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EVALUATION OF PILED-RAFT FOUNDATION BEHAVIOR DUE TO THE EFFECT OF EROSION AND SCOUR

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ملخص

عاده ماتستخدم الخوازيق في انتقال الحمل الي التربه القويه وفي حاله وجود الخوازيق داخل مجري مائي قد يحدث نحر(scour) يؤثر علي قدرة التحمل وعلي الهبوط وعلي قيمه الاحتكاك وايضا قيمه حمل الارتكاز في هذه الدراسه تم حساب قدرة تحمل خازوق واحد واربع خوازيق يعلوها لبشه مرتكزه مباشره علي التربه وايضا اربع خوازيق يعلوها لبشه غير مرتكزه علي التربه ومقارنه الهبوط الناتج في كل حاله. كما تم حساب تاثيرالنحر(scour) علي توزيع الحمل علي طول الخازوق وانتقاله اسفل الخازوق ومعرفه قيمه الهبوط الحادث بالنسبه للتربه الرمليه وايضا للتربه المختلفه الخواص وتم تحليل للنتائج للتربه الرمليه وللتربه مختلفه الخواص ووجد ان الهبوط يزيد مع زياده النحر(scour) وقدرة تحمل مجموعه الخوازيق تقل مع زياده عمق النحر والحمل المنقول عن طريق الاحتكاك (friction) يقل ايضا. وتم تحليل النتائج باستخدام برنامج plaxis 3D وكانت النتائج متوافقه مع نتائج العملي.

Abstract

Pile foundations are usually connected in groups of three or more piles. In the present study, the piles were tested in a setup under compressive axial loads. The strain along pile shaft and load at pile tip as well as the pile head loads were measured concurrently. Also, the load under pile cap transferred directly through pile cap to soil has been measured. The experimental program contained installing test piles in dense sand in a soil chamber exposed to compressive axial load. Though, three groups of testing were executed under axial compression. First group load test was carried out on single pile. Second group is four pile cap directly rested on soil. Third group is four piles cap non-rested on soil. The ultimate capacity for single pile and four piles cap directly rested as well as four piles cap non-rested on soil have been determined. Also, the effect of scour on the load distribution of the friction along the pile shaft and the load transferred to the tip of the piles as well as settlement of piles in dense sand different soil has been presented. Finite element analysis has been selected to develop numerical models to study the effect of scour depth on piled foundation. The analysis program consists of group of piles in sandy soil pile diameter (D=.15m) and pile length =1.5m).The analysis has been done for scour depths of pile diameter from 0.5D to 3.0 D. Also, the analysis program consists of group of piles in different soil properties pile diameter =.6m and pile length =15m.The analysis has been done with different scour depths of pile diameter from 0.0Dto 4.0 D. It is concluded that, the settlement of piled foundation increases with increasing scour depths. However, a decrease in the embedded depth of pile of foundation due to scour decreases the skin friction. In addition, conical local scour hole causes a negative skin friction (additional vertical stress). However, the values of ultimate load capacities of piles foundation decrease with increasing scour depths. The settlement of pile groups for piles cap rested on soil is less than that for pile

cap non-rested on soil. Fair agreement has been found between finite element analysis and experimental test result.

Keywords: Scour, skin friction, depth, conical, ultimate capacity, settlement, rested, non-rested

1. INTRODUCTION

The flow of water in rivers and streams causes erosion and scour around piled foundations. Scour may cause significant effect on the bearing capacity, settlement as well as load distribution (skin friction and end bearing) of the pile foundation. If the foundation embedment into the ground is not sufficient to account for erosion and scour that may occur over the life of the building, the building is vulnerable to collapse as shown in figure (1). Erosion and scour increase the un braced length of pile foundations and increase the bending moment to which they are subjected, and can overstress piles to be more susceptible to settlement as shown in fig (2). Thus, it is necessary to study this phenomenon and the performance of the pile-raft foundation cap directly rested on soil and performance of the pile-raft non-rested cap on soil.

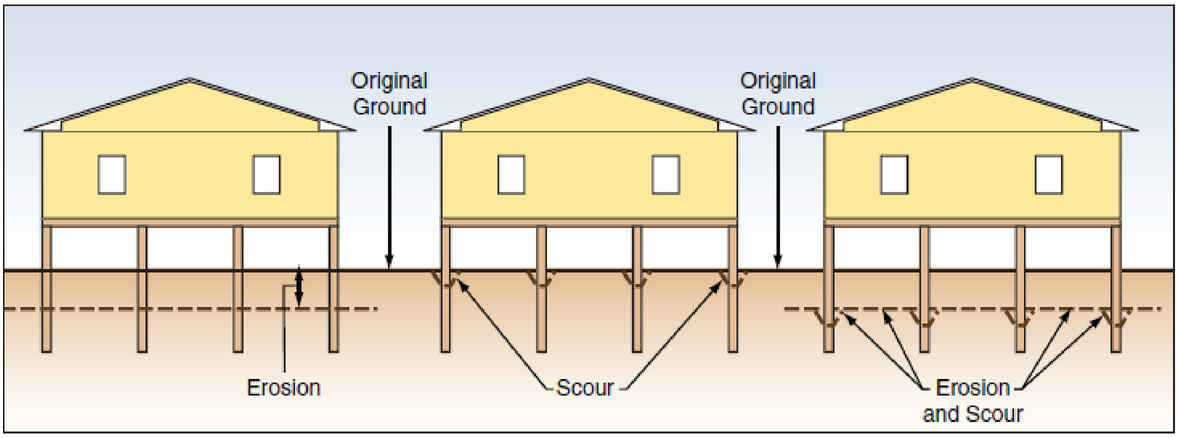


Figure (1) Distinguishing between coastal erosion and scour. A building may be subject to either or both.

Figure (2) Local scour damage at piles

Liang F.Y. (2011) studied the influence of scour on the ultimate capacity of the piled foundation by finite element analysis. The effects of scour depths on the presentation of piled foundations were analyzed. The numerical results showed that scour has negative effects on the ultimate capacity of the piles [1].

Eslami. A .et.al. (2012) presented an analysis of two and three dimensional finite element connected and non-connected pile-raft systems. The analyses include the investigation of the effect of different parameters such as piles spacing, embedment length, piling configuration and raft thickness. It was concluded that non-connected piled-raft systems can significantly reduce the settlements and raft internal bending moments by increasing the subsoil stratum stiffness [2].

Erion. P. and Yavuz Y. (2012) investigated the influence of the scour on pile bent load carrying capacity. Analysis results show that pile bents due to scouring height have decreased the load carrying capacity varying from 17.64 to 32.11% [3].

Jae H. P. et al. (2012) calculated scour bridges founded on pile foundations registered in the National Highway Bridge List of the capital district. The case studies consist of site study by boring test. Three of 12 pile foundation bridges showed the possible future susceptibility to scour with considerable reduction in the embedded depth of foundations due to scour [4].

Al- kadi, O. A. (2012) used the sake of validating software to estimate of different input parameters needed for modeling of different pile group system. Pile load test was performed on ALZEY Bridge in Germany. The effects of piles spacing and length to diameter ratio were studied [5].

Patil. J.D. (2013) presented the performance of piled raft foundation in sandy soil, clayey soil and, layered soil. Either experimentally or theoretically has been conducted on the behavior of piled raft foundation. A number of 3D numerical models have been developed but no effort is found to evolve analytical method based on numerical methods. Analytical methods were stated only to access the settlement of the piled raft foundation [6].

Ningombam T. S. and Baleshwar S. (2013) studied the behavior of piled raft in clay soils with the load sharing and pile spacing as well as pile length. However, raft if thickness increases has no significant effect on load sharing in soft soil. There is a significant effect of pile spacing on the behavior of piled raft. However, it was advised to keep the pile in larger spacing than the smaller one [7].

Cheng Lin (2017) investigated vertical soil stress at the pile under changing scour-hole dimension conditions. An analytical solution founded on Boussinesq's theory was developed for the stress distribution. The effects of scour-hole dimensions counting scour depth, width, and slope angles on the vertical soil stress distribution were discussed[8].

Elsamny M. K. et al. (2017-a) investigated the ultimate capacity, settlement and efficiency of pile groups in sandy soil. An experimental program was conducted to study the group efficiency. However, the experimental program consisted of testing single pile, pile groups of two, three and four piles in sand under axial compression load. The spacing between piles was kept three diameters of piles. The pile head loads, displacement, strains along the pile shaft were measured simultaneously. The obtained test results indicated that the ultimate capacity of single pile inside pile groups increases with increasing number of piles. However, the settlement of pile groups at the ultimate load was found to be more than that of the settlement of single pile. In addition, it was found that group efficiency of pile groups (2,3 and 4 piles) increases with increasing number of piles. However, for number of piles in pile group more than four no significant increase has been obtained. In addition, the group efficiency was found to be ranging from 1.25 -1.47 as by using chin method (1970) for the determination of ultimate capacity of piles [9].

Elsamny.M. K. et al. (2017-b) investigated the load shearing between soil and pile raft in cohesionless soil. An experimental program was conducted to study the distribution of applying loads. The experimental program consisted of testing single pile, pile groups of two, three, four, five and six piles in sand under axial compression load. It was found that the percentage of the transferred load of single pile at pile tip =13.5% from the ultimate capacity. Also, for pile groups (2, 3, 4, 5 and 6 piles) it was found that the percentage of loads transferred to the soil underneath the pile caps = 0.88 to 1.10 % from the ultimate bearing capacity. In addition, for pile groups (2, 3, 4, 5 and 6 piles) it was found that the percentage of loads transferred to the soil at pile tip = 4.20 to 2.53 % and transferred to the soil by friction = 94.70 to 96.59 % from the ultimate bearing capacity [10].

2. EXPERIMENTAL PROGRAM:

The experimental program was conducted to study effect of load shearing underneath the pile caps of pile groups and soil. The piles were tested in a setup under compressive axial loads. Load at pile tip, strains along the piles as well as the pile head loads were measured simultaneously. Furthermore, the load under pile cap transferred directly by pile cap to soil has been measured. The program consisted of installing test piles in dense sand, placing piles in a soil chamber subjected to compressive axial load. However, three groups of testing were performed in axial compression. First group load test was carried out on single pile. Second group was four pile caps rested on soil. Third group was four pile caps non-rested on soil. A nine-precast concrete cylindrical piles of 0.15 m diameter and 1.50 m length were fabricated the details of which are as follows in the test program:

- Group (1) – Single pile as shown in Fig (3).
- Group (2) – Group of four piles rested directly on soil as shown in Fig (4).
- Group (3) – Group of four piles non-rested on soil (clear distance=7.5cm under pile cap) as shown in Fig (5).

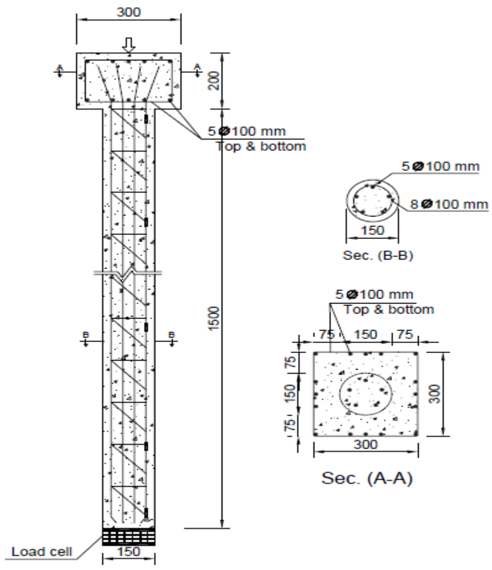


Fig (3) Concrete dimension and reinforcement, group (1) – single pile

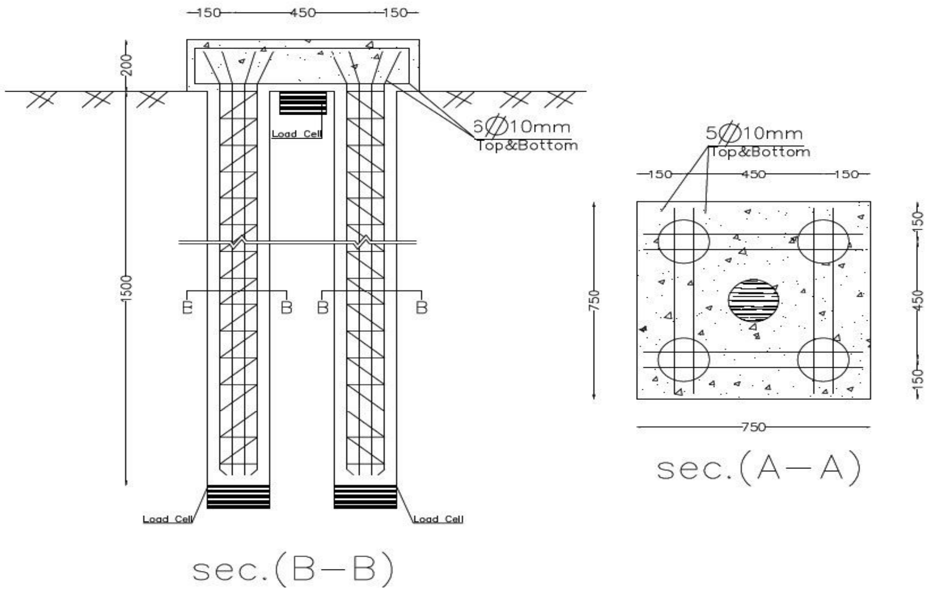


Fig (4) Concrete dimension and reinforcement, group (2) – group of Four pile cap directly rested on soil

