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the experimental results. In the case of specimens B300 and B400, although the curves are not fitted as good as those obtained for the pervious specimens, however, are still quite acceptable for engineering purposes. In Figure 3, the best-fit curves also are in a very good agreement with the experimental curves both in loading and unloading stages. This indicates that the criterion established for cohesive soils is also valid for non-cohesive fine-grained soils. Further investigations are required for stronger verification of this criterion however.

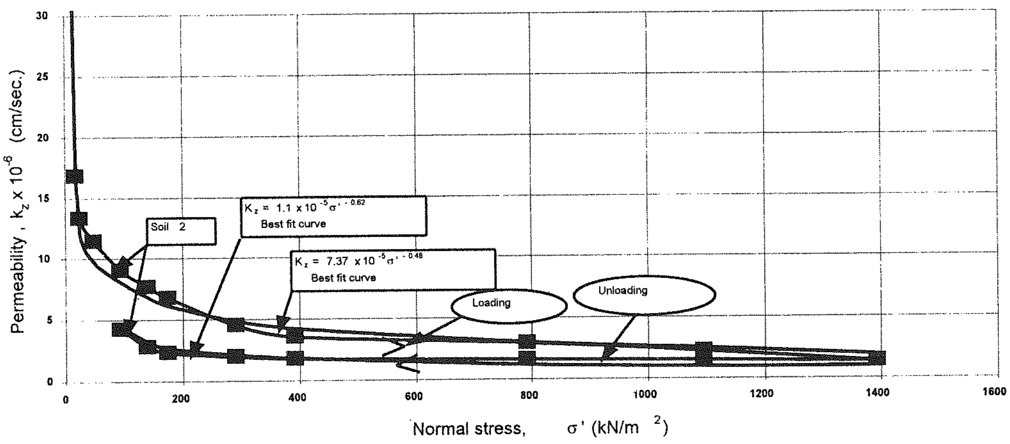

Figure (3) Stress dependent permeability of soil 2 and best fit curves.

4-Conclusions

Routine laboratory permeability measurements can be considered reliable only in cases where permeability is relatively insensitive to stress or where the stress dependence is known and can be corrected.

In the case of normally consolidated clay and non-cohesive fine-grained soils the coefficient of permeability is highly stress dependent. If the stress dependency of permeability is ignored in seepage and other effective stress analyses, considerable errors may be involved

Permeability decreases as the stress is increased and vice versa. However, a certain hysteresis is clearly seen. It is more convenient to replace "stress – dependent permeability" with "stress history – dependent permeability".

The equation $k = k_0(\sigma' / p_a)^{-b}$ is a useful criterion to introduce the stress dependent permeability characteristics of both cohesive and also fine grained non-cohesive soils in engineering practice. The b value needs some further investigations, however.

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Table (2) The physical properties of soils 1 and 2, and bentonite.

Soil No.	D_{max} mm	Passing 76 μ	w_l %	PI %	C_{cu}	C_u	γ_d kN/m^3	k_o cm/sec.	USCS class.
1	2	3	--	NP	0.8	4.0	16.0	3.6E-2	SP
2	4.7	11	--	NP	1.9	6.0	15.5	7.4E-5	SM
B	--	100	140	85	--	--	--	--	CH

This test was repeated for three times for increased accuracy. The initial and the average final characteristics of the specimens are summarized in Table 4. The test results, which are the average of three tests, are plotted in permeability - stress axes in Figure 3.

Table (3) Compositions of sand (soil 1) bentonite mixtures.

Specimen No.	B gr	S gr	w gr	γ_o gr/cm^3	e_o %	S_{ro}
B100	8.0	124.0	26.6	1.97	0.58	>99
B200	16.1	107.8	29.8	1.91	0.71	>99
B300	24.1	91.8	33.1	1.85	0.93	>99
B400	32.2	75.7	36.4	1.79	1.11	>99

Table (4) Initial and final characteristics of specimens prepared with soil 2.

Specimen condition	σ kN/m^2	γ_d kN/m^3	w %	e	S_r
Initial	0.0	15.5	16.6	0.67	64.5
Final	16.0	16.7	21.4	0.56	100.0

2-Theoretical Approach

In order to compare field measurements with theory, the ground water pressure profiles were established analytically solving the Equation 1. The stress dependency of permeability was considered substituting the k value from the Equation 7 (power law) in the basic Equation 1, and the final differential equation was obtained as follows:

$$dh_p / dz + C_1[(\gamma_{sat} / \gamma_w)z - h_p]^{Cc / Ck} = 0 \qquad (8)$$

in which h_p and C_l are piezometric head and a con-stant factor, respectively. This equation was solved numerically for C_c / C_k values ranging from 0.1 to 4.0, and plotted on Figure 1.

