# A MODEL STUDY OF PILE GROUPS IN CLAY

FARUK BUDAK

June, 1986

Boğaziçi University

A MODEL STUDY OF PILE GROUPS

IN CLAY

by

FARUK BUDAK

B.S. in C.E., Boğaziçi University, 1983

Submitted to the Institute for Graduate Studies in Science

and Engineering in partial fulfillment of

the requirements for the degree

Master of Science

in

Civil Engineering



Boğaziçi University

1986

A MODEL STUDY OF PILE GROUPS

Approved by:

Date of approval: 2 June1986

Examination Committee:

Prof.Dr. Ergün TOĞROL

Doç.Dr. Turan DURGUNOĞLU

Doç.Dr. Turgut ERSOY

Doç.Dr. Erol GÜLER
(Thesis supervisor)

Doç. Dr. Vural ALTIN

BOĞAZİCİ UNİVERSİTY

Bebek, İstanbul

# ACKNOWLEDGEMENTS

I am indebted to Doç. Dr. Erol GÜLER for overall supervision of the study, consistent encouragement and helpful criticism in preparetion of this thesis.

I acknowledge the help offered by the laboratory technician Erdal YILMAZ and vital advice from Mech. Engineer Mikdat DEDE.

I am also grateful to the officials of Security Service of İstinye Shipyard for their help in developing the model system.

FARUK BUDAK

İstanbul, June 1986

ABSTRACT

A model scale study of the bearing capacity of single piles and pile groups with a small L/D ratio in clay under vertical and axial loads has been performed. It was investigated how the pile spacing, driving energy, number of piles and time factor effects the bearing capacity of pile groups. Experimental results have been compared with the theoretical values. In this study for the interpretation of test results, a statistical analysis method called Latin Square was used.

# ÖZET

Kil zemin içerisinde kazık gruplarının taşıma gücü hakkında bir model çalışma yapılmıştır. Kazıklar arasındaki mesafenin, çakma enerjisinin, gruptaki kazık sayısının ve zaman factörünün taşıma gücünü nasıl ve ne oranda etkilediği araştırıldı. Deneysel sonuçlar teorik değerlerle karşılaştırıldı. Bu çalışmada, Latin karesi adı verilen bir istatistik analiz yöntemi deneylerin yorumlanmasında kullanıldı.

TABLE OF CONTENTS

| | PAGE |
|---|---|
| ACKNOWLEDGEMENTS | v |
| ABSTRACT | vi |
| özet | vii |
| LIST OF FİGURES | viii |
| I. INTRODUCTION | 1 |
| II. A STATE OF ART SUMMARY | 3 |
| 2.1 Summary | 3 |
| 2.2 General | 3 |
| 2.2.1 Introduction | 3 |
| 2,2.2 Definition of piles | 4 |
| 2,2.3 Pile installation | 5 |
| 2.2.4 Pile driving formulae | 6 |
| 2.3 A review of previous research on single piles | 8 |
| 2.4 A review of previous research on pile groups | 19 |
| III. METHOD OF THE MODEL STUDY | 27 |
| 3.1 General | 27 |
| 3.2 Soil bed | 31 |
| 3.2.1 Simulation of half space | 31 |
| 3.2.2 The sub-soil material and preparation of the bed | 32 |
| 3.2.3 Uniformity of the bed | 33 |
| 3.2.4 Properties of the soil | 33 |
| 3.3 Measurement of the load-displacement mechanism | 36 |
| 3.3.1 General | 36 |
| 3.4.2 Slow ML test | 37 |

3.4.3 Swedish Cyclic test 37

3.4.4 CRP test 38

3.5 Driving equipment 39

3.6 Testing programme 40

3.6.1 Major topics 40

3.6.2 The setting up of Latin Square 40

3.6.3 Testing routine 42

IV. INTERPRETATİON OF RESULTS AND COMPARİSON 46

4.1 Introduction 46

4.2 Single piles 46

4.2.1 General 46

4.2.2 Theoretical bearing capacity of a single model pile 46

4.2.3 Test results 47

4.3 Pile groups 48

4.3.1 The tests of 2x2 pile group 48

4.3.2 The tests of 3x3 pile group 49

4.4 Efficiency in groups 50

4.5 Block failure of model pile groups 51

4.6 Results of Latin Square 52

4.7 Comparison of the theoretical and experimental results 56

4.8 Regions of Linear-elastic behaviour 58

V. CONCLUSİONS 63

APPENDIX I. Calculation of Compaction energy 65

APPENDIX II. Water content change along the depth 66

APPENDIX III. Hydrometer anaļysis of the clay 67

APPENDIX IV. Calculation of driving energy 68

APPENDIX V. Figures 70

REFERENCES 80

LIST OF FIGURES


| number | p age |
| --- | --- |
| 1. The load-displacement curve of first single pile test. | 70 |
| 2. The load-displacement curve of second single pile tests | 71 |
| 3. The load-displacement curve of third single pile test. | 72 |
| 4. The load-displacement curves of single pile tests. | 60 |
| 5. The load-displacement curve of first 2x2 pile group test. | 73 |
| 6. The load-displacement curve of second 2x2 pile group test. | 74 |
| 7. The load-displacement curve of third 2x2 pile group test. | 75 |
| 8. The load-displacement curves of 2x2 pile group tests. | 61 |
| 9. The load-displacement curve of first 3x3 pile group test. | 76 |
| 10. The load-displacement curve of second 3x3 pile group test. | 77 |
| 11. The load-displacement curve of third 3x3 pile group test. | 78 |
| 12. The load-displacement curves of 3x3 pile group tests. | 62 |
| 13. The load-displacement curve of 2x2 pile group with 5D spacing | 79 |

# CHAPTER L: INTRODUCTION

## 1.1. GENERAL

Piled foundations have been used for well beyond the last thousand years. "Piled foundations are universally accepted as the traditional form of foundation in 'bad' condition........ Numereous examples are known of failure of piled foundation but the successful ones must also be remembered, most of the ancient city of Amsterdam, dating from fourteenth century, is still standing on original wooden piles...."(Little 1961). Also, many o of the monumental buildings along the shores of Istanbul are standing on wooden piles.Before the seventeenth century, since there was an abounding supply of timber and cheap labour, as many piles were driven as the ground would take. Resulting settlements were of little concern, since the prevalant types of structure could withstand considerable amounts of differential settlement without any damage.( Terzaghi and Peck 1948). The industrial revolution created a demand for heavy but expensive structures within which the cost of piled foundations became an item of consequence. Since then, various empiriçal pile driving formulae have been developed in order to forecast the minimum number of piles necassary to support a given structural load.

## 1.2. IMPORTANCE OF MODEL STUDIES

In Engineering experimenting is the best and most reliable method to obtain enlightening results and fruitful conclusions.

An engineer always takes into account two things, mainly: safety and economy. In many cases experiments have proven to be inapplicable for the main reason of their being uneconomical. But this disadvantage of experimenting is overcome by the introduction of models. Models serve the purpose of experimenting and are more advantageous due to economy, ease of handling and availability at any time.

The cost of making full-scale tests of groups and the need for a bed of homogenous soil of considerable depth prevents full-scale comparative tests of groups with different numbers and spacing of piles. Therefore model piles and model groups are more preferable in Foundation Engineering.

In spite of uncertainity concerning the scale effect between soil models and prototypes, model tests have been used in various investigations of group action in order to overcome the problems causedby high loading, inconsistency of sub-soil material, and the diffuculty of controlled-testing procedures associated in the field. The major published model-scale investigations in clay were carried out by Whitaker (1957,1960), Sowers (1961), and Saffery and Tate (1961). All these tests were carried out to investigate the performance of pile groups under ultimate load conditions and consequently focus on the prediction of failure loads.

This study was designed to investigate the ultimate bearing capacity (UBC) of model single piles and pile groups with asmall L/D ratio in clay, under vertical and centric loading. Results obtained have been compared with the various idealised analytical and experimental results available and agreement has been found. The behaviour of single piles under axial load and the interaction problem between closely spaced piles have been investigated.

design systems have necessarily beeh developed from experiences depending on emprical assumptions coupled with attemped generalizations of sets of consistence.

During the past two decades important advances have been made in investigation technique using instrumented piles to record the manner in which friction piles transfer their load to the supporting soil( Tomlinson 1970, Cooke and Price 1973, Butteṙfield and Johnston 1973, Kuizomi and Ito 1967 mainly ).

2.2.2. DEFINITION OF PILES

Essentially a pile is an elongated body ( or columnar ) installed in the ground for the purpose of transmitting forces to the ground.

The purpose of any foundation is to transmit loads orforces to the ground without excessive settlement. A piled foundation is used where it is necessary to carry the load to an underlying stratum through a weak or compressible material or through water. In a typical case the decision to use piling would probably be made if the site investigation showed a bed of rock, gravel or compact sand beneath depozits of alluvial silt, soft clay or peat which too expensive to remove or to excavate through.

When a pile carries a substantially axial force directed on to its head, as in the case of a vertical pile beneath the building, it is called a "bearing pile". Piles are also used for resisting horizantal forces or moments as in a dolphin or a port. Where they are called upon to resist upward forces they may be called tension or anchor piles. "Sheet" piles are installed in rows and are shaped so that the sides of each pile interlock with those of its neighbours to form a continuous bulkhead.

Piling is often used in deep beds of clay. The pile is supported in this case mainly by the adhesion or frictional action of the clay on the surface of the pile shaft. Such piles are termed "friction piles".

All piles obtain support from both the frictional forces on the surface of their shaft and from direct bearing on their bases or points, but generally one of these components and the division into "end-bearing" and "friction" piles is simply a convenient terminology.

2.2.3. PILE INSTALLATION

To describe in detail the various forms of piling equipment would be lenghty and out of keeping with the main purpose of this study. However, a short summary will be given.

Drop hammers are widely used for driving piles, the hammer being lifted on a rope by a which and allowed to fall onto pile head by releasing a clutch on the winding drum, the falling hammer dragging the rope and reversing the motion of the drum. Sometimes provision is made for the hammer to be released from the rope by a trigger, allowing it to fall freely.

Power hammers are operated by steam , compressed air or internal combustion (diesel).

Pile driving by vibration has been employed during the last two decades as an alternative to hammens. In this method the pile is vibrated in a vertical direction by a unit rigidly connected to the pile head. The vibration is comminicated to the soil immediately around the pile, causing a reduction in shear strength and the pile sinks into the ground under its own weight and that of the vibrator unit.

If the driving is carried out incorrectly, breaking or crushing of the pile can occur. Steel piles can be damaged at the pile head and if overdriven against an obstruction, the lower end may be computed or bent out of the straight.

To prevent damage of the head of a concrete pile from hammer blows a "helmet" is used, often called a "cushion" in contact with the concrete.

THe choice of hammer weight depends on the plant in use, but with drop hammers and single acting hammers, it is desirable that the weight of the hammer should be at least half that of pile

## 2.2.4. PILE DRIVING FORMULAE

The effort needed by anyone driving a stake or a rod into the ground depends on the "resistance" of the ground. For nearly two centuries engineers have applied this idea to pile driving and many mathematical expressions termed "driving formulae" or "dynamic formulae" have been devised for calculating the resistance.

Driving formulae are simple idealizations of a complex event. They are based on the action of the hammer on the pile in the last stage of its embedment and this can be presented by some simple mechanicle principle.

It is assumed that:

(a) The hammer and the pile may be treated as impinging particles,

(b) the hammer gives up its all energy on impact,

(c) on impact a resistance R to the motion of the pile is immetiadetely generated which remains constant while the pile moves a distance's!

The available energy of the hammer is WH and the work done in overcoming the resistance is'Rs', so that:

$$W.H=R.s$$

where W: weight of the hammer

H: height of fall of the hammer

R: the driving resistance

s: the net distance the pile is driven by a blow.

This is an elemantary formula known as the Sanders formula.

A.M.Wellington in 1888 assumed that:

"Under the hammer blow the resistance increases to the value R in an elastic manner as the pile is displaced, remains constant for further displacement and then falls to zero in an elastic manner as the pile rebounds."

Thus, $W.H=R.(s+c/2)$

Where 'c' is the elastic displacement of the pile head.

For drop hammers, the most famous formula is the Engineering News formula,

$$W.H=R.(s+25)$$

where H and s are measured in centimeter.

There have been several driving formulae given in the (X) literature. But the main idea is the following:

"All driving formulae owe their existance to the assumption that the driving resistance is equal to the ultimate bearing capacity (UBC in short) of the pile under static loading."

Hiley's formula is used in Britain more than any other. Variants of the Engineering-News formula are most commonly used in the United States, and other countries have other preferences.

(X) Whitaker, T. "The design of piled foundations", Pergamon PRess.

## 2.3. A REVIEW OF PREVIOUS RESEARCH ON SINGLE PILES

Numerous investigators have attempted to solve the basic problem of predicting the ultimate bearing capacity (UBC) of a single pile under axial load by semi-emprical methods based on the field measurements (Boonstra 1963, Cummings 1950, Master 1949 Mortensen 1948, Moore 1949, Plantama 1948).

Meyerhof (1951) summarized and reviewed the earlier wörk and developed on approximate theory for the UBC of deep and shallow foundations. His analysis were based on a study of the equilibrium of such systems, coupled with an approximate application of the Mohr-Coulomb failure criterion on an assumed kinematic failure mode(i.e. that the failure surfaces are either plane or logaritmic spirals) and that the effect of body forces can be linearly superimposed on the "weightless material" solutions. The theoretical results thus obtained were expressed in the same form as Terzaghi's well known equation for predicting the UBC for shallow footings (Terzaghi's 1943)

$$q_u = cN_c + p_o N_q + \gamma \frac{B}{2} N_\gamma \qquad (2.3.1)$$

where $q_u$: ultimate normal pressure at the base

$c$: apparent cohesion of the soil

$p_o$: overburden pressure at base level

$\gamma$: soil unit weight

$B$: width of the base

$N_c$, $N_q$ and $N_\gamma$ are bearing capacity factors which depend on the depthof embedment, shape and roughness of the base and the apparent angle of friction $\phi$ of the soil. Meyerhof found that his theoretical results when compared with the laboratory test results, generally agreed to within ± 15 %. A series of the lab.

test results on the ultimate bearing capaciy of a pile were also reported(Meyerhof 1951). He showed that the UBC of a pile driven in cohesive soil was found to increase with time, but the contribution made by end bearing remained constant.

For a saturated soil in undrained conditions (for $\phi_U=0, N_q=1, N_\gamma=0$ equation 2.3.1 simplified to

$$q_u = cN_c + p_o \qquad (2.3.2)$$

The model and full scale tests described by Meyerhof and Murdock (1953), Golder and Leonard (1954), Whitaker and Cooke (1966) recorded values of the ultimate end bearing of a pile which were within ±20% of the predicted value based on eq'ns (2.3.1 and 2.3.2) using $N_c=9$ and $p_o=\gamma L$

Summarising all the methods of determining the ultimate end load resistance available, Skempton (1959) also conducted full scale experiments in fissured London clay and evaluated 'c' (undrained cohesion at the level of the base) from the equation $q_u = N_c c_b$ using $N_c=9$. They found that values of c were smaller than the mean undrained shear strength obtained from triaxial tests which supported their postulate regarding the strength of a fissured clay. It was concluded that the shear strength obtained in the field test, where the volume of the soil failure zone was sufficiently large to induce a representative set of fissures, most truly represents the fissured strengh of the suil. It was therefore suggested that for calculating ultimate end bearing resistance a factor should be used with the above expression, i.

$$q_u = N_c \cdot c_b \cdot w \qquad (2.3.3)$$

where N: 9, the normally accepted bearing capacity factor for clay

c: the shear strength taken from the mean shear strength depth profile given by triaxial compression tests,

$w$ : a coefficient for mddifying c to give the equivalent 'fissured' strength ($0.7 \leqslant w \leqslant 1.0$)

Skempton (1966) commented that the conventional shear strengh measured in the laboratory was not necessarily the same as the strengh beneath a pile base.

The load carried by a pile can be considered as:

$$P = Q_s + Q_b \qquad (2.3.4)$$

where $Q_s$: frictional resistance contribution to P along the surface of the shaft,

$Q_b$: Base resistance contribution to P

Similarly the ultimate load carrying capacity $P_u$ of the pile can be written

$$P_u = Q_{su} + Q_{bu} \qquad (2.3.5)$$

where $Q_{su}$: ultimate frictional resistance

$Q_{bu}$: ultimate base load

The evaluation of $Q_{bu}$ has already been discussed. Incidentall Banarjee (1970) deduced from his elastic analysis that the end load carried by piles (of length L and diameter D), both compressible and rigid ($20 < L/D < 40$) was approximately 7.5 percent of the total load.

Thev values of $Q_{su}$ and $Q_{bu}$ obviously depend primarly upon the soil state immediately adjacent to the pile shaft, and therefore really on the whole story of load transfer between the pile shaft and the soil. The clay araund the shaft of bored piles may also have softened due to remoulding and swollen in the presence of free water. Meyerhof and Murdock (1953) found that the softened zone extended to about 2 in. from the shaft face of the bored cast-in-situ piles in London clay. The strenght of the clay in this zone was reduced by the initial increase in moisture content occurring during construction. Even less is known of the way in which effective stresses, which fundamentally govern the bahaviour of the whole system, are modified by the various disturbances particularly within the thin "skin" of soil immediately adjacent to the pile shaft.

Accepted practice is to express the ultimate shaft resistance of a pile in clay as;

$$Q_{SU} = A_S\, c\, \alpha \qquad (2.3.5)$$

where $A_S$: shaft surface area in contact with the soil

c: mean undrained cohesion over the lenght

$\alpha$: nondimensional adhesion factor ($0.2 \leq \alpha \leq 1$)

Therefore, the evaluation of $\alpha$ in the equ'n (2.3.5) is of prime importancein the determination of the contribution made by the sh shaft. Skempton (1959) recommended, after assessing many load tests in London clay, that the value of $\alpha$ would be 0.3 to 0.6 in this material for cast-in-situ piles. The reason for the reduction in shaft adhesion was thought to be mainly due to softening of the clay in the sides of the borehole, brought about thiefly by stress release and subsequent swelling (due to wetting)(Skempton 1966). Whitaker and Cooke (1966) suggested that the value given to the coefficient $\alpha$ ought to be assessed in relation to the method by which c was evaluated for the mean shear strenght depth profile, based usually on triaxial compression tests on specimens from tube samples, obtained via site investigation made in the current commercial manner. (They used $\alpha$ :0.44). Burland(1966) also found the value of $\alpha$ to be less than unity, and recommended that for design of bored piles in London clay, Values of $\alpha$ should be between 0.4 and 0.45 in accordance with Skempton (1966).

Tomlinson (1970) carried out an experimental investigation on adhesion of piles in stiff London clay using instrumented close ended 6.625 in (160 mm) outside diametem steel tubes, of lenghts ranging from 3 m, 4.5 m, 6 m long (L/D: 18,27,36). To find the effect of different installation methods, he used similar pairs of jacked and driven (by drop hammer), and maintened loading tests were performed on each. He found, after 28 days that the driven piles had considerably higher peak failure loads than the jacked piles. The rate of the load transference from the shaft was similar for jacked and driven piles, but in the latter case, the clay appeared to be bonded more strongly to the pile surface. The adhesion factor $\alpha$, derived from mean peak loads of the piles driven into stiff clay, decreased from about 0.85 for the shortest piles to 0.65 for the longest piles. The piles driven through soft clays into stiff clays showed adhesion factor of the same order as those driven into stiff clays only. Some of the soft clay was dragged down with piles driven through it into stiff clay, this extended up to 16 to 18 diameters and presumably caused some reduction in the value of $\alpha$ attributed to the stiff clay. In some cases it was found that the adhesion in this case was higher than that for piles driven into the stiff clay. This was merely due to the gap which existedaround the upper part of the shaft having been filled with particularly consolidated soft clay, the presence of a skin of sand or sandy clay increased the skin friction along the same section beyond that developed in the soft clay case.

