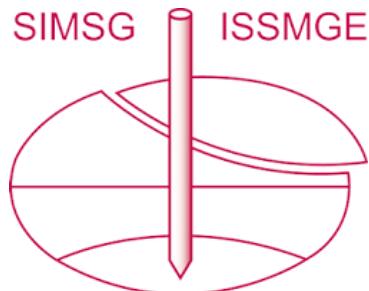


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



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Questions diverses

Various Questions

Sujets de discussion : Liaison chantier-laboratoire. Stabilisation des terres pour la construction d'habitations. Sols vacuolaires (loess).

Subjects for Discussion : Best laboratory-works relationship. Use of soils as a constructional material in housing projects. High void ratio soils (loess).

Président / Chairman :

Vice-Président / Vice-Chairman :

Rapporteur Général / General Reporter :

Membres du Groupe de discussion / Members of the Panel :

A. von Moos, *Suisse*,

M. Henry, *France*,

M. Mehra, *Inde*,

N. Denissov, *U.R.S.S.*; K. V. HELENELUND, *Finlande*; A. Lazard, *France*; G. Stefanoff, *Bulgarie*; W. Turnbull, *U.S.A.*

Discussion orale / Oral discussion :

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K. W. Helenelund, *Finlande*,
R. Haefeli, *Suisse*,
A. Lazard, *France*,
R. B. Peck, *U.S.A.*
G. Stefanoff, *Bulgarie*
W. J. Turnbull, *U.S.A.*
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M. MEHRA

Rapporteur Général, Division 7 / General Reporter, Division 7

verses ». Ce groupe de travail est présidé par M. von Moos, de la délégation suisse. J'en suis moi-même Vice-Président, mon nom est M. Henry, de la délégation française. Le Rapporteur est M. Rao de la délégation de l'Inde.

Les membres sont MM. Helenelund, de la délégation finlandaise, Lazard (France), Stefanoff (Bulgarie) Turnbull (Etats-Unis), et M. Denissov (U.R.S.S.)

Je passe la parole à M. von Moos.

Le Président :

Je remercie M. Henry de son aimable introduction.

I would like to say a few words about the work we are doing now. We have the intention to start with the first question, and when the panel discussion is finished we will continue with the discussion from the floor. Therefore I ask everybody who would like to make a contribution to bring his name written on a piece of paper, together with his country and the subject he would like to discuss. Afterwards we will continue with the third question — the second question will not be discussed today because of shortness of time; we have to leave this room at eleven o'clock.

Now I give the floor to the Delegate from India who will replace Prof. Mehra : Dr Rao, may I ask you to speak about the General Report?

M. K. L. RAO (Inde) pour le Rapporteur Général

In the unavoidable absence of Mr MEHRA, the General Reporter, I am just playing the role.

Le Vice-Président :

M. le Président, Mesdames, Messieurs, en tant que délégué du Comité d'organisation, j'ai l'honneur de vous présenter le groupe de travail chargé de la Section 7 « Questions di-

Of the eight papers received under this section, except for two the subjects were very varied, ranging from lining of canals to treatment of the foundations of buildings subject to seismic forces. Mr Mehra has given a summary of the papers in his general report. They were really miscellaneous in the true sense of the English word. On the subjects set for discussion, one was stabilised soil for use as a construction material. There is a very interesting paper on this subject — 7/4, by Litvinov and others, and it deserves very careful perusal. The knowledge on the subject can be summarised as follows :

(i) The soil can be stabilised economically by improving the grading of the soils by intermixing with other suitable soils wherever possible.

(ii) The stabilised soils can be used economically in the construction of roadways or runways and roads, if not for very high buildings.

(iii) Heavy clays, like "Tschernosems" of Russia and the "black cotton" soils of India, can be economically stabilised by using cement. However, we have to pretreat them with 4-5% of slaked lime in order to use their swell properties. As the Chairman has stated, due to lack of time this subject will not be taken up for discussion this morning.

The second subject is the high void soils. In this there is one interesting paper, 7/1. Though this soil occurs in many regions of the world there were very few contributions—actually only one contribution. In India we have had to deal with this type of soil very recently. In one of the States we had to build a dam on a high void and very dry soil. What we did in this case was to excavate and remove the soil for some depth and then saturate it by ponding and keeping the water for a length of time, thus trying to eliminate the excessive settlement that would have occurred when the dam was constructed without taking this precaution. There must be similar interesting experiences in other regions, and members are requested to contribute their discussions on this subject.

There is, however, another important subject—the relationship between the laboratory and the works. On this there have been no contributions. The panel committee felt that this is a very important subject, and you are particularly requested to make your contribution on this very interesting subject. In the modern practice of engineering we all know how close the relationship is between the laboratory and the design and construction engineer. In the field, for example, of hydraulics and river-valley structures one very interesting example in my experience was on designing protection works for a town called Dibrugarh in Assam, on the banks of a very mighty river, the Brahmaputra, which is about ten miles wide at that place. The town is a prosperous and very beautifully laid out town—it is known as French town. The river attacked with fury and in one year, about five years ago, devoured half of the town. Before the other half was finished, engineers were called in to deal with protective measures. The construction materials were 300 miles away and therefore we had to design taking into account the fact that the construction material was not available within easy reach and the period of construction was very short. Accordingly we designed five short spurs, over 200 ft. long, with the wooden spurs in between to protect a reach of nearly five miles. Many people said that this was a flimsy protection and it would not serve the purpose.

Then we went in for hydraulic models, and the models gave not only a great assurance that this would be successful but also gave valuable information in fixing the direction of the spurs, and the other dimensions like the nose, and so on. This was constructed, and the town was saved. This is a very valuable experience where the close liaison

between the laboratory and the designers was highly successful.

Similarly in concrete technology we know how in the modern practice of concrete construction it is absolutely essential that laboratory assistance be sought; in fact this assistance is so greatly valued now that the size of models is continuing to increase. I have seen some very beautiful structures at the ISMES Laboratory at Bergamo in Italy. I am sure that it is the laboratory that gave the courage to the Italian engineers to go ahead with that domeshaped structure, resulting in the mighty, beautiful Vianot dam, 261 metres high. Similarly I have seen some very beautiful experiments on full-scale models in Japan, subjecting these full-scale buildings to earthquake forces or to eccentric forces simulating earthquakes. So when we come to the subject of soil mechanics, or the application of soil engineering, we find a great necessity for ensuring a closer liaison between the laboratory and the field engineer and the field and design engineer. The reasons are two. One is that in the case of the soils, unlike concrete structures, we find the design cannot be finalised based on preliminary investigations. A large number of changes become necessary during construction. The soil varies so widely from what is given even by detailed preliminary investigations that it is necessary that the laboratory personnel be closely associated with the work during all stages of construction. Not only this; the progress of the science of soil mechanics and the possibilities of constructing economically in the future, again, depend on the observations that the laboratory personnel have to keep after structures are completed.

This is the theme of Dr Terzaghi's latest utterances. Again and again he emphasises how important it is for the development of the next stage of our science to keep these observations. Therefore, in view of this importance, we have got to define the relationship between the laboratory and the works very closely, in such a way as to take these two important factors into account. In other words, the laboratory research personnel must become more or less partners with the designer in this particular case; they must assume greater responsibility.

There is yet another important consideration. There is a tendency to develop appliances and equipment in soil mechanics which are not easily understandable by the general engineering personnel in charge of designs. It is not a healthy sign; in fact, if we import simplicity into the structure there are more chances of developing the science on more rapid lines. In support of this I quote only one example, the famous example of Pitot, the great French engineer. Pitot developed this small tube which is still the instrument that we use for measuring the velocity of water and determining discharges of the water flow. He discovered this small thing, and he said in very famous words, "I can never understand how this simple apparatus escaped the attention of the many investigators before me".

In a similar way, if we take this precaution not to load the science with too much mathematics and equipment it has a better chance of making the designer and the field engineer understand this intelligently and become partners in this closer liaison with the laboratory. It is about the specific details of how to achieve this liaison that the panel committee decided that we should have a discussion this morning.

Le Président :

Thank you, Dr Rao, for your very interesting contribution. We will start with the first question, best laboratory-works relationship and the first speaker is Mr Turnbull.

This discussion deals with the subject of the relationship between the soils laboratory and the construction work, but in a somewhat broader sense than indicated by the specific items mentioned by the General Reporter.

Soil mechanics as an important segment of the civil engineering profession is not being given the recognition due to it by the engineering profession as a whole. Those of us who are actively engaged in the practice of some phase of soil mechanics have not done a proper "selling" job of soil mechanics, as a "grown-up" segment of civil engineering, to other engineers, inclusive of architects and many so-called practical civil engineers, as well as to people of those closely allied groups dealing with construction problems such as contractors and construction equipment manufacturers.

It has been interesting to this discusser to note that over the years the writings of many eminent technical authorities on soil mechanics have been leaning more and more to the practical aspects. Such authorities include Terzaghi [1], Casagrande, Skempton, Peck [2] and Bjerrum. They repeatedly point out that the highly technical engineer well versed in theory may be of little use on design and construction unless he is mellowed with a considerable amount of actual construction experience backed up by specific soil mechanics evidence obtained from practical or applied research or by actual observation of prototype behavior. These same engineers amply verify their writings by their obvious eagerness continually to bolster their own personal knowledge by collecting physical facts of soil behavior.

It may well be then that the answer as to why soil mechanics is not regarded with complete respect by all engineers is being partially supplied by the current action of many of our soil mechanics authorities in emphasizing the practical application of soil mechanics to a greater extent than ever before. Some might ask the question "Is theory then to be ignored?" The answer is "of course not", but theory should be delimited or supplemented by experience and field evidence. It is quite possible that theory should be modified to fit specific conditions or in some cases discarded and replaced by a new theory.

We come then to the question "Of what does the practical application of soil mechanics consist?" The answer certainly does not lie in the actions of some so-called practical engineers and contractors, some of whom the discusser has encountered, who apparently figure that soils are basically "dirt" and there is nothing much that can be done to improve them as a construction material. Moisture control and proper compaction are regarded as an impractical idea of the theoretical fellows. Further, the answer certainly does not lie in the use of a soils laboratory as a "glamour" tool to furnish a ϕ (friction angle) and a c (cohesion value) which are obtained by unqualified testing personnel following some standardized test procedure which has no relation to the specific job conditions; the ϕ and c values are then used by a structural engineer in a standardized formula which probably bears very little relation to actual job requirements. Unfortunately such conditions still exist.

The application of soil mechanics can only be considered practical if it results in meeting at least the first three of the following factors : (a) a safe structure, (b) an economical structure commensurate with design requirements, (c) a structure which will meet the objectives of the project, and (d) a construction job on which the contractor has at least had a nominal profit. The latter item seems a bit incongruous; however, an unhappy contractor, that is, one without a profit, all too often develops problems which lead to defective work.

Rather than specifically covering the topics suggested by the General Reporter, the discusser will take the liberty of presenting a description of a procedure which in his opinion covers the subject matter and demonstrates the practical application of soil mechanics. This procedure is referred to as a fully integrated soil mechanics engineering operation. The operation involves the following phases of soil mechanics engineering investigational studies : (a) geologic investigations, (b) subsurface sampling and field studies, (c) laboratory testing, (d) design, (e) instrumentation, (f) construction, and (g) prototype studies. The entire operation should be supervised by an engineer well versed in all phases of soil mechanics study, the logical choice being the engineer responsible for the design. The study phases are discussed briefly in order.