3-Discussion

In Figure 1 the non–linearity of the variation of ground water pressure with depth is clearly observed. It is also seen that the pore water pressure profiles are nearly tangent, to some depth below the ground surface, say z / L = 0.3, to the hydrostatic line and then plunge sharply toward the piezometric head of the under draining layer. This discloses that if the stress dependency of permeability is ignored in seepage and other effective stress analyses, considerable errors may be involved. In this figure it is also observed that the experimental piezometric profiles are of comparatively the same patterns as those obtained from theory. However, reminding that the ground is redeposited, and referring to Table 1, the C_c / C_k ratio must have been about 0.87, while refe-rring to Figure 1 is estimated to be 1.5 to 2.7. It may be inferred that while the $k=k_o(\sigma'/p_a)^{-b}$ is a useful criterion to define the stress dependent permeability characteristics of soils, the b value may be somewhat different from the C_c/C_k ratio.

In Figures 2 and 3 the best-fit curves to the data points obtained from the laboratory tests are also shown. The equations governing these curves all are of the general form as $k=k_o(\sigma ' / p_a)^{-b}$. For the specimens B100 and B200 the best fit curves are nearly coincided to permeability of fine and high plasticity soils can be determined with sufficient precision by using the consolidation test. The falling head test is more suitable for silts, fine sands, and silty sands.

To conduct laboratory tests two types of sandy soils and a bentonite powder, whose physical properties are summarized in Table 2 were used. Where w_l = liquid limit, PI = plasticity index, C_{cu} = curveness coefficient, C_u = coefficient of uniformity, γ_d = dry density, and k_o = initial coefficient of permeability. The following tests were then carried out on the two soils and on mixtures of bentonite with sand (soil 1):

The soil 1 was mixed with 100, 200 and 300 kg/m^3 bentonite and molded in consolidation (oedometer) ring molds of 75 mm in diameter. The bentonite powder was first mixed with water. After the hydration of bentonite had been completed, the gel was mixed with the soil. The specimens had all the same water - bentonite ratio $w / B = 1.28$.The compositions by weight of the specimens are summarized in Table 3. Where S = sand, γ_0 = initial density, e_0 = initial void ratio, and S_r = initial degree of saturation. The specimens were subjected to consolidation tests (Head 1984). The test results, in terms of permeability against stress level, are shown in Figure 2. In this figure the zero stress level permeability values of the specimens were measured using a falling head permeameter which had been calibrated for head loss due to evaporation effect (ASTM 1994).

In the other test three specimens were prepared with soil 2 and subjected to the same falling head permeability tests. The specimens were initially the same in moisture content and dry density, and in turn in void ratio e_0. To prepare these specimens, soil 2 was molded in the standard permeability mold of 101.6 mm diameter and 107 mm height. The permeameter system was coupled with a uniaxial loading frame. Having saturated the specimens, they were subjected to axial stress levels varying from 0 to 1600 kN/m^2, and the specimen settlements were monitored. At each stress level the permeability test started after the soil settlement had been ceased.

