THE UNIVERSITY OF CALGARY

LOAD - DISPLACEMENT BEHAVIOR OF PILES

by

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ABSTRACT

This project involved performing laboratory tests on model piles to study the effect of load parameters on pile behaviour. Model instrumented piles were driven into a dry sand bed of uniform density prepared by pluviation. A data acquisition system was used to obtain load settlement readings from the pile.

The load parameters that were considered are loading history of the pile, cyclic loading on the pile and surcharge loading on the soil around the pile. The loading history and cyclic loading tests were conducted on piles with L/D ratios of 33, 26 and 20 whereas, the surcharge loading tests were conducted on piles having an L/D ratio of 26.

The effect of loading history was studied by conducting pile load tests in tension after compression and compression after tension. The effect of prior loading was obtained by comparing the tip, shaft and total resistance of the virgin loaded pile to that having a loading history. It was concluded that the ultimate failure load for piles having a loading history was significantly lower than those which had no prior loading. The pile shaft capacity was affected more than the tip capacity as a result of prior loading.

Slow repeated tensile loading on the piles was applied to study the pile displacement behaviour with an increase in the number of repetitions for various load ranges. It was concluded that repeated loads applied upto 10 % of the tensile failure

load caused negligible movements whereas, for higher load ranges the pile experienced a steady pull-out.

Surcharge loading was applied on the soil around the pile and its effect on the ultimate load, the tip and shaft capacity of the pile was studied. The effect of surcharge loading on the soil was assumed to simulate similar conditions as overburden pressure in the field. Hence, the variation of average unit shaft friction with depth was studied. Shaft resistance was found to increase steadily whereas, the tip resistance decreased with an increase in surcharge load. Plots of the average unit shaft resistance with depth show that shaft resistance increases with depth though at a decreasing rate.

Thirteen methods have been suggested in literature to obtain the pile failure load. In order to compare the pile failure loads obtained by different methods a criterion to predict the most probable failure load or mean failure load was devised using data from all the thirteen methods. Load deformation data from 101 pile tests were collected from literature and analysed to check the performance of each method individually against this criterion. Loads predicted by Chin's and Brinch-Hansen's 80 % criteria were higher and DeBeer's and Davisson's method were lower than the predicted most probable load. Single tangent and Double tangent methods gave good predictions of the ultimate load. Some settlement criteria were also considered and it was concluded that the definition of the predicted most probable failure load appeared to be satisfactory.

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To my parents

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NOTATIONS

A	=	cross-sectional area of the pile
A_P	=	area of the pile tip
b	=	bored pile
C	=	cohesive soil
c	=	cohesion of the soil
C_a	=	adhesion
CRP	=	constant rate of penetration test
d	=	driven pile
D	=	diameter of the pile
D_r	=	relative density of the soil
E	=	expanded base pile
F_s	=	total skin friction
f_s	=	average unit shaft friction
f_z	=	unit shaft resistance at depth, z
H	=	H-section pile
I	=	I-section pile
K	=	ratio of normal stress to vertical stress
K_A	=	active earth pressure coefficient
K_o	=	earth pressure coefficient at rest

K_P	=	passive earth pressure coefficient
K_u	=	theoretical uplift coefficient
K_s	=	average coefficient of earth pressure
L	=	length of embeddment of pile
L_{cr}	=	critical length of the pile
L/D	=	ratio of length to diameter of pile
$(L/D)_{cr}$	=	critical embeddment ratio
L_R	=	ratio of ultimate load predicted by a method to the most probable ultimate load
N	=	straight shafted pile
NC	=	non-cohesive soil
N_c^*	=	bearing capacity factor
N_q^*	=	bearing capacity factor
N_γ^*	=	bearing capacity factor
$N\sigma^*$	=	mean normal ground stress
Q	=	load on the pile head
Q_{max}	=	ultimate load of the pile
Q_P	=	total tip resistance
Q_u	=	ultimate uplift load
q	=	surcharge load
q_u	=	ultimate point bearing capacity of the pile
QML	=	quick maintained load test

R_1	=	ratio of gross pile head settlement at the most probable ultimate load to the diameter of the pile expressed as a percentage
R_2	=	ratio of incremental settlement to incremental load
r	=	radius of the pile
S	=	gross settlement of the pile
S_e	=	elastic compression of the pile
S_o	=	limiting settlement in modified hyperbolic method
S_u	=	settlement of the pile at the most probable ultimate load
SML	=	slow maintained load test
Y	=	modulus of elasticity of the pile material
δ	=	friction angle at the pile soil interface
$\Delta\sigma_R$	=	radial stress
$\Delta\sigma_v$	=	vertical stress
$\Delta\epsilon_R$	=	radial strain
$\Delta\epsilon_v$	=	vertical strain
γ'	=	effective unit weight of the soil
μ	=	roughness coefficient of the pile
ϕ'	=	effective angle of internal friction
σ_h'	=	lateral effective stress
σ_o	=	mean normal ground stress
σ_v'	=	effective overburden pressure

CHAPTER 1

INTRODUCTION

1.1 General

Piles have long been used as structural members to overcome the difficulties of founding on compressible soils. They were used in ancient times as timber stakes driven into the ground. However, it was not until early twentieth century that any research into the mode of pile behaviour and mechanism of its load transfer was initiated. Probably the first literature published on pile performance is *Piles and Pile Driving* edited by Wellington of the Engineering News (later to become Engineering News-Record) in 1893.

Pile foundations are used to transmit the load of a super structure directly to a hard sub-soil layer when the surface soil is too weak or compressible. Piles are also used to transmit inclined loads and in places where scour is likely to occur. Sometimes piles may be required to take uplift forces, as in offshore structures where the wave action and bouyancy induces tensile forces.

Probably one of the most important aspect of a pile foundation, is its ultimate capacity. The ultimate capacity of a pile can be evaluated either by empirical equations derived and used locally for a particular type of soil or by the use of static formulae, which require detailed knowledge of the strength and deformation characteristics of the soil, its variation in density and the moisture content of the soil.

Design of pile foundations, like other branches of geotechnical engineering is by no means an exact science. Design procedures, even today, are based on empiricism. As a result of the many uncertainties involved in predicting pile performance, it sometimes becomes necessary to conduct field tests, which are very expensive and may be difficult to carry out.

Another means of studying and understanding pile behaviour is by conducting model tests. Model laboratory tests, again, have their own limitations. However, these are inexpensive to carry out and have the advantage of being conducted in laboratory controlled conditions. There are many parameters that may affect the performance of a pile. These can be individually studied in model tests.

Field tests or laboratory tests have the same basic philosophy. A pile is pushed or driven into the soil and then a load test is conducted. On the basis of the load - displacement behaviour of the pile, the ultimate pile capacity is obtained. Initial portion of the pile load displacement curve is essentially linear and gradually turns non-linear with increase in load. There is a significant controversy regarding the way the ultimate capacity of a pile is obtained. Many investigators (Fellinius 1975,1980; Joshi and Sharma 1987) have given their own interpretation of the load settlement curve and have suggested different methods to evaluate the ultimate load.

Numerous factors affect the performance of piles. The pile installation method, the soil characteristics, the loading techniques, to mention a few. The variations in the pile displacement behaviour due to these, make it additionally difficult to have a universal criterion or method to determine the pile capacity.

The effect of prior loading accidental or otherwise, on the pile behaviour has been observed (Koreck and Schwarz 1988) to have a significant effect on pile performance but no indepth study into this criterion has been conducted. The effect of slow cyclic loading on sand has been known to cause soil degradation but some controversies remain unresolved.

The effect of overburden pressure on the pile capacity has not been understood very well. Vesic (1964) concluded that the unit shaft and tip resistance of a pile in sand becomes constant as depth increases due to arching effects. However, more recent reports (Kulhawy 1984) mention that the unit tip and shaft resistance keep increasing with depth though at a decreasing rate. Laboratory experiments to study the effect of overburden by applying a surcharge load on the soil around a pile has not been conducted to observe whether this effect is true or not.

1.2 <u>Scope of the Study</u>

This project involves the study of the effect of load parameters on the performance of model piles driven in sand. Model piles were driven in sand of uniform density, by the traditional hammer and pulley arrangement. The sand bed was pepared by pluviation. The load parameters that were considered were: loading history of the pile, repeated loading on the pile and surcharge loading on the soil around the pile. To determine the ultimate load of a pile a criterion involving all existing methods was devised. In order to check the performance of various methods given in literature a number of field pile load displacement curves were analysed. The settlement condition was used to check whether the defined criterion of ultimate load of pile was realistic or not.

1.3 Objectives

The objectives of this study were :

1. To study the effect of prior loading on the tip, shaft and total capacity of a pile seated in dry fine granular soil. Tests were conducted both in tension after compression and in compression after tension.

2. To study the effect of repeated tensile (one-way) loading on the pile in sand. To obtain the maximum repeated load that can be applied without the pile failing due to soil degradation.

3. To determine the effects of surcharge loading the soil on pile performance; and evaluate the variations of the ultimate load of a pile, and its tip and shaft capacity as a result of surcharge loading.

4. To study the variation of average unit shaft resistance with depth. The variation of $K^*\tan\delta$ with overburden pressure was studied.

5. To obtain the ultimate load of a pile by a simple criterion involving all available methods. A comparative study of the various methods, given to evaluate the pile failure load was made, based on load settlement data published in literature.

CHAPTER 2

LITERATURE REVIEW

Piles are vertical or slanted structural foundation members having a relatively larger length compared to its cross-sectional dimensions. A pile transmits loads to the subsoil in two ways : one is by bearing against the soil in the form of tip or end bearing capacity and another is by the frictional resistance all along the shaft termed as shaft capacity or frictional resistance of the pile. In end bearing piles the shaft capacity makes up a lesser fraction of the ultimate load than the tip capacity.

Detailed discussions on the mechanism of load transfer, pile point and shaft capacity and parameters affecting the pile performance are presented here.

2.1 Load Transfer and Failure Mechanism

The vertical surface of a pile in soil is subjected to horizontal earth pressures by the soil. With increasing vertical load frictional forces will be mobilized. Figure 2.1 shows the various types of load transfer mechanisms from pile to soil, when subjected to different types of loading. Figures 2.1a and 2.1c show the type of load transfer when a pile is vertically loaded in compression and in tension respectively. A small tip resistance may develop when piles are subjected to tension loading in saturated clays due to suction. Otherwise, in dry sands the tip resistance in uplift is essentially zero.

The effect of bending moments and lateral forces is shown in Figure 2.1b. The

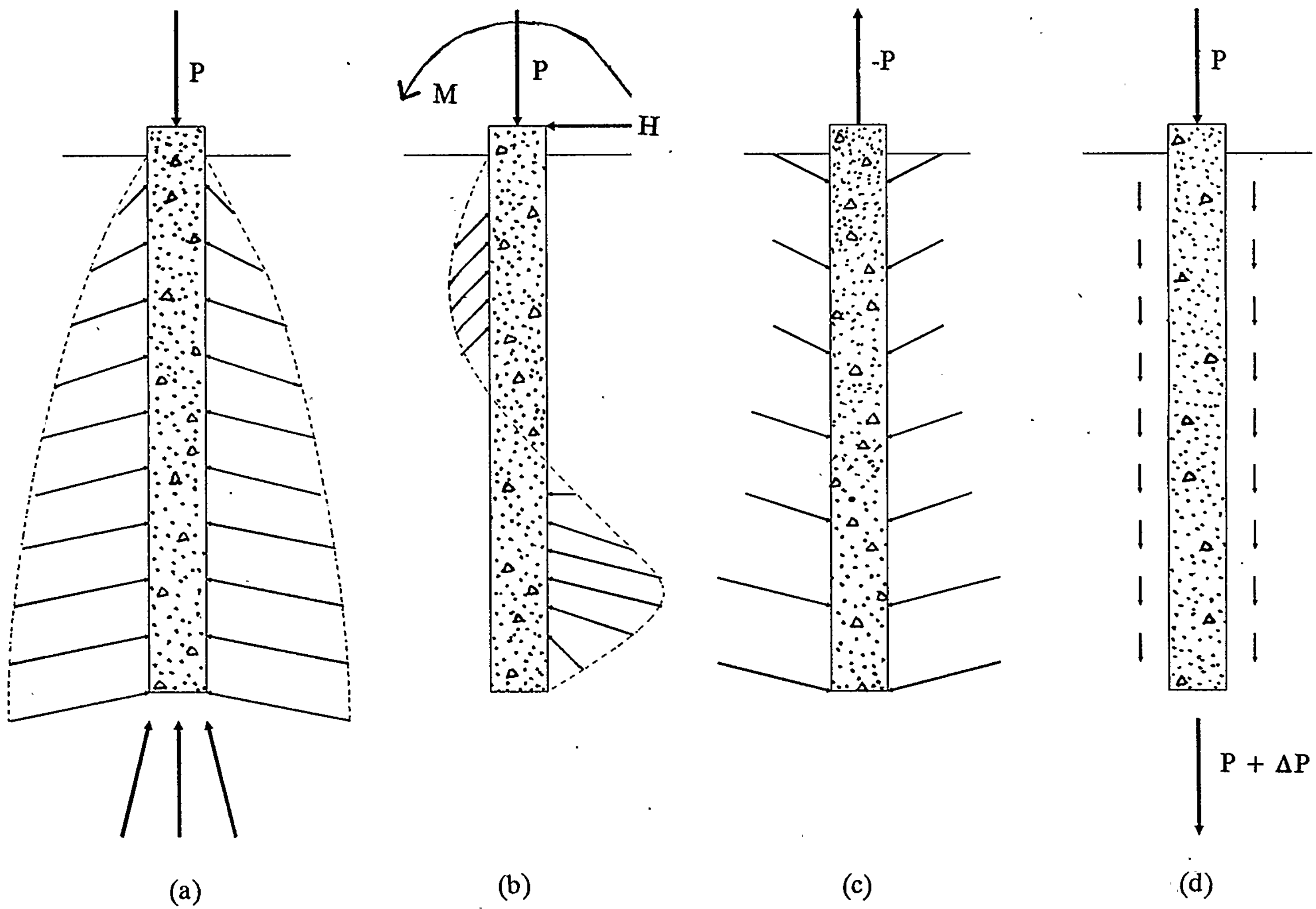


Figure 2.1 Load Transfer from Piles to Soil - Winterkorn and Fang 1975 *(adapted)*

load transfer as shown in Figure 2.1d occurs, when the upper layers of the soil is consolidating, as in clays. Here the soils transfer downward forces on the pile known as negative skin friction.

The mode of failure of a foundation depends on the shear strength of the soil and the type of pile used. In foundations, three kinds of failure modes are observed (Vesic 1967; Winterkorn and Fang 1975)

1) General shear failure of a foundation occurs with sliding surfaces and a definite failure load, as in shallow foundations. In such a failure, the soil properties are assumed to be such that a slight downward movement of footing develops fully plastic zones and the soil bulges out.

2) Local shear failure occurs when the lateral compression produced by the penetration of the footing is less than that required for the soil to bulge out. It occurs in soft or loose and compressible soil, where large deformations may occur below the footing before the failure zones are fully developed.

3) Punching shear failure, generally a characteristic of deep foundations in homogeneous soil, occurs when only lateral compression occurs and the shear stress mobilized is less than the shear strength of the soil.

The failure mode of a particular foundation is a function of the soil density and the ratio of foundation depth to width.

The ultimate capacity of a pile can be divided into two parts, the end bearing and shaft resistance.

2.2 Point Bearing Capacity

The general ultimate point bearing equation is given by,

$$q_u = cN_c^* + \sigma'_v N_q^* + \gamma' D N_\gamma^* \quad \text{.......................} \quad (2.1)$$

where, q_u = ultimate point bearing capacity of the soil

c = cohesion of the soil

σ_v'= effective overburden pressure at the pile point level = $\gamma' L$

γ' = effective unit weight of the soil at the pile point

D = diameter of the pile

L = Length of embeddment of the pile

N_c^*, N_q^* and N_γ^* are bearing capacity factors taking into consideration the shape and depth of the foundation.

In the above equation, for piles in sand, the value of c becomes zero and also the factor, $\gamma' D N_\gamma^*$ becomes negligible compared to $\sigma_v' N_q^*$ as the diameter of a pile is much smaller than the length. Hence the first and the last term in Equation 2.1 are neglected for design of piles in sand. Equation 2.1 thus reduces to,

$$q_u = \sigma'_v N_q^* \quad \text{...........................} \quad (2.2)$$

Various investigators, based on different assumed failure patterns under the tip have suggested different bearing capacity factors. Figure 2.2 reproduces a comparative study of the bearing capacity parameter, N_q^*, given by different investigators.

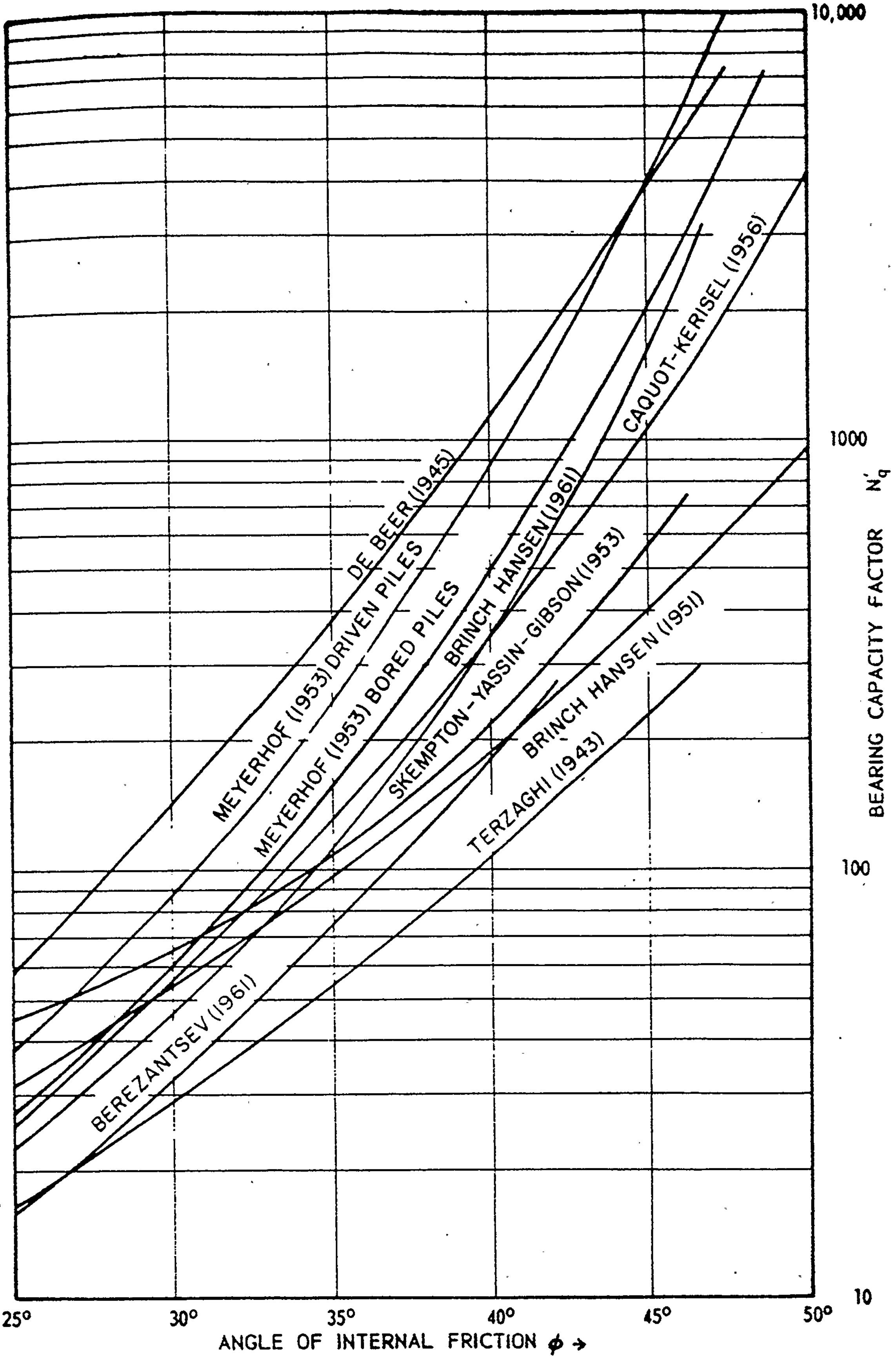
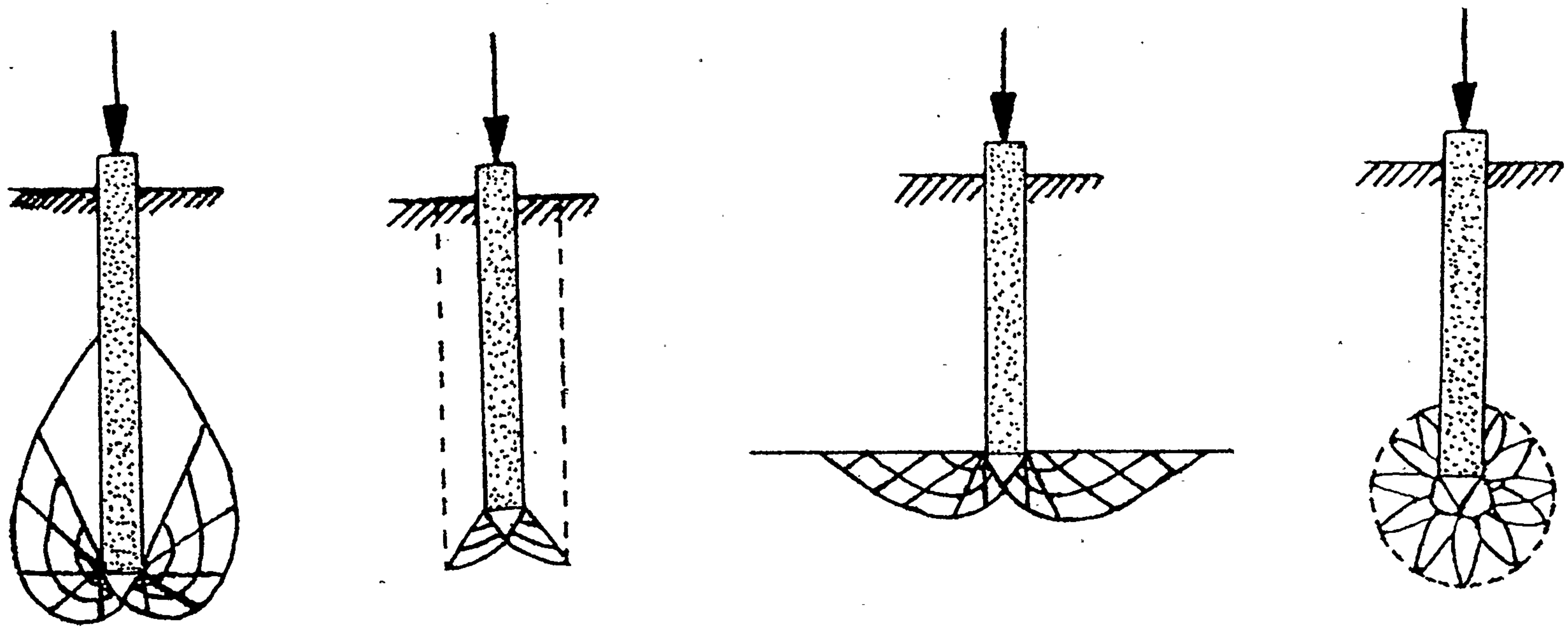


Figure 2.2 Bearing Capacity Factor, N_q^* versus Angle of Internal Friction, ϕ' - Vesic 1964

Solution for the point bearing resistance of piles was originally developed by Prandtl (1921) and Reissner (1924). It was a theoretical study into the penetration of a rigid stamp into an incompressible solid. This theory was used by Caquot (1934) and Buisman (1935) for the solution of the bearing capacity of deep foundations. De Beer (1945) and Jaky (1948) used a similar approach and obtained quite different bearing capacity factors. The failure pattern used by De Beer (1945) and Jaky (1948) are given in Figure 2.3a. Terzaghi (1945) assumed the same failure pattern for deep foundations as for shallow foundations replacing the soil above the pile tip level by an equivalent overburden pressure and ignoring the shear strength of the soil (Figure 2.3c). Berezantsev (1952, 1961) and Vesic (1963) consider the failure to occur beneath the pile tip only (Figure 2.3b). Berezantsev (1952, 1961) considers that the vertical stress acting at the level of the pile tip as smaller than the overburden pressure because of frictional forces acting on the cylindrical surface. Meyerhof (1953) assumed the same failure pattern as De Beer (1945) which considers the general shear failure pattern considering the shear strength of the soil also. Skempton *et al.* (1953) assumed a failure pattern which is simulated by an expanding spherical cavity (Figure 2.3d).

It is assumed that the pile tip resistance is proportional to the overburden pressure and that the tip capacity of a pile increases linearly downwards upto a certain depth. Vesic (1973) suggested that the point resistance is governed by the mean normal ground stress and not the effective overburden pressure as was previously assumed. The mean normal ground stress is given by,



(a) De Beer, Jaky, Meyerhof

(b) Berezantsev, Vesic

(c) Terzaghi

(d) Skempton, Yassin, Bishop

Figure 2.3 Failure Patterns in Soil under Pile Tip - Winterkorn and Fang 1975 *(adapted)*

$$\sigma'_o = \left(\frac{1+2K_o}{3}\right)\sigma'_v \quad \text{.......................... (2.3)}$$

where, K_o = earth pressure coefficient at rest

σ_v' = effective overburden pressure

σ_o' = mean normal ground stress

In the failure pattern assumed by Vesic (1977) there is a highly compressed conical wedge under the pile tip which, in dense sands, pushed the radial shear zone laterally. According to this theory,

$$q_u = cN_c^* + \sigma'_o N_\sigma^* \quad \text{....................... (2.4)}$$

where,

$$N_\sigma^* = \frac{3N_q^*}{(1+2K_o)} \quad \text{........................... (2.5)}$$

However, the full pile tip resistance is mobilized only after the pile has undergone a settlement of 10 to 25 % of the width of the pile. The pile tip resistance, in dense sands, does not increase linearly with depth due to arching effects (Vesic 1964) and due to decrease in ϕ' with an increase in the normal stress (Randolph 1985). It was reported (Vesic 1964) that the unit tip resistance increases linearly with depth only upto a depth of four times the diameter of the pile. At depths greater than fifteen times the diameter of the pile the tip resistance reaches a constant value asymptotically. This constant value also known as the ultimate tip resistance is a function of the relative density of the sand alone and is independent of the overburden pressure, σ_v'. Vesic (1964) mentions that in loose sand the ultimate tip resistance occurs at a depth of about 10 pile diameters in the soil, whereas, in dense sands it occurs at about 30 pile diameters. The average displacement required for a driven pile to reach this varies from 7.8 % of the pile diameter for loose sands to about 14.5 % of the pile diameter for dense sands.

However, this concept of limiting pile tip capacity has been disputed by Kulhawy (1984). Instead it is argued that the pile tip capacity keeps increasing with depth though at a reduced rate.

2.3 Shaft Resistance

The load transferred by skin friction has sometimes been neglected on the assumption that it contributes a very small fraction of the total pile capacity in cohesionless soil (Skempton *et al.* 1953). However, experience has shown this not to be correct and that skin friction may make a sizeable contribution to the capacity of a pile, especially if the pile is tapered. The load transfer through the pile walls is estimated by assuming development of adhesion and friction between the pile walls and the surrounding soil. Therefore, f_z, the unit shaft friction can be represented as such:

$$f_z = C_a + \sigma_h' \tan\delta \quad \ldots\ldots (2.6)$$

where, C_a = adhesion

$\sigma_h^{\prime}$ = the normal effective stress on the pile

K = ratio of normal stress to the vertical stress = $\sigma_h^{\prime} / \sigma_v^{\prime}$

δ = friction angle at the soil pile interface

Potyondy (1961) determined a relationship between δ, the friction angle at the pile soil interface and $\phi^{\prime}$, the angle of shearing resistance of the soil.

Vesic (1977) suggested the angle of friction at the pile soil interface to be independent of the initial soil density and the pile material. Since, the sand at the interface can be considered to be in a state of ultimate failure, the friction angle, δ, can be considered to be the same as $\phi^{\prime}$ at failure. Das (1984) concludes that δ varies in range of $0.5\phi^{\prime}$ to $0.8\phi^{\prime}$.

The determination of normal stress on the shaft is relatively complicated. The lateral earth pressure on the pile is assumed to be directly related to the vertical earth pressure, despite some reports (Shultze 1952) that the lateral distribution of load through skin friction does not necessarily agree with conventional earth pressure theories. Usually, earth pressure is assumed to vary from one half to the equivalent of the full vertical earth pressure for straight sided piles (Meyerhof 1956) and it approaches the passive earth pressure values in case of tapered piles (Ireland 1957; Nordlund 1963).However, in these studies it was assumed that the pile displaces the sand in a horizontal direction without any vertical deformation.

It was reported (Vesic 1970; Meyerhof 1976) that the ultimate unit shaft resistance, f_z, increases with depth upto a level where it reaches a maximum and then decreases to a minimum at the pile point. Accordingly, the corresponding local coefficient of earth pressure, K_z, on the shaft decreases with depth along the pile from a maximum near the top where K_z may approach the passive earth pressure coefficient to a minimum near the pile point where K_z may be less than the at rest earth pressure coefficient, K_o. There is no uniform variation of average coefficient of lateral earth pressure, K_s for piles in the field. For a particular ϕ' value, K_s can vary considerably from a lower limit of K_o for bored piles or piles jacked in loose sands to about four times this value or more for piles driven in dense sands due to dilatancy and other effects (Meyerhof 1976).

Das (1984) suggested some values to be used for design calculations of K

For bored or jetted piles,

$$K = K_o = 1 - \sin \phi'$$

For low displacement driven piles,

$$K = K_o \quad - \quad \text{lower limit}$$

$$K = 1.4K_o \quad - \quad \text{upper limit}$$

For high displacement driven piles,

$$K = K_o \quad - \quad \text{lower limit}$$

$$K = 1.8K_o \quad - \quad \text{upper limit}$$

As in the case of tip resistance, the shaft resistance also increases linearly with depth upto a certain depth beyond which it asymptotically attains a constant value. This constant value is a function of the initial sand density and is independent of the overburden pressure. The skin friction reaches a near constant value between 10 to 30 pile diameters (Vesic 1964). Again, as in tip resistance this limiting of shaft resistance has been disputed (Kulhawy 1984) and it is presently felt that the skin friction increases linearly with depth upto a certain point beyond which its increase with depth diminishes.

It is significant to note that the displacement needed to reach ultimate skin resistance is independent of the initial density of the sand and is equal to about 10 mm.

2.4 Uplift Capacity of Piles

Probably the first full scale field pile test loaded in tension was performed by Ireland (1957). Based on the data obtained from these tests, the following equation was given for the uplift capacity of piles.

$$Q_u = \frac{1}{2} K_S \pi D \gamma' L^2 \tan\phi' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.7)$$

where, Q_u = ultimate uplift load

K_s = average coefficient of lateral earth pressure

L = length of embeddment of piles

It was suggested that the maximum value of K_s to be considered for an initial estimate of Q_u as 1.75.

Meyerhof (1973) has suggested the equation

$$Q_u = (C_a + \sigma'_v K_u \tan\delta)\,\pi D L \quad \text{(2.8)}$$

where, K_u = theoretical uplift coefficient

Das *et al.* (1977) have suggested δ to vary from $0.4\phi'$ for very loose sands to about ϕ' for very dense sands.

Many researchers have given many formulae (Janbu 1971; Tejchman 1971; Coyle and Sulaiman 1967) but the basic equation remains the same as

$$Q_u = \frac{1}{2} K_u \gamma' L^2 \pi D \tan\phi' \quad \text{(2.9)}$$

The variation is only in the value of K_u. Meyerhof (1973) gives the value of K_u as between 0.5 and 1.0. Janbu (1971) gives the equation

$$Q_u = K_u \gamma' L^2 \pi D \quad \text{(2.10)}$$

where, $K_u = f(f_z, \mu)$

f_z is the maximum friction mobilized at a depth z

and μ is the roughness coefficient (≤ 1) of the pile

Tejchman (1971) gives Equation 2.11

$$Q_u = \frac{1}{2} \gamma' L^2 \pi N_t D \quad \text{(2.11)}$$

where, $N_t = f(\phi) = 0.15$

On the basis of experiments conducted, Das (1983) has concluded that the unit shaft resistance increases linearly with depth upto a critical depth beyond which it remains constant. This critical depth beyond which the uplift capacity remains constant, increases with relative density of compaction. On this basis, the following equation was given

$$Q_u = \frac{1}{2}\pi D\gamma' L_{cr}^2 K_u \tan\delta + \pi D\gamma' L_{cr} K_u \tan\delta\,(L - L_{cr}) \qquad (2.12)$$

L_{cr} is the critical length of the pile beyond which the pile capacity becomes constant.

Equation 2.12 might give estimates of the capacity of piles which are 18 to 20 % less than actual field capacity.

$$(L/D)_{cr} = 0.156 D_r + 3.58 \ldots (D_r \leq 70\%) \qquad (2.13)$$

$$(L/D)_{cr} = 14.5 \ldots (D_r > 70\%) \qquad (2.14)$$

where, $(L/D)_{cr}$ is the critical L/D ratio of the pile beyond which the unit uplift capacity of the pile remains constant.

D_r is the relative density of the soil

The critical L/D ratio beyond which the average shaft resistance in tension remains constant varies from 11 in medium dense sands to 30 in dense sands for smooth piles and is about 15 in medium dense sands for piles with rough surfaces (Choudhuri and Symons 1983).

There has been relative controversy regarding the compression and tension shaft resistance of piles. Broms (1963), Mohan *et al.* (1963) and Hunter and Davisson (1969) have observed that the tension shaft capacity is much less than the compression shaft capacity. Sparrow (1988) upon conducting laboratory tests concluded that the tension shaft resistance of a pile is lesser than the compression shaft resistance by about 15 %. On the other hand, Vesic (1970), the present API guidelines based on experiments conducted by Olson and Dennis (1982) and Dennis and Olson (1983) suggest there be no distinction in the shaft capacity of piles in tension and compression. Poulos and Davis (1980) and Kaniraj (1988) have suggested that the ratio of the shaft resistance in tension to that in compression to be in the range of 0.5 to 0.67.

2.5 <u>Residual Stresses</u>

Residual stresses are stresses that get locked in a pile when it is driven in the soil. These stresses may not get dissipated due to the lateral and vertical pressure of the soil around. Existence of residual stresses in driven piles is widely recognized (Gregersen *et al.* 1973; Hanna and Tan 1973; Holloway *et al.* 1975). Observations by Hunter and Davisson (1973) confirm the existence of substantial residual stresses in driven piles. However, the effect of residual stresses may be neglected as being negligibly small when the elastic compression of the pile (in case of rigid piles) is very small compared to the elastic compression of the soil.

