A Modification of Pile Unit Skin (f) of Frictional Resistance (Qs) on Clayey Layer at Central of Jakarta

by Putera Agung Maha Agung

A Modification of Pile Unit Skin *(f)* of Frictional Resistance *(Q_s)* on Clayey Layer at Central of Jakarta

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Keywords: frictional resistance, pile foundation, CPT, and PDA test.

Abstract: The evaluation of pile unit skin friction *(f)* is important and difficult in determining frictional resistance (Q_s) of single pile. A modification was conducted on curves and table design of *(f)* in clayey soil from Nottingham & Schmertmann and Schmertmann methods. PDA test was used as the valid data to compare the prediction analysis result using these methods. Resistance friction ($Q_{s(u)}$) and ($Q_{s(Ru)}$), respectively of single pile were obtained by 5 random points of CPT analyzed by Schmertmann methods and 2 points of PDA test evaluated by CASE and CAPWAP methods from study site. From the result of modification of unit skin friction analysis using CPT data approaching to the real data from PDA test results.

1 INTRODUCTION

Generally, cone penetration test (CPT) is extensively used in the building project. Cone penetration testing (CPT) is an easy, a fast and a reliable means of conducting building design site investigations for exploring soils and soft ground for support of foundations. For the small to medium project, engineer has to design with using CPT data only, so they must search the confirmatory data through some correlation between parameter from CPT data results to complete the shortage data. Thus, the all prediction for frictional resistance ($Q_{s(u)}$) parameter design of axial bearing capacity (Q_u) from CPT results must be examined by a loading test results at the field in determinating the safety factor (SF).

There are many methods for evaluation ($Q_{s(u)}$) based on CPT data (Aoki and De Alencar (1975), Clisby et al (1980), Schmertmann (1978), de Ruiter and Beringen (1979), Bustamante and Gianeselli (LCPC/LCP) (1982), Tumay and Fakhroo (1982), Philipponnat (1980), Price and Wardle (1982) methods. However, it was not for all methods showed the best capability (Titi and Abu-Farsakh, 1999) in predicting of ($Q_{s(u)}$) using (PPC) piles driven into a certain area. This research effort was focused on the applicability of one of CPT methods to predict the frictional resistance of piles from CPT data (Schmertmann, 1978). The predicted capacity ($Q_{s(u)}$) using CPT data was compared to the reference pile load capacity ($Q_{s(Ru)}$) obtained from the pile load test using PDA method. The CPT method were used to investigate the load carrying capacity of square precast prestressed concrete (PPC) piles of 30 cm in size driven into Jakarta clayey soil (SNI 2847, 2013). Criteria for acceptance of ($Q_{s(u)}$) (ultimate frictional resistance from CPT) should be equals or less than ≤ ($Q_{(sRu)}$) (from PDA results). Some previous studies for clayey soil, Cummings et al, (1950), Seed and Reese (1955), Bjerrum et al. (1958), Soderberg (1962), Begemann (1965), Tomlinson (1957), and Poulos (1989), etc were suggested to determine the unit skin friction *(f)* in determining the frictional resistance ($Q_{s(u)}$) of single pile by direct and or indirect approaches. This study proposed the modification parameter analysis for unit skin friction *(f)* in determining ($Q_{s(u)}$) of clayey soil layers at central of Jakarta.

2 METHODOLOGY

2.1 CPT Data

Soil investigation uses a set of CPT equipment complete with the auxiliary devices. The CPT is a light weight model with a compressive capacity of 2.5 tons. Conus used is friction cone type (biconus type) with a cross-sectional area unit of 10 cm^2, area of blanket 123 cm^2. CPT is carried out continuously at intervals of 20 cm depth penetration to show maximum cone (tip) resistance and maximum shear

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of 250 kg/cm^2, or up to a maximum depth of 30 meters. Data set presented are the cone tip resistance and the total friction (q_c) and ratio local friction (t_f) against cone tip resistance or (t_f/q_c) or friction ratio (F_R) values to a maximum depth of 30 meters. Table 1 shows the results of the soil investigation using CPT shows in the table below related with maximum depth of CPT, depth of hard soil, t_f and q_c values for all soil layers at the surface area. Table 2 identifies that the estimation of geostratigraphic profile using several methods (Begemann, 1965; Schmertmann, 1978; and Robertson and or et al, 1986; 1990). Table 3 shows 5 (five) CPT data (S-1 to S-5) provides the information of hard layer depth with (q_c) value between $60 < q_c <$ 150 kg/cm^2 and thickness of the bearing layer with (q_c) value > 150 kg/cm^2. Then, Figure 1 shows that geostratigraphic soil profile based on cone penetration test (CPT) of study site. Based on the 5 (five) CPT (S-1 to S-5), it could be indicated the condition of surface layers consisting of medium to stiff silty clay with cone resistance (q_c) values between 27 to 34 kg/cm^2 and at the depth between 0.40 to 3.60 m.

