대심도 연약지반에 항타매입된 PHC 말뚝의 선단지지력을 위한 CPT와 SPT법의 적용성 분석

Applicability of PDA and SPT-based methods for Toe Bearing Capacity of PHC Piles Driven in the Thick Deltaic Deposits

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개요 : 본 연구는 낙동강 하구 대심도 연약지반에 항타관입된 PHC 말뚝에 대하여 SPT 지지력 공식으로부터 계산된 선단지지력 값과 PDA 시험에서 얻어진 선단지지력 측정값을 비교하였다. 또한, SPT N값이 50이 넘는 경우에 대하여는 N값과 롯드 관입깊이의 선형관계를 가정하여 30cm 관입깊이에 해당하는 N값을 적용한 경우와 CPT q_c와의 상관성을 이용하여 q_c값으로부터 N값을 산정한 경우의 2가지 분석을 수행하였다. 그 결과, 본 연구에서 적용한 SPT 지지력 공식 모두 측정된 선단지지력 값과 차이가 났으며, SPT 지지력 공식은 대심도 연약지반에 항타 근입된 말뚝에 대하여 실제적인 설계를 수행할 때 신뢰하기 어려운 것으로 나타났다. 또한, N>50인 경우에 대하여 N값과 롯드 관입깊이의 선형관계를 적용하는 것은 지지력을 매우 과대평가하는 것으로 나타났다.

1. INTRODUCTION

The Standard Penetration Test(SPT) is one of the most common and economical field tests in soil investigation projects as it is simple and time-saving test. Besides main applications in soil characterization, attempts in estimating bearing capacity of piles by using the SPT N-vlaue have also been extensively studied for last few decades (e.g., Meyerhof, 1976; Decourt, 1995; Robert, 1997). In addition, using the test for estimating bearing capacity of piles has also been suggested in practical design codes and guidelines (e.g., AASHTO, 2007 CFEM, 2006) despite the SPT N-values show large variation. The Meyerhof (1976) method is commonly suggested for practical design in Korea (KSG, 2003).

Although several SPT-based methods were proposed for estimating bearing capacity of driven piles the reliability of the methods was uncertain for practical design, especially in deep sandy deposits. This paper is attempted to examine the reliability of some common SPT-based methods with an emphasis on the use of the SPT routinely performed in practice. First, three common CPT-based methods, which are often suggested in pile design manuals or specifications, were briefly reviewed. The analysis was then followed by a comparison analysis between toe bearing resistance obtained from the SPT-based methods and those obtained from the PDA test.

2. SPT-BASED TOE RESISTANCE

2.1 Correction of SPT N value

The common expressions of the normalized *(N_{60})* and the normalized corrected *$(N_1)_{60}$* of SPT N-value are highlighted in Table 1. It is noted from the table that the correction factor for overburden pressure (C_N) was adopted from the Liao and Whitman (1986) proposal. The correction factors C_E, C_B, C_S, and C_R in this study are taken as 0.9, 1, 1, and 1, respectively based on the hammer type and actual investigated depths.

Table 1. The normalized and normalized corrected SPT numbers

Expression	Parameter	Equation
$N_{60} = NC_EC_BC_SC_R$	N_{60} = normalized value; C_E, C_B, C_S, C_R = correction factors for energy, borehole diameter, sampling method, rod length, respectively (Skempton, 1986).	Eq. (1)
$(N_1)_{60} = C_N N_{60}$	$(N_1)_{60}$ = normalized corrected SPT number	Eq. (2)
$C_N = \left(\dfrac{p_a}{\sigma_v'}\right)^{0.5}$	C_N = correction factor for overburden pressure (Liao and Whitman, 1986), p_a = 100 kPa, σ_v' = effective stress (kPa)	Eq. (3)

2.2 Methods for SPT-based toe resistance

Table 2 briefly shows three considered methods that were proposed for driven piles in sandy deposits. It is noted from the table that only the unit toe resistance is taken into account in this study, since the SPT was not performed in the thick clay layer at the sites to fulfill the shaft resistance.

