A New Approach for Estimating Capacity of Driven Piles in Sand under Tensile Loading

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ABSTRACT

This paper is dedicated to estimate the capacity of tension piles installed by dynamic driving in cohesionless soil. Using a database comprising 23 closed-ended piles, a new design approach for closed-ended piles in sand under tensile loading was developed. The new approach was extended to open-ended piles using three methods and a database comprising 14 open-ended piles. The first method (method 1) is based on a plug indicator which accounts for the effect of incremental filling ratio and vertical effective stress at the pile tip. In the second method (method 2), open-ended pile is transformed to equivalent solid pile (considered as closed-ended pile with equivalent diameter equal to D*). The third method (method 3) is a combination of methods (1) and (2) with the required adjustments. Six independent well-documented full-scale pile load tests collected from literature were used as independent database to examine the validity of the proposed methods. Comparison of predicted capacities using the three proposed methods for open-ended piles with measured values for the collected six field cases showed good agreement when using the first method and suggested that this method had sound theoretical physical meaning.

KEYWORDS: Estimating capacity, Driven piles, Sand, Tensile loading, Closed-ended piles, Open-ended piles.

INTRODUCTION

Piles are usually used to support compressive loading. However, they may be used as a foundation system for floating platforms and electrical transmission towers to resist uplift loading. Piles are usually driven into soil either closed- or open-ended. However, open-ended piles are usually used in preference to closed-ended piles to reduce the cost of driving, especially for long piles penetrating sandy soils. Unfortunately, determination of compressive as well as tensile capacities of such piles becomes a difficult task due to the formation of soil plug inside the pile during driving.

In general, the ultimate uplift resistance of a vertical pile in sand is a matter of evaluating the magnitude of unit shaft friction distribution along a cylindrical failure surface (Kulhawy et al., 1983). The coefficient of lateral earth pressure K and the interface friction angle δ are considered as the main parameters in the evaluation of the unit shaft friction. Many factors affect the earth pressure coefficient, such as: soil properties, method of pile installation, loading type and pile shaft characteristics (Chauldhuri and Symons, 1983). Contribution of these factors with shortage in reliable full-scale experimental studies is considered as difficulties in theoretical formulation. Moreover, degradation of shaft friction during pile driving has been

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recognized for decades and generally referred to as friction fatigue (Heerema, 1980; Toolan et al., 1990; Alawneh, 1999).

Test results on the fully instrumented closed-ended Imperial College Pile (ICP) reported by Lehane (1992) as well as by Lehane and Jardine (1994) indicated that local unit shaft friction is controlled by Mohr-Coulomb failure criterion without limiting value as adopted in the API RP2A (2010) design guidelines and that the radial effective stress on the pile surface at ultimate loading ($\sigma`_{rf}$) can be divided into two components; the first component is the equalized stationary effective radial stress after installation and before pile loading ($\sigma`_{rc}$), while the second component is the change in the effective radial stress ($\Delta\sigma`_{rd}$) due to the tendency of sand to dilate during loading. The work of Lehane (1992), Lehane and Jardine (1994) and others formed the basis for developing new CPT-based design methods, such as: the Fugro-05 method (Kolk et al., 2005), Imperial College Pile (ICP-05) method (Jardine et al., 2005), Norwegian Geotechnical Institute (NGI-05) method (Clausen et al., 2005) and University of Western Australia (UWA-05) method (Lehane et al., 2005). Even though these methods in most cases yield reasonable estimates of measured pile capacity in the field, the effect of the degree of pile plugging for open-ended pile was not properly addressed in some of these methods.

When an open-ended pile penetrates down into sand, a soil column moves towards inside the pile in the initial stage of pile driving. If the height of the soil column equals the penetration length of the pile, the pile penetrates the soil in fully coring mode. Inner shaft friction between the soil core and the inner pile surface is mobilized as the pile penetrates down into soil. Mobilizing large magnitude of inner shaft friction may prevent additional soil from entering inside the pile; in this case, the pile is said to penetrate in fully plugged mode. Mobilizing small magnitude of inner shaft friction may partially prevent additional soil from entering inside the pile (partially plugged pile behavior during driving) (Paikowsky et al., 1989; Klos and Tejchman, 1977).

In light of the aforementioned introduction, it seems that a room for more contribution in this area is still available. This paper is dedicated to introduce a new design approach for driven piles (open- and closed-ended) in sand under tensile loading.

Alawneh (1999) Proposed Method

The uplift resistance of a single pile driven in sand is usually assumed to be dependent on the peak local unit shaft friction, which is related to the radial effective stress at failure. The following equations are usually used to evaluate the ultimate uplift shaft resistance of a vertical circular pile in sand:

$$Q_s = \pi Do \int_0^L \tau_{(z)}\, dz \qquad (1\text{-a})$$

$$\tau_{(z)} = K_{(z)}\, \sigma'_v \tan\delta \qquad (1\text{-b})$$

where:

Q_s = Ultimate uplift shaft friction capacity.
$\tau_{(z)}$ = Unit shaft resistance at depth z.
δ = Pile-sand friction angle.
σ'_v = Effective vertical stress at depth z.
z = Depth below ground surface.
L = Embedded length of pile.
$K_{(z)}$ = Lateral earth pressure coefficient at depth (z).
D_o = Pile outside diameter.

Alawneh (1999) proposed a method to estimate the ultimate uplift shaft resistance of a driven pile in sand based on a database of pile load tests collected from engineering literature. The database consists of 34 full-scale pile load tests and comprises 22 steel pipe piles (10 open-ended and 12 closed-ended), 11 concrete piles and 1 timber pile. These piles were driven into sand with relative density varying from 25% to 90%.

To account for the friction fatigue processes during pile driving, Alawneh (1999) proposed that the lateral earth pressure coefficient $K_{(z)}$ for a given soil horizon degrades from a peak value (K_{max}) near pile tip to a minimum value (K_{min}) as an exponential function of the length of the pile driven past that horizon:

$$K_{(z)} = K_{min} + (K_{max} - K_{min})\exp\left[-\mu\left(\frac{L-z}{Do}\right)\right] \quad (2)$$

where:
K_{min} = Minimum earth pressure coefficient.
K_{max} = Maximum earth pressure coefficient.
μ = Exponential decay rate.

Theories suggest that the value of K_{max} decreases with depth and increases with sand relative density (Toolan et al., 1990; Coyle and Castello, 1981). As such, Alawneh (1999) proposed the following correlation for K_{max} which considers these influencing factors:

$$K_{max} = 0.349\exp(0.029\,D_r)\left(\frac{Do+0.45}{2Do}\right)^{0.005D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84} \quad (3)$$

where:
σ'_{vtip} = Effective vertical stress at the pile tip.
D_r = Sand relative density.
P_a = Atmospheric pressure ($101.3\ kN/m^2$) .

