Numerical Evaluation of Long Pile's Compression Capacity

Mohamed Nabil Omar and Abid Abu Tair
Faculty of Engineering & IT. Structural Engineering Department, British University in Dubai, Dubai, UAE
Email: eng_m_nabil@yahoo.com, abid.abu-tair@buid.ac.ae

***Abstract*—Pile foundation is a system used to provide the stability of the structures, this is by transferring the structures reactions through the weak soil layers to the hard strata [1]. The piles may transfer the structure reaction by the friction between the pile and soil layers or the bearing between the pile and the hard soil strata or it may be a combination between the skin friction and end bearing. The compression pile capacity is one of the important aspects of any pile. During the design stage the pile capacity should be calculated accurately to provide a good estimation of the proposed pile foundation system. The common way used to estimate the pile compression capacity is to use the historical theoretical equations. In this research a numerical model of the pile will be modeled by using PLAXIS 2D, this is to estimate the pile compression capacity "the numerical pile capacity". This capacity will be compared with the actual pile compression capacity based on the static load test results.**

***Index Terms*—pile, numerical capacity, PLAXIS 2D**

I. Introduction

The most common procedure which is used to estimate the pile compression capacity is to use the theoretical equation to estimate the expected pile capacity. These theoretical equations and the soil parameters were based on a field experimental had been done on actual piles. Nowadays, the piling machines has been developed, where the pile depth can be reached to 60 to 80 m (deep piles) compared to 15 to 20 m in the previous decades. The performance of the long piles is different from the performance of the short piles, and this is very clear when it compression between the theoretical pile capacity and the piratical pile capacity which is derived from the results of static load test. Usually the practical pile capacity in case of long piles is greater that theoretical pile capacity by around 30 to 40%.

In this research an evaluation of the numerical pile capacity will be conducted and compared with the practical pile capacity which derived from the results of the static load test. The pile will be molded by using a finite element software *PLAXIS 2D*, all the soil parameters such as soil layers unite weight, internal angle of friction and soil young's modulus will be extracted from actual soil investigation report of the proposed case studies. The research will cover two experimental piles in different have been installed in different location in Dubai.

Manuscript received March 22, 2018; revised June 1, 2018.

II. Theoretical Pile Capacity

In this research two different examples of deep piles will be discussed in terms of the numerical pile capacity which will be calculated from the numerical model and the practical pile capacity which derived from the results of static load test. Both piles have been installed in Dubai, UAE, the classification of the soil layers are sand soil for the shallow layers and rock soil for the deep layers. The following section will cover the common theoretical equations which are used to calculate the pile skin friction resistance and the end bearing resistance in these type of soil.

A. Pile Capacity in Sand Soil

The classification of sand soil in Dubai is varied from medium dense to very dense sand soil. The following equations summarize the common theoretical method to estimate the compression pile capacity in sand soil.

$$P_U = P_{SU} + P_{BU} - W_P \tag{1}$$

where,

P_{SU} = Ultimate pile skin friction resistance

P_{BU} = Ultimate pile end bearing resistance

W_P = Pile weight

According to Michael Tomlinson and John Woodward (1977), the theoretical equation to calculate the compression pile capacity in the sand soil is,

$$Q_P = N_q \sigma'_{\upsilon o} A_b + \frac{1}{2} K_S \sigma'_{\upsilon o} \tan(\delta) A_S \tag{2}$$

where,

$\sigma'_{\upsilon o} =$ effective soil overburden pressure at the pile base.

$N_q =$ pile bearing capacity factor.

$A_b =$ the area of the pile base "cross sectional area".

$K_S =$ coefficient of the soil horizontal stress.

$\delta =$ the angle of friction between pile and soil.

$A_S =$ the area of the pile shaft.

The soil parameters N_q , K_s are empirical factors derived from the results of actual static load test, δ is obtained from the laboratory tests on the internal angle of

doi: 10.18178/ijscer.8.1.26-33

friction between the different soil types and the pile material such as steel or concrete. The value of the bearing capacity factor N_q was by Berezantzev et al. (1961) and it has been found that this coefficient is based on the drained angle of shearing resistance ϕ and the ratio between the pile penetration depth divided by the pile width or the pile diameter, this relationship is shown in Fig. 1. Vesic (1967) confirmed that N_q values give results which is almost near to the practical conditions in case of short pile (depth up to 15 to 20 m) [2], [3].

