A NOTE ON THE PORE PRESSURE PARAMETER B

NOTE SUR LE PARAMETRE B DE PRESSION INTERSTITIELLE

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SYNOPSIS

In triaxial testing, the ratio of change in pore pressure to change in cell pressure has been defined as the Pore Pressure Parameter B, and it can be shown that for many conditions this parameter is essentially equal to unity if the soil is fully saturated. Unfortunately, however, there are some cases where fully saturated soils can give B-values which are significantly less than unity. This is especially true for soils at high consolidation pressures, for stiff soils such as Ottawa sand and soil cement, and for some compacted clays which exhibit a stiff structure leading to a pseudo preconsolidation effect when loaded by small stress increments.

This paper describes both theoretical and experimental studies which were made to explain why low B-values are obtained in some situations even though the soil is fully saturated. The variation of B-values during the undrained shear stage of a triaxial test is also discussed.

INTRODUCTION

In many laboratory investigations it is often necessary to saturate the samples. A common laboratory method of checking whether a triaxial test specimen is fully saturated is to measure the pore pressure response to a small change in the cell pressure. This is usually accomplished by applying a back pressure to the drainage line while simultaneously keeping the cell pressure sufficiently high so that the desired effective stress is maintained. Lowe and Johnson (1960) presented the following theoretical equation for computing the amount of required pore water pressure necessary to change the degree of saturation of a sample to any desired value while maintaining the total volume and dry density of the sample constant:

$$u_s = u_i \left[\frac{(S_r - S_{r,i})(1 - H)}{1 - S_r(1 - H)}\right] \qquad (1)$$

where u_i is the initial pore pressure, usually atmospheric pressure (absolutely).

u_s is the required pore pressure (above u_i) to reach a degree of saturation S_r.

$S_{r,i}$ is the initial degree of saturation corresponding to pore water pressure u_i.

S_r is the final degree of saturation.

H is Henry's coefficient of solubility, a unitless ratio of the volume of air that can be dissolved in a unit volume of water. At room temperature it is approximately 0.02.

Substitution of $u_i = 14.7$ psi, $S_r = 1.0$, and $H = 0.02$ leads to an expression for the gage pore pressure required to reach 100% saturation from initial atmospheric pressure conditions:

$$u_{100} = 720(1 - S_{r,i}) \text{ psi} \qquad (2)$$

Knowing the initial degree of saturation and pore pressure, it is a straight forward calculation to determine the magnitude of pore pressure required to saturate the sample. However, sometimes these quantities, especially the initial degree of saturation, are not very accurately known. Also, sometimes, unless special precautions are taken, air from the triaxial chamber can diffuse through the membrane and come out of solution in the sample. Therefore, for these and other reasons, it is usually desirable to have a simple, convenient and independent method of checking that the samples are indeed saturated when they are intended to be saturated.

In triaxial testing, the most convenient method of checking whether or not the specimen is saturated is to measure the pore pressure response of an undrained sample resulting from a small change in cell pressure. The ratio of change in pore pressure, Δu, to change in cell pressure, $\Delta\sigma_3$, is called the pore pressure parameter B (Skempton, 1954). It is a function of the relative compressibilities of the soil structure and pore fluid as shown by the following theoretical equation presented by Skempton and Bishop (1954), Bishop and Henkel (1962), etc.

$$B = \frac{\Delta u}{\Delta \sigma 3} = \frac{1}{1 + \frac{nC_u}{C_d}} \qquad (3)$$

where n is the porosity of the soil sample.

C_u is compressibility of the pore fluid, $(\text{length})^2\ (\text{force})^{-1}$.

C_d is the compressibility of the soil structure, $(\text{length})^2\ (\text{force})^{-1}$.

The derivation of this equation omits the compressibility of the soil grains as being too small to be significant.

For a fully saturated soil, the compressibility of water is approximately 4.75 x 10^{-5} cm^2 per kg. Values of C_d for different soils are typically about two orders of magnitude greater than C_u. Thus, for many situations, the theoretical value of B computed from Eq. 3 for a saturated soil will be very close to unity; so close, in fact, that the deviation from unity could probably not be detected by normal experimental techniques.

