Proceedings of the Seventh Southeast Asian Geotechnical Conference, Hong Kong.Vol.1.
pp. 787-803. November 22-26. 1982

STRAIN RATES FOR UNSATURATED SOIL SHEAR STRENGTH TESTING

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SUMMARY The paper presents a theoretical model to estimate the time required to shear an unsaturated soil. More specifically, it gives the relationship between the time required for a desired degree of pore pressure dissipation during a drained test. The physical properties of the soil (i.e., the compressibility and coefficient of permeability) along with the drainage boundary conditions are the main factors affecting the strain rate. The equilization time required for an undrained strength test where the pore-water pressure is measured, is only slightly less than that required for a drained test since the high air entry disc controls pore pressure equilization. Consequently the time required to fail an unsaturated soil in the drained mode is a conservative estimated that can be applied to the undrained mode of testing.

The high air entry discs with low coefficients of permeability, used to separate the pore-air and pore-water pressures during unsaturated soil testing, are a major factor affecting strain rate. Based on the theoretical model, a parametric study was performed to illustrate the factors affecting the selected strain rate. Design charts are shown for two Hong Kong soils (i.e., decomposed granite and decomposed rhyolite). A general plot is presented that can be used for selecting the strain rate for testing any unsaturated soil. The results indicate that impeded drainage caused by the high air entry disc, has a significant influence on the selected strain rate.

INTRODUCTION

In recent years there has been an increased interest in the shear strength testing of unsaturated soils and defining strength in terms of fundamental shear strength parameters. The most common problems are slope instability associated with the reduction in soil suction in unsaturated soil slopes. Numerous strength testing procedures have been suggested and the strain rate of testing is a question which must always be addressed. To-date, the selected strain rates have been largely based on trial and error due to the lack of a theory for estimating strain rate (or time to failure).

HISTORY

It appears that prior to the 1960's unsaturated soils were tested in much the same manner as saturated soils using conventional testing equipment. Strain rates for testing unsaturated soils were relatively high. In the early 1960's, high air entry ceramic discs were placed between the sample and the lower pedestal in an attempt to measure the pore-water pressures separately from the pore-air pressure. Several empirical procedures were proposed for estimating the strain rate for testing (Bishop, Alpah, Donald and Blight, 1960; Lumb, 1966; Ruddock, 1966). Gradually, the testing equipment and procedures were modified such that the pore-air and pore-water pressures could be independently measured or controlled. The axis-translation technique (Hilf, 1956) was used to overcome the limitations associated with the pore-water pressure measuring system (i.e., cavitation at -1 atmosphere). A high air entry disc was commonly sealed onto the base pedestal while a coarse, low air entry disc was placed on the top of the soil sample. This allowed for the independent control or measurement of the pore-water and pore-air pressure, respectively. Similar testing equipment has been used by numerous researchers (Bishop and Donald, 1961; MIT, 1963; Gulhati and Satija, 1981; Ho and Fredlund, 1982).

Figure 1 shows the typical modifications applied to a conventional triaxial cell. The types of testing procedures that can be used for unsaturated soil testing are similar to the procedures used in saturated soil testing. First, the pore-water and pore-air pressures can be set and controlled while the sample is loaded to failure. This, in essence, is a consolidated drain (i.e., CD) type of test. Second, the soil sample can be allowed to come to equilibrium at a set of stresses and then the pore-air and pore-water pressures can be measured while the sample is loaded to failure. This, in essence, is a consolidated undrained test with pore pressure measurements (i.e., CU_p). A third testing procedure was proposed (Bishop and Henkel, 1962) in which the pore-water pressure was measured while the pore-air pressure was controlled. These tests are called constant water content tests (i.e., CW tests). For any of the above

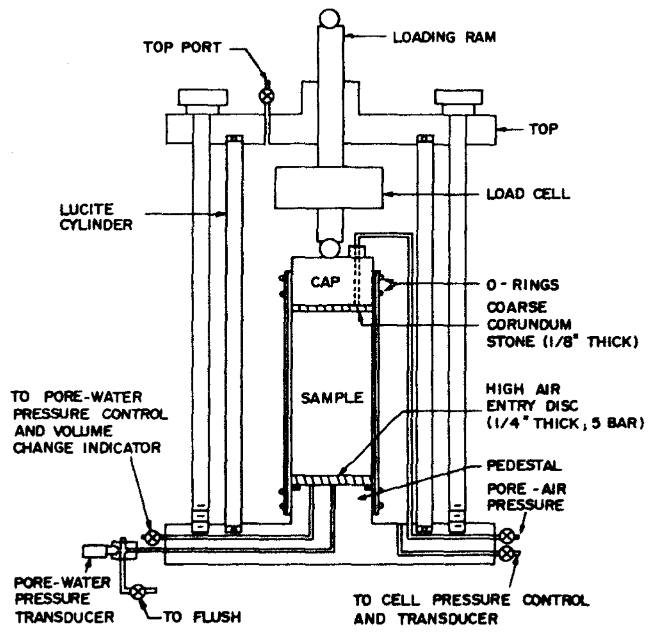


Fig. 1 - Modified triaxial cell for testing unsaturated soils.

procedures, theoretically the necessary stresses are known at failure and the shear strength results can be analysed. However, in all cases, it is important that the strain rate be sufficiently slow so as to produce accurate data.

