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Evaluation of the lateral response of single piles to adjacent excavation from CPT data

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Abstract: Excavation inevitably induces stress changes in the surrounding soil, which causes significant lateral movements, additional forces and bending moments in the existing pile foundations. To prevent or minimize damage to adjacent piles, this paper presents an actual full-scale instrumented study to examine the lateral response of existing piles to an adjacent test pit excavation. Cone Penetration Tests (CPTs) near the test piles before and after excavation are conducted and compared. A simple $p$-$y$ evolution model for lateral piles during excavation is proposed. The proposed model is defined by the pre-excitation and post-excitation $p$-$y$ curves from CPT data, which can provide better predictions by comparing with the field measured results. Additionally, for analysis of the pile behaviour after excavation, the observed lateral bearing capacity of full-scale tests are compared with those computed by the pre-excitation and post-excitation $p$-$y$ curves. The post-excitation $p$-$y$ curve can generally give a satisfactory prediction of the residual pile bearing capacity after excavation. The calculated results from the free-field cone parameters have a serious overestimate and are detrimental to the service design of pile foundations.

Keywords: piles; excavation; CPT; $p$-$y$ curve; lateral loads; full-scale tests

Introduction

Soil excavation is typically performed in urban densely populated cities to construct high-rise buildings and subways. Its detrimental effects on adjacent existing pile foundations have been widely recognized. Excavation inevitably induces the stress relief and lateral movement of the surrounding soil, which results in additional bending moments and
deflections in the adjacent piles. The impact is significant, particularly for piles constructed in soft clay. There have been several instances where the existing pile foundations were affected or damaged because of adjacent excavation, e.g., Amirsoleymani (1991), Finno et al. (1991), Chu (1994), and Kok et al. (2009). To estimate the excavation-induced pile response, empirical and numerical studies were conducted (Poulos and Chen 1996, 1997; Chow and Yong 1996; Guo 2003; Ong et al. 2009; Zhang et al. 2011). Poulos and Chen (1996, 1997) presented design charts for the lateral pile response adjacent to unsupported and supported excavations. Chow and Yong (1996) proposed a finite-element method where the pile was represented by beam elements, and the soil was idealized using the modulus of subgrade reaction. The numerical methods generated for excavation piles mainly include the complete three-dimensional analysis, which uses the finite-element method or finite-difference method (Pan et al. 2002; Mroueh and Shahrour 2002), and the two-stage analysis method (Poulos and Chen, 1996, 1997; Loganathan et al. 2001; Goh et al. 2003). The two-stage method is extensively acceptable for the pile design based on a Winkler model. A series of 1-g and centrifuge model tests has also been conducted to investigate the behaviour of single piles or pile groups in sands or clays considering lateral soil movements (Stewart et al. 1994; Poulos et al. 1995; Chen et al. 1997; Leung et al. 2000, 2003, 2006; Ong et al. 2006, 2009). Some of these are used to verify or amend the existing theoretical methods of passive piles. For the field tests, Finno et al. (1991) and Goh et al. (2003) reported the performance of pile groups and single piles adjacent to a deep supported excavation, respectively. Thasnanipan et al. (1998) and Kok et al. (2009) presented case studies of pile failure associated with open excavations in Bangkok soft clay and West Malaysia marine clay, respectively. The results of
these researchers lay a foundation for in-depth study.

However, previous studies pay little attention to the in situ soil stress changes induced by excavation unloading effects, which usually focus on the lateral soil movement. In stress terms, the soil stresses around the pile inevitably decrease because of the excavation unloading. Ong et al. (2006), and Leung et al. (2006) found the reduction in undrained shear strength by comparing the T-bar results prior to and after excavation in centrifuge model tests. In their study, they also concluded that post-excavation undrained shear strength values should be adopted to give a more accurate prediction of the excavation-induced pile response compared to pre-excavation undrained shear strength. However, the detailed stress relief characteristic corresponding to different unloading effects and its impacts on the lateral pile response are not well understood. Recently, Ong et al. (2015, 2016) reported a field case study on the prefailure and postfailure behaviours of an instrumented cast-in-place concrete pile group subject to lateral soil movement triggered by a slope failure. To date, field tests that demonstrate lateral pile responses during and after excavation are relatively scarce because it is practically impossible to instrument existing piles in the field.

This paper presents the results of an actual full-scale instrument study in Yangtze River multi-layer soils and evaluates the lateral response of single piles during excavation and after excavation from CPT data. The selected pit foundation excavation engineering near the Yangtze River presented a unique opportunity to perform the experimental study.