However, because air is extremely compressible compared to water, the presence of only a very small quantity of air in the pore fluid will result in a very much greater pore fluid compressibility and a B-value considerably lower than unity. Thus, a simple B-value test in which the cell pressure is increased a small amount and the change in pore pressure is measured, provides a convenient routine method to check that the test specimen is fully saturated.

Unfortunately, however, there are some instances in which the soil structure is so stiff that, even if the sample is fully saturated, the measured pore pressure response will indicate B-values sufficiently lower than unity to be alarming.

One example of low B-values associated with fully saturated soils has been reported by Wissa and Ladd (1965). They studied the shear strength of cement stabilized clayey silt by means of consolidated undrained triaxial compression tests using effective consolidation pressures as high as 55 kg per sq cm. To insure saturation, the samples were subjected to back pressures on the drainage line as high as 10 kg per sq cm. At low effective consolidation pressures below about 6 kg/sq cm, the measured B-values ranged from 0.90 to 0.99 with an average value of 0.95, whereas at the higher consolidation pressures up to 55 kg per sq cm, the measured B-values ranged from 0.55 to 0.90 with an average value of 0.76

However, the samples were known to be saturated because the same B-value was obtained for a wide range of back pressures. Had there been free air in the voids of the sample, it would have steadily compressed and dissolved under an increasing back pressure. This would have caused the B-value to be higher for high back pressure than for low back pressure, but this was not observed. It would appear that the combination of the cement additive and the high consolidation pressures produced a much stiffer soil structure than normally encountered, and this resulted in the low observed B-values.

In addition to this example, the writers have also observed cases where low B-values were obtained from fully saturated samples, both with saturated sands and with compacted clays at high consolidation pressures. In order to get a better understanding of this problem, some special tests were performed to determine how the compressibility factor, C_d, varies with different situations, and to obtain a better understanding of the pore pressure parameter B as an indicator of soil saturation.

COMPRESSIBILITY AND B-VALUES FOR SAND

Two soils were tested for this study. One soil was a fine- to medium-grained uniformly graded sand from the Sacramento River. The soil was washed and screened so that all particles were between the No. 50 and No. 100 sieves (0.297 to 0.149 mm). They were composed mainly of quartz and feldspar minerals, and were subangular to angular in shape. The other soil was the well-known Ottawa standard sand, well-rounded quartz particles with sizes between the No. 20 and No. 30 sieves (0.84 to 0.59 mm).

The volume changes measured from an isotropic consolidation test on a sample of each of these sands is shown on Fig. 1. Although

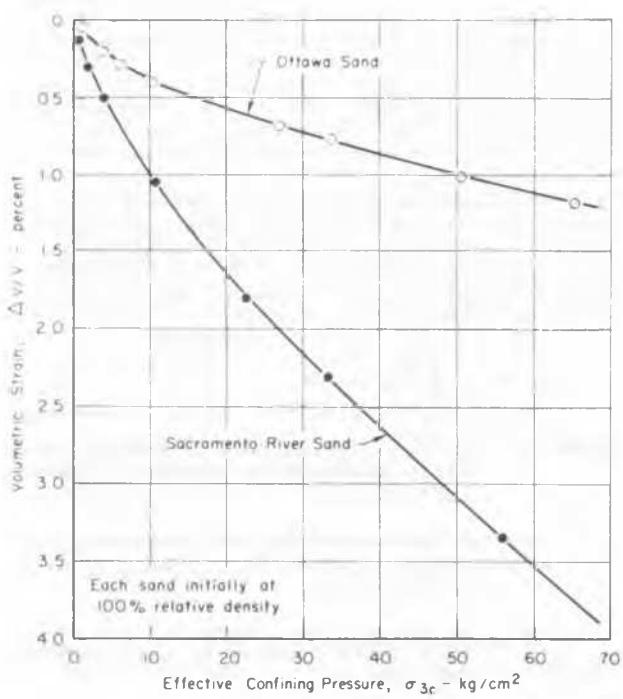

Fig. 1 Isotropic Consolidation of Two Dense Sands

both sands were compacted to an initially dense state, significant volume changes

developed as a result of the increased effective confining pressure, with the Sacramento River sand compressing more than the Ottawa sand. As would be expected, similar tests showed that loose sands compressed more than dense sands.