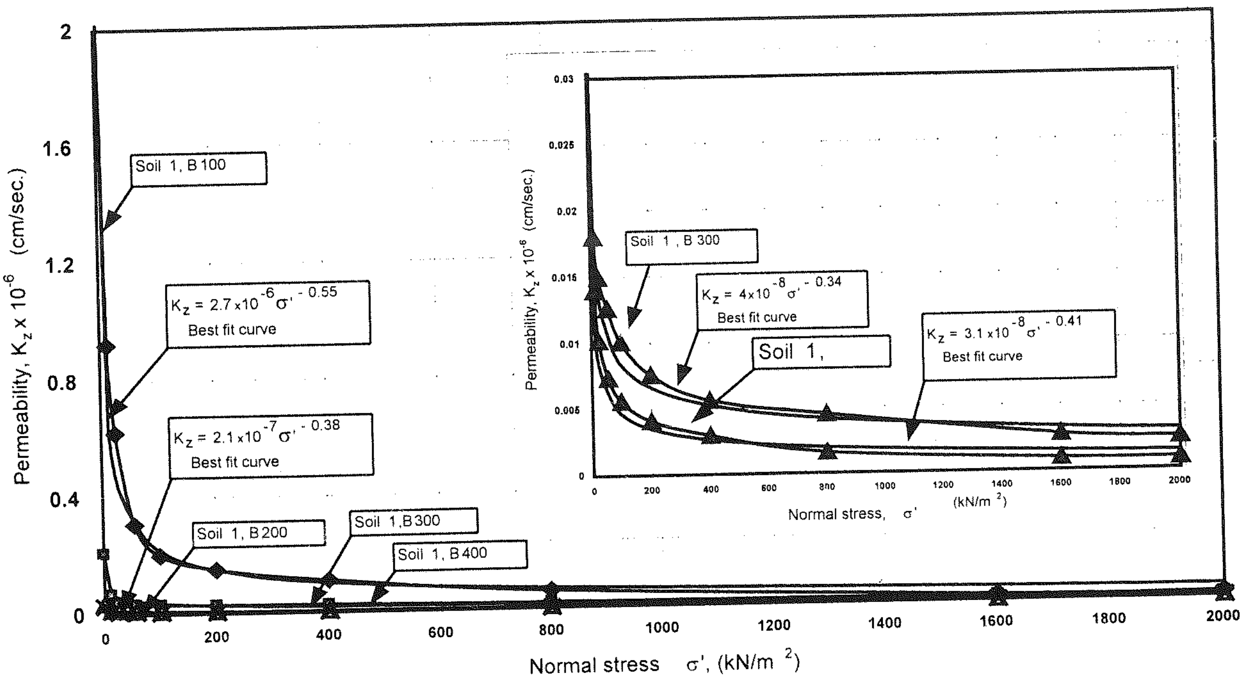

Figure (2) Stress dependent permeability of soil 1 and bentonite mixtures and best fit curves.

During a site investigation work on an area at the southern edge of London Clay it was observed that the ground water pressure pattern was governed by the layers of soil of high permeability values of the order of 10^{-4} cm / sec., compared with 10^{-8} to 10^{-10} cm / sec. for the high plasticity clay ground (Sadrekarimi 1988). The soil in this area is a redposited clay of high plasticity, and the average engineering properties are summarized in Table 1.

The presence within the clay ground of natural under-draining silty sand blankets, with nearly free drainage potential, encouraged the main author to evaluate the stress dependent permeability effects on downward ground water flow and pore water pressure buildup patterns. Accordingly, some standpipe and vibrating wire piezometers were prepared and installed at different depths below the ground level along several vertical profiles.

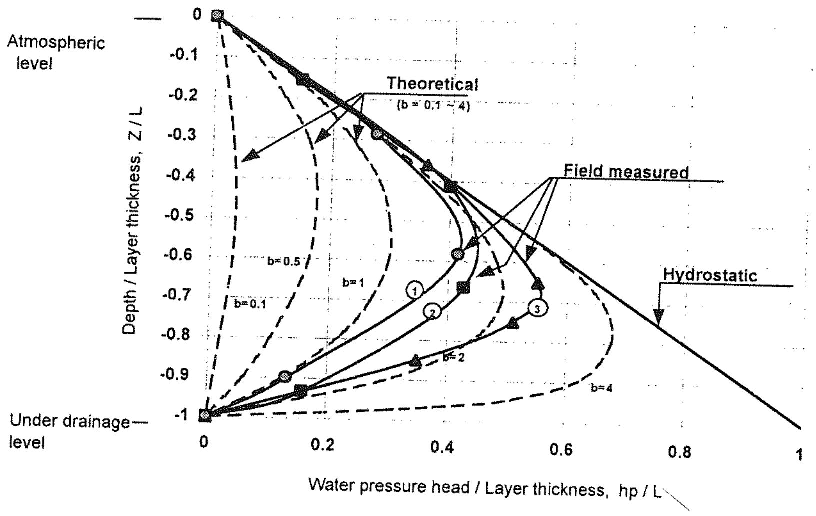

Figure (1) Ground watre pressure profiles, field measured and theoretical.

Table (1) Average engineering properties of the natural ground.

γ kN/m^3	w %	w_l %	PI %	C_c	C_s	e_0
18.9	32	82	50	0.37	0.14	0.85

After having reached the first time pressure equalization of piezometers (Hvorslev 1951) ground water pressure measurements were carried out for over one year. Standpipe piezometer equalization tests were also carried out to measure soil permeability (Premchitt & Brand 1981).