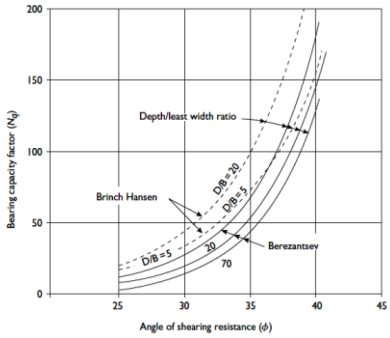

Figure 1. Pile bearing capacity factor.

The second part of equation 2 is used to estimate the skin friction resistance between the pile and the surrounded soil layers. The value of the skin friction parameter K_S is very critical and difficult to be evaluated, because it is depending on the stress history of the soil and the method of the installation. For example, the using of driven pile method lead to a significant increase in the horizontal soil stress from its original K_O value as shown in Table Ⅰ, on the other hand the using of bored pile technique can loosen the soil, and reduce the horizontal soil stress. This factor is governed by the following items;

- The stress history of the soil.
- The ratio between the pile penetration depth and the pile width or diameter.
- The shape and the stiffness of the pile.
- The pile material.

TABLE I. THE COEFFICIENT OF THE SOIL HORIZONTAL STRESS, K_S

Installation method	K_S / K_O
Driven piles, large displacement 15 mm	1.00 – 2.00
Driven piles, small displacement	0.75 – 1.25
Bored and cast-in-place piles	0.70 – 1.00
Jetted piles	0.50 – 0.70

The friction angle between the pile and the soil δ is obtained by factored the effective angle of shearing resistance ϕ of the soil as determined from the relationship with standard penetration test SPT values as shown in Fig. 2. This factor is depending on the pile surface material. Kulhawy (1984) established some values for this factor based on the soil and pile interface condition and it can be applying for the driven and bored piles as shown in Table II [4].

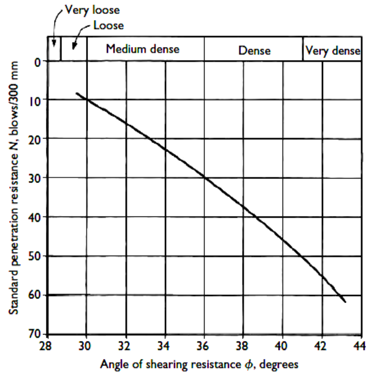

Figure 2. Relationship between standard penetration test N-values and angle of shearing resistance.

TABLE II. VALUES OF THE ANGLE OF PILE TO SOIL FRICTION FOR VARIOUS INTERFACE CONDITIONS PER KULHAWY (1984)

Pile / soil interface condition	Angle of friction between pile and soil δ
Smooth (coated) steel/sand	(0.5 – 0.7) ϕ
Rough (corrugated) steel/sand	(0.7 – 0.9) ϕ
Precast concrete/sand	(0.9 – 1.0) ϕ
Cast-in-place concrete/sand	(1.0) ϕ
Timber/sand	(0.8 – 0.9) ϕ

B. *Pile Capacity in Rock Soil*

For bored and cast-in-place piles which are drilled into rock soil layer act as friction and end bearing piles. Wyllie (1991) estimated the factors and coefficients which are governing the development of shaft friction through the rock socket depth. For the end bearing and pile settlement factors are summarized in the following items;

- The socket length to the diameter ration.
- The strength and modulus of elasticity of the rock layer.
- The base condition of the drilled pile hole with respect to the removal of the drilled material.
- Creep of the material at the rock / concrete interface.
- Settlement of the pile in relation to the elastic limit of the side-wall.

The soil layers around the pile shaft classification, has a significant impact on the skin friction resistance of the pile. On the other hand, the installation technique has an impact on the friction resistance as well. Wyllie (1991) stated that if the bentonite slurry used in the installation process of the pile, the rock socket shaft friction may be reduced by 25% compared to clean rock socket, unless pile load test done to verify the actual value of the friction resistance [5].

The friction resistance of the pile in the rock soil, is the resultant stress due to the friction between the pile material and the rock soil layers. The bond between the pile material and the rock soil layers is factor on the unconfined compression strength of the rock soil, the rock socket bond stress has been developed by Horvarth (1978), Rosenberg and Journeaux (1976), and Williams and Pells (1981) [5], [6]. The ultimate pile shaft resistance, in the rock soil can be estimated by using the following equation:

$$f_s = \alpha \cdot \beta \cdot q_{uc} \tag{3}$$

where;

$\alpha =$ reduction factor related to q_{uc} as shown in Fig. 3.

$\beta =$ correction factor related to the discontinuity spacing in the rock mass as shown in Figs. 3 & 4 [7].

