Discussion leader's report

Rapport de l'animateur

D.G. FREDLUND, University of Saskatchewan, Saskatoon, Sask., Canada

Tuesday, August 15, 1989 (San Martin Room)

CHAIRMAN'S INTRODUCTORY REMARKS BY
Professor R.K. Katti, Chairman

Learned panelists, distinguished delegates, ladies and gentlemen. I thank Prof. Bengt Broms, President ISSMFE and organizers of this conference for requesting me to conduct and chair this session on DS-7, the part related to swelling soils. This session is organized under the auspices of TC-6 committee constituted by the president with members from the following countries:

Chairman:	Prof. R.K. Katti	India
Secretary:	Mr. K.R. Saxena	India
Members:	Prof. G.E. Blight	S. Africa
	Dr. A.A.B. Williams	S. Africa
	Prof. A. Balasubramaniam	S. E. Asia
	Mr. Eugenio Retamal	Chile
	Mr. Silvano Trevisan	Argentina
	Dr. P.W. Mitchell	Australia
	Dr. Balu Iyer	Canada
	Mr. Xi-ling Huang	China
	Mr. M. Londez	France
	Mr. A. Stamatopoulos	Greece
	Dr. L. Rethati	Hungary
	Dr. C. Behnia	Iran
	Dr. N.V. Nayak	India
	Prof. Joseph G. Zeitlen	Israel
	Prof. Ing G. Scarpelli	Italy
	Prof. S.A. Ola	Nigeria
	Mr. G.M. Omanje	Nigeria
	Dr. V. Escario	Spain
	Prof. J.A. Jimenez Salas	Spain
	Mr. R. Driscoll	UK
	Dr. Robert L. Lytton	USA
	Prof. E.A. Sorochyan	USSR
	Mr. M.F.C. Warren	Zimbabwe

Terms of Reference:

1. Organize conference in India in first half of 1988.
2. To propose reference test procedures for identification and classification of swelling soils and evaluation of swelling pressure and heave.

The committee has fulfilled the terms of reference as described below:

a) Organized the 6th International Conference on expansive soils in December 1987 in New Delhi, India under the auspices of Central Board of Irrigation and Power, New Delhi, India, supported by Indian Geotechnical Society.

b) To deal with second term of reference a committee was constituted with Mr. C. Sudhindra as Chairman to draw up a comprehensive technical questionnaire. The questionnaire was circulated to more than 2000 people all over the world. Answers to the questions provided a reasonable basis for preparing a status paper for discussion at the time of TC-6 committee meeting held at Ashoka Hotel New Delhi on 27th November 1987. This status paper was prepared by a committee with Mr. C. Sudhindra as Chairman, Mr. A.A.B. Williams as member and Mr. K.K. Moza as Secretary. The committee meeting was convened on 27th November 1987.

For the benefit of the geotechnical community, I am briefly giving below some salient aspects about the technical content of 6th International Conference on Expansive soils held at New Delhi, 1987.

The technical committee of the 6th I.C.E.S. made it a point to utilize this conference to indicate, a) progress made in the area of application of various principles and technologies to analyze, design and construct civil engineering structure in expansive soil regions, b) to identify persons who have worked on different aspects of expansive soils and request them to contribute papers bringing out up-to-date thinking, and c) organize invited lectures from persons who have worked in this field so that they can indicate direction to be followed to conduct fundamental research, basis research, technology development and engineering aspects in years to come so that a rational basis can be provided for dealing with problems in the area of expansive soils. In addition by invitation were obtained from different countries enumerating the nature of expansive soils present in their country and the methods in use for stable construction in their country on such deposits.

The following sessions were organized: (i) identification, mineralogy and structure of expansive soils, (ii) shear strength, consolidation and earth pressure, iii) swelling potential, swelling pressure and heave, (iv) problems and remedial measures associated with use of expansive soils, case histories, and (v) poster session.

The papers did stress the role of montmorillonite clay minerals in development of swelling pressure. Swelling pressure of a bentonite clay mineral can be as high as 6000 kg/cm^2. Water surrounding and inside the interlayer of clay mineral is in different state.

Some papers stressed the existence of close relation between estimation of heave and suction potential. However, many workers expressed the need to give a fresh thought to application of stress state variable concept for a swelling pressure and heave device. Use of total stress approach for a given equilibrium condition for a saturated expansive soil below the swelling pressure range has been indicated.

For saturated expansive soil under a given stress level, close relation exists between swelling-swelling pressure-void ratios-shear strength, (mostly in the form of cohesion) has been focused by some research workers.

Taking into consideration relation between swelling pressure versus heave and void ratios versus cohesion, some papers described how the cohesive non-swelling soil layer technology can be used to prevent swelling pressure and heave and use it for civil engineering construction in expansive soils.

Indian experience in use of CNS in river valley project in deep seated black cotton soil area for the past 14 years has been brought out.

There is a need to develop instrumentation to measure suction potential at clay mineral level and develop a rational approach to relate suction potential to heave, swelling pressure, lateral pressures, void ratio and shear strength, etc.

Method of analysis, design and construction can be evolved in civil engineering practice and methods can be confidently utilized only when the engineers can predict shear strength and deformation under different stress levels based on the measurement of simple soil properties.

There is a need to develop earth pressure, bearing capacity, stability analysis theories based on both total stress and stress state variable theories. Both soil suction potential and cohesion are resultant effect due to interaction between charged surfaces of soil and dipolar nature of water. Encourage research workers to study the expansive soil water system in the light of physical, chemical, electrical and minerological structural phenomena.

TC-6 committee after deliberating have come out with some guide lines. Dr. A.A.B. Williams will present the highlights on this topic.

TODAY'S PANEL DISCUSSION WILL BE CONDUCTED AS PER PROCEDURE SET UP BELOW:

A. Prof. R.K. Katti, Chairman: Brief remarks

B. General Report (Prepared by R.L. Handy, K.R. Saxena and R.K. Katti) to be presented by R.K. Katti who is present. A Detailed write-up will be incorporated in the proceedings.

C. Prof. A.A.B. Williams: Highlights of deliberations and recommendation of TC-6 committee meeting held at New Delhi 27th Nov'87.

D. Prof. D.G. Fredlund: Remarks on evaluation of heave and swelling pressure.

E. Fu. Hua. Chen: Total and differential heave observed and predicted.

F. Prof. J.A.J. Salas: Identification of swelling soils.

G. Prof. D.G. Fredlund will conduct floor discussion as discussion leader. He will compile the discussion and the writeup will appear in the proceedings.

H. Prof. Abelev will take over as Chairman to conduct DS - session on collapsible soils.

The chairmanship for this session was shared as follows:

i) the Swelling Soils portion by Professor R.K. Katti of India.

ii) the Collapsible Soils portion by Professor M. Yu Abelev of the USSR.

The General Reporter for the Swelling Soils papers presented to the conference was Professor R.K. Katti of India. Following the presentation of the general report on swelling soils, each of the panelists added a few comments on swelling soils. Mr. A.A.B. Williams of South Africa presented a report from the Technical Committee on Swelling Soils. The report dealt primarily with attempts toward standardization in the measurement of swelling pressure(shown at end of presentation to DS-7).

Next, supplementary remarks on the measurement of swelling pressure were added by Professor D.G. Fredlund of Canada.

DR. D.G. FREDLUND

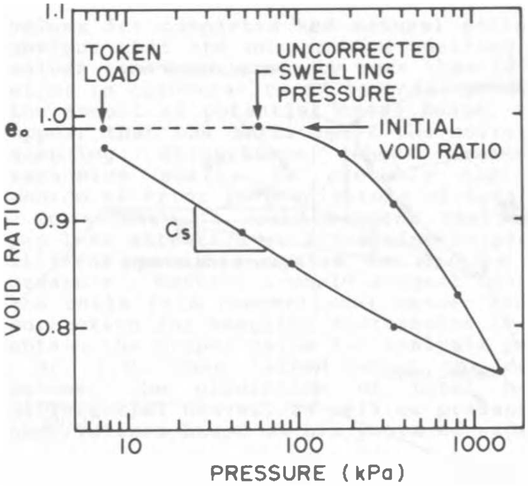
I would like to add a few comments on the interpretation and measurement of swelling pressure. In fact, the comments are by way of concerns that I have about how we obtain the swelling pressure of a soil.

I have noticed in the literature, over the past few years that researchers have attempted to construct a stiffer and stiffer measuring system in an attempt to measure the so called, "Best-Swelling Pressure." I would like to suggest that even the stiffest measuring system will never measure the "correct" swelling pressure. And the reason is related to a phenomenon called "disturbance" of the soil structure. A "disturbance" which is the result of a stress reversal being applied to the soil structure.

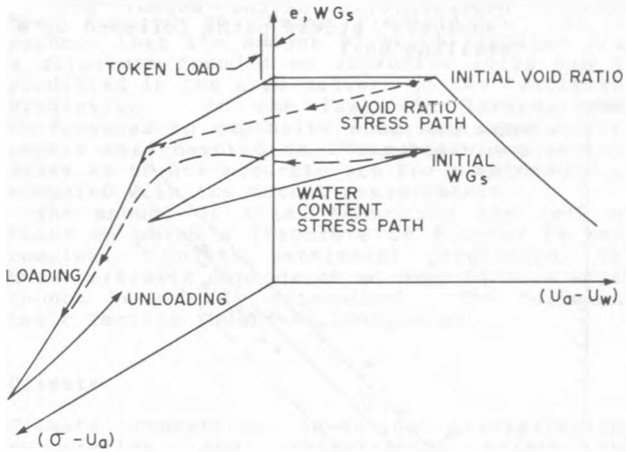
Researchers learned this lesson on saturated soils (with negative pore-water pressures) in the 1930's when attempting to determine a value for the so-called preconsolidation pressure. It seems we have to "re-learn" the same truth once again for "swelling pressure".

Let us consider the consolidation test data shown in Figure 1. The test was run using the "constant volume" test procedure. Suppose I were to ask you to interpret the data. One of the first questions you might ask would be, "Is the soil normally consolidated?" And I would respond, "That is what I want you to tell me through your interpretation of the laboratory data."