Table 2. Unit toe resistance from the SPT-based methods

Method	Unit toe resistance	Parameter	Equation
Meyerhof (1976)*	$r_t(KPa) = \dfrac{40(N_1)_{60} D_b}{D} \le r_l$ $r_l = 400(N_1)_{60}$	$(N_1)_{60}$ = normalized corrected SPT number, D_b = embedded depth into bearing stratum, D = Pile diameter, r_l = limit resistance.	Eq. (4)
Decourt (1995)*	$r_t(KPa) = K_b(N_{60})$	K_b=325 for sand, = 205 for sandy silt, =165 for clayey silt, = 100 for clay.	Eq. (5)
Robert (1997)	$r_t(KPa) = 190(N_1)_{60}$		Eq. (6)

* the method suggested in CFEM (2006)

3. CPT AND SPT TESTS AT THE STUDY SITES

3.1 General ground conditions

The study sites in this research were the Myeongji (MJ) and Shinho (SH) Residential

Complexes in the Nakdong River delta (NRD), where the ground has been reclaimed since the 1990s. The layering variation at the sites can briefly be described as follows: The fill layer of about 5 m thick was placed on the original ground surface and followed by loose silty sand (upper sand), soft to medium silty clay (upper clay), loose to dense sand (lower sand),and sandy gravel on bed rock. Thin clayey silt is mostly sandwiched in the lower sand layer. The thickness of the silty clay layer varies from 16 to 20m at the study site. Site characterization from CPTu data shows that the lower sand layer is slightly overconsolidated deposits (1< OCR <2) and the density of the sands varies significantly from loose to dense state ($20 < D_r < 80$). The cone tip resistance is up to 40 MPa in dense sands.

3.2 CPT and SPT profiles

During the soil investigation stage of the projects, the SPT was performed in association with boring logs. The test was performed using conventional donut hammer and cathead system. In dense to very dense sands, the test was stopped when the SPT number reached 50 even the penetration was less than 30 cm. Many SPT locations have been performed at the study sites, however four typical locations that were experimentally associated with the CPT and PDA test are examined in this study. Fig. 1 shows the CPTu-SPT profiles at the study locations in which the MC2-2 was at the MJ site and the others (SO2-1, SO3-2, SO5-3) were at the SH site. It is noted from the figure that some SPT-N values were limited to 50 with the penetration < 30 cm.

