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THE DEPENDENCE OF BEARING VALUE ON DIAMETER OF DRIVEN STEEL PILES IN SAND

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مدى تأثير قطر الركائز المدقوقة في الرمل على قدرة تحملها

ملخص تم إجراء تجارب الضغط الرأسي على ركائز مفتوحة ومغلقة النهاية وذات أقطار مختلفة . بتحليل نتائج التجارب تم دراسة تأثير قطر الركائز وشكل نهايتها على قدرة التحمل. وبحساب قدرة التحمل باستخدام تسعة طرق ومقارنة النتائج المحسوبة مع نتائج التجارب أشارت المقارنة إلى أن قدرة الركائز المحسوبة أقل من قدرتها المحددة في التجارب كما أن قدرة تحمل الركائز مفتوحة النهاية تقل عن نظيرتها مغلقة النهاية كلما زاد قطر الركيزة .

Abstract **Axial compression tests were performed on open- and closed-ended piles with different diameters. The results of these tests have been analyzed considering the effects of pile diameter and pile end condition on the pile's bearing capacity. Nine methods to determine axial pile capacity based on the soil properties were compared with the measured results. The results obtained show that piles driven in dense sand can develop significantly higher bearing capacities than the calculated ones (e.g. API). In sand, open-ended piles typically develop lower capacities compared to closed-ended piles as the pile diameter increases.**

Introduction

The increasing use of piles of small diameter in underpinning historical buildings and foundations of new structures needs more understanding of the effects of the diameter size on the bearing capacity value of this type of piles in sand. The bearing capacity of piles in sandy soil has been under investigation for years. Many tests have been carried out with instrumented piles to measure the variation of axial pile load with depth (Lehane et al. 1993). However very little reliable data for the pile behavior in dense sand, and the effect of pile scale are available (Chow 1995). Lings (1997) has made an attempt to study the influence of pile diameter on shaft resistance. Design methods for axially loaded piles in sand based on theoretical and empirical approaches still show major variations; because of a large number of factors that affects the bearing capacity value.

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Also Abu Keifa (1998) reported that, the standard penetration test does not predict the actual behavior of soil. Meyerhof (1983) concluded, based on static cone and standard penetration test results that factors such as size of the piles, bearing layer thickness, and method of construction must be considered in estimating the ultimate bearing capacity of piles in sand. The unit weight and the shear parameters (internal friction angle and cohesion) cannot be measured directly during in situ investigation in cohesionless soils, therefore the empirical design methods are used in estimating the bearing capacity (Robert 1997).

Full-scale load tests on both open-ended and closed-ended 2 m long steel pipe piles with different diameters were driven in dense sand at Islamic University of Gaza Campus, as part of a research program.

Soil Conditions

The site is located close to the southwest corner of the university campus. Figure 1 shows the soil profile and the lightweight penetrometer test results (DIN 4094). These results show almost uniform sand conditions. This allows for decreasing the boundary factors effects in pile bearing capacity results. The upper 2.8 m consists of fine to medium sand, overlying clay layer with about 4 m thickness. A summary of soil properties is given in Table 1. The sand layer may be classified according to the Unified Classification System as poorly graded sand (SP).

Testing Equipment and Procedure

Ten steel piles were driven at the Islamic University of Gaza. Figure 2 shows a plan view of nine of the pile test locations. The eight traditional piles on the corner worked as reaction/anchor piles during the load tests. The reaction piles were 12 m deep. The 10 steel pipe piles were constructed as test piles with 2 m long; 5 of them are closed-ended pipe piles and the other 5 are open-ended pipe piles. Each pile was jacked into the sand for about 20 cm to control its verticality and then using the head of the drilling machine as a hammer facilitated a full length pile driving.

All pile load tests were performed in almost similar field conditions, using a high capacity hydraulic jack and dial gauges accurate up to 0.01mm to measure the axial movement of the pile head. The piles have an outer diameter ranging from 9.1 cm to 32 cm and are closed ended with a 45 degrees conical tip. The pile was manufactured using steel tubes. The piles were spaced at more than three-pile diameter in general.

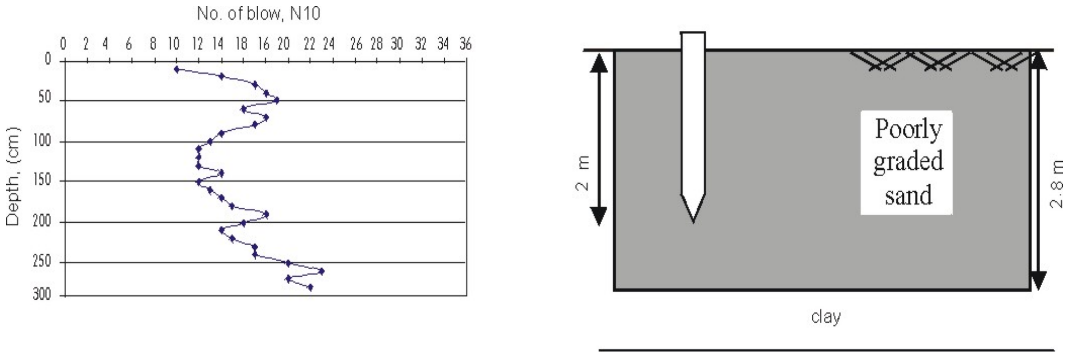

Figure 1: Typical light weight penetration test result and soil profile

Table 1: Soil properties

Sand layer		Clay layer	
Uniformity coefficient	1.83	Liquid limit	27.7%
Gradation coefficient	0.81	Plasticity index	10.9%
Natural water content	4.3%	Water content	17.19%
Bulk unit weight	17.7 kN/m^3	Cohesion C_u	35 kPa
Internal friction angle	34°	Unit weight	20.5 kN/m