You might note that the "uncorrected" swelling pressure of the soil was 60 kPa and then attempt to discern the meaning of this value. But you might also attempt to perform some type of correction to the data in an attempt to account for the fact that a soil never exhibits a distinct break upon reloading. Rather, the soil always shows a curvature when approaching the loading curve. And so a correction of the type proposed by Casagrande in the 1930's might be applied to the data as shown in Figure 2. The question may then be asked as to whether the corrected pressure should be thought of as a "corrected" swelling



a) Conventional procedure for plotting 'constant volume' oedometer data



b) 'Ideal' stress path representation of 'constant volume' oedometer data

Figure 1 Typical test results on a swelling clay using a 'constant volume' testing procedure

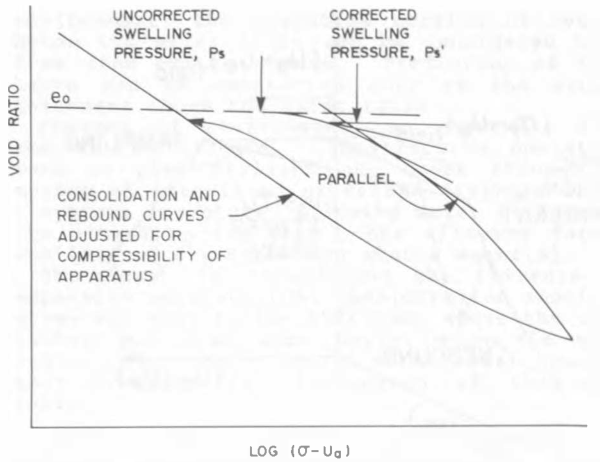


Figure 2 Empirical construction procedure to correct for the effect of 'sampling disturbance'

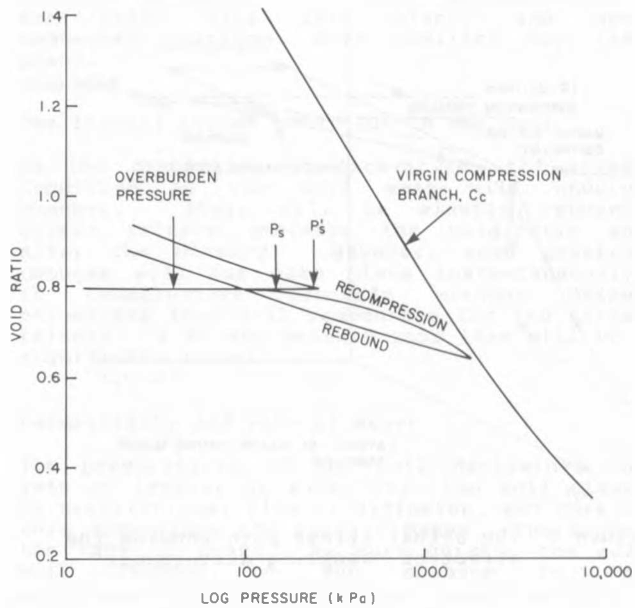


Figure 3 Relationship between the recompression curve and the virgin compression branch for a typical swelling soil

pressure or as a preconsolidation pressure. The answer to this question can only be confirmed if one knows the location of the virgin compression branch for the soil (Figure 3) and the overburden pressure and the pore-water pressure. The insitu pore-water pressure is negative and likely unknown.

There will be a break in the slope of the loading curve when the soil is loaded onto the recompression branch and a second break when the soil is loaded onto the virgin compression branch. In either case, there will be a gradual curvature associated with the changes of slope associated with each branch of the consolidation curve. In order to interpret the laboratory data, even for a saturated soil, it is necessary to perform a correction for the effect of sampling disturbance. Such a correction is always required when a soil has been loaded, unloaded and then reloaded.

The interpretation of the data could better be performed by plotting the data on a three-dimensional type plot with the horizontal axes designated as the net normal stress $(\sigma - u_a)$ and matric suction $(u_a - u_w)$ (Figure 4). The results then illustrate that when a sample in the oedometer is immersed in water, the initial suction of the soil is exchanged for a total stress. Water goes into the soil, increasing its water content while the void ratio is maintained as a constant value. As the soil approaches the recompression or the virgin compression branch, it will undergo a gradual curvature unto the new loading curve. This gradual curvature is referred to as "sampling disturbance" and must be accounted for through some type of correction procedure.

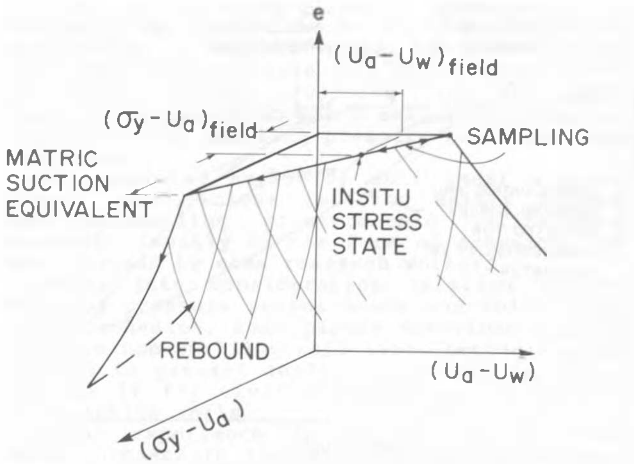


Figure 4 An 'ideal' stress path representation for a 'constant volume' oedometer test

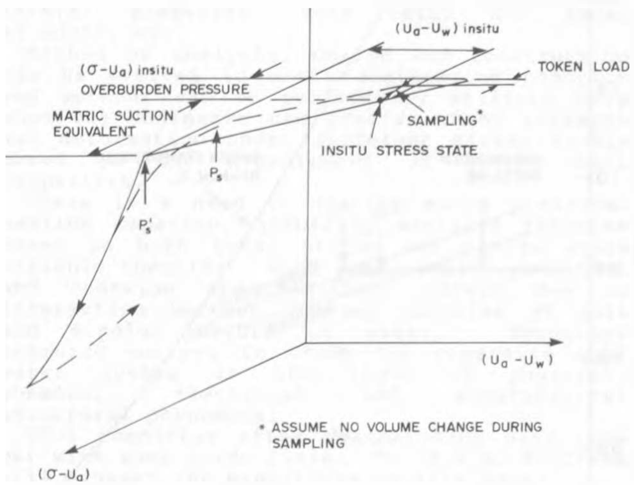


Figure 5 The actual stress path showing the effect of 'sampling disturbance'

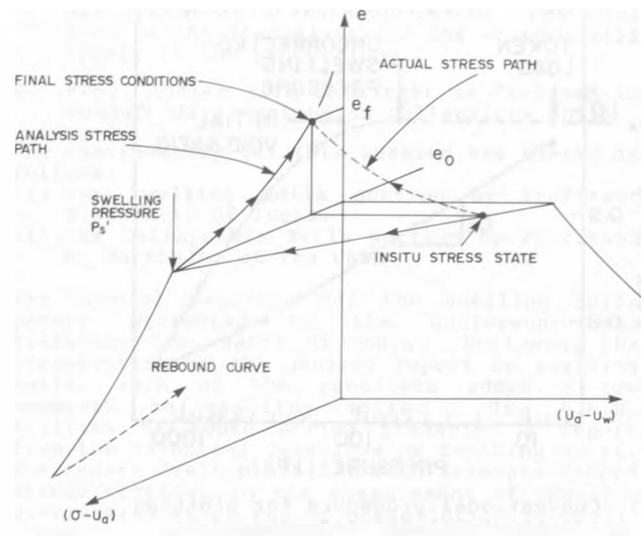


Figure 6 Representation of the 'actual' and 'analysis' stress paths followed by a swelling soil

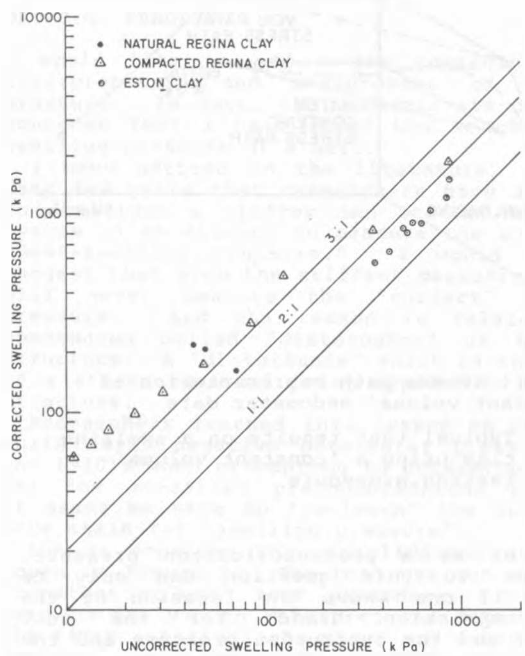


Figure 7 Change in swelling pressure due to the correction for 'sampling disturbance'

Figure 5 shows the meaning of the stress values which can be identified from the "constant volume" oedometer test. The uncorrected swelling pressure has little or no meaning. The corrected swelling pressure indicates the combined magnitude of the total overburden pressure and the matric suction transferred onto the total stress plane. This later value can be referred to as the "matric suction equivalent" of the soil. It is important to take the effect of "sampling disturbance" into account, particularly if the data are to be used for the prediction of total heave.

Figure 6 shows the stress path that a soil would undergo if wetted through water ingress in the field (i.e., the dash line). However, it is easier to use a stress path which follows the total stress plane when predicting total heave. In other words, to obtain the correct prediction of heave, it is necessary to completely rebound from the corrected swelling pressure values. Only in this way will the final void ratio of the soil will equal that experienced in the field (i.e., e_f). Failure to take into account this fact, (i.e., sampling disturbance) a prediction of heave could be 50 to 100% lower than what might occur in the field if the soil were completely wetted. This is a serious omission often made by geotechnical engineers.

Figure 7 shows a comparison between uncorrected and corrected swelling pressure values for compacted and natural soils. It is obvious that the uncorrected swelling pressure values can be in error by more than 100%. This error in turn results in an under-prediction of the amount of potential total heave. It would appear that the omission of the correction for sampling disturbance when dealing with expansive soils, is probably the greatest source of error in predictions of total heave.

In summary, I would suggest that we should pay less attention to attempting to produce the stiffest possible system to measure swelling pressure. Rather, I would suggest that we test the soils in a conventional manner and apply a correction for sampling disturbance in order to obtain the proper value for analysis purposes.

Mr. F.H. Chen talked about the difference between the prediction of total heave and differential heaves, as well as presenting some observations based on his years of experience.