Butterfield and Johnston (1973) investigated the stress acting on an instrumented steel pile (100 mm. external diameter, 4 m. long L/D:33) penetrating at a constant rate in London clay.

The adhesion factors $\alpha$ which they found are for driving and extraction respectively. These varied along the shaft, but in general, appeared to be reduced very slightly with increasing distances from the pile toe, although remaining consistently higher in the upper 15 diameters of the pile length. It is noticable that their mean $\alpha$ value applicable at any penetration was rather lower for extraction (ranging from 0.6 to 0.8) than for driving (ranging from 0.7 to 1.2). It iz worthwhile mentioning at this stage that the higher adhesion factor $\alpha$ near the pile toe may also relate to the problem of $q_u$ higher than $c_u$ found by many investigators, as diccussed in the earlier part of this section.

Cooke and Price (1973) plooted their measured shear strength distrubition predicted by an elastic analysis. The predicted and observed results were in reasonable aggrement, except near the pile toe where the observed values were higher. They found that the mean value of adhesion factor $\alpha$ was 0.46 in penetration at crp, which was within the expected range. They also noticed that the increase in $\alpha$ with depth in the load test was consistent with the observations of stress transfer at working loads.

Butterfield and Johnston (1973) observed that at penetration depths below 4 pile diameters, the numerical value of the total radial stress was to range between 4 to 8 times the value of $c_u$ profile of the soil and which corresponded reasonably with the available elastic-plastic anaysis(Butterfield and Banarjee 1970). From their measured vertical shear and radial stress, recorded locally on the pile shaft, they were able to predict an effective interface friction angle ($\delta$) which varied between $4^{o}$ and $20^{o}$ with 72 % of 200 measured values falling within the range of $10^{o} \pm 3^{o}$.

So far the governing factors for assessing the UBC of the toe and the shaft resistance of a friction pile in clay have been discussed. The selection of a factor of safety for calculating the working load is influenced by both the need for safety against catastrophic failure, and in some circumstances, particularly in pile groups, by settlement considerations. Whitaker and Cooke (1966) demonstrated that the components of shaft and base resistance are mobilesed independently during the penetration of a pile. For a given proportion of mobilizationof ultimate shaft resistance, settlement increased with shaft diameter and full mobilisation occured at a head displacement of between 0.5 % and 1.0 % of the shaft diameter. This appeared to be independent of the shaft length for both plain and enlarged base piles. They proposed that the working load be evaluated by the following expression:

$$\frac{P_u}{F} = \frac{Q_{su}}{F_s} + \frac{Q_{bu}}{F_b} \qquad (2.3.7)$$

where F: overall factor of safety to obtained load

$F_s$: factor of safety for shaft resistance

$F_b$: factor of safety for base resistance

The investigations of Burland (1966) showed that there is a unique non-dimensional relationship between load ratio ( $q/q_u$) and settlement ratio ($\rho/B$) over a wide range of depths at a given site up to one third of the ultimate load (q). The settlement relationship was approximately linear and expressible as;

$$\rho/D = K.(q/q_u) \qquad (2.3.8)$$

where $\rho$: settlement of pile

D: diameter of pile

K: a constant determinable from the slope of a non-dimensional plot of the plate load test.

(for London clay they found that K:0.02 approximately). Their field observations have shown that the magnitude of the long term consolidation settlement of deep foundations, in the heavily over consolidated London clay, is usually compared with the short term settlement.

Whitaker and Cooke (1966) observed that the consolidation settlement of an axially loaded pile was usually negligible when compared with the immediate settlement. The time-dependent settlement of a single pile due to the dissipation of excess pore water poressure was analysed approximately by Poulos and Davis (1968). When a single pile is loaded, the major part of the settlement apperantly occurs immediately even under the assumption of completely undrained conditions, although the rate of consolidation of the single pile-soil system (as expected) was generally slower than that of a surface footing of the same base dimensions. Butler and Morton (1970) showed, from the results of load tests on single piles in clay that the settlement of a single pile consisted of two parts, one recoverable and the other a residual "permanent set". They established an essentially unique relation between the load ratio ($P/P_u$) and the settlement performance of a particular type of pile on a specific soil, which suggested that the settlement of a pile became largely non-recoverable beyond a critical value of load ratio around 0.6 to 0.7. However there was still a linear relation between load ratio and recoverable settlement beyond one third of the ultimate load. Cooke and Price (1973) loaded their piles within the working load range (0.3 and 0.57 times ultimate load) and observed that the system behaved elastically in both loading and unloading.

Ottoviani (1975) has produced elastic analysis using a three dimensional Finite Element technique by incorparating parameters embeded length of pile, depth of rigid layer and compressibility ratios,(ratio of Young Modulus of the pile material to that of the soil). Results he presented like stiffness characteristic, shear and vertical stresses distribution agree well with those given by Mattes and Poulos (1969) and Butterfield and Banarjee (1971a). He has also produced some graphs of a particular case study showing the distrubition of principal stresses $\sigma_1$, $\sigma_3$ and $\sigma_v$(vertical stress) against shear stress in dimensionless form. Above mentioned stresses become negligible at a distance of less than 3 diameters below the pile and in the zone above the pile base. The minor principal stresses $\sigma_3$ as well as $\sigma_v$ was tensile. It is also noticable that the $\sigma_1$ and $\sigma_v$ distribution was similar below the pile base but different in the area above the pile base where $\sigma_v$ was very small.

Marsland (1971) conducted in-situ plate loading tests to investigate the "elastic modulus" of fissured London clay at two different site for a comparison with results obtained from laboratory triaxial tests. It should be added that determination of elastic "moduli" is by no means a "standard" triaxial test technique. He found that moduli determined from the reloading cycles on his plates made at a half of their UBC were appreciably greater than those obtained from first loading. The ratios of the reloading to the first loading E (Young modulus) varied from 1.3 at a depth of 6 m. to 1.8 at a depth of 25 m. Values of K were obtained from the equ'n 2.3.8 and found to be 0.006 to 0.009 as against 0.01 to 0.02 given by Burland (1966). Cooke and Price

(1973) evaluated the soil shear modulus (G) from the measured shear stress and corresponding shear strain along the pile shaft which they plotted as E against $[E=2(1+\mu)G, \mu=0.5]$ depth. It showed that E increased with depth with a mean value of 11 $MN/m^2$. They actually measured the soil movement around their pile as it was jacked into London clay and during a subsequent load test. The soil movements were measured by an inclonometer during installation and testing, and showed that the movement was greatest within two pile diameters of the shaft was also detected at distances up to 10 pile diameters from the shaft. The volume of the surface heave was approximately two thirds of the embedded volume of the pile and maximum upward movement occurred at a radius of 1.5 times the pile diameter.

Koizumi and Ito (1967) also investigated the changes in pore water pressure developed in a normally consolidated clay due to the effect of pile driving. The pore water pressure recorded at various depths immediately after full penetration of a pile were very low near the soil surface and increased linearly with depth reaching the peak value at elevations some where above the tip of the pile. The normal stresses on the pile face was almost equal to the induced pore water pressure. The excess pore pressure decreased sharply with distance from the interface and reached zero at 6 diameters from it. Tomlinson (1970) observed that the pore pressure developed around driven piles in London clay was lower than that developed by jacked piles and that the former were dissipated more quickly. He explained the phonomenon by the fact that rebound in pile driving might result in the opening up of fissures in the clay and thus a more rapid dissipation of excess pore water pressure. Consequently there would also be a gain ih strength through the reconsolidation of the clay around the pile shaft. The highest pore pressure in cases of both driven and jacked piles, were recorded at 2 diameters away from the pile surface.

Butterfield and Banarjee (1970) presented an analysis of the pore pressure due to an idealised model of piles installed in a saturated soil. Its dissipation with teme and the resulting time dependent variation of pile load carrying capacity. To produce this elastic-plastic analysis they assumed that:

(i) The total stress changes in the elastic range that of an ideal incompressible elastic material characterized by a shear modulus (G) and Poisson's ratio ($\mu$)(:0.5); and in the plastic range, according to the Von Misses yield criterion $\tau_{oct}$ = constant

(ii) The pore pressure ($\Delta u$) increases due to change in the total stress as

$$\Delta u = \Delta \sigma_{oct} + A^{*}.\Delta \tau_{oct} \qquad (2.3.11)$$

and

$$A^{*} = (3A-1)/\sqrt{2} \qquad (2.3.12)$$

where $\sigma_{oct}$ and $\tau_{oct}$ representoctahedral stres components and A is the conventional pore pressure parameter.

(iii) The model allows the excess pore pressure to dissipate with time according to the Biot (1941) theory. They found that for normally consolidated clay the mean total stress level at the interface within the soil body, at the pressure face was approximately 5.5 $c_u$ and the maximum pore pressure ($\Delta u$) varied typically between 4 to 6 times $c_u$ (depending on the value of A) These decreased linearly becoming negligible at the elastic-plastic boundary (approximately 5.5 D from the centre of the pile). Thel also showed that the immediate increase in effective radial stress at the pile face was nearly zero ünd that the ultimate load capacity of the driven pile might increase by 6 to 10 times its value immediately after its having been driven.

Chandler (1968) suggested that the effective lateral stresses on the pile surface control the magnitude of the friction-particularly when the rate of loading was slow enough to ensure drained conditions in the clay along the pile shaft. When a pile is loaded the probable mechanism of deformation involves the simple shear of a narrow cylinder of clay immediately around the pile shaft.In this zone the effective normal pressure would be the horizantel effectivestress $\sigma_h'$ and the drained strength of the clay around the pile shaft might be presented:

$$\tau = c' + \sigma_h' \tan \phi' \qquad (2.3.13)$$

and considering the whole length of a pile L and diameter D.

$$Q_s = \pi D \int_0^L (c' + \sigma_h' \tan \phi')\, dL \qquad (2.3.14)$$

In terms of the ultimate resistance per unit area for, the above equ'n 2.3.13 could be expressed as;

$$f_u = c' + \bar{K}_o \bar{\sigma}_v' \tan \phi' \qquad (2.3.15)$$

where $\bar{K}_o$ and $\bar{\sigma}_v$ are the mean values of K and the effective vertical stress $\sigma_v'$ respectively. Results of maintened load tests in London clay which were compared with this hypothesis using effective stress parameters obtained for remoulded softened specimens, shoved a raasonable agreement with the upper limit of the recorded values

## 2.4. A REVIEW OF PREVIOUS RESEARCH ON PILE GROUPS

During the past three decades attempts have been made both experimentally (mainly on models), and analytically, to understand the behaviour of pile groups. For financial reasons very few full scale field investigations have been carried out to study the comparative behaviour of pile groups and single piles in detail. Khe usual engineerihg practice is to assess the UBC of the pile group by multiplying the ultimate capacity of a single constituent pile by number of piles in the group together with an emprical "efficiency factor" which has been derived from either an "efficiency formulae" or some "rule of thumb". Terzaghi and Peck (1948) considered the use of efficiency form to be contrary to good design, since such formulae do not really take into account tha various parameters which are known to influence the group behaviour. They suggested that for design purposes the behaviour of the block composed of the soil and piles within the perimeter of the outer piles (subsequently referred to as a "block") should be examined, especially in the case of closely spaced pile groups. Peck,Hanson and Thornburn (1953) described a simple method of design in which the failure of this "block" is determined by using precisely the same method that has been described earlier for a single isolated pile or pier.

Whitaker (1857) performed model tests on pile groups using pointed 0.125 in (3 mm) diameter brass rods to form floating capped pile groups of 3x3,5x5, 7x7, 9x9 sizes. The length to diameter ratios of the piles used were 12, 24, 36 and 48 and the caps were rigid. The clay was a remoulded brown London clay with an undrained shear strength of about 4.5 to 9.5 $kN/m^2$. He principally investigated the relationship between failure criteria of pile group, the number of piles in a group, their spacing and the way in which the load was being shared amongst the piles in the group, "block failure" (when failure was accompanied by the formation of slip plane joining the perimeter piles in plan) and "local failure"(associated with local penetration of some or all piles into the soil). For groups of piles with a specific L/D ratio and number he found a unique value of spacing for each group at which the mechanism of failure changed. For spacings closer than this value, failure took place as a "block failure" whereas at wider spacing failure seemed to be "local". He observed that the transition from "block" to "local" failure took place at s:2.25 D for L/D:48 in 9x9 groups and at progressively closer spacing for shorter piles and smaller groups. For some groups the transition points were beyond the dimensions he used in his experiment (i.e. 1.5 pile diameter spacing). He found, in general, that the efficiency (the ratio of the avarege load per pile when failure of a group occurred to the load at failure of a comparable single pile) decreased more rapidly when the spacing was smaller than that causing block failure. The efficiency of a group increased gradually with larger spacing and became unity at spacings greater than 7 pile diameters. He also observed that the change in failure mechanism was related to the settlement ratio of a group as defined above expect for 3x3 groups; maximum settlement ratio occurred at the ultimate and half the ultimate load for a group of a given size at the "transition spacing" between block and local failure, and it decreased rapidly with closer spacing and less rapidly with wider spacing . Observing the load distribution between the piles in a group, he found that 1/3rd the failure load the corner pile in a group took the largest proportion of the applied load relative to the centre one. The proportion of the total load taken by individuals piles was distrubuted more uniformly with increases in spacing.

Whitaker (1960) extended his investigation on model scale pile groups in clay to cover the influence of rigid ground contacting caps. He used the same size of piles as in his previous investigation except that he confined his work to L/D ratio 48. He found the major influence of the ground contacting cap to be the generation of "block failures" even at spacing up to 4 diameters compared with about 2.25 diameters for the floating capped groups. The settlement ratios for 3x3 pile groups with ground contacting caps were found to be nearly the same as those with floating caps. However for larger groups, thesettlement ratios for ground contacting groups were found to be higher by almost 20 % than those for corresponding floating capped groups.

Sowers (1961) also performed model to study the behaviour of pile groups in a homogeneous clay.Tests were carried out using piles of 0.5 and 1.25 in diameter with embedded lengths of 12, 24 and 36 diameters in square groups of 2, 4, 9 and 16 piles. They found that in all cases, the avarage load per pile (in a group) was less than that of a single isolated pile at the same value of displacement. The effect of a 3x3 group was as low as 50 % with piles at 1.5 diameter spacing. At about 2 diameter spacing pile groups were observed to fail as a "unit" (i.e. block failure) with the soil between piles moving downwards. With the piles at larger spacings efficiency increased for almost all groups, and failure occurred in individuals. AT one third of the ultimate load on the group of 3x3 piles (s:2.5 D) the corner piles carried 3 times as much load as the centre pile. The percentage difference between the shared loads became smaller with increased pile spacing. They interpreted this unequal distrubition as a reflection of elastic deformation of the soil by analogy with the contact pressure distrubition under a rigid block resting on an elastic half space. The contact pressure are greatest at the outside corners and at least at the centre of the block.

Saffery and Tate (1961) performed model tests on floating cap pile groups in remoulded London clay on 3x3 pile group. The dimensions of their model piles were 6 mm diameter with L/D ratio 12, 18, 24 and 30. Their results were in general agreement with those previously reported by Whitaker and Sowers. They found the efficiency of 3x3 pile group not to be effected by eccentric loading within a range of eccentricities up to two thirds of pile spacing. The mean seettlement at failure under eccentric loading (e:2/3s) could be upto twice the settlement measured for failure occurring under axial loading on similar groups, but difference between the settlement due to the axial and eccentric loading was much smaller at half the ultimate load. They explained this phonemonen in terms of a change in group behaviour whilst the applied load increased from working load to ultimate load. Under all conditions of loading, they found, the settlement ratio of the 3x3 pile groups to be independent of the length of pile. During axial loading the distrubition of load was of course symmetrical, but eccentric loading , the row of piles nearest to the load application point reached failure, whilst the row furthest from that point carried quite negligible loads.

Tate (1963) also reported a further scale model investigation of pile groups in clay using 50 mm diameter piles of 22 in length The report itself is more of an instructive discussion of pile group behaviour at model scale than a contribution to any specific aspect of pile group problems. One important feature mentioned was that due to the driving subsequent piles belonging to a group, the disturbance of the soil surrounding embedded pile(s), tended to change the load capacitl of individual piles. This however depended on the pattern of driving. The magnitude of such group action was primarly depended on spacing and was noticeable for spacings up to 8 diameter.

Koizomi and Ito (1967) carried out an experimental programme on pile foundations using a 300 mm dia. instrumented piles 5.55m long with a 3 diameters spacing between piles in 3x3 groups. They performed cyclical loading tests on a single pile foundation, a group pile foundation (3x3) and a spread footing, the dimensions of which were identical with those of the cap. Their results showed that the single pile foundations had ü well defined failure load, at which the skin friction reached the shearihg strength almost simultaneously along whole pile shaft. For the group the load was carried mostly by the corner piles with the least load being borne by the centre pile and failure proceeded progressively from outer to inner piles. The adhesion at failure on both the single piles and the corner of group was approximately equal to the undrained cohesion of the original soil. The settlement equal to the undrained cohesion of the original soil. The settlement of the pile group was considerably greater than that of a single pile at the same average pile load within the working load range.

Brand (1972) conducted a serèes of full scale tests on 2x2 cap-bearing pile groups embedded in a normally consolidated soft Bangkok clay using timber pile of 15 cm dia. and 6 m. long. In their test series they also conducted loading tests on single piles, free-standing pile groups and on spread footings of identical sizes to the pile cap. Suprisingly they found that the value of settlement ratio was less than unity for piles spaced at 5 D apart. However, the settlement ratios for narrowly spaced piles were in good ggreement with the theoretical results.

Approximate elastic analyşis of axially loaded pile groups have been recently published by Poulos (1968), Barvashov (1968), Poulos and Mattes (1971 a and b) and Butterfield and Banarjee (1971 a,b). Poulos presented a basic analysis for settlement interaction between two identical in elastic media, and by superposition the increase in settlement of a symmetrical pile group was predicted. Poulos (1968) published a work on rigid pile groups with a rigid and flexible cap, whereas Poulos and Mattes (1971 a,b) further improved their analysis by taking into account of the compressibility ratio between the piles and the soil. However, all of their analysis have been for floating cap pile groups. Butterfield and Banarjee (1971a) extended their analytical method to solve the problem of rigid and compressible pile groups with floating caps, taking into account of the other piles comprising the group in an elastic finite layer and spaced in an arbitrary manner. All of these analysis showed that the settlement ratios of pile groups were strongly influenced by the ratios of length to diameter, spacing to diameter, the ratio of the thickness of elastic layer to the pile length and the number and the arrangement of piles in a group.

Butterfield and Banarjee (1971b) extended their studies further to encompass the interaction between compressible pile groups and a rigid bmooth pile cap resting on the soil surface. Their results showed that the load displacement characteristics of similar pile groups with typical floating or contacting caps were not very different. Depending on the group size and pile spacing the ground contacting cap increased the stiffness of the system by 5% to 15%.

Banarjee (1975a) extended his work on the application of elastic theory to the problem of deep foundations and has produced a method of analysis associated with the design of a cap bearing pile group, subjected to eccentric load. He showed that in the case of short flexible piles, the cap supports as much as 60% of the applied load (either vertical load or moment), whereas in the case of long rigid piles the proportion supported could be as low as 18%. He found that the minimum value of eccentricity required to produce tension at the cap soil interface or that on the pile head is always larger than 1/6th of the width of the foundation.

Cooke (1975) presented quite a practical approach for assessing the settlement of the friction pile foundation using elastic analysis. He postulated a mechanism for load transfer through the friction pile shaft to the supporting soil. He developed expressions to estimate the base as wellas the shaft settlements as a function of elastic moduli (E, G, $\mu$). These expressions were further simplified by incorparating the corresponding ultimate resistance which finally allows one to use emprical relationships such as $E/\tau$. Comparing his expression for the shaft settlement $[\rho_s = -3Q_s / 2\pi L E \log_e(2n)]$ with that given by Davis and Poulos(1968) he showed that the settlement of the pile carrying a given load P within the elastic range in a cohesive soil is virtually independent of the pile diameter.

