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SWELLING OF COMPACTION SHALE

by

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A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES
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THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled "SWELLING OF COMPACTION SHALE" submitted by BHAGAWATISWARA IYER BALASUBRAMONIAN in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

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ABSTRACT

This dissertation deals with the swelling characteristics of compaction shales in relation to the chemical composition of the water that causes swelling. One dimensional swelling tests are conducted on undisturbed samples to study the magnitude and the rate of development of swelling pressure, the coefficients of primary and secondary swelling and the pressure swell relationship.

The existing literature on the swelling characteristics of soils has been reviewed. From previous studies on pure clay minerals, the mechanism of swelling has been attributed to both mechanical and physico-chemical effects. In nature, the soil is present as a mixture of various types of soil minerals and therefore a general mechanism of swelling of any soil should include both of the above mentioned factors.

The various factors that control the swelling characteristics of soils have been clearly defined by past research. By testing undisturbed soil samples from the same block, the compositional factors and other initial conditions of the soil can be maintained constant and the influence of the composition of the water that causes swelling can be isolated. Swelling tests on identical samples were conducted in distilled water and their natural pore water in this study. The samples used in this study had high initial salt content.

The natural pore water of the samples was extracted by a high pressure mechanical squeezer which was found more suitable for the compaction shales than other existing methods of pore fluid extraction.

The swelling pressure of all the samples tested did not develop instantaneously on contact with an external source of water. This is due to the compliance of the measuring system or the presence of a compressible pore fluid. The usefulness of the observation of the rate of development of swelling pressure of the soils tested has been investigated.

It was quite surprising to find that the swelling pressure of two of the shales tested (which were homogeneous clayey shales) gave equal magnitudes of swelling pressure irrespective of the composition of the ambient water. When allowed to swell, the sample in distilled water swelled more than the sample in its natural pore water. A study of the composition of the pore water of the soil sample at the end of the constant volume swelling pressure test and after a certain amount of swell had been permitted has helped in explaining this behaviour.

The swelling characteristics of the shales studied here have been explained in terms of the true effective stress, σ^* defined as

$$\sigma^* = \sigma - u - (R - A)$$

where (R-A) is the physico-chemical component, and it may be estimated by means of the double-layer repulsive force equation.

The swelling test results on one of the shales have been used to predict the swelling in the field due to a spillway excavation. The prediction of the increase in water content due to the excavation was found to be quite good at depths unaffected by weathering. At shallow depths due to weathering the water content had increased much more than that predicted from swelling data on non-weathered shale.

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CHAPTER I

INTRODUCTION

1.1 General

Swelling soils create many problems in connection with spillways, shallow foundations, underground pipes, stability of slopes, etc. Attempts in the past have been made to develop methods of identifying and classifying swelling soils, to explain the mechanism of swelling and to understand the factors that control or affect the swelling behaviour of soils. These studies were based mostly on tests on compacted soils and on a few undisturbed soils. Generally these laboratory studies have not been successful in predicting field behaviour. In this thesis a study has been undertaken in order to obtain insight into the behaviour of natural soils in their undisturbed state.

Among the soils in Western Canada that have been recognised as highly swelling are many of the compaction shales (for example, Morgenstern, 1967) from Alberta, Saskatchewan and Manitoba. All of these shales are essentially fully saturated compared to the problem soils reported from Australia, India, Israel and South Africa, which are partly saturated. In this study, tests were conducted on undisturbed samples of Bearpaw shale from the South Saskatchewan River Project, Saskatchewan and Morden shale from the Morden dam site, Manitoba.

Most research in the past has been confined to the magnitude of swelling pressure or the ultimate swell of soils. Since few studies have been concerned with the rate of swelling, special attention was given to this aspect of the problem.

The mechanism of swelling has been explained in the past by soil scientists and soil engineers as due to mechanical and physico-chemical effects. This study has been directed to explain the limits of applicability of existing concepts to natural soils. The swelling characteristics of the soils studied here have been explained on the basis of the true effective stress, σ^*, which is equal to σ-u-(R-A). The limitation of the generality of this approach to all soils and the difficulty in the accurate evaluation of the (R-A) force have also been discussed. It is shown here that σ^* for a soil in equilibrium at a water content and a particular fabric is a unique value irrespective of the chemical environment.

Finally, a field case of swelling has been analysed using laboratory test data and the limitation of these tests is assessed by comparing the observed values with the predicted ones.

1.2 Scope of the investigation

Past research has clearly brought out the various factors that control the swelling behaviour of soils. Of these, the chemical composition of the water that enters the soil in relation to the natural pore water in the soil, has been studied in this programme. The samples were allowed to swell in distilled water and its natural pore water. A comparison of two initially "identical" (undisturbed) samples would isolate the influence of the chemical environment on the swelling properties of the natural soils.

The composition of the natural pore water was obtained by extracting the pore water by means of a high pressure mechanical squeezer and analysing it.

The swelling pressure at constant volume, the rate of development of swelling pressure with time, the coefficients of primary and secondary swelling and the pressure-swell relationship were determined from these tests.

Tests have been conducted only in a one-dimensional consolidometer with double drainage.

The results obtained on Morden shale were applied to the prediction of heave and the increase of water content of a spillway cut at the Morden dam, Manitoba. The prediction of water content was quite good at depths not influenced by weathering. The predicted heave was too high compared to the observed one.

The swelling pressure of the soil samples, even if they are fully saturated does not develop instantaneously on contact with external water. This has been interpreted as possibly due to compliance of a compressible pore fluid.

The fabric of the compaction shales tested has been found to conform to the cluster concept introduced by Olsen (1960).

1.3 Organisation of the thesis

Chapter 2 contains the survey of the available literature concerned with the problem of the swelling characteristics of soils.

Chapter 3 gives a brief description of the soils tested.

Chapter 4 describes the pore water analysis. This chapter includes a description of the various methods available for pore water extraction, the choice of the high pressure mechanical squeezer for this study and a brief account and results of the chemical analysis of the pore water of the soils used in the swelling tests.

Chapter 5 describes the apparatus, the main testing programme and lists the various tests conducted on each soil sample.

Chapter 6 gives the interpretation for the time dependent development of swelling pressure for the fully saturated shales tested in this programme.

Chapter 7 deals with the results of the swelling tests and their discussion.

Chapter 8 introduces the concept of the true effective stress and its application to the swelling behaviour of the soils tested.

Chapter 9 describes a practical case of swelling of a spillway cut at the Morden dam and the comparison of the actual field measurements with the predicted ones based on laboratory test results.

Chapter 10 includes conclusions drawn from this study and suggestions for future research.

Appendix A describes the double layer repulsion equation.

Appendix B deals with the working principles of the atomic absorption spectrophotometer.

Appendix C describes the procedures for the chemical tests described in the thesis.

CHAPTER II

PREVIOUS STUDIES ON SWELLING OF SOILS

2.1 General

The variety and the wide distribution of the problems in swelling soils mentioned in Chapter I is evident from the amount of published literature on this topic. There are papers on identification and classification of the problem soils, on the factors that affect or control the swelling properties of soils, on solving an immediate practical problem and on the fundamental studies, etc. To attempt to review all of them would be impractical. Hence an attempt has been made here to discuss only those papers providing essential background for the present study.

Some of the terms used in these papers are given first to facilitate further discussion.

2.2 Definitions

Swelling: If a soil sample, when brought in contact with an external source of water, increases in volume by drawing in water, that phenomenon is called swelling. This is a time-dependent process.

Immediate rebound: Any volume increase due to a decrease of stress without entry of external water may be termed immediate rebound. This is purely an undrained phenomenon and is not generally time-dependent.

Total Rebound: The total rebound is the sum of the immediate rebound and swelling. Once the soil sample is taken out of the ground, the immediate rebound takes place instantly, and hence only the swelling process can be studied in the laboratory. Since the swelling process is a volume increase associated with an increase in water content, the samples to be used for swelling tests should be undisturbed, care being taken not to change their water content during sampling, transportation and storage.

The Swelling Pressure: When a soil sample is allowed to come in contact with water, the pressure that is to be applied to it to maintain constant volume is defined as the swelling pressure. This definition is quite vague and hence has led to different methods of obtaining the swelling pressure in the laboratory.

The Swelling Potential: The swelling potential of a soil is a measure of its ability to swell. Many workers have used different techniques to define the potential of a soil to swell, particularly for identification purposes as will be discussed in Section 2.3.

The Percent Swell: This is the increase in the volume of the sample expressed as a percentage of its initial volume.

Free Swell: If the sample is allowed to swell at "no load", it is usually referred to as a free swell. But during the laboratory free swell test, the weight of the top porous stone, loading block, etc., would contribute a small load of the order of 0.01 tsf. The magnitude of free swell is also expressed as a percentage of the initial volume of the sample.

There have been at least two quite different ways of defining the initial conditions of the soil. The soil scientists have usually adopted the air dry state of the soil as its initial state. In engineering usage, the condition of the soil just before swelling due to, say, unloading is referred to as its initial state. This would be characterized by using undisturbed samples from the field. For studying the swelling characteristics of man made embankments or fills, soils compacted to field specifications have also been used.

The Coefficient of swell: This is a term similar to the coefficient of consolidation for the rate of primary swelling.

2.3 Identification and Classification of swelling soils

Not all soils create problem due to swelling when brought into contact with an external source of water. The problem soils have been recognised as clayey soils containing mainly the minerals of the montmorillonite group. A rigorous identification of the presence of these objectionable clay minerals requires facilities for making microscopic examinations, x-ray diffraction studies, differential thermal analysis, etc., (Lambe, 1952; Lambe and Martin, 1953; Brown, 1961, etc.). Since these facilities are not commonly available in all soil mechanics laboratories, attempts have been made by many workers to identify these soils using simple routine tests.

Holtz and Gibbs (1956) have defined the swelling potential of the soil as the total volume change of a soil from its air dry state to saturated conditions at 1 psi surcharge load. This they have correlated with the colloid content ($\% < 1\ \mu$ size), the plasticity index and the shrinkage limit. They have recommended that these three factors be used in a combined manner with logic and judgement.

Altmeyer (1956) has criticized the use of percent colloids if it is not known whether the colloids are of montmorillonite, illite or kaolinite groups. The maximum moisture content reached by a soil on soaking being the field moisture equivalent (FME) and the minimum moisture content on drying being the shrinkage limit, Altmeyer feels that the volume change of a soil between these two limits of water content would be more representative of its swelling and shrinkage characteristics than the plasticity index as recommended by Holtz and Gibbs (op. cit.). Together with the above, an actual expansion test under typical load conditions is sought by him to get the actual volume change.

Seed, Woodward and Lundgren (1962) have criticized the use of the shrinkage limit to predict the swelling potential as the shrinkage limit values of various soils fall in a very small range and an error in its value of ±3% is quite common due to procedural factors. They quite rightly felt that the combined use of colloid content, plasticity index and shrinkage limit to predict the swelling behaviour of soils would only result in confusion. These workers have defined the swelling potential of a soil as the percent swell under 1 psi surcharge load of a sample compacted at optimum moisture content to maximum density in the standard AASHO test. They have succeeded in arriving at a relationship between the percent clay sizes and the activity of the soil with its swelling potential from a number of tests on a wide variety of compacted artificial materials. Based on these tests, the authors have presented a plot of activity versus percent $< 2\ \mu$ fraction showing the percent swell lines of 1.5, 5 and 25 as a basis for the classification of the soils having low, medium high and very high degrees of expansion. A comparison of natural soils reported by Holtz and Gibbs (1956) in this plot has shown a very good fit.

Li (1963) has reported that the use of specific surface of the soils together with its cation exchange capacity and exchangeable cations (especially sodium) might be used to define the activity of the clay minerals instead of the percent $< 2\ \mu$ fraction.

The method suggested by Seed, Woodward and Lundgren (1962) has been extended by Ranganatham and Satyanarayana (1965) who have recommended the use of % clay sizes and swell activity [defined as Swell activity = $\frac{\Delta(\text{liquid limit} - \text{shrinkage limit})}{\Delta(\%\ \text{clay sizes})}$] to define the swelling potential of a soil. They recommended the use of shrinkage index over plasticity index since the liquid limit is the upper limit of water content of a swelling soil and the shrinkage limit is the water content at its minimum volume attained by drying, and their difference should indicate the volume change characteristics of a soil.

From tests on ten compacted soils, Ladd and Lambe (1961) have developed a PVC rating system (potential volume change) which depends upon the % heave of a compacted soil with a surcharge load of 200 psf, plasticity index, water content at 100% humidity and the percent volume change resulting from drying a soil from its field moisture equivalent to its shrinkage limit.

Seven of the ten soils described in the paper by Ladd and Lambe (op. cit.) have been considered to check the validity of the classification systems noted above. Table II.1 summarizes this study. It can be seen that the method of Holtz and Gibbs (1956) puts the margin between the various categories of swelling at much lower % free swell values than the other three methods. The comparison between the other three methods seems to be quite good even though none of them was able to identify the Siburua shale as a very critical one.

A major limitation of all these methods is that they have all considered the effect of amount and the type of clay (indirectly), but have consistently neglected the effect of the composition of the water present in it. The above mentioned classification methods are based on the consistency limits, the % free swell, etc., all of which are controlled to some degree by the pore fluid composition. Hence, a better classification may be one that incorporates effects of the pore fluid as well.

Using one of the above methods or developing a newer one does not solve an engineer's problem completely. A soil may have high swelling potential, but how much it swells is of importance to engineers. This will depend on the environmental factors.

2.4 The factors that control the swelling behaviour of soils

The swelling behaviour of a soil is fully defined if its swelling pressure, rate of swell and the pressure-swell relationship are known. The factors that control these may be grouped as follows.

1. Compositional factors:
    - (a) The type of clay;
    - (b) The amount of clay;

(c) The initial pore water composition.

2. Environmental factors:
   (a) The initial water content;
   (b) The initial density;
   (c) The initial degree of saturation;
   (d) The initial soil structure;
   (e) The stress history;
   (f) The availability and composition of the water that enters the soil.
   (g) Temperature.
3. Procedural factors:
   (a) Size and shape of specimen;
   (b) Amount of disturbance to the sample;
   (c) The measurement of swelling pressure;
   (d) The measurement of magnitude of swell.

2.4.1 Compositional Factors

The Swelling potential of a soil is governed by the compositional factors which have already been described in Section 2.3. These are considered again below and the influence of the composition of the initial pore water which was not given much attention by previous workers is elaborated.

(a) The type of clay

In general, montmorillonites, vermiculites, etc., are the problem clay minerals. These minerals with negative charge deficiency and well defined cleavage planes are known to show high affinity towards water. These water molecules are attracted towards the basal planes resulting in an increased interparticle spacing, dependent upon the surface area available for swelling.

(b) The amount of clay

It is obvious that increasing amounts of those clay minerals mentioned in 2.4.1(a) would add to the swelling potential of the soil.

(c) The initial pore water composition

The initial pore water salinity is a factor which has not been given the attention it deserves. The fact that the chemistry of the system affects the swelling behaviour of soils has led many people to vary the concentration of the external water entering the sample. A better understanding of what actually happens is possible only if the concentration of the pore water and its relation to the external water is known.

Some attempts have been made to vary the initial pore water salinity of the water used for compaction (Ladd, 1959), but the effect of the salinity of the water on the structure of the compacted soil was not studied in detail. Ladd concludes that the presence of salts decreases the amount of swelling.

2.4.2 Environmental Factors

(a) The initial water content

As per the methods adopted by the soil scientists, if a sample is air dried and then allowed to swell by imbibing water, the swelling pressure and the amount of swell would decrease as the initial water content of the sample increases.

Most of the papers published in the engineering literature refer to tests conducted on compacted material. Here care must be taken before making any conclusions, since changing the water content at compaction alters the dry density, degree of saturation and the soil structure. Hence tests conducted on compacted soils do not permit isolating the effects of water content or the other factors mentioned above on its swelling behaviour. These tests give the combined effect of all these factors.

From their studies on compacted soils, Holtz and Gibbs (1956) and Seed and Chan (1959) conclude that increasing water content decreases the swelling pressure and the amount of swell. Holtz and Gibbs (op. cit.) also describe a different kind of test. They compacted two samples under identical conditions. One of them was allowed to swell on contact with water whereas the other was maintained at constant volume by applying an external restraining pressure for some time. When allowed to swell, the latter specimen swelled less than the sample which was allowed to swell from the as compacted state without any restraint. All the factors except the initial water content and degree of saturation have remained the same, because during the period of restraint the compacted soil with an initial negative pore water pressure is free to draw in water from the external source increasing its water content and degree of saturation. This would lead one to the general conclusion that increasing water content would decrease the amount of swell.

Even with a saturated soil which would swell on contact with water, as swelling takes place, the water content would increase and it is obvious that with swell, the swelling pressure and the (remaining) amount of swell would decrease.

(b) The initial density

Holtz and Gibbs (1956) and Seed and Chan (1959) conclude from their studies that the lower the initial dry density the lower would be the amount of swell and the swelling pressure. This again is quite obvious since a saturated soil with decreasing dry density would have an increased particle spacing which results in decreasing the swelling pressure and the amount of swell, all other things being equal.

(c) The initial degree of saturation

This factor cannot be isolated, because it is directly related to the dry density and the water content. A variation of the degree of saturation keeping the dry density constant is possible only by varying the water content. Similarly, keeping the water content constant a variation of the degree of saturation can be effected only by varying the dry density.

The influence of the effect of the degree of saturation at constant dry density has already been discussed under Section 2.4.2(a). Holtz and Gibbs (1956) conducted swelling tests from the as compacted state and after soaking the sample at constant volume for the some time. The former case gave a higher magnitude of swell than the latter, but the difference gradually decreased with increasing degree of saturation.

(d) The initial structure

This factor has been studied by a number of workers. The effect of initial structure on the swelling characteristics has been reported by Barber (1956); Seed and Chan (1959); Seed, Mitchell and Chan (1962).

Two soil samples compacted at the same moisture content and the same dry density using static and kneading compaction were tested for swelling characteristics. The percent swell of both these samples dry of optimum was almost the same whereas for the sample compacted wet of optimum the one compacted statically swelled more. It is known from the works of Lambe (1958a), Seed and Chan (1959) that kneading compaction tends to create a more oriented structure and static compaction a random structure. However, such structural variations generally can only be obtained at water contents greater than the optimum and the samples compacted dry of optimum, irrespective of the type of compaction used, tends to give a more random structure.

Seed and Chan (op. cit.) have reported swelling tests on two compacted soils, one compacted dry of optimum and the other compacted wet of optimum. Both of these samples were brought to the same water content on soaking at constant volume. When allowed to swell, the samples initially compacted dry of optimum with a more random structure and a random distribution of particles gave higher swelling pressures. All of the tests mentioned above tend to give higher values of swelling pressures and greater magnitudes of swell for a soil with a random structure when compared to the same soil with a more oriented structure.

Parcher and Lin (1965) have reported results of the amount of swell in the vertical and horizontal directions of samples compacted in a mould 1.4" diameter and 2.8" high using kneading and static compaction. Their results show that the static compaction gives more swelling in both directions which agrees with the findings of the earlier workers. But when they tested samples compacted using either of these methods in the vertical and horizontal directions, they found that the horizontal strain always exceeded the vertical strain irrespective of the moulding moisture content. At moisture contents less than the optimum, the resulting random structure should have given more or less the same magnitude of strain in both directions and at moisture contents greater than optimum, where the particles tend to get oriented in a direction perpendicular to the direction of compaction, the vertical swell should have been more than the horizontal swell. The greater unit swelling in the horizontal direction has been explained by Parcher and Lin as probably due to large differences in the absorption characteristics or bonding forces of edge surfaces compared to the basal surfaces of the particles. But Rogatkina (1967) has conducted tests on samples in directions parallel to and perpendicular to the plane of orientation of an artificially prepared soil with oriented structure. These tests show that the magnitude of swelling in a direction perpendicular to the clay surface to be more than that in the direction of the plane of orientation. If this is the case, the author feels that the results of Parcher and Lin could only be explained as due to the influence of the side of the mould in which the samples have been compacted and the author believes that if the soil is compacted in larger moulds and the swelling sample is prepared out of the central portion and tested in different directions, the results would conform to the structure of the sample due to compaction.

(e) The stress history

The influence of stress history has been studied by Seed, Mitchell and Chan (1962). These authors have tested two identically compacted specimens. The first one was allowed to swell at 1 psi surcharge load. The second one was kept at a higher load for some time and then allowed to swell against 1 psi. The latter sample swelled less than the former one. This effect could as well be due to a change in the degree of saturation as was the case with the tests of Holtz and Gibbs (1956).

(f) The availability and composition of the water that enters the soil.

This factor has been studied by Ladd (1959); Seed, Mitchell and Chan (1962) and others. In general, increasing the concentration or valency of the cation of the entering solution has been found to cause a decrease in the amount of swell. Ladd has reported that the rate of swelling was not affected in tests on compacted clays allowed to swell freely.

Limited availability of water becomes an important issue only for the field case. In the laboratory tests water is abundant and swell maximized accordingly.

(g) Temperature

The influence of temperature on the swelling characteristics has been studied by Yong, Taylor and Warkentin (1962). They report that increasing temperature results in an increase in swelling pressure and the magnitude of swell.

Woodward, Clyde and Associates (1967) report that the temperature may be a prominant factor in the field. The influence of an increase in temperature by 1°C may be approximated as a head of 3' of water which the author considers to be much too conservative.

2.4.3 Procedural Factors

This Section is presented in four subsections. Various workers have chosen to use different dimensions for the samples used for swelling tests. The amount of disturbance which the sample had been subjected to is another factor. These two are described in the first two subsections. Even though a large amount of literature has been published on swelling characteristics of various soils, there seems to have been little, if any, uniformity of method of testing. This seems especially true in the method of determination of the swelling pressure of soils. This and the methods adopted for measuring the magnitude of swell are explained in the last two subsections.

(a) Size and shape of specimen

The influence of the size of the specimen has been reported by Palit (1953), Salas and Serratosa (1957); Woodward, Clyde and Associates (1967). Palit found that the swelling pressure of the sample increased with its thickness whereas Salas and Serratosa observed constant magnitude of swelling pressure for all thicknesses of soil samples. Salas and Serratosa have controlled the amount of swelling of samples of different thicknesses during the swelling pressure measurement, whereas Palit has used a proving ring fixed to a cross-head for all sample thicknesses. The deflection of the proving ring due to the swelling pressure from the sample results in equivalent amounts of swell in the sample. The percent swell in each sample due to the above reason would vary inversely as the thickness of the soil sample. It is known that with increasing swell the swelling pressure* would decrease, and this accounts for the observation of Palit of increasing swelling pressure with thickness.

Palit has also found that the percent swell decreased with increasing thickness of the sample. This, the author believes, would have resulted due to the frictional resistance offered by the sides of the sample ring.

Woodward, Clyde and Associates (1967) have tested two samples compacted in 2" and 4" diameter moulds and the percent swell in the 4" diameter sample was found to be smaller than the one in the 2" mould. Due to compaction the soil particles tend to orient themselves in a direction perpendicular to the compactive effort. The facility with which this orientation takes place would depend upon the diameter of the mould, the orientation becoming better in larger diameter moulds due to less restraint. Thus for the same compactive energy, the 2" sample would have a more random structure than the 4" sample resulting in a greater amount of swell.

Seed, Mitchell and Chan (1962) report that the sample faces must be perfectly plane, otherwise erroneously low swelling pressures would be measured.

(b) <u>Amount of disturbance to the sample</u>

Different aspects of this factor have been studied by various workers. Peterson and Peters (1963) have tested samples from the field

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*The swelling pressure is that pressure corresponding to zero volume change of the sample. With any amount of swell allowed, the pressure that is measured is only an equilibrium pressure corresponding to that much swell.

from the weathered and unweathered zones of the Bearpaw shale. They found that the magnitude of swell of the weathered samples to be much smaller than that of the unweathered samples. This is quite obvious since the weathered samples have already experienced substantial swelling in the field.

The amount of swell of an undisturbed sample of soil has been reported by Parcher and Lin (1965) to be very much smaller than the same soil compacted.

The finding of Holtz and Gibbs (1956); Barber (1956); Dawson (1956); Seed, Mitchell and Chan (1962) that the swelling pressure drops very much if a small volume increase is allowed illustrates the importance of taking samples with minimum disturbance from the field and taking care not to allow any change in water content during sampling, transportation and storage. Hvorslev (1949) stated:

> "--- a moderate increase in water content is not as serious as a corresponding decrease, since the test results (permeability, consolidation and shear strength) will be on the safe side for practical applications, and since the sample can be restored to its original condition by allowing it to consolidate under stresses approximating those in the undisturbed ground ---"

This statement is valid if only the above mentioned properties are of interest. The determination of the swelling pressure is very sensitive to even small increases in water contents. Hence very great care must be experienced while preparing samples for swelling tests.

At this stage, it is worth mentioning that samples with no change in water content cannot be prepared from varved clays or clays with silt seams especially from great depths. Usually migration of water takes place from the silt region to the clay zone, resulting in unavoidable disturbance to the sample. Testing these materials with minimum delay after removal from the ground is advised.

(c) Methods of determining the swelling pressure of samples in the laboratory

The methods for the determination of the swelling pressure of a sample of soil in the laboratory may be grouped as follows:

(i) The free swell test

In this method, the sample of soil is brought in contact with water and is allowed to swell at "no load" and then the soil is gradually consolidated back to its original volume by adding loads. In this procedure, the swelling pressure is defined as that stress which would consolidate the sample back to its original volume. (Dyregrov and Hardy, 1962; Hardy, 1965).

(ii) The constant volume test

In this method, the sample volume is maintained constant throughout the test by varying the load on the sample as required. In this method, the swelling pressure is defined as the final equilibrium stress on the sample.

Maintenance of constant volume may be achieved in different ways. Palit (1953) and Alpan (1957) have used a proving ring to measure the swelling pressure of the soil. The measurement of swelling pressure using a proving ring necessitates some deformation of the ring resulting in an equal amount of swelling in the soil and it has been mentioned earlier that with swell the measured swelling pressure decreases. Seed, Mitchell and Chan (1962) have shown this variation by measuring the swelling pressure using proving rings of different stiffnesses. Using very stiff proving rings or load cells in conjunction with rigid porous stones may be the optimum approach, but it should not be overlooked that the accuracy of the measurement of small loads is lost if stiff load measuring devices are used. This leads to the null-type measuring system wherein a small (measurable) volume change is allowed and is then balanced by adjusting the load on the sample, thus maintaining constant volume.

Komornik and Zeitlen (1965, 1970) explain a method of measuring lateral swelling pressures. This consists of a ring of stainless steel with three electrical wire conductors stretched around the outside of the membrane. This again works on the principles of the proving ring and thus has the disadvantage mentioned above. But the ones used by these authors seem to be quite stiff with a lateral movement of only $\pm 3 \times 10^{-4}$ cm for stresses up to 4.00 tsf. A null-type of measuring technique for lateral pressures has been described by Brooker (1964).

(iii) Loaded swell test

In this method a number of identical samples are subjected to different loads and brought in contact with an external source of water. The resulting volume changes are then plotted against the corresponding stresses and the stress corresponding to zero volume change is then taken as the swelling pressure. (Skempton, 1961; Gizienski and Lee, 1965; Noble, 1966; Matyas, 1969).

(iv) Measurement of pore water suction

Skempton (1961) and Matyas (1969) recommend the use of the initial pore water suction which, for saturated soils, would be equal to their swelling pressure. These authors have used triaxial tests for this purpose. The consolidometer developed by the Anteus Corporation (Lowe et al, 1964) permits the measurement of suction and also the measurement of one-dimensional swell on release of pressure.

(v) Comparison of the various methods of measuring the swelling pressure

Fig. 2.1 shows the stress paths followed by a sample during the determination of swelling pressure.

The free swell test method is shown in Fig. 2.1(a). The initial condition of the point is given by * . The sample is brought in contact with water and as water enters the sample, the initial suction of the sample and hence its effective stress decrease and the sample swells to point 1. After swelling is complete, the soil is subjected to loads and is gradually consolidated back to its original volume. The effective stress increases during the process and is shown by the curve 1-2. The pressure corresponding to the original volume (hence the void ratio) is taken as the swelling pressure. In the Fig. 2.1(a), the curve A represents the test conducted on a sample in its own pore water. A sample of soil allowed to swell from an effective stress to a lower value and then reconsolidated back to the initial effective stress comes to equilibrium at a void ratio generally lower than the initial value. This yields an average compression index over the stress range slightly larger than the average swell index. Hence, the measured swelling pressure is generally less than the original effective stress. The difference between the measured and actual swelling pressures increases with the difference between the two void ratios, before swelling and after recompression. Curve B represents the same test conducted in distilled water. Any dilution of the pore water of the sample, initially having a high salt content, will result in an amount of swell larger than the swell in its own pore water, and a reconsolidation of this sample may give a swelling pressure larger than the actual swelling pressure. Hardy (1965) mentions that the free swell test may yield smaller or higher values of swelling pressure compared to the initial effective stress.

The loaded swell test has been favoured by many workers in comparison to the other two due to the simplicity in conducting the test and in applying calibration corrections to it. Palit (1953); Ladd and Lambe (1961); Seed et al (1962) and Baker and Kassiff (1968) report that the sample does not develop the swelling pressure immediately on inundation. Later, it is shown that the present work on saturated samples also supports this. This has been argued as due to the presence of a compressible pore fluid (Chapter VI). If this is true, a sample subjected to, say, half the swelling pressure at the instant of inundation, is sure to consolidate initially and later swell and reach a final constant volume. This final volume change is plotted against the applied pressure and the stress corresponding to no net volume change is read as the swelling pressure of the soil. In the e-log σ' plot, the stress path followed by a sample during the loaded swell test is shown in Fig. 2.1(b), and since the average compression index is always greater than the average swelling index, this swelling pressure would always be less than the initial effective stress. Skempton (1961) reports the results on one soil sample giving a swelling pressure from this test to be less than initial suction of the soil directly measured.

Comparison of the constant volume swelling pressure method and the loaded swell test can be clearly seen from the results of Komornik and Zeitlen (1970). They have measured swelling pressures of compacted soil samples in the vertical and horizontal directions in a specially designed consolidometer. The lateral pressure has been measured by using a very rigid stainless steel ring and the vertical pressure by means of a loaded swell test. They have reported higher lateral swelling pressures when lateral movement is restrained. Swelling pressure at constant volume of a saturated unconfined sample, by definition, would be equal in magnitude to the initial pore water suction. This, being a hydrostatic force, would act in all directions with the same magnitude. In the case of partly saturated soils, the swelling pressure would be some function of the pore air and pore water pressure which are both independent of direction. Hence, one would expect the same magnitude of swelling pressure for any soil sample in all directions. (The magnitude of swell may differ in different directions because this is governed by the structure of the soil which may be anisotropic). Komornik and Zeitlen, as explained earlier, have used two different techniques for the measurement of the vertical and lateral swelling pressures. For reasons explained in the previous paragraph, the loaded swell test would always yield values less than (or at the most equal to) the swelling pressure at constant volume, and this is consistent with the observations of Komornik and Zeitlen.

In the constant volume swelling test, load on the sample is gradually varied as the sample tries to swell so as to maintain constant volume throughout the test. The void ratio and the average effective stress within the sample are thus maintained constant. In the e-log σ' plot, the sample remains at the same point throughout the test.

The direct measurement of suctions of very high magnitudes is not possible due to difficulties associated with cavitation in the measuring system. Resort is usually made to the axis translation technique (for eg. Olson and Langfelder, 1965). An external pressure is applied to the sample and the pore water pressure is measured. From these two known pressures the initial suction can be estimated. In this test also, no change in the void ratio or the effective stress is permitted and the sample remains at the same point throughout the test in the e-log σ' diagram. (Fig. 2.1(c)).

Direct measurement of suction has been strongly favoured by Matyas (1969) over other procedures. The sophisticated equipment needed for this test, has hindered its wide acceptance in many laboratories. It should not be forgotten that the suction in the pore water would be equal in magnitude to the swelling pressure only for a saturated soil with the pore pressure parameter $B = 1$. For partly saturated soils, the pore water suction would be more than the constant volume swelling pressure and this difference increases with decreasing degree of saturation (Pufahl, 1970).

The constant volume method has been adopted in this laboratory. Even though Matyas (1969) claims that the accuracy of measurement using a lever or hydraulically loaded oedometer is less than the direct suction measurement, the author found these instruments quite sensitive enough. Both the lever and hydraulically loaded consolidometers have been used in this laboratory. The details of the machines and experiments would be given in Chapter V.

(d) The measurement of the magnitude of swell

The magnitude of swell of a sample has been measured either by allowing the sample to swell freely on contact with water with no load on it or after restraining the swelling partly or fully for some time and then allowing the sample to swell freely. It has already been mentioned that partial or full restraint decreases the measured total swell compared to free swelling for compacted soils. This effect has been found to decrease with increasing degree of saturation. (Holtz and Gibbs, 1956). Therefore, for a fully saturated soil this effect may be neglected. For a partly saturated soil, a choice of testing could be made based on whether it is the change of water content or the change of volume that one is after.

2.5 The mechanism of swelling

A saturated soil may change its volume due to any of the following three processes (Bailey, 1965): particle diminution,particle deformation or particle reorientation.

Particle diminution could probably be an important factor during compression of soil at high pressures.

Particle deformation consists of elastic and plastic strains at contact points as well as flexure of platy minerals such as mica. Particle deformation of most sand particles is inconsequential. On the other hand, particle deformation of flaky minerals could be substantial.

Particle reorientation is probably a major factor in the volume change of particulate systems at pressures of interest to engineers. In general, this includes rotation and translation of flocculated particles resulting in a change of fabric as well as a displacement of parallel particles to a new spacing, where the fabric remains essentially constant.

Terzaghi (1929) attributed the swelling of soils to particle deformation alone. This explanation was quite satisfactory for coarse grained materials and for fine grained materials which are chemically inactive. But in those clays with high surface areas, this did not fit well. The swelling of such soils was found to be much more than anticipated from particle deformation considerations alone and to be dependent on the chemistry of the water. Realisation of the importance of the system chemistry led many to explain the swelling mechanism as solely dependent on physico-chemical considerations (Salas and Serratosa, 1953; Bolt and Miller, 1955; Bolt, 1956; Warkentin, Bolt and Miller, 1957; Warkentin, 1958: Lambe, 1958b; Ladd, 1959; Aylmore and Quirk, 1959; Aylmore and Quirk: 1960; Blackmore and Miller, 1961; Ruiz, 1962; Warkentin and Schofield, 1962; Yong and Warkentin, 1966). In these considerations the swelling pressure has been explained as a net repulsive force resulting from double-layer expansion.

Olson and Mesri (1970) have conducted tests on different sedimented soils to study the influence of the property of the external water on their compressibility characteristics. They conclude (as did Bailey, 1965) that even though both mechanical and physico-chemical effects influence the compressibility of clays, only one or the other usually dominates for a particular soil. They found that the kaolinites are controlled by the mechanical effects whereas smectite are physico-chemically controlled and illites occupy an intermediate position. This is an important contribution in the sense that unlike the earlier workers, these authors have tried to search for both mechanical and physico-chemical effects. Their findings on pure clays could be extended to the natural soils to recognize that due to the presence of a wide range of types of minerals within a soil, one would expect them to exhibit both mechanical and physico-chemical behaviour.

Lambe (1958b) has described the force systems in (fully saturated) clays in terms of:

$$\bar{\sigma} = \sigma - u = I + (R-A)$$

where $\bar{\sigma}$ = effective stress

σ = total stress

u = pore water pressure

$(R-A)$ = sum of all repulsive and attractive long range electrical forces between the soil particles

and I = steric interaction or contact pressure

In a later publication, Lambe (1960) (also in Bailey, 1965), from considerations of vertical static equilibrium over a unit area of soil-water-air arrived at the force equation given as follows:

$$\sigma = \bar{\sigma}\, a_m + u_a\, a_a + u_w a_w + (R-A)$$

where a_m, a_w, a_a = ratio of area of mineral-mineral contact, water-mineral or water-water contact and air-mineral or air-air contact to the unit area

and $\bar{\sigma}$, u_a, u_w = average stress on the portion of the unit area a_m, a_a or a_w

For a saturated soil with negligible a_m, this equation reduces to

$$\sigma = \bar{\sigma}\; a_m + u_w + (R-A)$$

This equation has been investigated by Bailey (1965) in relation to volume change characteristics of remoulded clays and physico-chemical environments. The equation has been rewritten here as

$$\sigma = \sigma' + u$$
$$= \sigma^* + u + (R-A)$$

where σ' = the apparent effective stress

$= \bar{\sigma}$ of Lambe

σ^* = the true effective stress

It has been shown in Chapter VIII that the variation of σ^* with water content for the swelling soils studied here is a unique curve irrespective of the chemical environment. It has also been demonstrated in that chapter that a change in (R-A) without a change in σ^* does not cause a volume change, if the fabric of the soil remains unchanged. Thus σ^* being the stress that affects the volume change behaviour of soils (resulting from a particle deformation and a reorientation without a change of fabric) it was thought appropriate to call it the "true" effective stress. The limits of generality of applying σ^* to study the volume change characteristics of all soils and the difficulties involved in assessing the (R-A) with precision have been discussed in Chapter VIII.

The present work relies on the basic logic put forth by Bailey (1965) that both mechanical and physico-chemical factors are to be considered in explaining the swelling behaviour of soils and not as isolated factors. Taking a purely mechanical approach does not explain why the properties of montmorillonitic clays depend on the chemistry of the external water. The physico-chemical approach which has been quite widely used also fails in explaining why a soil with a random structure should swell more than a soil with a more oriented structure as reported by Seed and Chan (1959); Seed, Mitchell and Chan (1962); Komornik and Zeitlen (1965, 1970). A purely physico-chemical approach would predict a larger amount of swell and swelling pressure for an oriented structure compared to a random structure.

2.6 Prediction of behaviour of the soil in the field

With the aid of the available procedures for classifying expansive soils, it is possible to identify the soils that possess undesirable expansion characteristics; but, these methods do not provide a means for predicting actual amounts of heave and damage that might occur in the field. Therefore the individual soil samples have been tested under estimated field conditions and loads. (Gizienski and Lee, 1965). They tested soil samples compacted to the field dry density at the field moisture content. The relationship between percent swell and pressure was obtained by subjecting these samples to different surcharge loads and measuring the final volume change. These results when compared with a small scale test revealed that the swell has been overestimated by about three times. The difference has been explained as due to the difference of soil structure and pore water concentrations between the field and laboratory soils.

Jennings and Knight (1957) have developed a method of estimating total heave based on two consolidation tests, one done on a sample at its natural moisture content and the other on an undisturbed sample initially saturated and allowed to swell and then subjected to consolidation. This method relies on the existence of a unique e-log p' curve. The irreversible nature of volume change characteristics of soils makes one question this method which relies on the recompression curve of a freely swelled soil to predict the swell due to decrease in effective stress in that material.

McDowell (1956, 1959) recommends the prediction of the percent swell from a family of curves (for different percent free swells) of percent swell against pressure. Noble (1966) proposes the use of a family of curves (for different water contents) of percent swell against pressure. In both these cases, it goes without saying that these curves are obtained from tests on compacted soils and how well these would explain the field swell is to be seen.

Peterson (1958) shows the laboratory swell curve and the field swell curve of a Bearpaw shale sample. The field swell was very much more than the laboratory rebound curve. The discrepancy has been attributed to the secondary part of the swelling in the field as compared with the small time tests in the laboratory.

The above mentioned papers comparing the predicted and actual field swell, have all been conducted on disturbed materials. [Peterson (op. cit.) has used undisturbed materials for testing. But, once he has consolidated them to very high pressures before testing for the swelling characteristics, the soil structure, density, moisture content, initial pore water salinity, etc., have been so much altered that these could also have been considered only as a disturbed material]. These failures in extending laboratory studies to a better understanding of field behaviour clearly illustrates the importance of the study of undisturbed samples of soil.

2.7 Rate of swell

There is very little published work on the rate of heaving of laboratory samples. DeWet (1957) has developed an analysis similar to Terzaghi's theory of consolidation. Blight (1965) used this approach to predict the rate of swelling of a soil in the field from the known values of coefficient of swelling obtained from the laboratory tests on undisturbed samples, the profile of excess pore pressure due to superimposed load and drainage conditions. The agreement found was very good.

2.8 Development of the present problem for research

A careful study of the published literature reveals the following facts:

(a) An accurate estimation of the swelling behaviour of natural soil in the field is impossible from laboratory tests unless the tests have been made on undisturbed materials.

(b) Of the various factors that control the swelling behaviour, the initial pore water salinity and its relation to the external water needs more study.

(c) The mechanism of swelling of natural soils has to be explained based both on mechanical and physico-chemical effects.

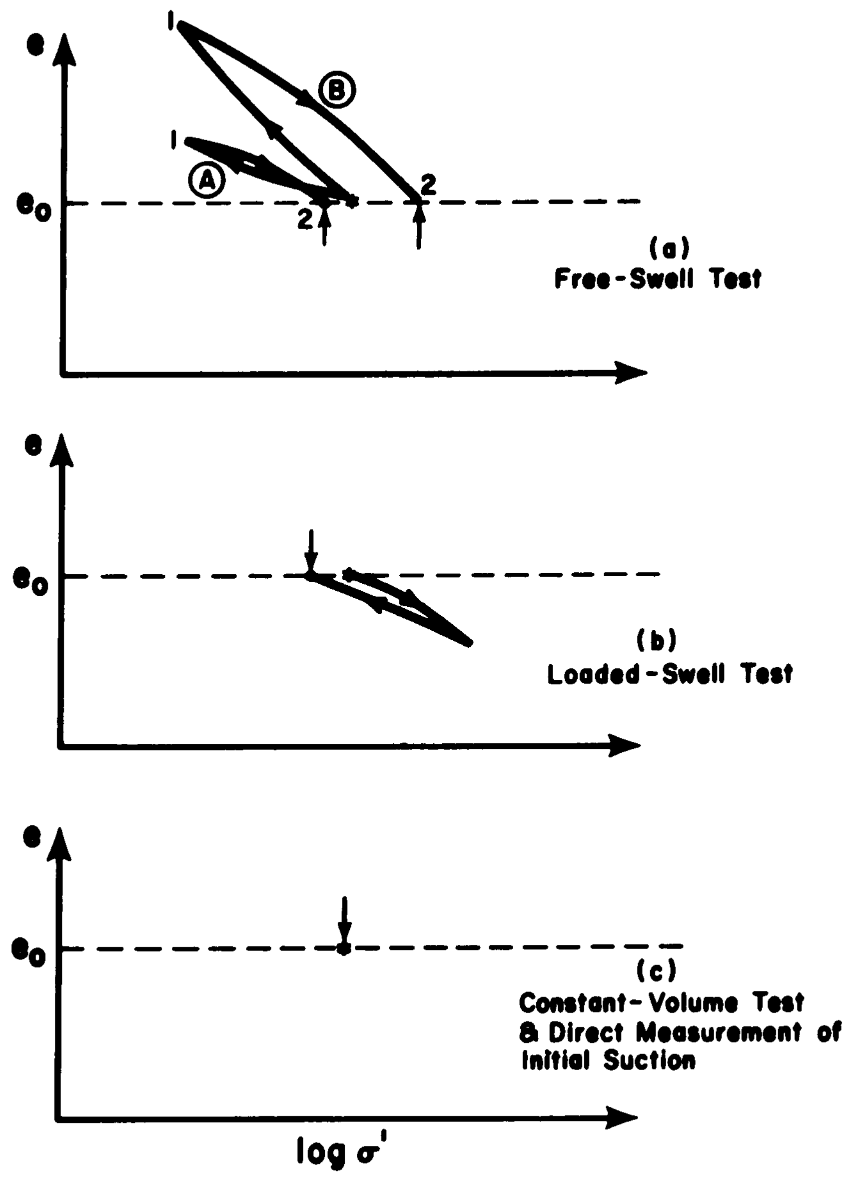
Recognition of these has led to the present study of the swelling characteristics of natural materials in their undisturbed state in both distilled water and their natural pore water.

TABLE II.1

COMPARISON OF VARIOUS METHODS OF IDENTIFICATION AND CLASSIFICATION OF SWELLING SOILS

Sample No	Name	Ladd & Lambe	Holtz & Gibbs	Seed et al	Ranganatham et al	Free* Swell
1	Iredell clay	very critical	very high	high	very high	95
2	Vicksburg Buckshot	very critical	very high	high	high	75
3	Texas black clay	critical	very high	high	-	75
4	Siburua Shale	critical	very high	high	high	135
5	Keyport soil	marginal	medium	low	low	43
6	Boston blue clay	non-critical	medium	low	low	15
7	Guelph sandy clay	non-critical	very high	low	low	35

$$\text{*Free swell} = \frac{\text{final volume - Initial dry volume}}{\text{Initial dry volume}} \times 100\%$$



* Initial eff. stress and Actual swelling pressure

↓ or ↑ Swelling pressure measured according to the particular method

FIG. 2-1 DIFFERENT METHODS OF DETERMINING SWELLING PRESSURE

CHAPTER III

DESCRIPTION OF SOILS

3.1 General

The materials that were used in this testing programme were the Bearpaw shale, the Morden shale and an Edmonton shale. This chapter includes a general description of these samples, their index properties and the results of some of the preliminary tests done on them.

3.2 The Bearpaw Shale

The samples of Bearpaw shale used in the tests were obtained from the South Saskatchewan dam site through the Prairie Farm Rehabilitation Agency (PFRA). The geology of this formation is described by Peterson (1954, 1958). In general, the Bearpaw shale is a marine deposit of upper cretaceous age. These shales are very uniform, medium grey in colour and have the appearance of a dense clay. This material is highly plastic. The whole formation may be generally divided into three parts or zones. The upper zone is the weathered zone. This is very soft, having many joints and some slickensides. Gypsum crystals are usually found in this region. The lower most zone is the hard zone. These are rather uniform hard clays having no slickensides. Lenses of sand and silt are encountered erratically in the hard shale zone. The transition layer in between is called the intermediate zone.

The testing programme was started on a block sample of Bearpaw shale (SSRP.TP.4103, West Wall, Sample #4,from a depth of 5.5 - 6.8 ft.). This sample had numerous slickensides and gypsum crystals in it. No sand or silt lenses were present. Preparation of samples from this block was very difficult because of the close net work of slickensides. This sample will be referred to further in this thesis as Bearpaw shale - block sample.

The second lot of Bearpaw shale samples consisted of 6" diameter driven samples over the depths of about 16-23 ft. in the weathered formation (RD 2300) and from depths of 68-72 ft. in the intact hard shale formation (RD 2299) . These samples were also obtained from the South Saskatchewan River dam and were supplied by the PFRA. The top formation was completely weathered and broken. The deeper samples were hard and uniform with lenses of sand present throughout the sample. These samples, for further reference, will be called Bearpaw shale - weathered and intact respectively.

3.3. Morden Shale

The Morden shale is a formation very similar to the Bearpaw shale and the Pierre shale of the Northern U.S.A. (Peterson, 1954). This shale is also a marine deposit of upper cretaceous age.

The Morden shale samples were received from beneath the spillway slab of the Morden dam. The samples used for the tests described later were taken from 3" diameter tube samples (AH 101) collected from the left bank of the spillway cut. These samples were also supplied by the PFRA. The Morden shale from this area was extremely uniform and very plastic. It was easy to prepare samples from them.

3.4 Edmonton Shale

For comparison purposes, a fresh water deposit of similar age was also used. The grey clay shale used in the swelling tests is from the Edmonton formation. The geology of this formation has been explained in detail by Ower (1960). The most notable feature of this member is the large number of coal seams. This formation consists, apart from the coal seams, of bentonitic terrestrial sediments. The shales are dull in colour, mainly brown or grey.

The samples used in the swelling tests were obtained from the Devon slide area using a pitcher sampler. The formation at the place of study consists mainly of brown and grey clay shales at the top with a carbonaceous clay shale below separated by a thin bentonite layer. A detailed log of the soil profile may be seen in Eigenbrod (1972).

The samples were quite plastic: they were very uniform and easily workable.

3.5 Sample Characteristics

Classification tests were conducted on all three shales (ASTM, 1971). The consistency limit tests were conducted on air dry samples, ground and allowed to pass through a No. 40 sieve. Appropriate quantities of distilled water have been added to these samples and they were allowed to equilibrate for about a day before being used for the Atterberg limit tests. Limited availability of these samples did not permit a detailed study on the variation of the liquid limit with aging and method of preparation.

The cation exchange capacity of these samples was determined by the Ammonium Acetate leaching method and the specific surface was found employing the ethylene glycol monoethyl ether (EGME) technique. (See Appendix C for a brief description of these two tests.) The mineralogical composition of the shales was determined using x-ray diffraction techniques (on glycolated samples). The <2μ portion of the total soil was separated by hydrometer analysis. The slides required for the x-ray analysis were prepared by air drying the suspension from the hydrometer test on glass plates. The results of these tests and the procedure employed by the Alberta Research Council (who did the interpretation of the x-ray results) are also included in Appendix C.

The results of these tests are presented in Table III.1. The presence of montmorillonite in the Morden shale might possibly have been missed due to the presence of mixed layer (illite - montmorillonite) minerals, as indicated by the x-ray patterns of this material given in Appendix C.

The Edmonton shale and the intact Bearpaw shale have similar % <2μ , cation exchange capacity and specific surface. This indicates that probably their mineralogical composition are similar. A low liquid limit value for the Edmonton shale has resulted from the presence of $CaCO_3$ in the soil (Chapter VII). The presence of such a cementing agent has affected its swelling characteristics also as discussed later.

TABLE III.1

SOIL CHARACTERISTICS

Soil	Representative values of											
	Liquid Limit	Plastic Limit	Plasticity index	Natural water content %	Degree of satu. %	Specific gravity of soil particles	%<2μ	(4) Cation Exchange capacity meq/100 gm of soil	(4)# Weighted Sp. sfce of total soil m^2/gm	Mineralogical Composition %M*	%I*	%K*
Bearpaw Block sample	103.6	29,8	73.8	30.3-31.8	88-100	2.83	48	65.3	266	45	45	10
Bearpaw weathered	61.8	25.7	36.1	~29	92-98	2.73	44	30.	203	40	50	10
Bearpaw intact	74.6	29.2	45.4	23.9-25.1	99-100	2.84	28	53.4	130	25	60	15
Morden shale	67.0	29.0	38.0	18.8-21	94-98	2.69	45	68	153	0^+	70	30
Edmonton shale	43.5	22.8	20.7	21.9	~100	2.65	28	52.6	151	-	-	-

* M - Montmorillonite
I - Illite
K - Kaolinite
(4) average of 4 samples
\# weighted sp. sfce. of total soil = sp. sfce. of <2μ x proportion of <2μ fraction in total soil
\+ the presence of montmorillonite might have been missed due to the presence of mixed layer minerals

CHAPTER IV

PORE WATER ANALYSIS

4.1 Introduction

The study of the chemistry of pore water has been of interest for some time to soil agronomists, petroleum engineers, sedimentologists, etc. Only recently have the soil engineers been making use of techniques developed in these areas in order to get a better understanding of the influence of the chemical environment on the engineering properties of soils. A number of methods have been developed for the extraction of pore fluid from soil. These methods are mentioned briefly in the following pages. The method that is used in this study is explained in some detail. Some of the observations made and the results of the chemical analysis done on the pore water from the Bearpaw, Morden and Edmonton shales are also included in this Chapter.

4.2 Different methods of extraction

(a) Dilution method or saturation extract

This method requires the accurate determination of the original moisture content of the sample and is based on the assumption that all salts of the pore water are precipitated during drying and redissolved when distilled water is added to the dry sample. The diluted sample is mixed thoroughly with distilled water, allowed to sit for a while (usually overnight) and then is filtered or the water extracted by applying a suction (Swarzenski, 1959 and Soil Science Manual #421).

(b) Leaching

The soil is dried and then mixed with a known quantity of distilled water. The suspension is heated to boiling and allowed to stand overnight. It is then washed a number of times with hot distilled water. The filtrates and washings are combined and analysed (Emery and Rittenberg, 1952).

(c) Centrifuging

The samples are centrifuged in closed plastic tubes for a certain time until a sufficient quantity of water is extracted for chemical analysis (Swarzenski, op. cit.; Rittenberg, 1963; Powers, 1957).

(d) Immiscible liquid displacement

Interstitial water is flushed out with a water immiscible liquid such as Paraplex G-60, G-61 and G-62 (available from Rohm and Hass Company, Washington Square, Philadelphia, Pa., U.S.A.). Extraction is made with a filter press by forcing an immiscible liquid through the piston by either piston or gas pressure (Scholl, 1963).

(e) Gas extraction method

Gas of relatively high humidity can be made to pass through water saturated specimens and expel interstitial water. Gas expulsion of the interstitial water from a sediment sample is carried out in a filter press fitted with a gas inlet valve. Gas is admitted under pressure from a reservoir tank or is drawn by a vacuum through the sediment; the extruded water drains through a discharge tube at the base of the press into a vessel covered to prevent evaporation (Scholl, op. cit.).

(f) Low pressure mechanical squeezing

Interstitial water from recent sediments can be squeezed with a screw jack operated by hand. The water is collected at the bottom and stored in air-tight containers (Siever, 1962). In place of the screw jack, low pressure gas has also been used to obtain water from recent sediments (Hartmann, 1965).

(g) High pressure squeezers

The working principle of this method is the same as for low pressure mechanical squeezing. Here the pressure applied to the piston is as large as needed to extract pore water from compacted sediments. The water that is squeezed is collected at the base of the apparatus directly into a syringe which is transferred to sealed bottles that are stored in a cool place until the specimens are tested (Manheim, 1966).

4.3 Comparative study of the various methods of pore water extraction

The immiscible liquid displacement method or the gas extraction method are useful only for sandy or silty soils and the low pressure mechanical squeezing is good only for relatively soft sediments such as recent ocean deposits. The centrifuging method is reported to be quite satisfactory with sands and silts (Swarzenski, 1959; Powers, 1957) and has been found successful also with finer materials (Rittenberg et al, 1963). But since most of the laboratory centrifuges cannot separate the clay and the fluid efficiently Siever (1962) doubts the applicability of this method for clayey deposits.

Leaching and saturation extract are very similar in their approach. The water content of the soil is increased by adding distilled water to such a limit that the water could be extracted by filtering or by application of a small suction. In these methods, the concentration of the pore water at its natural moisture content is calculated from the measured concentration of the extracted water and the corresponding diluted water content. However, there is a serious limitation to these approaches.

By varying the quantity of distilled water added, a number of soil samples were prepared at different moisture contents from the same soil type. Water was extracted from each of these samples using a small suction and these waters were analysed. Fig. 4.1 shows the variation of the cations (expressed as moles/litre) with the inverse of the moisture content, after Khan (1967). From this Figure it may be seen that the concentration of the extracted water is not directly proportional to the inverse of the moisture content over the range studied. This makes it impossible to calculate the salinity of the pore water at its natural moisture content (especially if it is low) from the known salinity at the diluted water content. Thus the estimation of the salinity at the natural moisture content is impossible using this method, if the water cannot be extracted by a small suction without dilution.

In the case of the high pressure mechanical squeezer criticisms regarding the dilution of the expressed water due to the entry of double layer water and due to the membrane effect of highly consolidated clayey sediments have been raised by Siever, Beck and Berner (1965); Engelhardt and Gaida (1963); Chilinger, Ricke and Sawabini (1967).

Clay minerals have a net negative charge on their surface which is compensated by exchangeable cations. The cation concentration gradually decreases with distance from the surface of the clay reaching a concentration equal to that of the external free solution at large distances. The anion concentration at the surface of the clay is zero and it gradually increases to that of the external solution again at large distances. As this clay is compressed, the free pore water will be removed first and the electrolyte-poor double layer water is left behind*. Any further squeezing would result in a gradual decrease in salt content of the squeezed water with decreasing particle spacing, due to the entry of the double layer water.

The solution squeezed out from the clay samples originally flows through larger pore channels, and with increasing pressure as the particles get closer, the flow channels become narrower. At very small particle spacings, due to the overlapping of the double layers containing high concentration of cations, the passage of anions and therefore salt, is hindered. Thus clays, especially those with high cation exchange capacities, act as a membrane preventing flow of salts through them at low void ratios.

