

CPT-based dynamic p-y analysis method for seismic design of piles embedded in clay

Garam Kim^{1a}, Jiyeong Lee^{2b}, Jonghyeog Yoon^{2c}, Qaisar Abbas^{2d} and Junhwan Lee^{*2}

¹Research Institute of Korea Electric Power Corporation, Naju 58263, Republic of Korea

²School of Civil and Environmental Engineering, Yonsei University, 50 Yonsei-ro, Seodaemun-gu, Seoul, Republic of Korea

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Abstract. In this study, a dynamic p-y curve method for the seismic design of laterally loaded pile in clay was proposed focusing on the direct application of the cone penetration test (CPT) result. Key motivation of this study was to fully utilize the continuous and in-situ profiling capability of CPT, which was particularly effective for multi-layered, heterogeneous soil conditions and offshore environment. The exerted dynamic load response of pile was expressed and obtained by introducing the static stiffness and damping of clay into the dynamic p-y curve function, both formulated as a function of the cone resistance. The proposed CPT-based dynamic p-y curve function was employed in the pseudo-static analysis based on an equivalent static-load approach. A calculation algorithm was prepared to implement the proposed method, following the procedure of the pseudo-static analysis. Case examples were selected, including centrifuge tests and field load tests, and adopted to compare measured and predicted dynamic pile load responses. The compared results of the case examples confirmed that the proposed method was effective and beneficial for the seismic design of piles in clay.

Keywords: clay; cone penetration test; pseudo-static analysis; p-y analysis; seismic design of pile

1. Introduction

The seismic stability of piles is an important design consideration to ensure the sustainable performance of structure during lifetime including unexpected event of earthquake. Common approaches for the seismic design of pile include pseudo-static analysis, response spectrum analysis, and time-history analysis (ISO 2004, ICC 2012). Among these, the pseudo-static analysis is widely used and popular in practice due to the simplicity in input and calculation algorithm, as well as the compatibility to the conventional static design methods (Tabesh and Poulos 2001, Zhang *et al.* 2019, Mitra *et al.* 2021, Tott-Buswell *et al.* 2022). In the pseudo-static analysis, dynamic loads are converted into an equivalent static load, which is then used for load-transfer and displacement analysis, ensuring that an induced displacement falls within tolerable range.

For the displacement analysis of laterally loaded piles, the p-y curve method based on the beam-on-elastic foundation model is frequently adopted. In this method,

soils are modeled as a series of nonlinear springs, for which the load response is defined using a functional relationship between lateral soil resistance (p) and induced displacement (y). Several p-y curve functions have been proposed (Matlock 1970, Reese *et al.* 1975, Dunnavant and O'Neill 1989, API 2010, Lombardi *et al.* 2017, Zhang *et al.* 2020). For clays, the p-y curve functions by Matlock (1970) and Reese *et al.* (1975) have gained popularity, with the undrained shear strength (s_u) being a key input parameter. The accuracy and reliability of calculated results are largely dependent on the s_u profile adopted in the analysis. The determination of s_u requires a series of experimental procedures including sampling, testing and measurement interpretation, all subjected to various uncertainties.

The direct application of in-situ testing method is an effective approach for foundation design and p-y analysis, offering reduced experimental uncertainties (Titi *et al.* 2000, Tumay and Kurup 2001, Lee and Randolph 2011, Mir *et al.* 2017). Lee *et al.* (2010) proposed a mechanical relationship between the lateral soil resistance around laterally loaded pile and the cone resistance (q_c) obtained from the cone penetration test (CPT), assuming that both are influenced by the horizontal effective stress. Several CPT-based p-y analysis methods have been proposed, where the p-y curve is defined as a function of q_c for both static and cyclic loading conditions (Suryasentana and Lehane 2014, 2016, Kim *et al.* 2016, Boufia 2017; Li *et al.* 2017, Liu *et al.* 2021, Kementzetzidis *et al.* 2022). However, little attention has been given to the direct application of q_c to establish the dynamic p-y curve for the seismic design of pile (Suryasentana and Lehane 2016, Kim and Lee 2022, Kementzetzidis *et al.* 2022, Tott-Buswell *et al.* 2022). For establishing the dynamic p-y curve, it is important to consider dynamic properties such as the elastic

*Corresponding author, Professor

E-mail: junlee@yonsei.ac.kr

^aSenior Researcher, Ph.D.

E-mail: garamkim@kepco.co.kr

^bPh.D. Student

E-mail: jiyeong12@yonsei.ac.kr

^cPh.D. Student

E-mail: yjh97@yonsei.ac.kr

^dPh.D. Student

E-mail: abbasqaisar935@gmail.com

modulus and damping of soil. These properties can be expressed using a complex stiffness that is comprised of static stiffness and damping components (Kim *et al.* 2022, Tott-Buswell *et al.* 2022). The direct application of q_c into the p-y method can be particularly beneficial for offshore piles, as sampling and testing procedures are often limited in offshore environments.

