SUDAN UNIVERSITY OF SCIENCE & TECHNOLOGY

College of Graduate studies

Numerical Analysis and Design of Bored Concrete Pile Foundations (case study: Soba Bridge Piles)

التحليل العددي والتصميم للأساسات الخازوقية الخرسانية المحفورة

(دراسه حالة: خوازيق كبري سوبا)

A Thesis Submitted to the Department of Structural Engineering in the Partial Fulfilment of the Requirements for the Degree of Master of Sciences of in Civil Engineering (Structures)

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Dedication

To the soul of my father

My lovely mother

My sisters, brothers and friends

Acknowledgments

I would like to express my special thanks to my dear supervisor,

Dr. Allia Mohammed Osman, for her brilliant ideas,

endless support and guidance throughout this study.

I am grateful for her patience and continuous encouragements and generous supply of references and leading supervision

I would like to express my gratitude to my family.

Sincere thanks to my friends for their precious friendship and continuous

support. Especially, the friends who look after me in bad times.

Abstract

Bored concrete piles are mostly used in Sudan Khartoum area for tall building, bridges and other heavily structures where the soil under which is very weak.

The purpose of this study is to analyze and design bored piles under general conditions of load in Khartoum area, particularly at Soba bridge site, to study how pile transform these loads to the ground. how these loads are distributed in each pile within the group piles and design it.

Using manual empirical formulas and computer software Allpile 6.5 to analyze different pile diameters (1.2,1.4,1.6and 1.8m), different pile lengths (25.9,32.9, and 35.9) and different numbers of pile groups (1x4,2x4 and3x4) subjected to vary load combinations to study the effect of change in diameter, length and number of piles in groups on pile behavior.

The results of settlement, deflection, axial, lateral load and moment on each pile in group were obtained to compare the results from manual empirical formulas with numerical software Allpile 6.5 and with the actual results which the bridge was designed.

The study has shown that the results of numerical software Allpile 6.5 analysis, manual empirical formulas calculations analysis and actual results are almost the same for the vertical load results. However, about the results of moment and lateral load the actual results which show large different compared to the software Allpile 6.5 and manual empirical formulas calculations results which they are often close.

The results indicate that the decrease in length effects on increase the settlement about 14.14% and the increase of diameter effect on the decrease the settlement by 22.59%. but the change in length does not influences on deflection. Also Increase the number of piles in the group decrease the loads applied on each pile.

finally, the pile has been designed calculating the reinforcement for safety and durability. Comparison of this with the actual design shows that more pile length is required because the actual length is 25.9m for 1X4 group, the satisfied length for the design requirement in this research is 35.9m for 1X4 group.

التجريد

الخوازيق الخرسانيه المحفورة (bored concrete piles) اكثر استخداما في السودان ,منطقة الخرطوم, للمباني العاليه والجسور والمنشآت الثقيله التي تقام على تربه ضعيفه .

الغرض من هذه الدراسه هو تحليل وتصميم الخوازيق الخرسانية المحفورة المطبق عليها القوي المختلفة في منطقة الخرطوم بالتحديد عند موقع كبري سوبا ودراسة كيفية نقل الخازوق للاحمال الي التربه وكيفيه توزيع هذه الاحمال على كل خازوق داخل مجموعه الخوازيق وتصميمه .

بإستخدام المعادلات التجريبه اليدويه و برنامج حاسوبيAllpile6.5 لتحليل اقطار المختلفه للخازوق (1.6,1.4,1.2و1.8 متر) و الاطوال المختلفه للخازوق (32.9,25.9و 35.9 متر) و ايضا وعدد مختلف من مجموعات الخوازيق (4x3,4x1 و4x2) معرضة لمجموعة من الأحمال المتغيرة لدراسة تأثير التغير في القطر والطول وعدد الخوازيق في المجموعه على سلوك الخازوق.

تم الحصول على نتائج الهبوط والإنحراف والقوي المحورية والقوي العرضية لكل خازوق في المجموعة و ذلك لمقارنة نتائج المعادلات التجريبيه اليدويه بنتائج البرنامج الحاسوبي Allpile6.5 و ايضا بنتائج الفعلية للتصميم.

هذه الدراسة أظهرت أن النتائج من التحليل العددي بإستخدام برنامج حاسوبيAllpile6.5 و النتائج بإستخدام المعادلات التجريبة اليدوية والنتائج الفعلية للتصميم تقريباً متساويه بالنسبة للقوة الراسية . لكن بالنسبه لنتائج العزوم و القوي الافقية أظهرت النتائج الفعلية للتصميم فرق كبير عن نتائج التحليل بإستخدام برنامج حاسوبيAllpile6.5 و المعادلات التجريبة اليدوية التي تكون في الغالب متقاربة.

أشارت النتائج الي أن تقليل الطول يؤثر في زياده الهبوط حوالي 14.14% وأيضا زيادة القطر تؤثر في الهبوط 22.59%, ولكن وجد ان تغيير الطول لا يؤثر في الانحراف . وايضا زيادة عدد الخوازيق في المجموعة يقلل من القوى المطبقة على الخازوق .

وأخيراً تم تصميم الخازوق تحت الاحمال وحساب التسليح المناسب للامان والديمومة. أظهرت المقارنه مع التصميم الفعلي توجد حوجه الي زياده طول الخازوق لأن الطول الفعلي هو 25.9 متر لمجموعة 1X4 , لكن طول الكافي لمتطلبات التصميم في هذا البحث هو 35.9متر لمجموعة1X4 .

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List of Symbols:

$(A_s)_i$ surface area of the pile in the layer.

A'_s area of shaft that is effective in developing skin friction.

A_p area of the pile tip.

A_p area cross section of pile.

A_s effective surface area of the pile in contact with the soil.

B width of the pile.

c cohesion of the soil at depth z.

d center to center pile spacing.

D depth of the piles below ground level.

D_s diameter of bored pile.

E modulus of elasticity in the pile material.

E_p modulus of elasticity of bored pile.

E_p modulus of elasticity of the pile material.

E_s modulus of elasticity of soil at or below the pile point.

E_u deformation modulus for the undrained loading conditions.

$f_{(z)}$ variation of with depth.

f_s average unit skin friction between the sand and the pile surface.

f_s unit friction resistance at any depth.

Gs shear modulus of soil.

H thickness of the soil layer.

H_i thickness of layer i

I moment of inertia of the pile.

I_p influence factor.

k coefficient of horizontal subgrade reaction.

K earth pressure coefficient.

K_0 earth pressure coefficient at rest.

K_p Rankine's coefficient of passive pressure.

L length of pile.

n number of layers in which the pile is installed.

N number of piles in group

n_1, n_2 number pile in group in each direction.

N_c bearing capacity factor.

N_c*, N_γ* , N_q* bearing capacity factor .

$N_{\sigma*}$ bearing capacity factor.

P perimeter of the pile section.

p' pressure on soil.

p_{oz} effective overburden pressure at depth z.

p_{oz} the effective overburden pressure at a depth z.

P_z passive resistance at any depth.

Q_{all} allowable load-carrying capacity for each pile.

Q_g ultimate load for group piles.

q_n net foundation pressure.

q_p ultimate bearing capacity of the soil at the pile tip.

Q_p point (or base or tip) resistance of the pile.

Q_s shaft resistance develop by friction (or adhesion) between the soil and

Q_u ultimate load for single pile.

q_{wp} point load per unit area at the pile point.

Q_{wp} load carried at the pile point under working load condition.

Q_{ws} load carried by frictional (skin) resistance under working load

R_1 ratio of moment of inertia of drilled shaft section to moment of inertia.

R Stiffness factor

tan δ' coefficient of friction between sand and the pile material.

T Stiffness factor

y Deflection of the pile.

α' effective vertical pressure at the pile tip.

γ' effective unit weight of sand.

δ' soil – pile friction angle = $0.8\phi'$.

λ friction capacity factor.

μ_s Poisson s ratio of soil.

σ_z average effective vertical stress imposed on the soil layer due to the net foundation pressure q_n at the base of the equivalent raft foundation

σ'_0 average effective overburden pressure.

σ_0 mean effective normal ground stress at the level of pile point.

$\overline{\sigma}_0$ mean effective vertical street for the embedment length.

$\bar{\sigma}_v$ effective vertical pressure at that depth thus unit skin friction.

$\sigma'_{(0)i}$ effective normal stress in the layer.

υ shear stress.

υ_c ultimate shear stress in concrete.

ϕ' effective soil friction angle.

ϕ' effective friction angle of the bearing stratum.

Δ average Volumatic strain in the plastic zone below the pile point

ΔL increment pile length over which p and f are taken to be constant.

Chapter One

General Introduction

Chapter One

General Introduction

1.1 Introduction:

The use of pile is oldest method of solving the soft soil problem to carry heavy load from tall building, bridges. piles are used to transmit the load to stronger layer, when this layer is in deeper depth.

When the load is too heavy to carry by single pile then the group of pile is necessary to carry the loads. Many researchers study pile and group of pile under loads to determine the capacity of piles to carry loads , most researcher depending their research on the field observations; in-situ full-scale and laboratory model tests were widely conducted on vertically and laterally loaded piles (including piers and drilled shaft etc.) to computing bearing capacity.

Central of Khartoum which the case study area are alluvium deposits, they grade with depth from clay near the ground surface in to silts and sand with gravel down to Nubian formation (Al-amery,2005). These formations are either exposed or covered by the recent quaternary formations. They cover large areas in Northern, Central Western and Eastern Sudan. Several important heavy structures such as bridges across the rivers and high-rise buildings in Khartoum are supported on these formations (Elsharief, 2014).

Shallow soil cannot carry heavy structure loads therefore, pile foundation is used.

1.2 Problem statement:

Analysis pile and pile group under general conditions how the pile carry the general conditions loads and transform it to the ground as single pile? also how the pile behave under these loads inside the group of pile? Most researchers analyze piles and group of piles and verify the results by compression with the experimental observations field. In this study the results of theoretical calculations will be compared with results of software program and actual results. That will help to verify the accuracy results.

1.3 Research objective

The aim of the study is to analyze numerically and manually and design pile for different diameters and lengths

Objectives of the study are:

1. To learn how to analyze and design bored pile manually.
2. To analyze and design bored pile using computer software.
3. To verify the accuracy of results by comparison with actual soba bridge pile foundation analysis and design results.

1.4 Methodology:

- Comprehensive literature review on bored piles analysis and design based on published papers, books…. etc.
- Formulating theoretical framework collecting necessary data studying the manual methods and selecting and studying the selected software package (Allpile6.5).
- Applications of the manual and software methods to analyze the case study problem and obtaining results.
- Analysis discussion and verification of results by comparison with known results.
- Conclusions presenting recommendations and writing up the dissertation.

1.5 Outlines of thesis:

Chapter One Presents the introduction writing the problem statement, the objective, the methodology and outlines of thesis.

Chapter Two Contains the theoretical background and literature review.

Chapter Three Gives details of methods of calculating capacity of vertical single pile in sand or clay soil, methods of analysis of lateral single pile. Methods of analysis group pile by simple equation and software program (All pile 6.5).

Chapter Four Presents the case study and result of analysis by theoretical equations and software. For a single pile calculating bearing capacity, ultimate lateral load for several diameters and lengths. also, analyze pile group under the static vertical load, lateral load, moment for different diameter, length and number of piles in group of pile.

Chapter Five Illustrates graphically the results for analysis and discussion the effect of change pile diameters and lengths on the vertical capacity and ultimate lateral load of the pile. the distribution load of each pile in a pile groups for manual calculations, All pile software, actual results.

Chapter Six Gives summary and conclusions of the current work and the recommendations.

Chapter Two

Historical Background and Literature Review

Chapter Two

Historical Background and Literature Review

2.1 Introduction:

Piles are columnar elements in a foundation which have the function of transferring load from the superstructure through weak compressible strata or through water, onto stiffer or more compact and less compressible soils or onto rock. They may be required to carry uplift loads when used to support tall structures subjected to overturning forces from winds or waves. Piles used in marine structures are subjected to lateral loads from the impact of berthing ships and from waves. Combinations of vertical and horizontal loads are carried where piles are used to support retaining walls, bridge piers and abutments, and machinery foundations." (TOMILSON ,2008).

2.2The following list identifies some of the conditions that require pile foundations (Vesic, 1977)

- When one or more upper soil layers are highly compressible and too weak to support the load transmitted by the superstructure, piles are used to transmit the load to underlying bedrock or a stronger soil layer, as shown in Figure 2.1a.
- When bedrock is not encountered at a reasonable depth below the ground surface, piles are used to transmit the structural load to the soil gradually. The resistance to the applied structural load is derived mainly from the frictional resistance developed at the soil–pile interface. (See Figure 2.1b.)
- When subjected to horizontal forces (see Figure 2.1c), pile foundations resist by bending, while still supporting the vertical load transmitted by the superstructure. This type of situation is generally encountered in the design and construction of earth-retaining structures and foundations of tall structures that are subjected to high wind or to earthquake forces.
- In many cases, expansive and collapsible soils may be present at the site of a proposed structure. These soils may extend to a great depth below the ground surface. Expansive soils swell and shrink as their moisture content increases and decreases, and the pressure of the swelling can be considerable. If shallow foundations are used in such circumstances, the structure may suffer considerable damage. However, pile foundations may be considered as an alternative when piles are

extended beyond the active zone, which is where swelling and shrinking occur. (See Figure 2.1d) Soils such as loess are collapsible in nature. When the moisture content of these soils increases, their structures may break down. A sudden decrease in the void ratio of soil induces large settlements of structures supported by shallow foundations. In such as, pile foundations may be used in which the piles are extended into stable soil layers beyond the zone where moisture will change.

- The foundations of some structures, such as transmission towers, offshore platforms, and basement mats below the water table, are subjected to uplifting forces. Piles are sometimes used for these foundations to resist the uplifting force. (See Figure 2.1e.)
- Bridge abutments and piers are usually constructed over pile foundations to avoid the loss of bearing capacity that a shallow foundation might suffer because of soil erosion at the ground surface. (See Figure 2.1f.)" (DAS,2007).

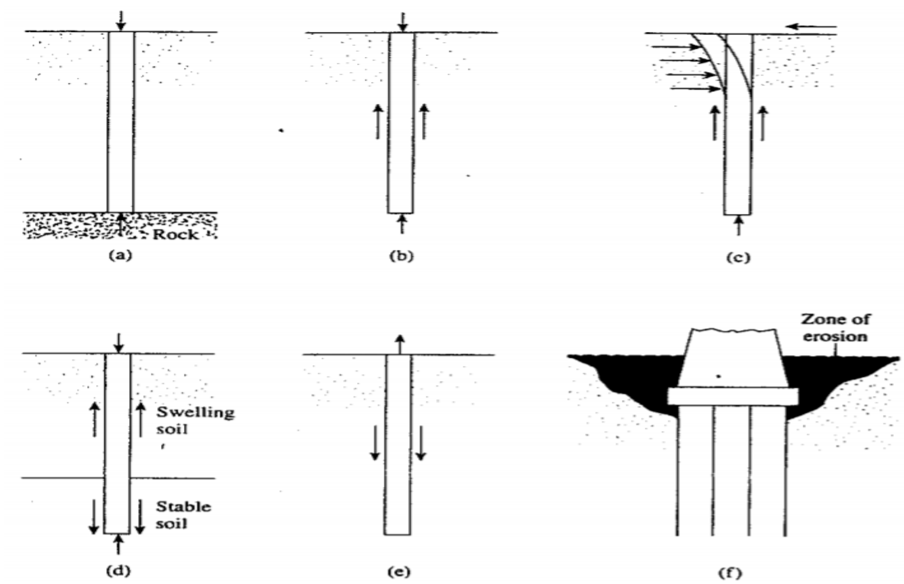


Figure (2.1) Condition that require the use of pile foundations (DAS,2007)

2.3 Type of piles:

Type of pile classified according to several way that will show below:

2.3.1 Classification based on mode of transfer of loads:

2.3.1.1 End bearing piles:

These piles transfer their load on to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile (see figure (2.2). In this case, the ultimate capacity of the piles depends entirely on the load-bearing capacity of the under-lying material; thus, the piles are called point bearing piles.

2.3.1.2 Friction piles:

Carrying capacity is derived mainly from the adhesion or friction of the soil in contact with the shaft of the pile (see figure 2.2). When no layer of rock or rocklike material is present at a reasonable depth at a site, point bearing piles become very long and uneconomical.

These piles are called friction piles, because most of their resistance is derived from skin friction.

2.3.1.3 Combined piles:

These piles transfer loads by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft. the ultimate load carried by the pile is equal to the sum of the load carried by the pile point, and the load carried by the skin friction.

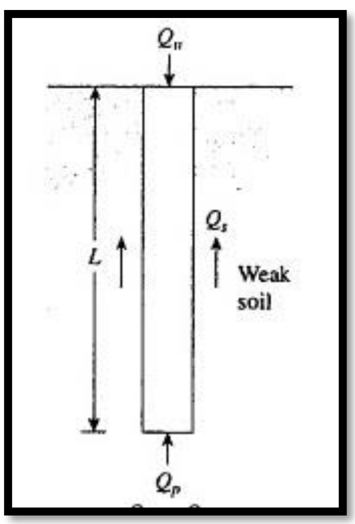


Figure (2.2) transfer of loads in pile

2.3.2 Classification according to material used:

There are four types of pile according to materials used: (Tomlinson ,2008).

2.3.2.1 Timber piles:

Timber piles are made from tree trunk after proper trimming. the timber used should be straight, sound and free defects. Steel shoes are provided to prevent damage at the top pile during driving. The length of pipe sleeve should be at least five times the diameter of the pile.

Timber pile blew the water table have generally long life. However, above the water table these are attacked by insects.

2.3.2.2 Concrete pile:

concrete piles may be divided into two basic categories:

(a) precast piles.

(b) cast-in-situ piles.

(c) Prestressed concrete

(a) Precast piles can be prepared by using ordinary reinforcement, and they can be square or octagonal in cross section. Reinforcement is provided to enable the pile to resist the bending moment developed during pickup and transportation, the vertical load, and the bending moment caused by a lateral load. The piles are cast to desired lengths and cured before being transported to the work sites.

(b) Cast-in-situ, or cast-in-place, piles are built by making a hole in the ground and then filling it with concrete. Various types of cast-in-place concrete piles are currently used in construction, and most of them have been patented by their manufacturers. These piles may be divided into two broad categories:

(a-1) cased.

(b-2) uncased.

Both types may have a pedestal at the bottom. Cased piles are made by driving a steel casing into the ground with the help of a mandrel placed inside the casing. When the pile reaches the proper depth, the mandrel is withdrawn, and the casing is filled with concrete.

Figures 2.3a, 2.3b, 2.3c, and 2.3d show some examples of cased piles without a pedestal. Figure 2.3e shows a cased pile with a pedestal. The pedestal is an expanded concrete bulb that is formed by dropping a hammer on fresh concrete. The uncased piles are made by first driving the casing to the desired depth and then filling it with fresh concrete. The casing is then gradually withdrawn. Figures 2.3f and 2.3g are two types of uncased pile, one with a pedestal and the other without.

(c) Prestressed concrete piles can also be prestressed using high-strength steel pre-stressing cables. The ultimate strength of these cables is about. During casting of the piles, the cables are pre tensioned to about, and concrete is poured around them. After curing, the cables are cut, producing a compressive force on the pile section.

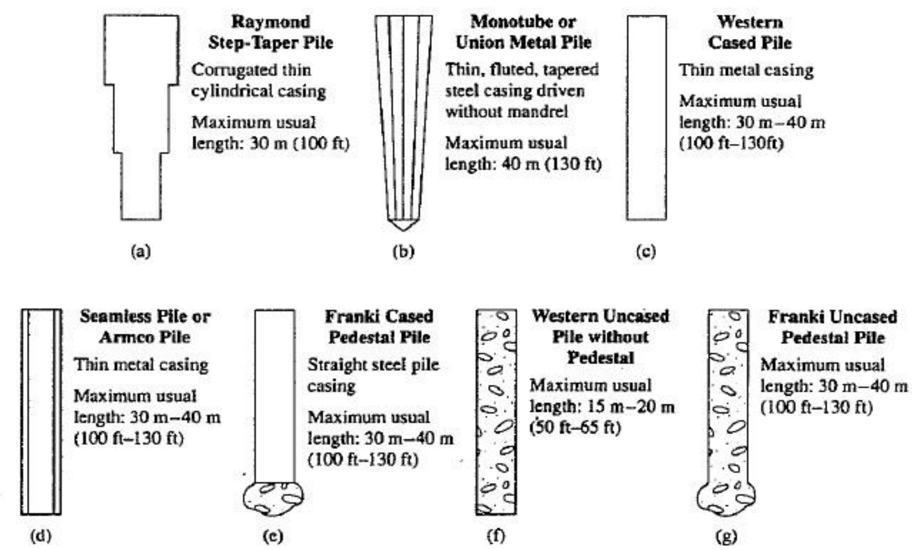


Figure (2.3) Cast in place concrete piles (DAS,2007).

2.3.2.3 Steel piles:

Steel piles have the advantages of being robust, light to handle, capable of carrying high compressive loads when driven on to a hard stratum, and capable of being driven hard to a deep penetration to reach a bearing stratum or to develop a high skin- frictional resistance, although their cost per metre run is high compared with precast concrete piles. They can be designed as small displacement piles, which is advantageous in situations where ground heave and lateral displacement must be avoided. They can be readily cut down and extended where the level of the bearing stratum varies; also, the head of a pile which buckles during driving can be cut down and re-trimmed for further driving. They have a good resilience and high resistance to buckling and bending forces.

Types of steel piles include plain tubes, box-sections, H-sections, and tapered. (Tomlinson ,2008).

2.3.2.4 Composite piles:

Combination of different materials in the same of pile. As indicated earlier, part of a timber pile which is installed above ground water could be vulnerable to insect attack and decay. To avoid this, concrete or steel pile is used above the ground water level, whilst wood pile is installed under the ground water level (see figure 1.7).

2.3.3 Classification based on method of installation:

A simplified division into driven or bored piles is often employed.

2.3.3.1 Driven piles:

Driven piles are displacement piles. In the process of driving the pile into the ground, soil is moved radially as the pile shaft enters the ground.

2.3.3.2 Driven and cast - in- situ piles:

These piles are formed by driving a casing with a closed bottom end into the soil The casing is later filled with concrete. The casing may or may not withdraw. (ARORA,2004)

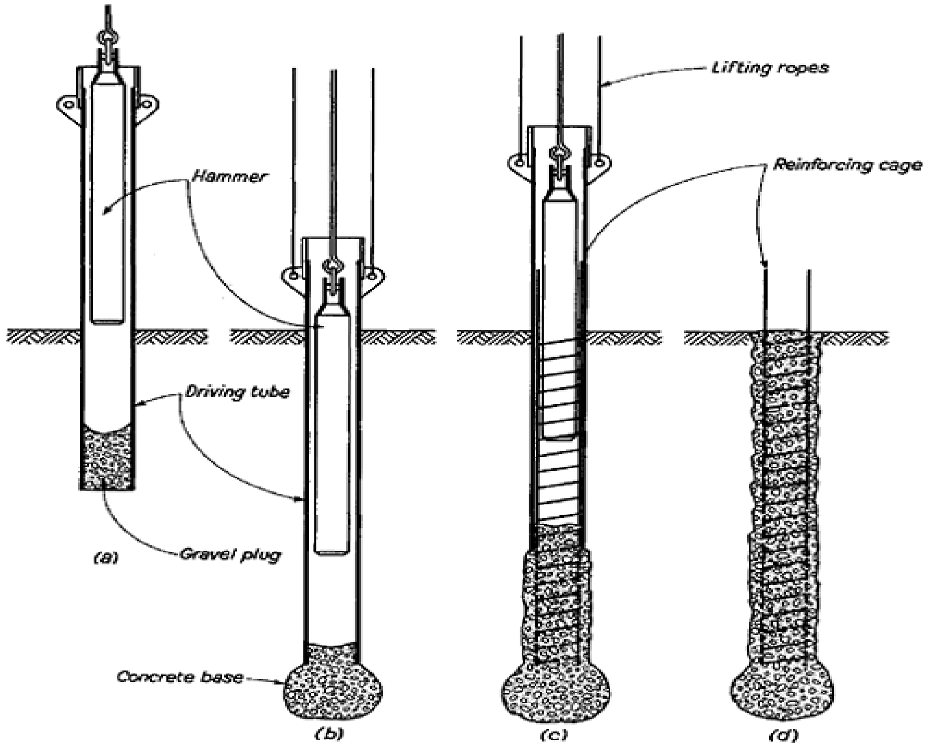


Figure (2.4) Stages in installing pile Driven and cast - in- situ pile (Tomilson ,2008) (a) Driving piling tube. (b) Placing concrete in piling tube. (c) Compacting concrete in shaft. (d) Completed pile (Tomlinson ,2008).

2.3.3.3 Bored piles:

Bored piles a void is formed by boring or excavation before piles is produced. Piles can be produced by casting concrete in the void (figure 2.6). Some soils such as stiff clays are particularly amenable to the formation of piles in this way, since the bore hole walls do not require temporary support except cloth to the ground surface. In unstable ground, such as gravel the ground requires temporary support from casing or bentonite slurry. Alternatively, the casing may be permanent, but driven into a hole which is bored as casing is advanced. A different technique, which is still essentially non-displacement, is to intrude, a grout or a concrete from an auger which is rotated into the granular soil, and hence produced a grouted column of soil. (Tomlinson,2008)

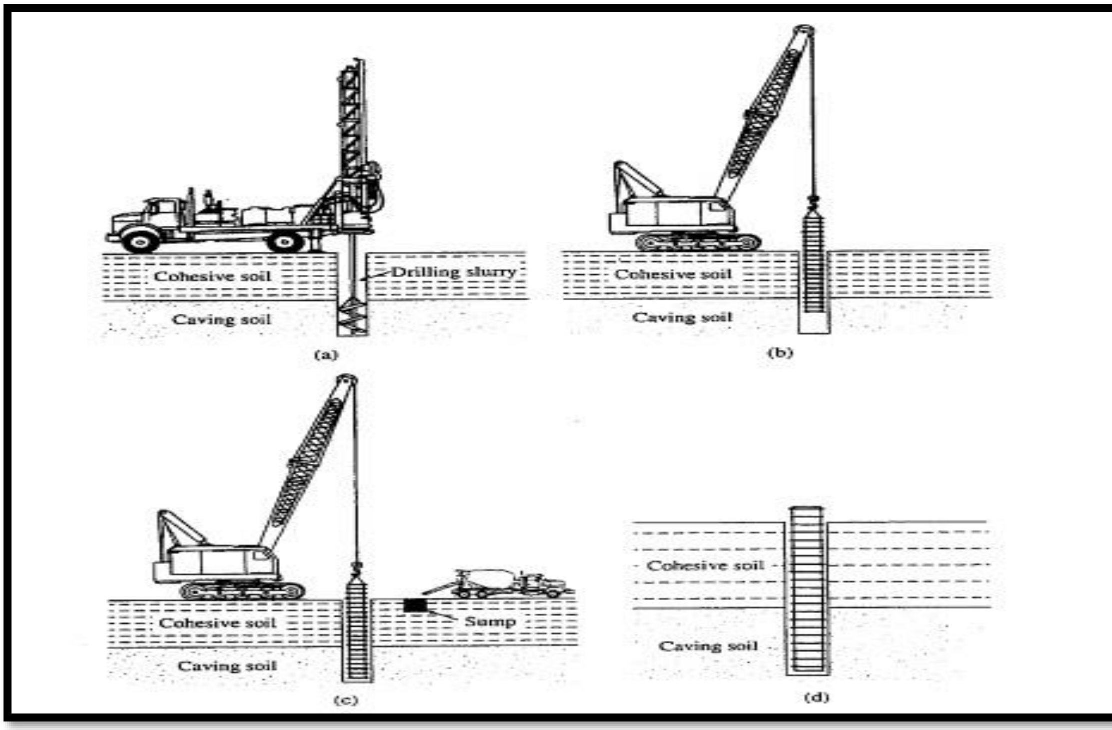


Figure (2.5) method of construction (a) drilling to full depth (b)placing rebar cage (c) placing concrete(d) completed bored.

2.3.4Classification based on displacement of soil:

Based on the volume of the soil displacement during installation, the piles can be classified into two categories: (Tomlinson ,2008).

2.3.4.1Displacement piles:

All driven piles are displacement piles as the soil is displaced laterally when the pile is installed. the installation may cause heaving of the surrounding ground. Precast concrete and closed – end pipe piles are high displacement piles. Steel H- piles are low displacement piles.

2.3.4.2Non- displacement piles:

Bored piles are non- displacement piles. As the soil is removed when the hole is bored, there is no displacement of the soil installation. The installation of these piles causes very little change in the stresses in the surrounding soil. (Tomlinson ,2008).

2.4 Previous studies:

(Basile ,2003)

Reviews of available computer programs for pile group analysis Estimation of geotechnical parameters using some applications in both the linear and nonlinear, attention is focused on correlations between these parameter and commonly available in-situ test data , also compare the application of available numerical methods with practical problem involving real soil .Effect of soil non linearity on pile group response, as measured experimentally and as predicted by numerical analysis . the main advantage of non-liner group analysis system over linear is reduction of loads in large groups.

(Elsharief. 2007)

Summarizes the outcome of a research program carried out in Sudan to provide guidelines for the design of bored concrete piles in expansive soils. Design parameters were developed. The parameters under consideration are the adhesion, bearing capacity and uplift factors. The results of the full-scale tests showed that the compressive axial capacity of piles installed in expansive soil were significantly reduced by wetting. This resulted in recommending higher factor of safety (minimum 4.0), showed that the adhesion factor was 0.45 for moisture content below the plastic limit and linearly increased with moisture above the plastic limit. The end bearing capacity factor was back calculated from instrumented full-scale load tests and a value of 9 was attained. Uplift factor was found to be 0.2 from model pile tests.

(Ivsic,2013)

Analyzes the bearing capacity and settlement of bored piles, as the most frequently used type of piles in local practice. Empirical methods based on geotechnical soil parameters for capacity estimation, introduce some simplifications which lead to neglecting certain elements of a complex pile-soil interaction. On the other hand, the results of pile field testing methods are a direct summary consequence of the overall complex conditions on pile-soil contact, the comparison of empirical procedures and in-situ tests conducted to determine bearing capacity and settlement of bored piles in soft soils. as results obtained by calculation methods are generally much higher that the bearing capacity values obtained by in-situ testing.

(Elsharief. 2014)

Summarizes the geotechnical characteristics of five bridge sites in Khartoum and the approaches used by the designers for estimating the bearing capacity of the piles. The designs were compared with the results from pile load tests carried out in the bridge sites. The analysis has shown that the approaches used for estimating the pile capacities in the NF are very conservative and un-realistic , this evaluation has shown that alternative design approaches or improvement in the used ones are needed for the designers to come out with a technically viable and sound design for piles socketed in the NF. Alternative design approaches or improvements of the currently used designs are needed.

(Poulos,2014)

Presented for the analysis of general three-dimensional pile groups. The features of the pile-group model may include battered piles, different pile sizes, non-uniform pile sections, soil nonlinearity, soil inhomogeneity, and pile-soil-pile interaction. A typical six-pile group is analyzed and compared with results from three other computer programs for pile group analysis that are based on different approaches. This method is then used to analyze field and laboratory tests on groups of battered and vertical piles. The computed solutions are shown to be in good general agreement with the measured data. The present approach is shown to be at least as good as some of the computer programs currently available for pile-group analysis. Full-scale field tests and a lab and the results showed reasonably good agreement with the measured values.

(Zhng ,2015)

Present a simplified approach for nonlinear analysis of the load displacement response of a single and pile groups embedded in multilayered soils. The relationship between shaft displacement and skin friction was presented, a hyperbolic model was used to capture the relationship between skin friction and relationship between end resistance and end displacement pile-soil relative displacement developing along the pile-soil interface. The model of an individual pile in pile groups were proposed. Computer program was developed using the proposed models. comparison of load-settlement responses demonstrated that the proposed method is good agreement with the field observed behavior.

(Priya,2017)

Carry out parametric analysis of a group of piles by analyzing using finite element method (The STAAD Pro software) and comparing the results obtained using empirical equations (Brom’s method and Vedic’s method). The piles are modelled as linear elements. The effect of soil structure interaction is considered by assuming it as vertical and horizontal soil spring (Winkler soil spring). The pile group is subjected to both vertical and horizontal forces. Brom`s methods which gave accurate results can be adopted for small scale projects and when software is not available for the analysis.

Chapter Three

Analysis and Design of bored Pile

Chapter Three

Analysis and Design of bored Pile

3.1 Introduction:

In this chapter a general expression for the theoretical analysis using empirical formulas also, the software computer program Allpile 6.5. to analysis and design single, group pile under vertical, lateral load and moment is given and determine the piles settlement and deflection, while materials for piles can be precisely specified, and their fabrication and installation can be controlled to conform to strict specification and code of practice requirements.

3.2 Load transfer mechanism for piles:

The load transfer mechanism from a pile to the soil is complicated. To understand it, consider a pile of length L, as shown in Figure 3.1a. The load on the pile is gradually increased from zero to Q(z=0) at the ground surface. Part of this load will be resisted by the side friction developed along the shaft, Q_1, and part by the soil below the tip of the pile, Q_2. Now, how are Q_1 and Q_2 related to the total load? If measurements are made to obtain the load carried by the pile shaft, Q_z at any depth z, the nature of the variation found will be like that shown in curve 1 of Figure (3.1b). The frictional resistance per unit area at any depth z may be determined as

$$f(z) = \frac{\Delta Q_{(z)}}{P(\Delta Z)} \qquad (3.1)$$

If the load Q at the ground surface is gradually increased, maximum frictional resistance along the pile shaft will be fully mobilized when the relative displacement between the soil and the pile is about 5 to 10 mm, irrespective of the pile size and length L. However, the maximum point resistance will not be mobilized until the tip of the pile has moved about 10 to 25% of the pile width (or diameter). (The lower limit applies to driven piles and the upper limit to bored piles). At ultimate load (Figure 3.1d and curve 2 in (Figure 3.1b), Q(z=0) = Qu. Thus,

$$Q_1 = Q_s \qquad (3.2)$$

$$Q_2 = Q_p \qquad (3.3)$$

At ultimate load, the failure surface in the soil at the pile tip (a bearing capacity failure caused by Q_p is like that shown in Figure 3.1e. Note that pile foundations are deep foundations and that the soil fails mostly in a punching mode, as illustrated previously in Figures 3.2e. That is, a triangular zone, I, is developed at the pile tip, which is pushed downward without producing any other visible slip surface. In dense sands and stiff clayey soils, a radial shear zone, II, may partially develop. Hence, the load displacement curves of piles will resemble those shown in Figure 3.2e. (Das,2007)

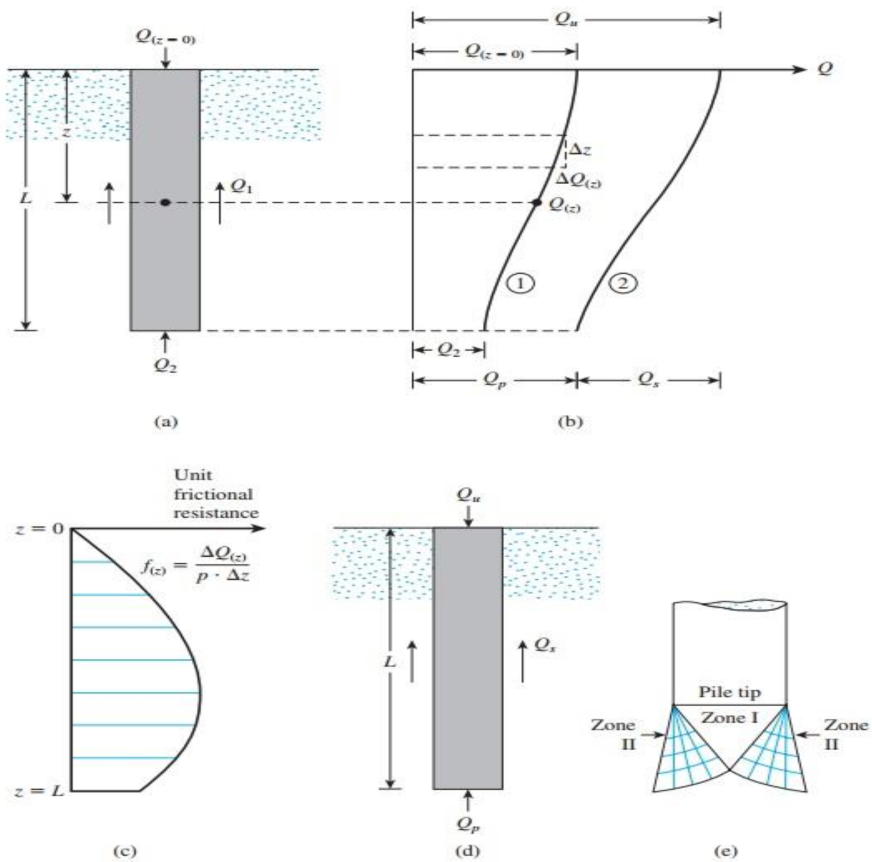


Figure (3.1) Load transfer mechanism for piles (Das ,2007)

3.3 Load carrying capacity of single piles:

Like shallow foundation, a pile foundation should be safe against shear failure and the settlement should be within the permissible limits. The methods for estimating the load carrying capacity of a pile foundation can be grouped into the following three categories:

3.3.1 Dynamic formula:

The ultimate capacity of pile driven in certain type of soil is related to the resistance against penetration developed during driving operation. pile to further penetration by driving depend upon the energy imparted to the pile by the hummer. it is tacitly assumed that the load carrying capacity of pile equal to the dynamic resistance during driving.

3.3.2 Pile load test:

load test may be carried out either on a driven pile or a cast-in-situ pile. Load tests may be made either on a single pile or a group of piles. Load tests on a pile group are very costly and may be undertaken only in very important projects.