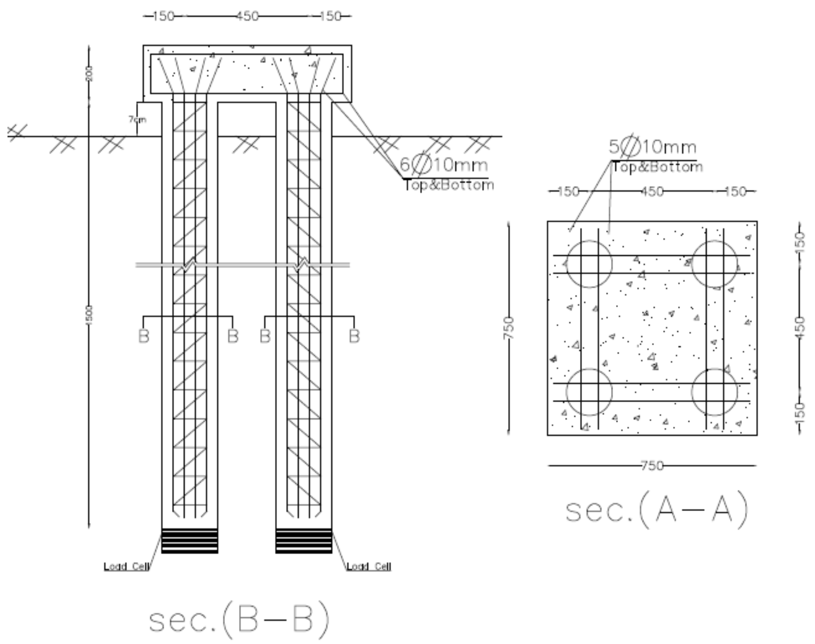


Fig (5) Concrete dimension and reinforcement, group (3) – group of Four pile cap non-rested on soil

2.1. Strain Gauges

The strain gauges were used on the longitudinal steel reinforcement for internal measurements. The strain gauges used were manufactured by TOKYO SOKKI KENKYUJO CO. LYD.

2.3 Piles Casting

All cylindrical piles were casted in tubes (forms) are shown in Figures (6) to (7). A mechanical vibrator was used, and all cylindrical piles were cured.

Fig (6) Pile reinforcement details

Fig (7) Tubes (forms) cylindrical pile

2.4. Theoretical Ultimate Capacity of Piles

Before execution of piles, estimation of pile load capacity is done by theoretical formula. The theoretical pile capacities have been calculated by using Egyptian code (2001) for single pile. The calculated theoretical ultimate capacity of single pile Qu=30 KN .and the calculated theoretical ultimate capacity of four pile rested on soil, non-rested on soil Qu=120 KN.

2.5 Testing of Pile

Three pile load tests were performed according to Egyptian code. However, Pile groups were tested as shown in Table (1).

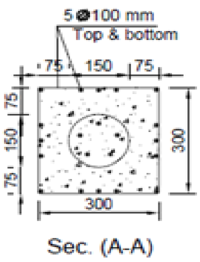
Table(1) Tested pile groups.

Test No	Theoretical Ultimate load (Kn)	Test load kN	Pile diameter (m)	Pile length (m)	No of pile
Group (1) single pile	30	1.50*30=45	0.150	1.50	1
Group (2) Four pile cap directly rested on soil	120	1.75*120=210			4
Group (3) Four pile cap non-rested on soil	120	1.75*120=210			4

In this present study three pile load tests were performed. The reaction load was performed by a system of jacking bearing against dead load of a loading frame. Loading frame was manufactured to resist the expected maximum loads that might occur during the test as shown in Figure (8). A hydraulic jack system comprising a 100 kN jack, was used in the test as shown in Figures (9). The load was measured at underneath the pile caps and the tip of pile by an 800 kN load cells connected to the data acquisition system as shown in Figures (10) and (11). In the present study all of three groups were loaded in twelve increments according to Egyptian code (2001). Settlement of the piles was measured by dial gauges. Each load increment was maintained till settlement rate was observed less than 0.25 mm per hour. However, load cells were placed at the tip of piles and underneath the pile cap to measure the transferred load to soil. In addition, strains readings along pile shaft were recorded. Tables (2) and (3) show the load increments in the test.

Table(2) Increment of load and interval time for group (1) according to Egyptian code.

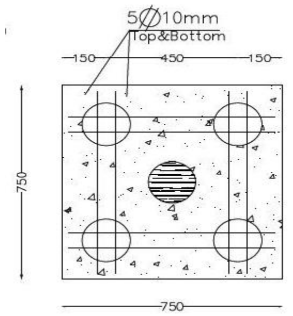
	Load%	Time	Load (kn)
Loading	25	1hr	7.5
	50	1hr	15
	75	1hr	22.5
	100	3hr	30
	125	3hr	37
	150	12hr	45
Un loading	Load%	Time	Load(kn)
	125	.25hr	37
	100	.25hr	30
	75	.25hr	22.5
	50	.25hr	15
	25	.25hr	7.5
	0	4hr	0



Sec. (A-A)

Table(3) Increment of load and interval time for groups (2) and (3) according to Egyptian code.

	Load %	Time	Load(kn)
loading	25	1hr	40
	50	1 hr	80
	75	1hr	120
	100	3 hr	160
	125	3 hr	200
	150	3 hr	240
	175	12hr	280
Un loding	Load %	Time	Load KN
	150	.25hr	240
	125	.25hr	200
	100	.25hr	160
	75	.25hr	120
	50	.25hr	80
	25	.25hr	40
	0	4hr	0



sec.(A—A)


Fig (8) Loading frame

Fig (9) Loading Jack

Fig (10) Data acquisition system Fig (11) load cell

2.5.1 PILE LOAD TEST GROUP (1) – SINGLE PILE.

The pile was surrounded in the sand such that the total embedment depth of the pile was 150 cm after filling the soil chamber with 15cm compacted layers of sand using mechanical compactor as shown in Figure (12). Furthermore, the vertical displacements of the pile cap were measured by four dial gauges with accuracy of 0.01 mm as shown in Figure (13). The measurements of load at top of pile were recorded using dial gauge at the top. Dial gauges readings were taken for each loading increment for settlement measurement. However, load cell was placed at the tip of pile to measure the transferred load. In addition, strains readings along pile shaft were recorded.

Fig (12) Placing compacted soil around tested pile
For group (1) – single pile

Fig (13) Dial gauges' setup for group (1) (single pile)

2.5.2 PILE LOAD TEST OF GROUP (2) – FOUR PILES CAP DIRECTLY RESTED ON SOIL.

The piles were surrounded by compacted sand such that the total embedment depth of the piles was 150 cm after filling the soil chamber with 15cm compacted layers of sand using mechanical compactor as shown in Figure (14). Furthermore, the vertical displacements of the pile cap were measured by four dial gauges with accuracy of 0.01 mm as shown in Figure (15). The measurements of load at top of piles were recorded using four dial gauges at the top. Dial gauges readings were taken for each loading increment for settlement measurement. However, load cells were placed at the tip of piles and underneath the pile cap to measure the transferred load. In addition, strains readings along pile shaft were recorded.

Figure(14) Placing compacted soil around tested piles for group (2)- four pile cap directly rested on soil

Figure (15) Dial gauges' setup for group (2) – four piles cap directly rested on soil and loading jack.

5.2.3. PILE LOAD TEAT OF GROUP (3) FOUR PILE CAP NON-RESTED ON SOIL.

The cap non-rested on soil has spacing under pile cap .5D =7.5cm. The piles were surrounded by compacted sand such that the total embedment depth of the piles was 150 cm after filling the soil chamber with 15cm compacted layers of sand using mechanical compactor as shown in Figure (16). Furthermore, the vertical displacements of the pile cap were measured by four dial gauges with accuracy of 0.01 mm as shown in Figure (17). The measurements of load at top of piles were recorded using four dial gauges at the top. Dial gauges readings were taken for each loading increment for settlement measurement. However, load cells were placed at the tip of piles and underneath the pile cap to measure the transferred load. In addition, strains readings along pile shaft were recorded.

Figure(16) Placing compacted soil around tested piles for group (3)- four pile cap non-rested on soil.

Figure (17) Dial gauges' setup for group (3) – four piles cap non-rested on soil and loading jack.

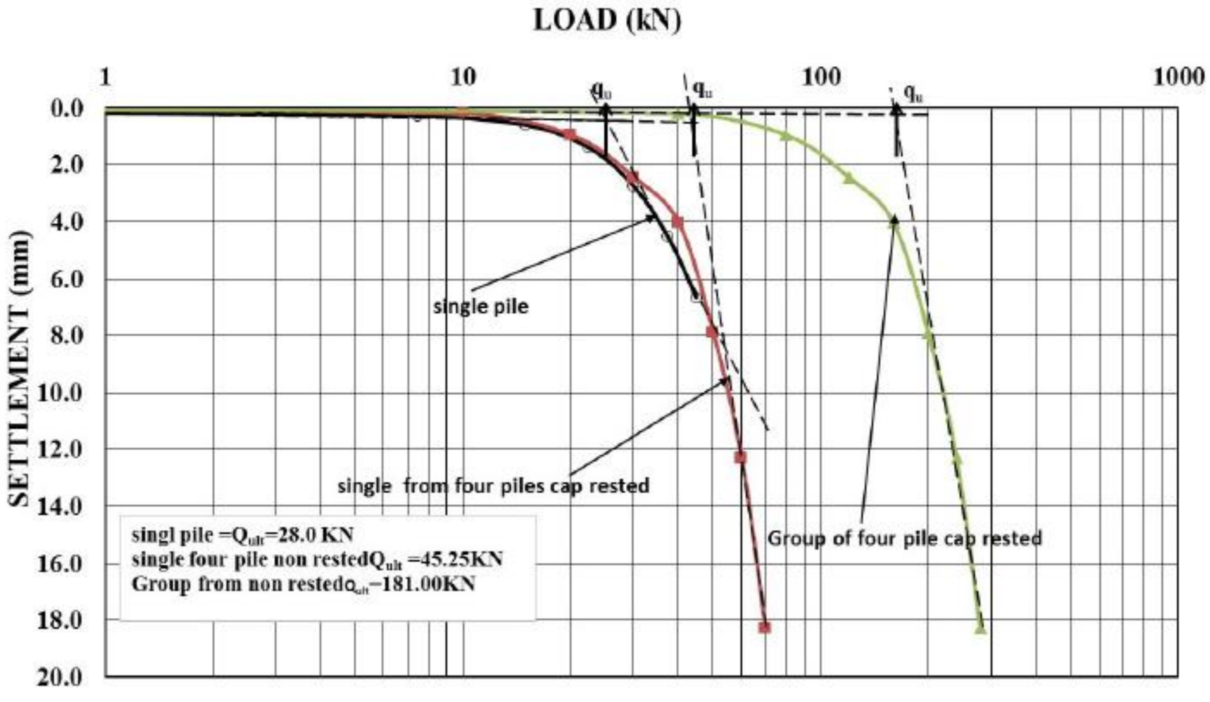
6. Determination of Ultimate Capacity of Piled-Raft

6.1. Determination the ultimate pile load capacity

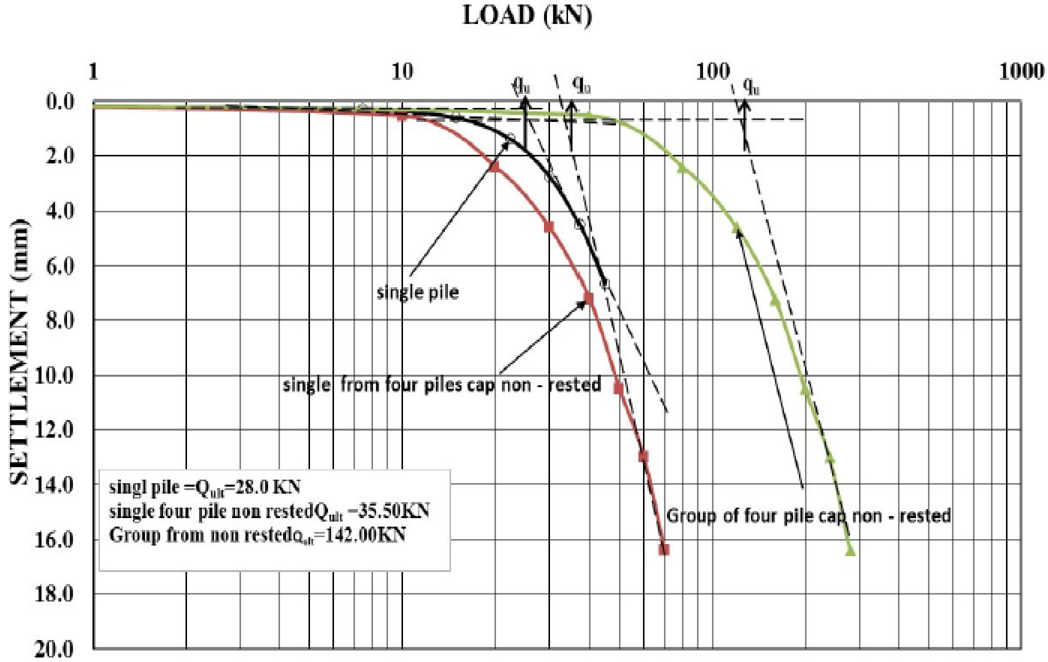
Determination the ultimate pile load capacity has been done by using Modified Chin Method (1970) and Tangent– Tangent Method as follows:

6.1.1. Excremental results for group (1) – single pile, group (2) – four pile cap directly rested on soil and group (3) – four piles cap non-rested on soil.