CPT data normally can be used to determine the other of soil parameter using some correlations required for deep foundation design, especially in determining frictional resistance of single pile ($Q_{s(u)}$). From some previous design reports and studies, Schmertmann (1978) method was selected and considered suitable for study area, besides the method was widely used CPT methods in determining bearing capacity (Q_u) in Indonesia.

Table 1: The results of CPT data.

CPT Point	Maximum depth (m)	Depth (m) with (q_c) > 100kg/cm^2	t_f value of depth (m) with (q_c) > 100 kg/cm^2	(q_c) value at the depth 1.00 m (kg/cm^2)	Ground water table (GWL) (m)
S-1	10.80	10.40	1720	23	- 6.40
S-2	11.20	10.60	1728	65	- 7.20
S-3	11.60	10.20	800	31	- 6.00
S-4	11.00	10.20	1445	15	- 5.60
S-5	11.20	10.00	1469	33	- 5.40

2.2 PDA Data

The PDA test is to evaluate the ultimate load carrying-capacity (Q_u) of the pile, the integrity / integrity of the pile and pole drop. PDA test normally are used as field test additional to replace the conventional loading test. Data results of PDA test was obtained from 2 locations of pile foundation system configuration at a sub-structure of BPJS Building of Central Jakarta project (See Appendix).

All calculation results of ($Q_{(sRu)}$) by PDA test actually uses top force pile and velocity signals, obtained using instrumentation or device, such as :

Table 2: The results of geostratigraphic profile from CPT.

CPT point	Depth of soil layer	Average (q_c) value (kg/cm^2)	Average (t_f) value (kg/cm^2)	Average (F_R) value (kg/cm^2)	Description	Remarks
S-1	0.00 – 0.20	-	-	-	-	Used concrete
	0.40 – 1.60	27	1.31	4.89	Medium silty clay	
	1.80 – 3.40	32	1.36	4.09	Stiff silty clay	
	3.60 – 7.00	26	1.85	7.48	Medium clay	
	7.20 – 10.00	45	1.91	4.63	Stiff silty clay	
	10.20 – 10.40	106	1.34	1.05	Dense silty sand	
	10.60 – 10.80	210	3.25	1.91	Very dense silty sand	q_c > 150 kg/cm^2
S-2	0.00 – 0.60	-	-	-	-	Used concrete
	0.80 – 3.20	40	1.33	3.64	Stiff silty clay	
	3.40 – 5.60	21	1.30	6.35	Medium clay	
	5.80 – 8.40	36	1.52	4.05	Stiff silty clay	
	8.60 – 10.40	77	2.86	3.89	Very stiff silty clay	
	10.60 – 11.20	197	1.83	1.06	Very dense silty sand	q_c > 150 kg/cm^2
S-3	0.00 – 0.40	-	-	-	-	Used concrete
	0.60 – 2.20	33	0.81	2.49	Stiff silty clay	
	2.40 – 6.80	23	0.62	2.83	Medium silty clay	
	7.00 – 8.80	51	1.07	2.17	Medium dense silty sand	
	9.00 – 10.80	90	1.12	1.33	Dense silty sand	
	11.00 – 11.60	205	1.83	0.95	Very dense silty sand	q_c > 150 kg/cm^2
S-4	0.00 – 0.60	-	-	-	-	Used concrete
	0.80 – 3.60	29	1.15	4.53	Stiff silty clay	
	3.80 – 6.60	15	0.83	5.47	Medium silty clay	
	6.80 – 9.20	37	1.74	4.76	Stiff silty clay	
	9.40 – 10.20	86	4.00	5.02	Very stiff silty clay	
	10.40 – 11.00	197	2.30	1.19	Very dense silty sand	q_c > 150 kg/cm^2
S-5	0.00 – 0.40	-	-	-	-	Used concrete
	0.60 – 2.20	34	1.32	4.09	Stiff silty clay	
	2.40 – 6.80	23	0.71	3.29	Medium silty clay	
	7.00 – 7.60	35	1.44	3.97	Stiff silty clay	
	7.80 – 10.20	78	3.13	4.34	Very stiff silty clay	
	10.40 – 11.20	197	2.24	1.24	Very dense silty clay	q_c > 150 kg/cm^2

Table 3: The results of hard and bearing layers from CPT.

