# Soil Properties – General

Propriétés des sols—Générale

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*Président*: J. KÉRISEL (France)

MESSIEURS, J'OUVRE LA SESSION NUMÉRO UN qui est consacrée aux propriétés générales des sols. Je vous rappelle que le Rapporteur général est le Professeur Jennings de Johannesburg. Nous commencerons par un exposé du Rapporteur général, Professeur Jennings.

*General Reporter*: J. E. JENNINGS (Republic of South Africa)

Every good administrative officer should have three baskets on his desk. The obvious ones are the IN and OUT. But there is an important basket called PENDING and into this the wise officer will put papers dealing with subjects which he considers too difficult or too involved for immediate attention. I am not the first Gencral Reporter to observe that Division 1 falls into this class; no less a man than the President of our Society, Professor Casagrande, made the same observation when he was General Reporter for this session. My division deals with a wide diversity of subjects and in reporting on them I ask you to forgive an unavoidable disjointedness in my presentation.

My first subject will be that of *engineering geology* which is concerned principally with the soil profile and the processes which have been involved in its development. Engineering geology knows no boundary between rocks and soils, between intact or unjointed and fissured or shattered soils and rocks, or even between materials above or below the water table. The engineering geologist presents a qualitative picture; he puts his feet up on the desk and tries to reason out why the observed conditions are found in nature and he makes shrewd guesses or judgments on how the materials will behave when man's activity changes the conditions. While such men, of course, are influenced by their own previous experiences and are also very prone to their own pet philosophies, or hobby horses, they are generally not committed to any of the mathematical theories or special laboratory procedures into which the soil mechanics engineer so frequently attempts to compel the behaviour of the soil with which he is dealing. Soil mechanics seems to be tending away from engineering geology and it is time the two subjects were brought together again. Soil is a geological material which has been subjected to weather changes and to stresses and deformations in its past history. These have led to soil features such as fissuring, slickensiding, preconsolidation, possibly under reversals of stress, residual horizontal pressures, tectonic stresses. We may well ask whether these variables are being taken into account in theories and procedures which start out with the soil as an intact material occurring in nature under isotropic stresses. Clearly these are matters deserving deep study.

My next main subject will be *soil exploration, sampling, and* in-situ *testing*. We have, of course, the excellent manuai on this subject prepared by Hvorslev, and the additions that have been made since its publication, notably the penetrometer, the vane, and the pressuremeter. The results of our laboratory tests can only be as good as the samples we recover and we know that the most important materials are those which tend to appear as gaps in our borehole records. I am aware of significant developments in soil sampling but there is no mention of many of these in our proceedings. If these conferences are to achieve their technical objectives, then such developments must be brought to our attention and the administrative process we adopt must accomplish this end in a simple and routine way. I will speak more on this later in my address.

We come now to the *physical chemistry* of the soil system. This is concerned with the interaction of the three phases—solids, water, and gas—in a soil. A great deal of work is being done on this subject throughout the world and while we are fortunate in having an outstanding paper by *Reséndiz (1/22)*, which I hope will provoke considerable discussion, I must remark on the absence of contributions by persons such as Rosenqvist, Martin, Winterkorn, and others of like interests with whom I am at present not acquainted. I realize it is dangerous to single out my colleagues in this way, particularly when one of these three good friends sits with me on the platform. I anticipate severe criticism for doing so and I beg that they do not waste too much time in scolding me. Dr. Reséndiz has presented us with a number of very important ideas giving us physical principles which will help our understanding of a wide range of problems. extending from frost heave through to the swelling of clays which causes heave of structures. What we as engineers would like to hear from the *fundis*, that is the Zulu word for the wise men of all knowledge and medicine, is whether this work of Reséndiz is so important that we should devote the midnight oil to its understanding. We need to understand, at least in principle, what is happening as changes take place in the air and water content of a clay system. This understanding will help us in the solution of practicable problems which, after all, is our main task.

I move now to the next important set of problems which have found their place in Division 1—the *problems of soils which are partly saturated*. Oddly, these problems are concerned with the greater part of expenditure on construction work and yet they have only been accorded a portion of one division of this Conference. As with saturated soils, the problems of partly saturated soils are concerned with four basic issues:

1. the amount of heave or settlement which will occur when a subsoil changes in water content,
2. the rate at which these changes take place,
3. the change which occurs in the strength of the soil,
4. the structural measures which should be used when such events take place.

The work of the recent conference at Texas A & M College is significant and this was devoted almost entirely to the question of heaving, but the important question of additional settlement due to collapse of grain structure should not be ignored.

Expressed in current terminology, the problems of most partly saturated soils are associated with an increase in pore pressure with time, after completion of the building or structure. This is in direct opposition to the case of structural loads on fully saturated soils where the pore pressure decreases with time. In the former case the soil strength will decrease with time, and we are concerned mainly with long-term effects, whereas in the latter case "end-of-construction" conditions present the most adverse situations and the immediate or short-term strengths are the most important in design.

These problems of partly saturated soils are recognized in a general way in much of our engineering practice. In addition to the papers in Division 1, which deal with additional settlement occurring when some loaded partly saturated soils become wetted, we also find three papers in Division 6 which deal with the subject. *Scherrer (6/25)* and *Sowers, et al. (6/29)* deal with rockfill dams and the mechanics involved in sluicing the rock with water, and *Holestöl, et al. (6/13)* describe the settlement of a dam foundation subsoil which was initially partly saturated. These cases are also probably problems of "collapse settlements."

Finally in Division 1 of the Conference we find a number of papers which do not seem to fall into any particular group, e.g., the problems of special soils in particular areas of the world, such as the muskegs of Canada. These are special local problems and the Conference must provide place for them. Such problems may well develop and become more important: at one stage the problems of partly saturated soils were viewed as a local condition but with the passage of time, they have increased in importance and scope so that they can be viewed as a special branch of the study.

This leads me to the most important part of my summarized report, and I wish to make certain recommendations for the Executive Committee. In these I carry the support of my friend, Professor Ter-Stepanian, who unfortunately has not been able to be present at this Conference. My proposals are as follows:

1. That the name of Division 1 be changed to "Basic General Factors in Soil Mechanics" and that this division be used for all papers that can find no convenient place in any other division.
2. That new divisions be created as follows: (*a*) engineering geology, exploration, and sampling; (*b*) physical chemistry of soil systems; (*c*) problems of partly saturated soils. This will mean we have four divisions instead of the present one. If the suggestion is adopted we will probably also wish to subdivide other divisions. I very much doubt whether the other existing divisions will fragment into as many as four each, but it does mean more divisions, which I submit will be a good thing.
3. That the General Reporters for each division at the next Conference should be appointed within the next 12 months.
4. That it should be first duty of a General Reporter to keep himself informed on developments within his field over the inter-conference period. National Committees should assist this by sending relevant papers in the fields to the General Reporters.
5. That 12 months before the next conference, the General Reporter should submit to the Executive Committee two or three names (which should include his own) of persons who would be best able to prepare a long paper for the next conference in the subject under consideration.
6. That from the names submitted by all of the General Reporters, the Organizing Committee should draw up a list of persons who would be invited to write papers describing and reviewing the work in a particular number of selected fields. These must be review reports, and should not aim at presenting new findings made by the authors.
7. That the practice of receiving and publishing shorter papers from the National Societies should continue but the number might be somewhat reduced. Contributions of important information published elsewhere could rely on the review report making mention of their work.

These proposals are put for your consideration. The object of our Conference is a technical one: to permit us to be brought up to date in the whole field once each four years. We should seek a method for doing this in a simple and routine fashion.

*(Profesor Jenning's General Report appears on pp. 215–19.)*

Président Kérisel

Je remercie le Professeur Jennings de son exposé à la fois très clair et très ramassé, et je donne immédiatement la parole aux membres du groupe de discussion.

*Panelist*: A. W. Skempton (Great Britain)

Reference is frequently made to the limitations, and even the irrelevance, of laboratory tests in predicting full-scale behaviour. On the other hand, vast numbers of laboratory tests are carried out every year in the belief that their results are useful in the design of engineering works. It may therefore be helpful to attempt some clarification of this rather paradoxical situation.

At the outset it must be emphasized that there are indeed many cases in which tests on undisturbed samples give satisfactory, or reasonably satisfactory, results. Within this category are found, for instance, the whole range of stability problems involving moderately homogeneous saturated clays which contain few joints or fissures and show a "ductile" type of failure (i.e., a flat-topped stress-strain curve in the region of peak strength). Most normally consolidated and lightly overconsolidated clays fall into this class, as well as the majority of boulder clays. Where a "first-time" slip occurs in such materials, the peak strength of undisturbed samples is usually within 15 per cent of the strength as deduced from a stability analysis. Bishop and Bjerrum (1960) summarize more than twenty cases where such accordance has been obtained from foundation failures of footings and embankments and loading tests, seven cases of base failure in strutted excavations, and another four cases involving end of construction slips in cuttings. Less data are available for long-term failures, in these materials, but well-documented records have been published by Sevaldson (1956), by Skempton and Brown (1961), and by Kjaernsli and Simons (1962). In all three cases the measured effective

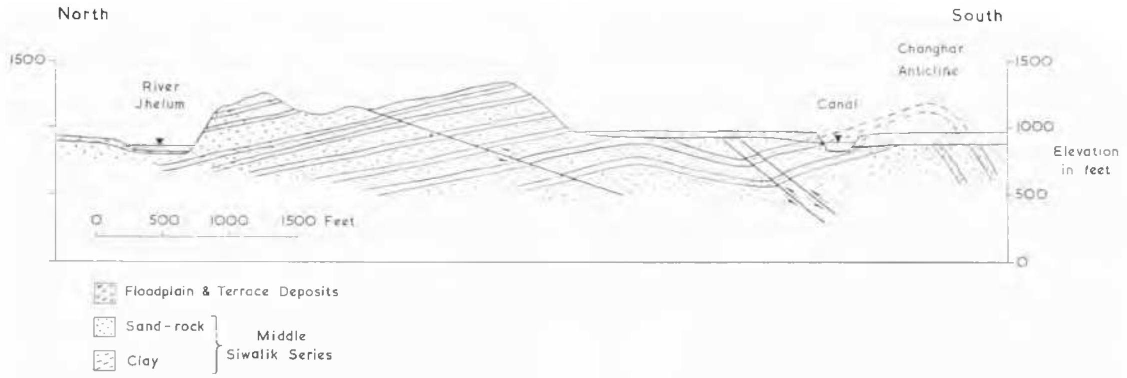



FIG. 1. Mangla Dam—cross-section at south abutment.

stress parameters (peak strength), when used in conjunction with pore pressures observed in the field, lead to errors in the calculated factor of safety not exceeding 10 per cent.

There are, however, numerous conditions where important discrepancies can arise between laboratory and field data and, for simplicity, these may be considered under five headings. (The rather large class of problems in which the fault lies not so much with the tests as with the theories used to translate the test results into practical terms is not included.)

1. In the first group the laboratory tests are wrong in principle, as they are measuring properties fundamentally irrelevant to the problem in hand. A conspicuous example is the use of undrained tests to predict the long-term stability of clay slopes. In London Clay, errors of up to 300 per cent can arise from this misapplication (Skempton, 1948) and even larger errors have been reported for other overconsolidated clays (see the summary by Bishop and Bjerrum, 1960). This kind of mistake is not often made now, although in the not very distant past it has certainly led to much confusion.

2. In the second group the tests are of the right type but are not correct in detail. For instance, they may be carried out at a rate which is too fast, by several orders of magnitude, as compared with the rate of strain operating in the field. This rate effect can be pronounced in undrained strength tests, as shown by Casagrande and Wilson (1951) and other investigators, yet it is not always recognized.

3. In the third group the tests are of the right type and correct in detail, but they are carried out on the wrong material. As an illustration of what might happen in problems of this category I will refer to the geological conditions at the site of Mangle Dam. Here the Siwalik series (Fig. 1), consisting of alternating hard clays and soft sandstones, have been folded into a sharp asymmetric anticline (Fookes, 1965) and, during folding, many of the clay strata were sheared parallel to the bedding planes. The shear zones are typically only a few inches thick, but they can be found at very considerable distances from the crest of the anticline, and the clay in the shear zones has had its strength reduced almost to the residual value. Moreover, the residual strength is far less than the peak strength of the unsheared clay. Thus if the stability of the foundations and of the deep cuts for the spillway had been based on ordinary drained tests carried out on the unsheared clay, dangerously misleading results would have been obtained, even if the tests were correct in every detail and used correctly in stability analyses. Evidently it is the strength in the shear zones which controls design, and the stability analyses must take into account the geometry of the strata. A further illustration, of a similar nature, is provided by the Walton's Wood landslide (Fig. 2). In a manner typical of old landslides, this contains many slip surfaces, and if anything is done to reduce stability, by excavating near the toe, or by building an embankment on

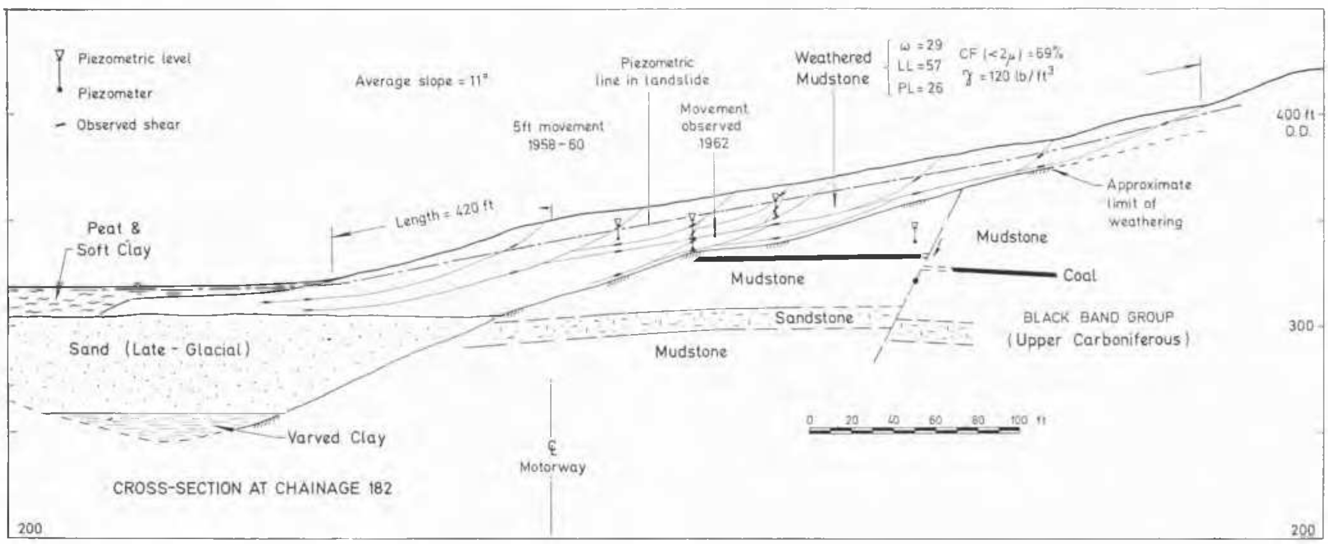
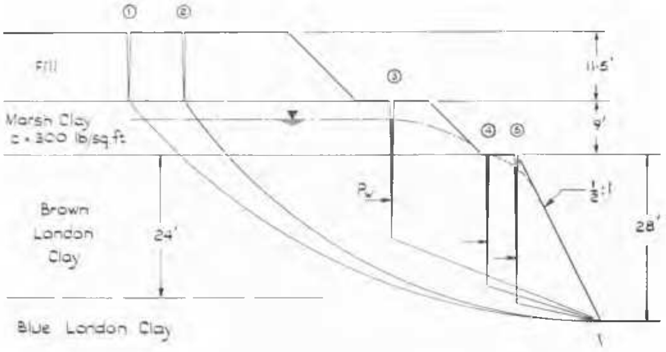



FIG. 2. Walton's Wood landslide—cross-section at chainage 182.

the slope, movement will be re-activated along one or more of these slip surfaces. Now, in taking samples from boreholes, it is easy to miss the slip surfaces, as they are less than one inch thick and can be separated by distances of several feet. Drained tests on samples from the body of clay gave the peak strength parameters $c' = 320$ lb/sq.ft. and $\phi' = 21°$, and using these values the factor of safety of the slope would have been calculated to be 2.5. In fact, the slope had a factor of safety of 1.0, and this result could be predicted with an error of less than 10 per cent by using the strength as measured on the slip surfaces, or by using the residual strength of the clay, both corresponding to the parameters $c' = 0$ and $\phi' = 13°$ (Skempton, 1964).




| Case | | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|
| London Clay | Strength for F = 1·0 $\bar{s}$ | 1100 | 1120 | 1060 | 1240 | 1430 |
| | Measured strength $c$ | 2000 | 2000 | 2200 | 2400 | 2500 |
| | $\bar{s}/c$ | 0·55 | 0·56 | 0·48 | 0·52 | 0·57 |

FIG. 3. Bradwell—Reactor No. 1—excavation analysis of slip 1.

