Research Article

Determination Permeability Coefficient from Piezocone

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The permeability coefficient of soil profile is one of the problems concerned by engineers, and the determination of permeability coefficient method mainly relies on the laboratory permeability test and field pumping test, but these tests are time-consuming and inefficient, and especially the permeability coefficient of soil under the condition of partial drainage was difficult to determine; in this paper, the modern digital CPTU technology was used. Dimensional permeability K_T was defined by using the dimensionless normalized cone tip resistance Q_r , friction factor F_r , and pore pressure ratio B_q , these parameters enable plots of B_q - Q_r , F_r - Q_r , B_q - F_r to be contoured K_T and hence for permeability coefficient. The relationship has been applied to Nanjing 4th Yangtze river bridge, and compared with laboratory penetration test. The results indicate that the method can accurately determine the permeability coefficient of soil under partial drainage condition and provide the theoretical basis for engineering application.

1. Introduction

With the development of foundation pit engineering to larger and deeper and the surrounding environment is increasingly complex, especially in the city downtown area, some excavations close to the high-rise buildings, some excavations close to the subway tunnel. At the same time, the groundwater treatment around the excavation cannot be avoided, and the environmental problems around the excavation caused by dewatering are becoming more and more serious. How to arrange dewatering to reduce the impact as much as possible is the key issue for many scholars and technicians. In order to reasonably arrange the dewatering well, we must grasp permeability characteristics of each soil accurately. The key parameter of permeability characteristics is the permeability coefficient. The permeability coefficient is the relative strength index of soil. It is a basic parameter and it must be used for seepage calculation; therefore, the accurate determination of soil permeability coefficient is a very important work, and it plays an important role in the success of the excavation. The current methods for determining the permeability coefficient are routine laboratory permeability test, field pumping test, water pressure test, and so forth. Routine laboratory permeability test is divided into constant

head permeability test and variable head permeability test. The laboratory permeability test is widely used, mainly because of its easy operation and simple equipment; however, it has some disadvantages: (1) soil disturbance, (2) inaccurate permeability test, especially sand inclusion or interbed, (3) not simulating field boundary conditions, and (4) the heavy workload and high cost. These drawbacks make much difference for the permeability coefficient between field test and laboratory test.

Piezocone penetration test (CPTU) is a time-saving, little disturbance, convenient and economic method for permeability coefficient of soil. The permeability coefficient of the soil was able to relatively accurately obtain, especially mixed-layer or thin interbed experts and scholars at home and abroad dedicated to the study method that can accurately determine the permeability coefficient.

2. Methods

Since the advent of multifunction, a lot of the relationships between permeability coefficient and multifunctional CPTU parameters have been proposed. The main methods of permeability coefficient of cohesive soil are divided into

FIGURE 1: Tip local conditions. Cone expansion stress is q_t .

two categories: (1) to estimate permeability coefficient based on the soil classification and (2) to estimate permeability coefficient based on the pore pressure dissipation test. The correct tip resistance q_t , pore pressure ratio B_q and sleeve friction f_s were modified to determine the type of soil [1–3] and (Olsen [4]). The permeability coefficient was estimated from soil classification [5]. And by the pore pressure ratio B_q [6] and by the side friction f_s to determinate the permeability coefficient was directly determined by using the cone tip resistance and side friction [7, 8].

View of the calculation of consolidation coefficient is less than $7.1 \times 10^{-5} \text{ m/s}^2$, the consolidation coefficient of the cohesive soil can be determined by the pore pressure dissipation in the completely undrained condition, and then the permeability coefficient can be determined by the consolidation coefficient according to the relationship between permeability coefficient and consolidation coefficient. However, the consolidation coefficient between $7.1 \times 10^{-5} \text{ m/s}^2$ and $1.4 \times 10^{-2} \text{ m/s}^2$ was taken for the part of drainage condition, so the pressure dissipation was not used. The partial drainage state boundary range $B_q Q_t < 1.2$, $Q_t F_r < 0.3$, $B_q / F_r < 4$ was proposed. Based on the partial drainage condition given by Elsworth [9], the permeability coefficient was put forward to determine by using CPTU data.

2.1. Cavity Expansion Theory. For the saturated clay under undrained conditions, cylindrical cavity expansion stress was proposed by using [10]

$$\sigma_r = \sigma_{v0} + \frac{4}{3} S_u \left[1 + \ln \left(\frac{G}{S_u} \right) \right], \quad (1)$$

where G is shear modulus, $G = E/2(1 + \mu)$, μ is Poisson's ratio, E is elastic modulus, S_u is undrained shear strength, kPa, and σ_{v0} is overburden stress, kPa.

