Pore pressure from penetrometer and hydraulic conductivity

Pression de pore à parlir de penetrometre et de la conductivité hydrolique

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ABSTRACT: This study analyzes the mechanism of the measured pore water pressure response of the cone penetration test during penetration and correlates the measured pore water pressure to the hydraulic conductivity of soils. The test results are compared and verified with the existing field test data and numerical results which are based on the coupled theory of mixtures with a modified Cam Clay model. From this study it is found that a hydraulic conductivity is obtained from the back analysis of excess pore pressure of Piezocone penetration test.

RÉSUMÉ: Cet étude analyse le mechanism des mesures de la pression de pore obtene par un penetrometre et sa correlation avec la conductivité hydrolique du sol. Les resultats de ces experiences ont éte compare et verifier contre des resultats de chantier et des resultats numeriques basés sur la theorie de milanges et le CAM model modifié des argiles. Il a éte conclu que la conductivité hydrolique peu être calcule á partir de analyse de la pression de pore.

1 INTRODUCTION

Traditionally, the estimation of the hydraulic properties (coefficient of consolidation or hydraulic conductivity) from the PCPT(Piezocone Penetration Test) is made from the pore pressure dissipation test data which are obtained by arresting the Piezocone (Torstensson 1977). However, this method requires substantial time for dissipation test, and is not possible to obtain the continuous hydraulic conductivity profile. Typically, it takes several hours for the dissipation test at one point. Without the dissipation test, the whole procedure for one PCPT takes about 1.5 to 2 hours for a 30 m penetration, and as a result only a limited number of dissipation tests can be carried out in a day.

A method is proposed here for determining the hydraulic conductivity of soils utilizing the coupled theory of mixtures without requiring arrest of the cone tip. This procedure also provides the possibility of a real time continuous hydraulic conductivity profile from PCPT.

2 BACKGROUND

2.1 *Essentials of pore pressure response*

Traditional methods for estimating the hydraulic conductivity of soils from piezocone penetration tests are based on pore pressure dissipation test data (Torstensson 1977). Essentially, the pore pressure response from the piezocone penetration test should follow the curves shown in Figure 1. This figure shows a conceptual and a slightly exaggerated excess pore pressure response of the soil element which is located at the projected center line of the piezocone penetration route. Initially, the piezocone tip is located far above this soil element, and there is no excess pore pressure. As time progresses, the penetrating cone tip comes closer to the soil element, and the induced stress of the penetrating cone tip gradually builds up the stress over this soil element. This will result in the excess pore water pressure (which may be a linear or non-linear increase). As the penetrating cone tip comes closer to this soil element, severe disturbance occurs and high excess pore pressure occurs. When the penetrating cone tip stops at this soil element, there will be an immediate drop in the excess pore pressure due to the reduced axial force. In the mean time, the interaction of the pore pressure between the near field and far field takes place (near field: location radially close to the cone tip; far field: location radially far from the cone tip), and results in the small increase or decrease in the pore pressure. As previously known, the pore pressure during penetration is at a maximum at the cone face (typically known as the u_1 position). Thus, if one has the porous element at the cone shoulder location (typically known as the u_2 position), a small increase of pore pressure is expected because of the pore water inflow from the u_1 position (high pore pressure position). This explanation is for the normally consolidated soils. The pore pressure response of an overconsolidated soil is beyond the scope of this paper.

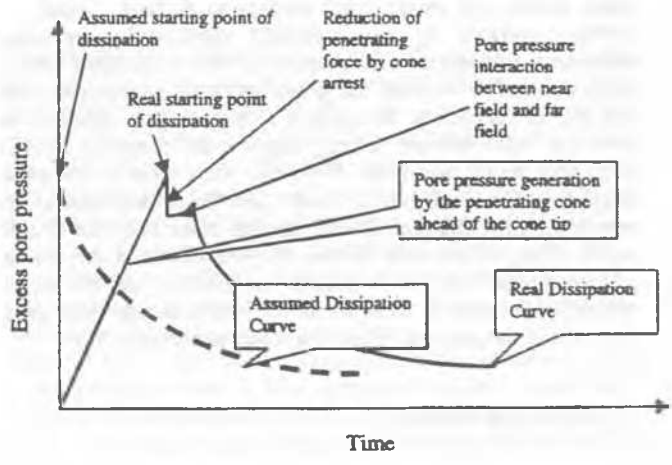

Figure 1. Schematics of Excess Pore Pressure Generation During Piezocone Penetration Test