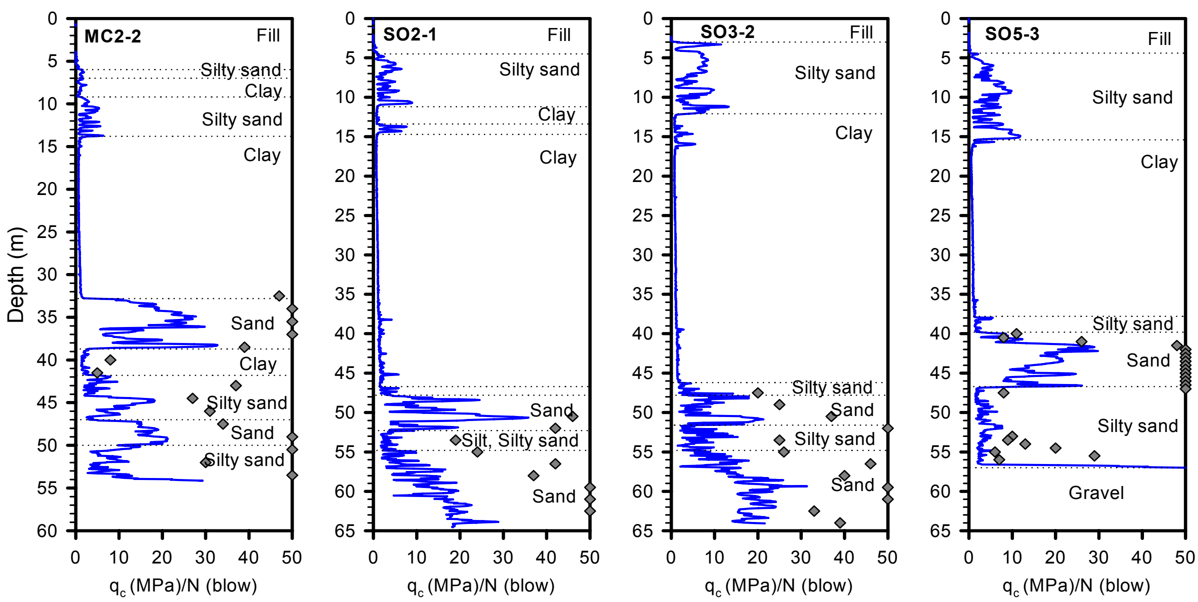


Fig. 1 CPT and SPT profiles at the study locations

4. PDA TEST AT THE STUDY SITES

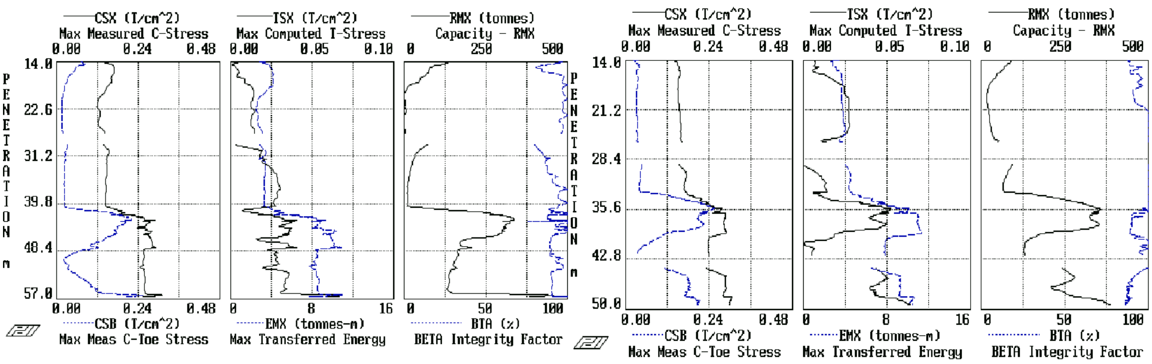
Five PHC closed-ended piles (600 mm outer diameter, B-type) were driven with the PDA test, which is to evaluate bearing capacity and drivability of the long PHC piles at the study sites. Two piles named as MJ-2 were driven at the same location of MC-2 and three piles named SH-2, SH-3, and SH-5 were driven at SO2-1, SO3-2, and SO5-3 locations, respectively. The PDA test was performed through the driving process, starting from the first stroke until the final depth. The piles were driven by using a hydraulic impact hammer of 16 ton capacity. All the piles were driven successfully up to designed depths. Table 3 briefly shows the drivability of the pile at the study sites. 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. Fig. 2 shows two typical PDA diagrams at the MJ and SH sites.

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.



a) MC2-2 location b) SO5-3 location

Fig. 2 Typical PDA-diagrams at the study locations

5. COMPARISON OF ANALYSIS RESULTS

5.1 Interpolation of the SPT N values > 50

As routine performance in dense to very dense sands, the test was stopped when the blow count (N) reached 50 even the penetration was less than 30cm. It is difficult to exactly determine the compressibility of the soil in this case in comparison with the case that the blow count is obtained with the full penetration of 30cm. Thus, two simple techniques were used in this study to interpolate the actual blow count, as follows: (1) correlation with cone tip resistance from the CPTu; (2) simple linear interpolation ($N = 30x50/S_{50}$) where S_{50}= penetration (cm) at N = 50.

For the first technique, a correlation between the normalized SPT number (N_{60}) and the cone tip resistance (q_c) was first carried out for the N ≤ 50 as shown in Fig. 3. It is shown from the figure that the ratio of q_c/N_{60} varies from 0.1 (lower bound) to 1 (upper bound), however it falls mainly into the range of 0.2 to 0.65 with the best fit ratio of 0.418. This best fit ratio is then used to approximately extrapolate the N_{60} values of the SPT numbers N> 50.

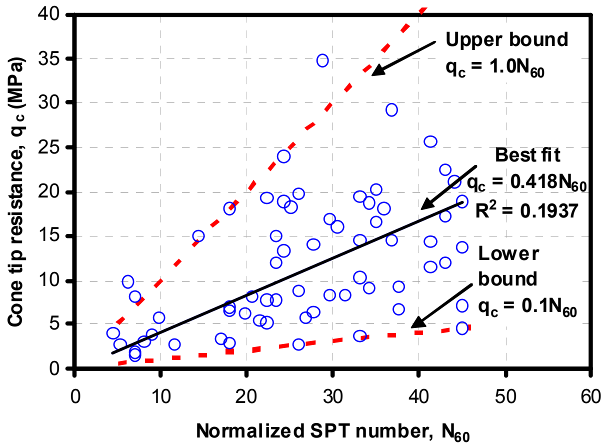


Fig. 3 Correlation between q_c and N_{60}

5.2 Comparative results

After getting full profiles of the N_{60} and $(N_1)_{60}$, the estimated toe resistances from the methods were obtained as shown in Figs. 4, 5, and 6 for Meyerhof (1976), Decourt (1995), and Robert (1997), respectively. In the figures, the legends SPT(1) and SPT(2) are denoted for the SPT-based methods associated with interpolating technique (1) and (2), respectively. It emerges from the figures that none of the methods give good agreement result with the PDA method.

