Prediction of tip and shaft capacities of driven closed end pile in sand using CPT results

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Dutch Cone Penetration Test (CPT) is one of the most important field tests used to predict the ultimate capacity of driven piles. In this paper a comprehensive database for load tests on driven piles is used to examine the relationship between (CPT), end resistance value q_c and ultimate base resistance, q_b, in both D/10 settlement of pile head and plunging failure, where D is pile diameter. Also using (CPT) results to evaluate shaft capacity in both tension and compression loadings. In this study the pile capacities determined using three different methods were compared with a new suggested method. Appling the new suggested method for prediction of driven pile capacity shows: (1) the q_b (predicted)/q_b (measured) is 1.10 in the mean value for D/10 failure while q_b (predicted)/q_b (measured) for plugging failure is almost 1.02; (2) there is no trend of q_b/q_c with absolute pile diameter; and (3) the shaft capacity in tension is lower than compression loading by about 20 %.

يعد تقدير تحمل خوازيق الدق مغلقة القاع بأستخدام نتائج تجارب المخروط الهولندي الاستاتيكي واحدة من أهم أهتمامات باحثي ميكانيكا التربة منذ مايقرب من ٥٠ عام لكن معظم هذه الأبحاث تعطي تقدير اما بالزيادة او النقص مقارنة بنتائج تجارب التحميل علي هذة الخوازيق. في هذا البحث تم محاولة إيجاد صيغة مستحدثة لتقدير حمل أرتكاز الخازوق و ذلك بأستخدام المتوسط الحسابي لمقاومة الاختراق للمخروط القياسي و ذلك للمتوسط الحسابي للمقاومة لمسافة تعادل ضعفي قطر الخازوق أعلي منسوب أرتكاز الخازوق و القيمة الدنيا للمقاومة في مسافة تقدر بضعفي قطر الخازوق أسفل منسوب أرتكاز الخازوق. هذة الصيغة أعطت تقارب نسبي بين القيمة التقديرية و القيمة الفعلية المحسوبة من تجارب التحميل. و قد أعطت هذة الصيغة النسبة بين قدرة الارتكاز المحسوبة/القدرة المقاسة =١,١٠ وذلك لهبوط كلي للخازوق قدرة ١٠% من قطر الخازوق. كما اعطت هذة الصيغة النسبة بين قدرة الارتكاز المحسوبة/القدرة المقاسة =١,٠٢ وذلك عند الأنهيار الكامل للخازوق. أظهرت هذة الدراسة أيضا أنة لا توجد علاقة تربط بين *النسبة* q_b/q_c و قطر الخازوق. من هذة الدراسة أيضا تم أستنتاج أن مقاومة الأحتكاك الجانبي للخوازيق في الشد أقل منها في الضغط بحوالي ٢٠ %. كما أعطت الدراسة أيضا معاملات يمكن أستخدامها لتقدير قدرة تحمل الخوازيق في كل من الشد و الضغط.

Keywords: Shaft capacity, Tip resistance, driven pile, CPT

1. Introduction

Based on the method of installation, piles are classified as either driven or bored. The strong direct relationship between the end bearing resistance of closed end driven pile and the CPT end resistance, q_c has been recognized for many years and still arises because the similarity between their penetration process. The main problem of evaluating the tip pile load and also the tension capacity of the shaft using static cone penetration test results, q_c is how the tip resistance affected by the penetration into the layer under investigation. A number of alternative methods exist to predict the tip resistance, q_b of the driven pile in sand based on the results of cone penetration tests, CPT. The geometric similarity of piles and CPT instrument suggests that during steady penetration (or at the plugging load maintained load test), q_b should equal q_c as predicted by continuum analysis methods such as cavity expansion solutions Randolph et al. 1994 and strain path method Baligh, 1985. However a number of authors have suggested that reduction factors should be applied to cone resistance, q_c such that $q_b = a\, q_c$ where $a \leq 1$. This reduction factor can be attributed to the following items:-

1.1. Partial pile embedment through the bearing strata

Partial pile embedment through the bearing strata, L/D since the pile has a greater diameter than a CPT instrument, a deeper embedment from the ground surface, or into a hard layer, is required to mobilize the full strength of that layer. Prior to sufficient penetration, q_b will be less than q_c since the weak layer is still be felt by the pile tip eg. Meyerhof, 1976; Valsangkar and. Meyerhof, 1977. Also, since the L/D ratio of a CPT exceeds that of pile, the ratio of shaft to base area is higher, and hence of Q_s/Q_b. Analysis of the interaction between the shaft and tip resistance offers a mechanism by which the surcharge on the soil surrounding the base of a CPT is higher than that occur around the base of pile, leading to corresponding decrease in q_b / q_c Winterkorn and Fang 1975; Borghi et al. 2001.