From above discussions, it is found that the dissipation test is usually analyzed with an incorrect initial time and magnitude of initial pore pressure. As the drainage condition deviates more and more from the assumption of fully undrained condition, the reliability of assumed dissipation curve is less and less. Senneset

et al. (1988) pointed out this aspect by questioning the validity of the magnitude of the initial pore pressure when B_q is less than 0.4, where, B_q is the ratio of excess pore pressure to net cone resistance. The value $B_q = 0.4$ corresponds roughly for clayey silt. A small value of B_q implies a higher hydraulic conductivity. For these soils, one can expect a substantial drop of Δu_o due to the pore water pressure dissipation during penetration and pore water pressure interaction between the far field and the near field. Thus, the validity of Δu_o is questionable for soils which have a higher hydraulic conductivity. In other words, for relatively higher permeable material, the penetration process of PCPT is a partially drained condition, and not a fully undrained condition. This may cause a substantial difference between the peaks of the two curves in Figure 1.

Elsworth (1993) showed that B_q varies with the coefficient of consolidation when B_q is less than 0.5. This phenomenon may also be due to the fact that for higher permeable soils (B_q<0.5), the validity of initial excess pore pressure is inaccurate since in reality the drainage condition is not that of a fully undrained condition. The results of Senneset et al. (1988) and Elsworth (1993) showed that the conventional method for determination of the hydraulic conductivity is desirable when the hydraulic conductivity is low enough and the interaction of pore pressure between the far field and near field is minimized.

Another aspect of the piezocone induced excess pore water pressure regime is that the measured penetration pore pressure is the result of simultaneous generation and dissipation. Figure 1 shows that pore pressures are generated around an advancing piezometer tip. Thus, the measured pore pressures at the piezometer tip are the sum of the early generated and dissipated pore pressures and newly generated pore pressures. This leaves some doubt as to the magnitude of the initial pore pressure (Kurup and Tumay, 1997). The error induced from the above difficulties may be negligible or may be significant depending on the soil condition. Therefore, an improved method for estimating the hydraulic conductivity from piezocone data is needed.

2.2 *New approaches*

New approaches to estimate the hydraulic conductivity from PCPT induced steady state excess pore pressure were proposed by Elsworth (1993, 1998), and Manassero (1994). Elsworth (1993, 1998) estimated the hydraulic properties of soils with a linear elastic soil model and dislocation method. Manassero (1994) correlated B_q and hydraulic conductivity by semi-empiricism. Manassero's correlation (1994) is obtained from a linear correlation between the given hydraulic conductivity data and the B_q coefficient. Manassero's method may therefore be used only when one has existing data for the hydraulic conductivity and the B_q parameter. From the view point of computational cost, the linear elastic model and the semi-empirical approach are quite efficient. Due to the non linear behavior of soils at the vicinity of the cone tip and the poor data available for the relationship between the hydraulic conductivity and the B_q parameter, one needs to incorporate the elasto-plastic large strain approach in the analysis of the piezocone penetration test.

3 PROPOSED METHOD

The method proposed here is based on the analysis of the steady state pore pressure during the piezocone penetration test, so that the full interaction between the piezocone and the soil is considered. As a result, the problems related to the initial time and initial magnitude of the pore pressure discussed in the previous section are naturally eliminated. The formulation of the coupled field equations for soils, using the theory of mixtures in an updated Lagrangian reference frame, based on the principle of virtual work, is used in this work. An axi-symmetric finite element program that is capable of simulating the behavior of soils with the advancement of the piezocone tips is developed and used in this work. Finally, the results of the proposed method are compared with well-documented field test data and results obtained using the LSU calibration chamber system.

The piezocone penetrometer typically penetrates into the ground with the speed of 2 cm/sec, which induces complete failure of soils around the cone tip. Researches have shown that the strain at the vicinity of the cone tip ranges between ten percent to more than a hundred percent (Levadoux and Baligh 1986; Kiousis et al. 1986; Voyiadjis and Abu-Farsakh 1997). Thus the penetration of the piezocone is essentially a time dependent large strain problem. Considering the complexity of the piezocone penetration test, both large strain theory and visco-plasticity are desirable features for the analysis of the problem. However, in this study the isotropic, elasto-plastic large strain approach with time dependent loading condition is adopted for simplicity. Also, the modified Cam clay model is used in this study to describe the plastic behavior of soils.

