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Comparative Study between Design Methods and Pile Load Tests for Bearing Capacity of Driven PHC Piles in the Nakdong River Delta

낙동강 삼각주에 항타된 PHC말뚝의 지지력을 위한 재하시험과 지지력 공식의 비교연구

Dung, N. T.[1]

Chung, S. G.[2] 정 성 교

Kim, S. R.[3] 김 성 렬

Chung, J. G.[4] 정 진 교

요 지

우리나라에서는 암반 및 자갈층과 같은 단단한 층 내에 깊은 기초를 매입하는 것이 일반적이다. 그러나 Chaophraya(Bangkok)와 Mississippi강 삼각주에서 실시되고 있는 것과 같이, 대심도 낙동강 삼각주 퇴적토에서도 말뚝의 지지층으로써 중간 깊이에 위치하는 모래 및 모래질 자갈층을 고려할 필요가 있다. 이 연구는 이 지역에서 PHC 말뚝을 요구하는 깊이까지 항타할 때, 말뚝의 지지력을 위한 적절한 평가법을 찾고자 하였다. 지반조사는 두 현장의 5개소에서 실시되었다. 말뚝의 지지력은 지반조사 결과를 이용하고 CPT에 근거한 평가법과 여러 다른 해석법을 적용하여 계산되었으며, 상호 비교가 이루어 졌다. 항타된 5개의 말뚝에 대해 매입된 전 깊이에 걸쳐 잘 알려진 PDA시험이 체계적으로 수행되었다. 여러 평가법에 의하여 계산된 지지력은 PDA 및 정재하 시험결과와 함께 비교되었다. 그 결과, 주면마찰력은 set-up 효과에 따라 지배적으로 영향을 받으며, 장시간 경과 후에는 β법에 의한 결과와 좋은 일치를 보였다. 선단 지지력은 과소평가되는 PDA시험 보다는 정재하시험결과에 근거하여 적절한 평가법을 선정하였다. 최종적으로, CPT결과를 이용하여 이 지역에 적합한 지지력의 평가법을 도출하였다.

Abstract

Deep foundations have been popularly installed in hard stratum such as gravels or rocks in Korea. However, it is necessary to consider sand or sandy gravel layers that locate at the mid-depths as the bearing stratum of piles in the thick Nakdong River deltaic deposits, as done in the Chaophraya (Bangkok) and Mississippi River deltas. This study was focused on the finding of suitable methods for estimating bearing capacity when driving prestressed high-strength concrete (PHC) piles to a required depth in the deltaic area. Ground investigation was performed at five locations of two sites in the deltaic area. Bearing capacity of the driven piles has been computed using a number of proposed methods such as CPT-based and other analytical methods, based on the ground investigation and comparison one another other. Five PDA (pile driving analyzer) tests were systematically carried out at the whole depths of embedded piles, which is a well-known useful technique for the purposes. As the results, the bearing capacities calculated by various methods were compared with the PDA and static load testing results. It was found that the shaft resistance is significantly governed by set-up effects and then the long-term value agrees well with that of the β method. Also, the design methods for toe resistance were determined based on the SLT result, rather than PDA results that led to underestimation. Moreover, using the CPT results, appropriate methods were proposed for calculating the bearing capacity of the piles in the area.

Keywords : Bearing capacity, PHC pile, Drivability, CPT, PDA, Sand

1 Member, PhD Student, Dong-A Univ., School of Civil Engrg., Busan, Korea
2 Member, Prof., Dong-A Univ., School of Civil Engrg., Busan, Korea, sgchung@dau.ac.kr, Corresponding Author
3 Member, Assistant Prof., Dong-A Univ., School of Civil Engrg., Busan, Korea
4 Member, Associate Prof., Pusan Information Technology College, Dept. of Civil Engrg., Busan, Korea

1. Introduction

In the west marginal lands of Busan city and its vicinity, which are located in the mouth of the Nakdong River, reclamation works have been started to develop industrial and residential complexes, since early 1990s. Although the new development was complete, most of the developed lands have been lying vacant for a long time. As unusually soft and thick clay was deposited in the area, the high construction cost for deep foundations was a prablem for the housing development. The costly foundations come from long piles (e.g. steel pipe piles) founded on rocks and/or gravel layer, sometimes from bitumen coating.

Case histories of friction pile in the other deltaic areas, such as Chaophraya river delta (Phien-wej, 2006) and Mississippi River deltas (Blessey, 1976), are interesting to be observed. These deltas are well-known as unusually deep deposits where bedrock exists at the depths of 400-500 m and 1-2 km respectively. And also, deep foundations have mostly been installed into the medium or dense sand layers in both areas. Therefore, it is necessary to consider whether friction piles in sand layer is possible to be applied in the Nakdong River estuary.

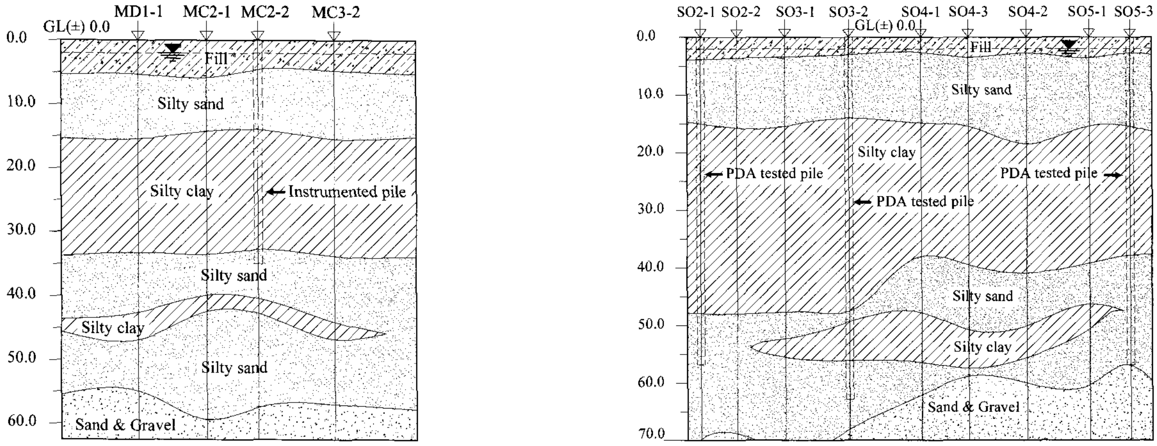
The aim of this study is to examine bearing capacities of piles at various depths in the sand layers of the Nakdong River deltaic deposit. For this, two locations were chosen in two different sites. A comprehensive geotechnical investigation was performed to determine soil parameters for pile design, using high-capacity CPT equipment, BST (borehole shear test) etc. A number of CPT-based design and other analytical methods were applied to calculate the bearing capacity and then the calculated results were compared with PDA analysis and static loading test (SLT) results. Unlike the steel pipe piles that are usually used in the area, the PHC piles of 600 mm in diameter were chosen and driven up to gravelly sand layer overlying the lower sand layer or to the sand layer. PDA (pile driving analyzer) tests were systematically performed for every blow during pile driving. Based on the results of PDA tests, the bearing capacity (resistance) as well as the drivability was analyzed. Finally, by comparing the measured and calculated bearing capacities, appropriate design methods are proposed to evaluate the proper bearing capacity in the area.

2. Study Sites and Ground Conditions

The study sites were Myeongji (MJ) and Shinho (SH) areas in the Nakdong River estuary, as shown in Fig. 1. Ground investigation for pile design was performed in the MJ and SH sites and the soil profiles are shown in Fig. 2. The fill of about 5 m thick was placed on the original ground surface and followed by loose silty sand (upper sand), soft clayey silt (upper clay), dense sand (lower sand) and sandy gravel on bed rock. Thin clayey silt mostly is sandwiched in the lower sand layer.

