Performance of Single Piles in Line groups Installed in Sand

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ABSTRACT

Performance of single pile in a line bored pile group in sand

Ujjal Chakraborty

Group action of piles driven in sand has been a challenging research subject in the field of geotechnical engineering for the last 50 years. In the literature, planar geometry was the only governing factor to estimate the group capacity, ignoring the contributions of the pile cap rigidity and sequence technique for driving the piles in a group. Accordingly the available theories failed to provide accurate prediction.

A numerical model was developed to incorporate the above-mentioned ignored parameters. Furthermore, the distribution of the column load on the individual piles under the pile cap was examined. The results of this analysis showed that in a line pile group, the interior piles will carry more load than those at the exterior boundaries, depending on the rigidity of the pile cap. For a relatively rigid cap, these phenomena existed at a much lesser extent.

A design procedure is proposed to determine the pile group efficiency for a given sequential bored piles installed in sand. An optimum cap thickness of a pile group is suggested to achieve the most economical design. Considering the results obtained from the numerical model an analytical model was developed to predict the group efficiency of line pile group. The predicted values by the proposed formula compared well with the numerical test data. Design procedure is given for practical use.

ACKNOWLEDGEMENTS

I am indebted to my Professor Dr. Adel M. Hanna for his patience, guidance and support throughout my M.A.Sc study. His constant inspirations always encourage me to put my best effort and that leads to the successful completion of this study and submission of the thesis in the present form.

I would like to dedicate this work to the departed soul of my beloved parents. Their blessings and firm belief for my higher education have made this effort possible. Special thanks go to my elder brother Sagar for his constant patience and the moral support throughout my endeavors.

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LIST OF SYMBOLS

A_p = Cross section area of pile tip

A_s = Surface area of pile shaft

p = Perimeter of pile

D = Diameter of pile

D_R = Relative density of sand

e = Void ratio

e_{max} = Maximum void ratio

e_{min} = Maximum void ratio

f_s = Average unit skin friction

K_0 = Coefficient of earth pressure at-rest

Q_p = Point resistance of pile

Q_s = Skin friction of pile

Q_u = Ultimate bearing capacity of pile

n = Number of piles in row

m = Number of piles in column

η = Group efficiency of pile

ϕ = Angle of internal friction

s = Centre to centre spacing of the pile

γ_d = Dry unit weight of sand

γ_{sat} = Saturated unit weight of sand

σ = Normal stress

τ = Shear Stress

ψ = Dilations angle

μ = Pore water pressure

ν = Void ratio

K_o = Earth pressure factor

η_g = Geometric efficiency

η = Load efficiency of shaft

CHAPTER-1

INTRODUCTION

1.1 General:

The design of pile foundations is both an art and science. Piles are structural members made of timber, concrete or steel, are used to transfer surface loads to lower levels in the soil mass. This load may be resisted by the pile shaft or directly through the pile tip. Pile foundations are known to provide higher capacity, earthquake resistance and to significantly reduce the settlement of the structure. Piles may be designed to carry axial and lateral loads, also used as anchors e.g. for docks and hangers or subjected to high lateral loads (offshore structure). It is then important to select the most suitable type and the method of installation for the proposed piles for the given ground and loading conditions. The engineer should be able to predict the capacity of these piles, the spacing and to determine whether the piles will act as a group or individual piles.

According to the method of installation, piles can be classified as displacement piles (driven & jacked) and non-displacement piles (bored piles). For the displacement piles, soil displaces radially and vertically during piles installation. Displacement piles are favorable for soil compaction in case of cohesion-less soils and heave in case of cohesive soils. For non-displacement piles, soils are excavated then piles are installed in the predetermined holes. Driven piles are favorable in terms of higher capacity and better performance.

When several piles are clustered, it is reasonable to expect that the soil will be pressured laterally due to the side friction on the pile shaft or the point bearing. The superimposed lateral pressure produced by both the pile shaft and the tip is depending on both the pile load and spacing. In case of high loading there will be a shear failure in the soil mass or the pile group will experience excessive settlement or failure of the pile material. The stress intensity due to the overlapping stress zones will obviously decrease with the increase of pile spacing. However, large spacing will require larger cap, which in return will reduce its rigidity. Pile cap may be installed in contact with the ground or above the ground, in the case of offshore platforms.

In the literature, design theories are focused on problems related to single piles rather than group of piles. This is due to mainly the difficulty in performing laboratory or field tests on pile groups. Accordingly, the group capacity was taken as the sum of the individual piles capacities. Thus the group efficiency (η) is given as follows:

$$\eta = \frac{Q_g}{\sum Q_s} \qquad 1.1$$

Where:

η = Efficiency of the pile group

Q_g = Capacity of the pile group

Q_s = Capacity of a single pile

1.2 Problem Determination:

The available theories in the literature did not provide accurate and reliable predictions of the pile group capacity and the group efficiency. This can be explained as follows:

a) Most of these theories consider the planner geometry of the pile group (i.e. pile spacing, pile diameter and pile arrangement) as the only parameter governing the performance of these piles. Accordingly, other factors such as; pile cap, pile soil interaction, loading condition, and load distribution over the individual piles, pile length and soil property was ignored.

b) There are two components of pile-soil-pile interaction (O' Nill 1983), first; installation effects, which consists of alteration of soil stresses around the pile shaft due to driving of the neighbouring piles, second; is the mechanical effect, which consists of strain superposition in the soil mass and alteration of failure zones due to simultaneous loading of the piles. While most of the available theories have considered the mechanical effects, they totally or partially ignored the installation effect on the pile-soil interaction.

c) Cap thickness in a pile group is a very important factor in estimating the group efficiency, which was ignored in most of the available theories.

d) The available experimental data in the literature (Combefort (1953), Kezdi (1957), Hanna (1963), Kishida & Meyerhof (1965), Meyerhof and Ranjan (1973b) etc.) have been used only to provide quantitative value of the group efficiency and to examine the effect of the planner geometry.

1.3 Research Objectives:

The objectives of the present research are summarized as follows:

(a) To review the pertinent literature on the topic of pile group efficiency for displacement and non-displacement piles in sand, subjected to axial loading.

(b) To evaluate the reliability of the existing theoretical models of pile group efficiency.

(c) To examine the effects of the governing factors that believed to affect the pile group efficiency.

(d) To develop a numerical model using the finite element technique to analyze the problem stated.

(e) To develop a new formula to predict group efficiency of line pile groups in cohesion-less soil.

CHAPTER -2

LITERATURE REVIEW

2.1 General

In this chapter an extensive literature review of group pile behavior was undertaken. Research on group efficiency of pile subjected to axial and lateral loading in cohesionless soil has been a challenge for the geotechnical engineers for the last 50 years. The majority of this research deals with the group behavior, only small number provides design information that can be readily applied in practice. The results of the present review have established the need for more design guidance.

At the earlier time, the capacity of pile groups was taken as equal to the sum of the capacities of the individual piles. However, in practice, when piles are placed close to each other, the stresses transmitted to the soil through neighboring piles will overlaps, resulting in a considerable change of the group capacity.

2.2 Historical Development

The Converse-Labarre (Bolin 1941) formula is one of the most widely used one for evaluating pile group behavior, and the AASHTO (1990) Bridge Specifications give it as a "suggestion" for frictional piles. According to this formula, the efficiency, i.e. Percentage of isolated pile value, is equal to

$$\eta = 1 - \frac{\alpha}{90}\left(\frac{(n-1)m + (m-1)n}{m.m}\right) \qquad 2.1$$

Where:

η = Group efficiency

m = no. of rows

n = no. of columns

$\alpha = \tan^{-1}\left(\frac{d}{s}\right)$

s = center-center spacing of piles

d = Pile diameter

By using Converse-Labaree formula, Figure 2.1 shows that, if the spacing is fixed i.e. s = 3d then group efficiency decreases with the increase of the number of rows.

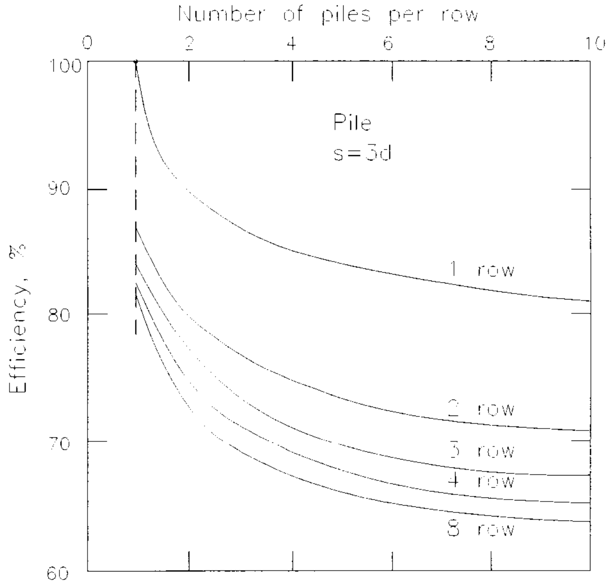


Figure 2.1: Group efficiency according to the Converse-Labarre formula (after Garg 1979)

Los Angeles group action, 1944 equation can be expressed as:

$$\eta = 1 - \frac{D}{\pi d n_1 n_2}[n_1(n_2 - 1)] + n_2(n_1 - 1) + \sqrt{2}(n_1 - 1)(n_2 - 1)] \qquad 2.2$$

Seiler and Keeney, 1944

$$\eta = \left\{1 - \left[\frac{11d}{7(d^2 - 1)}\right]\left[\frac{n_1 + n_2 - 2}{n_1 + n_2 - 1}\right]\right\} + \frac{0.3}{n_1 + n_2} \text{ Where d is in ft} \qquad 2.3$$

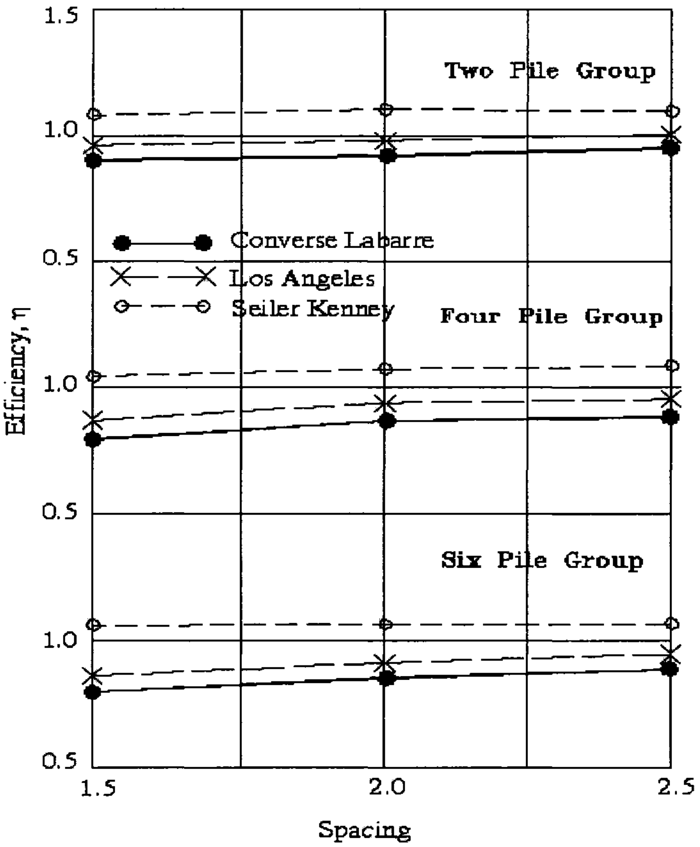


Figure 2.2: Pile group Efficiency from Various Formulas (After Garg, 1979)

Figure 2.2 presents a comparison of the above-mentioned three methods; it can be noted that for spacing 1.5d to 2.5d the overall efficiency of group pile is higher for Seiler Kenney formula as compared to Los Angeles and Converse Labarre formula.

Equation 2.1, 2.2 and 2.3 were developed under the geometric condition of the pile group i.e. number of piles, center-center spacing of the piles in-group. Furthermore, they did not consider the other important parameters, such as: length of the pile, soil condition, pile head conditions, group size, settlement of group and single piles, etc.

Feld (1943) suggested a method by which the load capacity of the individual piles in a group embedded in sand could be assigned. According to this method, the capacity of a pile is reduced by 1/16 by each adjacent diagonal or row pile. The technique is well explained in Figure 2.3, which shows the layout plan of the group of piles and the distribution of the load. Based on this method, different loads will be assigned to different piles in the group. Table 2.1 presents the load distribution reduction factor of each pile in a group.

Table 2.1 Ultimate bearing capacity (After Feld, 1943)

Pile Type	No of piles	No of adjacent piles/pile	Reduction factor for each pile	Ultimate capacity
A	1	8	1-8/16	$0.5Q_u$
B	4	5	1-5/16	$2.75Q_u$
C	4	3	1-3/16	$3.75Q_u$

$\sum = 6.5Q_u$

$= Q_{g(u)}$

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} = 6.5\frac{Q_u}{9Q_u} = 72\% \qquad 2.4$$

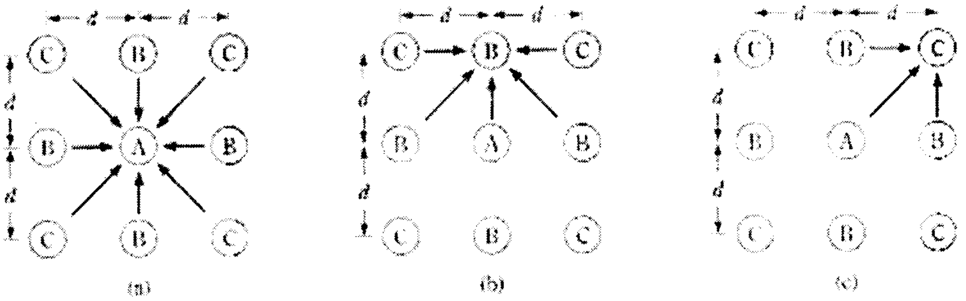


Figure 2.3: Feld's (1943) method for estimation of the group capacity of frictional pile.

It can be noted that Feld's model considered only the planar geometry of the group i.e. pile spacing, pile diameter and the number of piles in-group. The model also considered the reduction factor of each pile in-group. It did not however consider the other parameter such as: cap condition, soil condition, type of loading and pile-length to diameter ratio (L/d).

Terzaghi and Peck (1967) assumed the pile group and the enclosing soil to form a rigid block, (Figure 2.4) to transfer the load to the soil beneath the block (or a pier) shaft and tip, and further, it fails as a unit. The capacity of this "pier" is computed as follows:

$$P^{f^{group}} = abp^{f} + 2l(a+b)^{\tau_s} \qquad 2.5$$

Where:

$P^{f^{group}}$ = Capacity of group piles

p^{f} = capacity of single pile

a = length of the block

b = width of the block

l = length of the pile

τ_s = Average unit shear strength of the soil within the block shaft.

Furthermore, Terzaghi and peck reported that.

a. Group efficiency in case of pile group in single row relatively equals to or less than unity. It will increase with the increase of the number of piles and the spacing between piles, Figure 2.5a.
b. Group efficiency of closely spaced group piles will increase with the number of piles and spacing. Maximum efficiency will be achieved at a spacing of 3d, Figure 2.5b.

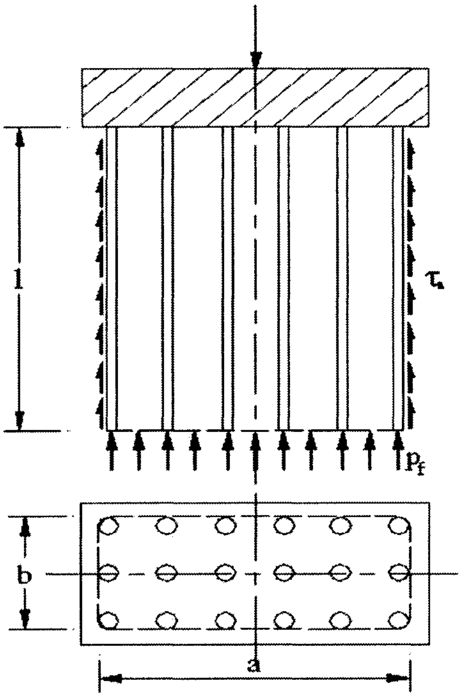


Figure 2.4: Bearing value for pile-groups (After Terzaghi and Peck, 1967)

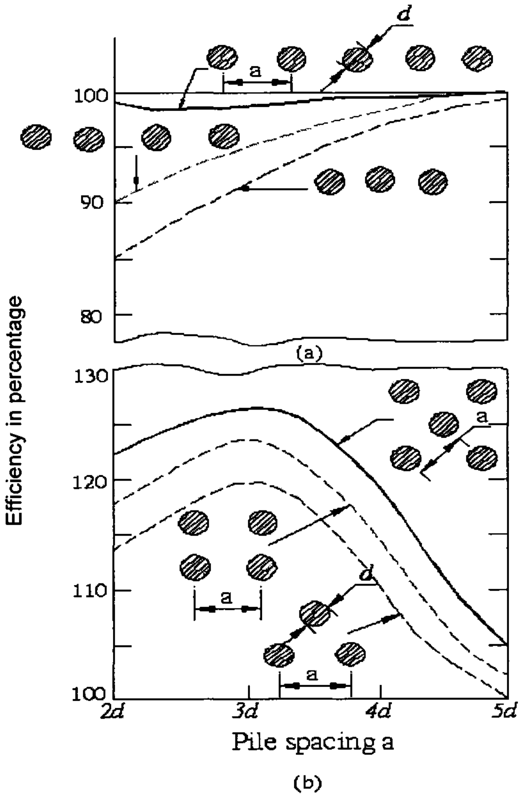


Figure 2.5: Group efficiency for pile rows (a) and closed groups (b) in medium sand (After Terzaghi & Peck, 1967).

Meyerhof (1959) conducted full-scale load tests of pile groups in sand up to failure. He has examined that the ultimate group load in driven piles with center-center spacing of about 2 to 4 pile diameters is greater than that of the sum of the ultimate load of the single piles. In 1965 Kishida and Meyerhof conducted an investigation on the group efficiency driven in sand. They reported that if the angle of shearing resistance ϕ is greater than 40^0 the group efficiency is always less than unity. For $\phi=30^0$ and 35^0 they found that the group efficiency is grater than unity. Figure 2.6 shows the variation of group efficiency with the value of ϕ , number of piles in the group and the spacing of piles in the group.

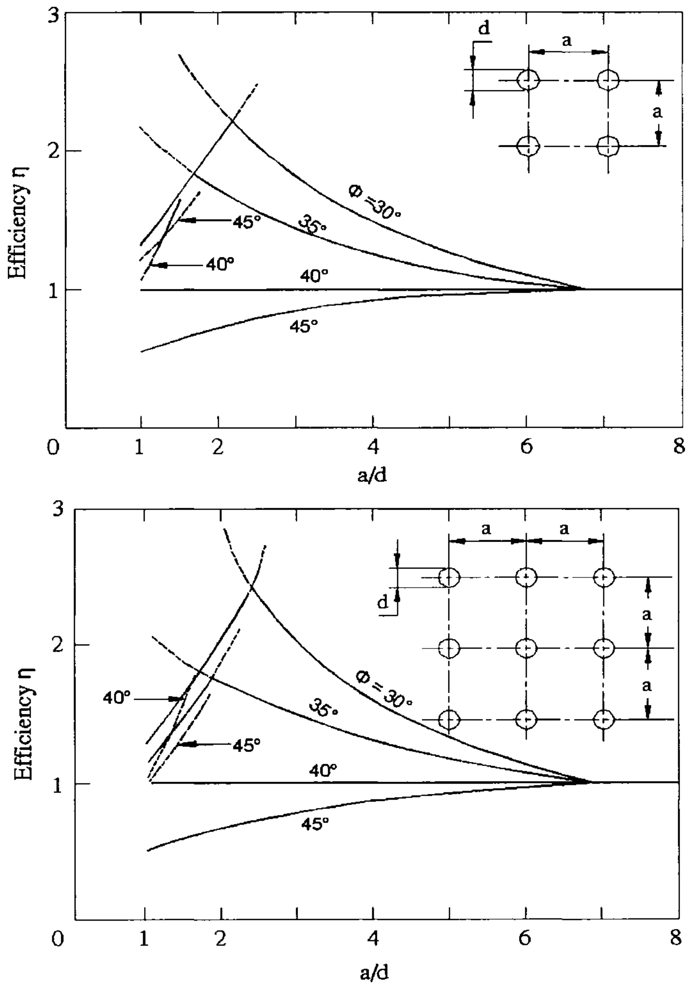


Figure 2.6: Group efficiency in sand, (After Meyerhof, 1960; and Kishida, 1964.)

Mayerhof (1960) conducted a model investigation on large and small group of piles in loose sand and soft clay. The results of this study are summarized in Figure 2.7.

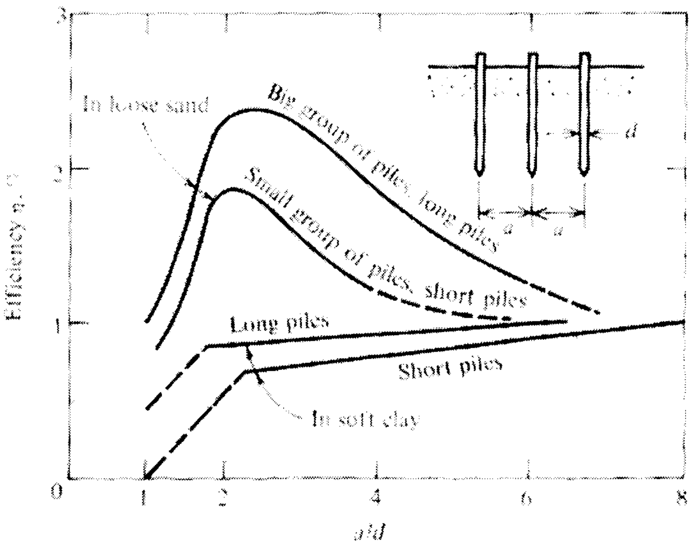


Figure 2.7: Group efficiency in loose sand and soft clay. (After Meyerhof, 1960)

It can be noted from Figure 2.7:

(a) Larger pile group with long pile driven in loose sand has efficiency (η)>2, when a/d is equal to 2, here a is the center-to-center spacing.

(b) Small pile group with short pile in loose sand has efficiency (η)<2, when a/d is equal to 2.

(c) Efficiency of pile group driven in sand is greater than the same group driven in clay.

For the determination of group efficiency, Mayerhof considered the spacing of piles in the group, arrangement of piles under the cap, pile length, angle of shearing resistance of the sand, settlement of the individual and the pile group etc. However, he did not consider the group size and the loading condition effect on pile group performance.

Vesic (1967) reported that the efficiency of the pile group increases with the increase of the pile diameter and slightly decreases due to an increase in the pile spacing. Vesic (1980b) reported the results of the capacity of a single pile and a pile group tested in cohesive soil in Houston. He observed that there was more scatter in the predictions of the capacity of the pile group than the single piles tested at the same site. He further reported that there is a tendency in the available methods towards over predicting the pile group capacity. Vesic also observed that the results of the field test on the pile group showed that the pile in-group plunged individually into the ground with no signs of block failure or increase in its efficiency with time.

Chellis (1969) and Vesic (1981) recommended that the group effect should be taken into consideration only for the shaft load component. Based on this postulate, the group load Q_g could be expressed as:

$$Q_g = \sum Q_p + \eta_s \sum Q_s \qquad 2.6$$

Where:

Q_p = Ultimate point load

Q_s = Ultimate shaft load

η_s = Load efficiency on shaft $= \eta_s'.k$ 2.7

η_s' = Geometric efficiency

K = Group interaction factor

Vesic (1967) suggested that for a group of m x n piles, the ratio $\frac{\sum Q_s}{\sum Q_0}$ is equal to the ratio $\frac{Q_s}{Q_0}$ of any individual pile in-group.

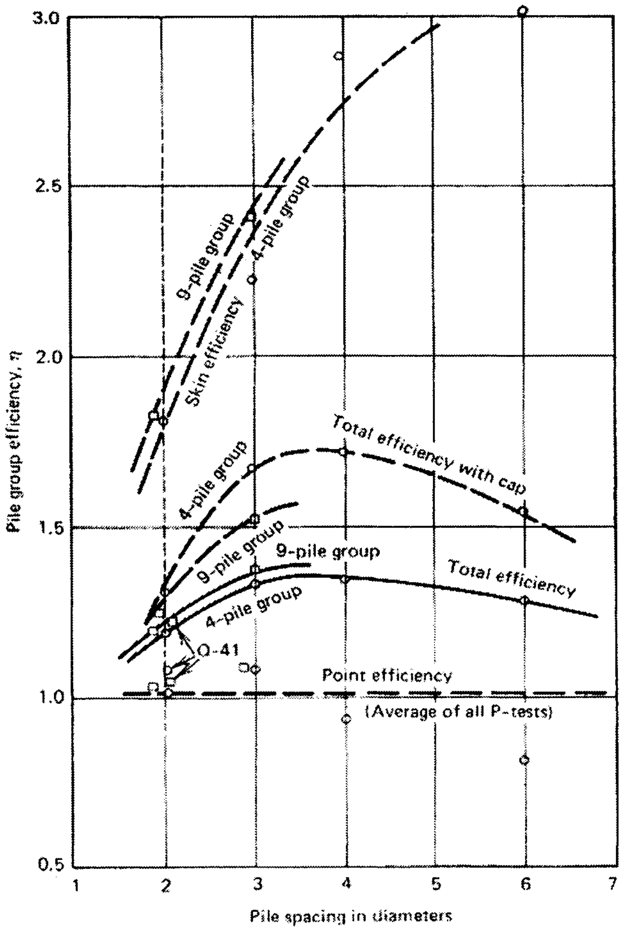


Figure 2.8: Pile group efficiencies (Vesic, 1968)

Figure 2.8 represents the relation between the pile spacing in group and pile efficiency. From this Figure it can be noted that:

(a) The maximum efficiency of 4 piles groups with cap embedded in homogeneous soil is 1.7 at pile spacing equals 3d to 4d. By increasing the pile spacing more than 4d tends to decrease the overall efficiency.

(b) Skin efficiency is much higher than that for the point efficiency & it reaches a maximum value 3 at pile spacing of 5d, which imply that point loads are unaffected by the group action.