1.2. Local in homogeneity

Kraft 1990, proposed that a reduction factor should be account for local inhomogeneities.

1.3. Absolute pile diameter

Jardine and Chow 1996 suggested a design method for offshore piles; they recommended a reduction factor based on pile diameter.

1.4. Partial mobilization

Lee and Salgado 1999 presented reduction factor on CPT resistance to account for partial mobilization of q_b by noting that the definition of q_b normally relates to a given settlement, rather than that plunging load required for continuous penetration.

1.5. Residual stress

After the final blow of installation the pile head rebounds. A large displacement is required to unload the pile base than to reverse the shaft friction. Therefore, when the pile head reaches a state of equilibrium with zero loads, the lower part of the pile remains in compression. A proportion of the base load is locked in and balanced by negative shaft friction. Chow 1996 showed that approximately 50 % of the ultimate base capacity is presented as a residual stress. Because it is practically impossible to evaluate the residual loads Lee, J. et al. (2003) suggested using design values of base and shaft capacity without any corrections.

2. Geotechnical site characterization

Due to the complex pile soil interaction anticipated in the series of testes, a comprehensive geotechnical investigation program was carried out to accurately define the soil profile at the test region. The site is located at Naphtha Refinery, Alexandria, Egypt. This investigation consisted of conventional sampling and laboratory testing as well as in-situ testing. In-situ tests included Standard Penetration Testing SPT, Vane shear Testing VST, and Dutch Cone Testing CPT. Laboratory testing performed on the field samples determined particles size distribution, Atterberg limits, shear strength, and consolidation characteristics. Geology and ground conditions at the site are presented in fig. 1. The soil condition within the explored depth, which is 20.0 m consists of six different layers. The top layer is engineering fill, comprising brown fine to medium dense sandy extends to a depth 3.0 m overlies a black greenish very soft silty clay with shell fragments of 7.25 m thick. The third layer is 4.6 m of brownish to grey fine to medium dense very thin layers of cemented sand, the fourth layer is brownish medium to coarse sand stone its thickness is 1.6 m underlain by green stiff clay with thickness of 1.60 m followed by brownish sand stone extended to the end of boring. The ground water level is near the ground level. Cone Penetration Test CPT Tests have been made close to the location of the boring; the generated chart is presented in fig. 2.

3. Prediction of unit base resistance

In this paper we are dealing with deeply embedment piles where as the pile is driven to a depth greater than or equals 8 pile diameter through the bearing strata, Meyerhof 1976 and Meyerhof et al. 1977. Also, the residual load has not been corrected, because this is practically impossible to do in practice. We

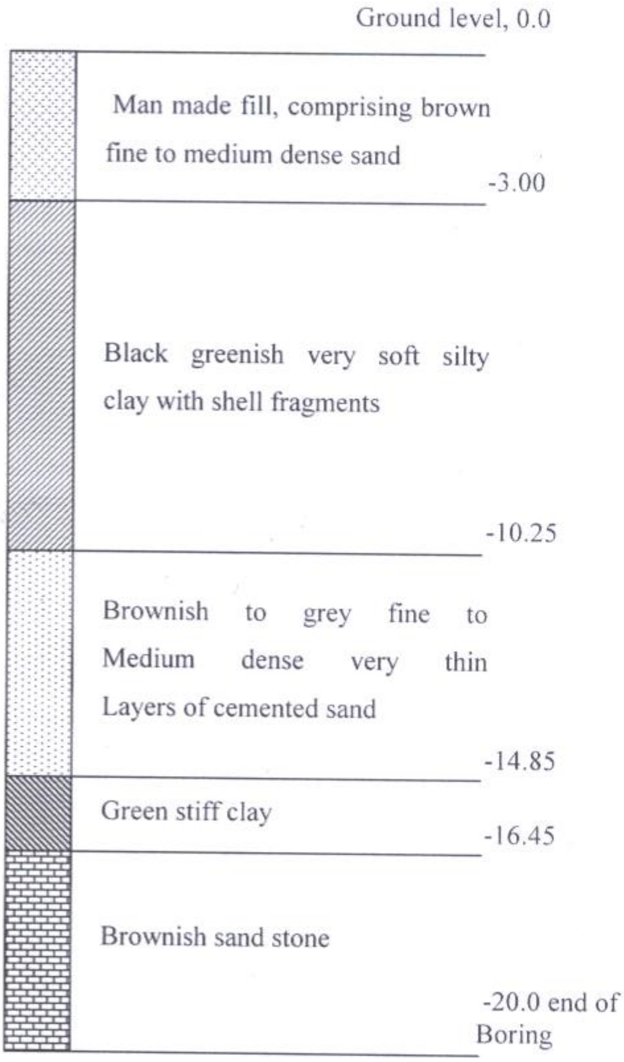

Fig. 1. Soil profile at the site.