The slope of the consolidation curves at any point is the compressibility, C_d. Values of compressibility of these sands are shown on Fig. 2. For comparison, the compressibility for Sacramento River sand at other densities is also shown.

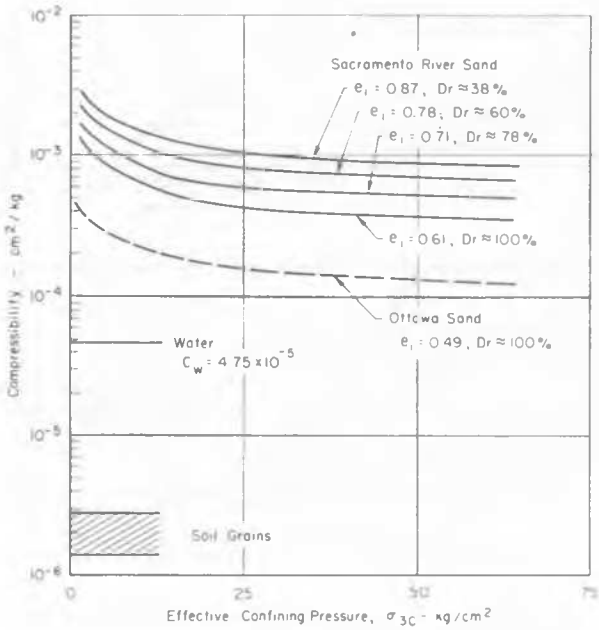

Fig. 2 Compressibility of Two Sands

Also shown for comparison is the compressibility value of water, as well as the range of values for typical soil forming minerals given by Skempton and Bishop (1954). It can readily be seen that the compressibility of these minerals is insignificantly small. However, the compressibility of water is not extremely small compared to the compressibility of dense Ottawa sand at high effective confining pressures.

To check the agreement between Eq. 3 and the measured B-values for a saturated, fairly compressible soil for computing the B-value, it is instructive to consider the following example for Sacramento River sand at about 75 percent relative density, D_r, and isotropically consolidated to an effective confining pressure of 3 kg per sq cm. At this relative density, e = 0.698, n = 0.412 and, from Fig. 2, $C_d = 14 \times 10^{-4}$ cm²/kg. Substitution of these data into Eq. 1 gives B = 0.986.

Under most routine test conditions, this value would be considered to be sufficiently close to unity to indicate complete saturation. The results of such a B-value test on a sample of Sacramento River sand consolidated to the above-mentioned density and confining pressure condition is presented on Fig. 3.

