Estimating Pile Axial Bearing Capacity by c-φ Derived from Pressuremeter Test, after T-L. Gouw (2021)

Volcanic Cohesive Soil Behaviour under Static and Cyclic Loading, after Sumartini et al. (2021)

New Solutions to Geotechnical Challenges for Coastal Cities, after Chu et al. (2021)

GEOTECHNICAL ENGINEERING

EDITOR-IN-CHIEF

2020 - Present
Dr. Kuo Chieh Chao
Asian Institute of Technology
Thailand

2010 - 2019
Dr. Teik Aun Ooi (Malaysia)
Prof. A. S. Balasubramaniam (Australia)

EDITORIAL ADVISERS

Prof. Za-Chieh Moh (Taiwan)
Dr. Teik Aun Ooi (Malaysia)
Prof. A. S. Balasubramaniam (Australia)
Dr. Edward W. Brand (U.K.)
Prof. Kwet Yew Yong (Singapore)
Prof. Harry G. Poulos (Australia)
Dr. Chin-Der Ou (Taiwan)
Dr. Suttisak Soralump (Thailand)
Dr. Wen Hui Ting (Malaysia)
Dr. John Chien-Chung Li (Taiwan)
Dr. Chung Tien Chin (Taiwan)
Prof. Pedro Seco E Pinto (Portugal)
Prof. Dennes T. Bergado (Philippines)

EDITORIAL PANEL

Prof. Paulus P. Rahardjo
Parahyangan Catholic University
Indonesia

Dr. Dominic Ong
Griffith University
Australia

Dr. Phung Duc Long
Long GeoDesign
Vietnam

Dr. Limin Zhang
Hong Kong University of Science and Technology
Hong Kong

Dr. Apiniti Jotisankasa
Kasetsart University
Thailand

Dr. Darren Chian
National University of Singapore
Singapore

Dr. Erwin Oh
Griffith University
Australia

Dr. Jie-Ru Chen
National Chi Nan University
Taiwan

Dr. Avirut Puttiwongrak
Asian Institute of Technology
Thailand
GEOTECHNICAL ENGINEERING

Paper Contribution, Technical Notes, and Discussions

*Geotechnical Engineering* is the official journal of the Southeast Asian Geotechnical Society and the Association of Geotechnical Societies in Southeast Asia. It is published four times a year in March, June, September, and December and is free to members of the Society. Please visit our website at http://www.seags.ait.ac.th for the membership information.

SEAGS & AGSSEA encourage the submission of scholarly and practice-oriented articles to its journal. The journal is published quarterly. Both sponsors of the journal, the Southeast Asian Geotechnical Society and the Association of Geotechnical Societies in Southeast Asia, promote the ideals and goals of the International Society for Soil Mechanics and Geotechnical Engineering in fostering communications, developing insights and enabling the advancement of the geotechnical engineering discipline. Thus, the publishing ethics followed is similar to other leading geotechnical journals. The standard ethical behavior of the authors, the editor, and his editorial panel, the reviewers, and the publishers is followed.

Before you submit an article, please review the guidelines stated herein for the manuscript preparation and submission procedures. The paper template is available upon request.

Geotechnical Engineering Journal accepts submissions via electronic. The manuscript file (text, tables, and figures) in both words and pdf format, together with the submission letter, should be submitted to the Secretariat and copied to the Chief Editor of Geotechnical Engineering Journal. Email: s-a-journal@ait.ac.th.

The guidelines for authors are as follows:

1. The manuscript including abstract of not more than 150 words and references must be typed in Times New Roman 9 on one side of A4 paper with a margin of 25 mm on each side. The abstract should be written clearly stating the purpose, the scope of work, and procedure adopted together with the major findings including a summary of the conclusions.

2. The paper title must not exceed 70 characters including spaces.

3. The maximum length of papers in the print format of the Journal is 12 two-column pages in single-spaced in Times New Roman 9 including figures and tables. A Journal page contains approximately 1,040 words. Authors can approximate manuscript length by counting the number of words on a typical manuscript page and multiplying that by the number of total pages (except for tables and figures). Add word-equivalents for figures and tables by estimating the portion of the journal page each will occupy when reduced to fit on a 160 mm x 240 mm journal page. A figure reduced to one-quarter of a page would be 260 word-equivalents. When reduced, the figure must be legible and its type size no smaller than 6 point font (after reduction).