**Site description and test setup**

The test site is located in Jingjiang city of Jiangsu province near China’s Yangtze River
(Fig. 1). The project, which is named the Jingjiang cultural centre, plans to centralize the construction of a theater, a library, a museum and commercial buildings in this site. Fig. 2 shows a top view of this project at a stage of foundation pit excavation when the upper 6-m-deep soil layer has been completely dug up for a double basement. According to the engineering requirements, four foundation pits (Pits 1-4) will be excavated at different times. In this study, the full-scale instrument test was conducted for the construction of foundation Pit 2, as shown in Fig. 2. The test piles (P1 and P2) were installed prior to the excavation of the upper 6-m-deep soil layer for the basements and tended to be above ground surface after the soil was removed. The bored pile is 1 m in diameter and 30 m in effective embedded length with a C30 concrete grade. The concrete cover was 50 mm. The steel reinforcement consisted of 16 Φ25 main bars equally spaced around the circumference with Φ8 spiral links at 100 mm pitch. The reinforcement extended to the full pile length.

Fig. 3 shows the measurement points with the locations of the test piles and test pit excavation corresponding to Fig. 2. The test pit excavation is part of the excavation of the foundation, so it does not affect the construction sequence of the entire excavation project. As shown in Fig. 3, a series of cone penetration tests (CPTs) was conducted to examine the changes in soil stress and pile response before and after the test pit excavation. To reflect the actual soil parameters around the piles, CPT-1 and CPT-4 were near the test piles. CPT-2 and CPT-3 were conducted to identify the soil distribution of the site as a supplementary reference. In these four CPT tests, CPT-4 was performed after the test pit excavation was completed. The other three CPT holes were finished before the excavation. Test piles P1 and P2 were distributed inside and outside the retaining wall (piles), respectively. As shown in Fig.
3, the retaining wall divided the test site into an excavation impact area and a free field area (not disturbed). Test pile 1 (P1) outside the retaining wall had the bearing characteristics in free field without excavation impacts. Pile P2 was located 1.5 m behind the excavation line. The excavation zone of the test pit for the study was 5 m long, 5 m wide, and 4 m deep, which can be accurately implemented using an excavator. Fig. 4(a) shows the cross section of the excavation, relative position of P2 and the excavation. An electronic-level displacement mirror and an in-pile inclinometer of 30 m length were installed in pile P2, as shown in Fig. 3 and Fig. 4(b). Additionally, an in-soil inclinometer was installed in the parallel position with pile P2 to observe the soil displacements. To monitor the pile behaviour during excavation, pile P2 was instrumented with strain gauges to measure the bending moments along the pile shaft. In detail, fifteen pairs of strain gauges were distributed on the front and rear sides of the upper pile at a vertical interval of 1 m, and seven pairs were distributed along the lower half of the pile at a vertical interval of 2 m. The detailed layout and installation of strain gauges are shown in Fig. 5.

In situ and laboratory tests were performed to determine the geotechnical properties of the test site. The soil characterization is shown in Fig. 6. The subsoil profile along the effective pile depth mainly consists of four layers: upper silty sand, middle muddy-silty clay, middle silty sand and silt, and bottom silty clay; the thickness values are $h=2.5$ m, 5 m, 3.5 m, and 17 m, respectively. The actual pile length is over 30 m with a sand-socketing layer below the silty clay layer. Those strata are widely distributed in the Yangtze River basin, China. The muddy-silty clay or silty clay easily produces large lateral movements during soil excavation and induces significant bending moment and deflection to the existing piles.
**Results of cone tests**

Obtaining excavation effects on surrounding soils is a complicated engineering problem, particularly in nonhomogeneous sites. As a quick in situ test method, the cone penetration test (CPT) can obtain the soil stress conditions of different soil layers in a continuous penetration process. The scope of this cone test results is two-fold: first, to observe the changing rules of soil stress conditions before and after an adjacent test pit excavation; and second, to evaluate the lateral response of single piles considering excavation effects and develop a simple analysis method from the CPT perspective.

The CPT $q_c$ profiles, which were measured at different locations before excavation, and the corresponding soil behaviour type, which was measured using Robertson et al. (1986) classification method, are shown in Fig. 7. Most soils within the depth of 2.5-7.5 m are identified as clay to silty clay. The similarity in cone resistance for CPT-1, CPT-2, and CPT-3 suggests that the test site is approximately homogeneous and has no significant variability in the soil. CPT-2 and CPT-3 inside the retaining wall are consistent with CPT-1 outside the retaining wall in the free field, which presents an advantage to directly compare the differences between post-excavation and pre-excavation.

