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Applicability of the SPT-based Methods for Estimating Toe Bearing Capacity of Driven PHC Piles in the Thick Deltaic Deposits

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Abstract

Standard Penetration Test (SPT)-based design methods for pile foundations have been extensively used in Korean practice. However, their applicability for local application has seldom been examined, particularly for the thick deposits, such as in the Nakdong River deltaic area. This paper examines the applicability of three common SPT-based methods to deep sandy deposits in the delta. Routine SPT data, in which the test was completed for each of the $N = 50$ blows at less than 30 cm penetration in the last two increments of 15 cm, were adopted. A special SPT was conducted to examine the general relationship between penetration and blow counts in dense sands. Based on the special SPT and another well-documented case study, a simple linear extrapolation was developed to estimate data equivalent to N > 50. PDA (Pile Driving Analyzer) tests were also carried out to evaluate energy efficiency of the donut hammer used for the routine SPT. Energy correction factor (C_E) was determined as 0.9. Using the corrected *N*-values, the estimated SPT-based toe resistances were compared with data obtained from PDA tests, field load tests on piles, and a CPT (Cone Penetration Test)-based method. Results indicate that the SPT-based methods generally manifest a rather low reliability. The recommended Meyerhof's method is deemed applicable for the preliminary design.

Keywords: *toe resistance, standard penetration test N-value, PDA test, CPT, PHC pile*

1. Introduction

Standard Penetration Test (SPT) is one of the most common and economical field tests used during ground investigation. Apart from its main applications in soil characterization, SPT *N*-value has also been extensively used for designing structural foundations and other earth structures, particularly, for the bearing capacity of piles (Meyerhof, 1976; Shioi and Fukui, 1982; Decourt, 1995; Robert, 1997). Thus, SPT-based methods are included in a number of design codes or manuals (AASHTO, 2007; CFEM, 2006; FHWA, 1996), including in the Korean code (KGS, 2003).

In the last half century, a number of studies were carried out to evaluate the reliability of SPT, its influence factors, and other related criteria. Studies indicate that since the SPT *N*-values include some uncertainties, they are appropriate for use in preliminary designs (Schmertmann, 1978; Kulhawy and Mayne, 1990; Clayton, 1993). Few studies concentrated on the reliability of SPT-based methods for estimating the bearing capacity of piles (Lee and Yun, 1992; Shariatmadari *et al.*, 2006), and these studies likewise indicate that the reliability is rather inconsistent and largely depends on local practices, as well as on local geological conditions. Furthermore, it is not known whether SPT-based methods, with the use of routine practice data, could be reliably applied for piles driven into deep sandy deposits, such as the Nakdong River deltaic area.

The purpose of this paper is to examine the applicability of SPT-based methods in estimating toe bearing capacity of long PHC (Pretensioned spun High strength Concrete) piles driven in the Nakdong River deltaic area. Initially, SPT and CPT tests were carried out at the Myeongji (MJ) and Shinho (SH) sites, from which several PDA (Pile Driving Analyzer) and static loading tests on PHC piles were conducted at great length. PDA tests were also carried out to evaluate energy efficiency of the donut hammer used for routine SPT, with their results consequently used to correct SPT *N*-values. Since the routine SPT was completed at each of the $N = 50$, even for less than 30 cm penetration during the last two increments of 15 cm (N = 50 / <30), a special SPT was additionally conducted at the MJ site. This aimed to examine the relationship between the penetration and blow counts, and to evaluate equivalent data for N/30. The toe bearing capacity of driven PHC piles was calculated using three common SPT-based methods, after which data were compared with those obtained from PDA tests, static loading tests, and the best CPT-

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based method for the sites (Dung *et al.*, 2007). Some discussions and recommendations are further presented in applications of SPT-based methods for pile foundations in the deposits.

2. SPT N-value and Toe Resistance

2.1 SPT and its Corrected N-values

According to ASTM D 1586 – 99 (similarly, with KSF 2307, 2007), the performance of SPT is completed with the occurrence of one of the following: (*i*) a total of 50 blows have been applied during any one of the three 0.15 m increments; (*ii*) a total of 100 blows have been applied; (*iii*) there is no observed advance of the sampler during the application of 10 successive blows of the hammer; and (*iv*) the sampler advanced the complete 0.45 m without the limiting blow counts occurring as described in the first three conditions. The sum of the number of blows required for the second and third 15 cm (6 in.) of penetration is termed as "standard penetration resistance" or the "*N*-value".

The normalized *N*-value to the standard energy ratio of 60% and corrected for a number of effects is often expressed by the following form (Skempton, 1986):

$$N_{60} = NC_E C_B C_S C_R \tag{1}$$

where C_E, C_B, C_S, and C_R are the correction factors for hammer energy efficiency, borehole diameter, sampling method, and length of drill rod, respectively.

The N_{60}-value is then corrected for overburden pressure effect as generally expressed by the following form:

$$(N_1)_{60} = C_N N_{60} \tag{2}$$

where C_N is the correction factor for effective overburden pressure. There are several expressions for the C_N in literature; however, the expression initially proposed by Liao and Whitman (1986) and later suggested in ASTM D 6066 - 96 is used for this study, which is as follows:

$$C_N = (p_a/\sigma'_v)^{0.5} \tag{3}$$

where σ'_v= vertical effective pressure at the SPT test point and p_a reference pressure = 100 kPa.

