TECHNICAL PAPER

JOURNAL OF THE SOUTH AFRICAN INSTITUTION OF CIVIL ENGINEERING
ISSN 1021-2019
Vol 60 No 1, March 2018, Pages 44–55, Paper 1410

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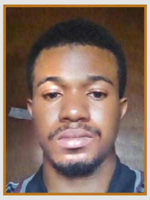

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Keywords: comparative analysis, cone penetration test, pile capacity, univariate analysis of variance, post-hoc tests

Comparative analysis of methods of pile-bearing capacity evaluation using CPT logs from tropical soils

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This study presents the results of a comparative analysis of the performance of eight direct cone penetration test (CPT) methods in predicting the ultimate load-carrying capacity of a 300 mm diameter end-bearing pile, evaluated at defined soil depths using CPT logs obtained for various locations in the south-east and south-south regions of Nigeria. The methods used are the Schmertmann (1978), LCPC (Bustamante & Gianeselli 1982 – Laboratoire Central des Ponts et Chausees), De Ruiter and Beringen (1979), Tumay and Fakhroo (1982), Price and Wardle (1982), Philipponnat (1980), Aoki and De Alencar (1975), and the Penpile methods (Clisby *et al* 1978). The results of univariate analysis of variance indicated that the interactions between location and treatment (methods of pile capacity prediction), depth and treatment, and location and depth were statistically significant for cohesionless soils, but that the interaction between depth and treatment was not statistically significant for cohesive soils. Also, post-hoc tests (Least square difference and Bonferroni methods) showed that the LCPC and Philipponnat methods are best suited for cohesionless soils, while the LCPC, Tumay and Fakhroo (1982) and De Ruiter and Beringen (1979) methods are recommended for cohesive soils.

INTRODUCTION

One of the main steps for the safe and economic design of foundations is the determination of ultimate bearing capacity. The four approaches generally used for bearing capacity determination are static analysis, in-situ testing methods, full-scale loading tests and using presumed values recommended by codes and handbooks. Among these, the in-situ methods have in recent years become more popular, and this is attributed to the rapid development of in-situ test methods, testing instruments, improved understanding of soil behaviour and the subsequent insight into the limitations and inadequacies of some conventional laboratory testing methods (Eslami & Gholami 2006; Shooshpaasha *et al* 2013). Among the in-situ methods, the CPT method is simple, fast and relatively economical, and supplies continuous records with depth. Its results are interpretable on both an empirical and analytical basis, and a variety of sensors can be incorporated in its use. When compared to the standard penetration test (SPT) method, the CPT method is more popular owing to the many problems and limitations associated with the performance and interpretation of the results obtained from the SPT (Eslami & Gholami 2006). Such problems and limitations associated with the SPT methods are due to the fact that it does not provide continuous data, and therefore important information on weak seams may be missed. It has limited applicability to cohesive soils, gravels, cobbles and boulders. Its progress is also slower than other in-place tests, because of incremental drilling, testing and sample retrieval.

In this regard, the authors are seeking to appraise the performance of the eight generally adopted methods of estimating pile capacity from CPT logs on tropical soils of Group B subgroup (b) and Group C subgroups (b) and (c) (see Table 1). This is aimed at finding out if some or all of these eight methods are applicable to tropical soils since they were originally developed for soils in temperate regions, but are often used for tropical soils as there are no documented methods solely developed for tropical soils.

BACKGROUND

Tropical and temperate soils

Tropical soils vary in type and composition, as is the case with soils in the temperate regions of the world, but tropical soils possess certain peculiarities for which their

Obeta IN, Onyia ME, Obiekwe DA. Comparative analysis of methods of pile-bearing capacity evaluation using CPT logs from tropical soils. J. S. Afr. Inst. Civ. Eng. 2018:60(1), Art. #1410, 12 pages. http://dx.doi.org/10.17159/2309-8775/2018/v60n1a5

Table 1 Classification of residual soils

Group: Major group	Group: Sub-group	Common pedagogical names	Examples	Means of identification	Comments on likely engineering properties and behaviour
Group A Soils with a strong mineralogical influence	(a) Strong macro-structure influence	Kaolinitic saprolites	Highly weathered rocks from acidic or intermediate igneous or sedimentary rocks	Visual inspection	This is a very large group of soils (including the saprolites) where behaviour (especially in slopes) is dominated by the influence of discontinuities, fissures, etc.
	(b) Strong micro-structure influence	Oxisols	Completely weathered rock formed from igneous or sedimentary rocks	Visual inspection and evaluation of sensitivity, liquidity index, etc	These soils are essentially homogenous and form a tidy group much more amenable to rigorous analysis than group (a) above. Identification of nature and role of bonding (from relic primary bonds to weak secondary bonds) important to understanding behaviour.
	(c) Little or no structural influence		Soils formed from very homogenous rocks	Little or no sensitivity, uniform appearance	This is a relatively minor sub-group. Likely to behave similarly to moderately over-consolidated soils.
Group B Soils strongly influenced by normal clay minerals	(a) Smectite (montmorillonite) group	■ Black cotton soils ■ Black soils ■ Tropical black earths ■ Grumusols ■ Vertisols	Black cotton soils and many similar dark-coloured soils formed in poorly drained conditions	Dark colour (grey to black) and high plasticity suggest soils of this group	These are normally problem soils found in flat and low-lying areas, having low strength, high compressibility, and high swelling and shrinkage characteristics.
	(b) Other clay minerals				Likely to be a very minor sub-group.
Group C Soils strongly influenced by clay minerals essentially found only in residual soils	(a) Allophane sub-group	■ Volcanic ash soils ■ Andosols/ andisols ■ Andepts	Soils weathered from volcanic ash in the wet tropics and temperate climates	Position on plasticity chart and irreversible changes on drying	Characterised by very high natural water content and Atterberg's limits. Engineering properties are generally good, though in some cases high sensitivity may make earthworks difficult.
	(b) Halloysite sub-group	■ Tropical red clays ■ Latosols ■ Oxisols ■ Feralsols	Soils often derived from volcanic material, especially tropical red clays	Reddish colour, well drained topography and volcanic origin are useful indicators	These are generally fine-grained soils of low to medium plasticity and low activity. Engineering properties are generally good (there is often some overlap between halloysite and allophane clays).
	(c) Sesquioxides – gibbsite, goethite, haematite	■ Lateritic soils ■ Laterites ■ Ferralitic soils ■ Duricrusts	Laterites or possibly some red clays referred to as lateritic clays	Non-plastic or low plasticity materials, generally of granular or nodular appearance	This is a very wide, poorly defined group ranging from silty clay to coarse sand and gravel. Behaviour ranges from low plasticity silty clay to gravel. These materials are the end products of a very large weathering process.