The minimum value of earth pressure coefficient (K_{min}) is reached for very long piles in loose sand and can be measured either from pile load tests or from a theoretical consideration, that adopts a value of K equal to the Rankine's active earth pressure coefficient (K_a). Alawneh (1999) suggested a value of K_{min} equal to 0.23 (the minimum back calculated K value in the database used in his study) and a constant value for the parameter μ which was 0.03.

Alawneh (1999) method has two shortcomings; the use of constant value for the parameter μ which is 0.03 regardless of pile diameter. Randolph et al. (1994) suggested using a value of μ in the range of 0.05 and stated that there is an indication that μ decreases with increasing pile diameter, reflecting the need for determining more reasonable value for the parameter μ in terms of pile diameter. Also, the open-ended piles were converted in Alawneh (1999) study into equivalent closed-ended piles by multiplying the measured shaft friction capacity by 1.25 regardless of the incremental filling ratio. Shaft friction capacity of a driven open-ended pile in sand increases with decreasing the incremental filling ratio (IFR) and in the case of IFR = 0.0, an open-ended pile behaves as a closed-ended pile. That is to say that the shaft friction capacity is equal to that of a similar closed-ended pile as concluded by Igoe et al. (2011).

Plug Indicators and Estimation of Average Incremental Filling

Plugging of open-ended driven piles in sand can be quantified by two parameters. These are the incremental filling ratio (IFR) and the plug length ratio (PLR):

$$IFR = \Delta L_p/\Delta L \quad (4\text{-a})$$

$$PLR = L_p/L \quad (4\text{-b})$$

where ΔL_p = change in the soil plug length during an increase in pile penetration ΔL, L_p = total length of soil column inside the pile at the end of pile driving and L = final penetration depth of the pile. Using (IFR) is not that easy, because it needs a continuous measurement of the change in the plug length during pile driving, which is not practical. As a result, it is easier to obtain PLR than IFR during pile driving.

If IFR = 1, the pile penetrates in coring mode and if IFR = 0, the pile behaves as a closed-ended pile (fully plugged). IFR between 0.0 and 1.0 indicates that the pile is partially plugged during driving. Figure (1) shows a schematic representation of different plugging modes.

Paik et al. (2003) proposed the following relationship between PLR and the incremental filling ratio at final penetration (FFR):

$$FFR = 1.09\,PLR - 0.22 \quad (5)$$

For driven piles in dense and very dense sand, Lehane et al. (2005) proposed the following relationship for the average incremental filling ratio (i.e., plug length ratio):

$$(IFR)ave. = \min.\left(1, \left(\frac{D_i}{1.5}\right)^{0.2}\right) \quad (6)$$

where:
D_i = Pile inner diameter in meters.

For piles in loose and medium-dense sand, Lehane et al. (2005) did not give an expression for the average incremental filling ratio. However, model pile tests on calibration chamber conducted by Paik and Lee (1993) and Paik et al. (2003) showed that piles driven in loose and medium-dense sand plug earlier than piles in dense and very dense sand. The driving pressure (i.e., plug capacity) which pushes the soil column inside the pile during driving is larger for piles in dense sand. As a result, driven piles in dense sand may experience higher plug length ratios than those in loose and medium-dense sands. Table (1) summarizes test results conducted on model piles in the calibration chamber under different initial vertical and horizontal effective stress conditions. The model pile was driven in sand with varying relative density.

Table 1. Soil properties, pile characteristics and measured PLR for model piles tested by Paik and Lee (1993) and Paik et al. (2003)

Reference	Pile	D_r (%)	σ'_v (kPa)	σ'_h (kPa)	Initial earth pressure coefficient	PLR (measured)	Ratio (Ψ)
Paik et al. (2003)	HHL	90	98.1	39.2	0.4	0.76	1
	MHL	56	98.1	39.2	0.4	0.70	0.92
	LHL	23	98.1	39.2	0.4	0.62	0.82
Paik and Lee (1993)	NT	90	98.1	39.2	0.4	(0.81-0.85) (0.83)*	1
	HS	90	98.1	68.7	0.7		
	VL	90	68.7	39.2	0.57		
	DL	56	98.1	39.2	0.4	0.76	0.91
	DS	23	98.1	39.2	0.4	0.69	0.83

*average value.

The test piles HHL, MHL, LHL, NT, HS, VL, DL and DS had outer and inner diameters of 4.27 cm and 3.65 cm, respectively with an embedded length of 0.59 m for DL, NT, HS, VL and DS test piles and 0.76 m for HHL, MHL and LHL test piles. The relative density was 90% for HHL, NT, HS and VL piles, 56% for MHL and DL piles and 23% for DS and LHL piles. From the measured PLR, the ratio of PLR for piles in loose sand to that in dense sand (Ψ) can be taken equal to 0.92 for piles in medium-dense sand and 0.82 for piles in loose sand (for piles in dense and very dense sand, Ψ = 1.0). As such, the average incremental filling ratio can be obtained by modifying Equation (6) as follows:

$$(\mathrm{IFR})\mathrm{ave.} = \min.\left(1, \Psi\left(\frac{D_i}{1.5}\right)^{0.2}\right) \quad (7)$$

In this study, this correction is used to estimate the average incremental filling ratio for some open-ended piles driven in loose and medium-dense sands.

Database

Alawneh (1999) used a database which comprised closed- and open-ended piles. Alawneh (1999) converted open-ended piles into equivalent closed-ended piles by multiplying the measured shaft capacity by 1.25, which is one of the shortcomings of Alawneh (1999) approach. In this study, the database used by

Alawneh (1999) (tension piles) was separated into two categories. The first one for closed-ended piles comprised 23 closed-ended piles, while the second one for open-ended piles comprised 14 piles (10 piles used by Alawneh (1999) and 4 new piles collected from literature). Tables (2) and (3) summarize pile characteristics, soil properties and load measurements for the closed- and open-ended pile groups, respectively.

Analysis of Database for Closed-Ended Piles

The analysis of the database summarized in Table (2) for closed-ended piles was made using μ values of 0.03 as used by Alawneh (1999), 0.05 according to Randolph et al. (1994) and a proposed value given by:

$$\mu = -0.1 \log_{10}(\mathrm{D_o}),\ 0.00 \le \mu \le 0.05 \qquad (8)$$

For each μ value, the K_{max} value for each pile in Table (2) is back-calculated and the results are summarized in Table (4). The best correlation which fits the back-calculated K_{max} values is obtained using the statistical package SPSS and the results are summarized in Table (5) for the three cases: case (1) when μ = 0.03, case (2) when μ = 0.05 and case (3) when μ is calculated from Equation (8).