Table 1. Unit Toe Resistance from SPT-based Methods

Method	Unit toe resistance	Parameter
Meyerhof (1976)[(1), (2)]	$r_t(\text{kPa}) = \dfrac{40(N_1)_{60}D_b}{D} \le r_l$ $r_l(\text{kPa}) = 400(N_1)_{60}$ for $D_b/D \ge 10$	$(N_1)_{60}$ = normalized corrected SPT number, D_b = embedded depth into bearing stratum, D = pile diameter, and r_l= limit resistance.
Decourt (1995)[(1), (3)]	$r_t(\text{kPa}) = K_b(N_{60})$	K_b = 325 for sand, 205 for sandy silt, 165 for clayey silt, and 100 for clay.
Robert (1997)[(3)]	$r_t(\text{kPa}) = K_b(N_1)_{60}$	K_b= 190 for sand

Note: [(1), (2)] and [(3)] indicates CFEM (2006), KGS (2003), and GGEH (2001) respectively, in which the methods are included.

2.2 SPT-based Toe Resistance Methods

A number of SPT-based methods were proposed to estimate the bearing capacity of driven piles. However, only three methods commonly considered in practical design are taken into account in this study, as shown in Table 1. As the SPT data in the clay and upper sand layers are not available for evaluation of the shaft resistance along the piles, only toe resistance is therefore considered for the lower sand layer.

3. Field Soil Tests

3.1 Locations and Ground Conditions

Figure 1 shows the locations of the study sites, the Myeongji (MJ) and Shinho (SH) residential complexes in Busan City. The locations are situated at the coastline of the deltaic area. For land development, a landfill of about 5m thick was constructed and finally completed in the late 1990s (MJ site) and in mid-2000 (SH site). Housing construction only began a few years ago for the MJ site.

The ground conditions of the sites are briefly described as follows. The fill layer is 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. One of its prominent features is a thin clayey silt layer that is mostly sandwiched in the lower sand layer. The thickness of the silty clay layer (upper clay) varies from 16 to 20 m. Groundwater levels at the sites are approximately located 2.5 m below the ground surface.

3.2 The SPT and CPT Profiles

SPT was performed in boreholes of NX size, which were drilled using a rotary wash boring machine with drill rods of Type A (41.3 mm outer diameter and 5.7 kg/m). The SPT was manually operated using a standard donut hammer-cathead system in which the rope was rolled 2.25 turns in the clockwise direction. The common testing procedure was largely identical to the ASTM standard D 1568-99; however, during the last two increments of 15 cm, the test was completed whenever the blow number reached 50 even at a penetration of less than 30 cm (50 /

Fig. 1. Study Locations in the Nakdong River Delta

<30). Due to the thick clay layer known as “Busan clay” (Chung *et al*., 2002), SPT was performed only in the lower sand layer (at least below 30 m), which was considered as the bearing stratum of piled foundations.

At the same time, Cone Penetration Test (CPT) was also performed at the sites using a 20-ton capacity CPT equipment. The test was carried out using an electrical cone of 60° apex angle with base cross-sectional and sleeve friction areas of 15 cm^2 and 225 cm^2, respectively. A porous element was mounted immediately behind the cone shoulder to measure induced pore water pressure (u_2). Average penetration rate was 20 mm/s, as recommended by ISSMFE (1989). A total of 16 SPT - CPTU adjacent tests were conducted at the two sites, in which the distance between SPT and CPTU was less than 5 m at each test location. Fig. 2 shows the typical five SPT-*N* and CPTU-q_c profiles at the sites, in which the first two profiles (MA1P-5, MC2-2) belong to the MJ site and the others (SO2-1, SO3-2, and SO5-3) to the SH site.

3.3 Special SPT and PDA Tests

Apart from the main field soil tests, a special SPT was designed to examine the general relationship between penetration and blow counts until full penetration of 45 cm was achieved; it was carried out at the MD1P-2 location of the MJ site (Fig. 1). Drill rod penetration was continuously measured for each blow. For this test, an automatic hammer system (hammer mass = 63.5 kg, fall height = 76 cm) was used together with the drill rods of Type A connected to the standard sampler and for the casings of NX size. Results are shown in Figs. 3 and 4, and consequently analyzed.

For the calibration of the hammer energy efficiency in the routine SPT, PDA test was additionally carried out at a neighboring site in the delta area where the same equipment and operators were employed. Calibration test procedures conformed to the ASTM D 4633-05. Table 2 provides the summarized results from the test.

4. Correction of the Measured *N*-values

4.1 Correction Factors for Energy Efficiency and Other Effects

In this study, the correction factors C_B, C_S, and C_R in Eq. (1) were taken as unity (Skempton, 1986) since the test was performed in relation to the borehole diameter of NX size, standard sampler, and drill rod larger than 30 m.