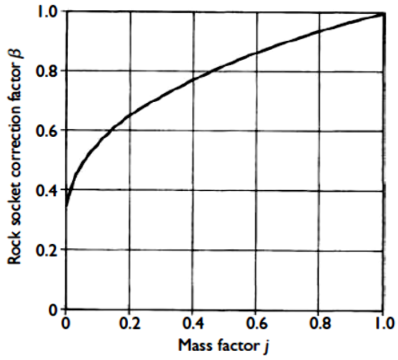

Figure 3. Reduction factors for discontinuities in rock mass (after Williams and Pells).

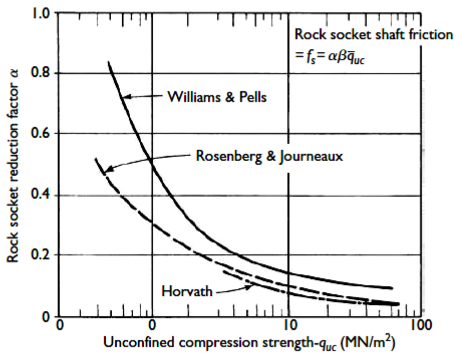

Figure 4. Reduction factors for rock socket shaft friction.

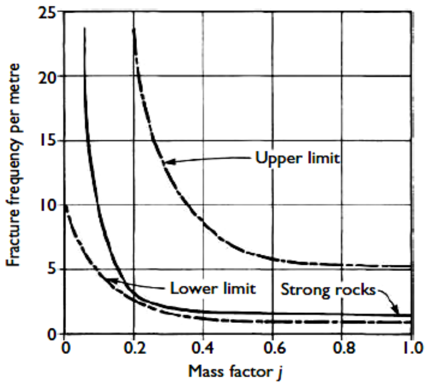

Figure 5. Mass factor value (after Hobbs).

The Williams and Pells (1981) curve in Fig. 5 is higher than the other two curves, but the factor is having the same value in all curves and it is factored on the mass factor, which is the ratio between the modulus of elasticity of the rock mass and the intact rock. If the mass factor cannot be identified from the loading test, it can be estimated with respect to the rock quality designation (RQD) or the discontinuity spacing quoted by Hobbs (1975) as shown in Table III:

TABLE III. Mass Factor J Value with Respect to RQD and the Discontinuity Spacing

RQD (%)	Fracture frequency per meter	Mass factor j
0 - 25	15	0.2
25 - 50	15 - 18	0.2
50 - 75	8 - 5	0.2 – 0.5
75 - 90	5 - 1	0.5 – 0.8
90 - 100	1	0.8 – 1.0

The used criteria to estimate the pile ultimate bearing resistance in case of the pile capacity is a combination between friction and bearing resistance. Both resistances are based on correlations between the pile static load test and the result of filed test in rock formations or laboratory tests. the following is the equation which is used to calculate the pile base resistance for the driven and bored piles:

$$q_b = 2N_\phi \cdot q_{uc} \tag{4}$$

where the bearing capacity factor N_ϕ is equal to:

$$N_\phi = \tan^2(45 + \frac{\phi}{2}) \tag{5}$$

The bearing resistance of the pile in weak rock soil influenced by the drilling technique. The use of percussive drilling machines causes a formation of a soft sludge at the bottom level of the drilled pile shaft. This is not only weakening the base resistance, it makes a difficulty to identify the accurate classification of the rock soil and the correct estimate the soil parameters at the base level as well. For weathered mudstones, siltstones and shales undisturbed samples must be collected during the investigation stage and shear strength tests should be applied, the test results will be used to calculate the base resistance [8].

For the moderately weathered mudstones, siltstones and shales uniaxial compression tests should be made on the rock cores samples to estimate the compression strength. The pile base resistance can be calculated from the uniaxial compression test results by using the relationship between and RQD as shown in Table IV:

TABLE IV. Ultimate Base Resistance of Piles Related to the Uniaxial Compression Strength of the Intact Rock and the RQD of the Rock Mass

RQD (%)	q_{uc}	C	ϕ
0 - 70	$0.33q_{uc}$	$0.1q_{uc}$	30
70 - 100	$0.33 - 0.8q_{uc}$	$0.1q_{uc}$	30 - 60

It is recommended that the pile bearing resistance which is calculated based on the previous method, should be adopted with caution due to the risk of high base settlement. Normally a reduction factor 20% is used to reduce the pile bearing resistance. For low safety factors in the calculation of pile compression capacity, influenced the resulting shaft settlement and it could break the bond between the rock soil and the pile material. This will affect directly the calculated pile capacity especially when the pile is general type and its capacity is the combination between the skin friction and end bearing. Therefore, it is recommended to use a reduction factor equal to 30% to 40% to the high value of pile skin friction resistance [9], [10].

