PERFORMANCE OF BORED PILES IN ALLUVIAL SOILS OF BANGLADESH

BY

MOHAMMAD ABU SADEQUE

A Project Report

Submitted to the Department of Civil Engineering, Bangladesh University of Engineering and Technology in partial fulfilments for the Degree

of

MASTER OF ENGINEERING

Department of Civil Engineering
Bangladesh University of Engineering and Technology
Dhaka

February 1989

PERFORMANCE OF BORED PILES IN

ALLUVIAL SOILS OF BANGLADESH

A PROJECT REPORT

BY

MOHAMMAD ABU SADEQUE

Approved as to style and content by:-

M. Hossain Ali

(Dr. M. Hossain Ali) Chairman
Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

A.M.M. Safiullah 15/2/89

(Dr. A. M. M. Safiullah) Member
Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

(Dr. M. Zoynul Abedin) Member
Associate Professor,
Dept. of Civil Engineering,
BUET, Dhaka.

ABSTRACT

Available literature related to fundamental concept of piling, especially the behavior of bored pile and soil-pile interaction in different soil conditions of Bangladesh have been studied. The study reveals that in most of the cases neither the proper pile driving technique, as suggested by the code of practice is followed nor the quality control is assured. As such, the present study suggests a full scale pile load test prior to service piling work.

In this study the data of five pile load tests on three different sites of Dhaka city are analysed and compared with the existing theoretical results. The variable considered are critical depth, loosening effect of soil and ground water level. Significant variations are noticed between experimental and theoretical results.

ACKNOWLEDGEMENT

I wish to express my appriciation for the help and constractive comments given by my supervisor, Dr. M. Hossain Ali, Professor of Civil Engineering Department for the preparation of this dissertation. I am also greateful to Dr. A. M. M. Safiullah, Professor of Civil Engineering Department, Dr. Md. Humayun Kabir, Professor of Civil Engineering Department, Dr. M. Zoynul Abedin, Associate Professor of Civil Engineering Department for their help and sincere co-operation.

I would like to extend my thanks to all teachers of the Bangladesh University of Engineering and Technology for their kind help and sincere co-operation throughout the course of my study at this University. I am grateful to M. Ahmed & Co. and Soil Tech International Ltd. for their co-operation in data collections.

Finally, I express my sincere and profound gratitude to my mother, wife and brothers who have been very patient and understanding throughout the course.

DECLARATION

I hereby declare that the project work submitted herewith was performed by me and has not been submitted for any other degree previously.

(MOHAMMAD ABU SADEQUE)

LIST OF NOTATION

A_p = Area of pile base.

B = Least dimention of pile.

C = Cohesion of soil.

C_a = Shear strength between soil and pile.

D = Depth of embedment of pile.

d = Diameter of pile shaft.

D_r = Relative density of soil.

K = Co-efficient of earth pressure.

K_a = Co-efficient of active earth pressure.

K_p = Co-efficient of passive earth pressure.

K_o = Co-efficient of earth pressure at rest.

L_w = Liquid limit.

L_p = Plastic limit.

N = Number of blows in standard penetration test.

N_c, N_γ, N_q = Bearing capacity factors for deep foundation.

Q = Total pile capacity.

Q_e = Load carried by pile end only.

Q_f = Shaft frictional force at pile failure.

Q_p = Point bearing load at failure.

Q_s = Load carried by pile shaft only.

Q_u = Ultimate pile capacity.

q = Surcharge.

S_u = Undisturbed undrained shear strength.

Z_c = Critical depth of the embeded pile.

α = Adhesion factor.

γ = Unit weight of soil.

δ = Angle of wall friction between pile and soil.

ϕ = Angle of shearing resistance.

σ_v = Effective vertical pressure.

σ_h = Effective horizontal pressure.

C O N T E N T S

LIST OF FIGURES

LIST OF THE TABLES

CHAPTER-1

INTRODUCTION

Piling is both an art and science. The art lies in selecting the most suitable type of pile and method of its installation for the ground conditions and the type of loading. Science enables the engineers to predict the behaviour of the piles once they are installed in the ground and subjected to loading. This behaviour is influenced profoundly by the methods used to install the piles and it can not be predicted solely from the physical properties of the piles and of the undisturbed soil. A knowledge of the available type of piling and method of constructing piled foundations is essential for a thorough understanding of the science of their behaviour. A pile foundation, even a single pile, is statically indeterminate to a very high degree. The chance of a precise analysis of a pile is thus even more remote than is true for most problems in geotechnical engineering Empirical knowledge and the results of pile load tests at the actual site are usually adopted for the solution to a given pile foundation problem. There are a number of good state of art treatments of deep foundations. But most of these refers to behaviour of single piles, as is evident from Fig. 1.1

Since Dhaka soil strata is devoid of any stone or rocky layer, it has been a common practice so far to transfer the building load to a deeper strata of soil. Here, in the field of piling engineering, bored piles or cast in situ piles are most common because of their suitability with the site conditions and also because of easier method of their installation. Excessive noise, vibration etc. do not permit installation of a driven pile. Thus although bored piles are used quite frequently, here there are no sufficient field report and pile load test results which is must for rational analysis of a pile. In most cases, reliance is made on load test results to determine the load carrying capacity of bored piles. However, considerable load test results in different types of soils are needed to develop rational methods for design of bored piles.

In this project work it has been tried to examine design data of some piles with their capacity obtained from pile load test results. The piles

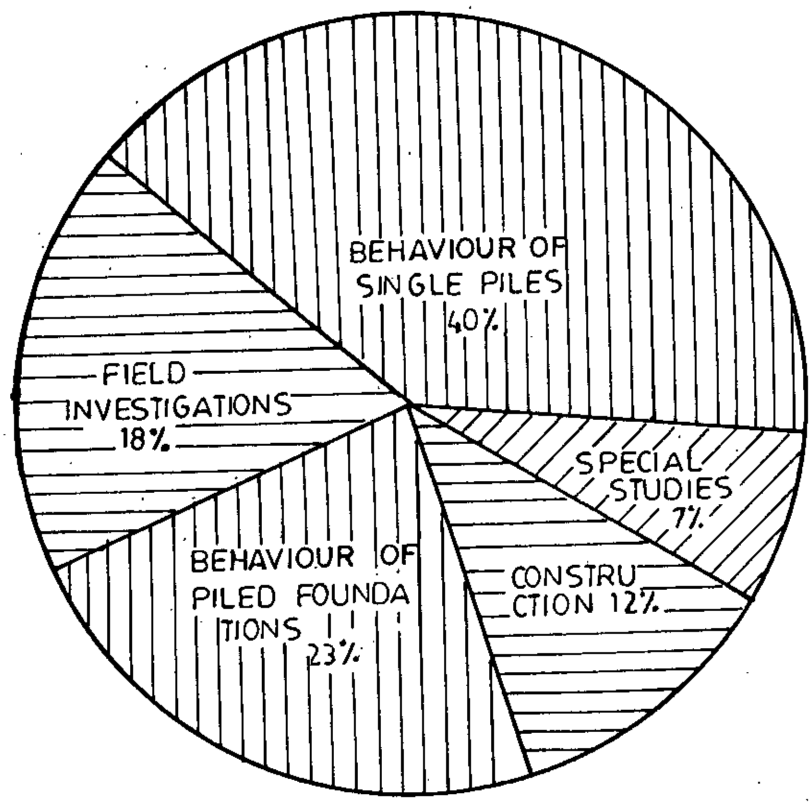


FIG. 1-1. DISTRIBUTION OF SUBJECTS IN CONTRIBUTIONS DEALING WITH PILES PRESENTED AT SOIL MECHANICS CONFERENCE (AFTER NAYEK[1])

have been analysed considering the available design criteria and present construction technique of bored piles. Here attempts have been made to determine the effect of installation method on actual capacity of piles and also pile-soil behavior of Bangladesh soil context. Finally in the light of load test results the causes of variation between the theoretical capacity and actual capacity have been evaluated.

CHAPTER-2

OBJECTIVE

Dhaka city, assumed a new dimension, after the liberation of Bangladesh in 1971, due to the fact that the city has overnight turned into a National Capital. As is normal with any Capital city of a developing country, Dhaka is in the process of fast changing from a small provincial Capital into a city of tall buildings. The surge of tall buildings is being strongly felt since 1978 or so.

Though the pile foundation has been adopted extensively for the tall building so far and there is every reason to assume that it will continue to remain so for the future buildings as well, there has not been any serious study into the factor of safety of such piles. It has been observed that the factor of safety assumed ranges from 2.5 to 3.0 from designer to designer apparently without giving due importance to the consequences in many cases.

And as such, It is felt that a study on the correlation of theoretical and actual capacity of piles will help designers to arrive at a consistant value of factor of safety with a subsequent benifit in safety and economy.

With the above in view, the objectives of the present study are as follows:-

1. To analyse the pile theoretically with the halp of sub-soil investigation report.
2. To predict the ultimate load carrying capacity by studying time-load-settlement curve obtained during field load test.
3. To compare and correlate the pile load capacity obtained from theoretical analysis with that of the result from load test.
4. To find out the causes of variations (if any) of the two results.
5. To draw a conculasion regarding the theoretical pile capacity in context with Bangladesh soil.

CHAPTER-3

LITERATURE REVIEW

3.1 Introduction

The capacity of a pile is determined on the basis of following two basic considerations,

(i) The structural capacity of the pile to support the load coming on it.

(ii) The support provided by the surrounding and underlying soil or rock.

The pile capacity is the smaller of the two values arrived from the above two considerations.

Structural capacity is governed by the permissible stresses in the pile materials. Generally building codes stipulate the maximum allowable material stresses. However, values based on different codes may differ greatly and usually they tend to be conservative. The recent practice is to design the piles as columns. However, it is necessary to realise that the factor of safety for piles when designed as columns should be higher than that allowed for columns in superstructures. In case of a superstructure the accuracy of the column straightness and alignment is assured within relatively narrow limits, and these columns are inspected after they are casting and also they are available for maintenance. But in case of piles, the alignment and straightness of piles are much less controllable. Concrete in cast in situ piles cannot be inspected. The environmental conditions under which piles are placed are usually more severe and also driving of piles introduces residual stress of unknown magnitudes. These facts clearly point to the need to have higher factor of safety when piles are designed as columns.

When the pile capacity is determined on the basis of the support provided by the surrounding and underlying soil or rock, a number of factors affecting the properties of surrounding and underlying soils must be considered. The degree to which the surrounding and underlying soils are affected are determined by the type of pile, the type of soil, method of installation of piles etc. The two types of piles, viz; precast and bored affect the surrounding and underlying soil differently mainly because of their different installation methods. In this Chapter the literature concerning the effect of installation of bored cast in situ piles on the surrounding and underlying soil will be considered.

The effect of installation of bored piles in clay soil have been studied largely in relation to adhesion between the pile and the soil. The adhesion has been found to be less than the cohesion of soil mainly because of softening of the clay immediately adjecent to the soil surface. This softening may arise from three causes. These are;

(a) Water poured into the boring to facilitate operation of the cutting tool.

(b) Migration of the water from the body of the clay toward the less highly stressed zone around the borehole.

(c) Absorption of moisture from the wet concrete.

Factor (a) may be eliminated by using good drilling technique and (b) can be minimized by carrying out the drilling and concreting operation as rapidly as possible.

Mayerhof and Murdock [8] measured the water contents of the clay immediately adjecent to the shaft of a bored pile in London clay and found an increase of nearly 4% of water content at the contact surface, although at a distance of 3 inch from the shaft, the water contents had not altered. This increase should be a maximum value, as the hole was drilled by hand and took two to three days to complete. For London clay Skempton[7] showed that an increase in water content of only 1% results in a 20% reduction in ratio of C_a/C_u, where C_a is the undrained shear strength of soil after installation of pile shaft and C_u is the original undrained shear strength. For a 4% increase in water content, C_a/C_u is reduced by about 70%.

Construction problem may also arise with bored piles, such as -

(a) Caving of the borehole, resulting in necking or misalignment of the pile.

(b) Aggregate separation within the pile.

(c) Buckling of the pile reinforcement.

Such structural defects may be difficult to detect since a load test may not reveal any abnormal behaviour, especially if the load is only taken to the design load.

There is relatively little quantitative information on the effects of installation of bored piles in sand or in any other cohesionless soils. Such piles usually require a casing and/or drilling fluids to support the walls of the bore during sinking of the hole. Subsequent withdrawal of the casing while concreting the shaft is likely to disturb and loosen the soil to some extent. Also some loosening is liable to occur at the bottom of the pile as a result of bailing the hole, And when this is done under water, the upward surge on withdrawal of the bailer can loosen the soil for several feet below and around the pile. Thus in calculating the load capacity of a bored pile in sand, Tomlinson[4] suggests that the ultimate value of angle of shearing resistance ø should be used, unless the pile is formed in a dense gravel when the 'surging' effect may not take place. If heavy compaction can be given to the concrete at the base of the piles, then the disturbed and loosen soil may be recompacted and value of ø for the dense state should be used. However, if the shaft is obstructed by the reinforcing cage, such compaction may not be possible.

3.2 Bearing Capacity of Piles

The ultimate bearing capacity or the ultimate bearing resistance, Q_u, of a pile is defined as the maximum load which can be carried by a pile and at which the pile continues to sink without further increase of load. The allowable load, Q_a, is the safe load that can be applied to a pile after taking into account the ultimate bearing resistance, the permissible settlement and overall stability of the pile foundation.

The bearing capacity of piles can be estimated in a number of ways which are all based on one of the followings:

(i) Static formulae, requiring a knowledge of the failure mechanism and the shear strength of the supporting soil.

(ii) Dynamic formulae, which equate the energy required to drive a pile to the static load carrying capacity.

(iii) Static field penetration test especially the Dutch cone penetration test, in which the failure mechanism of soil is similar to that in a pile.

(iv). Wave equation, and

(v) Direct load test on piles.

In this study only bored piles are considered and the most relevent methods are reviewed here in the following sections.

3.2.1 Pile Capacity by Static Formulae

The static formulae estimate the bearing capacity of a pile considering the properties of the medium through which it passes. The static load 'Q' supported by a pile can be thought of as being the sum of the frictional force on the pile shaft, Q_s and the load carried by the pile toe, Q_e.

Thus $Q = Q_e + Q_s$ (3.1)

The load supported by the pile at failure Q_u, is given by

$$Q_u = Q_p + Q_f \quad (3.2)$$

where, Q_u = Ultimate pile capacity

Q_p = Point bearing load at pile failure

Q_f = Shaft frictional force at pile

It should be noted here that, in practice, Q_p and/or Q_f might be less than the maximum values of the Q_p maximum and Q_f maximum.

Failure of a pile occurs when the sum of the two components of resistance i.e. Q_p and Q_f is a maximum, and the maximum values of each do not necessarily occur at the same vertical settlement of the pile.

The maximum point bearing load is usually calculated using following bearing capacity equation for deep foundations,

$$Q_p = A_p\ (cN_c + 0.5\gamma BN_\gamma + qN_q) \qquad (3.3)$$

where A_p = Area of the pile tip

Q_p = Point or toe resistance

c = Cohesion of the soil at pile toe

γ = Average unit weight of the soil

B = Least dimension of the pile tip

q = Surcharge

D = depth of embedement of pile

N_c, N_γ, N_q = bearing capacity factors for deep foundation.

The maximum total shaft friction can be expressed as

$$Q_f = C_a A_s \qquad (3.4)$$

where C_a = adhesion between soil and pile

A_s = Surface area of embeded pile shaft

The proportions of the maximum values of end resistance and shaft resistance mobilized when a pile fails will depend on the soil strength and on the stress-strain characteristics of the pile soil system. Piles which are installed entirely in clays are likely to resist applied loads largely by shaft friction unless the length/diameter ratio is very small and this might also be true for piles in homogeneous sand deposits. Such piles are commonly referred to as "friction piles". When the end bearing capacity of a pile is very high, however, the end resistance will predominate at failure and such a pile is generally termed an "end-bearing pile". The most common form of end bearing pile is one which penetrates a soil of low strength and has its toe situated in a stratum of relatively high strength.

(a) End Resistance for Cohesive Soils:-

In purely cohesive saturated soils, the minimum ultimate load is reached when the pile is loaded under undrained conditions. This is the well-known $\phi=0$ condition (SKEMPTON[7]), which gives $N_\gamma = 0$ and $N_q = 1$. The maximum end resistance of the pile then becomes :

$$Q_p \max = A_p (CN_c + \gamma D) \qquad (3.5)$$

If the weight of the pile is assumed equal to the weight of soil displaced during installtion, the net maximum end load is:

$$Q_p \max = A_p CN_c \qquad (3.6)$$

The value of N_c under undrained conditions has been determined in past investigations by both experimental and analytical methods. The theoretical analysis of MEYERHOF[8] resulted in a value of Nc between 9.3 and 9.8 depending on the frictional resistance developed at the pile toe. SKEMPTON[7] found from full-scale experiements that $N_c = 9$ was sufficiently accurate for the calculation of the maximum end resistance for bored piles in London clay, and several other subsequent investigations have tended to support this value for stiff clays.

(b) Shaft Resistance for Cohesive Soils:

The adhesion on the shaft of a pile in a cohesive soil is found to be directly related to the undrained shear strength of the soil by the relationship:

$$Q_f \max = A_s \, \alpha \, S_u \qquad (3.7)$$

where α = adhesion factor,

S_u = undisturbed undrained shear strength,

A_s = area of pile shaft

The installation of a pile in a soft clay will cause remoulding of the soil in the vicinity of the pile resulting in a decrease in soil strength. A regain of the strength usually occurs gradually but is often not complete until a considerable time after pile installation. In stiff clays, this regain in shear strength with time is usually very slow and complete regain is sometimes never achieved.

The adhesion factor, α, between clay and a pile shaft has been found to vary from unity to about 0.3, its value decreasing with increasing undrained strength of the soil. This general trend of decreasing adhesion factor with soil strength are shown in Fig. 3.1[6].

The explanation of why the adhesion factor for a clay is less than unity is that the shear strength between the clay and the material of the pile is lower than that of the clay alone. It is not clear, however, why α should decrease with increasing soil strength. TOMLINSON[6] suggested that this is because the driving of a pile results in a hole slightly larger than the pile diameter. When the soil has a high shear strength, the enlarged hole remains open without lateral support and the soil does not readily flow back around the pile and as such the adhesion between the pile and the soil is less than the shear strength of the soil. In case of soft clay, however, the soil will flow to fill any space close to the newly driven pile to give complete contact between the pile and the clay with a consequent higher adhesion factor.