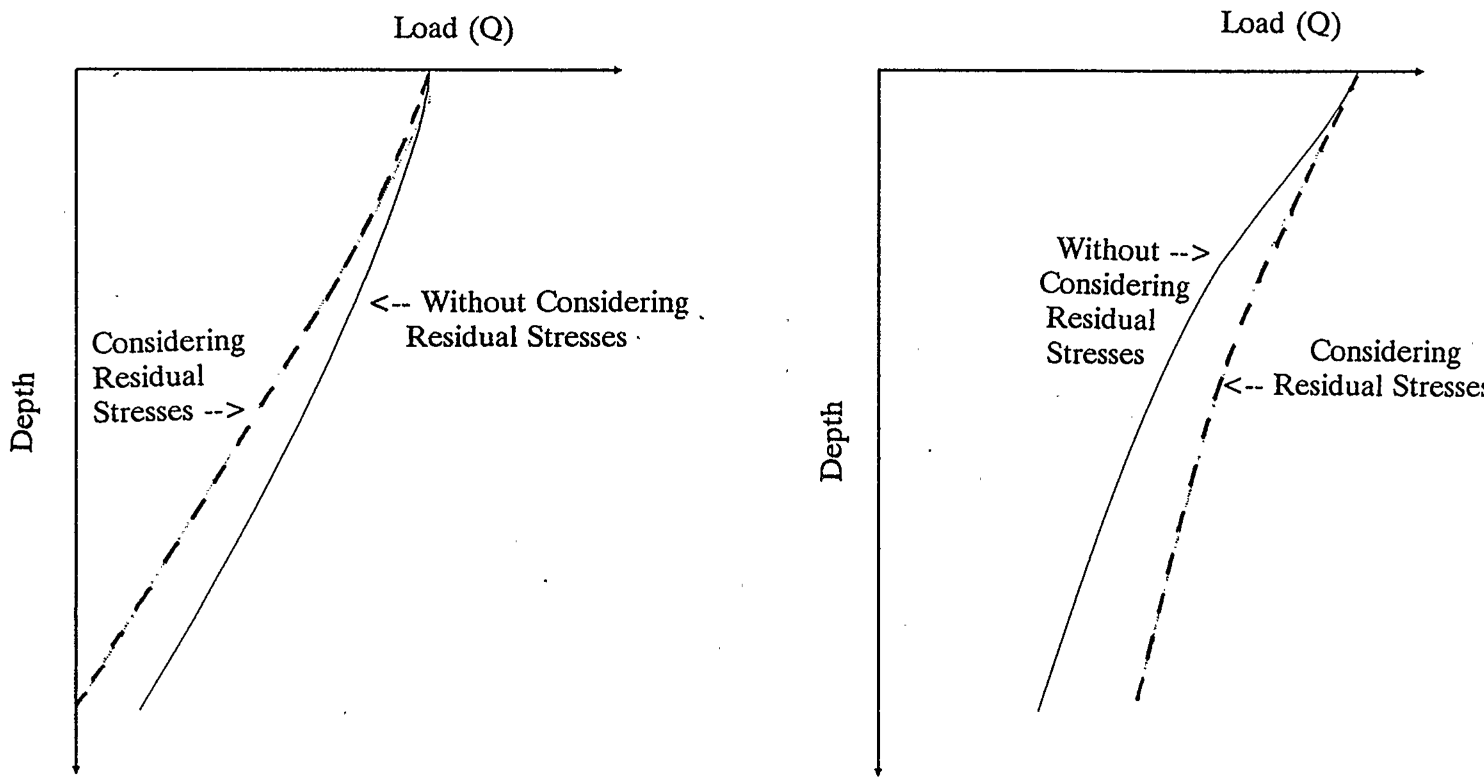
Residual load adjustments do not affect the ultimate tensile and compressive capacity of a pile, but affect the magnitude of the tip load and shaft resistance separately. Compressive stresses are likely to exist in the lower part of a pile. These depend on the pile-soil system only, and is independent of the impact pile driving apparatus. When a residual point load remains after driving, a portion of the total point bearing capacity has already been mobilized (Poulos and Davis 1980). Hence, if the residual forces are neglected during a test and the gauges and load cell are zeroed before a test, the tip load will be lesser than the actual value. The shaft resistance recorded in this case will be greater than actual. Figures 2.4a and 2.4b gives the effect of residual load on the pile shaft and tip capacity.

If residual stresses are neglected, the tip will show some resistance, in tension tests, and consequently the uplift shaft resistance will be recorded lower than the actual value.

Sparrow (1988) measured the residual stresses and concluded that these were insignificant. But this was attributed to the small length and rigidity of the pile.

Tan and Hanna (1974) found a difference of 34 % in the pile tensile capacity as a result of residual stress. Kraft (1991) however, concludes that the effects of residual stresses in the pile after installation have much less impact on the capacity than indicated by Tan and Hanna (1974).

The exact mechanism of residual stresses is not understood completely and their effects are not readily taken into account. However, as Poulos and Davis (1980)



(a) Tension

(b) Compression

Figure 2.4 Effect of Residual Stresses on Load Transfer of Piles - Hunter and Davisson 1969 *(adapted)* mentions, recognition of its effect may at least resolve some anomalies in some load tests.

2.6 Parameters affecting Pile Load Displacement Behaviour

The behaviour of piles in soil is dependent on many parameters most of which are interlinked and a variation in one may cause a significant difference in others. This is one of the prime reasons why soil is very difficult to model. The basic parameters that affect pile behaviour, can be broadly divided and discussed under four major categories (Kraft 1991) :-

1. Soil parameters
2. Pile parameters
3. Installation method
4. Load parameters

2.6.1 Soil Parameters

The soil parameters are some of the important parameters that affect the tip and shaft capacity of piles. The soil properties that are of significance are, the earth pressure coefficient, K, the stiffness modulus of the soil, the compressibility modulus, soil friction angle, soil-pile friction angle and stratigraphic changes (Kraft 1991).

The earth pressure coefficient, K - The earth pressure coefficient, K, of a soil is the ratio of the lateral effective stress to the vertical effective stress at a particular point. The earth pressure coefficient of the soil is generally obtained for 3 cases. The active earth pressure coefficient, K_A, when the soil exerts a tensile force on the surface under consideration. In this case, the soil has a tendency to move away from the surface. It is in unstable equlibrium and a little force may lead to soil separation from the surface cause cracks to form. The at rest earth pressure coefficient, K_0, is used when the soil is undisturbed. Following is a relationship when the soil is normally consolidated:

$$K_0 = 1 - \sin\phi' \quad \ldots\ldots (2.15)$$

The passive earth pressure coefficient, K_P, is used when the soil exerts a compressive force on the surface under consideration. Depending on the method of installation of a pile the earth pressure coefficient may vary between K_0 and $1.8K_0$ (Das 1984). During driving a pile, the lateral stresses increase and may approach the passive condition where,

$$K = K_P = \frac{1+\sin\phi'}{1-\sin\phi'} \quad \ldots\ldots (2.16)$$

But as the pile toe penetrates, the lateral stresses tend to decrease due to unloading and vibration "fluffing" of the soil as the pile advances (Kraft 1991). Nauroy and Le Tirant (1983) and Smits (1982) have reported similar conclusions. The stress reduction in this case is much more in the case of dense sands than in the case of loose sands. However, the effect of unloading vibrations is much more in loose sands than in dense sands. Both phenomena of stress reduction as toe penetrates in the soil and unloading vibration "fluffing" of the soil have an effect in lowering the earth pressure coefficient as a result of pile installation.

Nauroy and Le Tirant (1983) measured the lateral stresses in model tests in sands. It was observed that the lateral stresses near the pile vary between 0.6 to 2.85 times the initial lateral stress, depending on the compressibility of the sand, the displacement characteristics of the pile and the method of installation.

Pile loading also affects the lateral stress along the shaft. In loose sands, due to contraction, the lateral stress and hence the earth pressure coefficient, K, increases. The magnitude of stress, the density and the overconsolidation ratio determine the tendency of a soil to exhibit contraction or dilation.

The soil friction angle, ϕ' - The soil friction angle, ϕ', is an intrinsic property of any soil that plays a significant role in soil characterization. It depends on the fabric of the soil and is affected by a variation in soil density. The lateral earth pressure coefficient, K, is also dependent on ϕ'. Probably one of the major parameter in determining the tip and shaft capacity of a pile is the angle of internal friction.

The factor N_q^* in Equation 2.2 is a logarithmic function of ϕ' in which every unit increment of ϕ' causes an exponential rise in N_q^*. Hence, a small variation in ϕ' causes a wide variation of tip capacity.

The pile shaft capacity is given by Equation 2.6. In this equation both q_s and δ are related to ϕ'.

The installation of pile in a non-cohesive soil tends to alter the soil fabric at the pile soil interface. The effect on the soil fabric is largely dependent on the mode of installation. A driven pile will affect the soil fabric much more than a bored pile. The installation procedure also affects the grain size distribution and the density of the soil, due to crushing of the grains. Hence, pile installation affects the ϕ' value significantly. The effect of grain crushing is smaller for finer particles than for coarse grained ones. Again, rounded particles are less affected than angular ones.

The installation process orients the soil particles along the shaft with their long axis parallel to the shaft. On the basis of published literature, Kraft (1991) concluded that fabric changes along the shaft may result in a reduction of upto 5° in the soil friction angle. Near the pile toe, changes in fabric may occur due to local shear failure. The effect of fabric changes, may be upto 2° in friction angle (Miura et al. 1984)

Soil-pile friction angle - The interface between the soil and pile plays an important role in determining the shaft capacity and in turn, the axial capacity of a pile (Potyondy 1961). Failure of granular soils generally occur at the pile soil interface, unless the pile is very rough, in which case, the soil may fail within itself, a short distance away from the interface.

The pile soil friction angle has been evaluated by a large number of researchers (Potyondy 1961; Yoshimi and Kishida 1981; Datta *et al.* 1980). On the basis of a large number of data Kraft (1991) has concluded that the ratio of pile soil friction angle, δ, to the angle of internal friction of the soil, ϕ', can be assumed for steel piles to be 0.7 for siliceous sand and 0.6 for calcareous sand. Potyondy (1961) has reported a wide variation in the ratio of δ to ϕ', in dense sand. It varies from 0.543 for smooth piles to 0.99 for rough concrete.

2.6.2 Pile Parameters

Different types of piles are used depending on the specific usage, available technology and economic constraints of a project. Apart from pile material variation, there can be variations in cross-sectional area and length. There are four basic materials that are used for making piles.

Timber - Timber piles are prepared by trimming the tree trunks. ASCE manual of practice no. 17 (1959) categorizes timber piles into three major categories as A, B and C depending on their strength and other characteristics. Some of the timber suitable for making piles are Douglas fir, pine, spruce, larch etc. in soft woods and teak, beech, oak, greenheart etc. in hard woods. Timber piles were extensively used in pile work in previous centuries and early part of this century.

Steel - Today steel piles are preferred over timber ones. A steel pile compared to timber is much more strong and does not undergo any decay. Rolled H - sections and welded or seamless pipes are the commonly used steel piles. Wide flange steel sections and I - section piles are also usually used. Steel piles with H - section are useful on sites where driven pile are required but the soil displacement has to be kept low.

Concrete - Nowadays, for heavy structures and offshore structures, where large diameter piles are required, concrete piles are preferred due to their low cost and durability. Moreover, large diameter piles can be cast in situ. Concrete piles can be used cast in situ or precast or prestressed.

Composite piles - Composite piles are made of combinations of concrete and steel or concrete and timber. However, these piles have an inherent problem. The problem is of effecting a proper joint between the two materials. Hence, these piles are not so common.

The shape of the various types of piles used can be cylindrical, tapered, under-reamed or corrugated depending on soil conditions, material of construction and method of pile installation available.

The cross-sectional area of piles may be cylindrical, square, hexagonal, octagonal, I-section or H-section. The shape of the piles again depends upon, whether high displacement or low displacement piles are required. For example, a driven I-section or H-section pile is generally low displacement compared to a closed ended cylindrical pile.

Some of the major pile parameters that affect the performance of piles in soil are :

Pile compressibility - The relative soil-pile compressibility is a function of the diameter, length, wall thickness (in case of pipe piles) of the pile and the stiffness modulus of the soil. Hanna and Tan (1973) and Tan and Hanna (1974) reported a decrease in shaft resistance of model piles when the pile compressibility increased by six times. However, Tan and Hanna (1974) mentioned that this difference in capacity is due to the difference in the residual load locked in the pile during installation. This difference in the residual load is due to difference in pile compressibility. A more compressible pile has more residual stresses locked due to driving, as compared to a rigid pile.

If the soil shows deformation softening or strain softening behaviour (ie. the load transfer decreases with an increase in pile movement) the ultimate pile capacity gets affected by the relative pile-soil compressibility (Kraft 1991). However, most reported load transfer data do not show the occurrence of deformation softening in sands (Hanna and Tan 1973; Tavenas 1971; Beringen *et al.* 1979)

Pile Diameter - Meyerhof (1983) concluded that the ultimate shaft resistance of piles is practically independent of pile diameter. Hettler (1982) reported that the shaft resistance of piles embedded in sand, subjected to tensile loads decreased with an increase in the pile diameter. The magnitude of this decrease lessened for loose soil. It was also reported that for loose soils, the unit shaft resistance was almost independent of the pile diameter. The unit tip resistance is also independent of pile diameter. However, the total tip resistance is proportional to the square of the diameter.

Pile Displacement - The shaft resistance of a closed end pile (high displacement) is higher than that of an open end pile, embedded in dense sands. The ratio of the shaft resistance of a closed end pile to that of an open end pile can be in region of 1.5 (Beringen *et al.* 1979). Vesic (1964) reported that the shaft resistance of a driven full displacement pile can be about 1.4 times that of an almost zero displacement pile (for eg. buried pile). Low displacement piles cause lesser disturbance during installation, whereas, a high displacement pile may tend to densify the soil around the pile and hence, increase the lateral stress all along the pile.

Pile Length - The pile tip capacity increases linearly with the pile length upto a penetration of 10-30 pile diameters. The unit resistance does not increase continually as implied by classical theory (Neely 1990). However, various researchers reported different values of length to diameter ratios for various limiting tip resistance (Kerisel 1961; Vesic 1963;1964).

The unit shaft resistance also increases with depth, but the increase is more of a parabolic nature than triangular. Meyerhof (1976) suggests that for design purposes this increase can be taken upto 15 to 20 times the pile diameter, beyond which the pile capacity may be assumed to be constant. Hanna and Tan (1973) and Tan and Hanna (1974) found for model piles in loose sand the average unit shaft resistance increased, almost linearly upto 40 pile diameters and then remained constant. It was also reported that the shaft resistance increased more rapidly near the surface than at a larger depth. Das and Seely (1975), Vesic (1963) and Vesic (1964) have also reported similar trends. Hence, for homogeneous soils, the unit shaft and toe resistance of a pile can be expected to increase with depth though at a decreasing rate. The decrease in rate of increase of shaft resistance with depth is attributed to the arching action of dense sands (Vesic 1967) as result of increasing overburden pressure. Similar explanation for toe resistance can also be advanced. With an increase in the tendency of soil to shrink, it becomes more compressible. As the soil compressibility increases the rate of increase of the toe resistance decreases (Fleming *et al.* 1985; Vesic 1977). Some of the other reasons for the downward curvature of the stress may be due to the decrease in the soil and the soil-pile friction angles with depth as a result of an increase in effective overburden pressure and an increase in the lateral soil stresses from at rest condition due to displacement caused by pile installation (Kraft 1991).

2.6.3 Installation Method

Pile installation plays a significant role in determining the ultimate capacity of the pile. For example, a driven pile may show a higher shaft and tip capacity than a bored pile of the same diameter, length and material. Piles, on the basis of the method of installation may be classified into bored, driven, jacked, vibrated, jetted or tremie piles. Broadly, there are two classes of piles.

Displacement piles - These are piles whose installation cause significant soil displacement both laterally and vertically. Generally, displacement piles are driven into the soil. There can be two types of displacement piles.

a) those in which a solid or hollow closed ended pile is driven into the ground, and

b) those in which a pile like body is driven and then withdrawn, leaving a void. This void is consequently filled with slurry (Whitaker 1976).

In either method the soil around the pile is displaced, unlike in non-displacement piles where the soil is taken out. These piles are prefabricated steel pipe piles or steel and concrete shell piles. Steel I-section or H-section piles are used where the disturbance of soil has to be kept to a minimum, as in sensitive clays.

The effect of driving is more on clayey soils than on sands. De Mello (1969) classifies the effect of pile driving in clayey soils into 4 major categories :-

a) Remoulding or partial structural alteration of the soil surrounding the soil.

b) Alteration of the stress state in the soil in the immediate vicinity of the pile. In clays, a zone of soil extending to about two diameters away from the pile shaft gets remoulded.

c) The generation and subsequent dissipation of excess pore water pressure around the pile.

d) The immediate loss of shear strength of the soil. The phenomenon of strength regain in the soil is a long term one. Subsequently, however, the strength is found to increase due to thixotropic regain and an increase resulting from local consolidation of the soil due to dissipation of excess pore water pressure.

Due to pile driving or forced pushing by other means, the soil around the pile is compacted by displacement resulting in permanent rearrangement and some crushing of the particles. In loose soils, this results in an increase in the relative density of the soil. Robinsky and Morrison (1967) conducted model tests to study the zone of effect in sands. It was concluded that in very loose sand, the soil movement extends upto 3 to 4 pile diameters from the side of the pile and 2.5 to 3.5 diameters below the tip. In medium dense sands the respective zones of influence are 4.5 to 5.5 pile diameters from the shaft and 3 to 4.5 pile diameters below the pile tip. It was also observed that compaction below a pile tip is followed by sand movements adjacent to the pile shaft. These movements decrease the sand density at the sides. Hence, in loose and medium dense sands, due to compaction the tip resistance increases.

So below the tip the density of the sand increases and hence the friction angle also increases. Kishida (1967) observed the effect of driving to be about 7D around the pile, where, D is the pile diameter. Within this zone, he assumed the angle of internal friction ϕ' to increase linearly from the insitu value ϕ'_1 to ϕ'_2. The relationship between ϕ'_1 and ϕ'_2 is as such

$$\phi'_2 = \frac{\phi'_1 + 40}{2} \qquad (2.17)$$

Non-Displacement piles - Non-displacement or low displacement piles are formed by removing the soil and then putting the pile in place. Bored piles are a type of non-displacement piles. Non-displacement piles are formed in two ways:

a) Either an open - ended tube is forced into the ground until it reaches the bearing stratum. Soil may enter the tube which is removed and the tube filled with concrete, or

b) a bore hole is formed in the soil and temporary casing put in it. The slurry concrete is poured and then the casing is withdrawn.

These types of piles possess the advantages of prefabricated displacement pile. These piles also cause very limited compression or displacement of the soil.

In clays, the adhesion of clay particles is found to be less than the undrained cohesion before installation. This may be attributed to softening of clay around the pile due to following reasons (Poulos and Davis 1980):

a) Absorption of moisture from the wet concrete

b) The movement of water from the clay body to around the borehole and

c) Water poured into the borehole for ease of cutting.

Meyerhof and Murdock (1953) observed that the water content at the shaft of a bored pile as 4 % more than that in the body of the clay. This test was conducted on London Clay. On the same type of clay, Skempton (1959) presented relationships which show an increase in water content by 1 % cause a 20 % reduction in the ratio of C_a to C_u, where C_a is the adhesion and C_u is the undrained cohesion. The ratio reduces by about 70 % if the water content increases by 4 %. In granular soils some loosening is liable to occur below a pile as a result of baling. Tomlinson (1975) suggest that value of angle of internal friction ϕ', to be used in cases of design of bored piles in sand. In sands Clemence and Brumund (1975) observed that 20 to

30 % of the design axial load in end bearing drilled piers was carried by the pile skin. Touma and Reese (1974) concluded that the skin frictional capacity of a bored pile can be about 70 % of that of a driven pile.

2.6.4 Load Parameters

The load parameters that may affect the pile load displacement behaviour are:-

Loading History - The loading history of a pile typically means whether or not a pile has been subjected to any kind of loading accidental or intentional before the pile has actually been loaded. Such kind of loading are frequent in fields where lot of other parameters like movement of machinery, malfunctioning of driving apparatus etc. may sometimes result in accidental loading. One type of the common incidental loading occurs in group piles. Where pile groups are required, single piles are driven one by one in predetermined positions. Since the piles are driven in close proximity of each other with a typical spacing of 3 to 5 times the pile diameter, the driving of the second and subsequent piles exert an upward force on the first pile. Similarly, all piles driven subsequently affect the previously driven piles. In most cases, visible uplift of piles result and generally, the pile driver goes back and redrives all the other piles. It has been concluded by many researchers (Chan and Hanna 1980; Gudehas and Hettler 1981) that soil undergoes significant degradation when subjected to prior loading. Most research has been conducted with respect to cyclic loading, but the loading history can be considered synonymous to single cycle loading. In saturated soils, this loss of strength occurs primarily due to generation of pore water pressure at the pile soil interface. In dry sands the soil strength degradation occurs due to destruction of interparticle bonds and the realignment of soil structure parallel to the direction of shear strain and change of mean effective stress at the pile soil interface.

Koreck and Schwarz (1988) have conducted some tests and reported significant decrease in the pile ultimate capacity as a result of prior loading. This decrease was in both tensile and compressive capacity of the pile, when subjected to prior compressive and tensile loading respectively.

Type of load test performed - Pile load tests are conducted to ensure that failure does not occur before a particular load is achieved. It is also conducted to determine the ultimate load, which when reduced by a factor will yield the working load, and to determine the load settlement behaviour of a pile. Some of the types of load tests performed for the evaluation of the compression capacity of a pile are:

Maintained load test: In the maintained load test, there are usually two methods to apply loads in stages. The precise loading and unloading to be followed is specified by various building codes.

The standard loading procedure or the slow maintained load test specified in ASTM D1143 (1990) specifies that "unless failure occurs first, load the pile to 200 % of the anticipated pile design load for tests on individual piles or to 150 % of the group design load for tests on pile groups, applying the load in increments of 25 %

of the individual pile or group design load. Maintain each load increment until the rate of settlement is not greater than 0.01 inch (0.25 mm) /hr but not longer than 2h. Provided that the test pile or pile group has not failed, remove the total test load anytime after 12 h if the butt settlement over a one hour period is not greater than 0.01 inch (25mm); otherwise allow the total load to remain on the pile or the pile group for 24 hours. After the required holding time, remove the test load in decrements of 25 % of the total test load with 1 hour between decrements".

Another method specified by the ASTM D1143 (1990) is to "apply the load increments of 10 to 15 % of the proposed design load with a constant time interval between increments of 2½ mins or otherwise specified. Add load increments until continuous jacking is required to maintain the test load or until the specified capacity of the loading apparatus is reached, whichever occurs first, at which stop jacking. After a 5 min interval or as otherwise specified, remove the full load from the pile." This method is known as the quick maintained load test.

Cyclic loading : The third important method for application of loads is the cyclic loading. The application of loads in cyclic loads is the same as that for slow maintained load tests. After the application of loads equal to 50, 100 and 150 % of the pile design load, each load has to be maintained for 1 hr. Each load has to be removed in decrements equal to loading increments, allowing for 20 min between decrements. After removing each total applied load, reapply the load to each preceding load level in increments equal to 50 % of the design load, allowing 20 min between increments.

Constant rate of penetration : The constant rate of penetration test is another method specified by ASTM for the determination of compressive capacity of piles. In this method, the applied load needs to be varied as necessary to maintain a pile penetration rate between 0.25 mm/min to 2.5 mm/min depending on the soil. The pile penetration is to be continued "until no further increase in the load is necessary for continuous pile penetration at the specified rate unless the specified capacity of the loading apparatus is reached".

Apart from these the ASTM D1143 (1990) specifies a number of other methods to obtain the axial compressive capacity like the loading in excess of standard test method, constant time interval loading and the constant settlement increment loading method for individual piles.

Uplift capacity of piles : The ASTM D3689 (1990) specifies a few methods to obtain the uplift capacity of piles.

Some of the important methods are the standard loading procedure or the slow maintained load test, the cyclic loading test, the constant rate of uplift method, the constant time interval loading and the quick load test method for individual piles.

The slow maintained load test, the cyclic loading test, the quick load test methods for individual piles for uplift capacity of piles are the same as for the compressive capacity of piles.

The constant rate of uplift of piles to be used varies between 0.5 to 1.0 mm/min.

Tests were conducted by Sparrow (1988) to compare the pile failure capacity obtained by these three methods. It was concluded that the maintained load tests being more time dependent than the constant rate of penetration test, result in additional settlement occurring with time at a particular load. Hence, at the early settlement stages the constant rate of penetration test show a higher load than the maintained load test. Between the two types of maintained load tests, the quick maintained load test gave a higher load than the slow maintained load test. However, all three plots merged after a certain displacement and hence the failure load recorded was the same.

The failure curves were not taken to displacements required for mobilization of full shaft friction (Vesic 1964), hence, conclusive evidence of how the shaft resistance vary with the type of test was not known. It was also observed that a greater percentage of the total load is in end bearing in the constant rate of penetration method.

Cyclic or Repeated Loading : When a soil sample is subjected to cyclic or repeated loading, there is a net change in the state of stress at the end of the cycle as compared to the beginning. It was observed (Bishop and Henkel 1953) that upon a cycle of undrained loading and unloading on a saturated soil sample, the pore water pressure does not return to its original value.

Gregersen et al. (1973) concluded that when a pile is unloaded after loading then there is a change in the residual loads in the pile. Therefore, if subsequent repetitions of loading on the pile result in a progressive change in the residual load, it can be expected that either the residual load will reach a limiting value and thus the pile will approach an equilibrium state or a change in pile behaviour will be brought about (Chan and Hanna 1980). The main factors that affect the behaviour of piles subjected to repeated loads are, number of repetitions, the load applied as a percentage of the pile ultimate load, amplitude of load, frequency of loading, depth of embedment and soil characteristics.

Chan and Hanna (1980) after conducting a series of experiments concluded that repeated loading brings about significant changes in both the rate of movement and pile loads in the shaft. It was reasoned that due to cyclic loading the normal stress at the pile shaft decreases due to soil degradation. The degradation of soil resistance due to this loading occurs due to an increase in induced pore water pressure, destruction of interparticle bonds, realignment of particles (Kraft 1990; Krosch and Reese 1980; Chan and Hanna 1980). Cyclic loading results in minor adjustments of the soil particles every time the load is cycled.

Broms (1972) has however, concluded that below a certain critical load, piles can resist an infinite number of load cycles without failure. Puech *et al.* (1982) showed that even though large deformations may occur under cyclic loading, pile loading to failure subsequent to cyclic loading will yield pile capacities close to the initial pile capacity. This is due to the dilative response of the soil that results in an increase in the lateral effective stress on the pile.

Surcharge load on sand around the pile : The effect of surcharge loading on the soil around a pile is similar to application of an overburden pressure in the field.

The lateral pressure on the pile due to a surcharge load, q, on the top may be $K(\gamma' z + q)\tan\delta$, varying with depth upto a certain level.

Analytically, this will give the total skin friction as,

$$F_s = (2\pi r)\int_0^L K(\sigma'_v + q)\tan\delta\, dz \ldots\ldots\ldots\ldots (2.18)$$

With an increase in the length of a pile the average unit shaft resistance increases then becomes constant beyond a critical depth, L_{cr} (Vesic 1964). Assuming this to be true in this case,

for a pile $L > L_{cr}$,

$$F_s = (2\pi r)\int_0^{L_{cr}} K(\gamma' z + q)\tan\delta\, dz + (2\pi r)\int_{L_{cr}}^{L} K(\gamma' L_{cr} + q)\tan\delta\, dz \ldots\ldots (2.19)$$

upon integrating and simplifying,

$$F_s = (2\pi r)\,[K\gamma'\frac{L_{cr}^2}{2} + qLK + K\gamma' L_{cr}(L - L_{cr})]\tan\delta \ldots\ldots\ldots\ldots (2.20)$$

For a pile $L < L_{cr}$,

$$F_s = (\frac{\pi r K L^2 \gamma'}{2} + 2\pi r q L)\tan\delta \ldots\ldots\ldots\ldots (2.21)$$

The tip resistance should be given by the equation

$$Q_p = A_p(\gamma' z + q)\, N_q^* \quad (2.22)$$

But, this equation cannot be said to be applicable for estimation of the pile tip capacity as Vesic (1969) has shown that punching action occurs beneath a loaded pile and arching occurs around a pile and above the tip, which may prevent the full effect of surcharge load along the lower lengths of the pile. The decrease in the internal friction of the soil with depth will also decrease the pile capacity (Randolph 1985).

2.7 Model Tests

Model tests, in any inexact science, is a common phenomena, where experimentally determined curves and values are obtained and used to develop theories as well as empirical correlations and to check and modify already existing theories. Model tests can be of two types (Whitaker 1957).

Model tests may be conducted in the field or in the laboratory. In field model tests the pile is usually one fifth to one twentieth of full size. The pile testing is done depending on the funds available and the size of the bed of soil. Laboratory model tests are conducted in artificially made beds under laboratory controlled conditions. Here all the parameters can be controlled and the pile behaviour can be studied more accurately. One of the major limitations of laboratory tests is the lack of simulation of the field conditions.

Laboratory tests are conducted in test tanks where the chamber size itself may have an effect on the pile performance. There is a lot of debate as to what the ratio of chamber size to pile diameter should be in order to avoid significant effect on the pile performance. Schmertmann (1976) has reported that up to a ratio of 34, a significant effect of chamber exist, whereas, Parkin and Lunne (1982), for loose sands, suggest diameter ratios of 21.4 to be sufficient and for very dense sands, diameter ratios of 50 may be required to avoid side effects. Meyerhof (1959) mentions that when the chamber diameter is ten times that of the pile diameter, most of the effect on pile performance dessipates and this ratio can be considered for designing laboratory test tanks. However, later work, since the early eighties, the problem of the influence of finite dimensions of the chamber has been critically analysed (Parkin and Lunne 1982; Jamiolkowski *et al.* 1985; Belloti *et al.* 1985; Schnaid and Houlsby 1990) without reaching any definitive conclusion. Baldi *et al.* (1982) in order to quantitatively account for these boundary effects, developed correction factors to be applied to the measured cone resistance. For a chamber size 34 times the pile diameter the correction factors for a normally consolidated sand ranged from 1.0 for medium dense sands to 1.18 for dense sands. However, it was later felt that the correction procedures stated by Baldi *et al.* (1982) were very approximate and arbitrary (Sparrow 1988). Actually calibration chambers designed nowadays pertain to either of four categories (Table 2.1). In nature, a mixture of these types are found to occur. The natural soil behaves in a manner which is neither constant stress nor constant volume but something in between.

Another parameter in laboratory tests that is of interest is the pile dimension itself. The pile dimensions should be such that it is not affected by sand grains

TABLE 2.1

BOUNDARY CONDITIONS IN A CALIBRATION CHAMBER

Type	Radial	Radial	Vertical	Vertical	
	$\Delta\sigma_R$	$\Delta\epsilon_R$	$\Delta\sigma_v$	$\Delta\epsilon_v$	
B1	0		0		constant stress
B4	0			0	
B3		0	0		
B2		0		0	constant volume

$\Delta\sigma_R$ - Radial stress σ_v - Vertical stress

$\Delta\epsilon_v$ - Vertical strain ϵ_R - Radial strain

locally and that it should yield quantitative data relevant to the field. Vesic (1964) suggests that a pile diameter of 38 mm (1.5 inches) should be considered as an absolute minimum, in laboratory tests to have results of any significance.

One of the basic problems with laboratory tests is the simulation of field soil conditions. The soil in the field is generally heterogeneous not only in deposition but also locally due to the presence of pockets of stiffer or softer soils and due to the water table. Laboratory tests are performed on specimens of freshly reconstituted sands whose fabric is very much different from that of natural soil deposits. Natural soil deposits have a highly developed structure, by the phenomena of creep, cementation etc.(Ghionna and Jamiolkowski 1991; Dusseault and Morgenstern 1979; Mesri 1987; Mitchell and Solymar 1984). These effects are primarily due to aging and it is difficult to quantify the influence of these on pile performance in soil. Laboratory tests are generally conducted on clean, uniform sands whereas, natural deposits are neither clean nor uniform. Almost invariably, all natural sand deposits contain some fines which may significantly affect results.

Hence, before building any theory full scale tests may be necessary. Laboratory tests are, however, good to understand pile behaviour of homogeneous soil deposits at a relatively low expense.

Model tests have been conducted by various investigators (Vesic 1963,1964; Kerisel 1961,1964; Sparrow 1988) to study pile performance in controlled conditions. Vesic (1963, 1964) conducted tests in a cylindrical pit of 2.5 m diameter and 6.7 m deep. The water table in the pit was varied by a sump and a 200 ton capacity reaction frame permitted vertical and horizontal loading of model piles. Cylindrical piles of 54 mm, 102 mm, 171 mm diameter were used.

Hanna and Tan (1973) performed model tests in a square chamber of length 0.61 m and a height of 2.44 m. Piles of diameter ranging from 15.9 mm to 28.1 mm were suspended in the empty container and the sand bed was poured around the pile. A lever system allowed the top of the pile to be held rigid or to float during sand preparation.

Model tests on piles were conducted by Kerisel (1961, 1964) in a circular sand bed 6.4 m in diameter and 10.2 m in depth. Steel piles of diameter ranging from 40 mm to 320 mm were used.

Robinsky and Morrison (1964) performed model tests in sand to study the displacement and compaction around driven piles. Tests were conducted in uniformly deposited sand of relative density 17 % and 37 %, in a chamber of size 0.502 m × 0.711 m in cross-section and 0.813 m high. Three instrumented piles were tested, two of constant cross-section and one was tapered.

One of the basic problems encountered while testing piles in sand is to obtain sand deposits of uniform density. Rad and Tumay (1987) suggested the method of pluviation by which a sand deposit of uniform density can be obtained.

Sparrow (1988) conducted model tests in a test tank of diameter 0.9 m and length 2.3 m. A sand bed of uniform density was prepared by pluviation. Closed ended steel pipe piles of diameter 50.8 mm were driven into the sand by a hammer falling from a height of 0.3 m.

2.8 Evaluation of Pile Ultimate Load from Load Settlement Plots

Load - Settlement plots obtained from pile load tests are smooth curves that have an initial linear portion and a final non-linear portion. As the curve generally has a smooth transition, it becomes difficult to define a particular point at which failure can be said to have occurred. Many methods have been suggested for "failure" of the pile soil system. Some of the methods are based on actual inspection of the load settlement curves, whereas, others are based on idealization of the actual load settlement plot as a hyperbola or any other mathematical model. Due to the varied nature of these analyses and their inherent assumptions it is difficult to determine the true failure load. In cases of plunging failure most of these methods give the same value of failure load, whereas, the results may vary within wide limits for piles which continually settle with an application of the load. From literature some thirteen methods have been identified and considered in this study.

2.8.1 Single Tangent Method

In this method the load versus settlement curve is drawn. The straight line portion at the end of the curve which indicates the plastic range of failure of the soil, is identified. This straight line portion is extended backwards to meet the load axis, as shown in Figure 2.5, to give the ultimate load, Q_{max}.

This method can be used for all types of tests.

2.8.2 Double Tangent Method

Two tangents to the load settlement curve, one at the initial portion and the other at the final portion of the curve are drawn. The load corresponding to the intersection of the two tangents (Figure 2.6) is the ultimate load.

This method can be used for all types of tests.

2.8.3 Van der Veen's Method

Van der Veen (1953) suggested that the shape of the load settlement is scale dependent and therefore an analysis on the basis of inspection of the load settlement curve may give erroneous results. According to him, the load settlement data may be represented in a form which can be expressed as:

$$Q = Q_{max}(1 - e^{-aS}) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (2.23)$$

where, Q is the load on the pile head

Q_{max} is the ultimate load

S is the gross settlement of pile under load Q

a is a coefficient which influences the shape of load settlement curve

Equation 2.23 can be rewritten as:

$$S = -\frac{1}{a}\ln(1 - Q/Q_{max}) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (2.24)$$

A plot of S against ln (1 - Q/Q_{max}), shows that the curve consists of two straight lines as shown in Figure 2.7. Therefore, up to a certain load, the coefficient defining the slope of the straight line has a value a_1 and later changes to a value a_2. It has been suggested (Chakraborty 1989) that this change may be due to the exceedence of precompression to which the soil has already been subjected to, before the load test was performed.