III. EXPERMENTAL PILES

In this research, two experimental piles have been installed in different locations in Dubai. These piles will be discussed in details in terms of numerical and practical pile capacity. Table V summarize the piles' specifications.

TABLE V. PILES DETAILS

Pile Type	Pile C.O.L [m]	Pile Toe Level [m]	Pile Length [m]	Pile Diameter [mm]
P1	-4.85	-57.0	52.15	1,500
P2	+3.375	-31.0	34.375	900

Tables VI & VII show the soil layers' classifications for each experimental piles.

TABLE VI. SOIL LAYERS' CLASSIFICATIONS OF PILE TYPE P1

No.	Soil Layer	Layer Depth [m]	Soil parameters
1	Silty Sand and Calcrenite	3.15	E = 30,000 KN/m2 γsat = 18.5 KN/m3 γun sat = 18.5 KN/m3 φ = 34° υ = 0.3
2	Very Weak Sandstone	15.0	E = 200,000 KN/m2 γsat = 19.0 KN/m3 γun sat = 19.0 KN/m3 φ = 40° υ = 0.3
3	Calcisiltite and Gonglomerate	69.0	E = 400,000 KN/m2 γsat = 19.5 KN/m3 γun sat = 19.5 KN/m3 φ = 40° υ = 0.3

TABLE VII. SOIL LAYERS' CLASSIFICATIONS OF PILE TYPE P2

Soil Layer	Layer Depth – m Elevation of Engineering Parameters bottom of each layer (m DMD)		Engineering Parameters				
	Avg. Thick - m	To	Unit Wt, KN/m^3	E - MPa	Poison Ratio	C' - KPa	Ø °
Silty fine sand	13.00	-10.00	18	25	0.35	0	34
Dense to very dense sand	0.7	-10.70	18	50	0.35	0	36
Calacrenite / Sandstone	3.3	-14.00	22	200	0.3	70	32
	2.0	-16.00	22	200	0.3	100	32
	2.0	-18.00	22	200	0.3	80	32
	2.0	-20.00	22	200	0.3	60	32
	2.0	-22.00	22	75	0.3	20	27
	2.0	-24.00	22	75	0.3	27	27
	4.0	-28.00	22	150	0.3	60	32
	5.0	-33.00	22	250	0.3	120	32
	5.0	-38.00	22	250	0.3	130	32
Sandstone	5.0	-43.00	22	400	0.3	85	34

IV. NUMERICAL PILE CAPACITY

A. Numerical Analysis Procedure

A finite element software will be used to model each pile, the software which will be used is PLAXIS 2D version 8.6. The pile model will be modeled by using axisymmetric option and the materials which represent the pile and the surrounded soil will be modeled by using Mohr-Coulomb option. Prescribed settlement will be applied to the pile head and the force – settlement curve will be plotted to predict the numerical pile capacity [11].

The major aspects in the modeling process are the graphical boundaries, soil layers' classification and parameters and the pile material - soil layers' friction angle. These factors will be discussed in details in the following sections:

1) Graphical boundaries

Fig. 6 shows the model's graphical boundaries which should be followed in the modeling process.

The boundaries are as follows;

- D is the pile radius.
- Two layers of mesh transition, each layer width is equal to 3D.
- L is the pile depth.
- The model dimensions are equal to 2.5L for the model depth and 2.0L for the model width.
- The horizontal and vertical displacement of the model edge are not allowed.

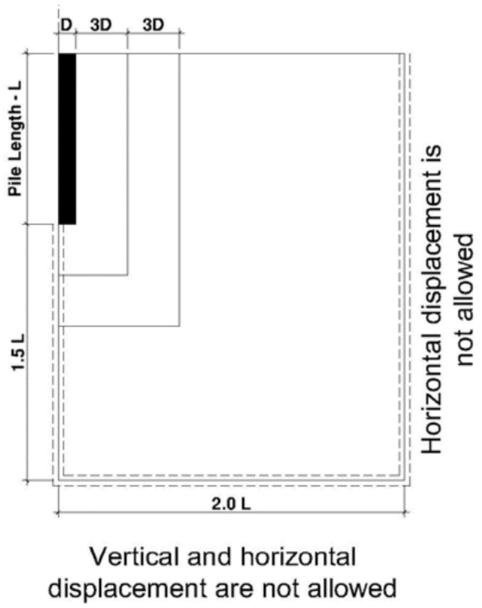

Figure 6. Graphical boundaries for the pile model.