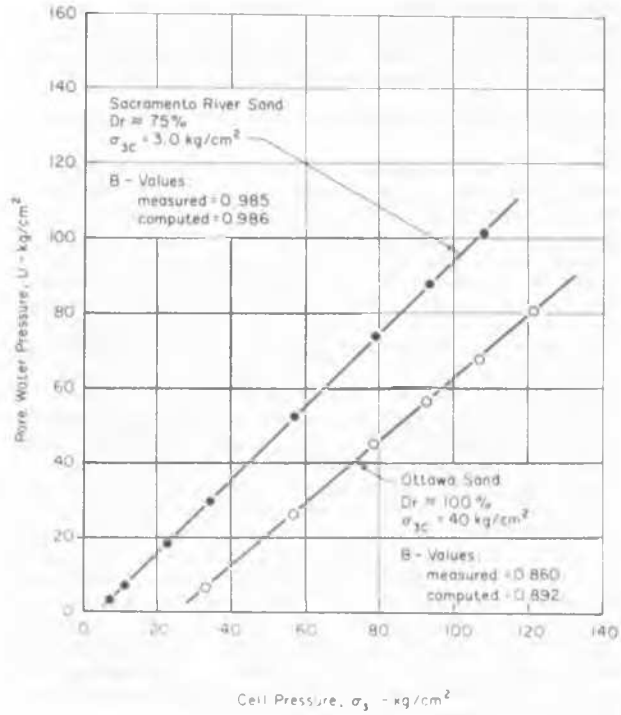

Fig. 3 Results of B-Value Tests on Two Saturated Sands

This sample was prepared by boiling the sand under a vacuum, and then transferring the sand completely under water into a water-filled forming jacket. The sample was, therefore, completely saturated when it was formed. Furthermore, a back pressure of 1 kg per sq cm was applied during the consolidation stage to further insure that saturation was maintained. The cell pressure was increased in a series of steps from an initial value of 4 to a maximum value of 107 kg/cm², and the resulting pore pressure was measured after each step. The data form a straight line plot of pore pressure vs cell pressure, which further indicates that the sample was completely saturated. The slope of this line defines the B-value equal to 0.985. This is remarkably close to the theoretical value of 0.986 computed by Eq. 3.

As indicated on Fig. 2, the compressibility of Ottawa sand is much lower than the compressibility of Sacramento River sand and, furthermore, the compressibility decreases with increasing consolidation pressure. The results of a similar theoretical and experimental investigation for dense Ottawa sand, isotropically consolidated to an effective stress of 40 kg per sq cm, are also illustrated on Fig. 3. In this case, the measured B-value was only 0.860, which was considerably less than the approximate value of unity conventionally associated with saturated soil. However, this experimental value agrees reasonably well with the computed theoretical value of 0.892. Thus, these tests illustrate that the simple rule of thumb of assuming that B

must be unity for a saturated soil is not necessarily always correct, and B-values considerably less than unity may be expected if the soil structure is particularly stiff.

COMPRESSIBILITY AND B-VALUES FOR COMPACTED CLAY

Another example in which low B-values were obtained from saturated samples was observed during an investigation of the shear strength of compacted clays at high confining pressures. The clays were compacted wet of optimum to about 95 percent of the standard Proctor density. The initial degree of saturation was about 93 percent. It was essential for the test program that the samples be fully saturated prior to testing. Calculations based on Eq. 2 indicated that a back pressure of 60 psi would be sufficient to insure full saturation. This was verified with tests on samples consolidated to low effective confining pressures. Samples tested using back pressures of 60 psi were found to have measured B-values sufficiently close to unity to indicate that they had been completely saturated. However, as the consolidation pressure increased, the measured B-values decreased and became as low as 0.85 for consolidation pressures of 1000 psi. This trend was verified on a large number of samples in spite of extensive precautions and other indications that the samples were fully saturated.

In all tests, the specimens were completely immersed in mercury to prevent any air from diffusing from the triaxial cell through the membrane and into the sample. In some cases, the back pressure was increased to as high as 240 psi, which was four times the amount required for saturation by Eq. 2, and yet, the same B-value was obtained for the higher back pressure as for the low back pressure cases. In many tests, a B-value was determined at low consolidation pressures, and at several intermediate pressures up to the final consolidation condition. In all cases, the measured B-values steadily decreased from values in excess of 0.95 at low pressures to values as low as 0.85 at high pressures.

Some special consolidation tests were performed on saturated samples to investigate the experimental and theoretical variation of B with increasing consolidation pressure. The volume changes which developed for different consolidation pressures for an isotropic stress consolidation test are shown on Fig. 4(a). Fairly large stress increments were used for this test which is typical of the loading procedure in most consolidation testing.

The slope of the consolidation curve at any point is the structural compressibility, C_d, for that particular stress level. A plot showing values of C_d from this test is shown on Fig. 5. Using these values of C_d from the large stress increment test, values of B were computed for different consolidation stress levels and are shown on Fig. 6(a). Also shown on Fig. 6(a) are the measured B-values obtained from supposedly saturated samples of the compacted clay consolidated to different effective confining pressures. It is seen