Fig. 8 presents the compared soil resistance profiles of CPT-1 and CPT-4 relative to the 4.0-m-deep test pit excavation. After the pit excavation, the soil stress was disturbed, and the cone resistance profiles significantly changed. The corresponding CPT $q_c$ profiles are related to the different soil stress paths and lateral movements. As shown in Fig. 8, two categories of excavation effects in this study can be summarized: horizontal unloading effect and vertical...
unloading effect. The horizontal unloading effect that resulted from the excavation-induced lateral soil stress relief is mainly concentrated above the excavation surface. The cone resistance of the silty sand (Fig. 6) at 0-2.5 m deep has a large decrease because of the horizontal unloading effect. However, there was a small change for muddy-silty clay at 2.5-4.0 m deep. In this test, the muddy-silty clay is poor sensitive to the horizontal unloading, which is similar to the vertical unloading effect. The excavation of a 4-m-deep test pit inevitably generates a vertical stress relief on the soil region below the excavation surface. Hence, the cone resistance of the surrounding soils decreases in different degrees. It can be concluded that the vertical unloading more strongly affects silty sand than muddy-silty clay in Fig. 8. The former CPT $q_c$ profile has a significant decrease because of the vertical unloading effect. These results can be explained by Mohr-Coulomb strength equation (Mitchell 1976) as follows.

$$\tau = \sigma \tan \phi + c \quad (1)$$

where $\tau$ is the soil shear strength, $\sigma$ is the normal stress, $c$ is the soil cohesion, and $\phi$ is the internal friction angle.

As presented in Eq. (1), the shear strength of muddy-silty clay is mainly controlled by the cohesion ($c$). Hence, when it suffers unloading effects of the normal stress ($\sigma$) in this experiment, its shear strength remains a blunted response. Conversely, the shear strength of silty sand is mainly controlled by the normal stress ($\sigma$) instead of the cohesion ($c$). A significant reduction in shear strength occurred during an excavation-induced unloading. In addition, the CPT cone resistance ($q_c$) is positively correlated with the soil shear strength.
(Robertson 2009). Hence, the change in $q_c$ of silty sand (7.5~11 m) appeared more obvious than that of muddy-silty clay (2.5~7.5 m) subject to the excavation unloading effect, as shown in Fig. 8. The cone resistance of muddy-silty clay keeps small changes. Beyond the depth of 11 m, the effect of the test pit excavation is almost non-existent, and the CPT $q_c$ profile remains unchanged. The impact depth from the excavation surface in this test is almost twice the depth of the pit excavation ($\approx 2h$), which is consistent with the existing research results (Jia and Xie 2008; Qin and Jiang 2008).

**Numerical analysis**

A simplified numerical program based on the finite-element method was used to analyse the lateral pile response. The program is named PYGMY (proposed by the University of Western Australia, Stewart 2000) and has been described elsewhere (Lee 2008; Randolph and Gourvenec 2011). In the PYGMY program, the pile is modelled by beam elements, and the pile-soil interaction is idealized using a series of nonlinear soil springs, also called $p$-$y$ curves. Measured free-field lateral soil movements from field tests are used as input values.

**CPT based $p$-$y$ formulations**

For this study, the CPT-based $p$-$y$ curves by Li et al. (2017) for soft clays and Suryasentana and Lehane (2014) for sands were used, which were directly generated from CPT data. The cone resistance $q_c$ from Fig. 8 was used as an input. The $p$-$y$ formulation of Li et al. (2017) for soft clays is given by

$$p = \frac{0.5N_c}{N_k}D(q_c - \gamma z)[\frac{100y}{(0.215q_c / p_a - 1.25)D}]^{1/3}$$

(2)
\[ N_c = 3 + \frac{N_k \gamma' z}{q_c - \gamma z} + \frac{0.5z}{D} \quad (z < z_r) \]
\[ N_c = 9 \quad (z \geq z_r) \]
\[ z_r = \frac{6D}{\gamma' - \frac{N_k D}{q_c - \gamma z} + 0.5} \]

where \( p \) is the lateral soil resistance per unit length of the piles; \( y \) is the lateral pile deflection; \( z \) is the depth from the ground surface; \( z_r \) is the limit depth; \( \gamma \) and \( \gamma' \) are the unit weight and effective unit weight of soils, respectively; \( D \) is the pile diameter; \( q_c \) is the cone resistance; \( p_a \) is the atmospheric pressure (0.1 MPa); \( N_k \) is the cone factor; \( N_c \) is the bearing capacity factor.