Escario (1980) modified a direct shear machine (Figure 2). The pore-air and pore-water pressures could be independently controlled and the axis-translation technique used for a wide range of soil suction. The main advantage of the direct shear test is the relatively short length of drainage path which allows for more rapid pore-pressure equilization.

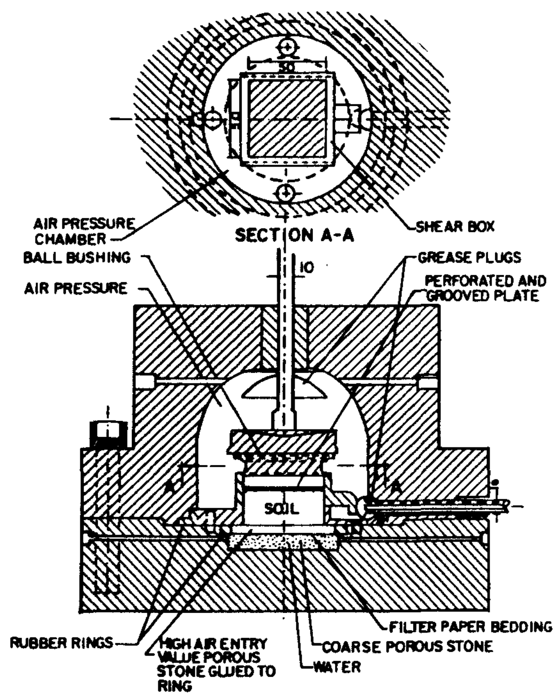


Fig. 2 - Modified direct shear apparatus for testing unsaturated soils (From Escario, 1980).

Researchers have realized the need to perform strength tests on unsaturated soils at an extremely slow rate in order to ensure equilization or dissipation of induced pore pressures (Donald, 1961; Satija and Gulhati, 1979). However, no theoretical procedure was available for predicting satisfactory strain rates. Rather, it seemed most appropriate to assess a strain rate (or "time to failure") by some trial and error procedure. Donald (1961) suggested that the effect of strain rate on maximum deviator stress be used as a criterion to approximate a strain rate. Test data (Bishop and Henkel, 1962) showed the relationship between strain rate and deviator stress and illustrated a levelling off a deviator stress below a particular strain rate. This gives insight into an upper limit for failing an unsaturated soil sample but will not necessarily be sufficiently slow to ensure pore pressure equilization or dissipation. Satija and Gulhati (1979) suggested that for constant water tests (i.e., CW tests), changes in matric suction (i.e., $(u_a - u_w)$) would be reasonable indicator of an appropriate strain rate. Samples of compacted Dhanaurhi clay were tested at rates varying from 0.2% per minute to 0.0032% per minute. The strain rate at which the pore-water pressures remained essentially unchanged was selected as an appropriate strain rate (Figure 3). It was concluded that a strain rate of 0.04% per minute was sufficiently slow for constant water content tests on Dhanaurhi clay. It was also concluded that a strain rate of one-fifth of the above value be used for consolidated drained tests. The Dhanaurhi clay has a liquid limit of 48.5%, a plastic limit of 25.0% and a percent clay size of 25.0%.

STRAIN RATE THEORY

The strain rate for shear strength testing should be sufficiently slow to avoid non-uniformity in the pore-water pressure distribution within the sample. This is true for testing both saturated and unsaturated soils. For undrained triaxial tests on saturated samples, the axial strain rate is selected to ensure at least 95% equilization of the induced pore-water pressure. Similarly, 95% equilization of the induced pore-water pressure is sought for drained tests. Gibson and Henkel (1954) applied the consolidation theory to a triaxial sample and formulated a theoretical method for approximating the "time to failure" required for a drained test. The procedure has been widely accepted for estimating the strain rate for triaxial testing (Bishop and Henkel, 1962).