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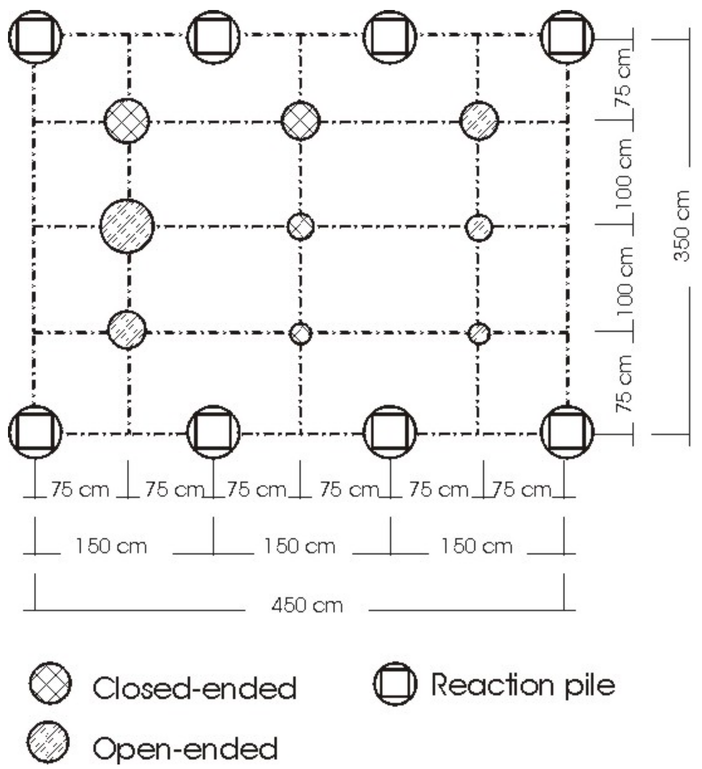
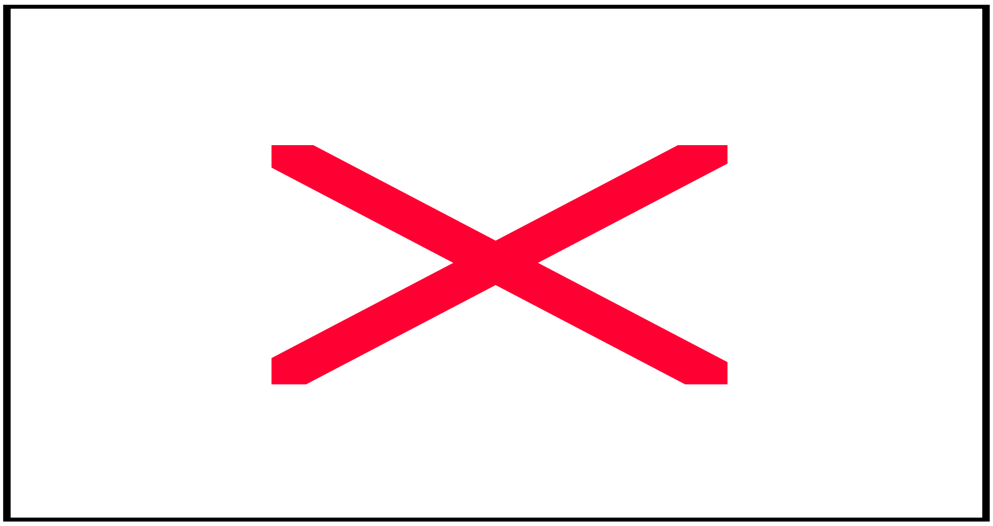


Figure 2: Pile test locations

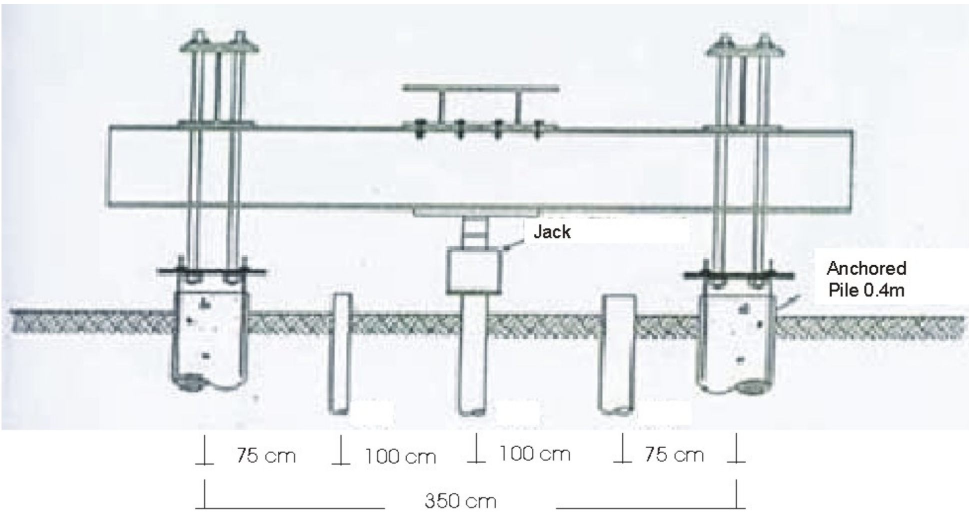
Tensile steel test was performed to estimate modulus of elasticity of pile material E=15GN/m^2 (150 t/cm^2). Shear box test was performed using steel plate (represents the pile material) in the upper half of the shear box and the sandy soil was placed in the lower half with the same field density. The interface friction angle between pile material and the sandy soil was found as 28 degrees.

