THE EFFECT OF IMPROVING SURROUNDING SOIL ON FRICTION CAPACITY OF PILE

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by
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تأثير تحسين التربة المحيطية على قابلية تحمل الاحتكاك للركيزة

رسالة مقدمة

إلى قسم الهندسة المدنية في جامعة النهرين

وهي جزء من متطلبات نيل شهادة الماجستير في الهندسة المدنية

من قبل

محمد محمود سلمان الجبوري

بكالوريوس ،٢٠٠٩

رجب ١٤٣٧

نيسان ٢٠١٦

Supervisor Certification

I certify that this thesis entitled **“The Effect of Improving Surrounding Soil on Friction Capacity of Pile”** was prepared by **“Mohammed Mahmud Salman Al-Jeboury ”** under our supervision at College of Engineering / Al- Nahrain University in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering.**

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Committee Certificate

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Abstract

The improving of the pile surrounding soil still insufficiently studied. Many researchers treated with the surrounding soil to improve the pile capacity for resisting the lateral loads or liquefaction. Many methods of improvement are adopted to increase pile capacity .Compaction and replacing surrounding soil are adopted in this study as improvement method. Experimental work was executed on pile model using a compression machine. At first tests, were performed on soil to determine soil properties. The bored and driven methods of installation are performed in two different ways. Steel bar model were used with two different embedded lengths (10,15) cm. This model was tested at first stage without improvement for both method of installation (bored, driven) and for two lengths for each one for studying the behavior of pile model in natural case. The next stage was to replace and compact the surrounding soil ,different diameter were used for compacting surrounding soil , and with length equal to embedded length of pile model without extending under pile model tip. For each diameter of improvement around pile model ,the model tested for two method of installation and for two embedded length. The load-settlement curve are plotted and the value of pile capacity was extracted. These results give indication for the effect of improvement surrounding soil on pile capacity. HANSEN equation was adopted to estimate the pile capacity and compare it with the results obtained from experimental work with respect to the expected behavior of pile and improved soil block surrounded pile .The discussion of results are summarized. Clear observation are formed , that is the best choice for improvement are located at tight limits around pile, because the high apparent cohesion of soil particles that appear due to compaction ,which makes compacted soil and pile behave as one block and that will rise the value of tip

load resistance. In this study the best choice for improvement was (43 mm) this which approximately equal to distance of (D) from face of pile with respect to pile model diameter (D=13mm)and with ((D improvment/D pile)equal to3.3)) . This improvement achieve best result by increasing both tip load and skin friction, by which pile capacity will increase for driven pile case

1-from (0.015kN) of natural case to (0.047 kN) for (15 cm) embedded length which is mean two times increment in pile load capacity .

2-(0.0095kN) of natural case to (0.033kN) for (10 cm) embedded length one and half time increment in pile load capacity

And for bored pile case

1- (0.0058kN) of natural case to (0.114kN) for (15 cm) embedded length increment in pile load capacity

2-(0.0038kN) of natural case to (0.075kN) for (10 cm) embedded length which is mean (13) times increment in pile load capacity. Any increasing in diameter of improvement will not give increasing in tip load resistance and the increment will occur just in skin friction of compacted soil block and around soil for the case of bored pile. Conclusion of this study are summarized about the effect of compaction and replacement on properties of surrounding soil properties and it effect on pile load capacity.

CONTENTS

Chapter Three experimental work

Chapter Four PRESENTATION AND DISCUSSION OF TEST RESULTS

Chapter Five CONCLUSIONS AND RECOMMENDATIONS

References

Notations and Symbols

The major symbols used in the text are listed below; others are defined as they first appear.

Q_T	Ultimate pile capacity
Q_S	Ultimate shaft capacity
Q_b	Ultimate base capacity
τ_a	Soil –pile shear strength
C_a	Cohesion of soil
σ_n	normal stress between soil and pile
δ	angle of friction between pile and soil.
K_o	coefficient of lateral earth pressure
P	pile perimeter
L	Pile length
A_b	Area of pile base
σ_{vb}	vertical stress of soil at level of pile base
γ	Unit weight of soil
D	Pile diameter of circular pile
$\mathrm{N_C}, \mathrm{N_q}, \mathrm{N_\gamma}$	Bearing capacity factors
ø	Angle of internal friction
$ø_{max.}$	Maximum angle of internal friction
N_q	Bearing capacity factor
$ø_{c.v}$	angle of internal at constant volume
R_D	Relative density
$Q\&R$	Constant founded by researchers
A	factor (that can be considered as the peak friction angle at unit confining pressure)
σ_C	he confining pressure
M	parameter determine experimentally

G_s	SPECIEFIC GRAVITY
D_{max}	Maximum particle size
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
σh	Horizontal stress
S/D	Ratio of spacing to diameter
r_0	Radius of layer
r_m	the radius of layer drawdown
υ	Poisson's ratio
n	constants which is determined from a set of triaxial tests.
R_f	failure ratio usually between(0.75-1.0). $R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}}$
L/D	The ratio of pile length to pile diameter
$\emptyset_{new}$	The effected angle of friction due to compaction
$D_{improvement}$	Diameter of improved area

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Dedication

To My Family (Especially My Beloved Mom dad and wife)…I Owe to Them For All Things I Got.

To My Teachers (From First One Who Taught Me How to Read and Write until Last One Who Taught Me How to Search)…I Owe to Them For All Things I knew.

I Dedicate to You All My Modest Efforts….

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“In the Name of Allah, the Most Gracious, the Most Merciful”

Before anything pride and praise be to Allah for all things. Thanks for Allah who led me to light in the critical times.

I would like to express my deepest thanks and grateful to my supervisor “**Dr. ABDULAZIZ RASSOL AZIZ** ” for his useful advices, help, guidance, and I greatly appreciated.

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My special thanks go to my family who has given me all that I needed and encouraged me to achieve this thesis.

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My thanks are also expressed to my partners and friends in ministry of defense and in Iraqi armed force .

Finally, I would like to thank all who helped me to achieve my goal in one way or another.

Mohammed m. salman

Abbreviations

CHAPTER ONE

Introduction

1.1 General

Piles are structural members of wood, concrete and/or steel used for transferring surface loads to lower soil deposit. This transferring may occur by vertical distribution of the load along the pile shaft or a direct transmit of load to a lower stratum by the pile point, a vertical distribution of the load is made by using a skin friction (or floating) pile and a direct Load application is made by tip or end-bearing pile. This distinguish is purely convenience, since all piles carry load by both side resistance and point bearing except the case of the pile penetrates a highly soft soil to a solid base (**Bowles,1996**).

Piles are generally used for two purposes, the first one is to increase the load-carrying capacity of the foundation and the second one is to reduce the settlement of the foundation. These purposes are accomplished by transporting loads through a soft stratum to a harder stratum at a greater depth or by distributing loads along the stratum by friction along the pile shaft or by some combination for both, in case of the load is distributed from the pile to the supporting soil is of interest (**Reese et. al., 2006**).

The use of scale models in geotechnical engineering presents the advantage of simulating complicated systems under controlled conditions and the opportunity to gain insight into the fundamental mechanisms running in these systems. In many circumstances (e.g. a static pile load test), the scale model may afford a more economical option than corresponding full-scale test. (**Ali, 2009**)

There are many methods to improve soil and suggested to increase soil bearing capacity or improves the pile capacity (stone column, vibro replacement, compaction surrounding soil….etc) all these methods are used to improve the pile capacity or improve soil capacity. In this thesis improving soil will be treated by replacing and compact the surrounding soil. And observe the behavior of pile model after improving surrounding soil. Many research and projects are executed in improving surrounding area by vibro-replacement or compaction. In this thesis the improvement of surrounding soil around pile will be studied this need appear because of crowding surround-area with cable trench and existing structures pile which lead to make the improving impossible for all surround-area. Compaction is the operation of increasing the density of a soil by compressing the particles together and decreasing in the volume of air, there is no noticeable change in the amount of water in the soil. In the construction of fills and embankments, loose soil is placed in layers ranges between 75 and 450 mm in thickness, each layer is being compacted to a specified standard by using instrument like rollers, vibrators or rammers. Generally, the higher degree of compaction presents the shear strength and the lower will present the compressibility of the soil. An engineering filly one in which the soil has been chosen placed and compacted to a suitable specification with the object of achieving a particular engineering Recruitment, generally depend on past experience. The aim is to ascertain that the resulting fill operations properties that are available for the function of the fill (***GRAIC*, 2003**). The dry density of a soil after compaction depends on the water content and the energy provided by the compaction instruments (referred to as the compactive effort) .

1.2 Thesis Objectives

The objectives of this thesis are:

1.To search for the answer of the question "what is the relation between improving surrounding soil of pile and friction capacity?"

2. Find the relation between the pile skin friction and the area of the improved soil ,to locate the best choice to improve the soil around pile (D improved /D pile) with different values and the economic choice.

3. find most suitable method with improvement (driven or bored) and discuss why.

4. The change in soil characteristics (c, Ø) before and after improvement

5.Explain mechanism of increasing the pile load capacity in both cases (bored, driven) due to the experimental work and analyze it according to Hansen equations.

1.3 Thesis Layout

This thesis is presented in the following four chapters:

Chapter one: Give a summary about the piles, the effect of compaction on soil and thesis objectives and layout.

Chapter two: Give brief a literature review and experimental work and numerical research on pile capacity and surrounding soil of pile and the relation between the soil state (medium, dense) sand on pile capacity and review the load settlement test results for each case and the effect of compaction on pile capacity in case lateral load and effect of soil structure on pile capacity.

Chapter three: Is devoted to present the experimental works used to get soil properties and apparatus used to get result for load-settlement test and set up model used as pile model and techniques for execution of compaction around pile.

Chapter four: Shows the result obtained in this study load-settlement curves and behavior of pile model in each case of compaction area. Then using **Hansen's equations** to verify the results obtained experimentally. The discussion for results are included in this chapter

Chapter five: In this chapter the thesis conclusions are obtained and summarized, recommendations are mentioned for future studies and for field works .

CHAPTER TWO

Literature review

2.1 General

Pile capacity determinations are very complicated. A large number of different equations applied, (**Bowels, 1996**). Evaluation of pile bearing capacity is still an object of many researches. Many researches mentioned that, the calculated pile bearing by conventional methods often gives slight agreement with the load capacity test results (**Kraft, 1991; Randolph et al., 1994**). Pile can be classified as two type according to method of installation (bored and driven) Usually 'Driven' piles are driven into the ground by applied force which causes stresses can be considered in the piles. The forces and accelerations noticed in the pile during driving are recorded using a data logger called Pile Driving Analyzer(PDA). Driven piles are commonly installed by many methods. Piles may be driven or jacked into the ground, a number of different methods of driving may be used(Dropping weight, Diesel hammer, Vibratory methods of pile driving Jacking**)** many advantage of driven pile can be observe ,mentioned some of these advantages, construction structure operations not effected by ground water ,projection above ground level advantageous to marine structure, very long length can be driven **,(FATHI,2012*)***. Many methods can be used for installation of bored pile. The simplest method is usage of an auger to remove the soil and replace it with concrete and reinforcement. This is a good method, although it suffers from two disadvantages : spoils and the relatively low load-bearing capacity of the piles. These disadvantages can be partially decreased by using pipe auger piles, which to some extent displace the soil and result in reduced quantities of spoil and raised values of load-bearing capacities.

It is also possible to use a piling method that does not result in spoils and achieves relatively high load-bearing capacities, i.e. soil displacement piles. A variety of intermediate solutions between these variants is also common.

- Bored piles can be installed with low vibration , directly against adjacent structures without causing damage.
- Bored piles installation reduce noise as possible as, making them ideally suited to use in densely-populated areas.
- Bored piles are not suitable for using it in weak soil strata, such as peat.
- It is considered costly type because It takes longer to bore a pile than to drive one, bored pile costs more than installing prefabricated piles or vibro piles.

Bored piles usually used in densely-populated areas, where noise hindrance and the risk of subsidence of adjacent buildings are important considerations. They are also frequently used to expand industrial buildings, as they usually contain vibration-sensitive equipment. Recently advices for the use of bored piles for the foundations of a multifunctional centre in Heemskerk to prevent the subsidence plaguing the buildings in the vicinity.(**FATHI,2012)**

2.2 Static Methods for Estimation of Pile Capacity in Sand.

A pile subjected to load parallel to its axial axis it will carry the load partially by shear generated along the shaft and partly by normal stresses produced at the base of pile as shown in Figure (2-1) (**Fleming et al., 2008**).

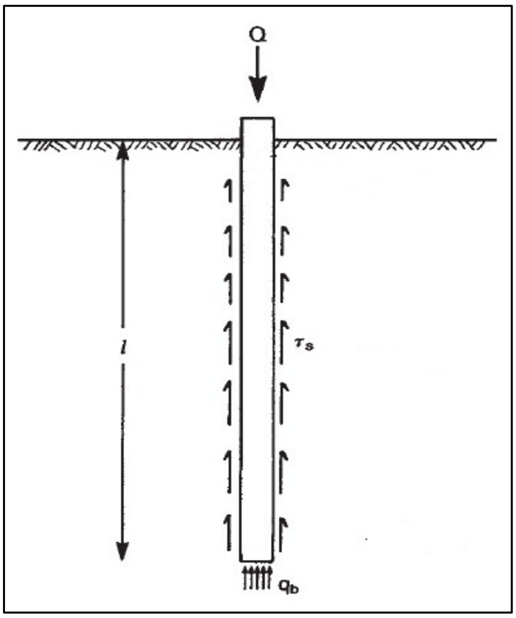


Figure (2-1). Axially loaded pile (after Fleming et al., 2008)

The ultimate capacity Q_T of compression loaded pile will be equalled to summation of the shaft capacity Q_S and base capacity Q_b, thus

$$Q_T = Q_S + Q_b \qquad \dots (2\text{-}1)$$

Where:-

Q_S = ultimate shaft capacity

Q_b = ultimate base capacity

Q_S can be calculated approximately by integration of the pile-soil shear strength τ_a over the surface area of the shaft. τ_a is obtained from Coulomb expression (**Poulos and Davis,1980**):

$$\tau_a = C_a + \sigma_n . \tan \delta \qquad \dots (2\text{-}2)$$

Where:-

τ_a = pile-soil shear strength,

C_a = cohesion = 0 (sandy soil),

σ_n = normal stress between soil and pile

δ = angle of friction between pile and soil.

σ_n: depend on vertical stress and coefficient of lateral pressure it can be estimated by following expression :

$$\sigma_n = K_0 . \sigma_v \qquad \ldots (2\text{-}3)$$

Where:-

K_o= coefficient of lateral earth pressure, and

σ_v= vertical stress.

Thus,

$$\tau_a = K_s . \sigma_v . \tan\delta \qquad \ldots (2\text{-}4)$$

and

$$Q_S = \int_0^L P . \tau_a . dz = \int_0^L P . K_s . \sigma_v . \tan\delta . dz \qquad \ldots (2\text{-}5)$$

Where:-

P = pile perimeter, and

L =length of pile shaft.

tip capacity can be expressed as term of bearing capacity for the shallow foundation as the following equation :

$$Q_b = A_b (c . N_C + \sigma_{vb} . N_q + 0.5 . \gamma . D . N_\gamma) \qquad \ldots (2\text{-}6)$$

Where:-

A_b = area of pile base,

c = cohesion of soil = 0 (sandy soil),

σ_{vb} = vertical stress of soil at level of pile base,

γ = unit weight of soil,

D = pile diameter for circular piles or width of square piles, and

N_C, N_q, N_γ = bearing capacity factors.

The tendency in estimate of tip bearing capacity of piles include neglect the diameter or width of the pile, hence the equation will be:

$$Q_b = A_b(\sigma_{vb}.N_q) \quad \dots (2\text{-}7)$$

So, the total capacity of pile Q_T will be:

$$Q_T = \int_0^l P.K_s.\sigma_v.\tan\delta\,.dz + A_b(\sigma_{vb}.N_q) \quad \dots (2\text{-}8)$$

The skin friction capacity of pile is mobilized at small displacement (0.5-2)% from pile diameter, while tip capacity is mobilized at much greater displacement than of skin friction capacity which is typically (10)% of pile diameter (**Fleming et al., 2008**).

2.3 <u>Skin Friction Factor</u>

Skin friction of pile is controlled by coefficient of earth pressure and angle of friction δ between soil and pile. The value of K_s is critical to the estimation of the shaft friction and is the most complicated to determine reliably because it depends on the stress happen on soil and changes when pile installed (**Tomlinson and Woodward, 2008**). Driving the pile will increase the horizontal soil stress from the original K_o value; while boring process will head for decreasing the soil which in turn will lead for reducing the horizontal stress.

The value of δ can be determined from particular test for the pile material, but for the cases of unavailable tests , it can be suggested equal to $\emptyset_{c.v}$ (**Fleming et al., 2008**).

Borms, (1966) related the value of K_s and δ to the angle of shearing resistance of the soil as listed in Table (2-1) (**Madabhushi et al., 2010**).

Table (2-1) Values of K_s and δ proposed by Broms (1966) (after Madabhushi et al., 2010).

Pile Material	δ_{cv}	K_s	
		Low relative density	High relative density
Steel	20°	0.5	1.0
Concrete	0.75 ϕ'	1.0	2.0
Wood	0.66 ϕ'	1.5	4.0

Vesic, (1967) analyse data of load tests on driven steel piles and suggested a relation between $K_s \tan \delta$ and ø as shown in Figure (2-2).

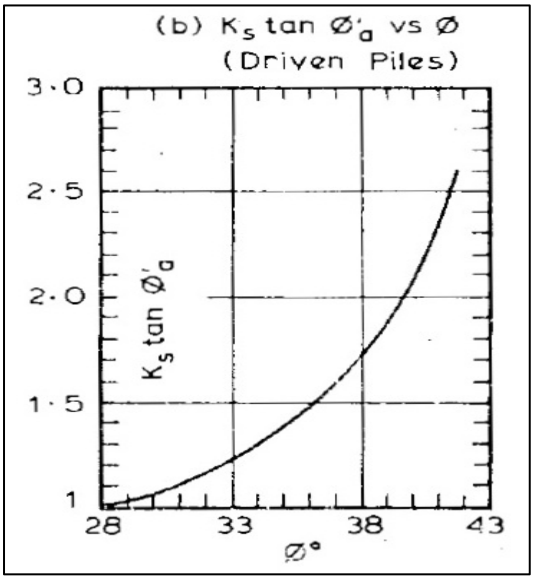


Figure (2-2.) The relationship proposed by Vesic (1967) for driven piles between $K_s \tan \delta$ and ø (from David and Poulos, 1980).

Meyerhof, (1976) made extensive work by collecting and analysing data to present the effect of installation method and the angle of internal friction on the value of $K_s \tan \delta$ as shown in Figure (2-3). The driven piles have the highest value while, the bored piles lowest values due to loosening which may occur by boring process.

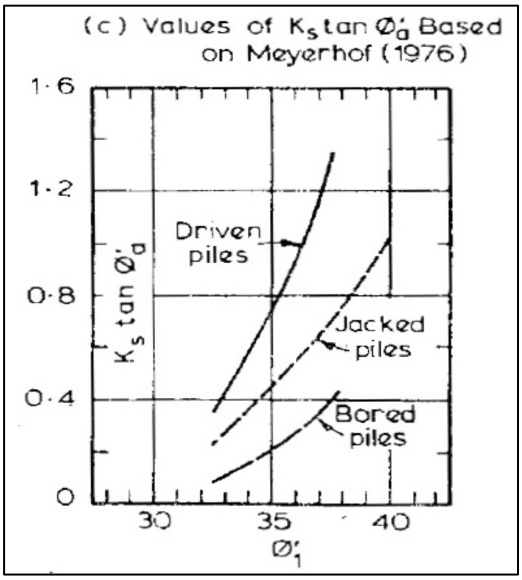


Figure (2-3) Effect of installation and angle of internal friction on the value of $K_s\ tan\ \delta$ as proposed by Meyerhof (1976) (after David and Poulos, 1980).

Kraft, (1991) presented an idea for estimation K_s, depends purely on relative density of the soil and effective area ratio of the pile (full or partial displacement) (**Randolph et al., 1994**). The proposed variation of K_s, shown in Figure (2-4), is depending on field test data, assuming interface friction angles of ($\delta = 0.7\ .\ ø_{max.}$) for silica sand and ($\delta = 0.6\ .\ ø_{max.}$) for calcareous sands, where $ø_{max.}$ is the peak friction angle for the soil.

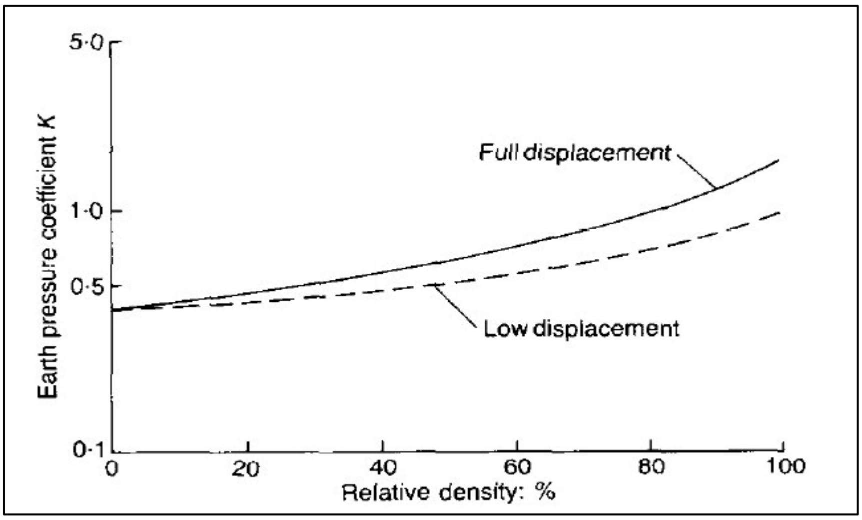


Figure (2-4) Relative density effect on K_s value proposed by Kraft (1991) (after Randolph et al., 1994).

Fleming et al., (1992) proposed taking K_s as a constant proportion of N_q to calculate the decreasing friction angle with an increasing stress level by the following equation:

$$K_s = 0.02. N_q \qquad \dots (2\text{-}9)$$

2.4 <u>Effects of Stress Level on Shear Strength of the Soil</u>

Differences of maximum friction angle in the standard shear tests with normal or confining pressure had been referred by different researchers (**Veiskarami et al., 2011**). This difference achieved by changing stress is called "stress level effect" and seem as one of the major factors causing scale effect. The shear strength of the sandy soil majorly depends on angle of internal friction and this friction angle is widely dependent on stress level. There is a problem arise with question 'did this variation have a great effect on bearing capacity' and " if it did, what value of friction angle should be used for safe and economical design". Many researchers studied the variation of friction angle of sand with stress level (**Akoobi, 2012**).

Lee and Seed, (1967) presented data from Ottawa dense sand and dense Sacramento River sand reported decrease in friction angle with an increment in confining pressure.

Bolton, (1986) collect data of triaxial and plane strain shear tests for 17 types of sand from different places and by analysis of these data; simple equations was proposed for triaxial and plane strain tests which correlate the maximum mobilized friction angle with the mean stress level:

$$\o_{mobilized} = \o_{c.v} + 5\ I_R \quad \text{(in plane-strain condition)} \qquad \dots (2\text{-}10)$$

$$\o_{mobilized} = \o_{c.v} + 3\ I_R \quad \text{(in triaxial condition)} \qquad \dots (2\text{-}11)$$

$$I_R = R_D(Q - \ln(\sigma)) - R \qquad 0 \le I_R \le 4 \qquad \dots (2\text{-}12)$$

Where:-

$\varnothing_{c.v}$ = angle of internal at constant volume,

R_D = relative density of sand,

$Q\&R$ = constants and will be discussed below, and

σ = the mean effective stress.

The suggested equation results by **Bolton** compared with laboratory results is shown in Figure (2-5). **Bolton** suggested that the constant in the above two equations ($Q\&R$) can assume 1and 10 respectively, while **Salgado et al., (2000)** performed a series of laboratory tests on Ottawa sand with fines content range from (0-20%) by weight and provide equation in the same form of **Bolton's equations** but the values of constants ($Q\&R$) are different from **Bolton's constants** and for a broad range of fine content. Summary of theses constants as suggested by **Salgado et al.** is shown in Table (2-2).

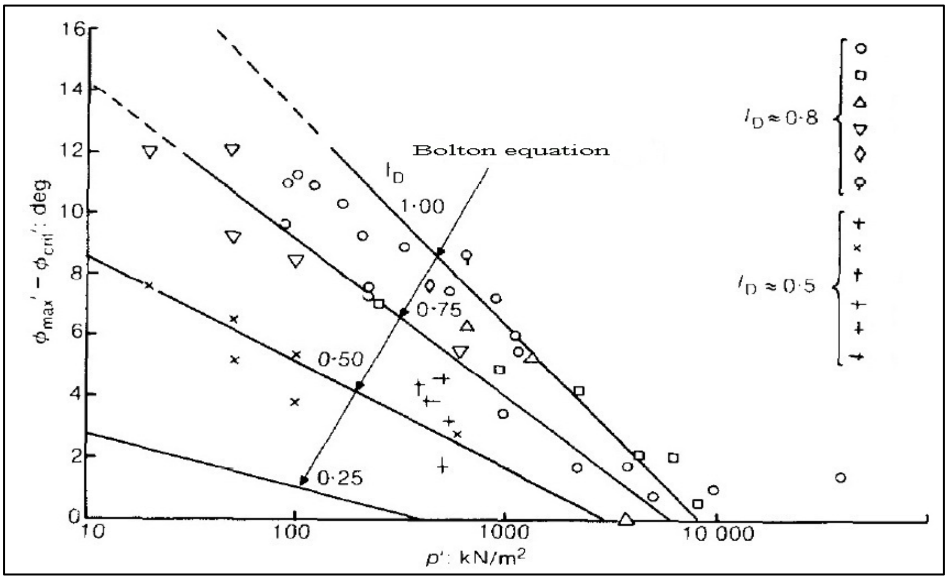


Figure (2-5). Triaxial test data for sands failing at various mean effective stresses comparing with Bolton's equation (after Bolton, 1986)

Table (2-2). The constants Q&R as proposed by Salgado et al., (2000).

Silt (%)	Q	R
0	9.0	0.49
5	9.0	-0.5
10	8.3	-0.69
15 (R_D > 38%)	11.4	1.29
15 (R_D < 38%)	7.9	0.04
20 (R_D > 59%)	10.1	0.85
20 (R_D < 59%)	7.3	0.08

Two experimental direct shear tests on Monterey sand and Danish Normal sand were conducted by **Gan et al., (1988)** showed a reduction tendency in friction angle with an increase in applied normal stress.

Clark, (1998) suggested equation for the decreasing of angle of internal friction with stress level from series tests of drained triaxial compression on silica sand as shown in Figures (2-6) and Figure (2-7):

$$ø = A(\sigma_C)^M \qquad \ldots (2\text{-}14)$$

Where:

A = is a factor (that can be considered as the peak friction angle at unit confining pressure) Figure (2-6).

σ_C = is the confining pressure, and

M = parameter determined experimentally.