It is an iterative procedure and plots of S against ln (1 - Q/Q_{max}) have to be obtained for each value of Q_{max}. The Q_{max} value for which the curve appears to consist of one- or two- straight line(s), is considered as limit load. In this way, this method seems to avoid any kind of personal bias in determining the ultimate load.

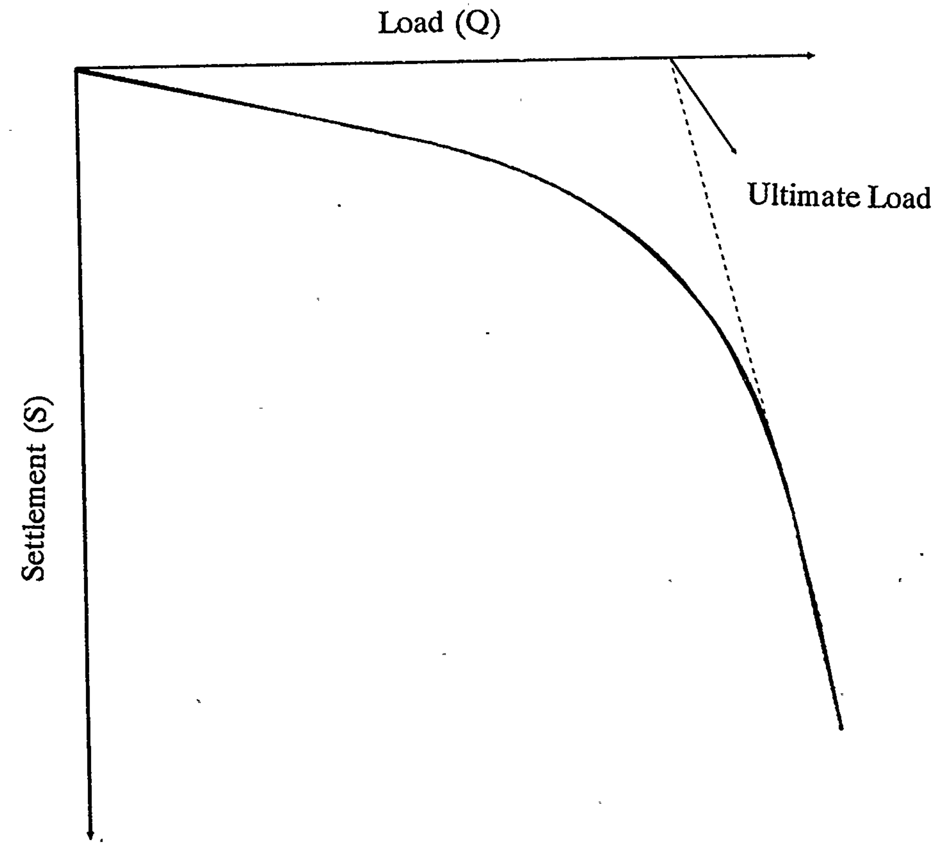


Figure 2.5 Single Tangent Method for Ultimate Load of Piles

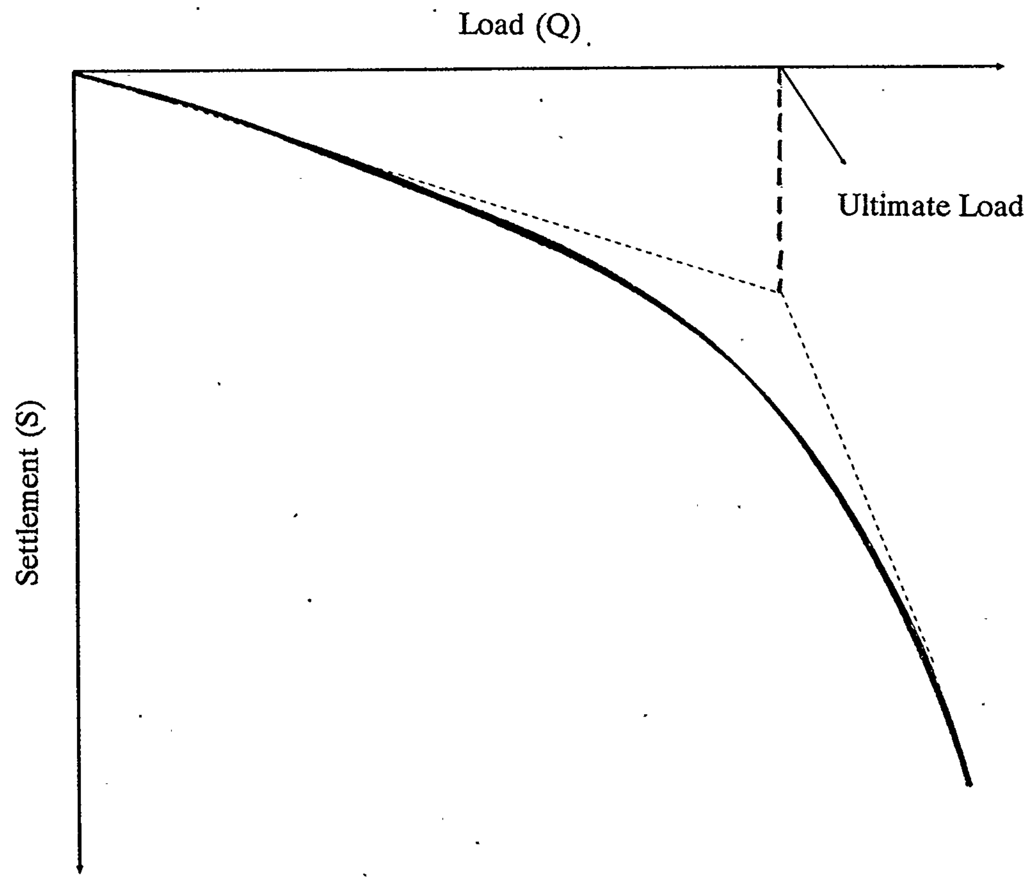


Figure 2.6 Double tangent Method for Ultimate Load of Piles

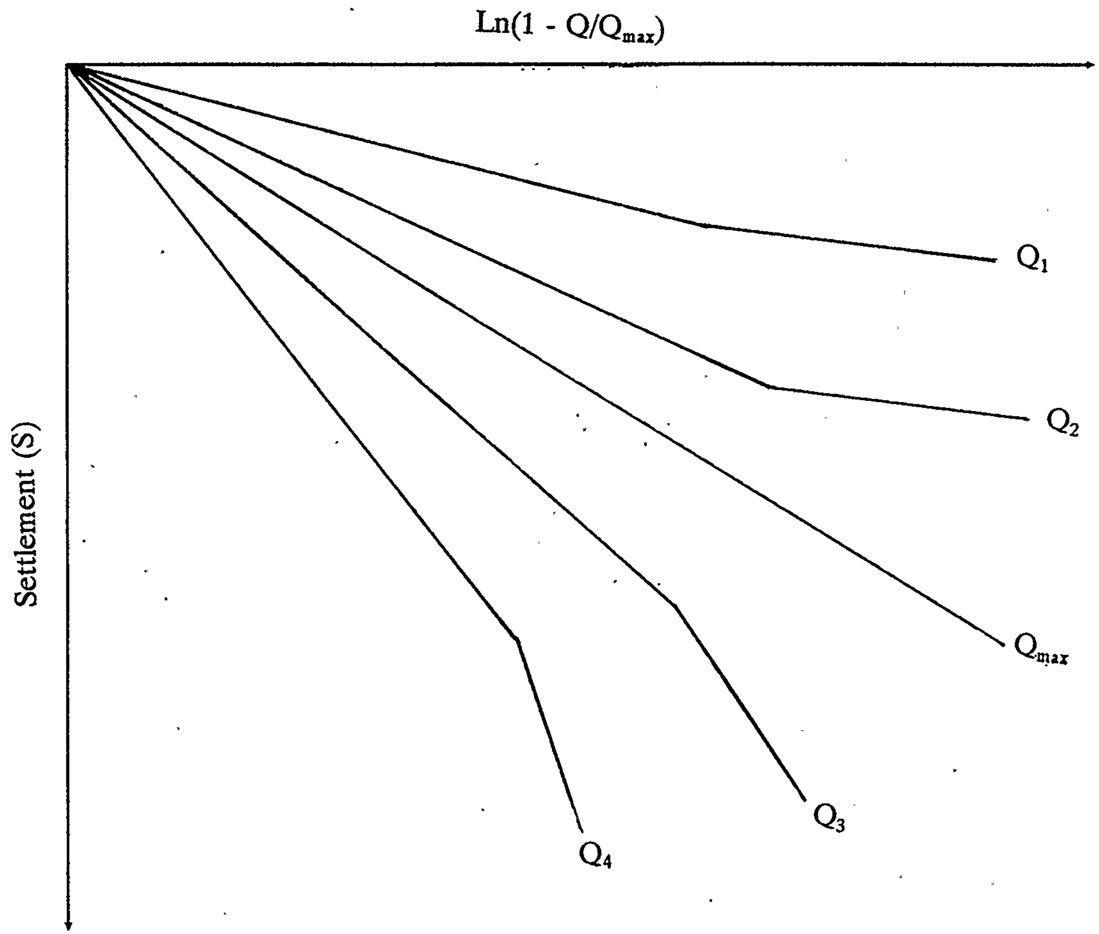


Figure 2.7 Van der Veen's Method for Ultimate Load of Piles

This method can be used for all types of tests.

2.8.4 De Beer's method

De Beer (1967) suggested a method for determining the ultimate load for piles tested in slow maintained load test. He suggested that the load settlement curve be drawn on a double logarithmic scale. The data point in such a plot fall on two straight lines. The load corresponding to the intersection of these two straight lines indicate the ultimate load (Figure 2.8)

2.8.5 Brinch-Hansen's 80 % criteria

In this method suggested by Brinch-Hansen (1963), a curve for plots of settlement against square root of settlement over load is drawn. A straight line fit is made for points lying in the later portion of the curve. If C_1 denotes the slope of the straight line and C_2 the intercept of the ordinate $\sqrt{S}/Q$, the ultimate load of the pile and the corresponding limiting settlement, S_o can be calculated from the following equations:

$$Q_{max}=\frac{1}{2\sqrt{C_1C_2}} \quad \text{(2.25)}$$

$$S_o=\frac{C_2}{C_1} \quad \text{(2.26)}$$

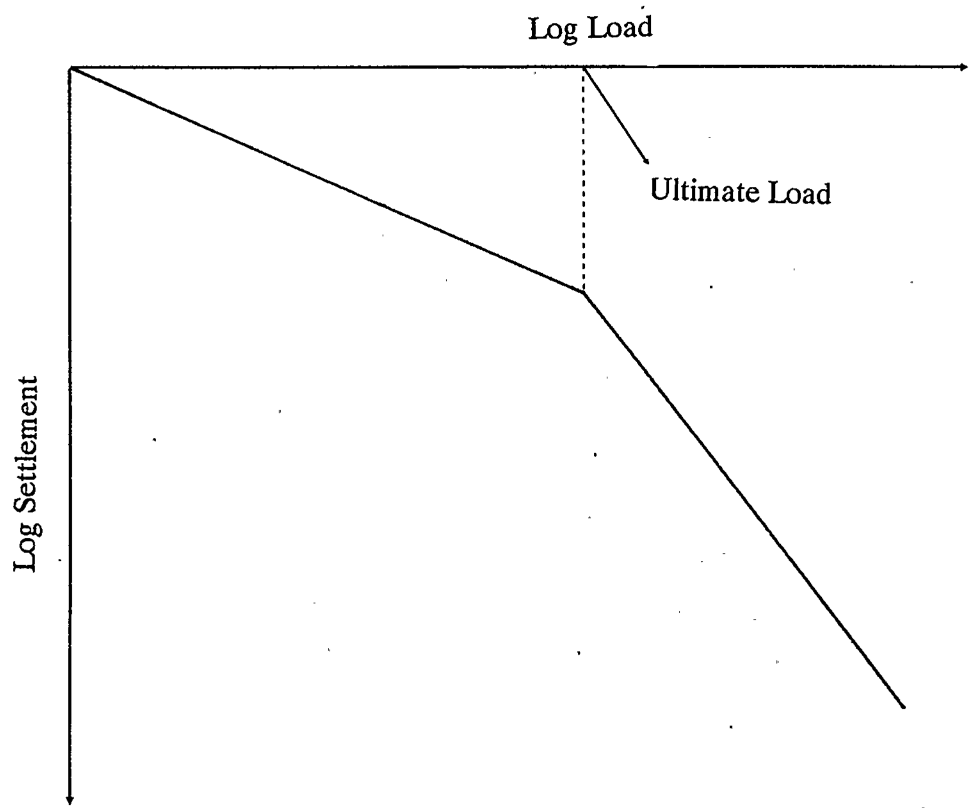


Figure 2.8 De Beer's Method for Ultimate Load of Piles

Figure 2.9 gives a graphical description of the above.

This method has not been suggested for any particular method of testing and is generally used for all types of loading.

2.8.6 Brinch Hansen's 90 % Criteria

Brinch Hansen (1963) suggested this method for tests conducted under constant rate of penetration of the pile. It is suggested that the failure load, Q_{max} is that load which gives twice the value of the settlement of the pile head as compared to the value of settlement obtained for 90 % of the load Q_{max} on the pile head (Figure 2.10). In this method, the load Q_{max} and corresponding settlement are obtained from the load settlement curve by trial and error.

This method has been suggested for the constant rate of penetration test.

2.8.7 Fuller and Hoy's Method

In this method suggested by Fuller and Hoy (1970), a straight line having a slope of 0.129 mm/kN and tangential to the load settement curve is drawn (Figure 2.11). The load corresponding to the tangent point is the ultimate load. This method was suggested and is generally used for piles tested by quick maintained load test.

2.8.8 Butler and Hoy's Method

This method suggested by Butler and Hoy (1977) is very similar to the Fuller and Hoy's method. The basic difference between this and the Fuller and Hoy's method is that, in this the initial linear portion of the load settlement curve is

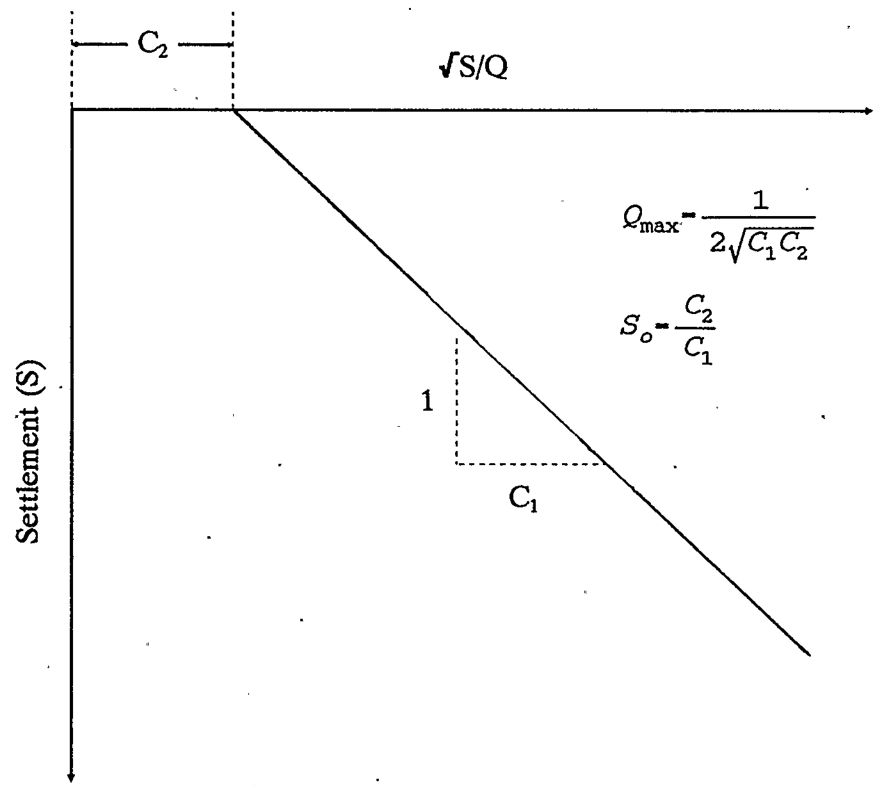


Figure 2.9 Brinch - Hansen's 80 % Criteria of Piles

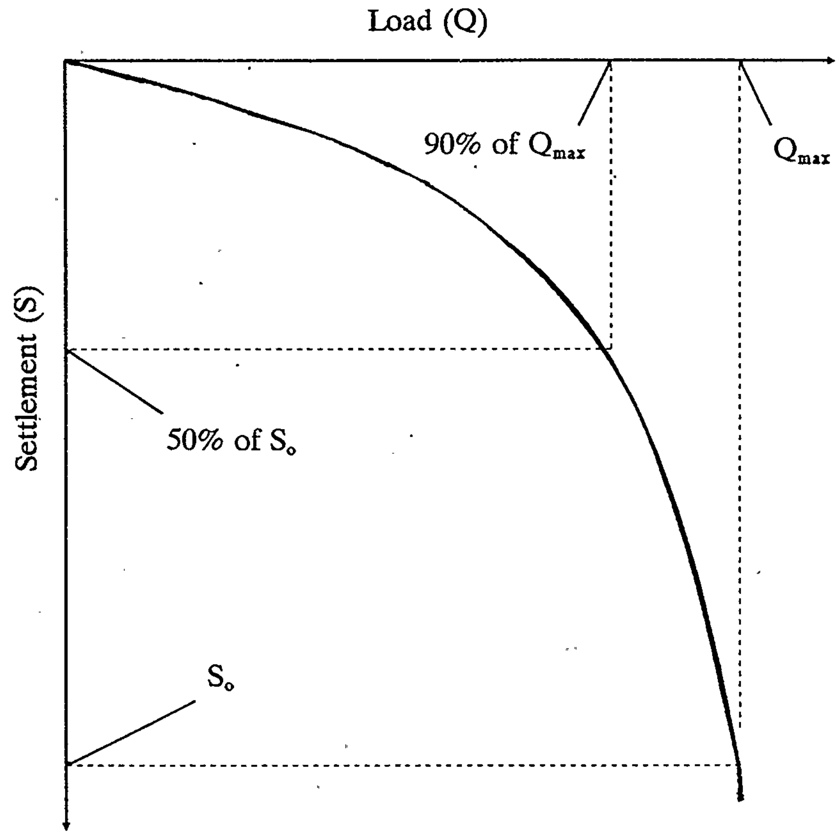


Figure 2.10 Brinch - Hansen's 90 % Criteria of Piles

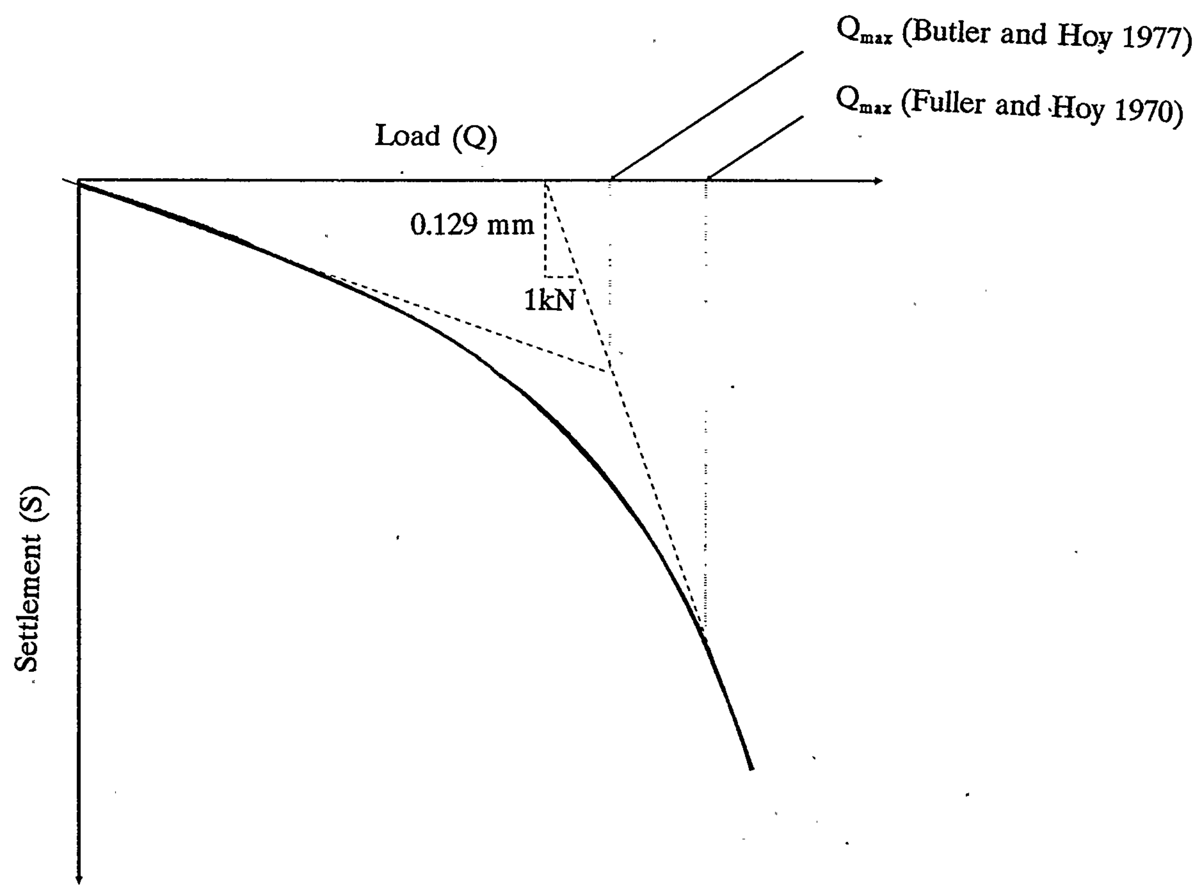


Figure 2.11 Fuller and Hoy's & Butler and Hoy's Method for Ultimate Load of Piles

extended to meet the straight line having a slope of 0.129 mm/kN and tangential to the load settlement curve. The load corresponding to the point of intersection of the two straight lines, is the ultimate load of the pile (Figure 2.11). This method was suggested for the determination of ultimate load for piles tested under quick maintained load tests.

2.8.9 Davisson's Method

This method, suggested by Davisson (1973), unlike other methods considers the effect due to the length of the piles. The ultimate load is defined as the load corresponding to the gross settlement, which exceeds the elastic compression of pile when considered as a free column, by a value of 4.0 mm plus a factor depending on the diameter of the pile. The following procedure may be used to determine the ultimate load on the pile.

The elastic compression, S_e, of the pile is computed as such:

$$S_e = \frac{QL}{AY} \qquad (2.27)$$

where, Q is the load on the pile head

L is the length of pile

A is the cross-sectional area of pile

Y is the modulus of elasticity of the pile material

Different values of S_e, thus obtained, from Equation 2.27 are plotted (Figure 2.12) on the load settlement curve as a straight line A-A$'$ with an intercept on the

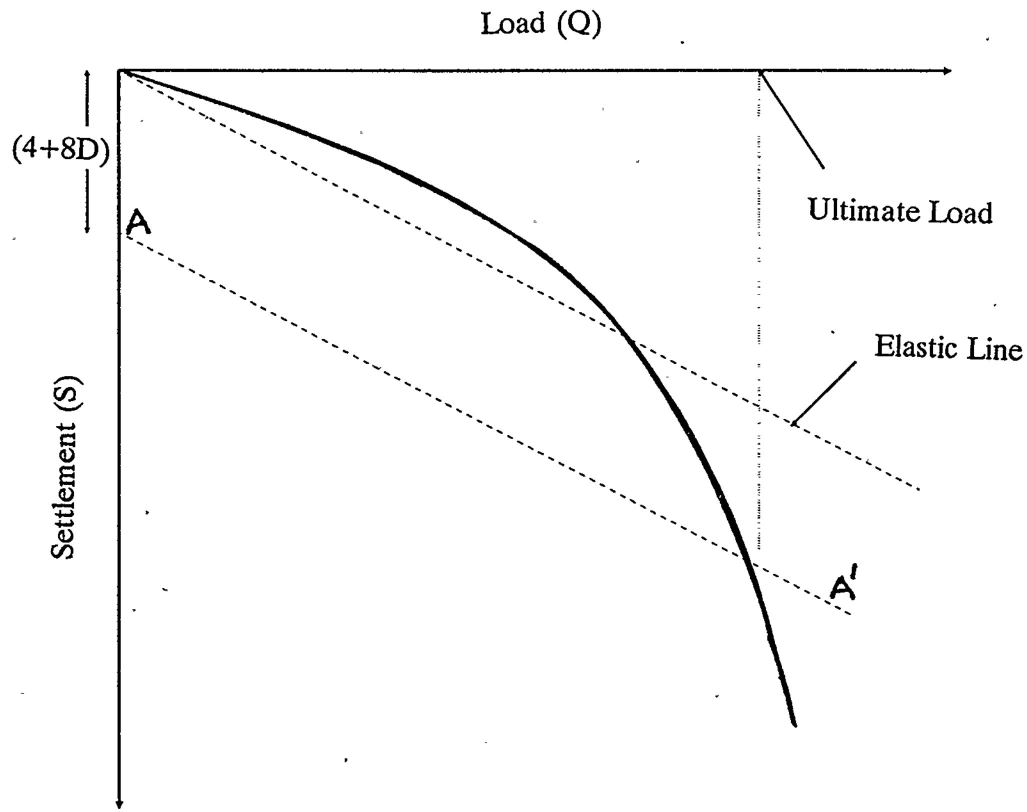


Figure 2.12 Davisson's Method for Ultimate load of Piles

settlement axis equal to (4 + 8D) mm, where D is the diameter of the pile in meters. This settlement intercept is an estimate of the tip movement required to mobilize the tip resistance of the pile. The load corresponding to the point of intersection of the line A-A′ with the load settlement curve is regarded as the failure load on the pile.

This method was suggested for quick maintained load tests.

2.8.10 Carroll's Method

This method, suggested by Carroll (1987) uses the concept of creep behaviour of soil for determining the ultimate load on the pile. Creep, is the continuing deformation of a material under a particular sustained load. This method can be used only for piles tested under slow and quick maintained load tests. Creep is measured as the settlement occuring during the period of maintaining a load increment on the pile head. Ultimate load of the pile is defined by creep limit ie. the load at which creep starts to increase rapidly.

Carroll's method can be described as follows.

From the load test data, the creep deformation versus load is plotted. Two tangents are drawn, one at the beginning portion of the curve and the other at the later portion. The bisector of the angle between the two tangents is extended to meet the curve (Figure 2.13). The load corresponding to the point of intersection of the bisector of the two tangents and the curve is considered as the failure load on the pile.

2.8.11 Mazurkeiwicz's method

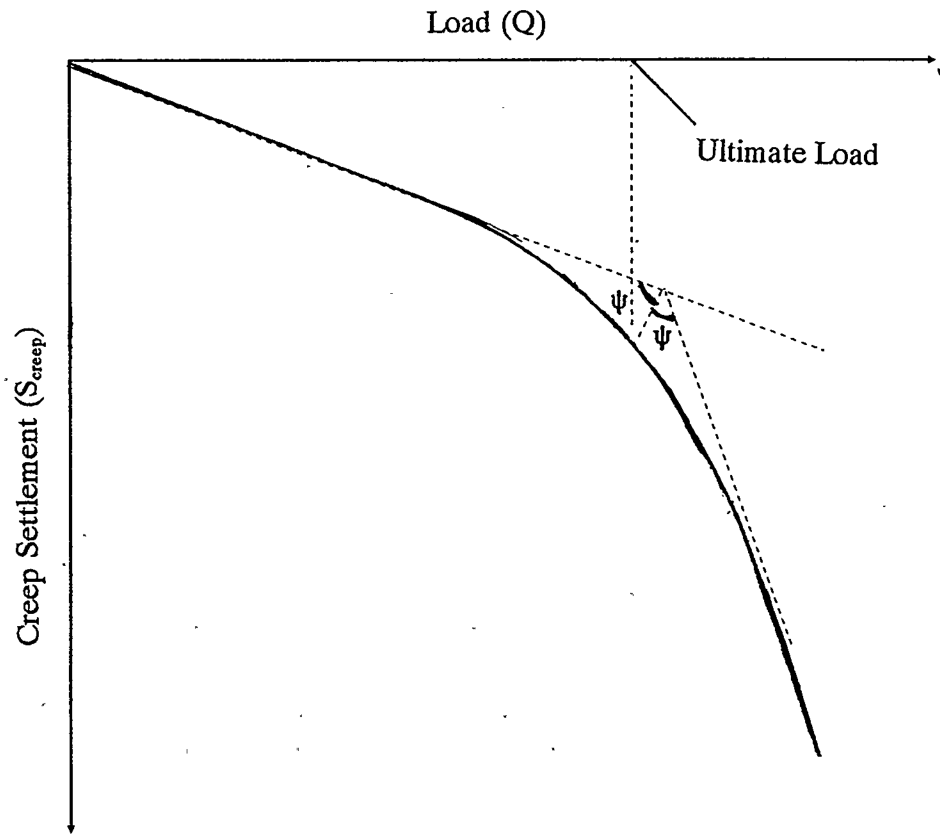


Figure 2.13 Carroll's Method for Ultimate Load of Piles

Mazurkeiwicz (1972) gave a method which allows the ultimate load to be determined for a pile which has not been tested to failure, but has been tested to a load close to the ultimate. On the load settlement curve, a set of equal gross settlement values of pile head are chosen. The set is preferably chosen at the later part of the load settlement curve. From each chosen settlement point, a line parallel to the load axis is drawn to intersect the load settlement curve. From each intersection point on the load axis, lines inclined at 45° to the load axis are drawn to intersect the extension of the next vertical line above load axis (Figure 2.14). Through these points of intersection on vertical lines, a straight line is drawn. The load corresponding to the intersection of this straight line with load axis is regarded as the ultimate load.

This method can be used for all types of load tests.

2.8.12 Chin's Method

Chin (1970) presented a method by which the ultimate load of a pile can be evaluated from the results of a load test without having to load the pile to failure. Chin (1970) concluded that the load (Q) -settlement (S) relationship is hyperbolic in nature and that a plot of settlement over load (S/Q) against settlement (S) is linear. Therefore,

$$\frac{S}{Q}=C_3S+C_4 \qquad (2.28)$$

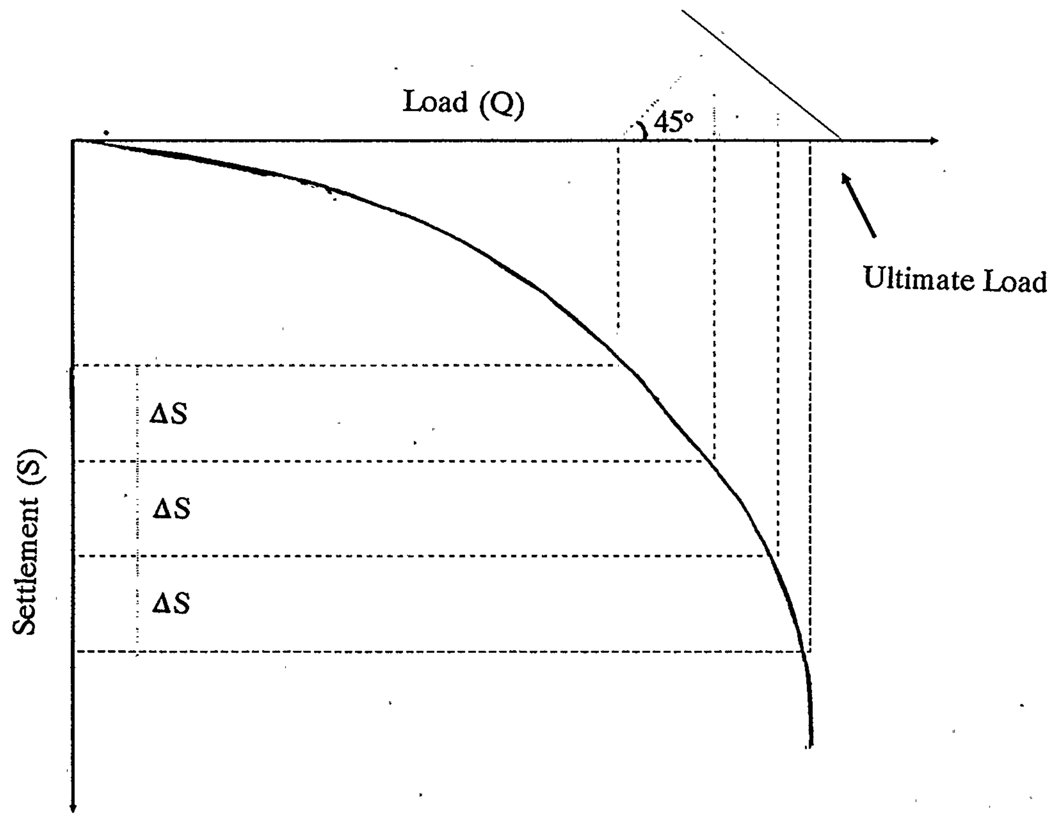


Figure 2.14 Mazurkeiwicz's Method for Ultimate Load of Piles

where, C_3 and C_4 are constants. The curve is made of two straight lines (Figure 2.15). The first straight line portion corresponds to the initial stages of loading associated with the build up of shaft friction. As the pile approaches failure, the plot shows another straight line having a much shallower slope. The inverse of the slope of the second line indicates the ultimate load of the pile.

The estimated failure load so obtained is found to be higher than that causing failure. For about 300 samples tested, the average ratio of the estimated value to the actual value was about 1.20. Hence a correction factor of about 0.2 may be necessary when Chin's method is used (Tan Swan Beng 1970)

This method can be used for all types of tests.

