MAIN SESSION 4
4-e SESSION PLENIERE
ЧЕТВЕРТОЕ ПЛЕНАРНОЕ ЗАСЕДАНИЕ

PROBLEMS OF SOIL MECHANICS AND CONSTRUCTION ON SOFT CLAYS AND STRUCTURALLY UNSTABLE SOILS
(collapsible, expansive and others)

Chairman: Prof. G.A.Leonards (USA), General Reporters: Prof. L.Bjerrum (Norway), Prof.G.D.Aitchison (Australia).
Participants: P.La Rochelle (Canada), Ch.C.Ladd (USA), K.V.Helenelund (Finland), A.W.Bishop (U.K.) G.Ter-Stepanian (USSR), B.B.Broms (Sweden), J.L.Justo (Spain), M.N.Goldstein (USSR), V.Escario (Spain), R.Rosen (Sweden), J.L.Kogan (USSR) B.I.Dalmatov (USSR)

Chairman Prof.G.A.Leonards (USA)

Main Session IV has two parts and two general reports: part I deals with the in situ properties of soft,saturated clays,and part 2 is concerned with the changes in properties of partly saturated soils especially "collapse" and swelling phenomena upon imbibing water. Due to its great importance, the Organizing Committee has arranged for the presentation of a summary of the general report prepared by the late Dr.Bjerrum. This will be followed by some Comments by our esteemed President, Prof.Ralf Peck. We will then proceed with the presentation of a summary of Dr.Aitchison's general report by his colleague from Australia, Dr.Gerrard. After the intermission there will be discussion on the questions raised by the General Reporters.

In Chapter I of his report,Bjerrum reviewed the principles for identifying and classifying deposits of Soft,saturated clays,including:

1/ The geological history,
2/ The variation of water content and Atterberg limits with depth.
3/ Description of the clay fabric based on visual examination in the wet and partly dried condition.
4/ The variation of undrained shear strength with depth; e.g. the vane shear strength.
5/ The existence of a P_c value, indicating a threshold in the compressibility of the clay,as determined by consolidation tests.

It is hoped that in the future all case records of structures on soft ground will include at least this amount of information to identify the clay deposit.

The existence of a P_c-value may be due to an earlier effective stress greater than the overburden pressure. P_o,i.e. an overconsolidated clay; or, to the formation of brittle cementing bonds /for example,the precipitation of calcium carbonate/,i.e. cemented clays; or to the effects of secondary compressions proceeding over long periods of time, which Bjerrum called "aged", normally consolidated clays. "Young", normally consolidated clays are deposits not affected by any of the above factors; e.g.recent delta deposits,clays consolidated by man-made fills, or marine clays in the last stages of leaching which are disturbed due to the resulting volume change. A "Young" clay is characterized by Pc=Po. Bjerrum emphasized the importance of aging effects, which depend upon the plasticity index,as shown in <u>SLIDE I.</u>

In Chapter 2 Bjerrum considered the effects of sampling and sampling disturbance Upon unloading,an uncemented clay is not fully able to resist swelling initially,and this resistance is further reduced with time due to creep effects. The swelling is accompanied by a redistribution of water from the more disturbed outer portions of the sample towards the core so that even if the total volume is maintained inchanged the properties of the sample are altered. For samples of high quality, i.e.thewmechanical disturbance of the sample is relatively small, Bjerrum recommended reconsolidation under Ko conditions prior to shear testing. The effect of such reconsolidation,and of time after sampling, are illustrated in SLIDES 2 and 3 A similar effect of time after sampling on the measured value of Pc was published in the Proceedings of the ASCE Settlements Conference in 1964/. Cemented clays are relatively insensitive to the above swelling effects.

On Chapter 3 Bjerrum turned his attention to the Shear strength of Soft Clays, a topic which was the subject of his doctoral dissertation and which probably remained closest to his heart throughout his active carreer. He began with Hvorslev's /1937/ concept of effective friction and effective cohesion, introduced the observations of Schmertmann, et al /1960-64/ that the effective cohesion is mobilized at much smaller strains than the effective friction and added his interpretations of the effects of time and anisotropy to develop an integrated mechanistic picture of the shear strength of soft, saturated clays. This picture forms the basis of his recommendations for selecting strength parameters to apply in practical problems and an attempt will be made to present its essential concepts.

Although Hvorslev's effective friction is proportional to the existing effective stress, and the effective cohesion is proportional to the equivalent consolidation pressure, it was demonstrated by Schmertmann, et al that their characteristics are different in two main respects: effective friction is mobilized at relatively large strains but is stable with time, while effective cohesion is mobilized at small strains but yields when subjected to sustained shear stresses. From these facts Bjerrum made the following deductions:

1/ After a clay stratum is deposited, mobilized cohesion is gradually transferred to effective friction due to creep in the cohesive bonds until, after a long period of time has elapsed, the existing shear stresses are carried entirely by effective friction. If a shear stress is suddenly applied, its critical value–i.e. the shear strength–can be formulated as follows:

$$\tau_{cr} = \tau_\alpha + \left\{(\sigma_\alpha \tan\phi_e - \tau_\alpha) D_m + k \cdot P_e\right\} \ldots\ldots \quad (A)$$

where, K and ϕ_e are the Hvorslev parameters.

τ_α and σ_α are the existing normal and shear stresses on the plane.

Pe is the equivalent consolidation pressure=Po.

Dm is a factor which is a measure of that fraction of the total available friction in excess of τ_α that has already been mobilized, and is usually less than I.

The mobilization of effective cohesion is assompanied by a breakdown of the soil structure. In undrained shear excess pore pressures develop and failure occurs; in drained shear pronounce yielding takes place at the same stress level. Thus until a breakdown in soil structure occurs, the drained and undrained shear strengths are identical and the maximum value of $\Delta\tau$ that can be added in the short term= the besmin egu A.

2/ If a shear stress $\Delta\tau$ is added to τ_α such that $\tau_\alpha + \Delta\tau < \tau_{cr}$, $\Delta\tau$ will initially be carried entirely by effective cohesion. However, creep will begin at a rate that depends on the magnitude of $\Delta\tau$ in relation to k.Pe; the larger $\Delta\tau$ the faster the creep rate. With time there will be a gradual transfer of shear stress from cohesion to friction and the creep rate will decrease. If $\Delta\tau$ is the available effective friction equilibrium will eventually be established; however, if it is greater, the creep rate gradually decreases to a constant value until a critical shear strain is reached. At the critical shear strain the soil structure breaks down, excess pore pressures develop, and failure occurs. This phenomenon, which Bjerrum called the "time effect", can be considered simply in terms of the time required for creep to reach a critical strain; the time duration is related to the shear stress level as shown in SLIDE 4.

Bjerrum also used equation /A/ to explain the effects of anisotropy. The initial stresses in the ground on any plane are:

$$\sigma_\alpha = P_o(\cos^2\alpha + K_o \sin^2\alpha) \qquad (B)$$

$$\tau_\alpha = P_o(1 - K_o)\sin\alpha\cos\alpha$$

where, Po is the vertical overburden pressure and Ko is the coefficient of earth pressure at rest. K, ϕ_e and Pe=Po can be determined in the conventional way, and Dm can be back figured from a single test in which τ_{cr} is measured. Bjerrum cited some results of a series of simple shear tests recently carried out at NGI by Dr.Soydemir in which predicted and measured anisotropy effects were compared, SLIDE 5, with good agreement. Note that anisotropy depends on both direction and sense of the applied shear stress, i.e. whether it is in the same or in the opposite direction as the existing shear stress, and on the relative magnitude of K.Pe to tan ϕ_e - i.e. the plasticity of the clay. This is illustrated in SLIDE 6, which shows the ratio of Su(α) to Su /vane/ as a function of α and plasticity.

Based on the foregoing principles, it is possible to take high quality undisturbed samples, reconsolidate them in the laboratory under the existing stress conditions, and perform series of tests to evaluate the effects of time, and of anisotropy, on the available shear strength. However Bjerrum recommended in Chapter 4 that at the present time practice should consist of applying empirically determined correction coefficients to the vane shear strengths. His final Chapter is a digest of case records which from the basis for determining these correction coefficients.

In this general report to the ASCE Conference at Purdue in June of 1972 Bjerrum analyzed 14 embankment failures with the result shown on SLIDE 7. In the present report he collected 9 footing failures and 5 load tests as shown on SLIDE 8, and 10 failures of temporary cuts or unsupported excavations, SLIDE 9. He demonstrated that in all these cases neglecting the effects of anisotropy was inconsequential for highly plastic clays and slightly on the conservative side for clays of low plasticity. Accordingly, for embankments, footings and temporary cuts Bjerrum recommended correcting the vane share strength for time effects only, as shown on SLIDE 10. A review of 12 bottom heave failures in strutted excavations led him to conclude that the same correction factors are applicable.

Bjerrum analyzed the results of passive earth pressure measurements on 4 strutted excavations in Oslo, where the applied shear stresses, SLIDE 11, are opposite to the existing ones. The result was that a correction factor on the vane shear strength of about 0.5 had to be applied to obtain agreement between the measured and predicted passive pressures. As the plasticity of the soils was low /Ip 25/, the correction factor must be due mostly to the effects of anisotropy, without applying the correction factor the predicted passive pressures were on the unsafe side by a factor of 2, a result which deserves careful attention in design practice.

Bjerrum also discussed the stability of permanent cuts and of natural slopes, and he outlined some principles for predicting settlements due to consolidation, but my summary of his general report will conclude with a few remarks concerning his interpretation of the bearing capacity of friction piles in soft clays.

Bjerrum maintained that the empirical shaft adhesion factor /a/ obtained from loading tests, and generally assigned a value of I in soft clays by a number of investigators, may NOT be used to predict the long-term bearing capacity of friction piles. This is due to the fact that a is dependent on the rate of loading /decreasing at slower loading rates/ and that long-term creep effects result in a transfer of stress from effective cohesion to effective friction which, along with consolidation of the clay, increases the bearing capacity of the pile. Bjerrum analyzed three loading tests in which the short-term effect of the rate of loading was measured, <u>SLIDE 12</u>. He recommended that similar relaxation tests be conducted routinely during load testing of piles to obtain an indication of the reduction in adhesion at slower rates of loading. In the long-term the bearing capacity will increase, especially-Bjerrum felt-in clays with high values of effective friction /i.e. low plasticity/, but he made no recommendations at this time concerning the prediction of the magnitude of this increase.

In conclusion, I urge that you study carefully this last report by Dr. Bjerrum. I believe it points out many avenues for fruitful research and- as Dr. Bjerrum pointed out- no responsible engineer should use an empirical design procedure without fully understanding the principles on which it was based. In retrospect it seems to me that Dr. Bjerrum may have had a premonition of coming events because be poured every fibre of his energy and of this penetrating thought into his last general report. I thank you very much for your kind attention . Now I wish to invite the delegates to take part in the discussion. Before the beginning of the discussion I wish to call upon Prof. Ralph Peck to make some comments.

Prof. Ralph B. Peck

It is impossible to comment on this session without feeling deeply the absence of Laurits Bjerrum. When you have the opportunity to read and study Bjerrum's report, I am sure you will agree with me that it is one of his crowning achievements. It is surely not the last word on problems of soil mechanics and construction on soft clay, because it raises questions as yet unanswered. But in my judgment it is the clearest and most comprehensive picture we now have of the subject.

The orderly development, so apparent in the state-of-the-art report, was not easily achieved. Those of you who attended the Purdue Conference in early summer of 1972 and the European Conference in Madrid earlier that year will notice that many of the same ideas were expressed by Bjerrum in his reports at those conferences, but that the concepts were still in a formative stage with several inconsistencies and speculations. The report to the present conference is based on several additional field and laboratory investigations carried out specifically to provide basic information regarding some of the concepts.

It was my good fortune, in the period between the Madrid conference and Bjerrum's death, to meet him in Montreal every two months in connection with the James Bay Hydroelectric Project. At each meeting he had new thoughts or a new draft of his state-of-the-art report, and we discussed them earnestly and sometimes heatedly. I wish I could convey to you how much of his life and effort went into the preparation of this report and how carefully he examined each aspect from all points of view.

Some of the conclusions must be used with caution. A major point is the necessity for reconsolidating undisturbed samples under the stress conditions to which they were subjected in the ground. Bjerrum emphasizes quite correctly that this procedure is valid only for the very best undisturbed samples, samples that have not swelled to the point that any inelastic behavior has taken place. Otherwise, the reconsolidation will result in reduction of water content, in higher strengths than exist in the field, and in unconservative results. Hence, the reconsolidation procedure goes hand in hand with the most expert sampling. In many parts of the world, undisturbed samples of the necessary quality are not customarily obtained and use of the reconsolidation procedure will lead to serious errors.

In the comparison of field results with safety factors computed in accordance with conventional procedures, Bjerrum stated quite positively that there is nothing wrong with the basic principle of the presently used methods of computing the stability. This may not be true, and errors in stability calculation may be responsible for some of the discrepancies pointed out in the report. The critical circle determined in an undrained stability analysis may not coincide with the actual failure surface. We recognize that the direction of the planes of failure in an undrained test are determined by an inherent frictional resistance of the material, possibly represented by Hvorslev's effective friction angle. Yet, in undrained stability analyses, we proceed as if the position of the surface of sliding corresponded to a frictionless material. The effect of this discrepancy does not seem to have been fully investigated as yet.

In the state-of-the-art report, Bjerrum points out that the full scale loading test at Ellingsrud throws some light on the problem of progressive failure. The test embankment did not fail at a factor of safety of 1.12 based on laboratory peak values. The clay was highly sensitive. Hence, Bjerrum concluded that progressive failure is a factor of minor importance. Nevertheless, in 1971, in connection with the James Bay Project, a test embankment on sensitive clays failed at a factor of safety of about 1,6 and

Bjerrum felt obliged to recommend that embankments on the extrasensitive materials found at that project should be designed with an allowance for progressive failure.The corrections for the time effect and for anisotropy were insufficient to account for the discrepancy.

Hence,as I pointed out earlier,the report is undoubtedly not the last word with respect to the stability of soft clays. Nevertheless, it demands the attention of every person who deals with such materials and it deserves fully to be called a masterpiece. We are indebted to Laurits Bjerrum for this magnificent contribution

Chairman Prof.G.A.Leonards

Thank you very much Prof. Peck. I would like to ask Dr.Gerrard to present us with his report.

Dr. Gerrard C.M. /Australia/

Mr.Chairman,Ladies and Gentlemen,as you are aware Dr.Aitchison was unable to attent the Conference because of some important unforeseen commitments. He asked me to extend his deepest apology for this together with his hope and desire for the success of the conference and his personal greetings to the many of you here who know him.

You will see that Dr.Aitchison's report is largely of a philosphical nature and in presenting this summary I will endeavour to adhere to what I understand to be his main ideas. Because of the time requirements the discussion will be general and specific papers will not be referred to as such.

By way of introduction we should firstly consider what is meant by structurally unstable soils. For the purposes of this report it is accepted that structurally unstable soils are those that exhibit significant departures from the simplistic stress-deformation rules of basic soil mechanics. However, the point should be made that it may be more profitable for soil mechanics communications to be based on a recognition of the general complexity of soil behaviour and the widespread occurence of structurally unstable soils.

Various authors have reported structural instability in sands due to liquefaction, collapse,and internal erosion,the causual mechanisms being dynamic loading,wetting and loading,or leaching. In silts,frost heave, collapse and expansion are reported due to temperature change and netting and loading.

For clays structural instability may be in the form of expansion,shrinkage and cracking, internal erosion, or liquefaction these forms being caused by moisture change,solute concentration changes or flows,and localized initiation of failure (A full description is given in Table 1).Attention in this report is focussed almost entirely on collapse and shrinking and swelling.

The above descriptions of structural instability lay emphasis on material susceptibility. However,in developing a national framework for communication,emphasis should be placed on the processes of structural instability including the nature of discontinuities in the stress-deformation behaviour. These processes involve the complex interaction of the intrinsic soil properties, applied stress levels and external factors, particularly environment. Structural instability may be categorized according to the relative contributions made by the applied stress levels and external factors as indicated in Table 2 and Fig.1.

It cannot be emphasised too strongly that structurally unstable soils can only be defined by reference to processes in terms of:

a) The structural awangement of the particles, and composition of the constituents of the soil.

b) the history of the environmental status including moisture content, pore pressure, temperature,and the nature of the electrolyte,and

c) the history of previous loading together with a definition of the proposed applied load.

Not unexpectedly, a survey of the literature indicates a relactance by engineers to accept the complexities of such a definition, particularly with regard to the role of environment. Engineering judgements are usually invoked as expediency measures and this has limited scientific development.

Having discussed the question of the definition of structurally unstable soils a second and interrelated question should be considered. This is whether any special circumstances exist that are inhibiting the evolution of the state-of-the art of structurally unstable soils as compared to that for structurally stable soils.

Certain logical stages in the form of a fully integrated study are necessary in the refinement of soil mechanics knowledge and these are set out in fig.2A and summarized below:

a) observations of a phenomenon or its manifestations,
b) formulation of a hypothesis with quantifiable parameters leading to the definition of a physical law that could duplicate such manifestations,
c) gathering experimental evidence in terms of parameters to confirm or modify this law,
d) use of the physical law as a design procedure,and
e) observation of engineering structures in terms of the above parameters.

Structurally stable soils have less complex interactions and it is often possible to introduce short cuts in the above procedure by introducing proven design methods and by using soil identification test results directly in design. These short cuts are shown in figures 2B and 2C. Hence,advances in the state-of the art for structurally stable soils can often be obtained by the integration of scattered results.

The interaction of environmental factors does not permit these types of short cuts to be used for structurally unstable soils. However, the conduct of fully integrated studies is very difficult because of the lack of precision in soil identification procedures, the vanety and problem of adequately quantifying external controls and soil properties, and lack of adequate instrumentation for some field measurements. For this reason appropriate short cuts must be used to solve immediate problems with the penalty of severely limiting advances in the state-of-art. The first of these short cuts, as used by many of the authors, is shown in fig.20 and consists of recognizing the indices of potential soil instability and removing or modifying the offending soil layers.