(c) There is a small difference between the 4 and 9 group pile efficiencies except for the cap contribution, where the total overall efficiency of the 9-pile group is higher than that of the 4-pile group.

The equation proposed by Vesic is useful for calculation of the group efficiency of pile group where a considerable percentage of loads are carried through skin friction. For point bearing pile group, the term $\frac{\sum Q_s}{\sum Q_0}$ equals to zero & accordingly the group efficiency equal to unity. The proposed formula by Chelli's 1969, Vasic (1981), accounts the frictional effect, geometry and group interaction, but they did not consider the pile cap, to the overall capacity of the pile group.

Hanna et al (1972) studied the behavior of groups of plate anchors embedded in sand. They examined the group efficiency, load distribution among the anchors of the group, sand movement among the group, group size and configuration. They examined that the efficiency of the group was generally less than 100% for $H/B \leq 12$ and $S/B \leq 4$.

Where

S= the center-to-center spacing between the anchors

H=depth of anchor

B=diameter of anchor

Hanna et al (1972) also concluded that the load distribution among the anchors is almost uniform at a low load level, and the central core of the anchor carries the least load when approaching the failure point. It should be mentioned here however that they did not consider the cap condition, size of the group and the state of the sand packing.

Tejchman (1973) study the capacity and pile group efficiency, Pile action in-group and load distribution pattern within piles in-group. He performed laboratory tests on model test of pile groups driven in loose and compacted sand. He examined various types of pile groups; square; rectangular; and single row. Piles were made of pre-cast concrete having a square cross section (35x35 mm). Tests were conducted on pile groups of different arrangements including: 2x2 & 3x3 Square groups, 2x4 rectangular groups, and 1x4 line groups. The relative density (D_r) of the loose and the dense sand were 0.226 and 0.557 respectively. The piles were spaced at 2B, 3B, 4B, 5B, 6B and 9B. In these tests, piles were loaded until the ultimate capacity of the group was reached. He concluded the following:

a) For both dense and loose conditions, reducing the spacing between piles in a group leads to an increase of the group efficiency for all group arrangements except for the (1x4) line group embedded in dense sand.

b) Settlement of pile groups is greater than single piles under the same load. This becomes more pronounced by increasing the number of piles.

c) In case of loose sand, maximum group efficiency (η) was reached at pile spacing (s) equals to two pile width (B), decreasing linearly by increasing the pile spacing until it reaches the efficiency equal to unity at $S>6B$.

While Tejchman (1973) focused his study on the effect of pile spacing, type of soil, and pile group arrangement, he ignored the contribution of the method of installation, order of pile driving, pile length diameter ratio (L/d), and cap condition.

Garg (1979) conducted a number of field tests on single piles and two, four and six pile groups spaced at 1.5D, 2D and 2.5D; the pile diameter (D) of 150 mm and the embedment depth (L) of 3000 mm were used all over the testing program. He considered two cap condition i.e. cap resting directly on soil and freestanding cap. In this study, he focused on providing an approach for the design of bored undreamed pile groups and also to study the load-displacement mechanism of pile groups. The test results are summarized in Figures 2.9 and 2.10.

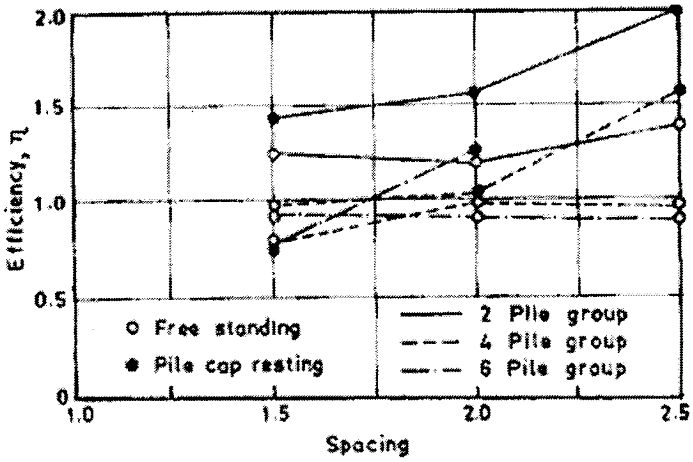


Figure 2.9: Effect of pile spacing on group efficiency (Garg, 1979)

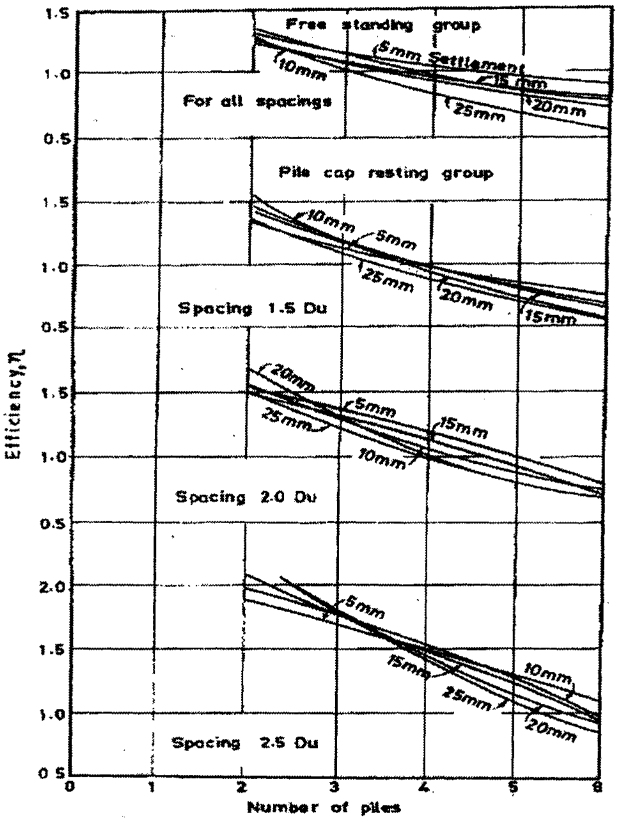


Figure 2.10: Pile group efficiency versus the number of piles in the group (Garg, 1979)

From the Figures 2.9 and 2.10 it can be noted that:

(a) The group efficiency increases due to an increase of pile spacing, for the case of pile cap resting condition, Figure 2.09.

(b) Group efficiency decreases with the increase of the number of piles for both freestanding and resting cap conditions, Figure 2.10.

(c) Load carrying capacity of pile group for cap resting condition is higher than those for freestanding condition, for same settlement value.

The study was limited to square pile group arrangements.

Pauls and Davis (1980) defined the pile group efficiency as:

$$\frac{1}{\eta_g{}^2} = 1 + \frac{(m-n)^2 . Q_0{}^2}{Q_B{}^2} \qquad 2.8$$

Where:

η_g = group efficiency

Q_0 = the ultimate load capacity of the single pile

Q_B = the ultimate load capacity of a pile group

m = number of piles in the row

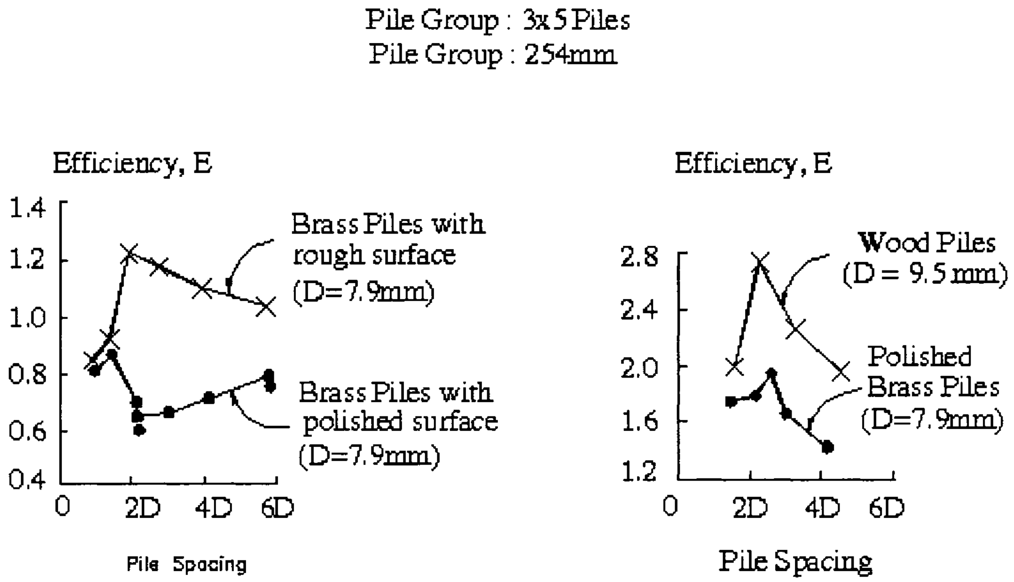
n = number of piles in the column

The proposed empirical equation takes on consideration the planar geometry of group i.e. pile spacing, pile diameter & number of pile in-group. It did not consider the other parameter like: cap condition, soil condition & pile-length to diameter ratio (L/D).

Broms (1981) has tested brass piles with rough surface (D=7.9mm), wooden piles (D=9.5mm) and brass piles with polished surface (D=7.9mm). He reported that the capacity of concentrically loaded parallel piles is normally larger than the capacity of the single pile under the same unit load. The efficiency (η) of the pile group is, in general larger then 1 in contrast to pile groups with friction piles in clay. This increase of the group capacity due to the increase in the number of piles in the group is normally not considered in the design. If the relative density of the sand is initially low, the sand around the adjacent piles will be compacted during the pile driving where, spacing of the piles is less then 7 to 8 pile diameters. From figure 2.11 he further concluded that:

(a) In dense sand, for fixed pile group of 3x5 and pile length of 254mm, the efficiency of rough surface piles was larger than that of polish surface, and the maximum efficiency was found at spacing of 2D. Figure 2.11(a)

(b) In loose sand wooden piles group showed higher efficiency than that of polish brass piles, and the maximum efficiency was found at spacing 3D. Figure (b)



(a) Dense Sand (b) Loose Sand

Figure 2.11: Capacity of pile groups in sand (after Broms, 1981).

Broms considered only the planar geometry of group i.e. pile spacing, pile diameter, the number of pile in-group and the soil condition. He did not consider however the other parameter such as: cap condition, type of loading and the pile-length to diameter ratio (L/D).

O Neill (1983) reported on a number of problems concerning pile group behavior utilizing the experimental data and analytical techniques available in the literature. He defined the pile-soil-interaction as consisted of two major components; namely the installation effects, which consist of soil stress changes due to piles installation closely to each other, and the mechanical effect, which consists of the superposition of pile strains and alternative of their zone due to the simultaneous loading of one or more neighboring piles. Most of the pile-soil-pile models are concerned with the second component and always neglect the installation effects, which makes them incomplete source of judgment that the designer cannot rely on.

He further reported that there is no mathematical expression can be considered as a reliable tool for the evaluation of η. This is due to mainly that none of these models has incorporated all the factors that govern the problem stated.

It also indicated that excessive error can be committed if pile driving order, geometric position of the piles in the group, influence of pile cap, rate of pore pressure dissipation, and variations of soil conditions across piles in the group are not taken into consideration while evaluating the load distribution among piles within a group.

Sayed and Bakeer (1992) proposed formula for the calculation of group efficiency of pile group's

$$\eta_g = 1 - \left(1 - \eta_s^{'}.K\right)\rho \qquad 2.9$$

Where

η_g = group efficiency

K = interaction factor

$\eta_s^{'}$ = geometric efficiency

$\eta_s^{'} = P_g / \Sigma P_p$

P_g = perimeter of the pile group

ΣP_p = the summation of the perimeters of the individual piles in group

$$\rho = \text{Friction factor } \frac{\sum Q_s}{\sum Q_0} = \frac{Q_s}{Q_0} \qquad 2.10$$

This empirical equation is extremely useful for computing the efficiency of pile group where a considerable percentage of loads are carried through the skin friction. For point bearing pile group, the term ρ becomes practically equals to zero. The proposed formula accounts for the friction effect, geometry and group interaction.

This formula considers new parameters; to include the geometric efficiency, the group interaction factor and friction factor. Geometric efficiency represents the planer geometry of the pile group. The frictional factor represents the effect of the three-dimensional characteristics, it accounts for the pile length as well as the property of the soil. The group interaction factor is a function of the method of installation, pile spacing and type of soil. Its values range from 0 to 1 according to the relative density of sand.

The equation didn't take an account the cap condition (Cap in direct contact with soil or freestanding) and distribution of load over the individual pile-group.

Hanna and Ghaly (1993) have conducted experimental investigations on single and group of vertical screw anchors installed in dense, medium and loose sand. Model tests were performed on anchor group 1x3, 2x2, 2x3, 3x3 with variable installation depths of 200mm, 400mm, 600mm, and 800mm and various angle of shearing resistance (φ) of sand of 42^0, 36^0 31^0. Table 2.2 and Figure 2.12 summarize these results. They reported that for medium and loose sands, the group efficiency increases with the anchor spacing and decreases with the group size. This was true for all tested depths, as the group efficiency is not much affected by the spacing between anchors for group installed in dense sand to relatively very shallow depths. Efficiency was more than 100% for group installed into relatively deep depths; it decreases with increasing the spacing.

Table 2.2: Efficiency η of anchor groups (After Hanna and Ghaly 1993)

Installation Depth (mm)	Group Configuration	$\phi = 31^0$			$\phi = 36^0$			$\phi = 42^0$		
		Spacing			Spacing			Spacing		
		3B	4B	5B	3B	4B	5B	3B	4B	5B
200	1x3	0.86	.94	1.00	0.89	0.95	1.02	0.98	1.03	1.04
200	2x2	0.83	0.88	0.97	0.85	0.93	1.00	0.97	0.99	1.05
200	2x3	0.79	0.84	0.89	0.81	0.86	0.92	0.91	0.93	0.96
200	3x3	0.71	0.78	0.82	0.78	0.82	0.88	0.87	0.91	0.94
400	1x3	0.85	0.91	0.97	0.88	0.92	0.98	1.11	1.1	1.12
400	2x2	0.82	0.87	0.95	0.84	0.90	0.96	1.10	1.08	1.08
400	2x3	0.77	0.80	0.87	0.79	0.83	0.89	1.05	1.05	1.04
400	3x3	0.70	0.73	0.78	0.72	0.79	0.85	1.01	1.02	1.00
600	1x3	0.82	0.88	0.96	0.88	0.95	1.02	1.25	1.18	1.10
600	2x2	0.78	0.86	0.94	0.84	0.93	1.01	1.21	1.16	1.09
600	2x3	0.72	0.78	0.82	0.79	0.84	0.9	1.14	1.09	1.01
600	3x3	0.64	0.70	0.74	0.71	0.79	0.86	1.10	1.02	0.96
800	1x3	0.80	0.87	0.91	0.83	0.91	1.04	1.4	1.31	1.19
800	2x2	0.76	0.82	0.89	0.79	0.89	0.98	1.36	1.28	1.18
800	2x3	0.70	0.72	0.79	0.73	0.80	0.89	1.29	1.20	1.09
800	3x3	0.60	0.65	0.71	0.68	0.75	0.82	1.28	1.18	1.08

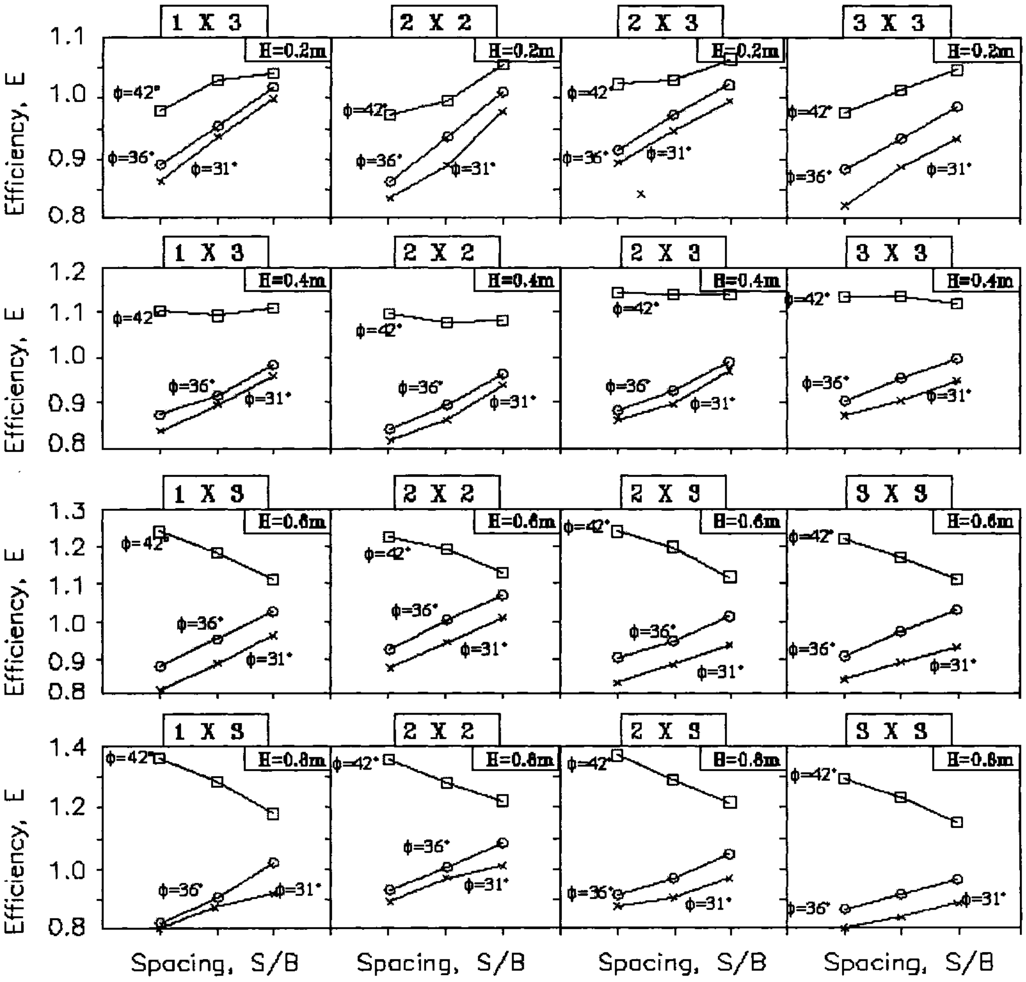


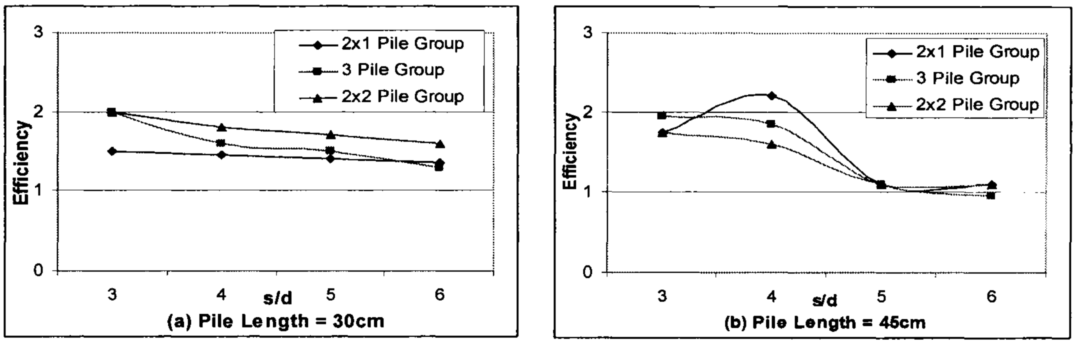
Figure 2.12: Efficiency of group anchors vs. Spacing (After Hanna and Ghaly 1993)

a) It can be noted from these results that for the installation depth of a group anchor between 200mm to 400mm, the efficiency increases linearly, where S/B is varies from 3 to 5. Efficiency will be higher when the angle of shearing resistance (φ) of the sand is higher and it decreases with the decrease of the angle of shearing resistance.

b) For the installation depth of a group anchor between 600mm to 800mm the efficiency decrease linearly, when the angle of shearing resistance (φ)=42^0, where S/B varies within the range of 3 to 5. In case of (φ) =36^0, =31^0 group efficiency increased linearly from less than unity to greater than unity, where S/B varies form 3 to 5.

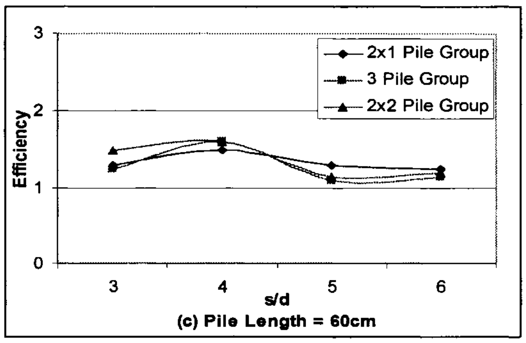
For the determinations of group efficiency Hanna and Ghaly considered e spacing of the anchors within the group, angle of shearing resistance and the anchor depth, however they did not consider the sequence of installation, pile head condition and the loading condition.

Chattopadhyay (1994) conducted a number of model test on pile groups in order to analyze their uplift capacity. Model piles had a diameter (d) of 19mm and an embedment length ranging between 300mm to 600mm. Three pile groups consisting of 2x1 line group, 3 triangular groups, and 2x2 square groups were tested for pile spacing ranging between 2.3d to 6d. The First series of the test was conducted on the dry sand with D60 = 95mm, D10 = 48mm, and cu = 1.98. For the second series of the test, locally available blackish grey clayey site soil having 8% clay and 92% silt was use. Model test results in dry sand are shown in Figure 2.13.



(a) Pile Length = 30cm

(b) Pile Length = 45cm



(c) Pile Length = 60cm

Figure 2.13: Efficiency of pile groups versus spacing of piles driven in sand (After Chattopadhyay 1994)

It was reported that:

(a) For the same spacing, in case of uplift loading of the three pile groups: 2x1 line, 3 triangular, and 2x2 square group, the pile head displacement which caused uplift failure was less for smaller depths of embedment.

(b) For all depths of embedment as well as for all group arrangements, the pile group efficiency was greater than unity. This value decreased by increasing the pile spacing, and reached unity at pile spacing of 6d.

(c) Maximum group efficiency was reached at a depth of embedment of 30cm and pile spacing equals 2.3d.

In this study, Chattopadhyay considered the spacing of piles, pile length & thee type of small group of pile. He did not consider friction factor, cap condition of group & loading of pile.

Liu et al (1994) conducted a large-scale field test on pile groups in order to analyze the cap-pile-soil interaction effects on their side and point resistances. They also studied the influence of pile spacing on the soil reaction beneath cap. Soil around the pile's shaft and beneath their caps consisted of soft mucky. Steel pipe piles of 0.1 m in diameter and 4.5 m length were used. During these tests, load cells were installed along the shafts in order to measure the load at the pile top, as well as the side and point resistances.

Based on the above observations, it was found that the conventional pile group efficiency (η) models do not take into consideration the effect of cap-pile-soil interaction. Therefore, separate coefficient of group efficiency was introduced for the side resistance (η_s), point resistance (η_p), and for the soil reaction beneath pile cap (η_c). These coefficients are calculated as follows:

$$\eta_s = G_s.C_s \tag{2.11}$$

$$\eta_p = G_p.C_s \tag{2.12}$$

$$\eta_c = \frac{\eta_c^{ex}.A_c^{ex}}{A_c} + \frac{\eta_c^{in}.A_c^{in}}{A_c} \tag{2.13}$$

Where,

G_s = side resistance effect coefficient

C_s = cap effect coefficient of side resistance ($C_s = 1$ for high-rise cap)

G_p = point resistance effect coefficient of pile group

C_p = cap effect coefficient of point resistance ($C_p = 1$ for high-rise cap)

η_c^{ex}, η_c^{in} = soil reaction effect coefficients of pile group in external and internal districts of cap

A_c^{ex}, A_c^{in} = the net areas of external and internal districts of cap

A_c = total net area of cap

The above-mentioned coefficients can be calculated as function of the pile group geometry as follows:

$$G_s = 1.2 - 1.2D/S \tag{2.14}$$

$$C_s = 1 + 0.12\left(\frac{S}{D}\right)^{\frac{1}{2}} - 0.5\left[\left(\frac{B_c}{L}\right) - Ln\left(0.3\frac{B_c}{L} + 1\right)\right] \tag{2.15}$$

$$G_p = 6\frac{D}{S} - Ln\left(e - \left(6 - \frac{S}{D}\right)\frac{1}{6}\right) \tag{2.16}$$

$$C_p = 1 + 0.1\left(\frac{D}{S}\right)Ln\left(0.5\frac{B_c}{L} + 1\right) \tag{2.17}$$

$$\eta_c^{ex} = \frac{\left(\frac{S}{D}+2\right)}{8} \qquad 2.18$$

$$\eta_c^{in} = 0.08\left(\frac{S}{D}\right)\left(\frac{B_c}{L}\right)^{\frac{1}{2}} \qquad 2.19$$

Where,

S = pile spacing

D = pile diameter

$\frac{B_c}{L}$ = The ratio of the width of cap to the length of pile

e = the natural logarithm base, let e = 2.718

The proposed model carries the advantage of considering the cap-pile-soil interaction, which was ignored by the conventional model. However, the complexity of calculation is considered as a major shortcoming.

Geddes and Murray (1996) reported that the ultimate lateral capacity of pile group depends on the length-to-diameter ratio of pile, pile frictional angle, pile group geometry, spacing of piles in group and sand placement density. It was noted that the group efficiency increases with an increase in pile spacing, for L/d = 12, there will be no group effect at spacing 6d and above. For L/d = 38, the group efficiency decrease with an increase in piles in-group configuration.

Mukherjee (1996) carried out an experimental investigation in order to study the pullout capacity of piles per pile groups in sand. The results of his study showed that the behavior aspects of pile per groups in terms of several parameters involved. The experiment was carried out in a segmented aluminums tank of size 900mm x 900mm x

1100mm width, length and depth respectively. Model piles of 25.4mm diameter were tested under different embedment length, pile spacing and group geometry. The values used for the embedment length were 600mm, 750mm and 900mm and for pile spacing were 75mm, 100mm, and 125mm. Line group of 1x2, triangular group of 1x3, square group of 2x2 and rectangular group 2x3 were tested. Based on the results of his study, the following conclusion can be drawn:

(a) For any type of group arrangement, group efficiency is proportional to pile diameter/pile length and spacing/pile diameter ratio.