2.8.13 Modified Hyperbolic Method

The modified hyperbolic method, suggested by Rollberg (1976) and subsequently modified by Chakraborty (1989) assumes that the load-settlement curve of a pile begins at the origin as a hyperbola and after a certain limiting value of settlement, S_o, the shape of the curve changes from a hyperbola to a straight line. The start of the straight line portion of the curve is evaluated as a tangent to the hyperbola at the settlement $S = S_o$ (Figure 2.16). The equation of the load settlement curve as per Rollberg (1976) can be expressed as,

$$Q=C_o+b_oS\ldots S_o\leq S\leq\infty\ldots\ldots\ldots\ldots(2.29)$$

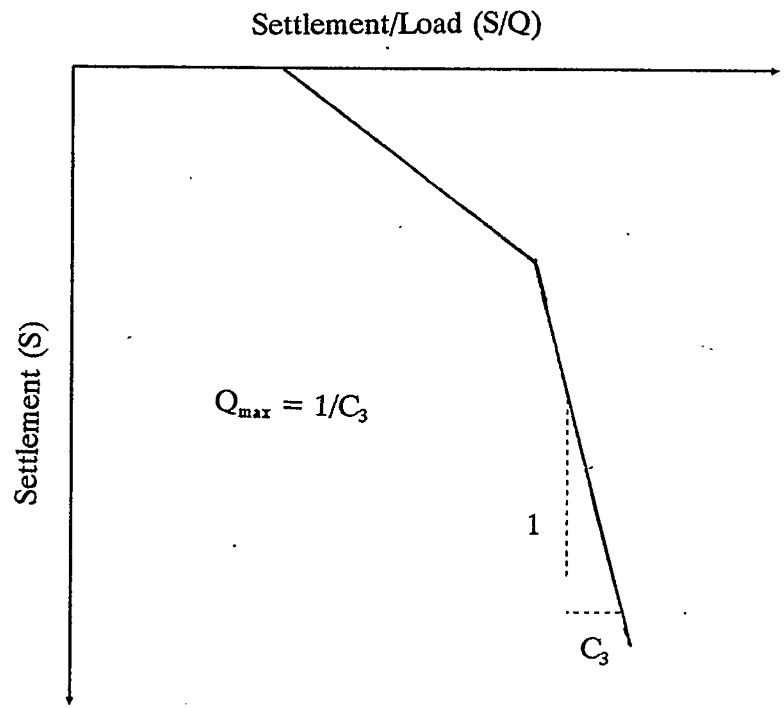


Figure 2.15 Chin's Method for Ultimate Load of Piles

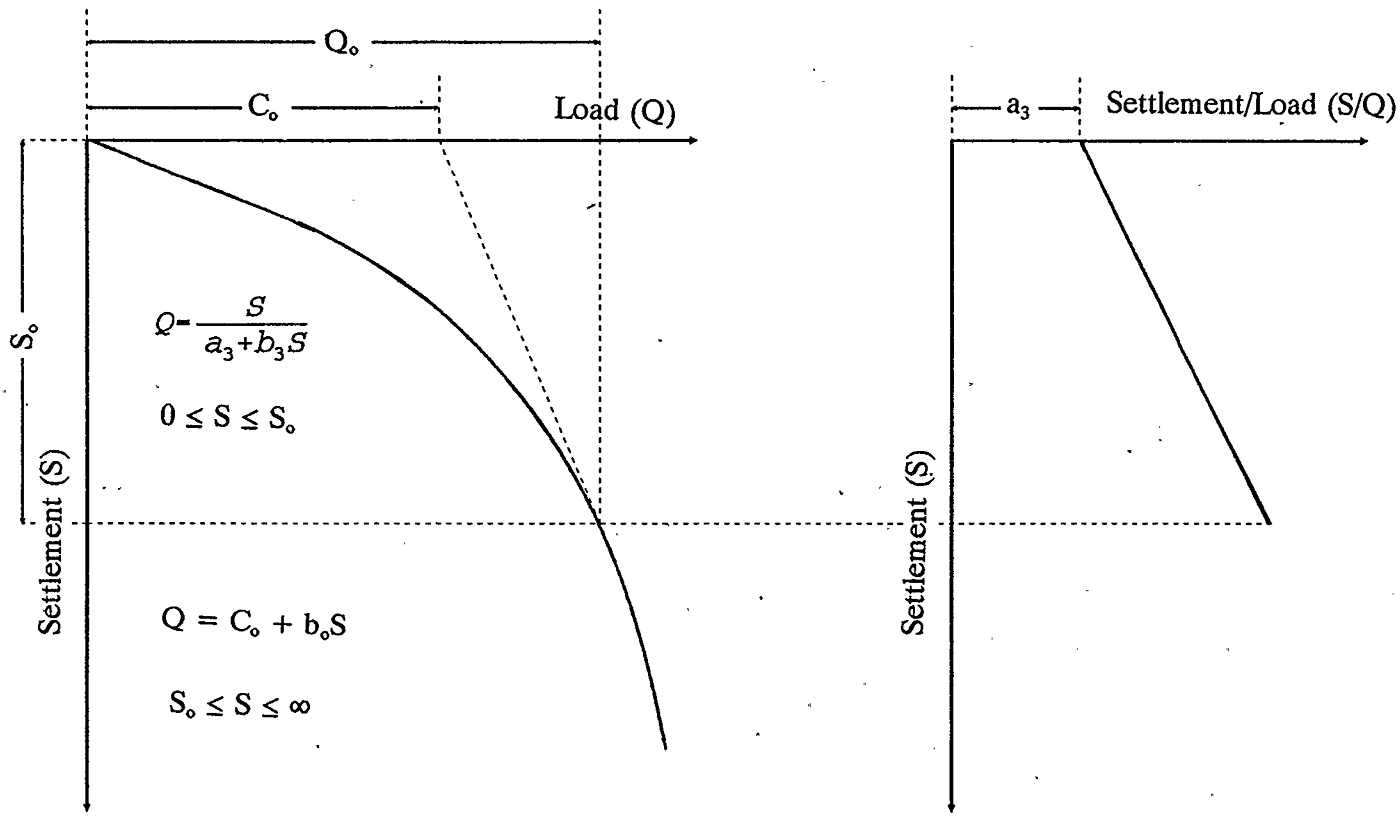


Figure 2.16 Modified Hyperbolic Method for Ultimate Load of Piles

and

$$Q=\frac{S}{a_3+b_3S}\ldots 0\leq S\leq S_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.30)$$

$$\therefore \frac{S}{Q}=a_3+b_3S \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.31)$$

Hence, a plot of S/Q against S yields b_3 as the slope and a_3 as its intercept. However, the value of S_o should be known beforehand.

The evaluation of S_o is explained below.

Case a: Maximum pile settlement (S_{max}) is larger than S_o

Here, S_o is to be determined from the load test data. Generally, the transition of the load settlement curve from a hyperbola to a straight line occurs rather smoothly. Hence, limit settlement, S_o, must be assumed. With this value of S_o, C_o and b_o are evaluated as described in Figure 2.16. Using Equation 2.29, the load settlement curve can be drawn to represent in a normal (Q,S) coordinate system. By comparing this load settlement curve, with the actual load settlement curve obtained, the estimation of S_o can be improved by trial and error.

Case b: Maximum pile settlement (S_{max}) is less than S_o

This case implies that the load test has not been carried up to failure. The value of limit settlement, S_o, is dependent upon the shape of the load settlement curve between the origin and S_o. According to Rollberg (1976), the closest form of relationship between the limit settlement, S_o, and parameters a_3 and b_3 in the double logarithmic scale coordinate system is given by,

$$\log S_o = d_1 + d_2 \log a_3 - d_3 \log b_3 \pm Y \qquad (2.32)$$

Values of coefficients d_1, d_2, d_3 and mean deviation Y, for different pile and soil types as suggested by Rollberg (1976) are presented in Table 2.2. Thus, the relationship between S_o and the coefficients a_3, b_3 can be expressed in the general form:

$$S_o = \frac{10^{d_1} a_3^{d_2}}{b_3^{d_3}} \qquad (2.33)$$

The limiting settlement, S_o, can be evaluated from Equation 2.33 and the corresponding limiting load can be evaluated from Equation 2.31.

With the load settlement data, a plot is made between S and S/Q as shown in Figure 2.16. A straight line fit is made for the points lying on the initial portion of the plot. The intercept of the straight line with the abscissa gives the value of a_3 and b_3 is the slope of the line. Depending on whether the soil is cohesive or cohesionless the values of d_1, d_2 and d_3 are considered from Table 2.2.

The modified hyperbolic method is probably the only method that does not require load test data upto or close to failure. All the other methods make use of the final non linear portion of the load settlement curve, where the curve shows a trend

TABLE 2.2

COEFFICIENTS FOR DETERMINATION OF S_O

Pile Type	Soil Type	Coefficients			MeanDe-viation
		d_1	d_2	d_3	Y
Driven	Noncohesive	0.374	0.329	0.745	0.15
Driven	Cohesive	1.251	0.407	0.44	0.052
Bored	Noncohesive	0.802	0.836	0.75	0.182

of increase in rate of settlement. This method makes extensive use of data in the initial portion of the curve. However, after a preliminary analysis of 32 pile test data, Chakraborty (1989) observed that if the measured field data of load and settlement are used directly in calculating the ultimate load, the modified hyperbolic method gives much higher values of ultimate load than those given by other methods. Hence, some refinement to the above method were suggested by Chakraborty (1989).

From the load test data the load settlement curve is drawn upto a gross settlement of nearly 1.5 % of the stem diameter of the pile. A set of loads with equal increment is chosen on the load axis. The load intervals are so chosen that the settlement corresponding to the final load should be close to 1.5 % of pile diameter. The gross settlement of pile head corresponding to each load increment is read from the load settlement. The load settlement data thus obtained is used to determine the ultimate load.

From the literature review it was concluded that detailed study of the effect of loading history and repeated loading on the performance of piles has not been done. Some research (Koreck and Schwarz 1988; Sparrow 1988; Chan and Hanna 1980) has been conducted to study and quantify these effects but considerable debate regarding these still exist.

The simulation of overburden pressure in the field by applying a surcharge load on the soil in a laboratory has not been done in the past. The average unit tip and shaft resistance with increase in the overburden pressure had been assumed to become constant but no study could conclude exactly at what depth. Some investigators (Kulhawy 1984) have reported that it never becomes a constant.

In order to effectively analyse the tests results it is necessary to define a criterion to determine the ultimate load of piles and to check the performance of the various methods given in literature, against this criterion.

CHAPTER 3

APPARATUS, TESTING AND TEST PROCEDURE

The pile testing apparatus and the procedure of sand filling has been described in detail by Sparrow (1988). However, a summary of the same is given here. Some modifications to the apparatus has been carried out to apply surcharge loads on the soil.

3.1 Test Apparatus and Testing Material

3.1.1 Sand Properties

The sand used for testing was air dried masonry sand with medium grain size distribution. It had a uniformity coefficient of 2.42 and effective size of 0.19 mm. The soil classification as per the Unified Soil Classification is SP-SM. The maximum and minimum dry densities as per ASTM D4253-83 (1983) and ASTM D4254-83 (1983) respectively were 17.20 kN/m^3 and 15.60 kN/m^3.

3.1.2 Pile Design

The pile was constructed of cold rolled mild steel seamless pipe of 50.8 mm diameter and 3.05 mm wall thickness. The modulus of elasticity of the steel was 203 GPa. The elastic deformation of the pile was calculated to be about 0.1 mm for L/D ratio of 33, and was hence neglected. The pile was assembled in eight sections using threaded connections. All sections except one were strain gauged. Each pile section had an inside slot 38 mm wide where the wall thickness was reamed to 0.64 mm for the strain gauges to be mounted. At each section two strain gauges were mounted diagonally opposite to neutralize the effect of eccentricity in loading (Figure 3.1). The strain readings from each section were recorded for each test. However, here they are not reported, as the strain readings were not required for this study. They may however, be used for a later study.

The pile was assembled in sections to vary the L/D ratio along the length. The pile tip consisted of a 20 kN load cell to measure compressive loads. The load cell was cylindrical in shape and had a flat surface at the bottom end. The top of the load cell was threaded to fit into the last pile section. The load cell had a length of 101 mm (4 inches) and a diameter of 50.8 mm. Once the load cell was fixed in place, it became an integral part of the pile like other pile sections. The length of the load cell was considered when calculating the pile L/D ratio.

A pile cap was used to facilitate driving as well as compressive testing. For the uplift tests a ball and socket coupling was used. This replaced the pile cap and was screwed in turn to the bottom of a load cell which also acted as a LVDT. The ball and socket connection eliminated the eccentric loading during uplift. The load cell measured the top load on the pile whereas the displacement of the pile was measured by the LVDT. The load cell and the LVDT were an integral part of a MTS system. The LVDT was hung from a frame. The net displacement of the frame for a load of 5 kN was calculated to be only 0.04 mm and was hence neglected from

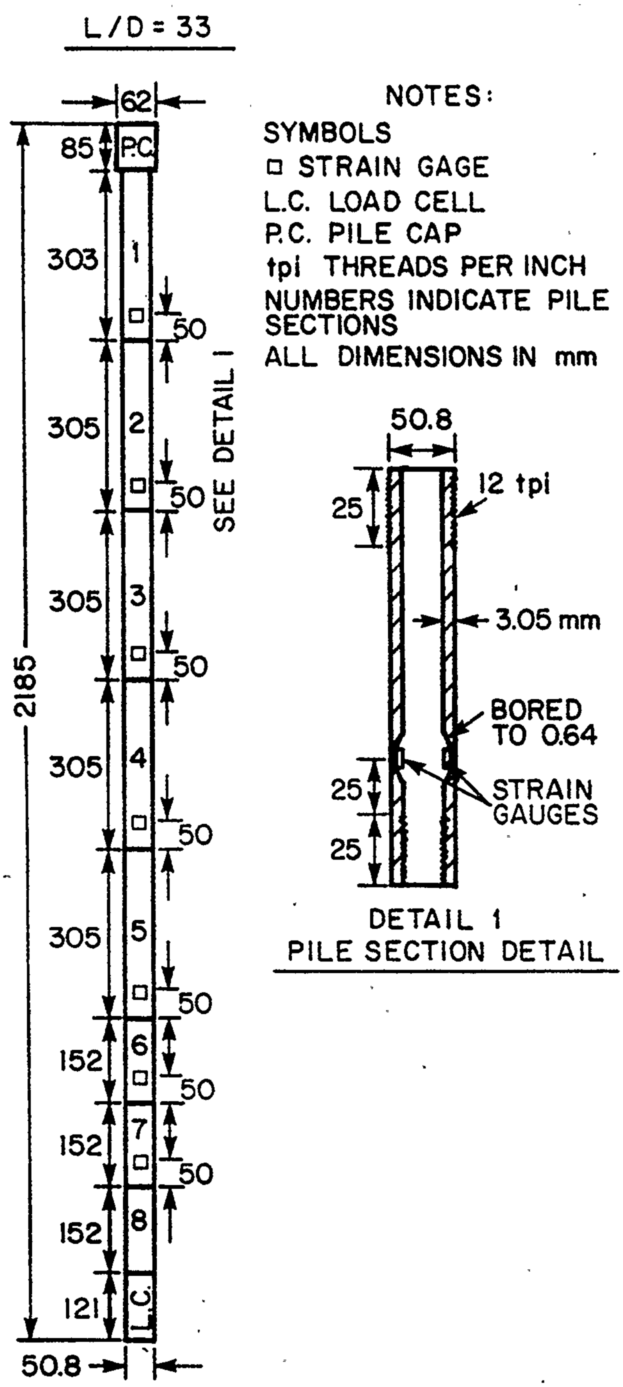


Figure 3.1 Pile Details - Sparrow 1988 *(adapted)*

analysis.

3.1.3 Test Tank Design

The test tank consisted of two parts, the base and the shell. The shell was circular and fabricated from 16 gauge sheet metal in two sections which were then bolted together. The outer shell was reinforced with horizontal flats welded to the sides of the tank. Additional reinforcement for tensile loads were provided with angle irons welded all along the length of the shell from the top lip of the tank to the bottom base. This was done for the tank to resist the tensile and hoop stresses generated when the sand around the pile was surcharge loaded. The base consisted of 9.5 mm sheet metal reinforced at the bottom with a 38 mm × 38 mm × 6.4 mm steel channel spaced 300 mm.

The ratio of the diameter of the test tank to the pile diameter was 17.7.

3.1.4 Design of Rubber Balloon and Top Lid of the Tank

In order to apply surcharge loads, a donut shaped annular rubber balloon of about the same outer diameter as the tank was fabricated. The balloon was encased within a flexible nylon covering giving it, its required shape. The covering and the balloon had an annular hole of diameter 76.2 mm for the pile to be driven through it. Air was pumped inside the balloon for application of surcharge loads (Figure 3.2).

The top of the test tank had a reinforced lid that held the tank under pressure (Figure 3.3). After the balloon was placed the lid was placed on the top of the tank

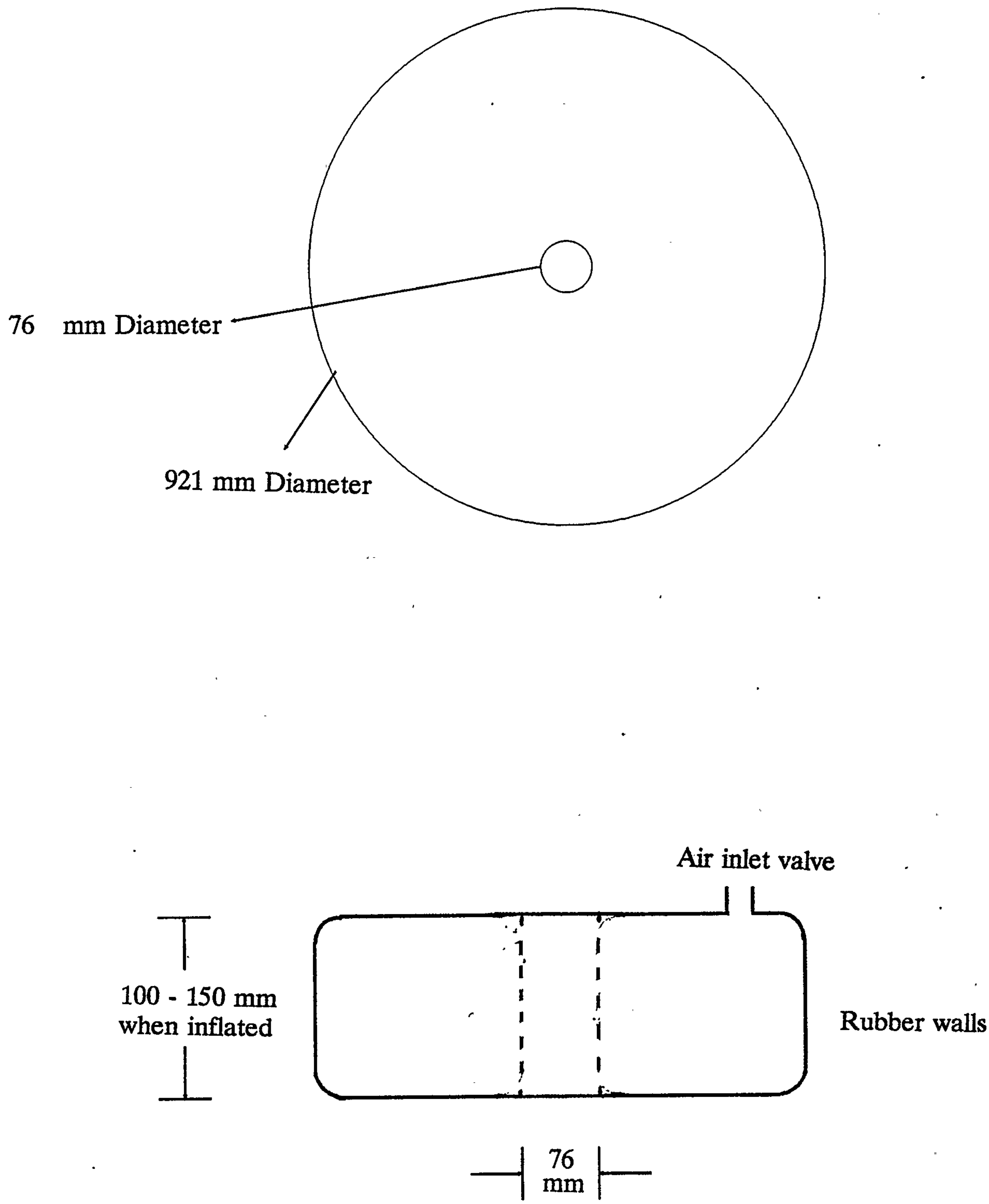


Figure 3.2 Balloon for Applying Pressure on the Sand

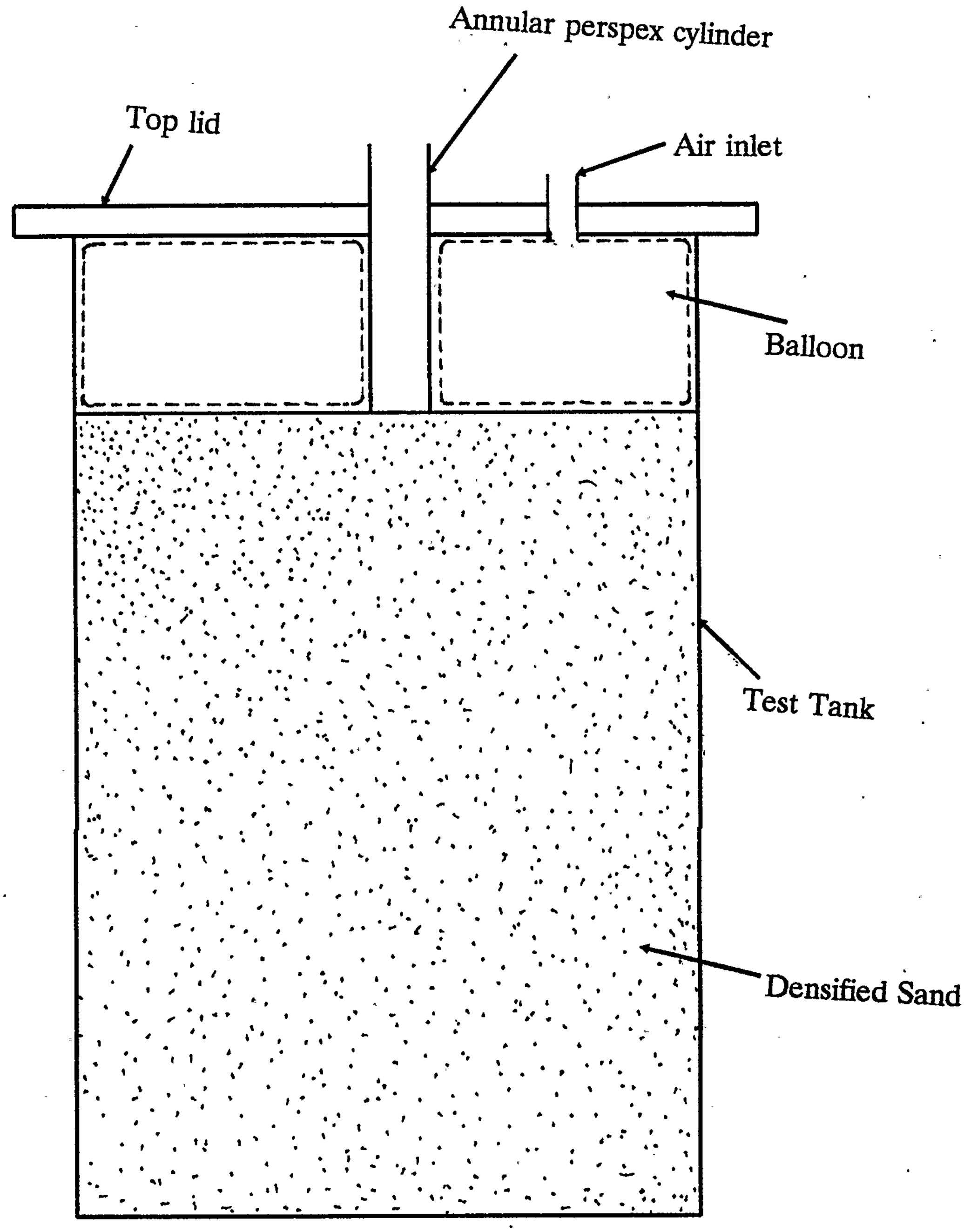


Figure 3.3 Test Tank with the Pressure Application System

and tightened with nuts and bolts to the tank. The lid had an annular hole of diameter 101.6 mm in the middle for the pile, and a smaller hole away from the centre, of diameter 25.4 mm, to let the air pipe to the balloon. An air inlet connected to the balloon passed through the top lid to an air pressure regulator and a safety valve to an air compressor. The regulator was used for applying different air pressures to the sand.

There were some severe problems associated with the balloon. Before applying pneumatic pressure it was decided that hydraulic pressure should be applied. Water was filled into the balloon and hydraulic pressure was applied. The balloon started leaking and the whole sand bed became wet. The leaking balloon had to be sent back to its original suppliers in Toronto and this delayed the start of the testing by more than a month.

3.2 Sample Preparation

3.2.1 Pluviation

The sand bed was prepared by pluviation or raining technique (Rad and Tumay 1987). A schematic diagram of the apparatus used to prepare the sand bed is given in Figure 3.4. The top tank measuring 2.13 m in height and 0.89 m in diameter was filled with sand. The test tank was then wheeled to below the top tank and aligned in position. A top plate, 0.87 m in diameter was then placed on the sand in the top tank. Cables were attached through the pulley in the top tank to hooks welded on the top plate. A sand diffuser consisting of 2 horizontally placed seives

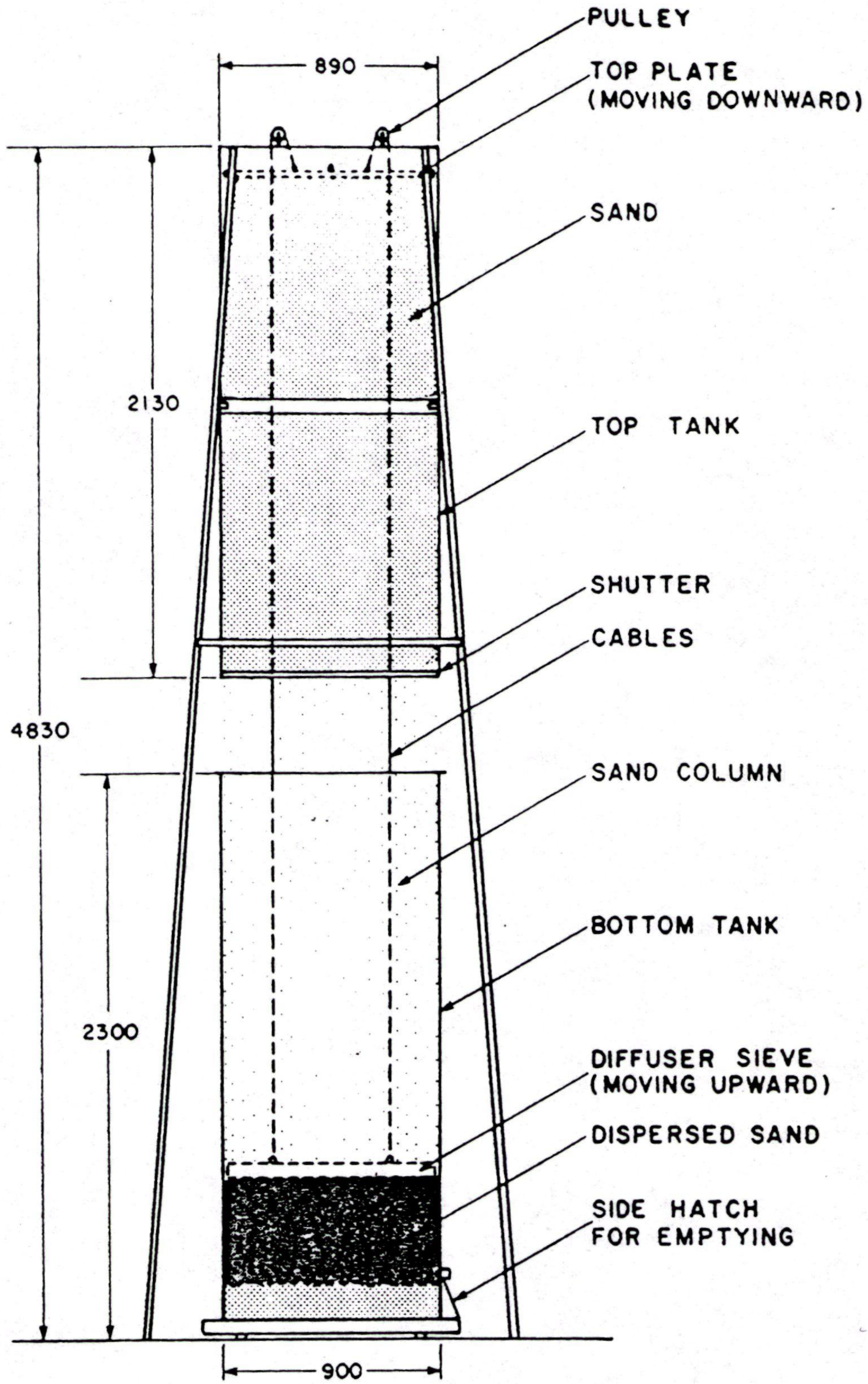


Figure 3.4 Schematic Diagram of Sample Preparation Appartus - Sparrow 1988

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rotated 45° with respect to each other, 50 mm apart with a nominal seive opening size of 6.35 mm was hung by the cables inside the test tank.

Once the whole setup was ready, a canvas envelope was attached from the bottom section of the top tank to the top part of the test tank. This canvas prevented the dust particles from flowing outside the setup once the shutter attached to the bottom of the top tank was opened. The shutter consisted of a 9.5 mm thick steel plate with 12.7 mm diameter holes uniformly spaced 63.5 mm center to center and a total porosity of 3.61 %. The base of the top tank had a similar pattern. When the shutter was opened, the two sets of holes lined up and the sand rained from the top tank. As the sand fell from the top tank, the top plate moved down and along with it moved up the diffuser. Since, the two tanks had the same diameter the decrease in the height of sand in the top tank was about the same as the increase of the height of sand in the bottom tank. Hence, the height of fall of sand grains from the diffuser to the bed below remained constant at 0.394 m. This resulted in samples of homogeneous cross-section and uniform density of 16.75 kN/m^3, a relative density of 73.8 % and a friction angle of 39°. However, twice while preparing the sand, the diffuser got stuck in the sand bed decreasing the height of fall of sand. This produced medium dense sand deposits with a unit weight of about 16.40 kN/m^3 and a relative density of 52.4 %. The friction angle for the medium dense sand was 37°. These two cases occurred during tests to be conducted for loading history. Tests on medium dense sand deposits were consequently carried out and are here referred to as dense and medium dense sand deposits respectively.

In order to test the uniformity of density with depth, Sparrow (1988) conducted cone penetrometer tests. It was concluded that the sand density was nearly constant except for slight variation which were attributed to disturbances caused due to handling of the tank.

Numerous difficulties were encountered while filling up of the sand. Quite often the strainer got stuck in the sand bed or the wires connecting the strainer to the top plate slipped from the pulleys in the top tank. Once during filling these wires snapped and the filling operation had to be aborted. On an average for every two filling operation done one had to aborted due to a malfunction of the apparatus.

3.2.2 Surcharge Loading

Before driving the pile, the balloon for surcharge loading was placed on the sand. In the inner annulus of the balloon a perspex tube was inserted just fitting in the inside of the balloon. The tube wall had a thickness of 3 mm. Once the balloon and the tube was placed in position, the top lid was carefully lowered in place, passing the perspex tube and the air tube of the balloon through it. The lid was then bolted to the test tank. The perspex tube aided in guiding the pile to the center of the tank as well as preventing the balloon from gripping against the pile.

3.2.3 Pile Driving

Once the sand sample was prepared, the top tank was removed and the pile driving apparatus set over it. The driving apparatus consisted of the traditional rope and pulley arrangement. The hammer consisted of a hollow steel cylindrical bucket of weight 105 N, dropping from a height of 0.3 m. After setting the pile driving apparatus in place, the pile was passed through two guides to ensure pile penetration to be straight. The hammer slided along a set of rollers and had a near free fall to the pile head from the requisite height.

3.2.4 Data Acquisition

Data acquisition was accomplished using a Datascan and Labtech Notebook software package and a personal computer. The strain readings on the gauges on the pile, the load readings at the tip of the pile and the load and displacement of the ram of the MTS were recorded through the Datascan. Data were recorded every 20 seconds. A plotter was setup to record the output of the two load cells against the pile head displacement.

3.3 Test Procedures

3.3.1 Loading History

The model piles were tested no less than 36 hours after being driven. The test tank was then moved to the MTS frame and aligned such that the pile was directly below the ram. To study the behaviour of piles having loading history as compared to those in virgin loading case, 20 pile load tests were conducted. The test tank was filled up 8 times for these tests. Tests were conducted for three L/D ratios of 33, 26 and 20. These L/D ratios correspond to pile lengths of 1.67m, 1.33m and 1.02m respectively. For each L/D ratio two series of tests were performed. The first series consisted of a tension test followed by two compression tests. The first compression test simulates the condition where a driven pile is lifted up by the virgin tension test. The second compression test simulates the condition where the disturbed pile has been nearly pushed back or redriven to its initial position. In the second series of tests, the loading history is reversed and a tension test is conducted after the completion of a compression test on the pile (Table 3.1).

For density measurements the tank with the sand was weighed after the tests. A one hour gap was maintained between two consecutive tests. Piles having L/D ratios of 33 and 20 were tested in dense sand (relative density = 73.8 %) whereas, piles having L/D ratio of 26 were tested in dense and medium dense sand (relative density of 73.8 % and 52.4 % respectively).

TABLE 3.1

SUMMARY OF PILE TESTS CONDUCTED FOR LOADING HISTORY

Pile Designation	L/D ratio	Density	Type of Test
TP1	33	Dense	C - T
TP2	33	Dense	T - C1 - C2
TP3	26	Dense	C - T
TP4	26	Dense	T - C1 - C2
TP5	26	Medium Dense	C - T
TP6	26	Medium Dense	T - C1 - C2
TP7	20	Dense	C - T
TP8	20	Dense	T - C1 - C2

Note:

C = Virgin compression test

T = Tension test

C1 = First compression test after tension

C2 = Second compression test after tension

3.3.2 Cyclic or Repeated Loading

Repeated loading test was conducted for one way tension loading only. The virgin tension failure load obtained in series 1 was considered.

Repeated lading tests were conducted for piles with L/D ratios of 33, 26 and 20. For each L/D ratio, six pile load tests were conducted. For one L/D ratio of 33 pile tests were conducted at two frequencies of 0.1 Hz and 0.05 Hz. A total of 24 pile load tests were conducted, 6 tests for each L/D ratio and frequency of 0.1 Hz and 6 tests for L/D = 33 and a frequency of 0.05 Hz. The sand tank was filled up four times for these tests. Repeated loading tests were conducted on the pile with loads ranging from 0 to 10 %, 0 to 12.5 %, 0 to 16.6 %, 0 to 20 %, 0 to 25 % and 0 to 33 % of the tensile failure load. These correspond to factors of safety or load factors of 10, 8, 6, 5, 4 and 3 respectively. The type of wave form used was essentially square in nature. It took a fraction of a second to build up. The load then remained applied for its duration of 10 or 20 seconds following which it released the load with in a fraction of a second. All tests were conducted on dry sand having a density of 16.75 kN/m^3, relative density of 73.8 % and a ϕ' of 39°.

Two percent of the pile failure load was applied as a seating load to keep the system in tension.

3.3.3 Surcharge Loading

The last parameter that was experimentally studied was the effect of surcharge loading on the soil around a pile. Experiments were conducted to study the effect of surcharge loads on the pile shaft and tip capacity. A pile of L/D ratio of 26 was used. Surcharge loads of 34.5 kPa (5 psi), 69 kPa (10 psi), 103.5 kPa (15 psi), and 138 kPa (20 psi) were used. A total of eight pile load tests were conducted and four times the sand tank was filled up. All tests were conducted on dense sand having a density of 16.75 kN/m^3, relative density of 73.8 % and a ϕ' of 39°.

After the sand bed was prepared by pluviation, the balloon was placed on the top of the sand in the tank. Then slowly, and carefully manoeuvring around, the top lid of the tank was placed in position and bolted to the lip of the test tank. The pile was then driven and air was pumped in the balloon to the requisite pressure. The encased balloon applied the pressure on to the sand. The system was left as such for 36 hours. After 36 hours the pile was tested. Piles were tested for both compressive and tensile capacity. The rate of penetration and uplift was maintained at 0.5 mm/min. Figure 3.5 shows a photograph of a pile in surcharge loaded soil ready for testing.