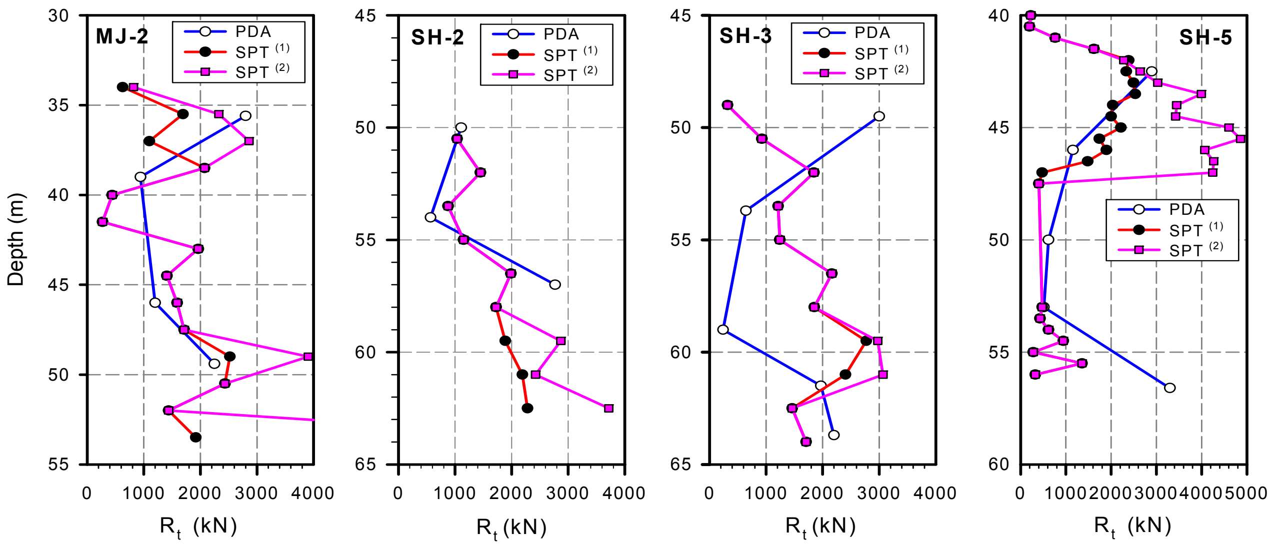


Fig. 4 Comparison of toe resistances from Meyerhof (1976) method

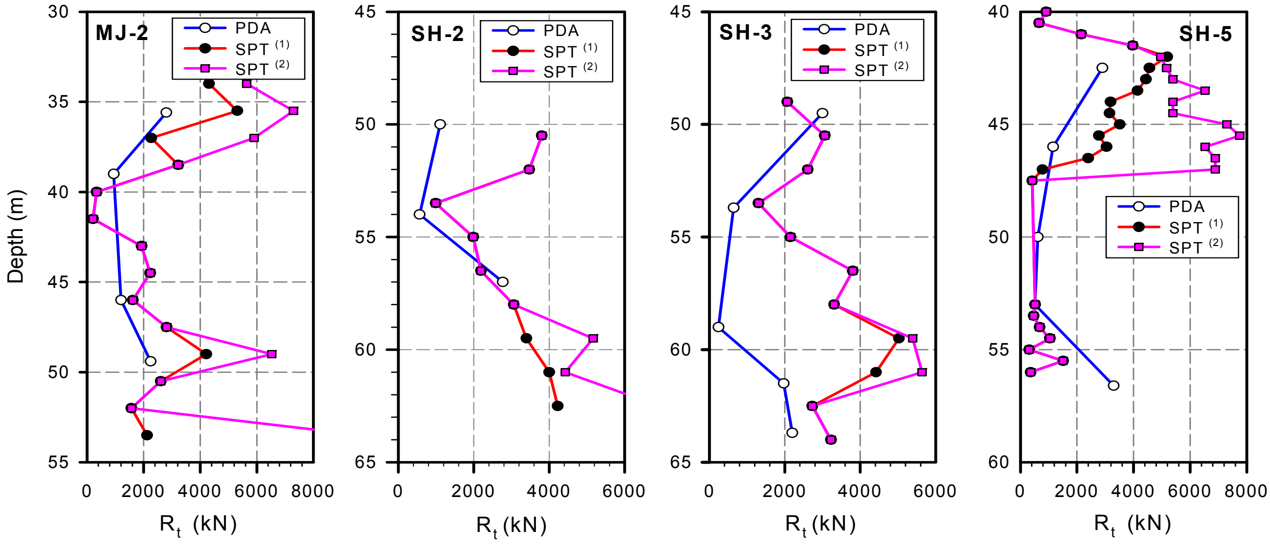


Fig. 5 Comparison of toe resistances from Decourt (1995) method

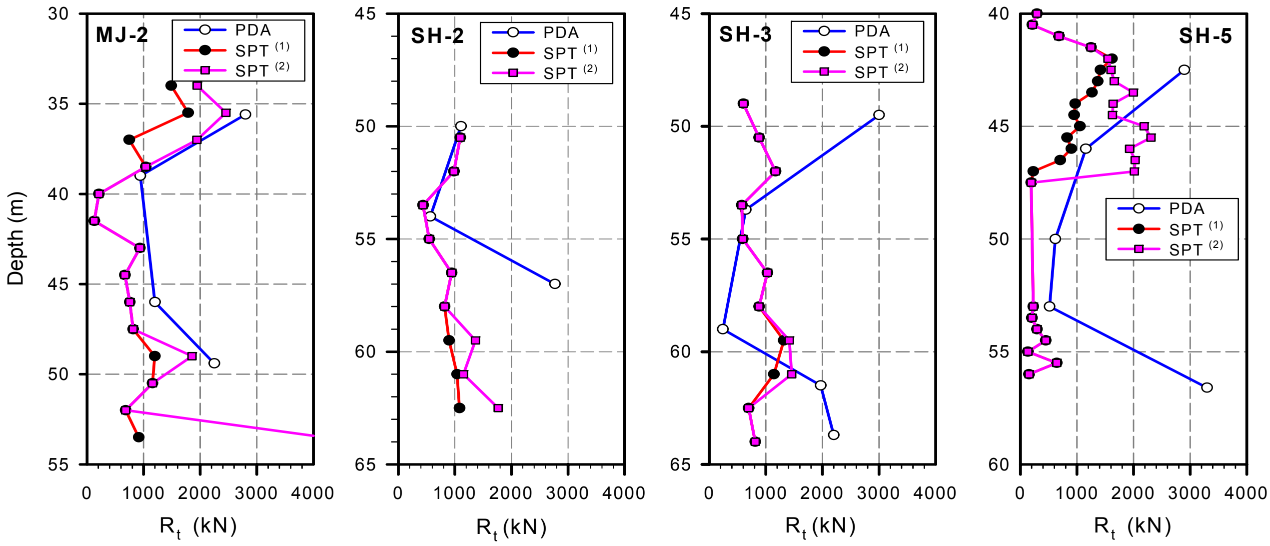


Fig. 6 Comparison of toe resistances from Robert (1997) method

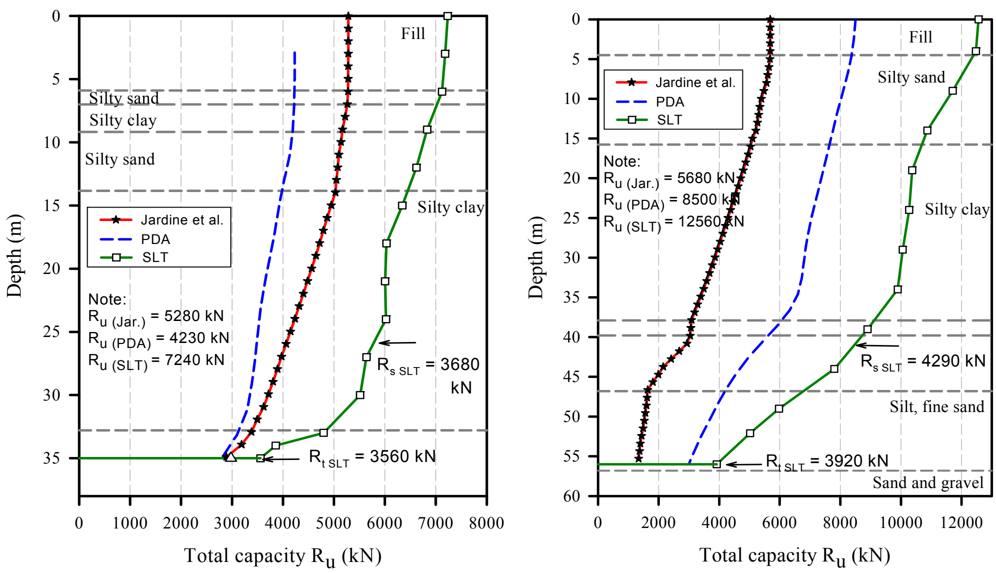
It appears from Fig. 4 that the Meyerhof (1976) method gives relatively closed toe resistances to that obtained from the PDA method (except pile SH-3) in the upper part of the profiles. However, the method tends to underestimate the toe resistances when embedded depth increases significantly. The underestimation in large embedded depths comes mainly from the "critical depth" concept used by the method. The shortcoming of the "critical depth" concept has been pointed out by several researchers (e.g., Fellenius & Altaee, 1995; Altaee et al. 1993; Randolph, 1993), as the main reason was that the residual load was ignored during the interpretation of the load distribution along the pile in static loading test.

As shown in Fig. 5, the Decourt (1995) method strongly overestimated the toe resistance in dense and very dense sands (upper and lower parts of the profiles), however the method tends to give matched results in loose silty sand and sandy silt deposits (the middle part of the profiles). Fig. 6 shows that the Robert (1997) method mostly underestimates the toe resistance at the sites.