# CHAPTER 3: THE METHOD OF THE MODEL STUDY

## 3.1. GENERAL

The basic philosophy of model study, especially in Soil Mechanics, has been clearly explained by Roscoe (1968). The general approach to model investigations can be divided into two groups; the first and most predominant type is that in which tests are made at model scale to examine the assumptions that have been adopted in theoretical analysis of prototype problems. ".....The intention is to cummit blunders on a small scale so that profits can be made on a large scale. This approach often brings to light unpredictable diffucilties, inspires confidence, and provides valuable experience at minimal cost....."(Roscoe 1968). But the results obtained from this type of model study may be misleading in predicting prototype behaviour if the principles of similitude are not satisfied. The second approach of model investigation is to determine and satisfy the principle of similitude so that the behaviour of a prototype may be correctly predicted from the model study. For this to be achieved it is necessary to assess not only all the physical quantities that are relevant to the problem, but also to reduce them to a working minimum by selecting the most significant parameters. Consequently, the most valuable results are obtained from a model test which can incorporate both of these approaches.

Because of the complicated nature of all important soil parameters and tha inhomogenity and variability of in-situ soil deposits, the scope for examining any analytical solutions in relation to prototype behaviour is very limited on field scale tests. Additionally such a procedure would be extremely expensive and slow, since each field problem usually has features, and there is little prospect of controlling, the precise conditions under which such tests have been made. Therefore there is still great scope for the proper use of model tests in Soil Mechanics.

Roscoe (1968) has discussed the detail of the similarity condition in Soil Mechanics model studies. This is restated below in order to discuss the model system adopted in this study. Considering a small element of volume V and the surface area of a large body of a saturated soil where;

- n: the porosity
- $\gamma_s$: weight of unit volume of the soil material
- f: unit weight of the pore fluid
- i: hydrolic gradient
- u: porewater pressure on the boundary of the element
- k: permeabilty
- z: depth below water table to point under consideration

Now let's consider two such homologous elements corresponding to the prototype and model whose linear scale ratio is "h", then the volume of the prototype is calculated as $V_p = h^3 V_m$. Here the subscripts "p" and "m" refer to the prototype and model respectively. Assuming that the material used in the model has similar stress-strain behaviour to that of the prototype material then;

$\alpha$ : stress scale factor

$\beta$ : strain scale factor

$S_t$ : time scale factor

$\rho_s$ : scale factor of unit weight of solid

$\rho$ : scale factor of bulk unit weight

$\rho_f$ : scale factor of unit weight of pore fluid

For similarity, all stresses, including porewater pressure must be to the scale $\alpha$, and consequently all forces to the scale $h^2\alpha$. Scaling the self weight of the solid phases requires that

$$\frac{\gamma_{sp}(1-n_p)\ V_p}{\gamma_{sm}(1-n_m)\ V_m} = h^2\alpha \qquad (3.1.1)$$

if $\gamma_{sp} = \rho_s \gamma_{sm}$ is used, the equ'n 3.1.1 becomes

$$\alpha = h\rho_s \frac{1-n_p}{1-n_m} \qquad (3.1.2)$$

Likewise, scaling the uplift of the solid phase (1-n)V requires

$$\alpha = h\rho_f \frac{1-n_p}{1-n_m} \qquad (3.1.3)$$

and the scaling the self weigth of the luquid V requires that

$$\alpha = h\rho_f \frac{n_p}{n_m} \qquad (3.1.4)$$

From eq'ns 3.1.3 and 3.1.4, it is evident that

$$n_p = n_m \qquad (3.1.6)$$

and from eq'ns. 3.1.2 and 3.1.3

$$\rho_f = \rho_s \qquad (3.1.6)$$

Considering the bulk unit weight $\gamma$ of a satureted soil can be stated as

$$\gamma = n\gamma_f + (1-n)\gamma_s \qquad (3.1.7)$$

$$\gamma_p = \rho\gamma_m \qquad (3.1.8)$$

therefore $\rho_f = \rho_s = \rho$ (3.1.9)

If tne requirements of eq'n 3.1.5 and 3.1.9 are satisfied, then eq'ns 3.1.1, 3.1.3 and 3.1.4 can be reduced to

$$\alpha = h\rho \qquad (3.1.10)$$

Eq'ns of similarity developed so far have ignored any effects of changes in pore water pnessure, i.e. time scale factor. Now it is necassary to assume that the fluid flow in both materials governed by Darcy's law Q: KiA where Q is the volume of fluid flowing per unit time through an area A. IT is essential that the flow nets can be similar at instants of time (with a time scale $S_t$) and equipotential surfaces can be represented as

$$u_p - \gamma_p z_p = \alpha u_m - \rho_f \gamma_{fm} h z_m = \text{constant}$$

which fulfils the condition of equ'n 3.1.9 and 3.1.10. Since the hydraulic gradient is given by $i = \frac{d}{dL}(u - \gamma_f z)$ and both u and $\gamma_f$ are now to a scale of $\alpha$ , it is evident that $i_p = (\alpha/h) i_m$

Due to consolidation, the flow net generally changes with time. to investigate this effect itis necessary to consider that the volume (V) of the element alters by $\Delta V$ in time $\Delta T$ and that the change in volume equals to the net volume of water that has emigrated from the element. When the two homologous elements in the prototype and model are considered, the changes of volume $\Delta V_m$ and $\Delta V_p$ for a corresponding time $\Delta T_m$ and $\Delta T_p$ must be such that:

$$\Delta V_p = h^3 \beta \Delta V_m \qquad (3.1.11)$$

and from Darcy's law

$$\frac{\Delta V}{\Delta T} \alpha\; kiA \qquad (3.1.12)$$

i.e. $$K \alpha \left(\frac{1}{ih}\right) \times \frac{\Delta T}{\Delta T} \qquad (3.1.13)$$

Consequently, the scale $\mu$ for the permeabilities of the prototype and model mèdia must be in the ratio;

$$M = \frac{h}{\alpha} \cdot \frac{1}{h^2} \cdot h^3\beta \cdot \frac{1}{S_t} = \frac{h^2\beta}{\alpha S_t} \qquad (3.1.15)$$

if the displacements are large, geometrical conditions of similarity will only be maintained if the scale factor (SF) for strain $\beta$:1, and by replacing the fluid by water, eq'n 3.1.9 can be written as $\rho = \rho_s = \rho_f = 1$ and this reduced Eq'n. 3.1.10 to $\alpha = h$

Roscoe further discussed the similarity condition with the respect to the critical state concept. He concluded that identical strain behaviour may be obtained from the same soil in different initial states (i.e. $n_p \neq n_m$), when it is subjected to pdoperly scaled stress paths. However, when $n_p \neq n_m$, the self weigth (body force) is not significant, the obvious solution is to use the prototype material in the same initial state as in the model (i.e. $n_p = n_m$) and to impose the same stresses on the model as on the prototype, thereby ensuring that $M$ remains constant. Under this condition $\alpha = \beta = 1$ and the identity of the strain curves in the model and prototype is ensured. Therefore from the eq'n 3.1.15, it can be shown that all the same scale factors between the model and prototype should then be proportional to $h^2$.

## 3.2 SOIL BED

### 3.2.1 SIMULATION OF HALF SPACE

Previous investigators (Whitaker 57,60, Saffery and Tate 1961, Sowers 61) investigated the behaviour of model pile groups using quite small containers of remoulded clay each individually prepared for a sets of tests on a specific size of pile diameter. For example, Saffery and Tate (1961) used a cylindrical container 48 diameter to 60 dia. deep for 3x3 pile groups of 12 D, 18 D and 30 D (where D represents pile diameter) embedded lengths and spacings varying between 1.5 D and 5 D ; whereas Sower (1961) used a container of 3 ft in diameter and 4 ft high for each test on pile groups consisting of 2, 4, 9 and 16 piles of 0.5 in and 1.5 in diameters 12,24, and 36 pile diameter embedded lengths spaced at 1.5 to 5 D apart. Consequently, the results they have presentdd were undoubtedly influenced by some interaction with the small size of containers used.

In this study a cylindrical container 380mm in diameter (15 D) and with a heigth of 360mm (14 D) was used.

3.2.2 THE SUBSOIL MATERIAL AND PREPARATION OF THE BED

Although it is possible to prepare large clay beds with uniform moisture content by careful hand punning of small size grinding clay, such beds are unsatisfactory in a number of reasons that is production of subsequent identical beds is very dubious; the manufacturing process is both tedious and uninformative; the inclusion of small hard nodules is difficult to avoid; and both the stress history and the stress state in the bed are totally unknown.

Whitaker (1957, 1960) well mixed London clay with additional water and filled into cylindrical brass containers with loose bottoms of brass plate. Each container was slightly overfilled to form a mound above the rim and was covered immediately with a sheet of polythene held down in contact with tha clay. The container was then stored from 4 to 6 days before use. Immediately before a test, the mound was cut off by means of a wire and the surface struck level with the upper edge of the container.

Butterfield used an apparatus which is ,in fact, a giant doubly drained oedometer consolidating clay slurry. This type of clay bed preparation pdocedure has been developed to overcome many prdblems of hand-made clay bed preparation. By controlling the drainage, adequately homogeneous beds with different moisture contents and undrained cohesive strehgths have been obtained. The whole preparetion pdrocedure for a 1 m-depth bed could take up to about three months.

As the project was intended to investigate the behaviour of model pile systems in a clay soil, remoulded clay from Kilyos Uskumru Köy basin was chosen as a suitable material.

The clay was obtained from a local brick factory producing reflecter brick for inner walls of furnaces, in the form of being grinded and passing completely through no 200 sieve.

A big mixer (Picture 1) was used to mix the clay with the optimum water content value of 25.7 percent. After having mixed, it was poured on the plates (Photo 2) and let in wait approximately for 24 hours with a wet towel on preventing the clay from the loss of humidity. Thun the mixed clay was compacted into the container according to the number of blows calculated in regard to the standard Proctor Energy (see Appendix I). As the final step the upper and lower lids were shut and a hydrolic constant pressure of 4 kg/cm$^2$ obtained from the triaxial test system, was applied into the container for 24 hours.

3.2.3 UNIFORMITY OF THE BED

To check the uniformity of the clay bed, as prequisitive tests, 37 mm diameter samples were taken along the depth at three different locations. The uniformity of the bed was checked both visual inspection and measurement of water content. It was found that the technique of preparetion was satisfactory and water con tent of the filling varied the range 29.9 to 31.8 percent(see App.II)

3.2.4 PROPERTIES OF THE SOIL

The general properties of the soil investigated by standard

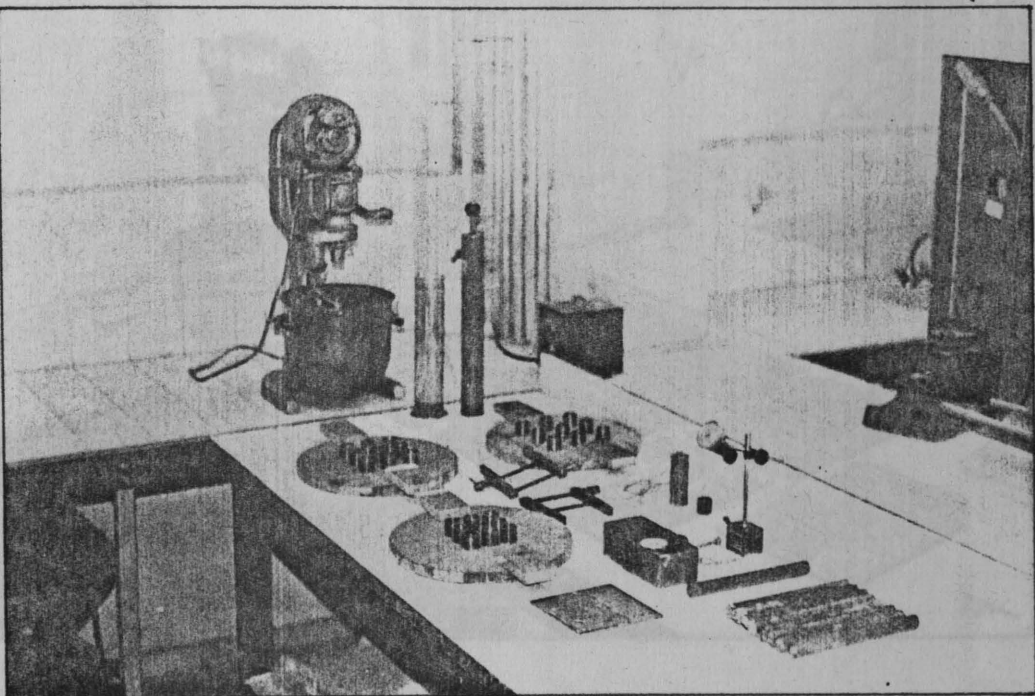


Photo 1. General equipment of the experiments.

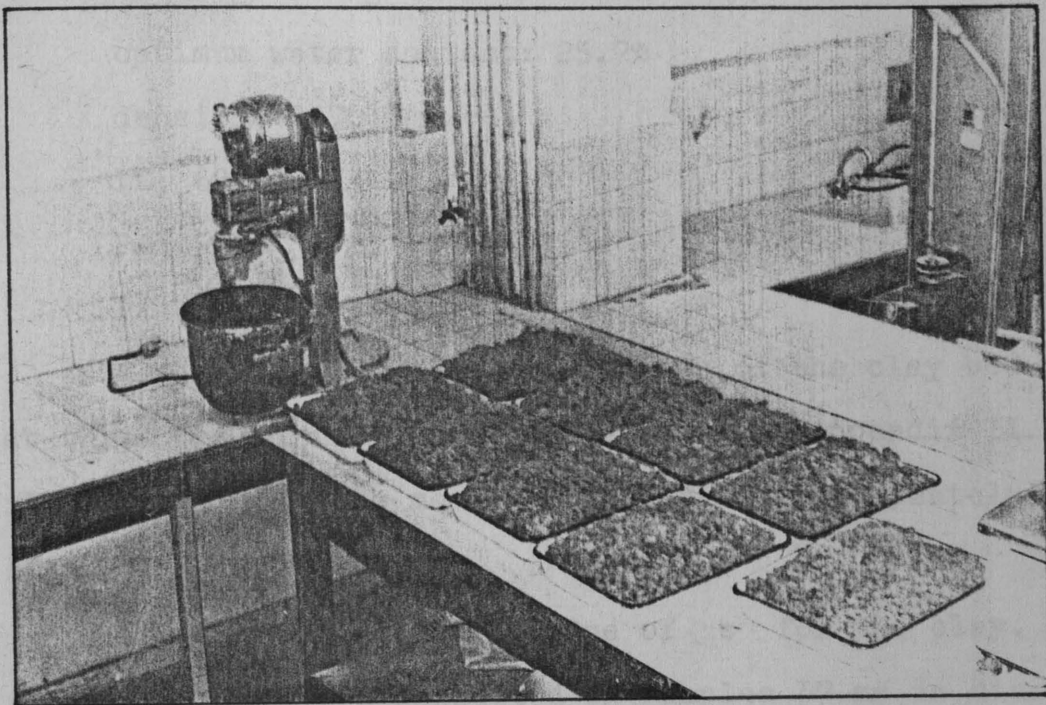


Photo 2. The clay mixed with water at optimum content.

laboratory tests. The results of Sieve and Hydrometer analysises (see Appendix III) shoved that the soil has a percentage of 20.1 of clay.

soil: yellow remoulded clay from Kilyos Uskumru Köy basin passing completely through no 200 sieve (after grinding in a brick factory)

optimum water content: 25.7%.

density: 1.701 gr/cm$^3$

LL: 69 %

PL: 29 %

PI: 40 %

The mean water content profiles of the clay bed obtained at three different locations are shown in Appendix II. This table illustrates that the mean waetr content approximately equals to 30 %.

To obtain a reliable value of c for the clay, undrained triaxial tests were conducted on samples 37 mm in diameter and 75 mm heigth. The cell pressures were 2.5, 3.5 and 5 kg/cm$^2$. The mean c value was found to be 0.2 kg/cm$^2$.

3.3 MEASUREMENT OF THE LOAD-DISPLACEMENT MECHANISM

3.3.1 GENERAL

Piles are test loaded for three main purposes:

1. to determine the load-settlement relationship, particularly in the region of working load,
2. To serve as a proof test to ensure that failure doesnot occur before a load selected multiple (factor of safety) of the chosen working load.
3. To determine the real ultimate bearing capacity for checking of the value calculated from pile driving formula

Many different test methods are used in the current practice. The most commonly used test method in North America is the Slow Maintained-Load test (Slow ML test) recommended by the American Society for Testing and Materials (ASTM). Another well known test is the Constant-Rate-Of-Penetration test (CRP Test) which is first devised and presented by Whitaker (1957,1963) and Whitaker and Cooke (1961). A third test method is the Swedish Cyclic test. These three tests can be said to present basic test types. A combination is the Quick Maintaned Load test which can be achieved a considerable saving of cost and time as the test can be completed during one working day.

3.4.2 SLOW ML TEST

The pile is loaded in eight equal increments to 200% of the anticipated working load of the pile. Then, the load is removed in four equal decrements. Each load is to be maintaned until the rate of settlement is decreased to 0.3 mm/hr.,i.e., 0.05 mm/10 min or for 2 hours, whichever occurs first. The 200% load is to be maintaned in 24 hrs. The test takes about 70 hours or more to perform, depending on conditions.

3.4.3 SWEDISH CYCLIC TEST

The pile is first loaded at a certain small load, equal to about one-third of the anticipated load of the pile. It is then unloaded to one-half of this value. This is repeated 20 times and as each individual cycle takes 20 min, the loads will be higher than in the first. This goes on 20 cycles for each load combination until "failure" is reached. (for example 40-20,60-30, 80-40, 100-50 tons....)

### 3.4.4 CONSTANT-RATE-OF-PENETRATION TEST

Generally, in load testing of soil structures, there is time dependence of the displacement produced by a load increment due to (i) soil consolidation (i.e. dissipation of pore water pressure) and (ii) the secondary creep deformation of the soil skeloton under constant effective stresses.

The extremes of behaviour due to consolidation can be spanned by creating either very "rapid" loading (i.e. undrained) conditions or very "sldw" (drained) conditions. Any influence of secondary creep can be detected by perfor ming slow tests at different rates. In the absence of "creep" such slow tests should produce identical results. Such an investigation is only possible in pile systems by using constant rate of displacement testing; on which the pile head velocity is the independent variable. The Maintained load tsts are not at all suitable for this purpose. Therefore all load tests on single piles and pile groups in this study were performed at a constant rate of penetration (displacement) (CRP). The CRP test has been used by many investigators (Whitaker and Cooke 1966, Saffery and Tate 1961, Tomlinson 1970, Butterfield and Johnston 1973) to explore the UBC of pila systems.

Whitaker (1957,1963) first devised and described the basic principle of such tests applied to piles. Inthe CRP test, the pile head is forced to penetrate the soil at a constant velocity, normally 0.5 mm/min and the force required to achieve the penetration is continuously recorded as a dependent variable.

Whitaker (1963) mentioned that the purpose of CRP test was to determine the UBC of the pile, and the force penetration curve obtained from this test did not represent an equilibrium relationship between load and corresponding settlement found by maintained load test.

The main merit of the CRP test is that it gives a result of ultimate load which is generally capable of interpretation without diffucult.

The settlement of a pile head obtained under CRP testing should be the immediate settlement as commented on by Whitaker and Cooke (1961), however it has already been shown that the major part of the settlement of a single pile occurs on immediate loading under undrained conditions (Whitaker and Cooke 1966, Poulos and Davis 1968)

## 3.5 DRIVING EQUIPMENT

The model piles used in experiments are hollow steel pipes having an outer diameter of 26 mm, a thickness of 2 mm and a length of 10 D. The pile tips produced to make a cone of $45^{\circ}$ were attached to one end and the other ends (pile heads) were closed with a circular metalic plate having the same diameter and a thickness of 2 mm.

For the operation of model pile driving, a simple tripod like driving system with an ordinary revolving reel, was intented After the calculations, a cylindrical metal mass was produced as well for the sake of the idea that it be used as a free-fall hammer.