(b) In a pile group at failure, the central pile carries the least load while the corner pile carries the highest load.

(c) In this investigation; the pile diameter was same all over the testing program and the order of pile driving and the pile-cap-soil interaction was not considered.

Das (1998) conducted a study to determine the capacity of pile groups. He reported that, when piles are placed close to each other, the stresses transmitted by the piles to the soil will overlap and reduce the capacity of the pile group. In practice, the minimum center-to-center spacing, s taken as 2.5d, and in ordinary situations, it is actually about 3-3.5d.

Das defined the efficiency of pile group as

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} \qquad 2.20$$

Where:

η = Group efficiency

$Q_{g(u)}$ = The capacity of the pile group

Q_g = The capacity of individual pile

Thus:

$$\eta = \frac{2(n_1 + n_2 - 2)d + 4d}{\rho n_1 n_2} \qquad 2.21$$

Where:

n_1 = number of rows

n_2 = number of columns

d = center-center spacing of piles

ρ = Perimeter of individual pile

Das's (1998) empirical equation considered only the planar geometry of the group i.e. pile spacing, pile diameter and the number of pile in-group. It did not however consider the other parameter such as: cap condition, soil condition, type of loading and pile-length to diameter ratio (L/d).

Seidel (1998) considered each individual pile in-group supported by the surrounding and underlying soil. Conversely, the pile imposed a reason of stress influence on the soil; this stress will decrease with the increase of spacing of the pile. For piles in-group, the stresses will overlap, and the capacity of the soil to support the piles may be reduced. For closely spaced piles, both freestanding and capped groups give very similar response. However, for the freestanding pile group, once individual behavior takes over for greater spacing, the rate of increase of the efficiency decreases. For the capped groups, the rate of increase of the group efficiency continues as before because of larger pile cap influence.

In any event, pile group capacity may be less than the sum of the capacities of the individual piles, which make up the group, furthermore:

$$P_{ug} \neq nP_{up} \qquad 2.22$$

P_{ug} = capacity of the pile group

P_{up} = capacity of the individual pile

N = number of piles in the group.

The model incorporates the effects of the soil condition, pile cap and planar geometry of group i.e. pile spacing, pile diameter and number of pile in-group. It did not however consider the other parameters such as: type of loading and pile-length to diameter ratio (L/D).

Mokwa (1999) conducted several experimental studies for computing the efficiency of pile groups. He introduced some factors that most significantly affect the overall group efficiency, these are:

a) Pile spacing
b) Group arrangement
c) Group size
d) Pile head fixity
e) Soil type and density
f) Pile displacement

Mokwa however did not consider the loading condition of the pile, pile driving sequence, length of pile, number of pile under the cap and the settlement of the individual pile and the pile group.

Ismael (2001) conducted tests on single pile and group of 3 piles & 4 piles in sand spaced at 2d and 3d respectively. He found that 70% of the ultimate load was transmitted through the side friction that was uniform along the pile shafts. The calculated pile efficiency was 1.22 and 1.93 for piles spaced at two and three pile's diameter respectively. However, loose sands and where high water level and soil relaxation occurs, efficiency <1 may be obtained.

The study did not take into consideration the effects of the number of piles on the group efficiency. Neither the cap-pile-soil interaction nor the pile length-diameter (L/d) ratio was considered for the group efficiency evaluation.

Zhang (2001) developed a data based on the field and model test results reported by Whutaker (1960), Sowers et al. (1961), Kishida (1967), De Mello (1969), Brand et al (1972), Vesic (1970, O'Neill (1983) Liu et al (1985) and other investigators. In this data, the piles spacing varies from 2 to 6 pile's diameter, the pile cap was either freestanding or in contact with ground. He reported that pile group with cap-ground contract generally exhibits larger efficiency because of the cap-soil interaction. The group efficiency factor (η) of most of the freestanding pile groups was greater than unity in cohesionless soil due to soil densification, and smaller than unity in cohesive soil. Group efficiency factor of long piles group with cap-ground contact are essentially taken to be same as those for freestanding pile group.

Hanna et al (2004) developed a model by Artificial Neural Network to predict the group efficiency of pile foundation on sand. Predication of group efficiency by the model was more accurate than that of the other conventional model. The governing parameter used in the model was method of installation, soil condition, type of loading, pile cross-section, pile length/diameter ratio, pile spacing/diameter ratio and pile arrangement. The model didn't give information about the distribution of load along the individual pile in-group. The sequence of pile driving was not considered in that model.

2.3 Discussion

Based on the literature review reported herein on the performance of pile groups driven in cohesion-less soil, it can be concluded that the empirical formulas available for prediction of the pile group efficiency (η) have serious shortcomings. Eventually the studies reported had overlooked some of the governing parameters, which are believed to affect the performance of these piles; the parameters include: pile spacing, pile length, group arrangement, cap condition, method of installation, order of pile driving, and soil characteristics. This is evident in the discrepancies among the results produced by different formulae; (Bolin, 1941; Polous and Davis, 1980; Sayed and Bakeer, 1992; Liu et al, 1994; Das, 1999).

Therefore, the present investigation has been directed to develop a numerical model to simulate the problem stated, taking into consideration the parameters mentioned-above. Furthermore, analytical model will be developed for predicting the group efficiency for line pile groups in sand.

CHAPTER-3

NUMERICAL MODEL

3.1 General

This study is directed to the prediction of the efficiency of pile groups embedded in loose to dense sands. For this purpose, a numerical model was developed to simulate the problem stated. After validation, the numerical model was then used to generate data, which was used to predict the different behaviors of pile group. In this model, the effects of the pile cap thickness, pile spacing, group geometry, and the angle of shearing resistance of the sand on the group efficiency will be incorporated that were ignored in the past.

3.2 Numerical model

The finite element method has proven to be a powerful technique to solve varieties of science and engineering problems. In recent years, finite element technique has become very popular in the field of geotechnical engineering to provide solution of various problems including soil-structure interaction, earth and earth retaining structures.

In this investigation, finite element technique was used to model both the soil and the pile group that allows rigorous treatment of soil–pile-cap interaction.

3.2.1 Mesh Generation

The mesh generator require a general meshing parameter which represents the average element size, l_e. In the model, l_e parameter is calculated from the outer geometry dimensions $(x_{min,} x_{max,} y_{min} y_{max})$ and a Global coarseness setting as defined in the mesh:

$$l_e = \sqrt{\frac{(x_{\max} - x_{\min})(y_{\max} - y_{\min})}{n_c}} \qquad 3.1$$

Where, n_c is the global coarseness number.

Distinction was made between the five levels of global coarseness of the mesh: very course, course, medium, fine and very fine. The selection of mesh global coarseness reflects a compromise between an acceptable degree of accuracy and computing time. In the present study global coarseness was taken as medium. The average element size and the number of generated triangular elements depend on this global coarseness setting. No. of elements for various types of global coarseness is given in Table 3.1.

Table 3.1: Number of elements for global coarseness

Global Coarseness	**n_c**	**No of elements**
Very course	25	50
Course	50	100
Medium	100	250
Fine	200	500
Very fine	400	1000

In this investigation, meshes composed of 15-node triangular elements have proven to provide finer distribution of stress-strain and thus more accurate results. Figure 3.1 shows a typical mesh before load application. It should be noted that the mesh has smaller elements in the region near the piles and pile cap where rapid changes in the stress/strain is expected. Figure 3.2 shows the deformed mesh after applying the load.

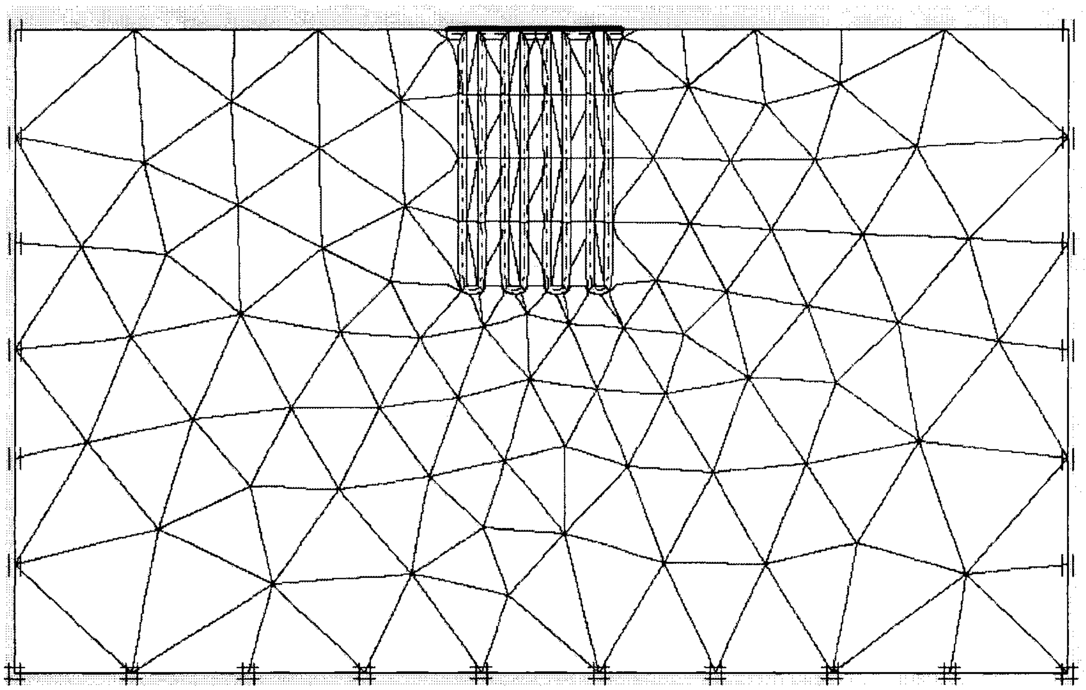


Figure 3.1: Mesh generation before load application (1x4 group)

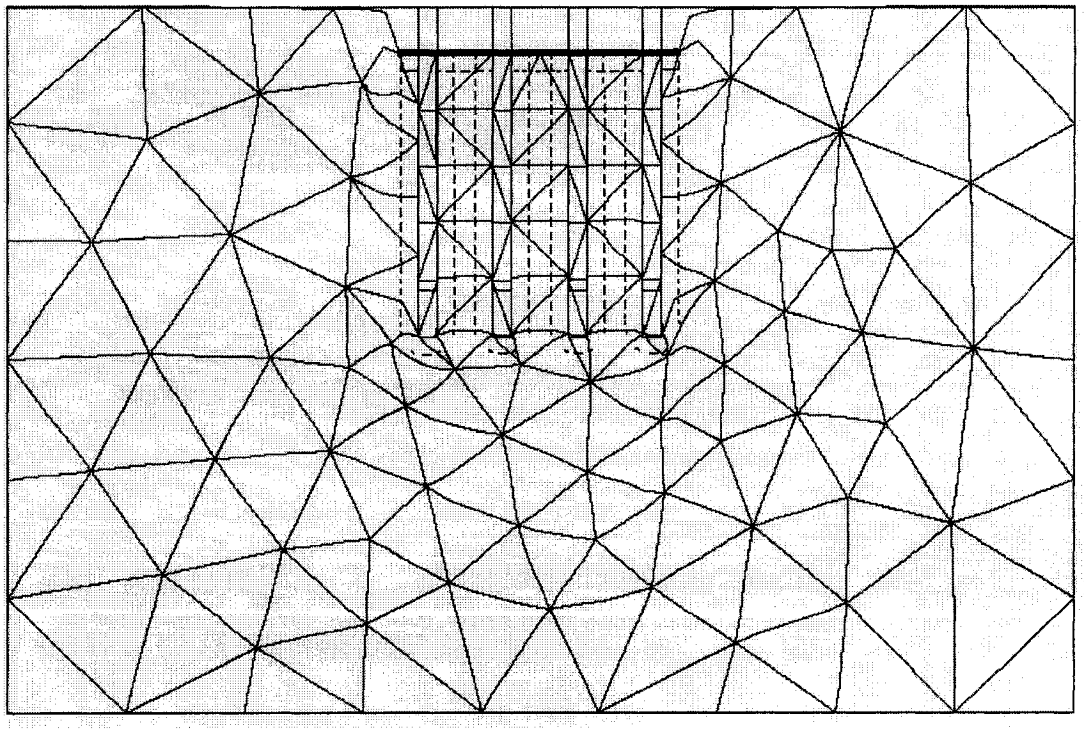


Figure 3.2: Deformed Mesh for 1x4 pile group

A trial calculation on half and full size mesh revealed reasonable agreement, with a maximum level of tolerance of 3 to 5%. Due to that level of tolerance, some elements did not show symmetry.

3.2.2 Boundary conditions

Two dimensional finite element meshes were developed, having a width ≥ 50 times the pile diameter or width of the pile, and a height = L+1.5L (L is the length of pile). The sides of the mesh were hinged support and the bottom was fixed. The parameters incorporated in this model are the angle of shearing resistance, pile spacing, cap thickness, axial load and pile length. The plan of the pile groups and the respective caps considered in this study were shown in Figure 3.3. The pile cap was modeled as a diaphragm wall made of steel material. Pile material was same as the pile cap. The loading was given in single stage but the sequence of construction was multistage. Water level was not considered in this model i.e. the sand was considered dry. The basic geometry & boundary conditions of pile group is shown in Figure 3.4.

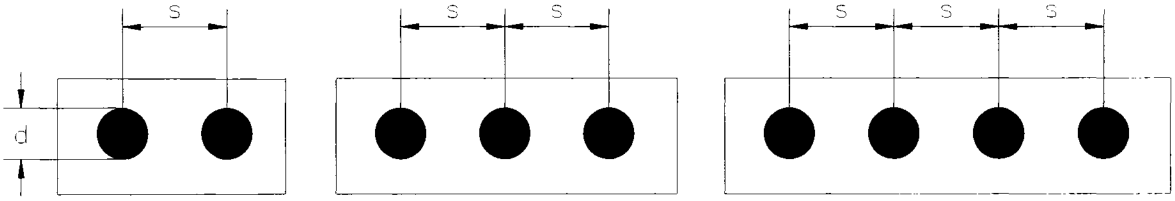


Figure 3.3: Plan of 1x2, 1x3 & 1x4 pile group

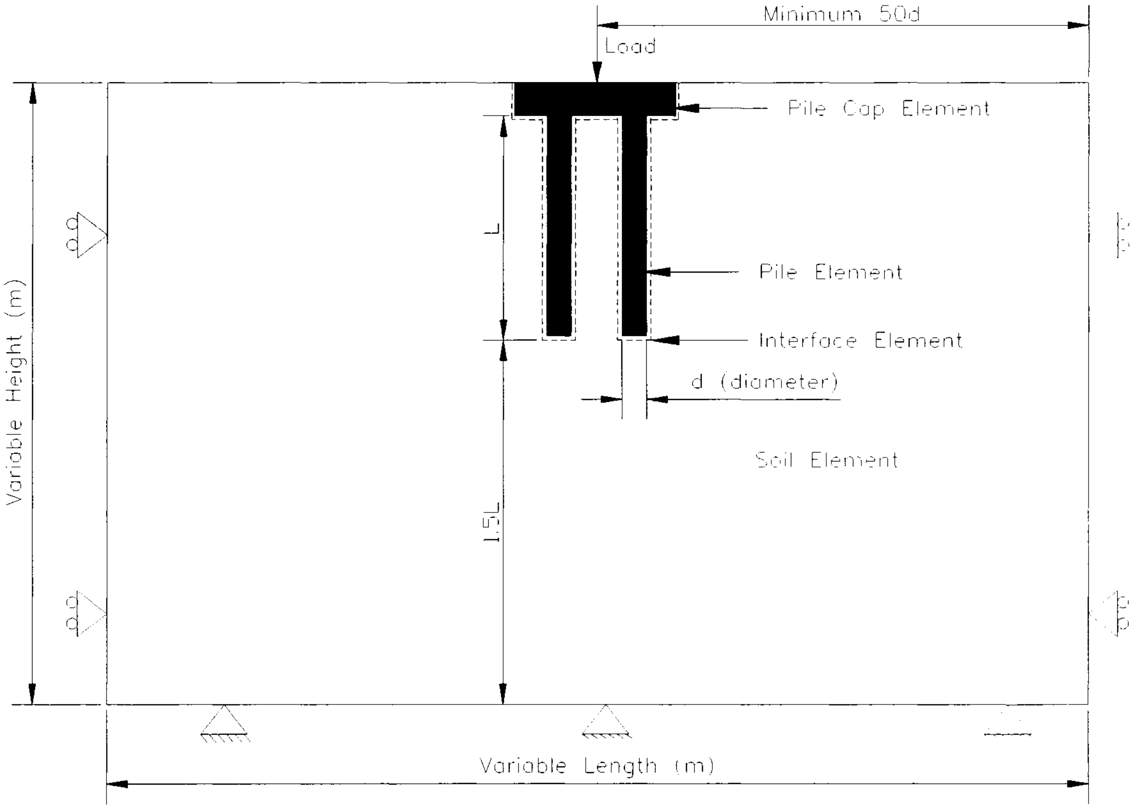


Figure 3.4: Typical geometry and boundary conditions of the numerical model.

3.2.3 Constitutive laws

The elastic-plastic Mohr-Coulomb model was used in the present study. The model involved five input parameters:

E =Young modulus

ν = Poisson's Ratio

ϕ = Angle of shearing resistance

c = Cohesion

ψ = Dilatancy angle

Mohr-Coulomb equation represents the relation between the shear and the normal stresses of the soil mass. It can be expressed as:

$$\tau = c + \sigma \tan(\phi) \qquad 3.2$$

Where,

τ = Shear stress

σ = Normal stress

c = Cohesion

ϕ = Angle of shearing resistance

Clay soils tend to show little dilatency (ψ = 0) apart from the heavily over-consolidated layers. The dilatency of sand depends on both the density and on the frictional angle. For quartz sands the order of magnitude is $\psi = \phi - 30$. However, for ϕ-value less than 30°, the angle of dilatency is mostly zero. A small negative value of ψ is only realistic for extremely loose sands.

This Mohr-Coulomb model represents a "first-order" approximation of sand behavior. For each layer, estimation of the average stiffness should be made. This will allow the computations to be relatively fast. Besides the five model parameters mentioned above, initial soil conditions and void ratio played an essential role in the sand deformation. Figure 3.5 shows the yielding stress condition for Mohr-Coulomb's envelope.

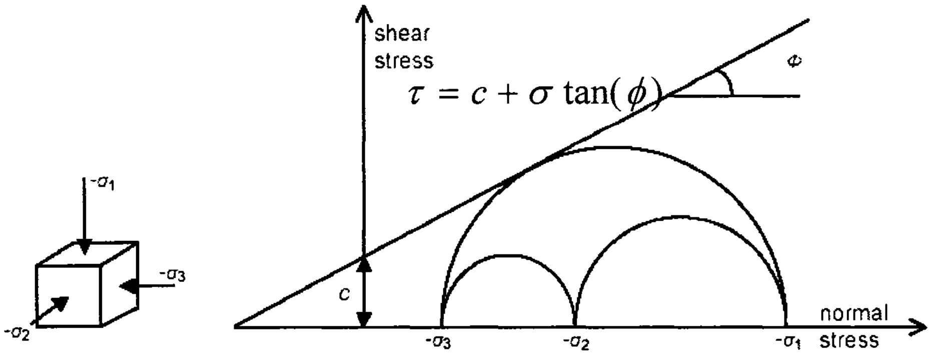


Table 3.5: Coulomb's envelope

3.2.4 Soil Elements

The soil consists with single layer of sand. 15-node triangular elements were used to model the soil material and other volume of clusters. It provided a fourth order interpolation for the displacements and the numerical integration of gauss point (stress points). These elements produce high quality stress analyses for difficult problems. Figure 3.6 shows the node and stress points of 15-nodes triangular element. Input data sets for soil are given in Table 3.2.

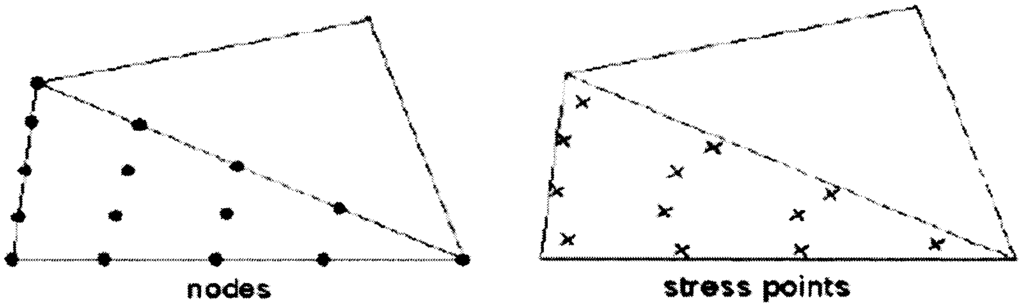


Figure 3.6: Position of nodes and stress points in soil elements (material model, plaxis-8)

Table 3.2: Soil element properties

Parameter	Sand Layer	Unit
Material model	Mohr-Coulomb	-
Type of material behavior	Drained	-
Soil unit weight (γ_{dry})	Variable	Kn/m^3
Soil unit weight (γ_{sat})	Variable	Kn/m^3
Young's modulus (E_{ref})	Variable	Kn/m^2
Poisson's ratio (v)	variable	-
Angle of shearing resistance (ϕ)	Variable	^{0}
Strength reduction factor inter. (R_{inter})	.67	^{0}

3.2.5 Pile and pile cap Elements

Elastic steel material was used to simulate the pile and pile cap material. Tables 3.3 and 3.4 show the material properties of pile and pile cap. Weight of the pile cap was relatively small which was ignored during model analysis. The cap was modeled as a plate with flexural rigidity (bending stress) EI and axial stiffness EA. From these two parameters equivalent plate thickness d_{eq} was calculated from the following equation:

$$d_{eq} = \sqrt{12\frac{EI}{EA}} \tag{3.3}$$

Table 3.3: Properties of the pile material

Parameter	Value	Unit
Type of behavior	Linear Elastic	
Material Unit weight (γ_{dry})	76.87	Kn/m^3
Equivalent thickness (D)	Variable	m
Young's modulus (E_{ref})	2x10^{11}	Kn/m^2

Table 3.4: Properties of the pile cap material

Parameter	Value	Unit
Type of behavior	Linear Elastic	
Normal stiffness (EA)	Variable	Kn/m
Flexural rigidity (EI)	Variable	Kn/m^2
Equivalent thickness (D)	Variable	m

3.2.6 Interface Elements

Interface was made of 10-node elements, with 5 stress points. Figure 3.7 shows connection between the soil and the interface elements. The interface elements were assumed to behave elastically to simulate the interface between the soil and the pile, where a range of roughness varies from smooth and fully rough. The roughness level of the interaction elements was taken by choosing a suitable value for the strength reduction factor (R_{inter}).

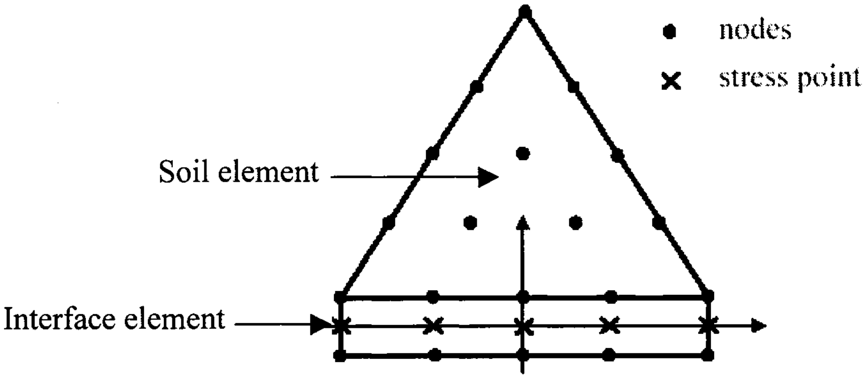


Figure 3.7: Distribution of nodes and stress points in interface elements and their connection to soil elements

The interface properties are calculated from the soil properties in the associated data set and the strength reduction factor by applying the following rules:

$$\tan\varphi_i = R_{inter}\tan\varphi_{soil} \le \tan\varphi_{soil} \qquad 3.4$$

$$\psi_i = 0 \text{ for } R_{inter}<1, \text{ otherwise } \psi_i = \psi_{soil}$$

Where,

φ_i = Frictional angle at the interface

φ_{soil} = Frictional angle of the soil.

ψ_i = dilatency angle of the interface

ψ_{soil} = dilatency angle of the soil

Values of the strength reduction factors are given in the table 3.5 for different materials. The R_{inter} value for the present model was taken = 2/3.

Table 3.5: Same values of strength reduction factor (R_{inter})

Material	R_{inter}
Sand/Steel	2/3
Clay/Steel	0.5
Sand/Concrete	1-0.8
Clay/Concrete	1-0.7

Each interface was assigned a 'virtual thickness', an imaginary dimension used to define the material properties of the interface. In the present investigation, virtual thickness was taken = 0.1.

3.2.7 Stage Construction

Stage construction is one the most advanced type of loading input. In the present numerical model stage construction procedure was introduced. The stage construction enables an accurate and realistic simulation of the various loading, construction and excavation processes. In the present numerical analysis stage construction in the pile considered as the sequence of pile driving. This was never being used in the numerical analysis of group pile. In this study, it was considered for the first time and used in the modeling. Figure 3.8 shows the sequence of the pile construction in the numerical model. The construction can start any side of the group. In case of square and rectangular pile group, the sequence of pile driving may be side-by-side or diagonal.

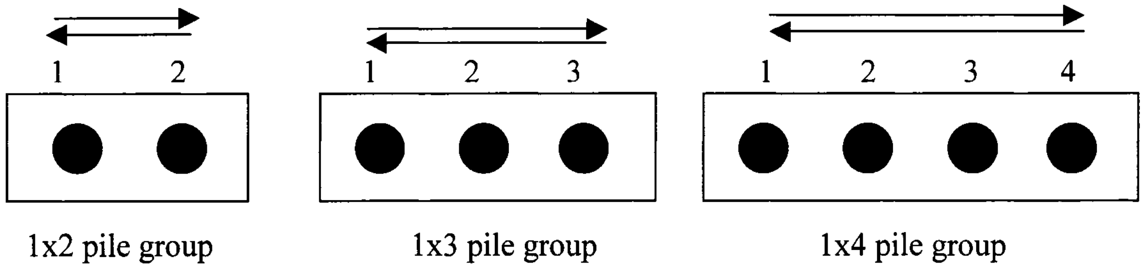


Figure 3.8: Sequence of stage construction for 1x2, 1x3 and 1x4 pile groups

3.3 Numerical Analysis

PLAXIS is a special purpose two-dimensional finite element computer program used to perform deformation and stability analyses for various types of geotechnical engineering applications such as single and group of piles, pavement, retaining wall, static and dynamic analysis of compaction. In the present study, plane strain numerical model was developed using a uniform cross section and stress state and loading scheme over a length perpendicular to the cross section (z-direction). Deformation along the z-axis was assumed to be zero. Figure 3.9 shows the basic geometry of the plan strain model. The program used a convenient graphical user interface that can quickly generate a geometry model and finite element mesh.