To study these two effects, Engelhardt and Gaida (op, cit.) tested pure montmorillonite and kaolinite samples mixed with various salt waters. These samples were subjected to consolidation tests and

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*In reality, it should be remembered that there is no sharp demarcation between free water and double layer water and there exists no means of physically separating these two components.

the water that was squeezed out during consolidation was collected and analysed. No change of concentration of water was found with kaolinite samples. But with pressure definite changes of concentration was reported with montmorillonite. The concentration of the pore water squeezed out decreased with increasing pressure. This the writers attribute to the expulsion of electrolyte-free double layer water out of the sample and to the ion filtering ability of electrically charged clay particle surfaces. However, the filtering effect has been reported to be negligible by others even over a large pressure range for the soils they tested. (Manheim, 1966; Manheim and Bischoff, 1969.) Lutz and Kemper (1959) show the dependence of the concentration of the squeezed solution on the squeezing pressure (and hence the particle spacing) provided the soils is saturated with monovalent ions. If calcium is the predominant ion, the concentration remains constant over a large range of pressure in the case of Wyoming Bentonite.

The experience cited above leads to the following conclusion. There are soils with high cation exchange capacities which are satisfied with mainly monovalent ions exhibiting pressure dependency on the concentration of the water squeezed out. The only way to determine whether the pore water of the particular soil under study could be extracted by high pressure squeezing technique or not, is by squeezing the soil at various pressures and analysing the water for salinity. The maximum pressure up to which no change of salinity is found is taken as the safe squeezing pressure and any pressure less than this value may be used for regular pore water extraction purposes for that soil.

4.4 Choice and description of the method of extraction of pore water and chemical tests

The Bearpaw shale and the Morden shale are both highly overconsolidated clayey deposits. From the discussion of the various methods described above, it is quite obvious that the high pressure mechanical squeezer is the only one applicable for these materials.

A pore water squeezer similar to the one described by Manheim (1966) has been built for this study. A drawing of it is shown in Fig. 4.2.

The parts of the apparatus are rinsed at least three to four times with distilled water and then with acetone. (The rubber parts are not to be rinsed with acetone.) Once in a while, the parts in the base (except the rubber ones) were washed with concentrated hydrochloric acid which in turn was removed by a number of washings with distilled water. These parts (especially the disc containing the porous stone) were kept dipped in a large container of distilled water (sometimes slightly heated) for a few hours. Finally, they are rinsed well with alcohol and allowed to dry completely before next use.

Since only representative samples are needed for the pore water analysis, the trimmings of various soil samples, obtained while preparing samples for the swelling tests, were used in this study. The samples were first compacted using loads of about 7,500 psi and once almost all the air is removed out of the sample, the load was brought down to the safe pressure for the sample, and the squeezing of the pore water started.

The pore water that was squeezed out was collected directly into a syringe connected to the base, care being taken to see that no air is entrapped in the syringe. This water was then transferred into tightly corked glass bottles and stored in a refrigerator until a sufficient number of samples have been collected for chemical analysis.

The water was analysed for its pH using a pH meter[*] and total salinity using a solu bridge[**]. The solu bridge measures the conductivity of the water which may be converted into total salts present using standard charts. (See United States Department of Agriculture Handbook No. 60.) The anions are usually determined using titration methods. (Soil Science Manual #421) For the first set of pore water specimens tested, the only measurable anion was sulphate. From then onwards the pore waters were tested only for cations and the salts were assumed to be sulphates of these cations. The concentration of cations was measured using an atomic absorption spectrophotometer. (Soil Science Manual #421) . (See also Appendix B) .

4.5 Description of various tests and their results

The quantity of the pore water that is needed to conduct all the above mentioned tests is about 5 cc. The first of the tests that was conducted was to find whether the concentration of the first 5 to

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[*] Metrohm pH meter model E388 with Fischer Electrode No. 13-639-92.

[**] Beckman RB3R104 solu bridge with glass pipette type conductivity cells with platinum electrodes CEL-G01, CEL-G1 , and CEL-G10, with cell constants 0.1, 1.0 and 10.0 respectively.

6 cc of the pore water squeezed out differs from the concentration of all the water that could be squeezed out at that pressure. Tests were conducted on samples of Bearpaw shale weathered and the grey clay shale sample from Devon. The results are given in Table IV.1.

The variation found from these tests is quite small and is well within the accuracy that could be obtained from the chemical tests and the usual variation between the individual samples used for squeezing tests.

The cation concentration could be determined correct to about 0.1 ppm using the spectrophotometer. But the range of concentration of cations which could be obtained is very small and very low, usually between 1 and 8 ppm for the monovalent ions and 1.25 and 4 for the divalent ones. The concentration of the pore water of the samples tested in this series are so high that they have to be diluted 1,000 to 5,000 times, to bring the concentration of the water within the measurable range of the machine. This dilution brings the accuracy of the estimation of the concentration of natural pore waters down to about 100 to 500 ppm. Further, even if there is any portion of the double layer water also being squeezed out with the free pore water, the first 5 or 6 cc of the water squeezed out would be less contaminated compared to the rest. Therefore, it was concluded that only the first 5 or 6 cc of pore water need be collected and used for chemical analysis made to define the pore water of the soil samples.

The next set of tests conducted was to find the dependency of the concentration of the pore water on the squeezing pressure. The results of tests on the block sample of Bearpaw shale (see Chapter III for description of this sample) are given in Fig. 4.3. The total concentration remained almost constant up to pressures of about 3,000 psi and dropped above that pressure. (No water could be squeezed out at pressures less than about 700 psi.)

The water that is being squeezed out at any pressure will contain partly the free pore water and partly the double layer water. The free pore water may be defined as the water that is not chemically bound to the clay particles whereas the double layer water is bound to the surface of the clay. For this reason, the pore water can flow freely and the double layer water is less mobile. In the high pressure squeezer test, in the presence of the external load, some of the outer layers of the bound water also become mobile. Hence, the water that is being squeezed out will contain mainly or all the free pore water plus a part of the double layer water. The force with which the double layer water is bound to the clay particle surface varies approximately as exponentially with the inverse of the distance. If an external pressure is applied on this clay sample, the portion of the double layer water bound to the clay surface with pressures less than or equal to that magnitude, also become mobile. Thus the amount of water that is being squeezed out from the double layer water will depend upon the external pressure. This mixing would be reflected as a decrease of the total salinity of the extracted water. This explains what is observed in Fig. 4.3 in the pressure range 700 - 3,000 psi. Above 3,000 psi, entry of larger proportion of double layer water might have decreased the total salinity. Salt filtering due to the membrane effect does not explain this situation because at all pressures, the sample was kept only until the first 5 to 6 cc of water has been squeezed and the size of the flow channels, therefore, has remained essentially the same at all pressures.

Fig. 4.4 shows the variation of the percentage of any individual cation in terms of the total against squeezing pressure. This figure shows a very gradual decrease of the percentage of the monovalent ions with increasing pressure. If the distribution of the individual ions in the double layer is considered, one would expect a larger ratio of divalent to monovalent cations on the surface of the clay which gradually decreases to that value of the external solution (free pore water) at very large distances. As more and more of the double layer becomes mixed with the extracted free pore water with increasing pressure, the percent divalent ions of the total would be expected to increase with a corresponding decrease of the monovalent ions, as seen in Fig. 4.4.

Gapon's equation (for eg. Yong and Warkentin, 1966) gives the relationship of the ratio of the concentration of the individual exchangeable cations to the ratio of the same ions of the outside (free) solution. Based on this equation, if only the outside solution is being squeezed out, without affecting the double-layer water, the ratio $(Na + K)_0/\sqrt{(Ca + Mg)_0}$ would remain constant throughout the pressure range. But Fig. 4.5 shows a very slight decrease of this ratio in the pressure range 700 - 3,000 psi and a rapid decrease thereafter.

It has already been mentioned that no water could be squeezed out of the Bearpaw shale samples at pressures less than 700 psi. The swelling pressures of these samples was found to be in the order of 25 psi. Therefore, from the definition of the swelling pressure, any pressure greater than that should cause consolidation expelling the pore water from the soil. Thus, the squeezing pressure could have been much lower than 700 psi. However, this requires the use of undisturbed samples for squeezing tests. The lack of sufficient undisturbed material for squeezing test prohibited the author from using undisturbed samples for these tests, hence representative samples and pressures this high had to be used.

All the above mentioned results lead to the conclusion that there is no absolute separation between the free and double layer water, and at any pressure the water that is being squeezed out is a mixture of the free pore water and the outer layers of the double layer water. But all these curves seem to flatten out at low pressure ranges (3,000 - 700 psi) becoming asymptotic to the "true free pore water". Since no water could be squeezed out at pressures less than 700 psi, the water squeezed out at that pressure is accepted as the free pore water in this study. This, the author agrees, is not the "true pore water", but is the best substitute to it, compared to the water that could be obtained by any other method of pore water extraction. Therefore, as a general rule the minimum pressure at which sufficient water, as required for the chemical analysis, could be squeezed out may be taken as the squeezing pressure. For further studies pressures in the order of

700 psi was used for Bearpaw and Morden shales.

The results of the chemical analysis of the pore water conducted on different samples are given in Table IV.2. The results clearly indicate the paleosalinities of the individual clays. The Bearpaw and Morden shales, which are marine deposits, show very high concentration of salt compared to that in the rather fresh water deposit of the Devon shale.

Another interesting feature that can be seen is the variation of the individual cations between the weathered and the unweathered zones of the Bearpaw shale. The deeper intact Bearpaw shale has a high concentration of Na ions whereas the weathered Bearpaw shale is higher in divalent ions and poorer in Na . The Na rich intact shales showed higher swelling and swelling pressure than the divalent rich weathered shales.

TABLE IV.1

CATION ANALYSIS ON SAMPLES OF PORE WATER COLLECTED AT 6 CC INTERVALS

S No.	PORE WATER	CATIONS IN PPM			
		Na	K	Ca	Mg
	SAMPLE: BEARPAW SHALE: BLOCK				
1	First 6 cc	1650	30	555	950
	Second 6 cc	1650	27	520	1110
	Third 5 cc	1550	23	510	950
2	First 6 cc	1710	33	480	910
	Second 6 cc	1900	28	520	950
	Third 5 cc	1210	25	493	910
	SAMPLE: GREY CLAY SHALE: EDMONTON				
1	First 6 cc	327	18	190	65
	Second 8 cc	300	16	178	55

TABLE IV.2

PORE WATER COMPOSITION

SOIL SAMPLE	pH	CONDUCTIVITY MILLIMHOS/CM	NORMALITY OF SOLUTION	CATION CONCENTRATION PPM*			
				Na	K	Ca	Mg
Bearpaw Block	7	13	0.13	2677	53	274	215
Bearpaw Weathered	-	9.8	0.08	1612	28	513	963
Bearpaw Intact	6.6	11.5	0.115	2960	-	380	122
Morden Shale	-	18.6	0.27	4200	210	430	265
Grey Clay Shale-Devon	-	1.8	0.02	314	17.5	184	55

* Average of not less than 4 samples.

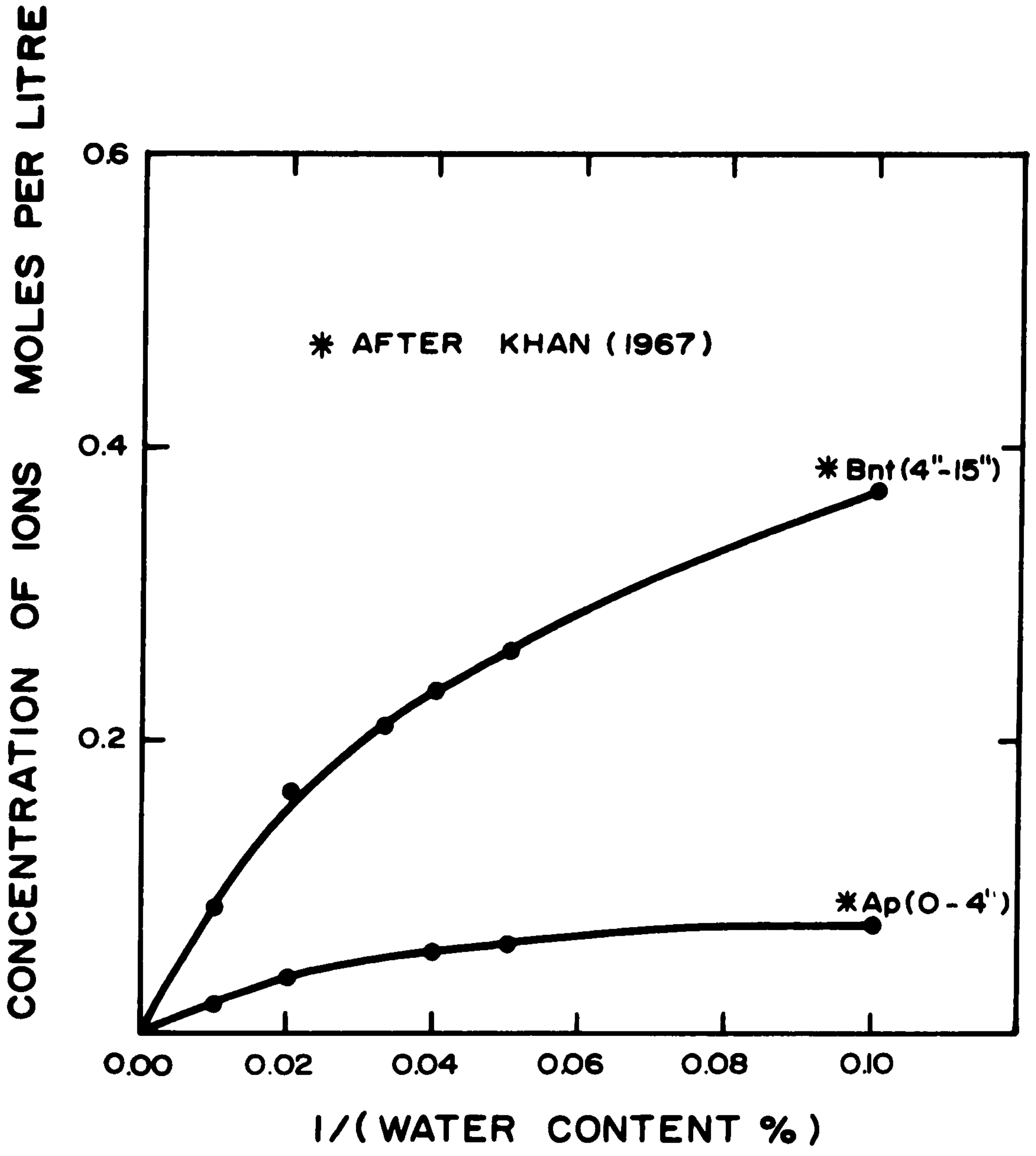
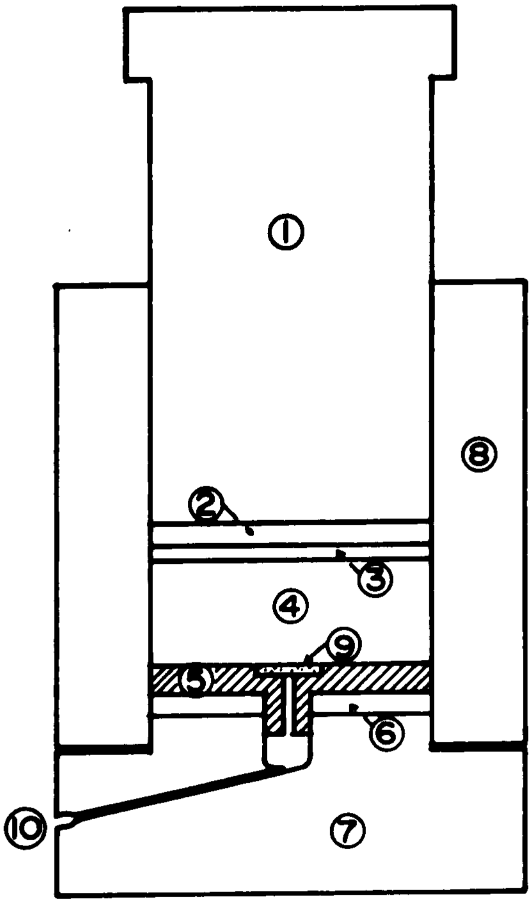


FIG. 4-1 VARIATION OF Na$^+$ ION CONCENTRATION WITH INVERSE OF THE WATER CONTENT



① STEEL PISTON WITH CHROMIUM PLATING
② RUBBER DISC
③ TEFLON DISC
④ SOIL SAMPLE
⑤ STAINLESS STEEL DISC
⑥ RUBBER RING
⑦ BASE - STAINLESS STEEL
⑧ STAINLESS STEEL CYLINDER
⑨ POROUS NICKEL
⑩ PORE WATER OUTLET - CONNECTION FOR SYRINGE

SCALE - HALF SIZE

FIG. 4-2 PORE WATER SQUEEZER

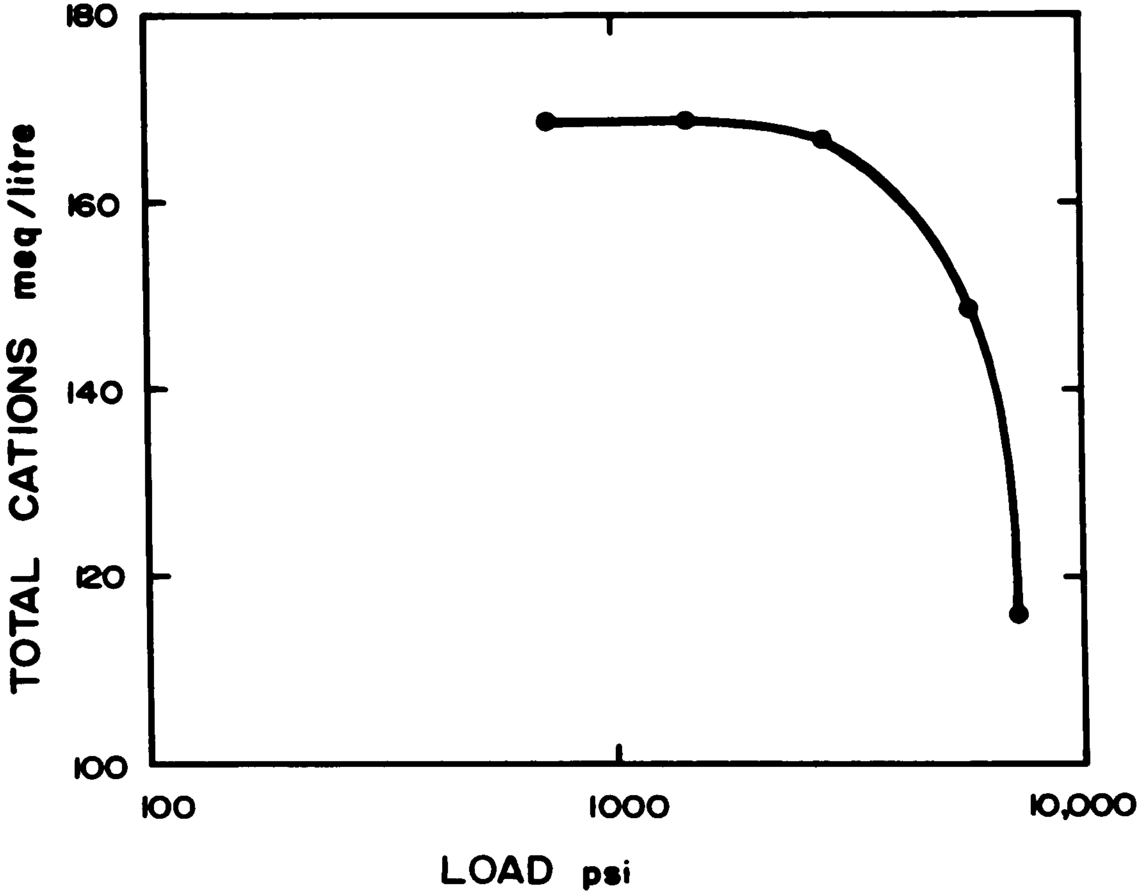


FIG. 4-3 VARIATION OF TOTAL CATIONS WITH SOUEEZING PRESSURE

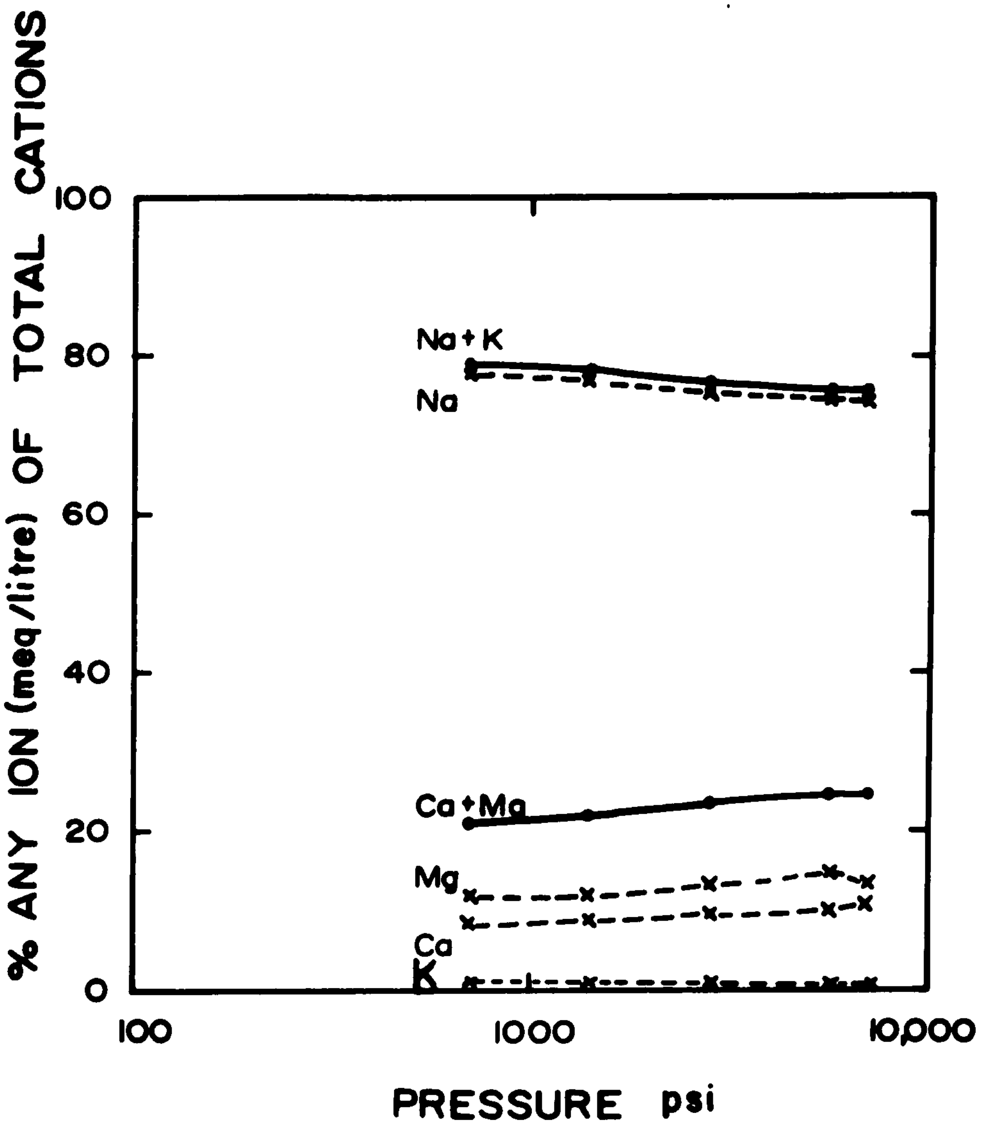


FIG. 4-4 VARIATION OF PERCENTAGE OF ANY ION WITH SQUEEZING PRESSURE

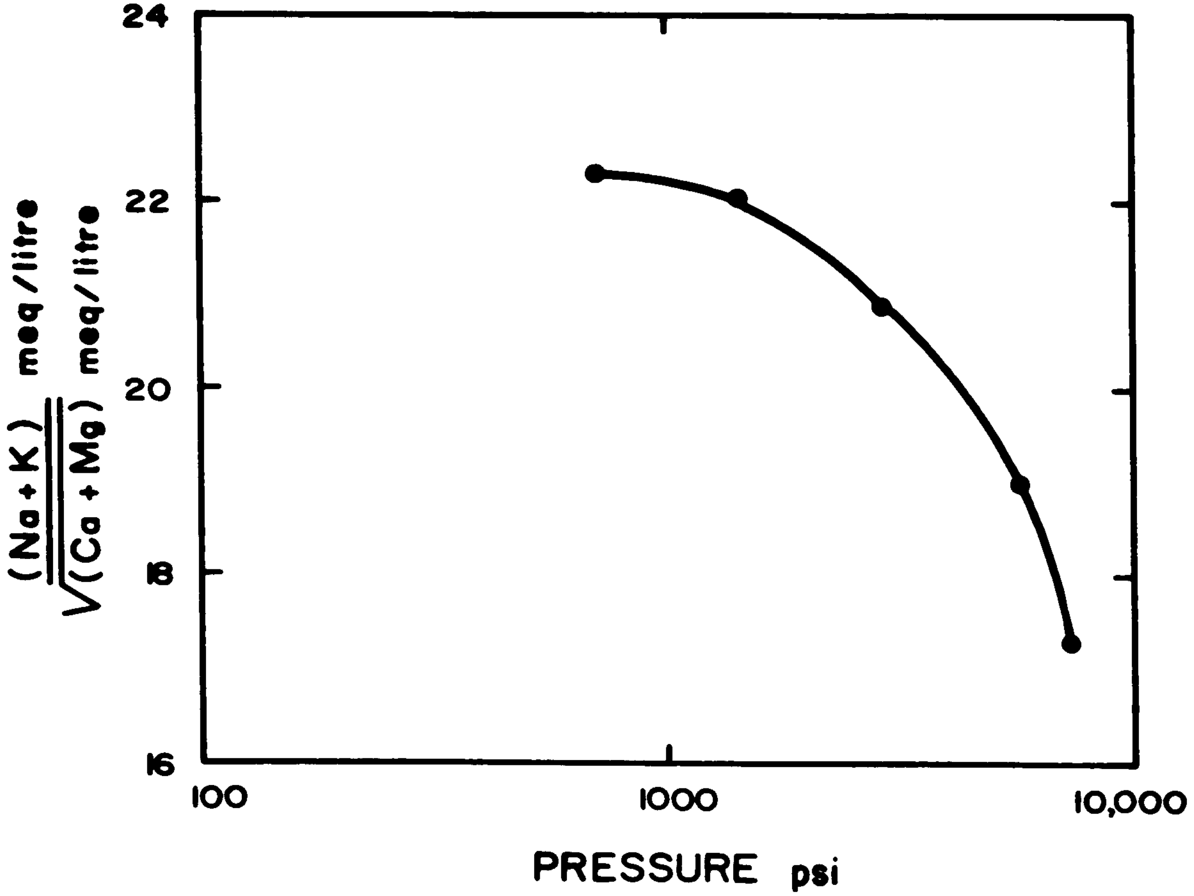


FIG. 4-5 VARIATION OF $\frac{(Na + K)_o}{\sqrt{(Ca + Mg)_o}}$ WITH SQUEEZING PRESSURE

CHAPTER V

LABORATORY TESTS

5.1 General

Only one dimensional swelling characteristics of soils were studied in this work. Tests were conducted in conventional lever loaded consolidometers as well as in a special hydraulically loaded consolidometer. The latter machine - the Anteus Corporation consolidometer is described briefly in Section 5.2. Drainage was allowed into the sample in the vertical direction from both the top and bottom of the specimen.

To investigate the influence of the composition of the water that enters the soil sample on its swelling characteristics, tests were conducted using distilled water and the natural pore water of the soil sample.

A general description of the tests are given in Section 5.3 to 5.8, and a summary of the various tests conducted on individual soil samples is given in Table V.1.

5.2 The Anteus Corporation Consolidometer

The whole assembly is divided into two chambers - a sample chamber and a loading chamber - by a roll seal to which is attached a stainless steel plate. This plate provides the means by which load is applied to the porous stone on top of the sample. The roll seal, which is made up of a rubber impregnated fabric, provides virtually frictionless operation and is capable of tolerating small amounts of tilt without detrimental effect on the test.

The loading chamber is connected to the loading system through a load valve (3) (Fig. 5.1). The load on the sample may be applied either through a sliding reservoir for small loads or from an external air pressure supply through a load regulator in the high pressure ranges.

The water that comes out of the specimen during the consolidation test flows into the saturation reservoir and similarly the water that flows into the sample during the swelling test is drawn from the saturation reservoir. Back pressure, if used, is applied through this reservoir and the pressure is regulated by means of the back pressure regulator. When no back pressure is applied during the test, the back pressure vent should be kept open. The equaliser valve allows a pressure equal to the back pressure being applied on to the sample, (through the loading chamber) so that during the initial back pressure application, the effective stress of the sample does not change.

The bottom porous stone is sealed from the sample chamber and is connected to a pore pressure measuring device. In this laboratory, by means of a T connection, this lead is taken to either a pressure transducer or to a U - tube manometer through a null-indicator.

When setting up this machine for a consolidation or a swelling test, care should be taken to get rid of any air entrapped in the loading chamber and the connecting line between the sliding reservoir and the loading chamber. The dome containing the loading chamber is removed from the base. The sliding reservoir is raised to a height above the dome. By opening the load pressure vent valve and the load valve, pressure is applied onto the loading plate. The loading plate is allowed to move downwards by about half an inch. By a gentle uniform push of the loading plate up, any air entrapped in the loading chamber or the connecting line could be forced out. The zero condition of the sliding reservoir corresponds to that position when the difference in head between the water levels in the saturation reservoir and the sliding reservoir is equal to the weight of the loading system. The zero elevation of the sliding reservoir may be obtained by adjusting the water level in the saturation reservoir with the sliding reservoir in the zero elevation, so that there is no movement of the loading disc as judged by the dial indicator at the top. During this process, it must be insured that the pore water and the permeability valve are kept closed.

The actual operation of this machine will be given when each particular test is described in the following sections.

5.3 Calibration of the machines

The two porous stones and the two filter papers cut to the size of the porous stones were boiled in the cell water to be used in the swelling tests. The machines are then calibrated using these porous stones and filter papers. The machines are subjected to at least four cycles of loading and unloading until a unique calibration curve is obtained. The whole set-up is maintained with a load equal to the anticipated swelling pressure of the soil sample to be tested, until the sample is prepared and is ready to be tested. This method was followed to minimize the effect of any creep in the filter papers or other parts of the measuring system.

5.4 Preparation of the swelling samples

In the conventional consolidometers, the sample is cut in the consolidometer ring itself. In the Anteus Corporation machine, the sample is cut out of the block using a sample cutter and is then transferred to the teflon coated ring before testing.

In either case, the sample cutter is lubricated inside and outside using a silicon grease spray. The cutter is weighed accurately. It is gradually worked into the sample block by carving the soil to slightly larger dimensions than the diameter of the cutter, ahead of the cutting edge and pushing the cutter into the sample using mild pressure. Once the cutter has been advanced completely, the soil is cut out at the bottom using a thin blade to free the soil in the cutter from the parent block. The soil is trimmed on both sides using very sharp blades, taking great care not to apply any pressure on the samples. The sample is prepared to a thickness less than (by about 25%) the depth of the consolidometer ring in order to provide space for the samples to swell. The weight and dimensions of the sample are determined carefully.

The trimmings obtained during the sample preparation are used for water content determination. The balance of the trimmings were preserved in air tight bags in the humidity and temperature controlled room for later use in pore water analysis and other tests described in Chapter IV.

5.5 Determination of the constant volume swelling pressure

The load (approximately equal to the anticipated swelling pressure) that was maintained on the consolidation machine (after the calibration was over) is now released. The cell water is drained out. The consolidometer ring with the soil is set up in the machine, care being taken to see that there is good contact between the soil and the two porous stones. The dial reading at zero load and zero time is noted. The cell is filled with the appropriate water - the pore water or the distilled water. As the sample tries to swell, load is applied on the sample to maintain constant volume. With increasing load, the new dial gauge reading to account for the calibration of the machine is calculated. The magnitude of the load and the time of application are noted. The load is continuously varied until a final equilibrium value is reached which causes no further volume change with time. This load is maintained for at least 1 week - in some cases up to six weeks before further testing is continued on these samples.

Further tests conducted on the samples after the swelling pressure determination may be grouped as follows: the free swell test (Section 5.6) or the tests for defining the pressure swell relationship (Section 5.7) or some special tests which will be mentioned in the appropriate places.

5.6 Free-swell test

The pressure on the sample was decreased from the swelling pressure to zero and the sample was allowed to swell. The deflection is measured with time until the primary part of swell is complete and the slope of the secondary swell is well defined as seen from the plot of amount or percent swell against the square root of time. The amount of swell is corrected for the calibration of the machine.

5.7 Pressure-swell relationship

After the determination of the swelling pressure, the pressure swell relationship of the soil sample may be defined by using either a strain controlled or a stress controlled test.

In the strain-controlled test, the sample is allowed to swell in steps of 1 or 2 percent swell and the load on the sample is adjusted to maintain it at that volume. In this method, the new dial reading to be reached corresponding to the amount to which the sample is permitted to swell is estimated. The pressure on the sample is decreased to zero and the sample is allowed to swell. Once the expected dial reading has been reached, loads are gradually applied on the sample to maintain the volume of the soil at that level. This process is continued until the final equilibrium load is reached. The magnitude and the time of application of the load are noted to facilitate determination of the rate of development of the equilibrium pressure corresponding to the various percentages of swell of the sample.

In the stress-controlled test, the load on the sample is decreased from its swelling pressure in steps. At each load, the swell is measured with time until the primary part of swelling is complete and the secondary part of swelling well defined.

The strain-controlled test has an advantage over the other because in this test the percent swell corresponding to any load is the ultimate swell under that load. But the stress-controlled method provides information on the rate of swelling at various stress levels which is not determined by the other method. However, the rate at which the equilibrium pressure develops gives a good approximation of the rate of swelling at those stress levels. (See footnote below)

5.8 Termination of tests

The samples are weighed at the end of the test. The final water content is determined for most of them. Some of the samples were squeezed to extract their pore water for chemical analysis.

5.9 Summary of tests

Table V.1 gives the summary of the tests conducted on individual samples, the soil type, location and the cell water used. The numbering system for the samples used in this table is as follows: the first number refers to the soil type (see footnote under Table V.1), the next number refers to the sample number, followed by the type and number (as used in this laboratory) of the apparatus (CH, WF and Ant referring to Clockhouse, Wykeham Farrance and Anteus consolidometers respectively), the last two letters defining the cell water used - either distilled water (dw) or the natural pore water (pw).

The strain-controlled test as described above assumes that the compression and swelling indices are equal.

TABLE V.1

SUMMARY OF TESTS ON VARIOUS SAMPLES

Sample No.	Depth ft	Magnitude of Sw. Press	Coeff. of Sw. Press	Free Swell	Pressure Swell Relation	Other tests
1-2-CH5-dw	5.5- 6.8	✓		✓		
1-2-WF4-pw	5.5- 6.8	✓		✓		
1-3-CH5-dw	5.5- 6.8	✓	✓	✓		
1-3-WF4-pw	5.5- 6.8	✓	✓	✓		
1-4-CH5-dw	5.5- 6.8	✓	✓		✓	
1-4-WF4-pw	5.5- 6.8	✓	✓		✓	
2-5-WF4-dw	30	✓				
3-5-CH5-dw	16 -16.6	✓	✓			
3-5-CH6-dw	22 -22.7	✓				
4-6-CH6-dw	68 -69.9	✓	✓			(a)
4-7-CH6-pw	72 -73	✓	✓		✓	
4-6-CH7-dw	72 -73	✓	✓		✓	
4-1-Ant8-pw	68 -69.9	✓	✓		✓	
4-1-Ant9-pw	68 -69.9	✓	✓		✓	
4-1-Ant10-dw	68 -69.9	✓	✓		✓	
4-1-Ant11-dw	68 -69.9	✓	✓		✓	
5-8-CH6-dw	9.5-10.3	✓	✓		✓	
5-8-CH7-pw	9.5-10.3	✓	✓		✓	
5-9-CH6-dw	10.5-11.3	✓	✓		✓	
5.9-CH7-pw	10.5-11.3	✓	✓		✓	
5-10-CH6-dw	8.5- 9.3	✓	✓			(b)
5-10-CH7-dw	8.5- 9.3	✓	✓			(b)
5-8-CH5-dw	9.5-10.3	✓				(c)
5-9-CH5-dw	9.5-10.3	✓				(c)
5-10-CH5-dw	8.5- 9.3	✓				(c)
5-11-CH5-dw	10.5-11.3	✓				(c)
5.14-CH5-dw	5.5-7	✓	✓		✓	(c)

Soil Types:
1: Bearpaw Shale-Block sample
2: Edmonton shale
3: Bearpaw shale weathered
4: Bearpaw shale-intact
5: Morden shale

Other tests:
(a) Comparison of coefficient of swelling pressure with coefficients of consolidation and swelling
(b) Variation of coefficient of secondary swelling with the load decrement ratio
(c) Study on dilution of pore water of sample after full mobilisation of swelling pressure

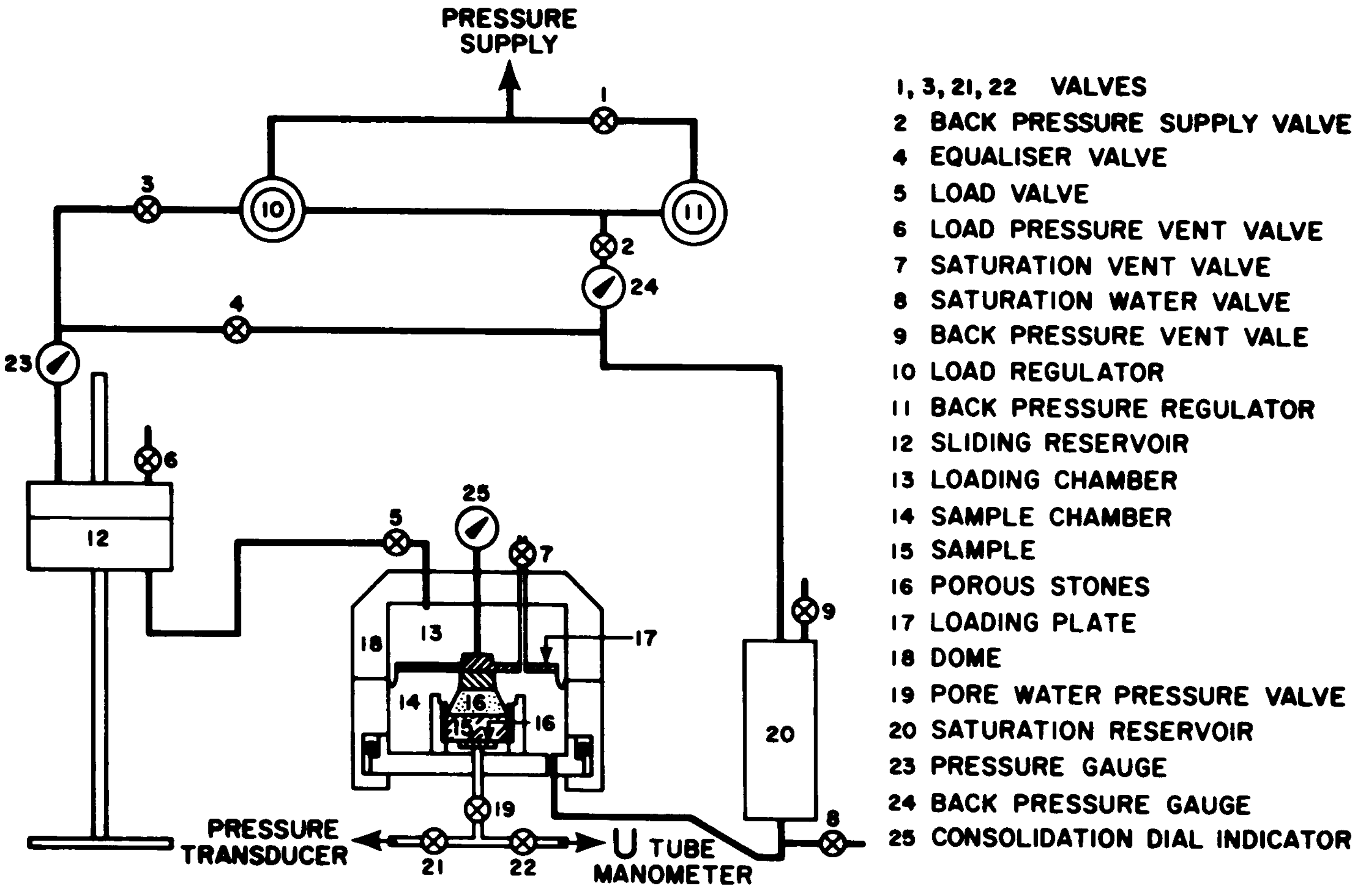


FIG. 5-1 ANTEUS CORPORATION CONSOLIDOMETER - A SCHEMATIC DIAGRAM

CHAPTER VI

INTERPRETATION OF THE RATE OF DEVELOPMENT OF SWELLING PRESSURE FOR FULLY SATURATED SOILS

6.1 Introduction

It is known that swelling pressure under constant volume conditions does not develop instantaneously on inundation but that it takes time and that the percent swelling pressure plotted against time gives a curve similar to the consolidation curve. (Palit, 1953; Ladd and Lambe, 1961; Baker and Kassiff, 1968; Komornik and Zeitlen, 1970.)

The only attempt, to the author's knowledge, to explain the delay in the response of the swelling pressure has been made by Baker and Kassiff (op. cit.). They have developed a theory for the development of swelling pressure of partly saturated soils which invokes many assumptions. The effective stress that controls the volume change characteristics has been assumed by them as

$$\sigma' = \sigma - u_a - \chi (u_a - u_w) \qquad (6.1)$$

where σ' = effective stress
σ = total stress
u_a = pore air pressure
u_w = pore water pressure

and χ = a factor having values ranging from 0 for dry soils to 1 for fully saturated soils

That this is the unique value of effective stress controlling volume change characteristics of partly saturated soils has been questioned by many workers. (For eg. Jennings and Burland, 1962) Baker and Kassiff (1968) ignored arguments on this point and accepted $\frac{\partial \sigma'}{\partial t} = 0$ as a condition for no volume change where σ' is defined by Equation 6.1. They assumed further that χ is equal to the degree of saturation, S, even though Bishop and Henkel (1962) claim that a simple common relationship does not exist between these two parameters. Now, according to Baker and Kassiff, the Equation for no volume change is

$$\frac{\partial \sigma'}{\partial t} = 0 \tag{6.2}$$

which when substituted in Equation 6.1 yields

$$\frac{\partial \sigma}{\partial t} = \frac{\partial u_a}{\partial t} - \frac{\partial}{\partial t}[\chi(u_a - u_w)] \tag{6.3}$$

The pressure on the liquid and gas phases are related by the expression

$$u_a = u_w + \frac{2T_s}{R} \tag{6.4}$$

where T_s = surface tension of water

and R = radius of curvature of gas bubble

Substituting Equation 6.4 for $(u_a - u_w)$ in Equation 6.3, the expression for $\frac{\partial \sigma}{\partial t}$ becomes

$$\frac{\partial \sigma}{\partial t} = \frac{\partial u_w}{\partial t} + 2T_s \frac{\partial}{\partial t}\left[\frac{1}{R}\right] - \frac{\partial \chi}{\partial t} \cdot \frac{2T_s}{R} - 2T_s \chi \frac{\partial}{\partial}\left[\frac{1}{R}\right] \qquad (6.5)$$

In order to simplify this Equation, the author made another assumption that $(u_a - u_w)$ and hence R remain constant during the swelling pressure determination. The swelling pressure considered here is that pressure at which no volume change is allowed. But during this process, an increase in the water content does take place in a partly saturated soil reducing the soil suction which in turn causes a variation of the radius of the gas bubble. However, by making this assumption and substituting S for χ, the authors arrive at the governing equation as

$$\frac{\partial \sigma}{\partial t} = \frac{\partial u_w}{\partial t} - \frac{\partial S}{\partial t} \cdot \frac{2T_s}{R} \qquad (6.6)$$

where S = degree of saturation

It should be noted in the above Equation that Baker and Kassiff permit changes in the degree of saturation with time, but expect $(u_a - u_w)$ and hence R to remain constant which is impossible.

Expressing u_w as a function of the initial pore water pressure plus a change in u_w due to a change in water content and a change in the external stress, σ as

$$u_w = u_{wo} + a\,(w - w_o) + b\,\sigma \qquad (6.7)$$

and S in terms of W_w, the weight of water in a unit volume of soil as

$$\frac{\partial S}{\partial t} = \frac{1}{\gamma_{w.e}}\,\frac{\partial W_w}{\partial t} \qquad (6.8)$$

and substituting these expressions in Equation 6.6, they get

$$\frac{\partial \sigma}{\partial t} = C\,\frac{\partial W_w}{\partial t} \qquad (6.9)$$

where $C = \frac{1}{\gamma_w(1-b)}\left[\frac{a}{G_s} - \frac{2T_s}{eR}\right]$

u_{wo} = the initial pore water pressure

a = the slope of the moisture content suction curve

b = a factor expressing the fraction of the applied pressure which is effective in changing the pore water pressure

e = void ratio

G_s = specific gravity of soil solids

and γ_w = unit weight of water

Substituting $\frac{\partial\sigma}{\partial t}$ from Equation 6.9 and $\frac{\partial S}{\partial t}$ from Equation 6.8 in Equation 6.6, one gets

$$\frac{\partial u_w}{\partial t} = \frac{C'}{C} \frac{\partial\sigma}{\partial t} \qquad (6.10)$$

where $C' = C + \frac{2T_s}{e.\gamma_w.R}$

If the compressibility of soil solids and water are neglected and if the permeability k is assumed constant, the changes in moisture with time during the transient flow state may be expressed as follows (Scott, 1963):

$$\frac{\partial W_w}{\partial t} = k \frac{\partial^2 u_w}{\partial t^2} \qquad (6.11)$$

where W_w = weight of water in a unit volume of soil

Substituting Equations 6.9 and 6.10 in Equation 6.11, Baker and Kassiff get the expression for $\frac{\partial u_w}{\partial t}$ as

$$\frac{\partial u_w}{\partial t} = c_{sp} \frac{\partial^2 u_w}{\partial t} \qquad (6.12)$$

where c_{sp} = coefficient of swelling pressure

= $C'K$

For saturated clays as R tends to ∞ , $C = C'$. But with b tending to unity with saturation, C and C' and hence c_{sp} tend to ∞ . This means that the swelling pressure of a saturated soil sample develops instantaneously. This has been shown not to be true by Palit (1953) and also in the present study on samples with degrees of saturation close to 100%.

A theory for the rate of development of swelling pressure for saturated soils has been developed below and its validity has been checked by experiments.

6.2 Theory

6.2.1 General

The swelling pressure of a saturated soil is equal in magnitude to the initial suction in the pore water, u_i . At the outset, the sample is in equilibrium with a pore water pressure of $-u_i$ or a positive effective stress of $+u_i$, as shown in Fig. 6.1(a). The instant the sample is brought in contact with an external source of water at zero pressure, the pore pressure at the boundaries of the sample (top and bottom surfaces for the consolidometer samples) suddenly increases to zero (Fig. 6.1(b)). This results in a decrease of the average effective stress within the sample causing the sample to swell. The swelling is arrested by adding external loads such that the volume of the sample remains constant during the constant volume swelling pressure test. The addition of the external load at any instant on the sample may be visualised as being applied to keep the average effective stress within the sample constant, if the swelling and compression indices are assumed to be equal, as mentioned under Section 6.2.4.

6.2.2 Governing Equations

Within the framework of the Terzaghi theory of consolidation, the governing equations for a change in pore pressure within the sample are (Scott, 1963):

$$c_{vc} \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - \frac{\partial \sigma}{\partial t} \quad \text{during consolidation} \qquad (6.13a)$$

$$c_{vs} \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - \frac{\partial \sigma}{\partial t} \quad \text{during swelling} \qquad (6.13b)$$

where $c_{vc,vs}$ = coefficients of vertical consolidation and swelling respectively

u = pore water pressure

z = dimension in the direction in which flow of water and resulting increase in volume occur

and σ = total stress on the soil

6.2.3 Boundary and initial conditions

The instant the sample is set up in the consolidation machine, the effective stress within the sample at all depths is equal to $+u_i$

i.e. $$\sigma'_{z,o} = +u_i \qquad -H \leq z \leq H \qquad (6.14a)$$

If the sample in inundated at time ε, the pore water pressure at the two boundaries will become zero and remain zero for all times thereafter.

$$u_{H,t} = u_{-H,t} = 0 \qquad t > \varepsilon \tag{6.14b}$$

This also means that the effective stress at the boundaries is equal to the external load.

i.e. $$\sigma_t = \sigma'_{H,t} \tag{6.14c}$$

Due to symmetry about the centre line of the sample, there is no flow of water across the centre line of the sample,

i.e. $$\frac{\partial u}{\partial z} = 0 \text{ at } z = 0 \tag{6.15a}$$

or $$\frac{\partial \sigma'}{\partial z} = 0 \text{ at } z = 0 \tag{6.15b}$$

since $\frac{\partial \sigma'}{\partial z} = 0$, $-H<z<H$ for consolidation specimens.

6.2.4 Assumptions

By assuming that the coefficients of volume compressibility and swelling are the same for compaction shales at stress levels very close to their swelling pressure, the condition of no volume change at any time may be satisfied by stating that there is no change in average effective stress with time.

i.e. $$\frac{1}{2H} \int_{-H}^{H} \frac{\partial \sigma'}{\partial t} . dz = 0 \qquad (6.16)$$

where 2H = thickness of the sample with double drainage.

During the process of development of the swelling pressure the external load on the sample is so adjusted so as to allow no net volume change. But the present set-up requires some response (an increase or decrease of the volume of the sample in the order of about 0.01 - 0.02% of its original volume) before the external load is adjusted accordingly. The magnitude of the volume change thus imposed on the sample being small, its effect on the coefficients of consolidation or swelling will be negligible. Hence it is reasonable to assume that the coefficients of consolidation and swelling are equal. This assumption reduces Equations 6.13a and 6.13b to a single Equation 6.13c.

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - \frac{\partial \sigma}{\partial t} \qquad (6.13c)$$

where $c_v = c_{vc} = c_{vs}$

6.3 Solution

This theory also leads to an instantaneous development of the swelling pressure for saturated soils.

The deviation found between this theoretical prediction and experimental observations could possibly be due to two reasons, (i) the compliance of the measuring system and (ii) a compressible pore fluid (Byrne, 1971).

Palit (1953) has used a proving ring to measure the swelling pressure of soil samples. This method requires some deformation of the proving ring and a corresponding swelling of the soil sample. (Actually the swelling of the soil sample would exceed the deformation of the proving ring by an amount equal to the compression of the porous stones, filter paper, etc., at any load.) Thus in such cases, the development of the swelling pressure would not be instantaneous, but would be a function of the swelling of the soil sample, thereby yielding a rate of development of swelling pressure comparable to the coefficient of swelling.

In the present set-up, the sample volume has essentially been kept constant by adjusting the loads on the sample. During this process, it was observed that at any time a load larger than the one applied (but less than the swelling pressure of the soil sample) caused consolidation of the sample below its initial volume. This is possible only if the pore pressure parameter $B \neq 1$. $B=1/(1+n.C_w/C_c)$; where n is the porosity of the soil and C_w and C_c are the compressibilities of the pore water and the soil skeleton respectively (Skempton, 1954). B could be less than 1 with a very stiff soil having a compressibility comparable to that of C_w or if the pore fluid is compressible. Wissa (1969) reports a value of

10^{-3} cm^2/kg for uncemented soils not heavily overconsolidated. For the two compaction shales tested in this programme, the compressibility of the soil skeleton, in the pressure range near their swelling pressures, was found to be about 5 x 10^{-3} cm^2/kg. This value being two orders of magnitude larger than the compressibility of pure water (5.5 x 10^{-5} cm^2/kg), would yield a value of B equal to 1 for all n. This would lead one to conclude that the pore fluid should be compressible in order to obtain a B less than 1. The compressibility of the pore water may arise due to the presence of air in the pore space. A saturated soil, by definition has no free air present in it. Air if present in solution in pore water does not contribute to the compressibility. Thus, a compressible pore water has to have some free air in it. The initial negative pore water pressure would be in magnitude equal to the swelling pressure of the sample times B at the time full swelling pressure has been mobilised. The swelling pressures of the Bearpaw and Morden shale samples were in the range of 2-5 tsf, indicating initially they were at pore water suctions of similar order of pressures. These negative pore water pressures being less than -1atm. could cause cavitation and any air in solution would separate as free bubbles. It is suspected that it is this phenomenon that contributes to a compressible pore fluid during the measurement of the swelling pressure of these undisturbed samples.

Thus, the rate at which the swelling pressure develops may be a function of the rate at which the compressibility of the pore fluid changes, which is itself a function of the pore water pressure at any time. This could possibly be why the time for development of swelling pressure is comparable to that of swelling, both yielding coefficients (of swelling pressure and swelling respectively) of equal magnitude. The coefficient of swelling pressure, c_{sp}, is the coefficient calculated from the rate of development of swelling pressure curve, whereas the coefficient of swelling is the coefficient derived from the conventional swelling tests. This coefficient of swelling pressure for the samples in distilled water will be influenced by the rate of diffusion of the salts from the pore water into the ambient water. The block Bearpaw shale and the Morden shale samples yielded similar c_{sp} values in distilled water which indicates that the coefficient of diffusion for these materials is probably very small (see Chapter VIII).

To investigate the use of the development of swelling pressure with time curve the following tests were conducted on an intact Bearpaw shale: 4-6-CH6-dw. The sample was first subjected to a constant volume swelling pressure test. The results of the rate of development of swelling pressure are shown in Fig. 6.2 in a $\sqrt{t}$ plot and in Fig. 6.2a in a log t plot. From the time for 50% development of swelling pressure, using a $T_{50} = 0.196$, the coefficient of swelling pressure c_{sp}, has been calculated and is given in Table VI.1. The magnitude of the swelling pressure was 2.89 tsf. The sample was then allowed to swell to 2.00 tsf and the deflection readings were taken with time until the primary part of swelling was complete and the secondary swelling well defined*. (Figs. 6.3 and 6.3a). The coefficient of swelling was calculated similar to the evaluation of the coefficient of consolidation. The sample was then consolidated to 4.00 tsf (Figs. 6.4 and 6.4a) and the coefficient of consolidation was estimated. The sample was then subjected to another cycle of swelling and consolidation and the corresponding coefficients estimated as before (Figs. 6.5, 6.5a, 6.6 and 6.6a). The results of these tests are tabulated in Table VI.1. The value of c_{sp} can be seen to lie between the values of c_{vc} and c_{vs} which are themselves very close. This suggests that the development of swelling pressure curve may be used to evaluate the coefficient of swelling pressure, c_{sp}, which is in magnitude comparable to the coefficient of swelling of the soil sample. This statement has been substantiated from the tests on other samples also, which were subjected to small (about 30% of swelling pressure) changes in effective stresses.

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* For these samples and others described later, this control was kept based upon the $\sqrt{t}$ plot, in which a demarcation between the primary and secondary swelling was more abrupt and clearly defined.