3.1 *Theory of mixtures*

The drainage condition around the penetrating cone tip is neither the fully drained nor the fully undrained. This condition is called the partially drained condition or the transient flow condition. For the transient flow condition it can be presumed that the pore pressure is a function of the hydraulic conductivity and other parameters such as material stiffness. Voyiadjis and Abu-Farsakh (1997) implemented the coupled theory of soil-water mixtures to the effective stress and pore water pressure, and derived the coupled equations of mixture in an updated Lagrangian reference frame. Here, similar equations are derived in a simpler manner. From the principle of virtual work in an updated Lagrangian reference frame (Bathe 1996, Voyiadjis and Abu-Farsakh 1997), Equation (1) is obtained:

$$\begin{aligned}
&\int_{{}^nV} D^*_{ABCD}\,(\Delta_n e_{CD} + \Delta_n \eta_{CD})\,\delta_n e_{AB}\, d^nV \\
&+ \int_{{}^nV} D^*_{ABCD}\,\Delta_n e_{CD}\,\delta_n \eta_{AB}\, d^nV \\
&+ \int_{{}^nV} ({}^n\sigma_{AB}{}^s + {}^nP_w\,\delta_{AB})\,\delta_n \eta_{AB}\, d^nV \\
&+ \int_{{}^nV} J^s X^s_{A,a} X_{B,b}\,\delta_{ab}\,\Delta P_w\,(\delta_n e_{AB} + \delta_n \eta_{AB})\, d^nV \\
&= {}^{n+1}R - \int_{{}^nV} ({}^n\sigma_{AB}{}^s + {}^nP_w\,\delta_{AB})\,\delta_n e_{AB}\, d^nV
\end{aligned} \tag{1}$$

where, ${}^nV$ is the volume of the element at the nth configuration, δ is increment, ${}^{n+1}R$ is the external force at the $(n+1)^{th}$ configuration, ${}^n\sigma_{AB}$ is the Cauchy stress, ${}^ne_{AB}$ is the linear strain at the nth configuration that is expressed ½ $({}^nu_{A,B} + {}^nu_{B,A})$, and ${}^n\eta_{AB}$ is the non-linear strain at the nth configuration that is given by ½ $({}^nu_{K,B} - {}^nu_{K,A})$, D^*_{ABCD} is the modified elasto plastic stiffness, $X^s_{A,a} = \partial\, {}^{n+1}X_A / \partial\, {}^nX_a{}^s$, P_w is the pore water pressure, D_{abcd} is the elasto-plastic modulus, and J^s is the Jacobian.

Equation (1) is the equation of equilibrium of the external and internal forces in an updated Lagrangian reference frame expressed in terms of the effective stress and pore water pressure. So far, the relationship between the pore water pressure and the hydraulic conductivity is not shown. This relationship can be derived from Prevost (1980) and Voyiadjis and Abu-Farsakh (1997) such that:

$$\begin{aligned}
&J^s C^s_{ij}\,\varepsilon_{ij} - J^s C^{s\,-1}_{ij}\, C^{s\,-1}_{ij}\; X_{D,a}{}^s\;(\partial/\partial X_D) \times [(n_w/\rho_w)\, K_{AB}{}^{ws}\, X_{a,A}{}^s \\
&\qquad (\partial P_w/\partial X_B - \rho_w B_B)] = 0
\end{aligned} \tag{2}$$

In Equation (2), C^s_{ij} is $X_{K,I}{}^S X_{K,J}{}^S$, ε_{ij} is the strain rate tensor, $X_{a,A}{}^s$ is $\partial\, {}^{n+1}X_a / \partial\, {}^nX_A{}^s$, B_B is $b_b/X^s_{b,B}$, b is the body force vector, J^s is the Jacobian, p^s is the mass density of the soil, ρ^w is the mass density of the water, K^{ws} is the hydraulic conductivity tensor, ρ_w is the density of water, v^w is the water velocity, and P_w is the pore water pressure. Using Equations (1) and (2), the coupling of the stress, deformation, pore water pressure, and hydrau-

lic conductivity is obtained. Thus, by solving Equation (1) and (2), one can compute the hydraulic conductivity of the soil.