For bored piles, the adhesion factor is usually found to be lower than for driven piles, and it has been suggested that this is because of the reduction of lateral pressure by the process of boring the hole. SKEMPTON[7] recorded values of α

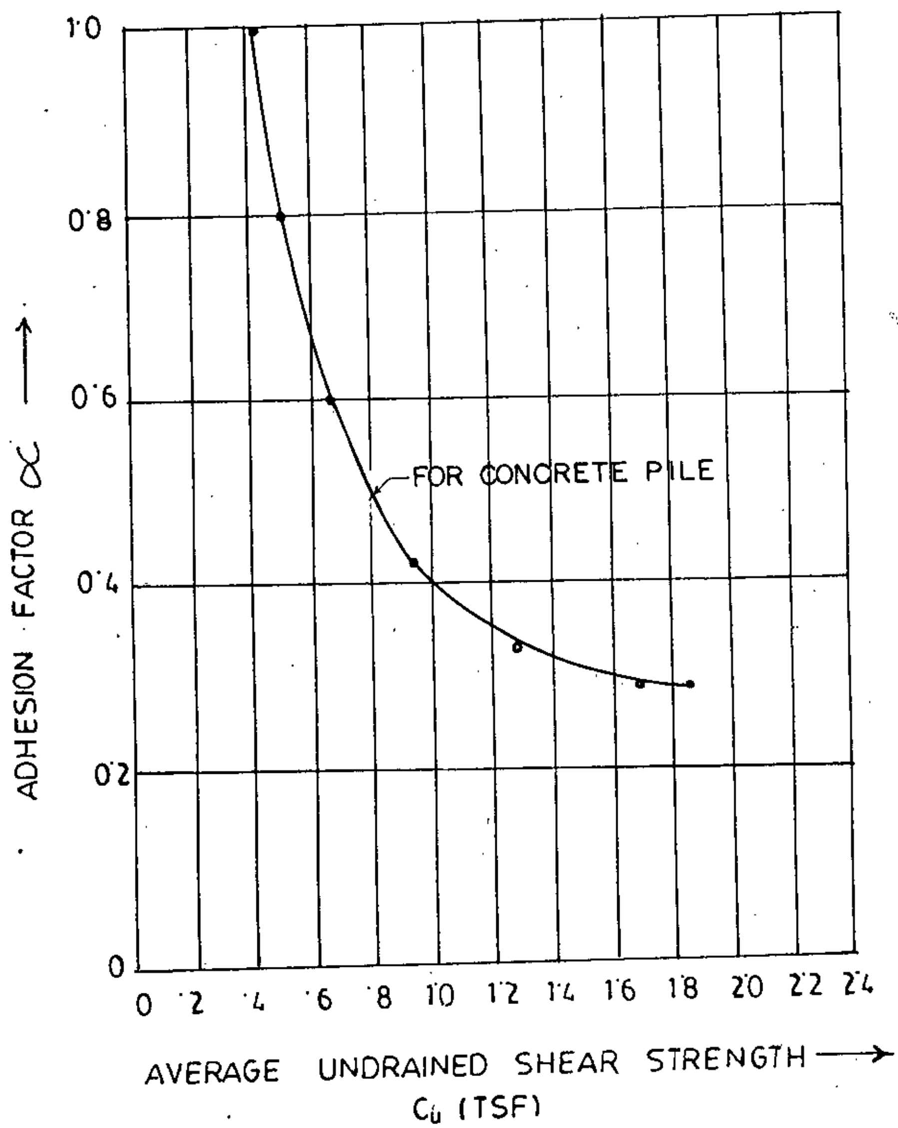


FIG. 3.1 . ADHESION FACTORS VS UNDRAINED SHEAR STRENGTH (AFTER NAYEK[1])

between only 0.3 and 0.6 for bored piles in London Clay. Low adhesion factors for bored piles have been attributed to other causes; particularly to softening of the clay at the sides of the bore hole due to moisture content increase during boring and concreting. Although there exists no satisfactory explanation of the mechanism of adhesion between a clay and the shaft of a pile, reasons for variations in C_a with certain properties of the pile shaft are not difficult to establish. A tapered pile, for example, will normally develop higher adhesion than an uniform section as a result of the better contact between the pile and the surrounding soil. It has been found that dissipation of the pore pressures after pile installation results in increased effective stresses between the soil and the pile surface because of decreased moisture content of the clay. For this reason, because of their relatively high permeabilities, piles of timber and concrete usually have higher adhesion than those of steel which do not permit excess pore pressure to dissipate readily.

Bored piles can be divided into two broad categories depending on method of installation, viz; (1) piles installed by boring carried out with bentonite and (2) piles installed by boring carried out without bentonite slurry to support the sides of bore holes.There will be softening effect on clay where the clay has an opportunity to absorb moisture from the concrete. For bored cast in situ concrete piles where bentonite mud is used to stabilize the sides of the bore holes, adhesion factor can be taken as unity for soft clays. For very soft to stiff clays adhesion factor varies from 1 to 0.3 and on average 0.54 can be taken for hard clays. For bored piles, upper limit of adhesion is taken as about 0.95 because no passive pressure develops in case of bored piles.

According to Tomlinson[6] the effect of drilling is to cause a relief of lateral pressure on the walls of the hole. This results in swelling of the clay and there is a migration of pore water towards the exposed clay face to cause softening of clay. If bentonite is used to support the sides of the borehole, the migration of porewater from clay due to relief of lateral pressure, and flow of water from any fissures, will not occur, but the bentonite may not be entirely removed from the interface between the clay and the concrete as the latter is placed and there will thus be the effect of a soft slurry on the contact face.

The effect of softening on the skin friction of bored piles in London clay was studied by Skempton[7], who suggested that the adhesion factor ranges from 0.3 to 0.6 for a number of load test results. He recommended a value of 0.45 for normal conditions where drilling and placing concrete followed a reasonably rapid sequence. Tomlinson[6] recommends a lower adhesion value for bored piles where there may be a long delay between drilling and placing the concrete.

(c) End Resistance for Cohesionless Soils:

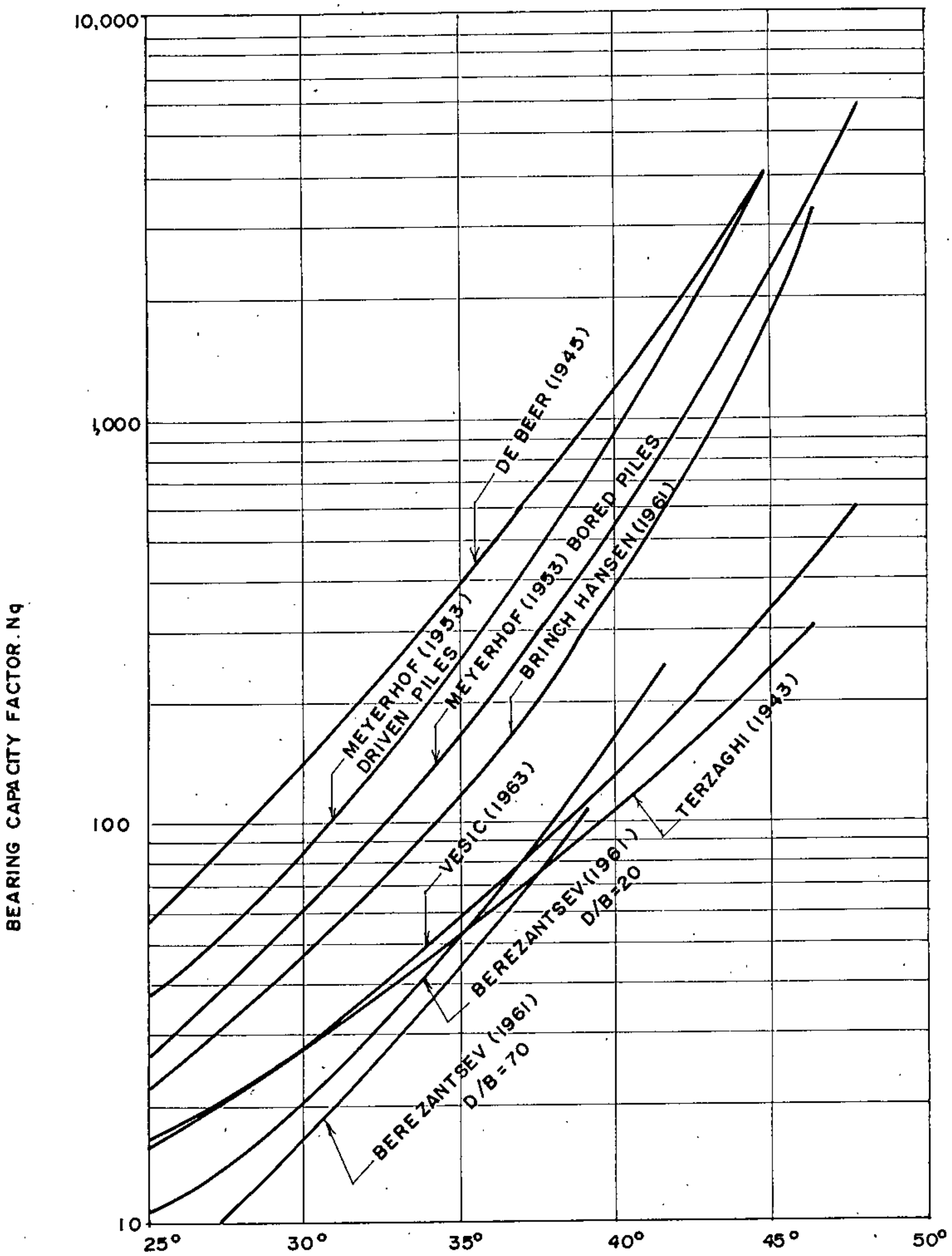
For a pile with its lower end embedded in a sand or gravel layer, where c = 0, the bearing capacity of the end becomes :

$$Q_p \max = A_p (\gamma B/2 N_\gamma + \gamma D N_q) \qquad (3.8)$$

Since for all but large diameter bored piles, B/2 is small compared with D, this approximates to :

$$Q_p \max = A_p \gamma D N_q \qquad (3.9)$$

The relationship between Nq and ø has been studied by many investigators, such as De Beer, Meyerhof, Vesic, Verezantsev etc. (Fig. 3.2.) Comparing the



ANGLE OF INTERNAT FRICTION Ø

FIG: 3.2. BEARING CAPACITY FACTOR FOR CIRCULAR DEEP FOUNDATION IN SAND*

*CITED BY BRAND[5]

observed and the theoretical values, NORDLUND* . and VESIC* found that the N_q and ϕ relationship proposed by BERZANTAEV* (Fig.3.2) takes into account the ratio B/D which conforms most closely to the results of load tests.

Further research of VESIC[8] has shown that the calculations of base resistance from the equation becomes invalid when the penetration depth/width ratio (D/B) for driven piles exceeds some value between 10 and 20 due to arching effect in the soil (fig.3.3 and fig. 3.4). It is not known whether this same limitations are also applicable to bored piles, but in any case the method of installing the pile has an important effect on the base resistance. The values of N_q largely depend on the relative density of soil adjecent to pile subsequent to pile installation. Method of installation has profound influence on the relative density of soil. These facts have been taken into account in the fig. 3.5 by Berezantsev's Nq vs. ϕ.

The Standards Association of Australia in their 1976 code on piling have given separate values of N_q for Driven and Bored piles (Table-3.1)

* Cited by BRAND[5]

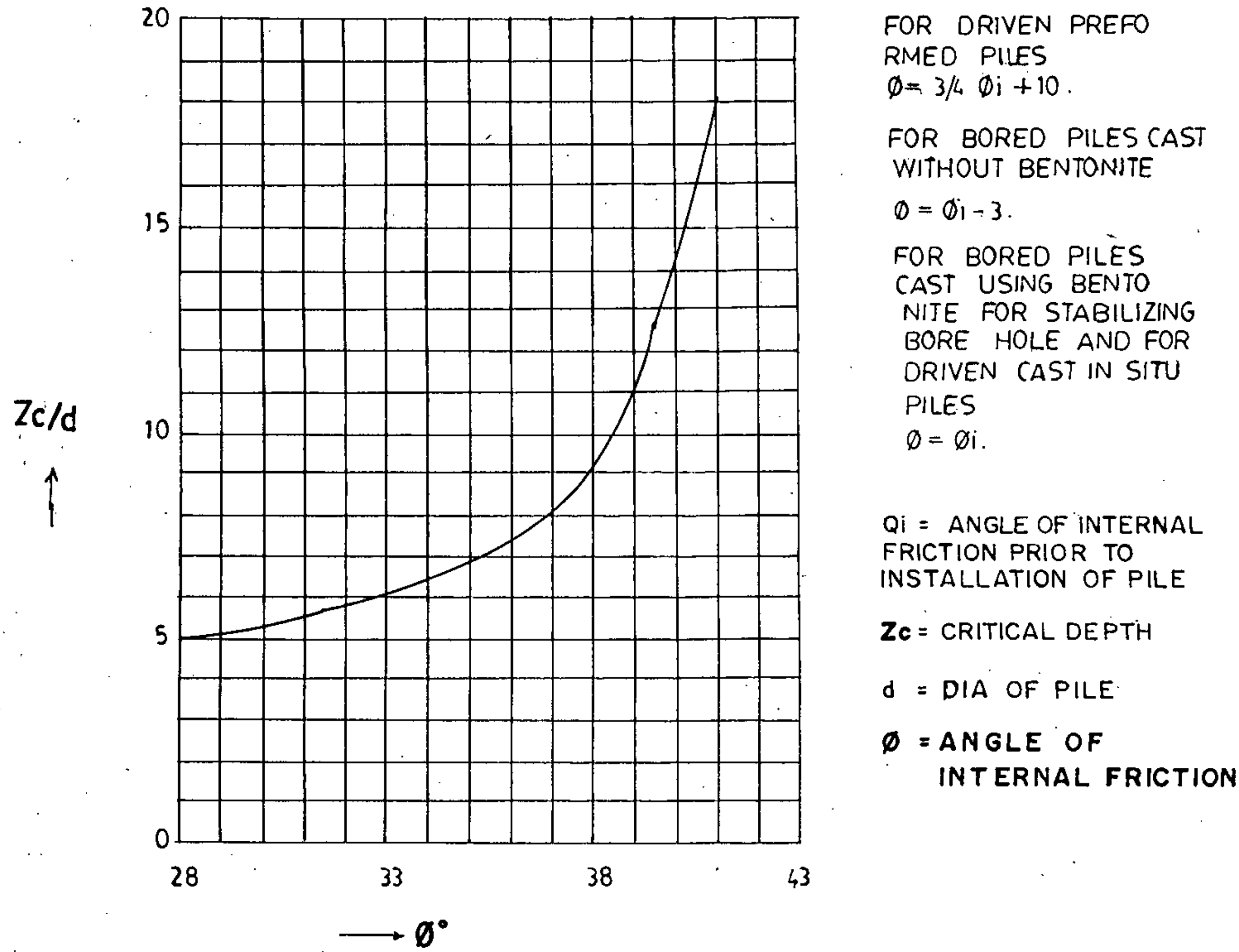


FIG:3.3 VALUES OF Zc/d FOR PILES IN SAND.
(AFTER POLOUS AND DAVIS[2])

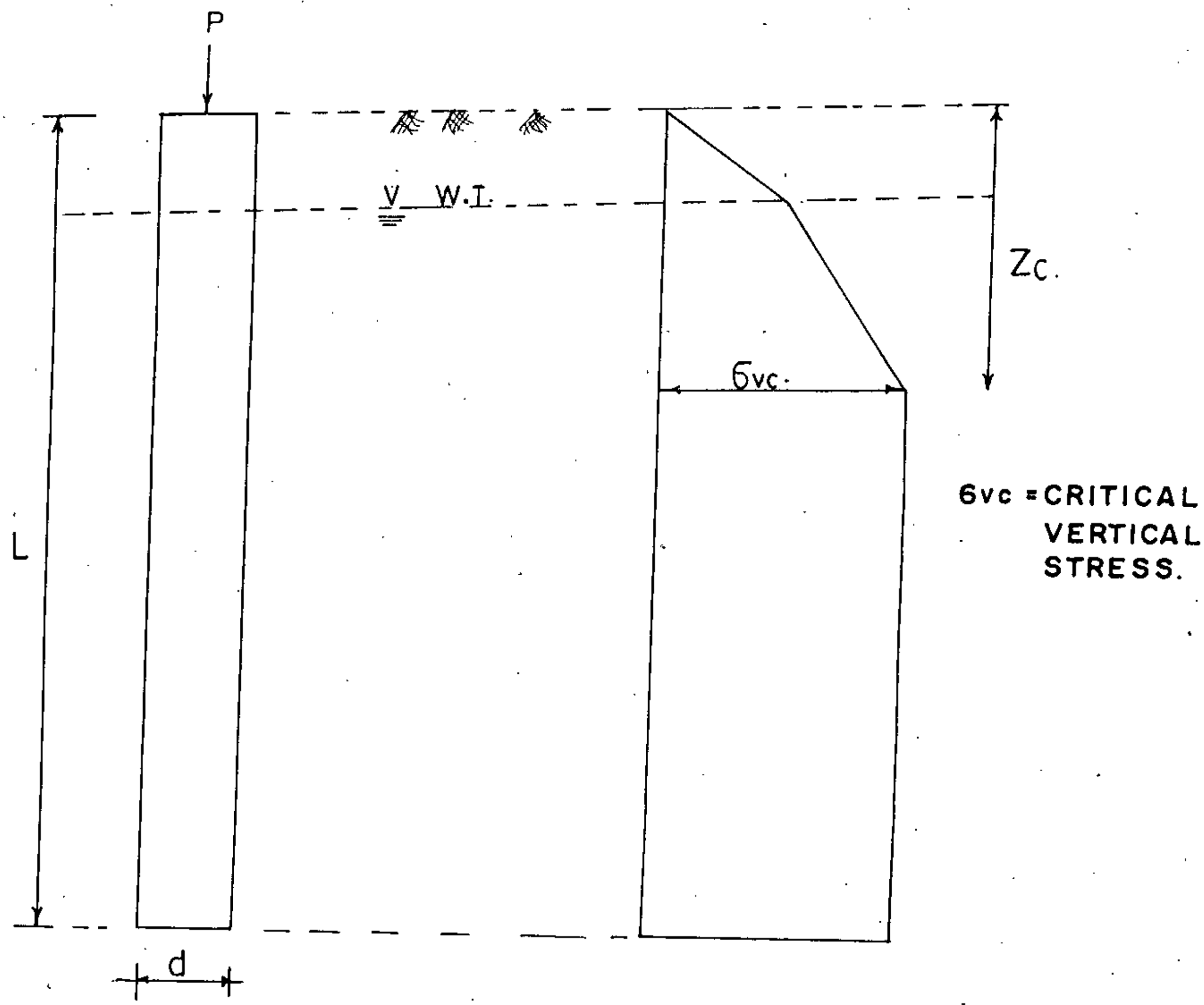


FIG :3 .4. SIMPLIFIED DISTRIBUTION OF VERTICAL STRESS ADJECENT TO PILE IN SAND. (AFTER POLOUS AND DAVIS (2))