The geotechnical profiles for both the sites are shown in Figs. 3 and 4, in which the presented soil parameters were adopted for pile design. The soil parameters of the sand were determined based on CPTU data: The unit weight was obtained based on the soil classification system proposed by Robertson et al. (1986); the effective friction angle was obtained from the chart of the relationship between effective stress and cone tip resistance proposed by Robertson & Campanella (1983a). However, most of the soil parameters of the clay were determined from laboratory tests. The undrained shear strength, S_u, was determined from both laboratory (CIU) and field (CPT and Field vane) tests. The corrected vane strength by the method of Aas et al. (1986) is close to $0.22\sigma'_{v0}$,

Fig. 1 Locations of study sites



(a) MJ site

(b) SH site

Fig. 2. Boring logs and soil profile

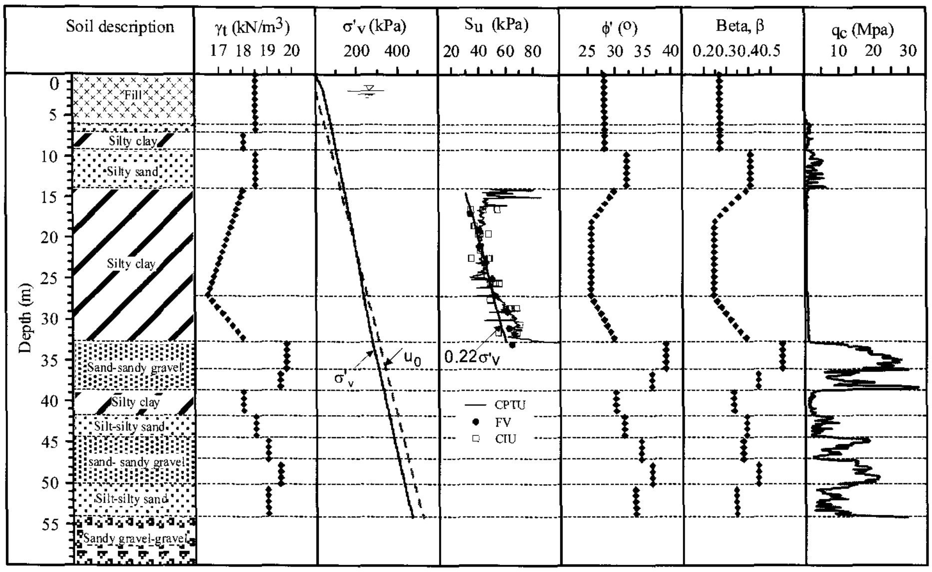


Fig. 3. Geotechnical profile at MC2-2

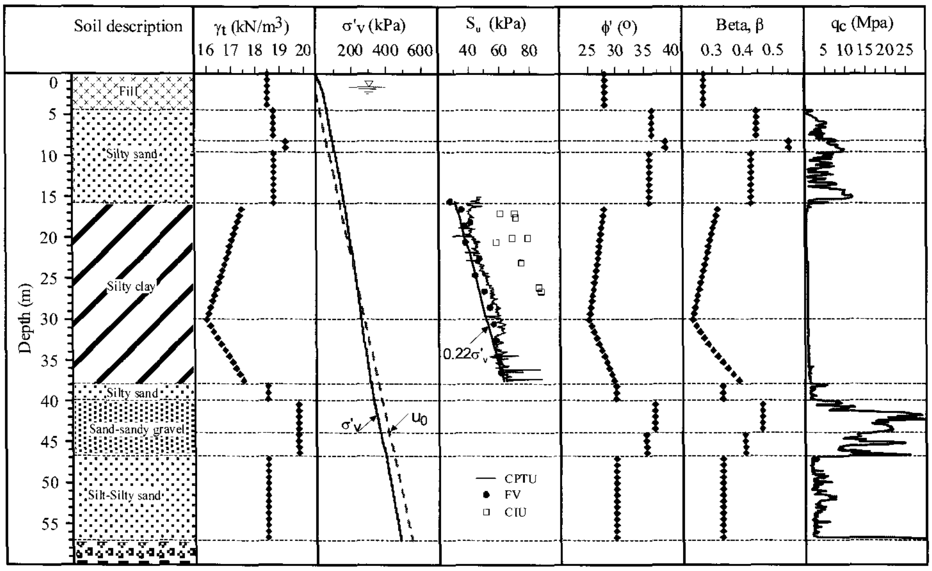


Fig. 4. Geotechnical profile at SO5-3

as indicated by Chung et al. (2006). Due to the recent fill, the maximum excess pore pressure of about 20 kPa existed at the middle of the upper clay in both sites.

3. Evaluation Methods for Pile Bearing Capacity

A number of pile design methods have been developed to calculate bearing capacity by using CPT or CPTU data (named as CPT-based design methods), which are summarized in Table 1. In the table, r_s and r_t are unit shaft and toe resistances and other parameters such as f_s, q_c, q_t, and q_E are shaft, cone tip, corrected cone tip and effective cone tip resistances respectively. The details of each

Table 1. CPT-based design methods

Method	Unite shaft resistance	Unit toe resistance
Schmertmann (1975)	$r_s = Kf_s$ Clay: (0.25≤K≤1.25) Sand: (0.25≤K≤1.25)	$r_t = C_{OCR} q_{ca}$ Influence zone: 8D above, 4D below pile toe
European (1979) (DeRuiter and Beringen, 1979)	Clay: $r_s = \alpha S_u = 0.05\alpha q_c$ α = 1 (NC clay), 0.5 (OC clay) Sand : $r_s = \min(f_s, q_c/300)$	Sand $r_t = C_{OCR} q_{ca}$ (Smert. method) Clay: $r_t = N_c S_u$ r_t = 15 MPa)
French (1982) (Bustamente and Gianeselli, 1982)	$r_s = Kq_c \le J$ K, J are given in table from original paper	$r_t = Cq_{caa}$ Infl. Zone (1.5D, 1.5D) Clay: 0.45 ≤ C ≤ 0.55 Sand: 0.40 ≤ C ≤ 0.50
Mayerhof (1976,1983)	$r_s = K_f f_s$ (K = 1)* $r_s = C_c q_c$ (Cc = 0.5)	$r_t = C_1 C_2 q_{ca}$ Influence zone: 4D above, 1D below pile toe. C_1,C_2 are function of diameter, embedded depth into bearing stratum
Tumay & Fakhroo (1981)	$r_s = Kf_s$ $K = 0.5 + 9.5e^{-90f_s}$ (fs in MPa)	$r_t = C_{OCR} q_{ca}$ (Schmert. method) Influence zone: 8D above, 4D below pile toe
Eslami-Fellenius (1997) (E-F method)	$r_s = C_s q_E$ ($q_E = q_t - U_2$) C_s = 0.004 – 0.08 depends on soil type	$r_t = C_t q_{Eg}$ q_{Eg} = geometric average of qE over infl. zone of 8D above, 4D below. C_t = 1/(3D) if D≥ 400mm
Prince and Wardle (1982)	$r_s = \alpha f_s$ (α = 0.53 driven pile)	$r_t = k_{b2} q_{ca}$ k_{b2} = 0.35(driven pile) q_{ca} arithmetic average over an infl. zone of 4D above and 4D below pile toe
Aoki and De Alencar (1975)	$r_s = \alpha_1 q_{ca}/F_{S2}$ (r_s ≤ 120 kPa) α_1 = 1.4–6% depends on soil type, F_{s2} = 3.5 (selected from given table)	$r_t = q_{ca}/F_b$ (r_t ≤ 15MPa) F_b = 1.75 (concrete pile, from given table). q_{ca} = arithmetic average over infl. zone of 4D above and 4D below pile toe
Philipponnat (1980)	$r_s = \alpha q_{ca}/F_S$ (α = 1.25 driven pile) F_s = 50–200 depends on soil type	$r_t = k_b q_{ca}$ (k_b = 0.35–0.5 depends on soil type). Infl. zone 3D and 3D
Jardine et al. (2005) (ICP method)	Sand: $r_s = \sigma'_{rf}$ tan δ_{cv} Clay: $r_s = \sigma'_{rf}$ tan δ_f Details are given in reference	$r_t = q_c\left[1 - 0.51\log\left(\frac{D}{D_{CPT}}\right)\right]$ D: pile diameter, D_{CPT} = cone diameter

* The method will be used for comparison with the PDA analysis.