TABLE VI.1

COMPARISON BETWEEN COEFFICIENTS OF CONSOLIDATION, SWELLING AND SWELLING PRESSURE

Stress	Coeff. of swelling, c_{vs} ft^2/year		Coeff. of cons., c_{vc} ft^2/year	
tsf	$\sqrt{t}$ method	log t method	$\sqrt{t}$ method	log t method
2.89-2.00	1.66	1.49		
2.00-4.00			2.79	2.50
4.00-2.00	2.12	1.75		
2.00-4.00			2.90	2.33

SAMPLE: 4-6-CH6-dw

Swelling Pressure = 2.89 tsf

Coeff. of Sw. pr., c_{sp} = 2.33 ft^2/year ($\sqrt{t}$ method)

= 1.94 ft^2/year (log t method)

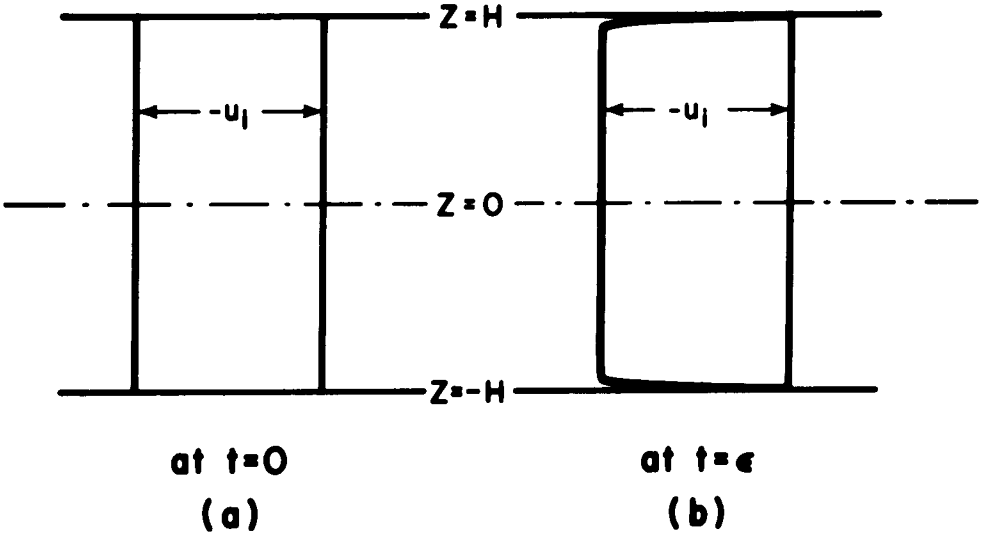

FIG. 6-1 VARIATION OF PORE PRESSURE WITH DEPTH IN SAMPLE

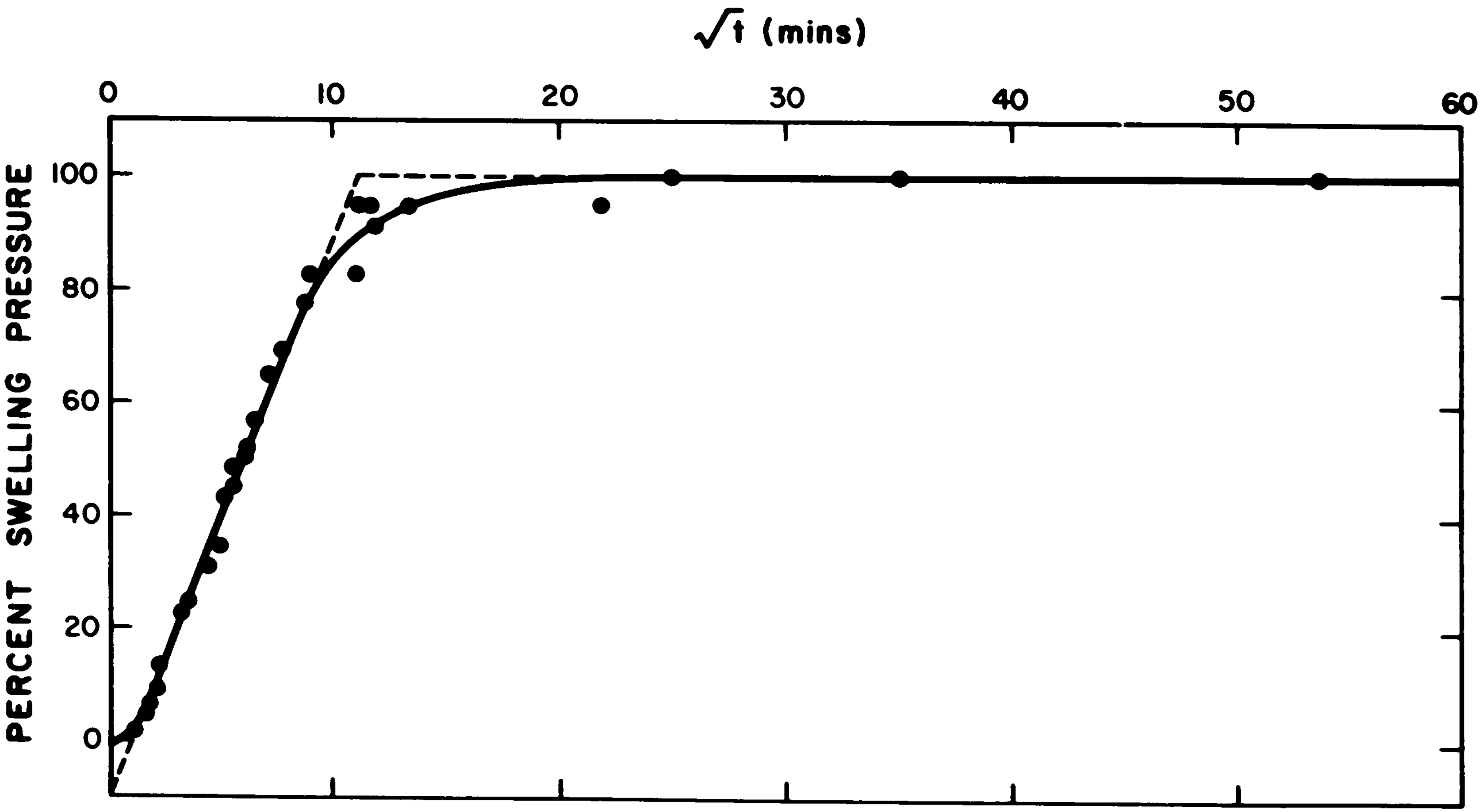

FIG. 6-2 DEVELOPMENT OF SWELLING PRESSURE WITH TIME: SAMPLE 4-6-CH6-dw

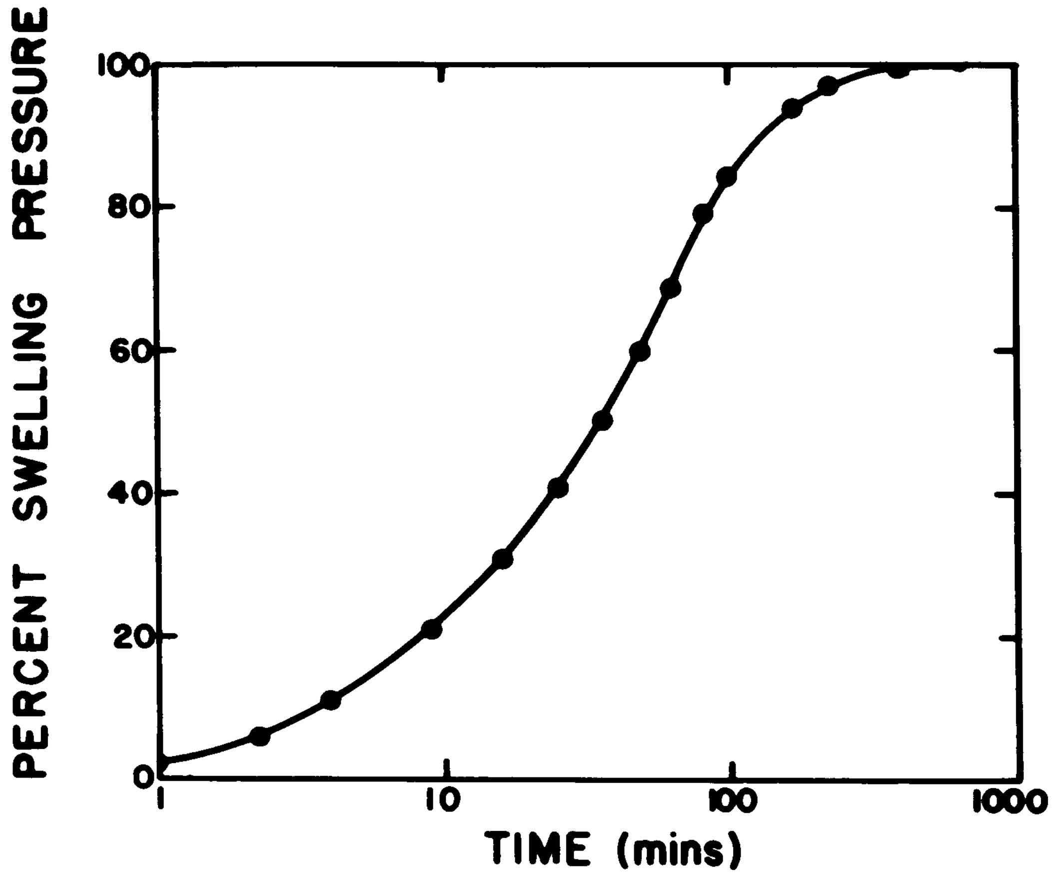

FIG. 6-2a DEVELOPMENT OF SWELLING PRESSURE WITH TIME:
SAMPLE 4-6-CH6-dw

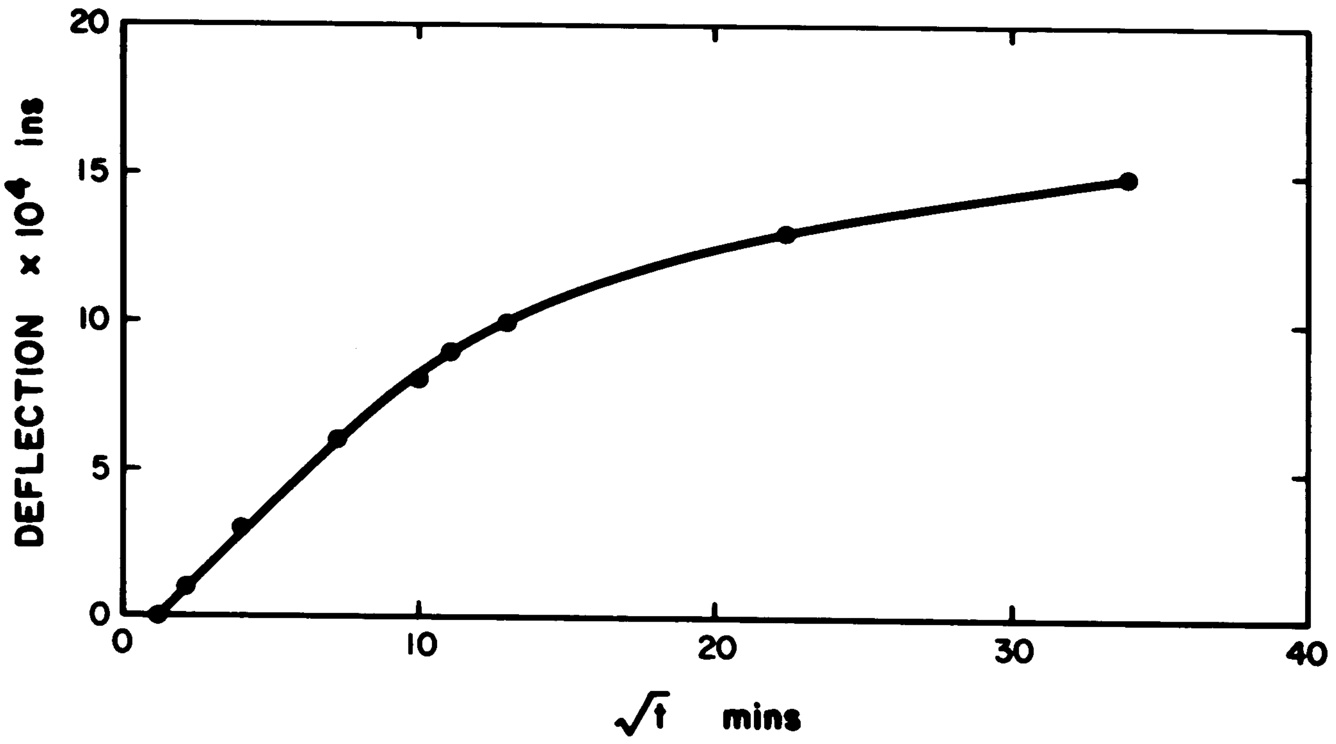

FIG. 6-3 SWELLING TEST (1): 2.89 - 2.00 tsf.

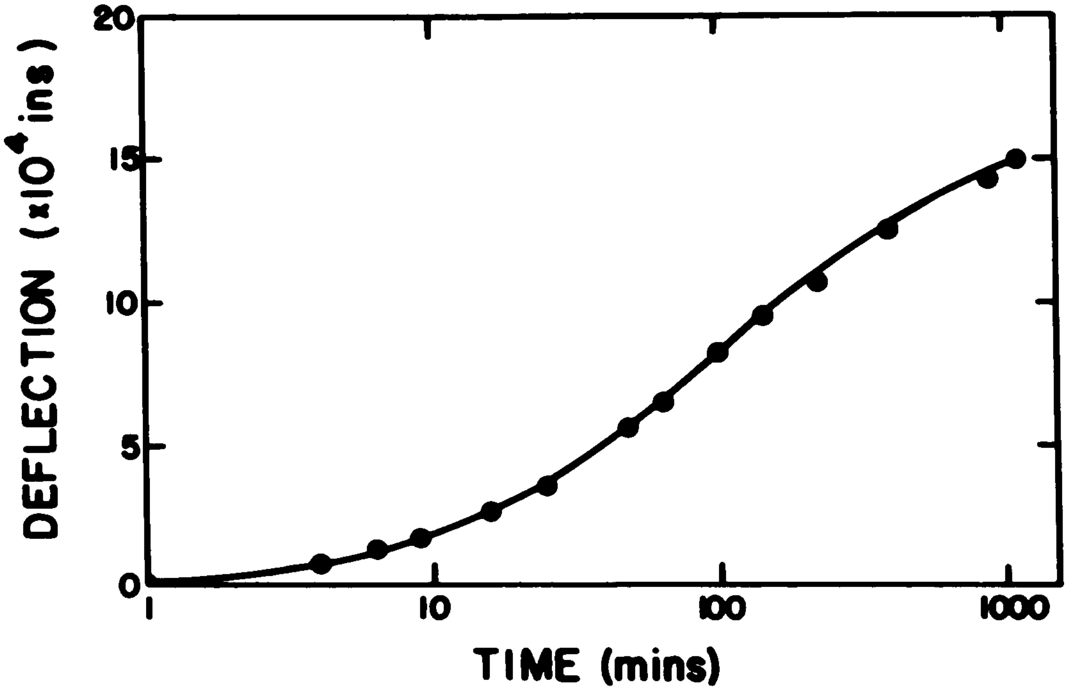

FIG. 6-3a SWELLING TEST (1): 2.89-2.00 tsf.

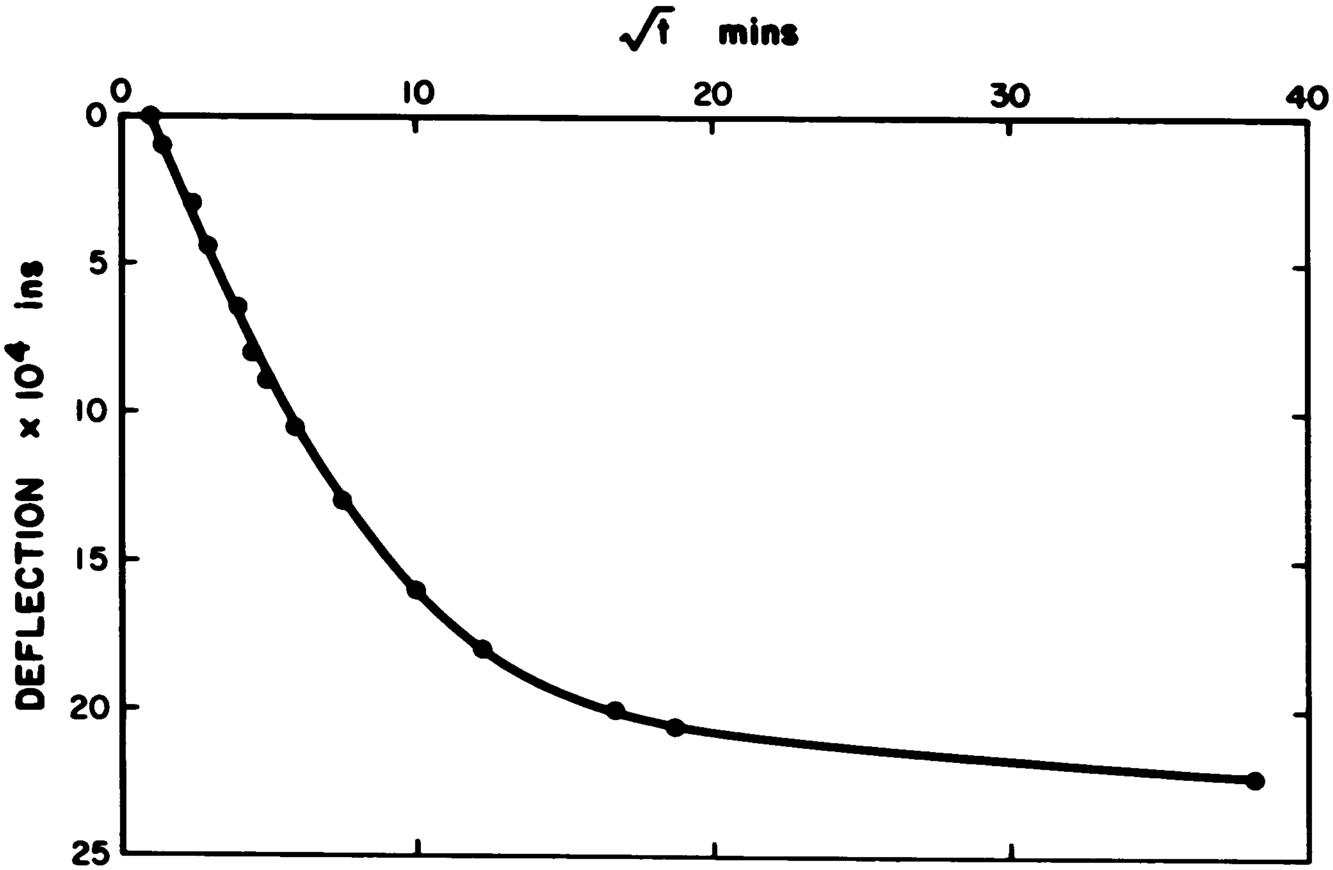

FIG. 6-4 CONSOLIDATION TEST (1): 2.00 - 4.00 tsf

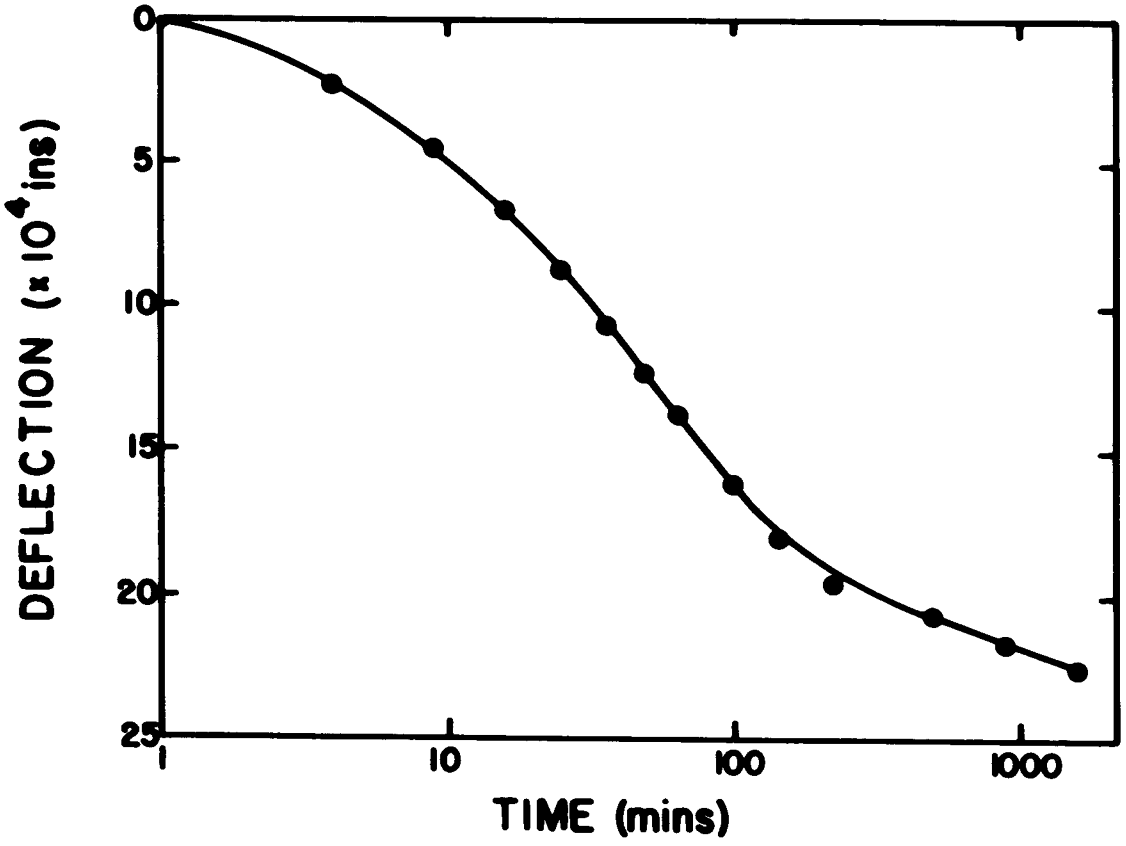

FIG. 6-4a CONSOLIDATION TEST (1): 2.00-4.00 tsf.

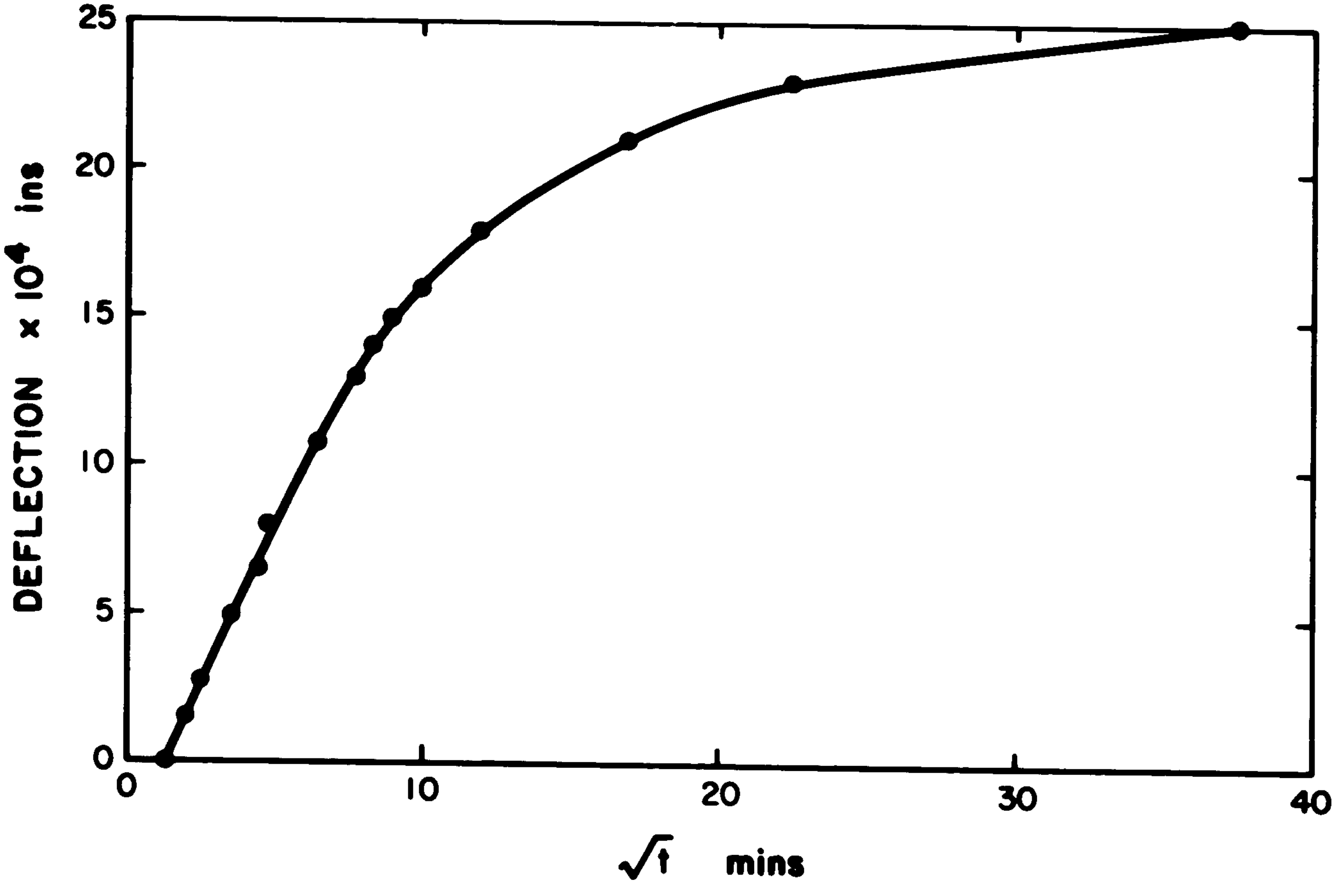

FIG. 6-5 SWELLING TEST (2): 4.00 - 2.00 tsf

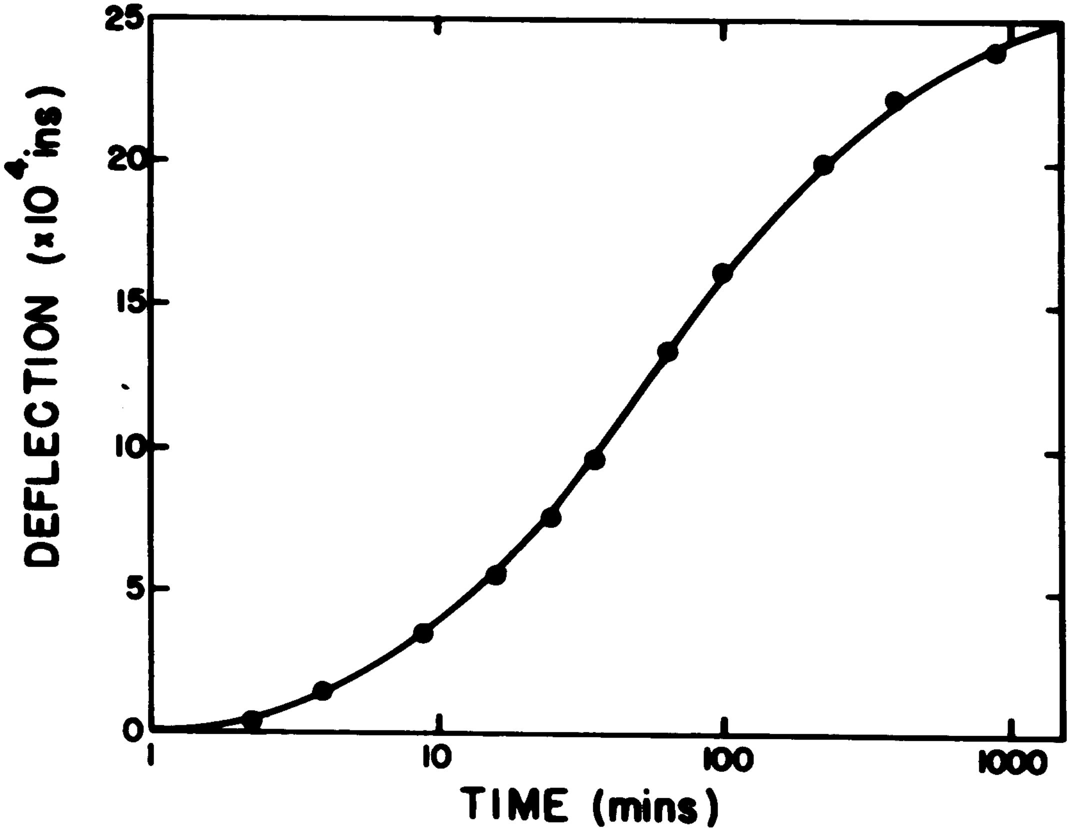

FIG. 6-5a SWELLING TEST (2): 4.00-2.00 tsf.

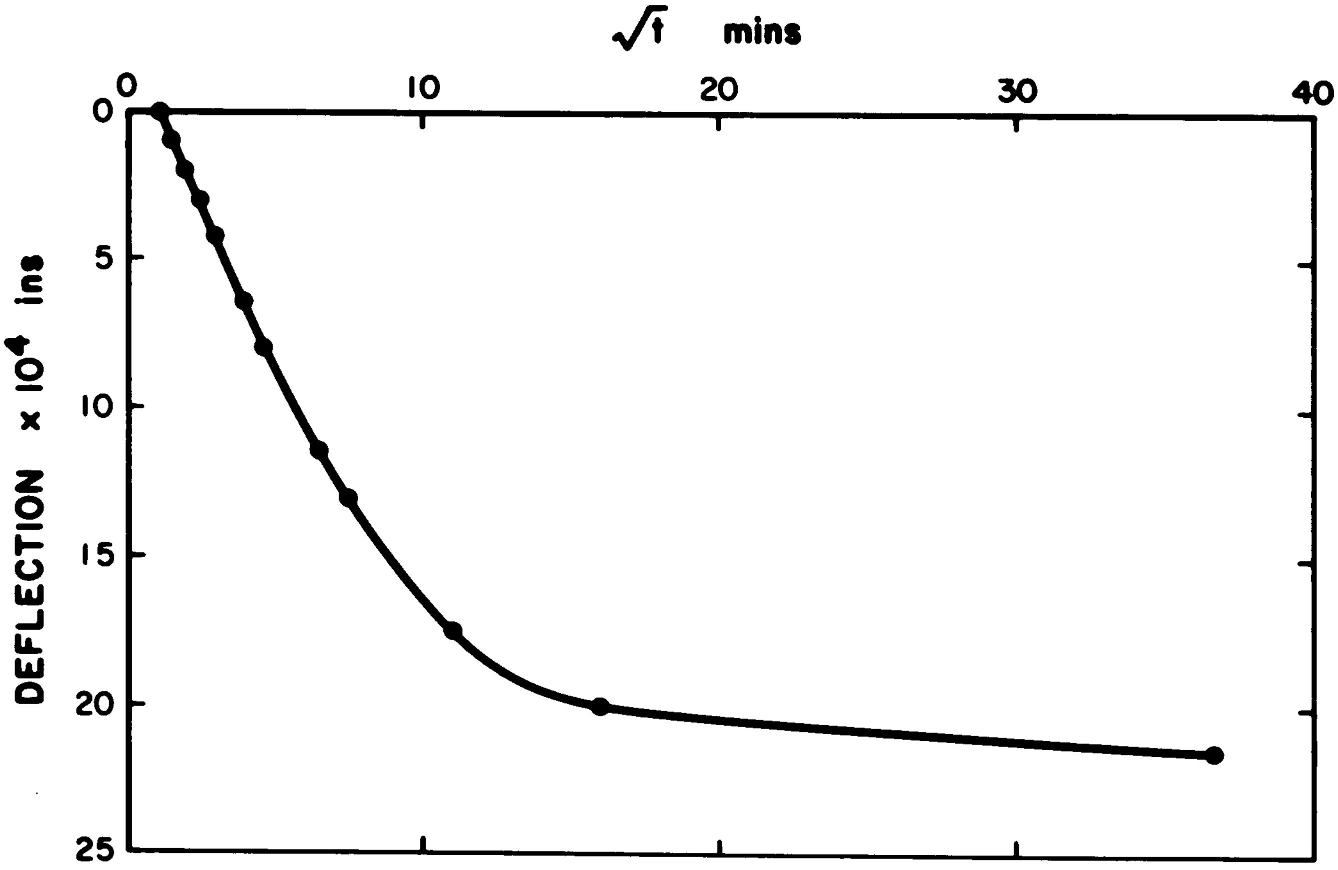

FIG. 6-6 CONSOLIDATION TEST (2): 2.00 - 4.00

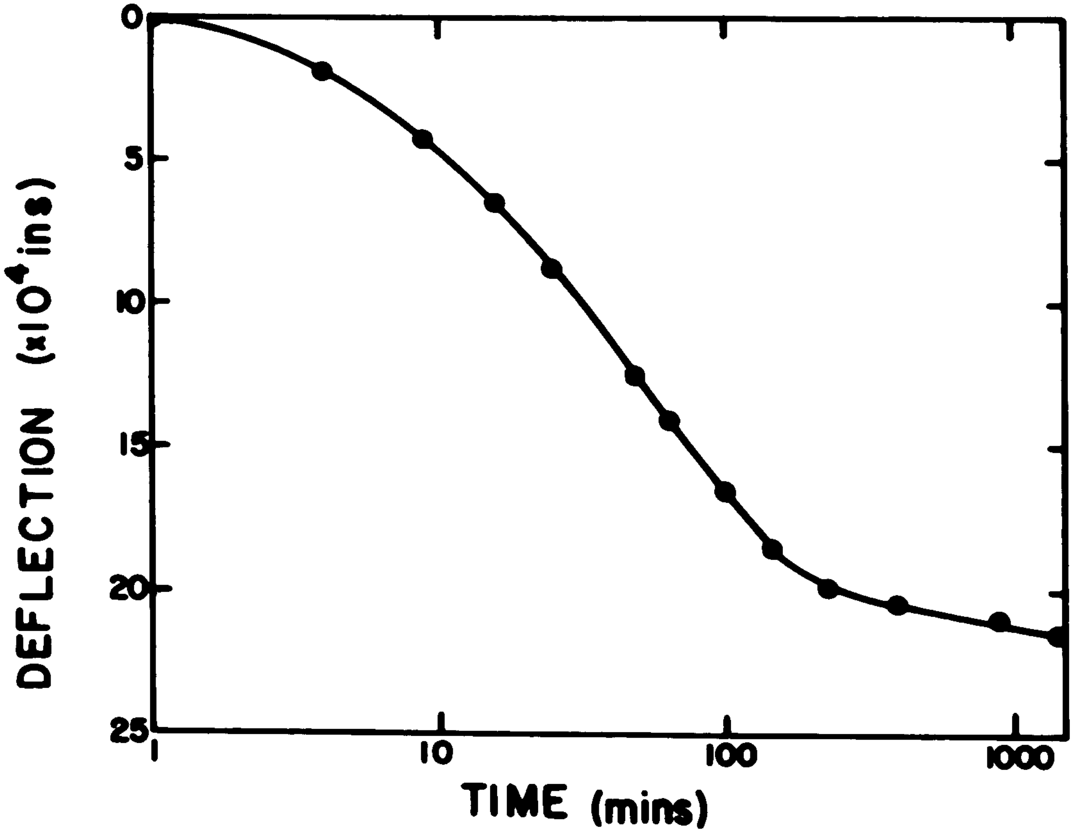

FIG. 6-6a CONSOLIDATION TEST (2): 2.00-4.00 tsf.

CHAPTER VII

RESULTS AND DISCUSSIONS

7.1 General

This chapter discusses the results obtained from the swelling tests described in Chapter V. Some special tests conducted to substantiate any argument proposed are also included in this chapter. A mechanism of swelling is proposed which has greater generality than others put forward by earlier workers.

The total number of tests conducted in this study is small. Each test was chosen depending upon the earlier results. Moreover, since the tests in this series were conducted on natural materials, the results for one material are not directly comparable with the others. Hence, the presentation that follows is subdivided on the basis of the materials tested.

A description of the materials has already been given in Chapter III and the details of the samples tested are noted in Chapter V.

7.2 Bearpaw shale-block sample

(i) Samples 1-2-WF4-pw and 1-2-CH5-dw

These two samples were tested for their swelling pressure in natural pore water and distilled water respectively. No observations on the time rate of development of the swelling pressure were made in this series, but it was noted that even though these samples were almost fully saturated (with degrees of saturation ranging between 90 and 100%), the swelling pressure did not develop instantaneously on inundation. The magnitudes of swelling pressure of these samples were 1.75 tsf (for sample 1-2-WF4-pw) and 1.71 tsf (for sample 1-2-CH5-dw). It was surprising to observe that the cell water did not have any effect on the magnitude of the swelling pressure. These samples were maintained under their respective swelling pressures for at least 6 weeks (after the full swelling pressure had been mobilised) to see whether the swelling pressure would change with time; but it stayed constant. The samples were then subjected to a free swell test to see the influence of the cell water on the magnitude of swell. The test was continued for about 10 days by which time the primary part of swelling was complete and the secondary part well defined, as seen by a clear change of slope in the $\sqrt{t}$ plot. The sample in distilled water 1-2-CH5-dw swelled 15% compared to 13% for sample 1-2-WF4-pw. The coefficient of swelling in distilled water was higher than the value for the sample in its natural pore water. (Fig. 7-1 and Fig. 7-1a shows the percent swell plotted against $\sqrt{t}$ and log t respectively.) Table VII-1 gives these results together with the results of the chemical analysis of the final pore water from these samples and their cell water. Because the ground water table at the time of sampling was not known, the initial effective overburden pressure could not be evaluated to be compared with the measured swelling pressure. (This is the case with all samples of Bearpaw shale.) From this table, one can see that the concentration of the cell water and the pore water of sample 1-2-WF-pw had increased. By mistake, the water level in the cell was maintained constant to balance evaporation by adding the artifically prepared pore water solution instead of distilled water. The concentration of sample 1-2-CH5-dw had decreased by imbibing cell water, as might be expected. Unfortunately, no tests were made on the cell water of sample 1-2-CH5-dw to see whether it bore any comparison with the final pore water of the sample.

The results of these tests led to further testing of the same soil in its natural pore water and distilled water to check the measurement of its swelling pressures and also to inspect the rate of development of the swelling pressure.

(ii) Samples 1-3-WF4-pw and 1-3-CH5-dw

The results of tests on these samples are given in Table VII.2. The magnitudes of swelling pressures were 1.32 tsf for the distilled water sample and 1.38 tsf for the sample in its own pore water. The coefficient of swelling pressure, c_{sp}, was 1.77 ft^2/year for sample 1-3-CH5-dw and 7.70 ft^2/year for 1-3-WF4-pw from the $\sqrt{t}$ plot. The above values from the log t plot were 1.66 and 8.2 ft^2/year respectively. (At the commencement of the test, the contact between sample 4-3 and the two porous stones was not good and therefore, the initial pressure readings for that sample were missed. Hence, not much reliance can be placed on the value of c_{sp} for the sample 1-3-WF4-pw.) The samples were then allowed to swell at "no load" (Fig. 7-3) and the coefficients of swelling for samples 1-3-WF4-pw and 1-3-CH5-dw were found to be 0.17 and 0.495 ft^2/year respectively from the $\sqrt{t}$ plot. Sample

1-3-CH5-dw swelled 15% of its original volume against 11% for sample 1-3-WF4-pw. No chemical analysis of the final pore water was made. The water contents of the samples were determined very carefully and the degree of saturation was found to be 100% for both of these samples.

(iii) Samples 1-4-WF4-pw and 1-4-CH5-dw

These samples were tested in order to determine the magnitudes of swelling pressure, the coefficient of swelling pressure, c_{sp}, and the pressure-swell relationship using a strain controlled procedure. The results of these tests and their initial moisture content, degree of saturation, etc., are given in Table VII.3. The rate of development of swelling pressure is shown against $\sqrt{t}$ in Fig. 7.4 and against log t in Fig. 7.4a. The pressure-swell relationship is obtained by allowing the sample to swell in increments of 2% of its original volume and observing the equilibrium pressure at each percent swell. This relationship is shown in Fig. 7.5 on a natural scale and in Fig. 7.6 on a semi-log scale. Sample 1-4-WF4-pw was allowed to swell at "no load" (after a controlled swell test up to 8%) and it swelled about 1% more. Sample 1-4-CH5-dw was allowed to swell at "no load" from its 14% swell value and it swelled another 16% in about 3 months. The test was stopped at that stage and the data are presented in Fig. 7.7. The samples were squeezed at the end of the test. The final pore water and the cell waters were analysed and these results are also shown in Table VII.3. The final pore water and cell water of the distilled water sample were very close to each other in composition, indicating free mixing of the pore water with the cell water once swelling was permitted. The concentration of the pore water and the cell water of the sample 1-4-WF4-pw had increased initially for the reasons explained earlier for sample 1-2-WF4-pw. Later the cell water was diluted to the original concentration of the pore water of the sample which slightly decreased in the next 8 months. (This decrease is possibly due to small losses of salt water from the cell due to overflowing.)

(iv) Discussion of results on Block sample

The results of the three sets of tests described above indicate that for this sample of soil, the swelling pressure at constant volume is independent of the cell water used within the time allowed, whereas once allowed to swell, they behave differently. The sample in distilled water swells more than the other sample at the same pressure or conversely, the distilled water sample shows higher equilibrium pressures for the same amount of swell. The fundamental difference between the constant volume swelling pressure test and the tests for the pressure swell relationship is that in the latter, mass inflow of water from the cell into the sample is permitted causing the sample to increase in volume whereas in the former, by varying the pressure on the sample, the sample volume was maintained constant throughout, requiring no mass inflow or outflow of water into or from the soil sample. If the sample surface was acting as a semi-permeable membrane, an osmotic pressure would be developed between the cell and pore water due to the difference in their salt concentrations resulting in a higher swelling pressure for the sample in distilled water than for the sample in its own pore water. In the case of the Bearpaw shale-block sample since no difference in the swelling pressure was measured between the distilled water sample and the pore water sample, the surface of the soil sample does not act as a semi-permeable membrane. If there had been free mixing of the pore water of the soil sample and the distilled water in the cell, the pore water in the distilled water sample would have been diluted. This would result in an added component to the swelling pressure due to an additional double layer repulsive force. (See Appendix A for the double layer repulsion equations and a sample calculation.) During the constant volume swelling pressure test on a saturated soil, through a process of diffusion, the pore water could be diluted, resulting in an added repulsive force between the adjacent particles. The fact that even after waiting for 6 weeks, the swelling pressure of the distilled water sample did not increase, leads one to conclude that the coefficient of diffusion for this soil sample is so small that, in the time scales that are of concern here, the effect of diffusion is nil. Once the sample is allowed to swell, there is a mass inflow of water causing free mixing of the pore water with the cell water which results in diluted pore water in the case of the distilled water samples. This causes them to swell more because of the added repulsive force between the particles, as seen from the double layer repulsion calculations. Summarising the above discussion, the concentration of the pore water of these soil samples was not affected by the cell water under condition of no volume change and hence there was no dependence of the swelling pressure on the cell water, at least, within the time scale mentioned. When a small amount of swelling was allowed, by permitting some inflow of the cell water into the sample, the pore water of the distilled water sample became diluted resulting in higher amounts of swell for these samples compared to the pore water samples. This statement can be supported by analysing the composition of the pore water from two soil samples tested in a distilled water environment - one being squeezed immediately after the determination of the swelling pressure and another after allowing some amount of swell. Since no more undisturbed samples could be prepared out of the block sample of Bearpaw shale, this testing was done on samples of Morden shale which also exhibited a similar behaviour.

Morden shale samples 5-10CH5-dw and 5-11CH5-dw, having an initial pore water concentration of 270 meq/l were tested for their swelling pressure at constant volume. These samples were placed in their respective consolidation rings, using a grease other than the silicon spray grease used with the other samples. This resulted in friction between the ring and the sample preventing the soil from swelling. These two samples did not start swelling immediately on contact with water in the consolidometers. To make sure it was due to side friction, the author increased the load on these samples to see whether it would cause consolidation. This had no effect on the soil samples up to quite large pressures, because the applied load was carried mainly by the side friction and not by the sample itself. For these two samples, the swelling pressure was defined as that load above which the sample started to consolidate. Thus the pressures measured were the true swelling pressure of the sample plus the side friction and for that reason the swelling pressure of these two samples are found to be more than for the other Morden shale samples from the same depth. But, even though the swelling pressure thus measured is erroneous, at all times the sample volume had been maintained constant for both of these samples, which was the prime requirement of this test. After determining the swelling pressure, the cell water was drained out and the sample removed from the apparatus as quickly as possible. These samples were squeezed and the salinity of their pore waters was determined using the solu bridge described in Chapter IV. The pore water concentration of the two samples was found to be 240 and 270 meq/l. The concentration of the cell water from the two tests was found to be 0 and 2.5 meq/l. These values indicate that the concentration of the pore water and the cell water have not changed during the determination of the swelling pressure at constant volume. Two other samples of Morden shale, 5-8-CH5-dw and 5-9-CH5-dw, were tested for their swelling pressures. The magnitudes of swelling pressure were 2.89 tsf and 3.07 tsf respectively. Sample 5-8-CH5-dw was then allowed to swell 1% and the equilibrium pressure was found to be 1.97 tsf. The sample was then squeezed and the pore water concentration was found to be 14 meq/l. Sample 5-9-CH5-dw was allowed to swell by 0.15% and was then consolidated back to its original volume and the new equilibrium pressure at original volume was found to be 3.75 tsf, indicating an increase in swelling pressure of 0.68 tsf due to the added component of double layer repulsive force associated with the dilution of the pore water of the soil sample. When squeezed, the pore water was found to have 20 meq/l salt in it. The concentration of the cell water at the end of the test was found to be 14 and 15.5 meq/l for samples 5-8-CH5-dw and 5-9-CH5-dw respectively. These results indicate that during the determination of swelling pressure at no volume change the pore water of the sample was not affected by the cell water within the time allowed in the laboratory (from 1-6 weeks for these and other samples). Once a very small volume change is permitted, allowing an inflow of cell water into the soil sample, there is a rapid mixing of the pore water with the cell water. It shall be noted that the final concentration of the pore water of samples 5-8-CH5-dw and 5-9-CH5-dw is not due to the entry of an amount of distilled water from the cell, equal to the allowed increase in the volume of the sample, but is due to a very fast diffusion of the salts of the pore water in all the cell water. This results in a final salt concentration of the pore and cell water that is essentially equal. The results of tests on samples 5-8-CH5-dw, 5-9-CH5-dw, 5-10-CH5-dw and 5-11-CH5-dw, described above are tabulated in Table VII.4.

The behaviour described above may result from two possible reasons described in the following pages.

(i) Slow diffusion rate, and

(ii) Partial sealing of the surface of the soil sample due to the method of sample preparation.

The coefficient of diffusion of the homogeneous - Block Bearpaw and Morden-shale samples may be very low. This would result in equal magnitudes of swelling pressure for these samples in distilled water and their own natural pore water since the concentration of pore water of these samples remains unchanged within the time allowed (up to six weeks). A direct measurement of the diffusion coefficient (D) of these shales has not been made. Therefore, D can only be estimated from other published data on clayey materials. Rosenquist (1955) has given the diffusion coefficient of three illitic materials at a water content of 30%. He conducted diffusion tests using a heavy water. The Aserum clay had a D value of 0.014 ft^2/year (0.034 cm^2/day) and a specific surface of 130 m^2/gm and the clay from Torup had a D value of 0.02 ft^2/year (0.05 cm^2/day) and a specific surface value of 30 m^2/gm. If the size (area) of the pores alone controls the diffusion coefficient, then the D values should vary as the inverse of the square of their specific surfaces. Using the (weighted) specific surface areas of Bearpaw shale of 260 m^2/gm, the diffusion coefficient for the block samples of Bearpaw shale can be calculated as 0.0035 or 0.00024 ft^2/year. Even using the lower value of D, in a period of 6 weeks (during which the magnitude of the swelling pressure of sample 1-4-CH5-dw did not increase) the average pore water concentration of a sample of thickness 3/4 in. with double drainage should have dropped to about 80% of its original value. This dilution of the pore water would yield a double layer repulsive force of approximately 0.04 tsf. This means that, in a period of 6 weeks, the Block Bearpaw shale samples in distilled water should have increased their swelling pressures by about 0.04 tsf. The fact that this did not happen indicates that the coefficient of diffusion estimated is too high, at least by one or two orders of magnitude. This is quite possible because the Bearpaw shale sample has a very high cation exchange capacity compared to Rosenquist's samples and the distribution of the fines in the pore space of larger particles would also differ between the two soils. The influence of assuming lower diffusion coefficients is clearly demonstrated in Figs. 7.28 and 7.29. Fig.7.28 shows the variation of average salt content within the sample against time and Fig. 7.29 illustrates the corresponding increase in the double layer repulsive force, p_y, (calculated as explainedin Chapter VIII and Appendix A) with time (for a sample 3/4 in. thick with double drainage). From these figures, one can see that acceptance of a lower diffusion coefficient might account for no increase of the swelling pressure of sample 1-4-CH5-dw and why the concentration of the pore water of samples 5-10-CH5-dw and 5-11-CH5-dw did not change in a comparatively short time. But the assumption of such a low diffusion coefficient does not explain how the pore water concentration of samples 5-8-CH5-dw and 5-9-CH5-dw suddenly decreased to a final concentration equal to that of the ambient water, with a small (0.15% to 1.0%) amount of swell. Such a small volume increase, could not possibly be assumed to have opened up flow channels throughout the sample which were initially small.

The orientation of the soil particles, parallel to the surface of the sample caused by the cutting of the sample by a blade (Martin, 1966) could account for the peculiar phenomena discussed above. Such an orientation of the clay particles on the surface might possibly create a temporary seal, on the surface of the soil specimens against diffusion of salts. It has been shown later that these shale samples are made up of clusters (of closely packed clay particles) and a large intercluster region most of which is filled with free pore water. The openings of the free pore space for such an arrangement would be much larger than for a soil with a homogeneous (at microscopic level) fabric. This would result in a much larger diffusion rate. But the diffusion for the block Bearpaw shale and the Morden shale samples have been found to be negligible during the determination of swelling pressure at constant volume. An orientation of the clay particles parallel to the surface, would reduce the coefficient of diffusion in a direction perpendicular to the direction of orientation. Since the samples were almost fully saturated, there was no mass inflow of ambient water during the swelling pressure measurement at constant volume. Once a small swell is permitted, the inflow of water could have possibly opened some flow channels through the surface. This would result in a much faster diffusion and hence a dilution of the pore water of the soil samples in a much smaller time. Fig. 7.28 and 7.29 show calculations on the decrease in salt content and a corresponding increase in the double layer repulsive force with arbitrarily assumed coefficients of diffusion higher than the ones predicted from Rosenquist's (1955) data. These figures clearly indicate that a higher diffusion rate could account for the fast dilution of the pore water of the samples 5-8-CH5-dw and 5-9-CH5-dw once a small swelling has been permitted. The presence of such a temporary seal initially which gets opened on permitting a small volume change can be investigated through a direct permeability test. The orientation of the soil particles on the surface of the sample will result in a much smaller permeability of the soil which would increase once the surface seal is broken. This test should be conducted with very great precision at controlled temperature and humidity conditions using a very small hydraulic gradient. Such a direct permeability test was not performed in this study. It should be emphasized that the time required for this test for the shales with very low permeability in the order of 10^{-7} to 10^{-11} cm/sec (Peterson, 1954) is very large.

The tests on the intact Bearpaw shale sample (discussed later) in which almost full diffusion occurred within the time required for the development of swelling pressure also supports the above described hypothesis. These samples had sand or silt seams on their surface. The absence of a continuous seal of oriented clay particles on their surface has resulted in a much faster diffusion for these samples comparable to that of the Block Bearpaw shale and Morden shale once their surface seal is opened up.

The nature of the pressure-swell relationship for the Bearpaw shale samples is also of interest. The results for samples 1-4-WF4-DW and 1-4-CH5-dw on a semi-log plot (Fig. 7.6) show a straight line relationship for the former sample whereas it is a curved line for the distilled water sample. A detailed discussion of this figure is reserved for the next chapter, where they are considered in terms of the general mechanism of swelling that is introduced.

7.3 The Edmonton Shale

Only one sample 2-5-WF4-dw was prepared from this material for the study of its swelling characteristics. This sample has been obtained from a depth of 30 ft. below ground level. The natural moisture content of the sample was found to be 21.9% and the degree of saturation 100%. The sample did not swell on contact with water. The comparison of the various properties of the soil samples tested in this programme indicates the reason for such behaviour. Table III.1 shows that the Edmonton shale and the intact Bearpaw shale have almost similar % clay, CEC and specific surface values, and therefore probably similar mineralogical compositions, whereas the liquid limit and plasticity index of the Edmonton shale are much smaller than those of the intact Bearpaw shale. The answer for this difference lies in the method of preparation of the samples for each of these tests. The addition of the deflocculating agent and the treatment with H_2O_2 prior to the hydrometer analysis, subsequent washings of this soil with ammonium acetate and ethanol for the CEC analysis and repeated washings and centrifuging with calcium chloride solutions and alcohol-distilled water mixtures for the specific surface analysis breaks down the soil into single particles, i.e. breaks all interparticle cementation. The samples prepared for the liquid limit tests, which have not been subjected to such severe pretreatments, would retain a major proportion of any cementation that was present in the natural conditions. Slaking tests done on this material have shown increases in water content as would be expected from its plasticity index values (Eigenbrod, 1972). Treatment of the various shales with hydrochloric acid indicated presence of large proportions of $CaCO_3$ in this Edmonton shale compared to the other shales. The presence of any such cementation would tend to suppress its swelling characteristics. It was found that the liquid limit of this shale after treatment with concentrated (5N) hydrochloric acid did not increase over the initial value. Even though all the calcium carbonates cementation would likely have been destroyed by the acid treatment the author feels that some of the montmorillonites might have also been dissolved by this severe treatment (Jackson, 1960).

7.4 Bearpaw shale-weathered

Two samples from depths 16-23 ft.(3-5-CH5-dw and 3-5-CH6-dw) having initial water contents of about 29% were tested in distilled water and they gave swelling pressures of 0.18 and 0.14 tsf respectively. The magnitudes of swelling pressure were so small that this material was not used for any further studies. These samples were sandy (containing about 20% of material retained in No. 200 sieve) and were quite broken. The water content of the sample might have increased during sampling and storage, and thus the samples would give very low swelling pressures compared to what would be expected of a sample from these depths. These materials were quite variable because of weathering and any comparison of data from them would have been difficult.

7.5 Bearpaw shale - intact

(i) Samples 4-7-CH6-pw, 4-6-CH7-dw, 4-1-Ant8-pw, 4-1-Ant9-pw, 4-1-Ant10-dw and 4-1-Ant11-pw

The next set of tests conducted were on intact Bearpaw shale samples from depths of 68-69.9 ft. and 72-73 ft. Two samples 4-7-CH6-pw and 4-6-CH7-dw were prepared from the depth 68-69.9 ft. and were tested in their own natural pore water and distilled water respectively. The swelling pressure and the rate of development of swelling pressure were noted. They were then subjected to stress-controlled swelling tests to define their pressure-swell relationship. The swelling under any pressure is continued until the primary part of swelling is complete and the secondary swelling well defined (as seen in a $\sqrt{t}$ plot). The results of these tests are tabulated in Table VII.5. Fig. 7.8 shows the development of swelling pressure with time for these samples and Figs. 7.9a, b and c show the three stages of swelling of these samples. The coefficients of swelling pressure and of swelling determined by the $\sqrt{t}$ and the log t fitting methods were very close in all cases. (See for example Table VI.1, VII.1, VII.2 and VII.3.) Therefore only the $\sqrt{t}$ plots and the coefficients derived from them have been presented for these samples and for the other samples described later. The pressure-swell relationship is shown in Fig. 7-12.

Samples 4-1-Ant8-pw, 4-1-Ant9-pw, 4-1-Ant10-dw and 4-1-Ant11-dw were prepared from the depths of 72-73 ft. and were subjected to similar tests as above. The results are given in Table VII.6 and Figs. 7-10, 7-11a, b, c and 7-12. During the course of the stress-controlled swelling test, samples 4-1-Ant10-dw and 4-1-Ant11-dw were disturbed and these tests had to be discontinued.