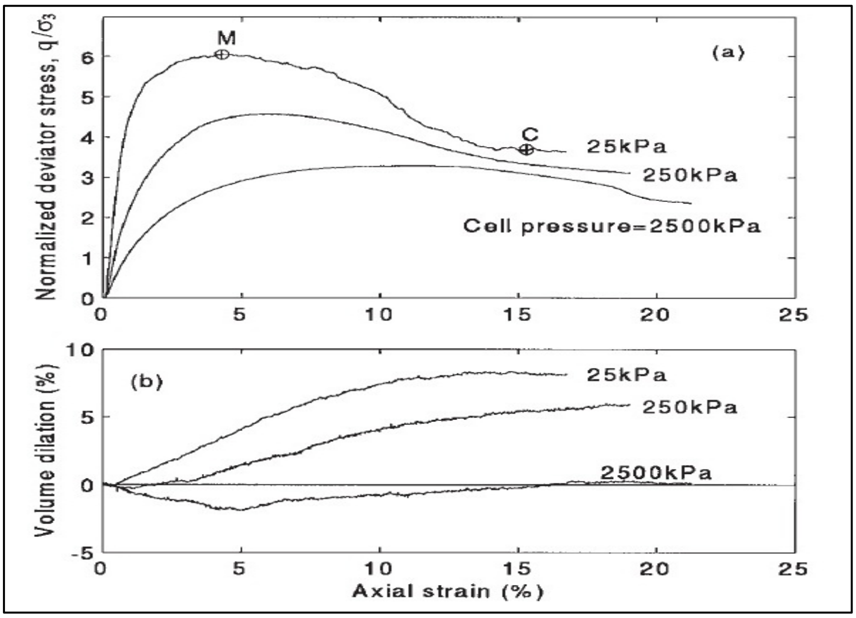


Figure (2-6). Results of triaxial test data on dense silica sand (after Clark, 1998).

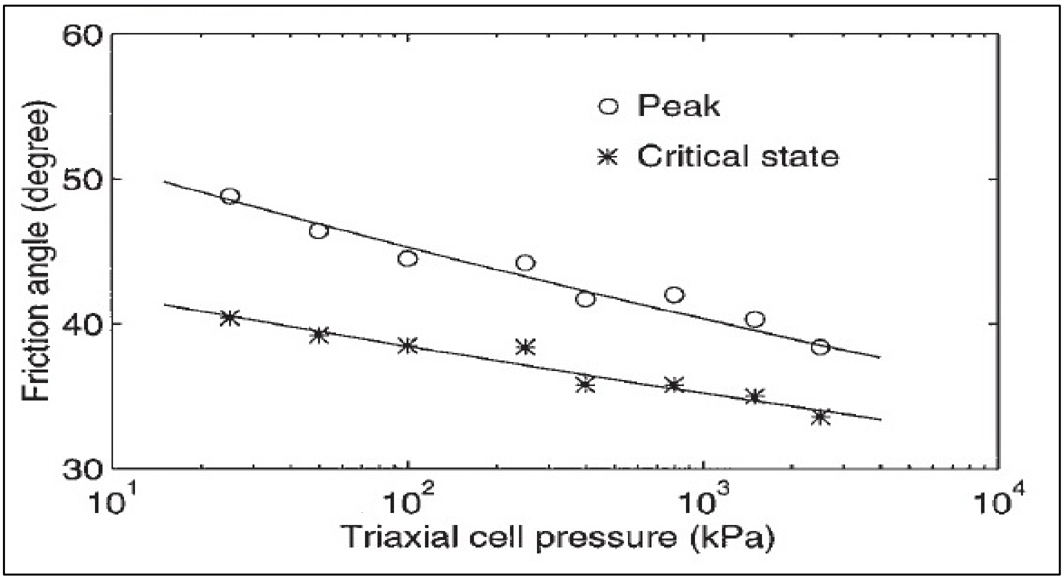


Figure (2-7) Decrease of peak and friction angles with increasing cell pressure (after Clark, 1998).

2.5 Effect of Stress Level on Bearing Capacity of Soil

Many researchers have noticed and stated that the bearing capacity of shallow foundations does not increase without limit and N_{γ} in equation (2-6) decrease when the foundation dimensions increase (**Veiskarami et al., 2011**).

The reduction of N_γ comes from the reduction of the mobilized angle of internal as foundation size increase (i.e. stress level increase).

Early experimental researches of this factor in sand were mostly interested with small foundation model tested. It was achieve that N_γ decreases with increasing footing width or diameter, and this is now widely recognized as the "foundation size effect" (**De Beer, 1963**). **De Beer** suggested that the standard strength of sands should be nonlinear, with friction angle decreasing as stress level increases, rather than linear as in the conventional Mohr-Coulomb criterion (**De Beer, 1963; Yamamoto et al., 2009**).

Clark, (1998) used centrifuge model to study settlement of shallow foundation on hard soil (i.e. dense sand) and noticed a reduction in bearing capacity factor N_γ as acceleration in the centrifuge increases for simulation a wide range of field stress levels. Change of bearing capacity factor with footing size from **Clark** centrifuge test is shown in Figure (2-8).

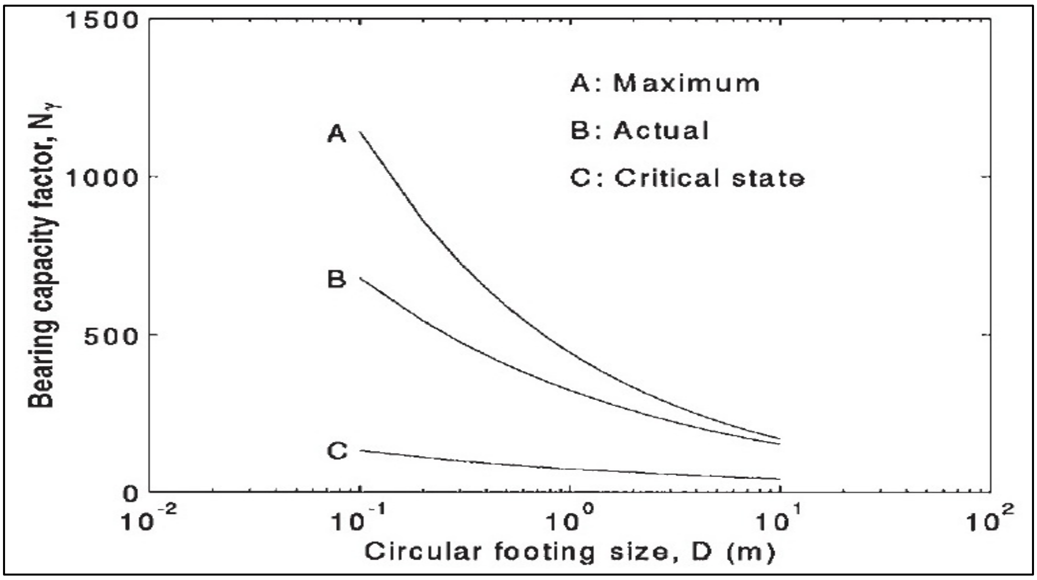


Figure (2-8). Variation in bearing capacity factor with footing size (after Clark, 1998).

Kumar and Khatri, (2008) noticed that the bearing capacity factor N_γ reduce as footing width increases until to a certain value beyond which N_γ have to take a constant value as angle of internal friction become less sensitive to stress level as shown in Figure (2-9).

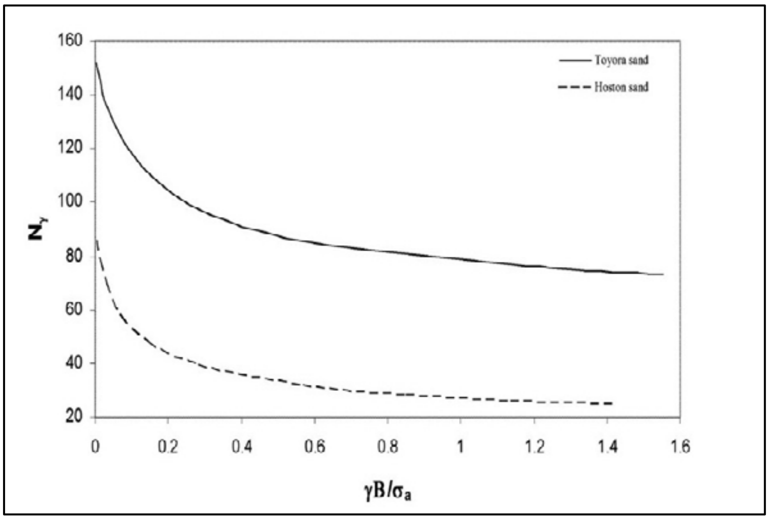


Figure (2-9) Variation in bearing capacity factor with footing size (after Kumar and Khatri, 2008).

In case of deep foundations, few works on such effect are available in literature. **Meyerhof, (1983)** compared with the methods of predication the ultimate bearing capacity of piles in sand for each static cone and (SPT) with the results of pile load test for both bored and driven piles of different sizes and embedment ratio in the sand bearing layer. From these comparisons, an empirical decreasing factor of ultimate end bearing resistance in sand was founded, which decreases with greater pile base. To justify the decrease of reduction factor as pile base increase, **Meyerhof** mentioned that "This decrease of the values of R_b of the unit point resistance with greater pile base diameter, D, at a given embedment ratio in the bearing stratum may be explained by the reduction of the effective angle of internal friction, especially in dense sand, at the base with increasing overburden pressure".

The reduction factor proposed by Meyerhof is the same for both driven and bored piles and explained in equation (2-15).

$$R_b = \left(\frac{D+0.5}{2D}\right)^m \quad \leq 1 \;\; for\; D \geq 0.5\, m \qquad \ldots (2\text{-}15)$$

Where:-

D = pile base diameter.

m = an index which may roughly be taken as m=1 for loose sand, m=2 for medium dense sand, m=3 for dense sand. This equation is described better in Figure (2-10).

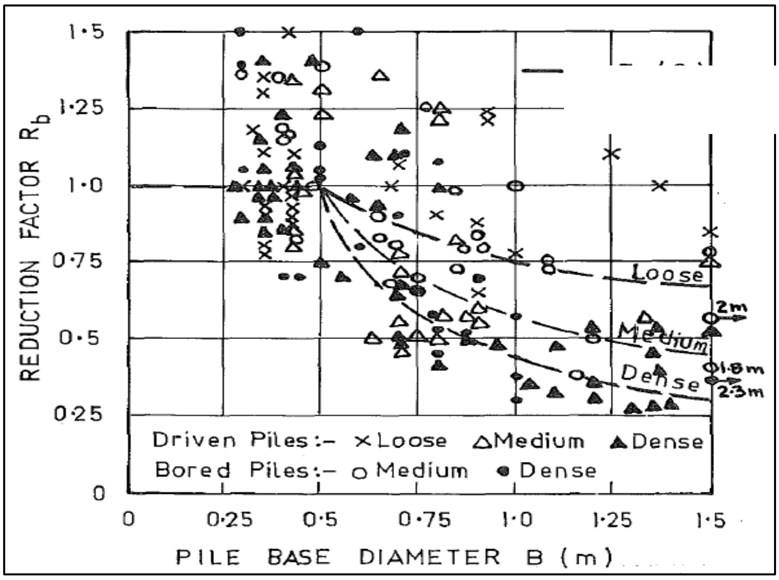


Figure (2-10). Meyerhof empirical reduction factor (after Meyerhof, 1983).

Neely, (1990) discuss data from 100 pile load tests on expanded base piles and observed the reduction of bearing capacity factor N_q as length of pile increase as explained in Figure (2-11).

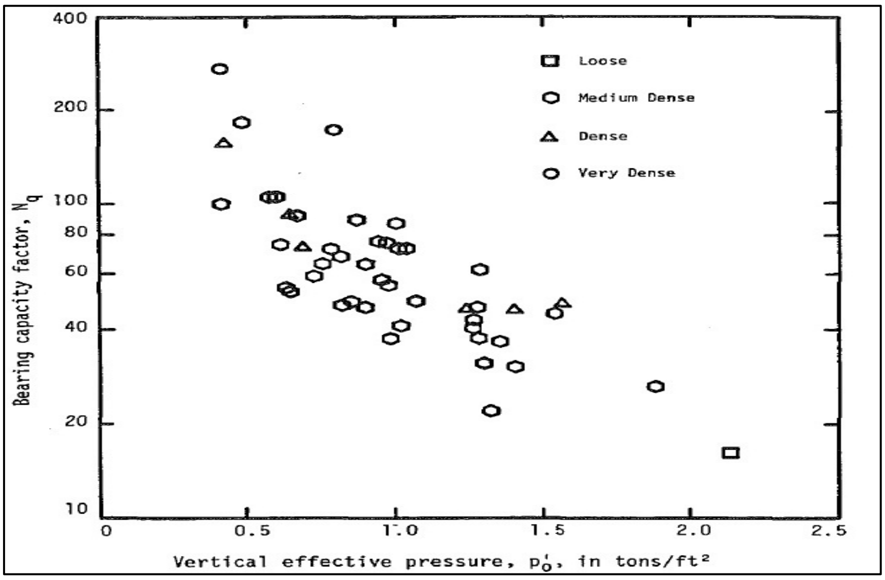


Figure (2-11). Reduction of bearing capacity factor N_q with vertical effective stress (after Neely, 1990).

Craig and Sabagh, (1994) analysed the results from centrifuge test on axially loaded pile in Mersey River sand and observed a reduction of bearing capacity factor N_q with rising stress level as shown in Figure (2-12).

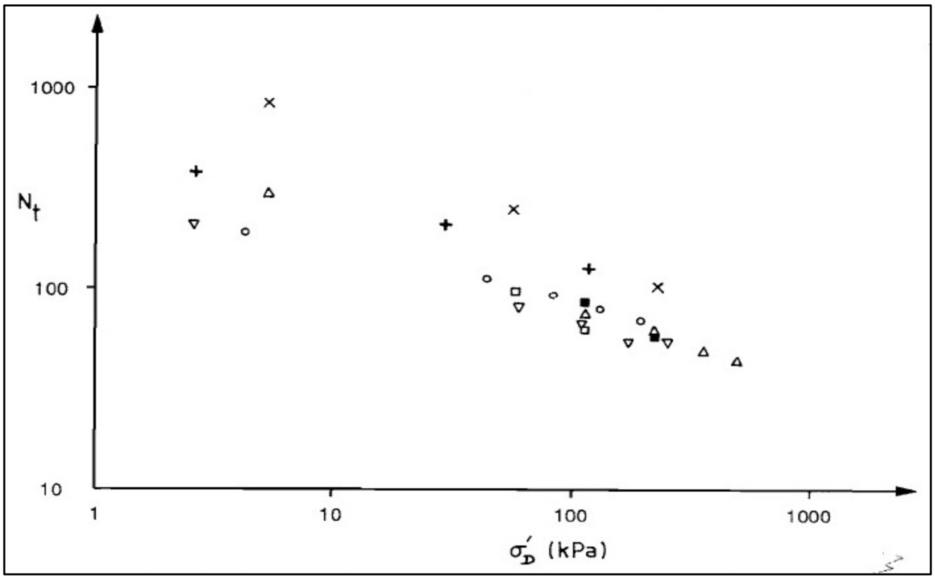


Figure (2-12) Reduction of bearing capacity factor N_q with overburden pressure (after Craig and Sabagh, 1994)

2.6 Soil Behavior around Pile

2.6.1. Material Properties

Model pile tests performed in three types of soil : Chiibishi sand, a skeletal carbonate beach sand from Okinawa, Japan; Dogs Bay sand, a skeletal carbonate beach sand from the west coast of Eire, which has been used by many researchers **Golightly and Hyde 1988; Coop 1990; Yasufuku and Hyde 1995; White and Bolton 2004; Tarantino and Hyde 2005 ؛and Toyoura sand,**

Japanese standard silica sand has been comoned used for research. The physical Characteristics of the soils are explain in Table (2-3) and grain size distributions in Figure (2-13). The factor coefficients of uniformity ($CU=D_{60}/D_{10}$) and range of void ratios are shown in Figure (2-14) for each material tested and some additional materials for comparison such as: Quiou sand, **Golightly(1989)** Shirasu a volcanic soil, a decomposed granite (**Hyodo et al. 1998**), and another silica sand, Aio sand (**Hyodo et al. 2002)**.

Table (2-3). The physical properties of the materials(after Hyodo,2002)

Material	Specific gravity G_s	Maximum particle size D_{max} (mm)	50% particle size D_{50} (mm)	Particle size relative to model pile diameter D_{50}/D	Void ratio e_{max}	Void ratio e_{min}	Carbonite content $CaCO_3$ (%)
Chiibishi sand	2.83	2.0	0.68	44	1.574	0.983	96
Dogs Bay sand	2.72	2.0	0.22	136	2.451	1.621	94
Toyoura sand	2.64	1.5	0.25	120	0.973	0.635	_

As it clear, that crushable soils such as Quiou, Dogs Bay, and Chiibishi sands have a large range of void ratios with respect to silica sands such as Toyoura and Aio. Figure (2-15) shows a comparison of grain shape and variability for the three sands used for the model pile tests. It can be seen that Toyoura sand particles are subrounded while the Chiibishi and Dogs bay sand particles vary from platey to biogenic rounded.

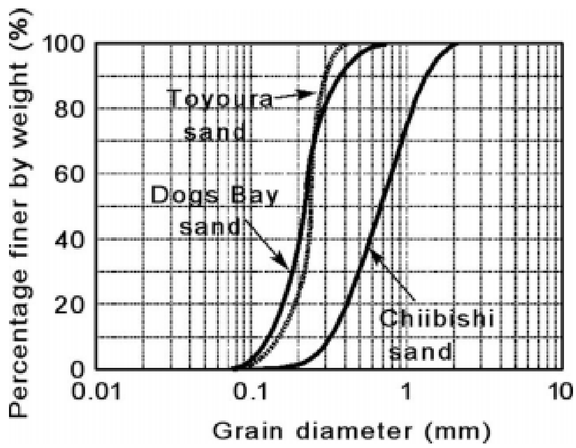


Figure (2-13). Grain size distribution

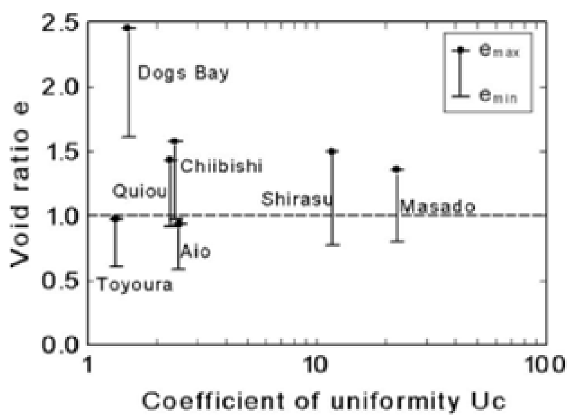


Figure (2-14). Range of void ratio

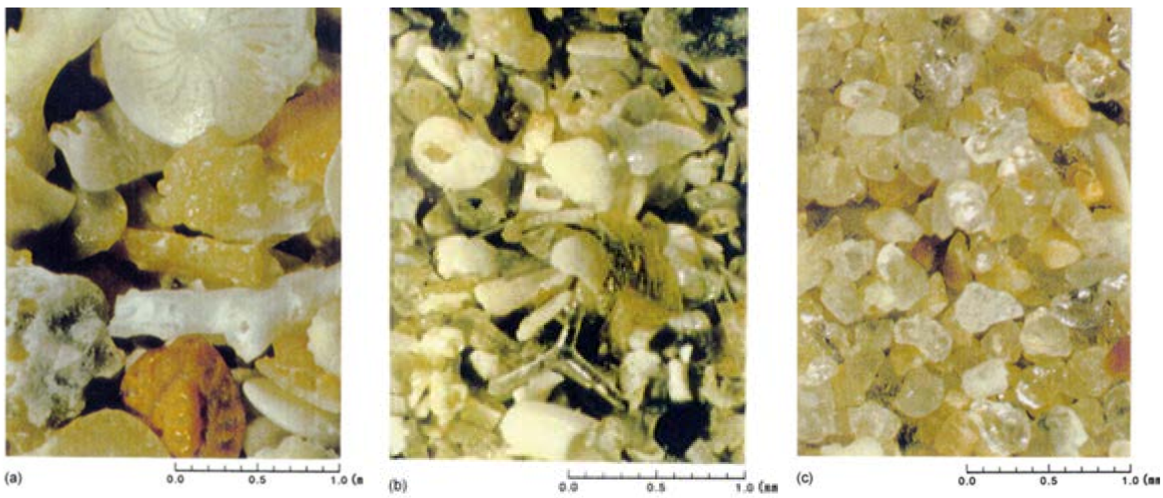


Figure (2-15). (a) Chiibishi sand, (b) Dogs Bay sand and (c) Toyoura sand

In order to observe the behavior of the soil around the pile an additional three samples of Chiibishi sand were generated with layers of coloured particles collected to the sand as it was being created. After testing, the samples were humidifying to make adequate suction for them to be self-supporting and they were then sectioned using a straight edge and photographed. The testing conditions for the sectioned samples were $\sigma v = \sigma h = 400kpa$, K=1.0, and Dr=90% and the variations in end bearing and skin friction with settlement for the sample loaded to S/D=3.0 are shown in Figures (2-16a and 2-16b) (**Kuwajima, Hyodo, Hyde, 2009**).

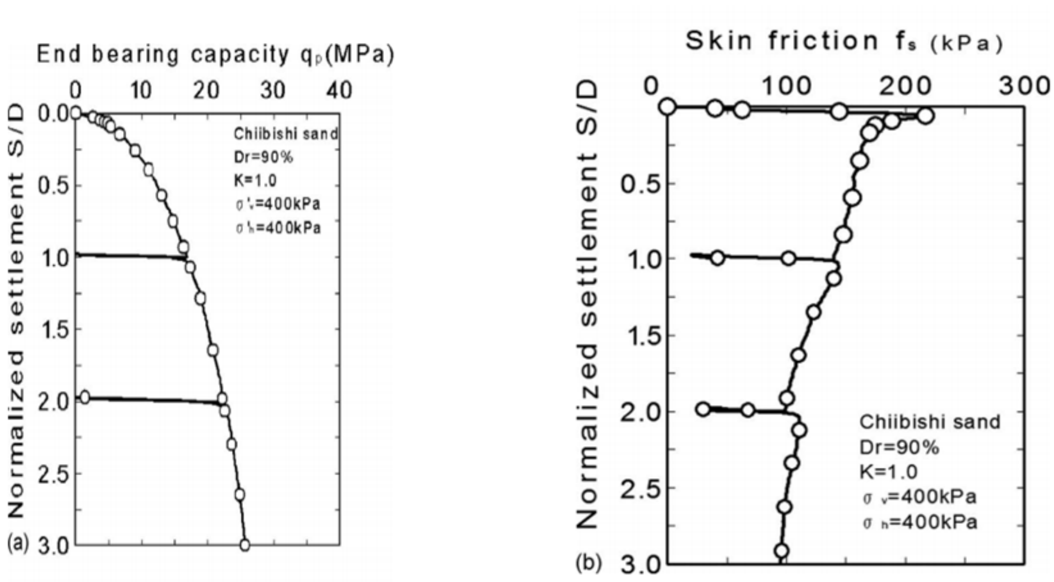


Figure (2-16).(a).End bearing capacity versus normalized settlement σv=σh=400kpa = settlement σv=σh=400kpa 0,K=1.0, Chiibishi sand and (b)skin friction versus ,K=1.0, Chiibishi sand

2.6.2 Pile Tip Soil Behavior

The samples of Chiibishi sand were sectioned and photographed at S/D=0.5, 1.0, and 3.0 as shown in Figures (2.17a, 2.17b and 2.17c) respectively. In addition samples of Dogs Bay and Toyoura sand were sectioned at S/D=1.0 Figures (2.17d and 2.17e). The distance between the coloured layers was 10 mm. It can be seen in Figures (2.17a, 2.17b and 2.17c) for Chiibishi sand that a spherical plastic zone was formed at the base of the pile which extend with increasing S/D and a degraded layer of broken particles developed around the pile as S/D increased.

Comparison of Figures (2.17b, 2.17d and 2.17e) for S/D=1.0 shows that slight heave occurred in the layers adjacent to the pile for Toyoura sand but this was suppressed in the case of the two carbonate sands. In the case of Toyoura sand the harder silica grains were moved sideways generating the heave. In the case of the carbonate sands the crushing caused a densification and corresponding contraction of the sand and thus eliminated the heave

It can be cleared in Figure (2.17c) that in case at a displacement S/D=3.0 there is still no proof of heave for the Chiibishi sand. There are clearly two recognized mechanisms of failure for crushable and relatively rigid grained materials. In the case of the crushable sand will not appear to be any proof of the figuration of a **Meyerhof (1976)** of any type failure technique of shear bands expands away from the pile tip, which is usually cited in text books and underlies the classical bearing capacity solutions for end bearing. The disappear of distinct shear bands refer that the penetration mechanism is more closely approximated by a continuum cavity expansion type of analysis. However, the deformation type also refer that the pile penetration is partly accommodated by volume contraction in the near field as well as continuum shearing toward the far field. It is therefore important to introduce a crushability factor into the calculation of bearing capacity. (**Kuwajima, Hyodo, Hyde, 2009).**

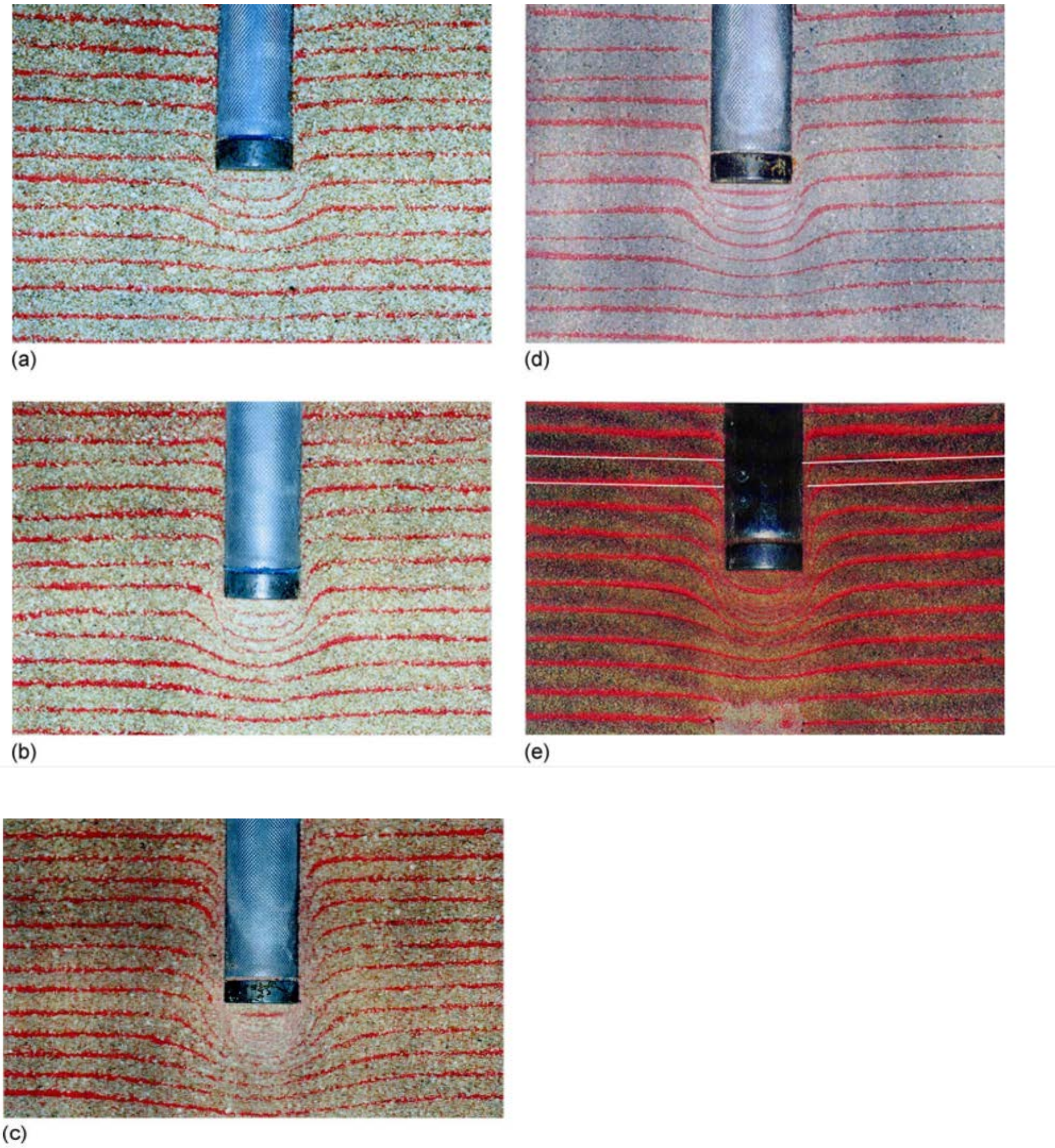


Figure (2-17)(a)Chiibishi sandS/D=0.5;(b)Chiibishi sandS/D=1.0;(c)Chiibishi sandS/D=3.0;(d)Dogs Bay sandS/D=1.0; and(e)and Toyoura sandS/D=1.0(after Hyde, 2009)

2.6.3 Pile Shaft Soil Behavior

To discuss the shearing zone surrounding the pile in carbonate sands a different sample of Chiibishi sand was made as strips just after the maximum skin friction was approached to with S/D=0.05. In this case the sample was consolidated to 400 kPa with a K_0value of 1.0. Figure (2-18) shows the layers after stripped and Figure (2-19) then present a graphic of the pile layer deformations, the amount of layer drawdown at the pile face. Table (2-4) shows these values averaged for all layers shown in Figure (2-18). Similar measurements were performed for the Chiiibishi sand at S/D=1.0 and 3.0 but above the initial location of the tip of the pile. Displacement measurements were averaged on layers above the initial location of the tip of the pile in order to prevent the distortions due to pile end bearing penetration. The resulting displacement of the layers due to skin friction alone was very little with $\delta/$ D values.