During the driving process, for having the piles driven vertically, wooden guides were used. Taking the three spacing between the piles into consideration, 2 cm thick cylindrical wooden plates were cut in diameter of the container and on them holes were opened to put 5 cm-long iron rings, each with an inner diameter 1 mm greater than that of piles.

Also, during the driving process, a metal cab was used so that the pile heads would not possibly undergo harm nor they would be deformed.

## 3.6 TESTING PROGRAMME

### 3.6.1 MAJOR TOPICS

The study covered the three major topics below:

A. To determine the UBC of single piles with a small L/D ratio under vertical loads applied axially and related to driving energy and time.

B. to determine the UBC of pile groups under vertical loads axially and the effects of driving energy, pile spacing, number of piles and time on UBC.

C. The interaction between individuals in a group.

D. To determine the region of Linear-elastic behaviour (working load capacity) of single pile and pile groups.

### 3.6.2 THE SETTING UP OF LATIN SQUARE

Since the aim of this study was to find out how the bearing capacity of single piles and pile groups was effected by driving energy, pile spacing, number of piles in a group and time these four factors were taken as the independent variables. The UBC, on the other hand, was the dependent variable. To be able to understand how much these four independent variables would effect the bearing capacity, Latin Square ( a statistical analysis method), set up with three levels of the variables, was used as an experimental method. This method had been used widely in some soil mechanics before. (X)investigations before(X) latin square might be given as follows:

| | $S_1$ | $S_2$ | $S_3$ |
|---|---|---|---|
| $n_1$ | 1st.Exp.($E_1 t_1$) | 2nd. ($E_2 t_2$) | 3rd. ($E_3 T_3$) |
| $n_2$ | 1st.Exp.($E_3 t_2$) | 2nd. ($E_1 t_3$) | 3rd. ($E_2 t_1$) |
| $n_3$ | 1st.Exp.($E_2 t_3$) | 2nd. ($E_3 t_1$) | 3rd. ($E_1 t_2$) |

where S: spacing between two piles from center to center

$S_1$: 1.5 D (diameter), $S_2$: 2 D, $S_3$: 2.5 D

n: number of piles in a group

$n_1$: single pile, $n_2$: 2x2 pile group, $n_3$: 3x3 p.g.

E: driving energy

$E_2$: the value corresponding to the same driving energy found after calculations.(see App IV)

$E_1$: half of the calculated energy

$E_3$: one and a half of the calculated energy.

t: time factor

$t_1$: considered to be zero (however, right after the driving process, since the container was put in the compression test machine given a constant rate of penetration and the measurement gages were set up in nearly 15 min.,the time zero happened to be 1/4 hrs.)

$t_2$: 2 days (48hrs) after driving

$t_3$: 5 days (120 hrs) after driving.

(X) -Kumbasar and Toğrol,1966, "Zemin cinsininpenetrasyon mukavemetine etkisi"

-Tümay,M.,1973,"Correlation-Regression of soil type, Preconsolidation stress and Granulometric parameters with swelling of Fine-grained soils", Proc. of the 3rd Int. Conf. on Expansive Soils, vol 5

[illegible] M., 1975 "Donmus zeminin kayma mukavemeti",Ph.D.Thesis

### 3.6.3 TESTING ROUTINE:

One of the three guides giving the necessary spacing for the experiment was placed on the container and fixed with two clamps (Photo 3 ). Then, the tripod-like driving system was placed over the container in a way that it provides a vertical fall of the hammer.

Taking the distance between the pile head and the hammer constant, the hammer was let fall free from the required fall height, and the pile was driven. After driving process, the clamps and wooden guide were taken out (Photo 4 ) and with regard to the necessary waiting time, the container was placed into the compression test machine which can give a constant-rate-of-penetration at a wide range of velocity.

During the loading process two different gages were used (Photo 5 ) and so the measurements were obtained. The container was arisen upwards with a velocity of 0.o2 mm/min.Here might be seen a reverse situation compared to tne nature. The deformation of the pile, that is, the penetration into the clay was a relative behaviour. The load gage placed on the pile head was, in fact, a steel ring with an extensiometre inside it.

The distance taken by the container during its movement upwards measured in milimetre witn an extensiometre placed on the edge of the container. The value read from the gage on the pile head was a negative rising because of the compression in the gage. The difference between them was the actual displacement of the pile (or group) (Photo 5).

After changing the displacement measurements read from the load gage on the pile head into real loads, a load-displacement curve has been obtained.

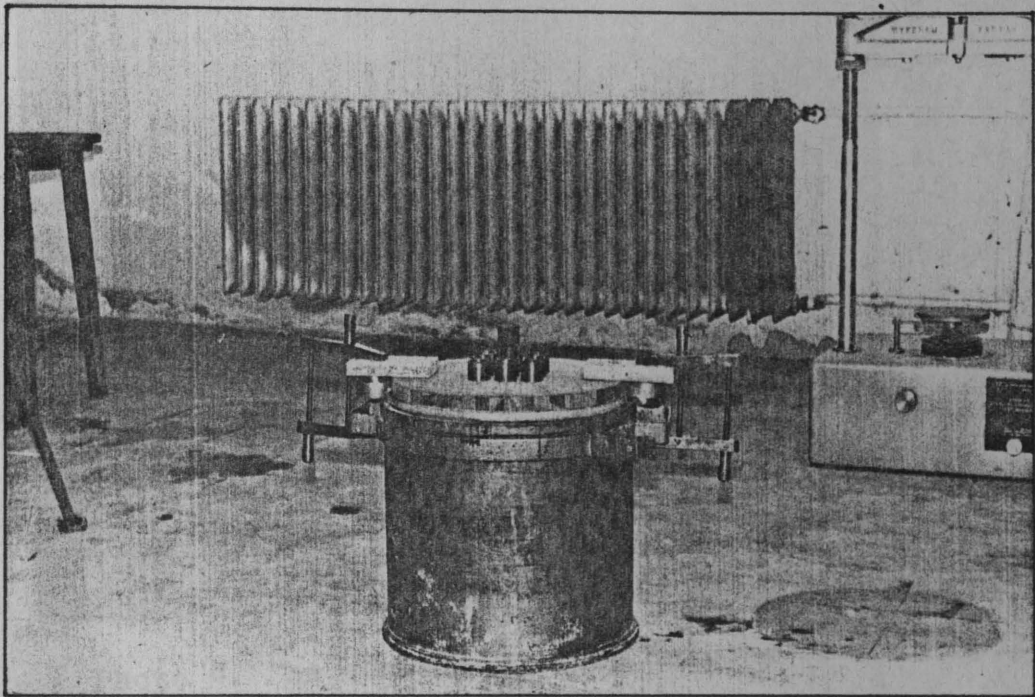


Photo 3. The container with a guide before driving process

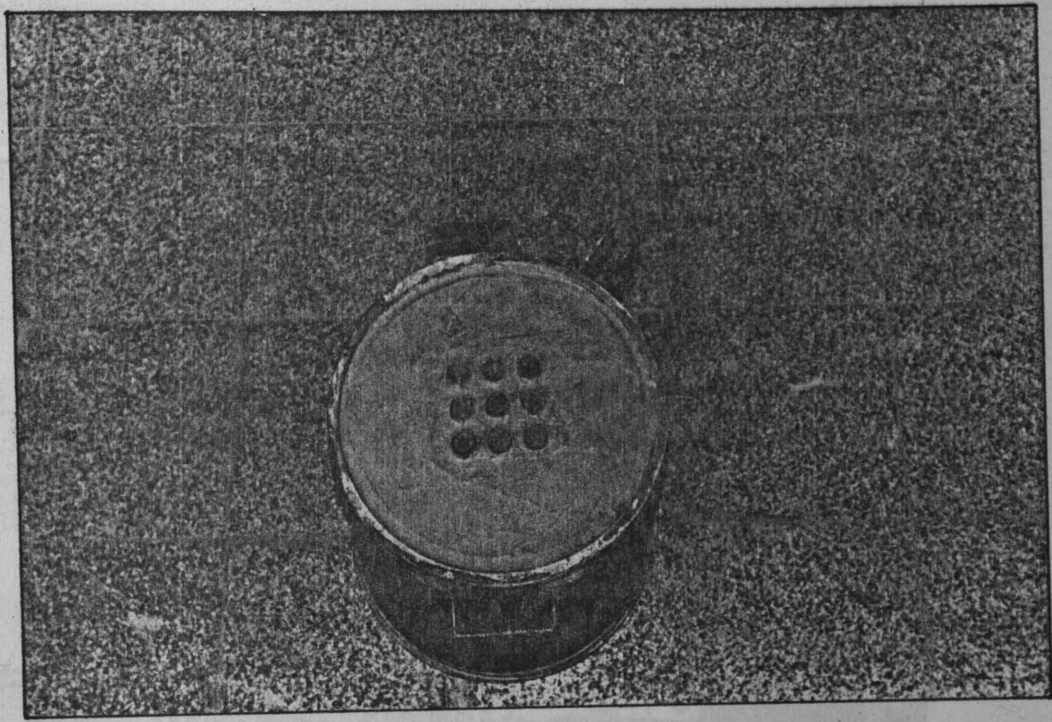


Photo 4. ~he container after driving of a 3x3 pile group

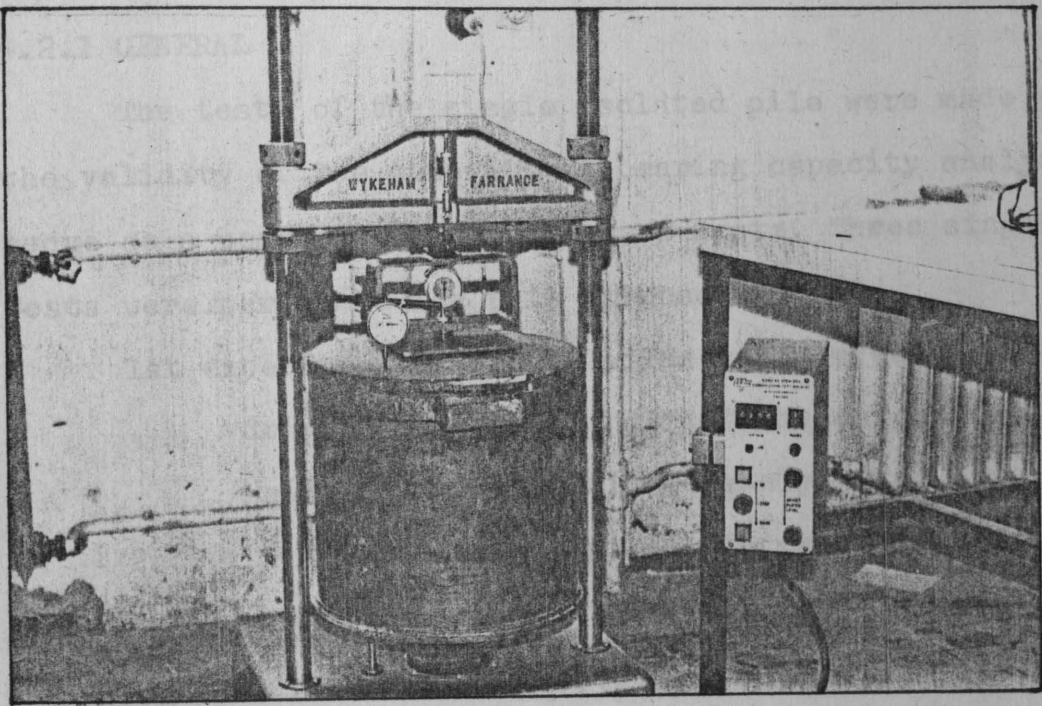



Photo 5. Test-loading of the container with piles

# CHAPTER 4: INTERPRETATION AND COMPRASION OF RESULTS

## 4.1 INTRODUCTION

In this chapter, the experimental results and theoretical values presented and after a comparison they are interpreted.

## 4.2 SINGLE PILES

### 4.2.1 GENERAL

The tests of the single isolated pile were made to determine the validity of the theoretical bearing capacity analysis and to sedve as a comrarison for the group tests. Three single pile tests were performed for this purpose.

1st experiment of single piles was performed with

driving energy: $E_1$ (half of the calculated energy)

waiting time : $t_1$ (immediately after driving)

2nd experiment ofsingle piles was performed with

driving energy: $E_2$ (the calculated energy)

waiting time : $t_2$ (2 days after driving)

3rd experiment of single piles was performed with

driving energy: $E_3$ (one and a half of the calculated)

waiting time : $t_3$ (5 days afterdriving)

### 4.2.2 THEORETICAL BEARING CAPACITY OF A SINGLE MODEL PILE

As it was explained in Chapter 2, Ultimate end-bearing resistance of a single pile can be calculated by means of the formula 2.3.3:

$$q_u = w \, N_c \, c_b$$

where $N_c$ :9, the normally accepted BC factor for clay

c : the shear strength taken from the mean shear strength depth profile given by triaxial compression tests

w : a coefficient for modifying $c_b$ changing between $0.7 \leq w \leq 1.0$

Taking $N_c$ :9, c: 0.2 kg/cm and w:1

$q_u$ :1.80 kg/cm$^2$

Now, the ultimate base resistance of pile is

$Q_{bu} = q_u A_b$ :1,80.π.(2,6/2):(1,80).(5.31)

$Q_{bu}$ = 9,56 kg.

The ultimate shaft resistance of a pile can be calculated by using eq'n 2.3.5

$Q_{su} = \alpha c\ A_s$

where $A_s$ : shaft surface area in contact with the soil

c : mean cohesion over the length

$\alpha$ : non-dimensional adhesion factor ($0.2 \leq \alpha \leq 1$ typically)

After substituting the values of $A_s$: 212,5 cm$^2$, c: 0.2 kg/cm$^2$ and $\alpha$:1 (chosen according to the value of c ) in the eq'n 2.3.5

$Q_{su}$: 212,5 x 0.2 x 1

$Q_{su}$: 42,5 kg

The load carried by the pile is the sum of the ultimate shaft resistance (frictional resistance) and the base resistance given in the eq'n 2.3 5

$P_u = Q_{su} + Q_{bu}$

$P_u$ = 42,5+9,56 = 52,06 kg.

4.2.3 RESULTS

The load-displacement curve which was obtained by loading of single pile driven and tested by the variables of the first experiment shows that, (Figure 1) the UBC reached is 61,12 kg. The Linear-elastic behaviour is valid in the regioh up to 33 kg and consistutes 54 percent of UBC.

The UBC of the second experiment (Figure 2) is 54.96 kg. The Linear-elastic behaviour region is observed in the region up to 45 kg. This meets to 81 %of UBC.

In the third test, The UBC is (58,42 kg and in the region up to 44 kg Linear-elastic behaviour is seen (figure 3). THis consists of 75 percent of the UBC.

4.3 PILE GROUPS

4.3.1 THE TESTS OF 2x2 PILE GROUP

The three tests which were performed for 2x2 pile group take place in the second row of Latin squares. Each test was performed according to the values in the cell and the spacing values which come across to the columns in which they exist. (see chapter 3.6.2 )

1st experiment of 2x2 group was performed with

driving energy: $E_3$ (one and a half of the calculated)

waiting time : $t_2$ (2 days)

spacing : $S_1$ (1.5 Ddistance between piles from center to center)

The load-displacement curve which was obtained by loading of 2x2 group driven and tested by these variables shows that, the UBC is 226.8 kg. The Linear-elastic behaviour is valid in the region up to 120 kg and this constitutes 53 percent of the UBC in this test. (Figure 5 ).

2nd experiment of 2x2 group was performed with

driving energy: $E_1$

waiteng time : $t_3$ (5 days)

spacing : $S_2$ (2 D )

and the UBC in this test is 197.7 kg. The Linear-elastic behaviour is observed in the region up to 141 kg. This meets to 71 percent of UBC. (Figure 6 )

3rd experiment of 2x2 pile group was performed with

driving energy: $E_2$

waiting time : $t_1$ ( zero time)

spacing : $S_3$(2.5 D )

UBC in this test is 216.36 kg and in the region up to 141 kg. Linear-elastic behaviour is seen. This consist 65 percent of the UBC. (Figure 7 )

Later another test was made to obtain a comrarison and to be able to contribute to the interpretation of test results. In this test the variables of the third test were used, but the only change was the doubling of the spacing to 5 d. There was an increase at UBC with the increase of spacing and 240.7 kg UBC was reached. Linear-elastic behaviour was seen up to 176 kg. which consists of 73 percent of UBC of this test.

### 4.3.2 THE TESTS OF 3x3 PILE GROUP

The tests which were made for 3x3 pile group take place in the third row of Latin square.

1st experiment of 3x3 pile group was performed with

driving energy: $E_2$

waiting time : $t_3$ (5 days)

spacing : $S_1$ (1.5 D)

UBC : 447.12 kg

Linear-elastic behaviour region limit: 250 kg

2nd experiment of 3x3 group was performed with (Figure 10 )

driving energy: $E_3$

waiting time : $t_1$ (zero time)

spacing : $S_2$ (2 D )

UBC : 498.06 kg

linear-elastic behaviour region limit: 274 kg

3rd experiment of 3x3 group was performed with (Figure11)

driving energy: $E_1$

waiting time : $t_2$ (2 days)

spacing : $S_3$ (2.5 D )

UBC: 471.24 kg

Linear-elastic behaviour region limit: 290 kg

## 4.4 EFFICIENCY IN GROUPS

Several efficiency formulas have been developed in the past 30 years for the behaviour of pile groups. These formulas do not take into account the length of piles and the effect of varied and complex soil conditions. Among these methods The Converse-Laberra Formula which is the best known is derived under the assumption that the area of the pile available for developing shear is reduced by the influence of adjacent piles in the same row as the subject pile, and by the closest pile of the adjacent row. For two piles

$$\text{Efficiency} = \frac{\text{Total circumference} - 2 \times (\text{non-acting area})}{\text{Total circumference}}$$

For 'n' piles in a single row and 'm' piles in 'n' rows

$$\text{Efficiency} = \eta = 1 - \arctan\frac{D}{2S}\left[\frac{m(n-1) - (m-1)\,n}{90\ mn}\right]$$

where D: diameter of a pile

S: spacing between two piles from cehtre to centre

For 2x2 pile group Efficiehcy factors of each spacing is as follows

for $S_1$ : 1.5 D $\eta_1$ : 0.795

$S_2$ : 2 D $\eta_2$ : 0.844

$S_3$ : 2.5 D $\eta_3$ : 0.874

S : 5 D $\eta$ : 0.915

and the theoretical group values for 2x2 groups:

$Q_{eff}$ = (number of piles in a group) X (theoretical UBC of a single pile) X (efficiency factor)

for

$S_1$ : 1.5 D $Q_{eff}$ : 152.03 kg

$S_2$ : 2 D $Q_{eff}$ : 161.4 kg

$S_3$ : 2.5 D $Q_{eff}$ : 167.14 kg

S : 5 D $Q_{eff}$ : 175.0 kg

For 3x3 pile groups, Efficiency factor for each spacing and the bearing capacity values related with that efficiency factors have been calculated as the following:

for $S_1$ : 1.5 D $\eta_1$ : 0.726

$S_2$ : 2 D $\eta_2$ : 00792

$S_3$ : 2.5 D $\eta_3$ : 0.832

and the bearing capacity values:

for $S_1$ : 1.5 D $Q_{eff}$ : 312.4 kg

$S_2$ : 2 D $Q_{eff}$ : 341.3 kg

$S_3$ : 2.5 D $Q_{eff}$ : 358.5 kg

## 4.5 BLOCK FAILURE OF MODEL PILE GROUPS

The experimental results and the interpretation of Latin square remind the possibility of "block failure" case. Therefore byconsidering the groups to behave as a'unit' for 1.5 D and 2D spacings, the theoretical bearing capacities were calculated to make a comparison. The formula given by Terzaghi (1943)

$$Q_g = q_d BL + D_f(2B+2L).s$$

where B: width of the pile group

L: lehgth of the pile group

s: avarage shearing resistance of soil per unit area ($s = c + p\tan\phi$)