(ii) Discussion of results on intact Bearpaw shale

The samples of intact Bearpaw shale used in the swelling tests were less homogeneous materials than the block sample of Bearpaw shale discussed earlier. These samples contained a number of silt or sand seams in a clay matrix. In materials like this, migration of water from the silt to the clay matrix cannot be avoided, especially if the samples have been kept for a long time after removal from ground. This would result in a slightly higher water content in the clay zone compared to the silty or sandy region. Such a sample when tested for its swelling pressure in the conventional consolidometers using coarse porous stones, would draw some water into the sample during setting up of the sample, resulting in a much smaller swelling pressure than the actual. (Using such coarse porous stones with the more homogeneous clayey shales do not really cause a problem. The permeability of those shales is so low, of the order of 10^{-9} to 10^{-11} cm/sec. that in the time required for setting up a sample in these machines significant inflow of water does not take place.) Even though no details regarding the ground water table were available to the author, he would have expected much higher swelling pressures than the ones measured from these samples. The use of porous stones with high air entry values (greater than the anticipated swelling pressure) might have resolved this problem. But a migration of the water from the silt or sand zone into the clay matrix could not be avoided in this case, because these samples had been tested after a time lapse from removal from the ground. A careful measurement of the water contents in the silt zone and the clay matrix indicated that the water content in the clay zone was about 1.5% greater than that of the silt zone. Fig. 7.13, shows the relation between the initial water content of the clay zone and the measured swelling pressure of three samples in their natural pore water. The figure shows an almost linear relationship on a semi-log plot, with the swelling pressure decreasing with increasing water content. This line has a slope of 0.043 which is comparable to the swell index of the sample 1-4-WF4-pw (0.075 from Fig. 7.6).

All of these samples of intact Bearpaw shale had sand or silt seams on their surface. The higher swelling pressure measured on these samples in distilled water environment compared to the pore water environment must have resulted due to the dilution of the pore water of the former samples. This dilution during the constant volume swelling pressure test, the author suspects, must have resulted from a faster diffusion through the silt seams on the surface of these samples.

Fig. 7-12 shows the percent swell versus log of pressure for all of the five intact Bearpaw shale samples and sample 1-4-WF4-pw. The slope of this curve for sample 1-4-WF4-pw can be seen to be very high compared to the others. This sample has been prepared out of the block sample of Bearpaw shale whereas the others have been prepared from deep intact Bearpaw shale (driven) samples. Besides the difference between the soil types, the major factor that accounts for the difference in their slopes is that sample 1-4-WF4-pw has been subjected to a strain-controlled swelling test and the others to stress-controlled swelling tests. In the strain-controlled swelling test, the samples are allowed to swell a known amount and the equilibrium pressure at that level determined. This load is maintained on the sample for sufficient length of time (at least about a week) to ascertain that at that pressure no further volume change takes place. In the stress-controlled swelling test, the pressure on the soil sample is decreased in steps and the dial readings are noted with time until the primary part of swelling is complete and the secondary swelling well defined. Given more time, at the same pressure, the samples would have swelled more. Thus the percent swell that is determined in a stress-controlled swelling test is not the ultimate swell at that pressure, whereas the strain-controlled test gives the ultimate swell at the corresponding equilibrium pressures. Results of tests on samples 5-10-CH6-dw and 5-10-CH7-dw, discussed later in this chapter, substantiate the statement made here that the final volume reached by the samples subjected to a stress-controlled swelling test is very close to the predicted one from a strain-controlled test on the same material. Hence, the slope of the line for sample 1-4-WF4-pw is steeper than the others in Fig. 7-12. Samples 4-1-Ant10-dw and 4-1-Ant11-dw are swelling tests in distilled water and therefore yield steeper lines than samples 4-7-CH6-pw, 4-1-Ant8-pw and 4-1-Ant9-pw, as expected. The stress-controlled swelling test does have one advantage over the strain-controlled test, because it facilitates the determination of the coefficients of swelling of the soil sample at various stress levels.

The coefficients of swelling pressure, c_{sp}, determined from the swelling pressure development test and the various coefficients of swelling at various stress levels for samples 4-7-CH6-pw, 4-6-CH7-dw, 4-1-Ant8-pw, 4-1-Ant9-pw, 4-1-Ant10-dw and 4-1-Ant11-pw have been tabulated in Tables VII.5 and VII.6. From this table, one can see that there is no special trend in the coefficient of swelling with stress release and that c_{sp} is always an underestimate of the true rate of swelling. This may be partly attributable to the non-homogeneity of these samples. The time required for setting up samples in the consolidometers would influence the rate of development of swelling pressure as well as the magnitude. This effect would be amplified in the case of samples with high coefficients of permeability or with the samples like the intact Bearpaw shale mentioned above which have sand or silt seams on their surface. Hence, as the results indicate, a conventional swelling test on these samples will be more indicative of the true coefficient of swelling unless high air entry value porous stones are used. Comparison of Tables VII.5 and VII.6 show a more uniform variation of the coefficient of swelling for the samples included in the latter table, and this reflects the advantage of using teflon-coated frictionless rings over the consolidation rings of the conventional consolidometers. The coefficient of swelling seems to decrease gradually with decreasing stress for samples 4-1-Ant8-pw, 4-1-Ant9-pw, 4-1-Ant10-dw and 4-1-Ant11-dw.

7.6 Morden Shale

(i) Samples 5-8-CH6-dw and 5-8-CH7-pw

Two samples of the Morden shale, 5-8-CH6-dw and 5-8-CH7-pw were prepared out of 3" diameter drive samples from depths 9.5 - 10.3 ft. These samples were tested for their magnitudes of swelling pressure, the rate of development of swelling pressure and the pressure-swell relationship using a strain controlled test. The magnitudes of swelling pressure were almost the same for both of these samples. (3.00 tsf for 5-8-CH6-dw and 3.07 tsf for 5-8-CH7-pw.) The sample 5-8-CH6-dw was then allowed to swell 0.3% of its original volume and was then consolidated back to its original volume. The new equilibrium pressure at original volume had now increased to 3.5 tsf. The small amount of swelling resulted in a dilution of the pore water which accounts for an additional swelling pressure component due to the additional double-layer repulsive force. The coefficients of swelling pressure estimated from these swelling pressure development tests were found to be 5.92 ft^2/year for sample in distilled water and 6.90 ft^2/year for sample in its own pore water (Fig. 7.14). The pressure-swell relationship using the strain-controlled swelling test is performed as follows. The final dial reading to be reached corresponding to any expected percent swell is computed. The pressure on the sample is reduced to zero and the sample is allowed to swell. When the deflection dial reading has reached the anticipated value, loads are gradually applied to the sample keeping track of the time of application of these loads. (The time, at which the deflection dial gauge read the estimated final dial reading, was taken as zero time and the time of application of various loads to maintain constant volume was counted from that time.) This process is continued until the final equilibrium load - at which no further volume change takes place is determined. A plot of the development of equilibrium pressure with time is used to evaluate the coefficient of swelling at the corresponding (equilibrium) pressure level (coefficient of equilibrium pressure, c_{ep}) in a manner similar to determining the coefficient of swelling pressure, c_{sp}, from the swelling pressure development test. (The value of T_{50} for the development of equilibrium pressure is 0.27 against 0.194 for the evaluation of c_{sp} since the initial pore water pressure distribution would be sinusoidal for the development of equilibrium pressure test.) A sample curve of equilibrium pressure development is shown in Fig. 7.15. The results of these tests on samples 5-8-CH6-dw and 5-8-CH7-pw have been tabulated in Table VII.7. Fig. 7.16 gives the percent swell against log of pressure for these samples.

(ii) <u>Samples 5-9-CH6-dw and 5-9-CH7-pw</u>

Two more deep samples of Morden shale (from 10.5 - 11.3 ft.) were tested in distilled water and its natural pore water. The magnitude of swelling pressure and the rate of development of swelling pressure were determined as for earlier samples. The pressure-swell relationship of this sample was obtained using stress-controlled swelling tests so that the coefficients of swelling of these samples at various stress levels may be calculated and be compared with the coefficients of swelling predicted from the development of equilibrium pressure test for samples 5-8-CH6-dw and 5-8-CH7-pw. The results of tests on samples 5-9-CH6-dw and 5-9-CH7-pw are tabulated in Table VII.8. The development of swelling pressure with time is plotted in Fig. 7.17 and the pressure swell relationship is shown in Fig. 7.16. The results of the stress controlled swelling tests are shown in

Figs. 7.18a, b, c and d.

(iii) Samples 5-10-CH6-dw and 5-10-CH7-dw

Two samples of Morden shale were prepared from depths 8.5 - 9.3 ft. and tested in distilled water to study the effect of the amount of unloading on the coefficient of secondary swelling. The swelling pressure of these samples was first determined and then the samples 5-10-CH6-dw and 5-10-CH7-dw were allowed to swell to 0.3 and 0.1 of their respective swelling pressures. The coefficient of swelling pressure and coefficient of swelling were evaluated and are given in Table VII.9. The test results are shown in Figs. 7.19 and 7.20.

(iv) Sample 5-14-CH5-dw

A sample of Morden shale from a depth of about 5 ft. was prepared and tested in distilled water. Careful determination of the moisture content was made to investigate whether there is any correlation between the observed swelling pressure and the swelling pressure that would be predicted for this sample from that of samples 5-8-CH6-dw and 5-9-CH6-dw and their water contents. The rate of development of swelling pressure was also measured for this sample. The magnitude of swelling pressure was found to be 1.07 tsf. The coefficient of swelling pressure was 1.13 ft^2/year from a $\sqrt{t}$ plot. (Fig. 7.21.) The sample was then subjected to stress controlled swelling tests. The pressure-swell relationship is given in Fig. 7.22. The results on this sample are tabulated in Table VII.10.

(v) Discussion of results of tests on the Morden shale samples

The samples of Morden shale used in this programme were uniform, homogeneous and intact. They showed no dependency of the magnitude of the swelling pressure on the composition of the cell water, as was the case with the block sample of Bearpaw shale. Following arguments similar to those put forward in Section 7.2, this would mean that the surface of these samples did not act like a semi-permeable membrane and possibly (a major portion of) its pore water was in some sort of equilibrium with interparticle bonds making it immobile during the constant volume swelling pressure determination. It has already been mentioned in Section 7.2 that once a small amount of swelling is allowed, a rapid mixing of the pore water with the cell water takes place. This causes a dilution of the pore water of the sample. This dilution increases the swelling pressure of the distilled water sample by an amount equal to the added double-layer repulsive force. This accounts for the increase of the swelling pressure of sample 5-8-CH6-dw from 3.00 to 3.50 tsf. Once the pore water of the distilled water sample has been diluted they consistently give higher equilibrium pressures at equal strains than the sample in its own pore water. The pressure-swell relationship of samples 5-9-CH6-dw and 5-9-CH7-pw are much flatter than those for samples 5-8-CH6-dw and 5-8-CH7-pw since the latter samples were subjected to strain-controlled swelling tests, whereas the others were subjected to stress-controlled swelling tests. From Fig. 7-16, it is evident that the sample in its own pore water (5-8-CH7-pw) gives a straight line relation whereas the distilled water sample shows a curved line relationship. The next chapter includes a discussion of this curve.

Table VII.7 and Fig. 7-23 give the values of coefficients of swelling at various stress levels estimated from the rates of development of swelling pressure or the equilibrium pressures. The magnitude of the coefficient of swelling can be seen to decrease with decreasing stress levels. With decreasing stress, the resulting increase in the sample volume would be expected to give higher permeabilities, but the increase in the value of the coefficient of volume swelling may account for the decrease in the coefficients of swelling.

Table VII.8 includes the results of tests on samples 5-9-CH6-dw and 5-9-CH7-pw. The natural moisture content of these samples was found to be 18.8%. These samples have been obtained from beneath the spillway slab of the Morden dam. The excavation for the construction of the spillway inlet channel was made in 1953. The water content of the samples as of that date, at the elevation from which samples 5-9-CH6-dw and 5-9-CH7-pw have been obtained, was about 20%. Compared to that, the soil should have increased in water content due to swelling caused by the spillway excavation. On the contrary, the water content of these samples are seen to be less than the moisture content as of 1953. This might possibly be due to some drying before this testing, and this explains a higher magnitude of swelling pressure. The coefficient of swelling pressure, c_{sp}, estimated from the swelling pressure development test (= 6.55 ft^2/year) is of the same order as for samples 5-8-CH6-dw and 5-8-CH7-pw. The coefficients of swelling obtained from the stress-controlled test for samples 5-9-CH6-dw and 5-9-CH7-pw are also shown in Figs. 7-23. It could be seen that even though there is a great amount of scatter of the values of coefficients of swelling, the coefficients calculated from the stress-controlled test are always in excess of the values obtained through the strain-controlled swelling test from the rate of development of equilibrium pressure. Samples 5-8-CH6-dw and 5-8-CH7-pw being subjected to strain-controlled swelling test, would yield a greater amount of swell at any pressure compared to samples 5-9-CH6-dw and 5-9-CH7-pw. This would enhance the coefficient of volume swelling, m_{vs}, value of the samples 5-8-CH6-dw and 5-8-CH7-pw resulting in a lower coefficient of swelling provided the change in the permeability between these sets of samples is less pronounced than the change in the value of m_{vs} . Further, during the stress-controlled test, each stress is being maintained only until the secondary swell at that stress is clearly defined. As mentioned earlier, complete strain under that stress has not been attained. This would surely affect the coefficients of swelling under subsequent stress levels, which would then be a function of the present overburden stress and the previous (unloading) stress history of the soil. If this is true, the coefficients of swelling evaluated from the strain controlled swelling test is more representative of the true coefficient of swelling for the soil sample. Further, using the stress-controlled swelling test, one finds that the primary part of swelling takes place quite fast, but only for a short while, with the secondary swelling continuing at a slow rate for a long time. In the strain-controlled test, the coefficient of swelling calculated gives a constant uniform rate at which the full equilibrium pressure is being mobilised; and this might probably be considered as a weighted average of the primary and secondary rate of swelling. For this reason, the author feels,that may be it is this coefficient of swelling that one has to take into account in predicting long time field swell. The discrepancy found between these two sets of values is so small that a choice of either may not lead to significant errors, at stress levels closer to the swelling pressure.

The c_{sp} estimated for samples 5-10-CH6-dw and 5-10-CH7-dw are comparable to that of the other Morden shale samples. The coefficient of swelling of both these samples was very much smaller than the c_{sp}. These samples have been subjected to very large decrements of pressure compared to the other Morden shale samples and therefore have yielded much smaller coefficients of swelling than samples 5-8-CH6-dw and 5-9-CH6-dw. The sample 5-10-CH7-dw which was subjected to a larger pressure decrement than the sample 5-10-CH6-dw shows a lower c_{vs} than the latter sample.

Fig. 7.22 gives the swelling pressure of the samples 5-8-CH7-dw, 5-9-CH7-pw and 5-14-CH5-dw against their initial water content. A straight line relationship is seen as expected. But the slope of the line gives an index of 0.088 against the C_s of 0.075 for these materials from the strain-controlled swelling data. The difference is within allowable experimental error.

7.7 The swelling characteristics of compaction shales: a discussion of results.

In the present study, swelling pressures of three compaction shales were measured in distilled water and their natural pore water environment. The Bearpaw shale block and the Morden shale samples gave equal swelling pressure in both of these cell waters. If the response of the swelling pressure is considered to be due to the negative pore water pressure (piezometric) plus the osmotic component of the pore water with respect to the outside (cell) water, the swelling pressure of the sample in distilled water should have been more than the sample in its own pore water. For the latter sample, the osmotic component would be equal to zero whereas the osmotic component of the former sample will vary depending upon the concentration of the pore water. But such an osmotic pressure would be developed only if the surface of the soil sample acts as a semipermeable membrane. Further, it was noticed that the swelling pressure of the Bearpaw shale block sample did not increase even after waiting for a reasonably long time (about 6 weeks) after full swelling pressure has mobilised. Tests on Morden shale samples showed similar trends. This means that, in periods of time of concern here, the dilution of the pore water for these samples by the external water was almost absent. This statement is supported further by the direct measurement of the pore water salinity of samples 5.10-CH5-dw and 5-11-CH5-dw, before and after the swelling pressure measurement. Thus the difference between the concentration of the pore water and the cell water has not contributed to the magnitude of the swelling pressure of these samples.

For the Bearpaw shale-intact sample, the swelling pressure in distilled water was found to be more than that for the same material in its natural pore water environment. No analysis of the pore water was made of these samples immediately after the swelling pressure measurement. But what has happened in these samples could very well be explained from the tests on sample 5-9-CH5-dw. This Morden shale sample was tested in distilled water. After measuring the swelling pressure the sample was allowed to swell by about 0.15% and then was reconsolidated back to its original volume. The swelling pressure at the original volume had by then increased to 3.75 tsf from 3.07 tsf. This sample was removed from the machine and squeezed. The concentration of the squeezed pore water was found to have decreased from its original pore water concentration of 270 meq/l to 20 meq/l. At the same time, the outside cell water, which was free of salt at the start of the test, had increased its salinity to 15.5 meq/l. This test indicates that once diffusion is less impeded, a rapid mixing of the pore water with the cell water takes place and both arrive at the same salinity. The same would have happened in the case of the Bearpaw shale intact samples. This sample was quite heterogeneous compared to the Bearpaw shale block and the Morden shale samples. The Bearpaw shale intact samples had a number of silt and sand seams present in a matrix of plastic clay. The sand seams were present in almost all cases on the sample surface itself, which would enhance the rate of diffusion. Here the final pore and cell water concentrations become equal for samples initially in distilled water in a short time. Thus the difference in the measured swelling pressure, between the samples in distilled water and natural pore water cannot be attributed to the concentration difference between the pore water and the cell water, but it rather arises from a concentration difference between the free pore water and the bound water within the soil. The availability of unbound water within the soil, at least for the soils under consideration in this study, has been established in Chapter IV. With reference to Fig. 4.3, if the concentration of the water squeezed out is (practically) independent of the squeezing pressure at pressures less than 3,000 psi, it implies that the water that comes out at these pressures is not chemically bound to the soil.

Scanning electron microscope photos were taken on Morden shale samples* from a depth of about 11 ft. and 5 ft. and these photos are shown in Fig. 7.25. The soil sample could be seen from these photos as made up of clusters (a close assembly of few clay particles) with large intercluster void spaces (Fig. 7.25a, b, d and e)**. A general orientation of the soil particles (clusters) is also seen in photos 7.25d and e. Between the two soil samples the one from a depth of 5 ft. showed a large quantity of tiny gypsum crystals in the large void spaces (Fig. 7.25c). Some of the soil particles from the 11 ft. depth can be seen to be bent still retaining some of the locked in

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*These samples were prepared by the freeze drying method (for eg. Gillott, 1969). A detailed description of sample preparation can be obtained from Chattopadhyay (1972).

**From tests to study the hydraulic conductivity of saturated clays, Olsen (1960) concluded that the soil particles are not oriented within the soil sample with uniform spacing but are present as clusters formed by several particles (Fig. 7.24).

energy noted by Bjerrum (1967). (Fig. 7.25f and g). The absence of such bent particles, and the presence of a large amount of gypsum crystals in the top soil indicate possible weathering up to about 5 ft. below the present ground level in this shale.

7.8 The rate of secondary swell

Those samples subjected to a stress controlled swelling test have been maintained at different stress levels until the primary part of swelling was complete and the secondary swelling well defined. In a plot of amount of swell against square root of time the (initial part of the) secondary swelling yields a straight line, which has been used to calculate the rate of secondary swelling to see whether it bears any relation to the other properties of the materials tested. The coefficient of secondary swelling has been calculated herein as follows.

$$\text{coefficient of secondary swelling} = \frac{(\%\ \text{swell})_1 - (\%\ \text{swell})_2}{t_1^{1/2} - t_2^{1/2}}$$

where $(\%\ \text{swell})_1$ corresponds to time t_1 and $(\%\ \text{swell})_2$ to t_2. This definition by the above definition has a dimension of $(t)^{-1/2}$. Such coefficients of secondary swelling have been calculated for all the samples subjected to stress controlled swelling. These values have been tabulated in Table VII.11.

Samples of Bearpaw shale block which have been subjected to free swell tests have yielded values of coefficients of secondary swelling in the range of 15 to 25 x 10^{-3} $\text{min}^{-1/2}$. Samples of intact

Bearpaw shale gave much smaller values for their first stage of unloading with the load decrement ratio of about 0.3. The coefficient increased for subsequent stages of unloading. A similar trend is also seen with the Morden shale samples (5-9-CH6-dw and 5-9-CH7-pw). The samples of Morden shale 5-10-CH6-dw and 5-10-CH7-dw which are subjected to larger load decrement ratio yielded comparatively higher coefficients of secondary swelling than samples 5-9-CH6-dw and 5-9-CH7-pw which were subjected to a load decrement ratio of 0.35

In these tests, as explained earlier, each pressure has been maintained on the samples only up to the time required to define the slope of the secondary swelling curve. This means that the total swell under any particular stress was not complete before it was subjected to a subsequent unloading. Therefore the coefficients of secondary swelling for the first unloading would be more representative of the material and the coefficients of secondary swelling for the subsequent unloadings do not represent the true secondary swelling at those stresses.

In general, in all these soils, the magnitude of the coefficient of the secondary swelling does not seem to be very much affected by the composition of the cell water.

Fig. 7.26 shows the variation of the coefficient of secondary swelling plotted against the load decrement ratio on a natural scale and the same data plotted in a log-log scale are shown in Fig. 7.27. A straight line relationship is seen in the log-log plot for the deep Morden shale samples in the range of the load decrement ratio tested in this programme.

TABLE VII.1

RESULTS OF TESTS ON SAMPLES 1-2-WF4-pw AND 1-2-CH5-dw

SWELLING TESTS

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of satu. %	Cell water composition salts in 1000 cc	Swelling Pressure tsf	Actual Time for 100% Sw. Pr. Minutes	Free Swell in 10 days	Coeff. of Swelling c_{vs} ft^2/year	$\frac{\text{Sec. swell}}{\text{prim. swell}}$ ratio
1-2-WF4-pw	0.625	31.8	0.901	100	8.25g Na_2SO_4+ 13.75g $MgSO_4$	1.75	-	13	$0.27^*(0.28)^+$	0.20
1-2-CH5-dw	0.705	31.8	1.024	88	No salt	1.71	-	15	$0.40^*(0.30^+)$	0.13

CATION CONCENTRATION STUDY

Sample	$Na^@$	$K^@$	$Mg^@$	$Ca^@$
Original cell water: 1-2-WF4-pw	3250	0	1950	0
Final cell water : 1-2-WF4-pw	5100	40	2538	212
Original pore water: both samples	2643	39	221	331
Final pore water : 1-2-WF4-pw	3571	54	1286	518
Final pore water : 1-2-CH5-dw	395	9	10	40

* values found from the $\sqrt{t}$ fitting method.
\+ values found from a log t fitting method.
@ all values in ppm.

TABLE VII.2

RESULTS OF TESTS ON SAMPLES 1-3-WF4-pw AND 1-3-CH5-dw

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of Satu. %	Cell water composition Salts in 1000 cc	Swelling Pressure tsf	Coeff. of Swelling pressure c_{sp} ft^2/year	Actual time for 100% Sw. Press. mins.	Free Swell in 7 days %	Coeff. of Swelling C_{vs} ft^2/year	$\frac{\text{Sec. Swell}}{\text{Prim. Sw.}}$	Final degree of Satu. %
1-3-WF4-pw	0.75	30.5	0.951	90.8	8.25gNa_2SO_4+ 2.75g $MgSO_4$	1.38	7.70* (8.2)	750	11	0.17*	0.10	100
1-3-CH5-dw	1.0	30.3	0.974	88.0	No salt	1.32	1.77* (1.66)	2000	15	0.495*	0.05	100

* coefficients calculated from the $\sqrt{t}$ fitting method.
( ) coefficients obtained from the log t fitting method.

TABLE VII.3

RESULTS OF TESTS ON SAMPLES 1-4-WF4-pw AND 1-4-CH5-dw

SWELLING TESTS

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of Satu. %	Cell water Composition salts in 1000 cc	Swelling pressure tsf	Actual Time for 100% Sw. Press mins.	Coeff. of Sw. Pr. c_{sp} ft^2/year
1-4-WF4-pw	0.625	30.5	0.897	96.2	8.25gNa_2SO_4+ 2.75g$MgSO_4$	2.07	1000	0.62* (0.61)
1-4-CH5-dw	0.75	30.5	0.909	95.0	No salt	2.00	1900	0.62* (0.62)

Sample	Equilibrium pressures for swells of 2%	4%	6%	8%	10%	12%	14%
1-4-WF4-pw	0.60	0.21	0.07	0.02	-	-	-
1-4-CH5-dw	1.16	0.79	0.54	0.46	0.34	0.23	0.20

CATION CONCENTRATION STUDY

Sample	Salinity# Normality	Na@	K@	Ca@	Mg@
Original conc. of pore water	0.130	2643	39	221	331
Cell water: 1-4-CH5-dw- Dec.1969	0.020	366	10	4	15
Cell water: 1-4-CH5-dw- Aug.1970	0.019	250	5	7	6
Final conc. of pore water: 1-4-CH5-dw	-	264	2	7	5
Original cell water:1-4-WF4-pw:Nov.1969	0.150	2925	1	31	-
Cell water: 1-4-WF4-pw-Dec.1969	-	5500	70	375	2812
Cell water: 1-4-WF4-pw-Aug.1970	-	1850	75	51	825
Final conc. of pore water: 1-4-WF4-pw	-	2000	56	340	950

* coefficients calculated from the $\sqrt{t}$ fitting methods

\# from solu bridge measurements

( ) coefficients calculated from the log t fitting methods

@ all values in ppm

TABLE VII.4

STUDY ON DILUTION OF PORE WATER OF MORDEN SHALE SAMPLES

Sample	Depth below ground level ft.	Initial water content %	Swelling pressure tsf.	Squeezed at % vol. change	Eqm. pr. at the final vol. tsf.	Final pore water composition meq/l	Final cell water composition meq/l
5-8-CH5-dw	9.5-10.3	20.09	2.89	1.0	1.97 at 101% vol	14	14
5-9-CH5-dw	9.5-10.3	20.45	3.07	0.15	3.75 at 100% vol	20	15.5
5-10-CH5-dw	8.5-9.3	20.68	4.14*	0.	-	240	0
5-11-CH5-dw	10.5-11.3	18.82	5.21*	0.	-	270	2.5

* high swelling pressures measured due to friction of the consolidometer ring

TABLE VII.5

RESULTS OF TESTS ON SAMPLES 4-7-CH6-pw AND 4-6-CH7-dw

SWELLING TESTS

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of satu. %	Cell water composition salts in 1000 cc	Swelling Pressure tsf	Actual Time for 100% Sw. Press. ins.	Coeff. of Sw. Press* c_{sp} ft^2/year
4-7-CH6-pw	0.625	25.1	0.715	99.8	9.26gNa_2SO_4+ 2.51g $MgSO_4$	0.19	4700	0.52
4-6-CH7-dw	0.625	25.1	0.713	99.9	No salt	0.72	2000	0.85

COEFFICIENTS OF SWELLING FROM STRESS CONTROLLED TESTS

Sample 4-7-CH6-pw			Sample 4-6-CH7-dw		
Stress range tsf	c^*_{vs} ft^2/year	$\frac{\text{Sec.swell}}{\text{Prim. swell}}$	Stress range tsf	c^*_{vs} ft^2/year	$\frac{\text{Sec.swell}}{\text{Prim. swell}}$
0.19-0.12	0.67	2.50	0.72-0.48	1.25	0.80
0.12-0.06	2.79	3.50	0.48-0.28	0.70	1.40
0.06-0.01	2.79	7.50	0.28-0.01	0.70	3.30

* coefficients calculated from the $\sqrt{t}$ fitting method

TABLE VII.6

RESULTS OF TESTS ON SAMPLES 4-1-Ant8-pw, 4-1-Ant9-pw, 4-1-Ant10-dw AND 4-1-Ant11-dw

SWELLING TESTS

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of satu. %	Cell water composition salts in 1000 cc	Swelling Pressure tsf	Actual Time for 100% Sw. Press. ins.	Coeff. of Sw. Press* c_{sp} ft^2/year
4-1-Ant8-pw	0.75	23.9	0.683	99.2	9.26g Na_2SO_4+ 2.51g $MgSO_4$	1.18	1440	0.37
4-1-Ant9-pw	0.75	24.3	0.691	100	"	0.52	1440	0.39
4-1-Ant10-dw	0.75	24.3	0.691	100	no salt	1.18	1440	0.43
4-1-Ant11-dw	0.75	23.9	0.678	100	no salt	1.53	1000	0.90

COEFFICIENTS OF SWELLING FROM STRESS CONTROLLED TESTS

	Sample 4-1-Ant8-pw		Sample 4-1-Ant9-pw			Sample 4-1-Ant10-dw		Sample 4-1-Ant11-dw	
Stress range tsf	c^*_{vs}	$\frac{\text{Sec. swell}}{\text{prim. swell}}$	c^*_{vs}	$\frac{\text{Sec. swell}}{\text{prim. swell}}$	Stress range tsf	c^*_{vs}	$\frac{\text{Sec. swell}}{\text{prim. swell}}$	c^*_{vs}	$\frac{\text{Sec. swell}}{\text{prim. swell}}$
1.18-0.52	2.00	0.53	-	-	Sw.Pr.-0.96	1.57	0.8	1.57	0.7
0.52-0.235	2.05	2.00	1.57	1.00	0.96-0.57@	1.24	2.7	1.24	1.6
0.235-0.01	1.00	1.50	1.24	1.60					

* coefficients calculated from the $\sqrt{t}$ fitting method in ft^2/year

@ The test had to be stopped due to disturbance to the machine

TABLE VII.7

RESULTS OF TESTS ON SAMPLES 5-8-CH6-dw AND 5-8-CH7-pw

SWELLING TESTS

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of satu. %	Cell water composition salts in 1000 cc	Swelling Pressure tsf	Actual Time for 100% Sw. Press. mins.	Coeff of Sw. Press* c_{sp} ft^2/year
5-8-CH6-dw	0.75	19.5	0.554	94.7	no salt	3.00	225	5.92
5-8-CH7-pw	0.75	19.7	0.540	97.9	13.43g Na_2SO_4+ 2.79g $MgSO_4$	3.07	200	6.90

COEFFICIENTS OF EQUILIBRIUM PRESSURE, c_{ep}

Sample 5-8-CH6-dw			Sample 5-8-CH7-pw		
% Swell	Eqm. pr. tsf	c_{ep}^* ft^2/year	% Swell	Eqm. pr. tsf	c_{ep} ft^2/year
0.0	3.5^+	5.92	0.0	3.07	6.90
0.3	3.0	-	1.0	1.68	-
1.3	1.93	-	2.0	1.14	3.37
2.3	1.39	3.37	4.0	0.45	2.56
4.3	0.79	2.24	6.0	0.18	0.38
6.3	0.46	1.09	8.0	0.07	-
8.3	0.30	1.19			

* all coefficients calculated from the $\sqrt{t}$ fitting method

\+ The sample was swelled to 100.3% volume and then reconsolidated to original volume

TABLE VII.8

RESULTS OF TESTS ON SAMPLES 5-9-CH6-dw AND 5-9-CH7-pw

SWELLING TESTS

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of satu. %	Cell water composition salts in 1000 cc	Swelling Pressure tsf	Actual Time for 100% Sw. Press. mins.	Coeff of Sw. Press* c_{sp} ft^2/year
5-9-CH6-dw	0.75	18.8	0.53	95.5	no salt	4.61	600	6.55
5-9-CH7-pw	0.75	18.8	0.52	97.4	13.43g Na_2SO_4+ 2.79g $MgSO_4$	4.63	180	6.55

COEFFICIENT OF SWELLING FROM STRESS CONTROLLED SWELLING TESTS

	Sample 5-9-CH6-dw		Sample 5-9-CH7-pw	
Stress range tsf	c^*_{vs} ft^2/year	Sec. swell / prim. swell	c^*_{vs} ft^2/year	Sec. swell / prim. swell
Sw. pr.-3.00	4.98	0.67	4.98	0.55
3.00 -1.50	5.33	0.60	4.76	1.0
1.50 -0.75	5.45	0.27	4.03	0.4
0.75 -0.30	4.37	0.10	3.95	0.25
0.30 -0.01	-		-	

* all coefficients calculated from the √t fitting method

TABLE VII.9

RESULTS OF TESTS ON SAMPLES 5-10-CH6-dw AND 5-10-CH7-dw

Sample No.	Sample thickness ins.	Initial water content %	Initial void ratio	Initial degree of satu. %	Swell press. tsf.	Actual time for 100% Sw. press mins.	Coeff. of Sw. pr.* c_{sp} ft^2/year	Load decrement ratio	Coeff. of swelling* c_{vs} ft^2/year	$\frac{\text{Sec. swell}}{\text{prim. sw.}}$	Coeff. of Sec. Sw. min$^{1/2}$ x 10^3
5-10-CH6-dw	0.75	20.0	0.552	97.3	3.75	120	11.3	0.70	0.98	0.75	16.3
5-10-CH7-dw	0.75	20.0	-	-	3.71	130	11.3	0.90	0.82	1.0	40.0

*coefficients calculated from the $\sqrt{t}$ fitting method

TABLE VII.10

RESULTS OF TESTS ON SAMPLE 5-14-CH5-dw

Initial water content	Initial void ratio	Initial degree of saturation	Composition of cell water	Swelling pressure tsf	Coeff. of sw. pr. c_{sp} ft^2/year	Coeff. of swelling for stress range in tsf. :		
						1.07-.75	.75-.37	.37-.01
21.0	0.576	98.0	dist. water	1.07	1.33	0.76	0.72	0.70

TABLE VII.11

VALUES OF COEFFICIENTS OF SECONDARY SWELLING

Sample	Initial water content	Pressure range tsf	Coeff. of sec. swelling $\text{min}^{-1/2} \times 10^3$
1-2-WF4-pw	31.8	1.75-0.01	15
1-2-CH5-dw	31.8	1.71-0.01	25
1-3-WF4-pw	30.5	1.38-0.01	20
1-3-CH5-dw	30.5	1.32-0.01	15
4-7-CH6-pw	25.1	0.19-0.12	0.53
		0.12-0.06	0.73
		0.06-0.01	2.0
4-6-CH7-dw	25.1	0.72-0.48	0.52
		0.48-0.28	1.33
		0.28-0.01	7.5
4-1-Ant8-pw	23.9	1.18-0.52	1.5
		0.52-0.235	2.1
		0.235-0.01	2.1
4-1-Ant9-pw	24.3	0.52-0.235	0.8
		0.235-0.01	1.1
4-1-Ant10-dw	24.3	1.18-0.96	1.1
		0.96-0.57	3.3
4-1-Ant11-dw	23.9	1.53-0.96	1.0
		0.96-0.57	2.2
5-9-CH6-dw	18.8	4.61-3.00	1.1
		3.00-1.50	2.1
		1.50-0.75	2.4
		0.75-0.30	4.0
5-9-CH7-pw	18.8	4.63-3.00	0.83
		3.00-1.50	3.3
		1.50-0.75	3.6
		0.75-0.30	3.0
5-10-CH6-dw	20.0	3.75-1.125	16.3
5-10-CH7-dw	20.0	3.71-0.37	40.0

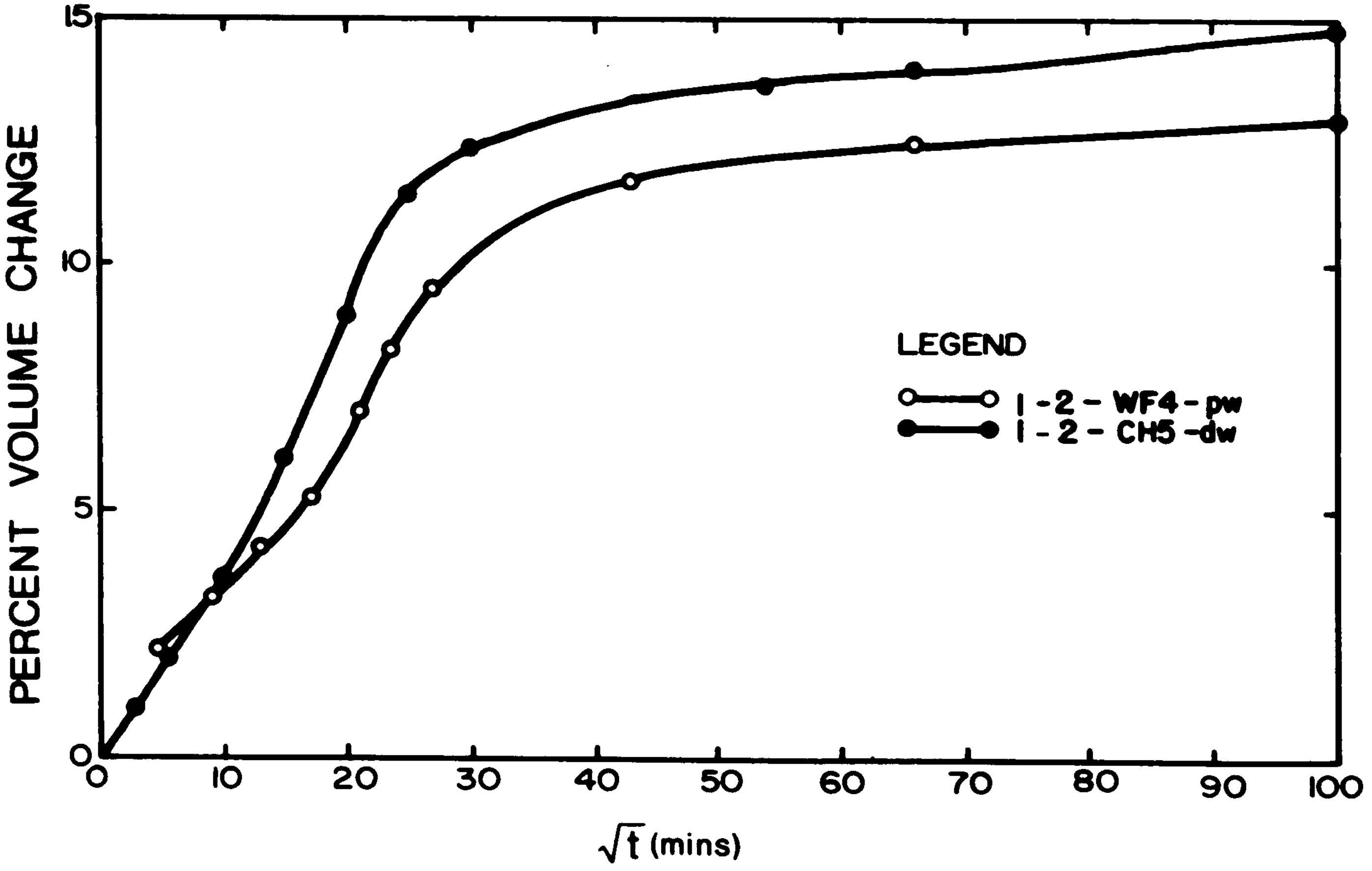


FIG. 7-1 FREE SWELL TEST - BLOCK BEARPAW SHALE

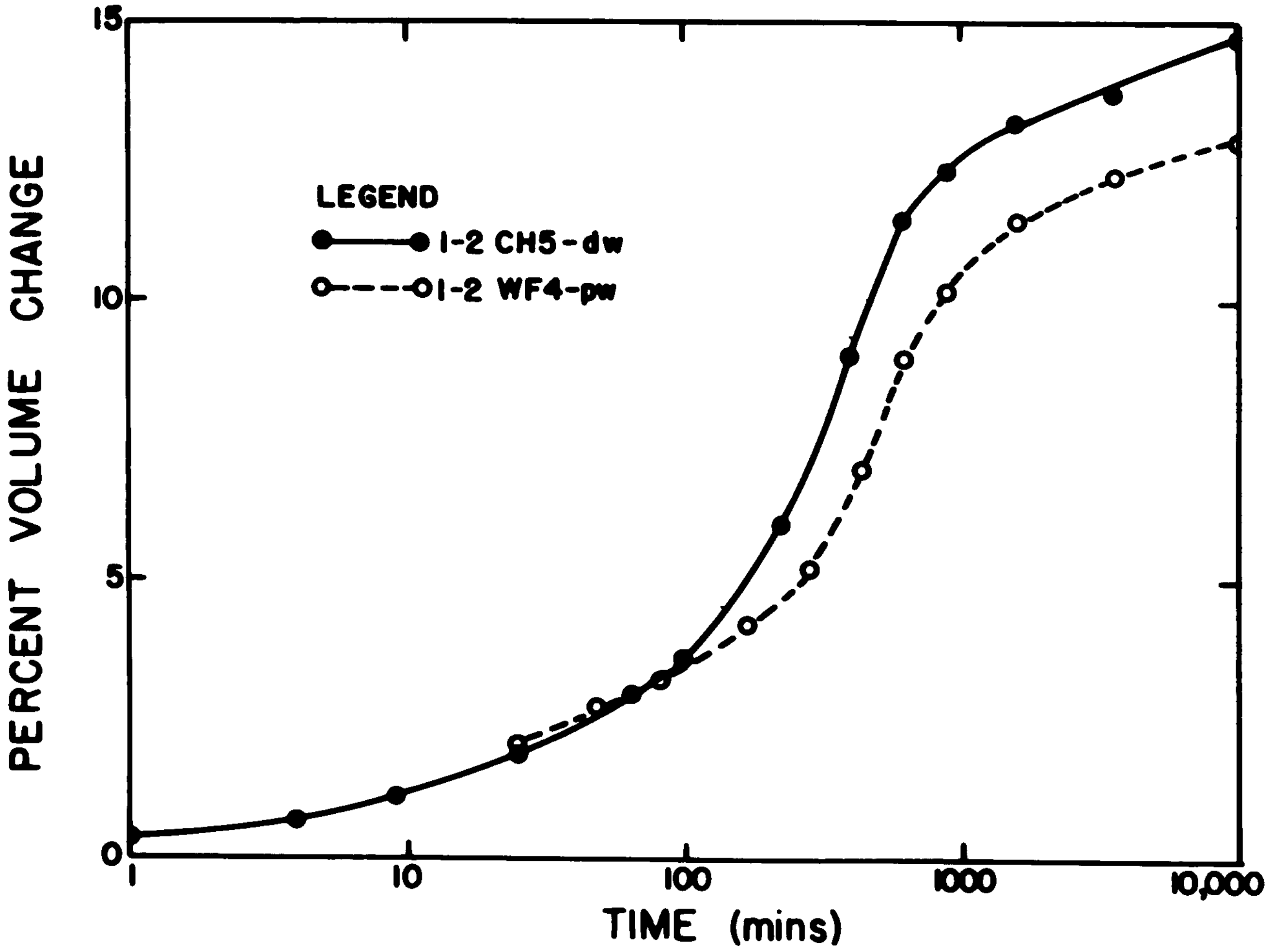


FIG. 7-1a FREE SWELL TEST - BLOCK BEARPAW SHALE

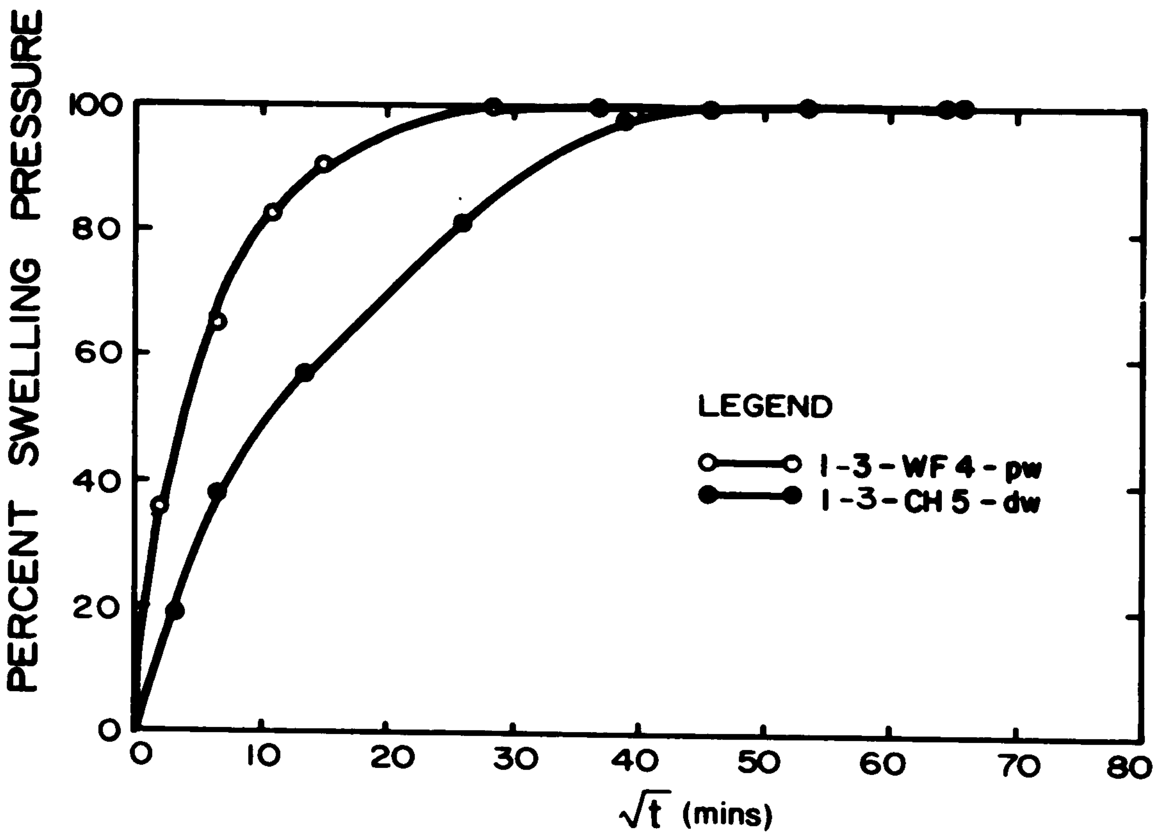


FIG. 7-2 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - BLOCK BEARPAW SHALE

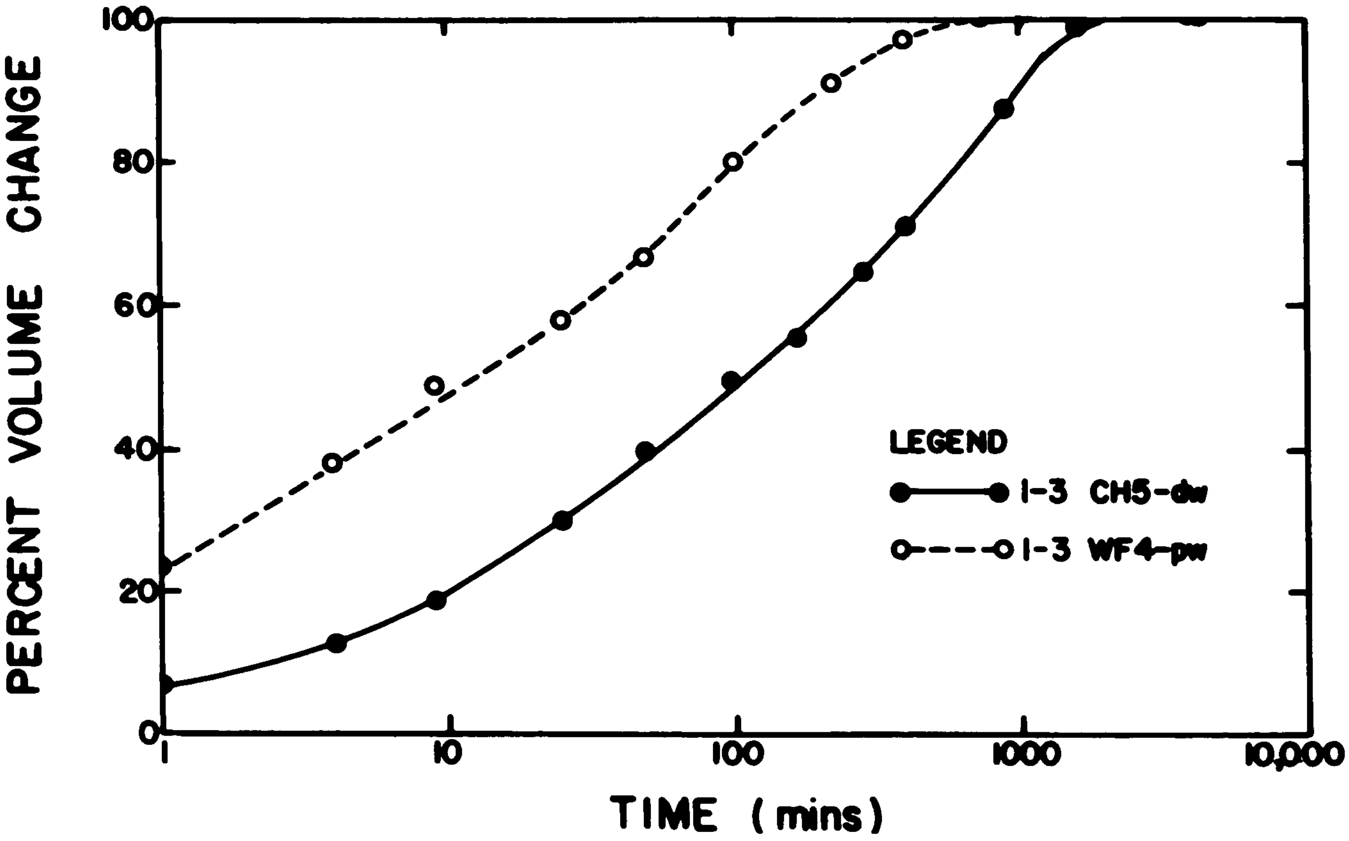


FIG. 7-2a DEVELOPMENT OF SWELLING PRESSURE WITH TIME - BLOCK BEARPAW SHALE

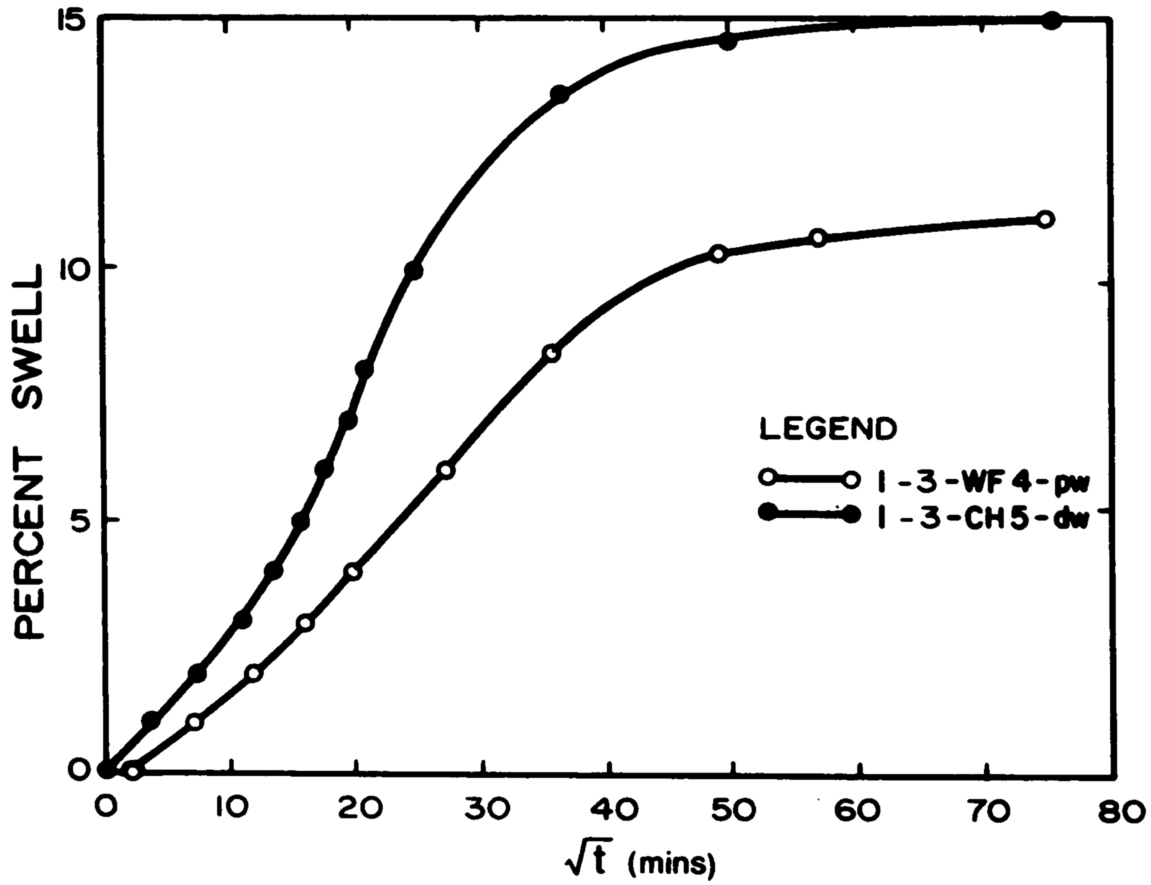


FIG. 7-3 FREE SWELL TEST - BLOCK BEARPAW SHALE

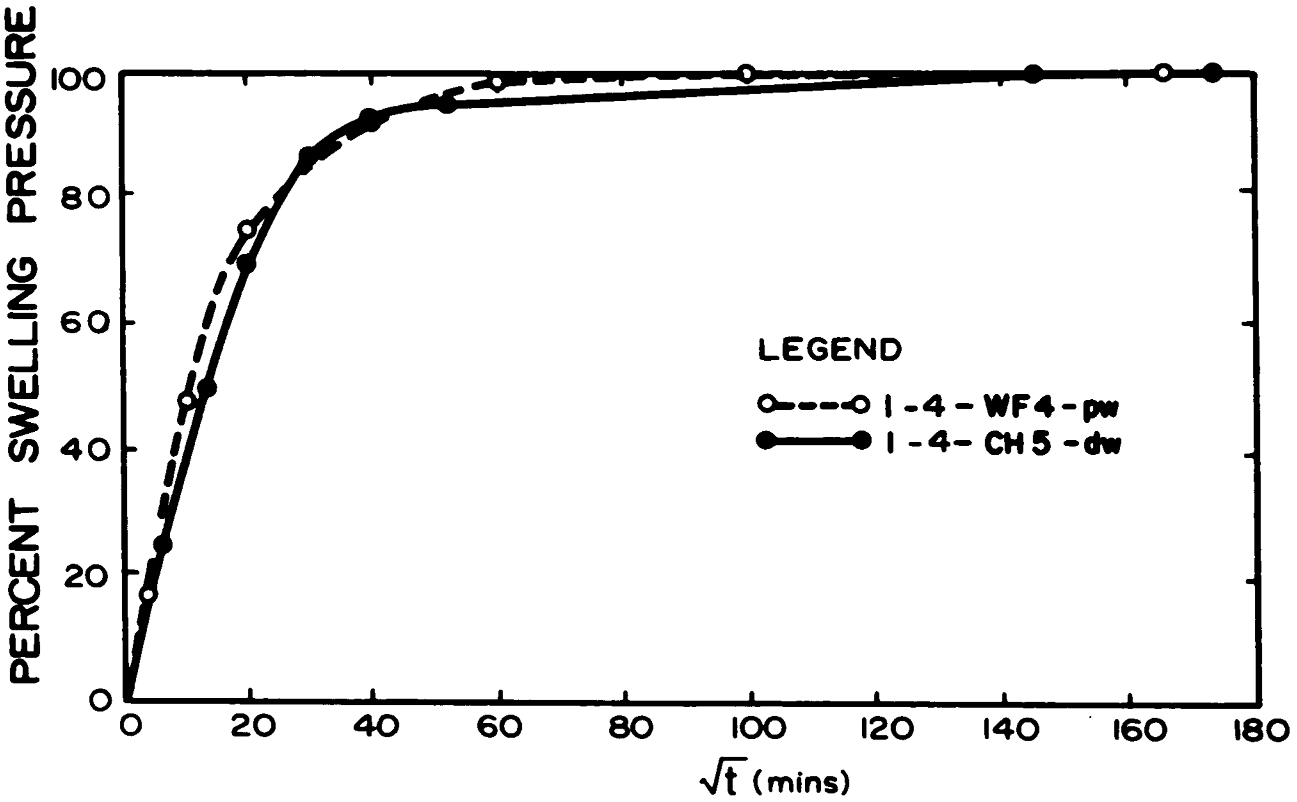


FIG. 7-4 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - BLOCK BEARPAW SHALE