CHAPTER 4

TEST RESULTS AND DISCUSSION

Discussion is presented in this chapter in two parts: the first part describes the results of experiments conducted on model piles to study the effect of load parameters individually on pile performance. A criterion to evaluate the pile ultimate capacity, involving all the methods applicable for a particular method of testing was evolved. The second part describes the analysis of 101 pile load test data on 93 piles collected from literature. These load test data correspond to piles tested at widely scattered geographical locations. This analysis was done to check the ability of the various methods to evaluate the pile ultimate load. Some settlement criteria were considered to check the performance of the defined criterion.

4.1 Evaluation of Load Parameters

4.1.1 Loading History

As described earlier (section 3.3.1), two series of tests were conducted. The first series consisted of a virgin tension test followed by two successive compression tests. The second series consisted of a virgin compression test followed by a tension test.

The following two types of comparisons between the test results obtained from the two series of tests were made :

(i) the virgin tension in series 1 and the tension test after the compression failure in series 2

(ii) between the virgin compression test in series 2 and the first and second compression tests after a tension failure in series 1.

Compression Test

Total Load : The load versus pile head settlement for the piles in dense sand are shown in Figures 4.1, 4.2 and 4.4 respectively for L/D ratios of 33, 26 and 20. The load settlement relationships for the pile in medium dense sand (L/D = 26) are presented in Fig. 4.3. Several methods have been suggested (Fellinius 1975,1980; Joshi and Sharma 1987) for the determination of ultimate load from the load settlement curve. Table 4.1 gives the ultimate load of the piles as determined by 7 different procedures applicable for constant rate of penetration test. To determine the ultimate load of a pile a criterion was defined. As per this criterion, the arithmetic mean value of the ultimate load obtained from all the methods was calculated first. Those values which were more than 1.5 times or less than 0.5 times this mean value were discarded. This was done to eliminate the effect of methods which predict unrealistically high or low values. Hence, the probable range within which the failure load would lie was realised. The mean of the remaining values was calculated again and the process of elimination of extreme values was repeated until no further elimination was necessary. This final mean value was defined as the mean failure load or the probable failure load. This iterative process was used to determine the ultimate load of all the piles.

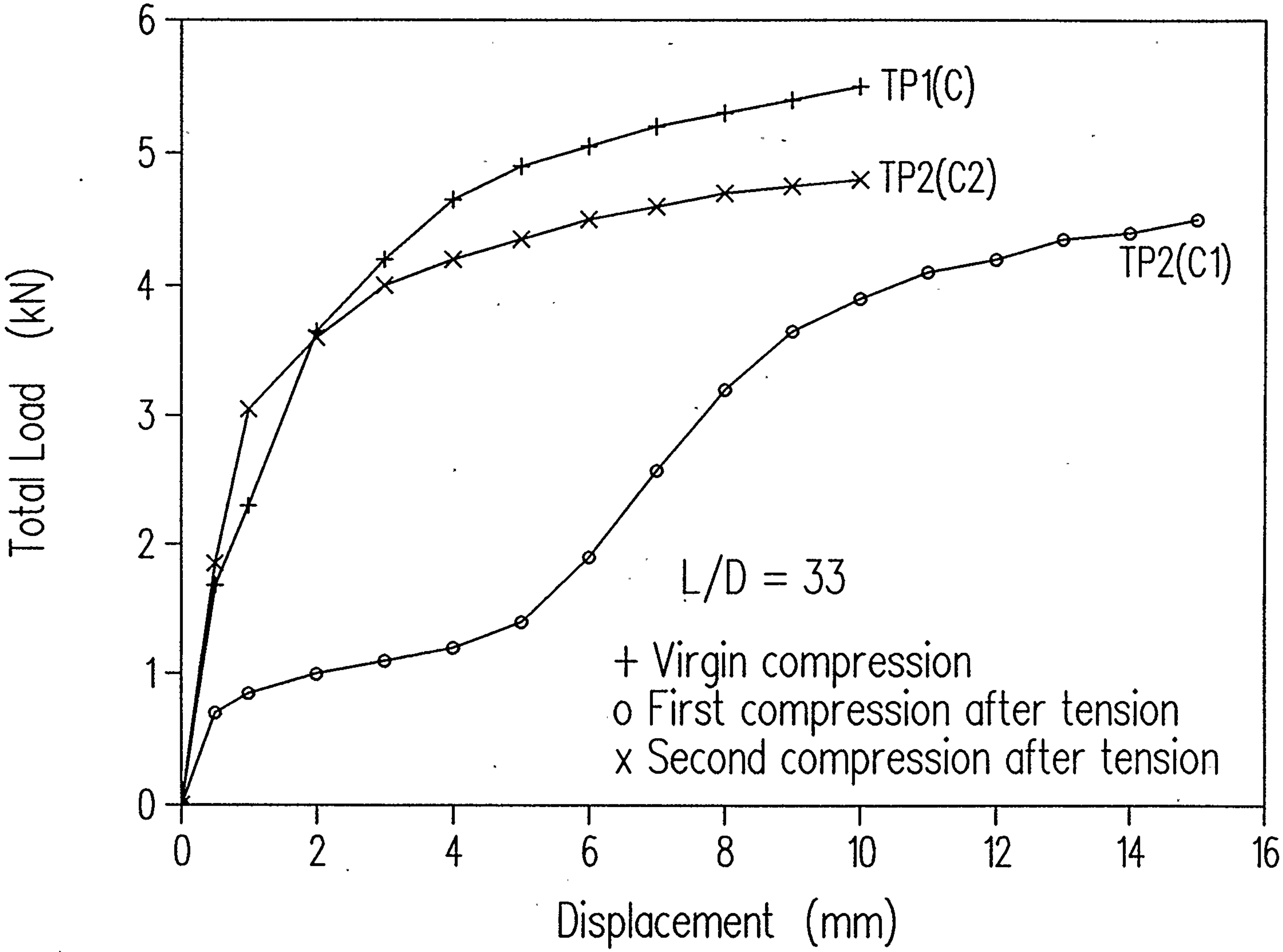


Figure 4.1 Load - Displacement Plots for Piles (L/D = 33) in Dense Sands

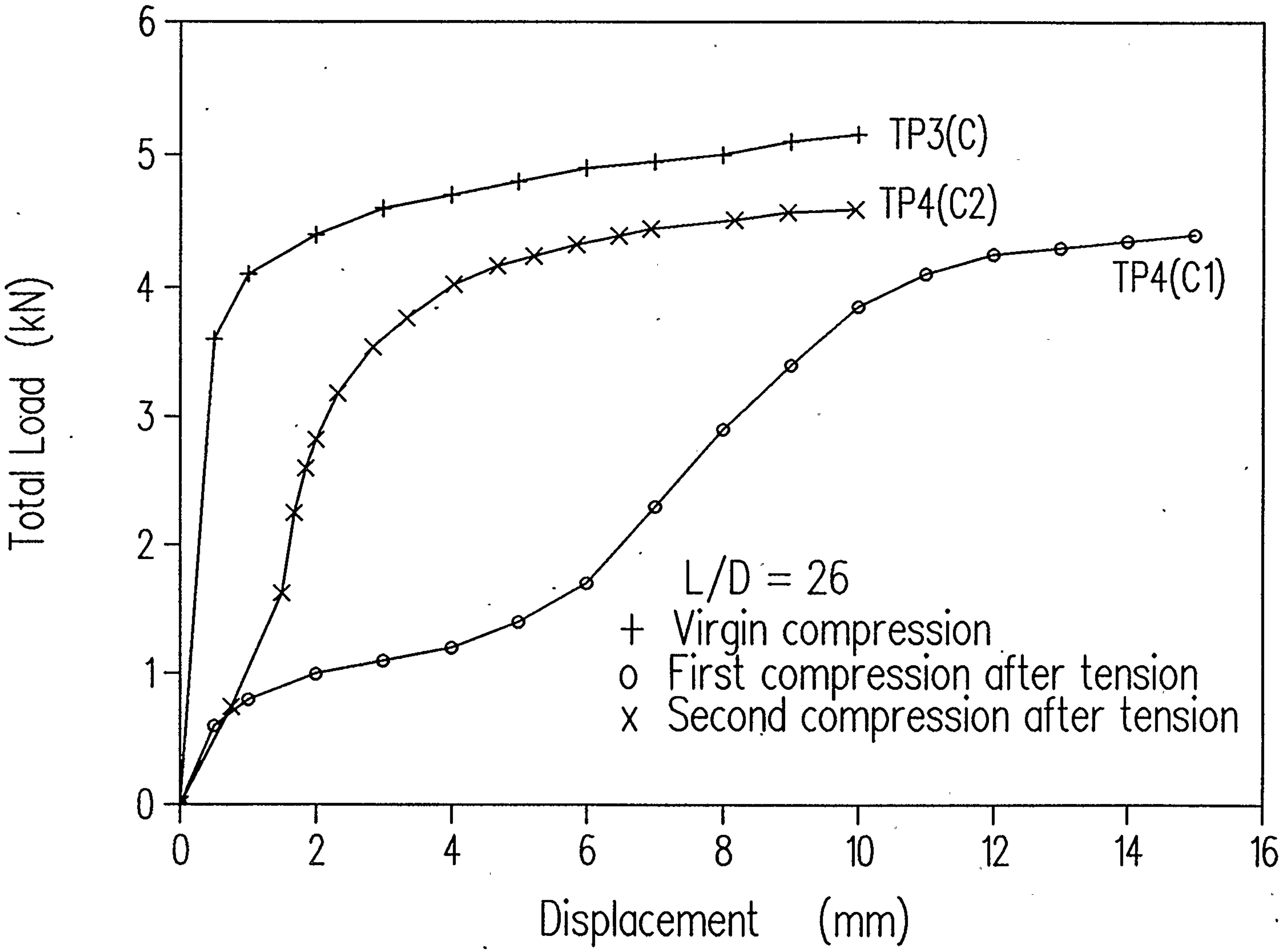


Figure 4.2 Load - Displacement Plots for Piles (L/D = 26) in Dense Sands

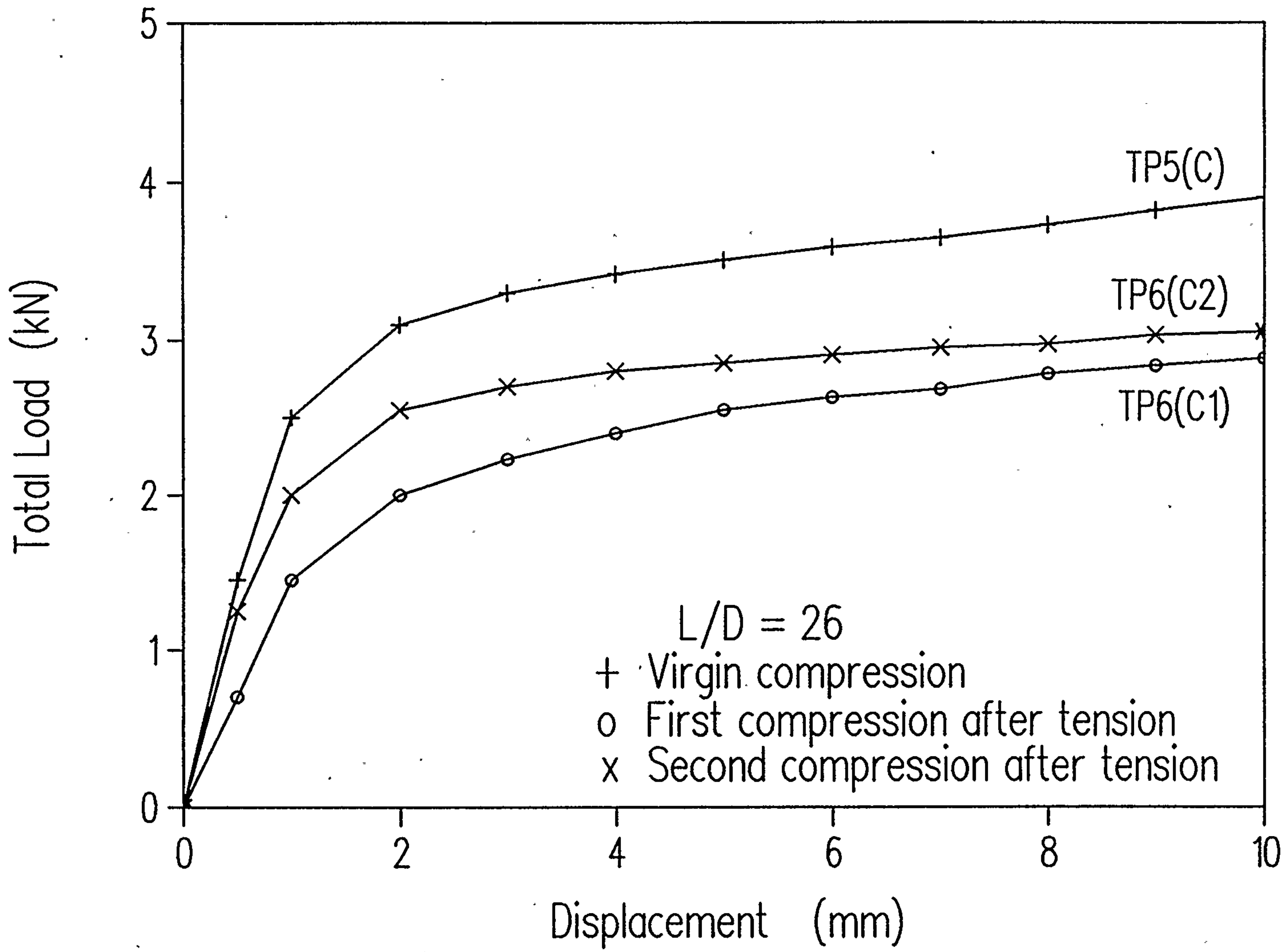


Figure 4.3 Load - Displacement Plots for Piles (L/D = 26) in Medium Dense Sands

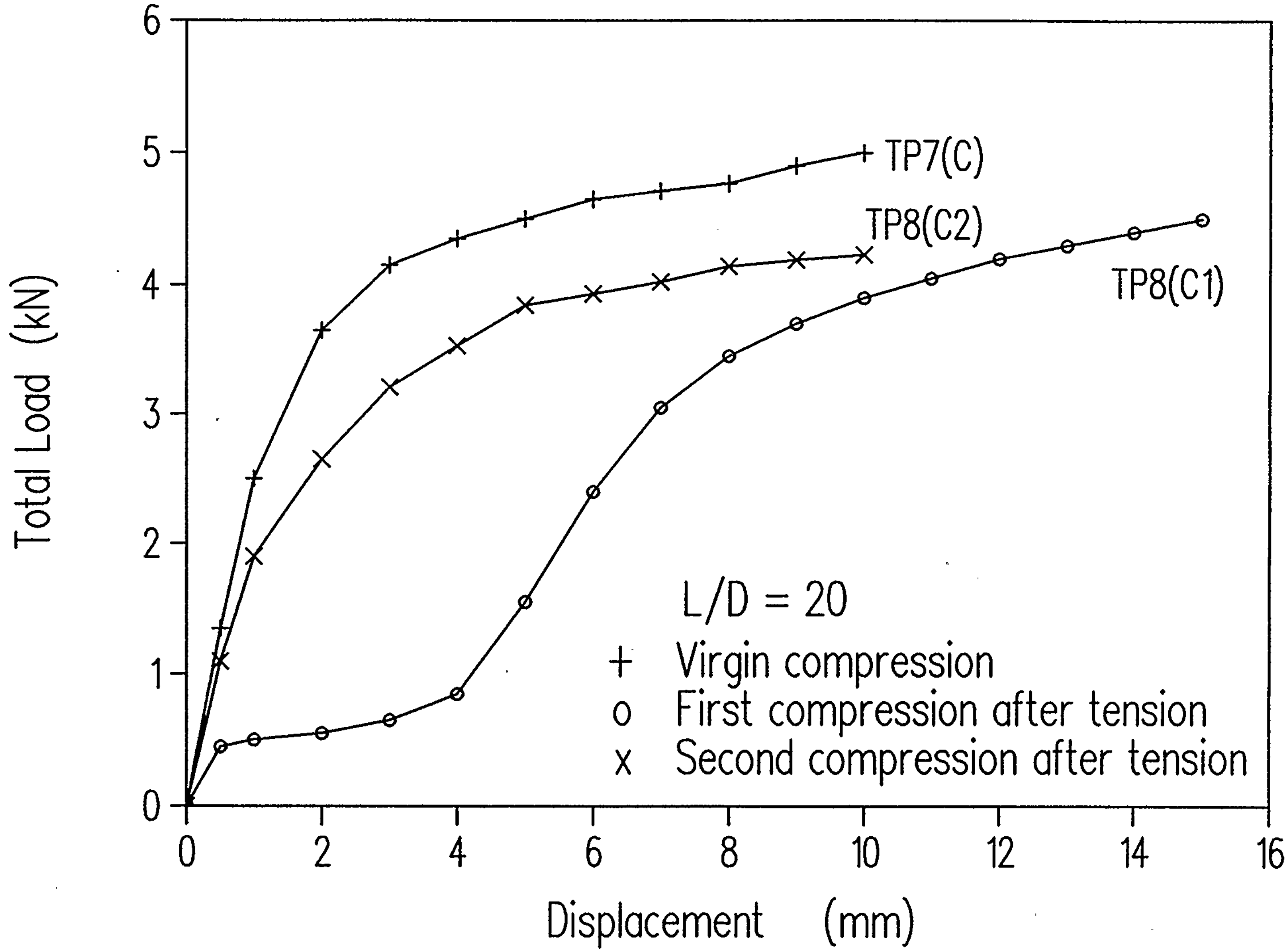


Figure 4.4 Load - Displacement Plots for Piles (L/D = 20) in Dense Sands

From the load settlement curves plotted in Figures 4.1, 4.2 and 4.4 it was concluded that piles in first compression after a tension test show a lag in picking up of the total load. Generally, displacement of approximately 5 mm was found necessary for picking up load in these compression tests. An examination of data in Fig. 4.1, 4.2 and 4.4 show that the load settlement curves in these tests, conducted immediately after the tension tests, are not typical compression test type. In these tests the pile settled considerably first under minimal initial load and then the settlement reduced significantly on subsequent loading, giving rise to a step curve. No standard procedure for the evaluation of the failure load could be used for these data. Instead in these cases, the settlement criteria of the pile was said to govern.

TABLE 4.1

COMPRESSIVE FAILURE LOAD (kN) OF PILES HAVING LOADING HISTORY

NAME	ST	DT	MAZ	CHIN	BH90	BH80	VAN	MEAN (kN)	SETT (mm)
TP1(C)	4.60	4.75	5.82	6.60	5.50	5.94	5.40	5.52	9.40
TP2(C2)	4.33	4.40	5.00	5.19	4.80	4.70	4.95	4.77	9.40
TP3(C)	4.44	4.46	5.20	6.07	4.30	5.26	5.10	4.98	8.10
TP4(C2)	4.12	4.30	4.35	5.30	4.50	4.98	4.70	4.61	9.60
TP5(C)	3.10	3.20	4.20	4.81	3.95	4.26	4.10	3.95	> 10
TP6(C2)	2.70	2.74	3.15	3.31	2.80	3.04	3.05	2.97	8.64
TP6(C1)	2.30	2.35	2.85	3.42	2.80	3.18	2.75	2.81	8.37
TP7(C)	4.15	4.27	5.00	5.60	5.00	5.15	4.85	4.86	8.75
TP8(C2)	3.68	3.80	4.44	4.64	4.30	4.22	4.30	4.20	8.70

Note:

C = Piles in virgin compression

C1 = First compression after tension

C2 = Second compression after tension	ST = Single Tangent
BH90 = Brinch-Hansen's 90 % criteria	DT = Double Tangent
BH80 = Brinch-Hansen's 80 % criteria	MAZ = Mazurkeiwicz's method
VAN = Van der Veen's method	CHIN = Chin's method
MEAN = Mean failure load	SETT = Settlement at mean failure load

The failure load of the stepped load settlement curve (first compression) was evaluated considering that the settlement for the mean ultimate load of the second compression test, after tension test, as being the same as that of the first compression test after tension. The failure load of piles in second compression was more than the failure load in first compression (Figures 4.1 - 4.4). However, in the case of medium dense sand (relative density = 52.4 %) no step type curve and the load settlement curve was observed (Figure 4.3) similar to the virgin compression curve.

For all L/D ratios and densities load-settlement plots corresponding to piles that have already failed in tension show a decrease in the ultimate failure load when compared to piles in virgin compression. This decrease was in the range of 7.4 % to 24.8 %. Koreck and Schwarz (1988) conducted experiments of a similar nature. They have presented plots to demonstrate a decrease in the ultimate capacity for piles having loading history. The decrease in the ultimate compression load due to an initial tension loading on the pile was around 20 % for a pile with a L/D ratio of 38.5.

The settlements corresponding to the mean failure loads are presented in Table 4.1. The mean failure settlement for L/D = 26 in medium dense sand could not be evaluated as the mean ultimate load was beyond the load up to which the test was carried.

According to Vesic (1964) the settlement needed to attain ultimate load was about 14 % of the pile diameter for medium dense, dry sand and about 14.5 % of the pile diameter for dense dry sand. The average settlement for failure in virgin compression tests obtained in this study were more than 20 % of the pile diameter for medium dense sands and in the range of 15.9 % to 18.5 % of pile diameter for dense sands. For the load settlement plots reported by Koreck and Schwarz (1988) the failure settlement for virgin compression was about 8 % of the pile diameter whereas, for piles having loading history it was around 20 %. In the present study such wide variation in the failure settlement was not observed. Since the data reported by Koreck and Schwarz (1988) is from the field and the soil profile has not been given, the reasons for this variation could not be found out. However, since the experiments conducted here were under laboratory controlled conditions where it is known that the soil is of uniform density these results are expected to be more reliable.

Tip Load : Figs. 4.5, 4.6 and 4.8 show that in dense sand increasing tip load was mobilized as the pile displacement increased. Tip load-displacement plots for piles in medium dense sand and a L/D ratio of 26 are presented in Fig. 4.7. The tip load- settlement curves have been plotted considering the pile weight of 150 N as always acting at the tip as a seating load. Except in the case of first compression test

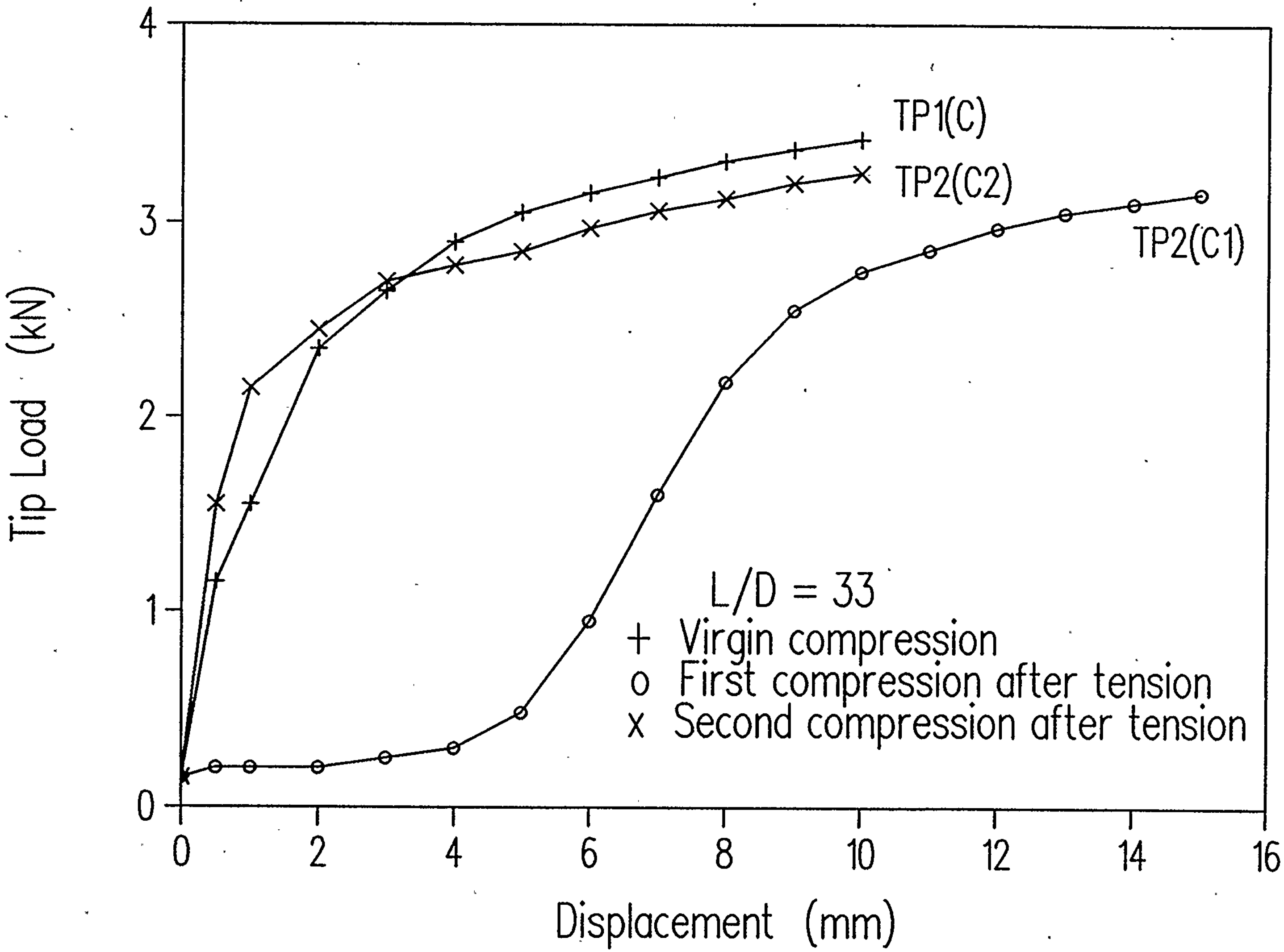


Figure 4.5 Tip Load - Displacement Plots for Piles (L/D = 33) in Dense Sands

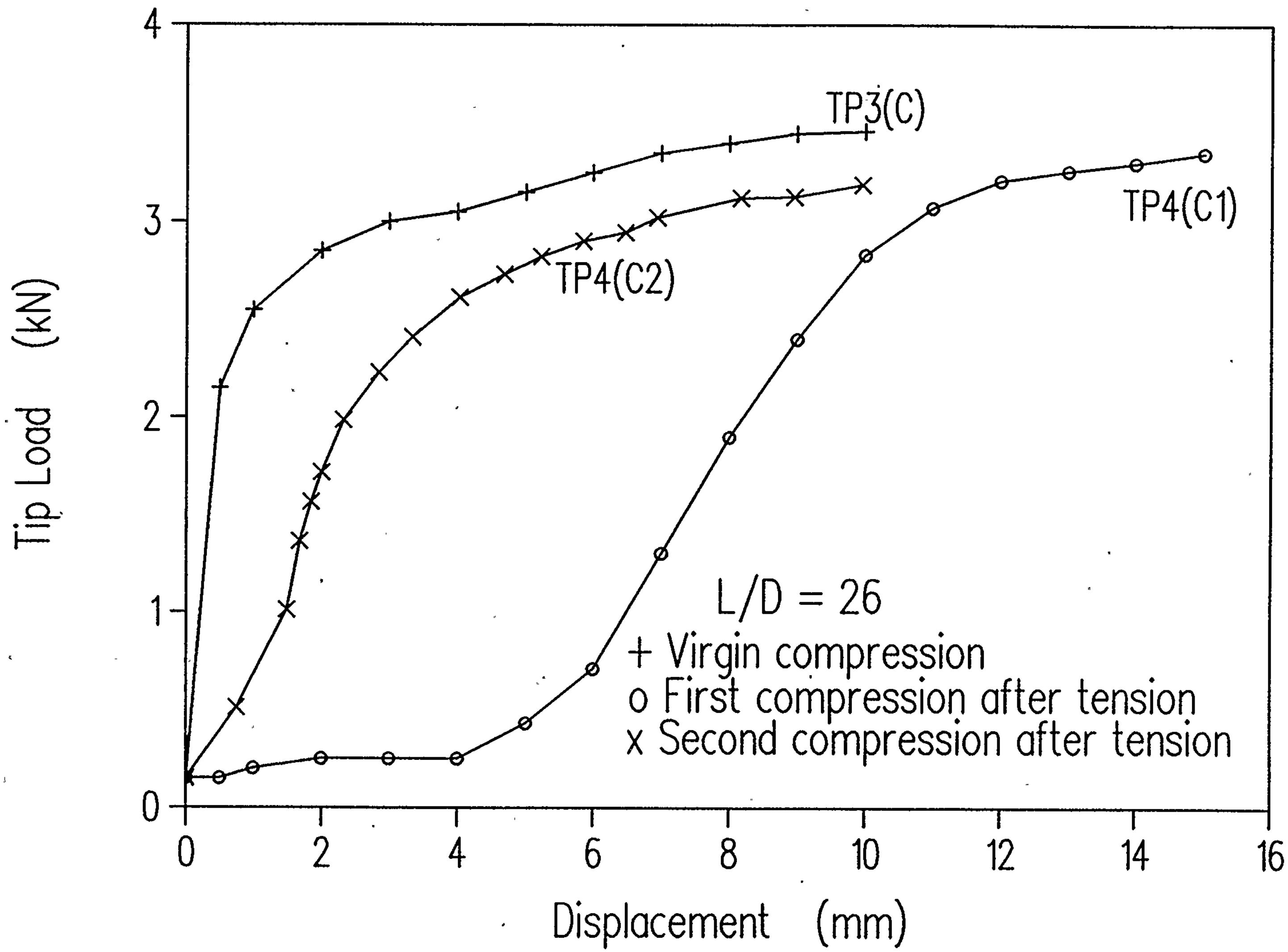


Figure 4.6 Tip Load - Displacement Plots for Piles (L/D = 26) in Dense Sands

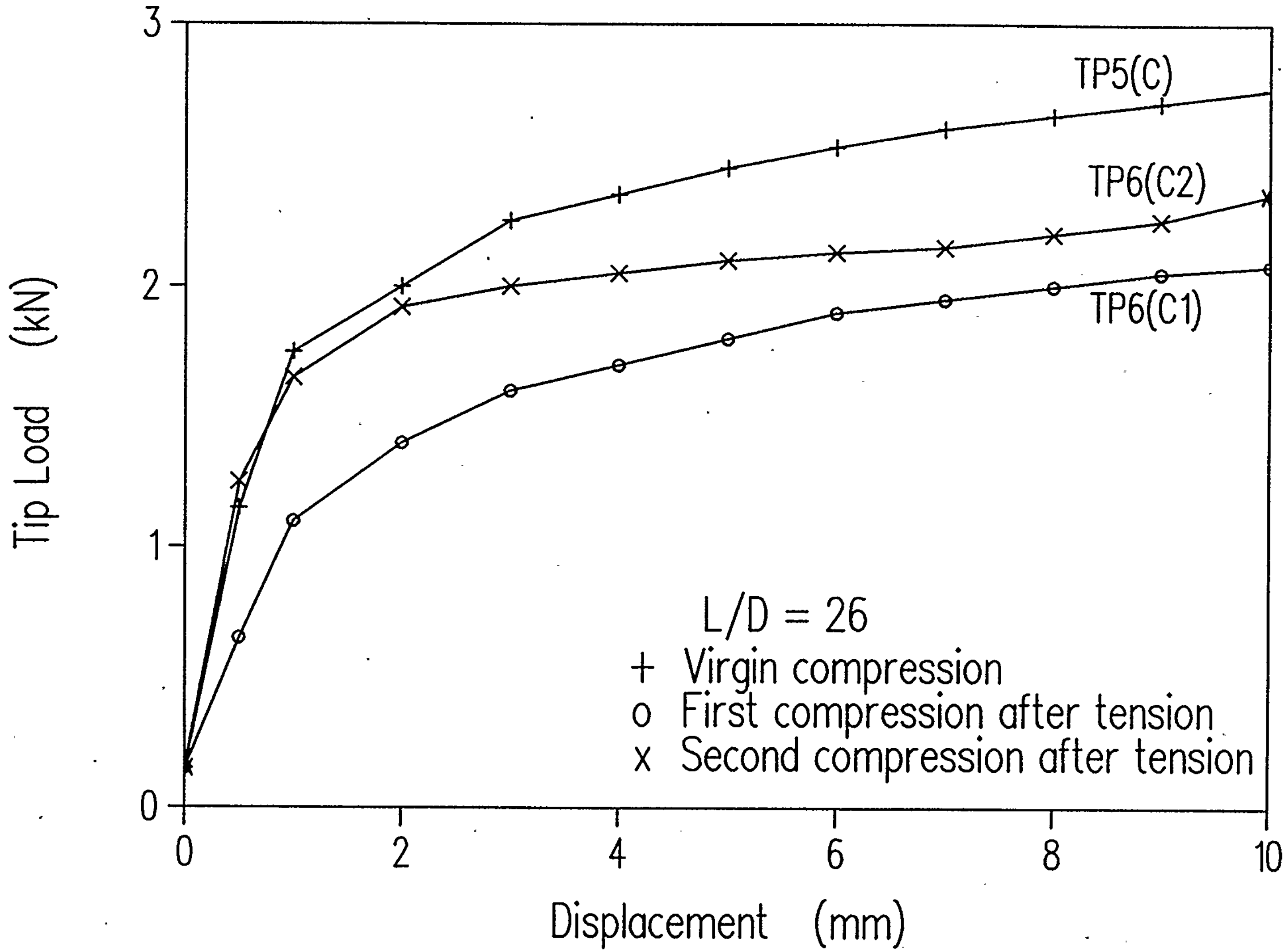


Figure 4.7 Tip Load - Displacement Plots for Piles (L/D = 26) in Medium Dense Sands

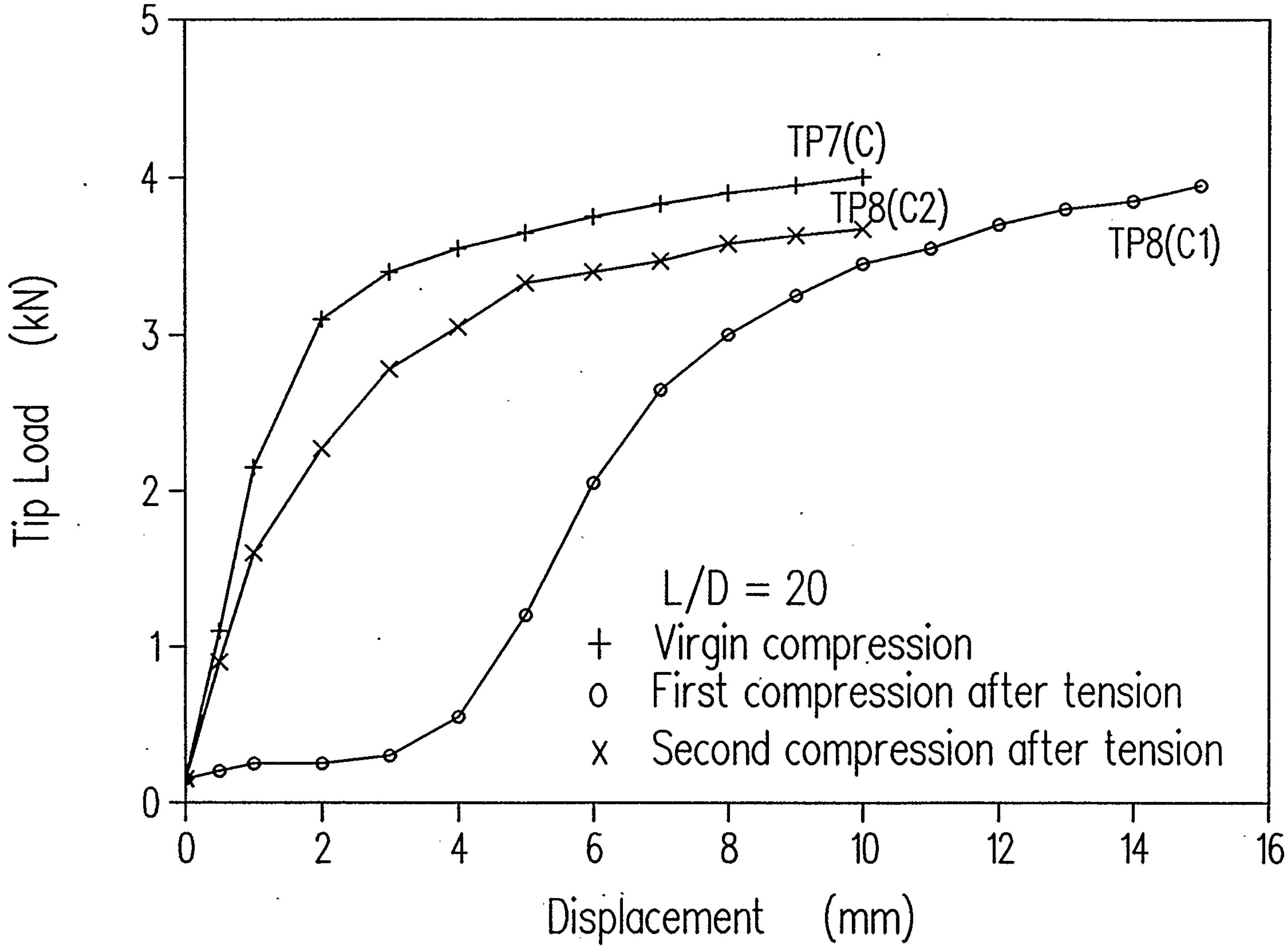


Figure 4.8 Tip Load - Displacement Plots for Piles (L/D = 20) in Dense Sands

after a tension test, the tip load was generally found to be mobilized at around 5 mm displacement or at about 10 % of the pile diameter. Table 4.2 shows the tip load and shaft load mobilized at the settlement corresponding to mean ultimate load. The tip load mobilized for piles having loading history was in the range of 78 to 94 % of tip load for piles under virgin compression, for dense sands. This range was about 91 % to 94 % of the virgin capacity, for piles under second compression as compared to 78 to 82 % of the virgin capacity for piles in first compression after tension. In the case of piles in first compression after a tension loading the pile tip did not seem to carry much load until after 4-5 mm of displacement. The stepped nature of the total load versus displacement curve was due to the lack of mobilization of the tip load for the first 5 mm of displacement. This may be due to the creation of a small loose pocket of loose sand just below the pile tip when a pile is loaded in tension. This loose sand pocket was likely to be the cause for some initial displacement of the pile necessary before the full tip load could be mobilized. Figure 4.9 gives a sketch of the formation of loose pocket of sand below the pile tip for a pile that has already failed in tension.