Table 2. Analytical design methods

Resistance	Method	Unit resistances	Soil type	Reference
Shaft resistance (β methods)	Burland	$r_s = (K\tan\delta)\sigma'_v$	clay-sand	Burland (1973)
	Fellenius	$r_s = \beta\sigma'_v$ **	clay-sand	Fellenius (1991)
Toe resistance	Janbu	$r_t = cN_c^* + q'N_q^*$	clay-sand	Janbu (1976)
	Vesic	$r_t = cN_c^* + \sigma'_0 N_\sigma^*$	clay-sand	Vesic (1977)
	Kulhawy	$r_t = B\gamma N_\gamma^* + \sigma'_{zD} N_q^*$	clay-sand	Kulhawy et al. (1983)
	Fellenius	$r_t = N_t \sigma'_{z=D}$	clay-sand	Fellenius (1991)

**The method will be used for comparison with the PDA analysis and modification of the Eslami-Fellenius (1997) method (which will be mentioned as the β method in the late sections).

method can be referred to the listed references.

A number of other analytical methods have also been developed and widely applied to estimate pile bearing capacity. In this paper, all available methods could not be mentioned in detail, however, some common methods used for this study are briefly described in Table 2.

4. Calculation of Bearing Capacity Based on Evaluation Methods

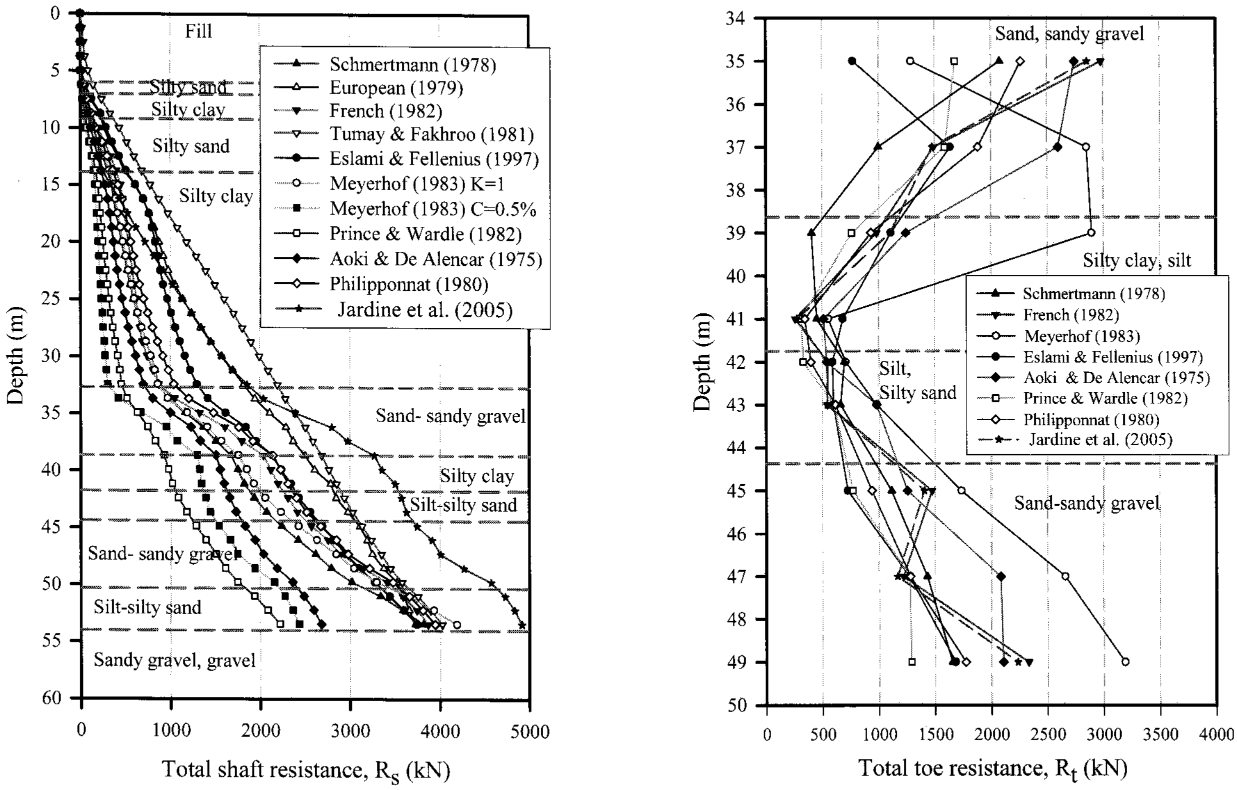
Bearing capacity was calculated under the following conditions: (a) Concrete cylinder pile with diameter of 600mm; (b) Soil classification was performed based on borehole logs data and CPTU data (by Eslami-Fellenius (1997) method); (c) The clays and sands are assumed as normally consolidated (NC) soils (Chung et al. 2002) and therefore parameters were taken for NC soils; and (d) An arithmetic average of cone tip resistance in the influence zone, q_{ca}, was used for all methods except the Eslami-Fellenius (1997) method which adopts geometric average of q_E.

The coefficient, $\beta = K \cdot \tan\delta$ that was proposed by Burland (1973) can be expressed as Eq. (1):

$$\beta = \left(\frac{K}{K_0}\right) K_0 \tan\delta = \left(\frac{K}{K_0}\right)\left(1-\sin\phi'\right)\tan\left(0.8\phi'\right) \tag{1}$$

The average ratio, K/K_o, was taken averagely as 1.60 for driven piles (large displacement piles) and the friction angle, δ, between pile and soil was taken as $0.8\phi'$, which is the average value of $\delta = (2/3 \sim 1)\ \phi'$. The values of β and N_t (toe bearing capacity coefficient) were linearly interpolated from an approximate range of β and N_t (Fellenius, 1991).

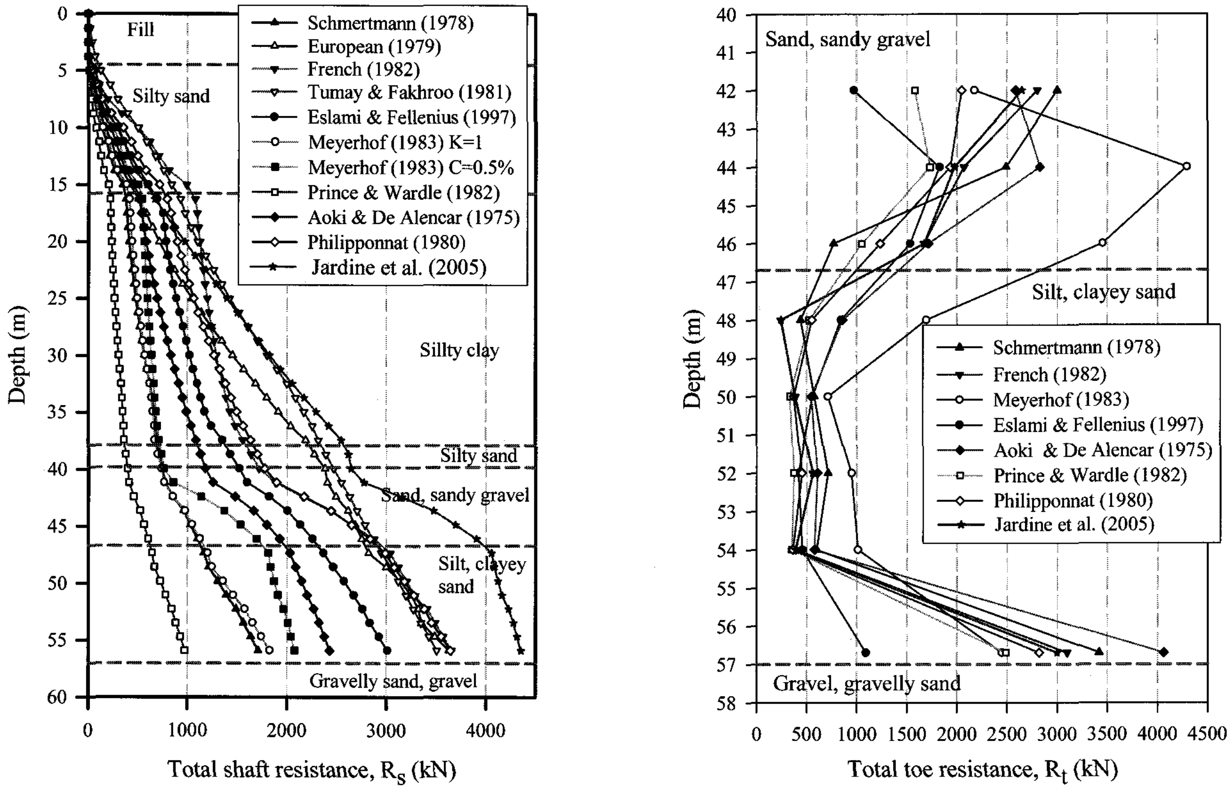
Figs. 5 and 6 show calculated results from the CPT-based methods for MC2-2 and SO5-3 locations. It is noted that the results significantly vary due to differences in the methods, rather than differences in the soil properties at the sites. In Figs. 5 (a) and 6 (a), the Jardine et al. (2005) and Prince and Wardle (1982) methods usually give largest and smallest shaft resistance values, respectively. The different magnitude increases significantly when the