$q_d$: UBC, per unit area, of a rectangular loaded area with depth L

After substituting the values for 2x2 pile group

at 1.5 D spacing:

$$Q_g = 1.8x(2.5\ D)x(2.5\ D)+26(2x2.5\ D+2x2.5\ D)(0.2+0.0017\tan\phi)$$

$$Q_g = 76.05 + 139.25 = 215.3 \text{ kg}$$

at 2 D spacing:

$$Q_g = 1.8x(3D)x(3D) + 26x(2x3D+2x3D)x(0.206)$$

$$Q_g = 109.51+167.1 = 276.6 \text{ kg}$$

Now the same calculations for 3x3 pile group

at 1.5 D spacing:

$$Q_g = 1.8x(4D)x(4D) + 26x(2x4D+2x4D)x(0.206)$$

$$Q_g = 194.7+222.8 = 417.5 \text{ kg}$$

4.6 RESULTS OF LATIN SQUARE

The single pile tests were conducted to form a basis of comparison. Because the spacing has no contribution in single pile tests, the effect of spacing will not be included in our first analysis. After replacing the UBC values obtained for each test at Latin square (The values in the second and third rows of Squares were obtained by dividing the experimental UBC by total number of piles in that group):

| | $E_1$ | $E_2$ | $E_3$ | 'n'totals |
|---|---|---|---|---|
| $n_1$ | 61.12 | 54.96 | 58.42 | 174.5 |
| $n_2$ | 49.42 | 54.07 | 56.7 | 160.19 |
| $n_3$ | 52.36 | 49.68 | 55.34 | 157.33 |
| totals | 162.9 | 158.66 | 170.46 | 492.07 |

| | $E_1$ | $E_2$ | $E_3$ | 't'totals |
|---|---|---|---|---|
| $t_1$ | 61.12 | 54.07 | 55.34 | 170.53 |
| $t_2$ | 52.36 | 54.96 | 56.7 | 160.02 |
| $t_3$ | 49.42 | 49.68 | 58.42 | 157.52 |
| | 162.9 | 158.71 | 170.46 | 492.07 |

The ansum of squares for each variable are computed as follows

forspacing $\sum S = \frac{167.5^2}{3} + \frac{159.72^2}{3} + \frac{164.85^2}{3} - \frac{492.07^2}{9} = 10.43$

for number of piles $\sum n = \frac{174.5^2}{3} + \frac{160.19^2}{3} + \frac{157.33^2}{3} - \frac{492.07}{9} = 56.19$

for energy $\sum E = \frac{162.9^2}{3} + \frac{158.71^2}{3} + \frac{170.46^2}{3} - \frac{492.07^2}{9} = 23.64$

for time $\sum t = \frac{170.53^2}{3} + \frac{160.02^2}{3} + \frac{157.52^2}{3} - \frac{492.07^2}{9} = 28.14$

Total $= (61.12)^2 + (54.96)^2 + (58.42)^2 + (49.42)^2 + (54.07)^2 + (56.7)^2 + (52.36)^2 + (49.68)^2 + (55.34)^2 - (492.07^2/9) = 118.47$

The ahalysis-of-varience table is the following:

| | sum of squares | Degrees of freedom | Mean square | F ratio | |
|---|---|---|---|---|---|
| number of piles | 56.418 | 2 | 28.209 | 5.32 | $> F_{.75} = 3.00$ |
| energy | 23.818 | 2 | 11.909 | 2.24 | |
| time | 28.14 | 2 | 14.07 | 2.65 | |
| residual | 10.602 | 2 | 5.301 | - | |
| Total | 118.978 | | | | |

As can be seen from the F ratio column, in a comparison at 75 percent confidence level, 'n'(number of piles) only has a meaning as obvious, Driving energy (E) and time (t) have no meanin at this confidence level. So it can be concluded that the levels of driving energy and waiting time chosen for these tests, didn't have a meaningful effect to the UBC of model piles.

Another analysis can be conducted to investigate how the independent variables effect the stress-strain relationship of model piles. The part of stress-strain relation that is relevant to the pile design is its elastic region. So if the values of the load at which the stress-strain (load-displacement) curve deviates from the linear zone are placed in the latin square, the following is obtained.

| | $E_1$ | $E_2$ | $E_3$ | time totals |
|---|---|---|---|---|
| $n_1$ | 33 | 45 | 44 | 122 |
| $n_2$ | 40.99 | 35.33 | 35.61 | 111.93 |
| $n_3$ | 17.4 | 17.8 | 18.9 | 54.1 |
| totals | 91.39 | 98.13 | 98.51 | 288.03 |

| | $E_1$ | $E_2$ | $E_3$ | 'n' totals |
|---|---|---|---|---|
| $t_1$ | 33 | 35.33 | 18.9 | 87.23 |
| $t_2$ | 17.4 | 45 | 35.61 | 98.01 |
| $t_3$ | 40.99 | 17.8 | 44 | 102.79 |
| 't' totals | 91.39 | 98.13 | 98.51 | 288.03 |

The analysis-of-varience table is the following:

| | sum of squares | Degrees of freedom | Mean square | F ratio | |
|---|---|---|---|---|---|
| number of piles | 895.124 | 2 | 447.56 | 15.65 | > $F_{.75}$ = 3.00 |
| energy | 10.696 | 2 | 5.34 | 0.18 | $F_{.50}$ = 1.00 |
| time | 42.352 | 2 | 21.17 | 0.74 | |
| Residual | 57.179 | 2 | 28.59 | - | |
| Total | 1005.351 | | | | |

The fact that, F ratio values obtained for 'E' and 't' were smaller than the value 1.00 for 50 percent confidence level which was already a small value to compare, shows all those independent variables (E and t) have no effect or contribution to the limit loads for Linear stress-strain behaviour. To obtain obtain the same results from two different analysises supports the fact that, the different values chosen for driving energy and time effect for these model tests are unsuitable to make a contribution. The reason is that, the time period for driving process provides time enough for dissipation of pore water pressure (lasting approximately 4 hrs for driving of a 3x3 pile group). So it is very hard to investigate the effects of driving energy and time factors within a single drop hammer driving system in such kind of model tests.

Excluding the single pile tests, 2x3 Latin Squares were set up and analysis-of-varience were made.

2x3 Latin Square analysis of which UBC of each test divided by number of piles in each group is given as:

| | $S_1$ | $S_2$ | $S_3$ | 'S'total |
|---|---|---|---|---|
| $n_2$ | 56.7 | 49.42 | 54.07 | 160.19 |
| $n_3$ | 49.68 | 55.34 | 52.36 | 157.38 |
| 'n' totals | 106.38 | 104.76 | 106.43 | 317.57 |

The analysis-of-variance table is given as:

| | sum of squares | Degrees of freedom | Mean square | F ratio | |
|---|---|---|---|---|---|
| number of piles | 1.316 | 1 | 1.316 | 0.062 | $< F_{.75} = 2.57$ |
| spacing | 0.902 | 2 | 0.451 | 0.02 | |
| residual | 42.309 | 2 | 21.154 | - | |
| Total | 44.528 | | | | |

Again F ratio values in a comparison at 75 percent confidence level have no meaning. Another 2x3 Latin Square analysis made using Linear-elastic behaviour region limits obtained by visual inspection is the follwing:

| | $S_1$ | $S_2$ | $S_3$ | 'S'total |
|---|---|---|---|---|
| $n_2$ | 30 | 40.97 | 40.97 | 111.94 |
| $n_3$ | 27.77 | 30.44 | 32.22 | 90.43 |
| 'n' totals | 57.77 | 71.41 | 73.19 | 202.37 |

The analysis-og-variance table is given as:

| | sum of squares | Degrees of freedom | Mean | F ratio |
|---|---|---|---|---|
| number of piles | 77.11 | 1 | 77.11 | 8.07 |
| spacing | 71.16 | 2 | 35.58 | 3.72 |
| residual | 19.10 | 2 | 9.55 | - |
| Total | 167.37 | | | |

F ratios of both number of piles and spacing effect at 75 percent confidence level have a meaningful effect to Linear-elastic behaviour region limits. For number of piles, this relation is more dominant.

Briefly, from the results of 2x3 Latin Square analysis, it was found that the UBC was not effected by the levels of variables chosen. Whereas a good relation was found between number of piles, pile spacing and Linear-elastic behaviour region.

4.7 COMPARISON OF THE THEORETICAL AND EXPERIMENTAL RESULTS

The theoretical UBC of a single pile has been calculated as equal to 50.2 kg. It was seen that tha average of maximum bearing capacity of single piles obtained in three tests is 15 percent in excess with regard to calculated theoretical value.

In 2x2 pile groups, for 1.5 D spacing, the experimental value of 226.8 kg is 42 percent in excess with regard to the theoretical one calculated by the Converse-Labarre group efficiency formula (159.6 kg). But again, the experimental UBC is 2 percent greater according to the theoretical one calculated according to Terzaghi's block failure formula. In this case, an existance of a great difference between the group efficiency value and the experimental result's being 2 percent greater according to the theoretical "block failere" value, shows that a "block failure" case is valid for 1.5 D spacing of 2x2 pile group. For 2 D spacing the experimental result (197.7 kg) is 16 percent greater when compared with group efficiency value. In addition, the experimental result is 32 percent less than the block failure value(290.2 kg). That means there is no block failure at 2D spacing. For 2.5D spacing, the experimental result (216.36 kg) is 23 percent greater according to the group efficienc y value (175.5 kg). A result (240.6 kg) which is 30 percent greater according to group efficiency value, has been obtained in 5D spacing test which was performed to make a comparison and which was not seen in Latin Squares. It was observed that percentage of the difference between experimental results obtained for four different spacing and theoretical values increase with the increase of spacing.

The results obtained from 3x3 pile group tests are the following. For 1.5D spacing, the experimental result (447.1 kg) is 36 percent in excess with regard to the theoretical group efficiencl value. The former- on the other hand, is 2 percent in excess with regard to block failure value. Therefore, the fact that, a "block failure" case is valid for 1.5 D spacing is concluded. For 2 D spacing, the experimental result with regard to theoretical group efficiency value is 40 percent in excess (357.8 and 498.06 kg). The block failure seems to have been brought on the platform due to excessive divergence of 40 percent and according to the calculations, the experimental result has reached 82 percent of group ~~efficienc~~ failure value (with missing 18 percentage)

This migth be interpreted as; the spacing that the type of group failure changes is a very close point to 2 D spacing for 3x3 group. For 2.5 D spacing, the excess in the experimental result is 25 percent with regard to the group efficiency value and this is nearly the same with the excess of 23 percent obtained at 2.5 D spacing of 2x2 pile group.

The difference of 15 percent between the experimental results and theoretical values of single piles has remained the same for 2 D spacing in 2x2 pile group (16 %). But, as for 2.5 D spacing of groups, it has increased to 23-25 percent and it is noticable that this result increases as the pile spacing increases. This increase migth be interpreted as the outcome of the smaller dimensions of the container effect the UBC of pile groups. In 2x2 pile group, a block failure has been seen at 1.5 D spacing. In 3x3 pile group, however, the spacing that the type of group failure changes is seen close to 2 D spacing.

## 4.8 REGIONS OF LINEAR-ELASTIC BEHAVIOUR

Butterfield (1979) has presented the linearity of his load-displacement curves that, "this was remarkably good to a nominal working load of 0.5 $Q_{ult}$, where $Q_{ult}$ for an 'N' pile group was arbitrarily assessed at Nxthe load capacity for a single similar pile and, in fact, the linearity extended to about 0.6x$Q_{ult}$..."

The Linear-elastic regions inspected visually are given as:

| | | $S_1$ | $S_2$ | $S_3$ | Mean ratio |
|---|---|---|---|---|---|
| 2x2 p.group | LEBR limit | 120 | 141 | 141 | |
| | ratio to $Q_{ult}$ | .57 6 | 0.67 | 0.67 | 0.64 $Q_{ult}$ |
| 3x3 p.group | LEBR limit | 250 | 274 | 290 | |
| | ratio to $Q_{ult}$ | 0.53 | 0.58 | 0.61 | 0.57 $Q_{ult}$ |

Taking the average of the two mean ratio of Linear-elastic behaviour region limits to $Q_{ult}$ :

$$(0.64 + 0.57)/2 = .60\ Q_{ult}$$

The ratio is satisfactorly in agreement with the comment given in the literature.

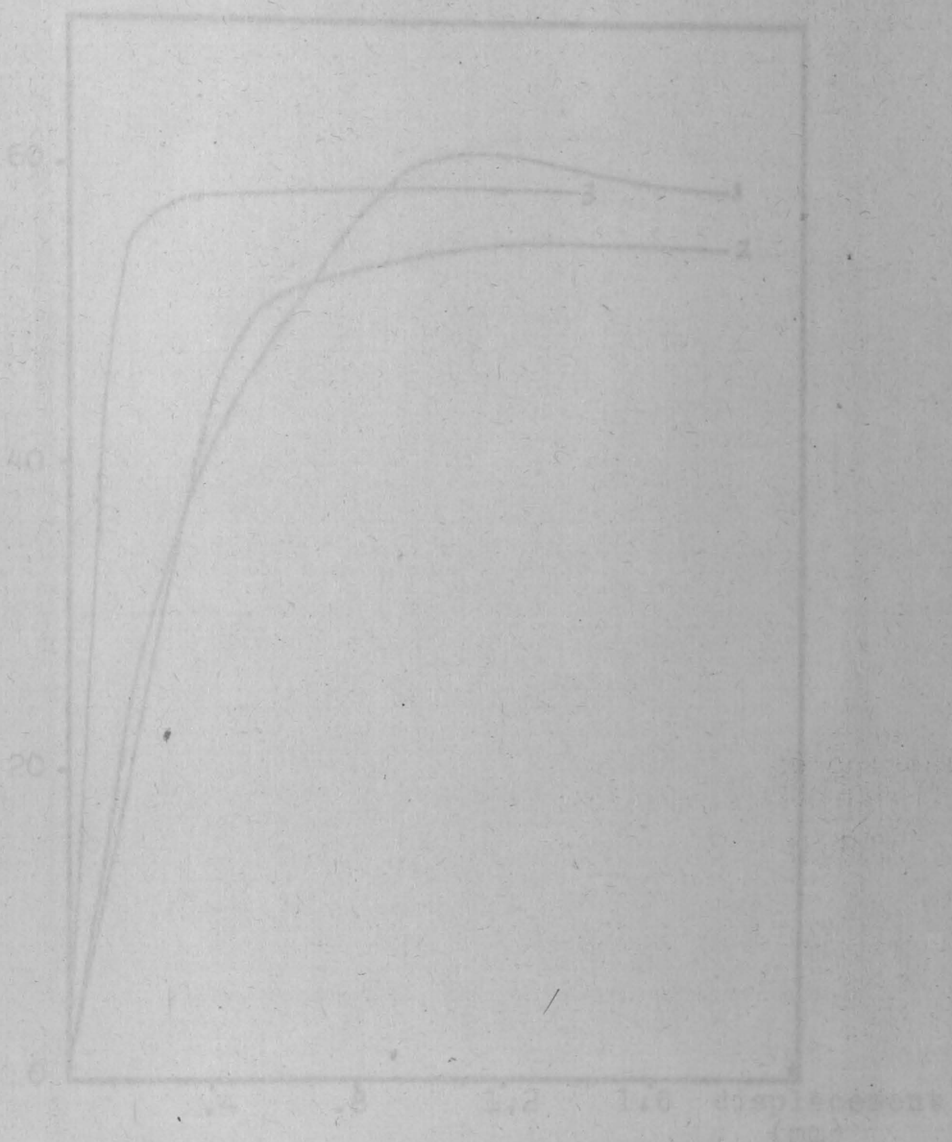



Figure 4. The load-displacement curve
of Single pile tests.

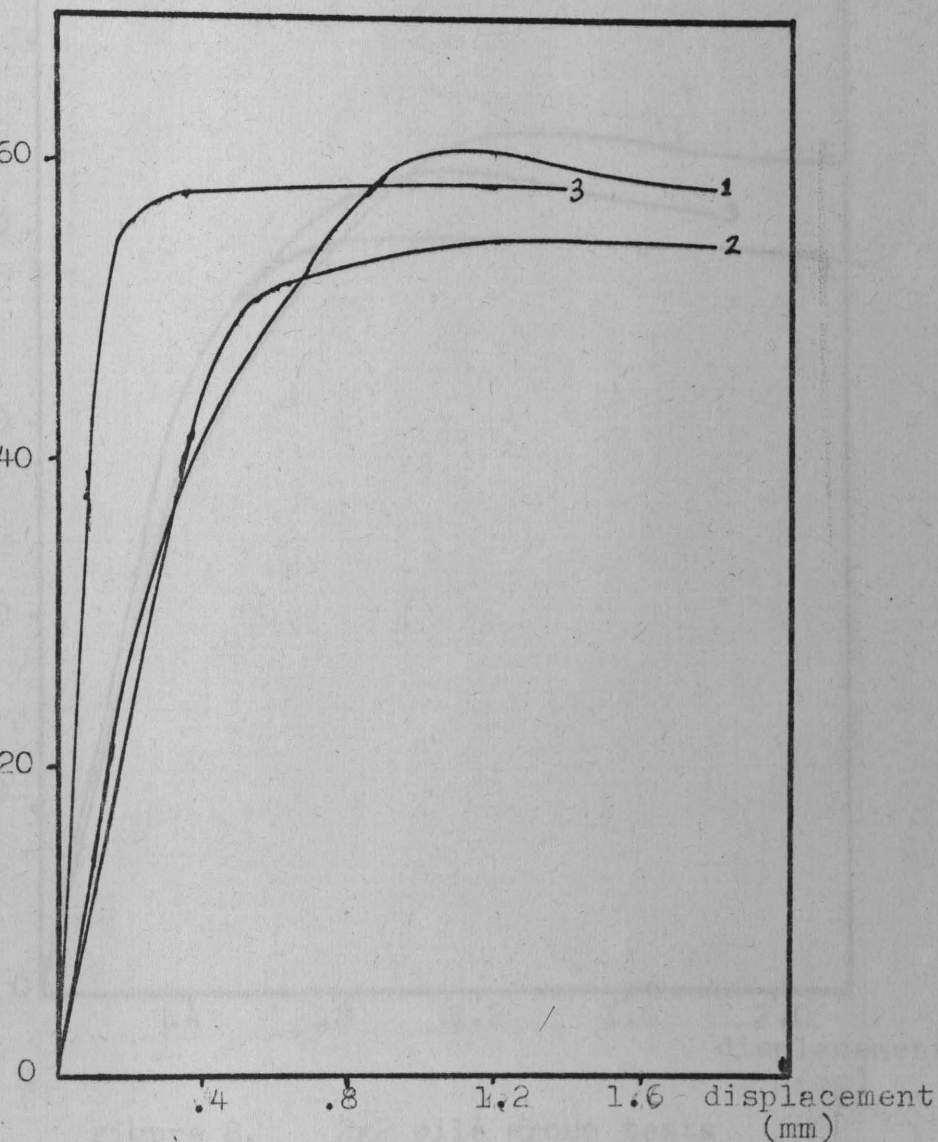



Figure 4. The load-displacement curve of Single pile tests.