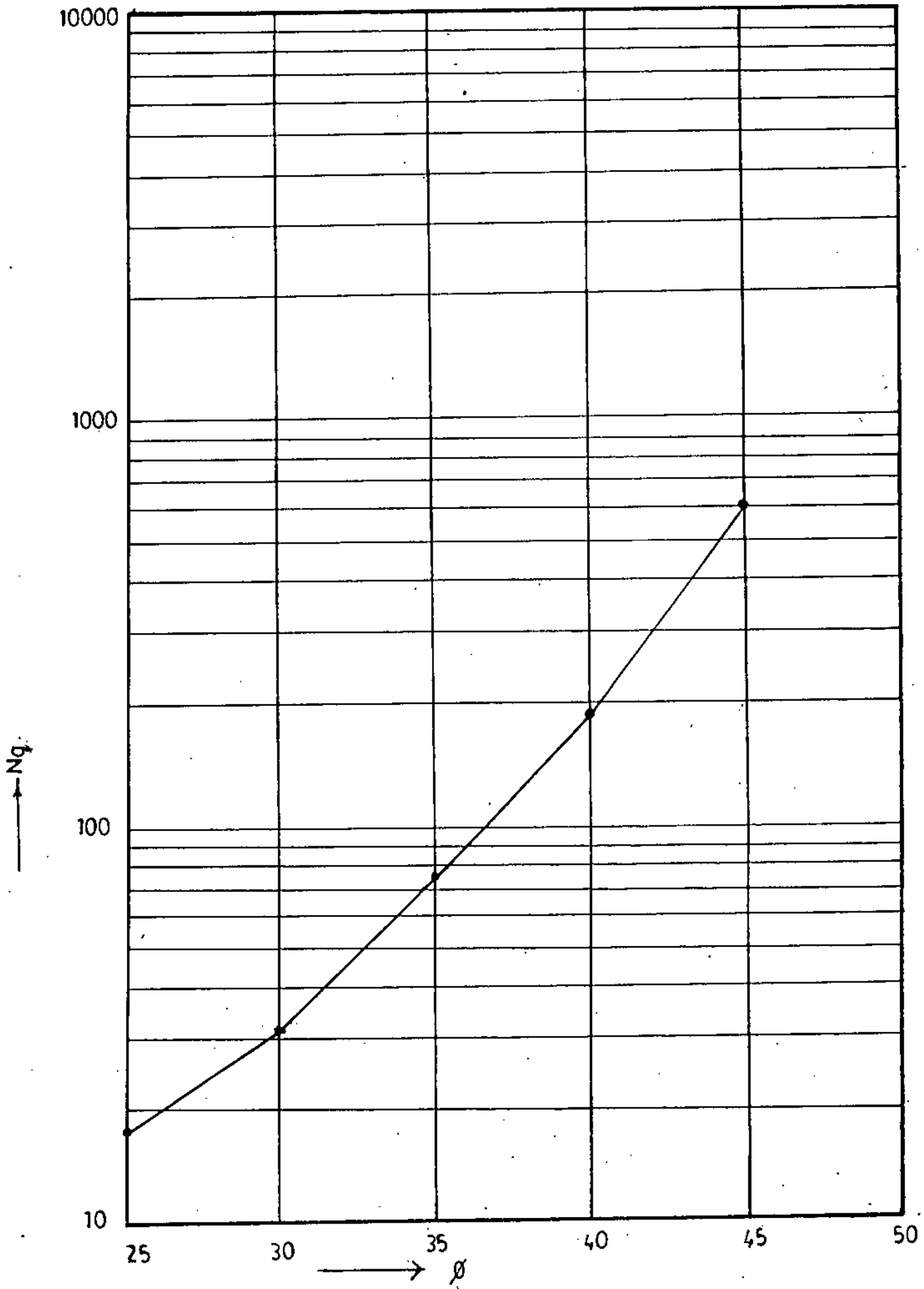


FIG. 3.5. RELATIONSHIP BETWEEN Nq AND Ø
(AFTER TOMLINSON [6])

Table-3.1 : Values of Nq with relative density of sand.*

Soil description	Nq Values	
	Driven pile	Bored pile
Loose		
Dr = 0.2 to 0.4	60	25
Medium		
Dr = 0.4 to 0.75	100	60
Dense		
Dr = 0.75 to 0.9	180	180

This practice of assigning different values of Nq for driven and bored piles in loose to medium dense looks more realistic and desirable. From the relation $Q_p = Ap\gamma DN_q$ it is seen that there is a rapid increase in Q_p for high value of ϕ (hence N_q), giving high values of end resistance.

However, published pile test results* indicate that the maximum value of end bearing is 100 Tsf. Earlier it was thought that, this limiting behavio-

* Cited by Tomlinson [6]

ur was autributed to some form of arching effect. But a more rational explanation lies in the variation of friction angle, ø with confining pressure. Bolton* discusses the strength and dilatancy characteristics of sand and shows that the bearing capacity of a deep strip footing does indeed appear to asymptote towards a limiting value (of around 93 Tsf) when the variation of ø with confining pressure is allowed for. The same approach as used by Bolton* may be used, to estimate the bearing capacity of deep circular footings.

VESIC[8] confirmed, with field tests, the tendency for unit resistance to increase with depth to some limiting value. He noted that even though the rate of increase sharply decreases at some 'critical' depth, there was an additional increase with further penetration. This critical depth was estimated as being between 10 pile diameters for loose sands and 20 for denser sands. In 1977, Meyerhof and Valsankgar[10] obtained additional laboratory evidence of a limiting value for unit point resistance. The study also showed that the'critical' depth for submerged sands is 1.6 times greater than that for dry sand. This

* Cited by VESIC[8]

increased critical depth is probably caused by buoyancy effects.

Since for dense sand, small variations in ϕ make large difference in the value of Nq, accuracy in assessing the end bearing capacity in sand depends mainly on the accurate determination of the value of ϕ. At present, perhaps the most satisfactory way to estimate ϕ is from standard penetration tests.

(d) Shaft Resistance for Cohesionless Soils:

The frictional resistance between the soil and a pile shaft is usually expressed in terms of the effective vertical pressure $\sigma_v = \gamma D$ for a homogeneous deposit, as :

$$Q_f = A_s k \sigma_v \tan\delta \qquad (3.10)$$

where k = coefficient of earth pressure σ_h/σ_v

δ = angle of friction between pile and soil.

The value of k depend on angle of friction (ϕ) for a soil and is different for active,passive and at rest states of strees. For at rest condition MEYERHOF* suggested that it varied between 0.5 for loose sands to 1.0 for dense sands. These values would seem to infer that the earth pressure on the pile shaft is sometime as low as the active value and never goes much above the 'at rest' value. This suggestion, however, is difficult to reconcile with the volume changes which must take place in the sand at the face of the pile when failure occurs. Before the pile is driven or a hole is bored, the sand exists in the 'at rest' condition where the lateral pressure is given by :

$$\sigma_h = k_o\sigma_v = (1-\mathrm{Sin}\phi)\sigma_v \qquad (3.11)$$

The boring of a hole would probably cause this to drop to the active condition, where :

$$k=k_a= (1-\mathrm{Sin}\phi)/(1+\mathrm{Sin}\phi) \qquad (3.12)$$

But lateral displacement that take place during the driving of a pile would cause lateral pressures which approaches the passive condition, where :

$$k = k_p =(1 + \mathrm{Sin}\phi)/(1 - \mathrm{Sin}\phi) \qquad (3.13)$$

* Cited by BRAND[5]

Even if k drops from k_o for the undisturbed sand to k_a after a hole is bored or the tube of a driven cast-in-situ pile is withdrawn, the shear displacements which occur at the pile surface during pile loading would cause volume change in the sand to bring about changes in the value of k. While it can be seen that loose sands might compact during shear so that the active earth pressure might be reached, dense sand would almost certainly dilate and the pressure might well be close to the passive value when the shear displacements are large enough for the pile to have failed. The most useful guide to the value of the angle of wall friction for a given pile material can be found in POTYONDI and BROOM*. They determined the ratio of δ/ϕ for the common materials by conducting laboratory direct shear tests and the results are shown in Table- 3.2 and Table- 3.3.

It is worth noting that the full scale and model pile tests carried out in sand by VESIC* indicated that the unit skin friction does not increase linearly with depth as predicted by the equation $Q_s = A_s k \sigma_v \tan\delta$. VESIC* showed that at some

* Cited by Tomlinson (6)

penetration depth between 10 and 20 pile diameters, a peak value of unit skin friction is reached. Thus the above equation gives increasingly unsafe values as the penetration depth exceeds about 20 diameters. Research has not yet established whether the peak value is a constant in all conditions, or is related to factors such as soil grain size or angularity. At present a peak value of 1(one)Tsf is used for straight sided piles. In many cases the skin friction of a pile in cohesionless soil is only a small proportion of its total resistance to compression loading and where piles are driven deeper then 20 diameters it may be satisfactory to use values given in Table- 3.4 as average skin friction over the whole shaft.

Table- 3.2 : Values of δ/ϕ for various materials in cosntact with dense sand[6]

Material	Surface finish	Value of δ/ϕ	
		Dry Sand	Saturated Sand
Steel	Smooth (Polished)	.54	.64
	Rough (rusted)	.76	.80
Wood	Parallel to grain	.76	.85
	Right angle to grains	.88	.89
Concrete	Smooth (from metal formwork)	.76	.80
	Grained(from timber formwork)	.88	.88
	Rough (Cast on Ground)	.96	.90

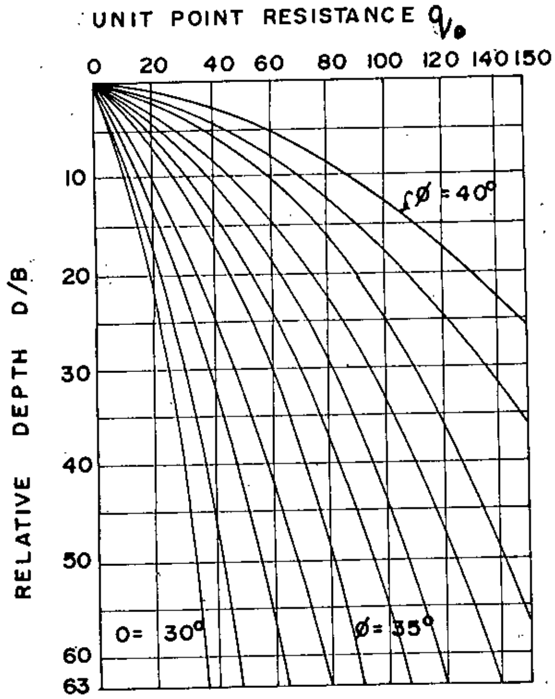
Table- 3.3: Values of δ, for various pile material[11]

Pile Materials	Values of δ
Steel	20
Wood	0.67ϕ
Concrete	0.75ϕ

Table -3.4: Average skin friction for straight sided piles in cohesionless soils[4]

Relative Density	Average Unit skin friction (tsf)
Less than 0.35 (loose)	0.10
0.35-0.65 (medium-dense)	0.10-0.23
0.65-0.85 (dense)	0.23-0.65
More than 0.85 (Very dense)	0.65-not more than 1.00

According to a study by Coyle and Castello[12], the co-efficient of earth pressure, K and the bearing capacity factor Nq are dependent upon overburden pressure or depth. As a result, there is not a strong justification for the individual determination of N_q or K. In fig. 3.6 the plot of unit point resistance, q_o, versus relative depth D/B is presented. Relative depth is determined



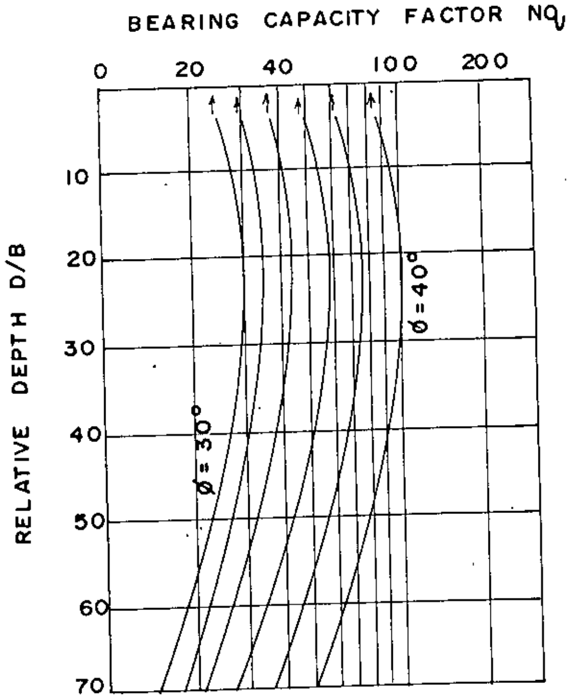
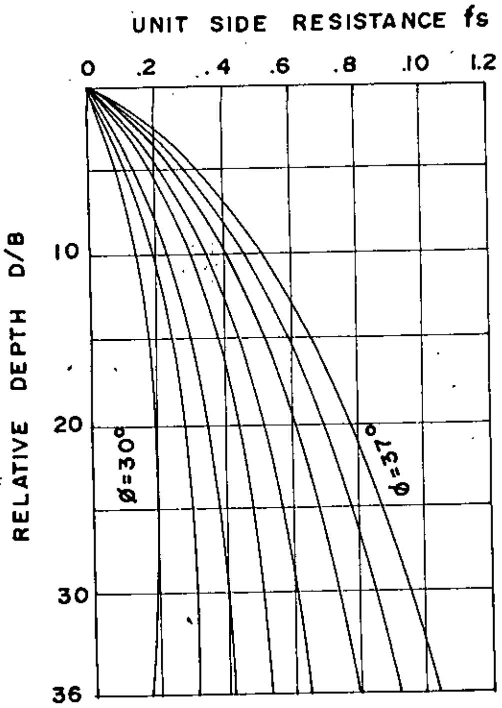


FIG.3.6. RELATIONSHIP BETWEEN UNIT POINT RESISTANCE AND RELATIVE DEPTH

FIG. 3.7. RELATIONSHIP BETWEEN N_q AND RELATIVE DEPTH



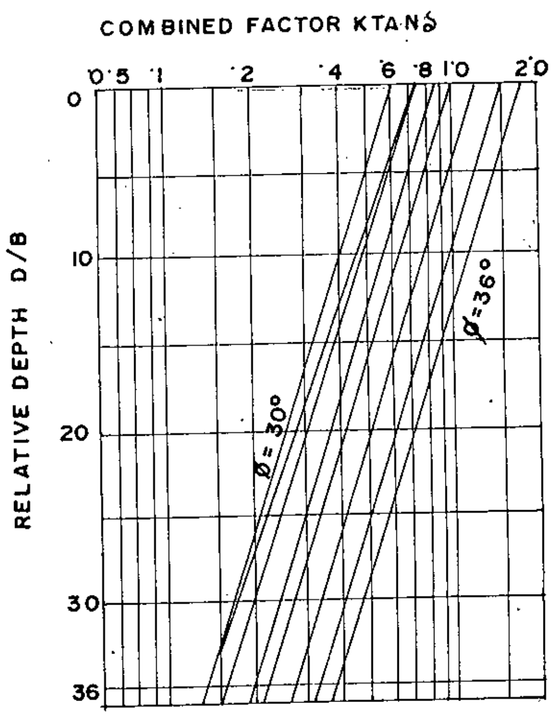


FIG.3.8. RELATIONSHIP BETWEEN UNIT SIDE RESISTANCE AND RELATIVE DEPTH

FIG. 3.9. RELATIONSHIP BETWEEN K TAN δ AND RELATIVE DEPTH

(AFTER MEYERHOF AND MURDOCK[3])

using the depth at the pile end point. At penetration of 60 pile diameter, q_o has not reached a constant value. A reduced rate of increase is indicated below 15 pile diameters of penetration for loose sands, and below approximately 30 pile diameters for denser sands. Below this critical depths the relationship between q_o and relative depth is nearly linear.

Fig. 3.7 shows a plot of the bearing capacity factor N_q (Log scale) versus relative depth. In general the N_q, values increase with increasing friction angle. It can be seen that the N_q values increase from zero penetration to a maximum value at roughly 20 pile diameter. At deeper penetrations the N_q values seem to decrease linearly with depth.

Fig. 3.8 shows the plot of unit side resistance fs, versus relative depth. The effect of increasing side resistance with increasing friction angle is indicated. Below approximately 10 pile diameters of penetration for loose sands, a practically linear relationship between fs, and relative depth is indicated. A similar relationship is suggested for denser sands, at deeper penetrations.Fig. 3.9 shows the plot of the combined factor K $\tan\delta$ (log scale) versus the relative depth. The K values correspond to a partially developed passive earth pressure coefficient at shallower depths, and decrease to the order of magnitude of active coefficients as the relative depth increases. Therefore the parameters which provide the best design correlations for piles in sand are the relative depth (depth to diameter ratio) and the sand friction angle[3].

According to FJELLERUP[13] the creep deformation may be the cause and the effect is the mobilization of extra friction. This can only take place at the expense of point resistance. He concluded that for large diameter bored piles - (a) there are considerable increase in ultimate shaft friction with pile age and (b) an increase in ultimate shaft friction occurs due to ground water lowering. However the ultimate load was not found at the largest ground water level lowering.

3.2.2 Ultimate Load from Dutch Cone Test:

The cone penetration test has been used in Europe for many years for the determination of the engineering characteristics of subsoils used for the design of piles. A detailed description of the instrument setup and of the test methods can be found in BEGEMANN*.The setup consists essentially of a 60° cone of 10 cm length and an accompaning 'friction jacket' 150 cm^2 in area; they measure independently the end resistance and local friction respectively.