Source Wesley (2009)

location is a contributing factor. They are located between the Tropic of Cancer and the Tropic of Capricorn (Punke 2014), with a large variety of them being residual soils. They are residual as a result of intense chemical weathering initiated by climatic circumstances that break down clay minerals into hydrated oxides of aluminium and iron, whereas clay is fairly stable in temperate regions. Also, in the warm and humid climates typical of the tropics, the time required to chemically alter a rock material is considerably less than in temperate climates (Morin & Todor 1976). Residual soils are different from the sedimentary soils of the temperate regions which are formed through a depositional process wherein soils are deposited in a marine/lake environment. Two important factors that lead to a degree of homogeneity and predictability of sedimentary soils that are absent from residual soils are:

- The sorting process that takes place during erosion, transportation and deposition of sedimentary soils, which tend to produce homogeneous deposits, and
- The stress history, which is a prominent factor in determining the behavioural characteristics of sedimentary soils, and leads to the convenient division of these soils into normal and over-consolidated materials, unlike residual soils for which mineralogy, and not stress history (a concept which has not much if any relevance), is the important factor (Wesley 2009; Nnadi 1987). The general climatic requirements for the formation of residual soils are an average annual rainfall of at least 1 200 mm and a daily temperature in excess of 25°C (Gogo-abite 2005). The classification of residual soils is presented in Table 1, while other peculiar tropical soils, such as laterites, tropical red clays, peats and black cotton soils are discussed in the following paragraphs.

Laterites

Rahardjo *et al* (2004) described laterites as a product of the in-situ weathering of igneous, sedimentary and metamorphic rocks under unsaturated conditions. According to Information Technology Associates USA (ITA USA 2011), "Chemical and microscopic investigations have shown that lateritic clay differs from those commonly found in temperate regions. It does not contain the hydrous silicate of alumina, but is a mechanical mixture of fine grains of quartz with minute scales of hydrates of alumina. Lateritic clay is easily soluble in acid while clay is not, and after treating laterite with acids the alumina and iron leave the silica as a residue in the form of quartz. The great abundance of alumina in some varieties of laterite is a consequence of the removal of the fine particles of gibbsite, hydrargillite and diaspore from the quartz by the action of gentle currents of water."

A summary of the geotechnical properties of laterites is given in Table 2.

Table 2 Geotechnical properties of laterites

Geotechnical property	Behavioural trend
Density	Density increases with depth in the laterite deposits. The upper layers are more porous and possess a high void ratio as a result of weathering and leaching of soil minerals, while the lower layers have a lower porosity and void ratio, as a result of finer particles resulting from the breakdown of the upper granular structure by the removal of the sesquioxides cementing agents (Rahardjo *et al* 2004; Townsend *et al* 1973).
Compressibility	The compressibility of laterite soils is generally low and its compaction results in improved unit weight, void ratio and compression index (Ogunsanwo 1990).
Permeability	The permeability reduces with increase in depth, as a result of the leaching of fines combined with the effect of weathering, which fills up the pores in the lower deposits. Compacted laterites also show a reduction in permeability as the degree of saturation increases (Nnadi 1987).
Shear strength	The factors that affect the shear strength of laterites are degree of weathering, mineralogical and chemical composition, and water content. The shear strength parameters generally increase with depth as weathering decreases with depth (Rahardjo *et al* 2004). Increase in kaolinite content increases cohesion, increase in sesquioxide content increases the internal angle of friction, but increase in moisture leading to inundation results in drastic reduction of stability capacity and leads to shear failure (Gogo-abite, 2005).

Tropical red clays

Tropical red clays are formed by the weathering of various types of rocks which range from granite, volcanic lavas and ashes (Anon 1990). The colour of the soils could be brown, reddish brown or pure red, and they contain iron sesquioxides and halloysites, kaolinite and/or allophane clay minerals (Culshaw *et al* 1992). A feature of these red clays is that they possess enough cohesion to enable an unsupported block of the soil to be trimmed or carved in-situ, and this is as a result of their negative pore pressures which increase their effective strength and enhance their ability to stand unsupported. Geotechnically they are unusual in that, despite having high clay content (over 50%), intact specimens are friable and comprise an open, weakly bonded structure of silt-sized clay particles resulting in high void ratios and low densities. They dry rapidly on exposure and are prone to stress-sensitive collapse (Culshaw *et al* 1992; Hobbs *et al* 1992). Considering the subject of sample disturbance which arises as a result of the various methods of sampling, Terzaghi and Peck (1967) reported that the value of overconsolidation ratios (OCRs) of temperate soils (using traditional sampling methods), estimated via the method of Schmertmann (1953), lies between 0.3 and 0.7. However, Culshaw *et al* (1992) observed that the tropical red clays of Kenya have OCRs greater than 1, and as such they concluded that the method of Schmertmann cannot be used to quantify the effect of sample disturbance for tropical soils. This they attributed to the variable bonding strength of the bonded red clays, and the extent to which this bonding strength is reduced by sampling cannot be ascertained without comparative experimental data. In addition, it has been observed by Wesley (2009) that tropical red clays of Java, Indonesia, when plotted on a conventional e-logP graph, behave as moderately overconsolidated soil, but when the consolidation behaviour of the same soil is plotted on a linear compression vs P graph, the curves generated are reasonably close to linear and the evidence of yield stress disappears. He also reported that sample remoulding has no effect on the compression behaviour (e vs logP plot) of the red clays, because they exist naturally in a dense, unstructured state that is close to their liquid limit.

Tropical peats

Tropical peats are organic soils deposited in mires of the tropics and subtropics lying between latitudes 35°N and 35°S, including those at high altitudes (Andriesse 1988). They possess ash contents in the range of 0–55% (Wust *et al* 2003). A study of the tropical peat deposits of Tasek Bera, Malaysia, by Wust *et al* (2003) showed that the soil could not fit into any of the classification systems provided for peats found in the temperate regions of the world. Wust *et al* (2003) gave the following reasons for this peculiar behaviour of tropical peats:

- Temperate and boreal peats are often dominated by bryophytes and shrubs. Root penetration is thus shallow and decomposition rates are often low. In contrast, tropical peatlands have various tree species with roots penetrating the organic deposits for several metres. Rates of biomass production and primary decomposition are high. Subsurface input of organic matter from decaying roots and root exudates is therefore much greater in tropical than in temperate peat deposits. Hence, the rubbing test and examination of liquid extracted from tropical peat lead to incorrect characterisation of texture, which is often fibric because of woody components.
- Existing classification schemes for temperate and boreal peats are based on selected characteristics for specific uses in the fields of agriculture, engineering, energy, etc, rather than having a generic approach. This results in a lack of correlation between field observations and laboratory test results, because field investigations are often regarded as less important than laboratory results, and also limits the interpretation of data collected in the field.
- Classifications of organic soil for agricultural purposes are based on a control section. Hence, a 5 m thick peat deposit would be classified according to the upper 50 cm section of the profile, ignoring the nature and origin of the underlying deposit. A full description and characterisation of the complete stratigraphic section is required for classification.

It is therefore obvious why tropical peats are not captured in Table 1 and are thus not considered in this work.