3.2 *Numerical simulation*

The numerical simulation is conducted using an axisymmetric finite element analysis with the mesh shown in Figure 2. The piezocone penetrometer is assumed to be infinitely

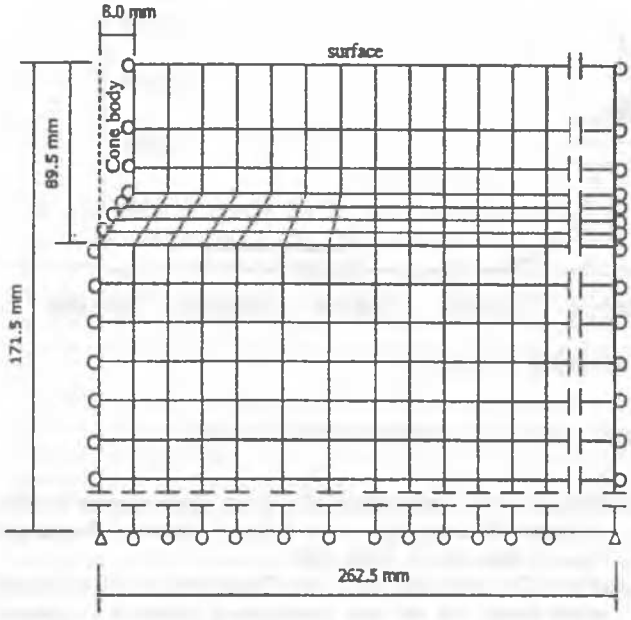

Figure 2. Mesh for Finite Element Analysis

stiff and tensile stresses are not allowed to develop along the centerline boundaries.

The continuous penetration of the piezocone is simulated by applying an incremental vertical penetration rate of the cone 2 cm/sec. This is the same rate as the piezocone penetration, and it allows the partial pore pressure dissipation during the penetration. The input parameters used in this study are obtained from typical oedometer tests and triaxial compression, and shown in Table 1. As in a typical modified Cam Clay model, no fudge parameters are involved in the simulation.

Table 1. Materials Properties

Parameter	Value	Units
Compression index, λ	0.11	dimensionless
Recompression index, κ	0.024	dimensionless
Initial void ratio, e_o	1.0	dimensionless
Poisson's ratio, ν	0.3	dimensionless
Slope of critical line, M	1.16	dimensionless
Unit Wt. of soil, γ_t	1.8	ton/m^3
Depth (from ground surface)	20	m
Unit Wt. of water, γ_w	1.0	ton/m^3

3.3 *Hydraulic conductivity estimation procedure*

With the assumed hydraulic conductivity matrix, the excess pore pressure at the piezometric-element location can be computed. This computed value is compared to the measured value. If these two values are within 10 % of each other, the assumed hydraulic conductivity is considered a good estimation of the hydraulic conductivity of the soil. This general trial and error method is time consuming, and therefore a good initial hydraulic conductivity matrix must be chosen.

3.4 *Comparison with Test Results*

Three piezocone penetration tests were also conducted at the LSU calibration chamber for K-50 (mixture of 50% kaolinite and 50% sand) and K-33 specimen (mixture of 33% kaolinite and 67% sand) for evaluation of this approach.

Three penetration tests were carried out in the soil specimens. The hydraulic system used for the cone penetration consists of dual pistons, double acting hydraulic jacks on a collapsible frame. The frame is mounted on top of the upper lid of the chamber and allows for penetration of the sample in a single stroke of 640 mm or less. Such a single stroke continuous penetration is desirable especially in saturated cohesive specimens where stress relaxation and pore pressure dissipation can occur during a pause between strokes. The penetration depth is measured using an electronic analog to digital converter depth decoding system. All tests are conducted at the standard penetration rate of 2 cm/sec. A total of three penetration tests are performed. Tests 1 and 2 are performed for the two different piezo-element configurations, the u_1, u_3, u_4 configuration (See Figure 3 for piezo-element configuration) and the u_2, u_3, u_4 configuration, respectively. Test 3 has the same configuration as Test 2. The main purpose of Test 3 is to check the repeatability of the tests. These three calibration chamber test results were averaged and shown in Figure 4.

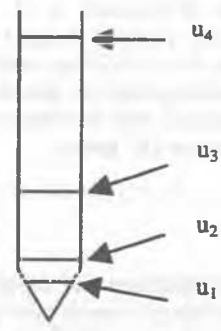

Figure 3. Location of Piezo-element tips

Test results were collected from well-documented piezocone penetration tests. Test data from the literature were normalized to an undrained shear strength 60 kPa, the shear strength of the specimen tested in the LSU calibration chamber in this study. This normalization of the field data is based on the fact that the induced excess pore pressure is proportional to the undrained shear strength from the Cavity Expansion Theory (Vesic, 1972). Through this normalization, the dependency of pore water pressure to strength or stiffness of the material is expected to be reduced. Experimental results are plotted in Figure 4 against the theoretical results as discussed previously.