Table 2 on the calibrated results for three types of hammers obtained from the PDA test shows that the average Energy Transfer Ratio (ETR) obtained from the tested donut hammer varies from 53.9 to 55.9%. These ratios closely agree with those obtained from other Korean studies (Lee *et al*., 1992; Park *et al*., 1994; Lee *et al*., 2005), which suggest that average energy transfer ratio for donut hammers used in Korean application can be averagely taken as 49.5~53.3%. Based on the measured results, an average ratio of ETR = 54% is obtained for this study. Thus, $C_E = ETR/60 = 0.90$ and $N_{60} = 0.9$ N.

4.2 Extrapolation Technique for Terminated *N*-values

Results from the special SPT performed at MD1P-2 are shown in Fig. 3 adjacent to the CPTU-q_c profile. The test was carried out only in the denser upper part of the lower sand layer at intervals of 1 m. Fig. 4 shows two typical profiles of penetration-blow counts where SPT *N*-values are larger than 50 at 33.5 and 35.5 m

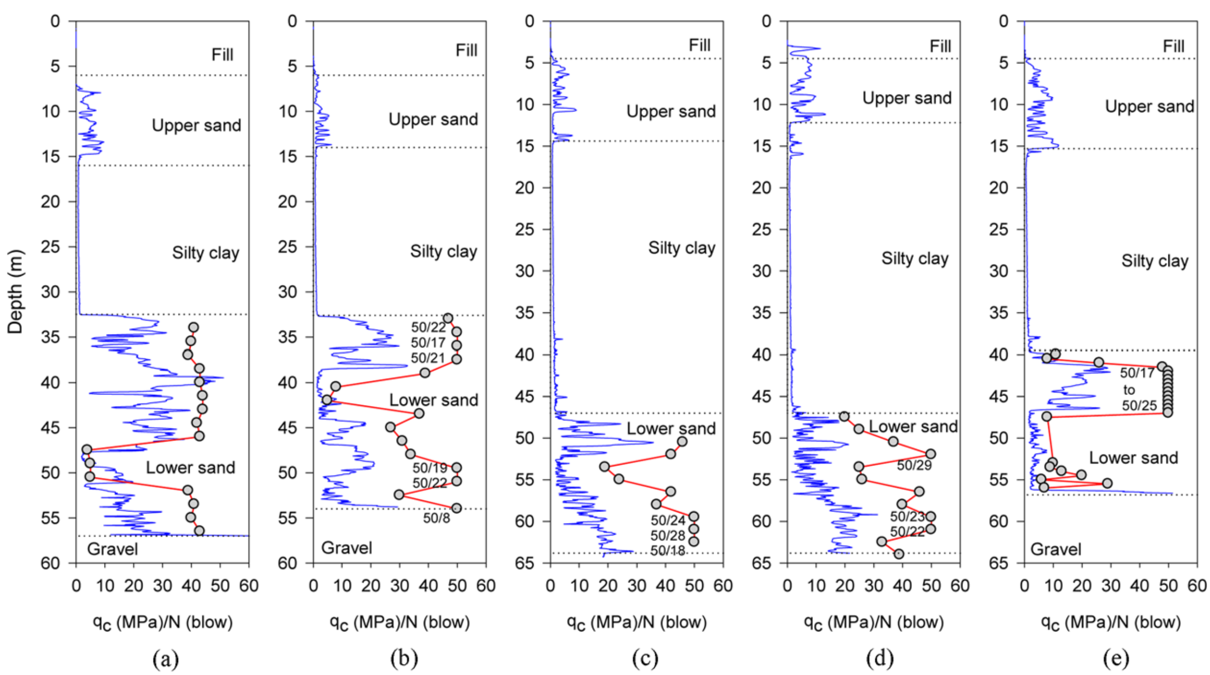

Fig. 2. Typical SPT-*N* and CPTU-q_c Profiles at the MJ and SH Sites: (a) MA1P-5, (b) MC2-2, (c) SO2-1, (d) SO3-2, (e) SO5-3

Table 2. Calibrated Results from the SPT Equipment Used

Hammer type		Safety		Donut		Automatic
Depth	GL-13.5m	GL-30.0m	GL-9.0m	GL-4.5m	GL-1.0m	
Boring No.	BH3-68	BH3-76	BH3-102	BH3-106	BH3-114	
Soil type	Weathered soil	Weathered rock	Weathered soil	Sandy gravel	Sandy gravel	
EMX (tf·m)	Range	0.022~0.033	0.021~0.034	0.021~0.032	0.021~0.032	0.020~0.034
	Avg.	0.027	0.026	0.027	0.026	0.027
ETR (%)	Range	45.6~68.4	43.5~70.5	43.5~66.3	43.5~66.3	41.4~70.6
	Avg.	55.9	53.9	55.9	53.9	55.9

Note: *EMX* (measured energy) = $\int F(t)V(t)dt$; *ETR* (Energy transfer ratio) = *EMX/PE,* where *PE* = potential energy of the hammer ($PE = WH = 0.04826\ t_f m$).

