NOTE TO USERS

Page(s) not included in the original manuscript are unavailable from the author or university. The manuscript was microfilmed as received.

75-76

This reproduction is the best copy available.

Modeling Pile Group Efficiency in Cohesionless Soil Using Artificial Neural Networks

Mary Helmy

A Thesis

In

The Department

Of

Building, Civil, and Environmental Engineering

Presented in Partial Fulfillment of the Requirements
For the Degree of Master of Applied Science at
Concordia University
Montreal, Québec, Canada

October 2002

ABSTRACT

Modeling Pile Group Efficiency in Cohesionless Soil Using Artificial Neural Networks

Mary Helmy

For the past few decades, the subject of pile group action has been of interest to many researchers in the area of foundation engineering. This is because piles are often used in groups to transmit structural loads down to deep soil strata and to reinforce the surrounding soil. When piles are widely spaced, their response to loading is based on their individual stiffness and the single pile failures are more likely to happen in this case. However, closely placed piles interact with each other through the surrounding soil upon loading and block failures are more likely to occur in this case.

Several empirical and analytical methods have been used to resolve the complexity of estimating the ultimate bearing capacity of pile groups. These methods have resulted in simple and practical design equations that calculate the pile group efficiency, which is the ratio between the bearing capacity of a pile group and the summation of bearing capacities of the individual piles in the group. Most of these design equations accounts only for the parameters that describe the planner geometry of the pile group, such as pile diameter, pile spacing, and pile arrangement. However, other parameters that may have serious impact on the pile group efficiency, such as the soil condition, pile cap condition, pile length, method of pile installation, and type of pile loading, were completely ignored. Moreover, existing design equations cannot be easily updated when the results of new laboratory and/or field tests become available.

Therefore, the objective of this research is twofold: first, to evaluate the reliability of existing design theories; and second, to develop a new model that eliminates the shortcomings of the existing theories. To fulfill the first objective, the results of several laboratory and field tests were obtained from the literature and compared with the pile group efficiency calculated using the existing design theories. This comparison revealed the inadequate accuracy of these theories in addition to their contradictory predictions.

To fulfill the second objective, artificial neural networks (ANN), one of the artificial intelligence techniques, was used to develop a computer model that predicts pile group efficiencies. This model benefits from the actual data that are available in the literature to link the pile group efficiency variable with several governing parameters, such as the method of pile installation, soil condition, cap condition, type of loading, pile cross section, pile length / diameter ratio, pile spacing / diameter ratio, and pile arrangement. Validating the ANN model using a set of data that is different from the one used in model development has indicated that the ANN model has better performance characteristics (i.e. efficiency, consistency, and accuracy) than existing design theories. In addition, the developed ANN model can be easily updated when new data becomes available and further extended to accommodate new design parameters.

ACKNOWLEDGEMENT

I would like to acknowledge my supervisor Dr. A. M. Hanna for his tremendous effort in advising, teaching, and supporting me throughout this research. I would like also to greatly thank my husband and my son David for their continuous encouragement and extreme love.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

Symbol	Description
ANOVA	Analysis of variance
ANN	Artificial neural networks
BPNN	Back propagation neural network
Q_s	capacity of a single pile
Q_g	capacity of the pile group
R2	Coefficient of determination
r	Correlation coefficient
ρ	Friction factor
$\acute{\eta}s$	Geometric efficiency
K	Interaction coefficient
α	Level of significance
e	Load eccentricity
MAE	Mean absolute error
m	Number of pile per column
N	Number of piles
n	Number of piles per row
Pg	Perimeter of pile group
Pp	Perimeter of single pile
D	Pile Diameter
η	Pile group efficiency
L	Pile length
S	Pile Spacing
B	Pile width
Dr	Relative density
RMSE	Root mean square error
Φ	Soil friction angle
γ	Unit weight of soil

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Pile foundations are extensively used in bridges, high-rise buildings, towers, and many other structures. It is often recommended to use this type of deep foundations rather than shallow foundations when the upper soil layer is highly compressible and cannot support heavy structural loads. In this case, piles are used to transmit these loads to a stronger and deeper soil stratum. Also, in high-rise buildings and towers subjected to horizontal loads resulting from wind or earthquakes, piles are required to support these forces by bending. In case of expensive or collapsible soils where the use of shallow foundations may cause a considerable damage to the structure, pile foundations are extended beyond the active zone in order to transmit the structural load to a more stable bearing stratum.

The two main categories of piles are displacement piles (driven and jacked piles) and non-displacement piles (bored piles). For the displacement piles, soil moves radially and also vertically while piles are installed. This movement causes a favorable compaction effect in case of cohesionless soils and heave in case of cohesive soils. For the non-displacement piles, soil is excavated then piles are installed. As a result, lateral stresses are reduced but the favorable effects of compaction resulting from soil displacement are eliminated.

In practice, piles are generally used in groups in order to transmit the structural load to down deeper, stronger soil strata and also to reinforce its surrounding soil. Two different classes of pile groups exist: First, when individual piles are widely spaced, their response to loading depends on their individual stiffness and their failure in this case may happen to each pile individually. Second, piles are situated closely enough to each other that the response of a given individual pile to loading is influenced by loading upon neighboring piles (group action). In this case piles interact with each other through the surrounding soil and block failure may occur.

Most of the developed analytical methods are concerned with isolated single piles rather than pile groups; this may be because of the complexity of the assessment of pile group behavior under different loading conditions. The two major problems concerning pile group design are:

a) Evaluation of pile group efficiency (η)

$$\eta = Q_g / \Sigma Q_s$$

Where Q_g = capacity of the pile group

Q_s = capacity of a single pile.

b) Evaluation of increased pile-head flexibilities of group piles with respect to single piles using a settlement ratio (S_r)

$$S_r = \delta_g / \delta s$$

Where δ_g = settlement of a pile group

δ_s = settlement of a single pile.

This research focuses on the evaluation of pile group efficiency (η) for both displacement and non-displacement piles subjected to axial loading and placed in loose or dense sandy soil.

1.2 PROLBLEM DEFINITION

Many researchers have developed theoretical models for the evaluation of pile group efficiency based on the results of field and laboratory tests. However, these models could not provide engineers with a reliable prediction of the pile group capacity because of the following reasons:

a. The nature of pile-soil-pile interaction consists of two components (O' Neill 1983): first, installation effects, which consist of alteration of soil stresses due to driving piles closely to others, and second, mechanical effects, which consist of strain superposition in the soil mass and alteration of failure zones due to simultaneous loading of neighboring piles. Most of the existing pile-soil-pile interaction models consider only the mechanical effects and ignore the installation effects of the pile-soil-pile interaction.

b. There are several factors that affect the pile group action phenomena, such as soil characteristics, cap-pile-soil interaction, type of loading and pile length (i.e. depth of embedment). However, most of existing theoretical models are concerned only

with the planar geometry of pile groups (i.e. pile spacing, pile diameter, and pile arrangement) and do not lake into consideration any of the previous factors.

c. The developed theoretical equations are considered as static models that cannot be easily updated when a new set of data are obtained either from laboratory or field tests. This fact restrict the extensibility of these models and result in their obsoleteness

1.3 RESEARCH OBJECTIVES

The objectives of this research can be summarized as follows:

1. To conduct a literature review on the existing theoretical models for pile group efficiency in cohesionless soil and evaluate the reliability of these models by comparing their predicted values with the values obtained from the different field and laboratory tests conducted in the literature.
2. To determine the governing factors that affect the pile group efficiency based on some statistical analyses and parametric studies of the existing model test data.
3. To develop a new computer model that incorporates all the governing factors and actual data to determine the pile group efficiency for piles in cohesionless soils. The model can be easily updated to provide design engineers with relatively accurate prediction for pile group efficiencies.

1.4 THESIS ORGANIZATION

This thesis is organized as follows:

Chapter 2 Literature Review: This chapter presents a brief discussion on the existing theoretical models proposed by many researchers for calculating pile group efficiency. It also presents the different model tests (field and laboratory tests) carried out during experimental investigations along with their results.

Chapter 3 Analysis: This chapter presents a comparison between pile group efficiency values calculated using the different theoretical models and those resulting from model tests. Also, the statistical analysis and parametric study carried out to determine the governing factors that affect pile group efficiency are presented.

Chapter 4 Proposed Methodology: This chapter introduces artificial neural networks (ANN), their different uses, components, and operations. It also presents the development of an ANN model for predicting pile group efficiency and comparing the results of this model against those of the theoretical models.

Chapter 5 Conclusions and Recommendations: This chapter highlights the contributions of this research and suggests recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The determination of the group efficiency (η) of piles subjected to axial loading in cohesionless soils has intrigued and challenged engineers for a very long time and it still occupy a place of high importance in geotechnical engineering. Over three decades ago, the bearing capacity of pile groups was perceived to be equal to the sum of the bearing capacities of the individual piles, but in practice, when piles are placed close to each other, the stresses transmitted to the soil through the piles will overlap, resulting in a considerable change of their bearing capacities, which is known by pile group efficiency.

Later on, with the development of the instrumentation in geotechnical engineering and powerful computers, researchers started to examine the problem from the theoretical and experimental point of view. Several data were collected, exchanged, and analyzed in order to resolve the complexity of this problem. This chapter presents the model tests and the theoretical equations reported in the literature to estimate the pile group efficiency along with their pros and cons.

2.2 HISTORICAL DEVELOPMENT

The literature of pile group efficiency can be classified into two categories:

a) Model tests: which include the large-scale field tests or small-scale laboratory tests carried out on single piles and pile groups subjected to axial loading.

b) Theoretical models: which were proposed by many researchers to estimate pile group efficiency.

The following subsections present each of these categories in a chronological order.

2.2.1 MODEL TESTS

In order to analyze the bearing capacity of pile groups under central and eccentric loading, Kishida et al (1969) conducted a number of model tests on freestanding groups and piled foundations. Many parameters were involved in this study including: number of piles (N), pile spacing (S), load eccentricity (e), and type of soil.

Model piles had a diameter of ½ in, embedment lengths of 11 in for freestanding groups and 12 in for piled foundations. Tests were conducted on single piles as well as on pile groups including: 2 x 2 and 3 x 3 square groups embedded in loose (soil friction angle $\varphi = 35^{\circ}$) and medium dense sand (soil friction angle $\varphi = 43^{\circ}$) with different eccentricities of the load applied to the pile cap. Test results are presented in Figures 2.1 and 2.2 as a plot of the applied load (Ib) versus settlement (in) for all test groups as well as different eccentricities of load in cases of freestanding pile groups and piled foundations respectively. For the above-mentioned conditions, test results showed that:

a) For free standing pile groups and under central load, the total bearing capacity of pile groups in loose sand increases by decreasing their spacing and it reaches its

maximum value at a pile spacing (S) equals twice the pile diameter (2D), where pier failure occurs. On the contrary, in dense sand the bearing capacity decreases by deceasing pile spacing and pier failure may not occur due to dilatancy effects.

b) For piled foundations and under central load, the total bearing capacity of pile groups in loose sand increases by increasing their spacing and pier failure occurs at a pile spacing equals to three times the pile diameter. For dense sand, the same observations were made.

c) For both freestanding pile groups and piled foundations, small eccentricities have no significant effect on the bearing capacity of pile groups, while greater eccentricities tend to decrease their bearing capacities because smaller point resistances are produced due to the reduction of the effective base area.

d)

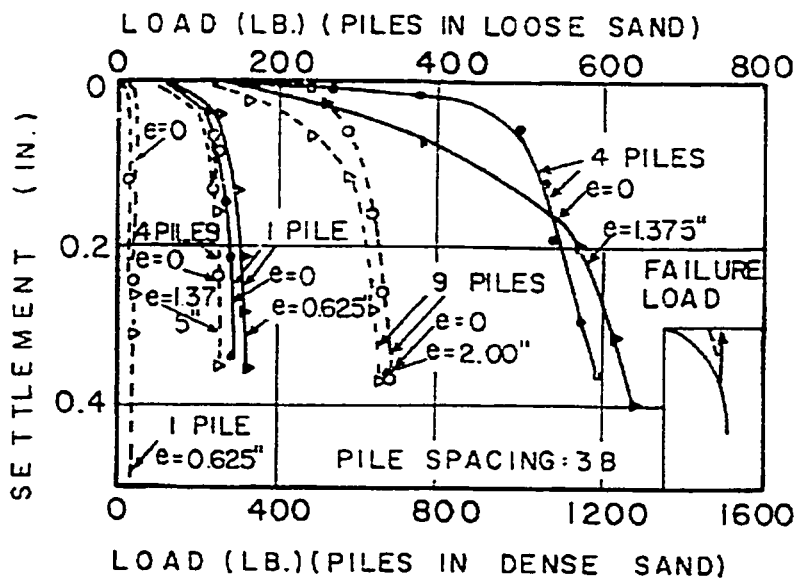


Figure 2.1: Load-settlement relationships for different pile groups and load eccentricities in case of freestanding piles after Kishida et al (1969)

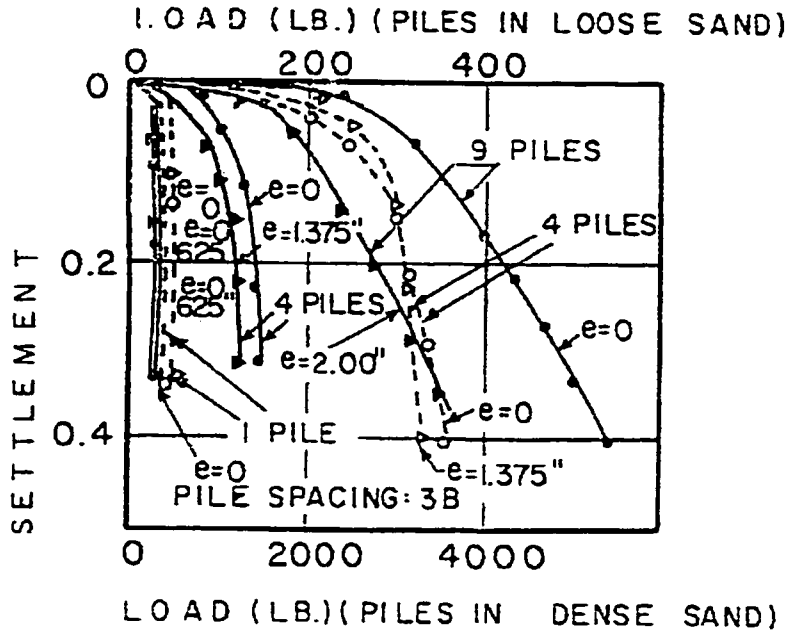


Figure 2.2: Load-settlement relationships for different pile groups and load eccentricities in case of piled foundation after Kishida et al (1969)

Vesic (1968) conducted a large scale model test on 4 and 9 square pile groups with and without caps in order to study their behaviour when embedded in cohesionless soil. Model piles had a diameter (D) of 4 in, an embedment length of 60 in, and spaced at 2 to 6d centre to centre. These piles were jacked into dry and submerged sand at two soil conditions. First, homogeneous medium dense sand with relative density (Dr) = 65%. Second, a two layer deposit consisting of an upper layer of very loose sand with Dr = 20% and a lower layer of a very dense sand with Dr = 80%.

Figure 2.3 presents a plot between the pile spacing and the pile group efficiency. From this figure it can be clearly noticed that:

a) Maximum overall efficiency of four pile groups with cap embedded in homogeneous soil was 1.7 reached at a pile spacing equals 3 to 4D. Increasing the pile spacing tends to decrease this overall efficiency.

b) Skin efficiency is much higher than the point efficiency and it reaches a maximum value of 3 at a pile spacing equals to 5D. This means that the point loads is unaffected by the group action.

c) There exist very small differences between the 4 and 9 pile group efficiencies except for the case of cap contribution where the total overall efficiency of the 9-pile group is higher than that for the 4-pile group.

Measurements of axial load distribution along piles in the group during and after applying the load showed that there exists a degree of uniformity of the load distribution among piles especially for the 4 square pile group. For the 9-pile group, there was a difference between the amount of load carried by each pile according to its position in the group. In other words, centre piles carried about 36% more than the average load while edge piles carried 3% more than the average and the corner pile carried 12% less than the average.

This study was limited to square pile groups and did not consider the effect of other group arrangements on the group efficiency of piles. Also, it did not consider the order of pile driving while examining the load distribution along piles.

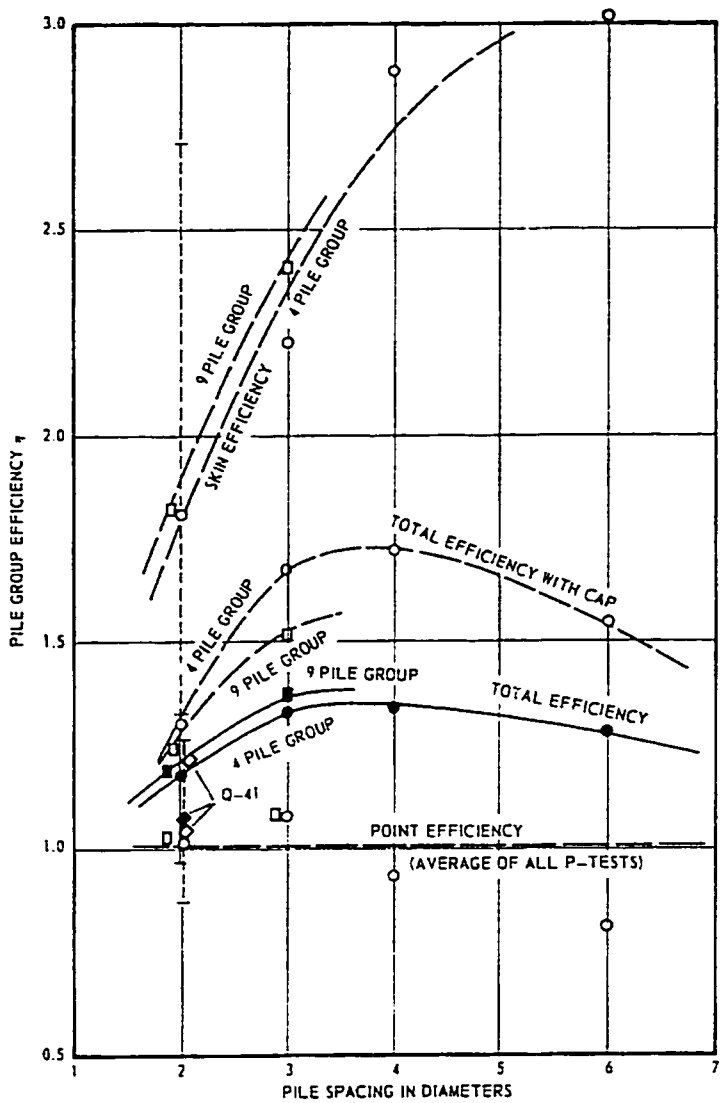


Figure 2.3: Pile group efficiency versus pile spacing for the four and nine pile groups after Vesic (1968)

Tejchman (1973) conducted a number of model tests on pile groups, in order to analyze their group efficiencies in cohesionless soils. Piles were constructed as precast concrete piles having a square cross section with dimension B = 3.5 cm and a total length of 60 cm, their depth of embedment was 52.5 cm. Model tests were conducted on pile

groups of different arrangements including: 2x2 and 3x3 Square groups, 2x4 Rectangular group, and 1x4 line group for two types of soil: Loose sand with relative density (Dr) =0.226 and Medium dense sand with relative density (Dr) =0.557. Also, different values of pile spacing (S) were used including: S = 2B, 3B, 4B, 5B, 6B, and 9B. In these tests piles were loaded until the ultimate bearing capacity similar to that of the loading test for a single pile was reached. For the above-mentioned conditions, test results showed that:

a) For both loose and dense sand conditions, increasing the number of piles in a group and narrowing the spacing between them leads to an increase in their group efficiencies for all group arrangements except for the (1x4) line group embedded in dense sand.

b) In general, settlement of pile groups is greater then that for single piles. This difference becomes clearer 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 this spacing till it reaches the efficiency corresponding to the sum of the bearing capacities of individual piles at S >6B.

d) For loose sand and in case of (1x4) pile group, the group efficiency (η) was increased by very small amounts by decreasing the pile spacing (S) so that the efficiency in this case may be assumed to correspond to the sum of the bearing capacities of individual piles.

e) For medium dense sand, the group efficiency (η) had lower values than those obtained for loose sand, and its value decreased by narrowing the spacing between piles in case of (1 x 4) line group.

In this study many parameters were used in the analysis of pile group efficiency including: pile spacing, type of soil, and pile group arrangement. Test results showed that these parameters have great influence on the group efficiency of piles. While based on the earlier reported experimental investigations, many other parameters have considerable effects. These parameters are: 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 respectively. All of the tested piles had a diameter of 15 cm and an embedment depth (L) of 300 cm. These tests were performed for two cap conditions: cap resting directly on soil, and freestanding cap.

The purpose of these tests was to provide an approach for the design of bored undreamed pile groups and also to study the load-displacement mechanism of pile groups. These tests consisted of loading piles and pile groups with incremental loadings using hydraulic jacks. Strain gauges were connected at many points within pile groups in order to determine the load-displacement characteristics at these points. Test results showed that:

a) There is no definite failure load for a single pile, however the ultimate load was determined through time-displacement plot.

b) For pile groups, the increase in pile spacing tended to increase the pile group efficiency, for pile cap resting condition. The effect of pile spacing on the group efficiency is presented in Figure 2.4.

c) Group efficiency decreases with the increase of the number of piles for both freestanding and resting cap conditions as it is shown in Figure 2.5.

d) The load taken by a certain pile group for pile cap resting condition is higher than that for free standing condition, for the same settlement value.

e) When evaluating the capacity of pile groups for pile cap resting condition, only the contribution from the outer rim of the pile cap should be considered.

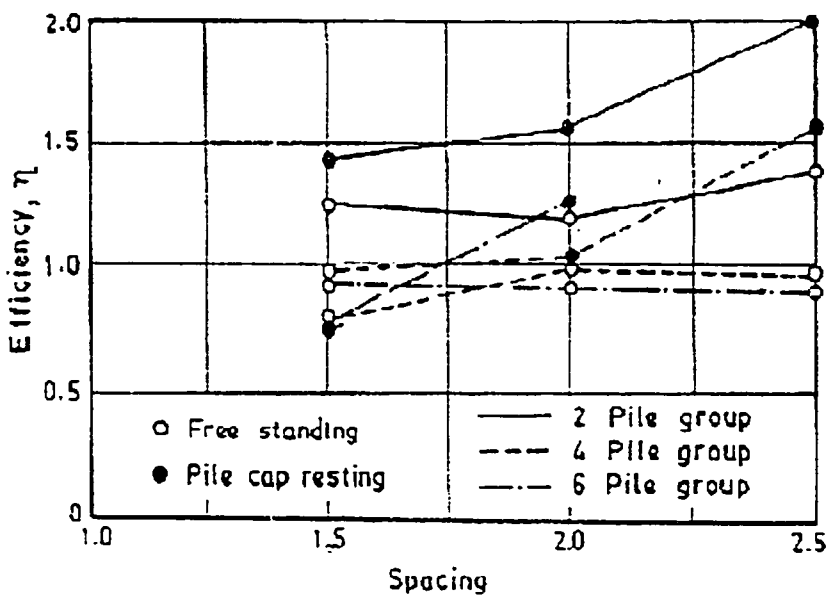


Figure 2.4: Pile group efficiency versus pile spacing for the two cap conditions after Garg (1979)

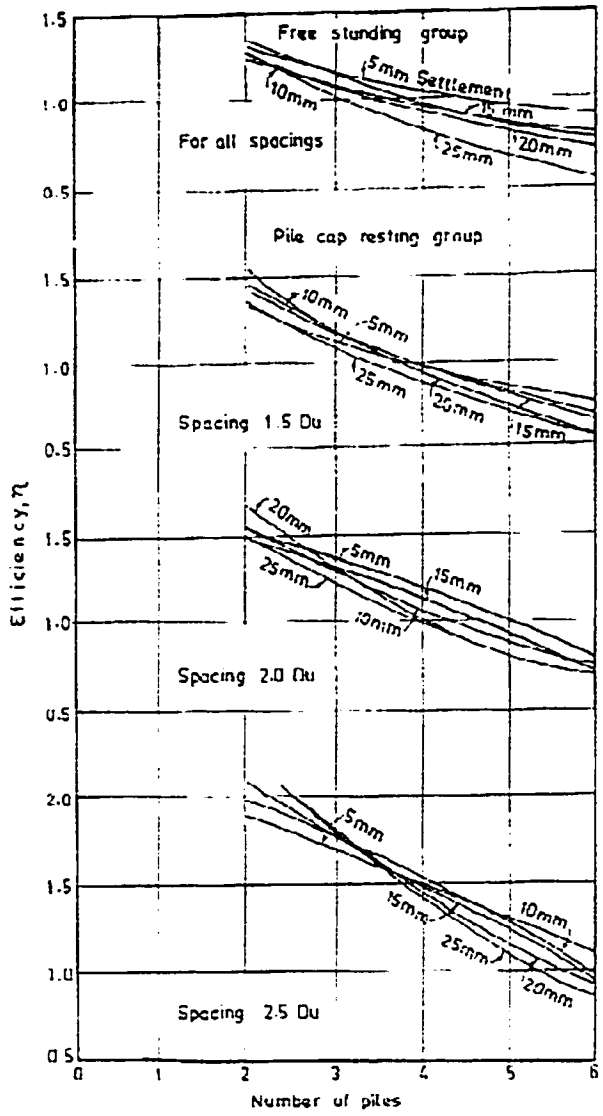


Figure 2.5: Pile group efficiency versus number of piles for the two cap conditions after Garg (1979)

This study has the following shortcomings:

a) Limited to square pile group arrangement.

b) It was concluded that the contribution of the pile cap couldn't be defined as a percentage of the total load carried by the pile group. This fact was changed later by Chen et al (1993).

O Neill (1983) defined a number of problems concerning pile group behaviour through some recent experimental data and analytical technics. First problem defined is that the pile-soil-pile interaction consists of two major components: First, installation effects which consists of soil stress changes due to piles installation closely to each other. Second, mechanical effects which consists of the superposition of pile strains and alteration of their failure zone due to simultaneous loading of one or more neighbouring pile. Most of the pile-soil-pile models are concerned with with the second component and always neglect the installation effects, which makes them incomplet source of judgement that the designer can not rely on.

The second problem is that no mathematical model can be considered as a reliable tool for the evaluation of η, this may be because none of these models could assess all the factors that affect its computation.

The third problem defined can be summarized as follows: Excessive errors can be commited 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.

Liu et al (1985) carried out a large-scale field test on 51 bored pile groups in a loose sandy soil in order to analyze the effect of cap-pile-soil interaction on their behaviour. This study considered the effects of many parameters involved. these parameters include:

a) Pile diameter (D = 125-330mm).

b) Pile length (L =8 to 23D).

c) Pile spacing (S = 2 to 6D).

d) Number of piles (N = 2 to 16).

e) Pile arrangement (square, rectangular, and one line).

f) Position of pile cap (in direct contact with soil, freestanding).

g) Long-term loading.

During the tests, load was applied gradually to the piles and the pile groups using hydraulic jacks. For the above mentioned test conditions, test results showed that:

a) For pile groups, if $L \geq 1.5Bc$ (where Bc is the width of the pile cap, Bc = 9D), side resistance continues to increase with settlement until it reaches the peak value for single piles. This behaviour is called "settlement hardening". But if $L < 1.5Bc$, pile groups behaviour is similar to that of single piles, this is called "settlement softening".

b) The average side resistance of pile groups with low-set cap is smaller than that for high-rise cap, so the pile cap has a "weakening effect" on the side resistance.

c) The point resistance of pile groups with high-rise cap is smaller than that of the same groups with low-set cap. This means that the pile cap has a "strengthening effect" on the point resistance.

A new calculation model was proposed in order to evaluate the group efficiency of bored pile groups. This model was based on introducing separate pile group efficiency factors for side and point resistances and adding the effect of cap-pile –soil interaction in terms of the soil reaction beneath the pile cap. The side and point group efficiencies ηs and ηp respectively are calculated as follows:

$$\eta s = Gs * Cs \tag{2.1}$$

$$\eta p = Gp * Cp \tag{2.2}$$

$$Gs = \frac{\alpha}{Ln(e + 1 - \frac{r+m}{2m})} \tag{2.3}$$

$$Gp = \frac{8}{(\frac{Sa}{d} - 3)^2 + 9} \tag{2.4}$$

$$Cs = 1 + 0.1\frac{Sa}{d} - 0.9\frac{Bc}{L} \tag{2.5}$$

$$Cp = 1 + 0.2\frac{Sa}{d} \cdot \frac{Bc}{L} \tag{2.6}$$

Where

Gs, Gp: coefficient considering the effect of pile-soil interaction on side and on point resistance respectively.

Cs, Cp: coefficient considering the effect of cap-pile-soil interaction on side resistance and on point resistance respectively.

For high rise cap;

$Cs = Cp = 1$.

e : base of natural algorithm.

r, m: number of rows and number of piles in each row respectively.

Bc = SQT (length of cap (a) x width of cap (b).

If a / b $\geq$ 9, then Bc = 3d.

α: dimensionless coefficient.

The present study has the advantage over many previous model tests conducted by researchers in considering many parameters that were ignored in earlier studies and may have influence on the behaviour of bored pile groups in sandy soils. Also, a new calculation method was proposed for the pile group efficiency (η) based on separate coefficients for both side and base resistances. This new model takes into consideration the cap-pile-soil interaction effects, which were completely ignored by the conventional pile group efficiency theoretical methods based on the "equivalent pier" model. But, it carries the disadvantage of the complexity of calculation of the group efficiency based on the proposed model, also, it can not be considered as a quick and easy tool that can be used by design engineers as it includes too much manual work.

Briaud et al (1989) conducted a field loading test on 5 driven single piles and a group of 5 driven piles. The soil where the piles were driven was stiff silty clay with some layers of sand. Piles used were steel close ended pipe piles having an outside diameter of 27.3cm and instrumented with strain gages along the pile's shafts as well as top and toe load cells. Single piles were loaded with load increments of 45 KN while holding the load for 30 minutes and recording the instruments every 5 minutes. Load increments of 267

KN were hold for 30 minutes and instrumentation as well as displacements were recorded every 5 minutes. Test results for single piles and for the pile group showed that:

a) Pile group efficiency (η) with respect to the total plunging load had a value of 0.99. This value is in a good agreement with conventional practice, but does not agree with previous model test results conducted for the same type of soil.

b) At failure, pile group efficiency (η) for the frictional load was 1.83. This is due to the increase in effective vertical stress induced by driving piles in a group.

c) At failure, pile group efficiency (η) for the point load was 0.67. This is due to the loss of prestressing under driven piles due to the driving of a new pile.

d) Pile group efficiency (η) does not depend on pile spacing and the type of soil only, but it depends on the load distribution along the piles as well as the depth of embedment.

Based on this study, pile group efficiency (η) depends on pile spacing, type of soil, distribution of loads along piles, and depth of embedment. Many researchers found that the method of installation, group arrangement, and cap-pile-soil interaction have great influence on the pile group efficiency. In other words, the evaluation of pile group efficiency (η) should be based on all the above-mentioned parameters in order to achieve reliable results.