A less drastic, and hence more versable, approach is that of the stress-path method is shown in fig.2E. Here the correct order and the specific stress response is used directly in design. One of the features of the stress- path method is that it allows the proper use of the effective stress principle. It is recommended that the effective stress principle should not be discounted in structurally unstable soils. The appropriate form of this is given in equation V,1.

We shall now briefly consider the particular forms of structurally unstable soils known as collapsing soils and expansive and shrinking soils.

For collapsing soils, although there is acumulating data on methods of modification or removal (i.e. following the stages shown on fig.20), there have been few if any attempts to conduct a fully integrated study such as illustrated in fig.2A. The state-of-the art is therefore not well advanced. One practical way of improving this situation would be by the adoption of the stress-path method as outlined in fig.2E. The data thereby obtained would allow the engineer to choose one of four possible courses of action:

a) to design for collapse as quantified,
b) to design for avoidance of collapse by precluding the operation of the relevant environmental control,
c) to induce collapse prior to construction, or
d) to remove or modify the soil susceptable to collapse.

In turning to expensive soils it is important to note the significant role that changes in solute suchon can play in producing large deformations. This can be clearly seen in fig.56. On the subject in general there has been a lack of progress within recent years and many unfounded assumptions are still common. This appears to be due to the lack of integration of site investigations, laboratory studies and design procedures together with a reluctance to gather data in quantifiable stress-deformation terms.

Two areas of research on expansive clays particularly require further effort. The first is concerned with rate processes with the development of an appropriate mathematical model and the conduct of fully integrated study as in fig.2A. The second involves the gathering of facts, in quantifiable stress-deformation terms, which define the intrinsic properties of the soils with respect to each external control and which also define the nature and magnitude of each such external control.

In conclusion it is recommended that further discussion be related to the following topics:

1. The present and potential availability of facts in terms of stress-deformation parameters concerning:
   a) the intrinsic properties of structurally unstable soils, including their fabric, and
   b) the nature, magnitude and occurence of each environmental control.

2. The present and potential availability of mathematical or physical models capable of representing the stress-deformation behaviour of structurally unstable soils as related to typical engineering projects. Models should be available for:
   a) collapsing soils, having due regard to discontinuities in stress-deformation response patterns, and
   b) expansive and shrinking soils including the development of physical discontinuities during volume change.

3. The feasibility of undertaking a number of fully integrated studies on a range of structurally instable soils. Major research centres with sufficient capacity to undertake such studies could nominate theri topic of interest to the International Society who may decide to provide financial support for approved projects. As well as conducting an integrated study each research centre would serve as a co-ordinating centre for research on that topic.

The recognition of a number of principal research centres should be coupled by a regional approach to the problems of structurally unstable soils. The definition of land areas of similar environmental controls and intrinsic soil properties would allow some extrapolation of the results of the fully integrated studies. Such extrapolation will be necessary because of the cost of repeating integrated studies. However, it should be emphasised that any form of regionalism should be accompanied by a common of communication in terms of quantifiable stress-deformation parameters. This should ensure that the current parochal form of regionalism is not perpetuated. Thank you Ladies and Gentlemen.

Chairman Prof. Leonards G.A.

Thank you very much Dr. Gerrard for making acquainted with the report of Prof. Aitchison.
Ladies and Gentlemen, it is time for 15 Minutes intermission. After intermission we shall begin our discussion

Intermission

Chairman Prof.G.A.Leonards

I would like to call on Prof.La Rochelle from Canada.

During the last years of his life, Dr.Bjerrum became more and more attracted by the Canadian clays and by the differences in their behaviour when compared to the Norwegian clays; this is evidenced by the elaborate comments on the cemented Canadian clays which have been included in his General Report.

The Reporter has specifically expressed the wish that the discussion on his report "be devoted to a general illumination and enlargement of the recent findings and advances with special reference to their application in practice". I certainly do not have the pretention of shedding a general illumination on the General Report, however, I believe that some of the recent findings of our group at Laval University, Quebec, Canada, in relation to the sampling techniques and their implications deserve to be brought to the attention of this audience.

It is only during the last decade that Canadian engineers became really aware of the presence and the practical meaning of the cementation bonds in the sensitive clays found in eastern Canada. The reason for such a late appreciation of a fundamental characteristic of our clays lies mainly in the fact that, although the cementation bonds are more resistant to the disturbance resulting from a release to the confining in situ stresses, they are probably more sensitive to distortion than the ordinary bonds found in non-cemented clays. When comparing results of tests made on block and tube samples, it was found that even the thin-wall tube stationary piston sampler produced a distortion large enough to destroy most of the cementation bonds (La Rochelle and Lefebvre 1971); the difference in brittleness was such that the peak value of drained compression shear strength measured on the tube samples corresponded more or less to the residual strength measured on block samples. The unconfined compression strengths obtained on tube samples were only half the value given by tests on block samples.

The cause for such a large difference was traced back to the distortion which occurs during sampling. A laboratory study on block samples has shown that the lateral strain produced by sampling when taking into account the total volume change resulting from the intrusion of the tube into the clay mass is six times as high as the strain required to destroy the cementation bonds. From these data, it was evident that the area ratio of the samplers should still be much lower than the 10% usually considered acceptable; there are of course practical limitations to the thinness of tubes which may be used, but other possibilities for decreasing the area ratio of the sampling tubes exist.

The area ratio, as illustrated in figure 1a, is increased by the inside clearance which, moreover, has the disadvantage of allowing a lateral expansion of the sample inside the tube. As a result, a great part of the change of volume due to the intrusion of the tube the clay will take place towards the inside of the tube sampler so that the clay sample will be squeezed in around the cutting edge as illustrated in figure 1b. This effect was thought to be one of the main causes of the disturbance occurring during sampling. Some fifteen years ago, our Swedish colleagues (Kalstenius 1958) have shown that the angle of attack of the cutting edge should be as small as possible in order to avoid disturbance. It was decided therefore to shape the cutting edge at a very small angle (figure 1c) and eliminate the inside clearance. It was thought that most of the volume change would thus take place towards the outside of the sampler. As for the elimination of the inside clearance no appreciable friction was expected to develop inside the tube since sensitive clays are self-lubricating. This contention was confirmed by the small pressure required to extrude the sample from the sampler. However, an unexpected difficulty appeared when it was realized that most of the commercially available tubes had either a slightly taper shape or a slightly oval cross-section with the main axis rotating by up to 45° from one end of the tube to the other. A careful selection of the tubes then had to be made and only those satisfying rigorous geometrical tolerances were retained.

Much better samples could be obtained with these re-shaped 8 cm diameter sampling tubes. However, the same variability of quality within the re-shaped sampling tube was found as in the standard sampler; this variability of quality was mentioned by the General Reporter. When the 80 cm long usable sample is cut in 8 ten-centimeter long specimens (Fig.2), it is found that only the specimens numbered 4,5 and 6 on figure 2 are invariably good, number 7 is often good, 3 is seldom good, 1,2 and 8 are always disturbed. This disturbance may be attributed either to the intrusion of the piston into the clay mass or to the release of stresses with the suction being applied in the cavity as the sampler is pulled out. Continuous sampling only reduced the disturbance to specimen number 3.

The results of unconsolidated undrained (UU) tests are compared in figure 3 with the average vane strength obtained on the site of Saint-Alban in Quebec. Considering the results obtained at the lower level of 22 feet deep, it is seen that the measured undrained shear strength is nearly twice as great as the vane strength. This ratio is moreover about the same as that measured when comparing block and standard tube samples from other sites. It has been the usual practice to consider that the standard tube samples were of an acceptable quality of the UU tests on these samples gave shear strength values comparable to the vane strength. It is seen that in the present case, UU tests on samples taken with reshaped tubes give much higher values of shear strength.

The results obtained at the upper level of 9 feet are not as good since a small upward pulling force was intentionally applied to the piston during the lowering of the sampling

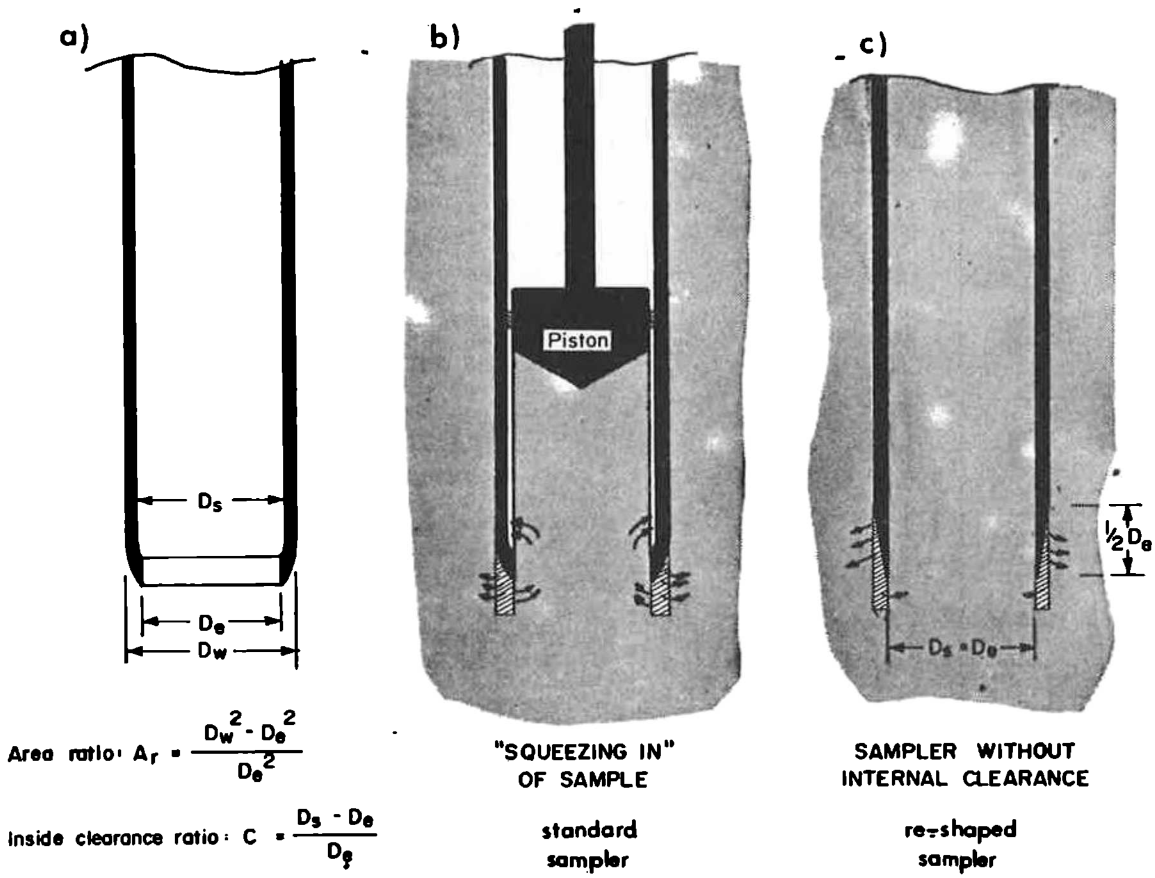

FIGURE 1 - SAMPLING TUBES WITH AND WITHOUT INTERNAL CLEARANCE

tube, thus increasing the "squeezing in" effect mentioned above. The peak of the stress-strain curve was eliminated, as shown on figure 4, with a drastic reduction of the brittleness of the clay.

As the General Reporter has mentioned, a reconsolidation of the sample under the in situ stresses should rectify an appreciable part of the disturbance due to sampling. However, in the case of the cemented clays it was thought that the reconsolidation of the clay would have a negligible effect since it is generally agreed that, once broken, the cementation bonds will not reform. It was then surprising to realize that a reconsolidation of the samples under the in situ stresses restored appreciably the brittleness of the clay as illustrated on figure 4. In the present case, the K_o value on the site, as measured by hydraulic fracturing, was equal to 1.0, so that the isotropical consolidation corresponds to the in situ conditions. The same effect was observed on samples from the lower level (Fig.5) where the strains at failure are lower than 1% for both UU and CIU tests, and where the reconsolidation of the clay results in a marked increase of the brittleness. The changes of volume measured during reconsolidation was much larger in the upper level samples with an average of 4.1% than in the lower level samples with an average of 1.2%, which illustrates the difference of disturbance at the two levels.

No direct comparison was made with standard samplers, however there is some evidence from our past studies where standard samplers were used that the beneficial effect of reconsolidation is far less important, probably du to the partial destruction of the cementation bonds during sampling. It would then seem that the re-shaped sampler described above is more competent in preserving the cementation bonds in our sensitive clays.

The General Reporter has attributed the decrease of strength of the samples to a migration of water from the outer more disturbed and more compressible zone to the central core where negative pore pressure results from sampling; some differences of 3% to 4% were measured in the water content through cross-sections of clay samples from Drammen. The migration of water was also evidenced in the Norwegian clays by the fact that the compression strength obtained from samples which were trimmed and tested in the field immediately after sampling were appreciably higher than the values measured on samples three days after sampling.

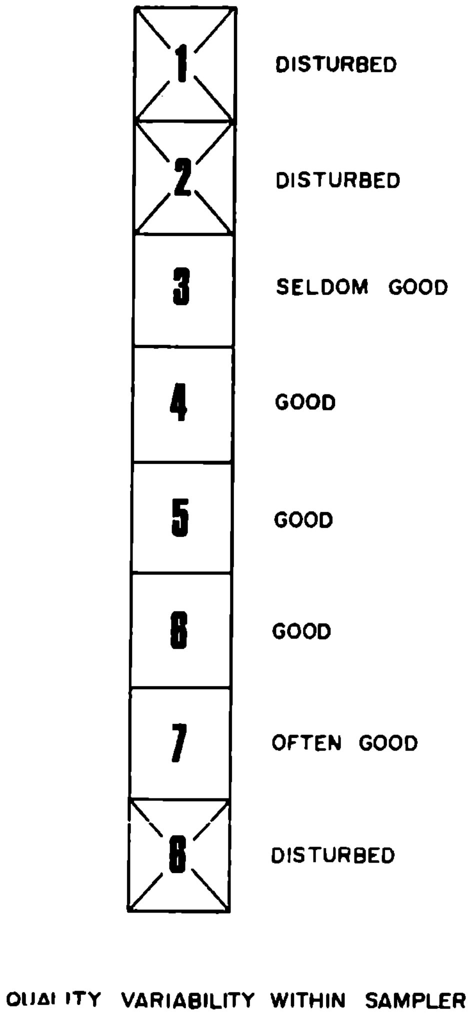

Figure 2.Quality variability of clay specimens within a tube sampler

A similar study made on a cemented Canadian clay from Saint-Alban did not show similar tendencies; no consistent differences in water content between the outer zone and the central core of the samples could be detected even after three weeks of storage and no consistent differences in the compressive strength were observed between the samples tested in the field immediately after sampling or few days later in the laboratory. This may be attributed to the fact that when the cementation bonds are preserved during sampling,they prevent the swelling from taking place.

Good quality sampling does have some importance mainly in problems of settlement. However, when dealing with stability problems on our sensitive clays,it would seem that the quality of sampling has much less practical consequence. During the last few years,the concept that the residual strength is applicable in stability problems in sensitive clays has gained more and more weight in view of the evidence being accumulated.

For example,figure 6 illustrates the results of drained and consolidated undrained tests with pore pressure measurements made on tube and block samples (La Rochelle and Lefebvre 1971). The tests on block samples show a pronounced peak strength followed by a rapid decrease down to an approximately constant value obtained at strains of 10% to 15%. The peak value corresponds to the hump-shaped envelope which represents the bonded strength, while the constant values of strength obtained at large strains correspond to the dotted line which is considered to represent the residual strength envelope. The tests on specimens trimmed from tube samples taken with a standard sampler give peak strengths only slightly higher or equal to the residual strength measured on block samples,and the residual strength of tube samples was approximately the same as that of block samples.

These results were obtained during investigations of natural slope failures. When applying the effective strength parameters taken from these two envelopes to the analysis of four well-defined natural slope failures, it was found that the use of the parameters corresponding to the peak-strength envelopes grossly overestimates the safety factors while the residual strength parameters gave realistic factors of safety very close to unity.

More recently,the failure of a test embankment built on a soft sensitive cemented clay foundation in Saint-Alban was analyzed using the same approach but with the undrained residual strength. When analysing the embankment on the basis of vane strength with the usual assumptions,a factor of safety of 1.3 was found where the plasticity index of the clay is 23%. Hence the relationship between the plasticity and the factors of safety computed on the basis of the vane strength, as suggested by the General Reporter,cannot explain the observed discrepancy in the present case. However when using the residual undrained strength values measured by UU tests on tube samples,the factor of safety obtained is equal to 1.00.

Although the validity of the residual undrained strength approach as applied to the short term stability problems is not yet proven, it seems to offer some promise in that regard. There are some unpublished data which tend to show that embankment failures on higher plasticity clays could be explained by that approach without having to resort to the plasticity vs vane strength relationship. The results of these studies have been submitted for publication in the Canadian Geotechnical Journal.

As the residual strength is not too sensitive to sampling disturbance,useful results may be obtained for stability purposes by testing standard tube samples whenever the residual strength applies to the problem under study. For that purpose,it is preferable but not really essential that the cementation bonds be preserved.