Black cotton soils

Black cotton soils (BCS) are expansive soils (Tomlinson & Boorman 1999) that are found in the north-eastern part of Nigeria, Cameroon, Lake Chad Basin, Sudan, Ethiopia, Kenya and South Zimbabwe. The

Table 3 Summary of CPT pile design and prediction methods

CPT pile prediction method	q_b (unit end bearing capacity)	f (unit shaft friction)
Schmertmann (1978)	$q_b = \dfrac{q_{c1} + q_{c2}}{2} \leq 15$ MPa	In clay: $f = k_c f_s \leq 120$ Kpa $\quad k_c = 0.2 - 1.25$
		In sand: $Q_s = k\left[\sum_{d=0}^{8D} \dfrac{d}{8D} f_s A_s + \sum_{d=8D}^{L} f_s A_s\right]$ Where K depends on d/D ratio
De Ruiter and Beringen (1979)	In clay: $q_b = N_c S_u \leq 15$ MPa $S_u = \dfrac{q_{ca}}{N_k}, N_c = 9, N_k = 15 \text{ to } 20$	In clay: $f = aS_u \leq 120$ KPa A = 1 for NC clay and 0.5 for OC clay
	In sand: Similar to Schmertmann (1978)	In sand: $f = min\begin{cases} fs \\ \dfrac{q_{ca}}{300} (compression) \\ \dfrac{q_{ca}}{400} (tension) \\ 120 \text{ KPa} \end{cases}$
LCPC (Bustamante and Gianeselli 1982)	$q_b = k_{b1} q_{eq}(tip)$	$f = \dfrac{q_{eq}(side)}{k_{s1}} \leq max\ k_{s1} = 30 - 150$ depending on soil type, pile type and installation procedure
Tumay and Fakhroo (1982)	Similar to Schmertmann (1978)	$f = mf_{ca} \leq 72$ KPa $m = 0.5 + 9.5e^{-0.009fca}$ where f_{ca} is average friction in KPa
Aoki and De Alencar (1975)	$q_b = \dfrac{q_{ca}(tip)}{F_b} \leq 15$ MPa F_b depends on pile type = 1.75 for PPC driven piles	$f = \dfrac{q_{ca}(side)a_1}{F_{s2}} \leq 120$ KPa $a_1 = 1.4 - 6$ depends on soil type while F_{s2} depends on pile type = 3.5 for PPC driven piles
Price and Wardle (1982)	$q_b = k_{b2} q_{ca}(tip)$ where k_{b2} depends on pile type = 0.35 for driven piles	$f = a_2 f_s$ where $a_2 = 0.53$ for driven piles
Philipponnat (1980)	$q_b = k_{b3} q_{ca}(tip)$ where k_{b3} depends on soil type = 0.4 for sand, 0.45 for silt and 0.5 for clay	$f = \dfrac{q_{ca}(side)a_3}{F_{s2}}$ where a_3 depends on pile type = 1.25 for PPC driven piles and F_{s2} = 50–200 depending on soil type
Penpile (Clisby *et al* 1978)	$q_b = \begin{cases} 0.25 q_{ca}(tip) \text{ for pile tip in clay} \\ 0.125 q_{ca}(tip) \text{ for pile tip in sand} \end{cases}$	$f = \dfrac{f_s}{1.5 + 14.47 f_s}$ where f and f_s are in MPa
q_{ca} = average q_c values over a specified zone that depends on the method		

Source Abu-Farsakh and Titi (2004)

soil is also found in India, Australia and the south-west of the United States of America, South Africa, Israel and other places where the annual evaporation exceeds the precipitation (Chen 1975). They contain high amounts of montmorillonite and are very problematic in construction (either for building of houses or for the construction of roads) as a result of differential settlement arising from the shrink-swell behaviour of the soil. The swelling occurs when polar molecules, such as water or organic molecules, adsorb onto the tip of the soil layers in the inter-planar space. The expansion of the soil interlayer and swelling is thought to be primarily influenced by the type of exchangeable cations present in the aqueous solutions that come into contact with the clay (Luckham & Rossi 1999; Zhou 1995). Cracks measuring 70 mm wide and up to 3 m deep have been reported in places with high deposits of BCS (Adeniji 1991).

MATERIALS AND METHODS

Data collection and modification

Eight CPT data (logs) in kg/cm^2 comprising 320 data points were collected from

MTN Nigeria telecommunications Ltd. The logs were generated from results of CPT tests carried out for the purposes of designing suitable foundations for telecommunication towers across the Anambra, Imo and Delta states of Nigeria. The depth of the logs ranged from 0.25–9.75 m below ground level. The data from Anambra was code-named ANAM 1-4, that from Imo was named IMO 1-3, and the Delta data was named DELTA.The soils upon which the tests were carried out were classified according to the predominant soil type, whether cohesive or cohesionless. The respective ultimate end-bearing pile capacity values Q_u in KN were then computed for each location and depth, using the methods summarised in Table 3 on page 47, while assuming that the piles to be installed are 300 mm diameter piles. These calculations were done for depths of 4.5, 4.75, 5.0, 5.25, 5.5, 6.5, 6.75, 7.0, 7.25 and 7.5 m for each location, while neglecting skin friction f_s, because the parameters for its compuation were not available, and by implication considering the piles as strictly end-bearing piles. The unavailability of data for computing skin friction was as a result of the dutch cone penetrometer models used by the contractors. The calculated pile capacities are presented in Figures 1 to 8.

Methods of estimating pile capacity from the CPT test

Several authors have worked on the direct methods of estimating pile capacity using the CPT approach. The authors and their approaches are presented below:

The Schmertmann (1978) method

This is based on a summary of the work on model and full-scale piles presented by Nottingham (1975). The method is as described in Equation 1:

$$q_t = \frac{q_{c1} + q_{c2}}{2} \tag{1}$$

Where: q_t is the unit tip bearing capacity of the pile, q_{c1} the minimum of the average cone tip resistances of zones ranging from 0.7D to 4D below the pile tip (where D is the pile diameter and q_{c1} is determined by the minimum path rule) and q_{c2} is the average minimum cone tip resistances over a distance 8D above the pile tip. Thus the zone 8D above 0.7D–4D below the pile tip represents the failure surface, which is approximated by a logarithmic spiral. A limitation of the Schmertmann method