(a) shaft resistance

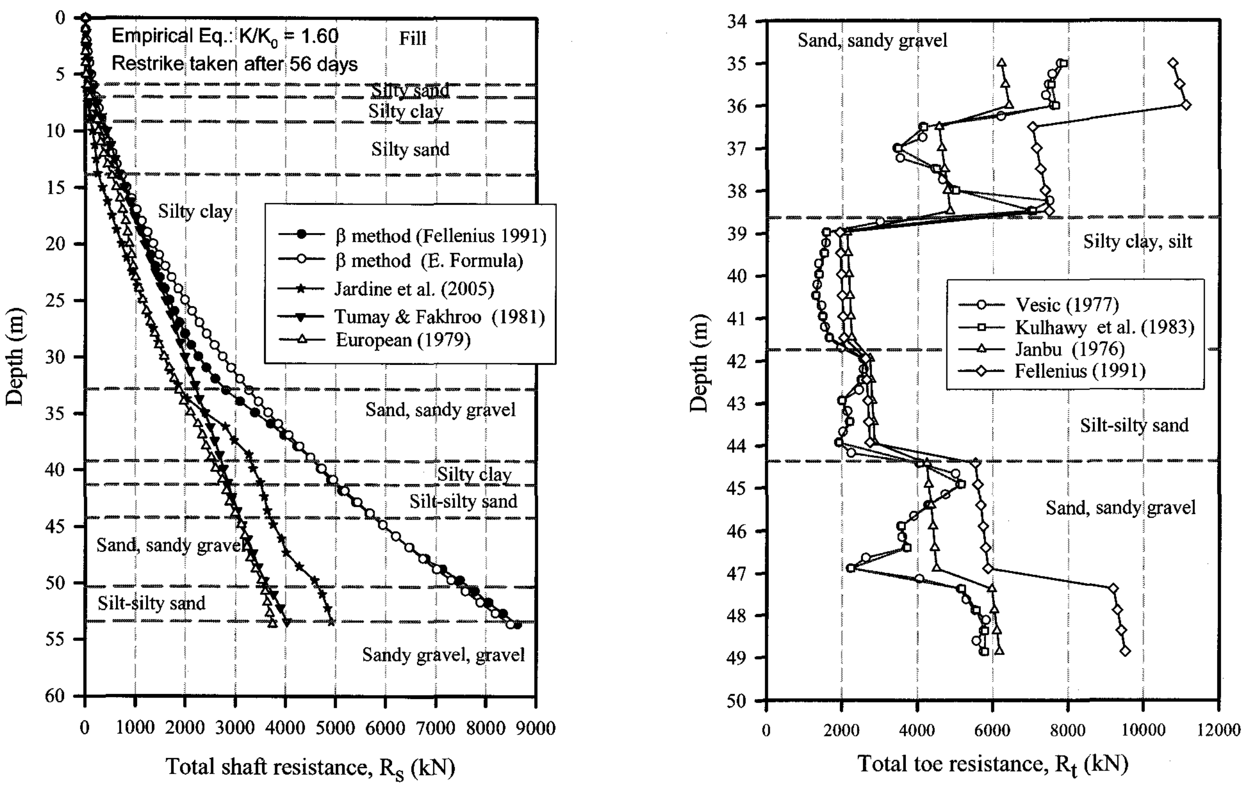
(b) toe resistance

Fig. 5. CPT-based total shaft and toe resistances at the MC2-2



(a) shaft resistance (b) toe resistance

Fig. 6. CPT-based total shaft and toe resistances at the SO5-3



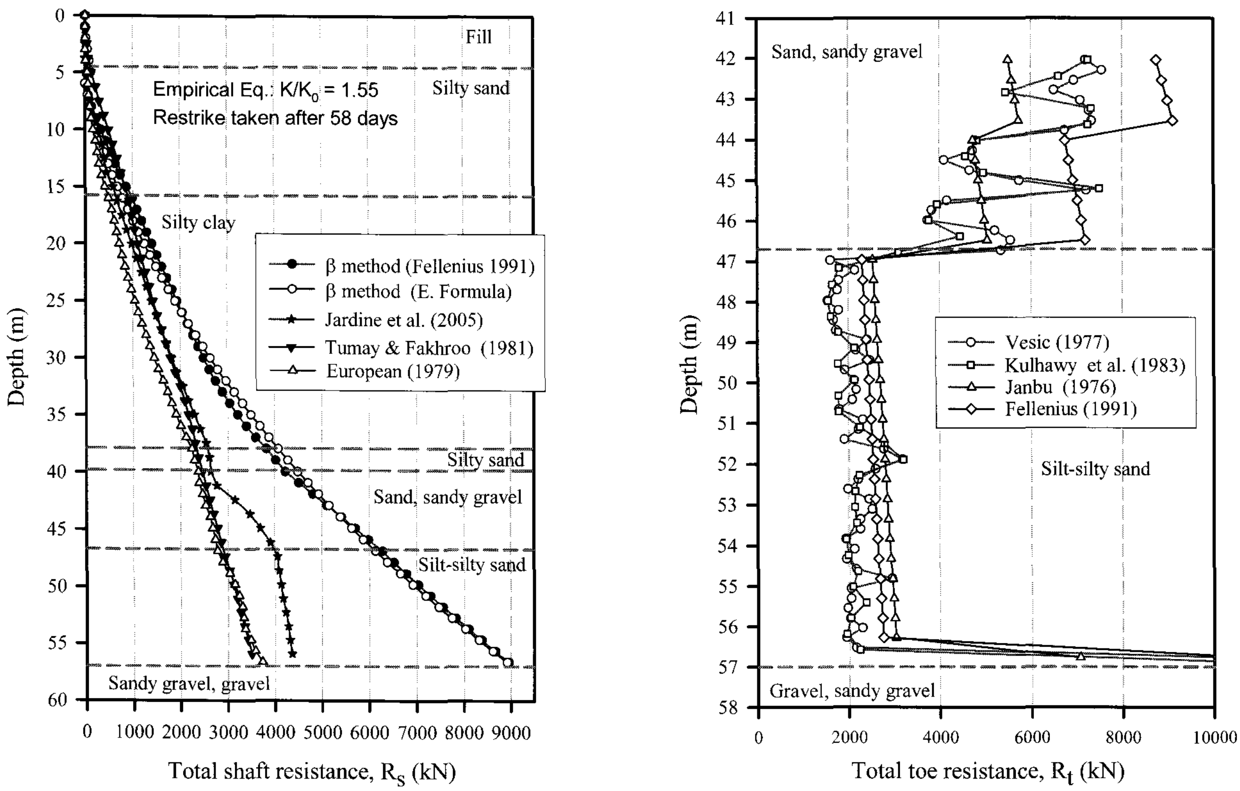
(a) shaft resistance (b) toe resistance

Fig. 7. Shaft and toe resistances from the analytical methods at the MC2-2

pile goes deeper into the sand layers and the maximum difference is up to 2500 kN at the study locations.

