A numerical study on the applicability of the piezocone interpretation methods for hydraulic conductivity of clays

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1 INTRODUCTION

Despite all the available laboratory tests and field measurement techniques, piezocone (CPTu) penetration and dissipation tests have found singular popularity amongst the geotechnical engineers to measure soil hydraulic conductivity parameters. These techniques are fast, cheap and less complicated compared to other existing laboratory and field tests. However, the CPTu test relies heavily on the particular interpretation method adopted to estimate hydraulic conductivity parameters, i.e. the coefficient of consolidation or the permeability.

The interpretation of dissipation data is sensitive to a number of criteria, e.g. testing equipment, testing methods and subsurface conditions. Figure 1 shows a CPTu probe and the most common positions of porous/filter elements for the measurement of pore pressures.

Available CPTu interpretation methods are mainly based on the empirical, semi-empirical, and analytical approaches. The numerical methods are rarely employed to propose or validate these interpretation methods or they are based on numerous simplifying assumptions which in turn create uncertainties.

A major concern associated with the currently available interpretation methods is their reliability. Insufficient records of dissipation test data together with the experimentally-measured soil hydraulic conductivity parameters exist to facilitate a meaningful comparison. Changes in the boundary conditions governing each case study, coupled with inadequate theoretical and numerical models to simulate the piezocone penetration and dissipation processes, also complicates the assessment of the interpretation methods using numerical techniques.

In this paper, the two most commonly applied piezocone interpretation methods for estimating permeability parameters are investigated. A rigorous numerical model is used to simulate both piezocone penetration and dissipation steps. A coupled pore-fluid pressure analysis is carried out taking into account large deformation and finite strain contact kinematics at the interfaces of the cone and soil. Realistic initial boundary conditions are imposed to the numerical model to ensure its robustness. The numerical model is verified through comparison with other numerical models. The verified numerical model is then used to evaluate the reliability of the most commonly used interpretation methods. The focus of this research is primarily on the data interpretation step. The dissipation data interpretation methods of Teh & Houlsby (1991) and Sully & Campanella (1994) and their corresponding theoretical frameworks are discussed below.
2 DATA INTERPRETATION METHODS

2.1 Typical Dissipation Curve Method (Teh & Houlsby, 1991)

Analogous to Tetzlaff's one-dimensional theory of consolidation, attempts have been made to obtain typical dissipation curves by normalizing the dissipation data via normalized parameters. It is common practice to plot 'Normalised Excess Pore Pressure' against proposed modified time factors. The excess pore pressure is normalised by the following formula known as 'Degree of Dissipation' or 'Normalised EPP':

\[ U_{50} = \frac{u - u_0}{\Delta u} \quad (1) \]

where \( u_0 \) is the initial hydrostatic pore pressure; \( u \) is the initially induced excess pore pressure at the beginning of the dissipation; and \( \Delta u \) is the pore pressure at time \( t \). Other Normalised EPP criteria have been proposed (Gupta & Davidson 1986; Senneset et al. 1982; Teh 1987), but not widely used. In this study, the normalised excess pore pressures at the location of filter elements \( u_0 \) and \( u_50 \) are expressed as \( U_0 \) and \( U_50 \) respectively.

The time factors proposed by various researchers either differ in the parameters they are defined by or by their literally unique value. Tornberg (1977) suggested that the coefficient of consolidation be interpreted at 50% dissipation time using the following Equation:

\[ c_h = \frac{T_{50}}{t_{50}} \quad (2) \]

where \( c_h \) is the horizontal coefficient of consolidation; \( T_{50} \) is the time factor from the theoretical solutions corresponding to 50% dissipation; \( t_{50} \) is the measured time at 50% dissipation; and \( r_H \) is the equivalent penetrometer radius for the spherical model or penetrometer radius for the cylindrical model. This time factor was later modified to enforce the effect of soil stiffness on the EPP distribution around a penetrating cone. Consequently, Teh & Houlsby (1991) expressed a new time in the form of:

\[ T = \frac{c_h \cdot r^2}{I^2} \quad (3) \]

where \( I \) is the rigidity index, which is a factor of soil shear stiffness on the consolidation process \( (I = G/\nu) \). Other normalization factors are also proposed by Teh (1987), Baligh & Levasdoux (1986), Teh & Houlsby (1991), and Abu-Farsakh et al. (1998) to obtain typical dissipation curves. The majority of these methods propose unique dissipation criteria utilizing estimations of the soil hydraulic parameters.