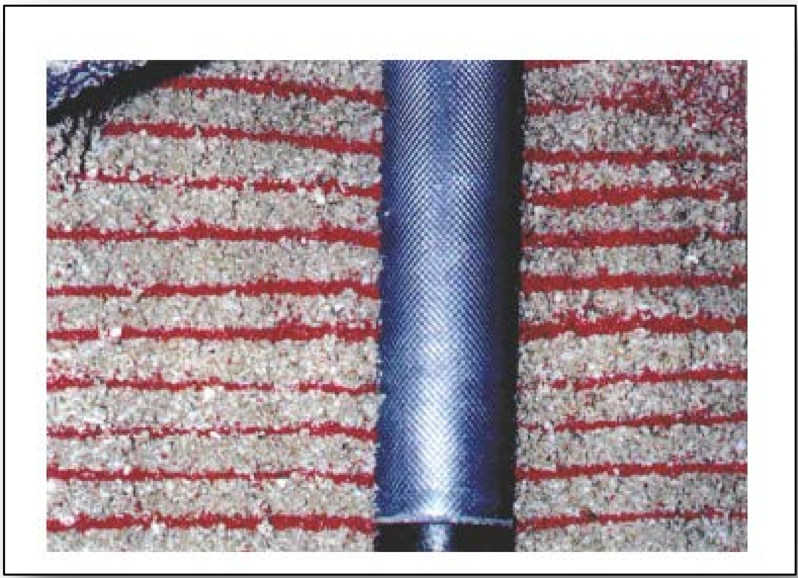


Figure (2-18). Chiibishi sandS/D=0.05(after Hyde, 2009)

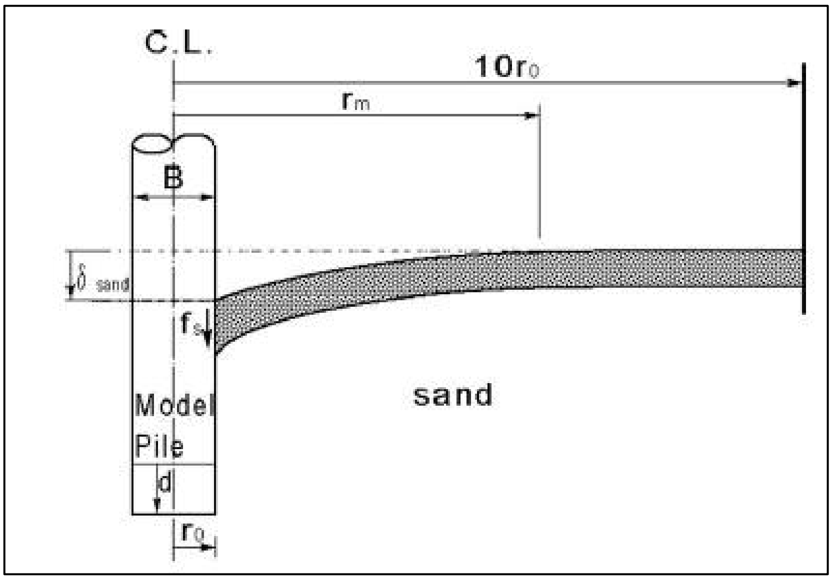


Figure (2-19). Schematic of pile layer deformation. (after Hyde, 2009)

Table (2-4). Sand layer displacement. (after Hyde, 2009)

S/D	**r_0 (mm)**	**δ_{sand} (mm)**	**r_m (mm)**	**δ/D**	**r_m/r_0**
0.05 (f_{max})	15	1.5	34	0.05	2.3
1.0	15	18	38	0.06	2.5
3.0	15	19	41	0.06	2.7

small shear deformation was transformed to the surrounding soil. **Coop et al. (2004)** using ring shear apparatus on circular soil samples stated that particle breakage during shear continued over very large shear strains. An increment in fines near to the pile was also noticed in these model pile experiments. As the grains in the annulus near to the pile continued to break after the maximum value of skin friction f_s the value of skin friction decreased similar tests performed by **Tanaka et al. (1995)** using smooth pile produced smaller deformation in the surrounding soil. The maximum skin friction for all sands happen at a normalized displacement S/D of less than 0.1. At this point the carbonate sands generally less skin friction values than the silica sand, displacement beyond caused a rapid reduction in skin friction for all three substances. Although there was an clearly difference between the maximum values of skin friction at low displacements, this difference had all but disappear at S/D =3. At higher lateral stresses, however, the less crushable Toyoura sand produced higher skin frictions. It can be explained that the interface zone of degraded particles surrounding to the pile contracts especially for served under initial loading. In the case of a driven pile therefore one would not expect a sharp peak in the skin friction but one might wait to see this in a bored pile Samples of Chiibishi sand were sectioned and photographed. It was noticed that a spherical plastic zone was formed at the tip of the pile which expanded with increasing S/D and a degraded layer of broken particles increase around the pile as S/D increased. Large values of the Marsal particle breakage factor were restricted to a zone extending outwards to one pile radius An end bearing capacity modification factor has been suggested which is a function of soil compressibility and degree of penetration. The factor was shown to decrease with increasing soil compressibility and increase with normalized penetration S/D (**Kuwajima, Hyodo,Hyde, 2009**).

2.7 THE EFFECT OF SURROUNDING SOIL STRESS AND DENSIFICATION

(**AKOOBI, 2012**) used the finite element method to simulate the cases of axially loaded bored piles embedded in dense and medium sand. Two mathematical models using **STATISTICA** computer package in form of simple formulas are suggested depending upon the obtained results to simplify the analysis and to expect the capacity of piles readily with excellent regression.

To understanding the results are got the properties of sand which treated should be reviewed as shown in Tables (2-5, 2-6, 2-7 and 2-8).

Table (2-5) Non-linear hyperbolic properties of medium sand. (after Al-Kubaisy,2004).

Parameters	Soil
Unit weight γ (kN/m^3)	14.5
Coefficient of at rest earth pressure, k_o	0.463
Cohesion intercept c (kPa)	0
Max. angle of internal friction Ø (deg.)	32.5
Poisson's ratio υ	0.3
Modulus of elasticity	Variable
Nonlinear parameters	
K	250
K_{UR}	450
n	0.6
R_f	0.8
E_f (kPa)	0.1 of pre failure ratio
$Ø_{c.s}$ (deg.)	23.86
$R\ .D$ (%)	39

Table (2-6) Non-linear interface parameters for Medium sand-pile interface (after Al-Kubaisy, 2004).

Material	δ	K_I	n	R_f
Soil-pile interface	29.4	6132	0.36	0.9

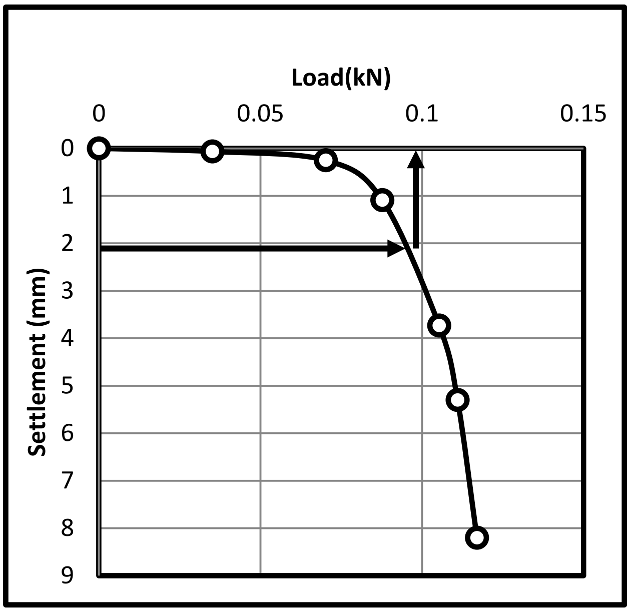
Table (2-7) Non-linear interface parameters for Medium sand-pile interface (after Haddad, 1997).

Material	$\boldsymbol{\delta}$	$\boldsymbol{K_I}$	$\boldsymbol{n}$	$\boldsymbol{R_f}$
Soil-pile interface	33.5	23300	0.27	0.9

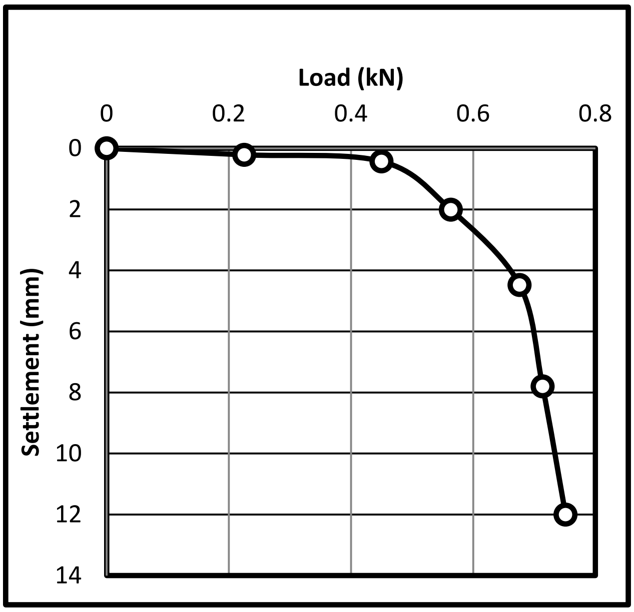
Table (2-8) Non-linear hyperbolic properties of dense sand. (after Haddad, 1997)

Parameters	**Soil**
Unit weight γ (kN/m^3)	15.6
Coefficient of at rest earth pressure, k_o	0.4
Cohesion intercept c (kPa)	0
Max. angle of internal friction Ø (deg.)	37
Poisson's ratio υ	0.3
Modulus of elasticity	Variable
Nonlinear parameters	
K	950
K_{UR}	1150
n	0.45
R_f	0.8
E_f (kPa)	0.1 of pre failure ratio
$Ø_{c.s}$ (deg.)	25
$R\ D$ (%)	70

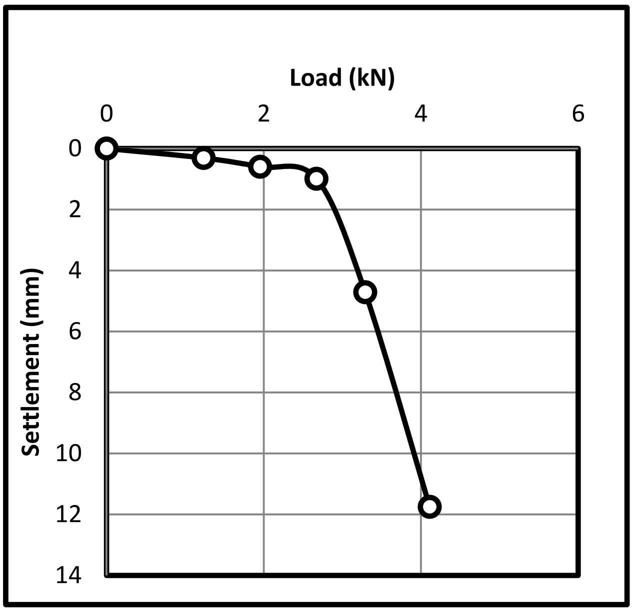
Figures (2-20, 2-21 and 2-22) shows that the load-settlement relations approximately have the same trend shape for both relative densities of sand (dense and medium sand) with all embedment ratios. At the beginning of pile load process, the response of pile settlement seem to be very close to linear relation as results of small settlement value. Beyond this stage and with continuing loading operations , the non-linear behavior of soil appears and formed a visible curvature as soil elements start to fail causing a significant increment in rate of settlement and make a hyperbolic shape for load-settlement relation it can be seen from load-settlement curves for both dense and medium sands ,that the punching type failure is control for all stresses ranges . and as embedment depth increases pile capacity will increase for all range of stresses. This observation gives first notice about the fallacy of critical depth , also these curves shows that the effect of sand state (medium or dense) is quite clear, and pile capacity for pile embedded in dense sand are larger than pile embedded in medium (1.75-2) time for all ranges of stress as the followed plot. See in Figure (2-20).



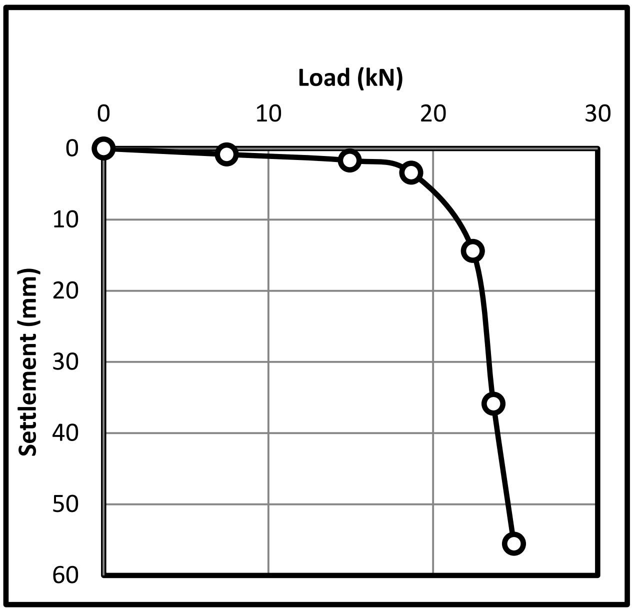
(a) D = 2 cm & L = 30 cm.



(b) D = 4 cm & L = 60 cm.



(c) D = 8 cm & L = 120 cm.



(d) D = 16 cm & L = 240 cm.

Figure (2-20) Load-settlement curves for piles with $(L/D) = \mathbf{15}$ embedded in dense sand.(after Akoobi,2012)

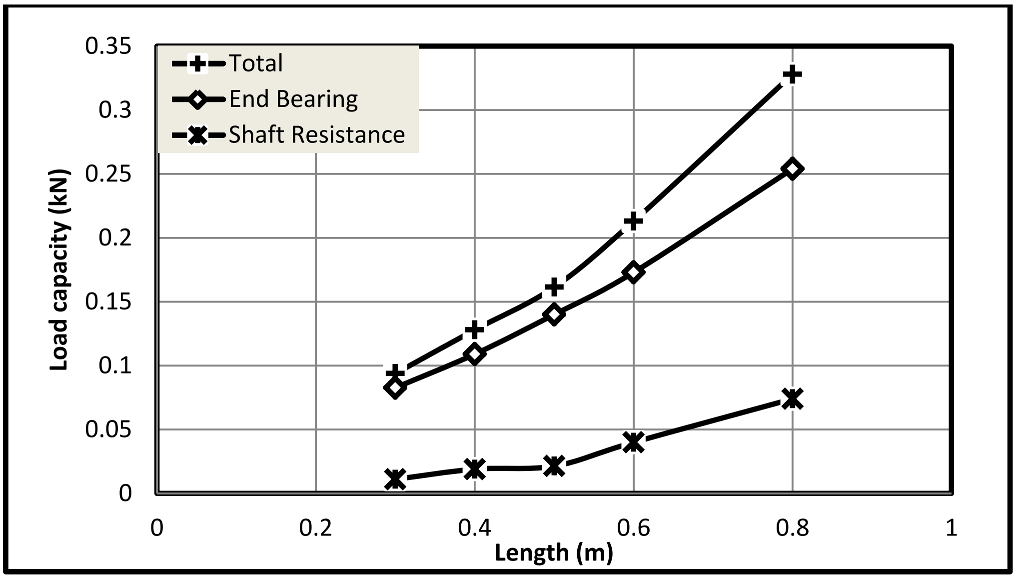


Figure (2-21) load capacity vs. length for pile embedded in dense sand & D=2 cm.

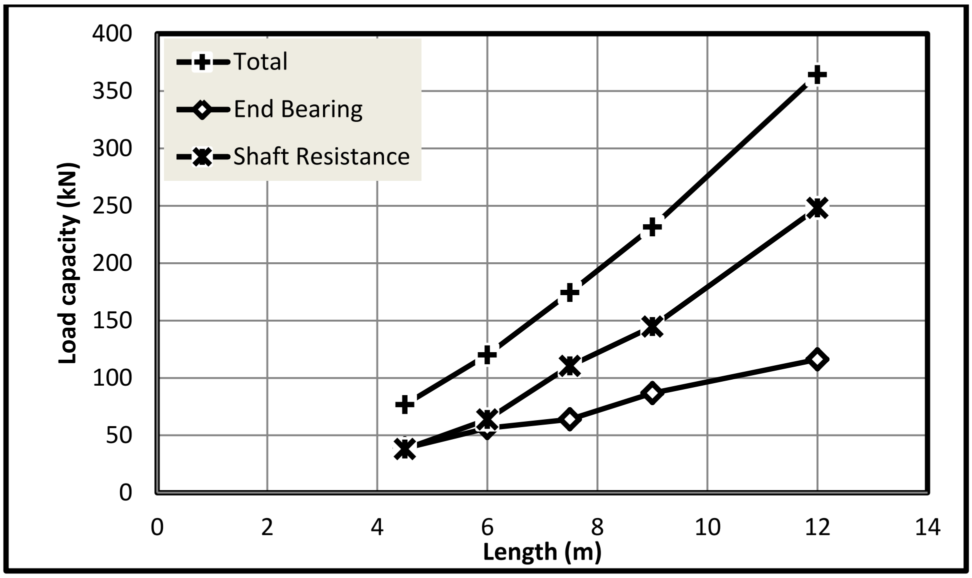


Figure (2-22) Load capacity vs. length for pile embedded in medium sand & D=30 cmafter (Akoobi, 2012)

From previously shown plots, it can be seen that the end bearing capacity is much larger values than the skin friction resistance in laboratory dimensions. and the shafts resistances contributed with small share of total capacity of pile. When lengths of piles increase all the end bearing, skin friction, and total capacities are increased. The increase in end bearing with length gives a observation about increasing in bearing capacity factor as embedment ratio increase. The reason for that the mobilized end bearing capacity is much higher than skin friction resistance may be because of high friction angle mobilized in such low stresses and the dependency of end bearing capacity of piles on angle of internal friction In case of higher stresses range the mobilized end bearing capacity begin to become less than shaft resistance and the difference between them become larger as lengths of piles increase. This behaviour also belong to the decreasing of internal friction angle due to increasing in stress level because increment in dimensions. Same behaviour can be observed for medium sand. In low stress level, the tip bearing capacity is larger than skin friction resistance and as stress level increases, the difference between end bearing and skin friction decreases until shaft resistance is larger than end bearing. It can be observed also, that the end bearing capacity of medium sand is less than the end bearing capacity of dense sand by a significant amount but the skin friction of dense sand is higher than the skin friction resistance of medium sand this is attributed to the difference between angles of adhesion between pile and surrounding soils. From the observations explained above, it can be concluded that the stress level has a clear effect on end bearing capacity and insignificant effect on shaft resistance and the care should be taken in extrapolating the results from a model pile in small scale dimensions (low stress level) to field dimensions (high stress level) and for such extrapolating a stress level factor should be used for safe and economical design for pile in sand. Pile dimension have noticeable effect on pile capacity increasing the pile length means more stress produced at the pile interface along its length and will lead to rise in pile capacity. Also, increasing the diameter of pile leads to increasing in the surface area of the pile shaft contact with the neighboring soil which in turn will lead to increase in the skin friction. In addition, increasing the diameter of pile means more tip load resistanc. (**Akoobi, 2010**)

2.8 The Effect of Improving Surrounding Soil

Improvement of horizontal bearing capacity by composite ground foundation method the composite ground foundation is a new type of foundation that noticeably improves the horizontal bearing capacity by considering the mechanical interaction effect of the improving ground and pile which are installed as one body. Conventional, the ground and foundation structure are considered as separate models, for example, in the case of pile foundation, the load resistance characteristics of soft ground and pile are considered independently in the analysis (**Maeda, 2006**). New construction methods are being studied in order to restrain horizontal displacement and make the number of piles less, and consequently, decrease the construction's total cost, using Deep-Mixing-Method (DMM) which strengthens ground resistance by pouring cement in circumferential ground. The "composite ground figure (2-23) foundation method," that is defined herein, is a foundation practice which expects positive effect of interaction between the improved ground in-situ and the existent pile .Presently, the composite ground foundation which is under development in Japan can be divided in two types according to load/resistance characteristics as illustrated in and Figure (2-24). Figure (2-23) is the most basic type, where all layers of soft ground are improved as part of pile foundation. The whole block of improved ground is considered as a mass that does not move or deform and the increase of its stiffness and strength contributes to the improvement of pile's resistance and restoration force characteristics. The study of its use is already published by the authors (**Maeda, et al., 2007)**

On the other hand, Figure (2-24) explain the foundation structure with deep bearing layer where only the soft ground near the surface that controls most of foundation's horizontal resistance is improved in order to rise the horizontal bearing capacity characteristic of foundation (**Maeda, et al., 2001, 2006**). In this case, since it is suggested that the improved ground resists load with the pile foundation as one body, its deformation and movements is allowed. According to traditionally studies, the located of bearing capacity characteristics of Type II foundation, such as the load specialization and deformation due to mechanical interaction of pile and improved ground as well as the improvement of horizontal bearing capacity, is in considerable progress.

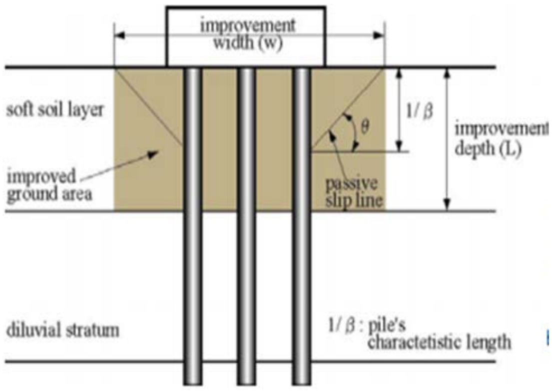


Figure (2-23). Type I basic type (Maeda, el.at., 2007)

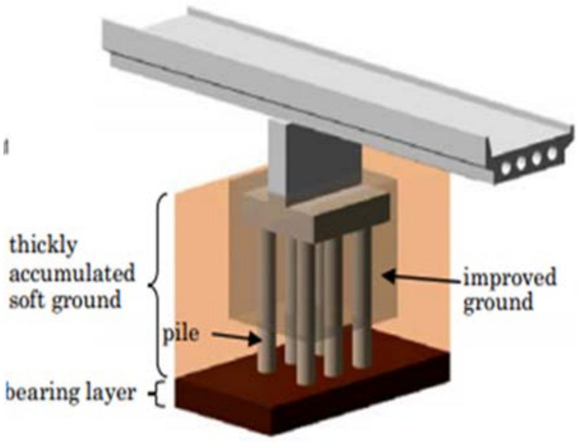


Figure (2-24). Type II floating type (Maeda, el.at., 2001, 2006)

The load-displacement characteristics of composite ground foundation is studied in order to improve its horizontal bearing capacity by carrying out horizontal loading tests of Type I and Type II foundations which have different bearing capacity mechanism. Depend on these results, composite ground foundation has noticeably improved horizontal bearing capacity regard to ordinary pile .In the case of Type I composite ground foundation, the crosswise improvement area is determined from the relation of pile's characteristic length 1/βand passive slip area economic advantage is possible since displacements based on tests are sufficiently small and the improvement area can be decrease more . Displacement inhibitor effect of composite ground foundation is well performed. According to in-situ and laboratory loading test results of Type II composite ground foundation, horizontal bearing capacity higher than ordinary pile can be expected because of improved ground's high stiffness as well as passive resistance in front and frictional resistance in the side. Sufficient composite effect is assured since improved ground and steel pipe pile behave as one body when displacement is small approximately within 1% of foundation width On the other side, when displacement begins to exceed 1% (in-situ) or 5% (laboratory) of foundation width, the behavior of improved ground and pile as one body fails and displacement increases, although, decrease in bearing capacity is not noticed (**Maeda, 2006**).

2.8.1 The Effect of compaction on soil.

Soil compaction is the physical consolidation by applied force that's destroys structures reduces porosity limits air and water infiltration and increase resistance.**(Wolkowski,2008*)*.** Compaction is usually an economical method of improving the bearing capacity of site soils. It may be accomplished by excavating to some depth, then carefully backfilling in controlled lift thicknesses, each of which is compacted with the appropriate compaction equipment. The backfill soil may be the excavated soil dried (or wetted) as necessary, possibly mixed with an admixture such as cement or lime, with or without fly ash or sand filler; or it may be imported soil from a nearby borrow pit.**(Bowels,1996).** The effect of compaction can be expressed by increasing bulk density as soil aggregate are pressed closer. Resulting greater mass per unit volume this produced more firm soil. Soil strength is a measure of ability of soil to resist deformation from an applied force soil, strength become more as soil particle be more tightly together increase of soil strength is as results of soil compaction. **(Wolkowski,2008).** **(Hosseini,2013)** studying the effect of compaction on soil sample with the following properties shown in table (2-9):

Table (2-9) soil physical properties after(Hosseini,2013)

Material	G_s	D_{50} (mm)	γ_{dmax} (kN/m^3)	USCS Classification
Babolsar sand	2.772	0.20	16.68	SP
Ottawa sand [2]	2.660	0.74	17.00	SP
filter	2.710	1.50	18.62	SP

the following conclusions obtained:

Internal friction angle decreases with increasing normal stress. For instance, at relative compaction of 93%, friction angles of Babolsar sand, dry filter, and saturated filter decreased 2.6˚ (from 38.1˚ to 35.5˚), 18.5˚ (from 49.9˚ to 31.4˚), and 19.8˚ (from 49.3˚ to 29.5˚), respectively, as the normal stress increased from 1 kg/cm2 to 16 kg/cm2. In other words, due to 15 times increase in σv, friction angles of Babolsar sand, dry filter, and saturated filter decreased about 7%, 37%, and 40%, respectively.