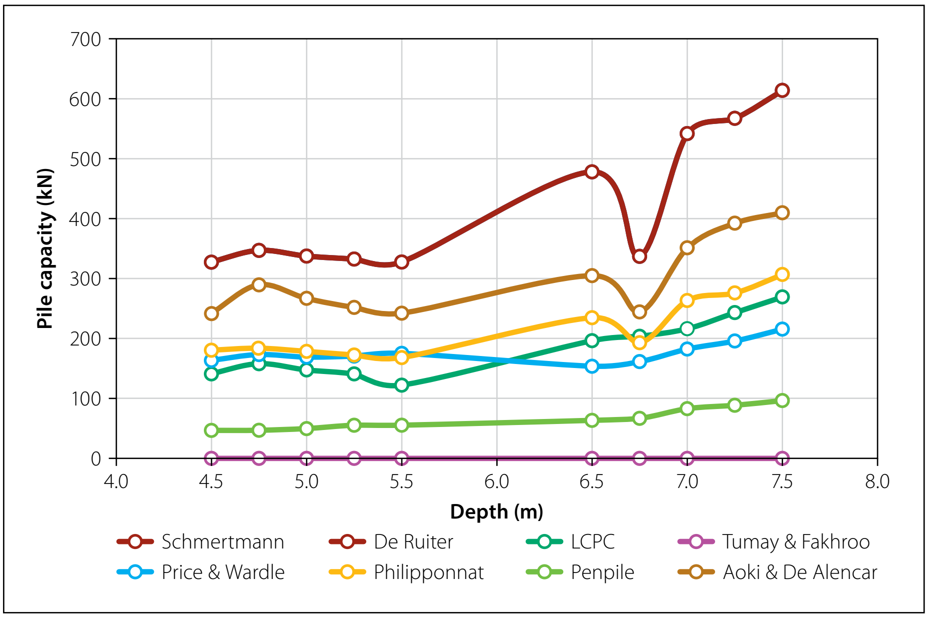

Figure 1 Performance of pile capacity methods across depths at IMO3

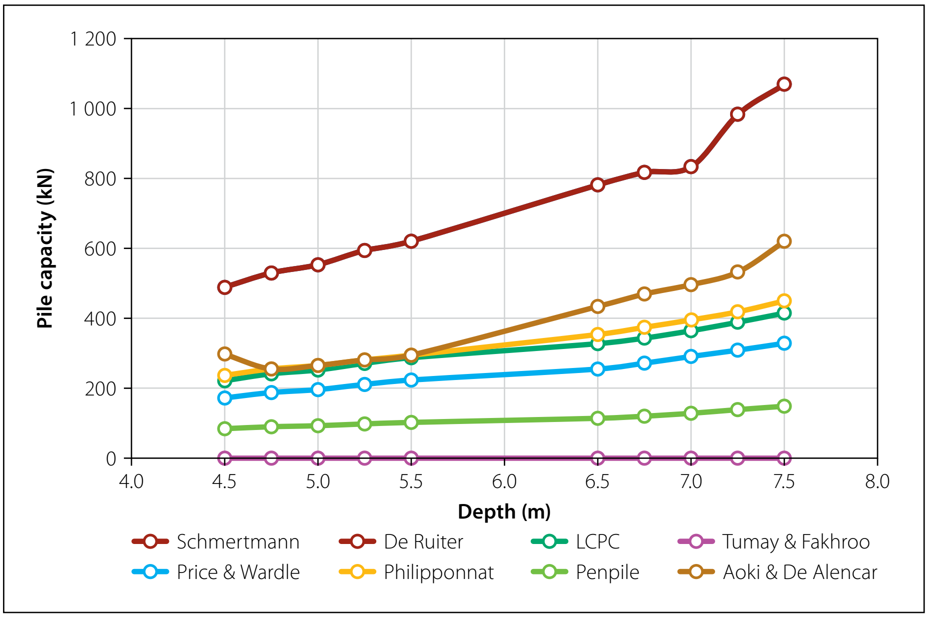

Figure 2 Performance of pile capacity methods across depths at ANAM1

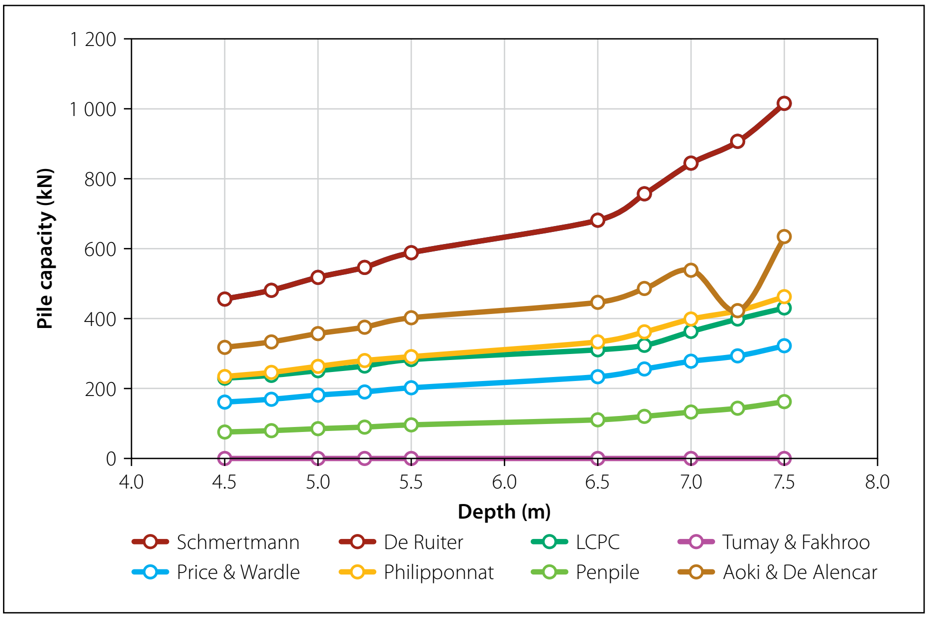

Figure 3 Performance of pile capacity methods across depths at ANAM4

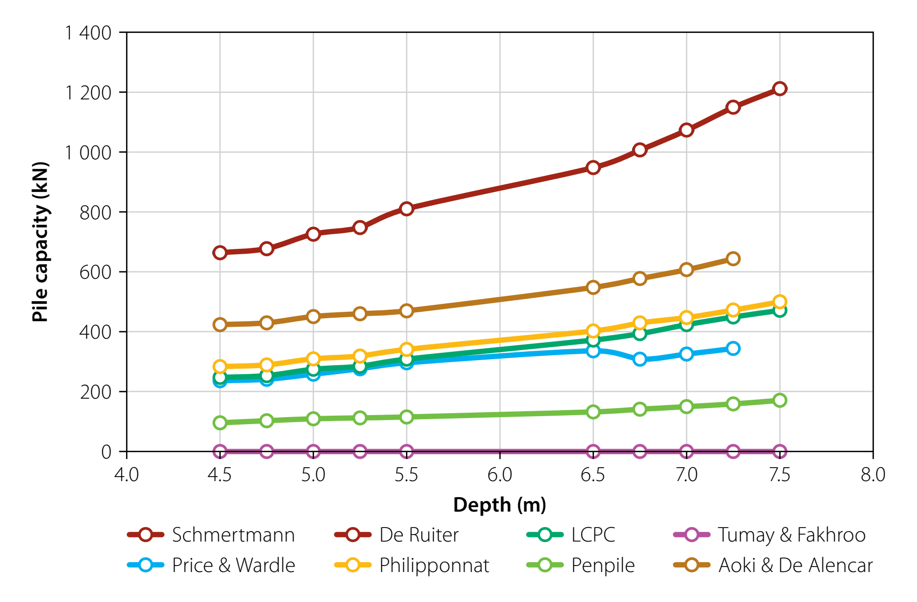

Figure 4 Performance of pile capacity methods across depths at ANAM2

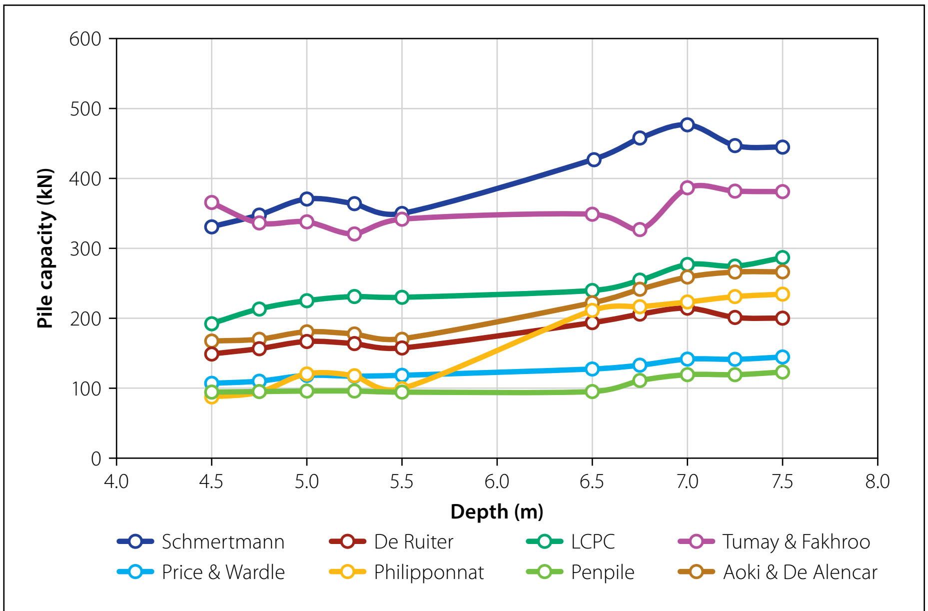

Figure 5 Performance of pile capacity methods across depths at ANAM3

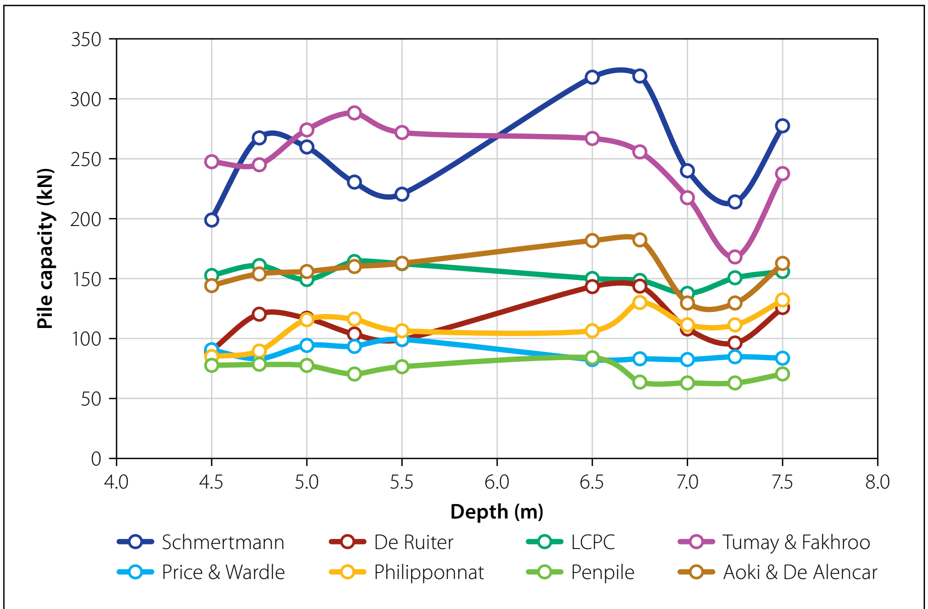

Figure 6 Performance of pile capacity methods across depths at DELTA

is that the skin friction cannot exceed 120 kPa and the method of determining skin friction varies for different pile types, especially in clay soils.

De Ruiter and Beringen (1979) method

De Ruiter and Beringen (1979) presented a method based on experience gained in the North Sea by Fugro Consultants International (Eslami & Fellenius 1995), which is similar to Schmertmann's method for determining q_t in sands, but differ in clays. In clays q_t is given by Equation 2, where N_c is the bearing capacity factor, S_u the undrained shear strength of each soil layer and N_k the cone factor.

$$q_t = N_c S_u(tip) \text{ where } S_u(tip) = \frac{q_c(tip)}{N_k} \quad (2)$$

The LCPC (Bustamante & Gianeselli 1982) method