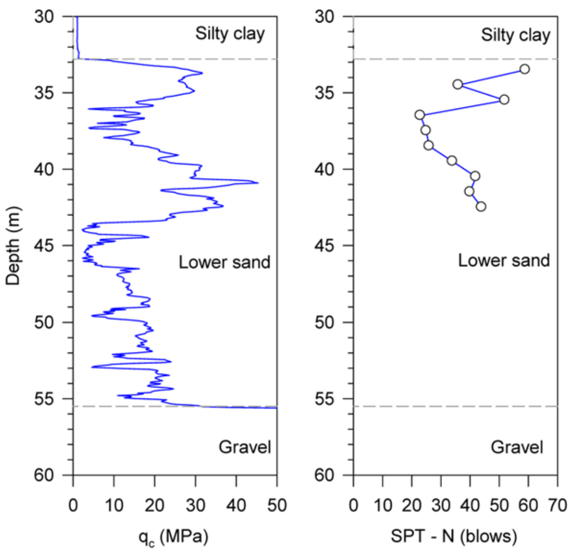

Fig. 3. SPT & CPT Profiles at MD1P-2

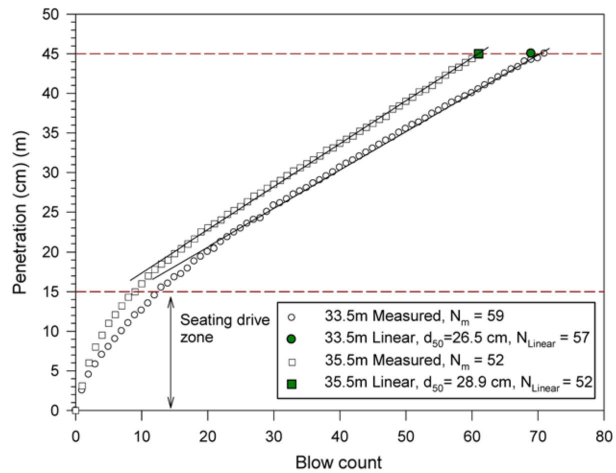

Fig. 4. Two Penetration-blow Count Profiles at MD1P-2

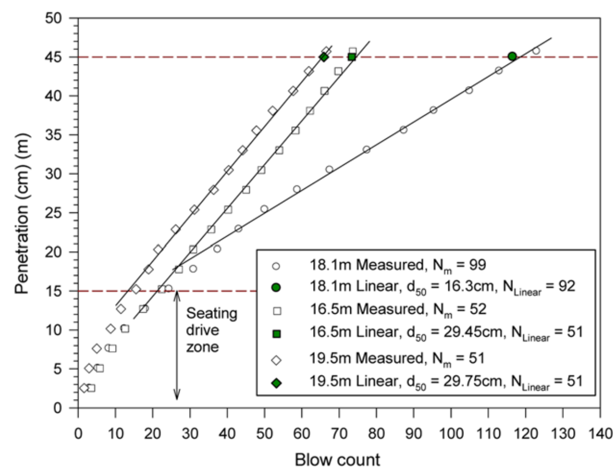

Fig. 5. Three Penetration-blow Count Profiles at Seward Site (Data from Daniel, 2000)

depth. Data indicate that after the seating drive zone (the first 15 cm), the relationships seem to be largely linear.

To verify the above result, a well-documented experimental data is provided. Daniel (2000) experimentally performed a comprehensive study on correlation between Large Penetration Test (LPT) and SPT at Seward Site, Alaska, USA, where the deposit consists of silty and sandy gravels in the flood plain. For these tests, a safety hammer of 63.5 kg (fall height = 76 cm) was used, and the blow counts were recorded at intervals of 1 in (2.54 cm). Fig. 5 shows three typical profiles of penetration-blow counts, which indicate a similar trend to our study results (Fig. 4).

As shown in Figs. 4 and 5, the entire curves of penetration versus blow counts generally exhibit a hyperbolic relationship similar to the load-displacement curve observed from the plate load test or pile static loading test. However, it is interesting to note that after passing the seating drive zone (the first 15 cm), penetration versus blow count curves become approximately linear. For *N*-values counted after the seating drive zone, a simple linear extrapolation equation can thus be developed:

$$N_{Linear} = 50 \times 0.3 / d_{50} \tag{4}$$

where N_{Linear} = equivalent blow count per 0.3 m penetration, d_{50} (m) = measured penetration when the test is terminated at $N = 50$.

The mechanism of driving the SPT sampler into dense sandy deposits is similar with driving open-ended piles. Since the sampler is rather small and short (length = 9 times of diameter), its total shaft resistance is significantly smaller than the total base resistance. In addition, due to cyclic loading, soil elements around the sampler walls become fatigued and loosened (White, 2005; White and Lehane, 2004); thus, shaft resistance decreases with increasing number of blows. In dense sand, the hollow sampler is quickly plugged during penetration in the seating drive zone; consequently, base resistance steadily increases. Similarly, total resistance increases steadily after the seating drive zone, showing a slight nonlinear curve typical of those illustrated in Figs. 4 and 5.

From the recorded data, the penetrations at $N = 50$ blows (d_{50}) were obtained and the linear extrapolated values (N_{Linear}) were calculated using Eq. (4). The extrapolated values (solid symbols) are plotted together with the measured values, as shown in Figs. 4 and 5, which show that the extrapolated values are slightly smaller than those measured at full penetration of 45 cm. The discrepancy between the measured and extrapolated values becomes larger in dense gravel–sands; however, SPT for such deposits is noted as less reliable. As a consequence, it can be stated that the linear extrapolated technique can conservatively be used for practical design. It is applied herein to evaluate equivalent N-values larger than 50.