Friction coefficient decreases with increasing normal stress. For instance, at relative compaction of 93%, friction coefficients of Babolsar sand, dry filter, and saturated filter decreased 0.072 (from 0.785 to 0.713), 0.579 (from 1.177 to 0.598), and 0.600 (from 1.152 to 0.552), respectively, as the normal stress increased from 1 kg/cm2 to 16 kg/cm2. In other words, due to 15 times increase in σv, friction coefficients of Babolsar sand, dry filter, and saturated filter decreased about 9%, 49%, and 52%, respectively.

Internal friction angle increases with increasing relative compaction. For instance, at normal stress of 1 kg/cm2, friction angles of Babolsar sand, dry filter, and saturated filter increased 5˚ (from 38.1˚ to 43.1˚), 2.8˚ (from 49.9˚

to 52.7˚), and 2.8˚ (from 49.3˚ to 52.1˚), respectively, as the relative compaction increased from 93% to 100%.

Friction coefficient increases with increasing relative compaction. For instance, at normal stress of 1 kg/cm2, friction coefficients of Babolsar sand, dry filter, and saturated filter increased 0.15 (from 0.785 to 0.935), 0.118 (from 1.177 to 1.295), and 0.117 (from 1.152 to 1.269), respectively, as the relative compaction increased from 93% to 100%

Proposed two variable functions can be used to estimate the internal friction angle and friction coefficient by substituting the values of normal stress and relative compaction and therefore no additional test is needed. It is better to use curved envelope instead of linear envelope for sands, because it provides a much better fit to the data. Saturating the specimens caused the nonlinear envelopes to have more curvature, so decrease in internal friction angles and friction coefficients increased. It also caused the linear envelopes to have smaller υ and larger c values compared with linear envelopes of air dried specimens. **(Hosseini,2013)**

2.8.2. Reference Projects for Vibro Compaction

1. Soil improvement by means of Vibro Compaction: Fort Calhoun Nuclear Station.

The construction of the Nuclear Station Fort Calhoun neighbouring to the Missouri River in Nebraska started in 1968. It was founded on open steel pipe piles with a diameter of 50cm. The subsoil consisted at most of fine sands with changed silt content in the upper part and with depth increasing in-situ density (from loose to medium dense). In the depth of about 20m rock is encountered As result of the investigations it was found that by means of the vibro compaction method both clean and silty sands could be more densified successfully up to a depth of 20m. For the successful densification of the silty sands a tighter grid, or the addition of imported clean sand, resulted beneficial .

2-Golden Ears Bridge in British Columbia, Canada. Means of the presented ground improvement methods the risk of ground liquefaction around pile foundations can be reduced. Thus a more economical design of pile foundations is possible (**Sondermann, Wehr, 2011**) - (**Deepak raj, S.RGandhi, 2004**) studied the effect of compaction on lateral capacity of pile by testing aluminum single pile model driven in tank filled with sand and compact soil to limited range and to limited diameter around pile with respect to magnitude of (t) which is obtained from following equation (2-16)

$$t = \sqrt[5]{EI/N}. \qquad \ldots(2\text{-}16)$$

EI=flexural rigidity of pile

N=co- effiecient of subgrade reaction

The density of soil was varied from loose sand to higher density with limited depth to value of (t) two dial gauge was fixed on pile model to observe the readings of loads and displacements. They record increasing in lateral load capacity of pile as shown in Figure (2-25) below:

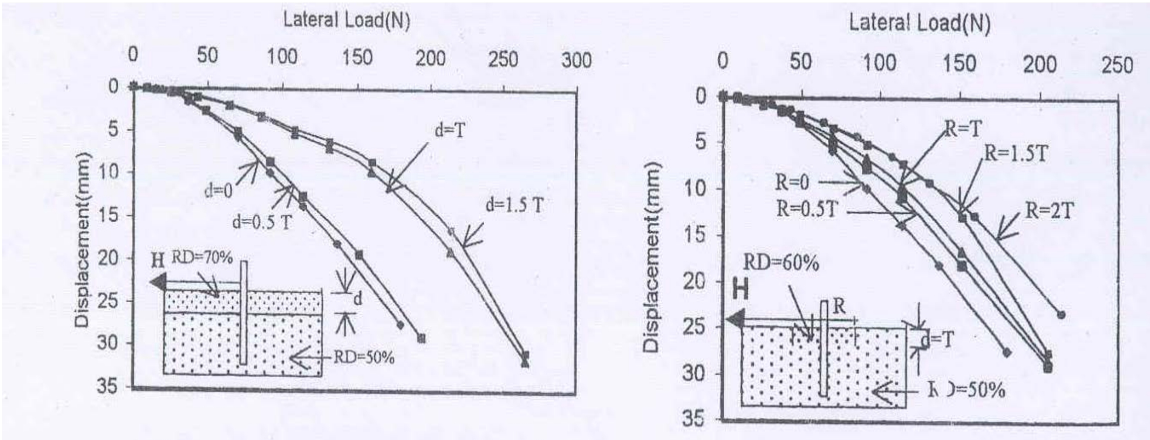


Figure (2-25). Show the effect of improving soil by compaction on lateral pile capacity(DEEPAK RAJ,S.RGANDHI,2004)

2.9 <u>Summary</u>

1. The extrapolating of results from small scale model or using a small scale model to study the behavior of piles embedded in medium or dense sand are not correct if the stress level effect is neglected. So, the stress level effect should be incorporated and taken with care in such stress level dependent soils .
2. For piles embedded in medium and dense sand, the stress level has a obviously effect on end bearing but this effect is not significant on shaft resistance of these piles .
3. The distribution of shear stresses in soil-pile interface along pile shaft is random , non-linear and its tendency to rise with increasing overburden pressure .
4. By means of the methods of ground improvement mentioned previously the risk of ground liquefaction around pile foundations can be reduced. Thus a more possibly economical design of pile foundations .
5. While vibro compaction is suitable in coarse-grained soils, vibro replacement is applicable in both fine-grained and mixed soils.

6. Bearing capacity of the model pile increases with increasing the rate of loading. The relationship between the compressive Bearing capacity and the loading rate can be represented by a Straight line on a log-log plot .
7. Sand density significantly affects the relationship between the bearing capacity and the loading rate when compared to depth to-diameter ratio, which has aslight effect on this relationship.
8. (**DEEPAK RAJ,S.RGANDHI,2004)** conclude there is rise in lateral pile capacity corresponding with increase in density of soil and this increment is for limit range there is not much increment beyond this limit. There is observed increment with increase the depth of compacted soil the increment is limited to specific range also.

    In this study the replacing of surrounding soil of pile and compact it with the optimum moisture content to noticing the effect of compaction on friction capacity and change the area of improving surrounding soil will be observed and observe the change in pile capacity and finally make comparison between the results (with, without) improvement to choose the best result for improving the friction capacity.

CHAPTER THREE

Experimental work

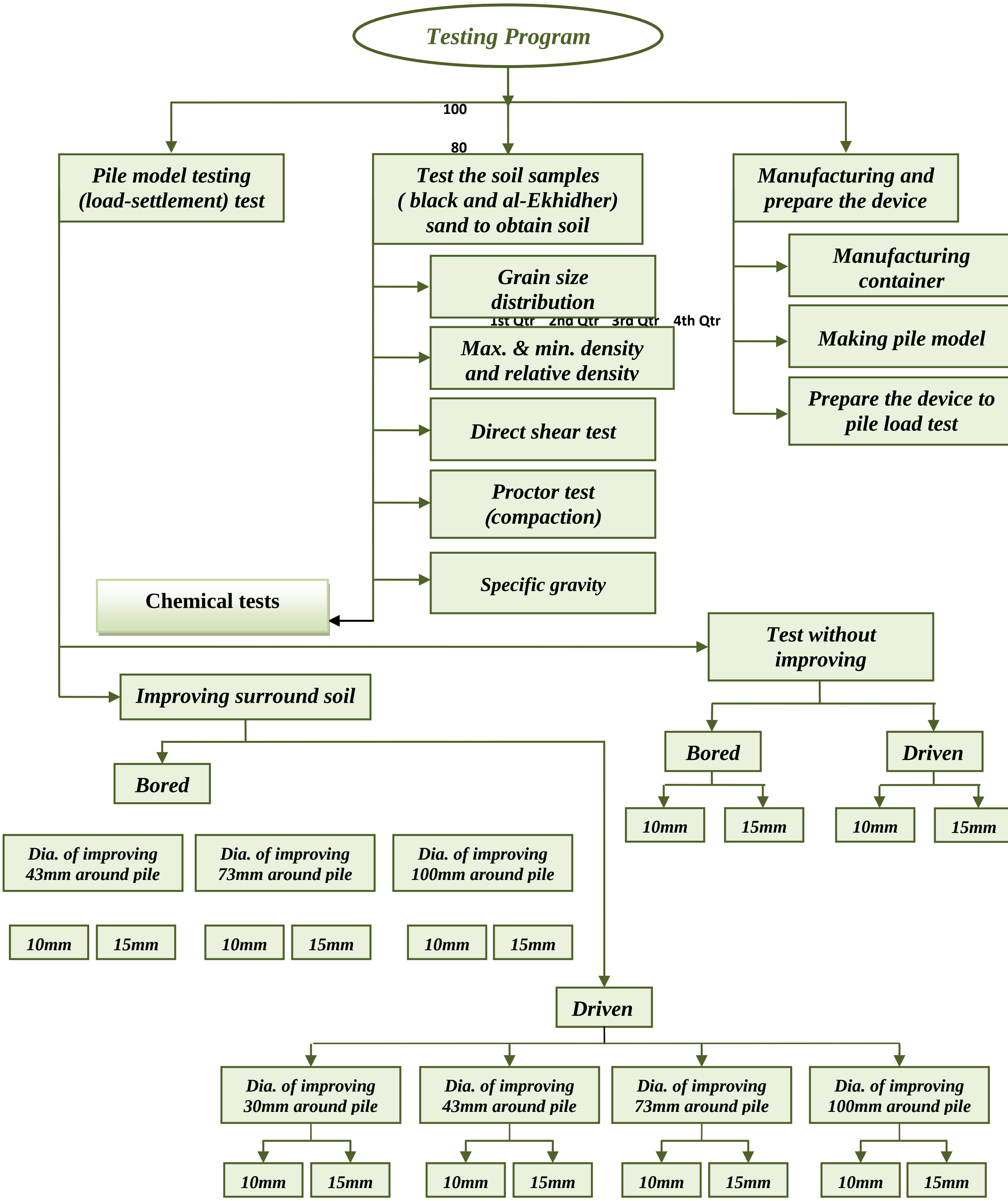
3.1 Introduction

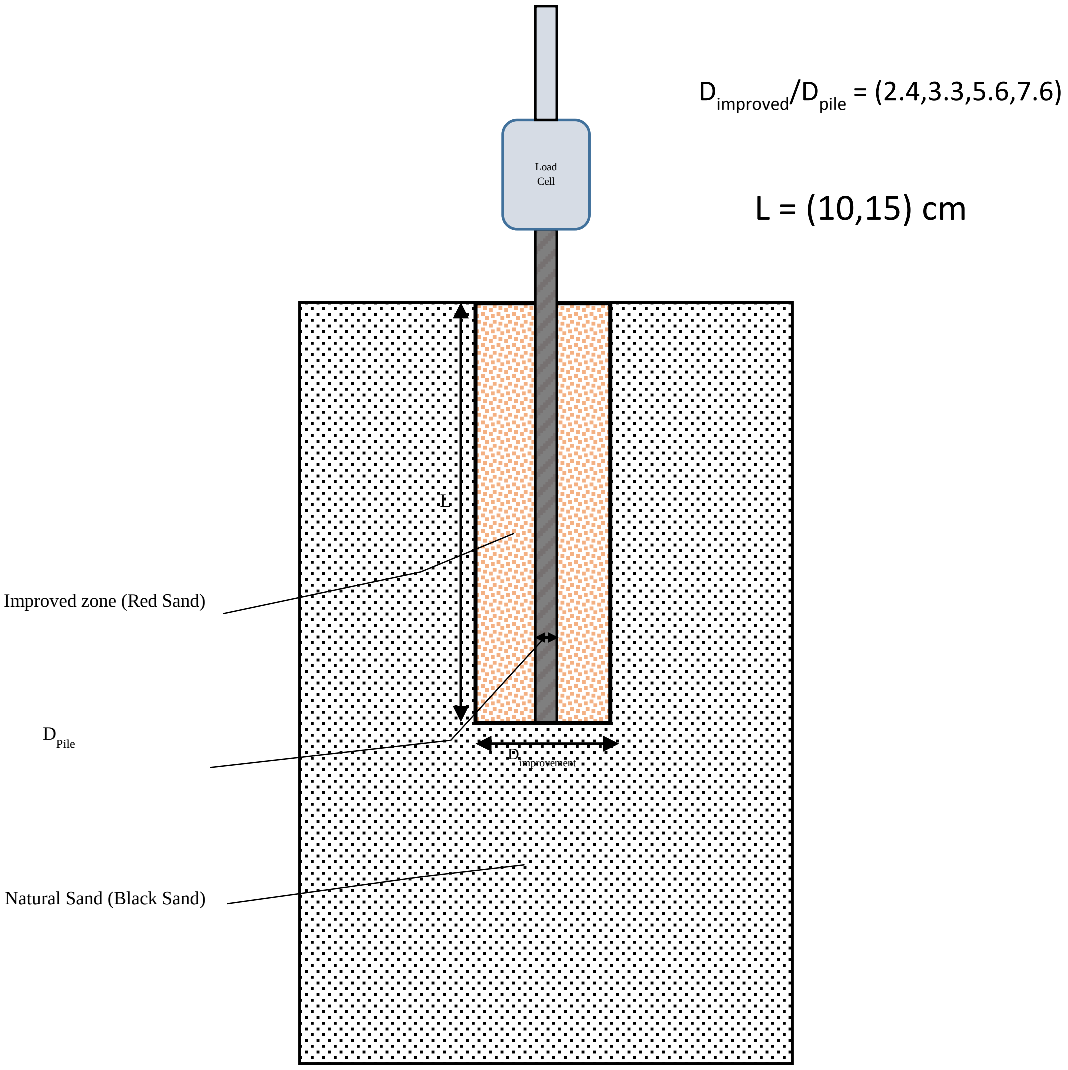
In this chapter, the testing program , material test and manufacturing of some apparatus were explained in which were requested to perform the test program. In order to study the effect of improvement surrounding soil ,bored and driven piles were performed in different methods without improving surrounding soil , then different cases of improvement pile surrounding soil were performed. Plotting the test results as load-settlement curves have been done in this chapter.

Improvement: indicate to replace the surrounding soil to more coarse soil which has more friction than the original one (which considered natural soil) and compact it to get the maximum density, which can be obtained in according to proctor test results . Adding optimum moisture content obtained from proctor test to the soil that need to be improved.

Finally, pile load-settlement test was adopted for studying and discussing the effect of improving surrounding soil of pile on the load capacity.

3.2 Testing Program the following flow chart explain the stages of experimental work





Figure(3-1). The pile model compression device system

3.3 Material Used and Soil Characterization

In this study two types of soil are used (black sand and al- Ekhidher sand) the selection of two different types because of considering the black sand as the original natural soil, and al-Ekhidher sand as replacing soil which will be compacted later.

Two type of soil samples are obtained from local market in sufficient amount to perform the standard test for determining the physical properties of these two types of soil. The details of these properties are listed in following Tables(3-1)and (3-2):-

Table (3-1). BLACK SAND properties (type 1)

Index property	Value
Specific gravity (Gs)	2.61
D_{10} (mm)	0.116
D_{30}(mm)	0.211
D_{60}(mm)	0.337
Coefficient of uniformity(C_u)	2.91
Coefficient of curvature(C_c)	1.14
Maximum dry unit weight (kN/m^3)	17.42
Minimum dry unit weight (kN/m^3)	14.20
Maximum void ratio	0.803
Minimum void ratio	0.475
Angle of internal friction (ϕ) at R.D=12%	29
(C) VALUE	0
Soil classification (USCS)	SP

Table (3-2). AL-EKHIDHER SAND properties (type 2)

Index property	Value
Specific gravity (Gs)	2.54
D_{10} (mm)	0.096
D_{30}(mm)	0.231
D_{60}(mm)	0.407
Coefficient of uniformity(C_u)	4.26
Coefficient of curvature(C_c)	1.36
Maximum dry unit weight (kN/m^3)	17.56
Minimum dry unit weight (kN/m^3)	14.03
Maximum void ratio	0.776
Minimum void ratio	0.418
Angle of internal friction (ϕ)at R.D=86%	44
(c) VALUE	0
Soil classification (USCS)	SP-SM
Maximum unit weight due to compaction (kN/m^3)	18.5

Table (3-3). Chemical test for ekhidhur sand

PH	SO_3%	TSS%	Organic%	Gypsum%
8.16	3.2	4.4	5.1	5.7

3.3.1 Soil Characterization

Standard laboratory tests are carried out on soil to determine soil characterization including the following tests:-

1. Specific gravity
2. Grain size distribution.
3. Maximum and minimum dry unit weight
4. Direct shear test.
5. Moisture-density relation (compaction) test (proctor test)

3.3.1.1 Specific Gravity

Specific gravity tests were performed in accordance with (ASTM- D854-00-) standard test for specific gravity of soil soild by water pycnometer

3.3.1.2 Grain Size Distribution

Sieve Analysis was performed in general accordance with (ASTM) D422 standard test method for particle –size analysis of soils.

Table (3-4). Sieve and hydrometer analysis for ekhidhur sand

Sieve Analysis											
Clay, %	Silt, %	Sand, %	Gravel, %	Passing #200	D_{10}	D_{30}	D_{60}	C_u	C_c	Symbol	Description
-	-	93	0	7	0.096	0.231	0.409	4.26	1.36	SP-SM	Poorly-graded sand with silt

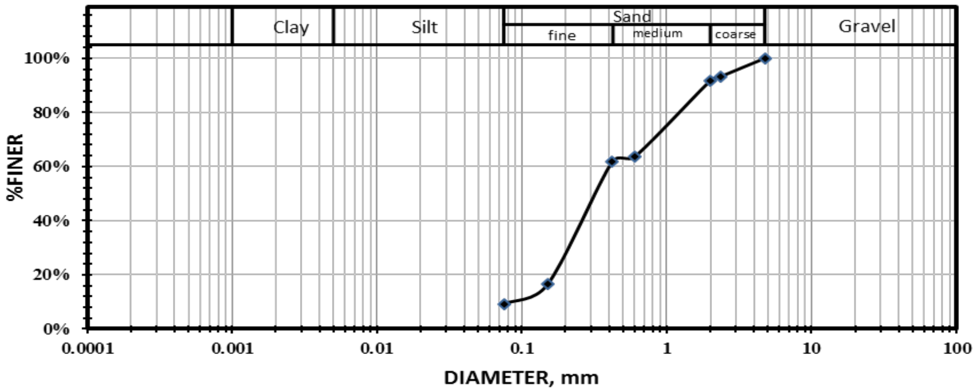


Figure (3-2). Grain size distribution for al ekhidher sand.

Table (3-5). Sieve and hydrometer analysis for black sand

Sieve & Haydrometer Analysis											
Clay, %	Silt, %	Sand, %	Gravel, %	Passing #200	D_{10}	D_{30}	D_{60}	C_u	C_c	Symbol	Description
-	-	97	0	3	0.116	0.211	0.337	2.91	1.14	SP	Poorly-graded sand

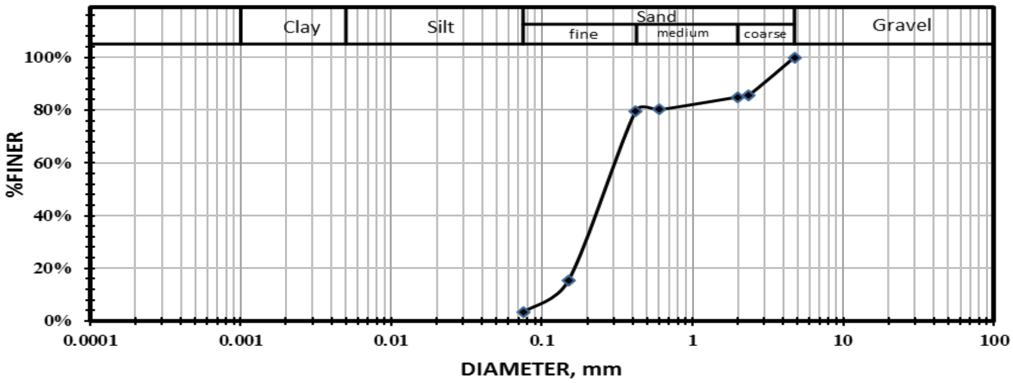


Figure (3-3). Grain size distribution for black sand

3.3.1.3 <u>Maximum and Minimum Dry Density</u>

ASTM D 4254 standard test method for maximum and minimum density. Minimum density test is done using a mold with diameter of 15.2 cm and height of 16.5cm. The funnel used has an opening of 1.27 cm and was maintained at a constant height of (1.25-2.5) cm during filling. The minimum dry density was then calculated as the mass of soil retained in the mold divided by the volume of the mold.

ASTM D 4253 standard test method for maximum index density and unit weight of soils and calculations of relative density.

Maximum density test carried out by uses a mold with diameter 15.2 cm and height 16.5cm. The mold was filled with soil and surcharge plate and weights installed and fixed on the top of mold then placed all on vibrated table at a frequency of 60 hertz. Plate surcharge of (14 kPa) is used to prevent the sand particles from moving in the mold during the vibration. The soil mass after vibration must be weighed to determine soil mass and the change of soil height should be notice too to determine the maximum dry density.

3.3.1.4. Direct Shear Test

Direct shear test is carried out in accordance with the ASTM-3080-90. The minimum specimen width should not be less than 10 times the maximum particle size diameter and the minimum initial specimen thickness should not be less than six times the maximum particle diameter. The test was performed with four different axial stresses (27.24, 54.48, 108.97 and 217.93) kPa for the al-ekhidher sand and (27.24, 54.48, 108.97 and 217.93) kPa for black sand after recording the readings of horizontal displacements we get the following results shown in Figure (3-4).

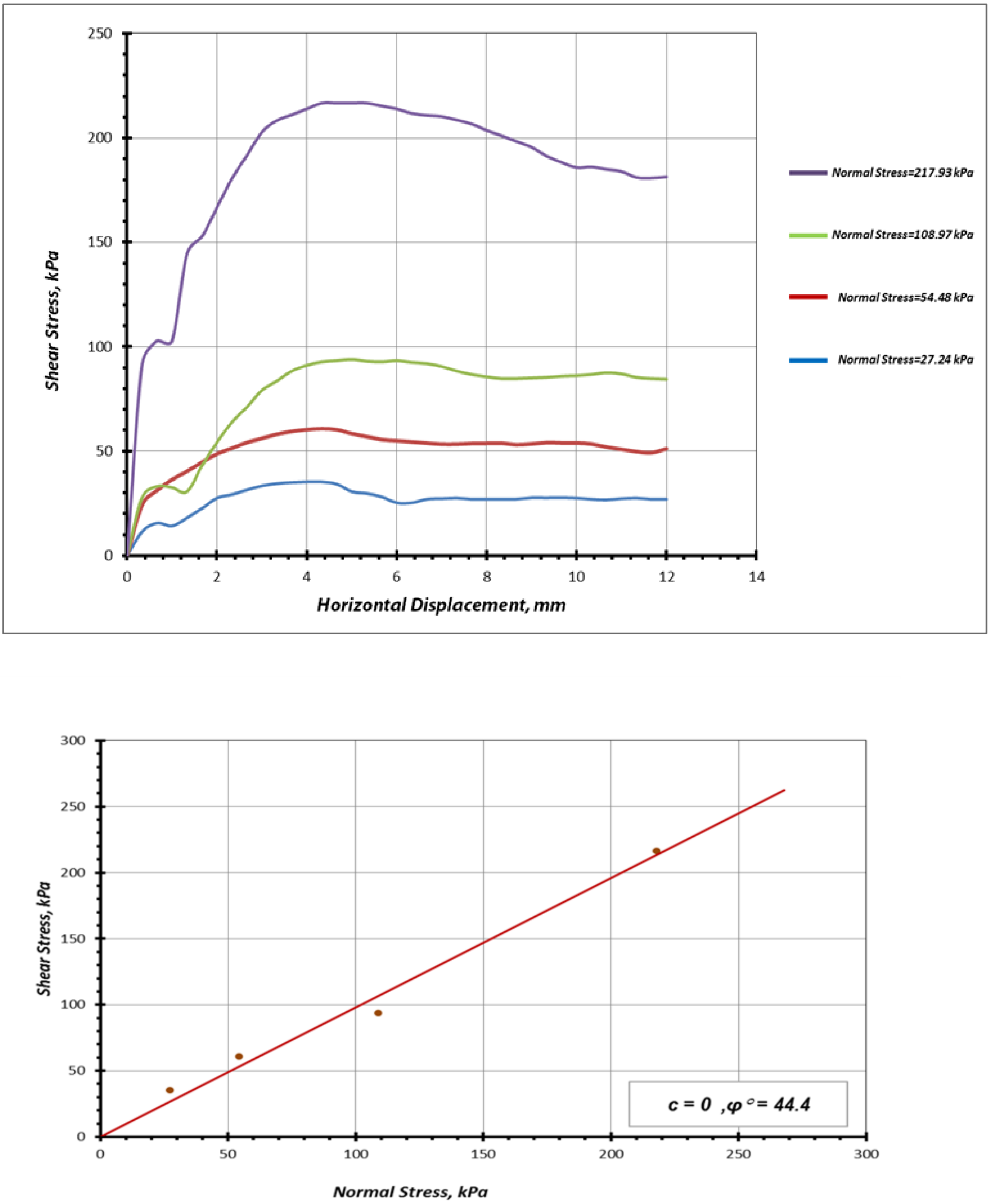


Figure (3-4). Direct shear results sf Al-Ekhidher sand.

