Analytical approach to analysis of piles in liquefied sand using design geotechnical parameters predicted from field pile load tests

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ABSTRACT

Liquefaction can lead to significant, often permanent, lateral or vertical deformations, imposing substantial kinematic loads on pile foundations. This results in a reduced capacity of the piles to resist vertical or lateral loads. The simplified design and analysis of laterally loaded piles are commonly conducted using the nonlinear "Winkler foundation approach." In this method, lateral soil-pile interaction is modelled using arrays of nonlinear springs, known as p-y curves. The development of CPT-based p-y curves has gained momentum in recent years. However, only a few of these models consider effective stress in their structure, and their ability to account for the liquefied state of soil has not been thoroughly investigated. In this study, a simplified method is introduced to back-calculate pile responses from large-scale pile tests conducted by GIKEN LTD. under various liquefaction states. Using the results from Borehole Pressure Meter tests, an initial framework for modifying existing CPT-based models was introduced. This framework was then implemented in a 1D-FEM analysis to assess the performance of each model against the results of large-scale experimental tests. The results demonstrate a good agreement between the outcomes obtained from 1-D nonlinear beam analysis using the Winkler model and the experimental data.

Key words: *p-y curves, Winkler Model, Liquefaction, Borehole pressuremeter, 1D FEM analysis*

1. Background and objective

1.1. Background

Earthquake induced liquefaction can cause large lateral deformations which exerts significant kinematic loads on horizontally and vertically loaded piles. Sudden buildup of excess pore water pressure leads to the decrease in shaft friction, tip bearing capacity, or lateral subgrade reaction, leading to a reduction in the pile's capacity. This could eventually lead to structural failure of piles under different mechanism of bending, shear, buckling or large subsidence (Haldar and Babu, 2010; Kheradi et al., 2019).

Existing codes (Japan Road Association Code, NEHRP, and Eurocode 8) of practice considers simplified procedure for design and analysis of piles in liquefied soil considering partial factors on load, materials and focused on bending moments on piles considering non liquified soil applies passive earth pressure to the pile while liquified layer only exerts up to 30% of drag load to soil.

To ensure the structural integrity and safety of pile-supported structures, it is imperative to appropriately consider the effects of liquefaction in their design. This consideration is crucial to prevent unprecedented failures, especially in seismic events.

The soil-structure interaction of pile-supported structures subjected to earthquake-induced liquefaction has been shown to be influenced by material and geometrical nonlinearity. Simplified procedures for designing pile foundations under lateral and vertical loads, such as the limit equilibrium methods (Dobry and Abdoun, 2001), and the nonlinear Winkler foundation method, have been practiced for a long time. In this method, lateral soil-pile interaction is modeled using arrays of nonlinear springs, known as p-y curves.

Several p-y curves have been introduced and are well understood for non-liquefied soil conditions. Some of the most commonly used p-y models include the American Petroleum Institute (API)'s sand model (API, 2010) and Matlock's model for clay (Matlock, 1970). A notable challenge in implementing the API model involves determining the physical and mechanical properties of soils, particularly internal friction angles and cohesion, based primarily on correlations using results from Standard Penetration Test (SPT) N values.

Only in recent years has the development of Cone Penetration Test (CPT)-based p-y curves, which rely on cone resistance, gained momentum. Among these, only a few were derived from full-scale pile load tests, and the rest through Finite Element Method (FEM) analysis. However, most of these equations are developed for mainly static or dynamic conditions with reduced resistance.

The impact of liquefaction on soil-pile interaction and the evolution of p-y curves have yet to be thoroughly investigated. Moreover, the influence of liquefaction on soil-structure interaction, soil reaction forces, and CPT resistances is not as well understood as the p-y backbone curves suggested by various researchers for non-liquefied conditions. Furthermore, only a few existing p-y curves include effective stresses or the effective saturated unit weight in their calculations, and their ability to account for the liquefied state of soil has not yet been thoroughly investigated. Therefore, it is necessary to evaluate the performance of existing CPT-based p-y models and their ability to accommodate the liquefaction state. Additionally, the effect of excess pore water pressure on p-y curves needs to be explored.

1.2. Objective and framework of study

In this study, a CPT-based p-y analysis method is proposed for the seismic design of piles in liquefiable sand. The objective is to establish an effective seismic design method for laterally and vertically loaded piles by incorporating physical model test results into the design process. Initially, the force-deformation response of the pile is determined from large-scale tests, and Borehole Pressuremeter Test (BPT) results are computed. p-y curves are back-calculated from pile responses under different liquefaction states.

These results are then used to assess the performance of various existing API and CPT-based p-y models. A modification to p-y models is proposed to include the effect of liquefaction on soil-pile interaction. Subsequently, the p-y models are integrated into a 1D Finite Element Method (FEM) to simulate the response of large-scale model tests. Further modifications are applied to the proposed p-y models to refine their accuracy.

2. Testing program and site condition

2.1. Testing program

A series of horizontal load tests, borehole pressuremeter tests, and CPT tests were carried out at GIKEN's Kochi trial site, mainly for the purpose of achieving the geotechnical parameters of liquefied ground (Toda et al. 2024).