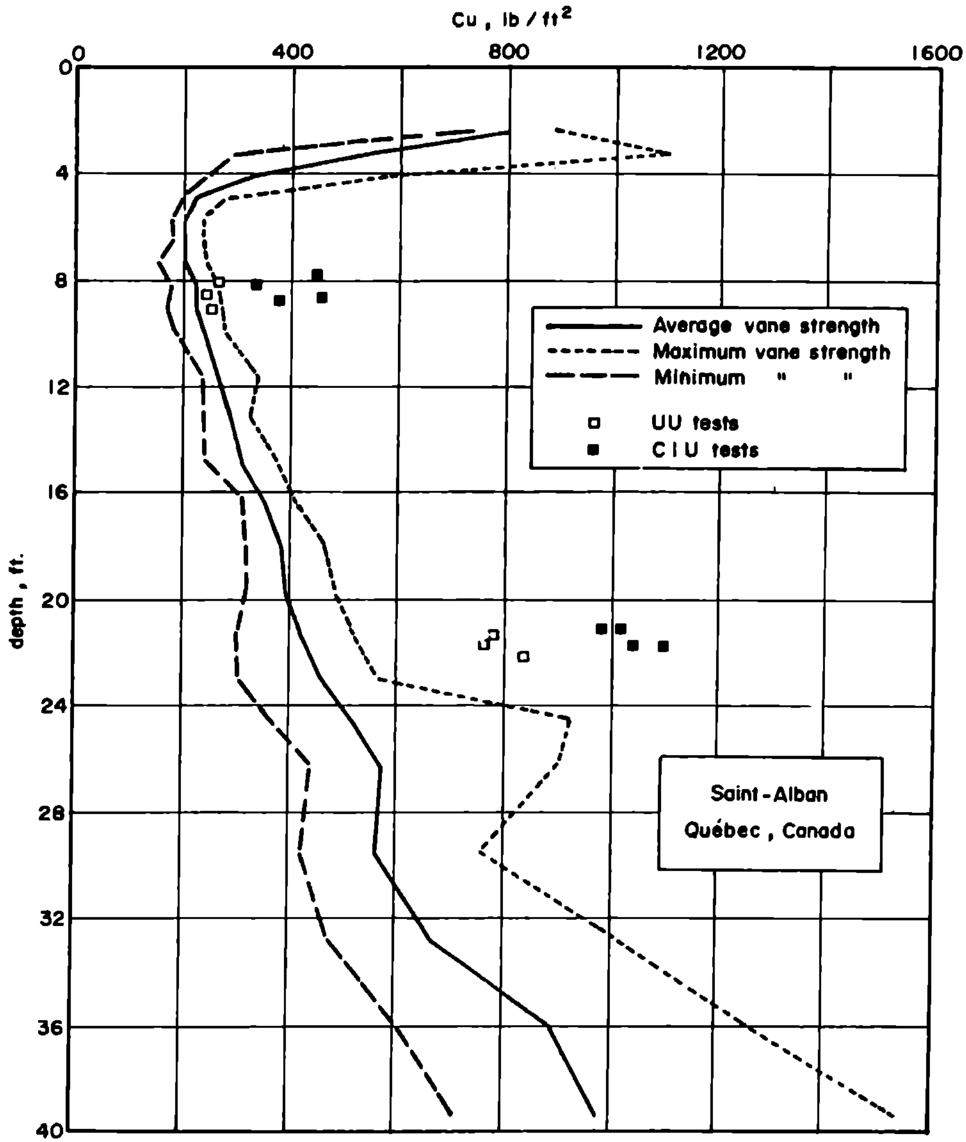

FIGURE 3 - COMPARISON BETWEEN SHEAR STRENGTHS MEASURED BY VANE TESTS AND BY UU AND CIU TESTS

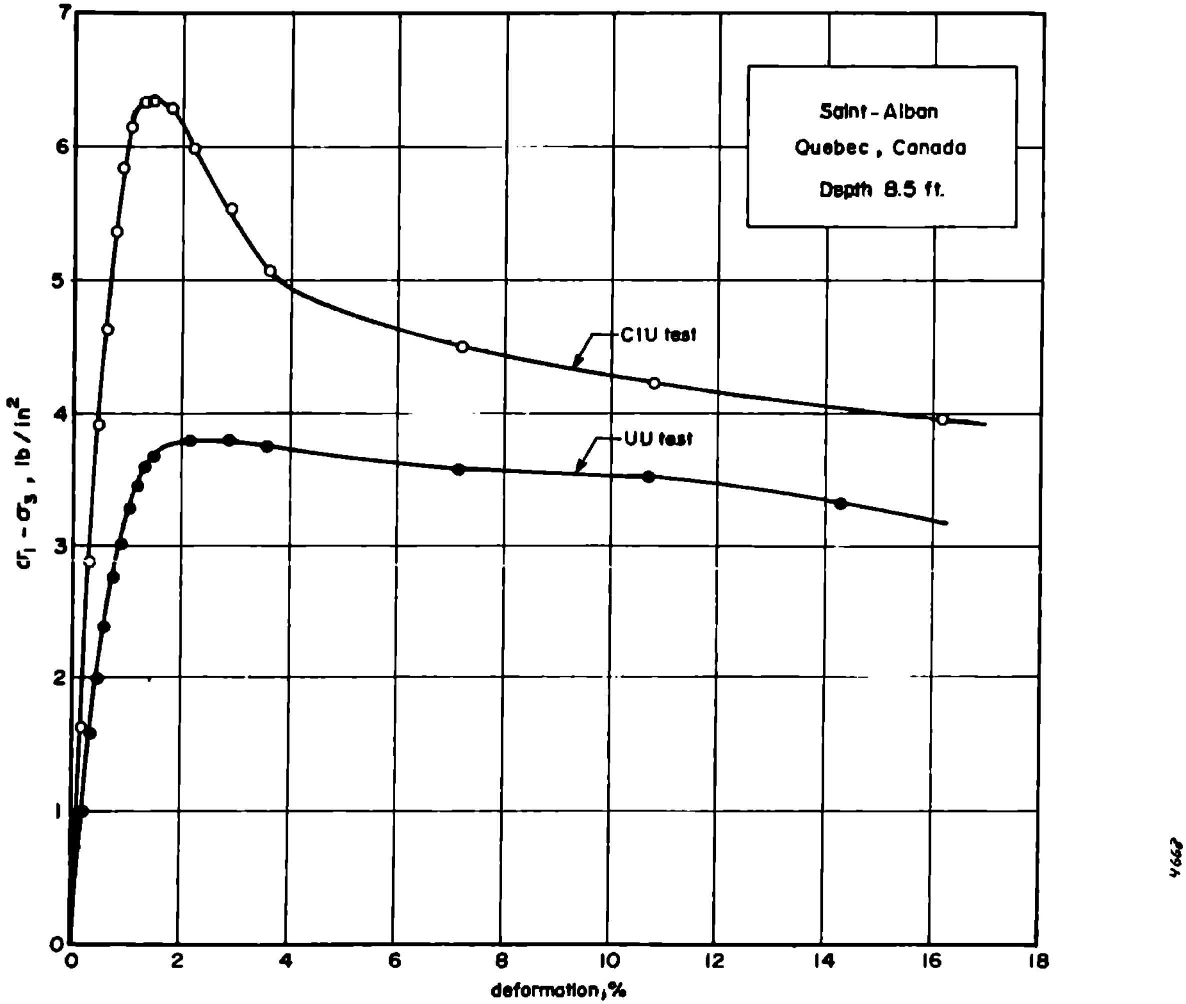

FIGURE 4 - STRESS-STRAIN CURVES OF UU AND CIU TESTS ON SAMPLES FROM THE UPPER LEVEL (9 ft. DEEP)

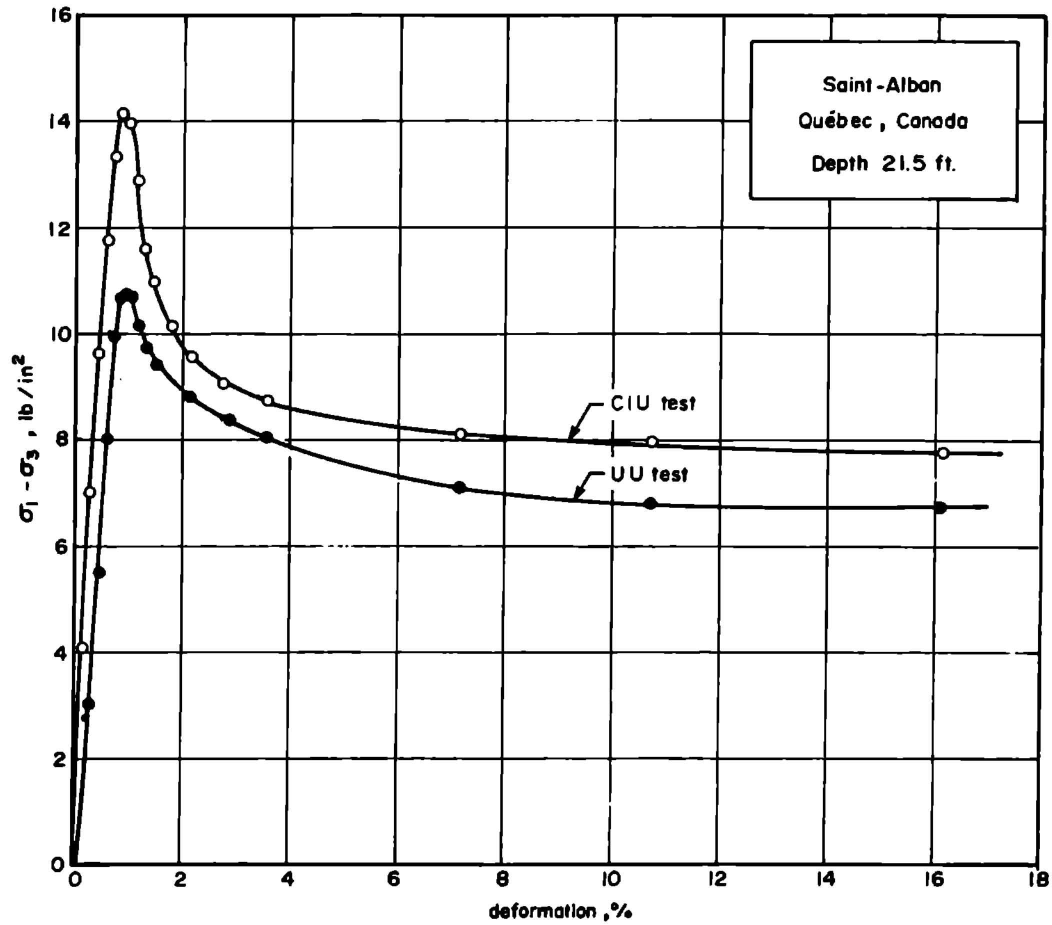

FIGURE 5 - STRESS-STRAIN CURVES OF UU AND CIU TESTS ON SAMPLES FROM THE LOWER LEVEL (22 ft. DEEP)

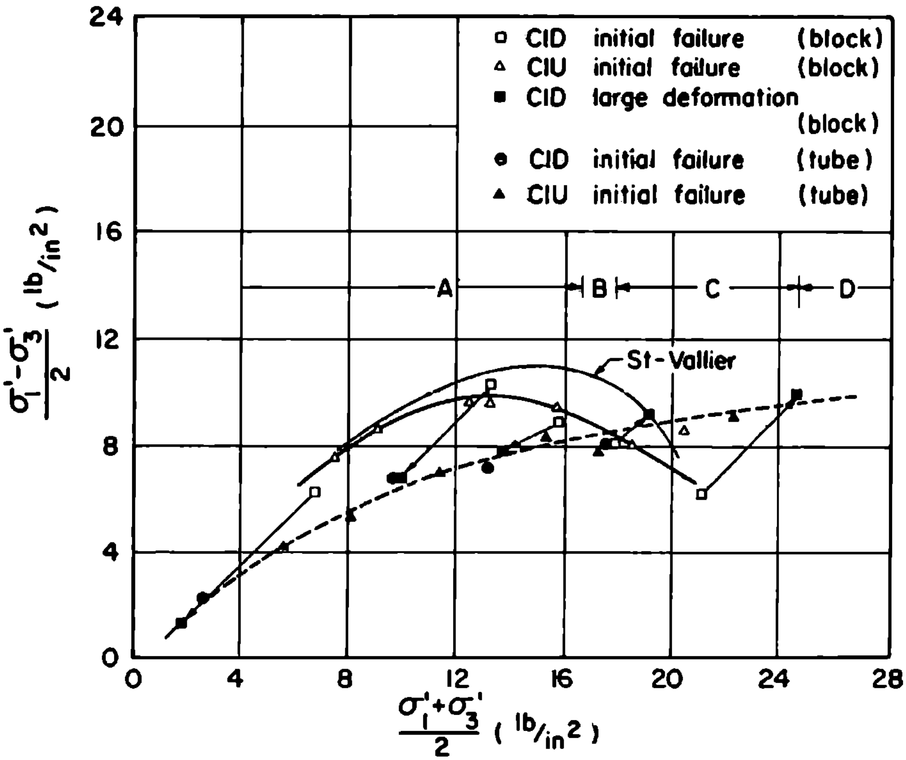

FIGURE 6 - EFFECTIVE STRESS ENVELOPES OF CEMENTED CHAMPLAIN CLAYS

More work will be required before the residual undrained strength approach may be considered valid; however if this proved to be the case, it would eliminate many uncertainties involved in the present methods and techniques of design, at least in the case of cemented sensitive clays.

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Chairman Prof. G.A.Leonards

Thank you very much Prof. La Rochelle. I would like to ask now Prof.Charles Ladd from Massachusetts Institute of Technology, Cambridge.

Prof. Charles C.Ladd /USA/

It is an honor to be invited by the Chairman of the Session to discuss Dr.Bjerrum's outstanding State-of-the-Art Report. My discussion is concerned with the field vane test and the measurement of undrained strength anisotropy as applied to embankments on soft clay.

Application of the Field Vane Test

Part A of Table I summarizes theoretical considerations regarding the field vane test. Because of these factors,it is concluded that the field vane is a Strength Index Test that requires a field calibration for each clay deposit. Bjerrum (1972) developed a correction factor to be applied to measured field vane strengths based on data from 14 embankment failures. Bjerrum's State-of-the-Art Report to this Session recommends use of the same correction factor for several other types of stability problems,as listed at the bottom of Table I.

Figure 1 plots Bjerrum's field vane correction factor versus plasticity index of the foundation clay. The field cases used to develop the relationship are shown as circles. Data from seven other embankment failures are also plotted. The recommended correction to measured field vane strengths becomes more important with increasing plasticity index. For example, the vane strength of a clay with a plasticity index of 100% is multiplied by

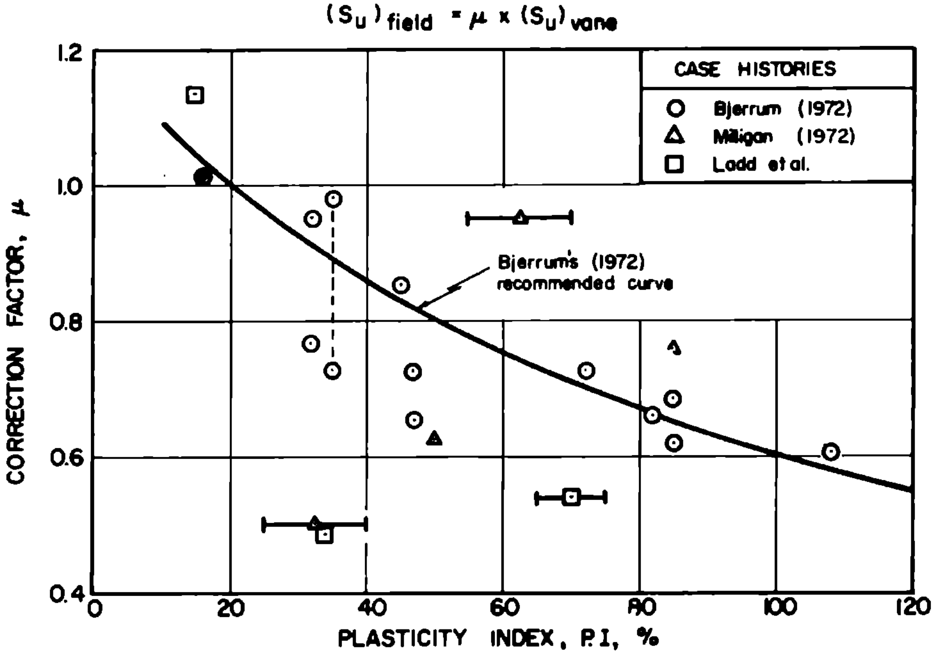

FIG. I FIELD VANE CORRECTION FACTOR VS. PLASTICITY INDEX DERIVED FROM EMBANKMENT FAILURES

<u>Table I Field Vane Test</u>

A. THEORETICAL CONSIDERATIONS

1. Shearing on vertical cylindrical surface:
   (a) Causes severe rotation of principal planes.
   (b) Produces stress system unlike any mode of failure of practical interest.
2. Failure occurs in very short time
3. Insertion of vane head may cause significant disturbance to surrounding clay

<u>Conclusion</u>

The field vane is a <u>Strength Index Test</u> requiring field calibration

B. EMPIRICAL CORRELATIONS FOR SOFT CLAYS

1. Bjerrum (1972) developed correction factor derived from embankment failures (see Fig. 1)

   (s_u)Field $= \mu \times (s_u)$ vane

   where $\mu = f$(plasticity index of clay)

2. Bjerrum (1973) recommends same correction factor for:
   (a) Bearing capacity of footings
   (b) Bottom heave of strutted excavations
   (c) Stability of temporary cuts and unsupported excavations

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0.6 to obtain the corrected field strength. This trend is attributed by Bjerrum to the increased importance of strain rate effects with highly plastic clays.

Table II summarizes the writer's recommended use of the field vane test and outlines some of its limitations for embankment stability. The writer concurs with Bjerrum's recommendation that field vane tests should always be performed and would use his correction factor to obtain design strengths subject to the restrictions set forth in part A3 of Table II.

These restrictions are imposed because there are case histories for which Bjerrum's correction factor is in error by 25 to 50% (see Figure 1). This can lead to unsafe or uneconomical designs. Also the field vane test can not provide stress-strain data for deformation analyses. Nor can it predict changes in s_u with consolidation that is required for embankment designs involving stage construction. Under these situations, the writer believes that one should turn to the use of shear tests performed on samples reconsolidated in the laboratory.