Performance direct shear test to al-Ekhedher sand after compaction and getting the following results in Figure (3-5)

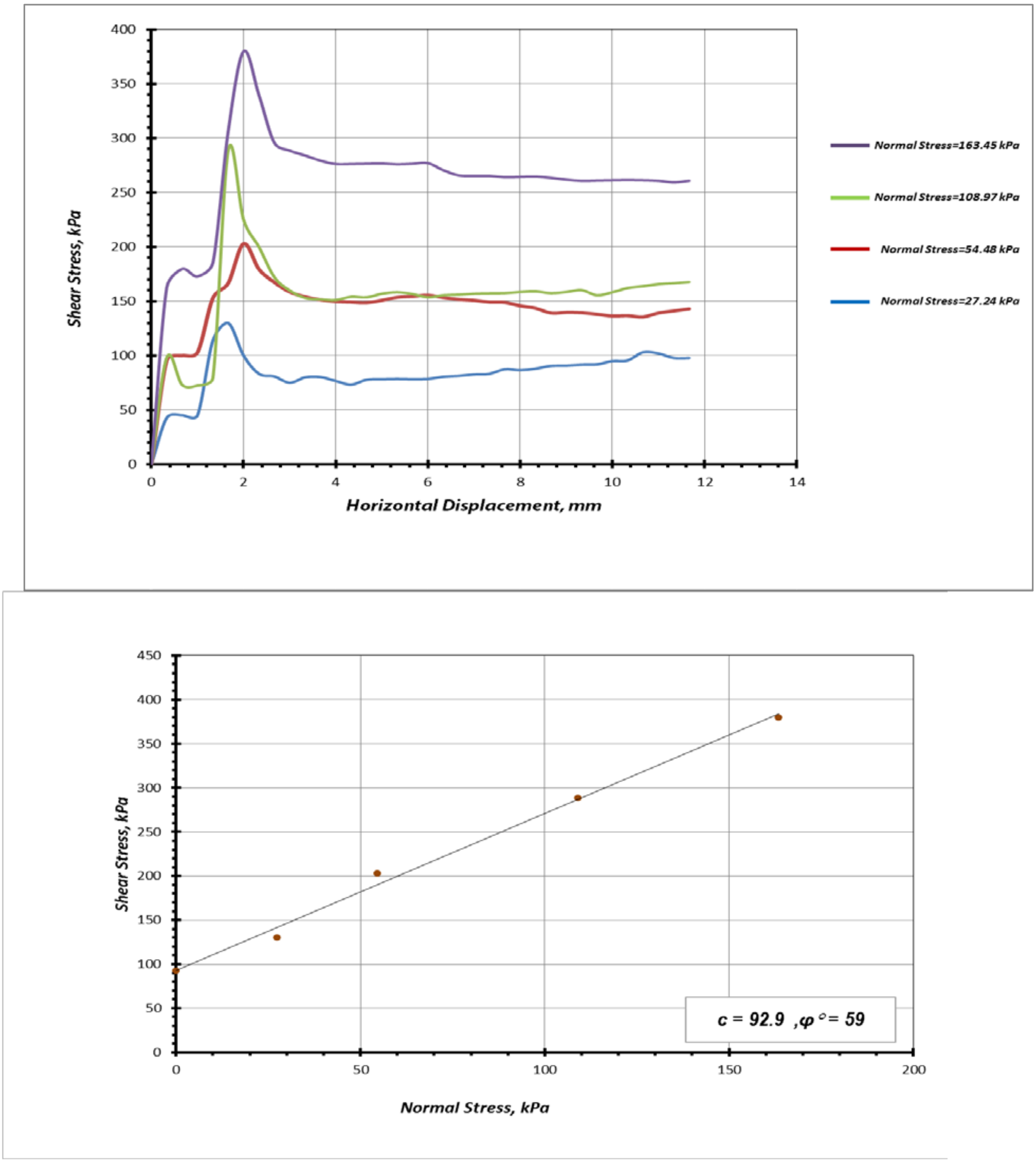


Figure (3-5). The direct shear results for the compacted sand (al-Ekhidher)

3.3.1.5. Moisture-Density Relation (Compaction) Test (Proctor Test)

This laboratory test carried out to determine relation between the moisture and dry density of soil for specified compaction effort. This test and method was developed by ***R.R. Proctor*** in 1933for this reason it is also known as "proctor" test this test performed with accordance to ASTM D 698. Two types of proctor test are routinely performed (1) standard proctor test (2) modified proctor test. In this study performance the standard test by using a mold with volume of about (944) cm^3 filled with soil, and soil compacted by 5.5 Ib hammer falling a distance of one foot into mold filled with soil and soil were layered by three equal layers of soil each one compacted by subjected to 25 drops of hammer each trial addition of specified water content for each trial ,then the moisture content which give maximum density was chose as optimum moisture content. And the following results was obtained as shown in Figure (3-6) and Figure (3-7) shows the performance of proctor test.

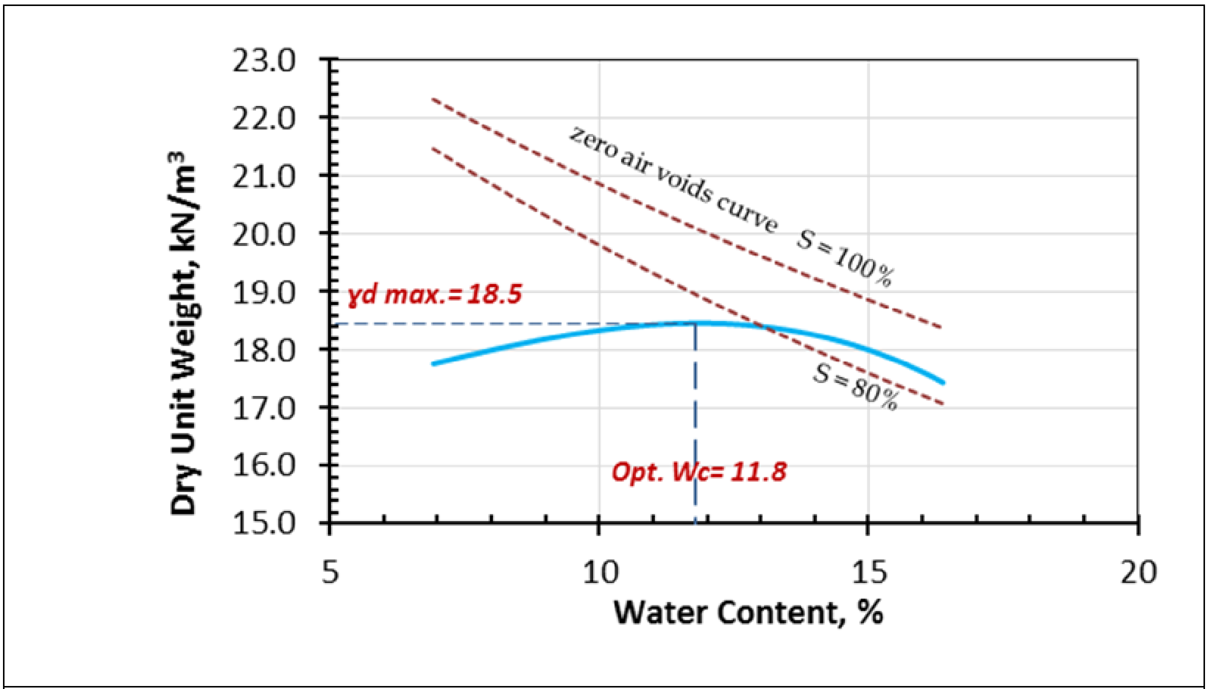


Figure (3-6). Compaction test for al-ekhidher sand

Figure (3-7). Performance of Proctor test

By this test the optimum moisture content and maximum dry density can be get . which will used for replacing soil later (al-Ekhidhar sand) the optimum moisture content for al-ekhidhar sand was estimated to be equal to (11.8%).

3.4 Apparatus and Equipment

In this study, some special apparatus and equipment are needed to achieve the requirements of tests that performed to obtain load-settlement curves in order to compare results of improving the surrounding soil of pile and without improvement . Some of this apparatus were manufactured and the other were developed to be compatible with the requirement of test program.

3.4.1 Model Pile Details

A deformed steel reinforcement bar covered with cement mortar with the following details in table (3-6) are used as pile model .

Table (3-6). Properties of reinforcement steel bar.

Material	Reinforcement steel bar
fy (MPa)	642
Modulus of elasticity (MPa)	200000
Length (mm)	250
Diameter (mm)	13

A steel reinforcement bar has a length of 25 cm are used as a model of pile and a screw in a model head was made to connect with main load cell of device as shown in figure (3-8).

Figure (3-8). Pile model.

3.4.2 Raining Frame and Technique

RAINING TECHNIQUE : raining technique is the method of layering sand in laboratory that is supposed to approximate natural depositional processes in an environment. Method in which a specific initial density is reached, controlled by many factors including, drop height, uniformity of sand rain and depositional intensity. The influence of drop height during dry technique on relative density has been discussed by many authors; some authors seem to think that the influence of height has a significant effect on relative density(**Kolbuszewski 1948a, 1948b; De Beer 1970; Tatsuoka, 1982; Creswell,et) al., 1999; Katagiri et al., 2000**)

The influence of drop height seems greatest in the range of (0-50) cm. Further increase in height produced little or negative effect. **Rad and Tumay (1987)** presented results show that for a given drop height, the dispersion sand leaving the diffuser at tip of the cone affect the resulting relative density. A more falling sand rain results in higher relatives densities.

Raining system consists of:

1- Raining steel frame: it’s made up from steel-square cross section pipe installed as two Column with height of (180) cm welded and connect together by horizontal beam a length (100) cm this frame used to fixed other parts on it.

2- Rope: this rope used for controlling the height of funnel.

3- Pulley.

4- Funnel: made up of plastic with exit opening with diameter (2.5) cm using for filling the mold with sand as pouring device. .

5- Steel mold: with dimensions height (120) mm and diameter (153) mm.

6- Measuring tape to measure the height of funnel. Figure (3-9) show the rain fall system and figure (3-10) show the rain fall sand system results for black sand.

Figure (3-9). The rain fall system

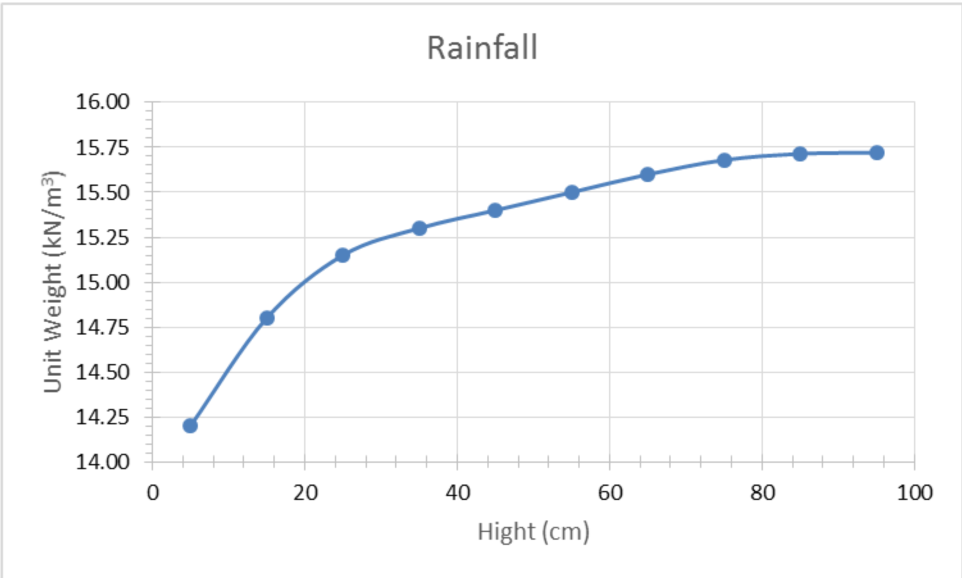


Figure (3-10). The rain fall sand system results for black sand.

The minimum height are chosen which can be obtained from the raining technique to get the minimum density, then the funnel was raised (10) cm in each step to increase the height of falling sand. in each step increasing observed that the height of drop Accompanied by increasing the density of sand until approaching (75) cm as height of drop. A insignificant increasing in density was noticed.

3.4.3 Steel Container

A steel box with dimensions (35*35*35) cm made of steel plates with thickness (0.5) mm welded on frame of steel square -section pipe with dimensions (2*2) cm . The front side of container including (10) mm glass to be facing front of container . At the bottom of container the center of it was located and hole drilled this hole used to fix it onto compression device. This container was made in accordance with the following requirement:

1- Effect of sides of container walls may strongly reduce the vertical stress with depth, to avoid side friction of walls; the ratio of the container height to the diameter must be equals or less than one (**Tarnet 1999 Garnier 2001 and 2002**)

2- Smooth walls are needed to be joined to minimize arching and side friction effect due to limited container (**sizeKutter (1994), Lai et.al (2002) and Teymur and Madabushi (2003)**).

3- To eliminate any rigid boundary resulting from pile driving in loose sand, the bulb of stress around pile is about (7D), this distance should be considered in design (**Kishida, 1967**). This container will fill with soil and install in the pile load-settlement measuring device. Figure (3-11) shows the steel container .(**ALI,2012)**

Figure (3-11). The steel container.

3.4.4 Pile Load–Settlement Testing Device

The 50kN automatic electromechanical compression machine "CONTROLS, Italy" model 70-T0108/E/EZ used as the compression machine used for applying loads and records settlement of pile model .The main properties of it is as listed in Table (3-7):

Table (3-7) . properties of compression machine

Max. load capacity	50 kN
Displacement rate	0.01-51mm/min
Load rate	1-10000 N/sec
Horizontal span	380 mm
Ram travel	100 mm
Maximum vertical span	800 mm

A 2.5 kN load cell" CONTROLS, Italy" is used for measuring the loads applied on pile model with accuracy of (+/- 0.0001) kN.

Displacement transducers of 10 cm full scale with accuracy of 0.001 mm "Mitutoyo, Italy" is used for measure the settlement. the container was installed on the base of compression machine then filled with sand reaching to the desired density. The model of pile connects to the load cell system and the rate of settlement was selected as 1.27 mm/min(**ASTM,1995**) .At the start of the test the load cell and displacement transducers start to display the readings of loads and displacement of the moving shaft under container respectively on screen of the device . The readings was recorded then plotted load –settlement curve to estimate the pile capacity of pile model. The compression machine is shown in Figure (3-12).

(a)display of the device

(b) the compression machine connect with pile model

Figure (3-12). The compression machine

3.5 <u>Pile Load-Settlement Test Procedure</u>

The container fixed onto the device by connecting it with the drilled hole at bottom of container. The black sand was considered as the original natural soil , after completing the rainfall process and limiting the densities of soil Corresponding to each high ,soil was prepared with a dry unit weight of 14.20

kN/m^3 at height of fall equal to 5 cm. The funnel is filled with sand to pour it in the container freely with uniform way. The sand was distributed spirally ended at center. When the sand rises in the container with a certain thickness, the funnel should be raised with distance equals to the soil layer thickness that raised in the container to maintain the required distance for the falling, to achieve the desired density after completing the final layer. The top surface was scraped and leveled to get as near as possible to flat surface. Pile model is connected to device directly to the load cell by screw manufactured at top of the model. The steel arm which load cell and pile model were connected with it, will be downed by using screw approaching to the desired depth . The main aim of test is to determine the behavior of pile model before and after improvement of surrounding soil wherefore at first the pile model was tested as driven and bored pile and with (100) mm and (150) mm length for each case was mentioned. Pile model is connected to the device of measuring load-settlement. Sand was prepared to the test by raining technique for two different length (100-150 mm) and methods(with improvement-without improvement), for each case (driven- bored).

3.6 Pile Models Installation Procedure

3.6.1 Bored Pile Procedure:

The case of bored , pile model was fixed at load cell for desired length as mentioned previously ,either (100) mm or (150) mm the. Bed of sand was prepared carefully with desired density, to approach few millimeters under toe .The pile model will uninstall and the soil under toe will be bedded with desire density , then the installation process of model piles is performed to achieve to the desired length again , because the soil zone which is under the model pile toe does not have the same desired density ,because the sand particles cannot arrive beneath the toe zone with the required energy. The method of filling the container with sand was continuing until the container filled with sand ,then the process of leveling surface was performed . The model of desire length was embedded in sand .The rate of settlement for the device was chosen and operated . The load-settlement readings will appear on device display, this recorded readings will be plotted as load-settlement curves Figure (3-13) below shows the bored model installation. .(**ALI,2012)**

Figure (3-13). Bored pile model installation

3.6.2 <u>Driven Pile Procedure:</u>

The arm of device is lifting to distance provide enough length for both pile model length and load cell which connected each other. Soil will prepare with desired density by raising the funnel with the desired height and filling the container with soil by dropping it with desired height according to raining sand technique results. The pile was connected to load cell of device. Pile model will reach to soil surface. Start with the lowering arm of device, which makes the model penetrates soil until desired length. Choosing the rate of settlement has been done. The device was operated ,pile model will start penetrating the sand. The load-settlement readings will appear on device display, this recorded readings will be plotted as load- settlement curves Figure (3-14) below shows the driven pile installation. (**ALI,2012)**

Figure (3-14). Driven pile model installation

3.7 <u>Improvement Surrounding Soil Performance</u>

In the case of improving surrounding soil (PVC) pipe needed to use as casing pipe. this pipe with different diameters ranges (32 ,43 ,73,100) mm considered as a limits of improving area and ($D_{improvement}$ /D_{pile}) equal to (2.4, 3.3,5.6,7.6) respectively. Figure (3-15) shows the casing pipe using in case of improving surrounding soil (bored –driven). This pipe Specifies with line around the pipe to determine the length of embedded part in sand.

Figure (3-15). Casing pipe using in case of improving surrounding soil (bored –driven)

3.7.1 Performance Improvement soil in driven Case:

In driven case the pipes were chosen with a length equal to the desired length of model pile (100mm or 150mm) this determines by line around casing pipe. Al-Ekhidher sand (type 2) were prepared with optimum moisture content that was obtained from proctor test. The case pipe (PVC) was installed with required diameter and desire length ,in considering with the centralization the pile model in case pipe (pile will be at the center of pipe). The deposits of soil are prepared with desire density and soil will be raised in container ,with certain thickness according to rain fall technique results. After filling the container with black sand (type 1), .Starting to fill the case pipe with al-Ekhidher sand as equal layers approximately (3cm for each layer) and start to compact it with steel bar with equal blows distributed equally and with spirally around pile model (20-40) blows according to pipe diameter, till getting no more response for compacting blows. Rising the the case pipe in conjunction with compaction process to facilitate extracting the casing pipe, and to interlocking the improvement block with surrounding soil and to prevent adhesion between casing pipe and compacted soil. The arm of device was lowered to the desired length ,the pile model will penetrate the improved zone. The device was operated and the readings for load-settlement was recorded then plotted the load-settlement curves.

3.7.2 Performance Improvement soil in Bored Case:

Casing pipe with different diameters were used , length about (7-8) cm and longitudinally separated from one side, this splitting is to make it easier to extract it after compacting surrounding soils, because the existing pile model. The improving surrounded(replaced) soil was compacted by layers with the existed casing pipe with the same level of the natural soil without improving . The procedure of improving operation requires preparing the soil for bored pile by the same way of performing without improvement . When the natural soil arrive at pile model toe casing pipe longitudinally splitting from one side was installed .After installation the casing pipe the filling the container with natural soil Continues in surrounding zone. When natural soil layer reaches at top of casing pipe al-Ekhidher sand (improved soil) were added with optimum moisture content inside the casing pipe and compacting it with the same procedure (same technique and same number of blows) as in the tests in driven piles . All this steps are carried out with existing pile model and the casing pipe was pulled at the same time, the improved sand was compacted. This procedure performed frequently till the sand arrives sand to desired length. This pulling of case is to prevent the adhesion between the compacted sand and the case pipe , the casing pipe was extracted by opening its split longitudinally and extracting it out. After preparing the sand , model and the improving zone, the device was operated with the rate of settlement previously chosen and start to record readings of load-settlement for specific range. When the rate of settlement increases obviously with slightly increment in load that gives indication on pile capacity resisting. Figures (3-16) and (3-17) shows the step of improvement and installation for both case (driven-bored) in improved and without case.

(a) (b)

Figure (3-16). Driven model pile in case of improvement surrounding soil

(a) (b)

(c)

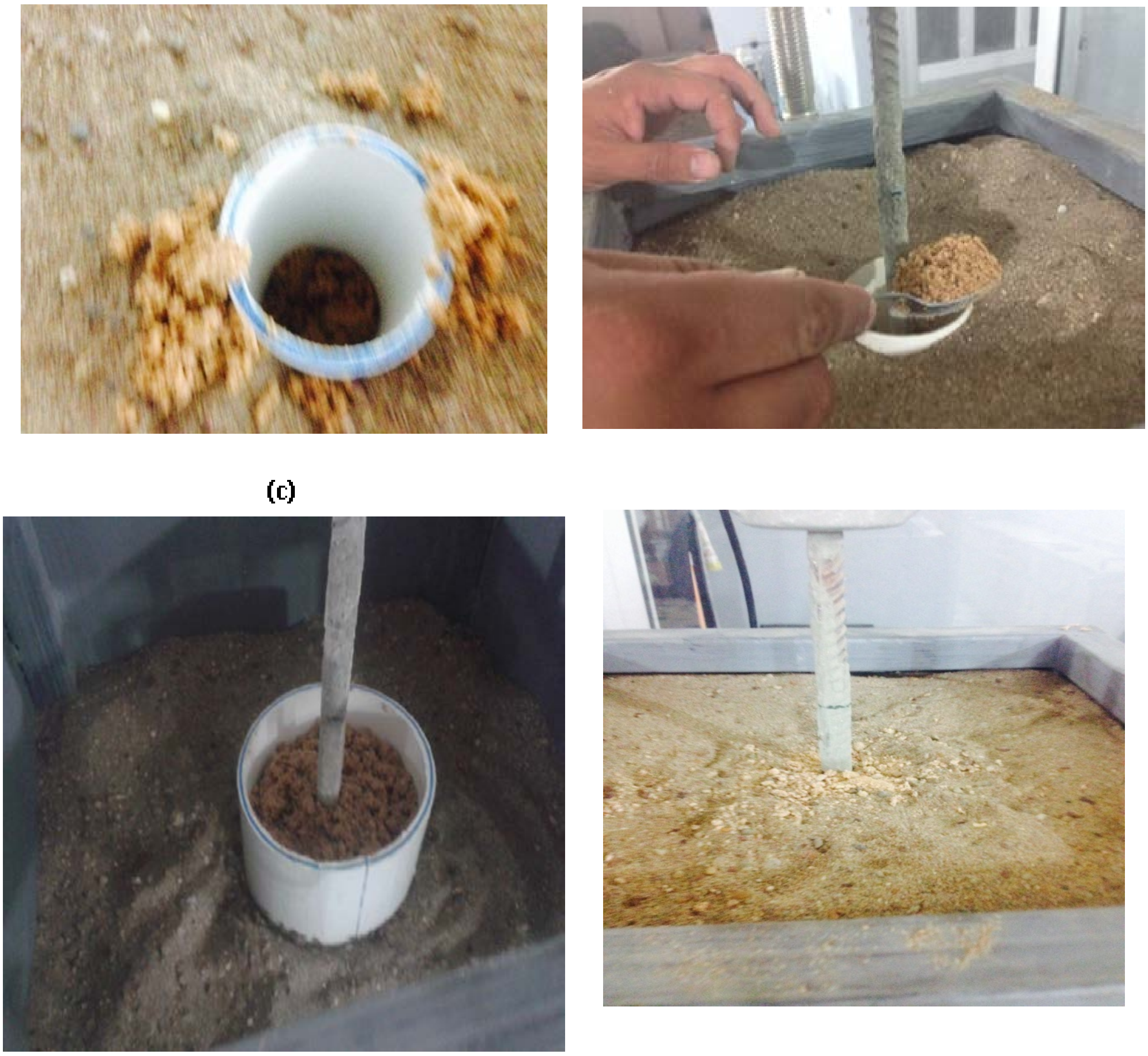


Figure (3-17). Bored pile model with improvement surrounding soil

CHAPTER FOUR

Presentation and discussion of test results

4.1 Introduction

This chapter presents results of pile load tests carried out on pile model to study and investigates the behavior of pile after compacting pile surrounding soil along pile length ,for both (driven and bored) pile and for two different Instilled length. The soil compacted does not extend beneath the pile tip. The purpose of restrict the improvement just about surrounding soil is observe the friction improvement due to the compaction.

The black sand (type 1) was chosen to be the Peripheral soil to be less intense as possible The container has been filled with sand at the lowest possible density using rain-fall technique. On this basis height (5 cm) has been chosen for sand dropped from funnel. For simulating Peripheral soil as loose as possible. PVC pipe use as cases to contains the soil type (2) which is adding as replacing soil of surrounding soil. A steel bar were used for compact soil with equal blows on each layer. Each layer has height of (3cm) driven pile and bored pile are performed with special technique corresponding to method of installation of pile model in container as explain in chapter three (experimental wok).

This chapter is studying the variation of pile load capacities value after observation and discuss load settlement curve for each test, depending on load and settlement readings obtained from each test. Each test simulates a case of improvement, length and installation. These three parameters are changes respectively to notice their relationship with each other and its effect on pile-load capacity. The two tangent methods are adopted to estimate the pile load capacity from the tests results. The pile load capacities are compared for each case. Increment ratio and effect of length is studying and discussing the final magnitudes for all cases are collected in table as summary. The investigation discussed the feasibility of using the compaction as improvement choice to increase the friction factors capacity of pile and the most effected method of installation by compaction ,and the most effective for economic compaction diameter around pile.

4.2 Effect of Load Rate and Instillation Method on Improved Soil Block

Method of installation of pile has effected on surrounded soil. Commonly driving of pile densifies the soil around pile except the dense soil that is may be loosen by driving pile, This affect appears clearly in driving pile due to hammering down or by jacking pile. The main aim of this thesis is to discuss the affect of compaction surrounding soil on pile load capacity, so it is important that the improved soil structure must be kept for this purpose. Constant rate of penetration was chosen to drive the pile model down to avoid vibration and affect of hammering on surrounded. For the bored pile this problem was not appear because the nature of installation of bored pile keeps the improved soil block as one block and that is why the affect of compaction on piles surrounded soil gives more increment in the pile load capacity for bored type. The affect of compaction of surround soil combined with keeping the improved soil block coherent appears obviously in bored pile type and lesser in driven pile .

4.3 The Effect of Surrounding Soil Compaction on Pile Load Capacity

4.3.1 Effect of Surrounding Soil Compaction on Driven Pile Load Capacity

Many tests are carried out to compare the different cases that this study examined as driven pile set. At first step pile-load test performed on pile model without improvement to simulate the natural case this case is performed for two different lengths. The first test results was for tested model without improvement and with driven length equal to 15cm and the results was as shown in Figure (4-1) the estimated pile load capacity was (0.015kN) and Figure (4-2) shows the driven pile model loaded in improved surrounding soil.

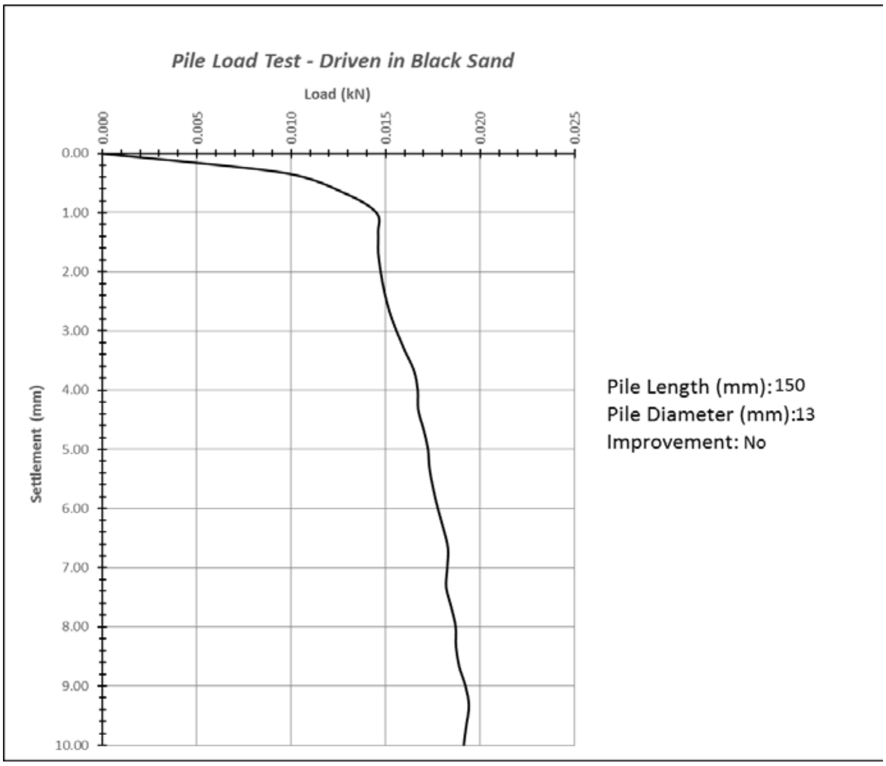


Figure (4-1). Relationship of Driven Pile model (150mm) loaded in natural black sand with settlement

Figure (4-2). Driven pile model loaded in improved surrounding soil

The second test was performed on pile model with length (100mm) as driven pile in soil without improvement the estimated pile load capacity was (0.0095kN) as shown in Figure (4-3).