MR. F.H. CHEN

In recent years, a great deal of attention has been focused on "Heave Prediction". It is assumed that the amount of heave expected from a structure founded on expansive soils can be predicted in the same pattern as the Settlement Prediction. In the last six International Conferences on expansive soil, at least twenty papers were devoted on the subject. Questions arise as to how accurate are the predictions as compared with the actual measurements.

The amount of total heave and the rate of heave on which a structure is founded is very complex. Unlike settlement prediction, the heave estimate depends on so many factors which cannot be readily determined. The following basic factors should be considered:

Climate

Climate conditions involving precipitation, evaporation, and transpiration affect the moisture in the soil. The depth and degree of desiccation affects the amount of swell in a given soil horizon. Climate condition partially affects the desiccation.

Thickness of expansive soil stratum

In most cases, the thickness of the expansive soil stratum extends down to great depth and the practical thickness is governed by the capability of surface water penetrating into the stratum. For practical purposes, we assume a depth of 15 feet, and in some extreme cases, surface water penetrates through the seams and fissures of expansive clays to as much as 30 feet.

Depth to water table

Soil below the water table. However, in the case of claystone bedrock, due to fine-grained structure and the nearly impermeable nature, water is not able to penetrate and saturate the material. The only access of water into the stratum is through its seams and fissures. Consequently, when taking moisture content of submerged claystone, one may found in the vicinity of seams and fissures, while relatively dry conditions can be found elsewhere. Under fairly stable and unchanging environment, the claystone portion of bedrock below the water table can be considered to be free from volume change. Prediction of total heave can be considered only on the stratum thickness above the water table.

Changes of environment can, however, alter the entire picture. Construction operations such as pier drilling can break through the system of interlaced seams and fissures in the claystone structure, allowing water to saturate the otherwise dry area, thus allowing further swelling of the otherwise stable material.

Therefore, in considering the thickness of expansive soil stratum, consideration should be given not only to the thickness above the water table, but also some depth below the water table. This depth should be at least equal to that of possible fluctuation of the water table.

Nature and degree of desiccation of the soil

The predicted amount of heave depends on the initial condition of the soils immediately after construction. If the excavation is allowed to expose for a long period of time, desiccation will take place, and upon subsequent wetting, more swelling may take place.

The initial stress condition in the soil

On the completion of excavation, the stress condition in the soil mass will undergo changes. There will be elastic rebound. Stress release increase the void-ratio and alter the density. However, such physical changes will not take place instantaneously. If construction proceeds without delay, structural load will compensate for the stress release. I do not believe this item will be a significant amount.

Permeability and rate of heave

The permeability of the soil determines the rate of ingress of water into the soil either by gravitational flow or diffusion, and this in turn determines the rate of heave. The higher the rate of heave, the more quickly the soil will respond to any changes in the environmental conditions, and thus the effect of any local influences will be emphasized. At the same time, the higher the permeability the greater the depth to which any localized moisture will penetrate, thus engendering greater movement and greater differential movement. Therefore, the permeability is an important factor and the higher the permeability, the greater the probability of differential movement.

Extraneous influence

The above mentioned basic factors, although difficult to predict, still theoretically can be evaluated. Extraneous influence at the same time is totally unpredictable. The supply of additional moisture will accelerate heave. For instance, if there is an interruption of the subdrain system to allow the sudden rise of a perched water table. The development of the area, especially residential construction, will contribute to a drastic rise of perched water table.

Various methods have been proposed to predict the amount of total heave under a given structural load. These are the Double Oedometer method, the Department of Navy Method, the South African method, and the Del Fredlund method. All suggested methods have limitations. The Double Oedometer method developed by Jennings and Knight is based on the concept of effective stress and has received wide attention.

The Department of Navy in its Design Manual outlined a procedure for estimating the magnitude of swelling that may occur when footings are built in expansive soils. The shortcome of this procedure is that it is assumed that at a lower depth, the soil will not swell as much as at an upper depth due to overburden pressure. In fact, even at depth 8 ft., the overburden pressure amounts to only, at most, 1 kip/ft.^{2} which is small in highly expansive soil areas.

The Van der Merwe method, commonly known as the South African method, started by classifying the swell potential of soil into very high to low categories. The assign potential expansive (P.E.) expressed as in in./ft. of thickness based on the following:

Table 1

Swell Potential	Potential Expansion (P.E.) in./ft.
Very High	1
High	1/2
Medium	1/4
Low	0

Assume the thickness of an expansive soil layer or the lowest level of ground water. Divide this thickness to several soil layers with variable swell potential, and calculate the total expansion.

Professor Fredlund used the results of oedometer tests in terms of void-ratio to predict the total heave.

Many empirical methods have been established to predict heave. The input data commonly needed consists of classification data such as Atterberg limits, initial water content, dry density, and percent clay content.

Johnson and Snethen introduced the suction method for heave prediction. They concluded that the suction method is simple, economical, expedient, and capable of simulating field conditions.

The search by various investigators for a reliable method for predicting total heave is probably affected by the concept of ultimate settlement in the theory of consolidation. For many years, engineers have been familiar with the calculation of ultimate settlement and differential settlement of a structure founded on clay, and it assumed that the total heave can also be predicted. There are some fundamental differences between the behavior of settling and heaving soil. Some of them are as follows:

Settlement of clay under load

Settlement of clay under load will take place without the aid of wetting, while expansion of clay will not be realized without moisture increase.

Total amount of heave

The total amount of heave depends on the environmental conditions, such as the extent of wetting, the duration of wetting, and the pattern of moisture migration. Such variables cannot be ascertained, and consequently, any total heave prediction can be entirely erroneous.

Differential settlement

Differential settlement is usually described as a percent of the ultimate settlement. However, in the case of expansive soils, one corner of the building may be subjected to maximum heave due to excessive wetting while another corner may have no movement. Therefore, in the case of swelling soils, differential heave can equal the total heave. No correlation between differential and total heave can be established.

A proposed reservoir will have an approximate capacity of 10 million gallons with a plan dimension of 185 feet by 360 feet. Subsoil beneath the proposed reservoir consists essentially of claystone and sandstone bedrock with shallow clay overburden at one end. The site was investigated by drilling 32 deep exploratory borings. A total of 108 swell tests were performed to determine the swelling potential and swelling pressure of the subsoil at various depths. The water table was measured at depth 12 to 33 feet below ground surface.

Test results indicate that the swelling potential under design pressure varies from nil to 7.5%. Swelling pressures tested at natural moisture content ranges from 1,600 psf to 22,000 psf, while the air dried samples showed swelling pressure 13,000 psf to 75,000 psf. For heave prediction, it was assumed that there is a possible absolute maximum value. The following data were used:

Table 2

Liquid Limit	56 to 67%
Plasticity Index	32 to 41%
Dry Density	95.7 to 110.8 psf
Swelling Potential under 1,000 psf Load	1 to 4%
Swelling Pressures	5,000 to 26,000 psf
Natural Moisture Content	11.5 to 21.1%
Maximum Swelling Potential under Most Adverse Conditions	9%
Maximum Swelling Pressure under Most Adverse Conditions	35,000 psf

By using Van der Merwe method, the Department of Navy method, and others, this predictive total heave is as follows:

For maximum condition, assuming 9% potential heave and 20 feet for a depth of heave gives 11 inches. For minimum conditions, assuming 4% potential heave and 6 feet for depth of heave gives 1.0 inches.

The heaving will take place at the eastern portion of the reservoir, and assuming no heaving will take place at the western portion of the reservoir, the heave value of 11 and 1 inches should be considered as differential heave.

It is my contention that the maximum differential heave predicted for the reservoir should be 11 inches. A rational value used for design purposes should be the average of maximum and minimum or 5.5 inches.

On further examination, it was felt that if the proposed drain system is properly installed and the adversed drying and wetting conditions do not take place, differential heave should not be as much as predicted. It was decided that the structure engineer should design the reservoir for only 2.75 inches of differential heave. The structure engineer protested at this figure, claiming that the cost of such design will be out of reach. We finally settled to design for only 1.5 inches.

The reservoir was built in 1981, latest survey indicated that differential heave is less than 1/4 inches.

The above case indicates clearly the drastic difference between predicted heave and actual heave. In this case, it was on the safe side, the reverse can also happen. Finally, I wish to point out the legal responsibility of a geotechnical engineer in giving a definite figure for heave measurement. It is a dangerous area for practicing engineers to touch.

The General Reporter for the collapsible soils papers presented to the conference was Mr. W.R. Mackechnie of Zimbabwe.

MR. W.R. MACKECHNIE

(submitted separately)

Following the presentation of the general report on collapsing soils, a discussion period was lead by Professor D.G. Fredlund of Canada. Introductory remarks were made on the relationship between swelling soils and collapsing soils, followed by the consideration of three questions by the panelists and persons from the audience.

D.G. FREDLUND

The two types of soils being discussed today are found primarily in arid regions of the world. If we consider a map of the world (Figure 8) we note that about 33% of the earth's surface can be classified as arid or semi-arid (i.e., the potential annual evaporation significantly exceeds the precipitation). This is a significant portion of our land surface.

I would also note that the soil mechanics discipline had its inception and growth in the more humid environments of the world. As such, the soil mechanics science has catered primarily to regions where the soils are essentially saturated and the pore-water pressures are positive. Our textbooks primarily provide information on the behavior of saturated soils and as such teach us saturated soil mechanics. But today, we are discussing situations where the pore-water pressures are negative and where an understanding of unsaturated soil mechanics is

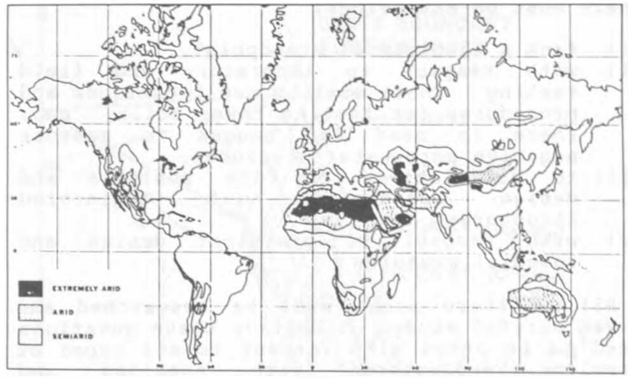


Figure 8 Map showing the arid regions of the world (Meigs, 1953)

required for engineering practice.