Furthermore, the variation in the tip load might not only be due to the loosening of the sand below the pile tip but also disturbances caused in the sand by tension loading as well as by the movement of sand beneath and around the pile tip. The exact state of affairs of the soil just below the pile tip after the pile has been loaded is not known and need to be studied.

The settlement corresponding to the mean failure load for piles in medium

TABLE 4.2

TIP AND SHAFT CAPACITY OF PILES HAVING LOADING HISTORY

	TIP CAPACITY (kN)				SHAFT CAPACITY (kN)				
NAME	MEAS U-RED	MEYER H-OF (1976)	POULOS & DAVIS (1980)	TOMLIN SON (1986)	MEASU- RED	BROMS (1966)	MEYER H-OF (1976)	POULOS & DAVIS (1980)	TOMLINS ON (1986)
TP1(C)	3.32	7.5	3.6	2.9	2.2	1.66	1.84	4.28	11.98
TP2(C2)	3.03				1.74				
TP2(C1)	2.64				1.28				
TP3(C)	3.3	7.5	3.6	2.93	1.68	1.3	1.45	2.96	9.44
TP4(C2)	3.1				1.51				
TP4(C1)	2.65				1.15				
TP5(C)		3.8	2.1	1.95		1.28	1.43	2.36	3.94
TP6(C2)	2.14				0.83				
TP6(C1)	1.98				0.83				
TP7(C)	3.86	7.5	3.6	2.95	1	1	1.11	1.98	7.26
TP8(C2)	3.52				0.68				
TP8(C1)	3.17				0.6				

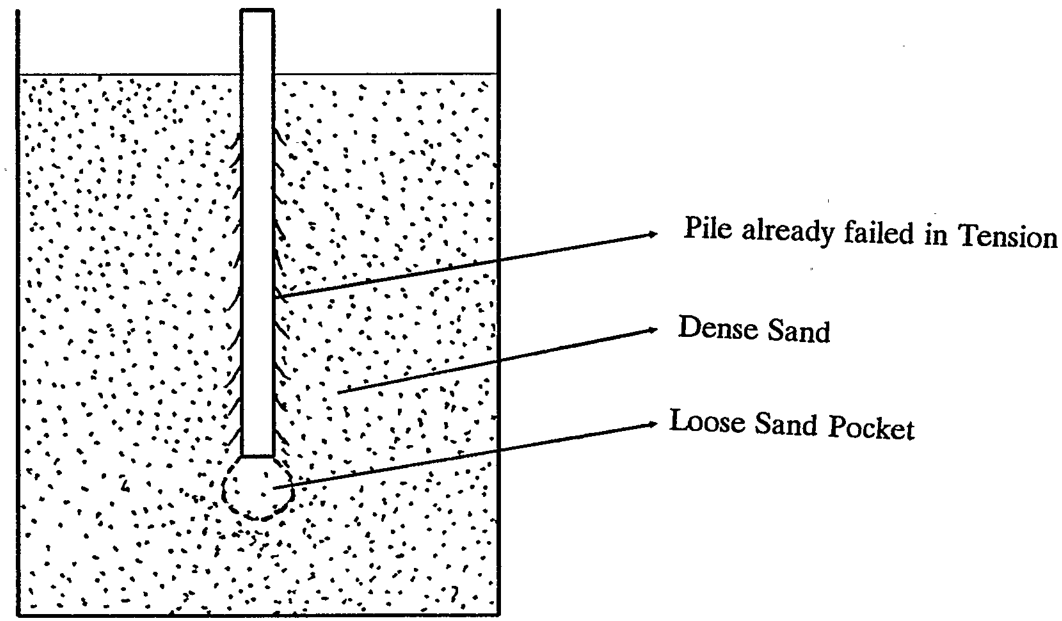


Figure 4.9 Formation of Loose Pocket Under the Pile Tip due to Prior Loading of Pile in Tension

dense sand (Figure 4.3) could not be evaluated as the failure load was higher than the load to which the pile was carried. Due to this, the variation range for tip loads for piles im medium dense sands having loading history was not known. For pile tested in medium dense sand the step shaped curve was not observed. The tip load - displacement curve obtained was similar to the virgin compression curve. For piles having loading history Koreck and Schwarz (1988) have also not obtained a step load settlement curve. This might be because the soil used there had a lower density. Moreover, their tests were field tests where exact soil conditions are not known. The movement of sand particles in the pocket below the tip, in medium dense sand, probably does not cause substantial change in density of the material. Also a larger volume of sand beneath the tip was possibly affected in medium dense sand since arching effects are likely to be minimal here as compared to those in dense sands. In dense sands arching effects would allow filling of cavity with relatively loose sand immediately below the pile tip, when the pile is tested in tension. Pile capacities for L/D ratios of 33, 26, and 20 and densities 16.75 kN/m^3 and 16.40 kN/m^3 have been evaluated by 3 different methods (Table 4.2). Method given by Meyerhof (1976) gives a higher value of the tip resistance than that measured. Tomlinson's (1986) method predicts marginally lower values, whereas, recommendations of Poulos and Davis (1986) predict the tip capacity closest to the measured value for piles driven and tested in medium dense as well as dense sand.

Shaft Resistance : Figs. 4.10 - 4.13 are plots of the shaft resistance versus the

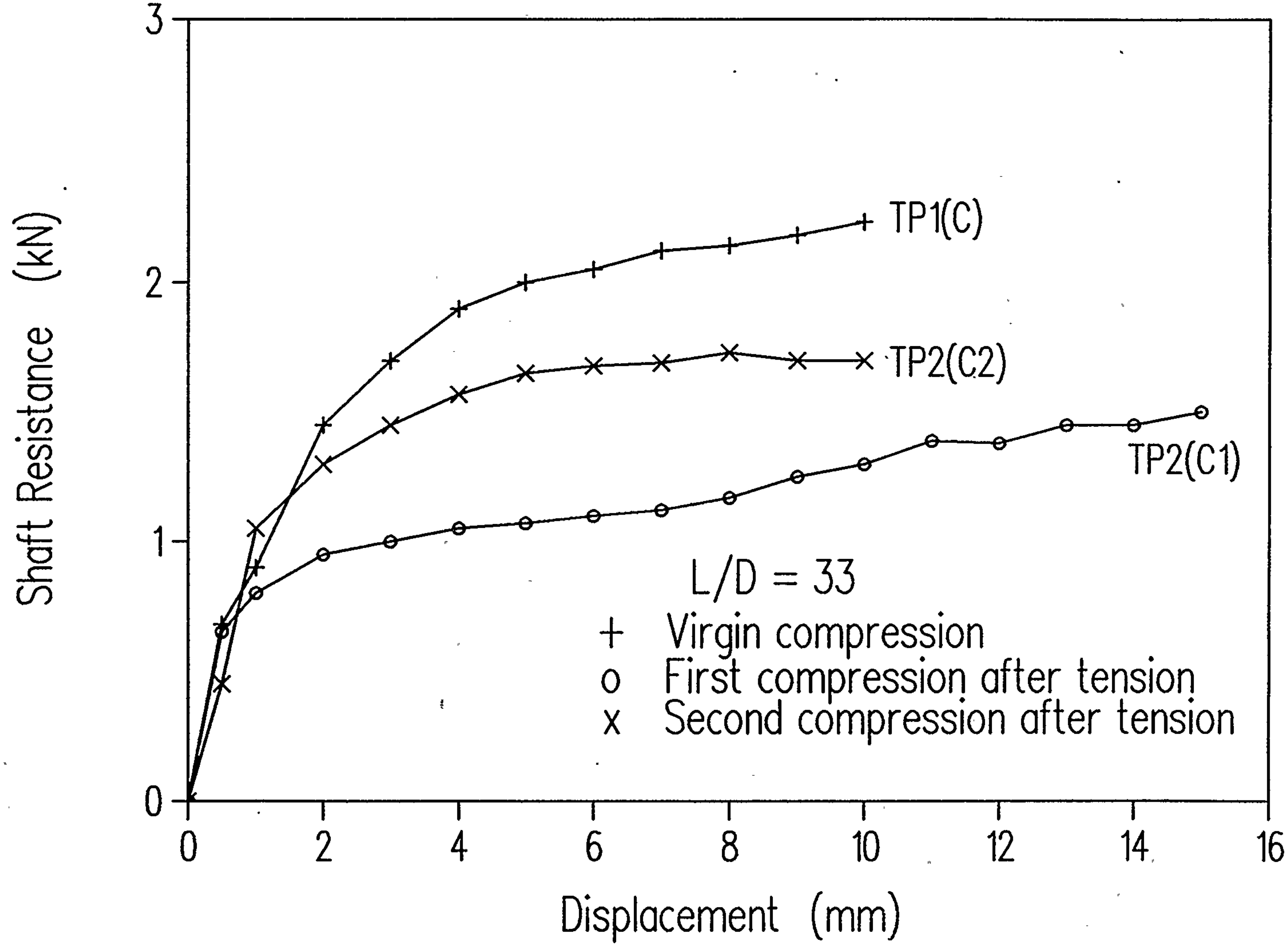


Figure 4.10 Shaft Resistance - Displacement Plots for Piles (L/D = 33) in Dense Sands

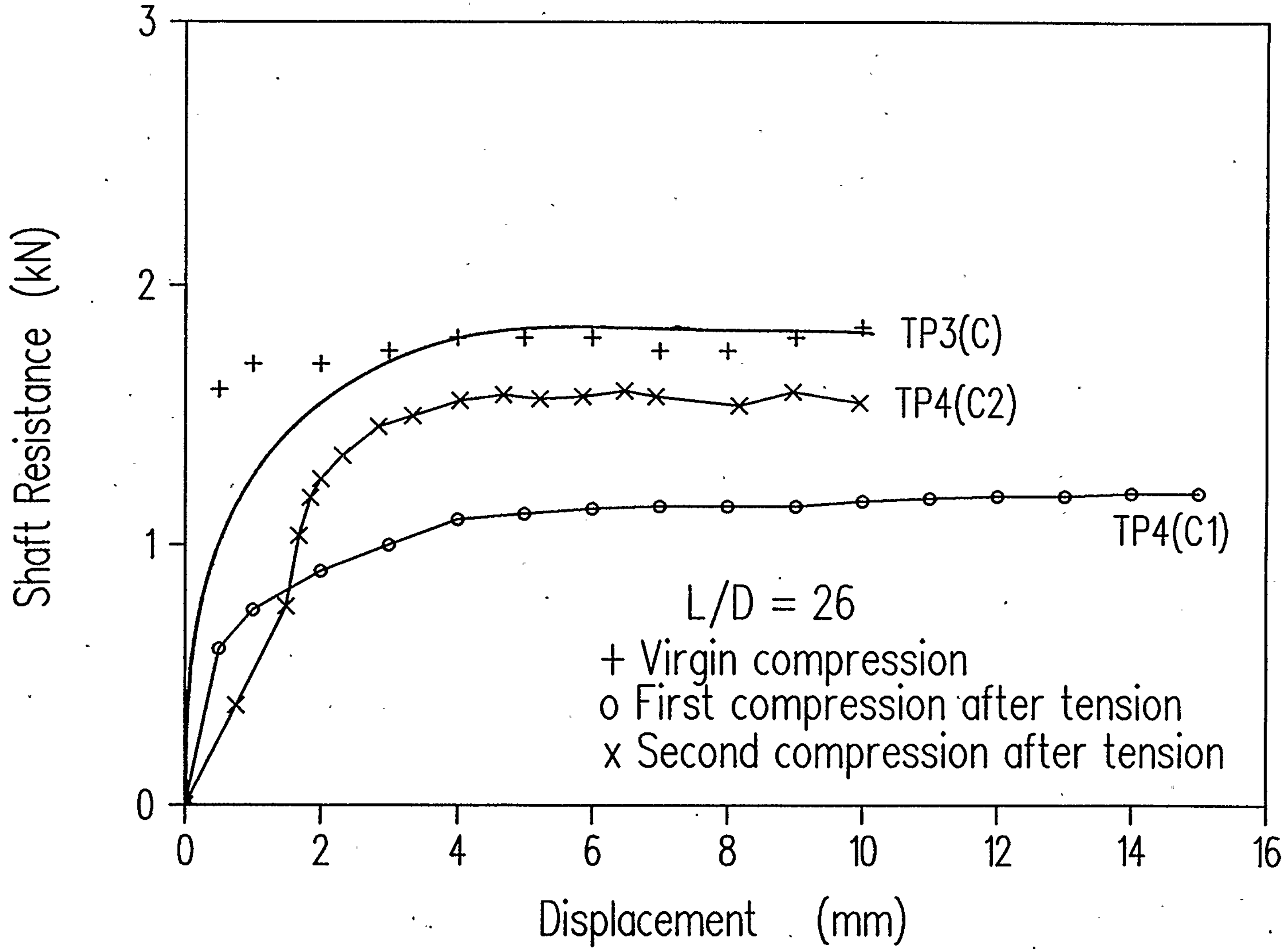


Figure 4.11 Shaft Resistance - Displacement Plots for Piles (L/D = 26) in Dense Sands

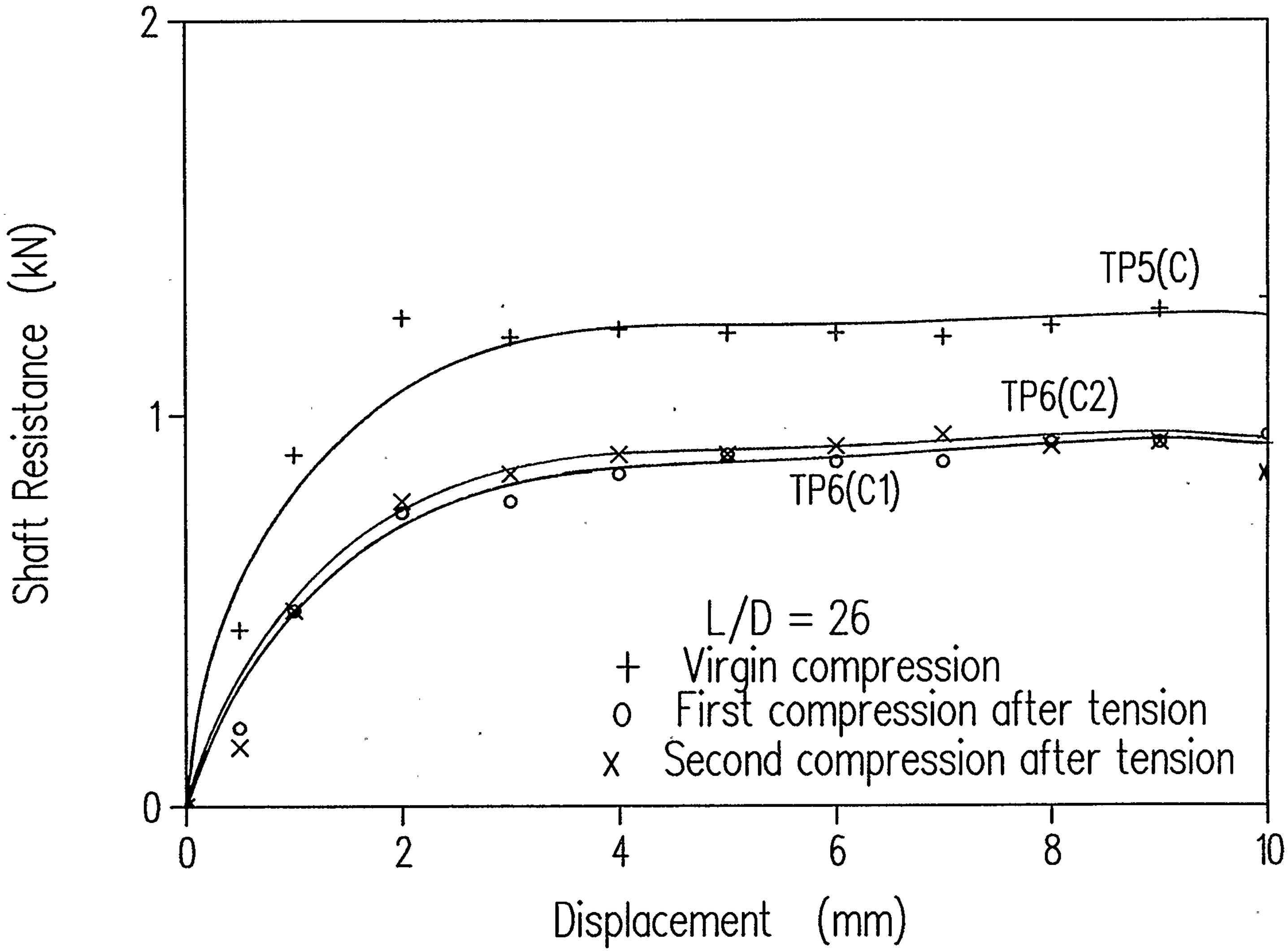


Figure 4.12 Shaft Resistance - Displacement Plots for Piles (L/D = 26) in Medium Dense Sands

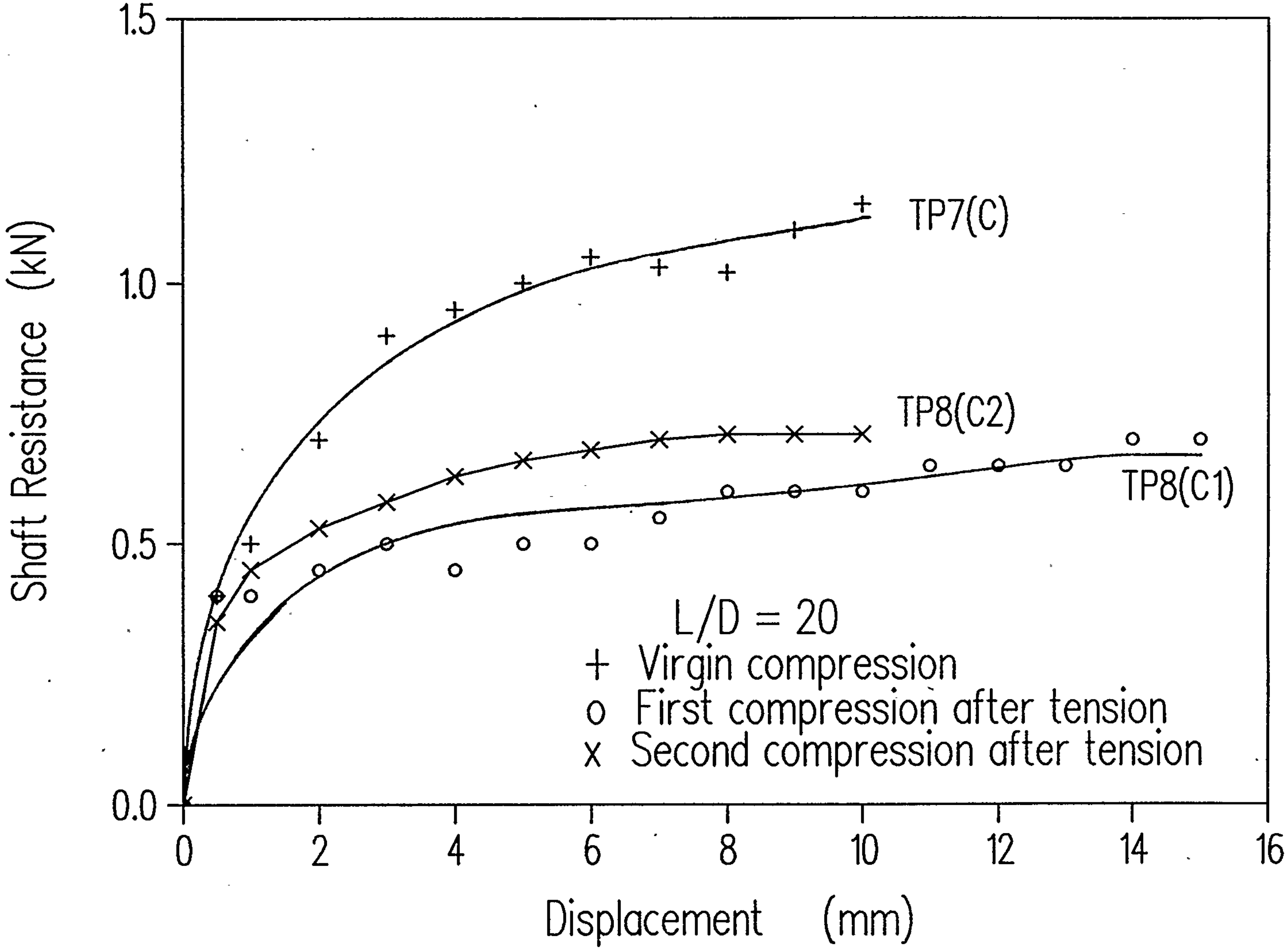


Figure 4.13 Shaft Resistance - Displacement Plots for Piles (L/D = 20) in Dense Sands

pile head displacement for piles having L/D ratios of 33, 26 and 20. Some of the plots for shaft resistance versus displacement have been approximated to curves of best fit (Figure 4.11-4.13). Less shaft resistance was mobilized in case of piles which were pushed back after tension loading (second compression) and even lesser for piles which were not pushed back (first compression) after a tension loading (Figure 4.10-4.13). A wide variation of shaft resistance as compared to tip capacity was observed. The maximum and minimum shaft resistance mobilized for piles having loading history were 90 % and 58 % of the virgin shaft resistance. As a result of prior loading, shaft resistance was much more affected than the tip resistance. This might be explained partially by considering the driving stresses that were locked in a pile. These driving stresses tend to get modified when a pile is loaded. Hence the initial state of piles under virgin compression and piles having a loading history are not identical. This change in the initial condition may result in variations in the pile behaviour (Tan and Hanna 1974; Chan and Hanna 1980). Such a change in the initial state affects the pile soil interface reaction. But due to the pile being rigid the effect of residual stresses may not be that much. Apart from the difference in the initial state of residual stress, another factor that can cause the variation in shaft resistance is the loosening of sand around the pile during a tension test. During the tension test the pile undergoes axial tensile strain. These tensile forces create tensile strains on the soil at the pile soil interface thus reducing the mean effective stress. This will cause a reduction in the pile capacity after a tension failure. The increase in the capacity after one compression subsequent to tension test is due to the densification of the soil due to the compression or the release of some of the axial tensile strain induced during a tensile test. The second compression, hence, showed a higher shaft resistance. This effect was predominant in high density sand, and became negligible for medium dense sand. This was because in medium dense sands the effect due to loading history on the sand density around the pile was less. The axial tensile strain induced at the pile soil interface due to tension loading is less.

The shaft resistances for L/D ratios of 33, 26 and 20 and densities of 16.75 kN/m^3 and 16.40 kN/m^3 have been predicted in Table 4.2 as per the recommendations of Meyerhof (1976), Broms (1966) (using the ß method), Poulos and Davis (1980) and Tomlinson (1986). Method given by Meyerhof (1976) assuming $K_s = 1.8\ K_o$ and $\delta = 0.8\ \phi'$ (maximum values as per Das (1984)) gave values of shaft resistance close to the measured values. A comparison of the measured values and those obtained from the tests in this study show that both Meyerhof (1976) and Brom's (1966) method gave good estimates of the shaft resistance at lower L/D ratios but at higher L/D ratio they both seemed to under predict the ultimate shaft resistance. Poulos and Davis (1980) and Tomlinson (1986) seemed to overpredict the shaft resistance values. To be on the safer side, Meyerhof's (1976) method should be used for determining the shaft resistance of the pile.

Tension Test

As no tip load is generated in tension tests the total load is the same as shaft resistance. The total load versus pile head displacement for piles in dense sands at L/D ratios of 33, 26 and 20 are shown in Figs. 4.14, 4.15 and 4.17 respectively. Fig.

4.16 gives the total load against pile head displacement as obtained for L/D = 26 for medium dense sand.

Methods recommended for evaluation of ultimate load of piles are for compressive loads on piles. However, here these standard procedures were used for piles in tension. The pile failure load obtained by various standard procedures applicable for the constant rate of penetration tests the mean failure load and the corresponding failure displacement are presented in Table 4.3.

A significant decrease in the failure load was obtained when piles tested in virgin tension was compared to those which have a loading history. Sparrow (1988) and Koreck and Schwarz (1988) had reported a significant decrease in the ultimate load of piles in tension that have prior loading history.

TABLE 4.3

TENSILE FAILURE LOAD (kN) OF PILES WITH LOADING HISTORY

NAME	ST	DT	MAZ	CHIN	BH90	BH80	VAN	MEAN (kN)	SETT (mm)
TP2(T)	1.38	1.35	1.43	1.53	1.38	1.46	1.45	1.42	8.40
TP1(T)	0.91	0.91	0.98	0.98	0.96	0.97	0.94	0.95	7.50
TP4(T)	0.83	0.85	0.92	0.98	0.91	0.91	0.87	0.90	8.27
TP3(T)	0.74	0.74	0.77	0.80	0.75	0.76	0.78	0.76	7.50
TP6(T)	0.80	0.81	0.85	0.86	0.82	0.84	0.84	0.83	6.81
TP5(T)	0.51	0.52	0.60	0.67	0.58	0.61	0.63	0.59	7.19
TP8(T)	0.50	0.50	0.50	0.50	0.48	0.58	0.51	0.51	9.63
TP7(T)	0.27	0.27	0.28	0.29	0.27	0.27	0.27	0.27	7.00

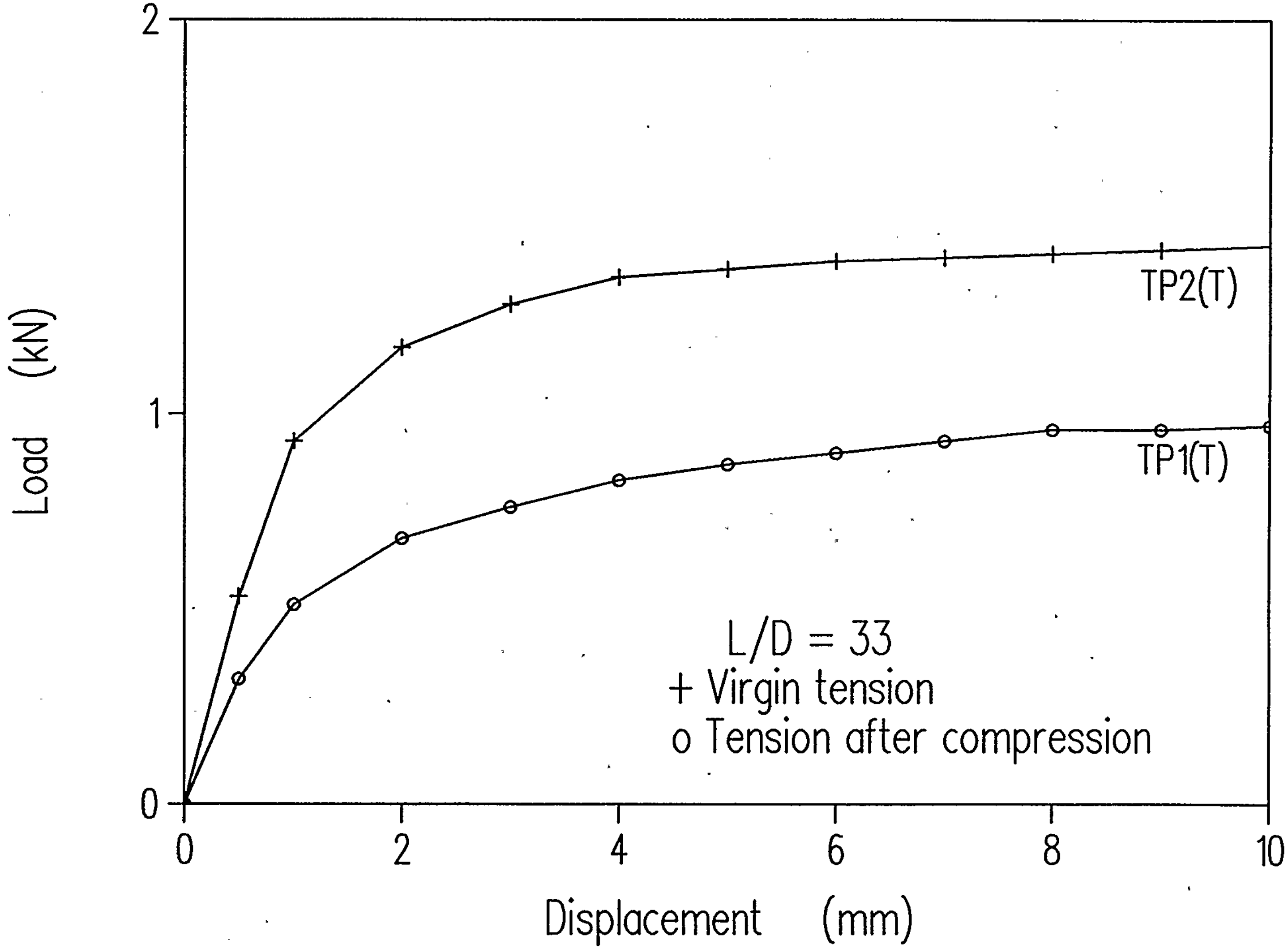


Figure 4.14 Tension Load - Displacement Plots for Piles (L/D = 33) in Dense Sands

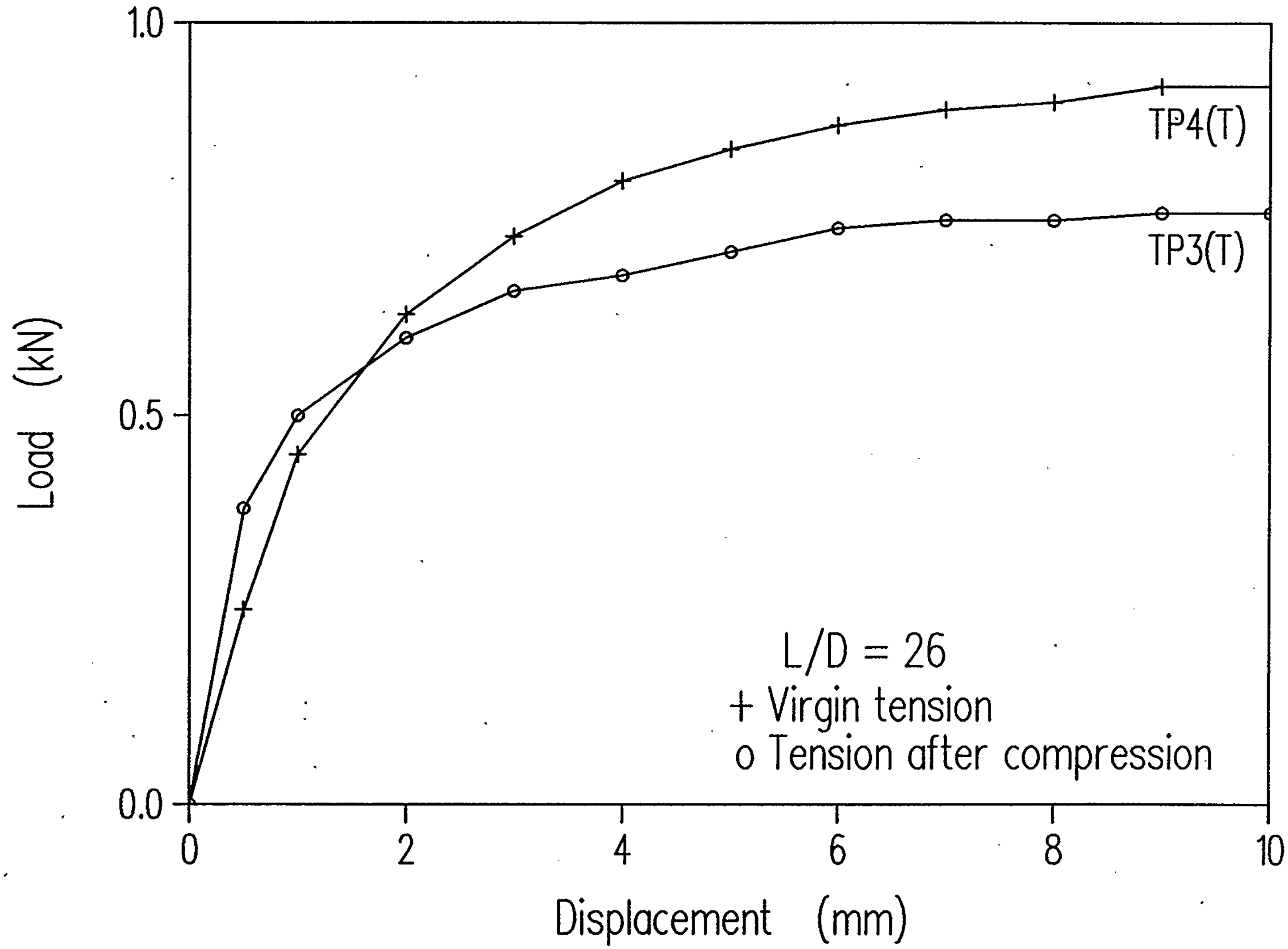


Figure 4.15 Tension Load - Displacement Plots for Piles (L/D = 26) in Dense Sands

