預鑽孔植入 PC 樁於排水土層之受壓承載力評估
Evaluation of Axial Capacity of Pre-bored PC Piles in Drained Soils

陳梅竹 陳俊宏 陳逸駿*
Maria Cecilia M. Marcos, Chung-Hung Chen, Yit-Jin Chen*

摘要

本研究蒐集 50 筆排水土層之預鑽孔植入式 PC 樁受軸向壓載試驗，用以評估代表性樁底承載力及樁身摩擦力之分析模式。為評估樁之實際破壞載重，本研究採用 L_2 詮釋法。依分析統計結果顯示，排水土層之理論分析模式於預估底承力時產生相當高估的情形，而預估樁身摩擦力時則有低估的現象，但隨著樁深度增加其預估摩擦力趨近量測值。另透過載重試驗資料之回饋分析，預鑽孔植入式 PC 樁於分析樁身摩擦力之建議側向係數因子 (K/K_o) 亦示於文內。
關鍵詞：載重試驗，排水土層，預鑽孔植入 PC 樁，樁身摩擦力，樁底承載力

Abstract

Axial compression capacities of 50 pre-bored precast concrete piles under drained loading condition are examined. Predicted tip capacity and side resistance are assessed using representative analytical models. L_2 method is adopted to acquire the measured capacities. From the analysis results, the predicted model overestimates the tip capacity whereas side resistance is underestimated but the ratio of predicted/measured is convergent to 1.0 as the pile length increases. The stress factor (K/K_o) for pre-bored PC piles is developed from the back-analysis of available field load tests.
Keywords: load tests, drained soils, pre-bored PC piles, side resistance, tip capacity

I. INTRODUCTION

Driven piles and drilled shafts are among the most commonly used pile foundations worldwide. In recent years, the use of pre-bored PC piles has increased due to a number of advantages including low vibration, noise, and cost with comparable capacity with other types of piles. Research on evaluating drilled shaft (or driven pile) behavior and capacity has been progressing [1-5], while less effort has been done to explicitly evaluate the behavior of pre-bored PC piles. With the apparent differences in construction methods, the evaluation of pile capacity (including tip capacity and side resistance) for various piles substantially varies.

Axial compression capacity of pre-bored PC piles is provided by the tip capacity generated from the bearing strength of soil beneath the tip and side resistance supplied by the soil shearing resistance over the side area of the pile, as demonstrated in Fig. 1. Theoretically, tip capacity in compression is given by:

$$Q_{tcp} = q_{ult} \times A_{tip} \tag{1}$$

In which Q_{tcp} = predicted tip capacity, A_{tip} = pile tip area, and q_{ult} = ultimate bearing capacity. The Terzaghi-Buisman equation has been used for the general solution of q_{ult} [6] as:

$$q_{ult} = cN_c + qN_q + 0.5\gamma BN_\gamma \tag{2}$$

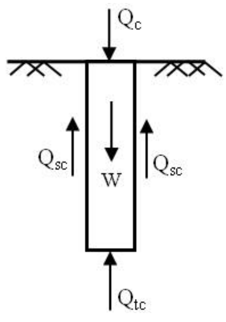

Fig. 1 Force equilibrium diagram in compression

中原大學土木工程系
*Corresponding author. E-mail: yjc@cycu.edu.tw
Department of Civil Engineering, Chung Yuan Christian University, Chung-Li, Taiwan, R.O.C.
Manuscript received 3 March 2011; revised 13 July 2011; accepted 18 July 2011

In which c = soil cohesion, γ = soil unit weight, B = pile diameter, q = vertical stress at pile tip, and N_c, N_γ, N_q = bearing capacity factors. The popular ultimate bearing capacity equation, (q_{ult}) has been improved by several researchers [6-8] to extend the situation to actual field conditions. Additional modifiers that include foundation shape (s), depth (d) and rigidity (r) have been introduced. Considering these modifiers for circular drilled shafts, the general form of the bearing capacity equation for drained compression tip resistance is derived as:

$$q_{ult} = \bar{q} N_q \zeta_{qs} \zeta_{qd} \zeta_{qr} + 0.3 \bar{\gamma} B N_\gamma \zeta_{\gamma r} \quad (3)$$

In which $\zeta_{qs}, \zeta_{qd}, \zeta_{qr}$ = modifiers of N_q for foundation shape, depth, and soil rigidity, respectively, $\zeta_{\gamma r}$ = modifier of N_γ for soil rigidity, and $\bar{q}$ and $\bar{\gamma}$ = effective vertical stress and soil unit weight, respectively. The detailed values of N_γ, N_q and modifiers are presented elsewhere [9]. Recently, a re-evaluation of the tip capacity of drilled shaft was studied [5] using a large amount of field load test data in drained soils and presented that measured tip capacity is much less than the predicted capacity.