Table 3. Linearly Extrapolated Values for N> 50

Borehole	Depth (m)	N/d_{50} (blow/cm)	N_{Linear} (blow)
MC2-2	34.5	50/22	68
	36.0	50/17	88
	37.5	50/21	71
	49.5	50/19	79
	51.0	50/22	68
SO2-1	59.5	50/24	63
	61.0	50/28	54
	62.5	50/18	83
SO3-2	59.5	50/23	65
	61.0	50/22	68
SO5-3	42.0	50/25	60
	42.5	50/24	63
	43.0	50/23	65
	43.5	50/19	79
	44.0	50/23	65
	44.5	50/23	65
	45.0	50/17	88
	45.5	50/16	94
	46.0	50/19	79
	46.5	50/18	83
	47.0	50/18	83

Note: the N_{Linear} values were rounded to be integers.

4.3 Linear Extrapolated *N*-values

Figure 2 shows five typical soil profiles at the sites where the PDA and SLT were elaborately executed. In this analysis, the last four locations (i.e., Figs. 2b~2e) where both PDA and CPT profiles were fully made available are taken into consideration. Table 3 shows the measured N-values, at which the test was completed at 50 blows with $d_{50} < 30$ cm, and consequently, with the linearly extrapolated values (N_{Linear}) based on Eq. (4). This indicates that the extrapolated N-values vary from 54 to 94.

5. Toe Resistances

5.1 Toe Resistances based on CPT and Other Methods

To examine applicability of SPT-based methods, toe resistances obtained from CPT, PDA, and Static Loading Test (SLT), which are known as the more reliable methods, were compared with each other.

5.1.1 CPT-based Toe Resistances

It is known that the PDA-based method (CAPWAP analysis) could be the most reliable field test used to determine pile bearing capacity if SLT is not performed on piles. Dung *et al*. (2007) reported a comparison of shaft and toe resistances obtained from various CPT-based and PDA-based methods at the sites. Results indicate that among 10 common CPT-based methods, toe resistance obtained from the Aoki and De Alencar (1975) method provided the best agreement for those obtained from the PDA test. A further study carried out by Hung *et al*. (2008) also drew the same conclusion based on a similar analysis using a total of 172 PDA and CPTU test data (without adjacent SPT data available). Therefore, the Aoki and De Alencar (1975) method (hereafter denoted as "CPT-based") are applied herein, the result of which is compared with those from the SPT-based methods.

5.1.2 Toe Resistances from PDA and SLT

PDA test was performed for four closed-ended PHC piles of 600 mm in diameter. One pile was driven at MC2-2 location (named here as pile MC2-2) and three others at the locations of SO2-1, SO3-2, and SO5-3, and with similar names assigned. The piles were successfully driven using a hydraulic impact hammer with the maximum potential energy of 24 $t_f \cdot m$. PDA test was systematically performed through the driving process from the first until the final stroke. Piles were then re-struck 39 days, 42 days, 56 days and 59 days after the End of Initial Driving (EOID). Further details on the PDA results were reported by Dung *et al*. (2007).

Piles MC2-2 and SO5-3 were well-instrumented with strain gauges to monitor development of residual loads with time. An O-cell was installed at the bottom of the pile SO5-3. Static Loading Test (SLT) and O-cell test were then applied to the piles MC2-2 and SO5-3 at 164 days and 219 days after driving, respectively. Further details of the two piles can be inferred from the reports of Kim *et al*. (2006), Dung (2008), and Fellenius et al. (2009). An additional instrumented pile was later installed at location MA1P-5 in the MJ site, which also included an O-cell at the bottom of the pile. Further details of pile MA1P-5 can be inferred from Kim *et al*. (2008). Table 4 shows the toe and shaft resistances obtained from the PDA, SLT, and O-Cell tests on the three instrumented piles.

It should be noted that PDA-based toe resistances illustrated in Table 4 were taken from the EOID. It is because the hammer energy was insufficient to fully mobilize the shaft resistance of the long PHC piles that were subjected to large residual loads. Toe resistances obtained from the re-strikes were therefore smaller than with the EOID. As shown in Table 4, PDA-based toe resistances matched relatively well with those obtained from the SLT and O-Cell tests. It can also be inferred that if hammer energy was sufficient to fully mobilize the piles during the re-strikes, PDA-based toe resistances would have been closer to the data on the SLT and O-Cell tests. This is in good agreement with existing conclusions supporting PDA-based resistances, which can be reliably used for practical applications (Likins *et al*., 1996; Likins and Rausche, 2004).