It is now about 25 years since special attention began to be given to expansive soils with the recognition that their behavior appeared to deviate from that of classical saturated soil mechanics. It is only a few years ago since special attention was given to the behavior of collapsible soils, recognizing that their behavior deviated from that of classical soils. While we are not discussing residual soils in this session it is worthy to note that it is only about seven years ago since serious attention was given to residual soils because of their unique behavior. Once again, these soils commonly have negative pore-water pressures.

At the ASCE Conference in Hawaii on Construction and Design Problems Related to Residual Soils, it was interesting to hear a past graduate student from the United States explain the problems he had when attempting to apply the saturated soil mechanics theories to residual soils in his country. The message was that classical soil mechanics was satisfactory only when the pore-water pressures were positive and the soils were saturated. However, it was suggested that extensions to these theories were required for most cases involving residual soils.

All the above soil types have one thing in common. They do not accurately adhere to many of the concepts and behavioral patterns taught by classical, textbook saturated soil mechanics. Therefore, there is need for extensions to classical soil mechanics which will accommodate these needs.

We could ask, "What do these soils (i.e., swelling and collapsing soils) have in common?" And I would say that from a basic standpoint what they have in common is the fact that they all have negative pore-water pressures. They may be unsaturated, but more fundamentally, their pore-water pressures may be highly negative, even many atmospheres negative. We know that swelling soils have very negative pore-water pressures and more recently it has been recognized that collapsible soils have negative pore-water pressures.

This fact (i.e., negative pore-water pressures) has soil mechanics ramifications and requires extensions to our science in several areas:

There must be extensions:

i) from a theoretical standpoint,
ii) with respect to laboratory and field testing. There must be new techniques and procedures for testing these soils, and, there is need for gauges to measure negative pore-water pressures,
iii) to formulations for the analysis and design associated with engineered structures, and
iv) with respect to practical design and remedial measures

All of these areas must be researched and given careful study. I believe these questions need to be asked with respect to all types of problems encountered with swelling and collapsible soils in engineering practice. Not just with respect to volume change problems but also with respect to shear strength, seepage, lateral earth pressure, bearing capacity and other problems.

We ought to be able as geotechnical engineers to address the same type of questions for unsaturated soils with negative pore-water pressures as we ask for soils that are saturated.

I have noted that over the last 25 years, researchers in expansive soils have come together each four years, for a conference on expansive soils and have made presentations which go something like this. "Here are some examples of heave of engineered structures in our country; and here is what we do about them." And each four years the process is repeated. However, there appears to be no attempt to develop a consistent theoretical framework or basis for analysis and design. "Theory is the glue" which can bring together the thinking and understanding from all countries, and I believe there is a need to come to a common consensus on the theory for soils with negative pore-water pressures.

For purposes of our discussion period, I have put together three questions to which I would solicit a response from the panelists and the audience.

The first question is:

1. a) In what ways are expansive soils and collapsible soils the same?
   b) In what ways are expansive soils and collapsible soils different?

Your answers can be made with respect to: i) theory, ii) testing procedures, iii) methods of analysis, design or remedial measures.

The second question is with respect to expansive soils:

2. a) What are some recent advances with respect to understanding the behavior of <u>expansive</u> soils and handling these soils from a practical standpoint?
   b) What are the greatest needs for future research relative to <u>expansive</u> soils?

The third question is with respect to collapsible soils:

3. a) What are some recent advances with respect to understanding the behavior of <u>collapsing</u> soils and handling these soils from a practical standpoint?
   b) What are the greatest needs for the future relative to research on <u>collapsing</u> soils?

Due to a shortage of time, the response to the third question was limited. In general, there appeared to be a consensus that it was logical to consider both swelling and collapsing soils simultaneously. Possibly this should be given consideration with respect to future conferences.

DR. J. GRAHAM, VISITING PROFESSOR, UNIVERSITY OF HONG KONG

The discussion leader (D.G. Fredlund) asked whether the processes taking place in expansive soils are similar to or different from those in collapsible soils. I suggest they are different.

"Expansive" soils are also "compressive" soils when current stresses exceed the swelling pressure. However, this compressive process is inherently different from "collapse". We should distinguish between "compressive" (or "contractant") and "collapsible" behaviour.

For some years I have been testing a saturated 50/50 mixture of sand and bentonite proposed for use in the Canadian Nuclear Fuel Waste Management Program. Examination of the applicability of the effective stress principle has shown that this saturated sand-bentonite mixture wants to reach an equilibrium relationship between the applied "effective" stress ($\sigma - u_w$) and specific volume V (Graham et al., 1989. This "Swelling Equilibrium Line" (SEL) is similar in many ways to the Normal Consolidation Line (NCL) in normal soil mechanics. However, it is postulated that movement along the SEL is reversible (i.e., unloading does not in the long term produce an unload-reload line as in normal soils).

In Figure 9 for example, a saturated specimen at A will not be in equilibrium under its mean total stress, p, and pore-water pressure, u_w, measured on external pressure transducers. Instead, an unbalanced net Repulsive minus Attractive force (R-A) between the particles will draw water into the soil and cause

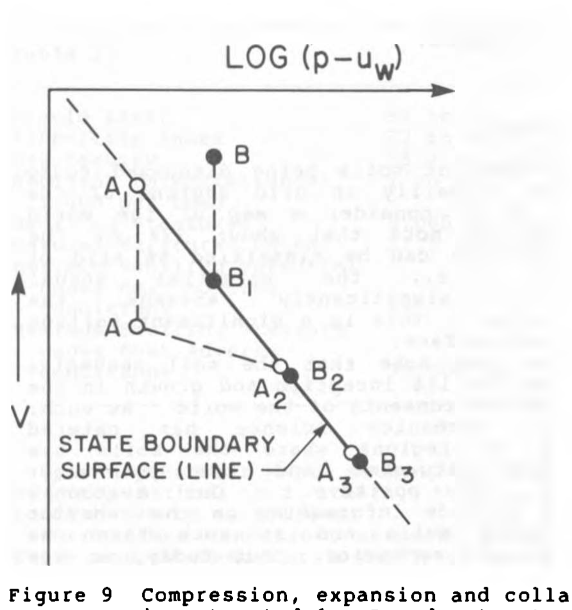


Figure 9 Compression, expansion and collapse in saturated 1 - D oedometer tests

expansion to A_1. Alternatively, if $(p - u_w)$ is increased from A, the soil will compress successively to A_2 and then A_3. Similarly, expansion will take place from A_3 to A_1 and beyond when $(p - u)_w$ is decreased and sufficient time is allowed for the specimen to reach equilibrium. This is essentially a reversible process or in other words, swelling clays are also compressive clays.

It is possible to think of wetting-drying and freezing-thawing processes that will take a soil structure in the short term from the SEL to drier "states" such as A. Subsequent wetting or unloading permits the soil to recover its SEL relationship and thereafter to produce expansive or compressive volume strains as the stresses or pore-water pressures change. With this approach, clays are restricted to regions of the log $(p - u_w)$, V-graph on or below the SEL which therefore becomes a state boundary line.

It is also conceivable in certain circumstances that a soil can exist in Figure 9 at a state such as B above the SEL. This condition exists in very loose water-deposited sands, in marine clays that have subsequently had salt-rich porefluid replaced by freshwater, and in clays where evaporation of porefluid from the ground surface produces cementation. Processes have not been identified that can consistently take a soil to less-dense states outside its current state boundary surface. Metastable structures seem mostly to be associated with changes in soil chemistry.

If clay at B is loaded, it will experience rapid decrease in volume (that is, "collapse") to B_1. It will thereafter react more slowly with compression along the SEL to B_2 and B_3. If the earlier postulation is correct and the clay is expansive, compression along the SEL is reversible. Unloading will produce expansion to B_2, to B_1 and indeed, to A_1 or beyond. However, it will not be possible to return to B. The collapse mechanism that produced the volume changes from B to B_1 is not reversible. Similar behaviour can be proposed for unsaturated soils. Figure 10 (redrawn from Fredlund and Morgenstern (1977)) shows a state boundary surface SBS) in V, $\log(\sigma - u_a)$, $(u_a - u_w)$-space. When tested with $(u_a - u_w)$ constant, an unsaturated specimen such as C inside the SBS, will exhibit stiff, small-strain behaviour to C_1 and then more compressible (but not collapsible) behaviour to C_2. If at this stage the volume is held constant and the suction $(u_a - u_w)$ is increased, then the total stress $(\sigma - u_a)$ must decrease to C_3 to keep the stress path on the SBS. Subsequent increase of $(\sigma - u_a)$ with $(u_a - u_w)$ again held constant produces further compression to C_4. As in Figure 9, it is possible to have specimens such as D which are metastable with a V, $\log(\sigma - u_a)$, $(u_a - u_w)$ state lying outside the SBS. An increase in $(\sigma - u_a)$ will initially cause rapid compression (collapse) to D_1, followed by the normal compression process from D_1 to D_2. If the soil

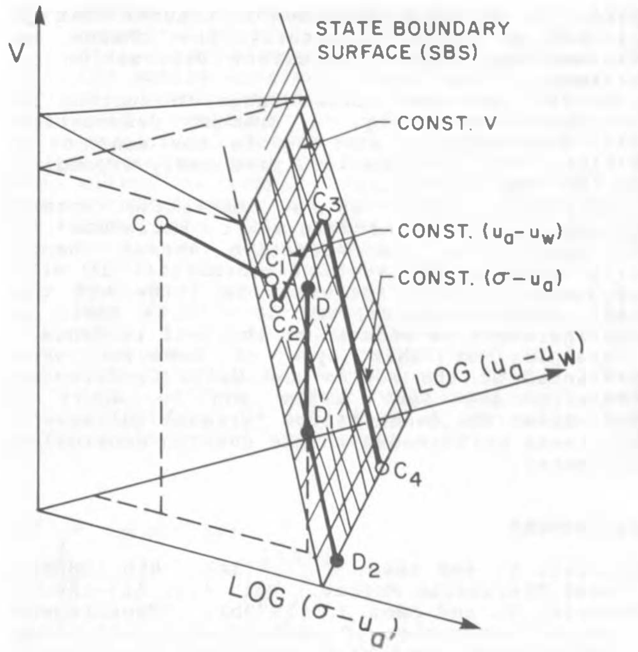


Figure 10 Compression, expansion and collapse in unsaturated soil

is expansive like the sand-bentonite discussed earlier, unloading will permit long-term expansion to D_1, C_2, C_1 and beyond, but not to D.