The importance of geologic studies has been repeatedly emphasized by many engineers, with Dr Terzaghi [3] probably being the most emphatic. It is only common sense that soil mechanics engineers cannot properly interpret field conditions without knowing how Mother Nature laid down the sediments and rocks and then in many instances subsequently seriously distorted them to the extent that the safety and successful operation of the engineering structure could be seriously endangered unless the geologist detected the true condition so that it could be recognized in the design.

The taking of proper samples, predeterminable as to type, for study and testing in agreement with requirements of design must be accomplished.

The proper laboratory tests meeting the particular objectives and peculiar conditions existing at the project site should be accomplished. Casagrande [4] repeatedly points out the uselessness and even damaging results of using test data which do not meet the requirements of the project being studied.

Under an integrated operation the design engineer has the very great advantage of having available a well-detailed geologic report and knowledge that the data from the field sampling and laboratory testing are meaningful and representative, since they have been conducted specifically with due consideration to project conditions. Completion of the geologic, soil-sampling, and laboratory-testing studies in time for full utilization by the design engineer is particularly essential.

Development of instrumentation and its proper installation are very important phases of the integrated operation.

Construction is usually supervised by others than the design engineer; however, close liaison should be maintained and the construction engineer should be made fully aware of all design requirements, particularly those which are most critical with respect to building a safe and satisfactory project.

We come now to the last and most often neglected phase of the integrated operation. It is essential that instrument readings be taken at predetermined intervals of time, properly recorded, fully analyzed, and carefully reported immediately so that modifications in design or in construction procedures may be made if so indicated. This is the source of some of the most valuable soil mechanics engineering evidence.

In discussing what has been referred to as an integrated soil mechanics engineering operation, it is realized that an ideal situation is represented. Its full attainment by any group or organization is seldom realized; however, many groups are active in at least two phases and a goodly number in three or more phases of the complete operation. In any case, it is very important for the responsible organization to be fully cognizant with the quality of the work done in those phases accomplished by others.

The Soils Division of the U.S. Army Engineer Waterways Experiment Station has been fortunate in that on some projects for the Mississippi River Commission, Corps of

Engineers, all phases of the integrated operation have been handled, excepting construction, and in this phase liaison has been maintained. In other civil and military works projects for the Corps of Engineers the operation has been less complete; nevertheless, it is established practice wherever possible to maintain close liaison on those phases of the integrated operation not handled directly. Wherever possible, the end product, after about five years' operation of the structure, is the prototype analysis report. In this report, soil profiles and conditions prior to construction, soils classification, soils test values, design requirements, actual conditions obtained in construction, and behavior data are brought together for general analytical study. Often important conclusions in regard to future design criteria can be derived.

By following generally along the procedures just delineated, it has been a source of great satisfaction to the discusser on numerous occasions to demonstrate to other engineers, not well versed in soil mechanics, and to contractors the utility of the practical application of soil mechanics. One of the most graphic experiences of this type was with a contractor on a major project attempting to drive a sheet pile cutoff to depths up to 170 feet in a sand-gravel-cobble alluvium. At about the 100-foot depth the contractor was unable to drive the piling to bedrock. He closed down operation claiming changed conditions, namely, the presence of boulders up to 25 feet in greatest dimension. The discusser, having complete knowledge of the geology, explorations, laboratory tests, and in particular the contractor's operations, was able to determine that a cobble bed, later verified by borings, was being created below the end of the piling through which eventually the piling would not penetrate. When the contractor's procedural operations were corrected to prevent the formation of the cobble bed, the piling was seated into bedrock to the maximum depths without undue difficulty.

The use of published "standards" for sampling, testing, and classification is briefly mentioned. There are many arguments for and against the current use of presently existing standards and even the development of more standards. Those against, among other things, claim that standards lead to oversimplification, and in some cases improper analysis, of many soil mechanics and foundation problems which could lead to defective construction. The proponents of standards maintain that all engineers and inspectors cannot be specialists in soil mechanics; consequently, the use of standards is a boon to them and results in better and safer projects. This discusser feels that the advantages in the use of standards outweigh the disadvantages, primarily because of the unfortunate fact that the majority of engineering projects involving soil mechanics problems are still being supervised and inspected by engineers and inspectors who do not have an adequate background or experience in soil mechanics.

It is generally accepted among soil mechanics engineers that much more needs to be done in basic and applied research to improve and strengthen fundamentals, and the individual engineer undoubtedly realizes his need to gain experience in his field and add evidence to his knowledge; however, too many of us have tended to neglect the need of demonstrating to other engineers in the construction field that the application of soil mechanics to projects involving soil materials not only is practical but also is necessary if the project is to be constructed properly to meet design requirements.

In summary, it is believed that all engineers actively engaged in practicing soil mechanics engineering should make a personal effort as well as join in group effort to demonstrate to those who are skeptical of the practicability of soil mechanics that through proper application of basic soil-mechanics principles and knowledge, engineering projects

can be constructed in a safer and quite often more economical manner.

Références :

- [1] TERZAGHI, Karl (1959). « Soil Mechanics in Action ». *Civil Engineering*, vol. 69, pp. 33-34.
- [2] PECK, Ralph B. (1961). « The Art and Science of Foundation Engineering ». Presented at meeting of Soil Mechanics and Foundations Division, Illinois Section, *American Society of Civil Engineers*.
- [3] TERZAGHI, Karl, « Presidential Address, Opening Session, Fourth International Conference on Soil Mechanics and Foundation Engineering ». *Proceedings*, vol. III, London. 1957.
- [4] CASAGRANDE, Arthur (1960). « Discussion of Requirements for the Practice of Applied Soil Mechanics ». *Proceedings*, Congress of First Pan-American Soil Mechanics Conference, vol. III.

Le Président :

Thank you Dr Turnbull.

M. LAZARD (France)

J'ai compris que la discussion d'aujourd'hui concernait, d'une manière générale, les relations entre maître d'œuvre et mécaniciens du sol ou géotechniciens bien que ces termes soient encore assez peu précis; parmi les mécaniciens du sol, je comprends les géologues purs.

Bien entendu, je me placerai dans le cas d'un maître d'ouvrage qui sait qu'il doit faire appel à la mécanique des sols. Je réduirai donc un peu le problème, le prenant à un niveau moins élevé que celui auquel vient de se placer M. Turnbull.

Le sujet étant extrêmement vaste, et pour me limiter, je voudrais traiter quatre points particuliers, en donnant à chaque partie, d'ailleurs, une importance très différente.

— Dans la première je voudrais traiter des relations avec les laboratoires, et plus spécialement des questions de sondage et de ce qu'on peut attendre d'un laboratoire;

— Dans une deuxième partie, je voudrais traiter des problèmes difficiles qui nécessitent une expertise avec des ingénieurs conseils de Mécanique des sols;

— Dans une troisième partie, je parlerai de l'appel d'un maître d'ouvrage dans des cas difficiles également aux entreprises spécialisées de Mécanique des sols;

— Pour terminer enfin, je voudrais dire quelques mots des dépenses qu'il faut faire, ou consentir pour la Mécanique des sols.

Première Partie. — Relations avec les laboratoires, en particulier, que peut-on attendre des sondages et des résultats que vont nous donner les laboratoires ?

Bien entendu, la première question consiste à caractériser du mieux possible les couches sur lesquelles on va avoir à asséoir un ouvrage. A ce sujet, je voudrais me référer aux paroles très pertinentes qu'a prononcé à la séance inaugurale M. le Prof. Caquot. Il a insisté sur le fait qu'il fallait, autant que possible, s'entendre nettement sur la terminologie des mots, car malheureusement nous en sommes encore à un point où ces questions ne sont pas très bien définies, et il a indiqué également que lorsque l'on fait un sondage, il faut absolument qu'on obtienne des chiffres.

Or, sur ce point, je ne suis peut-être pas entièrement d'accord avec lui. En effet, dans des pays comme la France, où la construction est déjà bien étendue, on est souvent amené à étendre un ouvrage à côté d'ouvrages connus, et le premier problème consiste seulement à se renseigner à et s'assurer que les couches ne sont pas très différentes de celles que l'on

connaît, et pour lesquelles on connaît la meilleure solution à appliquer. Dans ces conditions, les chiffres n'ont pas une très grande importance.

Au contraire, quand on se trouve dans des sols nouveaux, ou quand le problème est entièrement différent de celui pour lequel on connaît bien la solution, il est, bien entendu, nécessaire de demander des chiffres aux mécaniciens du sol. Mais quels chiffres demander ? Là, je suis bien d'accord avec M. Turnbull pour dire que c'est le maître d'ouvrage qui doit préciser à l'avance au laboratoire ce qu'il veut obtenir. Mais encore faut-il faire un certain nombre de distinctions.

S'il s'agit, par exemple, de déterminer des tassements, je crois que la Mécanique des sols, actuellement, est bien avancée et peut nous donner, dans la plupart des cas, d'excellents renseignements et d'excellentes précisions. Le problème, à mon avis, est entièrement différent s'il s'agit d'obtenir des résultats qu'on puisse utiliser dans les calculs en vue d'obtenir une résistance. Sur ce point, mon opinion personnelle est assez pessimiste, en ce sens que dans de nombreux cas l'expérience prouve journellement que les meilleures théories, les plus avancées à l'heure actuelle, et qui nécessitent souvent un appareil mathématique extrêmement complexe, donnent des résultats fréquemment pessimistes : la nature est meilleure enfant et donne, elle, des résultats deux ou trois fois supérieurs.

Malheureusement, il est d'autres cas, au contraire, où les théories ont plutôt tendance à être trop optimistes, et à donner des résultats qui sont du mauvais côté de la sécurité.

A ce sujet, je voudrais signaler une différence entre le congrès de Londres et le congrès de Paris. A Londres, il m'avait semblé que régnait un optimisme presque béat : tous les rapporteurs généraux nous annonçaient : « Dans deux ou trois ans toutes les questions seront à peu près résolues ». Quatre ans ont passé, et je crois que le congrès de Paris donne des résultats qui sont davantage dans le sens du scepticisme.

A tel point qu'hier un de mes excellents camarades me faisait remarquer, après la très intéressante discussion sur les pieux et la comparaison entre les pieux vraie grandeur et les pénétromètres, qu'on avait peut-être eu tort, dans l'organisation de nos discussions, de n'inviter à la tribune que de très grands « pontifes ». Car ils ne sont généralement appelés que dans des cas extrêmement difficiles, hors de la pratique ordinaire, si bien qu'à les écouter on pourrait presque craindre que tout soit hors des règles. Or, bien entendu, il y a tous les cas courants, où il n'est pas nécessaire de faire appel à un grand maître, et où les résultats, ma foi, sont convenables.

Pour en revenir aux chiffres, je ne sais pas ce que vous faites quand vous avez fait appel au laboratoire, mais il me semble qu'il y a une pratique qui n'est pas entrée dans les mœurs, c'est celle d'exiger des laboratoires qu'ils nous donnent les chiffres de dispersion de leurs propres essais. D'abord, dans tout essai, il y a une dispersion, une équation personnelle du laboratoire ou de ceux qui opèrent; de plus des échantillons prélevés à différents endroits donnent des différences souvent très grandes.

Je suis, vous le savez, très sensible aux questions de dispersion, et l'autre jour j'ai fait une petite communication (voir p. 325), à la demande de M. Biarez, pour indiquer comment les statistiques devaient intervenir dans nos calculs pour prendre en compte ces phénomènes de dispersion.