The pseudo-static analysis based on the dynamic p-y curve considers wave propagation and energy dissipation within soil to capture the dynamic response of pile, influenced by soil nonlinearity and damping. For the dynamic p-y curve, mobilized lateral soil resistance is determined by dynamic stiffness, which encompasses both static stiffness and damping (Novak 1991, El Naggar and Bentley 2000, Varun *et al.* 2009, Yang *et al.* 2011, Hussein and El Naggar 2021). The static stiffness is given by static p-y curve, and damping is determined by damping coefficient and load frequency. While s_u governs the dynamic p-y analysis for clay in similar manner to the static p-y analysis, the CPT-based dynamic p-y method for clay was not yet specifically established due to the lack of direct correlation between q_c and dynamic p-y function parameters.

In this study, a dynamic p-y curve method is proposed for the seismic design of piles in clay, focusing on the direct application of q_c from CPT. It is aimed at developing a unified design procedure based on in-situ test results, to eliminate multi-step experimental and interpretation process required in the conventional approach. The dynamic load response of pile is described by introducing static stiffness and damping of clay based on the framework of the dynamic p-y curve. The p-y curve function is formulated as a function of the cone resistance to fully utilize the continuous and in-situ profiling capability of CPT. A calculation algorithm is developed to implement the proposed method following the procedure of pseudo-static analysis.

2. Methods of p-y analysis for laterally loaded piles in clay

2.1 Static p-y curve

For the displacement analysis of laterally loaded piles, the p-y analysis based on the beam-on-elastic foundation model is widely adopted where soils are assumed as a series of discrete springs. The load-displacement response is then specified by the p-y curve that is given as a certain functional expression. Various p-y curve models were proposed for piles in clay under static and cyclic loading conditions (Matlock 1970, Reese *et al.* 1975, Dunnavant and O'Neill 1989). The main components that characterize the p-y curve are the ultimate lateral soil resistance (p_u) and the shape function of the curve. p_u represents the soil resistance exerted by surrounding soil and can be obtained using the undrained shear strength and the bearing capacity factor given as the following relationship:

$$p_u = N_c \cdot s_u \cdot D \quad (1)$$

where s_u = undrained shear strength; N_c = bearing capacity factor; and D = pile diameter. N_c is an empirical parameter

that increases linearly with depth and becomes constant below certain limit depth. Matlock (1970) and Reese *et al.* (1975) suggested the following equations of N_c for soft and stiff clays, respectively

$$N_c = 3 + \frac{\gamma'_z}{s_u} + \frac{J_z}{D} \leq 9 \quad (2)$$

$$N_c = 2 + \frac{\gamma'_z}{s_{ua}} + \frac{2.83z}{D} \leq 11 \quad (3)$$

where s_{ua} = average undrained shear strength; γ' = effective unit weight of soil; z = depth; and J = empirical parameter equal to 0.5 and 0.25 for soft and stiff clays, respectively.

For piles embedded in clay, the p-y curve function proposed by Matlock (1970) has been often used in practice as adopted in various design specifications (API 2010, DNV 2013) given as follows

$$\frac{p}{p_u} = 0.5 \left(\frac{y}{y_{50}} \right)^{1/3} \quad (4)$$

where p = lateral soil resistance per unit length; p_u = ultimate lateral soil resistance; y = lateral displacement of pile; y_{50} = reference lateral displacement = $2.5 \varepsilon_{50} D$; and ε_{50} = strain value at 50% of failure stress in triaxial test. The p-y curve of Eq. (4) by Matlock (1970) is shown in Fig. 1(a). Reese *et al.* (1975) also proposed the p-y curve for piles in stiff clay as shown in Fig. 1(b). In the p-y curve by Reese *et al.* (1975), softening after peak was considered, and the curve was divided into five piecewise sections that were formulated with functions for each section. For all these methods, the characterized values and depth profile of the soil strength parameter are crucial to obtain the proper and reliable prediction for the lateral load response of pile.

2.2 Dynamic p-y curve

The dynamic p-y curve is defined by the dynamic or complex stiffness, expressed given as the following relationship

$$p_d = K_d \cdot y \quad (5)$$

where p_d = dynamic lateral soil resistance; K_d = dynamic stiffness; and y = lateral displacement of pile. K_d consists of static stiffness and damping components given as follows (Gazetas 1991, Novak 1991, El Naggar and Bentley 2000)

$$K_d = K_1 + i \cdot K_2 = K + i \cdot \omega \cdot C \quad (6)$$

where K_1 and K_2 = real and imaginary parts of complex stiffness K = true stiffness; i = imaginary unit; ω = angular load frequency = $2\pi f$; f = load frequency; and C = damping coefficient. The true stiffness, K , given by the real part of the complex stiffness represents the static stiffness and thus the same as that for the static p-y curve. The imaginary part represents the energy dissipation by wave propagation from pile and damping.