2) Soil layers' classification and parameters

The soil layers' classifications and soil parameters estimated and derived from the soil investigation report of each location. soil layers' modulus of elasticity, cohesion, soil unit weight and the angle of internal friction of each layer are the major important factors which should be identified and used in the pile model.

3) Pile - soil friction relationship

One of the important factors which has a significant impact on the pile skin friction resistance is the pile - soil interface condition. This skin friction factor is depending on the following items:

- Soil layers' classification.
- Pile material.
- The installation technique.

For example, the using of bentonite slurry in the pile installation has a negative impact on the skin friction resistance compared to the other techniques like bored pile or CFA technique. This is because the using of bentonite slurry generates a smooth surface between the pile and the surrounding soil.

Generally, the reduction factor of skin friction resistance due to interface condition has a value between 1.0 to 0.5, in this research the used reduction factor for sand soil layers is 0.8 and the value of the rock soil layers is 0.9 (see Table I).

B. Numerical Piles Capacities

Prescribed settlement has been applied to the pile head, this is to plot the relationship between the radial compression pile capacity in the horizontal axis and the pile settlement in the vertical axis. From this relationship curve, the value of numerical pile capacity has been calculated based on the British standard BS 8004: 1986 recommendation. The ultimate pile capacity defined as that load which produce a settlement of the pile head equal to 10% of the pile diameter or pile width [12].

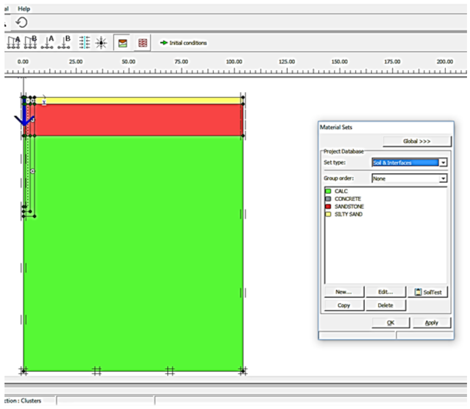

Figure 7. Numerical Model of Pile Type P1.

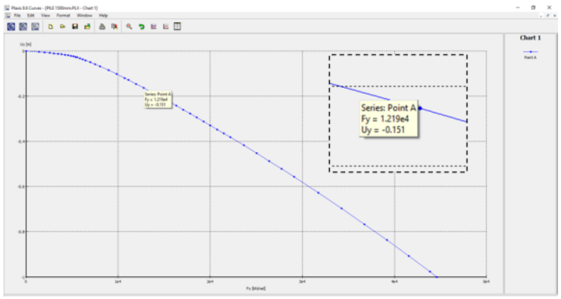

Figure 8. Pile Load – Settlement Relationship Curve of Pile Type P1.

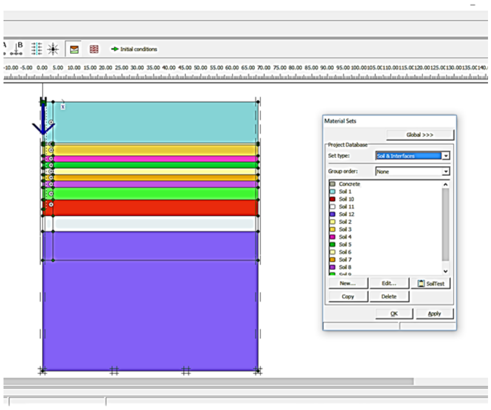

Figure 9. Numerical Model of Pile Type P2.

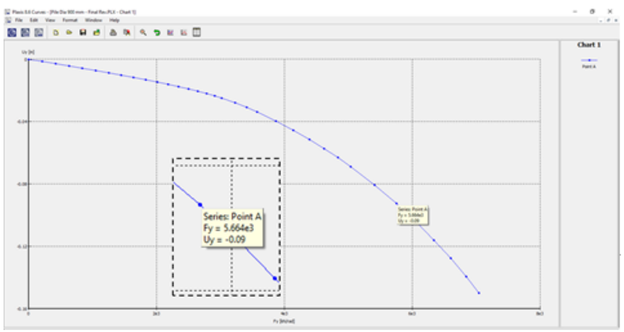

Figure 10. Pile Load – Settlement Relationship Curve of Pile Type P2.

Calculating the numerical pile capacity by using the load - settlement relationship curve from Fig. 7 & 8.

- 10% of the pile diameter = 150 mm = 0.15 m
- From the previous figure, Fy = 12,190 KN/rad at displacement equal to 150 mm

$$Q_u = Fy \times 2\pi = 12{,}190 \times 2\pi = 76{,}553 KN$$

$$Qw = \frac{Q_u}{F.O.S} = \frac{76{,}553}{2} = 38{,}277 KN$$

From the previous equations, the calculated numerical compression pile capacity by using PLAXIS 2D is **38,277 KN**.