The LCPC or the French method was developed by Bustamante and Gianeselli (1982) and is based on the analysis of 197 pile load tests with a variety of pile types. In this method both the unit tip bearing capacity q_t and the unit skin friction f of the pile are obtained from the cone tip resistance q_c. The sleeve friction f_s is not used (Titi *et al* 1999) and this is unlike the other methods. The unit tip bearing capacity q_t is predicted from Equation 3 for which the average q_c, is determined within a zone of 1.5D above and 1.5D below the pile tip, and K_b is an empirical bearing capacity factor varying from 0.15 to 0.6 depending on the soil type and pile installation procedure. The method appears suitable for all pile and soil types.

$$q_t = k_b q_c(avg) \quad (3)$$

The Tumay and Fakhroo (1982) method

Tumay and Fakhroo (1982) proposed a method for predicting the ultimate pile capacity of piles in clay soils. Their method is similar to the Schmertmann method, and the unit tip bearing capacity q_t is determined from Equation 4 where q_{c1} is the average of q_c values 4D below the pile tip, q_{c2} is the average of the minimum q_c values 4D below the pile tip, and q_a is the average of the minimum of q_c values 8D above the pile tip. They suggested an upper limit of 15 MPa for the unit pile tip bearing capacity q_t. Hence, in using this method, the pile capacity cannot exceed 15 MPa.