Chattopadhyay (1994) conducted a number of model tests on pile groups in order to analyze their uplift capacity. Model piles had a diameter (D) of 19mm and an embedment length ranging between 300 and 600 mm. Three pile groups consisting of: 2 x 1 line

group, 3 triangular group, and 2 x 2 square group were tested for pile spacing ranging between 2.3D to 6D. The first series of the test was conducted on dry sand with D60 = 95 mm, D10 = 48 mm, and cu = 1.98. For the second series of the test, locally available blackish grey clay silt soil having 8% clay and 92% silt was used. Test results in dry sand soil showed that:

a) For the same pile spacing, in case of uplift loading of the three pile groups: 2 x 1line group, 3 triangular group, and 2 x 2 square group, the pile head/cap displacement which caused uplift failure was much less for smaller depths of embedment.

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

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

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 shafts 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. Also, vertical deformation of soil was detected by using special equipments for two pile cap conditions (Low-set and high rise cap) while varying loading conditions.

In order to verify the cap-pile-soil interaction effects on the side resistance of pile groups, Figure 2.6 shows the mean side resistance values measured during the tests and plotted against settlement in case of low-set cap. From this figure the following observations were made:

a) In case of low set cap, the measured mean side resistance increases by increasing pile spacing. In other words, the cap-pile-soil interaction has a reduction influence on the side resistance of pile groups.

b) For the same settlement value, the mean side resistance for single pile is greater than that for a pile group.

c) At the middle of the shaft, the side resistance of pile groups is higher than that for single piles. This may be due to that the soil becomes more compacted, thus, causing "strengthening effect". By increasing pile spacing, strengthening effect may disappear and the side resistance of pile groups becomes closer to that of single pile.

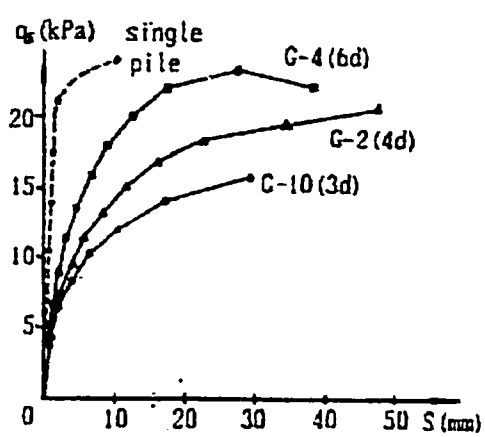


Figure 2.6: Mean side resistance versus settlement in case of low-set cap after Liu et al. (1994)

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 coefficients of group efficiency (η) effect were 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:

$$\eta_s = G_s C_s \tag{2.7}$$

$$\eta_p = G_p C_p \tag{2.8}$$

$$\eta_c = \eta_c^{ex} A_c^{ex} / A_c + \eta_c^{in} A_c^{in} / A_c \tag{2..9}$$

Where,

G_s = side resistance effect coefficient of pile group

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:

$$G_s = 1.2 - 1.2\ D/S \tag{2.10}$$

$$G_p = 6\ D/S - Ln\ (e - (6 - S/D) / 6) \tag{2.11}$$

$$C_s = 1 + 0.12\ (S/D)^{1/2} - 0.5\ [(B_c/L) - Ln\ (0.3\ B_c/L + 1)] \tag{2.12}$$

$$C_p = 1 + 0.1\ (S/D)\ Ln\ (0.5\ B_c/L + 1) \tag{2.13}$$

$$\eta_c^{ex} = (S/D + 2) / 8 \tag{2.14}$$

$$\eta_c^{in} = 0.08\ (S/D)\ (B_c/L)^{1/2} \tag{2.15}$$

Where,

S = pile spacing

D = pile diameter

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 group efficiency model carries the advantage of considering the cap-pile-soil interaction, which was ignored by the conventional models. But the complexity of calculation can be considered as its major shortcoming.

Mukherjee (1996) carried out an experimental investigation in order to study the pullout capacity of piles / pile groups in sand. This study included the behaviour aspects of piles / pile groups in terms of several parameters involved. The experiment was carried out in a segmented aluminium tank of size 900 mm x 900 mm x 1100 mm deep, filled with sand as foundation medium. Model piles of 25.4 mm diameter were tested in a number of 57 tests varying: embedment length, pile spacing and group geometry. The values used for the embedment length were 600 mm, 750 mm and 900 mm and for pile

spacing were 75 mm, 100 mm and 125 mm. Line groups of 1x2, triangular group of 1x3, square group of 2x2 and rectangular group 2x3 were tested. Axial movements of instrumented pile were measured during uplift loading up to failure, failure surface was obtained during the tests by specifying some breakage points of a very fragile material placed around tested piles.

In order to supplement his experimental work, a numerical analysis using finite element method was carried out. The analysis consisted of 8 nodes rectangular isoparametric elements; pile groups were simulated to a single pile with some perforations in the wall. Those perforations were made in such a way that the total cross sectional area of any level equal to the sum of the areas of cross sections of piles in the group, uplift capacities were obtained by considering failure surface profiles. Test results ended with the following conclusions:

a) Uplift capacity of single piles increase by 20 to 25% for an increase of the embedment ratio from 24 to 30 and from 30 to 36.
b) Uplift capacity increases with the increase of pile spacing if the embedment length and group arrangement are considered to be constant. On the other hand, for a particular pile spacing and group arrangement, the uplift capacity increases with the increase of embedment length.
c) Uplift capacity increases with the increase of the number of piles in a group.
d) Uplift capacity increases in a triangular arrangement more than in a 1x3 arrangement.
e) For a certain group arrangement, group efficiency $\eta \alpha d/l$ and $\eta \alpha s/d$.

f) For a single pile, the ultimate uplift load occurred at an axial movement of 20 to 30% of pile diameter. For a group, it occurred at 5 to 15% of group size.

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

This study has some shortcomings that can be summarized as follows:

a) For all the 57 tests carried out by the author, the pile diameter was the same, which means that the effect of varying this parameter was not considered.

b) Order of pile driving was not taken into consideration for the evaluation of ultimate load transferred to piles.

c) Cap-pile-soil interaction effects were not considered for the evaluation of uplift capacities for both single piles and pile groups.

Ismael (2001) proposed a program of field tests on bored piles and pile groups under axial loading in a cemented sand soil. These tests consisted of a compression test on 2 identical piles, a tension test on 2 other single piles and a compression test on 2 pile groups of 5 piles each with a spacing of two and three pile diameters respectively. Analysis on test results was carried out and came to the following results:

a) Group settlement is larger than the settlement of single piles in the elastic range.

b) Elastic settlement increases with the width of pile group Bg.

c) At larger loads, the settlement of single piles exceeds that of the groups as it approaches failure at relatively smaller loads.

d) At failure, single piles in compression resisted most of the applied load by shaft resistance.

e) No difference was reported between tension and compression axial load distribution for single piles as well as for pile groups.

f) Shaft resistance component increases due to group effect; this fact explains the high values of group efficiencies for piles spaced at two and three pile diameters.

g) The group factor (ratio of the settlement of pile group to the settlement of single pile) increases as the pile group width increases.

Based on this study, the following shortcomings are listed:

a) This study was limited to 5 piles in-group and it did not take into consideration the effects of increasing or decreasing the number of piles on their group efficiency (η).

b) Neither the cap-pile-soil interaction effects nor the variation in the pile length-diameter (L/D) ratio was considered for the group efficiency evaluation.

2.2.2 THEORETICAL MODELS

Terzaghi and Peck (1948) proposed a theoretical model in order to estimate the ultimate capacity of pile groups in case of block failure. This equation can be expressed as follows:

$$Q_g = q_b B L + D_f (2B + 2L) s \qquad (2.16)$$

Where,

Q_g = ultimate capacity of pile group

q_b = Ultimate capacity per unit area of a rectangular loaded area with dimensions $BxLxD_f$.

B = width of pile group.

L = Length of pile group.

s = average shearing resistance of soil per unit area.

It has been demonstrated that block failure does not occur unless the number of piles in-group is relatively large and unless they are embedded in silt or soft clay. In addition, a pile group can be considered safe against such kind of failure if the total design load, which can be computed as the number of piles multiplied by the ultimate bearing capacity per pile, does not exceed $Q_g / 3$.

According to Chellis (1961) one of the earliest group efficiency formulas used to asses the reduction in the load bearing capacities of piles due to group action is Converse-Labarre (Bolin 1941) formula contained in the Uniform Building Code of the International Conference Of Building Officials And Specifications of the American Association of State Highway Officials. This formula can be expressed as follows:

$$\eta = 1 - \frac{\phi\,[(n-1)\,m + (m-1)\,n]}{90\,n\,m} \qquad (2.17)$$

where,

m = number of rows

n = number of piles/ row

Φ = ATAN D/S

S = pile spacing

D = pile diameter

Based on the above-mentioned expression, it can be clearly noticed that Converse-Labarre formula considers the group action based on the relative spacing between piles and the pile diameter regardless neither the pile length nor the soil properties variation with depth.

A more recent variation of equation 4 is the so-called "Los Angeles Group Action Method" this method can be expressed as follows:

$$\eta = 1 - \frac{D\,[m\,(n-1) + n\,(m-1) + (n-1)(m-1)\sqrt{2}]}{\pi\,\mathrm{S}\,m\,n} \qquad (2.18)$$

Another method used to assess the reduction of load bearing capacity of piles in-group is that proposed by Feld (1943). This method consists of reducing the load bearing capacity of each pile in the group by one- sixteenth in order to consider the effect of each neighbouring pile in the same row or at the diagonal of the pile in question. Based on this method, different loads will be assigned to the piles, while by using the previously mentioned equations all the piles will have the same loads as their load bearing capacities will be reduced by the same value.

Based on the assumptions of Converse-Labarre theory, an empirical equation was proposed by Seiler & Kenney (1944) in view of the satisfactory agreement that was found between model tests on pile groups and this theory. This equation can be expressed as follows:

$$\eta = 1 - \frac{11\,\mathrm{S}\,(n+m-2)}{7\,(S^2-1)(n+m-1)} + \frac{0.3}{n+m} \qquad (2.19)$$

where S is in ft.

Sayed and Bakeer (1992) proposed a new formula for the evaluation of group efficiency of axially loaded pile groups. This formula was based on the premise that the group effect should be taken into consideration for the shaft component only.

$$\eta = 1 - (1 - \eta_s^{'} \cdot K) \cdot \rho \qquad (2.20)$$

where,

η = group efficiency

ρ = friction factor.

K= interaction factor

$\acute{\eta}_s$ = geometric efficiency = $P_g / \Sigma P_p$

P_g = perimeter of the pile group

ΣP_p = sum of the perimeters of the individual piles.

$$\eta_s^{'} = 2 * \frac{[(n-1)\,\mathrm{S} + D] + [(m-1)\,\mathrm{S} + D]}{\pi\, n\, m\, \mathrm{D}} \qquad (2.21)$$

It should be noted that the previous equation takes into consideration several parameters that were not considered in earlier studies. These parameters are: the geometric efficiency ηs, the group interaction factor K and the friction effect ρ. For the geometric efficiency ηs, this parameter is responsible for the planar geometry effect the pile group, the friction factor ρ introduces the effect of the three dimensional characteristics of the proposed formula and takes into account the pile length as well as the properties of the embedding soil. Its values usually range from 0 for end bearing piles to 1 for friction piles. The group interaction factor is a function of the method of installation, pile spacing and type of embedding soil. Its values range from 0 to 1 according to the relative density of the sand or the consistency of the clay. The new formula can assess both the short term and the long-term group efficiencies for soils that may experience shear strength changes through time. This assessment can be accomplished through the all driving analyser or through pile dynamic measurements made at the end of driving which are used to measure changes in the friction factor ρ through time. In order to validate the predictive capabilities of the proposed formula, a full-scale test data used by Kishida (1967) were used for a 2x2 and 3x3 pile groups. Computed values of ηg using the proposed formula were compared against test results and good agreement between both values was found.

The new proposed formula has the advantage over the earliest formulas in that it takes into account the three-dimensional geometry rather than its planar geometry only. Also, it introduces the new friction factor (ρ), which accounts for the soil characteristics and the

depth of embedment, but in practice, this new factor can only be determined for different types of soil but for different depths of embedment no definite values were recommended. The interaction factor (K) proposed by the author is limited to driven and jacked piles, its values may differ significantly for bored piles. Also, it cannot be determined for different pile cap conditions (cap in direct contact with soil or freestanding).

Das (1998) proposed an empirical model in order to calculate the group efficiency of frictional piles subjected to axial loads. In this model the pile group acts as a block and the group efficiency is calculated as follows:

$$\eta = \frac{2\,S\,(n+m-2)+4\,D}{n\,m} \qquad (2.22)$$

where,

n = number of piles per row.

m = number of piles per column.

S = pile spacing.

D = pile diameter

The present proposed empirical model for the calculation of the group efficiency takes into consideration only the planar geometry of the group (pile spacing, pile diameter, and number of piles). Therefore, it carries the disadvantage of not considering other parameters like: cap condition, soil condition, type of loading, and pile-length to diameter ratio (L/D).

2.3 DISCUSSION

Based on the above literature review, it can be concluded that the mathematical models used for the calculation of the pile group efficiency (η) are lacking accuracy and consistency. This is evident in the conflict among the formulas proposed by Bolin (1941), Sayed and Bakeer (1992), and Liu et al (1994). For model tests, many researchers overlooked some of the parameters that are believed to have a great influence on the pile group efficiency (η). Examples of these parameters are: pile spacing, pile length, group arrangement, cap condition (cap in direct contact with soil or freestanding), method of installation, order of pile driving, and soil characteristics.

Therefore, the purpose of this research is to make the best use of all the data available from field and laboratory tests in building a model that calculates pile group efficiency (η) based on the previously mentioned parameters. This model will provide an accurate and quick estimate for the group efficiency of piles subjected to axial loading in cohesionless soils.

CHAPTER 3

ANALYSIS

3.1 GENERAL

Full-scale field tests yield 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. Therefore, the chapter presents the analysis of existing design theories for pile group efficiency using the results of several laboratory tests and few field tests that were available in the literature. The objective of this analysis is to evaluate the differences between these design theories and the measured data and to quantify their relative reliability. Also, this chapter presents the analysis of measured data obtained from different sources in order to evaluate their completeness and consistency. This evaluation is necessary for the development of the proposed model for pile group efficiency, which is presented in the following chapter.

3.2 ANALYSIS OF DESIGN THEORIES

In order to evaluate the existing design theories for pile group efficiency (η), adequate experimental data are required. In this section, the data obtained from the tests conducted by Mukherjee (1996), Vesic (1967), Tejchman (1976), Chattopadhyay (1994), Garg (1979), Kishida and Meyerhof (1965), Kezdi (1957), and Liu et al (1985) are utilized in

comparing the following design theories: Das (1998), Converse-Labarre (1941), Los Angeles Group Action (1944), Seiler and Keeney (1944), and Sayed and Bakeer (1992), which were presented in Chapter 2. These comparisons are tabulated in Appendix A and discussed in details as follows:

3.2.1 Mukherjee Experiment

Mukherjee (1996) conducted a laboratory test on different pile groups that have pile length (L) equal to 0.6, 0.75, and 0.9 m, pile spacing (S) equal to 0.075, 0.105, and 0.125 m, and pile arrangement equal to 1x2, 1x3, 2x2, 2x3, and 3x3. The results of comparing the experimental group efficiencies and those calculated using different design theories are listed in Table A.1 and plotted in Figure 3.1 (a) to 3.1 (i). Based on these results, the following observations were made:

a) The group efficiency (η) reaches its maximum values when calculated using equations 2.21 & 2.22 (Sayed & Bakeer 1992 and Das 1998) for all pile lengths as well as for all group arrangements. While Converse-Labarre and Los Angeles group action theoretical models (eq.2.17 & 2.18) yield η values less than unity and better agrees with the measured ones.

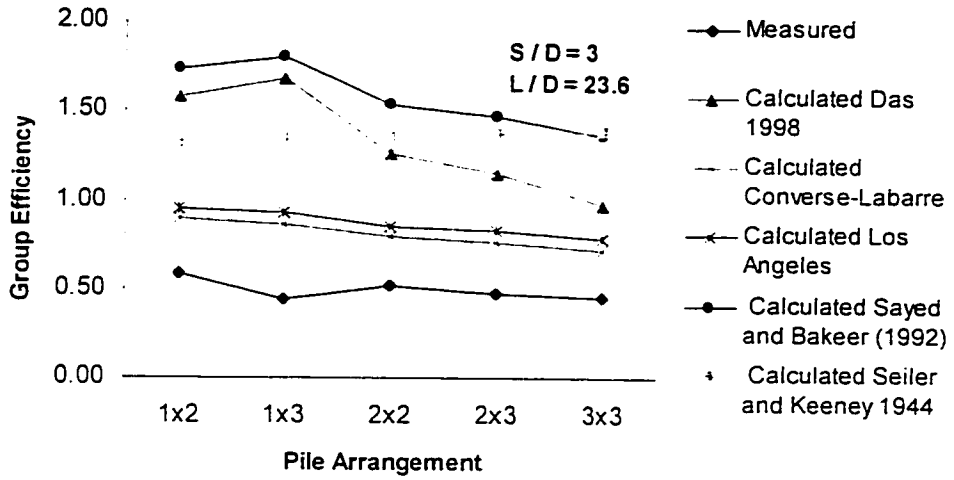
b) The group efficiency (η) calculated using all of the existing theories except Sayed and Bakeer (1992) increases by increasing the pile spacing- diameter (S/D) ratio. This may be because all of these models consider only the planar geometry of the group and does not account neither for the variation of the soil conditions nor for the pile-soil interaction effects.

c) None of the existing design theories could assess the variation in the pile length-diameter (L/D) ratio.

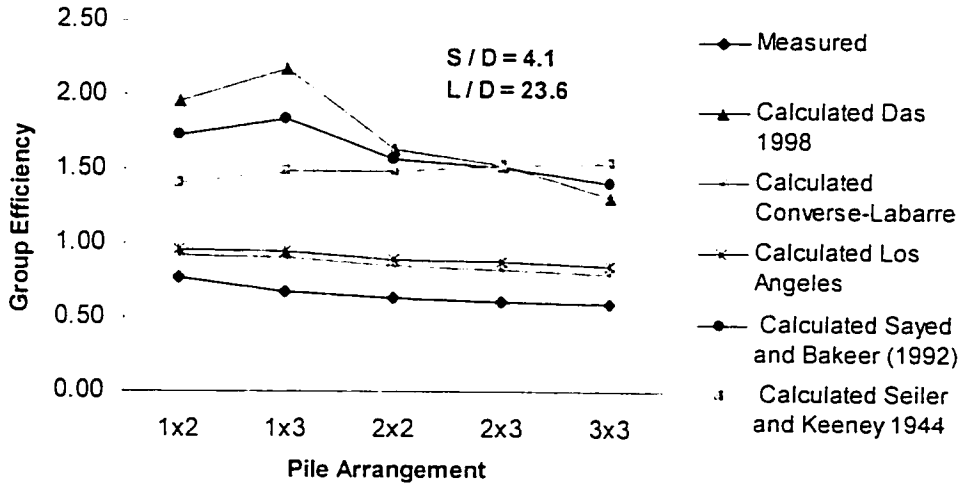
d) For uplift loading, the group efficiency (η) calculated using Sayed and Bakeer (1992) becomes closer to the measured values by increasing the pile spacing as well as the number of piles.

e) The discrepancies between the measured and calculated group efficiency values using Sayed and Bakeer (1992) model are very large in case of line groups (1x2 and 1x3). This may be because this efficiency model was mainly verified for square and rectangular arrays so its results in case of line groups are not reliable.

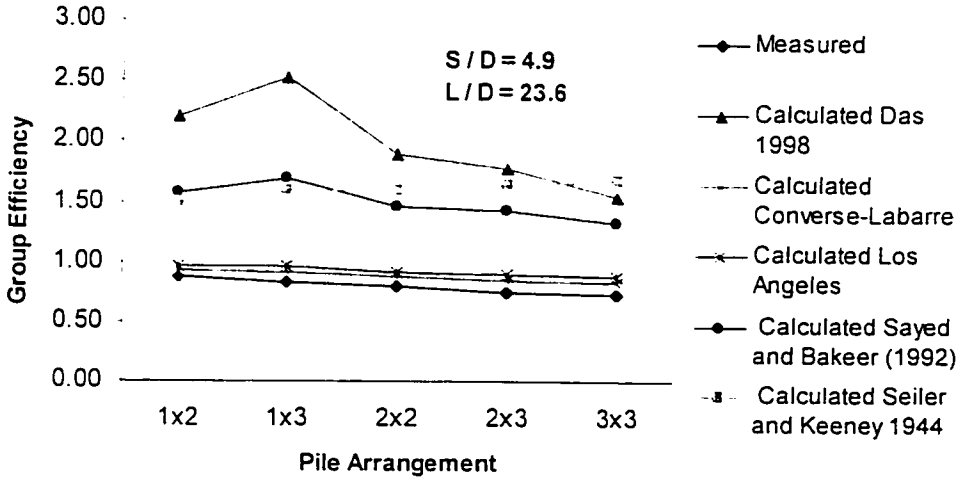
f) In case of uplift loading, for the same pile length-diameter (L/D) ratio and (S/D) ratio, the group efficiency (η) calculated using all of the existing design theories except Seiler and Kenney (1944) decreases by increasing the number of piles. While in case of compression loading the group efficiency (η) calculated using Sayed and Bakeer (1992) increases by increasing the number of piles as will be seen in the following tests.


2.00
1.50
1.00
0.50
0.00
Group Efficiency
S / D = 3
L / D = 23.6
1x2
1x3
2x2
2x3
3x3
Pile Arrangement
Measured
Calculated Das 1998
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

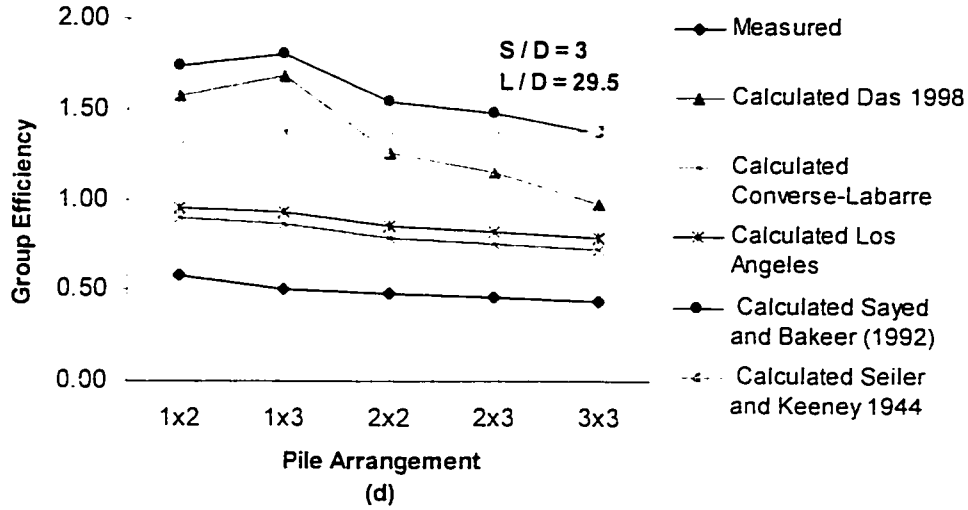
(a)


2.50
2.00
1.50
1.00
0.50
0.00
Group Efficiency
S / D = 4.1
L / D = 23.6
1x2
1x3
2x2
2x3
3x3
Pile Arrangement
Measured
Calculated Das 1998
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

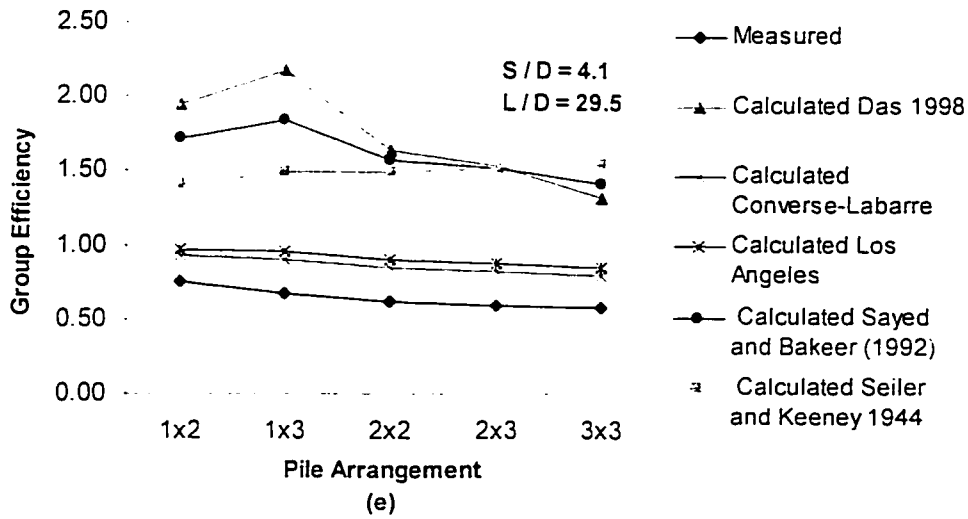
(b)


3.00
2.50
2.00
1.50
1.00
0.50
0.00
Group Efficiency
S / D = 4.9
L / D = 23.6
1x2
1x3
2x2
2x3
3x3
Pile Arrangement
Measured
Calculated Das 1998
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

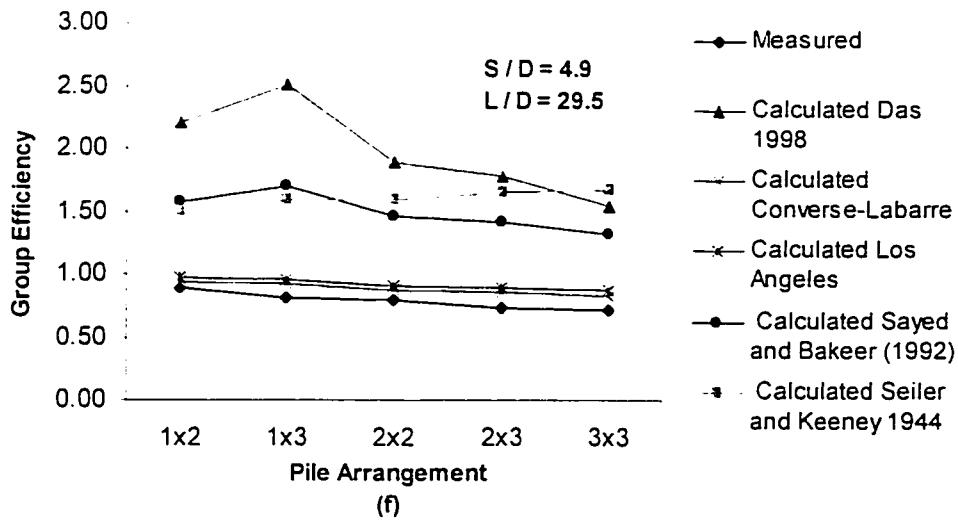
(c)


2.00
1.50
1.00
0.50
0.00
Group Efficiency
S / D = 3
L / D = 29.5
1x2
1x3
2x2
2x3
3x3
Pile Arrangement
Measured
Calculated Das 1998
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

(d)


2.50
2.00
1.50
1.00
0.50
0.00
Group Efficiency
S / D = 4.1
L / D = 29.5
1x2
1x3
2x2
2x3
3x3
Pile Arrangement
Measured
Calculated Das 1998
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

(e)


3.00
2.50
2.00
1.50
1.00
0.50
0.00
Group Efficiency
S / D = 4.9
L / D = 29.5
1x2
1x3
2x2
2x3
3x3
Pile Arrangement
Measured
Calculated Das 1998
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

(f)

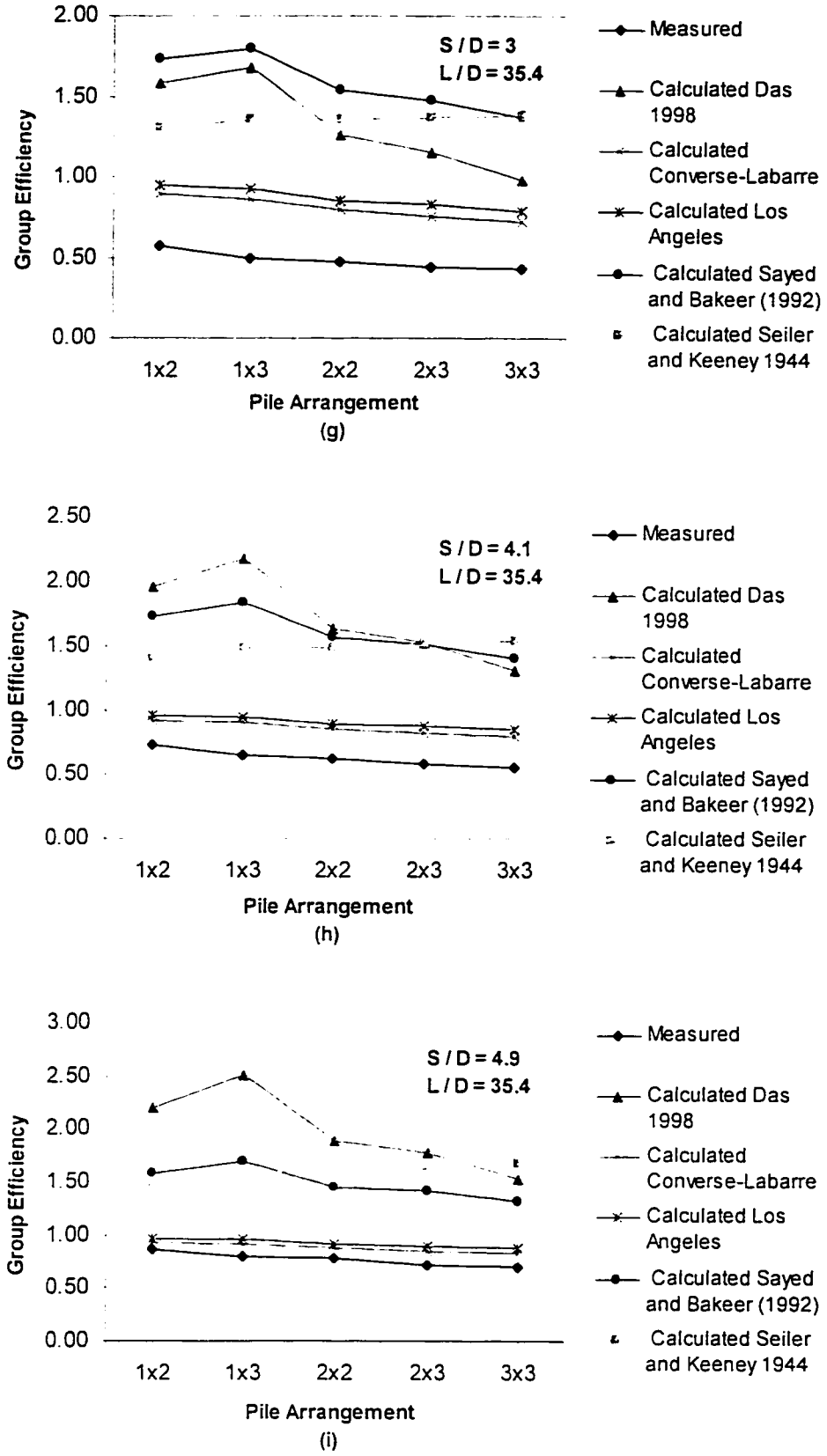


Figure 3.1: Comparing the results of Mukherjee experiment with different design theories

3.2.2 Vesic Experiment

Vesic (1969) conducted high quality model tests on different pile groups in loose sand. These tests were conducted for square pile arrangements (2x2 and 3x3). pile spacing - diameter ratio (S/D) = 2, 3, 4, 6, and unit weight of soil ranging between 14.5 and 15.2 kN/m^3. The results of comparing the measured group efficiencies and those calculated using different design theories are listed in Table A.2 and plotted in Figure 3.2 (a) and (b). Based on these results, the following observations were made:

a) For 2x2 group arrangement. (L/D) =15.2. and unit weight of soil (γ) =14.8 KN/m^3. the group efficiency (η) calculated using Sayed and Bakeer (1992) best agrees with the experimental values. for the rest of the theoretical design models. their exists large discrepancies between the measured and calculated values. are found.

b) None of the existing design theories except Sayed and Bakeer (1992) could assess the variation in the unit weight of soil (γ). This may be this model takes into account the variation in soil conditions while the rest of the design theories consider only the planar geometry of the group.

g) The group efficiency (η) calculated using all of the existing theories except Sayed and Bakeer (1992) increases by increasing the pile spacing- diameter (s/D) ratio. This may be because all of these models consider only the planar geometry of the group and does not account neither for the variation of the soil conditions nor for the pile-soil interaction effects.

c) At S/D $\geq$ 3, the calculated group efficiency using Seiler and Kenney (1944) yields inaccurate results.