4. A fourth group of problems in which laboratory tests may be misleading is perhaps a variant of the preceding group. It concerns the effect of minor structural and depositional features; and the errors involved may be considered as arising from a scale effect. The tests are of the correct type but, in fact, they are made on samples which are too small to reveal the properties of the mass. In a lecture given last week at Laval University, Professor Rowe quoted some remarkable examples of the wide discrepancies between conventional laboratory values and actual field values of the coefficient of consolidation in laminated and layered clays. He also showed that the errors could be reduced by testing larger samples, and that substantially correct values were obtained from *in-situ* constant head piezometer tests using the technique suggested by Gibson (1963). Another illustration of scale effect is provided by fissured clays. The ordinary laboratory tests are made on specimens which may be smaller than the spacing of the fissures, and usually when a specimen is seen to contain a fissure plane it is rejected before testing. Yet the over-all strength and behaviour of the clay in the field must be influenced by the fissures (including in this term the small joints frequently present in stiff clays). Thus, in the investigation of a short-term slip in a deep excavation in the London Clay at Bradwell (Fig. 3), Skempton and LaRochelle (1965) were able to show that the actual strength in the field was only 55 per cent of the strength as measured in conventional undrained tests on small (1½-in. diameter) specimens. These tests were of the correct type, but they led to an appreciable overestimate of strength due to the relatively high rate of strain used in the laboratory. In addition, however, the strength of specimens tested at the correct rate of strain still exceeded the field strength by more than 30 per cent, because the specimens were solely representative of the intact clay. In other words the presence of fissures in the clay mass reduced the strength to a value approximately two-thirds of the laboratory result.

5. With long-term slips the position is worse, since fissures can lead to local overstressing and progressive failure. Here we encounter a fifth group of problems in which discrepancies can exist between field and laboratory data, characterized by the presence of a mechanism (such as progressive failure) not induced in normal testing procedures. As an example, I can quote a "first-time" slip in a London Clay cutting which occurred 49 years after excavation, with an average strength along the slip surface corresponding to a residual factor of about 80 per cent (Skempton, 1964). In such cases, large errors would be involved if the peak strength parameters were used for estimating long-term stability, even with very slow drained tests and an accurate knowledge of the pore water pressures.

The pattern that emerges from this brief examination of the problem is clear. Laboratory tests are carried out on relatively small specimens and their use in full-scale problems is based on the assumption that they are representative of the stratum from which the samples were taken. If the stratum is homogeneous and the samples are undisturbed then the laboratory tests will be reliable provided, of course, they are correctly performed and designed to measure relevant properties. If the stratum is not homogeneous there is a serious possibility that the samples themselves may not be representative, in which case the test results cannot be reliable, or that the specimens may be too small fully to invoke the effects of fissures, joints, laminations, and other features which tend to weaken the mass, to induce progressive failure, and to increase the permeability.

It seems obvious, therefore, that more attention should be given to studying the materials as they occur in nature, and to the development of quantitative methods for describing their structure. By these means it should be possible to bridge the gap which all too often separates results obtained in the laboratory from those observed in the field.

## REFERENCES

BISHOP, A. W., and L. BJERRUM (1960). The relevance of the triaxial test to the solution of stability problems. *A.S.C.E. Research Conference on Shear Strength of Cohesive Soils* (Boulder), pp. 437–501.

CASAGRANDE, A., and S. D. WILSON (1951). Effect of rate of loading on the strength of clays and shales at constant water content. *Géotechnique*, Vol. 2, pp. 251–63.

FOOKES, P. G. (1965). Orientation of fissures in stiff overconsolidated clay of the Siwalik system. *Géotechnique*, Vol. 15, pp. 195–206.

GIBSON, R. E. (1963). An analysis of system flexibility and its effect on time-lag in pore-water pressure measurements. *Géotechnique*, Vol. 13, pp. 1–11.

KJAERNSLI, B., and N. SIMONS (1962). Stability investigations of the north bank of the Drammen River. *Géotechnique*, Vol. 12, pp. 147–67.

SEVALDSON, R. A. (1956). The slide in Lodalen, October 6th 1954. *Géotechnique*, Vol. 6, pp. 167–82.

SKEMPTON, A. W. (1948). The rate of softening of stiff fissured clays, with special reference to London Clay. *Proc. Second International Conference Soil Mechanics and Foundation Engineering*, Vol. 2, pp. 50–3.

——— (1964). Long-term stability of clay slopes. *Géotechnique*, Vol. 14, pp. 77–101.

SKEMPTON, A. W., and J. D. BROWN (1961). A landslide in boulder clay at Selset, Yorkshire. *Géotechnique*, Vol. 11, pp. 280–93.

SKEMPTON, A. W., and P. LAROCHELLE (1965). The Bradwell slip; a short-term failure in London Clay. *Géotechnique*, Vol. 15, pp. 221–42.

*Panelist*: I. TH. ROSENQVIST (Norway)

During the London Conference I acted as the General Reporter and I made a plea for studies in the physical chemical properties of soils which I said were necessary. I was criticized by some of my colleagues who said that I wanted to change the Conference into a conference of physics instead of a conference on soil mechanics. Since then an increasing number of valuable papers on the study of the physical chemical properties and physical chemical factors involved in the soil properties have been published and there is a considerable amount of literature outside of the field of soil mechanics that deals with such problems.

The importance of the physical chemical factors of soils is now well established. Here today, I feel that I can best illustrate this by discussing some of the papers presented to this Conference, especially the paper by *Reséndiz (1/22)*, "Considerations of the Solid-Liquid Interaction in Clay-Water Systems." This paper refers to what I personally believe to be one of the most important aspects of soil mechanics, namely, the importance of a thorough understanding of the fundamentals of the soils, specifically clay. Soil mechanics is an engineering science, and it has dealt mainly with the bulk properties, as if the soil had been plastic in nature, as just mentioned here by Dr. Skempton. What is actually happening on the microscopic or molecular level has been ignored. Engineers, being practical men, are not interested in the geological history or atomic arrangement of the material they are dealing with, if they can get sufficient data to enable them to solve their problems. However, without the fundamental study of the forces acting in the molecular and atomic states, there is a limit to what our science can achieve. We may soon come to a standstill if we do not extend our studies beyond pure engineering. Consequently, the papers presented at this Conference on the physical aspects of soils ought to raise considerable interest.

Although I may disagree with Dr. Reséndiz on certain details, I would recommend his paper for thorough study. When the author says that the permanent dipole of the water molecule is responsible for most of the abnormal properties of water, this may only be part of the picture. Certainly, the innermost water nearest to the interfaces of the clay minerals will have other types of hydrogen bonding. In free water, in the interaction between the permanent dipoles, the Keesome forces may be responsible for about two-thirds of the total van der Waal attraction, whereas the London forces and field induction may be responsible for the other one-third. In the clay water system, the water molecules seem to be influenced by the potential field from the minerals to a much greater extent than in free water. This has been illustrated and very well clarified by a group of German scientists around Professor Armin Weiss. At last year's meeting of the German Ceramics Society, Dr. Weiss and his co-workers presented good reasons for assuming that the water layers nearest to the mineral surface in the clays have undergone changes to such an extent that the number of positively charged $H_3O^+$-ion and $OH^-$-ion groups are very much higher than in free water. This fact should be remembered for all studies of soil water systems. Together with studies of diffusion properties and nuclear magnetic resonance, it indicates that the properties of part of the water in clay water systems may differ so much from the normally known properties that, for instance, the double-layer theory of Gouay-Chapman cannot be used without important qualifications.

Dr. Reséndiz is arguing against the concept that the density of the water in the vicinity of clay particles should be lower than normal. His arguments are based upon the idea that the drift of interstitial molecules should increase the density. I think this point ought to be clarified further. For instance, experimental data from Purdue University by Low and Anderson and several others, and recently by Dr. Lang at the Berkeley Conference on Clay Mineralogy, also seems to have shown experimentally that the density is in fact decreasing and not increasing as you approach the clay minerals. There are opposing factors, all of them affecting the mechanical properties of the soils. The ordering effect due to the underlying mineral lattice should certainly decrease the density. If it had not been for the disturbing factor of the diffusely distributed counter ions, a density close to that of ice would be expected, although the configuration of the water molecule does not fit with the ice lattice. Because of large cations and the anions, the structure would be broken and a higher density expected. Whether this will result in a density below or above unity is not theoretically clear, and I do not think that there are theoretical reasons to state that a density below unity is unlikely as long as good experimental physicists present the opposite views.

Reséndiz's paper stated that the plasticity of mixtures of clay and water is explained in terms of a stable relatively thick layer of viscous water around the mineral particles. This was also the original idea of Dr. Terzaghi, and it has been discussed several times in our literature. Although this view differs from that of many other eminent scientists, I will remind you of the work reviewed by Robert Martin at the Massachusetts Institute of Technology. Personally, I agree that plasticity is best understood in terms of adsorbed water although I will not agree that the water is relatively thick. Diffusion data seem to indicate that this layer is relatively thin, only a few tens of angstroms, as compared with the average size of the pores which are in the order of hundreds of angstroms. I do not think that the increased viscosity of water due to small amounts of bentonite should be taken as a clue to the rigidity of the water but rather as evidence of edge or corner to plane attractions between the flaky minerals combined with fairly thin layers of adsorbed water.

The paper presented by *Arnold (1/3)* on "The Effect of Surface-Ion Attractive Forces on the Permeability of Bentonite" is another important contribution to our understanding of the fundamental forces in clay minerals. Consolidation tests on normal commercial bentonites indicate that interparticle spacing between 30 and 10 angstroms represents a rather unstable distance with a collapsing of the structure and a spontaneous reduction of spacing. The fact that the coefficient of permeability and the void ratio do not show any unique relationship to void ratios fitting with the interparticle spacing demonstrates the importance of colloidal physical properties in clay materials.

I am glad that this Conference has contributed to this field in such a valuable way. I think that it will open our eyes to the need for further study I suggested several years ago.

*Panelist*: F. P. SILVA (Brazil)

Our contribution to this panel discussion concerns the geotechnical properties of some of the residual soils of southern Brazil.

Our experience with these soils is mainly in connection with soils derived from *in-situ* weathering of gneiss, basalt, and sandstone. Information about these soils has been submitted to previous conferences and the main characteristics of their profiles have been described by Vargas (1963) at the Second Panamerican Conference on Soil Mechanics and Foundation Engineering. Besides those three types of soils, Vargas suggested a fourth type that should also be considered a residual soil. This one is the product of *in-situ* weathering of the sedimentary tertiary deposit where the city of São Paulo is located. It is most interesting to observe that this fourth type is very similar, in aspect and engineering properties, to the upper "porous" zone of residual soils derived from the *in-situ* weathering of gneiss.

One feature common to all these soils is their upper "porous" zone. Soils of this zone may attain thicknesses of up to 10 m or more and are important from an engineering standpoint because they serve as a foundation for light buildings and have to be carefully analysed when present at sites where large construction jobs are to be located.

Soils of this "porous" zone are characterized by a high void ratio, a low degree of saturation, and a low "equivalent" preconsolidation pressure. They are, therefore, very compressible and sometimes extremely sensitive to additional settlement when wetted under load. In one case of airport pavement construction we had an opportunity to deal with a soil from sandstone that had a natural void ratio of 0.9 and a degree of saturation of only 20 per cent. The equivalent preconsolidation pressure was only 0.5 kg/sq.cm. and when subjected to saturation after stabilizing under a certain consolidation pressure, settlements would again take place up to twice the settlement which had already occurred. In another case, in which a dam was to be constructed on weathered basalt soil, we measured void ratios in the natural condition as high as 1.7 and preconsolidation pressures as low as 0.4 kg/sq.cm. This soil was also extremely compressible and sensitive to saturation. In this case the lateral slopes of the valley were gentle, and although the maximum thickness of compressible material was 4 m, no abrupt changes in thickness were observed. Therefore, no differential settlements of objectionable magnitude were to be anticipated.

Because of the properties light buildings founded directly on this upper zone may start to crack a long time after construction. Immediately after construction initial equilibrium is attained very quickly, because the settlement of the soil is practically simultaneous with the load application. However a slow change in the water content of the soil begins to take place under the covered area, and additional settlements develop until a second equilibrium is reached under the new moisture condition. The building may stand perfectly during the first phase of settlement but show cracks in the second phase. As a consequence, the practice of excavating the porous soil and backfilling the excavation with the same soil properly compacted to form a foundation is common in the construction of road and airport pavements, buildings, and dams. In the case of the airport pavement construction mentioned above, the porous material was excavated to a depth of 1 m and used as backfill. The C.B.R. value changed from 2 per cent in the natural, soaked condition to 70 per cent the compacted, soaked condition.

Most of the soil material that is used in earthwork engineering in this region of Brazil comes from the porous upper zone of *in-situ* weathered rocks. These soils make good material for fill construction. When compacted at about 1 per cent below standard Proctor optimum moisture content to degrees of compaction of 95 to 100 per cent, they develop fairly good strength, low compressibility, and low permeability. On the other hand, when compacted to these conditions they seem to be somewhat sensitive to cracking. One such crack was observed in a 60-m-high rolled fill dam in which the borrow material was a residual soil from gneiss. The crack was probably caused by differential settlements within the fill because the abutments were fairly steep and the foundation was not compressible. A few settlement plates were installed in this dam but none sufficiently close to the zone where the crack occurred. Adopting the measured values that were closest to this zone, we found a differential settlement of 32 cm over a length of 40 m or approximately 1 : 120. This means that the fill compacted in the condition described above is still quite rigid and it would be advisable to make the fill more plastic without impairing its other properties. Incidentally, collecting more data pertaining to cracks in earth dams is desirable so that the conditions under which cracks develop may be better understood. Keeping in mind the recent work by Leonards and Narain (1963) on the subject, it would be desirable to correlate actual behaviour related to cracking with other soil properties.

Another matter of interest to this audience concerns the development of pore pressures in compacted embankments of these soils. We have, in many instances, observed negative pore water pressures that become more negative (tension increases) with increasing load. This happens in the initial stages of the loading. With continuing load application a maximum water tension is reached, then it begins to decrease and eventually positive water pressures appear. This seems to occur only in soils with low initial degrees of saturation. For higher degrees of saturation only positive pressures are observed. To interpret this behaviour, we consider the soil to be represented by a capillary tube partially filled with water. Compressing the soil would correspond, in the model, to compressing the wall of the capillary tube reducing its diameter, with the resulting increase in water tension. When the tube becomes saturated, new external load applications generate positive pressures in the water.

Finally, I would like to mention the case of a fine compressible material mixed with larger fragments. An example of this is a dam partially founded on talus material in gneiss soil. The talus deposit was quite heterogeneous, with boulders and large fragments of partially weathered gneiss distributed at random in the soil mass. There was no certain way to predict the settlement of this mass of soil under load. Several consolidation tests were made on the finer material using 10-cm-diameter specimens. The compressibility of the fine material was high and to account for the presence of the boulders, estimates of the settlement were made by considering an "effective compressible thickness" of the total mass equal to one-half the real thickness. With this assumption, an estimate was made of a maximum value of 0.8 m for the settlement. A benchmark and a settlement plate were installed to measure the settlement of this part of the foundation and the final value observed for the settlement, sometime after construction, was only 35 cm. This result, smaller than the estimated value, proves that the percentage of large fragments was high enough to control the behaviour of the whole mass. In cases like this, predictions of soil behaviour are difficult and it appears that only expensive tests on large soil masses will be of any use.

REFERENCES

LEONARDS, G. A., and J. NARAIN (1963). Flexibility of clay and cracking of earth dams. *Jour. Soil Mechanics, A.S.C.E.*, March, 1963.

VARGAS, M. (1953). Some engineering properties of residual clay soils occurring in Southern Brazil. *Proc. Third International Conference Soil Mechanics and Foundation Engineering.*

——— (1963). Projeto e comportamento da barragem Euclides da Cuna. *Proc. Second Panamerican Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 331–46.

*Panelist*: J. G. ZEITLEN (Israel)

I particularly wish to congratulate the General Reporter, Professor Jennings, on not only a lucid summing up of the field of "Soil Properties—General," but on his engineering approach towards the evaluation of the contributions. I would like to discuss some of the special problems of partly saturated soils. From the viewpoint of the engineering requirements of soil studies we are interested in the rate of movement, amount of movement, strength, and an engineering solution. Parameters often measured include soil moisture, density, stress-strain relation, and strength. In recent years increasing use has been made of the soil science approaches, including physico-chemical factors, which certainly have much to contribute. We should not feel inhibited about continuing in this direction because of the limitations still existing in applying such theoretical and scientific work directly to engineering problems but, on the contrary, increase our efforts towards finding the connections and exploiting them. Good progress is being made in studying the condition of partly saturated soils in the field, increasingly utilizing such soil science concepts as suction measurements to determine the free water energy. Information is thus obtained which can be correlated not only with soil moisture, but as far as possible with other characteristics. Particular note should be made of the achievements of the Road Research Laboratory in England and of the C.S.I.R.O. in Australia in the direction of both field measurement and theoretical evaluation of suction. One difficulty is that soil moisture is not measured directly, but this may not be too serious if suction properties can be better correlated with actual engineering properties, such as stress-strain behaviour. Thus, future progress and information involving moisture parameters will involve not only quantitative determinations, but also a study of the nature of the pore water and the stresses acting in various phases of water and clay minerals.

Professor Skempton has pointed out the necessity of evaluating the characteristics of the undisturbed soil for the actual field conditions concerned, taking into account the entire soil mass structure, complete with fissures. The change in characteristics after the soil has been subjected to repeated strains has also been discussed. Soil structure is important because the bulk of engineering problems involve stress-strain relations at stress value well below rupture conditions. However, in the usual determinations of design parameters, such as strength, results are obtained and utilized without considering the structure of the soil. The role of bound water, mineralogy of particles, and the interparticle arrangement and forces influence the over-all behaviour of the soil mass in a way analogous to the manner in which location, nature, and strength of connections determine the behaviour of a structural frame or truss. For practical reasons in design we have little choice but to continue with only phenomenological determinations on soil samples but it should be recognized that in principle we are testing small models of structures with the possibility that they may not be representative of field conditions even though composition, density, and moisture conditions are similar.