Mais de plus, et là je rejoins partiellement M. Turnbull, il est souvent difficile, pour quelqu'un qui n'est pas un spécialiste de la Mécanique des sols, de savoir exactement comment manipuler les chiffres qu'on lui donne, tels que les cohésions, les angles de frottement, de savoir s'il faut prendre les valeurs c ou φ , barrées ou non barrées, avec des ' , soulignées, etc... tout ce que nous entendons ces jours-ci, des

coefficients de pression des pores dans lesquels on fait intervenir l'eau, l'air, j'avoue que je suis souvent un peu effaré et je ne m'y retrouve pas très bien. Je suppose qu'il en est de même pour beaucoup de maîtres d'ouvrage qui ne sont pas des spécialistes, et je crois qu'il est indispensable que, le laboratoire ayant discuté avec le maître d'œuvre de ce que l'on peut tirer des essais, il lui indique la nature des essais à faire, savoir s'ils doivent être drainés, non drainés, rapides, lents, etc...

Une autre question qui peut se poser est de savoir où l'on doit faire ces sondages. Nous avons eu, à ce sujet au Comité français de Mécanique des sols, une discussion assez intéressante : certains pensaient que les sondages, quand il y a à reconnaître une fondation, par exemple, devaient être faits à côté de la fondation; d'autres pensaient au contraire qu'on devait les faire dans le site même où devait avoir lieu la fondation. Ceux qui étaient pour les sondages à l'extérieur de la fondation faisaient état au moins de deux expériences. C'est le cas de souterrains, et je pense en particulier, au futur souterrain qui reliera un jour la France à la Grande-Bretagne, sous le Pas-de-Calais, et où il est, bien entendu, désirable que les sondages et les prélèvements ne soient pas faits dans l'axe du futur souterrain, car le jour où l'entrepreneur arriverait à cet endroit-là le chantier serait peut-être inondé.

Un autre cas signalé est celui où l'on risque une nappe artésienne. Bien entendu également il ne s'agit pas que le sondage risque de noyer le chantier avec l'eau qui vient du sous-sol.

Mais dans les autres cas, il me paraît qu'il vaut beaucoup mieux faire les sondages à l'emplacement même de la fondation, car malheureusement la mère nature, comme le disait M. Turnbull, est très capricieuse. Dans des endroits très rapprochés, très souvent, les couches plongent, ou bien on trouve subitement des poches de sable ou de vase intercalées entre des couches et qui n'apparaissent pas à côté.

Le problème est particulier, par exemple pour nous, dans les chemins de fer, où nous sommes obligés de faire des ouvrages dans nos remblais, et nous ne pouvons attaquer vraiment la fondation que quand l'ouvrage est commencé. Dans les études préliminaires nous devons nous contenter souvent de faire des sondages aux extrémités. L'expérience quotidienne nous apprend que nous avons souvent des surprises désagréables quand le chantier est attaqué.

Une autre question qui pourrait être intéressante est de savoir à quelle entre-distance on doit faire ces sondages, quelle est la « grille » à laquelle on doit avoir recours pour planter les sondages. A ce sujet, le congrès est marqué par un excellent rapport de nos amis Suisses; prévoyant une autoroute le long du lac Léman ils ont fait un nombre de sondages et d'études considérable, qu'impose maintenant la rigidité des nouveaux travaux tels que les chemins de fer et les autoroutes dont le tracé est extrêmement peu flexible, et pour lesquels on est obligé aujourd'hui de passer dans des terrains difficiles que nos anciens, qui ne connaissaient pas la Mécanique des sols, évitaient soigneusement.

Je connais une étude faite en France récemment, dans des lieux que visiteront les Congressistes qui iront dans le Midi, par notre grande société l'Électricité de France, où des sondages avaient été implantés, je crois, tous les six cents mètres, pour la création d'un canal de 15 km de long, ce qui n'était pas mal. Il a semblé, d'après tous les sondages, que le terrain était très homogène. Il n'empêche qu'il y a eu des ennuis en certains endroits, où la mère nature avait cru bon de changer ses couches d'une manière exagérée, entre deux points de sondage.

Deuxième Partie. — Je voudrais maintenant en venir très brièvement aux cas difficiles, quand nous sommes obligés de faire appel à des mécaniciens du sol spécialisés. Bien entendu

du, ce sont toujours des cas d'espèce. On fait appel à eux quand le cas se présente d'une manière très différente de ce qui était attendu, lorsqu'il y a une surprise tout à fait fortuite; alors, il est difficile de donner des directives générales, et l'on fait appel tant aux géologues qu'aux mécaniciens du sol, avec lesquels il faut naturellement que la collaboration soit la plus complète.

Il n'est pas question de leur poser le problème, de leur payer des honoraires, et d'attendre leur réponse. Il faut que la question soit discutée continuellement, tant au cours des prélèvements que des essais, pour que l'on puisse conduire le mécanicien du sol, qui n'a peut-être pas la connaissance de toutes les données du problème, vers la solution la meilleure.

Troisième Partie. — Maintenant, je voudrais faire mention des études que nous demandons aux entreprises spécialisées de fondations, et c'est peut-être, en ce qui nous concerne, le cas le plus fréquent. Les conditions sont telles qu'un nombre considérable de solutions peuvent se présenter à l'esprit, qui toutes, certainement, donneront satisfaction, mais qui toutes peuvent conduire à des dépenses extrêmement différentes. Là, il est bon d'être très au courant des dernières nouveautés des entreprises, pour pouvoir leur demander leur collaboration la plus efficace.

Je fais allusion, naturellement, aux problèmes de stabilisation, d'injection avec les produits les plus nouveaux tels les résines, les coulis ternaires, etc.

Quatrième Partie. — Pour en finir avec ma longue intervention, je voudrais que l'on puisse tout à l'heure discuter un peu des dépenses qu'il est bon de consacrer aux études de géotechnique. J'ai fait faire une petite enquête en France, dont je vais vous communiquer les résultats, qui m'ont, je dois le dire, légèrement surpris.

Dans la construction de grands bâtiments, il semble qu'en France les dépenses soient relativement faibles, et ne dépassent pas 2 pour mille de l'ensemble de la dépense de la construction. C'est peut-être un peu faible, mais je répète qu'en général en France les sols sont bons, et l'on construit très souvent autour de zones déjà bien connues.

A l'autre extrême, j'ai l'exemple des dépenses qui ont été consacrées aux études pour le plus grand barrage en terre construit en France, celui de Serre-Ponçon, dans la vallée de la Durance, où les dépenses ont atteint le chiffre, qui me paraît considérable, de 8 % : 6 % ont été consacrés aux études préalables, et 2 % aux études de contrôle pendant la construction. Il s'agissait, évidemment, d'un cas très particulier, puisque, vous le savez, sous le lit même du barrage, il y avait 100 mètres d'alluvions qu'il a fallu barrer par des injections.

E.D.F. me signale que sur un chantier moyen, les dépenses sont de l'ordre de 2 %. A priori, 2 % me semble un chiffre raisonnable pour des études, mais j'aimerais avoir, si possible, l'avis de mes collègues à cette table, et éventuellement dans la salle.

Le Président :

Je remercie M. Lazard pour son exposé, et je donne la parole à M. Helenelund.

M. K. W. HELENELUND (Finlande)

The laboratory-works relationship depends to a large degree upon the project in question and the type of engineering organisation concerned. In a large organisation like a State railway, road or power company, where the design, the field and laboratory investigations and also the construction work is done within the same organisation, the prerequisites for an effective laboratory-works relationship are generally good. The soil mechanics department makes the first subsoil explorations, normally at an early stage, when

the design department has prepared preliminary plans, often including alternative construction sites. On the basis of the results of these investigations the definite site is chosen and the plans for detailed investigation of the more complex areas are made. The quality of the finished project and the difficulties during construction to a large extent depend on the effectiveness of co-operation between the design and soil mechanics departments in this period.

The project, or certain parts of the project, are also discussed with representatives of the construction department. During construction the soil mechanics department may have to make complementary investigations, for instance soundings or borings for individual piles, or perhaps for sub-surface explorations for temporary installations, inspections of foundation pits, and so on. In some cases it is necessary to maintain a special field laboratory for control tests, for instance, during the construction of earth dams. A special moving laboratory, a trailer or rail laboratory, which can be placed at or near to the construction site, is often practical. Some organisations have a central laboratory with heavy equipment and smaller laboratories in different districts. For instance, the Board of Roads and Waterways in Finland have a central laboratory in Helsinki and 12 small laboratories for routine and control tests in different parts of the country. Such a district laboratory may serve as a field control laboratory during construction in its neighbourhood.

When the preliminary investigations are made by an independent organisation like a private consulting firm or independent soil mechanics laboratory, the co-operation between the laboratory and the construction site is often unsatisfactory. The contract between the soil mechanics firm and the organisation which is responsible for the construction may be limited to preliminary investigations only. In this case the consulting firm may not have anything to do with the construction at all. Even if the firm on its own initiative can visit the construction site and follow the whole job, the soil mechanics firms are often too busy to have time enough for an effective control of the construction. Several failures have occurred under similar conditions. The project has been altered without asking the soil mechanics consultant ; or the assumptions on which the design is based do not correctly represent the actual conditions during construction. As an example may be mentioned the quay construction in Viipuri, with stonewalled timber caissons back-filled with sand filling on a place where the subsoil consists of soft clay and silt layers to moraine bottom layers about 15 metres under sea level.

According to the recommendations given by the soil mechanics consultant, the caissons had to be placed on gravel filling reaching to the bottom moraine. The excavation of the soft clay and silt layers and the filling of gravel was, however, not sufficiently controlled, as the supervisor thought the presence of a thin silt layer under the caissons would not be dangerous. So the caissons in reality were built on a gravel bed resting on a thin silt layer above the hard moraine bottom and as a result the quay construction failed during the back-filling operation. The causes of the failure and the responsibility for the costs of the reconstruction were later on analysed by the law court.

A drawback which I believe is common for many soil mechanics investigations is the shortage of time. In some cases a report or preliminary recommendation is wanted only a few days after the beginning of the subsoil explorations, or after signing the exploration contract. It may be necessary to make limitations in the sounding and sampling programme, and time-consuming laboratory tests, for instance consolidation tests. The need for complementary investigations later on is, of course, even greater than normal in this case. More time is often needed for the design period as a whole. The situation may, however, be better in those countries where

it is the custom to make all drawings and specifications ready before the construction job is started in the field.

Finally I should like to quote some statements made by Prof. Terzaghi, his conclusion to the paper *Consultants, Clients and Contractors* :

" ...in most engineering organizations, design and supervision of construction are still divorced, though this fact may be camouflaged by a small soil mechanics department with no function other than providing the design department with the basic data for design. If a consultant is invited by an engineering organization with such an administrative setup to cooperate on a project in the design stage, he should watch his step. First of all, he should turn down the assignment unless it involves the duty to remain in active contact with the project until the end of the period of construction, and to inspect the job whenever he considers it necessary. In order to be able to perform his duty he must get detailed weekly reports informing him of all those observational facts which have a significant bearing on the validity of the design assumptions. Such a report can be prepared only by a competent soils engineer who stays on the job permanently. Second, if the consultant accepts the assignment, he should find out as soon as possible whether or not the inspection of the construction operations on the job is satisfactory. If he arrives at the conclusion that the inspection is inadequate and his efforts to ameliorate the condition are unsuccessful, he should submit his resignation, leaving no doubt concerning the reasons which compelled him to do so. " (p. 14).

Le Président :

Thank you for your contribution.