Pile load tests on a single pile or a group of piles are conducted for the determination of:

- Vertical load bearing capacity.
- Settlement.
- Uplift load capacity,
- Lateral load capacity.

Generally, load tests are made to determine the bearing capacity and to establish the load settlement relationship under a compressive load. The other two types of tests may be carried out only when piles are required to resist large uplift or lateral forces.

3.3.3 Static methods:

the static methods give the ultimate capacity of an individual pile, depending upon the characteristics of the soil. The ultimate load

capacity is given by

$$Q_u = Q_p + Q_s \qquad (3.4)$$

$$Q_p = q_p\, A_P \qquad (3.5)$$

$$Q_S = q_s\, A_s \qquad (3.6)$$

The ultimate bearing capacity (q_p) of the soil at the pile tip can be computed from the bearing capacity equation similar to that for a shallow foundation, Terzaghi (1943) was the first to present a comprehensive theory for the evaluation of the ultimate bearing capacity of rough shallow foundations. (Arora, 2004)

According to Terzaghi's equations (Das,2007)

(for shallow square foundation)

$$q_u = 1.3c`qN_c + q`N_q + 0.4\gamma B\, N_\gamma \qquad (3.7)$$

(for shallow circle foundation)

$$q_u = 1.3c`qN_c + q`N_q + 0.3\gamma B\, N_\gamma \qquad (3.8)$$

Because the width to Depth for pile is relatively small, then $\gamma D\, N_\gamma{}^*$ may be dropped from the equation without introducing error.

$$q_p = c`N_c{}^* + qN_q{}^* \qquad (3.9)$$

The point bearing of the pile is

$$Q_p = q_p A_p = A_p(c`N_c{}^* + qN_q{}^*) \qquad (3.10)$$

3.3.1.1 Meyerhof 's method for estimating Q_p:

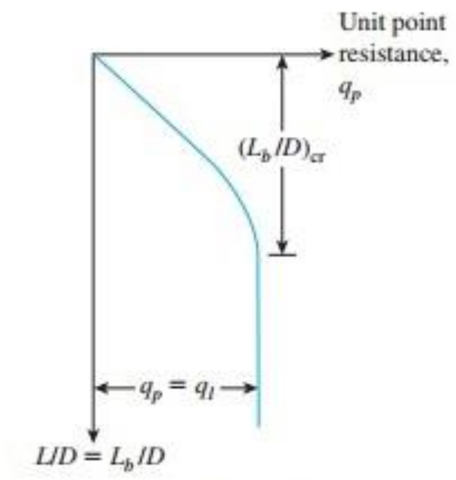
- **Sand (c=0):**

The point bearing q_p, of a pile in sand generally increases with the depth of embedment in the bearing stratum and reaches a maximum value at an embedment ratio of

$$L_b/D = (L_b/D)_{cr} \qquad (3.11)$$

L_b is equal to the actual embedment length of the pile. Beyond the critical embedment ratio $(L_b/D)_{cr}$, the value of q_p remain constant (q_p = q_1) .

That is as shown in figure (3.2)



Figure(3.2) Nature of variation of unit point resistance in a homogeneous sand(Das,2007)

For piles in sand c = 0 and equation 3.10 simplifies

$$Q_p = A_p \text{ q } N_q{}^{*} \qquad (3.12)$$

The bearing capacity factor $N_q{}^{*}$ depend upon the angle of shearing resistance (ϕ/) is shown in (figure 3.3), however Q_p should not exceed the limiting value $A_p\ q_1$.

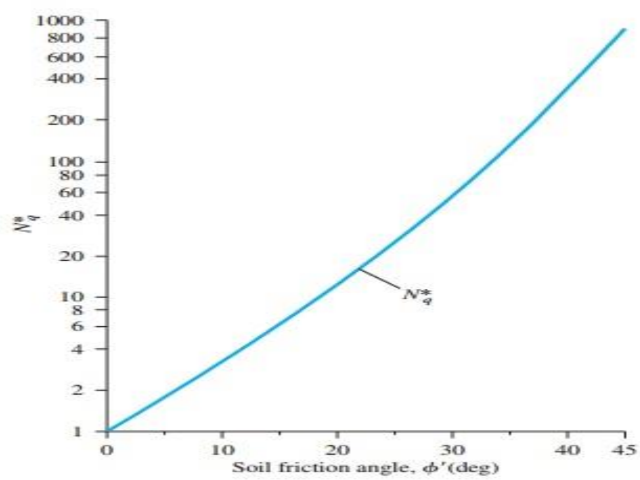


Figure (3.3) Variation of the maximum values of Nq* with soil friction angle ϕ′ (Das,2007)

3.3.1.3 methods for estimating Q_s :

- **For Sand (c =0)**

The frictional or skin resistance Q_s of pile may be written as

$$Q_s = \sum p \, \Delta L \, f \qquad (3.13)$$

the unit skin friction for a straight – sided pile depends upon the soil pressure acting normal to the pile surface and coefficient of friction between the soil and the pile material figure (3.5). (Arora, 2004)

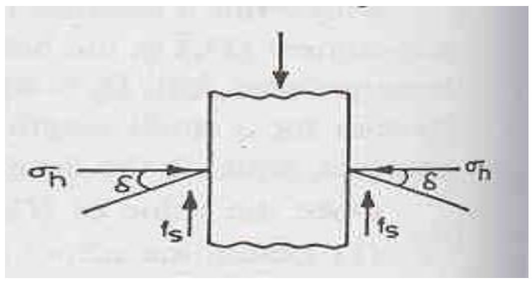


Figure (3.4) Unit frictional resistance for piles in sand (Arora, 2004)

From figure (3.4) the soil pressure normal to the vertical pile surface is horizontal pressure (σ_h) and is related to the effective vertical soil pressure as

$$\sigma_h = k\,\sigma_v \qquad (3.14)$$

$$f_s = \sigma_h tan\delta' \quad \text{or} \quad f_s = k\,\grave{\sigma}_h tan\delta' \qquad (3.15)$$

approximate value of K can obtain from the following equation. (Das,2007)

$$K = 1 - \sin\phi' \qquad (3.16)$$

The value of K generally varies between 0.3 and 0.75. average value of 0.5 is usually adopted. K value is given in table (3.1) (Das,2007)

the value of δ from various investigation appear to be in range from 0.5ϕ′ to 0.8 ϕ′

Table (3.1) K value (Das,2007)

Type of pile	K
Bored or jetted	$K_0 \approx 1-\sin\phi'$
Low – displacement driven	$K_0 \approx 1-\sin\phi'$ to $1.4K_0 = 1.4(1-\sin\phi')$
high– displacement driven	$K_0 \approx 1-\sin\phi'$ to $1.8K_0 = 1.8(1-\sin\phi')$

$$Q_s = f_s\ PL = (k\ \sigma_h tan\delta')\ PL \tag{3.17}$$

As stated earlier, the effective vertical pressure ($\grave{\sigma_v}$) increases with depth only up to the critical depth below the critical depth, the value of σ_v remains constant.

The frictional resistance (Q_s) can be expressed as (Arora, 2004)

$$Q_s = \sum_{i=1}^{n} K(\grave{\sigma_v})_i\ \tan\delta'\ (A_s)_i \tag{3.18}$$

The Equation (3.18) can be written as

$$Q_s = \sum_{i=1}^{n} K \tan\delta' \text{ (area of } \grave{\sigma}_v \text{ diagram) * pile perimeter} \tag{3.19}$$

The ultimate load for pile can be written for the sand soil as equation (3.4)

$$Q_u = q'\ N_p\ A_p + \sum_{i=1}^{n} K\,(\grave{\sigma_v})_i\ \tan\delta'\ (A_s)_i \tag{3.20}$$

3.4 Allowable load for pile:

After the total ultimate load-carrying capacity of a pile has been determined by summing the point bearing capacity and the frictional (or skin) resistance, a reasonable factor of safety should be used to obtain the total allowable load for each pile,

$$Q_{all} = \frac{Q_u}{F_s} \tag{3.21}$$

F_S is factor of safety the factor of safety generally used ranges from 2.5 to 4, depending on the uncertainties surrounding the calculation of ultimate load. (Das,2007)

3.6 Analysis of pile to resist lateral loading:

Piles are frequently subjected to lateral load and for example quay and harbor structure where horizontal force are cause by the impact of ships during berthing and weave action , structure subject to wind load ,earth quake ,or pile supported earth-retaining structure .the problem of laterally loaded pile embedded in soil is closely related to the beam on an elastic foundation . A beam can be loaded at one or more point along its length, where in the case of piles the external loads and moments are applied at or above the ground surface only.

A vertical pile resists a lateral load by mobilizing passive pressure in the soil surrounding it. (Das,2007)

The degree of distribution of the soil's reaction depends on

(a) The stiffness of the pile,

(b) The stiffness of the soil, and

(c) The fixity of the ends of the pile.

3.6.1Winkler's Hypothesis:

Most of the theoretical solution for laterally loaded pile involve the concept of subgrade reaction or otherwise termed as soil modulus which based on Winkler's assumption that a soil elastic medium may be approximated by a series of closely spaced independent elastic springs. (Murthy.1969)

based on this assumption

$$k = \frac{p' \text{ (KN /m)}}{y(m)} \qquad (3.22)$$

using the theory of beam on elastic foundation we can write

$$EI\frac{d^4x}{dz^4} = p' \qquad (3.23)$$

Based on Winkler's model

$$p' = -k\ y \qquad (3.24)$$

the sign negative because the soil reaction is in the direction opposite that of the pile deflection

combining the equations (3.64) and (3.65) gives for zero axial load:

$$EI\ \frac{d^4x}{dz^4} + k\ y = 0 \qquad (3.25)$$

3.6.2 Laterally loaded piles can be divided into two major categories:

3.6.2.1Short or rigid piles:

The lateral load required to cause failure of soil along the pile length (shear failure in the soil) before failure of the pile. (Das,2007).

the pile then rigid and its capacity governed by the soil resistance. (Poulos,1980)

A short rigid pile unrestrained at the top and having a length to width ratio of less than 10 to 12. The short rigid pile will fail by rotation when the passive resistance of the soil at the head and toe are exceeded figure (3.8a). rigid pile restrained at the head by a cap will fail by translation figure (3.8b). (Tomilson,2008)

3.6.2.2Long or elastic piles:

The failure mechanism of an infinitely long pile is different. The passive resistance of the lower part of the pile is infinite, and thus rotation of the pile cannot occur, the lower part remaining vertical while the upper part deforms to a shape shown in Figure (3.5a). Failure takes place when the pile fractures at the point of maximum bending moment, and for the purpose of analysis a plastic hinge capable of transmitting shear is assumed to develop at the point of fracture. In the case of a long pile restrained at the head, high bending stresses develop at the point of restraint, e.g. just beneath the pile cap, and the pile may fracture at this point (Figure 3.5b). (Tomilson,2008)

The ultimate lateral resistance may be determined by the yield moment of pile which may be reached before full mobilization of the ultimate soil resistance. pile failure happened before the soil, lateral capacity of pile governed by pile characteristics, long piles would generally fail by bending. (Polous.1980)

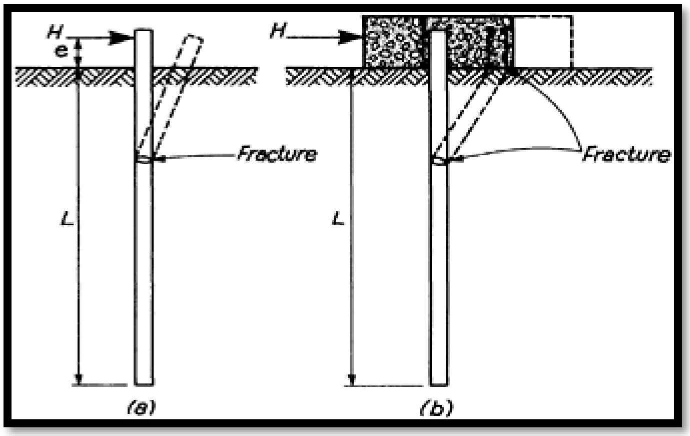


Figure. (3.5) Long vertical pile under horizontal load
(a) Free head (b) Fixed head (Tomilson,2008)

3.6.3 Calculating the ultimate resistance to lateral loads :method to analysis single pile under lateral load:

3.6.3.1 BrinchHasen's method

3.6.3.2 Brom's method

The first step is to determine whether the pile will behave as a short rigid unit or as an infinitely long flexible member. This is done by calculating the stiffness factors R and T for the combination of pile and soil. The stiffness factors are governed by the stiffness (EI value) of the pile and the compressibility of the soil. The latter is expressed in terms of a 'soil modulus', which is not constant for any soil type but depends on the width of the pile B and the depth of the loaded area of soil being considered.

In the case of a stiff over-consolidated clay, the soil modulus is generally assumed to be constant with depth. (Tomilson,2008)

$$R = \sqrt[4]{\frac{EI}{kB}} \qquad (3.26)$$

$$T = \sqrt[5]{\frac{EI}{n_h}} \qquad (3.27)$$

Values of the coefficient of modulus variation n_h were obtained directly from lateral loading tests.

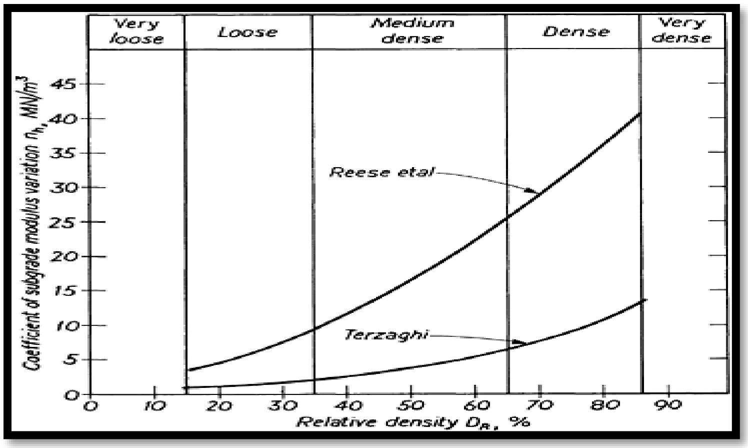


Figure (3.6) Relationship between coefficient of modulus variation and relative density of sands (Tomilson,2008)

Representative value of n_h Table (3.2) (Das,2007)

Soil	n_h kN/m³	n_h lb/in³
Dry or moist sand		
Loose	1800–2200	6.5–8.0
Medium	5500–7000	20–25
Dense	15,000–18,000	55–65
Submerged sand		
Loose	1000–1400	3.5–5.0
Medium	3500–4500	12–18
Dense	9000–12,000	32–45

Having calculated the stiffness factors R or T, the criteria for behavior as a short rigid pile or as a long elastic pile are related to the embedded length L as shows in table (3.3):

Table (3.3) Stiffness factor

Pile type	Soil modulus	
	Linearly-increasing	Constant
Rigid (free head)	$L \leq 2T$	$L \leq 2R$
Elastic (free head)	$L \geq 2T$	$L \geq 3.5R$

When $L \geq 5T$ the pile considered to be long pile. for $L \leq 2T$ the pile considered to be rigid. (Tomilson,2007)

3.6.3.1 Ultimate resistance of short rigid piles Brinchhansen's method:

Can be used to calculate the ultimate lateral resistance of short rigid piles. The method is a simple one which can be applied both to uniform and layered soils. The resistance of the rigid unit to rotation about point X in Figure (3.7) a is given by the sum of the moments of the soil resistance above and below this point.

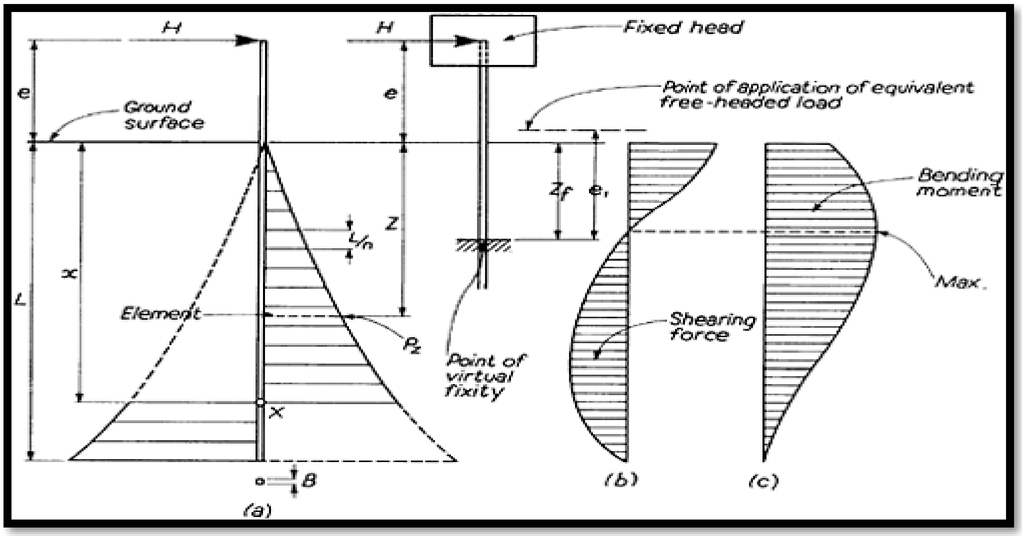


Figure. (3.7) BrinchHansen's method for calculating ultimate lateral resistance of c) Bending moment b) Shearing force diagram((short pile (a) Soil reactions diagram (Tomilson,2008)

The passive resistance diagram is divided into a convenient number n of horizontal elements of depth L/n. The unit passive resistance of an element at a depth z below the ground surface is then given by

$$P_Z = P_{OZ}K_q + CK_c \qquad (3.28)$$

and K_{qz} andK_{cz} are the passive pressure coefficients for the frictional and cohesive components respectively at depth z.

BrinchHansen has established values of K_q and K_c in relation to the depth z and the width of the pile B in the direction of rotation, as shown in figure (3.8)

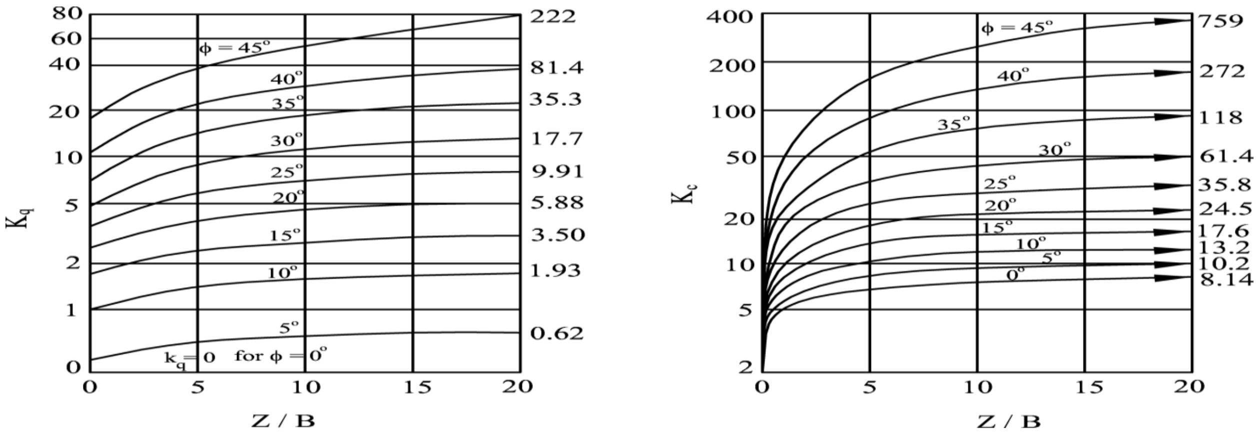


Figure (3.8) Brinch Hansen's coefficients K_q and K_c (Tomilson,2008)

The total passive resistance on each horizontal element is $p_Z(e+z)B$ and, by taking moments about the point of application of the horizontal load,

$$\sum M = \sum_{z=0}^{z=x} p_z \frac{L}{n}(e+z)B - \sum_{z=x}^{z=L} p_z \frac{L}{n}(e+z)B \qquad (3.29)$$

The point of rotation at depth x is correctly chosen when the passive resistance of the soil above the point of rotation balances that below it. Point X is thus determined by a process of trial and adjustment. If the head of the pile carries a moment M instead of a horizontal force, the moment can be replaced by a horizontal force H at a distance e above the ground surface where M is equal to H×e. Where the head of the pile is fixed against rotation, the equivalent height e1 above ground level of a force H acting on a pile with a free head is given by

$$e_1 = \frac{1}{2}(e + z_f) \qquad (3.30)$$

where is the height from the ground surface to the point of application of the load at the fixed head of the pile Figure (3.7), and z_f is the depth from the ground surface to the point of virtual fixity or the point of zero shear .

Having obtained the depth to the center of rotation from equation (3.29), the ultimate lateral resistance of the pile to the horizontal force H_u can be obtained by taking moments about the point of rotation, when

$$H_u(e+x) = \sum_0^x p_z \frac{L}{n} B(x-z)B + \sum_x^{x+L} p_z \frac{L}{n} + (z-x)B \qquad (3.31)$$

The final steps in Brinchhansen's method are to construct the shearing force and bending moment diagrams Figure (3.7 b and 3.7 c). The ultimate bending moment, which occurs at the point of zero shear, should not exceed the ultimate moment of resistance M_u of the pile shaft. The appropriate load factors are applied to the horizontal design force to obtain the ultimate force H_u.

3.6.3.2 Brom's method :

Brom's (1965) developed a simplified solution based on the assumption of:

1.Shear failure in soil, which is case for short piles and

2.Bending of pile, governed by the plastic yield resistance of the pile section which applicable to long pile. **(**Das, 2007).

Brom`s provide solution for both short and long piles installed in cohesive and cohesionless soil. He considered pile fixed or free to rotate at the head. (Murthy,1969).

- **Ultimate resistance of long piles:**

The passive resistance provided by the soil to the yielding of an infinitely long pile is infinite. Thus, the ultimate lateral load which can be carried by the pile is determined solely from the ultimate moment of resistance M_u of the pile shaft.

simple method:

ultimate lateral load free headed pile $H_u = M_u/(e + z_f)$ (3.32)

ultimate lateral load fixed headed pile $H_u = 2M_u/(e + z_f)$ (3.33)

z_f should be taken as 1.4R for stiff, over-consolidated clay and 1.8T for normally consolidated clay, granular soils and silt. (Tomilson,2008)

*** Cohesion less soil: c = 0**

For long piles in cohesion less soils the soil reactions and bending moments for free-headed piles are shown in Figure (3.29 a). The maximum bending moment on the pile shaft occurs at the point where the shearing force is zero.

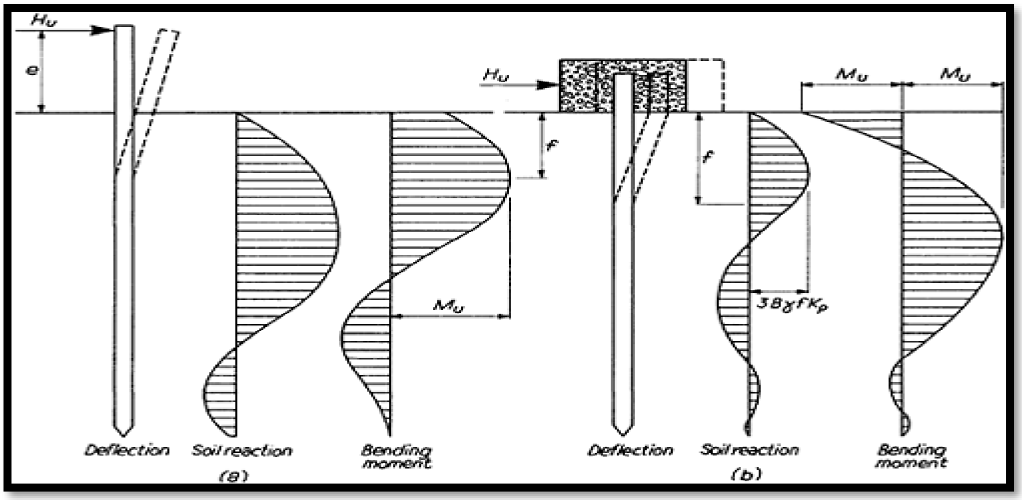


Figure (3.9) Soil reactions and bending moments for long pile under horizontal load in cohesionless soil (Brom`s method) (a) Free head (b) Fixed head (Tomilson,2008)

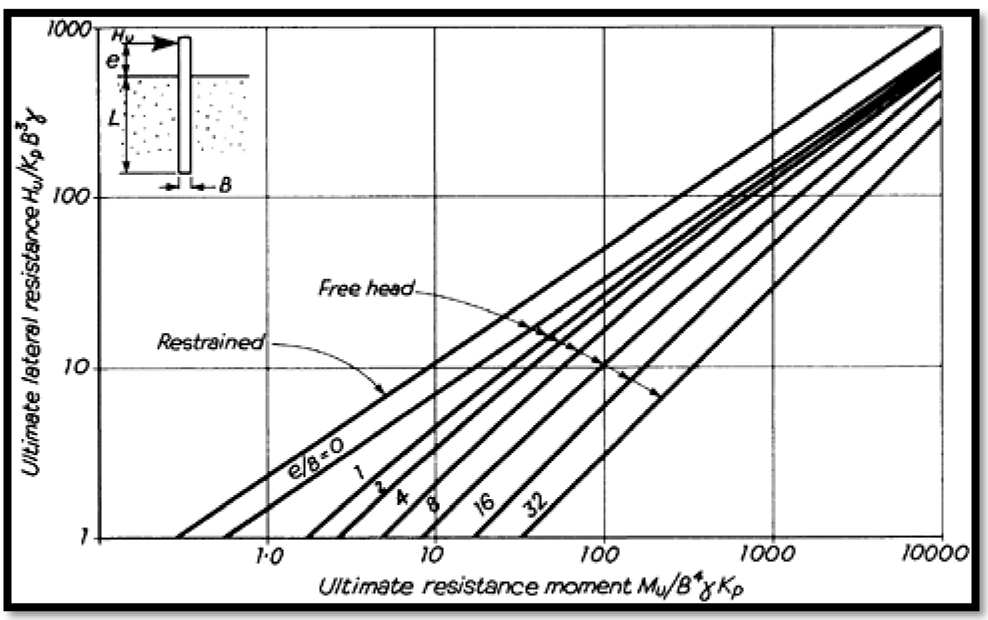


Figure (3.10) Ultimate lateral resistance of long pile in cohesionless soil related to ultimate resistance moment (Brom`s method) (Tomilson,2008).

Brom`s has established the graphical relationship between $H/K_p\gamma B^3$

and M_u / (γKpB^4)shown in Figure (3.21). These graphs can be used to determine the ultimate lateral load Hu.

3.7 Pile Group:

3.7.1 Group action of piles:

In most cases pile group is used in to transmit the structural load that supported by several piles acting as a group to the soil. a pile cap is constructed over the group pile) The load acts on the pile cap which distributes the load to the pile see figure (3.11).

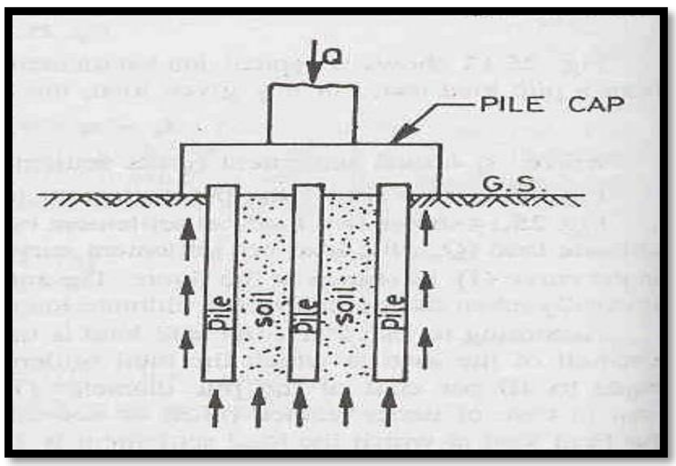


Figure (3.11) Pile cap (Arora,2004)

The load carrying capacity of a pile group is not necessarily equal to the sum of the capacity of the individual piles. Estimation of the load carrying capacity of a pile group is a complicated problem. When the piles are spaced enough distance apart, the group capacity may approach the sum of the individual capacities. if the piles are closely spaced, the stresses transmitted by the piles to the soil may overlap, and this may reduce the load carrying capacity of the piles figure (3.12) and figure (3.13) .

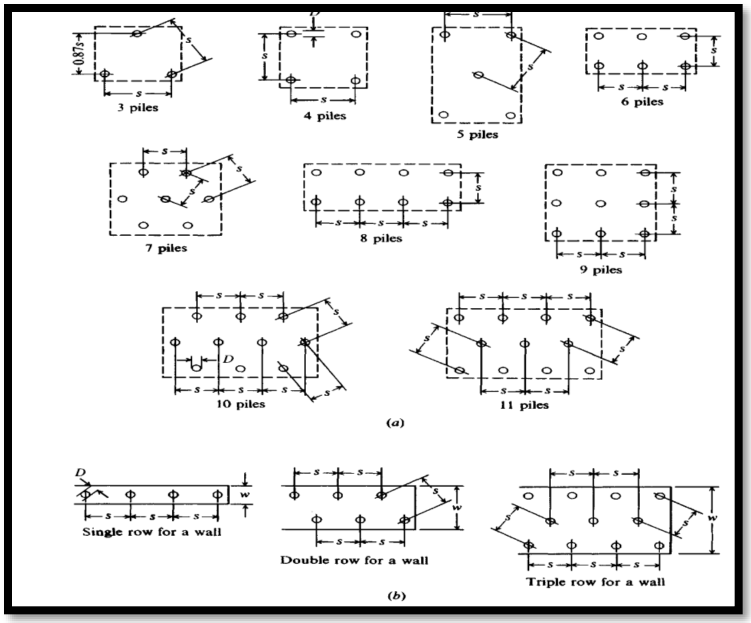


Figure (3.12) Typical pile-group patterns: (a) for isolated pile caps; (b) for foundation walls (Bowls,1997)

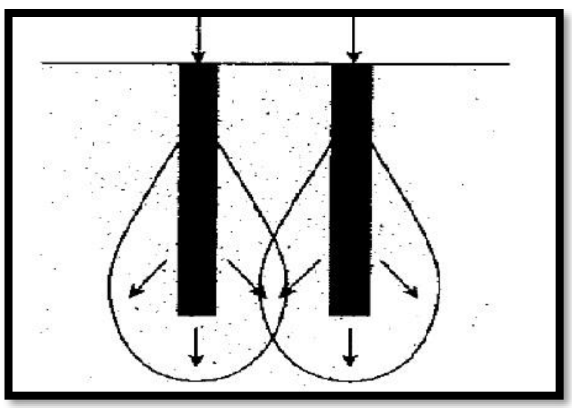


Figure (3.13) soil overlap (Das.2007)

3.7.2The efficiency (η_g) of a group piles: are defined as the ratio of the ultimate load of the group to the sum of individual ultimate loads. (Das ,2007)

$$\text{Thus } \eta_g = \frac{Qg(u)}{NQu} x100 \tag{3.34}$$

$$\eta_g = \frac{2(n1 + n2 - 2)d + 4d}{p\, n1\, n2} \tag{3.35}$$

if $\eta_g \geq 1$ in that case the piles will behave as individual piles

$$Q_g\ (u) = \sum Q_u \qquad (3.36)$$

if $\eta_g < 1$ then in this case

$$Q_g\ (u) = \eta_g \sum Q_u \qquad (3.37)$$

3.8 General analysis of pile group:

In general, a pile group may be subjected to simultaneous axial load, lateral load, moment and possibly, torsional. Method of analyzing the pile group shown below:

3.8.1Simple statically method:

That ignore the presence of the soil and consider the pile group as purely structural system. Traditional design method has relied on consideration of the pile group as simple statically – determinate system, ignoring the effect of the soil. one such method which may be employed either graphically or analytically. Considering, for simplicity, load and pile having a pinned head, the steps in this method are as follows:

- assuming each pile to take an equal share of the vertical load on the cap and assuming the vertical load in pile caused by moment in the cap, to be proportional to the distance x, the vertical pile loads are calculated as

$$V_i = \frac{V}{N} + \frac{Mxi}{\sum_{i=0}^{n} \mathrm{Iy}}\ \mathrm{x} + \frac{Myi}{\sum_{i=0}^{n} \mathrm{Ix}}\ \mathrm{y} \qquad (3.38)$$

- There then a residual horizontal force H which is assumed to be equally distributed between each pile in the group.

It should be noted that this method cannot take in to account different conditions of fixity at the pile head and always assume zero moment at the head of each pile group is thus obtained. (Polous,1980)

- Pile group resist horizontal load by bending:

A group of vertical piles subjected to horizontal load H applied at the top of the pile is showing in figure (3.14). the piles are assumed to be fixed at the top and bottom.

Shear per pile. (Macginley,2009)

Lateral load in each pile = H/N (3.39)

Moment in each pile = H h_1/(2N) (3.40)

Where N is the number of pile and h_1 is length of pile between fixed end.

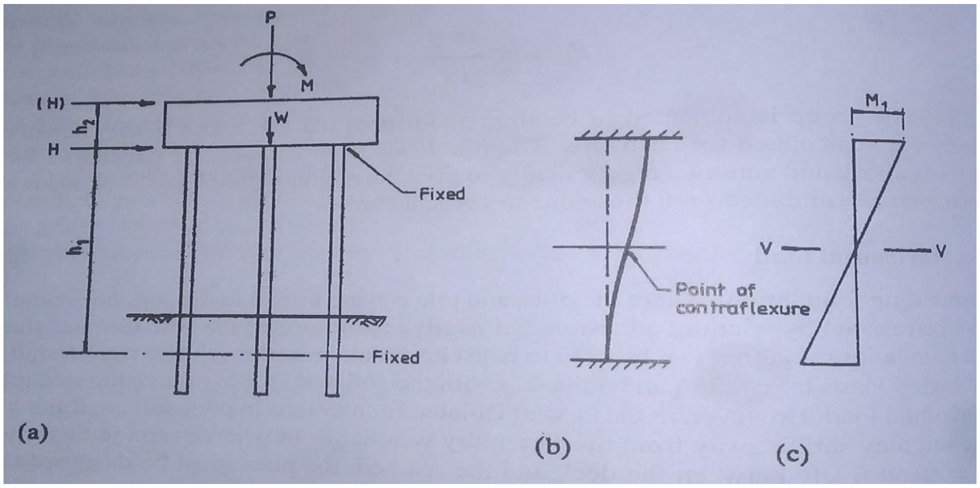


Figure (3.14) (a) pile group (b) deflection (c) moment diagram

(Mcginley,2009)

3.9 Allpile6.5 software analysis:

The program Allpile6.5 from CivilTech software analyze pile load capacity efficiently and accurately. Allpile6.5 handle all type of pile, the program can perform the following calculation:

- Vertical capacity and deflection.
- Vertical capacity and settlement.
- Group vertical and lateral analysis.
- Static and cyclic condition.
- Negative and zero friction.

In this research negative and zero friction, cyclic condition not considering

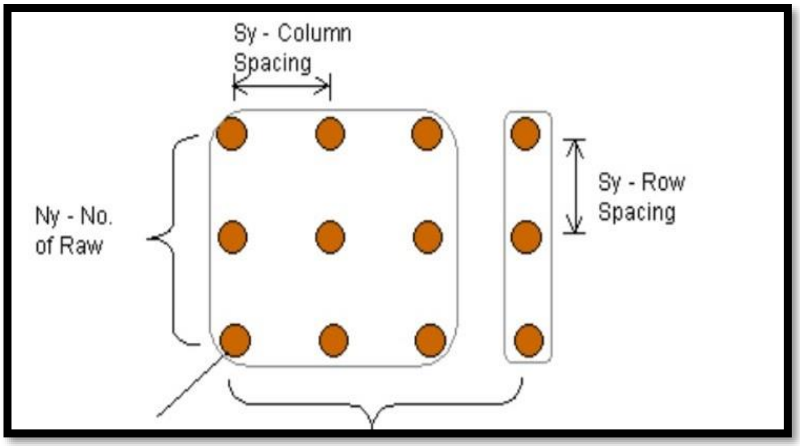


Figure (3.15) group pile for vertical analysis

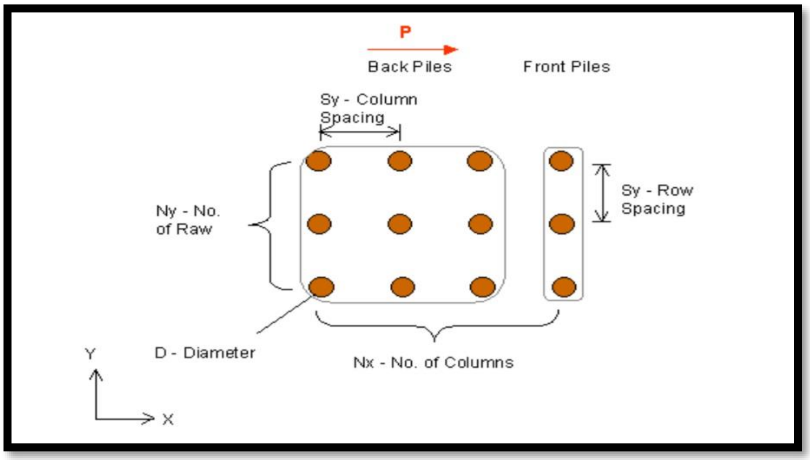


Figure (3.16) group pile for lateral load

3.9.1 Lateral load analysis:

The method utilized in the laterally loaded pile program, is based on the theory of subgrade reaction discussed above see equations (3.22 and 3.4).

3.9.1.2 p-y concept of lateral load transfer:

when the basic beam – column is inserted vertically as pile shaft , the method of analysis Allpile6.5 consider the soil surrounding the shaft as a set of nonlinear elastic spring a depicted in figure (3.17) .this assumption is attributed to Winkler (1967) , and it states that each spring act independently , the behavior of one spring has no effect on any of the adjacent spring .