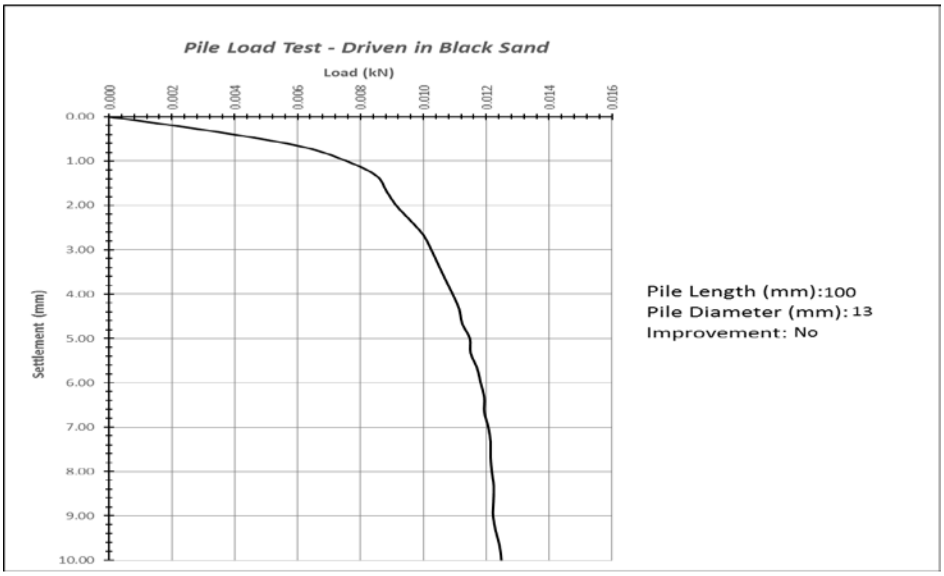


Fig (4-3). Relationship of Driven Pile model (100mm) loaded in natural black sand with settlement

4.3.1.1 Improvement Surround Soil with (32mm) ($D_{IMPROVEMENT}/D_{PILE}$=2.4) Diameter around Pile Model

Test performed on pile model with length (150mm) and improvement surrounding soil with diameter (32mm) the calculated pile load capacity (0.0195kN) as shown in Figure (4-4).

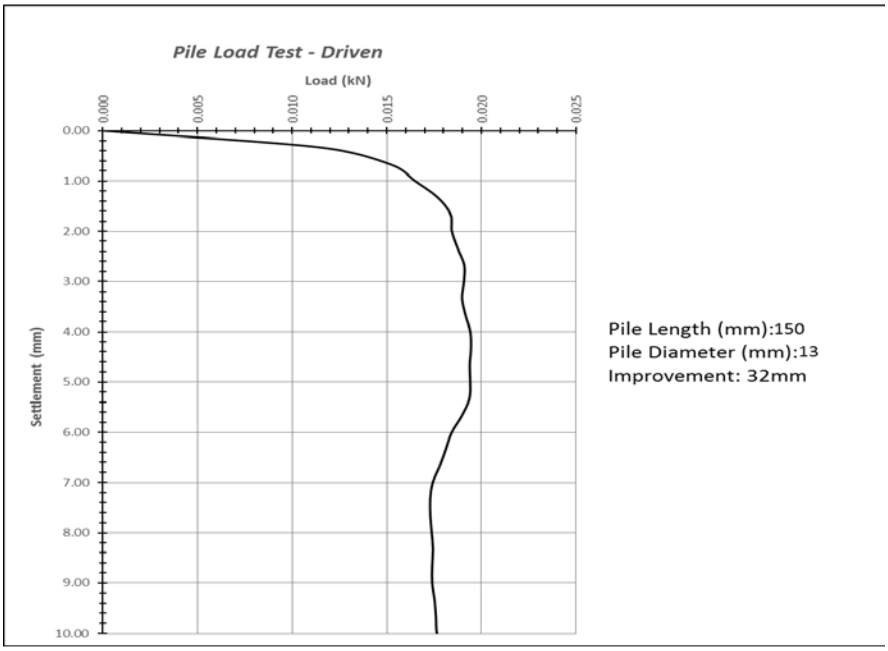


Figure (4-4). Relationship of Driven Pile model (150mm) loaded in natural black sand improved soil diameter (32mm) around pile with settlement

Other test carried out on pile model with length (100mm) and compacted soil surround pile with diameter (32mm) calculated pile load capacity was (0.0114kN) as shown in Figure (4-5).

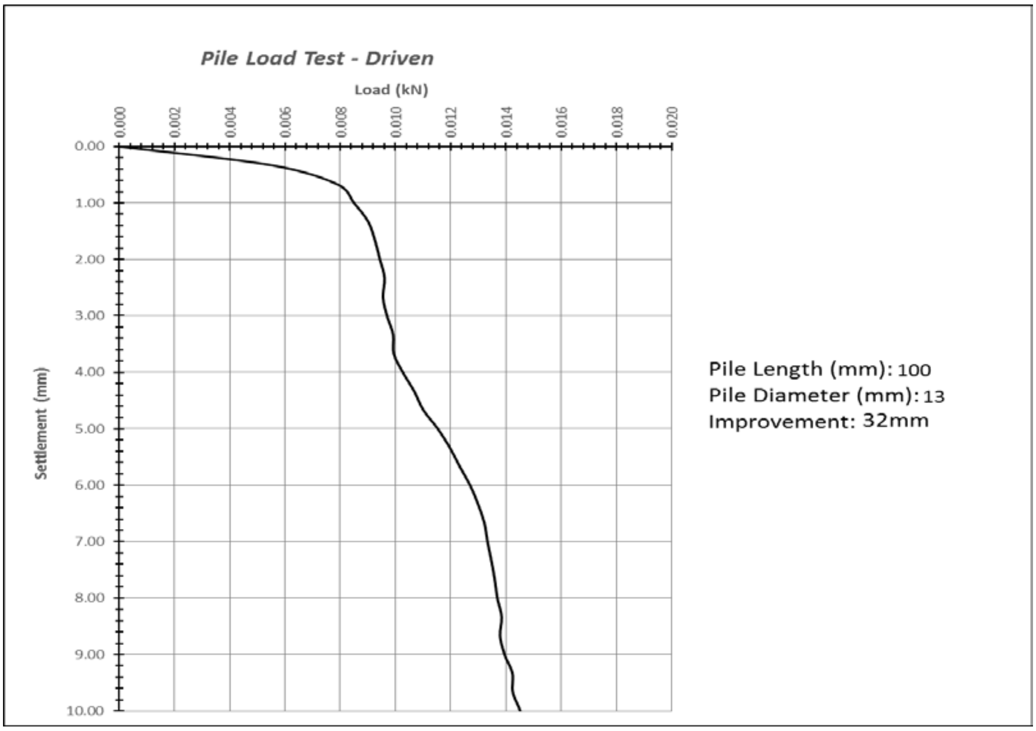


Figure (4-5). Relationship of Driven Pile model (100mm) loaded in natural black sand improved soil diameter (32mm) around pile with settlement

4.3.1.2 Improvement Surround Soil with (43mm) ($D_{IMPROVEMENT}/D_{PILE}$=3.3) Diameter around Pile Model

The next stage of improvement was improve the surround soil with wider diameter the new diameter was increase to be (43mm) this range of increment in improvement accompanied by noticeable rise in pile load capacity. Tested pile model with length (150mm) produced pile load capacity (0.047kN) as shown in figure (4-6).

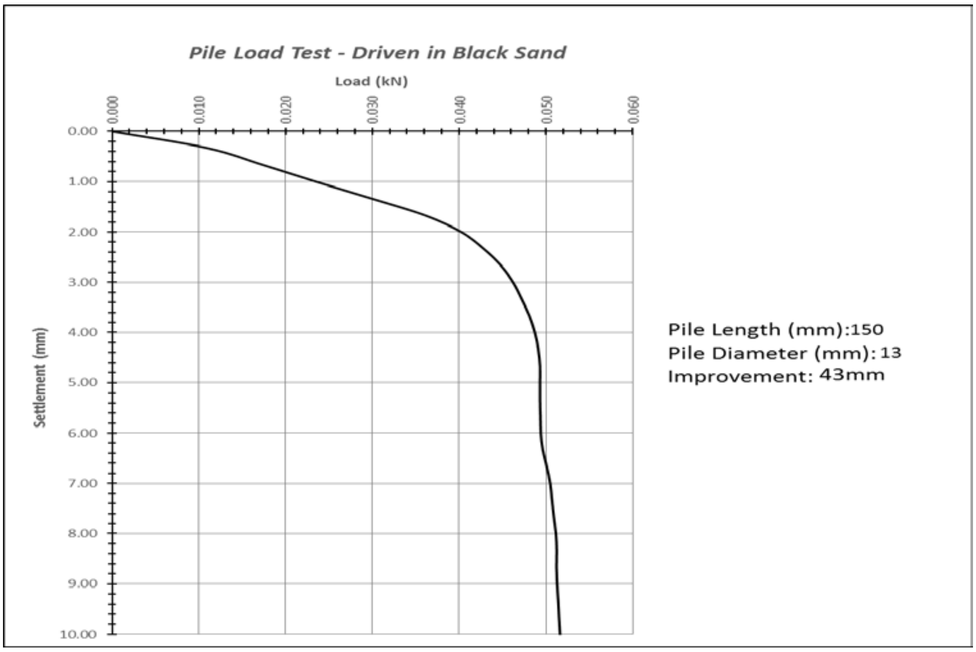


Figure (4-6) Relationship of Driven Pile model (150mm) loaded in natural black sand improved soil diameter (43mm) around pile with settlement

Other test performed on pile model with length (100mm) and compacted surround soil with diameter (43mm) the calculated pile capacity was (0.033kN) as shown in Figure (4-7).

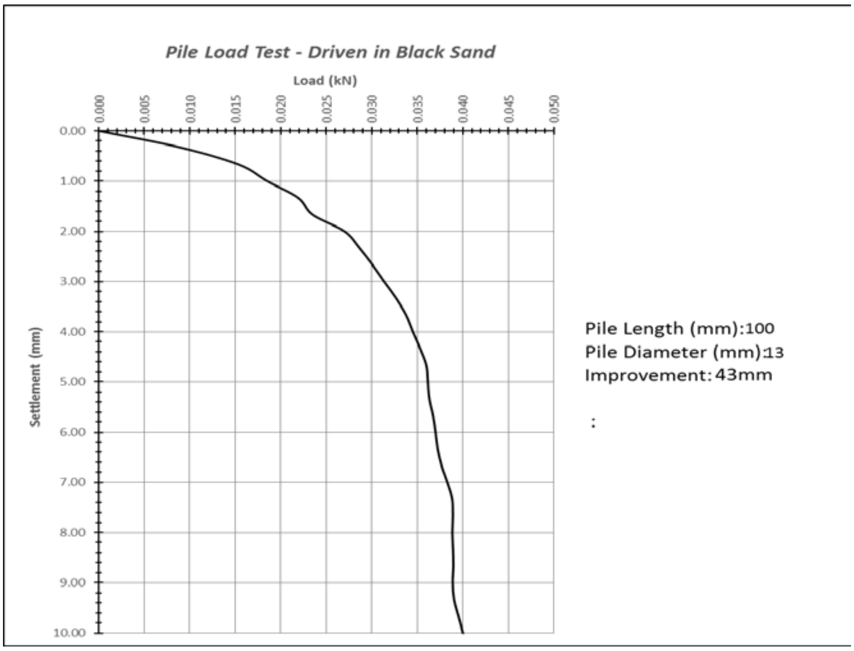


Figure (4-7). Relationship of Driven Pile model (100mm) loaded in natural black sand improved soil diameter (43mm) around pile with settlement

4.3.1.3 Improvement Surround Soil with (73 mm) ($D_{IMPROVEMENT}/D_{PILE}$=5.6) Diameter around Pile Model

Other increment in diameter of improved soil around pile model executed in this stage the improvement soil block was increased to be with diameter (73mm) around pile model. First test was carried out with pile model length (150mm) the calculated pile load capacity in this test was (0.067kN) as shown in Figure (4-8) below.

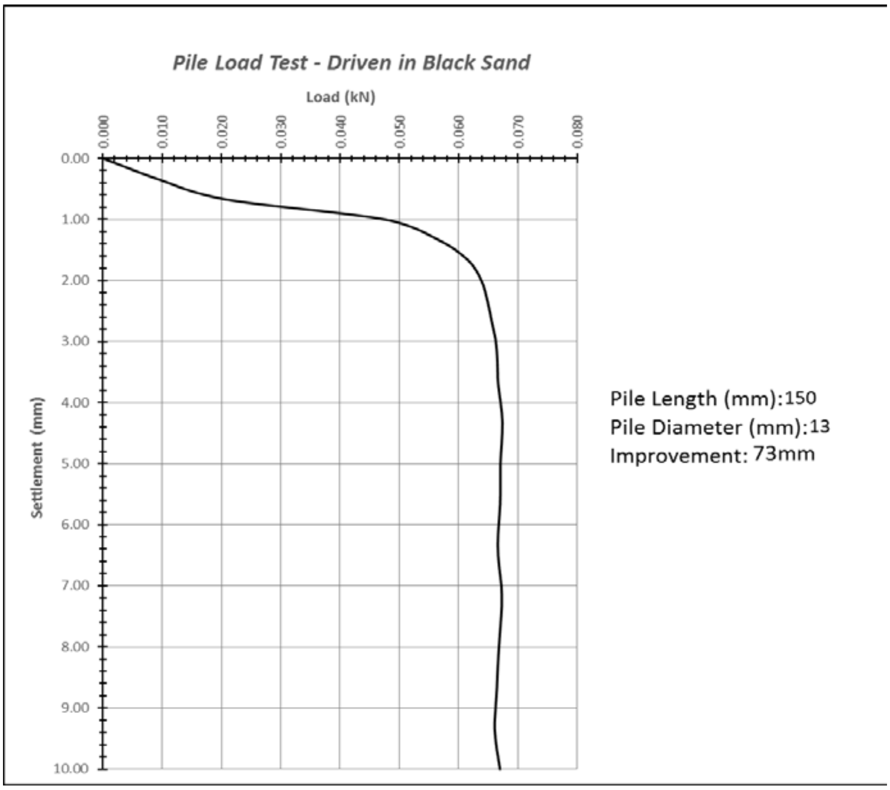


Figure (4-8). Relationship of Driven Pile model (150mm) loaded in natural black sand improved soil diameter (73mm) around pile with settlement

Other test was performed on pile model with length (100mm) the calculated pile load capacity was (0.056kN) as shown in Figure (4-9).

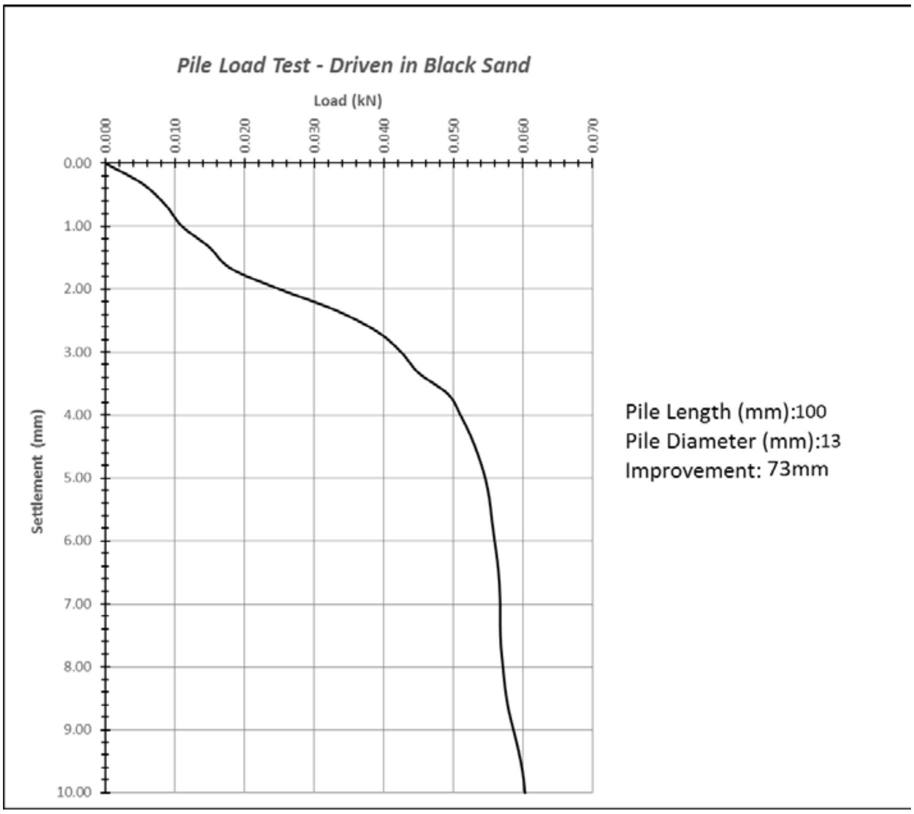


Figure (4-9). Relationship of Driven Pile model (100mm) loaded in natural black sand improved soil diameter (73mm) around pile with settlement

4.3.1.4 <u>Improvement Surround Soil with (100mm) ($D_{IMPROVEMENT}/D_{PILE}$=7.6) Diameter around Pile Model</u>

Last stage of increment in diameter of improved soil around pile model was performed with diameter of (100 mm) this increment considered last stage for improvement for reasons will be explained in discussions of results .as usual test performed for two length of pile model test carried out for (150mm) the calculated pile load capacity was (0.07kN) as shown in Figure (4-10).

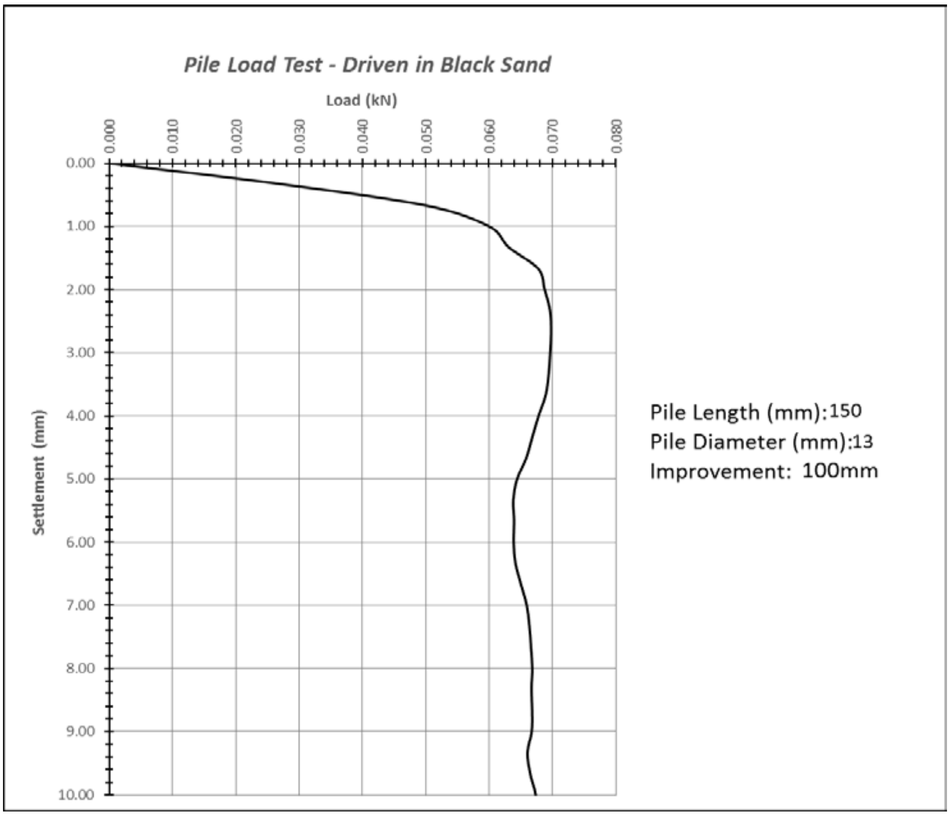


Figure (4-10). Relationship of Driven Pile model (150mm) loaded in natural black sand improved soil diameter (100mm) around pile with settlement

Other test was performed for (100 mm) the calculated pile load capacity was (0.058kN) as shown in Figure (4-11).

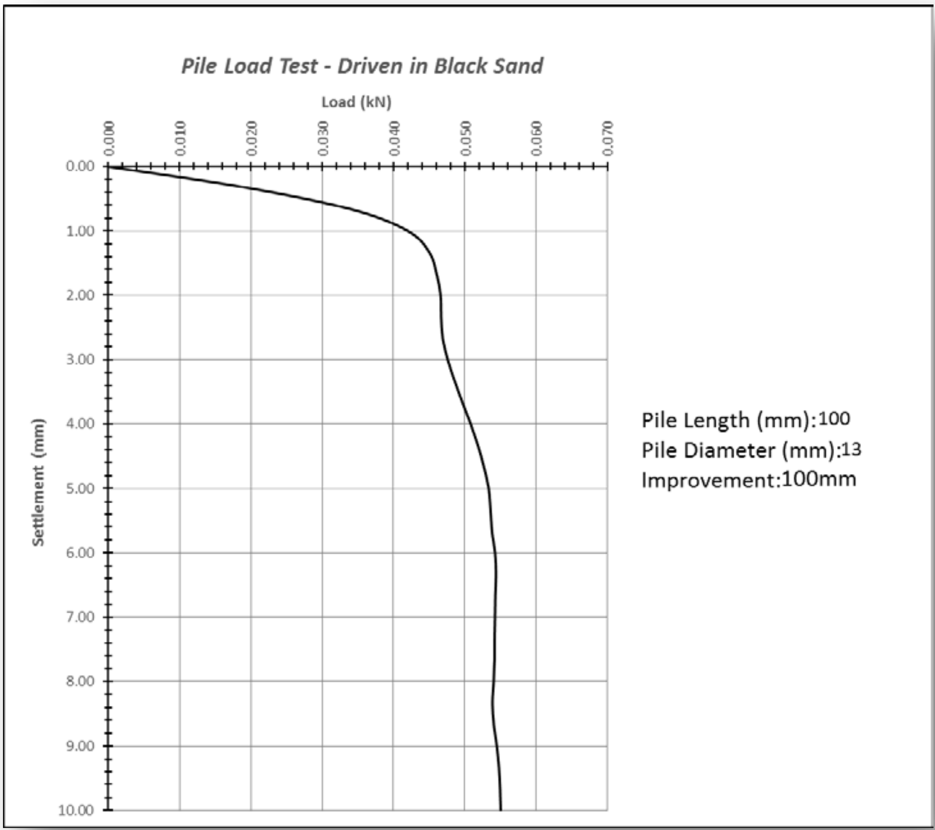


Figure (4-11). Relationship of Driven Pile model (100mm) loaded in natural black sand improved soil diameter (100mm) around pile with settlement

After determine the pile-load capacity for each test results curve plot relation was plotted between the diameter of improvement around pile and pile load capacity for driven pile case to determine the behavior of effect curve with the increasing the diameter of improvement area as shown in Figure (4-12) ,this figure show that the pile load capacity has increase in pile load capacity at the first stage of improvement (32mm)($D_{IMPROVEMENT}/D_{PILE}$=2.4). The increasing in pile load capacity at (43mm) ($D_{IMPROVEMENT}/D_{PILE}$=3.3) diameter of improvement has clear effect on pile load capacity the value of pile load capacity increase approximately to (0.047, 0.33) kN for (150,100) length respectively after improve the surround soil compare with the natural case. Clear increase in pile load capacity at (73mm) ($D_{IMPROVEMENT}/D_{PILE}$=5.6) pile model capacity increase to (0.067 and 0.056 kN) for (150,100 mm) respectively. The same behavior observed in case of (100mm) ($D_{IMPROVEMENT}/D_{PILE}$=7.6) pile load capacity increase to (0.07 and 0.058 kN) for (150 ,100mm) respectively in case of improvement (100 mm) And its has little effect on the pile load capacity drove in sand appears in magnitude of pile load capacity but both have same behavior with respect to improvement ,That's because of effect of compaction appear by rearrange the soil particles and increase the interlocking between particles and make the compacted block firmer and that naturally increase friction between the block and pile model. The value of cohesion of soil particles will increase by compaction that's lead to increase the friction factor between the compacted block and pile model. The density of soil around pile model increase that is lead for increase friction, but this effect according to experimental work may inserted in specific range around pile mode so beyond limits of (43mm) this effect start to be lighter. But there is Reverse effect appears because of driving the pile model presents in collapse almost part of soil compacted block and divided into parts. The increment in pile load capacity was clear at diameter of (43mm) ($D_{IMPROVEMENT}/D_{PILE}$=3.3) improvement surrounding soil and there is an obvious effect at (73mm) ($D_{IMPROVEMENT}/D_{PILE}$=5.6) improvement surrounding soil but the most effected and economic choice was (43mm) ($D_{IMPROVEMENT}/D_{PILE}$=3.3) because the increment in the pile load and for economic consideration .

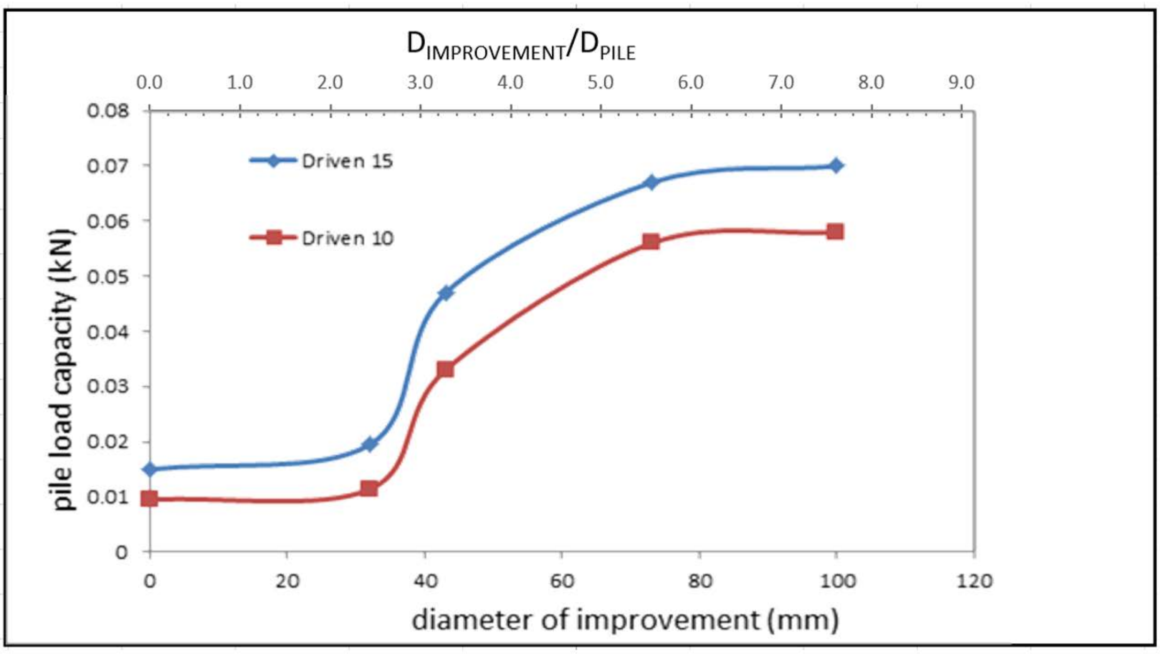


Figure (4-12). The relationship between pile load capacity and diameter of improvement Surrounding soil

4.3.2 The Effect of Surrounding Soil Compaction on Bored Pile Load Capacity

As stated previously, the process of keeping the improving soil as block and working as one body, a distinct effect on increasing pile load capacity ,and because of method of installation of pile model in case of bored pile which is based on compact the surrounding soil in existing pile model which is lead to keep the soil block as one block. Figures (4-13) and (4-14) show the bored pile model installation. In this phase of testing executing improvement with (32mm) diameter cannot be executed because the impossibility of execution with available tools. Load-settlement curve are plotted for natural case (without improvement) its limited on filled container with soil type 1 (black sand). Test performed for two length of pile (100 and 150) mm the estimated pile load capacity for the pile model with length (150mm) was (0.0058kN) as shown in figure (4-15).