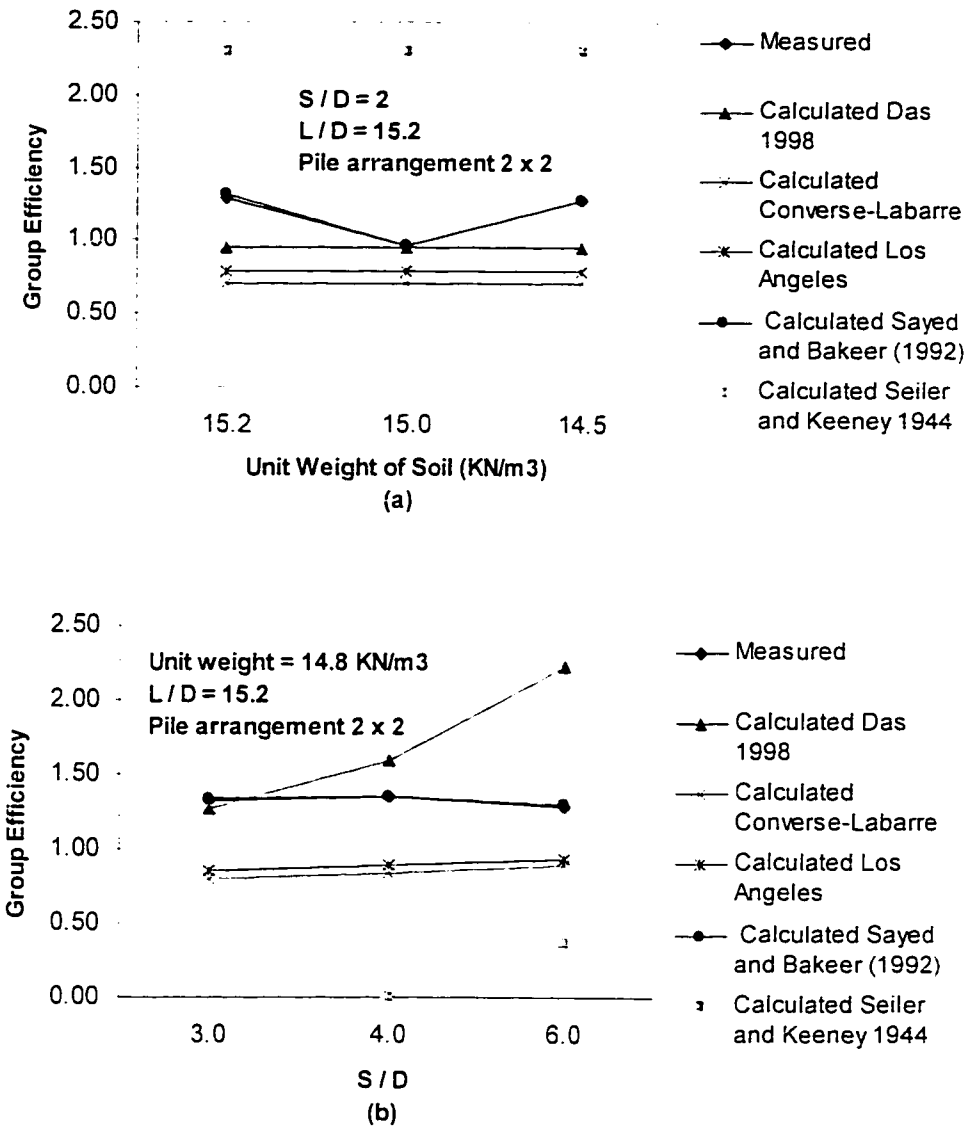


Figure 3.2: Comparing the results of Vesic experiment with different design theories

3.2.3 Tejchman Experiment

Tejchman (1973) conducted a number of laboratory tests on pile groups in both loose and dense sands. These tests were conducted on square and rectangular arrangements of 2x2, 2x4, and 3x3. Pile groups had pile spacing to pile diameter ratios (S/D) equal to 2, 3,

4.5, and 6. The results of comparing the experimental group efficiencies based on the above-mentioned tests and those calculated using different design theories are listed in Tables A.3 and A.4 and plotted in Figures 3.3 (a) to (d) and 3.4 (a) to (d) for dense and loose sand. Based on these figures, the following observations were made:

a) In case of loose sand, the experimental group efficiency increases by decreasing the pile spacing; this may be because of the increased lateral stresses against piles as the soil becomes more compacted due to driving neighboring piles.

b) In case of dense sand, the experimental group efficiency decreases by decreasing the pile spacing this may be because of the resulting dilatancy effects.

c) For both loose and dense sand conditions, large discrepancies between the measured and calculated group efficiency values using Sayed & Bakeer 1992 were found in case of line group (1x4). This may be because this model was verified for square and rectangular arrays so that its results in case of line groups are not reliable.

d) For both loose and dense sand conditions, the group efficiency (η) calculated using all of the existing theories except Sayed and Bakeer (1992) increases by increasing the pile spacing- diameter (s/D) ratio. This may be because all of these models consider only the planar geometry of the group and does not account neither for the variation of the soil conditions nor for the pile-soil interaction effects.

e) For both loose and dense sand conditions, the group efficiency (η) calculated using both Sayed and Bakeer (1992) as well as Seiler and Kenney (1944)

increases by increasing the number of piles. While it decreases by increasing the number of piles when calculated using the rest of the design theories.

f) None of the above-mentioned empirical models except Sayed and Bakeer (1992) model could assess the difference between the loose and dense sand conditions.

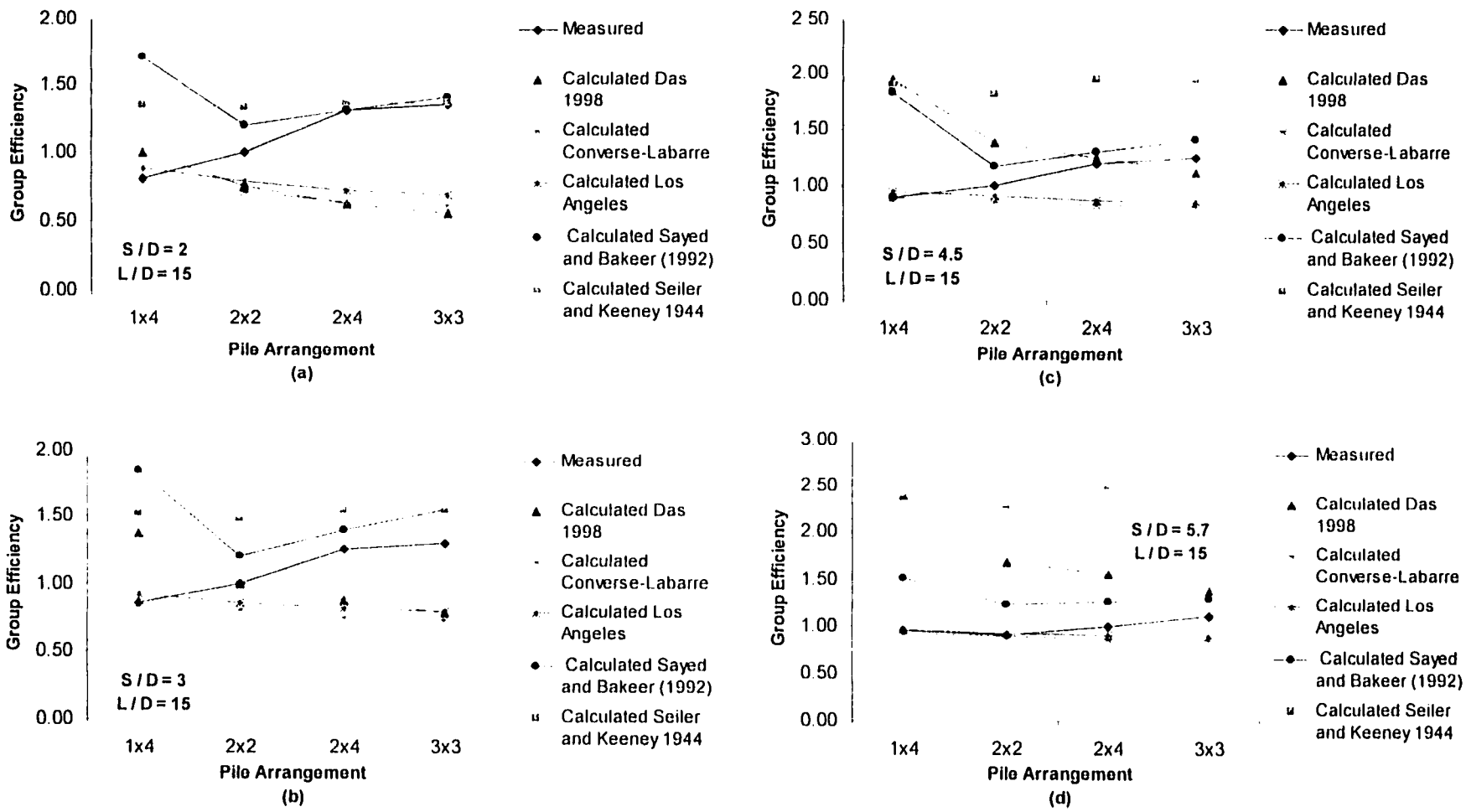


Figure 3.3: Comparing the results of Tejchman experiment for dense sand with different design theories

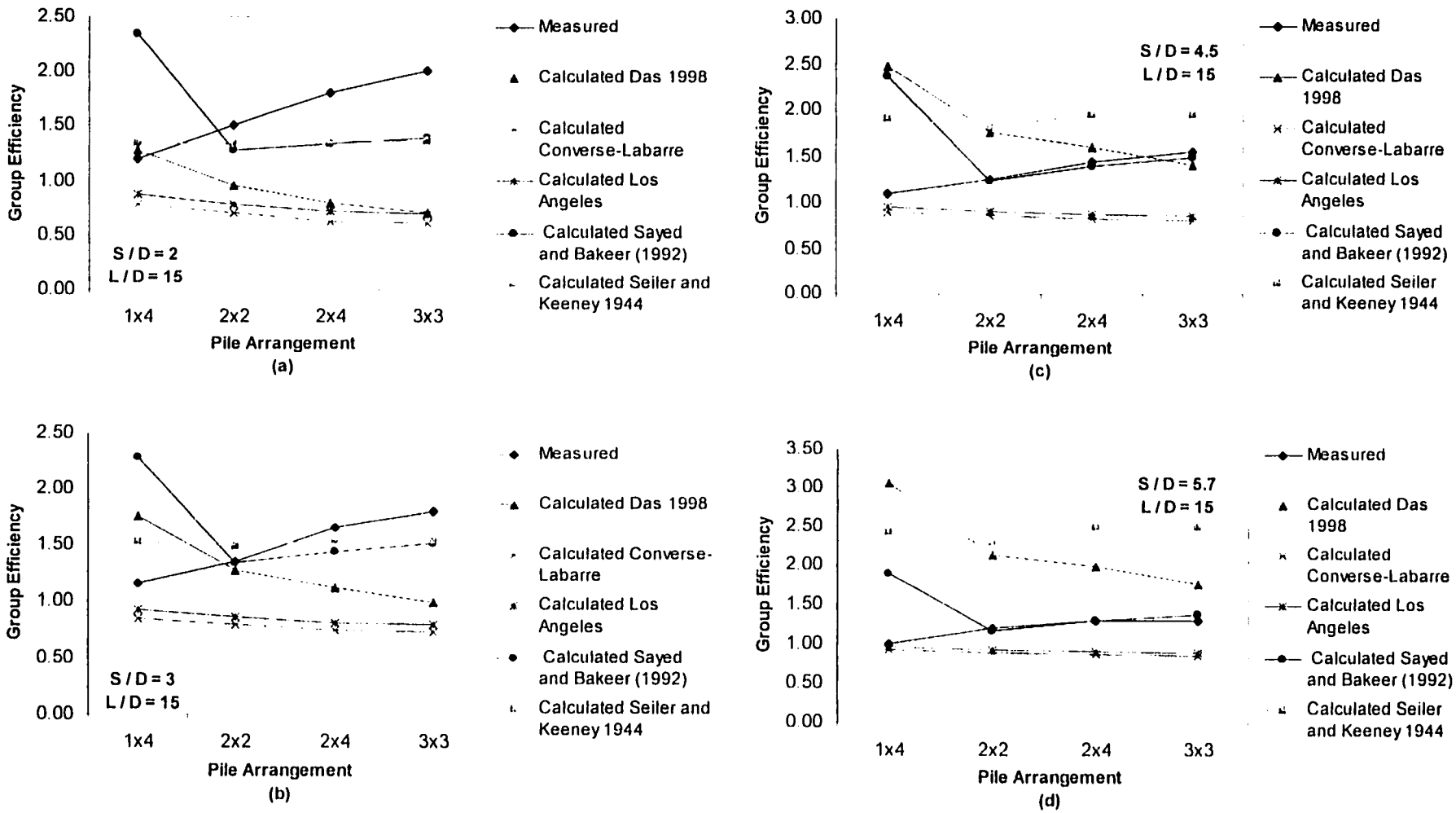


Figure 3.4: Comparing the results of Tejchman experiment for loose sand with different design theories

3.2.4 Chattopadhyay Experiment

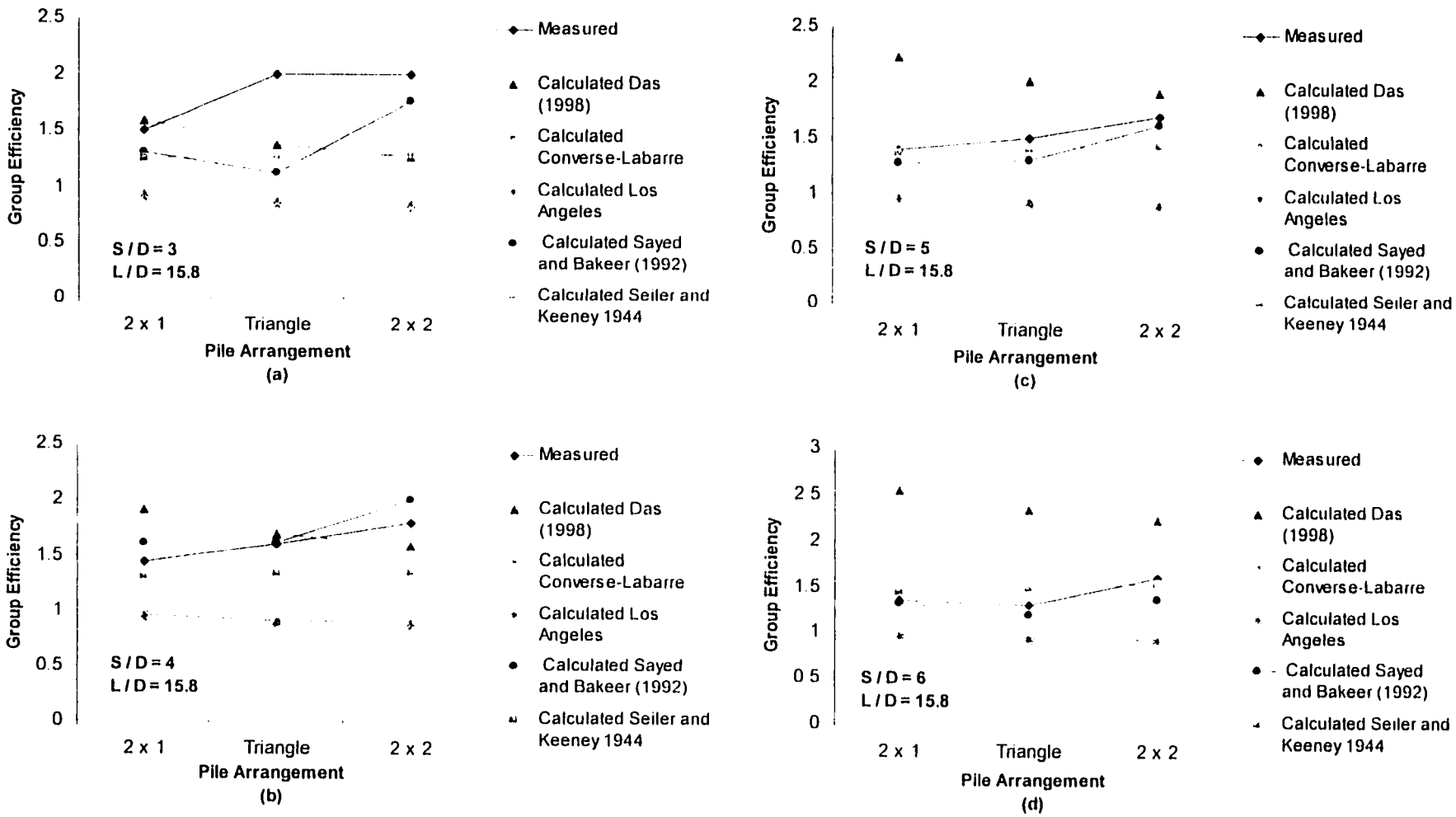
Chattopadhyay (1994) conducted several laboratory tests on pile groups driven in loose sand with three different arrangements: line (1x2), triangular (1.5x2), and square (2x2). Pile groups had different pile spacing-diameter (S/D) ratios varying between 2, 4, 5, and 6, and pile length-diameter ratio (L/D) = 15.8. The results of comparing the experimental group efficiencies based on the above-mentioned tests and those calculated using different design theories are listed in Table A.5 and plotted in Figure 3.5 (a) to (l). Based on these results, the following observations were made:

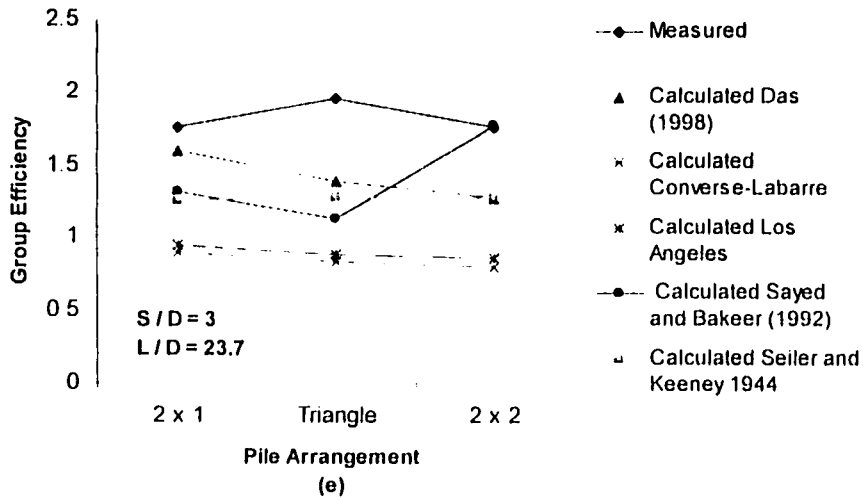
a) The experimental group efficiencies increase by decreasing the pile spacing this may be because of the increased lateral stresses against piles when closely spaced as well as the soil becomes more compacted as a result of driving neighboring piles.

b) None of the existing design theories could assess the variation in the pile length-diameter (L/D) ratio.

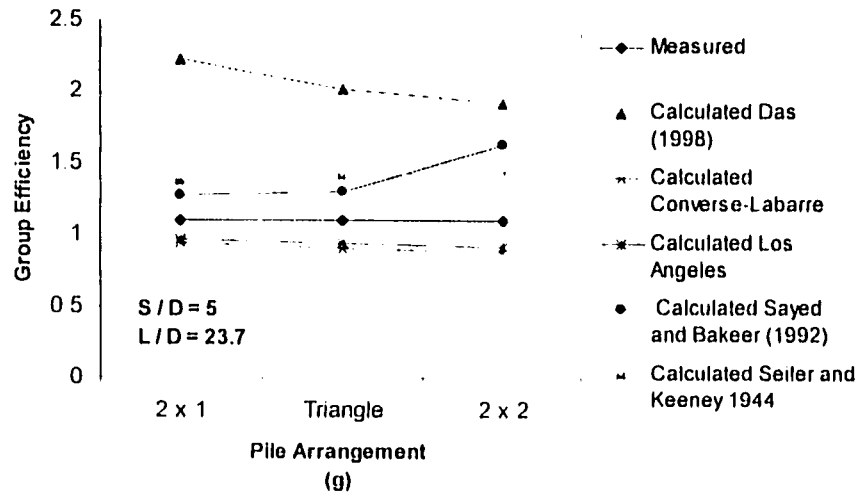
c) The group efficiency (η) calculated using Sayed and Bakeer (1992) as well as Seiler and Kenney (1944) increases by increasing the number of piles. While it decreases by increasing the number of piles when calculated using the rest of the design theories.

d) The group efficiency (η) calculated using all of the existing theories except Sayed and Bakeer (1992) increases by increasing the pile spacing- diameter (S/D) ratio. This may be because all of these models consider only the planar geometry of the group and does not account neither for the variation of the soil conditions nor for the pile-soil interaction effects.

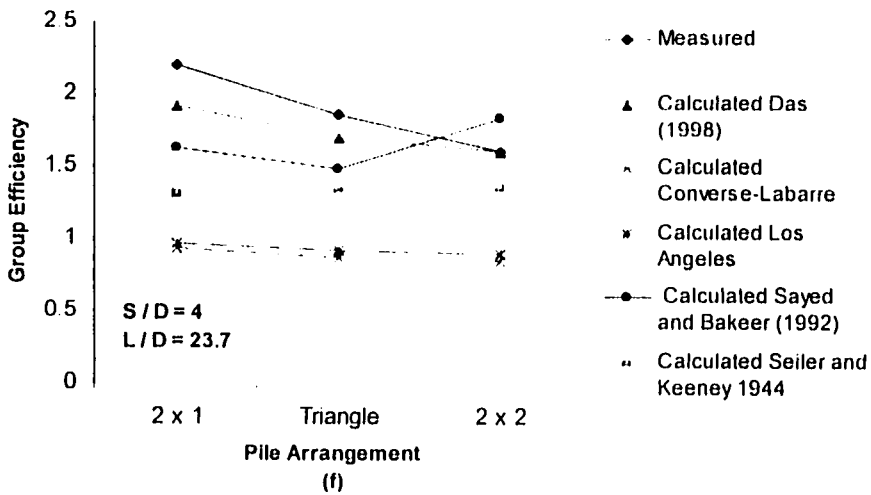

Group Efficiency
2.5
2
1.5
1
0.5
0
S / D = 3
L / D = 15.8
2 x 1
Triangle
2 x 2
Pile Arrangement
(a)
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944
S / D = 4
L / D = 15.8
(b)
S / D = 5
L / D = 15.8
(c)
3
S / D = 6
L / D = 15.8
(d)


2.5
2
1.5
1
0.5
0
Group Efficiency
S / D = 3
L / D = 23.7
2 x 1
Triangle
2 x 2
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

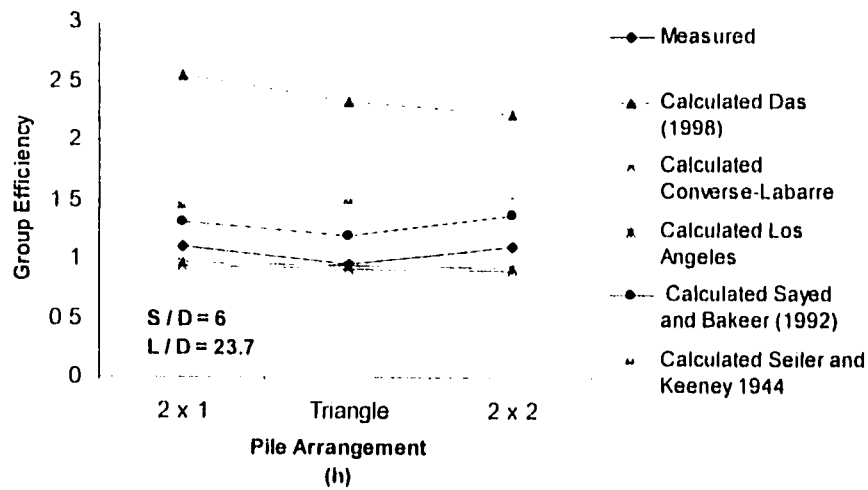
(e)


2.5
2
1.5
1
0.5
0
Group Efficiency
S / D = 5
L / D = 23.7
2 x 1
Triangle
2 x 2
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

(g)


2.5
2
1.5
1
0.5
0
Group Efficiency
S / D = 4
L / D = 23.7
2 x 1
Triangle
2 x 2
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

(f)


3
2.5
2
1.5
1
0.5
0
Group Efficiency
S / D = 6
L / D = 23.7
2 x 1
Triangle
2 x 2
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and Keeney 1944

(h)

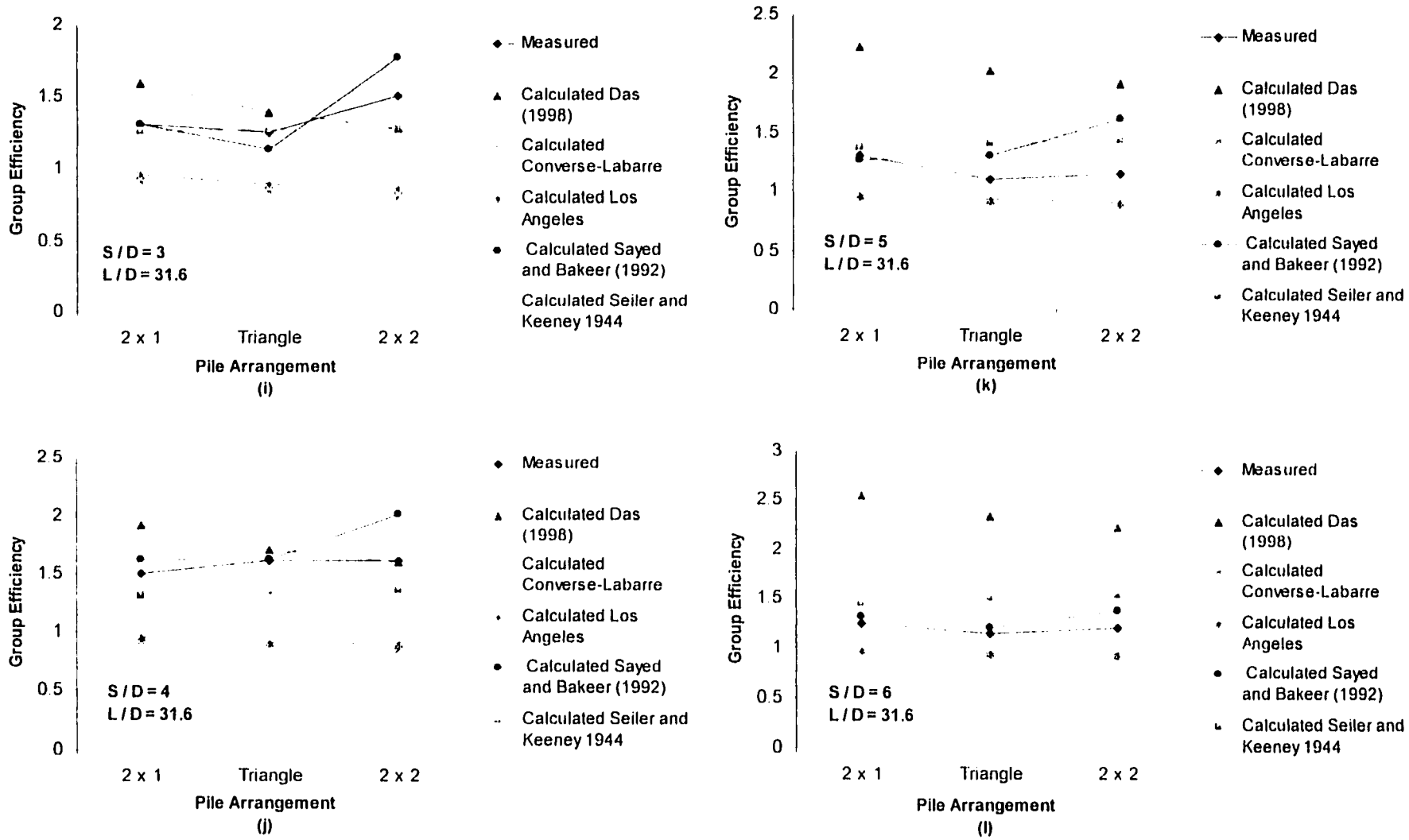


Figure 3.5: Comparing the results of Chattopadhyay experiment with different design theories

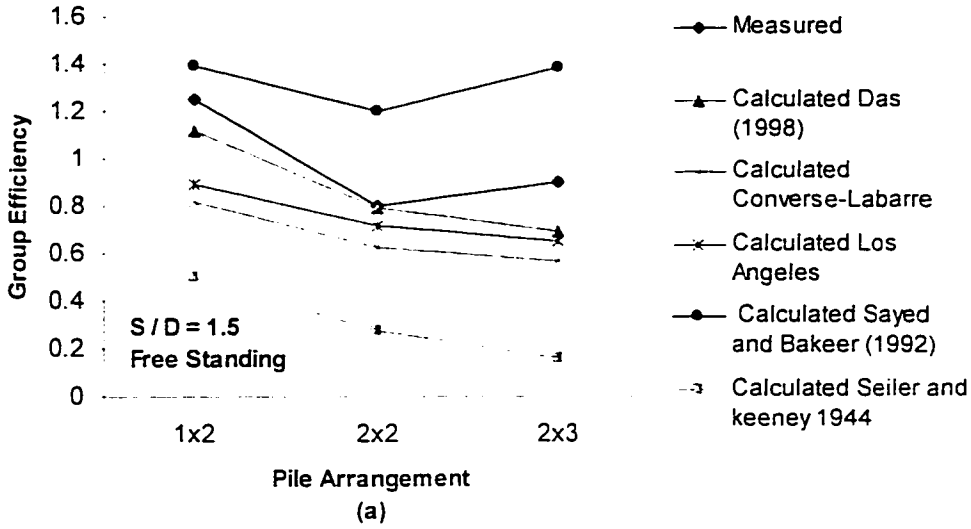
3.2.5 Garg Experiment

Garg (1979) conducted a full-scale field test on bored piles in loose sand with two cap conditions: freestanding and cap resting on soil. These piles had different pile spacing-diameter (S/D) ratios varying between 1.5, 2, and 2.5, and pile length -diameter ratio (L/D) = 20. Three pile arrangements including: 1x2, 2x2, and 2x3 were used. The results of comparing the experimental group efficiencies based on the above-mentioned tests and those calculated using different design theories are listed in Table A.6 and plotted in Figure 3.6 (a) to (f). Based on these results, the following observations were made:

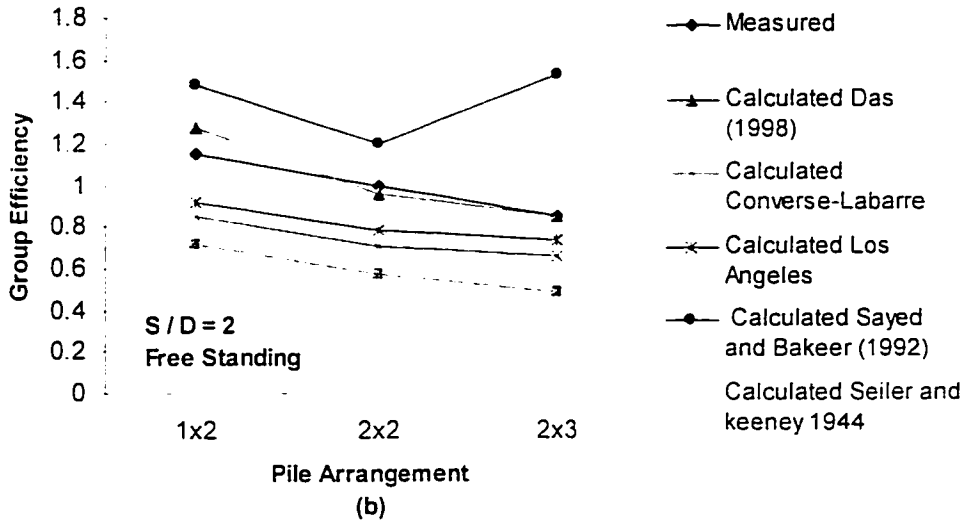
a) The pile group efficiency calculated using Converse-Labarre and Los Angeles Code theoretical models lies between lies between .56 and .94. For short-bored undereamed pile groups embedded in silty sand and in case of freestanding condition, these models can be used for predicting the group efficiency as they usually yield η values less than unity which holds true for bored piles.

b) The group efficiency calculated using Sayed and Bakeer theoretical model lies between 1.3 and 1.7. These values are always higher than the experimental values for both of the freestanding and cap resting conditions. This may be because the pile-soil interaction factor K values for bored piles differ significantly from those used for driven and jacked piles.