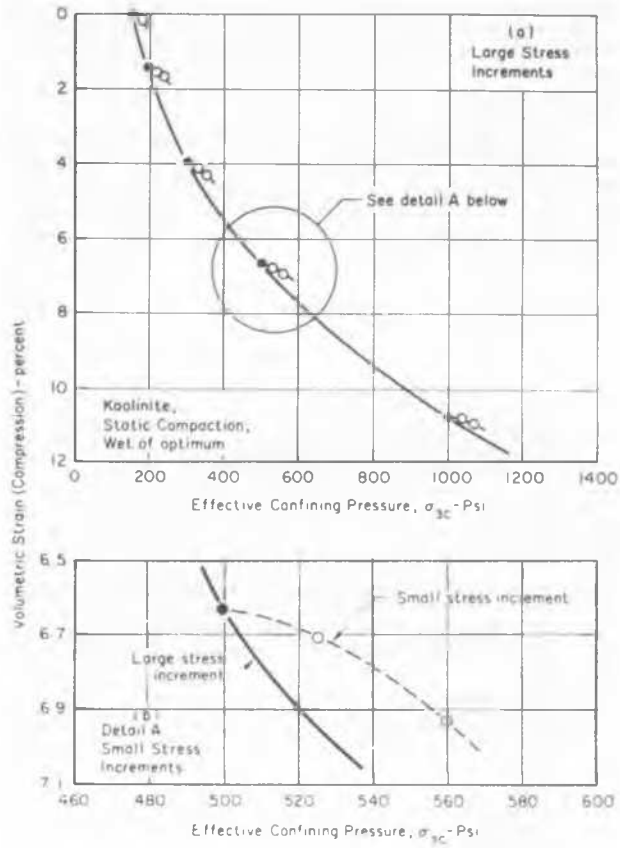

Fig. 4 Isotropic Consolidation of Compacted Clay

that the theoretical B-values computed from the large stress increment ratio consolidation test data are much greater than the actual measured values. At 1000 psi consolidation pressure, the theoretical B-value was 0.97, whereas the measured B-value was only 0.86. This discrepancy was too large to ascribe to normal experimental errors, and another explanation was required.

The data shown on Fig. 4(a), which was used to compute the above-described theoretical B-values, was obtained from a consolidation test performed in the usual manner by using loading with large stress increments. However, the changes in effective stress which occur during a B-value test on a saturated sample must be very small even if large cell pressure increments are used.

Leonards and his collaborators (1959, 1961, 1964) have shown that the shape of the consolidation curve for clay soil can be greatly effected by the load increment ratio that is used for the test. If very small load increment ratios are used, the samples appear to be initially much less compressible than if large load increments are employed. Finally, however, as the load is increased by small increments, the consolidation curve steepens