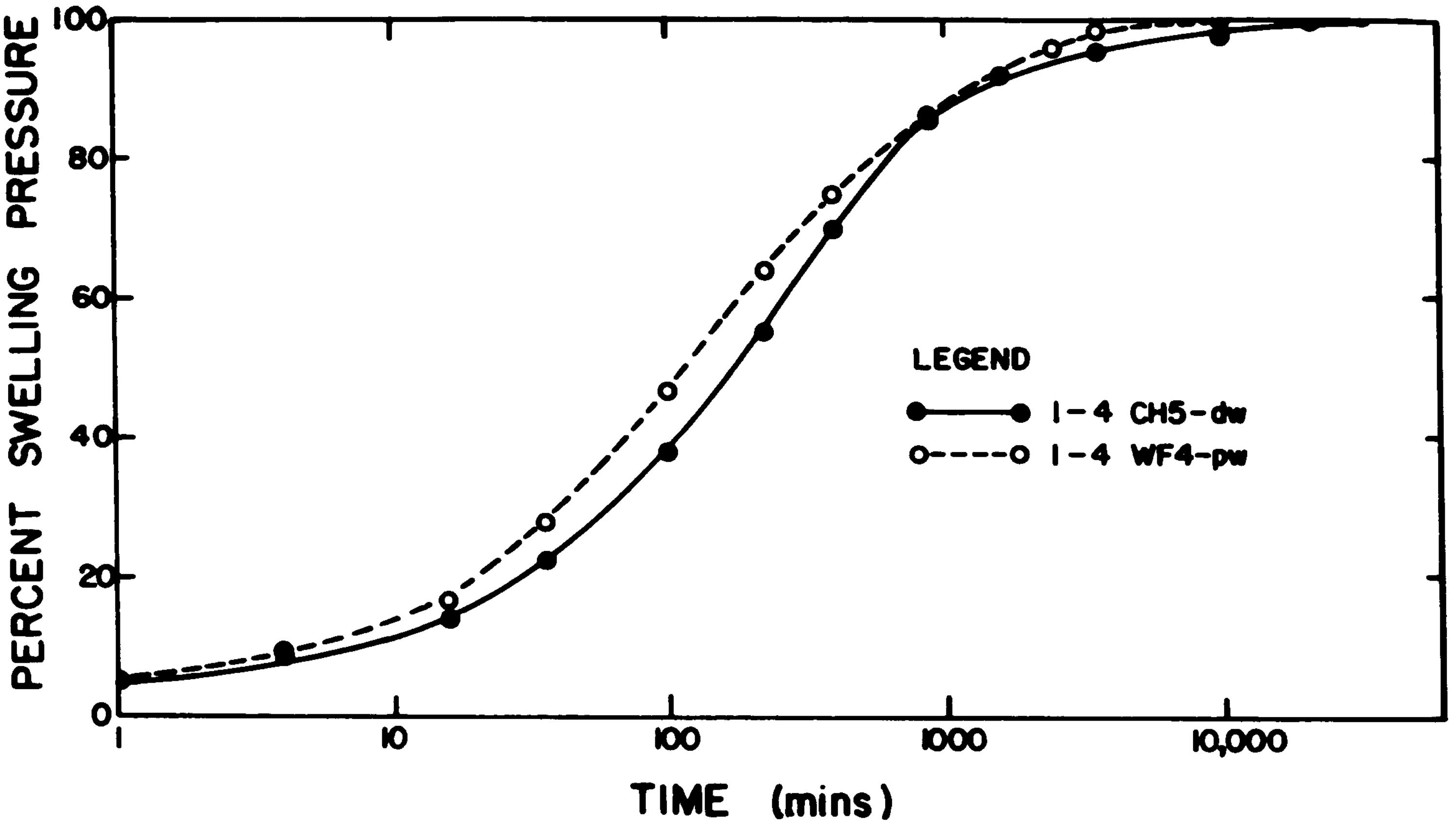


FIG. 7-4a DEVELOPMENT OF SWELLING PRESSURE WITH TIME - BLOCK BEARPAW SHALE

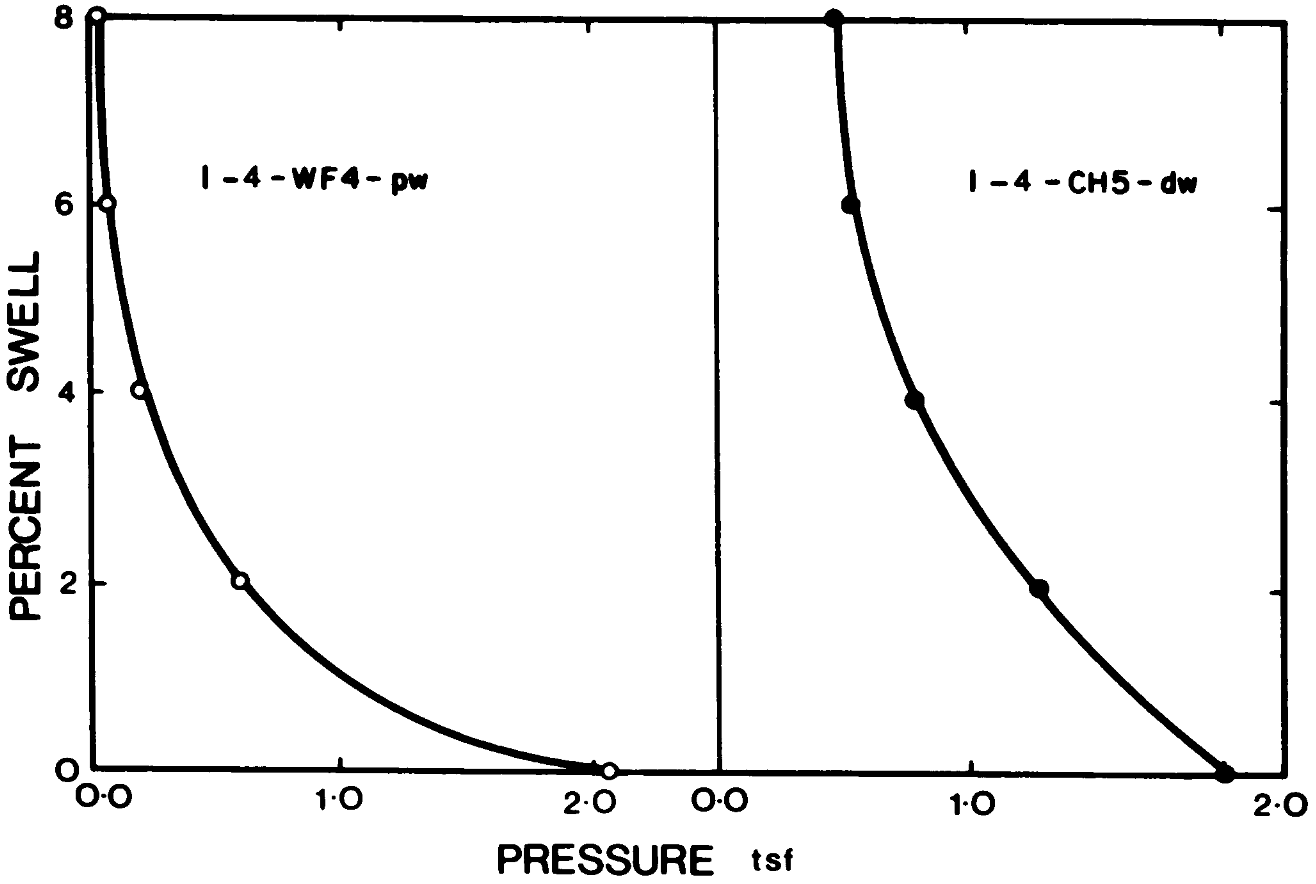


FIG. 7-5 PRESSURE - SWELL RELATIONSHIP - BLOCK BEARPAW SHALE

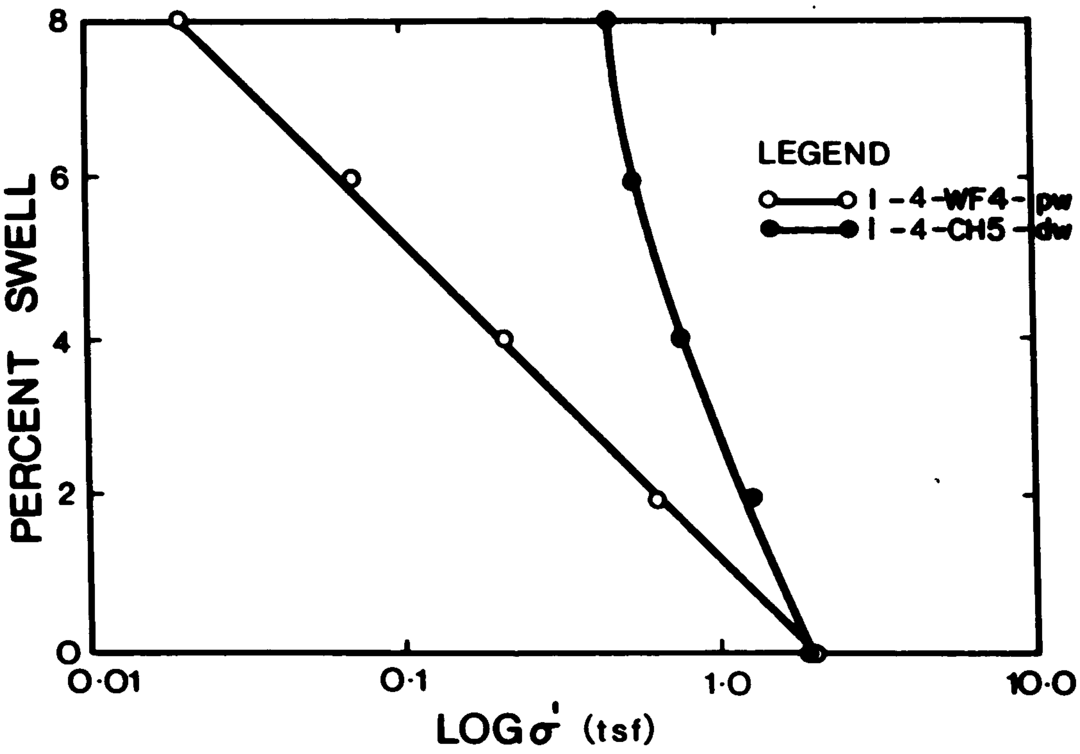


FIG. 7-6 PRESSURE - SWELL RELATIONSHIP - BLOCK BEARPAW SHALE

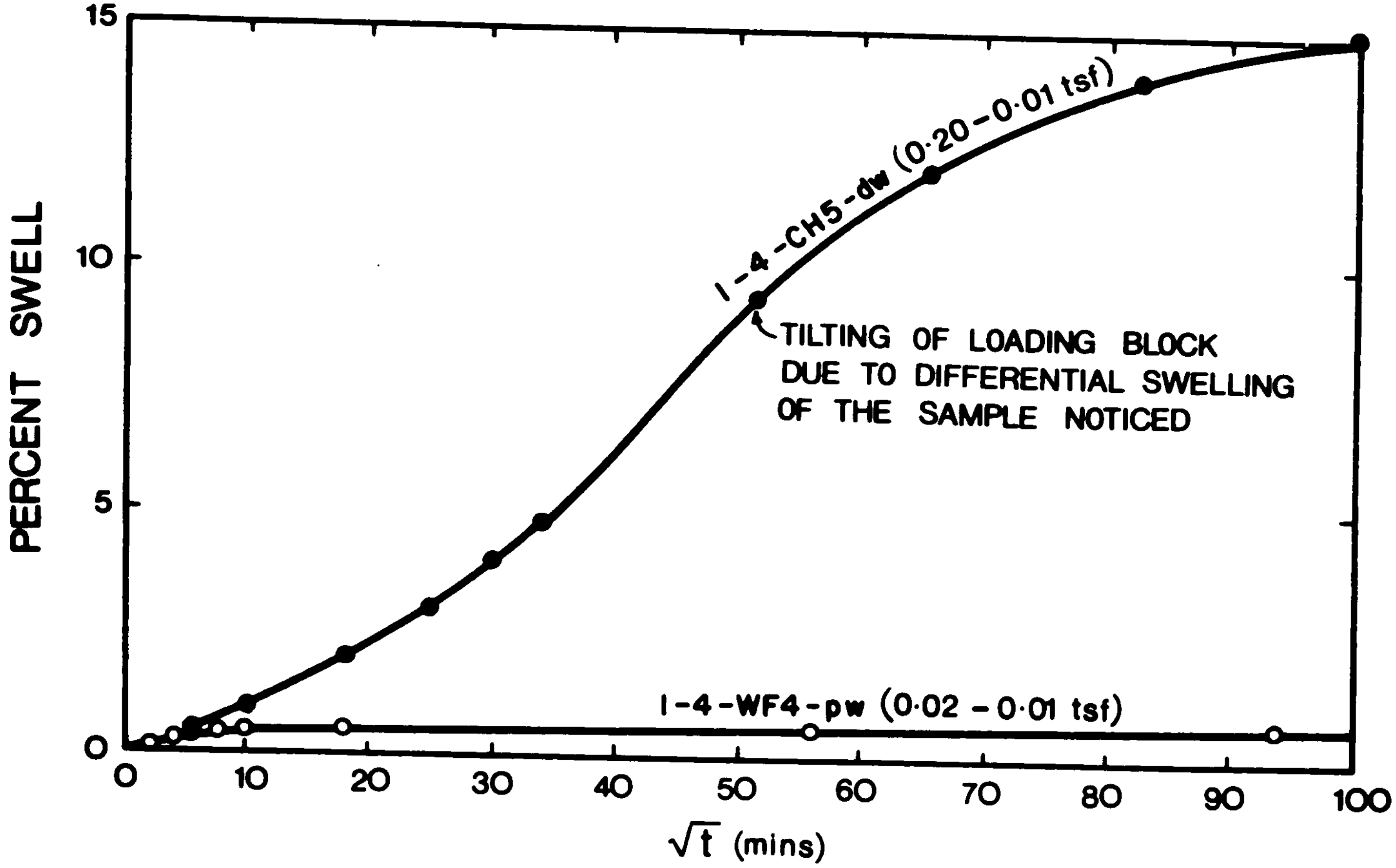


FIG. 7-7 FINAL STAGE FREE SWELLING - BLOCK BEARPAW SHALE

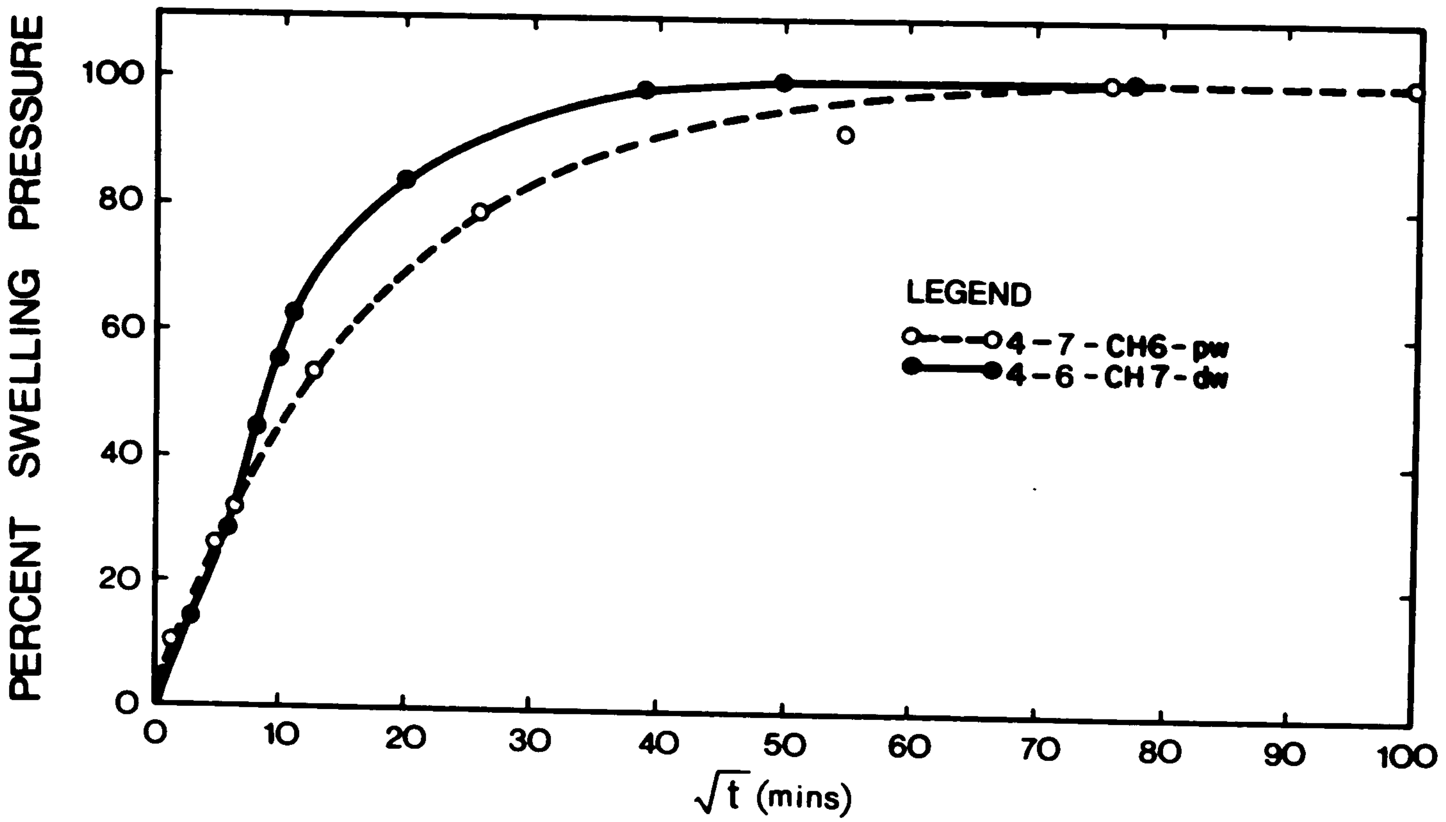


FIG. 7-8 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - INTACT BEARPAW SHALE

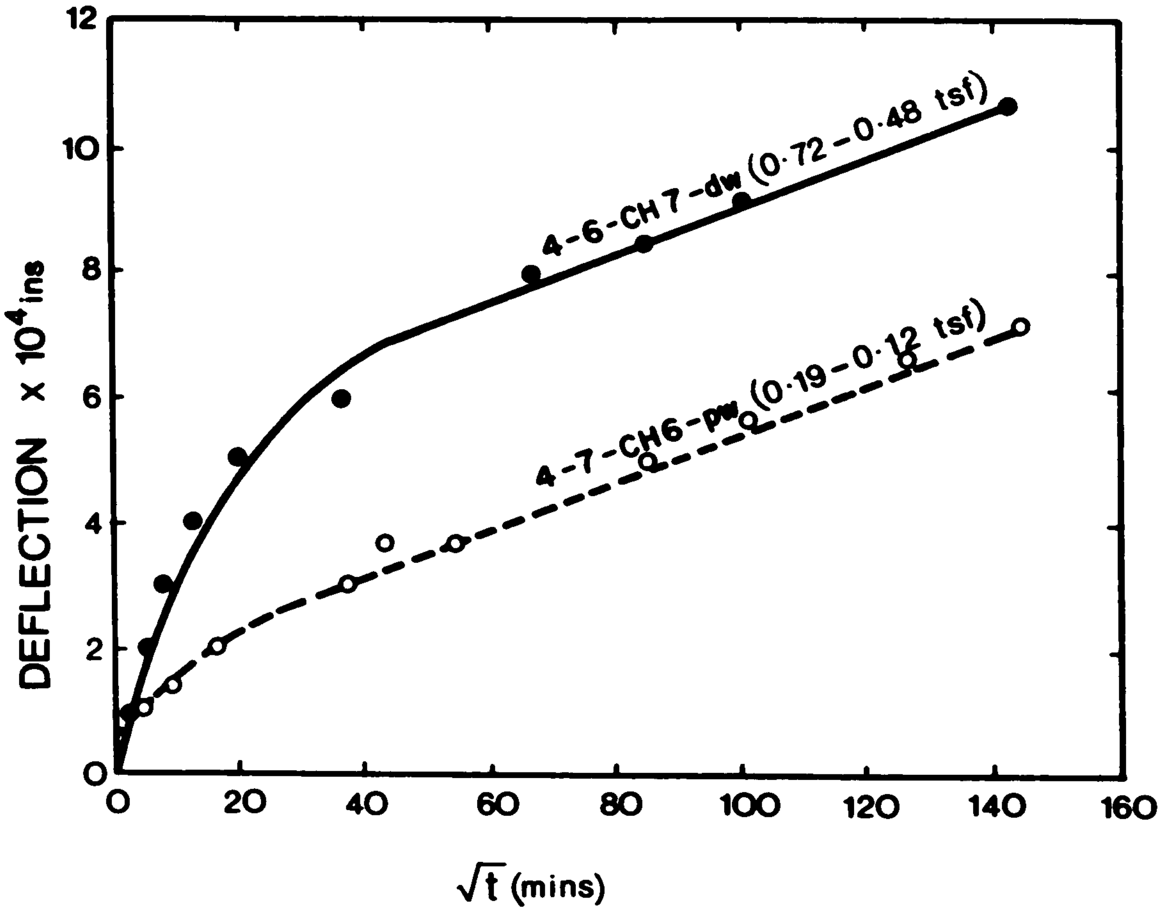


FIG. 7-9a STRESS-CONTROLLED SWELLING TEST (1) - INTACT BEARPAW SHALE

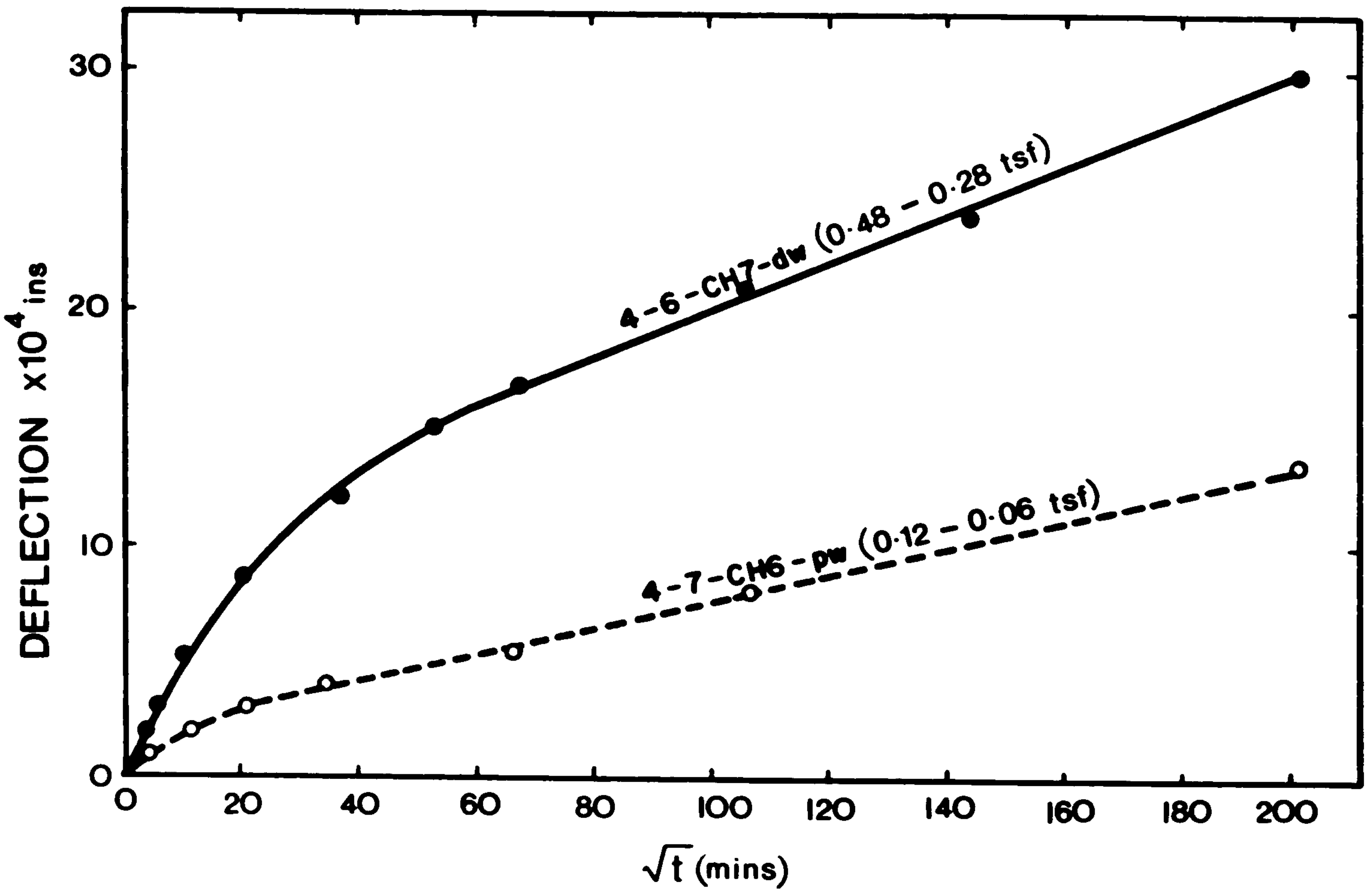


FIG. 7-9b STRESS-CONTROLLED SWELLING TEST (2)-
INTACT BEARPAW SHALE

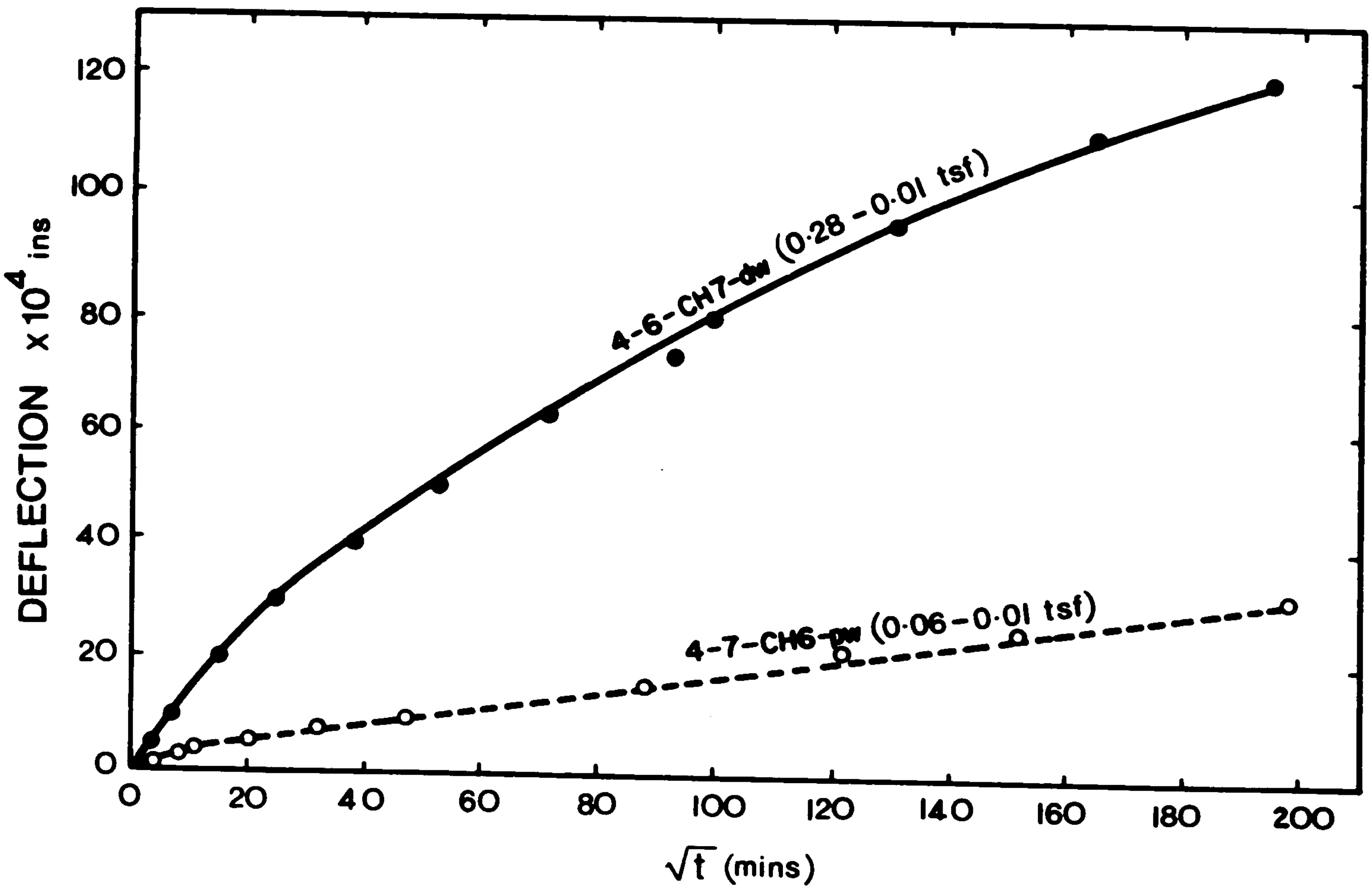


FIG. 7-9c STRESS-CONTROLLED SWELLING TEST (3)- INTACT BEARPAW SHALE

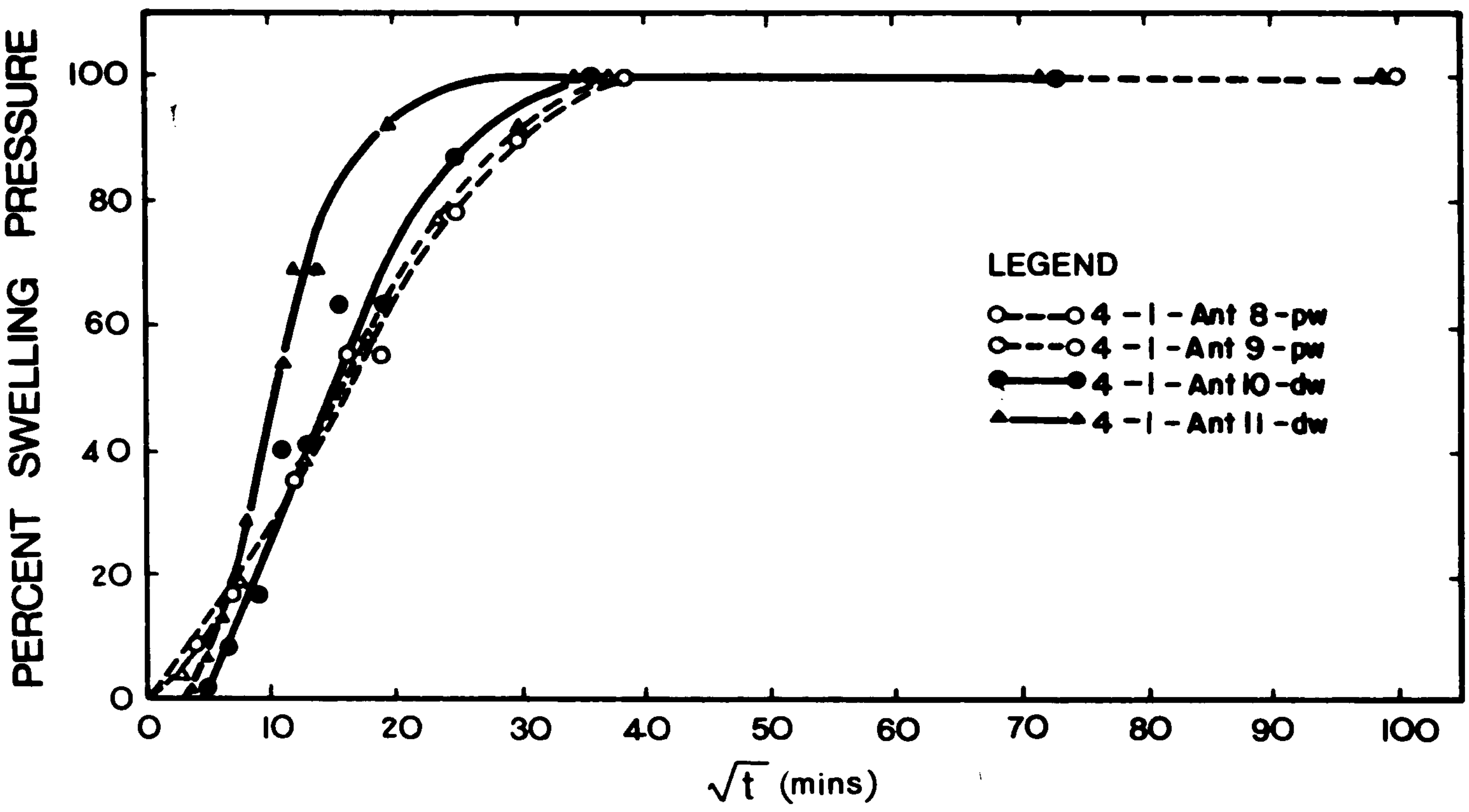


FIG. 7-10 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - INTACT BEARPAW SHALE