In general, a representative value of \( N_k = 15 \) is adopted for Jiangsu silty clays.

The \( p-y \) formulation of Suryasentana and Lehane (2014) for sands is

\[ p = p_u \left\{ 1 - \exp\left[ -6.2\left( \frac{z}{D} \right)^{-12} \left( \frac{\gamma}{D} \right)^{0.89} \right] \right\} \]  

\[ p_u = 2.4\gamma' z D \left( \frac{q_c}{\gamma' z} \right)^{0.67} \left( \frac{z}{D} \right)^{0.75} \]

where \( p_u \) is the ultimate lateral soil resistance, and other parameter values are the same as before.

The CPT depth of this experiment was approximately 22 m \((\approx 22D)\). When we determined \( p-y \) curves of different layer soils using Eqs. (2) and (5), the average cone resistance \( q_c(i) \) of each layer was calculated according to the following equation.

\[ q_c(i) = \frac{1}{h_i} \int_0^{h_i} q_c(z)dz = \frac{1}{h_i} \sum_{j=1}^{N} q_c(j) \Delta z(j) \]

where \( h_i \) is the soil thickness of layer \( i \); \( j \) is the sublayer corresponding to layer \( i \); \( N \) is the
number of sublayers; $\Delta z(j)$ and $q_c(j)$ are the thickness and cone resistance of sublayer $j$, respectively.

In this study, Eq. (5) is used for the silty sand layer and silty sand and silt layer, and Eq. (2) is used for the silty clay layer and muddy-silty clay layer. Fig. 9 presents a set of representative $p-y$ curves at typical depths for different soils calculated from the $q_c$ profiles in Fig. 8. The excavation unloading effects on the $p-y$ curves associated with the lateral pile response are well reflected. It can be seen that the post-excitation $p-y$ curves of silty sand ($z=1$ m), muddy-silty clay ($z=5$ m), and silty sand and silt ($z=8$ m) in Fig. 9 have different degrees of decline compared to the $p-y$ curves before excavation.

Then, the $p-y$ curves and measured lateral displacements from the in-soil inclinometer (Fig. 3) were used as the input to the PYGMY program to analyse the lateral pile response. The program also requires an input on the pile flexural rigidity $EI$, pile length and diameter, etc. The detail pile properties are presented in Table 1.

**Results of the analysis**

The measured and predicted pile deflection and bending moment profiles using CPT data from the free field and after excavation are shown in Fig. 10. The measured lateral soil displacement is also drawn in Fig. 10, which shows a large lateral movement in the muddy-silty clay layer. The measured bending moments of the test pile were calculated from the strain gauges attached to the pile shaft using the following equation suggested by Rollins et al. (1998):
\[ M = \frac{EI(\varepsilon_t - \varepsilon_c)}{d_s} \]  

(8)

where \( \varepsilon_t \) and \( \varepsilon_c \) are the tensile and compressive strains at the location of the strain gauges, respectively, \( EI \) is the pile flexural rigidity, and \( d_s \) is the horizontal distance between the strain gauges.

The measured pile deflections in this study were both derived from using direct inclinometer data and twice integrating bending moments, respectively. A seventh-order polynomial about the axial coordinate of the pile shaft was used to least-squares fitting the measured bending moments from strain gauges. Then, the pile deflections were calculated by twice integrating the bending moments. As shown in Fig. 10, in order to examine deformations of the pile head, two constraints for the pile deflection at the pile-toe and 10 m-depth below the surface were applied, and those data were directly read from the inclinometer. It is noted that the pile deflections from the two approaches are basically consistent.

As shown in Fig. 10, using the \( p-y \) curves from the CPT data before and after excavation, the calculated pile deflection and bending moment undergoing lateral soil movements are significantly different from each other and deviate from the measured results. The maximum displacement in the pile head that was calculated using CPT data after excavation is 11.1% larger than the result using CPT data from the free field. Likewise, the computed bending moment from CPT data after excavation is 22% higher in the maximum positive bending moment and 10.4% lower in the negative bending moment compared to the values calculated using CPT data from the free field. Fig. 11 also presents the calculated rotation curves of the
pile using the two different $p$-$y$ curves because the rotation is usually a control standard of free-head single piles. There is a larger bias in pile head rotation compared to the deflections and bending moments. The use of CPT data from the free field may produce a 300% deviation compared to the result from CPT data after excavation. In other words, if we only pay too much attention to the lateral soil movement in practice and ignore the release in soil stress because of excavation effects, great security risks for the on-service performances of the piles during excavation will be occur. Additionally, the predicted lateral pile behaviour in Fig. 10 using these two $p$-$y$ models before and after excavation remains unsatisfactory. The former underestimates the measured pile deflection and bending moment, and the latter is the opposite. The correctness of the lateral response considering excavation depends on the appropriate $p$-$y$ model being used.