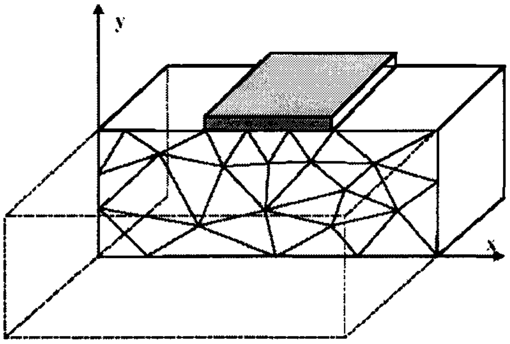


Figure 3.9: Plane strain model condition

3.4 Validation of the model

In order to validate the numerical model the experimental model by Tejchman (1973) for 1x4 group pile was considered. Parameters and results of the Tejchman, 1973 model and the numerical model are shown in Table 3.6 and Table 3.7. A comparison was made between the parameters and the results of the Tejchman model and the present investigation.

Table 3.6: Mechanical properties of Tejchman, 1973 model

	Test Parameter									
Test Number	Angle of shearing resistance	Bulk density γ_0	Void ratio e	Poisson ratio	Pile Width (d) m	Pile Spacing (S) m	S/D	L/D	Pile Group	Pile Length (L) m
1	28	1.51	0.759	0.2	0.035	0.070	2.0	15.0	1X4	0.525
5	28	1.51	0.759	0.2	0.035	0.105	3.0	15.0	1X4	0.525
9	28	1.51	0.759	0.2	0.035	0.159	4.5	15.0	1X4	0.525
1	32.5	1.6	0.664	0.3	0.035	0.070	2.0	15.0	1X4	0.525
5	32.5	1.6	0.664	0.3	0.035	0.105	3.0	15.0	1X4	0.525
9	32.5	1.6	0.664	0.3	0.035	0.159	4.5	15.0	1X4	0.525

Table 3.7: Comparison of the test results with Tejchman, 1973 model

		Ultimate Capacity		**Efficiency**	
Test Number	Angle of shearing resistance	Test Result (KN)	Numerial model result (KN)	Test Result	Numerial model result
1	28	3.3	3.36	1.2	0.96
5	28	3.4	4.9	1.15	0.85
9	28	3.6	6.3	1.1	0.96
1	32.5	7.8	6.3	0.8	0.88
5	32.5	9	9.1	0.85	0.90
9	32.5	10.5	11.55	0.9	0.91

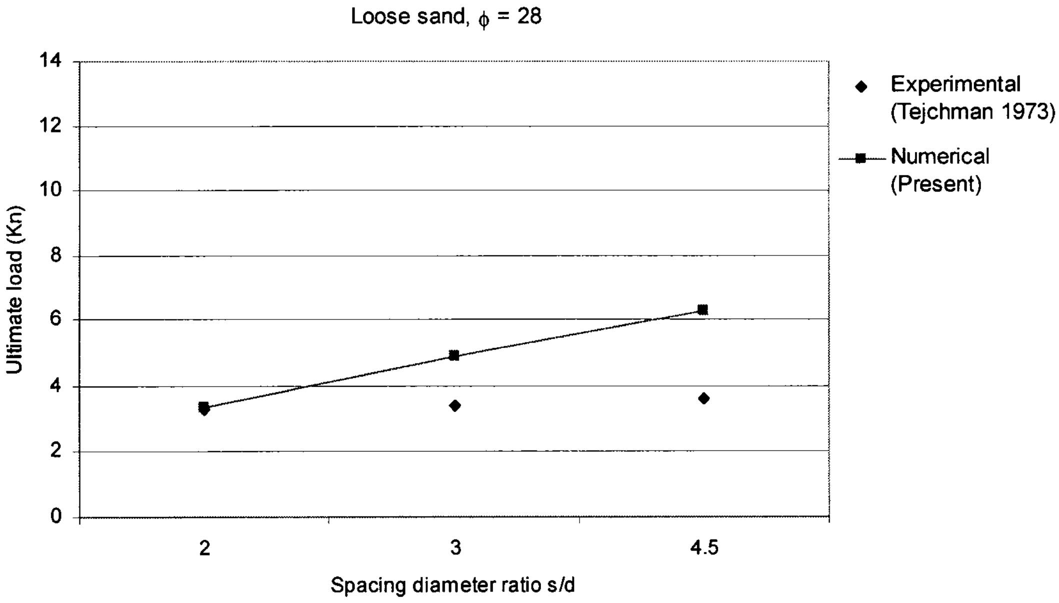


Figure 3.10: Comparison of Numerical and Tejchman, 1973 model for loose sand

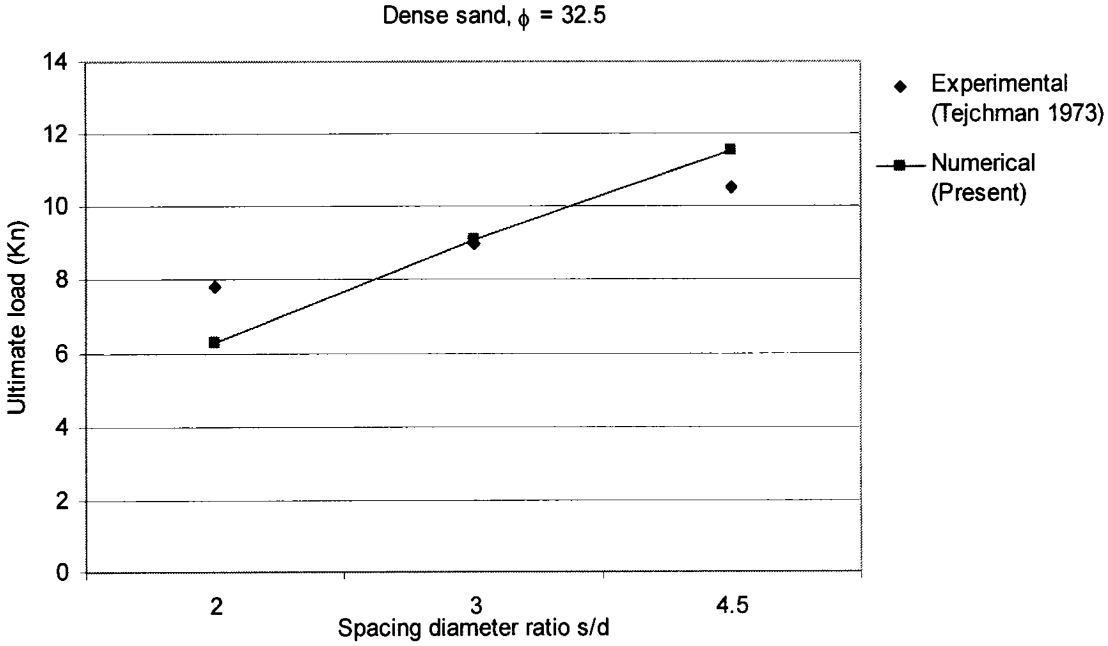


Figure 3.11: Comparison of Numerical and Tejchman, 1973 model for dense sand

From the comparison of the test and experimental model following observation were made:

a) For loose sand Figue3.10, the bearing capacity of the numerical model was much higher compared with the experimental model in case of s/d ratio more than 3. It is due to the fact that in the numerical model strain in the z direction is considered zero and hence it gives higher bearing capacity.

b) For dense sand Figure 3.11, the bearing capacity of the numerical and experimental model was very close in case of s/d ratio varies from 2 to 3. In case of s/d = 4.5, the bearing capacity of the numerical model was little higher.

c) For pile group efficiency Table 3.7 gave a very good comparison. Group efficiency calculated in the experimental model was higher because in the experimental model the individual pile bearing capacity was not considered. It only takes single pile bearing capacity then multiplied by the no of pile in the group.

From the above observations, it may be concluded that the numerical model was able to predict the group behavior of pile foundation successfully.

3.5 Parametric analysis

Preliminarily tests were performed to establish the sensitivity of the parameters, which was considered in this analysis for 1x2, 1x3 and 1x4 line pile groups.

3.5.1 Effect of Loading

The behavior of pile group under axial compressive loading is a complex problem as compared to the case of single pile, and has not been adequately examined and understood. In this investigation, line pile groups 1x2, 1x3 and 1x4 were tested under variable loading. The purpose of the analysis is to investigate the bearing capacity variation with variable axial compressive load. The pile cap thickness, angle of shearing resistance, pile length and pile spacing in the group were unchanged as 2.5d, 30°, 15d and 3d respectively. Test results for the 1x2, 1x3 and 1x4 line pile group are shown in Table 3.8. In the Table variation of tip resistance of individual in a group due to the change of applied load is given, it also shows the three-point deflection of pile cap due the various loading.

Table 3.8: Test results for Variable load

Length of pile (L) = 0.525m

Diameter of the pile (d) = 0.035m

Spacing-diameter ratio (s/d) = 3

Angle of shearing resistance (ϕ) = 30^{o}

Test No.	**Applied load (Kn) Q**	**Pile tip resistance (Kn)**				**Cap Deflection (mm)**		
						Point		
		Pile-1	**Pile-2**	**Pile-2**	**Pile-4**	**1**	**2**	**3**
1x2 Pile Group								
1	1.000	0.356	0.356			1.213	1.224	1.209
2	1.200	0.421	0.420			1.563	1.574	1.559
3	1.600	0.564	0.541			2.367	2.373	2.354
4	2.000	0.705	0.661			3.727	3.731	3.722
5	2.500	0.879	0.827			4.357	4.345	4.308
6.	3.000	1.051	0.991			5.544	5.526	5.481
7.	**4 Failed**	1.417	1.347			8.226	8.187	8.121
1x3 Pile Group								
8	2.000	0.444	0.579	0.444		1.974	1.976	1.923
9	2.500	0.543	0.722	0.549		2.608	2.597	2.532
10	3.000	0.644	0.872	0.657		3.262	3.250	3.183
11	3.500	0.747	1.017	0.763		3.929	3.908	3.833
12	4.000	0.852	1.165	0.863		4.638	4.606	4.520
13	4.500	0.955	1.319	0.975		5.369	5.335	5.247
14	5.000	1.063	1.480	1.086		6.146	6.116	6.123
15	5.500	1.166	1.636	1.196		6.924	6.898	6.905
16	6.000	1.270	1.793	1.306		7.721	7.694	7.701
17	7.000	1.490	2.115	1.521		9.455	9.427	9.434
18	**8 Failed**	1.589	2.269	1.618		10.563	10.529	10.537
1x4 Pile Group								
19	6.000	0.905	1.362	1.312	0.882	5.444	5.434	5.376
20	7.000	1.046	1.600	1.546	1.016	6.540	6.535	6.483
21	8.000	1.187	1.835	1.781	1.141	7.663	7.671	7.632
22	9.000	1.335	2.071	1.998	1.277	8.797	8.798	8.750
23	10.000	1.481	2.309	2.206	1.424	10.056	10.068	10.030
24	**11 Failed**	1.525	2.377	2.263	1.478	10.661	10.713	10.716

The variation of bearing capacity and deflection of the pile cap for 1x2, 1x3 and 1x4 pile groups, while loads are increasing shown in Figure 3.12 (a) to (f).

For group pile 1x2, axial loads on the cap were varied from 1 to 4Kn. Variation of the load and deflection is shown in the Figure 3.12 (a) and (b). It is shown in the Figure 3.12 (a) that the soil body was collapsed at axial load 4Kn. For further analysis, the applied load for pile group 1x2 considered 3Kn to be on the safer side.

For group pile 1x3, applied load was increased from 2Kn to 8Kn. The soil body was collapsed at the axial load 8Kn. From load 2 to 6Kn, the variation of the tip resistance and deflection at three points of pile cap increased linearly. However, from 7KN onwards it didn't follow the linearity and hence applied load was taken 6Kn. Variation of the load and deflection is shown in the Figures 3.12 (c) and (d).

For group pile 1x4, applied load was increased from 6Kn to 11Kn, at axial load 11Kn the soil body was collapsed. From load 6 to 9Kn, the variation of tip resistance and deflection at three points of pile cap increased linearly but from 9Kn to onward it didn't follow the linearity, so applied load was chosen 8Kn. Variation of the load and deflection is shown in the Figure 3.12 (e) and (f). From the above analysis the following observation were made:

a) For any type of line pile group, before reaching the ultimate load tip resistance of the piles increase linearly with the increase of load.

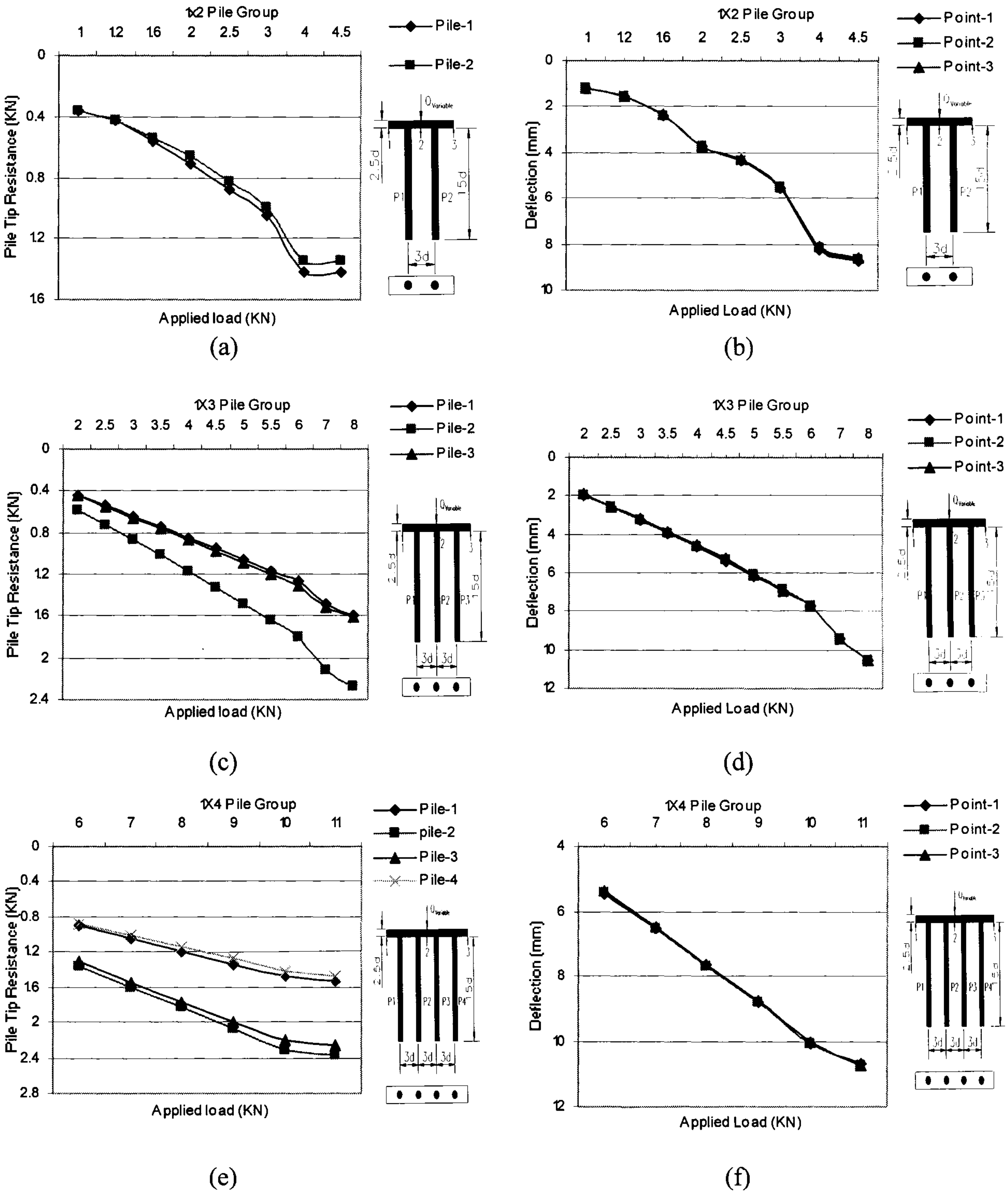
b) Although the study considered the effective cap thickness but tip resistance for individual pile in-group is not same. For 1x2 pile group, the pile-1 gives higher tip resistance than pile-2. This phenomenon was due to the stage construction and pile soil interaction between the piles in-group.

c) For pile group 1x3, the tip resistance of the middle pile is higher than the other two corner piles. Pile-1 and pile-3 took almost the same load.

d) For pile group 1x4, it was found that middle two piles gave higher tip resistance than the outer two piles.

e) Pile cap was considered as rigid, so the deflection of the three points for all the pile group arrangement is all most same.

f) Pile cap exhibits steady or constant deflection after the ultimate load because after the ultimate load tip gave a constant resistance.



(a) (b)

(c) (d)

(e) (f)

Figure 3.12: Tip resistance and deflection of 1x2, 1x3 & 1x4 group piles for variable loading

3.5.2 Angle of shearing resistance

Angle of shearing resistance has a great influence on the group pile capacity as well as efficiency. The value of shaft resistance of piles depends on the level of mobilization of the angle of shearing resistance. In the present study, three types of sand were used where angle of shearing resistance of were 30°, 32.5° and 35°. In this analysis the pile cap thickness, length of pile, spacing of the group pile was fixed and values were 2.5d, 15d and 3d respectively. Axial compressive load 3, 6 and 8Kn was applied for 1x2, 1x3 and 1x4 pile groups.

The detail test results of tip resistance of individual pile in a group due to the change of angle of shearing resistance are given in Table 3.9. It also shows the variation of three-point deflection of the pile cap

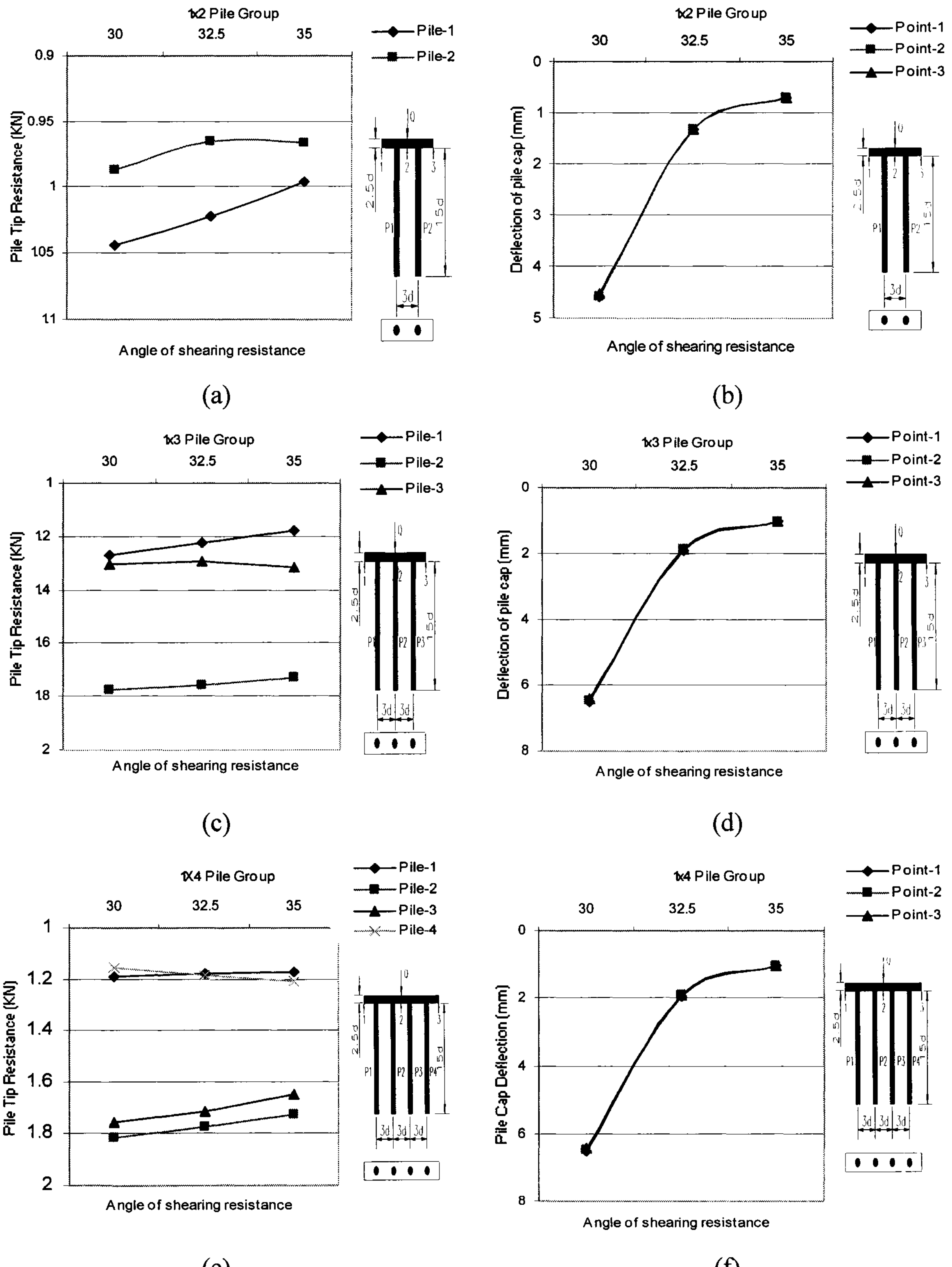
Table 3.9: Test results for Variable angle of shearing resistance

Length of pile (L) = 0.525m

Diameter of the pile (d) = 0.035m

Spacing-diameter ratio (s/d) = 3

Test No.	**Applied load (Kn) Q**	**Angle of shearing resistance**	**Pile tip resistance (Kn)**				**Cap Deflection (mm)**		
							Point		
			Pile-1	**Pile-2**	**Pile-2**	**Pile-4**	**1**	**2**	**3**
1x2 Pile Group									
25	3	30	1.044	0.987			4.588	4.577	4.544
26	3	3.2.5	1.022	0.965			1.329	1.330	1.325
27	3	35	0.996	0.967			0.704	0.707	0.705
1x3 Pile Group									
28	6	30	1.267	1.775	1.302		6.503	6.473	6.398
29	6	3.25	1.224	1.758	1.294		1.892	1.879	1.852
30	6	35	1.179	1.731	1.315		1.022	1.017	1.003
1x4 Pile Group									
31	8	30	1.193	1.817	1.755	1.154	6.494	6.482	6.429
32	8	3.2.5	1.179	1.775	1.713	1.185	1.938	1.919	1.886
33	8	35	1.173	1.729	1.651	1.208	1.048	1.037	1.017



(a) (b) (c) (d) (e) (f)

Figure 3.13: Tip resistance and deflection of 1x2, 1x3 & 1x4 group piles for loose to dense sand

The variation of pile tip resistance with the variation of angle of shearing resistance for 1x2, 1x3 and 1x4 pile groups are shown in the Figure 3.13 (a), (c) and (e), one the other hand the variation of cap deflection with the variation of angle of shearing resistance are shown the Figure 3.13 (b), (d) and (f). Findings of the analysis are given below:

a) Figure 3.13 (a) and (b) show the variation of tip resistance and three-point deflection of pile cap with angle of shearing resistance 30°, 32.5° and 35° whereas load, spacing and cap thickness were constant. Applied constant load was 3Kn. It is shown in the Figure 3.13 (a) tip resistance decreases with increase in angle of shearing resistance, because shaft resistance of the individual pile increases with the increase of angle of shearing resistance.

b) Due to the high compressibility of loose sand, pile cap deflection is very high for angle of shearing resistance 30° but in case of angle of shearing resistance 32.5° and 35°, deflections are small and very close for a constant axial load.

c) Figure 3.13 (c) and (d) shows the variation of tip resistance and three point deflection of pile cap for angle of shearing resistance 30°, 32.5° and 35°. Applied load, spacing and cap thickness are constant. Applied constant load was 6Kn. Middle pile gave higher tip resistance. The variation of the tip resistance and three-point deflection pattern was same as group pile1x2.

d) Figure 3.13 (e) and (f) shows the variation of tip resistance and three point deflection of pile cap for angle of shearing resistance 30°, 32.5° and 35° at a constant load, spacing and cap thickness. Applied constant load was 8Kn. Middle two piles gave higher tip resistance. The nature of the three-point deflection of pile cap was same as group pile 1x2 & 1x3.

e) Percentage of load distribution along the pile in-group is not same. However, the piles close to the applied load have taken higher load than that of the outer pile in the group. It happened because part of the applied load is directly transmitted through the piles, which are close to the applied load.

f) Three points deflection of pile cap for all the pile group arrangements are all most the same since pile cap was considered as rigid.

3.5.3 Pile cap Thickness

The stiffness of the pile cap influences the distribution of structural loads to the individual piles. The Rigidity of group pile mostly depends on the thickness of the cap and the cap material property. The thickness of the pile cap increases skin friction and end-bearing resistance and also influences the distribution of structural loads to the individual piles. In the present study various values of thickness of the pile cap were; 0.1d, 1d, 1.5d, 2.5d and 3.5d, where d is the diameter or width of the pile. In the study the angle of shearing resistance, length of pile, spacing of the group pile were fixed and the values were; 30°, 15d and 3d respectively. Applied axial load were 3, 6 and 8Kn for 1x2, 1x3 and 1x4 pile groups respectively. Table 3.10 shows the detail test result of tip resistance and deflection of pile cap for the variable cap thickness.