M. LAZARD

On m'a demandé de dire quelques mots sur la façon dont la Mécanique des sols est appliquée dans les Chemins de Fer Français. Vous savez que les Chemins de Fer Français sont la plus grande entreprise de France. Je pense que c'est à ce titre que quelques renseignements peuvent être intéressants.

Dès mon retour du Congrès de Rotterdam, je me suis occupé de créer une équipe de pénétromètre, et comme les lieux d'accès pour les Chemins de Fer sont souvent très difficiles — on ne peut y accéder que par les voies ferrées — il a fallu monter le pénétromètre sur rails, ce qui a pris quelque temps. Dès 1951, l'équipe a commencé à fonctionner, et depuis deux agents sont affectés à cet appareil : ils se promènent constamment dans toute la France, partout où de grands projets sont envisagés.

Plus récemment, il y a maintenant un an et demi, nous avons pu doter chacune des Régions (entre lesquelles le Chemin de Fer Français est divisé) de petites sondes extrêmement commodes à utiliser, de sorte que non seulement nous avons le pénétromètre mais nous pouvons prélever de petites « carottes ».

D'autre part, les Chemins de Fer Français n'ont pas de laboratoire spécialisé en mécanique des sols, et je le regrette très vivement. Nous ne sommes pas aussi bien dotés que certaines régions des Chemins de Fer Britanniques, par exemple.

Nous faisons appel, comme je vous l'ai dit, aux laboratoires officiels de France, et à tous les ingénieurs conseils de France, quand il y a des problèmes difficiles.

Certains problèmes, quelquefois, sont spécifiques aux Chemins de Fer. Il en est ainsi, par exemple, du problème du renversement des fondations qui soutiennent les caténaires des lignes électrifiées. J'ai fait, au Congrès de Londres, une communication sur ce sujet, et hier soir je faisais encore des

essais, pas très loin de Paris, essais qui d'ailleurs n'ont pas donné entièrement satisfaction.

Dans le domaine du personnel qui, chez nous, s'occupe de Mécanique des sols, j'ai suivi tous les Congrès depuis Rotterdam ; mon collaborateur, M. Carpentier, a été à celui de Londres, et au Congrès de Paris nous sommes trois dans cette salle qui participons aux travaux. L'un d'entre nous a même publié un livre sur les fondations.

Je crois que c'est tout ce qu'il y a à dire sur le sujet, Monsieur le Président.

Le Président :

Je remercie M. Lazard.

I am very sorry I cannot call for contributions from the floor ; perhaps at the end of the second question we could continue.

I would like now to take the third subject, *high void ratio soils*, and I ask Mr Stefanoff to start the discussion.

M. G. STEFANOFF (Bulgarie)

La question des sols de loess et d'autres sols pas suffisamment consolidés est traitée également dans quelques autres sections des rapports (1/57), (3 A/5), (3 A/20) et (3 A/8).

Les rapports présentés ont confirmé que les sols de loess de différents sites géographiques ont des qualités différentes et que plus particulièrement les uns s'affaissent quand on les trempe et les autres ne s'affaissent pas. Voilà pourquoi il est naturel que le saut dans les courbes dans la Fig. 5 du rapport (1/57) de E. Schultze et P. Kotzias soit trop petit, parce que le loess du Rhin ne s'affaisse pas. D'autre part, comme on peut le voir dans le rapport de A. Beles et I. Stanculesco (3 A/5) Fig. 2 et dans le rapport de W. G. Holtz et J. W. Hilf (3 A/20) Fig. 9, le loess roumain comme le loess du bassin du Missouri s'affaissent énormément, ce qui est dangereux pour les bâtiments qu'ils supportent.

Dans les mêmes rapports et dans celui de M. Litvinov (7/4) sont décrites les mesures que l'on doit prendre pour empêcher l'affaissement du loess. Plusieurs des méthodes sont élaborées et utilisées avec grand succès en U.R.S.S. [1, 2, 3, 4]. Dans les rapports mentionnés ci-dessus est démontrée l'importance des études préalables pour constater le degré de possibilité d'affaissement. C'est dans le rapport (7/7) que nous avons proposé une méthode rapide de sa détermination en laboratoire. En outre, je partage l'opinion de notre Rapporteur Général, que pour chaque loess on doit voir si les conditions de la méthode proposée sont remplies.

Le rapport de A. B. Brink et B. A. Kantey (3 A/8) confirme l'opinion des autres auteurs [1, 2] : l'affaissement peut se produire aussi dans d'autres sols comme dans le cas d'un granit en décomposition.

Références :

- [1] ABELEV, J. M. (1948). *Principes de recherches et de constructions sur sols macroporeux*, Moscou.
- [2] DENISSOV, N. J. (1953). *Propriétés constructives des terres de loess* (2^e édition), Moscou.
- [3] ABELEV, J. M. and ASCALONOV, V. (1957). « Stabilisation des fondations de constructions sur terres loessoides ». *Proc. 4th. Intern. Conf. on Soil Mechanics and Foundation Engineering*, London.
- [4] LITVINOV, M. (1959). « Main Requirements on Designing and Construction Methods for Thermal Soil Stabilization ». *Publishing House of Academy of Construction and Architecture of U.S.S.R.*

Le Président :

Je vous remercie, M. Stefanoff.

Je donne maintenant la parole à M. Henry.

Dans son aménagement, la Compagnie Nationale du Rhône rencontre souvent, notamment en bordure du fleuve, des dépôts limoneux très légers : le cas s'est produit par exemple pour la partie amont du Canal d'amenée de la chute de Montélimar.

La composition granulométrique moyenne des limons en cause est la suivante :

Sable moyen (0,2 à 0,4 mm)	20 pour cent
Sable fin (0,1 à 0,2 mm)	35 —
Sable très fin (0,05 à 0,1 mm)	25 —
Silt (0,005 à 0,05 mm)	15 —
Argile	5 —

D'après une opinion assez répandue, les dépôts hydrauliques seraient compacts et denses. En l'espèce il n'en est rien, les densités sèches étant très faibles : ces densités sont en général comprises entre 1,3 et 1,4 avec un minimum de 1,22 .

La question se posait de savoir si on pouvait conserver un tel sol en soubassement de digues. De toute façon, un pareil sol est fort sensible à la pression de courant et il faut le protéger à ce sujet par un filtre efficace, mais de tels sables peuvent en outre présenter un effondrement de leur structure, lorsqu'ils sont à la fois saturés et soumis à un cisaillement. Dans ce cas, le maintien sous digue d'une lame du limon en cause non remanié, aurait pu donner lieu à un tassement appréciable, de l'ordre d'une vingtaine de centimètres, susceptible de désorganiser la digue, et peut-être d'ouvrir un chemin d'eau dangereux. Il a donc été procédé à des essais pour savoir si le sable en cause était susceptible du phénomène d'effondrement de structure.

On a exécuté des essais de laboratoire sur des échantillons non remaniés et saturés, qui ont été soumis à des chargements progressifs ou brutaux ainsi qu'à des vibrations. Le tassement maximal a été de 4,5 pour cent et on n'a rien observé qui pût ressembler à un effondrement de structure.

Les limons en cause sont inondés à peu près tous les ans. L'inondation doit avoir pour effet de les saturer, et lors de la décrue l'abaissement de la nappe doit entraîner une surcharge appréciable tant que l'équilibre n'est pas établi. Ce processus naturel constitue un mode de chargement des limons en cause, accompagné d'une saturation, et cependant, malgré les inondations répétées, les sols en question sont restés extrêmement légers.

On ne s'est pas contenté de cette expérience naturelle, et on a exécuté divers essais *in situ*. Le limon en cause étant chargé par un remblai en sable et gravier présentant un talus de 3/2, on a taillé le limon à 45° à partir du pied de talus. On obtenait ainsi des conditions de cisaillement beaucoup plus graves que dans les digues, et le limon dont la partie basse était saturée, a parfaitement résisté.

On a essayé de provoquer l'effondrement de la structure en faisant au voisinage des explosions : il ne s'est produit aucun effondrement, ni même aucun tassement.

En présence de ces résultats négatifs, nous avons admis que le danger d'effondrement de la structure n'existant pas pour ces limons, ou tout au moins qu'il ne pouvait se manifester que pour des charges très supérieures à celles de nos digues. Il restait donc à pallier les risques dus à la pression de courant. Des essais préliminaires ayant montré que les sables et graviers, qui se trouvent partout sous le limon de surface, sont susceptibles de jouer le rôle de filtre par rapport au limon en cause, il a semblé que le limon non remanié pouvait être laissé sous la digue, pourvu qu'il soit bloqué par des couvertures en sable et gravier d'épaisseur suffisante. La lame de limon qui est ainsi laissée en soubassement de digue est soumise à un écoulement à peu près

horizontal qui se fait avec un gradient très modique, de l'ordre de 5/1.

Les résultats ont été favorables, aucun tassement n'ayant décelé dans les digues.

Il n'en reste pas moins que l'on peut se poser les deux questions suivantes :

1. Existe-t-il un critère pratique susceptible de faire prévoir qu'un sol vacuolaire est, ou non, susceptible de s'effondrer lorsque, étant saturé, il est soumis à un cisaillement ?

2. Ainsi que l'a fait remarquer l'un des auteurs de rapports, le prélèvement d'échantillons intacts de tels sables est très difficile, et malgré toutes les précautions prises, le simple transport entre le chantier et la laboratoire est susceptible d'altérer l'échantillon. Par ailleurs, on n'est jamais bien certain d'obtenir en laboratoire une saturation complète.

Dans ces conditions, et c'est là la question que je pose, vaut-il mieux, pour caractériser ces sols à l'égard du risque d'effondrement dans le cas de saturation et de charge, procéder à des essais de laboratoire ou à des essais *in situ* ?

Le Président :

Je vous remercie, M. Henry. Messieurs, vous avez entendu ces deux questions. Je donne maintenant la parole à M. Denissov, spécialiste pour les loess.

M. DENISSOV (U.R.S.S.)

I should like to say something about the nature of soil subsidence which is due only to soil moisture increasing without any change of stress. It is wrong to think that subsidence due to water infiltration is a characteristic property of loess only. One knows that other unsaturated soils except clear sands, as a rule, with low liquid limit, have this property; and it is known also that some typical loess has not this property.

What is the reason for the subsidencial properties of loess and other analogous soils? One thinks that there is a high void ratio of soils, but it is not true, because many clays have a high void ratio but have no subsidencial properties. Subsidence as a result of water infiltration can be characteristic for the under-consolidated soils only. Only soil with cohesion can be in an under-consolidated state. The subsidence can occur if the bond between particles is not waterproof.

It is well known that the terms "normally consolidated" and "over-consolidated" are common and useful in modern soil mechanics. It is a pity that the term "underconsolidated" is not often used in our practice, for it is as convenient as the former two. The under-consolidated state of soils in natural conditions is very dangerous for construction, especially for irrigation. One knows that the subsidence near an irrigation canal can reach a depth of two or three metres. The under-consolidated state of soils is due to their formation in dry climatic conditions. The soils can be in the underconsolidated state until they can keep a low water content.

Water infiltration in these soils is accompanied by a drop of strength and additional consolidation occurs. If the degree of saturation is near unity, the soil becomes normally consolidated and loses subsidencial properties. If in this new condition the water content decreases without any shrinkage (which is characteristic of soils with low liquid limit) they remain in a normally consolidated state. In the process of loading these soils they will not be able to consolidate effectively and the curve of consolidation will be above the curve of normally consolidated soils. Therefore an unconsolidated state of soils can also be due to the construction work.