To overcome this defect and provide an accurate prediction, the realistic $p$-$y$ model of excavation soils should be further proposed. Fig. 12 presents an attenuated evolution of the conjecture $p$-$y$ relationship based on the pre-excavation and post-excavation $p$-$y$ curves from the CPT data with the silty sand ($z=1$ m) and muddy-silty clay ($z=5$ m) as examples. It shows that the shape of the $p$-$y$ curve under unloading effects resembles a strain-softening curve, whereas the conventional $p$-$y$ curves usually have a hyperbolic shape. This result is easy to understand: when the surrounding soil transits from the pre-excavation state to the post-excavation state, the $p$-$y$ curves continue changing according to the intermediate process curves in Fig. 12. The soil resistance will cross different intermediate $p$-$y$ curves step by step with the increase in displacement. For brevity, the description of evolution steps in Fig. 12 assumes that the soil resistance falls into an adjacent $p$-$y$ curve in each step, which
corresponds to the identical displacement (approximately uniform rate). The actual feature of evolution steps from the pre-excavation $p-y$ curve to the post-excavation $p-y$ curve should be complicated with multiple factors, such as embedded pile depth and excavation rate. As shown in Fig. 12(a), an attenuated evolution curve that corresponds to the excavation process is delineated by connecting the intermediate point of each ladder step. The end of this evolution curve falls to the post-excavation curve and continues along the post-excavation curve. This entire attenuated evolution curve can be simplified as a simple evolution $p-y$ model with three-gradient lines. Compared with the initial soil condition, the ultimate residual resistance in Fig. 12(b) is a reaction to the stress relief caused by soil excavation, which can be obtained from literature (Kirkpatrick and Khan 1984). To a limit state, the end of the second line segment in the attenuated $p-y$ model will fall to the line of ultimate residual resistance and subsequently remain constant. Any post-excavation $p-y$ curve under different excavation conditions will intersect the second line segment. An assumed post-excavation curve under the actual post-excavation in this experimental was drawn in Fig. 12(b) for ease of understanding.

**Simple evolution $p-y$ models during excavation**

According to the previous analysis, the $p-y$ curves during excavation can be characterized as the attenuated type using the three-gradient line models. More fortunately, the initial pre-excavation $p-y$ curve and post-excavation $p-y$ curve from the CPT data can be used to easily determine the evolution characterization when the surrounding soil transits from the free-field state to the excavation state.

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Attenuated p-y model subject to excavation unloading effects

For the unloading effects, the attenuated p-y evolution model can be expressed as follows, and its schematic diagram is illustrated in Fig. 13.

\[
\begin{align*}
  p &= k_1 y & \quad & y \leq y_1 \\
  p &= p_1 + k_2 (y - y_1) & \quad & y_1 < y < y_2 \\
  p &= p_{po}(y) & \quad & y \geq y_2
\end{align*}
\]

(9)

In Fig. 13, \(p_u\) is the ultimate soil resistance of the free field before excavation; \(p_r\) is the ultimate residual soil resistance with disturbance or stress relief, which corresponds to a yield displacement of soil \((y_r)\); \(p_1\) is the maximum soil resistance under excavation unloading effects, which corresponds to the terminal displacement of the initial portion \((y_1)\) where the soil resistance begins to decay; \(p_2\) and \(y_2\) are the terminal soil resistance and displacement of the second portion line, after which the soil resistance will enter the post-excavation \(p-y\) curve. The subscript \(po\) in Eq. (9) denotes the post-excavation. \(k_1\) and \(k_2\) are the initial stiffness and post-peak stiffness of the excavation \(p-y\) curve. For \(y > y_r\), the stiffness tends to zero. The absolute advantage of this model is that it can be directly determined by the pre-excavation and post-excavation \(p-y\) curves from CPT, which is convenient for routine practice.