The dynamic effect within soil is described using a series of springs and dashpots placed in a parallel fashion as shown in Fig. 2. According to El Naggar and Bentley (2000), the inner soil zone near pile is mainly related to the soil nonlinearity while the outer zone allows damping

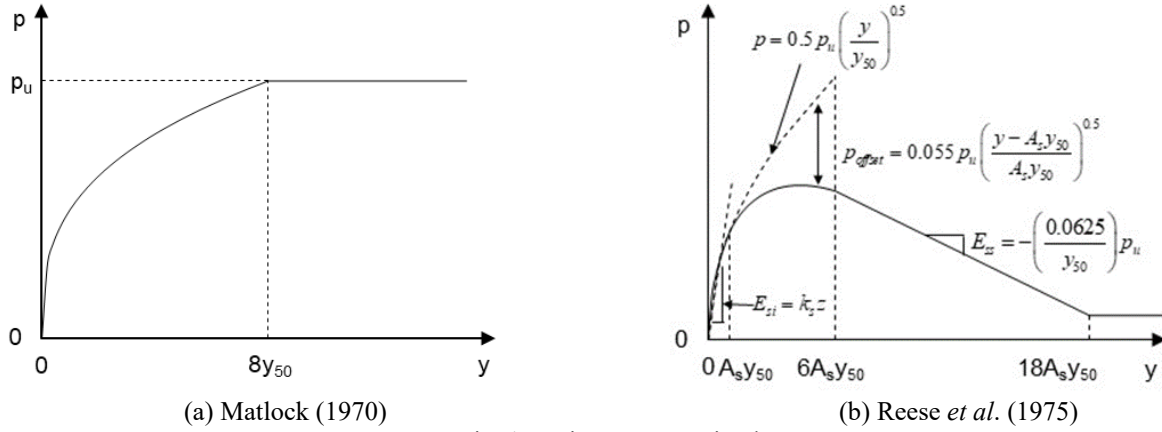


Fig. 1 Static p-y curves in clay

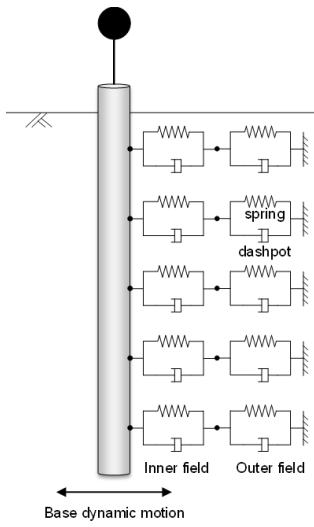


Fig. 2 Spring-dashpot model for pile-soil response under dynamic loading condition

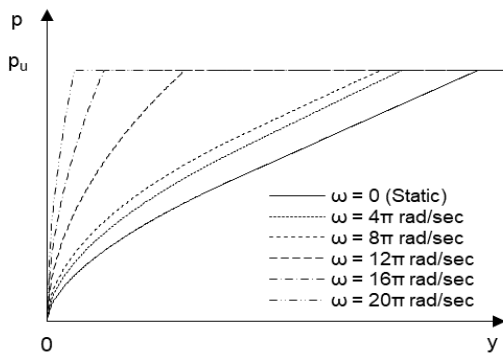


Fig. 3 Dynamic p-y curves with angular frequency (El Naggar and Bentley 2000)

through the soil medium. In general, the dominant component for the seismic design of pile is the radiation damping that is caused by reduced kinetic energy through the process of wave-energy discharging.

For flexible piles embedded in a homogeneous soil, K and C in Eq. (6) can be expressed given as follows (Gazetas 1991)

$$K = DE_s \left(\frac{E_p}{E_s} \right)^{0.21} \quad (7)$$

$$C = DE_s \left(\frac{E_p}{E_s} \right)^{0.21} \left[1.6 \frac{\beta}{\omega} + 0.35D \left(\frac{E_p}{E_s} \right)^{0.17} V_s^{-1} \right] \quad \text{for } \omega > \omega_s \quad (8)$$

where D = pile diameter; E_s and E_p = elastic moduli of soil and pile, respectively; β = damping of soil; $\omega_s = (\pi/2)V_s/H$; V_s = shear wave velocity of soil; and H = soil layer thickness.