The schematic view of the soil tank, as shown in liquefaction testing apparatus in the **Fig. 1**, reveals that the tank is equipped with a water supply system. This system allows water to flow from the bottom of the tank through a network of pipes, equipped with nozzles, at a certain flow rate. This flow rate correlates with the upward hydraulic gradient and the degree of soil liquefaction. Silica sand NSK-40, with a mean grain size (D50) of 0.23 mm, was used to fill the tank, layered over a gravel layer that served as a filter. The physical properties of the soil are detailed in Table 1.

For the experiments, a closed-end box-shaped pile, measuring 300×300 mm (refer to **Table 2**), was employed. This pile was outfitted with strain gauges, earth pressure sensors, inclinometers, and pore water pressure gauges along its length.

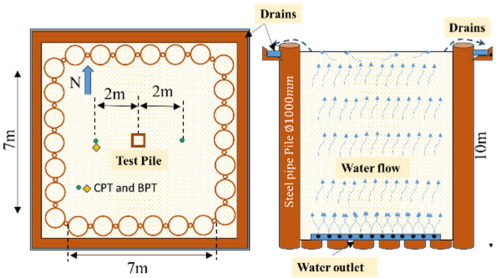


Fig. 1. Schematic view of liquefaction testing apparatus

LVDT sensors were installed at two different elevations directly above the loading position to measure the horizontal displacement of the pile. The horizontal load tests were conducted under three different liquefaction states, characterized by varying Ru=$\Delta u/\sigma'_{v0}$ values. Where Δu is the excess pore water pressure, σ'_{v0} symbolizes the effective stress. **Fig. 2** illustrates the test pile and its instrumentation.

Table 1. Physical properties of the soil

Soil Type	Maximum Void ratio (e_{max})	Minimum Void Ratio(e_{min})	Saturated unit weight (Υ_{sat})	Wet Unit weight (Υ_{wet})	Poisson's ratio (ν)
	(-)	(-)	(kN/m³)	(kN/m³)	(-)
NSK-40 Sand	0.891	0.531	18.331	14.275	0.300

Table 2. Physical properties of steel pipe pile

Cross section	Material	Young modulus (E)	Poisson's ratio (ν)	Section area (A)	Second moment of inertia (I)
		(kN/m²)	(-)	(m²)	(m⁴)
Box Shape (H300×B300)	Steel	2.06E+08	0.300	1.171E-02	1.837E-04

Additionally, a series of borehole pressuremeter tests were conducted at a depth of 2m under different liquefaction states. Furthermore, a series of cone penetration tests were carried out on the east side, west side, and center of the soil tank to evaluate the ground condition under various liquefaction states. The summary of these tests is presented in **Table 3**.

2.2. Ground condition and Testing procedure

In order to achieve homogeneous ground conditions, the following procedure was adopted before conducting the tests. Initially, water was flowed from the bottom of the tank at a certain flow rate. This was done to ensure that the hydraulic gradient reached the critical hydraulic gradient, causing the soil to start liquefying (Ru = 1). Subsequently, Cone Penetration Tests (CPT), Borehole Pressuremeter Test (BPT), and Horizontal Load Tests (HLT) were conducted.

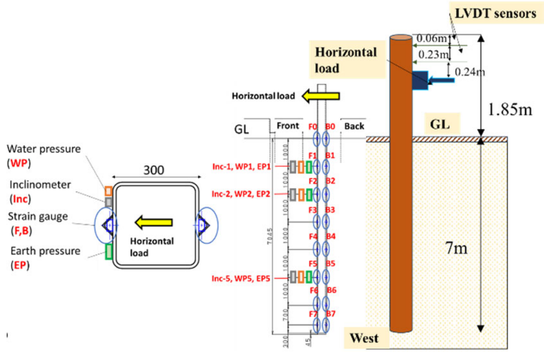


Fig. 2. Horizontal load tests and its instrumentations

Table 3. Summary of experiments conducted in this study

Item	Test	EPPR (Ru= $\Delta u/\sigma'_{v0}$)
1	Horizontal load Test (HLT)	0, 0.3, 0.6
2	CPT Tests	0, 0.3, 0.6, 0.9
3	Borehole Pressuremeter Test	0, 0.3, 0.6, 0.9

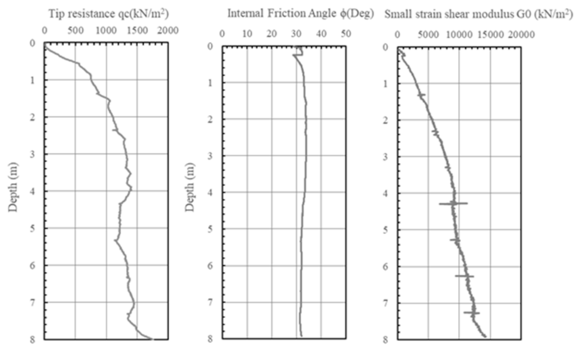


Fig. 3. CPT results, internal friction angle (∅) and the distribution of the small strain shear modulus