Figure 3-a shows a photograph of the arrangement consisting of two parallel beams adjacent to each other connected with two perpendiculars I beams at either end, which are connected to the 4 anchored piles. This assembled steel frame is resting on two I-beams as supports. The hydraulic jack is located directly over the pile head and below the middle connected I beams. Figure 3-b illustrates the position of pile installations.

The tests were performed in relatively dry sand material. Compression loads were applied using the hydraulic jack. The pile movements were measured with three dial gauges located on a rigid plate (3 mm thick), which was positioned on the pile head. The gauges were



(a)



(b)

Figure 3: Pile test arrangements

connected to two reference beams. The hydraulic jack was controlled manually using a pump with a dial gauge readings taken by the jack operator. The procedures commonly used to perform load tests are in accordance with ASTMD 1143-81.

Loads were applied in equal increments of about 20 kN (2 tons), 10% of the expected maximum load, and maintained for a holding period of around 20 minutes. The movement rate was controlled not to exceed 0.0025 mm/min. Failure was decided when the pile movement increased and the maximum applied load cannot be reached.

Load-Settlement Behavior

Figure 4 shows the results of closed-ended pipe pile with 20.3 cm inner diameter test. The load-settlement curve indicated that the failure was reached. For the final load of (160 kN), the head settlement was (20.4 mm). It is important to note that the residual settlement was very high (19.03 mm). For the rest of the piles, a rather similar behavior was observed. Some of these piles are re-tested after about one month; it was observed that the maximum load was almost the same.

Diameter Response of Single Pile

A limited number of researches have been presented in the literature about the pile diameter effect on its bearing value. Meyerhof (1983) has compared the results of load tests of different pile sizes with their estimated ultimate bearing capacities in sand. In this study full-scale tests were performed and analyzed by means of nine methods for pile bearing capacity estimation.

Test Results

Table 2 summarizes pile loading test results. The results are separated into two groups according to the pile type. The failure load was defined as the maximum load reached. The test results for two types of steel pipe piles were plotted in Figure 5. An examination of this figure together with Table 2 shows that at a diameter less than 11.7 cm the measured pile load capacities were close to each other for open-ended and closed-ended piles.

The rate of increase of pile load capacity with the increase of its diameter is designated R and is defined as the difference in pile load capacity divided by the difference in outer diameter for two piles in a sequence. The calculated rate values are plotted in Figure 6. For the current test results, the rate increases with increase in pile diameter. It is interesting to note that, for open-ended pile at the first stage, the rate is not sensitive to change in pile diameter.