El Naggar and Bentley (2000) proposed a dynamic p-y curve as the following equation

$$p_d = (K_1 + iK_2)y = p_s \left[c_1 + c_2 a_0^2 + c_3 a_0 \left(\frac{\omega y}{D} \right)^{c_4} \right] \leq p_u \quad (9)$$

where p_s = static lateral soil resistance; p_u = ultimate lateral soil resistance; a_0 = dimensionless frequency = $\omega r/V_s$; r = pile radius; V_s = shear wave velocity of soil; and c_1 , c_2 , c_3 , and c_4 = curve fitting parameters. c_1 was taken as unity to provide p_d equal to p_s for ω of 0. As indicated by Eq. (9), the resistance response that stiffens with increasing load frequency can be considered. The characteristic shape of the dynamic p-y curve given by Eq. (9) with ω is shown in Fig. 3. As ω increases, a stiffer lateral load repose of pile is obtained. Note that the p-y curve for $\omega = 0$ in Fig. 3 indicates the static p-y curve.

Varun et al. (2009) also proposed the dynamic p-y curve with K_d given as follows

$$p = K_d y = [K_s \cdot k'(a_0) + i \cdot a_0 \cdot C(a_0)] y \quad (10)$$

where K_s = static stiffness; $k'(a_0)$ = frequency-dependent stiffness coefficient; $C(a_0)$ = frequency-dependent damping coefficient; and a_0 = dimensionless frequency = $\omega D/V_s$. In Eq. (10), C is expressed as

$$C = 1.85 \cdot a_0 \cdot E_s \quad \text{for } a_0 < 1 \quad (11)$$

$$C = 1.85 \cdot E_s \quad \text{for } a_0 > 1 \quad (12)$$

where E_s = elastic modulus of soil. Eqs. (11) and (12) were functions fitted to represent the increase and convergence of the radiation damping coefficient with a_0 .

3. Proposed dynamic p-y analysis method using CPT

3.1 Description of CPT-based p-y curve

For laterally loaded piles in clay, the ultimate lateral soil resistance for the p-y curve is obtained given as a function of the undrained shear strength (s_u). In this study, to evaluate s_u from CPT results, the following q_t - s_u correlation proposed by Lee *et al.* (2010) was adopted

$$s_u = \frac{q_t - u_0}{N_e} = \frac{q_e}{N_e} \quad (13)$$

where q_t = cone resistance corrected with area ratio for saturated clay; u_0 = hydrostatic pore pressure; q_e = effective cone resistance = $q_t - u_0$; and N_e = effective cone factor = 16 ± 3 . Among several methods to evaluate s_u using CPT results (Vesic 1972, Baligh 1975, Teh and Houlsby 1991, Yu *et al.* 2000), Eq. (13) was selected as it can take full advantage of the direct application of CPT. As indicated by Eq. (13), q_t and u_0 are only required to obtain s_u , both of which can directly be obtained from CPT. In most other correlations, an in-situ stress needs to be additionally identified and adopted into the estimation procedure of s_u , which lowers the effectiveness of direct CPT application.

Based on the q_t - s_u correlation of Eq. (13), Kim *et al.* (2014) proposed the normalized static p-y curve function for piles in clay given as follows

$$p = 0.5 \frac{N_c}{N_e} q_e D \left(\frac{y}{y_{50}} \right)^{1/3} \quad (14)$$

where y_{50} = limit displacement = $\varepsilon_{50} \cdot D$ and D = pile diameter. As given by Eqs. (2) and (3) for N_c , the values of N_c/N_e vary with depth to the limit depth and converge to 0.5625. The limit strain, ε_{50} , for y_{50} in Eq. (14) is obtained as

$$\varepsilon_{50} = 0.185 \left(\frac{q_e}{p_A} \right)^{-1.124} \leq 0.02 \quad (15)$$

where p_A = reference pressure = 100 kPa. Eq. (15) was derived based on the values of ε_{50} recommended in the Matlock method.

3.2 Formulation of dynamic p-y curve function

For the dynamic load response, both static stiffness and damping should be taken into account, as given by the complex stiffness of the p-y function in Eq. (5). In this study, the CPT-based p-y function of Eq. (14) was adopted to describe the true stiffness in the real part of the complex stiffness. For the damping component in the imaginary part of the complex stiffness, Eqs. (11) and (12) proposed by Varun *et al.* (2009) were adopted, which can reflect the effect of load frequency on the dynamic response of pile.