After the dissipation test data is obtained, a dissipation curve can be depicted by normalizing the EPP and the proposed time factor based on the selected method. This curve is expected to coincide with the typical dissipation curve of the selected method. Then, for any particular degree of dissipation, for example 50% dissipation, \( U_{50} \), a unique time factor exists \( (T_{50}) \) for a particular filter element. Using equation (3), the horizontal coefficient of consolidation can be estimated.

This category of interpretation method is only suitable for monotonically decreasing dissipation curves, known as 'backbone' curves. Therefore, for dilative dissipation curves with initially increasing pore pressure, this method is not an alternative and cannot be applied directly. Dilative dissipation curves can be observed if the clay is stiff or over-consolidated, the porous element is located behind the cone, the soil is fissured or cracked, or the filter element is not well-saturated or is dissaturated during penetration.

Subsequently, some correction schemes were proposed to remake monotonically decreasing dissipation curves from the dilative curves. These methods are categorized as 'data transition methods' discussed below.

2.2 Data Transition Method of Sully & Campanella (1994)

Semi-empirical data transition techniques have been proposed to correct a dilative dissipation curve to a monotonically decreasing curve. This enables application of the 'typical dissipation curves method' to estimate soil hydraulic parameters. The 'log-time' correction scheme and the 'root-time' correction scheme of Sully & Campanella (1994) are the two more prevalent transition techniques used by geotechnical engineers to interpret the dilative dissipation data. In either scheme, the corrected normally-decreasing dissipation data is used by the 'typical dissipation curve' method for hydraulic conductivity measurements.

3 NUMERICAL MODELLING

In this study, penetration of a reference piezcone from the ground surface to a specific depth is simulated taking advantage of axisymmetry. The coupled soil-pore fluid analysis is carried out. The penetrating cone is modelled as an impermeable rigid body. The soil is modelled as a nonlinear elastoplastic material. The plastic response of the soil is modelled using the isotropic hardening modified cam-clay. Large deformation formulation has been used to study cone penetration in fine-grained soils which allow for separation and sliding of a finite strain. Mohr-Coulomb frictional contact is applied to represent interactions between the cone and ground.
a constant rate of 2 cm/s. A master/slave, surface to surface approach is used to detect contact between two surfaces. The rigid piezocone is chosen as the master surface and the soil as the slave surface. To avoid surface penetration, the slave surface was discretised using finer mesh which helps the solution convergence. Surface-based contact is available for fully saturated porous media in Abaqus, however, flow in the direction of the contact surface is set equal to zero and only flow normal to the contact surfaces is included for nodes located on the interface. The 'penalty approach' contact formulation is implemented to ensure contact constraints. A node-to-surface contact discretisation is adopted to minimize the penetration at the contact surface. The classical isotropic Coulomb friction model is used to define the frictional behaviour of the contact surfaces. For more details about this frictional model you can refer to Abaqus 6.7 User Manual.

### 3.1 Finite Element Mesh

The finite element model of a piezocone penetration problem is shown in Figure 2. The reference cone with a surface area of 10 cm² and a cone angle of 60 degrees is modelled using eight-node reduced integration axisymmetric (CAX8R) elements. The soil is discretised using eight-node axisymmetric reduced integration quadratic (bilinear) pore pressure (CAX8RP) elements to obtain adequate accuracy.