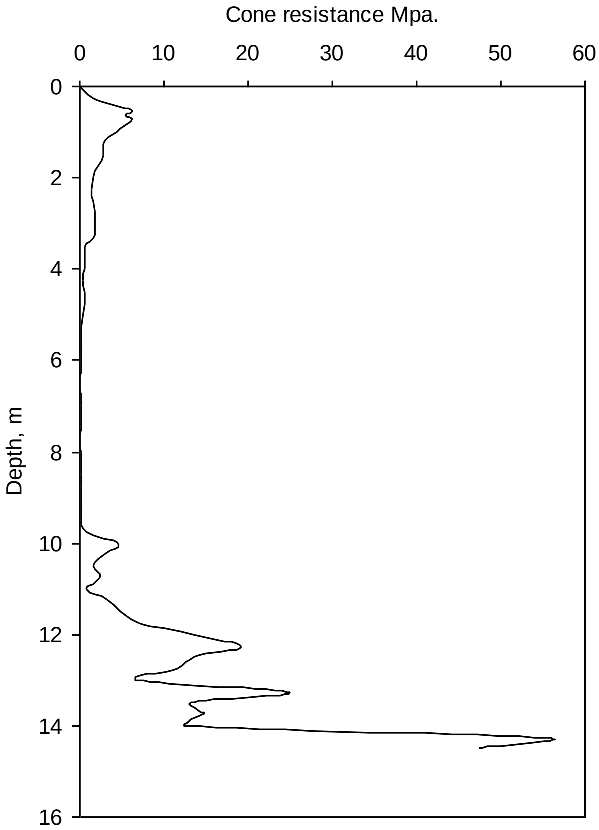

Fig. 2. CPT results for the site.

base our suggested design values of shaft and base resistance on the value measured without any correction for residual loads, as proposed by Lee et al. 2003. Unit base resistance, q_b, has been evaluated according to two failure modes: D/10 pile head settl-ement, and plunging failure. Plunging capacity is clearly defined in some tests, at which a constant penetration resistance is reached, and the maximum applied load has been chosen. This represents an under estimation, which in most cases is only by a few percent if compared to an extrapolated curve. In this study we suggest an alternative method to estimate the tip resistance using CPT results. In this method we consider that, q_b, is the average value of, q_I, and q_{II} where, q_I is the average of cone resistance over a height of two pile diameters above the level of the pile toe and, q_{II} is the minimum value of cone resistance below the point of the pile tip within a depth of two pile diameters. Some of reported methods of evaluating the tip resistance using the CPT results are considered in this study to compare the obtained results using the suggested method and the other methods. Chow et al. 1996 by averaging CPT over 1.5 pile diameters above and below the pile tip, Van der Veen et al. 1957, suggested that , the average cone resistance in sand is taken over a distance of 3.75 B above and 1.0 B below the point level where B is the pile diameter, Sanglerat 1972, proposed that the value of point resistance may be calculated as $q_b = 0.5\ (q_I + q_{II})$ where q_I is the average cone resistance over a depth of 8 pile diameters above the largest section of the pile base, q_{II} is the average of cone resistance below the point of pile tip for a depth of 3.5 diameters and the presented method is considered to predict the pile tip resistance. The predicted tip capacity of pile using different methods and measured tip resistance using the two modes of failure are presented in table 1 and figs. 3 and 4. As can be seen from these Figures it is clearly observed that the proposed method to predict the tip pile resistance is well scattered around a line of $q_{predicted} = q_{measured}$ and the best relationships can be evaluated from these figures are presented in table 2. This table indicates that the D/10 settlement tip resistance is considered 10 % lower than q_c which is attributed to partially mobilized of tip resistance. Where as the plugging capacity is considered as q_c thus is due to totally mobilized of q_b.