There are two important soil parameters to obtain the damping coefficients (C) in Eqs. (11) and (12); the elastic modulus of soil (E_s) and dimensionless frequency (a_0). The a_0 represents to the load frequency (ω) normalized with the shear wave velocity of soil (V_s). Both E_s and V_s can be correlated to and obtained as a function of the cone

resistance (q_t). In this study, the following correlation between V_s and q_t proposed by Mayne and Rix (1995) was adopted

$$V_s = 1.75 \cdot q_t^{0.627} \quad (16)$$

where q_t = cone resistance in kPa. Note that Eq. (16) was originally suggested for q_c based on the regression of data for V_s and q_c while 87% of the data were for the corrected cone resistance, q_t . For E_s , the following relationship proposed by Bowles (1996) was adopted

$$E_s = N_s \cdot s_u \quad (17)$$

where N_s = correlation parameter in the ranges of 100 to 500 and 500 to 1500 for soft and stiff clays, respectively. As s_u can be expressed as a function of q_t from Eq. (13), E_s of Eq. (17) becomes

$$E_s = \frac{N_s}{N_e} q_e \quad (18)$$

where N_e = effective cone factor = 16 ± 3 ; q_e = effective cone resistance = $q_t - u_0$; and u_0 = hydrostatic pore pressure.

Using the formulations of V_s and E_s with q_t , the damping coefficient (C) of Eqs. (11) and (12) can be rewritten as

$$C = 1.85 \left(\frac{\omega D}{1.75 q_t^{0.627}} \right) \frac{N_s}{N_e} q_e \quad \text{for } a_0 < 1 \quad (19)$$

$$C = 1.85 \frac{N_s}{N_e} q_e \quad \text{for } a_0 > 1 \quad (20)$$

This yields that the CPT-based dynamic p-y function is established given as follows

$$p_d = \left[\frac{0.5 \frac{N_c}{N_e} q_e D \left(\frac{y}{y_{50}} \right)^{1/3}}{y} + i 1.85 \left(\frac{\omega D}{1.75 q_t^{0.627}} \right)^2 \frac{N_s}{N_e} q_e \right] y \quad \text{for } a_0 < 1 \quad (21)$$

$$p_d = \left[\frac{0.5 \frac{N_c}{N_e} q_e D \left(\frac{y}{y_{50}} \right)^{1/3}}{y} + i 1.85 \left(\frac{\omega D}{1.75 q_t^{0.627}} \right) \frac{N_s}{N_e} q_e \right] y \quad \text{for } a_0 > 1 \quad (22)$$

where y_{50} = limit displacement = $\varepsilon_{50} \cdot D$; D = pile diameter; ε_{50} = limit strain of Eq. (15); ω = load frequency; N_c/N_e = cone factor ratio; N_s = correlation parameter of Eq. (17); and q_t and q_e = measured and effective cone resistances, respectively.