In the case of water infiltration the new subsidence will occur. It will be accompanied by an additional settlement of the structure, and may be a failure.

The soils with subsidencial properties we find in regions

with dry climatic conditions. They have low water content which is an important index of the possibility of keeping the unconsolidated state of soils.

I can say to my colleague Mr Henry that we have methods for determination of the degree of under-consolidation. These are described in our literature and they are based on the viewpoint about subsidence as external phenomena of the process of transformation of the soil from underconsolidated state to the more stable normally consolidated state.

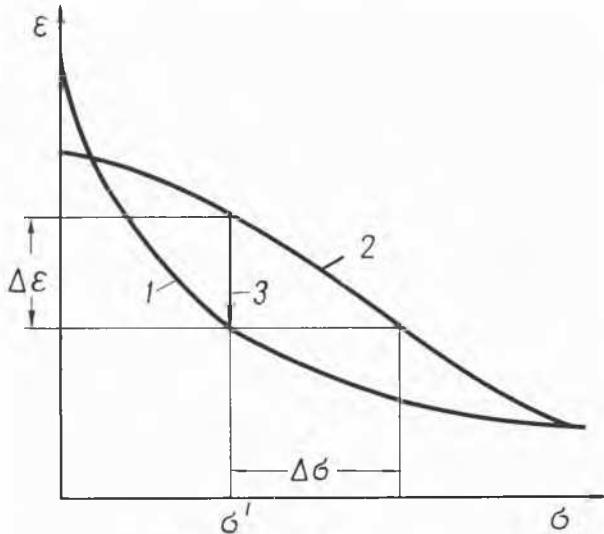


Fig. 1

Fig. 1 gives the idea of this process. Curve 1 corresponds to the consolidation of the soil in the saturated state and this curve illustrates the process of normal consolidation. Curve 2 corresponds to the low moisture state of this soil, and a considerable part of this curve passes above curve 1. This part of curve 2 corresponds to the under-consolidated state. Water saturation of the underconsolidated soil under pressure σ' produces additional consolidation which is illustrated by the vertical line 3. It gives the decrease of void-ratio equal to $\Delta\epsilon$. In order to make this decrease of void-ratio take place only as a result of pressure increase, this increase must be equal to $\Delta\sigma$. In other words the saturation of underconsolidated soil of low moisture has its equivalent as pressure $\Delta\sigma'$.

In connection with wide distribution of loess and other underconsolidated soils in many countries of the world it is desirable and necessary to carry out the broad exchange of the research results and of the opinions of specialists of these countries. This discussion, in which many specialists of Bulgaria, France, U.S.S.R., and the U.S.A. and of many other countries would take part, would undoubtedly be fruitful.

Le Président :

Thank you for your remarks. Mr Stefanoff would like to continue the discussion.

M. STEFANOFF

Je voudrais confirmer l'opinion de M. Denissov, et dire que c'est également l'avis des autres auteurs ; par exemple, dans le rapport de MM. A.B.A. Brink et B.A. Kanley (rapport 8 de la section 3 A) il est confirmé que l'affaissement peut se produire aussi même dans un granit en décomposition.

Maintenant, je donnerai une réponse à la deuxième question

de M. Henry. Je serai très bref. Les résultats des recherches sont sûrs lorsqu'on les effectue au laboratoire et également *in situ*. Les essais de laboratoire sont beaucoup plus faciles à effectuer, et c'est pourquoi on doit toujours les faire, mais quelquefois les résultats ainsi obtenus ne sont pas assez clairs, en raison des motifs invoqués par M. Henry, et principalement en raison de la destruction des échantillons. Dans ce cas, on est obligé de faire également des essais *in situ*.

On est certain qu'un sol de loess peut ou non s'affaisser quand les résultats des essais *in situ* confirment les résultats obtenus au laboratoire. En outre, je voudrais dire que le sol cité par M. Henry n'est pas un loess. Je vous remercie, Monsieur le Président.

Le Président :

Merci bien, M. Stefanoff. And now I would like to ask Mr Turnbull to give some ideas about the problems of loess in the Mississippi Valley.

M. W. J. TURNBULL

I was very much interested in the panel paper presented by Mr Denissov on the experience in Russia with loess as a typical high-void-ratio soil. I was author of a paper entitled "Utility of Loess as a Construction Material" for the Second International Conference on Soil Mechanics and Foundation Engineering held in Rotterdam in 1948, which was published in volume V, pages 97-103, of the Proceedings of the conference. The paper was based on experience gained during six years of laboratory and field investigations for and the design and construction of 44 earth dams. These earth dams were of homogeneous sections consisting of loess soil placed on foundations of in-situ loess soils ranging from 30 to 200 feet in thickness. The dams ranged from 35 to 85 feet in height. I will cite a few outstanding features encountered in construction which verify several of the points made by Mr Denissov.

A dual problem existed concerning estimates of settlement of the loess soil foundation. One was the estimation of settlement in the dry under the load of the dam, and the second was the additional settlement which could be expected upon raising the water in the reservoir. Predicted settlements ranged from 2 to 6 feet, depending upon the height of the dam and the thickness of the foundation ; it was further estimated that approximately half the settlement would develop in the dry and the remainder would develop upon saturation. In actuality, in most instances the proportional settlement in the dry and saturated conditions was about equal to that estimated, but the total settlement tended to be greater than that estimated. Some of the dams existed at least a year in the dry, and the settlement was in reasonably consistent order. Upon the raising of the water in the reservoir, settlement began to occur quite rapidly as the saturated zone passed under the dam. This settlement was not as uniform as that in the dry, the most pronounced instance being the record of one survey marker on the crest of one of the dams showing a drop of approximately 6 inches in 24 hours. A careful check was made of the survey, and the conclusion was reached that the rapid settlement had actually taken place. The only way in which this might be accounted for is a possible large-scale arching in the area under the bench mark and the sudden breakdown of this arch. In numerous instances of high settlement along canals, borings indicated that the loess consolidated under its own weight under the rising ground-water mound. In several instances, lenticular cavities up to a foot in thickness were found above the saturated loess soil. This undoubtedly was produced by arching of the dry soil over the saturated soil as a result of the soil consolidating under the ground-water mound. A condition similar to this may have developed under the dam.

Another instance of pronounced settlement was in the case of one dam about 65 feet high with 40 feet of water in the reservoir where the crown of the dam in a distance of 200 feet settled vertically a maximum of 5 feet. This undoubtedly occurred as a result of alluvial or coalluvial deposited soils in an old, deep stream channel which was not detected by the investigational borings.

I attribute the success of these loess embankments in resisting these high differential settlements to the fact that the moisture content of the dam during compaction was deliberately, on the average, kept at least two percentage points above standard Proctor optimum. For silty loess materials, this meant a material which was fairly soft and plastic but which retained sufficient stability under side slopes of 1 vertical to 3 horizontal, both upstream and downstream.

Another phase which may be of interest was the large-scale field study conducted on the stability of natural loess slopes. The problem involved was the determination of the slopes to be used and their heights. This was important because with cuts ranging up to 135 feet deep, a great difference in the amount of excavation was involved, depending on the steepness of the slopes. The field experiment consisted of excavating slopes 4 vertical on 1 horizontal to such a height that sloughing started. Predictions, based on laboratory direct shear tests on undisturbed samples of loess soil and on stability calculations by a slip-circle method, were made of the height to which the slope could be cut and still remain stable. It was found that a very reasonable check was obtained between the predictions and the actual heights to which the 4-on-1 slope could be cut. When the height was found at which the slope would not stand on 1 horizontal to 4 vertical, a slope of 1 horizontal to 2 vertical was used and carried to a height where sloughing began. These large-scale field tests accompanied by the laboratory investigations confirmed the use of very steep slopes on this project and further confirm the steep natural slopes found in loess regions.

Le Président :

Thank you, Mr Turnbull. La parole est à M. Woodward.

M. R. J. WOODWARD (Etats-Unis)

In its relatively short life, soil mechanics has developed from an empirical art to a more sophisticated and polished science, and more progress will be made in this direction as a result of basic research, now being emphasised. The need of a realistic approach toward the solution of practical problems, however, must not be forgotten in the excitement of the development of new theories.

Every construction project must go through a stage of preliminary studies, followed by design, construction, and post-construction performance observations. A soil and foundation engineer can and should play a very important role throughout these four phases of a construction project. In performing his services the consulting soil engineer has to deal with architects, other engineers, contractors and frequently with individuals who have no technical knowledge. In the following paragraphs we shall try to outline a typical example of the type of services a consulting soil engineer is often called upon to offer to the construction industry.

Recently an architect asked us to investigate a site upon which several buildings were to be erected for a new high school ; the major portion of the site was to be converted into playgrounds. Prior to contacting us the architect had planned the configuration of the buildings, their relative positions with respect to each other, and the elevations and gradients of the playgrounds.

We carried out our work in two phases. In the first phase we proposed to determine by means of a few borings and

seismic traverses the general soil and rock profile of the site, and using this information to evaluate the proposed plan. As a result of this first phase of the investigation it was suggested that the architect change the building locations and the elevations and gradients of the playgrounds. These suggested changes would eliminate a considerable amount of rock excavation, and would decrease the amount of fill and excess cut material to be wasted.

Following this discussion the architect drew a new plan. Detailed sub-surface investigations were made at the new locations of the buildings. The necessary testing was performed to determine the engineering properties of the soil and to establish criteria for foundation design. On the basis of the data presented to them, the architect and his structural engineer were convinced that the system of piers and grade beams originally contemplated would not be necessary. A new plan was drawn, utilising continuous strip footings with ground-floor slabs on grade.

As a result of this two-phase investigation, substantial savings were realised in the project.

The client also retained us to supervise grading during construction, including the control of the fill placement procedures.

This example presented was chosen because of its simplicity and the modest size of the project. One can easily extrapolate and realise the importance and need for a similar approach to more complicated and expensive projects. In our opinion, however, an engineer in private practice has the duty of offering his services to small as well as large projects. He should provide a safe and economical design even though the project is small and relatively unexciting.

We would like to emphasise the desirability of preliminary studies in co-operation with the architect and the structural engineer. Such studies should be carried out before final plans are made, and should determine the best possible use of the site for the project in question.

A soil and foundations engineer is often confronted with a predetermined situation, and is forced to become, to the best of his ability and resourcefulness, an accessory after the fact. This can lead to very interesting and exciting moments, however, which do not always bring out the best utilisation of the site from the technical point of view.

Le Président :

Je vous remercie beaucoup pour votre contribution. Puis-je demander à M. Haefeli de prendre la parole ?

M. R. HAEFELI (Suisse)

Dans le domaine des sols de grande porosité, je propose de distinguer entre les 3 catégories principales suivantes :

1. Les sols dont les grains sont transportés par l'air et déposés dans l'air comme agent (medium) de sédimentation (*loess proprement dit* et certains « volcanic ashes »).

2. Les sols organiques dont les grains sont transportés par l'eau et sédimentés dans l'eau, comme p. ex. *la tourbe et les dépôts organiques limoneux*.

3. Les sols de sédimentation chimique, comme la *craie lacustre*.

Indépendamment de l'agent de transport et de sédimentation (l'air ou l'eau), une loi générale domine la porosité de ces dépôts : le maximum de la porosité est réalisé quand la vitesse de l'agent de sédimentation (air ou eau) est nulle (air tranquille).