In Figs. 5 (b) and 6 (b), the calculated toe resistances have a similar trend to that from the cone tip resistance except the Meyerhof (1983) method. The toe resistances vary significantly when pile toe is located in the sand layers and the values tend to be much closer when pile toe is located in the silty clay or silt layers. As the soil is coarser, the difference in the toe resistance generally becomes larger. Figs. 5 (b) and 6 (b) show that there is no distinctive difference but a constant trend of the toe resistance between the methods; however, the Eslami-Fellenius (1997) method usually gives smallest values while the Meyerhof (1983) method the largest values for both sites. The maximum difference in the toe resistances is approximately 2000 kN in dense sand layers.

Figs. 7 and 8 show the shaft and toe resistances calculated by the analytical methods for both sites. The shaft resistances from the analytical methods were compared with those of the European (DeRuiter and Beringen, 1979), Tumay and Fakhroo (1981) and Jardine et al. (2005) that gave largest values in the CPT-based methods. It is shown from Figs. 7 (a) and 8 (a) that the shaft resistance obtained from the β methods is significantly larger than the largest resistances obtained from the CPT-based methods. Considering that the β values were statistically derived from ultimate bearing capacity of static loading tests for friction piles (Fellenius, 2006a), the other methods lead to the underestimation. In addition, the shaft resistance from the β methods significantly increases with depth, compared to the others, when the pile goes deeper into the sand layers. It is because the β values are directly interpolated from the effective friction angle which is usually high in sand layers. Figs. 7 (b) and 8 (b) show that the toe resistances obtained from the methods of Janbu (1976), Vesic (1977) and Kulhawy et al (1983) are quite similar to one another, while the Fellenius (1991) method gives the largest values. In general, toe resistances obtained from these methods are usually 2 to 3 times larger than those obtained from the CPT-based methods.



(a) shaft resistance

(b) toe resistance

Fig. 8. Shaft and toe resistances from the analytical methods at the SO5-3

5. Pile Driving Analyzer (PDA) Test

5.1 Methodology

Five PHC piles (600 mm outer diameter, B-type) were driven to evaluate bearing capacity and drivability of long PHC piles at the sites: two piles that called MJ-2 at the borehole MC2-2 and three piles that called SH-2, SH-3, and SH-5 at the boreholes SO2-1, SO3-2, and SO5-3, respectively. The PDA tests were performed through the driving process, starting from the first stroke until the last meter depth. The piles were closed-end, driven by a hydraulic impact hammer having a weight of 16 ton. All the piles were driven successfully up to designed depths.

5.2 Drivability Analysis

A summary of pile drivability from the sites is given in Table 3. Figs. 9 and 10 show typically the results obtained from PDA tests on the two piles. It is very interesting to note that the tendency of the stress and capacity curves (CSX, RMX) matched well with the cone tip resistance curves shown in Figs. 2 and 3. The allowable compressive and tensile stresses of the piles are 0.48 t/m^2 and 0.102 t/m^2, respectively. As shown in Table 3, the maximum stresses (CSX, CSB and TSX) of the piles induced from driving process are all less than the allowable ones.

The quake value, which is the movement between the pile and the soil required to mobilize fully plastic resistance, is important to analyze the drivability of the pile. It was observed that the maximum quake values (Q) at the MJ and SH sites were 3.5 mm (0.58% of the pile diameter) and 6.9 mm (1.15% of the pile diameter), respectively. The quake value at the pile toe is known to be related to pile diameter and it is usually within the range of 1% of the pile diameter (Fellenius, 2006a). It could therefore be stated that the piles were driven within the range of reasonable values.

Table 3. Summary of drivability

Location	Monitored Depth (m)	Ram height (m)	F.P (mm)	Q (mm)	CSX (t/cm^2)	CSB (t/cm^2)	TSX (t/cm^2)	EMX (t-m)	ETR (%)	RMX (ton)	BTA
MJ-2*	14.0-35.0	0.2-0.8	3	2.54	0.29	0.23	0.031	7.89	62	320	88
MJ-2	14.0-49.4	0.2-0.8	4	3.50	0.31	0.27	0.065	10.4	81	370	86
SH-5	14.0-56.7	0.2-0.8	3	6.90	0.33	0.26	0.071	10.0	78	430	81
SH-3	14.0-63.7	0.2-0.8	5	2.60	0.34	0.25	0.061	10.1	79	300	79
SH-2	13.7-57.1	0.2-0.8	2	2.54	0.26	0.24	0.096	10.3	80	406	91

* The instrumented pile for static loading test.
Where F.P = Final penetration (mm/impact); Q = Quake value at final depth; CSX and CSB = maximum compressive stress at pile head and pile toe, respectively; TSX = maximum tensile stress along the pile; EMX = maximum driving energy measured at pile head; ETR = Energy translation ratio; RMX = Total resistance by the Case method; BTA = Integrity of pile material.

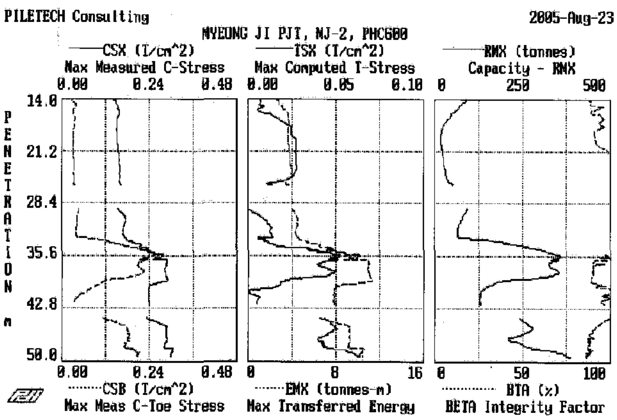


Fig. 9. PDA diagrams at the MC2-2

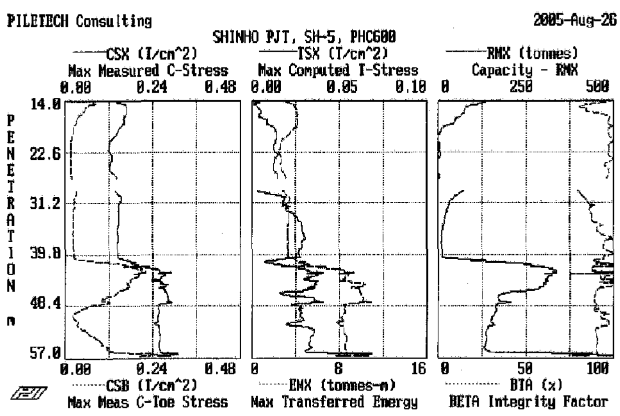


Fig. 10. PDA diagrams at the SO5-3

It is noted from Figs. 9 and 10 that the tensile stress induced along the piles in clay layers increased with depths. High tensile stress was developed in clay layers even under the ram height of about 0.2-0.3 m, while normal ram height of about 0.5-0.8 m was applied in the sand layers. The maximum energy translation ratios (ETR) from the pile are 80% in average, except the instrumented pile which was driven very carefully to avoid any damage of strain gages. An average BTA value of 85% was recorded, indicating that the piles were driven in high integrity. Conclusively, the parameters have proved that without damages, the PHC piles are able to be driven up to desirable depths in the area.