Apart from its usefulness for estimation of soil strength and compressibility, the Dutch cone test is invaluable for pile design. The reasoning may be that the mode of failure of the soil as the cone is advanced is virtually identical to that of an actual pile. The cone resistance can be used as a guide to evaluate the point resistance of a pile and the local friction reading can be used to estimate the shaft resistance. Adjustments are necessary, however, to the friction reading measured for clays, since a certain amount of disturbance would have been caused by the cone penetration prior to the friction measurement and, also because of the adhesion factor between the clay and the material of the friction jacket. This factor might be different from that of the clay and the material of the pile. Pore pressure development

* Cited by BRAND[5]

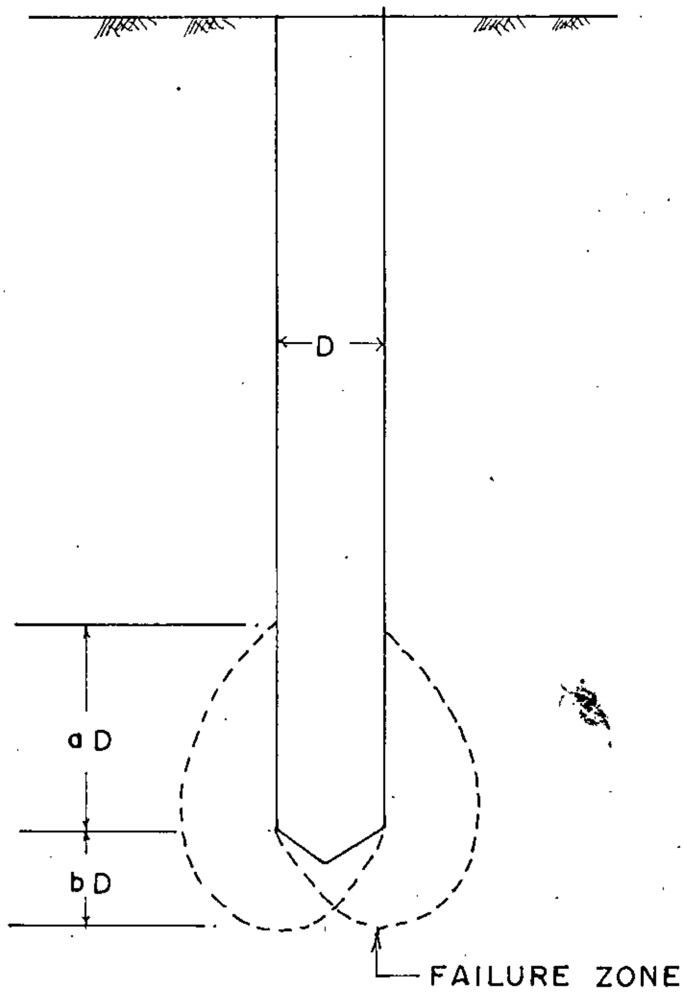


FIG. 3.10. END BEARING FROM DUTCH CONE RESISTANCE. (AFTER BRAND[5])

during testing further complicates the interpretation for clays, and, generally speaking, the existing experience is still insufficient for reliable predictions of ultimate pile load in clays.

In the application of Dutch cone to pile design it is assumed that the dimensions of the shear surface in the vicinity of the pile toe, and which determine the maximum end resistance, are proportional to the equivalent diameter, D, of the pile. The shear zone is taken as extending to a vertical distance aD above the pile toe and bD below it, and the unit end resistance of the pile is then taken as the average of the cone resistance over the depth (a + b)D as shown in Fig.3.10. The difficulty in the application of this technique is the selection of the appropirate values of parameters a and b. VAN DER VEEN* suggested that a and b have values from 1.5 to 12 and from 1 to 2 respectively He suggested that the most probable values for 'a' and 'b' in the Amsterdam area were 3.75 and 1.0 respectively.

A recent investigation on different aspects of

* Cited by BRAND[5]

Dutch cone test by PHAM* has adequately demonstrated the extreme usefulness of this instrument in the conditions which prevail in the Bangkok area. Undrained strengths and soil compressibilities can be estimated with sufficient accuracy for design purposes and it is possible to establish the soil profile over a large area with a minimum of borehole data. An analysis of the limited amount of pile load test data led PHAM to propose the ultimate load of a driven pile in the Bangkok area in the form:

$$Qu = 1.4\ Qf_1 + 0.7\ Qf_2 + \lambda Qc \qquad (3.14)$$

where Q_c = pile end resistance from average cone resistance.

Qf_1, Qf_2 = total frictional forces on pile in soft clay and stiff clay respectively as obtained from friction jacket.

λ = a constant (=1/3 for stiff clay and 1.0 for sand)

* Cited by BRAND[6]

3.2.3 Ultimate Load from Pile Tests

Generally two types of pile load test are performed in the field. One is for the determination of the pile capacity by giving load upto failure on the test pile and the other one is for checking the design load by giving 1.5 to 2.0 times design load on a service pile.

(a) Tests to Failure:

The loading to failure of full-scale test pile is certainly the most satisfactory basis for the estimation of ultimate load carrying capacity of population installed in similar subsoil conditions. Even if pile tests to failure are not carried out on all sites where piles are installed, sufficient test data is necessary for a given type of pile and given subsoil conditions to enable sensible predictions to be made for future piles on the basis of the static and dynamic formulae and the Dutch cone test.

It cannot be overemphasized that the only way to know with certainty the performance of a particular type of pile in specific soil conditions is to carry out a series of load tests to failure. It is a common practice that load tests are all too often restricted to a maximum applied load of twice the design load. Only when failure is reached can sufficient information be gained to enable safe and economical design to be achieved with that type of pile in future.

(b) Tests Not to Failure:

For pile load tests not carried to failure, methods of extrapolation have been proposed for the determination of the ultimate load capacity. One method, suggested by DAVISSON* also referanced by Peck* . Davisson's limit value is defined as the load corresponding to the movement which exceeds the elastic compression of the pile by a value of 0.15 inch (4 mm) plus a factor equal to the diameter of the pile divided by 120 (Fig. 3.11). The Davisson's limit was developed in conjunction with the wave equation analysis of driven piles and has gained widespread use in phase with the increasing popularity of this method of analysis. Another method is suggested by CHIN*.

* Cited by FELLENIUS(17)

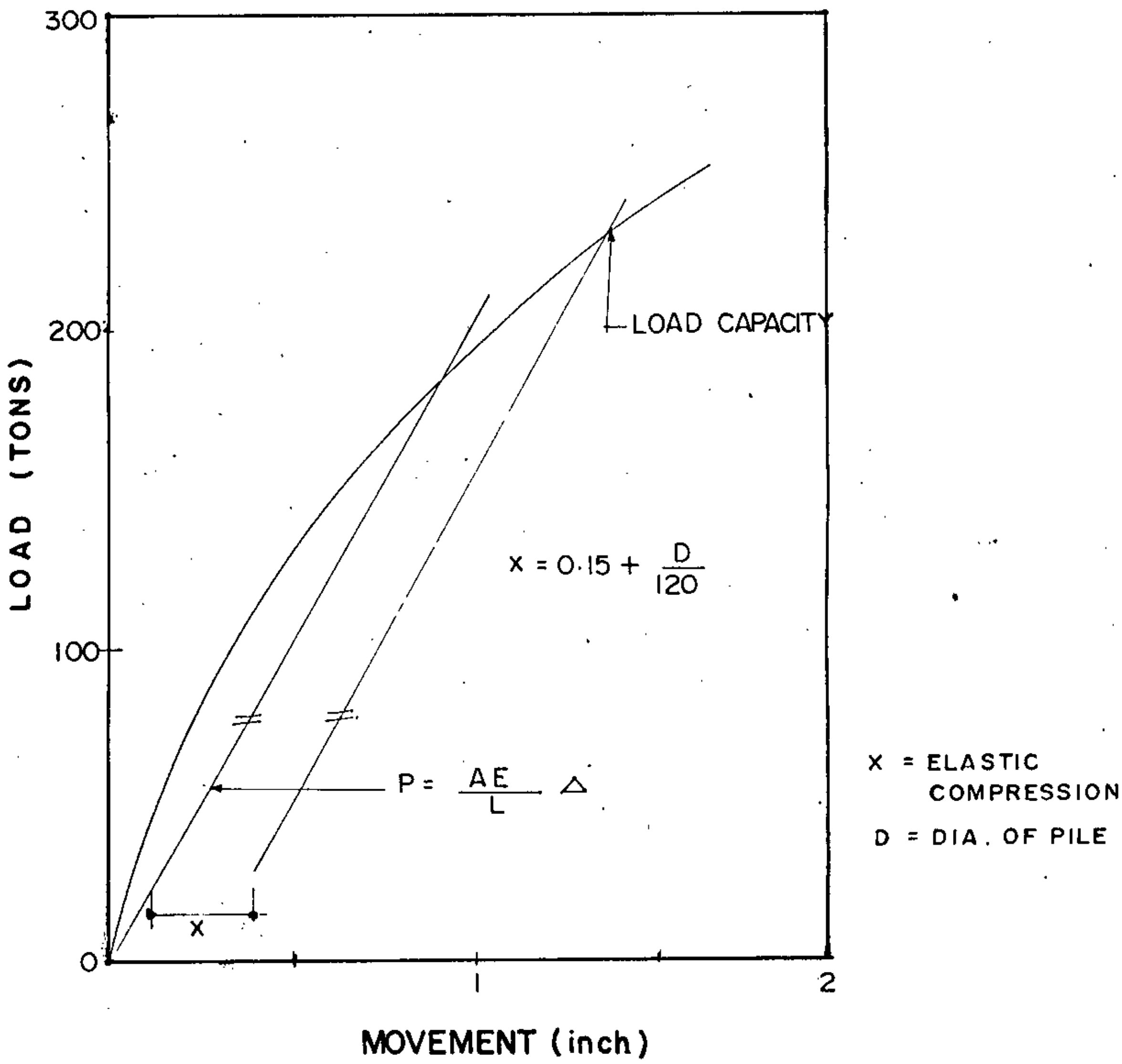


FIG : 3.11. DAVISSON'S METHOD OF ESTIMATING PILE CAPACITY (AFTER FELLENIUS[17])

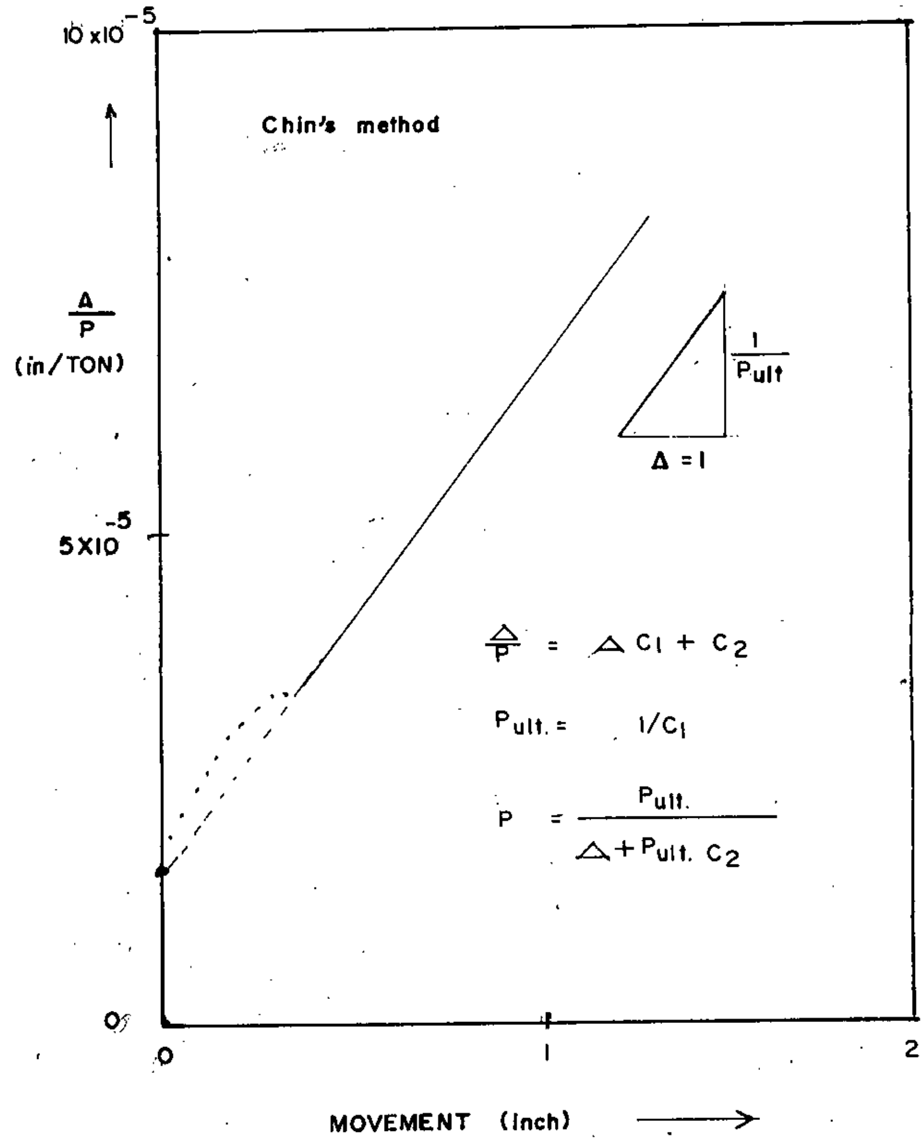


FIG.3.12. CHIN'S METHOD FOR ESTIMATING ULTIMATE CAPACITY OF A PILE. (AFTER FELLENIUS (17))

He observed that the stress-strain curves for direct shear and triaxial tests are approximately hyperbolic as are the load-settlement relationships for piles. By the Chin's method, each load value is divided with its corresponding movement value and the resulting value is plotted aganist the movement as shown in Fig. 3.12, after some initial variation, the plotted value fall on a straight line. The inverse slope of this line is the Chin's failure load.

Pile Load Tests

(1) Criteria of Failure

In order to measure, specify or discuss the ultimate load capacity of a pile it is necessary to establish what is to be understood by 'failure'. Where a maximum load is reached which either drops or is sustained as the pile settlement is increased, the definition of failure presents no problem as long as the settlement at which this state is reached is tolerable. For many piles, as with most spread footings, this ideal failure criterion cannot be applied (BRAND et al,[5] and it becomes necessary to define failure in terms of some rather arbitrary value of the pile settlement. It is impossible to establish one maximum permissible settlement for all piles under all circumstances, and the many existing criteria of failure based on allowable settlement have generally been established to take account of the worst combination of circumstances.

TERZAGHI* suggested that the criterion of failure for a single pile should be taken as a settlement of 0.1D. This will lead to extremely large settlements, for large diameter piles under their design loads. Such a criterion also has the disadvantage that it does not differentiate between elastic and plastic settlement, for it is the latter which truly determines the imminent onset of large vertical settlements for small load increase.

For piles which are essentially friction piles, it would perhaps seem logical to define failure as the settlement at which the maximum shaft resistance is mobilized. This settlement is generally small compared to that required to mobilize end resistance.

* Cited by BRAND[5]

Table-3.5 : Some codes and specifications for Pile load tests.

City or Organization	Settlement Criterion	Test Load / Design Load (%)	Max. Permissible Settlement in./ton	Other Settlement Limits	Time settlement Limits	Safety Factor	Load Test Req.d Over ton
New York City	Net	200	0.01	1 in. gross	0.001 in in 43 hr.	2	30
Cleveland, Ohio	Gross	200	0.01	1 in. max.	none in 24 hr.	2	60
Chicago	Net	200	0.01	-	none in 16 hr.	2	-
Washington, D.C.	Net	200	0.01	1 in. gross	none in 24 hr.	2	40
Boston, Mass	Net	200	0.5 in. (total)	no sign of failure	-	2	-
AASHO	Net	200	0.25 in. (total)	-	48 & 60 hr.	-	-
Dept. of Highways Ohio	Net	upto 300	0.25 in. (total)	rate of settlement <0.003 in. /ton	none in 6 hr.	variable-	

The allowable settlement of a pile under the design load is given by many codes. Examples of these criteria for a number of representative code is given in Table -3.5. It can be seen that a factor of safety of 2.0 on the working load is commonly specified for the definition of allowable settlement. Some codes base their criterion of acceptability on total settlement, some on plastic settlement, and some on a combination of the two.

A logical criterion of acceptability of settlement for a pile would take into account the type of pile (end bearing and/or friction) in question. Because of the large discrepancy in the settlements at which the maximum end and shaft resistances of piles are mobilized, BURLAND et al* proposed design criterion established on the basis of separate consideration of end and shaft resistances. This appraoch will almost certainly lead to safer and more economical foundations in most cricumstances as long as the mechanism by which the particular pile carries the applied load is well understood(5).

* Cited by BRAND(5)

(2) Types of Load Test

(a) Maintained Load Test: Of the two types of load test employed for testing piles, the maintained load (ML) test is by far the most common. The procedure adopted is to apply static loads in increments of the anticipated working load. Increments of 0, 25, 50, 75, 100, 0, 100, 125, 150, 175 and 200% of the working load are often employed. Each load is maintained until the settlement has ceased or has diminished to an acceptable rate or until a certain time period has elapsed. The working load and twice the working load are maintained on the pile for 24 hours or sometimes longer. If the load is increased to failure, this is done by reducing the increments where failure is imminent so that the ultimate load capacity can be accurately measured.

(b) Constant Rate of Penetration Test:
The constant rate of penetration (CRP) test was first proposed by WHITAKER* who suggested that a pile could be treated as a probe used for measuring soil strength. The test is

* Cited by BRAND[5]

carried out by continuously loading the pile so that it penetrates the soil at a constant rate while the load is measured continuously. The rate of penetration selected is usually that used in shearing soil samples in the unconfined compression test (0.0012 in/min), but the rate does not significantly affect the ultimate load (WHITAKER,*). As the ultimate load capacity is approached, very little increase in load is required to maintain a constant rate of penetration, and the ultimate bearing capacity is reached when the continuous vertical movements result in no increase in the penetration resistance.

Good agreement has been found to exist between the ultimate loads measured by the ML and CRP tests of the CRP test, however, has been voiced by ELLISON et al* on the grounds that it does not represent the type of loading to which a pile is subjected during its working life. They also reported that this test tended to over-estimate the ultimate load capacity of bored piles in London Clay.

* Cited by BRAND[5]

It is obvious that the settlement recorded for a given applied load in the CRP test will always be lower than the comparative settlement for the ML test, because no time is permitted for plastic settlement under sustained load; this is a disadvantage of the CRP test. Otherwise, the CRP test has the great advantage that it can be carried out very quickly. As long as sufficient experience is gathered with the CRP test in the prevailing subsoil conditions, and if initial correlation studies are made, the CRP test is generally to be preferred to the ML test.

(3) Estimation from Load-Settlement Curve: Where no strain measurements are taken in the pile from which to determine Qp and Qf an estimation of these separate resistance can be made on the basis of the load-settlement curve. VAN WHEELE* observed from many load tests on piles driven into a sand stratum that the elastic compression of the soil at the pile toe was directly proportional to the applied load (ML test). He showed that, since the total

* Cited by BRAND[5]

settlement of a pile is composed of plastic settlement plus the elastic compressions of the pile and soil, the elastic compressions could be taken as the elastic rebound of the pile after unloading.

In order to separate end bearing and shaft friction by the method of Van Wheele, it is necessary to load the pile to failure and to measure elastic rebound after each load increment. The elastic compression of the subgrade is equal to the elastic rebound minus the elastic shortening of the pile. The elastic shortening is determined either by direct measurement or from an assumed distribution of load along the pile length. BULLEN* assumed a linear distribution of skin friction with depth and found that the elastic shortening of a pile is equivalent to that obtained by compressing the pile with an axial load varying between 1/3 to 2/3 of the total shaft friction, depending on the distribution of the frictional force. He suggested, that elastic shortenings computed on the basis of an axial load equal to one half of the shaft load gives values which are acceptable estimates of actual elastic compressions.

* Cited by FELLENIUS[17]

JAIN & KUMAR* proposed a relaxation technique for use with Van Weele's method. The shaft friction is first calculated on the assumption of no elastic shortening. This computed shaft load is then used to determine the elastic shortening, which is then used to determine the shaft load, etc. Successive approximations of this kind eventually lead to compatible values of shaft and end load.