In connection with the stress-strain behaviour of soils under different conditions, shear and consolidation parameters are being found with increasing accuracy and refinement, utilizing the latest developments in instrumentation and electronics. Mathematical tools and methods have been developed which, with the increasing use of computers, can be more easily applied to actual field cases. During recent years we have also seen both soil science specialists and soil mechanics researchers pay increasing attention to the study of clay minerals and clay water by means of physico-chemical concepts. However, we are still a long way from solving the problem of how to correlate more quantitatively the physico-chemical properties with the engineering properties determined in the usual testing programme to find design parameters.

Examples of the influence of soil structure may include the following based particularly on experience in Israel:

1. Differences of appreciable magnitude are found in the swelling characteristics of undisturbed and remoulded soils. Also, even for similar specimens wetted under various applied loads, it has been indicated that the volume change *versus* pressure relationship is a different function depending on whether consolidation or expansion is occurring, reflecting a breakdown in structure under applied load.
2. As a number of investigators have found, there is a large variation in shrinkage characteristics depending on whether the soil is remoulded or in its undisturbed condition. This is most obvious in shrinkage limit determinations.
3. The permeability of individual samples taken from unsaturated clay may differ from the permeability of the clay mass by several orders of magnitude, particularly in relation to the uppermost zone of the soil where cracking and "shattering" is most pronounced following climatic changes.
4. Utilization of a compacted fill composed of a soil or soft rock with its own structural characteristics may produce a far from uniform material. In one case in California a series of small samples, carved from a cubic foot sample of "homogeneous" decomposed granite fill, were tested and produced densities ranging from 118 to 128 lb/cu.ft.

PRÉSIDENT KÉRISEL

Je remercie les panelistes pour l'ensemble des exposés. Nous allons maintenant faire un arrêt de 15 minutes.

*(There followed a brief intermission.)*

PRÉSIDENT KÉRISEL

Pour la dernière séance, les six orateurs qui prendront la parole sont inscrits sur le tableau. Le premier sera le Professeur Tsytovich, suivit par Mme Grigorian, M. Majtényi, M. Hampton, le Professeur Buchanan, et le Dr. Croney.

G. TER-STEPANIAN (U.S.S.R.) (Presented by N. A. TSYTOVICH (U.S.S.R.))

Professor Jennings points out the necessity of a more precise definition of the problems to be covered by Division 1—Soil Properties, General. His criticism of the division as it stands is well founded. It seems important to discuss this question not for the sake of the present Conference which is now under way but for future ones. Papers presented to this division cover a very wide range of problems, partly because of the gradual and excessive shortening of the number of divisions in the soil mechanics conferences over the years. There were 16 divisions at the Cambridge Conference, 12 at Rotterdam, 8 at Zurich, 6 at London, 7 at Paris, and 6 again at Montreal. This trend seems to contradict the great increase in the volume and variety of information which characterizes modern soil mechanics.

It is noteworthy that the number of technical sessions at the last three conferences was greater than the number of divisions (9 at the London Conference, 8 at Paris, and 9 at Montreal). This enabled the number of divisions to be increased without complicating the operation of the conferences. Perhaps we can consider it a rule to allocate one technical session to one division.

It would be advisable at future conferences to have a separate division on engineering geology, covering soils, their occurrence, classification, description, exploration, sampling, and so on. One of the main tasks of this division must be analyses of the engineering consequences of geological facts, of the engineering interpretation of geological structure, and, especially, of the misuse of soil mechanics. The importance of the last problem, the misuse of soil mechanics, was emphasized by the late Professor Terzaghi (1961a, 1961b) in his last papers. There must be an antidote for the enthusiasm of using mathematical solutions in such geological conditions which exclude the possibility of inevitable simplifications connected with this application. If this proposal is adopted, Division 1—Engineering Geology, with its analysis of the misuse of soil mechanics will serve to some degree as a purgatory to other divisions—circles of paradise where the proper use of this science will be demonstrated.

It should be noted that the problems of engineering geology have been mastered for a long time by Soviet specialists and that several problems of engineering geology were worked out in the U.S.S.R. earlier than anywhere else. Monographs by Professors Saverensky (1937) and Popov (1951) on engineering geology are among the best textbooks in this field. A monograph on permafrost was published by Professor Sumgin in 1927, and the fundamentals of the mechanics of frozen soils were formulated by Professors Tsytovich and Sumgin in 1937. The mechanics of collapsive loessial soils were developed by Professors Abelev (1931) and Denisov (1946). The high sensitivity to remoulding of some post-glacial structural clays was established by Professor Ter-Stepanian (1941) in 1935 and published 5 years earlier than the excellent investigations made independently by Professor Rosenqvist (1946).

We have much evidence of the important role which engineering geology plays in the solution of problems in soil mechanics. It is worthwhile to touch upon the reverse problem of the relation of soil mechanics to engineering geology, and even to the solution of problems in tectonics. This great realm of application of soil mechanics remains almost untouched.

REFERENCES

ABELEV, YU. M. (1931). Provisional instruction for the design and construction of structures on loessial soils. *Stroitel'naya promyshlennost'*, No. 7, pp. 368–70 (in Russian).

DENISSOV, N. YA. (1946). *On the nature of subsidences in loessial loams.* Izd. "Sovetskaya nauka," Moscow (in Russian).

POPOV, I. V. (1951). *Engineering geology.* Moscow, Gosgeolizdat; 2nd rev. ed. Izd. Moskovskogo Universiteta, 1959 (in Russian).

ROSENQVIST, I. TH. (1946). Om leires kvikkagtighet. Statens vegvessen, Veglaboratoriet, *Meddelelse*, Vol. 4, No. 3, pp. 29–36.

SAVERENSKY, F. P. (1937). *Engineering geology.* Moscow-Leningrad, Gosnauchtekhisdat (in Russian).

SUMGIN, M. I. (1927). *Permafrost of soil within limits of the USSR.* Vladivostok; Moscow-Leningrad, 2nd ed. Isd. Akademii nauk SSSR, 1937 (in Russian).

TER-STEPANIAN, G. I. (1941). Geotechnical investigations of Yoldia structural clays. Trudy nauchno-issledovatelskogo sektora Leningradskogo otdelenya tresta glubinnykh rabot, vyp. 2, Moscow-Leningrad, Stroiizdat (in Russian).

TERZAGHI K. (1961a). Enginering geology on the job and in the classroom. *J. Boston Society Civil Engineers*, Vol. 48, pp. 97–109.

——— (1961b). Past and future of applied soil mechanics. *J. Boston Society Civil Engineers*, Vol. 48, pp. 110–39.

TSYTOVICH, N. A., and M. I. SUMGIN (1937). *Fundamentals of mechanics of frozen soils.* Moscow-Leningrad, Izd. Akademii nauk SSSR (in Russian).

A. A. GRIGORIAN (U.S.S.R.)

Experimental field research carried out for some time in the loess regions of the U.S.S.R. permits some essential theoretical and practical conclusions to be made (Grigorian, 1964; Golubkov, 1959; "Some Questions," 1963).

It was found on the basis of field experiments that the deformation zone under footings in loess soils is not deep, about 1.0 to 1.5$B$, where $B$ is the width of footing. The deformation zone is part of the soil massif that is in a state of stress under the footing, and in which an essential part (nearly 90 per cent) of the total volume deformation of compaction takes place. Soil collapse due to overburden pressure upon wetting actually begins at a depth of 6–9 m, where the overburden pressure is of the order of 1.0 kg/sq.cm.

The different principles of development of collapse under a footing load and under the overburden pressure are the reasons for dividing the soil conditions in loess regions into two types: (1) sites where collapse under the footing load is possible, and collapse under overburden pressure is practically impossible; (2) sites where both types of collapse are possible. This division is of great importance in the treatment of foundation soils and in the choice of construction measures to provide strength, stability, and safe use of buildings. We have suggested construction methods for different kinds of collapse, and these have been given in *The Norms and Rules for Designing Bases and Foundations.*

The division of soil conditions in loess regions into two types allows greater economy in the design of footings of small width (for example, strip footings). It is quite possible, in the case of soil conditions corresponding to the first type, to eliminate collapse of the soils in a small deformation zone; i.e., for footing widths less than 2.0 m to eliminate collapse in a layer with thickness of 2.0 m under the base of footing. In the U.S.S.R. the collapse of a soil layer having a thickness of 2.0 m is easily eliminated by surface tamping with heavy tampers weighing 3 tons.

Strength, stability, and the safe use of buildings and structures erected in soil conditions of the second type are provided by using different methods of eliminating soil collapse in the whole stratum (which often reaches a thickness of 15 to 30 m) or by using construction procedures which permit significant non-uniform deformation.

In 1964–5, a panel-type house, having flexibility in two directions, was designed by Moscivil Project and the Institute of Foundations. Rigid three-dimensional blocks (towers) on single footings constitute the foundation elements of this flexible construction. The towers are 3.0 m × 2.4 m and are placed in two rows spaced at intervals of 6 m. Other construction elements are supported by these towers. The peculiar feature of this method of construction is the possibility of levelling the structure by means of jacks. Jacks are placed in special niches made in the footing blocks of the towers. One experimental section of the aforementioned building was erected without partitions, engineering communications, etc., and tested in 1964 (Abelev, *et al.*, 1964).

At the beginning of 1965 we tested a flexible house ready for maintenance and erected on collapsing soils. The test was carried out by saturation of the foundation soil under the bearing towers at the side of the front flank and corner. As a result of wetting the foundation soil under a corner tower, the differential settlement between the corner tower and the nearest tower was 25 cm. Total settlement of the corner tower was 41 cm. The tangent of the angle of rotation of the external panels between the corner tower and the next nearest one was 0.04 after wetting. External panels were not connected with bearing towers by means of rigid connections, but were overlapped. After soil collapse, the joints between panels widened 5 cm. The differential settlement of 25 cm. did not cause damage to the house. Jacks were used to lift the towers, the maximum load on the jacks being 300 tons. During lifting of the towers most of the cracks and joints closed. The remaining cracks were repaired and the building was ready for use.

The construction technique described herein has been developed further in the U.S.S.R. to include building foundations located on non-uniform compressible soils.

### REFERENCES


——— (1963). Some questions of building construction on loess soils. *Materials of All-Union Scientific Conference of Higher School.* Voronesch, 1963.

ABELEV, Y. M., I. D. BESPALI, E. A. SOROCHAN, D. L. SCHAPIRO, *et al.* (1964). Investigations of a panel type house and foundation soils with collapsible properties. *Foundations, Footings, and Soil Mechanics,* No. 1.

GOLUBKOV, V. N. (1959). Investigation zone under test plates. *Foundations, Footings, and Soil Mechanics,* No. 2.

GRIGORIAN, A. A. (1964). *Manual on designing bases and foundations for buildings to be built on collapsible soils.* Moscow.


S. MAJTÉNYI and M. I. ESRIG (U.S.A.)

The article by *Reséndiz (1/22)* represents an interesting approach to the behaviour of claywater systems. It appears, however, that some points need amplification. These include the structure of the clay water system, the forces acting in the system, and some details of the double-layer theories.

At the outset, it is appropriate to fix some ideas about the origin of the charge on clay particles. Details of the concepts discussed below were presented by van Olphen (1963), Esrig and Majtényi (1965a), Fripiat (1963), and Lambe (1958):

1. *Charges due to imperfections in the crystal structure.* Most often imperfections in clay crystals occur as a result of isomorphous substitution. For example, Al, Mg, and Fe are often substituted in the octahedral sheets for the higher valency Si, while in the tetrahedral layers Mg may be substituted for Al. The structure that results exhibits a net negative excess charge, which is balanced by exchangeable cations on the mineral or in the adjacent liquid phase.

2. *Charges due to selective, preferential adsorption of ions on the solid phase from the liquid phase.* The surface of the clay crystal can adsorb preferentially either positive or negative ions. The resulting surface charge is balanced by ions of the opposite sign that are in the liquid phase. The adsorption abilities of the surface depend on the surface properties and the ions in the electrolyte.

3. *Charges on the edges of clay particles.* The crystal structure is discontinuous at its edges. There exists either an exposed cation with an excess positive charge or hydroxyl radicals which can produce either neutral or negative charges. If the surrounding electrolyte solution has an acidic pH, then the hydroxyl radicals at the edges can be decomposed and the edge charge is positive. When the electrolyte solution is basic, hydroxyl ions in the liquid phase can complete the crystal structure, producing zero or excess negative charges on the edges. At the edges, the excess charge can also be satisfied by any of the ions in the electrolyte, independent of how the ions fit into the crystal structure. An ionic bond joins these ions to the crystal.

It is important to recognize that a charge on the solid phase of a clay water system will not, in general, originate from only one of the above sources. In the case of kaolinite, Schofield, Ekka, and Fripiat (1963) observed that the cation exchange capacity cannot be explained completely by isomorphous substitution. Furthermore, the total charge on kaolinite and some other clay minerals has been found to be pH dependent, probably because of the dependence on the pH of the electrolyte solution of the edge charges and of the adsorption ability of the other surfaces. However, on clay particles, the charge resulting from isomorphous substitution is frequently more important than that resulting from other causes.

Another problem which needs further study is the structure of water near clay surfaces. It is recognized that a great number of forces act on the water molecules adjacent to the clay surface. The electric field of the charges and multipoles on the clay surface attracts and orients the water dipoles (Esrig and Majtényi, 1965a). The thermal motion and the presence of exchangeable cations and anions disturb this orientation. The adsorption forces on water can vary with the distance from the clay surface and from point to point on the clay surface. DTA and vapourization tests under vacuum indicate that not all water molecules are bounded to the clay surface in the same manner. According to the strength of the adsorbing forces, Fripiat (1963) distinguishes between chemisorbed water and physically adsorbed water. Chemisorbed water exhibits a higher than usual degree of dissociation which influences the electrical conductivity and chemisorption properties of the clay surfaces. Chemisorbed water is held on the crystal structure by weak, short-range chemisorption and van der Waal's forces. Physically adsorbed water is held by van der Waal's forces and hydrogen bonds.

Recently, Wu (1964) studied the bonding between clay minerals and water using nuclear magnetic resonance. He stated that his experiments indicate that the water close to the surface of the clay has a structure different from that of the ice at temperatures below 0 C. These data suggest that the thermal motion, exchangeable cations, and special structure of the clay surface make the development of an ice-like crystal structure in the water improbable, even though the water layers adjacent to the clay surfaces are probably strongly oriented, particularly at points where the adsorption force is due to hydrogen bonding.

Anderson and Low's experimental data (1958) suggest that the water density decreases towards the clay surface. They also show that the density of water is a function of the exchangeable cation on the clay surface. They found that at a distance of 10 Å from the surface the densities of water of lithium-, sodium-, and potassium-bentonite were 0.975, 0.972, and 0.981 gr/cu.cm., respectively. Martin (1960) concluded that Anderson and Low's data are acceptable and in agreement with experiments from other sources.

It appears from Anderson and Low's data that an analysis that considers only the electrostriction phenomenon is incomplete. Their data show that the water density decreases towards the clay surface instead of increasing in accordance with the theory presented by Reséndiz. In the writers' opinion, the apparent contradiction between the theory and the experimental data would be resolved if a more complete and complex force system could be considered (Bolt and Miller, 1958; Esrig and Majtényi, 1965a).

With regard to the confidence that can be placed in existing double-layer theories, the writers agree with Reséndiz's opinion. The most popular theory is the Gouy-Chapman double-layer theory (Esrig and Majtényi, 1965a, 1965b; Babcock, 1963; van Olphen, 1963). This theory recognizes that a separation of charges exists at the solid-liquid interface and states that the part of the double layer in the liquid phase has a diffused structure due to the dispersive forces. Debye and Hüeckel assumed the applicability of the Boltzmann statistics and the Poisson equation for developing an equation for the distribution of charges in the diffused part of the double layer. They obtained relatively simple charge density and potential distribution functions by integrating the linearized form of the Poisson-Boltzmann equation. Later the integration technique was improved by several investigators (Babcock, 1963; Esrig and Majtényi, 1965a, 1965b).

It is true that these double-layer theories consider only idealized conditions. One of the earliest analyses of the applicability of the Gouy-Chapman theory was presented by Bolt and Peech (1953). They argued that the affinity of clay surfaces for water does not affect significantly the charge distribution in the double layer. They indicated that usually the charge density on clay surfaces is very small and that most of the surface is available for water adsorption. Later Bolt (1955) analysed the effect of the energy terms neglected when the Poisson-Boltzmann equation is integrated to yield workable charge density and potential distribution functions. He found that the effects of dielectric saturation, polarization of ions, and ionic interaction are negligible or balancing when the surface charge density is less than $2 \times 10^{-7}$ meq/sq.cm.

Presently, the Gouy-Chapman theory enjoys a great popularity among soil scientists. It has been applied in the clay compression and swelling studies cited by Reséndiz and has yielded satisfactory results in many cases. However, the applicability of the Boltzmann equation is probably limited when small capillaries are considered. For small capillaries the statistical considerations assumed in its development are not necessarily correct. However, no practical, better theory is currently available.

REFERENCES


ANDERSON, D. M., and P. F. LOW (1958). The density of water adsorbed by lithium-, sodium-, and potassium-bentonite. *Proc. Soil Science Society America.* Vol. 22, pp. 22–4.

BABCOCK, K. L. (1963). Theory of the chemical properties of soil colloidal systems in equilibrium. *Hilgardia* (University of California), Vol. 34, No. 11.