The ultimate load capacity for single pile, group (2) – four pile cap directly rested on soil and group (3) – four piles cap non-rested on soil was determined by the slope modified chin method and the tangent-tangent method from load settlement readings at the point of intersection of the initial and final tangents of the load settlement curve. A comparison between ultimate capacities of piles for single pile group (1) and single pile inside groups (2) and (3) from tangent-tangent method is shown in Figures (18) and (19). A comparison between ultimate capacities of piles for single pile group (1) and single pile inside groups (2) and (3) from Modified Chin method as shown in Figures (20) and (21). The ultimate load for single pile calculated in this study was determined by different theoretical approaches. The values of ultimate capacities and ultimate capacities of single pile and single pile inside groups from different methods are listed in Table (4).



Figure(18). Comparison between ultimate capacities of piles for single pile group (1) and single pile inside groups group (2)–four piles cap rested on soil from Tangent-Tangent Method.



Figure(19) Comparison between ultimate capacities of piles for single pile group (1) and single pile inside groups group (2)–four piles cap non-rested on soil from Tangent - Tangent Method.

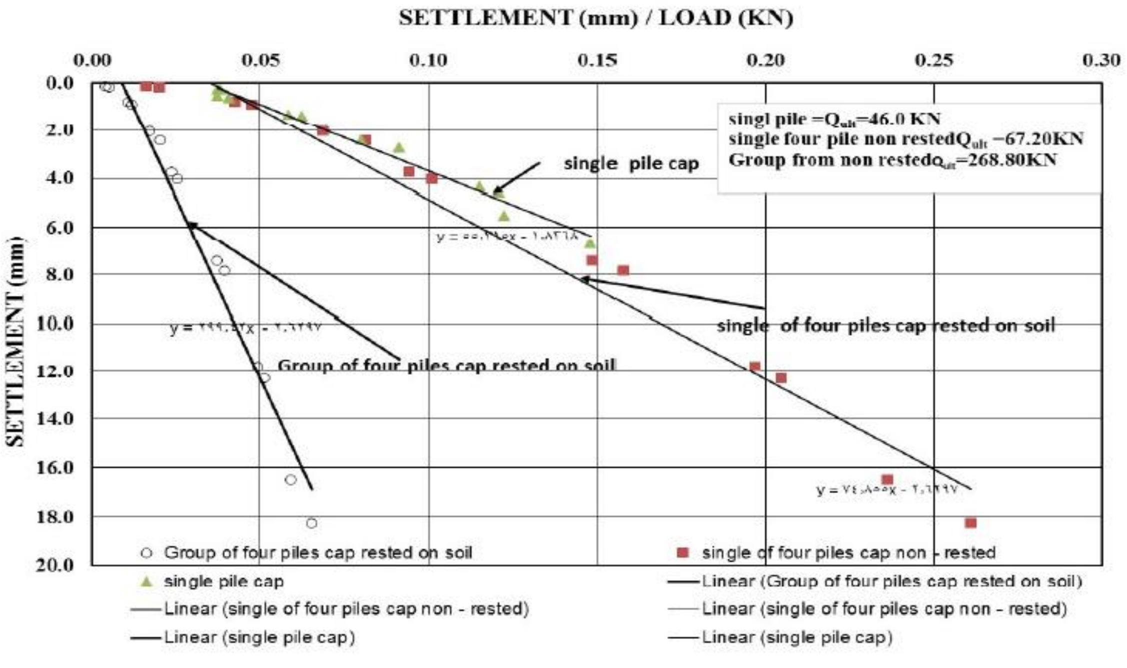


Figure-20**.** Comparison between ultimate capacities of piles for single pile group (1) and single pile inside groups group (2)–four piles cap rested on soil from Modified chin method.

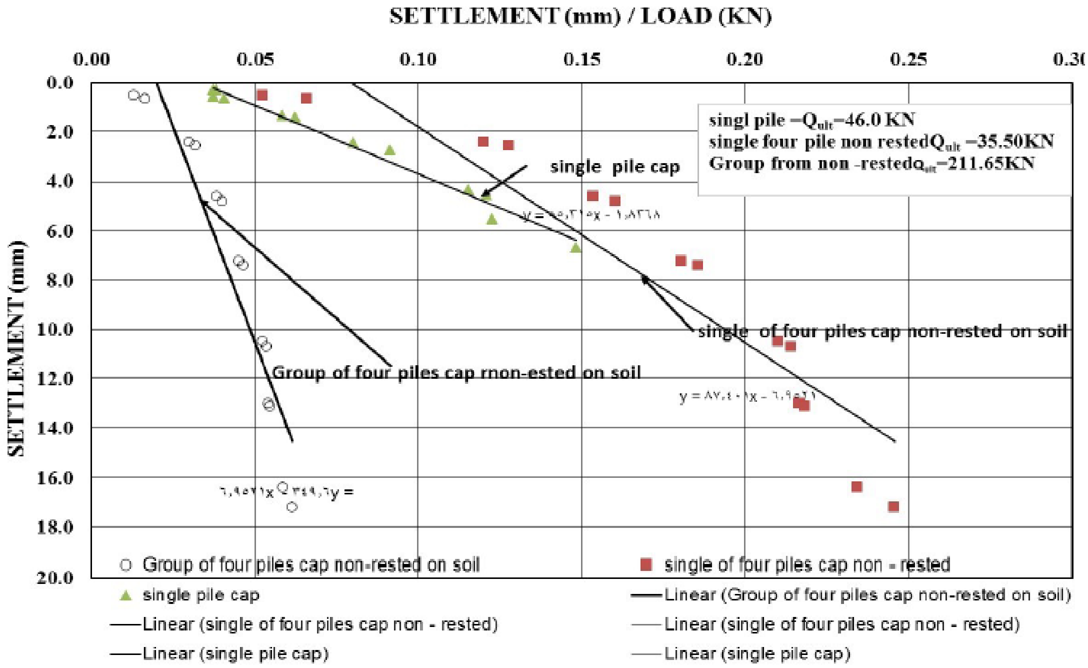


Figure (21) Comparison between ultimate capacities of piles for single pile group (1) and single pile inside groups group (2)–four piles group cap non-rested on soil from Modified Chin method.

Table (4) Ultimate capacities and ultimate capacities of single pile and single pile inside groups by theoretical methods and experimental methods.

Grou	Ultimate load (Qult) from theoretical Methods (kN)	Ultimate load (Qult) from pile load test (kN)		Ultimate capacities of single pile and single pile inside groups (Qult) from pile load test (kN)	
	Egyptian code (2001)	Tangent method (1991)	Modified Chin method (1970)	Tangent-Tangent method (1991)	Modified Chin method (1970)
Single pile	30.00	28.00	46.00	28.00	46.00
Four piles cap rested	120.00	181.00	268.80	45.25	67.20
Four piles cap non-rested	120.00	142.00	211.65	35.50	52.75

6.2. Experimental results for group (1) single pile, group (2) – four pile cap directly rested on soil and group (3) – four piles cap non-rested on soil.

The ultimate load capacity for group (2)–four piles cap rested on soil was determined by the modified chin method and the tangent-tangent as shown in Figure (22).

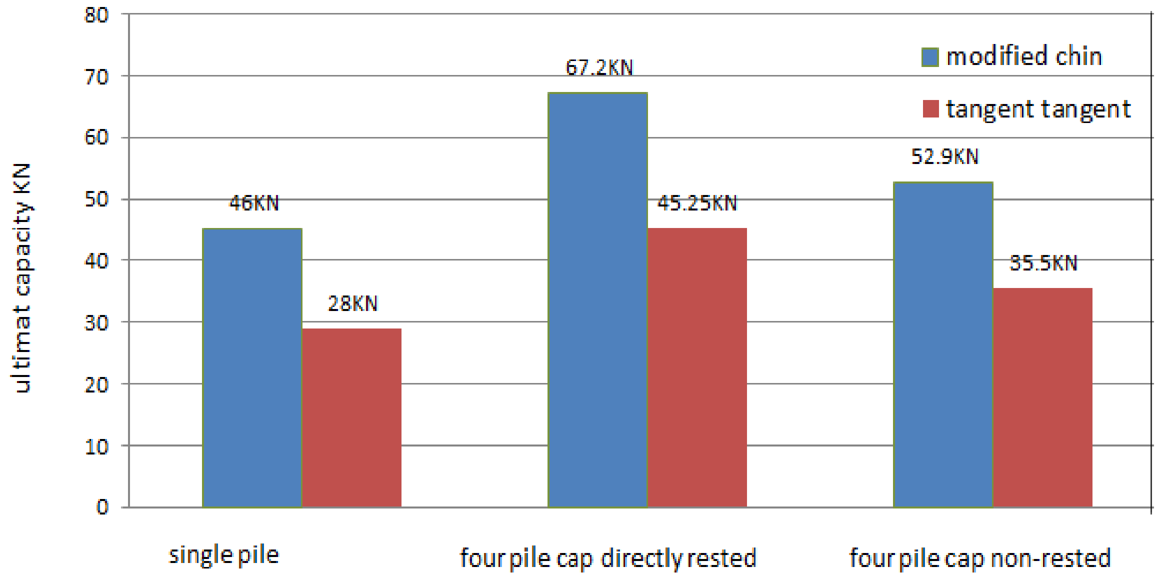
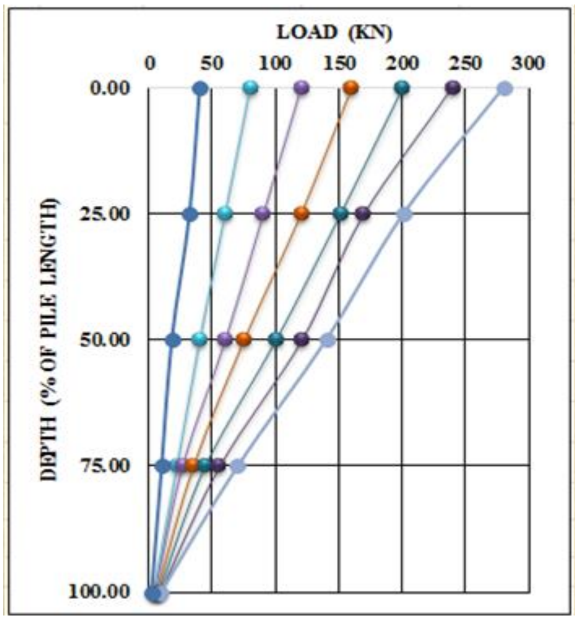


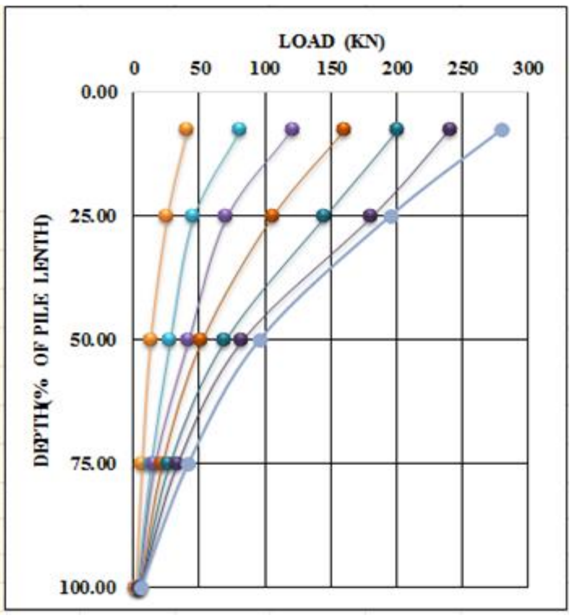
Fig (22) Comparison between the ultimate capacities for single pile, single pile inside groups of four pile rested and four pile non rested by Tangent-Tangent method and modified Chin method (1970)

6.3. Experimental results for group (1) single pile, group (2) – four pile cap directly rested on soil and group (3) – four piles cap non-rested on soil.