4. Figures: Line art should be submitted in black ink or laser printed; halftones and color should be original glossy art. Figures should be submitted at the final width, i.e., 90 mm for one column and 185 mm for two columns. The font of the legends should be in Times New Roman and should use capital letters for the first letter of the first word only and use lower case for the rest of the words. Background screening and grids are not acceptable.

5. Each table must be typed on one side of a single sheet of paper.

6. All mathematics must be typewritten and special symbols identified. Letter symbols should be defined when they first appear.

7. The paper must have an introduction and end with a set of conclusions.

8. Practical applications should be included, if appropriate.

9. If experimental data and/or relations fitted to measurements are presented, the uncertainty of the results must be stated. The uncertainty must include both systematic (bias) errors and imprecisions.

10. Authors need not be Society members. Each author’s full name, Society membership grade (if applicable), present title and affiliation, and complete mailing address must appear as a footnote at the bottom of the first page of the paper.

11. Journal papers submitted are subject to peer review before acceptance for publication.
12. Each author must use SI (International System) units and units acceptable in SI. Other units may be given in parentheses or in an appendix.

13. A maximum of five keywords should be given.

14. References
   


15. Discussions on a published paper shall be made in the same format and submitted within six months of its appearance and closing discussion will be published within twelve months.

For additional information, please write to:

Dr. Kuo Chieh Chao
Hon. Secretary-General
Southeast Asian Geotechnical Society
Email: s-a-journal@ait.ac.th
Website: http://www.seags.ait.ac.th

Ir. Peng Tean SIN
Hon. Secretary-General
Association of Geotechnical Societies in Southeast Asia
Email: pengtean@gmail.com
Website: http://www.agssea.org
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>List of Papers</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Estimating Pile Axial Bearing Capacity by $c$-$f$ Derived from Pressuremeter Test</td>
<td>1-10</td>
</tr>
<tr>
<td>By T-L. Gouw</td>
<td></td>
</tr>
<tr>
<td>2. Volcanic Cohesive Soil Behaviour under Static and Cyclic Loading</td>
<td>11-18</td>
</tr>
<tr>
<td>By W. O. Sumartini, H. Hazarika, T. Kokusho, and S. Ishibashi</td>
<td></td>
</tr>
<tr>
<td>3. Reliability Assessment on Deep Braced Excavations Adjacent to High Slopes in Mountain Cities</td>
<td>19-25</td>
</tr>
<tr>
<td>5. The Use of the Observational Method in Deep Excavations for the Realization of a Residential Compound in a Complex Hydrogeological Context</td>
<td>35-42</td>
</tr>
<tr>
<td>By M. Carassini, F. Bucci, and A. Antiga</td>
<td></td>
</tr>
<tr>
<td>By A. Arsyad, A. B. Muhiddin, T. Harianto, E. Budianto, P. Sangle, and A. B. Firmansyah</td>
<td></td>
</tr>
<tr>
<td>7. Application of Distributed Fibre Optic Sensor (DFOS) in Bi-directional Static Pile Load Tests</td>
<td>48-54</td>
</tr>
<tr>
<td>By S. C. Lee, B.P. Tee, M. F. Chong, H. Mohamad, K. A. Ang, and P. P. Rahardjo</td>
<td></td>
</tr>
<tr>
<td>8. Numerical Study of Ground Surface Settlement Induced by Diaphragm and Buttress Installation</td>
<td>55-60</td>
</tr>
<tr>
<td>By A. Lim and P. Hsieh</td>
<td></td>
</tr>
<tr>
<td>9. New Solutions to Geotechnical Challenges for Coastal Cities</td>
<td>61-66</td>
</tr>
<tr>
<td>By H-D. Do, V-N. Pham, A-D. Pham, P-N. Huynh, and E.Oh</td>
<td></td>
</tr>
<tr>
<td>By C. M. Chow and Y.C. Tan</td>
<td></td>
</tr>
<tr>
<td>12. Compressibility Behaviour of Sapric Peat in Double Drainage Constant Rate of Strain (CRS) Test</td>
<td>95-100</td>
</tr>
<tr>
<td>By D. N. D. Unoi, A. Hasan, A. G. Amuda, and F. Sahd</td>
<td></td>
</tr>
</tbody>
</table>