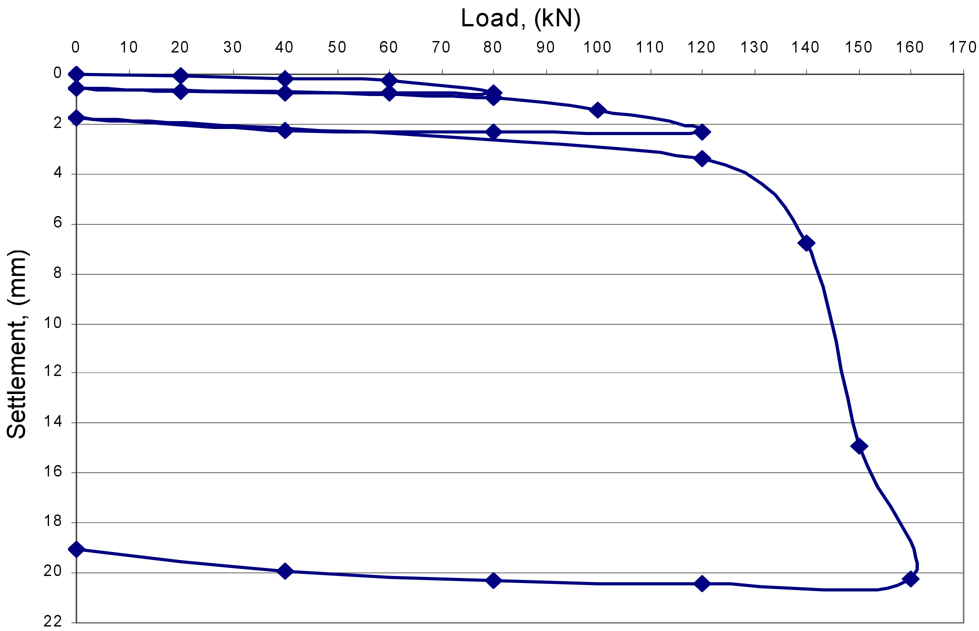


Figure 4: Settlement versus load for closed-ended pile

Table 2 presents also the percent difference between the open-ended and closed-ended pile load capacities, determined as the difference between the two load bearing values divided by the bearing value of open-ended pile of the same diameter. As the diameter increases the pile load capacity of closed-ended pile is higher than the open-ended pile with an average 26.5%. Alawneh et al. (1999) concluded that the closed-ended driven pipe piles showed approximately 24% increase in skin resistance compared with the open-ended piles.

Table 2: Test results and percent differences in pile load capacity

Pile diameter (cm)		Open-ended pile		Closed-ended pile		Difference %
Inner	Outer	Q_0 * (kN)	R (kN/cm)	Q_c * (kN)	R (kN/cm)	
7.6	9.1	40	4.4	40.5	4.4	1.25
10.2	11.7	52	4.7	56	6.1	7.69
15.2	17.3	80	4.9	104	8.5	30
20.3	22.3	130	10	160	11.2	23.07
30.5	32	210	8.3	257	10	22.4

Note:* Q_0, Q_c = Failure pile capacity for open-ended and closed-ended pile respectively.

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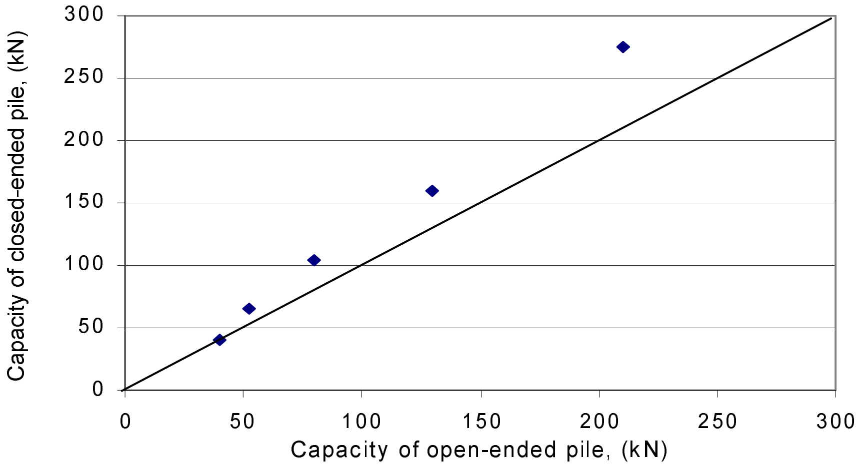

Figure 5: Measured closed-ended versus open-ended pile

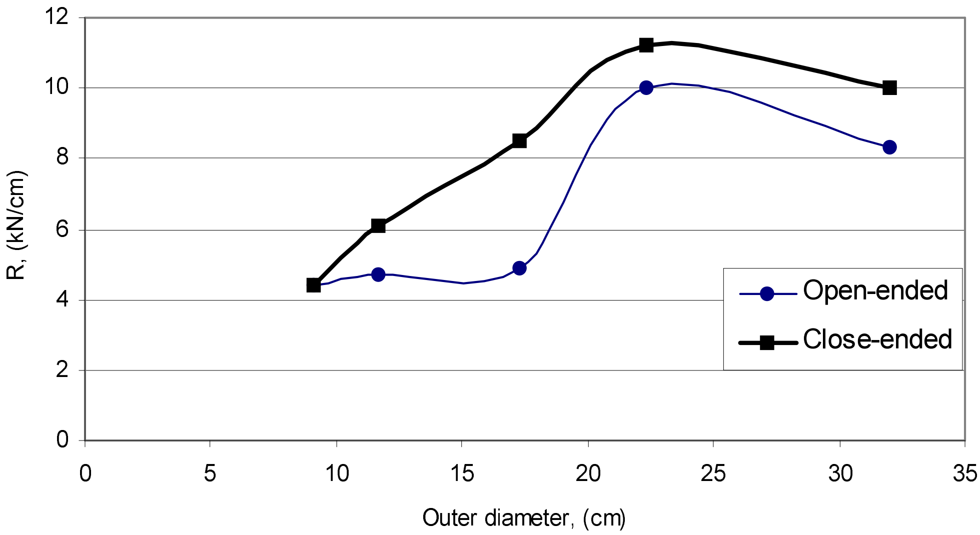

Figure 6: The rate of capacity increasing with respect to diameter

Theoretical Design Methods

Several methods are currently used in engineering design practice. In some of these methods field test data are used in determining bearing values (i.e. cone penetration test and standard penetration test). In others laboratory testing results are used to calculate the required parameter values, such as the earth pressure coefficient K, and the bearing capacity factor N_q. Mainly refer to the American Petroleum Institute (1993) procedure, American Society of Civil Engineers book adapted from U.S. Army Corps of Engineers (ASCE 1994), and Bowles (1997) textbook for detailed procedure of ultimate bearing capacity calculations. For pile foundation with ratio of pile embedded length to pile diameter greater than 5 the N_γ term in the bearing capacity formula is small compared with N_q term and can safely be neglected (Kulhawy 1984).