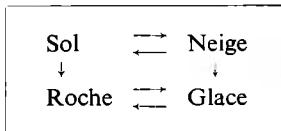
Cette règle est respectée d'une façon extraordinaire dans le dépôt le plus connu de l'atmosphère : la neige.

Exemple. — La neige polaire, une neige à gros cristaux et avec vitesse de sédimentation 0, a une porosité d'environ

97 à 98 pour cent, correspondant à un indice de vide e max. ~ 50 . Les mêmes cristaux de neige ou leurs débris déposés par un vent fort, peuvent former une couche de neige dont l'indice des vides est jusqu'à 30 fois plus petit.

Les porosités données par le Prof. Zeevaert pour les « volcanic ashes » du Mexique sont du même ordre de grandeur que celles des neiges courantes des régions alpines. Et plus encore : les phénomènes de tassement primaires et secondaires et surtout les phénomènes rhéologiques sont, eux aussi, très semblables à ceux de la neige. C'est pour cette raison que j'ai l'impression que pour l'étude des sols de grande porosité, la connaissance approfondie de la mécanique de la neige serait d'un grand secours. En termes plus généraux, je ne doute pas que la rhéologie de la neige et de la glace, qui est plus compréhensible et mieux développée que celle du sol, pourrait jeter un peu de lumière sur tous les phénomènes très complexes des sols où le facteur « Temps » intervient, d'autant plus que les films d'eau d'attraction moléculaire qui enveloppent les grains les plus fins se comportent comme une des différentes formes de la glace.

Il ressort de ce qui précède que, dans les investigations futures de rhéologie des sols, *l'influence de la température* promet d'être spécialement intéressante. C'est aussi pour cette raison que l'on peut s'imaginer un laboratoire futur qui unirait sous son toit les mécaniques suivantes :



Si, à première vue, cette suggestion ne simplifie pas les choses, il ne faut pas oublier que c'est surtout l'étude des analogies et des parallèles — symboles de l'unité autant que des polarités de la nature — qui permet d'approfondir nos connaissances.

A cette occasion, permettez-moi — en terminant ma petite intervention — de féliciter M. le Prof. R. Legget du Canada d'avoir fait un premier pas sur ce chemin d'un développement futur possible en organisant des sections parallèles des sols et des neiges dans le cadre du Building Research à Toronto. J'espère bien que le prochain Congrès au Canada donnera l'occasion de savourer les premiers fruits de cette collaboration entre les spécialistes de la mécanique du sol et de la neige.

Le Président :

Je remercie M. le Prof. Haefeli pour sa synthèse, à la fin de notre discussion.

Je remercie tous ceux qui ont contribué à cette discussion, et je déclare cette dernière section de notre conférence terminée.

(La séance est levée à 11 heures 30.)

Interventions écrites / Written Contributions

M. E. BOTEA (Roumanie)

Le loess et les sables macroporiques occupent des surfaces très grandes du territoire de la République Populaire Roumaine.

La grande sensibilité de ces sols en contact avec l'eau pose des problèmes très difficiles à résoudre quand on doit y fonder toutes sortes de constructions, depuis les canaux d'irrigation jusqu'aux grands complexes industriels pour lesquels les charges transmises par les poteaux atteignent 1 000 tonnes (sur la figure, on peut voir l'ampleur des crevasses de plus de 50 cm de largeur qui ont apparu dans le voisinage d'un canal d'irrigation).

Bien que l'on connaisse des procédés pour traiter ces sols quand on veut assurer la stabilité des constructions qu'ils supportent, procédés qui ont été appliqués chez nous pour résoudre différents problèmes, les phénomènes internes



Fig. 2

qui conduisent au tassement d'effondrement sont encore insuffisamment connus. Par exemple, on ne peut encore expliquer assez clairement le fait que pendant l'inondation de haut en bas du sol, les tassements supplémentaires obtenus sont beaucoup plus grands que dans le cas de l'inondation du même sol de bas en haut (cruies de la nappe souterraine). Nous pensons que dans ce cas, la force d' entraînement hydro-dynamique joue un rôle important.

Des recherches sont entreprises à l'heure actuelle à l'Institut des Recherches Hydrotechniques de Bucarest pour établir les causes qui conduisent à l'effondrement du loess en contact avec l'eau.

Ces recherches sont exécutées d'après le principe recommandé par M. Caquot, par l'examen d'une seule variable à la fois, recherches que je veux vous signaler brièvement.

Un premier résultat a été obtenu en étudiant l'effet du rythme de pénétration de l'eau libre dans le sol. On a constaté que l'effet d'effondrement a la même amplitude, même si l'eau arrive avec une vitesse réduite, que si le terrain est inondé instantanément.

Nous continuons les recherches en ralentissant la pénétration de l'eau par des succions contrôlées, ce qui a montré l'importance des phénomènes superficiels sur le processus d'effondrement du sol.

Pour établir les paramètres de calculs — tenant compte que les caractéristiques des sols loessoïdes présentent des grandes différences même sur des surfaces assez réduites — (porosité qui varie entre 42 et 55 pour cent, nous avons entrepris des études pour résoudre le problème avec des moyens statistiques.

Enfin, on doit souligner l'importance des recherches et des études sur les propriétés des sols loessoïdes en chaque site, parce que bien qu'ayant la même origine, ces sols ont des propriétés toutes différentes d'un lieu à un autre.

Drainage problems in structure foundations of capillary water not connected with Groundwater Table

Water in soils is not uniform. The various forms of this

water have different properties. On the basis of three mutually connected physical properties of groundwater — its mobility, forces determining this mobility and water distribution in soil pores [1], we have evolved four forms for all kinds of ground-water.

Forms and condition of groundwater applied for sand soil drainage aims

Form of water	Main features of groundwater Relation of water to soil strength in road beds		
	1. Water mobility.	2. Predominant influence of forces on formation and motion of water.	3. Water distribution in soil pores
I Vaporous water	Water vapour moves in soil pores from places with higher vapour tension to places with lower tension or with air flow.	Difference in vapour tension in different soil layers.	Does not cause deformation
II Cohesive water	1. Highly cohesive — it may move and undergo evaporation. 2. Loosely cohesive — it may move, evaporating or as film from one particle to another.	Absorptive forces (on surface between mineral and porous solution).	In non-dilative soils does not cause deformation.
III Capillary water	1. Capillary — separated or capillary — immobile condition of groundwater (pendular condition); it may move, evaporating or film motion of loosely cohesive water. 2. Capillary- mobile condition of groundwater (funicular condition); it moves under influence of capillary forces in any direction from moist to dry parts of soil. 3. Capillary- highly mobile condition of groundwater (capillary condition proper).	Capillary (meniscus) forces (at surface separating porous solution — gas phase).	Causes deformation. Water drainage as yet not worked out technically. Separate accumulations of water around contact of particles connected together by their edges. Fine pores entirely filled with water. All pores filled with water, only loose intervals or hollows left free.
IV Free water	1. Seepage condition of free groundwater with complete saturation of soil with water, its motion is in conformance with Darcy Law. 2. Free groundwater in soil flow condition, its motion is in conformance with Darcy Law.	Gravity forces (forces of own weight).	Causes deformation. In sand and sandy clay soils water is easily drained.

Table 1 shows that it is necessary to remove capillary water from the soils. Where sand is the material for individual members of the construction, for instance — in foundation beds, roads and airfields, capillary water has an important effect, in aiding deformation and destruction.

Groundwater or atmospheric water penetrating into the sand layer of roads or into the sand ballast of railways, first of all becomes capillary water, which cannot be removed from the sand by the usually employed drains. An important feature of capillary water is that it is similar to absorbed water and does not flow out at excavations (wells, holes, drains) and adheres in the soil pores by surface tension, which exceeds the force of gravity.

For moving capillary water horizontally the difference of soil layers by their classification is of great importance as the capillary water moves from layers with coarse fractions into layers consisting of smaller fractions.

For explaining these phenomena, let us consider the motion of a column of water introduced into different capillaries (Fig. 3).

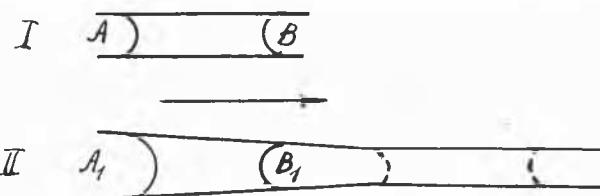


Fig. 3 Motion of water from large-size capillaries to smaller ones.

In the first capillary tube (I) the forces acting at menisci

A and *B* will be the same, and, according to the Laplace formula, equals :

$$K - \frac{2\alpha}{r_a} = K - \frac{2\alpha}{r_b} \quad \dots \quad (1)$$

where

K = normal pressure of liquid for plane surface;

$r_a = r_b$ = radius of curvature of menisci;

α = surface tension of water.

If we decrease the capillary at the left side of *B* in such a way that r'_b becomes less than r_a , or if we take a tapered tube (II), equation (1) changes into :

$$K - \frac{2\alpha}{r_a} > K - \frac{2\alpha}{r'_b} \quad \dots \quad (2)$$

as $\frac{2\alpha}{r'_b} > \frac{2\alpha}{r_a}$. Consequently, the surface pressure of meniscus

A_1 becomes higher than the surface pressure of meniscus B_1 . The water will move from the wider pores to the narrower ones, correspondingly from the coarse soil to the finer fractions [2].

For lowering the moisture content in sand base courses of roads and other structures where the water may accumulate (during spring thawing of soils, for example) drainage of a new design, which we call soil moisture content reducers, with application of drainage material of finer size than the dried soil, is needed.

For ensuring the discharge of capillary water delivered into the moisture content reducers, the following should be taken into account.

The water remaining in the soils when ensuring complete drainage is distributed unequally : at the bottom in the layer of a thickness equal to the height of capillary ascent for the given soil is the capillary water; above the capillary water zone is the absorbed water with the soil pores mainly free. Investigations have shown that in case of delivery from the top of any amount of water into the capillary zone of the soil with free water drainage, the same amount flows out of the soil [3]. Thus, for operation of the moisture content reducer it is necessary to place finer drainage material into the deepened drainage trench than the dried layer in such a way that its thickness exceeds the height of the capillary ascent of the drainage material and that its toe is below the toe of the dried soil layer; for example, by h_k , where h_k is the capillary ascent in the drainage material.

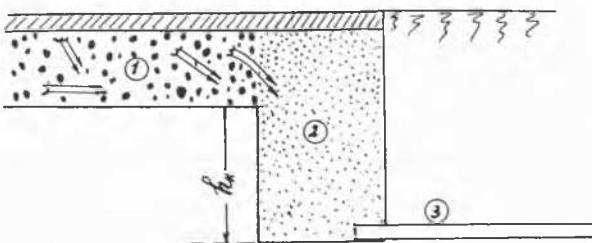


Fig. 4 Diagram of installation of soil moisture content reducers:
1. coarse sand; 2. fine sand; 3. water discharge;
→ movement of capillary water.

Références :

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M. T.E. PHALEN (Etats-Unis)

The factors put forth by Mr Turnbull during the oral discussion are certainly worth deeper thought and consideration. During the past few years, several problems concerning an improper approach to foundation and soils problems by engineers and architects who have not analysed and investigated the problems thoroughly have been noted by many soils engineers. A typical example of this was a series of relatively light stucco, concrete block, and structural concrete buildings that was proposed to be founded on a compacted fill of eight to ten feet which was underlaid by a variable layer of soft yellow clay from four to twenty feet in thickness. One of the structures was so designed that geometrically the northerly half of the structure was founded on caissons, and the other half on spread footings. The other structures were founded on spread footings. With the fact that the soft yellow clay would be sensitive to settlements which could seriously affect the rigid type structure. In the ensuing conversations with the architect and engineer, it became apparent that no attempts had been made to examine the differential settlements due to the overburden of fill and the effects of these settlements on the brittle structure and that the choice of caissons in one half of one unit had no rational basis for selection, particularly when differential settlements are considered.