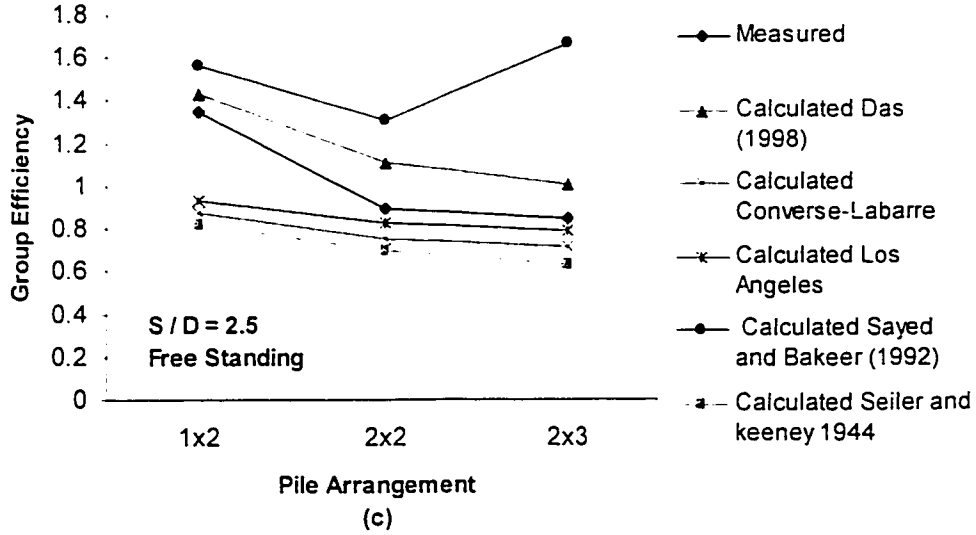
c) The group efficiency (η) calculated using all of the existing theories except Sayed and Bakeer (1992) increases by increasing the pile spacing- diameter (S/D) ratio. This may be because all of these models consider only the planar geometry of the group and does not account neither for the variation of the soil conditions nor for the pile-soil interaction effects.


1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Group Efficiency
S / D = 1.5
Free Standing
1x2
2x2
2x3
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and keeney 1944

(a)


1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Group Efficiency
S / D = 2
Free Standing
1x2
2x2
2x3
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and keeney 1944

(b)


1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Group Efficiency
S / D = 2.5
Free Standing
1x2
2x2
2x3
Pile Arrangement
Measured
Calculated Das (1998)
Calculated Converse-Labarre
Calculated Los Angeles
Calculated Sayed and Bakeer (1992)
Calculated Seiler and keeney 1944

(c)

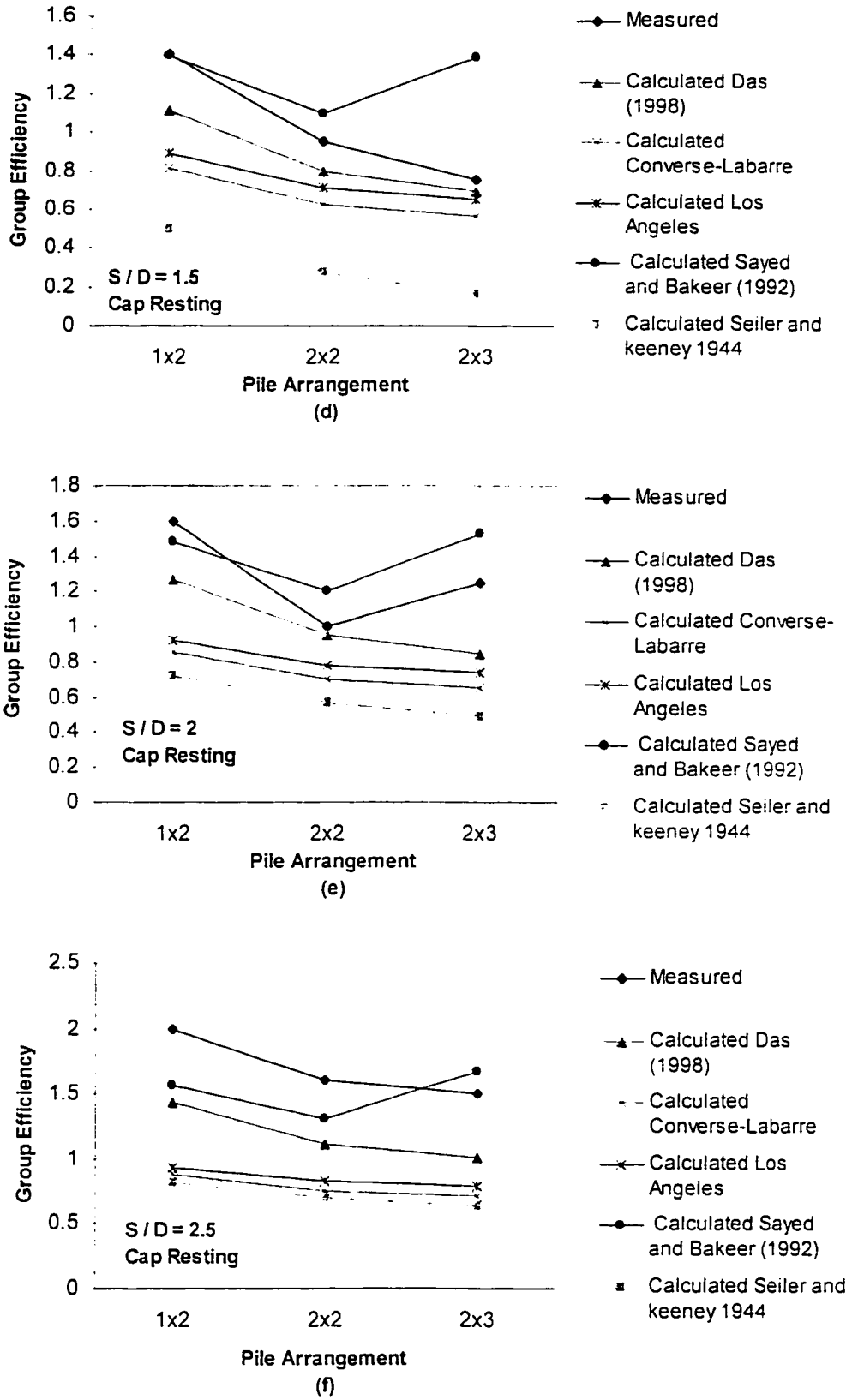


Figure 3.6: Comparing the results of Garg experiment with different design theories

d) None of the existing theoretical models could differentiate between the freestanding and cap resting conditions. This may be because based on earlier studies, it was cautioned against increasing the overall bearing capacity of the group as a result of cap rigidity due to the increased probability of erosion and loss of support, which may happen because of the settlement of the soil surrounding the cap.

e) In case of cap resting condition, the measured group efficiencies as well as the calculated ones are so much higher than in case of freestanding groups, this may be because in this case the cap contributes by increasing the overall group capacity. This is in good agreement with what was reported in the literature.

f) The group efficiency (η) calculated using Sayed and Bakeer (1992) increases by increasing the number of piles. While it decreases by increasing the number of piles when calculated using the rest of the previously mentioned design theories including Seiler and Keneey model (1944), this may be because this test was conducted for bored piles.

3.2.6 Kishida & Meyerhof Experiment

Kishida and Meyerhof (1965) conducted laboratory tests on jacked piles in loose and dense sands with two cap conditions: freestanding and cap resting on soil. Freestanding groups had a pile length-diameter ratio (L/D) = 22, while for groups with caps resting on soil, (L/D) = 24. Mainly square arrangements of 2x2 and 3x3 were used, and for all pile groups the pile spacing-diameter (S/D) ratio varied between 2, 4, and 6. The results of comparing the measured group efficiencies based on the above-mentioned tests and those

calculated using different design theories are listed in Tables A.7 and A.8 and plotted in Figures 3.7 (a) to (d) and 3.8 (a) to (d) for loose and dense sand. Based on these results. the following observations were made:

a) In case of loose sand and for freestanding groups. the experimental group efficiency increases by decreasing the pile spacing: this may be because of the increased lateral stresses against piles when closely spaced as well as the soil becomes more compacted as a result of driving neighboring piles. For cap resting condition, the opposite procedure happens.

b) In case of dense sand and for freestanding condition both of the measured and calculated group efficiency values decrease by decreasing the pile spacing. this may be because of dilatancy effects.

c) In case of dense sand and for freestanding condition. the calculated group efficiency using Converse-Labarre (Bolin 1941) and Los Angeles group action (1944) models yields the closest values to the measured ones. For cap resting condition, these values are so much lower than the measured ones: this may be because the above-mentioned models do not take into consideration the increased group capacities resulting from cap contribution.

d) None of the existing theoretical models could differentiate between the freestanding and cap resting conditions. This may be because based on earlier studies, it was cautioned against increasing the overall bearing capacity of the group as a result of cap rigidity due to the increased probability of erosion and loss of support, which may happen because of the settlement of the soil surrounding the cap.

e) The group efficiency (η) calculated using Sayed and Bakeer (1992) increases by increasing the number of piles. While it decreases by increasing the number of piles when calculated using the rest of the previously mentioned design theories.

f) None of the above-mentioned empirical models except Sayed and Bakeer model (1992) could assess the difference between the loose and dense sand conditions.

g) The calculated group efficiency (η) using all of the existing design theories including Sayed and Bakeer (1992) increases by increasing the pile spacing-diameter ratio. This may be because for this test, piles were jacked while for the other tests piles were driven or bored.

h) In case of cap resting condition, the measured group efficiencies as well as the calculated ones are so much higher than in case of freestanding groups and it reaches 6.5 in case of 3x3 group arrangement, this may be because in this case the cap contributes by increasing the overall group capacity. For that reason these data records will be disregarded from the proposed methodology which will be presented in the following chapter since it may affect the accuracy of its results.

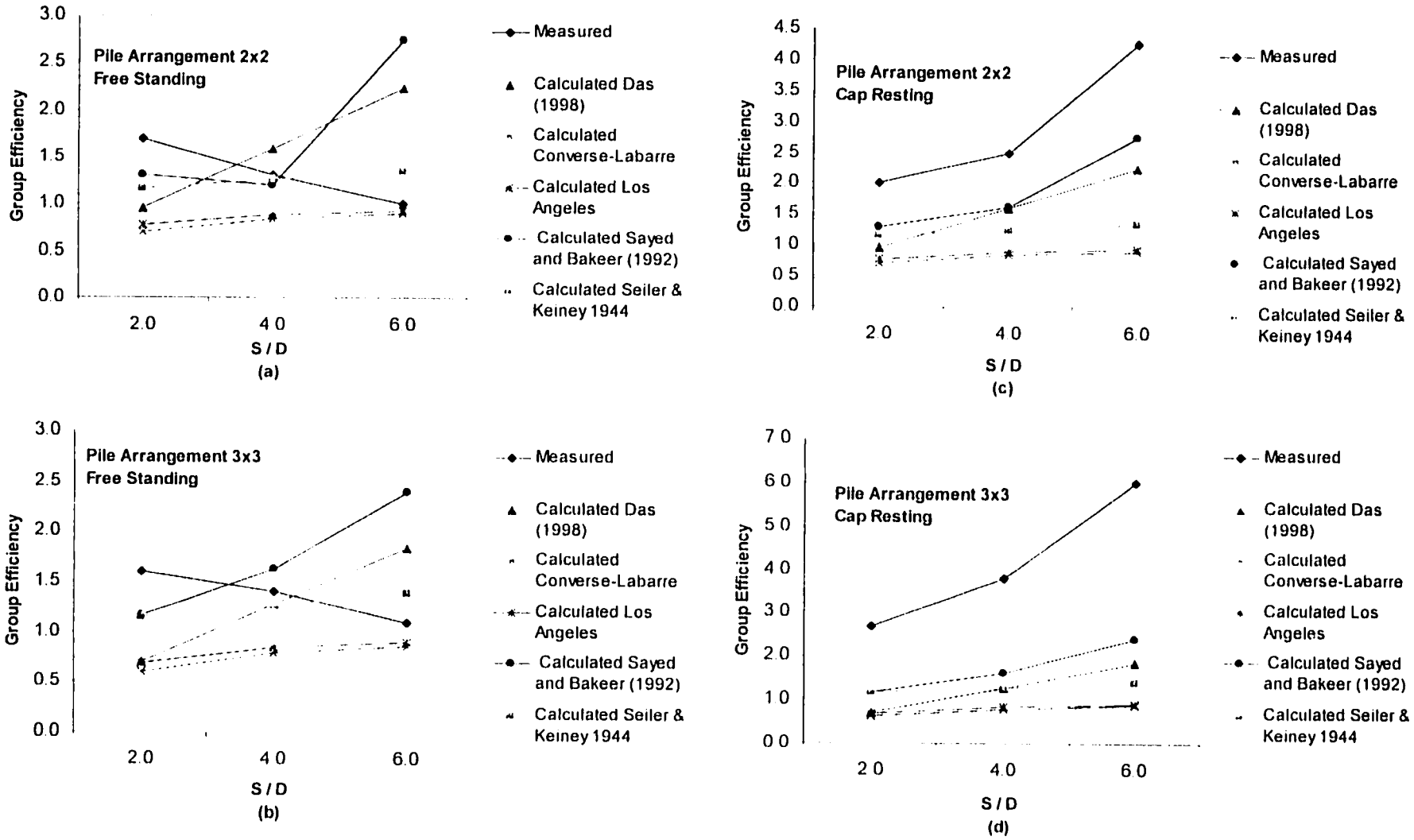


Figure 3.7: Comparing the results of Kishida & Meyerhof experiment for loose sand with different design theories

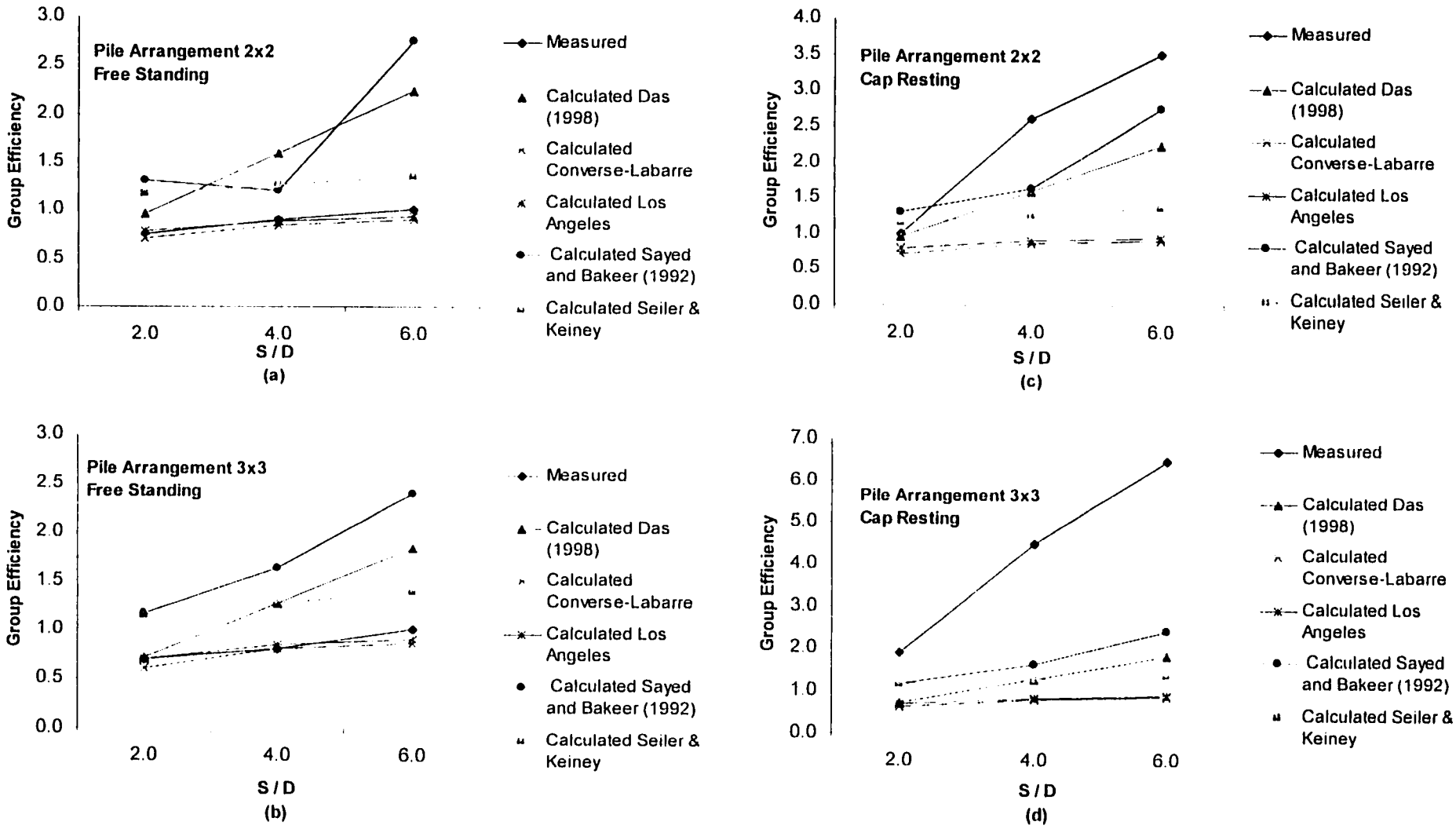


Figure 3.8: Comparing the results of Kishida & Meyerhof experiment for dense sand with different design theories

3.2.7 Kezdi experiment

Kezdi (1957) conducted a laboratory test on freestanding driven pile groups in loose sand, these piles had different pile spacing-diameter ratio (S/D) varying between 2, 3, 4, and 6, and pile length-diameter ratio (L/D) = 20. In these tests two group arrangements including: 1x4 and 2x2 were used. The results of comparing the experimental group efficiencies based on the above-mentioned tests and those calculated using different design theories are listed in Table A.9. Based on these results, the following observations were made:

a) The calculated group efficiency (η) using Seiler and Keeney model (1994) gives the closest values to the measured ones in case of $S/D < 3$. While for $S/D \geq 3$, this model could not produce group efficiency (η) values with a fair degree of accuracy.

b) The group efficiency (η) calculated using Sayed and Bakeer model (1992) gives the closest values to the measured ones for all pile spacing especially in case of square arrangements. While for line groups, large discrepancies between the experimental and calculated values were found. This may be because this model was mainly verified for square and rectangular arrays and does not account for other group arrangements.

c) The calculated group efficiency (η) using Sayed and Bakeer model (1992) increases by increasing the pile spacing (S) for $S/D \leq 4$, but for $S/D > 4$, η decreases by increasing the pile spacing (S).

d) The calculated group efficiency (η) using all of the existing design theories except Sayed and Bakeer (1992) increases by increasing the pile spacing-diameter (S/D) ratio.

e) The group efficiency (η) calculated using both Converse-Labarre and Los Angeles group models gives values less than unity for all test conditions. This is for the same reasons mentioned previously.

f) The experimental group efficiency (η) values increase by decreasing the pile spacing-diameter (S/D) ratio. This may be because of the increased lateral stresses against the soil as a result of the interference of the soil compaction zones around the piles in the group due to driving neighboring piles.

3.2.8 Liu experiment

Liu et al (1985) carried out a large-scale field test on bored pile groups in a loose sandy soil. These piles had different pile length-diameter (L/D) ratios varying between: 8, 13, 18, and 23. Also, many pile spacing-diameter (S/D) ratios including: 2, 3, 4, and 6 were used. These tests were carried out for the following group arrangements: 1x4, 1x6, 2x2, 2x4, 3x3, and 4x4, with two cap conditions: Freestanding and Cap resting. The results of comparing the experimental group efficiencies based on the above-mentioned tests and those calculated using different design theories are listed in Table A.10. Based on these results, the following observations were made:

a) For (L/D) =18 and 3x3 group arrangement, The calculated group efficiency (η) using all of the existing design theories except Sayed and Bakeer (1992) increases by increasing the pile spacing-diameter (S/D) ratio.

b) None of the existing design theories could assess the difference between the freestanding and cap resting conditions.

c) None of the existing design theories could assess the variation in the pile length-diameter (L/D) ratio.

d) The experimental group efficiency values decrease by increasing (S/D) ratio in case of freestanding condition. while in case of cap resting condition: these values increase by increasing (S/D) ratio.

e) In case of 1x4 and 1x6 line groups. large discrepancies were found between the experimental and calculated group efficiency values using Sayed and Bakeer model (1992). This may be because this model was mainly verified for square and rectangular arrays.

3.2.9 Discussion

Based on the comparisons presented above. it can be concluded that:

a) The empirical model proposed by Sayed and Bakeer (1992) best agrees with the experimental tests results incase of square or rectangular arrangements. But. for line groups, large discrepancies between model and experimental results were found. This may be because this model was mainly verified for square and rectangular arrays.

b) In case of dense sand and for freestanding condition. the empirical model proposed by Sayed and Bakeer (1992) gives closer values to the experimental results especially by increasing the number of piles. While in case of loose sand and for cap resting condition. the calculated group efficiency using the above-

mentioned model becomes closer to the measured ones by decreasing the number of piles. This model yields group efficiency values between 1.2 and 1.7 for both loose and dense sand conditions.

c) The empirical model proposed by Sayed and Bakeer (1992) carries the advantage over the rest of the empirical models in that it considers the three dimensional geometry of the group as well as the variation in the soil conditions and the pile-soil-interaction effects expressed by the coefficient K. But it is limited to driven and jacked piles and cannot be applied for bored piles.

d) The empirical models proposed by Converse-Labarre (1941) and Los Angeles group yield group efficiency values close to each other and always less than unity for any number of piles as well as for any pile spacing-diameter (S/D) ratio. These models best fit bored pile groups, while in case of driven piles, the above-mentioned models yield much lower group efficiency (η) values than the experimental results.

e) None of the existing design theories could assess the difference between the freestanding and cap resting conditions. This may be because based on earlier studies, it was cautioned against increasing the overall bearing capacity of the group as a result of cap rigidity due to the increased probability of erosion and loss of support, which may happen because of the settlement of the soil surrounding the cap.

f) None of the existing design theories except Sayed and Bakeer model (1992) could assess the difference between the loose and dense sand conditions.

g) None of the existing design theories could assess the variation in the pile length-diameter (L/D) ratio.

h) The group efficiency calculated using the empirical model proposed by Seiler and Keeney (1944) increases by increasing the pile spacing-diameter (S/D) ratio for all group arrangements. But, this model could not predict the group efficiency (η) values to a fair degree of accuracy for S/D ≥ 3.

i) The group efficiency (η) calculated using all of the existing design theories except Sayed and Bakeer (1992) increases by increasing the pile spacing-diameter (S/D) ratio. This may be because all of these models consider only the planar geometry of the group and does not account neither for the variation of the soil conditions nor for the pile-soil interaction effects.

j) The calculated group efficiency (η) using Sayed and Bakeer model (1992) increases by increasing the number of piles in case of compression loading, while for uplift loading, η decreases by increasing the number of piles.

k) The calculated group efficiency (η) using Converse-Labarre model (1941), Los Angeles model, and Das model (1998) decreases by increasing the number of piles (N).

l) In general, the calculated group efficiency (η) using Seiler and Keeney model (1944) increases by increasing the number of piles (N). But, at S/D≥ 3, this model yields inaccurate results.

m) In case of cap resting condition, the measured group efficiencies as well as the calculated ones are so much higher than in case of freestanding groups, this may

be because in this case the cap contributes by increasing the overall group capacity. This is in good agreement with what was reported in the literature.

n) The data records resulting from Kishida and Meyerhof model (1965) will be removed from the proposed network proposed in the following chapter since it yield very high group efficiency values that are inconsistent with the other tests results. and consequently may lead to inaccurate results.

3.3 ANALYSIS OF EXPERIMENTAL DATA

In order to explore the quality of the experimental data obtained from the previous experiments, a statistical analysis and a parametric study were carried out. The objective of the statistical analysis is to evaluate the associations between the different parameters and the pile group efficiency. These associations assist in determining whether the data set is inconsistent and/or missing some important parameters. Also. a parametric study is carried out to verify the effect of some parameters on the pile group efficiency when all the other governing parameters are constant.

In general, there are two different statistical methods that can be used to assess the association between two variables according to their types (Moore and McCabe 1993): measuring the coefficient of correlation and performing the analysis of variance (ANOVA) tests. The coefficient of correlation measures the strength of the linear association between two quantitative variables. This coefficient takes a value between −1 and +1. It is negative if one variable tends to increase as the other variable decreases. and positive if the two variables tend to increase or decrease together. The analysis of

variance (ANOVA) test compares several population means and consequently measures the association between one quantitative variable and another qualitative variable that is used to split the quantitative variable into several populations. This test has a null hypothesis "Ho" that the population means are all equal and an alternative hypothesis "Ha" that at least one population mean is different. The correlation analysis and the ANOVA tests were carried out using the Minitab Statistical Software Release 12 and discussed below.

3.3.1 Correlation Analysis

The records used in this analysis, which counts for 176 records, are presented in Table B.1 in Appendix B. The parameters involved in this analysis were: pile group efficiency (η), unit weight of soil (γ), soil friction angle (φ), number of piles (N), pile length-diameter ratio (L/D), and pile spacing-diameter ratio (S/D). Table 3.1 shows the developed correlation matrix (only the lower triangle is shown because of symmetry), which displays the calculated coefficient of correlation (shown in the top of each cell) between each pair of quantitative parameters. This correlation matrix also displays p-values (shown below the coefficients of correlation) for the hypothesis test of the coefficient of correlation being zero. This hypothesis test is a two-tailed test that has a null hypothesis (Ho: $r = 0$) and an alternative hypothesis (Ha: $r \neq 0$), where "r" is the correlation coefficient. Using a level of significance $\alpha = 0.05$, the common practice in hypothesis tests (Sinich 1994), all coefficients of correlation that have p-value greater than or equal 0.05 are considered insignificant.

	φ	γ	L/D	S/D	N
γ	**0.80** 0.00				
L/D	**0.32** 0.00	**0.06** 0.45			
S/D	**0.22** 0.00	**0.10** 0.19	**0.12** 0.12		
N	**-0.04** 0.64	**0.18** 0.02	**-0.23** 0.00	**-0.06** 0.47	
η	**-0.45** 0.00	**-0.48** 0.00	**-0.50** 0.00	**-0.13** 0.09	**-0.04** 0.60

Table 3.1: Correlation matrix

The last row in the above-mentioned correlation matrix contains the associations of all quantitative variables with the response variable "η". In this row, only the association with "L/D", "φ" and "γ" is considered significant, while the associations with "N" and "S/D" are considered insignificant. The insignificance of "S/D" is contrary to what was reported in the literature as it was noted by many researchers that the pile spacing-diameter ratio (S/D) has considerable impacts on the pile group efficiency. The reason that the correlation matrix shows this insignificance may be due to the fact that the correlation analysis measures the association between only one independent variable and the response variable regardless of the other independent variables. This sometimes underestimates the effect of the independent variable, since this variable may have a local effect that changes when the values of the other variables change. Therefore, a parametric study will follow this analysis to demonstrate the exact effect of "S/D" on the pile group

efficiency (η), while other parameters, such as soil condition and pile arrangement, will be used in the ANOVA test to consider the effect of "φ", "γ", and "N" in a different way.

The highest coefficient of correlation in the above matrix is between "φ" and "γ", which confirms the fact that by increasing the angle of soil friction resulting from pile driving, the soil becomes more compacted and its unit weight is increased. The negative association between "η" and "L/D", which means that the ultimate bearing capacity of groups with longer piles is lower than that for groups with short piles, is contrary to what was reported in the literature. Chellis (1961) stated that pile groups with deeper depth of embedment are believed to provide a larger area for the shear resistance of soil in the bounding perimeter of the group, which increases their ultimate bearing capacity. Therefore, another parametric study will be carried out to accurately examine the impact of changing "L/D" on "η".

3.3.2 Analysis of Variance (ANOVA)

Table 3.2 shows the results of seven ANOVA tests that were carried out in order to measure the association between the response variable and the following qualititative parameters: method of installation, cap condition, type of loading, type of test, soil condition, pile cross section, and pile arrangement. The last two columns of this table show the significance of the ANOVA test, while the other columns show some descriptive statistics (i.e. the mean and standard deviation) about each value of the qualititative parameter.