Calculating the numerical pile capacity by using the load - settlement relationship curve from Fig. 9 & 10:

- 10% of the pile diameter = 90 mm = 0.09 m
- From the previous figure, Fy = 5,664 KN/rad at displacement equal to 90 mm

$$Q_u = Fy \times 2\pi = 5{,}664 \times 2\pi = 35{,}570 KN$$

$$Qw = \frac{Q_u}{F.O.S} = \frac{35{,}570}{2} = 17{,}784 KN$$

From the previous equations, the calculated numerical compression pile capacity by using PLAXIS 2D is **17,784 KN**.

V. Practical Pile Capacity

The practical pile capacity has been evaluated by using the results of static load test of each pile. The maximum static load test did not reach to the ultimate limit, so an empirical method used to evaluate the practical pile capacity from non-destructive load test results.

A. Prediction of Pile Capacity from Non-Destructive Static Load Test - Chin's Method

Static load testing is the most reliable procedure to evaluate the actual pile compression capacity. This method involves physical loading of the pile by using concrete cubes with specific dimensions and weight at specific time interval and monitoring the pile settlement of the pile head until failure. The applied load should be increased gradually up to the maximum value of the applied load or up to the maximum allowable pile settlement (pile failure point) then the load should decrease gradually as well. The results of the static load test are plotted as load - settlement curve. And the failure load is calculated, the failure load is the load where the pile is subjected to excessive settlement under small or no load increase.

1) Chin's method

Chin's method (Chin and Vail, 1973) is the developed procedure to predict the ultimate pile capacity from the results of non-destructive static load test. The method is applied by assuming that the load-settlement relationship is hyperbolic, and the ultimate pile capacity can be predicted by plotting a curve between the (settlement / test load) in the vertical axis and the (settlement) in the horizontal axis. Then plot the best fit line through the data points. The ultimate pile capacity is derived from the inverse slopes of this line.

$$\frac{\Delta}{Q} = C_1 \cdot \Delta + C_2 \tag{6}$$

$$Q_u = \frac{1}{C_1} \tag{7}$$

where:

Δ = pile displacement

Q_u = ultimate pile capacity

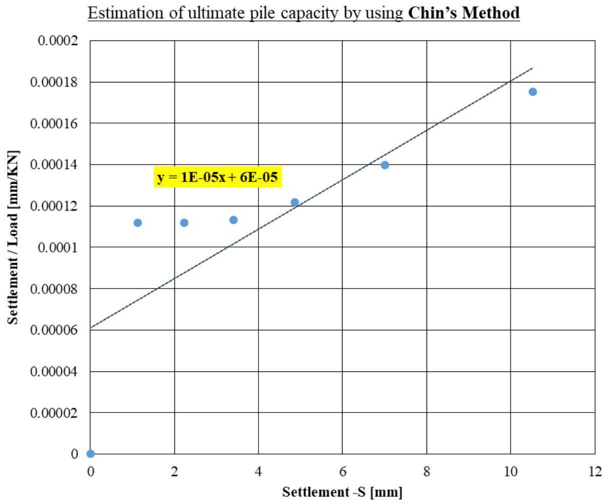

Figure 11. Chin's Digram – Pile Type P1.

Fig. 11 plotted based on the static load test results as shown in Table VIII. From Fig. 11, the equation which represent the best fit line through the data points is:

$$\frac{\Delta}{Q} = C_1 \cdot \Delta + C_2 = 1.0E-05 \cdot \Delta + 0.00006$$

$$Q_u = \frac{1}{C_1} = \frac{1}{1.0E-05} = 100{,}000 KN$$

$$Q_w = \frac{Q_u}{F.O.S} = \frac{100{,}000}{2.5} = 40{,}000 KN$$

TABLE VIII. Static Load Test Results – Pile Type P1

Load - P [KN]	Settlement - S [mm]	Settlement / Load [mm/KN]
0	0	0
10000	1.12	0.000112
20000	2.24	0.000112
30000	3.4	0.000113333
40000	4.87	0.00012175
50000	7	0.00014
60000	10.52	0.000175333

TABLE IX. STATIC LOAD TEST RESULTS – PILE TYPE P2

Load - P [KN]	Settlement -S [mm]	Settlement / Load [mm/KN]
0	0.793	0
2240	1.100	0.000491071
4490	2.960	0.000659243
6780	4.850	0.000715339
8970	7.060	0.000787068
11210	8.935	0.000797056
13460	11.000	0.000817236
15690	12.900	0.00082218
17940	14.850	0.000827759
20180	16.800	0.000832507
22430	19.100	0.000851538