5.3 Bearing Capacity Estimated from PDA Test

In order to verify the reliability of the CPT-based and the analytical methods, bearing capacity at 17 depths in the sand layers was obtained from CAPWAP analysis. Fig. 11 shows results from CAPWAP analysis, in which the EOID and restrike shaft resistances for all the depths are shown in Fig. 11 (a). It is noted that the CAPWAP shaft resistances shown in Fig. 11 (a) have a very similar trend with that from the CPT-based methods shown in Figs. 5 (a) and 6 (a). The shaft resistance increases significantly as the piles penetrate deeply into the lower dense sand layers (Fig. 11 (a)), however, the toe resistance sensitively changed depending on the density of sand (Fig. 11 (c)). Consequently, it is shown that the total resistances roughly increase with depth (Fig. 11 (d)), which means that the shaft resistance considerably governs the total resistance.

6. Comparison between the Experimental and Calculation Methods

6.1 Comparison between CAPWAP and Calculated Resistances

It is known that bearing capacity obtained from CAPWAP analysis is reliable before a static loading test is performed. Therefore, in order to estimate the applicability of the calculation methods in the sites, CAPWAP resistances obtained from the PDA tests were compared with the calculated resistances. The ratios of calculated resistances to the CAPWAP resistances are shown in Fig. 12, where the ratios of $R_s/R_{s,PDA}$, $R_t/R_{t,PDA}$, and $R_u/R_{u,PDA}$ indicate the shaft (Fig. 12 (a)), the toe (Fig. 12 (b)), and the total resistances (Fig. 12 (c)), respectively.

As shown in Fig. 12 (a), the shaft resistance ratios from

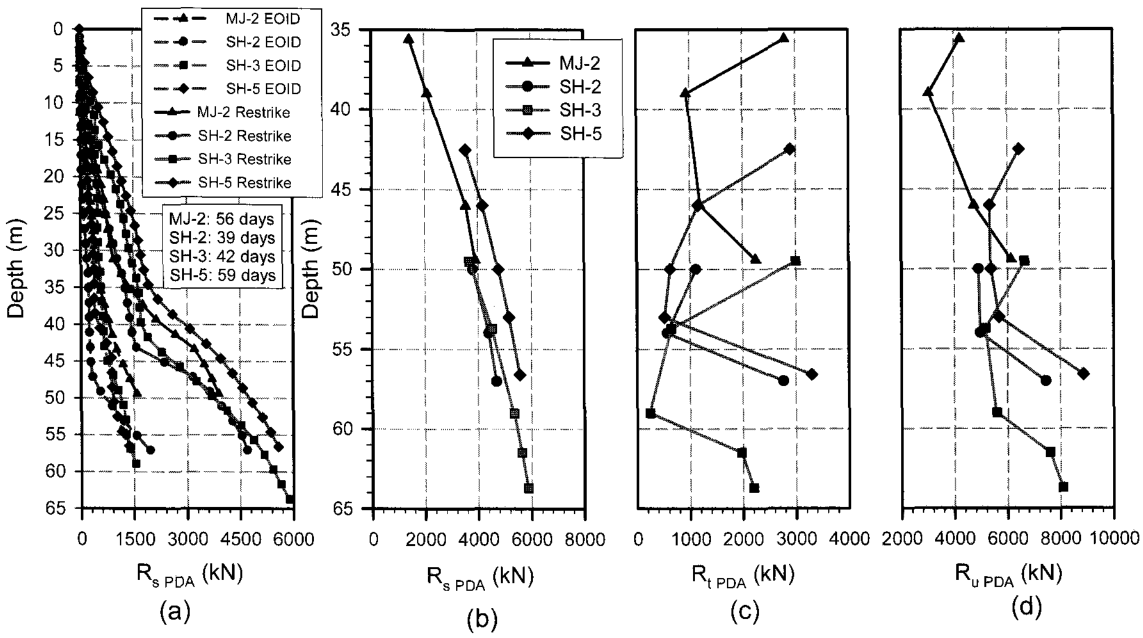


Fig. 11. CAPWAP analysis results of the piles

the CPT-based methods are almost less than unity, whereas only the β method gives the ratio of 1.5 to 2. It implies that though the shaft resistance of the pile was not fully mobilized for the entire depths (Kim et al, 2006), most of the CPT-based methods underestimate the shaft resistance compared with the CAPWAP results. Among them, the Prince & Wardle (1982) and β methods usually give the smallest and largest ratios, respectively.

Fig. 12 (b) shows that the toe resistance ratios are so scattered, ranging from 0.5 to 2 for the CPT-based methods and from 2.5 to 4.5 for the other analytical methods. The results of Eslami-Fellenius (1997) and Fellenius (1991) usually give the smallest and largest ratios, respectively.

Fig. 12 (c) shows that the total resistance ratios from the CPT-based methods are almost in the range of 0.5 to 1. It could be stated from Fig. 12 (c) that the CPT-based methods usually underestimate pile bearing capacity at the study locations. The bearing capacity would be more underestimated if the shaft resistance were fully mobilized due to soil set-up effect. In general, the methods of Prince & Wardle (1982) and Jardine et al. (2005) give the smallest and largest total resistance ratios, respectively. However, it should be noted that the CAPWAP results significantly depend not only on restrike time after the end of driving but also on the soil profile conditions.

6.2 Comparison of the CAPWAP, Static Loading Test, and Calculated Results

It is worthwhile that the bearing capacity indicated previously is compared with the result of the static loading test (SLT) which was performed at MJ-2 location. Fig. 13 presents a comparison of shaft resistances between the previous results and the SLT result. It is recognized from Fig. 13 that the shaft resistance from the SLT considerably agrees with that of the β methods, rather than the results of CAPWAP and Jardine et al. (2005). The shaft resistance from the β methods is a little smaller up to 13 m and then larger than that of the SLT at the lower depths. If the shaft resistance at the lower part were fully mobilized (in fact, the lower part of the pile was not fully mobilized (Kim et al, 2006)), the agreement between them would become better. Though the method of Jardine et al (2005) showed the best agreement with the CAPWAP (restrike data), it still underestimates shaft resistance compared with the SLT data.

Fig. 14 presents a simulated toe resistance-movement (t-z) curve for the instrumented pile at the MJ-2, associated with toe resistances from the CPT-based and the analytical