The pore water pressure profiles along 3 selected sections are shown in Figure 1. In this figure the non-linearity of the variation of ground water pressure with depth is clearly observed. Note that the dashed curves are obtained from theory and will be explained later in this paper.

1-2-Laboratory works

In order to evaluate the permeability – effective stress relations in cohesive and non-cohesive soils laboratory tests were carried out. In the current research the falling head test (Head 1984) with loading facility and standard consolidation test (ASTM 1994) were used as the direct and the indirect methods of permeability measurement, respectively.The

$$k = k_0(\sigma' / p_a)^{-Cc/Ck} \quad (7)$$

In the case of over-consolidated soil, where the stress level is less than the pre-consolidation pressure, the C_c is replaced by the swell index C_s. However, this equation indicates that compression and/or swell indexes govern the stress dependency behavior of clay soil. The pre-consolidation pressure is also a significant factor, which affects soil permeability. It has been proposed that the permeability of soft ground changes significantly after the soil yields (Chai & Bergado 1993, Tavenas & Leroueil 1980). This means that the pre-consolidation pressure is an inflection point in the stress dependent permeability characteristic of a soil.

Bromhead & Vaughan (1980) have postulated linear void ratio – mean effective stress σ' and void ratio – log permeability relationships and obtained the logarithmic law as ln $k/k_0 = -b\sigma'$; and with linear void ratio – log effective stress and void ratio – log permeability relationships obtained the power law as $k / k_0 = \sigma'^{-b}$, where b is a constant factor. It is important to note that the former of these two equations should be used cautiously, because, void ratio – mean effective stress relationship is generally non-linear (Verdugo & Ishihara 1996, Reimer & Seed 1997).

In the case of naturally fractured and fissured hard formations there are also ample evidences proving dominant effect of effective stress field changes over mass conductivity. Fractures and fissures are other factors affecting mass permeability. Fissuring contributes to increase the overall permeability (Nonveiller 1981). The permeability of fissured reservoirs has been postulated to be highly sensitive to changing effective stress (Hubert et al. 1993). There are numerous cases where a well starts producing at a high rate, but within a short time its production declines without a clear explanation (Da Part 1990). This can be attributed to the fact that the rapid drainage of the fracture and fissure system decreases pore fluid pressure which in turn increases effective stress, leading to close up of the fissure system. Walsh (1981) modeled fracture permeability versus effective stress based on asperity contacts between opposing surface, which was subsequently appraised by Hubert et al. (1993) using field parameter. In the case of over-consolidated fissured clay the fissures do not affect the insitu permeability of the soil. This view is supported by the available field evidence, which indicates that the rate of pore water pressure equilibration in cuttings, despite the fissured structure of the clay, is very slow (Skempton 1977). Holt (1989) found experimentally that the permeability reduction of sand stone during hydrostatic and non-hydrostatic loading below the yield level is near the $e^3 / (1 + e)$ behavior expected from the Carman-Kozney equation (Carman 1956). After yielding, however, the permeability decreases sharply.

While with the advent of, for example, the finite element method, problems involving anisotropy and non-homogeneity are well analyzed, the seepage problems associated with anisotropy and non-homogeneity caused by the stress field are far from clear, this is because the stress dependency of soil permeability is not well established.

In the current paper the stress dependency of permeability is reviewed. The results of a field investigation to establish pore water pressure profile and ground water flow pattern are presented; and the dominant effect of stress dependent permeability characteristics of soil is demonstrated and pointed out. Then the procedure and results of a laboratory experimental work to evaluate stress dependency of soil permeability is introduced. Finally, referring to the site and laboratory works the nature of stress dependent permeability of soil is discussed.

1-Experimental Works

1-1-Field works

are interrelated; the coefficient of permeability k can be expressed as a function of any two of the three aforementioned parameters e, w, and s_r (Leong & Rahardjo 1997). As a general rule, permeability increases with increase of degree of saturation, and when the soil is fully saturated the w or s_r effect vanishes. Accordingly, the void ratio e will be the unique dominant factor affecting permeability.