**Cover Photographs**

1. Estimating Pile Axial Bearing Capacity by $c$-$f$ Derived from Pressuremeter Test
   By T-L. Gouw
2. Volcanic Cohesive Soil Behaviour under Static and Cyclic Loading
   By W. O. Sumartini, H. Hazarika, T. Kokusho, and S. Ishibashi
3. New Solutions to Geotechnical Challenges for Coastal Cities
   By C. M. Chow and Y.C. Tan
Estimating Pile Axial Bearing Capacity by c-ϕ Derived from Pressuremeter Test

T-L. Gouw

1Associate Professor, Universitas Katolik Parahyangan, Bandung, Indonesia
E-mail: gtloffice@gmail.com

ABSTRACT: Due to its rather brittle nature, retrieving undisturbed samples of Jakarta cemented greyish stiff clay, often found at a depth of 30 to 120 m, is very difficult. Good and reliable effective shear strength parameters, i.e., c' and ϕ' values, obtained from triaxial test are hardly available. By modifying cavity expansion theory, Gouw (2017) was able to derive these effective shear strength parameters through Pressuremeter in situ stress strain curve. It was found Jakarta cemented clay exhibiting a drained behaviour when loaded. Its effective cohesion, c', values are linearly increasing with depths, averaging from around 95 kPa at 20 m to around 475 kPa at 100 m depth, while its effective friction angle ϕ' values are within 20°–30°, averaging to around 24°. The values found to be similar to the values derived from CIU triaxial compression test from relatively good undisturbed samples. This paper presents the methodology in deriving the shear strength parameters and then applying the derived Pressuremeter c' and ϕ' values to estimate the pile axial bearing capacity through finite element simulation and comparing it with the commonly known SPT method applied in Jakarta.

KEYWORDS: Pressuremeter, Modified cavity expansion theory, Effective shear strength parameters, Pile axial capacity.

1. INTRODUCTION

By far, Pressuremeter test is the only known in-situ geotechnical testing device capable to generate a stress-strain curve of in-situ soils, somewhat similar to the stress-strain curve obtained from triaxial or direct shear test in soil laboratories. By simulating the Pressuremeter, hereinafter abbreviated as PMT, test through modification of cylindrical cavity expansion theory and matching the resulting stress strain curve with the actual PMT data curve, Gouw (2017) was able to derive effective shear strength parameters, i.e., c' and ϕ' values, of Jakarta cemented stiff clay. His research showed that Jakarta cemented clay, known of its rather brittle nature, exhibiting a drained behaviour when loaded under the PMT test. The effective cohesion, c', values were found to be linearly increasing with depths, averaging from 95 kPa at 20 m depth to around 475 kPa at 100 m depth, while its effective friction angle ϕ' values are within 20°–30°, averaging to around 24°. The values found to be similar to the values derived from CIU triaxial compression test from relatively good undisturbed samples. This paper presents the PMT testing principle, the traditional PMT parameters, the modified cavity expansion formulas used, a case study in deriving c and ϕ of Jakarta cemented clay, and application of the values obtained to estimate pile axial bearing capacity through finite element simulation, finally comparing the result with the commonly known SPT method applied in Jakarta local practice.

2. PRESSUREMETE R TEST AND ITS PARAMETERS

Pressuremeter test is conducted by inserting a cylindrical membrane into a carefully prepared borehole to a determined test depth where the cylindrical membrane is then pressurized against the borehole wall and the subsequent volume expansion (Menard PMT) or the radial expansion (OYO PMT) of the cylindrical membrane is measured. Figure 1 shows the schematic diagram of PMT.

If the pressure is applied by pumping de-aired water into the cylindrical membrane, the actual pressure or stress acting on the borehole wall needs to be corrected against membrane resistance and against the hydrostatic pressure from the manometer level to the centre of the membrane. In Menard PMT the volume of expansion is corrected against the expansion of the hose to deliver the water from the control unit to the membrane. In OYO PMT, also known as Elastmeter, the radius of expansion is corrected against the reducing membrane thickness when pressurised.