Meanwhile, a widely known effective stress analytical model for analyzing drained side resistance is the β method. The original β method [10] has been modified by Kulhawy *et al.* [8] and is given by

$$Q_{scp} = \pi B \left(\frac{K}{K_o} \right) \sum_{n=1}^{N} \bar{\sigma}_{vn} K_{on} \tan \left[\bar{\varphi}_n \cdot \frac{\delta}{\bar{\varphi}} \right] t_n \quad (4)$$

In which Q_{scp} = predicted side resistance, K = coefficient of horizontal soil stress, K_o = in-situ K, $\bar{\sigma}_v$ = vertical effective stress, $\bar{\phi}$ = effective stress friction angle, δ = soil-shaft interface friction angle, β = K tan δ, B = shaft diameter, n = number of soil layers, and t = thickness of soil layer.

In this method, side resistance is a function of horizontal effective stress ($\bar{\sigma}_{ho}$), effective stress friction angle (δ) at soil-shaft interface, and shaft geometry. Kulhawy *et al.* [8] examined the available load test data and presented that the stress factor (K/K_o) for drilled shafts is generally less than 1 and is dependent on the construction method and its influence to the in-situ stress. Later, Chen *et al.* [3] re-assessed the K/K_o for drilled shafts under different construction methods and larger values than the previous study [8] have been established. In order to apply this factor to pre-bored PC piles, the dry construction method for drilled shaft [3] having a K/K_o = 1.03 is adopted.

In this study, evaluation of pile tip capacity and side resistance of pre-bored PC piles is carried out on numerous load tests case histories in drained soils by imposing the representative analytical models. The applicability and reliability of these models to pre-bored PC piles are examined. Specific design recommendations in using the analytical models are thereafter presented.

II. DATABASE OF LOAD TESTS

The database developed in this study is consisted of 50 field axial load tests conducted at 18 sites in Taiwan at different times. Based on the predominant soil condition along the shaft length, the load test data are primarily in drained condition. All of the selected load tests were conducted on circular pre-bored precast concrete piles and have almost complete geological information and load- displacement curve.

The L_1-L_2 method proposed by Hirany and Kulhawy [1, 11], which is a graphical construction method, is adopted to measure the axial compression capacity. The method presents reasonable results for pile design based on previous studies [4, 12]. As shown in Fig. 2, the load-displacement curve can generally be simplified into three distinct regions: initial linear, curve transition, and final linear. Point L_1 (elastic limit) corresponds to the load (Q_{L1}) and butt displacement (ρ_{L1}) at the end of the initial linear region, while L_2 (failure threshold) corresponds to the load (Q_{L2}) and butt displacement (ρ_{L2}) at the initiation of the final linear region. Q_{L2} is defined as the "interpreted failure load or capacity" because beyond Q_{L2}, a small increase in load gives a significant increase in displacement. The research study by Chen and Fang [4] defined that Q_{L2} occurs on average, at 4%B for drained compression loading. With the interpreted compression capacity, the measured tip capacity (Q_{tcm}) and measured side resistance (Q_{scm}) can be deduced from the load-distribution curve along the shaft length. A statistical result [13] revealed that tip resistance and side resistance are on average 20% and 80%, respectively, of Q_{L2}. In the absence of the load-distribution curve, this result is adopted to infer Q_{tcm} and Q_{scm}.