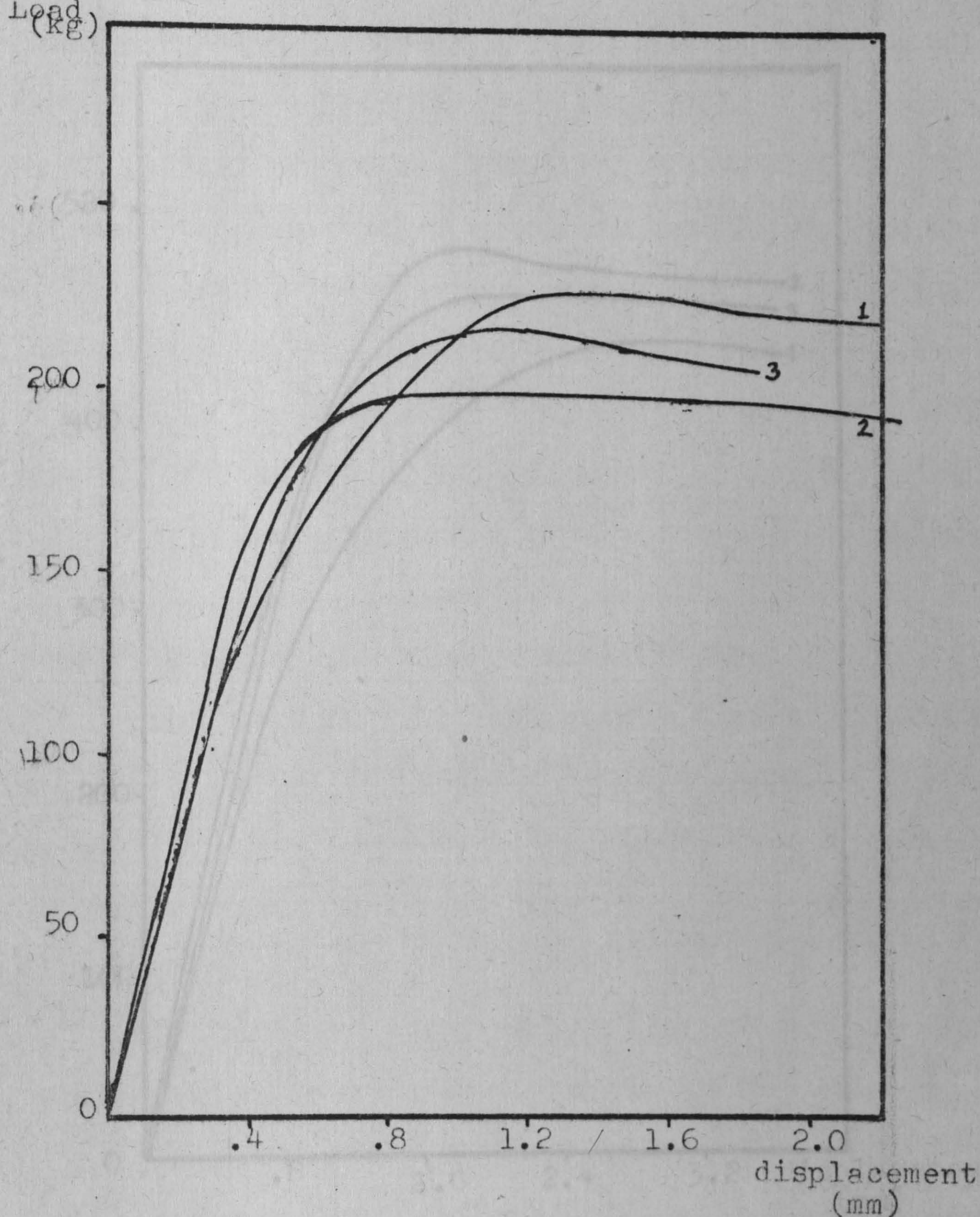



Figure 8. 2x2 pile group tests

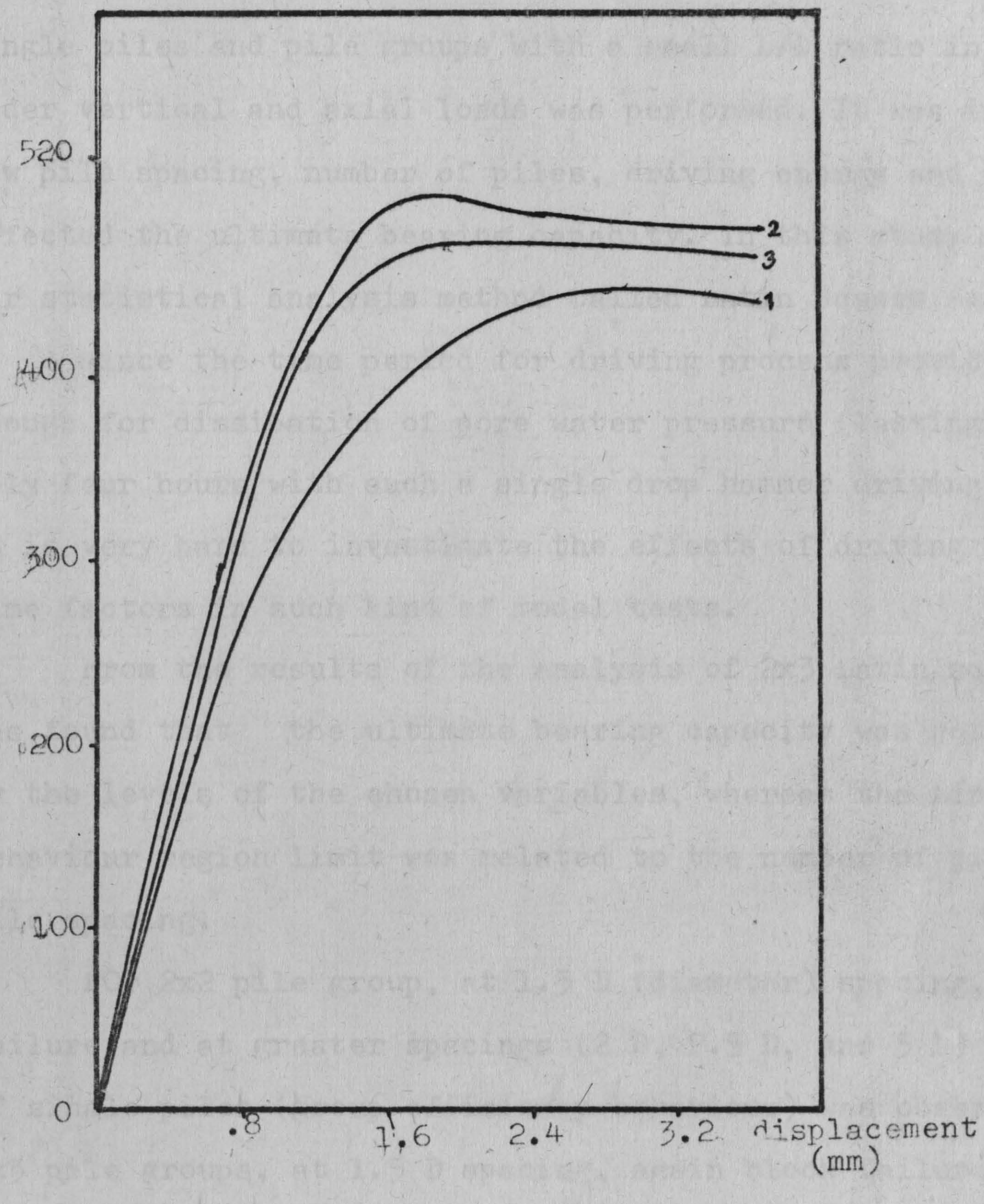



Figure 12. The load-displacement curves of 3x3 pile groups

# CHAPTER 5. CONCLUSIONS

A model scale study of the ultimate bearing capacity of single piles and pile groups with a small L/D ratio in clay under vertical and axial loads was performed. It was investigated how pile spacing, number of piles, driving energy and time factor effected the ultimate bearing capacity. In this study a particular statistical analysis method called Latin Square was used.

Since the time period for driving process provides time enough for dissipation of pore water pressure (lasting approximately four hours with such a single drop hammer driving system), it is very hard to investigate the effects of driving energy and time factors in such kind of model tests.

From the results of the analysis of 2x3 Latin square, it was found that the ultimate bearing capacity was not effected by the levels of the chosen variables, whereas the Linear-elastic behaviour region limit was related to the number of piles and pile spacing.

FOr 2x2 pile group, at 1.5 D (diameter) spacing, block failure and at greater spacings (2 D, 2.5 D, and 5 D) failure of single piles (group efficiency behaviour) was observed. For 3x3 pile groups, at 1.5 D spacing, again block failure was seen. The spacing that the type of group failure changes was close to 2 D spacing. The failure of single piles was observed for 2.5 D.

The Linear-elastic behaviour limits obtained by visual inspections are satisfactorily in aggreement with the comment given in the literature and equals to:

0.6 x Number of piles in group x the load capacity of a single pile

Since the soil container was of limited size, an increase in experimental bearing capacity has been observed with increasing pile number and increasing spacing between piles.

APPENDIX I

(Calculation of Compaction energy)

The nominal compaction energy supplied at proctor tests:

$$CE = \frac{\text{number of blows(75) x weigth of hammer(24.5N) x heigth of drop(.3}}{\text{Volume } (9.44x10^7}$$

$CE = 593.7\ kJ/m^3 = 593700\ J/m^3$

Volume of the container: $0.03532\ m^3$

If the volumes are replaced to find number of blows for the same energy:

$$593700\ Joule/m^3 : \frac{X \times 24.5N \times 0.305m}{0.03532\ m^3}$$

where X equals to the number of blows necessary to give same energy

X : 2807 blows

for 9 layers of compaction:

x : 2807/9 : 313 blows for each layer.

APPENDIX II

(Water content change along the depth of the container)

| | Depth(cm) | 1st loc. | 2nd loc. | 3rd loc. |
|---|---|---|---|---|
| bottom | 0-2 | 29.5 | 29.9 | 30.1 |
| | 2-4 | 30.2 | 29.4 | 29.7 |
| | 4-6 | 29.7 | 29.5 | 29.9 |
| | 6-8 | 30.6 | 30.1 | 30.1 |
| | 8-10 | 29.4 | 30.1 | 29.8 |
| | 10-12 | 29.8 | 29.1 | 29.5 |
| | 12-14 | 29.9 | 29.4 | 30.2 |
| | 14-16 | 30.5 | 29.8 | 30.5 |
| | 16-18 | 30.1 | 30.3 | 29.8 |
| | 18-20 | 29.5 | 28.9 | 29.9 |
| | 20-22 | 30.2 | 30.1 | 30.6 |
| | 22-24 | 29.6 | 30.4 | 30.3 |
| | 24-26 | 30.1 | 29.3 | 30.2 |
| | 26-28 | 30.2 | 29.8 | 29.2 |
| | 28-30 | 30.1 | 29.8 | 30.3 |
| | 30-32 | 30.9 | 29.5 | 30.5 |
| | 32-33 | 31.5 | 32.2 | 32.7 |
| | 33-34 | 42.0 | 42.5 | 43.9 |

APPENDIX III

Grain size analysis-hydrometer method

| Time of reading | Elapsed time min. | Temp. t°C | Actual Hyd. reading | Corr. Hyd reading | % Finer | Hyd. corr. only for menis. R | L from table | L/t | K from table | D, mm | % Finer |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 9.48 | | | | | | | | | | | |
| 9.49 | 1 | 15 | 53 | 43.9 | 86.92 | 54 | 7.4 | 7.4 | .0141 | .038 | 20.60 |
| .50 | 2 | 15 | 49 | 39.9 | 79.0 | 50 | 8.1 | 4.05 | " | .028 | 18.72 |
| .51 | 3 | 15 | 45 | 35.9 | 71.0 | 46 | 8.8 | 2.93 | " | .024 | 16.85 |
| .52 | 4 | 15 | 43 | 33.9 | 67.12 | 44 | 9.1 | 2.275 | " | .021 | 15.91 |
| .56 | 8 | 15 | 39 | 29.9 | 59.2 | 40 | 9.7 | 1.2125 | " | .015 | 14.03 |
| 10.03 | 15 | 15.5 | 36 | 27.0 | 53.46 | 37 | 10.2 | 0.68 | " | .012 | 12.67 |
| .18 | 30 | 15.5 | 33 | 24.0 | 47.52 | 34 | 10b7 | 0.36 | " | .008 | 11.26 |
| .48 | 60 | 16 | 29 | 20.1 | 39.8 | 30 | 11.4 | 0.19 | " | .006 | 9.43 |
| 11.48 | 120 | 17 | 25 | 16.3 | 32.27 | 26 | 12.0 | 0.10 | .0141 | .004 | 7.65 |
| 13.48 | 240 | 19 | 22 | 13.7 | 27.13 | 23 | 12.5 | 0.052 | .0136 | .003 | 6.43 |
| 17.48 | 480 | 20 | 18 | 10 | 29.8 | 19 | 13.2 | 0.0275 | .0134 | .002 | 4.69 |

G of solids: 2.70 a: 0.99

Dispersing agent: Na SiO Amount: 4 % in 125 ml

Wt. of soil, W : 50

Zero connection: 8 Meniscus correction: 1.0

# GRANÜLOMETRİ EĞRİSİ

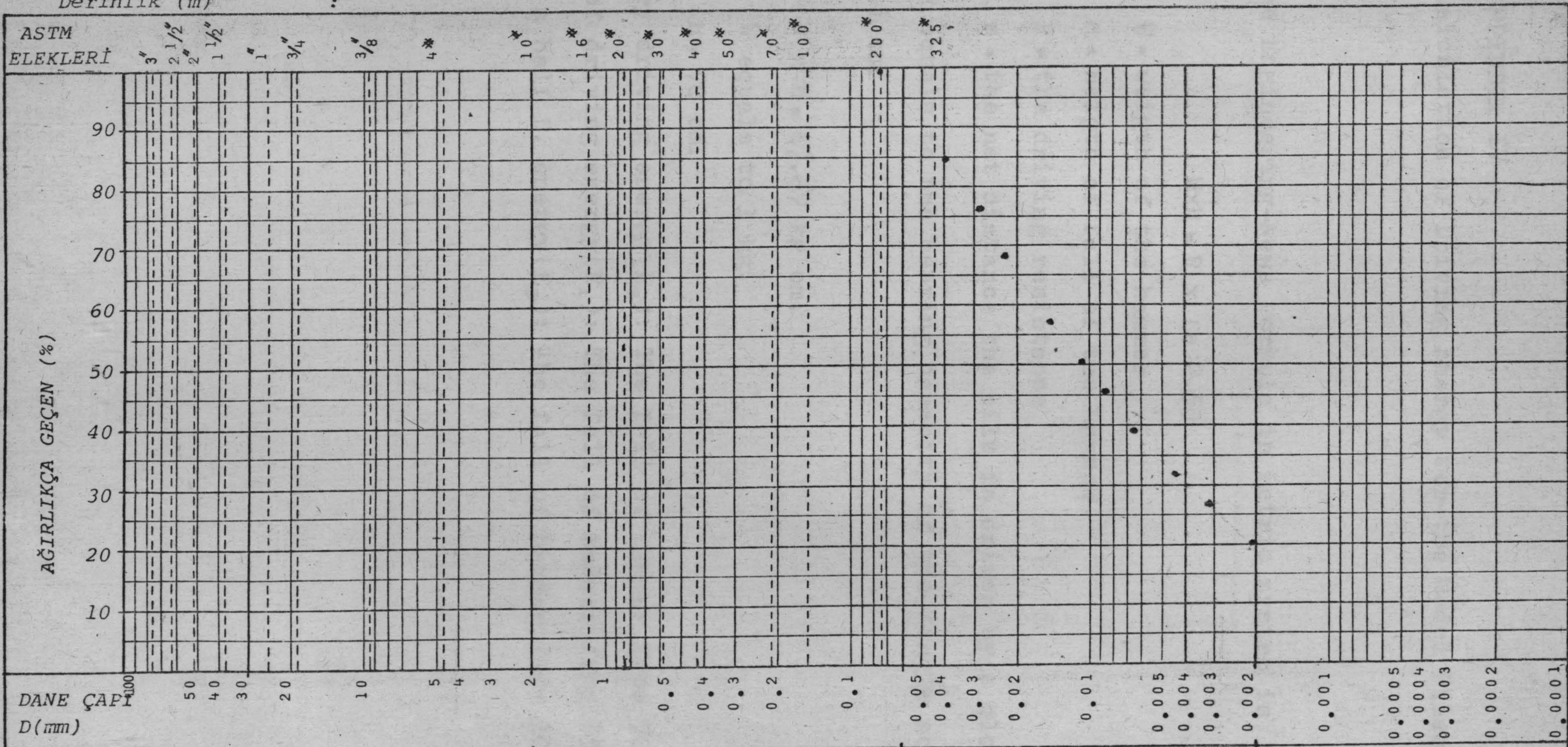
Nümunenin Geldiği Yer:

Sondaj No. :

Nümune No. :

Derinlik (m) :

Rapor No.:




| ZEMİN | TAŞ | ÇAKIL | | | KUM | | | SİLT | | | KİL | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | KABA | ORTA | İNCE | KABA | ORTA | İNCE | KABA | ORTA | İNCE | KABA | ORTA | İNCE |

TAŞ : %

ÇAKIL: %

KUM : %

$D_{60}$ :

$D_{10}$ :

NOT:

# APPENDIX IV

## Calcũlation of Driving Energy for the Model Piles:

The Engineering-News Formula in metric system is

$$W x H = R \times (s\ 2.5)$$

where W = weigth of the hammer

H = heigth of fall of the hammer

R = the driving resistance

s = the net distance the pile is driven by a blow.

Taking R equals to the bearing capacity of the single model pile and s = 0.1 cm.

W x H = 67.67 kg cm.

Taking W equals to 1 kg.