I suggest we restrict the terms "compressive" or "contractant" to compressions such as those shown in Figures 9 and 10 on the state boundary surface. In expansive clays, these compressions will be partly or totally reversible. The term "collapsible" should be restricted to metastable states outside the state boundary surface. Compressions associated with collapse mechanisms are not reversible.

REFERENCES

Fredlund, D.G. and Morgenstern, N.R. (1977). Stress state variables for unsaturated soils. Journal of the Geotechnical Engineering Division ASCE 103, 447-466.

Graham, J., Saadat, F., Gray, M.M., Dixon, D.A. and Zhang, Q.-Y. (1989). Strength and volume change behaviour of a sand-bentonite mixture. Canadian Geotechnical Journal 26, 274-281.

DR. VENTURA ESCARIO

The word collapse means a sudden or at least very fast failure or large deformation of a material.

There are many kinds of collapse, but in this intervention I will refer only to that originated by a decrease in the capillary suction of a soil.

If a partly saturated soil is flooded, it sufferes a quite sudden and large change of suction from the value it originally had to zero. It is, as a consequence, natural that if it had a loose structure, the change be followed by a quite important deformation or collapse.

But if the same total change in suction is introduced gradually, a gradual deformation will also develop arriving to the same or a similar total deformation, presumably depending on the type of soil.

Therefore, this is a completely normal phenomenon where the deformation introduced is a function on the effective stress change originated by the suction increment: it will be large if such increment is large and the soil structure loose, and it will be small if the increment is reduced or the soil is dense.

Examples of this type of behavior were presented at the Moscow and Haifa Conferences (Escario and Sâez 1973a and b) where I introduced the denomination "gradual collapse", for tests performed with the suction controlled oedometer.

REFERENCES

Escario, V. and Sâez, J. (1973a). 8th ICSMFE. Oral Discussion Moscow. Vol. 4.2, 123-124.

Escario, V. and Sâez J. (1973b). "Measurement of the properties of swelling and collapsing soils under controlled suction", Int. Conf. on Expansive Soils, Haifa, 195-199.

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Objections were put forward during this discussion concerning the possibility of:
a) prognosis of behaviour of expansive soils.
b) classification of expansive and collapsible soils.

Indeed, these problems cannot be solved without a thorough study in soil microstructure (i.e., in its fabric and in structural reasons of its behaviour in collapse and swelling. The necessity of microstructural considerations found understanding in this Conference (compare DS-1 and also Rizkallah and Keese, 1989, 1/25, Alonso et al., 1989, 7/3).

a) The most probable structural reasons of swelling clay are:

i) diffuse layer repulsion, p_R, between structural elements, resulting in soil suction, which energy or pressure may be calculated theoretically, obtaining a good agreement with the measured values. This part of swelling may be identified with the "primary" mechanism (Alonso et al., 1989, 7/3),

ii) particle delamination with time due to rebound and/or to increased water content of an expansive clay during swelling. This may be identified with the second mechanism (Alonso et al., 1989, 7/3), which is "responsible for long term behaviour and is a result of hydration of active clay minerals".

Indeed, with particle delamination there increases the amount of hydrating water, (i.e., the hygroscopic water content, w_h) (Fig. 11), measured at the relative humidity $p/p_o = 0.95$. It is due to a higher specific surface of thinner particles: water sorption is proportional to the external specific surface

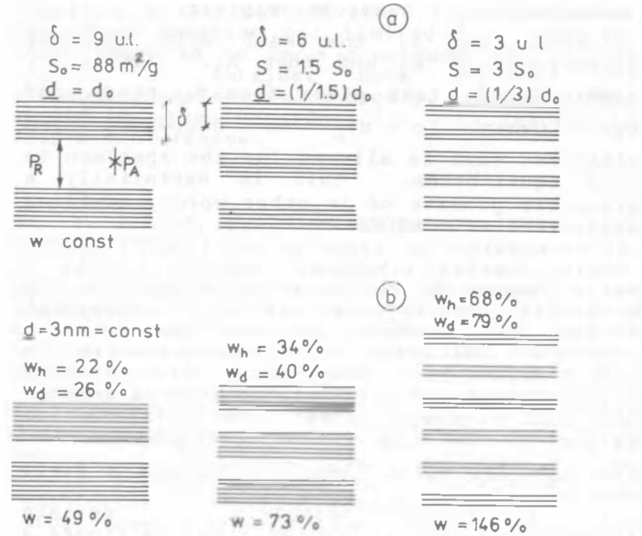
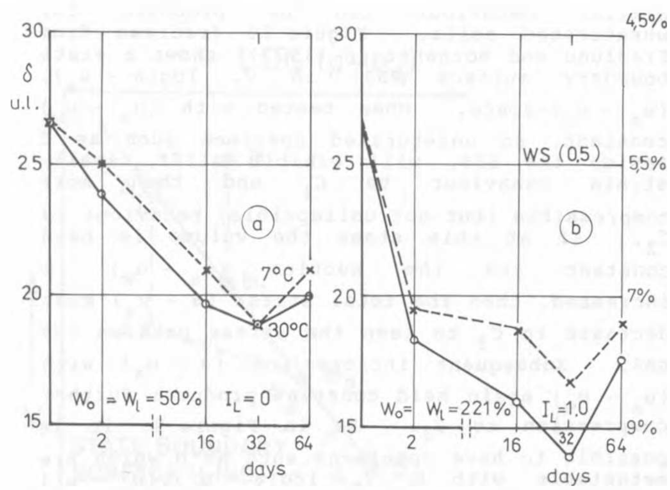


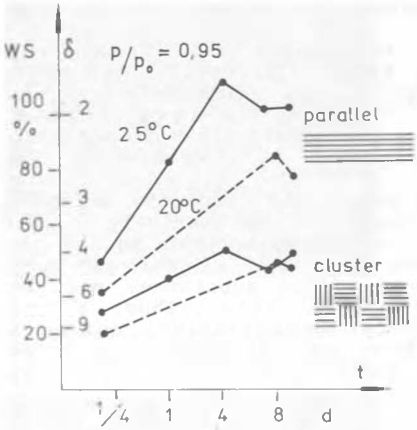
Figure 11 Delamination in clay swelling (a) at constant volume and water content, W, (b) at constant interparticle distance, 2d. σ is the particle thickness in number of unit layers, (i.e., nanometers in dry state), S is the external specific surface, p_R is the diffuse layer repulsion, p_A is the van der Waals attraction, W_h is the hygroscopic water content, measured at $p/p_o = 0.95$ (strongly bound water). W_d is the diffuse layer water.

and is inversely proportional to the average particle thickness. Delamination was observed when a Na-bentonite was stored for a prolonged time at a higher water content (Fig. 12). It may be the reason of an excessive clay

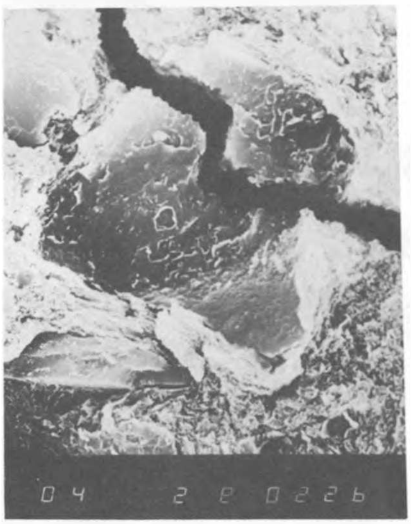


a) Particle thickness at plastic limit   b) Particle thickness at liquid limit

Figure 12 Change in particle thickness, σ, with time (Stepkowska and Olchawa, 1987)



a) Water sorption versus particle arrangement for Na-bentonite



b) Scanning electron micrograph (2,000x) of a grain of parallel structure within the matrix of cluster structure (Brebent bentonite Na, picture breadth 50 μm)

Figure 13 Influence of microstructure (parallel particles or clusters) and temperature on the water sorption, thus on clay swelling with time: Na-bentonite (Stepkowska and Jefferies, 1983).

swelling, exceeding values observed in short-term study in the laboratory (see e.g., Alonso et al., 1989, 7/3 their Fig. 4). Also Bucher et al., (1989, 7/5) observed a kind of stepwise swelling pressure development in medium scale experiments on MX-80 bentonite and in Montigel.

The discrepancy between swelling as measured in laboratory and as observed in field, may be due to following microstructural reasons:

i) Time effect: stepwise increase in swelling with time is being observed, most probably due to particle delamination. This may not be considered in short-term laboratory study.

ii) Effect of particle arrangement: parallel smectite particles delaminate and swell more readily, causing a higher swelling of the entire specimen, than that of a sample of cluster structure (Fig. 13), provided this swelling is not restricted by geometrical hindrances of the neighboring elements. This indicates

iii) effect of sample size: small samples of parallel structure delaminate more readily than the big ones, where parallel structure is trapped inside that of restricted swelling (cluster structure). Indeed it was observed that water sorption may depend on sample size (Fig. 14). Thus, there is also a discrepancy between clay swelling in laboratory and in field. Also Rizkallah and Keese (1989, 1/25) state, that amount of settlement and coefficient of subsidence C_s decrease when sample height increases.

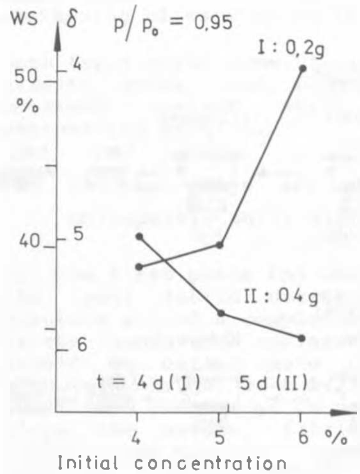


Figure 14 Influence of sample size on water sorption, thus on swelling of Na-bentonite: suspensions of the initial concentration indicated were prepared, dried and their water sorption was measured (Stepkowska and Jefferis, 1983)

The effects mentioned above may be easily observed as a change in water sorption, WS, or water retention, WR, at various relative humidities, (e.g., p/p_o = 0.5 and 0.95. WS and WR are proportional to specific surface, thus inversely proportional to particle thickness, σ. Clays indicating big differences between WS and WR, as well as big differences in these values at various relative humidities, may have high and variable swelling properties. Otherwise clay has a stable particle thickness and its swelling is due only to diffuse layer repulsion and/or rebound of rigid particles (kaolinite).

Values of SW and WE measured in various clays are quoted in (Stepkowska, 1987).