BOLT, G. H. (1955). Analysis of the validity of the Gouy-Chapman theory of the electric double layer. *J. Colloid Science,* Vol. 10, pp. 206–18.

BOLT, G. H., and M. PEECH (1953). The application of the Gouy theory to soil-water systems. Discussion by C. E. Marshall. *Proc. Soil Science Society America,* Vol. 17, pp. 210–22.

BOLT, G. H., and R. D. MILLER (1958). Calculation of total and component potentials of water in soil. *Trans. Am. Geophys. Union,* Vol. 39, No. 5, pp. 917–28.

ESRIG, M. I., and MAJTÉNYI (1965a). *A feasibility study of electrokinetic processes for stabilization of soils for military mobility purposes.* No. 2 Report. *An analysis of the electro-osmotic phenomenon in soil capillary systems.* Cornell University Soil Engineering Series, June, 1965.

——— (1965b). Discussion, "Review and some studies of electrokinetic phenomena" by D. ZASLAVSKY and I. RAVINA. Conference on Soil Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas, August, 1965.

FRIPIAT, J. J. (1963). Surface properties of alumino-silicates. *Clays and Clay Minerals, Proc. Twelfth National Conference* 1964.

LAMBE, T. W. (1958). The structure of compacted clay. *Proc. A.S.C.E., J. Soil Mechanics Division,* Vol. 85, No. 1654.

MARTIN, R. T. (1960). Adsorbed water on clay. *Proc. Ninth National Conference Clays and Clay Minerals,* 1962, pp. 28–70.

VAN OLPHEN, H. (1963). *An introduction to clay colloid chemistry.* New York, Interscience Publishers.

WU, T. H. (1964). A nuclear magnetic resonance study of water in clay. *J. Geophysical Research,* Vol. 69, pp. 1083–91.


D. HAMPTON and E. J. YODER (U.S.A.)

*Kantey and Morse* (*1/12*) are to be congratulated on their efforts to inform the profession of the need for a statistical approach to soil sampling. A recognition of the need for such an approach and active research in this general area has been carried on in the United States since the late 1950's. This pioneering effort has continued under the general direction of Professor T. H. Thornburn, of the University of Illinois. However, the research efforts at Purdue University also have direct bearing on the paper by the authors. The first publication from this group was Hampton, Yoder, and Burr (1962).

Kantey and Morse spent a great deal of time developing a methodology for determining the minimum number of samples required to adequately ascertain the suitability of a soil stratum for use in highway or other earth structures. As outlined in their paper, the technique shown would be most effective for borrow materials or for compacted soils which had been previously processed. It does not help one to solve the problem of obtaining design parameters which characterize the properties of a subgrade or a dam foundation, for example. The authors assumed that engineering soil maps were available. In most areas of the world such will not be the case.

Hampton, Yoder, and Burr (1962) have developed a statistical analysis technique which can be used to define the engineering properties of soils in various areas, to determine if there are significant differences in the engineering properties of the soils in the various areas, and to predict the number of tests necessary to define the engineering properties of the soils to any given degree of accuracy. In areas in which reliable engineering soils maps are not available, all three of the aforementioned items of information are needed in order to develop sufficient data to construct engineering soils maps, and to determine the minimum number of samples necessary to define the engineering properties of the soil.

The analysis of variance model proposed is

$$Y_{ijklm} = U + C_i + D_j + CD_{ij} + B_{k(ij)} + H_l + HC_{il} + HD_{jl} + HCD_{ijl} + HB_{lk(ij)} + E_{m(ijkl)}, \quad (1)$$

in which $Y_{ijklm}$ = value obtained from a given test, $U$ = true mean value for the population; $C_i$ = between geographic areas true effect; $D_j$ = topographic effect; $B_{k(ij)}$ = between boring true effect in the *C-D* cells; $H_l$ = between horizons true effect; and $E_{m(ijkl)}$ = error true effect of repeat measurements. The other terms denote interactions between the main effects while the subscripts are variables representing the levels of effort for the various effects.

Based on the analysis of variance model proposed, the effect of the factors on the engineering property of interest was noted and evaluated quantitatively. From a quantitative estimate of the variance components the number of samples necessary to obtain a given degree of accuracy was determined and presented in graphical form.

The engineering soil properties investigated in this study were liquid limit, plasticity index, plastic limit, optimum density, optimum moisture content, R–value (from Hveem Stabilometer test), swelling pressure, C.B.R., per cent finer than 0.074 mm, per cent finer than 0.002 mm, and unconfined compressive strength. The results of this study were displayed in graphical form.

In so far as variability of soil properties is concerned, the ultimate consideration should obviously be concerned with the effect of this variability upon the structure (highway, embankment, bridge, dam, or building) which is to be constructed on the soil in question. The researchers at Purdue University have pointed out that soil variability is a function of the property being measured. In some cases high variability was encountered with strength values, but on the other hand the soil variability was much less when considering optimum moisture content. In this connection it is noted that the authors have dealt only with the *B* horizon of the soil deposits. In the case of highways (at least high type highways) the *B* horizon may be the least significant of the soils encountered. This is brought about by the grade and alignment requirements which result in the *B* horizon being encountered only at the transitions between cut and fills. Thus, the variability of the parent material itself is considered to be the most significant factor.

As a result of the work at Purdue University, it has been concluded that the use of one-point compaction and Atterberg limit tests is justified. Further, it has been concluded that, for pavements, design on the basis of soil classification or some other simple procedure is justified, because of the large variation in design thicknesses which might occur within a given area due to the variation in the parameter which forms the basis for design. In contrast, the performance of pavements over given soil areas strongly indicates that thickness and quality design of pavements should be constant for a given soil map unit, regardless of the variability of test properties within the unit as determined by present testing techniques. The variability of the aforementioned test properties strongly suggested that a statistical approach to pavement design should be considered.

Coupled with the above, and still of great significance, is the matter of quality control during construction. This latter factor, coupled with the variability of natural deposits, suggests that great refinement in the exploration phase is not justified.

The writers are of the opinion that for those areas in which soil maps are not available, an approach using the analysis of variance given above will provide the maximum amount of information per unit cost.

REFERENCE

HAMPTON, D., E. J. YODER, and I. W. BURR (1962). Variability of Brookston and Crosby Soils. *Proc. 41st Annual Meeting, Highway Research Board.*

S. J. BUCHANAN (U.S.A.)

*(Oral contribution not subsequently submitted for publication.)*

D. CRONEY (Great Britain)

The General Reporter has spoken of the need for a more highly developed theoretical approach to the problem of frost heave. Progress has been rather slow in this field. R. K. Schofield in 1935 developed an equation relating soil suction to freezing point depression and Mr. Coleman and the author used this method in the 1950's to escribe to any subzero temperature gradient an associated equilibrium suction gradient. In turn, if the appropriate relation between suction and moisture content were known, the equilibrium suction gradient could be expressed as an unfrozen moisture gradient.

The problem of estimating the rate of magnitude of frost heave is, however, essentially a permeability problem. The permeability in the freezing front itself probably controls the rate of heave.

Although attempts are being made, I believe, in Canada and elsewhere to study the unsaturated permeability of partially frozen soils, the results are more likely to be of interest from the light they throw on the mechanisms involved than from the quantitative point of view.

We have been studying the problem in England in a semi-empirical manner using typical road pavement sections laid into the floor of a large refrigerated room in which any required temperature gradient can be obtained. The soil under study extends to a depth of 6 ft and provision is made for adjustment of water-table level within that range. The sections are fully instrumented for temperature, pore water pressure, and heave, and provision is also made for measuring the change of strength of the soil and pavement layers during and after freezing and for metering the amount of water drawn into the freezing zone during any time period.

The results of the first 4 years' work with this facility will be published shortly. They will, I hope, provide detailed experimental data to which any subsequent theoretical work can be applied.

CHAIRMAN KÉRISEL

Je remercie les orateurs et je demande maintenant au Rapporteur général de bien vouloir résumer ses conclusions.

GENERAL REPORTER JENNINGS

Mr. Chairman, Gentlemen, may I breathe a great sigh of relief. I am so pleased to find at the end of this session that I am still in a whole state and have not been cut to pieces by some irate author who feels that I have misinterpreted his paper in my General Report.

The discussion, both from the Panel and from the contributors, has been exactly what I had hoped for, namely it has given emphasis to aspects of engineering geology, particularly in so far as these relate to the structural features of the soil. Professor Skempton's comments are of the greatest importance: we need to look at the structure of our soils, and to pay particular attention to the physical planes of weakness which so largely determine soil behaviour. He was right to point out that these thoughts do not represent the overthrow of what we previously knew or were doing, but only mean that we must differentiate in our interpretation of tests and predictions of behaviour between soils which are structured and soils which are unstructured. This is very important.

I do hope we will see these thoughts extended to cover other problems of engineering geology of soils and rocks. These thoughts must take into account the whole history of the material, as closely as we are able to estimate it. Naturally many of these factors cannot be measured or stated precisely and at times we will be forced to return to the classical geologists' approach. There is a need sometimes simply to put our feet on our desks and think.

There can be no doubt in anybody's mind of the importance of physical chemistry as it applies to the soil. I hope Dr. Rosenqvist and other physical chemists will return home with the idea that we, as engineers, believe that this is very important. But we have to wait for them to show us how these studies will help us towards a better understanding of the likely behaviour of soils. We should not ask the physical chemists to come forward with detailed predictions: what they should aim to provide is a general background understanding of our problems. I would like to ask the physical chemists of our Conference to take advantage of the offer made earlier by Dr. Legget, to spend some of their evenings using the rooms which have been made available for the small meetings which the organizers hope will arise during the Conference. If I may be so presumptuous, may I ask someone to take the lead in this, and look to Dr. Rosenqvist. I am aware that many in this audience would like to participate in those discussions. Can we please ask our physical chemists to try to come to agreement amongst themselves, at least on principles, and also can they please seek practical implications of their work. If they will do this then we, as engineers, will spend our midnight hours understanding what they are doing.

I come next to the problems of partly saturated soils which have been greatly emphasized in the discussions; I estimate that about one-half of our time has been concerned with partly saturated soils. I would like to re-emphasize that the bulk of expenditure on construction is concerned with work which lies within the range of partly saturated soil. This is a subject which deserves greater attention from all countries of the world including those with a so-called high water table.

Finally, Sir, if I may pick up a point made by Professor George Zeitlen, we meet at this Conference for the purpose of studying a subject which will be of use to us for constructing engineering work. Our object is the end product; the work and the study in between is merely the means of reaching this point and is not the end in itself. Let us never forget this.

(*The remarks of the General Reporter for Session 1 presented to the Closing Session appear on pp. 591–92*)

CHAIRMAN KÉRISEL

Messieurs, la première session est maintenant terminée, mais avant de lever la séance je voudrais remercier particulièrement nos quatre distingués panelistes de l'exposé de leur contribution, et surtout marquer le caractère de clarté, le caractère très constructif des conclusions de notre Rapporteur général que je remercie tout particulièrement pour le merveilleux travail qu'il a produit.

## WRITTEN CONTRIBUTIONS CONTRIBUTIONS ÉCRITES

M. APPENDINO (Italy)

I would like to illustrate the results of a comparison made between the settlements calculated with the aid of the Ménard pressuremeter and those measured on two loading tests using two rigid loading plates with unusual sizes of 1.50 and 3.00 m (Figs. 4 and 5). As can be observed from the results reported in Fig. 6 good agreement exists in this case between the calculated and measured settlements. This test is part of the study of foundations for the two 600-MW steam power units presently under construction in Italy at La Spezia.

The soil to be studied is formed from metamorphic rock consisting of alternate layers of quartzites and sericitic schists (typical formations found in the Apennines) extremely corrugated and decomposed. In particular, the schists are decomposed in such a way that they are now transformed into soils ranging from gravel to clayey silts, even though the stratification of the original rock is maintained. The decomposition is not limited to the upper layers, but reaches a depth of several meters (Fig. 7).

While the compactness and the heavy preconsolidation of the soil led us to believe that it might have a considerable bearing capacity, we were concerned that the differential settlements which occurred might be larger than those permitted by the high rigidity of boiler structures and turbo-

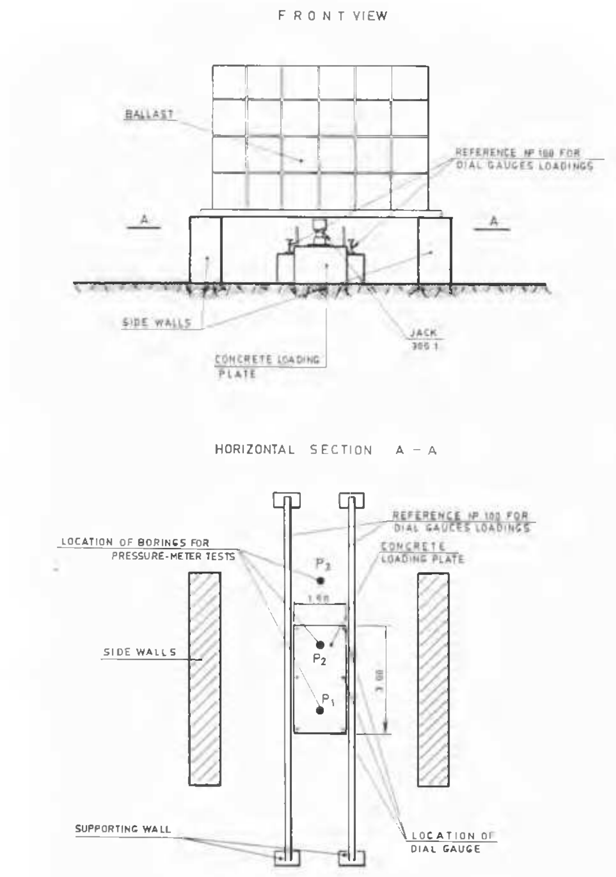



FIG. 4. Loading test equipment.

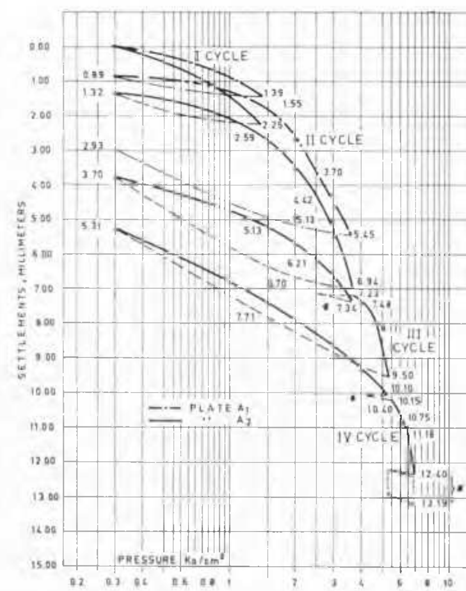



FIG. 5. Vertical settlements *versus* pressure. Alternate load and unloading cycles.

generator foundations. The differential settlements are caused essentially by the considerable heterogeneity of the soil in

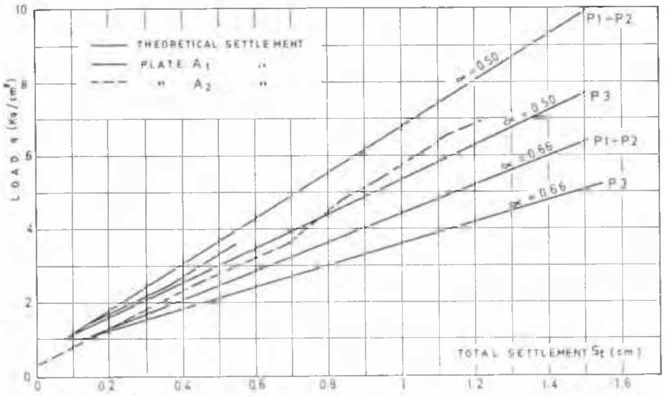



FIG. 6. Comparison between theoretical settlements ($S_t$) calculated according to the Ménard formula and those measured by loading tests.

$$S_t = [(1 + \mu)(q - \gamma D)R_0/3E_i][\lambda(0.5B/R_0)^\alpha + 0.5Bq/4.5E_{ed}$$

where $\mu$ = Poisson ratio, $q$ = load/sq.cm., $B$ = foundation width, $R_0$ = factor, $\gamma$ = soil weight per unit volume, $D$ = excavation depth, $\lambda$ = shape factor, $\alpha$ = factor depending on soil nature ($\alpha$ = 0.50 for silty and sandy soil, 0.66 for clayey soils), $E_i$ = average of pressuremeter moduli, $E_{ed}$ = average of oedometric moduli (herein assumed equal to $E_i$), $P_1$, $P_2$ $P_3$ = borings made as shown in Fig. 4 in the loading plate $A_2$ area.

both the vertical and horizontal directions, a heterogeneity due to the chaotic stratigraphy and the alternate strata of quartzites, which are not very compressible, with schists, which are much more compressible. Because of this we can expect high relative settlements even if the absolute value will not be large.

As it was impossible to obtain undisturbed samples for laboratory œdometric compression tests, calculations were based on a series of *in-situ* tests using the Ménard pressuremeter. The number of these tests was sufficient to give a complete knowledge of the heterogeneity throughout the whole foundation area down to a depth of about 30 m. To check the settlements calculated from the data supplied by the pressuremeter, we performed the two load tests reported above. The sizes of the plates used for these two tests were sufficiently large to minimize the variation caused by the local heterogeneity derived from the stratification. From the collected data, the maximum allowable load of 3.00 kg/sq.cm. on the soil was established to obtain differential settlements lower than the maximum allowable limit of 0.3 per cent.