The HLT tests were carried out in a an incremental loading manner, as stipulated by JGS 1831-12, a standard guideline for such tests. Similarly, the BPT tests were conducted according to the guidelines of JGS 1531-2012. **Fig. 3** displays the results of the CPT test at Ru=0, which was conducted on the east side of the testing pile. Accompanying these results are the internal friction angle (φ') and the distribution of the small strain shear modulus

(G_0). These parameters were computed using the following equations, which are not detailed here but are integral to interpreting the test results.

$$\varphi' = 17.6 + 11\log\left[(q_t - \sigma_{v0}/P_a)(P_a/\sigma'_{v0})^n\right] \quad (1)$$

$$G_0 = (\gamma/g)\left((10^{0.55I_c+1.68})(q_t - \sigma_{v0}/P_a)\right)^{0.5} \quad (2)$$

In this context, q_t represents the corrected cone resistance, while σ_{v0} denotes the total overburden stress. Pa stands for atmospheric pressure, σ'_{v0} symbolizes the effective stress, g is the gravitational acceleration and γ soil unit weight. Additionally, I_c refers to the soil behavior type index.

2.3. Ground condition and Testing procedure

To assess modified CPT-based p-y models accounting for liquefaction effects, a series of 1D finite element analyses on laterally loaded piles using the Winkler model were performed at excess porewater ratios (Ru) of 0, 0.3, and 0.6, where Ru=0 signifies non-liquefied soil. The soil profile was divided into 8 layers, each 1m thick, to enhance accuracy. The pile was modeled using beam elements supported by p-y springs, as illustrated in **Fig. 4**.

2.4. Response of pile and back calculation of p-y curves

To calculate the force-displacement of the pile at a given load, the pile is modeled as a beam. According to the Euler-Bernoulli beam theory, the bending moments and shear forces (internal forces), along with the lateral soil pressure (external force), are computed. These calculations are based on the equations presented in the schematic overview of **Fig. 5.** In these equations, y represents the displacement at depth z, M is the bending moment, V is the shear force, p denotes the soil resistance per unit length, EI is the flexural rigidity, and θ is the rotational slope of the beam. Please note that strain gauges, which are for obtaining pile curvatures, are utilized to calculate the moment distribution.

This calculation is derived from a polynomial fit of the strains recorded along the pile. Conversely, inclinometers are employed to measure the pile rotations. These measurements are used to correct the pile rotations that are calculated from strains. Additionally, the p-y curves were back-calculated from the shear force distribution values.

2.5. Overview of existing p-y curves

In general, p-y springs, which are used in the Winkler spring model, are typically derived for a specific location using full-scale pile load tests or, alternatively, through 3D Finite Element (FE) testing. Numerous p-y models have been introduced to date. One notable example is the API method, which was originally introduced by Reese et al. (1974) based on a series of field tests on small diameter piles (D = 0.61 m) with large aspect ratios (L/D = 34.4). This method was later updated by O'Neill & Murchison in 1983 and is currently included in the API guidelines (API, 2011). As a result, it has become the standard approach for predicting lateral pile displacements in cohesionless soils.

However, the API method derives soil strength from an equation that includes the angle of internal friction (φ'), a value obtained indirectly through empirical equations resulting from in-situ tests (e.g., CPT, SPT). The extreme sensitivity of the API method to the selection of the empirical equation for φ' exacerbates the uncertainty of p-y results. To circumvent the difficulties of selecting appropriate strength parameters, CPT-based p-y models, which use the cone resistance (qc), have been introduced. An overview of CPT-based p-y models used in this study is summarized in **Table 4**.

Models by Dyson & Randolph (2001) and Li et al. (2014) are not based on a broad range of Pile Load Tests (PLTs) representative of both flexible and rigid piles. Although Li et al. (2014) varied the aspect ratio and rigidity, their tests were limited to dense siliceous sands, and only six tests were conducted. Suryasentana & Lehane (2014; 2016) utilized 3D FEM modeling and varied the relative density between 28% and 97%, thereby enhancing the applicability of their methods.