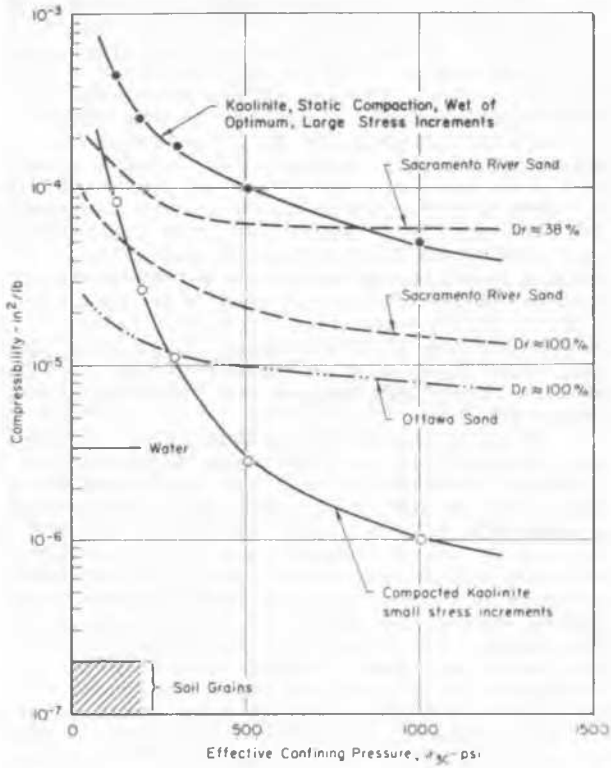

Fig. 5 Compressibility of Sand and Clay

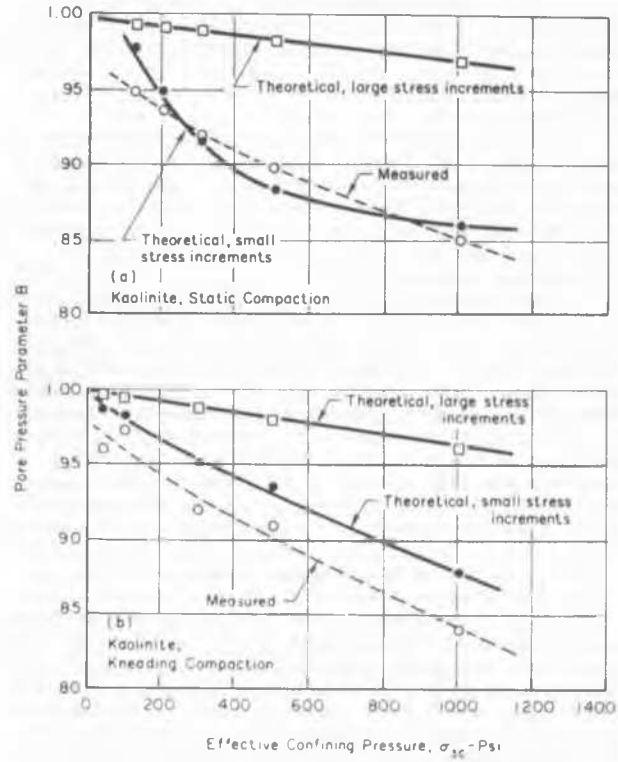

Fig. 6 Theoretical and Measured B-Values for Compacted Clay

and eventually coincides with the large load increment curve. This behavior, under small load increments, leads to an apparent or pseudo preconsolidation effect. Similar effects have also been reported by Raymond (1966), Bjerrum (1967), and Brezezinski (1968), when soil was allowed to consolidate for a long time at one static effective stress, and this appears to be a common behavior of many clay soils.

Therefore, because clay soils are less compressible when loaded in small increments than when loaded by large increments, and because a B-value test involves only a very small change in effective stress, it follows that the appropriate value of C_d should be a value obtained from a small load increment consolidation test. The results of such a test are also presented on Fig. 4. The sample was saturated by means of back pressure and consolidated to equilibrium under a nominal effective confining pressure of 150 psi. At this time the confining pressure was increased by a very small amount. The drainage line was kept open to a sensitive burrette, and the amount of pore water which was expelled under this small load increase was observed. When equilibrium had been reached, a second small stress increment was applied and the corresponding volume changes noted as before. This process was repeated several times until sufficient data had been obtained to define the shape of the consolidation curve under small load increments. A large stress increment was then applied and the sample allowed to consolidate to the new and considerably higher effective stress condition. After reaching equilibrium, the small stress increment process was again repeated.

The consolidation data for the small stress increment test is also shown on Fig. 4(a), superimposed on the large stress increment consolidation curve. An enlarged detail of one set of small stress increment data is shown on Fig. 4(b). It is seen that this clay soil behaves similarly to the behavior described by Leonards and others. Under the initial, very small stress increments, the slope of the consolidation curve is much flatter than obtained from the large stress increment curve. However, even under small stress increments, the consolidation curve eventually steepens and joins with the large stress increment curve.

The tangent to the small stress increment curve at the beginning of each set of data was used to define a more appropriate value of C_d for use in Eq. 3 to compute a theoretical B-value. Values of C_d computed for both the small and large stress increment tests are also shown on Fig. 5. The compressibility of compacted clay under small stress increments is not only much less than under large stress increments, but at high confining pressures, it is even less compressible than water.

In fact, it is only about one order of magnitude more compressible than the soil grains themselves. In view of these data, it is not surprising that the measured B-values were found to be very low at high confining pressures, even though the soil was known to be saturated.

Theoretical values of B computed from Eq. 3 using C_d data obtained from the small stress increment data on Fig. 4 are shown on Fig. 6(a) where they are compared with the measured B-values. In this case, the agreement between the theoretical and measured values is very good.