* Cited by BRAND[5]

CHAPTER-4

PILE CAPACITY BY THEORETICAL ANALYSIS

4.1. Introduction

The exact analysis of a pile theoretically is impossible because of higher degree of indeterminancy and of unpredictable behaviour. Pile may be analysed theoretically in many ways considering the empirical relations and suggestions offerred by numerous authors. This chapter deals mainly with the classification of piles, the method of analysis of pile load capacity, the effect of installation method on pile load capacity and the factors on which the load carrying capacity of a pile depend.

4.2. Description of the Piles

Depending on the diameter, piles may be classified as micro-piles, mini-piles, small diameter piles and large diameter piles. The term micro-pile is commonly used for piles having diameter less than approximately 250 mm, although sometimes, the term is restricted to those with diameter 150 mm or less. The term mini pile is used to cover the range of piles having 150 to 250 mm diameter. Piles are referred to as small-diameter when diameter is less than 600 mm and large-diameter when diameter is greater than this nominal size.

The piles considered in the present study are small diameter, bored, cast in place concrete piles. The piles are of diameter 450mm and 500mm with lengths from 15.25m to 21.34m. The method of boring was mostly rotary drilling method and bentonite slurry was used instead of complete casing. A short length (4m to 5m) steel casing was used at the top of the hole in drilling and concreting operation which was removed after completion of the concrete pouring operation. After completion of the drilling upto required depth the bore hole was washed by water circulation method before concreting. The concrete having 5 inch to 7 inch slump was poured with the help of a tremie pipe successively, by keeping the tip of the tube within the concrete so as to avoid the contamination.

4.3 Analysis of Piles

The exact calculation of the load carrying capacity of a pile is a complex matter which is based partly on theoretical concepts but mostly on empirical methods based on experience. The practice of calculating the ultimate load carrying capacity of a pile based on the principles of soil mechanics differs greatly from the application of these principles to shallow spread foundations. In the latter case the entire area of soil supporting the foundation is exposed and can be inspected and sampled to ensure that its bearing characteristics conform to those deduced from the results of exploratory boreholes and soil tests. For spread foundations virtually the whole mass of the soil influenced by the bearing pressure remains undisturbed and unaffected by the constructional operations. Thus the safety factor against general shear failure of the spread foundation and its settlement under the deisgn working load can be predicted from a knowledge of the physical characteristics of the undisturbed soil with a degree of certainty which depends only on the complexity of the soil stratification. The location of the failure surface for a deep foundation is

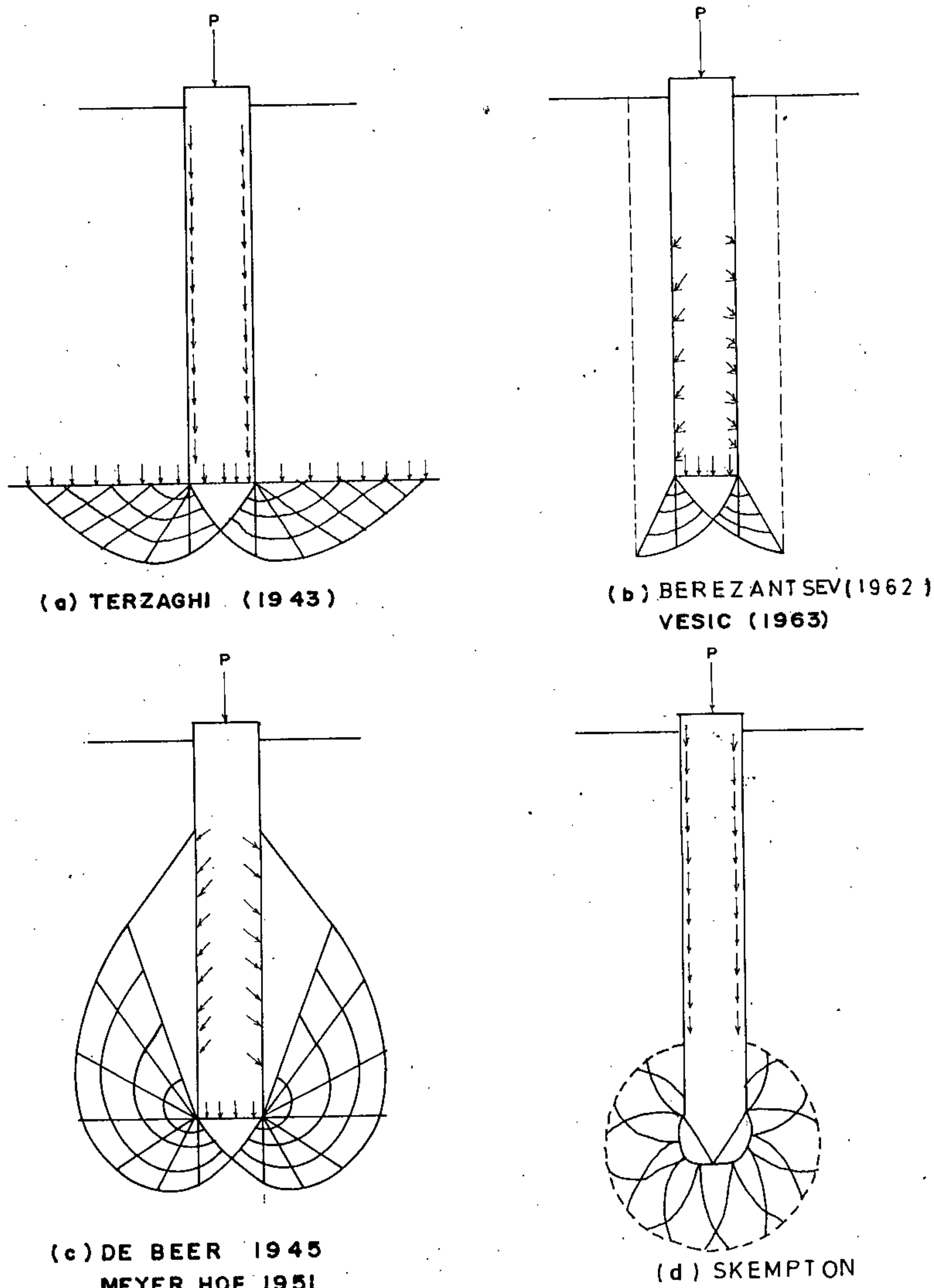


FIG. 4.1. ASSUMED FAILURE PATTERNS UNDER DEEP FOUNDATIONS (AFTER LAMBE [14])

less well known than for shallow foundations, and depending on the location and shape of the surface assumed, investigators have calculated various values for bearing capacity factors. Fig. 4.1 shows some of the pattern of failure that have been assumed in the theoretical analysis.

The conditions which govern the supporting capacity of the piled foundation are quite different. No matter whether the pile is installed by driving with a hammer, by jetting, by vibration, by jacking, screwing or drilling, the soil in contact with the pile face from which the pile derives its support by skin friction and its resistance to lateral loads, is completely disturbed by the method of installation. Similarly, the soil beneath the toe of a pile is compressed or loosened to some extent which may affect significantly its end bearing resistance. Changes takes place in the conditions at the pile soil interface over periods of days, months or years which materially affect the skin friction resistance of a pile. These changes may be due to the dissipation of excess pore water pressure, to the relative effects of friction and cohesion which in turn depend on the relative pile-to-soil movement and to chemical or electro-chemical effects caused by the hardening of the concrete or the corrosion of the steel in contact with the soil[6]. The sub-soil strata of the piles used in the present investigation are shown in Figs. 4.2 to 4.6. Some of the physical and engineering properties of the soils are also indicated in the figures.

4.3.1. Piles in Sand:

Piles in sand have been analysed on the basis of simplified formulae. The skin friction values are calculated using the following formulae:

$$Q_f = As\ K\sigma_v\ \tan\delta \qquad (4.1)$$

where Q_f = Shaft frictional force at pile failure

A_s = Area of Shaft

K = Co-efficient of earth pressure

σ_v = Effective vertical presure and

δ = Angle of wall friction between pile and soil

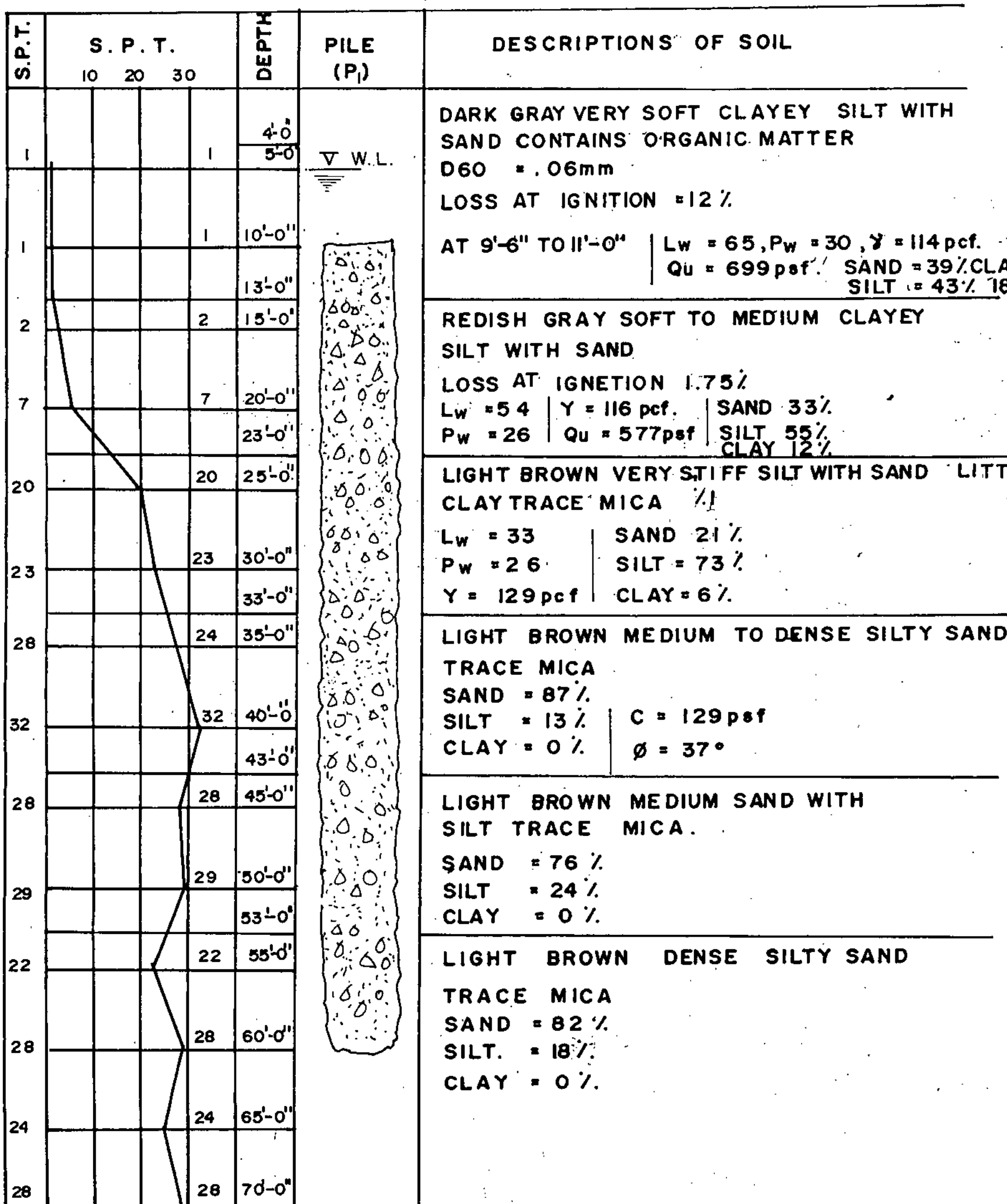


FIG. 4.2. SUB SOIL INVESTIGATION BORE LOG AT LOCATION OF PILE P.1

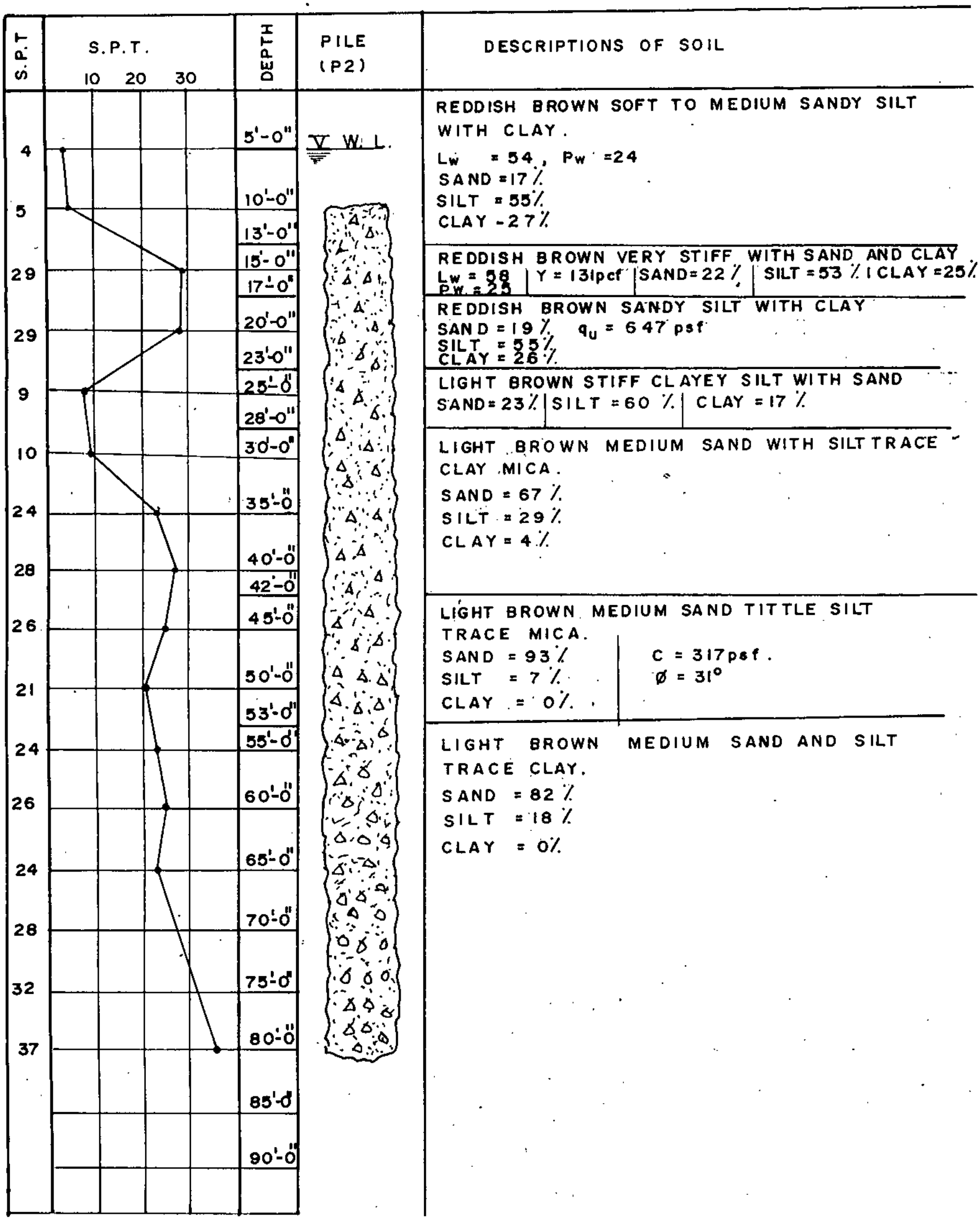


FIG.4.3. SUB SOIL INVESTIGATION BORE LOG AT LOCATION OF PILE P.2

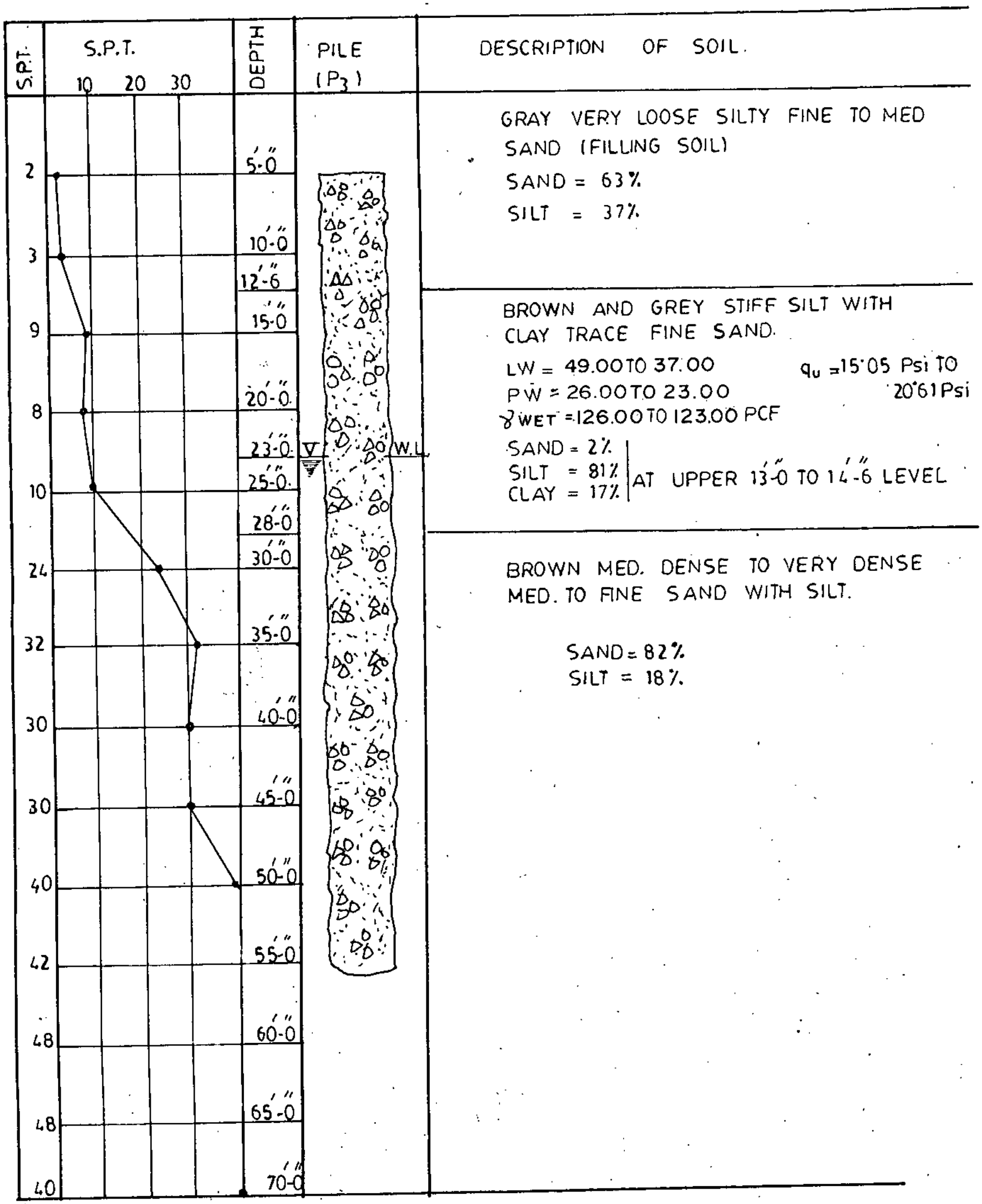


FIG : 44 SUBSOIL INVESTIGATION BORELOG AT LOCATION OF PILE P 3

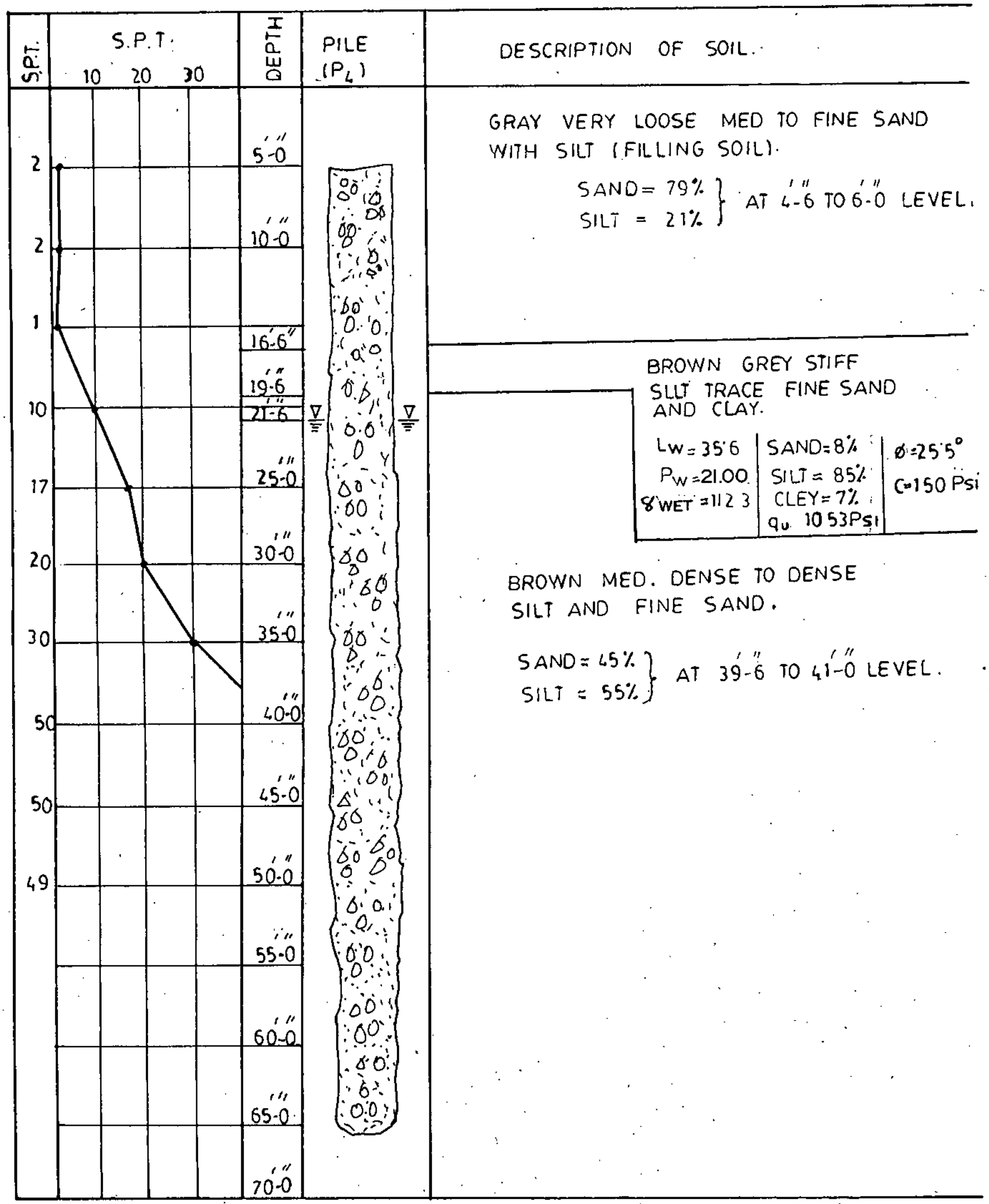


FIG :4-5. SUB SOIL INVESTIGATION BORELOG AT LOCATION OF PILE P. 4

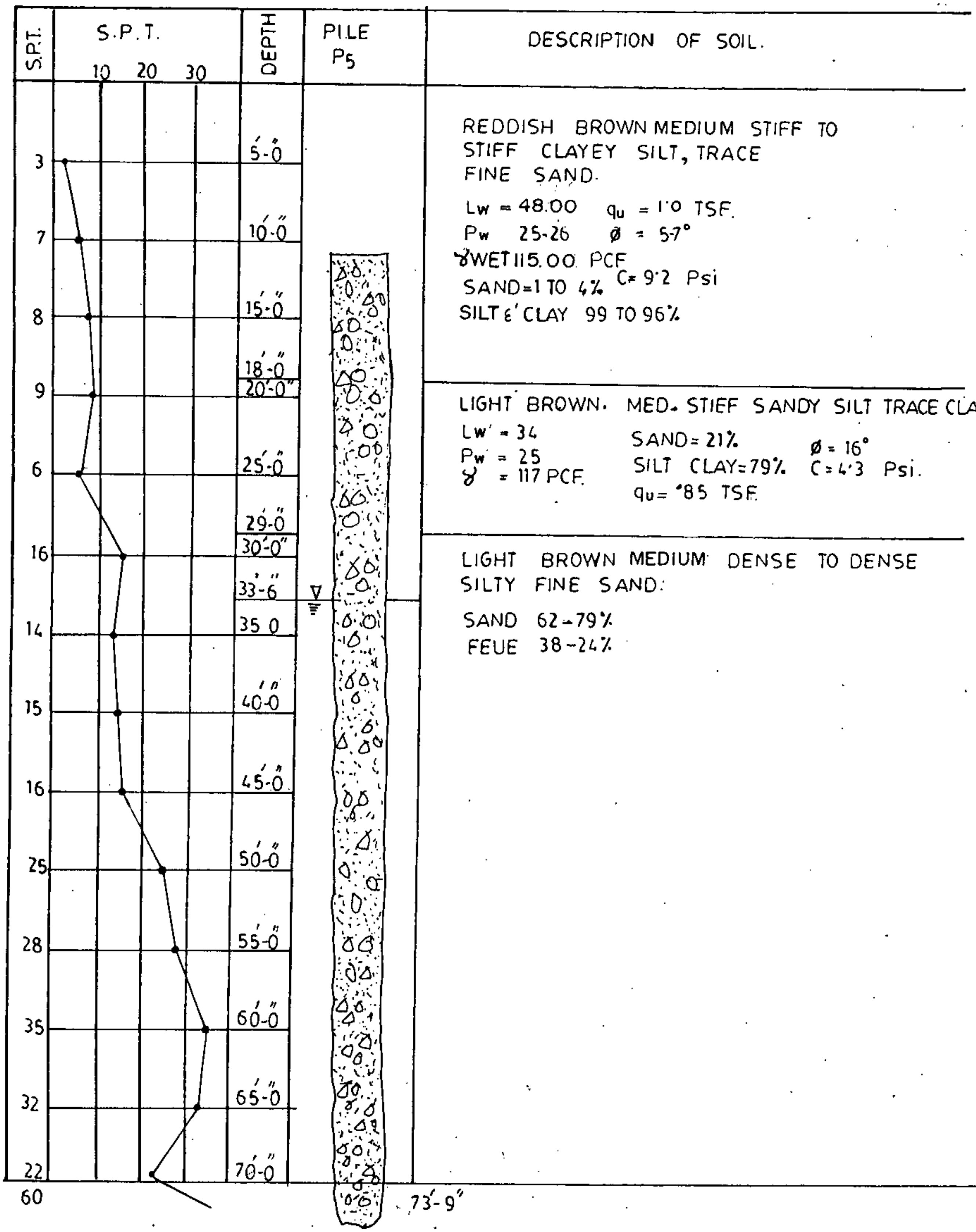


FIG:4.6. SUB SOIL INVESTIGATION BOREHOLE AT LOCATION OF PILE P.5

Taking the coefficient of earth pressure at rest equal to unity (K=1) and the value of δ = .75ϕ[6], the expression stands for

$$Q_f = A_s \sigma_v \tan(.75\,\phi) \qquad (4.2)$$

The vertical effective stress σ_v was considered beyond the critical depth.
The end bearing resistance for cohesionless soil

is determined from the formula as follows:

$$Q_e = A_p \sigma_v N_q \qquad (4.3)$$

wheer Q_e = Load carried by pile end only

A_p = Area of pile base

N_q = Bearing capacity factor

The values of N_q are taken from the relation proposed by Berezantsev* and σ_v is taken as constant beyond critical depth. The ϕ value are taken from Sub-soil investigation (Fig. 4.2 to Fig. 4.6). In case of unavailability of ϕ value, empirical relation suggested by Pack, Hanson and Thornburn[16] (Fig. 4.7), is used.

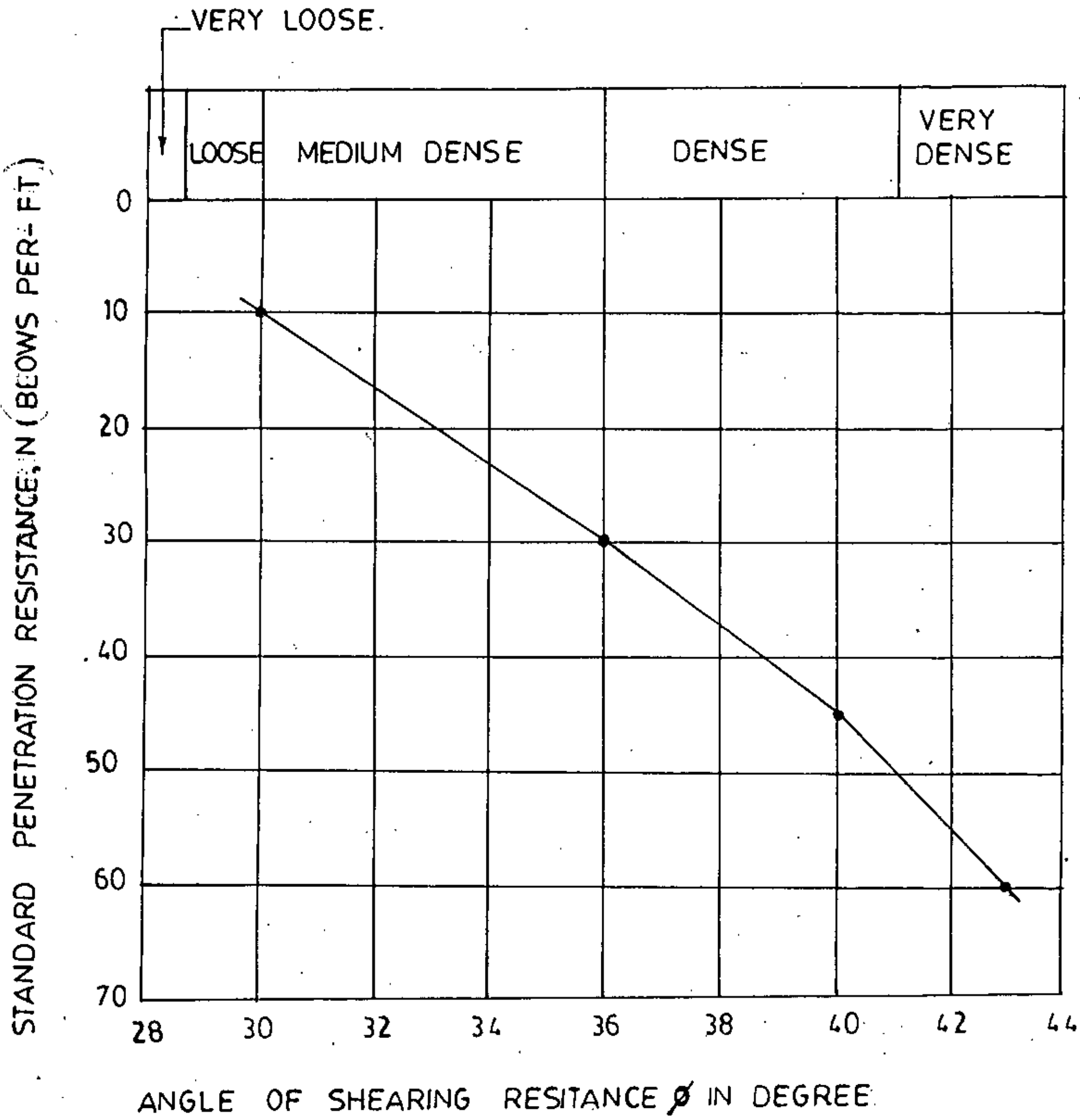


FIG:4.7. RELATIONSHIP BETWEEN ANGLE OF SHEARING RESISTANCE AND S.P.T. VALUE OF COHESION LESS SOIL (AFTER PECK, HANSON AND THORN BURN[6])

4.3.2. Piles in Clay:

The pile portion embeded in clay has been analysed using the simplified relations and empirical formulae. The shaft resistance for cohesive soil is calculated from the relation:

$$Q_f = A_s \alpha C \tag{4.4}$$

where α = Adhesion factor and

C = Cohesion of soil

The value of α is taken from Fig. 3.1 and the values of c were determined from unconfined compression test or triaxial test. Empirical relation of C = N/16 Tsf[4] was also used for estimation of cohesion in case of nonavailability of test data. The locations of water table were considered in the design are shown in the soil borelog (Fig. 4.2 to Fig. 4.6.)

In determining the shaft resistance of a pile segment the average values of ϕ or c were taken. For very soft or compressible layer the frictional resistance of shaft was ignored. In case of soil lying below water table, the submerged unit wt. of the soil was considered. In calculating the end resistance the ϕ-value considered is the average of the values between one diameter below the tip of the pile and 3-diameter above the tip of the pile[4] as is considered in case of a Dutch cone penetration test.

CHAPTER-5

PILE LOAD TEST PROCEDURES

5.1. Introduction

This chapter describes the different procedures for load test of a pile. Three types of load tests such as Maintained load (ML) test, constant rate of penetration (CRP) test and method of equilibrium test are mainly concerned. The relative advantages and disadvantages or the suitability of the three methods are also compared.

5.2 Test Procedure

Two principal types of pile load test are commonly carried out for compressive loading on piles; the maintained load test and the constant rate of penetration test. In constant rate of penetration test (CRP) the compressive force is progressively increased to cause the pile to penetrate the soil at a constant rate until failure occurs and in maintained load test (ML) the load was increased in stages to some multiple of working load, 1.5 times to twice and the time-settlement curve is ploted at each stage of loading and unloading. The ML test may also be taken to failure by progressively increasing the load in stages.

Another common procedure for compression test is the method of equilibrium. The principle is to apply to the pile, at each stages of the test, a load slightly higher than the required load and then to decrease the load to the desired value. In this case the rate of settlement diminishes much more rapidly than with the maintained load and equilibrium is reached in a matter of minutes rather than hours.

The CRP test procedure is best suited to determine the ultimate bearing capacity of a pile and is therefore applied only to preliminary test piles or research-type investigations[13]. The CRP test is not, howevere, suitable for checking the compliance with the specification or requirements for the maximum settlement at given stages of loading. The ML test is best suited for contract work, particularly for proof loading test on working piles. The load at each stage is held for a minimum period of one hour or beyond this period if the rate of settlement has not decreased to less than 0.1 mm in 20 minutes . If it is desired to obtain the utlimate load on a preliminary test pile, it is useful to adopt the ML method for upto twice the working load and then to continue loading to failure at a constant rate of penetration. A further modification of the ML test consists of returning the load to zero after each increment. This form of test is necessary if the net settlement curve is used as the basis of defining the failure load.

CRP and ML test use the similar type of loading arrangements and pile preparation. Suitable load arrangements for applying the load to the pile is using hydraulic jacks and the reaction beam. The loading arrangement is done either by the tension piles, kentledge or cable anchors. The minimum clearance between the pile and the reaction support systems is to be maintained to avoid the induced horizontal pressures from the supports having an appreciable effect on the skin friction and the base load of the test pile. It is sometimes uneconomical to space the supports so widely apart that all effects are eleminated. If closer spacing is necessary the contribution of these surcharge effects should be calculated and allowed for in the interpretation of the test results.