5.2 SPT-based Toe Resistances

The toe bearing capacity of driven closed-ended PHC piles of 600 mm diameter was estimated using the corrected *N*-values (i.e., N_{60}, $(N_1)_{60}$). SPT-based toe resistances from the three methods were then plotted together with the values obtained from the PDA-based, CPT-based, and SLT methods, as shown in Fig. 6. It appears that the CPT-based resistances match rather well with the PDA-based values, except for some points at SO3-2. The discrepancy was probably due to the heterogeneity of soil in the horizontal direction at the site, which made the PDA-based resistances smaller in the middle part. The relatively good agreement between CPT-based, PDA-based, and SLT results, as shown in Table 4 and Fig. 6, further confirms previous studies (Dung *et al*., 2007; Hung *et al*., 2008); that is, both PDA-based and CPT-based design approaches are appropriate for the design of the driven PHC piles in the thick delta. The methods of Decourt (1995) and Robert (1997), which were based on SPT, appear to have produced larger and smaller toe resistances, respectively, in comparison with the PDA-based and CPT-based values. The Meyerhof (1976) method appears to be the best among the SPT-based methods, giving close resistances not only to the PDA-based, but also to the CPT-based values.

Table 4. Toe and Shaft Resistances Obtained from the Instrumented Piles

Pile	Length (m)	Applied test	PDA-based resistance		SLT-based resistance	
			R_t (kN)	R_s (kN)	R_t (kN)	R_s (kN)
MA1P-5	35.0*	PDA, O-Cell	2520	930	2480	2830
MC2-2	35.0	PDA, SLT	2800	1430	3250	3680
SO5-3	56.6	PDA, O-Cell, SLT	3300	5570	3920	8640

*Including 3m excavation from the original ground surface.

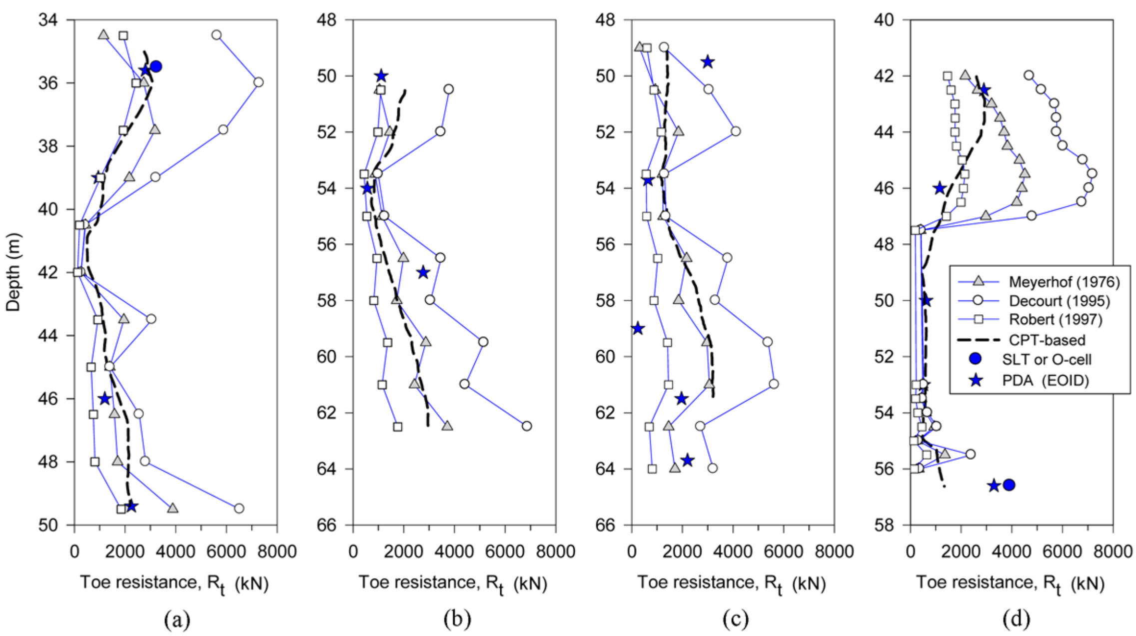

Fig. 6. Toe Resistances from Different Methods: (a) MC2-2, (b) SO2-1, (c) SO3-2, (d) SO5-3

As shown in Fig. 6, while the Meyerhof (1976) method achieved toe resistances comparable to those obtained from either PDA-based and CPT-based methods, even at depths as deep as 60m ($D_b/D \geq 10$), the method provided rather smaller results than those from the two methods in the upper part of the lower sand layer (i.e., in the embedded depths of $D_b/D < 10$ into the bearing stratum). Thus, it seems that smaller toe resistances have resulted directly from the influence of the embedded depth ratio defined for the unit toe resistance (Table 1). For example, in many cases where we were unable to drive the piles into a very hard stratum, toe resistance was negligible because $D_b \approx 0$. In reality, however, piles could contribute a significant amount of toe resistance. To avoid inconsistencies, limited unit resistance was applied to the whole bearing stratum (the lower sand layer), and thus, unit toe resistance can be simply given as follows (denoted as the modified Meyerhof method):

$$r_t\,(\text{kPa}) = 400(N_1)_{60} \tag{5}$$

Figure 7 shows a comparison between the toe resistances obtained from the PDA- and CPT-based methods, the original and modified Meyerhof (1976) methods, for the depths up to the embedded depth (D_b) of 10 times pile diameter (D) into the lower sand layer. It appears that the modified toe resistances were closer to the CPT-based values than with the original values.