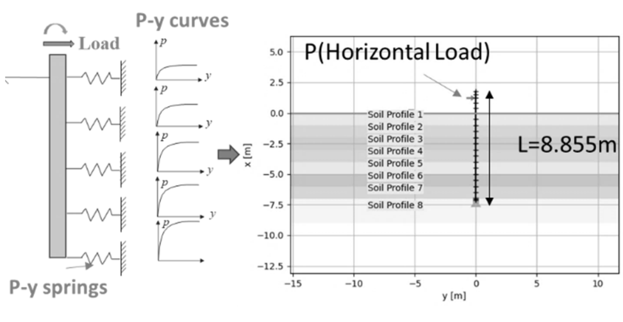


Fig. 4. Schematic diagram of 1D FEM of Laterally loaded pile

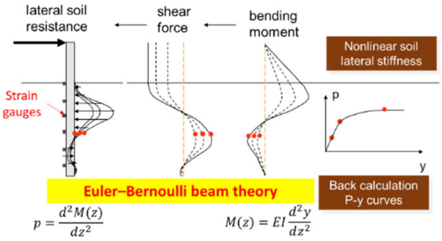


Fig. 5. Derivation of force deflection and p-y curves

Table 4. API- and CPT-based p-y methods used in this study

p-y model	p-y expression
API (2011)	$\frac{p}{p_u}$=A×tanh($\frac{k\times z}{A\times p_u}$×y) , p_u=min[p_{us}; p_{ud}] p_{us}=(C_1×z+C_2×D) ×$\acute{\gamma}$×z p_{ud}=(C_3×D×$\acute{\gamma}$×z) $C_1 = \frac{(\tan\beta)^2 \times \tan\alpha}{\tan(\beta-\varphi)} + K_0\left(\frac{\tan(\varphi)\times\tan\beta}{\cos\propto\times\tan(\beta-\varphi)} +\right.$ $\left.\tan\beta\times(\tan(\varphi)\times\sin\beta - tan\alpha)\right)$ $C_2 = \frac{\tan\beta}{\tan(\beta-\varphi)} + K_a$ $C_3 = K_a((\tan\beta)^8 - 1) + K_0\times\tan(\varphi)\times$ $(\tan\beta)^4$ $K_0 = 0.4$, $\alpha=\frac{\varphi}{2}$, $\beta = 45 + \frac{\varphi}{2}$ and $K_a = (1 -$ $\sin\varphi)/(1+\sin\varphi)$
Dyson & Randolph (2001)	p=2.84×D×($\acute{\gamma}$×D) ×$(\frac{q_c}{\acute{\gamma}\times D})^{0.72}$×$(\frac{y}{D})^{0.64}$
Li et al. (2014)	p=3. 6×D×($\acute{\gamma}$×D) ×$(\frac{q_c}{\acute{\gamma}\times D})^{0.72}$×$(\frac{y}{D})^{0.66}$
Suryasentana & Lehane (2014)	p=2.4×D×($\acute{\sigma_v}$) ×$(\frac{q_c}{\acute{\sigma_v}})^{0.67}$×$(\frac{z}{D})^{0.75}$× [1-exp(-6.2×$(\frac{z}{D})^{-1.2}$×$(\frac{y}{D})^{0.89}$]
Suryasentana & Lehane (2016)	p=4.5×G_{max}×y, $\frac{y}{D}$ <0.0001 P=2.4×D× ($\acute{\sigma_v}$) ×$(\frac{q_c}{\acute{\sigma_v}})^{0.67}$×$(\frac{z}{D})^{0.75}$× [1-exp(-8. 9×$(\frac{z}{D})^{-1.25}$×$(\frac{y}{D})^{-1.2}$×$(\frac{\sigma_v-u_g}{\acute{\sigma_v}})$]<$q_c$×D, 0.0001 < $\frac{y}{D}$ <0.01

In the **Table 4**, p represents lateral soil resistance per length of pile, and y is deflection φ' represents internal friction angle, D represents pile diameter, q_c represents the cone resistance, while σ_v denotes the total overburden stress. Pa stands for atmospheric pressure, $\acute{\sigma_v}$ symbolizes the effective stress and γ soil unit weight. G_{max} is the small strain shear modulus.

2.6. Procedure for Determining p-y Curves from BPT Results

The technical advantages of using Borehole Pressure Meter results for calculating p-y curves, as opposed to relying on in-situ pile test results, include more controlled and consistent measurements. They are generally more cost-effective and less invasive, causing minimal site disturbance, and offer enhanced accuracy. Furthermore, deriving p-y curves from HLT tests may be associated with certain errors. Therefore, in this study, a modification to existing API and CPT-based p-y curves is initially proposed using BPT test results. This modification involves the introduction of p and y multipliers, which consider the effect of liquefaction.

Briaud, Smith, and Meyer developed a method using BPT results for calculating p-y curves for laterally loaded piles. The resistance of a laterally loaded pile comprises two components: the frontal resistance, denoted as Q, which results from the development of passive resistance on the face of the pile, and the friction resistance, F, arising from shear resistance along the sides of the pile (**Fig.6a**). These two elements collectively contribute to the pile-soil interaction (Briaud et.al.,1985, Smith1983).

$$P=F+Q \tag{3}$$

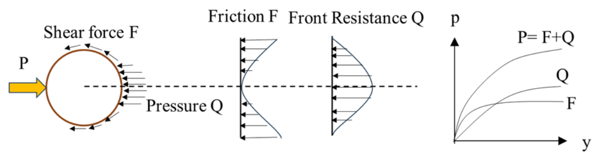
$$y=y\,(BPT) \times (D_{pile}/D_{BPT}) \tag{4}$$

$$F=\tau\times D_{pile}\times S_F\times(1/\beta) \tag{5}$$

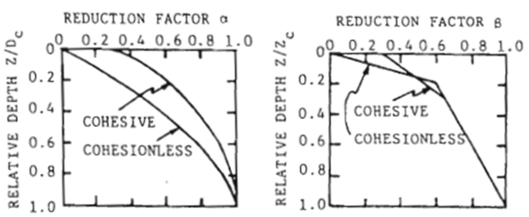
$$\varepsilon_\theta = (\Delta R/R) \tag{6}$$

$$\tau = \varepsilon_\theta \times (dP(BPT)/d\varepsilon_\theta) \tag{7}$$

$$Q=P(BPT)\times D_{pile}\times S_Q\times(\alpha/\beta) \tag{8}$$



(a)