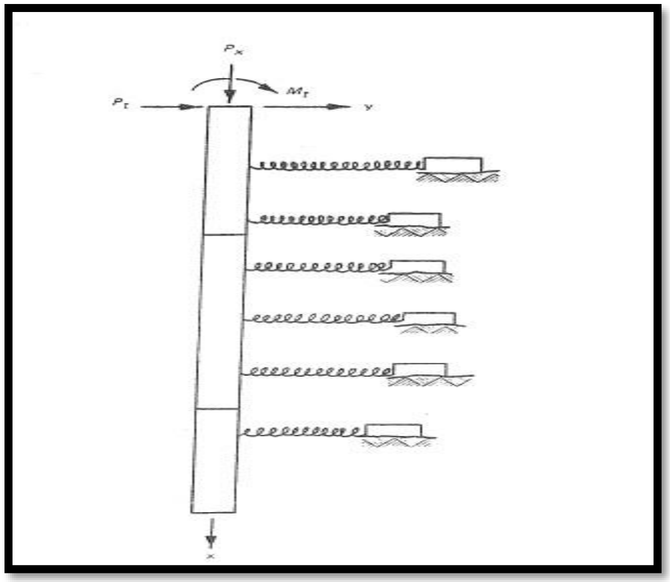


Figure (3.17) model of pile system with soil represented as set of nonlinear elastic spring (Reese, 1984).

in the analysis the response of spring can take as either linear or nonlinear. the approach in program Allpile6.5 is to treat the springs as nonlinear with their response represented by curves which relate soil resistance p to pile deflection y. in general , these curves are nonlinear and depend on several parameters including depth , pile geometry , shear strength of soil , and type of loading (static or cyclic).

A typical p-y curve is shown in figure (3.18).

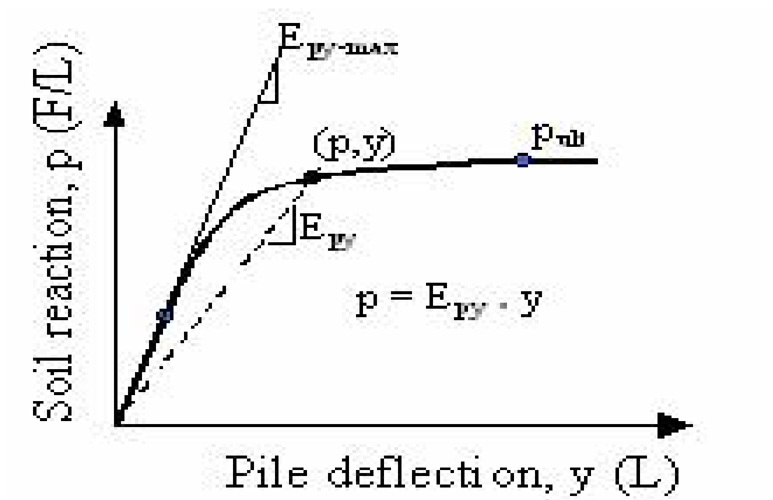


Figure (3.18) characteristic shape of p-y curve (Reese, 1984)

The soil modulus E_s is defined as –p/y and is taken as the secant modulus to a point on p-y curve as shown in figure (3.18). Because the curve is strongly nonlinear, the soil modulus changes from initial stiffness Es_1 to ultimate stiffness p_u/y_u. as can be seen the soil modulus Es is not a constant except for small range deflection.

All pile directly program solve the nonlinear different equations representing the behavior of the pile – soil system to lateral (shear and moment) loading conditions in finite difference formulation using Reese’s p-y method of analysis. For each set of applied boundary loads the program performs an iterative solution which satisfies static equilibrium and achieves an acceptable compatibility between force and deflection (p and y) in every element. The program uses the four nonlinear differential equation to perform the lateral analysis, equation (3.25).

3.9.2The Allpile6.5 software input steps:

Allpile6.5 can be divided into six input pages:

First Pile type page: you can select the pile type there are twelve different type to choose.

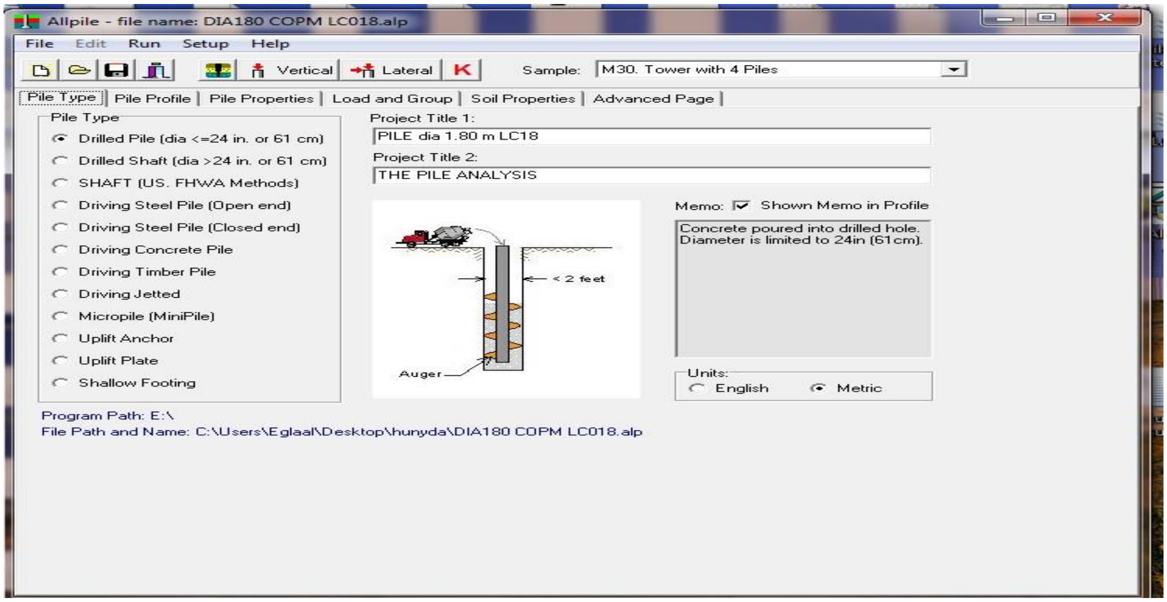


Figure (3.19) Pile type page

Second input pile profile page: input the pile length, distance from the ground level, surface angle and batter angle.

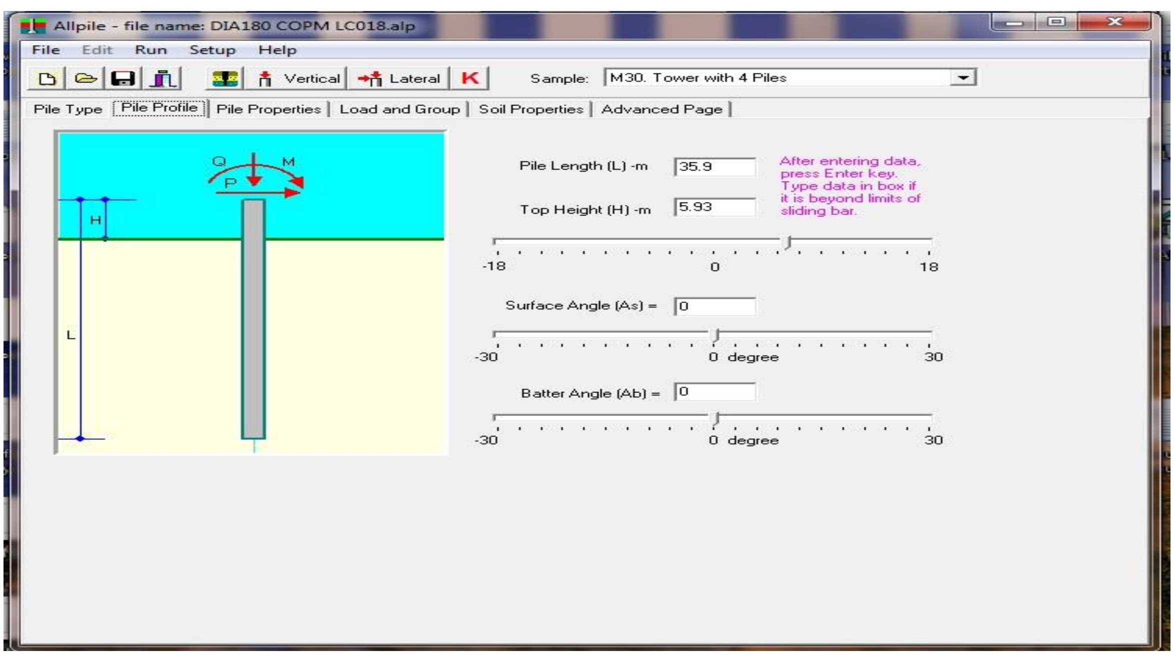


Figure (3.20) pile profile page

Third input pile property page: in this page input the data through the section is (width, area, perimeter, moment of inertia, elastic modulus, depth of pile,…..)

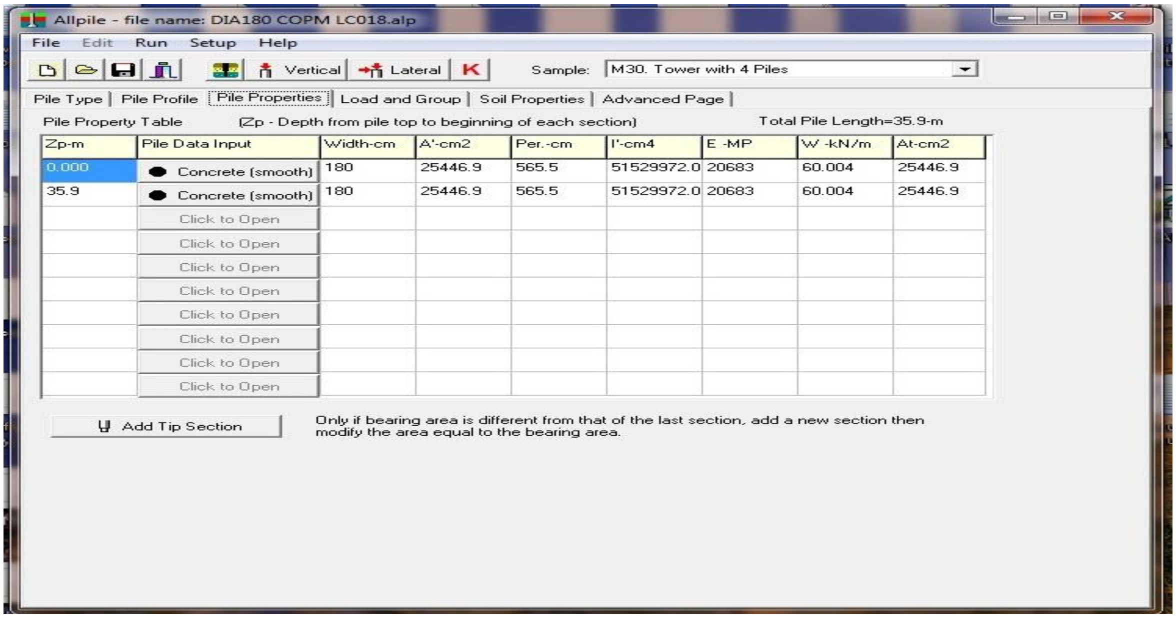


Figure (3.21) pile property page

Fourth input load and group: select the pile configuration that most fit to analysis (single pile, group pile or tower foundation), determine static or cyclic load and input vertical load, shear force (lateral load) , moment , torsion .

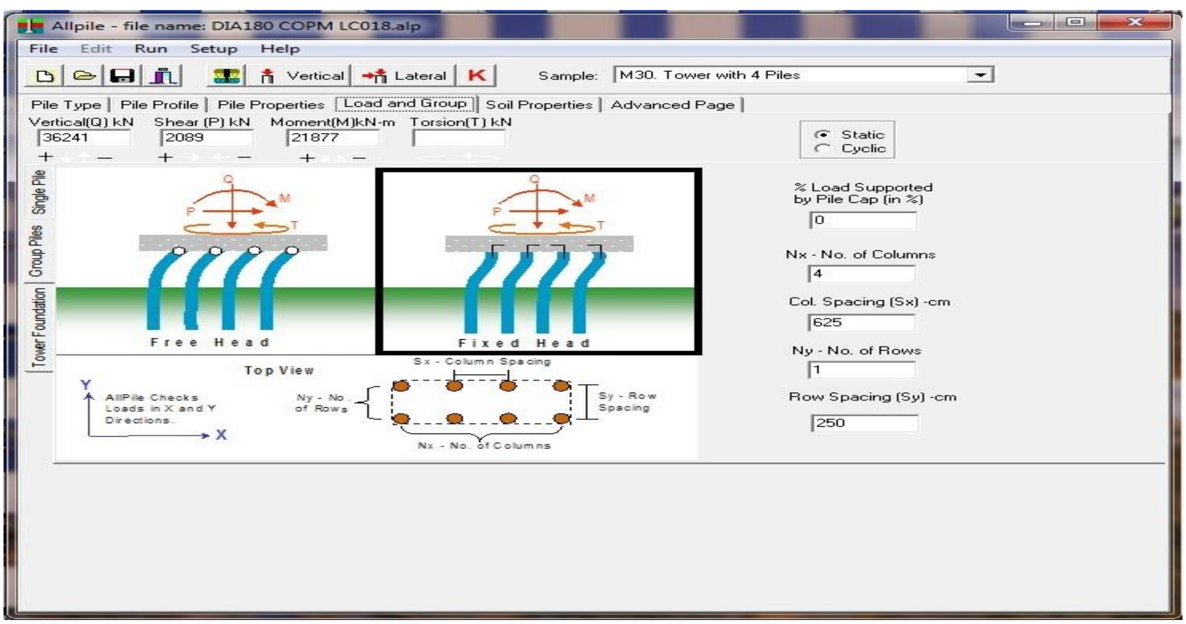


Figure (3.22) load and group page

Fifth input soil property page: input the parameters of the soil (angle of friction,) and must determine the water level.

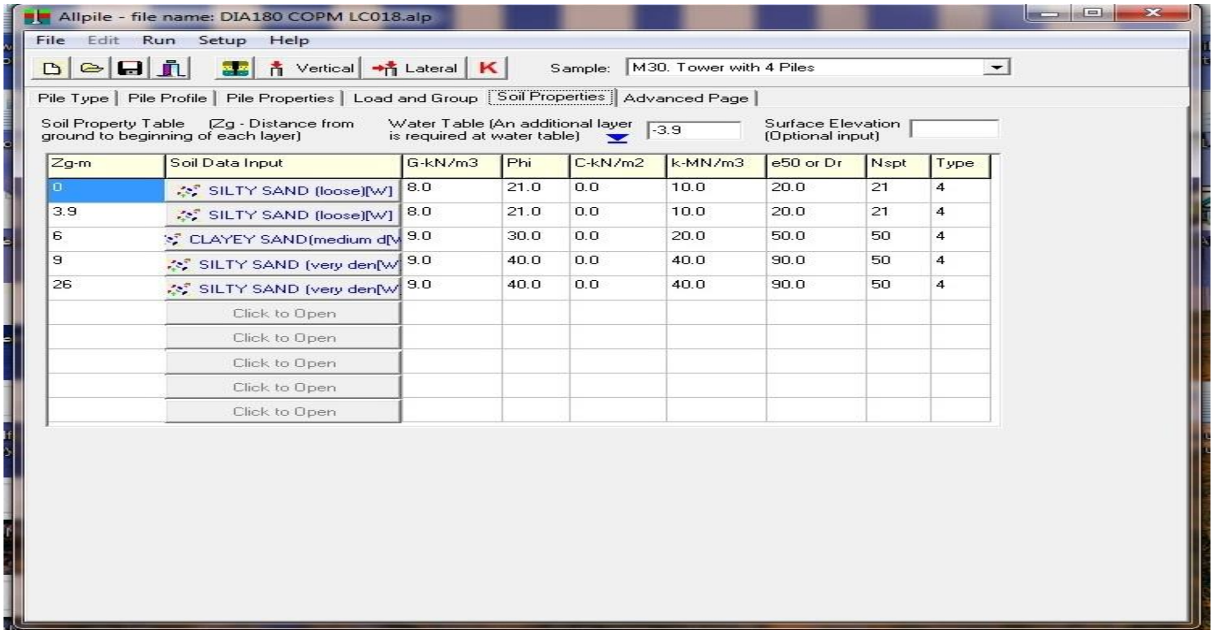


Figure (3.23) soil property page

Sixth advance page: this page allows to assign analysis parameter like factor of safety for vertical and lateral loads, resistance limit, allowable deflection.

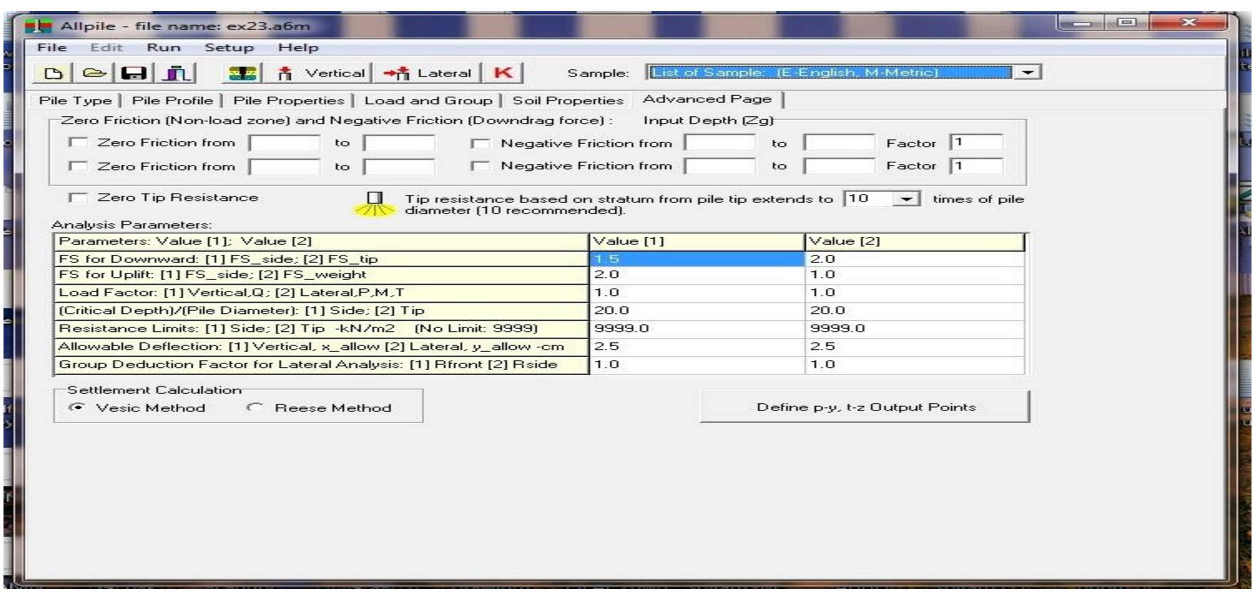


Figure (3.24) advance page

3.9.3This program provides the result as follows:

Vertical analysis results: click on (vertical analysis) will display a panel that allows to choose the different type of result from analysis. for this analysis all lateral load components are ignored and only vertical load considering.

Lateral analysis results: click on (lateral analysis) will display a panel that provides several choices.

3.10 Design of bored pile:

in this research will design reinforce concrete pile under axial load moment and lateral load. when subjected to axial load and moment the reinforced concrete piles are considered as columns the effective lengths for various conditions of end restraint are given in BS 8110 respectively. (Tomilson,2008). But for lateral load will design as beam (Winkler's assumption).

Chapter (4)

Case Study Analysis and Design

Chapter Four

Analysis Results and Design of Soba Bridge Piles

4.1 Introduction:

In this chapter pile foundation of soba bridge have been studied. soba bridge contains 13 piers. piers from1 to 3in west shore, from11to 13 in east shore and from 4 to 10 which in river have been studied in this research.

In this study several diameters (1.20,1.6,1.40,1.80 m), several lengths (25.9,32.9 ,35.9m) and several pile groups (1x4,2x4,3x4) subjected to load combinations have been analyzed using manual empirical formula and software Allpile6.5.

Soba Bridge: this bridge located on the Blue Nile which connect east soba area with west soba area in Khartoum see figure (4.1) the bridge length is 571m and width is 6 ways. Published soil investigation recommendations for bored pile is 1.2m diameter and 30 m length but the actual analysis using 1.8 m for diameter and 25.9m for length and 1x4 for number of piles in group.

Figure (4.1) soba bridge location

4.2 Properties of soil for soba bridge:

Soba bridge soil properties shown in table (4.1) which is inside the river.

Table (4.1) Properties of soil for soba bridge

Description	Depth	Properties	Value
Silty sand (loose)	6m	c'	0
		ϕ'	21 deg
		γ'	8 KN/m3
		K	1000 KN/m3
		K0	0.5
Clayey sand (medium dense)	3m	c'	0
		ϕ'	30 deg
		γ'	9 KN/m3
		K	2000 KN/m3
		K0	0.45
Silty sand (very dense)	17m and more	c'	0
		ϕ'	40deg
		γ'	9 KN/m3
		K	4000 KN/m3
		K0	0.35

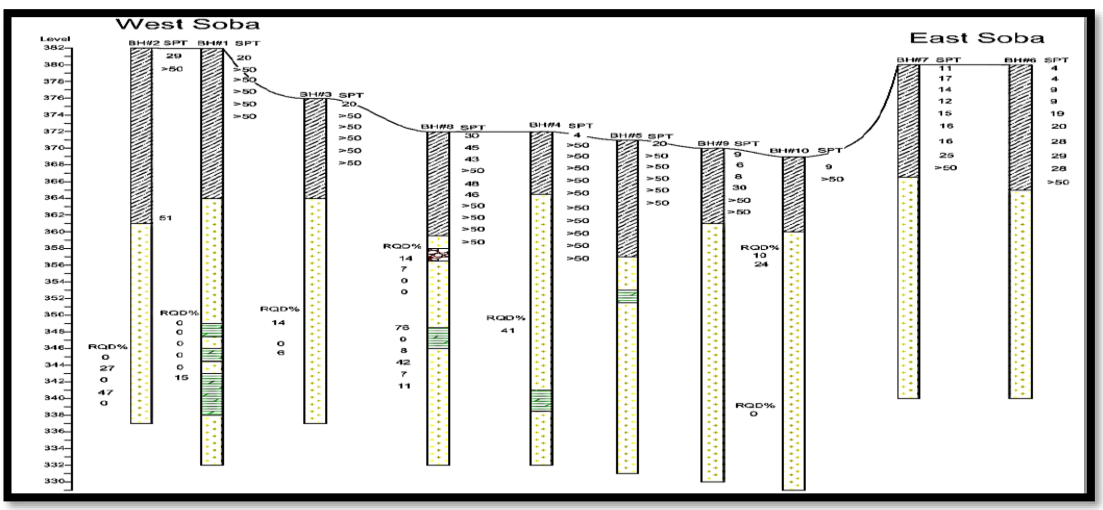


Figure (4.2) Subsurface profile of soba bridge site (Elsharief,2014)

4.3Bearing capacity of vertical pile with several diameter and length:

To analysis and design pile it necessary calculates Bearing Capacity of pile to know how much that pile can carry load without failure. Bearing capacity calculations by manual methods and with All pile soft were show below.

4.3.1Pile bearing capacity manual calculation:

use equations 3.12, 3.17 and 3.20 from previous chapter to calculate the ultimate capacity of an individual pile. to calculate the allowable load for pile, use the equation 3.21and compare the result with soil report and All pile 6.5 result.

Tables from (4.2) to (4.4) describe the pile bearing capacity results from soil report, manual calculations and All pile 6.5 software analysis for Lengths 39.5m ,32.9m and 25.9m respectively. (F.S is factor of safety).

Table (4.2) Bearing capacity of vertical pile length 35.9

#	Diameter m	Manual result		All pile result	Soil report result	
1-	1.20 m	Q_p	13069.52	17110.006		
		Q_s	3117.9	5053.283		
		Q_u	16187.42	22163.289	Q_{all}	9500 KN
	F.S (2 for Q_p and 1.5 for Q_s)	Q_{all1}	8617.01	7724.648		
	F.S (3 for Q_p and 1.5 for Q_s)	Q_{all2}	6435.106			
	F.S (2.5for Q_p and Q_s)	Q_{all3}	6474.96			
2-	1.40 m	Q_p	17789.079	27246.727		
		Q_s	3637.51	6128.66		
		Q_u	21426.59	33375.395		
		Q_{all1}	11324.34	11533.709		
		Q_{all2}	8354.699			
		Q_{all3}	8570.636			
3-	1.60 m	Q_p	23234.72	38250.762		
		Q_s	4157.156	7039.301		
		Q_u	27391.876	45290.063		
		Q_{all1}	14394.85	15565.975		
		Q_{all2}	10516.344			
		Q_{all3}	10956.75			
4-	1.80 m	Q_p	29406.4	48411.109		
		Q_s	4676.80	7918.659		
		Q_u	34083.20	56329.805		
		Q_{all1}	17828.56	19304.514		
		Q_{all2}	11672.85			
		Q_{all3}	13633.28			

Table (4.3) Bearing capacity of vertical pile length 32.9

#	Diameter m	Manual results KN		All pile6.5 results KN
1-	1.20 m	Q_p	13069.52	17095.324
		Q_s	2520.32	4167.684
		Q_u	15589.84	21263.008
		Q_{all1}	8218.474	7365.515
		Q_{all2}	6036.72	
		Q_{all3}	6235.93	
2-	1.40 m	Q_p	17789.079	26291.209
		Q_s	2940.436	4928.36
		Q_u	20729.5	31219.57
		Q_{all1}	10859.38	10735.081
		Q_{all2}	7889.98	
		Q_{all3}	8291.806	
3-	1.60 m	Q_p	23234.72	34862.352
		Q_s	3360.49	4458.969
		Q_u	26595.2	39321.320
		Q_{all1}	13863.47	13404.371
		Q_{all2}	9985.23	
		Q_{all3}	10638.085	
4-	1.80 m	Q_p	29406.4	42905.840
		Q_s	3780.59	6125.547
		Q_u	33186.96	49121.387
		Q_{all1}	17230.76	16788.166
		Q_{all2}	12322.5	
		Q_{all3}	13274.78	

Table (4.4) Bearing capacity of vertical pile length 25.9

#		Manual results KN		All pile6.5 results KN
1-	1.20 m	Q_p	13069.5	14180.772
		Q_s	1370.617	2206.919
		Q_u	14440.137	16387.691
		Q_{all1}	7451.638	5609.692
		Q_{all2}	5270.25	
		Q_{all3}	5776.05	
2-	1.40 m	Q_p	17789.079	19301.685
		Q_s	1599.927	2574.651
		Q_u	19388.127	21879.219
		Q_{all1}	9964.738	7463.710
		Q_{all2}	6995.726	
		Q_{all3}	7755.25	
3-	1.60 m	Q_p	23234.72	25210.344
		Q_s	1827.502	2942.836
		Q_u	25062.22	28153.180
		Q_{all1}	12841.11	9580.582
		Q_{all2}	8963.24	
		Q_{all3}	10024.88	
4-	1.80 m	Q_p	29406.4	31906.834
		Q_s	2055.93	3310.408
		Q_u	31462.33	35217.242
		Q_{all1}	16080.6	11959.774
		Q_{all2}	11172.75	
		Q_{all3}	12584.9	

4.4 Analysis group of pile:

load combination of soba bridge which subjected from bridge to piers. is given in table (4.5). the LC01, LC05, LC11 and LC18 is load combination which used in analysis and design because is maximum load combination. Figure (4.4) shows the number and spacing between piles in different groups (1x4,2x4,3x4).

Table (4.5) Load combinations (A&A soba bridge project)

LC	Vertical load KN	Lateral load KN	Moment KN.m
LC01	43608	0	10427
LC02	40685	0	7127
LC03	43088	0	13734
LC04	40939	0	10019
LC05	41191	0	29735
LC06	39109	0	19388
LC07	40953	0	30954
LC08	39546	0	28624
LC09	42215	344	8690
LC10	39779	344	10986
LC11	41874	344	11621
LC12	40059	344	11217
LC13	40201	344	24779
LC14	38466	344	16157
LC15	40072	344	26145
LC16	38882	344	24188
LC17	27199	283	5838
LC18	36241	2089	21877
LC19	28189	2089	21877
LC20	27199	2089	16921
LC21	27199	850	12670
LC22	40201	520	33172
LC23	30800	0	18021
LC24	31159	0	21305
LC25	27919	1651	16980
LC26	30800	429	24957

Figure (4.3) Piers bridge

4.4.1 Pile group vertical analysis:

Calculating group efficiency for all diameters using equation (3.34) to determine how the pile group behave according to η_g the results from table (4.6) represent that all pile behave as individual pile in group pile.

Table (4.6) $\eta_g$values

Diameter	η_g group 1x4	$\eta_g$for group 2x4
1.8	1.9	1.27
1.6	2.18	1.40
1.4	2.44	1.58
1.2	2.81	1.8

The spacing between piles in the groups 6.25 that is distance of piers which supporting the bridge deck that for1x4 group for another group the spacing shown in figure (4.4). The spacing is more than 3D. using equation (3.38) to distribute the vertical load in each pile in the group.

4.4.2 Pile group lateral analysis: using equation (3.27) to determine the stiffness factor, according to table (4.7) find the lengths for all diameters greater than 5T then the pile describe as long pile.

Table (4.7) stiffness factor T for all diameters

Diameter m	T	5T
1.8	4.63	23
1.6	4.19	20.95
1.4	3.78	18.9
1.2	3.34	16.7

the equation from (3.39) distribute the lateral load, because pile is long using equation (3.33) to distribute the and moment in each pile in the group manually see tables from (4.10 to 4.64).

The limiting settlement criteria sometimes specified the net settlement should not be more than 25mm (Arrora,2004), The settlement should not be more than 25mm (Bowls, 1997). limits the lateral deflection at ground level to 25mm (Das,2007).

4.5 Calculate maximum moment and ultimate lateral load on pile.

4.5.1 Brom`s method:

Using the equation (4.1) and (4.2) to determine the yield moment for each diameter which is shown in table (4.7).

Table (4.8) shown ultimate lateral load which to calculated from equation (4.4) .

$$\text{My} = f_y * Z \qquad (4.1)\ \text{(Murthy, 1969)}$$

$$= f_y * \pi D^3/32 \qquad (4.2)$$

For concrete assume the yield strength = $f_y = f_{cu}$ = 30 N/mm^2

$$M_y = 30(\pi D^3/32) \quad \text{KN.m} \qquad (4.3)$$

Table (4.8) maximum moment in pile Brom`s method

Diameter m	M_y
1.8	17176.6
1.6	12063.7
1.4	8081.74
1.2	5089.38

Ultimate horizontal load from figure (3.10), table (4.9) shows ultimate l horizontal load values of for different diameters.

Table (4.9) ultimate lateral load in pile Brom`s method

Diameter (m)	H_u (KN)
1.8	11022.48
1.6	7962.6
1.4	5927.04
1.2	1555.2

4.5.2 Brinchhansen`s method using spread sheet:

Calculating ultimate lateral load from spread sheet using brinchhansen method the figures from (A15) to (A18) shown values of ultimate lateral load for different diameters and lengths the table (4.10) represent the ultimate lateral load in pile using brinchhansen method.

Table (4.10) ultimate lateral load and maximum moment brinchhansen method in pile

Diameter (m)	H_u (KN)
1.8	9711.5
1.6	9039.5
1.4	8356.3
1.2	7664.9

4.6The results:

The results of analysis the pile group of pile under four maximum load combinations (LC01, LC05, LC11, LC18), for diameter (1.2, 1.4, 1.6 and 1.8 m) and three different length (25.9, 32.9 and 35.9 m) by manual calculation ,Allpile6.5 software analysis and results report from A&A .

tables from (4.11 to 4.18) for L = 39.9m for 1x4 group pile results.

tables from (4.19 to 4.26) for L = 39.9m for 2x4 group pile results

tables from (4.27 to 4.34) for L = 32.9m for 1x4 group pile results.

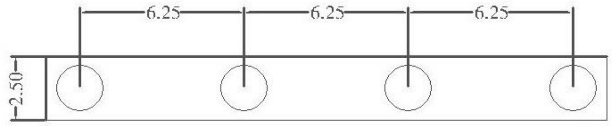
tables from (4.35 to 4.42) for L = 32.9m for 2x4 group pile results.

tables from (4.43 to 4.50) for L = 25.9m for 1x4 group pile results.