H = 70 cm.

necessary driving energy($E_2$): The fall of hammer from 70 cm heigth

half of driving energy($E_1$): The fall of hammer from 35cm heigth

one and a half D. energy($E_3$): The fall of hammer from 105cm heigth

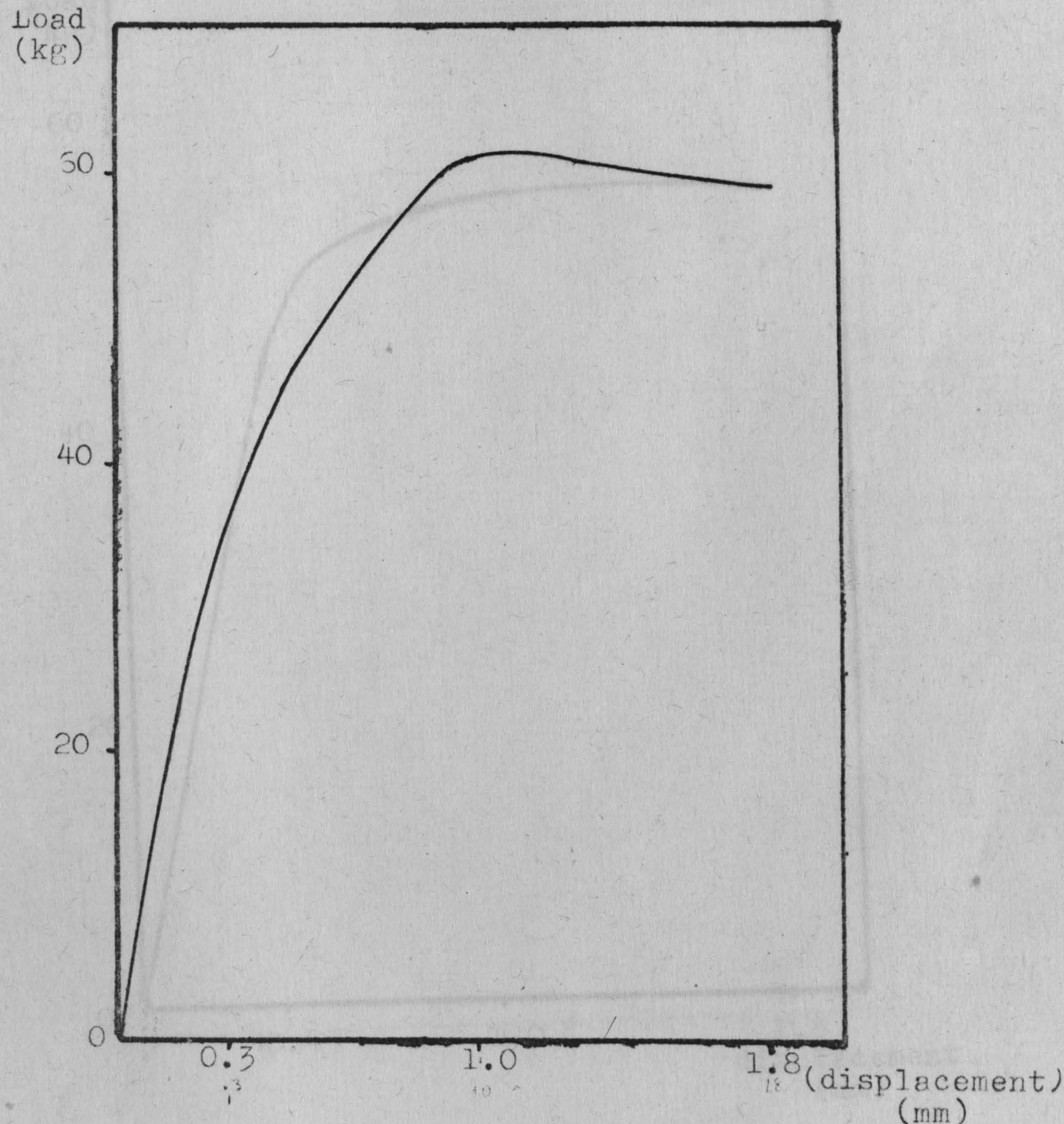



Figure 1. 1st single pile tests
load-displacement curve

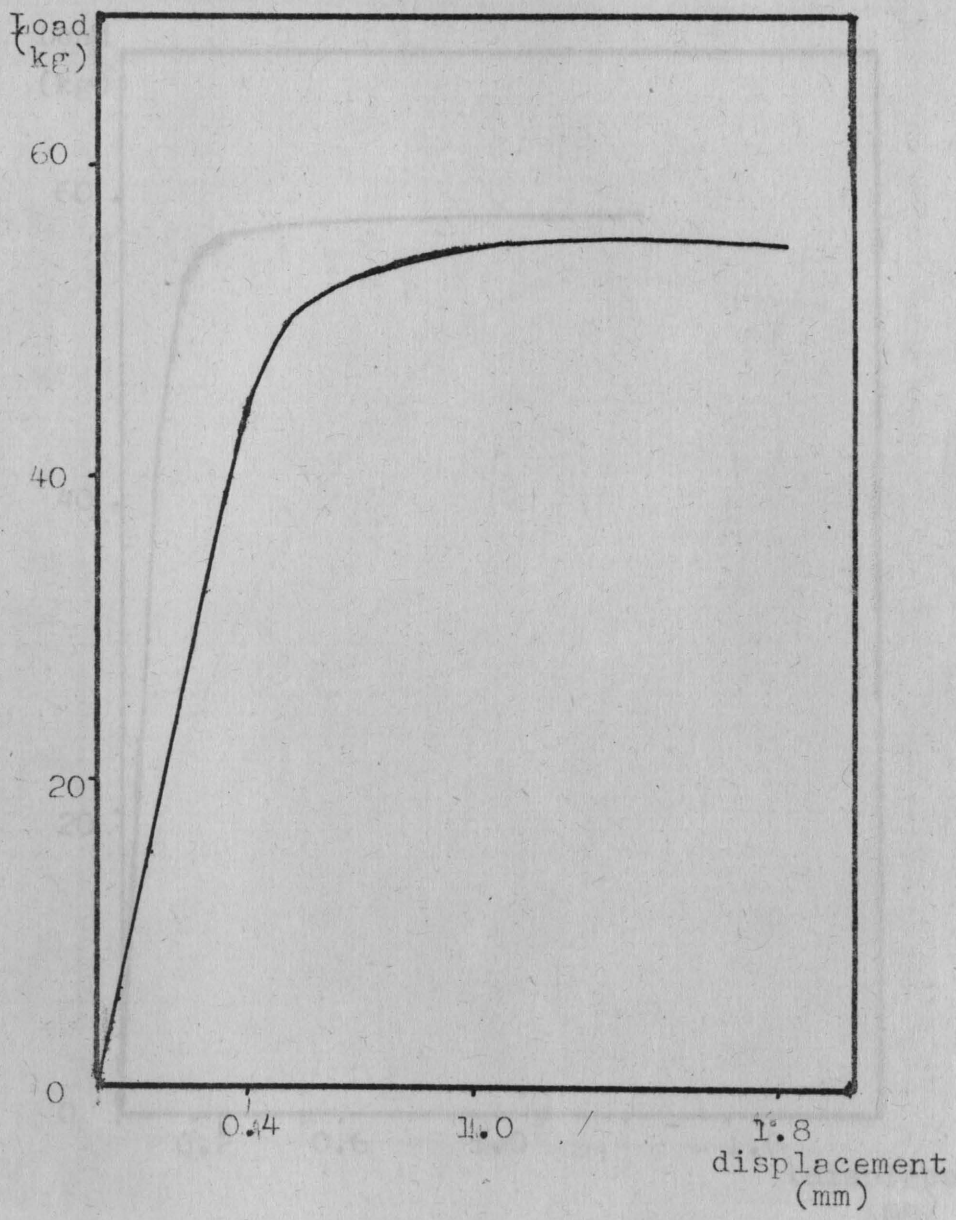



Figure 2. The load-displacement curve of 2nd single pile test.

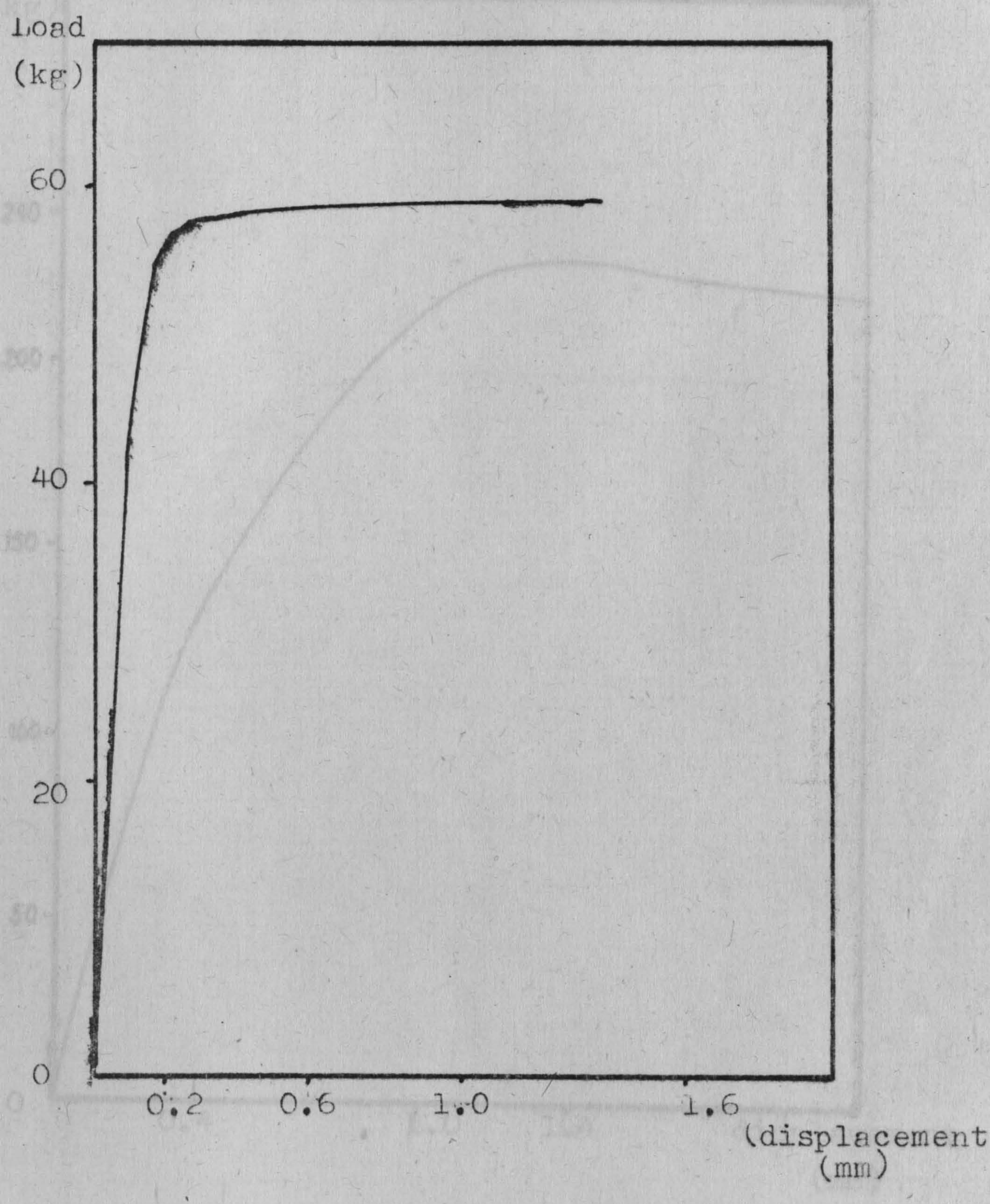



Figure 3. The Load-displacement curve
of 3rd. single pile test.

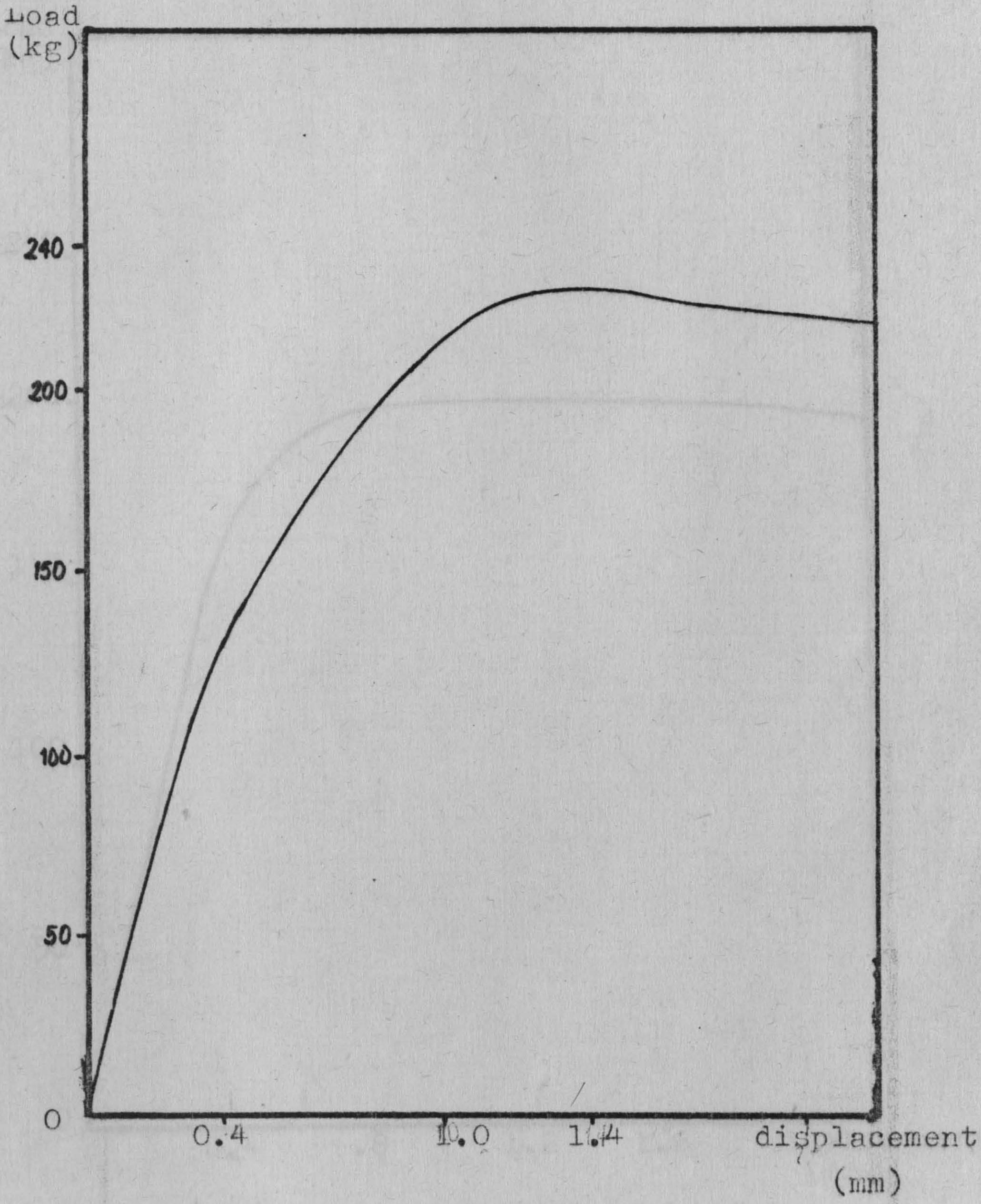



Figure 5. The Load-displacement curve
of 1st 2x2 pile group test.

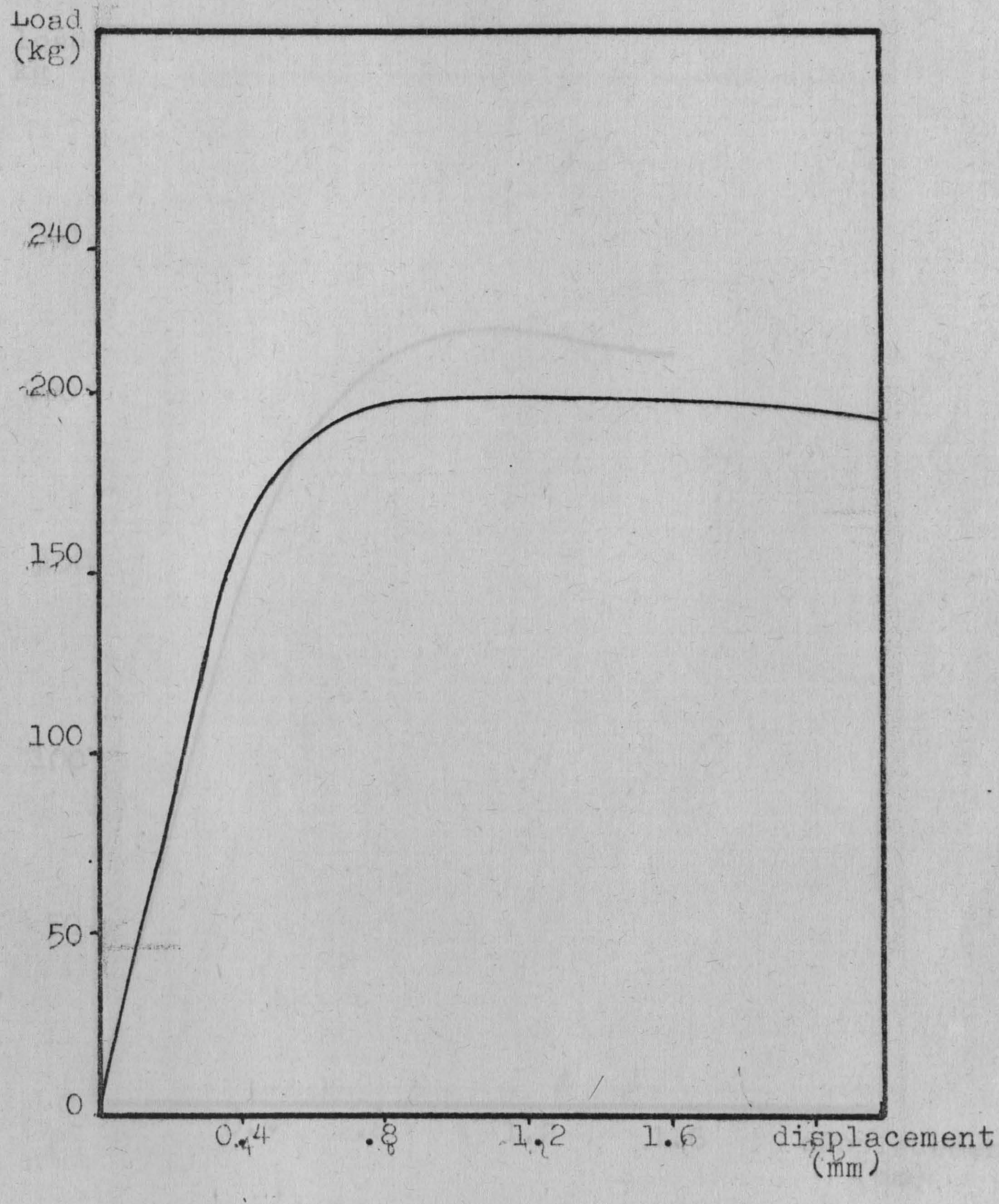



Figure 6. The load-displacement curve of 2nd 2x2 pile group test.

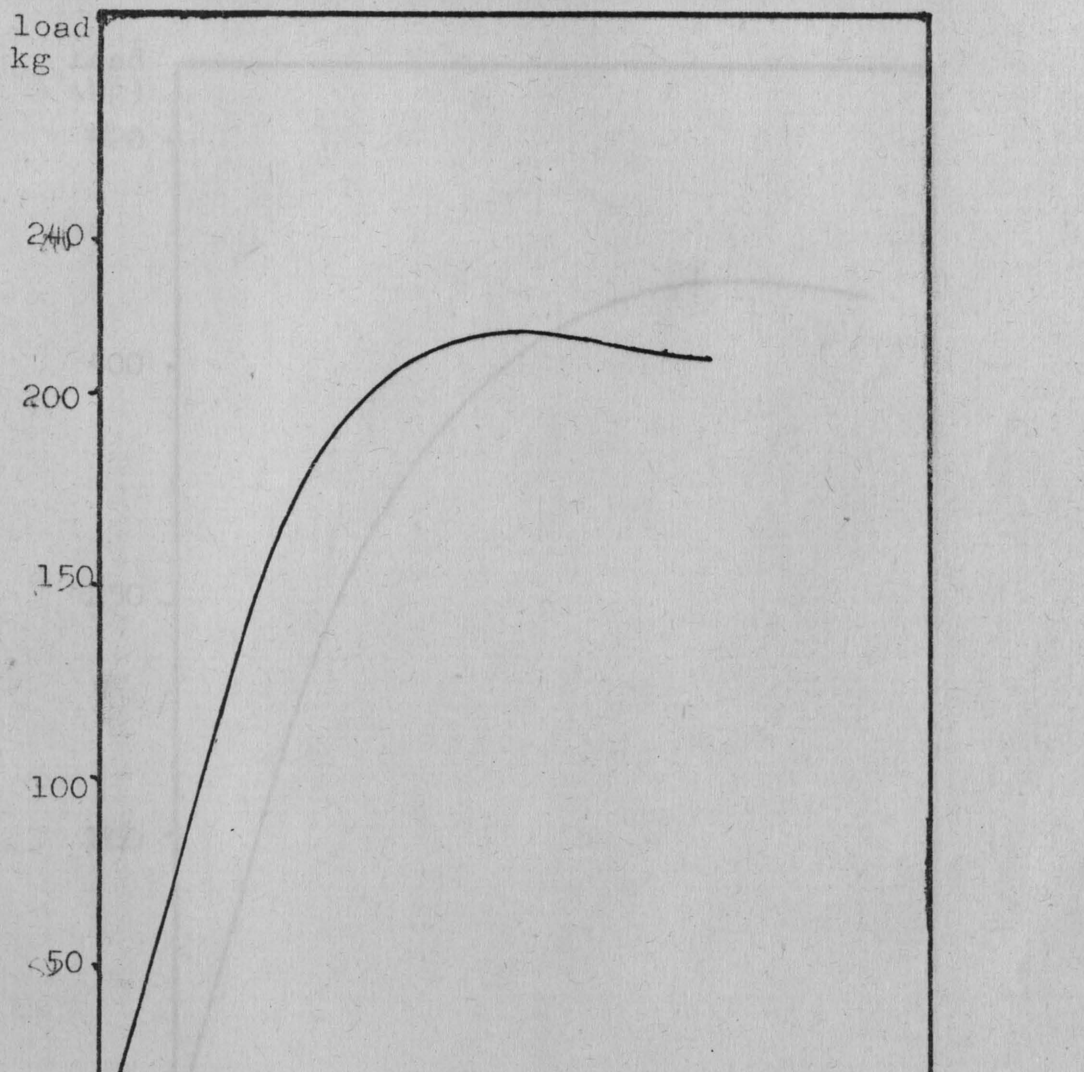



Figure 7. The load-displacement curve of 3rd 2x2 pile group test.