The corrected volume or radius is then converted into radial strain of the borehole wall. The resulting corrected radial strain data is then plotted. Figure 2 shows the typical stress strain curve obtained from PMT test.
Traditionally six parameters are obtained from the PMT stress-strain curve, i.e.: $P_0$, $P_y$, $P_L$, $K_m$, $E_m$, and $G$ (Baguelin et al, 1972, 1978; Gambin, 1980, 1995; Gambin and Frank, 2009; Clayton et al, 1982; Briaud, 1992; Clarke, 1995). The parameters are described below (refer to Figure 2 for some notations):

- Horizontal pressure, $P_o$, is the pressure when the membrane first touches the borehole wall, i.e. first point at the beginning of linear or elastic part of PMT curve. This pressure is interpreted as soil total horizontal pressure at rest, i.e.,
  \[ P_o = \sigma_{vo}^* k_o + u_o \]

- Yield pressure, $P_y$, is the end of the linear curve and the beginning of the non-linear or plastic part of the PMT curve.

- Limit pressure, $P_L$, is the ultimate pressure of PMT curve where soil start to ‘flow’, i.e. radial strain keeps on increasing at relatively constant pressure. In practice, limit pressure is hardly achieved, and to obtain this $P_L$ value, the test curve must be extrapolated in a logarithmic plot as shown in Figure 3 below.

- Horizontal subgrade reaction, $K_m$, obtained through linear part of the test curve, i.e.:
  \[ K_m = \frac{\Delta P}{\Delta R} = \frac{P_y - P_o}{R_y - R_o} \]

 where $R_y$ is cavity radius at $P_y$ and $R_o$ is cavity radius at $P_o$.

- Soil deformation or stiffness modulus, $E_m$:
  \[ E_m = (1+\nu) \frac{R_y+R_o}{2} K_m \]

 where $\nu$ is Poisson ratio of the soil, usually taken as 0.33.

- Shear Modulus, $G$:
  \[ G = \frac{E_m}{2(1+\nu)} \]

3. MODIFIED CAVITY EXPANSION FORMULAS

The expansion of cylindrical cavity can be divided into elastic and plastic zone as illustrated in Figure 4. By using Mohr Coulomb failure criterion and radial stress vs modulus of deformation, depicted in Figure 5, Mecsi derive equations to calculate the cohesion, c, and friction angle, $\phi$, of soils from PMT test data. His equations are:

The expansion of cylindrical cavity can be divided into elastic and plastic zone as illustrated in Figure 4. By using Mohr Coulomb failure criterion and radial stress vs modulus of deformation, depicted in Figure 5, Mecsi derive equations to calculate the cohesion, c, and friction angle, $\phi$, of soils from PMT test data. His equations are:
\[
\sigma_u = \frac{2c}{\tan \phi}
\]  
\[\xi = \frac{1-\sin \phi}{1+\sin \phi}\]  
\[E_s = E_0 \left(\frac{\sigma_c}{\sigma_0}\right)^\beta \]  

where \(E_0\) is deformation modulus at a cavity pressure of \(\sigma_c\), \(E_s\) is deformation modulus at a reference pressure \(\sigma_0 = 100\) kPa as shown in Figure 5, coefficient \(\beta\) is rigidity index.

\(E_s\) is deformation modulus at a cavity pressure of \(\sigma_c\), deformation modulus at a reference pressure \(\sigma_0 = 100\) kPa as shown in Figure 5, coefficient \(\beta\) is rigidity index.

The radius where the soil is still in compression is defined as radius of compression (plastic) zone, \(r_c\), and formulated as:

\[r_c = r_0 \left(\frac{\sigma_c + c \cot \phi}{\sigma_0 + c \cot \phi}\right) \left(\frac{1+\sin \phi}{2\sin \phi}\right) \]  

where \(r_0\) = cavity radius at cavity pressure \(\sigma_0\) and \(\sigma_c\) is horizontal or radial stress at boundary of compression zone which is defined as:

\[\sigma_p = \frac{\sigma_0}{\beta} \left[1+\xi - \sqrt{(1+\xi)^2 - 2(1-\xi)2\beta \xi \sigma_o} + \sigma_{ho}\right] \]  