Comparison between distribution of load at pile tip from load cell measurements and along pile shaft measured from strain gauges measurements group (2) – (four piles cap directly rested on soil) and group (3) as well as (four piles cap non-rested on soil) is shown in Figs (23a)to (23b).



Figure(23a) Distribution of load at pile tip from load cell and along pile shaft measured from strain gauges group (2) – (four pile cap directly rested on soil)



Figure(23b) Distribution of load at pile tip from load cell and along pile shaft measured from strain gauges group (3) – (four pile cap non-rested on soil)

7. Settlement of the pile groups

The Settlement at ultimate capacities of single pile and pile groups rested and non-rested based on the results of Tangent-Tangent Method and Modified Chin method are shown in Table (5) and Figure (24).

Table (5) Settlement at ultimate capacities of single pile and pile groups rested and non-rested based on the results of Tangent-Tangent Method and Modified Chin method.

Group	Settlement (mm)	
	Modified Chin	Tangent- Tangent
Single pile	6.17	2
Four pile rested	13.96	4.25
Four pile non rested	16.65	6.15

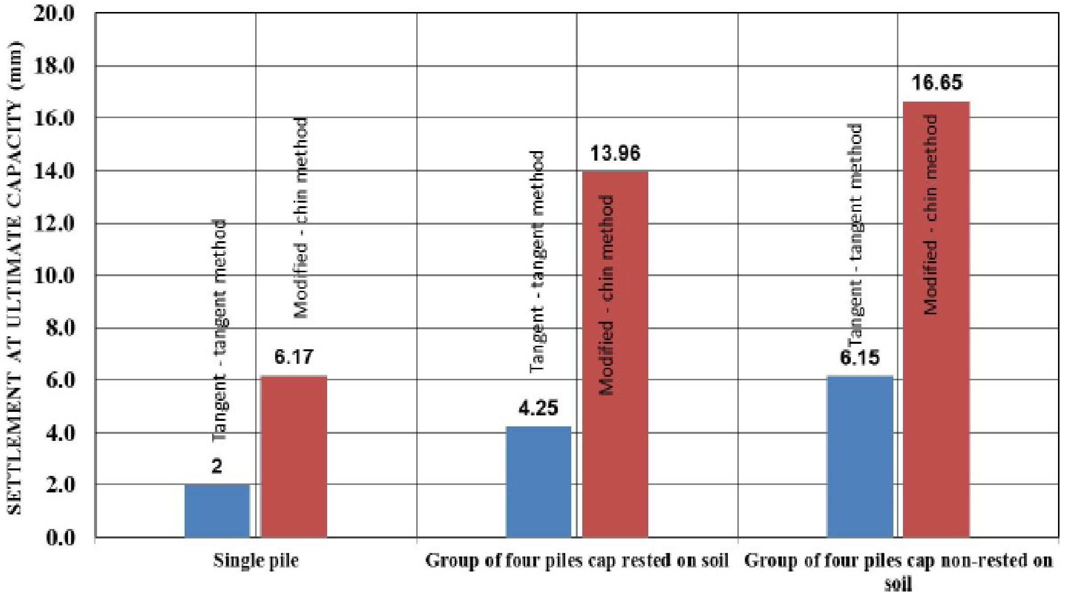


Fig (24) Comparison between settlement for single pile group (1) and single inside pile group (2) - four piles cap directly rested as well as group (3)–four piles cap non-rested obtained from loading tests by from Tangent-Tangent Method (1991) and Modified Chin method 1970).

8. FINITE ELEMNT ANALYSIS.

In the present study finite element analysis was used for single pile and pile groups. The analysis is done by using 3D Plaxis program in which the soil is simulated by a semi-infinite element isotropic homogeneous elastic material. A model with a fixed yield surface as perfectly-plastic is assumed. The stress states are assumed purely elastic. The Mohr-Coulomb model in used. These parameters with their standard units are listed below:

E: Young's modulus [kN/m2] =20x106kN/m2 - ν: Poisson's ratio [-] = 0.2 -

Φ: Friction angle [°] = 36° - C: Cohesion [kN/m2] =0 - α: Dilatancy angle [°] =6.
Using this model of soil, the stress relationship of soil is linear and elastic.
Figures (25) to (30) show 3D deformed mesh and vertical displacement for group (1) – group of single pile, group (2) four pile cap directly rested on soil and group (3) four pile cap non-rested on soil.

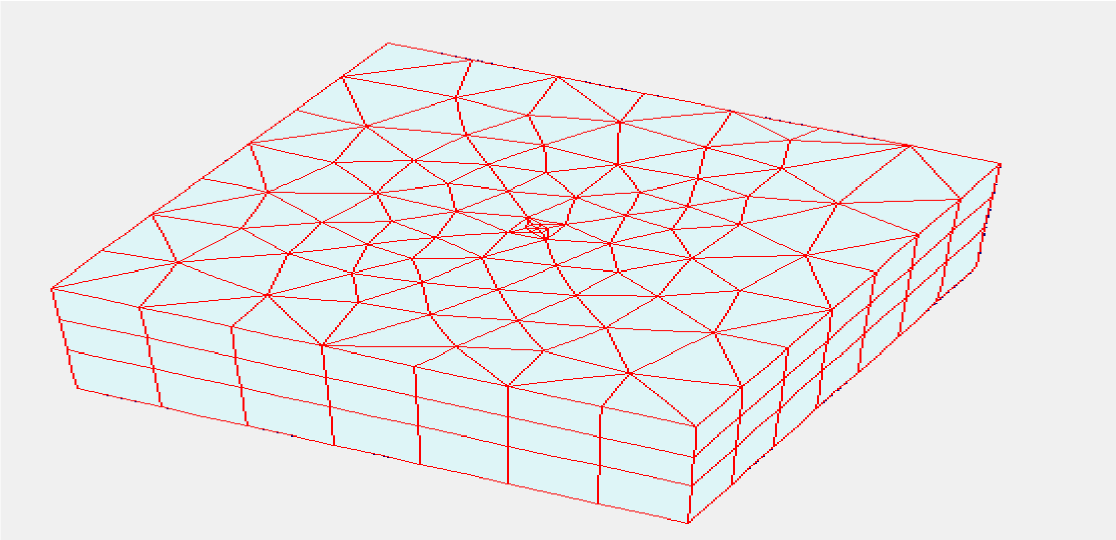


Fig (25) 3D deformed mesh for group (1) – group of single pile

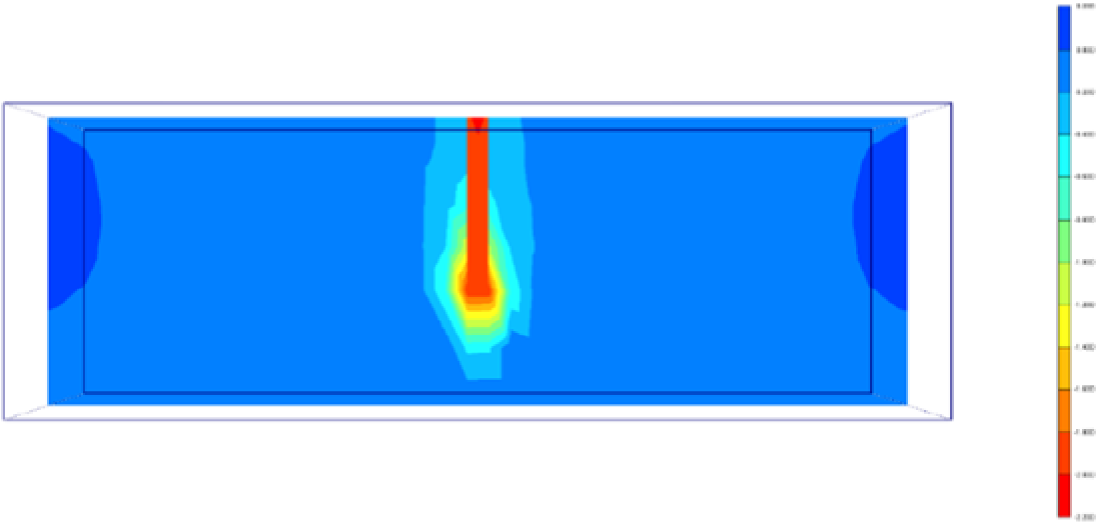


Fig (26)Vertical displacement for group (1) – group of single pile.

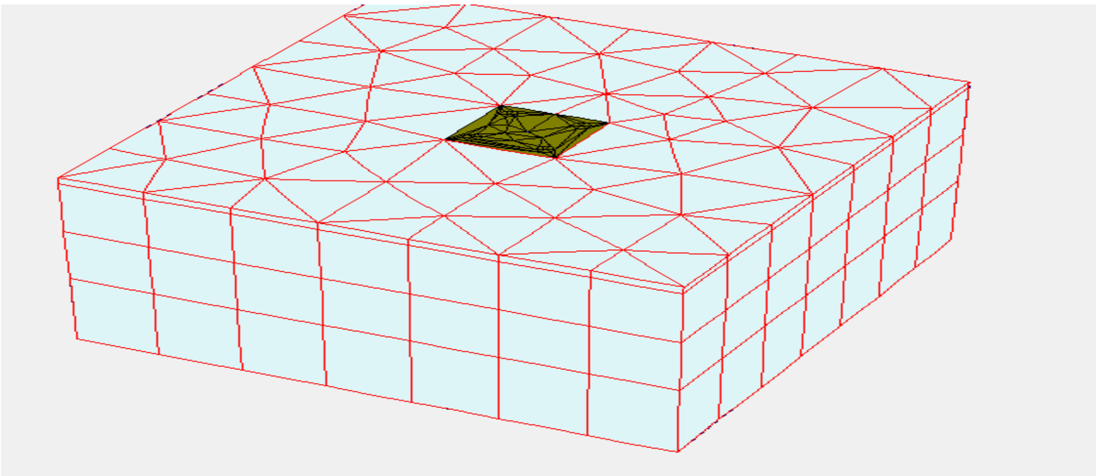


Fig (27) 3D deformed mesh for group (2) – group of four pile cap directly rested on soil.

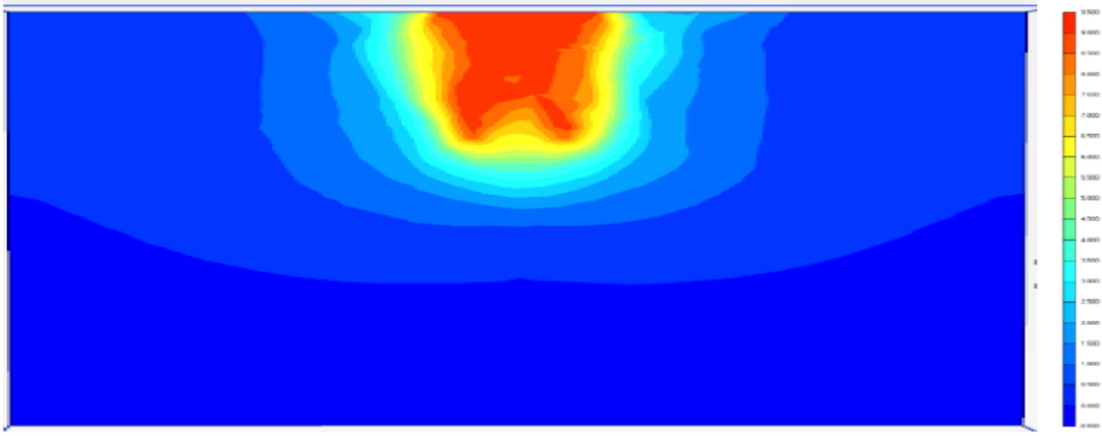


Fig (28) Vertical displacement for group (2) – group of four pile cap directly rested on soil.