The construction of the two power plants is presently under way and the applied foundation loads at some areas are 50 per cent of the total. The absolute settlements observed up to now, four of which are shown on Fig. 8, have a lower value than anticipated, but the differential settlements are of the same order of magnitude as calculated. Fig. 5 shows the curves of the settlements obtained versus time at four representative points.

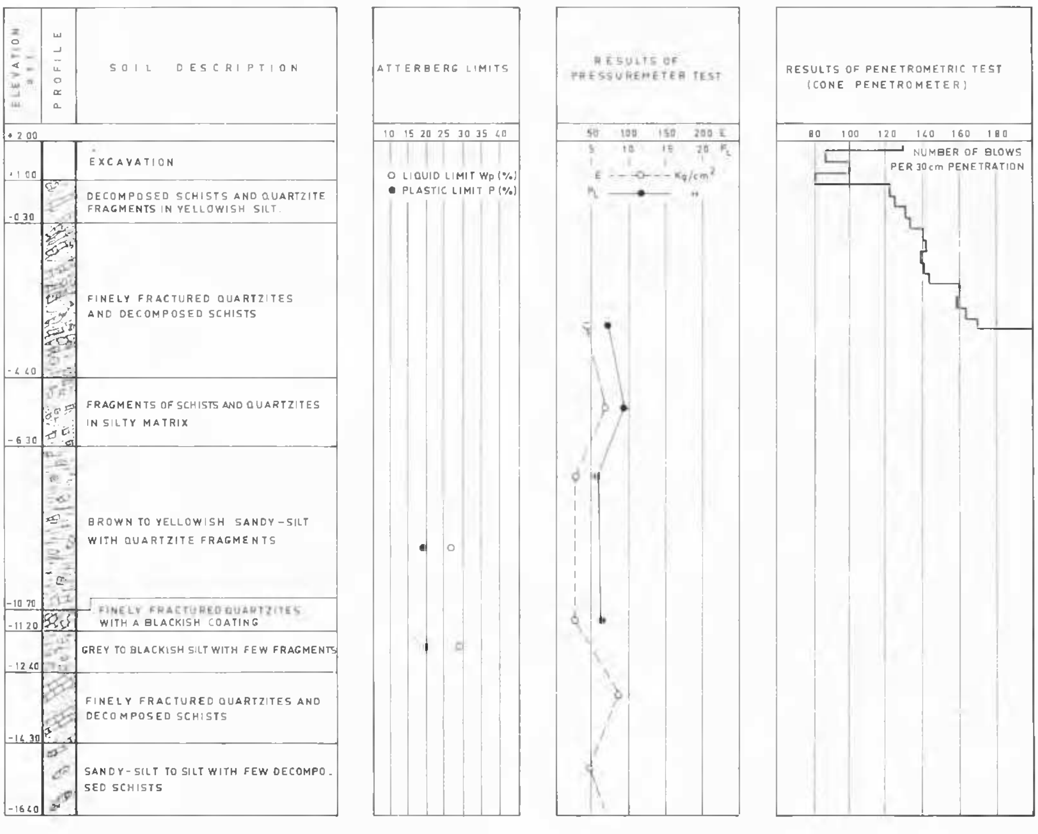



FIG. 7. Typical stratigraphy and geotechnical details.

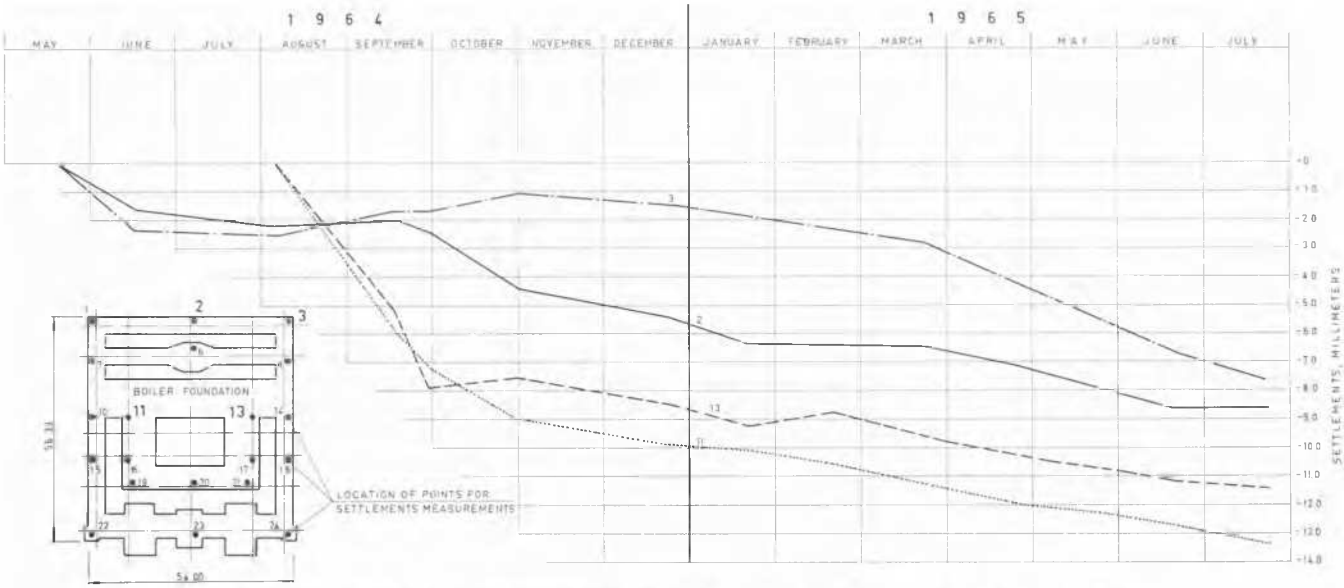



FIG. 8. Curves of settlements *versus* time at four representative points in boiler foundation.

V. I. FERRONSKY and G. K. BONDARIK (U.S.S.R.)

In the reports by *Shashkov (4/19)* and others which discuss the application of sounding to the study of soil building properties, solutions of individual problems using this method are described. However, the efficiency and scope of the application of sounding can be considerably extended if it is used in conjunction with other methods.

On the basis of static soundings we have worked out a complex of penetration and logging methods that allows detailed and reliable information to be obtained on the profile and properties of loose deposits without boring, sampling, and laboratory tests. The complex includes: static sounding with near-bottom tensio-electric transducers of frontal resistance and friction of the soil, gamma-ray logging, gamma-gamma logging, neutron-neutron logging, and a sampler for taking soil samples from the required depth.

Experimental tests of the complex have been carried out using the SUGP-10 type self-propelled apparatus designed to sink rods and truck-mounted recording equipment (Fig. 9). The apparatus has a double-action hydraulic cylinder with an automatic reverse device and a hollow bar for passing rods. This system develops an axial pressure of up to 10 tons which is of sufficient magnitude to sink rods with diameters of 50 mm to a depth of 25 m in loose deposits. The speed of rod sinking can be regulated within a wide range of 0 to 6 m/min that ensures the very high efficiency of the installation. The sounding transducers, 60 mm in diameter, for recording various indices of physico-mechanical properties of soils are screwed on the end of the first rod. The cable communicating channel is mounted in the rods. When the rods are joined, the cable is connected by means of plug-and-sockets. The information transmitted from the transducers while the probes are being sunk is recorded continuously by the apparatus located on the ground surface.

It is common knowledge that the physico-mechanical properties of soils are estimated from the analysis of samples taken in a borehole or other opening. As the properties of layers differing in lithology vary within rather wide limits both in strike and depth, the results of analysis of soil samples taken at individual points at certain intervals of depth may be of an accidental nature. Obtaining continuous diagrams of changes in rock properties throughout the whole depth of the profile under investigation, when the measuring transducers are sunk statically, makes it possible to estimate objectively, and more accurately, the degree of the uniformity and persistence of rocks using the given property as measured *in situ*. In this case, the recorded properties are therefore much more representative.

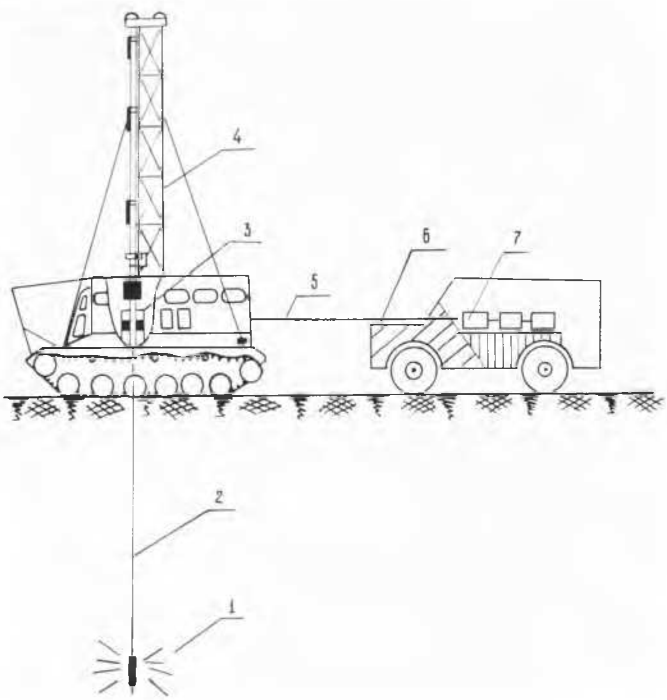



FIG. 9. SUGP-10 apparatus for rod sinking with recording equipment. 1, sounding transducer; 2, rod, 3, hydraulic cylinder; 4, mast for supporting rods; 5, communicating channel; 6, apparatus station; 7, recording apparatus control channel.

As an example, typical penetration and logging diagrams obtained during the static sinking of sounding transducers are shown in Fig. 10. It is interesting to note that sands, sandy loams, and clayey loams are accurately distinguished

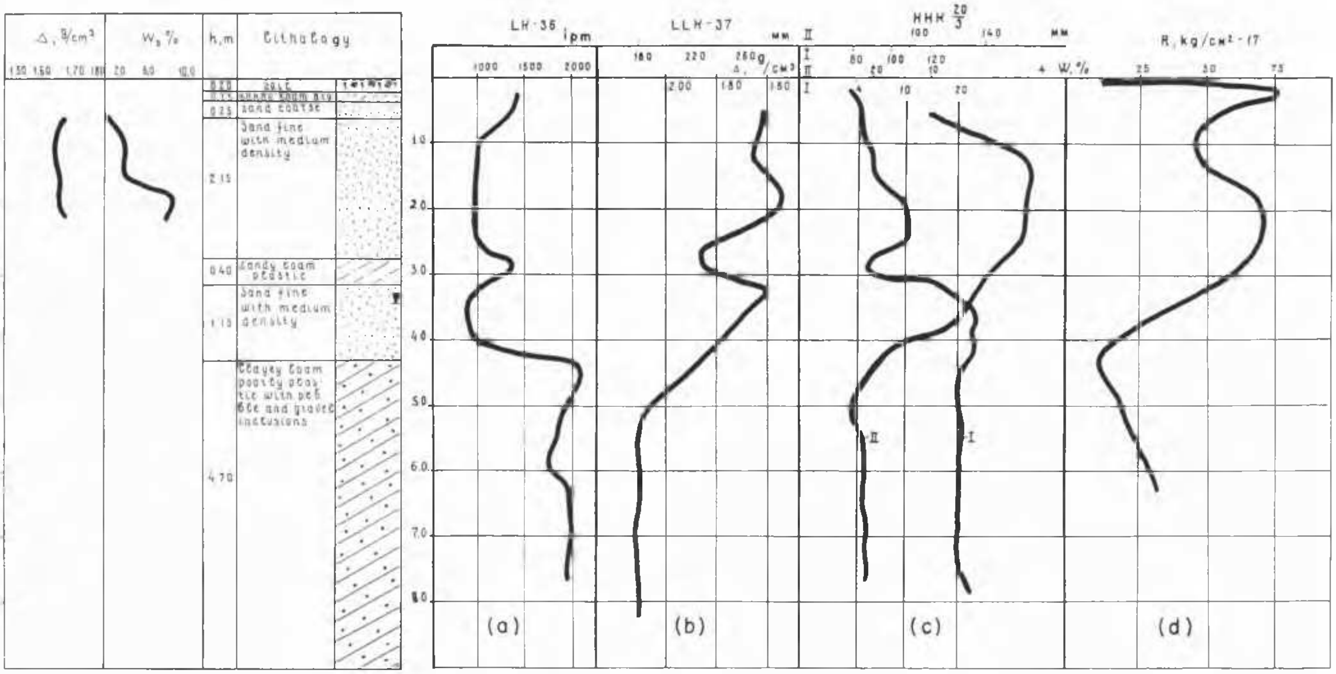
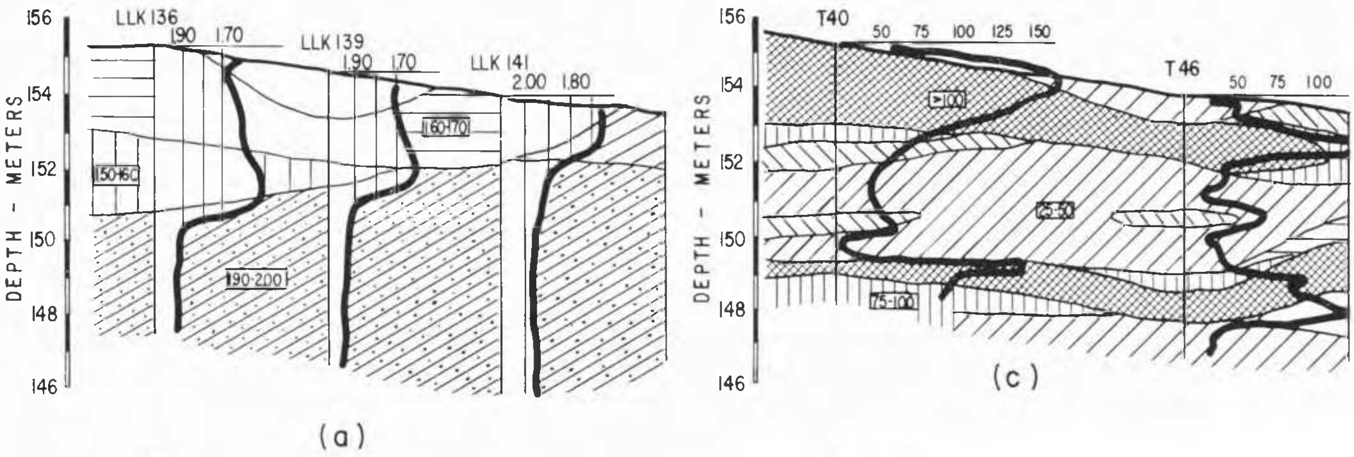



FIG. 10. Typical recording diagram for sandy-clayey glacial profile. (a) Gamma-ray logging diagram (ipm = impulses per minute). (b) Gamma-gamma logging diagram. (c) Neutron-neutron logging diagram: (i) probe length 5 cm; (ii) probe length 45 cm. (d) Probe sinking frontal resistance diagram.





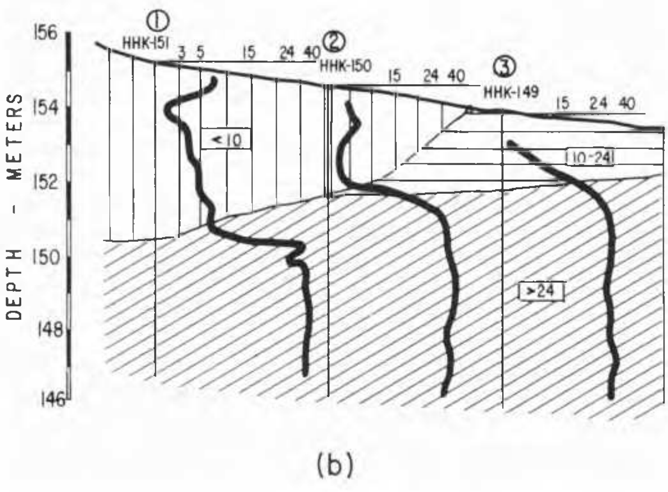



FIG. 11. Distribution of properties in soil profile. (a) Unit weight in g/cu.cm. (LLK represents gamma-gamma logging. (b) Water content in per cent (HHK represents neutron-neutron logging). (c) Cone sinking frontal resistance in kg/sq.cm. (T represents tensiogram).

on the basis of the natural gamma-background diagram. Using gamma-gamma and neutron-neutron loggings applied for the first time under the conditions of static sinking of logging probes without boring, it proved to be possible to interpret quantitatively, with a high degree of accuracy, the unit weight and moisture content of soils throughout the profile. From the diagram of the sounding point sinking resistance, which is recorded with a near-bottom tensio-electric transducer, and a qualitative estimate of strength properties, an attempt is made to determine the modulus of soil deformation by reducing static logging to solving the Kelvin problem of a force applied within space.

To establish the lithological profile of sandy-clayey deposits, the fundamentals of the complex interpretation of data on the cone frontal resistance, soil friction on the side surface of the sounding transducer, and gamma-gamma, neutron-neutron, and gamma-ray loggings have been worked out. Lithological layers are distinguished by their physico-mechanical properties: unit weight, moisture content, strength, and natural radioactivity.

The complex interpretation of diagrams with recordings of various indices of soil properties allows us to establish accurately and objectively the lithological profile of rocks without sampling. Geological profiles showing the distribution of rock properties in the profile plane can be constructed from penetration and logging diagrams.