CPT point	Hard layer with $60 < q_c < 150$		Bearing layer with $q_c > 150$		Remarks
	Depth (D) (m)	q_c values (kg/cm^2)	Depth (D) (m)	q_c values (kg/cm^2)	
S-1	$10.20 < D \leq 10.40$	$106 < 150$	$10.60 < D \leq 10.80$	210	
S-2	$8.60 < D \leq 10.40$	$77 < 150$	$10.60 < D \leq 11.20$	197	
S-3	$9.00 < D \leq 10.80$	$90 < 150$	$11.00 < D \leq 11.60$	205	
S-4	$9.40 < D \leq 10.20$	$86 < 150$	$10.40 < D \leq 11.00$	197	
S-5	$7.80 < D \leq 10.20$	$78 < 150$	$10.40 < D \leq 11.20$	197	

piezoelectric accelerometers and bolt-on of strain transducers attached to the pile near its top. PDA conditions and calibrates these signals and velocity. Using case method solutions, the PDA calculates the results on BPJS Building project site (Figure 2) described in the following section. PDA test were applied in two piles load testing sites for axial bearing capacity and one pile for lateral displacement. To help examine the soil conditions and design calculation, the results of the investigations which nearest the PDA pile tested for CPT-S1 and CPT-S2 as shown as Table 1; 2; and 3.

Pile testing data results are shown from Table 4 to 6. Table 4 shows the existing pile data for evaluation of bearing capacity and settlement. Table 4 shows the maximum compression and tension stresses results from field testing. Table 5 identifies the PDA test data reading result from field test and after analysis using manually CASE analysis and CAPWAP *(Case Pile Wave Analysis Program)* software. Both measurements and analyses generate the end bearing capacity (Q_b) and shaft friction (Q_s). However, this study will discuss shaft or skin friction *(f)* only as shown as with *"RED ELLIPSE-CIRCLE"* on the data of Table 6. This frictional resistance from PDA test ($Q_{(sRu)}$) or *"Actual Values"* will be compared with the calculation using CPT data ($Q_{s(u)}$) or *"Estimation Values"* for this study.

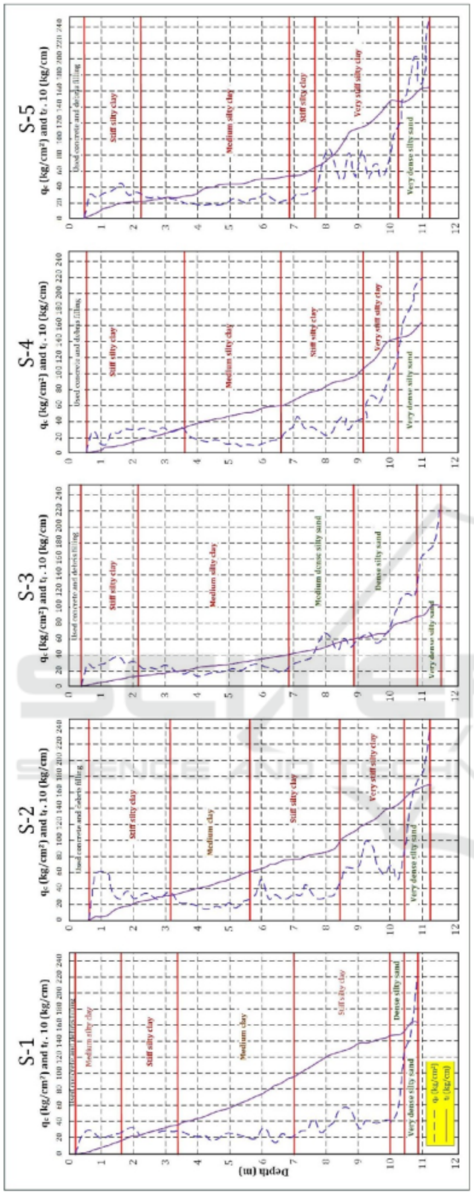

Figure 1: Soil profile based on (CPT) parameters.

Table 4: Pile data for PDA test.