Under undrained conditions, the excess pore pressure caused by cavity expansion can be calculated using the average total stress increment $\Delta\sigma_m$

$$\Delta u = u_2 - u_0 = \Delta\sigma_m = \frac{1}{3} (\Delta\sigma_r + 2\Delta\sigma_\theta) = \frac{4}{3} S_u \ln \left(\frac{G}{S_u} \right), \quad (2)$$

where u_2 is cone shoulder pressure, u_0 is hydrostatic pressure, and q_t is the correct cone tip resistance.

In the CPTU penetration process, the horizontal stress is assumed to be equal to the cavity expansion stress. For the cohesionless soil $c = 0$, the normalized friction ratio F_r and pore pressure were established relationships, and noticed the sleeve friction, $f_s = \mu \sigma'_h$, $\sigma'_h = \sigma_h - u_2$, c , ϕ were the soil strength parameters. The friction factor is assumed to be $\mu = \tan \phi$ as shown in Figure 1

$$f_s = (\sigma_h - u_2) \times \tan \phi. \quad (3)$$

2.2. Dimensionless Parameters. CPTU sounding yields profiles of the correct tip cone resistance, q_t , cone shoulder pore pressures, u_2 , and sleeve friction, f_s , with depth. These dimensional metrics may be defined as normalized magnitudes of tip resistance, Q_t , pore pressure ratio, B_q , and friction ratio, F_r , as

$$\begin{aligned} Q_t &= \frac{q_t - \sigma_{v0}}{\sigma'_{v0}}, \\ B_q &= \frac{u_2 - u_0}{q_t - \sigma_{v0}}, \\ F_r &= \frac{f_s}{q_t - \sigma_{v0}}, \end{aligned} \quad (4)$$

where σ'_{v0} is the effective overburden stress, kPa, u_0 is hydrostatic pressure, kPa, and u_2 is cone shoulder pressure, kPa.

If the adhesion of the cone sleeve is assumed to be equal to the undrained shear strength, $f_s = S_u$, then the magnitudes of nondimensional cone metrics of end-bearing, sleeve friction, and pore pressure ratio may be determined by substituting (1) and (2) into (4) to yield

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} = \frac{\ln(G/S_u)}{1 + \ln(G/S_u)} \quad (5)$$

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} = \frac{4S_u}{3\sigma'_{v0}} \left[1 + \ln \left(\frac{G}{S_u} \right) \right] \quad (6)$$

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} = \tan \phi \left(1 + \frac{1}{Q_t} - B_q \right) \quad (7)$$

$$B_q Q_t = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \cdot \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} = \frac{4S_u}{3\sigma'_{v0}} \ln \left(\frac{G}{S_u} \right). \quad (8)$$

FIGURE 2: Geometry of process zone surrounding advancing penetrometer.

FIGURE 3: The soil profile of north anchorage of the Yangtze river 4th bridge ①₀ = Clay, ①₁ = silty clay, ①₂ = silt, ②₃ = silty clay with fine sand interbed, ②₃ = bedrock.

FIGURE 4: Layout diagram of CPTu holes in Yangtze River four bridge north anchorage.

FIGURE 5: Results of CPTu.

It is assumed that the penetration of a standard cone has a stable speed (i.e., 20 mm/s). According to the fluid continuity theorem and the Darcy law, the water volume is equal to the cone penetration volume in unit time, as shown in (9) and u_r is the hydrostatic pressure where $r = r_h$ it is assumed that the permeability coefficient has a small variation in the penetration process. At the same time, when $r_h \rightarrow \infty$ and $r_0/r_h \rightarrow 0$, the pore pressure is hydrostatic pressure u_0 , namely, (10) calculation diagram as shown in Figure 2. Consider

$$u_2 - u_r = \frac{\gamma_w}{4\pi K_h r_0} \left(1 - \frac{r_0}{r_h}\right) dV = \frac{U r_0 \gamma_w}{4K_h} \left(1 - \frac{r_0}{r_h}\right) \quad (9)$$

$$u_2 - u_0 = \frac{\gamma_w}{4\pi K_h r_0} dV = \frac{U r_0 \gamma_w}{4K_h} \quad (10)$$

Take into consideration

$$B_q Q_t = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \cdot \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} = \frac{u_2 - u_0}{\sigma'_{v0}}, \quad (11)$$

and assumption that

$$B_q Q_t = \frac{1}{K_T} \quad (12)$$

Obtain

$$K_h = \frac{U r_0 \gamma_w \cdot K_T}{4 \cdot \sigma'_{v0}}, \quad (13)$$

where U is penetration rate of m/s; r_0 is cone radius m; K_T is the dimensionless permeability index; K_h is permeability coefficient, m/s.

FIGURE 6: Contoured plots of B_q - Q_t , F_r - Q_t , and B_q - F_r .