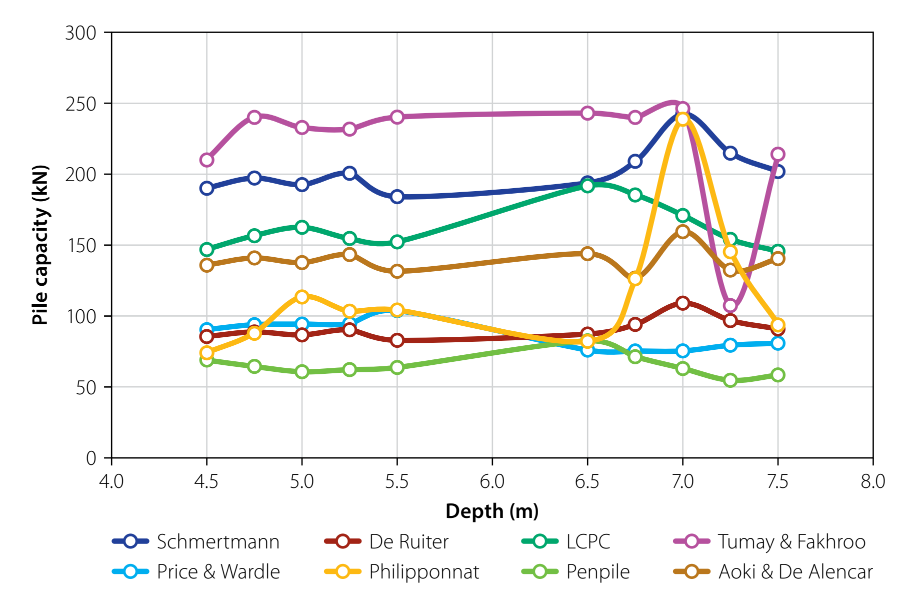

Figure 7 Performance of pile capacity methods across depths at IMO1

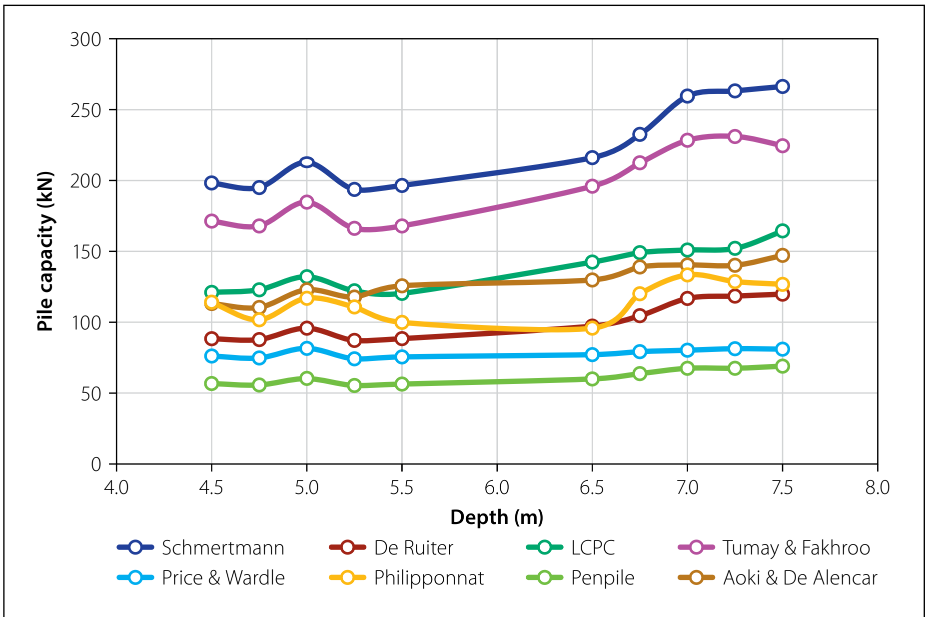

Figure 8 Performance of pile capacity methods across depths at IMO2

$$q_t = \frac{q_{c1} + q_{c2}}{4} + \frac{q_a}{2} \quad (4)$$

The Aoki and De Alencar (1975) method

The method of determining the ultimate load carrying capacity of a pile from CPT data as presented by Aoki and De Alencar (1975) is given by Equation 5. The unit tip bearing capacity q_t is obtained as follows:

$$q_t = \frac{q_{ca}(tip)}{F_b} \quad (5)$$

Where: $q_{ca}(tip)$ is the average cone tip resistance around the pile tip, and F_b is an empirical factor which depends on the pile type. F_b ranges from 3.5 to 1.75, depending on whether the pile is a bored, Franki, steel or precast pile. The upper limit for q_c is also 15 MPa. The method can only be applied to sands, silts and clays.

The Price and Wardle (1982) method

The relationship $q_t = K_b q_c$ was established by Price and Wardle (1982) as a means of determining the ultimate load-carrying capacity of a pile from CPT data. The factor K_b is equal to 0.53 for driven piles and 0.62 for jacked piles. Similarly, an upper limit of 15 MPa is also imposed on the determined value of the ultimate end-bearing pile capacity. The method is thus primarily applicable to jacked, bored and driven piles in stiff (London) clay.

The Philipponnat (1980) method

Philipponnat (1980) established that $q_t = K_b q_{ca}$ where K_b varies from 0.35–0.5 depending on whether the soil is gravel, sand, silt or clay. The cone tip resistance q_{ca} is determined from Equation 6.

$$q_{ca} = \frac{q_{ca}(A) + q_{cb}(B)}{2} \quad (6)$$

Where: $q_{ca}(A)$ is the average cone tip resistance within a distance 3D above the pile tip and $q_{cb}(B)$ is the average cone tip resistance within a distance 3D below the pile tip. The method is applicable to gravels, sands, silts and clays.

The Penpile method

The Penpile method (Clisby *et al* 1978) is applicable to sands and clays, and the unit tip bearing capacity of the pile

$$q_c = \begin{cases} 0.25q_{ca}(tip) f\, or\ pile\ tip\ in\ clay \\ 0.125q_{ca}(tip) f\, or\ pile\ tip\ in\ sand \end{cases}$$

Where: q_c is the average of three cone tip resistances close to the pile tip.

A tabular presentation of all the direct methods of estimating pile capacity using the CPT logs is as shown in Table 3 (p 47).

Data analysis

A statistical package for social science (SPSS 16.0) was used to analyse the data generated. The data, comprising pile capacity values, pile capacity methods, depth of pile, and location, was coded into the SPSS software. Statistical parameters like mean, standard deviation, univariate analysis of variance, and post-hoc tests such as least square differnce (LSD) and Bonferroni comparisons were executed.

Statistical parameters for choosing the most suitable method

The statistical parameters of interest which will aid in determining which of the pile capacity methods is most suited for the area of interest are the general linear model univariate analysis of variance and the post-hoc tests.

The general linear model (GLM) is a model which adjusts for interactions of covariates with the given factors, while the univariate GLM is the version of the general linear model used to implement the analysis of variance (ANOVA) test. The process deals with a situation where there is one dependent variable and one or more independent variables. The general linear model (GLM) implementation of ANOVA supports parameters such as "main effect" and "interaction effect". The main effect is the direct effect of an independent variable (depth, location of pile and method of analysis) on the dependent variable (pile capacity), while the interaction effect is the joint effect of two or more independent variables on the dependent variable. The key statistic in ANOVA is the F-test which tests for difference in group means, i.e. testing if the means of the groups formed by values of the independent variable or combinations of values for multiple independent variables are significant. If the group means do not differ significantly it implies that the independent variable did not have an effect on the dependent variable. But if the F-test shows that the independent variable is related to the dependent variable, then post-hoc tests are used for further examination.

Post-hoc analyses are usually concerned with finding patterns and or relationships between subgroups of sampled populations that would otherwise remain undetected and undiscovered using earlier statistical methods (Wikipedia 2014). The tests are designed for situations where a significant omnibus F-test with a factor that consists of three or more means has been obtained, and additional exploration of differences among means is needed to provide specific information on which means are significantly different from one another. Two popular post-hoc tests that will be considered in this work are the least significant difference (LSD) test and the Bonferroni test. The LSD test explores all pair-wise comparisons of means comprising a factor using the equivalent of multiple T-tests, i.e. it determines what the smallest difference between means would be for the comparison to be statistically significant. A limitation of the LSD is that it makes no adjustment for the fact that multiple comparisons are being made. The Bonferroni test, on the other hand, multiplies each of the significant levels from the LSD test by the number of tests carried out and, as such, mean differences that were significant in LSD could become non-significant after the completion of the Bonferroni tests.