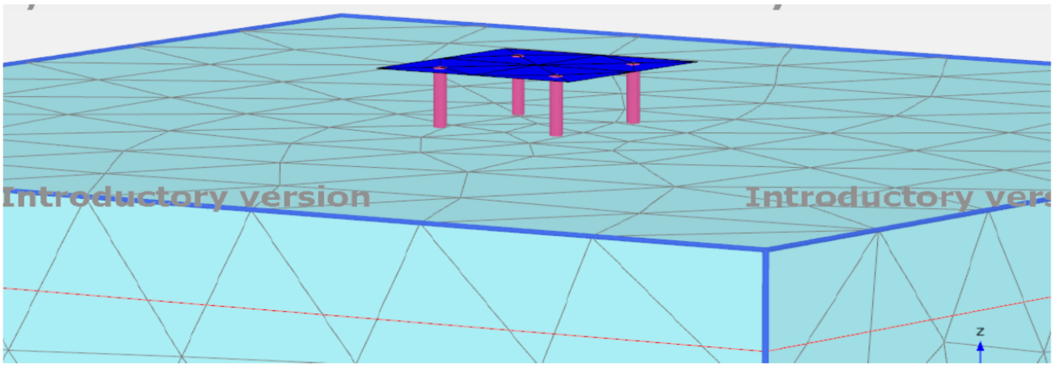


Fig (29) 3D deformed mesh for group (3) –group of four piles cap non-rested on soil.

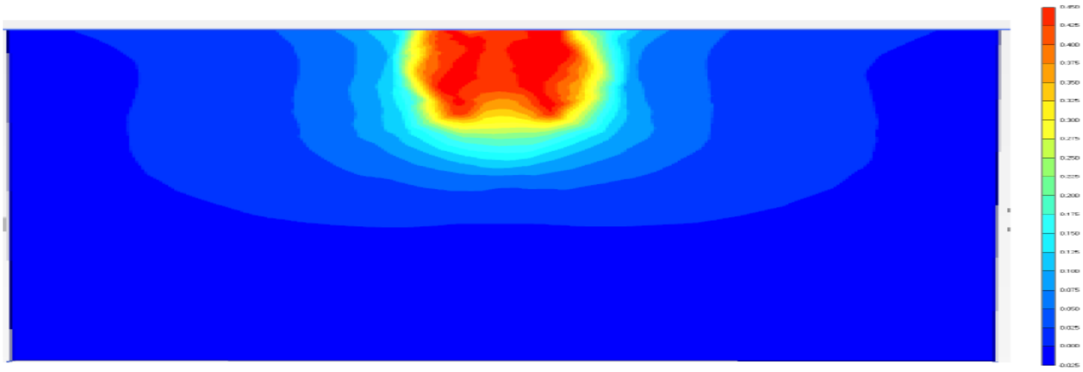


Fig (30) Vertical displacement for group (3) group of four piles cap non- rested on soil.

9. Comparison between the Settlement at Ultimate Capacities of piles by Finite Element Method and Those Obtained from Pile Load Tests.

In the present study a comparison between settlement for single pile group (1) and single inside pile group (2) (four piles cap directly rested) as well as group (3) (four piles cap non-rested) obtained from loading tests from tangent - tangent Method and Modified Chin method and finite element analysis is shown in Table (6) and figures (31) to (32).

Table (6) Settlement at ultimate capacities of single pile and four pile cap directly rested on soil and four pile cap non-rested on soil based on the results of tangent-tangent Method and Modified chin method and by using finite element analysis.

GROUP	Settlement (mm)		
	Finite element analysis	Tangent-tangent Method	Modified chin method (1970)
Group(1) Single pile	1.85	2.00	6.17
Group (2) - four piles cap rested	3.13	4.25	13.96
Group (3) - four piles cap non-rested	4.01	6.15	16.65

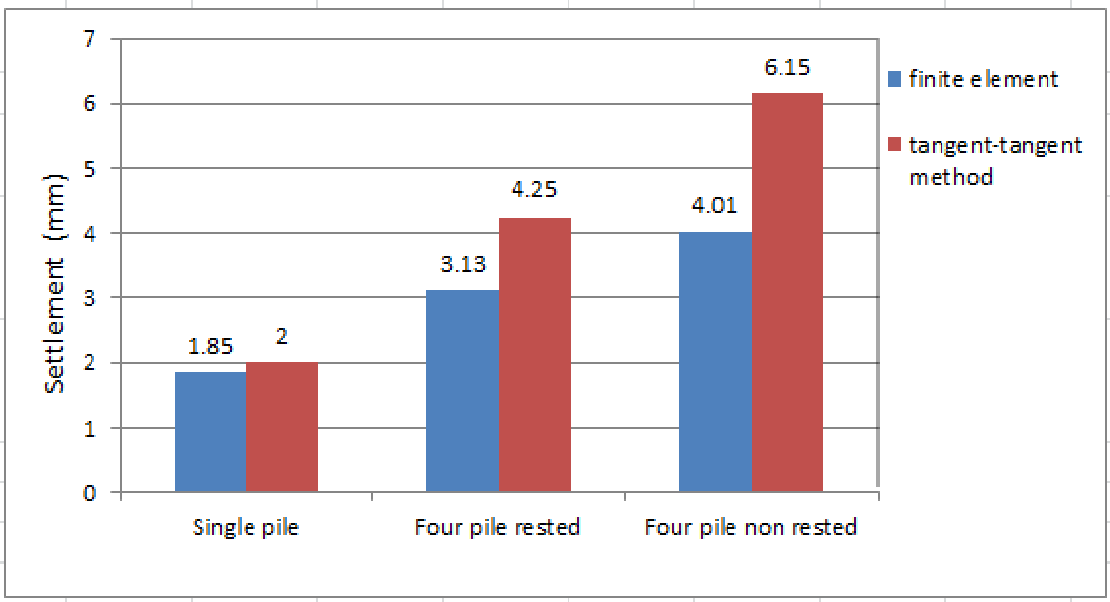


Fig (31) Comparison between the settlement at ultimate capacity by finite element and that obtained by and tangent-tangent method for group (1) single pile and group (2) for pile directly rested on soil.group(3) for pile cap non-rested on soil .

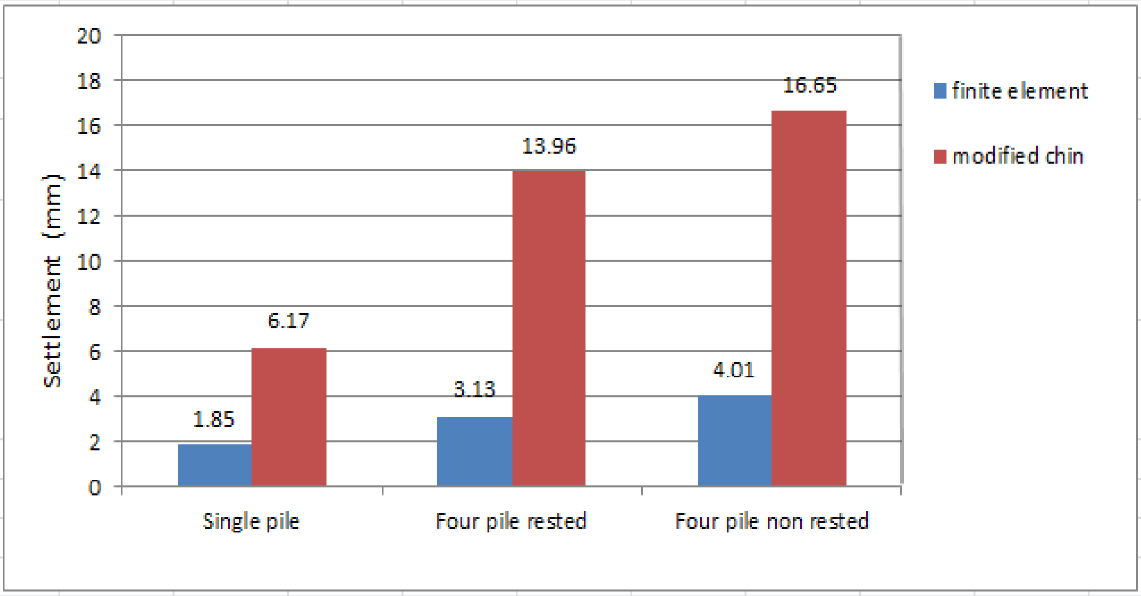


Fig (32) Comparison between the settlement at ultimate capacity by finite element and that obtained by and modified chin method for group (1) single pile and group (2) for pile rested on soil.group(3) for pile non-rested on soil .

10. EFFECT OF SCOUR ON PILED RAFT SHAFT RESISTANCE.

The flow of water in rivers and streams causes scour around piled foundations. Scour damage at piled foundation is called local scour as shown in Fig (33). A conical hole is then formed at the upper soil surface, which is usually called conical local scour as shown in Fig (34). Fig (35) shows the influence of local scour on the effective stresses acting on pile.

Fig (33) Local scour damage at piled foundation.

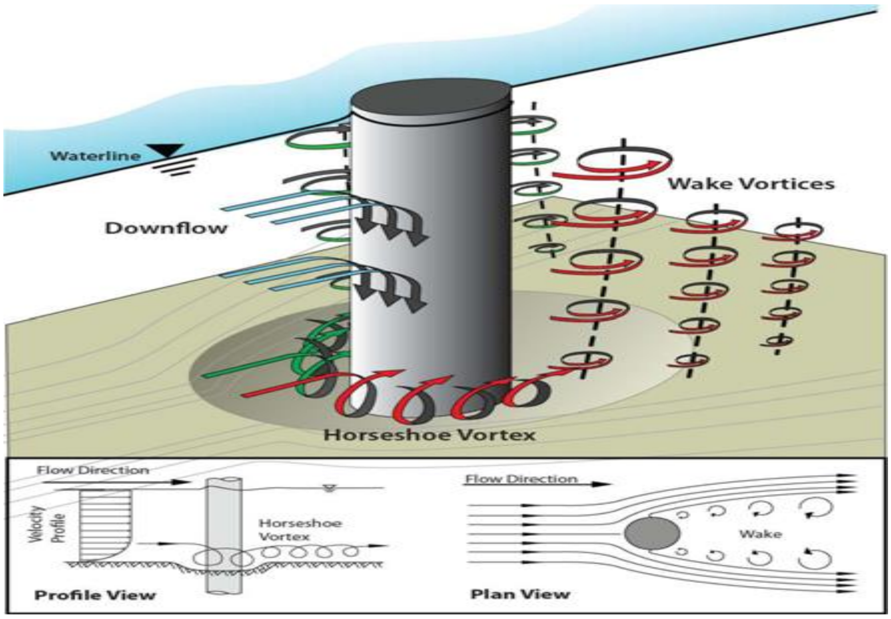


Fig (34) Diagram shows the formation of a local scour hole around a pile.

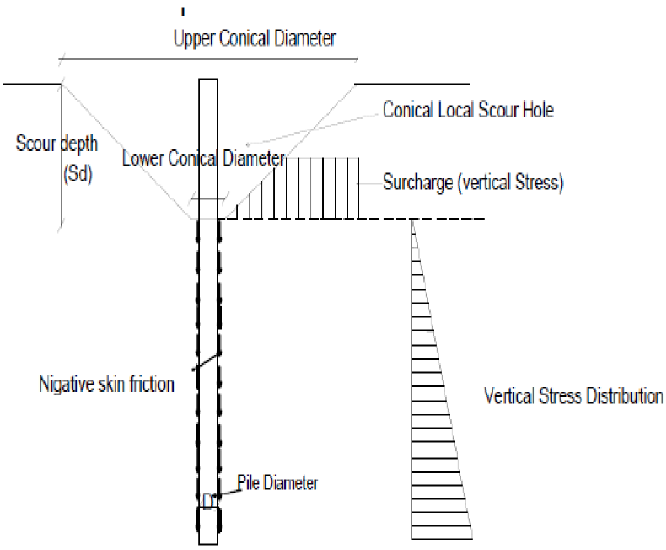


Fig (35) The influence of conical scour on effective stress.

11. Effect of Scour Depth on Friction around Shaft of Pile for the experimental program (dimension L=1.5m&D=.15m).

A finite element model was developed to predict the effect of scour depth on friction around shaft of piles (negative skin friction and positive skin friction) of the pile foundation, In addition conical local scour hole causes negative skin friction (additional vertical stress) along pile shaft.

Fig (36) and table (7) shows the relationship between net skin friction around shaft of piles and effective pile depth. From these fingers, it can be concluded that a decrease in the embedded depth of pile of foundation due to scour decreases skin friction.