Unlike previous studies, the cone is introduced at the top of the soil mesh in this simulation. As a result, the in-situ stress and the hydrostatic pore pressures are calculated more realistically. The soil mesh is refined in the vicinity of penetrating cone to avoid solution divergence.

This numerical model incorporates highly non-linear analysis. To avoid divergence, soil elements and mesh dimensions need to be refined in the vicinity of penetrating cone to avoid solution divergence. The mesh, in this case, is refined in the vicinity of penetrating cone to avoid solution divergence.

### 3.2 Permeability vs Coefficient of Consolidation

The available interpretation methods obtain estimates of the horizontal coefficient of consolidation while finite element numerical models require the soil permeability parameter to be provided as an input. Based on the one-dimensional Terzaghi’s consolidation theory (Terzaghi et al. 1996), hydraulic conductivity of a soil can be related to its coefficient of consolidation via the following Equation:

\[ \kappa = \frac{K}{m C} \]

where \( m \) is the coefficient of volume change, \( C \) is the density of pore fluid. The compressibility coefficient can be determined via oedometer tests or through Menard pressuremeter tests. Empirical CPT estimations are also available but not highly reliable. For numerical calculations, equations which relate \( m \) to its corresponding coefficient of consolidation for a strain hardening clay material are sought. Subsequently, the following Equations are derived to express Equation (4) in terms of MCC soil parameters:

\[ (v_h)_c = \frac{3(1 + \epsilon_0)}{(1 + 2K_{OCR})} \left( \frac{\epsilon_0}{\lambda} \right) \rightarrow OCR = 1 \]

\[ (v_h)_c = \frac{3(1 + \epsilon_0)}{(1 + 2K_{OCR})} \left( \frac{\epsilon_0}{\kappa \
abla} \right) \rightarrow OCR > 1 \]

in the above formulation, \( \lambda = \) compression index; \( \kappa = \) recompression index; \( K_{OCR} = \) in-situ coefficient of earth pressure; \( \epsilon_0 = \) initial void ratio; \( p' = \) volumetric stress.

### 4 MODEL VERIFICATION AND RESULTS

Reliability of the presented piezocone penetration finite element model is verified through comparison with the numerical model of Abu-Farsakh et al. (1998). The model of Abu-Farsakh et al. (1998) is one of the few verified coupled numerical models which accounts for the soil nonlinearity, large deformations, and contact mechanics of the problem. Parameters used to carry out the simulation are listed in Table 1.

As illustrated in Figure 3a, b relatively good agreements exist between the numerical results of Abu-Farsakh et al. (1998) with the finite element model of this study. For both the normally consolidated and overconsolidated models predict similar results at both \( \omega_1 \) and \( \omega_2 \) positions.

### 5 ASSESSMENT OF INTERPRETATION METHODS

The reliability of the numerical models predict similar results. The reliability of the interpretation methods of typical different transition are investigating aims to identify uncertainty within these methods.

#### 5.1 Reliability of Ty娚 Methods

The interpretation of Houlshby (1991) is evaluated (6) which relates perimetric loads and state, undrained penetration of a normally consolidated state of strain and strain path to a constant rate of 2 cm/s.

\[ (v_h)_c = \frac{3(1 + \epsilon_0)}{(1 + 2K_{OCR})} \left( \frac{\epsilon_0}{\lambda} \right) \rightarrow OCR = 1 \]

Figure 2 Geometry, finite element mesh, and boundary conditions.
numerical models require the
eter to be provided as an
one-dimensional Terzaghi's
hydration can be related to its coeffi-
the following Equation:
\[
\frac{b_{0}}{(1 + 2K_{v})} \Rightarrow OCR = 1
\]
\[
\frac{b_{0}}{K_{v}} \Rightarrow OCR > 1
\]
\[
\alpha = \text{compressibility coefficient}
\]
\[
K_{v} = \text{in-situ coefficient of
general void ratio; } \rho' = \text{volumetric}
\]

ATION AND RESULTS

ated piezocone penetrometer fi-
ted through comparison of
Abu-Farsakh et al. (1998)
numerical model for the cone face element and (b) cone shoulder element locations.