Parameter	Value	N	Mean	StDev	F	P
Method of Installation	Bored	31	1.27	0.36	2.45	0.09
	Driven	121	1.11	0.46		
	Jacked	24	1.27	0.30		
Cap Condition	Direct Contact	25	1.46	0.31	15.38	0.00
	Freestanding	151	1.11	0.43		
Type of Loading	Compression	95	1.29	0.34	21.73	0.00
	Uplift	81	1.00	0.47		
Type of Test	Field Test	39	1.35	0.41	10.19	0.00
	Lab Test	137	1.11	0.43		
Soil Condition	Dense	67	0.76	0.24	191.70	0.00
	Loose	109	1.40	0.33		
Pile Cross Section	Circular	136	1.11	0.44	8.24	0.01
	Square	40	1.33	0.36		
Pile Arrangement	1.5x2	12	1.45	0.36	5.92	0.00
	1x2	27	1.21	0.42		
	1x3	9	0.65	0.15		
	1x4	13	1.22	0.39		
	2x2	54	1.23	0.42		
	2x3	15	0.76	0.29		
	2x4	9	1.37	0.24		
	3x3	37	1.15	0.45		

Table 3.2: ANOVA test results

The P-value in the last column of Table 3.2 is the probability of accepting or rejecting the null hypothesis "Ho". If the P-value is less than the significance level (α) then reject Ho, while if it is greater than or equal α then accept Ho. The F-value is the ratio between the difference of the means of values in the explanatory qualititative parameters and the difference in these means due to data errors. A high F-value means that the existing parameter has a significant effect on the response variable.

Based on Table 3.2 it can be clearly noticed that the cap condition, type of loading, type of test, soil condition, pile cross section, and pile arrangement have significant effects on the pile group efficiency (η) as they all give P-values < 0.05 and relatively high F-values. Details of these effects are discussed below. On the other hand, the method of installation is considered insignificant with respect to pile group efficiency, as it has relatively high P-value and low F-value. This differs with what was reported in the literature, where many model tests showed that bored piles result in lower group efficiency than jacked and driven piles. According to Meyerhof (1976), the ultimate group capacity of bored pile groups in sand under lied by a weak deposit should be taken as about two-thirds of the sum of the bearing capacities of single piles. This may be because of the overlap of point shear zones without soil compaction which leads to a reduction of the individual pile capacities by one half for S/D =3, also some reduction in the shaft resistance may be expected.

The insignificance of the method of installation in the previous ANOVA test may be due to the fact that the analysis of variance evaluates the changes in the average and standard deviation of the response variable corresponding to each value of one independent variable regardless of the impact of other independent variables (one way ANOVA). In order to accurately assess the effect of the method of installation on the pile group efficiency, two ANOVA tests were carried out using constant values for the soil condition and cap condition variables as shown in Table 3.3.

Parameter	Value	N	Mean	StDev	F	P
Method of Installation	Bored	31	1.2658	0.3643	4.43	0.014
	Driven	60	1.4775	0.3264		
	Jacked	18	1.3994	0.2009		

(a) For Loose Soil Condition

Parameter	Value	N	Mean	StDev	F	P
Method of Installation	Bored	12	0.9967	0.1778	13.6	0
	Driven	60	1.4775	0.3264		
	Jacked	12	1.3275	0.1902		

(b) For Loose Soil Condition and Freestanding Cap Condition

Table 3.3: ANOVA test results for method of installation

The ANOVA test results for the method of installation in case of loose soil shown in Table 3.3 (a) clearly indicate that bored piles result in the lowest pile group efficiency while driven piles result in the highest pile group efficiency. This is in agreement with what was reported in the literature as mentioned earlier. This conclusion was also confirmed in the ANOVA test for the method of installation in case of loose soil and freestanding cap condition shown in Figure 3.3 (b), which has a higher significance than the first test.

For the type of test as well as the pile cross-section, no design theory considered these parameters in its formulation. But based on Table 3.1, field tests yield higher group efficiency values than lab test. Also, square cross sections give higher η values than circular cross sections. Both of these two parameters will be considered in the development of the proposed model presented in the following chapter.

For the cap condition parameter, the direct contact condition yields a higher efficiency of the pile group (η = 2.00) than the freestanding condition (η = 1.11). This is in a good agreement with what was reported in the literature. According to Hansbo (1993) the group capacity in case of direct contact condition is higher than that in case of freestanding condition because of the contribution of the pile cap, that has two components: a) the bearing capacity of the cap itself and b) its surcharge effect on the pile shaft. Also, according to Chen et al. (1993), if the pile cap is resting on soil, the group efficiency will be relatively increased by a certain amount expressed by "Pc/ P", where "P_c" is the load carried by cap and "P" is the total load applied to the group. The ratio "Pc/P" was found to be very much dependant on the group geometry (i.e. "S/D" and "L/D"), the soil characteristics beneath the cap, and the method of installation.

For the type of loading parameter, the case of compression loading yields a higher group efficiency (η = 1.51) than the case of uplift loading (η = 1.0). This is in a good agreement with what was reported in the literature, where Chaudihuri et al (1982) reported a sharp reductions in η at all S/D when uplift loading is applied.

For the soil condition parameter, piles in loose sand yield a higher efficiency of the pile group (η = 1.5) than those in dense sand (η = 0.97). This is in a good agreement with what was reported in the literature, where piles driven in loose sand compact the surrounding soil because of the vibration effect resulting from the driving process, while those driven in dense sand loosen the surrounding soil due to dilatancy effect. According to Lo (1967), the ultimate group efficiency is a combined result of the degree of

compaction as well as the degree of pressure bulb overlapping. Also, based on several experimental investigations, the pile group efficiency was observed to be greater than 100% in case of loose sand and lower than 100% in case of dense sand. Since the soil condition parameter is significant, this parameter will be used in model development instead of "φ" and "γ".

For the pile arrangement parameter, the 3x3 arrangement yields the highest group efficiencies (η = 1.56) followed by rectangular and square arrangements with smaller number of piles, while line arrangements yields the lowest group efficiencies (η = 0.65). This is in good agreement with what was reported in the literature, where O Neill (1983) reported that line groups produce lower group efficiency values than square groups for the same number of piles. This is because the soil becomes compacted in only one dimension with line groups. Also, based on several model compression tests on vertically loaded rectangular groups, higher values of "η" occurred by increasing the number of piles. The number of piles (N) parameter will not be considered in the proposed model development since the pile arrangement parameter accounts automatically for the number of piles. It should be noted that group efficiencies corresponding to pile arrangements 1x6 and 4x4 cannot be accepted since these values were obtained from only two data records. These records will be removed from the data set used for the proposed model development because considering very small samples may lead to unreliable results. Also, the data records of Vesic's test in which the unit weight of soil is the only variable will be removed, since the unit weight of soil (γ) will not be considered as a governing parameter in the model development as discussed earlier in this chapter.

3.3.3 Parametric Study

Based on the results of the correlation matrix and the ANOVA test, it has been proven that statistical tests could not correctly assess the variation in the group efficiency (η) with either pile spacing-diameter ratio (S/D) or pile length-diameter ratio (L/D). For these reasons, a parametric study will be carried out in order to assess these two relationships by changing each of these two parameters while fixing the values of the remaining parameters.

For example, in case of loose sand condition, freestanding cap, compression loading, 1x4, 2x2, 2x4, and 3x3 group arrangements, and L/D = 15, the pile group efficiency (η) decreases by increasing the pile spacing-diameter ratio (S/D) as it is shown in Figure 3.9. This is in good agreement with what was reported in literature, because the high efficiency in sand is apparently because of the increased lateral stresses against the piles when closely spaced, and also the soil becomes more compacted as a result of driving neighboring piles. These two factors tend to increase the shear resistance between the pile and the soil resulting in an increase of the overall side resistance of the pile group (O Neill 1983). Also, according to Lo (1967), the vibration resulting from pile driving tends to make the soil more compacted, and if only compaction is to be considered the group efficiency is expected to exceed 100%. But, actually the group efficiency is considered to be a function of:

(a) Soil compaction resulting from the driving process.

(b) Degree of pressure bulb overlapping.

(c) Pile spacing.

Based on Meyerhof (1976) when piles are closely driven, an overlap of compaction zones near the piles will be created; this overlap increases mainly the skin resistance, which may exhibit equivalent pier failure at small spacing.

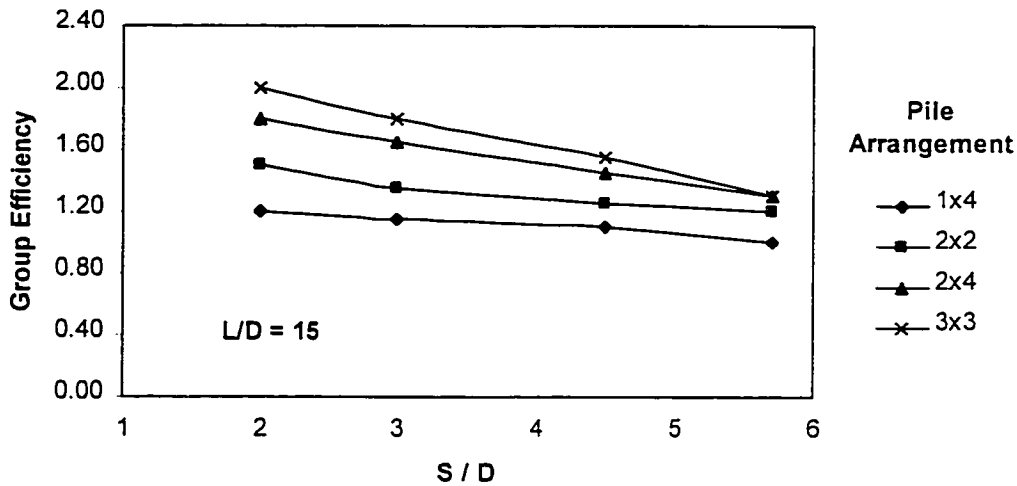


Figure 3.9: The effect of S/D on group efficiency (η) in case of loose sand and L/D = 15

For the same test parameters mentioned above but in case of dense sand, decreasing the pile spacing does not increase the group efficiency as it is shown in Figure 3.10. This is in good agreement with what was reported in the literature. According to Lo (1967), when piles are driven in dense sand, the sand surrounding the pile will be loosened due to dilatancy, also the overlap of stress zones will reduce the load capacity of the individual piles in the group.

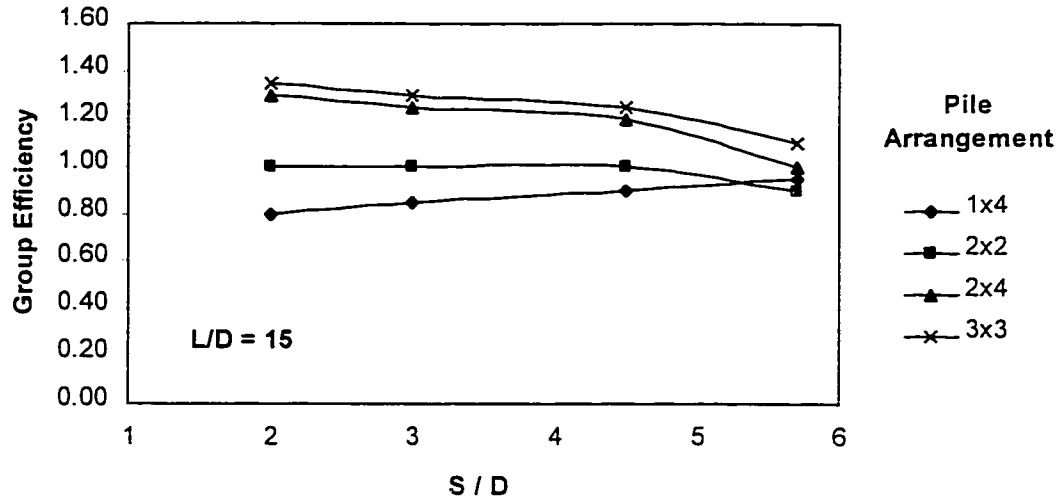


Figure 3.10: The effect of S/D on group efficiency (η) in case of dense sand and L/D = 15

Another example which presents the relationship between the group efficiency (η) and the pile spacing is for the case of loose sand, freestanding cap, L/D =20, compression loading, and 1x4 and 2x2 group arrangements. In this case, the group efficiency increases by decreasing the pile spacing as shown in Figure 3.11.

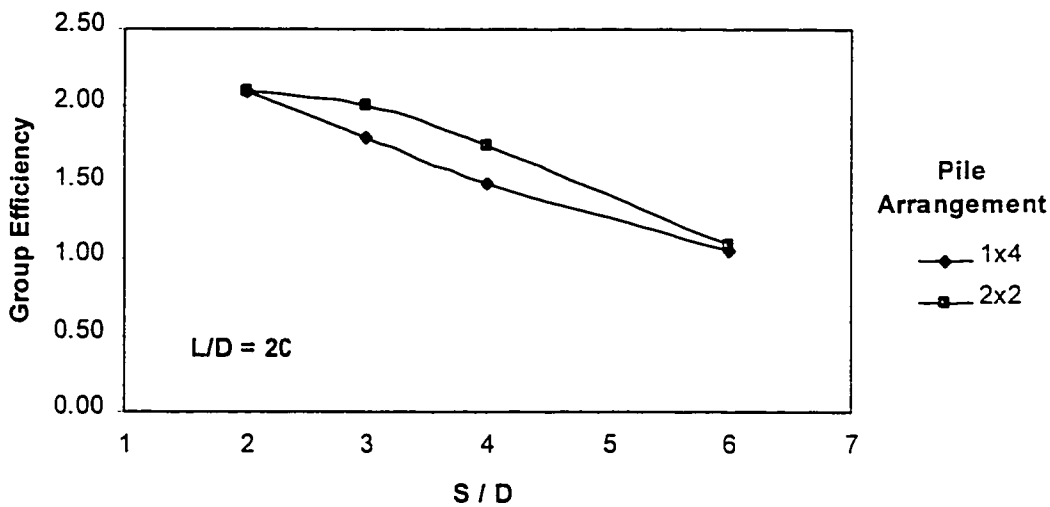


Figure 3.11: The effect of S/D on group efficiency (η) in case of loose sand and L/D = 20

NOTE TO USERS

Page(s) not included in the original manuscript are unavailable from the author or university. The manuscript was microfilmed as received.

75-76

This reproduction is the best copy available.

CHAPTER 4

ARTIFICIAL NEURAL NETWORKS MODEL

4.1 GENERAL

Artificial Intelligence (AI) is the area of computer science concerned with making computers smarter. More specifically, AI is the collection of technologies that permit a computer to mimic the human mind in its main functions like thinking and reasoning. Since the early 1950s, research in AI has grown rapidly to provide many powerful tools in different fields of science Examples of these tools are expert systems, neural networks, and case-based reasoning systems. The value of these tools is coming from the fact that standard data processing tools do not work with some kind of problems, but with AI tools these problems can be tackled (Frenzel 1987).

Artificial intelligence tools, such as expert systems (ES), artificial neural networks (ANN), and case-based reasoning (CBR), can be applied to an incredibly wide range of problems. Any problem that doesn't lend itself to an algorithm solution is a candidate for AI technologies. Since algorithms need specific pieces of data to solve the problem, many non-numerical problems containing uncertainty and ambiguity do not fit the algorithmic process. As it turns out, there are many situations in this world of ours that are disorganized or imperfect to that we lack complete information about. AI can deal with such problems, often producing a satisfactory solution. AI applications were generated in different fields of science like, medicine, geology, chemistry, computer science, and civil engineering which also include many fields like, structural engineering,

construction engineering and management, environmental engineering, and geotechnical engineering. These applications solve a variety of problems such as diagnosis, fault detection, prediction, interpretation, monitoring, instruction, planning, and design (Frenzel 1987; Boussabaine 1996; Watson 1997).

4.2 ARTIFICIAL NEURAL NETWORKS

Among the different technologies of AI, artificial neural networks (ANNs) are considered promising management tools that have capabilities particularly suited for analogy-based decision problems. The parallel and distributed structure of ANNs along with their capabilities of generalization, fault tolerance, adaptive and associative performance, ability to perform dynamic and real-time functions, and their limited requirement of software, ensure their appropriateness for many practical applications in geotechnical engineering. Below are some examples for the use of ANNs in solving different problems in geotechnical engineering in general and in piled foundation in particular.

Goh et al. (1995) developed a back propagation neural network (BPNN) to evaluate the frictional capacity of piles (*fs*) driven in cohesive soils using input data that was collected from load tests. Several experiments using different network settings were carried out in order to obtain the most reliable neural network model. Based on a comparison between the measured and predicted frictional capacity of piles using ANN model, it was concluded that the neural network was more successful in modeling the

non-linear relationship between *fs* and the other parameters than the conventional methods

Goh (1995) developed a back propagation neural network (BPNN) to model the complex relationship between seismic soil parameters and its liquefaction potential. The input database consisted of 85 patterns collected from 42 liquefied sites and 43 sites that did not liquefy after earthquakes. Validating the performance of the BPNN model using a set of data that was not used in model development showed a higher rate of success (95%) than the conventional methods, which produced a success rate of 84%.

Teh et al (1997) developed a back propagation neural network to estimate the static pile capacity from dynamic stress wave data. The database of 37 dynamic pile tests extracted from commercial CAPWAP analysis reports were used. Initially, a neural network was used to estimate the total static pile capacity. Then, a second network was set up to predict the resistance distribution along the pile shaft, and a third network was set up in order to predict the damping and quake parameters as well as the soil resistance distribution. The output data resulting from all the three networks were compared to the static pile capacities resulting from CAPWAP analysis. Based on this comparison, it has been demonstrated that the neural network approach was a reliable method for predicting the static pile capacity based on stress wave data with a good degree of accuracy. Also, another advantage of using this approach over the traditional curve matching techniques is that it provides the design engineers with a real-time prediction of pile capacity in the field.

Abu Kiefa (1998) developed a General Regression Neural Network (GRNN) in order to determine the ultimate capacity of piles driven into a cohesionless soil. In this study, data collected from 59 load tests covering a number of parameters was used. Three GRNN models were developed to estimate the total pile capacity, the tip pile capacity, and the shaft pile capacity. The results of the three networks and those obtained from using empirical methods were compared with actual data. This comparison showed high values of the coefficient of determination (R^2 =0.95) for the neural network models and low values ranging from 0.52 to 0.63 for the empirical methods.

Kim et al. (2002) conducted a back propagation neural network (BPNN) model as well as a sequential neural network (SNN) model in order to predict the lateral load- deflection relationship of group piles. In order to verify the applicability of these networks, a total of 146 model tests results were used for model development. Many input parameters including: sand condition, constraint condition of the head of a pile (free or fixed), group arrangement, and type of loading (central or eccentric) were considered. In order to demonstrate the predictive capabilities of the two ANN models, statistical measures, such as the covariance and the root mean square error were calculated. Based on these measures, the BPNN and SNN models could predict the lateral load-deflection relationship of laterally loaded group piles with reasonable accuracy, and the SNN model was more reliable than the BPNN model.

Shahin et al. (2002) developed an ANN model to predict the settlement of shallow foundations because existing models could not assess the settlement with a fair degree of accuracy. Many factors were considered in the developed model, such as distribution of applied stresses, soil compressibility, stress-strain history of soil, and difficulty of obtaining undisturbed samples. The database used for model development and validation of the proposed model consisted of 189 records obtained from the field tests that are available in the literature. Comparing the values predicted by the developed model with the measured ones indicated that the ANN model outperforms the conventional models as it gives relatively high coefficient of determination (R^2 =0.819) and low mean absolute error.

4.3 COMPONENTS AND OPERATION OF ANN

An artificial neural network is a collection of interconnected computational elements called neurons that have performance characteristics in common with biological neurons (Fausett 1994). This brain-like structure makes ANN models superior to knowledge-based models and mathematical models in solving problems that involve intuitive judgment, posses high degree of non-linearity, and contain time-dependant data. Three-layered back propagation neural networks (BPNNs) were selected for the development of the proposed model. This is because of the ability of these types of ANNs to approximate any non-linear function and to map unknown relationships between inputs and outputs (Hornik et al. 1989).

Figure 4.1 shows a typical three-layered back propagation neural network with N_i, N_j, and N_k neurons in its input, hidden, and output layers respectively (Fausett, 1994). The connection between any two neurons has a weight (W_{ij} or W_{jk}) that represents the importance of the input of this connection relative to the inputs of the connections entering the same neuron. Each neuron (i) in the input layer has an activation function F (i.e. transfer function) that transforms the input of this neuron (I_i), which comes from a data pattern, into an output (O_i). A binary sigmoid function (also called logistic sigmoid function) is commonly used in BPNNs because of its ability to transform different types of attributes. This function takes the following form:

$$F = \frac{1}{1 + e^{-g_i I_i}} \tag{4.1}$$

Where g_i is the gain of the sigmoid transfer function (i.e. steepness parameter) of neuron i.

The exact form of the sigmoid function is not particularly important, it is merely important that the function be monotonically increasing and bounded by both lower and upper limits. In the presented function, the lower limit is 0 as I_i approaches negative infinity, and the upper limit is 1 as I_i approaches positive infinity.

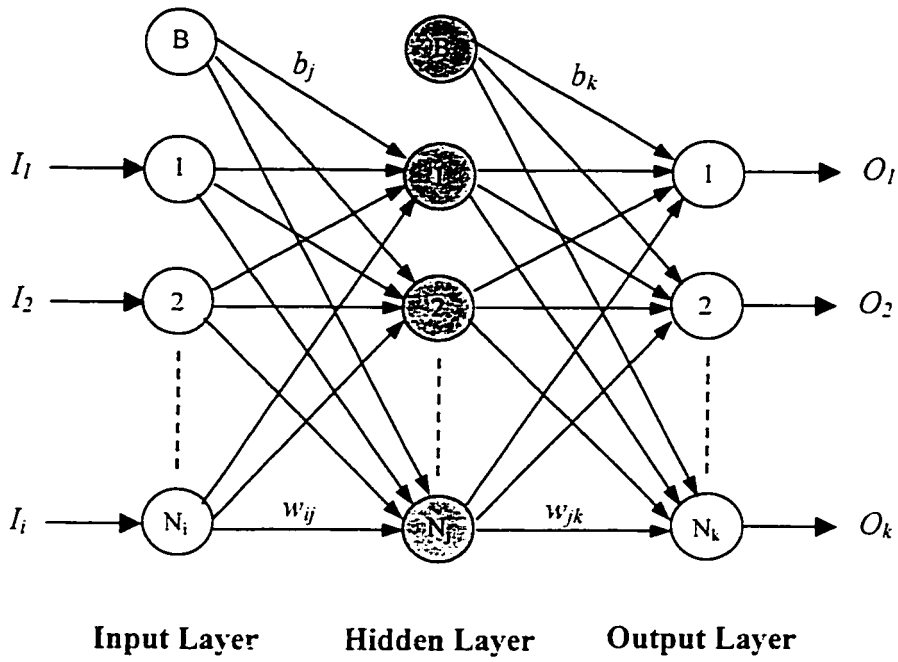


Figure 4.1: The architecture of three-layered ANNs

Each neuron (j) in the hidden layer has an input (Ij) that is equal to the summation of the weighted outputs of the previous layer and the weight on the connection with the bias neuron. A bias neuron acts exactly as a regular neuron that has an input equal to 1.0 to indicate a general inclination. The net input is calculated as follows:

$$I_j = b_j + \sum_{i=1}^{i=N_i} W_{ij} \times O_i \qquad (4.2)$$

Where b_j is the weight on the connection between the bias neuron of the input layer and the neuron j of the hidden layer.

Hidden neurons have also activation functions that transform their net inputs into outputs to feed forward the neurons of the output layer. Each neuron (k) in the output

layer transforms its net input (calculated using equation 4.2) into an output (O_k) that is compared with a target value (T_k) obtained from the data pattern. The error of this neuron (E_k) is calculated as follows:

$$E_k = T_k - O_k \tag{4.3}$$

All the errors of the output neurons are propagated backward to adjust the weights on the connections between the hidden layer and the output layer as follows:

$$W_{jk}\ (new) = W_{jk}\ (old) + \Delta W_{jk} \tag{4.4}$$

$$\Delta W_{jk} = \alpha\ E_k\ F'(I_k)\ O_j \tag{4.5}$$

Where α is the learning rate that indicates the amount of adjustment to the old weight. and F' is the first derivative of the transfer function with respect to the input signal.

Weights on the connections between the input layer and the hidden layer are adjusted in the same manner after calculating the error at each hidden neuron j as follows:

$$E_j = \sum_{k=1}^{k=N_k} W_{jk} \times E_k \tag{4.6}$$

The above-mentioned process is repeated for all the data patterns, which are divided into two sets: a training set and a testing set. The data patterns of the training set are used

to refine network weights in several training cycle (one training cycle means passing all the data patterns of the training set into the network). While, the data patterns of the testing set are used to evaluate the network performance after each training cycle to check the network ability to manipulate patterns that are different from those used in training. Hundreds and thousands of training and testing cycles are required to obtain the network weights that minimize the overall network error. This error can be presented in two forms:

1. Root means square error (RMSE) of testing data patterns, which is calculated as follows:

$$RMSE = \sqrt{\frac{\sum_{p=1}^{p=N_{testing}} \sum_{k=1}^{k=N_k} E^2_{kp}}{N_k N_p}} \tag{4.7}$$

2. Mean absolute error (MAE) of testing data patterns, which is calculated as follows:

$$MAE = \frac{\sum_{p=1}^{p=N_{testing}} \sum_{k=1}^{k=N_k} \left| E_{kp} \right|}{N_k N_p} \tag{4.8}$$

4.4 DEVELOPMENT OF ANN MODELS

For the development of any ANN model, four phases should be considered: identification phase, collection phase, implementation phase, and verification phase. Some of these phases may be iterative in order to achieve the most reliable ANN model. For example, identification, collection, and implementation phases may be repeated after

the verification phase. The following sub-sections present a detailed explanation of applying the above-mentioned phases to the development of an ANN model that estimate pile group efficiency. Only the last iteration of the development cycle will be presented.

4.4.1 IDENTIFICATION PHASE

In order to identify the input neurons of the proposed ANN model, a thorough understanding of the parameters affecting the efficiency of pile groups in cohesionless soil and subjected to axial loading is necessary. This understanding was developed in three steps. First, earlier studies and model tests regarding the estimation of pile group efficiency were examined and the parameters involved in these studies and tests were considered. Second, two different statistical tests (i.e. correlation analysis and analysis of variance) were carried out to identify the importance and the significance of each of these parameters (refer to chapter 3). Since the results of these tests were contradictory to what was reported in the literature about some parameters, such as the pile spacing-diameter ratio (S/D) and the pile length-diameter ratio (L/D), the third step was carried out. In this step, a detailed parametric study was made to examine the influence of changing the value of each of these parameters on the value of pile group efficiency when the values of other parameters remain constant (refer to chapter 3).

These three steps indicated that the parameters that significantly affect the pile group efficiency (η) are: the method of pile installation, pile cap condition, type of pile loading, type of pile test, soil condition, pile cross section pile arrangement, pile length-diameter ratio (L/D), pile spacing-diameter ratio (S/D), unit weight of soil (γ), and soil friction

angle (φ). Because of the disadvantageous impact of considering a large number of parameters in the development of ANN model especially when the available data patterns are limited, some parameters that are believed to have the same impact on the pile group efficiency were eliminated. For example, both of the soil friction angle (φ) and unit weight of soil (γ) were disregarded because the soil condition (loose or dense sand) implicitly accounts for their impact on the pile group efficiency. It should be mentioned that many of the above-mentioned input parameters, such as type of test, pile cross section, and type of loading, were not considered before in any of the experimental or theoretical models that are available in the literature. However, these parameters were considered in the developed model based on the outcome of the data analysis presented in chapter 3.

Table 4.1 lists the output parameter and the nine input and parameters identified for the development of the ANN model along with their values or ranges of values that the model would be restricted to. Using different values or values outside these ranges may lead to erroneous results.

Parameter Type	Parameter Name	Values / Range	Number of Neurons
Input	Method of Installation	Driven - Jacked - Board	3
	Cap condition	Freestanding - Direct Contact	2
	Type of Loading	Compression - Uplift	2
	Type of Test	Field - Lab	2
	Soil Condition	Loose - Dense	2
	Pile Cross Section	Square - Circular	2
	Pile Arrangement	1.5x2 - 1x2 - 1x3 - 1x4 - 2x2 - 2x3 - 2x4 - 3x3	8
	Length/Diameter ratio (L/D)	8 - 35.4	1
	Spacing/Diameter ratio (S/D)	1.5 - 6	1
Output	Group efficiency (η)	0.43 - 2.23	1

Table 4.1: Input and output parameters of the developed ANN model

The seven symbolic parameters shown in Table 4.1 have to be transformed into numeric parameters because neural networks deal only with numbers (Nelson and Illingworth 1991). Two methods can be used to transform these attributes into a form suitable for ANN representation (Moselhi et al. 1991): a binary-value transformation and a continuous-value transformation. In a binary-value transformation, a symbolic attribute is replaced by a vector of binary attributes, each of which represents a single attribute value. Selected attribute value is assigned 1 while other values are assigned 0s. In a continuous-value transformation, a symbolic attribute is transformed into a numeric attribute that takes integer or real values; each of them represents a single attribute value. Attribute values have to be ranked in such a way that the value with the higher rank results in a higher value for the network output. The binary-value transformation results in a higher number of input neurons than the continuous-value transformation, which affects negatively the network performance especially when small number of training patterns is available. However, the binary-value transformation is more realistic and does not require ranking attribute values, which is a subjective task.

Therefore, the binary-value transformation was used with all the symbolic parameters of the developed model. The third column in Table 4.1 lists the number of neurons assigned to represent each parameter after considering the binary-value transformation of the symbolic parameter. This results in a total of 23 neurons in the input layer, while only one neuron in the output layer for the pile group efficiency.

4.4.2 COLLECTION PHASE

Data used for training and testing ANN models is the main source of knowledge acquisition in this kind of models. For the development of the proposed model, data were collected from the field and laboratory tests conducted by Kezdi (1957), Kishida and Meyerhof (1965), Tejchman (1973), Garg (1979), Liu (1985) Chattopadhyay (1994), and Mukherjee (1996). These data, which account for 196 records, were listed in Table B.1 in Appendix B. All these records were reviewed using Microsoft Access 2000 to check their completeness and consistency. Although ANN models, in general, can run when some data items are missed, it is not preferable to use incomplete or inconsistent data records in model training or testing because this may affect the ability of the model to learn and consequently result in inaccurate outputs. Table B.2 in Appendix B lists the 20 data records that were filtered out from the data set because of their either incompleteness or inconsistency with other records.

4.4.3 IMPLEMENTATION PHASE

For the implementation of the proposed model, the neural network simulator called "BrainMaker Professional 3.11" developed by California Scientific Group was used for training, testing, and validating the ANN model. This simulator was selected because of its ability to work with data having complex relationships and its affordability. Following are the five steps carried out for the model implementation.