In Eq. (9), the three-gradient line model was used to represent the load transfer behaviour subjected to excavation unloading effects. The following section outlines the approach to determine the initial stiffness \(k_1\) and post-peak stiffness \(k_2\) for cohesionless and cohesive soils, respectively. If points \((p_1, y_1)\) and \((p_r, y_r)\) are determined, \(k_1\) and \(k_2\) can be obtained.
Cohesionless soil

The soil resistance-displacement relationship for laterally loaded piles is typically represented by subgrade modulus methods, where the soil response is characterized by coefficient $k$. To date, various correlations between $k$ and Young's modulus of soils have been proposed. Based on the analysis of an infinite beam on an elastic foundation, Vesic (1961) suggested the following expression:

$$
k = \frac{0.65E_s}{1-\nu_s^2} \left(\frac{E_sD^4}{EI}\right)^{\frac{1}{2}} \quad (10)$$

where $E_s$ is the soil deformation (secant) modulus, $\nu_s$ is Poisson's ratio of the soil, and $EI$ is the pile flexural rigidity.

For sands, it is customary to assume that the modulus $E_s$ linearly varies with depth, and Decourt (1991) suggests the following correlation

$$
E_s = N_h z \quad (11)
$$

where $z$ is the depth below the ground surface and $N_h$ is the parameter related to soil properties. Typical values of $N_h$ for loose, medium and dense sands are 1.5, 5.0, and 12.5 MPa·m$^{-1}$, respectively.

Reese et al. (1974) studied the problem of laterally loaded piles in sands and recommended the following expression for the yield displacement of sands

$$
y_r = \frac{3D}{80} \quad (12)
$$

To determine the ultimate residual soil resistance $p_r$ of the attenuated $p$-$y$ model, $N_h = 1.5$
for loose sand (Decourt 1991) was selected in this study to determine the deformation modulus considering excavation effects. We substitute the expression for $E_s$ and $y_r$ into Eq. (10) and rearrange to obtain an expression for $p_r$

$$p_r = ky_r = \frac{1.95N_s zD}{80(1 - \nu^2)} \left( \frac{N_z D^4}{EI} \right)^{\frac{1}{12}}$$  \hspace{1cm} (13)

In addition, the transformation displacement $y_t$ of elasticity and plasticity for sand proposed by Reese et al (1974) is as follows:

$$y_t = \frac{D}{60}$$  \hspace{1cm} (14)

where $y_t$ was proposed on the basis that the sand response is elastic, but the excavation sand in Fig. 12a is understood as having entered the elastic-plastic stage of the hyperbolic $p-y$ curve for the free field. Therefore, it is not appropriate to directly use Eq. (14) in the attenuated $p-y$ model for excavation soils, where the stiffness have changed compared to the initial soil condition. By retaining the relationship between $y_t$ and the pile diameter in Eq. (14) and noting that $y_1 = 2y_t$ in the three-gradient line model considering excavation effect, the terminal displacement $y_1$ is expressed as

$$y_1 = \frac{D}{30}$$  \hspace{1cm} (15)

Therefore, substituted $y_1$ into the pre-excavation $p-y$ curve in Fig. 13, the maximum soil resistance $p_1$ for sands in the attenuated $p-y$ model can be determined.

In this study, the preceding expression was used for cohesionless soils.

_Cohesive soil_
To determine the ultimate residual soil resistance $p_r$ of clays, the theory about ultimate soil resistance $p_y$ was borrowed, which can always be related to the undrained shear strength $s_u$ of soils (Matlock 1970; Randolph and Housby 1984; Chen and Poulos 1994). In this study, the $p_y - s_u$ relationship proposed by Randolph and Housby (1984) using classical plasticity theory is used. They recommended the following expression:

$$p_y = 10.5s_u \quad (16)$$

When the soil is in a limit state of excavation-induced stress relief, Eq. (16) can be used to obtain $p_r$. The undrained shear strength $s_{ur}$ of the excavation soil considering stress relief is different from the undrained shear strength $s_u$ of in situ soil. The limit ratio, $s_{ur}/s_u$ proposed by Kirkpatrick and Khan (1984) is 0.55 for illites and 0.4 for kaolins, respectively. An average value $s_{ur}/s_u \approx 0.5$ is adopted in this study. Hence, the ultimate residual soil resistance $p_r$ for cohesive soil can be expressed as

$$p_r = \frac{p_y}{2} = 5.25s_u \quad (17)$$

The undrained shear strength $s_u$ in free field can be easily obtained by cone resistance. In addition, it is common practice to estimate the yield displacement $y_r$ and displacement $y_{50}$ for clays using the expressions of Matlock (1970):