Table II. USE OF FIELD VANE TESTS IN PRACTICE FOR EMBANKMENT STABILITY

A. RECOMMENDED APPLICATION WITH SOFT CLAYS

1. Perform field vane tests in order to obtain relative changes in s_u throughout the clay deposit (variations in vertical and lateral directions)
2. Determine average field vane s_u versus depth within each similiar geologic deposit.
3. Use Bjerrum's (1972) correction factor to obtain design s_u subject to following restrictions for clay deposits without prior experience:
   (a) Clay is fairly homogeneous without shells, sand lenses, varves, etc.
   (b) Consequences of a failure are not too important.
   (c) The project is relatively small

B. LIMITATIONS OF ABOVE APPROACH

1. There are clay deposits for which Bjerrum's correction factor is in error by large amounts (25-50%). This can lead to unsafe or uneconomical designs.
2. It does not provide stress-strain data for undrained deformation analyses
3. It can not predict changes in s_u with consolidation for embankment designs involving stage construction

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Major Test Variables for Consolidated-Undrained Laboratory Strength Tests

Table III lists three major variables that must be considered in a program of K_o consolidated-undrained laboratory strength tests.

Table III. MAJOR TEST VARIABLES FOR K_o CONSOLIDATED-UNDRAINED (CK_oU) LABORATORY TESTS

A. STRESS SYSTEM DURING SHEAR

1. Because of anisotropic stress-strain-strength behavior of clays, lab tests should simulate the in situ modes of failure
2. The variables are:
   (a) Magnitude of intermediate principal stress (σ'_2)
   (b) Rotation of principal planes (direction of σ_{1f} versus σ_{1c}).
3. Because of limitations of shear equipment, can only simulate certain types of in situ stress systems.

B. RATE OF LOADING

1. Because of strain rate effects, stress-strain-strength behavior is a function of rate of loading (rate of strain or load increment duration).
2. Time and equipment limitations (problems with leakage and friction) generally preclude test durations equal to field loading period.
3. There is evidence to show that strain rate effects are more pronounced in triaxial tests than under plane strain conditions.

C. CONSOLIDATION STRESSES

1. Should samples be reconsolidated to:
   (a) In situ stresses, where effects of sample disturbance are important except for very high quality samples;
   or (b) Greater than in situ stresses, where large strains cause changes in soil structure with cemented and quick clays

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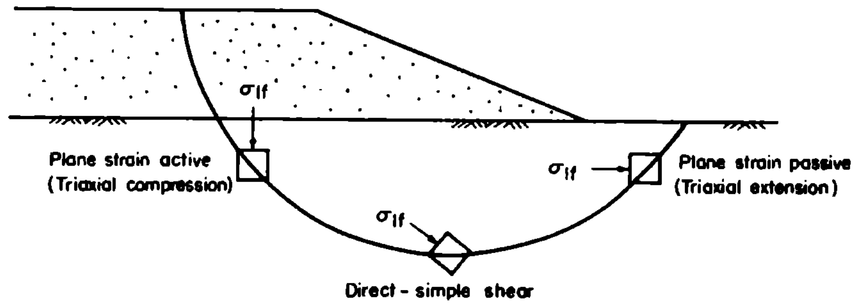
First, because most clays exhibit anisotropic strength behavior, the tests should simulate the in situ modes of failure. The variables are the magnitude of the intermediate principal stress and rotation of principal planes during shear. However, equipment limitations restrict the types of in situ stress systems that can be duplicated. Data illustrating the importance of anisotropy will be presented.

The rate of loading is important with plastic clays, especially as measured in triaxial compression tests with times to failure of less than one day. However, there is very little experience relating strain rate effects as measured in the laboratory to actual in situ behavior. Also, the rate of loading appears to be more important in triaxial tests than in plane strain tests. This area is still poorly understood.

Finally, one has to select the appropriate consolidation stresses in order to minimize the effects of sample disturbance while still maintaining as closely as possible the same soil structure as exists in the ground. A procedure for doing this will be presented.

Undrained Strength Anisotropy

Turning to undrained strength anisotropy, Table IV presents data from various types of strength tests on a normally consolidated marine clay of moderate sensitivity from Boston, Massachusetts. In the first two tests, σ_{1f} acts in the vertical direction, the strain



DETERMINATION OF S_u BASED ON	$\frac{S_u}{\bar{\sigma}_{vo}}$	DEFINITION OF S_u
Average from plane strain active, passive and direct - simple shear	0.215	$S_u = \tau_{ff}$
Average of triaxial compression and extension	0.205 0.24	$S_u = \tau_{ff}$ $S_u = 1/2(\sigma_1 - \sigma_3)_f$
Direct - simple shear	0.20	$S_u = (\tau_h)_{max}$

For normally consolidated Boston blue clay
$\tau_{ff} = 0.5\,(\sigma_1 - \sigma_3)_f \cos \bar{\Phi}$

FIG. 2 UNDRAINED STABILITY OF EMBANKMENTS FROM LABORATORY CK_oU TESTS

at failure is low, and the undrained strength ratio is high (0.34 and 0.33) for plane strain and triaxial compression respectively). Rotation of the principal planes to produce a horizontal failure surface, as in the direct-simple shear test, reduces the strength considerably. A 90 degree rotation of principal planes causes a further reduction in strength and increase in the strain at failure, with triaxial extension producing the lowest strength, 0.155, which is less than one-half of the vertical strength. Strengths from field vane and strip footing tests (Kinner and Ladd, 1973) are shown for comparison.

These data are typical of marine clays of low to moderate plasticity. The in situ undrained strength is highest for vertical cuts and active earth pressures, and lowest for bottom upheaval and passive earth pressure problems. Strengths for bearing capacity failures fall between these extreme values. Certainly no one test, either in the field or the laboratory, can be applied to all types of stability problems.

How can one consider anisotropy in determing the stability of embankments? Figure 2 shows the approach M.I.T. has been using quite successfully for several years, which

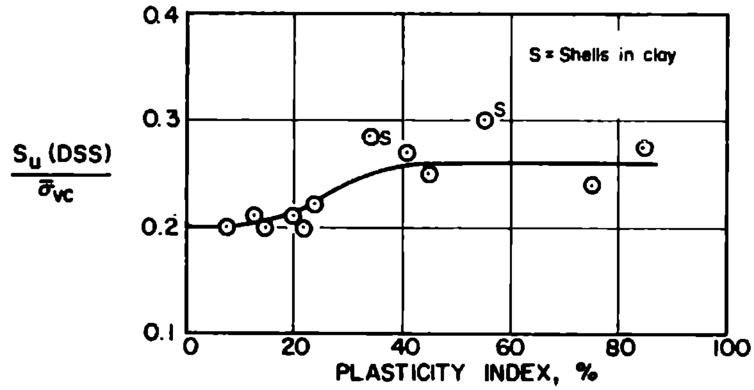


FIG. 3 UNDRAINED STRENGTH VS. PLASTICITY INDEX FROM CK_0U DIRECT - SIMPLE SHEAR TESTS ON NORMALLY CONSOLIDATED NON - LAYERED CLAYS

(From Ladd and Edgers, 1972)

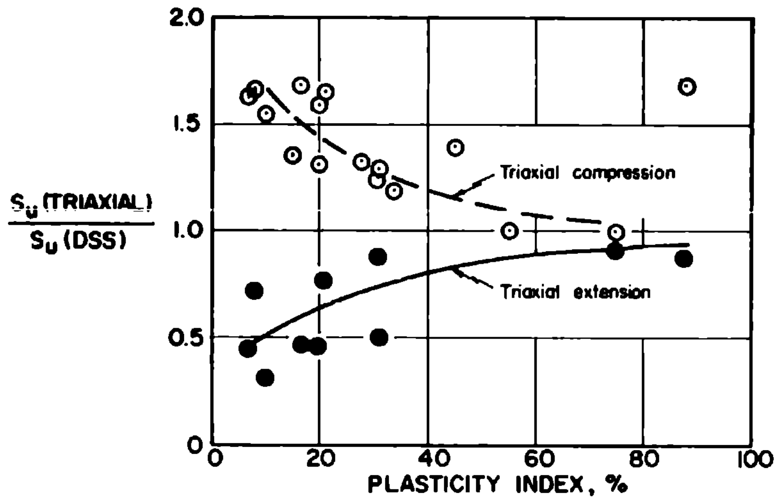


FIG. 4 UNDRAINED STRENGTH ANISOTROPY VS. PLASTICITY INDEX OF SOFT NON - LAYERED CLAYS

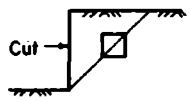
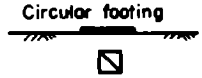
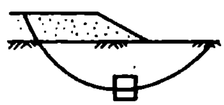
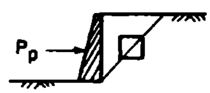
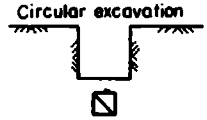
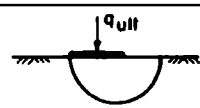
is similar to that now recommended by Bjerrum (1973). The three types of plane strain tests shown at the top of Figure 2 are performed, i.e. the active, the direct-simple shear, and the passive modes of failure. For normally consolidated Boston Blue Clay, the average strength from these three tests is 0.215. If plane strain equipment is not available, triaxial compression and extension tests might be used. Finally, laboratory data and field experience show that the direct-simple shear test gives a good to slightly conservative estimate of the average strength along a circular arc failure for non-layered soft clays.

Figure 3 shows strength data from direct-simple shear tests run on several normally consolidated clays. s_u/equal to $(\tau_h)_{max}$/ divided by the vertical consolidation stress $\bar{\sigma}_{vc}$ equals 0.20 to 0.22 for lean clays and increases to 0.24 to 0.28 for plastic clays. As stated above these strengths are thought to be equal to or slightly less than the average strength along the critical arc of an embankment failure.

Figure 4 compares strengths measured in triaxial compression and extension tests $/s_u = 0.5(\sigma_1 - \sigma_3)_f/$ with those from direct-simple shear tests. For lean clays, triaxial compression tests yield much higher values of s_u, and thus would greatly overestimate the stability of embankments. Conversely, triaxial extension strengths are much lower and they would yield factors of safety that are too low. With the more plastic clays, anisotropy becomes far less important.

TABLE IV UNDRAINED ANISOTROPY OF NORMALLY CONSOLIDATED BOSTON BLUE CLAY

(LL = 41 %, P.I. = 21 %)

TYPE OF TEST	IN SITU CONDITION	SHEAR STRAIN γ_f, (%)	$\frac{S_u}{\bar{\sigma}_{vo}} = \frac{"C"}{P}$
1 Plane strain active	 Cut	0.8	0.34
2 Triaxial compression	Circular footing 	0.5	0.33
3 Direct - simple shear		6	0.20
4 Plane strain passive	 P_p	9	0.19
5 Triaxial extension	Circular excavation 	15	0.155
6 Field vane		—	0.19
7 Model strip footing	 q_{ult}	—	0.26 (N_c = 5.14)

Note : $S_u = 0.5(\sigma_1 - \sigma_3)_f$ except for DSS and field vane tests

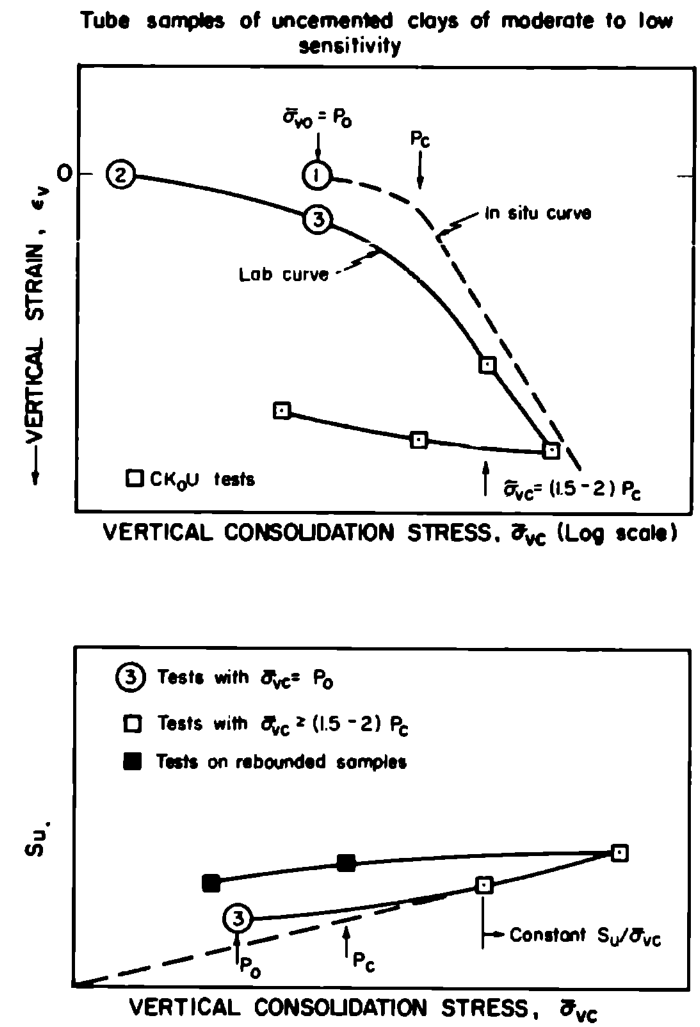
Laboratory Consolidation Stresses

Finally, the question arises as what consolidation stresses should be used for K_o consolidated-undrained laboratory tests. Bjerrum (1973) and others recommend that samples be reconsolidated to the in situ stresses. While this procedure may be the best approach with high quality samples of quick clays and naturally cemented soils, the writer does not recommend this procedure for typical tube samples of other types of clays.

The upper plot in Fig.5 shows hypothetical in situ and laboratory compression curves for a soft slightly overconsolidated clay. The in situ condition is shown by Point 1. Disturbance during tube sampling reduces the stresses to Point 2. Recompression to the in situ stresses, shown by Point 3, causes a significant decrease in volume, typically 3 to 8%. The undrained strength of this sample is therefore likely to be too high.

The procedure recommended by the writer is to consolidate samples into the virgin compression region, including tests on rebounded samples, as shown by the squares. The resulting strengths are plotted in the bottom figure. Tests are run on several normally consolidated specimens to check that s_u divided by the vertical consolidation stress is a constant. The tests on rebounded samples show how the strength will increase with overconsolidation ratio. These data are used in conjunction with the in situ stress history to compute undrained strengths for design. This approach has been successfully used on several major highway and flood control projects involving a variety of soil types (Ladd and Foott, 1973).

FIG. 5 RECOMMENDED CONSOLIDATION PROCEDURE FOR LABORATORY CK_oU TESTS ON SOFT CLAYS



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Chairman Prof. G.A.Leonards. Thank you Prof. Ladd, Mr.Helenelund, will you,please.

Mr.K.V. Helenelund /Finland/

The General Reporter Professor Leonards has asked me to give some comments on the application of Bjerrums method in design and construction of foundations and earth structures on soft and organic clays. In Finland we have generally used the reduction coefficients recommended by John Olsson as reported by Dr.Broms in the discussion of Session 1. This means that the undrained shear strength determined by cone tests or vane tests has been reduced by 20 to 40% depending on the value of the so called "fineness number",which is about equal to the liquid limit (but determined in a different way,by the cone test). This reduction corresponds fairly well to the method proposed by Bjerrum. However,in our country,we have used reduced shear strength values as a rule only in organic clays with a fineness number of liquid limit greater than 80.

It has some times been questioned if such a reduction is necessary in organic clays and what are the special properties of organic clays that make such a reduction necessary, if no similar reduction is made in non-organic clays. From this point of view I think that Bjerrum's method is a most logical extension and improvement of the old recommendation,which was based mainly on experience with railway embankments.

In the Speciality Session 6 Mr.Slunda presented some case records from three big slope failurer during the construction of the Saimaa canal on the boarder-line between Finland and USSR. The design of these slopes in normally consolidated and slightly overconsolidated varved clays was based on field vane tests with a factor of safety during the construction period of 1.30. According to subsequent investigations by a special committee, the main reason for the dic- between the calculated and the real factor of safety (1.0) was that normal vane tests (with high-diameter ratio =2) gave to high values for the undrained shear strength of the varved (laminated) clay and that the scatter of the strength values had not been taken properly into account.

If we use the reduction factor recommended by Bjerrum the factor of safety would have been 1.23 in stead of 1.30 as the plasticity index the clay was about 25. If we apply the additional anisotropy reduction factor according to figure no 9 in Bjerrums general report, the factor of safety would be a little less than 1.10 which already presents a fairly good approximation.

It must be stressed, however,that there are several types of anisotropic clays and that it is not possible to cover them all correctly by one single reduction factor. A more detailed analysis of the influence of strength anisotropy and the slope failures at the Saimaa Canal can found in Slungas report, which also discusses the application of the probability theory in slope stability predictions. It may be mentioned that the economic optimum value of the factor of safety in this case was found to be 1,30...1.50 depending on the costs of slope stabilization and reconstruction,risk of human losses and damage on heavy construction equipment and human lives.

Chairman Prof.G.A.Leonards. Thank you Mr. Helenelund. The next will be Prof. Bishop, England

Prof. A.W.Bishop /U.K./

The Chairman has asked me to make a few remarks about the problems of creep.

At Imperial College we have carried out a series of very long-term creep tests on axial symmetrically loaded specimens under both drained and undrained conditions. The maximum test duration has been 3 1/2 years and the rubber envelopes enclosing the samples have been surrounded by mercury to prevent any migration of pore fluid or gas. Some of the test data were reported at the last International Conference in Mexico (Bishop and Lovenbury,1969).

As we have carried out more tests on overconsolidated soils I will begin with these. They are also more important as a check on the concepts proposed by Dr.Bjerrum in his State-of-the-Art Report to this Session,since Hvorslev's cohesion term makes a much larger contribution to the strength of a heavily overconsolidated soil than is the case for a normally consolidated soil,and the cohesion term, in Dr.Bjerrum's view, is the term which depends principally on time. This view was also taken, if I remember correctly, by Professor Rowe,at the time of the 1957 International Conference.