For each case (combination of μ and K_{max}) in Table (5), tensile capacity of each pile in the database in Table (2) was calculated and the results are summarized in Table (6). The ratio of calculated to measured shaft capacity for case (3) varies between 0.72 and 1.4 with an average value of 1.05. In general, the statistics obtained from using the correlations for case (1) and case (3) are not significantly different from each other, because the database used by Alawneh (1999) has an average diameter of 0.45. However, the proposed value for μ in Equation (8) yields reasonable predictions, consistent with Randolph et al. (1994) who stated that there is an indication that μ value tends to decrease with increasing pile diameter. For pile diameter equal to 0.5 m, Equation (8) gives μ = 0.05 which is the value suggested by Randolph et al. (1994). For large-diameter pile; say in the range of 1.5 m for example, the use of μ = 0.05 or 0.03 may not yield good prediction for pile shaft friction capacity. In this case, the use of Equation (8) to calculate the parameter μ may be more appropriate.

Based on the results shown in Table (5), Equation (3) for closed-ended piles can be modified and written as:

$$K_{max} = 0.4 \exp(0.029\ D_r) \left(\frac{Do+0.45}{2Do}\right)^{0.005D_r} \left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84} \qquad (9)$$

This equation for K_{max} is used with μ value calculated from Equation (8).

Proposed Methods for Estimating Shaft Friction Capacity of Open-Ended Piles

Method (1): Based on a New Plug Indicator (M)

Al-Omari (2018) proposed a methodology for evaluating the magnitude of shaft capacity of an open-ended pile driven into sand using the following formulation:

$$\tau_{OE} = \tau_{CE} \times \mathrm{CF} \qquad (10\text{-a})$$

$$\mathrm{CF} = 1 - \frac{(1-M)}{L} \times z \qquad (10\text{-b})$$

where:

τ_{OE} = Unit shaft friction at depth (z) along the open-ended pile.

τ_{CE} = Unit shaft friction at depth (z) along a similar closed-ended pile.

CF = Correction factor which varies linearly with depth from a minimum value of M at the pile tip to a maximum value of 1.0 at the pile head. Referring to Equation (1-b), Equation (10-a) can be written as:

$$\left[K_{(z)}\right]_{O.E.} = \left[K_{(z)}\right]_{C.E.} \times [\mathrm{CF}] \qquad (11)$$

where:

$\left[K_{(z)}\right]_{O.E.}$ = Earth pressure coefficient at any depth (z) along the open-ended pile.

$\left[K_{(z)}\right]_{C.E.}$ = Earth pressure coefficient at any depth (z) along a similar closed-ended pile.

Table 2. Pile characteristics, soil properties and load measurements for the closed-ended piles (tension tests)

Site Name and (Reference)	Site No.	Serial No.	Pile No.	L (m)	D_o (m)	$\frac{L}{D_o}$	Average σ'_v (kPa)	D_r (%)	ϕ_{cv} (degree)	ϕ' (degree)	Measured Q_s (kN)	Pile Material	GWT location (m) $\gamma'_{ave.}$ (kN/m³)
Low-Sill Structure, Old River, Louisiana Mansur et al. (1958)	1	1	2	19.81	0.56	35.2	80.0	65	30	34	1680	Steel	2.86 (8.1)
		2	4	20.12	0.46	43.6	81.4	62	30	34	1780	Steel	2.86 (8.1)
		3	5	13.72	0.46	29.8	53.9	57	27	30	715	Steel	2.86 (7.9)
		4	6	19.81	0.51	38.7	79.5	55	30	34	1655	Steel	2.86 (8.0)
Arkansas River Lock and Dam No.2 Sherman et al. (1974)	2	5	G-2	13.04	0.35	36.8	87.0	70	30	36	(803)*	Conc.	2.74 (15.9)
		6	B-5	13.10	0.46	28.5	70.3	70	30	36	(1005) *	Conc.	0.76 (12.8)
Arkansas River Lock and Dam No.3 Sherman et al. (1974)	3	7	G-7	11.67	0.33	35.4	76.9	55	30	34	433	Timber	2.32 (13.2)
Arkansas River Lock and Dam No.4 Mansur and Hunter (1970)	4	8	1	16.18	0.38	42.5	79.8	63	30	34	828	Steel	0.76 (9.90)
		9	2	16.08	0.51	31.5	79.4	63	30	34	1000	Steel	0.76 (9.90)
		10	3	16.15	0.56	28.8	79.7	63	30	34	1080	Steel	0.76 (9.90)
		11	4	12.28	0.41	30.0	60.5	63	30	34	820	Conc.	0.76 (9.90)
		12	10	16.18	0.51	31.7	79.4	63	30	34	980	Steel	0.76 (9.90)
Jonesville Lock and Dam Sherman et al. (1974)	5	13	1	11.58	0.46	25.2	71.4	70	32	37	1128	Conc.	2.13 (12.3)
		14	2	13.72	0.46	29.8	81.9	70	32	37	1300	Conc.	2.13 (11.9)
		15	3	16.42	0.46	35.8	95.8	70	32	37	1116	Conc.	2.13 (11.6)
		16	4	13.72	0.46	29.8	81.9	70	32	37	1123	Conc.	3.05 (11.9)
Ogeechee, Georgia Vesic (1970)	6	17	H-16	15.00	0.46	32.6	88.8	73	31	40	1540	Steel	----- (11.8)
Dramman, Norway Gregersen et al. (1973)	7	18	A	8.00	0.28	28.6	45.0	25	30	31	94	Conc.	----- (11.2)
		19	D/A	16.00	0.28	57.1	88.7	25	30	31	254	Conc.	----- (11.1)
		20	E	23.50	0.28	83.9	125.7	25	30	31	287	Conc.	----- (10.7)
Hoogs and Holland Beringen et al. (1979)	8	21	II	6.75	0.36	18.8	54.4	90	33	43	1106	Steel	----- (16.1)
Labenne Site Lehane et al. (1993)	9	22	TP2	5.95	0.102	58.3	39.0	45	33	34	50	Steel	2.80 (13.0)
Hsina-Ta Yen et al. (1989)	10	23	5	34.30	0.61	56.1	160.5	50	30	33	2500	Steel	----- (9.40)

* Excluding clay layer.