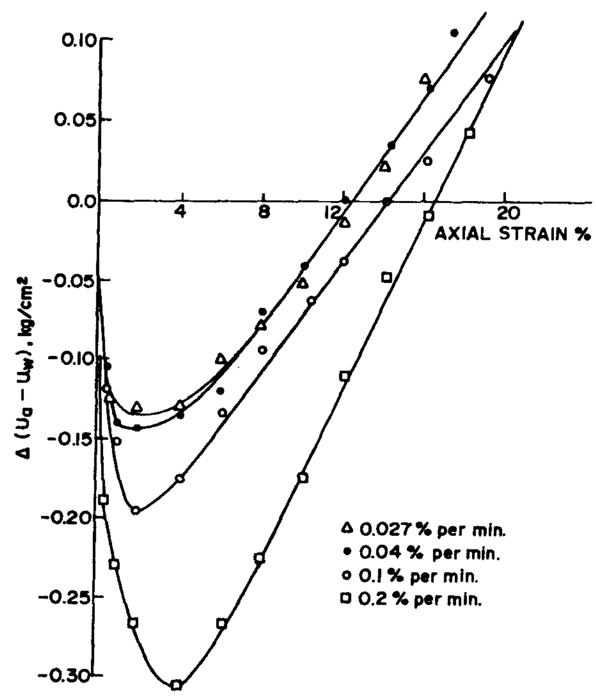


Fig. 3 - The effect of strain rate on measured pore-water pressures in compacted Dhanaurhi clay (From Satija and Gulhati, 1979).

The problem of a suitable strain rate for testing unsaturated soils is somewhat more complicated than for saturated soils. The first complication is the high air entry disc placed at the bottom of the soil sample. These discs have low coefficients of permeability (Table 1). The discs are kept saturated and allow the passage of water but not air. However, the low coefficient of permeability of the disc results in impeded flow in and out of the soil sample. In 1963, Bishop and Gibson presented a solution to the problem of impeded flow from saturated triaxial and oedometer specimens as a result of a low permeability disc placed next to the soil. The second complication is the physical properties of the unsaturated soil. The two properties of primary concern are the coefficient of permeability and compressibility of the unsaturated soil.

TABLE I

PROPERTIES OF HIGH AIR ENTRY CERAMIC DISCS*

Property	Air Entry Value	
	5 Bar	15 Bar
Thickness (cm)	0.622	0.306
Permeability (cm/sec)**	1.2×10^{-9}	7.9×10^{-11}

* Discs were manufactured by Soilmoisture Corporation, Santa Barbara, California, U.S.A.

** Permeabilities were measured by Fredlund (1973). "Volume Change Behavior of Unsaturated Soils", Ph.D. Dissertation, University of Alberta, Edmonton, Canada.

The choice of a strain rate must take into account the ability of the sample and the measuring system to dissipate excess pore pressures. Let us first consider the unsaturated soil sample. The first assumption made is that the coefficient of permeability with respect to the air phase is much larger than the coefficient of permeability with respect to the water phase. This will generally be the case once the air phase is continuous. Therefore, it is assumed that the pore-water pressure dissipation will control the strain rate.

In 1979, Fredlund and Hasan derived two partial differential equations that must be solved simultaneously to predict the pore-water and pore-air pressure dissipation in an unsaturated soil. The partial differential equation satisfying continuity of the water phase was written,

$$\frac{\partial u_w}{\partial t} = C_w \frac{\partial u_a}{\partial t} + c_v^w \frac{\partial^2 u_w}{\partial y^2} \quad (1)$$

where

u_w = pore-water pressure,

u_a = pore-air pressure,

t = time,

y = coordinate direction,

C_w = interactive constant associated with the water phase,

c_v^w = coefficient of consolidation with respect to the water phase.

During the strength testing of an unsaturated soil, the gradient in the air phase will generally be significantly less than in the water phase. For example, the air phase is relatively insensitive to loads applied to the soil due to its high compressibility. Considered another way, the pore-air pore pressure parameters will be smaller than the pore-water pore pressure parameters (Hasan and Fredlund, 1980). The result is that the gradient in the air phase will be much smaller than the gradient in the water phase. Therefore, for purposes of approximating the consolidation process, the partial differential equation can be written:

$$\frac{\partial u_w}{\partial t} = c_v^w \frac{\partial^2 u_w}{\partial y^2} \tag{2}$$

The equation now has the same form as the conventional Terzaghi consolidation equation. However, further consideration must be given to the coefficient of consolidation term.

$$c_v^w = \frac{k_w}{m_2^w \rho_w g} \tag{3}$$

where

k_w = coefficient of permeability with respect to the water phase,

ρ_w = density of water,

g = acceleration due to gravity,

m_2^w = slope of the matric suction (i.e., $(u_a - u_w)$ plot versus volumetric water content.

Volumetric Water Content Versus Suction Modulus

The modulus term, m_2^w, for unsaturated soils does not bear a direct relationship to the compressibility of the soil structure. In a saturated soil, the amount of water removed from a soil during consolidation is always equal to the volume change of the soil structure. Therefore, the modulus satisfying continuity is equal to the compressibility of the soil structure, m_v, For an unsaturated soil, the modulus arising in the consolidation derivation is the soil slope of the plot of volume of water in the soil versus matric suction (i.e., m_2^w).

$$m_2^w = \frac{G_s}{1+e_o} \frac{dw}{d(u_a - u_w)} \tag{4}$$

where

G_s = specific gravity of soil solids,

e_o = initial void ratio,

dw = change in gravimetric water content, and

$d(u_a - u_w)$ = change in matric suction.