End-Bearing Capacity

The following is a presentation of the end bearing capacity computations of piles. For various theoretical approaches, the ultimate end bearing capacity for piles in sand may be computed neglecting pile weight and N_γ term using the following equation:

[1] $q_{pu} = q^{\backslash} N_q F_q$

Where

$q^{\backslash}$ = effective soil vertical overburden pressure at pile point $\approx L\gamma^{\backslash}$.

$\gamma^{\backslash}$ = effective unit weight of soil along the pile embedded length L.

N_q = pile bearing capacity factor of surcharge component.

F_q = correction factor of surcharge component.

Table 3 lists the calculated end-bearing capacity of piles by various methods for sandy soil. Foray et al. (1998) showed that a very short soil plug is sufficient to develop a full end bearing capacity. As was also shown by Murff (1990) the plug length with two to three diameters of a steel pipe pile can carry the end bearing capacity of the internal soil plug (quoted from Foray et al. 1998). A pile of 2 m long is considered and the end bearing has been calculated for piles of various diameters assuming a fully plugged (or closed-ended) pile. In the American Petroleum Institute’s (API) recommendations for open-ended pile, the total resistance includes the internal shaft friction or the end bearing of the plug, whichever is less.

Skin Friction Computations

The maximum skin friction resistance that may be mobilized along the pile length may be determined by:

[2] $Q_s = A_s f$

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Table 3: Values of end resistance for driven closed-ended piles in sand having ϕ =34°.

Method	Reference paper	Procedure source	Nq	End resistance (kN)				
				D*= 0.0912	0.1166	0.1732	0.2232	0.3198
Meyerhof	Meyerhof 1976	ASCE 1994	110	11.6	24.24	79.45	139	285.35
Hansen	Hansen 1970	ASCE 1993	29.44	15. 96	26.02	57.11	94.44	192.12
Vesic	Vesic 1975, 1977	Bowles	89.55	12 . 99	21.23	46.85	77.8	159.71
General shear	Bowles 1968	ASCE 1994	36.49	8.44	13.79	30.44	50.55	103.77
Janbu's	Janbu's 1976	Bowles 1993	41.91	9.7	15.8	36	58.1	119.2
Nordlund	Vanikar 1986	ASCE 1994	55	3.77	7.88	25.8	49.52	101.65
API	API 1993	API 1993	32	7.4	12.1	26.7	44.3	91
Alawneh	Alawneh 2001	Alawneh	133	30.76	50.3	110.93	184.2	378.2
			145.3**	33.6	54.9	121.2	201.3	413.2

Note:*D= Diameter of pile in m,** Considering residual loads

where
f = unit frictional resistance
A_s = area of pile surface

The following methods are used for the calculation of unit frictional resistance of piles in sand: i-The American Petroleum Institute (API 1993), ii-Nordlund, iii-Beta (β) and iv- Alawneh. The recommendations of API (1993) for pile design in sand are used as follows:

[3] $$f = KP^{\backslash} \tan\delta$$

where
K = dimensionless coefficient of lateral earth pressure
= 1 for close-ended pile
= 0.8 for open-ended pile
$P^{\backslash}$ = the average effective overburden pressure along the pile
δ = friction angle between the soil and pile wall.

When calculating the unit frictional resistance (*f*) with increasing embedded depth, a limiting value $f_{\lim}$ is applied, which depends on the relative density of sand. Limiting values for *f* and ultimate end bearing capacity q_{pu} are specified with API. Typical values of limiting capacities of $f_{\lim}$ range from 47.8 to 67 kPa for loose sand and from 81 to 115 kPa for dense to very dense sand. Similar limiting end bearing values lie in the range 1900-2900 kPa for loose sand to 4800-12000 kPa for dense sand.

Table 4 lists the calculated skin friction capacity of the closed-ended pile. Comparison of methods show that skin friction varies from 4.38 kPa to 42.68 kPa for the used sand. The ultimate pile capacity is computed by adding the skin friction resistance Q_s (unit skin friction *f* integrated over the embedded surface area of the pile) to the end resistance Q_p. Therefore, the total bearing capacity is:

[4] $Q_u = Q_p + Q_s$

The minimum and maximum values of q_{pu} and f calculated above could be used to obtain a range of Q_u .

Table 4:Skin friction resistance calculation of driven closed-ended piles in sand having $\phi = 34^o$.