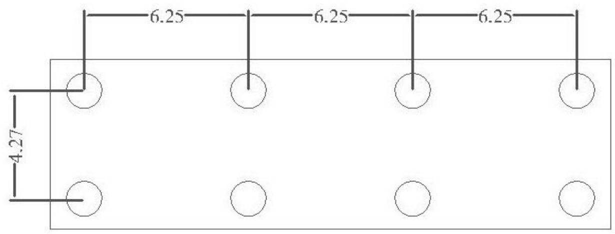
tables from (4.51 to 4.54) for L = 25.9m for 1x4 group pile results (report from A&A).

tables from (4.55 to 4.62) for l = 29.9m for 2x4 group pile results

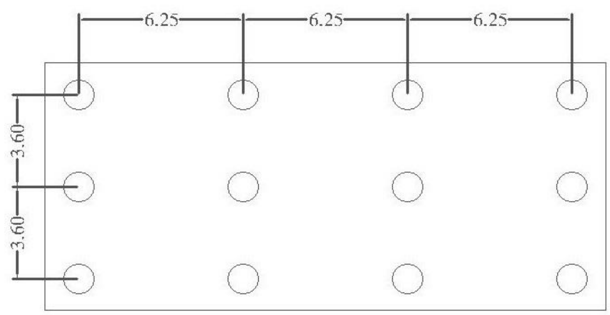
tables from (4.63 to 4.70) for l = 29.9m for 3x4 group pile results



1x4 pile group



2x4 pile group



3x4 pile group

FIGHURE (   ) GROUP OF PILE

Figure (4.4) shows different number and spacing for pile groups

Table (4.11) Load Combination (LC01) analysis results for 6.25 spacing L= 35.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	10900.96	3.37301	BACK	0.0	2.56	0.00330	10735.17	0.0	0.0	Exceeds the Allowable Capacity (Down) 7724.65-kN
	11402.50	3.65470	FRONT	0.0	2.45	0.00291	11402.5	0.0	0.0	Exceeds the Allowable Capacity (Down) 7724.65-kN
1.40	10900.96	2.17246	BACK	0.0	2.76	0.00238	10735.17	0.0	0.0	
	11402.50	2.17314	FRONT	0.0	2.59	0.00197	11402.5	0.0	0.0	
1.60	10900.96	1.62483	BACK	0.0	2.98	0.00181	10735.17	0.0	0.0	
	11402.50	1.73842	FRONT	0.0	2.76	0.00143	11402.5	0.0	0.0	
1.80	10900.96	1.33159	BACK	0.0	3.22	0.00145	10735.17	0.0	0.0	
	11402.50	1.41779	FRONT	0.0	2.92	0.00109	11402.5	0.0	0.0	

Table (4.12) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	30348.26	3.37358
1.40	45452.73	2.17280
1.60	61477.11	1.62506
1.80	76333.38	1.33176

Table (4.13) Load Combination (LC05) analysis results for 6.25 spacing L= 35.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	10294.78	3.04738	BACK	0.0	2.55	0.00328	9821.99	0.0	0.0	Exceeds the Allowable Capacity (Down)= 7724.65-kN
	11725.03	3.83967	FRONT	0.0	2.46	0.00291	11725.03	0.0	0.0	Exceeds the Allowable Capacity (Down)= 7724.65-kN
1.40	10294.78	1.97778	BACK	0.0	2.75	0.00237	9821.99	0.0	0.0	
	11725.03	2.44840	FRONT	0.0	2.60	0.00197	11725.03	0.0	0.0	
1.60	10294.78	1.49444	BACK	0.0	2.98	0.00181	9821.99	0.0	0.0	
	11725.03	1.81350	FRONT	0.0	2.76	0.00143	11725.03	0.0	0.0	
1.80	10294.78	1.23311	BACK	0.0	3.21	0.00145	9821.99	0.0	0.0	
	11725.03	1.47383	FRONT	0.0	2.93	0.00109	11725.03	0.0		

Table (4.14) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	30348.26	3.04893
1.40	45452.73	1.97872
1.60	61477.11	1.49507
1.80	76333.38	1.23359

Table (4.15) Load Combination (LC11) analysis results for 6.25 spacing L= 35.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	10467.34	3.13830	BACK	73.10	466.00	0.60100	10282.56	86	512.52	Exceeds the Allowable Capacity (Down) 7724.65-kN
	11026.31	3.44254	FRONT	98.90	606.00	0.72000	11026.31	86	512.52	Exceeds the Allowable Capacity (Down) 7724.65-kN
1.40	10467.34	2.33839	BACK	73.10	504.00	0.43300	10282.56	86	546.92	
	11026.31	2.53139	FRONT	98.90	640.00	0.48500	11026.31	86	546.92	
1.60	10467.34	1.74940	BACK	73.10	544.00	0.33000	10282.56	86	578.01	
	11026.31	1.88621	FRONT	98.90	681.00	0.35300	11026.31	86	578.01	
1.80	10467.34	1.26114	BACK	73.10	588.00	0.26400	10282.56	86	612.21	
	11026.31	1.35244	FRONT	98.90	722.00	0.26900	11026.31	86	612.21	

Table (4.16) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	30348.26	3.13891
1.40	43692.50	2.33878
1.60	61483.46	1.5316
1.80	76333.38	1.26133

Table (4.17) Load Combination (LC18) analysis results for 6.25 spacing L= 35.9 m:
Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	9058.06	2.43169	BACK	443.91	3150.00	4.96000	8710.218	522.25	3112.39	Exceeds the Allowable Capacity (Down) 7724.65-kN
	10110.35	2.95176	FRONT	600.59	4420.00	7.30000	10110.35	522.25	3112.39	Exceeds the Allowable Capacity (Down) 7724.65-kN
1.40	9058.06	1.61057	BACK	443.91	3110.00	2.78000	8710.218	522.25	3321.31	
	10110.35	1.92008	FRONT	600.59	4360.00	4.08000	10110.35	522.25	3321.31	
1.60	9058.06	1.24764	BACK	443.91	3290.00	2.00000	8710.218	522.26	3510.12	
	10110.35	1.45538	FRONT	600.59	4340.00	2.46000	10110.35	522.26	3510.12	
1.80	9058.06	1.04333	BACK	443.91	3560.00	1.61000	8710.218	522.25	3717.76	
	10110.35	1.20442	FRONT	600.59	4400.00	1.64000	10110.35	522.25	3717.76	

Table (4.18) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	30348.26	2.43273
1.40	45452.73	1.61117
1.60	61477.11	1.24806
1.80	76245.63	1.04366

Table (4.19) Load Combination (LC01) analysis results for 6.25 spacing L= 35.9 m:
Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	5200.75	0.97413	BACK Y	0.0	2.53	0.00340	6096.593	0.0	0.0	
	6175.10	1.25408	FRONT Y	0.0	2.29	0.00250	6253.601	0.0	0.0	
	5200.75	0.97413	BACK X	0.0	2.37	0.00279	6148.929	0.0	0.0	
	5701.25	1.11132	FRONT X	0.0	2.29	0.00249	6201.265	0.0	0.0	
1.40	5200.75	0.74900	BACK Y	0.0	2.29	0.00223	6097.823	0.0	0.0	
	6071.65	0.91121	FRONT Y	0.0	2.40	0.00161	6150.159	0.0	0.0	
	5200.75	0.74900	BACK X	0.0	2.54	0.00193	5993.151	0.0	0.0	
	5701.25	0.84109	FRONT X	0.0	2.40	0.00161	6045.487	0.0	0.0	

Table (4.20) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	59617.78	1.04030
1.40	89616.09	0.79371

Table (4.21) Load Combination (LC05) analysis results for 6.25 spacing L= 35.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	4435.23	0.78672	BACK Y	0.0	2.51	0.00339	7139.67	0.0	0.0	
	7213.81	1.62018	FRONT Y	0.0	2.30	0.00251	7287.942	0.0	0.0	
	4435.23	0.78672	BACK X	0.0	2.36	0.00278	7189.09	0.0	0.0	
	5862.52	1.15715	FRONT X	0.0	2.29	0.00249	7238.518	0.0	0.0	
1.40	4435.23	0.61721	BACK Y	0.0	2.66	0.00222	6058.557	0.0	0.0	
	6918.82	1.08411	FRONT Y	0.0	2.41	0.00162	6107.98	0.0	0.0	
	4435.23	0.61721	BACK X	0.0	2.54	0.00192	5959.709	0.0	0.0	
	5862.52	0.87162	FRONT X	0.0	2.40	0.00161	6009.133	0.0	0.0	

Table (4.22) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	59617.78	0.96075
1.40	89616.09	0.74007

Table (4.23) Load Combination (LC11) analysis results for 6.25 spacing L= 35.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment
1.20	4955.35	0.91082	BACK Y	36.55	230.00	0.31100	6103.24	43	256.26
	6041.26	1.21258	FRONT Y	49.45	283.00	0.30800	6227.192	43	256.26
	4955.35	0.91082	BACK X	36.55	216.00	0.25500	5855.336	43	256.26
	5513.15	1.05794	FRONT X	49.45	282.00	0.30700	5979.288	43	256.26
1.40	4955.35	0.80699	BACK Y	36.55	243.00	0.20300	5987.952	43	273.46
	5925.98	1.01566	FRONT Y	49.45	297.00	0.19900	6111.904	43	273.46
	4955.35	0.80699	BACK X	36.55	232.00	0.17600	5740.048	43	273.46
	5513.15	0.92412	FRONT X	49.45	297.00	0.19900	5864	43	273.46

Table (4.24) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	59617.78	0.98278
1.40	86351.53	0.86397

Table (4.25) Load Combination (LC18) analysis results for 6.25 spacing L= 35.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	4005.08	0.69100	BACK Y	221.96	1410.00	1.94000	6114.593	261.125	1556.20	
	6049.36	1.21509	FRONT Y	300.29	1990.00	2.88000	6027.617	261.125	1556.20	Deflection Exceed the maximum value 2.5 cm
	4005.08	0.69100	BACK X	221.96	1400.00	1.91000	6071.105	261.125	1556.20	
	5055.17	0.93657	FRONT X	300.29	1980.00	2.86000	5984.129	261.125	1556.20	Deflection Exceed the maximum value 2.5 cm
1.40	4005.08	0.54876	BACK Y	221.96	1470.00	1.23000	5202.97	261.125	1660.65	
	5832.33	0.86590	FRONT Y	300.29	1980.00	1.59000	5246.458	261.125	1660.65	
	4005.08	0.54876	BACK X	221.96	1410.00	1.08000	5115.994	261.125	1660.65	
	5055.17	0.72394	FRONT X	300.29	1970.00	1.59000	5159.482	261.125	1660.65	

Table (4.26) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	59617.78	0.80886
1.40	89616.09	0.63355

Table (4.27) Load Combination (LC01) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	10900.96	3.61801	BACK	0.0	2.56	0.00330	10735.17	0.0	0.0	Exceeds the Allowable Capacity (Down) 7724.65-kN
	11402.50	3.89701	FRONT	0.0	2.45	0.00291	11402.5	0.0	0.0	Exceeds the Allowable Capacity (Down) 7724.65-kN
1.40	10900.96	2.41593	BACK	0.0	2.76	0.00238	10735.17	0.0	0.0	
	11402.50	2.58921	FRONT	0.0	2.59	0.00197	11402.5	0.0	0.0	
1.60	10900.96	1.86576	BACK	0.0	2.98	0.00181	10735.17	0.0	0.0	
	11402.50	1.99794	FRONT	0.0	2.76	0.00143	11402.5	0.0	0.0	
1.80	10900.96	1.54305	BACK	0.0	3.22	0.00144	10735.17	0.0	0.0	
	11402.50	1.64570	FRONT	0.0	2.92	0.00109	11402.5	0.0	0.0	

Table (4.28) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	28997.26	3.61859
1.40	42381.86	2.41628
1.60	54163.41	1.86604
1.80	66451.70	1.54326

Table (4.29) Load Combination (LC05) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group 1x4:

Día (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	10294.78	3.31624	BACK	0.0	2.55	0.00329	9821.99	0.0	0.0	Exceeds the Allowable Capacity (Down) 7724.65-kN
	11725.03	4.10354	FRONT	0.0	2.46	0.00292	11725.03	0.0	0.0	Exceeds the Allowable Capacity (Down) 7724.65-kN
1.40	10294.78	2.21020	BACK	0.0	2.75	0.00237	9821.99	0.0	0.0	
	11725.03	2.70226	FRONT	0.0	2.60	0.00197	11725.03	0.0	0.0	
1.60	10294.78	1.71088	BACK	0.0	2.98	0.00181	9821.99	0.0	0.0	
	11725.03	2.08470	FRONT	0.0	2.76	0.00143	11725.03	0.0	0.0	
1.80	10294.78	1.39884	BACK	0.0	3.21	0.00144	9821.99	0.0	0.0	
	11725.03	1.68430	FRONT	0.0	2.93	0.00109	11725.03	0.0	0.0	

Table (4.30) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	28836.96	3.31783
1.40	42381.86	2.21119
1.60	54163.41	1.71163
1.80	67372.82	1.39941

Table (4.31) Load Combination (LC11) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	10467.34	3.38001	BACK	73.10	466.00	0.60100	10282.56	86	512.52	Exceeds the Allowable Capacity (Down) 7365.515KN
	11026.31	3.68750	FRONT	98.90	606.00	0.72000	11026.31	86	512.52	Exceeds the Allowable Capacity (Down) 7365.515KN
1.40	10467.34	2.26830	BACK	73.10	504.00	0.43300	10282.56	86	546.92	
	11026.31	2.45891	FRONT	98.90	640.00	0.48500	11026.31	86	546.92	
1.60	10467.34	1.75471	BACK	73.10	544.00	0.33000	10282.56	86	578.01	
	11026.31	1.89862	FRONT	98.90	681.00	0.35300	11026.31	86	578.01	
1.80	10467.34	1.43197	BACK	73.10	587.00	0.26400	10282.56	86	612.21	
	11026.31	1.54154	FRONT	98.90	723.00	0.26800	11026.31	86	612.21	

Table (4.32) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	28997.26	3.38064
1.40	42381.86	2.26869
1.60	54163.41	1.75500
1.80	67372.82	1.43219

Table (4.33) Load Combination (LC18) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group:1x4

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	9058.06	2.43169	BACK	443.91	3150.00	4.96000	8710.218	522.25	3112.39	Exceeds the Allowable Capacity (Down) 7365.515KN
	10110.35	2.95176	FRONT	600.59	4420.00	7.30000	10110.35	522.25	3112.39	Exceeds the Allowable Capacity (Down) 7365.515KN
1.40	9058.06	1.80836	BACK	443.91	3110.00	2.78000	8710.218	522.25	3321.31	
	10110.35	2.14862	FRONT	600.59	4360.00	4.07000	10110.35	522.25	3321.31	
1.60	9058.06	1.41571	BACK	443.91	3290.00	2.00000	8710.218	522.26	3510.12	
	10110.35	1.66543	FRONT	600.59	4340.00	2.46000	10110.35	522.26	3510.12	
1.80	9058.06	1.17295	BACK	443.91	3560.00	1.60000	8710.218	522.25	3717.76	
	10110.3	1.36343	FRONT	600.59	4390.00	1.63000	10110.35	522.25	3717.76	

Table (4.34) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	30348.26	2.43273
1.40	42381.86	1.80904
1.60	54163.41	1.41620
1.80	67372.82	1.17332

Table (4.35) Load Combination (LC01) analysis results for 6.25 spacing L=32.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment
1.20	5200.75	1.07133	BACK Y	0.0	2.53	0.00340	6096.593	0.0	0.0
	6175.10	1.39865	FRONT Y	0.0	2.29	0.00250	6253.601	0.0	0.0
	5200.75	1.07133	BACK X		2.37	0.00279	6148.929	0.0	0.0
	5701.25	1.23161	FRONT X		2.29	0.00249	6201.265	0.0	0.0
1.40	5200.75	0.81645	BACK Y	0.0	2.67	0.00223	6097.823	0.0	0.0
	6071.65	1.00297	FRONT Y	0.0	2.40	0.00161	6150.159	0.0	0.0
	5200.75	0.81645	BACK X		2.54	0.00193	5993.151	0.0	0.0
	5701.25	0.92172	FRONT X		2.40	0.00161	6045.487	0.0	0.0

Table (4.36) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	57112.86	1.14860
1.40	83734.42	0.86889

Table (4.37) Load Combination (LC05) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment
1.20	4435.23	0.85501	BACK Y	0.0	2.51	0.00339	7139.67	0.0	0.0
	7213.81	1.81424	FRONT Y	0.0	2.30	0.00251	7287.94	0.0	0.0
	4435.23	0.85501	BACK X	0.0	2.36	0.00278	7189.09	0.0	0.0
	5862.52	1.28667	FRONT X	0.0	2.29	0.00249	7238.518	0.0	0.0
1.40	4435.23	0.66843	BACK Y	0.0	2.66	0.00222	6058.557	0.0	0.0
	6918.82	1.20364	FRONT Y	0.0	2.41	0.00162	6107.98	0.0	0.0
	4435.23	0.66843	BACK X		2.54	0.00192	5959.709	0.0	0.0
	5862.52	0.95576	FRONT X		2.40	0.00161	6009.133	0.0	0.0

Table (4.38) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	57112.86	1.05535
1.40	83734.42	0.86889

Table (4.39) Load Combination (LC11) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment
1.20	4955.35	0.99816	BACK Y	36.55	230.00	0.31000	6103.24	43	256.26
	6041.26	1.34999	FRONT Y	49.45	283.00	0.30800	6227.192	43	256.26
	4955.35	0.99816	BACK X	36.55	216.00	0.25500	5855.336	43	256.26
	5513.15	1.16922	FRONT X	49.45	282.00	0.30700	5979.288	43	256.26
1.40	4955.35	0.76884	BACK Y	36.55	243.00	0.20300	5987.952	43	273.46
	5925.98	0.96969	FRONT Y	49.45	297.00	0.19900	6111.904	43	273.46
	4955.35	0.76884	BACK X	36.55	232.00	0.17600	5740.048	43	273.46
	5513.15	0.88201	FRONT X	49.45	297.00	0.19900	5864	43	273.46

Table (4.40) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	57112.86	1.08165
1.40	83734.42	0.82313

Table (4.41) Load Combination (LC18) analysis results for 6.25 spacing L= 32.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Moment	Note
1.20	4005.08	0.74607	BACK Y	221.96	1410.00	1.94000	6114.593	261.125	1556.20	
	6049.36	1.35286	FRONT Y	300.29	1990.00	2.88000	6027.617	261.125	1556.20	Deflection Exceed the maximum value 2.5 cm
	4005.08	0.74607	BACK X	221.96	1410.00	1.91000	6071.105	261.125	1556.20	
	5055.17	1.02649	FRONT X	300.29	1980.00	2.86000	5984.129	261.125	1556.20	Deflection Exceed the maximum value 2.5 cm
1.40	4005.08	0.59209	BACK Y	221.96	1470.00	1.23000	5202.97	261.125	1660.65	
	5832.33	1.00297	FRONT Y	300.29	1980.00	1.59000	5246.458	261.125	1660.65	
	4005.08	0.59209	BACK X	221.96	1410.00	1.08000	5115.994	261.125	1660.65	
	5055.17	0.78820	FRONT X	300.29	1970.00	1.58000	5159.482	261.125	1660.65	

Table (4.42) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	57112.86	0.87965
1.40	83734.42	0.68635

Table (4.43) Load Combination (LC01) analysis results analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	10900.96	4.88849	BACK Y	0.0	2.56	0.00330	10735.17	0.0	0.0	Exceeds the Allowable Capacity (Down) 5609.692 KN
	11402.50	5.27992	FRONT Y	0.0	2.45	0.00291	11402.5	0.0	0.0	Exceeds the Allowable Capacity (Down) 5609.692 KN
1.40	10900.96	3.57098	BACK Y	0.0	2.76	0.00238	10735.17	0.0	0.0	Exceeds the Allowable Capacity (Down 7463.710KN
	11402.50	3.81540	FRONT Y	0.0	2.59	0.00197	11402.5	0.0	0.0	Exceeds the Allowable Capacity (Down 7463.710KN
1.60	10900.96	2.79620	BACKY	0.0	2.98	0.00181	10735.17	0.0	0.0	Exceeds the Allowable Capacity (Down 9580.582 KN
	11402.50	2.97759	FRONT Y	0.0	2.76	0.00143	11402.5	0.0	0.0	Exceeds the Allowable Capacity (Down 9580.582 KN
1.80	10900.96	2.29916	BACK Y	0.0	3.22	0.00145	10735.17	0.0	0.0	
	11402.50	2.44576	FRONY	0.0	2.93	0.00109	11402.5	0.0	0.0	

Table (4.44) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	22183.28	4.45400
1.40	29555.26	3.57148
1.60	37982.17	2.79658
1.80	46610.07	1.54326

Table (4.45) Load Combination (LC05) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	10294.78	4.45195	BACK	0.0	2.55	0.00328	9821.99	0.0	0.0	Exceeds the Allowable Capacity (Down) 5609.692
	11725.03	5.54872	FRONT	0.0	2.46	0.00291	11725.03	0.0	0.0	Exceeds the Allowable Capacity (Down) 5609.69
1.40	10294.78	3.27886	BACK	0.0	2.76	0.00237	9821.99	0.0	0.0	Exceeds the Allowable Capacity (Down 7463.710
	10300.72	3.28167	FRONT	0.0	2.44	0.00164	11725.03	0.0	0.0	Exceeds the Allowable Capacity (Down 7463.710
1.60	10294.78	2.58197	BACK	0.0	2.98	0.00181	9821.99	0.0	0.0	Exceeds the Allowable Capacity (Down 9580.582
	11725.03	3.09679	FRONT	0.0	2.76	0.00143	11725.03	0.0	0.0	Exceeds the Allowable Capacity (Down 9580.582
1.80	10294.78	2.12462	BACK	0.0	3.22	0.00145	9821.99	0.0	0.0	
	11725.03	2.54092	FRONT	0.0	2.93	0.00109	11725.03	0.0	0.0	

Table (4.46) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	22183.28	4.45400
1.40	29555.26	3.28027
1.60	37982.17	2.58301
1.80	47457.87	2.12547

Table (4.47) Load Combination (LC11) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection Cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	10467.34	4.57260	BACK	73.10	466.00	0.60100	10282.56	86	512.52	Exceeds the Allowable Capacity (Down) 5609.692 KN
	11026.31	4.98361	FRONT	98.90	606.00	0.72000	11026.31	86	512.52	Exceeds the Allowable Capacity (Down) 5609.692 KN
1.40	10467.34	3.36125	BACK	73.10	503.00	0.26100	10282.56	86	546.92	Exceeds the Allowable Capacity (Down 7463.710 KN
	11026.31	3.63202	FRONT	98.90	640.00	0.48600	11026.31	86	546.92	Exceeds the Allowable Capacity (Down 7463.710KN
1.60	10467.34	2.64228	BACK	73.10	545.00	0.33100	10282.56	86	578.01	Exceeds the Allowable Capacity (Down 9580.582 KN
	11026.31	2.84111	FRONT	98.90	681.00	0.35400	11026.31	86	578.01	Exceeds the Allowable Capacity (Down 9580.582 KN
1.80	10467.34	2.17397	BACK	73.10	588.00	0.26500	10282.56	86	612.21	
	11026.31	2.33567	FRONT	98.90	723.00	0.26900	11026.31	86	612.21	

Table (4.48) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	22183.28	4.57343
1.40	29555.26	3.36181
1.60	37982.17	2.64269
1.80	47457.87	2.17430

Table (4.49) Load Combination (LC18) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	
1.20	9058.06	3.65487	BACK	443.91	3150.00	4.94000	8710.218	522.25	3112.39	Exceeds the Allowable Capacity (Down) 5609.692 KN
	10110.35	4.32578	FRONT	600.59	4430.00	7.31000	10110.35	522.25	3112.39	Exceeds the Allowable Capacity (Down) 5609.692 KN
1.40	9058.06	2.71868	BACK	443.91	3120.00	2.79000	8710.218	522.25	3321.31	Exceeds the Allowable Capacity (Down 7463.710
	10110.35	3.19182	FRONT	600.59	4360.00	4.09000	10110.35	522.25	3321.31	Exceeds the Allowable Capacity (Down 7463.710
1.60	9058.06	2.15880	BACK	443.91	3300.00	2.00000	8710.218	522.26	3510.12	Exceeds the Allowable Capacity (Down 9580.582
	10110.35	2.51768	FRONT	600.59	4340.00	2.47000	10110.35	522.26	3510.12	Exceeds the Allowable Capacity (Down 9580.582
1.80	9058.06	1.77859	BACK	443.91	3560.00	1.60000	8710.218	522.25	3717.76	
	10110.35	2.07212	FRONT	600.59	4400.00	1.64000	10110.35	522.25	3717.76	

Table (4.50) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.20	22183.28	3.65620
1.40	29555.26	2.71963
1.60	37982.17	2.15954
1.80	47457.87	1.77918

Table (4.51) Load Combination (LC01) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4 the actual result from pile group software:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN	Max Moment KN.M	Top Deflection cm
1.80	10404	0.55531	0.244	808.43	2.27
	11400	0.63263	0.244	824.18	2.28

Table (4.52) Load Combination (LC05) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4 the actual result from pile group software:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN	Max Moment KN.M	Top Deflection cm
1.80	8860.8	0.43989	38.612	750.30	2.106
	11706	0.65642	39.082	791.63	9.32

Table (4.53) Load Combination (LC11) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4 the actual result from pile group software:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN	Max Moment KN.M	Top Deflection cm
1.80	9913.5	0.51721	84.071	1894.6	4.48
	11024	0.60341	87.926	1965.5	9.327

Table (4.54) Load Combination (LC18) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 1x4 the actual result from pile group software:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN	Max Moment KN.M	Top Deflection cm
1.80	6928.6	0.32646	512.81	5063.7	2.136
	11001	0.60168	535.24	5226.9	4.73

Table (4.55) Load Combination (LC01) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	5200.75	1.59146	BACK Y	0.0	2.53	0.00341	6096.593	0.0	0.0	Exceeds the Allowable Capacity (Down 5609.692KN
	6175.10	2.05773	FRONT Y	0.0	2.29	0.00249	6253.601	0.0	0.0	Exceeds the Allowable Capacity (Down 5609.692KN
	5200.75	1.59146	BACK X	0.0	2.37	0.00279	6148.929	0.0	0.0	
	5701.25	1.82745	FRONT X	0.0	2.29	0.00249	6201.265	0.0	0.0	
1.40	5200.75	1.21501	BACK Y	0.0	2.67	0.00223	6097.823	0.0	0.0	
	6071.65	1.52408	FRONT Y	0.0	2.40	0.00161	6150.159	0.0	0.0	
	5200.75	1.21501	BACK X	0.0	2.54	0.00193	5993.151	0.0	0.0	
	5701.25	1.38963	FRONT X	0.0	2.40	0.00161	6045.487	0.0	0.0	
1.60	5200.75	0.98978	BACK Y	0.0	2.81	0.00156	5751.155	0.0	0.0	
	5994.07	1.20225	FRONT Y	0.0	2.52	0.00112	5855.827	0.0	0.0	
	5200.75	0.98978	BACK X	0.0	2.72	0.00141	5698.819	0.0	0.0	
	5701.25	1.12206	FRONT X	0.0	2.52	0.00112	5803.491	0.0	0.0	

Table (4.56) Group Pile Vertical Analysis (in Group):

Dia (M)	Total Allowable Capacity (KN)	Settlement cm
1.2	43906.46	1.70856
1.40	58579.05	1.30119
1.60	75938.45	1.05523

Table (4.57) Load Combination (LC05) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	4435.23	1.24924	BACK Y	0.0	2.51	0.00339	7139.67	0.0	0.0	
	7213.81	2.59101	FRONT Y	0.0	2.30	0.00251	7287.942	0.0	0.0	Exceeds the Allowable Capacity (Down 5609.692KN
	4435.23	1.24924	BACK X	0.0	2.36	0.00278	7189.09	0.0	0.0	
	5862.52	1.90511	FRONT X	0.0	2.29	0.00249	7238.518	0.0	0.0	Exceeds the Allowable Capacity (Down 5609.692KN
1.40	4435.23	0.96923	BACK Y	0.0	2.66	0.00222	6058.557	0.0	0.0	
	6918.82	1.84569	FRONT Y	0.0	2.41	0.00162	6107.98	0.0	0.0	
	4435.23	0.96923	BACK X	0.0	2.54	0.00192	5959.709	0.0	0.0	
	5862.52	1.44771	FRONT X	0.0	2.40	0.00161	6009.133	0.0	0.0	
1.60	4435.23	0.80262	BACK Y	0.0	2.81	0.00143	5849.088	0.0	0.0	
	6697.57	1.40562	FRONT Y	0.0	2.53	0.00112	5997.36	0.0	0.0	
	4435.23	0.80262	BACK X	0.0	2.71	0.00141	5898.512	0.0	0.0	
	5862.52	1.16623	FRONT X	0.0	2.52	0.00112	5947.936	0.0	0.0	

Table (4.58) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	43906.46	1.56733
1.40	58579.05	1.19775
1.60	75357.70	0.97624

Table (4.59) Load Combination (LC11) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	4955.35	1.47866	BACK Y	36.55	230.00	0.31100	6103.24	43	256.26	
	6041.26	1.99201	FRONT Y	49.45	283.00	0.30800	6227.192	43	256.26	Exceeds the Allowable Capacity (Down 5609.692KN
	4955.35	1.47866	BACK X	36.55	230.00	0.31100	5855.336	43	256.26	
	6041.26	1.99201	FRONT X	49.45	283.00	0.30800	5979.288	43	256.26	Exceeds the Allowable Capacity (Down 5609.692KN
1.40	4955.35	1.13351	BACK Y	36.55	243.00	0.20400	5987.952	43	273.46	
	5925.98	1.47084	FRONT Y	49.45	297.00	0.19900	6111.904	43	273.46	
	4955.35	1.13351	BACK X	36.55	232.00	0.17600	5740.048	43	273.46	
	5513.15	1.32282	FRONT X	49.45	297.00	0.19900	5864	43	273.46	
1.60	4955.35	0.92820	BACK Y	36.55	257.00	0.14200	5653.582	43	289.00	
	5839.51	1.15993	FRONT Y	49.45	312.00	0.13800	6025.438	43	289.00	
	4955.35	0.92820	BACK X	36.55	248.00	0.12900	5777.534	43	289.00	
	5513.15	1.07148	FRONT X	49.45	312.00	0.13800	5901.486	43	289.00	

Table (4.60) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	43906.46	1.60704
1.40	58579.05	1.22655
1.60	75357.70	0.99854

Table (4.61) Load Combination (LC18) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 2x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment	Note
1.20	4005.08	1.07168	BACK Y	221.96	1410.00	1.95000	6114.593	261.125	1556.20	
	6049.36	1.99595	FRONT Y	300.29	1990.00	2.89000	6027.617	261.125	1556.20	Exceeds the Allowable Capacity (Down 5609.692KN
	4005.08	1.07168	BACK X	221.96	1410.00	1.91000	6071.105	261.125	1556.20	
	5055.17	1.52439	FRONT X	300.29	1980.00	2.86000	5984.129	261.125	1556.20	
1.40	4005.08	0.84268	BACK Y	221.96	1470.00	1.23000	5202.97	261.125	1660.65	
	5832.33	1.43670	FRONT Y	300.29	1980.00	1.60000	5246.458	261.125	1660.65	
	4005.08	0.84268	BACK X	221.96	1410.00	1.08000	5115.994	261.125	1660.65	
	5055.17	1.16665	FRONT X	300.29	1970.00	1.59000	5159.482	261.125	1660.65	
1.60	4005.08	0.70455	BACK Y	221.96	1560.00	0.86300	5078.095	261.125	1755.06	
	5669.55	1.11339	FRONT Y	300.29	1970.00	0.94700	5165.071	261.125	1755.06	
	4005.08	0.70455	BACK X	221.96	1500.00	0.77900	5034.607	261.125	1755.06	
	5055.17	0.95298	FRONT X	300.29	1970.00	0.94600	5121.583	261.125	1755.06	

Table (4.62) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	43906.46	1.60704
1.40	58579.05	0.99793
1.60	75357.70	0.82484

Table (4.63) Load Combination (LC01) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 3x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment
1.20	3467.17	0.86888	BACK Y	0.0	2.50	0.00336	3943.713	0.0	0.0
	3996.05	1.06809	FRONT Y	0.0	2.27	0.00246	4048.385	0.0	0.0
	3467.17	0.86888	BACK X	0.0	2.35	0.00276	3686.336	0.0	0.0
	3800.83	0.99200	FRONT X	0.0	2.26	0.00246	3780.134	0.0	0.0

Table (4.64) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	65754.63	0.92880

Table (4.65) Load Combination (LC05) analysis results for 6.25 spacing L= 25.9 M:

Single Pile in Group 3x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment
1.20	2956.82	0.69749	BACK Y	0.0	2.49	0.00335	3915.867	0.0	0.0
	4465.05	1.26199	FRONT Y	0.0	2.27	0.00247	4514.473	0.0	0.0
	2956.82	0.69749	BACK X	0.0	2.34	0.00275	3482.007	0.0	0.0
	3908.34	1.03324	FRONT X	0.0	2.26	0.00246	4415.625	0.0	0.0

Table (4.66) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	65754.63	0.85646

Table (4.67) Load Combination (LC11) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 3x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment
1.20	3303.56	0.81163	BACK Y	24.37	152.00	0.20500	3365.548	28.66	170.80
	3893.01	1.02715	FRONT Y	32.97	187.00	0.20300	4016.959	28.66	170.80
	3303.56	0.81163	BACK X	24.37	143.00	0.16800	3613.452	28.66	170.80
	3675.44	0.94446	FRONT X	32.97	186.00	0.20300	3769.055	28.66	170.80

Table (4.68) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	65754.63	0.87690

Table (4.69) Load Combination (LC18) analysis results for 6.25 spacing L= 25.9 m:

Single Pile in Group 3x4:

Dia (M)	Vertical Load (KN)	Settlement cm	Lateral Load KN		Max Moment KN.M	Top Deflection cm	Manual Vertical Load	Manual Lateral Load	Manual Max Moment
1.20	2670.05	0.60955	BACK Y	147.97	920.00	1.24000	3063.571	174.08	1037.44
	3779.70	0.98399	FRONT Y	300.29	1250.00	1.66000	3823.189	174.08	1037.44
	2670.05	0.60955	BACK X	147.97	888.00	1.10000	3370.9	174.08	1037.44
	3370.12	0.83412	FRONT X	200.20	1250.00	1.65000	3428.884	174.08	1037.44

Table (4.70) Group Pile Vertical Analysis (in Group):

DIA(M)	Total Allowable Capacity (KN)	Settlement cm
1.2	65754.63	0.71746

4.7 Design of bored pile:

For design the pile using BS8110 code for LC18 load combination using chart for circular column in figure (4.5) and calculations in Table (4.71) to determine the area of steel and shear check.

The pile group designed 1x4 length 35.9 and diameter 1.8 to support all the load and the pile settlement and deflection in the design range.

Table (4.71) Calculation design for diameter 1.8 in 1x4 pile group.

<table>
<tr><td colspan="3">4.71 design calculations:</td></tr>
<tr><td colspan="3">Diameter of main reinforcement(Φ)=30mm
Diameter of link reinforcement=16mm
Cover=70mm
Compressive strength of concrete=30N/mm²
Yield strength of reinforcement =460 N/mm²
Maximum size of aggregate=20mm
Exposure condition=Mild
Bs 8110-1-1997
Load combination 18 (LC18)
N=1011.35KN, M=4400KN.M, Shear force = 600.59</td></tr>
<tr><td>3.8.1.6.1</td><td>1\calculate effective length:
Le = 0.85 lo
Le = 0.85 * 35.9 = 30.515 m
2\slenderness ratio:
λ= le/h=30.515/1.8=16.95
The pile is slender unbraced.
2\calculate deflection</td><td>BS 8110
equation 31

British Cement Association ,1989

BS 8110</td></tr>
</table>

3.8.1.8 Table 3.19 3.8.3.1	a u = βa h K assume k=1 $= \frac{\mathbf{1}}{\mathbf{2000}}(\frac{\boldsymbol{le}}{\boldsymbol{h}})^{\mathbf{2}} * h * k = \frac{1}{2000}(\frac{30.515}{1.8})^2 * 1.8 * 1$=0.258 3/calculate M add: M add=a u N=10110.35X0.258=2615. 1KN.m M=Mi +M add =4400+2615.1=7015. 11KN.m N/h2= $\frac{\mathbf{10110.35*1000}}{\mathbf{1800^2}}$ = 3.12 M/h3= $\frac{\mathbf{7015.11*10^6}}{\mathbf{1800^3}}$ =1.20 From chart in figure (4.5) 100As/Ac= 0.40 As =10178.76 mm2 15 T 30 mm Shear check: At the face support: Lateral load = 600.59 KN *For diameter 1.8 m* υ = 0.236 N/mm2 < $0.8\sqrt{fcu}$ = 0.8√30 =4.38 = 4.38N/ OK $υ_c$ =0.97 N/ mm2 from table (Table 3.8) υ = 0.236 N/mm2 < $υ_c$ =0.97 N/ mm2 OK Nominal link reinforcement A_{sv}/S_v = 0.4b / 0.87fy = $\frac{0.4x1800}{0.87\,x460}$ =1.799(T16@220mm C/C) A_{sv}/S_v =1.82 OK	equation 32 BS 8110 equation 32 British Cement Association ,1989 Mosely,1990 equation 33 BS 8110

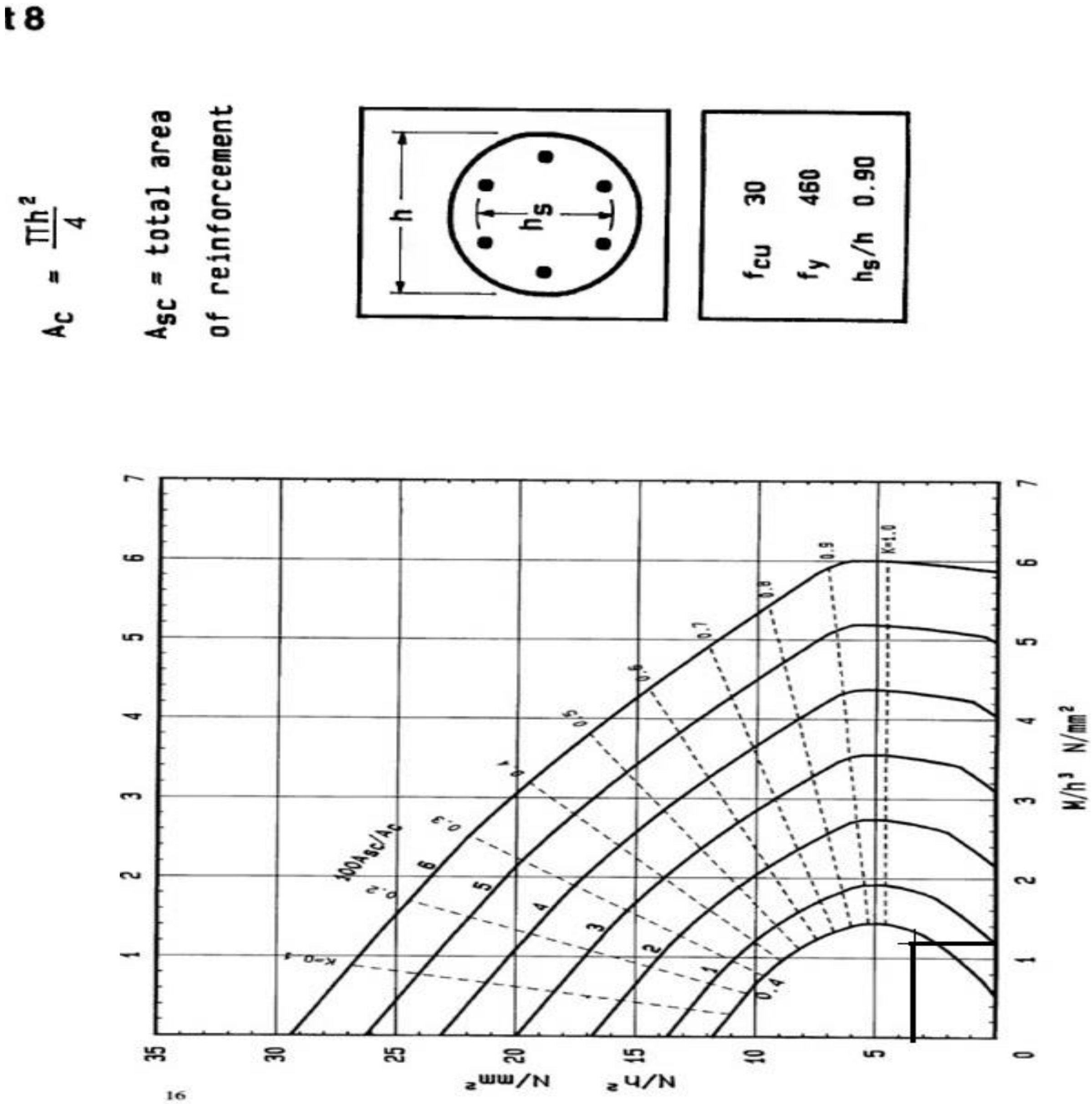


Figure (4.5) circle column design chart (BCA,1989)

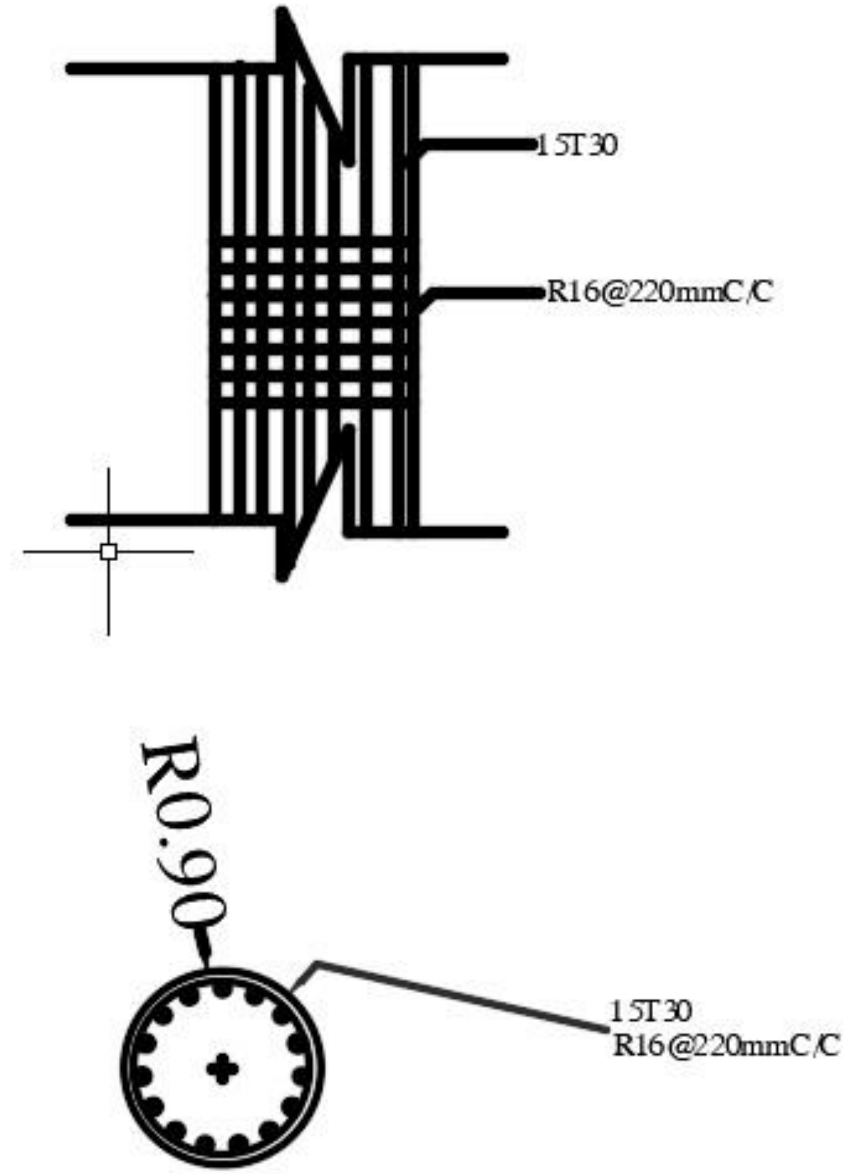


Figure (4.6) Pile Reinforcement Detail

Figure (4.6) Pile Reinforcement Detail

Chapter five

Analysis and Discussion of Results

Chapter Five

Analysis and Discussion of Results

5.1 Introduction:

In this chapter have been studied effects in change of length and diameter on the axial capacity and ultimate lateral load of single pile, also study the variability of pile (diameter , length , number) in groups effect on pile deformation (settlement , deflection) and loading distribution (vertical, lateral load , moment) using the result from manual calculation and All pile 6.5 software analysis compare with the actual results that had calculated (using PileGroup software) for case study soba bridge project.