The radial stress inside the compression zone (at radius \(r \leq r_c\)) is:

\[\sigma_r = \left(\sigma_p - \sigma_r\right) \cdot \left(\theta_1\right)^2 + \sigma_{ho}\]  

The induced radial strain, \(\varepsilon_r\):

\[\Delta \varepsilon_r = \frac{\sigma_{ref}}{(1-\beta)E_0} \left[\frac{\sigma_0}{\sigma_{ref.o}} - \left(\frac{\sigma_{ho}}{\sigma_{ref.o}}\right)^{1-\beta}\right] \]  

The induced radial displacement \(\Delta U_r\):

\[\Delta U_r = \frac{\Delta \varepsilon_r \cdot \Delta \varepsilon_{(1-\beta)}}{2} \cdot r_{(1-\beta)} \]  

With the above formulas, it is supposed to be able to derive the \(c\) and \(\phi\) of clayey soils by matching PMT test data curve with the calculated radial stress strain curve, i.e. matching \(\sigma_r\) vs. \(\varepsilon_r\) plot from PMT against \(\sigma_r\) vs. \(\varepsilon_r\) plot from the above cavity expansion formulas.

Gouw (2017) found that the above formulas could not match PMT data of Jakarta cemented stiff clay, especially in the plastic phase of the curve, i.e. the part after yield pressure \(P_y\). To match the test data curve, many trial and error were done. However, every trial could only partially match the PMT data curve and gave different set of \(\beta\), \(c\) and \(\phi\) values, i.e. no unique values could be obtained. On the same test data curve, each of the diagram in Figure 6 shows different values of rigidity index and \(c - \phi\) values! By modifying the deformation modulus function, i.e. modifying equation (8), Gouw (2017) was finally able to match the PMT test data curve and derive a more consistent values of \(c - \phi\) of Jakarta cemented stiff clay. The modified formula is as follows:

- **When PMT stress level is still within the linear range, i.e. within \(P_0\) to \(P_y\), equation (8) needs to be modified into:**

\[E_s = E_0 \left(\frac{\sigma_c}{\sigma_0}\right)^{0.5} \]  

- **When PMT stress level is above yield pressure \(P_y\),**

\[E_{sy} = E_{yo} \left(\frac{\sigma_{yo}}{P}\right)^{a_{yo}} \Rightarrow E_{sy} = m_y E_0 \left(\frac{\sigma_{yo}}{P}\right)^{a_{yo}} \]  

\(E_0\) = elastic soil deformation modulus at cavity pressure of \(\sigma_0\), \(E_s\) = \(E_0\) = pressure modulus as defined in equation (4) 
\(E_{sy}\) = plastic deformation modulus = \(\sigma_{sy}/E_s\) = cavity pressure at plastic part divided by its corresponding strain (from Pressuremeter test data) 
\(E_{yo}\) = yield factor 
\(\sigma_{yo}\) = cavity pressure at and above yield pressure 
\(a_{yo}\) = rigidity factor after yield pressure

To find both \(m_y\) and \(a_{yo}\), equation 8b is normalized as follows:

\[E_{sy} = m_y \left(\frac{\sigma_{yo}}{P}\right)^{a_{yo}} \]  

from PMT data calculate and plot \(E_{sy}/E_0\) vs. \(\sigma_{yo}/P_y\), the parameter \(m_y\) and \(a_{yo}\) can then be obtained by running power function regression analysis. Figure 7 shows one of the plotted test data. In this case,
m_y = 0.6151 and a_y = -2.06. Once parameter m_y and a_y are found, substitute these parameters to equation 8b.

Figure 7 Finding m_y and a_y from Pressuremeter Test Data

Figure 8 shows one of the results of PMT test curve matching with curve calculated from the modified equation (8), i.e. modified E function or modified cavity expansion model. The result shows that when the stiff clay is still in linear “elastic” range, the shear strength consists both cohesion and angle of internal friction (since the shear strength parameters are derived from Pressuremeter, it is notated as c_PMT and \( \phi_{PMT} \)). However, once the soil entering non-linear plastic part, the stiff clay lost its cohesion \( (c_{PMT} = 0 \text{ kPa}) \), and only the angle of internal friction \( \phi_{PMT} \) is working. It is also found that the angle of internal friction remains constant throughout the elastic and plastic phase, i.e. \( \phi_{PMT} = \phi_{PMT} \). The same outcomes are found from all the PMT test data.