The data presented on Figs. 4, 5 and 6(a) were obtained from samples of Kaolonite compacted wet of optimum by a static compaction method. Seed, Mitchell and Chan (1960) have shown that this type of compaction often leads to a flocculated soil structure, whereas kneading compaction to the same density and water content will produce a dispersed soil structure. They also showed that compressibility and other physical properties were greatly affected by the type of compaction used. In another study, Lee and Haley (1968) observed these same phenomena for the clay described herein.

The above-described special B-value tests were repeated on samples of the same clay prepared by kneading compaction. A summary of the measured and theoretical B-values for this clay are shown on Fig. 6(b). In this case, the sample prepared by kneading compaction was saturated and consolidated under an effective confining pressure of only 40 psi as compared to the consolidation pressure of 150 psi used for the statically compacted sample. However, as shown on Fig. 6(b), the same general behavior was observed in both cases. The theoretical B-values computed from large stress increment ratio consolidation tests were much too high. But, when the small stress increment test data was used, the theoretical and measured B-values were in tolerable agreement.

PORE PRESSURES DURING A SHEAR TEST

Skempton's (1954) well-known pore pressure parameter equation may be written as follows:

$$\Delta u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right] \qquad (4a)$$

$$\Delta u = B\Delta\sigma_3 + \bar{A}(\Delta\sigma_1 - \Delta\sigma_3) \qquad (4b)$$

$$\text{where } \bar{A} = AB, \text{ and } \bar{A} = A \text{ if } B = 1.0. \qquad (4c)$$

Eq. 4 is a convenient expression of the pore pressure response to changes in principal stresses, and in the above form, it is especially suited for use in interpreting data from the triaxial compression test. The parameters A or $\bar{A}$ relate changes in pore pressure to changes in deviator or shear stress. However, as shown by the above equations, it is important to know the value of the parameter B as well as the parameters A or $\bar{A}$ for a complete interpretation of the test data. As shown on Fig. 6, the simple rule of thumb that $B \approx 1.0$ for all saturated soil may lead to significant errors in some cases.

In previous paragraphs it was shown that, even for fully saturated soils, B-values significantly less than unity could be expected because of a stiff structure. For clays this stiff structure could be the result of a pseudo preconsolidation effect developed by consolidating for a long time at one effective confining pressure as illustrated by Leonards, et al. Leonards has further shown that this pseudo-stiff structure was destroyed after a certain amount of stress increments had been applied. Therefore, the question arose as to whether or not the shearing strains produced by a triaxial compression test to failure would destroy this pseudo-stiff structure and lead to higher B-values throughout the shearing stage of an undrained test than those measured under static consolidation conditions before the shearing stage commenced.

To investigate this possibility, a special isotropically consolidated undrained triaxial compression test was performed on a sample of the Kaolinite statically compacted to approximately the same water content and density as the previously described samples. The specimen was saturated using 70 psi back pressure and isotropically consolidated to a nominal effective stress of only 20 psi. At this stage, the measured B-value was 0.99. The sample was then further consolidated in increments at higher effective confining pressures, and the B-values were measured at the end of each increment. It was found that the B-values decreased steadily with increasing consolidation pressure in the same manner as shown on Fig. 6. At the final consolidation pressure of 1000 psi, the measured B-value was only 0.90.

The sample was then axially loaded undrained to beyond the failure condition at a controlled rate of strain of 0.05 percent per minute. At several intervals during the shearing stage of the test, a B-value was measured. During these B-value tests, the axial strain was stopped, but the axial load was not intentionally reduced. However, there was a small reduction due to creep relaxation, which was quickly recovered when the loading recommenced, but this appeared to have no significant influence on the behavior of the sample.

The results of these B-value tests at various axial strains are illustrated on Fig. 7. On the very enlarged scale of Fig. 7, the B-value data points show somewhat more scatter than desirable. Some of the scatter is probably due to small pore pressure changes which occur as a result of creep relaxation of the axial stress during the time interval in which the B-value measurements were being taken. It is noted that during all stages of the test, the B-values remained approximately the same as at the start of the test.