No.	Pile Number	Installation date	Pile dimension (cm)	Pile shape	Pile length (m)	Pile length at below of instrument (m)	Pile embedded length (m)	Weight of hammer (tonne)
1.	As 3 No. 58	26-10-18	□ 30 x 30	Square	10.50	9.90	9.50	1.20
2.	As. 4 No. 73	12-10-18	□ 30 x 30	Square	9.60	9.15	8.95	1.20

Figure 2: Piling and instalation of PDA test.

Table 5: Maximum compression and tension stresses.

No.	Pile Number	Height of drop hammer (m)	Impact energy (m)	Hammer efficiency (%)	Hammer type	Maximum compression stress (MPa)	Maximum tension stress (MPa)
1.	As 3 No. 58	0.75	0.52	57.78	Drop hammer (1.20 tonne)	14.5	1.80
2.	As. 4 No. 73	0.75	0.68	75.56	Drop hammer (1.20 tonne)	19.4	1.30

Table 6: PDA analysis data results (CASE & CAPWAP)

No.	Pile Number	PDA	CAPWAP ANALYSIS			Final set (mm/blow)	BTA (%)
			Bearing capacity (ton)	Friction resistance (ton)	End bearing resistance (ton)		
1.	As 3 No. 58	137	137.3	86.9	50.4	0.10	1.80
2.	As. 4 No. 73	166	166.7	97.5	69.2	0.10	1.30

2.3 Frictional Resistance using CPT

Estimating the frictional or skin resistance (Q_s) of piles in clay layers is almost as difficult a task as estimating that in sand layers, due to the presence of several variables that can not easily be quantified. Several methods for obtaining the unit frictional resistance of piles are described in the literature (Vijayvergiya and Focht, 1972, etc). However, in this study some analyses use Nottingham and Schmertmann (1975) and Schmertmann (1978) only in determining ($Q_{s(u)}$).

The correlation to evaluate unit skin friction *(f)* in clay (with $\phi = 0$) to be:

- Generally, axial bearing capacity equation:
  $$Q_u = Q_b + Q_s \quad (1)$$
- Frictional resistance from CPT data ($Q_{s(u)}$):
  $$Q_{su} = \sum f \, . \, p(\Delta L) = \sum \alpha' . f_c . p . (\Delta L) \quad (2)$$
- Unit skin friction can be determined by:
  $$f = \alpha' \, . \, f_c \quad (3)$$
- The variation of (α') with the frictional resistance (f_c) is shown in Figure 2.
- Where (α) equals an empirical adhesion factor. The approximate variation of the value of (α) is shown in Table 7. It is important to realize that the

values of a gives in Table 7 may vary somewhat, since (α) is actually a function of vertical effective stress (σ'_o) and the undrained cohesion (c_u).

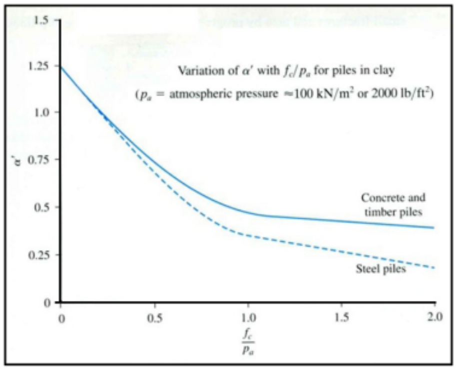

Figure 2: Variation of f_c / P_a and α' (Nottingham & Schmertmann (1975); Schmertmann (1978)).

Table 7: Variation of f_c / Pa and α
(Terzaghi, Peck, and Mesri (1996); Das, 2007).

$\frac{c_u}{p_a}$	α
≤ 0.1	1.00
0.2	0.92
0.3	0.82
0.4	0.74
0.6	0.62
0.8	0.54
1.0	0.48
1.2	0.42
1.4	0.40
1.6	0.38
1.8	0.36
2.0	0.35
2.4	0.34
2.8	0.34

Note: p_a = atmospheric pressure ≈ 100 kN/m² or 2000 lb/ft²

From the detail explanation above, in determining α value requires laboratory testing for undisturbed samples. However, for this study, soil investigation was carried out CPT only. Thus, this study conducted needs to propose a modification α values in determining pile unit skin *(f)* from the comparison between ($Q_{s(u)}$) and ($Q_{s(Ru)}$), so that it would be suitable for study site in determining (Q_u) of a pile.

3 RESULT AND DISCUSSION

Frictional resistance analysis based on CPT (Schmertmann, 1978) ($Q_{s(u)}$) and PDA (CASE and CAPWAP methods) ($Q_{s(Ru)}$) data are compared each other and obtained the Figure 3. In the field, the position of CPT-S1 is near PDA-3, and also S-2 closes to PDA-4, respectively.