Table 3. Pile characteristics, soil properties and load measurements for the open-ended piles (tension tests)

Site Name and (Reference)	Site No.	Serial No.	Pile No.	L (m)	D_o (m)	$\frac{L}{D_o}$	Average σ'_v (kPa)	D_r (%)	ϕ_{cv} (degree)	ϕ' (degree)	Measured Q_s (kN)	Pile Material	GWT location (m) $\gamma'_{ave.}$ (kN/m³)
Hoogsand, Holland Beringen et al. (1979)	8	1	I	7	0.36	19.4	55.2	90	33	43	816.8	Steel	----- (15.8)
		2	III	5.25	0.36	14.6	44.2	90	33	43	538.4	Steel	----- (16.8)
Padre Island McClelland Engineers, Inc. (1958)	11	3	A	14.83	0.51	29.1	77.5	59	30	34	480.8	Steel	----- (10.5)
British Columbia McCammon and Golder (1970)	12	4	1A	45.42	0.61	74.5	178.3	35	30	32	1636.8	Steel	----- (7.4)
North Sea (AD) Furgo (1969)	13	5	P16	30.5	0.61	50.0	143.4	60	30	34	2531.2	Steel	----- (9.4)
Blount Island Nottingham (1975)	14	6	322	22.56	0.27	83.6	59.6	59	30	34	702.4	Steel	----- (8.5)
Mustang Island Reese and Cox (1976)	15	7	1	21.03	0.61	34.5	95.1	82	33	38	(1823.2)*	Steel	----- (11.3)
		8	2	21.03	0.61	34.5	95.1	82	33	38	(1676.8)*	Steel	----- (11.3)
Dunkerque Brucy et al. (1991)	16	9	CS	11.6	0.324	35.6	70.5	60	30	34	388	Steel	----- (12.2)
Los Barrios Mey et al. (1985)	17	10	1	18	0.91	19.7	90	50	27	30	2500	Conc.	----- (10.0)
Euripides Kolk et al. (2005)	18	11	Ia	30.5	0.763	40.0	275.6	85	33	43	(3000)*	Steel	1.00 (10.5)
		12	Ib	38.7	0.763	50.7	318.7	95	33	42	(9750)*	Steel	1.00 (10.5)
		13	II	46.7	0.763	61.2	360.7	95	33	42	(11000)*	Steel	1.00 (10.5)
Dunkirik Jardine and Standing (2000)	19	14	R1	19.3	0.457	42.2	106.15	78	32	36	1450	Steel	4.00 (11.0)

* Excluding clay layer.

Table 4. Back-calculated K_{max} for different values of μ (closed-ended piles)

Site No.	Serial No.	Pile No.	ϕ_{cv} (degree)	δ_f (degree)	K_{max}		
					$\mu = 0.03$	$\mu = 0.05$	$\mu = -0.1\log_{10}(D_o)$
1	1	2	30	26.0	1.62	1.90	1.55
	2	4	30	26.0	2.17	2.64	2.26
	3	5	27	23.0	2.00	2.31	2.05
	4	6	30	26.0	1.82	2.18	1.81
2	5	G-2	30	26.0	1.97	2.30	2.22
	6	B-5	30	26.0	2.24	2.54	2.30
3	7	G-7	30	(30.0)*	1.03	1.19	1.17
4	8	1	30	26.0	1.50	1.81	1.68
	9	2	30	26.0	1.26	1.45	1.25
	10	3	30	26.0	1.21	1.37	1.17
	11	4	30	26.0	2.24	2.59	2.39
	12	10	30	26.0	1.22	1.41	1.21
5	13	1	32	28.0	2.20	2.50	2.25
	14	2	32	28.0	1.92	2.22	1.98
	15	3	31	28.0	1.20	1.40	1.24
	16	4	32	28.0	1.65	1.90	1.70
6	17	H-16	31	27.0	2.05	2.40	2.11
7	18	A	30	26.0	0.73	0.81	0.81
	19	D/A	30	26.0	0.54	0.63	0.63
	20	E	30	26.0	0.22	0.22	0.22
8	21	II	33	29.0	5.29	6.35	5.29
9	22	TP2	33	29.0	1.88	2.38	2.38
10	23	5	30	26.0	0.64	0.77	0.60

* Timber piles (δ_f was taken epual to ϕ_{cv}).

Table 5. Statistical analysis using different μ values for closed-ended piles

Case	μ	K_{max}	$\frac{\text{Predicted Shaft Capacity}}{\text{Measured Shaft Capacity}}$ Avg.	Max.	Min.	Sd.
1	0.03 Alawneh (1999)	$0.38\exp(0.029\,D_r)\left(\frac{D_o+0.45}{2D_o}\right)^{0.005D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84}$	1.05	1.37	0.70	0.19
2	0.05 Randolph et al., (1994)	$0.42\exp(0.029\,D_r)\left(\frac{D_o+0.45}{2D_o}\right)^{0.005D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84}$	1.01	1.31	0.66	0.18
3	$-0.1\log_{10}(D_o)$ (Proposed in this study)	$0.4\exp(0.029\,D_r)\left(\frac{D_o+0.45}{2D_o}\right)^{0.005D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84}$	1.05	1.4	0.72	0.19

Table 6. Ratio of calculated to measured shaft friction capacity using different μ values

Site No.	Serial No.	Pile No.	Measured Q_s (kN)	Calculated Q_s (kN) μ = 0.03	μ = 0.05	μ from Eq. (8)	$\frac{(Q_s)_c}{(Q_s)_m}$ μ = 0.03	μ = 0.05	μ from Eq. (8)
1	1	2	1680	1685.3	1606.5	1846.0	1.00	0.96	1.10
	2	4	1780	1265.7	1180.3	1286.3	0.71	0.66	0.72
	3	5	715	662.3	641.4	679.2	0.93	0.90	0.95
	4	6	1655	1158.2	1101.8	1225.3	0.70	0.67	0.74
2	5	G-2	803	765.1	735.7	719.4	0.95	0.92	0.90
	6	B-5	1005	965.7	954.2	994.8	0.96	0.95	0.99
3	7	G-7	433	563.0	539.4	519.2	1.30	1.25	1.20
4	8	1	828	892.8	833.1	844.6	1.08	1.01	1.02
	9	2	1000	1225.4	1184.0	1297.3	1.23	1.18	1.30
	10	3	1080`	1360.6	1327.4	1481.4	1.26	1.23	1.37
	11	4	820	746.0	720.8	738.0	0.91	0.88	0.90
	12	10	980	1232.9	1190.6	1305.2	1.26	1.21	1.33
5	13	1	1128	1097.9	1079.6	1129.8	0.97	0.96	1.00
	14	2	1300	1288.9	1247.5	1321.7	0.99	0.96	1.02
	15	3	1116	1525.5	1451.5	1558.0	1.37	1.30	1.40
	16	4	1123	1288.9	1247.5	1321.7	1.15	1.11	1.18
6	17	H-16	1540	1461.3	1399.9	1495.1	0.95	0.91	0.97
7	18	A	94	112.7	111.8	106.4	1.20	1.19	1.13
	19	D/A	254	239.9	233.5	224.9	0.94	0.92	0.89
	20	E	287	382.4	377.2	366.7	1.33	1.31	1.28
8	21	II	1106	981.9	923.2	960.4	0.89	0.83	0.87
9	22	TP2	50	57.2	50.8	48.4	1.14	1.02	0.97
10	23	5	2500	2305.8	2217.2	2571.3	0.92	0.89	1.03