(b)

Fig. 6. **(a)** Friction and frontal resistance component **(b)** α , β are

Reduction factors

The p-y curves can be calculated from below, where P is soil resistance on the pile expressed as force per unit length, P(BPT) the pressure meter pressure, D_{pile} pile diameter, D_{BPT} BPT diameter, S_F (=1 for square and =2 for round profile) and S_Q (=1 for square and = $\pi/4$ for round profile) are shape factors and α, β are Reduction factors and τ is Shear stress at soil pile interface , R is the radius of pressuremeter, ε_θ is circumferential strain at the wall of cavity.(**Fig. 6b**)

3. Results

3.1. Response of pile

Fig. 7 displays the response of the pile at a Liquefaction Ratio (R_u) of 0. This response was calculated using the procedure outlined in Section 2.3. The calculation involved utilizing the p-y data across each loading, derived from curves obtained from the pile responses at specific depths, as shown in **Fig. 8**.

The ultimate horizontal capacity of the pile reduces markedly with an increase in the excess pore pressure ratio. Additionally, the rate at which the stiffness of soil reaction forces reduces appears to escalate with higher values of Ru. For example, at Ru=0.6, the soil rapidly loses its stiffness evidenced by the flattening of the p-y curves as the pile's deflection increases, particularly at shallower depths.

3.2. Determining p-y curves from BPT tests

As explained in Section 2.5, data from Borehole Pressuremeter Test (BPT) can be utilized to understand the components of side shear resistance and passive pressure, which are integral to describing soil-pile interaction when the pile is laterally loaded. **Fig. 9a** presents the results of p-y curves derived from BPT test results at a depth of 2m, for Liquefaction Ratios (Ru) of 0, 0.3, 0.6, and 0.9. These data were subsequently fitted using a polynomial function.

From each fitted curve, the maximum p values and their corresponding y values were determined. These values were then normalized against the maximum p value and corresponding y values at Ru=0. Following this analysis, a linear correlation is proposed between the normalized maximum p values and the Ru values, as well as between the normalized corresponding y values and Ru. This approach helps in quantifying the relationship between the soil-pile interaction and the degree of soil liquefaction (**Fig. 9b)**. The followings are the initial settings are as follows

$$P_{max}/P_{max}(Ru = 0) = (0.9945 - 0.7445 \times Ru) \quad (6)$$

$$y_{max}/y_{max}(Ru = 0) = (0.9755 - 0.3965 \times Ru) \quad (7)$$

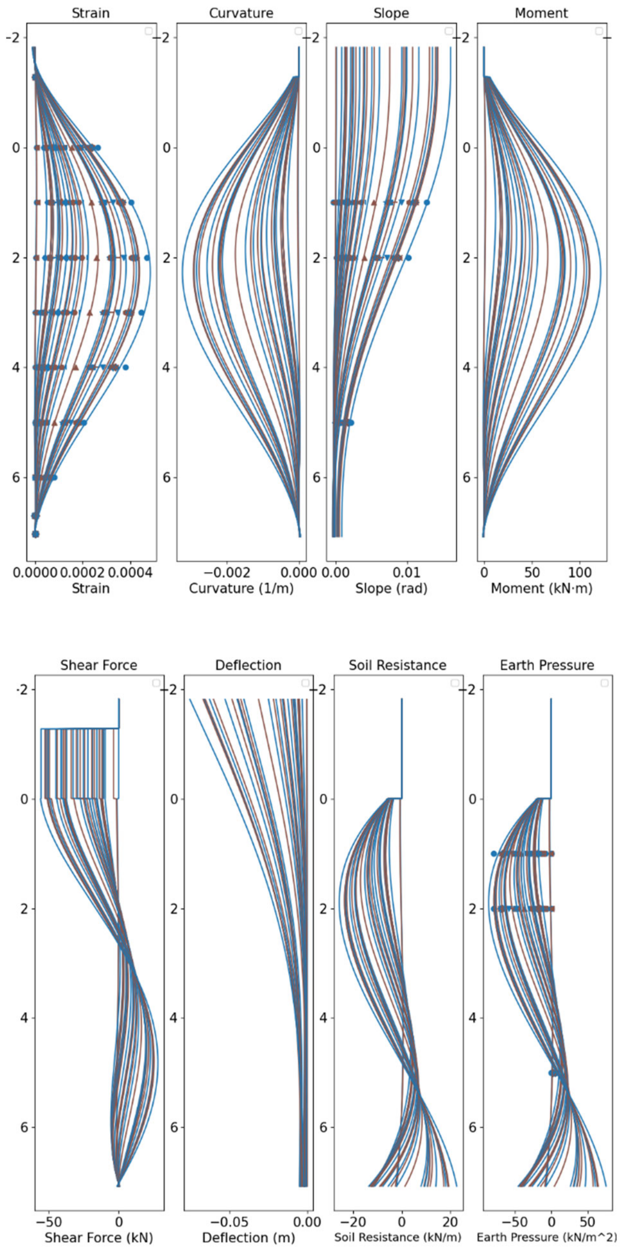


Fig. 7. Measured response of laterally loaded pile at (R_u=0)

3.3. Simulation of p-y curves from BPT tests

To evaluate the effectiveness of various Cone Penetration Test (CPT)-based p-y methods, load-deflection data from Borehole Pressuremeter Test (BPT) for four different Liquefaction Ratio (Ru) values, ranging from 0 (non-liquefied) to 0.9 (near-liquefaction state), were analyzed. The correlation in **Eq. 6** and **Eq. 7** was applied as a P and y multiplier to reflect the influence of liquefaction on the evolution of p-y curves. The results of this analysis are displayed in **Fig. 10**.