1. Data Division

In order to obtain a reliable ANN model, data patterns were divided randomly into three sets: 1) a training set that is used to refine network weights (74%), 2) a testing set that is used to measure the network ability to tackle unseen data while training (13%), and 3) a validation set that is used to evaluate the overall performance of the trained network (13%). These percentages were determined based on common practices and data availability. It should be noted that the function of the testing set is extremely important because ANN models tend to memorize the input-output relationships of the training records and give very low error in training, which is called over fitting model (Shahin et al. 2002). The testing set checks the ability of the ANN to generalization and gives realistic estimate of the model error. Also, the testing set is used to configure the optimal network architecture by determining the number of hidden nodes that has minimum error in testing.

Several trials were made to randomly select the data patterns constituting the training, testing, and validation sets. In these trials, the objective was to divide the data so that the probability distribution of the pile group efficiency remains almost the same in the three sets. This is necessary to guarantee a fair evaluation of the trained model. Table 4.2 shows the average and standard deviation of the group efficiency (η) of the training, testing, and validation sets that were used in model development. The closeness among the values of the average and standard deviation indicates the closeness of the probability distributions in the three sets.

Category	Size	Average	Std. Deviation
Validation Set	13%	1.157	0.442
Testing Set	13%	1.195	0.408
Training Set	74%	1.160	0.432

Table 4.2: The average and standard deviation of pile group efficiency in the different data sets

2. Network Architecture

Configuring the architecture of the neural network is one of the most important stages in model development. This configuration includes determining the number of hidden layers, the number of hidden neurons, and the network-learning rate that achieve the best model performance. As presented earlier, a three-layered back propagation network (input layer, one hidden layer, and output layer) was selected for model because of its ability to approximate any non-linear continuous function and map the unknown relationship between inputs and outputs provided that there are a sufficient number of connection weights (Hornik et al. 1989).

According to Flood (1994), there is no rule or specification that determines the number of hidden neurons, but this determination is more complex as the number of hidden layers is increased. A large number of hidden neurons may slow down the network training, but it may provide a greater potential to develop a solution surface that fits closely to that produced by the training patterns. A few numbers of hidden neurons may lead to an inaccurate ANN that cannot model complex relationships. To obtain the optimum number of hidden neurons, the equilibrium between having sufficient free

parameters (weights) that can approximate the non-linear function between inputs and outputs, and not having too much of them to avoid overtraining, must be done. Therefore, several ANN models were developed with varying number of hidden neurons and the RMSE and MAE of the testing set were calculated as shown in Figure 4.2. Based on this figure, the network with 17 hidden neurons was selected because it provided the minimum value for both RMSE and MAE of the testing set. Also, the smaller the number of hidden nodes, the smaller the number of connection weights, which is better because networks with large number of connection weights usually suffer from the problem of over fitting.

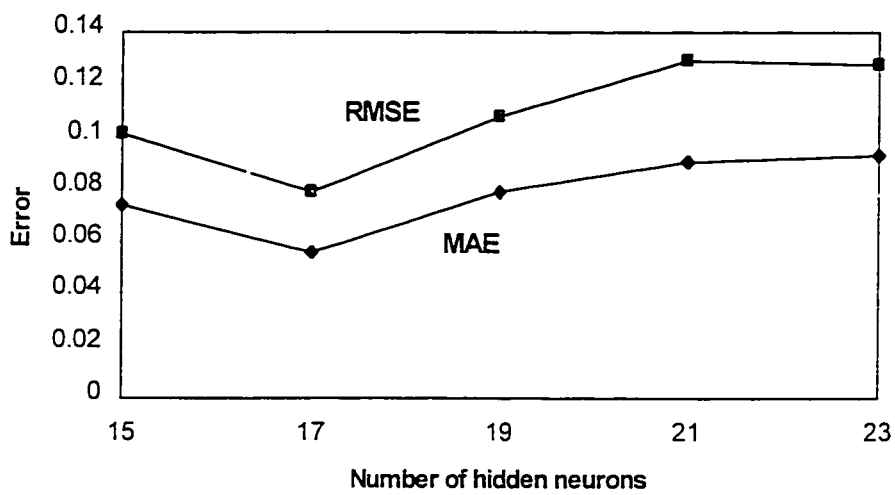


Figure 4.2: The RMSE and MAE for the testing set when different numbers of hidden neurons are used

Other network parameters, such as the learning rate, were determined automatically by the simulator based on the network performance during the training process. For example, the learning rate was set to change linearly from 1.0 to 0.1 according to the percentage of correct predictions (from 100% to 0%) in each training cycle. This set up is

efficient since high learning rates are recommended in the early training cycles to avoid lengthy training, while in advanced training cycles, low learning rates are recommended to fine tune network weights and achieve network stability (Yeh et al. 1993).

1. Weight Optimization

The process of optimizing connection weights in ANN model is equivalent to the process of parameter estimation in traditional empirical models. The main purpose of this process is to find connection weights that are able to approximate the non-linear relationships between input and output parameters. This is done as follows: First for each trial of number of hidden nodes, random initial weights and biases are generated; Second, the patterns of the training set are used to adjust connection weights using an initial value of the learning rate equal to 1.0; Third, the BrainMaker simulator automatically determines the learning rate of the following training cycle from 0.1 to 1.0 based on the percentage of correct predictions of the current training cycle; Fourth, the patterns of the testing set are used to calculate the RMSE and MAE of the network after each training cycle. These steps are repeated till predefined stopping criteria are satisfied.

2. Stopping Criteria

Several criteria can be selected by the developer prior to the weight optimization process, such as the maximum number of training cycles, a percentage of good patterns in testing, a value of network error in testing, a value of coefficient of determination, or combinations. In the developed model, training was carried out up to a 5000 training cycles. This number was determined based on network convergence while training.

Testing was carried out after each training cycle to evaluate the network performance using unseen patterns. The network that gave the minimum value for RMSE within the 5000 cycles was selected to represent the developed ANN model.

4.4.4 VALIDATION PHASE

The main purpose of model validation is to examine the model accuracy and efficiency. Model accuracy means its ability to provide correct answers when new cases that have not been used previously in model training or testing are encountered. Model efficiency means how competitive the model is with other existing model in terms of running time and capacity. Model validation also confirms the model's ability to generalize rather than simply memorize the input-output relationships of the training set. Data records that constitute the validation set were randomly selected by the developer a priori so that the probability distribution of pile group efficiency in the validation set is similar to those of the training and testing sets as shown earlier in Table 4.2.

The most widely used method for validating AI models compares the outcome of the developed model with the solutions of real-world cases that were not used in model development. The main advantage of this method it that it does not require any additional resources, such as human judgment, which is subjective or other systems, which may be erroneous. This method of validation was selected to evaluate the accuracy and the efficiency of the developed ANN model in predicting pile group efficiency.

Table 4.3 presents the comparison between the pile group efficiency measured in 23 actual load tests with those calculated using the ANN model. Several statistical indicators, such R^2, MAE, RMSE, and average percentage error, were calculated to evaluate the accuracy of the developed model. The high value of R^2 (0.724) and the low values of MAE, RMSE, and average percentage error (0.157, 0.232, and 13% respectively) shown at the bottom of Table 4.3 indicate clearly that accuracy of the ANN model. Also the experience with the development and validation of the ANN model showed its high efficiency with respect to the running time and updating / expanding effort, even with large number of data.

No.	Measured	Calculated	Error	Error^2	Percentage
1	1.6	1.4013	0.199	0.039	12.4%
2	1.1	1.2093	0.109	0.012	9.9%
3	1.28	1.6667	0.387	0.150	30.2%
4	1.25	1.2207	0.029	0.001	2.3%
5	0.46	0.5224	0.062	0.004	13.6%
6	0.68	0.5936	0.086	0.007	12.7%
7	1.6	1.385	0.215	0.046	13.4%
8	0.51	0.5325	0.023	0.001	4.4%
9	0.71	0.6283	0.082	0.007	11.5%
10	1.8	1.705	0.095	0.009	5.3%
11	2.2	1.4079	0.792	0.627	36.0%
12	1.45	1.5762	0.126	0.016	8.7%
13	1.15	1.2849	0.135	0.018	11.7%
14	1.2	0.986	0.214	0.046	17.8%
15	0.88	1.2049	0.325	0.106	36.9%
16	1.35	1.4694	0.119	0.014	8.8%
17	1	0.6564	0.344	0.118	34.4%
18	1.1	1.0014	0.099	0.010	9.0%
19	0.59	0.5365	0.054	0.003	9.1%
20	1.25	1.1983	0.052	0.003	4.1%
21	0.59	0.6085	0.019	0.000	3.1%
22	1.5	1.4444	0.056	0.003	3.7%
23	1.36	1.3622	0.002	0.000	0.2%
		R^2	MAE	RMSE	Average
		0.724	**0.157**	**0.232**	**13.0%**

Figure 4.3: Results of validating the developed ANN model

4.5 ANN VERSUS CONVENTIONAL MODELS

Many conventional methods used for the evaluation the efficiency of pile groups in cohesionless soils subjected to axial loading are presented in Chapter 2. These methods, which include: Das (1998), Converse Labarre (1941), Los Angeles group action (1944), Seiler and Kenney (1944), and Sayed and Bakeer (1992), are used to calculate pile group efficiency for the 23 cases used in validating the ANN model. Comparisons between the efficiencies measured in the 23 cases and those calculated using the five conventional methods were made. Table 4.4 presents the four statistical indicators R^2, MAE, RMSE, and average percentage error calculated for the ANN model and the five conventional methods. These indicators shows clearly that the ANN model fares much better than the conventional methods since it provide the highest coefficient of determination (R^2) and the lowest MAE, RMSE, and average percentage error. Figure 4.3 confirms this conclusion by plotting the measured efficiency versus the calculated one using the ANN model and the five conventional methods. The closer the plots to the diagonal line the better the accuracy of the method used. It is obvious that the ANN model provides the best fit to the measured data, while the other methods provide scattered plots.

Model	R^2	MAE	RMSE	Average%
ANN	0.7242	0.157	0.232	13.0%
Das 1998	0.0073	0.448	0.526	48.3%
Converse-Labarre	0.0075	0.452	0.546	37.3%
Los Angeles	0.0066	0.428	0.512	37.1%
Seiler and Keeney	0.0001	0.354	0.433	39.8%
Sayed and Bakeer	0.0818	0.498	0.620	65.6%

Table 4.4: The accuracy of the ANN model and the conventional methods in calculating pile group efficiency

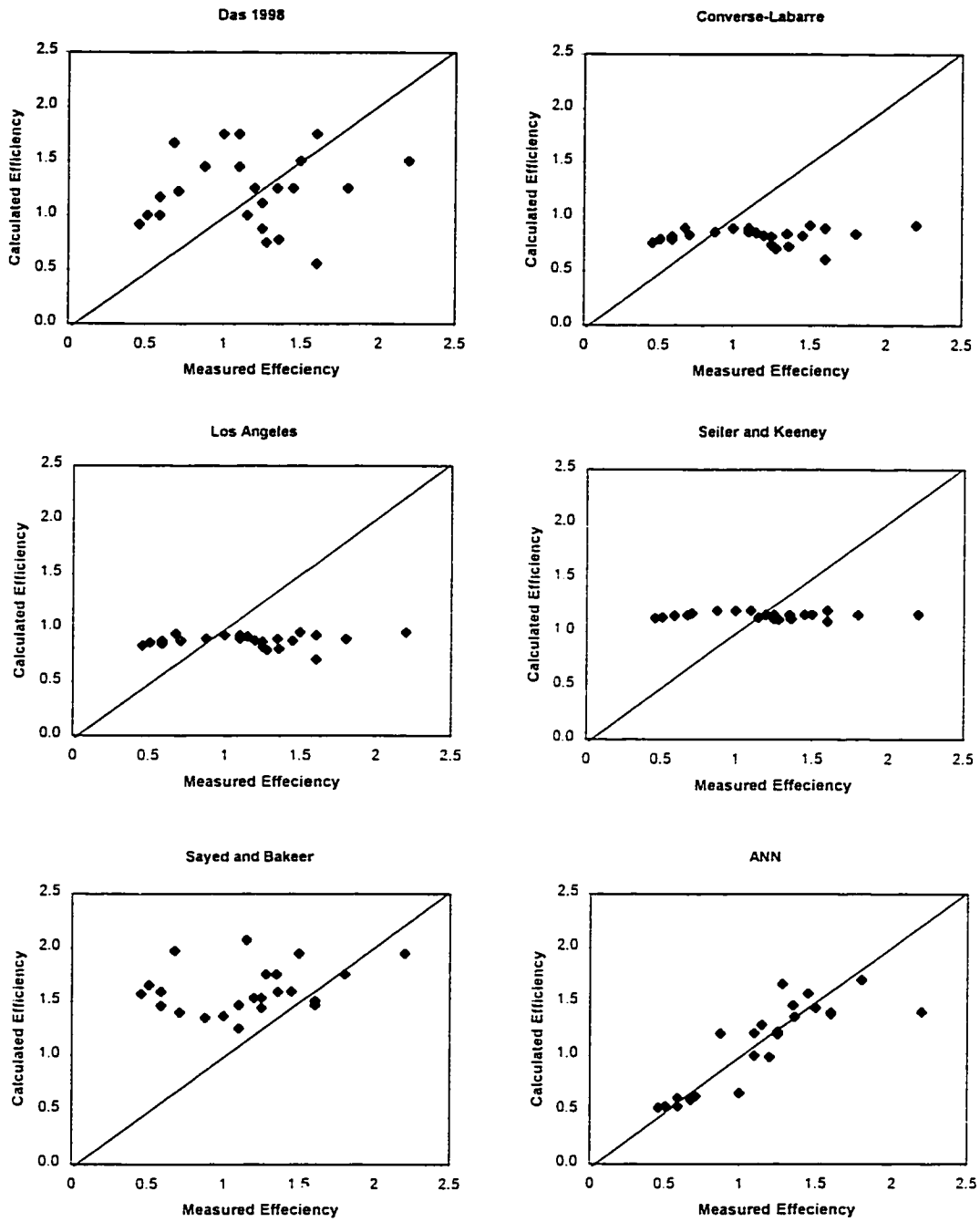


Figure 4.3: Comparing the ability of ANN model and the conventional methods to fit the measured data

Another way to present the reliability of the ANN model versus the reliability of the conventional methods is shown in Figure 4.4. In this figure, the results of comparing the predicted efficiency (i.e. calculated using the models) with the measured efficiency were plotted for different tolerance values. The tolerance value represents the maximum acceptable value for the absolute difference between the predicted efficiency and the measured one; the lower the tolerance value, the higher the prediction accuracy. Figure 4.4 shows clearly that the ANN model provides accurate predictions even when low tolerance values are allowed. For instance, when 20% tolerance value is allowed, the ANN model provide correct prediction in more than 80% of the cases while the conventional methods provide correct prediction in 45% of the cases at maximum. Therefore, the ANN model can be a design tool that assists foundation engineers in predicting pile group efficiency in an accurate and efficient manner.

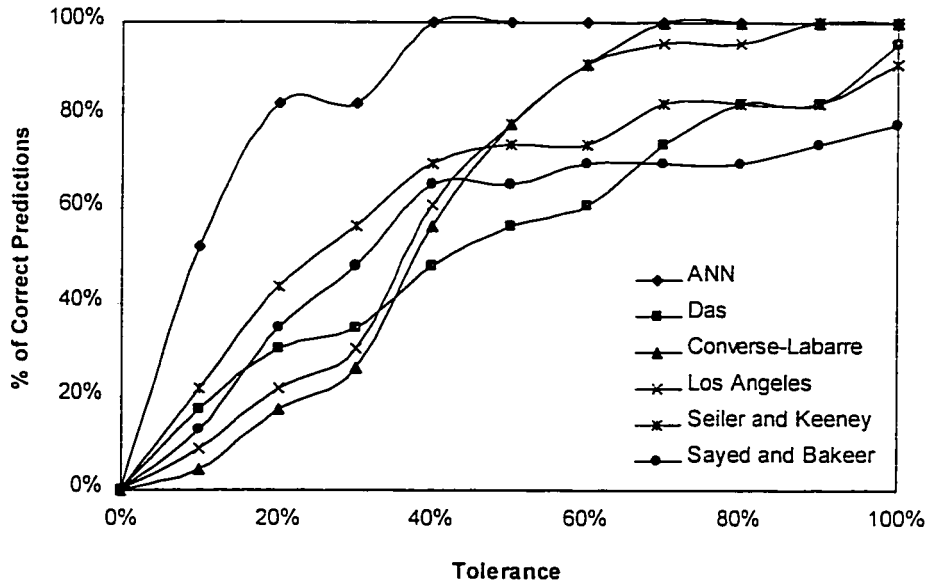


Figure 4.4: Comparing the accuracy of the ANN model with the conventional methods at different tolerance values

CHAPTER 5

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

Group efficiency analysis of piles subjected to axial loading in cohesionless soils, as in many situations in geotechnical engineering, is a complex problem that is not well understood. Therefore, many experimental and field tests have been conducted and several design theories have been developed to predict the group efficiency (η). This prediction is necessary for determining the bearing capacity of group piles as one unit compared to the sum of the bearing capacities of the individual piles. This research focused on developing a group efficiency model that provides the state-of-the art in this subject with accurate predictions of the dependant variable η based on many parameters involved.

The literature review of the group efficiency models has revealed many design theories. These theories are mostly mathematical models that attempted to solve the above-mentioned problem; the lack of physical understanding of the problem is usually supplemented by simplifying assumptions. In other words, most of these models like: Converse-Labarre (Bolin 1941), Los Angeles group action (1944), and Seiler and Kenney (1944), were based on relating the group efficiency to the spacing between piles and usually yielded η values of less than unity, regardless of the pile/ soil conditions, the

variation in the pile length-diameter (L/D) ratio, the cap condition (freestanding or cap resting), type of loading (compression or uplift), pile cross section (circular or square), soil conditions (loose or dense). Other models like Sayed and Bakeer (1992) have some limitations, for example, this model is limited to driven and jacked piles and cannot be applied to bored piles. Also, it carries the disadvantage of being limited to square and rectangular arrays and yields inaccurate results in case of line groups.

Therefore, the BPNN approach, which is categorized as an AI model, is introduced in order to provide geotechnical engineers with an accurate, realistic, and quick group efficiency model that eliminates the shortcomings of the state-of-the-art models. The basic idea of the BPNN model is similar to that used in the development of the conventional statistical models. In both cases, the purpose of the model is to establish a relationship between a historical set of model inputs and the corresponding outputs. This is achieved by presenting many examples of this input-output relationship during the learning phase while adjusting the connection weights in order to minimize the error between the measured group efficiency (resulting from field or laboratory tests) and the output predicted by the developed model.

In the present study, a total of 176 database records resulting from both field and laboratory tests were used for the training and the validation of the proposed BPNN model. For this model, three layers were used: input, hidden, and output layer. Only one hidden layer was considered as it was reported in the literature that increasing the number of hidden layers has a minor effect on the performance of any ANN used to model the

nonlinear relationship between the input and output parameters. The number of nodes for the input layer was 23 representing the input parameters identified to the network, while only one node representing the group efficiency was used for the output layer. In general, there exist no rule or specification controlling the number of hidden nodes, therefore, many trials was made by changing this number in order to achieve the minimum RMSE for the testing set. The learning rate used by the proposed BPNN was varied automatically between 1 and 0.1 by the Brain Maker simulator. Higher learning rates were used at the beginning of the training phase, while lower rates were used for the advanced training cycles. The training process was stopped when the RMSE for the testing set (testing while training) was minimized in the range of 5000 training cycles.

In order to identify the input parameters of the proposed network, a data analysis was carried out using two statistical measures; the correlation analysis and the analysis of variance (ANOVA) to study the relative importance of the factors that affect the group efficiency of piles subjected to axial loading in cohesionless soils. This analysis indicated that: Cap condition, Soil condition, Group arrangement, Type of loading, Type of test, Pile cross section, Pile length-diameter (L/D) ratio, Pile spacing-diameter (S/D) ratio, and Method of installation, are the most important factors affecting the response variable η.

The results of the BPNN model indicated that it could predict the group efficiency of piles with a fair degree of accuracy especially if compared with the conventional methods considered for an independent validation set. Also, one of the major advantages of ANNs is that it uses the database alone to determine the structure and parameters of the model, a

wider range of results can be obtained by only increasing the number of training patterns, it is a quick and reliable tool for estimating the group efficiency without performing any manual work. But it also has some shortcomings, like any other empirical model, it is limited to a certain range constrained by the data used in the model calibration phase, therefore for a wider range, the model has to be recalibrated, the lack of theory for it's development, and it's inability to explain the procedure used to obtain it's results based on the non linear relationship between the input and output parameters.

5.2 CONCLUSIONS

The contributions of this research are grouped into two areas: evaluation of the existing design theories and modeling the group efficiency using an ANN. Below is a highlight on the contributions made in each of these areas.

With respect to the evaluation of the existing design theories, five empirical models including: Converse-Labarre (Bolin 1941), Los Angeles group action (1944), Seiler and Kenney (1944), Das (1998), and Sayed and Bakeer (1992) were used in order to compare their results with the measured group efficiencies resulting from laboratory and field tests. Based on this comparison, the following conclusions were made:

1. None of the existing design theories except Sayed and Bakeer model (1992) could assess the difference between the loose and dense sand conditions.
2. None of the existing design theories could assess the variation in the pile length-diameter (L/D) ratio.

3. None of the existing design theories could assess the difference between the freestanding and cap resting conditions.
4. The empirical models proposed by Converse-Labarre (1941) and Los Angeles group yield group efficiency values close to each other and always less than unity for any number of piles as well as for any pile spacing-diameter (S/D) ratio.
5. The group efficiency calculated using Sayed and Bakeer (1992) is limited to driven and jacked piles and cannot be applied to bored piles.
6. The group efficiency calculated using Sayed and Bakeer (1992) is limited to square and rectangular arrays, while it yields inaccurate results in case of line groups.

With respect to modeling the group efficiency (η), a new approach that employs the use of ANN to predict the group efficiency of axially loaded piles in cohesionless soils was introduced. ANN is an AI approach that works with the specific knowledge encapsulated in a set of input data and can incorporate a transfer function to establish an input-output relationship in order to refine the required solution. This approach has eliminated most of the shortcomings of the conventional models presented in Chapter3. ANN contributed to solving the problems of the group efficiency models as follows:

1. ANN considers not only the planar geometry (pile spacing, pile diameter, and group arrangement) as the conventional models, but also the other parameters that were found to be governing the predicted group efficiency based on the results of the correlation analysis and the analysis of variance (ANOVA) statistical measures. This means that the effect of new parameters like: Cap condition, Soil condition, Type of

loading, Type of test, Pile cross section, Pile length-diameter (L/D) ratio, and Method of installation (neglected in the conventional models) is considered.

2. ANN is able to determine the structure of the model based on the presented data, for the conventional models, the structure of the model has to be determined first, and then the unknown parameters can be estimated. It has no limitations or assumptions incorporated and a wider range of results can be obtained only by increasing the number of training patterns.
3. The proposed ANN is not restricted to a certain group arrangement or a method of installation as the Sayed and Bakeer model (1992) is limited to square and rectangular arrays and can not be applied to bored piles.
4. The ANN provides a higher level of accuracy than the corresponding conventional models. This performance was proved to be satisfactory by testing two models using a set of validation patterns. The ANN yielded the highest coefficient of correlation (r^2 = 0.72) and the lowest MAE (= 0.157) and RMSE (= 0.23), while the conventional models yielded much lower coefficients of correlation ranging between 0.0001and 0.081, higher MAE ranging between 0.42 and 0 .49, and RMSE ranging between 0.23 and 0.62.

5.3 RECOMMENDATIONS FOR FUTURE RESEARCH

In order to enhance the capabilities of the ANN approach in modeling the group efficiency of piles subjected to axial loading in cohesionless soils, the following points need to be explored in the future research:

1. Validating the ANN model by operating it in a real-world environment where users can evaluate the system in various and real situations.
2. Introducing new input parameters to the network such as the stress history of soil in case of data availability.
3. Creating a new adjustment in the operation system of the model in order to be extended to a wider range of input data without the need to be recalibrated.
4. Investigating the case of piles subjected to negative skin friction forces.
5. Examining the case of batter piles and the effect of their angle inclination as an input parameter.

REFERENCES

Abu Keifa, M. A. (1998) "General regression neural networks for driven piles in cohesionless soils." Journal of Geotechnical and Geoenvironmental Engineering Division, ASCE, Vol. 124, No. 12, 1177-1185.

Bolin, H.W. (1941) "The pile efficiency formula of the Uniform Building Code." Building Standards Monthly, 10 (1), 4-5.

Boussabaine, A. H. (1996) "The Use of Artificial Neural Networks in Construction Management: A Review", Construction Management and Economics, E&FN Spon, Vol. 14, pp. 427-436.

Briaud, J. L., Tucker, L. M. and NG, E. (1989) "Axially loaded 5 pile group and single pile in sand" Proceedings of the 12th International Conference On Soil Mechanics And Foundation Engineering, I.C.S.M.F.E, Vol. 2, Rio De Janerio, August1121-1124.

Chattopadhyay, B.C. (1994) "Uplift capacity of pile groups", Proceedings of the 13th International Conference In Soil Mechanics And Foundation Engineering, I.C.S.M.F.E, Vol. 2, New Delhi, India, 539-542.

Chaudihuri, K. P. R., Ghataora, G. S., and Symons, M. V. (1982) "Uplift resistance of model piles and pile groups." Proceedings of the Second International Conference on Numerical Methods in Offshore Pilling, I.C. E., London.

Chellis, R. D. (1961) "Pile foundations." McGraw-Hill, New York.

Chen, Z. C., Xu, H., and Wang, J.H. (1993) "Cap pile interaction of pile groups." Proceedings of the Deep Foundations On Bored and Auger Piles conference, Van Impe (ed.), Balkema, Rotterdam, 133-141

Das, B. M. (1999) "Principles of foundation engineering." Brooks/ Cole Publishing Company, California, USA.

Fausett, L., (1994) "Fundamentals of Neural Networks: Architectures, and Applications", Prentice Hall.

Feld, J. (1943) "Discussion on friction pile foundations." Transactions, ASCE, Vol. 108, 115- 73.

Fleming, W. G. K., Weltman, A. J., Randolph, M. F. and Elson, W. K. (1992) "Piling Engineering" John Wiley and Sons, New York.

Flood, I., and Kartam, N. (1994) "Neural networks in civil engineering. I: Principles and understanding." Journal of Computing in Civil Engineering, Vol. 8, No. 2, 131-148.

Frenzel, L. E. (1987) "Understanding Expert Systems", Howard W. Sams & Co. Publishers.

Garg, K. G. (1979) "Bored pile groups under vertical loads in sand." Journal of Geotechnical Engineering Division, ASCE, Vol. 105(GT8), 939- 955

Goh, A.T.C (1995) "Empirical design in geotechnics using neural networks." Geotechnique, Vol. 45, No. 4, 709-714.

Goh, A.T.C (1995) "Seismic liquefaction potential assessed by neural networks." Journal of Geotechnical and Geoenvironmental Engineering Division, ASCE, Vol. 120, No.9, 1467-1480.

Hansbo, S. (1993) "Interaction problems related to installation of pile groups." Proceedings of the Deep Foundations On Bored and Auger Piles conference, Van Impe (ed.), Balkema, Rotterdam, 59-66.

Hornik, K., Stinchcomebe, M., and White, H. (1989) "Multilayer feedforward neural networks are universal approximators." Neural Networks, Vol. 2, No. 3, 356-366.

Ismael, N. F. (2001) "Axial load tests on bored piles and pile groups in cemented sands." Journal of Geotechnical Engineering Division, ASCE, Vol. 127(9), 766-773

Kezdi, A. (1957) "Bearing Capacity of Piles and Pile Groups", Proceedings of the Fourth International Conference of Soil Mechanics and Foundation Engineering, Vol. II, pp. 46-51.

Kim, B. T., and Kim, Y. S. (2002) "Lateral load-deflection modelling of group pile using artificial neural networks." Proceedings of the International Deep Foundation Congress, Orlando, FL, February.

Kishida, H. and Meyerhof, G. (1965) "Bearing capacity of pile groups under eccentric loads in sand", Proceedings of the 6th International Conference In Soil Mechanics And Foundation Engineering, I.C.S.M.F.E, Montreal, Vol 2, 270-274.

Liu, J.L., Huang, Q., Li, H., Li, X. and Hu, W. L. (1994) "Experimental research on bearing behaviour of pile groups in soft soil", Proceedings of the 12th International Conference In Soil Mechanics And Foundation Engineering, I.C.S.M.F.E, New Delhi, Vol. 2, 535-538.

Liu, J.L., Yuan, Z. L. and Shang, K.P. (1985) "Cap-pile-soil interaction of bored pile groups", Proceedings of the 11th International Conference In Soil Mechanics And Foundation Engineering, I.C.S.M.F.E, A. A. Balkema, Rotterdam (ed.), Vol. 3, San Francisco, CA, August, 1433-1436.

Lo, M.B., and Maguire, C. A. (1967) "An experimental study of the load distribution in pile groups in sand." Discussion, Journal of Geotechnical Engineering Division, ASCE, Vol. 3, No. 3, 353-354.

Masters, T. (1993) "Practical neural network recipes in C ++" Academic, San Diego.

Meyerhof, G. G. (1976) "Bearing capacity and settlement of pile foundations." Journal of Geotechnical Engineering Division, ASCE, Vol. 102, No. GT3, 197-227.

Moore, D. S. and McCabe, G. P. (1993) "Introduction to the Practice of Statistics", W. H. Freeman and Company.

Moselhi, O., Hegazy, T., and Fazio, P. (1991) "Neural Networks As Tools in Construction", Journal of Construction Engineering and Management, ASCE, Vol. 117, No. 4, 606-625.

Nelson, M. M. and Illingworth, W. T. (1991) A Practical Guide to Neural Nets, Addison-Wesley, New York.

O' Neill, M.W. (1983), "Group action in offshore pilling." Proceedings of the Geotechnical Practice In Offshore Pilling Conference, ASCE, Stephen Wright (ed.), New York, 25-64.

Poulos, H. G. and Davis, E. H. (1980) "Pile foundation analysis and design." John Wiley and Sons, New York.

Sayed, S. M., and Bakeer, R. M. (1992). "Efficiency formula for pile groups" Journal of Geotechnical Engineering Division, ASCE, Vol. 118 (2), 278- 299.

Seiler, J.F., and Kenney, W.D. (1944) "The efficiency of piles in groups." Wood Preserving News, Vol. 22, No. 11, 109-118.

Shahin, M. A., Maier, H. R., and Jaska, M. B. (2002) "Predicting settlement of shallow foundations using neural networks." Journal of Geotechnical and Geoenvironmental Engineering Division, ASCE, Vol. 128, No. 9, 785-793.

Sincich, T. (1994) "A Course in Modern Business Statistics", Macmillan College Publishing Company, Inc.

Teh, C. I., Wong, K. S., Goh, A. T. C., and Jaritngam, S. (1997) "Prediction of pile capacity using neural networks." Journal of Computing in Civil Engineering, Vol. 11, No.2, 129-138.

Tejchman, A. F. (1973) "Model investigations of pile groups in sand" Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99 (SM2), 199-217.

Terzaghi, K., and Peck, R. B. (1967) "Soil mechanics in engineering practice." John Wiley and sons, New York.