These values indicate that the ratios of WE/WS > 1 and W(0.95) to W(0.5) > 1.5 may be used for identification of swelling clays and those exhibiting the danger of slides.

b) The simplest way of classification of expansive is a direct measurement of their properties. Some indirect indications are suggested, (e.g., Moza, 1987) gives a review of published identification methods and classifications of expansive soils and he mentions following parameters: clay fraction content (< 2 μm), plasticity index and shrinkage limit, as well as specific surface area (see above WS and WR) and estimates therefrom the probable expansion.

Snethen (1984) proposes liquid limit, plasticity index and natural clay suction. Sridharan et al., (1985) uses the free swell index > 2 cm^3/g, (i.e., volume occupied in water by 1 g of oven dry soil containing kaolinite). Chen (1975) estimates swelling potential from plasticity index, see Moza (1987).

It is suggested here that the simple water sorption test and water retention test (WS test and WR test, Fig. 15) are included into the identification procedures and that more study in microstructure is done using (e.g., scanning electron microscopy).

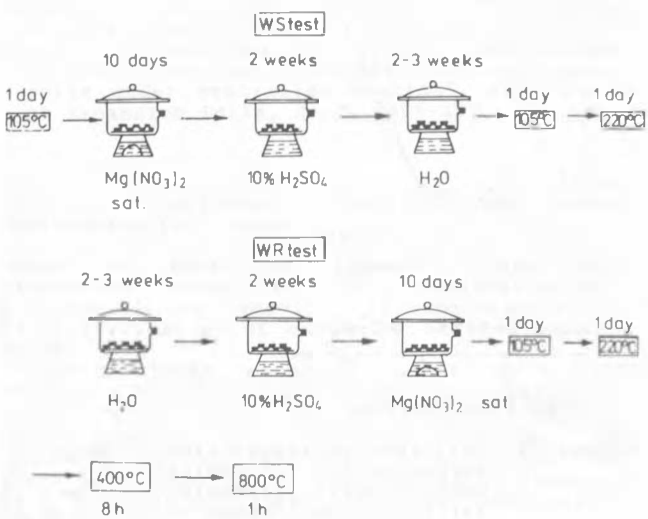


Figure 15 WS test and WR test

The same concerns the identification of collapsible soils, which property must depend primarily on microstructure and mainly on porosity, which must be high.

Ali et al., (1989, 7/2) mention here the following collapse - related (C_p) soil parameters: in situ dry unit weight, γ_d, water content, w_o void ratio, e_o, porosity, n_o, degree of saturation, S_r, and the plastic limit, with the highest correlation coefficient for C_p. They mention the following collapse criteria: (a) Gibb's (1961) collapse ratio, R, (b) Alfi's (1984) collapse parameter A and percent collapse (C_p) following the saturation under load (Jennings and Knight, 1957). Rizkallah and Keese (1989, 1/25) indicate that collapsible soils are "well sorted, fine grained elastic sediments as well as porous soils of a relatively low cohesion". Major effect is the internal unit fabric.

The following reasons of collapse may thus be mentioned:

i) disturbance of contact bonds and/or of cementation between structural elements (grains, aggregates), which form a loose structure. This results in filling of free pores (macropores) with soil material and is accompanied by increase in dry density.

ii) collapsible soils must indicate a loose structure with comparatively big macropores. Contacts between aggregates (grains) must indicate a small surface area and/or cementing substances must be water soluble (Fig. 16). Thus, either water (possibly water tension) and/or a definite load causes disruption of these bonds and a sudden settlement of the clay system.

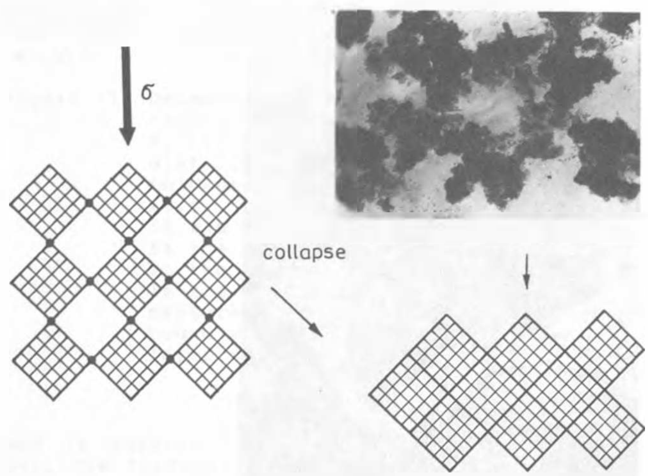


Figure 16 Schematic representation of collapse in soils of cemented aggregates and a transmission electron micrograph of CS-smectite (D.W. Thomson)

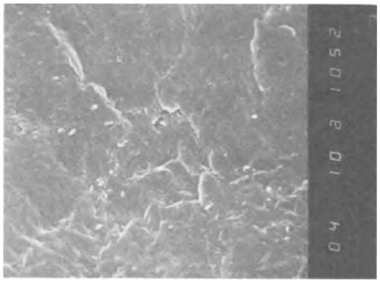
iii) change of arrangement of uncemented aggregates or grains from a loose structure to the densest packing of structural elements may happen when the intergrain friction is eliminated by water entering the system or it is overcome by the external load.

Clarification of the microstructural reason of collapse in the given collapsible soil and study of cementing material may be helpful in adequate prognosis of its behavior. The microstructural study, including scanning electron microscopy with simultaneous chemical analysis is strongly recommended.

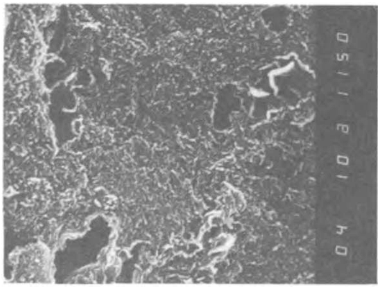
REFERENCES

K.K. Moza (1987) Identification Mineralogy and Structures of Expansive Soils. General Report, Technical Session I. 6th International Conference on Expansive Soils, New Delhi.

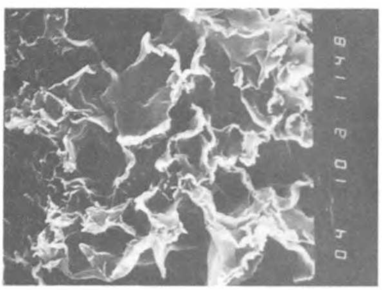
E.T. Stepkowska and S.A. Jefferis (1983) Study



a) parallel type structure



b) cluster type structure



c) floc type structure

Figure 17 Examples of three types of microstructure for some bentonitic clays at a magnification of 10,000x with a picture width of 10 μm.

in Microstructures of Clay Slurries. Arch. Hydrotechniki, 30, 193-211.

E.T. Stepkowska and A. Olchawa (1987) Influence of Water Content Temperature and Time on the Particle thickness in Bentonite (in polish). Prace Naukowe Instytutu Geotechniki Politechniki Wroclawskiej, Nr 52, Konferencje Nr 24, 137-142.

E.T. Stepkowska (1987) Physical Properties of Expansive Soils Related to Particle Thickness. 6th International Conference on Expansive Soils, Additional Papers, 55-62.

Panelist contribution
Contribution de panelist

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Several questions were posed by Dr Fredlund and my comments are as follows:

1.(a) In what ways are expansive soils and collapsible soils the same?

Both types of soil are initially in a non-saturated state and coupled with this, therefore, is the difficulty of applying theoretical concepts that have been developed in terms of effective stress.

Both types of soil can be either transported in mode of origin, or residual from the in situ weathering of various rocks.

Both types occur commonly in arid or semi-arid climate zones, and it is the increase of moisture content which brings about an engineering problem.

(b) In what ways are expansive soils and collapsible soils different?

In the first place the changes which occur in the soil fabric during the behaviour in question are of a completely different nature. In the 'sands with collapsible fabric', as they should be called more rightly, the micro-structure changes both by re-orientation of the particles and change in packing. In heaving clays the natural fabric may change very little, the relative orientation of particles changing only slightly. (The term 'soil fabric' is here used with reference to the spatial arrangement and orientation of the particles and the amount and shape of the voids.)

Further, the physico-chemical changes which result from the addition of water are thought to affect the external surface of the main particles in soils with a collapsible fabric, while in expansive soils it is the internal surface area where the major effect occurs.

The time scales over which the problems arise usually differ most significantly, heaving occurring over several years while collapse may occur in minutes in the extreme case. The amount of movement, too, can be significantly different, expansive soils increasing in volume by commonly 2 to 3 per cent, while collapsible soils may decrease by up to 10 per cent.

A major difference in behaviour is that expansive soils often cause as much of a problem through reversal of movement, when they are usually termed 'shrinkable soils'. The usual type of collapsible soil does not exhibit this continually reversible behaviour with change in moisture content.

Perhaps the points of difference are clearly summarised by saying that the problems are most often those concerning expansive clays, or of collapsible sands.

(c) Answers can be with respect to (i) theory (ii) testing procedures (iii) methods of analysis, design or remedial measures.

While many of the structural solutions may be the same, this is because the manifestation of the problem is one of differential movement.

The actual mechanisms involved are quite different as appears from the above comments, and it is thought that different theories are still needed, with different testing procedures and empirical methods to suit, for there to be significant progress in practical solutions to the problem.

2. EXPANSIVE SOILS

(a) What are some recent advances with respect to understanding the behaviour of expansive soils and handling these soils from a practical standpoint?

There now seems to be a trend towards the consideration of differential heave and its prediction, rather than the sole prediction of total heave, which is an important aspect of soil/structure interaction and adequate design. This is treated in some recent publications from Australia and in the proceedings of the Lagos Regional Conference.

Secondly, an important advance has been made with the production of a code of practice for the design of Residential Slabs and Footings - Australian Standard 2870-1986. This is aimed principally at expansive soils for which design guides for stiffened rafts are given to four categories of soil profile behaviour - those exhibiting slight (S), medium (M), high (H) or excessive (E) for predicted surface movement.

Recent advances too have resulted from consideration of soil suction as an important parameter controlling boundary conditions to the problem and affording a more rational approach to prediction of the effects of changed environment. This has been accompanied by more widespread use of techniques for the measurement of soil suction, with some simpler techniques emerging than the use of thermocouple psychrometers.