$$y_r = 20\varepsilon_{50 \text{dis}}D \quad (18)$$

$$y_{50} = 2.5\varepsilon_{50 \text{pre}}D \quad (19)$$

where $\varepsilon_{50}$ defines the soil strain at 50% of the maximum stress in laboratory triaxial tests. The subscripts dis and pre denote the disturbed and pre-excavation states, respectively.
Matlock (1970) suggested, the range of $\varepsilon_{50}$ is 0.005-0.02. Typical values of $\varepsilon_{50}$ for sensitive clay, disturbed (or remoulded) clay, and normally consolidated clay are 0.005, 0.02, and 0.01, respectively. For simplicity, 0.01 for the pre-excavation condition and 0.002 for the limit disturbed condition are used to build the evolution $p$-$y$ model in this study. The point $(p_r, y_r)$ along with point $(p_1, y_1)$ was used to determine the $k_2$ shown in Fig.13. Meanwhile, the $y_{50}$ expression for the free field in Eq. (19) was used to determine the terminal displacement $y_1$ of the initial portion of the three-gradient line model for brief.

If $y_1$ is determined and substituted into the pre-excavation $p$-$y$ curve (Eq. (2) for clays) in Fig. 13, the maximum soil resistance $p_1$ for clays in the attenuated $p$-$y$ model can be determined.

In this study, the preceding expression was used for cohesive soils.

**Comparison with measured results**

Fig. 14 presents the calculated lateral response using the proposed attenuated $p$-$y$ models in the PYGMY program compared to the measured lateral deflection and bending moment. In contrast to the results of conventional $p$-$y$ curves (see Fig. 10), the proposed excavation model in this study can better predict the lateral response of the test pile during excavation. In particular, the position and magnitude of the maximum bending moment and deflection are well predicted. The proposed $p$-$y$ model during excavation consider the “attenuated evolution” rule, and the shape of the soil resistance versus displacement curve is more accurate. Additionally, the proposed attenuated $p$-$y$ model is based on CPT data, which can account for complex soil conditions such as nonhomogeneous layers in practice.
Lateral response after excavation

For in-service piles adjacent to an excavation, the lateral loads imposed by lateral soil movements, which induce bending moments and deflections, will be eliminated over a period of time after the excavation. Then, the pile-soil system enters a new equilibrium stage. If the pile continues to suffer external horizontal loads on the pile head (active piles) after the excavation, its residual bearing characteristics should be considered. Hence, the lateral bearing capacity of the test piles after excavation was further calculated and compared using the $p-y$ curves from CPT data before and after excavation, respectively. The field lateral load tests were conducted for pile 1 and pile 2 (Fig. 3) to verify the calculated results.

The lateral load test was performed using a displacement controlled approach. The hydraulic jack was controlled with an electromechanical servo-valve and an electric hydraulic pump. Because the test piles would later be used as working piles, a maximum displacement of 6 mm was applied to protect the piles from damage. The lateral loads were applied to the pile head. Linear variable differential transducers (LVDTs) were installed on the pile face in the opposite direction of the loads. There were 10 loading steps. At the end of each step, displacement data from the LVDTs were collected.

Fig. 15 presents the load-displacement ($H_0 - y_0$) curves of the two test piles computed using PYGMY and the measured results. It can be seen that the lateral bearing capacity of the pile after excavation is reduced compared to the free field (before excavation). The calculated $H_0 - y_0$ relationships of the two piles are consistent with the measured results. It means, if we select the non-excavated soil parameter as the pile design parameter and do not consider the
effect of excavation, we will overestimate the bearing capacity of the pile foundations and bury the safety hazard. In this study, the residual bearing capacity of the pile after excavation decreases by 11.5% before excavation, which corresponds to a 6-mm displacement.

The predicted and measured bending moment profiles at two representative load levels ($H_0= 150$ kN, 270 kN) are shown in Fig. 16, and the agreement is also notably good for the $p$-$y$ curve after excavation. The prediction by the $p$-$y$ curve from the free field was underestimated along the entire pile length, and the maximum bending moments subjected to 150 kN and 270 kN were underestimated by 6.2% and 6.9%, respectively. The discrepancy between predicted and measured values increased with the increase in loads. However, the actual $p$-$y$ curve after excavation from the CPT data was well predicted compared to the measured curves. The maximum bending moment was located at approximately 4 m below the ground level. In addition, after excavation, the bending moments of the test pile increased, and the maximum bending moment more easily reached the limit cracking moment.