Fig 1 shows that for undistrubed London

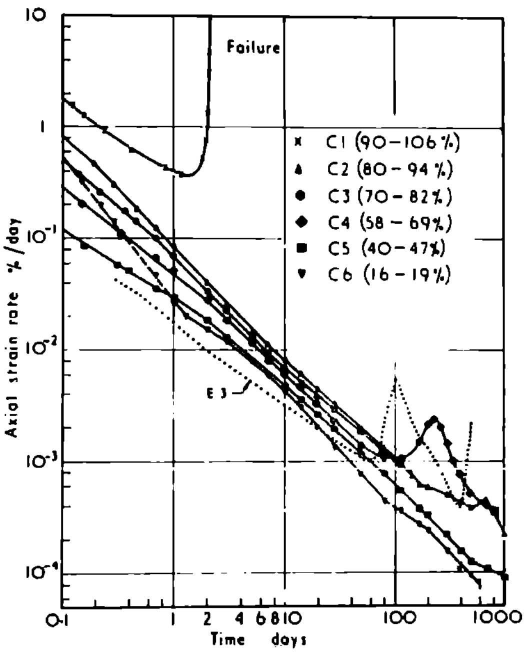

Fig 1: Strain rate as a function of time for various stress levels: drained tests on undisturbed London Clay.

Clay of PI 47%, only the sample under the very highest stress level showed a breakdown of structure leading to rupture. At stress levels below the range 80-94% (or weighted average 89%) the creep rate was decreasing steadily after 3 1/2 years. Yet much of this 89% of peak strength must have been contributed by cohesion, either of the Hvorslev type or due to some other form of interparticle bond.

If we consider the increase in strain with time (Fig.2) and accept the view that for drained conditions there is a critical strain at which rupture would occur,then it is possible to suggest that the strain rate of some of the samples at lower stress levels must at some future date increase and failure must occur. However, on the present evidence this date might not be reached within the life span of those of us in this hall. We may have to distinguish between creep of geological significance and creep of significance within the planned life of engineering works.

In the latter connection I would like to refer to the undrained creep of the overconsolidated Lias Clay,which forms the foundation of a large dam under construction in the UK. Here a 20 metre high trial embankment has been constructed within the profile of the proposed dam to test the strength and time dependence of the foundation and fill, which are of the same clay.

Fig.3 shows the decay of strain rate plotted against logarithm of time for this clay under undrained conditions in the laboratory, compared with the drained value for London Clay. The strain rates for the undrained tests are a little higher,but decay in the same way as for the drained tests. We have not yet carried out drained creep tests on the Lias itself.

When we had raised the trial bank almost to a critical height,we allowed it to creep at constant load and observed the deformations within the foundation. These are of course under plane strain conditions. However, the decay of the strain-rate was very similar to that observed in the laboratory triaxial tests, and the actual rates of strain were almost within the experimental scatter of the laboratory results (Fig.4).

Although the shear strain was of the order of 8% for a depth of the order of 7 metres, no overall failure occurred within the test period of about 4 months, and the embankment has now been incorporated in the permanent work of the dam, which has a factor of safety against undrained failure some 50% greater than the trial embankment.

We have also carried out long-term drained ~~creep tests on the almost normally consoli-~~ dated highly plastic clay from the foundation strata of the leaning tower of Piza (Fig.5) PI=47%. You will notice that even for samples at the 85% stress level (based on 1 week strength) the strain rate was decaying after 2 years. The fact that the Leaning Tower of Piza is still standing is consistent with the trend indicated by this observation. It may also suggest that cohesion is not entirely time-dependent as Bjerrum suggested. However, between 30 and 100 days all the samples at all stress levels and strains showed a change in strain rate which deviated from the standard logarithmic or other laws and which extended over several months. Extrapolation from shortterm tests would have been most misleading.

This discontinuity has been noted on other normally consolidated soils, and also on a rockfill made of mudstone (Fig.6). Here there is clearly interparticle contact, and it makes one question Bjerrum's assumption that stress and particularly strain related to interparticle contact are not significantly time-dependent.

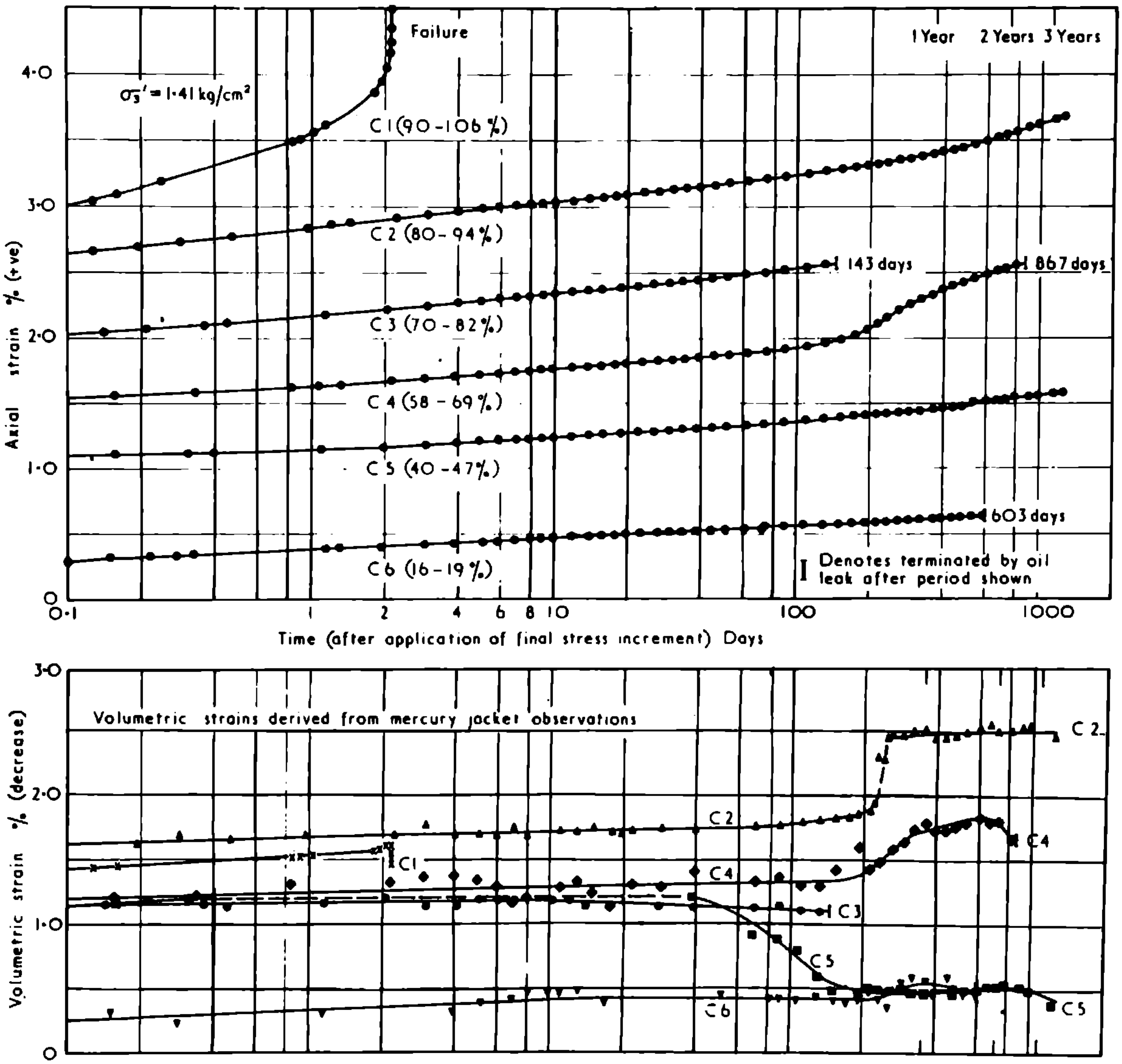

Fig 2: Drained creep tests on undisturbed London Clay from Hendon: σ_3' constant, σ_1' increased.

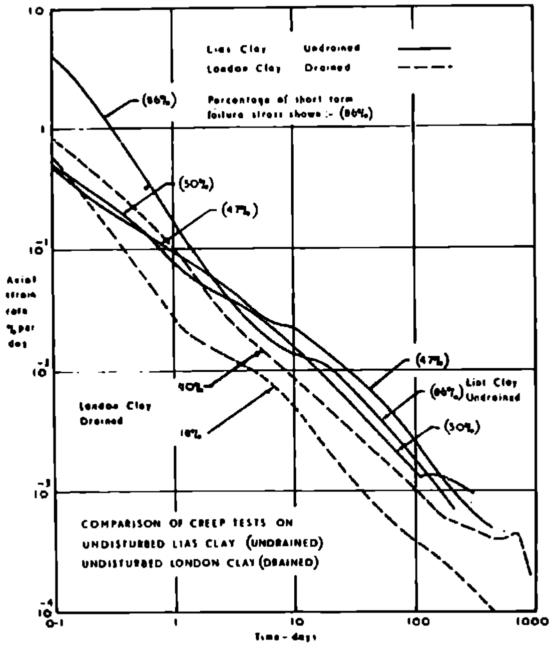

Fig 3: Comparison of creep tests on undisturbed Lias Clay (undrained) and undisturbed London Clay (drained).

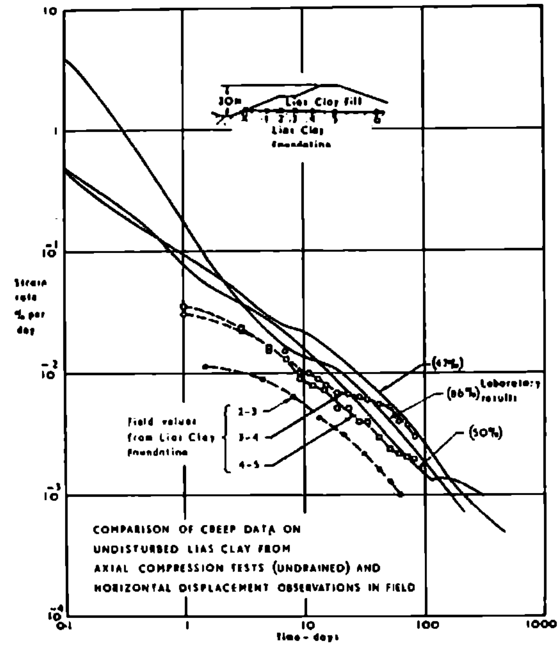

Fig 4: Comparison of creep data on undisturbed Lias Clay from axial compression tests (undrained) and horizontal displacement observations in field.

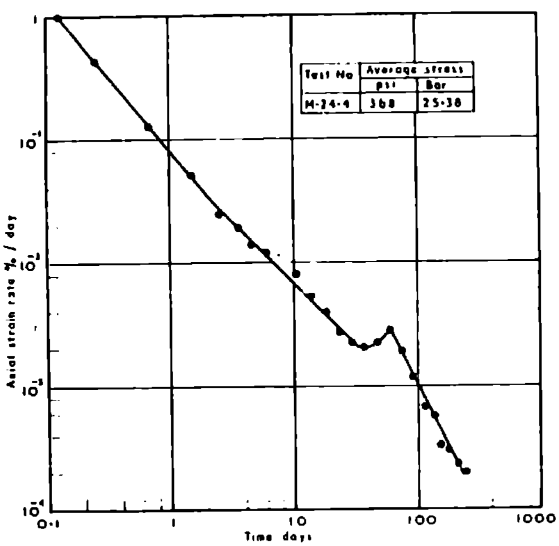

Fig 5: Strain rate as a function of time for various stress levels: drained tests on Pancone Clay.

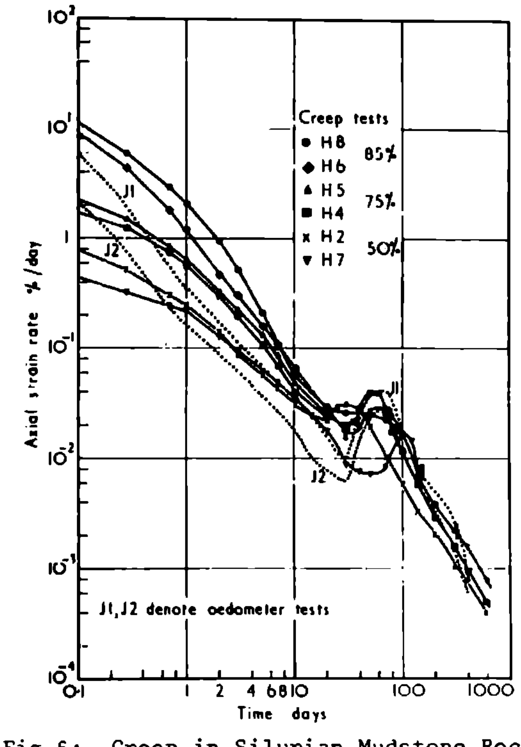

Fig 6: Creep in Silurian Mudstone Rockfill (particles < 70 mm nominal diameter) 0.6 m diameter sample loaded vertically with zero lateral yield.

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Chairman Prof. G.A. Leonards. Thank you Prof. Bishop. Now I pass the word to Prof. Ter-Stepanjan.

Prof. Ter-Stepanjan G. /USSR/

The late Dr. Bjerrum in his general report to the present conference has drawn the attention of experts in this science to the effect of time in the shear strength of clays. I like to present a summary of creep investigations carried out during last years.

Theory of creep of clay at shear is proposed, corroborated by experiments and illustrated by a rheological model. Process of creep under the constant shear stress τ consists of two phases-mobilization and rupture. A successive series of soil structures are formed and destructed in the process of creep; they are characterized by definite arrangement of particles, contacts between them and stresses that act in these contacts. Deformative properties are determined by a relationship between fluidity and rigidity of soil structures; these qualities are estimated by tangential F and normal R vectorial sums of corresponding components of interparticle forces. Transition from one structure to the other takes place jump-likely. In the mobilization phase the strain rate $\dot\gamma$ decreases directly with the stress age t

$$\dot\gamma = a\,\frac{\tau-\tau_p}{\eta}\,\frac{1}{t}$$

where a is a structural coefficient depending on soil deformability F/R, which is constant for each structure, τ_p is the creep limit and η - static viscosity equal to relation of dymanic viscosity to the stress age. Therefore the stress-strain curve in the mobilization phase consists of pieces of logarithmic curves; each of them is described by equation (Fig.1)

$$\gamma = a\,\frac{\tau-\tau_p}{\eta}\,\log_n t + C$$

In the rupture phase the strain rate increases directly with the stress age

$$\dot\gamma = a\,\frac{\tau-\tau_p}{\eta\, t_m^2}\,t$$

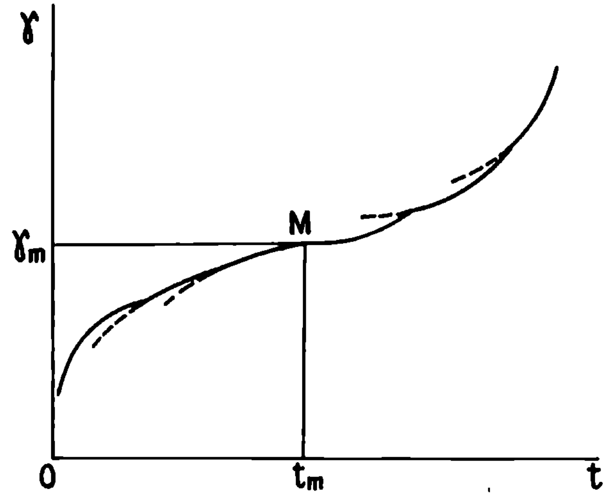

Fig.1. Creep curve for a clay at shear. M- mobilization limit

where t_m is the mobilization time corresponding to transition from mobilization to rupture phases. Therefore the stress-strain curve in the rupture phase consists of pieces of parabolas; each of them is described by equation (Fig.1)

$$\gamma = a\,\frac{\tau-\tau_p}{2\eta\, t_m^2}\,t^2 + C$$

The mobilization limit M corresponds to transition from one phase to the other; it occures after a certain mobilization strain γ_m takes place; this strain depends on the stress level

$$\gamma_m = \frac{\tau-\tau_p}{G_m},$$

where G_m is modulus of mobilization. A decrease of the strain rate takes place in the rupture phase at middle and low stress levels; the transition point is the stabilization limit S. The corresponding stabilization strain γ_s is linear with the stress

$$\gamma_s = \frac{\tau-\tau_p}{G_s},$$

where G_s is modulus of mobilization. The relationship between strain and strain rate is plotted in logarithmic scale on Fig.2; these intrinsic curves are characteristic for the rheological behaviour of clays. In the coordinate system $\tau - t\dot\gamma$ for the given normal stress test results are represented by straight lines described by equation

$$\tau = \tau_p + \eta\, t\dot\gamma$$

The creep process in clays is determined by four quantities which are linear with the normal stress: creep limit τ_p, static viscosity η, mobilization G_m and stabilization G_s moduli. A rheological model of creep of the

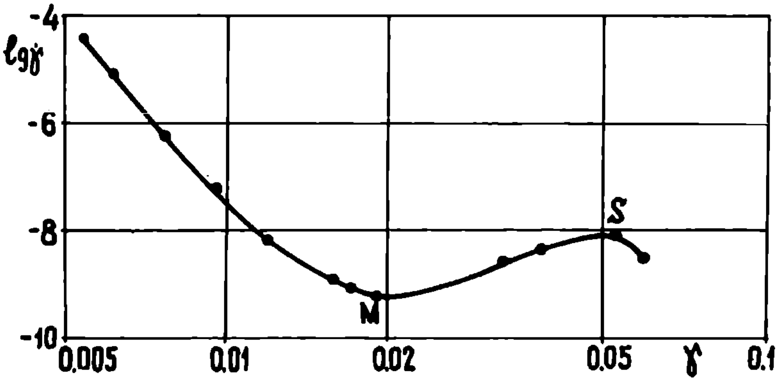

Fig.2. Intrinsic curve for a clay at shear. S–stabilisation limit

clay at shear is proposed (Fig.3). The model consists of an elastic Hook body connected in series with a pair, where a plastic St.