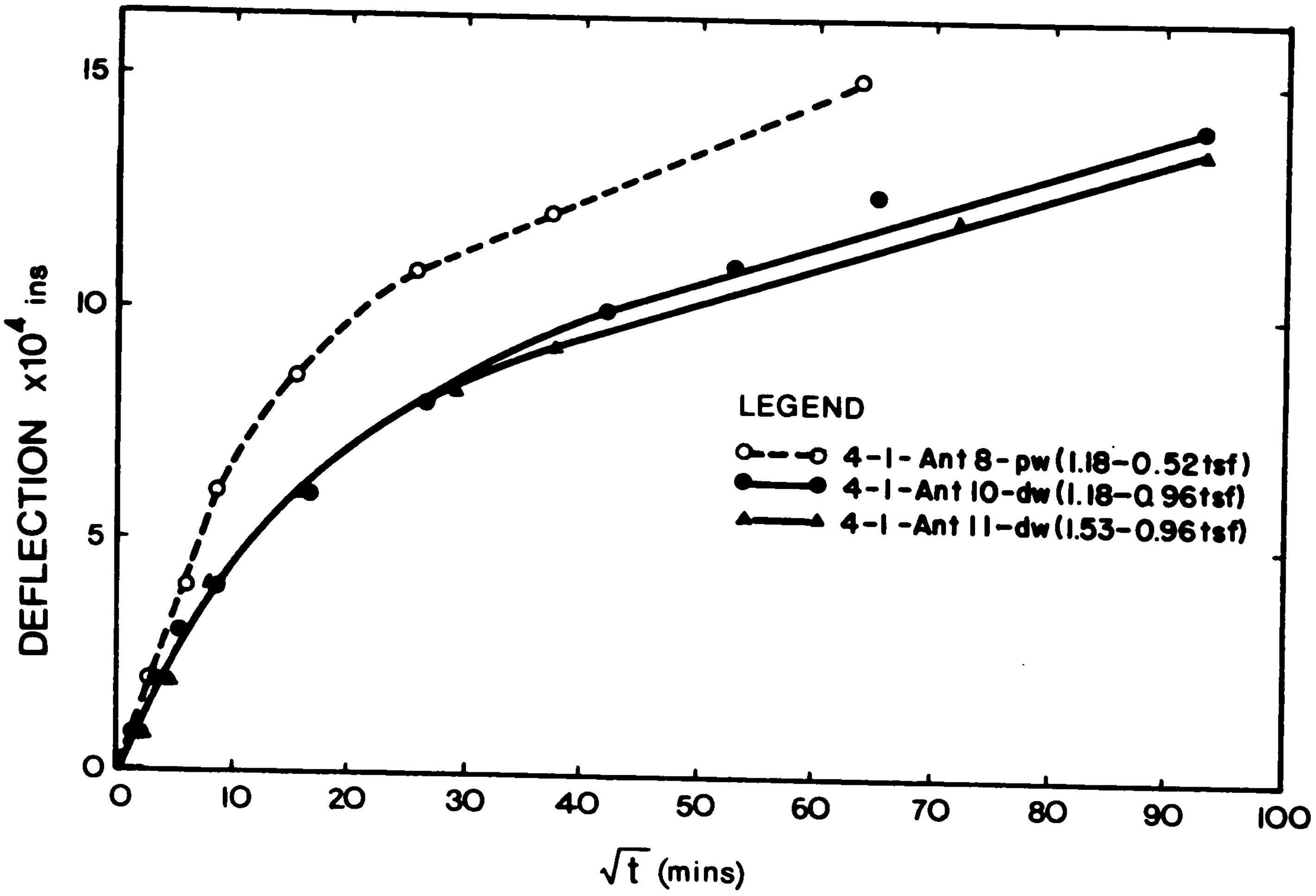


FIG. 7-11a STRESS-CONTROLLED SWELLING TEST (1)- INTACT BEARPAW SHALE

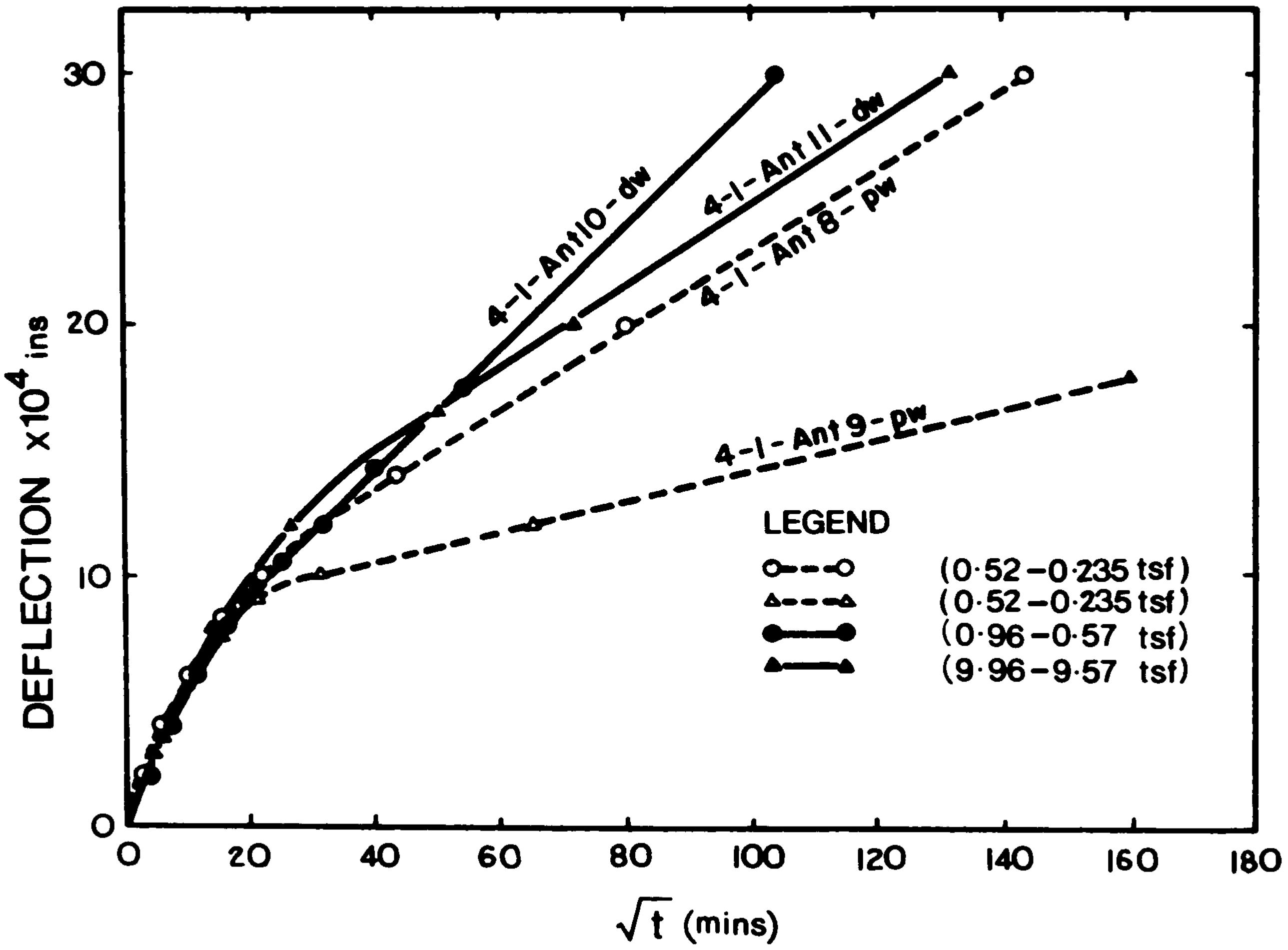


FIG. 7-11b STRESS-CONTROLLED SWELLING TEST (2)-
INTACT BEARPAW SHALE

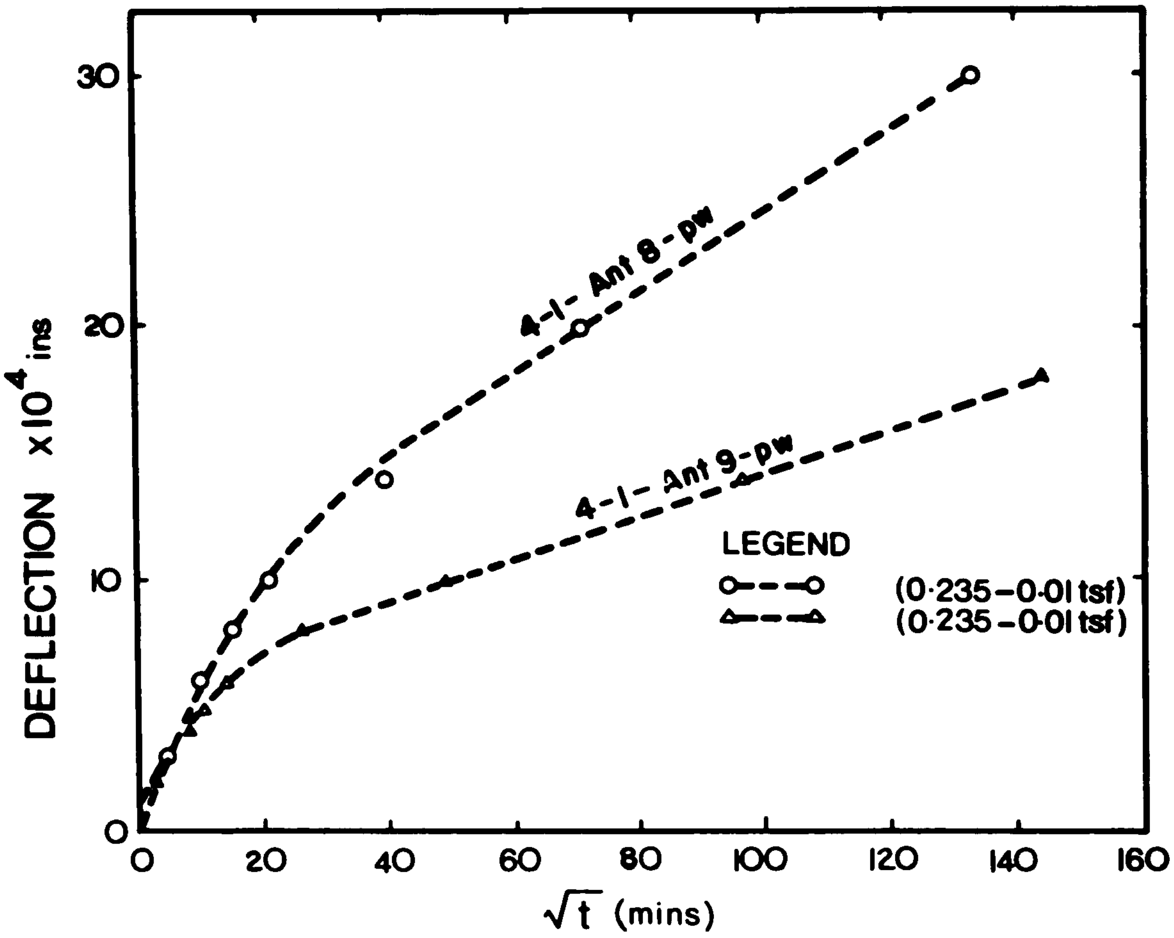


FIG. 7-11c STRESS-CONTROLLED SWELLING TEST (3)-INTACT BEARPAW SHALE

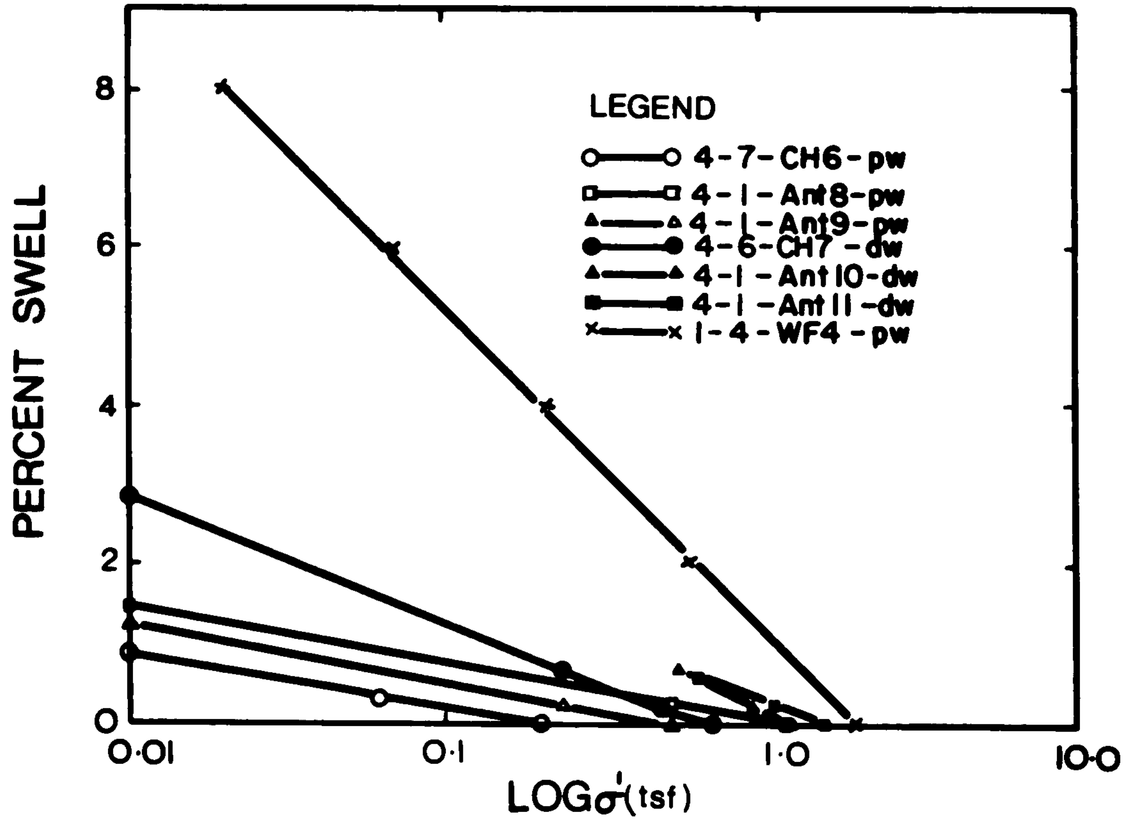


FIG. 7-12 PRESSURE-SWELL RELATIONSHIP - INTACT BEARPAW SHALE SMAPLES & 1-4-WF4-pw

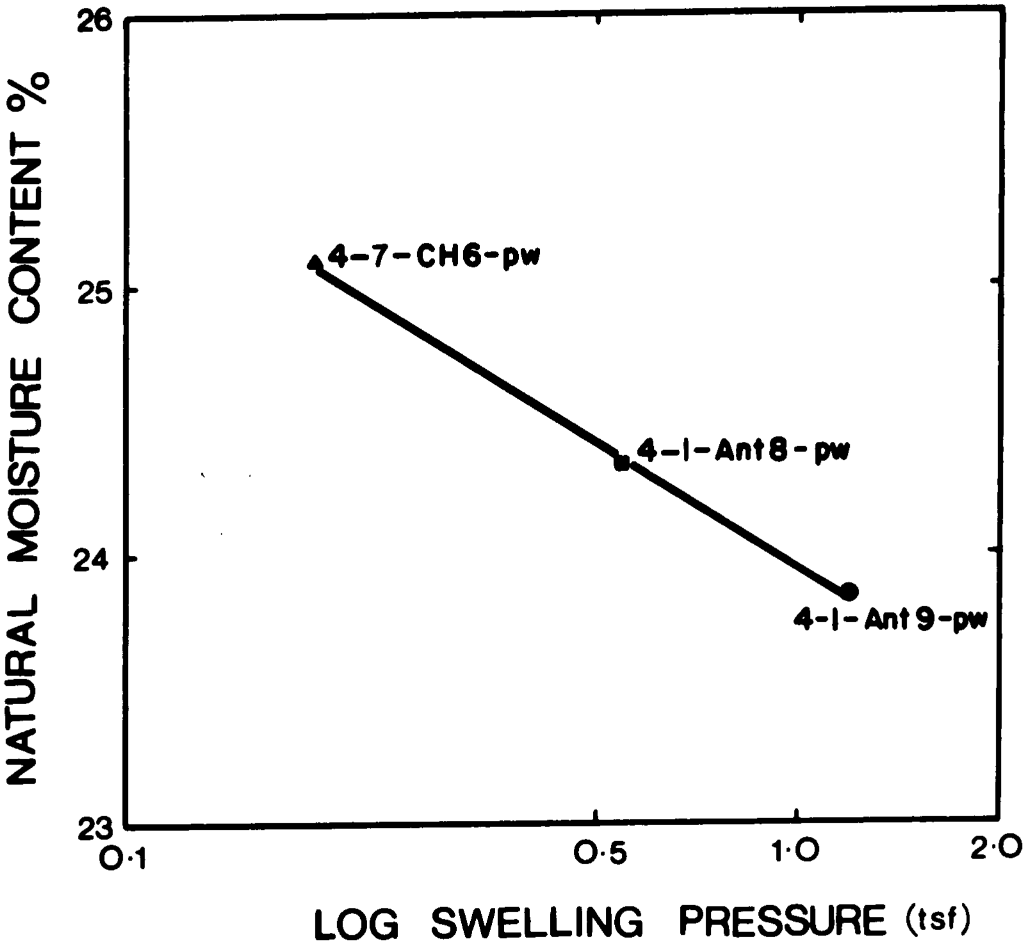


FIG. 7-13 WATER CONTENT-SWELLING PRESSURE RELATIONSHIP - INTACT BEARPAW SHALE

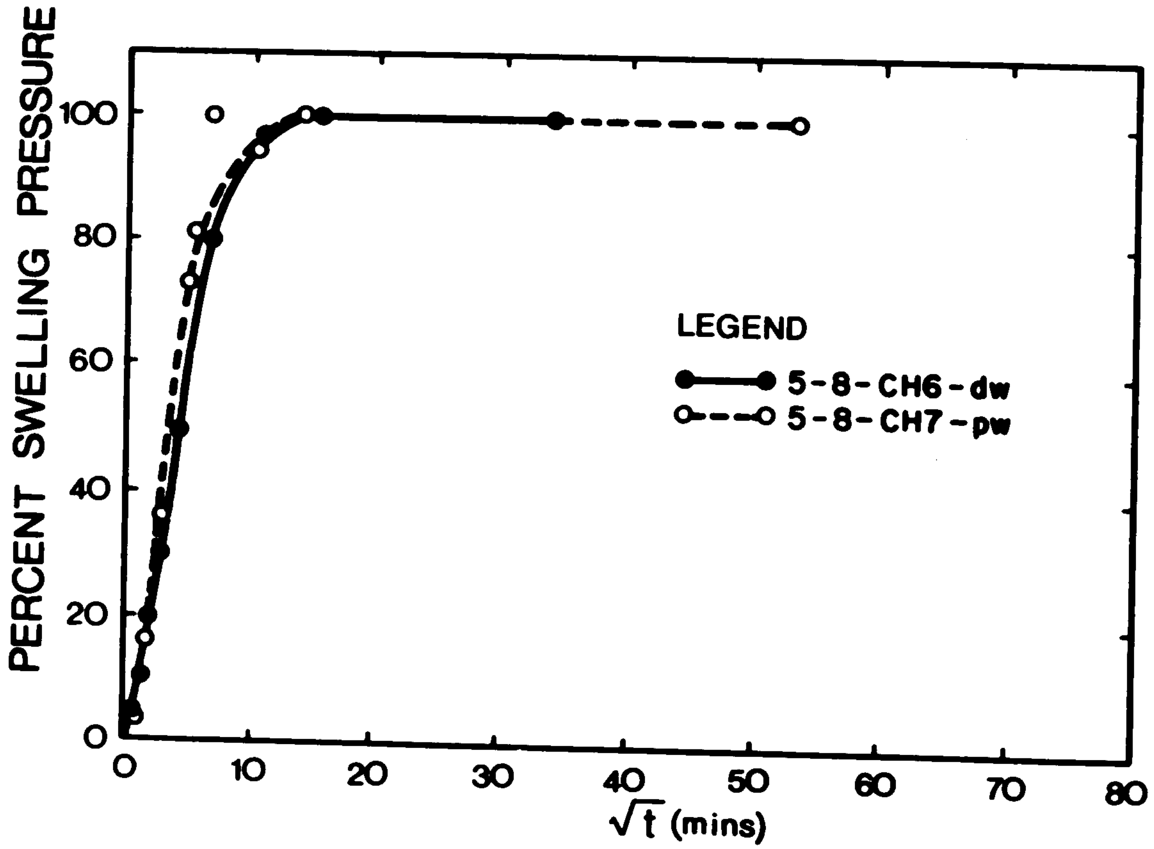


FIG. 7-14 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - MORDEN SHALE

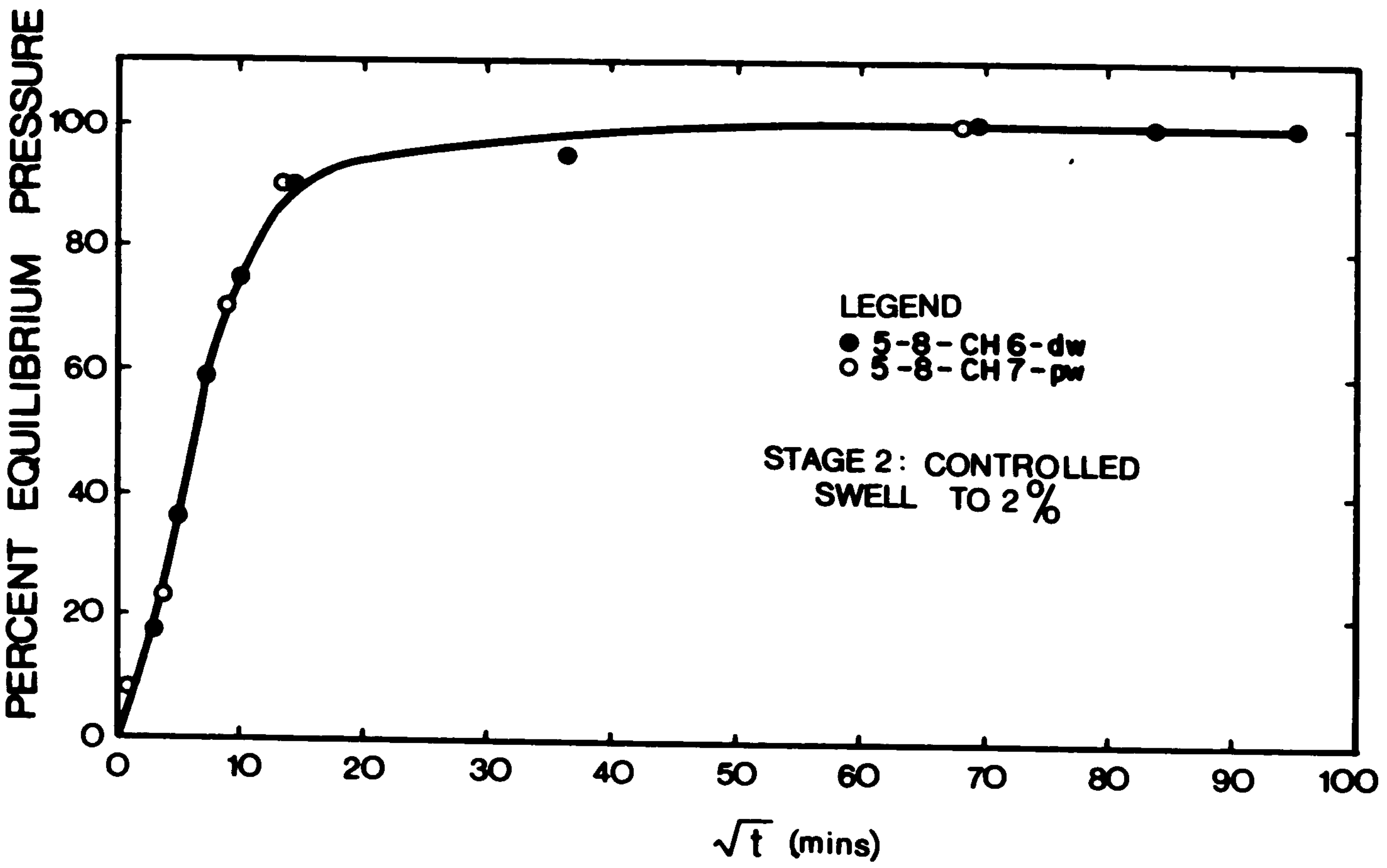


FIG. 7-15 DEVELOPMENT OF EQUILIBRIUM PRESSURE WITH TIME - MORDEN SHALE

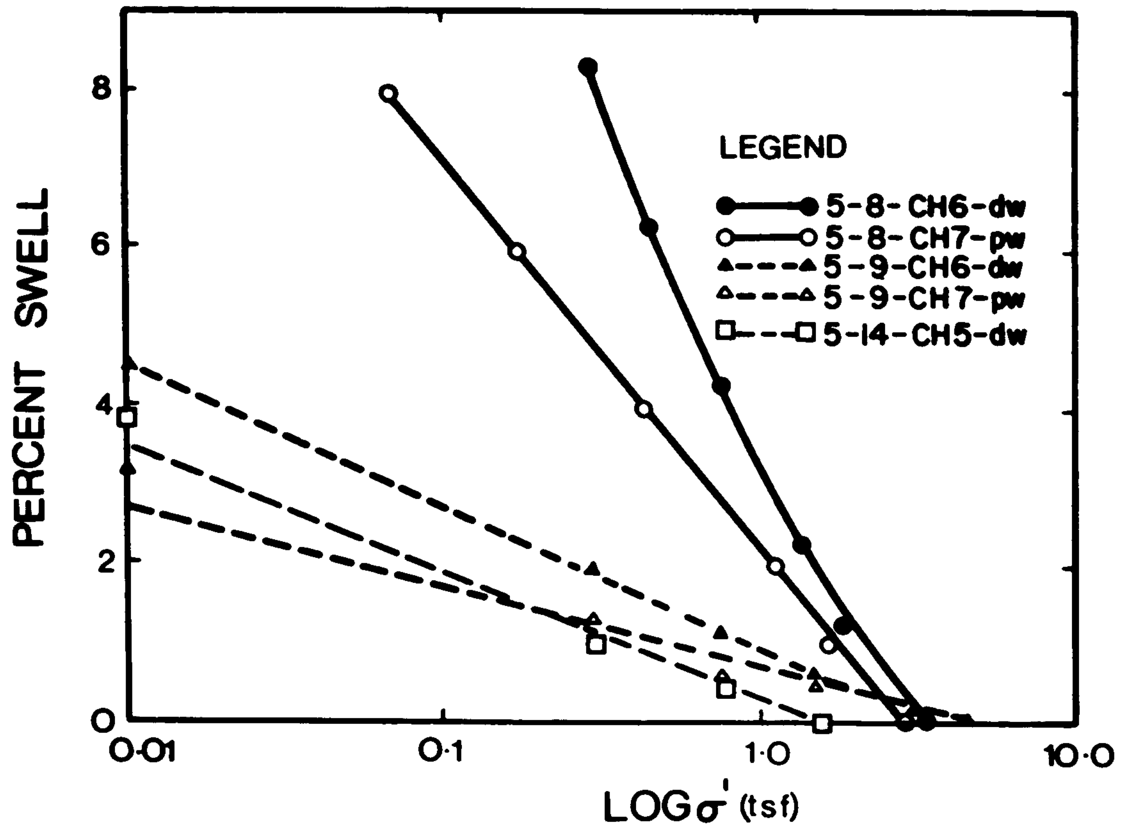


FIG. 7-16 PRESSURE - SWELL RELATIONSHIP - MORDEN SHALE SAMPLES

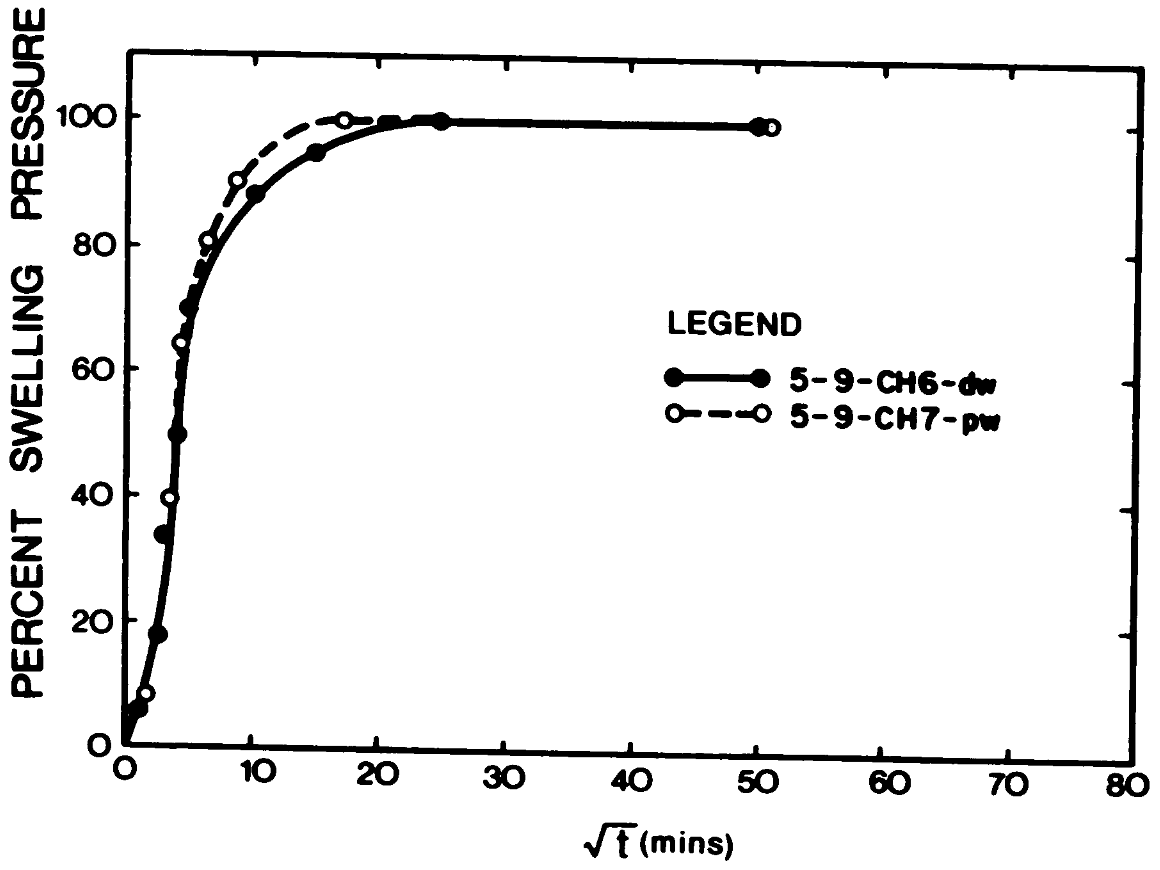


FIG. 7-17 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - MORDEN SHALE

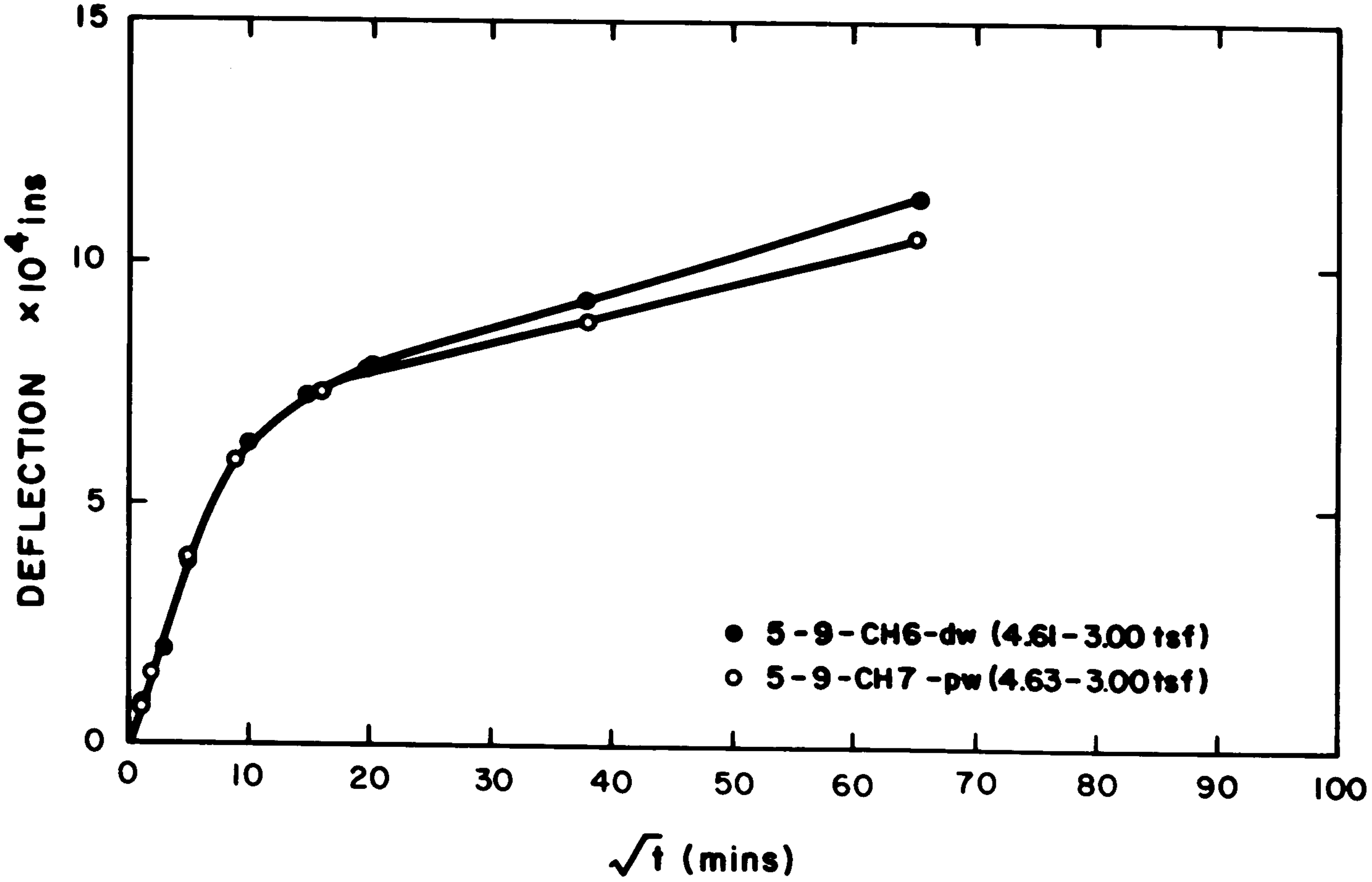


FIG. 7-18a STRESS - CONTROLLED SWELLING TEST (1) - MORDEN SHALE

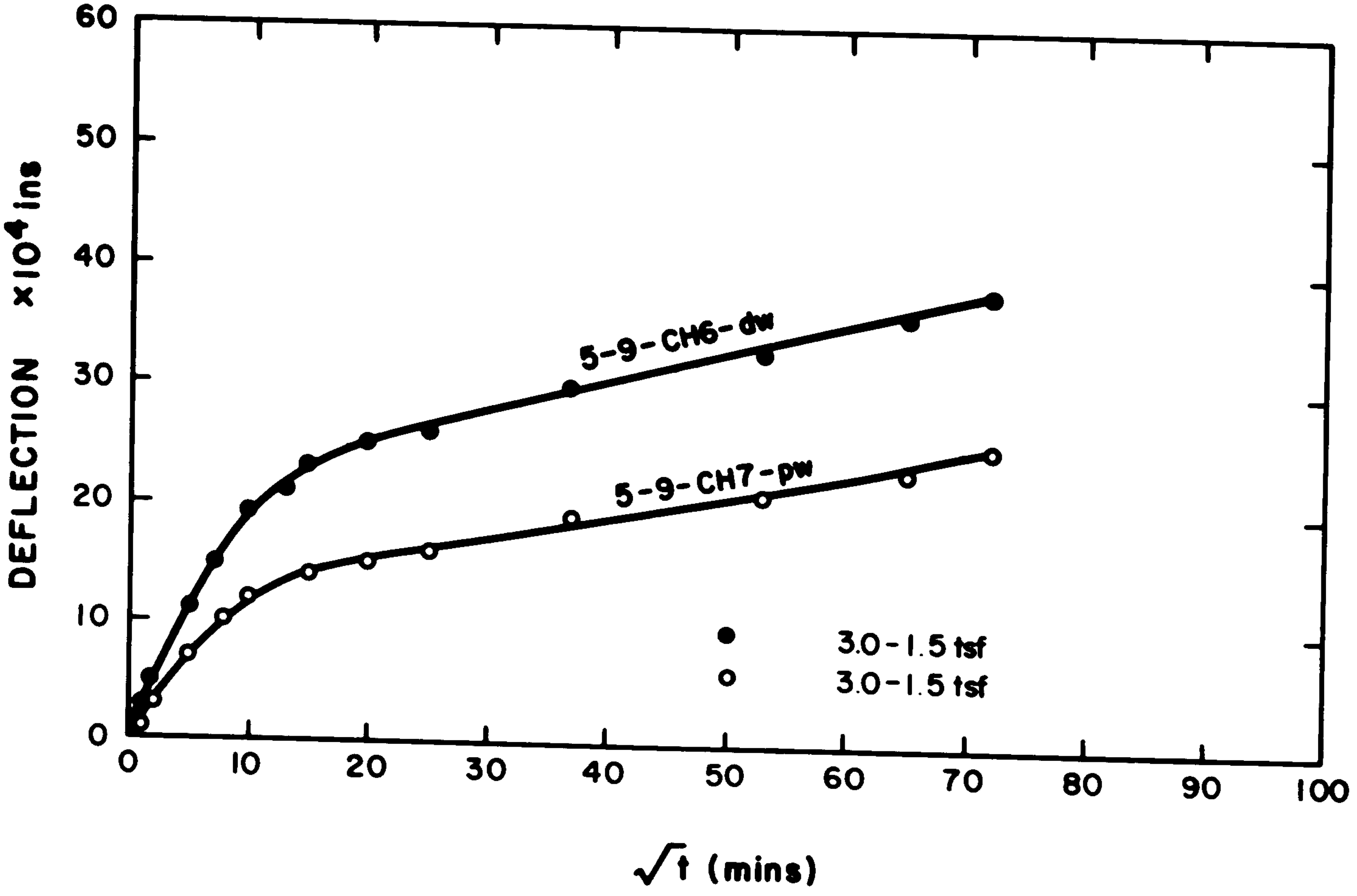


FIG. 7-18b STRESS - CONTROLLED SWELLING TEST (2) - MORDEN SHALE

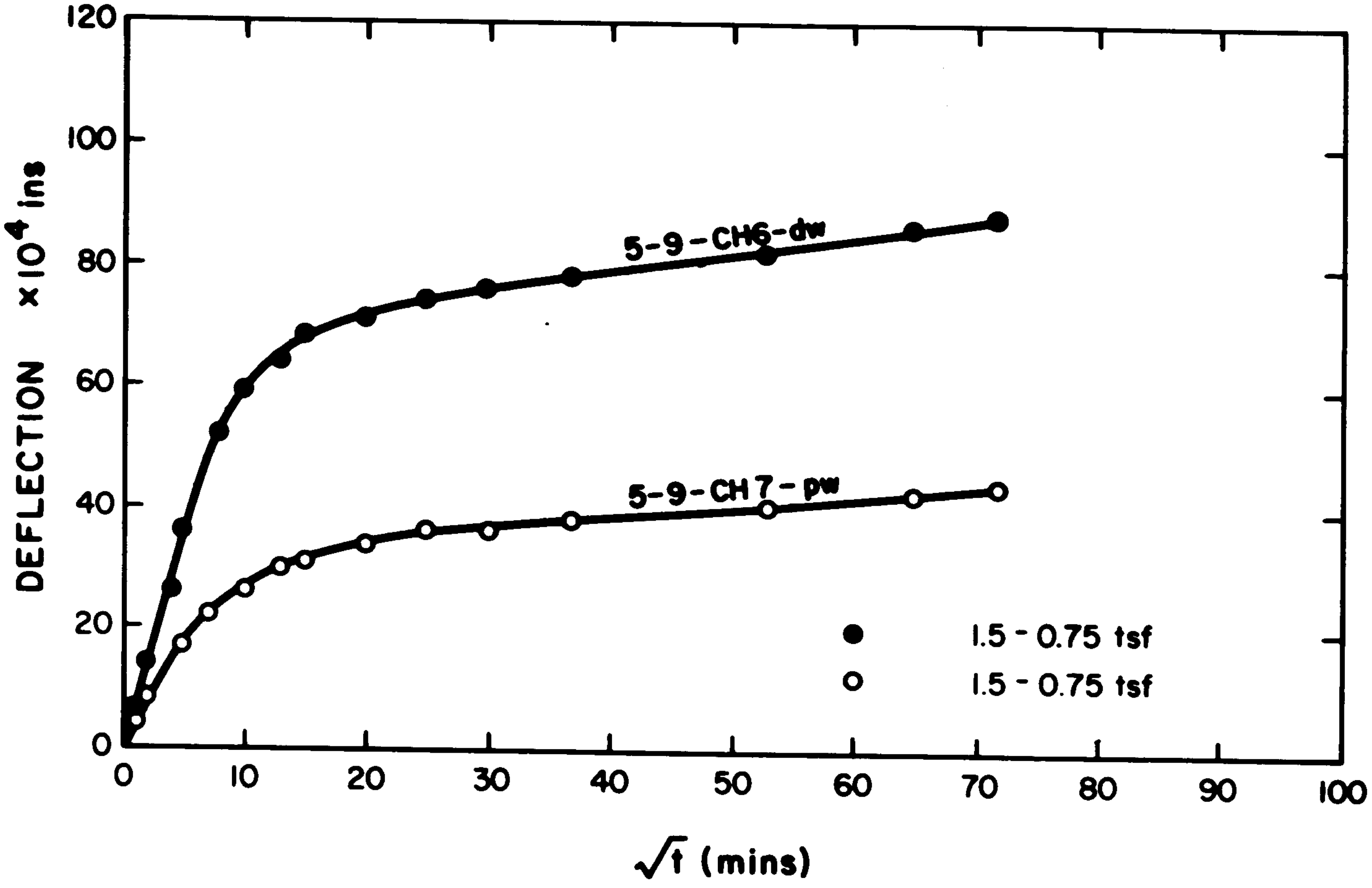


FIG. 7-18c STRESS - CONTROLLED SWELLING TEST (3) - MORDEN SHALE

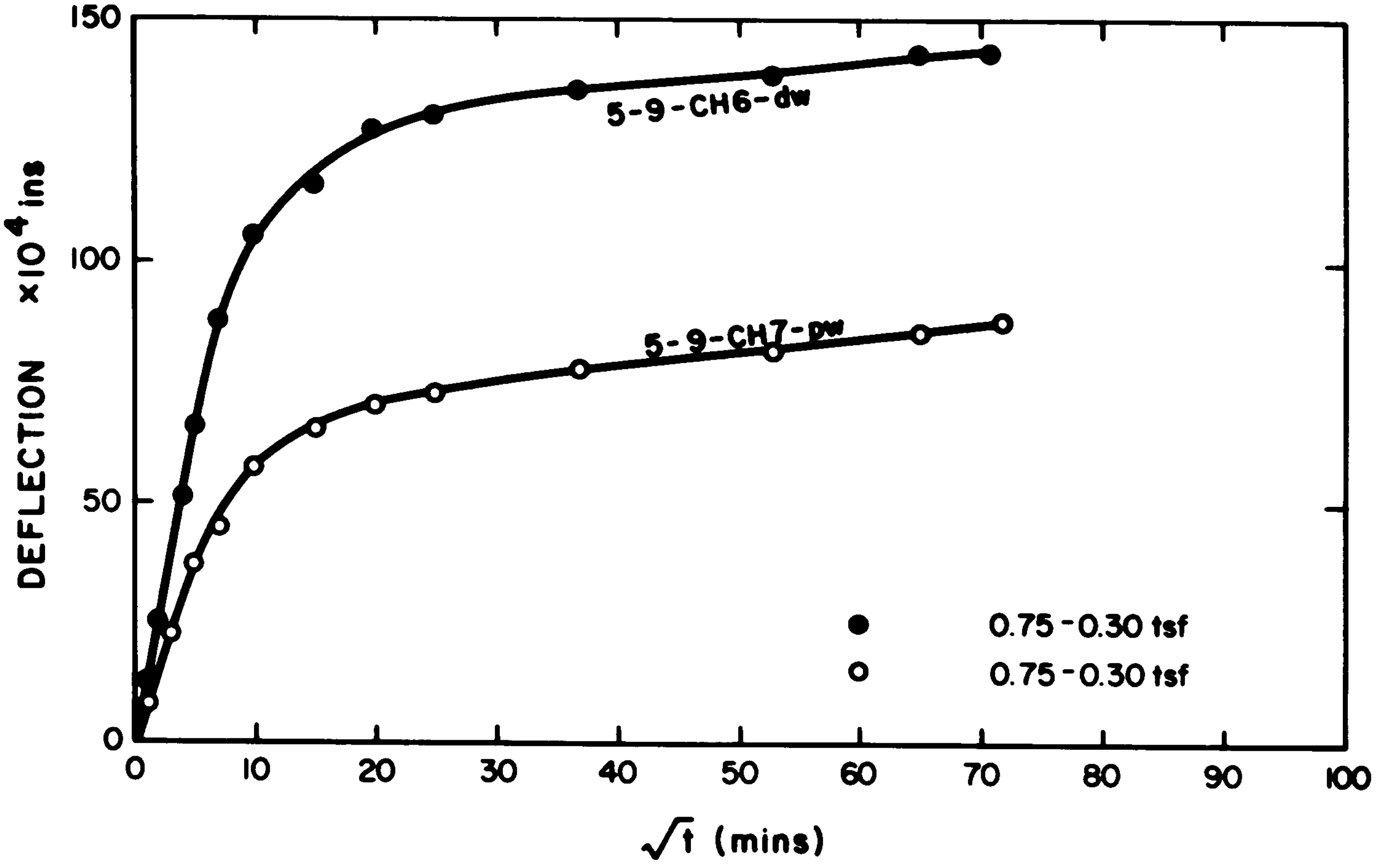


FIG. 7-18d STRESS - CONTROLLED SWELLING TEST (4) - MORDEN SHALE

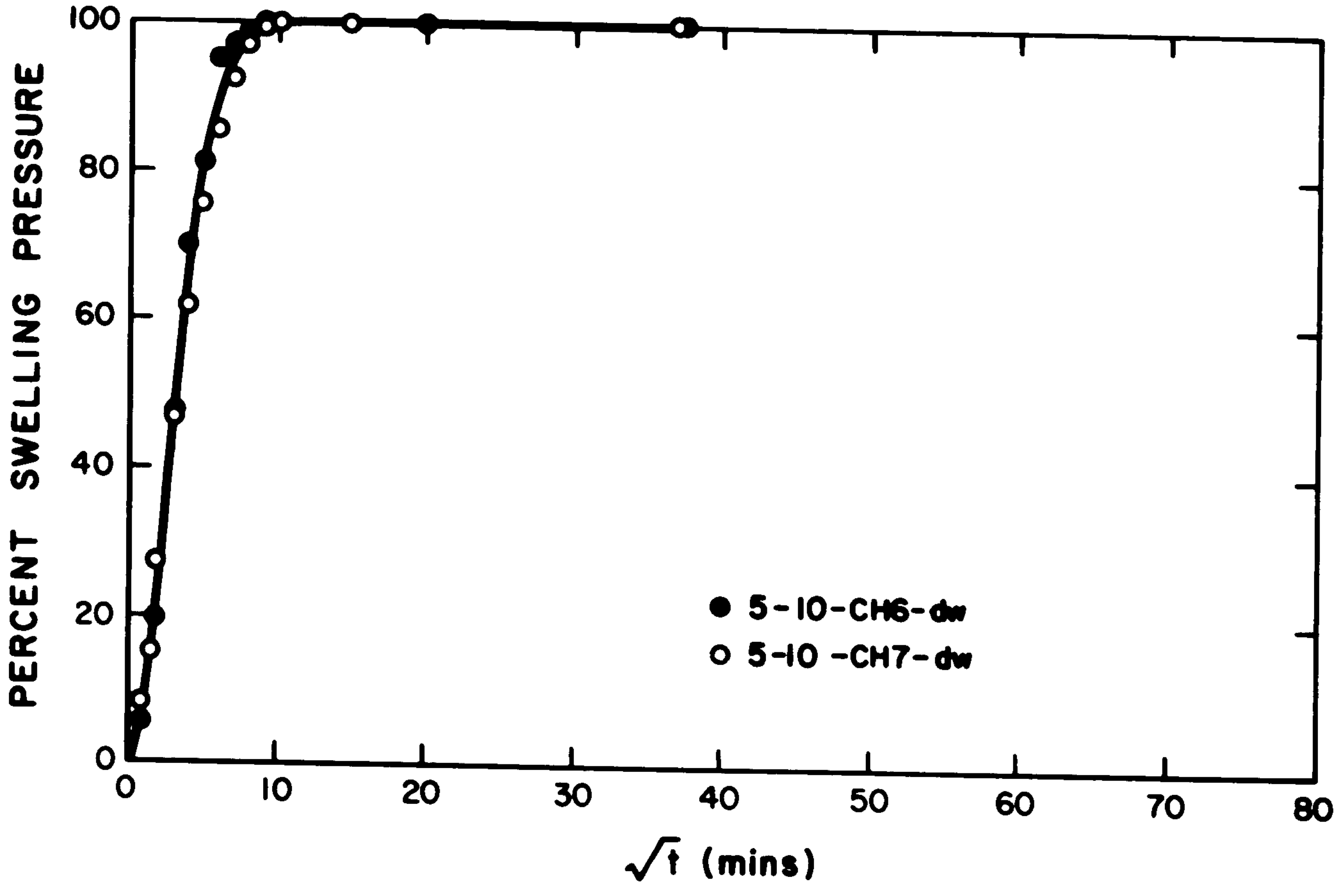


FIG. 7-19 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - MORDEN SHALE

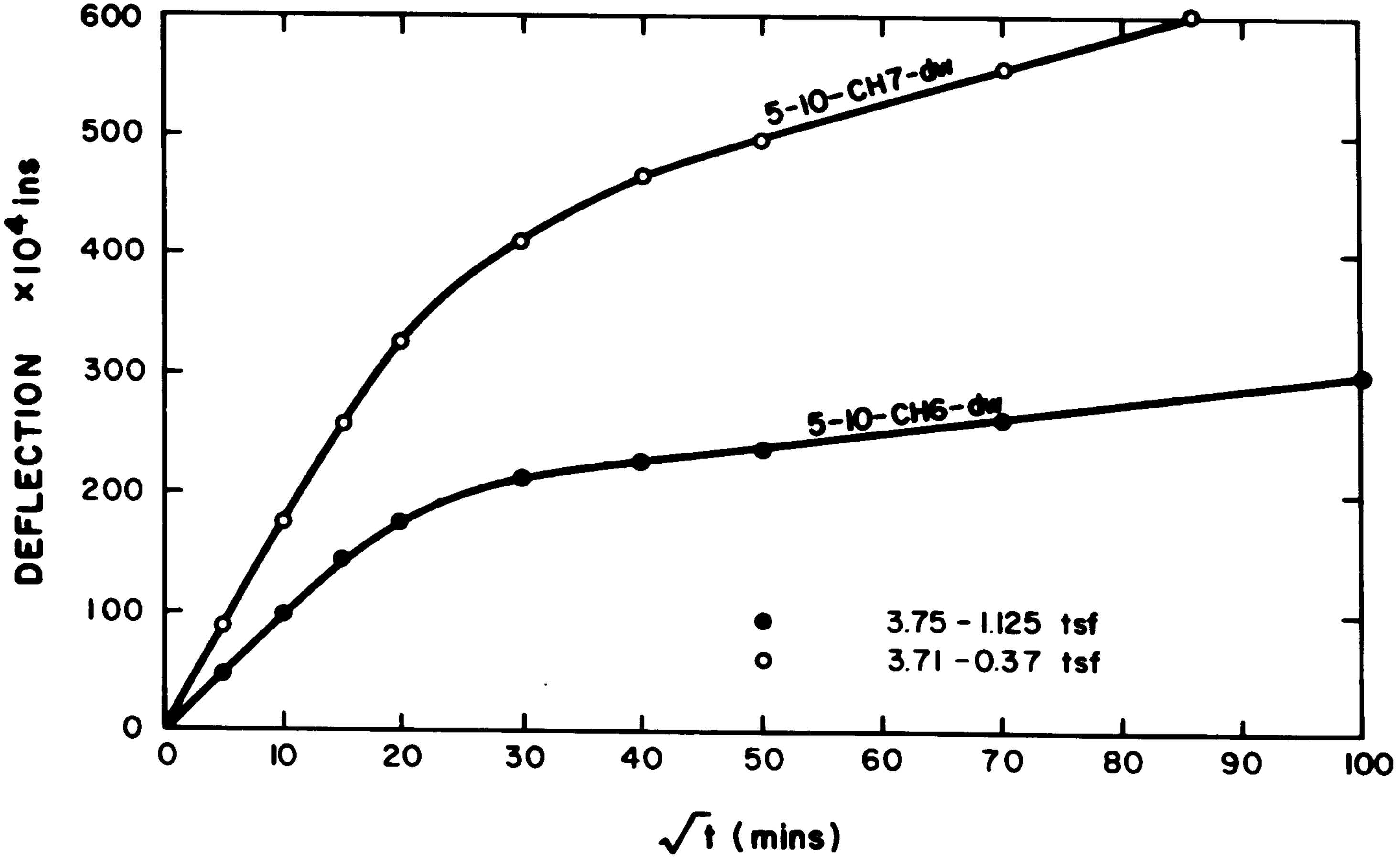


FIG. 7-20 STRESS - CONTROLLED SWELLING TEST - MORDEN SHALE

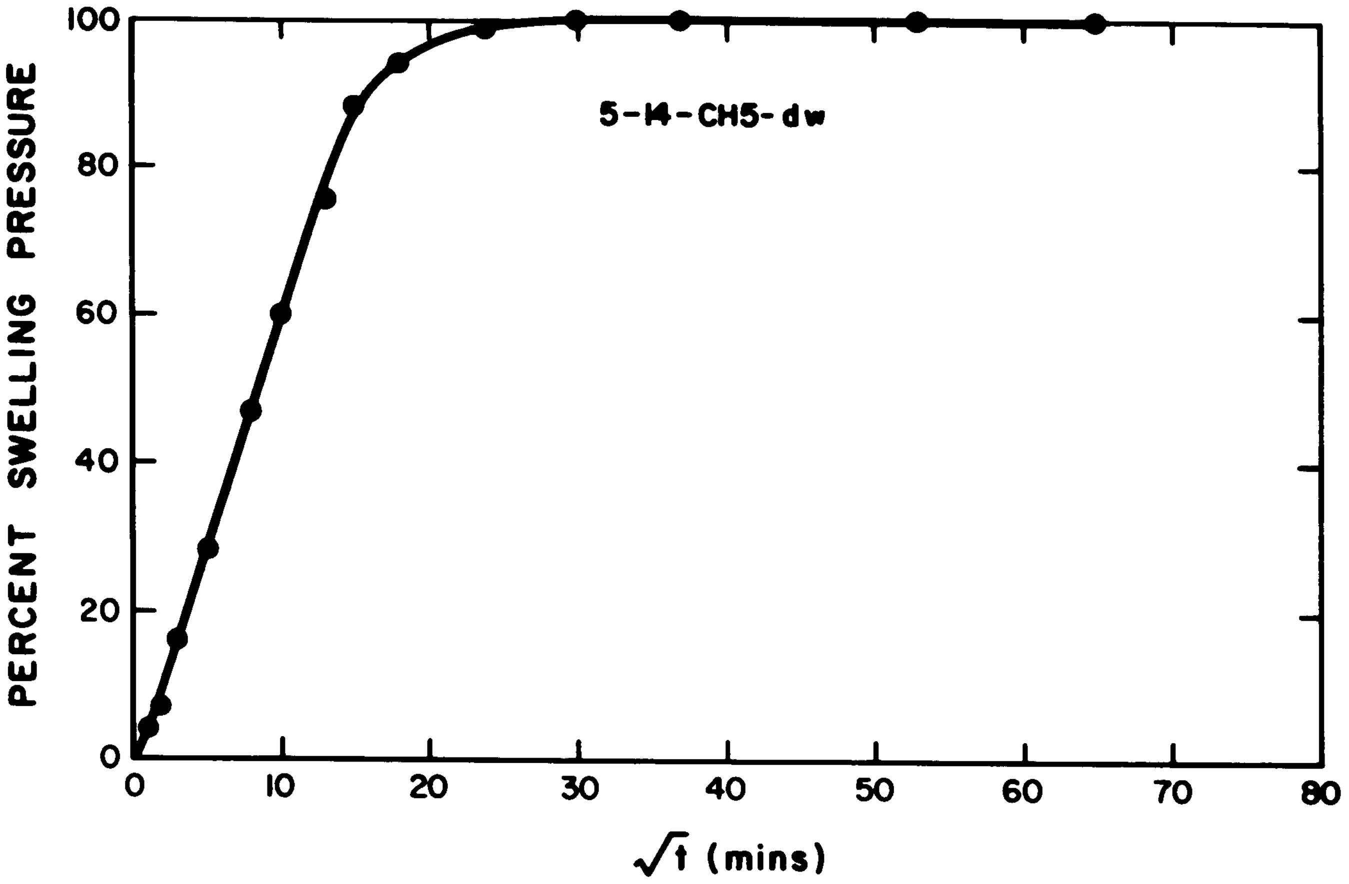


FIG. 7-21 DEVELOPMENT OF SWELLING PRESSURE WITH TIME - MORDEN SHALE

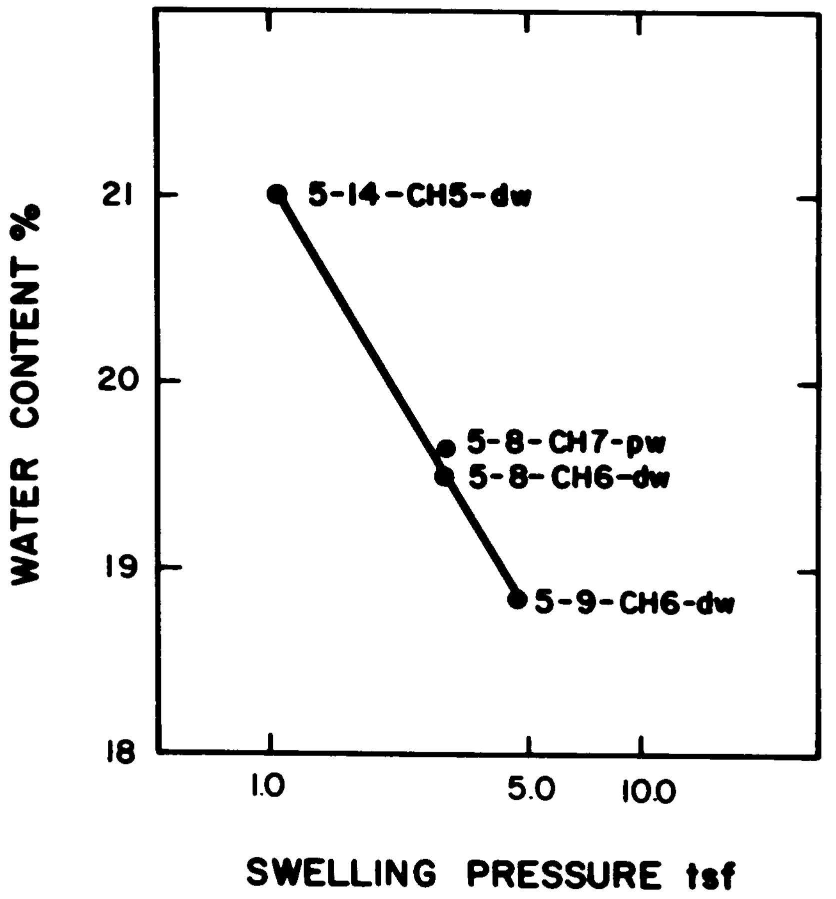


FIG. 7-22 WATER CONTENT - SWELLING PRESSURE RELATIONSHIP - MORDEN SHALE

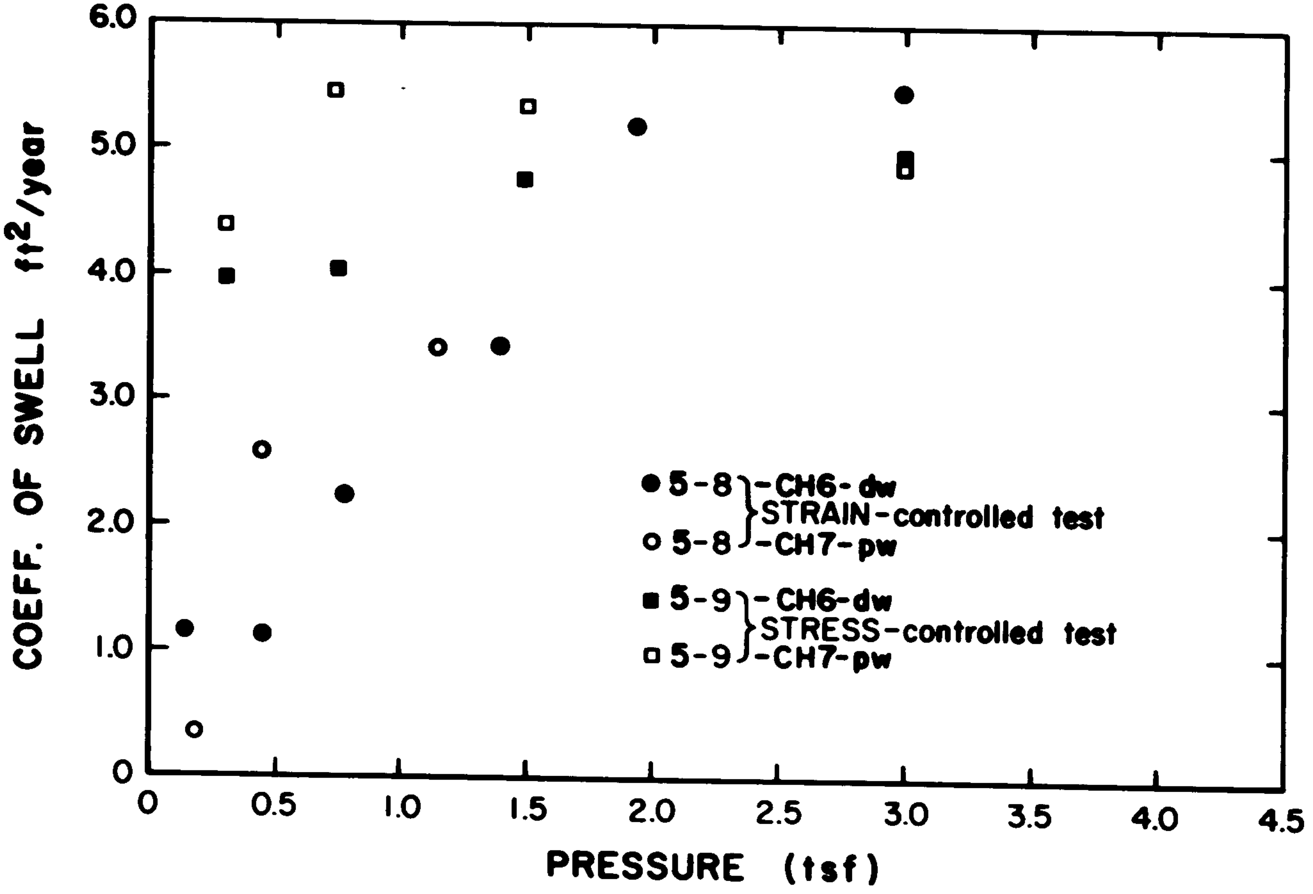


FIG. 7-23 PRESSURE - COEFFICIENT OF SWELLING RELATIONSHIP - MORDEN SHALE

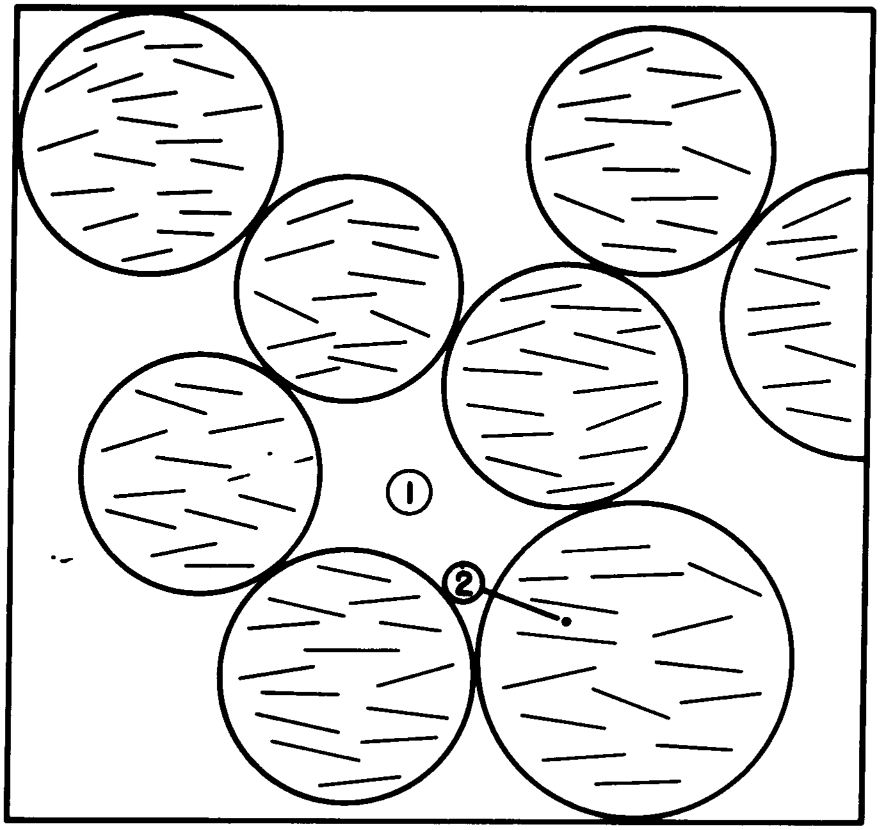
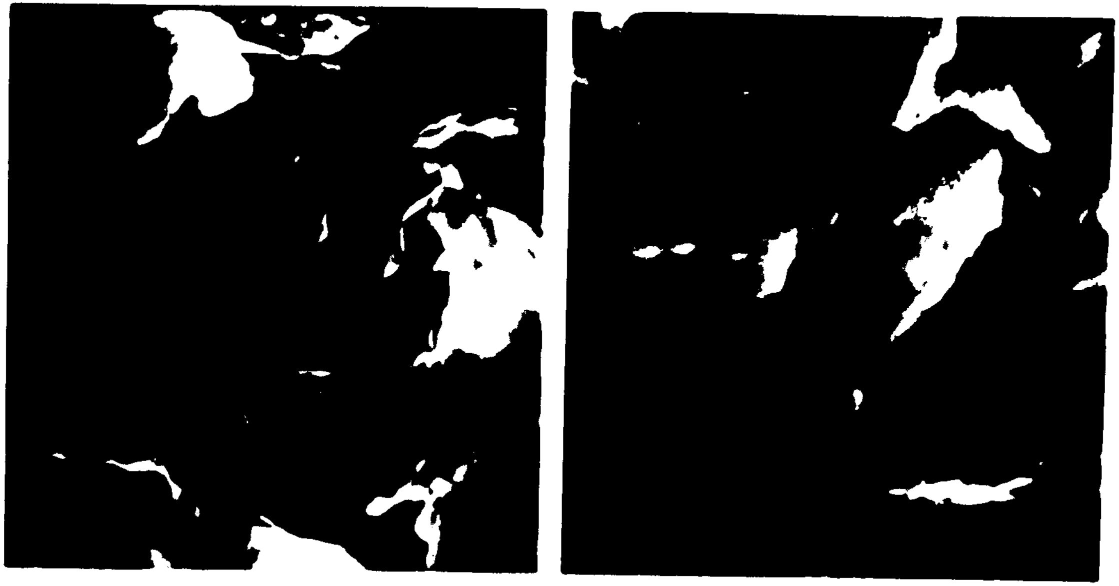


FIG. 7-24 CLUSTER MODEL (AFTER OLSEN, 1962)

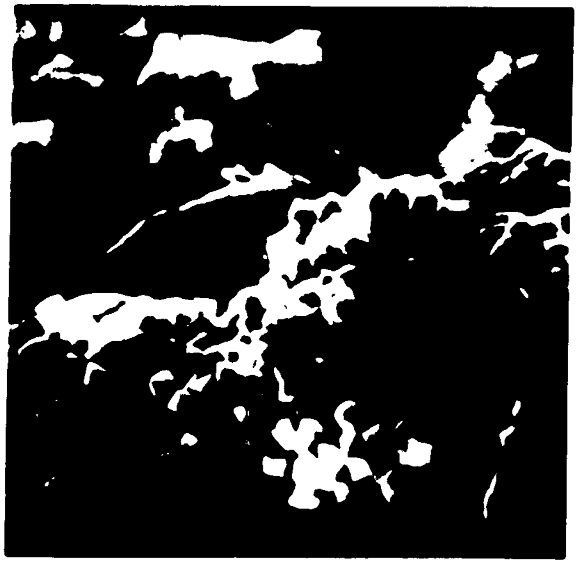


(a) ⊢⊣ 1 μm Horizontal plane (b) ⊢⊣ 0.5 μm Horizontal plane

(c) ⊢⊣ 2 μm Vertical plane

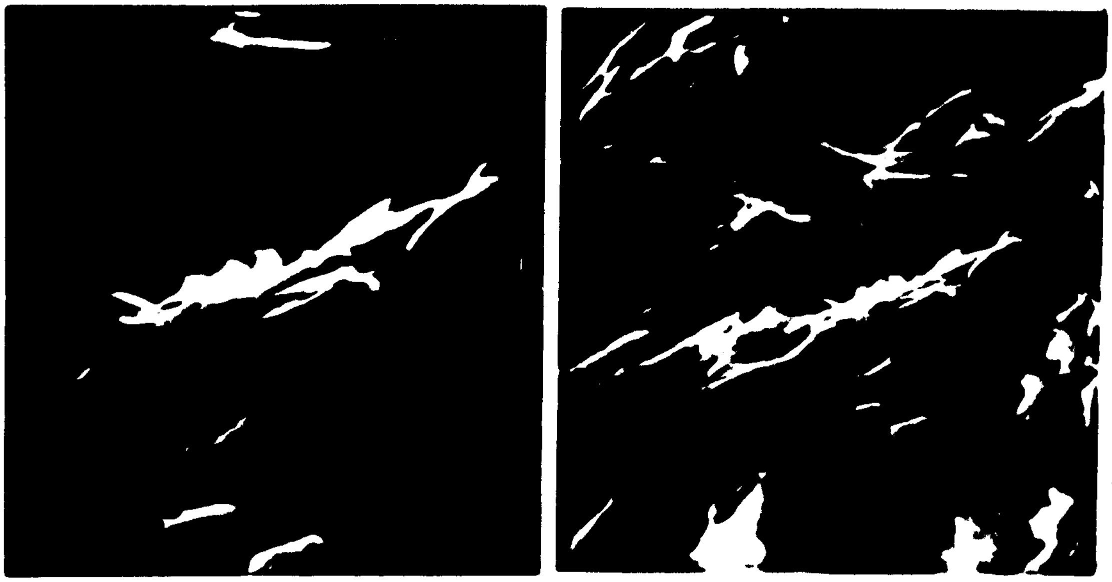
FIG. 7-25 a, b, c SCANNING ELECTRON MICROSCOPE PHOTOS OF MORDEN SHALE SAMPLES FROM 5ft. DEPTH.

(a) ⊢⊣ 1 μm Horizontal plane (b) ⊢⊣ 0.5 μm Horizontal plane

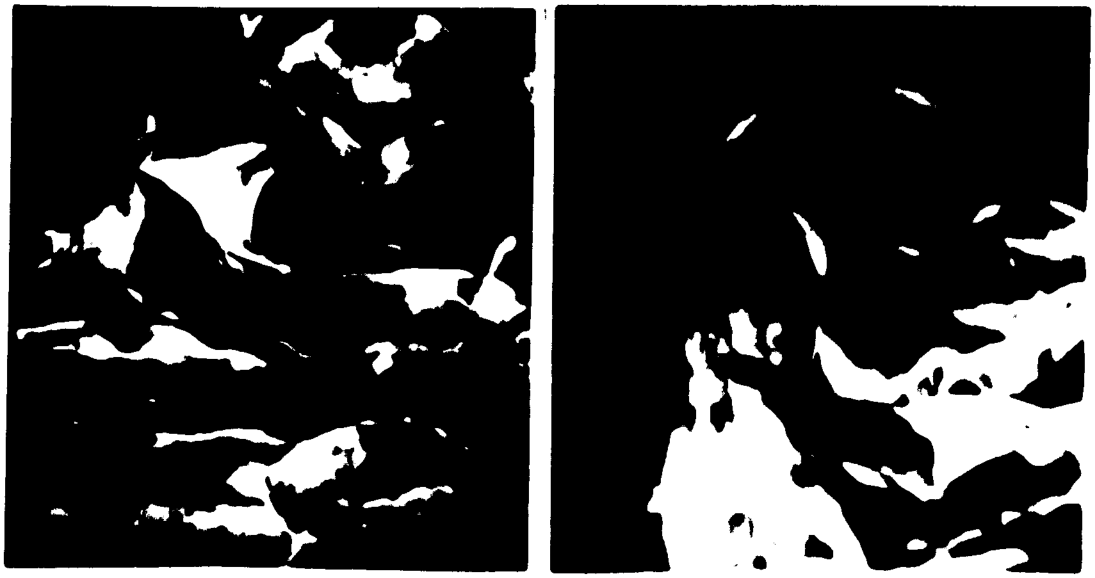


(c) ⊢⊣ 2 μm Vertical plane

FIG. 7-25 a, b, c SCANNING ELECTRON MICROSCOPE PHOTOS OF MORDEN SHALE SAMPLES FROM 5ft. DEPTH.

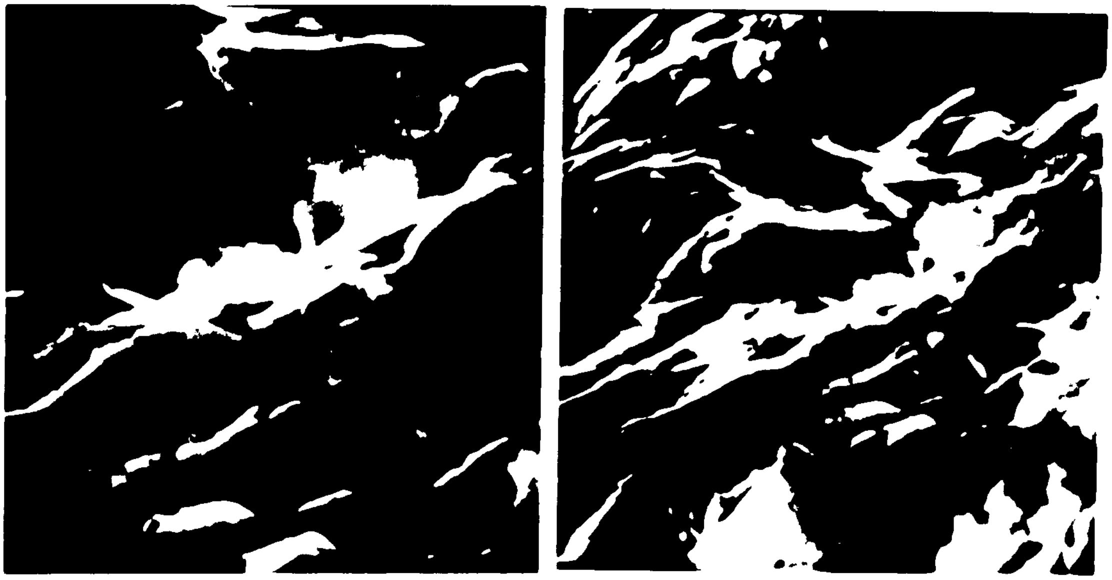


(d) 1 μm Vertical plane (e) 2 μm Vertical plane

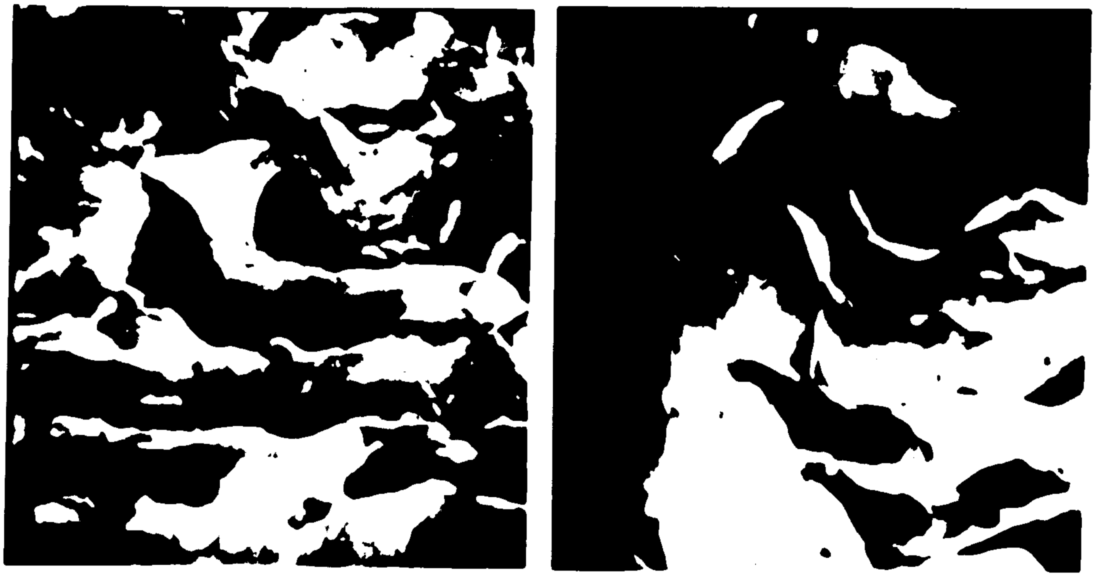


(f) 1 μm Horizontal plane (g) 1 μm Vertical plane

FIG. 7-25 d, e, f, g SCANNING ELECTRON MICROSCOPE PHOTOS OF MORDEN SHALE SAMPLES FROM 11ft. DEPTH.



(d) 1 μm Vertical plane (e) 2 μm Vertical plane



(f) 1 μm Horizontal plane (g) 1 μm Vertical plane

FIG. 7-25 d, e, f, g SCANNING ELECTRON MICROSCOPE PHOTOS OF MORDEN SHALE SAMPLES FROM 11ft. DEPTH.