Here is a typical example of what Mr Turnbull depicted as the result of the men in the field of soil mechanics not asserting and placing soil mechanics and foundation engineering in its proper perspective as a mature field of science. The typical example noted here is one of several such typical examples of how the profession has neglected the advancement of soil mechanics.

Mr Turnbull's remarks concerning the advancement of soil mechanics as a basic field of science, the apparent lack of proper professional status, and the corrective measures that can be applied are heartily endorsed by the writer.

M. T. E. PHALEN

This short summary will indicate recent investigation concerning another high void ratio soil other than loess. The facts outlined herein are based upon the author's experience over the last five years both in the field and from laboratory investigation. Separate papers on various topics have been prepared for publication.

Peat, unlike loess, is not an aeolian deposit, but rather macroscopic and microscopic fragments of decayed organic matter generally located in low-lying water sheds. Laboratory tests on the peats examined are summarized as follows :

Specific gravity of solids	1.9
Void Ratio	4.2
Porosity	0.79
Water Content	207 per cent
Degree of Saturation	93.2 —
Per cent Organic Matter of Dry Solids	42.2 —
Unit Weight (pounds per cubic foot)	70.1
Dry Unit	7.7

Atterberg Limit Tests were made but the results were meaningless due to the fibrous organic materials present.

The storage of samples was found to require attention so that original moisture content would be unaffected. Consequently all samples were stored in a room of constant 72°F. temperature and 100 per cent humidity and were rotated 180° every day. If this was not accomplished it was noted that the water content throughout the sample changed. The laboratory tests for consolidation were conducted on standard consolidation devices and certain series of samples of specially designed six inch diameter and twelve inches in height. In addition, foundations founded on variable depths of peat up to twenty-eight feet in depth were observed from a period of one month to eighteen months, and observations on the technique utilized to underpin these structures founded on this material has been carried on for three years. The laboratory tests were all made on completely remoulded samples under the following conditions.

1. Variable loads in standard consolidation apparatus.
2. Variable loads in special cylinders maintaining an atmosphere of 100 per cent moisture content.
3. Variable loads in special cylinders mixed with various percentages of gravel maintaining an atmosphere of 100 per cent moisture content.
4. Combinations of 2 and 3, but exposed to drying in air.

The duration of the tests lasted from a few minutes to six weeks.

A typical consolidation plot is shown in Fig. 5. The results of laboratory and field information show surprisingly good correlation as to the types of curves to be expected. The consolidation plots have been made on arithmetic, semilogarithmic, and logarithmic and have been plotted according to the technique recommended by Taylor for clays. From all of these plots the author has concluded that the logarithmic plots give the best results.

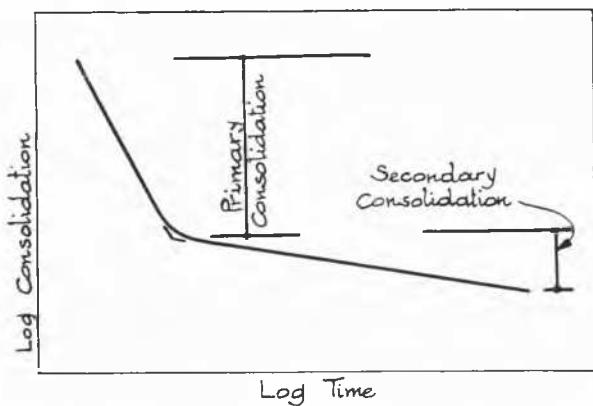


Fig. 5

Without going into specific detail the following equations depict our opinions as to the factors that affect the consolidation of peat.

$$C_t = C_1 + C_2 \quad (1)$$

where C_t is the total consolidation
 C_1 is the primary consolidation
 C_2 is the secondary consolidation

and $C_1 = f(P_c) \quad (2)$

and $C_2 = t^k \quad \dots \quad (3)$

where t is the time

k is the rate of consolidation

and where k becomes a function of

$$k = f \left[\frac{1 \ln (P_c, t, k_1, C_1)}{1 \ln t} \right] \quad (4)$$

$= k_1 = \text{coefficient of permeability}$

Our investigations indicate that the consolidation phenomena can be described by the following equation.

$$C = f(P_c, k_1, t) \pm f(g) + f(e, n, \Delta T) f(f_e) + f(d_r) \quad \dots \quad (5)$$

where

P_c is the surcharge

k_1 is the permeability

t is the time

g is the volume of gaseous products present

e is the void ratio

n is the porosity

ΔT is the change in temperature

d_r is the state of drying

f_e is the fibrous content

Our investigations also show that an approximate range of the coefficients of permeabilities are from 10^{-5} to 10^{-7} cm/sec. and that the approximate formula for coefficients of permeability derived from test results is quite adequate and is given as follows.

$$k_1 = \frac{C^2}{2 P t} \quad (6)$$

where

C is the total consolidation

P is the surcharge

t is the time

Much of this work has been correlated by an unpublished master's thesis conducted under the author by David Ghiglio at Northeastern University. Current work has recently been in studying the primary consolidation process, which should be completed within the year.

MM. I. V. POPOV, E. G. BORISSOVA, L. C. TANKAJEVA
(U.R.S.S.)

The Nature of Settling and Piping Phenomena in Loess Soils.

This paper is devoted to the investigations carried out by the Moscow University on karst and piping phenomena in loess loams of North Ural and settling and piping properties of loess soils in South Tadzhikistan.

The purpose of the investigations was to reveal the character of such bonds between the soil particles, the disturbance of which by water involves the development of piping and settling.

In accordance with the opinions of several authors stated mainly on the basis of general considerations, these phenomena may be caused by : the wedging action of the film water (N. J. Denissov), cement dissolving (S. S. Morosov), the simultaneous action of both these processes (I. V. Popov), etc.

The North Ural soils are heavy loams of alluvial and glacial origin, non sulphatic and non carbonatic. The content of light soluble salts amounted to 0,05-0,06 per cent. The clayey fractions of loams consisted of montmorillonite group minerals and hydromica. The organic matters content amounted to 0,2-0,3 per cent. Exchange capacity equals to 3-37 m-eq. for 100 g of soil. Exchangeable cations : 21-23 m-eq. Ca^{2+} , 4-6 m-eq. Mg^{2+} and 1-6 m-eq. H^+ .

The water attacking action on loams was investigated using two methods of treatment : (1) by repeated treatment of samples by distilled water (in 1:200 proportion) without saturation of water with CO_2 and by distilled water saturated with CO_2 ; (2) electrodialyzing the water suspension.

The water action results upon the samples was examined by filtrate chemical analysis, investigations of exchangeable cations composition, of clayey fractions mineralogical composition and of absorbing capacity of soils to water.

The content of mineral matters passing into solution after 10-times treatment amounted to 2.06-2.78 per cent when treated with distilled water and to 3.72-4.55 per cent when treated with water saturated with CO_2 ; this greatly contributed to the increasing of the soil common salts content. The filtrates investigations by means of the electronic microscope revealed that the clayey matter itself passes into the water solution in the form of semitransparent and non-transparent flakes having dimensions less than 0.3 m. These flakes were not revealed by the water turbidity and did not give any noticeable opalescence.

The micro-aggregate analysis of the loams after 10-times water treatment (and after treatment with CO_2 saturated water) showed the decreasing of clay particles content; after electrodialysis it showed the increasing of clay particles content. X-ray and thermal analyses of clayey fractions of loams before and after the dialysis did not reveal any change in their mineralogical composition and confirmed therefore that water brings about only dispersion action upon the soils, accompanied with the considerable replacement of the exchangeable Ca and Mg cations by the H ions (hydroxonium ion).

The increasing of the soils' hygroscopic moisture involved by the Ca and Mg cations replacement is caused in our opinion, by the wedging action of water, penetrating between clayey crystal packs, this water being the component of hydroxonium hydrated ion $\text{H}_3\text{O}\cdot\text{H}_2\text{O}$. The same is shown by the results of the loams' microaggregate composition analysis after their electrodialysis. In spite of the decreasing of the medium pH (pH 5.9) the content of clayey fractions is noticeably increased. On the basis of results obtained from the investigations the soils mentioned are referred to the soils with the coagulation structure type. Nevertheless, besides the bonds causing this type of structure, there are two more kinds of bonds, namely : the bonds inside the two - and three storey layers and interlayer bonds, caused by exchangeable cations (anchicrystalline bonds after I. V. Popov).

When water is acting upon the soils the molecular bonds are destroyed at first. Soil moistening causes the strength decrease; the increasing of the dead weight of the soil itself due to the pore water content may be the cause for soil caving in, forming karst caverns.

The destroying of anchicrystalline bonds in the mentioned soils requires, besides large quantities of water percolation, a considerable time.

Finally, the strongest bonds in these soils are the bonds inside the crystal packs of the clayey minerals; it is experimentally confirmed by the invariability of the clayey minerals' composition and by the negligible degree of clayey minerals decomposing by the electrodialysis.

The Tadjikistan loesses are represented by the light loams, coarse silted, with the plasticity index of 7.8. However, on the basis of the analyses carried out with the employing of dispersgators (natrium pyrophosphat), the content of particles less than 5 μ in these soils amounts to 15-17 per cent. Taking into account their mineral composition, the clayey fractions of these loesses may be regarded as hydromica and magnesium silicates. Their exchange capacity is about 37-38 m-eq. for 100 g of clayey particles. The carbonate content in the South Tadjikistan loesses amounts to 18-20 per cent. Among the soluble salts gypsum prevails. The

content of the latter is about 2 per cent. The content of readily soluble salts represented by natrium and magnesium chlorides and sulphates did not exceed 0.4 per cent.

Thus, these soils include the material for coagulation and crystallization bonds formation with the cementation of the dominating in these soils' quartzitic silt particles by common salts.

The experiments for investigation of various kinds of bonds were carried out as follows : the monolithic 10 g loess samples were placed in the medium capable of increasing or suppressing solubility (in the case of clay particles-peptization) of the given loess component, which can take part in bonds formation between loess particles. The comparison of the grain-size distribution of the formed suspensions allowed judgement of the real part played by this component in soil cementation. For instance, when investigating the cementation by gypsum, the soil samples were placed in the 2 per cent solution Mg SO_4 , suppressing the solubility of gypsum and in the 2 per cent solution MgCl_2 intensifying the solubility of gypsum. The investigations of clayey fractions' significance in the structural bonds formation was carried out by comparing the grain-size distribution for suspensions formed at various degrees of colloidal and anchicrystalline bonds destroying. For this purpose the samples were placed in solutions with various ionic strengths and the exchangeable cations composition was varied.

In the same way the possibility was studied, of the formation of relatively stable (against water action) bonds with carbonates and colloidal iron oxides participation forming together with the clayey particles mutual precipitation coagels.

As a consequence of the investigations carried out it may be stated that the majority of loess structural bonds are formed by clayey particles. Gypsum and another soluble salts take practically no part in structural bonds formation. Anchicrystalline bonds play no significant part either. Carbonates participate in mixed carbonate-clayey cement formation; they do not play a significant part in loess particles cementation and the colloidal iron oxides behave in the same way.