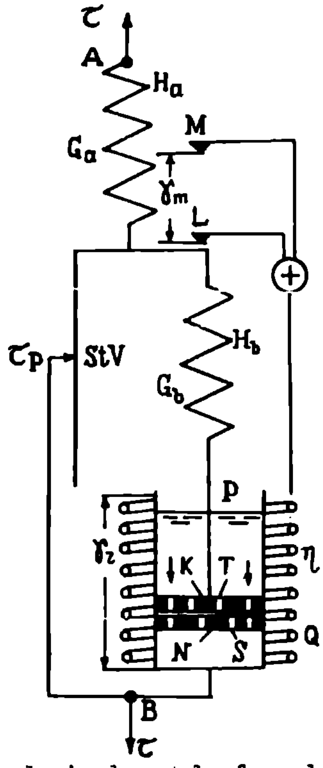

Fig.3. Rheological model of a clay at shear

Venant body is parallelly connected with an elasto-viscous relaxating destructible body having variable viscosity. It has a damper P with a coil Q through which cooling or heating liquid may flow thus increasing or decreasing the viscosity of the liquid in this damper. The rheological body models both phases-mobilization and rupture of the creep and the probabilistic character of the soil structures which are originated and destroyed in the creep process. Equations of the mechanical behaviour of the model in the mobilization and rupture phases are deduced; these equations adequately reflect the behaviour of soil at creep.

Chairman Prof.G.A.Leonards.
Thank you Prof. Ter-Stepanjan. The next will be Dr.Broms from Swedish Geotechnical Institut, Sweden

Dr.Bengt B.Broms /Sweden/

The following comments concerns a new type of drain which has been developed at the Swedish Geotechnical Institute (SGI) and the results from a large scale load test where the drain has been used.

The drain described in this report is a further development of the Kjellman cardboard drain (Kjellman, 1949) and it consists in principle of a grooved polyethene core which is wrapped in a wet-strong paper as shown in Fig. 1. The paper serves as a filter and prevents clogging of the canals formed by the grooves. The total thickness of the drain is 4 mm and its width is 100 mm.

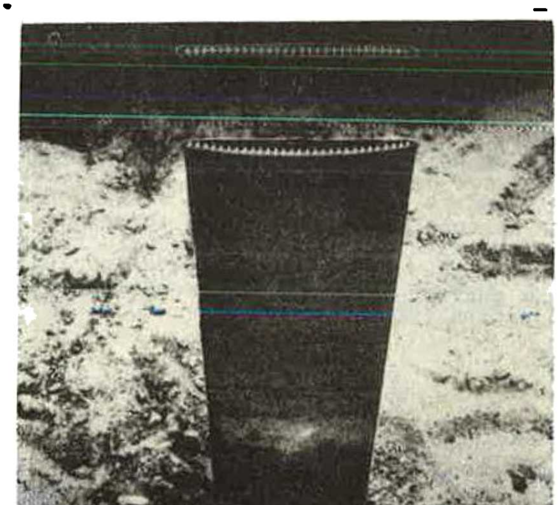

Fig. 1. Photograph of drain

The drains are inserted into the soil by a flat thin walled steel tube or mandril which is mounted on a standard front wheel loader as shown in Fig. 2. The maximum penetration force is 0,4 MN (4.0 metric tons). The maximum penetration depth is 12 m. Approximately 40 drains each 10 m long can be installed per hour. No special preparation of a site is normally required since the wheel loader can be provided with tracks with low contact pressure.

Fig. 2. Installation of drains

The drains are folded at the bottom of the flat steel tube when they are pushed down into the soil. Thereby the drains are kept in place when the driving tube is withdrawn.

The behaviour of the drains has been investigated at the SGI test field at Skå-Edeby located 25 km west of Stockholm. At this site the soil consists from the surface of a 1.0 m thick dry crust of stiff fissured clay, 3.0 m of postglacial, slightly organic clay and 10 m of glacial brown to grey varved clay. Bedrock or dense glacial moraine is normally found at a depth of 9 to 15 m below the surface. The shear strength as determined by field vane and fall-cone tests is about 7 kN/m^2 just below the dry crust. It increases to 10 to 20 kN/m^2 near the bottom. The water content of the clay decreases with depth from about 100 percent below the dry crust to about 60 percent at the bottom. The sensitivity of the clay is about 10. The soil properties have been described in detail by Hansbo (1960) and by Holtz and Broms (1972).

About 950 drains, each with a length of 10 m, were installed in the summer of 1972 within an area with 31 m diameter. The drains were placed in a triangular pattern at a spacing of 0.9 m. After the drains were installed a 1.5 m high circular gravel fill was placed in three lifts over the area. The fill was completed in nine days.

Settlements have been measured at the surface and at various depths under the fill during the loading and the subsequent consolidation of the clay. The pore pressure distribution below the fill and the dissipation of pore pressures were measured as well as the lateral displacements of the soil around the edge of the fill.

The observed surface settlements across the diameter of the test fill are shown in Fig. 3. The maximum settlement 141 days after the placement of the fill was 350 mm. It can be seen that the settlement at the center of the fill was slightly less than that at the quarter points.

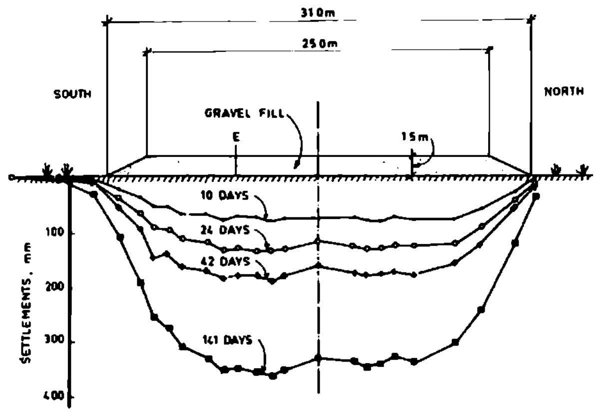

Fig. 3. Settlements below test fill

In Fig. 4 the settlement at the quarter point (Point E) of the drained area has been plotted as a function of time as well as the settlement of an adjecant 1.5 m high undrained test fill. The undrained fill, which was constructed in 1957, has the same height (1.5) as the drained fill. Its diameter (35 m) is however slightly larger. The behaviour of the undrained fill has been described in detail by Hansbo (1960) and by Holtz and Broms (1972).

The settlement rate of the drained fill has been analyzed by the method proposed by Kjellman (1948). A value of $c_h = 2.5\ c_v$ as indicated by oedometer tests was used in the analysis. Under this conditions the agreement between calculated and measured values was good.

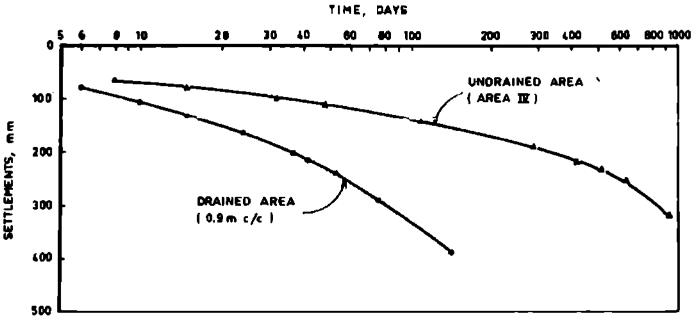

Fig. 4. Settlement - time relationship

It can also be seen from Fig. 4 that the settlement rate of the drained fill was approximately ten times that of the undrained fill which indicates that the drains were effective. Analysis of the test data indicate that the draining effect of a plastic drain is equivalent to that of a 16 to 18 cm diameter sand drain.

The plastic drain was developed at SGI by Mr Oleg Wager in cooperation with Örebro Pappersbruk AB and Perstorp AB. The field tests were planned and supervised by Mr Per Boman of SGI.

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Chairman Prof.A.G.Leonards.
Thank you very much Dr.Broms for your interesting discussion, the next will be Prof. Justo from Spain.

Prof. J.L.Justo/Spain/

In his excellent General Report, Professor Aitchison has mentioned our paper by saying (I quote him):
"A totally different viewpoint is that suggested in the paper by Salas et al in which collapse is shown to have occured in a gypseous soil following wetting by pure water-whereas wetting by-water saturated with gypsum produced relatively minor effects".

We should like to disagree in one small respect with the statement made by Dr.Aitchison by saying that collapse with water saturated with gypsum was rather important, although less than with distilled water.
In other words, it seems that in these gypseous soils there is an important collapse produced by wetting and another collapse produced by solution.

Chairman Prof.G.A.Leonards. Thank you Prof. Justo. The next contribution will be in charge of Prof. Goldstein.

Prof. M.N.Goldstein /USSR/

The lack of time doesn't allow me to consider many of very interesting items relating to this problem and so I would like to make only some remarks.
The most important of these questions is the inadequacy of present methods of settlement prediction. For instance on one of investigated building site the designed settlement of the collapsible layer with the thikness of about 16-18 m was under its own weight equal 30 cm, whereas the experimental wetting causes the real settlement 64 cm. But the cases are known when the theoretical settlement was more than real one.
The causes of such discrepancies are not yet known. One may suppose that the appearance of some wetted part in the soil body may essentially change the stress distribution in the soil comparing with the present theories.
Now when the rate of construction is constantly accelerating all the methods of foundation preparation which demand much time are unacceptable. So it seems to me that the present methods of wetting of soil prior to construction does not correspond to the industrial style of construction. But there are many possibilities of improving and accelerating this method as was shown by prof. Litvinov. The employment of long piles when the thikness of loessial soils is large is to my mind a very troublesome and uneconomical method of construction. Many experimental investigations must be carried out before this problem will be solved.
I have already published the opinion that all the man-made deposits of cohesive soils must be regarded as collapsible ones. Such fills are the typical examples of unconsolidated soils by prof. Denissov terminology.
It is significant that the collapsibility of

such soils gradually vanishes as the deposit is wetted by rainfall for a long time in natural conditions. So the main point in testing such soils is the determination of their settlement under wetting. The large and serious investigations conducted in our country by many engineers and geologists made it possible to solve many problems of construction on the loessial soils. The problems put forward by the general reporter will attract great attention of many investigators and we may be sure that they will be successfully solved in the nearest future.

Chairman G.A.Leonards.
Thank you Prof. Goldstein. The next will be Mr. Escario from Spain

Mr. V.Escario /Spain/

Collapse of soils is a phenomenon which at least originally had the meaning of a sudden settlement originated by flooding in certain soil types. There are different causes associated with this type of soil behaviour that I am not going to discuss. It seems nevertheless accepted by many investigators that one of the main reasons in many cases is the elimination of capillary forces due to saturation. As flooding supress very quickly the apparent cohesion originated by capillary forces, it is reasonable to expect a sudden soil deformation or the so called collapse. Nevertheless there may be situations in nature where complete flooding does not occur, but a gradual decrease of suction should produce the corresponding settlement without having to arrive to the maximum value due to full immersion.

For instance a house founded on a collapsible soil may settle after construction at least something due to the increase in moisture content, which will tend to achieve a new equilibrium value even if no flooding is followed.

I am also convinced that the settlements which undergo dry and poorly compacted embankments and which in some cases may last for many years are due, not only to conventional consolidation effects,but also to the gradual moisture content increase originated in the body of the fill when in tends to its equilibrium moisture, what may take many years to achieve.

It is therefore important to study the complete phenomenon by determining the whole curve relating settlement with suction decrease and not only the point corresponding to full submergence. Furthermore the influence of the stress history on the final result should also be possible to simulate.

As the same problem exists with swelling, several years ago (Escario,V.,1965) I started to apply suction to the soil samples by means of a mercury column. As this system is limited to a maximum negative pressure of -1 kg/om^2 I had to develop a different type of device (Escario,V.,1967) which has undergone several succesive improvements (Escario V., 1969) and in fig.1 is shown in its latest design as presented in the recent Haifa Conference on Expansive Soils. (Escario,V. and Saez, J.,1973).

APPARATUS FOR MEASURING SWELLING AND COLLAPSE OF SOILS UNDER CONTROLLED SUCTION

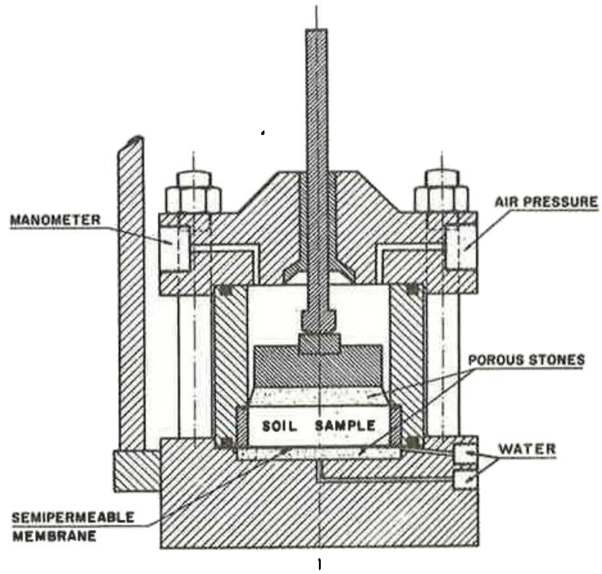

FIG. 1

As it may be seen the design is essentially based in the principle of the pressure membrane apparatus. A semipermeable membrane separates the sample from a source of water at atmospheric pressure, while an air (actually nytrogen) pressure equal to the desired suction is applied to the chamber. Vertical forces and deformations of the laterally constrained sample are measured through the vertical piston. With the design shown, suctions up to about-120 kg/cm^2 may be applied and we have now five units working in our laboratory.

I am not going to enter in the details of how we have solved the problems originated by the deformability of the apparatus itself and of the membrane,which may be seen in the above mentioned report. I am only going to show you some of the results obtained in studying the phenomenon of collapse.

The tests have been carried with a clayey sand of Madrid (arena de miga) of the following characteristics:

Liquid Limit	32
Plasticity Index	15
% passing no.200 sieve	15.5
Proctor max.density	1,98 T/m^3
Proctor optimum moisture	11%

All the samples have been compacted to the same density of 80% max. Proctor. The moisture content has been different in the three series of tests: 3%, 6% and 8%, all of them below the 11% optimum Proctor.

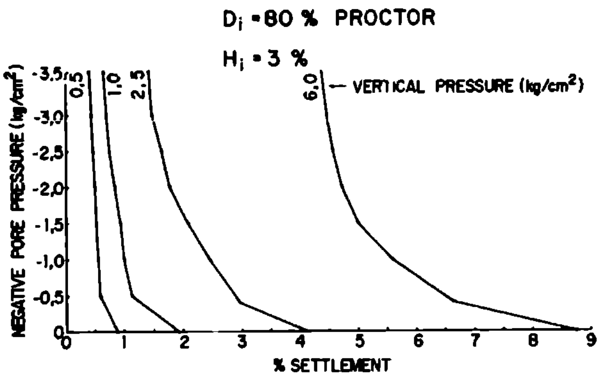

FIG. 2

In Fig.2 the 3% series is shown. Each curve corresponds to a sample which was initially consolidated undor the corresponding surcharge at the after compaction suction value ; then the suction was progressively decreased until completely flooded. As it may be seen, the settlements originated by the suction decrease process (initiated at the origin of each curve which includes soil plus membrane deformation) are not large until a relatively low suction value is attained,were upon the process is considerably accelerated leading to what might be perhaps called collapse. This tendency is more accentuated in the following set of curves (figs.3 and 4).

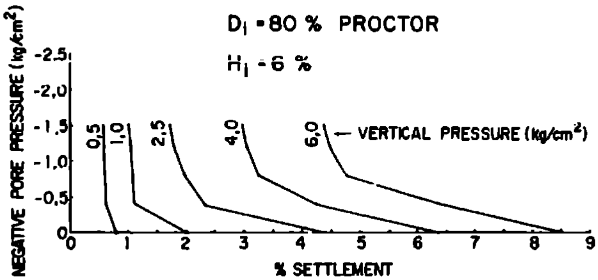

FIG. 3

By comparing the three sets of curves with each other after discarding the initial settlement it may be seen that, for each given vertical pressure,the settlements corresponding to a certain suction are larger the dryer the sample was compacted as it would be expected; but when complete flooding is attained they reach practically the same value, what does not appear so obvious for us.

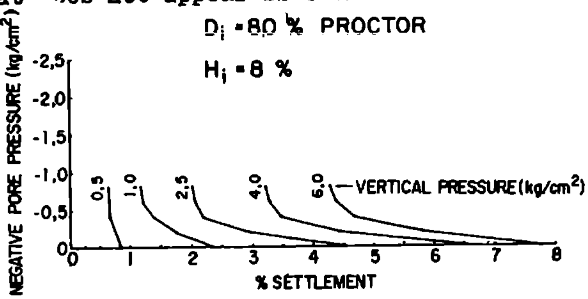

FIG. 4

Curves of this kind are necessary to define the soil behaviour at a given site and under given environmental conditions,either for swelling or collapse. As Dr.Aitchison says in his General Report,apparatus of the type shown are today the best approach to study the problem. But it is very surprising for us to see that Dr.Aitchison did not even mention the device we have developed in our laboratory since we have presented the results in well known International Meetings.

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Chairman G.A.Leonards.
Thank you Mr.Escario for your contribution. The next will be Mr. Rolf Rosen, Sweden.

Mr.Rolf Rosen /Sweden/

The new Parliament building of Sweden is situated in the central part of Stockholm on the west slope of the esker of Brunkeberg. The soil consists here mainly of sand and gravel. To examine the soil the Swedish penetration test was mainly used. The new Parliament building was prepared to be founded on firmly stratified sand and gravel with a pressure of (5.0 kp/cm²). In the south-west of the building site, i.e. near the central part of the esker, where the original groundlevel was around 20 m higher than the foundation level, was an area of ca 40x60 m found,where the sounding rod sank easily up to 12 m and up to 3 m without any turns on the rod, see fig.1.