As anticipated, the findings indicate that the American Petroleum Institute (API) model tends to overestimate the p-y values at smaller deflections. In contrast, the model developed by Suryasentana & Lehane (2016) appears to underestimate these values, particularly at smaller y values. This discrepancy highlights the varying performance of different models under conditions of soil liquefaction and emphasizes the importance of selecting an appropriate p-y method for accurate predictions in such scenarios.

3.4. Results of 1D FEM analysis

To assess the performance of the American Petroleum Institute (API) and Cone Penetration Test (CPT)-based models for cases of laterally loaded piles under different liquefaction states, a comparison was made between the predicted load-deflection, shear forces, and moments of the pile and the actual results from Horizontal Load Tests (HLT). The initial p and y multipliers functions, obtained in **Eq. 6** and **Eq. 7**, were modified to produce load-deflection results akin to those observed in the HLT tests (**Fig. 11, Fig. 12**).

Table 4. API- and CPT-based p-y methods used in this study

P-Y model	p-multiplier (p=p×p-multiplier)	y-multiplier (y=y×y-multiplier)
API (2011)	(0.9945 - 0.7445 × Ru)	(2.52 + 0.91 × Ru)
Dyson & Randolph (2001)	Same As API model	1
Li et al. (2014)	Same As API model	1
Suryasentana & Lehane (2014)	Same As API model	1
Suryasentana & Lehane (2016)	Same As API model	Same As API