In South Africa, in particular, there has been a large increase in the use of stiffened rafts, with articulated structures, as an economical solution to founding houses or light structures on heaving clays. While this form of construction may have been common in parts of the U S A and Australia it has only recently been employed elsewhere. This viable option has perhaps aided the research and development as well of a more economical solution to the anchored pile and suspended floor system for the housing market.

(b) What are the greatest needs for future research relative to expansive soils?

There are several needs in the present context, such as better methods for determining the parameters required for stiffened raft design, the "hang off" distance, the mound shape exponent, or the design soil modulus.

There is an over-riding need for the development of constitutive equations for expansive soil behaviour in terms of parameters that can be measured in a practical fashion. While promising progress has been made in a theoretical framework, a practical method for analysis and design which is generally applicable has not yet emerged.

There is still not enough known about soil/structure interaction so that overall engineering design can exploit the combined affect of foundation and superstructure, particularly for low-cost structures like mass housing. There seems to be a discontinuity in expertise between the structural engineer and the geotechnical engineer, except on major projects.

3. COLLAPSIBLE SOILS

(a) What are some recent advances with respect to understanding the behaviour of collapsing soils and handling these soils from a practical standpoint?

It is very difficult to recall any major advances of late in this field.

(b) What are the greatest needs for the future relative to research on collapsing soils?

One major need is again the development of constitutive equations describing the soil behaviour, in terms of measurable parameters. There is also a lack of well-documented case histories, or extensive full-scale experimental or follow-up work, which would provide data for verification of theoretical developments. Perhaps a major effort on collection of full sets of field data would help define particular unknowns that are as yet not clearly identified.

Contribution to discussion
Contribution à la discussion

A.A.B.WILLIAMS, Division of Building Technology, Pretoria, South Africa

Review

In following on the general report of Dr R K Katti, Chairman of Technical committee TC6 on Expansive Soils, about the activities and progress made, I have been asked to review the outcome of the meetings held in New Delhi before the Sixth International Conference on Expansive Soils in December, 1987, as well as any significant aspects which emerged from the proceedings.

A particular task of the Committee had been to propose reference test procedures (i) for the identification and classification of swelling soils and (ii) for the evaluation of swelling pressure and heave. On the first part, the returns from 16 different countries had been bound in a 200 page document, but no succinct conclusions were possible at the time. Secondly, a small sub-committee was set up to draft a test procedure which might become a standard - a task given to C Sudhindra, K K Moza and A A B Williams. They prepared the description of a method using standard laboratory consolidometers and a 'swell-under-load' technique, in which similar specimens of non-saturated clay were subjected to different loads and then inundated with water. The results were then presented in the form of a plot of "Percentage Swell" against the logarithm of "Applied Pressure". The linear relationship, which was so often observed, could then be used for interpretation of 'swelling pressure' (under no volume change) or 'the free swell' (under a nominal load or perhaps 1 kPa).

The data drawn from various sources (Ref 1,6,8) mentioned during the Sixth conference in Delhi seemed to confirm this linear relationship and the results are summarised in Figure 1. At that conference there seemed to be wide acceptance of this behaviour, with but few doubts about the general validity of a straight line plot. During the writer's recent visit to Chile, results of swelling tests on clays from Santiago also seemed to fit the picture and have been included in Figure 1. Some data given at this Conference by Signer et al (1989) have also been shown and seem to lend further confirmation.

The intent of the TC6 sub-committee had been to develop a standard procedure for testing the swelling characteristics of a heaving clay, so that the findings from different countries could be compared on a common basis. Several features also kept in mind were the need for relatively simple apparatus, so that many laboratories with unsophisticated facilities could perform the test, and the applicability of the results for practical design purposes. The outcome was the use of oedometers already in common practice, with the hope that methods of more detailed interpretation could develop from what appeared to be a fairly universal type of behaviour for swelling soils, much in the same way as in the standard consolidation test with a graphical construction or curve fitting technique.

The Sixth International Conference on Expansive Soils in New Delhi was attended by about 300 participants and about 90 papers are contained in the Proceedings (published by the Central Board of Irrigation and Power, Malcha Marg, Chanakyapuri, New Delhi-110 021). A significant aspect was the reporting of the distribution of expansive soils from more than fifteen countries where major problems with the

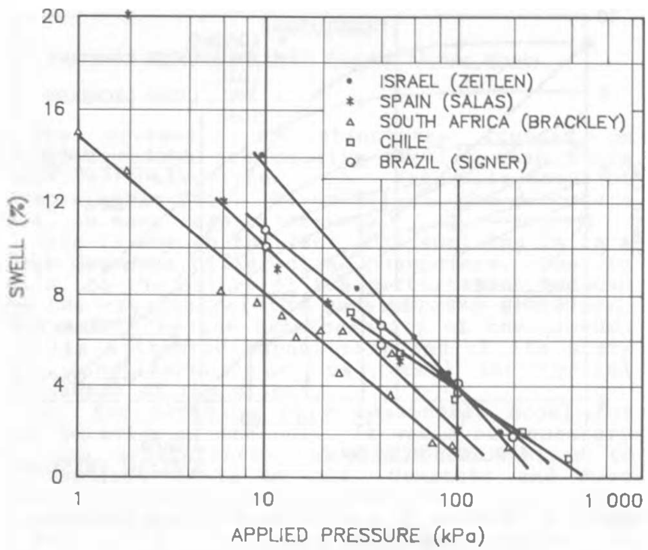

FIGURE 1: SWELL UNDER LOAD

cracking of structures was still being experienced. Buildings, canals, roads, tunnels, as well as heavily loaded structures, were still exhibiting severe distress over wide-spread areas. The problems in practice appeared to be as prevalent as they were at the time of the First International Conference on Expansive Clays in 1965, in spite of the progress made in understanding the mechanisms involved and in providing methods for prediction, analysis or design. One successful practical solution for canal linings developed in India was the use of a cohesive non-swelling layer (CNS) about 1 m thick over the active clay - very often a black cotton soil.

Discussion

In referring to the proposed swell-under-load test and the interpretation of the maximum swelling pressure which can be achieved with no volume change, some recent work by Pellissier (1989) has been aimed at establishing a procedure for obtaining consistent results. He has suggested that the set of similar samples (say, four) be loaded initially to 10 kPa and after equilibrium has been attained, with perhaps slight settlement, this condition be taken as the zero from which volume changes are measured when the load is changed and the samples are inundated. These procedural options for testing have been referred to by Iyer (1989) in the Delhi conference and reviewed by El-Sohby et al (1989) at this conference. Previous results published by Pidgeon (1987) are shown in Figure 2 which clearly illustrate the effect of different stress paths.

Several comparisons have been reported in the literature of the swelling pressures as determined by the various methods. Pellissier, using the initialized swell-under-load test mentioned above, found that fair agreement was obtained between the interpretation from this test and the direct measurement of swelling

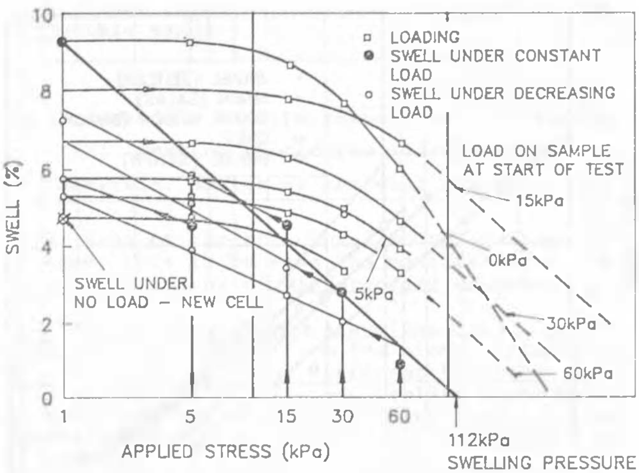

FIGURE 2: RESULTS OF STRESS PATH OEDOMETER TEST ON UNDISTURBED SAMPLES

pressure in a stiff frame, allowing no volume change. The deformability of the stiff frame, including discs and filters, must have an effect, but for ease of use in practice he suggested limiting the strain to a maximum of 2,5 per cent of the sample height when a complete dummy, using a steel disc, was loaded to 1000 kPa. This limit might have to be revised if dense materials were being tested, such as very soft rock weathered mudstones.

It is apparent that many factors still influence the determination, or rather interpretation of the swelling pressure, or even other swelling characteristics. There is a need, however, for some standard test procedure that can be used for practical purposes and for the interchange of data, and I like the present suggestion of Jim Graham (Australia) to call the semi-log plot a "swelling-equilibrium-line". The adoption of a standard name, or description, for what appears to be this universal type of simple behaviour might remove the resistance to accepting a term such as 'the swelling pressure' which is clearly defined in theoretical concept, but may involve complex testing or specialist interpretation. Because current usage of the phase is now so established it may well be pragmatical to talk of a 'swelling pressure' from the swelling-equilibrium-line (following initialized swell-under-load tests) as distinct from the true swelling pressure, which is still a matter for discussion or further study.

References

1. BRACKLEY, I J A (1983) The effects of density, moisture content, and loading on swelling of clays. CSIR, Pretoria, NBRI Special Report, BOU 66, 16p.

2. EL-SOHBY, M A, MAZEN, S O and ABOU-TAHA, M M (1989) Effect of apparatus deformability on swelling pressure. Proc Twelfth Int Conf on Soil Mech and Found Eng, Rio de Janeiro, Vol 1 p 589-592.

3. IYER, B (1987) Influence of procedural factors on the swelling behaviour of soils. Proc Sixth Int Conf on Expansive Soils, New Delhi, Vol 1, p 149-154.

4. PELLISSIER, J P (1989) The determination of swelling pressure in active clays. Engineering Structures and Information Technology Programme, Division of Building Technology, CSIR. Internal Report.

5. PIDGEON, J T (1987) The prediction of differential heave for design of foundations in expansive soil areas. Proc of Ninth African Regional Conference, Lagos, Sept 1987, 12p. (NBRI Reprint No. BOU 98).

6. SALAS J A J and ROY M (1987) Preliminary evaluation of the swelling danger - overview on the Spanish practice. Proc Sixth Int conf on Expansive Soils, New Delhi, India, p 217-224.

7. SIGNER S, MARINHO F A M, SANTOS N B AND ANDRADE, C M M (1989) Expansive and collapsible soils in semi-arid region. Proc Twelfth Int Conf on soil Mech and Found Engng, Rio de Janeiro, 1989, Vol 1, p647-650.