Table 3.10: Test results for Variable cap thickness

Length of pile (L) = 0.525m

Diameter of the pile (d) = 0.035m

Spacing-diameter ratio (s/d) = 3

Angle of shearing resistance (ϕ) = 30°

Test No.	**Applied load (Kn) Q**	**Cap thickness (m)**		**Pile tip resistance (Kn)**				**Cap Deflection (mm)**		
								Point		
				Pile-1	**Pile-2**	**Pile-2**	**Pile-4**	**1**	**2**	**3**
1x2 Pile Group										
34	3	.1d	0.004	1.257	1.159			5.676	5.696	5.597
35	3	1d	0.035	1.059	0.999			5.547	5.529	5.482
36	3	1.5d	0.053	1.055	0.994			5.554	5.536	5.490
37	3	2.5d	0.088	1.051	0.991			5.544	5.526	5.481
38	3	3.5d	0.123	1.051	0.991			5.547	5.529	5.484
1x3 Pile Group										
39	6	.1d	0.004	1.742	2.179	1.669		8.109	8.298	8.011
40	6	1d	0.035	1.290	1.821	1.324		7.741	7.723	7.641
41	6	1.5d	0.053	1.276	1.801	1.312		7.730	7.709	7.631
42	6	2.5d	0.088	1.270	1.793	1.306		7.721	7.694	7.612
43	6	3.5d	0.123	1.269	1.790	1.305		7.718	7.689	7.605
1x4 Pile Group										
44	8	.1d	0.004	1.703	1.943	1.739	1.603	7.799	8.335	7.836
45	8	1d	0.035	1.205	1.865	1.807	1.160	7.678	7.693	7.641
46	8	1.5d	0.053	1.192	1.845	1.789	1.146	7.668	7.678	7.635
47	8	2.5d	0.088	1.187	1.835	1.781	1.141	7.663	7.671	7.632
48	8	3.5d	0.123	1.186	1.834	1.779	1.140	7.663	7.671	7.631

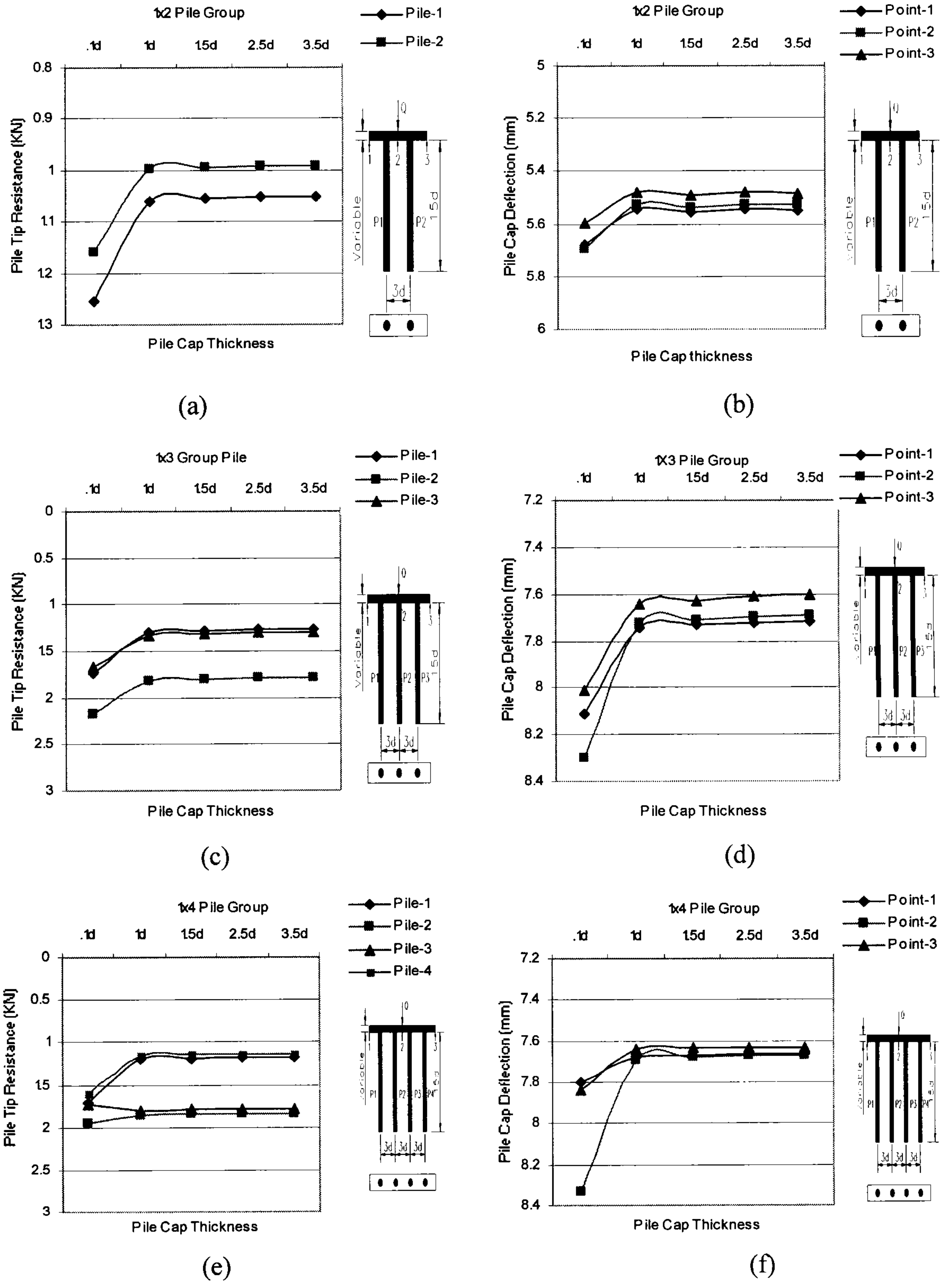


Figure 3.14: Tip resistance and deflection of 1x2, 1x3 & 1x4 group piles for variable cap thickness

Due to increasing the cap thickness, the variation of capacity and deflection of the pile cap for 1x2, 1x3 and 1x4 pile groups are shown in Figures 3.14 (a) to (f). From these Figures the following observations are made:

a) The distribution of column load over the individual piles in the group is not uniform; the piles closer to the center have taken higher load than that of the outer pile.

b) Figures 3.14 (a) and (b) show that tip resistance and three point deflection of pile cap was very high when the cap thickness was 0.1d. An increase in the cap thickness decreases the deflection of cap and tip resistance. It also exhibited that after pile cap thickness 2.5d there was no change of tip resistance and deflection. Pile cap didn't give any significant resistance incase of small thickness, maximum percentage of the applied load directly transferred to the individual pile and it gave higher tip resistance and cap deflection.

c) Figure 3.14 (c) and (d) show that tip resistance and three-point deflection of pile cap was very high when the cap thickness was 0.1d. An increase of the cap thickness decreases the deflection of the cap and tip resistance. It was also noted that point load gave much impact on the pile, which was just below the point load. It exhibited that after thickness 2.5d there was no change of tip resistance of the individual pile and deflection of the pile cap.

d) Figures 3.14 (e) and (f) show that tip resistance and three-point deflection of pile cap was very high when the cap thickness was .1d. An increase of cap thickness decreases the deflection of cap and tip resistance. It was also clearly exhibited that after thickness 2.5d there was a little change of resistance and deflection.

e) Rigidity of a pile group mostly depends on the thickness of pile cap. It can be noted that pile cap can be consider rigid if the cap thickness is equal or higher than vary 2.5 the diameter of the pile.

3.5.4 Group Spacing

Spacing between piles in a pile group is one of the key factors that affect the performance of the individual group pile. Group interaction between the piles increases due to a decrease of pile spacing, up to a limit, beyond which piles will breakdown or deflected. In the present study various spacing of pile groups as 2d, 3d, 4d and 5d were examined. In this study, the angle of shearing resistance, the pile length, cap thickness was fixed at 30^{o}, 15d and 2.5d respectively. Applied axial load was 3, 6 and 8 KN for 1x2, 1x3 and 1x4 pile groups respectively. Table 3.11 presents the results of tip resistance of the piles in the group and the deflection of the pile cap due to the change of spacing-diameter ratio.

Table 3.11: Detail test results for variable s/d ratio

Length of pile (L) = 0.525m

Diameter of the pile (d) = 0.035m

Spacing-diameter ratio (s/d) = Varies from 2 to5

Angle of shearing resistance (ϕ) = 30°

Test No.	**Applied load (Kn) Q**	**Center to center Spacing (s/d)**		**Pile tip resistance (Kn)**				**Cap Deflection (mm)**		
								Point		
				Pile-1	**Pile-2**	**Pile-2**	**Pile-4**	**1**	**2**	**3**
1x2 Pile Group										
49	3	2	0.070	0.893	0.873			9.338	9.317	9.270
50	3	3	0.105	1.051	0.991			5.544	5.526	5.481
51	3	4	0.140	0.964	0.958			3.164	3.162	3.136
52	3	5	0.175	0.933	0.910			2.046	2.028	1.985
1x3 Pile Group										
53	6	2	0.070	0.855	1.121	0.826		9.782	9.809	9.784
54	6	3	0.105	1.270	1.793	1.306		7.721	7.694	7.612
55	6	4	0.140	1.137	1.637	1.195		4.152	4.127	4.044
56	6	5	0.175	1.047	1.527	1.144		2.624	2.632	2.580
1x4 Pile Group										
57	8	2	0.070	1.703	1.943	1.739	1.603	10.019	10.015	9.963
58	8	3	0.105	1.205	1.865	1.807	1.160	7.663	7.671	7.632
59	8	4	0.140	1.192	1.845	1.789	1.146	4.041	4.044	4.001
60	8	5	0.175	1.187	1.835	1.781	1.141	2.510	2.511	2.467

The results of this series are presented in Figures 3.15 (a) to (f) respectively. It can be noted from these Figures that:

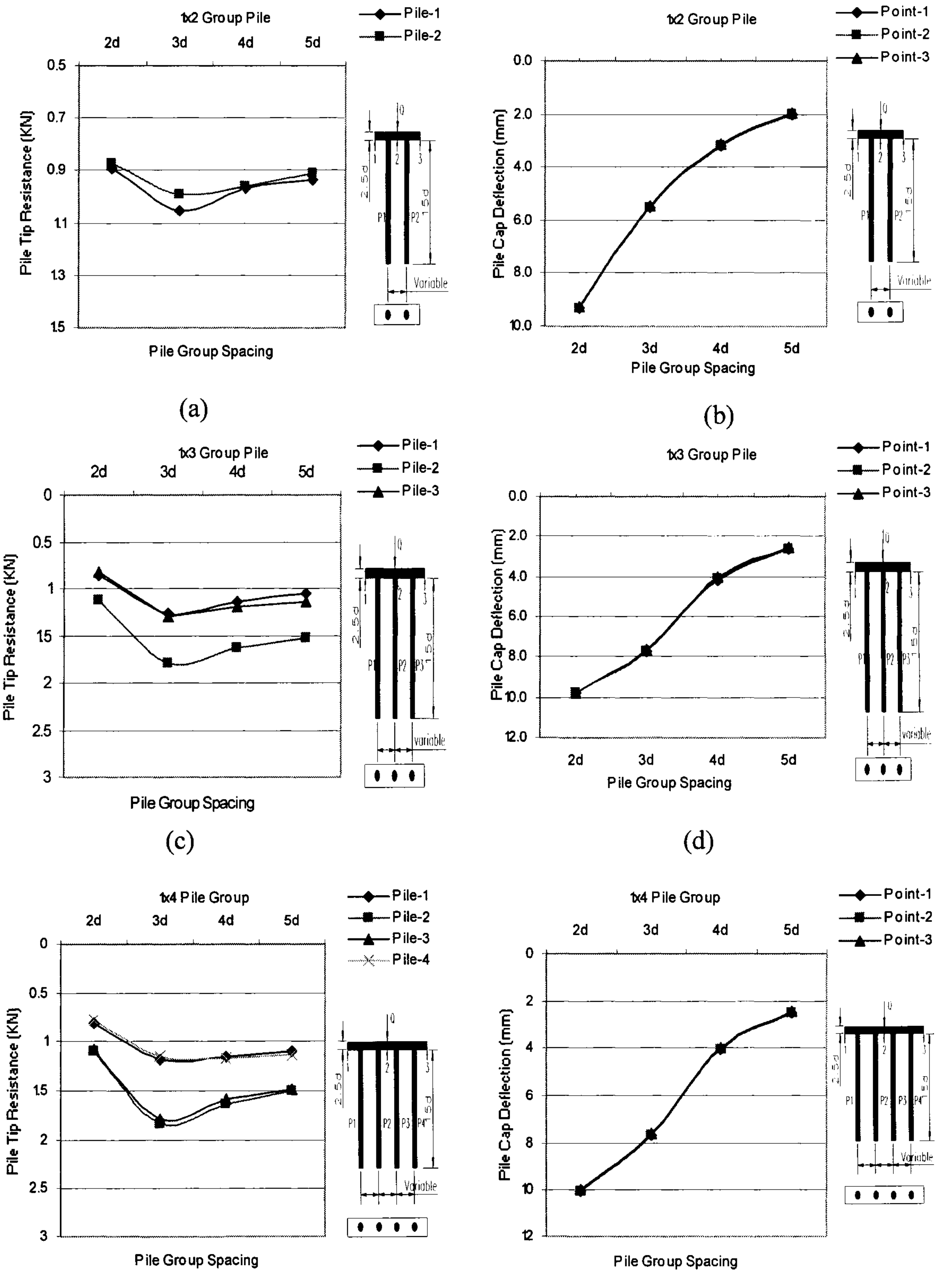
a) The load distribution on the individual piles is not uniform; pile closer to the applied load has taken higher percentage of load than outer piles.

b) Pile cap was considered as rigid; nevertheless, all caps were subjected to deflection.

c) Tip resistance increases due to the increase of pile spacing. Pile group gave maximum tip resistance when the spacing-diameter ratio was 3. In case of three-point cap deflection decreases with the increasing of pile spacing. Examining these relationships it can be noted that deflection decreases linearly with the increasing of pile spacing, up to a minimum critical value of 3d.

d) Figures 3.15 (c) and (d) show the variation of tip resistance and the three-point deflection on the pile cap. It can be noted that the maximum tip resistance is at 3d spacing. Similar behavior was observed for the cap of 1x2.

e) Figures 3.15 (e) and (f) show that tip resistance and three-point deflection of pile cap. It can be noted that for spacing 3d the pile group provided the maximum tip resistance. Deflection pattern due the change of spacing was same as for pile group of 1x3.



(a) (b)

(c) (d)

(e) (f)

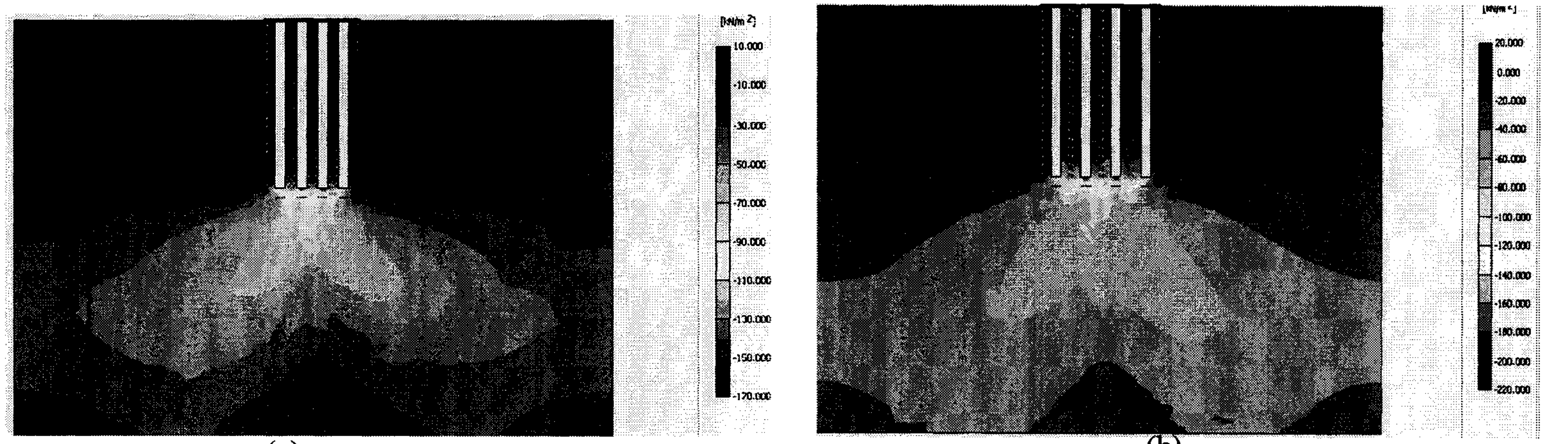
Figure 3.15: Tip resistance and deflection of 1x2, 1x3 & 1x4 group piles for variable spacing

3.5.4.1 Effective Horizontal and Vertical stress

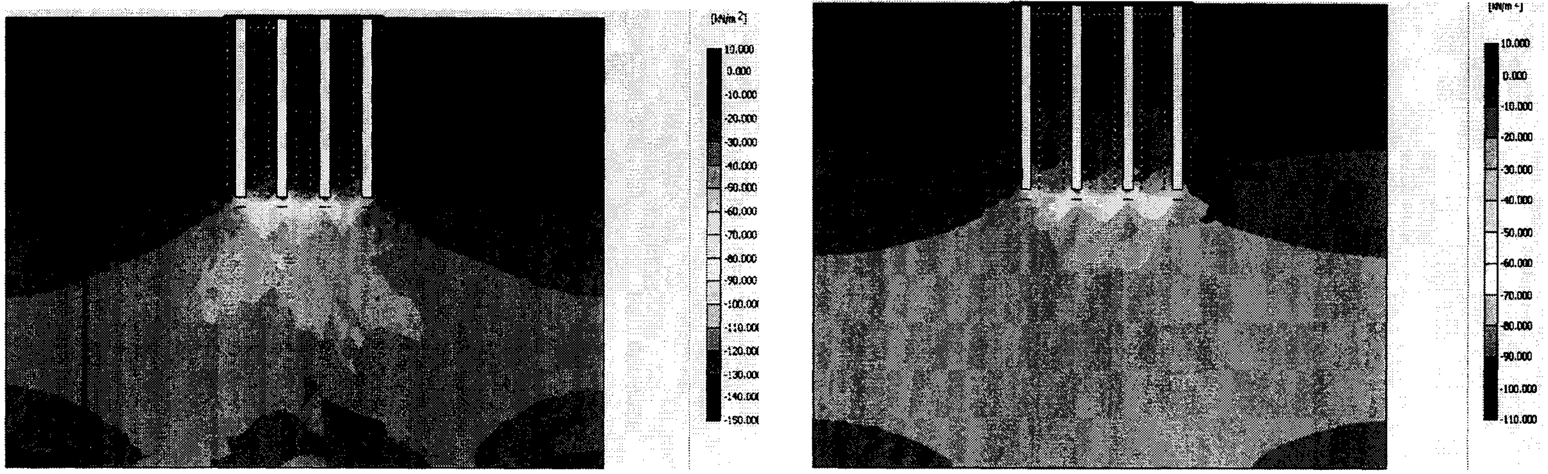
Horizontal and vertical stresses at the tip of the pile group depend upon the type of loading, spacing-diameter ratio, length-diameter ratio and the soil condition. In the present study spacing-diameter ratio was taken as 2, 3, 4 and 5 for a 1x4 line pile group. The angle of shearing resistance, length-diameter ratio, and cap thickness was 30^o, 15 and 2.5d respectively. Figures 3.16 (a) to (d) and Figures 3.17 (a) to (d) show the variation of effective horizontal and vertical stresses due to an increase of the spacing-diameter ratio from 2 to 5. It can be noted that:

a) Figure 3.16 (a) shows that the overlapped effective horizontal stress at the tip of the middle pile was higher as compare to the other piles in the group. This stress reduces due to the increase of pile spacing, see Figures 3.16 (b), (c) and (d).

b) Vertical stresses increases due to the decrease of s/d ratio. Nevertheless the stresses extend vertically due to the increase of s/d ratio, see Figure 3.17. For s/d > 3 there is no overlap of vertical effective stresses as shown in figure 3.17 (c) and (d), where individual piles in the group act as single piles and not as group. Again vertical stresses along the shaft of the individual pile are also increase with the increase of pile spacing.

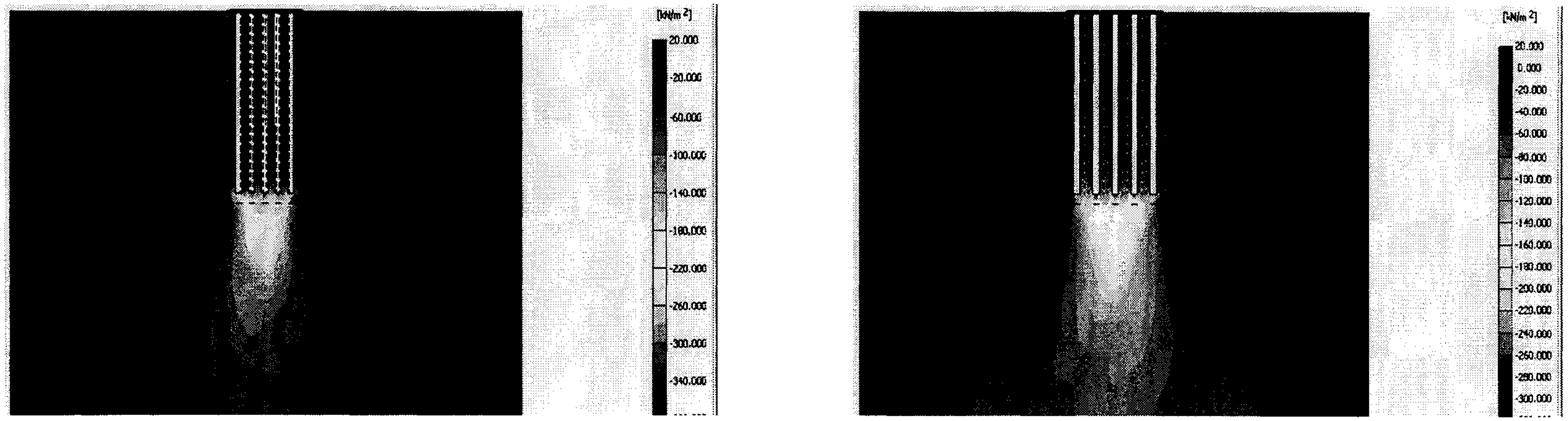


(a) S/d=2 (b) S/d=3



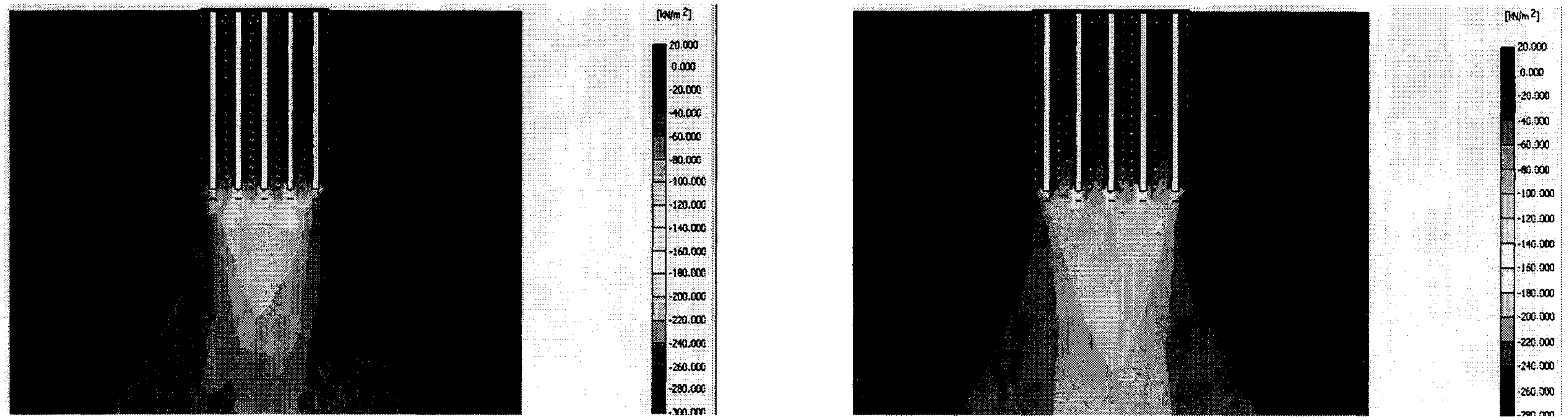
(c) S/d=4 (d) S/d=5

Figure 3.16: Effective horizontal stress distribution due to variation of spacing for 1x4 pile group



(a) S/d=2

(b) S/d=3



(c) S/d=4

(d) S/d=5

Figure 3.17: Effective vertical stress distribution due to variation of spacing for 1x4 pile group

3.5.4.2 Horizontal and Vertical displacement

Displacement of soil around the pile tip provides valuable information about the interaction of the piles in the group. 1x4 pile group was examined to analyze the displacement due the increase of s/d from 2 to 5. The angle of shearing resistance, L/d and the cap thickness was 30°, 15 and 2.5d. Figures 3.18 (a) and (b) show that for spacing/diameter ratio = 3, the horizontal and the vertical displacement at the tip of the 1x4 pile reached the maximum. This is due to the fact that at s/d = 3 the group pile provide higher bearing capacity due to the optimum overlapping of the pressure bulb at the tip of the pile group.