Therefore, the investigations of the South Tadjikistan loess structural bonds, which are quite different from North Ural loesses in their mineralogical and grain-size properties, led to the same conclusion — namely, that the loess structure-destroying when loess is interacting with water (the beginning of piping and settlement phenomena) is caused first of all by colloidal bonds-destroying.

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M. B. RZHANITSINE (U.R.S.S.)

Nous venons d'écouter le rapport intéressant de M. Mehra, concernant un domaine important du développement des constructions de fondations et de la consolidation du sol. Je voudrais vous dire, qu'en U.R.S.S., au cours des dernières années, le nombre de procédés de consolidation du sol, dans les constructions de fondations, s'est considérablement accru.

Actuellement, pour consolider le sol sablonneux, nous appliquons des procédés chimiques qui sont basés sur l'injection dans le sol de silicate de sodium, de résine synthétique, de solutions de sels.

Pour consolider les sols loessiques susceptibles d'affaissement, on emploie : la silicatisation, le procédé thermique, l'électrosilicatisation.

Pour le dégel et le compactage des sols (dans la zone de congélation éternelle) constamment gelés, sont employés, le chauffage à l'électricité et l'électro-osmose. En outre, cette dernière méthode est utilisée pour creuser des tranchées « à sec » dans des sols saturés argileux.

Le procédé le plus moderne est l'emploi des résines. Il s'agit de l'injection de solutions de certaines résines.

Le sol sablonneux, peu saturé, est transformé en pierre avec une résistance à la compression de 20 à 100 kg/cm². Ce procédé peut être employé pour consolider le sol sablonneux à coefficient de perméabilité cf. = $3 \cdot 10^{-4} - 5 \cdot 10^{-3}$ cm.sec.

En outre, l'injection des solutions de résines dans le sol loessique susceptible d'affaissement supprime son affaissement et lui communique une résistance de près de 10 kg/cm².

Ce procédé a été appliqué avec succès sur différents chantiers, par exemple pour la consolidation de sol sablonneux à Léningrad, sous les fondations du théâtre d'Opéra Kirov, à l'Usine Métallurgique de Lipetsk, et à autres endroits.

Nous pouvons être sûrs que, dans le proche avenir, la consolidation du sol trouvera une large application dans les constructions de fondations.

G. STEFANOFF, (Bulgarie)

Comme complément au rapport de F.F. Abey (7/1) je voudrais dire qu'en Bulgarie les normes de construction dans les zones sismiques ont varié cette année. Le coefficient sismique est déterminé sans que le soit encore le degré du rayon sismique correspondant au type du bâtiment et de sous-sol.

Au cours de la transformation du code se posa le problème suivant : dans les édifices hauts, les moments dus aux forces sismiques horizontales provoquent de grandes pressions sous les bords des fondations.

Il faut alors augmenter les pressions admissibles des sols. Ces augmentations résultent de ce que l'on admet des pressions jusqu'à la limite de destruction du sol, pressions calculées par exemple par la formule de Terzaghi.

Par rapport aux conditions normales, les pressions admissibles des sols purent être doublées pour des sols saturés, et multipliées par 5 pour des roches solides.

M. H.H. WEINERT (présentée en son nom par M. A.A.B. WILLIAMS (Afrique du Sud))

On a paper by A. Hamrol on « A quantitative classification of the weathering and weatherability of rocks ». (7/3).

In connection with the paper by Mr Hamrol I would like to refer you to the work of Dr H.H. Weinert of the South African Council for Scientific and Industrial Research, who has been studying the weathering of rocks.

Mr A. Hamrol proposes in his paper two simple methods to determine the state of weathering and the weatherability of a rock by direct laboratory tests. The state of weathering is determined from some expression involving the void ratio and the weatherability is indicated by the rate of absorption of moisture. Weathering is classified as "type I" (weathering excluding cracking of any kind) and "type II" (weathering consisting exclusively of cracking).

The use of the term "soundness" for a material depends on the purpose for which it is being considered. A rock which is either fresh or in any state of weathering may thus be sound for one purpose but unsound for another. Dr Weinert has carried out active research in the field of road materials and

the following discussion will deal with some of the general aspects.

The classification into "Type I" and "Type II" weathering does not seem to be entirely satisfactory. A twofold description of weathering is also used in South Africa as follows : *decomposition* referring to the alteration of the rock's minerals and *disintegration* to the mere break-up of the rock. It would appear that Type II weathering corresponds with the outlook on disintegration. It is difficult, however, to imagine how any weathered material can be classified as Type I because the weathering agents must have been able to penetrate a rock which has decomposed to any depth. New minerals are then formed which normally require more space than the original ones and this leads to further cracking and, consequently, to further penetration of the rock by any agent. Even in the case of mineral alterations within a rock due only to loss of pressure, cracking will occur subsequent to the formation of new minerals. Any type of weathering will lead to a destruction of the internal bond of the rock and no rock, however intensely weathered, can be imagined to exist without exhibiting cracks. The impression of no cracking may be obtained from rocks which weather to residual clay, but this is actually the most extreme form of destruction.

The void ratio is undoubtedly an important and handy means of expressing the internal structure and strength of a fresh or weathered rock. A certain degree of either disintegration or decomposition, however, can produce the same void ratio and yet, the performance of the two materials will be very different. The degree of decomposition is more liable to make a material unsound than the degree of mere disintegration.

The degree of decomposition is best expressed by the percentage of those secondary minerals which originate from primary ones under the influence of the atmosphere and hydrosphere. This can be done microscopically if one takes into account that the specific type of a clay mineral, e.g. montmorillonite or kaolinite, cannot safely be distinguished with a polarizing microscope. X-ray investigations must be applied for this purpose.

Studies on the method of determining the soundness of a material from its contents of secondary minerals, which was invested by Scott (1955) [1,5] were carried out in South Africa on a regional basis. This led to a fourfold classification of weathering stages of rocks for engineering purposes. This classification is :

Fresh : less than 15 per cent secondary minerals.

Weathered : 15 per cent — 35 per cent secondary minerals.

Badly weathered and residual soil : more than 35 per cent secondary minerals.

These percentages give an indication of the degree of decomposition. An impression of the degree of disintegration can be formed by the 10 per cent fines aggregate crushing test as invented by Shergold and Hosking (1955) [2, 5]. Further work on these aspects is still under way.

South Africa is a country with a rather wide range of climatic conditions. It has been found in this country that normal laboratory tests, like Atterberg Limits, abrasion, soaking, sodium sulphate or others, are not sufficient to predict the performance of a given material. The performance of material, tested to suit a particular purpose, e.g. as a road foundation material, can vary depending on where this material is used. The environmental conditions have been found to be almost as important as the conditions of the material. The climate at the site of use proved to be the most important factor in regard to the environmental conditions, being superior to the local topography. The potential evaporation of the warmest month and the annual precipitation, in particular, have turned out to be most decisive individual climatic factors and a ratio of them has been established to express their interaction. Temperature was found to be another im-

portant factor which must not be neglected and research is under way on its bearing on the above-mentioned ratio [3, 4, 5, 6].

The comparison of test results with the local climatic conditions was found necessary to give a more definite meaning to laboratory tests.

Mr A. Hamrol has made an important statement on the secondary cracks which originate in a rock sample which is prepared for testing and also states that such cracks can lead to unreliable test results. This effect will occur in all tests on rocks where cracking plays a role and this should always be kept in mind.

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M. ZOUKIANOV (U.R.S.S.)

Calculs et recherches concernant les phénomènes physiques dans les sols et les ouvrages à l'aide des intégrateurs hydrauliques, par la méthode des analogies.

Dans le domaine de la mécanique des sols et des fondations il existe une multitude de problèmes qui exigent des calculs mathématiques très compliqués. Les lois fondamentales des phénomènes et les conditions initiales de la formulation analytique sont connues; cependant la résolution pratique jusqu'à un résultat numérique se heurte à d'énormes difficultés et à beaucoup de travail.

Tels sont, par exemple, les problèmes de transmission de la chaleur en régime permanent et surtout la circulation des liquides dans des milieux poreux en régime transitoire, la diffusion ainsi que d'autres phénomènes de la physique mathématique. L'importance pratique de la résolution de ces problèmes est évidente. Ils sont liés avec les questions de la construction : facteur climatique, surtout dans les pays chauds et froids, filtration de l'eau dans les sols et dans les roches particulièrement en régime transitoire, tassements et stabilité des fondations, etc.

En U.R.S.S., actuellement, pour résoudre de pareils problèmes nous nous servons largement de la méthode des analogies hydrauliques.

En 1934, à Moscou, à l'Institut de Recherches Scientifiques pour les Constructions de Transport, j'avais proposé des appareils hydrauliques pour la résolution des schémas compliqués de thermotechnique. Depuis cette époque, l'appareillage et les applications ont été très développés.

Dans notre méthode des analogies hydrauliques les phénomènes étudiés (changements de température, pressions, concentrations, etc,) sont reproduits à une échelle déterminée des hauteurs et de la durée. On observe les changements de niveau de l'eau dans les tubes de verre de l'intégrateur.

En principe l'intégrateur se compose d'un réseau de récipients verticaux de différentes sections, réunis entre eux par des éléments de résistances hydrauliques fixées et d'une suite d'installations pour soumettre au calcul les conditions aux limites et des facteurs supplémentaires. A tout moment le processus du changement des niveaux de l'eau dans les récipients peut être arrêté et on enregistre les valeurs auxquelles on s'intéresse.

Avec notre appareillage on peut résoudre des problèmes à une, deux et trois dimensions. Les domaines à étudier peuvent être de n'importe quelle forme, le milieu peut être non homogène, les conditions initiales quelconques ainsi que les conditions des limites. On peut prendre en considération des sources et des puits.

Les phénomènes qu'on résoud, aux intégrateurs hydrauliques peuvent être représentés par l'équation différentielle dont l'expression générale est la suivante :

$$e \frac{\partial T}{\partial t} = \frac{\partial}{\partial x} \left(a \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left(b \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left(c \frac{\partial T}{\partial z} \right) + \mathcal{F}$$

où x, y, z, t sont les variables indépendantes.

e, a, b, c certaines fonctions positives de $x, y, z,$

\mathcal{F} une fonction de $x, y, z,$ et $t,$ et dans des cas particuliers de $T.$

Quelqu'un utilisant un intégrateur reste spécialiste en son domaine, mais peut résoudre des problèmes mathématiques compliqués sans être mathématicien.

Actuellement beaucoup de travaux différents sont effectués en appliquant la méthode des analogies hydrauliques. Elle est enseignée en U.R.S.S. dans nombre d'établissements supérieurs, parmi lesquels l'Université de Moscou. La fabrication des intégrateurs hydrauliques est organisée à l'échelle industrielle et ils sont employés dans nombre d'établissements scientifiques et d'enseignement.

L'utilisation de la méthode hydraulique pour la résolution des problèmes de la conductibilité de la chaleur a fréquemment figuré dans la littérature (Emanueli - 1923) (Moor - 1936). Il s'agissait de solutions particulières limitées par la mise au point de l'appareillage. Le travail théorique, exécuté par nous, l'élaboration des appareils, la résolution de problèmes à une grande échelle nous permettent à présent de recommander une application large de notre méthode des analogies hydrauliques dans les recherches et les calculs.

On peut trouver dans les rapports de Congrès internationaux et dans la littérature soviétique des renseignements plus amples sur ce sujet.

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