Vesic, A. S. (1968) "Experiments with instrumented pile groups in sand." Proceedings of the seventy-first Annual Symposium in Performance Of Deep Foundations, American Society For Testing And Material, ASTM STP 444, San Francisco, CA., 23-28 June, pp. 177-222.

Watson, I. (1997) "Applying Case-Based Reasoning: Techniques for Enterprise Systems", Morgan Kaufmann Publisher, Inc.

Yeh, Y., Kuo, Y., and Hsu, D. (1997) "Building KBES For Diagnosing PC Pile With Artificial Neural Network", Journal of Computing in Civil Engineering, ASCE, Vol. 7, No. 1, 71-93, 1993.

APPENDIX A

Comparisons of Design Theories

	Test Parameters											Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Unit Weight of Soil (γ) kN/m³	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	Measured	Calculated Das 1998	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and Keeney 1944	K	η_h	Calculated Sayed and Bakeer (1992)
1	37	16.2	0.0254	1	2	0.075	0.247	3.0	23.6	1x2	0.6	0.59	1.58	0.90	0.95	1.31	2.50	1.58	1.74
2	37	16.2	0.0254	1	3	0.075	0.247	3.0	23.6	1x3	0.6	0.44	1.68	0.86	0.93	1.35	2.50	1.68	1.80
3	37	16.2	0.0254	2	2	0.075	0.247	3.0	23.6	2x2	0.6	0.51	1.26	0.79	0.85	1.35	2.50	1.26	1.54
4	37	16.2	0.0254	2	3	0.075	0.247	3.0	23.6	2x3	0.6	0.47	1.15	0.76	0.82	1.37	2.50	1.15	1.47
5	37	16.2	0.0254	3	3	0.075	0.247	3.0	23.6	3x3	0.6	0.46	0.98	0.72	0.79	1.38	2.50	0.98	1.36
6	37	16.2	0.0254	1	2	0.105	0.345	4.1	23.6	1x2	0.6	0.77	1.95	0.92	0.96	1.41	2.00	1.95	1.73
7	37	16.2	0.0254	1	3	0.105	0.345	4.1	23.6	1x3	0.6	0.68	2.18	0.90	0.95	1.49	2.00	2.18	1.84
8	37	16.2	0.0254	2	2	0.105	0.345	4.1	23.6	2x2	0.6	0.64	1.63	0.85	0.90	1.49	2.00	1.63	1.57
9	37	16.2	0.0254	2	3	0.105	0.345	4.1	23.6	2x3	0.6	0.61	1.53	0.82	0.87	1.52	2.00	1.53	1.51
10	37	16.2	0.0254	3	3	0.105	0.345	4.1	23.6	3x3	0.6	0.59	1.31	0.80	0.85	1.54	2.00	1.31	1.41
11	37	16.2	0.0254	1	2	0.125	0.411	4.9	23.6	1x2	0.6	0.89	2.20	0.94	0.97	1.49	1.50	2.20	1.58
12	37	16.2	0.0254	1	3	0.125	0.411	4.9	23.6	1x3	0.6	0.82	2.51	0.91	0.96	1.59	1.50	2.51	1.69
13	37	16.2	0.0254	2	2	0.125	0.411	4.9	23.6	2x2	0.6	0.80	1.89	0.87	0.91	1.59	1.50	1.89	1.46
14	37	16.2	0.0254	2	3	0.125	0.411	4.9	23.6	2x3	0.6	0.75	1.78	0.85	0.89	1.64	1.50	1.78	1.42
15	37	16.2	0.0254	3	3	0.125	0.411	4.9	23.6	3x3	0.6	0.72	1.53	0.83	0.87	1.67	1.50	1.53	1.33
16	37	16.2	0.0254	1	2	0.075	0.247	3.0	29.5	1x2	0.75	0.57	1.58	0.90	0.95	1.31	2.50	1.58	1.74
17	37	16.2	0.0254	1	3	0.075	0.247	3.0	29.5	1x3	0.75	0.50	1.68	0.86	0.93	1.35	2.50	1.68	1.80
18	37	16.2	0.0254	2	2	0.075	0.247	3.0	29.5	2x2	0.75	0.48	1.26	0.79	0.85	1.35	2.50	1.26	1.54
19	37	16.2	0.0254	2	3	0.075	0.247	3.0	29.5	2x3	0.75	0.46	1.15	0.76	0.82	1.37	2.50	1.15	1.47
20	37	16.2	0.0254	3	3	0.075	0.247	3.0	29.5	3x3	0.75	0.43	0.98	0.72	0.79	1.38	2.50	0.98	1.36
21	37	16.2	0.0254	1	2	0.105	0.345	4.1	29.5	1x2	0.75	0.75	1.95	0.92	0.96	1.41	2.00	1.95	1.73
22	37	16.2	0.0254	1	3	0.105	0.345	4.1	29.5	1x3	0.75	0.67	2.18	0.90	0.95	1.49	2.00	2.18	1.84
23	37	16.2	0.0254	2	2	0.105	0.345	4.1	29.5	2x2	0.75	0.62	1.63	0.85	0.90	1.49	2.00	1.63	1.57
24	37	16.2	0.0254	2	3	0.105	0.345	4.1	29.5	2x3	0.75	0.59	1.53	0.82	0.87	1.52	2.00	1.53	1.51
25	37	16.2	0.0254	3	3	0.105	0.345	4.1	29.5	3x3	0.75	0.58	1.31	0.80	0.85	1.54	2.00	1.31	1.41
26	37	16.2	0.0254	1	2	0.125	0.411	4.9	29.5	1x2	0.75	0.88	2.20	0.94	0.97	1.49	1.50	2.20	1.58
27	37	16.2	0.0254	1	3	0.125	0.411	4.9	29.5	1x3	0.75	0.81	2.51	0.91	0.96	1.59	1.50	2.51	1.69
28	37	16.2	0.0254	2	2	0.125	0.411	4.9	29.5	2x2	0.75	0.79	1.89	0.87	0.91	1.59	1.50	1.89	1.46
29	37	16.2	0.0254	2	3	0.125	0.411	4.9	29.5	2x3	0.75	0.73	1.78	0.85	0.89	1.64	1.50	1.78	1.42
30	37	16.2	0.0254	3	3	0.125	0.411	4.9	29.5	3x3	0.75	0.71	1.53	0.83	0.87	1.67	1.50	1.53	1.33
31	37	16.2	0.0254	1	2	0.075	0.247	3.0	35.4	1x2	0.9	0.57	1.58	0.90	0.95	1.31	2.50	1.58	1.74
32	37	16.2	0.0254	1	3	0.075	0.247	3.0	35.4	1x3	0.9	0.49	1.68	0.86	0.93	1.35	2.50	1.68	1.80
33	37	16.2	0.0254	2	2	0.075	0.247	3.0	35.4	2x2	0.9	0.47	1.26	0.79	0.85	1.35	2.50	1.26	1.54
34	37	16.2	0.0254	2	3	0.075	0.247	3.0	35.4	2x3	0.9	0.44	1.15	0.76	0.82	1.37	2.50	1.15	1.47
35	37	16.2	0.0254	3	3	0.075	0.247	3.0	35.4	3x3	0.9	0.43	0.98	0.72	0.79	1.38	2.50	0.98	1.36
36	37	16.2	0.0254	1	2	0.105	0.345	4.1	35.4	1x2	0.9	0.73	1.95	0.92	0.96	1.41	2.00	1.95	1.73
37	37	16.2	0.0254	1	3	0.105	0.345	4.1	35.4	1x3	0.9	0.65	2.18	0.90	0.95	1.49	2.00	2.18	1.84
38	37	16.2	0.0254	2	2	0.105	0.345	4.1	35.4	2x2	0.9	0.62	1.63	0.85	0.90	1.49	2.00	1.63	1.57
39	37	16.2	0.0254	2	3	0.105	0.345	4.1	35.4	2x3	0.9	0.58	1.53	0.82	0.87	1.52	2.00	1.53	1.51
40	37	16.2	0.0254	3	3	0.105	0.345	4.1	35.4	3x3	0.9	0.56	1.31	0.80	0.85	1.54	2.00	1.31	1.41
41	37	16.2	0.0254	1	2	0.125	0.411	4.9	35.4	1x2	0.9	0.87	2.20	0.94	0.97	1.49	1.50	2.20	1.58
42	37	16.2	0.0254	1	3	0.125	0.411	4.9	35.4	1x3	0.9	0.80	2.51	0.91	0.96	1.59	1.50	2.51	1.69
43	37	16.2	0.0254	2	2	0.125	0.411	4.9	35.4	2x2	0.9	0.78	1.89	0.87	0.91	1.59	1.50	1.89	1.46
44	37	16.2	0.0254	2	3	0.125	0.411	4.9	35.4	2x3	0.9	0.72	1.78	0.85	0.89	1.64	1.50	1.78	1.42
45	37	16.2	0.0254	3	3	0.125	0.411	4.9	35.4	3x3	0.9	0.70	1.53	0.83	0.87	1.67	1.50	1.53	1.33

Table A.1: Comparing design theories based on the results of Mukhergee (1996) test.

	Test Parameters										Group Efficiency							
Test number	Unit Weight of Soil (γ) kN/m³	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	Measured	Calculated Das 1998	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and Keeney 1944	K	η_h	Calculated Sayed and Bakeer (1992)
P-41	15 2	0 1	2	2	0 2	0 658	2 0	15 2	2x2	1 524	1 29	0 96	0 705	0 78	2 29	2 54	0 955	1 31
P-46	15 0	0 1	2	2	0 2	0 658	2 0	15 2	2x2	1 524	0 97	0 96	0 705	0.78	2 29	0 89	0 955	0 97
P-45	14 5	0 1	2	2	0 2	0 658	2 0	15 2	2x2	1 524	1 28	0 96	0 705	0 78	2 29	2 37	0 955	1 28
P-42	14 8	0 1	2	2	0 3	0 987	3 0	15 2	2x2	1 524	1 33	1 27	0 795	0 86		1 98	1 273	1.33
P-43	14 7	0 1	2	2	0 4	1 316	4 0	15 2	2x2	1 524	1 35	1 59	0 844	0 89	0 00	1 64	1 592	1 35
P-44	14 8	0 1	2	2	0 6	1 974	6 0	15 2	2x2	1 524	1 29	2 23	0 895	0 93	0 36	1 05	2 229	1 29
P-91	14 4	0 1	3	3	0 2	0 658	2 0	15 2	3x3	1 524	1 21	0 71	0 606	0 69	2 51	2 9	0 707	1 23
P-93	15 0	0 1	3	3	0 2	0 658	2 0	15 2	3x3	1 524	1 17	0 71	0 606	0 69	2 51	2 57	0 707	1.18
P-92	14 9	0 1	3	3	0 3	0 987	3 0	15 2	3x3	1 524	1 37	0 99	0 727	0 79		2 76	0 990	1 38

Table A.2: Comparing design theories based on the results of Vesic (1969) test.

	Test Parameters										Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile Width (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	Measured	Calculated Das 1998	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and Keeney 1944	K	η_h	Calculated Sayed and Bakeer (1992)
1	32 5	0 035	1	4	0 070	0 230	2 0	15 0	1x4	0 525	0 80	1 00	0 779	0 88	1 35	3	1 273	1 705
2	32 5	0 035	2	2	0 070	0 230	2 0	15 0	2x2	0 525	1 00	0 75	0 705	0 78	1 33	2	0 955	1 20
3	32 5	0 035	2	4	0 070	0 230	2 0	15 0	2x4	0 525	1 30	0 63	0 631	0 72	1 36	3	0 796	1 31
4	32 5	0 035	3	3	0 070	0 230	2 0	15 0	3x3	0 525	1 35	0 56	0 606	0 69	1 36	4	0 707	1 40
5	32 5	0 035	1	4	0 105	0 345	3 0	15 0	1x4	0 525	0 85	1 38	0 846	0 92	1 52	2 5	1 751	1 844
6	32 5	0 035	2	2	0 105	0 345	3 0	15 0	2x2	0 525	1 00	1 00	0 795	0 86	1 49	1 5	1 273	1 20
7	32 5	0 035	2	4	0 105	0 345	3 0	15 0	2x4	0 525	1 25	0 88	0 744	0 81	1 54	2 5	1 114	1 39
8	32 5	0 035	3	3	0 105	0 345	3 0	15 0	3x3	0 525	1 30	0 78	0 727	0 79	1 54	3 5	0 990	1 54
9	32 5	0 035	1	4	0 158	0 520	4 5	15 0	1x4	0 525	0 90	1 94	0 896	0 95	1 90	1 75	2 474	1 832
10	32 5	0 035	2	2	0 158	0 520	4 5	15 0	2x2	0 525	1 00	1 38	0 861	0 90	1 82	1	1 756	1 17
11	32 5	0 035	2	4	0 158	0 520	4 5	15 0	2x4	0 525	1 20	1 25	0 826	0 87	1 95	1 5	1 596	1 31
12	32 5	0 035	3	3	0 158	0 520	4 5	15 0	3x3	0 525	1 25	1 11	0 815	0 86	1 95	2	1 419	1 40
13	32 5	0 035	1	4	0 200	0 658	5 7	15 0	1x4	0 525	0 95	2 39	0 917	0 96	2 43	1	3 047	1 512
14	32 5	0 035	2	2	0 200	0 658	5 7	15 0	2x2	0 525	0 90	1 68	0 890	0 92	2 29	0 95	2 138	1 23
15	32 5	0 035	2	4	0 200	0 658	5 7	15 0	2x4	0 525	1 00	1 55	0 862	0 90	2 51	1 1	1 978	1 26
16	32 5	0 035	3	3	0 200	0 658	5 7	15 0	3x3	0 525	1 10	1 38	0 853	0 89	2 51	1 3	1 759	1 28

Table A.3: Comparing design theories based on the results of Tajchman (1973) test on dense sand

	Test Parameters										Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile Width (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	Measured	Calculated Das 1998	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and Keeney 1944	K	η_h	Calculated Sayed and Bakeer (1992)
1	28	0 035	1	4	0 070	0 230	2 0	15 0	1x4	0 525	1 20	1 27	0 779	0 88	1 35	5	1 273	2 342
2	28	0 035	2	2	0 070	0 230	2 0	15 0	2x2	0 525	1 50	0 96	0 705	0 78	1 33	3	0 96	1 28
3	28	0 035	2	4	0 070	0 230	2 0	15 0	2x4	0 525	1 80	0 80	0 631	0 72	1 36	4	0 80	1.33
4	28	0 035	3	3	0 070	0 230	2 0	15 0	3x3	0 525	2 00	0 71	0 606	0 69	1 36	5	0 71	1 38
5	28	0 035	1	4	0 105	0 345	3 0	15 0	1x4	0 525	1 15	1 75	0 846	0 92	1 52	3 5	1 751	2 282
6	28	0 035	2	2	0 105	0 345	3 0	15 0	2x2	0 525	1 35	1 27	0 795	0 86	1 49	2 5	1 27	1 33
7	28	0 035	2	4	0 105	0 345	3 0	15 0	2x4	0 525	1 65	1 11	0 744	0 81	1 54	3 5	1 11	1 44
8	28	0 035	3	3	0 105	0 345	3 0	15 0	3x3	0 525	1 80	0 99	0 727	0 79	1 54	4 5	0 99	1 52
9	28	0 035	1	4	0 158	0 520	4 5	15 0	1x4	0 525	1 10	2 47	0 896	0 95	1 90	2 6	2 474	2 358
10	28	0 035	2	2	0 158	0 520	4 5	15 0	2x2	0 525	1 25	1 76	0 861	0 90	1 82	1 5	1 76	1 25
11	28	0 035	2	4	0 158	0 520	4 5	15 0	2x4	0 525	1 45	1 60	0 826	0 87	1 95	2 3	1 60	1 40
12	28	0 035	3	3	0 158	0 520	4 5	15 0	3x3	0 525	1 55	1 42	0 815	0 86	1 95	3	1 42	1 49
13	28	0 035	1	4	0 200	0 658	5 7	15 0	1x4	0 525	1 00	3 05	0 917	0 96	2 43	1 5	3 047	1 893
14	28	0 035	2	2	0 200	0 658	5 7	15 0	2x2	0 525	1 20	2 14	0 890	0 92	2 29	1	2 14	1 17
15	28	0 035	2	4	0 200	0 658	5 7	15 0	2x4	0 525	1 30	1 98	0 862	0 90	2 51	1 5	1 98	1 30
16	28	0 035	3	3	0 200	0 658	5 7	15 0	3x3	0 525	1 30	1 76	0 853	0 89	2 51	2	1 76	1 38

Table A.4: Comparing design theories based on the results of Tajchman (1973) test on loose sand

	Test Parameters											Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile diameter (D) m	Number of piles / row (N1)	Number of piles / column (N2)	Pile spacing (s) m	Pile Spacing (S) ft	S/D	Cap Condition	Pile Arrangement	L/D	Pile length (L) m	Measured	Calculated Das (1998)	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and keeney 1944	K	η_k	Calculated Sayed and Bakeer (1992)
1	30	0 375	1	2	0 56	1 850	1 5	freestanding	1x2	8	3	1 25	1 11	0 81	0 89	0 50	2 5	1 11	1 39
2	30	0 375	2	2	0 56	1 850	1 5	freestanding	2x2	8	3	0 8	0 80	0 63	0 71	0 28	3	0 64	1 20
3	30	0 375	2	3	0 56	1 850	1 5	freestanding	2x3	8	3	0 9	0 69	0 56	0 65	0 16	4	0 69	1 39
4	30	0 375	1	2	0 75	2 467	2 0	freestanding	1x2	8	3	1 15	1 27	0 85	0 92	0 72	2 5	1 27	1 48
5	30	0 375	2	2	0 75	2 467	2 0	freestanding	2x2	8	3	1	0 95	0 70	0 78	0 57	3	0 64	1 20
6	30	0 375	2	3	0 75	2 467	2 0	freestanding	2x3	8	3	0 85	0 85	0 66	0 74	0 49	4	0 85	1 53
7	30	0 375	1	2	0 94	3 084	2 5	freestanding	1x2	8	3	1 35	1 43	0 88	0 94	0 82	2 5	1 43	1 57
8	30	0 375	2	2	0 94	3 084	2 5	freestanding	2x2	8	3	0 9	1 11	0 76	0 83	0 70	3	0 80	1 31
9	30	0 375	2	3	0 94	3 084	2 5	freestanding	2x3	8	3	0 85	1 01	0 72	0 79	0 63	4	1 01	1 67
10	30	0 375	1	2	0 56	1 850	1 5	cap resting	1x2	8	3	1 4	1 11	0 81	0 89	0 50	2 5	1 11	1 39
11	30	0 375	2	2	0 56	1 850	1 5	cap resting	2x2	8	3	0 95	0 80	0 63	0 71	0 28	3	0 48	1 10
12	30	0 375	2	3	0 56	1 850	1 5	cap resting	2x3	8	3	0 75	0 69	0 56	0 65	0 16	4	0 69	1 39
13	30	0 375	1	2	0 75	2 467	2 0	cap resting	1x2	8	3	1 6	1 27	0 85	0 92	0 72	2 5	1 27	1 48
14	30	0 375	2	2	0 75	2 467	2 0	cap resting	2x2	8	3	1	0 95	0 70	0 78	0 57	3	0 64	1 20
15	30	0 375	2	3	0 75	2 467	2 0	cap resting	2x3	8	3	1 25	0 85	0 66	0 74	0 49	4	0 85	1 53
16	30	0 375	1	2	0 94	3 084	2 5	cap resting	1x2	8	3	2	1 43	0 88	0 94	0 82	2 5	1 43	1 57
17	30	0 375	2	2	0 94	3 084	2 5	cap resting	2x2	8	3	1 6	1 11	0 76	0 83	0 70	3	0 80	1 31
18	30	0 375	2	3	0 94	3 084	2 5	cap resting	2x3	8	3	1 5	1 01	0 72	0 79	0 63	4	1 01	1 67

Table A.5: Comparing design theories based on the results of Garg (1979) test

	Test Parameters											Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile diameter (D) m	Number of piles / row (N1)	Number of piles / column (N2)	Pile spacing (S) m	Pile spacing (S) ft	S/D	Cap Condition	Pile Arrangement	L/D	Pile length (L) m	Measured	Calculated Das (1998)	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler & Keiney 1944	K	η_k	Calculated Sayed and Bakeer (1992)
1	35	0.0127	2	2	0.0254	0.08	2.0	freestanding	2x2	22	0.28	1.7	0.95	0.70	0.78	1.16	2.5	0.96	1.31
2	35	0.0127	2	2	0.0508	0.17	4.0	freestanding	2x2	22	0.28	1.3	1.59	0.84	0.89	1.26	3	0.64	1.20
3	35	0.0127	2	2	0.0762	0.25	6.0	freestanding	2x2	22	0.28	1.0	2.23	0.89	0.93	1.36	4	2.23	2.74
4	35	0.0127	3	3	0.0254	0.08	2.0	freestanding	3x3	22	0.28	1.6	0.71	0.61	0.69	1.16	2.5	0.71	1.17
5	35	0.0127	3	3	0.0508	0.17	4.0	freestanding	3x3	22	0.28	1.4	1.27	0.79	0.84	1.27	3	1.27	1.62
6	35	0.0127	3	3	0.0762	0.25	6.0	freestanding	3x3	22	0.28	1.1	1.84	0.86	0.90	1.39	4	1.84	2.40
7	35	0.0127	2	2	0.0254	0.08	2.0	cap resting	2x2	24	0.30	2.0	0.95	0.70	0.78	1.16	2.5	0.96	1.31
8	35	0.0127	2	2	0.0508	0.17	4.0	cap resting	2x2	24	0.30	2.5	1.59	0.84	0.89	1.26	3	1.27	1.62
9	35	0.0127	2	2	0.0762	0.25	6.0	cap resting	2x2	24	0.30	4.3	2.23	0.89	0.93	1.36	4	2.23	2.74
10	35	0.0127	3	3	0.0254	0.08	2.0	cap resting	3x3	24	0.30	2.7	0.71	0.61	0.69	1.16	2.5	0.71	1.17
11	35	0.0127	3	3	0.0508	0.17	4.0	cap resting	3x3	24	0.30	3.8	1.27	0.79	0.84	1.27	3	1.27	1.62
12	35	0.0127	3	3	0.0762	0.25	6.0	cap resting	3x3	24	0.30	6.0	1.84	0.86	0.90	1.39	4	1.84	2.40

Table A.6: Comparing design theories based on the results of Kishida and Meyerhof (1976) test on loose sand

	Test Parameters										Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile diameter (D) m	Number of piles / row (N1)	Number of piles / column (N2)	Pile spacing (S) m	Pile spacing (S) ft	S/D	Cap Condition	Pile Arrangement	Pile length (L) m	Measured	Calculated Das (1998)	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler & Keiney	K	η_k	Calculated Sayed and Bakeer (1992)
1	43	0 0127	2	2	0 0254	0 08	2 0	freestanding	2x2	0 28	0 8	0 95	0 70	0 78	1 16	2 5	0 96	1 31
2	43	0 0127	2	2	0 0508	0 17	4 0	freestanding	2x2	0 28	0 9	1 59	0 84	0 89	1 26	3	0 64	1 20
3	43	0 0127	2	2	0 0762	0 25	6 0	freestanding	2x2	0 28	1 0	2 23	0 89	0 93	1 36	4	2 23	2 74
4	43	0 0127	3	3	0 0254	0 08	2 0	freestanding	3x3	0 28	0 7	0 71	0 61	0 69	1 16	2 5	0 71	1 17
5	43	0 0127	3	3	0 0508	0 17	4 0	freestanding	3x3	0 28	0 8	1 27	0 79	0 84	1 27	3	1 27	1 62
6	43	0 0127	3	3	0 0762	0 25	6 0	freestanding	3x3	0 28	1 0	1 84	0 86	0 90	1 39	4	1 84	2 40
7	43	0 0127	2	2	0 0254	0 08	2 0	cap resting	2x2	0 30	1 0	0 95	0 70	0 78	1 16	2 5	0 96	1 31
8	43	0 0127	2	2	0 0508	0 17	4 0	cap resting	2x2	0 30	2 6	1 59	0 84	0 89	1 26	3	1 27	1 62
9	43	0 0127	2	2	0 0762	0 25	6 0	cap resting	2x2	0 30	3 5	2 23	0 89	0 93	1 36	4	2 23	2 74
10	43	0 0127	3	3	0 0254	0 08	2 0	cap resting	3x3	0 30	1 9	0 71	0 61	0 69	1 16	2 5	0 71	1 17
11	43	0 0127	3	3	0 0508	0 17	4 0	cap resting	3x3	0 30	4 5	1 27	0 79	0 84	1 27	3	1 27	1 62
12	43	0 0127	3	3	0 0762	0 25	6 0	cap resting	3x3	0 30	6 5	1 84	0 86	0 90	1 39	4	1 84	2 40

Table A.7: Comparing design theories based on the results of Kishida and Meyerhof (1976) test on dense sand

	Test Parameters										Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile diameter (D) m	Number of piles / row (N1)	Number of piles / column (N2)	Pile spacing (S) m	Pile spacing (S) ft	S/D	L/D	Pile Arrangement	Pile length (L) m	Measured	Calculated Das (1998)	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and Keeney 1944	K	η_k	Calculated Sayed and Bakeer (1992)
1	35	0.019	2	1	0.057	0.188	3.0	15.8	2 x 1	0.3	1.5	1.59	0.90	0.95	1.25	1.5	1.59	1.31
2	35	0.019	2	1.5	0.057	0.188	3.0	15.8	Triangle	0.3	2	1.38	0.83	0.89	1.27	2.5	0.64	1.13
3	35	0.019	2	2	0.057	0.188	3.0	15.8	2 x 2	0.3	2	1.27	0.80	0.86	1.28	3.5	1.27	1.76
4	35	0.019	2	1	0.076	0.250	4.0	15.8	2 x 1	0.3	1.45	1.91	0.92	0.96	1.31	2	1.91	1.62
5	35	0.019	2	1.5	0.076	0.250	4.0	15.8	Triangle	0.3	1.6	1.70	0.87	0.91	1.34	3	1.27	1.62
6	35	0.019	2	2	0.076	0.250	4.0	15.8	2 x 2	0.3	1.8	1.59	0.84	0.89	1.35	3.5	1.59	2.01
7	35	0.019	2	1	0.095	0.313	5.0	15.8	2 x 1	0.3	1.4	2.23	0.94	0.97	1.37	1	2.23	1.27
8	35	0.019	2	1.5	0.095	0.313	5.0	15.8	Triangle	0.3	1.5	2.02	0.90	0.93	1.41	1.5	1.59	1.31
9	35	0.019	2	2	0.095	0.313	5.0	15.8	2 x 2	0.3	1.7	1.91	0.87	0.91	1.44	2	1.91	1.62
10	35	0.019	2	1	0.114	0.375	6.0	15.8	2 x 1	0.3	1.35	2.55	0.95	0.97	1.44	0.95	2.55	1.31
11	35	0.019	2	1.5	0.114	0.375	6.0	15.8	Triangle	0.3	1.3	2.33	0.91	0.94	1.50	1	1.91	1.20
12	35	0.019	2	2	0.114	0.375	6.0	15.8	2 x 2	0.3	1.6	2.23	0.89	0.93	1.53	1.2	2.23	1.37
13	35	0.019	2	1	0.057	0.188	3.0	23.7	2 x 1	0.45	1.75	1.59	0.90	0.95	1.25	1.5	1.59	1.31
14	35	0.019	2	1.5	0.057	0.188	3.0	23.7	Triangle	0.45	1.95	1.38	0.83	0.89	1.27	2.5	0.64	1.13
15	35	0.019	2	2	0.057	0.188	3.0	23.7	2 x 2	0.45	1.75	1.27	0.80	0.86	1.28	3.5	1.27	1.76
16	35	0.019	2	1	0.076	0.250	4.0	23.7	2 x 1	0.45	2.2	1.91	0.92	0.96	1.31	2	1.91	1.62
17	35	0.019	2	1.5	0.076	0.250	4.0	23.7	Triangle	0.45	1.85	1.70	0.87	0.91	1.34	2.5	1.27	1.48
18	35	0.019	2	2	0.076	0.250	4.0	23.7	2 x 2	0.45	1.6	1.59	0.84	0.89	1.35	3	1.59	1.83
19	35	0.019	2	1	0.095	0.313	5.0	23.7	2 x 1	0.45	1.1	2.23	0.94	0.97	1.37	1	2.23	1.27
20	35	0.019	2	1.5	0.095	0.313	5.0	23.7	Triangle	0.45	1.1	2.02	0.90	0.93	1.41	1.5	1.59	1.31
21	35	0.019	2	2	0.095	0.313	5.0	23.7	2 x 2	0.45	1.1	1.91	0.87	0.91	1.44	2	1.91	1.62
22	35	0.019	2	1	0.114	0.375	6.0	23.7	2 x 1	0.45	1.1	2.55	0.95	0.97	1.44	0.95	2.55	1.31
23	35	0.019	2	1.5	0.114	0.375	6.0	23.7	Triangle	0.45	0.95	2.33	0.91	0.94	1.50	1	1.91	1.20
24	35	0.019	2	2	0.114	0.375	6.0	23.7	2 x 2	0.45	1.1	2.23	0.89	0.93	1.53	1.2	2.23	1.37
25	35	0.019	2	1	0.057	0.188	3.0	31.6	2 x 1	0.6	1.3	1.59	0.90	0.95	1.25	1.5	1.59	1.31
26	35	0.019	2	1.5	0.057	0.188	3.0	31.6	Triangle	0.6	1.25	1.38	0.83	0.89	1.27	2.5	0.64	1.13
27	35	0.019	2	2	0.057	0.188	3.0	31.6	2 x 2	0.6	1.5	1.27	0.80	0.86	1.28	3.5	1.27	1.76
28	35	0.019	2	1	0.076	0.250	4.0	31.6	2 x 1	0.6	1.5	1.91	0.92	0.96	1.31	2	1.91	1.62
29	35	0.019	2	1.5	0.076	0.250	4.0	31.6	Triangle	0.6	1.6	1.70	0.87	0.91	1.34	3	1.27	1.62
30	35	0.019	2	2	0.076	0.250	4.0	31.6	2 x 2	0.6	1.6	1.59	0.84	0.89	1.35	3.5	1.59	2.01
31	35	0.019	2	1	0.095	0.313	5.0	31.6	2 x 1	0.6	1.3	2.23	0.94	0.97	1.37	1	2.23	1.27
32	35	0.019	2	1.5	0.095	0.313	5.0	31.6	Triangle	0.6	1.1	2.02	0.90	0.93	1.41	1.5	1.59	1.31
33	35	0.019	2	2	0.095	0.313	5.0	31.6	2 x 2	0.6	1.15	1.91	0.87	0.91	1.44	2	1.91	1.62
34	35	0.019	2	1	0.114	0.375	6.0	31.6	2 x 1	0.6	1.25	2.55	0.95	0.97	1.44	0.95	2.55	1.31
35	35	0.019	2	1.5	0.114	0.375	6.0	31.6	Triangle	0.6	1.15	2.33	0.91	0.94	1.50	1	1.91	1.20
36	35	0.019	2	2	0.114	0.375	6.0	31.6	2 x 2	0.6	1.2	2.23	0.89	0.93	1.53	1.2	2.23	1.37