5.2 vertical analysis results:

5.1.1 maximum settlement for 1x4 group of pile results:

Figure from (5.1) to (5.4) represent the effect of increase the pile diameter in maximum settlement for length 35.9 m for group 1x4, the comparison shows that the maximum settlement decreases about 26.43% with increase the pile diameters (1.2, 1.4,1.6, 1.8m). The increase of pile diameter effects in the increase end bearing capacity which decrease the settlement.

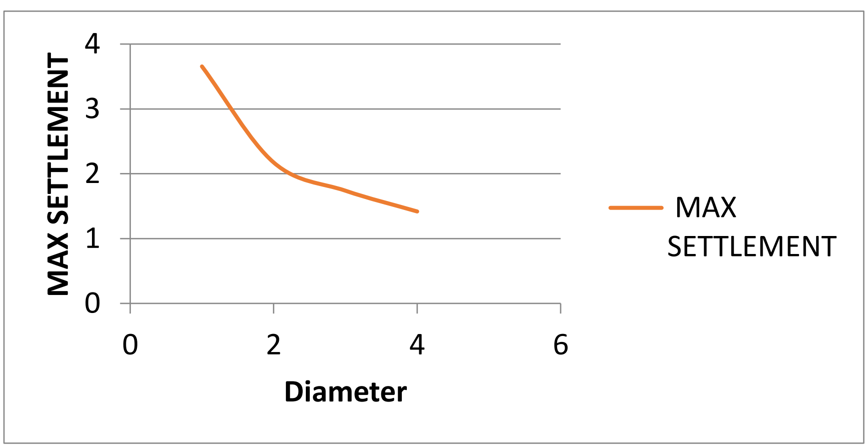
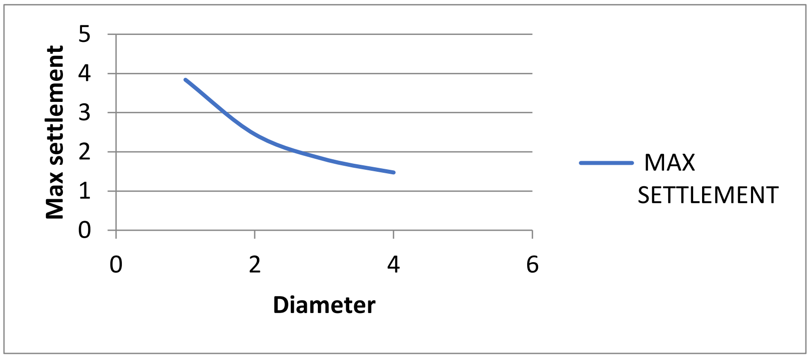


Table (5.1)

Diameter	Maximum settlement
1.2	3.6547
1.4	2.17314
1.6	1.73842
1.8	1.41779

Figure (5.1) L=35.9 LC01 1X4 group pile maximum settlement represent table (5.1) which from table (4.11)

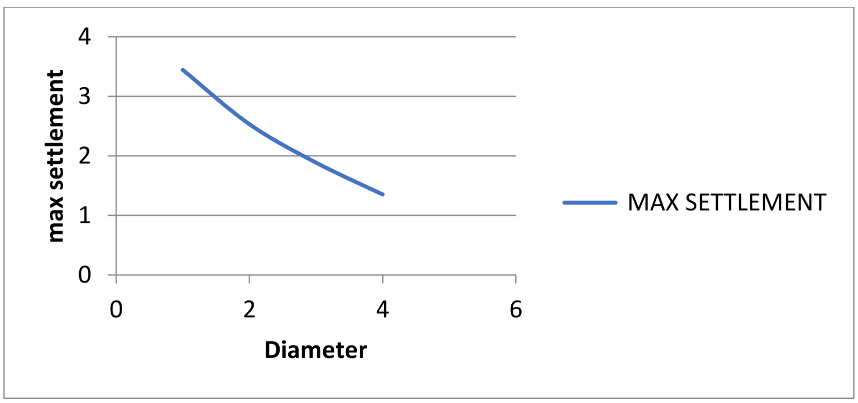
Table (5.2)



Diameter	Maximum settlement
1.2	3.83967
1.4	2.4484
1.6	1.8135
1.8	1.47383

Figure (5.2) L=35.9 LC05 1X4 group pile maximum settlement represent table (5.2)which from table(4.13)

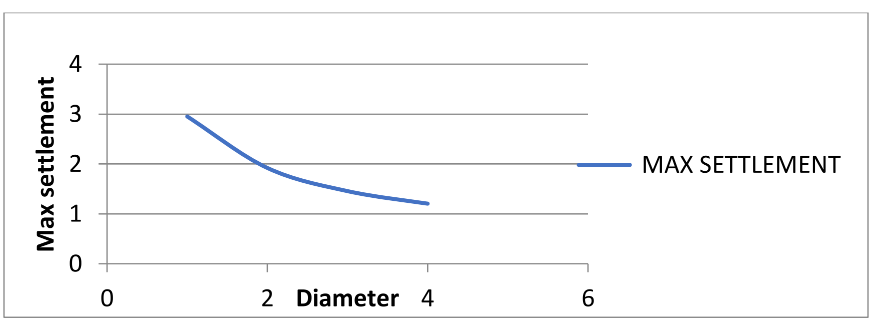
Table (5.3)



Diameter	Maximum settlement
1.2	3.44254
1.4	2.53139
1.6	1.88621
1.8	1.35244

Figure (5.3) L=35.9 LC11 1X4 group pile maximum settlement from table (5.3) which from table (4.15)

Table (5.4)



Diameter	Maximum settlement
1.2	2.95176
1.4	1.92008
1.6	1.45538
1.8	1.20442

Figure (5.4) L=35.9 LC18 1X4 group pile maximum settlement represent table (5.4) which from table (4.17)

Figure from (5.5) to (5.8) represent the effect of increase the pile diameter in maximum settlement for length 35.9, 32.9 and 25.9 m for group 1x4, the comparison show that the maximum settlement decrease 26.43% ,24.49% and 22.59% respectively with increasing of diameters of pile for all load combinations (LC01,LC05,LC11,LC18).

Decrease of the length effect on maximum settlement. decrease length of pile from 35.9 m to 32.5 m increasing maximum settlement about 14.14%, decrease the length from 35.9 m to 25.9 m increasing maximum settlement about 39.79%, also decrease from 32.9 m to 25.9 m increase the maximum settlement. The decrease of pile length effects in the decrease skin resistance which increase the settlement.

Table (5.5)

DIA	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	3.83967	4.10354	5.54872
1.4	2.4484	2.70226	3.28167
1.6	1.8135	2.0847	3.09679
1.8	1.47383	1.39998	2.54092

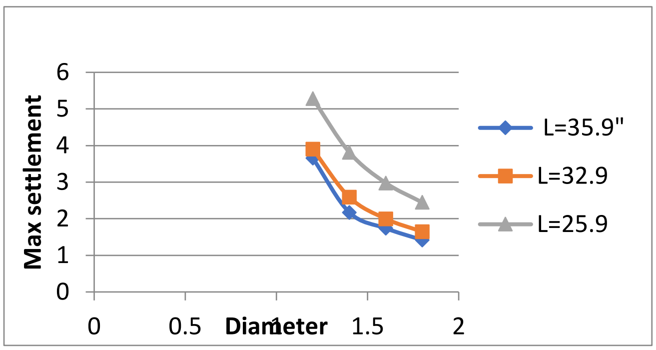


Figure (5.5) maximum settlement for L= (35.9 32.9 ,25.9) and Diameter = (1.2 , 1.4 , 1.6 ,1.8) for LC01 from tables (4.11 ,4.27and 4.43)

Table (5.6)

Dia	Maximum settlement 35.9	Maximum settlement 32.9	Maximum settlement 25.9
1.2	3.6547	3.89701	5.27992
1.4	2.17314	2.58921	3.8154
1.6	1.73842	1.99794	2.97759
1.8	1.41779	1.6457	2.44576

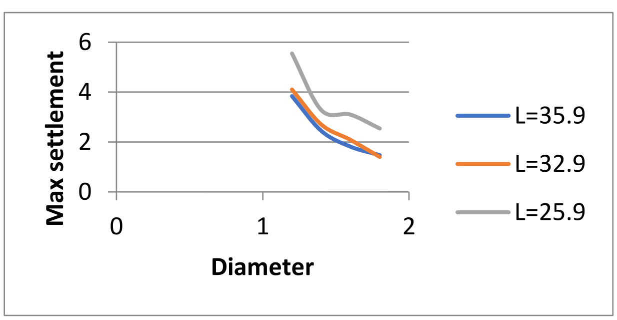
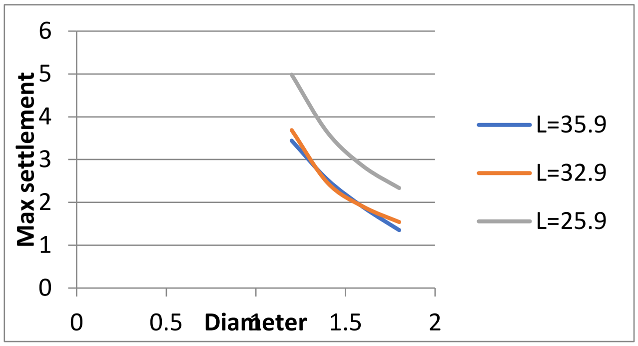


Figure (5.6) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2 , 1.4 , 1.6 ,1.8) for LC05 represent table (5.6) which from tables (4.13 ,4.29and 4.45)

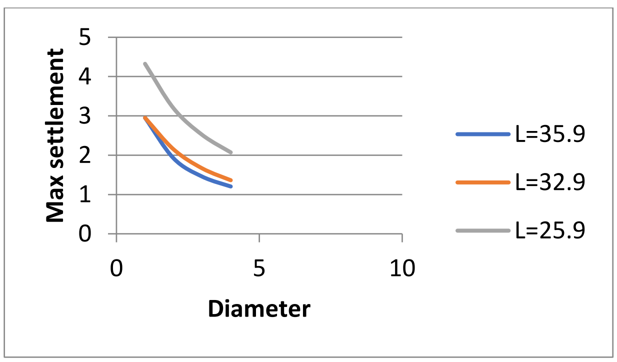
Table (5.7)



DIA	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	3.44254	3.6875	4.98361
1.4	2.53139	2.45891	3.63202
1.6	1.88621	1.89862	2.84111
1.8	1.35244	1.54154	2.33567

Figure (5.7) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2 , 1.4 , 1.6 ,1.8) for LC11 represent table(5.7) which from tables (4.15 ,4.31and 4.47)

Table (5.8)



DIA	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	2.95176	2.95176	4.32578
1.4	1.92008	2.14862	3.19182
1.6	1.45538	1.66543	2.51768
1.8	1.20442	1.36343	2.07212

Figure (5.8) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2, 1.4, 1.6 ,1.8) for LC18 represent table (5.8) which from tables (4.17 ,4.33and 4.49)

5.1.2 maximum settlement for 2x4 group of pile results:

Figure from (5.9) to (5.13) represent the effect of increase the pile diameter in maximum settlement for length 35.9 m for group 2x4, the comparison show that the maximum settlement decreases 27.2% with increase the pile diameters (1.2, 1.4,1.6, 1.8m).

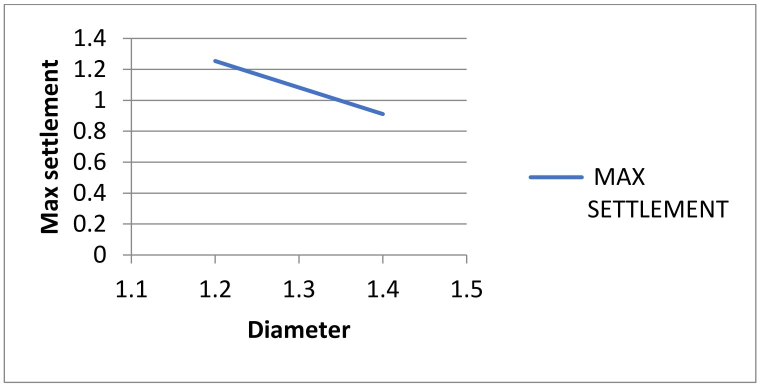


Table (5.9)

Diameter	Maximum settlement
1.2	1.62018
1.4	1.08411

Figure (5.9) L= 35.9 LC01 2X4 group maximum settlement represent table (5.9) which from table (4.19)

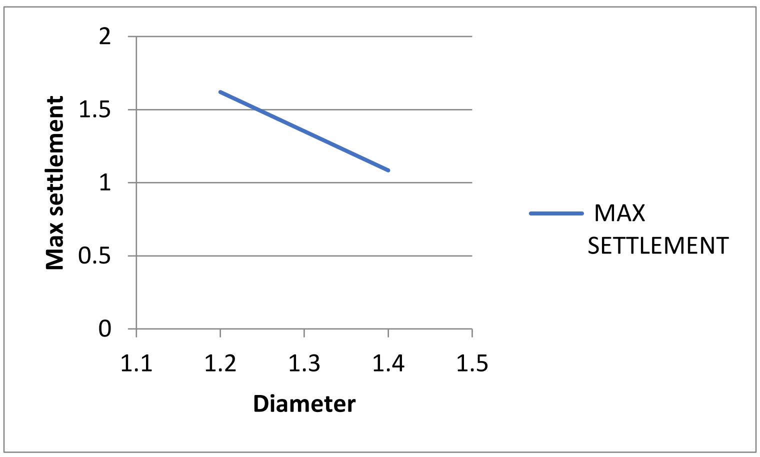
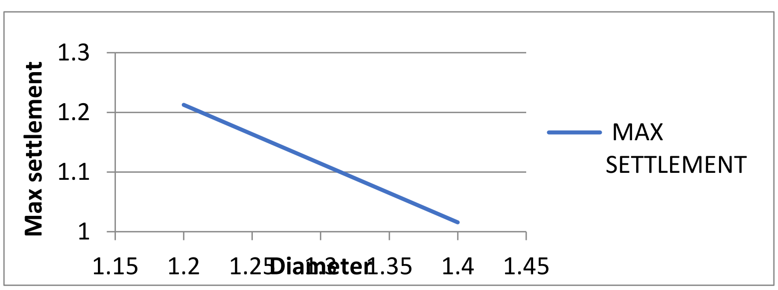


Table (510)

Dia	Maximum settlement
1.2	1.25408
1.4	0.91121

Figure (5.10) L= 35.9 LC05 2X4 group maximum settlement represent table (5.10) which from table (4.21).

Table (5.11)



Dia	Maximum settlement
1.2	1.21258
1.4	1.01566

Figure (5.11) L= 35.9 LC11 2X4 group maximum settlement represent table (5.11) which from table (4.23)

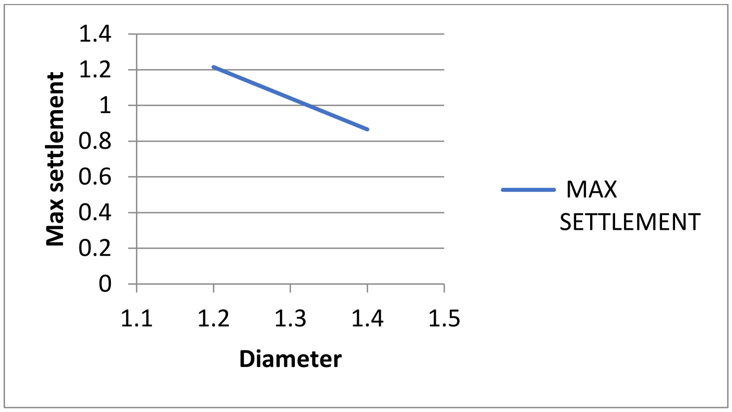


Table (5.12)

Diameter	Maximum settlement
1.2	1.21509
1.4	0.8659

Figure (5.12) L= 35.9 LC18 2X4 group maximum settlement represent table (5.12) which from table (4.25)

Figure from (5.9) to (5.12) represent the effect of increase the pile diameter in maximum settlement for length 35.9, 32.9 and 25.9 m for group 2x4, the comparison show that the maximum settlement decrease 27.2% ,28% and 25.85% respectively with increasing of diameters of pile for all load combinations (LC01,LC05,LC11,LC18).The compression between maximum settlement for group2x4 and 1x4 group shows maximum settlement decrease by 58.06%,62.63%,57.53% respectively to length (35.9, 32.9, 25.9 m).

Decrease of the length effect on maximum settlement. decrease length of pile from 35.9 m to 32.5 m increasing maximum settlement about 10.2%, decrease the length from 35.9 m to 25.9 m increasing maximum settlement about 33.1%, also decrease from 32.9 m to 25.9 m increase the maximum settlement about 25.5% for all diameters for group 2x4.

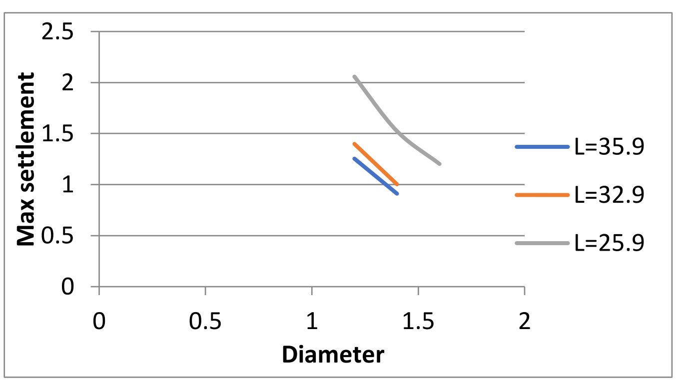
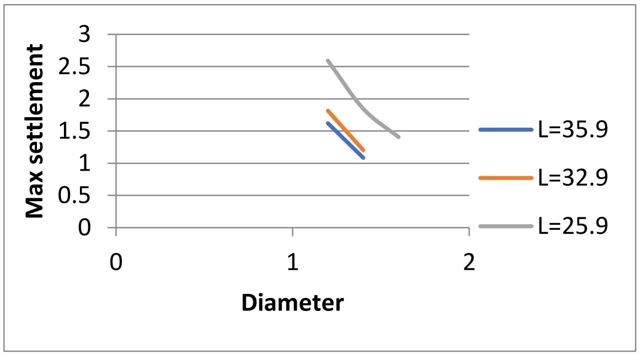


Table (5.13)

DIA	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	1.25408	1.39865	2.05773
1.4	0.91121	1.00297	1.52408
1.6			1.20225

Figure (5.13) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2 , 1.4 , 1.6 ,1.8) for LC01 represent table (5.13) which from tables (4.19,4.35and 4.55).

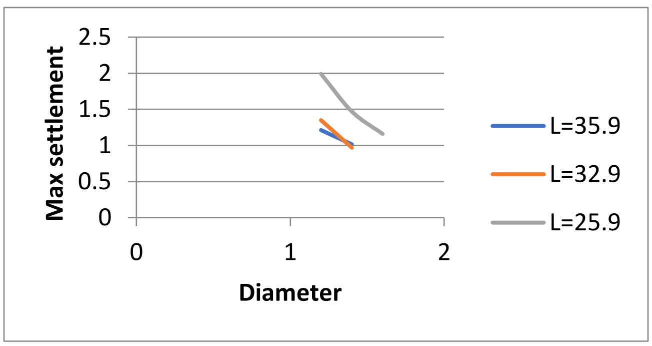
Table (5.14)



Diameter	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	1.62018	1.81424	2.59101
1.4	1.08411	1.20364	1.84569
1.6			1.40562

Figure (5.14) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2 , 1.4 , 1.6 ,1.8) for LC05 represent table (5.14) which from tables (4.21,4.37and 4.57).

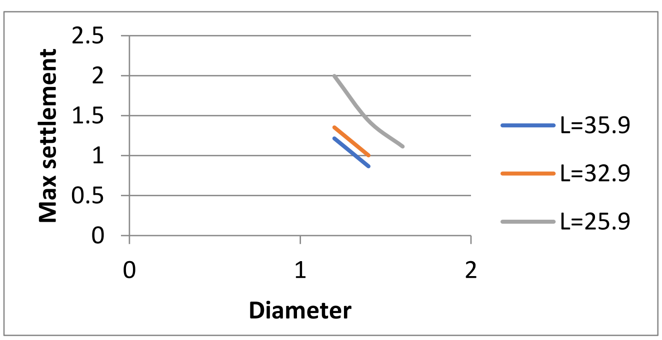
Table (5.15)



Diameter	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	1.21258	1.34999	1.99201
1.4	1.01566	0.96969	1.47084
1.6			1.15993

Figure (5.15) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2, 1.4, 1.6 ,1.8) for LC11 represent table (5.15) which from tables (4.23,4.39and 4.59)

Table (5.16)



Diameter	Maximum Settlement L=35.9	Maximum Settlement L=32.9	Maximum Settlement L=25.9
1.2	1.21509	1.35286	1.99595
1.4	0.8659	1.00297	1.4367
1.6			1.11339

Figure (5.16) maximum settlement for L= (35.9 ,32.9 ,25.9) and Diameter = (1.2, 1.4, 1.6 ,1.8) for LC18 represent table (5.16) which from tables (4.25,4.41and 4.61)

5.1.3 maximum vertical load results for pile group (1x4,2x4 3x4):

Figure from (5.17) to (5.22) represent the effect of increase number of pile and diameter in maximum vertical for group 1X4,2x4 and 3X4 the comparison between the Allpile6.5 and manual results showing below for combinations (LC01, LC05, LC11, LC18).

maximum vertical load decrease by 46.67 % and 35.28% with increasing of number of piles for 1x4 ,2x4 and 3X4.

the manual results increase more the All pile6.5 results respectively 2.93%,1.6%and 6.4% for group 1X4,2x4 and 3X4.

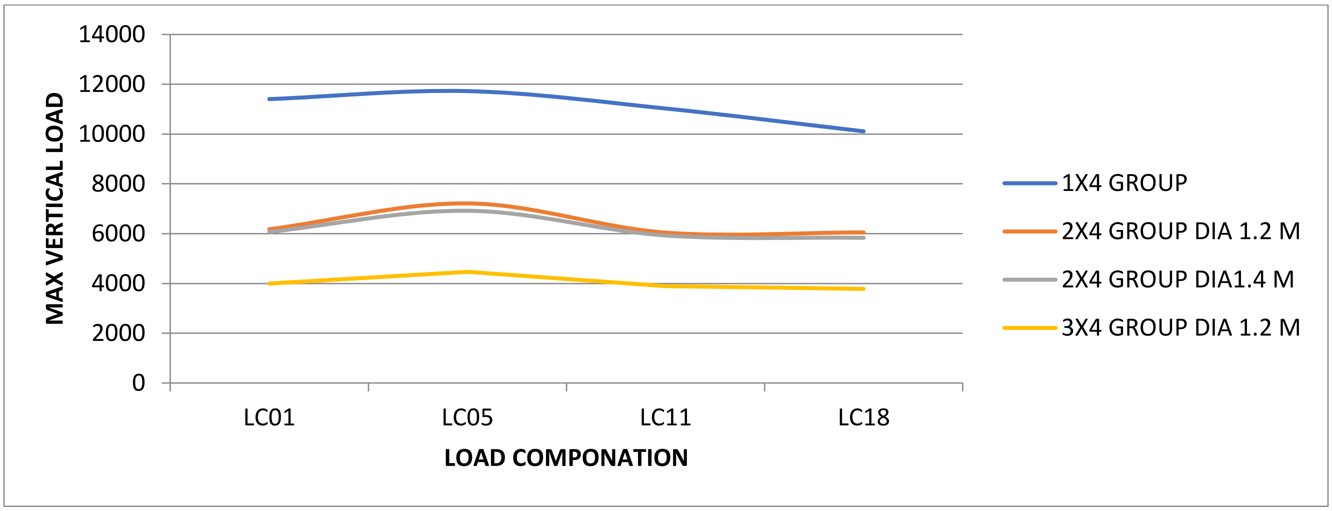


Figure (5.17) maximum vertical load with different pile groups Allpile6.5 results from table (5.17)

table (5.17)

Load combination	maximum vertical load for Allpile 1.2,1.4m pile group1x4	maximum vertical load for Allpile1.2 m pile group2x4	maximum vertical load for Allpile1.4 m pile group2x4	maximum vertical load for Allpile1.2 m pile group3x4
LC01	11402.5	6175.1	6071.65	3996.05
LC05	11725.03	7213.81	6918.82	4465.05
LC11	11026.31	6041.26	5925.98	3893.01
LC18	10110.35	6049.36	5832.33	3779.7

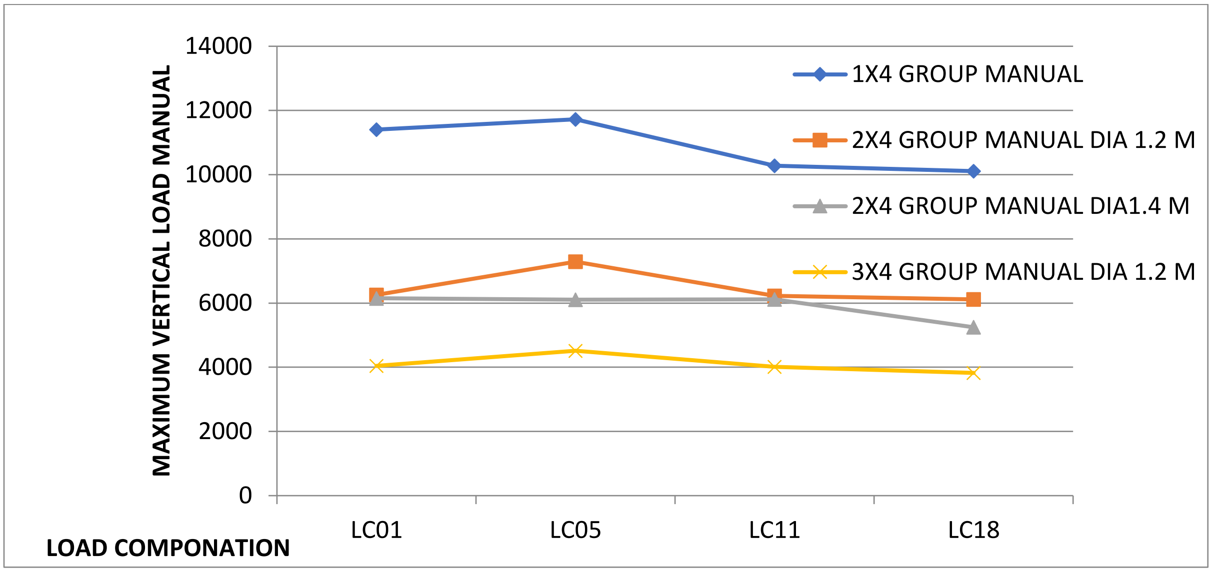


Figure (5.18) maximum vertical load manual results with different pile from table (5.18).

table (5.18)

Load combination	maximum vertical load manual for 1.2and 1.40m ,1x4 pile group	maximum vertical load manual for 1.2m ,1x4 pile group	maximum vertical load manual for 1.4m ,2x4 pile group	maximum vertical load manual for 1.2m ,3x4 pile group
LC01	6253.601	6175.1	6150.159	4048.385
LC05	7287.942	7213.81	6107.98	4514.473
LC11	6227.192	6041.26	6111.904	4016.959
LC18	6114.593	6049.36	5246.458	3823.189

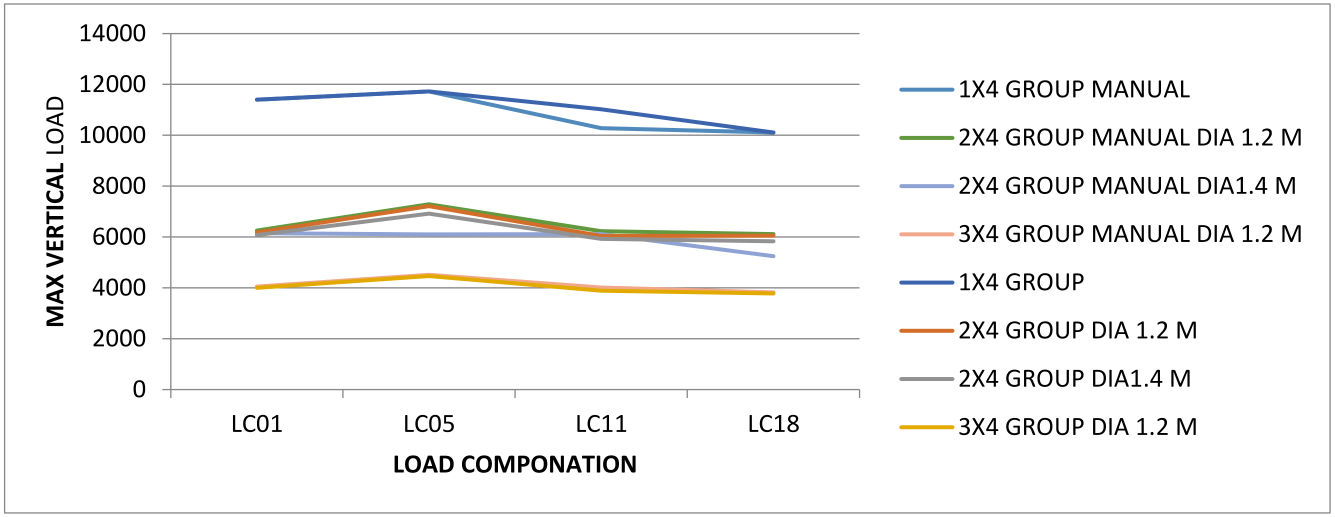


Figure (5.19) maximum vertical load manual and allpile6.5 results with different group pile for length 39.5m from table (5.19).

Table (5.19)

Load combination	maximum vertical load for Allpile for 1.2and 1.40m ,1x4 pile group	maximum vertical load manual for 1.2and 1.40m ,1x4 pile group	maximum vertical load for Allpile for 1.2m ,1x4 pile group	maximum vertical load manual for 1.2m ,1x4 pile group	maximum vertical load for Allpil for 1.4m ,2x4 pile group	maximum vertical load manual for 1.4m ,2x4 pile group	maximum vertical load for Allpile for 1.2m ,3x4 pile group	maximum vertical load manual for 1.2m ,3x4 pile group
LC01	11402.5	11402.5	6175.1	6253.601	6071.65	6150.159	3996.05	4048.385
LC05	11725.03	11725	7213.81	7287.942	6918.82	6107.98	4465.05	4514.473
LC11	11026.31	10282.56	6041.26	6227.192	5925.98	6111.904	3893.01	4016.959
LC18	10110.35	10110.35	6049.36	6114.593	5832.33	5246.458	3779.7	3823.189

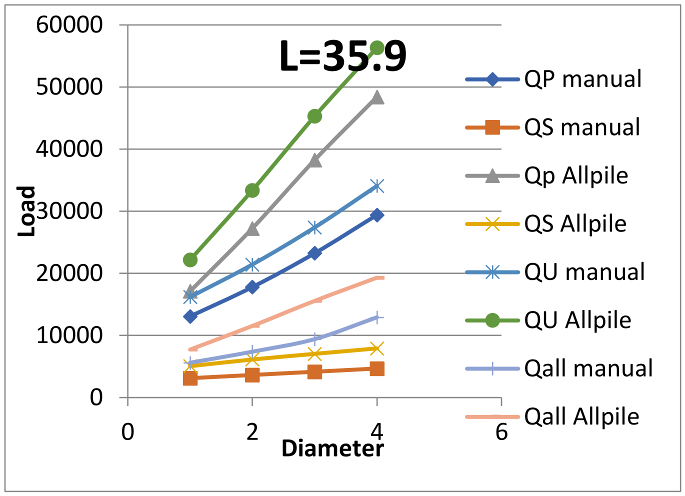


Figure (5.20) vertical load capacity manual and allpile6.5 results with different pile diameters from tables (4.2 to 4.4).

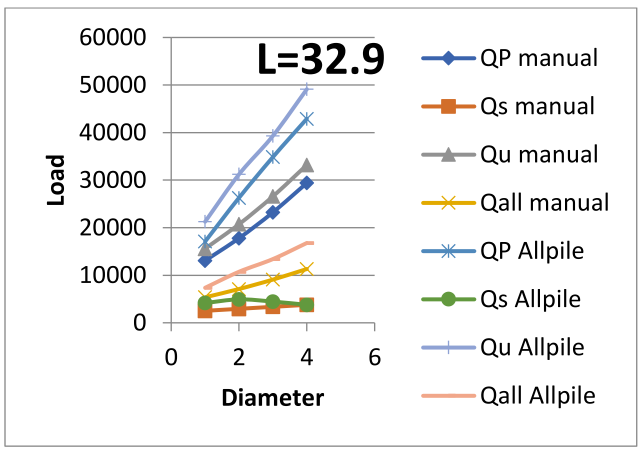


Figure (5.21) vertical load capacity manual and allpile6.5 results with different pile diameters from tables (4.2 to 4.4).

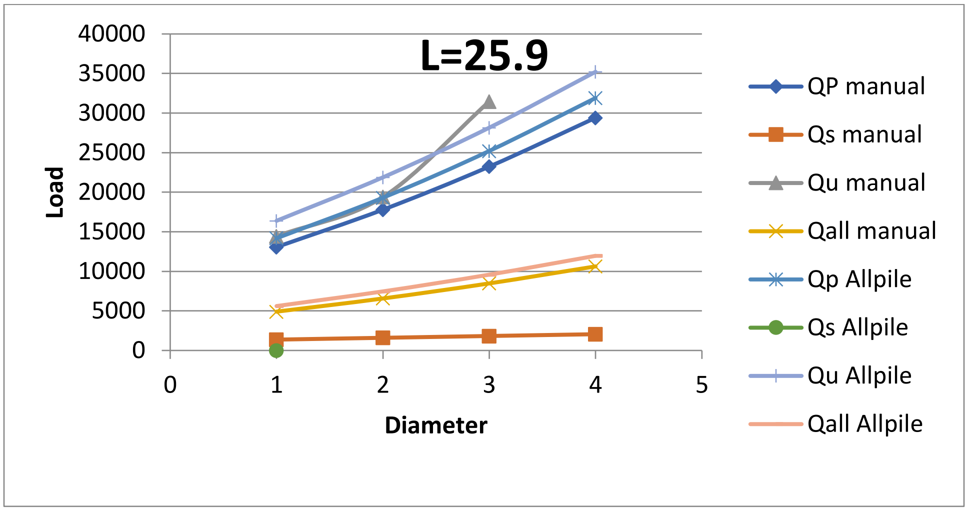


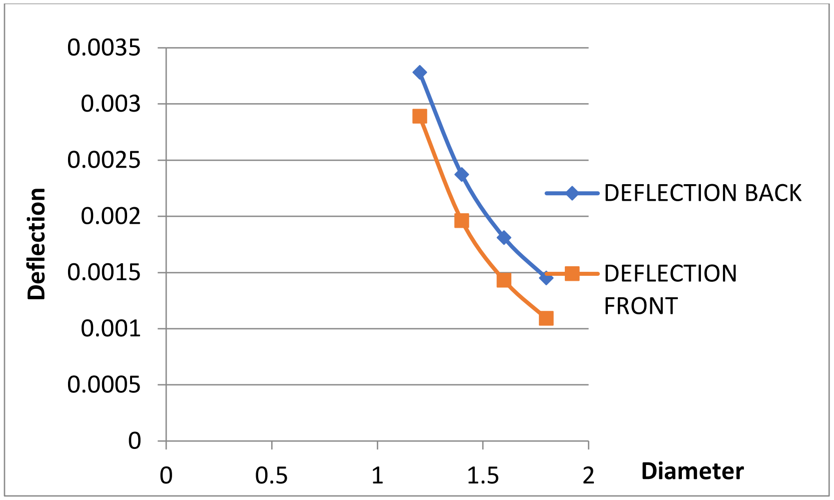
Figure (5.22) vertical load capacity manual and allpile6.5 results with different pile diameters from tables (4.2 to 4.4).