Table (7) Comparison between friction (shaft) along pile with scour depth of four pile non-rested on soil. L=1.5m& D=.15m

Scour Depth(m)	Friction around (shaft) of pile (kN)							
	Effective pile length							
	0.1m	0.3m	0.5m	0.7m	0.9m	1.1m	1.3m	1.5m
S=.5D	0.52	36.596	54.846	73.096	91.346	109.596	125	139
S=1D	-	30.25	48.23	65.125	80.23	98	115	132
S=1.5D	-	19.63	39.68	56.98	72.65	91.65	109.68	126.5
S=2D	-	-	26.87	45.254	65.89	85	104.6	121.5
S=2.5D	-	-	20	40.58	60.2	80.35	100.56	116
S=3D	-	-	21.58	32.5	51.23	72.25	97.56	110.5

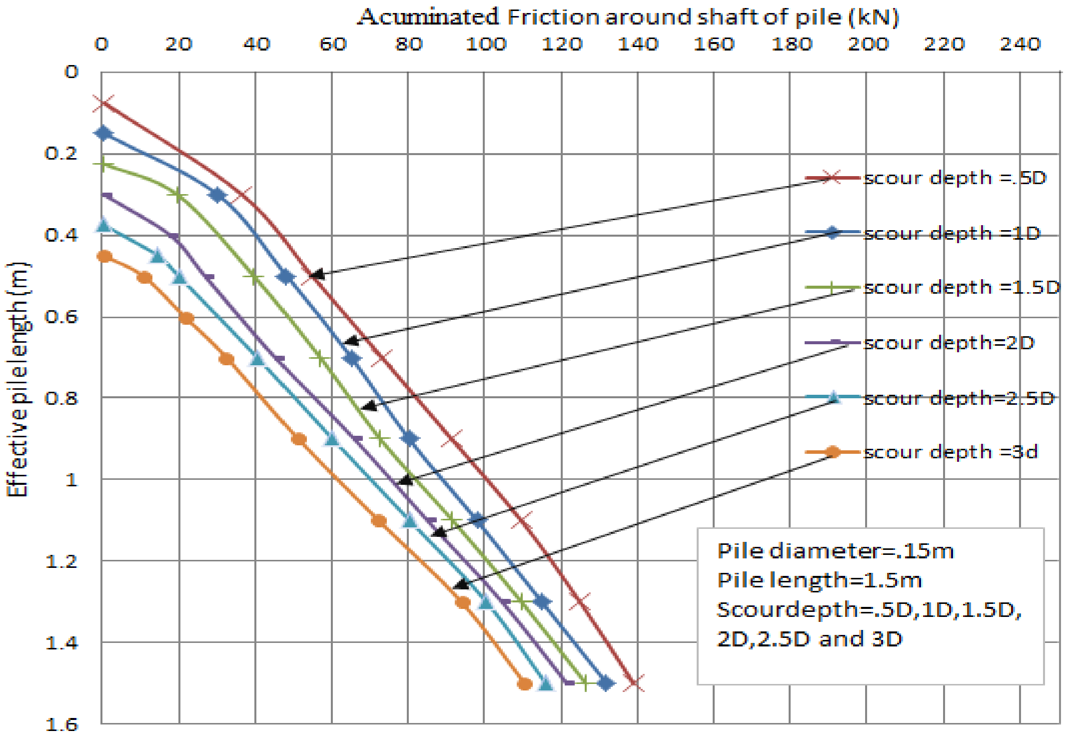


Fig (36) The relationship between the net skin friction around shaft of piles and effective pile depth for different pile diameters. L=1.5m& D=.15

13. Comparison between bearing capacity, Settlement and End bearing according to result of scour by using finite element method. (Dimension L=1.5m& D=.15m).

The settlement and scour depth for four piles cap non-rested on soil is shown in Table (5) & in Fig (37).

Fig (38) illustrates a comparison between ultimate capacities and scour depth, end bearing and scour depth for four pile cap non-rested on soil according to the table (8).

Table (8) Comparison between Settlement at ultimate capacities and ultimate capacity group of four pile cap non-rested on soil using finite element analysis. L=1.5m& D=.15m.

Scour depth (D)	Settlement (mm)	Ultimate Capacity(kN)	End bearing (kN)
.5D	9.14	165.218	26.28
1D	11.53	157.5	25.5
1.5D	11.75	151.7	25.2
2D	13.36	146.1	24.6
2.5D	15.20	140.2	24.2
3D	25.40	134.5	24

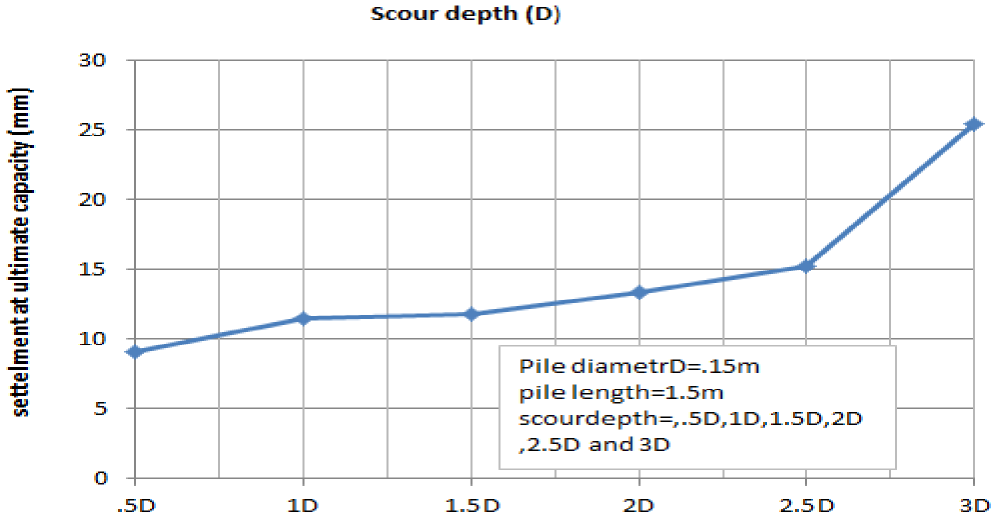


Figure (37) Relationship between the settlement at ultimate capacity and scour depth by finite element. L=1.5m&D=.15m

i. It can be shown that settlement increases with increasing scour depth.

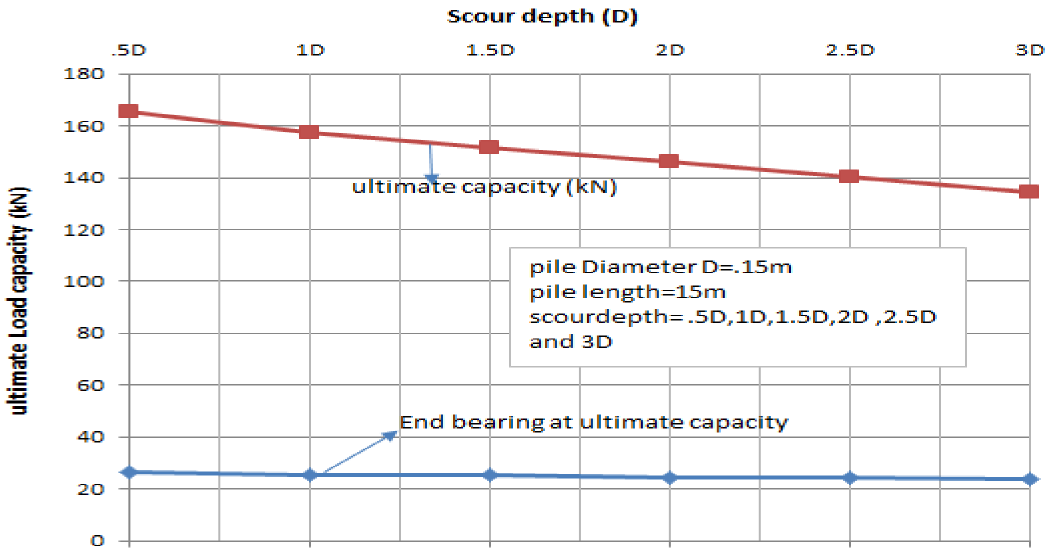


Figure (38) Relationship between ultimate capacity and scour depth and end bearing scour depth by finite element
L=1.5m&D=.15m for from pile group non-rested on soil

From the above the following on obtained:

i. The values of ultimate load capacities of piles foundation decrease with increases scour depths.
ii. The values of transferred loads at pile tip (end bearing) decrease with increasing scour depth.

14. Effect of Scour Depth on Friction around Shaft of Pile in different soil

A finite element model was developed to predict the effect of scour depth on friction around shaft of piles (negative skin friction and positive skin friction) of the pile foundation in different soil.

14.1. FINITE ELEMENT ANALYSIS.

The finite element method was selected in the present study to develop numerical models to study the effect of scour depth on ultimate capacity, settlement as well as load distribution (skin friction and end bearing) of the pile foundation. The finite element analysis has been performed using software PLAXIS 3D2014. The soil is simulated by isotropic homogeneous semi-infinite elastic material.

14.2. Numerical Program.

A numerical analysis was conducted to study the effect of scour depth on ultimate capacity, settlement as well as load distribution (skin friction and end bearing) of the pile foundation. Fig (39) illustrates a borehole for the selected investigated site in the present study. The analysis program consists of group of a pile cap with pile diameter =0.60m. And length 15m. The analysis has been done with different scour depths of 0.00 D 0.50 D, 1.00 D, 2.00 D, 3.00 D and 4.00 D where D = pile diameter. The analysis program is shown in Table (6). Soil material properties are shown in Table (7)

Table (6) analysis program

Group No	Scour Depth(m)	Pile length(m)	Pile diameter(m)
1	0.0D	L=15m	.6m
2	.5D		
3	1D		
4	2D		
5	3D		
6	4D		

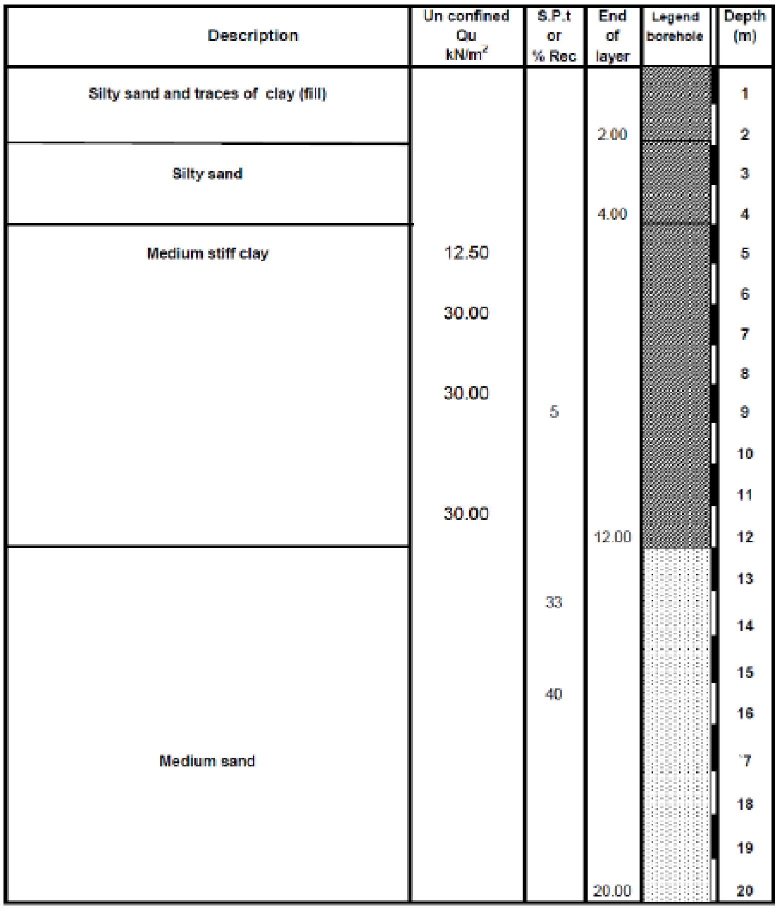


Fig (39) Borehole log for soil used in El- Maady, Cairo, Egypt project

Table(7) Soil material properties

Parameters	Thickness (m)	Description	Densityγ (KN/m^2)	Es (KN/m^2)	Poisson ratio, u	Cohesion Cu (KN/m^2)	Friction angle
Layer1	2.0	Silty sand of and traces clay(fill)	15.0	2000	0.3	0	0
Layer2	2.0	Silty sand	16.6	4000	0.4	12.5	0
Layer3	8.0	Medium to stiff clay	17.0	3000	0.3	25	0
Layer4	8.0	Medium sand	18.0	15000	0.25	0	33

15. Effect of Scour Depth on Settlement

Ultimate capacity and pile tip resistance and shaft friction as well as settlement at different scour depths are listed in Tables (8) to (10). Fig (40) shows the relationship between the values of settlement at ultimate load capacities and scour depth.