Method	Reference paper	Procedure source	Skin resistance (kN)				
			D= 0.091	0.117	0.173	0.223	0.32
Beta (β)	Meyerhof 1976, Poulous 1980	ASCE 1994	5.55	9.07	20.01	29.79	42.68
Nordlund	Vanikor 1986	ASCE 1994	4.38	6.72	9.56	9.94	10.93
API	API 1993	API 1993	5.39	6.89	10.24	13.2	18.91
Alawneh	Alawneh 2001	Alawneh 2001	7.3	9.34	13.9	17.9	25.6
			4.34*	5.6	8.2	10.62	15.2

Note: D = outer diameter of pile in (m),* Considering residual loads

Comments on Bearing Capacity Computations

The design methods described above were used to predict the bearing capacity of the test closed-ended piles included in the experimental program. It is obvious that there were a variation in the value of N_q determined by different methods, which affects the end-bearing value; this agrees with Coyle and Castello (1981) and Coduto (1994) (quoted from Robert 1997). On the other hand, comparison of methods indicates that Hansen, API and Vesic methods take account of a limiting effective stress while the general shear method and Janbu method ignore this stress. The critical depth applies to the Meyerhof and Nordlund methods for analysis of bearing capacity. It should be mentioned that Karft (1991) concluded that the critical depth concept does not exist, which is not in agreement with Meyerhof and others. The calculations indicate a significant difference between these methods.

Figure 7 shows the calculated pile resistance (capacity) with respect to the measured ultimate pile capacity of each test pile. Comparison of calculated and measured results shows that the measured pile capacity is higher than the calculated for most of the current design methods. Foray et al. (1998) showed that piles driven in dense sand can develop significantly higher bearing capacities than the calculated with API method. Figure 8 shows the influence of the pile diameter on the prediction of skin resistance to the toe resistance ratio. On the average, the ratio of calculated skin to toe pile capacity decreases with the increase in pile diameter. Obviously, the current methods show significant variations in the computation of bearing

capacity values, which reflect the variations of the assumed shape of the shear pattern and its extent above and below the pile tip and the variation in the limit of skin stress parameters.
Figure 9 shows the influence of pile diameter on the calculated to the measured pile capacity of each test pile. In general, the results of most design methods for piles in sand have highlighted the poor reliability of these design approaches; which agreed with Briaud and Tucker (1988) and Lings (1997). The ratio of calculated to measured resistance is, in general, less than 1 for small pile diameters which shows under-estimation of pile capacity. But the ratio increases with the increase in pile diameter. Lings (1997) studied the pile diameter influence on pile skin resistance and found that at very small pile diameter the calculated skin resistance is under-estimated. For this study the general trend of diameter influence on pile bearing capacity agrees with Lings (1997).

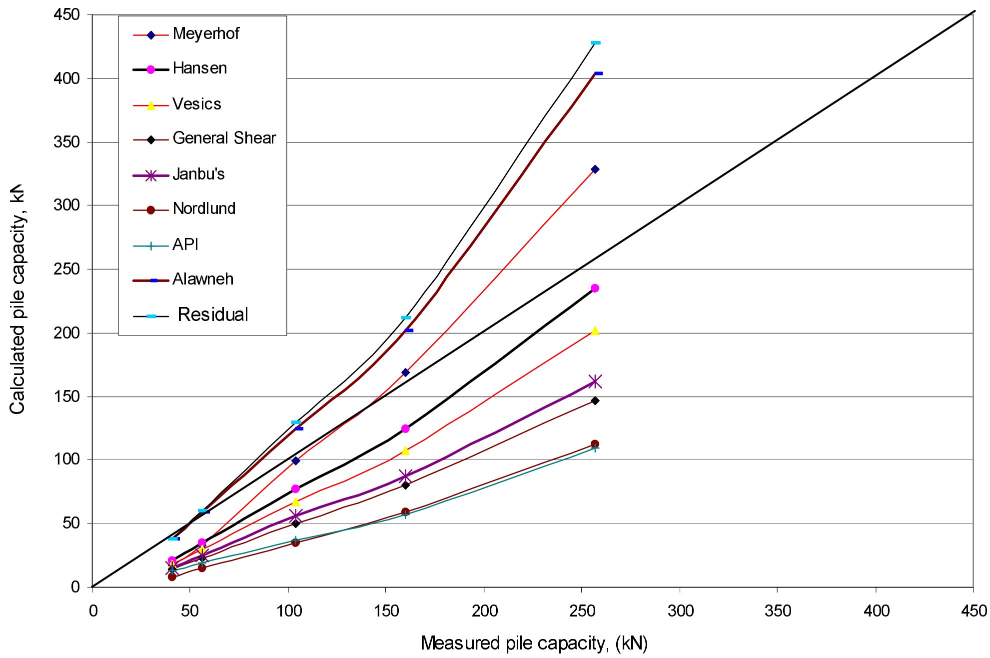


Figure 7: Measured versus calculated pile capacity by different methods

Evaluation of Displaced Volume

In order to study the factors that may affect the pile bearing capacity, the volume of soil displaced per unit length is calculated for open- and closed-ended piles. The influence of soil displacement on total pile capacity is shown in Figure 10. It should be noted that the rate of increase of closed-ended pile capacity decreases with the increase in the volume of displaced soil. For the open-ended piles the rate of increase in pile bearing capacity with the increase in the volume of displaced soil is small, but rapidly increases when the pile diameter becomes equal to or greater than 22.3cm.