RESULTS AND DISCUSSIONS

Comparative performance of the pile capacity determination methods

The figurative illustration of the comparative performance of the various pile capacity determination methods across the various depths and locations is shown in Figures 1 to 8, while the comparative

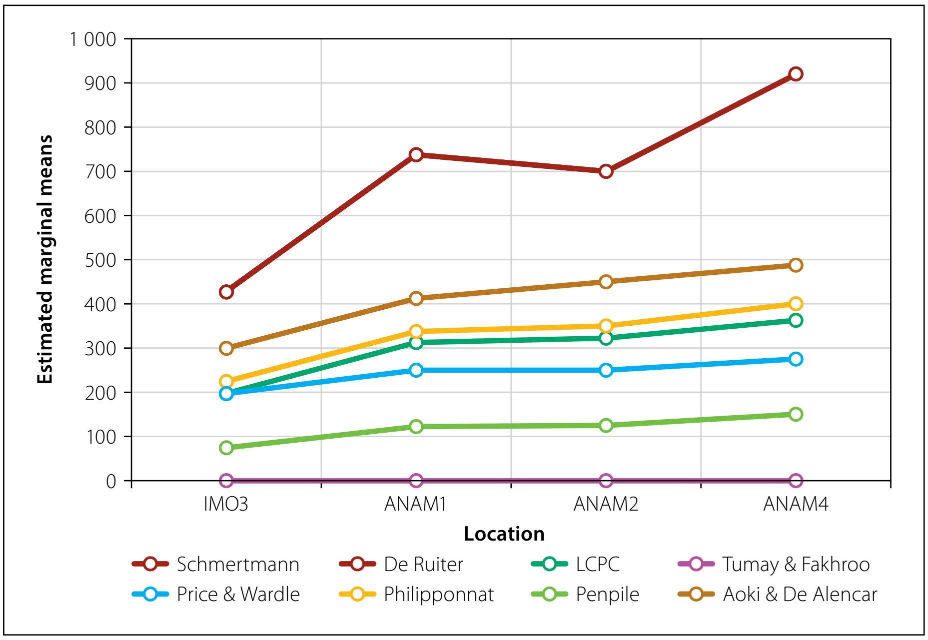

Figure 9 Comparative performance of pile capacity methods in cohesionless soils across all depths and locations

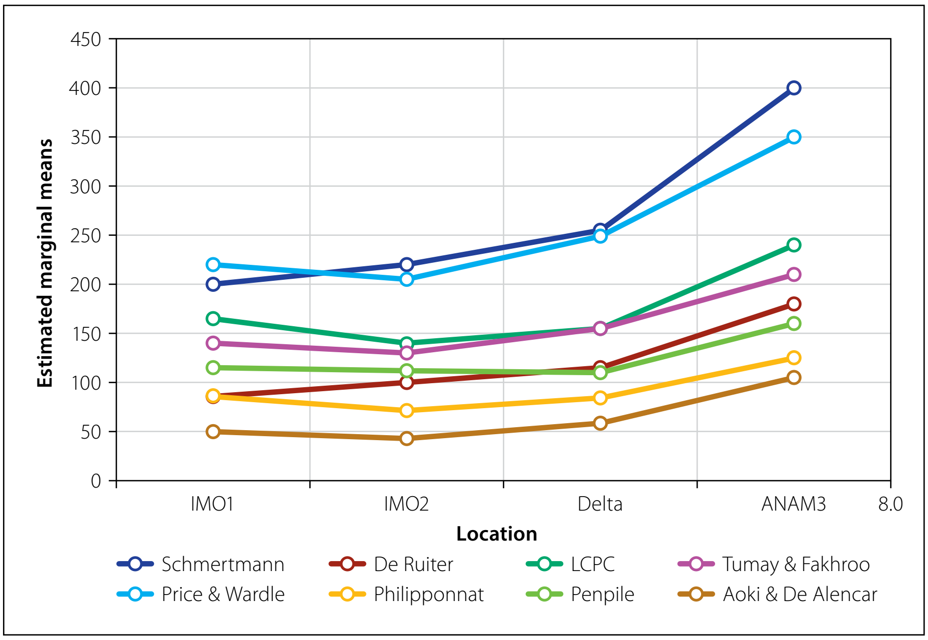

Figure 10 Comparative performance of pile capacity methods in cohesive soils across all depths and locations

summary of the pile capacity determination methods across locations is shown in Figures 9 and 10. In these figures the estimated marginal means of the pile capacities determined by the various methods are used as a basis of comparison. The estimated marginal means are basically the mean of a group or subgroup's measures of a variable in an experiment, and is important when comparing the means of unequal sample sizes where one takes into consideration each mean in proportion to its sample size. Thus, by observation, one can see that for cohesionless soils, the Aoki and De-Alencar, Philipponnat, LCPC, and Price and Wardle methods had central values which are also close (Figure 9). The other methods (Schmertmann and De Ruiter, Penpile, and the Tumay/Fakhroo methods) had either very high or very low values of pile capacity. For cohesive soils it is the LCPC, Tumay and Fakhroo, De Ruiter and Penpile methods that are in the central range (Figure 10). The others had either very high values implying lower reinforcements for the piles, or very low values implying very high reinforcements. The generally perceived good performance of the LCPC, Tumay and Fakhroo, De Ruiter and Penpile methods in clays is believed to be as a result of the fact that the LCPC considered almost all types of piles and

Table 4 UNIANOVA test between subject effects for cohesive soils

Dependent variable: Pile capacity		Degree of freedom	Mean square	F-statistic	Significance (0.05)
Source	Type III sum of squares				
Model	1.022E7[a]	104	98266.407	179.385	.000
Depth	45785.494	9	5087.277	9.287	.000
Location	445866.984	3	148622.328	271.309	.000
Treatment	1408059.670	7	201151.381	367.200	.000
Location* treatment	140814.543	21	6705.454	12.241	.000
Depth* treatment	38886.600	63	617.248	1.127	.264
Error	118324.190	216	547.797		
Total	**1.034E7**	**320**			

a: R squared = .989 (adjusted R squared = .983)

*: Versus

soil conditions in its derivation (197 pile load tests were carried out), while the Tumay and Fakhroo method had clays as its primary target. The De Ruiter method considered the popular pile-bearing capacity factor N_c with q_t determined from average values of cone resistances around the pile tip, while the Penpile methods considered the average of three cone tip resistances around the pile tip. Similarly, the Philipponnat method, just like the LCPC method, considered a wide range of soil types, and that could be the reason for its good performance.

Univanova test between subject effects (for cohesive and cohesionless soils)

The UNIANOVA tests show that a significant omnibus F-test with a factor that consists of three or more means has been obtained. The adjusted R-squared values for the models used are 0.983 and 0.987 for cohesive and cohesionless soils respectively, as can be seen in Tables 4 and 5. It can also be seen from these tables that the interactions between location and treatment (methods of pile capacity prediction), depth and treatment, and location and depth were statistically significant for cohesionless soils, but the interaction between depth and treatment was not statistically significant for cohesive soils. This implies that, for the cohesive soils under investigation, the effect of treatment on depth was not consistent (level of significance for depth vs treatment is 0.264), and this could be attributed to the complexities of the clay minerals, as all the layers were of varying property.