5 ASSESSMENT OF THE PROPOSED
INTERPRETATION METHODS

The reliability of the two more frequently applied
technical dissipation curve and the data
ation are investigated in this section. This sec-
ction aims to identify possible sources of error/un-

tainty within these interpretation methods.

5.1 Reliability of Typical Dissipation Curve

The interpretation method presented by Teh &
Housley (1991) is evaluated in this section. Equation
which relates permeability to \( e_{0} \) uses a purely
lent penetration and invokes a normally-

mized state of dissipation to the problem.

theses assumptions coincide with the conditions of those methods founded on the cavity expansion the-

and strain path methods:
\[
f_{w} (\text{NC}) = \frac{3(1 + e_{0})}{(1 + 2K_{v})(1 + 2K_{v}NC)} \rho_{0}^{*} \frac{b_{0}}{\alpha T_{v}}
\]

where \( e_{0} \) is the initial void ratio and \( \rho_{0}^{*} \) is the mean
ormal effective stress at 50% dissipation time for the
element adjacent to the piezcone filter element.

The applicability of the typical dissipation curve
method with the conditions mentioned above forms
our first scenario. The corresponding method is ap-
plied to a pre-defined numerical problem. The nu-

erical model is also used to estimate \( e_{0} \). We

should bear in mind that the methods of typical dis-
sipation curve are solely proposed to cases with a
monotonically decreasing dissipation data. Therefore,
the applicability of these methods is limited to
non-fissured soft normally-consolidated clays with
measurements carried out at the cone face filter ele-
ment (w).

Figure 4 presents the ‘normalised dissipation curve’
for an NC modified camlavy defined in Table
2. Referring to Figure 4, the modified time factor
corresponding to the 50% dissipation time \( T_{50} \) for
the cone face filter element is about 0.116 which is
relatively close to the value of 0.118 proposed by Teh &
Housley’s (1991) analytical solutions. For a \( T_{50} = 455 \) seconds and \( T_{50} = 0.118 \); a value of
\( 3.3 \times 10^{-3} \) cm²/s was estimated while the input value

\begin{table}[h]
\centering
\begin{tabular}{|c|c|c|}
\hline
Parameter & Value & Unit \\
\hline
Slope of virgin consolidation line & 0.3 & - \\
Slope of recompression line & 0.05 & - \\
Slope of critical state line M & 1.4 & - \\
Initial void ratio & 1.0 & - \\
Poisson’s ratio & 0.3 & - \\
Coefficient of lateral earth pressure & 0.5 & - \\
Dry density & 1.4 & t/m³ \\
Permeability & 10⁻⁹ & m/s \\
\hline
\end{tabular}
\caption{Soil input parameters for finite element analysis}
\end{table}
the changes in the stress state history of the soil element undergoing penetration.

The mean total stress is expected to remain unchanged during penetration with a standard rate of 2 cm/s as the undrained condition partially governs the process. During the penetration, a dramatic rise followed by a gradual decay in the induced pore pressure would lead to a momentary unloading-reloading state. Table 3 obtains records of the variation of mean normal effective stress during both penetration and dissipation steps for a clay with OCR equal to 1.6, 9. These values are collected from those elements in direct contact with the penetrometer.