It was supposed that a so called dead ice hollow filled with clay had been found, and sedimentary soils from the higher parts of the esker had been stratified over the clay. However,samples of the material showed that it consisted of very uniform sand,which was very loosely stratified, see fig.2.

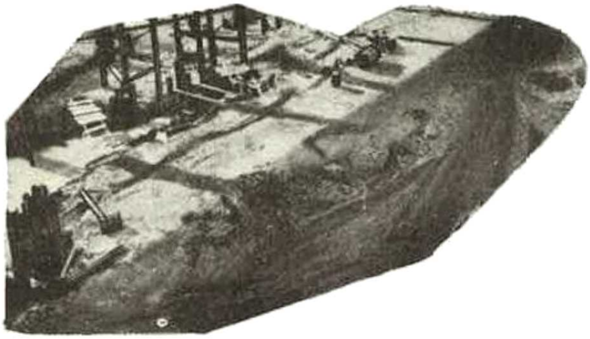

Fig. I

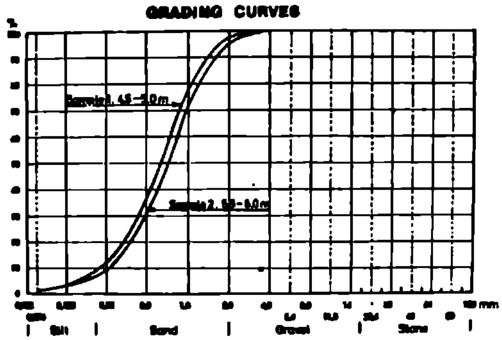

Fig. 2

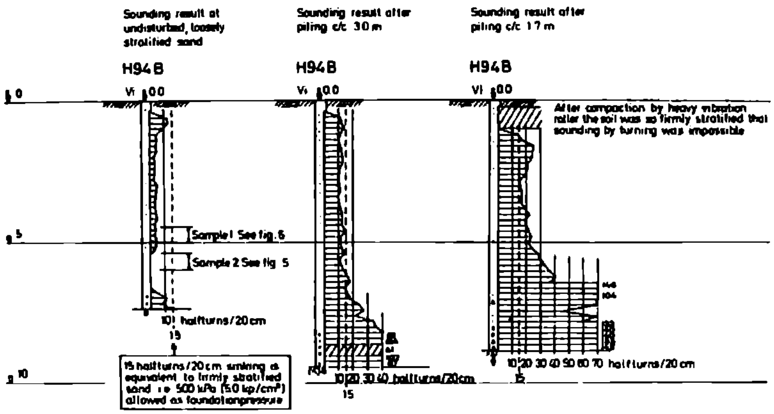

Fig. 3

It was thus necessary to strengthen the loosely stratified sand. "ith regard to the thickness of the loosely stratified layers, more than IO m., the soil was gradually compacted by concrete piles (25x25 cm.). The piling work started by driving the piles c/c 3.0 m. Each pile had a length corresponding to the thickness of the loosely stratified sand. In some parts especially in the outer parts of the area was this frequency enough, but for the greater part it was necessary to double the number of piles, see Fig. I.

For the entire compaction by piling, more than 200 piles were used to get the necessary compactness for foundation on slabs.
The following geological explanation of the origin of the loosely stratified sand is given. When an esker is formed it requires a tunnel with a very high water pressure in the ice , see Fig. 3.

Just outside the ice, the pressure ceases completely with the consequence that all the material transported by the ice river is deposited. The esker of Brunkeberg was formed ca 150 m below the water level. Probably a lot of material was deposited also into the ice-tunnel, so that a stopper arose. On account of the hig water pressure two waterstreams arose, one over and one under the stopping material. Through different speeds for the ice to melt, e.g. during summer and winter, and of the following different waterpressures a lot of esker hills were formed. Between the eskerhills could e.g. sand be stratified. Through the strong upward water stream that followed of the stopping the skeleton of the sand became very loose. Consequently, the way of forming is equivalent to the origin of quicksand. The loosely stratified sand has since kept its loosely stratifaction through the protecting archean, formed by the overlying sedimentary soils.

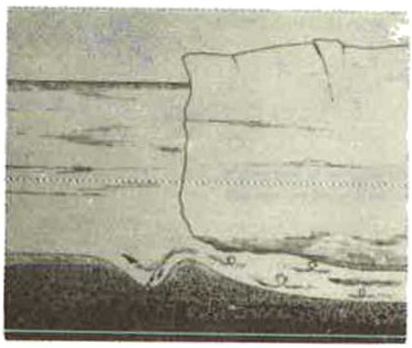

Fig.4.

Chairman G.A.Leonards. Thank you Mr.Rosen. Now I pass the word to Mr. Kogan from Moscow.

Mr.J.L.Kogan /USSR/

INFLUENCE OF MICROORGANISMS ON STRESSED STATE OF SOIL

Microbiological processes in soil are known to influence its mineralogical composition, structure and dispersion. However, V.Radina's investigations /1,2/ carried out in the Scientific Research Centre of the "Hydroproject" Institute have shown that microorganisms may have a considerable effect on the soil stressed state and therefore be the main cause of development of earth slides, damage of structure stability, formation of quick soil, volume expansion of soil etc.

Thus according to V.Radina's investigations the formation of a true quick soil is determined not only by the accumulation of new solid and fluid products of the microorganisms life activity in a water saturated dispersed medium, but it is also due to excess pore pressure, being the energy factor of the soil movement.

According to the medium conditions, the increase of gas content in soil released by microorganisms can be explained by the following:

a) soil pores dewatering-increase of relative humidity;
b) increase of soil volume-porosity increase;
c) pressure increase in restrained bubbles;
d) increase of gas content in pore water.

The increase of pressure in restrained bubbles and thus, in a particular functional relation, the increase of pore pressure, are caused by the intensity of the microbiological process of gas formation and by the drainage conditions. And if the pore pressure reaches the value of the total pressure, the soil expansion should take place.

V.Radina's experiments with the soil specimens from different regions of the Soviet Union including soils with characteristic properties showed that microorganisms developed pore pressures up to 4 kg/cm^2 under conditions favourable for their life activity. Our further investigations suggest that the pore pressure value may be even larger.

Other experiments with water saturated specimens of pure alluvial and eolian sands allowed V.Radina to show that at sowing the microorganisms and adding in them nutrient substratum, the undrained sand gained properties typical for true quick soil in the course of time (3-5 years), and its pore pressure increased up to a considerable value. Thus quick soil conditions of water saturated dispersed rock was for the first time reproduced in the laboratory, and thereby the possibility of controlling the process was proved.

The results of the investigations have opened up the possibility of solving a number of important problems in the field of geological engineering and in the different branchers of construction. They permit to develop rational ways of improving structural properties of true quick soils proceeding from the revealed conditions of their formation, from creating more effective methods of controlling the earth slides and other phenomena based on the appearance of excess pore pressure produced by microorganisms.

Unfortunately in this short communication I cannot dwell on the description of the experiments and the illustrative material and give examples showing that, to our mind, the life activity of microorganisms was the main cause of the development of slides and the formation of backpressure in the structure foundations.

With a great interest I have read the report of the canadian colleagues: E.Penner, W.Egen, J.Gillott /3/, who describe a case of the heave of a structure floor, erected on shale containing pyrite and some organic material. Comprehensive investigations revealed that the structure floor deformation was due to bacteriological weathering of the shale. The Authors write that they do not understand all the peculiarities of the weathering process that have led to the floor heave.

The effect of modern construction on the environment including the microstructure which is playing a significant role in the formation of the soil properties and its stressed state demands serious consideration. Study in this direction should provide a real possibility to bend enormous geological activity of microorganisms to the will and mind of Man.

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Chairman G.A.Leonards. Thank you Mr. Kogan. Now, at the end of our Session Prof.Dalmatov, Vice-Chairman will make summarizing remarks on the work of our Session. Prof. Dalmatov, will you please.

Prof.B.I.Dalmatov /USSR/

Dear Mr.Chairman, dear colleagues!

In his general paper submitted after his death Prof.L.Bjerrum presented profound analysis of the problems related to the mechanics of soft clay and structurally unstable soils as well as the questions connected with construction on these soils. Expounding briefly his paper in a most excellent way Prof. G.A.Leonards has thrown much light on the tasks facing the specialists in soil mechanics and foundation engineering which deal with the IVth session subject-matter.

An interesting paper of Prof.G.D.Aitchison which was kindly presented by Prof.C.M.Gerard raises also a great number of highly important problems.

The reports delivered by the major participants to the discussion have been complemented by very valuable information in the course of the discussion.

It is well known that there are an immense number of problems related to the solution to the questions of construction on soft and structurally unstable soils.Their enumeration alone would have taken a lot of time. So I would not rather go beyond the most general summary of further research tendencies. This summary included in the IVth session recommendations reads:

"It is necessary to carry out further collection and summing up of practical data on the behaviour of various structures on structurally unstable soils,to develop further the theoretical basis and field research procedures,designing and construction with due regard to particular features of aeolian settling,heaving,compressible and the similar types of soil".

I would like to dwell also on the problem of constructing buildings and structures on soft soils.

In his report Prof.M.N.Goldstein drew our attention to the fact that "structurally unstable soil" idea is of relative character. Structure unstability depends upon a type and force of stresses developed in the structure foundations.

The idea of "soft and compressible soil" is relative in not less degree. The heavier the structure is and the more is it sensitive to settlement non-uniformity,the greater number of medium quality soils should be regarded by the construction engineers as soft soils because these cannot be used as a natural foundation unless proper measures either to consolidate soils or to adapt the structure members to predicted non-uniform settlements are taken.

The Soviet Union standards recommend to design structure foundations on the basis of the strain limiting state, i.e.taking into account the limit non-uniformities of settlement. This point in our standards has enabled us in Leningrad to gather considerable experience in constructing the buildings with the height of up to 50 m. In some cases these buildings develop settlements of 40 cm and more.

It became possible to sum up this experience in construction due to the instrumental observations for soil deformations under the buildings under construction as well as in their vicinity. These observations carried out by Assistant-Professor S.N.Sotnikov (Leningrad Civil Engineering Institute) have showed that when erecting the 9-12-storeyed buildings on thick layer of soft soils (25-30 cm), these soils get deformed at a depth of more than 27 m below the foundation bed and at a distance of more than 30 m from the outer walls of the building erected on the foundation plate (observed by A.A.Sobenin)

It proves best of all the unacceptability of Winkler's hypothesis for evaluating the foundation plates although there were supporters of this hypothesis at the IIn session.

Research into the deformations of soft soils under the model foundations which was carried out by A.V.Golly (Leningrad Civil Engineering Institute) has shown that residual deformations of soil consolidation take place only within the layer approximately up to one width of the model square foundation. Below it the reversible deformations alone, i.e. elastic deformations and those of elastic aftereffect are developed. In Fig.1 the curve I is the graph showing total vertical displacements (settlements) of the ground bench-marks located at different depths under the foundation bed center; curve 2 is the graph of reversible deformations; dotted line 3 shows the lower border of the layer being consolidated.

The observed phenomenon can be taken into account when working out an evaluated scheme of the structure and foundation intersection. This scheme may be represented as an elastic semi-space with elasticity ratio of soil below the consolidation zone and as the soil limited layer undergoing the consolidation deformations.

Soft clay soils may sometimes cause a lot of trouble to the builders as a result of the foundation soil stability loss and the disturbance of soil natural structure when excavating pits for foundations.

Soft soil structure may be disturbed even under the buildings constructed scores of years ago. The observations show that in addition some settlements may develop even when comparatively small soil crumpling takes place. So in Leningrad it proved to be necessary to drive piles into the soil at a depth of 18 m at a distance of 3-4m from the building of the Kirov Academic Opera and Ballet Theater.

Hollow piles with the diameter of some 80cm were pressed into soil under static load. All the same there developed outer wall additional settlements which kept increasing over 3 years and have exceeded 10 cm.

Therefore it is necessary to carry out all-round research into the deformations of foundations made up of soft and structurally unstable soils with due regard to the conditions which appear when arranging the foundations and erecting and loading the structures as well as during the maintenance of the latter.

Thank you for your kind attention!

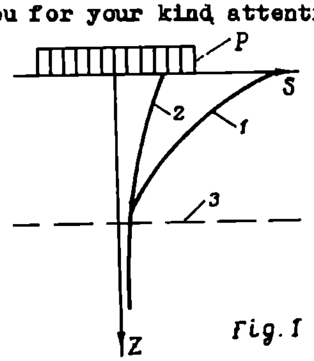

Fig. 1

Written contributions.

Dr.Ing.Rene VAN GANSE /Belgium/

My paper in Volume 2.2 of the Proceedings of this Conference, pages 233-237, has been mentioned by Prof.Aitchison, rather surprisingly as bringing out the value of lime stabilization in a loess in Belgium.

I would like to point out that the paper mentioned does not deal wish a particular Belgian loess soil, but with a wide variety of collapsible fine grain soils.

I have found necessary to underline the distinction which is always to be made between two different categories of affects brought about by lime in wet clayey soils, namely, the immediate amelioration of the soil properties, and eventually, later on, a long-term gain in strength after curing.

In earthworks operations in these soils, the important phenomenon is the immediate amelioration of the soil by a very small addition of lime, usually 1%.

This immediate amelioration is the consequence of a modification of the structure of the soil by the formation of sand-grain sized soil agglomerates which are stable in the presence of free water.

The notion of soil structure implicates the relative arrangements of soil particles, the bonds existing between particles, and the macroporosity between the soil aggregates.

The collapse phenomenon is known to be closely connected with the soil structure and with the tendency towards parallel orientation of clay particles under shear strains. These matters have been dealt with in the Main Session no.1 of this Conference, namely the structure of fine grain soils by Vyalov et al., Ter-Stepanian et al., Kulkarni, and Feda et al., as well as the consolidation of loess soils by Nuyens and Huergo.

The stability of the structure induced by lime in fine grain soils is a consequence of an instant adsorption of calcium and hydroxyl ions on the faces and edges of clay particles, linking these by water-resistant bonds which oppose parallel alignment.

The same instant effect has been found in the Centre de Recherches Routieres to be evolved as well by the hydroxides of other earth-alkaline elements, namely strontium and barium, which however do not produce long-term gains in strength, the pozzolanic reactions being different.

Carl Redel /USA/

Time-Strength Relationship in Remolded Clays

The action of kneading or remolding clays, alters their structural arrangement and results in the decrease of their shearing resistance. This phenomenon is known as sensitivity and is expressed as the ratio of the strength in the undisturbed state to that in the remolded state (Lambe and Whitman, 1969), at the same water content. The loss of strength is probably due to two different factors: the destruction of the orderly arrangement of the molecules in the absorbed layers and the injury to the clay structure (Terzaghi and Peck, 1967).

The sensitivity is generally determined in the laboratory by unconfined compression tests, and by vane shear tests in the field. The numerical values determined by the two procedures mentioned are not strictly comparable (Terzaghi and Peck, 1967).

A somewhat inverse phenomenon known as thixotropy, is the gain of strength with time, after remolding, at constant water content. This according to some (Terzaghi and Peck, 1967) is due to the rotation of the clay particles at constant volume to a more stable configuration, and according to others (Leonards, 1962 for example) is due to the development of an oriented absorbed water structure. Thixotropy cause some remolded clays to regain their full original strength (Tschebotarioff, 1951), whereas others appear to regain only a portion of their original strength (Wu, 1966). Further, according to Terzaghi and Peck (1967), the regain of strength is limited to the loss caused by the disturbance of the absorbed layers,while the remaining loss, probably caused by permanent alteration of the structure,is irrecoverable.

Sensitivity and thixotropic effects have been reported at length by Moretto (1948), Skempton and Northey (1952), Mitchel (1960) and others, but the implications of the shear strength recovery, of disturbed clays with time, on the solution of practical problems have been somewhat neglected. Depending on the material,the gain of strength may vary between negligible recovery to full recovery and the time required to complete the thixotropic effect may vary over a very wide range.

The writer has experimented by performing field vane shear tests in cohesive soils in the undisturbed and remolded states,plus additional tests after permitting the vane to rest umperturbed in the remolded soil for various increments of time. Is was found, for example, that the sentitive soft cohesive soils in the port of Ponce, Puerto Rico, regain their full undisturbed shearing strength within 15 minutes after remolding, whereas the sensitive soft cohesive deposits along the creeks and rivers in the vicinity of the City of São Paulo, Brazil, regain only a small portion of their shearing strength (about 8%) after 14 hours of rest. The available information is very scant and the influence of clay minerology, liquidity index, consolidation history, confining pressure and possibly other factors on the regain of strength of the remolded clays are not fully understood.

The practical importance of determining the undisturbed and remolded strength and the time-strength relationship after the disturbance or remolding has occurred, is illustrated below:

Example 1

Fill is to be placed on a sensitive soft cohesive soil and soon thereafter piles are to be driven. These would be driven from the newly placed fill, through the soft soil, remolding it, and into an underlying compact sand. Negative skin friction is expected to be imposed on the piles by the soft soil. Should the static analysis of negative friction on the piles be based on undisturbed strength, remolded strength or some value between these? Also, should there be a difference in the adopted shear strength of the soft soil, for piles to support a permanent or a temporary structure?

Example 2

A contractor is to cut into sensitive soft cohesive soil on which he is to place a filter course. Is the contractor to place the filter course immediately after excavation or will the disturbance, caused by excavation, mean excessive contamination of the filter and possible shear failure? Or is he to wait for the soil to regain strength, and then for how long is he to wait?

The practical problems to be solved are many and a great deal of research and field observation is required to further understand the phenomenon of time-strength gain of remolded clays. Special field vane shear tests, as described above,and similar laboratory vane shear tests copled with records of identification testing, overconsolidation ratio, confining pressures and mineral analyses will surely shed further light on this phenomenon.

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Mme KREMAKOVA-KARAPTCHANSKA, /Bulgarie/

En Bulgarie,les sols de loess couvrent de vastes surfaces. Ils occupent environ 1/11-eme du territoire du pays. Leur epaisseur moyenne est de 15 a 20 m.