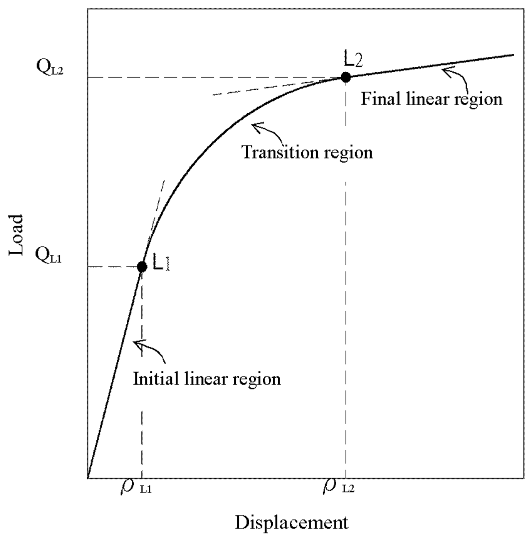

Fig. 2 Regions of load–displacement curve

Table 1 Basic information and pile capacity results

Pile No.	depth, D (m)	dia., B[a] (m)	D/B	Q_{tcm} (kN)	Q_{tcp} (kN)	Q_{tcm}/Q_{tcp}	Q_{scm} (kN)	Q_{scp} (kN)	Q_{scm}/Q_{scp}	$(K/K_o)_n$[b]
TP1	35.0	0.7	50.0	1019	9466	0.11	4077	3678	1.11	1.14
TP2	35.0	0.7	50.0	1043	9466	0.11	4171	3678	1.13	1.17
TP3	38.0	0.8	47.5	1619	18614	0.09	6476	6016	1.08	1.11
TP4	38.0	0.8	47.5	1599	18614	0.09	6397	6016	1.06	1.10
TP5	38.0	0.7	54.3	2058	7341	0.28	8232	4395	1.87	1.93
TP6	29.0	0.9	32.2	1588	6319	0.25	6350	2570	2.47	2.55
TP7	29.0	0.9	32.2	1521	6319	0.24	6084	2570	2.37	2.44
TP8	29.0	0.7	41.4	845	3823	0.22	3381	1999	1.69	1.74
TP9	29.0	0.7	41.4	698	3823	0.18	2793	1999	1.40	1.44
TP10	26.0	0.6	43.3	623	3761	0.17	2493	1756	1.42	1.46
TP11	26.0	0.6	43.3	770	3761	0.20	3081	1756	1.75	1.81
TP12	15.0	0.6	25.0	284	2278	0.12	1137	620	1.83	1.89
TP13	25.0	0.7	35.7	537	3492	0.15	2148	1862	1.15	1.19
TP14	25.0	0.7	35.7	510	3492	0.15	2038	1862	1.09	1.13
TP15	25.0	0.7	35.7	608	3492	0.17	2430	1862	1.31	1.34
TP16	25.0	0.7	35.7	596	3492	0.17	2383	1862	1.28	1.32
TP17	25.0	0.7	35.7	674	3492	0.19	2697	1862	1.45	1.49
TP18	25.0	0.7	35.7	461	3492	0.13	1842	1862	0.99	1.02
TP19	25.0	0.7	35.7	843	3492	0.24	3371	1862	1.81	1.87
TP20	25.0	0.7	35.7	545	3492	0.16	2180	1862	1.17	1.21
TP21	25.0	0.7	35.7	455	3492	0.13	1819	1862	0.98	1.01
TP22	30.0	0.7	42.9	1470	4273	0.34	5880	2273	2.59	2.66
TP23	32.0	0.6	53.3	958	3283	0.29	3832	2225	1.72	1.77
TP24	32.0	0.6	53.3	916	3283	0.28	3665	2225	1.65	1.70
TP25	32.0	0.7	45.7	1058	4468	0.24	4234	2596	1.63	1.68
TP26	32.0	0.6	53.3	708	3283	0.22	2833	2225	1.27	1.31
TP27	32.0	0.6	53.3	875	3283	0.27	3499	2225	1.57	1.62
TP28	32.0	0.7	45.7	992	4468	0.22	3969	2596	1.53	1.58
TP29	33.0	0.7	47.1	1470	11309	0.13	5880	3289	1.79	1.84
TP30	33.3	0.9	37.0	1960	18677	0.10	7840	4258	1.84	1.90
TP31	20.0	0.6	33.3	381	2341	0.16	1523	1042	1.46	1.51
TP32	20.0	0.6	33.3	490	2341	0.21	1958	1042	1.88	1.94
TP33	25.0	0.6	41.7	537	2493	0.22	2148	1587	1.35	1.39
TP34	24.0	0.7	34.3	674	3403	0.20	2697	1721	1.57	1.61
TP35	10.0	0.6	16.7	169	1731	0.10	674	286	2.36	2.43
TP36	10.0	0.6	16.7	163	1772	0.09	651	296	2.20	2.27
TP37	25.0	0.6	41.7	494	2601	0.19	1976	1627	1.21	1.25
TP38	20.0	0.6	33.3	414	2439	0.17	1654	1106	1.50	1.54
TP39	15.0	0.6	25.0	439	2115	0.21	1756	653	2.69	2.77
TP40	15.0	0.6	25.0	412	2115	0.19	1646	653	2.52	2.60
TP41	15.0	0.6	25.0	541	2298	0.24	2164	768	2.82	2.90
TP42	30.0	0.7	42.9	784	6307	0.12	3136	2870	1.09	1.13
TP43	30.0	0.7	42.9	651	6307	0.10	2603	2870	0.91	0.93
TP44	30.0	0.7	42.9	811	6585	0.12	3246	2930	1.11	1.14
TP45	30.0	0.7	42.9	596	4828	0.12	2383	2822	0.84	0.87
TP46	40.0	0.7	57.1	1156	7117	0.16	4626	4524	1.02	1.05
TP47	40.0	0.7	57.1	1107	7117	0.16	4430	4524	0.98	1.01
TP48	22.6	0.8	28.3	1607	5584	0.29	6429	2030	3.17	3.26
TP49	22.6	0.8	28.3	776	5584	0.14	3105	1972	1.57	1.62
TP50	22.6	0.8	28.3	892	5584	0.16	3567	1972	1.81	1.86