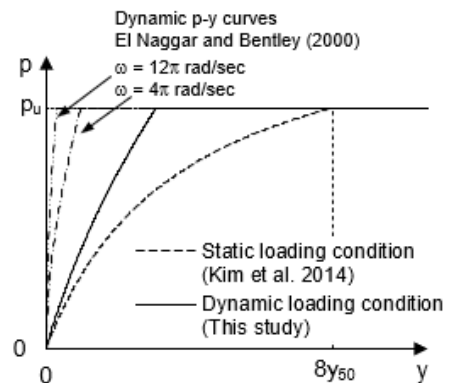


Fig. 4 Comparison of static and dynamic p-y curves in clay

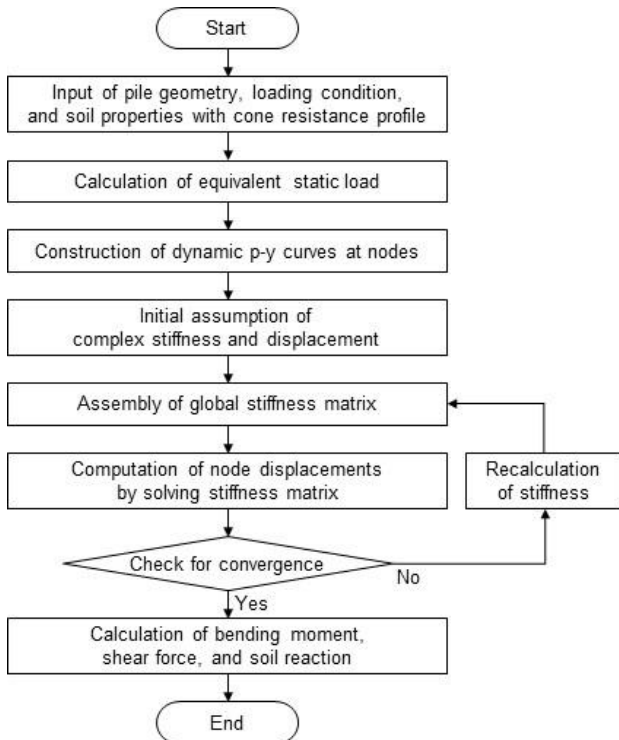


Fig. 5 Calculation algorithm for proposed CPT-based p-y analysis

The characteristic shape of the CPT-based dynamic p-y curve of Eqs. (21) and (22) proposed in this study is shown in Fig. 4, compared with the static p-y curve. Additionally, dynamic p-y curves from El Naggar and Bentley (2000) were also included in Fig. 4. Note that, if ω is equal to 0 in Eqs. (21) and (22), the p-y curve becomes the static p-y curve of Eq. (14). It is seen that the dynamic p-y curve shows a stiffer load response than the static p-y curve, due to the effect of damping.

3.3 Calculation algorithm for proposed dynamic displacement analysis

A calculation algorithm for the proposed CPT-based dynamic p-y analysis is presented in Fig. 5 where the procedure for the pseudo-static analysis based on an equivalent static load was considered. In this procedure, the equivalent static load is first evaluated for input into the p-y analysis. The equivalent static load can be obtained using inertial force from the superstructure or the seismic response coefficient given in the seismic design specifications (Bradley 2012, ASCE 2013, Charney 2015). The subsequent procedure of the analysis is then similar to that for the static p-y analysis.

The properties of pile and soil are required to construct the p-y curve at nodes that are assigned along the pile. For this purpose, the depth profile of the cone resistance is directly introduced and employed as main input for the p-y analysis. In this step, the p-y formulations of Eqs. (21) and (22) are utilized to obtain the dynamic load response of pile.

The global stiffness matrix of the pile is then assembled, including the dynamic stiffness that consists of static

stiffness and damping component. The lateral displacement of the pile is estimated iteratively by solving the global stiffness matrix. Once a calculated lateral displacement is determined by convergence, the bending moment, shear force, and soil reaction force are subsequently calculated.

4. Comparison and validation

4.1 Case example with centrifuge tests

To check the validity of the proposed CPT-based dynamic p-y analysis method, a case example was selected from the literature and adopted to compare with predicted results using the proposed method. The selected example was centrifuge tests by Boulanger *et al.* (1999), conducted at a centrifugal acceleration of 30g. The schematic view of the centrifuge test layout is shown in Fig. 6(a). The test was carried out in a flexible shear beam container of $0.7 \times 1.7 \times 0.7$ m, adopted to minimize the boundary effect under a dynamic loading condition.

The test model pile was made of aluminum, on the top of which a lumped mass was placed to consider the load of superstructure. In the prototype scale, the model pile represented a steel pipe pile with the diameter (D) of 0.67 m and the flexural rigidity ($E_p I_p$) of 417 MN·m². The test pile was embedded to the depth of 16.8 m in the prototype scale.

The upper mass of 49.1 Mg was attached to the pile at 3.81 m above the ground surface. An earthquake motion was applied at the base of the box, which was corresponding to the scaled version of Kobe earthquake recorded from Port Island in Hyogoken-Nambu, Kobe in 1995. Three input peak accelerations (a_{max}) of 0.016 g, 0.055 g and 0.20 g were adopted in the tests. The example of input motion for the case of $a_{max} = 0.20$ g is shown in Fig. 6(b).

The test soil specimens were prepared in two layers with the upper clay and lower sand layers. The upper layer was soft clay placed down to 6.1 m with the effective unit weight (γ') of 5.2 kN/m³. The lower sand layer was prepared at the relative density (D_R) and the effective unit weight (γ') of 75% to 80% and 10.1 kN/m³, respectively. The depth profile of s_u obtained from the torvane test is shown in Fig. 7. As CPT was not conducted in the centrifuge tests, the q_t profile was obtained from the s_u profile in Fig. 7 using Eq. (13) with the cone factor equal to 16. The depth profiles of s_u and q_t are shown together in Fig. 7.

For the case example in Figs. 6 and 7, the dynamic p-y analysis was performed through the calculation algorithm given in Fig. 5. The equivalent static loads (F_{eq}) were 24.1 kN, 67.4 kN, and 173.4 kN for the values of a_{max} equal to 0.05g, 0.14g, and 0.36g measured at the superstructure with the applied input base accelerations of $a_{max} = 0.016g$, 0.055g, and 0.20g, respectively. These values of F_{eq} were applied at the pile head, 3.81 m above the ground surface. For the pile zone embedded in the lower sand layer, the p-y curve of Eq. (9) proposed by El Naggar and Bentley (2000) was assigned.