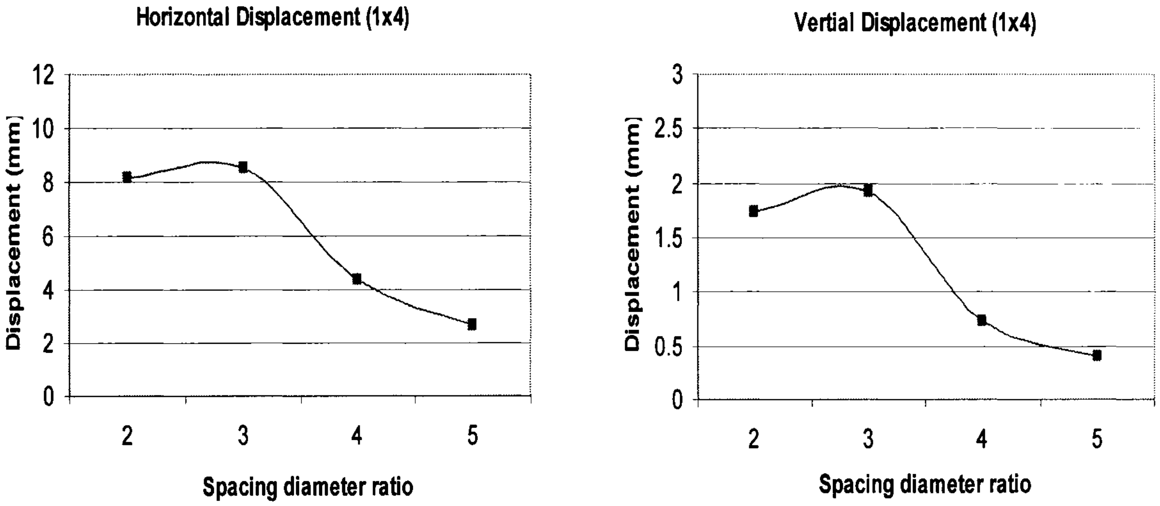


Figure 3.18: Effective horizontal and vertical displacement due to variation of spacing for 1x4 pile group

3.5.5 Shaft resistance of pile group

Length/diameter ratio and spacing/diameter and soil condition mostly control the group pile-soil interaction of a pile group. Figure 3.19, 3.20 and 3.21 show the change of shaft resistance change of L/d, s/d and angle of shearing resistance for 1x2, 1x3 and 1x4 line pile group. From these Figures the following observations are made:

a) From Figure 3.19, 3.20 & 3.21 it can be noted that, for any type of group arrangements, shaft resistance increases with the increase of s/d ratio.

b) The changes of the shaft resistance are relatively small for s/d ratio greater than 4. This is because when s/d>3 the interaction between the piles in the group is significantly reduced and thus the piles in the group will act as single piles.

c) Shaft resistance increases with the increase of L/d from 24 to 50 as shown in Figures 3.19, 3.20 and 3.21. Shaft resistance at s/d=3 gave the same results although L/d ratio was increasing. This is because at this spacing the pile soil intersection is optimum.

d) Figure 3.19, 3.20 and 3.21 show the shaft resistance in the line group pile increases with the angle of shearing resistance.

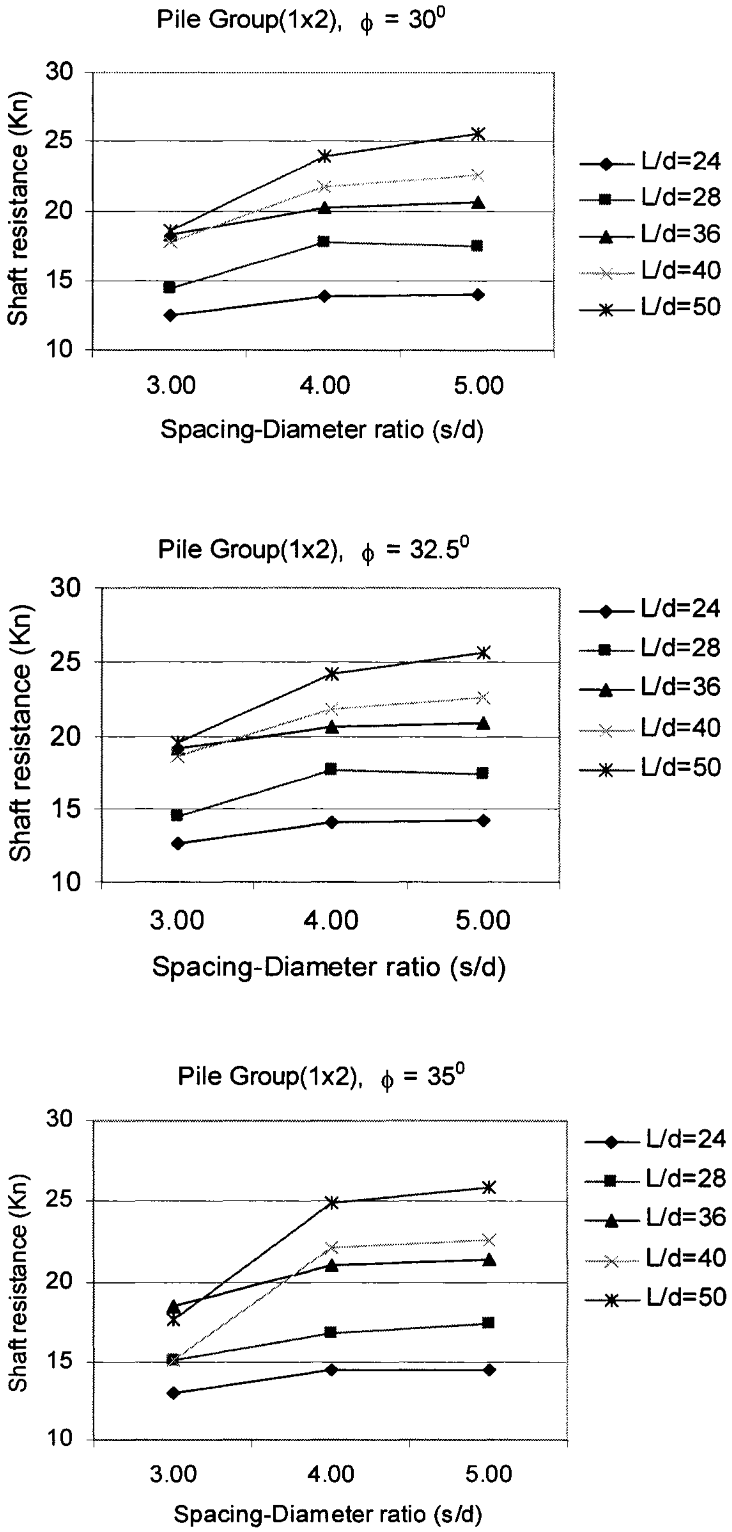


Figure 3.19: Variation of shaft resistance for 1x2 pile group

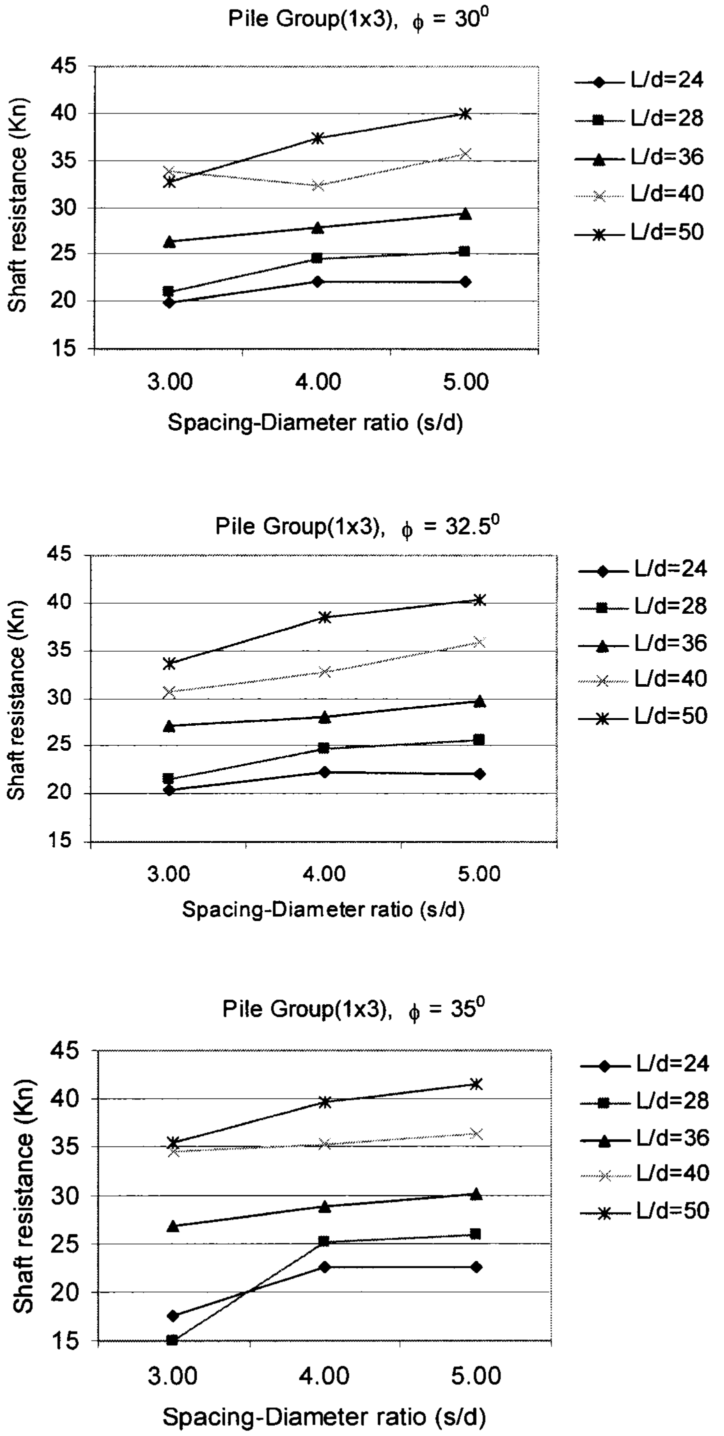


Figure 3.20: Variation of shaft resistance for 1x3 pile group

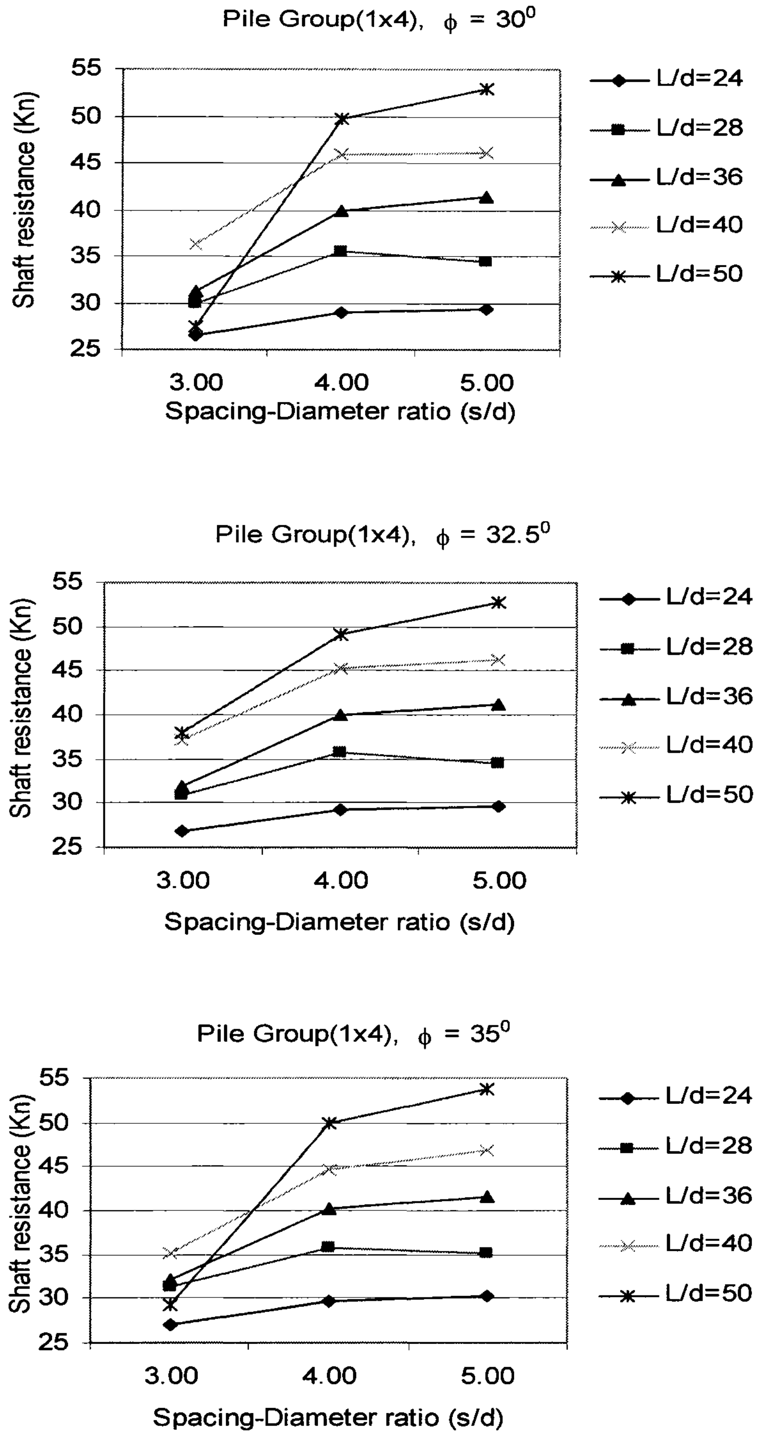


Figure 3.21: Variation of shaft resistance for 1x4 pile group

3.5.6 Group Efficiency

Efficiency of the pile group is an important factor to evaluate the effectiveness of the pile group. In the present investigation the group efficiency for 1x2, 1x3 and 1x4 pile groups was analyzed by changing the length/diameter ratio from 24 to 50, varying the spacing from 3d to 5d and the angle of shearing resistance from 30° to 35°.

3.5.6.1 Group Efficiency for variable length/diameter ratio

Length/diameter ratio has great impact on the efficiency of the group. The increase of the length/diameter ratio will increase the load capacity of the pile group as well as the individual piles in the group. Figure 3.22, 3.23 and 3.24 shows the variation of the group efficiency with the length/diameter ratio for 1x2, 1x3 and 1x4 pile groups. It can be noted from these Figures that the efficiency varies from 0.8 to 1.1. This is due to the fact that the efficiency was calculated from the individual tip and shaft resistance of pile. Furthermore:

a) The group efficiency of group decreases with the increase of L/d ratio.

b) The group efficiency increases due to an increase of the angle of shearing resistance.

c) Variation of L/d and the angle of shearing resistance with pile spacing greater than 4 exhibited almost the same group efficiency (0.90 to 0.95).

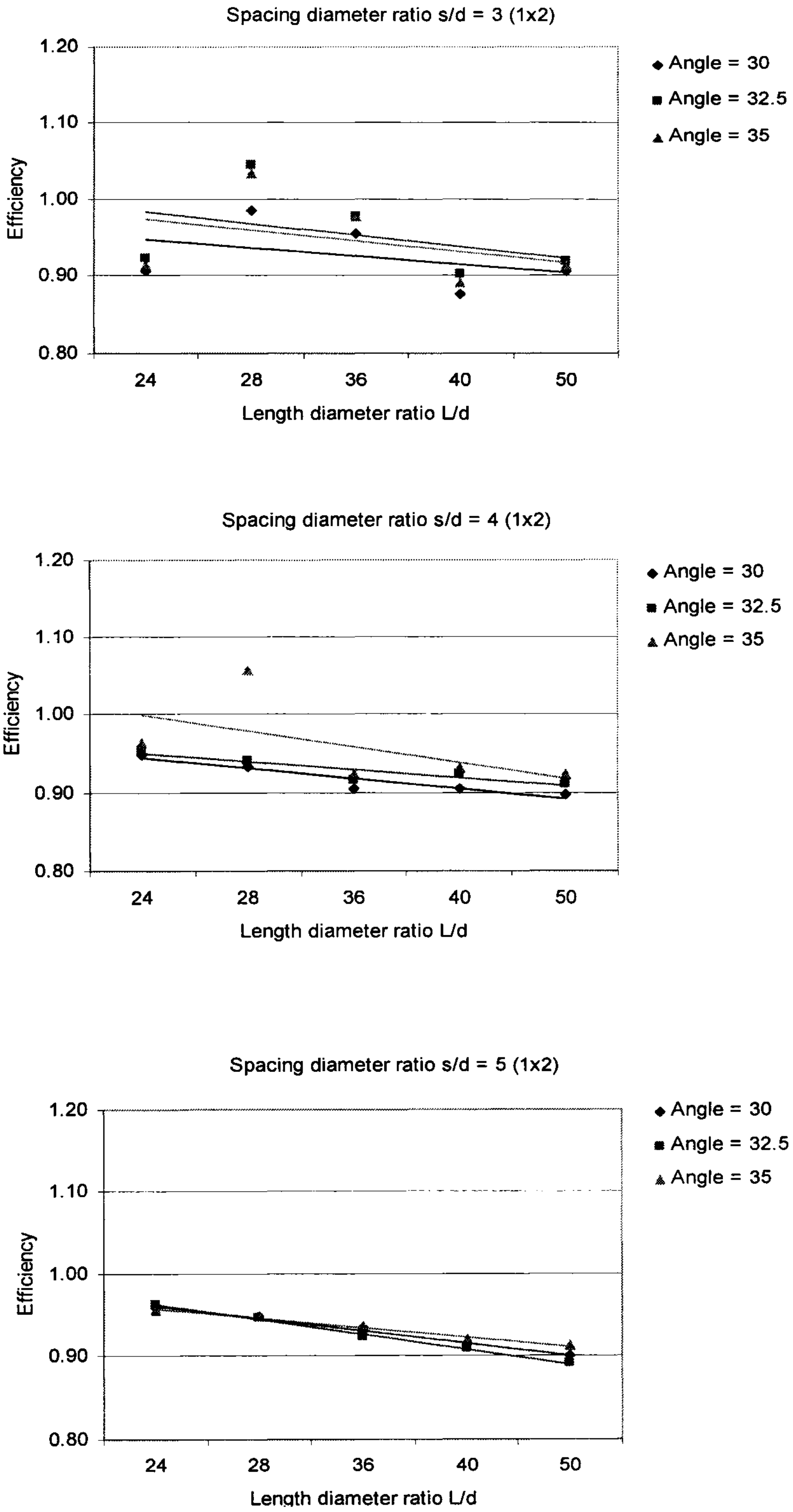


Figure 3.22: Efficiency for 1x2 pile group

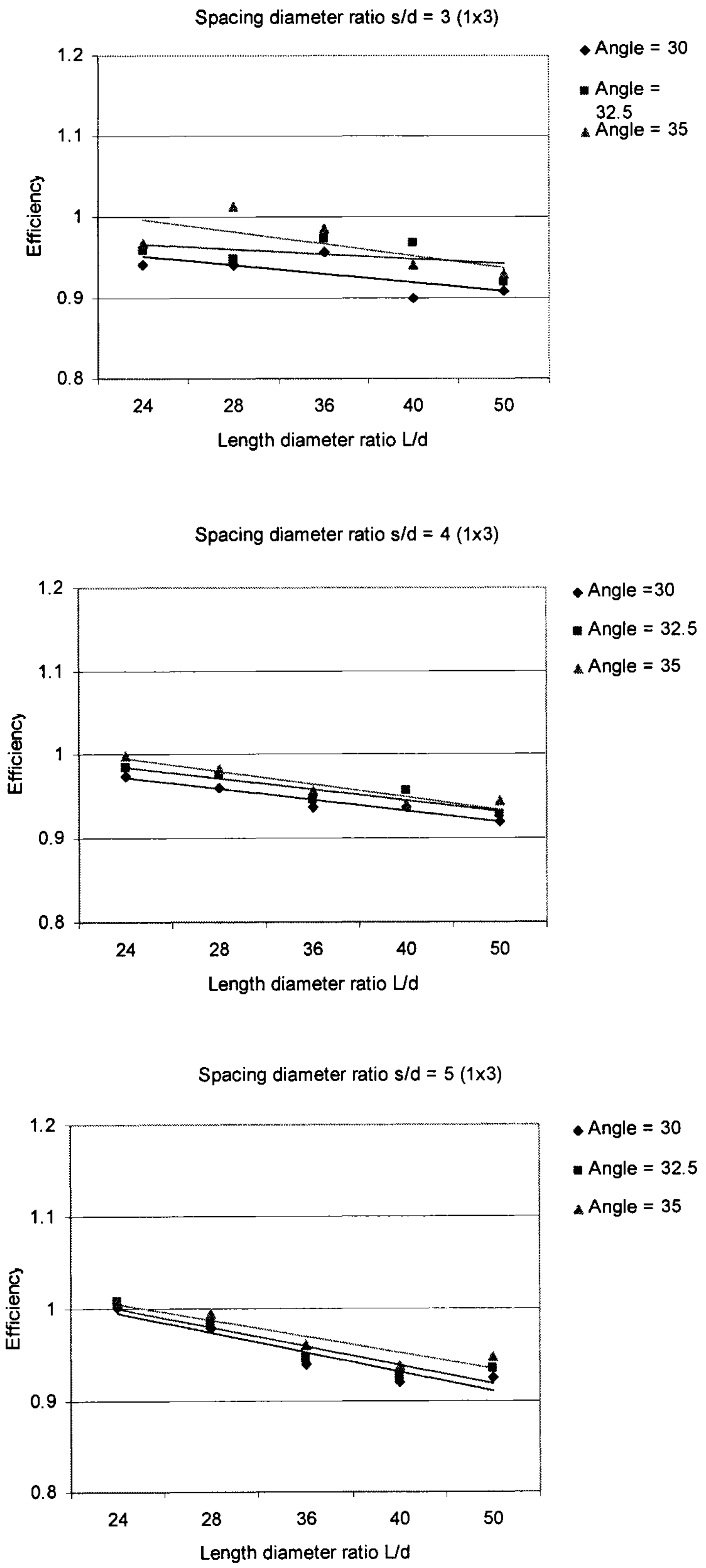


Figure 3.23: Efficiency for 1x3 pile group

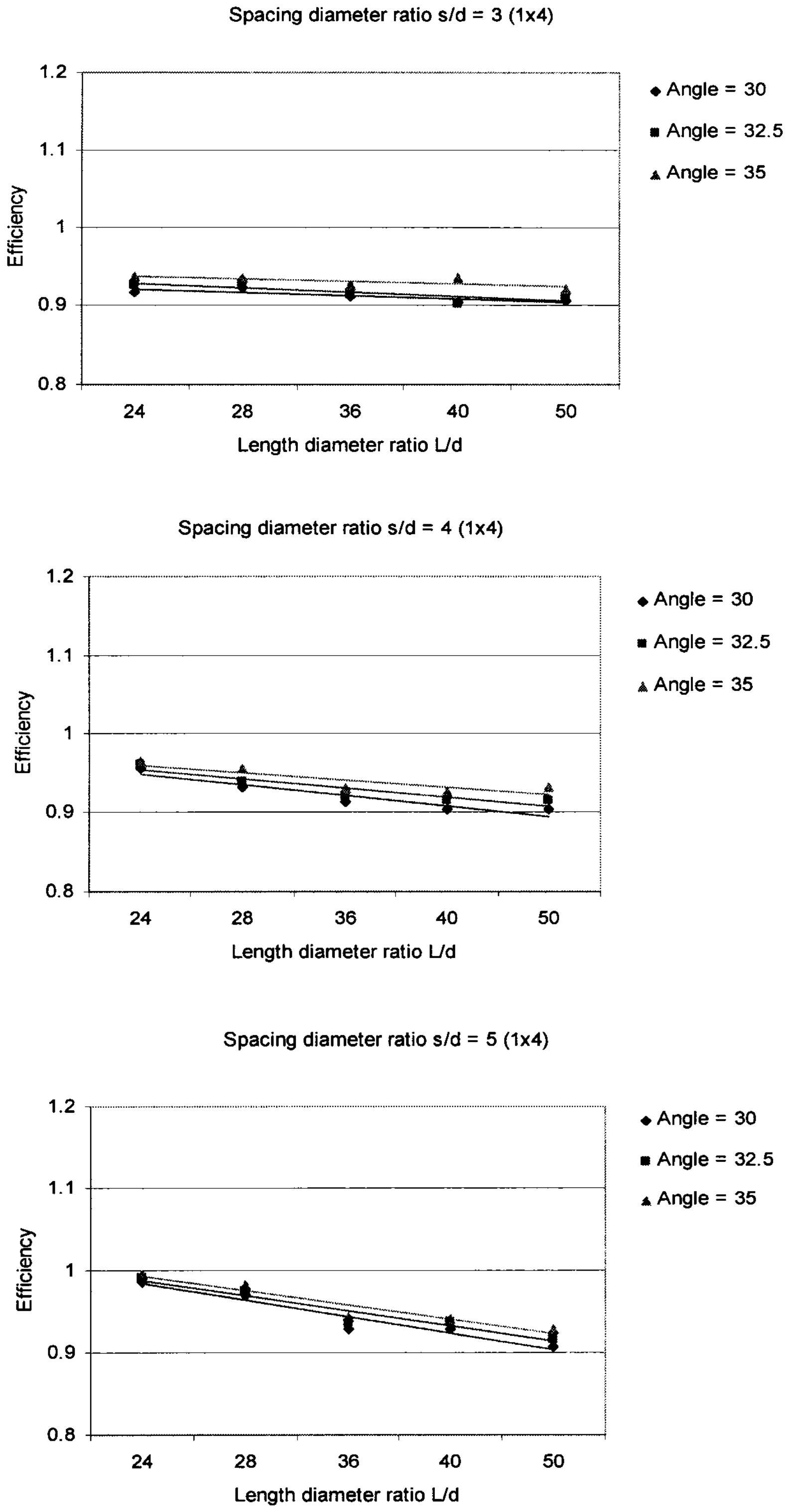


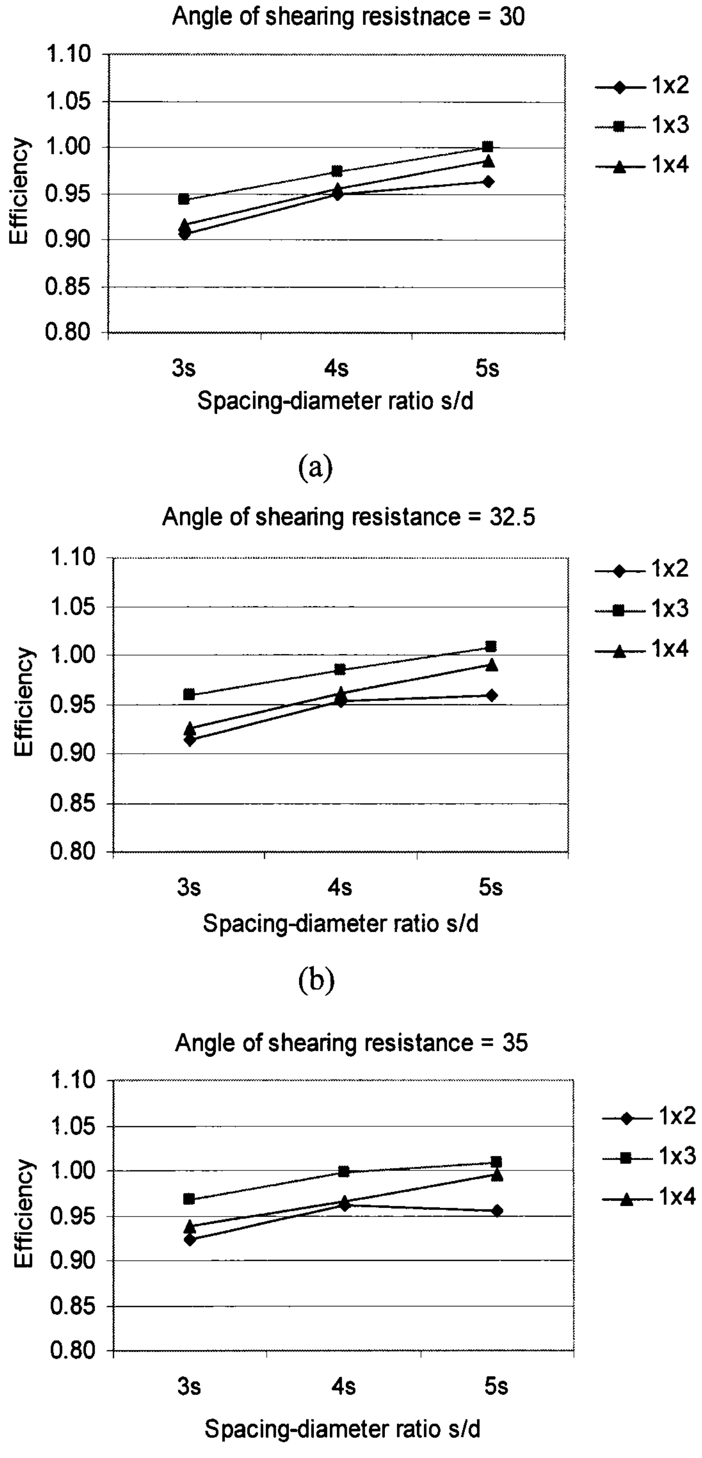
Figure 3.24: Efficiency for 1x4 pile group versus pile length/diameter ratios

3.5.6.2 Group Efficiency for variable spacing-diameter ratio

Spacing between the piles in a group has a great impact on the capacity of the individual piles. In section 3.4.4 the effect of spacing over various groups of piles was examined. In this section the spacing/diameter ratio varies from 3, 4 and 5 for line pile groups are examined. The angle of shearing resistance was assigned these values; 30°, 32.5° and 35°. Figures 3.25 (a), (b) and (c) shows the variation of the group efficiency with the variation of pile spacing in the group. From these Figures, it can be noted that:

a) From the Figures 3.25 (a) and (b) it was clearly observed that for angle of shearing resistance 30° and 32.5° the group efficiency for all group arrangements will increase with the increase of the spacing ($s/d \geq 3$). It also observed that due to the variation of s/d ratio from 3 to 5 the efficiency was varied from 0.9 to 1.1.

b) Figure 3.25 (c) shows that for dense sand with an angle of shearing resistance 35°, the variation of group efficiency for spacing diameter ratio $s/d \leq 4$ is less, it gave almost similar result.