It is noted that the simply linear interpolation method for N> 50 led to abruptly larger values of Rt. The linear interpolation technique commits the basic principle of soil mechanics that soils response significantly nonlinearly at high strain level, thus induced results from the technique would not be reliable. In comparison with the PDA profiles, the CPT-based interpolation technique show more reasonable trend than the linear based technique.

5.3 Reliability of the PDA results

The reliability of the PDA test has been extensively studied for several decades ago by comparing the bearing capacity obtained from the PDA and that obtained from the static loading test. Among plentiful publications on the topic, extensive studies carried out by Goble et al. (1980), Likins et al. (1996), and Likins and Rausche (2004) clearly show the reliability of the PDA method. It is reported from their studies that, by using sufficient hammer capacity and taking enough time for soil setup effects, the PDA test can provide very accurate bearing capacity of piles.



a) Pile MJ-2 b) Pile SH-5

Fig. 7 Bearing capacity of driven PHC pile from different methods

Dung (2008) performed a comparative study on bearing capacity of driven PHC pile at the study sites by using several method approaches. Figs. 7a and 7b show the comparison of ultimate bearing capacity obtained from a CPT-based method, PDA test, and static loading test for the instrumented piles MJ-2 and SH-5, respectively. The figures show that the toe resistances obtained from the PDA test were slightly smaller than that obtained from the static loading test. The relatively matched results from Dung (2008) study have further implied that the PDA test can reasonably predict the toe bearing capacity of driven piles.

6. CONCLUSIONS

Three common SPT-based methods were briefly reviewed in this study. The methods were then applied to estimating the toe bearing capacity of four experimentally driven PHC piles at the two sites in the Nakdong River delta. The corrected N values for N>50 were estimated by using two techniques. The comparison between the toe resistances of the PDA tests and the SPT-based methods showed that the Meyerhof (1976) method gives reasonable results in the upper part of the profiles and it tends to underestimate the toe resistance when the embedded depth into bearing stratum increases significantly. The Decout (1995) method strongly overestimates the toe resistance in dense to very dense sand, however it shows reasonable agreements in loose to medium sandy silt and silty sand. The Robert (1997) method strongly underestimates the toe resistance of the pile. In addition, the technique (2) for correcting N>50, which assumes the linear relation between N values and the penetration depth of rods, significantly overestimated the toe capacity.

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