There are established correlations between void ratio and effective stress, and between void ratio and permeability, from which effective stress – permeability relationships can be deduced. According to Kozeny (1927) and Carman (1956) the coefficient of permeability is linearly related to $e^3/(1+e)$. The Kozeny – Carman equation works well only for describing coarse-grained soils. However, an examination of the coefficient of permeability of a uniform Madison sand against $e^3/(1+e)$, $e^2/(1+e)$ and e^2 reveals that the coefficient of permeability is linearly related to $e^3/(1+e)$, $e^2/(1+e)$ and e^2, and it appears that all three relations are equally good (Das 1987). Casagrande (1932) has also introduced an empirical relation for k for fine or medium clean sand with bulky grains as follows:

$$k = 1.4\, k_0\, e^2 \tag{2}$$

where k_0 is the coefficient of permeability at a void ratio of 0.85 (Das 1987). Taylor (1948) proposed a relationship between permeability and the void ratio of clays as follows:

$$k = k_0 \times 10^{[-(e_0 - e)/C_k]} \tag{3}$$

where k_0 and e_0 are initial permeability and void ratio, respectively; k is the permeability corresponding to the void ratio e at the condition considered, and C_k is a constant. Several natural clays were used to study the variation of permeability during consolidation through laboratory tests. It was suggested that Taylor's formula is valid, and the constant C_k can be estimated as one half of the initial void ratio (0.5 e_0) of the soil (Tavenas et al. 1983). Further investigations disclosed that the approximate relation $C_k = 0.5e_0$ is also valid for both vertical and horizontal permeability (Leroueil et al. 1990). Equations 2 and 3 can also be written in terms of effective stress. For cohesionless soils Li & Wang (1998) have shown that variations of void ratio e versus mean effective stress σ' in the $e - (\sigma'/p_a)^\alpha$ plane is linear, where p_a is the atmospheric pressure serving as a reference pressure, and α is a material parameter around 0.7 to 0.8. Accordingly, the e and σ' are correlated as:

$$e = e_0 - (\sigma'/p_a)^\alpha \tag{4}$$

where e_0 denotes void ratio corresponding to $\sigma' = 0$. The combination of Equations 2 and 4 yields to:

$$k = 1.4\, k_0[0.85 - (\sigma'/p_a)^\alpha]^2 \tag{5}$$

In the case of normally consolidated clays, void ratio e and the logarithm of effective stress σ' are related by the compression index C_c. From which:

$$e_0 - e = C_c \log \sigma'/\sigma'_0 \tag{6}$$

where e_0 is the void ratio corresponding to the initial effective stress σ'_0, and σ' is the effective stress at the condition considered (Terzaghi & Peck 1967). Assuming $\sigma'_0 = p_a$, and combining Equations 3 and 6 leads to:

Stress Dependent Permeability Effects on Ground Water Flow

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Abstract

There are several geotechnical and geoenvironmental problems involve water flow through soils. The coefficient of permeability is the most important parameter that dominates the water flow pattern through soils, both in transient and steady state conditions. In the current paper the stress dependency of permeability and its effect on the ground water flow is evaluated. The results of a field instrumentation program to establish pore water pressure profile is presented and the dominant effect of stress dependent permeability characteristics of soil over ground water pressure and flow is demonstrated and compared with an analytical solution. Also, the procedure and results of a laboratory experimental work to evaluate stress dependency of permeability is elaborated. Finally, referring to the site and laboratory investigations the nature of stress dependent permeability of soil and the validity of the related equations are assessed.

Keywords

Stress, Permeabifity, ground water.

Introduction

Although stress dependent permeability has been recognized and investigated by several geotechnical and petroleum engineers (Kilmer et al. 1987, Bromhead & Vaughan 1980), the assumption of constant permeability in the current engineering practice is still common. This is most probably because of the fact that the engineers are not well aware of the importance and extent of the stress effects on permeability and in turn on the seepage and ground water flow pattern. This concept may be demonstrated by the well-known one-dimensional equation of continuity as follows:

$$k_z \partial^2 h / \partial z^2 + \partial k_z / \partial z \,.\, \partial h / \partial z = 0 \qquad (1)$$

where k_z is the coefficient of permeability in z direction and h is the total head (Das 1987). Equation 1 describes the steady flow condition for a given point along the z axis in the soil mass, and proves that wherever k_z changes along the z axis the pore water pressure profile generally will be nonlinear and also permeability dependent. If k_z is constant, equation 1 will be simplified to $\partial^2 h / \partial z^2 = 0$, which indicates that the pore water pressure profile in z direction is linear and independent of permeability. The authors have observed many cases in which considerable errors and discrepancies between theory and practice occurred when the permeability was assumed constant (e.g. Figure 1).

In the case of normally consolidated soil (no stress history, no fissures and cementation) the coefficient of permeability of a soil with defined structure, grain shape and size distribution, and particles composition, not only vary with direction but depends on the degree of saturation and the voidratio. Since void ratio e, water content w and degree of saturation s_r