note: a-use drilling diameter as pile diameter; b-back calculated K/K_o

Table 2 Statistical results of capacities and stress factor

Data type	Statistics	Q_{tcm}/Q_{tcp}	Q_{scm}/Q_{scp}	$(K/K_o)_n$ [a]
All data	n	50	50	50
	Range	0.09~0.34	0.84~3.17	0.87~3.26
	Mean	0.18	1.60	1.65
	SD	0.06	0.55	0.57
	COV	0.35	0.35	0.35
Average per site	n	18	18	18
	Range	0.09~0.34	0.99~2.59	1.02~2.66
	Mean	0.18	1.64	1.69
	SD	0.07	0.50	0.51
	COV	0.38	0.30	0.30

note: a-back calculated K/K_o

In addition, some load tests were terminated before reaching the interpreted load; thus the last portion of the load-displacement curve for test data are inferred using the "hyperbolic method", as suggested by Chen [2]. Pile diameter ranges from 0.50 to 0.80 m, while the shaft depth ranges from 10.0 to 40.0 m. The detailed analyses as well as the reference sources of these tests are too lengthy to list down, however they are summarized elsewhere [12].

Table 1 shows the result of predicted and measured tip capacity and side resistance as well as the ratios of measured and predicted values and the back-calculated K/K_o for pre-bored PC piles. For convenience, Table 2 provides a summary of the ranges of capacity ratios and stress factor, K/K_o with their coefficient of variation (COV), which is the standard deviation (SD) divided by the mean value. In addition, the average value per site is presented so as to avoid site bias brought on by the numerous amounts of load tests. The result of the statistics is also shown in Table 2.

III. COMPARISONS OF AXIAL COMPRESSION CAPACITY

1. Tip Capacity

The predicted tip capacity (Q_{tcp}) is calculated using Eqs. (1) and (3), while Q_{tcm} is interpreted using L_1-L_2 method, as described previously. The statistics in Table 2 demonstrate a mean Q_{tcm}/Q_{tcp} considering all data of 0.18 with a COV of 0.35. The average per site shows a quite small difference with all data, confirming a minimal site bias. Following the all data values, the measured results are only about 18% of the predicted results. This reveals an overestimation of the tip capacity. Fig. 3 shows the comparison between predicted and measured tip capacity and the result of the regression analysis showing a mean ratio of 0.15. Apparently, tip capacity in drained soils provides limited resistance under allowable design settlement and the use of Eq. (3) for tip capacity prediction is overestimated.

Similar phenomenon was encountered in previous studies for drilled shaft tip capacity in drained layers [5] and gravelly soils [14]. Former result in drained soils [5] presented a measured tip capacity of drilled shafts of only about 10%. The result of this study is obviously larger by around 8% than the previous one. This may be attributed to the grouting procedure applied in pre-bored PC pile upon installation which can improve the pile resistance and reduce sediments in the pile tip. Nevertheless, the cause of the overestimation of Eq. (3) may possibly be due to the effect of the model parameters such as N_q or $\bar{q}$.