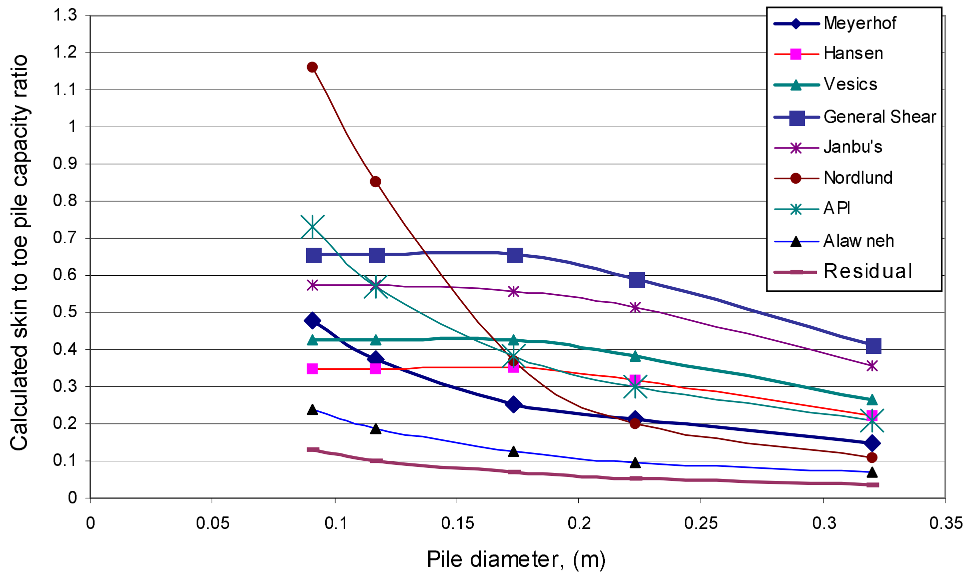


Figure 8: Comparison of the calculated skin to toe capacity with respect to closed-ended pile diameter

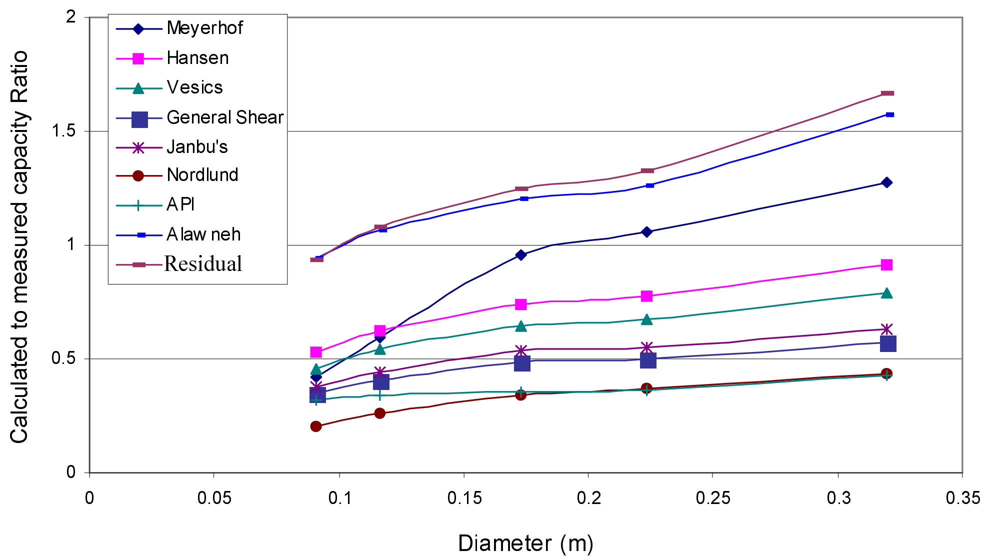
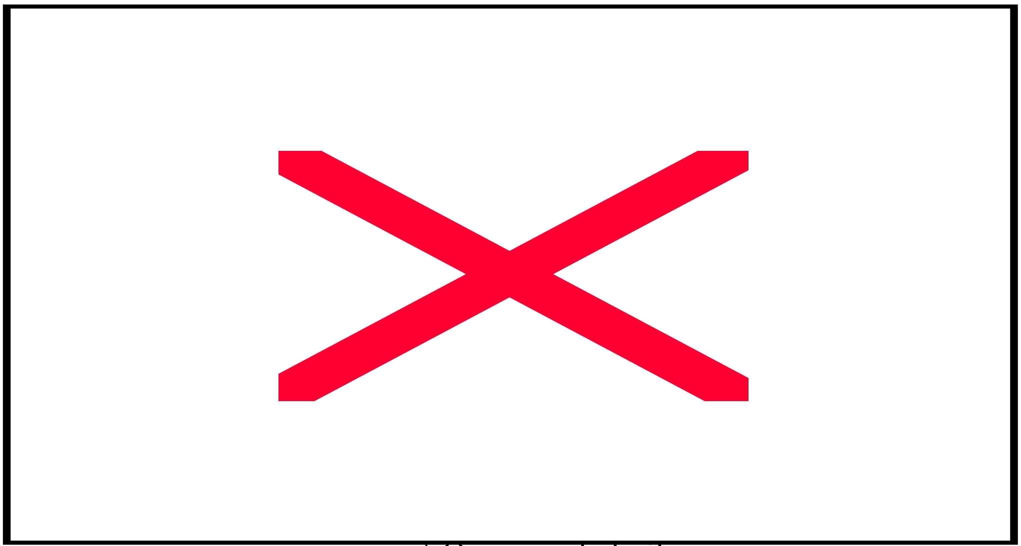
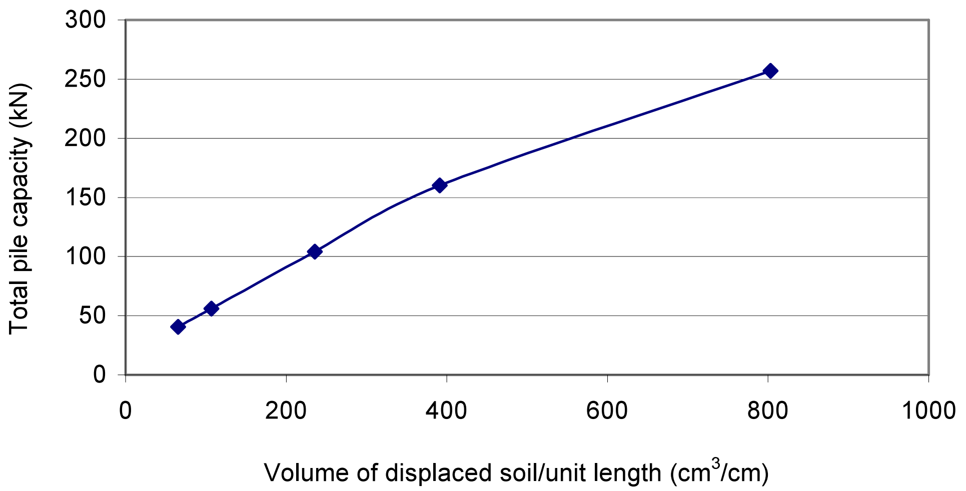