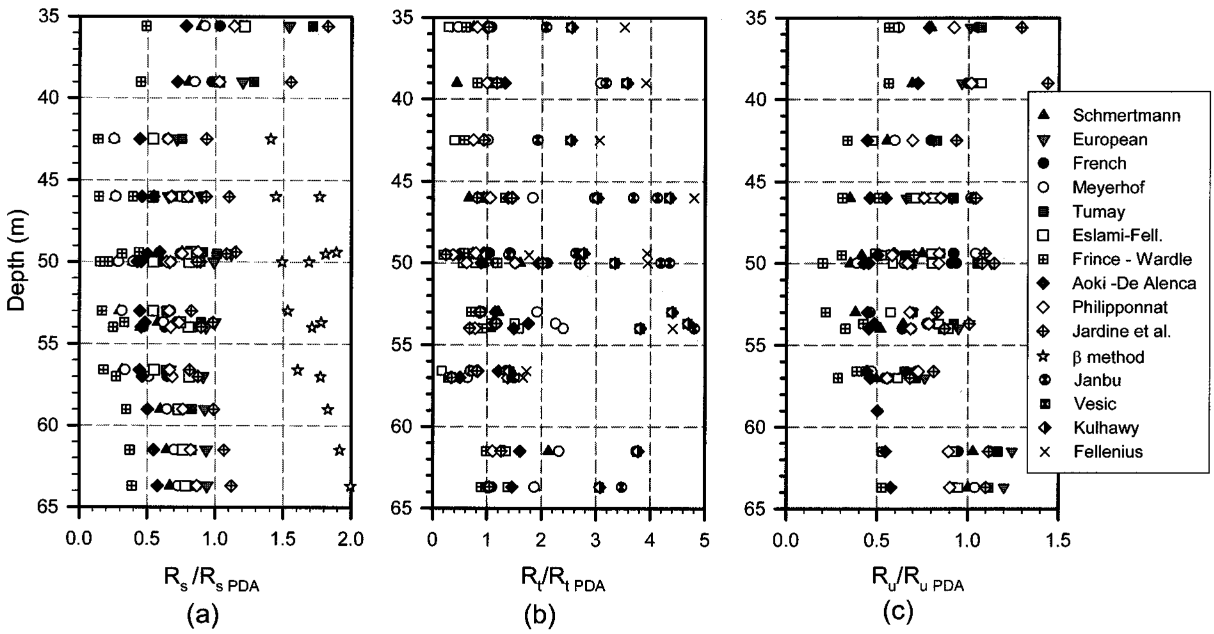


Fig. 12. Resistance ratios of the piles

methods given in Tables 1 and 2 (Kim et al. 2006). It is featured that there is no particular yield (failure) point on the curve that could be considered as ultimate toe resistance, but the toe resistance increases with increasing of toe movement. Therefore, the ultimate toe resistance from the SLT could be obtained from the curve at a toe movement value of 10 mm, as Fellenius' recommendation (2006a) for the design purposes. Because the PDA final stroke from the EOID was about 2-3 mm, it could be explained that the toe resistance obtained from the

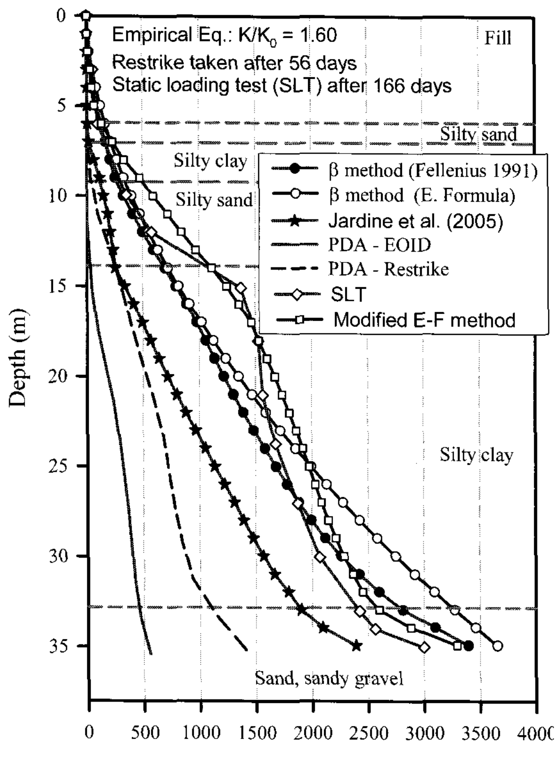


Fig. 13. Comparison of shaft resistances for the pile MJ-2

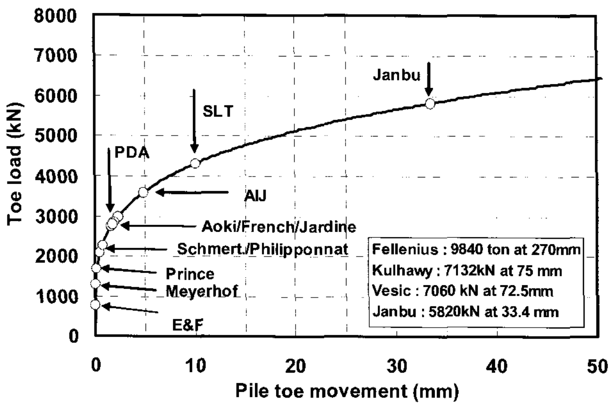


Fig. 14. Comparison of toe resistances between the SLT and the calculated methods (Data from Kim et al, 2006)

CAPWAP analysis is smaller than that from the SLT (R_{tPDA} = 2800 kN < R_{tSLT} = 4370 kN). It is also shown that if the pile toe movement were taken at about 2-3 mm, then toe resistance would be about 3000 kPa and it would be quite similar to that from the CAPWAP result. A group of three methods, i.e., Aoki and De Alenca (1981), French (1982) and Jardine at al. (2005) give similar toe resistances to the CAPWAP result.

Fig. 15 presents a comparison of total resistance among the CAPWAP, SLT, and Jardine et al (2005) methods. In addition, the toe resistance from the method of Aoki and De Alenca (1975) that agreed well with the CAPWAP result is also plotted. The predicted total resistances from CAPWAP analysis and the calculation methods are less than the SLT result. It can be recognized that the discrepancy is attributed to the difference in toe resistances. If the toe resistance is appropriately predicted and the β method (Fellenius, 1991) is adopted, then the total resistance could be determined without any difficulty in this case. Considering the safety for the design purpose, the toe resistance from the methods of Aoki and De

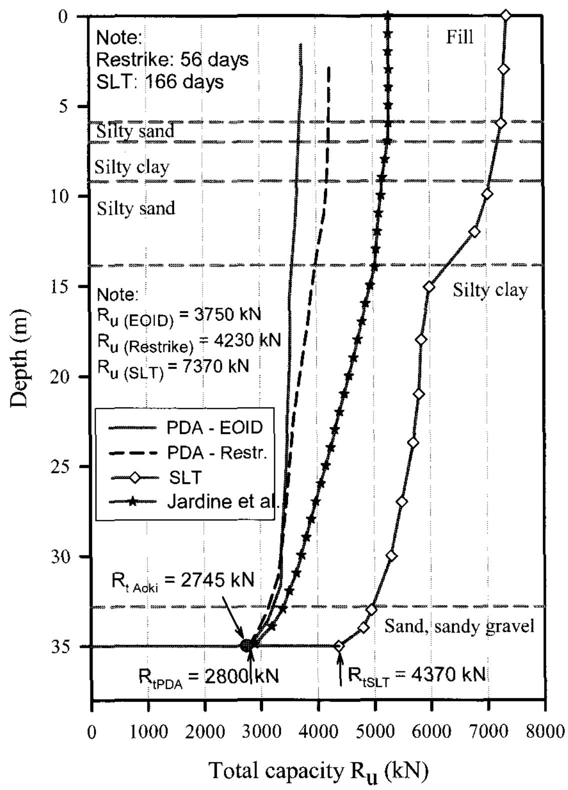


Fig. 15. Comparison of total bearing capacity for the pile MJ-2 (Data from Kim et al, 2006)

Alenca (1975), French (1982), Jardine at al. (2005) and AIJ (2001) and the shaft resistance from the β method would be reasonable.

7. Modification of Eslami-Fellenius Method (1997) for Shaft Resistance

It was previously shown that though the CPT has an advantage to obtain a continuous soil profile, the CPT-based methods underestimated shaft resistance at the study sites. It would be useful to modify a CPT-based method for estimating shaft resistance. Among the number of CPT-based methods, only Eslami-Fellenius (1997) developed the unit shaft resistance based on CPTU as follows:

$$r_s = C_s \cdot q_E \tag{2}$$

where C_s is shaft correlation coefficient, which depends on soil type obtained by their soil classification chart; $q_E = q_t - u_2$. However, the method also underestimated pile bearing capacity at the study sites. In order to correlate with the β values (Fellenius, 1991) that agreed well with shaft resistance from the SLT as above, a modified shaft coefficient C'$_s$ was proposed for the study sites by the following equation:

$$C_s' = \beta \cdot \frac{\sigma_v'}{q_E} \tag{3}$$

Table 4 shows the coefficients C_s and C'$_s$ for each soil type identified by the Eslami-Fellenius (1997)'s classification. Fig. 16 shows a comparison of shaft resistances for the methods of β (Fellenius, 1991), Eslami-Fellenius (1997) and modified Eslami-Fellenius (1997) for 7 locations in the study sites. Though the shaft resistances obtained from the Eslami-Fellenius (1997) method were usually equal to 50% of that obtained from the β method or the SLT, the modified shaft resistances match quite well with that obtained from the β method. It would be proper to use the modified shaft coefficients for practical design at the study sites when CPTU data are available.