It is clear from Table 3 that the maximum mean normal effective stress within a soil element during penetration always reaches a higher value than its value during the dissipation step for a particular element, irrespective of the soil stress history and the soil permeability value. This means that the consolidation process mainly occurs in the overconsolidated (elastoplastic) state of the soil adjacent to the cone when the soil has already undergone strain hardening during penetration. Hence, the coefficient of consolidation associated with the overconsolidated state is applied instead. To account for the partial drainage during penetration, the void ratio corresponding to 50% dissipation also substitutes the initial void ratio:

\[ (e_{50})_{OC} = \left[ \frac{3(1 + e_0)}{(1 + 2K_{OCR})} \right] P_{so} \left( \frac{k_{OCR}}{\gamma_o} \right) \]

\[ (e_{50}) = \left[ \frac{3(1 + e_0)}{(1 + 2K_{OCR})} \right] P_{so} \left( \frac{k}{\gamma_o} \right) \quad (7) \]

where \( e_0 \) represents the void ratio at 50% dissipation and \( K_{OCR} \) is the at-rest earth pressure corresponding to OC clay. \( K_{OCR} \) for an anisotropically OC clay is estimated by the following empirical formulas:

\[ K_{OCR} = 1 - \sin \phi' \quad \text{Jacky (1944)} \]

\[ K_{OCR} = K_{OCR} - OCR \sin \phi' \quad \text{Schmidt (1966)} \]

where \( \phi' \) is the soil drained friction angle assuming critical state condition (\( c' = 0 \)). The updated normalised dissipation curve by Equation (7) provides a time factor \( T_0 = 0.49 \). The resulting consolidation coefficient related to this numerically derived time factor would then be equal to 1.372 x 10^2 cm/s which reduces the error to 30% for this particular example.

Table 3 Evolution of mean normal effective stress during penetration and dissipation

<table>
<thead>
<tr>
<th>OCR</th>
<th>Maximum mean effective pressure during penetration (kPa) - (1)</th>
<th>Maximum mean effective pressure during dissipation (kPa) - (2)</th>
<th>Ratio (1)/(2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>86.2</td>
<td>71.18</td>
<td>1.21</td>
</tr>
<tr>
<td>6</td>
<td>408.3</td>
<td>373.3</td>
<td>1.09</td>
</tr>
<tr>
<td>9</td>
<td>623.34</td>
<td>526.56</td>
<td>1.19</td>
</tr>
</tbody>
</table>

It is obvious that the accuracy of the proposed typical dissipation curve methods is highly influenced by the assumptions that these methods are grounded on. These assumptions include, but are limited to, a solely one dimensional (radial) consolidation, fully undrained penetration, constant hydraulic properties during dissipation, and elastic consolidation state of dissipation.

Different time factors are derived if different soils with varying strength/stiffness are introduced to the problem. This variation indicates the dependency of dissipation process on unsure parameters within the normalisation factors of this method. Also review of the presented methods by various researchers shows non-unique typical dissipation curves which mainly emanates from the assumptions within these methods. Corrections in the form of ‘data transition methods’ are suggested for dilative dissipation curves which will be evaluated subsequently.

5.2 Reliability of Data Transition Methods

Semi-empirical data transition methods, also known as ‘time transition methods’ or ‘time shift methods’ are proposed to help interpretation of dissipation data with initially increasing EPP response. These methods are used in conjunction with the typical dissipation curve methods to estimate soil hydraulic properties.

The numerically-derived dissipation response of NC clay at the \( e_0 \) shoulder element position is shown in Figure 5. Dilation at the filter element behind the cone is maintained after a sufficient change of time. The resulting data is presented by Sully & Campanella (1964) on a log-time dissipation data.

![Figure 5](image-url)

**Figure 5** (a) Root-time transition scheme, and (b) log-time transition scheme by Sully & Campanella (1964) on a numerically-derived dissipation data.
These methods, also known as dissipation methods, are widely used for dissipation data transition schemes for the BHF dissipation data. The typical dissipation curves which mainly contain more accurate results for this particular clay are adjusted by the numerical model and the method of Teh & Houlsby (1991) to calculate $c_\text{c}$. The numerical results show the dissipation curves respond to the changes in OCR. The effect of overconsolidation on the CPTu dissipation response of 3X10^{-5} cm^2/s is illustrated in Figure 7. The numerical results show how the dissipation curves respond to the changes in OCR. This type of behaviour from OC soils is not considered in the theories proposing typical time factors for various dissipation degrees and as a result, the coefficient of consolidation estimated by these methods correspond to the value for an NC clay.