Figure 4 shows a typical relationship between the relevant moduli for a saturated and unsaturated soil. The uppermost curve is a conventional, one-dimensional consolidation curve reflecting a preconsolidation pressure and a subsequent parabolic shape. If matric suction were applied to an identical sample, the shape of the water volume change versus suction plot has a different shape. (Note that the gravimetric water content is multiplied by the specific gravity of the solids to make the ordinate scale comparable to void ratio). The initial portion of the curve is gradually curved since the volume change is isotropic (Fredlund, 1967). At a particular matric suction value, the soil commences desaturation. This desaturation continues as suction increases. As a result, the modulus with respect to a change in matric suction is considerably higher than the conventional compressibility of the soil structure. It is possible for the soil structure to be highly incompressible while the modulus with respect to matric suction (i.e., m_2^w) may be relatively high.

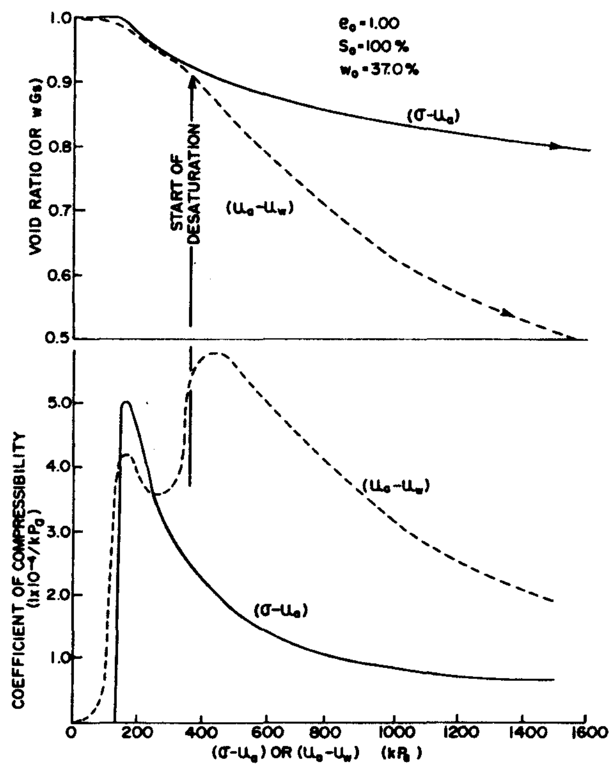


Fig. 4 - Typical volume change versus stress state variables (i.e., $(\sigma - u_a)$ and $(u_a - u_w)$) plots for an unsaturated soil

Coefficient of Permeability With Respect to the Water Phase

The coefficient of permeability of an unsaturated soil decreases rapidly as a soil desaturates. It is as though the air in the soil reduces the cross-sectional area through which water can flow. Childs (1969) explains the permeability of an unsaturated soil as follows: "The conductivity channels with the unsaturated material are those pores which are full of water at a particular suction corresponding to a chosen moisture content. The air-filled pores are ineffective since water can hardly pass through a pore without occupying it. It follows that the air-filled pores could be filled with solids, such as wax, without affecting the rate of flow of fluid through the remaining pores, and the porous material treated in this way must now be regarded as a new saturated material with the same properties as the original unsaturated material".

Figure 5 shows a typical plot of the coefficient of permeability with respect to the water phase versus volumetric water content for a fine sand (Elzeftawy and Dempsey, 1976). Although the soils are relatively coarse, the coefficient of permeability rapidly decreases as water content decreases (i.e., suction increases). The trends are similar for finer, silty and clayey soils. The calculated lines on Figure 5 are based on the theory proposed by Green and Corey (1971) and Elzeftawy and Dempsey (1976). The coefficient of permeability for all water contents (and degrees of saturation) can be computed if the saturated permeability and also the relationship between matric suction and water content are experimentally measured. Corey (1957) measured the coefficient of permeability of unsaturated soils and found that it varied as the fourth power of a saturation ratio.