The depth profiles of the bending moments were obtained using the proposed CPT-based dynamic p-y

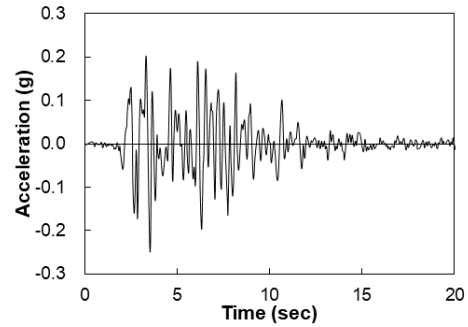
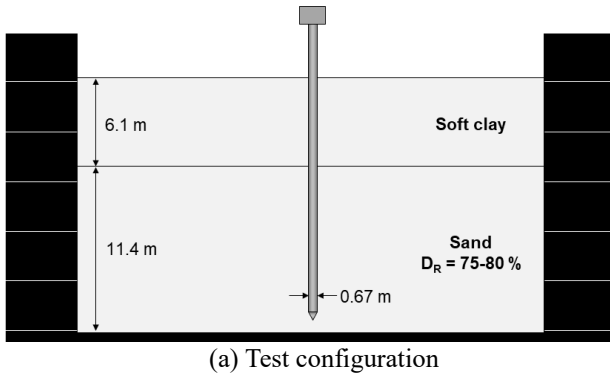


Fig. 6 Description of centrifuge test

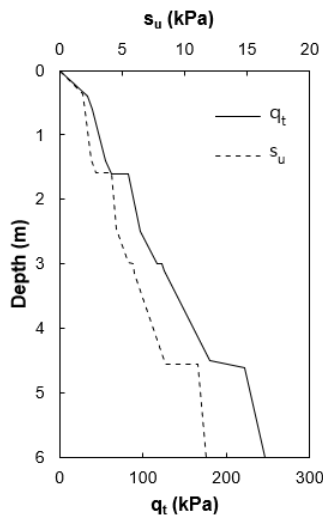


Fig. 7 Depth profiles of undrained shear strength and cone resistance for centrifuge test

analysis for each of $a_{max} = 0.016$ g, 0.055 g and 0.20 g, and plotted in Fig. 8(a). The measured results were also included and compared in Fig. 8(a). The measured values shown in Fig. 8(a) were those obtained when the maximum bending moment occurred. It was observed that the depth profiles of calculated bending moment using the proposed method showed close match to those measured for all the cases whereas slightly underestimated profile was observed for the case of $a_{max} = 0.020$ g.

Measured and predicted lateral displacement profiles with depth were also obtained and compared in Fig. 8(b). It was observed that the predicted lateral displacements using the proposed method showed good agreement with measured results for the cases of $a_{max} = 0.016$ g and 0.055 g. For the case of $a_{max} = 0.020$ g, on the other hand, the predicted displacements showed somewhat underestimated results compared to the measured results. Nonetheless, the results shown in Fig. 8 confirmed that the proposed method is sufficiently effective producing overall close match to measured results.

4.2 Comparison with field test results

The field dynamic pile load test of Blaney and O'Neill (1986) was selected and adopted to further compare

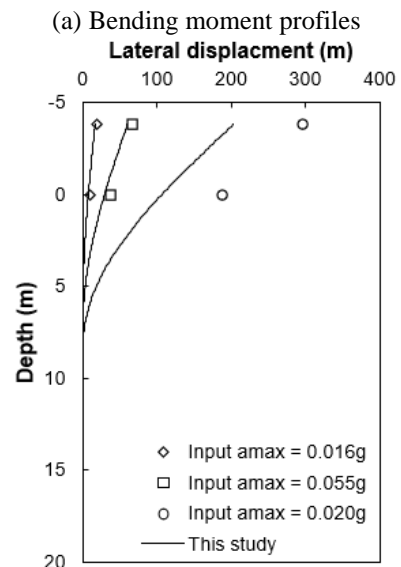
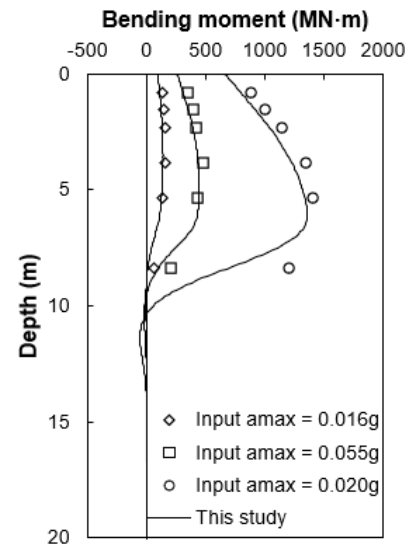


Fig. 8 Comparison of measured and predicted results

measured and predicted results. The test site was located in Houston, TX, US. The depth profile of CPT cone resistance at the test site is shown in Fig. 9, which was obtained from Mahar and O'Neill (1983). The soil at the test site was a stiff clay with the undrained shear strength (s_u) up to 380