As an example, the distribution of the unit weight, moisture content, and cone resistance presented in Fig. 11 are constructed from ten penetration and logging diagrams that were obtained along a line at a small test area near Moscow. These profiles of rock properties can serve as a sound basis for designing conventional footings and deep foundations.

The complex under consideration permits solving of the following engineering geological problems: subdivision of the profile of deposits using their strength and physical properties; estimation of the degree of rock uniformity and persistence using various indices of properties and data of thickness and strike; determination of the ground water table; determination of the unit weight and moisture content of rocks throughout the profile; estimation of rock strength properties. By using the method of static sinking of sounding transducers it is also possible, in principle, to conduct soil shearing vane tests, to carry out pressiometric tests, and to conduct activation analysis to estimate the chemical content of the rocks.

Application of penetration and logging methods on the basis of static sinking of sounding transducers in soil makes it possible to abandon such labour-consuming and expensive work as boring and hole sinking and use field methods to estimate the physico-mechanical properties of loose deposits *in situ* while investigating building sites and conducting engineering geological surveys at different scales.

E. Hannigan (U.S.A.)

*Reséndiz (1/22)* has presented an interesting theory of the nature of the adsorbed water in a clay water system. He has postulated that all water has an ice-like structure. In bulk water the thermal vibrations are such that the rigidity of the structure is destroyed. Near clay surfaces, a certain rigidity exists in the water structure and the mass density increases due to the presence of interstitial molecules.

It is well known that a certain activation energy (heat of fusion) is required to change water from its solid structure (ice) to bulk water at a given temperature. From this it can be postulated that the energy released during the formation of ice is caused by the re-orientation of the water molecules from a random structure to the lower energy state of the ice-like structure. If the adsorbed water in a clay water system were in an ice-like structure as suggested by the author it would seem reasonable to expect all pore water to freeze at 0 C. It has been found, however, that even at temperatures substantially below 0 C not all of the pore water is frozen (Lovell, 1958). This is taken to mean that some of the water is in an ordered phase (but different from ice) and exhibits a higher required activation energy to re-orient into the form.

The author's postulated increase in mass density of water near a clay surface has been seriously questioned by Low and Anderson (1958), who measured pore water specific volumes of 1.02–1.03 cu.cm./g in bentonitic soil.

Martin (1960) has discussed the following two major hypotheses offered for the nature of the adsorbed water: (1) a solid-like substance; (2) a two-dimensional fluid. Martin believes that both hypotheses adequately explain the bulk of experimental data available.

The hypothesis of a solid-like substance for the pore water means only that this water is more regularly oriented than bulk water. Each individual bond in such a system would be stronger than in bulk water. It must then be assumed that such water would exhibit a greater resistance to both normal forces and shear forces than normal water. Thus, the term, solid-like, is applied to the water.

The effect of the electrical fields is viewed differently in the two-dimensional fluid hypothesis. It is theorized that the bonds between water molecules are stronger than in bulk water but there are a smaller number of bonds. This decrease in the number of bonds is attributed to the restraint placed on the adsorbed water by the clay surface. This is interpreted by Martin to mean that there would be an increased resistance to normal forces but no increase, and possibly even a decrease, in resistance to shear.

Reséndiz also states that, "if water content is of the order of the liquid limit, the curve of strength *versus* water content is a straight line on semi-logarithmic paper." It is known that the void ratio *versus* the log of pressure relationship is linear over a relatively large range of water content. The void ratio is directly proportional to the water content in saturated soils. The undrained shear strength of saturated clays has been found to be proportional to the consolidating pressure. Thus, it would be expected that water content and the log of shear strength would show a linear variation not only near the liquid limit but over a large range of water contents. This has been borne out by laboratory testing (Leonards, 1962).

An approach different than the one taken by Reséndiz will be examined to explain the above phenomena. Rather than considering the adsorbed water to be of major importance, it will be considered merely a medium for transmitting the electrical forces present in a clay water system (Tan, 1957). The Coulombic forces between two charged plates (clay mineral platelets) are indirectly proportional to the dielectric constant of the transmitting medium. The author points out that the dielectric constant for adsorbed water has been observed to be as low as three near the clay surface whereas it is 80 for bulk water. The percentage of water which is oriented would logically increase at lower water contents. In such a case the dielectric constant of the medium separating the clay particles would decrease correspondingly. As the water content decreases the effective dielectric constant decreases and thus the Coulombic forces increase, with a corresponding greater bond between particles and improved shear strength.

If the foregoing hypothesis is correct, the role of the adsorbed water is minimized and similar behaviour should result no matter what the pore fluid may be, even if it is non-polar. Behaviour such as secondary time effects has been observed when the pore water was replaced by carbon tetrachloride (Leonards and Girault, 1961).

Recently, attempts have been made to replace the pore water with air without particle disturbance (Griffin, 1965; Hannigan, 1964; Leonards and Altschaeffl, 1964), to examine the mechanism of shear, and also to evaluate the effect of water. While such work is still preliminary in nature, it has been found that secondary time effects do exist, even in the absence of water, indicating even further that the role of the adsorbed water is only that of a medium for transmitting the forces of electrical fields (Leonards and Altschaeffl, 1964). Also, the replacement of water with air results in a higher shear strength, at the same void ratio. This can be attributed to the decrease in the dielectric constant of the pore fluid, from the relatively large value for water to unity for air.


REFERENCES

GRIFFIN, W. H. (1965). A study of strength relationships of critical-region-dehydrated kaolin clay sediments. M.S. thesis, Purdue University, School of Civil Engineering.

HANNIGAN, E. (1964). A critical region apparatus for the dehydration of clay soils without particle disturbance. M.S. thesis, Purdue University, School of Civil Engineering.

LEONARDS, G. A. (1962). *Foundation engineering*, p. 210. New York, McGraw-Hill.

LEONARDS, G. A., and A. G. ALTSCHAEFFL (1964). Compressibility of clay. *Proc. A.S.C.E.*, Vol. 90, SM5, Sept., 1964.

LEONARDS, G. A., and P. GIRAULT (1961). A study of the one-dimensional consolidation test. *Proc. Fifth International Conference Soil Mechanics and Foundation Engineering*, Vol. 1.

LOVELL, C. W., JR. (1958). *Temperature effects on phase composition and strength of partially frozen soil.* Civil Engineering Reprint, No. 141, Purdue University.

LOW, P. F., and D. M. ANDERSON (1958). The partial specific volume of water in bentonite suspension. *Proc. Soil Science Society America*, Vol. 22.

MARTIN, R. T. (1960). Adsorbed water on clay: A review. *Proc. Ninth National Conference Clays and Clay Minerals.*

TAN, T. K. (1957). *Structure mechanics of clays.* Harbin, China, Academia Sinica, Soil Mechanics Laboratory, Institute of Civil Engineering and Architecture.


L. I. KORZHENKO and V. B. SHWETS (U.S.S.R.)

A wide variety of eluvial soils, the product of *in-situ* weathering of rocky materials, and the variation in their building properties depend upon the type and the structure of a parent rock, on the types and some features of weathering processes taking place, and upon the degree and homogeneity of weathering. For several years the authors of this discussion, together with their scientific teams, carried out investigations of rocky soils in the Urals region. These investigations allowed determination of a number of the specific features in the formation of eluvial soils which distinguish them from ordinarily transported soils (Korzhenko, 1963; Shwets, 1964). A norm has been worked out which is based on the results of the investigations carried out (*Specifications*, 1964).

In the course of the investigations it was found that using current soil nomenclature for eluvial soils results, in a number of cases, in introducing definitions of terms and establishing a complex of investigations not inherent in the real nature of these soils. Accordingly, mistakes arise in assumptions of the most important properties such as compressibility, water permeability, strength, softening, and swelling. Particularly important discrepancies are observed in cases where weathering causes a combination of physical disintegration and chemical decomposition in the parent rock. A schematic diagram illustrating the formation of eluvial soils by different weathering processes and by their combination, is given in Fig. 12.

The specific features of such soils resulting from the processes of their formation made it necessary to develop a special classification on the basis of which the building nomenclature was drawn up (Kovzhenko, 1963; *Specifications*, 1964). The nomenclature is based on a quantitative evaluation of the degree of weathering of a parent rock determined by the degree of its disintegration and its strength. Additional nomenclature characteristics (the coefficient of structural strength $K_{sst}$, specific shear strength $\tau_0$, and the debris weathering ratio $K_w$) in combination with common characteristics (compression strength in a water saturated state and softening) allow us to clearly distinguish rocky soils from clayey ones, clayey soils from coarse debris ones, and coarse debris soils from rocky ones. The nomenclature developed for eluvial soils is given in Table I.

In the nomenclature two additional soils are distinguished —rotten stone and "suprolite." The qualitative difference of

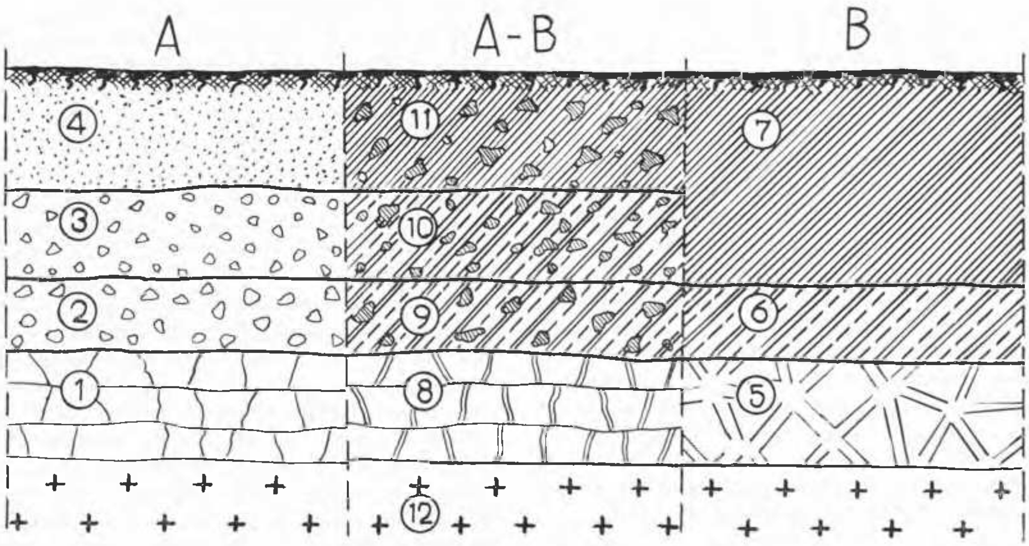



FIG. 12. Scheme of rock weathering processes. A, Physical weathering: (1) broken-up rock structure; (2) rubble; (3) gruss; (4) dust-like sand. B, Chemical weathering: (5) rotten material; (6) suprolite; (7) clayey soil (sandy loam, loam, clay). A–B, Combination of physical and chemical weathering: (8) rotten broken-up rock structure; (9) suprolite with rotten rubble; (10) suprolite with rotten gruss; (11) clayey soil with suprolite rubble and gruss; (12) rock.

TABLE I. TYPES OF ELUVIAL SOILS

| Soils | Distinguishing characteristics |
|---|---|
| | *Rocky soils* |
| 1. broken-up structure rock, including shales | Bedding in the form of displaced structures like dry laying. Temporary compression strength in a water saturated state is more than 50 kg/sq. cm. Softening ratio $K_s > 0.75$. |
| 2. rotten rock | Temporary compression strength in a saturated state is less than 50 kg/sq. cm. but more than 10 kg/sq. cm. Softening ratio $K_s \leqslant 0.75$. |
| including a rotten broken-up structure rock | Bedding in the form of undisplaced structures. |
| rubble-gruss rotten rock | Bedding in the form of accumulations of debris of different sizes with some remaining cohesion. |
| clayey rotten material | Bedding in the form of massive deposits. |
| | *Coarse debris* |
| 1. grussy, grubbly, rubble-grussy, with solid debris | Debris is not crushed and reduced in size by hand and does not soften in water. Weathering ratio $K_w < 0.5$. |
| 2. grussy, rubbly, rubble-grussy with rotten debris | Debris may be crushed, but not reduced in size by hand and partly softens in water. Weathering ratio $0.5 \leqslant K_w \leqslant 0.75$. |
| | *Clayey soils* |
| 1. weak soils-clays, loams, sandy loams. | Specific shear strength from uni-axial compression $\tau_0 \leqslant 1$ kg/sq.cm. The coefficient of structural strength $K_{sst} \leqslant 1.25$. |
| 2. solid soils—clayey, loamy sandy loamy suprolite | Specific shear strength from uni-axial compression $\tau_0 \leqslant 1$ kg/sq.cm. The coefficient of structural strength $K_{sst} \leqslant 1.25$. Debris may be reduced in size by hand and softens in water. |
| 3. gruss-rubbly with suprolites debris | Debris may be reduced in size by hand and softens in water. Weathering ratio $K_w > 0.75$. |

these soils is that a rotten material is a weakened rocky soil in which some of the less stable minerals are transformed into clayey products under the action of slight chemical decomposition, whereas suprolite is a product of more profound chemical transformation in which a considerable quantity of minerals is displaced by clayey materials, but where some crystal binding agents of chemically stable minerals have partly remained among separate grains. The presence of such braces in suprolites, forming a peculiar rigid honeycomb structure, makes them different from the usual clayey soil with a similar characteristic composition. The quantitative difference between a rotten material and suprolite is determined according to the nomenclature characteristics.

The nomenclature developed does not include sandy eluvial soils as they are relatively scarce. Designations of all types of eluvial soils must be supplemented by mentioning the type of parent rocky material; for coarse debris soils it is necessary to include a description of particle shape as well.

The weathering ratio $K_w$ and the coefficient of structural strength $K_{sst}$, are determined according to special procedures and by special apparatus (Korzhenko, 1963; Shwets, 1964; *Specifications*, 1964). On the basis of common terminology this nomenclature presents an opportunity for creating a better understanding among surveyors, designers, and builders in establishing the necessary investigations to evaluate correctly the natural properties of eluvial soils in designing, building, and working, and to make fuller use of their bearing capacity for structure foundations.

REFERENCES

——— (1964). *Specifications on characteristic features of engineering-geological surveying and designing of footings in eluvial soils.* Moscow, Publishing House of Building Literature.

KORZHENKO, L. I. (1963). *Footings and foundations under conditions of the Urals region.* Sverdlovsk Publishing House.

SHWETS, V. B. (1964). *Eluvial soils as structure footings.* Moscow. Publishing House of Building Literature.

F. B. DE MELLO (Brazil)

The ingenious development of the Dutch cone penetration method put forward by *Begemann (1/4)* uses a comparison of cone penetration resistances with local friction measurements made by means of a friction sleeve. Begemann purports to furnish an approximate identification of soil layers penetrated on the strength of a demonstration that the higher the angle of friction $\phi'$, presumed directly and intimately correlated with the soil texture, the greater will be the ratio between the two values. Indeed such a method would appear to fill a deeply felt deficiency inherent in the use of the cone penetrometer for subsoil exploration, and therefore runs the risk of coming into much wider and undiscriminating use than really intended or warranted. Hence I cannot help but deplore a concept which may come to cloak a practice, inherently unsound and unacceptable, whereby preliminary subsoil explorations may once again be used in place of close and careful inspection and identification of truly representative soil samples. The long and difficult struggle is still vividly remembered, a struggle during which wash-sampling techniques were rejected by the pioneers of modern soil mechanics, while searching for a subsoil exploration procedure which would yield adequate samples for identification of the soil type, and also permit an empirical measure of density and consistency. The procedure arrived at was the use of dry-sample boring combined with the measurement of dynamic penetration resistances.

The reader should realize that whereas the author suggests the present method for *preliminary* identification of soil layers, the limitations in principle and practice involved would merely allow us to accept its possible applicability for the *approximate* identification of soil layers in simple subsoil profiles that are already well-known in a preliminary fashion and where only the location of the main soil types remains to be established. One must guard against extending the intention of the author to encompass tacit acceptance of the use of the proposed method for the preliminary reconnaissance of a subsoil at a given site. The principle of basing any subsoil reconnaissance on methods that dispense with the visual and tactile identification of representative soil samples is totally unacceptable and must never be condoned. In the cases that served as a background for the author's interesting development, the indispensable preliminary subsoil reconnaissance is tacitly covered by the well-known upper subsoil profile that prevails in Holland which has been extensively investigated and described, and is geologically extremely simple.

The correct use of the Dutch penetrometer, whether or not it is improved with the local friction sleeve, continues to be as a complement to preliminary subsoil reconnaissance, at sites where the subsoil profile is comprised of relatively simple soil types, and more detail is required of *in-situ* strength and bearing capacity parameters. Since it is always annoying to interpret such parameters without recourse to any real knowledge of the strata tested, the author's proposal may be hailed as an interesting advance, reducing the uncertainties in the formulation of the probable soil type involved at each point. Hitherto the probable soil types were assumed on the basis of interpolation between dry-sample reconnaissance borings. Henceforth such assumptions may be less subject to statistical error, by using the indirect method proposed by the author.

Considering the strictly empirical nature of the correlations established by the author, it is rather unfortunate that he has not seen fit to analyse and present his data in a statistical fashion, so that average values and confidence limits may be established and evaluated. The data represented in Fig. 1, which are intended to furnish satisfactory evidence of a good correlation between the three methods of evaluating apparent cohesion in saturated clay layers ($\phi' = 0$), may stand closer scrutiny, especially if the number of cases and range of consistencies covered is extended, and if submitted to statistical correlation. At present, the correlations are only fair. To begin with, since the derivation of Eq (d) in the paper, $c_u = S/_{14}$, is recognizedly somewhat loose, a statistical correlation might well suggest a correction factor for a better fit on an average; for instance $c_u = S/_{17}$ appears to give a better fit of the data listed. Moreover, since the data are rather scanty, the scatter of approximately $\pm 40$ to 60 per cent around the average values may indicate too broad a confidence limit to stir immediate enthusiasm.