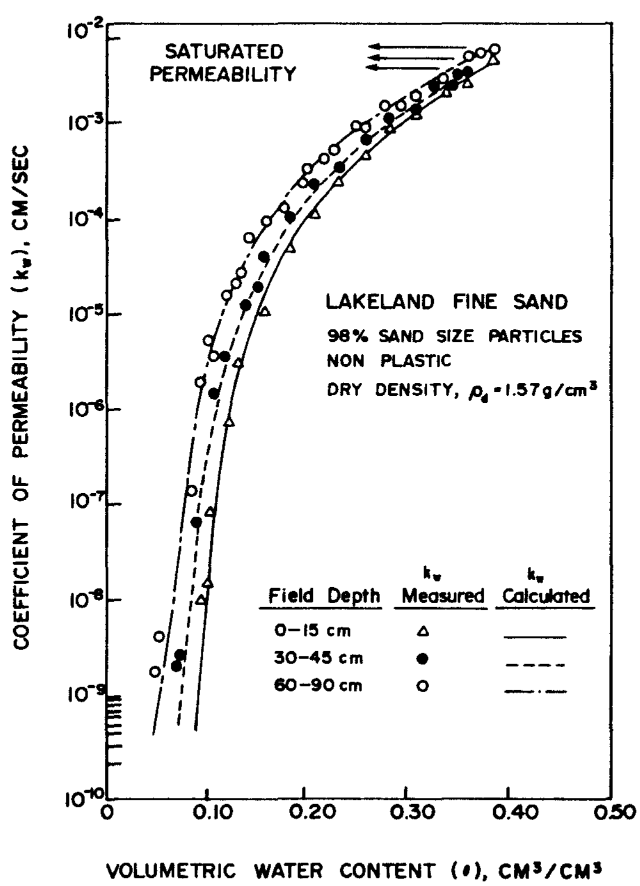


Fig. 5 - Typical plot of coefficient of permeability versus volumetric water content for a fine sand (From Elzeftawy and Dempsey, 1976).

$$k_w = k_{sat} \left[\frac{S-S_r}{1.0-S_r}\right]^4 \times 100 \qquad (5)$$

where

k_w = coefficient of permeability with respect to the water phase at any degree of saturation,

k_{sat} = saturated coefficient of permeability,

S = degree of saturation,

S_r = residual degree of saturation.

The residual degree of saturation S_r, has an absolute lower limit of 0.0 but generally lies in the range of 0.3 to 0.6. Figure 6 shows a plot of equation (5) for two values of S_r and several initial coefficients of permeability (i.e., saturated permeability values). The plot shows that generally the coefficient of permeability will decrease by 3 or 4 orders of magnitude as the soil desaturates due to an increase in matric suction. The decrease in permeability reduces the coefficient of consolidation by a similar amount

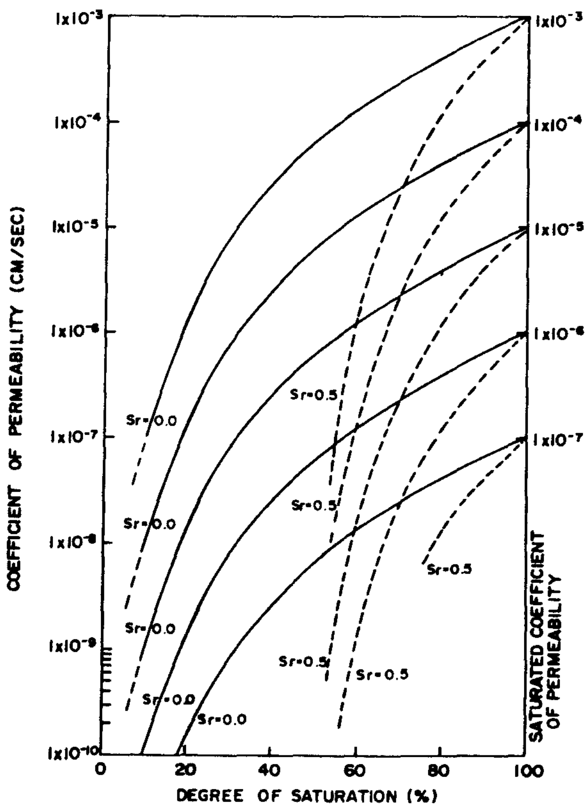


Fig. 6 - Relationship between degree of saturation and coefficient of permeability proposed by Corey (1957).

Effect of the High Air Entry Disc

The high air entry disc at the base of the soil sample has a low coefficient of permeability and affects the rate of movement of water in and out of the sample. For analysis purposes, the high air entry disc is assumed to be incompressible. To satisfy continuity between the soil sample and the high air entry disc, the pore-water pressure must be the same on each side of the interface and the gradient across the high air entry disc must be constant

$$k_w \frac{\partial u_w}{\partial y} = k_d \frac{u_w}{h_d} \qquad (6)$$

where

k_d = coefficient of permeability of the high air entry disc,

h_d = thickness of the high air entry disc.

Equation (6) applies for the interface between the soil and the high air entry disc, at all times.

Solution of the Impeded Flow Consolidation Problem

The solution to the problem of impeded drainage from an unsaturated soil is equivalent mathematically to the heat conduction problem of a slab initially at a uniform temperature, and cooling at its surface in accordance to Newton's law (Carslaw and Jaeger, 1959). The Fourier's series solution to this problem has been published by Gray (1945). The Fourier's equation shows that the properties of the high air entry disc appear in the solution in the form of an impedance type factor.

$$\lambda = \frac{k_d h'}{k_w h_d} \tag{7}$$

where

λ = impedance factor,

h' = length of the drainage path (i.e., length of the soil sample).

The solution can be written in terms of the "time to failure" required to maintain a selected degree of consolidation in the soil (Bishop and Gibson, 1963).

$$t_f = \frac{h^2}{\eta c_v^w (1-\bar{U}_f)} \tag{8}$$

where

t_f = time required to fail the sample,

$\bar{U}_f$ = average degree of dissipation of the induced pore-water pressure at failure,

$2h$ = the actual length of the specimen, whether drainage is from one or both ends,

$\eta = \frac{0.75}{(1 + 3/\lambda)}$ when drainage is from one end of the soil sample,

$\eta = \frac{3.0}{(1 + 3/\lambda)}$ when drainage is from both ends of the soil sample.