Table 7. Calculated M values from Eq. (12-b) and back-calculated n values (open-ended piles)

Serial No.	Pile No.	PLR	FFR Eq.(5)	$\overline{\text{FFR}}$ Eq.(13)	M Eq.(12-a)	Back-calculated n	M^n
1	I	0.66+	0.50	0.50	0.65	0.35	0.86
2	III	0.77+	0.62	0.38	0.37	0.26	0.77
3	A	0.87+	0.73	0.27	0.42	0.52	0.64
4	1A	0.68**	0.52	0.48	(1.89)* 1.0	(1.34)* 1.0	1.00
5	P16	0.76**	0.61	0.39	(1.25)* 1.0	0.90	1.00
6	322	0.65**	0.48	0.52	(1.17)* 1.0	(1.50)* 1.0	1.00
7	1	0.83**	0.68	0.32	0.67	0.62	0.78
8	2	0.82**	0.68	0.32	0.70	0.62	0.80
9	CS	0.67**	0.51	0.49	0.82	0.64	0.88
10	1	0.82**	0.68	0.32	0.61	0.36	0.84
11	Ia	0.99+	0.86	0.14	0.28	0.72	0.40
12	Ib	0.97+	0.84	0.16	0.48	0.91	0.51
13	II	0.95+	0.82	0.18	0.73	(1.10)* 1.0	0.73
14	R1	0.78+	0.63	0.37	0.87	0.76	0.90

* Taken as 1, + measured, **calculated from Eq. (7).

Table 8. Calculated tensile unit shaft friction at the tip of a large diameter closed-ended pile ($D_o \geq$ 0.9m), ($\sigma'_{v(tip)}/P_a$) =1, tan δ = 0.50

Sand description	D_r (%)	Unit shaft friction (kPa)	
		Tip	Average
Loose	25	41	20
Medium-dense	50	80	40
Dense	75	180	90
Very dense	90	277	138

Notes: If tan δ ≠ 0.5, multiply by (tan δ/0.5). For ($\sigma'_{v(tip)}/P_a$) ≠1, multiply by $\left(\sigma'_{v(tip)}/P_a\right)^{0.16}$.
For open-ended pile, multiply by [M^n], n from Equation (19). For compression loading, multiply by 1.25.

The magnitude of the minimum value of the correction factor at the pile tip (M) is defined as (Al-Omari, 2018):

$$M = \frac{(K_{tip})_{O.E.}}{(K_{tip})_{C.E.}} \tag{12-a}$$

Based on well documented five field cases collected from literature, Al-Omari (2018) concluded that the parameter M can be estimated from the following correlation:

$$M = (1.4\,\overline{\text{FFR}} - 0.11)\left(\frac{\sigma'_{vtip}}{P_a}\right), M \leq 1.0 \tag{12-b}$$

where $\overline{\text{FFR}}$ represents the degree of pile plugging and is given by the following equation:

$$\overline{\text{FFR}} = 1 - \text{FFR} \tag{13}$$

where FFR is the final filling ratio along the final few pile diameter of pile penetration.

The correction factor at the pile tip (M) can be considered as a new plug indicator which accounts for the effect of the incremental filling ratio at final penetration (degree of plugging during driving) and the magnitude of the vertical effective stress at the pile tip.

Kikuchi et al. (2007) reported pile load test on a large-diameter open-ended steel pipe pile termed (TP-5) with an outside diameter of 1.5 m and an inner diamater of 1.444 m. The measured plug length ratio was 1.0 and the vertical effective stress at pile tip was 798 kPa. Using Equation (5), the estimated FFR = 0.87. The calculated M value from Equation (12-b) is 0.57 which is much greater than the effective area ratio ($A_{r,eff}$) which can be calculated from:

$$A_{r,eff} = 1 - FFR\left(\frac{D_i^2}{D_o^2}\right) \qquad (14)$$

This indicates that for this particular field case, the M value is about three times the value of $A_{r,eff}$ ($A_{r,eff}$ = 0.19). The effective area ratio accounts only for the final filling ratio and ignores the stress level at the pile tip.

According to Al-Omari (2018), the lateral earth pressure coefficient at the pile tip of an open-ended pile can be assessed as:

$$\left(K_{tip}\right)_{O.E.} = \left(K_{tip}\right)_{C.E.}(1 - FFR) + FFR\,K_o \qquad (15)$$

where:

$K_o =$ At-rest earth pressure coefficient.

$\left(K_{tip}\right)_{O.E.} =$ Earth pressure coefficient at the tip of the open-ended pile.

$\left(K_{tip}\right)_{C.E.} =$ Earth pressure coefficient at the tip of a similar closed-ended pile.

In this study, the ratio of the earth pressure coefficient at the tip of an open-ended pile to that of a similar closed-ended pile is assumed to be:

$$\frac{\left(K_{tip}\right)_{O.E.}}{\left(K_{tip}\right)_{C.E.}} = \left[\frac{\left(K_{tip}\right)_{C.E.}(1-FFR)+FFR\,K_o}{0.40\exp(0.029D_r)\left(\frac{Do+0.45}{2Do}\right)^{0.005D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84}}\right]^n \qquad (16)$$

Eq. (16) can be written as:

$$\frac{\left(K_{tip}\right)_{O.E.}}{\left(K_{tip}\right)_{C.E.}} = [M]^n \quad ; 0.12 \leq M \leq 1.00 \qquad (17)$$

where n is a parameter to be determined based on the database for open-ended piles presented in Table (3). It should be stated that Al-Omari (2018) used a constant value for n which equals 1.

For open-ended piles, the value of the maximum earth pressure coefficient at the pile tip can be written as:

$$(K_{max})_{O.E.} = \left[0.4\exp(0.029D_r)\left(\frac{Do+0.45}{2Do}\right)^{0.005D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84}\right].[M^n] \qquad (18)$$

In this paper, it is proposed to use this equation together with Equations (8) and (12-b) to calculate the shaft friction capacity of an open-ended pile driven in sand. Using this approach, Table (7) summarizes the calculated M values from Equation (12-b) and the back-calculated n values to fit the measured capacities for the open-ended piles used in this study. The back-calculated n values in Table (7) were found to correlate well with L/D_o ratio. The best correlation for the back-calculated n values is:

$$n = 0.018\,\frac{L}{Do}; n \leq 1.00 \qquad (19)$$

Using this approach, the ratio of calculated to measured shaft friction capacity of the open-ended piles used in this study varies from 0.59 to 1.47 with an average value of 1.06 and a standard deviation of 0.32.