8. ZEITLEN J G et al (1987) Report on Expansive Soils from Israel Society of Soil Mechanics and Foundation Engineering. In document for First Meeting of the ISSMFE Technical Committee on Expansive soils (TC6), 28-29 Nov 1987, New Delhi, India, p10-15.

Discussion

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An apparatus for running direct shear tests under controlled suctions up to 150 kgf/cm^2 was developed by the author (Escario 1980) and several sets of test results have been presented and discussed (Escario and Sáez, 1986, 1987a, 1987b, Escario, 1988, Escario and Juca 1989).

As a first prototype, it presents some drawbacks that I have tried to overcome with the new model under construction, schematically presented in figure 1.

The shear box is again square of 50 x 50 mm. The air pressure chamber is cylindrical and the overall dimensions of the device are such, that it can be fitted on the normal types of commercial shear testing machines by replacing the standard shear box with the new cell.

The vertical load, up to 6 kgf/cm^2, is applied with air pressure in the upper chamber, through a Bellofram membrane. Several cells may therefore be held separatelly consolidating and reaching equilibrium suction conditions, for as many days as deemed necessary. They are then moved to the testing machine to be sheared. With the existing prototype model, consolidation and equilibrium had to be accomplished with the cell installed on the testing machine and, therefore, the output of the equipment was very small.

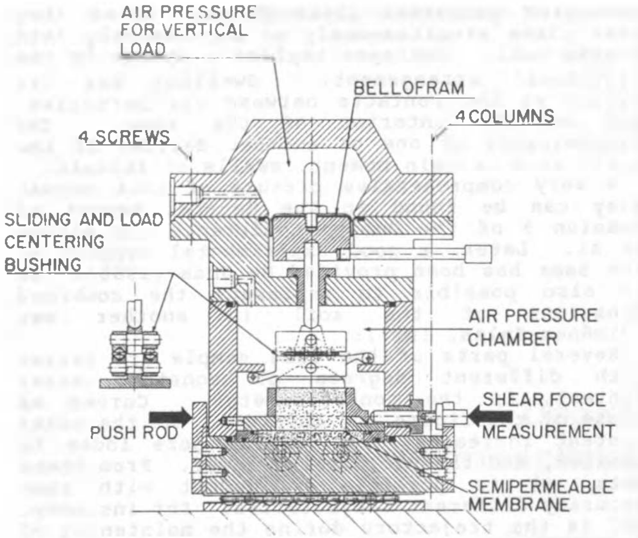

Figure 1.- New model of direct shear test device under controlled suction.

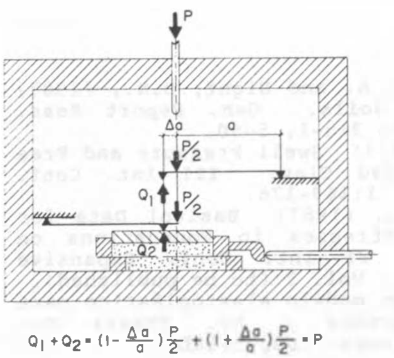

Figure 2.- Scketch of the working principle of the bushing for automatically centering the load.

A special bushing is provided that keeps the vertical load continuously at the center of the upper shear frame for any displacement. The working principle is shown schematically in figure 2: the vertical load P is split in two equal parts, each one acting on a horizontal lever, with an auxiliary sliding support at each side of the chamber.

This special bushing may be substituted by a normal one, as in the previous design, if it is considered unnecessary.

The device shown is now in a very avanced stage of construction and, therefore, its performance and structural strength have still to be checked.

References

Escario, V. (1980). "Suction controlled penetration and shear test". Proc.4th.Conf.Expansive soils, II, pp. 781-797. Denver: American Sociefy of civil Engineers.

Escario, V. & Juca, J.F.T. (1989). "Strength and deformation of partly saturated soils". 12th ICSMFE, Rio de Janeiro.

Discussion

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The movements of structures founded on swelling clays are usually smaller than those of the structures on deep layers of compressible soil. Nevertheless, the disorders are, in many cases, greater.

This is due to the fact that swelling is in a high degree a differential phenomenon. Due, in turn, to the nature of the perturbation induced by the creation of the superimposed structure, but mainly by the heterogeneity of the ground, of its intrinsic expansivity and of its state and conditions. Let us recall the typical landscape of the gilgai.

So, for arriving at a reasonable model for the behavior of the soil, it would be necessary to use geostatistic, as has been applied to collapsible soils by Ali, Nowatzki and Myes (1989) in their paper presented at this same session.

But it is unrealistic to think that we are going to perform enough number of swelling pressure determinations, for instance, to be able to apply these techniques, and it is necessary to admit that in actual practice, at least in my country, swelling pressure tests are mainly an index property. Even the swelling pressure is used, in fact, as an index, as the number of test performed at each site is usually not enough to give to the obtained values a real statistical significance.

Between the index tests that are routinely performed in Spain, I wanted to call your attention on the Lambe test, developed by Prof. T.W. Lambe at the MIT on request of the FHA in 1960.

The universal use of this test has been small. But in Spain where swelling clays are one of the most important geotechnical problems, it has been in use since its publication. Today it is an standard test extensively used and the general consensus is that it provides a very reliable indication of swelling danger.

It is to be noted that a somewhat peculiar characteristic of the clays of extensive areas in Spain is that they are basically illitic, with a small proportion of smectites, if any. But their seasonal movements are important. It has been extensively proved that the Lambe test is able to detect where there is danger, when other indices, such as the Atterberg Limits, fail.

The Lambe Standard Test gives three options for the initial moisture content of the sample. In practice we use only the one in equilibrium with an atmosphere at 50% of relative humidity (pF=6 approximately). The corresponding swelling pressure will be referred as SPL (Swelling Pressure Lambe).

With a data base of 161 samples of different sites in Spain, mainly in the South, it has been possible to raise the following correlations with swelling pressure.

(SPL in kilopascals):

$$SPL = -18.34 + 3.27 \cdot Wl \quad (R=0.60)$$

$$SPL = -3.20 + 18.44 \cdot Wi \quad (R=0.79)$$

$$SPL = -23.93 + 15.32 \cdot Wi^{0.9} \cdot \gamma_d/\gamma_w \quad (R=0.77)$$

$$SPL = 29.85 + 555.53 Wi/Wl \quad (R=0.52)$$

$$SPL = 8.80 + 6.38 \cdot Wp \quad (R=0.34)$$

We can see that there is a better correlation with the initial moisture content (Wi), and also with a composite variable of moisture content and the initial density (γ_d) of the sample, tamped according with the Lambe Standard.

According to the Lambe recommendations, real danger exists when SPL is greater than 157 kPa, with the above included correlations, the corresponding discriminating values would be:

$$Wl = 53.5 \qquad Wi = 8.7 \qquad Wi^{0.9} * \gamma_d/\gamma_w = 11.8$$

If we classify the samples of the said data base according with these criteria, and we compare the results with those of the Lambe test, we obtain the following percentages:

With the liquid limit, 70% of samples correctly classified, 9% mistakes on the safe side (non dangerous samples classified as dangerous), and 21% on the unsafe side. With the Initial Water Content, 80%, 8% and 12%, respectively; with the composite index, 84%, 7% and 9%.

From all this we advance the following conclusions:

i) Actual practice relies, in great proportion, on indices other than on parameters.
ii) The Lambe Test gives a valuable and significant indication of the swelling danger.
iii) Its Initial Water Content is an interesting predictor of the results of the test.
iv) It is in fact a crude determination of the Hygroscopic Moisture at pF=6
v) It is reasonable to expect that the Hygroscopic Moisture is a significative index about the properties of the soils into the unsaturated range.

As a possible answer to the request of the Discussion Leader about similitudes and differences between swelling and collapsible soils, the writer is of the opinion that, instead of speaking of soils, we should speak of phenomena. Swelling and collapse are uncoupled processes although very often they take place simultaneously or successively into a same soil. Collapse implies a change in the structural arrangement. Swelling has its origin at the contacts between the particles, and at the interior of the same. The predominance of one or another decides if the soil, at a certain moment, swells or shrinks.

A very comprehensive picture of this mutual play can be found in the General Report of Session 5 of the Dublin Conference, by Alonso et al. Later, a good experimental support of the same has been provided by Josa (1988). It is also possible to represent the combined behaviour of the soil in another way (Jiménez-Salas, 1987):

Several parts of the same sample are tested with different degrees of constant water content, in the consolidometer. Curves as those of the figure are obtained. As the water content increases, the initial pore index is greater, and the yield point lower. From these data, it is possible to deduct with some accuracy different trajectories. For instance, ABC is the trajectory during the moistening of a dry sample at constant volume. It can be seen that the Brackley (1973) effect (initial high swelling pressure and subsequent decrease of the same) is predicted.

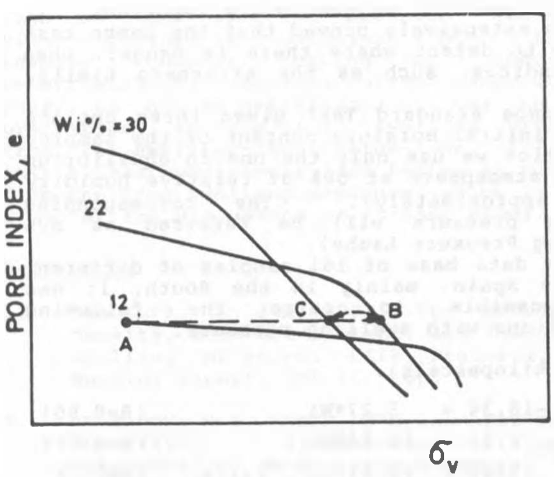

Figure 1 Illustration of the yield point as a function of water content

REFERENCES

Alonso, E.E., Gens, A. and Hight, D.N., (1987) Special Problem Soils. Gen. Report Sess. 5-IX ESCMFE-Dublin 3:5-1, 5-60.

Brackley, I.J., (1973) Swell Pressure and Free Swell in Compacted Clay. III Int. Conf. Expansive Soils. 1:169-176.

Jiménez-Salas, J.A., (1987) Basical Data for Calculation of Stresses in Foundations on Swelling Clays. VI Int. Conf. Expansive Soils. New Dehli. Vol. 2 (to be published).

Josa, A., (1988) Un modelo elastoplástico para suelos no saturados. Dr. Thesis Un. Politécnica Catalunya. (Unpublish).