Approximate properties of the unsaturated soil must be used to obtain an estimate of c_v^w. The properties of the high air entry disc will affect the impedance factor, λ. The suggested degree of dissipation of induced pore-water pressure is 95% but any value can be used. Equation (8) must be solved in order to compute the rate at which an unsaturated soil sample should be strained in the drained mode.

$$\dot{\varepsilon} = \frac{\varepsilon_f}{t_f} \tag{9}$$

where

$\dot{\varepsilon}$ = strain rate for failing the sample,

ε_f = strain of the sample at failure.

It is difficult to estimate the strain at failure, prior to performing the strength test. However, generally there is some knowledge of the shape of the stress versus strain curve and the point at which the strength peaks. This information is used to obtain an initial estimate of strain at failure, which may have to be revised as testing proceeds. In the case of a multi-stage type of test, the time required to bring the specimen to a maximum deviator stress for each "stage", must be evaluated.

TIME TO FAILURE BASED ON THEORY

Based on the proposed theory for selecting strain rates for unsaturated soils, a parametric study was performed to illustrate the factors affecting the "time to failure". An attempt was made to develop charts relevant for two Hong Kong residual soils; namely, decomposed granite and decomposed rhyolite. A range of values for the soil properties was assumed to demonstrate their effect on the "time to failure". In order to minimize the number of variables involved, the soil modulus, m_2^w, was kept constant at 0.6 x 10^{-3} per kPa. As well, a constant sample height of 14 cm (5.5 inches) was selected.

Figure 7 shows a plot of the thickness of the high air entry disc versus the "time to failure" for a soil with a coefficient of permeability, k_w, of 3 x 10^{-3} cm/sec. Contours of ratios of k_w/k_d for approximately 7 orders of magnitude are shown. The graph indicates that the "time to failure" would be extremely fast if there were no impedance due to the measuring system. The thickness of the high air entry disc has some effect on the "time to failure". However, by far the most significant factor is the coefficient of permeability of the high air entry disc. For example, a 5-bar, high air entry disc causes the "time to failure" to increase by more than 4 orders of magnitude over the time required from the standpoint of the soil. The graph clearly indicated that the permeability of the high air entry disc completely dominates the "time to failure" for this soil.

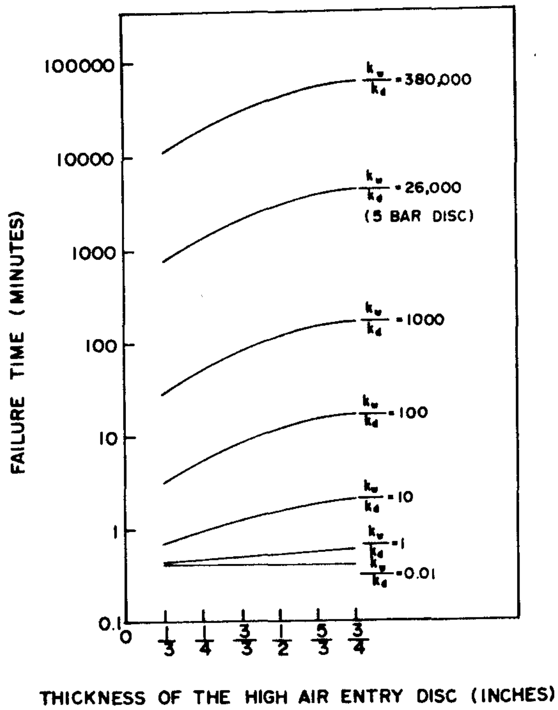


Fig. 7 - Thickness of high air entry disc versus "time to failure" for the soil coefficient of permeability of 3×10^{-3} cm/sec.