Table 9. Pile characteristics, soil properties and load measurements for piles TP-1, TP-2, TP-3, TP-5 and P.R.

Site	Pile	L (m)	D_O (m)	D_I (m)	$\sigma'_{v(tip)}$ (kN/m^2)	D_R (%)	ϕ' (degree)	δ (degree)	PLR	FFR	Q_S (KN)
Kwangyang Natural Gas Plant, Korea Ko and Jeong (2015)	TP-1 (Open-ended)	8.6	0.508	0.3884	91.9	40	33	29	0.44	0.20	579
	TP-2 (Open-ended)	11.4	0.711	0.5916	115.7	45	33	29	0.76	0.60	942
	TP-3 (Open-ended)	15.5	0.9144	0.7968	150.5	45	33	29	0.85	0.58	1959
Tokyo Port Bay Site Kikuchi et al. (2007)	TP-5 (Open-ended)	86	1.5	1.444	798	78	38	29	1	-	7000
Pigeon River Site, Indiana, USA Paik et al. (2003)	P.R. (Open-ended)	7	0.356	0.292	95	30-80	30 (loose) 40 (dense)	22.2	0.82	0.78	195
	P.R. (Closed-ended)	6.85	0.356	-	93.3	30-80	30 (loose) 40 (dense)	22.2	-	-	425

Table 10. $\frac{(Q_s)_c}{(Q_s)_m}$ and percent error for prediction using method (1)

Pile	$\frac{(Q_s)_c}{(Q_s)_m}$	% Error
TP-1	0.90	-10
TP-2	1.03	+3
TP-3	1.06	+6
TP-5	1.04	+4
(Pigeon River) Closed-ended	1.18	+18
(Pigeon River) Open-ended	1.51	+51
Mean	1.12	+12

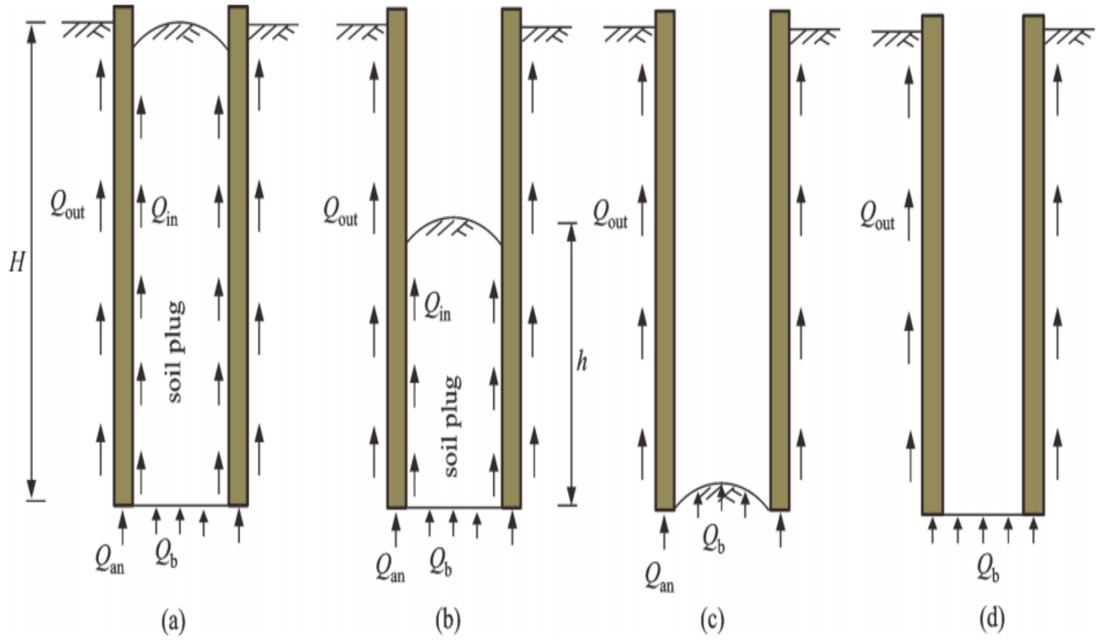

Figure (1): Modes of penetration: (a) fully coring mode, (b) partially plugged mode, (c) fully plugged mode for an open-ended pile and (d) for a closed-ended pile (Kikuchi, 2011)

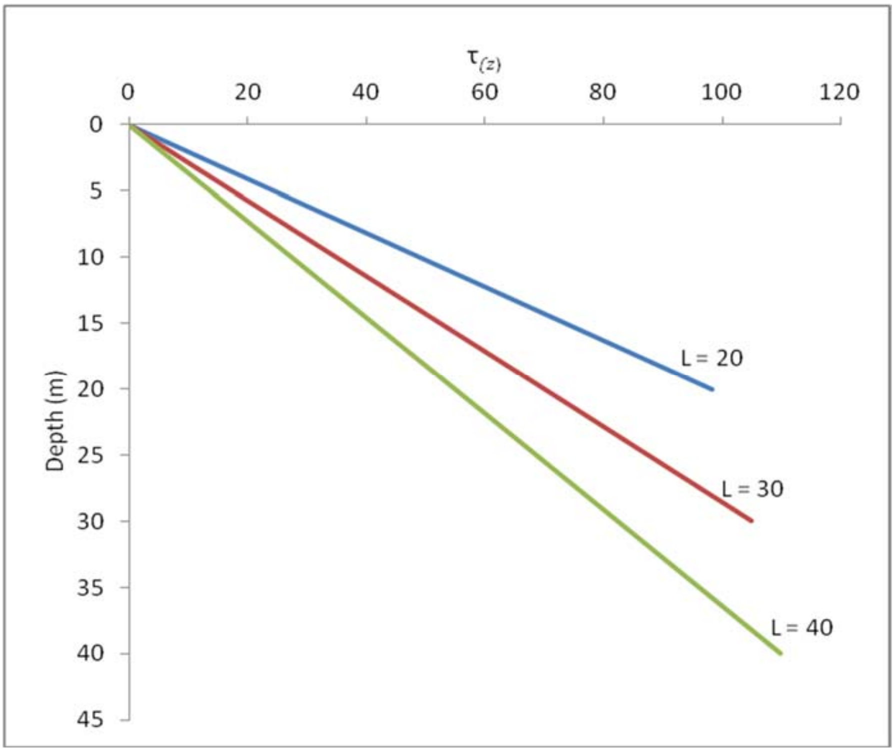

Figure (2): Variation of $\tau_{(z)}$ with depth for a hypothetical pile

Method (2): Based on Effective Pile Diameter Concept

To account for the reduced level of soil compaction during driving, Chow (1997) suggested the use of D^* (equivalent pile diameter) instead of D_o as follows:

$$D^* = \sqrt{D_o^2 - D_i^2} \quad (20)$$

On this basis, Equation (2) can be written as:

$$K_{(z)} = K_{min} + (K_{max} - K_{min})\exp\left[-\mu\left(\frac{L-z}{D^*}\right)\right] \quad (21)$$

Using this equation for $K_{(z)}$ and Equation (8) to calculate μ together with K_{max} from Equation (18), the ratio of calculated to measured shaft capacity for the open-ended piles used in this study varies from 0.50 to 1.58 with an average value of 1.06 and a standard deviation of 0.32.