2. Side Resistance

The predicted side resistance (Q_{scp}) is calculated using Eq. (4), while Q_{scm} is similarly interpreted using L_1-L_2 method. The statistics in Table 2 demonstrate a mean Q_{scm}/Q_{scp} considering all data of 1.60 with a COV of 0.35. The average per site shows a quite small difference compare with all data, confirming a minimal site bias, hence the all data values are analyzed. On the contrary, the result indicates an underestimation of side resistance. Fig. 4 shows the comparison between predicted and measured side resistance and the result of the regression analysis with a mean ratio of 1.39. The occurrence is likely due to the underestimation of soil strength parameters or some construction factors. One reason may be due to the assumption that the soil is normally consolidated, where in fact overconsolidation is possible especially for shallower depth, therefore K_o is likely underestimated for shorter piles. Furthermore, as previously mentioned, the stress factor (K/K_o) is set to 1.03 as per Chen *et al.* [3] result in drilled shaft for dry construction method. However, from the back-calculation result utilizing the field load test data, it can be seen in Table 2 that the K/K_o is obviously larger and has a mean value of 1.65. It can be regarded from the good

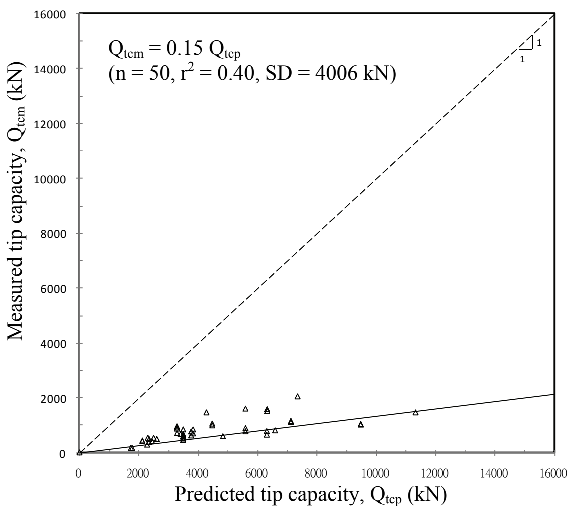

Fig. 3 Comparison of drained predicted and measured tip capacity

adhesion of the grouting materials, the pile, and the surrounding soil during grouting leading to a better side resistance. Therefore, the K/K_o result of this study is more suitable and suggested to use in pre-bored PC piles.

Figs. 5(a) and 5(b) show the relations of Q_{scm}/Q_{scp} and pile tip depth (D) and depth/diameter (D/B), respectively. It can be observed in both figures that the ratio Q_{scm}/Q_{scp} generally decreases and is converging to 1.0 as the depth of the pile increases. This phenomenon supports the above analysis that K_o has been underestimated because at a shallower depth, overconsolidated soil condition occurs and as the depth increases, the soil tends to be in a normally consolidated condition. The corresponding statistical data for Figs. 5(a) and 5(b) are demonstrated in Tables 3(a) and 3(b), respectively. The tables list the numbers of load tests (n) used for the specified depths, the mean capacity ratio, standard deviation, and the coefficient of variations (COV). In general, both tables demonstrate that the mean Q_{scm}/Q_{scp} decreases with depth.

IV. CONCLUSIONS

Fifty load test data are used for evaluating the axial compression capacity of pre-bored PC piles in drained soil condition. Representative analytical models are utilized to assess in detail the tip capacity and side resistance in both measured and predicted results. Based on the analyses, the following conclusions are reached and a design recommendation for stress factor is proposed.

1. The mean ratio of measured to predicted tip capacity is ranging from 0.15 to 0.18 indicating that bearing capacity theory unreasonably predicts the tip capacity and cannot be reliably applied to prediction for drained soils under tolerable design settlement.