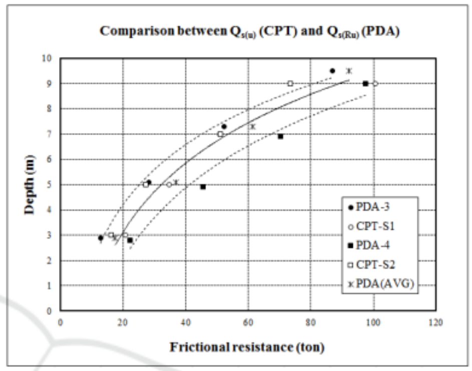

Figure 3: ($Q_{s(u)}$) and ($Q_{s(Ru)}$).

From Figure 3, parameter of *(f)* or *(f_c)* and α and or α' can be determined by back analysis using average of line equation of ($Q_{(s(Ru)}$) directly. Thus, to calculate *(f)* or *(f_c)* can be used the Equation (4) and (5), respectively:

$$\sum f = \frac{\overline{Q_{u(Ru)}}}{\sum p(\Delta L)} \quad (4)$$

$$\sum f_c = \frac{\overline{Q_{u(Ru)}}}{\sum \alpha' p(\Delta L)} \quad (5)$$

where $f = \alpha' . f_c$.

And, the results of modification of Nottingham & Schmertmann (1975) and Schmertman (1978) can be shown in Figure 4 and Table 8.

Since the study is only for concrete piles, the Figure 4 is applicable for deep foundation of pile made from reinforcement concrete or square precast prestressed concrete (PPC) piles system. And, Figure 4 is only valid for clayey layers at central of Jakarta. However, this study can be developed for another places in Indonesia which is same as Jakarta soil geostratigraphy.

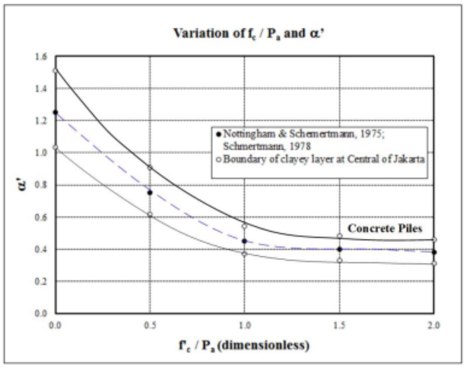

Figure 4: Modification relationships between variation of f_c / P_a and α' values

Table 8: Modification of Terzaghi et al (1996)

Terzaghi et al (1996)		Clayey layer of Jakarta Central
c_u/P_a	**α**	**Range of α**
0.1	1.00	0.83 – 1.21
0.2	0.92	0.76 – 1.11
0.3	0.82	0.68 – 0.99
0.4	0.74	0.61 – 0.90
0.6	0.62	0.51 – 0.75
0.8	0.54	0.45 – 0.65
1.0	0.48	0.40 – 0.58
1.2	0.42	0.35 – 0.51
1.4	0.40	0.33 – 0.48
1.6	0.38	0.31 – 0.46
1.8	0.36	0.30 – 0.44
2.0	0.35	0.29 – 0.42
2.4	0.34	0.28 – 0.41
2.8	0.34	0.28 – 0.41
Note: Pa = atmospheric pressure ≈ 100 kPa		

4 CONCLUSIONS

Study analysis investigations have indicated that unit skin friction *(f)* in determining frictional resistance (Q_s) on clayey layers of Jakarta Central reaches about ± 17 to 18 percent of the previous studies (Nottingham & Schmertmann, 1975; Schmertman, 1978). This matter, it can be shown in Figure 4 and Table 8, where α and or α' values on each modification has a certain range of these values. However, in application it should be considered with vertical effective stress (σ'$_o$) and the undrained cohesion (c_u) from laboratory data for undisturbed sample.

These values obtained were within ranges established by governmental authority regulations. Pile unit friction *(f)* showed significant changes during instalation for any time period. α and or α' values of CPT data proved more sensitive than c_u values changes for determining frictional resistance during the pilling. Losses of unit friction *(f)* can increase with pore water pressure around pile shaft.