Even though this method gives reasonable predictions, it has a major shortcoming. Regardless of the degree of pile plugging, the value of the earth pressure coefficient at the tip of an open-ended pile is the same and equals in magnitude the value of earth pressure coefficient at the tip of a similar closed-ended pile.

Method (3): Based on M and D* Concepts

As stated earlier, the major shortcoming of method (2) is that the earth pressure coefficient at the pile tip of an open-ended pile is the same of that for a similar closed-ended pile irrespective of the degree of pile plugging. To solve this problem, the lateral earth pressure at any depth $K_{(z)}$ is taken the smallest of the following two values:

$$K_{(z)} = \min.\left[\left(K_{(z)}\right)1, \left(K_{(z)}\right)2\right] \quad (22)$$

where:

$\left(K_{(z)}\right)1$ = Earth pressure coefficient at depth (z) from method (1).

$\left(K_{(z)}\right)2$ = Earth pressure coefficient at depth (z) from method (2), but by taking into account the degree of pile plugging during driving, by modifying Equation (20) as follows:

$$D^* = \sqrt{D_o^2 - \mathrm{PLR}\, D_i^2} \quad (23)$$

Using this equation for D* and Equation (8) for μ together with K_{max} from Equation (18), the ratio of calculated to measured shaft capacity for the open-ended piles used in this study varies from 0.60 to 1.47 with an average value of 1.0 and a standard deviation of 0.29. The statistics indicate that this method is not different from method (1) for the open-ended piles considered in this study. For longer piles, the two methods may yield different results.

Simplified Estimation of Shaft Friction Capacity of Large Diameter Pile

For closed-ended piles with diameter ($D_o \geq 0.9$ m), the value of μ from Equation (8) approaches zero and in this case, Equation (2) yields $K_{(z)} = K_{max}$:

$$K_{(z)} = K_{max} = 0.4\exp(0.029\, D_r)\left(\frac{Do+0.45}{2Do}\right)^{0.005 D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84} \quad (24)$$

The unit shaft resistance at the pile tip $\tau_{(tip)}$ is:

$$\tau_{(tip)} = K_{max}\,\sigma'_{v(tip)}\tan\delta = 40\exp(0.029\, D_r)\left(\frac{Do+0.45}{2Do}\right)^{0.005 D_r}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{0.16}\tan\delta \quad (25)$$

Along the pile, the local unit shaft friction varies linearly from zero at the pile head to a maximum value given by Equation (25) at the pile tip.

Figure (2) shows the variation of $\tau_{(z)}$ with depth for a hypothetical pile with an outside diameter equal to 1.0 m and an embedded length varying between 20 and 40 m, driven into sand with relative density equal to 50%. The effective unit weight was assumed equal to 10 kN/m^3.

Values of μ approaching zero do not mean that there is no friction fatigue during pile driving. As shown in Figure (2) and for a given soil horizon, $\tau_{(z)}$ decreases as the pile penetrates down and $\tau_{(z)}$ varies linearly with depth in a manner similar to that postulated by Toolan et al. (1990). The average unit shaft resistance is then the tip value divided by two.

The term in parentheses in Equation (25) which is a function of pile diameter can be ignored for large-diameter piles without introducing significant error. This term represents the contribution of sand dilation to shaft friction capacity during pile driving, which is small for large-diameter piles. So, Equation (25) can be furthermore simplified to:

$$\tau_{(tip)} = 40\,\exp(0.029\,D_r)\left(\frac{\sigma'_{v(tip)}}{P_a}\right)^{0.16} \qquad (26)$$

Table (8) summarizes the calculated unit shaft friction at the tip of a closed-ended pile for different sand relative densities and normalized effective vertical stress at the pile tip equal to 1.0. Pile-sand friction coefficient (tan δ) was taken equal to 0.5.

Application of the Proposed Method to Field Cases

Ko and Jeong (2015) described full-scale pile load tests (compression) conducted on three instrumented open-ended twin-walled piles at Kwangyang Plant site, Korea. The test piles, TP-1, TP-2 and TP-3 had different outer diameters and were driven into a hydraulic fill at the boundary of loose to compact sand to different embedded depths. The ground water table was at 2.5 m below the ground surface and the estimated sand relative density was in the range of 40-45%. The friction angle of the soil ϕ' was 33˚. The pile-soil friction angle was assumed to be 29˚. The effective unit weight of the sand above the GWT was taken 16 kN/m³ and for the sand below the GWT, it was taken 8.5 kN/m³. Pile characteristics, soil properties and load measurements are summarized in Table (9).

Kikuchi et al. (2007) performed pile load tests (compression) on two large-diameter open-ended steel pipe piles. The test site was located in Tokyo Port Bay, where the Tokyo Coastal Highway Bridge was planned to be constructed. The piles were driven to reach embedment depths of 73.5 m and 86 m for TP-4 and TP-5, respectively. The two tested piles were driven through layered soils with alternating clay and sand. However, the bearing strata for TP-4 and TP-5 were sandy gravel and sand layers, respectively. The estimated relative density of the sandy gravel layer was 95% and that for the sand layer was 78%. The GWT was at 7 m below the ground surface. The estimated friction angle of the sandy gravel and sandy layers was 42˚and 38˚, respectively. The effective vertical stress at the pile tip was 641 kN/m² for TP-4 and 798 kN/m² for TP-5. Pile-soil friction angle was taken 29˚. The measured shaft friction from a depth extending from 64.1 m to 73.5 m along TP-4 was 6700 kN. This range of depth includes a clay layer that extends from 63 m to 70 m. The measured shaft friction for the sandy layer that extends from 77 m to 86 m depth along TP-5 was 7000 kN. The proposed methods in this study can be used in this case on TP-5 only, since measurement of shaft friction capacity was made along sand layer (no clay), while load measurement made on TP-4 includes a clay layer that contributes to the measeured shaft capacity of the pile. Table (9) summarizes pile characteristics, soil properties, plug and load measurements for the TP-5 pile at this site.