Table 1
Predicted and measured values of tip pile resistance

Source	Pile dimensions, m		Predicted tip resistance, Mpa.	Measured tip capacity, Mpa.		Notes
	Length	Diameter		D/10 Failure	Plugging failure	
Present work	13.20	0.356	14.50	14.56	16.07	13.2 m pile tension test: Qt = 245.25 kN compression test: Q = 1667.7 kN maximum applied load = 1814.85 kN 13.7 m pile tension test: Qt = 294.3 kN compression test Q = 2256.3 kN maximum applied load = 2256.3 kN @ settlement 23 mm = 0.065 D
	13.70	0.356	16.30	20.1	20.1	
Lee and Salgado (18)	6.87	0.356	11.35	8.70	10.75	
Chow (8)	5.96	0.1016	13.81	10.85	10.85	Q_b not fully mobilized
	7.40	0.1016	15.10	11.85	11.85	
Altaee et al.(1), (2)	11.00	0.285	5.55	5.35	6.21	
	15.00	0.285	7.50	7.29	7.52	
Briaud et al. (7)	7.78	0.273	6.20	4.94	Notes	
Alekander , S. Vesic (3)	8.86	45.72	16.1	12.89		
	13.12	45.72	17.4	13.12		
Gergersen et al. (11)	8.00	0.28	2.80	2.61	2.85	8 m pile tension test: Qt = 92kN compression test: Q = 253 kN maximum applied load = 267 kN 16 m pile tension test: Qt = 240kN compression test Q = 451 kN maximum applied load = 462 kN
	16.00	0.28	4.70	3.43	3.61	
Yen et al. (26)	34.25	0.609	8.00	2.92	2.92	Tip of pile is located within clayey layer and tested to 2.5% D settlement
Lahane (16)	1.80	0.1016	6.10	4.30	4.30	Plugging and D/10 failures are not reached
	5.96	0.1016	4.47	4.70	4.70	

Table 2
Statistical summery for ratio of predicted/measured tip resistance of pile concluded from the current work and previous studies (R^2 is regression coefficient)

Source	D/10 mode of failure For tip resistance of pile		Plugging mode of failure For tip resistance of pile	
	$q_{predicted} / q_{measured}$	R^2	$q_{predicted} / q_{measured}$	R^2
Present study	1.10	0.82	1.02	0.83
Van der Veen, et al. (23)	1.135	0.71	1.05	0.76
Sanglerat (21)	1.32	0.76	1.18	0.75
Chow (8)	1.32	0.84	1.227	0.88

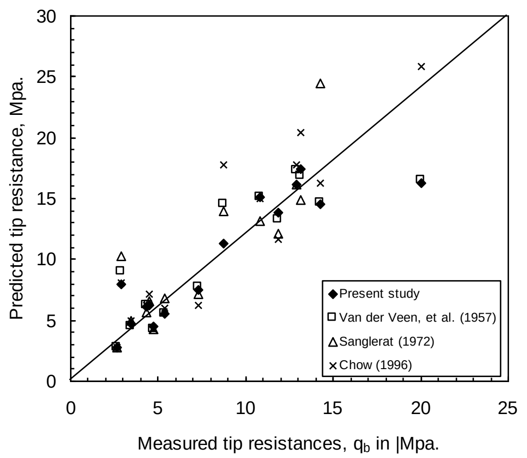

Fig. 3. Predicted versus measured tip resistances at D/10 settlement.

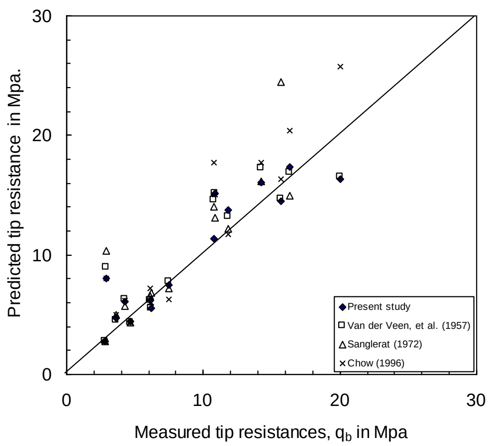

Fig. 4. Predicted versus measured tip resistances at plugging failure.

In order to investigate the scale effect on the tip pile resistance figs. 5, 6 are presented. The data introduced by Yen et al., 1989 is ignored because this was test pile located in a clay layer which is not captured in CPT profile as indicated by White 2003. These figures indicate the ratios of the measured to predicted tip resistance, q_b/q_c vs. pile diameters. It is clearly seen that no scale effect on q_b/q_c with absolute pile diameter is evident, this trend is reported by White 2003. However Jardine and Chow 1996 reported that the q_b/q_c ratio is inversely proportional with the pile diameter.