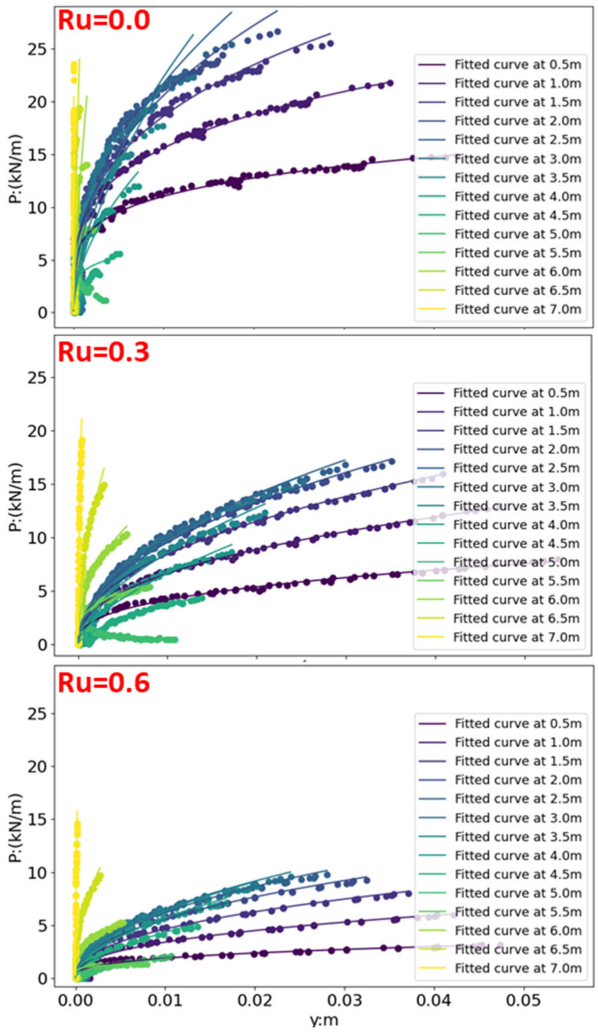


Fig. 8. P-y curves back calculated from the results of HLT tests

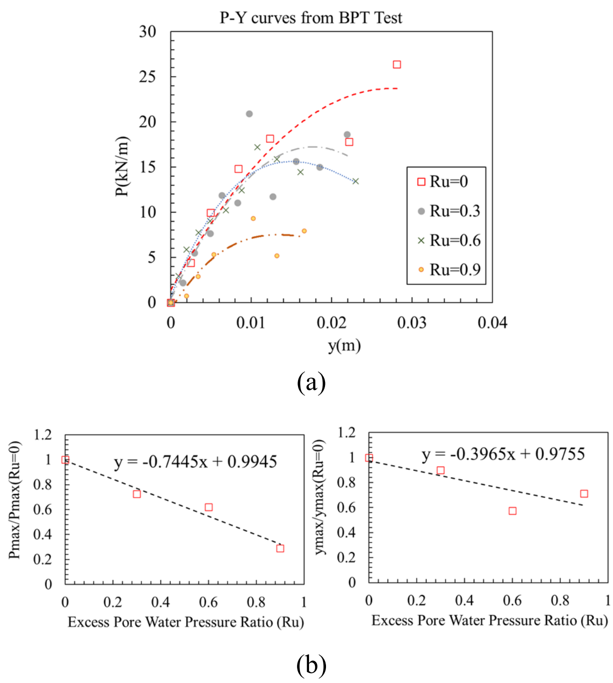


Fig. 9. (a) P-y curves (b) P_{max} and y_{max} correlation

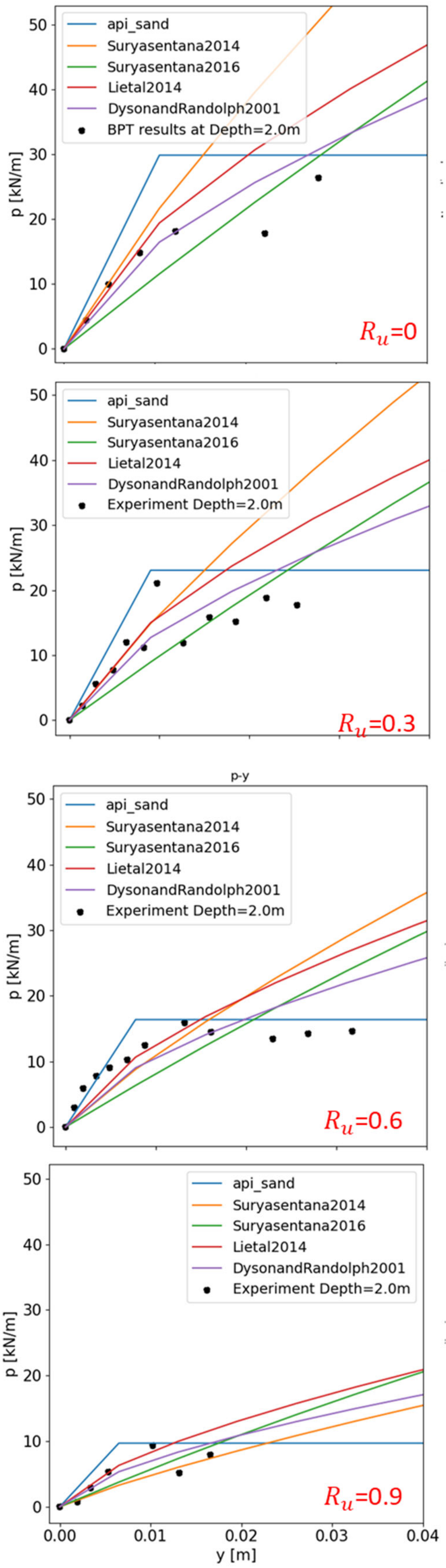


Fig. 10. Simulation of p-y from BPT at depth of 2m

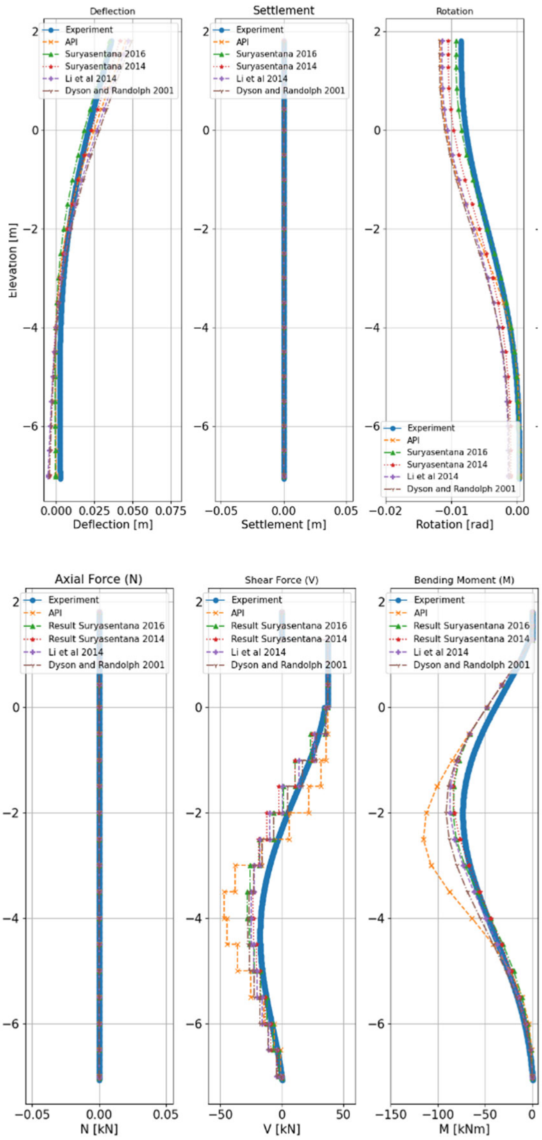


Fig. 11. Predicted pile response using API and CPT based p-y models at Ru=0

The analysis revealed that nearly all existing p-y models, including those based on CPT data, tend to overestimate the maximum values of moment and shear forces. Notably, the API model was found to overestimate these values by approximately 1.5 times. The proposed P and y multipliers from for each p-y model, which are designed to offer a more accurate reflection of the HLT test results, are summarized in **Table. 4**. This adjustment is crucial for improving the accuracy of p-y models in predicting the response of laterally loaded piles, especially in liquefied soil conditions.