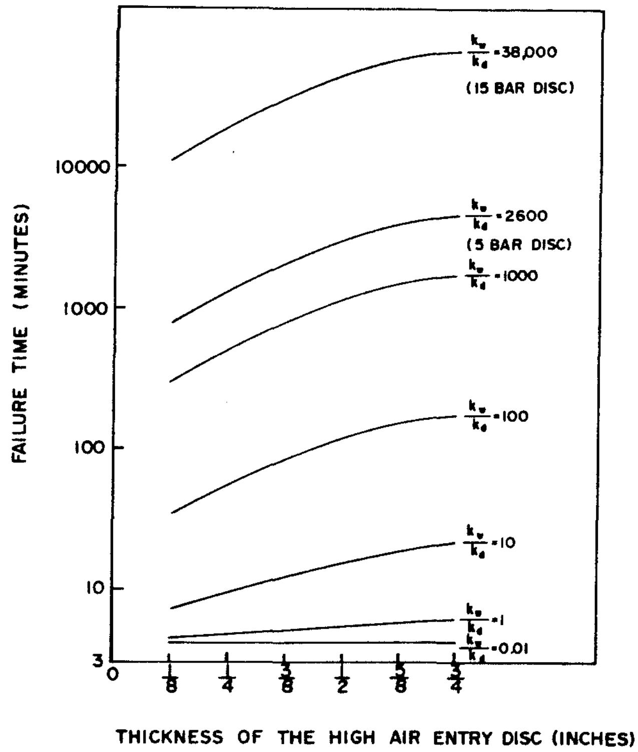
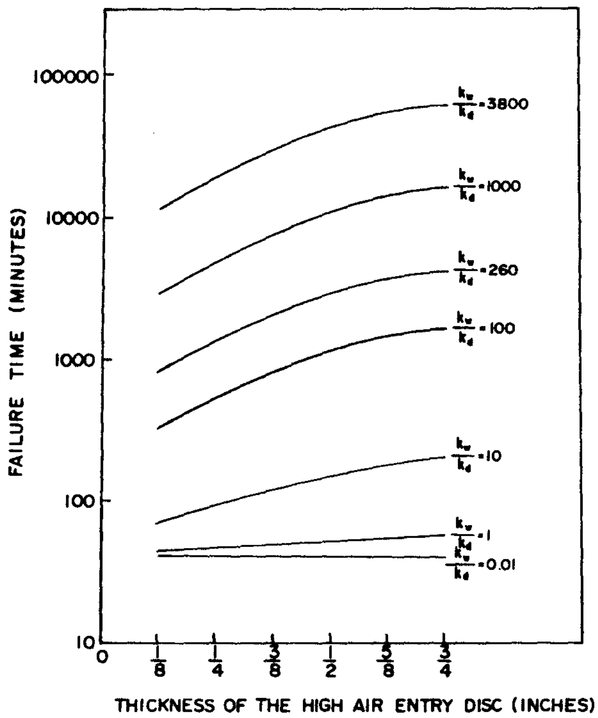


Fig. 8 - Thickness of high air entry disc versus "time to failure" for the soil ccefficient of permeability of 3×10^{-4} cm/sec.

Figure 8 and 9 show similar plots of the "time to failure" when the permeability of the soil is decreased by 1 and 2 orders of magnitude, respectively. The plots show similar trends to Figure 7; however, the effect of the high air entry disc is less pronounced. As anticipated, the lower coefficient of permeability of the soil, the less significant is the impedance due to the high air entry disc. Figures 7 and 9 inclusive, cover the anticipated range of the coefficients of permeability for the decomposed granites in Hong Kong. The graphs assist in the selection of a suitable strain rate.



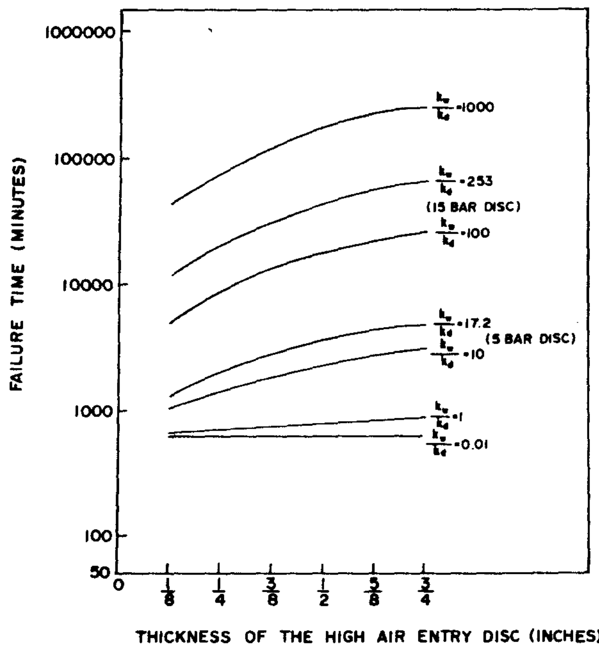


Fig. 9 - Thickness of high air entry disc versus "time to failure" for a soil coefficient of permeability of 3×10^{-5} cm/sec.

Fig. 10 - Thickness of high air entry disc versus "time to failure" for a soil coefficient of permeability of 2×10^{-6} cm/sec.

Figure 10 extends the coefficient of permeability of the soil by approximately one order of magnitude (i.e., 2×10^{-6} cm/sec). The decomposed rhyolites in Hong Kong were also tested and may have coefficients of permeability extending into this range.

A general plot (Figure 11) was prepared using the dimensionless impedance factor, λ, and the coefficient of consolidation with respect to the water phase, c_v^w. The dimensionless form of the graph allows the assessment of the "time to failure" for a wide range of soils and pore-pressure measuring system.