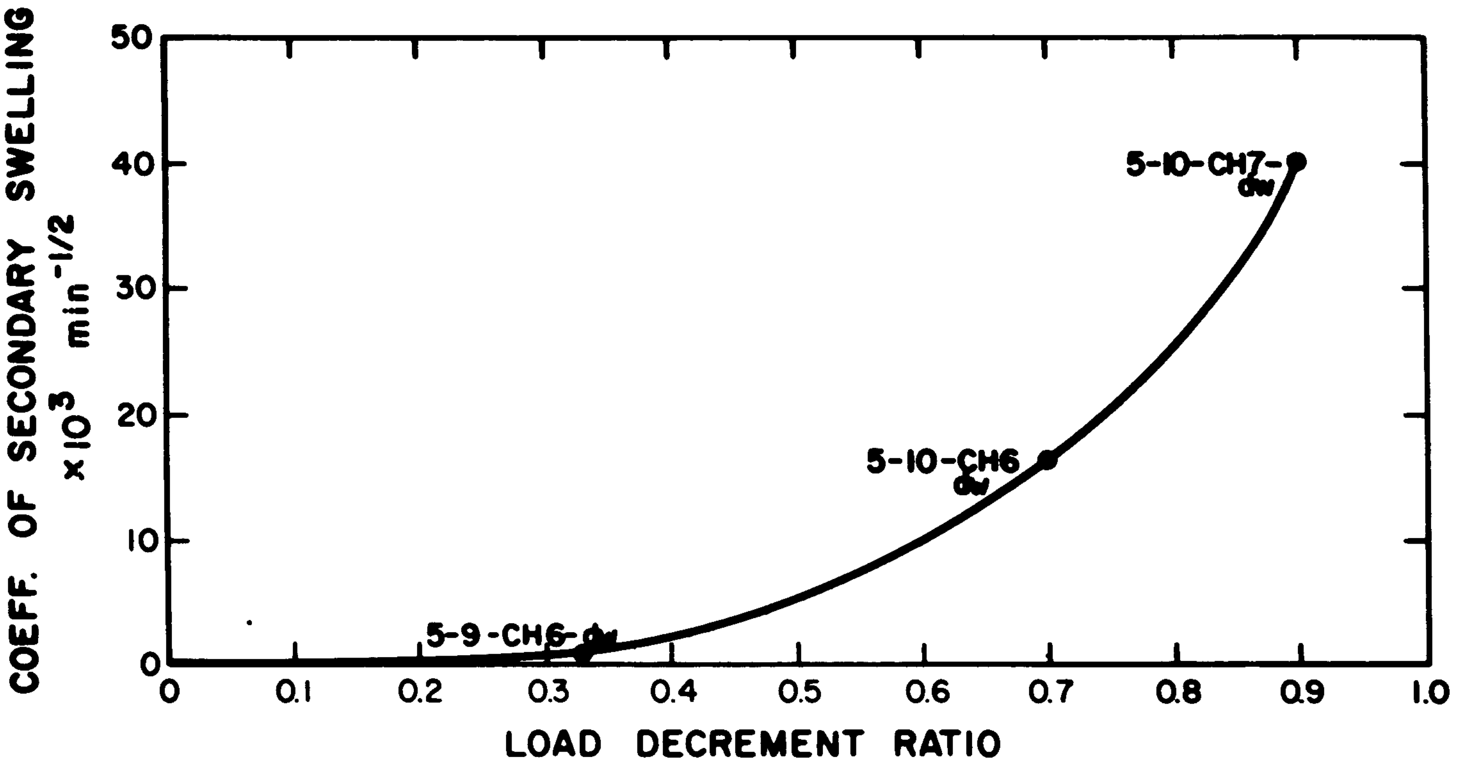


FIG. 7-26 VARIATION OF COEFF. OF SEC. SWELLING WITH LOAD DECREMENT RATIO

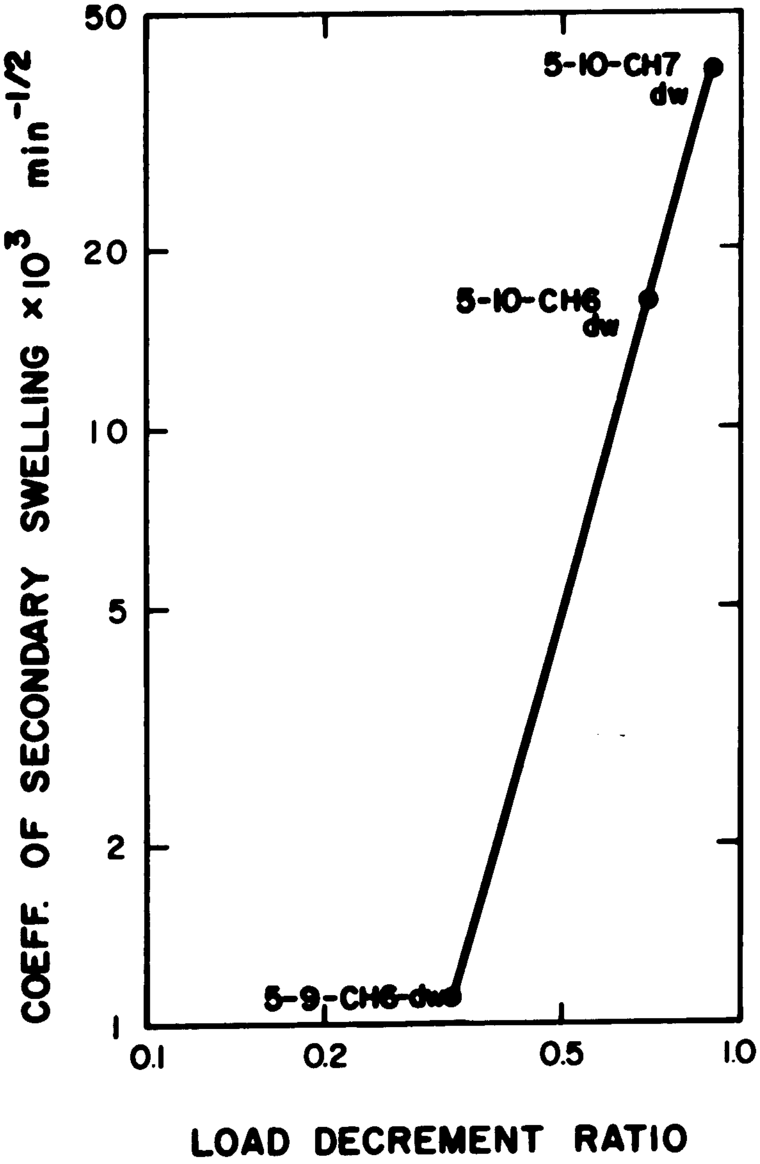


FIG. 7-27 VARIATION OF COEFF. OF SEC. SWELLING WITH LOAD DECREMENT RATIO

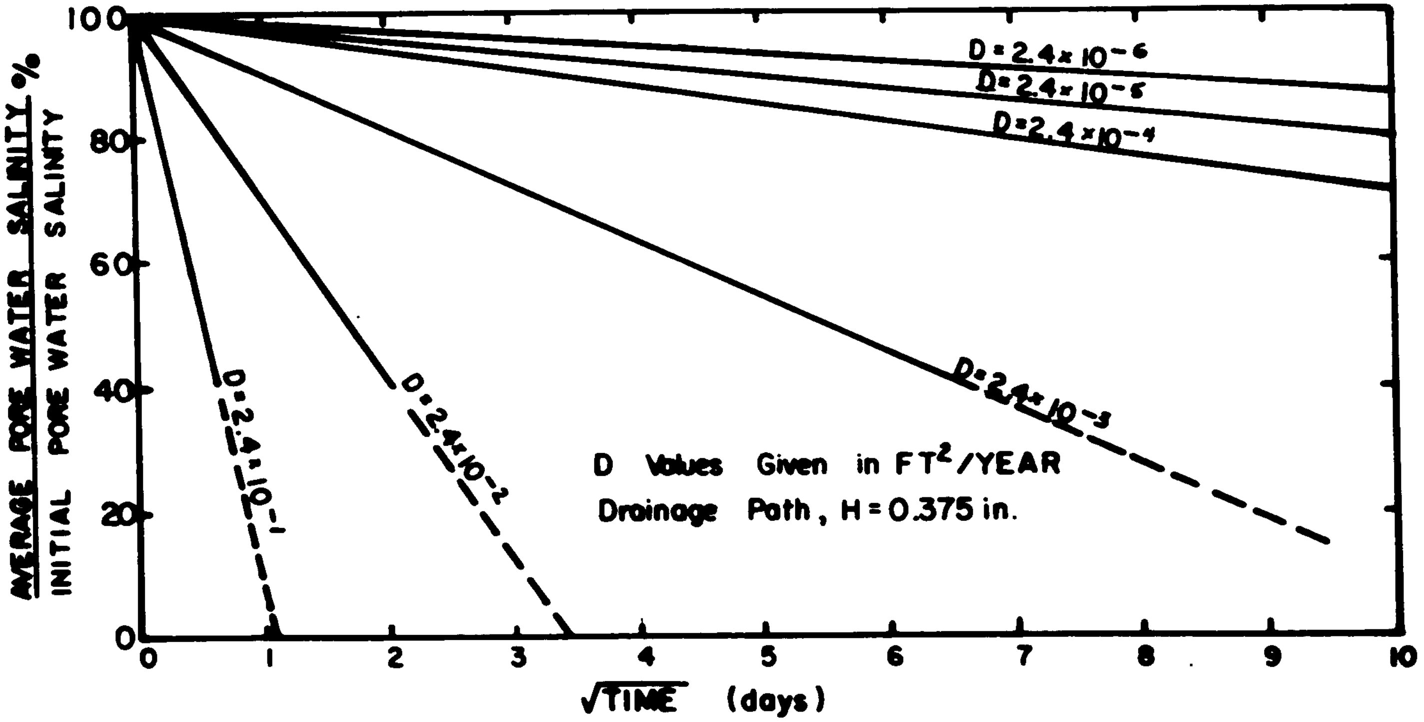


FIG. 7.28 VARIATION OF PORE WATER SALINITY WITH TIME

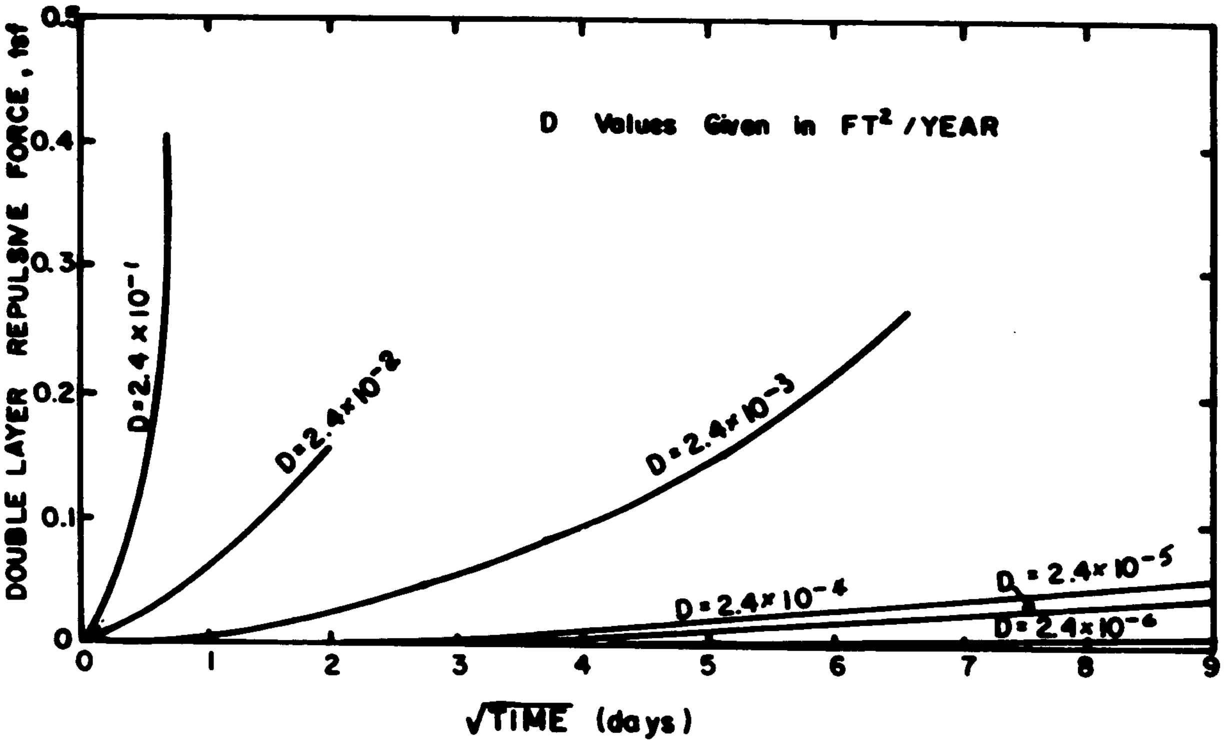


FIG. 7.29 INCREASE IN DOUBLE LAYER REPULSION WITH DIFFUSION-BLOCK BEARPAW SHALE

CHAPTER VIII

THE CONCEPT OF TRUE EFFECTIVE STRESS AND THE MECHANISM OF SWELLING

8.1 The Effective Stress

In the early 1920's, Terzaghi realised that the volume change characteristics of soils and their shear strength behaviour depend not on the total stress applied to the soil, but on the difference between the total stress and the pressure set-up in the fluid in the pore space. The concept of effective stress (for a fully saturated soil), as stated above, really consists of two statements.

(i) The effective stress is defined as:

$$\sigma' = \sigma - u \qquad (8\text{-}1)$$

where σ' = the effective stress

σ = the total stress

and u = pressure in the pore water.

(ii) The effective stress thus defined controls certain aspects of the behaviour of the soil, notably compressibility and shear strength.

Both of these statements are essential to the principle,for effective stress, defined as above, would be a useless quantity if it were not for its relation to soil behaviour.

For a saturated soil, from considerations of static equilibrium of the forces normal to a plane of unit area, one obtains the equation

$$\sigma = \sigma_c . a + u_w(1 - a) \tag{8-2}$$

where σ = the total normal stress acting on this plane

a = ratio of the area of mineral-mineral contact to the unit area (= a_m of Lambe, 1960)

σ_c = the average contact stress acting on the portion of the unit area, a(= $\bar{\bar{\sigma}}$ of Lambe, op. cit.)

and u_w = the pore water pressure acting on the portion of the unit area (1 - a) where there is water-mineral or water-water contact.

Equation 8-2 may be rewritten in terms of the average intergranular stress, σ_i', acting over the entire unit area as

$$\sigma = \sigma_i' + (1 - a)\, u_w \tag{8-3}$$

where $\sigma_i' = \sigma_c . a$ (8-3a)

Comparison of equations 8-1 and 8-3 shows that the intergranular stress is a function of the contact area whereas σ' does not depend upon a . The experiments by Laughton (reported in Bishop, 1960 and Skempton, 1961) have clearly demonstrated that the volume change characteristics of particulate materials are controlled by σ' and that they are independent of the value of a. Equation 8-2, which has been derived from principles of static equilibrium may now be rewritten as

$$\sigma = (\sigma_c - u_w)\,a + u_w \tag{8-4}$$

where $$\sigma' = (\sigma_c - u_w)\,a \tag{8-4a}$$

This equation means that it is the product of the average contact stress less the pore water pressure (buoyancy effect on the intergranular pressure) and the portion of the area over which there is mineral-mineral contact that controls the volume change behaviour of soils.

This approach has been found to be inadequate with clayey soils. Besides σ_c and u_w, there are other forces to be considered which arise from the peculiar nature of clays (a net change deficiency within the lattice structure and a high specific surface area) and the chemical environment. Following Lambe (1960)*, the equation for static equilibrium of normal forces perpendicular to a unit area for a saturated clay may be written as

$$\sigma = \sigma_c . a + u_w(1-a) + (R-A)\,(1-a) \tag{8-5}$$

---

*The notations used here are different from Lambe's (1960). His equations have been rewritten in terms of the convention followed in this thesis.

where (R-A) = the net interparticle stress due to the physico-chemical environment.

[Lambe (op. cit.) considers the physico-chemical component of the stress (R-A) as acting over the entire unit area under consideration, whereas here they are considered to be acting only in the interparticle region where there is no mineral-mineral contact.]

This equation may now be rewritten (similar to equation 8-4) as

$$\sigma = [\sigma_c - u_w - (R-A)]\ a + u_w + (R-A) \qquad (8\text{-}6)$$

According to Mitchell (1960), $u_w + (R-A)$ is the total pore water pressure**.

Now σ' defined in equation 8-1 as equal to $\sigma - u$, becomes

$$\sigma' = [\sigma_c - u_w - (R-A)].\ a + (R-A) \qquad (8\text{-}7)$$

Whether σ' defined as above governs the behaviour of clays or not needs investigation.

If a clay soil initially having a high salt concentration in its pore water so that the (R-A) component is equal to zero and at static equilibrium ($u_w = 0$) under a total stress is leached with a

---

**In all of the equations in this discussion, the pore water pressure u_w is defined as that pressure measured by a piezometer inserted into the soil mass and containing a fluid exactly identical with the fluid that would be squeezed from the soil mass if additional consolidation stresses are applied to the soil mass (Bailey, 1965).

very dilute salt solution, as the pore water concentration decreases (R-A) would increase. To maintain constant volume, the external load on the sample has to be increased* corresponding to the increase in (R-A). At the end of the test, when the soil sample has been leached completely, a new equilibrium condition would be reached. The final total stress on the sample would be in excess of the initial value by an amount equal to the increase in (R-A). Expressing these in equation form, initially,

$$\sigma = \sigma_i$$

$$u_i = 0$$

$$(R\text{-}A)_i = 0$$

$$\sigma'_i = \sigma_i - u_i = \sigma_i$$

At the end of the leaching test,

$$\sigma_f = \sigma_i + x$$

$$u_f = 0$$

$$(R\text{-}A)_f = x$$

$$\sigma'_f = \sigma_f - u_f = \sigma_i + x$$

where x is the increase in the (R-A) component measured. From the above, one can see that the effective stress σ', defined as $\sigma - u$ has increased during the leaching test whereas the volume of the sample

---

* This statement implies that during leaching, the fabric of the soil does not change. Leaching a quick clay may result in a decrease in volume due to the collapse of the initial flocculated structure to a more dispersed arrangement.

remained unchanged. This clearly indicates that σ' is not the effective stress that controls the volume change characteristics of all clayey soils. With reference to equation 8-6, one can see that

$$\text{initial } [\sigma_c - u_w - (R-A)_i].a = \sigma_i - u_i - (R-A)_i = \sigma_i$$

and

$$\text{final } [\sigma_c - u_w - (R-A)_f].a = \sigma_f - u_f - (R-A)_f = \sigma_i$$

The stress $[\sigma_c - u_w - (R-A)].a$ has not changed during the leaching process and neither has the sample volume (assuming no change in fabric).

It has been argued in Chapter II that a volume change of a soil sample may arise due to particle diminution, particle deformation and a particle reorientation with or without a change of fabric. All of these factors will contribute in different proportions to a volume decrease during consolidation. But the swelling of many fully saturated soil samples may be interpreted as resulting essentially from particle deformation and a particle reorientation without a change in fabric. In such cases, the above discussion demonstrates that it is $[\sigma_c-u_w-(R-A)].a$ that is effective in controlling volume change behaviour of soils. For granular soils, since $(R-A) = 0$, the stress that controls their swelling characteristics becomes $[\sigma_c-u_w].a$ which has been stated to be "the" effective stress for these soils (Section 8.4a). As was the case with the granular soils, for many active clays one can see that the volume change behaviour does not depend on the contact stress or the total pore water pressure $(u_w + R-A)$ or the value of a separately, but rather on the product of $[\sigma_c-u_w-(R-A)]$ and a . Therefore equation 8.6 has been rewritten in this thesis as

$$\sigma = \sigma^* + u_w + (R-A) \qquad (8\text{-}8)$$

where $$\sigma^* = [\sigma_c - u_w - (R-A)].a \qquad (8\text{-}8a)$$

Since this stress, σ^*, is effective in controlling the volume change characteristics of soils involving no change in fabric or particle diminution, it was thought appropriate to call it the "true effective stress" and the conventional effective stress, σ', is called the apparent effective stress.

8.2 Application of the above principles to the study of the swelling behaviour of soils

Comparison of equations 8-1 and 8-8 yields the relationship between σ' and σ^* as

$$\sigma' = \sigma^* + (R-A) \qquad (8\text{-}9)$$

In the above section it has been concluded that σ^* is the stress that governs swelling behaviour of many saturated soils. But a study of the soil behaviour in terms of σ^* is possible only if (R-A) can be estimated (or measured) with reasonable accuracy.

The physico-chemical components of interparticle forces are numerous (for eg. Lee, 1968) and it is not possible to calculate these forces accurately for any clayey soil . However, the (R-A) component of a sample could be suppressed, say, using a very high salt concentration for the ambient water, so as to measure the (R-A) of another identical sample separately. The (R-A) thus found for a sample experimentally may be used to check the accuracy of any method proposed for its prediction.

Pressure-swell relationships of undisturbed samples of the block Bearpaw shale and the Morden shale in their natural pore water and in distilled water have been obtained in this study. Following the discussion in the previous paragraph, a relationship of this kind can be employed to evaluate the (R-A) component for the sample in distilled water, only if the physico-chemical component for the sample in its own pore water is equal to zero.

The percent swell-log of equilibrium pressure relationship for the samples in pore water (1-4-WF4-pw and 5-8-CH7-pw) was found to be a straight line (Figs. 7.6 and 7.16). During the initial part of the swelling test, the concentration of the cell water for the sample 1-4-WF4-pw was increased to almost two times the intial value and a part of the straight line relationship for that sample was obtained with this concentration. At such high concentrations, to assume that the (R-A) component would be equal to zero seems quite reasonable. The pressure-swell relationship for that sample being identical in its own pore water and in a cell water of very high salinity, indicates that the (R-A) component for this sample (and therefore for the Morden shale sample in its own water) is zero. It should be noted here that such a straight line e-logσ^* curve will be obtained, only if the change in the fabric of the soil sample in the range of σ^* considered is negligible. The shale samples studied in this thesis are highly over-consolidated and within the range of pressure investigated here, the assumption of no change in fabric is reasonable. Thus for the samples considered here in their own pore water,

$$\sigma^*_{pw} = \sigma'_{pw} = \sigma_{pw}$$

since $u_{pw} = 0$

and $(R\text{-}A)_{pw} = 0$

(where the suffix 'pw' stands for the sample in its own pore water). This means that the percent swell-apparent effective stress plot is the same as the percent swell - σ^* plot for samples 1-4-WF4-pw and 5-8-CH7-pw. Both of these samples yield a swelling index value of 0.075 (Figs. 7.6 and 7.16) which is of the same order as values given by Olson and Mesri(1970) for kaolinites, illites and smectites in a non-polar liquid (carbon tetrachloride), for which $\sigma^* = \sigma'$ (Table VIII.3).

Sample 1-4-CH5-dw gave a swelling pressure at constant volume equal to that for sample 1-4-WF4-pw. At this stage, as mentioned earlier, the pore water concentration is unchanged. The physicochemical component being equal to zero, the swelling pressure would in magnitude be equal to the true effective stress, σ^*. Once swelling is permitted, due to the mixing of the pore water and the distilled water of the cell, the pore water concentration drops to a normality of 0.02 which is the concentration of the cell water at the end of the first stage of swelling. This dilution results in an increase in the equilibrium pressure at all values of swell greater than zero, over the corresponding equilibrium pressures for samples 1-4-WF4-pw, by a value equal to (R-A) for that swell.

$$\sigma_{dw} = \sigma^*_{dw} + (R-A)_{dw}$$

$$= \sigma_{pw} + (R-A)_{dw}$$

since $u_{dw} = 0$

That is, $\sigma^*_{dw} = \sigma_{dw} - (R-A)_{dw}$

$$= \sigma_{pw}$$

But it has been shown earlier that

$$\sigma_{pw} = \sigma^*_{pw}$$

therefore, $\sigma^*_{dw} = \sigma^*_{pw}$

This means that for a sample at equilibrium at any water content, the true effective stress, σ^* is a unique value irrespective of the physico-chemical environment and the value of σ^*_{dw} calculated as $\sigma_{dw} - (R-A)_{dw}$ (for $u_{dw} = 0$) should be exactly the same as σ^*_{pw}. Hence, the value of (R-A) estimated for a particular amount of swell and the difference between the equilibrium pressures in distilled water and pore water at that swell should be equal. Such (R-A) values determined experimentally have been given in Tables VIII.1 and VIII.2.

8.3 The Evaluation of (R-A)

The evaluation of (R-A), the physico-chemical component of the interparticle force has been discussed in detail by Bailey (1965) and Lee (1968). The correspondence found by Bailey (op. cit.) for Na-montmorillonites with very low concentrations (10^{-4} molar) at low pressures between the (R-A) force as predicted from double-layer repulsion theory and the apparent effective stress led the author to attempt to see whether the net (R-A) could be approximated from double-layer theory alone or not. The success of such an approach would only mean that the sum of all other repulsive forces and all attractive forces for these (monovalent rich swelling) soils is equal to zero.

8.4 The double-layer repulsive force

The equation for calculating the double-layer repulsive force (Py) is given in Appendix A. From this equation it can be seen that Py depends upon the concentration of the bulk solution (Co) valency of the ions and the interparticle distance (2d) for a given soil of known surface charge density.

The cell water has often been considered as the bulk solution by many (for eg. Yong and Warkentin, 1966). According to this approach either all the water within the soil is bound water or the concentration of the free pore water is the same as the cell water. This is not always the case. It has been mentioned that it is the relation between the double-layer water and the free pore water that affects the swelling pressure, and for this reason, it is the concentration of the free pore water that has to be used as Co for the calculation of Py. (Bolt, 1956, refers to the value of Co as that of the equilibrium solution which is pressed out upon loading of the clay specimen.)

For Bearpaw shale-block samples and Morden shale samples during swelling pressure measurements at no volume change, the salt concentration of the pore water remained unchanged, irrespective of the cell water used. Therefore the value of Co to be used is that of the natural pore water. Once a small amount of volume change is permitted, the pore water of the sample becomes diluted by the entry of the cell water for distilled water samples, reaching a pore water salinity equal to the final cell water concentration which had increased simultaneously. Therefore, at all magnitudes of swell other than 0 , Co would be the pore water salinity equal to that of the final cell water. For the Morden shale sample 5-8-CH6-dw, by allowing a volume increase and subsequent reconsolidation, dilution of pore water was induced and the corresponding (R-A) has also been measured experimentally at 0 percent swell.

The estimation of the interparticle distance has been discussed in detail by Bailey (1965). The estimation of the half-distance d from a knowledge of the water content and its specific surface is possible employing equation A-9, if an assumption is made that the particles are of infinite extent and are equidistant throughout the sample. This assumption eliminates any possible end effects, the presence of a dead volume etc., discussed by Bailey and for those reasons does not indicate the actual spacing between the clay particles. (Whether an estimation of actual interparticle distance is possible or not for a natural soil is itself debatable.) Using equation A-9, the particle half-spacing that would be calculated yields only an equivalent d .

Once this assumption has been made, d can be calculated once the water content of the soil sample and its specific surface are known. The specific surface that is to be used for a natural material would not be the specific surface of the clay fraction (<2μ fraction) alone, but should be a weighted value for the total soil. In Table III.1, the weighted specific surface has been calculated as follows. The <2μ portion of the natural soils was separated out using a conventional hydrometer test and was used for the surface area determination using the EGME technique. (See Appendix C.) This surface area multiplied by the proportion of the <2μ fraction is the value entered in Table III.1. This would be the average specific surface only if the total surface area thus predicted participates in swelling. In nature the clays are present as clusters (Fig. 7.25). Tests by Norrish (1954) as reported by Bailey (1965) clearly indicates that montmorillonites with Ca-cations do not swell beyond approximately 19 Å (with a 2d of about 10 Å) even in distilled water. This is of enormous importance in understanding the behaviour of the shales under investigation. Cation exchange capacity studies on these shales (see Appendix C), have clearly shown the presence of Ca, Mg, Na and K cations in the exchangeable positions. The presence of Ca in the exchangeable complex and the presence of the shale as clusters and the preferential attraction of Ca ions towards clays would all indicate that Ca is probably the predominant cation within the cluster. If this is the case, the clusters do not swell even in the presence of distilled water and any swelling that occurs due to physico-chemical effect is only due to the external surface of the clusters. An estimation of this external surface could be made from a knowledge of the mineralogical composition of the material and a proportion of the individual ions in the exchangeable complex (Pawluk, 1972). The total specific surface of the block Bearpaw shale is 266 m^2/gm (Table III.1). The clay fraction of the shale consists of montmorillonite (45%), illite (45%) and kaolinite (10%). The clay fraction had 34.6, 3.6, 18.8 and 8.3 meq/100 gm of soil of Na, K, Ca and Mg respectively in its exchangeable position. Assuming that the ratios of the external to total surfaces of various clays saturated with different ions given by Kinter and Diamond (1958) are valid for this shale, the external surface could be calculated as 116 m^2/gm for the block Bearpaw shale, which is about 43% of the total surface area given in Table III.1. This is the value that has been used as the effective specific surface for this shale. Such an estimate of the effective specific surface could not be made for the Morden shale samples since the mineralogical composition of this material could not be determined due to the presence of interlayering of illites and montmorillonites. Assuming that the values obtained for the block Bearpaw shale samples are reasonable for Morden shale as well (the geological history, cation exchange capacity, % clay, etc. being similar), the same proportion of external to total surface area has been used for this shale too. This assumption yields the effective area for the Morden shale of about 66 m^2/gm.

The double layer repulsive force, P_y values have been calculated as outlined in detail in Appendix A and are given in Tables VIII.1 and VIII.2 for the Bearpaw and Morden shale samples respectively. The agreement between the predicted and observed values is good for both shales. The accuracy of the prediction of the physico-chemical component of the interparticle force (R-A) as a double layer repulsive force, Py employing the method explained above is within about 15%.

The value of Py calculated for the natural soil with initial salinity yielded a value of zero for both of these shale samples. If Py was zero at 0% swell, it would mean that with increasing swell, d increases, and as Co remains constant for the samples in their own pore water, (R-A) would remain zero throughout. Thus the swelling curves of these two shale samples in their own natural pore water as plotted in Figs. 7.6 and 7.16, is actually expressed in terms of σ^* (since $\sigma' = \sigma^*$ for $(R-A) = 0$).

8.5 Soil parameters in terms of the true effective stress

Comparing equations 8.1 and 8.6, one gets an expression for the true effective stress, σ^*, as the apparent effective stress, σ' ($= \sigma - u$), minus the physico-chemical component, (R-A). In the earlier section this (R-A) component for the shales tested has been shown to be equal to the double layer repulsive force provided the effective specific surface could be estimated with reasonable accuracy.

The volume change characteristics of soils may now be studied in terms of σ^* as follows.

$$\sigma^* = \sigma' - (R-A) \qquad (8\text{-}7)$$

$$\frac{d\sigma^*}{de} = \frac{d\sigma'}{de} - \frac{d(R-A)}{de} \qquad (8\text{-}8)$$

Defining $\frac{de}{d\sigma^*} = a^*_{vs}$

$$\frac{de}{d(R-A)} = b_{vs}$$

and $\frac{de}{d\sigma'} = a_{vs}$

where a_{vs} is the (conventional) coefficient of swelling (or compressibility) and a^*_{vs} and b_{vs} are similar soil properties in terms of σ^* and (R-A) respectively. Equation 8-8 becomes

$$\frac{1}{a^*_{vs}} = \frac{1}{a_{vs}} - \frac{1}{b_{vs}}$$

i.e. $$a^*_{vs} = \frac{a_{vs} \cdot b_{vs}}{b_{vs} - a_{vs}} \qquad (8.8a)$$

The value of a_{vs} could be obtained by conventional tests and the coefficients a^*_{vs} and b_{vs} may now be found by the separation of σ' into σ^* and (R-A) as discussed in the previous section. The estimation of a_{vs} and b_{vs} for the Bearpaw shale block sample and Morden shale samples may be made using Figs. 8.1 and 8.2, since $\sigma^*_{pw} = \sigma'_{pw}$ for these two samples.

8.6 Concluding remarks

The swelling characteristics of many soils are governed by its true effective stress, σ^*. A change in (R-A) alone, without changing σ^* does not cause a change in volume of the sample provided the fabric of the soil remains unchanged as demonstrated in Section 8.1. The slaking tests results of Eigenbrod (1972) on a Bearpaw shale sample also support this. Eigenbrod conducted slaking tests on two "identical samples", one in distilled water and the other in its own natural pore water. The final water content reached by both of these samples was almost the same. The two samples subjected to slaking tests had the same initial σ^*_i and a final σ^*_f tending to zero. The change in σ^* being the same for both of these samples has resulted in an equal change in water content, irrespective of the external water used for the slaking test.

The validity of using σ^* to study the swelling behaviour of soils is further substantiated by Fig. 8.3. In this figure the particle or domain (cluster) half spacing is plotted against σ^* for the Na and Ca montmorillonites, taken from Bailey's (1965) report. The values of σ' and Py have been read from the graphs of Bailey and σ^* calculated as $\sigma' - Py$. All of these samples yield a straight line relationship in a semi-log plot with swelling indices in the range of 0.025 - 0.065 which are comparable to the swelling indices given in Table VIII.3. The difference in the value of the swelling indices of these materials is a result of the difference in the fabric between these three samples. The Na montmorillonite in 10^{-4}M NaCl solution had been preconsolidated to about 55 kg/cm^2 compared to the Na-montmorillonite in 10^{-3}M NaCl solution which had been consolidated only to 10 kg/cm^2. The lower consolidation pressure and the higher salt concentration for the latter sample and the higher valency of the cation for the Ca-montmorillonite have possibly resulted in a more flocculated structure for these samples which in turn has resulted in higher swelling indices for them than for the Na-montmorillonite in 10^{-4}M NaCl. Therefore, a unique d-σ^* relationship is not obtained for all of the three samples due to a change in their fabric. This figure indicates that the swelling of these soil samples could be studied in terms of σ^* and that σ^* could be estimated as σ' - Py and that the d - σ^* relationship in a semi-log plot yields a straight line. It should also be noted that σ^* tends to zero at very large particle half-spacings, about 80Å for the Na-montmorillonite in 10^{-4} NaCl (see Fig. IV.1. of Bailey, 1965) with corresponding water contents in the proximity of the liquid limit. This means that, at such high water contents, the shear strength of this soil (which is also dependent on σ^*) is negligible, which is a fact.

TABLE VIII.1

PREDICTED AND OBSERVED VALUES OF THE DOUBLE-LAYER REPULSIVE FORCE: BEARPAW SHALE-BLOCK

DATA:

Concentration of original pore water = 0.130N

Concentration of bulk solution: 1-4-WF4-pw = 0.150N

P_y for sample: 1-4-WF4-pw = 0.0 tsf

Concentration of bulk solution: 1-4-CH5-dw = 0.02N

x_0 = 3.68 A

Effective specific surface = 116 m^2/g

% Swell	water content w %	particle half - spacing d A	P_y tsf. (predicted)	(eq. pr. 1-4-CH5-dw)-(eq. pr. 1-4-WF4-pw) tsf
0	30.5	26.3	0.70	0.0
2	32.3	27.8	0.58	0.66
4	34.1	29.4	0.48	0.57
6	35.9	31.0	0.40	0.47
8	37.7	32.5	0.32	0.44

TABLE VIII.2

PREDICTED AND OBSERVED VALUES OF THE DOUBLE-LAYER REPULSIVE FORCE: MORDEN SHALE

DATA:

Concentration of original pore water = 0.271N

P_y for sample 5-8-CH7-pw = 0.0 tsf.

Concentration of bulk solution: 5-8-CH6-dw = 0.02N

x_o = 3.6 A

Effective specific surface = 66 m^2/g

% swell	water content w %	particle half-spacing d A	P_y tsf (predicted)	(eq. pr. 5-8-CH6-dw) - (eq. pr. 5-8-CH7-pw) tsf
0	19.5	29.5	0.53	0.50
2	20.6	31.2	0.43	0.37
4	21.7	32.9	0.35	0.38
6	22.8	34.5	0.28	0.35
8	23.9	36.2	0.22	0.25

TABLE VIII.3

VALUES OF SWELLING INDICES

Soil	Cell water	Swelling index	Source
Bearpaw shale -block	Natural pore water	0.075	Sample 1-4-WF4-pw
Morden shale	Natural pore water	0.075	Sample 5-8-CH7-pw
Kaolinite	Carbon tetrachloride	0.05	Olson and Mesri (1970)
Illite	Carbon tetrachloride	0.04	Olson and Mesri (1970)
Smectite	Carbon tetrachloride	0.03	Olson and Mesri (1970)

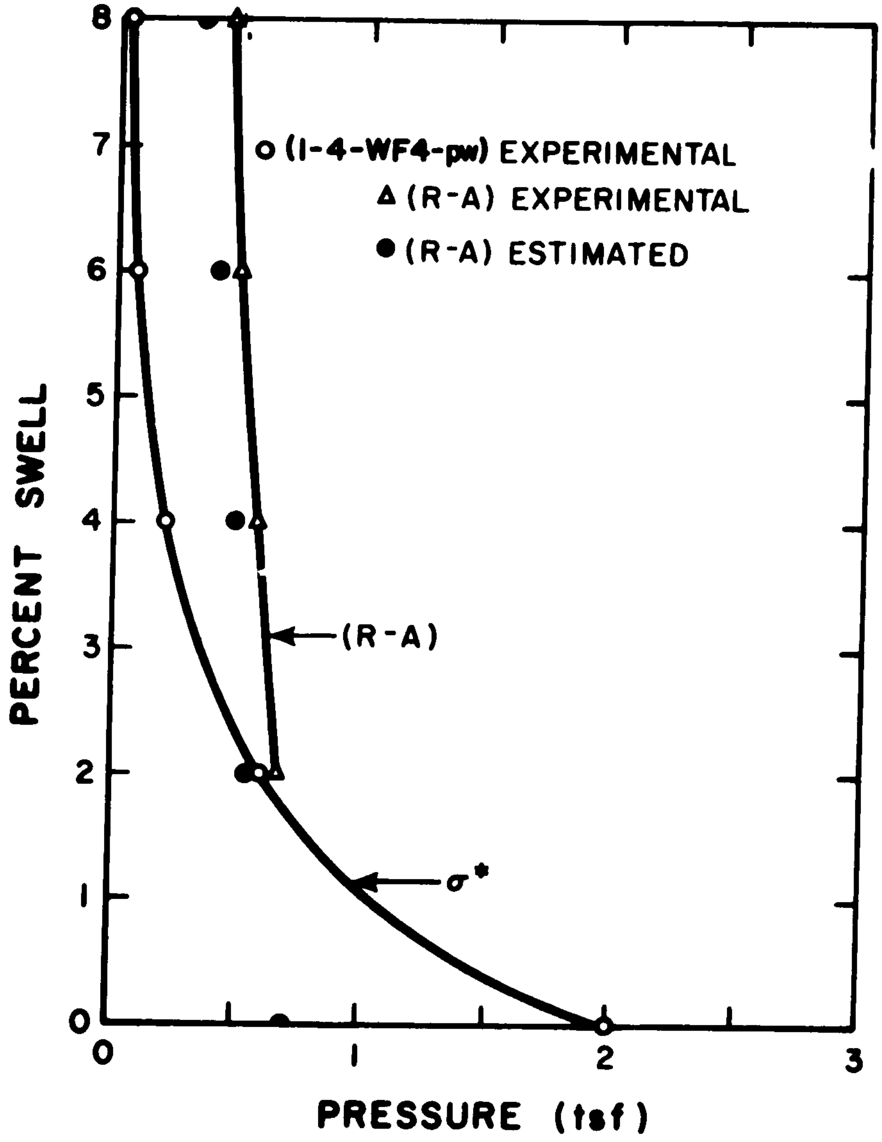


FIG. 8-1 PRESSURE - SWELL RELATIONSHIP - BLOCK BEARPAW SHALE

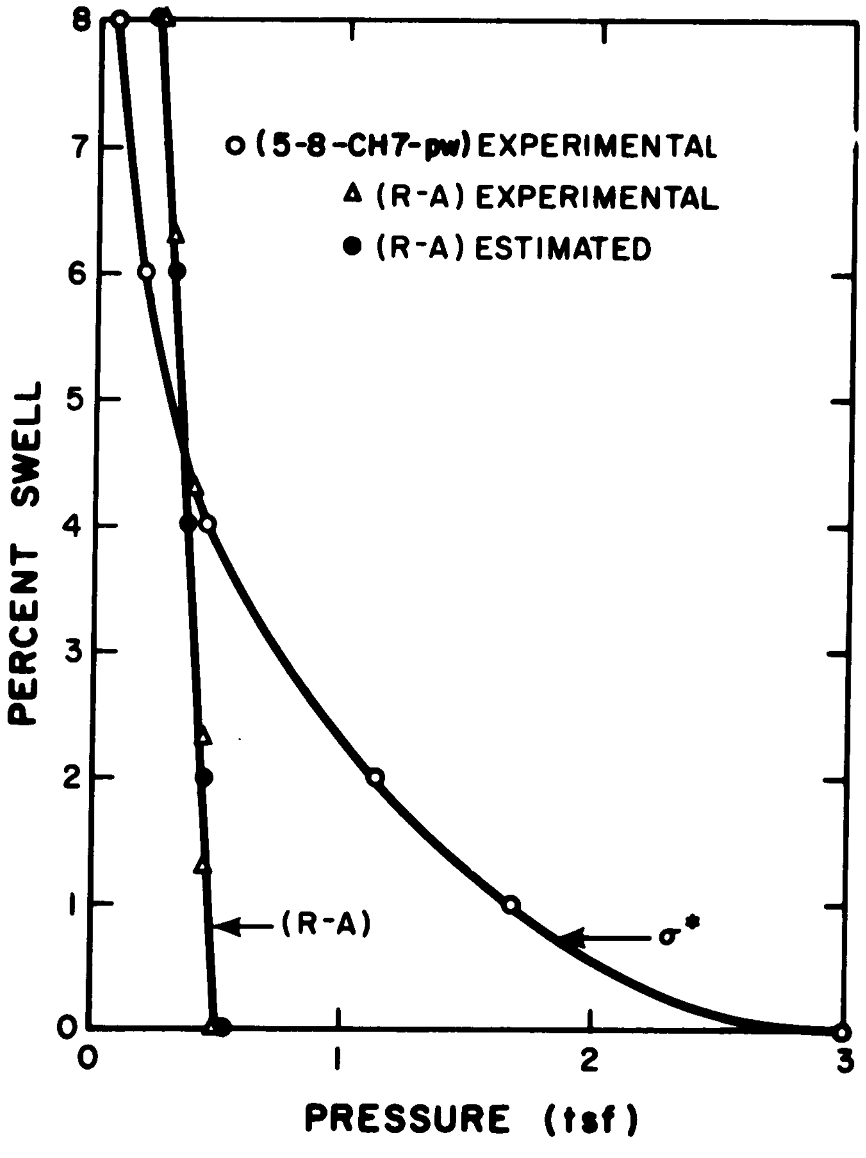


FIG. 8-2 PRESSURE - SWELL RELATIONSHIP - MORDEN SHALE

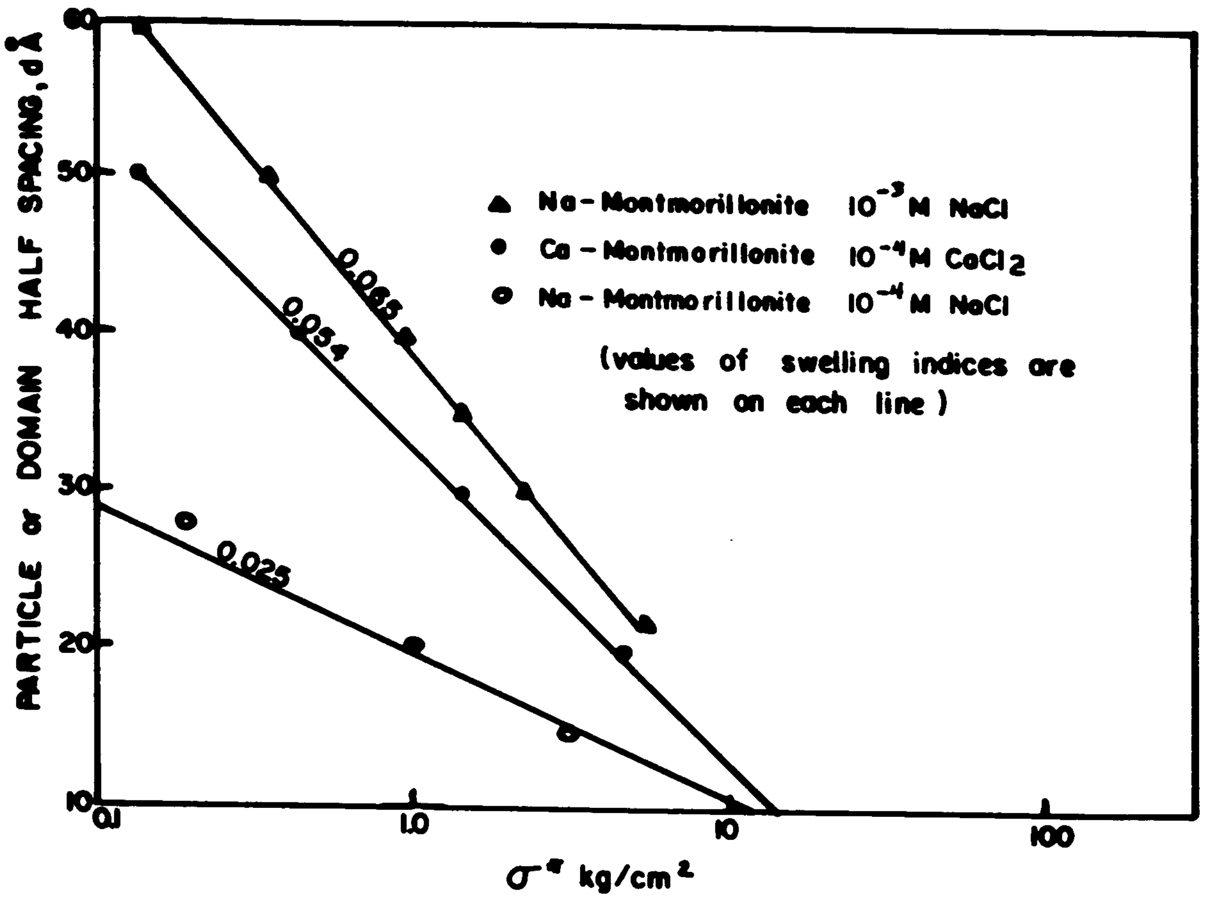


FIG. 8.3 d-σ^* RELATIONSHIP FOR THE SWELLING OF PURE MONTMORILLONITE

CHAPTER IX

PREDICTION OF SWELLING IN THE FIELD

9.1 General

The results of tests on Morden shale samples (mentioned in Chapter VII) have been used here to predict the swelling at various depths due to spillway excavations at the site of the Morden dam. The Morden dam is situated at Morden in the Province of Manitoba, Canada, about 75 miles southwest of Winnipeg. The details of the spillway excavations and the observations of the field swelling given in this chapter have been provided by Mr. N. Iverson, Chief, Soil Mechanics and Materials Division, PFRA.

9.2 Construction details and field observations

The original construction of the spillway was commenced in 1941 and it involved very little unloading (in the order of 1 to 3 ft.) in the area of the present inlet channel. The new construction in 1953 involved an excavation of a 30 ft. deep channel with 3:1 side slopes to elevation of about 1,072. Fig. 9.1 shows a typical section. The water level in the reservoir fluctuated around elevation 1,072 from 1953 onwards.

Four rebound gauges (RG1, RG2, RG3 and RG4) were installed in 1953 shortly after the excavation of the inlet channel was completed.

Fig. 9.2 shows a plan of the new spillway, the location of the various rebound gauges as well as the test holes. The gauges consisted of a series of telescoping pipes of varying lengths driven to predetermined depths in a drill hole. Fig. 9.3 shows a rebound gauge. Readings were continued in the above mentioned four gauges until 1963 when the gauges became inoperative due to severe rusting. In 1963, the two gauges near the bottom of the left slope of the channel, RG1 and RG2 were replaced. The cumulative heave and final water content profiles as of 1970 are shown in Fig. 9.4. The heave is still being measured at these two gauges at 1 year intervals.

Three holes were drilled on the left slope of the spillway channel in the vicinity of the rebound gauges in 1970. (See Fig. 9.2). The test holes AH-101 and AH-102 were drilled with a 4 in. hand auger. Continuous samples were obtained with 3 in. shelby tubes. These samples were ejected, classified and tested for water content. They were then wrapped with cheese cloth, waxed and stored in a humid room. The samples of Morden shale, described in the previous chapter, were prepared from the AH-101 hole.

9.3 Prediction of total heave and final water content

(i) Introduction

It is of concern here to see whether the laboratory test results on the samples of Morden shale would predict the heave or the increase in water content with depth due to the spillway excavation. To predict these, one needs to know the initial and final effective stresses, the swelling index and the rate of swelling.

(ii) The initial effective stress

The estimation of the initial effective stress is possible if the average density of the overlying soil and the ground water table is known. The average density of the soil could be approximated with reasonable accuracy from the known specific gravity of soil grains and initial water content data. But since no information is available regarding the initial ground water table, the pore pressure variation and hence the initial effective stress at any depth cannot be evaluated. The initial effective stress has therefore to be estimated indirectly.

The water content variation with depth at the time of excavation (1953) may be used to estimate the initial swelling pressure from the moisture content - swelling pressure relationship for the Morden shale (Fig. 7.22), obtained from laboratory tests. Here the assumption made is that the moisture content - swelling pressure relationship is unique for all depths and also extends beyond the range of moisture contents measured in the laboratory. The values of initial vertical effective stress computed as mentioned above are tabulated in Table IX.1, column 3. A closer study of Fig. 7.22 would reveal the limitation of this approach. In this figure a straight line relationship was obtained between three Morden shale samples, which have been prepared from depths 10.5 - 11.3 ft., 9.5 - 10.3 ft., and 5 - 5.7 ft. respectively in hole AH-101 (Table V.1). The electron microscope studies of the Morden shale samples exhibited evidence of chemical weathering at depths 5 ft. and above. This means that of the three samples mentioned above 5-14-CH5-dw is weathered, whereas the others are relatively unweathered. The depth of excavation for the spillway channel was of the order of 30 ft. That means that the depth from where the sample 5-14-CH5-dw has been prepared was about 35 ft. below the original ground level. At such depths the effect of weathering is essentially absent. Thus the above mentioned approach predicts the swelling pressure of the soil at the time of excavation, when at all depths of interest here, the soil was unweathered, by means of a relationship (Fig. 7.22) obtained from both weathered and unweathered samples.

The swelling pressure variation with depth may also be evaluated by another approach. The swelling pressure of any soil sample can be determined at the time of excavation, if a soil sample had been taken at that time (before any change of water content takes place) and tested. In this case, the samples had been taken 17 years after excavation by which time the water content had increased at all depths. (Water content measurements were made by the PFRA at the time of sampling in 1970 up to a depth of about 12 ft. below the present spillway level). The water contents of all the samples at the time of present testing were lower than the ones obtained in 1970 indicating possible drying during the storage period. The initial moisture content of a sample from a depth of 9 ft. below the present ground level was 19.75 in 1953 (at the time of excavation). This had increased to 21% in 1970. The samples 5-8-CH6-dw and 5-8-CH7-dw which were prepared from depth of 9.5 ft. gave water contents of 19.5 and 19.65% at the time of testing. These present water contents are very close to the ones that existed initially at these depths. Assuming that the change in water content during swelling and storage has not affected the sample properties, (no signs of weathering could be noticed at these depths through electron microscopic studies), the swelling pressure measured may be assumed to be equal to that which existed at these elevations at the time of the spillway excavation. By assuming hydrostatic equilibrium[+] at the start of excavation (and the ground water table at +1075[*]), the swelling pressure at any other depth D can be estimated as the swelling pressure at depth 9 ft. + (D-9.0) γ_{subm}. The initial effective stresses determined in this way are tabulated in Table IX.2, column 2.

(iii) <u>The final vertical effective stress</u>

For simplicity only point RG4 (Fig. 9.2) is being considered here. This point being symmetrical with respect to the cutting and quite far from the side slopes, the vertical effective stress would be equal to the overburden pressure due to the soil above plus the weight of the spillway slab less the pore water pressure. The pore water pressure may be calculated as the depth below the ground level times γ_w, since according to PFRA reports, the water level in the reservoir has fluctuated around the elevation 1,075 after 1953*. The bottom of the spillway cutting at RG 4 falls at an elevation of about 1,073.

[+] The calculation of the pore pressure from the vertical effective stress estimated by the earlier method yields an excess hydrostatic pressure distribution which is quite unlikely, since these shales are still swelling.

[*] Since no water table data near the present spillway cut was available, elevation 1075, has been taken as the ground water table at the location of RG4.

(iv) Prediction of final water content and heave

This may be calculated using the intial effective stress evaluated by one of the two approaches mentioned in subsection (ii) and the final effective stress, provided the swelling index of the sample is known. A study of the pore water composition with depth on the AH-101 samples showed that the salinity of the pore water is almost exactly the same at all depths. From 1953 to 1970, when these samples were taken, the water content of all the samples have very clearly increased and a constant pore water salinity would mean that the swelling had taken place in its own pore fluid. For this condition, the swelling index would be 0.075, as obtained from Fig. 7.16 for 5-8-CH7-pw. Using this value of swelling index, C_s, Δw_f, the final increase in water content can be calculated as follows.

$$\Delta w_f = \frac{C_s}{G_s} \log \frac{\sigma_{i*}}{\sigma_{f*}}$$

where G_s = specific gravity of soil particles

and $\sigma_{i,f}^*$ = initial and final true effective stress

and the change in thickness of any soil layer of initial thickness z and initial void ratio e_o would be,

$$\Delta z_f = z . \frac{C_s}{1 + e_o} \log \frac{\sigma_i^*}{\sigma_f^*}$$

9.4 Prediction of water content or heave at any time

From the Fig. 9.4, it can be seen that the heave due to the spillway cutting has stopped at a depth of about 7 ft. below the present ground level, but water content changes have been measured up to a depth of about 12 ft. Therefore, a depth of 20 ft. much larger than the above two has been taken as the distance to the impervious boundary from the top pervious boundary. That is, the drainage path has been taken as 20 ft. (=H). This value of H may be used to calculate the degree of swelling at any depth at any time from the value of time factor, T calculated as

$$T = \frac{c_{vs} \times t}{H^2}$$

where c_{vs} = coefficient of vertical swelling

The change in water content at any depth would be

$$\Delta w_{t-d} = \Delta w_{f-d} \times \frac{U_d}{100}$$

where U = degree of swelling

and suffix d refers to any particular depth.

The calculation of Δz at any depth at any time t is similar to the above. The total heave at any depth would then be the sum of all Δz for that depth and all depths below at any particular time.

9.5 Comparison of predicted and measured water content and heave

Following the methods described above, using coefficients of swelling of 3.0 ft^2/year (the average of the 5 coefficients of swelling estimated from the development of equilibrium pressure test for 5-8-CH7-pw, the final water content and heave and the water content and heave after 17 years (at 1970) have been calculated and given in Tables IX.1 and IX.2. (The initial effective stress in Table IX.1 has been calculated from the water content swelling pressure relationship for the Morden shale and in Table IX.2, the same has been calculated from the swelling pressure at 9 ft. as 3.00 tsf, that of 5-8-CH7-pw and calculating the effective stress at other points assuming a hydrostatic pore pressure variation.)

The water contents are much underestimated in Table IX.1. Table IX.2 gives reasonable agreement between the estimated and actual water contents at depth 7 ft. and below. At shallower depths, the measured water contents were always greater than the predicted ones. A higher water content at shallow depths could result from a possible dilution of the pore water (which has been ruled out by the pore water study which shows no influences) and/or by weathering. Scanning electron microscope examinations of the soil from shallow and deep depths, revealed the presence of a large quantity of tiny gypsum crystals within the top layers. This indicates possible chemical weathering of the soil at shallow depths (Morgenstern, 1970) which explains the higher water contents.