5.3 Reliability of the SPT-based Methods

To examine the reliability of the SPT-based methods, SPT-based toe resistances ($R_{t\text{-}SPT}$) versus PDA-based ($R_{t\text{-}PDA}$) and CPT-based ($R_{t\text{-}CPT}$) values were plotted; they are shown in Figs. 8 and 9, respectively. When the tested depths were different between the PDA and SPT, the $R_{t\text{-}PDA}$ value at the level of the SPT was linearly interpolated from the two closest $R_{t\text{-}PDA}$ values. Both comparisons generally indicate that the SPT-based values tend to be poorly correlated with the CPT and PDA-based methods. Particularly, it seems that from Figs. 6, 8, and 9, the Decourt (1995) method strongly overestimates the toe resistances as compared with the PDA- and CPT-based values. Meanwhile, the Robert (1997) method underestimates the values, especially in dense sands. The Meyerhof (1976) method seems to be better, as compared to the other two methods.

To be quantitatively clear, Table 5 is given to show the geometric mean values of $R_{t\text{-}SPT}/R_{t\text{-}PDA}$ and $R_{t\text{-}SPT}/R_{t\text{-}CPT}$, and the coefficients of determination [R^2 = 1 - (the sum of square errors)/(the

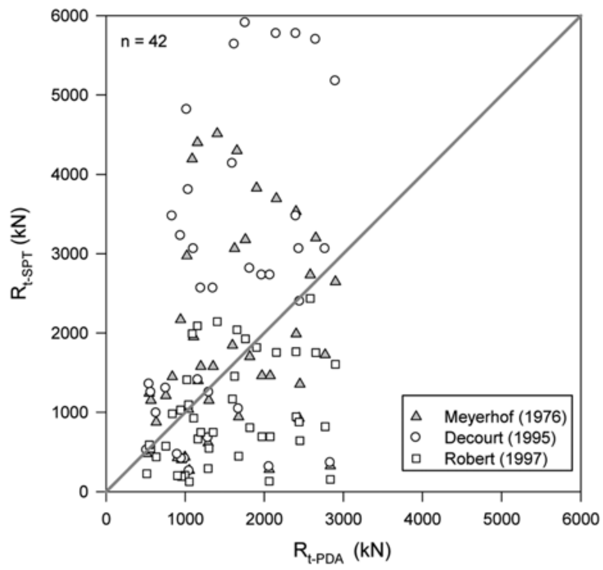

Fig. 8. $R_{t\text{-}SPT}$ versus $R_{t\text{-}PDA}$

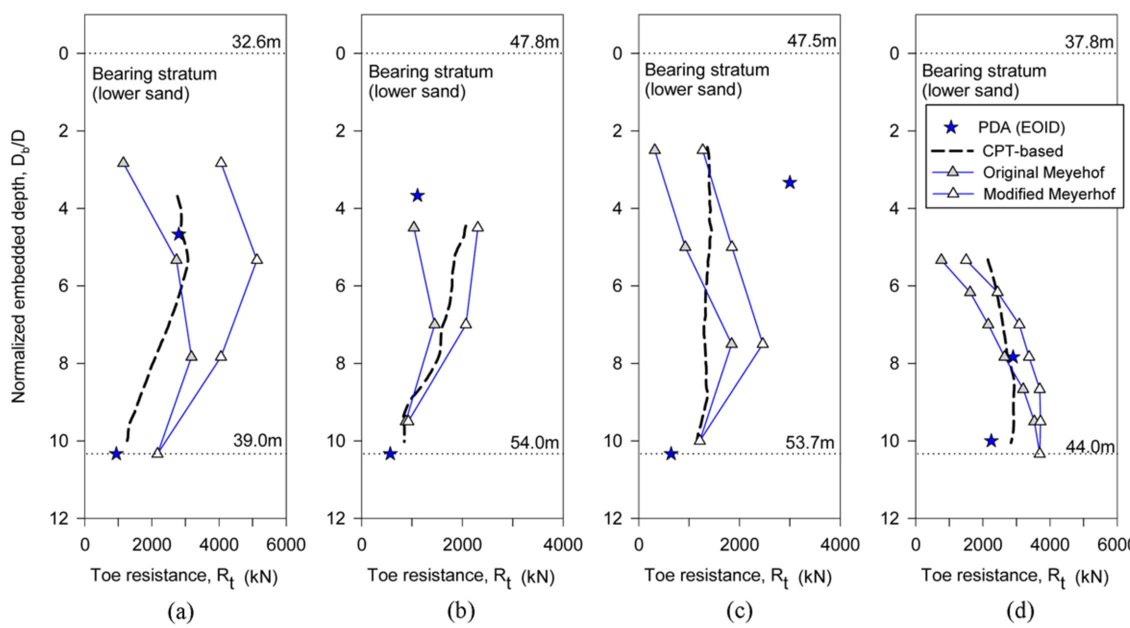

Fig. 7. Toe Resistances from the Original and Modified Meyerhof (1976) Methods: (a) MC2-2, (b) SO2-1, (c) SO3-2, (d) SO5-3