Substitute $B_q = 1/Q_t K_T$ into (7)

$$K_T = \frac{1}{B_q} \left(\frac{F_r}{\tan \phi} - 1 + B_q \right), \quad (14)$$

$$K_T = \frac{1}{Q_t (1 + 1/Q_t - F_r / \tan \phi)}, \quad (15)$$

where B_q is the pore pressure ratio, Q_t is the normalized tip resistance, F_r is resistance ratio, K_T is the dimensionless permeability index, and K_h is the permeability coefficient, m/s.

Equation (14) showed that the friction ratio F_r and pore pressure ratio B_q can calculate the dimensionless permeability index K_T . Equation (15) shows that K_T can be calculated by the frictional ratio F_r and the normalized tip resistance Q_t . According to $k_h = Ur_0 \gamma_w \cdot K_T / 4 \cdot \sigma'_{v0}$, the soil permeability coefficient can be calculated in partially drained conditions by using the obtained K_T values.

3. Engineering Examples

3.1. Engineering Overview. South and North Anchorage of the 4th Nanjing Yangtze River Bridge are located in the Yangtze River levee on both sides of south and North, belonging to the lower reaches of the Yangtze River alluvial floodplain. The site of north anchor of the 4th Nanjing Yangtze River Bridge is located in Nanjing fine mag Technology Co. Ltd. factory area, the ground elevation is about 5.5~6.1 m. The northern boundary of south anchor is apart from dyke and river more than 60 meters and 200 meters respectively. The north anchorage ground elevation is about 4.5~5.2 m, the site and the surrounding for forest, the southern boundary of north anchor is apart from the dyke and river about 120 m and 120 m respectively. The Quaternary loose sediment thickness is more than 60 m in north anchor.

3.2. Results of CPTU Test. From June 17, 2007, to June 23, 2007, the CPTU tests (six holes) were carried out in north anchor of Nanjing 4th Yangtze River Bridge. The soil profile of north anchorage of the Yangtze river 4th bridge is shown in Figure 3.

TABLE 1: Estimated soil permeability (k) based on normalized CPT soil behavior type (SBTn) by Robertson [2] (modified from Lunne et al., [11]).

SBTn zone	SBTn	Range of k (m/s)	I_c
1	Sensitive fine grained	3×10^{-10} to 3×10^{-8}	NA
2	Organic soils clay	1×10^{-10} to 1×10^{-8}	$I_c > 3.60$
3	Clay	1×10^{-9} to 1×10^{-9}	$2.95 < I_c < 3.60$
4	Silt mixture	3×10^{-9} to 1×10^{-7}	$2.60 < I_c < 2.95$
5	Sand mixture	1×10^{-7} to 1×10^{-5}	$2.05 < I_c < 2.60$
6	Sand	1×10^{-5} to 1×10^{-3}	$1.31 < I_c < 2.05$
7	Dense sand gravelly sand	1×10^{-3} to 1	$I_c < 1.31$
8	Very dense/stiff soil*	1×10^{-8} to 3×10^{-5}	NA
9	Very stiff fine-grained soil*	1×10^{-9} to 3×10^{-7}	NA

*Overconsolidated and/or cemented.

FIGURE 7: Curve of permeability coefficient changing with depth.

The hole position is shown in Figure 4. The CPTU test results were as shown in Figure 5. From Figure 6, it can be seen that the measuring points of the sand layer are located in the range of $0 < B_q Q_t < 0.2$, $Q_t F_r < 0.2 \sim 0.7$, $B_q / F_r < 4$, $F_r < 0.2$, which belongs to the section of the partial drainage conditions, so it can calculate the soil permeability coefficient according to (12)~(15).

Based on laboratory tests, the internal friction angle of slit is 30.9° , and the internal friction angle of fine sand is 32.5° and the internal friction angle of silty soil is 31° . The permeability coefficient related to the depth can be calculated by the equation (13). Figure 7 shows the comparison between the calculated results and the laboratory test ones, shown in

Figure 7. As seen from Figure 6, the permeability coefficients from the laboratory test and the CPTU test were consistent and the trend is basically same. The permeability coefficient using the presented method to calculate was in accordance with the scope of permeability coefficient in Table 1 [2].

4. Conclusion

The dimensionless penetration index K_T was derived through the normalized tip resistance Q_t , frictional ratio F_r , and the pore pressure ratio B_q relationship in partial drainage conditions; based on K_T , the permeability coefficient can be calculated. Comparing the results of laboratory test and the CPTU test, we can see the permeability coefficient in the range of 10^{-4} m/s– 10^{-7} m/s from the test results of soil under partially drained conditions and more than 10^{-4} m/s in complete drainage conditions and less than 10^{-7} m/s in undrained condition the permeability coefficient can be calculated through the pore pressure dissipation test.

The engineering application shows that the permeability coefficients obtained from the pumping test and CPTU test are identical. This method can provide the reference for practical foundation pit dewatering.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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