The comparison at deeper layers also indicates that the predicted water content is smaller than the observed. This may be due to a higher swelling index of the soil in the field where it swells in 3 directions compared to the one-dimensional swelling test.

The comparison of the predicted heave with the observed ones is very poor. The reliability of the field values itself is not very good, because at many depths, where an increase in water content has been measured no heave had been observed. This lack of correspondence between the two measurements does not permit a fair assessment of the accuracy of the prediction of heave in the field.

9.6 Calculation of change of water content as due to primary plus secondary swelling

It is of interest to see whether the water content as of 1970 could be predicted more accurately by considering the swelling as due to a primary plus a secondary part as seen in conventional swelling tests. From tests on sample 5-9-CH7-pw, the swell index from the primary part of swelling alone was found to be (approximately) 0.011. Using this swelling index and using a coefficient of primary swelling, c_{vs} of 4.4 ft^2/year, the water content as of 1970 can be estimated as explained in the earlier sections. This is given in Table IX.3a. For the calculation of secondary swelling the total period of 17 years (1953 to 1970) has been divided into time intervals of 0-4 1/2, 4 1/2 - 7 1/2, 7 1/2 - 10 1/2, 10 1/2 - 13 1/2 and 13 1/2 - 17 years, with the average time for these intervals being 3, 6, 9, 12 and 15 years. With the progress of (primary) swelling with time the effective stress at various depths decreases and the value of $\Delta\sigma/\sigma_i$* may be calculated from pore pressure isochrones (for example, Scott, 1963) for each of the above mentioned times. The coefficient of secondary swelling for each $\frac{\Delta\sigma}{\sigma_i^*}$ may be found from Fig. 7.26 or 7.27 which has been used to calculate Δw for the corresponding time interval. The total change in water content at any depth at any time due to secondary swelling would then be the sum of Δw at that depth up to that time. Such Σ Δw's calculated for the period of 17 years at all depths have been tabulated in Table IX.3b. The change in water content due to secondary swelling is an overestimate for the top layers and an underestimate in the deep layers. The failure of this approach could be either because the laboratory swelling tests plotted in Figs. 7.26 and 7.27 were conducted in distilled water,whereas in the field the soil swelled in its own pore fluid or because the relationship between the coefficient of secondary swelling and $\Delta\sigma/\sigma_i^*$ may not be unique for the soil at all depths. A significant limitation of this approach lies in the fact that soil samples subjected to any stress release do not swell forever, but come to an equilibrium after some time, however large that may be.

Peterson (1958) found that the laboratory swelling curve of a Bearpaw shale sample (using a conventional swelling test on a sample which has first been consolidated to very large pressures)was very flat compared to the field swelling curve. The swelling index of the field curve was about 10 times that of the laboratory one. At that time, Peterson concluded that this higher swell is due to secondary effects. It is quite true that the conventional laboratory test which considers primary swelling only can never predict the field swell. Neither can the secondary swell calculations be incorporated in the field swell calculations unless the final swell at any particular stress level is known. By conducting a strain-controlled test as the one described in this thesis, one obtains the ultimate swell under any particular load. This in combination with coefficients of primary and secondary swelling can be used to predict the swell at any time in the field.

9.7 Conclusions

The prediction of change of water content in the field using the swelling index from a strain-controlled swelling test gave good results in the deep layers. The soil at shallow depths have weathered and hence have increased in water content more than is predicted from the swelling tests.

Conventional (stress-controlled) swelling tests yield the coefficients of primary and secondary swelling and the swelling index for primary swelling alone. If the test is continued over a long period for each unloading (in the order of more than 6 to 8 weeks for a sample 0.75 in. thick) the total swell (primary plus secondary) could be determined. But, the strain controlled test on these samples gives information regarding the total swell under any load in a much shorter time (say, about a week). A strain-controlled test in combination with the stress-controlled test on identical samples would yield all the information needed for the swell-time relationship of the soil in the field.

TABLE IX.1

PREDICTION OF MOISTURE CONTENT AND HEAVE

Depth ft	w_i%	σ_i* tsf	σ_f* tsf	Δw_f%	U % (1970)	w % (1970)	w % field values (1970)
1	22	0.54	0.1	0.30	92	22.28	37
3	21.5	0.75	0.165	0.27	77	21.71	28.7
5	21.	1.07	0.231	0.27	63	21.17	24.7
7	20.5	1.50	0.297	0.29	50	20.65	23.25
9	19.75	2.88	0.363	0.37	38	19.89	22.5
11	19.50	3.00	0.425	0.35	29	19.60	21

Depth ft	e_o	ΔH(1970) ft	Heave (1970) ft	Field Heave (1970) ft
1	0.591	0.009	0.036	0.57
3	0.578	0.007	0.027	0.20
5	0.564	0.006	0.020	0.06
7	0.551	0.005	0.014	0.04
9	0.531	0.005	0.009	0.02
11	0.524	0.004	0.004	0.02

TABLE IX.2

PREDICTION OF MOISTURE CONTENT AND HEAVE

Depth ft	σ_i* tsf	σ_f* tsf	Δw_f%	w_i% (1953)	U % (17 yrs)	w % (1970)	w % field value (1970)
1	2.734	0.1	4.02	22	92	25.7	37.0
3	2.802	0.165	4.44	21.5	77	24.15	28.7
5	2.868	0.231	3.06	21	63	22.93	24.7
7	2.934	0.297	2.78	20.5	50	21.89	23.25
9	3.0	0.363	2.56	19.75	38	20.97	22.5
11	3.066	0.425	2.40	19.50	29	20.70	21

Depth ft	e_o	ΔH(1970) ft	Heave (1970) ft	Field Heave ft
1	0.591	0.125	0.414	0.57
3	0.578	0.090	0.289	0.20
5	0.564	0.066	0.199	0.06
7	0.551	0.048	0.133	0.04
9	0.531	0.043	0.085	0.02
11	0.524	0.042	0.042	0.02

TABLE IX.3a

PREDICTION OF MOISTURE CONTENT - PRIMARY SWELLING

Depth ft	σ_i* tsf	σ_f* tsf	Δw_f%	w_i%	U % (1970)	w % (1970)	w % Field Value (1970)- w 1970 predicted
1	2.734	0.1	0.57	22	94	22.54	14.46
3	2.802	0.165	0.63	21.5	82	22.02	6.85
5	2.868	0.231	0.44	21	70	21.31	3.39
7	2.934	0.297	0.40	20.5	59	20.74	2.51
9	3.00	0.363	0.37	19.75	49	19.93	2.57
11	3.066	0.425	0.34	19.5	40	19.60	1.40

TABLE IX.3b

PREDICTION OF MOISTURE CONTENT - SECONDARY SWELLING

Depth ft	σ_i^* tsf	σ_f^* tsf	Values of U and $\Delta\sigma/\sigma_i^*$ for t=3 yrs U	$\frac{\Delta\sigma}{\sigma_i^*}$	6 yrs U	$\frac{\Delta\sigma}{\sigma_i^*}$	9 yrs U	$\frac{\Delta\sigma}{\sigma_i^*}$	12 yrs U	$\frac{\Delta\sigma}{\sigma_i^*}$	15 yrs U	$\frac{\Delta\sigma}{\sigma_i^*}$
1	2.734	0.10	84	0.81	89	0.86	90	0.87	92	0.89	93	0.90
3	2.802	0.165	56	0.53	68	0.64	74	0.70	78	0.73	80	0.75
5	2.868	0.231	34	0.31	50	0.46	58	0.53	63	0.58	67	0.62
7	2.934	0.297	17	0.15	34	0.31	43	0.39	49	0.44	55	0.49
9	3.00	0.363	8	0.07	22	0.19	32	0.28	39	0.34	44	0.39
11	3.066	0.425	3	0.03	13	0.11	22	0.19	29	0.25	36	0.31

Values $10^3 \times C_{ss}$ in $\min^{-1/2}$ and Δw for

Depth ft	t = 3 yrs (0-4½) yrs C_{ss}	Δw	t = 6 yrs (4½-7½) yrs C_{ss}	Δw	t = 9 yrs (7½-10½) yrs C_{ss}	Δw	t = 12 yrs (10½-13½) yrs C_{ss}	Δw	t = 15 yrs (13½-17) yrs C_{ss}	Δw
1	28	25.48	35	9.19	38	8.04	39	7.08	40	7.38
3	7.6	6.9	12	3.15	16	3.39	19.5	3.54	22	4.06
5	1.0	0.91	3.7	0.97	7.6	1.61	8.5	1.54	11	2.03
7	0.5	0.46	1.0	0.26	2.1	0.44	3.1	0.56	4.9	0.90
9	0.2	0.18	0.7	0.18	1.0	0.21	1.3	0.24	2.1	0.39
11	0.1	0.09	0.2	0.05	0.7	0.15	0.8	0.15	1.0	0.18

Depth	Δw due to secondary swelling	Field w - Predicted w (1970) primary (From VIII.3a)
1	57.17	14.46
3	21.04	6.85
5	7.06	3.39
7	2.62	2.51
9	1.20	2.57
11	0.62	1.40

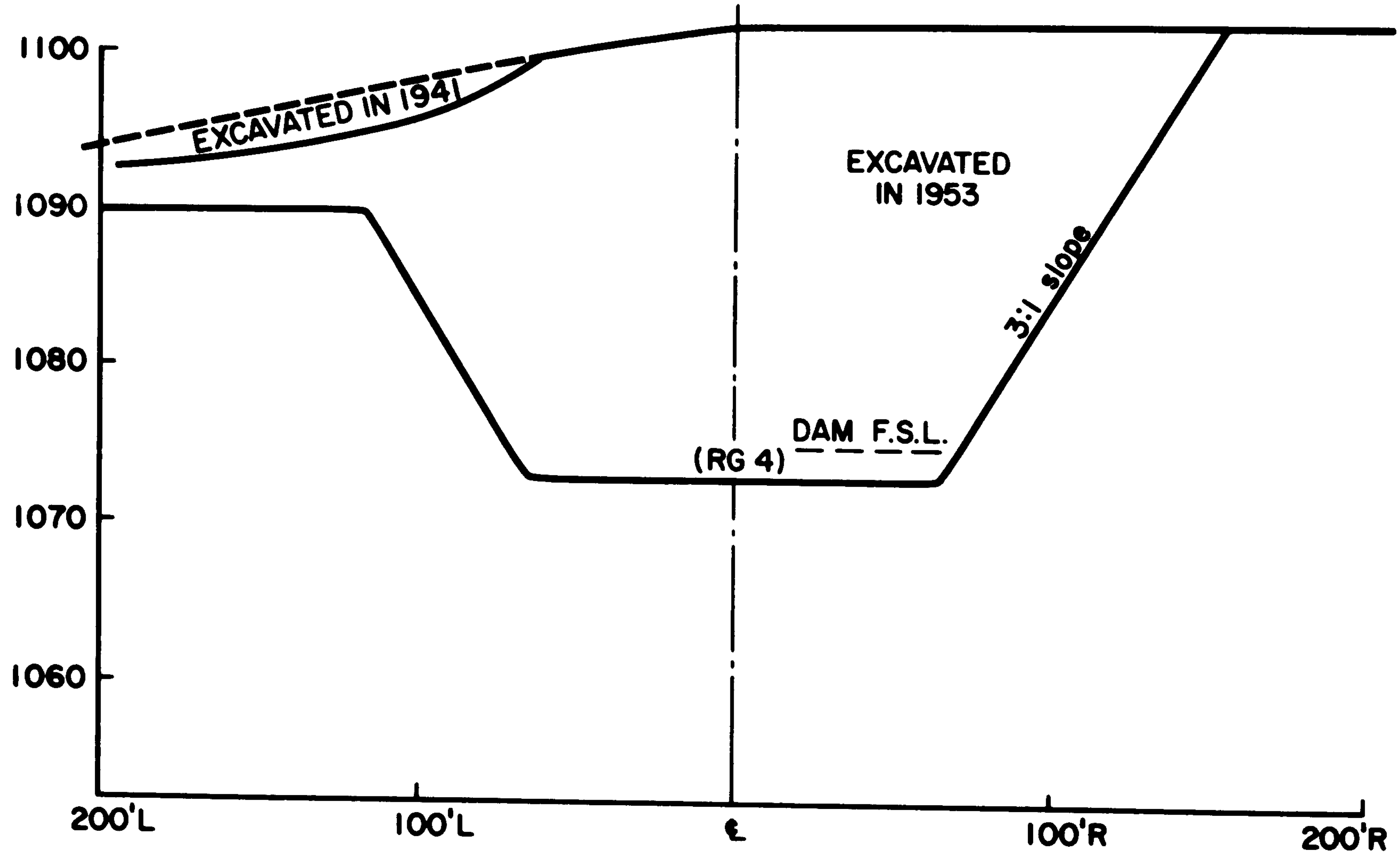


FIG. 9-1 TYPICAL SPILLWAY SECTION

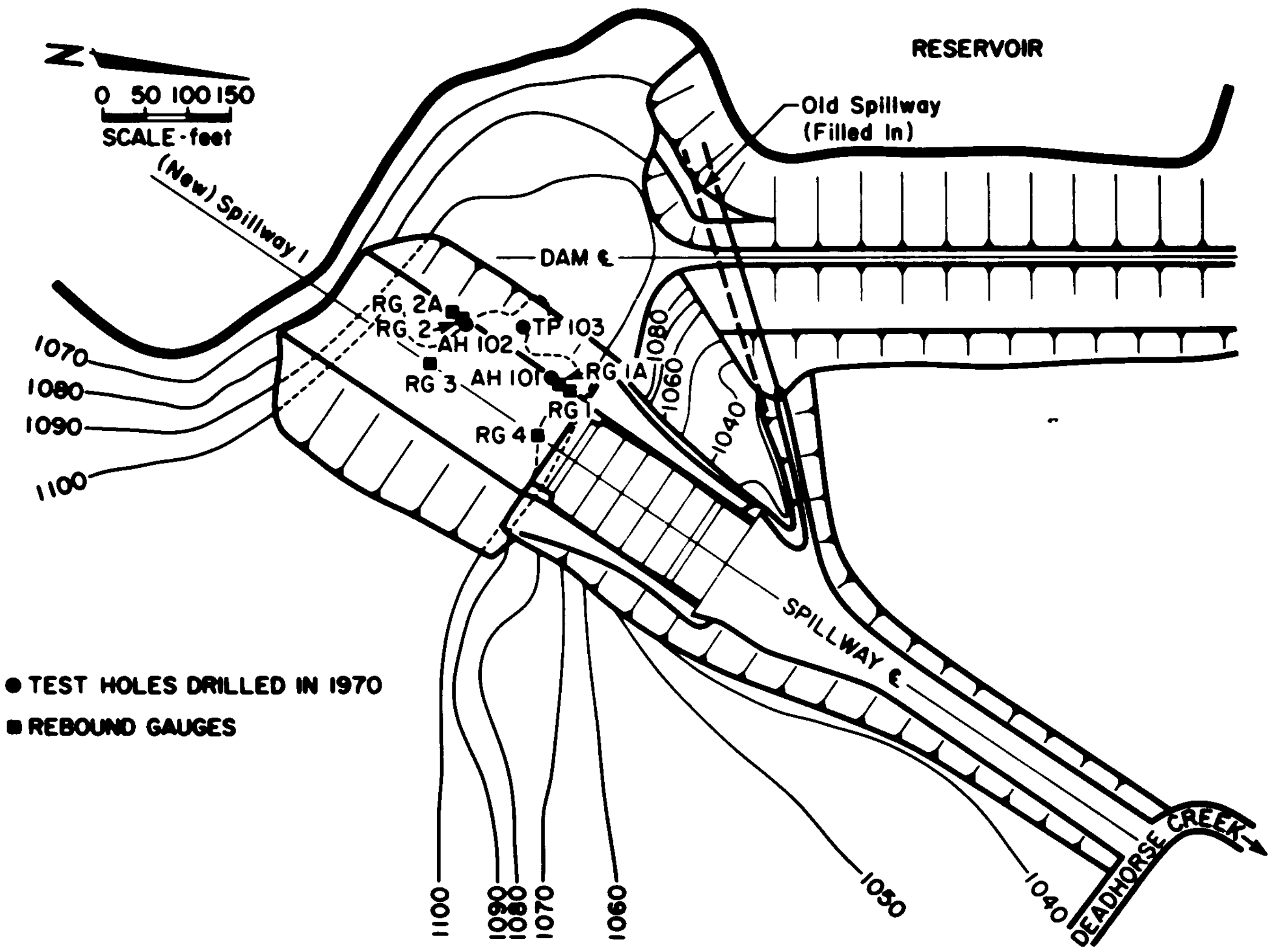


FIG. 9-2 PLAN SHOWING REBOUND GAUGES AND TEST HOLES

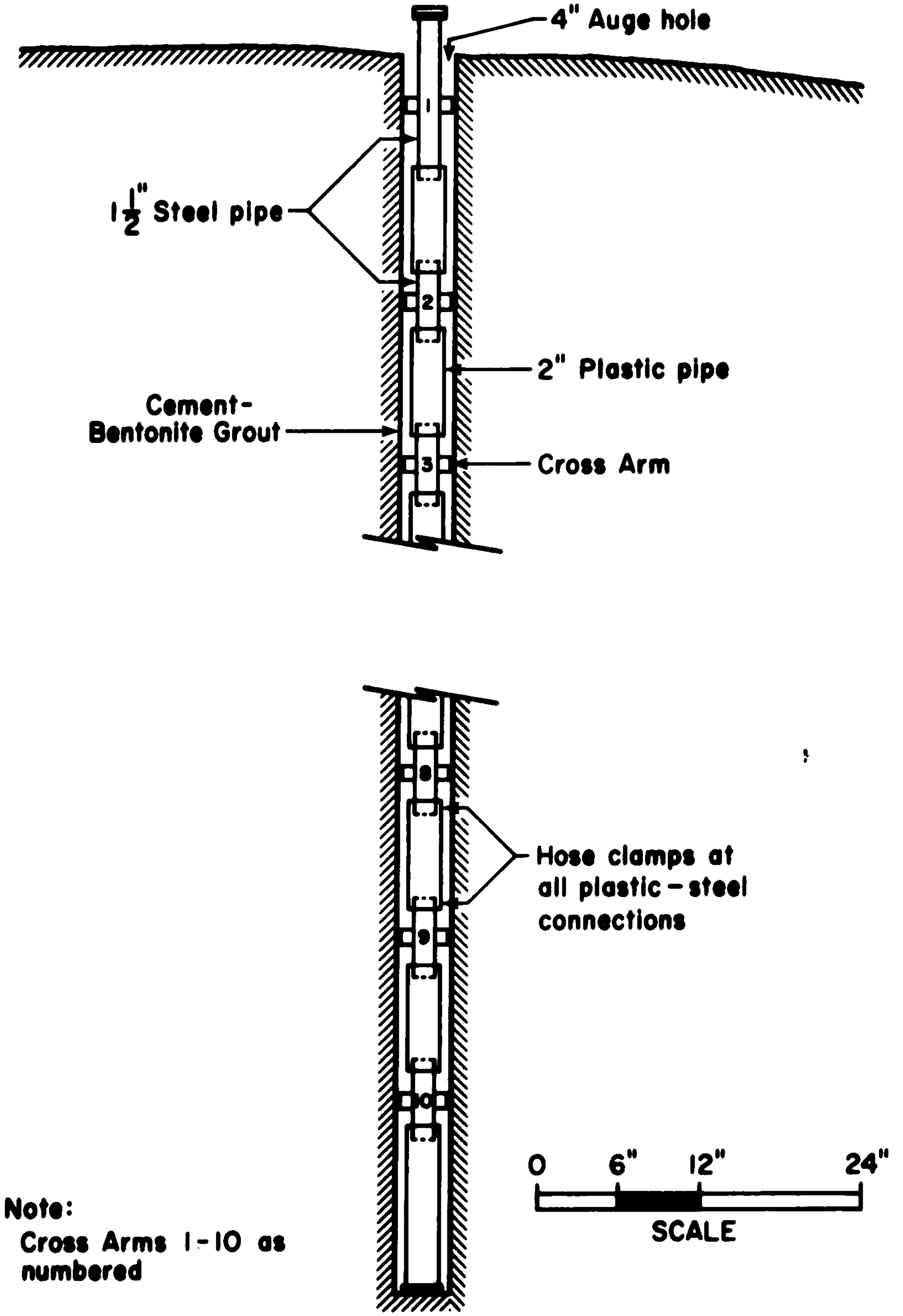


FIG. 9-3 REBOUND GAUGES

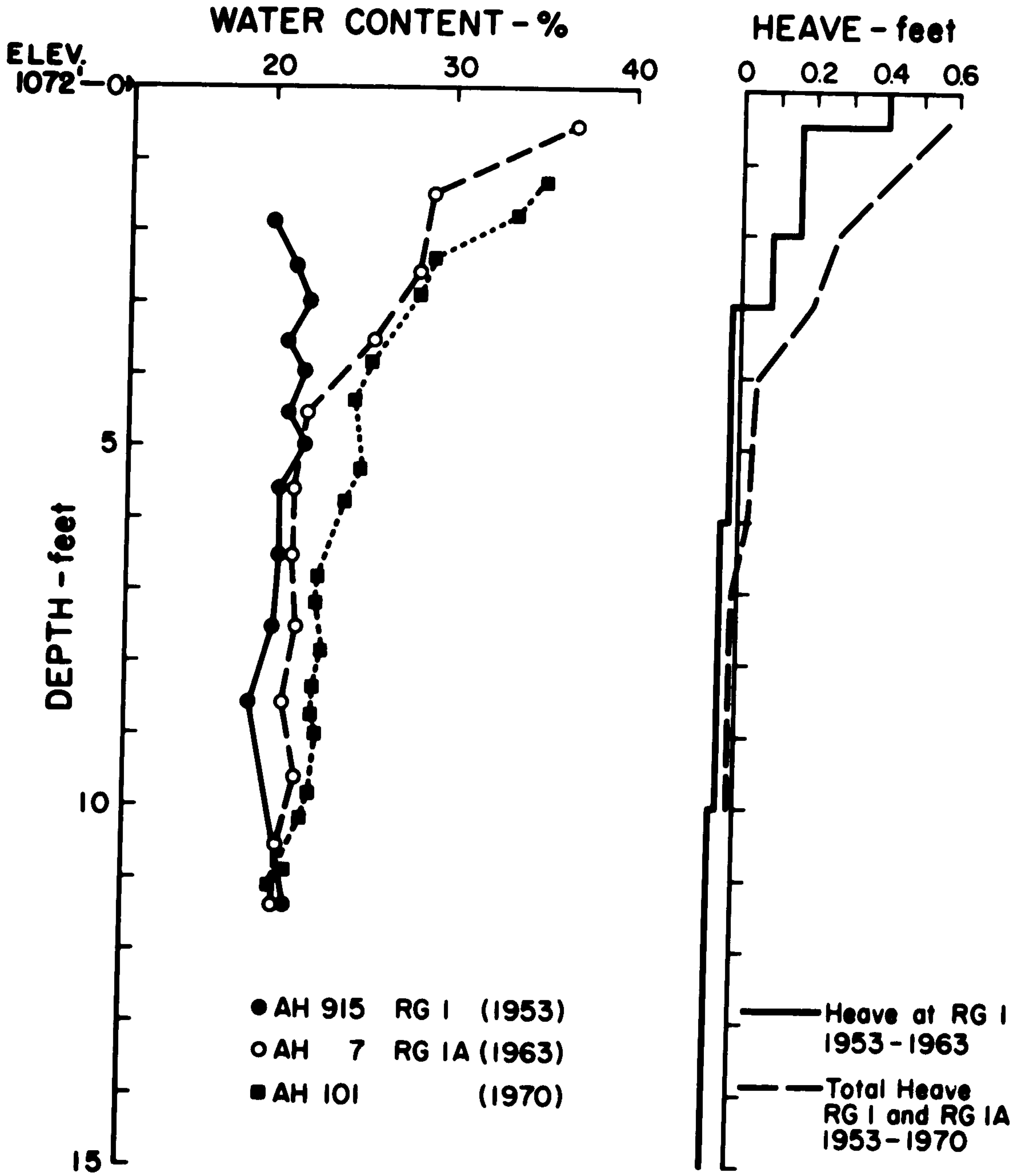


FIG. 9-4 WATER CONTENT AND HEAVE PROFILES: AT RG 1 AND RG 1A

CHAPTER X

CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

10.1 Conclusions

Based on the results of the present work and a discussion of the previous studies the following conclusions may be drawn.

1. Of the different methods available for determining the swelling pressure of soil samples, the sample remains at constant volume (i.e., at the same point in the e-log σ' plot, e-log σ^* plot to be precise) only during the constant volume test and when the suction of the pore water is found directly. Because of the difference in the magnitudes of the compression and swelling indices the loaded swell test and the free swell test may yield erroneously low swelling pressures. A free swell test on a marine shale in distilled water may give higher swelling pressures. All the four methods mentioned above would yield the same swelling pressure, only if, within the range of effective stress induced on the sample in the latter two methods, the compression and swelling indices are more or less equal in magnitude.

2. Among the various methods available for pore water extraction, the high pressure mechanical squeezer is the most suitable for compaction shales. The variation of the salt concentration in the pore water with the squeezing pressure should be investigated and the safe squeezing pressure determined.
3. Because of the non-linearity of the salt concentration - inverse of water content relationship, the dilution method of pore water extraction should not be used to study the pore water salt concentration of clayey sediments.
4. The swelling pressure of a soil even if it is fully saturated does not develop instantaneously. This delay mechanism may either be due to the compliance of the measuring apparatus and/or due to a compressible pore fluid. When the development of swelling pressure is plotted against time, it yields a curve similar to the consolidation curve. The coefficient of swelling pressure calculated from such a curve is comparable to the coefficient of swelling obtained from conventional swelling tests, with small load decrement ratios.
5. The effective stress in a fully saturated soil is defined by the equation

$$\sigma = \sigma^* + u + R-A$$

6. The volume change behaviour of soils associated with particle deformation and particle reorientation without a change of fabric are governed by the true effective stress, σ^* . The swelling behaviour of many fully saturated soils, particularly montmorillonitic compaction shales which are influenced primarily by the above two factors, is therefore controlled by σ^* alone.
7. A unique relationship will be obtained, irrespective of the physico-chemical environment, if the void ratio (or water content or percent swell) is plotted against σ^*, provided the associated change in fabric is small which was the case for the materials studied.
8. The evaluation of σ^* requires an estimate of the (R-A) component. This force may be approximated by the double-layer repulsive force for the swelling soils with sodium as the predominant cation, if the effective surface area of the soil can be estimated with reasonable accuracy. Since a direct estimation of this surface area is difficult, an indirect method has been adopted in this thesis which has yielded satisfactory results.
9. The surface of the clay soil sample does not act as a semi-permeable membrane causing an additional component of swelling pressure for marine shales in a

9. (Cont'd) distilled water environment. The surface of the sample behaves rather like an impermeable membrane in some shales or like a freely permeable membrane in others. In the former, due to the presence of an unknown barrier to diffusion, the pore water concentration is not affected by the ambient water (at least within the time scales discussed in the test programme). In the latter, the pore water concentration of the soil in a distilled water environment is decreased due to a free mixing of the pore water with the ambient water, both finally reaching the same concentration. Thus, in both of these samples, any difference in swelling pressure measured between two samples initially in distilled water and the natural water is not due to any concentration difference between the pore water and ambient water. The difference between the two cases arises because of the difference in the concentration of the bound water and the pore water.
10. The salt concentration of the pore water of those samples which exhibited no change in swelling pressure in distilled water and in its natural pore water is affected by a rapid mixing with the ambient water, once a small swelling of the soil sample is permitted.
11. The coefficient of swelling predicted from the rate of development of swelling pressure curve is an

11. (Cont'd) underestimate compared to the results from conventional tests on heterogeneous samples. (Use of high air entry porous stones is recommended for such samples.)

12. The coefficient of swelling from stress controlled (conventional) swelling tests is larger than the same predicted from the strain-controlled test (from observations of the rate of development of equilibrium pressure).

13. The strain controlled tests yield the ultimate swell under any load in a much shorter time compared to the stress-controlled tests.

14. The coefficient of secondary swelling in distilled water and pore water environments do not vary significantly.

15. The coefficients of secondary swelling gave a straight line relationship when plotted against the load decrement ratio $\Delta\sigma/\sigma_i$* for the deep Morden shales samples.

16. The increase in water content or heave in the field may be estimated reasonably well from the one dimensional swelling test only if, in the field, the swelling is more or less one dimensional. The magnitude of swelling in the field cannot be predicted as primary plus secondary unless the final

16. (Cont'd) swell at that load is known.

17. Calculation of increases in water content using the swelling index from the strain controlled swelling test compared favourably with the actual increases in water contents at large depths due to a spillway excavation. At shallow depths due to weathering the water content has increased beyond the magnitude predicted from swelling tests.

10.2 Suggestions for future research

1. The uniqueness of σ^* controlling other soil properties is worthy of further investigation. The residual shear strength τ_r divided by the true effective stress σ^* (calculated as $\sigma' - (R-A)$, where $(R-A)$ is equal to the double-layer repulsive force) has yielded a unique tan ϕ_r value for Kenney's (1967) test results. (Chattopadhyay, 1972).
2. In Chapter IX, it was found that the increase in water content due to weathering in the shallow depths could not be predicted from the swelling tests alone. It would be of great interest to study the influence of weathering quantitatively so as to enable engineers to predict the magnitude and rate of increase in water content in the field. Weathering in the field may occur due to many causes. The slaking tests

2. (Cont'd) conducted by Eigenbrod (1972) give information on the influence of physical weathering induced by alternate drying and wetting processes. More quantitative studies on similar lines may define more clearly the factors that influence swelling in-situ.

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APPENDIX A

THE DOUBLE-LAYER REPULSIVE FORCE

This appendix has been divided into four subdivisions. The equations for calculating osmotic repulsive force due to the double-layer interaction between two adjacent clay particles have been given first. (Bolt, 1956; Bailey, 1965). The derivation of these equations is based on several assumptions which have been discussed in detail by Low (1959). An examination of these assumptions and Low's criticisms has been included in the next Section. A detailed description of the various terms used in these equations and a sample calculation of the double-layer repulsive force using them are presented in the next two subsections.

A-1 Double-layer repulsion equations

$$P_y = 2\, C_o\, R\, T\, (\cosh y_c - 1) \qquad \text{(A-1)}$$

$$y_c = 2 \ln \frac{\cosh \Delta + 1}{\cosh \Delta - 1} \qquad \text{if} \quad y_c < 1 \qquad \text{(A-2a)}$$

$$y_c = 2 \ln(\frac{\pi}{\Delta}) \qquad \text{if} \quad y_c > 1 \qquad \text{(A-2b)}$$

$$\Delta = \kappa\, (x_o + d) \qquad \text{(A-3)}$$

$$x_o = \frac{4\ z\ C_o}{\kappa^2 \sigma} \cdot 10^3 \qquad \text{(A-4)}$$

$$\kappa = \left[\frac{8\ \pi\ e^2\ z^2\ n}{\varepsilon\ k\ T} \right]^{1/2} \qquad \text{(A-5)}$$

In the above equations,

P_y	denotes the double-layer repulsive force in kgm/cm^2 .
C_o	denotes the ion concentration in the bulk solution (moles/cc). (This will be equal to the molarity of the bulk solution x 10^{-3}.)
R	denotes the gas constant which has a value of 85 kg-cm/mole/°K .
T	denotes the absolute temperature in °K.
z	denotes the valency of the ions .
σ	denotes the surface charge density of the soil in meq/cm^2.
ε	denotes the dielectric constant of the pore fluid in esu units.
e	denotes the electric charge on an electron having a value of 4.8 x 10^{-10} esu.
n	denotes the volume concentration of cations and anions in the bulk solution in ions/cc. (This will be equal to the molarity of the bulk solution x 10^{-3} x N_A.)

N_A denotes Avogadro's number having a value of 6.0232×10^{23} mol/gm mole

k denotes the Boltzmann's constant

and d denotes the particle half spacing in cm.

A-2 Assumptions involved in the double-layer equations

Low (1959) gives the derivation of these equations and detailed discussion of the assumptions on which these equations are based.

In arriving at the above equations from the Boltzmann's equation (which forms the basic differential equation for the double-layer repulsion equations), it is assumed that the potential energy of the ions is affected only by the electrical field of the clay particles. However, there are additional components of energy in the interparticle region, such as the polarization energy of the ions in the electric field, the energy due to ion-ion and ion-water interaction and the energy of interaction of the ions to the atoms on the surface of the particles. Bolt (1955) has made a theoretical assessment of the effect of these additional energies and he arrived at the conclusion that their net effect should be small. This, the author believes, might possibly explain why Bolt (1956), Bailey (1965) and others found good agreement between their predicted and experimental values of interparticle force (especially for Na-montmorillonites with very low molar concentrations at low pressures).

The author agrees with Low (1959) and Bailey (1965) in that the double-layer repulsive force equations do not take care of all the forces in the particle region*. But in the absence of a complete understanding of all the forces between two adjacent clay particles, the double-layer repulsive force, as given by equations in Section A-2, has been shown (in Chapter VIII) to give the best estimate of the net (R-A) force between two clay particles, especially when the adsorbed cations are predominently monovalent.

A-3 Description of the various terms in the double-layer repulsion equations.

(i) The dielectric constant, ε .

The value of the dielectric constant of the double-layer water is taken to be 80 esu units which is that of free water. (See arguments in Section A-2.)

(ii) The surface charge density, σ and x_0

The surface charge density of the soil may be estimated from its cation exchange capacity (CEC) and the specific surface (S.S.) as follows. If the CEC is in meq/100 gm. of air dry soil and S.S. in m^2/gm,

$$\sigma = \frac{CEC}{100 \times SS \times 100^2} \quad meq/cm^2 \qquad (A-6)$$

From the equations in Section A-1, one can see that σ affects directly only the value of x_0 .

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*The presence of an interaction between the first few layers of water and the clay surface has been discussed in detail by Martin (1960).

The double-layer repulsive force as given by equation A-1 is, in essence, an osmotic repulsive force, due to the difference in the sum of the concentration of all the ions in the mid-plane between the particles and the outside bulk solution. The concentration at the mid-plane is estimated from the Boltzmann equation (Taylor, 1959), which assumes an exponential variation for the concentration of the ions. This assumption leads to a surface charge density of ∞ at zero distance from the particle surface to balance a theoretical cation density of ∞ at the particle surface. This is impossible in reality. Hence a correction factor is introduced in terms of x_0, which is a distance from the imaginary surface of infinite charge density to a surface having a charge density equal to the soil under consideration. This correction thus applies only to that portion of any soil having a charge deficiency and from whose surface the double layer develops. From the known cation exchange capacity and specific surface of the clay fraction ($<2\mu$ portion) of the shales under study, x_0 was estimated to be 3.68A for the Bearpaw shale block sample and 3.60A for the Morden shale samples. (It may be seen from Equation A-4 that x_0 does not depend upon the concentration of the external solution, but only on its valency.)

(iii) <u>The valency, z, and κ</u>

Substituting the appropriate values of ε, k, T, etc., in Equation A-5, the expression for κ becomes

$$1/\kappa = \frac{1}{z_m} \left(\frac{9.2 \times z_m}{N} \right)^{1/2} \mathring{A} \qquad \text{(A-7)}$$

When the concentration of the individual salts in the bulk solution is known, the net valency of the double-layer water at the mid-plane z_m, may be calculated as follows. As a first approximation, calculate z_m as

$$z_m = z_c = \frac{\text{weight of monovalent cations} + 2 \times \text{weight of divalent cations}}{\text{weight of (monovalent + divalent) cations}}$$

where z_c = valency of cations.

In the above expression, the assumption made is that the concentration of anions is nil and that the proportion of the monovalent and divalent cations in the mid plane is equal to that of the bulk solution. Using z_c as defined above, calculate y_c. By definition (for eg. Low, 1959), e^{y_c} and e^{-y_c} represent the concentration of cations and anions in the mid plane divided by the concentration in the bulk solution. The valency of the mid plane water can now be estimated as

$$z_m = \frac{z_c \times e^{y_c} + z_a \times e^{-y_c}}{e^{y_c} + e^{-y_c}}$$

where z_m = net valency of the mid plane water

and z_a = the valency of anions.

Using this new value of z_m, the value of κ can be calculated.

(iv) The interparticle half distance, d*

The next step is the evaluation of d . From the known water content of the soil and its specific surface, the particle half spacing may be evaluated using the following equation, (assuming uniformly oriented infinite particles).

$$d \text{ in } \mathring{A} = \frac{\text{water content } (\%) \times 100}{\text{specific surface } (m^2/gm)} \qquad \text{(A-9)}$$

The assumption of uniformly oriented infinite particles is not met within most cases and for that reason the value of d calculated as above does not indicate the actual spacing between the particles. A detailed study has been made by Bailey (1965) to investigate the effects of finite particles and non-parallel orientation.

Adoption of this realistic model is difficult at the present state of knowledge of the various factors involved, such as the influence of the dead volume, osmotic repulsive force between non-parallel particles and any attractive force between particles which might become important at very close spacings. Therefore, the author is forced to make the assumption of uniformly spaced infinite particles to calculate an equivalent d to be substituted in the double-layer equations.

As seen from equation A-9, the evaluation of this d necessitates a knowledge of the specific surface of the soil. It has

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* See also Chapter VIII.

been argued in Chapter VIII that the specific surface to be used in the above equation is the effective surface area and not the total surface area of a soil. The effective surface area of a pure Na-montmorillonite system would be the total surface area determined by the EGME technique (Appendix C). In the case Ca montmorillonites, it has been reported by Norrish (1956) that the soil is present as clusters and that the intracluster spacing does not increase during swelling. In such soils the external surface area of the clusters is to be used in equation A-9 for calculating d. This is possible only if the number of clay particles per cluster is known (Blackmore and Miller, 1961). The shales tested in this programme were also seen to have formed clusters. A direct evaluation of the external surface area of these clusters being difficult, an alternate indirect approach has been suggested in Chapter VIII, which has worked satisfactorily.

(v) Evaluation of P_y

Once κ, x_0 and d have been evaluated, the calculation of P_y is made by direct substitution of these values in Equations A-3, A-2 and A-1.

A-4 Sample calculations

Soil Sample: Bearpaw shale 1-4-CH5-dw

Effective specific surface = 116 m^2/gm

Water content at 0% swell = 30.5%

d at 0% swell = $\frac{30.5 \times 100}{116}$ = 26.3 Å

Normality of bulk solution = 0.02N

Composition of bulk soln. = 8.25 gms Na_2SO_4 + 2.75 gms of $MgSO_4$ in 1000 cc of solution

In 1000 cc solution, weight of Na = 2.67 gms

weight of Mg = 0.55 gms

$$z_c = \frac{2.67 \times 1 + 0.55 \times 2}{2.67 + 0.55} = 1.77$$

CEC of the <2μ portion = 65.2 meq/100 gms

Specific surface of <2μ portion = 546 m^2/gm

$$x_o = 3.68 \text{ Å}$$

$$1/\kappa = \frac{1}{1.17}\left(\frac{9.2 \times 1.17}{0.02}\right)^{1/2}$$

$$= 19.83$$

$$y_c = 1.463$$

$$e^{y_c} = 4.32$$

$$e^{-y_c} = 0.23$$

$$z_m = \frac{4.32 \times 1.17 + 0.23 \times 2}{4.32 + 0.23}$$

$$= 1.212$$

$$1/\kappa = 19.48$$

$$P_y = 0.70 \text{ tsf}$$

APPENDIX B

THE ATOMIC ABSORPTION SPECTROPHOTOMETER

Principles

Fig. B.1 shows schematically the atomic absorption spectrophotometer (for example, Walsh, 1960). The sample solution, containing the element to be determined (1) is aspirated through a short tube into the specially designed mixing chamber (2). The finely atomised droplets are mixed with air and acetylene gas and carried to the burner. Large drops settle down and drain out of the instrument. The mixture of air, acetylene and the sample solution is burnt in the 10 cm long slot type burner (3). The heat of the flame causes most of the compounds to dissociate, liberating the metal atoms to be determined. The flame serves two purposes; it delivers the sample to the light path of the spectrophotometer and produces the necessary dissociations.

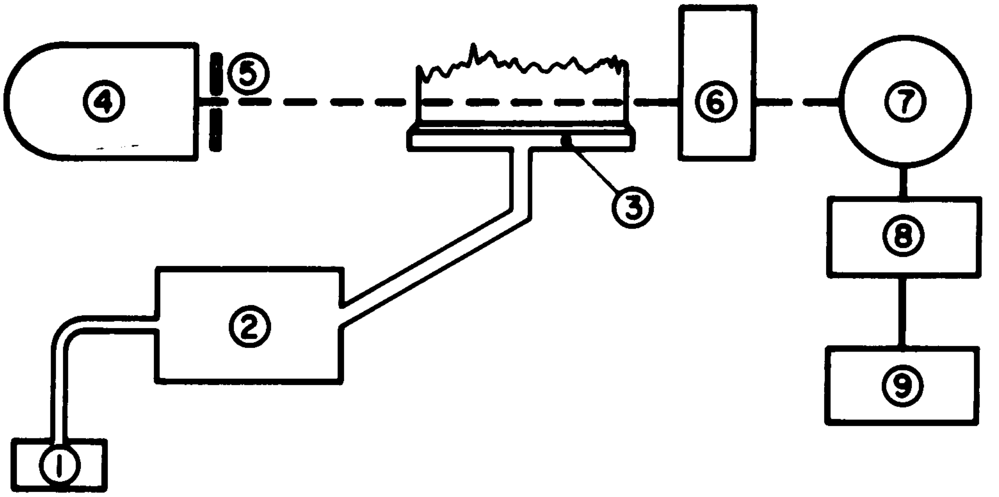
The source lamp (4) usually used in atomic absorption is a tube containing a hollow cathode made of the metal to be determined. The light from the source (4) is collimated (5) and passes through the flame immediately above the burner. The spectrum of the metal being determined contains one or more resonant wave lengths and these wave lengths will be absorbed by any atoms of this metal.

A monochromator (6) screens out all light except the selected wave length to which it has been set. The light from the monochromator is detected by a photomultipler (7), amplified (8) and read on a meter (9) calibrated directly in absorbance.

If the instrument is zeroed with no sample in the flame, the drop in intensity when the sample is introduced is proportional to the concentration of the particular metal in question.

Method of Operation

The unit in the University of Alberta has a plotter instead of a meter (9). First prepare standard solutions of K, Na, Mg and Ca and run these standards in the spectrophotometer and obtain calibration curves for each ion. The solution to be analysed may be then fed into the machine (properly diluted to fall in the range of concentration of the unit and pretreated properly with 15,000 ppm Sr solution for Mg and Ca studies to avoid interference of other atoms) and the concentration may be determined directly from the appropriate calibration curve.



① SAMPLE SOLUTION
② MIXING CHAMBER
③ BURNER
④ SOURCE LAMP
⑤ COLLIMATOR
⑥ MONOCHROMATOR
⑦ PHOTOMULTIPLIER
⑧ AMPLIFIER
⑨ METER OR RECORDER

FIG. B-1 SCHEMATIC DIAGRAM OF ATOMIC ABSORPTION SPECTROPHOTOMETER

APPENDIX C

CATION EXCHANGE CAPACITY AND SPECIFIC SURFACE AREA DETERMINATIONS

The procedures followed for the determination of the cation exchange capacity and the specific surface area of soils are given below (Soil Science Manual #421; Jackson, 1960).

C-1 The Cation Exchange Capacity (CEC)

The organic content of the soil sample is removed by H_2O_2 treatment. This sample is washed through a No. 200 sieve and the silt + clay fraction of the total soil that passed through the sieve openings is collected and air dried. Exactly 10 gm of this air dry soil sample is transferred into 100 ml plastic centrifuge tubes. About 1 teaspoonful of ground filter paper and 40 ml. of 1N NH_4Ac are added to the soil. The plastic tube is stoppered and shaken for several minutes and then allowed to stand overnight. The tube is shaken for another 15 minutes. The contents of the tube are then transferred to a 5.5 cm Buchner funnel in which a moist Whatman No. 42 filter paper has been seated by gentle suction. The tube and the stopper are rinsed into the funnel with NH_4Ac. The soil is washed with 4 successive 40 ml portions of NH_4Ac, with aspiration between each washing. The leachate is transferred to a 250 ml volumetric flask and made to mark with NH_4Ac. After mixing, this extract is used for the analysis of Na, K, Ca and Mg, using the Atomic Absorption Spectrophotometer. The concentration of the individual exchangeable cations thus obtained is usually expressed as meq/100 gm. of air dry soil.

The funnel containing the NH_4 saturated soil is replaced on the suction flask and is washed with 3 successive 40 ml portions of ethonol, with aspiration between each washing. The leachate is descarded. The suction flask is washed with distilled water and the funnel is replaced on it. The soil is now leached with 4 successive 50 ml portions of 1N NaCl. The leachate is transferred to a 250 ml volumetric flask and made to volume with distilled water.

Exactly 100 ml of this NaCl extract is pipetted into a 800 ml Kjeldahl flash. A few grains of zn, one small piece of parafffin wax, 5 gm MgO and 200 ml of cold tap water are added to the leachate. The content is distilled into a flask containing about 30-35 ml boric acid until about 200 ml liquid has collected. About 5 drops of a mixed indicator (0.5% Bromcresol green + 0.1% Methyl Red in 95% ethanol) is added to this liquid and it is then titrated with 0.05N H_2SO_4 to greyish end point that follows the disappearance of the blue colour. The CEC of the soil sample may now be calculated as

$$\text{CEC in meq/100 gm of soil} = \frac{(\text{ml } H_2SO_4 - \text{ml } H_2SO_4 \text{ for Blank}) \times N \times 100}{\text{st. of soil sample}}$$

$$= 1/2(\text{ml } H_2SO_4 - \text{ml } H_2SO_4 \text{ for Blank})$$

(For N = 0.05, wt. of sample - 10 gms)

The Blank is made up of a few grains of Zn one small piece of paraffin wax, 5 gm MgO and 200 ml of cold tap water. This is distilled into a flask containing about 30 - 35 ml boric acid. About 5 drops of the mixed indicator is added and is then titrated with 0.05N H_2SO_4 to greyish end point.

C-2 The Specific Surface area determination

The specific surface area of the soil is determined on the <2μ portion of the soil separated by the conventional hydrometer test. The suspension containing this fine fraction is air dried to a paste and is used for the specific surface area determination. This soil sample is saturated with calcium by repeated mixing with 1N $CaCl_2$. The excess $CaCl_2$ is removed by a number of washings with distilled water, 50:50 alcohol-water mixture and pure alcohol. The soil is centrifuged. The supernatant liquid is discarded. (Care should be taken not to lose any fines.) The soil is dried in an oven at 60°F overnight. The soil is ground to pass a 60 mesh sieve. About 0.5 gm of this soil is transferred to an aluminum can. Approximately 3 ml reagent grade Ethylene Glycol Monoethyl Ether (EGME) is added to this sample. The sample is transferred to a vacuum oven in which an EGME - $CaCl_2$ solvate is placed in culture dishes to maintain a constant EGME vapour pressure at sorption surfaces. The EGME soil slurry is allowed to equilibrate for only 30 minutes before the oven is evacuated. The sample is taken out and weighed in 4 to 6 hours after the first 45 minute evacuation. The weighing is continued every 4 hours after a 45 minute evacuation until a constant weight is reached. The specific surface of the soil sample may now be calculated as follows.

$$\text{The Sp. Sfce. in } m^2/\text{gm} = \frac{\text{weight of adsorbed EGME in gms}}{0.000286 \times \text{wt. of soil in gms}}$$

C-3 Computation of clay mineral percentages using x-ray diffraction analysis (used by the Alberta Research Council)

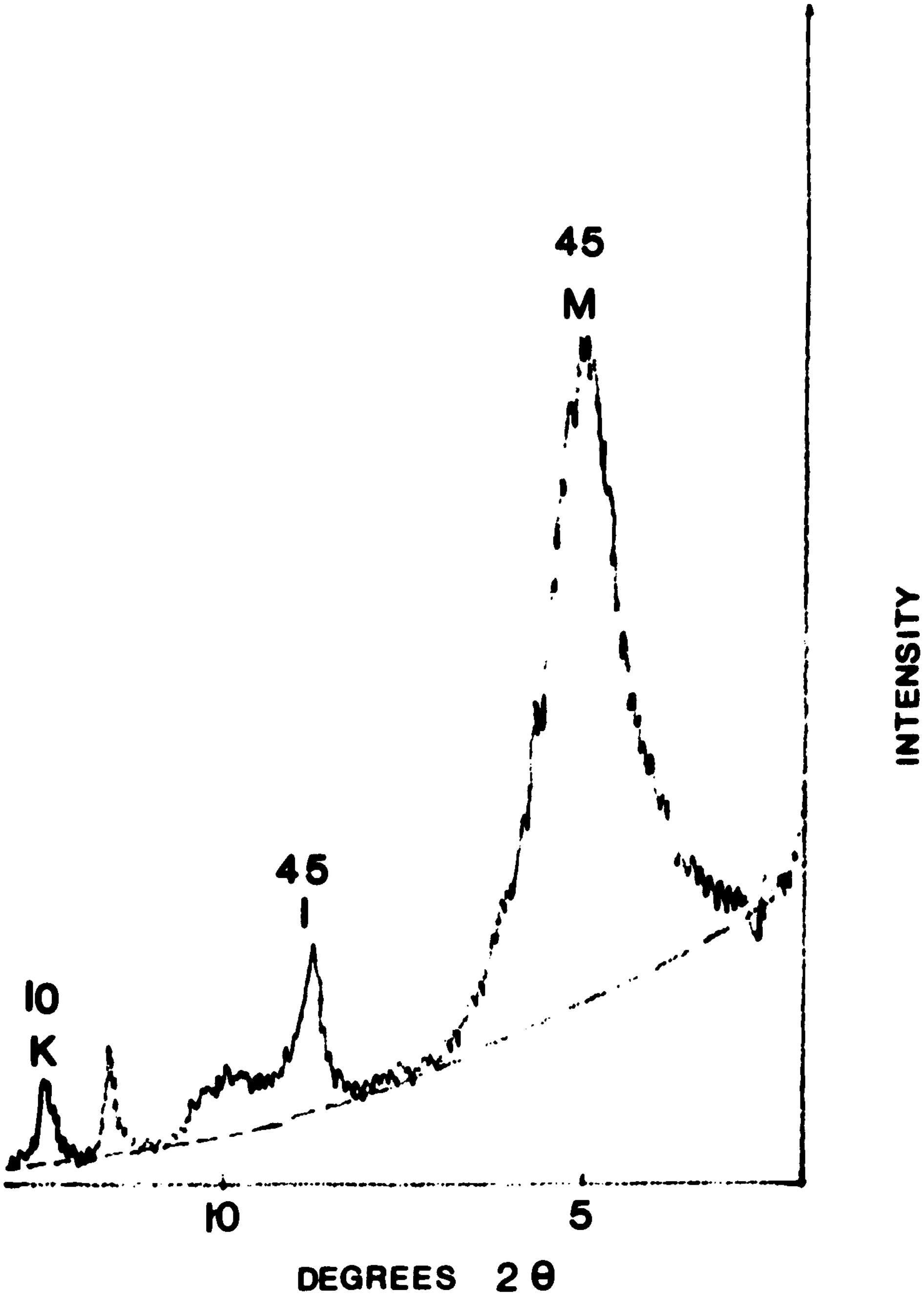
A smooth line is drawn beneath the first order peaks on the glycolated pattern and the untreated pattern. Record the heights of the 7, 10 and 17Å peaks from the glycolated pattern and of the 14Å peak from the untreated pattern. If the 7A peak height is not the same on both the glycolated and untreated patterns, multiply the 14A peak by (7A peak glycolated (7A peak untreated). Taking the form factor value of the 10A mineral as 1.0, the other form factors are calculated as follows:

17A form factor = 17Å peak height/4 x 10Å peak height

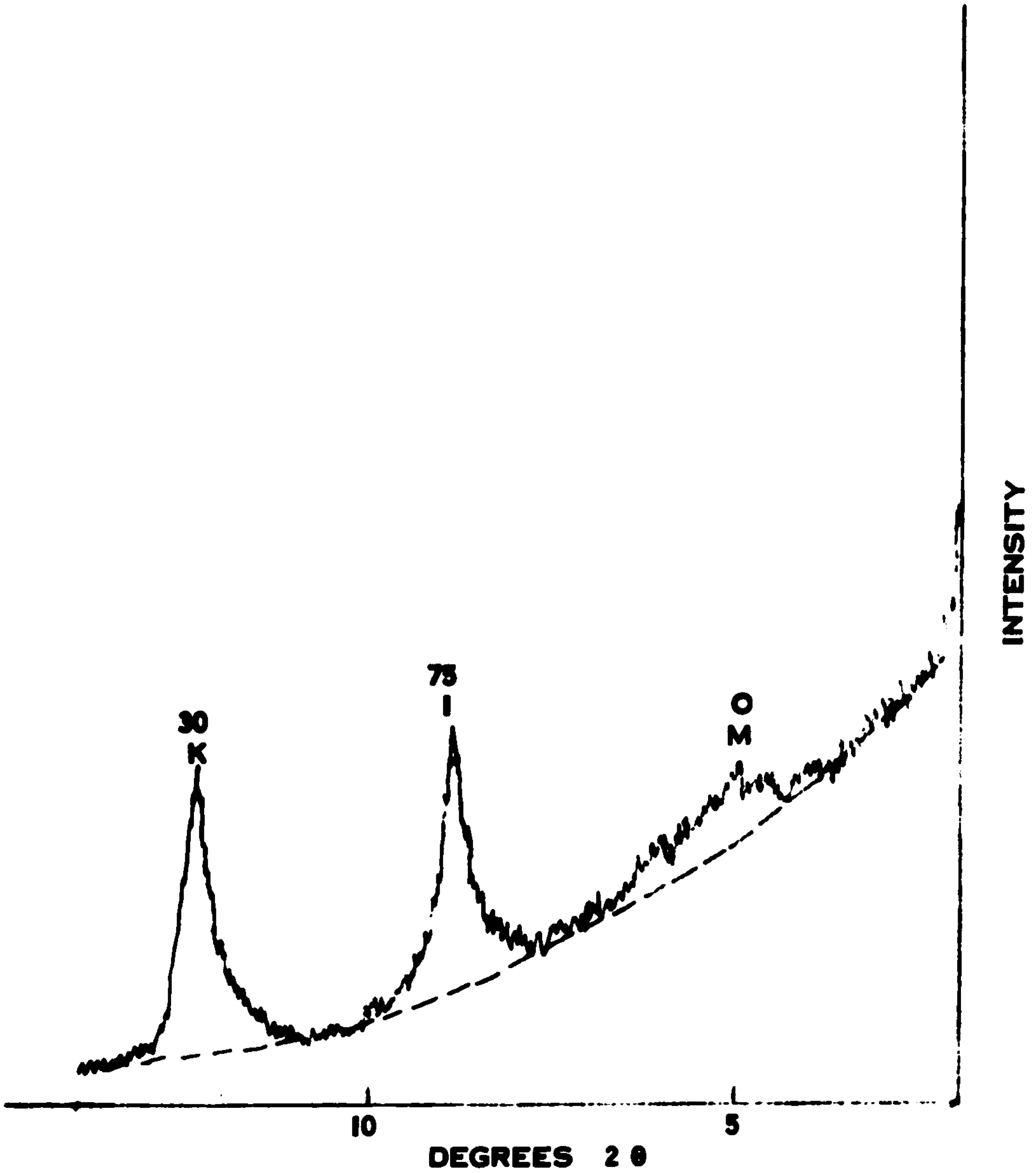
14A form factor = 14A peak height/3 x 10Å peak height

7Å form factor = 7A peak height/2.5 x 10A peak height

The proportion of each mineral may now be expressed as their form factor divided by the sum of the form factors expressed as a percentage.



X-RAY PATTERN OF BEARPAW SHALE-BLOCK
GLYCOLATED



X-RAY PATTERN OF MORDEN SHALE GLYCOLATED