La construction diverse et intensifiee sur ces sols durant les dernieres 15+20 annees en Bulgarie,a renforce l'interet pour leur etude plus approfondie.

S'appuyant sur un grand nombre de recherches en laboratoire et in situ,sur des observations systematiques des deformations des batiments et des ouvrages,sur l'experience acquise dans la pratique de construction et d'exploitation,de realisation de nouvelles solutions lors de la fondation, nous sommes arrives a de nouvelles conceptions dans les "Conditions techniques pour la projection dans des sols de loess" depuis 1966 et a l'elaboration de nouvelles normes et regles, qui sont en vigueur depuis 1972.

Nous nous arreterons en particulier sur quelques changements importants et quelques nouveautes dans ces normes.

Avant tout, les normes de projection dans des sols de loess sont etroitement liees aux normes de fondation dans d'autres sols. La pression admissible de sol n'est pas definie d'apres le degre de saturation d'eau, mais en fonction de l'indice de porosite et la consistance comme chez les autres sols.

Lors du calcul des deformations du sol de fondation, on prend en vue non seulement l'affaissement du sol, mais aussi le tassement des batiments. L'experience acquise par la pratique de construction et les resultats d'un nombre considerable d'essais en laboratoire montrent que le tassement dans la plupart des cas depasse l'affaissement. Cela s'observe dans 57% des cas envisages. Le tassement varie dans de larges limites de 1,5 a 17%, le plus souvent de 2 a 10%. Des observations faites a partir des essais effectues sur des batiments et des ouvrages, nous pouvons confirmer les conclusions faites d'apres des analyses en laboratoire. Par exemple, des tassements ont ete enregistres dans des silos de l'ordre de 12 a 40 cm, ce qui indique que ces deformations sont incluses dans le calcul des deformations du sol de fondation.

En general, ol faut souligner que nos sols de loess se caracterisent par une structure relativement fragile et par une petite densite qui, prises en combination avec d'autres facteurs comme par exemple- la charge verticale, le degre de saturation d'eau, la vitesse de pression etc., menent a de considerables deformations verticales. Un tableau est introduit dans les nouvelles normes concernant les deformations-limite totales du sol de fondation (tassement + affaissement).

On a note de meme quelques procedes nouveaux de fondation appliques avec succes chez nous, comme fondation de pieux (prefabriques, battus type "Franki", fores), mise en oeuvre d'un ecran a semelle en loess-ciment.

On a introduit plusieurs modifications lors de la realisation des travaux de prevention d'eau.

Dans les normes est relate un nouveau chapitre, dans lequel sont donnes des examples-solutions concrets de fondation dans la construction de quelques batiments et ouvrages largement effectues chez nous. Pour l'illustration de la methodologie de calcul sont donnes des exemples-solutions.

Prof. Molisz R. /Poland/

I would like to add some remarks concerning to the paper under the title "Field Studies of Embankment over Peat" which was written by Prof.Molisz as the senior author, Mr.Baran, Mr.Najder and me. In this paper was discussed among others comparison of long term consolidation coefficient in peat layer. Coefficient value calculated from the settlement observations of the test fill was twelvefold more than value obtained from laboratory tests.

As mentioned in our paper existing discrepancy in long term consolidation coefficient was due to disturbing of peat sample during sampling, extrusion and trimming. In addition, settlement effect of soft organic clay which is underlain peat was ignorated as well as influence of old fill.

In 1972 on the same area not far from the highway /approximately 400 m/ was built other test fill. It was constructed as a square shaped fill with side extends to 35m and 2,5m in high above ground level. Typical geotechnical profile shows 6.6m thick peat layer. Below peat occurs sand.

Peat was characterized with the same properties as peat which build subsoil under described highway.

The test fill was placed in layers rather regular.

The average long term consolidation coefficient taken from this test fill is equal 0.124. From laboratory tests the coefficient is equal 0.033. Such results indicate that coefficient obtained in situ is about fourfold more than obtained from laboratory.

In result of applying preloading layer 1.2m thick which remained on place about 70 days and reduction of load being equal 20% have been obtained reduction in the long term consolidation coefficient equal about 50%.

Summing up presented results can be stated that elimination such factors as initial loading on peat layer by old fill, and settlement which had been occured below peat caused important decreasing in difference between long term consolidation coefficients from field and laboratory tests.

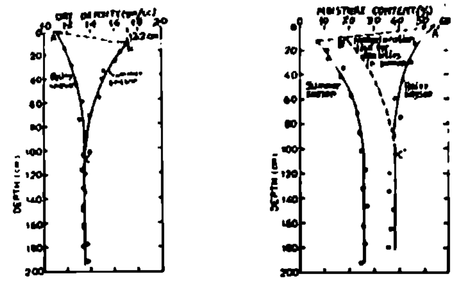
The rest existing difference can be explained as I have mentioned, by disturbing of peat sample during sampling.

Dr. R.K.KATTI and Dr.S.K.KULKARNI /India/

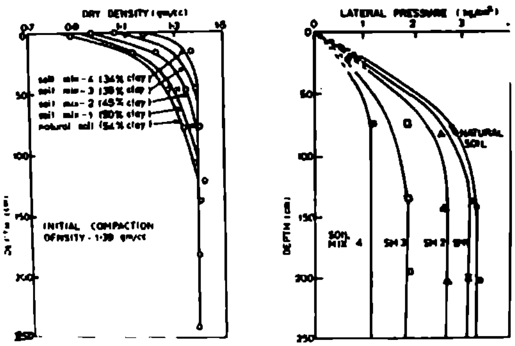
In this short report an attempt is made to highlight on certain important effects observed during the research work on expansive soils at IIT Bombay in relation to depth of soil mass and also with respect to the importance of clay minerals on swelling, swelling pressure and their interaction. To explain these effects a microstructure physical model coupled with micro-anchor concept is indicated.

Slide-1 shows the variation of dry density with depth during both extreme summer and rainy season conditions. It is clear that the density remains constant beyond a shallow depth of around 1 m or so. It is seen that the alteration in moisture content at a depth below about 1 m from summer to rainy season condition is not accompanied by change in dry density. The swelling pressure during saturated condition at this depth for the initial conditions corresponding to those in summer is around 1.6 kg/cm^2. This indicates that in rainy season condition eventhough there is an opportunity for the soil to saturate and swell the phenomenon of swelling is not observed beyond a depth of around 1 m. In other words, this high magnitude of swelling pressure is balanced due to reasons other than weight of overburden soil mass.

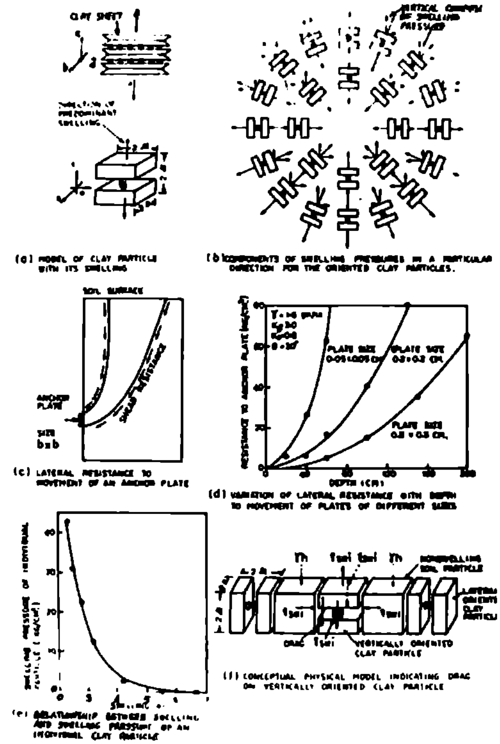
Slide-1 also shows the depth effects observed in the large scale tests conducted in the laboratory for varying clay contents. The



VARIATION OF DRY DENSITY AND MOISTURE CONTENT WITH DEPTH IN FIELD FOR SUMMER AND RAINY SEASON CONDITIONS.



VARIATION OF DRY DENSITY AND LATERAL PRESSURE WITH DEPTH IN LARGE SCALE TESTS FOR VARIOUS SOILS.

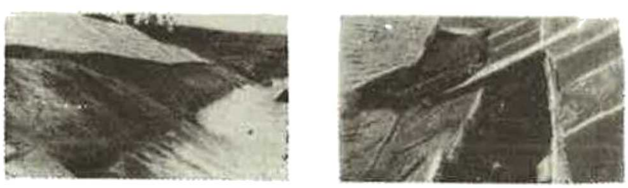


results indicate that the depth at which constant volume condition exists decreases with clay content. The lateral pressure with depth is a significant phenomenon to be noted since it deviates considerably from that for the conventional soils, viz. at a depth of about 1m only, the lateral pressure of the order of 3.3 kg/cm^2 is observed in case of natural soil. The vertical swelling pressures at the corresponding depths are nearly same as the lateral swelling pressures. These effects clearly indicate that the balancing of such high pressures may be attributed to the interaction of clay particles within the soil media.

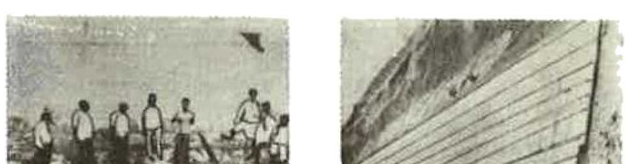
The physical model conceived to explain the effect is shown in Slide-2. In this case anisotropy of expansion of lattice of montmorillonitic clay particle is used to conceive development of swelling pressure in various directions. Using the mechanical analysis and idealizing the clay particles- in the present case 2 micron size-vertical and lateral equivalent pressure conditions are worked out for individual clay particles. Slide-2 also shows the relationship between swelling and swelling pressure of the individual clay particle as computed by the above method. This shows that a clay particle can exhibit a swelling pressure of 43 kg/cm^2 or more. Decrease in swelling pressure with increase in void ratio as observed in the tests may be attributed to expansion of clay particles in the void spaces.

Resistance to swelling of laterally oriented particles is conceived as a micro-anchor pushing against the soil as shown in Slide-2. Resistance to movement of the clay particles in the soil media as conceived by using Terzaghi's theory is also shown in Slide-2. This shows that resistance of 65 kg/cm^2 can develop even at a shallow depth of 60 cm for an anchor size of 0.05 x 0.05 cm. For the case of clay particles it may be even more.

In the light of the above theory, experiments were conducted both in the laboratory and the field to evaluate the efficacy of cohesive nonswelling soil layer to counteract the swelling pressure of a swelling soil media. Slide -3



CANAL LINING IN DISTRESS. WITHOUT COHESIVE NONSWELLING SOIL LAYER BACKING



CANAL LINING STANDING. WITH 1M. THICK COHESIVE NONSWELLING SOIL LAYER BACKING shows the effectiveness of 100 cm thick cohesive nonswelling soil layer in preventing the damage to lining of a canal slope. In the same slide the place where cohesive nonswelling soil layer is not used for lining and the distress has taken place is also shown.

These studies help in focussing the attention towards the fact that stress distribution in expansive soils is altogether different from conventional soils. This further may have considerable effect in developing the mechanics of expansive soil media.

Aldo R.REGINATTO /Argentina/

After paper 4/30 was submitted for publication additional research was carried out to determine how liquids are chemically modified by soils upon saturation,and viceversa, and the effect of such changes in collapse.

Results of this research confirm that the collapse phenomena upon saturation,for the soils analyzed,is due to dispersion of the clay fraction which makes up the bonds between the particles of macroporous collapsible soils. Test results shown on Tables 1A and 2A have been selected to illustrate the chemical interaction which took place between soils and saturating liquids.

<u>Variation of Soil Chemical Characteristics After Saturation</u>

Results of tests before and after saturation with different liquids indicate the following changes in the chemical characteristics of collapsible soils:

1) A marked reduction in the amount of soluble sodium cations occurs with saturation the other cations vary in a much lesser percentage. Consequently the SAR of the soil diminishes.

2) For the exchange cations, Na increases while Ca remains fairly stable after saturation with minor variations for other cations. The % exchangeable sodium of the soil increases after saturation.

3) The calcium carbonate content remains invariable. It is pointed out that in the soils tested this substance occurs mainly as small isolated concretions.

Table 1A shows results of tests on a typical collapsible soil sample from the Cordoba area in Argentina, before and after saturation.

<u>Variation of Chemical Characteristics of Liquids After Saturation</u>

The collapse susceptibility of the soils studied was measured with double oedometer tests (1), using fixed ring devices. The saturating liquids entered the samples through the bottom connection only,seeping upwards through a thickness of soil of about 20 mm. The samples of liquids after saturation were obtained with a hypodermic syringe from the upper part of the fixed ring devices, after seepage through the soil within the ring.

The following changes were noted in the liquids used to saturate the collapsible soil samples:

1) For drinking water,soluble Na content increases markedly, while K and Ca content increases to a much lesser extent. Consequently the SAR value of the liquid increases. pH levels change from slightly acid or slightly alkaline,to alkaline.

2) For domestic sewage,the increase in soluble sodium content is larger than for drinking water. Consequently the increase in the SAR value of the liquid is larger. pH levels remain more or less constant, well within the alkaline range.

3) For acidic water,the range of increase in soluble Na is similar in magnitude to domestic sewage, but there is also a marked increase in the soluble Ca content. Therefore the increase of the SAR value is the largest. A remarkable change in pH takes place, and the liquid becomes strongly alkaline.

Table 2A presents results of tests performed in liquids before and after saturating collapsible soil samples.

CONCLUSIONS

The main conclusions derived from the results obtained are:

1- An important exchange of soluble Na takes place between collapsible soils and saturating liquids. There is a marked reduction in the amount of soluble Na in the soil, and an increase of similar magnitude for the same cation in the liquid. Variations for other soluble cations are minor.

2- There is a marked increase in pH values for liquids which are acidic or slightly alkaline before saturation. If the initial pH value of the liquid is high before saturation it remains in the same order of magnitude after saturation.

3- The calcium carbonate content of the soil remains invariable before and after saturation.

4- The soluble sodium content of a soil,and the sodium dissolving capacity of a saturating liquid, play an important role in relationship to the susceptibility to collapse.

5- It is obvious that as a liquid solubilizes the soluble sodium in a soil, becoming strongly alkaline or remaining so if the initial pH is high,the clay dispersing properties of the solute are greatly increased.

6- Thus,the collapse noted for the soils tested is caused by dispersion of the clay fraction which makes up the bond between the particles. Therefore the sensitivity to dispersion or deflocculation of a soil in relationship to a given liquid governs the possibility and magnitude of collapse.

(1) See references on page 183,vol.2.2

TABLE 1A

Variations of Chemical Characteristics of Collapsible Soil Upon Saturation With Various Liquids

CONDITION OF SOIL	CATIONS IN SATURATION EXTRACT													SAR
	SOLUBLE CATIONS me/liter						EXCHANGE CATIONS me/100g							
	Ca	Mg	Na	K	Σ	Na%	Ca	Mg	Na	K	Σ	T	Na%	
Before saturation	14.0	8.0	8.0	0.8	30.8	25.0	9.4	1.0	0.6	1.0	12.0	12.0	5.0	2.2
After saturation w/domest. sewage	18.5	7.8	4.1	0.9	31.3	13.1	8.0	1.7	0.7	1.0	11.4	11.4	6.0	1.1
After saturation w/Drinking water	17.2	7.9	5.2	1.0	31.3	16.6	8.8	0.9	0.9	1.4	11.4	11.4	8.0	1.5

TABLE 2A

Variations of Chemical Characteristics of Liquids After Saturating Collapsible Soils

TYPE OF LIQUID	SOLUBLE CATIONS me/liter						RAS	pH
	Ca	Mg	Na	K	Σ	Na%		
Drinking water before saturation	1.9	3.1	o.65	o.08	5.73	11.3	o.3	7.6
Ditto, after satur.	3.2	3.3	26.0	1.1	33.6	77.4	10.0	7.9
Drinking water before saturation	1.9	3.1	0.5	0.1	5.6	8.9	0.3	6.8
Ditto, after satur.	2.5	1.0	4.5	0.3	8.3	54.0	3.4	7.9
Domestic sewage before saturation	2.5	0.4	1.4	0.4	4.7	30.0	1.0	8.5
Ditto, after satur.	3.6	1.2	9.3	0.5	14.6	63.0	6.0	9.0
Acidic water before saturation	0.9	0.4	0.37	0.61	2.28	16.1	0.5	5.7
Ditto, after satur.	3.1	3.4	34.0	0.52	41.02	82.9	13.0	8.0

E.T.HANRAHAN /Ireland/

'Soft ground must be strengthened by consolidation to render it capable of supporting heavy leads. Consolidation is normally effected by mechanical loading. The consequence of this loading may be a substantial reduction in volume of the subsoil and possibly large settlement of the surface and superstructure. However an undesirable component of the surface settlement may be movement due to overstressing or shear failure of the subsoil and it is important to avoid as far as possible this type of settlement. Also for t this reason it is essential that forecasts of settlement at the design stage make a clear distinction between the two types of settlement. The method of forecasting based on the e_g and e_k parameters (ref.1) has been found satisfactory for this purpose. During the construction period it is essential to accelerate the consolidation or strengthening process. This objective can best be achieved by using the expedient of small-diameter slotted plastic pipes which function as closely-spaced sand-piles (slide 1). This expedient has the further advantage that in deep déposits the strengthening process can be confined to a shallow surface crust, and the surface settlement is correspondingly reduced. In certain organic soils the consolidation process persists for extended periods following construction, and may be of sufficient magnitude to cause excessive distortion of the superstructure. The most satisfactory méthod of terminating this secondary settlement is undoubtedly preconsolidation, i.e. the removal of part of the load used for developing strength in the subsoil.

Organic soils frequently occur in ill-drained or low-lying locations.The loss of elevation which is associated with the partial removal of load may give rise to further problems, notably flooding. The use of lightweight fill (slide 2) as a replacement for the heavy material employed during the initial construction period is usually a satisfactory solution because it permits a reduction of load without loss of elevation.

Hanrahan, E.T., The e_g and e_k parameters. Proc.Roscoe Symposium. Cambridge Univ. England. Mar. 1971