Figure 9: Influence of closed-ended pile diameter on calculated to measured pile capacity ratio

The Dependence of Bearing Value on Diameter



a) Open- ended pile



b) Closed- ended pile

Figure 10: Measured total pile capacity versus the soil displacement volume

Pile Diameter

The driving of a closed-ended pile is usually associated with moving of larger amount of sand, which causes additional densification of the sand in comparison to an open-ended pile. Therefore, it is reasonable to expect that by additional densification axial stress will be increased and will lead to a corresponding increase in the lateral stress and for this reason the skin friction stress at the pile-soil interface is expected to be increased. While open-ended piles cause less densification and bring about small changes of lateral stresses. The unit skin friction capacity of a pile may be influenced by the lateral (radial) effective stress at the pile-soil interface (Foray et al. 1998, Chow 1995, and Randolph et al. 1991). Lehane et al. (1993) showed that the lateral effective stress acting on the pile shaft is comprised of two components. These are the lateral stress after installation and before loading and the additional component drived from sand dilation during loading.

For sands, Meyerhof (1983) concluded that ultimate end bearing capacity tends to be less for larger diameter driven piles, and indicated that ultimate unit skin friction is practically independent of pile diameter. The reduction in end bearing capacity has been related with a reduction of the effective angle of internal fraction $\phi^{\backslash}$ with larger diameter. However, Hettler (1982) concluded that the unit skin resistance of model tension piles in sand decreases as the pile diameter increases (quoted from Kraft 1991). Kraft (1991) mentioned that the relative influence of pile diameter decreased as the pile diameter increased. In this study, it was found that the rate of increase of the pile capacity decreases with greater pile diameter, and for a given pile diameter the rate of increase for the closed-ended piles is larger than that for open-ended piles.

Open-Ended Pile

Open-ended pipe piles are often driven into sandy soil to reduce driving resistance. The influential factors for the plug formation are pile diameter, sand density, embedded length, pile wall thickness, and installation procedures. Tests were performed on driven, open-ended and closed-ended steel pipe piles in sandy soil, in which the same soil conditions are used. Results of this study show that the pile diameter strongly influenced the plug behavior where the pile capacities of smaller diameter (less than 11.7 cm) have almost the same capacity of closed-ended pile. This means that fully plugged pile has stiffer response to loading in smaller diameter piles than in the larger diameter piles. The driven open-ended pile capacity is generally less than the similar closed-ended pile capacity. Lehane and Gavin (2001) shows that for partially plugged pile the plug stiffness is less than that of an equivalent closed-ended pile. Paikowsky

(1990) indicated that most open-ended piles is filling up with soil during driving, but it fails as closed-ended piles during static loading (quoted from Randolph et al. (1991). Randolph et al. (1991) pointed that, arching action occurs within the pipe pile which leads to high internal friction. Most researchers who studied plug load transfer mechanism of piles in sands agree that the open-ended piles behave as closed-ended piles when loaded statically (Karft 1991,Randolph et al. 1991,O'Neill et al. 1991, Raines et al. 1992).

Conclusions

- There is no clear evidence of maximum ' limiting' values of pile bearing capacity, in the range of pile diameter investigated. However, there was an increase in bearing capacity with the increase in pile diameter.
- The rate of increase in pile capacity decreases with the increase in pile diameter.
- The measured pile capacity is higher than the calculated one with the mostly used current design methods, i.e. API.
- The smaller diameter of open-ended piles exhibited a similar bearing capacity as the closed-ended pile with the same diameter.
- For relatively larger diameter of piles the capacity of closed-ended piles is larger than that of open-ended piles with the same diameter.

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