Table (8) Settlement at ultimate capacity at different scour depths

Pile diameter(D)	Settlement at ultimate capacity (mm) at scour depth s=					
	S=0.0D	S=.5D	S=1D	S=2D	S=3D	S=4D
D=0.6m	7.56	8.08	8.65	9.30	10.05	10.90

Table (9) Ultimate capacities at different scourdepth.L=15m&D=.6m

Pile diameter(D)	Ultimate capacities (kN) and end bearing at scour depth S=					
	S=0.0D	S=.5D	S=1D	S=2	S=3D	S=4D
D=0.60m	1551.08	1380.46	1242.42	1134.5	1034.5	951.74
D=0.60m	Pile tip resistance	Pile tip resistance	Pile tip resistance	Pile tip resistance	Pile tip resistance	Pile tip resistan ce
	1016.08	918.18	848.55	796.74	7.61.18	724.03

Table (10) Pile friction resistance at different scour depths. .L=15m&D=.6m

Pilediameter(D)	Pile friction resistance (kN)at scour depth s=					
	S=0.00D	S=0.5D	S=1D	S=2D	S=3D	S=4D
	Pile friction resistance	Pile friction resistance	Pile friction resistance	Pile friction resistance	Pile friction resistance	Pile friction resistance
D=0.60m	535.00	462.28	393.87	333.86	273.32	227.71

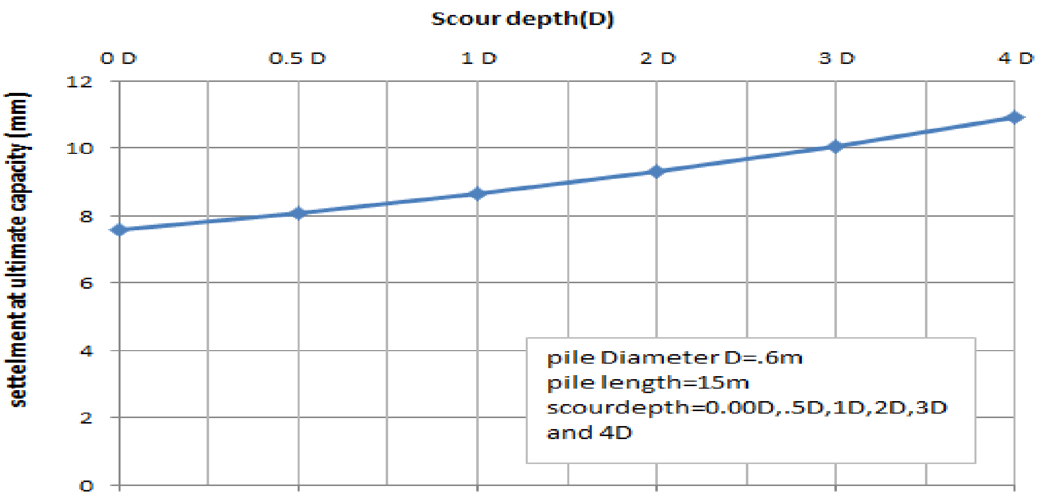


Fig (40) The relationship between scour depth and settlement at ultimate load capacities for pile diameter =.6m and L=15m.

From the above the following on obtained

i. The settlement of piled foundation increases with increasing scour depths.

16. Effect of Scour Depth on the Ultimate Pile Load Capacity and End Bearing Resistance of Pile

The ultimate load capacities obtained from the numerical analysis at different scour depths have been obtained. Fig (41) shows the relationship between the values of ultimate load capacities and scour depths and the end bearings (at pile tip) at ultimate capacities.

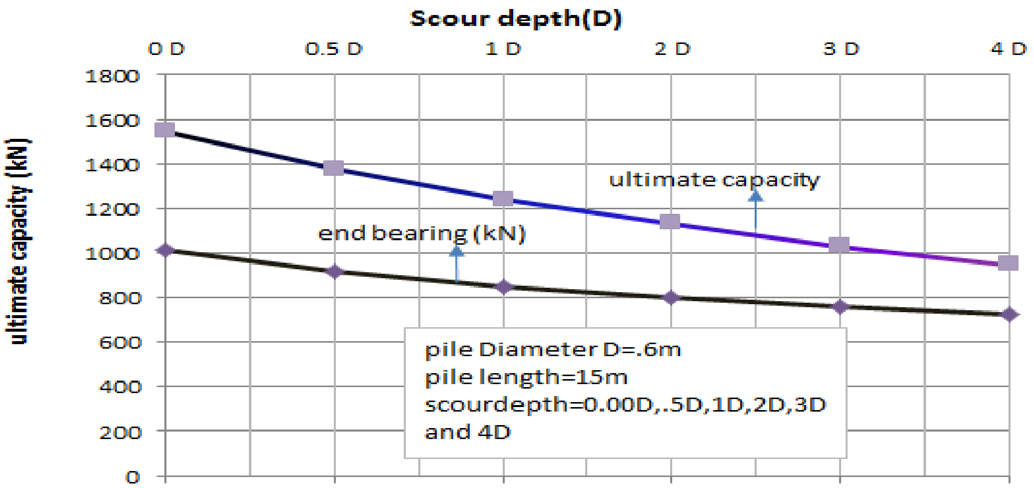


Fig (41) Relationship between ultimate capacity and end bearing at ultimate capacity and scour depth for pile diameter D=.6m and length L=15m

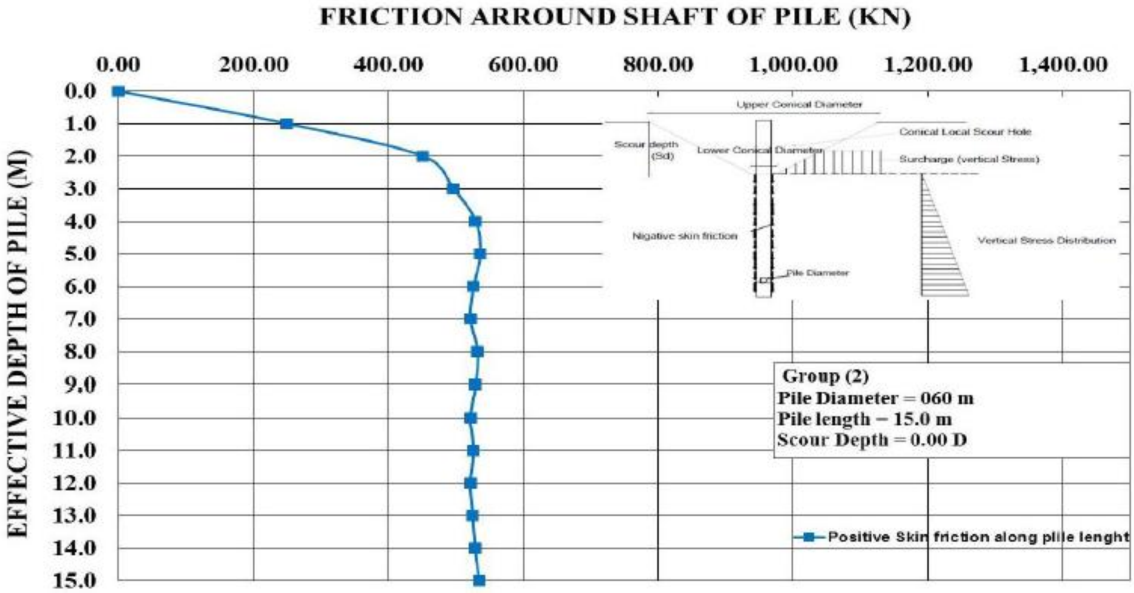
From the above the following on obtained:

i. The values of ultimate load capacities of piles foundation decrease with increasing scour depth.

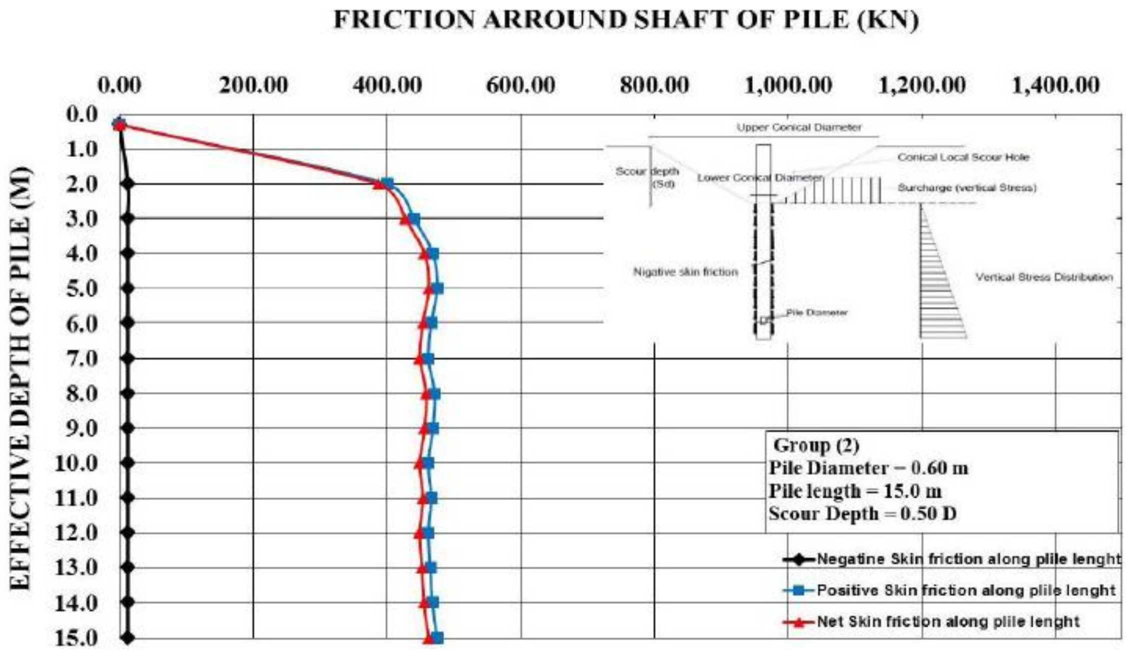
ii. The values of transferred loads to pile tip (end bearing) decrease with increasing scour depth

17. Effect of Scour Depth on Friction around Shaft of Pile

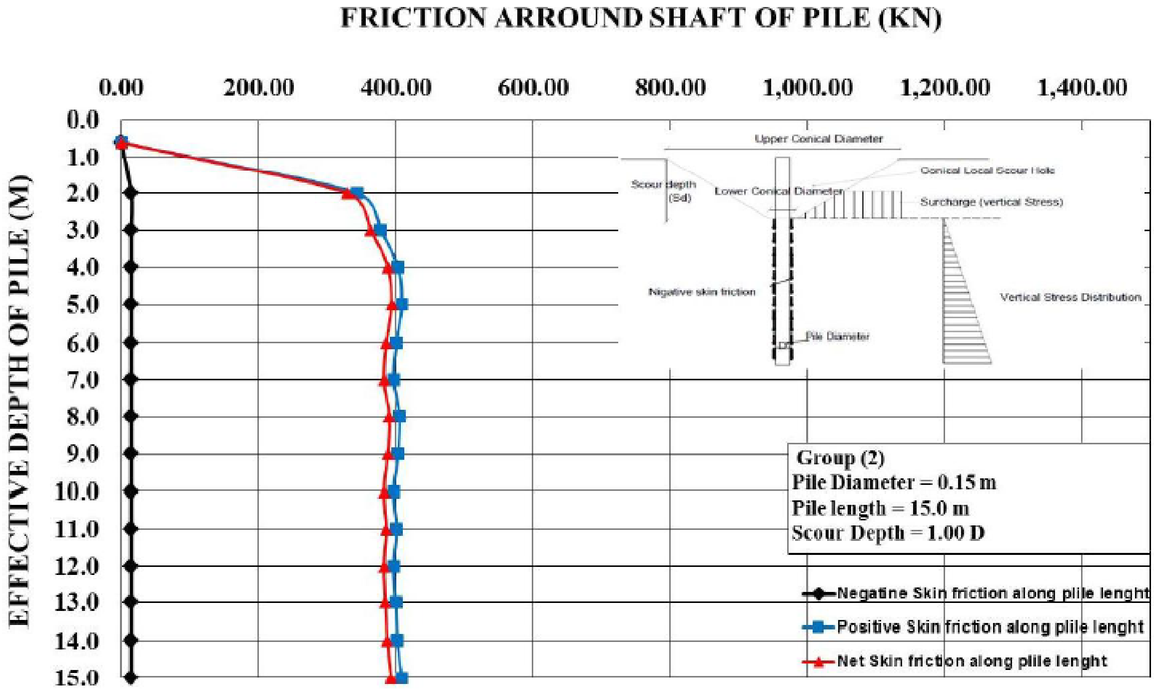
A finite element model was developed to predict the effect of scour depth on friction around shaft of piles (negative skin friction and net skin friction) of the pile foundation. In addition, conical local scour hole causes negative skin friction (additional vertical stress) along pile shaft. Figs (42) to (47) show some examples of the relationships between friction around shaft of piles (negative and net) and effective pile depth. Fig (48) shows the relationship between net skin friction around shaft of piles and effective pile depth. From these figures, it is concluded that, a decrease in the embedded depth of pile of foundation due to scour decreases skin friction.