4. CASE STUDY ON JAKARTA CEMENTED CLAY

A case study was carried out at a project site at Bendungan Hilir, central Jakarta, where many high-rise buildings are located. The following field and laboratory testings were carried out:

- 21 deep borings carried out between 90 to 120 m depths. SPT tests were taken at every 2 to 3.5 m intervals.
- 20 pre-borehole Pressuremeter tests conducted at cemented stiff clay layers.
- A total of 123 undisturbed samples for laboratory index properties tests, triaxial UU, triaxial CIU and consolidation tests.

Figure 9 to 10 show index and engineering properties of the subsoil. Stiff clay layer is found below 20 m depth, it exhibits an increasing SPT blow counts with depth, bulk unit weights vary within 16.5–18.5 kN/m³ (Figure 9). Plasticity index are mostly within 20 to 60%, water contents fall near the plastic limits, with liquidity indices less than 0.30, an indication of stiff clay (Figure 10). Void ratios of the stiff clay are found to be within 0.70-1.30, it has specific gravity of around 2.63, and water content averaging around 35% (Figure 11).
Figure 12 Pre-Consolidation Pressures and Oedometer Modulus

Figure 12 shows the pre-consolidation pressure and oedometer modulus. The pre-consolidation pressures appear increasing with depth. Comparing with the corresponding effective stresses, the over consolidation ratio of the stiff clay layers is found to be in the order of 2.0. The effective and total shear strength obtained from triaxial CIU tests are shown in Figure 13.

Figure 14 shows typical PMT test data match reasonably well with the curve derived from the modified cavity expansion theory described above. The black triangular dots show the PMT test data point and the dashed red lines show the curve obtained from modified cavity expansion theory. With this matching of curve, the c and \(\phi\) values of the tested cemented stiff clay can be derived. Note that the notation of PMT DB-xx/yy in the graphs means the PMT test conducted at borehole no xx at depth of yy meter. Figure 15 shows the PMT parameters derived from the test data, all the notations on the graphs are as defined before. The effective horizontal stress \(\sigma'_{ho}\) is obtained by subtracting PMT total horizontal pressure \(P_h\) with its corresponding hydrostatic groundwater pressure, as formulated in equation (2). It is important to show the value of effective horizontal stress here as it needs to be implemented in equations (10), (12), and (13).

Figure 13 \(c' - \phi'\) and \(c_u \) and \(\phi_u\) from Triaxial CIU Tests
Figure 14a PMT Test Data Points (black triangular points) vs Modified Cavity Expansion Theory (dashed red line)

Figure 14b PMT Test Data Points (black triangular points) vs Modified Cavity Expansion Theory (dashed red line)
a depth of 100 m, and it is clearly higher than the values obtained from CU triaxial test, be the undrained or drained cohesion. The lesser values of cohesion from triaxial tests are generally attributed to the brittle nature of the Jakarta cemented stiff clay which tends to suffer micro cracks resulted from the sampling process by thin wall tube sampler and during the preparation of the samples in the laboratory. The higher values of \( c_{\text{SMT}} \) is attributed to the cemented nature of the Jakarta stiff clay.

From all the above phenomena, it can be concluded or at least postulated that for Jakarta stiff clay, at the initial stage of Pressuremeter test the soil is in partially or near drained cohesion, as the radial stress and strain reaches its yield pressure, \( P_y \), the stiff clay is already in a fully drained condition. At and beyond this stage the soil at least is in a partially drained condition. At and beyond yield pressure, the induced tangential strain will be large enough to cause spacings within the clay particles move to a larger distance one another and possibly creates micro cracks within the soil structure, hence the clay start to lose its cohesion and left only with its angle of internal friction, at this stage the stiff clay is already in a fully drained condition. This postulated phenomenon is illustrated in Figure 18.
As found above, the strength parameters of the Jakarta cemented stiff clay derived from the PMT tests, \( c_{PMT} \) and \( \phi_{PMT} \), together with the PMT deformation modulus, \( E_{PMT} \), are linearly increasing with depth and can be written as follows. From 20 m to 100 m depth:

\[
\begin{align*}
  c_{PMT} (kPa) &= \frac{y (m)}{0.2106} \\
  E_{PMT} (kPa) &= \frac{y (m)}{0.0011}
\end{align*}
\]  

where \( y \) is depth in m.