In the present investigation the method of load test employed is Maintained load (ML) test for all the five piles.The load-time-settlement diagrams are shown in Fig.5.1 to 5.5. The increment of load and their maintained time were reduced towards the end of the test (after 200% design loading) so as to identify the failure load. The loading system that has been used in our piling was kentledge reaction system. In this sytem a platform was constructed over the pile supporting the two sides of the test pile by steel I-section (joist) and woodend plank. Sandbags were then placed over this platform to have the desired load on the pile head. The load was then applied into the pile head with the help of a load column and the mechanism used in this system (Hydraulic pump and Hydraulic jack system). The settlement of the pile was measured with the help of Dial gauge extensometers and reference beam. The increment of loading were followed mostly as 20%, 40%, 60% 80%, 100%, 120%, 140%, 160%, 180%, 200%, 220%, 240%, 260%, 280%, 300% of the design load. After loading

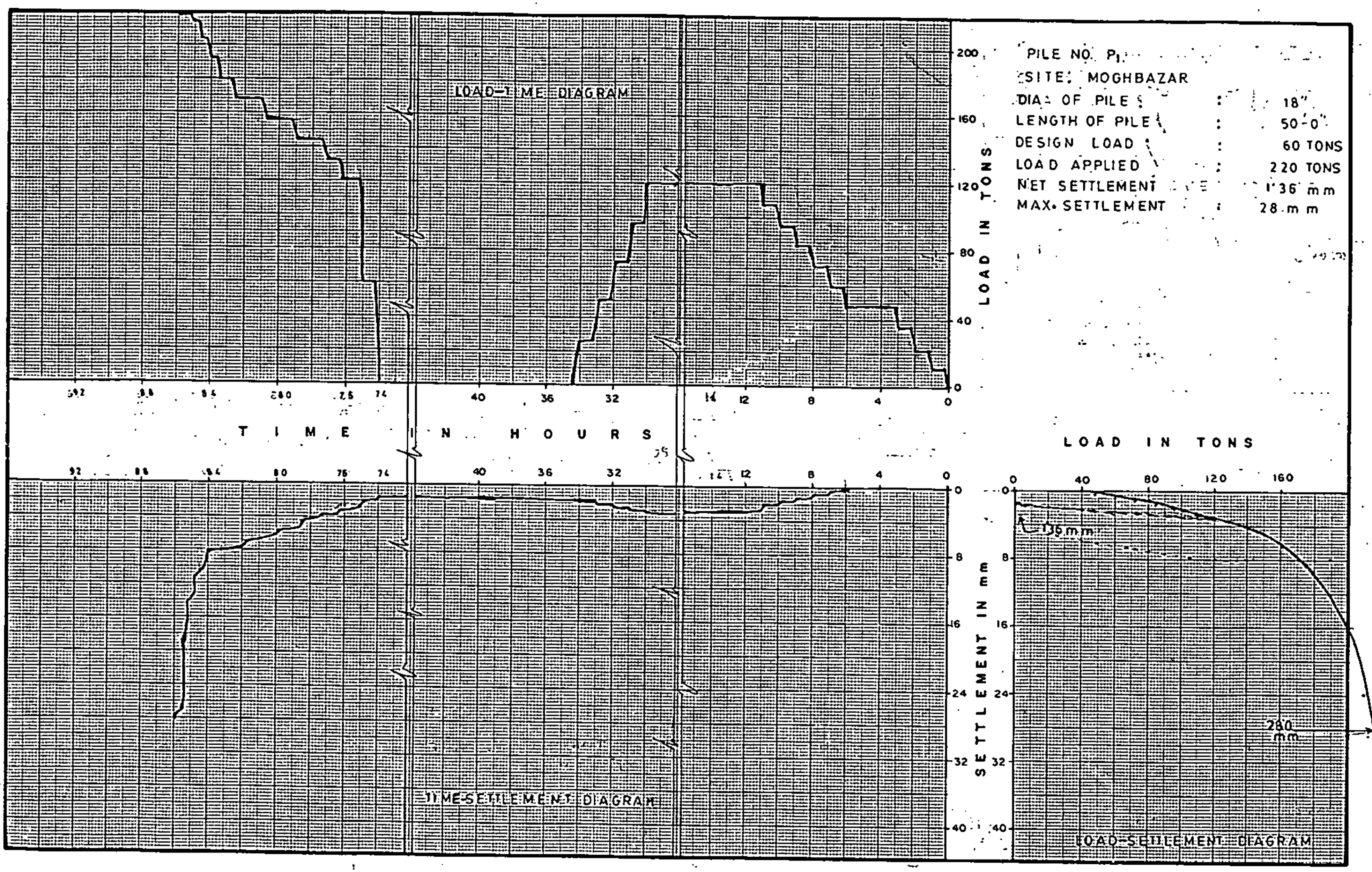


FIG. 5.1. LOAD-TIME-SETTLEMENT DIAGRAM

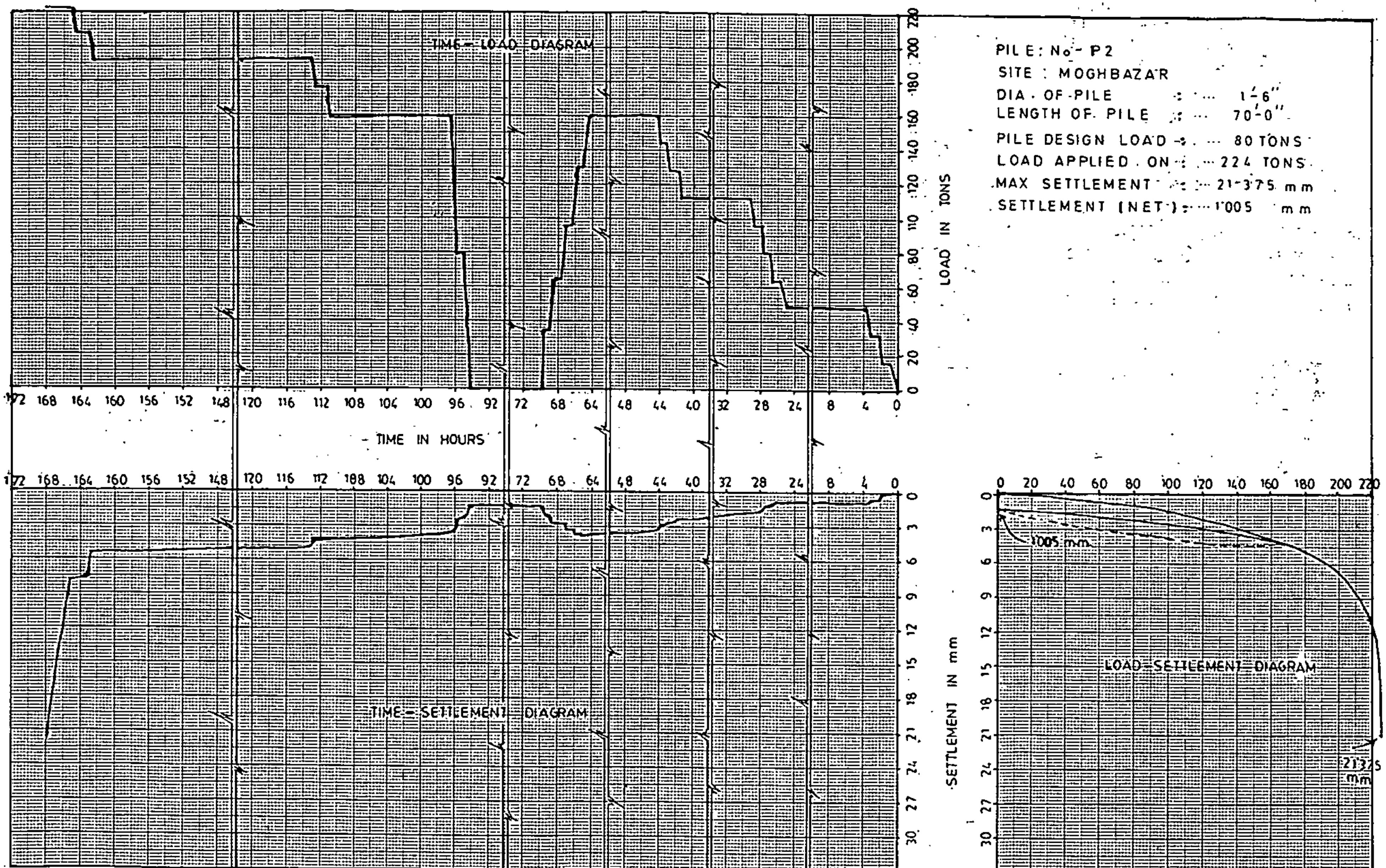


FIG. 5.2. LOAD- TIME- SETTLEMENT DIAGRAM

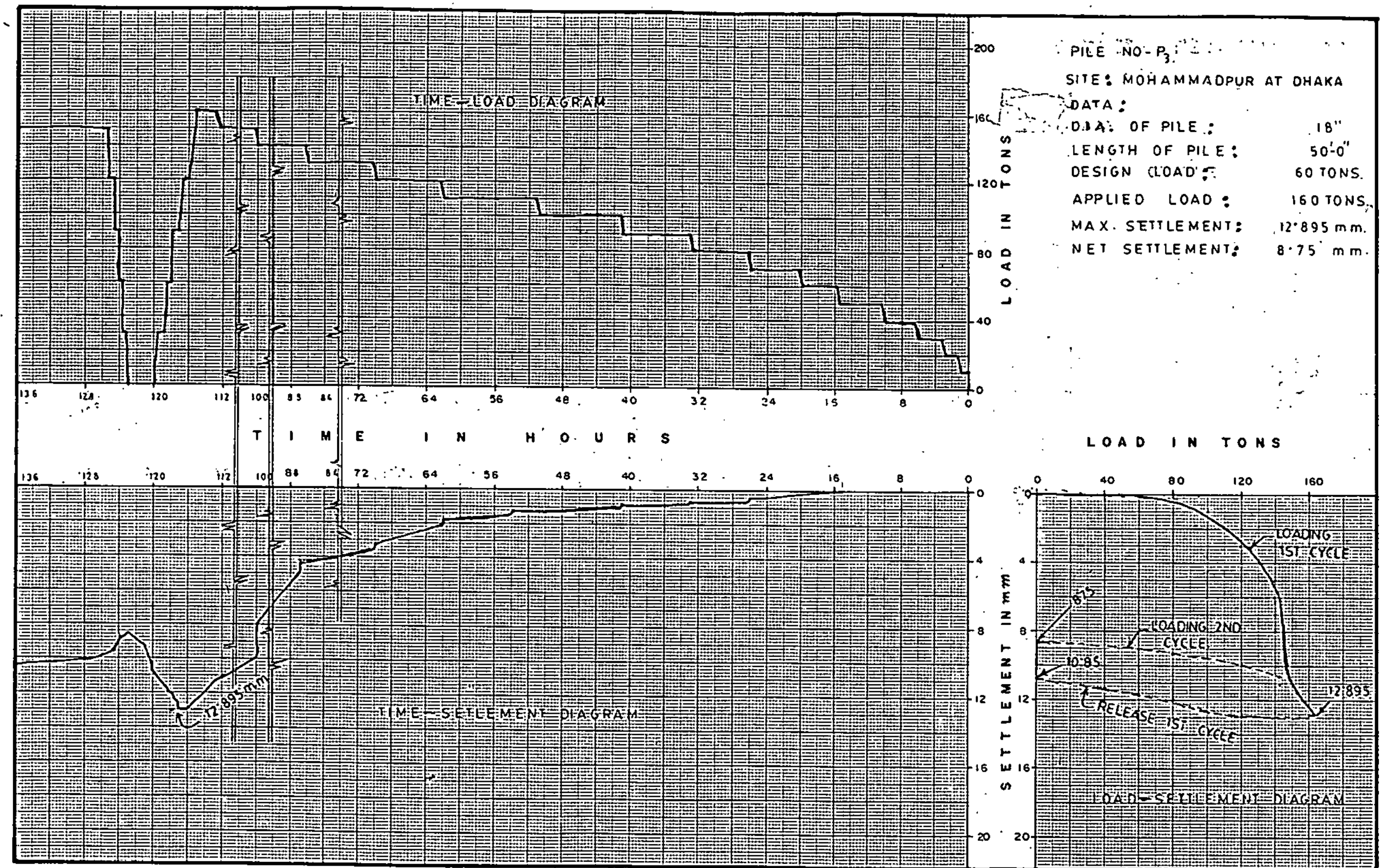


FIG. 5.3. LOAD-TIME - SETTLEMENT DIAGRAM

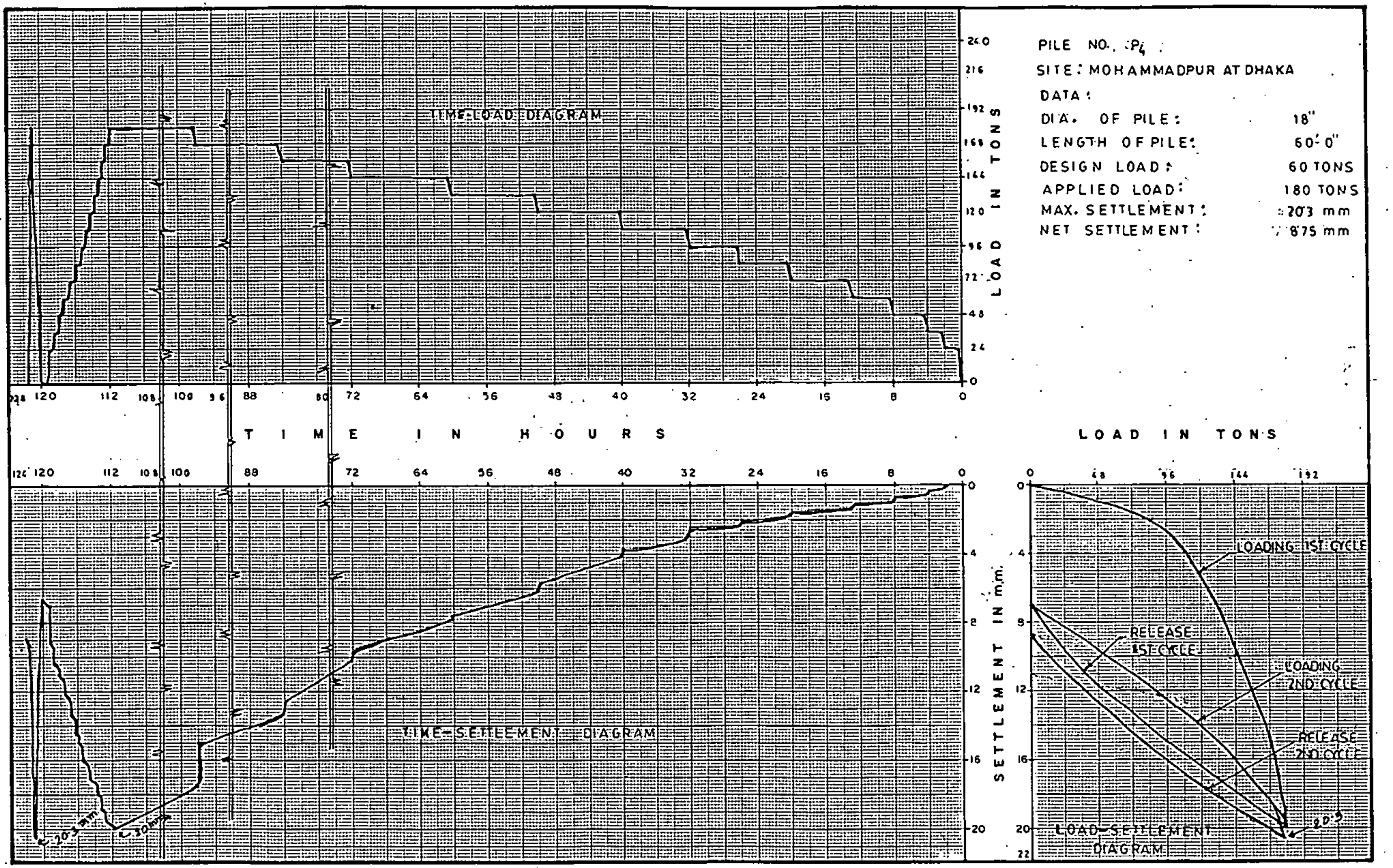


FIG.5.4. LOAD-TIME-SETTLEMENT DIAGRAM

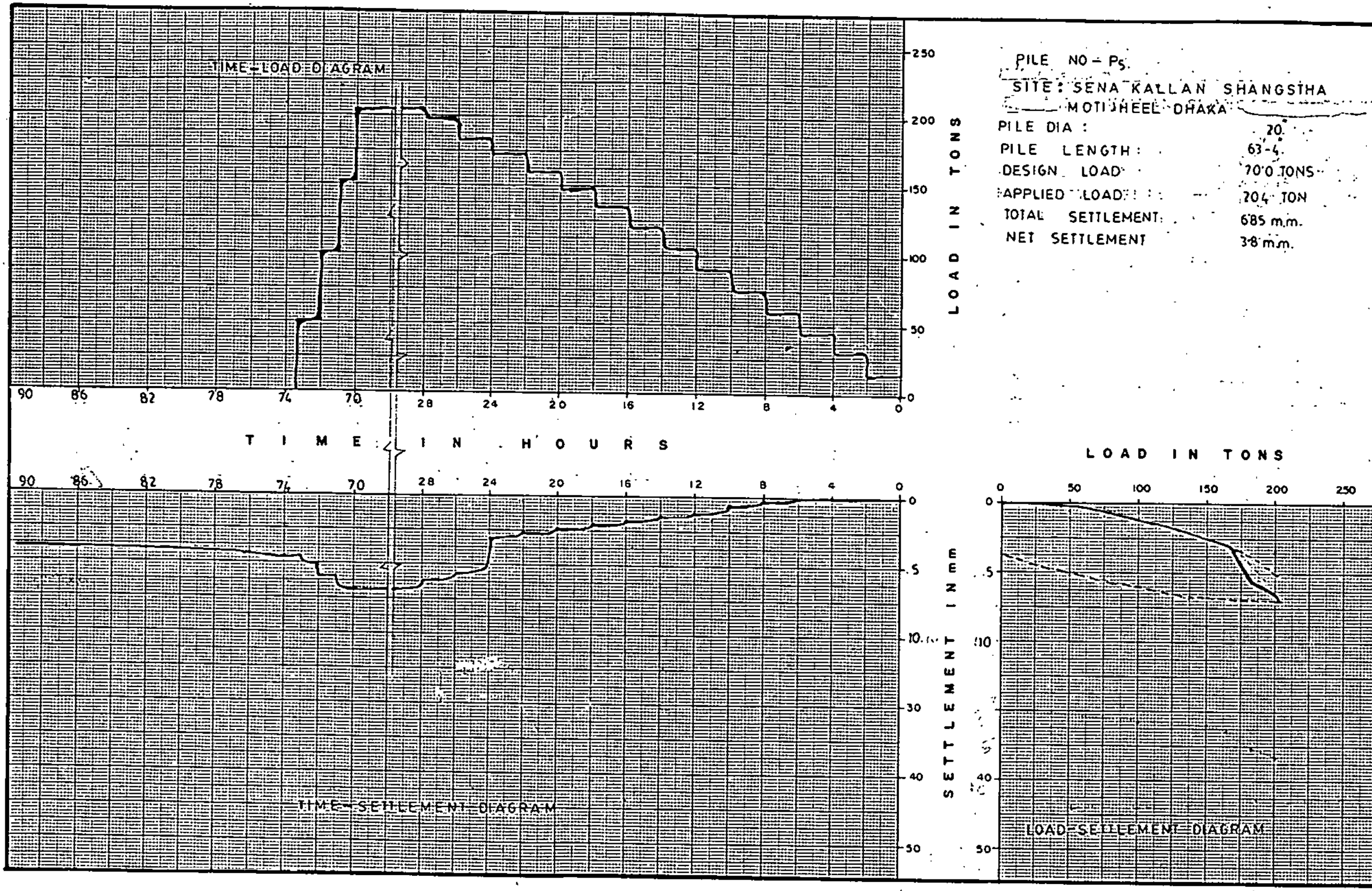


FIG. 5.5. LOAD-TIME-SETTLEMENT DIAGRAM

the loads were released in a decreasing rate. No load increment upto 300% of design load were maintained for less then one hour period. The time period maintained for 200% of design load for pile 1 to 5 were 18 hours, 22 hours, 12 hours, 10 hours and 2 hours respectively. Due to mechanical fault pile P_2 was subjected to load upto a period of 22½ hours at 60% of design load. For similar reasons, pile P_5 was kept under 180% of design load for a period of only 3 hours. It was not possible to apply a load more than 290% of design load due to the risky condition of the platform in pile P_2 and P_5.

CHAPTER-6

PILE LOAD TEST RESULTS

6.1 Introduction

The results of pile capacity obtained from the load test of five test piles from three different sites of Dhaka city are presented in this Chapter. Their theoretical analysis are also presented. Prediction of pile capacities from the theoretical analysis of the piles are based on different considerations and empirical relations.