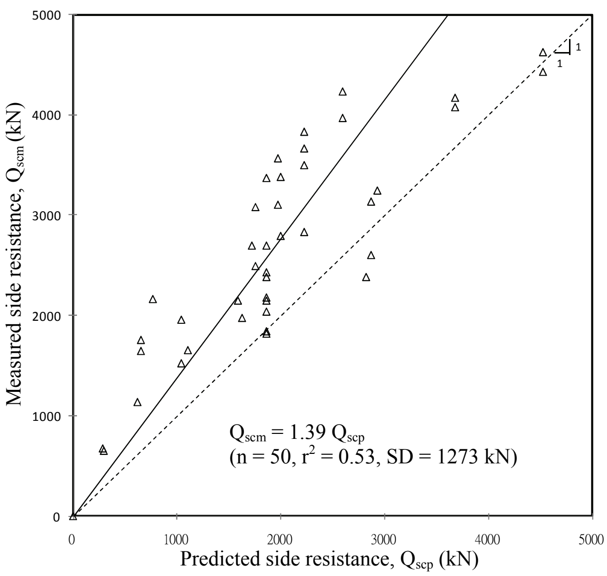

Fig. 4 Comparison of drained predicted and measured side resistance

Table 3 (a) Statistics of Q_{scm}/Q_{scp} with depth, (b) Statistics of Q_{scm}/Q_{scp} with D/B

(a)

Depth range (m)	10-15	16-20	21-25	26-30	31-35	36-40
n	6	3	16	10	10	5
Mean	2.40	1.61	1.53	1.51	1.52	1.20
SD	0.36	0.23	0.59	0.57	0.26	0.38
COV	0.15	0.14	0.39	0.38	0.17	0.31

(b)

D/B range	16-30	31-40	41-50	51-60
n	9	17	17	7
Mean	2.33	1.58	1.30	1.44
SD	0.53	0.51	0.30	0.35
COV	0.23	0.32	0.23	0.24

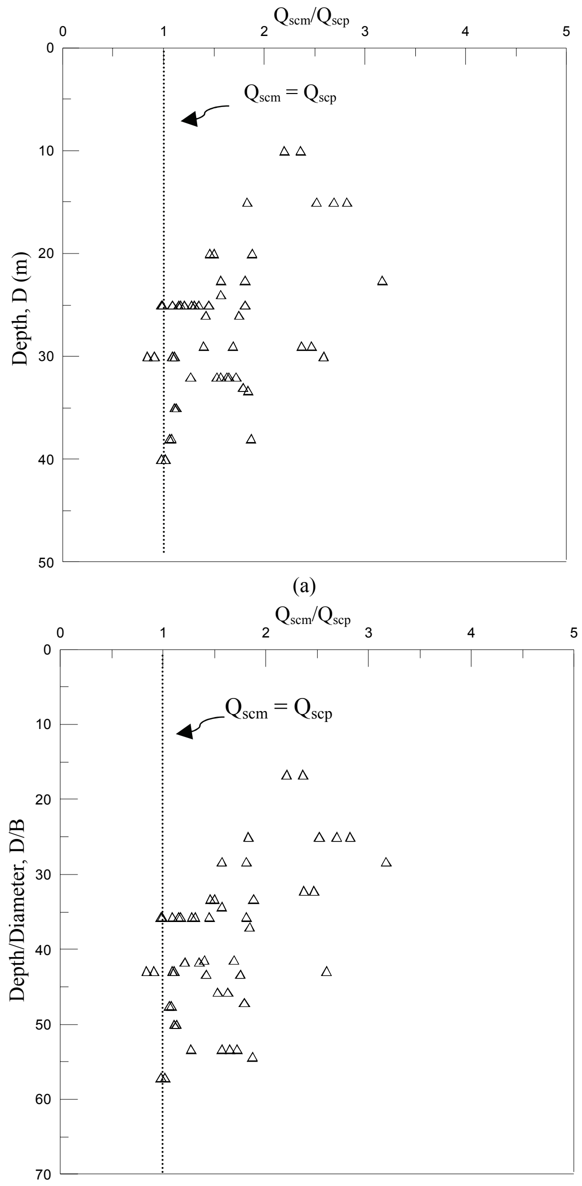

Fig. 5 Q_{scm}/Q_{scp} versus (a) depth and (b) depth/diameter

2. The bearing capacity equation parameters such as N_q or $\bar{q}$ should be improved if the predicted and measured results are to be compared and to be suitably applied for pre-bored PC piles.

3. The mean ratio of measured to predicted side resistance is between 1.39-1.60. The effective stress β method underestimated the side resistance for shorter shafts due to the underestimation of design parameters but can reasonably be applied for longer shafts.

4. A stress factor K/K_o of 1.65 is developed and proposed for pre-bored PC piles side resistance analysis under drained loading condition.

ACKNOWLEDGEMENT

This study was supported by the National Science Council of Taiwan, under contract number: NSC 100-2221-E-033-073-MY3.

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