5.2 Lateral load result:

5.2.1 deflection and maximum moment of lateral load for 1x4 group pile results:

Figure from (5.23) to (5.30) represent the effect of increase the pile diameter in deflection and maximum moment for lengths 35.9 ,32.9 and 25.9 m for group 1x4. maximum moment and deflection for back and front pile, also compare with diameters (1.2, 1.4,1.6, 1.8m). the maximum deflections decrease by 33.75% with increase the pile diameters also moment for back increase 7.5%. and front is increase 5.4%. The comparison between results from manual calculations and Allpile6.5 for maximum moment find that the manual maximum moment for back pile increase about 5.9% and 3.8% decrease for front pile about 15.74% and 23.12% from Allpile 6.5 results for LC11 and LC18 respectively.

Figure (5.23) LC05 1X4 group deflection for lateral load, maximum moment lateral for diameter (1.2, 1.4 , 1.6 ,1.8) and lengths (35.9,32.9,25.9) for Allpile results from table (5.23)



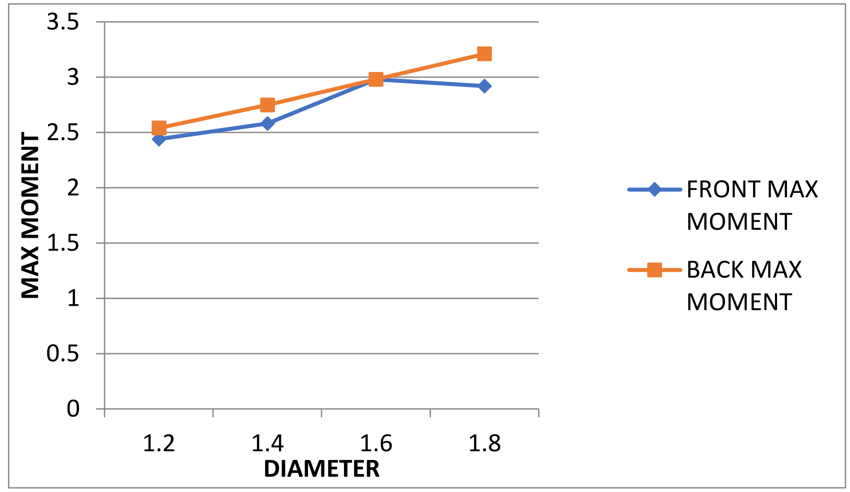
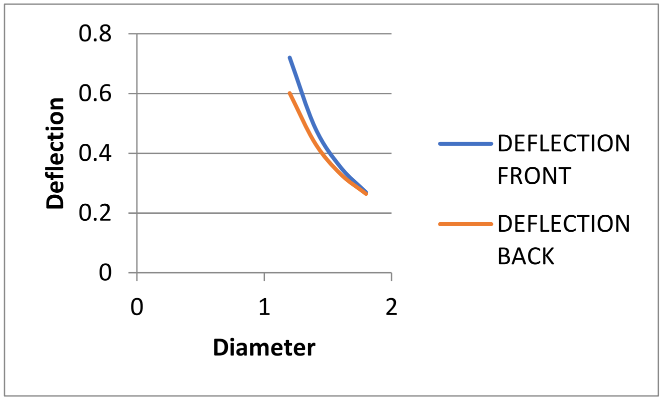


Table (5.23)

Diameter	1.2		1.4		1.6		1.8	
Maximum Moment	Back	Front	Back	Front	Back	Front	Back	Front
Lateral load	2.55	2.46	2.75	2.6	2.98	2.76	3.22	2.93
Deflection	0	0	0	0	0	0	0	0



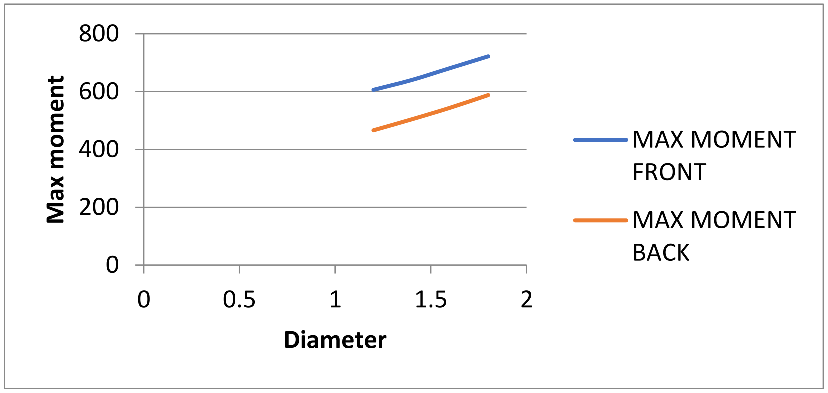
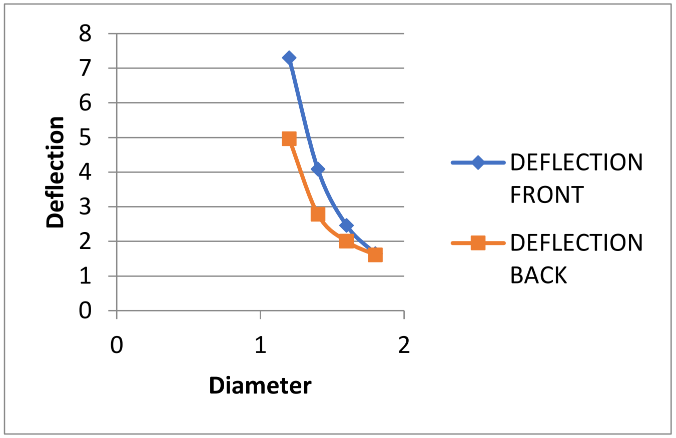


Figure (5.24) LC11 1X4 group deflection for lateral load , maximum moment lateral for diameter (1.2 , 1.4 , 1.6 ,1.8) and lengths (35.9,32.9,25.9) for Allpile results from table (5.24)

Table (5.24)

Diameter		*1.2*		*1.4*		*1.6*		*1.8*
	Back	Front	Back	Front	Back	Front	Back	Front
Maximum Moment	466	606	504	640	544	681	588	722
Lateral load	73.1	98.9	73.1	98.9	73.1	98.9	73.1	98.9
Deflection	0.601	0.72	0.433	0.485	0.33	0.353	0.264	0.269



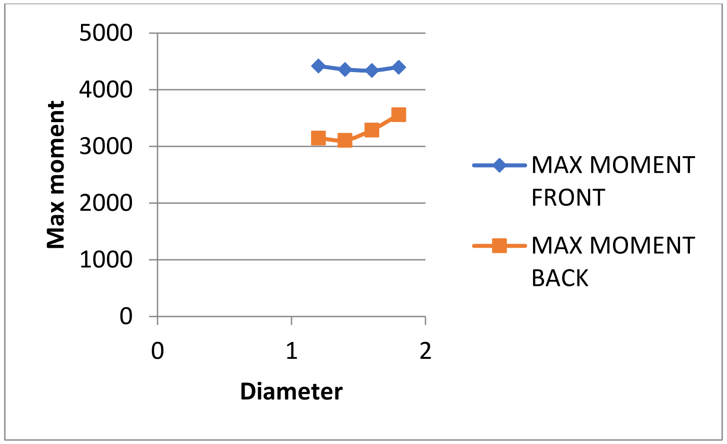


Figure (5.25) LC18 1X4 group deflection for lateral load , maximum moment for diameter (1.2 , 1.4 , 1.6 ,1.8) and lengths (35.9,32.9,25.9) for Allpile results from table (5.25).

Table (5.25)

Diameter		*1.2*		*1.4*		*1.6*		*1.8*
	Back	Front	Back	Front	Back	Front	Back	Front
Maximum Moment	3150	4420	3110	4360	3290	4340	3560	4400
Lateral load	443.91	600.59	443.91	600.59	443.91	600.59	443.91	600.59
Deflection	4.96	7.3	2.78	4.08	2	2.46	1.61	1.64

5.2.2 deflection and maximum moment of lateral load for 2x4 group pile results:

Figures from (5.26) to (5.29) represent the effect of increase the pile diameter in deflection and maximum moment for lengths 35.9 ,32.9 and 25.9 m for group 2x4. maximum moment and deflection for back and front pile, also compare with diameters (1.2, 1.4,1.6, 1.8m). the maximum deflections decrease by 37.59% with increase the pile diameters also moment for back increase 5.39%. and front is increase 4.75%. The comparison between results from manual calculations and Allpile6.5 for maximum moment find that the manual maximum moment for back pile increase about 9.5% and 9.1% decrease for front pile about 7.2% and 16.5% from Allpile 6.5 results for LC11 and LC18 respectively.

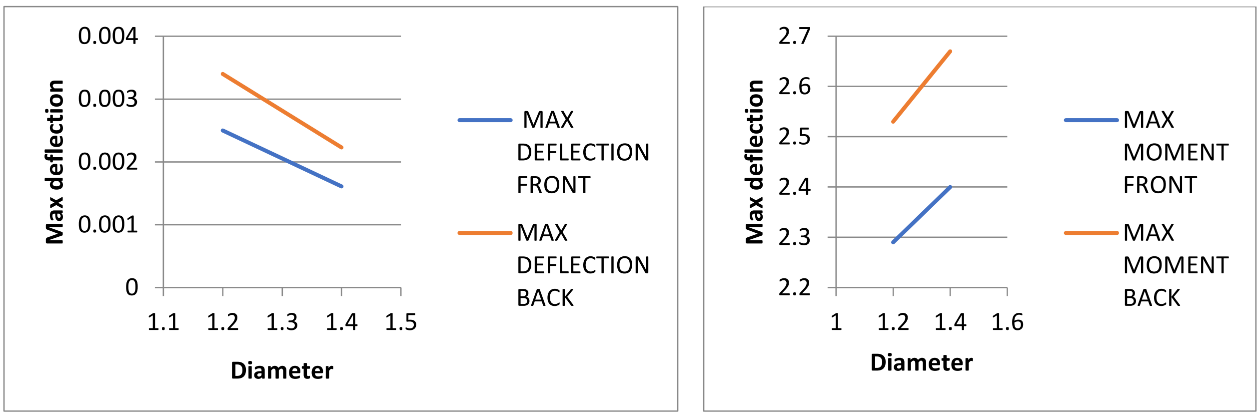
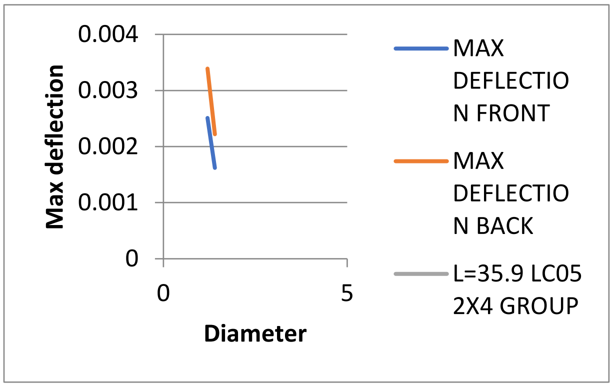


Figure (5.26) LC01 2X4 group maximum deflection, for diameter (1.2 , 1.4 , 1.6 ,1.8) for Allpile results from table(5.7)

Table (5.26)

Diameter (m)	1.2		1.4	
	Back	Front	Back	Front
Maximum Moment	2.53	2.29	2.67	2.4
Lateral load	0	0	0	0
Deflection	0.0034	0.0025	0.00223	0.00161



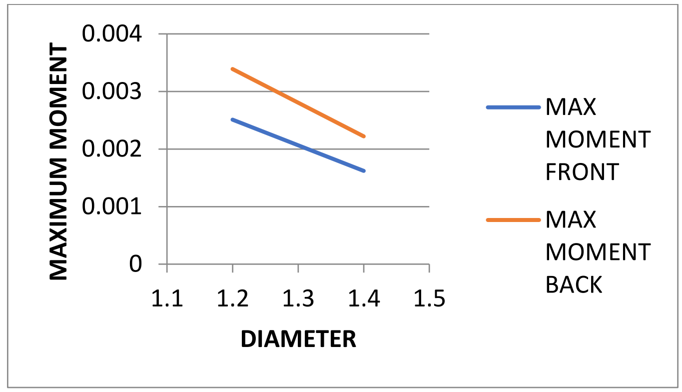
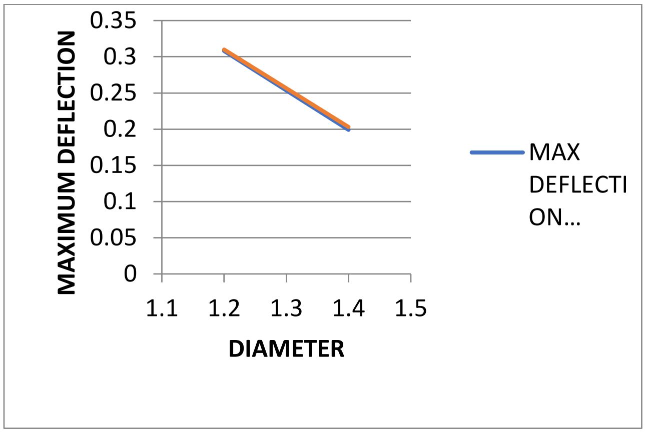


Figure (5.27) LC01 2X4 group maximum deflection, maximum moment for diameter (1.2, 1.4 , 1.6 ,1.8) for Allpile results from table (5.27)

Table (5.27)

Diameter (m)	*1.2*		*1.4*	
	Back	Front	Back	Front
Maximum Moment KN.m	2.51	2.3	2.66	2.41
Lateral load KN	0	0	0	0
Deflection cm	0.00339	0.00251	0.00222	0.00162



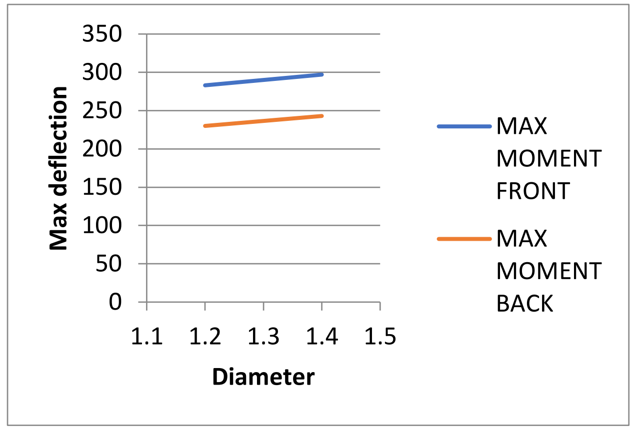


Figure (5.28) LC05 2X4 group maximum deflection, maximum moment, maximum moment for diameter (1.2 , 1.4 , 1.6 ,1.8) for Allpile results from table(5.28).

Table (5.28)

DIA (M)	1.2		1.4	
	Back	Front	Back	Front
Maximum Moment	230	283	243	297
Lateral load	36.55	49.45	36.55	49.45
Deflection	0.31	0.308	0.203	0.199

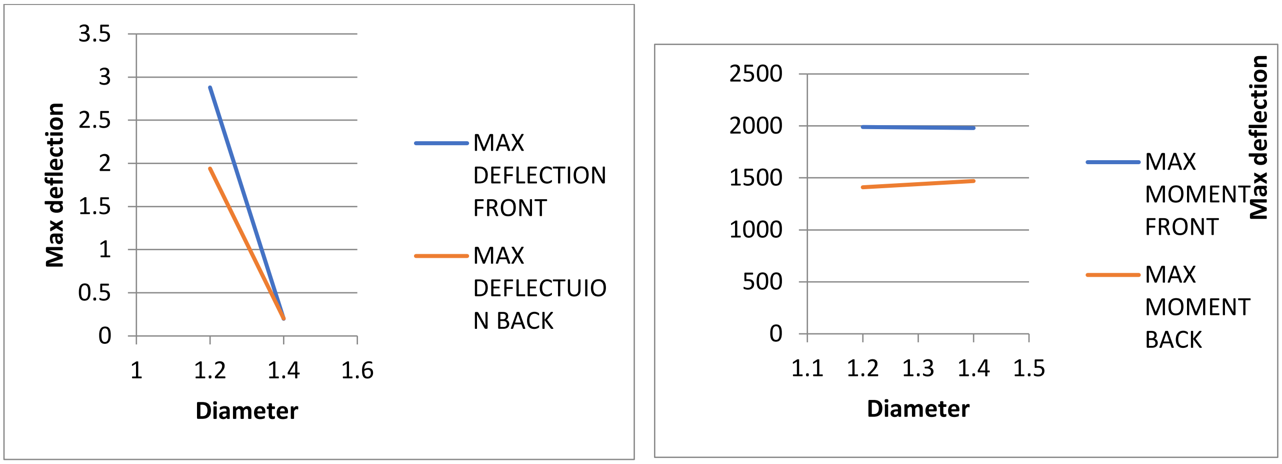


Figure (5.29) LC11 2X4 group maximum deflection, maximum moment for diameter (1.2, 1.4 , 1.6 ,1.8) for Allpile results from table (5.29).

Table (5.29)

Diameter(m)	1.2		1.4	
	Back	Front	Back	Front
Maximum Moment	1410	1990	1470	1980
Lateral load	221.96	300.29	221.96	300.29
Deflection	1.94	2.88	0.203	0.199

5.3 The results from manual calculation, both software Allpile6.5 and pile group software for 1x4 pile group with load combinationsLC01 LC05, LC11, LC18 :

Figure (5.30) represent the comparison of maximum vertical load results from two software program(Allpile6.5 ,pile group) and manual calculation of analysis 1x4 group of pile spacing 6.25 length 25.9 m , the comparison show that the result from Allpile6.5 and manual calculation typically same for all combinations, for pile group just results increase 8.09% for LC18 .

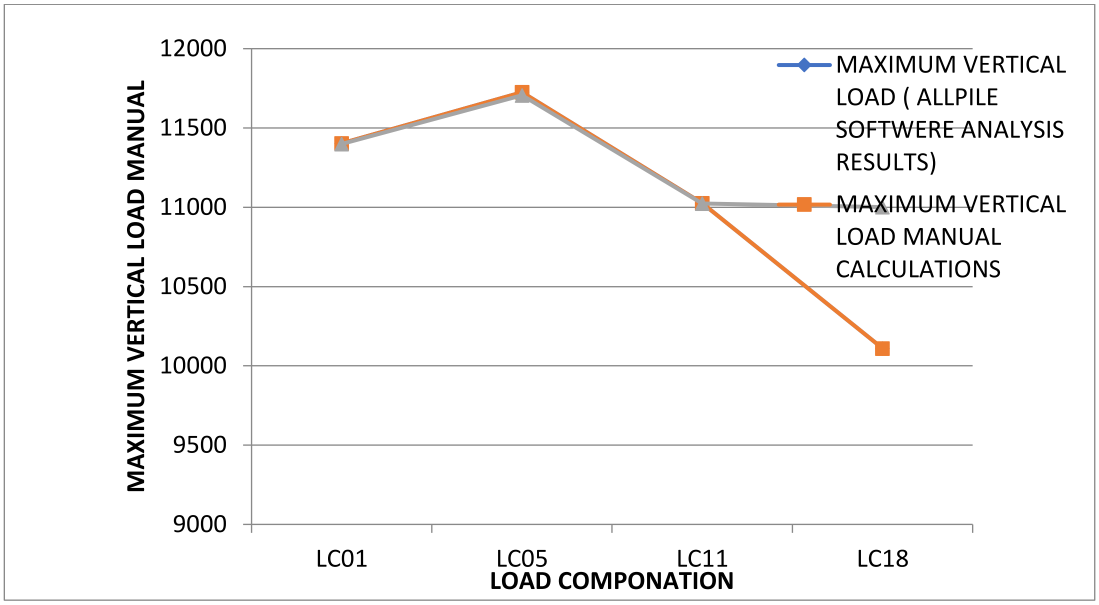


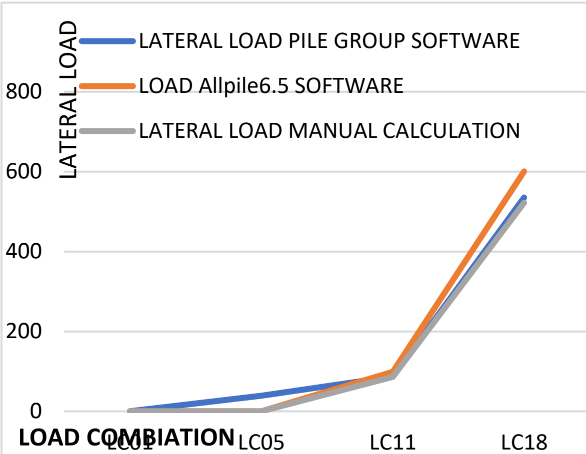
Figure (5.30) represent the comparison of vertical load manual and Allpile6.5 software and pilegroup software results with load combination LC01 LC05, LC11 ,LC18 from table (5.30).

Table (5.30)

LC	Maximum vertical load (ALLPILE softwar analysis results)	Maximum vertical load manual calculation	Maximum vertical load (PILE GROUP software analysis results))
LC01	11402.5	11402.5	11400
LC05	11725.03	11725	11706
LC11	11026.31	11026	11024
LC18	10110.35	10110.35	11001

Figure (5.31) represent the comparison of lateral load results, the comparison shows that the result from Allpile6.5 more than manual calculation by 13%, also more than pile group software by 10%.

Table (5.31)

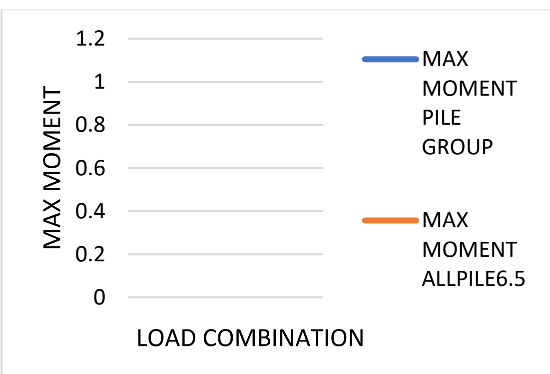


LC	lateral load PILE GROUP software	Lateral load Allpile software	lateral load manual calculation
LC01	0.244	0	0
LC05	38.612	0	0
LC11	87.071	98.9	86
LC18	535.24	600.59	522.25

Figure (5.31) lateral load for manual, Allpile6.5 software and pile group software results with load combination LC01 LC05, LC11, LC18 from table (5.12).

Figure from 5.32 represent the comparison of maximum moment results, the comparison shows that the result from Allpile6.5 more than manual calculation by 44% and 63% LC11and LC18, also more than pile group software by 43% and 90% LC11and LC18.

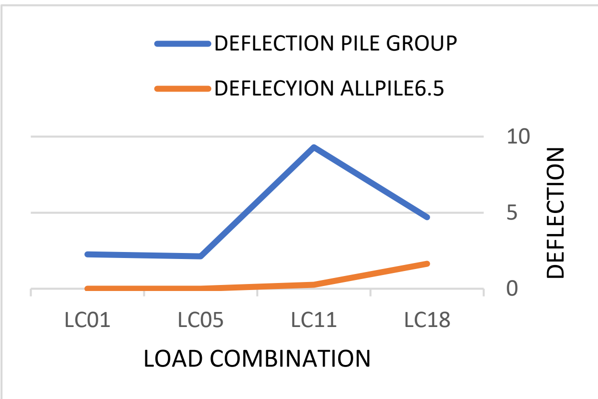
Table (5.32)



LC	maximum moment pile group software	maximum moment Allpile software	maximum moment manual calculation
LC01	824.18	3.22	0
LC05	791.63	3.22	0
LC11	1965.5	1102.31	612.76
LC18	5226.9	10110.4	3715.8

Figure (5.32) maximum moment for manual, Allpile6.5 software and pile group software results with load combination LC01 LC05, LC11, LC18 from table (5.13).

Table (5.34)



LC	deflection group software	deflectiond Allpile software
LC01	2.25	0.0019
LC05	2.13	0.0019
LC11	9.3	0.269
LC18	4.703	1.64

Figure (5.33) deflection for Allpile6.5 software and pile group software results with load combination LC01 LC05, LC11, LC18 from table (5.14)

Figure from (5.33) represent the comparison of deflection results, the comparison shows that the result from Allpile6.5 less than pile group software by 99% and 65% LC11and LC18.

Chapter Six

Conclusions and Recommendations

Chapter Six

Conclusions and Recommendations

6.2 Conclusions:

1. Decrease of the length effect on maximum settlement decrease length of pile from 35.9 m to 32.5 m increasing maximum settlement about 14.14%, decrease the length from 35.9 m to 25.9 m increasing maximum settlement about 39.36%, also decrease from 32.9 m to 25.9 m increase the maximum settlement about 31.08% for all diameters for group 1x4.
2. Increase of the pile diameter effect on reduction maximum settlement for length 35.9 m the maximum settlement decrease about 26.43%, for length 32.9 m the maximum settlement decrease about 24.49% and for length 25.9 m maximum settlement decrease about 22.59% for group 1x4 with increasing of diameters.
3. Decrease of the length effect on maximum settlement. decrease length of pile from 35.9 m to 32.5 m increasing maximum settlement about 10.2%, decrease the length from 35.9 m to 25.9 m increasing maximum settlement about 33.1%, also decrease from 32.9 m to 25.9 m increase the maximum settlement about 25.5% for all diameters for group 2x4.
4. Increase of the pile diameter effect on reduction of maximum settlement for length 35.9 m the maximum settlement decrease about 27.2%, for length 32.9 m the maximum settlement decrease about 28% and for length 25.9 m the maximum settlement decrease about 25.85% for group 2x4 with increasing of diameters.
5. The compression between maximum settlement for group2x4 and 1x4 group shows that maximum settlement for 1x4 less than maximum settlement 2x4 group by 58.06%,62.63%,57.53% respectively to length (35.9, 32.9, 25.9 m .
6. Increase number of pile and diameter decrease the maximum vertical load in each pile for (1X4,2x4 and 3X4) groups. the maximum vertical load on each pile decreased by 54.38% for 1x4 to 2x4, for 1x4 to 3x4 decrease by 64.95% and by 23.16 % for 2x4 to 3x4 group pile.
7. The manual empirical formulas maximum vertical load results increase more than All pile6.5 results respectively 2.93%,1.6%and 6.4% for all lengths.
8. Increase of the pile diameter effect on deflection and maximum moment for all lengths for group 1x4. the maximum deflections decrease by 33.75% with increase

the pile diameters also moment for back pile increase by 7.5%. and front pile is increase by 5.4%.

9. The comparison between results from manual empirical formulas calculations and Allpile6.5 for maximum moment find that the manual empirical formulas maximum moment for back pile increase by 5.9% and 3.8% and decrease for front pile about 15.74% and 23.12% for All pile 6.5 results for Lc11 and LC18 respectively.
10. Increase of the pile diameter effect on deflection and maximum moment for lengths 35.9 ,32.9 and 25.9 m for group 2x4. the maximum deflections decrease by 37.59% with increase the pile diameters also moment for back pile increase by 5.39%. and front pile is increase by 4.75%.
11. The comparison between results from manual empirical formulas calculations and Allpile6.5 for maximum moment find that the manual empirical formulas maximum moment for back pile increase about 9.5% and 9.1% and decrease for front pile about 7.2% and 16.5% from Allpile6.5 results for LC11 and LC18 respectively.
12. The comparison of maximum vertical load results from two software program(Allpile6.5 ,pile group) and manual empirical formulas calculation of analysis 1x4 group of pile spacing 6.25 length 25.9 m , the comparison show that the result from Allpile6.5 and manual empirical formulas calculation typically same for all combinations, for software pile group results increase 8.09% for only LC18 .
13. The Allpile6.5 lateral load results are more than manual empirical formulas calculation results by 13%, also more than pile group software by 10%.
14. The Allpile6.5 maximum moment results are more than manual empirical formulas calculation results by 44% and 63% LC11and LC18, also more than pile group software by 43% and 90% LC11and LC18.
15. The Allpile6.5 deflection results less than pile group software results by 99% and 65% LC11and LC18,
16. According to the analysis the increase of the length doesn’t effect on reduction of deflection or moment, lateral load on pile.
17. Increase of the diameter of pile reduces the deflection and increase the maximum moment on single pile.
18. the result of analysis by two software Allpile 6.5 is 2D analysis software and the PILEGROUP is 3D software and the manual empirical formulas calculation for

vertical load analysis is close but for lateral load analysis the result from manual empirical formulas and allpile6.5 is close but the pile group is large compare with the other two ways.

19. The pile group designed 1x4 length 35.9 and diameter 1.8 to support all the load and the pile settlement and deflection in the design range.

6.3 Recommendations:

Based on the results obtained it to recommended:

1. To use manual empirical formulas calculation preliminary studies of pile as single and pile group analysis in order to predict capacity and load subjected to pile for pile design.
2. To use Allpile6.5 software for all final analysis and design of bored piles.

For future studies it recommended to:

1. Analyze pile and group pile using 3D software program.
2. Analyze and design batter pile inside group pile and study the effect that on vertical and lateral load distribution.
3. Analyze and design group pile under earthquake load.

References

References

1. A and A for urban development soba bridge project data analysis 2013
2. Al-amery , Mukdad Mehmod ,2005 “Analysis and Design of piles in central of Khartoum using finite element method” Sudan University of Science and Technology ,College of Graduate .
3. British Cement Association ,1989 “Charts for the design of circular columns to BS8110” Century house Telford Avenue Crowthorne Berks RG45 6YS.
4. Elshareif, Ahmed M, 2007 “Guide For Design of Bored Concrete Pile in Expansive Soil OF Sudan ” www.reseachgate.net .
5. Elshareif, Ahmed M, 2014“Design Practice of bored piles in Nubian Formation case study foundation and bridges in Khartoum Bridges IN Khartoum” Building and Road Research Institute, University of Khartoum, Sudan Engineering Society Journal.
6. Arora, K.R,2004. “soil mechanics and foundation engineering (in si unit) reprint six edition standard publisher’s distribution”
7. Basile, Francesco.2003” Analysis and design of pile groups” www.researchgate.net.
8. Bowles, E Joseph, RE., S.E,1997. “Foundation Analysis and Design” Fifth Edition. The McGraw-Hill Companies, Inc
9. BS 8110-1: 1997 Structural use of concrete — Part 1: Code of practice for design and construction.
10. Das,M. Braja, 2007. “Principles of Foundation Engineering”, SI. Sixth Edition. Cengage Learning, Global Engineering: Christopher M. Shortt. THOMSON.
11. Elsharief, M. Ahmed. 2014. “Design Practice of Bored Piles in Nubian Formation case study: Foundation and Bridges in Khartoum” Sudan Engineering Society Journal.
12. Ivsic, Tomislav. 2013“Estimation of bored capacity and settlement in soft soil” www.reseachgate.net .
13. McGinley, T. J,2009 “Reinforcement concrete design theory and example “third addition. Tylor & Francis group 2 Park Square, Milton Park, Abingdon, Oxon OX14 4RN.
14. Mosely, H.W and Bungy, J.H. , 1990,”Design reinforcement concrete design “MACMMILLAN PRESS LTD 4th addition .

15. Murthy, V.N.S, 1969“Geotechnical engineering, principle and practices of soil mechanics and foundation engineering.
16. Nayak, V. Narayan, 2001.” foundation design manual for practicing engineer and civil engineering study”. fourth revised and enlarged Edith. Dhanpat Rai PuplicationsS (P) LTD.
17. Poulos, H.G., E.H. DAVIS, 1980. “Pile Foundation Analysis and Design”. rainbow – Bridge Book Co.
18. Poulos, H.G .2014 “Pile Group analysis: A study of two method“ www.reseachgate.net .
19. Priya, Harika.2017 “Comparative Study and Analysis of the Lateral and Vertical Loads of Pile Foundation” Department of Civil Engineering 1,2Lords Institute of Engineering and Technology, Himayathsagar, Hyderabad, India.
20. Reese, C. Lymon, 1984. “Laterally Loaded Piles and Computer Program COM624G” U. S. Army Engineer Division, Lower Mississippi Valley P. 0. Box 80, Vicksburg, Miss. 39180.
21. Ruigrok, J.A.T, 2010, “Laterally Loaded Piles Models and Measurements”, TU Delft Faculty of Civil Engineering and Geosciences Section of Geo-engineering
22. Tomilson,M. J 2008. “Pile Design and Construction Practice “Fifth edition. Taylor & Francis 2 Park Square, Milton Park, Abingdon, Oxon OX14 4RN.
23. Zhng, Qian-qing .2015 “Simplified non-liner approaches for response of a single pile and pile group considering progressive deformation of pile-soil system” research center of geotechnical and structural engineering, Shandong university, Jinan, china.

Appendix A

****

ALLPILE 6 results

VERTICAL ANALYSIS SUMMARY OUTPUT

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****

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Date: 2019/08/07 File: C:\Users\Eglaal\Desktop\ANALSYS DATA\FIXED SUPPORT\DIA 1800 mm\DIA 1.8 L=30\DIA 1.8 spacing 6.25m\DIA180 COPM LC01.alp

Title 1: PILE dia 1.80 m LC01

Title 2: THE PILE ANALYSIS

TOTAL LOADS:

Vertical Load, Q: 43608.0 -kN

Load Factor for Vertical Loads: 1.0

Loads Supported by Pile Cap: 0 %

PILE PROFILE:

Pile Length, L= 35.9 -m

Top Height, H= 5.9 -m

Slope Angle, As= 0

Batter Angle, Ab= 0.00 Batter Factor, Kbat= 1.00

GROUP PILES:

Group Configuration:

Fixed Head

Average Pile Diameter= 1.80 -m

Sx= 250 -cm

Sy= 625 -cm

Nx= 1 Ny= 4

1. Single Pile Vertical Analysis (in Group):

Vertical Load= 10902.00 -kN

Results:

Total Ultimate Capacity (Down)= 56329.80-kN, Total Ultimate Capacity (Up)= 5930.97-kN

Total Allowable Capacity (Down)= 19304.51-kN, Total Allowable Capacity (Up)= 3668.49-kN

At Work Load= 10902.00-kN, Settlement= 1.332-cm

At Work Load= 10902.00-kN, Secant Stiffness Kqx= 8186.18-kN/-cm

At Allowable Settlement= 2.500-cm, Capacity= 16763.35-kN

Work Load, 10902.00-kN, OK with the Capacity at Allowable Settlement= 2.50-cm, Capacity= 16763.35-kN

Work Load, 10902.00-kN, OK with the Allowable Capacity (Down)= 19304.51-kN

2. Group Pile Vertical Analysis (in Group):

Vertical Load= 43608.00 -kN

Results:

Total Ultimate Capacity (Down)= 225319.22-kN, Total Ultimate Capacity (Up)= 23723.89-kN

Total Allowable Capacity (Down)= 76333.38-kN, Total Allowable Capacity (Up)= 14673.95-kN

At Work Load= 43608.00-kN, Settlement= 1.33176-cm

At Work Load= 43608.00-kN Secant Stiffness Kqx= 32744.73-kN/-cm

At Allowable Settlement= 2.500-cm, Capacity= 67053.38-kN

Work Load, 43608.00-kN, OK with the Capacity at Allowable Settlement= 2.50-cm, Capacity= 67053.38-kN

Work Load, 43608.00-kN, OK with the Allowable Capacity (Down)= 76333.38-kN

FACTOR OF SAFETY:

FSside		FStip	FSuplif	FSweight
2.5	3.0	2.0	1.0	

Note: If program can't find result or the result exceeds the up limits. The result shows 9999.

****

ALLPILE 6
LATERAL ANALYSIS SUMMARY OUTPUT
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Date: 2019/08/07 File: C:\Users\Eglaal\Desktop\ANALSYS DATA\FIXED SUPPORT\DIA 1800 mm\DIA 1.8 L=30\DIA 1.8 spacing 6.25m\DIA180 COPM LC05.alp
Title 1: PILE dia 1.80 m LC05
Title 2: THE PILE ANALYSIS
PILE PROFILES:
Pile Length, L= 35.9 -m
Top Height, H= 5.9 -m
Slope Angle, As= 0
Batter Angle, Ab= 0.00
TOTAL LOADS:
Vertical Load, Q: 41191.0 -kN
Moment, M: 29735.0 -kN-m
Torsion, T: 0.0 -kN
Shear Load, P: 0.0 -kN
FACTORS AND CONDITIONS:
Load Factor for Vertical Loads: 1.0
Load Factor for Lateral Loads: 1.0
Loads Supported by Pile Cap: 0 %
Shear Condition: Static
GROUP PILE FOUNDATION:
Group Configuration:
Head Condition: Fixed Head (Cap with Restrained Connection)
Average Pile Diameter= 1.80 -m
Column Number, Nx= 1
Row Number, Ny= 4
Row Spacing, Sy= 6.25 -m
==================
Rt2= 52.08
RMAX= 1.50

rx2/RMAX= 52.08
===================

Y-Direction (Lateral Loading in Y Direction)
****

A. Back Pile, Critical Loading for Back Pile:
Deduction factor due to side effect, Rside= 0.45
Deduction factor due to front effect, Rfront= 0.44

1.Vertical Analysis:
Try Vertical Load= 10294.78 -kN
Try Results:
Total Ultimate Capacity (Down)= 56329.80-kN, Total Ultimate Capacity (Up)= 5930.97-kN
Total Allowable Capacity (Down)= 19304.51-kN, Total Allowable Capacity (Up)= 3668.49-kN
At Work Load= 10294.78-kN, Settlement= 1.23311-cm
Work Load, 10294.78-kN, OK with the Allowable Capacity (Down)= 19304.51-kN

2.Lateral Analysis:
Try Shear= 0.00 -kN
Deduction factor due to Group Effect, R= 0.20
Fixed Head Condition
Try Results:
Top Deflection, yt= 0.00145-cm
Max. Moment, M= 3.21-kN-m
Top Deflection Slope, St= 0.00000

B. Front Pile, Critical Loading for Front Pile:
Deduction factor due to side effect, Rside= 0.45
Deduction factor due to front effect, Rfront= 0.44

3.Vertical Analysis:
Try Vertical Load= 11725.03 -kN
Try Results:
Total Ultimate Capacity (Down)= 56329.80-kN, Total Ultimate Capacity (Up)= 5930.97-kN
Total Allowable Capacity (Down)= 19304.51-kN, Total Allowable Capacity (Up)= 3668.49-kN
At Workload= 11725.03-kN, Settlement= 1.47383-cm
Workload, 11725.03-kN, OK with the Allowable Capacity (Down)= 19304.51-kN

4.Lateral Analysis:
Try Shear= 0.00 -kN
Deduction factor due to Group Effect, R= 0.45
Fixed Head Condition

Try Results:
Top Deflection, yt= 0.00109-cm
Max. Moment, M= 2.93-kN-m
Top Deflection Slope, St= 0.00000

C. Final Results & Summary:
Max. Cap Settlement, Xmax= 1.47383-cm
Average Cap Settlement, Xaverage= 1.35347-cm
Differential Cap Settlement, Xdiff= 0.24072-cm
Cap Rotation, Rt= 0.005777238 Slope
Cap Rotation, Ra= 0.33100766 Degree
Lateral Cap Movement (Deflection), yt= 0.000-cm
Front Pile: Shear= 0.00-kN
Back Pile: Shear= 0.00-kN
Lateral Cap Movement, yt= 0.000-cm, OK with the Allowable Deflection= 2.500-cm
Max. Cap Settlement, Xmax= 1.474-cm, OK with the Allowable Deflection= 2.500-cm

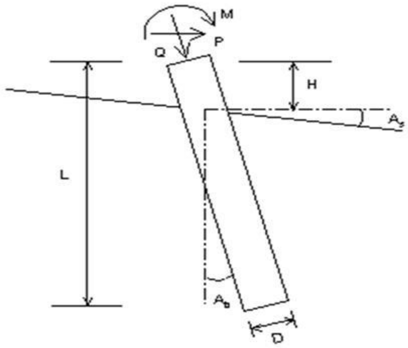
X-Direction (Lateral Loading in X Direction)
****
Nx<2, No Calculation. Please run single or tower pile analysis.
Note: If program can't find result or the result exceeds the up limits. The result shows 9999________________________________

Notes:
Q - Vertical Load at pile top
P - Lateral Shear Load at pile top
M - Moment at pile top
Xall - Pile top total settlement
yt - Pile top deflection
St - Pile top deflection slope (deflection/unit length)

The Max. Moment calculated by program is an internal moment of shaft due to the loading. Egineers
have to check whether the pile has enough moment capacity to resist the Max. Moment with adequate
factor of safety. If not, the pile may be damaged under the loading.