Figure (4-13). Bored pile model installation

Figure (4-14). Bored pile model test

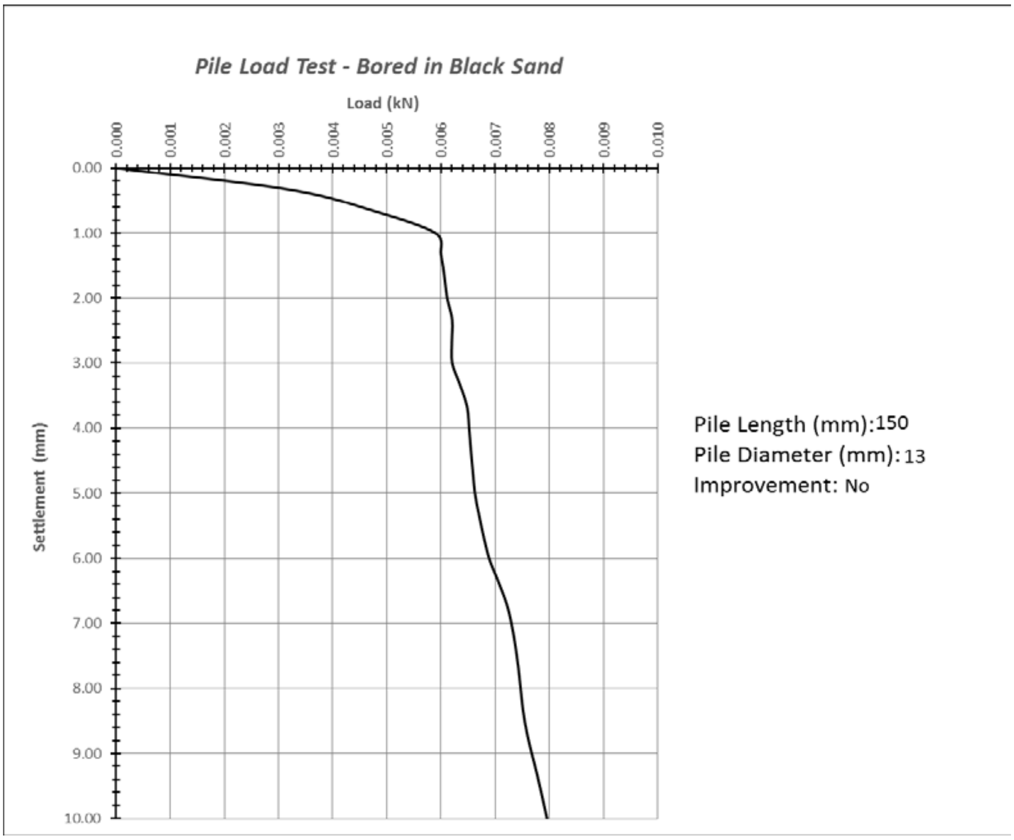


Figure (4-15). Relationship of bored Pile model (150mm) loaded in natural black sand with settlement

The other test for same conditions but with length (100mm) the calculated pile load capacity was (0.0038kN) as shown in Figure (4-16).

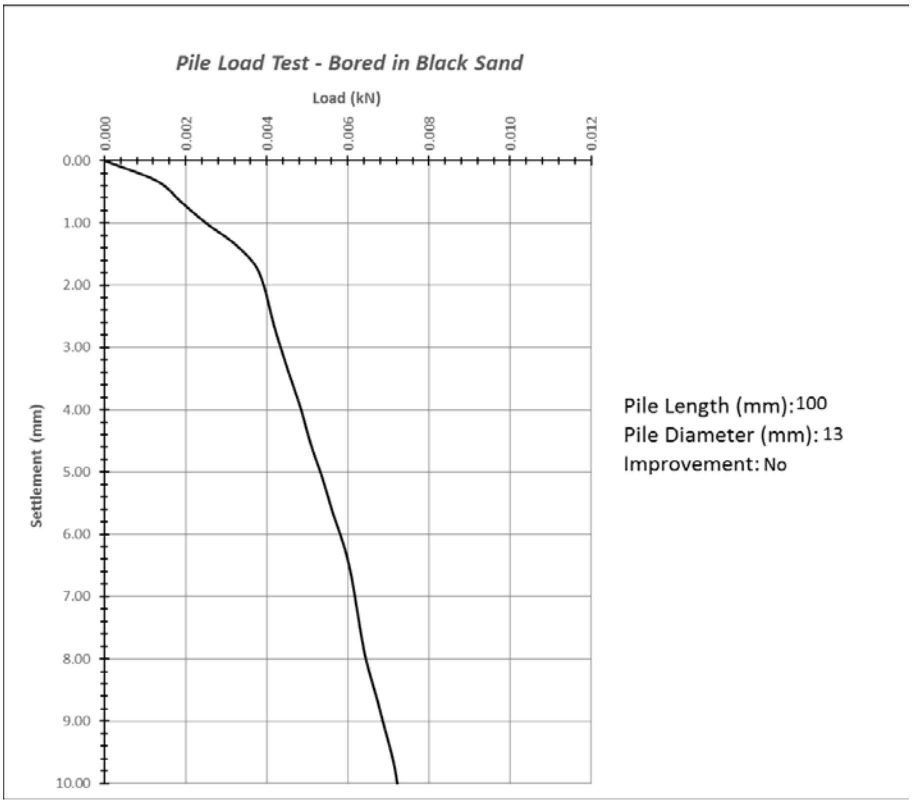


Figure (4-16). Relationship of bored Pile model (100mm) loaded in natural black sand with settlement

4.3.2.1 Improvement Surround Soil with (43mm) ($D_{IMPROVEMENT}/D_{PILE}$=3.3) Diameter around Pile Model

Soil surrounding pile model compacted with existing of pile model this will lead to more interlocking between the pile model and soil compacted and pile model which caused increase in pile load capacity. The area of compacted soil was located with diameter (43mm) and test carried out with two length also the estimated pile load capacity for (150mm) was (0.114kN) as shown below in Figure (4-17).

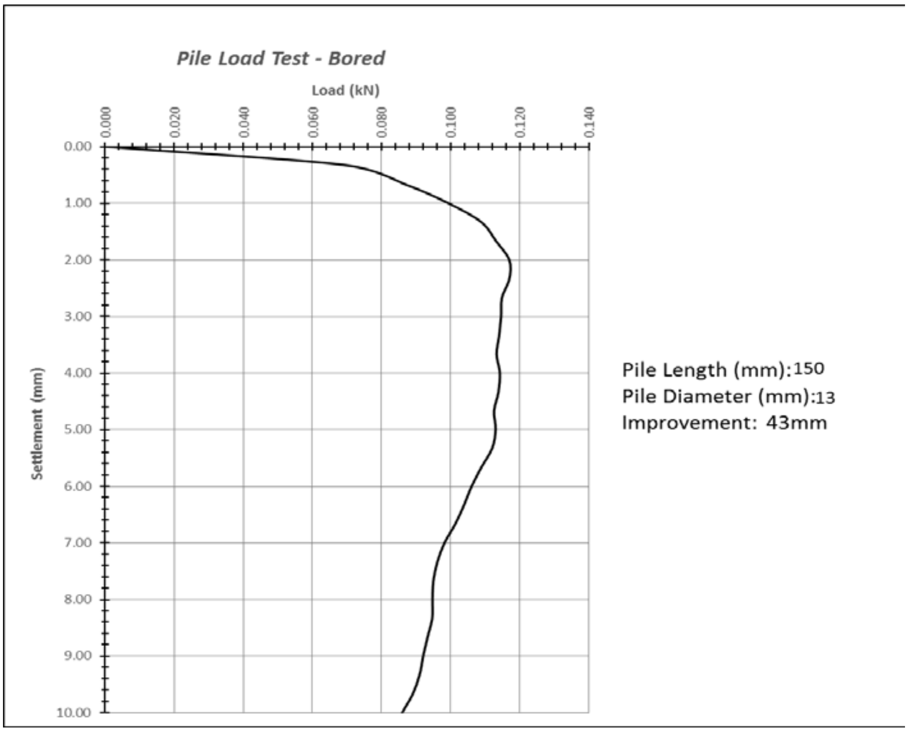


Figure (4-17). Relationship of bored Pile model (150mm) loaded in natural black sand improved soil diameter (43mm) around pile with settlement

Other test was carried out for (100mm) length the calculated pile load capacity was (0.073kN) as shown in Figure (4-18).

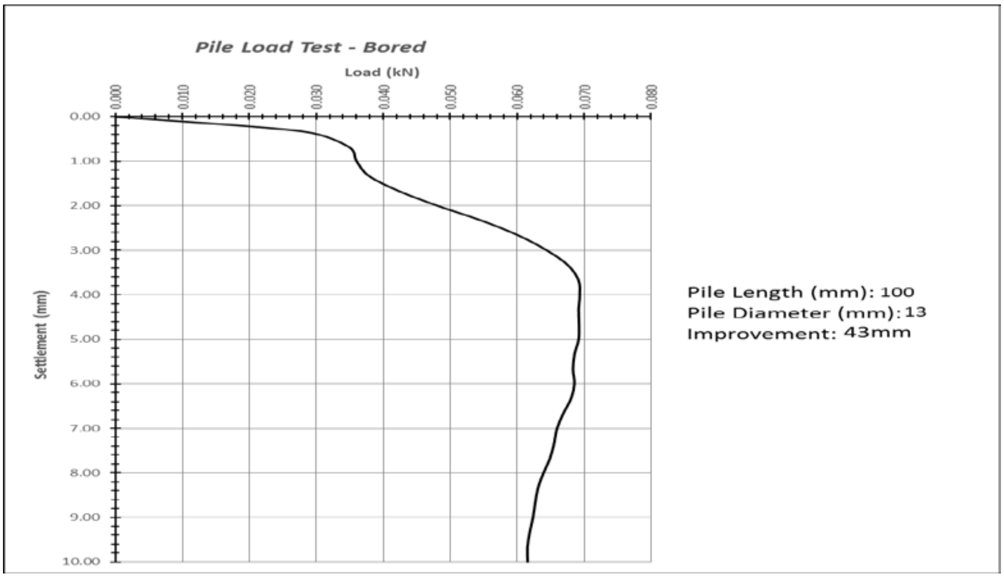


Figure (4-18). Relationship of bored Pile model (100mm) loaded in natural black sand improved soil diameter (43mm) around pile with settlement

4.3.2.2 Improvement Surround Soil with (73mm))($D_{IMPROVEMENT}/D_{PILE}$=5.6) Diameter around Pile Model

To investigate and compare the effect of compaction on both bored and driven pile the same stages of improvement are performed as in driven case. The diameter of improvement area was increased to (73mm))($D_{IMPROVEMENT}/D_{PILE}$=5.6) and tests are performed for two length also (150 and 100) mm the estimated pile capacity for (150mm) was (0.144kN) as shown in Figure (4-19).

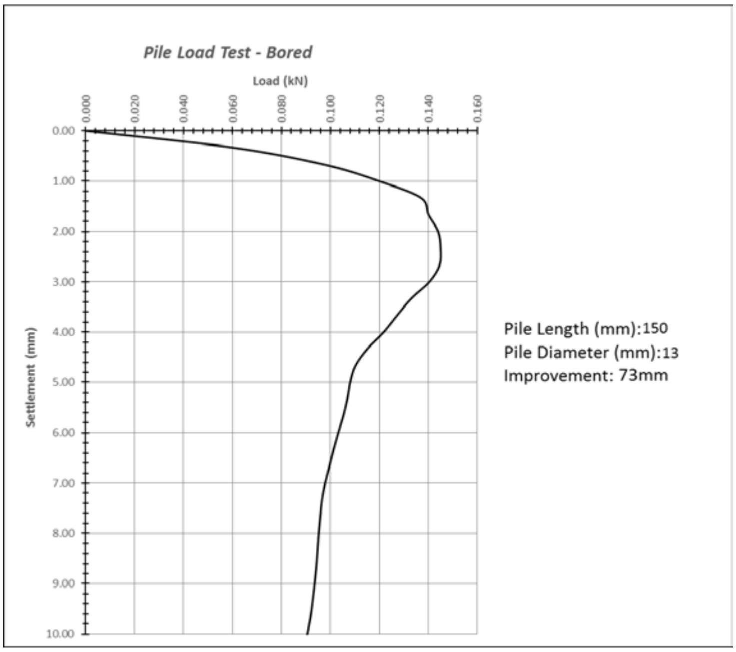


Figure (4-19). Relationship of bored Pile model (150mm) loaded in natural black sand improved soil diameter (73mm) around pile with settlement

The other test was performed for (100mm) the estimated pile load capacity was (0.103kN) as sown in Figure (4-20).

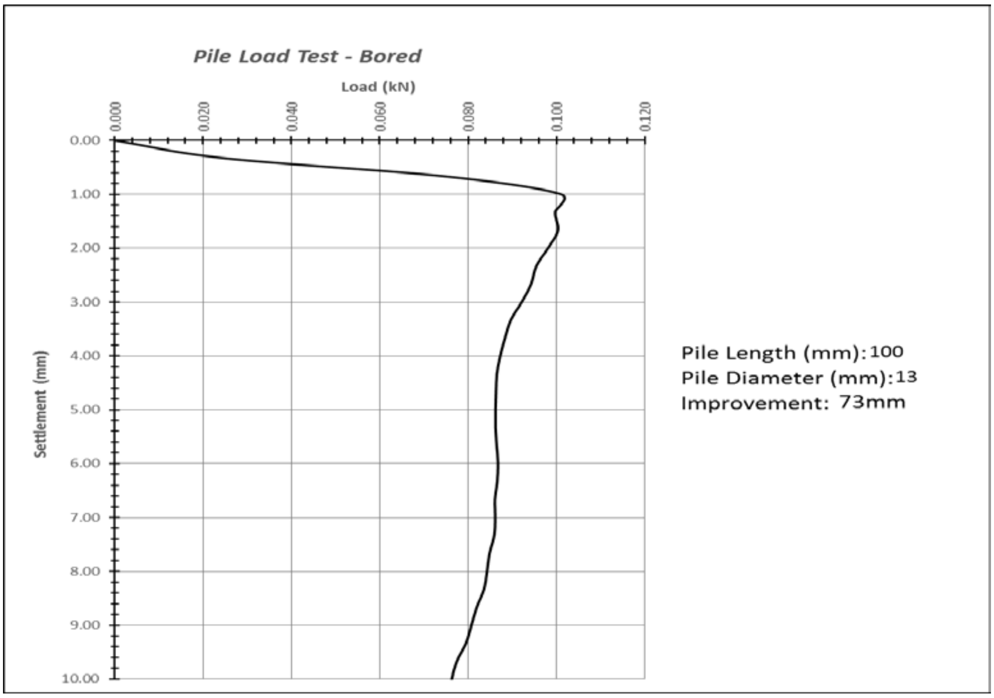


Figure (4-20). Relationship of bored Pile model (100mm) loaded in natural black sand improved soil diameter (73mm) around pile with settlement

4.3.2.3 Improvement Surround Soil with (100mm))($D_{IMPROVEMENT}/D_{PILE}$=7.6) Diameter around Pile Model

The last stage of improvement carried out by compact the surrounding soil with area limited by (100mm))($D_{IMPROVEMENT}/D_{PILE}$=7.6) diameter. The first test performed with length (150mm) the calculated pile load capacity was (0.150kN) as shown in Figure (4-21).

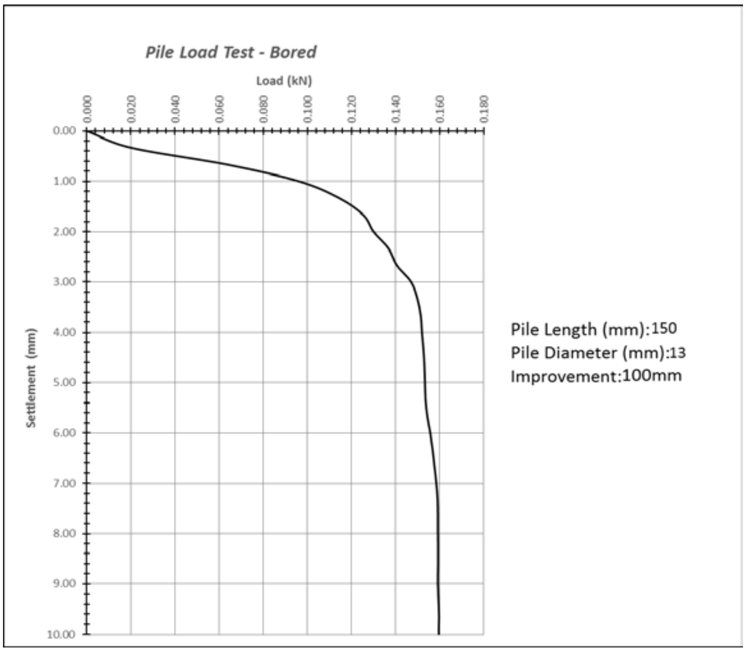


Figure (4-21). Relationship of bored Pile model (150mm) loaded in natural black sand improved soil diameter (100mm) around pile with settlement

The other test performed for length (100mm) the calculated pile load capacity was (0.110kN) as shown in Figure (4-22).

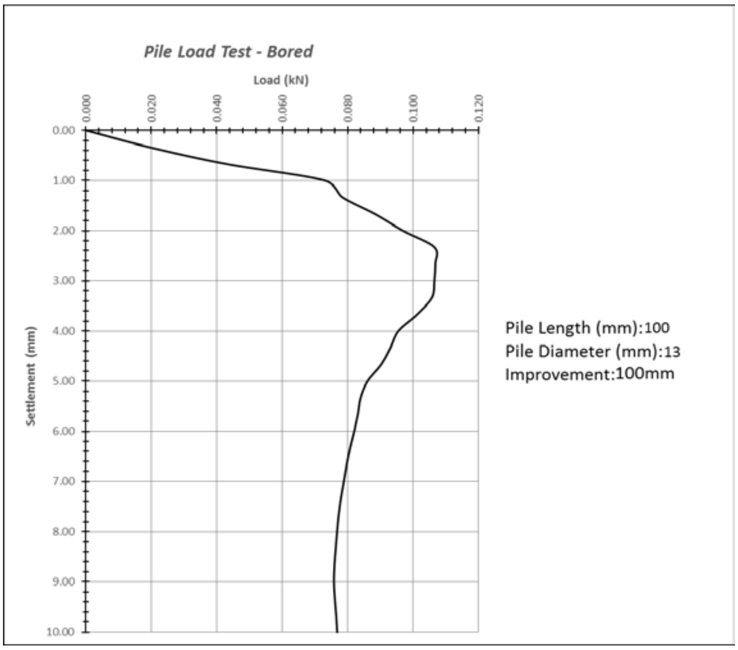


Figure (4-22). Relationship of bored Pile model (100mm) loaded in natural black sand improved soil diameter (100mm) around pile with settlement

For the case of bored pile notice was observed that most of curves has decrease in pile load in load-settlement curves that's explained by the area under the tip will not compacted of effected with the same degree of compaction with soil under block so this create weak zone under tip. After determine pile load capacity for each test result curve relation was plotted between the diameter of improvement and pile load capacity and the method of installation of pile and curves obtained in Figure (4-23).

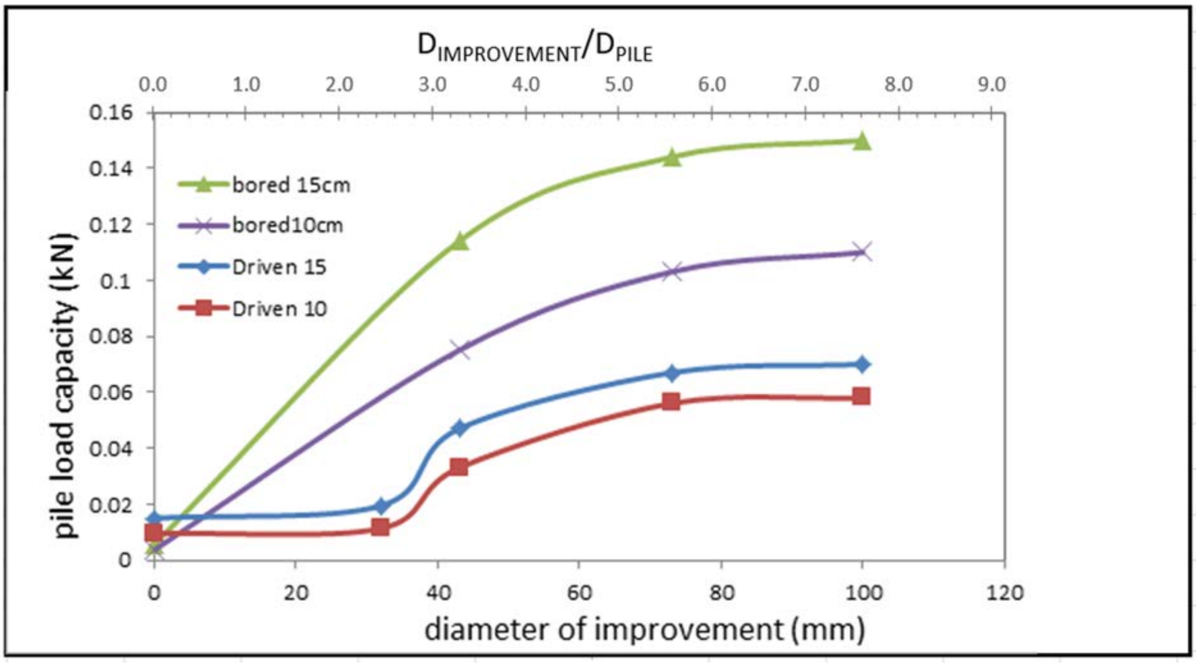


Figure (4-23). Relation between improvement soil diameter and pile load capacity due to two method of installation

From previous relation conclusion created that the effect of compaction appear clearly when the structure of compacted soil which surround pile still as one block, for that reason the compaction is more effective in case of bored pile that's belongs to compaction of surrounding soil performed in presence of pile model . That lead for more interfering between the pile model and compaction lead for more interlocking between soil particles which may be causing increase apparent cohesion of soil and make soil block firmer. This cohesion may be causing by cementing of compounds in the soil like(Fe_2O_3, $CaCO_3$, NaCl) which may increase the cohesion to high values , other reason causing the "Apparent cohesion" , because addition amount of water for compaction As soil dries out, water menisci form at grain contacts. These are under negative capillary pressure. “Matric suction stress” referred to when capillary forces are netted over a unit area, , which creates interparticle forces called "apparent cohesion" many reasons may cause this cohesion due to pore-pressure response during relatively fast (“undrained”) loading, increased total normal stress. A little bit of cohesion or apparent cohesion make big difference to the stability of sandy soil ,this can be demonstrate in several ways. The effect of apparent cohesion was in preventing shallow foundation sloughing in sand slope (**GRAY, ,1996**) . Stability of this structures is most effective parameter in rise of pile load capacity. And compaction in presence of pile model and withdraw casing pipe will lead to interfering between soil compacted block and surrounding soil . Although this block consider to be one firm block but it’s still a brittle structure and that’s what explain curves down after reach ultimate pile load capacity in bored pile case. At start for explain of effect of compaction on driven piles we must refer to that in driven piles improvement can be performed at diameter of (32mm) which it's unable to perform in bored pile with existing devices, due to the residual tight perimeter in existing of pile model. As mentioned previously the most effective factor causing increasing pile load capacity is remaining of improved soil block as one block. In case of driven pile this factor was decreasing clearly because of breaking soil improved block into parts due to driving of pile and loose soil surrounds improved soil block which not supported the block. so this block are breaking due to pushing of pile drive load and weak support of surround soil. If the improvement was with insufficient diameter (32mm))($D_{IMPROVEMENT}/D_{PILE}$=2.4) that's will lead for little effect because of small volume of improving block and breaking of this part will change it to small parts with small contact area with pile model and that's causing small value of frictional force that's appear in Figure (4-23).the stability and keeping of improved soil block as one block belong to increment in values of (C,Ø) and this may belong to effect of chemical components.

Figure (4-23) shows that the both driven and bored pile have same behavior after improving surrounding soil that appear obviously in increment of pile capacity after compaction surrounding soil specially in the both cases the most noticeable increment occurs in case of (43mm))($D_{IMPROVEMENT}/D_{PILE}$=3.3) of improving surrounding soil, and both increments occurs beyond this limit with limit that's explain important factor that's most effected range by compaction and most range effect on pile load capacity is the region surrounds pile with sufficient area. If the area of improving area around pile related with pile model diameter (D))($D_{IMPROVEMENT}/D_{PILE}$=3.3), According to our study distance estimated to be equal to ($D_{IMPROVEMENT}/D_{PILE}$=2.4) in case of (32mm). This distance will not provide sufficient region for frictional parts to bond and provided more friction, ($D_{IMPROVEMENT}/D_{PILE}$=5.6) distance will lead to slight increment compared with increment of pile load capacity due to)($D_{IMPROVEMENT}/D_{PILE}$=3.3). The same behavior applies on ($D_{IMPROVEMENT}/D_{PILE}$=7.6) distance of improving surrounding soil. Table (4-1) summarized the percentage of increment of pile capacity due to improvement surrounding soil :

Table. (4-1) the percentage of increment in pile load capacity compare with the natural case for both driven and bored pile .

Type	**L (mm)**	**($D_{IMPROVEMENT}/D_{PILE}$=2.4) Percentage of increment %**	**($D_{IMPROVEMENT}/D_{PILE}$=3.3) Percentage of increment%**	**($D_{IMPROVEMENT}/D_{PILE}$=5.6) Percentage of increment %**	**($D_{IMPROVEMENT}/D_{PILE}$=7.6) Percentage of increment %**
Bored	150	-	1865	2382	2486
	100	-	1321	2610	2794
Driven	150	30	213	346	366
	100	20	135	300	314

The figure(4-23) give indication that in both cases (driven-bored) that the difference between two length (100 and 150)mm is almost constant this belongs to the constant difference of length lead to decrease Perimeter area and that's lead to decrease in friction force.

The pile load capacity difference between bored and driven pile due to the behavior of compacted soil block in case of bored pile it worked as one block as shown in figure (4-24) and figure (4-25)

Figure (4-24). Improved soil block at the end of test show the soil block as one block

Figure (4-25). Improvement block with pile model behave as one block

keeping of compacted soil block as one body and the weak support of soil surround that block due to loose density all this made this block behave as one block and that's mean more friction force and more tip load. All these reasons lead to increase in the pile load capacity.