In short, even if the experimental data available to the author do warrant the presentation of a graph such as that of Fig. 2, none of the straight lines relating cone resistance, local friction, and percentage of soil particles $<16\mu$, could be proposed or interpreted as furnishing any more than an average relationship. Such average empirical relationships should always be accompanied, in my opinion, by graphs clearly indicating the plotted points and the statistically computed confidence limits that apply to each correlation. Presumably the data summarized in Fig. 5 constitute one of the approximately 250 points scattered over the Netherlands, collected by the author as field and laboratory evidence in support of Figs. 2, 3, and 4. Since there are nine sets of data for the desired correlations on grain size in the borings represented in Fig. 5, it is presumed that sufficient sets of data are available for interesting statistical correlations.

It is hoped that the author may see fit to furnish such data, and that the profession will be very cautious in employing the method proposed, with due regard not only to the limitations emphasized within the paper itself, but also to the objections in principle and to the empirical confidence limits as discussed briefly herein.

Y. NISHIDA (Japan)

*Arnold (1/3)* presents interesting data for the permeability of clay with a high void ratio. He shows the relationship of $\log_{10} k \propto \log_{10} e$, referring to its liquidity index. However, the better relationship expressed by $e = a + b \log_{10} k$ is found generally in many clays in the ordinal void ratio, where $a$ and $b$ are coefficients. I would like to point out that the coefficients $a$ and $b$ in the above expression have a linear relationship with the plastic index of each clay. According to my experiments (Nishida, 1961), $a \doteqdot 0.085\ (I_P) + 2.0$; $b = a/_{10}$; $I_P = \%$, where $I_P$ is the plastic index. This expression is valid for a wide range of $I_P$ from 5 per cent to 60 per cent.

REFERENCE

NISHIDA, Y. (1961). *Die Bauingenieur,* Dec., 1961.

E. PASZYC-STĘPKOWSKA (Poland)

The phenomenological description of clay behaviour does not seem to be satisfactory any more, and attempts are being made to explain this behaviour in terms of physico-chemical theories. The paper by *Arnold* (1/3) is an example.

Forces are presented below that may be calculated according to physico-chemical theories. Some conclusions are in agreement with Arnold's considerations.

It was assumed that:

1. The main reason for swelling of clay and for the swelling pressure is the double-layer repulsion (Bolt and Miller, 1955; Norrish and Rausell-Colom, 1961). This repulsion $p_R$ may be calculated from:

$$p_R = 2nkT\,(\cosh Y_d - 1) \tag{1}$$

where $n$ = concentration expressed as number of ions per cu. cm. of the solution away from the particle surface where the electric potential is zero; $k = 1.38 \times 10^{-16}$ erg/molec. °K; $T$ = temperature in °K; and $Y_d$ is a dimensionless parameter of the electric potential in the middle between two parallel clay particles, and is an exponential function of the distance $2d$ between the particles.

2. Above a certain water content, the distance between clay particles is a linear function of the water content. Norrish (1954) proved this for Na-montmorillonite. During the shearing process the average effective distance, $d$, between clay particles may be calculated from:

$$\bar{d} = (W - W_a)/\bar{S}, \tag{2}$$

where $W$ = water content, $W_a$ = water content when the entire theoretical surface of the clay is covered by monomolecular water layer, $\bar{S}$ = specific surface effective in the shearing process.

3. Remembering Lambe's reasoning (1960) in somewhat changed form, assume that any effective stress $\bar{\sigma} = \sigma - u$ in saturated clays is divided into two components. One part of it $k\bar{\sigma}$ (where $k < 1$ and positive) is carried by the swelling pressure $p_s$, that is caused mainly by double-layer repulsion $p_R$. The other part $(1 - k)\,\bar{\sigma}$ is carried at mineral to mineral contact as $\bar{\sigma}_m$.

$$\bar{\sigma} = |p_s| + \bar{\sigma}_m = k\bar{\sigma} + (1 - k)\bar{\sigma}. \tag{3}$$

4. The frictional force is proportional to the effective stress normal to the shear plane and is equal to zero when the normal stress is zero.

In a previous work (Paszyc-Stępkowska, 1960) the influence of exchangeable cation upon the shear strength was found to be similar to the influence of exchangeable cation upon the double-layer repulsion. Thus, as a working hypothesis, it was assumed that double-layer repulsion, $p_R$, is a reason for the shear strength in bentonite, in accordance with assumption 3, that every effective stress in soil is partially carried by the swelling pressure.

The only unknown value in Eq 1 for $p_R$ was half the interparticle distance, $d$. According to the working hypothesis mentioned above the average value of $\bar{d}$, effective in the shearing process, was calculated on the assumption that in Na-bentonite at a water content of 130 per cent the shear strength determined as $\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_{max}$ is equal to $p_R$. It is probable that at a high water content no other force may cause the comparatively high shear strength (0.2 kg/sq.cm.).

The value of $\bar{d}$ thus calculated permitted the determination of $\bar{S}$ from Eq 2 as equal to $1.2 \times 10^6$ sq. cm./g (theoretical total surface of Na-montmorillonite is $7.8 \times 10^6$ sq.cm./g and thus the calculation of $\bar{d}$ for all other water contents was possible. The $\bar{S}$ value was assumed as constant for bentonite samples with the following exchangeable cations: Na, Na + Ca, Ca, Mg, Fe, Al, and H.

Within a certain water content range the shear strength determined was in good agreement with the calculated $p_R$, but at low water contents the determined shear strength was higher, and this could not be explained by experimental error. It was concluded that the working hypothesis is valid: that the double-layer repulsion is the main factor in the shear strength of saturated swelling clays, but that another force is also acting and is important at low water contents. Van der Waal's attraction $p_A$ was found to be of adequate magnitude:

$$p_A = A/48\pi[1/d^3 + 1/(d + \delta)^3 - 2/(d + \delta/2)^3], \tag{4}$$

where $A$ = constant assumed usually as $10^{-12}$ ergs, $d$ = half the interparticle distance assumed as equal to the $\bar{d}$ value used in $p_R$ calculations, $\delta$ = particle thickness taken as $100 A°$.

This attraction must be overcome when clay particles are forced to move relative to each other. It is the interaction between atoms or molecules of interacting particles as permanent, induced, or temporary dipoles and it acts independently of any other force present in the system and particularly

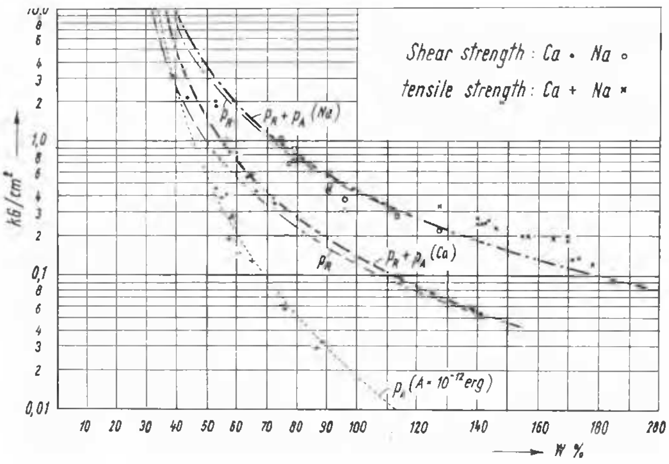



FIG. 13. Strength of bentonite—calculated and measured forces.

independently of double-layer repulsion, which has the character of osmotic pressure. Thus if both forces must be overcome in the shearing process, their sum may be correlated to the shear strength, though one is repulsion and the other is attraction. In Fig. 13 the calculated values of $p_A$, $p_R$, and $p_R + p_A$ are presented and compared with the measured shear strength and tensile strength of Na- and Ca-bentonites. There is a surprising agreement between the $p_R + p_A$ calculated results and the measured shear and tensile strength of Na-bentonite. Below a water content of 70 per cent there is a similar agreement for the shear strength of Ca-bentonite, but its tensile strength seems to be caused solely by van der Waal's attraction. Similar results were obtained for the other exchangeable cations mentioned above.

The conclusion is that there are serious indications that the deviator stress in swelling clays when no friction is present must overcome double-layer repulsion and van der Waal's attraction to cause shear. Tensile strength may be caused either by double-layer repulsion (Na-bentonite) or by van der Waal's attraction (Ca-bentonite).

The above conclusions were checked by shear strength measurements on triaxial test equipment of the Norwegian type with pore-pressure measurement at constant strain rate (2 per cent per hour; $k = 4 \times 10^{-7}$ cm/sec). Bentonite samples were compacted statically at various water contents ($S \cong 94$

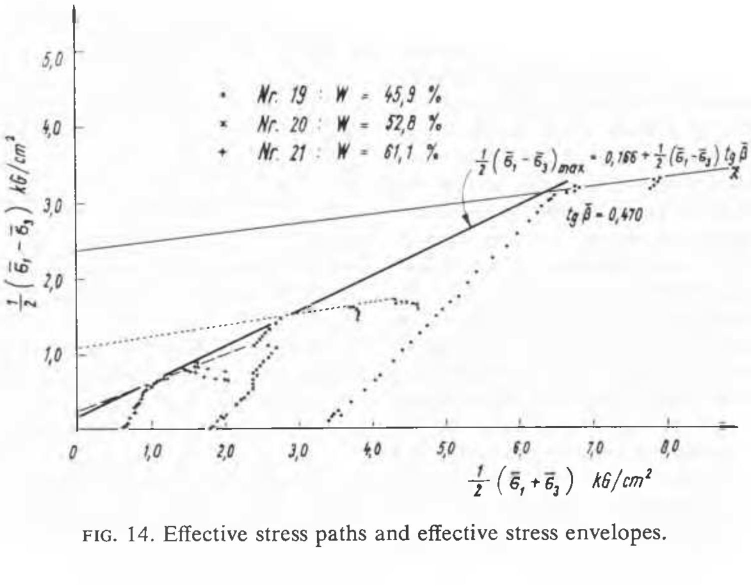



FIG. 14. Effective stress paths and effective stress envelopes.

per cent) and sheared. The initial lateral pressure ($\bar{\sigma}_{eq}$) was chosen so as to prevent water inflow or outflow from the sample ($u$ = atmos. pressure) and this condition was kept throughout the whole test performance ($u$ = const = 0, $\sigma_3$ varying).

The frictional component of the shear strength was evaluated as follows. At strains greater than 8 per cent the deviator stress was approximately constant; thus in six samples at various water contents the shearing process proceeded as mentioned only to 8 per cent strain, then $\sigma_3$ was increased by one kg/sq.cm. and the shearing process proceeded to 11 per cent strain at constant $\sigma_3$ and $u$ measured. After another increase in $\sigma_3$ by one kg/sq.cm. at 11 per cent strain the shearing process was stopped at 14 per cent. The straight line on the graph (Fig. 14) through the points $\frac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_{max}$ for the three shearing stages mentioned gave both the tan $\bar{\beta}_w$ and cohesion intercept $\bar{c}$. From these experimental results tan $\bar{\beta}_w$ as the function of water content was calculated as $\log \tan \bar{\beta}_w = -2.66+0.0359\,W$. Fig. 15 shows the relation between $\bar{c}$ and water content as compared with calculated $p_R$ and $p_A$ ($p_A$ was calculated assuming $A = 0.5 \times 10^{-12}$ erg). From this series of experiments the following conclusions were drawn.

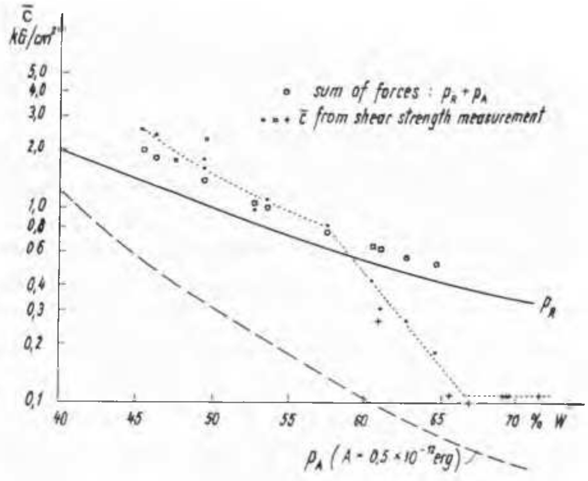



FIG. 15. Comparison between the calculated forces and measured cohesion, $\bar{c}$.

1. In the water content range of 45 to 60 per cent (plastic limit 59 per cent), $\bar{d}$ = 18 to 30 A°, $\bar{c}$ is created by the sum of $p_R + p_A$. (Within this $d$ range Arnold also assumes the parallel plate model as valid.) At higher water contents, $p_R$ gradually becomes inoperative in the shearing process.

2. Any effective stress is carried by swelling pressure and mineral-to-mineral contact (Eq 3). As $p_R$ increases (decrease in water content) the stress fraction carried by it increases (increase in k) and there is a decrease in mineral-to-mineral contact stress. The same is true for shear stress. Part of it is used to overcome friction, $\frac{1}{2}(1 - k')$ $(\bar{\sigma}_1 - \bar{\sigma}_3)$, and the other part, $\frac{1}{2}k'(\bar{\sigma}_1 - \bar{\sigma}_3)$, to overcome $p_s$, $p_A$, and particle-to-particle cementation (usually small as compared to $p_s$ and $p_A$).

$$\tfrac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_{max} = (\bar{c}_0 + p_s + p_A)\cos\bar{\phi}_w + \tfrac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)\tan\bar{\beta}_w \quad (5)$$

where $\bar{c}_0$ = interparticle cementation.

$k'$ is the fraction of the effective stress carried by $\bar{c}$ in the shear plane:

$$k' = \bar{c}/\tfrac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_{max}. \quad (6)$$

Thus the friction to be overcome by deviator stress is

$$\tfrac{1}{2}(1 - k')(\bar{\sigma}_1 - \bar{\sigma}_3) = \tfrac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)\tan\bar{\beta}_w = \tfrac{1}{2}(1 - k')(\bar{\sigma}_1 + \bar{\sigma}_3)\mu\, \mathrm{cso}\,\bar{\phi}_w \quad (7)$$

and

$$\tan\bar{\phi}_w = (1 - k')\,\mu = \tan\beta_w/\cos\bar{\phi}_w, \quad (8)$$

where $\mu$ is the friction coefficient found in these experiments as equal 0.66, k was called static distribution coefficient of the effective stress, and k′, the kinematic distribution coefficient of the effective stress in the shear plane.

3. Between the static effective stress carried by the given clay mass in equilibrium and the deviator stress necessary to cause shear there should be the following relation:

$$[k'\tfrac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_{max}]/\cos\bar{\phi}_w - k\bar{\sigma}_{eq} = p_A \quad (9)$$

4. Both the static and the kinematic coefficients (k and k′) seem to depend on the water content of the samples and on the particle preferred orientation. They increase with decrease in water content. The opposite is true for the effective angle of internal friction. This is in agreement with and simultaneously explains the changes in activity factor ($f(i) = \bar{\sigma}_{eq}/p_R = 1/k$) with water content as found by Arnold (his Fig. 6).

5. Part of the normal stress carried by $p_R + p_A$ in the shear plane at maximum deviator stress, both calculated and determined experimentally, was found to be very close to $2\bar{c}$:

$$\tfrac{1}{2}(\bar{c}_1 + \bar{\sigma}_3)_{calc} = [\tfrac{1}{2}(\bar{\sigma}_1 - \bar{\sigma}_3)_{max} - \bar{c}]/\tan\bar{\beta}_w \quad (10)$$

$$\tfrac{1}{2}k'(\bar{\sigma}_1 + \bar{\sigma}_3)_{calc} \cong \tfrac{1}{2}k'(\bar{\sigma}_1 + \bar{\sigma}_3)_{exp} \cong 2\bar{c}. \quad (11)$$

REFERENCES


BOLT, G. H., and R. D. MILLER (1955). Compression studies of illite suspensions. *Proc. Soil Science Society America*, Vol. 19 pp. 285–8.

LAMBE, T. W. (1960). A mechanistic picture of shear strength of clay. Research Conference on Shear Strength of Cohesive Soils, Colorado.

NORRISH, K. (1954). The swelling of montmorillonite. *Faraday Society Discussions*, No. 18, pp. 120–34.

NORRISH, K., and J. A. RAUSELL-COLOM (1961). Low angle X-ray diffraction studies of the swelling of montmorillonite and vermiculite. *Proc. Tenth National Conference Clays and Clay Minerals* Pergamon Press, 1963.

PASZYC-STĘPKOWSKA, E. (1960). Wpływ rodzaju jonu wymiennego na własności fizyko-mechaniczne bentonitu. *Archiwum Hydrotechniki*, vol. 7, No. 2, pp. 143–213. [The influence of exchangeable cation on the physico-mechanical properties of bentonite (English translation), Published for National Science Foundation, OTS 60-21566.]


G. P. RAYMOND (Canada)

I would like to make a brief comment on the permeability of fine-grained soils. At Queen's University we have performed falling head permeability tests on seven different soils consolidated under different loads in the oedometer. These tests have been conducted in a Bishop type oedometer to which a lever arm has been added so that the height of the soil specimen may be maintained constant during the permeability test (Fig. 16). This has the disadvantage that the load need not remain constant during the permeability test. On the other hand, the effects of secondary consolidation during the permeability test are eliminated. Fig. 17, which is a plot of excess head to a logarithmic scale against time to an arithmetic scale, shows a typical result obtained for one of five of the soils tested. As may be seen Darcy's Law is approximately valid for these five soils. One soil was an artificially deposited bentonite and the other four were inorganic clays, three artificially deposited and one undisturbed.