VERTICAL ANALYSIS

Figure 1



Drilled Pile (dia <=24 in. or 61 cm)

Loads:
Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

Vertical Load, Q= 43608.0 -kN
Shear Load, P= 0.0 -kN
Moment, M= 0.0 -kN-m

Profile:
Pile Length, L= 35.9 -m
Top Height, H= 5.9 -m
Slope Angle, As= 0
Batter Angle, Ab= 0

Soil Data:

Depth -m	Gamma -kN/m3	Phi	C -kN/m2	K -MN/m3	e50 or Dr %	Nspt
0	8.0	21.0	0.0	10.0	20	21
3.9	8.0	21.0	0.0	10.0	20	21
6	9.0	30.0	0.0	20.0	50	50
9	9.0	40.0	0.0	40.0	90	50
26	9.0	40.0	0.0	40.0	90	50

Pile Data:

Depth -m	Width -cm	Area -cm2	Per. -cm	I -cm4	E -MP	Weight -kN/m
0.0	180	25446.9	565.5	51529972.0	20683	60.004
35.9	180	25446.9	565.5	51529972.0	20683	60.004

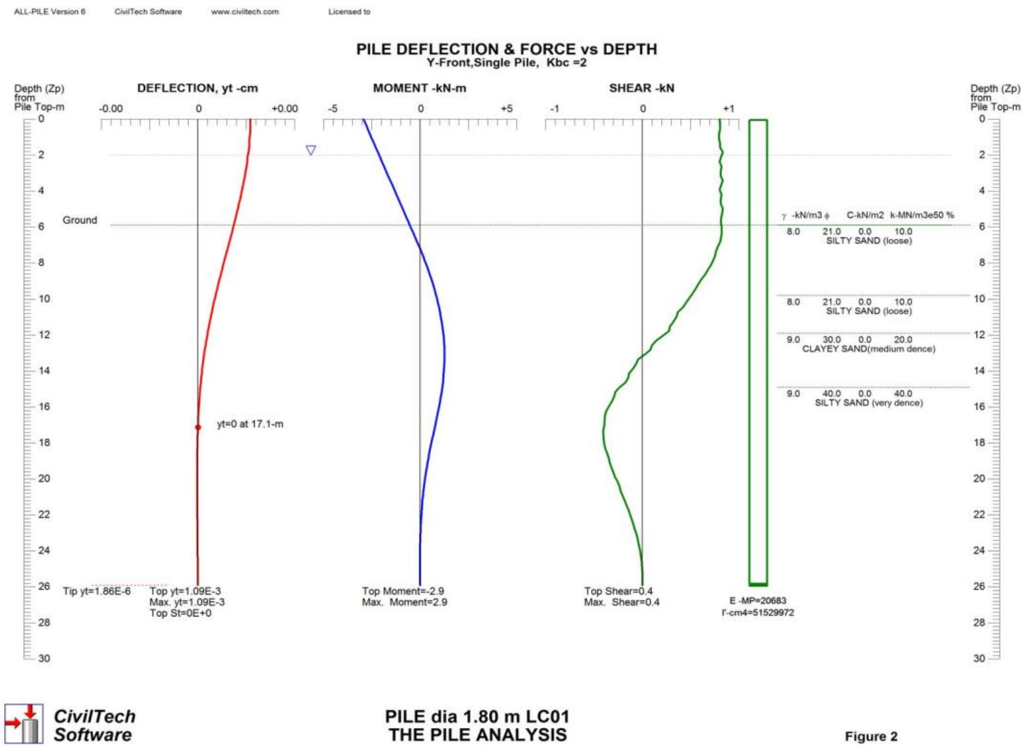
Vertical capacity:
Weight above Ground= 354.02 Total Weight= 1406.00-kN *Soil Weight is not included
Side Resistance (Down)= 7918.695-kN Side Resistance (Up)= 4524.974-kN
Tip Resistance (Down)= 48411.109-kN Tip Resistance (Up)= 0.000-kN
Total Ultimate Capacity (Down)= 56329.805-kN Total Ultimate Capacity (Up)= 5930.974-kN
Total Allowable Capacity (Down)= 19304.514-kN Total Allowable Capacity (Up)= 3668.487-kN
N/G! Qallow < Q

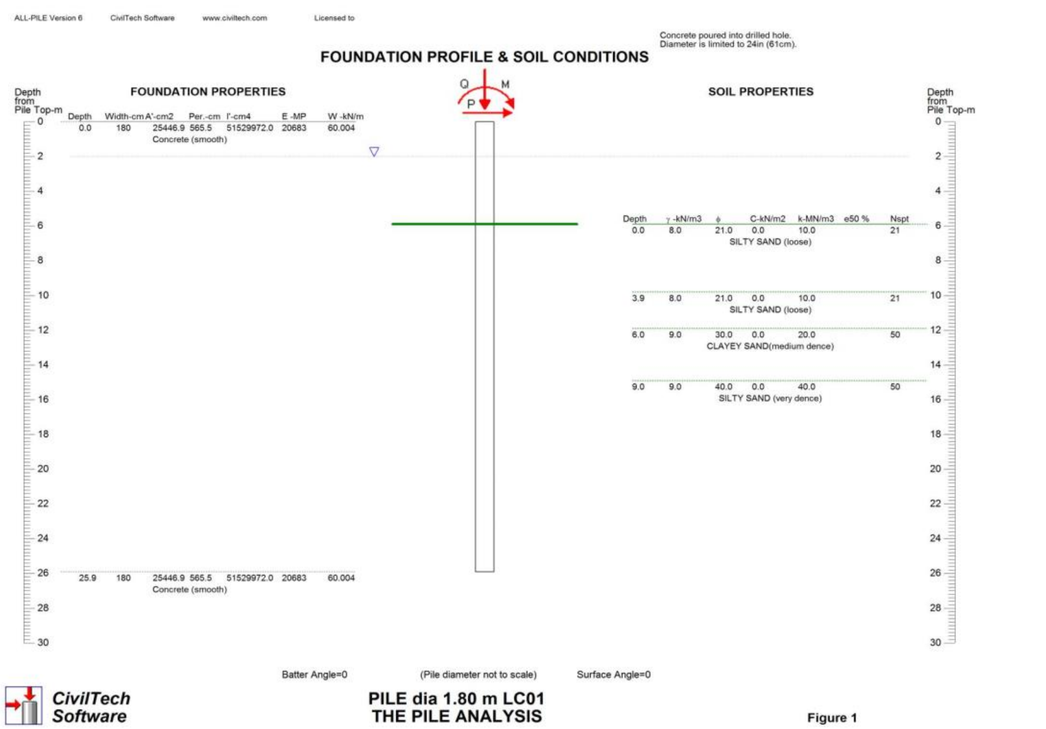
Settlement Calculation:
At Q= 43608.00-kN Settlement= 10.91451-cm
At Xallow= 2.50-cm Qallow= 16763.34570-kN

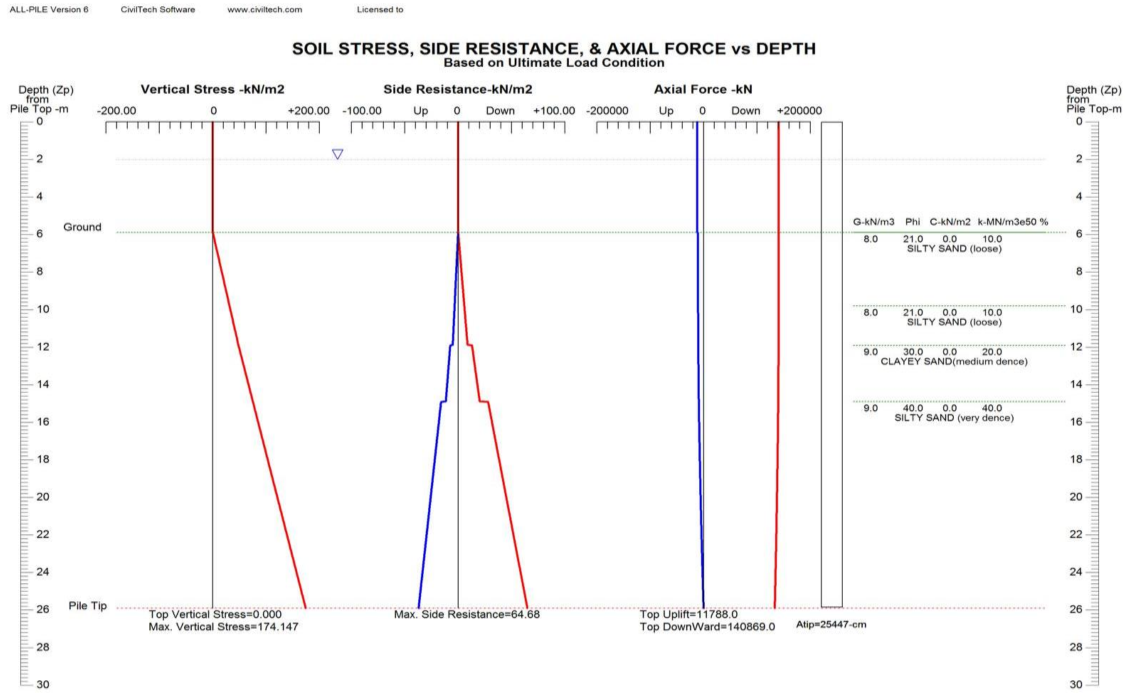
Note: If program can't find result or the result exceeds the up limits. The result shows 9999.

CivilTech Software

PILE dia 1.80 m LC01
THE PILE ANALYSIS


ALL-PILE Version 6 CivilTech Software www.civiltech.com Licensed to
PILE DEFLECTION & FORCE vs DEPTH
Y-Front,Single Pile, Kbc =2
Depth (Zp) from Pile Top-m
DEFLECTION, yt -cm
MOMENT -kN-m
SHEAR -kN
Ground
yt=0 at 17.1-m
Tip yt=1.86E-6
Top yt=1.09E-3
Max. yt=1.09E-3
Top St=0E+0
Top Moment=-2.9
Max. Moment=2.9
Top Shear=0.4
Max. Shear=0.4
E -MP=20683
I'-cm4=51529972
SILTY SAND (loose)
CLAYEY SAND(medium dence)
SILTY SAND (very dence)
CivilTech Software
PILE dia 1.80 m LC01
THE PILE ANALYSIS
Figure 2


ALL-PILE Version 6 CivilTech Software www.civiltech.com Licensed to
Concrete poured into drilled hole.
Diameter is limited to 24in (61cm).
FOUNDATION PROFILE & SOIL CONDITIONS
FOUNDATION PROPERTIES
SOIL PROPERTIES
Depth from Pile Top-m
Concrete (smooth)
SILTY SAND (loose)
CLAYEY SAND(medium dence)
SILTY SAND (very dence)
Batter Angle=0
(Pile diameter not to scale)
Surface Angle=0
CivilTech Software
PILE dia 1.80 m LC01
THE PILE ANALYSIS
Figure 1



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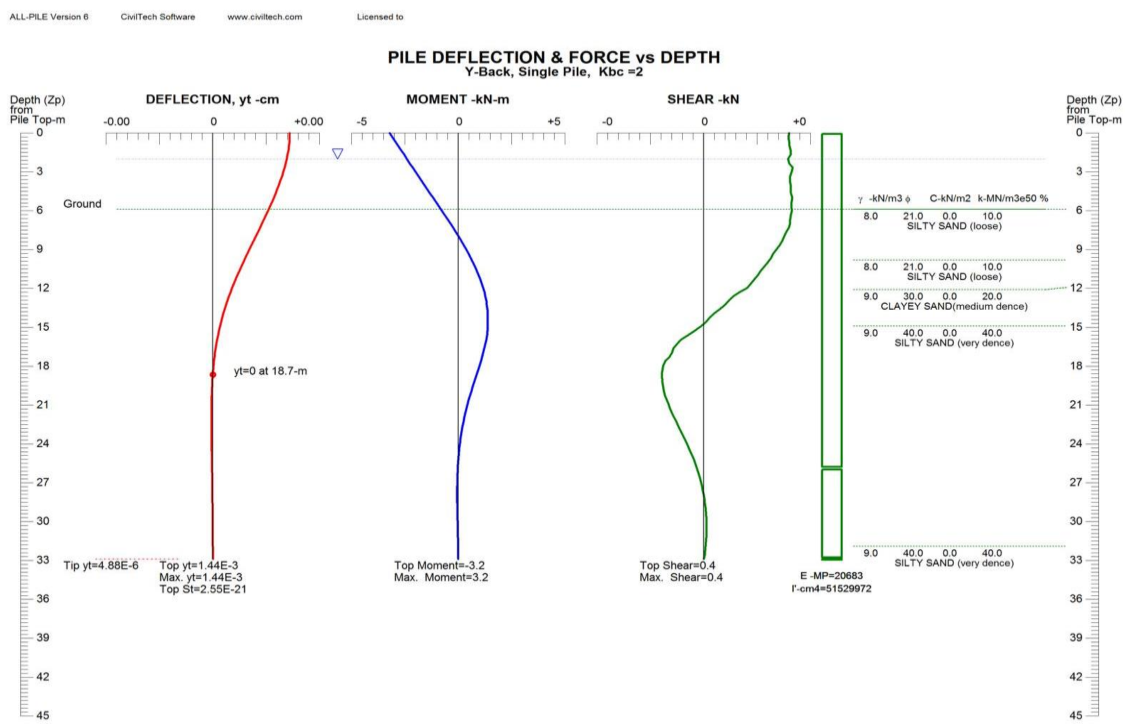
PILE dia 1.80 m LC01
THE PILE ANALYSIS

Figure 1



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PILE dia 1.80 m LC05
THE PILE ANALYSIS

Figure 2

capacity of pile manual calculate									
Qu =	Qp+Qs								
Qp =	qp*Ap	13075							
Qs =	Ki*σi* tan δ′								
depth	20 m	Diameter	1.2 m						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	98.1693
2	3	30	9	0.5	67.5	22.5	0.414	13.97	158.061
3	11	40	9	0.357	130.5	30	0.577	26.88	1115.14
									1371.37
	Qu =	14446							
	Qall 1=	7451.6							
	Qall2 =	5272.5							
	Qall3 =	5778.5							
Qp =	qp*Ap	17796							
depth =	20 m	Diameter	1.4 m						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	114.531
2	3	30	9	0.5	67.5	22.5	0.414	13.97	184.404
3	11	40	9	0.357	130.5	30	0.577	26.88	1300.99
									1599.93
	Qu =	19396							
	Qall 1=	9964.7							
	Qall2 =	6998.7							
	Qall3 =	7758.5							
Qp =	qp*Ap	23244							
depth	20 m	Diameter	1.6 m						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	130.892
2	3	30	9	0.5	67.5	22.5	0.414	13.97	210.785
3	11	40	9	0.357	130.5	30	0.577	26.88	1486.93
									1828.61
	Qu =	25073							
	Qall 1=	12841							
	Qall2 =	8967.1							
	Qall3 =	10029							

Qu =	Qp+Qs								
Qp =	qp*Ap	29418							
Qs =	Ki*σi* tan δ′								
depth	20 m	Diameter	1.8 m						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	147.254
2	3	30	9	0.5	67.5	22.5	0.414	13.97	237.133
3	11	40	9	0.357	130.5	30	0.577	26.88	1672.8
									2057.19
	Qu =	31475							
	Qall 1=	16081							
	Qall2 =	11178							
	Qall3 =	12590							
Qp =	qp*Ap	13075							
Qs =	Ki*σi* tan δ′								
depth =	27m	Diameter	1.2 m						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	98.1693
2	3	30	9	0.5	67.5	22.5	0.414	13.97	158.089
3	18	40	9	0.357	162	30	0.577	33.37	2265.36
									2521.62
	Qu =	15596							
	Qall 1=	8218.5							
	Qall2 =	6039.3							
	Qall3 =	6238.6							
Qp =	qp*Ap	17796							
depth	27m	Diameter	1.4						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	114.531
2	3	30	9	0.5	67.5	22.5	0.414	13.97	184.437
3	18	40	9	0.357	162	30	0.577	33.37	2642.92
									2941.89
	Qu =	20738							
	Qall 1=	10859							
	Qall2 =	7893.3							
	Qall3 =	8295.3							
Qp =	qp*Ap	23244							
depth	27m	Diameter	1.6						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs

1	6	21	8	0.641	24	15.75	0.282	4.338	130.892
2	3	30	9	0.5	67.5	22.5	0.414	13.97	210.785
3	18	40	9	0.357	162	30	0.577	33.37	3020.48
									3362.16
	Qu =	26606							
	Qall 1=	13863							
	Qall2 =	9989.5							
	Qall3 =	10642							
Qu =	Qp+Qs								
Qp =	qp*Ap	29418							
Qs =	Ki*σi* tan δ′								
depth	27m	Diameter	1.8						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	147.254
2	3	30	9	0.5	67.5	22.5	0.414	13.97	237.133
3	18	40	9	0.357	162	30	0.577	33.37	3398.04
									3782.43
	Qu =	33201							
	Qall 1=	17231							
	Qall2 =	12328							
	Qall3 =	13280							
Qp =	qp*Ap	13075							
depth	30	Diameter	1.2						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	98.1693
2	3	30	9	0.5	67.5	22.5	0.414	13.97	158.089
3	21	40	9	0.357	175.5	30	0.577	36.15	2863.16
									3119.42
	Qu =	16194							
	Qall 1=	8617							
	Qall2 =	6437.9							
	Qall3 =	6477.7							
Qp =	qp*Ap	17796							
depth	30	Diameter	1.4						
no	depth	ϕ	ϒ	k	σ	δ′	tan δ′	fs	Qs
1	6	21	8	0.641	24	15.75	0.282	4.338	114.531
2	3	30	9	0.5	67.5	22.5	0.414	13.97	184.437
3	21	40	9	0.357	175.5	30	0.577	36.15	3340.36

Table of MANUAL CALCULATION FOR VERTICAL LOAD 1X4 GROUP PILE

		NUMBER	MOMENT		SPACE	Ix			NUMBER	MOMENT		SPACE	Ix
LC01		4	10427	1.5	6.25	195.3	LC05		4	29735	1.5	6.25	195.3
FRONT	11402			0.5			**FRONT**	11725			0.5		
							VER load	41191					
													139
VER load	43608								4	29735	1.5	6.25	195.3
							BACK	**9822**			0.5		
		4	10427	1.5	6.25	195.3		41191					
BACK	10735			0.5									
	43608												
		NUMBER	MOMENT		SPACE	Ix			NUMBER	MOMENT		SPACE	Ix
LC11		4	11621	1.5	6.25	195.3	LC18		4	21877	1.5	6.25	195.3
FRONT	11026			0.5			**FRONT**	10110			0.5		
VER load	41874						VER load	36241					
						140							
		4	11621	1.5	6.25	195.3							
BACK	10283			0.5					4	21877	1.5	6.25	195.3
	41874						**BACK**	8710			0.5		
								36241					

Table of MANUAL CALCULATION FOR VERTICAL LOAD 2X4 GROUP PILE

2X4		NUMBER	MOMENT		SPACE	Ix	IY	2X4		NUMBER	MOMENT		SPACE	Ix	IY
LC01		8	10427	1.5	6.25	390.6	25.92	LC05		8	29735	1.5	6.25	390.6	25.92
P1	6253.6		3271	0.5	3.6			P1	7287.94		3089	0.5	3.6		
P2	6096.59							P2	7139.67						
P3	4648.4							P3	3009.81						
P4	4805.41							P4	3158.08						
P5	6201.27							P5	7238.52						
P6	6148.93							P6	7189.09						
P7	4753.07							P7	3108.66						
P8	4726.9							P8	3083.94						
VER load	43608	KN						VER load	41191	KN					
2X4		NUMBER	MOMENT		SPACE	Ix	IY	2X4		NUMBER	MOMENT		SPACE	Ix	IY
LC11		8	11621	1.5	6.25	390.6	25.92	LC18		8	21877	1.5	6.25	390.6	25.92
P1	6227.19		7747	0.5	3.6			P1	6114.59		2718	0.5	3.6		
P2	5855.34							P2	5984.13						
P3	4241.31							P3	2945.66						
P4	4241.31							P4	3076.12						
P5	6103.24							P5	6071.11						
P6	5979.29							P6	6027.62						
P7	4320.14							P7	3032.63						
P8	4489.21							P8	3010.89						
VER load	41874	KN						VER load	36241	KN					

manual pile group destibuted vertical load for group (3x4) for diameter1.2m

2X4		NUMBE	MOMENT	SPACE	SPACE	Ix	IY
LC01		12	10427	1.5	6.25	585.938	103.68
P1	4048.38		3271	0.5	3.6		
P2	3943.71			0			
P3	3219.62			1			
P4	3324.29						
P5	3686.34						
P6	3581.66						
P7	3616.55						
P8	3651.45						
P9	3849.91						
P10	3780.13						
P11	3254.51						
P12	3289.4						
VER load	43608						

2X4		NUMBE	MOMENT	SPACE	SPACE	Ix	IY
LC05		12	29735	1.5	6.25	585.9	103.68
P1	4514.47		3089	0.5	3.6		
P2	4415.62		0	0			
P3	2350.69			1			
P4	2449.54						
P5	3482.01						
P6	3383.16						
P7	3449.06						
P8	3449.06						
P9	3915.87						
P10	3915.87						
P11	2383.64						
P12	2416.59						
VER load	41191						

2X4		NUMBE	MOMENT	SPACE	SPACE	Ix	IY
LC11		12	11621	1.5	6.25	585.938	103.68
P1	4016.96		7747	0.5	3.6		
P2	3769.05			0			
P3	2962.04			1			
P4	3209.95						
P5	3613.45						
P6	3365.55						
P7	3530.82						
P8	3530.82						
P9	3608.62						
P10	3608.62						
P11	3044.68						
P12	3127.31						
VER load	**41874**						

2X4		NUMBE	MOMENT	SPACE	SPACE	Ix	IY
LC18		12	21877	1.5	6.25	585.9	103.68
P1	3823.19		2718	0.5	3.6		
P2	3736.21			0			
P3	2216.98			1			
P4	2303.95						
P5	3063.57						
P6	2976.6						
P7	3005.59						
P8	3034.58						
P9	3428.88						
P10	3370.9						
P11	2245.97						
P12	2274.96						
VER load	**36241**						

BrinchHasen method spraed sheet for diameter 1.2m

Appendix A Sample Results
BrinchHasen method spraed sheet for diameter 1.2m

Height of application above top of pile	5.9	m
Length of pile	20	m
Pile diameter	1.2	m
Depth about which rotation of pile occurs *(use negative numbers)*	-15.1	m
Number of slices **top soil**	10	
Depth of top soil	9	m
Number of slices **bottom soil** (max 10)	10	
Soil Properties — **Upper soil**		
Phi	21	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	8	kN/m^3
Bottom soil		
Phi	40	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	9	kN/m^3
Height of each element in top soil	0.9	
Height of each element in bottom soil	1.1	

PRESS TO SOLVE

Resistance to Horizontal Force = 7664.9 kN

	Slice Number	Top level of slice	Bottom level of slice	Z	Z/B	Kq	Poz	Kq.Poz	Kc	c'	c'.Kc	Pz	Pu	arm	M	M
Upper Soil	1	0	0.9	0.45	0.38	3.044343	3.6	11	5.11085618	0	0	11	11.836407	-14.7	173.6321068	173.6321068
	2	0.9	1.8	1.35	1.13	3.4125	10.8	37	10.6279945	0	0	37	39.8034	-13.8	548.0659563	548.0659563
	3	1.8	2.7	2.25	1.88	3.7275	18.0	67	13.8247676	0	0	67	72.4626	-12.9	932.54476	932.54476
	4	2.7	3.6	3.15	2.63	3.9925	25.2	101	15.466302	0	0	101	108.65988	-12.0	1300.585434	1300.585434
	5	3.6	4.5	4.05	3.38	4.225	32.4	137	16.7747761	0	0	137	147.8412	-11.1	1636.502312	1636.502312
	6	4.5	5.4	4.95	4.13	4.43125	39.6	175	18.0151842	0	0	175	189.5157	-10.2	1927.246773	1927.246773
	7	5.4	6.3	5.85	4.88	4.61875	46.8	216	19.0253242	0	0	216	233.4501	-9.3	2163.924881	2163.924881
	8	6.3	7.2	6.75	5.63	4.77125	54.0	258	19.7863592	0	0	258	278.2593	-8.4	2328.842555	2328.842555
	9	7.2	8.1	7.65	6.38	4.9025	61.2	300	20.4975733	0	0	300	324.03564	-7.5	2420.327553	2420.327553
	10	8.1	9	8.55	7.13	5.01825	68.4	343	21.1699279	0	0	343	370.70816	-6.6	2435.302462	2435.302462
Bottom Soil	11	9	10.1	9.55	7.96	26.91667	86.0	2313	118.137011	0	0	2313	3053.8035	-5.6	17007.62461	17007.62461
	12	10.1	11.2	10.65	8.88	27.875	95.9	2672	125.681882	0	0	2672	3526.8008	-4.5	15762.41926	15762.41926
	13	11.2	12.3	11.75	9.79	28.51458	105.8	3015	133.226753	0	0	3015	3980.3507	-3.4	13411.09564	13411.09564
	14	12.3	13.4	12.85	10.71	29.60625	115.7	3424	139.223031	0	0	3424	4519.6309	-2.3	10256.51206	10256.51206
	15	13.4	14.5	13.95	11.63	30.625	125.6	3845	144.76384	0	0	3845	5075.3588	-1.2	5934.744588	5934.744588
	16	14.5	15.6	15.05	12.54	31.8125	135.5	4309	150.30465	0	0	4309	5687.8841	-0.1	394.3133708	394.3133708
	17	15.6	16.7	16.15	13.46	32.95833	145.4	4790	155.845459	0	0	4790	6323.4518	1.0	6517.422739	6517.422739
	18	16.7	17.8	17.25	14.38	34.0625	155.3	5288	161.386268	0	0	5288	6980.4281	2.1	14873.02271	14873.02271
	19	17.8	18.9	18.35	15.29	35.29167	165.2	5828	165.975291	0	0	5828	7693.5128	3.2	24855.23822	24855.23822
	20	18.9	20	19.45	16.21	36.20833	175.1	6338	168.524771	0	0	6338	8366.5148	4.3	36232.65508	36232.65508
															161112.0231	161112.0231

BrinchHasen method spraed sheet for diameter 1.4m

BrinchHasen method spraed sheet for diameter1.4m

Height of application above top of pile	5.9	m
Length of pile	20	m
Pile diameter	1.4	m
Depth about which rotation of pile occurs *(use negative numbers)*	-15.1	m
Number of slices **top soil**	10	
Depth of top soil	9	m
Number of slices **bottom soil** (max 10)	10	
Soil Properties — **Upper soil**		
Phi	21	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	8	kN/m^3
Bottom soil		
Phi	40	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	9	kN/m^3
Height of each element in top soil	0.9	
Height of each element in bottom soil	1.1	

PRESS TO SOLVE

Resistance to Horizontal Force = 8356.3 kN

	Slice Number	Top level of slice	Bottom level of slice	Z	Z/B	Kq	Poz	Kq.Poz	Kc	c'	c'.Kc	Pz	Pu	arm	M	M
Upper Soil	1	0	0.9	0.45	0.32	3.017287157	3.6	11	4.684	0	0	11	13.68641455	-14.7	200.770465	200.770465
	2	0.9	1.8	1.35	0.96	3.341962482	10.8	36	9.81	0	0	36	45.47742545	-13.8	626.1934577	626.1934577
	3	1.8	2.7	2.25	1.61	3.615	18.0	65	12.68	0	0	65	81.9882	-12.9	1055.132804	1055.132804
	4	2.7	3.6	3.15	2.25	3.865	25.2	97	14.8	0	0	97	122.72148	-12.0	1468.893296	1468.893296
	5	3.6	4.5	4.05	2.89	4.083571429	32.4	132	15.94	0	0	132	166.70772	-11.1	1845.341956	1845.341956
	6	4.5	5.4	4.95	3.54	4.27	39.6	169	17.05	0	0	169	213.05592	-10.2	2166.634924	2166.634924
	7	5.4	6.3	5.85	4.18	4.444642857	46.8	208	18.09	0	0	208	262.0917	-9.3	2429.413184	2429.413184
	8	6.3	7.2	6.75	4.82	4.605357143	54.0	249	18.95	0	0	249	313.3485	-8.4	2622.515479	2622.515479
	9	7.2	8.1	7.65	5.46	4.740071429	61.2	290	19.63	0	0	290	365.516388	-7.5	2730.160746	2730.160746
	10	8.1	9	8.55	6.11	4.860714286	68.4	332	20.24	0	0	332	418.9158	-6.6	2751.994097	2751.994097
Bottom Soil	11	9	10.1	9.55	6.82	24.82142857	86.0	2133	108.3	0	0	2133	3285.43875	-5.6	18297.67663	18297.67663
	12	10.1	11.2	10.65	7.61	26.21428571	95.9	2513	115.2	0	0	2513	3869.4645	-4.5	17293.89497	17293.89497
	13	11.2	12.3	11.75	8.39	27.39285714	105.8	2897	121.7	0	0	2897	4461.06375	-3.4	15030.77425	15030.77425
	14	12.3	13.4	12.85	9.18	28.11607143	115.7	3252	128.2	0	0	3252	5007.500438	-2.3	11363.64664	11363.64664
	15	13.4	14.5	13.95	9.96	28.62678571	125.6	3594	134.6	0	0	3594	5534.903138	-1.2	6472.101393	6472.101393
	16	14.5	15.6	15.05	10.75	29.6625	135.5	4018	139.5	0	0	4018	6187.389863	-0.1	428.9416766	428.9416766
	17	15.6	16.7	16.15	11.54	30.53571429	145.4	4438	144.2	0	0	4438	6835.08375	1.0	7044.748978	7044.748978
	18	16.7	17.8	17.25	12.32	31.48214286	155.3	4888	149	0	0	4888	7526.908125	2.1	16037.39391	16037.39391
	19	17.8	18.9	18.35	13.11	32.60714286	165.2	5385	153.7	0	0	5385	8293.00725	3.2	26792.01003	26792.01003
	20	18.9	20	19.45	13.89	33.39285714	175.1	5845	158.5	0	0	5845	9001.94625	4.3	38984.5023	38984.5023
															175642.7412	175642.7412

BrinchHasen method spraed sheet for diameter 1.6m

BrinchHasen method spraed sheet for dimater 1.6m

Height of application above top of pile	5.9	m
Length of pile	20	m
Pile diameter	1.6	m
Depth about which rotation of pile occurs *(use negative numbers)*	-15.1	m
Number of slices **top soil**	10	
Depth of top soil	9	m
Number of slices **bottom soil** (max 10)	10	

PRESS TO SOLVE

Soil Properties

Upper soil		
Phi	21	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	8	kN/m^3

Resistance to Horizontal Force = 9039.5 kN

Bottom soil		
Phi	40	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	9	kN/m^3

Height of each element in top soil	0.9
Height of each element in bottom soil	1.1

	Slice Number	Top level of slice	Bottom level of slice	Z	Z/B	Kq	Poz	Kq.Poz	Kc	c'	c'.Kc	Pz	Pu	arm	M	M
Upper Soil	1	0	0.9	0.45	0.28	2.9969949	3.6	11	4.3632048	0	0	11	15.53642	-14.7	227.9088232	227.90882
	2	0.9	1.8	1.35	0.84	3.2810859	10.8	35	8.8491133	0	0	35	51.02745	-13.8	702.6135126	702.61351
	3	1.8	2.7	2.25	1.41	3.530625	18.0	64	11.826784	0	0	64	91.5138	-12.9	1177.720847	1177.7208
	4	2.7	3.6	3.15	1.97	3.766875	25.2	95	14.224364	0	0	95	136.6924	-12.0	1636.115301	1636.1153
	5	3.6	4.5	4.05	2.53	3.960625	32.4	128	15.299991	0	0	128	184.7869	-11.1	2045.466499	2045.4664
	6	4.5	5.4	4.95	3.09	4.14625	39.6	164	16.292353	0	0	164	236.4358	-10.2	2404.392118	2404.3921
	7	5.4	6.3	5.85	3.66	4.30375	46.8	201	17.257199	0	0	201	290.0383	-9.3	2688.459492	2688.4594
	8	6.3	7.2	6.75	4.22	4.4546875	54.0	241	18.141452	0	0	241	346.3965	-8.4	2899.104936	2899.1049
	9	7.2	8.1	7.65	4.78	4.5953125	61.2	281	18.899057	0	0	281	404.9757	-7.5	3024.895178	3024.8951
	10	8.1	9	8.55	5.34	4.7166875	68.4	323	19.519654	0	0	323	464.5749	-6.6	3051.943255	3051.9432
Bottom Soil	11	9	10.1	9.55	5.97	23.9375	86.0	2057	98.917759	0	0	2057	3621.074	-5.6	20166.93568	20166.935
	12	10.1	11.2	10.65	6.66	24.65625	95.9	2363	106.47205	0	0	2363	4159.411	-4.5	18589.75904	18589.759
	13	11.2	12.3	11.75	7.34	25.6875	105.8	2716	113.07852	0	0	2716	4780.958	-3.4	16108.6003	16108.60
	14	12.3	13.4	12.85	8.03	27.03125	115.7	3126	118.73717	0	0	3126	5502.049	-2.3	12485.93756	12485.937
	15	13.4	14.5	13.95	8.72	27.71875	125.6	3480	124.39582	0	0	3480	6124.957	-1.2	7162.065917	7162.0659
	16	14.5	15.6	15.05	9.41	28.264063	135.5	3828	130.05448	0	0	3828	6737.926	-0.1	467.107699	467.1076
	17	15.6	16.7	16.15	10.09	28.776563	145.4	4183	135.50817	0	0	4183	7361.505	1.0	7587.318242	7587.318
	18	16.7	17.8	17.25	10.78	29.704688	155.3	4612	139.66378	0	0	4612	8116.509	2.1	17293.64127	17293.64
	19	17.8	18.9	18.35	11.47	30.46875	165.2	5032	143.81938	0	0	5032	8856.169	3.2	28611.40172	28611.40
	20	18.9	20	19.45	12.16	31.234375	175.1	5468	147.97499	0	0	5468	9622.936	4.3	41673.80754	41673.80
															190005.1949	190005.1

BrinchHasen method spraed sheet for diameter 1.8m

BrinchHasen method spraed sheet for diametere 1.8m

Height of application above top of pile	5.9	m
Length of pile	20	m
Pile diameter	1.8	m
Depth about which rotation of pile occurs *(use negative numbers)*	-15.1	m
Number of slices **top soil**	10	
Depth of top soil	9	m
Number of slices **bottom soil** (max 10)	10	

PRESS TO SOLVE

Soil Properties

Upper soil		
Phi	21	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	8	kN/m^3
Bottom soil		
Phi	40	degrees
c'	0	kN/m^2
rate of change of c'	0	kN/m^2/m
Gamma	9	kN/m^3