8. Discussions

The importance of soil set-up effect was previously

Table 4. Shaft coefficients from the methods

β method	β	Eslami-Fellenius (1997)	C_s	C'$_s$
Clay	0.23-0.40	Soft sensitive soils	0.080	0.120
		Clay	0.050	0.080
Silt	0.27-0.50	Silty clay, stiff clay and silt	0.025	0.040
		Sandy silt and silt	0.015	0.035
sand	0.30-0.80	Fine sand or silty sand	0.010	0.045
		Sand to sandy gravel	0.004	0.010

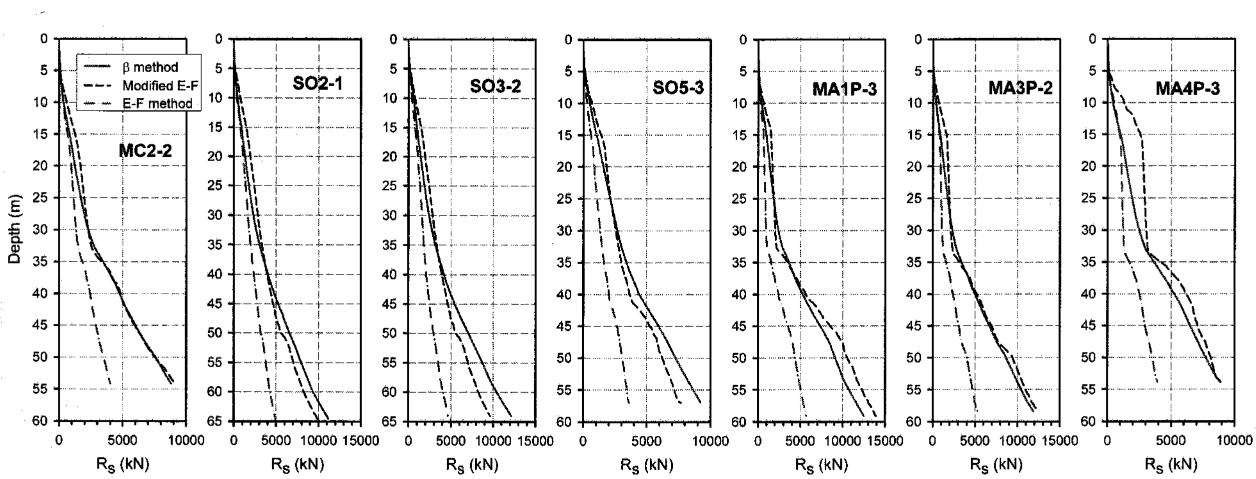


Fig. 16. Comparison of shaft resistances between the b, E-F and modified E-F methods

described in bearing capacity on piles. Therefore, it is meaningful to observe a few interesting testing results on piles. According to the study of Fellenius (2002) for a pile of 19 m long in sand deposit, the shaft resistance induced from long-term soil set-up could be continuously increased during 143 days after driving, which was shown in a series of CAPWAP analysis. The toe resistance of the pile from the soil set-up effect was also found increased simultaneously. It is also meaningful to mention about the recent study of Fellenius (2006b), which has the results on several full-scale, long-term tests performed since the 1960s through the 1990s, in several countries. The load transfer is governed by effective stress and very small movement results in mobilization of ultimate shaft resistance. And also, the pile toe resistance is determined by downdrag of the pile and the resulting pile toe penetration. Based on this study, it is difficult to say which method is appropriate for the toe resistance. In other words, the toe resistance is just the value depending on the toe movement, unlike the shaft resistance.

On the other hand, Murad and Titi (2004) evaluated the applicability of methods for bearing capacity using the static load test results of 35 PPC (precast prestressed concrete piles) driven piles. The piles had different sizes and lengths, and were failed during the SLT in the altered layers by sand and clay. According to the evaluated results, the European (1979) and French (1982) methods showed the best agreement with the SLT results. It is shown that the total shaft resistance from the SLT in the Louisiana area is much different from those evaluated previously in the Nakdong River delta. We can infer that this result did not consider the long-term effects and hence the shaft resistance must be underestimated.

Kim et al. (2006) also presented a persuasive evidence of pile bearing capacity influenced significantly from soil set-up effect in the Nakdong River delta. Especially, the significance was also emphasized where the soil profiles involve unusually thick and soft clay layers and the ongoing consolidation process was being taken place during the period until the restrike.

Herein, it would be worthwhile to remind Fellenius' valuable experiences (Fellenius, 2006a). There are many factors which cause various bearing capacity among the methods as well as from site to site, however, a number of key factors could be: (1) each method was usually developed based on a number of static loading tests in a local area or a number of places which can not be representative for all kinds of soils in the world; (2) the methods might have been developed from different procedures of static loading test (for example: slow or quick test) and interpreted from different failure criteria; (3) the residual load concept might not be considered properly among the methods. In addition, fully mobilized shaft resistance during soil set-up duration might not be considered or properly calculated so that many of the CPT-based methods underestimate shaft resistance compared with the SLT data; (4) the length of the influent zone above and below pile toe is not unified among the CPT-based methods. This factor could significantly make variant toe resistances among the methods in strongly layered soil profiles as the MJ and SH sites; (5)The CPT-based methods which were developed before the piezocone came in general use (all the CPT-based methods except the method of Eslami-Fellenius, 1997). They do not consider the more accurate measurement achievable with the piezocone.

Consequently, it can be said that the bearing capacity analyzed from PDA test is a reference value at any testing time, because it depends on soil set-up effect. Considering that the bearing capacity from a few CPT-based methods agreed well with those from the EOID values of PDA test, it would be inferred that most of the CPT-based methods are appropriate for the bearing capacity when piles are just driven. Because the drag load is important in a thick soft deposit, the empirical methods for shaft resistance should be better to be applied for the MJ and SH sites. The toe resistance increased with increasing the toe displacement (see Fig. 14), so that the methods equivalent to the displacement of 10 mm as usual should be proper to be chosen.

9. Conclusions

Several PHC piles were driven well up to 60 m depth at the MJ and SH sites in the Nakdong River deltaic area where pile toe is located at the top of middle dense sand and lower sand layers, followed by loose sand and soft clay layers. The PDA test was performed during the driving and restriking after a long duration. Using the PDA testing results, the drivability and bearing capacity were analyzed. A number of calculation methods for bearing capacity, which were developed based on CPT and experiments, were adopted. The computed results were compared with those from CAPWAP analysis and SLT data for an instrumented pile installed at the MJ site. The following conclusions and recommendations can be drawn from the study.

(1) The soil set-up effect was important on bearing capacity in the thick soft clay deposit. The PDA tests performed during a short duration after pile driving gave a good agreement with the shaft resistance calculated by the methods of Jardine et al (2005), French (1982) and Tumay and Fakhroo (1981). However, the shaft resistance from the β method agreed well with the SLT result that was performed long after pile driving and was much larger than the previous results.

(2) According to the SLT result, the t-z curve (Fig. 14) was governed by an exponential function. The toe resistance from the PDA tests (the restriked result) underestimated more than the SLT value at the toe movement of 10 mm. It was because the PDA results were taken from the toe movement of less than 5 mm. The methods of Aoki and De Alenca (1975), French (1982) and Jardine et al. (2005) gave better agreement with the PDA data. However, the analytical methods such as Fellenius (1991), Janbu (1976) Vesic (1977), Kulhawy et al (1983) overestimated ($2\sim3$) times the toe resistance from the PDA results. It would be very risky if we consider these methods for practical design.

(3) A modified method for shaft resistance was newly proposed based on the β method and Eslami-Fellenius method (1997), which has an advantage of easily calculating a continuous shaft resistance profile using the CPTU data. This method, as well as the β method, would be applicable to the drag loads that are critical in the thick deltaic deposit.

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