Table A.8: Comparing design theories based on the results of Chattopadhyay (1994) test

	Test Parameters										Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile width (D) m	Number of piles / row (N1)	Number of piles / column (N2)	Pile spacing (S) m	Pile spacing (S) ft	S/D	L/D	Pile Arrangement	Pile length (L) m	Measured	Calculated Das (1998)	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler & Keiney 1944	K	η_s	Calculated Sayed and Bakeer (1992)
1	30	0 1	1	4	0 2	0 66	2 0	20 0	1x4	2	2 1	1 27	0 78	0 88	2 43	2	1 00	1 22
2	30	0 1	2	2	0 2	0 66	2 0	20 0	2x2	2	2 1	0 95	0 70	0 78	2 29	4	0 75	1.44
3	30	0 1	1	4	0 3	0 99	3 0	20 0	1x4	2	1 8	1 75	0.85	0 92	45 55	1 75	1 38	1.31
4	30	0 1	2	2	0.3	0 99	3 0	20 0	2x2	2	2	1 27	0 80	0 86	40 62	3 5	1 00	1 55
5	30	0 1	1	4	0 4	1 32	4 0	20 0	1x4	2	1 5	2 23	0.88	0 94	-1 06	1 5	1 75	1.36
6	30	0 1	2	2	0 4	1 32	4 0	20 0	2x2	2	1 75	1 59	0 84	0 89	-0 81	3	1 25	1 61
7	30	0 1	1	4	0 6	1 97	6 0	20 0	1x4	2	1.05	3.18	0.92	0 96	0 26	1	2 50	1.33
8	30	0 1	2	2	0 6	1 97	6 0	20 0	2x2	2	1 1	2 23	0 89	0 93	0 36	1	1 75	1 17

Table A.9: Comparing design theories based on the results of Kezdi (1957) test

	Test Parameters											Group Efficiency							
Test number	Pile - Soil Friction Angle (δ)	Pile diameter (D) m	cap condition	Number of piles / row (N1)	Number of piles / column (N2)	Pile spacing (S) m	Pile spacing (S) ft	S/D	L/D	Pile Arrangement	Pile length (L) m	Measured	Calculated Das (1998)	Calculated Converse-Labarre	Calculated Los Angeles	Calculated Seiler and Keeney 1944	K	η_h	Calculated Sayed and Bakeer (1992)
1	30	0 25	cap resting	3	3	0 75	2 467	3 0	8 0	3x3	2	1 64	0 99	1 00	0 79	0 44	4	0 99	1 65
2	30	0 25	cap resting	3	3	0 75	2 467	3 0	13 0	3x3	3 25	1 69	0 99	1 00	0 79	0 44	4	0 99	1 65
9	30	0 25	cap resting	3	3	0 75	2 467	3 0	18 0	3x3	4 5	1 36	0 99	1 00	0 79	0 44	4	0 99	1 65
3	30	0 25	cap resting	3	3	0 75	2 467	3 0	23 0	3x3	5 75	1 15	0 99	1 00	0 79	0 44	4	0 99	1 65
5	30	0 25	cap resting	3	3	0 5	1 645	2 0	18 0	3x3	4 5	1 21	0 71	0 99	0 69	-0 16	5	0 71	1 56
6	30	0 25	cap resting	3	3	1	3 289	4 0	18 0	3x3	4 5	1 46	1 27	1 00	0 84	0 63	3	1 27	1 62
7	30	0 25	cap resting	3	3	1 5	4 934	6 0	18 0	3x3	4 5	2 23	1 84	1 00	0 90	0 78	1 5	1 84	1 39
8	30	0 25	Freestanding	3	3	0 5	1 645	2 0	18 0	3x3	4 5	1 05	0 71	0 99	0 69	-0 16	5	0 71	1 56
10	30	0 25	Freestanding	3	3	1	3 289	4 0	18 0	3x3	4 5	1 03	1 27	1 00	0 84	0 63	3	1 27	1 62
11	30	0 25	Freestanding	3	3	1 5	4 934	6 0	18 0	3x3	4 5	0 88	1 84	1 00	0 90	0 78	1 5	1 84	1 39
12	30	0 25	cap resting	1	4	0 75	2 467	3 0	18 0	1x4	4 5	1 49	1 75	1 00	0 92	0 49	4	1 75	2 32
16	30	0 25	cap resting	2	2	0 75	2 467	3 0	18 0	2x2	4 5	1 6	1 27	1 00	0 86	0 57	4	1 27	1 90
17	30	0 25	cap resting	1	6	0 75	2 467	3 0	18 0	1x6	4 5	1 41	1 80	1 00	0 91	0 41	4	1 80	2 37
13	30	0 25	cap resting	2	4	0 75	2 467	3 0	18 0	2x4	4 5	1 4	1 11	1 00	0 81	0 44	4	1 11	1 76
15	30	0 25	cap resting	4	4	0 75	2 467	3 0	18 0	4x4	4 5	1 19	0 80	0 99	0 76	0 38	4	0 80	1 48

Table A.10: Comparing design theories based on the results of Liu (1985) test

APPENDIX B

Data Used in Developing the ANN Model

Reference	Method of Installation	Cap Condition	Type of Loading	Type of Test	Soil Condition	Angle of Friction (ϕ)	Unit weight of soil (γ) in KN/m3	Pile length (L) in meters	Pile Diameter (D) in meters	Pile Cross Section	Pile Spacing (S)	L/D	S/D	Number of Piles (N)	Pile Arrangement	Group Efficiency (η)
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 2	20	2	4	1x4	2 1
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 3	20	3	4	1x4	1 8
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 4	20	4	4	1x4	1 5
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 6	20	6	4	1x4	1 05
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 2	20	2	4	2x2	2 1
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 3	20	3	4	2x2	2
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 4	20	4	4	2x2	1 75
Kezdi (1957)	Driven	Freestanding	Compression	Field test	Loose	30	14 5	2	0 1	square	0 6	20	6	4	2x2	1 1
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	0 28	0 013	circular	0 026	22	2	4	2x2	1 7
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	0 28	0 013	circular	0 052	22	4	4	2x2	1 3
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	0 28	0 013	circular	0 078	22	6	4	2x2	1
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	0 28	0 013	circular	0 026	22	2	9	3x3	1 6
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	0 28	0 013	circular	0 052	22	4	9	3x3	1 4
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	0 28	0 013	circular	0 078	22	6	9	3x3	1 1
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Dense	43	18	0 28	0 013	circular	0 026	22	2	4	2x2	0 8
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Dense	43	18	0 28	0 013	circular	0 052	22	4	4	2x2	0 9
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Dense	43	18	0 28	0 013	circular	0 078	22	6	4	2x2	1
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Dense	43	18	0 28	0 013	circular	0 026	22	2	9	3x3	0 7
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Dense	43	18	0 28	0 013	circular	0 052	22	4	9	3x3	0 8
Kishida & Meyerhof (1965)	Jacked	Freestanding	Compression	Lab test	Dense	43	18	0 28	0 013	circular	0 078	22	6	9	3x3	1
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 026	24	2	4	2x2	2
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 052	24	4	4	2x2	2 5
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 078	24	6	4	2x2	4 3
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 026	24	2	9	3x3	2 7
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 052	24	4	9	3x3	3 8
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 078	24	6	9	3x3	6
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 026	24	2	4	2x2	1
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 052	24	4	4	2x2	2 6
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 078	24	6	4	2x2	3 5
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 026	24	2	9	3x3	1 9
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 052	24	4	9	3x3	4 5
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 078	24	6	9	3x3	6 5
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	15 2	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 29
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	4	2x2	0 97
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 5	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 28
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 8	1 53	0 1	circular	0 3	15 3	3	4	2x2	1 33
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 7	1 53	0 1	circular	0 4	15 3	4	4	2x2	1 35
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 8	1 53	0 1	circular	0 6	15 3	6	4	2x2	1 29
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 4	1 53	0 1	circular	0 2	15 3	2	9	3x3	1 21
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	9	3x3	1 17
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	14 9	1 53	0 1	circular	0 3	15 3	3	9	3x3	1 37
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	15 2	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 29
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 02
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 48
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 8	1 53	0 1	circular	0 3	15 3	3	4	2x2	1 67
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 7	1 53	0 1	circular	0 4	15 3	4	4	2x2	1 72
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 8	1 53	0 1	circular	0 6	15 3	6	4	2x2	1 55
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 4	1 53	0 1	circular	0 2	15 3	2	9	3x3	1 32

Reference	Method of Installation	Cap Condition	Type of Loading	Type of Test	Soil Condition	Angle of Friction (ϕ)	Unit weight of soil (γ) in KN/m3	Pile length (L) in meters	Pile Diameter (D) in meters	Pile Cross Section	Pile Spacing (S)	L/D	S/D	Number of Piles (N)	Pile Arrangement	Group Efficiency (η)
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	9	3x3	1 18
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 9	1 53	0 1	circular	0 3	15 3	3	9	3x3	1 52
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 07	15	2	4	1x4	1 2
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 07	15	2	4	2x2	1 5
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 07	15	2	8	2x4	1 8
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 07	15	2	9	3x3	2
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 105	15	3	4	1x4	1 15
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 105	15	3	4	2x2	1 35
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 105	15	3	8	2x4	1 65
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 105	15	3	9	3x3	1 8
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 1575	15	4 5	4	1x4	1 1
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 1575	15	4 5	4	2x2	1 25
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 1575	15	4 5	8	2x4	1 45
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 1575	15	4 5	9	3x3	1 55
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 21	15	6	4	1x4	1
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 21	15	6	4	2x2	1 2
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 21	15	6	8	2x4	1 3
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Loose	28	14 5	0 525	0 035	square	0 21	15	6	9	3x3	1 3
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 07	15	2	4	1x4	0 8
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 07	15	2	4	2x2	1
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 07	15	2	8	2x4	1 3
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 07	15	2	9	3x3	1 35
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 105	15	3	4	1x4	0 85
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 105	15	3	4	2x2	1
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 105	15	3	8	2x4	1 25
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 105	15	3	9	3x3	1 3
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 1575	15	4 5	4	1x4	0 9
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 1575	15	4 5	4	2x2	1
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 1575	15	4 5	8	2x4	1 2
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 1575	15	4 5	9	3x3	1 25
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 21	15	6	4	1x4	0 95
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 21	15	6	4	2x2	0 9
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 21	15	6	8	2x4	1
Tejchman (1973)	Driven	Freestanding	Compression	Lab test	Dense	40	18	0 525	0 035	square	0 21	15	6	9	3x3	1 1
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 057	15 8	3	2	1x2	1 5
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 076	15 8	4	2	1x2	1 45
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 095	15 8	5	2	1x2	1 4
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 114	15 8	6	2	1x2	1 35
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 057	15 8	3	3	1 5x2	2
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 076	15 8	4	3	1 5x2	1 6
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 095	15 8	5	3	1 5x2	1 5
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 114	15 8	6	3	1 5x2	1 3
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 057	15 8	3	4	2x2	2
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 076	15 8	4	4	2x2	1 8
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 095	15 8	5	4	2x2	1 7
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 3	0 019	circular	0 114	15 8	6	4	2x2	1 6
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 057	23 7	3	2	1x2	1 75
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 076	23 7	4	2	1x2	2 2
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 095	23 7	5	2	1x2	1 1
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 114	23 7	6	2	1x2	1 1

Reference	Method of Installation	Cap Condition	Type of Loading	Type of Test	Soil Condition	Angle of Friction (ϕ)	Unit weight of soil (γ) in KN/m3	Pile length (L) in meters	Pile Diameter (D) in meters	Pile Cross Section	Pile Spacing (S)	L/D	S/D	Number of Piles (N)	Pile Arrangement	Group Efficiency (η)
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 038	23 7	2	3	1 5x2	1 85
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 076	23 7	4	3	1 5x2	1 85
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 095	23 7	5	3	1 5x2	1 1
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 114	23 7	6	3	1 5x2	0 95
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 038	23 7	2	4	2x2	1 75
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 076	23 7	4	4	2x2	1 6
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 095	23 7	5	4	2x2	1 1
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 45	0 019	circular	0 114	23 7	6	4	2x2	1 1
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 038	31 6	2	2	1x2	1 3
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 076	31 6	4	2	1x2	1 5
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 095	31 6	5	2	1x2	1 3
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 114	31 6	6	2	1x2	1 25
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 038	31 6	2	3	1 5x2	1 25
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 076	31 6	4	3	1 5x2	1 6
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 095	31 6	5	3	1 5x2	1 1
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 114	31 6	6	3	1 5x2	1 15
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 038	31 6	2	4	2x2	1 5
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 076	31 6	4	4	2x2	1 6
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 095	31 6	5	4	2x2	1 15
Chattopadhyay (1994)	Driven	Freestanding	Uplift	Lab test	Loose	35	14 5	0 6	0 019	circular	0 114	31 6	6	4	2x2	1 2
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 0762	23 6	3	2	1x2	0 59
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 1016	23 6	4	2	1x2	0 77
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 127	23 6	5	2	1x2	0 89
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 0762	23 6	3	3	1x3	0 44
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 1016	23 6	4	3	1x3	0 68
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 127	23 6	5	3	1x3	0 82
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 0762	23 6	3	4	2x2	0 51
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 1016	23 6	4	4	2x2	0 64
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 127	23 6	5	4	2x2	0 8
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 0762	23 6	3	6	2x3	0 47
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 1016	23 6	4	6	2x3	0 61
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 127	23 6	5	6	2x3	0 75
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 0762	23 6	3	9	3x3	0 45
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 1016	23 6	4	9	3x3	0 59
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 6	0 0254	circular	0 127	23 6	5	9	3x3	0 72
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 0762	29 5	3	2	1x2	0 57
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 1016	29 5	4	2	1x2	0 75
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 127	29 5	5	2	1x2	0 88
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 0762	29 5	3	3	1x3	0 50
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 1016	29 5	4	3	1x3	0 67
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 127	29 5	5	3	1x3	0 81
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 0762	29 5	3	4	2x2	0 48
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 1016	29 5	4	4	2x2	0 62
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 127	29 5	5	4	2x2	0 79
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 0762	29 5	3	6	2x3	0 46
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 1016	29 5	4	6	2x3	0 59
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 127	29 5	5	6	2x3	0 73
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 0762	29 5	3	9	3x3	0 43
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 1016	29 5	4	9	3x3	0 58
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15 89	0 75	0 0254	circular	0 127	29 5	5	9	3x3	0 71

Reference	Method of Installation	Cap Condition	Type of Loading	Type of Test	Soil Condition	Angle of Friction (ϕ)	Unit weight of soil (γ) in KN/m3	Pile length (L) in meters	Pile Diameter (D) in meters	Pile Cross Section	Pile Spacing (S)	L/D	S/D	Number of Piles (N)	Pile Arrangement	Group Efficiency (η_f)
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.0762	35.4	3	2	1x2	0.57
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.1016	35.4	4	2	1x2	0.73
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.127	35.4	5	2	1x2	0.87
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.0762	35.4	3	3	1x3	0.49
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.1016	35.4	4	3	1x3	0.65
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.127	35.4	5	3	1x3	0.80
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.0762	35.4	3	4	2x2	0.47
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.1016	35.4	4	4	2x2	0.62
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.127	35.4	5	4	2x2	0.78
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.0762	35.4	3	6	2x3	0.44
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.1016	35.4	4	6	2x3	0.58
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.127	35.4	5	6	2x3	0.72
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.0762	35.4	3	9	3x3	0.43
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.1016	35.4	4	9	3x3	0.56
Mukherjee (1996)	Driven	Freestanding	Uplift	Lab test	Dense	37	15.89	0.9	0.0254	circular	0.127	35.4	5	9	3x3	0.70
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.225	20	1.5	2	1x2	1.25
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.3	20	2	2	1x2	1.15
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.375	20	2.5	2	1x2	1.35
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.225	20	1.5	4	2x2	0.8
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.3	20	2	4	2x2	1
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.375	20	2.5	4	2x2	0.9
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.225	20	1.5	6	2x3	0.9
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.3	20	2	6	2x3	0.85
Garg (1979)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.375	20	2.5	6	2x3	0.8
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.225	20	1.5	2	1x2	1.4
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.3	20	2	2	1x2	1.6
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.375	20	2.5	2	1x2	2
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.225	20	1.5	4	2x2	0.95
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.3	20	2	4	2x2	1
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.375	20	2.5	4	2x2	1.6
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.225	20	1.5	6	2x3	0.75
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.3	20	2	6	2x3	1.25
Garg (1979)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3	0.15	circular	0.375	20	2.5	6	2x3	1.5
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	2	0.25	circular	0.75	8.0	3.0	9	3x3	1.64
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	3.25	0.25	circular	0.75	13.0	3.0	9	3x3	1.69
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.75	18.0	3.0	9	3x3	1.36
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	5.75	0.25	circular	0.75	23.0	3.0	9	3x3	1.15
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.5	18.0	2.0	9	3x3	1.21
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	1	18.0	4.0	9	3x3	1.46
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	1.5	18.0	6.0	9	3x3	2.23
Liu (1985)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.5	18.0	2.0	9	3x3	1.05
Liu (1985)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	1	18.0	4.0	9	3x3	1.03
Liu (1985)	Bored	Freestanding	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	1.5	18.0	6.0	9	3x3	0.88
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.75	18.0	3.0	4	1x4	1.49
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.75	18.0	3.0	4	2x2	1.6
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.75	18.0	3.0	6	1x6	1.41
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.75	18.0	3.0	8	2x4	1.4
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14.5	4.5	0.25	circular	0.75	18.0	3.0	16	4x4	1.19

Table B.1: The entire set of data obtained from the literature

Reference	Method of Installation	Cap Condition	Type of Loading	Type of Test	Soil Condition	Angle of Friction (φ)	Unit weight of soil (γ) in KN/m3	Pile length (L) in meters	Pile Diameter (D) in meters	Pile Cross Section	Pile Spacing (S)	L/D	S/D	Number of Piles (N)	Pile Arrangement	Group Efficiency (η)
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 026	24	2	4	2x2	2
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 052	24	4	4	2x2	2 5
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 078	24	6	4	2x2	4 3
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 026	24	2	9	3x3	2 7
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 052	24	4	9	3x3	3 8
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Loose	35	14 5	0 3	0 013	circular	0 078	24	6	9	3x3	6
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 026	24	2	4	2x2	1
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 052	24	4	4	2x2	2 6
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 078	24	6	4	2x2	3 5
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 026	24	2	9	3x3	1 9
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 052	24	4	9	3x3	4 5
Kishida & Meyerhof (1965)	Jacked	Direct contact	Compression	Lab test	Dense	43	18	0 3	0 013	circular	0 078	24	6	9	3x3	6 5
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	15 2	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 29
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	4	2x2	0 97
Vesic (1969)	Jacked	Freestanding	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	9	3x3	1 17
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	15 2	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 29
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	4	2x2	1 02
Vesic (1969)	Jacked	Direct contact	Compression	Lab test	Loose	35	15	1 53	0 1	circular	0 2	15 3	2	9	3x3	1 18
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14 5	4 5	0 25	circular	0 75	18 0	3 0	16	4x4	1 19
Liu (1985)	Bored	Direct contact	Compression	Field test	Loose	30	14 5	4 5	0 25	circular	0 75	18 0	3 0	6	1x6	1 41

Table B.2: Data records removed due to their inconsistency

No.	Install	Cap	Loading	Test	Soil	Section	Arrangement	L/D	S/D	Efficiency
1	Bored	Freestanding	Compression	Field	Loose	Circular	2x2	20	1.5	0.8
2	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	23.6	5	0.82
3	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	23.6	5	0.75
4	Jacked	Direct contact	Compression	Lab	Loose	Circular	3x3	15.3	3	1.52
5	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	18.0	6.0	2.23
6	Bored	Direct contact	Compression	Field	Loose	Circular	1x4	18.0	3.0	1.49
7	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	13.0	3.0	1.69
8	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	23.6	3	0.44
9	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	23.0	3.0	1.15
10	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	15.8	5	1.4
11	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	22	4	1.3
12	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	35.4	4	0.62
13	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	29.5	3	0.57
14	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	23.7	5	1.1
15	Driven	Freestanding	Compression	Lab	Loose	Square	2x2	15	3	1.35
16	Driven	Freestanding	Compression	Lab	Dense	Square	2x4	15	2	1.3
17	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	15.8	3	2
18	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	35.4	4	0.55
19	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	23.7	6	1.1
20	Bored	Freestanding	Compression	Field	Loose	Circular	3x3	18.0	2.0	1.05
21	Driven	Freestanding	Compression	Lab	Dense	Square	3x3	15	2	1.35
22	Bored	Freestanding	Compression	Field	Loose	Circular	2x2	20	2	1
23	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	29.5	4	0.62
24	Driven	Freestanding	Compression	Lab	Dense	Square	1x4	15	6	0.95
25	Jacked	Freestanding	Compression	Lab	Dense	Circular	3x3	22	2	0.7
26	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	15.8	4	1.45
27	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	35.4	4	0.73
28	Jacked	Freestanding	Compression	Lab	Dense	Circular	3x2	22	4	0.9
29	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	29.5	3	0.43
30	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	22	2	1.7
31	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	18.0	4.0	1.46
32	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	35.4	5	0.70
33	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	31.6	5	1.1
34	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	35.4	3	0.49
35	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	29.5	5	0.79
36	Driven	Freestanding	Compression	Field	Loose	Square	2x2	20	6	1.1
37	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	23.7	2	1.75
38	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	35.4	3	0.43
39	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	31.6	6	1.2
40	Driven	Freestanding	Compression	Lab	Loose	Square	1x4	15	4.5	1.1
41	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	29.5	4	0.67
42	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	15.8	3	1.5
43	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	23.6	3	0.45
44	Driven	Freestanding	Compression	Field	Loose	Square	2x2	20	3	2
45	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	29.5	4	0.75
46	Driven	Freestanding	Compression	Lab	Dense	Square	2x2	15	6	0.9
47	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	35.4	3	0.47
48	Bored	Direct contact	Compression	Field	Loose	Circular	1x2	20	2.5	2
49	Driven	Freestanding	Compression	Lab	Loose	Square	3x3	15	4.5	1.55
50	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	23.7	4	1.85
51	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	29.5	5	0.88
52	Jacked	Freestanding	Compression	Lab	Loose	Circular	3x3	15.3	2	1.21
53	Driven	Freestanding	Compression	Lab	Dense	Square	2x2	15	4.5	1
54	Driven	Freestanding	Compression	Lab	Loose	Square	3x3	15	6	1.3
55	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	23.6	3	0.47
56	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	23.6	4	0.61
57	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	15.3	6	1.29
58	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	31.6	5	1.15
59	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	35.4	5	0.87
60	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	31.6	4	1.6
61	Bored	Freestanding	Compression	Field	Loose	Circular	2x3	20	1.5	0.9
62	Driven	Freestanding	Compression	Lab	Dense	Square	1x4	15	2	0.8
63	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	23.6	5	0.8
64	Bored	Direct contact	Compression	Field	Loose	Circular	2x4	18.0	3.0	1.4
65	Bored	Direct contact	Compression	Field	Loose	Circular	2x2	20	1.5	0.95
66	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	35.4	5	0.80
67	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	22	6	1
68	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	15.3	3	1.33
69	Jacked	Freestanding	Compression	Lab	Loose	Circular	3x3	22	6	1.1
70	Jacked	Freestanding	Compression	Lab	Loose	Circular	3x3	15.3	3	1.37
71	Driven	Freestanding	Compression	Lab	Loose	Square	1x4	15	6	1
72	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	31.6	2	1.5
73	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	29.5	3	0.48
74	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	35.4	3	0.44

No.	Install	Cap	Loading	Test	Soil	Section	Arrangement	L/D	S/D	Efficiency
75	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	31.6	2	1.3
76	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	31.6	6	1.15
77	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	29.5	5	0.81
78	Bored	Freestanding	Compression	Field	Loose	Circular	3x3	18.0	4.0	1.03
79	Driven	Freestanding	Compression	Lab	Dense	Square	1x4	15	3	0.85
80	Bored	Freestanding	Compression	Field	Loose	Circular	2x2	20	2.5	0.9
81	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	29.5	5	0.73
82	Driven	Freestanding	Compression	Lab	Loose	Square	1x4	15	3	1.15
83	Jacked	Freestanding	Compression	Lab	Loose	Circular	3x3	22	4	1.4
84	Bored	Direct contact	Compression	Field	Loose	Circular	2x2	20	2	1
85	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	23.7	6	0.95
86	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	23.6	3	0.59
87	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	15.8	6	1.35
88	Bored	Freestanding	Compression	Field	Loose	Circular	1x2	20	2.5	1.35
89	Driven	Freestanding	Compression	Lab	Dense	Square	2x2	15	3	1
90	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	15.8	3	2
91	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	15.8	6	1.3
92	Bored	Freestanding	Compression	Field	Loose	Circular	2x3	20	2.5	0.8
93	Jacked	Direct contact	Compression	Lab	Loose	Circular	2x2	15.3	2	1.48
94	Driven	Freestanding	Compression	Lab	Dense	Square	1x4	15	4.5	0.9
95	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	35.4	4	0.58
96	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	23.7	2	1.95
97	Driven	Freestanding	Compression	Field	Loose	Square	1x4	20	3	1.8
98	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	23.6	5	0.89
99	Bored	Freestanding	Compression	Field	Loose	Circular	1x2	20	1.5	1.25
100	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	35.4	3	0.57
101	Driven	Freestanding	Compression	Lab	Loose	Square	3x3	15	2	2
102	Driven	Freestanding	Compression	Lab	Dense	Square	2x2	15	2	1
103	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	15.8	5	1.7
104	Driven	Freestanding	Compression	Lab	Loose	Square	3x3	15	3	1.8
105	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	15.8	5	1.5
106	Driven	Freestanding	Compression	Field	Loose	Square	1x4	20	2	2.1
107	Bored	Direct contact	Compression	Field	Loose	Circular	2x3	20	1.5	0.75
108	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	31.6	2	1.25
109	Driven	Freestanding	Compression	Field	Loose	Square	2x2	20	4	1.75
110	Bored	Direct contact	Compression	Field	Loose	Circular	2x2	18.0	3.0	1.6
111	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	23.7	3	1.75
112	Driven	Freestanding	Compression	Lab	Dense	Square	3x3	15	3	1.3
113	Bored	Direct contact	Compression	Field	Loose	Circular	1x2	20	2	1.6
114	Driven	Freestanding	Compression	Lab	Loose	Square	2x2	15	6	1.2
115	Jacked	Freestanding	Compression	Lab	Dense	Circular	3x3	22	6	1
116	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	23.7	4	1.6
117	Driven	Freestanding	Compression	Lab	Loose	Square	2x4	15	3	1.65
118	Driven	Freestanding	Compression	Lab	Dense	Square	2x4	15	6	1
119	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	31.6	4	1.5
120	Driven	Freestanding	Uplift	Lab	Loose	Circular	1.5x2	15.8	4	1.6
121	Bored	Direct contact	Compression	Field	Loose	Circular	2x3	20	2.5	1.5
122	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	29.5	3	0.50
123	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	29.5	4	0.58
124	Jacked	Direct contact	Compression	Lab	Loose	Circular	2x2	15.3	3	1.67
125	Bored	Freestanding	Compression	Field	Loose	Circular	2x3	20	2	0.85
126	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	8.0	3.0	1.64
127	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	23.6	4	0.64
128	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	35.4	4	0.56
129	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x2	23.6	4	0.77
130	Bored	Direct contact	Compression	Field	Loose	Circular	2x2	20	2.5	1.6
131	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	23.7	5	1.1
132	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	23.7	5	1.1
133	Driven	Freestanding	Compression	Field	Loose	Square	1x4	20	4	1.5
134	Driven	Freestanding	Compression	Field	Loose	Square	2x2	20	2	2.1
135	Driven	Freestanding	Compression	Lab	Loose	Square	2x4	15	2	1.8
136	Jacked	Freestanding	Compression	Lab	Dense	Circular	3x3	22	4	0.8
137	Jacked	Direct contact	Compression	Lab	Loose	Circular	2x2	15.3	4	1.72
138	Driven	Freestanding	Compression	Lab	Loose	Square	2x2	15	4.5	1.25
139	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	31.6	6	1.25
140	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	18.0	2.0	1.21
141	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	35.4	5	0.72
142	Jacked	Direct contact	Compression	Lab	Loose	Circular	2x2	15.3	6	1.55
143	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	35.4	5	0.78
144	Bored	Direct contact	Compression	Field	Loose	Circular	1x2	20	1.5	1.4
145	Jacked	Freestanding	Compression	Lab	Dense	Circular	2x2	22	2	0.8
146	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	31.6	5	1.3
147	Driven	Freestanding	Compression	Lab	Loose	Square	2x2	15	2	1.5
148	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	23.6	5	0.72
149	Jacked	Direct contact	Compression	Lab	Loose	Circular	3x3	15.3	2	1.32
150	Driven	Freestanding	Compression	Field	Loose	Square	1x4	20	6	1.05
151	Driven	Freestanding	Compression	Lab	Loose	Square	1x4	15	2	1.2
152	Driven	Freestanding	Compression	Lab	Loose	Square	2x4	15	6	1.3
153	Bored	Direct contact	Compression	Field	Loose	Circular	2x3	20	2	1.25

Table B.3: The data set used for training and testing the developed ANN model

No.	Install	Cap	Loading	Test	Soil	Section	Arrangement	L/D	S/D	Efficiency
1	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	15.8	6	1.6
2	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	23.7	6	1.1
3	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	15.3	2	1.28
4	Driven	Freestanding	Compression	Lab	Dense	Square	3x3	15	4.5	1.25
5	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	29.5	3	0.46
6	Driven	Freestanding	Uplift	Lab	Dense	Circular	1x3	23.6	4	0.68
7	Jacked	Freestanding	Compression	Lab	Loose	Circular	3x3	22	2	1.6
8	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x2	23.6	3	0.51
9	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	29.5	5	0.71
10	Driven	Freestanding	Uplift	Lab	Loose	Circular	2x2	15.8	4	1.8
11	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	23.7	4	2.2
12	Driven	Freestanding	Compression	Lab	Loose	Square	2x4	15	4.5	1.45
13	Bored	Freestanding	Compression	Field	Loose	Circular	1x2	20	2	1.15
14	Driven	Freestanding	Compression	Lab	Dense	Square	2x4	15	4.5	1.2
15	Bored	Freestanding	Compression	Field	Loose	Circular	3x3	18.0	6.0	0.88
16	Jacked	Freestanding	Compression	Lab	Loose	Circular	2x2	15.3	4	1.35
17	Jacked	Freestanding	Compression	Lab	Dense	Circular	2x2	22	6	1
18	Driven	Freestanding	Compression	Lab	Dense	Square	3x3	15	6	1.1
19	Driven	Freestanding	Uplift	Lab	Dense	Circular	3x3	23.6	4	0.59
20	Driven	Freestanding	Compression	Lab	Dense	Square	2x4	15	3	1.25
21	Driven	Freestanding	Uplift	Lab	Dense	Circular	2x3	29.5	4	0.59
22	Driven	Freestanding	Uplift	Lab	Loose	Circular	1x2	31.6	4	1.5
23	Bored	Direct contact	Compression	Field	Loose	Circular	3x3	18.0	3.0	1.36

Table B.4: Data records used in validating the developed ANN model