Resistance to Horizontal Force = 9711.5 kN

Height of each element in top soil 0.9
Height of each element in bottom soil 1.1

	Slice Number	Top level of slice	Bottom level of slice	Z	Z/B	Kq	Poz	Kq.Poz	Kc	c'	c'.Kc	Pz	Pu	arm	M	M
Upper Soil	1	0	0.9	0.45	0.25	2.981212	3.6	11	4.113988	0	0	11	17.386429	-14.7	255.0471814	255.0471814
	2	0.9	1.8	1.35	0.75	3.233737	10.8	35	8.101462	0	0	35	56.577469	-13.8	779.0335676	779.0335676
	3	1.8	2.7	2.25	1.25	3.465	18.0	62	11.16079	0	0	62	101.0394	-12.9	1300.308891	1300.308891
	4	2.7	3.6	3.15	1.75	3.675	25.2	93	13.29197	0	0	93	150.0282	-12.0	1795.736306	1795.736306
	5	3.6	4.5	4.05	2.25	3.865	32.4	125	14.80106	0	0	125	202.86612	-11.1	2245.591042	2245.591042
	6	4.5	5.4	4.95	2.75	4.035	39.6	160	15.68805	0	0	160	258.85332	-10.2	2632.363575	2632.363575
	7	5.4	6.3	5.85	3.25	4.19	46.8	196	16.56037	0	0	196	317.66904	-9.3	2944.577619	2944.577619
	8	6.3	7.2	6.75	3.75	4.33	54.0	234	17.41801	0	0	234	378.7884	-8.4	3170.203279	3170.203279
	9	7.2	8.1	7.65	4.25	4.4625	61.2	273	18.18354	0	0	273	442.4301	-7.5	3304.654269	3304.654269
	10	8.1	9	8.55	4.75	4.5875	68.4	314	18.85697	0	0	314	508.3317	-6.6	3339.396217	3339.396217
Bottom Soil	11	9	10.1	9.55	5.31	22.61111	86.0	1943	91.63054	0	0	1943	3847.9815	-5.6	21430.66011	21430.66011
	12	10.1	11.2	10.65	5.92	23.83333	95.9	2284	98.34546	0	0	2284	4523.1615	-4.5	20215.47941	20215.47941
	13	11.2	12.3	11.75	6.53	24.52778	105.8	2594	105.0604	0	0	2594	5135.7488	-3.4	17304.00738	17304.00738
	14	12.3	13.4	12.85	7.14	25.27778	115.7	2923	111.3924	0	0	2923	5788.2825	-2.3	13135.495	13135.495
	15	13.4	14.5	13.95	7.75	26.5	125.6	3327	116.4223	0	0	3327	6587.6085	-1.2	7703.05624	7703.05624
	16	14.5	15.6	15.05	8.36	27.36111	135.5	3706	121.4522	0	0	3706	7338.0038	-0.1	508.708147	508.708147
	17	15.6	16.7	16.15	8.97	27.97222	145.4	4066	126.4821	0	0	4066	8050.2098	1.0	8297.148796	8297.148796
	18	16.7	17.8	17.25	9.58	28.37917	155.3	4406	131.512	0	0	4406	8723.6139	2.1	18587.18489	18587.18489
	19	17.8	18.9	18.35	10.19	28.9125	165.2	4775	136.1168	0	0	4775	9454.3008	3.2	30543.77178	30543.77178
	20	18.9	20	19.45	10.81	29.7375	175.1	5206	139.8107	0	0	5206	10306.988	4.3	44636.21277	44636.21277
															204128.6365	204128.6365

lateral load and moment manual distributionm
using equations 3.33 for and3.39

dia 1.8					
T= (EI/nh)^1/5		4.631925	1x4 group	shear	moment
nh)	Ec	I	for LC11	86	612.211
5000	20700000	0.515	for LC18	522.25	3717.76
zf	8.337464911				
h1	14.23746491				
dia 1.6					
T= (EI/nh)^1/5		4.1901637	1x4 group	shear	moment
nh)	EI		for LC11	86	578.019
5000	20700000	0.312	for LC18	522.25	3510.12
zf	7.542294605				
h1	13.44229461				
dia 1.4					
T= (EI/nh)^1/5		3.7884568	1x4 group	shear	moment
nh)	EI		for LC11	86	546.927
5000	20700000	0.1885	for LC18	522.25	3321.31
zf	6.819222237				
h1	12.71922224				
dia 1.2					
T= (EI/nh)^1/5		3.3439852	1x4 group	shear	moment
nh)	EI		for LC11	86	512.524
5000	20700000	0.101	for LC18	522.25	3112.39
zf	6.019173309				
h1	11.91917331				
dia 1.2					
T= (EI/nh)^1/5		3.3439852	2x4group	shear	moment
nh)	EI		for LC11	43	256.262
5000	20700000	0.101	for LC18	261.13	1556.2
zf	6.019173309				
h1	11.91917331				

dia 1.4			2x4 group	shear	moment
T= (EI/nh)^1/5		3.7884568	for LC11	43	273.463
nh)	EI		for LC18	261.13	1660.65
5000	20700000	0.1885			
zf	6.819222237				
h1	12.71922224				

Appendix B

Sample of results

River Pile Group

Time and Date of Analysis

Date: May 22, 2013 Time: 15:06:58

***** COMPUTATION RESULTS *****

SOBA BRIDGE-PILE GROUP ANALYSIS

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : Load Case 01
LOAD TYPE : Special, Sp

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

GROUP NO	P-FACTOR	Y-FACTOR
1	0.9772	1.0000
2	0.9549	1.0000
3	0.9549	1.0000
4	0.9772	1.0000

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, KN	HOR. LOAD Y, KN	HOR. LOAD Z, KN
43608.0	0.00000	0.00000

MOMENT X , M- KN	MOMENT Y, M- KN	MOMENT Z, M- KN
0.00000	10427.0	3271.00

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL , M HORIZONTAL Y, M HORIZONTAL Z, M

Page 1

River Pile Group

* PILE TOP REACTIONS *

PILE GROUP	AXIAL, KN	LAT. y, KN	LAT. z, KN	MOM x, KN- M	MOM y, KN- M	MOM z, KN- M	STRESS, KN/ M**2
**********	***********	***********	***********	***********	***********	***********	****************
1	1.1400E+04	-0.2417	-7.1986E-03	3.1472E-08	13.659	808.43	7372.9
2	1.1068E+04	0.2387	-7.7542E-03	3.1472E-08	13.726	816.55	7271.5
3	1.0736E+04	0.2477	-4.2881E-04	3.1472E-08	13.745	821.85	7160.1
4	1.0404E+04	-0.2447	1.5382E-02	3.1472E-08	13.714	824.18	7038.0
MINIMUM	1.0404E+04	-0.2447	-7.7542E-03	3.1472E-08	13.659	808.43	7038.0
Pile N.	4	4	2	1	1	1	4
MAXIMUM	1.1400E+04	0.2477	1.5382E-02	3.1472E-08	13.745	824.18	7372.9
Pile N.	1	3	4	1	3	4	1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR M	DEFLECTION z-DIR M	BENDING MOMENT z-DIR KN- M	BENDING MOMENT y-DIR KN- M	SHEAR FORCE y-DIR KN	SHEAR FORCE z-DIR KN	SOIL REACTION y-DIR KN/ M	SOIL REACTION z-DIR KN/ M	TOTAL STRESS KN/ M**2	FLEXURAL RIGIDITY z-DIR KN- M**2	FLEXURAL RIGIDITY y-DIR KN- M**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-2.2900E-02	-1.0300E-06	-1040.0	-0.5640	-11.300	-3.5100	-73.900	-0.9630	4480.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	19.880	28.000	0.0000	0.0000
2	-2.2800E-02	-1.0200E-06	-1040.0	-0.5600	-11.200	-3.5000	-73.500	-0.9620	4350.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	20.160	28.000	0.0000	0.0000
3	-2.2800E-02	-1.0200E-06	-1040.0	-0.5610	-11.100	-3.5000	-73.300	-0.9630	4220.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	20.160	28.000	0.0000	0.0000
4	-2.2800E-02	-1.0100E-06	-1040.0	-0.5630	-11.100	-3.5300	-73.500	-0.9750	4090.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	19.880	28.000	0.0000	0.0000
Min.	-2.2900E-02	-1.0300E-06	-1040.0	-0.5640	-11.300	-3.5300	-73.900	-0.9750	4090.0	7.0400E+06	7.0400E+06
Pile N.	1	1	1	1	1	4	1	4	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR M	DEFLECTION z-DIR M	BENDING MOMENT z-DIR KN- M	BENDING MOMENT y-DIR KN- M	SHEAR FORCE y-DIR KN	SHEAR FORCE z-DIR KN	SOIL REACTION y-DIR KN/ M	SOIL REACTION z-DIR KN/ M	TOTAL STRESS KN/ M**2	FLEXURAL RIGIDITY z-DIR KN- M**2	FLEXURAL RIGIDITY y-DIR KN- M**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-2.2900E-02	3.8300E-04	33.800	17.500	210.00	0.1880	57.700	1.2300	8220.0	7.0400E+06	7.0400E+06
x(M)	19.880	0.0000	23.520	0.0000	18.200	25.200	19.880	17.080	12.600	0.0000	0.0000
2	-2.2800E-02	3.8300E-04	33.500	17.400	209.00	0.1870	57.500	1.2300	8070.0	7.0400E+06	7.0400E+06
x(M)	0.0000	0.0000	23.520	0.0000	18.200	25.200	20.160	17.080	12.600	0.0000	0.0000
3	-2.2800E-02	3.8300E-04	33.400	17.400	209.00	0.1870	57.300	1.2300	7930.0	7.0400E+06	7.0400E+06
x(M)	0.0000	0.0000	23.520	0.0000	18.200	25.200	20.160	17.080	12.600	0.0000	0.0000
4	-2.2800E-02	3.8300E-04	33.400	17.500	209.00	0.1870	57.900	1.2400	7810.0	7.0400E+06	7.0400E+06
x(M)	0.0000	0.0000	23.520	0.0000	18.200	25.200	19.880	17.080	12.600	0.0000	0.0000
Max.	-2.2800E-02	3.8300E-04	33.800	17.500	210.00	0.1880	57.900	1.2400	8220.0	7.0400E+06	7.0400E+06
Pile N.	2	1	1	1	1	1	4	4	1	1	1

LOAD CASE : 2

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River Pile Group

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR M	DEFLECTION z-DIR M	BENDING MOMENT z-DIR KN- M	BENDING MOMENT y-DIR KN- M	SHEAR FORCE y-DIR KN	SHEAR FORCE z-DIR KN	SOIL REACTION y-DIR KN/ M	SOIL REACTION z-DIR KN/ M	TOTAL STRESS KN/ M**2	FLEXURAL RIGIDITY z-DIR KN- M**2	FLEXURAL RIGIDITY y-DIR KN- M**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-5.3300E-02	-1.0500E-06	-2330.0	-0.5650	-28.500	-3.2500	-163.00	-0.8310	4210.0	7.0400E+06	7.0400E+06
x(M)	0.0000	20.160	12.880	23.800	25.480	18.480	17.080	20.720	28.000	0.0000	0.0000
2	-5.3300E-02	-1.0500E-06	-2320.0	-0.5680	-28.700	-3.2300	-161.00	-0.8180	4080.0	7.0400E+06	7.0400E+06
x(M)	0.0000	20.160	12.880	23.800	25.480	18.480	17.080	20.720	28.000	0.0000	0.0000
3	-5.3200E-02	-1.0500E-06	-2310.0	-0.5680	-28.600	-3.2300	-161.00	-0.8190	3960.0	7.0400E+06	7.0400E+06
x(M)	0.0000	20.160	12.880	23.800	25.480	18.480	17.080	20.720	28.000	0.0000	0.0000
4	-5.3100E-02	-1.0400E-06	-2320.0	-0.5710	-28.600	-3.2600	-162.00	-0.8310	3830.0	7.0400E+06	7.0400E+06
x(M)	0.0000	20.160	12.880	23.800	25.480	18.480	17.080	20.720	28.000	0.0000	0.0000
Min.	-5.3300E-02	-1.0500E-06	-2330.0	-0.5710	-28.700	-3.2600	-163.00	-0.8310	3830.0	7.0400E+06	7.0400E+06
Pile N.	1	1	1	4	2	4	1	1	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR M	DEFLECTION z-DIR M	BENDING MOMENT z-DIR KN- M	BENDING MOMENT y-DIR KN- M	SHEAR FORCE y-DIR KN	SHEAR FORCE z-DIR KN	SOIL REACTION y-DIR KN/ M	SOIL REACTION z-DIR KN/ M	TOTAL STRESS KN/ M**2	FLEXURAL RIGIDITY z-DIR KN- M**2	FLEXURAL RIGIDITY y-DIR KN- M**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-5.3300E-02	3.7600E-04	80.400	16.400	463.00	0.2000	118.00	1.1400	1.2500E+04	7.0400E+06	7.0400E+06
x(M)	20.160	0.0000	23.800	0.0000	18.480	25.480	20.720	17.080	12.880	0.0000	0.0000
2	-5.3300E-02	3.7600E-04	80.600	16.300	459.00	0.2020	116.00	1.1400	1.2400E+04	7.0400E+06	7.0400E+06
x(M)	20.160	0.0000	23.800	0.0000	18.480	25.480	20.720	17.080	12.880	0.0000	0.0000
3	-5.3200E-02	3.7600E-04	80.400	16.300	458.00	0.2020	116.00	1.1400	1.2200E+04	7.0400E+06	7.0400E+06
x(M)	0.0000	0.0000	23.800	0.0000	18.480	25.480	20.720	17.080	12.880	0.0000	0.0000
4	-5.3100E-02	3.7600E-04	80.600	16.400	460.00	0.2020	117.00	1.1500	1.2100E+04	7.0400E+06	7.0400E+06
x(M)	0.0000	0.0000	23.800	0.0000	18.480	25.480	20.720	17.080	12.880	0.0000	0.0000
Max.	-5.3100E-02	3.7600E-04	80.600	16.400	463.00	0.2020	118.00	1.1500	1.2500E+04	7.0400E+06	7.0400E+06
Pile N.	4	1	2	1	1	2	1	4	1	1	1

LOAD CASE : 5
CASE NAME : Load Case 05
LOAD TYPE : Special, Sp

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

GROUP NO	P-FACTOR	Y-FACTOR
1	0.9772	1.0000
2	0.9545	1.0000
3	0.9545	1.0000
4	0.9767	1.0000

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, KN	HOR. LOAD Y, KN	HOR. LOAD Z, KN
41191.0	0.00000	0.00000
MOMENT X , M- KN	MOMENT Y, M- KN	MOMENT Z, M- KN
0.00000	29735.0	3089.00

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL , M	HORIZONTAL Y, M	HORIZONTAL Z, M
5.48155E-03	-0.0211850	1.06738E-03
ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
1.32228E-05	1.15479E-04	2.29193E-03

THE GLOBAL STRUCTURAL COORDINATE SYSTEM

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X, M	DISP. Y, M	DISP. Z, M	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	6.5642E-03	-2.1309E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
2	5.8424E-03	-2.1226E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
3	5.1207E-03	-2.1144E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
4	4.3989E-03	-2.1061E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
MINIMUM	4.3989E-03	-2.1309E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
Pile N.	4	1	1	1	1	1
MAXIMUM	6.5642E-03	-2.1061E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
Pile N.	1	4	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X, KN	FOR. Y, KN	FOR. Z, KN	MOM X, KN- M	MOM Y, KN- M	MOM Z, KN- M	STRESS, KN/ M**2
1	1.1706E+04	-0.2341	-7.5243E-02	8.0574E-08	38.612	750.30	7288.4
2	1.0777E+04	0.2264	-4.0669E-02	8.0574E-08	38.900	766.68	6981.8
3	9847.3	0.2496	1.6112E-02	8.0574E-08	39.055	780.40	6665.6
4	8860.8	-0.2419	9.9800E-02	8.0574E-08	39.082	791.63	6318.1
MINIMUM	8860.8	-0.2419	-7.5243E-02	8.0574E-08	38.612	750.30	6318.1
Pile N.	4	4	1	1	1	1	4
MAXIMUM	1.1706E+04	0.2496	9.9800E-02	8.0574E-08	39.082	791.63	7288.4
Pile N.	1	3	4	1	4	4	1

River Pile Group

THE PILE COORDINATE SYSTEM (LOCAL AXES)

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x, M	DISP. y, M	DISP. z, M	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	6.5642E-03	-2.1309E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
2	5.8424E-03	-2.1226E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
3	5.1207E-03	-2.1144E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
4	4.3989E-03	-2.1061E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
MINIMUM	4.3989E-03	-2.1309E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
Pile N.	4	1	1	1	1	1
MAXIMUM	6.5642E-03	-2.1061E-02	1.0674E-03	1.3223E-05	1.1548E-04	2.2919E-03
Pile N.	1	4	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL, KN	LAT. y, KN	LAT. z, KN	MOM x, KN- M	MOM y, KN- M	MOM z, KN- M	STRESS, KN/ M**2
1	1.1706E+04	-0.2341	-7.5243E-02	8.0574E-08	38.612	750.30	7288.4
2	1.0777E+04	0.2264	-4.0669E-02	8.0574E-08	38.900	766.68	6981.8
3	9847.3	0.2496	1.6112E-02	8.0574E-08	39.055	780.40	6665.6
4	8860.8	-0.2419	9.9800E-02	8.0574E-08	39.082	791.63	6318.1
MINIMUM	8860.8	-0.2419	-7.5243E-02	8.0574E-08	38.612	750.30	6318.1
Pile N.	4	4	1	1	1	1	4
MAXIMUM	1.1706E+04	0.2496	9.9800E-02	8.0574E-08	39.082	791.63	7288.4
Pile N.	1	3	4	1	4	4	1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR M	DEFLECTION z-DIR M	BENDING MOMENT z-DIR KN- M	BENDING MOMENT y-DIR KN- M	SHEAR FORCE y-DIR KN	SHEAR FORCE z-DIR KN	SOIL REACTION y-DIR KN/ M	SOIL REACTION z-DIR KN/ M	TOTAL STRESS KN/ M**2	FLEXURAL RIGIDITY z-DIR KN- M**2	FLEXURAL RIGIDITY y-DIR KN- M**2
1	-2.1300E-02	-2.8800E-06	-977.00	-1.5800	-10.500	-9.7900	-69.000	-2.6900	4600.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	17.920	17.080	19.880	28.000	0.0000	0.0000
2	-2.1200E-02	-2.8600E-06	-969.00	-1.5700	-10.400	-9.7700	-68.400	-2.6900	4230.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	19.880	28.000	0.0000	0.0000
3	-2.1100E-02	-2.8600E-06	-963.00	-1.5700	-10.300	-9.7700	-67.900	-2.6900	3870.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	19.880	28.000	0.0000	0.0000
4	-2.1100E-02	-2.8400E-06	-961.00	-1.5700	-10.200	-9.8500	-67.800	-2.7300	3480.0	7.0400E+06	7.0400E+06
x(M)	0.0000	19.880	12.600	23.520	25.200	18.200	17.080	19.880	28.000	0.0000	0.0000
Min.	-2.1300E-02	-2.8800E-06	-977.00	-1.5800	-10.500	-9.8500	-69.000	-2.7300	3480.0	7.0400E+06	7.0400E+06
Pile N.	1	1	1	1	1	4	1	4	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE DEFLECTION BENDING MOMENT SHEAR FORCE SOIL REACTION TOTAL FLEXURAL RIGIDITY

Page 13

Soil report recommendations

- The prevailing sub-soil conditions.
- The magnitudes of loads imposed by the superstructures.
- The total and differential tolerable ground settlement under the foundation.
- The economics of alternative foundation design, and
- The capability and experience of available foundation construction contractors.

Taking these factors into account with reference to particular project under consideration, the bored concrete pile foundation type is recommended to support the proposed bridge at the investigated site:

6.1- Pile foundation:

Consideration has been given to the use of bored piles as a feasible foundation alternative at this particular site. These are suitable for the rock formation encountered at the present site. The bored piles have the advantage that local contractors are available. Piles of 60 to 120 cm diameter are recommended. The length of the pile will depend on the magnitude and direction of the forces acting on the pile and the soil conditions within the pile zone. The piles should rest on or be embedded into the Nubian sandstone formation at the depth of 30.0 m below the existing ground level or the river bed.

The allowable bearing capacity of individual bored pile was calculated according to *AllPile* software program and a summary of the computed safe pile's bearing capacity is given below for piles of variable diameters and 30.0m length.

Table(4) : Pile allowable bearing capacity

Pile diameter (m)	0.6	0.8	1.0	1.2
Pile embedment (m)	30.0	30.0	30.0	30.0
Allowable bearing Capacity (kN)	3000	4900	7100	9500

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Web site: www.esd-sudan.com. E-mail: esd.eng.co@gmail.com - esdengco@yahoo.co

6.2- General Recommendations:

- Depending on the magnitudes of anticipated superimposed loads, the appropriate size of pile (length, cross-section or diameter) and the number and spacing of piles in each group may be decided according to the foundation design method applied.
- Bored piles are normally constructed by advancing a hole of appropriate size to the required depth by auger drilling, or any other suitable technique followed by lowering down the reinforcement cage and gradually filling the hole with concrete. Below the groundwater level or in the offshore sub-structure trimme pipes or casings should be used.
- It must be pointed out that only static vertical bearing capacity, using factor of safety of 3 for end bearing and 2.5 for side friction, of the piles were considered in the analysis and thus the values given above should be considered as estimates and guideline for foundation design. To confirm the theoretical calculations of the allowable bearing capacity of single piles, it is strongly recommended that few pile-loading tests be carried out. The pile load test should be carried out on a typical pile loaded to a minimum of 1.5 times the calculated design load.

7.0- Conclusions and Recommendations:

The site investigation has been made for (A&A for Urban Development) at Soba town, in Khartoum state for the proposed Soba Bridge. A total of ten (10) boreholes ranging between 40 to 50m, as suggested by the client, were drilled in this project. Field as well as laboratory tests were conduced on the soil samples and the summary of the results are presented in this report. The results indicated that the site is dominated by alluvial soils that are followed by Nubian formation towards the bottom of the boreholes. The alluvial deposits are clayey silt (ML), silty clay (CL), clayey sand (SC) and silty sands (SM, SP-SM, or SP), whereas the Nubian formation is comprising of sandstone and mudstone. The chemical test results indicated alkaline soils free from harmful

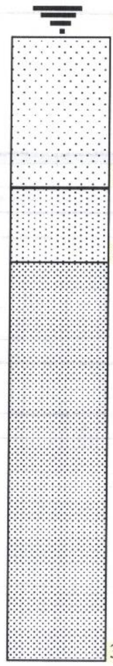
Soil properties

WHATLING CONSULTANTS	Project No	Sheet No.	Rev.
	11001	C02.4/6	A

Calculation Sheet

Project Soba Bridge	Made by PFW	Checked by
Element Pile Design - Piers P4 to P10	Date 21.05.13	Date

Design Section - 1800 Pile



Description	Stratum thickness (m) & Level (m)		Unfactored P-y data	Factored p - y data	
	370				
Silty Sand (Loose)	6.0	c' =	0	0	kN/m^2
		φ' =	21	17.1	deg
		γ' =	8	8	kN/m^3
		K =	10000	10000	kN/m^3
		Ko =	0.5		Ref Tomlinson 5th Ed. Pg 170
	364				
Clayey Sand (Medium dense)	3.0	c' =	0	0	kN/m^2
		φ' =	30	24.8	deg
		γ' =	9	9	kN/m^3
	361	K =	20000	20000	kN/m^3
		Ko =	0.45		Ref Tomlinson 5th Ed. Pg 170
Silty Sand (Very Dense)	17.0	c' =	0	0	kN/m^2
		φ' =	40	33.9	deg
		γ' =	9	9	kN/m^3
		K =	40000	40000	kN/m^3
		Ko =	35		Ref Tomlinson 5th Ed. Pg 170
	344				

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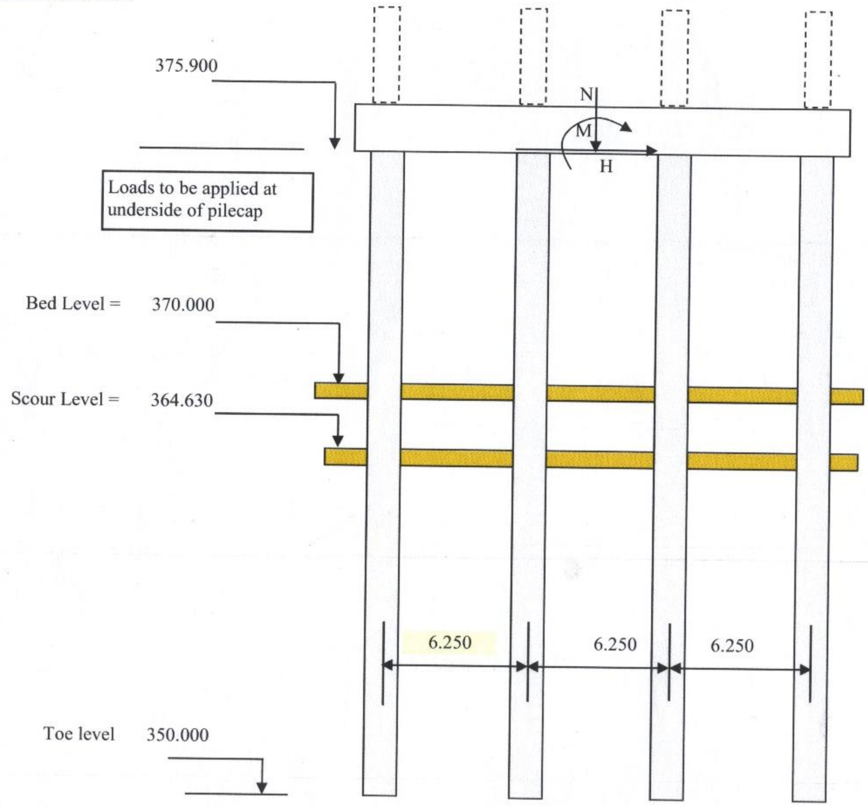
Transverse Section of pile group

WHATLING CONSULTANTS Calculation Sheet	Project No 11001	Sheet No. C02.4/8	Rev. A
Project Soba Bridge	Made by PFW		Checked by
Element Pile Design - Piers P4 to P10	Date 21.05.13		Date

6.2 Transverse Section



Tables of load combinations

WHATLING CONSULTANTS	Project No	Sheet No.	Rev.
	11001	C02.4/16	A
Calculation Sheet			
Project Soba Bridge	Made by PFW	Checked by	
Element Pile Design - Piers P4 to P10	Date 21.05.13	Date	

Combination 1 - ULS

LC01	**6 Lanes HA - Both Footways - 2 Spans Loaded - Max Longitudinal**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	7753			0.075	1.345	581	10427
Footway	605			0.075	0	45	0
Totals	**43608**	**0**	**0**			**3271**	**10427**

LC02	**6 Lanes HA - Both Footways - 1 Span Loaded - Max Longitudinal**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	5022			0.825	1.345	4143	6754
Footway	413			0.825	0	340	
Totals	**40685**	**0**	**0**			**7127**	**6754**

LC03	**6 Lanes HA+HB - Both Footways - 2 Spans Loaded - Max Longitudinal**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	7232			0.075	1.899	542	13734
Footway	605			0.075	0	45	0
Totals	**43088**	**0**	**0**			**3232**	**13734**

LC04	**6 Lanes HA+HB - Both Footways - 1 Span Loaded Max Longitudinal**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	5276			0.825	1.899	4353	10019
Footway	413			0.825	0	340	0
Totals	**40939**	**0**	**0**			**7337**	**10019**

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WHATLING CONSULTANTS	Project No	Sheet No.	Rev.
Calculation Sheet	11001	C02.4/17	A
Project Soba Bridge	Made by PFW	Checked by	
Element Pile Design - Piers P4 to P10	Date 21.05.13	Date	

Combination 1 - ULS

LC05	**4 Lanes HA - One Footway - 2 Spans Loaded - Max Transverse**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	5638			0.075	4.625	423	26077
Footway	302			0.075	12.1	23	3658
Totals	**41191**	**0**	**0**			**3089**	**29735**

LC06	**4 Lanes HA - One Footway - 1 Span Loaded - Max Transverse**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	3652			0.825	4.625	3013	16892
Footway	206			0.825	12.1	170	2497
Totals	**39109**	**0**	**0**			**5827**	**19388**

LC07	**4 Lanes HA+HB - One Footway - 2 Spans Loaded - Max Transverse**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	5400			0.075	5.055	405	27296
Footway	302			0.075	12.1	23	3658
Totals	**40953**	**0**	**0**			**3071**	**30954**

LC08	**4 Lanes HA+HB - One Footway - 1 Span Loaded - Max Transverse**						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	4089			0.825	6.39	3373	26127
Footway	206			0.825	12.1	170	2497
Totals	**39546**	**0**	**0**			**6187**	**28624**

WHATLING CONSULTANTS	Project No	Sheet No.	Rev.
	11001	C02.4/18	A

Calculation Sheet

Project	Made by	Checked by
Soba Bridge	PFW	
Element	Date	Date
Pile Design - Piers P4 to P10	21.05.13	

Combinations 2 to 5 - ULS

LC09	6 Lanes HA - Both Footways - 2 Spans Loaded - Braking/Traction						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	6461			0.075	1.345	485	8690
Footway	504			0.075	0	38	0
Horizontal			344	13.4		4606	0
Totals	**42215**	**0**	**344**			**7772**	**8690**

LC10	6 Lanes HA - Both Footways - 1 Span Loaded - Braking/Traction						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	4185			0.825	1.345	3453	5629
Footway	344			0.825		284	0
Horizontal			344	13.4		4606	0
Totals	**39779**	**0**	**344**			**10986**	**5629**

LC11	6 Lanes HA+HB - Both Footways - 2 Spans Loaded - Braking/Traction						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	6120			0.075	1.899	459	11621
Footway	504			0.075		38	0
Horizontal			344	13.4		4606	0
Totals	**41874**	**0**	**344**			**7747**	**11621**

LC12	6 Lanes HA+HB - Both Footways - 1 Span Loaded - Braking/Traction						
	V	Ht	Hl	el	et	Ml	Mt
Dead	35251			0.075		2644	
Traffic	4464			0.825	1.899	3683	8477
Footway	344			0.825		284	0
Horizontal			344	13.4		4606	0
Totals	**40059**	**0**	**344**			**11217**	**8477**

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WHATLING CONSULTANTS	Project No	Sheet No.	Rev.
Calculation Sheet	11001	C02.4/20	A
Project Soba Bridge	Made by PFW	Checked by	
Element Pile Design - Piers P4 to P10	Date 21.05.13	Date	

Combinations 2 to 5 - ULS

LC18	**0.2x(4 Lanes HA - One Footway) - 2 Spans Loaded - Ship Impact**						
	V	Ht	Hl	el	ht	Ml	Mt
Dead	35251			0.075		2644	
Traffic	940			0.075	4.625	70	4346
Footway	50			0.075	12.1	4	610
Horizontal		2089	0		8.1	0	16921
Totals	**36241**	**2089**	**0**			**2718**	**21877**

LC19	**Min Dead Load - 0.2x(4HA+Footway) - 2 Spans Loaded- Ship Impact**						
	V	Ht	Hl	el	ht	Ml	Mt
Dead	27199			0.075		2040	
Traffic	940			0.075	4.625	70	4346
Footway	50			0.075	12.1	4	610
Horizontal		2089			8.1	0	16921
Totals	**28189**	**2089**	**0**			**2114**	**21877**

LC20	**Min Dead Load - No Live Load - Ship Impact**						
	V	Ht	Hl	el	ht	Ml	Mt
Dead	27199			0.075		2040	
Traffic	0			0.075	0	0	0
Footway	0			0.075	0	0	0
Horizontal		2089			8.1	0	16921
Totals	**27199**	**2089**	**0**			**2040**	**16921**

LC21	**Min Dead Load - No Live Load - Transverse Wind**						
	V	Ht	Hl	el	ht	Ml	Mt
Dead	27199			0.075		2040	
Traffic	0			0.075	0	0	0
Footway	0			0.075	0	0	0
Horizontal		850	0		14.9	0	12670
Totals	**27199**	**850**	**0**			**2040**	**12670**

LC22	**4 Lanes HA - One Footway - 2 Spans Loaded - Transverse Wind**						
	V	Ht	Hl	el	ht	Ml	Mt
Dead	35251			0.075		2644	
Traffic	4699			0.075	4.625	352	21731
Footway	252			0.075	12.1	19	3048
Horizontal		520	0		16.15	0	8392
Totals	**40201**	**520**	**0**			**3015**	**33172**

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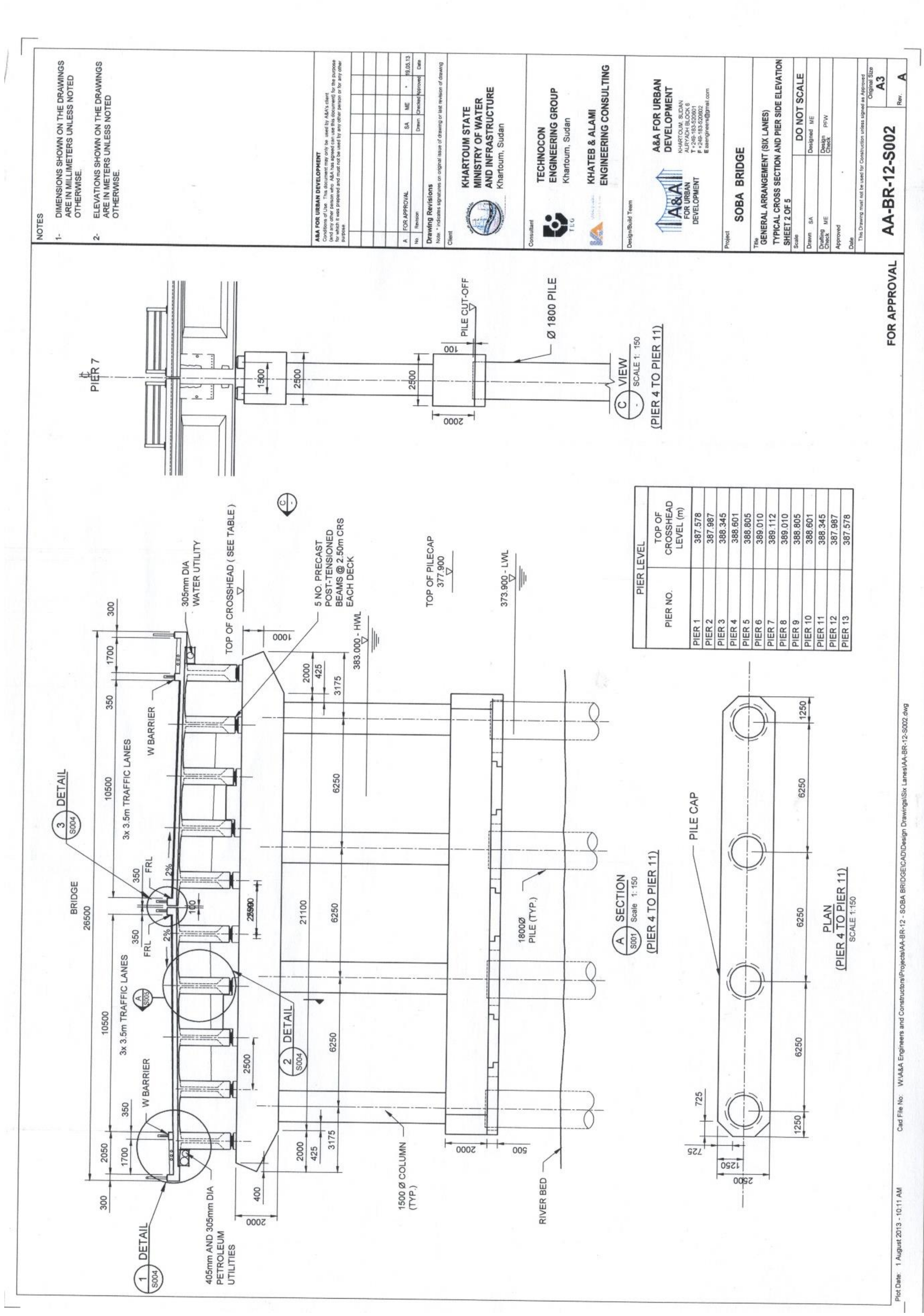
NOTES
1- DIMENSIONS SHOWN ON THE DRAWINGS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE.
2- ELEVATIONS SHOWN ON THE DRAWINGS ARE IN METERS UNLESS NOTED OTHERWISE.
PIER LEVEL
PIER NO. | TOP OF CROSSHEAD LEVEL (m)
PIER 1 | 387.578
PIER 2 | 387.987
PIER 3 | 388.345
PIER 4 | 388.601
PIER 5 | 388.805
PIER 6 | 389.010
PIER 7 | 389.112
PIER 8 | 389.010
PIER 9 | 388.805
PIER 10 | 388.601
PIER 11 | 388.345
PIER 12 | 387.987
PIER 13 | 387.578
KHARTOUM STATE MINISTRY OF WATER AND INFRASTRUCTURE Khartoum, Sudan
TECHNOCON ENGINEERING GROUP Khartoum, Sudan
KHATEB & ALAMI ENGINEERING CONSULTING
A&A FOR URBAN DEVELOPMENT
SOBA BRIDGE
GENERAL ARRANGEMENT (SIX LANES) TYPICAL CROSS SECTION AND PIER SIDE ELEVATION SHEET 2 OF 5
DO NOT SCALE
AA-BR-12-S002
FOR APPROVAL
SECTION Scale 1:150 (PIER 4 TO PIER 11)
PLAN (PIER 4 TO PIER 11) SCALE 1:150
VIEW SCALE 1:150 (PIER 4 TO PIER 11)
PIER 7
PILE CUT-OFF
Ø 1800 PILE
PILE CAP
RIVER BED
1500 Ø COLUMN (TYP.)
1800Ø PILE (TYP.)
TOP OF CROSSHEAD (SEE TABLE)
TOP OF PILECAP 377.900
373.900 - LWL
383.000 - HWL
5 NO. PRECAST POST-TENSIONED BEAMS @ 2.50m CRS EACH DECK
305mm DIA WATER UTILITY
405mm AND 305mm DIA PETROLEUM UTILITIES
3x 3.5m TRAFFIC LANES
W BARRIER
BRIDGE 26500
DETAIL