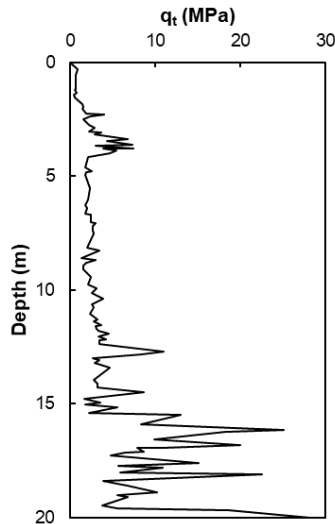


Fig. 9 Depth profile of CPT cone resistance at test site

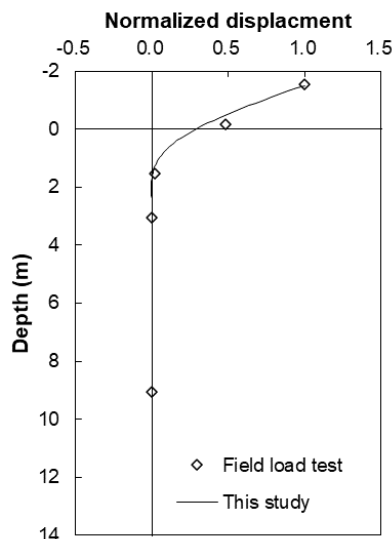


Fig. 10 Comparison of measured and predicted displacement profile for field test case

kPa within the embedded pile length of 13.4 m. The saturated unit weights (γ_{sat}) of the soil were 19.8 kN/m^3 down to 8.3 m below the ground surface and 20.9 kN/m^3 for depths from 8.3 to 14.6 m. The test pile was steel pipe pile with an outer diameter (D) of 0.273 m and the wall thickness of 9.27 mm. An upper mass of 61.52 kN was attached to the pile head, and the center of gravity was 2.21 m above the ground surface. For the test pile, sweep tests were conducted with the frequency from 15 Hz to 1 Hz.

The dynamic p-y analysis was conducted using the q_t profile shown in Fig. 9. The equivalent static load (F_{eq}) was 2.67 kN, which was the maximum sweep load observed from the field load test. The displacement profile calculated from the proposed method was obtained and compared with the measured results in Fig. 10. For comparison, the normalized displacement was adopted, which was indicated as a ratio of lateral displacement to that obtained at the pile head.

The measured displacement in Fig. 10 was the peak displacement obtained at the resonant frequency of 2.2 Hz. As shown in Fig. 10, the predicted results using the proposed p-y analysis were in close agreement with the measured results, showing the similar range of the active pile length of 1.5 m below the ground surface. The results in Fig. 10 again confirms the effectiveness of the proposed CPT-based analysis for the seismic design of piles in clay.

5. Conclusions

In this study, a CPT-based dynamic p-y curve method for the seismic design of laterally loaded piles in clay was proposed. The dynamic p-y curve function for the proposed method was formulated as a function of the cone resistance to fully utilize the continuous and in-situ profiling capability of CPT. It is supposed that the proposed approach is particularly effective and beneficial for piles embedded in multi-layered, heterogeneous soil conditions and offshore environments.

The dynamic load response of pile was considered by introducing the static stiffness and damping of clay into the dynamic p-y curve function. Both the static stiffness and damping were defined as a function of the cone resistance. The dynamic p-y curve was formulated with the dynamic stiffness using the complex formulation of real and imaginary parts corresponding to the static stiffness and damping, respectively. The static stiffness was derived from the CPT-based static p-y curve with the effective cone factor, and damping was given by radiation damping based on the correlation of the cone resistance to the elastic modulus and shear wave velocity.

The proposed CPT-based dynamic p-y curve was incorporated into the pseudo-static analysis where an equivalent static load was applied. The dynamic p-y curve yielded a stiffer load response than the static p-y curve due to damping. A calculation algorithm was established following the procedure of the pseudo-static analysis. The properties of the pile and soil were necessary to construct the dynamic p-y curve at different depths, which was defined by utilizing directly input q_c profile.

Case examples were selected, including centrifuge tests and field load tests, and adopted to compare measured and predicted dynamic load responses of piles. The measured and predicted dynamic load responses of the piles were in close agreement for both cases. The results of the case examples demonstrated that the proposed CPT-based dynamic p-y method effectively captured the increase in lateral soil resistance with depth and was suitable for the seismic design of piles in clay. As the proposed procedure is valid in clay, when mixed soil types of clay and sand exist, additional incorporation of a method valid for sand should be additionally introduced.

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