(a)

(b)

(c)

Figure 3.25: Efficiency of pile group versus spacing/diameter ratios

3.5.7 Load distribution along the pile group

In all the existing design theories, it was assumed and taken for granted that individual pile in a group will carry the same load. In the present investigation, the analysis was conducted over 1x2, 1x3, 1x4, 1x5 and 1x6 line pile groups for variable spacing, angle of shearing resistance, length diameter ratio and fixed pile cap thickness. Figure3.26 shows the average distribution of point load over above mentioned pile groups for s/d=3, L/d = 24 to 50 and for $\phi = 30^{\circ}$ to 35°. Figure 3.28 presents the variation of the load distribution of 1x2, 1x3 and 1x4 piles groups for spacing diameter ratio 4 and 5. From these Figures, it can be noted that:

a) Pile in the central portion of the group carries higher percentage of load than the outer piles.

b) For 1x2 pile group, both the piles have taken almost the same percentage of load.

c) For 1x3 pile group, pile no. 2 took 36% of the total load and pile 1 and 3 have taken 32% each.

d) For 1x4 pile groups, the internal two piles i.e. pile no. 2 and 3 took 27% each of total load and outer two piles 1 and 4 have taken 24% and 22% of total load respectively.

e) For 1x5 pile group, pile no. 3 took 23.4% of the total load whereas piles no. 2 and 4 have taken 22% each. In addition, pile no. 1 and 5 took 16.3% each.

f) For 1x6 pile group, pile no. 3 and 4 took 19% each of the total bearing capacity. Here pile no. 2 and 5 took 18% each of the total loads. Furthermore, piles no. 1 and 6 have taken 13% each of the total bearing capacity of the pile group.

g) From Figure 3.27, it can be noted that the variation of load distribution due the change of spacing for 4d and 5d did not make much difference in the individual capacity of the pile in the group.

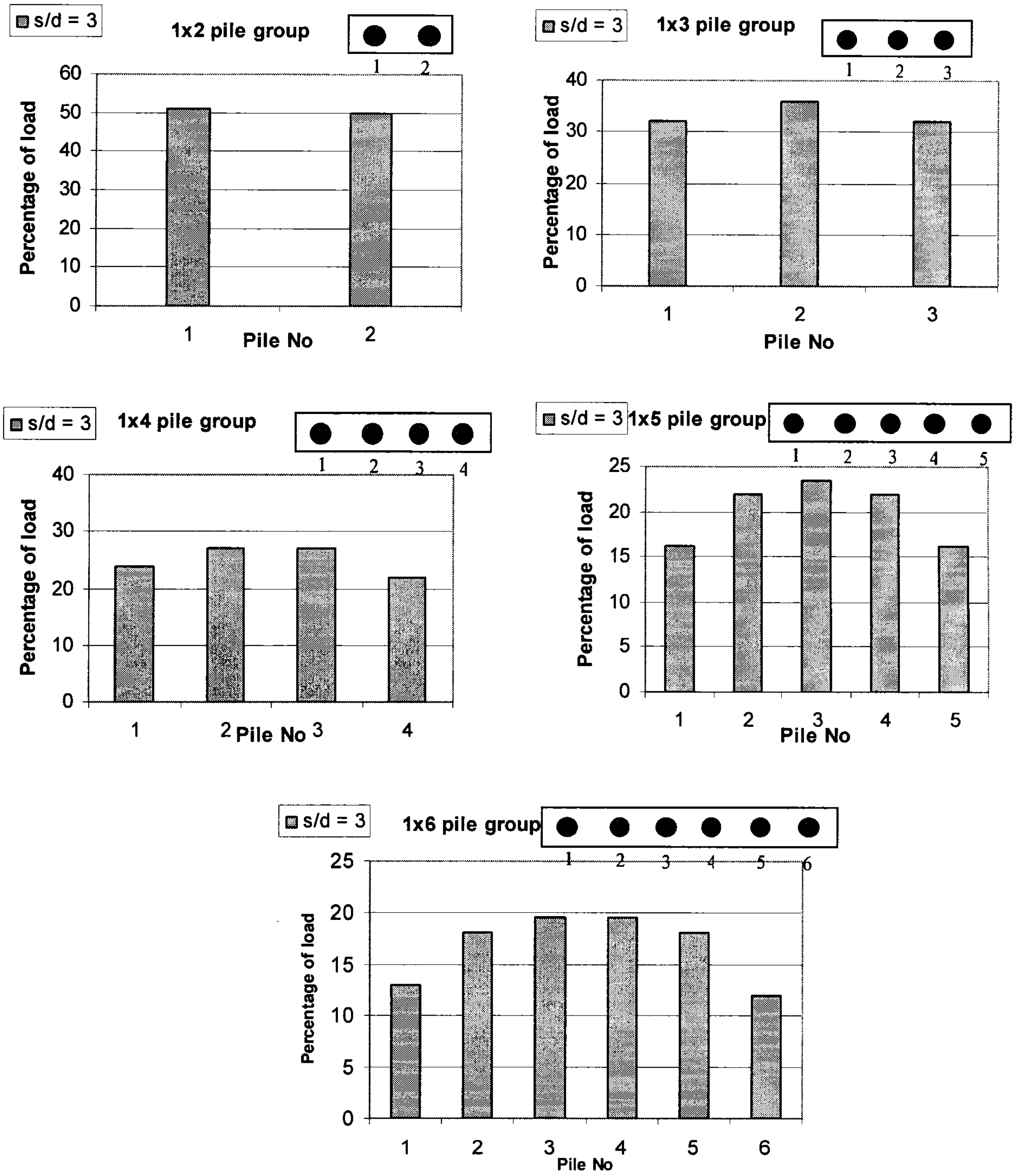


Figure 3.26: Load distribution pattern in line pile group for s/d = 3

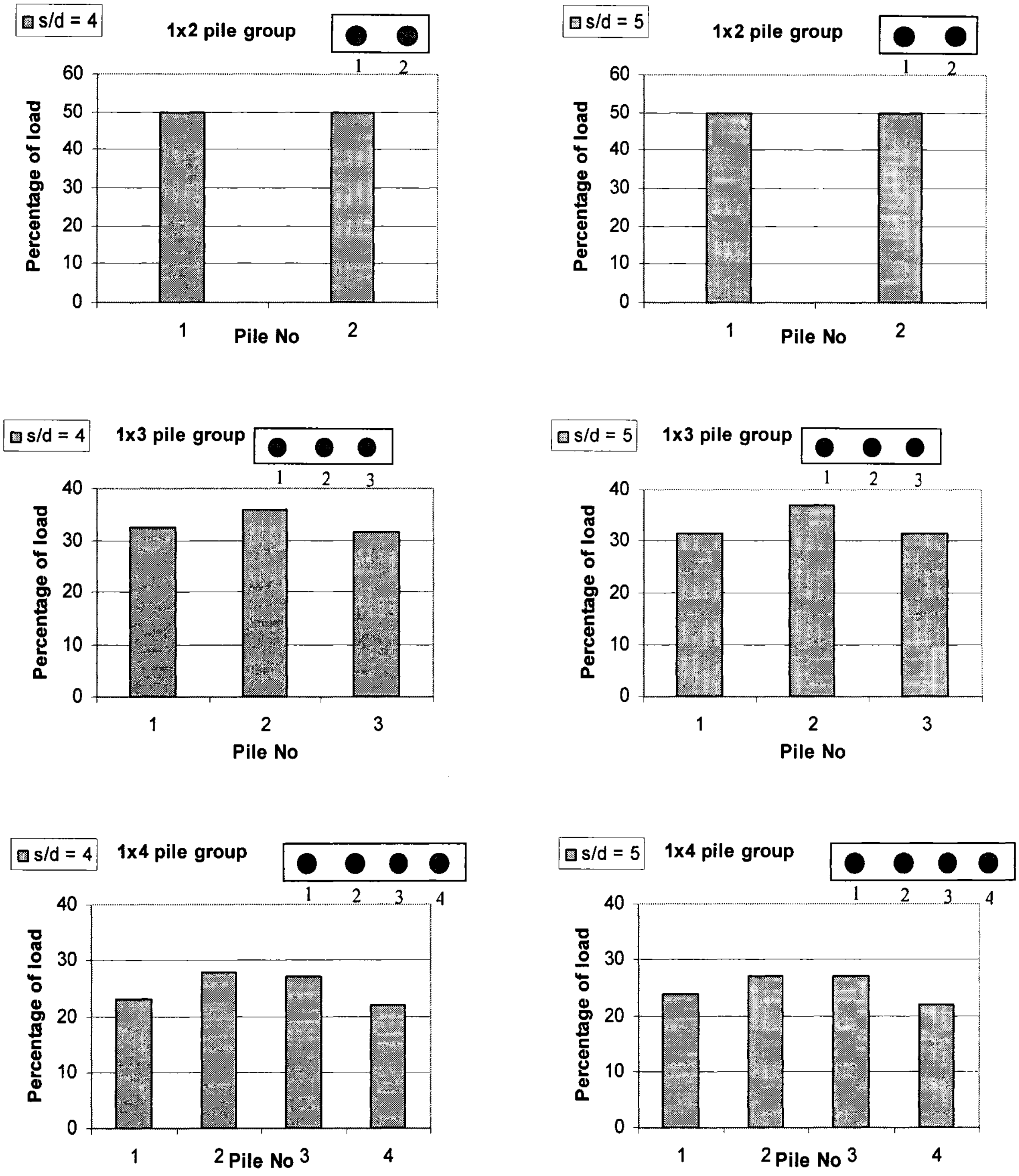


Figure 3.27: Load distribution pattern in line pile group for s/d = 4 and 5

3.6 Proposed Design Consideration

In the design of pile group, and based on the result of the present investigation the following recommendations are made:

a) Spacing-diameter ratio should be in the range of 2.5 to 3.5 for ϕ = 30 to 35 and L/d = 24 to 50 and for fixed cap thickness 2.5d.

b) The thickness of the pile cap should be more than 2.5 times of the diameter or width of the single pile.

c) Sequence of construction should be side-by-side.

d) Distribution of load on the individual piles should be considered in the design of pile group. Figure 3.28 presents the percentage of load distribution in the 1x2, 1x3, 1x4, 1x5 and 1x6 pile groups.

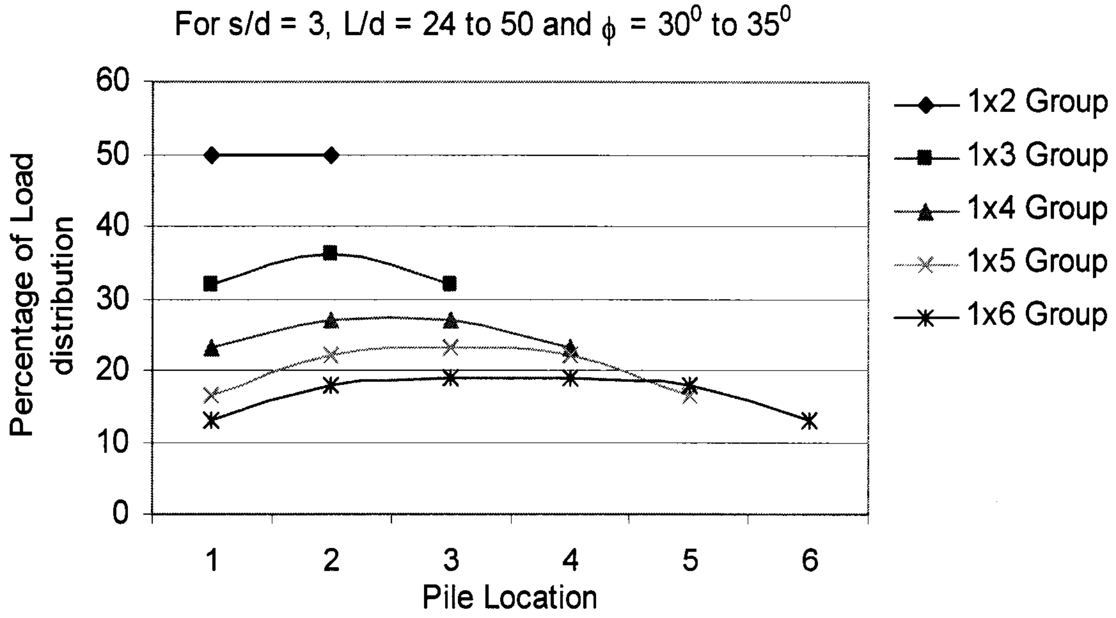


Figure 3.28: Proposed load distribution chart

CHAPTER 4

ANALYTICAL MODEL

4.1 General

Full-scale field tests provide more reliable results than laboratory model tests. However, full scale tests carry the disadvantage of being limited to the soil conditions of the specified test location in addition to the fact that the cost of these tests is very high and the chances of conducting parametric study using these tests is very limited. On the basis of the present numerical model results, a new formula was proposed for the group efficiency of line bored pile group in sand. The proposed formula was based on the work of Sayed and Bakeer 1992 for driven and jacked piles. The derivation of the formula is based on the concept that the group effect can be taken into consideration for the load component taken by shaft, tip and pile cap.

In this chapter comparison of the existing design theories with the proposed formula for pile group efficiency is also presented.

4.2 Mathematical Model

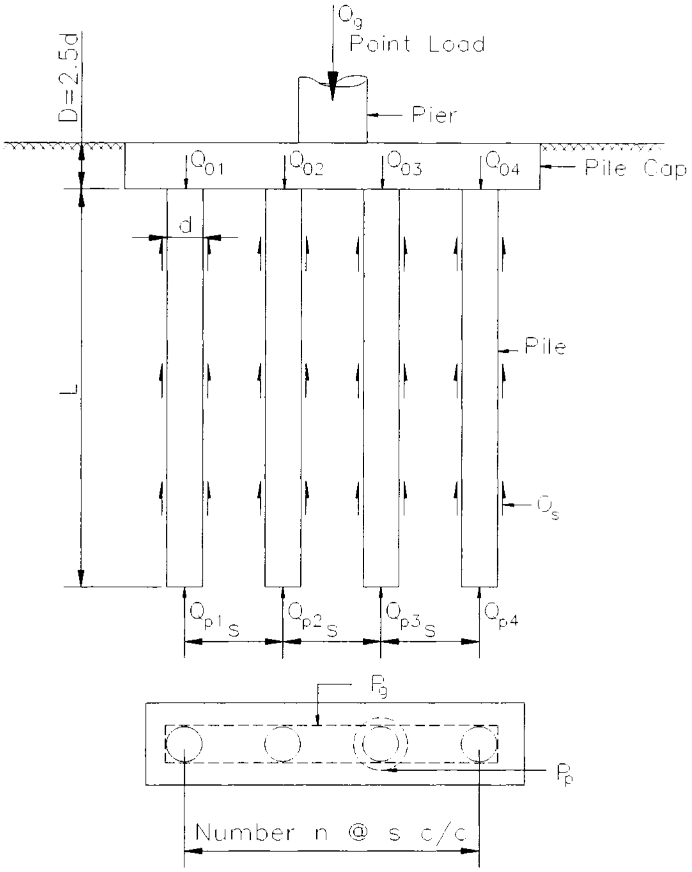
The pile group shown in Fig. 4.1 consists of 1 row of *n* piles and is subjected to a vertical and centric loading, the efficiency of the pile group is defined as ratio of group capacity and the sum of the load capacity of the individual pile in the group, as follows:

$$\eta_g = \frac{Q_g}{\sum Q_{0n}} \qquad 4.1$$

Where Q_g = The group load capacity

Q_{0n} = The single pile capacity

η_g = Pile group efficiency



d = Pile diameter

s = Center to center spacing

P_g = Perimeter of the pile group

P_p = Perimeter of the single pile

Q_g = Ultimate load of pile group

Q_0 = Ultimate load of single pile

Q_s = Ultimate load of shaft

Q_p = Ultimate tip resistance

n = Number of pile in a row

D = Thickness of the cap

L = Length of the pile

Figure 4.1: Typical pile group arrangements

The applied load on the individual piles is divided into two components: shaft load Q_s and the tip load Q_p

$$Q_0 = Q_p + Q_s \tag{4.2}$$

Considering the geometry of the line group, s/d ratio, L/d ratio and the soil property the proposed group efficiency can be expressed as:

$$\eta = 1 - \left(1 - \eta_g * K_0 * \rho\right) * \sin(\frac{L}{d}) \tag{4.3}$$

Where,

η = Group efficiency

η_g = Geometric efficiency Factor

K_0 = Earth pressure coefficient

ρ = Pile group interaction factor

$\frac{L}{d}$ = Length/diameter ratio

4.3 Parameters of the Model

The parameters introduced in this model will be evaluated individually in this section.

4.3.1 Geometric efficiency factor

This factor takes into consideration the planar geometry, number of piles and the diameter of the pile in the group. For 1 x n pile group arrangement, the geometric efficiency η_g can be expressed as:

$$\eta_g = \frac{p_g}{\sum p_p} \quad 4.4$$

$$p_g = 2\{[(n-1)s + d] + d\} \quad 4.5$$

$$p_p = \pi . d \quad 4.6$$

The notations of the P_g and P_p are given in the Figure 4.1, thus for a line group the geometric efficiency factor is:

$$\eta_g = 2\left\{\frac{[(n-1)s + d] + d}{\pi . n . d}\right\} \quad 4.7$$

For square piles, π is replaced by 4 and d is replaced by b, where b is the width of the pile.

4.3.2 Earth pressure factor

The proposed formula was developed for bored pile, so the earth pressure was considered at-rest condition as follows:

$$K_0 = 1 - \sin(\phi) \tag{4.8}$$

Where,

K_0 = Coefficient of at-rest earth pressure

ϕ = Angle of shearing resistance of soil.

4.3.3 Group interaction factor

Group interaction factor in a pile line group is mostly depends on the center-to-center spacing and length/diameter ratio. Based on the results obtained from the numerical model in the present investigation, the proposed interaction factors for variable s/d ratio are presented in Table 4.1.

Table 4.1 Group interaction factor

Spacing/diameter Ratio (s/d)	Interaction factor (ρ)
3	1
4	0.9
≥ 5	0.8

4.3.4 Analysis of Proposed Mathematical Model

To compare the proposed design formula for pile group efficiency (η), adequate experimental data are required. Due to the unavailability of experimental data for bored line pile group, the numerical test results were compared with the proposed analytical model.

The numerical test results for different pile groups that have pile length/diameter ratio (L/d) 24, 28, 36, 40 and 50, pile spacing/diameter ratio (s/d) 3, 4, and 5, and the pile arrangements 1x2, 1x3 and 1x4 were compared with the proposed mathematical model. These results are listed in Tables A.1, A2 and A3 and plotted in Figures 4.2, 4.3 and 4.4. Comparing with the results, the following observations were made:

a) For any type of line group arrangements the group efficiency (η) calculated by the proposed formula (Eq.4.3) is 0.8 to 1.1. Efficiency increase with the increase of the No. of pile in group

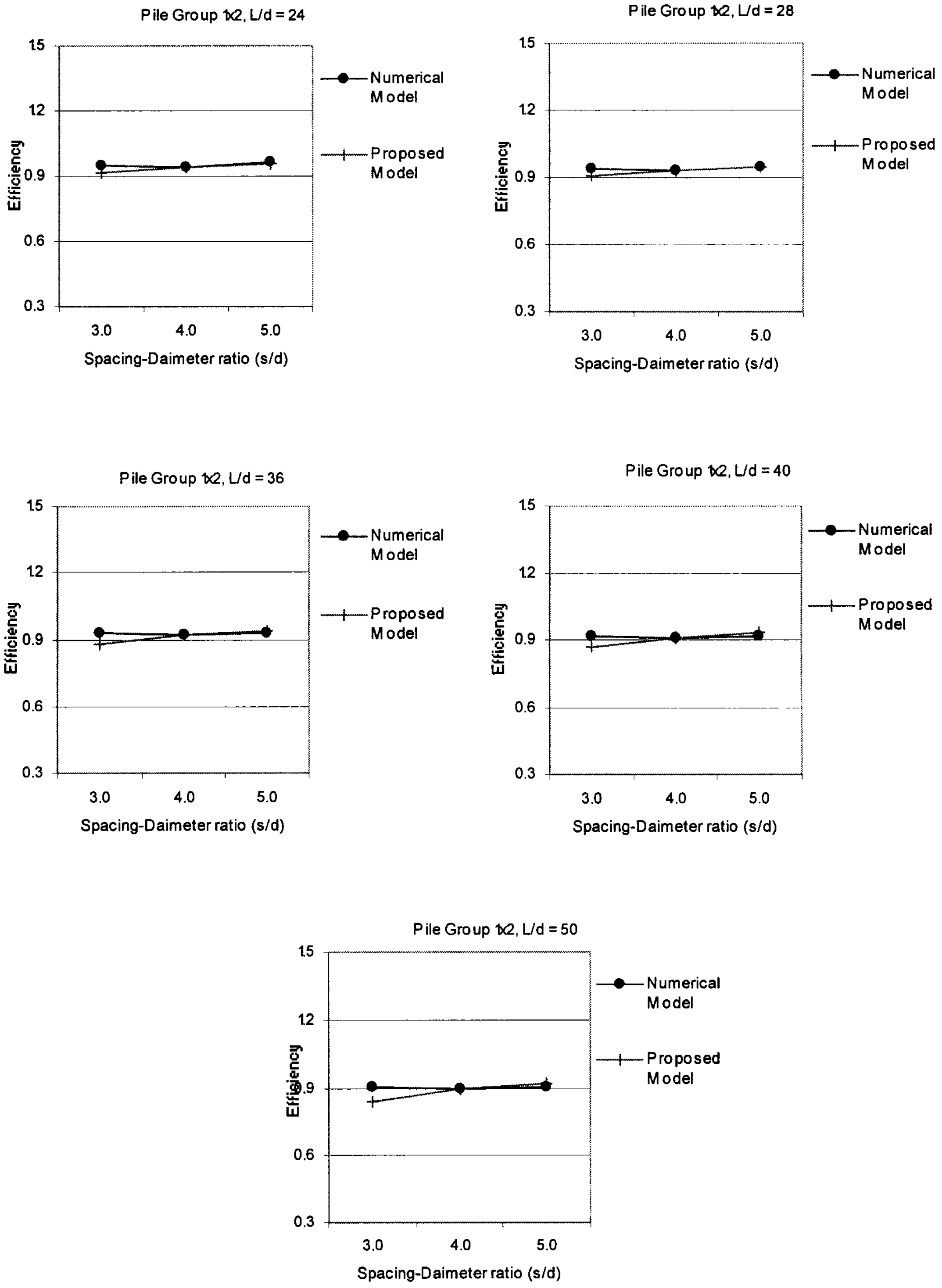
b) The efficiency increases with the increase of s/d ratio, up to a value of s/d = 4, beyond which the efficiency remains unchanged. This is due to the fact that at higher values of s/d, the group action over the piles in-group diminishes.

c) Efficiency decreases with the increase of the length-diameter ratio.

d) In the proposed formula the effect of soil condition, s/d ratio, L/d ratio and pile soil interaction are introduced. This explains the good agreement achieved with the results obtained from the present numerical model, which utilize the same parameters.

e) The proposed formula is only valid for bored line pile group.

f) This formula is applicable for of cohesion-less soil.