In addition to this special test, B-values were measured on a number of other specimens after failure and before the axial load had been removed. In all cases, the B-values measured after failure were much lower than unity, and were approximately the same as observed immediately after consolidation and before the axial loading stage commenced.

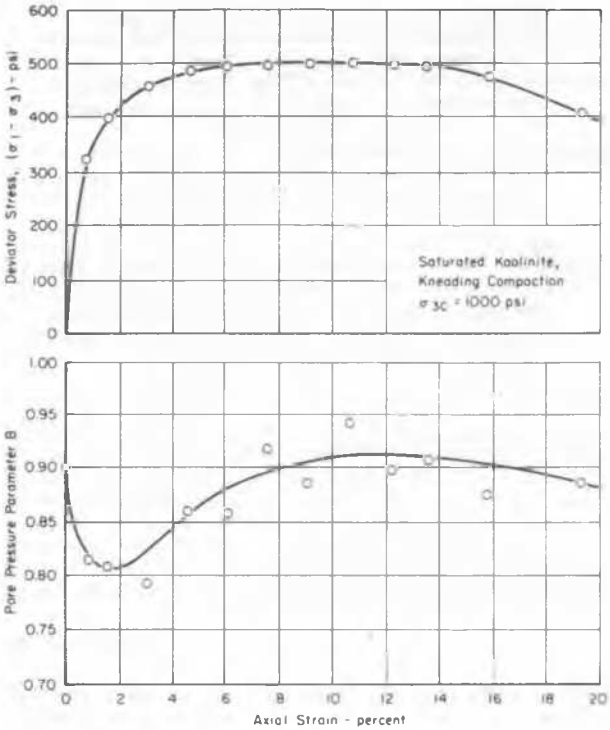

Fig. 7 B-Values Measured at Various Stages During Loading of an Undrained Triaxial Test

Therefore, in spite of the scatter shown on Fig. 7, it is felt that the data do indicate a definite trend and serve to demonstrate that for these conditions, the B parameter never attains a high value close to unity at any strain during the axial loading stage of the test. Therefore, B-values measured at the end of the isotropic consolidation stage should be appropriate to other subsequent conditions of loading during the undrained test, for example, at failure.

CONCLUSIONS

Several examples have been cited where the B-values of saturated soils are considerably less than unity. These examples have been obtained from direct measurements and confirmed by theoretical calculations based on Eq. 2. Therefore, the common rule of thumb that $B \approx 1.0$ for all saturated soil may be erroneous and misleading in some situations.

The principal factor contributing to a low B-value is a stiff soil structure represented by a low value of compressibility. The compressibility of all soils decreases with increasing confining pressure. Some stiff soils such as Ottawa sand and soil cement have very low compressibilities at all confining pressures. Clay soils represent a special case that sometimes may be misleading. When consolidated using large stress increments, the compressibility appears to be relatively high. However, when loaded by means of very small stress increments, the clay soil can be very stiff and exhibit very low compressibility values, which for moderately high confining pressures, may be less than the compressibility of water.

A comparison of the compressibility values of compacted clay and two types of sand is presented on Fig. 5, which illustrates the large stiffening effect caused by loading the compacted clay samples with small stress increments. Because the changes in effective stress are very small during a B-value test, it follows that the appropriate soil compressibility associated with the B-value measurement is the one corresponding to the small stress increments. Using these compressibility data, theoretical B-values were computed which were considerably lower than normally associated with saturated soil, but which, nevertheless, agreed with the values which were actually measured.

Measured B-values at different stages of axial loading indicate that the B-value measured at the end of consolidation is appropriate for any subsequent undrained loading condition.

It is therefore concluded that one must exercise caution in using the results of a B-value test to determine whether or not a soil sample is saturated. Since a low B-value may not necessarily indicate a non-saturated condition, it is recommended that one or more independent checks should also be used. One such check is to use a back pressure in excess of that predicted by Eq. 2. Another check is to use the procedure followed by Wissa and Ladd (1965) of measuring the B-value using several successively higher back pressures. A measured B-value which is independent of back pressure indicates 100 percent saturation.

ACKNOWLEDGEMENTS

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