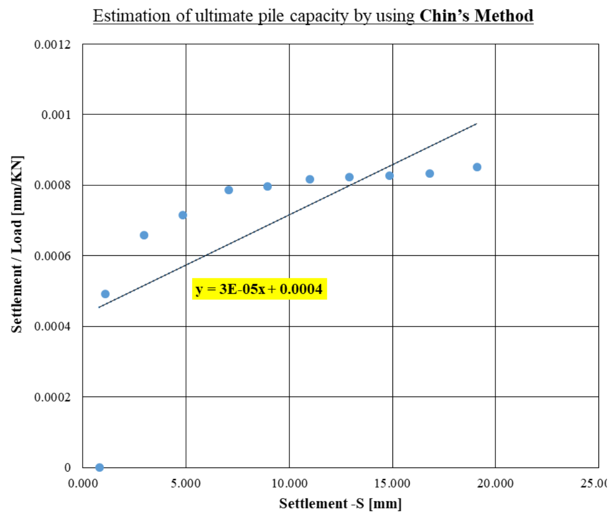

Figure 12. Chin's Digram – Pile Type P2.

Fig. 12 plotted based on the results of static load test results as shown in Table IX. From Fig. 12, the equation which represent the best fit line through the data points is:

$$\frac{\Delta}{Q} = C_1 \cdot \Delta + C_2 = 3.0E-05 \cdot \Delta + 0.0004$$

$$Q_u = \frac{1}{C_1} = \frac{1}{3.0E-05} = 33{,}333KN$$

$$Q_w = \frac{Q_u}{F.O.S} = \frac{33{,}333}{2.5} = 13{,}333KN$$

VI. COMPARISON BETWEEN PRACTICAL AND NUMERICAL PILE CAPACITIES

Table X and Figs. 13 & 14 summarize the theoretical, numerical and practical pile compression capacity of each case:

TABLE X. PRATICAL AND NUMERICAL PILE CAPACITIES

Pile Type	Practical Capacity [KN]	Theoretical Capacity [KN]	Numerical Capacity [KN]
P1	40,000	27,670	38,277
P2	13,333	9,015	17,784

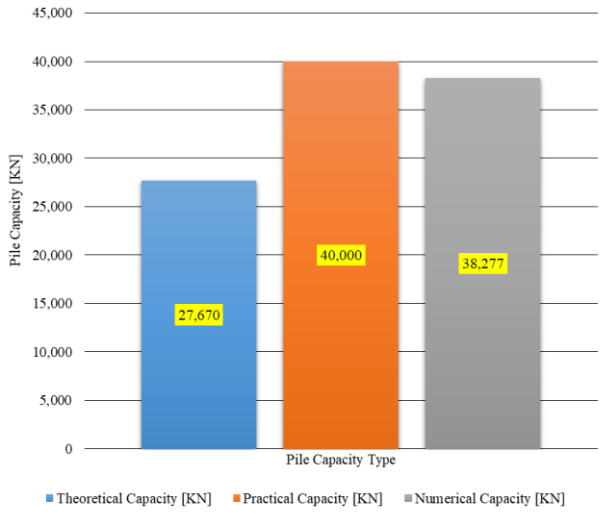

Figure 13. Pile Capacities – Pile Type P1.

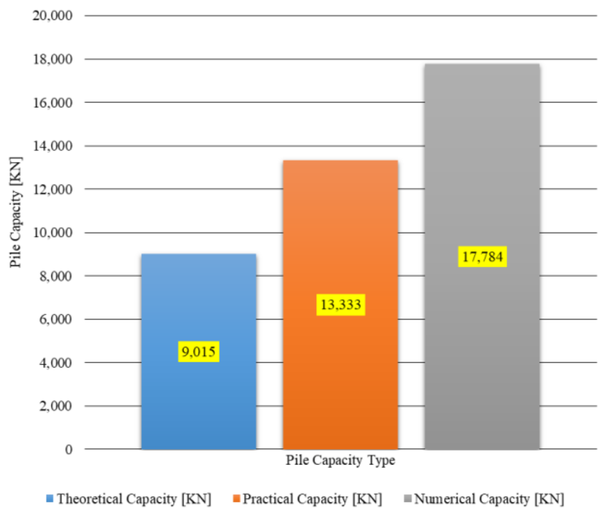

Figure 14. Pile Capacities – Pile Type P2.