### 6 Conclusion

The piezocone penetration and dissipation processes are modelled numerically by taking into account the dependency of parameters within the method. Also review of the literature shows that these methods are introduced to the geotechnical practice. These methods are checked numerically below.

Numerically simulated dissipation data of a soil with isotropic coefficient of consolidation equal to 2.81X10^{-3} cm^2/s and a rigidity index of 1.0 is selected to check the accuracy of the above-mentioned correction schemes. Figure 5 shows (a) the root-time correction, and (b) the log-time correction carried out on the dissipation data at the $n_\text{p}$ porous element to change the dilative dissipation curve into a uniformly decreasing curve.

The values of $t_\text{R}$ and $c_\text{R}$ evaluated from these data transition methods are listed in Table 4. The 50% dissipation times are further applied to the method of Teh & Houlsby (1991) to calculate $c_\text{c}$.

Although the root-time correction scheme obtains more accurate results for this particular problem, the transition method does not seem to satisfactorily be adjusting the dilative dissipation data for this case. The modified time factor calculated by the numerical model is also notably different from the value proposed by Teh & Houlsby (1991) for $n_\text{p}$ pore pressure measurements. Equation (7) is used to calculate the $c_\text{c}$ value for the numerical model. It is worthwhile to note that in evaluating the time factors, a rigidity index of 1.0 provides more accurate estimations of the horizontal coefficient of consolidation for both the numerical model and the method of Teh & Houlsby (1991).

Applicability of the data transition methods to interpret dissipation data for the overconsolidated soils seems irrelevant as their theoretical framework to estimate the hydraulic characteristics of cohesive soils is only valid to the normally- to lightly overconsolidated soils. In accordance with the axiomatic theory of consolidation (Equation (8)), the rate of dissipation is in direct proportion with the derivatives of the 'first-order' and also the 'second-order' derivatives of the excess pore pressure in the radial direction expressed as:

$$ u = c \left[ \frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right] + c_\text{c} \frac{\partial^2 u}{\partial z^2} $$

(8)
account the geometric nonlinearity, soil-piezcone contact interface, material non-linearity, and coupled analysis of the multiphase system. The numerical results demonstrate good agreement with the previous numerical model of Abu-Farsakh et al. (1998).

Accuracy of the two most broadly used interpretation methods to estimate the soil hydraulic characteristics of clays via piezcone dissipation data are studied. Results of this evaluation revealed that the typical dissipation curve methods by Teh & Houlshby (1988, 1991) are mainly grounded on idealistic assumptions which can lead to relatively large disagreements while estimating actual hydraulic parameters. These assumptions may include but are not limited to a fully-undrained piezcone penetration and dissipation (no volume change), one-dimensional (radial) consolidation, linear elastic-perfectly plastic soil material with constant coefficient of consolidation throughout the dissipation step, no frictional contact at the soil-penetrometer interface, and negligible effects of initial and boundary conditions on the problem solution. Proposed normalization factors cannot appropriately neutralize the effects of soil stiffness/strength parameters as well as the above-mentioned influential components of the problem. Consequently, non-unique time factors are derived for various soil parameters or different conditions of dissipation.

Data transition methods share the majority of the shortcomings inherent in the methods of typical dissipation curve as their theoretical basis are analogous. Though these correction schemes facilitate interpretation of dissipation data with dilative response, but their estimation can include errors in orders of magnitude (Table 4).

To summarize, the dissipation process of the EPP within fine-grained soils is highly influenced by the soil stiffness and strength parameters, initial and boundary conditions, or even practical considerations such as the rate of penetration. Accordingly, it is somewhat impractical to derive typical dissipation curves with comprehensive applications to all soils. Subsequently, necessity to introduce a new interpretation method that can cover the effect of all the leading factors mentioned above is found to be essential.

REFERENCES


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