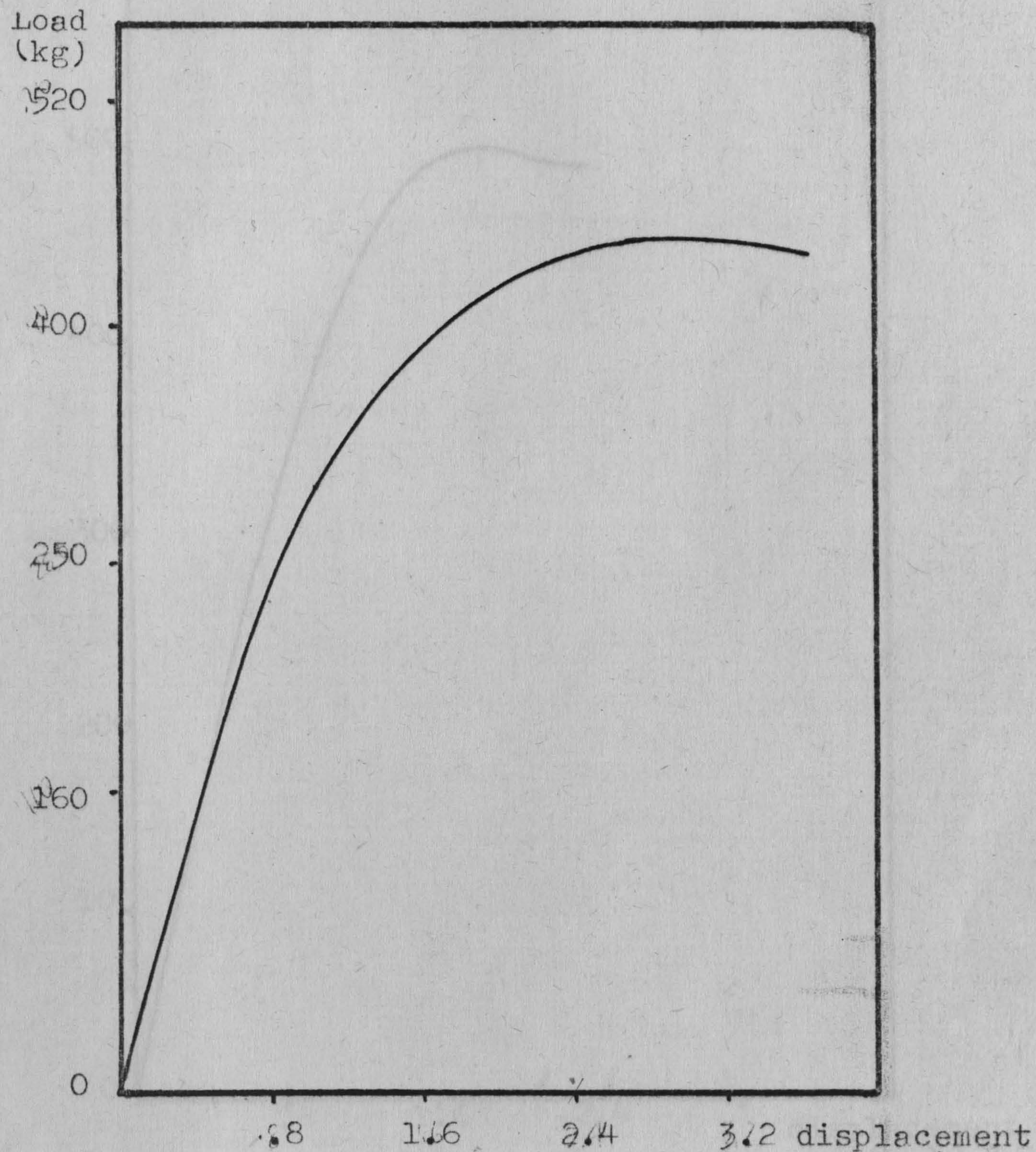



Figure 9. The ladd-displacement curve of 1st 3x3 pile group test.

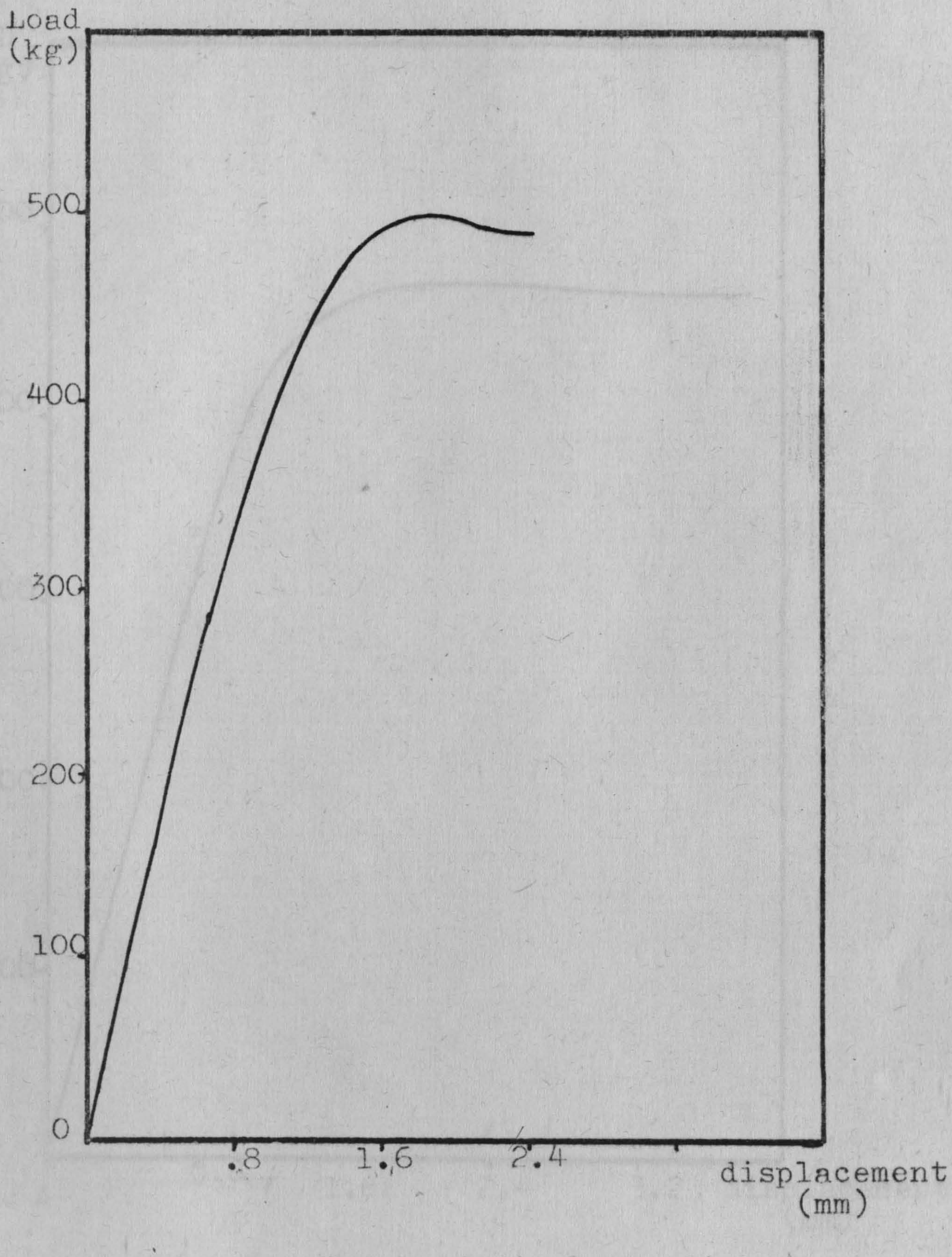



Figure 10. The load-displacement curve
of 2nd 3x3 pile group test.

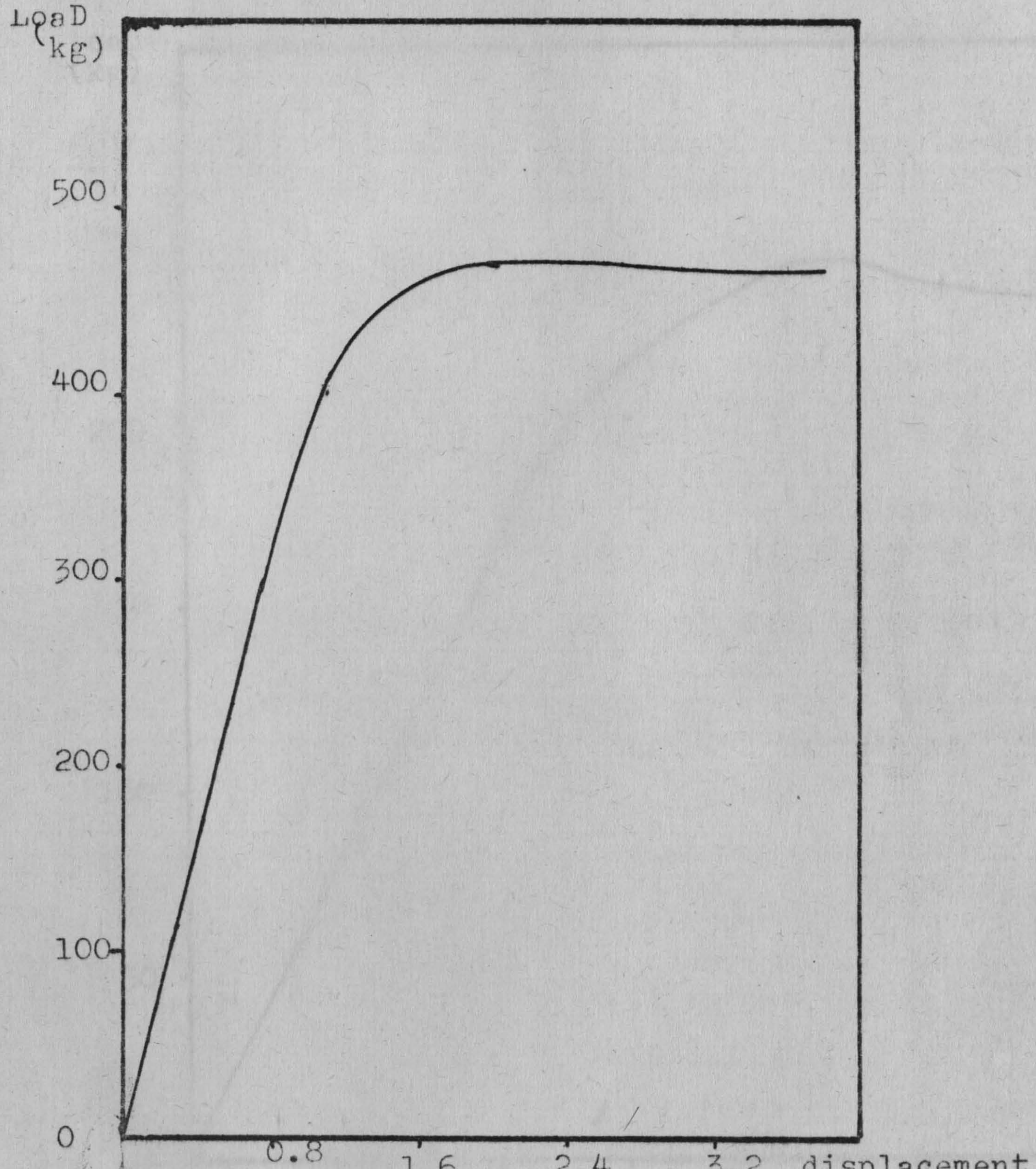



Figure 11. The load-displacement curve of 3x3pile group test 3

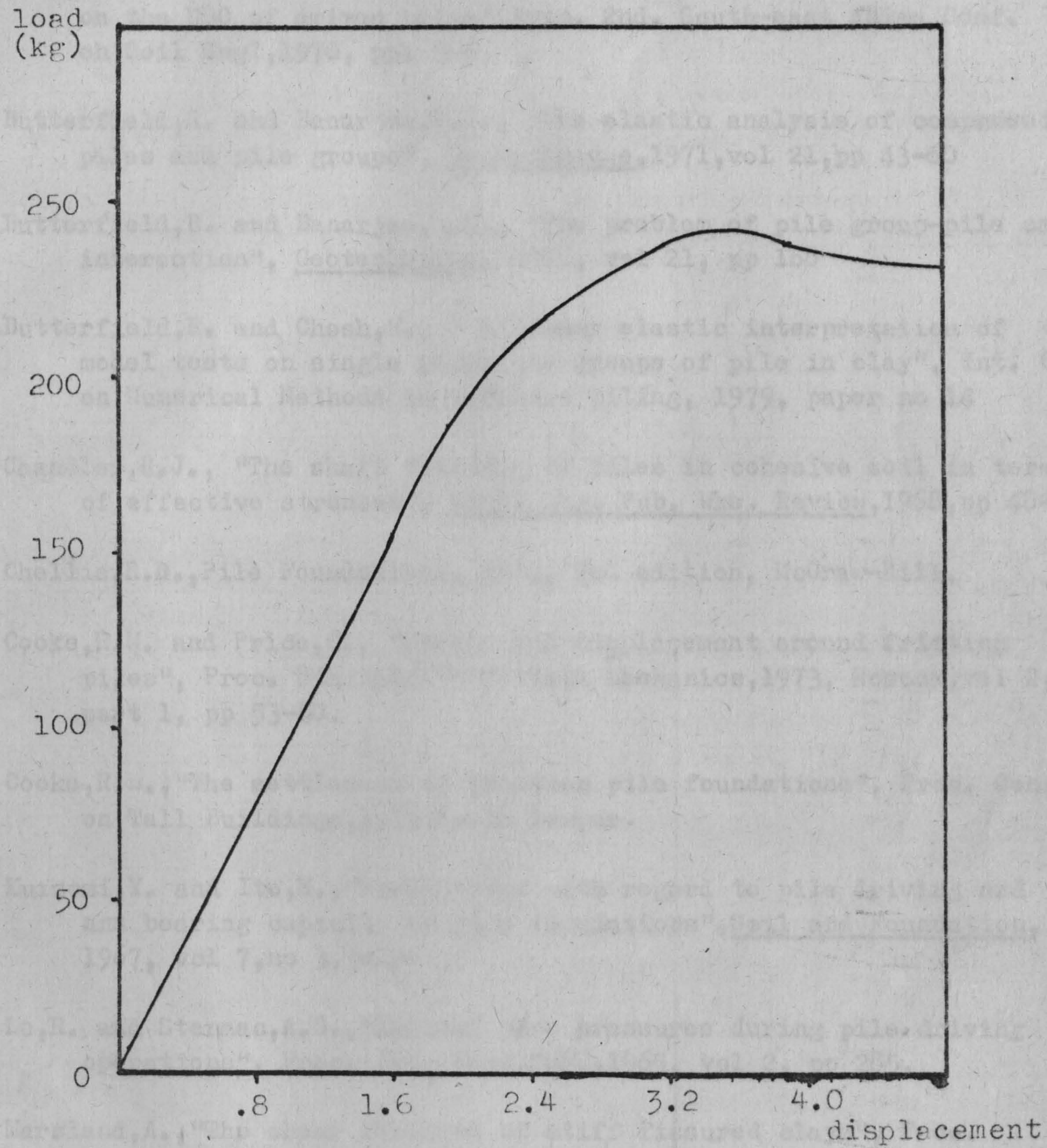



Figure 13. The load-displacement curve

of 2x2 pile group with 5 D

REFERENCES


1. Brand,E.W.,"Load tests on small foundations in soft clay",ASCE,1972, vol 1,part 2, pp 903-948

2. Butterfield,R. and Banarjee,B.K.,"The effect of pore water pressure on the UBC of driven piles",Proc. 2nd. South-east Asian Conf. oh Soil Eng?,1970, pp. 385

3. Butterfield,R. and Banarjee,B.K., "The elastic analysis of compressible piles and pile groups", Geotechnique,1971,vol 21,pp 43-60

4. Butterfield,R. and Banarjee,B.K., "The problem of pile group-pile cap interaction", Geotechnique, 1971, vol 21, pp 168

5. Butterfield,R. and Ghosh,N., " A linear elastic interpretation of model tests on single piles and groups of pile in clay", Int. Conf. on Numerical Methods in Offshore piling, 1979, paper no 14

6. Chandler,R.J., "The shaft friction of piles in cohesive soil in terms of effective stresses", Civil Eng. Pub. Wks. Review,1968,pp 48-51

7. Chellis,R.D.,Pile Foundations, 1961, 2nd edition, McGraw-Hill,

8. Cooke,R.W. and Price,G., "Strain and displacement around friction piles", Proc. 8th Int. Conf. Soil Mechanics,1973, Moscow,vol 2, part 1, pp 53-60.

9. Cooke,R.W.,"The settlement of friction pile foundations", Proc. Conf. on Tall Buildings,1975,Kuala Lumpur.

10. Kuizomi,Y. and Ito,K.,"Field tests with regard to pile driving and and bearing capacity of pile foundations",Soil and Foundation, 1967, vol 7,no 3,Tokyo

11. Lo,R. and Stermac,A.G.,"Induced pore pressures during pile driving operations", Proc. Int. Conf.SMFE,1965, vol 2, pp 285.

12. Marsland,A.,"The shear strength of stiff fissured clays", Proc. Roscoe Memorial Symp., 1971,Cambridge, pp 59-68.

13. Mattes,A. and Poulos,H.G.,"Settlement of single compressible piles", ASCE SMFE,1969, vol 95, SM 1, 189.

14. Mattes,A. and Poulos,H.G., "An analysis of end-bearing pile", Geotechnique,1969, vol 19,no 2.

15. Meyerhof,G.G.,"The UBC of foundations", Geotechnique,1951,pp 201-232

16. Meyerhof,G.G.," A study of the UBC of piles", Annales Ins.Tech., Paris,1953, pp 63-64.

17. Meyerhof,G.G. and Murdock,L.J., "An investigation of the UBC of the some bored and driven piles in clay", Geotechnique,1953 vol 3,pp 267-282.

18. O'neill and Reese, "Behaviour of bored piles in Beaumont clay", ASCE SMFE, SM 2.

19. Ottoviani,M.,"Three dimensional Finite element analysis of vertically loaded pile groups", Geotechnique,1975,vol 25,no 2.

20. POulos-H.G. and Davis,E.H.," The settlement behaviour of single axially loaded incompressible piles and piers", Geotechnique, 1968, vol 18, pp 351_371.

21. Poulos,H.G.,"Analysis of the settlement of piles", Geotechnique, 1968, vol 18, pp 449_471.

22. Roscoe-,"Soil and model tests", Jour. of Strain analysis, 1968, vol 3, no 1,pp 57-64.

23. Saffery,M.R. and Tate,A.P.K.," Model tests on pile groups in a clay soil with particular reference to the behaviour of the group when it is loaded eccentrically", Proc. 5th Int. Conf. on Soil Mechanics and Foundation Eng., 1961,Paris,vol 2,pp 129

24. Seed,H.B. and Reese,L.," The action of soft clay along friction piles", Transection ASCE,1957, vol 122, pp 731_754.

25. Skempton,A.," Cast in-situ piles in London clay", Geotechnique, 1959, vol 9, pp 153-173.

26. Sowers," The bearing capacity of friction pile groups in homogeneous clay from model studies",Proc. 5th Int.Conf. SMFE,1961, vol 2, pp 155.

27. Tomlinson,M.J."Some effects of pile driving on skin friction", Behaviour of piles, 1970, Inst. of Civil Engineers, London, pp 107-114.

32. Whitaker,T.,"Experiments with model pile groups",Geotechnique, 1957, vol 7, pp 147-167.

29. Whitaker,T.,"The constant rate of penetration test", Proc. Inst. Civil Engr.,1963,pp 119-123.