**Conclusion**

An actual full-scale instrument study in Jiangsu soil deposits was presented to investigate the excavation effects on the lateral pile response. The evaluation models direct from CPT data for the pile behaviour during and after excavations are proposed and verified. Based on the results, the following conclusions can be drawn:

1) The reaction of the cone tip resistance ($q_c$) corresponding to an excavation activity was obtained, and the horizontal unloading and vertical unloading effects were examined. The correctness of CPT profiles can provide a good premise to evaluate the lateral response of
2) The shortcomings of conventional $p$-$y$ models in lateral pile response to an excavation have been overcome. Based on the pre-excavation and post-excavation $p$-$y$ curves from CPT data, the attenuated $p$-$y$ model of single piles during excavation was proposed. The calculated pile deflection and bending moment were proven to be consistent with the measured results.

3) The use of post-excavation $p$-$y$ curves can generally provide a satisfactory prediction of the lateral pile behaviour after excavation. The $p$-$y$ curves from the free-field CPT data will induce a serious overestimate. In practice, the actual soil condition must be obtained using CPT after excavation to accurately predict the residual lateral bearing capacity of pile foundations.

The results of this paper indicate a basic framework for properly accounting for the effects of the excavation unloading on the behaviour of single piles from CPT data. Based on the developed attenuated $p$-$y$ model, the lateral pile response due to an adjacent excavation can be accurately evaluated. However, the tests are carried out on the basis of limited data for specific considered cases, and a broader set of studies are needed to enhance the findings from this study.

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References

Amirsoleymani, T. 1991. Elimination of excessive differential settlement by different
methods. Proceedings, the 9th Asian Regional Conference of Soil Mechanics and

subjected to lateral soil movement. Proc, 8th Int Conf on Computer Methods and
Advances in Geomechanics, 3: 2311-2316.


Decourt, L. 1991. Load-deflection prediction for laterally loaded piles based on N-SPT
values. Proceedings, 9th Pan-American Conference on Soil Mechanics and Foundation
Engineering, 549-556.

934-955.


12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering,
Cambridge, Mass., USA, Verlag Gluckauf GMBH. Essen (Germany), 2: 2215-2220.


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86x47mm (600 x 600 DPI)
Fig. 3. Plan layout of test piles and CPTs

Not disturbed area

Retaining wall (piles)

CPT-3 3 m 1 m 0.5 m 2.5 m
CPT-2

In-soil inclinometer

Pile 2

Excavation impact area 1.5 m

CPT-4

PLAN VIEW

Excavation line

Not to scale

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121x181mm (300 x 300 DPI)
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- **Silty sand** \( \gamma = 18.6 \text{ kN/m}^3 \), SPT N = 7, Poisson's ratio = 0.25

- **Muddy-silty clay** \( \gamma = 17.5 \text{ kN/m}^3 \), \( s_u = 23.7 \text{ kPa} \)
  \( c = 10.2 \text{ kPa} \), Poisson's ratio = 0.4

- **Silty sand & Silt** \( \gamma = 17.9 \text{ kN/m}^3 \), SPT N = 8, Poisson's ratio = 0.25

- **Silty clay** \( \gamma = 19.7 \text{ kN/m}^3 \), \( s_u = 39.2 \text{ kPa} \)
  \( c = 22.5 \text{ kPa} \), Poisson's ratio = 0.35

Penetration depth: 22 m

125x122mm (300 x 300 DPI)
Fig. 7. Results of cone tests before the excavation and the corresponding soil behaviour type (SBT) based on Robertson et al. (1986) classification method

123x101mm (300 x 300 DPI)
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<table>
<thead>
<tr>
<th>Pile type</th>
<th>Effective pile length $L$</th>
<th>Diameter $D$</th>
<th>Flexural rigidity $EI$</th>
<th>Steel reinforcement</th>
<th>Reinforcement ratio $\rho$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bored pile</td>
<td>30 m</td>
<td>1000 mm</td>
<td>$0.16 \times 10^7$ kN·m$^2$</td>
<td>16φ25 mm</td>
<td>0.95%</td>
</tr>
<tr>
<td>Bored pile</td>
<td>Elastic modulus of reinforcement $E_r$</td>
<td>Concrete grade C30</td>
<td>Protective thickness $d_p$</td>
<td>Elastic modulus of concrete $E_c$</td>
<td></td>
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<tr>
<td></td>
<td>$2.0 \times 10^8$ kN/m$^2$</td>
<td></td>
<td>50 mm</td>
<td>$3 \times 10^7$ kN/m$^2$</td>
<td></td>
</tr>
</tbody>
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