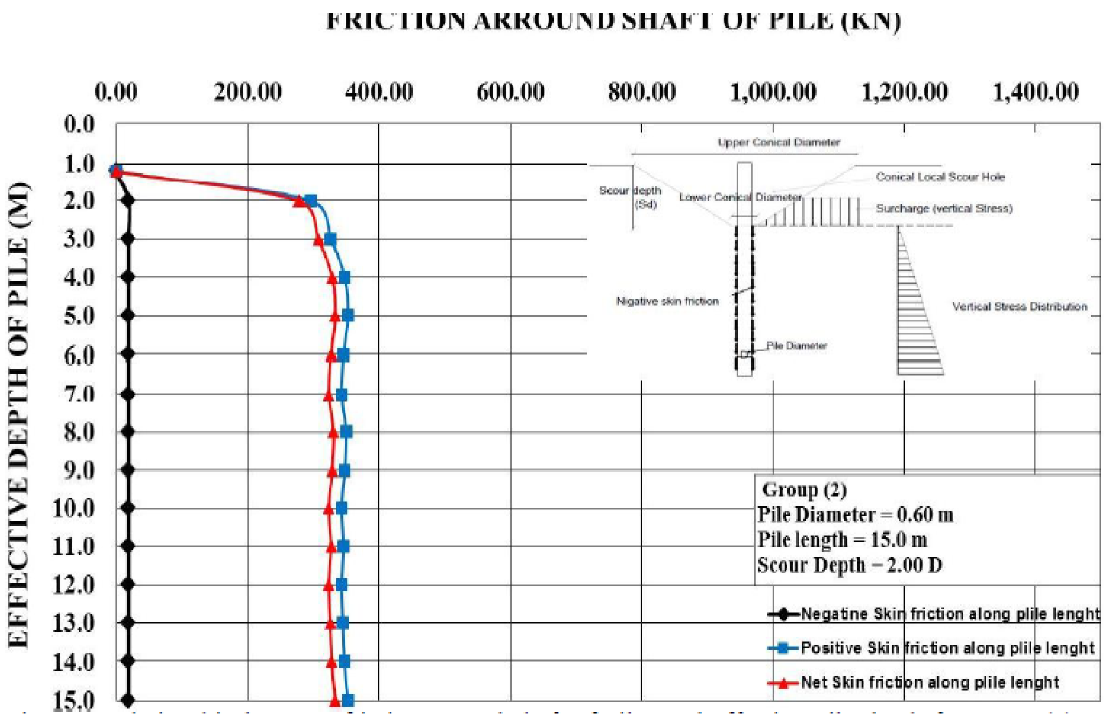
Figs (42) Relationship between friction around shaft of piles and effective pile depth for pile diameter =0.60 m at scour depth=0.0 D.



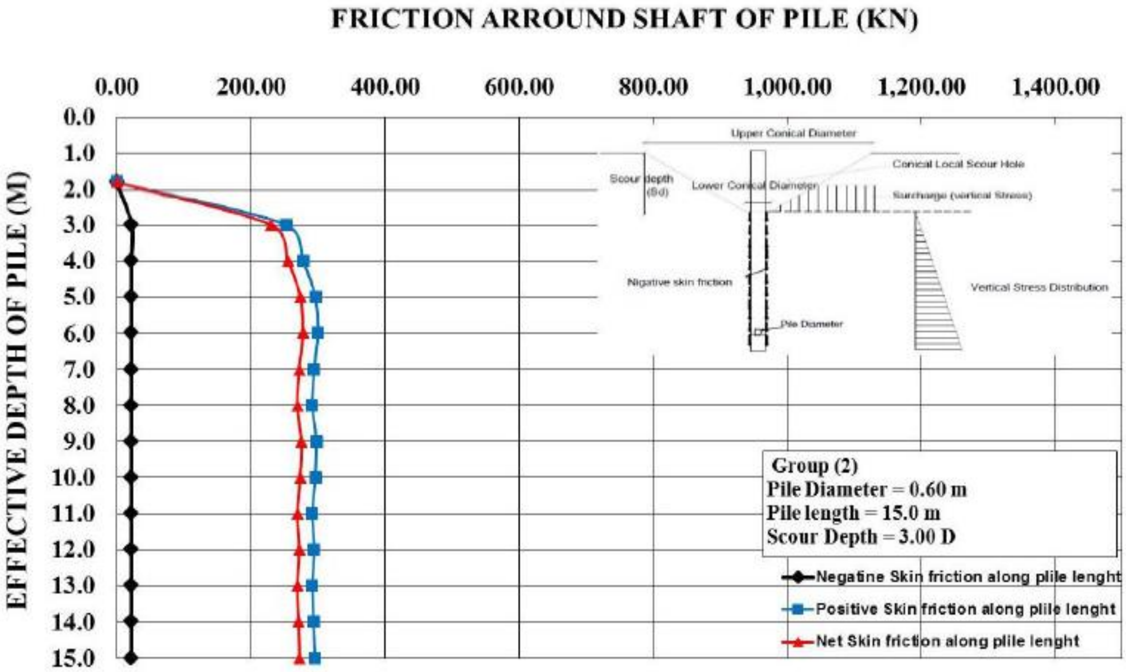
Figs (43) Relationship between friction around shaft of piles and effective pile depth for pile diameter =0.60 m at scour depth=0.5 D.



Figs (44) Relationship between friction around shaft of piles and effective pile depth for pile diameter =0.60 m at scour depth=1D



Figs (45) Relationship between friction around shaft of piles and effective pile depth for pile diameter =0.60 m at scour depth=2.0 D.



Figs (46) Relationship between friction around shaft of piles and effective pile depth for pile diameter =0.60 m at scour depth=3.0 D.

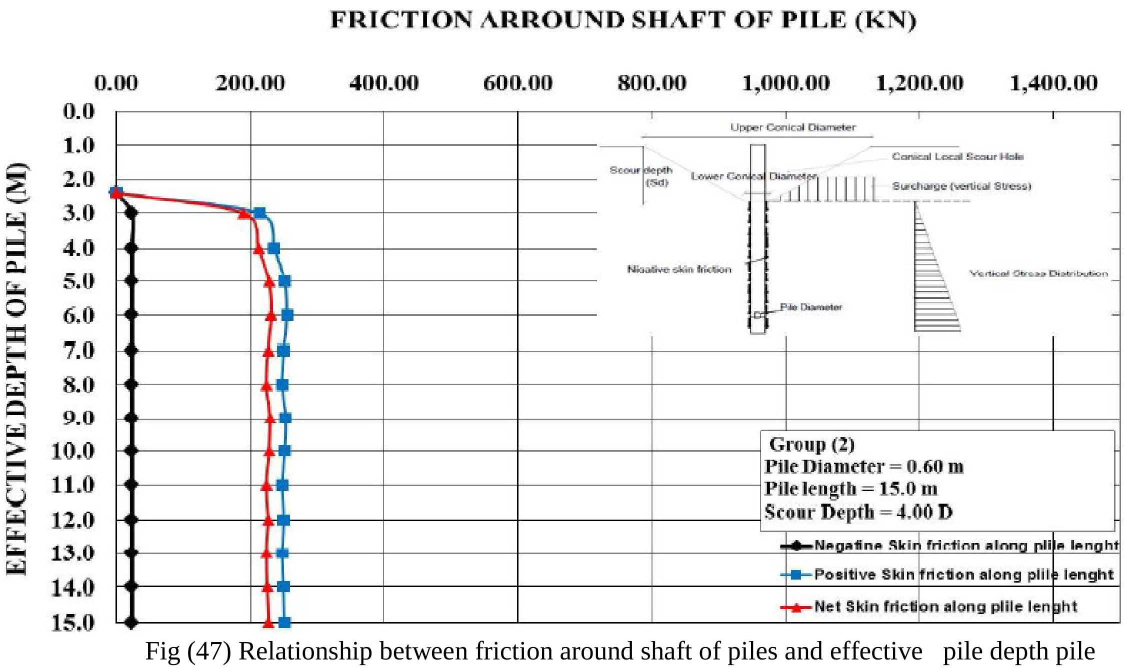


Fig (47) Relationship between friction around shaft of piles and effective pile depth pile diameter =0.60 m at scour depth=4.0 D.

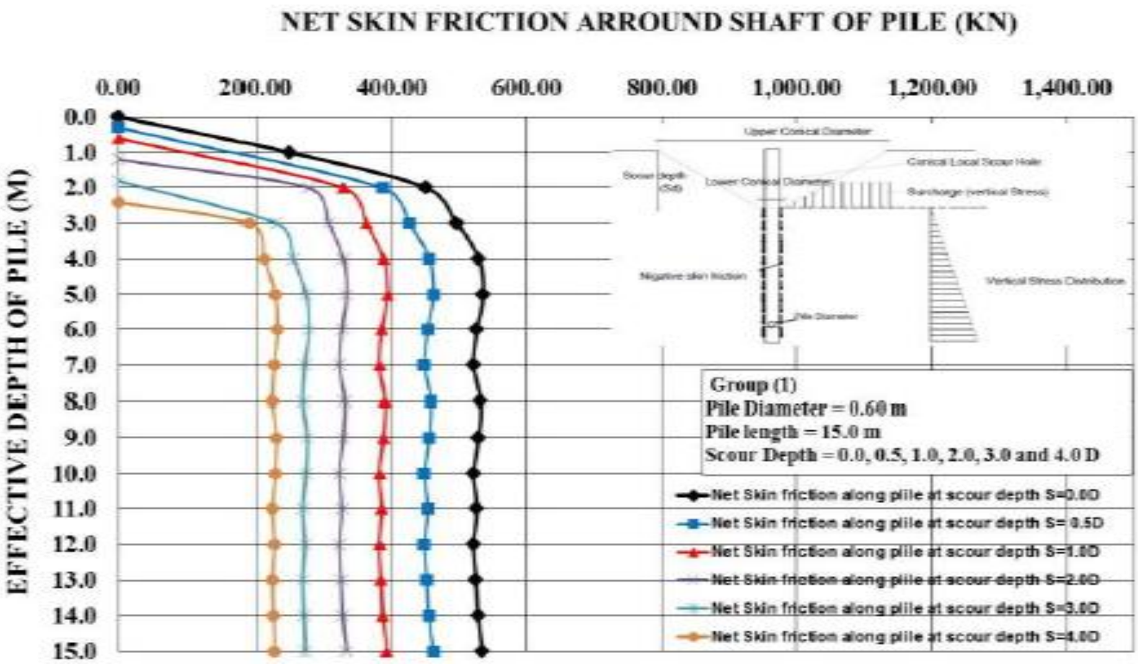


Fig (48) The relationship between the net skin friction around shaft of piles and effective pile depth for different pile diameters.

From the above the following on obtained

i. A decrease in the embedded depth of pile of foundation due to scour decreases the skin friction.

18. CONCLUSIONS

From the present study, the following conclusions are obtained:

i. The ultimate capacity of group of four piles cap directly rested on soil is more than the ultimate capacity group of four pile cap non-rested on soil.
ii. The load transferred to soil underneath pile cap rested on soil was found to be 7.98% from the ultimate load capacity. However, the load transferred to soil by friction was found to be 88.27% from the ultimate load capacity. In addition, the load transferred to soil at pile tip was found to be 3.75% from the ultimate load capacity.
iii. A reduction in the embedded depth of pile of foundation owing to scour decreases the net skin friction
iv. The settlement of pile groups for pile cap rested on soil is less than that for pile cap non-rested on soil.
v. The values of transferred loads at pile tip (end bearing) decreases with increasing scour depths.
vi The values of ultimate load capacities of piles foundation decrease with increasing scour depth.
vii. Fair agreement has been obtained between the experimental values and finite element analysis.

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