Post-hoc tests

The ANOVA test results shown in Tables 4 and 5 only show that the interactions between treatment/location, depth/treatment and depth/location are significant. It does not compare the interactions between the various treatments, thus the essence of the post-hoc tests. The LSD and Bonferroni tests were carried out for the two groups of soils and the results shown in Tables 6 to 9, with some of the results for the methods with significant differences omitted. The omissions were made because the exhaustive tables are only a repetition of what is presented, as is the case with Tables 6 and 8 where only the mean differences between the De Ruiter and Penpile methods are not significant. From Tables 6 and 7 one can see that both the LSD and Bonferroni tests agree for cohesive soils. The results

Table 5 UNIANOVA test between subject effects for cohesionless soils

Dependent variable: Pile capacity		Degree of freedom	Mean square	F-statistic	Significance (0.05)
Source	Type III sum of squares				
Model	5.920E7[a]	131	451877.577	179.637	.000
Depth	1605171.150	9	178352.350	70.901	.000
Location	1666758.884	3	555586.295	220.864	.000
Treatment	1.699E7	7	2427403.560	964.975	.000
Depth* treatment	1176787.123	63	18679.161	7.426	.000
Location* treatment	1209323.771	21	57586.846	22.893	.000
Depth* location	224085.719	27	8299.471	3.299	.000
Error	475431.182	189	2515.509		
Total	**5.967E7**	**320**			

a: R squared = .992 (adjusted R squared = .987)

*: Versus

Table 6 Post-hoc tests (LSD) for pile capacity methods in cohesive soils

Multiple comparisons						
Pile capacity LSD		Mean difference (I-J)	Standard error	Significance	95% Confidence interval	
(I) Treatment	(J) Treatment				Lower-bound	Upper-bound
De Ruiter	Schmertmann	–148.75*	3.899	.000	–156.44	–141.06
	LCPC	–52.08*	3.899	.000	–59.77	–44.39
	Price and Wardle	–132.09*	3.899	.000	–139.78	–124.39
	Philipponnat	27.31*	3.899	.000	19.62	35.00
	Penpile	–4.60	3.899	.239	–12.29	3.09
	Aoki and De-Alencar	46.03*	3.899	.000	38.34	53.72
	Tumay and Fakhroo	–37.25*	3.899	.000	–44.95	–29.56
Penpile	Schmertmann	–144.15*	3.899	.000	–151.84	–136.46
	De Ruiter	4.60	3.899	.239	–3.09	12.29
	LCPC	–47.48*	3.899	.000	–55.17	–39.79
	Price and Wardle	–127.48*	3.899	.000	–135.18	–119.79
	Philipponnat	31.91*	3.899	.000	24.22	39.60
	Aoki and De-Alencar	50.63*	3.899	.000	42.94	58.33
	Tumay and Fakhroo	–32.65*	3.899	.000	–40.34	–24.96

Based on observed means
The error term is "mean square" (error) = 304.098

* The mean difference is significant at the 0.05 level

obtained from the De Ruiter and Penpile methods are not significantly different (note parameters without asterisks under the mean difference column, whose significant values are greater than 0.05). In addition to that, the Bonferroni test also shows that there are no significant differences between the Schmertmann and Price/Wardle methods, between the LCPC and Tumay/ Fakhroo methods, and between the Philipponnat and Aoki/De-alencar methods. One can therefore say that the methods of De Ruiter and Penpile, and the methods of LCPC and Tumay/Fakhroo may be best suited for tropical cohesive soils. Considering Tables 8 and 9, it is also evident that the LSD and Bonferroni tests agree that there are no significant differences between the methods of Schmertmann and De Ruiter. In addition, Bonferroni's method also shows that there are no differences between the LCPC and Philipponnat methods. It is therefore the authors' opinion that, considering safety and economy, the LCPC and Philipponnat methods performed better for tropical cohesionless soils.

Table 7 Post-hoc tests (Bonferroni) for pile capacity methods in cohesive soils

Multiple comparisons						
Pile capacity Bonferroni		Mean difference (I-J)	Standard error	Significance	95% Confidence interval	
(I) Treatment	(J) Treatment				Lower-bound	Upper-bound
Schmertmann	Price and Wardle	16.66	7.048	.523	–5.55	38.88
De Ruiter	Penpile	–4.60	7.048	1.000	–26.82	17.61
LCPC	Tumay and Fakhroo	14.83	7.048	1.000	–7.39	37.04
Price and Wardle	Schmertmann	–16.66	7.048	.523	–38.88	5.55
Philipponnat	Aoki and De-Alencar	18.72	7.048	.233	–3.49	40.94
Penpile	De Ruiter	4.60	7.048	1.000	–17.61	26.82
Aoki and De-Alencar	Philipponnat	–18.72	7.048	.233	–40.94	3.49
Tumay and Fakhroo	LCPC	–14.83	7.048	1.000	–37.04	7.39

Based on observed means
The error term is "mean square" (error) = 993.418

* The mean difference is significant at the 0.05 level

Table 8 Post-hoc tests (LSD) for pile capacity methods in cohesionless soils

Multiple comparisons						
Pile capacity LSD		Mean difference (I-J)	Standard error	Significance	95% Confidence interval	
(I) Treatment	(J) Treatment				Lower-bound	Upper-bound
Schmertmann	De Ruiter	.0000	11.21497	1.000	–22.1226	22.1226
	LCPC	394.2745*	11.21497	.000	372.1519	416.3971
	Tumay and Fakhroo	682.2902*	11.21497	.000	660.1677	704.4128
	Price and Wardle	454.4270*	11.21497	.000	432.3044	476.5496
	Philipponnat	367.9948*	11.21497	.000	345.8722	390.1173
	Penpile	578.4968*	11.21497	.000	556.3742	600.6193
	Aoki and De-Alencar	285.5832*	11.21497	.000	263.4607	307.7058
De Ruiter	Schmertmann	.0000	11.21497	1.000	–22.1226	22.1226
	LCPC	394.2745*	11.21497	.000	372.1519	416.3971
	Tumay and Fakhroo	682.2902*	11.21497	.000	660.1677	704.4128
	Price and Wardle	454.4270*	11.21497	.000	432.3044	476.5496
	Philipponnat	367.9948*	11.21497	.000	345.8722	390.1173
	Penpile	578.4968*	11.21497	.000	556.3742	600.6193
	Aoki and De-Alencar	285.5832*	11.21497	.000	263.4607	307.7058

Based on observed means
The error term is "mean square" (error) = 2515.509

* The mean difference is significant at the .05 level

Table 9 Post-hoc tests (Bonferroni) for various CPT pile prediction methods in cohesionless soils

Multiple comparisons						
Pile capacity Bonferroni		Mean difference (I-J)	Standard error	Significance	95% Confidence interval	
(I) Treatment	(J) Treatment				Lower-bound	Upper-bound
Schmertmann	De Ruiter	.0000	11.21497	1.000	–35.5376	35.5376
De Ruiter	Schmertmann	.0000	11.21497	1.000	–35.5376	35.5376
LCPC	Philipponnat	–26.2797	11.21497	.564	–61.8174	9.2579
Philipponnat	Schmertmann	–367.9948*	11.21497	.000	–403.5324	–332.4571
	De Ruiter	–367.9948*	11.21497	.000	–403.5324	–332.4571
	LCPC	26.2797	11.21497	.564	–9.2579	61.8174

Based on observed means
The error term is "mean square" (error) = 2515.509

* The mean difference is significant at the .05 level

CONCLUSION

It is therefore possible to conclude from the preceding discussions, and from Figures 9 and 10, that the LCPC and Philipponnat methods are best suited for cohesionless soils, as they are the two methods whose predicted pile capacity values are consistent and average, compared with the Schmertmann and De Ruiter methods whose values are very high. It can also be concluded that the LCPC, Tumay/Fakhroo and De Ruiter methods gave the best results and are thus recommended for cohesive soils within the study area. Specifically, the LCPC method is, interestingly, suitable for both classes of soils and this agrees with the studies on Louisiana soils by Titi *et al* (1999).

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