5. **ESTIMATING PILE AXIAL CAPACITY**

The shear strength and the deformation modulus of the stiff cemented clay obtained from PMT data are applied to estimate pile axial bearing capacity through finite element analysis by using the axisymmetric model in Plaxis 2D software. The input parameters are presented in Table 1. The finite element model is shown in Figure 19. Figure 20 shows the resulted pile load settlement curve. By applying the ultimate load criterion set in the Indonesian Geotechnical standard (SNI 8640:2017) which set the ultimate load as the load at pile head settlement of 4% pile diameter, the ultimate pile capacity can be estimated.

<table>
<thead>
<tr>
<th>Table 1 Plaxis Input Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth, ( y ) (m)</td>
</tr>
<tr>
<td>--------------------------------</td>
</tr>
<tr>
<td>0 -10</td>
</tr>
<tr>
<td>-10 -20</td>
</tr>
<tr>
<td>-20 -100</td>
</tr>
</tbody>
</table>

Groundwater level = -15m

Pile Properties
- Modeled as linear elastic material
- Pile Diameter = 1.5 m
- Pile Depth = 70 m from ground surface

Figure 17 \( c_{PMT} \) and \( \phi_{PMT} \) Triaxial Undrained \( c_u - \phi_u \)

Figure 18 Radial Expansion causing Micro-cracks
4% of 1.5 m pile diameter is 60 mm pile head settlement, from figure 19, it can be found that the ultimate capacity of the pile is:

\[ Q_{ult\_PMT} = 30,395 \text{ kN} \]

Figure 20 shows the idealised SPT profile to calculate the pile axial bearing capacity from the following formula:

\[ Q_{ub} (kN) = m N_s A_s + n N_b A_b \]  \hspace{1cm} (17)

where \( m = 6 \) = friction coefficient, \( n = 40 \) = base coefficient, \( N_s \) is SPT blow count along the pile shaft, \( N_b \) is the SPT blow count at pile base; \( A_s \) is the pile skin area and \( A_b \) is the pile base cross sectional area.

Based on this approximate SPT formulas commonly adopted in Jakarta practice, the ultimate bearing capacity of the same pile size found is:

\[ Q_{ult\_SPT} = 30,610 \text{ kN} \]

It can be seen the PMT and the SPT results give similar values of estimated pile axial capacity.

6. CONCLUDING REMARK

To derived \( c \) and \( \phi \) values of Jakarta stiff clay from PMT data, Mecsi model needs to be modified. The deformation modulus need to be divided into two parts as written in Equation (8a) and (8b). With this modified E function, cavity expansion theory can then be applied to derive the shear strength parameters.

PMT test in Jakarta stiff clay initially exhibits partially drained condition and then gradually become fully drained condition when reaching and beyond its yield pressure. The \( c \) and \( \phi \) values obtained from Pressuremeter test are effective stress parameters. The Pressuremeter test can reveal the effect of cementation of Jakarta stiff clay which appear in a higher value of cohesion which cannot be captured by triaxial test due to the difficulty in obtaining a good ‘really’ undisturbed Jakarta stiff clay samples by normal thin wall tube sampler.

The axial pile bearing capacity calculated by finite element method with strength and stiffness parameters derived from PMT test is comparable with the calculated bearing capacity of SPT formula commonly used in Jakarta’s practice.

Further research is necessary to make sure whether the theory derived in this study can be applied to estimate the strength parameters of other soil types. It will be good if PMT test data can be done in conjunction with instrumented pile load test data tested to failure, with this the theory can be further verified.

7. ACKNOWLEDGEMENT

The author would like to thank Prof. Paulus. P. Rahardjo, and Prof. A. Aziz Djajaputra for their valuable guidance during the research. To Prof. H. Moeno, R. Karlinasari PhD and S. Herina, for their feedbacks. To GEC and PT. Pondasi Kisocon Raya for providing necessary data for the research. Finally, high appreciation also attributed to Universitas Katolik Parahyangan for facilitating the research.

8. REFERENCES