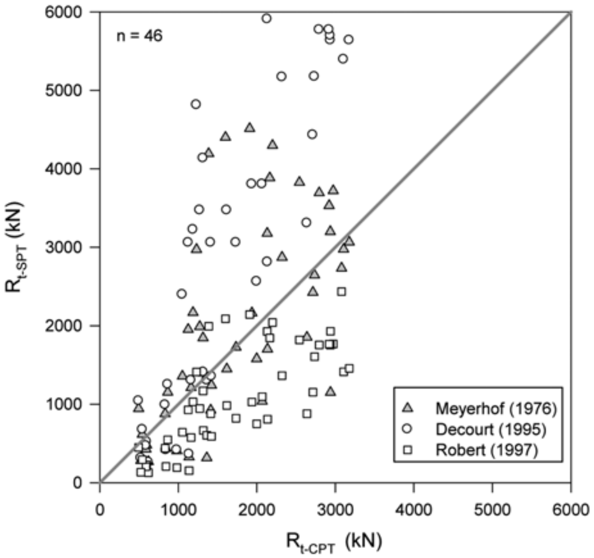

Fig. 9. $R_{t\text{-}SPT}$ versus $R_{t\text{-}CPT}$

total sum of squares)]. It should be noted that the coefficient of determination was determined based on the equal line ($R_{t\text{-}PDA}$ = $R_{t\text{-}SPT}$; $R_{t\text{-}CPT}$ = $R_{t\text{-}SPT}$), where $R_{t\text{-}PDA}$ and $R_{t\text{-}CPT}$ are considered as basic values for the comparison. It appears that the reliability of SPT-based toe resistances is very low since most of the coefficients were negative. It is probable that the high overestimation have resulted from the Decourt (1995) method, which was mainly due to the use of the normalized SPT number, N_{60}, rather than the normalized corrected one, $(N_1)_{60}$. That is, N_{60} value does not include the factor C_N which becomes as small as 0.5 in the deep deposits as the cases presented herein. However, the underestimation resulting from the Robert (1997) method may have come directly from the use of the rather small ratio of $r_t/(N_1)_{60}$ = 190. This fact was also highlighted by Poulos (2001). Therefore, it is not recommended that these two methods be used for thick and inhomogeneous deposits, such as the Nakdong River deltaic area.

As opposed to the Decourt (1995) and Robert (1997) methods, the Meyerhof (1976) method seems to produce relatively reasonable toe resistances with mean values of $R_{t\text{-}SPT}/R_{t\text{-}PDA}$ and $R_{t\text{-}SPT}/R_{t\text{-}CPT}$ are 1.04 and 0.87, respectively. However, the coefficient of determination remains very low (i.e., R^2 = 0.0044 and 0.4434, respectively), which indicates that even the Meyerhof method has low reliability compared to the CPT- and PDA-based methods. Nevertheless, this result supports data mentioned in many practical design manuals. The Meyerhof (1976) method is applicable to preliminarily determine the toe bearing capacity of driven piles, provided that SPT N-values are reliably obtained. In addition, practicing engineers may use the limited unit toe resistance, Eq. (5), for sand layers in preliminary designs.

Table 5. Mean Values of toe Resistance Ratios and Correlation Coefficients

Method	$R_{t\text{-}SPT}/R_{t\text{-}PDA}$		$R_{t\text{-}SPT}/R_{t\text{-}CPT}$	
	Mean	R^2	Mean	R^2
Meyerhof (1976)	1.04	0.0044	0.87	0.4434
Decourt (1995)	1.58	-0.1887	1.42	-0.0004
Robert (1997)	0.54	-0.9316	0.46	-0.4829

6. Conclusions

This study evaluated whether three common SPT-based methods for toe bearing capacity of piles are applicable for deep sandy deposits in the Nakdong River estuary area in Busan City, Korea. To achieve this, routine SPT were carried out in two sites. The test was terminated whenever the blow number reached 50 in the last two 0.15 m increments. A special SPT was performed to develop a method for extrapolating terminated N-values. In addition, PDA tests were also conducted to determine energy correction factors for the routine SPT. The corrected N-values (i.e. for N_{60} or $(N_1)_{60}$) were used to estimate the toe bearing capacity of driven PHC piles. Results were compared with those obtained from the PDA, SLT, O-cell tests, and a CPT-based method. The conclusions drawn from this study are as follows:

1. From the special SPT and another well-documented case study, an approximate linear relationship between penetration and blow counts was found for the last two 15 cm increments, which was used for extrapolating the terminated N-values of 50. Linear extrapolated values (N_{Linear}) appear to be slightly smaller than those obtained from the measured data. It was also found that energy efficiency of the donut hammer used at the sites was on average 54% of the theoretical free-fall energy. Thus, energy correction factor was determined as C_E = 0.90.
2. In general, the toe bearing capacity estimated from the SPT-based methods has low reliability in comparison with those obtained from CPT-based methods, PDA, SLT, and O-cell tests. The Decourt (1995) and Robert (1997) methods seem to highly overestimate and underestimate the toe bearing capacity, respectively; unlike the Meyerhof (1976) method, which particularly provided closer values to those obtained from the CPT-based, PDA-based methods, and field load tests.
3. A modified Meyerhof method was proposed based on other reliable methods that were adopted herein; that is, the limited unit resistance (r_l) is considered for the whole bearing stratum instead of using embedded depth ratio to up to D_b/D = 10 in the Meyerhof (1976) method. It is recommended that the modified Meyerhof method be used for the toe bearing capacity of the driven PHC piles for preliminary design, particularly, in the thick deltaic deposits.

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