Soil block in bored case keep it structure constant approximately as shown in figure (4-26).

Figure (4-26). Soil block in bored case keep it structure constant approximately

4.4 Results Summary

Table (4-2) the summary of experimental tests results

Improving case	Pile type	Pile length (cm)	Pile capacity (kN)	Settlement (mm)	Stress (MPa)
Without improving	Bored	15	0.0058	0.42	0.0436
		10	0.0038	1	0.028
	Driven	15	0.015	0.4	0.113
		10	0.0095	1.05	0.120
Improving dia. (43mm)	Bored	15	0.114	0.4	1.28
		10	0.075	0.8	0.376
	Driven	15	0.047	1.2	0.345
		10	0.033	1.19	0.245
Improving dia.(73mm)	Bored	15	0.144	0.8	1.084
		10	0.103	1	0.828
	Driven	15	0.067	1.1	0.504
		10	0.056	3.6	0.421
Improving dia.(100mm)	Bored	15	0.110	1.3	1.35
		10	0.150	1.2	1.13
	Driven	15	0.07	0.8	0.527
		10	0.058	1	0.0399
Improving dia.(32 mm)	Bored	15	0.0195	0.4	0.146
		10	0.0114	0.4	0.085

From previous tests results clear observation on the route of compaction effecting on pile load capacity we also connect it with the distance from pile face to the end of improvement area and related to pile model diameter as following :

1. (43mm) distance presents ($D_{IMPROVEMENT}/D_{PILE}$=3.3).

2. (73mm) presents ($D_{IMPROVEMENT}/D_{PILE}$=5.6).

3. (100mm) presents ($D_{IMPROVEMENT}/D_{PILE}$=7.6)

4. (32mm) presents ($D_{IMPROVEMENT}/D_{PILE}$=2.4) (executed only in driven case).

For the previous test results ,for the both cases (bored-driven) its seem to be that the best choice and economic for improving surround soil is at)($D_{IMPROVEMENT}/D_{PILE}$=3.3) pile face at this ratio the pile load capacity increased as following :

1. For pile length (15cm) driven case pile capacity will increase by213%
2. For pile length (10cm) driven case pile capacity will increase by135%
3. For pile length (15cm) bored case pile capacity will increase by1865%
4. For pile length (10cm) bored case pile capacity will increase by1321%

The increments in pile load capacity beyond a ($D_{IMPROVEMENT}/D_{PILE}$=3.3).Will be insignificant compare to the first increment occur. The settlement also increases due to improvement of surround soil. The table (4-2) shows that there is increment in settlement for all cases of improvement almost less than (10%) of pile model diameter except in ($D_{IMPROVEMENT}/D_{PILE}$=5.6) driven case for embedded length (10cm). This increment can be explained as the weight of compacted soil increases ,the loose soil under compacted soil block will compress , this will lead for moving of all block and during the test loading of the pile model .

4.5 comparison of experimental results with Hansen equation for computing pile load capacity.

The tendency in estimate of tip bearing capacity of piles include neglect the diameter or width of the pile, hence the equation will be:

$Q_b = A_b(\sigma_{vb}.N_q)$……….(2-7)

So, the total capacity of pile Q_T will be:

$Q_T = \int_0^l \rho.K_s.\sigma_v.\tan\delta\,.dz + A_b(\sigma_{vb}.N_q)$ ……………(2-8)

The N_q factor used in this equation is belong to ***Meyerhof*** equation which it the same used by Hansen . One of the early sets of bearing-capacity equations was proposed by ***Terzaghi (1943***) as shown in Table (4-3). These equations are similar to Eq**.** ***Terzaghi*** used shape factors noted when the limitations of the equation were discussed. ***Terzaghi's*** equations were produced from a slightly modified bearing-capacity theory development.

Table(4-3). Bearing capacity equations factors for many author after(bowles,1996)

Bearing-capacity equations by the several authors indicated

Terzaghi (1943). See Table 4-2 for typical values and for $K_{p\gamma}$ values.

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5\gamma B N_\gamma s_\gamma \qquad N_q = \frac{a^2}{a\cos^2(45+\phi/2)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

For:	strip	round	square
s_c =	1.0	1.3	1.3
s_γ =	1.0	0.6	0.8

Meyerhof (1963).* See Table 4-3 for shape, depth, and inclination factors.

Vertical load: $q_{ult} = cN_c s_c d_c + \bar{q}N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma$

Inclined load: $q_{ult} = cN_c d_c i_c + \bar{q}N_q d_q i_q + 0.5\gamma B' N_\gamma d_\gamma i_\gamma$

$$N_q = e^{\pi\tan\phi}\tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

Hansen (1970).* See Table 4-5 for shape, depth, and other factors.

General:† $q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q}N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$

when $\phi = 0$

use $q_{ult} = 5.14 s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 1.5(N_q - 1)\tan\phi$

Vesić (1973, 1975).* See Table 4-5 for shape, depth, and other factors.
Use Hansen's equations above.

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 2(N_q + 1)\tan\phi$

*These methods require a trial process to obtain design base dimensions since width *B* and length *L* are needed to compute shape, depth, and influence factors.(***Bowels,1996***)

Depending on data obtained from experimental work Hansen equation was applied to get the bearing capacity under tip. There is important factor must take into consideration is the method of installation on surrounding soil hence pile load capacity **.** ***Horn (1966)*** presented test results of sand prior and after pile driving, significant densification of the sand was noticed for distance as large as eight diameters away from the center of the pile.

Kishida (1967) proposed a simple method of estimating the effects of driving pile in loose sand in vicinity of the tip; it was assumed as in Figure (4-27) that the diameter of the compacted zone around a pile is 7 times the diameter and angle of internal friction changes linearly with distance from the original value of $\phi 1$ at a radius (r = 3.5D) to a maximum value of $\phi 2$ at the pile tip. The relationship between $\phi 2$ and $\phi 1$ is taken to be as:-

$\phi 2 = (\phi 1+40)/2$ …. (2.7)

(ϕ_1 and ϕ_2) = angle of internal friction before and after driving process.

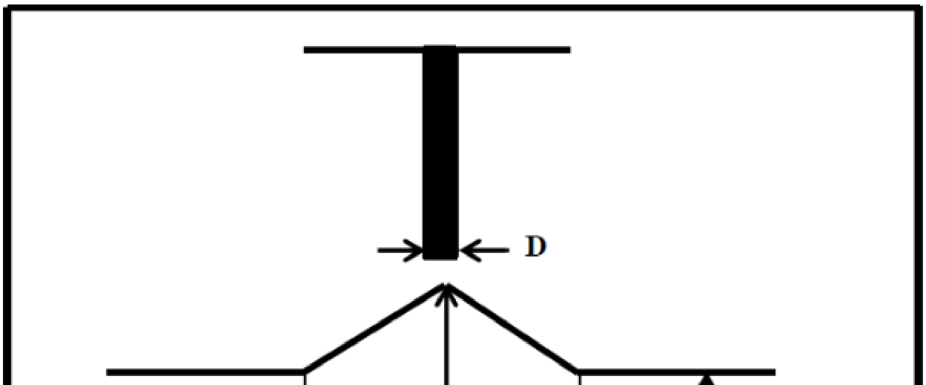


Figure (4-27) Effect of pile driving on angle of internal friction after (ALI ,2012)

By applying the data obtained from experimental work into the (2-8) equation and with respect to affect of installation method on soil surrounding pile the following results were obtained :

4.5.1without improvement case:

In this case (2-8) equation are applied in both cases driven and bored with respect to method of installation and its affect on surrounding soil. The method of installation may densify or loosen the surrounding soil in case of driven pile as previously mentioned. The surrounding soil will be densified while in bored case the surround soil will be loosen.This affect appear clearly by increasing or decreasing the value of (Ø) and this value will be applied into the (2-8) equation as will be shown next section

4.5.1.1driven pile :

the following data will be use as following :

1-Ø=29° for loose medium sand this value was effected by driving process and this value will be for tip load calculations :

$$\emptyset new = \frac{\emptyset + 40}{2} \ldots\ldots\ldots$$ (***Berazantzev,1961)***

For skin friction calculations :

$\emptyset new = 3/4\ \emptyset + 10$

The new values of Ø will be (34.5°) for tip load calculations and (31.75°) for skin friction calculations

2- unit weight =14.20 kN /m^3

3-area of pile model tip =1.33 $*10^{-4}$ m^2

4- k_0 = 1-sin 31.75 =0.473

5-N_q=31.285 according to Meyerhof equation for N_q

6- $\sigma v = 2.13$ kN /m^2 for length 15 cm, $\sigma = 1.42$ kN /m^2 for length 10 cm

After application these data in equation(2-8) we get the following results

1- for length 15cm

q_{ult}=0.0126 kN using equation and q_{ult} =0.015 kN by experimental work

2- for length 10cm

qult =0.00869 kN by equat$ion and qult=0.0095 kN experimental work

4.5.1.2bored case:

the following data will be used

1-$\emptyset = 29°$ For loose medium sand this value effected by boring process this value will be for tip load calculations and skin friction calculations

$\emptyset_{new}$ =$\emptyset - 3$…………….*(**berazantzev**,**1961**)*

The new value will be used in the equation for calculations pile load capacity is(29°)

2- unit weight =14.20 kN/m3

3-area of pile model tip =1.33 $*10^{-4}$ m^2

4- k_0 = 1-sin 26=0.561

5-N_q=11.8 according to Meyerhof equation for N_q

6- $\sigma v = 2.13$ kn/m^2 for length 15 cm, $\sigma = 1.42$ Kn/m^2 for length 10 cm

After application these data in equation(2-8) we get the following results

1- for length 15cm

q_{ult}=0.0069 Kn using equation and q_{ult} =0.0058Kn by experimental work

2- for length 10cm

qult =0.00387 kN by equation and by experimental work qult=0.0038 kN

4.5.2 improvement case:

For driven case the equation adopted to calculate the pile load capacity is not compatible to this case because of collapsing of compacting soil block surrounding pile due to pile driving process. For bored case behavior of one block for both pile model and surrounding ,idea was adopted for this case ,adoption of this idea belong to the loose soil around improved area and the effect of compaction on improved soil that's make it behave as one block the other reason to adopt this idea is the cohesion between pile model and improved soil surrounding it ,this will make it behave as one block because of the high value of cohesion . This idea mean the tip area will be equal to summation of model tip and surround soil.The compaction operation will affect soil under tip and make it denser which make the tip load higher than the friction between the around soil and improved soil block in the tight range around pile model. In the wider range the of improving soil around pile. tip resistance will increase because of increasing the area subjected to tip resistance . At this stage the difference between the pile load capacity obtained by equation and ,pile load capacity obtain from load settlement curves that plotted corresponding to load settlement tests performed was noticed . The equation will give capacity approaching the experimental tests. The pile model and compacted soil which surrounding it will not give values of pile capacity as equation as one block failure behavior like in tight range of improving surrounding soil case. When the diameter of improving soil surround pile increase the difference in pile capacity value with experimental increase also. Suggestion of using angle of internal friction (∅) effected by compaction and casing the compacted soil was adopted .The soil around soil compacted block treated as effect by casing and it value of (Ø) considered to be decreased . The soil under tip considered to be effected by compaction and it value will increase .The following results show the effect of compaction on behavior of surrounding soil

4.5.2.1 replace and compact soil (43 diameter) around the pile.

For case of pile will slid out from improved block data used for this case is as following:

1-Ø=59° for compacted soil

2-unit weight for compacted soil=18.5kN/m^3

3-value of (C)=92.9kPa

4-Nq =31.285 for Ø=34.5° under tip effected by compaction

5-$\sigma v = 1.85$ Kn/m^2 for length 10 cm

σv=2.775 Kn/m^2 for length 15 cm

6- area of pile tip =1.33*10^{-4}

7-k_0=0.142 for the angle of internal friction (59°)

If we applied this data into (2-8) with respect to assumption that the pile will slip out the soil compacted block the equation will be as shown:

$=(2.775*31.285)*1.33*10^{-4}+(0.15/2*18.5*0.142*\tan 59+92*0.58)*0.15*0.013\pi$

=0.344 kN

And

q_{ult}=0.228 kN for 10 cm length

for pile model with length embedded with 15 cm

while in experimental work the results was (0.114) kN that's lead us to refuse the explanation of failure the pile by sliding out from soil block. Which make another explanation adopted this depends on explain the failure occur as one block for both the pile model and compacted soil. The data will be used for this case as following:

1-Ø=34.5 for soil under tip which it effected by compaction ,Ø=26 for soil surrounding the soil surrounding compacted soil which is effected by casing which make the angle of internal friction lessen .

2-unit weight for compacted soil=14.20kN/m^3

3-value of (C)=0Kpa

4-Nq =31.285 for Ø=34.5° under tip effected by compaction

5-$\sigma v = 2.13$Kn/m^2 for length of 15 cm

σv=1.42Kn/m^2 for length 10 cm

6- area of pile tip =$1.452*10^{-3}$ m^2

7-k_0=0.561 for the angle of internal friction (26°) surround soil block.

If we applied tis data into (2-8) with respect to assumption of behavior as one block equation will be as shown:

1- 15 cm length :

$=(2.13*31.285)*1.452*10^{-3}+(0.15/2*0.561*14.2*\tan(59)*(0.15*0.043*\pi$

=0.116 kN

While in experimental work the estimated pile load capacity was (0.114 kN)

2-10 cm length :

$=(1.42*31.285)*1.452*10^{-3}+(0.10/2*0.561*14.2*\tan(59)*(0.1*0.043*\pi$

=0.073 kN

While in experimental work curves the calculated pile load capacity was (0.075 kN)

<u>4.5.2.2 replace and compact soil (73 diameter) around the pile.</u>

behavior of one block was adopted also in this case data will be used was as following :

1-Ø=34.5 for soil under tip which it effected by compaction ,Ø=26 for soil surrounding the soil surrounding compacted soil which is effected by casing which make the angle of internal friction lessen .

2-unit weight for compacted soil=14.20 Kg/m^3

3-value of (C)=0Kpa

4-Nq =31.285 for Ø=34.5° under tip effected by compaction

5-$\sigma v = 2.13$Kn/m^2 for length of 15 cm

σv=1.42Kn/m^2 for length 10 cm

6- area of pile tip =4.185*10^{-3} m^2

7-k_0=0.561 for the angle of internal friction (26°) surround soil block.

If we applied tis data into (2-8) with respect to assumption of behavior as one block equation will be as shown:

1- 15 cm length :

=(2.13*31.285)*4.185*10^{-3}+(0.15/2*0.561*14.2*tan(59)*(0.15*0.073*π

=0.313 kN

While in experimental work the calculated pile load capacity was (0.144 kN)

2-10 cm length :

=(1.42*31.285)*4.185*10^{-3}+(0.10/2*0.561*14.2*tan(59)*(0.1*0.073*π

=0.201 kN

While in experimental work the calculated pile load capacity was

(0. 103 kN)

4.5.2.3 replace and compact soil (100 diameter) around the pile.

behavior of one block was adopted also in this case data will be used was as following :

1-Ø=34.5 for soil under tip which it effected by compaction ,Ø=26 for soil surrounding the soil surrounding compacted soil which is effected by casing which make the angle of internal friction lessen .

2-unit weight for compacted soil=14.20 kN/m^3

3-value of (C)=0kPa

4-Nq =31.285 for Ø=34.5° under tip effected by compaction

5-$\sigma v = 2.13$Kn/m^2 for length of 15 cm

σv=1.42Kn/m^2 for length 10 cm

6- area of pile tip =4.185*10^{-3} m^2

7-k_0=0.561 for the angle of internal friction (26°) surround soil block.

If These data applied into (2-8) with respect to assumption of behavior as one block equation will be as shown:

1- 15 cm length :

=(2.13*31.285)*7.853*10^{-3}+(0.15/2*0.561*14.2*tan(59)*(0.15*0.1* π

=0.569 kN

While in experimental work the calculated pile load capacity was (0.150 kN)

2-10 cm length :

=(1.42*31.285)*7.853*10^{-3}+(0.10/2*0.561*14.2*tan(59)*(0.1*0.1*π)

=0.369 kN

While in experimental work the calculated pile load capacity was (0. 110 kN)

From previous results , there is a difference between the equations results and experimental tests , especially when the tip area of block increase.as results tip resistance increase until be larger than the skin friction of pile model with the improved soil block. As a logical consequence the failure expected to be between the failure as one block and slid out of pile model failure. The following table shows the details of distribution of pile load capacity terms :

Table (4-4) show the details of pile capacity terms for (15 cm)

Case for 15cm length	Pile load capacity by experimental (kN)	Pile load capacity theoretical (kN)	Skin friction of pile model with block(kN)	Skin friction of block with surround soil (kN)	Tip load for whole block (kN)
Improvement $D_{IMPROVEMENT}/D_{PILE}=3.3$	0.114	0.116	0.328	0.0201	0.0958
Improvement $D_{IMPROVEMENT}/D_{PILE}=5.6$	0.144	0.313	0.328	0.0342	0.278
Improvement $D_{IMPROVEMENT}/D_{PILE}=7.6$	0.150	0.569	0.328	0.0468	0.522

Table (4-5) show the details of pile capacity terms for (10 cm)

Case for 10cm length	Pile load capacity by experimental (kN)	Pile load capacity theoretical (kN)	Skin friction of pile model with block(kN)	Skin friction of block with surround soil(kN)	Tip load for whole block (kN)
Improvement $D_{IMPROVEMENT}/D_{PILE}=3.3$	0.075	0.079	0.218	0.00895	0.07005
Improvement $D_{IMPROVEMENT}/D_{PILE}=5.6$	0.103	0.201	0.218	0.0152	0.186
Improvement $D_{IMPROVEMENT}/D_{PILE}=7.6$	0.110	0.369	0.218	0.0208	0.348

From previous results there is different between the equations results and experimental work observed. Because the effect of improvement will decrease with increase of diameter of improvement around pile model this not appear in equation. On the contrary, in case of equation the pile capacity increases with increasing diameter. Specially increasing the tip resistance which as shown previously. Tip resistance formed most part of pile capacity. In case of behavior of one block. There is other observation that's the zone under tip is not compacted at same degree of compaction not like the zone under compacted area surrounding pile which it effected clearly by compaction and that's explain why the toe of pile model extract from soil compacted block specially in large diameter of improvement . The most effected range of improvement inserted by limited of (43 mm) Improvement $D_{IMPROVEMENT}/D_{PILE}$=3.3 for instance:

For the case of improvement (73 mm) $D_{IMPROVEMENT}/D_{PILE}$=5.6 the skin friction of whole compacted soil block with length (15 cm) has been equal (0.0342 kN) if it is added to the tip load resistance for case of (43 mm) $D_{IMPROVEMENT}/D_{PILE}$=3.3 the result would be (0.139 kN) and this value approaching to the results obtained from load-settlement test which is equal(0.144 kN). In the case of improvement (100 mm) if the same previous procedure used the results would be (0.142 kN) which is approaching from the result obtained from load settlement test (0.150 kN). This notice guides for observation that is the effect of improvement and behavior of one block as results of this improvement is inserted by tight limits surround pile. From the previous results we notice that the best choice for improvement is with limit to(43 mm) $D_{IMPROVEMENT}/D_{PILE}$=3.3 surrounding pile because any increment beyond this limit will not effect on pile tip load resistance with wide range .the increment in improvement diameter around pile will cause increment in skin friction of compacted soil block with surrounding soil with slight increment in tip load resistance.

CHAPTER ONE

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CHAPTER TWO

REVIEW of LITRETURE

CHAPTER THREE

EXPERIMENTAL WORK

CHAPTER FOUR

PRESENTATION AND DISCUSSION OF TEST RESULTS

CHAPTER FIVE

CONCLUSIONS & RECOMMENDATIONS

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CHAPTER FIVE

Conclusions and recommendations

5.1 Conclusions

1. Compaction will affect the soil parameters and will lead to increase the apparent cohesion and soil interlocking which results an increasing soil strength.
2. Compaction effect on soil surrounding the pile and soil under tip. The case installation of replacing soil surround pile will reduce the angle of internal friction ,while it increase under tip because of compacting blows.
3. The method of installation of pile has important rule in keep the consistency of soil block, which have most important effect on behavior of pile and soil surrounding it .
4. The effect of compaction appears obviously when compacted soil block surrounded pile still as one structure as it observed in bored case (in this study) .
5. The collapsing of soil compacting block as in case of driven pile will lead to increase the pile load capacity due to the friction generated between the pile skin and parts of collapsing block.
6. The increment in soil cohesion and interlocking results a high adhesion between pile and surrounding soil and that lead to behave as one block .
7. The increase in diameter of improvement results increase in pile load capacity due to increase in tip load resistance as result of behave as one block for both (pile and compacted soil block)
8. The effective range of improvement inserted in the close limits around pile model in this study (**$D_{IMPROVEMENT}/D_{PILE}=3.3$**) which is cause increment

in tip load resistance for soil block (model –compacted soil) any increase beyond this limit will cause slight increment in pile load capacity due to increment in skin friction of soil block with soil contact it . In this case the pile capacity will increase in as following :

For pile length (15cm) driven case pile capacity will increase by213%

For pile length (10cm) driven case pile capacity will increase by135%

For pile length (15cm) bored case pile capacity will increase by1865%

For pile length (10cm) bored case pile capacity will increase by1321%

9. The soil under pile model tip will not compacted at same degree of soil under replacing soil surrounding pile. This create a weak zone under pile model tip and that's explain the decreasing of pile load in the pile load – settlement curves after approaching the maximum load point in case of improvement of bored pile .
10. According to this study ,the operation of improving pile surrounding soil leads to high increment in pile load capacity ,and this also leads to reducing the number of piles or pile length ,that required to specific load capacity, this lead to economic benefits

6.2 Recommendations

1. Studying the effect of compaction on pile tip load capacity purely.
2. Using different shapes as models to studying the effect of shape of pile model .
3. Studying reliability of this study on prototype .
4. Studying the effect of compaction surround soil of pile in pile group.

5. Studying the effect of compaction of surround soil of pile on pull load capacity.
6. Studying the chemical material content on compaction results .
7. Studying the effect of soil improvement on settlement with respect to improvement in pile load capacity .
8. Studying the effect of compaction on friction capacity and behavior of soil block in case of dense sand or clay around the block.
9. Studying the effect of improvement surrounding soil on pile capacity with existing of water.

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الخلاصة

يعتبر موضوع تحسين الترب المحيطة ودراسة سلوك الركائز بعد التحسين موضوع مثير للأهتمام الباحثيـــن حيث تناول الباحثيـــن موضوع تحسيـن الترب المحيطة بالركائز لغرض تحسين قابلية تحمل الركائـــز للأحمال الجانبيـــة ومواجهة الزلازل وما ينتج عنها من أحمال موجية تبنى الباحثين عدة طرق لتحسيـــن سلوك التربـــة وقابلية تحمل الركائز اما في دراستنا هذه فقد تم اعتماد اسلوب استبدال التربة المحيطيـة ورصها كطريقة لتحسين التربة المحيطية .تم اجراء الفحوصات المختبرية على نموذج ركيزة مصغر يربط الى جهاز ضغط لدراسة سلوك الركيزة قبل وبعد التحسين . تم اجراء الفحوصات المختبرية الخاصـــة بالتربـــة لمعرفة خصائص التربة موضوعة البحث وتحديد ميزاتها .تم تنفيذ طريقة تنصيب الركائز بكلا الطريقتين (الغرز – الحفر) ومماثلتها بطريقتين مختلفتين.

تم استخدام نموذج الركيـزة المكون من قضيب حديد وبطولي دفن مختلفين هذا النموذج فحص بالمرحلة الاولى بدون تحسيـــن لدراسة السلوك في حالة التربة الطبيعية بدون تحسين ولطولين مختلفين لكل حالـة نصب ركيـــزة (غرز –حفر).المرحلة التاليـــة تضمنت فحص النموذج بعد تحسين التربة المحيطة بـه بالاستبدال والرص وتم تغييـــر محيط التربـــة بالنموذج بأقطـــار مختلفة وبطول يساوي لطول النموذج المغروز بالتربـــة لضمـــان عدم امتداد عملية استبدال التربة لما تحت قدم الركيزة .لكل قطر من اقطار التحسين حول النموذج كان النموذج يفحص لكلا طريقتي النصب (الحفر –الغرز) ولطولين مختلفين لكل طريقة .تم استخراج قيم (الحمل-الهطول) لكل تجربـــة واستخدام هذه القيـــم برسم العلاقة بينهما وبالتالي استخراج قابلية تحمل التربة لكل حالة. حيث كانت هذه القيم وتغيراتها تعطي مؤشرا عن مدى تأثر سلوك الركيزة بعمليـــة التحسيـن .

تم اعتماد معادلة (هانسن) لغرض تخمين قابلية تحمل النموذج ومقارنة هذه النتائج مع النتائج المستحصلة من الفحوصات المختبريـــة مع الاخذ بنظر الاعتبار السلوك المتوقع لكل من نموذج الركيزة وكتلة التربة

المحسنة .وبعد الدراسة والمقارنات بين النتائج تم التوصل لفكرة ان افضل خيار للتحسين ينحصـــر بمدى ضيق حول الركيزة وذلك لان التماسك العالي سيؤدي الى تصرف التربة المحسنة والركيزة ككتلة واحدة مما يؤدي الى رفع قيمة مقاومة قدم الكتلة المحسنة ككل .في هذه الدراسة فان افضل خيار لتحسين التربة تمثل في تحسين التربة بمحيط (٤٣ ملم) حول الركيزة بما يماثل مسافة تساوي قطر نموذج الركيـــزة من وجه الركيزة وبما يساوي نسبة (قطر المنطقة المحسنة / قطر نموذج الركيزة =٣،٣) .حيث سينتج عنها زيادة في احتكاك التربة المحيطية مع كتلة التربة المحسنة وزيادة بقابلية تحمل قدم التربة المحسنة ايضا. حيث ستؤدي الى في حالة الركائز المغروزة:

١- رفع قيمة تحمل الركيزة من (٠،٠١٥) في حالة التربة الطبيعية بدون تحسين الى (٠،٠٤٧) لطـــــول (١٥ سم) وبنسبة زيادة تقدر ب ٢١٣%.

٢- رفع قيمة تحمل الركيزة من (٠،٠٠٩٥) في حالة التربة الطبيعية بدون تحسين الى (٠،٠٣٣) لطول (١٠سم) مدفون في التربة وبنسبة زيادة تقدر ١٣٥%.

اما في حالة الركائز المحفورة:

١- (٠،٠٠٥٨) حالة التربة الطبيعية بدون تحسين الى (٠،١١٤) لطول (١٥ سم) مدفون بالتربة وبنسبة زيادة تقدر ١٨٦٥%

٢- رفع قيمة تحمل الركيزة من (٠،٠٠٣٨) الى (٠،٠٧٥) لطول (١٠ سم) في حالة الركائز المحفورة وبنسبة زيادة تقدر ١٣٢١%. أي زيادة بعد حدود التحسين هذه ستعطي زيادة في قابلية تحمل الكتلة المحسنة للاحتكاك وليس في قابلية تحمل القدم للأحمال خصوصا وان الاخير يعتبر العامل الاكثر تأثيرا في زيادة قابلية تحمل للكتلة المحسنة. وفي نهاية الرسالة تم تلخيص هذا التأثير واظهار هذا التأثير على قابلية تحمل التربة الركيزة للأحمال.

<u>Appendix</u>

Figure (A.1) Istrument Used for Testing Material

Appendix

Figure (A.2) Direct Shear Test

Appendix

Figure (A.3) The Soil Improvement Block