Figure 4 shows reasonably good agreement between the test data and predicted results. The agreement is good considering that these data are from different countries for different soils of different properties such as stiffness etc. As discussed earlier, the test data covers a wide range of soils with hydraulic conductivities ranging from 10^{-9} m/sec to 10^{-6} m/sec. One can see from Figure 4 that the change in the excess pore pressure is very small when the hydraulic conductivity is less than 10^{-9} m/sec or larger than 10^{-6} m/sec. Therefore, the drainage condition is practically that of a fully undrained condition for the hydraulic conductivity of lower than 10^{-6} m/sec, and that of a fully drained condition for the hydraulic conductivity of higher than 10^{-6} m/sec. A simpler approach such as drained cavity expansion or undrained cavity

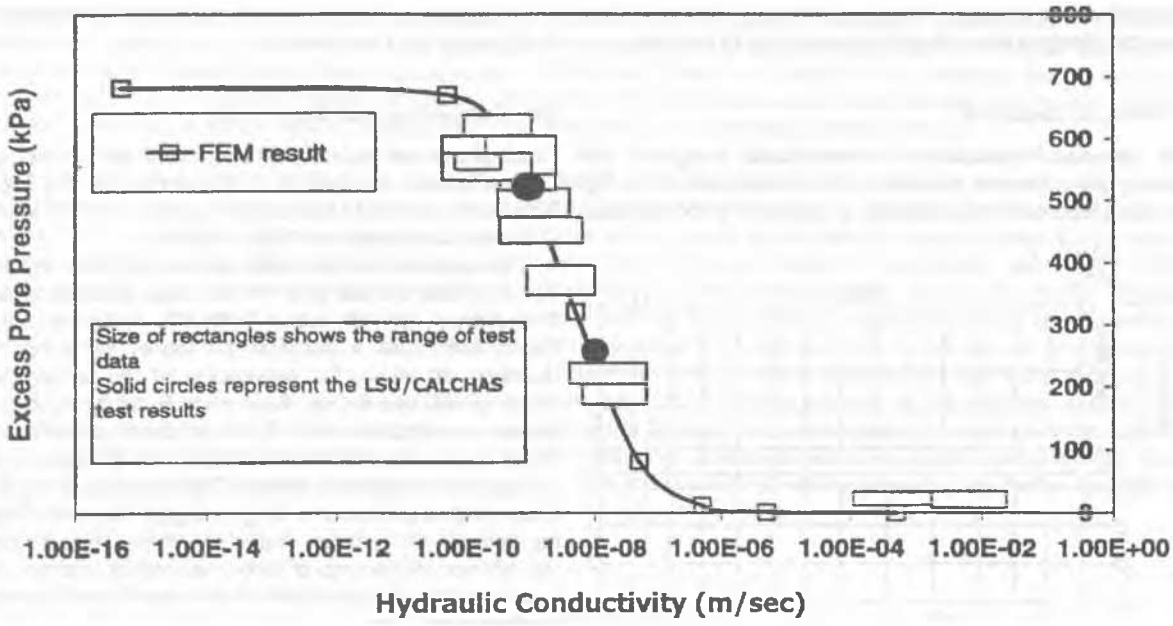

Figure 4. Relationship between Hydraulic Conductivity and Excess Pore Pressure

expansion may be applicable. However, for soils in between these boundaries, the partial drainage effect is not negligible, so such simpler approach may not provide reliable results. One may recall that the work of Senneset et al. (1988) and Elsworth (1993) showed that the partially drained condition is obtained when B_q is less than 0.4 or 0.5.The work presented here gives similar results by showing that the excess pore pressure during the piezocone penetration test is affected when the hydraulic conductivity is lower than 10^{-9} m/sec.

4 CONCLUSIONS

A new theoretical interpretation and experimental verification of the cone penetration induced excess pore pressures is presented. The large strain coupled theory of mixtures formulation using an updated Lagrangian reference frame is adopted in this work. Using this theory and the presented numerical simulation technique, the cone penetration induced excess pore pressure is predicted and compared with existing test data.

From this study, the following conclusions may be made:

The test data agree well with the theoretically predicted results. Therefore, the potential exists for use of this method to interpret of the continuous pore pressure measurements, and it may be possible to use this approach for the real time analysis for the hydraulic conductivity of saturated soil.

Two threshold hydraulic conductivities are obtained as 10^{-9} m/sec and 10^{-6} m/sec for the undrained condition and the free drainage condition, respectively. The coupled theory of mixtures should be used to predict the behavior of soils within the range of these threshold values.

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