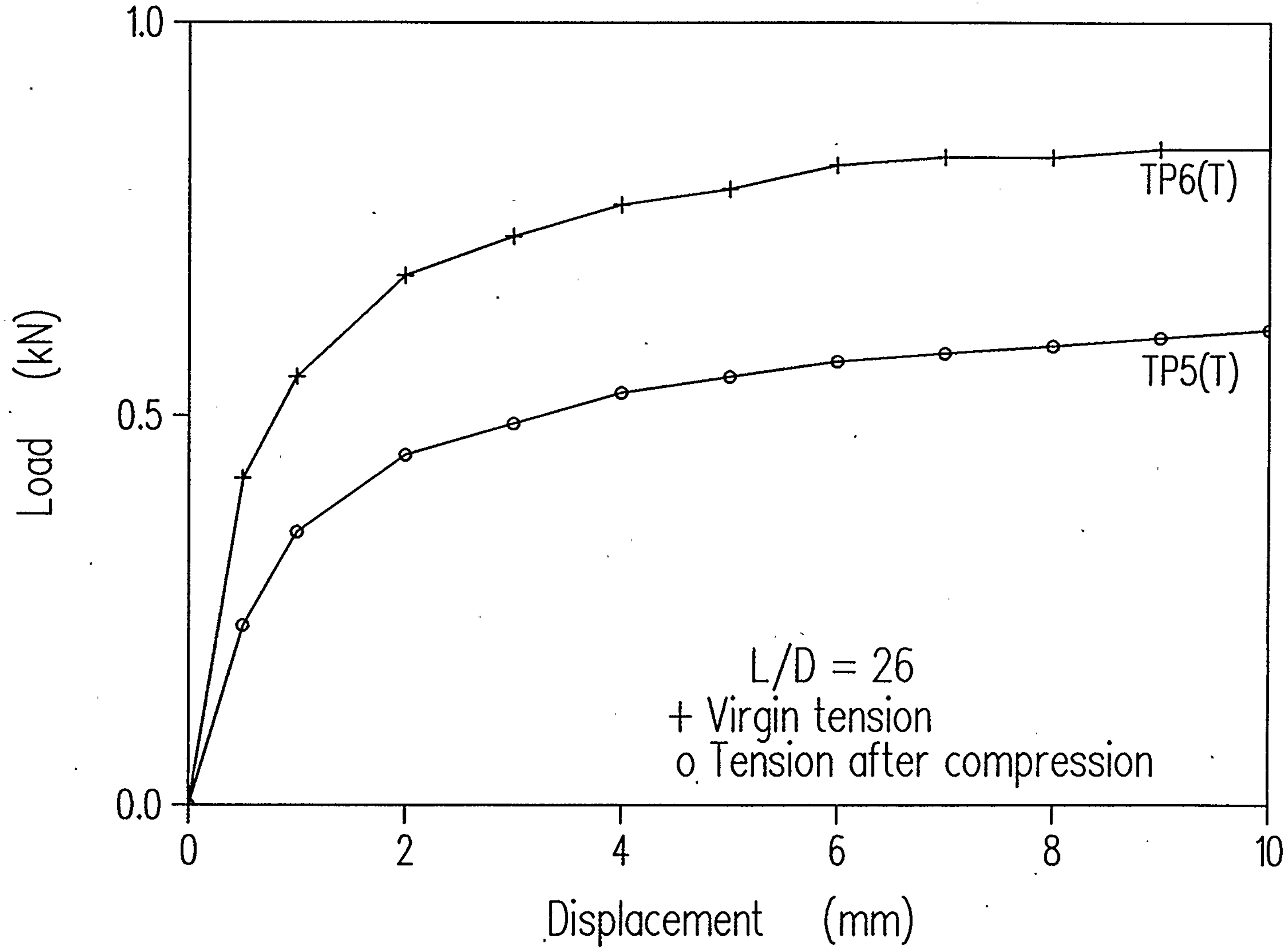


Figure 4.16 Tension Load - Displacement Plots for Piles (L/D = 26) in Medium Dense Sands

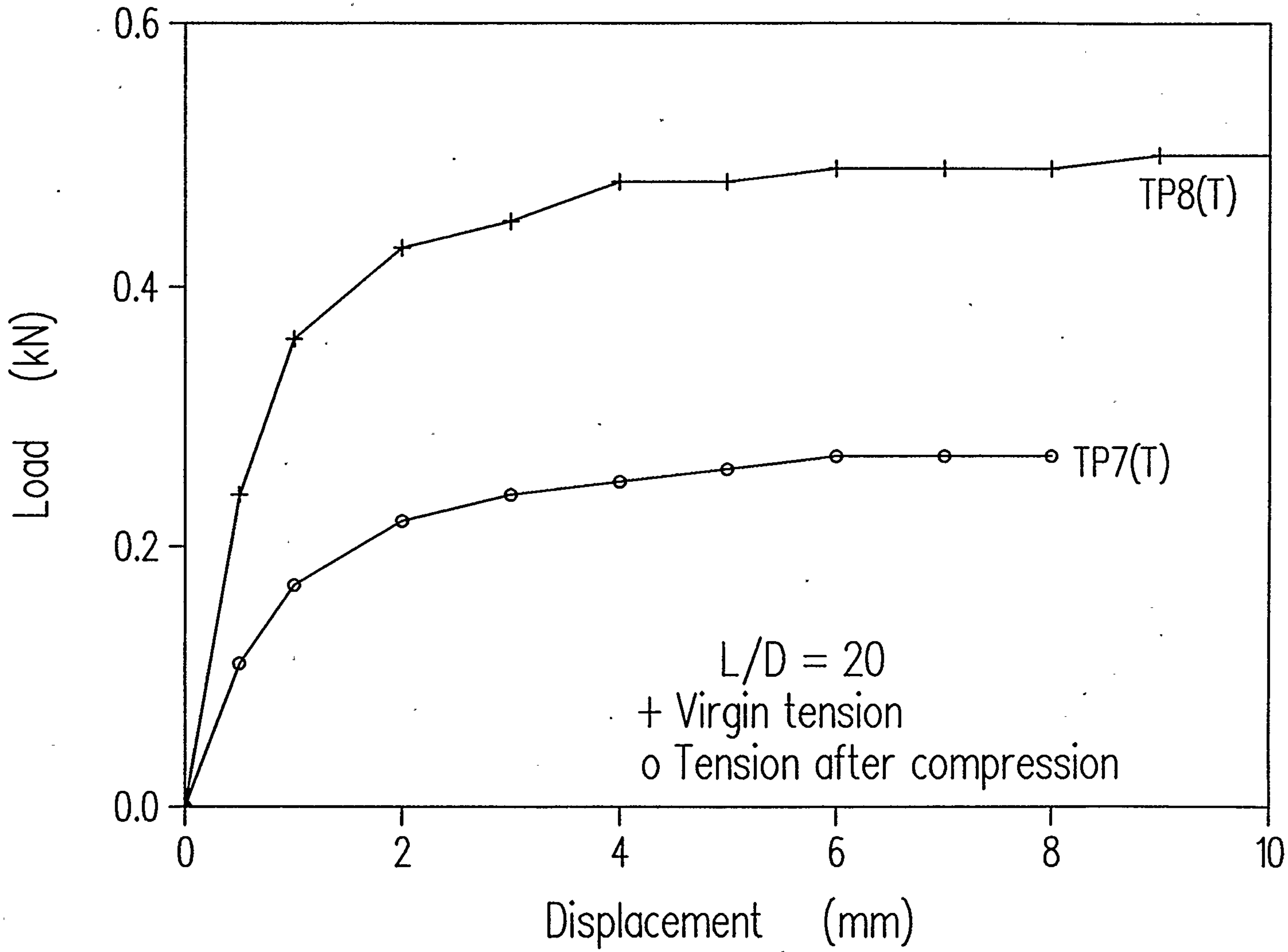


Figure 4.17 Tension Load - Displacement Plots for Piles (L/D = 20) in Dense Sands

Note:

T = Piles in tension	ST = Single Tangent
BH90 = Brinch-Hansen's 90 % criteria	DT = Double Tangent
BH80 = Brinch-Hansen's 80 % criteria	MAZ = Mazurkeiwicz's method
VAN = Van der Veen's method	CHIN = Chin's method
MEAN = Mean failure load	SETT = Settlement at mean failure load

Here the percentage decrease in mean failure load was more in the case of piles having a L/D ratio of 20 than in the case of piles with L/D ratio of 33. The decrease in the mean failure load obtained was in the range of 16 % to 47.5 % of the virgin tension load. Results of Koreck and Schwarz (1988) show a decrease of about 24 % for a pile with L/D ratio of 38.5.

Table 4.4 compares the value of tensile failure load obtained by three different methods mentioned in literature. As compared to the measured values Das's (1984) method predicted values which are less than 50 % of the measured values. Tejchman (1971) method predicts values of pile tensile capacity very close to Das's value. Thus, both Das's and Tejchman's method underpredict the tensile capacity. Good prediction is obtained by Coyle and Sulaiman's (1967) method, though the prediction of this method is slightly higher than those measured.

The settlement at mean failure load ranges between 13 % to 19 % of the pile diameter. From the load displacement plots reported by Koreck and Schwarz (1988) it was concluded that failure occurs at a movement around 17 % of the pile diameter for piles in virgin tension and 19 % for piles having a previous loading history.

TABLE 4.4

COMPARISON OF TENSILE CAPACITY OF PILES

Name	Measured	Das (1984)	Tejchman (1971)	Coyle and Sulaiman (1967)
TP2(T)	1.42	0.48	0.56	1.52
TP4(T)	0.90	0.35	0.35	0.94
TP6(T)	0.83	0.22	0.34	0.86
TP8(T)	0.50	0.24	0.21	0.56

The shaft resistances in compression and tension can be compared in Tables 4.2 and 4.3. Mohan et. al (1963) and Hunter and Davisson (1969) have shown the tension shaft resistance as being significantly lower than compression shaft resistance. It has been suggested (Poulos and Davis 1980; Kaniraj 1988) that the ratio of shaft resistance in tension to that in compression as generally in the range of 0.50 to 0.67. The previous version of API RP2A design guidelines distinguished between the pile shaft capacity for tensile and compressive piles. The tensile shaft capacity was typically around 70 % of the compressional shaft capacity of the piles. However, Olson and Dennis (1982) on the basis of field data concluded that no systematic difference between the tensile and compressive shaft capacity exist. In light of these data the present revised American Petroleum Institute guidelines (Randolph 1985) suggests that there be no distinction in the shaft capacity of piles in tension and compression. Vesic (1970) also suggests that no significant difference exist between the pile shaft capacity in tension and in compression. The present study indicated that the ratio of the tension shaft resistance to the compression shaft resistance is in the range of 51 % for L/D = 20 to 64.5 % for L/D = 33. There is one main aspect of tension loading that leads to lower shaft resistances than for compression loading. It is the probable reduction in the mean stress level caused by applying an upward load on the pile. It means that axial tensile straining of the pile shaft leads to corresponding axial strains in the soil resulting in a decrease of the effective stress.

4.1.2 Cyclic or Repeated Loading

Repeated loading of piles embedded in soil causes soil degradation and affect the pile performance. An attempt was made to determine the behaviour of piles in dry sand subjected to repeated tensile loads upto which piles can be loaded without the soil undergoing significant degradation.

The number of repetitions versus displacement expressed as a percentage of the pile diameter for piles subjected to repeated loads at 0.1 Hz are presented in Figures 4.18, 4.19 and 4.20 for piles with L/D ratios of 33, 26 and 20 respectively. A comparison of the responses of the pile when cyclic loads were applied at frequencies of 0.1 Hz and 0.05 Hz show that the number of cycles required for a fixed displacement of piles at higher load ranges was almost identical. The behaviour of the pile changed at lower load ranges. At lower load ranges it was observed that for a particular displacement a higher number of cycles is required, if the frequency of loading is lower. Moreover, the slope of the plot changed sharply and became steeper for load ranges varying between 0 to 33 % and 0 to 20 % of the tensile failure load.

Figure 4.18, 4.19 and 4.20 show that L/D ratio had less effect on the number

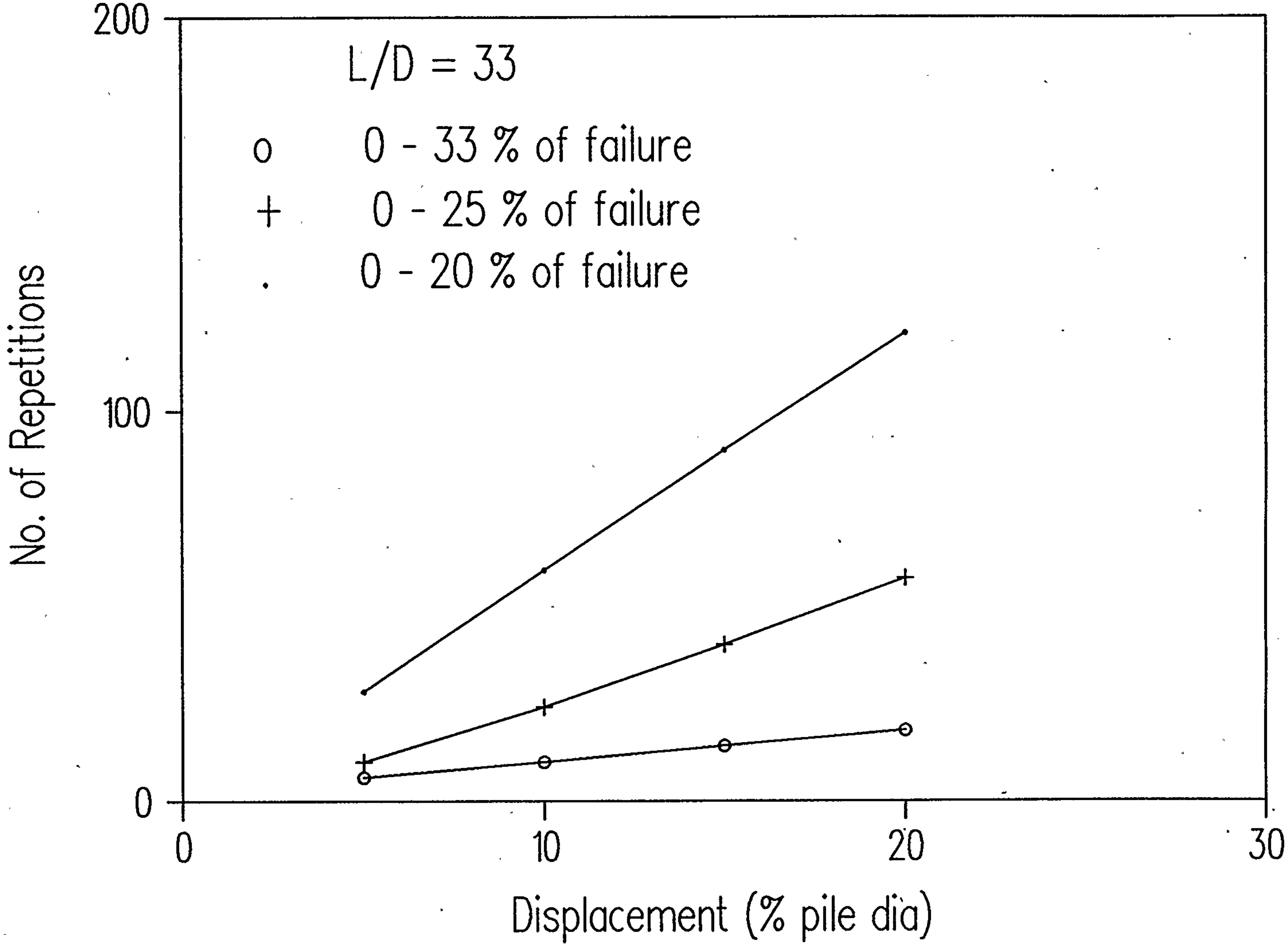


Figure 4.18 Displacement - Number of Repetitions Plot for Piles with L/D = 33

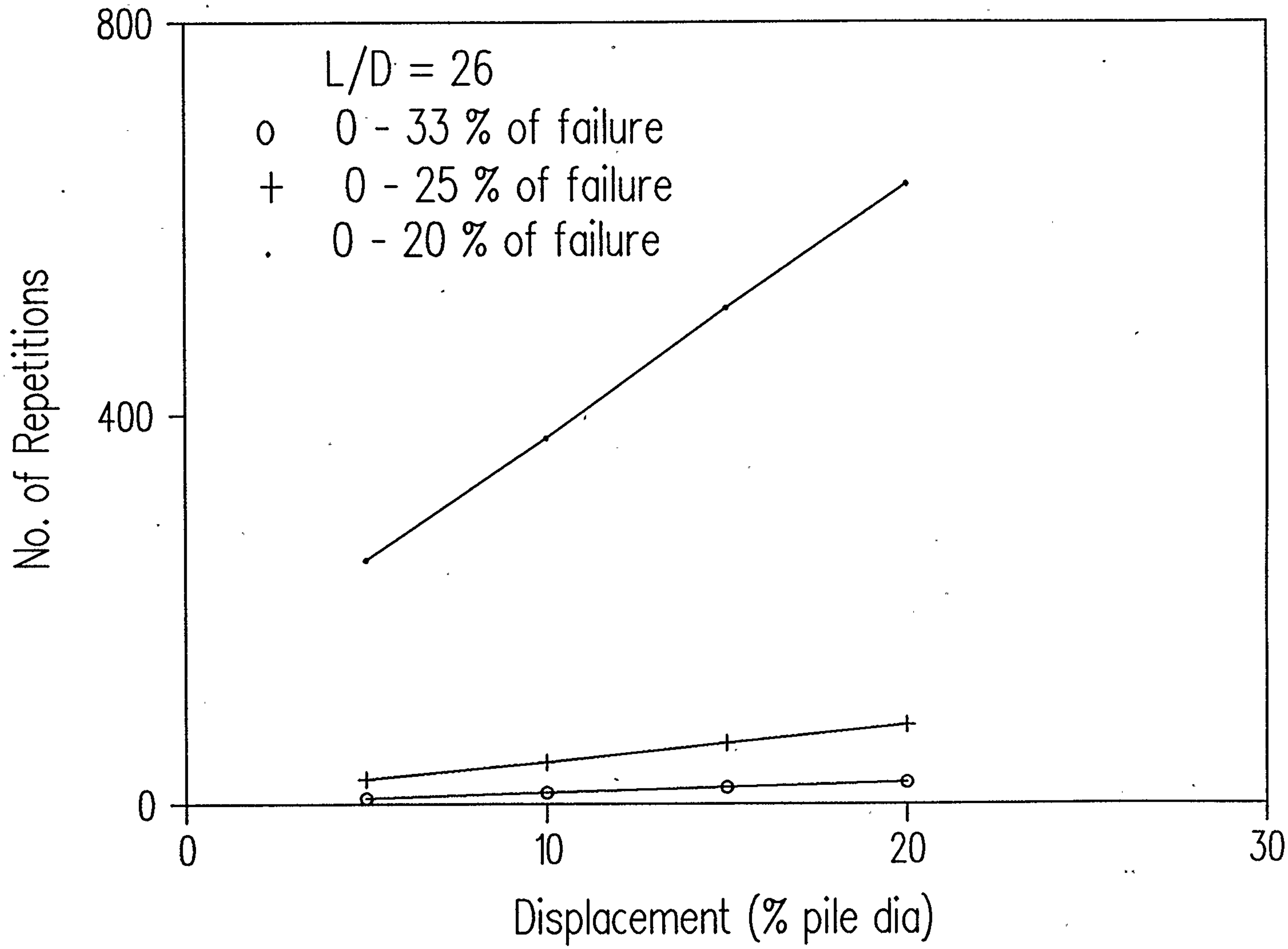


Figure 4.19 Displacement - Number of Repetitions Plot for Piles with L/D = 26

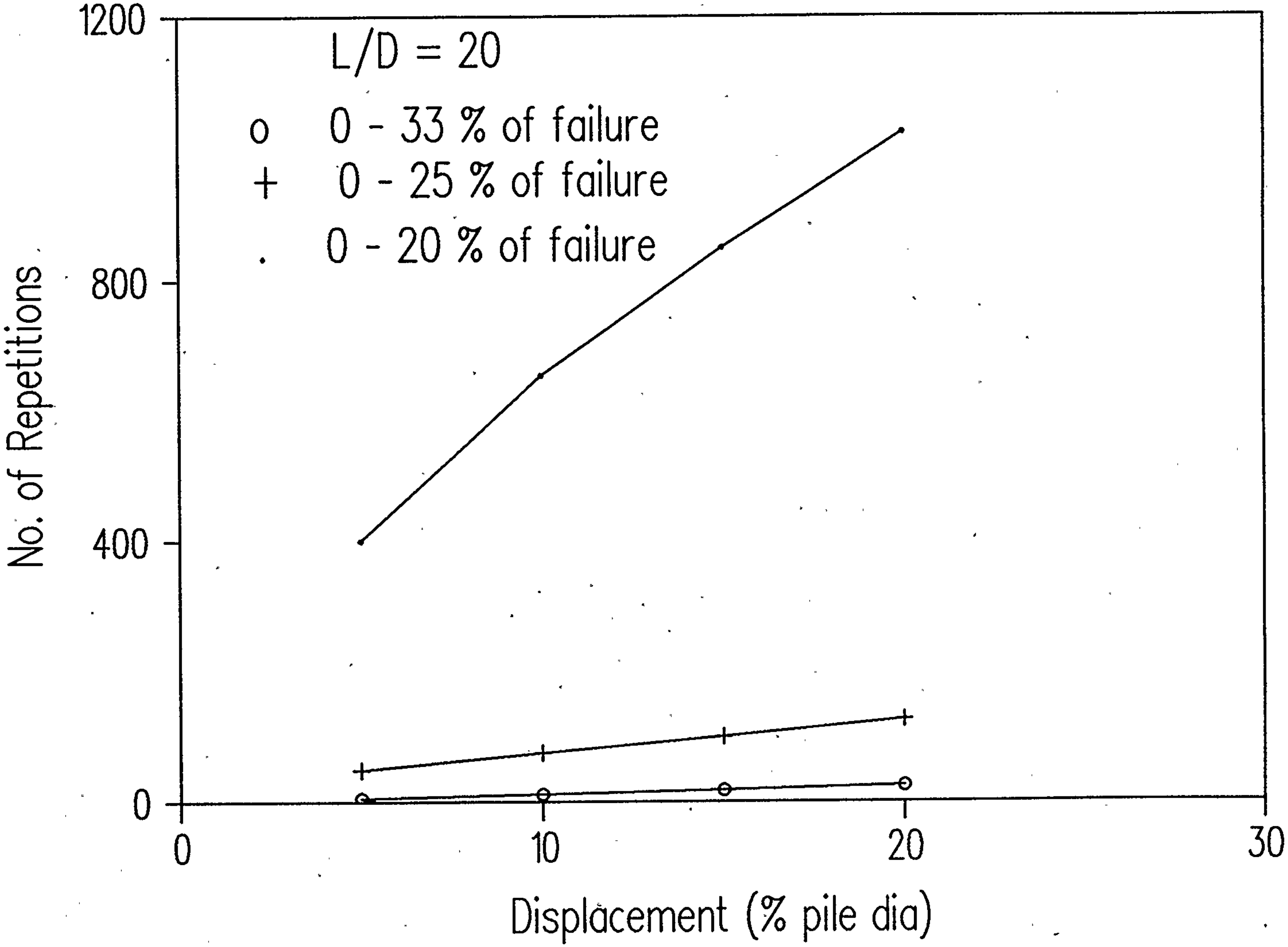


Figure 4.20 Displacement - Number of Repetitions Plot for Piles with L/D = 20

of cycles - displacement plots for a cyclic load range between 0 to 33 % of the tensile failure load. However, at lower load ranges of 0 to 20 % and 0 to 25 % the number of repetitions required for a particular displacement increases quite significantly as the L/D ratio decreases. A plot of number of repetitions against factor of safety or load factor is presented in Figure 4.21. It was observed from this figure that at higher load factor or factor of safety the effect of L/D increased significantly. At lower load factors for the same displacement all piles had to undergo almost same number of cycles.

While conducting the test it was observed that at very low load ranges of 0 to 10 % (for L/D = 33) and 0 to 12.5 % (for L/D ratios of 26 and 20) of the tension failure load the pile movement was negligibly small (less than 0.5 mm) even after 2000 cycles and did not seem to increase. However, at higher load ranges the pile had a tendency to steadily move upwards. This transition phase from negligible movement to steady pull out occurs in the range beyond 0 to 10 % of the tensile failure load. Chan and Hanna (1980) conducted compressive cyclic load tests of upto 10 % of the static capacity. It was observed that even for that range, after about 45,000 cycles the sand showed signs of degradation and pile pullout. Chan and Hanna (1980) and Gudehas and Hettler (1981) reported that significant degradation of sand occur when repeated loading is applied on piles. This may result in significant loss of resistance and steady pullout of the pile. This decrease in strength of dry sand might be due to rearrangement of sand particles in a direction parallel to the direction of shear strain, the partial destruction of interparticle bonds and due to grain crushing (Chan and

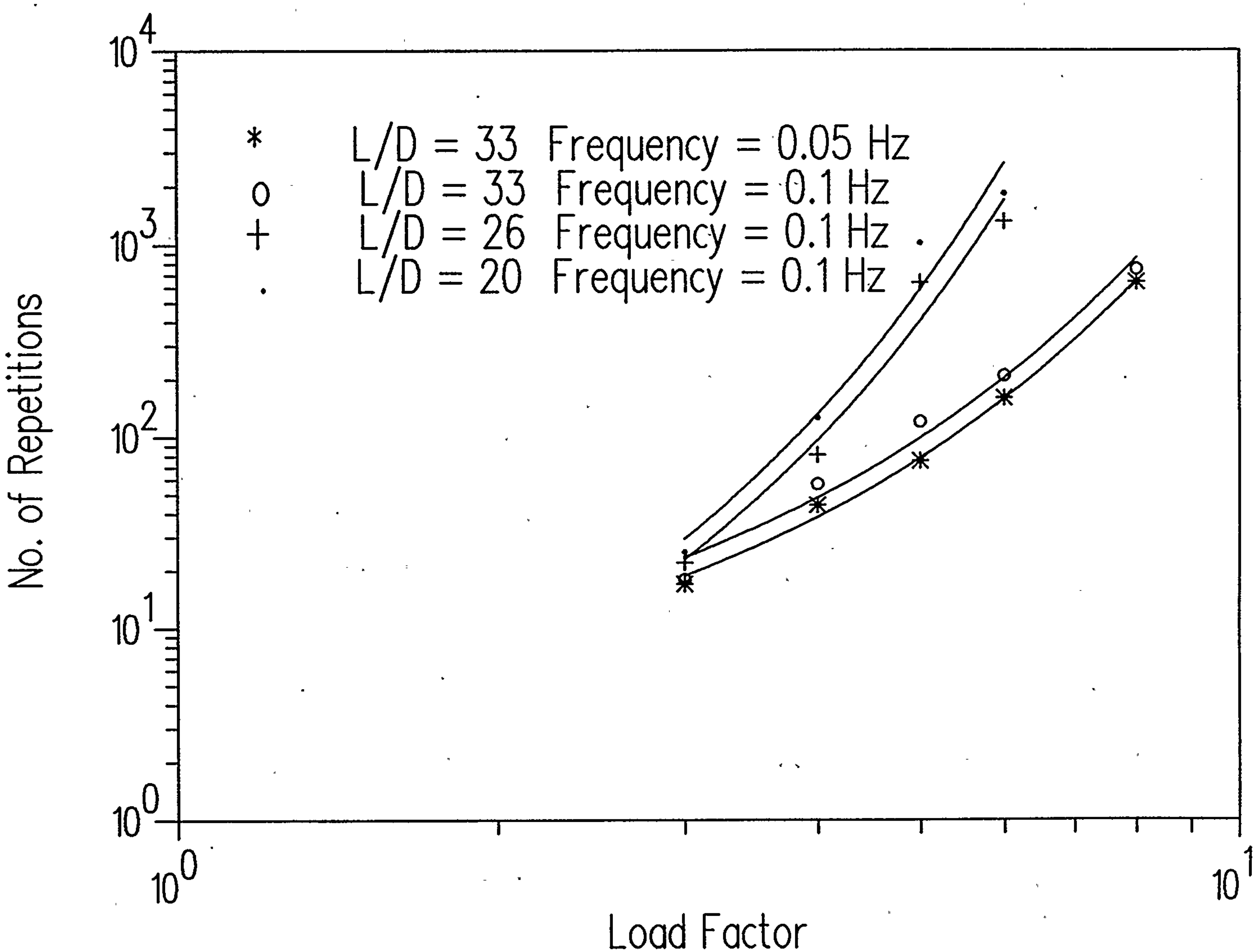


Figure 4.21 Number of Repetitions versus Load Factor

Hanna 1980; Kraft 1990).

The results of this study indicate that the pile pullout steadily increased with the number of repetitions but only when the load range was higher than 10 % of the tension failure load. It might therefore be concluded that in dry sand even in its dense state (relative density = 73.8 %) the resistance for tension failure under cyclic loads was low and that in the field if repeated loads are applied, then these should be limited to a maximum of 10 % of the tensile failure load to avoid steady pullout of the pile. This seems to agree with Broms's (1972) conclusion that below a certain critical load piles can resist an infinite number of load cycles without failure. For lower L/D ratios like 26 and below, this limit may be increased to 12.5 % of the tension failure load.

4.1.3 Surcharge Loading

Compressive Tests

Total Load : The effect of surcharge loading on the soil around the piles was studied at pressures of 34.5 kPa (5 psi), 69 kPa (10 psi), 103.5 kPa (15 psi) and 138 kPa (20 psi). Figure 4.22 gives a plot of displacement against total load for different pressures. A comparative study of the load displacement curves for the four pressures could be made.

Table 4.5 gives an evaluation of the compression failure load under surcharge loading.

TABLE 4.5

COMPRESSIVE FAILURE LOAD UNDER SURCHARGE LOADING

SURCHARGE kPa (psi)	ST	DT	MAZ	CHIN	BH90	BH80	VAN	MEAN (kN)	SETT (mm)
34.5 (5)	4.80	4.85	5.55	5.88	5.40	5.51	5.60	5.37	11.70
69.0 (10)	3.36	3.50	3.90	4.07	3.60	3.73	4.00	3.74	11.05
103.5 (15)	3.62	3.70	4.40	4.80	4.20	4.23	4.20	4.16	10.2
138 (20)	4.00	4.12	4.90	5.00	4.50	4.50	4.50	4.50	10.0

Note:

ST = Single Tangent

DT = Double Tangent

MAZ = Mazurkeiwicz's method

CHIN = Chin's method

MEAN = Mean failure load

BH90 = Brinch-Hansen's 90 % criteria

BH80 = Brinch-Hansen's 80 % criteria

VAN = Van der Veen's method

SETT = Settlement at mean failure load

As in the case of loading history, here also the failure load was evaluated by seven different methods available for piles tested under constant rate of penetration. The mean failure load was obtained by the same iterative procedure of obtaining the mean and then discarding those values which are more than 1.5 times or less than 0.5 times this mean value and then obtaining the mean again.

A comparative study of the pile ultimate load showed that the failure load for piles in soil under 34.5 kPa (5 psi) had the highest load of 5.37 kN (Table 4.5). Figure 4.22 shows that after this there was a marked decrease of the failure load for a pressure increase from 34.5 kPa to 69 kPa. At pressures beyond 69 kPa the failure

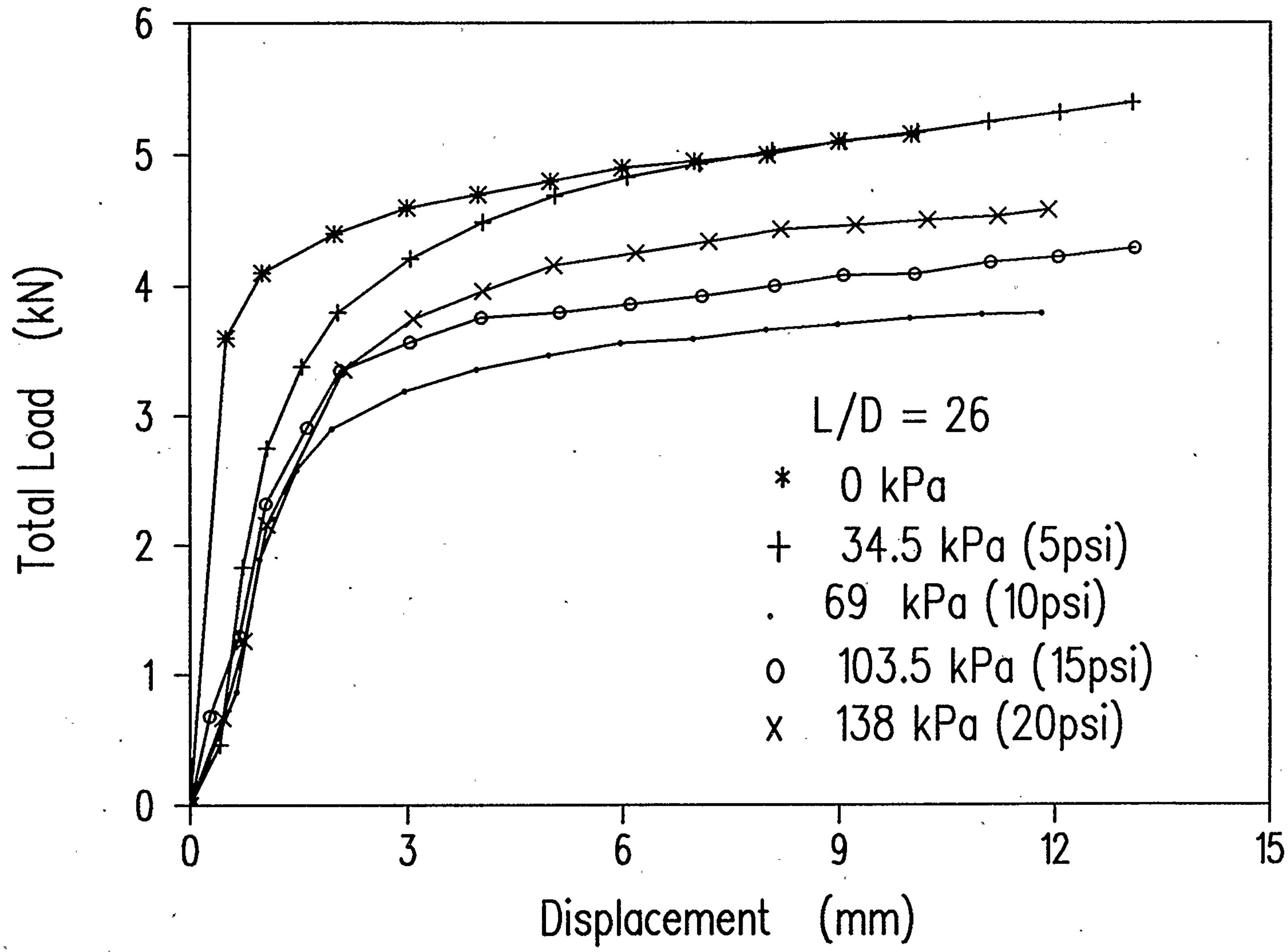


Figure 4.22 Total Load - Displacement Plots for Piles in Soil under Surcharge Loads

load increased slowly. This initial decrease was in the range of 30 % and showed a change in the soil behaviour.