Paik et al. (2003) described compression load tests performed on a 356 mm outside diameter, instrumented twin-walled open-ended pile (wall thickness = 32 mm) and a closed-ended pile of the same diameter. The open-ended pile was driven to 7 m and the closed-ended pile to 6.85 m through a gravelly sand deposit at Pigeon River, Indiana, USA. A 2 m thick fill was removed before pile installation. The ground water table was at a depth of 3 m below the ground surface. The first 3 m of the gravelly sand were in a loose state (D_r = 30 %) and the rest of the deposit was in a dense state (D_r varies from 70% to 90% at the pile tip). The estimated friction angle of the sand at the site using CPT-data and the correlation by Lee et al. (2004) was 30˚ and 40˚ for loose and dense sand, respectively. As reported by Paik et al. (2003), the pile-sand friction angle (δ) was 22.2˚. The estimated unit weight of the loose sand above GWT is

16.5 kN/m³ and the saturated unit weight of the dense sand is 21.2 kN/m³. Load measurements and soil-pile characteristic are also summarized in Table (9).

For each pile in Table (9), pile shaft capacity was calculated using method (1) and the ratios of calculated to measured pile capacity are summarized in Table (9). Table (9) also summarizes the percent error for each case which is defined as:

$$\%\text{Error}\,\frac{\text{calculated capacity}-\text{measured capacity}}{\text{measured capacity}} * 100\% \quad (27)$$

Positive mean error means that pile capacity is over-estimated, while negative mean error implies that pile capacity is under-estimated.

In general, method (1) gives an acceptable prediction, indicating that the method has sound physical and theoretical meanings.

The use of method (2) yields similar results as method (1) for the cases in Table (9); this is because $[K_{(z)}]_1$ is smaller than $[K_{(z)}]_2$ for all cases and thus $[K_{(z)}]_1$ controls pile capacity. On the other hand, method (2) significantly over-estimates pile capacities mainly, because in this method, the earth pressure coefficient is the same for open- and closed-ended piles.

CONCLUSIONS

Based on the work conducted in this study, the following conclusions are drawn:

1. The driving pressure (i.e., plug capacity) which pushes the soil column inside an open-ended pile during driving is larger in dense sand than in loose or medium- dense sand. As a result, driven piles in dense sand may experience higher plug length ratios than those in loose and medium-dense sand. Piles in loose sand (D_r = 23%) may experience PLR equal to 82% of that for piles in dense sand. For piles in medium-dense sand (D_r = 56%), this percentage is about 92%.
2. The parameter μ which represents the exponential decay rate of unit shaft friction as the pile tip penetrates down can be expressed as a function of pile diameter. The magnitude of μ decreases as the pile diameter increases.
3. For large-diameter piles, small values of μ (close to zero) do not mean that there is no friction fatigue process during pile driving. They simply mean that local unit shaft friction varies linearly with depth and sliding triangle is the best to describe this variation as the pile tip penetrates down. As such, friction fatigue effect is incorporated as the pile tip penetrates down, but in a manner nearly close to that postulated by Toolan et al. (1990).
4. For driven closed-ended piles in sand, the magnitude of the maximum earth pressure coefficient at the pile tip is about 15% greater than that in Alawneh (1999) study, when μ is taken as a function of pile diameter.
5. For open-ended piles, method (1) is the best among the three proposed methods in this study. Using an independent database comprising six full-scale pile load tests collected from literature, the ratio of predicted (using method 1) to measured shaft capacity varies from 0.9 for TP-1 to 1.51 for the Pigeon River pile (open-ended). Method (3) yielded reasonable and comparable results to method (1). On the other hand, shaft capacities of these six piles were over-estimated when using method (2), mainly because in this method, K_{max} is the same for open- and closed-ended piles and the piles are relatively short.
6. It is believed that method (2) which is based on equivalent pile diameter concept (D*) will give reasonable predictions for long piles with low final filling ratio (FFR) along the last few diameters of pile penetration and relatively high plug length ratio (PLR) along the upper locations of the pile.

LIST OF ABBREVIATIONS

Abbreviation	Description
$A_{r,eff}$	Effective area ratio
CF	Correction factor which varies linearly with depth
Di	Inner diameter of the pile
D_o	Outer diameter of the pile
D_r	Sand relative density
D*	Equivalent pile diameter
IFR	Incremental filling ratio
$(IFR)_{ave}$	Average incremental filling ratio
FFR	Final filling ratio
K	Lateral earth pressure coefficient
K_o	Lateral earth pressure coefficient at rest
$K_{(z)}$	Lateral earth pressure coefficient at depth (z)
$[K_{(z)}]_{O.E.}$	Earth pressure coefficient at any depth (z) along the open-ended pile
$[K_{(z)}]_{C.E.}$	Earth pressure coefficient at any depth (z) along a similar closed-ended pile
$(K_{(z)})_1$	Earth pressure coefficient at depth (z) from method (1)
$(K_{(z)})_2$	Earth pressure coefficient at depth (z) from method (2)
$(K_{tip})_{O.E.}$	Earth pressure coefficient at the tip of the open-ended pile
$(K_{tip})_{C.E.}$	Earth pressure coefficient at the tip of a similar closed-ended pile
K_{max}	Maximum lateral earth pressure coefficient
K_{min}	Minimum lateral earth pressure coefficient
L	Pile embedded length
L_p	Total length of soil column inside the pile at the end of driving
ΔL	Increment of pile penetration length
ΔL_P	Increment of soil plug length
M	Correction factor at the pile tip
P_a	Atmospheric pressure (101.3 kPa)
PLR	Plug length ratio
Q_s	Uplift shaft friction capacity
$(Q_s)_c$	Calculated shaft friction capacity
$(Q_s)_m$	Measured shaft friction capacity
z	Depth below ground surface

GREEK ABBREVIATIONS

Abbreviation	Description
γ'	Effective unit weight
$\gamma'_{ave.}$	The average effective unit weight
Ψ	The ratio of PLR in loose or medium-dense sand to PLR in dense sand
φ'	Effective soil friction angle
φ'_{cv}	Effective constant volume friction angle
μ	Exponential decay rate
δ	Pile-sand friction angle
δ_f	Pile-sand friction angle at failure

σ'_h	Horizontal effective stress
σ'_{rf}	Radial effective stress at failure
σ'_{rc}	Radial effective stress after installation and before loading
$\Delta\sigma'_{rd}$	The change in the effective radial stress during loading due to dilation
$\sigma'_{v.tip}$	Vertical effective stress at the pile tip
σ'_v	Vertical effective stress
τ_{lim}	Limiting unit shaft resistance
$\tau_{(z)}$	Unit shaft resistance
τ_{OE}	Unit shaft friction at depth (z) along the open-ended pile
τ_{CE}	Unit shaft friction at depth (z) along the closed-ended pile
τ_{tip}	Unit shaft resistance at the pile tip

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