Pile Group 1x2, L/d = 24

Pile Group 1x2, L/d = 28

Pile Group 1x2, L/d = 36

Pile Group 1x2, L/d = 40

Pile Group 1x2, L/d = 50

Figure 4.2: Comparison of group efficiency for 1x2 pile group (s/d variable)

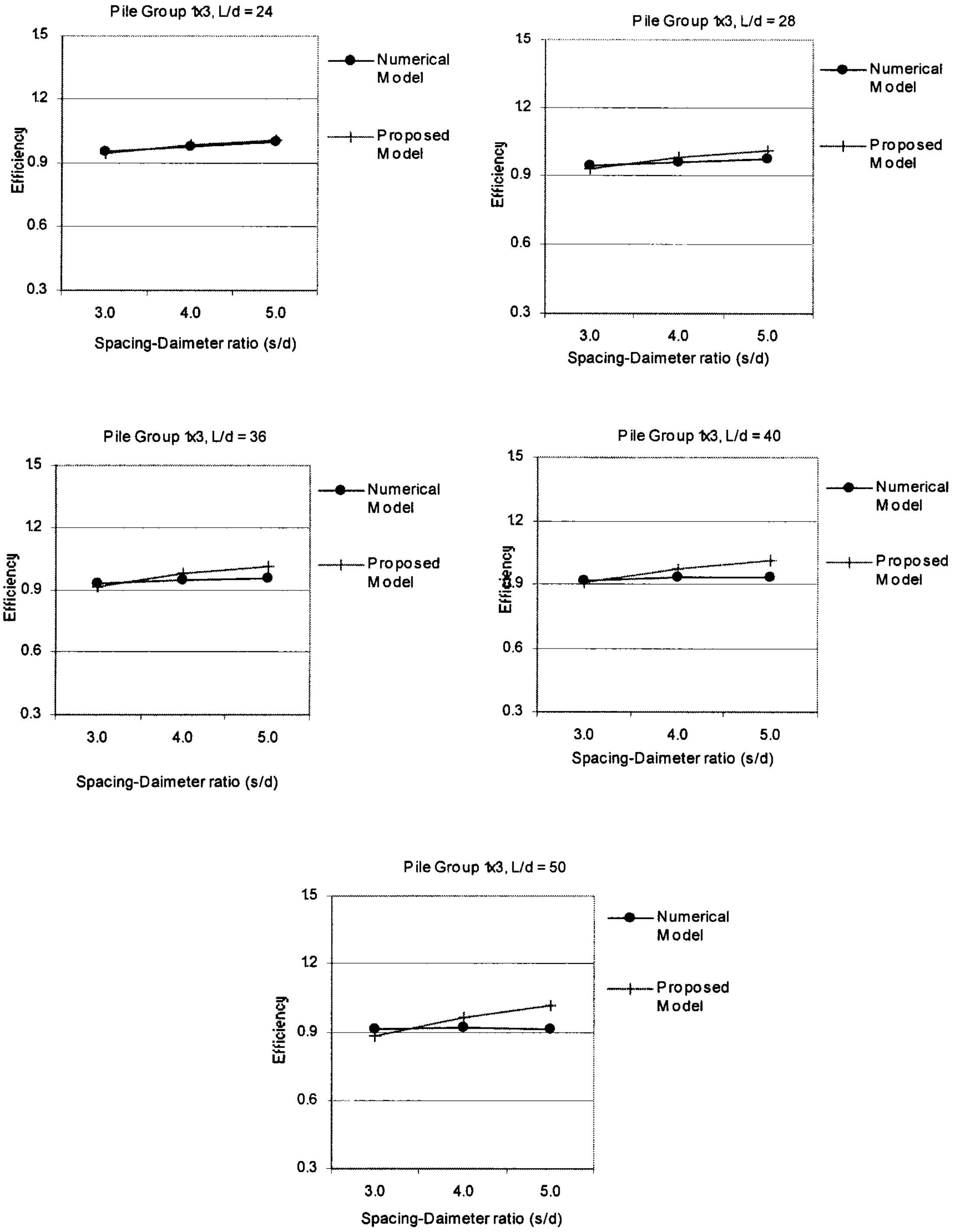


Figure 4.3: Comparison of group efficiency for 1x3 pile group (s/d variable)

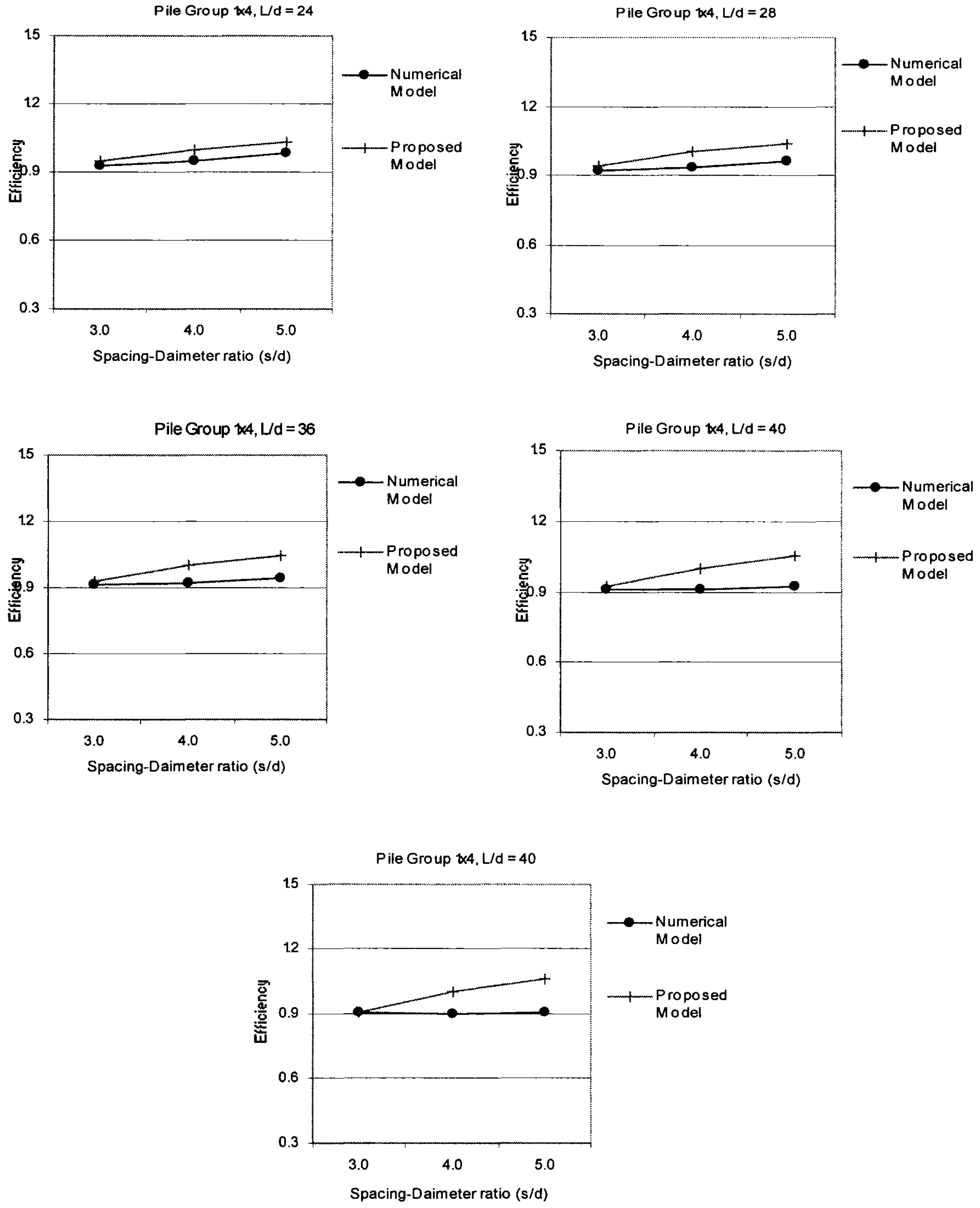


Figure 4.4: Comparison of group efficiency for 1x4 pile group (s/d variable)

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Two dimensional finite element models were developed to examine the performance of the pile shaft resistance, tip capacity, pile cap deflection and the group efficiency under vertical compressive load on cohesion-less soil. A parametric study was performed on the parameters believed to govern this behavior. An analytical formula was developed for practicing use to predict the group efficiency of line pile group. The results produced by the analytical formula compared well with the results obtain by the numerical model. The following conclusions are made:

1. Pile cap rigidity has an effect on the displacement of group pile, which was not taken in account in the previous models. It was found that; 2.5d to 3d cap thickness gave uniform displacement of the pile group.
2. Distribution of applied point load over the pile group never uniform. The piles, which are closer to the point load carry higher load than the outer pile in-group. For odd line pile group the middle pile takes highest share of the load, while for even line pile groups the central two piles take the highest share of the load. In all cases, the load on the individual piles decreases for the pile in the outer direction.
3. Sequence of pile installation has an effect on the capacity on the individual pile in-group. Pile, which is installed first, produce more cap deflection than that of which are installed later. It happened due to the progressive compaction of the surrounding soils.

4. Pile group give high bearing capacity when the center to center spacing of the individual pile is 3d, for small pile group the effective spacing-diameter ratio should be 3 and for large pile group it may be 3-3.5.
5. The existing formulas did not take into account the effect of L/d ratio and the angle of shearing resistance of the soil. The proposed formula it is easy to use, and it is proven to predict accurate line pile group efficiency for cohesion-less soil of bored piles.
6. For line pile group efficiency varies from 0.8 to 1.1. The efficiency of pile groups decreases with the increase of length-diameter ratio. Dense sand produces higher efficiency than that of loose sand.

5.2 Recommendations for future research

1. To conduct experimental investigation on the distribution of the load on the individual piles in the group.
2. Three-dimensional modeling would significantly advance the area of pile groups of any shapes.
3. Present investigation can be extended to the groups subjected to lateral loading.

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Appendix

Table A.1: Comparing the proposed formula over Numerical test Results (1x2 group)

	Test Parameter											Efficiency	
Test Number	Soil Frictional Angle(ϕ)	Pile Diameter (D) m	Number of piles per Row (N1)	Number of piles per Column (N2)	Pile Spacing (S) m	Pile Spacing (S) ft	S/D	L/D	Pile Arrangement	Pile Length (L) m	η_g	Numerical Test Result	Proposed model
1	30	0.0254	1	2	0.076	0.250	3.0	24	1x2	0.6	1.59	0.95	0.92
2	30	0.0254	1	2	0.076	0.250	3.0	28	1x2	0.7	1.59	0.94	0.91
3	30	0.0254	1	2	0.076	0.250	3.0	35	1x2	0.9	1.59	0.93	0.88
4	30	0.0254	1	2	0.076	0.250	3.0	39	1x2	1	1.59	0.91	0.87
5	30	0.0254	1	2	0.076	0.250	3.0	50	1x2	1.27	1.59	0.90	0.84
6	32.5	0.0254	1	2	0.076	0.250	3.0	24	1x2	0.6	1.59	0.96	0.89
7	32.5	0.0254	1	2	0.076	0.250	3.0	28	1x2	0.7	1.59	0.94	0.88
8	32.5	0.0254	1	2	0.076	0.250	3.0	35	1x2	0.9	1.59	0.93	0.85
9	32.5	0.0254	1	2	0.076	0.250	3.0	39	1x2	1	1.59	0.92	0.83
10	32.5	0.0254	1	2	0.076	0.250	3.0	50	1x2	1.27	1.59	0.91	0.80
11	35	0.0254	1	2	0.076	0.250	3.0	24	1x2	0.6	1.59	0.97	0.87
12	35	0.0254	1	2	0.076	0.250	3.0	28	1x2	0.7	1.59	0.96	0.85
13	35	0.0254	1	2	0.076	0.250	3.0	35	1x2	0.9	1.59	0.95	0.81
14	35	0.0254	1	2	0.076	0.250	3.0	39	1x2	1	1.59	0.93	0.80
15	35	0.0254	1	2	0.076	0.250	3.0	50	1x2	1.27	1.59	0.92	0.75
16	30	0.0254	1	2	0.102	0.333	4.0	24	1x2	0.6	1.91	0.94	0.94
17	30	0.0254	1	2	0.102	0.333	4.0	28	1x2	0.7	1.91	0.93	0.93
18	30	0.0254	1	2	0.102	0.333	4.0	35	1x2	0.9	1.91	0.92	0.92
19	30	0.0254	1	2	0.102	0.333	4.0	39	1x2	1	1.91	0.91	0.91
20	30	0.0254	1	2	0.102	0.333	4.0	50	1x2	1.27	1.91	0.89	0.89
21	32.5	0.0254	1	2	0.102	0.333	4.0	24	1x2	0.6	1.91	0.95	0.92
22	32.5	0.0254	1	2	0.102	0.333	4.0	28	1x2	0.7	1.91	0.94	0.91
23	32.5	0.0254	1	2	0.102	0.333	4.0	35	1x2	0.9	1.91	0.93	0.88
24	32.5	0.0254	1	2	0.102	0.333	4.0	39	1x2	1	1.91	0.92	0.87
25	32.5	0.0254	1	2	0.102	0.333	4.0	50	1x2	1.27	1.91	0.91	0.84
26	35	0.0254	1	2	0.102	0.333	4.0	24	1x2	0.6	1.91	1.00	0.89
27	35	0.0254	1	2	0.102	0.333	4.0	28	1x2	0.7	1.91	0.98	0.88
28	35	0.0254	1	2	0.102	0.333	4.0	35	1x2	0.9	1.91	0.96	0.85
29	35	0.0254	1	2	0.102	0.333	4.0	39	1x2	1	1.91	0.94	0.83
30	35	0.0254	1	2	0.102	0.333	4.0	50	1x2	1.27	1.91	0.92	0.80
31	30	0.0254	1	2	0.127	0.417	5.0	24	1x2	0.6	2.23	0.96	0.96
32	30	0.0254	1	2	0.127	0.417	5.0	28	1x2	0.7	2.23	0.95	0.95
33	30	0.0254	1	2	0.127	0.417	5.0	35	1x2	0.9	2.23	0.93	0.94
34	30	0.0254	1	2	0.127	0.417	5.0	39	1x2	1	2.23	0.92	0.93
35	30	0.0254	1	2	0.127	0.417	5.0	50	1x2	1.27	2.23	0.90	0.92
36	32.5	0.0254	1	2	0.127	0.417	5.0	24	1x2	0.6	2.23	0.96	0.93
37	32.5	0.0254	1	2	0.127	0.417	5.0	28	1x2	0.7	2.23	0.95	0.92
38	32.5	0.0254	1	2	0.127	0.417	5.0	35	1x2	0.9	2.23	0.93	0.90
39	32.5	0.0254	1	2	0.127	0.417	5.0	39	1x2	1	2.23	0.91	0.89
40	32.5	0.0254	1	2	0.127	0.417	5.0	50	1x2	1.27	2.23	0.90	0.87
41	35	0.0254	1	2	0.127	0.417	5.0	24	1x2	0.6	2.23	0.96	0.90
42	35	0.0254	1	2	0.127	0.417	5.0	28	1x2	0.7	2.23	0.95	0.89
43	35	0.0254	1	2	0.127	0.417	5.0	35	1x2	0.9	2.23	0.94	0.86
44	35	0.0254	1	2	0.127	0.417	5.0	39	1x2	1	2.23	0.93	0.85
45	35	0.0254	1	2	0.127	0.417	5.0	50	1x2	1.27	2.23	0.91	0.82

Table A.2: Comparing the proposed formula over Numerical test Results (1x3 group)

	Test Parameter											Efficiency	
Test Number	Soil Frictional Angle(ϕ)	Pile Diameter (D) m	Number of piles per Row (N1)	Number of piles per Column (N2)	Pile Spacing (S) m	Pile Spacing (S) ft	S/D	L/D	Pile Arrangement	Pile Length (L) m	η_g	Numerical Test Result	Proposed model
47	30	0.0254	1	3	0.076	0.250	3.0	28	1x3	0.7	1.70	0.94	0.93
48	30	0.0254	1	3	0.076	0.250	3.0	35	1x3	0.9	1.70	0.93	0.91
49	30	0.0254	1	3	0.076	0.250	3.0	39	1x3	1	1.70	0.92	0.90
50	30	0.0254	1	3	0.076	0.250	3.0	50	1x3	1.27	1.70	0.91	0.88
51	32.5	0.0254	1	3	0.076	0.250	3.0	24	1x3	0.6	1.70	0.97	0.91
52	32.5	0.0254	1	3	0.076	0.250	3.0	28	1x3	0.7	1.70	0.96	0.90
53	32.5	0.0254	1	3	0.076	0.250	3.0	35	1x3	0.9	1.70	0.95	0.88
54	32.5	0.0254	1	3	0.076	0.250	3.0	39	1x3	1	1.70	0.95	0.86
55	32.5	0.0254	1	3	0.076	0.250	3.0	50	1x3	1.27	1.70	0.94	0.84
56	35	0.0254	1	3	0.076	0.250	3.0	24	1x3	0.6	1.70	1.00	0.89
57	35	0.0254	1	3	0.076	0.250	3.0	28	1x3	0.7	1.70	0.98	0.87
58	35	0.0254	1	3	0.076	0.250	3.0	35	1x3	0.9	1.70	0.97	0.84
59	35	0.0254	1	3	0.076	0.250	3.0	39	1x3	1	1.70	0.95	0.82
60	35	0.0254	1	3	0.076	0.250	3.0	50	1x3	1.27	1.70	0.94	0.79
61	30	0.0254	1	3	0.102	0.333	4.0	24	1x3	0.6	2.12	0.97	0.98
62	30	0.0254	1	3	0.102	0.333	4.0	28	1x3	0.7	2.12	0.96	0.98
63	30	0.0254	1	3	0.102	0.333	4.0	35	1x3	0.9	2.12	0.95	0.97
64	30	0.0254	1	3	0.102	0.333	4.0	39	1x3	1	2.12	0.93	0.97
65	30	0.0254	1	3	0.102	0.333	4.0	50	1x3	1.27	2.12	0.92	0.97
66	32.5	0.0254	1	3	0.102	0.333	4.0	24	1x3	0.6	2.12	0.98	0.95
67	32.5	0.0254	1	3	0.102	0.333	4.0	28	1x3	0.7	2.12	0.97	0.95
68	32.5	0.0254	1	3	0.102	0.333	4.0	35	1x3	0.9	2.12	0.96	0.93
69	32.5	0.0254	1	3	0.102	0.333	4.0	39	1x3	1	2.12	0.94	0.93
70	32.5	0.0254	1	3	0.102	0.333	4.0	50	1x3	1.27	2.12	0.93	0.91
71	35	0.0254	1	3	0.102	0.333	4.0	24	1x3	0.6	2.12	0.99	0.93
72	35	0.0254	1	3	0.102	0.333	4.0	28	1x3	0.7	2.12	0.98	0.91
73	35	0.0254	1	3	0.102	0.333	4.0	35	1x3	0.9	2.12	0.96	0.89
74	35	0.0254	1	3	0.102	0.333	4.0	39	1x3	1	2.12	0.95	0.88
75	35	0.0254	1	3	0.102	0.333	4.0	50	1x3	1.27	2.12	0.93	0.86
76	30	0.0254	1	3	0.127	0.417	5.0	24	1x3	0.6	2.55	1.00	1.01
77	30	0.0254	1	3	0.127	0.417	5.0	28	1x3	0.7	2.55	0.97	1.01
78	30	0.0254	1	3	0.127	0.417	5.0	35	1x3	0.9	2.55	0.95	1.01
79	30	0.0254	1	3	0.127	0.417	5.0	39	1x3	1	2.55	0.93	1.01
80	30	0.0254	1	3	0.127	0.417	5.0	50	1x3	1.27	2.55	0.91	1.01
81	32.5	0.0254	1	3	0.127	0.417	5.0	24	1x3	0.6	2.55	1.00	0.98
82	32.5	0.0254	1	3	0.127	0.417	5.0	28	1x3	0.7	2.55	0.98	0.97
83	32.5	0.0254	1	3	0.127	0.417	5.0	35	1x3	0.9	2.55	0.96	0.97
84	32.5	0.0254	1	3	0.127	0.417	5.0	39	1x3	1	2.55	0.94	0.96
85	32.5	0.0254	1	3	0.127	0.417	5.0	50	1x3	1.27	2.55	0.92	0.96
86	35	0.0254	1	3	0.127	0.417	5.0	24	1x3	0.6	2.55	1.01	0.95
87	35	0.0254	1	3	0.127	0.417	5.0	28	1x3	0.7	2.55	0.99	0.94
88	35	0.0254	1	3	0.127	0.417	5.0	35	1x3	0.9	2.55	0.97	0.92
89	35	0.0254	1	3	0.127	0.417	5.0	39	1x3	1	2.55	0.95	0.92
90	35	0.0254	1	3	0.127	0.417	5.0	50	1x3	1.27	2.55	0.94	0.90

Table A.3: Comparing the proposed formula over Numerical test Results (1x4 group)

	Test Parameter											Efficiency	
Test Number	Soil Frictional Angle(ϕ)	Pile Diameter (D) m	Number of piles per Row (N1)	Number of piles per Column (N2)	Pile Spacing (S) m	Pile Spacing (S) ft	S/D	L/D	Pile Arrangement	Pile Length (L) m	η_g	Numerical Test Result	Proposed model
91	30	0.0254	1	4	0.076	0.250	3.0	24	1x4	0.6	1.75	0.93	0.95
92	30	0.0254	1	4	0.076	0.250	3.0	28	1x4	0.7	1.75	0.92	0.94
93	30	0.0254	1	4	0.076	0.250	3.0	35	1x4	0.9	1.75	0.92	0.93
94	30	0.0254	1	4	0.076	0.250	3.0	39	1x4	1	1.75	0.91	0.92
95	30	0.0254	1	4	0.076	0.250	3.0	50	1x4	1.27	1.75	0.90	0.90
96	32.5	0.0254	1	4	0.076	0.250	3.0	24	1x4	0.6	1.75	0.93	0.92
97	32.5	0.0254	1	4	0.076	0.250	3.0	28	1x4	0.7	1.75	0.93	0.91
98	32.5	0.0254	1	4	0.076	0.250	3.0	35	1x4	0.9	1.75	0.93	0.89
99	32.5	0.0254	1	4	0.076	0.250	3.0	39	1x4	1	1.75	0.92	0.88
100	32.5	0.0254	1	4	0.076	0.250	3.0	50	1x4	1.27	1.75	0.92	0.85
101	35	0.0254	1	4	0.076	0.250	3.0	24	1x4	0.6	1.75	0.94	0.90
102	35	0.0254	1	4	0.076	0.250	3.0	28	1x4	0.7	1.75	0.93	0.88
103	35	0.0254	1	4	0.076	0.250	3.0	35	1x4	0.9	1.75	0.93	0.85
104	35	0.0254	1	4	0.076	0.250	3.0	39	1x4	1	1.75	0.93	0.84
105	35	0.0254	1	4	0.076	0.250	3.0	50	1x4	1.27	1.75	0.92	0.81
106	30	0.0254	1	4	0.102	0.333	4.0	24	1x4	0.6	2.23	0.95	1.00
107	30	0.0254	1	4	0.102	0.333	4.0	28	1x4	0.7	2.23	0.94	1.00
108	30	0.0254	1	4	0.102	0.333	4.0	35	1x4	0.9	2.23	0.92	1.00
109	30	0.0254	1	4	0.102	0.333	4.0	39	1x4	1	2.23	0.91	1.00
110	30	0.0254	1	4	0.102	0.333	4.0	50	1x4	1.27	2.23	0.90	1.00
111	32.5	0.0254	1	4	0.102	0.333	4.0	24	1x4	0.6	2.23	0.95	0.97
112	32.5	0.0254	1	4	0.102	0.333	4.0	28	1x4	0.7	2.23	0.94	0.97
113	32.5	0.0254	1	4	0.102	0.333	4.0	35	1x4	0.9	2.23	0.93	0.96
114	32.5	0.0254	1	4	0.102	0.333	4.0	39	1x4	1	2.23	0.92	0.95
115	32.5	0.0254	1	4	0.102	0.333	4.0	50	1x4	1.27	2.23	0.91	0.94
116	35	0.0254	1	4	0.102	0.333	4.0	24	1x4	0.6	2.23	0.96	0.94
117	35	0.0254	1	4	0.102	0.333	4.0	28	1x4	0.7	2.23	0.95	0.93
118	35	0.0254	1	4	0.102	0.333	4.0	35	1x4	0.9	2.23	0.94	0.92
119	35	0.0254	1	4	0.102	0.333	4.0	39	1x4	1	2.23	0.93	0.91
120	35	0.0254	1	4	0.102	0.333	4.0	50	1x4	1.27	2.23	0.92	0.89
121	30	0.0254	1	4	0.127	0.417	5.0	24	1x4	0.6	2.71	0.98	1.03
122	30	0.0254	1	4	0.127	0.417	5.0	28	1x4	0.7	2.71	0.96	1.04
123	30	0.0254	1	4	0.127	0.417	5.0	35	1x4	0.9	2.71	0.94	1.05
124	30	0.0254	1	4	0.127	0.417	5.0	39	1x4	1	2.71	0.92	1.05
125	30	0.0254	1	4	0.127	0.417	5.0	50	1x4	1.27	2.71	0.91	1.06
126	32.5	0.0254	1	4	0.127	0.417	5.0	24	1x4	0.6	2.71	0.99	1.00
127	32.5	0.0254	1	4	0.127	0.417	5.0	28	1x4	0.7	2.71	0.97	1.00
128	32.5	0.0254	1	4	0.127	0.417	5.0	35	1x4	0.9	2.71	0.95	1.00
129	32.5	0.0254	1	4	0.127	0.417	5.0	39	1x4	1	2.71	0.93	1.00
130	32.5	0.0254	1	4	0.127	0.417	5.0	50	1x4	1.27	2.71	0.91	1.00
131	35	0.0254	1	4	0.127	0.417	5.0	24	1x4	0.6	2.71	0.99	0.97
132	35	0.0254	1	4	0.127	0.417	5.0	28	1x4	0.7	2.71	0.98	0.96
133	35	0.0254	1	4	0.127	0.417	5.0	35	1x4	0.9	2.71	0.96	0.96
134	35	0.0254	1	4	0.127	0.417	5.0	39	1x4	1	2.71	0.94	0.95
135	35	0.0254	1	4	0.127	0.417	5.0	50	1x4	1.27	2.71	0.92	0.94