The other two soils, a lake marl and an amorphous-granular peat, which were tested in the undisturbed state, did not obey Darcy's Law: that is, plots similar to Fig. 17

were curved rather than straight. The tests for these two soils have been fitted with reasonable success to the relationship suggested by Hansbo (1960): that is, $v = \mu i^n$, where $v =$ the velocity of flow, $\mu =$ a constant, $i =$ the hydraulic gradient, and $n =$ a constant. Both $\mu$ and $n$ were found to vary with void ratio. The results for these latter two soils suggest that the time factor in consolidation theory is not always dependent on the inverse of the length of the drainage path squared. It should be added that the peat was known to contain gas within the sample and thus is not a fully saturated soil. It is not known whether the same explanation is valid for the lake marl.

E. TOGROL (Turkey)

I was very much interested in the investigation of soil colour measurement in relation to the structural analysis of soil by *Coleman (1/6)*. I have been carrying out a study somewhat along these lines for the last two years, and it might be pertinent to make a brief comment on it.

In our experiments colour was considered in terms of trichromatic coefficients—brightness, saturation, and dominant hue wavelength. Since only two of the three trichromatic coefficients are independent, five values were obtained for each sample. Our results indicate that measurement of colour of a number of samples could be a valuable tool in soil exploration.

The results of measurements were factor analysed to classify the 50 samples obtained from six borings. All borings were made in a landslide area near Istanbul. The area had

FIG. 16. Apparatus used for the falling head permeability tests.

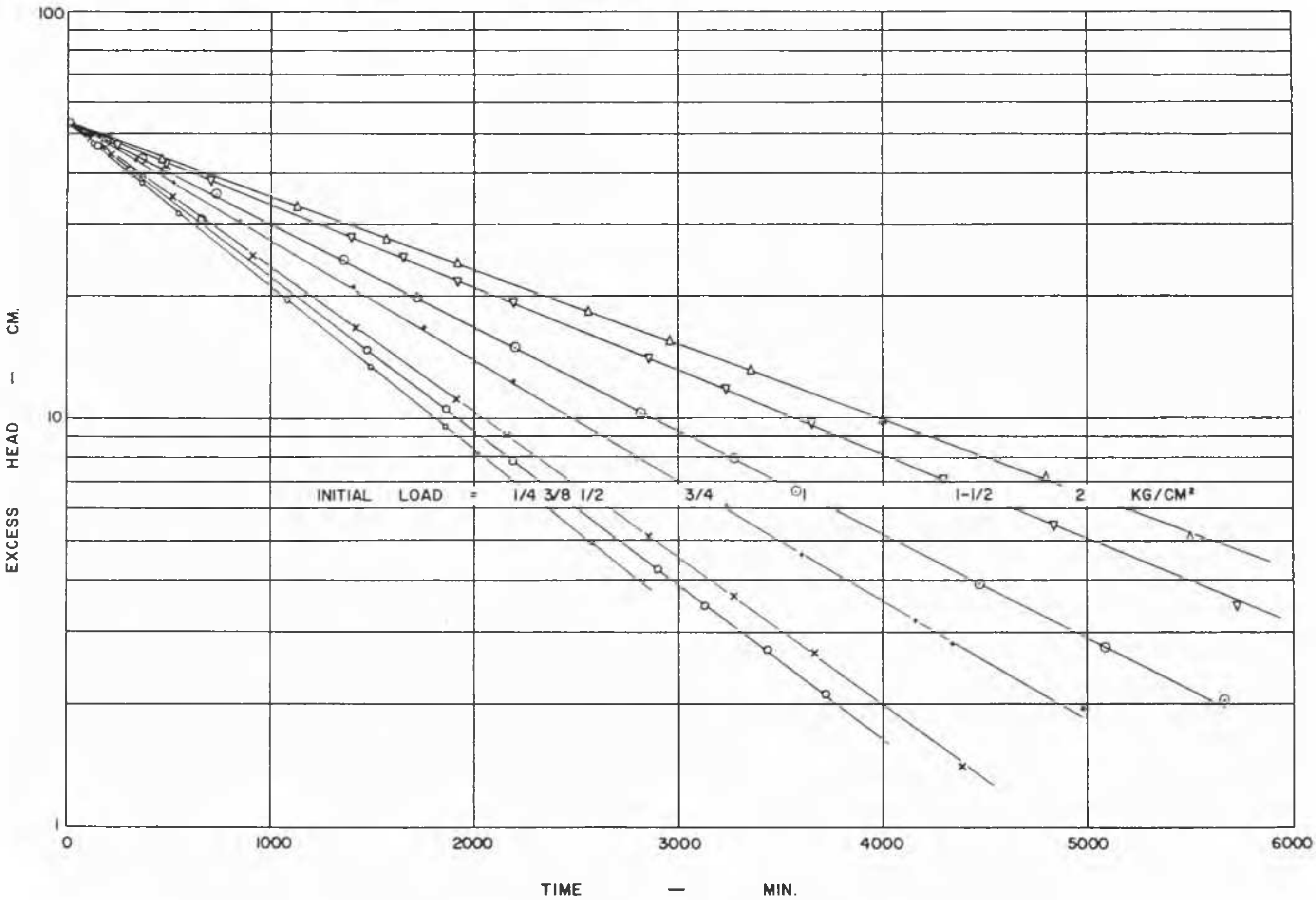



FIG. 17. Typical results from falling head permeability tests.

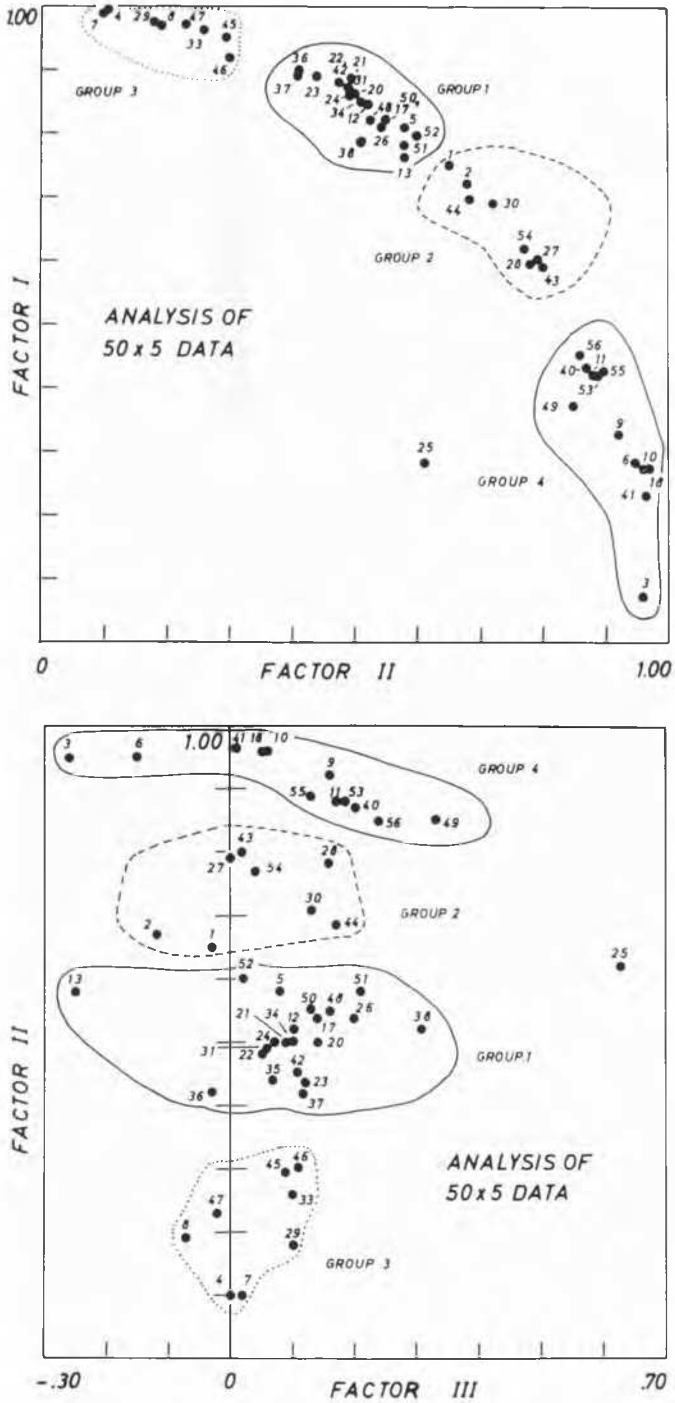



FIG. 18. Analysis of 50 $\times$ 5 data. A, Factor I *vs.* Factor II; B, Factor II *vs.* Factor III.

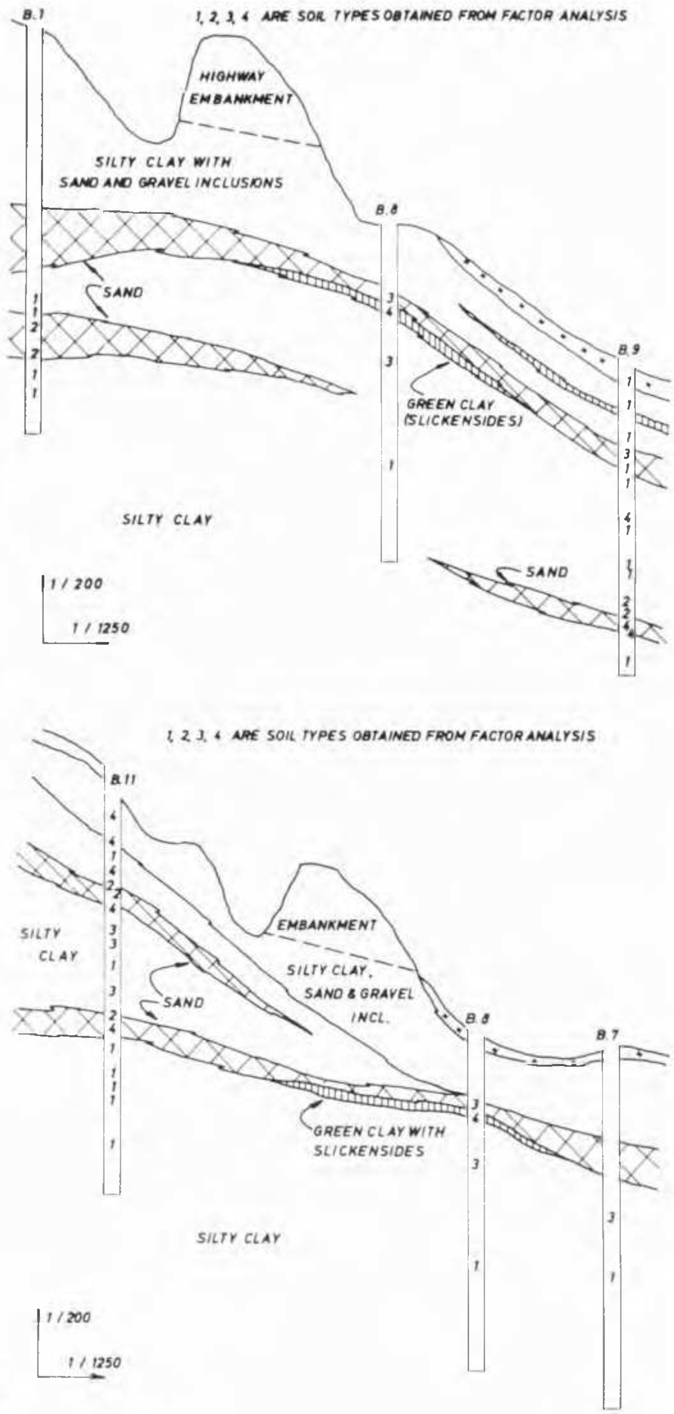



FIG. 19. Soil types obtained from factor analysis shown on soil profiles obtained by conventional means. A, Profile 1; B, Profile 2.

been thoroughly investigated through a number of borings and inspection pits. Oven-dried samples passing No. 40 sieve were used. Colour measurements were carried out on a Lovibond-Schofield Tintometer at the Soil Mechanics Laboratory of Istanbul Technical University. Five colour values were employed for each sample, namely, two trichromatic coefficients, $x$ and $z$, brightness, dominant hue wavelength, and saturation. A data matrix of 50 $\times$ 5 was obtained with rows representing 50 samples and the columns representing the five colour values. Each sample was defined with a vector in the space determined by the 5 colour axes. The angular distance between two sample vectors defines their similarity (Imrie and van Andel, 1964). This angle could be 0° indicating identical samples or 90° corresponding to complete dissimilarity. Cosines of these theta angles between any two samples were calculated into the 50 $\times$ 50 matrix. The examination of such a large matrix, even when it is symmetrical may be an eyewearing and even an impossible task. Thus, factor analysis was employed in the evaluation of interrelations between the samples.

In applying factor analysis to our data, by the nature of the analysis we assumed that the seeming variety among the samples could be determined by a small number of factors. Weights of each sample in respect to these common factors were used in grouping them. Factor analysis differs from statistical prediction in that the factors and weights that are given to each sample may not have a direct physical interpretation. However, they could still be used to classify samples qualitatively. The centroid method was employed in the evaluation of the factor matrix and the factor matrix obtained was rotated with varimax method of rotation. Thus, three factors were determined. All computations were carried out on I.B.M. 1620 in the Computing Center of Istanbul Technical University.

Fig. 18 graphically represents the grouping of our data in respect to the three factors. If samples are grouped correctly then the groups should fit into the soil profile determined by usual means. It is seen from Figs. 19A and 19B that the soil groups determined by factor analysed colour data correspond very closely to the actual strata. Group 1 is the largest of all groups and represents the main stratum of fissured clay. Group 2 is silty sand and observed in thin bands throughout the area. Group 3 is a rusty-looking gravel, sand, and clay combination observed lying just over the sandy silty layer (Group 2). Group 4 is the portion of the profile in which the sliding is occurring. It is greener and more plastic than the main stratum, contains slickensides, and is usually found under the sand-silt layer.

As a final analysis, 27 samples representing three of the borings were analysed by taking into account the five colour values along with their liquid and plastic limits. The results yielded the same sort of groupings as before.

It is interesting to note that the complete colour data when factor analysed could define the different strata more successfully than any one of the parameters considered.

REFERENCE

IMBRIE, J., and T. H. VAN ANDEL (1964). Vector analysis of heavy-mineral data. *Bull. Geological Society America*, Vol. 75, No. 1.

A. VAN WAMBEKE (Belgique)

En complément à la communication de *Meigh and Greenland* (*1/16*) je voudrais signaler une application importante du pressiomètre dans l'étude des fondations des ouvrages de rachat de la chute de Ronquières (dénommés, en raccourci, "plan incliné") en Belgique. Ce n'est pas le moment de parler de ce chantier que beaucoup d'entre vous ont pu, par ailleurs, visiter. Je dirai simplement que le terrain d'assise se situe à la limite du dévonien et du silurien et que si ce dernier s'est avéré d'excellente qualité, le dévonien a par contre réservé quelques surprises. L'importance des charges et la limitation assez draconienne des tassements différentiels entre appuis adjacents ont rendu nécessaire une exploration fouillée du terrain de fondation. La mauvaise qualité de la roche rendait indispensable l'utilisation d'essais in situ et dans un tel terrain il ne pouvait s'agir que d'essais pressiométriques.

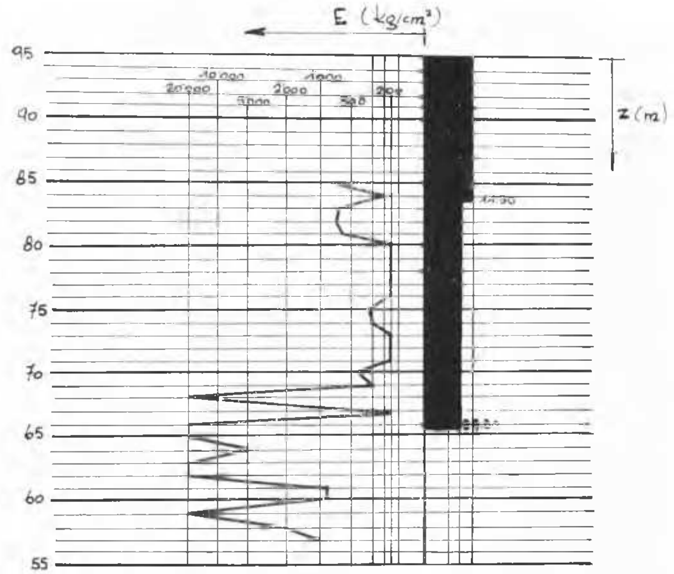


FIG. 20. Les modules pressiométriques $E$ en fonction de la profondeur z.

Les fondations de la plupart des ouvrages sont constituées par des puits de grand diamètre (2 à 3,75 m nominal) et présentent parfois un profil discontinu avec diamètres décroissant en profondeur; à chaque ressaut de discontinuité est ainsi réalisé un épaulement qui contribue à améliorer la tenue de la fondation en mobilisant mieux que ne peut le faire le frottement latéral la capacité de résistance du terrain contigu au fût.

Les profils pressiométriques donnant les modules pressiométriques $E$ en fonction de la profondeur $z$ ont permis de déterminer la longueur de chaque puits ainsi que le niveau où un épaulement pouvait être prévu (fig. 20).