VII. CONCLUSION

Proper soil investigation from specialist soil test laboratory during the design stage is essential, to provide a suitable information about the soil layers' classifications and soil parameters such as soil unite weight, internal angle of friction, cohesion and the modulus of elasticity of each soil layer. The importance of the soil parameters is very significant especially if the numerical method will be used in the design stage to identify the compression pile capacity.

The theoretical equations which are used to estimate the compression pile capacity is derived from practical studies had been done from a long time, these practical studies covered the short pile which has a depth range between 15 to 20 m. Nowadays, the development of the piling equipment allow the pile to reach to a depth range between 60 to 80 m. Due to this development, and from the new practical studies, it was found that there is a significant difference between the estimated pile capacity based on the theoretical equation and the practical pile capacity which is derived from the results of static load test, where the practical capacity is almost grater that the theoretical capacity by 30 to 40%. On the other hand, the differences between the numerical pile capacity and the practical pile capacity especially in long piles case is very less, it is around 5 to 10%. Therefore, it is recommended to use the numerical method in the concept design stage which give more accurate results compared to the theoretical method.

Finally, this research provides a comparison between the theoretical, practical and numerical pile capacities has been installed and tested in Dubai. The research result is that the practical pile capacity is higher than the theoretical pile capacity by around 40%. And the numerical pile capacity is higher than the theoretical pile capacity by around 35%. This will provide the ability to reduce the cost of the piles foundation system by around 30%. As well as, reducing of the required pile's materials which is considered as a sustainable practice for our environment.

References

[1] D. Youventharan, "Introduction to pile foundation," Kuantan: Penerbit University Malaysia Pahang, 2009.

[2] O. Aun, *The Loading Behavior of Long Piles*, 1st ed. University of Sheffield, 1980.

[3] J. Bowles, *Foundation Analysis and Design*, 1st ed. New York: McGraw-Hill, 1977.

[4] H. Poulos and E. Davis, *Pile Foundation Analysis and Design*, 1st ed. New York: Wiley, 1980.

[5] M. Tomlinson and J. Woodward, *Pile Design and Construction Practice*, 1st ed. London: Taylor & Francis, 2008.

[6] R. Chellis, *Pile Foundations*, 1st ed. New York: McGraw-Hill, 1961.

[7] B. Das, *Theoretical Foundation Engineering*, 1st ed. Ft. Lauderdale, FL: J. Ross Pub, 2007.

[8] W. Guo, *Theory and Practice of Pile Foundations*, Boca Raton, FL: CRC Press, Taylor & Francis Group, 2013.

[9] R. Liang and F. Zhang, *Deep Foundations and Geotechnical in Situ Testing*, 1st ed. Reston: ASCE, 2010.

[10] B. Look, *Handbook of Geotechnical Investigation and Design Tables*, London: Taylor & Francis, 2007.

[11] R. Brinkgreve, W. Broere, and R. Al-Khoury, "PLAXIS: 2D-version 8," in *Autocad Magazine*, 1st ed. vol. 64, Lisse: Balkema, 2004, pp. 48-49.

[12] "British standard code of practice for foundations," BS 8004: 1986, British Standards Institution, pp. 84-109, 1986.

Mohamed Nabil Omar was born in Egypt in 1983. He received the B.Sc. in civil engineering from Suez Canal University, Port Said, Egypt in 2005. And the M.Sc. in structural engineering from British University in Dubai, Dubai, UAE in 2017. He joined to the teaching team of the faculty if engineering in the British University in Dubai in 2017 as doctor assistant. His main areas of research interest are structural engineering innovations and geotechnical engineering.

Mr. Mohamed Nabil is a member of U.A.E society of engineers, the Egyptian Engineers Syndicate and the Institution of Structural Engineers (IStructE) - Graduate Member. He awarded the prize of best paper in the 3rd international sustainable buildings symposium (ISBS 2017), Dubai, UAE.

Abid Ul Razzaq Abu-Tair, professor of structural engineering at the British University in Dubai and visiting research scholar of the Manchester School of Mechanical, aerospace and civil engineering. He is a chartered civil engineer of the UK Engineering Council and a fellow of the Institution of civil engineers in the UK. He is also a senior fellow of the UK Higher Education Academy, a qualification of teaching and learning and educational management. He was awarded the Henri Adam Diploma by the Institution of Structural Engineers UK and the silver award by the Permanent Way Institute UK for paper published in their journals.

He worked at three UK Universities and three in the Middle East. His major research areas relate to the structural performance, sustainability and asset management with emphasis on durability and rehabilitation of concrete structures.