6.2. Presentation

The theoretical pile capacity has been determined considering the basic equation as described in chapter-4. Since the variables in the general equation do not have any unique value for perticular soil-pile condition, the analysis does not yield an unique solution. In the theoretical analysis the bearing capacity factors and other variables as suggested by the different authors and investigators such as H.G.POLOUS and E.H.DEVIS[2], M.J.TOMLINSON[4], VESIC[8], MAYERHOF and VALSANKGAR[10] etc. were considered.

Table-6.1 shows the identification of piles with their location and specification. Table-6.2 shows the comparison of the load test results with the theoretical results considering the design parameters suggested by H.G.POLOUS and E.H.DEVIS[2]. Similarly Table-6.3 contains the theoretical values with design parameters suggested by M.J.TOMLINSON[6] and Tables-6.4 & 6.5 shows the theoretical values according to M.J.TOMLINSON and MAYERHOF[3] respectively. Table- 6.6 contains the theoretical values by taking into consideration the critical depth concept. The pile capacity from the load test values for all the five tables are calculated using the Devisson's prediction method* on actual load settlement curve. The concrete cylinder stregth was considered to be 2500 psi in calculating the axial deformation of the pile to apply in Devisson's prediction method. In calculating the theoretical capacity of piles N-values from soil investigation report i.e. field value was considered rather than the corrected N-values as suggested by Tomlinson[6].

* Cited by Fellenius, B.H.[16]

Table-6.1 : Identification of Piles

Sl.No.	Pile No.	Location of the site of pile	Pile Length (L)(ft.)	Pile Diameter (D)(inch)
1	2	3	4	5
1	P_1	Mogh Bazar	50	18
2	P_2	Mogh Bazar	70	18
3	P_3	Mohammadpur	50	18
4	P_4	Mohammadpur	60	18
5	P_5	Motijheel	63	20

Table 6.2 :Comparison of test and Theoritical capacity of Piles.

Sl.No.	Pile No.	Pile Capacity from theoritical analysis Qu(theo)* (Ton)	Pile Capacity from load test value Qu(test) (Ton)	Variations of the two results. $\frac{Qu(test)-Qu(theo)}{Qu(test)} X100$
1	2	3	4	5
1	P_1	60	215	72%
2	P_2	94	225	58%
3	P_3	122	163	25%
4	P_4	176	170	3.5%
5	P_5	189	204 **	8%

* The values of column 3 obtained on the basis of calculation suggested by H.G. Poulos & E.H. Davis[2] (1.2)

** The load 204T may not be the ultimate capacity. The pile may take more load.

Table 6.3 :Comparison of test and Theoritical capacity of piles.

Sl.No.	Pile No.	Pile Capacity from theoritical analysis Qu(theo)* (Tons)	Pile Capacity from load test value Qu(test) (Tons)	Variations of the two results. $\frac{Qu(test)-Qu(theo)}{Qu(test)} X100$
1	2	3	4	5
1	P_1	84	215	61%
2	P_2	139	225	38%
3	P_3	113	163	31%
4	P_4	124	170	27%
5	P_5	22	204	9%

* The values of column-3 obtained by Tomlinson's method[6] with a modification of considering ø at loose condition though the method of drilling during pile installation were rotary circulation method.

Table 6.4 :Comparison of test and Theoritical capacity of piles.

Sl.No.	Pile No.	Pile Capacity from theoritical analysis Qu(theo)* (Tons)	Pile Capacity from load test value Qu(test) (Tons)	Variations of the two results. $100X \frac{Qu(test)-Qu(theo)}{Qu(test)}$
1	2	3	4	5
1	P_1	228	215	6%
2	P_2	277	225	23%
3	P_3	286	163	75%
4	P_4	283	170	66%
5	P_5	389	204	90%

* The values of column 3 obtained on the basis of calculation suggested by M.J. Tomlinson[6].

Table 6.5 : Comparison of test and Theoritical capacity of piles.

Sl.No.	Pile No.	Pile Capacity from theoritical analysis Qu(theo)* (Tons)	Pile Capacity from load test value Qu(test) (Tons)	Variations of the two results. (Qu(test)-Qu(theo))/Qu(test) X100
1	2	3	4	5
1	P_1	148	215	31%
2	P_2	230	225	2%
3	P_3	182	163	12%
4	P_4	162	170	5%
5	P_5	277	204	36%

* The values of column-3 obtained using the increased critical depth over Tomlinson's value for submerged sand as suggested by Meyerhof and Valsankgar.C

Table 6.6: Comparison of test and Theoritical capacity of piles.

Sl.No.	Pile No.	Pile Capacity from theoritical analysis Qu(theo)* (Tons)	Pile Capacity from load test value Qu(test) (TONS)	Variations of the two results. (Qu(test)-Qu(theo))/Qu(test) X100
1	2	3	4	5
1	P_1	198	215	8%
2	P_2	287	225	28%
3	P_3	154	163	6%
4	P_4	171	170	0.6%
5	P_5	234	204	15%

* The values of column 3 obtained using Tomlinson's critical depth value suggestion for loose soil condition and ignoring submerged effect in calculatiang the effective vertical stress.

CHAPTER-7

DISCUSSIONS

7.1 Introduction

The observation of the work done in this study is analysed and discussed in this chapter. The discussion is made in two sub-headings; one is general discussion and the other is discussion on results. General discussion includes mainly the overall methods and considerations of design and execution of the project piling and their limiations. Discussion on results includes the criticism of the different theoretical approach for determining pile capacity; the variation of test results from theoretical predictions and factors influencing pile capacity.

7.2 General

Theoretical pile load capacity were calculated using the methods suggested by different authors such as Tomlinson(4,6), Whitaker, Poulos and Devis(2) and using the soil parameters as suggested by Tomlinson, Vesic, Berezantzev, Meyerhof and Terzaghi. Pile load capacity from load tests were calculated following different code of practice. In the calculation of pile capacity, the effect of load released by the excavation of the soil for load test upto cut off-level was not considered. The submerged unit wt.of soil as per soil investigation report, the principle of critical depth and maximum permissible end resistance and frictional resistance etc. as discussed in Chapt.3 were taken into consideration.

The degree of disturbance of soil surrounding the pile and below the end of the pile is difficult to predict. As such considerable judjement has been applied to estimate the effect of loosening of the soil and also of the bentonite slurry on the theoretical capacity of piles.

It has been considered that for piles in sand, the skin friction of the shaft and end bearing of the pile are independent of the depth i.e. overburden pressure beyond the critical depth but dependent only on the shear strength and relative density of the soil.

Because of the effects of overburden presure the results of the pile capacity will vary with time and climatic conditions because of change in ground water level. When ground water level is at higher position, the effective overburden pressure becomes less due to bouyant weight of the soil and vice versa. In this project work when load test was carried out, the position of the water level was not same as that reported in the soil investigation report and was used to estimate the pile capacity. As a result, descripancy between the theoretical and actual pile capacities may exist, which could not be ascertained quantitatively.

Soil investigation report does not provide all the design parameters needed for the analysis of a pile. In case of unavailability of useful data the empirical relations and experiences of the previous authors were used.

When load test is performed the total load on the platform may not be transfered to the pile for all time of load application and it may cause an increased overburden pressure or surcharge in the vicinity of the pile. Hence inaccurate estimate of the carrying capacity of the pile would occur which could not be correctly accounted for in present calculations.

The test result is very much dependent on the test procedure and the method which is used to evaluate the test results. Moreover, workmanship and construction method is a major point affecting the carrying capacity of a pile. For example excess bentonite lessen the friction etc. and insufficient bentonite causes siltation and caving, hence reducing the end bearing resistance and damaging the uniformity of the hole and resulting in over estimation or under-estimation of pile capacity.

The ground water level in Dhaka city lies between the depth of approximately 18'-0" at Uttara and 60'-0" at New Eskaton according to Dhaka WASA report in the year 1985. In the sites at Moghbazar, Mohammadpur and Motijheel the ground water table lies below 37'-0" as per Dhaka WASA report of 1982 and below 45'-0" asper report of 1985. Therefore, in the analysis of pile capacity consider the submerged unit weight of soil below the above mentioned depth. It may be possible that position of water table shown in the soil investigation report is not actual ground water table position rather the entraped water within the soil.Sometimes it was observed that the quantity of concrete that should have been required for construction of a pile theoretically did not conform with the actual requirements. This may arise mainly due to variation in shrinkage factor and the relative dimension of bore hole with respect to pile dia.

The load test procedures actually followed does not conform strictly to any standard method or codal rule and the same applies to the case of load application and load duration. The method of settlement measurement such as fixing of dial gauge with reference beam and extension rod, length of the reference beam and distance of the support of the kentledge pad also does not always ensure the requirements of standard procedure. Both dial gauge extensometer and optical leveling method from a remote reference point should have been used for better results The load transfer system should be as much as concentric as possible by using spherical seating (ball and socket) arrangement.

On the light of above discussion it is apparent that an accurate estimate of pile capacity can not be expected from load test results in our country. Considerable study and research are needed to establish the degree of confidence level and probability of accepting this type of load test results.

The test results as well as the results obtained from the theoretical analysis and other data relating to piles such as pile location, pile length, pile dia. etc. are shown in Table 6.1 to table-6.6. The load test values given in the tables are those predicted by Devisson's method from load-settlement curve ploted in plain graph paper. The prediction of ultimate load capacity are also checked by the method proposed by Brinch Hanson's 90% criterion method and Butler and Hay's method. But among these three results the Davisson's one seems to be more consistent for all the piles. The percent variation column in the tables indicates the difference of the load capacity of pile determined by different methods with the theoretical results.

The theoretical capacity as shown in Table 6.2 has been calculated considering that the critical depth to pile dia ratio (Zc/d) varied from 5 to 20 for loose to very dense sands. For bored pile $\phi = \phi'-3$ was used for determining Zc/d and Nq values as suggested by Poulos and Davis[2]. For determining shaft resistance ϕ-values were taken $\phi = \phi'$ in all cases. For end bearing, $\phi=\phi'-3$, as suggested by Poulos and Davis[2], was taken because of loosing effect. The angle of friction between the pile materials i.e. concrete and the soil, δ, was taken as 0.75 ϕ as suggested by Broms[11]. In all cases maximum limiting values of skin friction of 1.0 Tsf and end bearing of 100 Tsf has been considered[*].

In Table-6.3 the theoretical capacity has been calculated considering that the critical depth varied from 10 to 20 times pile diameter as suggested by Tomlinson[4] for loose to dense sandy soil. Due to the loosening effect as stated above the ϕ-values are estimated considering the soil as loose and $\phi = 28^\circ$ was taken in calculating the skin friction. A value of $\phi = 30^\circ$ for calculating the end bearing of the pile was selected because of the compacting effect due to the self wt. of the raw concrete column (pile shaft). The angle of friction between the concrete and the soil, δ, was taken as

0.75 ø as was in previous cases. In all the cases the Nq values were taken according to the curve suggested by Berezantzev(fig.-3.5).

The theoretical pile capacity in Table-6.4 were calculated considering the critical depth as in Table-6.3 but the values of ø were taken as in situ values ignoring the effect of loosening as suggested by Tomlinson[4]. In this case that the critical values of depth (Z_c) are equal both for bored and driven piles. The values of δ in this case also were taken as 0.75 ø.

According to Meyerhof and Valsankgar[10] critical depth for submerged sands is 1.6 times higher than that for dry sands. In Table-6.5 the values of critical depth were taken 1.6 times more than that suggested by Tomlinson for submerged sand. For sand layer overlain by a clay layer, the increased value of critical depth has been used in pile capacity calculation. The values of ø as before, were taken as 28° and 30° and δ = .75 ø. In case of partly submerged sands, the increased critical depth were also considered.

In Table-6.6 also the values of ø and δ were taken

28°, 30° and .75 ø. It was considered that no water table is present within the critical depth and hence no effect of submergence of soil was considered in calculating the effective vertical pressure.

7.3 On Results

It is difficult to determine accurately the capacity of a pile considering the large number of variables affecting the pile capacity. The present investigation is aimed at obtaining a better method of pile capacity prediction on the basis of load test results of piles in certain selected locations. In Table-6.2 it is found that there is a wide variation between the theoretical and test pile capacity for pile P_1 and P_2. In case of P_3, P_4 & P_5 the theoretical results have the reasonable agreement with the tested results. The variation in the former is probably due to the consideration of low values of critical depth Z_c assumed in calculation. The critical depth concept is based on uniform soil layers. In the Moghbazar area, a 10' to 15' clay layer rests over a loose sand layer. The critical depth assumed for this location considering a loose sand layer. This might have resulted in low values of the pile capacity. In Mohammadpur and Motijheel site, pile were resting upon a very dense sandy layer and $\phi = \phi' - 3$ were used allowing a little loosening effect. As a result almost entire benifit of the end bearing might have been acheived which should be the major part of total capacity and therefore the two values of measured and calculated capacities are in close aggrement. This is contrary to the results found in Moghbazar site.

In Table-6.3 the predicted values of pile capacity and the load test results do not show a good agreement. Although the critical depth of 10 to 20 times pile diameter (higher values of critical depth for loose sand) were considered the ϕ- values were taken for the loose condition of the soil. The submerged unit wt. of the soil below the water position as shown in the soil investigation report were used in caculation. But the location of water table may not be as high as shown in the soil investigation report. In soil investigation report of piles P_1 and P_2 the ground water table was shown at a shallow depth (Fig. 4.2 to 4.6). As a result the effective vertical pressure is decreased and gives low pile capacity and the variations becomes as high as 61% and 38%. In Mohammadpur and Motijheel site the ground water levels are located as low as the critical depth. Hence small variation is found in the value of effective vertical pressure. And as such, the variation in the pile capacities by theoretical and experimental methods are found to be small compared to Moghbazar site. The variation is 31% and 27% and 9%. It is to be noted here that all the results in Table-6.2 and Table-6.3 obtained from theoretical analysis are in lower side excepting the result of Motijheel site and therefore we can say that these two criterion for calculating pile capacity are conservative and underestimate the actual capacity of piles.

The theoretical results of pile capacity shown in the Table 6.4 are much higher than that of load test values. The trend of results of pile capacity in this table are completely reverse to that of results in Table-6.2 and table-6.3. The main difference between the results of Table-6.3 and Table-6.4 lies in the consideration of ø values in the later. The ø values to be chosen considering 'disturbed or undisturbed' condition of the end bearing soil. In this case ø values are taken considering undisturbed or 'in situ' condition of the soil which may not be practically true. Since the ø values were considered for undisturbed state of the soil the capacity became higher.

In Table-6.5 it is clear that the theoretical and load test capacities are close to each other. Agreement may be due to consideration of higher values of critical depth (1.6 times) and ø-values in loose condition.

Of all the results, (in the five tables) the theoretical results in Table 6.6 are very close to that of the load test value except the result of pile P_2 which shows 28% higher value. This may be due to the fact that the pile P_2 was concreted 12 hours later after the completion of the bore hole. Still 28% variation in pile capacity may be taken as within close limit. The other values show fair agrement.

In Motijheel site though the theoretical results is very much similar to that of Mohammadpur site but variation is more in case of P_5. The cause of this variation may be that the applied load in P_5 is not failure load.

CHAPTER-8

CONCLUSION AND RECOMMENDATION FOR FUTURE STUDIES

The following conclusion may be drawn from the present study:-

1. Because of non-homogeneous character of soil, the exact analysis for a pile load capacity is not possible solely with the help of a soil investigation report.
2. Full scale pile load tests are required to estimate the pile load capacity.
3. The sub-soil investigation should be as much informative as possible so as to provide all the necessary details of the soils including water table variation with time, geology of the soils etc.
4. The factors that influence the capacity of a pile in a particular site is the method of drilling, water table location, amount of bentonite used in construction etc.

Finaly, with the limited data obtained here, the author could not draw any correlation equation between the theoretical results and the actual finding from test pile. However, the present study has been concluded that this study will not be an end in itself but will initiate far more serious studies into the problems of more accurately pridicting the bearing capacity of bored piles for Dhaka city area.

REFERENCES:

1. Nayak, N. V. 1985, "Foundation Design Manual Enlarged and Fully Revised", Third Edition, Dhanpat Rai & Sons, PP. 125-126.

2. Polous, H. G. and Davis, E. H. 1980, "Pile Foundation Analysis and Design", John Willey & Sons, New York, PP. 15-17.

3. Meyerhof, G. G. and Murdock L. J. 1953, "An Investigation of the Bearing Capacity of Some Bored and Driven Piles in London Clay", Geotechnique Vol. 3, No.7, PP. 267-282.

4. Tomlinson, M. J. 1975, "Foundation Design and Construction", Pitman Books Limited, London, Third Edition.

5. Brand, E. W. and Apichai, J. S. 1972, "Performance of Some Driven and Cast in Situ Piles in Bangkok Clay", A. I. T. Research Report No. 20.

6. Tomlinson, M. J. 1977,"Pile Design and Construction Practice", View Point Publication by the Cement and Concrete Association, PP. 129.

7. Skempton, A. W. 1959,"Cast in Situ Bored Piles in London Clay", Geotechnique Vol. 9, No. 4, PP. 153-173.

8. Vesic, A. S. 1970,"Test on Instrumented Piles, Ogeechee River Site", Proceedings of the American Society of Civil Engineers, Vol. 96, No. SM-2, PP. 561-584.

9. Fleming, W. G. K., Weltman, A. J., Randolph, M. F. and Elson, W. K. , 1985,"Piling Engineering", John Willey & Sons, New York, PP. 99.

10. Meyerhof, G. G. and Valsangkar, 1977,"Bearing Capacity of Piles in Layered Soils", Proceeding of the IX International Conference on Soil Mechanics and Foundation Engineering, Vol. 1 PP. 645-650.

11. Broms, B, 1966, "Methods of Calculating the Ultimate Bearing Capacity of piles, a Summery", Sols-Soils 5, PP. 18-31.

12. Coyle, H. M. and Castello, R, 1981, "New Design Correlations for Piles in Sand", Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 107, No. GT.7, PP. 980-983.

13. Fjellerup, F. E., 1977, "Load Test on Large Bored Piles in Sand", Proceedings of the IX International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, PP. 716.

14. Lambe, T. W. and Whitman, R. V., 1969, "Soil Mechanics", Wiley Eastern Private Limited, New Delhi, First Edition, PP. 502.

15. Weltman, A. J., DOE and CIRIA Piling Development Group, "Pile Load Testing Procedures", Report PG-7, PSA Civil Engineering Technical Guide 25, PP. 38.

16. Peck, R. B., Hanson, W. E. and Thornburn, T. H., 1974, "Foundation Engineering", Second Edition, John Wiley , New York, PP. 310.

17. Fellenius, B. H., 1980, "The Analysis of Results from Routine Pile Load Tests", Ground Engineering, U.K. PP. 19-21.