4. Prediction of shaft resistance

Local shaft friction (τ_f) is found to have a strong correlation with the CPT end resistance, q_c. This correlation which has been

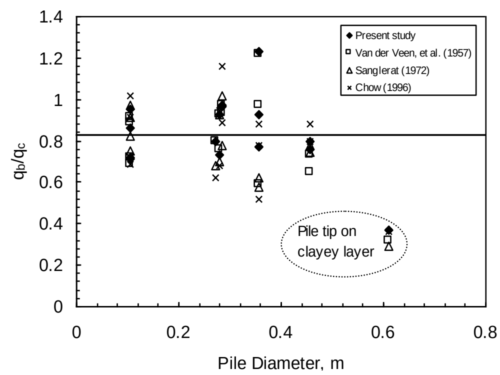

Fig. 5. Normalized pile base resistances versus pile diameter (failure D/10 settlement).

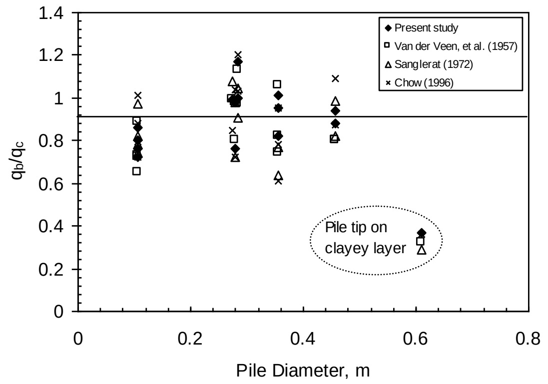

Fig. 6. Normalized pile base resistances versus pile diameter (plugging failure).

observed directly in instrumented field tests e.g. Lehane et al. 1993 has been employed successfully in well known design methods such as that reported by Bustamante and Gianiselli 1982 .The total shaft resistance may be evaluated using the simplified eq. (1)

$$Q_s = \pi D f \int_0^L q_c \, dL. \qquad (1)$$

Where Q_s is total shaft resistance, q_c is the tip cone resistance, f is the average normalized friction value, D is the pile diameter and L is the pile embedment length. The normalized shaft resistance was obtained using CPT sounding, and the results of calculating values of normalized friction using equation 1 in both tension and compression are presented in table 3. From this table it is clear that the normalized value in tension is slightly lower than that in compression loading by an average value about 20 %, this result agrees with the work done by Lehane et al. 1993, they observed that the tension capacity of the pile is 20 % less than the compression capacity, also De Ruiter et al. 1979 stated that the friction coefficient in tension is smaller than that in compression by about 25 %. Also Denicola and Randolph 1993 and Jardine et al. 2005 stated that the shaft friction that can be developed in tension is smaller than that which can be mobilized in a pile loaded in compression. Table 3 presents an average of the normalized value of 0.00555 and 0.00692 for tension and compression loading respectively which may be considered to estimate the maximum shaft capcity of tension and compression loading. These values are almost doubled the values suggested by De Ruiter et al. 1979.

Table 3
Normalized shaft resistances in tension and compression loadings

Source	Normalized shaft resistance in tension q_s/q_c	Normalized shaft resistance in compression q_s/q_c
Alekander , S. Vesic (3)		0.00733 0.00731 0.0061 0.0059 0.00645
Lee and Salgado (18)	0.0063	0.0078
Lahane et al. (16)	0.0071	0.0061 0.0084
Gergersen et al. (11)	0.0043 0.0051	
B.J. Jardine et. al. (13)	0.004	
Present work	0.00588	0.0067
Average value of f	0.0055	0.0069

5. Conclusions

1. In order to estimate the driven pile tip resistance, q_b, using CPT results, we can consider the average value of, q_I and q_{II} where, q_I is the average of cone resistance over a height of two pile diameters above the level of the pile base and, q_{II} is the minimum value of cone resistance below the point of the pile tip within a depth of two pile diameters.
2. There is no scale effect on q_b / q_c ratio with absolute pile diameter.
3. The normalized friction value in tension is slightly lower than that in compression loading by an average value about 20 %.
4. The normalized friction value of 0.0055 and 0.0069 for tension and compression loadings respectively may be used for evaluating the shaft resistance in tension and compression loading.

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