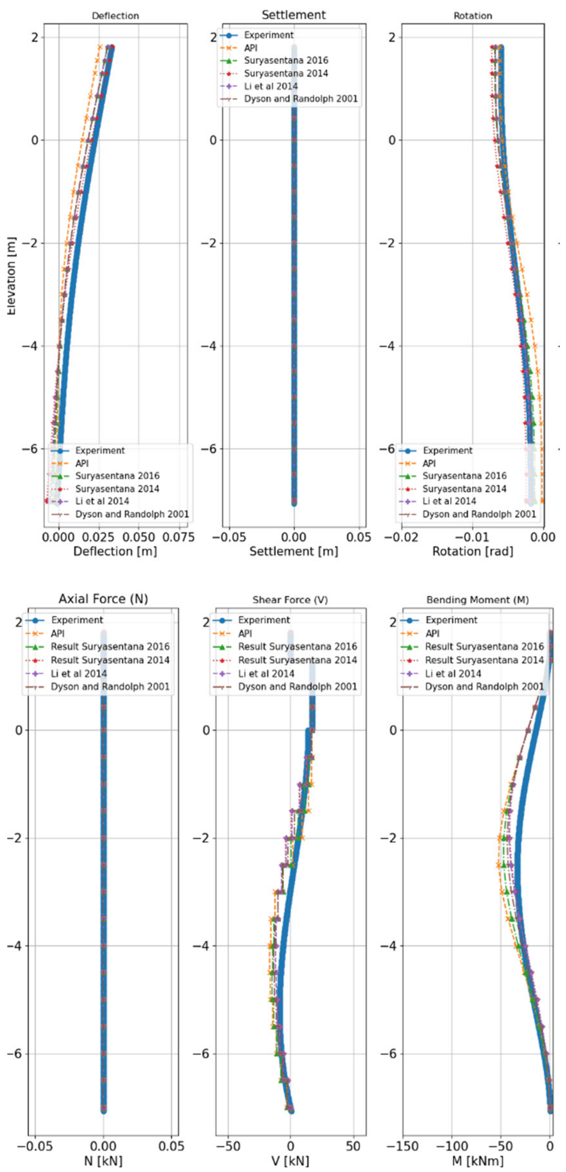


Fig. 12. Predicted pile response using API and CPT based p-y models at Ru=0.6

4. Discussion

4.1. Back-calculating p-y curves from HLT Tests

A method was developed to back-calculate p-y curves from laterally loaded pile data, utilizing Euler–Bernoulli beam theory. The precision of the calculated p-y data is directly related to the availability of diverse types of recordings and the complexity of the fitting function. As explained in section 3.1, that back-calculated p-y curves for different liquefaction states were similar to those in non-liquefied states, simple p and y multipliers can be effectively used to incorporate the effects of liquefaction into p-y models.

4.2. Back-calculating p-y curves from BPT Tests

As shown in section 3.3 and **Fig. 9**, initial modifications, using p and y multipliers, to existing API and CPT-based p-y models, can be recommended by incorporating Borehole Pressuremeter Test (BPT) results obtained under various liquefaction states. This approach helps to minimize errors associated with traditional HLT test result derivations. Furthermore, since the modified p-y model integrates Ru values, it can be applied to both liquefied and non-liquefied cases.

4.3. Performance of modified p-y models

The results from the 1D analysis, considering various liquefaction states, suggest that existing CPT-based models likely underestimate the soil's stiffness (**Fig. 11**). This underestimation, in turn, leads to an overprediction of the moments and shear forces within the pile. Despite this, the CPT-based models correspond more closely with the actual pile measurements compared to the API method. Furthermore, p-y models in particular API tends to underestimate deflections relatively. To address this issue, the initial function for the y-multiplier is modified to more closely match the deflections observed in experiments. However, to draw more definitive conclusions, it is essential to verify the models' accuracy across a broader spectrum of pile aspect ratios (L/D), from short to slender piles.

5. Conclusions

In this study, a simplified method was introduced to back-calculate pile responses from large-scale pile tests conducted by GIKEN LTD. under various liquefaction states. Using the results from Borehole Pressuremeter tests, an initial framework for modifying existing CPT-based models was introduced. This framework was then implemented in a 1D-FEM analysis to assess the performance of each model against the results of large-scale experimental tests.

The consistency in p-y curve shapes across different liquefaction states indicates that simple p-y multipliers are a viable approach for integrating liquefaction effects into existing models. This study introduces an initial p-y multiplier derived from Borehole Pressuremeter Test (BPT) results as a preferable alternative to those obtained from Horizontal Load Test (HLT) results, which may contain errors. Additionally, 1D analysis reveals that existing CPT-based models tend to underestimate soil stiffness in liquefied conditions, resulting in overpredictions of moments and shear forces. However, despite this underestimation, CPT-based models align more closely with actual pile measurements than the API model, highlighting their potential for more accurate pile behavior predictions in liquefied soils.

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