DISCUSSION

It is evident from the results of the parameter study that the impeded drainage due to the presence of the high air entry disc is generally the governing factor in assessing the "time to failure" for unsaturated soil testing. Little can be done to increase the rate of testing. The high air entry discs generally range between 1/8 inch and 3/8 inches in thickness. The thinner discs will allow a slightly increased rate of testing. However, the thicker discs are superior because they greatly reduce the rate at which air diffuses through the disc and collects below the disc. A flushing system should be used periodically (i.e., every few hours) to remove diffused air. It may also be possible in some cases to use 2 bar ceramic discs and significantly increase the rate of testing. Of course, the air entry value of the disc must not be exceeded.

The modified direct shear apparatus provides the advantage of using a thin soil sample. This could assist in expediting testing, particularly on extremely low permeability soils.

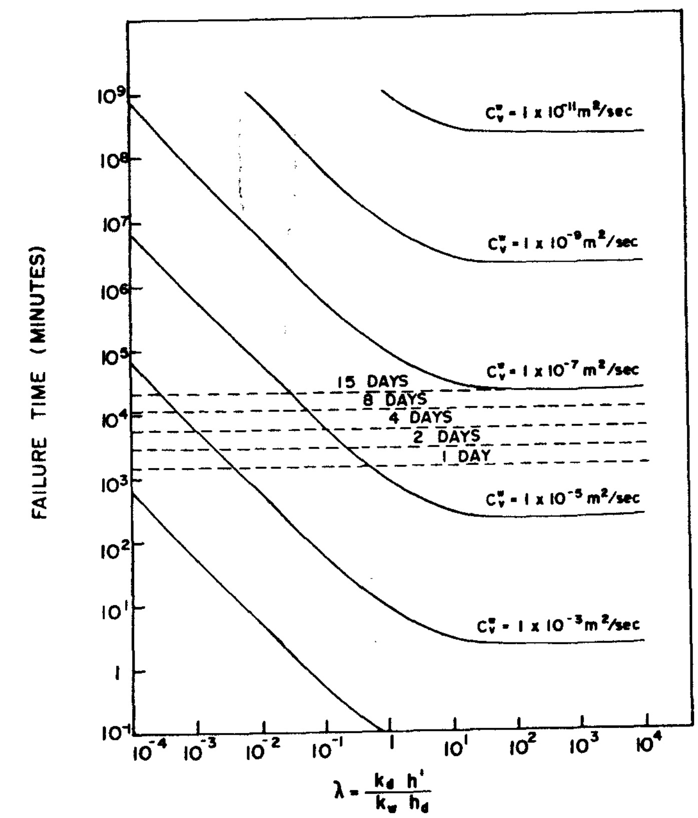


Fig. 11 - Dimensionless impedance factor versus "time to failure" for various coefficients of consolidation for the soil.

Caution is expressed regarding using strain rates which are too high since sufficient time is not allowed for the dissipation of excess pore-water pressures. Even when an unsaturated soil appears relatively pervious, the strain rate will still need to be relatively slow due to the high air entry disc.

Water can more rapidly flow from the soil out through the high air entry disc than vice versa. Therefore, if a soil tends to dilate during shear, it is extremely difficult to get water to flow upward through the low permeability disc into the soil. The problem can be somewhat alleviated by using a significant axis translation to ensure positive pore-water pressures and by flushing desired water below the high air entry disc every few hours (Fredlund, 1975).

An extensive laboratory strength testing program was conducted on decomposed granite and decomposed thyolite samples from Hong Kong (Ho and Fredlund, 1982). A multi-stage type testing procedure was used. Conservative estimates of strain rate were computed. The strain rate was essentially the same for both soils and ranged from 0.001% per minute to 0.004% per minute. Part of the reason for the low strain rate was the fact that the soils reached their peak strength at low strains. For example, the peak strength for stage I generally occurred between 3 and 5 percent strain. Peak strength occurred at approximately 1 to 3 percent strain for stages II and III. The test data (Ho and Fredlund, 1982) showed reasonable angles of friction but there was not direct experimental verification of the strain rates. It is suggested that a study involving higher strain rates would be useful in assessing whether testing could be expedited. It may also be possible to use a higher strain rate (i.e.,approximately two times as fast) for stage I than for stages II and III in multi-stage type testing.

Since the "time to failure" is strongly controlled by the properties of the high air entry disc, equation (8) can also be used to estimate the strain rate for undrained shear strength testing. The equilization time for undrained shear strength testing should be somewhat less than that required for the complete dissipation process. However, a rigorous analysis of the undrained testing procedure is lacking at present. It would be necessary to model the response characteristics of the pore-water pressure measuring system. It is suggested that the time required for full dissipation in a drained test be used as a conservative estimate of the "time to failure" required for full equilization of induced pore pressures in a sample under drained conditions.

It is still difficult to accurately assess all soil properties required to estimate the strain rate for testing unsaturated soils. However, the proposed theory indicates the major factors affecting strain rate and assists in obtaining an approximate "time to failure".

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support received from Fugro (Hong Kong) Ltd., in the course of conducting the above research.

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