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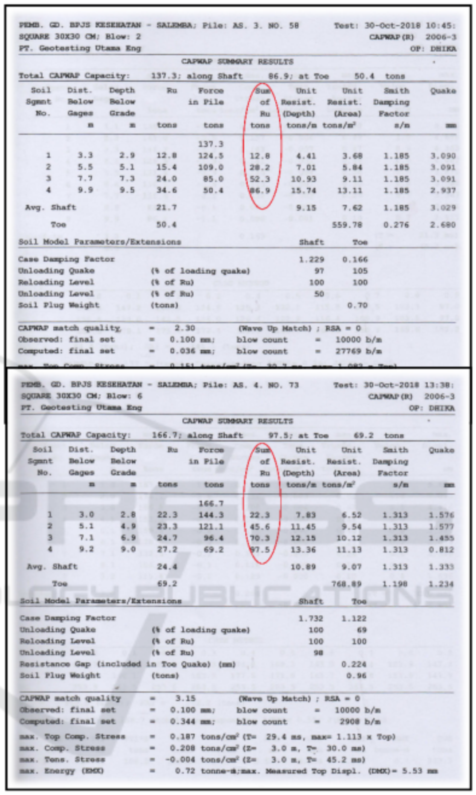
APPENDIX

PDA DATA



PEMB. GD. BPJS KESEHATAN - SALEMBA; Pile: AS. 3. NO. 58 — Test: 30-Oct-2018 10:45:
SQUARE 30X30 CM; Blow: 2 — CAPWAP(R) 2006-3
PT. Geotesting Utama Eng — OP: DHIKA

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 137.3; along Shaft 86.9; at Toe 50.4 tons

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru tons	Force in Pile tons	Sum of Ru tons	Unit Resist. (Depth) tons/m	Unit Resist. (Area) tons/m²	Smith Damping Factor s/m	Quake mm
				137.3					
1	3.3	2.9	12.8	124.5	12.8	4.41	3.68	1.185	3.090
2	5.5	5.1	15.4	109.0	28.2	7.01	5.84	1.185	3.091
3	7.7	7.3	24.0	85.0	52.3	10.93	9.11	1.185	3.091
4	9.9	9.5	34.6	50.4	86.9	15.74	13.11	1.185	2.937
Avg. Shaft			21.7			9.15	7.62	1.185	3.029
Toe			50.4				559.78	0.276	2.680

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.229	0.166
Unloading Quake	(% of loading quake)	97	105
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Soil Plug Weight	(tons)		0.70

CAPWAP match quality = 2.30 (Wave Up Match) ; RSA = 0
Observed: final set = 0.100 mm; blow count = 10000 b/m
Computed: final set = 0.036 mm; blow count = 27769 b/m

PEMB. GD. BPJS KESEHATAN - SALEMBA; Pile: AS. 4. NO. 73 — Test: 30-Oct-2018 13:38:
SQUARE 30X30 CM; Blow: 6 — CAPWAP(R) 2006-3
PT. Geotesting Utama Eng — OP: DHIKA

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 166.7; along Shaft 97.5; at Toe 69.2 tons

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru tons	Force in Pile tons	Sum of Ru tons	Unit Resist. (Depth) tons/m	Unit Resist. (Area) tons/m²	Smith Damping Factor s/m	Quake mm
				166.7					
1	3.0	2.8	22.3	144.3	22.3	7.83	6.52	1.313	1.576
2	5.1	4.9	23.3	121.1	45.6	11.45	9.54	1.313	1.577
3	7.1	6.9	24.7	96.4	70.3	12.15	10.12	1.313	1.455
4	9.2	9.0	27.2	69.2	97.5	13.36	11.13	1.313	0.812
Avg. Shaft			24.4			10.89	9.07	1.313	1.333
Toe			69.2				768.89	1.198	1.234

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.732	1.122
Unloading Quake	(% of loading quake)	100	69
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	98	
Resistance Gap (included in Toe Quake) (mm)			0.224
Soil Plug Weight	(tons)		0.96

CAPWAP match quality = 3.15 (Wave Up Match) ; RSA = 0
Observed: final set = 0.100 mm; blow count = 10000 b/m
Computed: final set = 0.344 mm; blow count = 2908 b/m
max. Top Comp. Stress = 0.187 tons/cm² (T= 29.4 ms, max= 1.113 x Top)
max. Comp. Stress = 0.208 tons/cm² (Z= 3.0 m, T= 30.0 ms)
max. Tens. Stress = -0.004 tons/cm² (Z= 3.0 m, T= 45.2 ms)
max. Energy (EMX) = 0.72 tonne-m; max. Measured Top Displ. (DMX)= 5.53 mm

A Modification of Pile Unit Skin (f) of Frictional Resistance (Qs) on Clayey Layer at Central of Jakarta

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