The settlement required to cause failure at surcharge pressures are typically in the range of 20 % to 23 % of the pile diameter. It was therefore concluded that the average settlement required for piles tested in dense sands is higher in the case of surcharge loading by about 6.5 % to 9.5 % of the pile diameter.

Tip Load : Figure 4.23 gives the variation of tip load against displacement. This tip load was the actual load occuring at the tip including the weight of the pile (0.15 kN).

The variation of the tip load explains the anomalous variation of the total load against displacement. From Figure 4.23 it can be concluded that the tip load decreases with an increase in surcharge loading or the overburden pressure. This decrease is only marginal as the pressure increases from 0 kPa to 34.5 kPa (5 psi). As the overburden pressure increases from 34.5 kPa (5 psi) to 69 kPa (10 psi) the tip resistance decreases significantly. As the overburden pressure increases further from 69 kPa (10 psi) to 138 kPa (20 psi) the pile tip resistance decreases though marginally.

In a pile-soil system as the overburden pressure increases the pile experiences negative skin friction. This negative skin friction is very large in the case of soft compressible soils and might cause the pile to settle on its own. Calculations of negative skin friction as per Zeevaert (1983) show that piles can be subjected to downdrag forces of 2.89 kN for an overburden pressure 34.5 kPa, and 9.44 kN for an

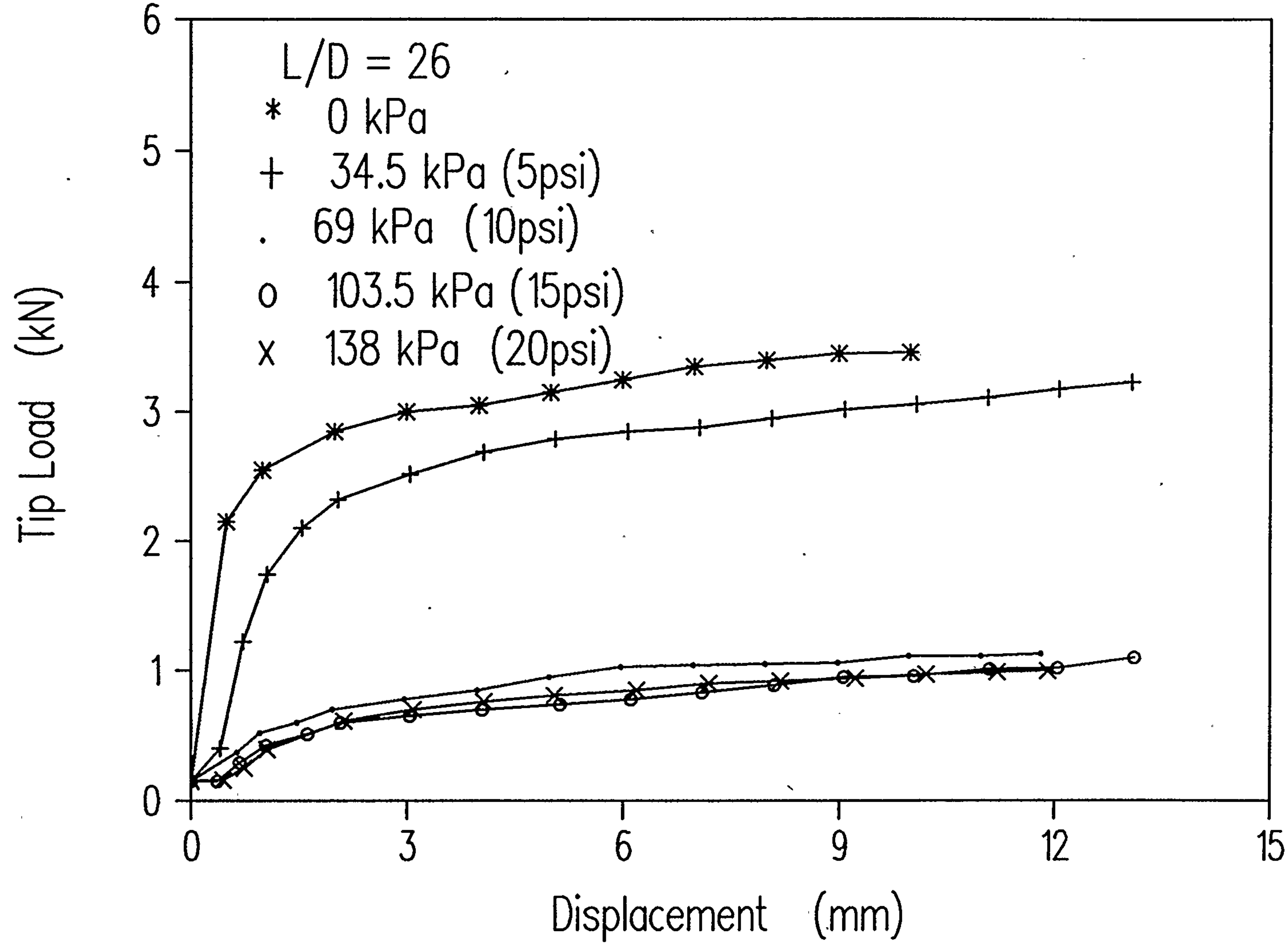


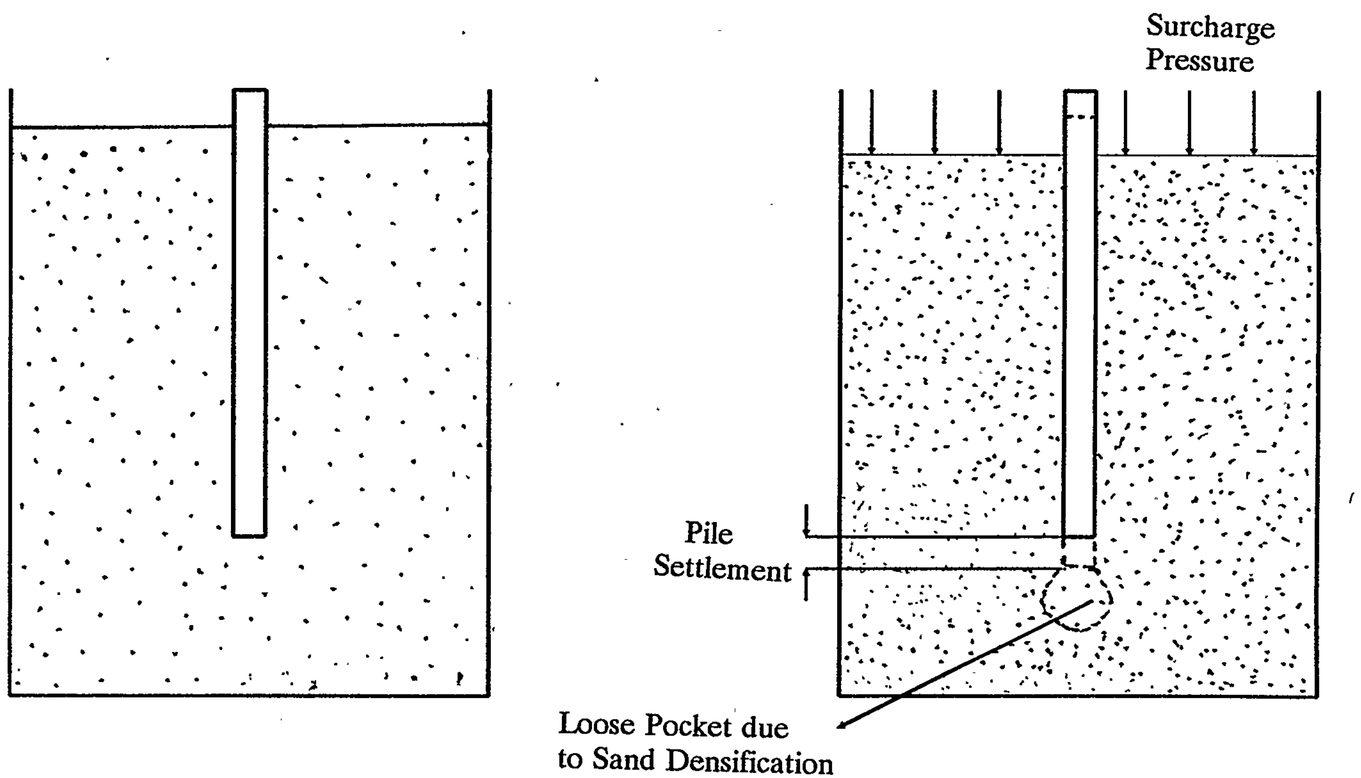
Figure 4.23 Tip Load - Displacement Plots for Piles in Soil under Surcharge Loads

overburden pressure of 138 kPa. But this solution is possible only when the pile soil characteristics remain constant with depth, that the soil is soft and compressible (as in clays) and the pile is end bearing.

Begemann (1969) has concluded that for piles in sand the maximum negative friction is limited to the pulling resistance of a pile embedded in it. He recommends to adopt a negative skin friction of 75 % of the total pulling force computed, since the process of pile extraction takes place under relative movement velocity which is much higher than that existing in a long term process.

The ultimate load of uplift for the piles subjected to overburden (Table 4.8) is in the range of 1.0 kN to 1.4 kN. For these forces to be developed in compression the pile displacement required is less than 1 mm (Figure 4.22). Hence the pile displacement due to the compression of the sand is insignificant; whereas, the sand all round the pile undergoes densification. Figure 4.24 gives a schematic presentation of this effect.

Soil all around the pile underwent densification. As the sand densified the height of the sand column decreased and the balloon at the top enlarged. Similar densification also occurred at a distance below the pile tip. As the sand just below the tip was not under pressure due to surcharge it did not have any tendency to densify. This left a pocket of relatively loose sand just below the tip (Figure 4.24) . This is the possible reason for the decrease in the tip capacity when a surcharge load was applied. As this pressure increased the loose pocket of sand continually became looser and variation of sand density increased from just below the tip to further



(a) Pile before applying Surcharge Load

(b) Pile after applying Surcharge Load

Figure 4.24 Formation of Relatively Loose Sand Pocket Under Pile Tip due to Surcharge Loading below. It could be expected that the relative decrease in sand density with pressure would fall, as pressures increased. Probably, at a pressure of 69 kPa (10 psi), the sand below the tip was very loose and a significant variation of tip capacity would not occur at pressures beyond 69 kPa (138 psi).

TABLE 4.6

TIP AND SHAFT CAPACITY OF PILES UNDER SURCHARGE LOADING

Surcharge Pressure kPa (psi)	Measured Tip Capacity	Measured Shaft Capacity
34.5 (5)	3.00	2.37
69.0 (10)	0.96	2.78
103.5 (15)	0.86	3.30
138.0 (20)	0.81	3.69

Shaft Resistance : Figure 4.25 gives the plot of shaft resistance against displacement for piles subjected to various overburden pressures. Unlike the variation of tip resistance, here it was observed that the shaft resistance increased steadily from a lower overburden pressure of 34.5 kPa (5 psi) to 138 kPa (20 psi). Table 4.6 gives the measured shaft failure load for surcharge pressures of 34.5 kPa (5 psi), 69 kPa (10 psi), 103.5 kPa (15 psi), 138 kPa (20 psi).

The unit shaft resistance for 34.5 kPa (5 psi) pressure increased by about 41 % over the shaft resistance for piles in soil with no pressure. The percentage increase for higher pressures of 69 kPa (10 psi), 103.5 kPa (15 psi), 138 kPa (20 psi) were about 65 %, 96 % and 120 % over the shaft resistance of piles at zero overburden pressure (Table 4.6).

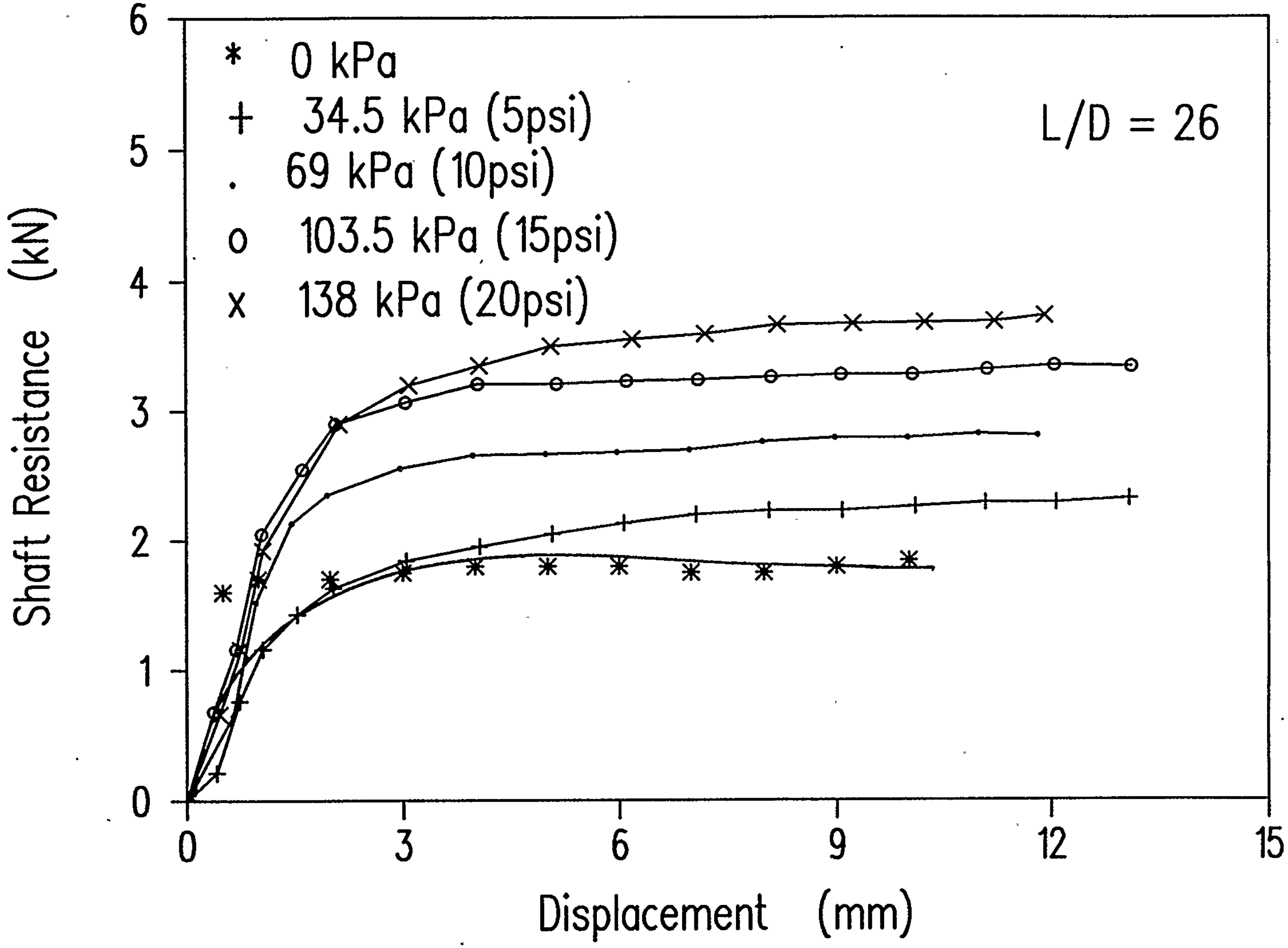


Figure 4.25 Shaft Resistance - Displacement Plots for Piles in Soil under Surcharge Loads

The variation of the average unit shaft and tip resistance with the effective L/D ratio of the pile was studied. Analysis was done assuming a pile embedded to a depth equal to the effective height of the soil due to the surcharge load plus the actual length of the pile in the soil. This gave pile lengths upto 9.56 m. Thus, the L/D ratios of the effective length of the pile due to surcharge loading plus its actual length was 26 for no overburden pressure, 66.5 for 34.5 kPa (5 psi) overburden pressure, 107.1 for 69 kPa (10 psi) overburden pressure, 147.6 for 103.5 kPa (15 psi) overburden pressure and 188.2 for 138 kPa (20 psi) overburden pressure. Table 4.7 gives the average unit shaft resistance and the unit tip resistance for pile of these L/D ratios as was obtained in the laboratory simulated conditions of overburden pressure. The average unit shaft and tip resistance have been obtained assuming the total shaft and tip resistance (Table 4.6) to be acting uniformly all along the pile length of 1.32 m (L/D = 26). Table 4.7 shows that the average unit shaft resistance of piles does not attain a constant value but seems to increase even at large L/D ratios. On the otherhand the average unit tip resistance decreases with an increase in L/D ratio. The reason for this may be due to the formation of a loose pocket just below the pile tip due to the application of the overburden pressure as explained earlier.

Figure 4.27 gives the variation of average unit shaft resistance with depth. The depth equivalent of surcharge pressures was calculated in a similar manner as for retaining walls.

TABLE 4.7

UNIT SHAFT AND TIP CAPACITY (kN/m²) OF PILES UNDER SURCHARGE LOADING

L/D ratio	Unit Shaft Resistance	Unit Tip Resistance
26	7.97	15.66
66.5	11.24	14.23
107.1	13.19	4.55
147.6	15.66	4.08
188.2	17.51	3.84

The overburden pressure was assumed to act horizontally and equally along the entire length of the pile as a force rectangle. The soil pressure was assumed to act as a force triangle.

The point of application, below the pile top, of the summation of the two forces was considered and was added to the effective height of the soil due to overburden to get the effective depth of application of the pressure. Figure 4.27 shows that the pile shaft resistance increased with depth though not linearly. It increased sharply for an initial depth beyond which its increase decreased with depth. The variation of unit shaft resistance with depth shows that the average unit shaft resistance does not attain a constant value even when piles with high L/D ratio (L/D = 188.2) are used, but it seems to increase at a decreasing rate. This seems to agree with Kulhawy's (1984) conclusion that ultimate shaft resistance is a fallacy and that shaft resistance increases continually with depth.

Tension Test

The plots of tensile load against displacement are presented in Figure 4.26. The tensile load is synonymous to the shaft resistance of piles in tension. From Figure 4.26 it was observed that the tensile capacity of a pile increased as the surcharge loading increased. Table 4.8 gives the mean tensile failure load and the settlement at which it occurred. Figure 4.27 gives the plot of tensile shaft resistance against depth. It was observed that the tensile shaft capacity increased upto a certain depth but its increase decreased with depth. However, it was also observed that the rate of increase of the tensile shaft capacity with depth was much less than the compressive shaft capacity. The percentage increase of the tensile capacity for surcharges of 34.5 kPa (5 psi), 69 kPa (10 psi), 103.5 kPa (15 psi), 138 kPa (20 psi) over that of zero surcharge pressure (Table 4.8) were 32 %, 45 %, 59 % and 84 % respectively.

TABLE 4.8

TENSILE FAILURE LOAD UNDER SURCHARGE LOADING

SURCHARGE kPa (psi)	ST	DT	MAZ	CHIN	BH90	BH80	VAN	MEAN (kN)	SETT (mm)
34.5 (5)	1.00	1.00	1.00	1.02	0.99	1.01	1.01	1.00	4.15
69.0 (10)	1.07	1.08	1.11	1.17	1.08	1.10	1.07	1.10	5.70
103.5 (15)	1.18	1.19	1.23	1.29	1.16	1.24	1.21	1.21	5.55
138 (20)	1.37	1.38	1.41	1.43	1.39	1.40	1.40	1.40	6.00

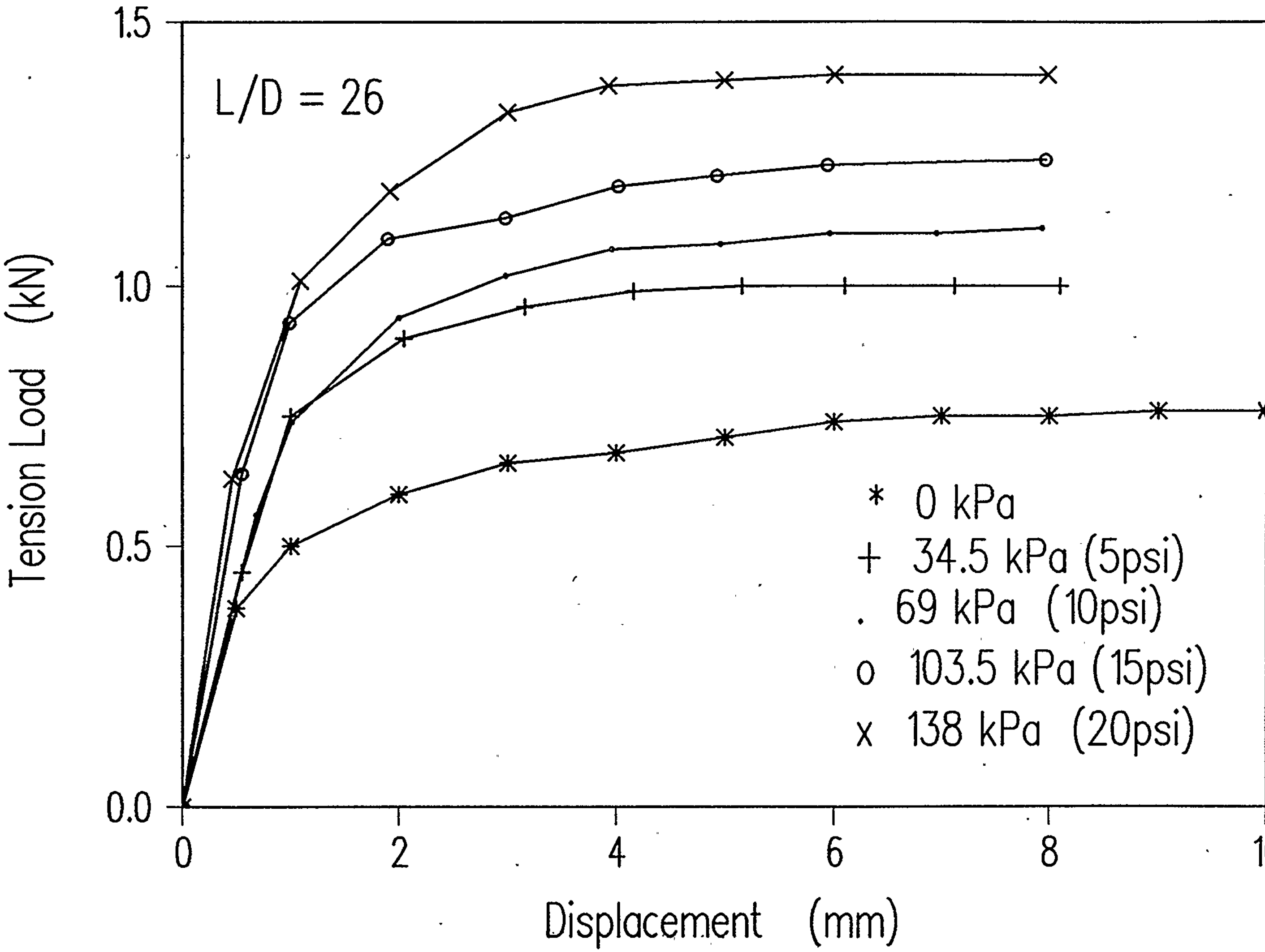


Figure 4.26 Tensile Load - Displacement Plots for Piles in Soil under Surcharge Loads

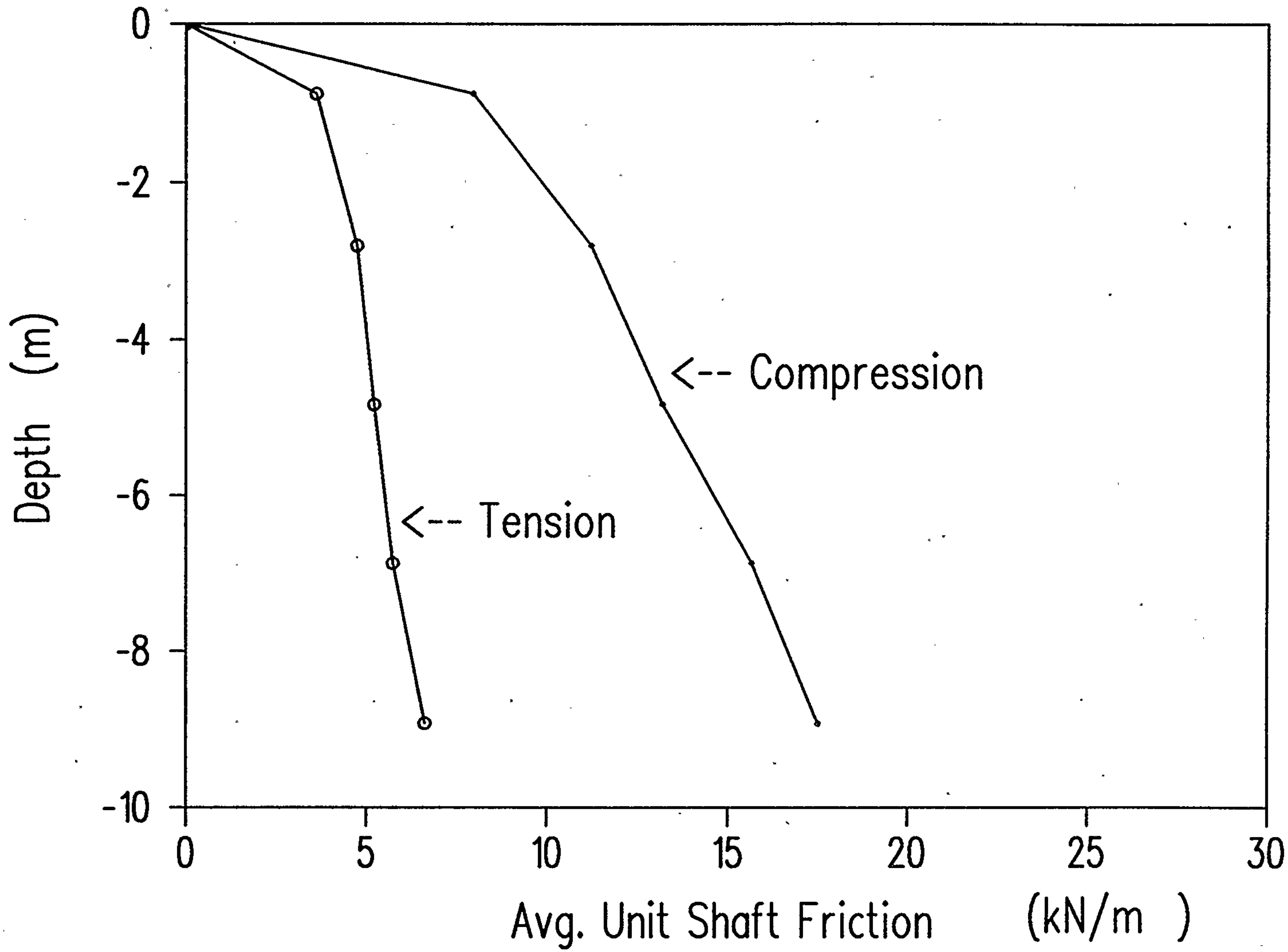


Figure 4.27 Unit Shaft Resistance - Depth Plot for Piles in Tension and Compression

Note:

ST = Single Tangent

DT = Double Tangent

MAZ = Mazurkeiwicz's method

CHIN = Chin's method

MEAN = Mean failure load

BH90 = Brinch-Hansen's 90 % criteria

BH80 = Brinch-Hansen's 80 % criteria

VAN = Van der Veen's method

SETT = Displacement at mean failure load

The uplift displacement required for piles to fail in tension with surcharge loading was between 8 % of the pile diameter and 12 % of the pile diameter whereas, for piles without any surcharge loading, the piles require an uplift displacement of about 15 % of the pile diameter.

Table 4.9 gives the variation of average unit shaft resistance of piles in tension with L/D ratio. The L/D ratio of the piles was evaluated as in the case of piles in compression, discussed in a earlier section.

As in case of compression shaft resistance here also it shows that the average unit shaft resistance of piles does not become constant with larger depth, but has a tendency to keep increasing with depth though at a decreasing rate.

From the experimental results as well as the analysis it is clear that the unit shaft resistance keeps increasing with depth both in tensile and compression capacity of a pile. An analysis was done to check the variation of individual components of the average unit shaft resistance.

$$f_s = K_s * \tan\delta * \sigma_v'$$

TABLE 4.9

AVERAGE UNIT SHAFT RESISTANCE OF TENSION PILES UNDER SURCHARGE LOADING

L/D ratio	Unit Shaft Resistance (f_s)
26	3.61
66.5	4.74
107.1	5.22
147.6	5.74
188.2	6.64

Of the three parameters that f_s depends K_s, δ and σ_v', the most difficult to ascertain are K_s and δ. σ_v' can be measured. Hence the variation of K*tanδ with either of the two other parameters was attempted. Since f_s in this case is known from measured values and σ_v' is the average pressure applied between the top and the bottom of the pile. The pressure acting on the top of the pile is the surcharge load applied. The pressure at the bottom of the pile is the summation of surcharge load and the pressure due to the sand. It is assumed that the vertical effective stress at the pile tip is equal to the product of the density and the length of the pile. A plot of σ_v' against f_s was made to study the variation of K*tanδ. K*tanδ is the slope of the plot. Figure 4.28 gives this plot. It can be observed that the value of K*tanδ decreases with an increase in overburden pressure for both cases of piles in tension

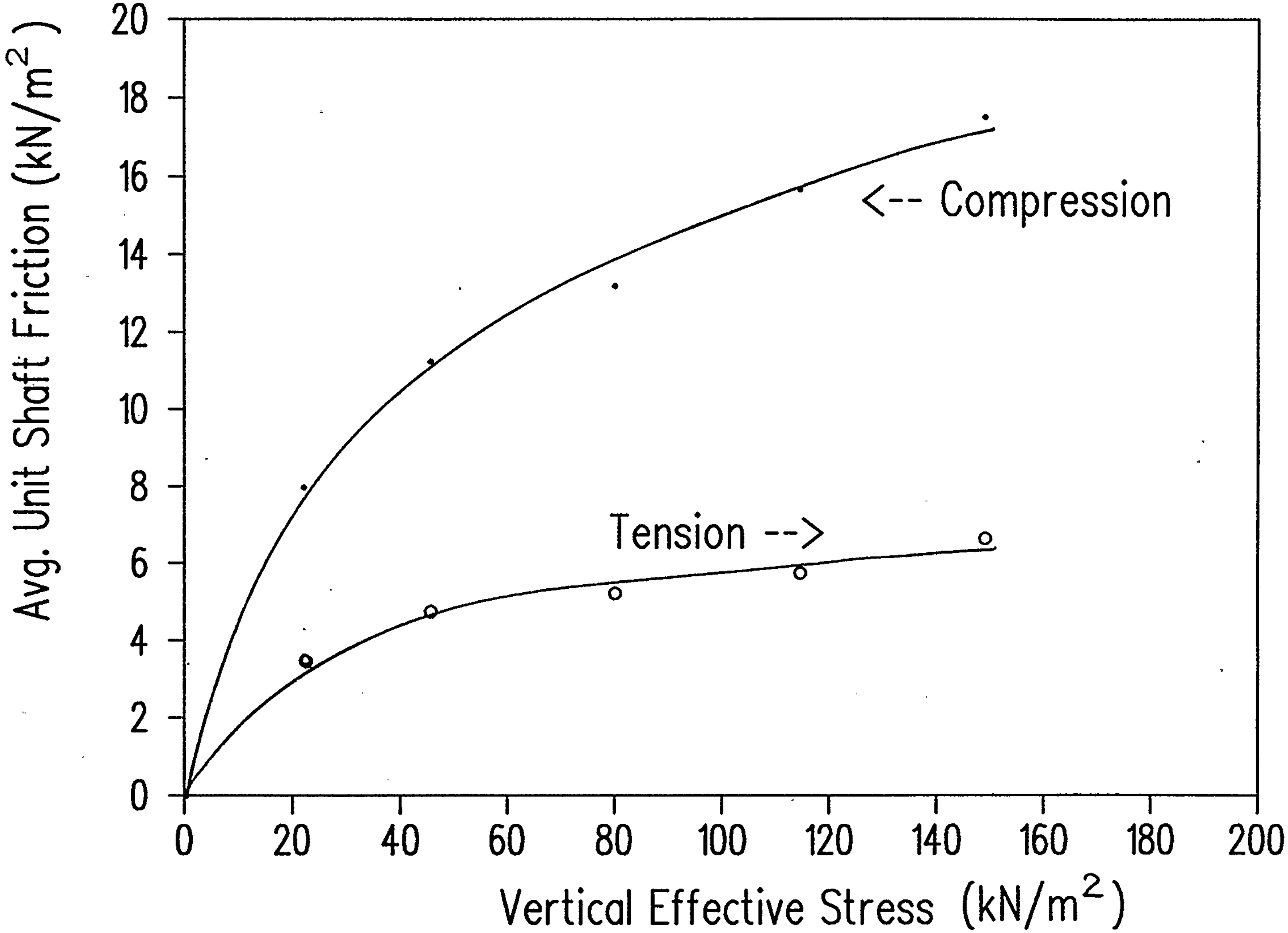


Figure 4.28 Unit Shaft Resistance versus Overburden Pressure

and compression for the range of effective vertical stress considered. Its rate of decrease is more for lower overburden pressures and as the overburden pressure increases the decrease in the value of K*tanδ becomes less.

4.2 Interpretation of Pile Load Test Data

Field data of pile load tests have been analysed to check the validity of the definition of the mean or probable failure load. The range of prediction of the ratio of various existing methods to evaluate the pile ultimate capacity to that of the defined mean or probable failure load was calculated and plotted. The definition of the mean or probable failure load was also checked against the settlement criteria for all kinds of piles. The reason for doing this analysis is to compare the failure loads obtained by various methods and thus obtain a criteria of failure.

101 pile load test data on 93 piles tested at widely scattered geographical locations have been analyzed. The piles tested by slow and quick maintained load test methods were classified into the following groups: driven straight shaft piles, bored straight shaft piles, driven expanded base piles, bored expanded base piles, and I-section and H-section piles. Piles tested by constant rate of penetration method were considered as a separate group. The ultimate load of each pile was determined by several methods.

The results of all the methods were analyzed to determine the range of prediction of the methods. The data on piles tested by slow and quick maintained load test methods were analyzed to determine the differences in pile load capacity obtained by these methods.

The piles have been separated into different groups as explained. The number of piles in each category were:

Driven straight shaft piles - 27 piles

Bored straight shaft piles - 29 piles

Driven expanded base piles - 10 piles

Bored expanded base piles - 11 piles

I-section & H-section piles - 5 piles

Piles tested by CRP method - 11 piles

The probable ultimate load of pile should be known to compare the ultimate load predicted by the various methods. Therefore, ultimate load of each pile was first determined by different appropriate methods. To obtain the most probable ultimate load of the pile an iterative procedure was used. First the mean of the ultimate loads determined by the several methods was computed. To determine the range in which the probable ultimate load will lie, the values which were more than 1.5 times and less than 0.5 times the mean value were discarded. For the remainder of the values the mean was computed again and the process of elimination of the extreme values from the mean value was repeated. The mean value finally arrived at from such an iterative process was considered to be representative of the most probable ultimate load of the pile. Usually no more than two iterations were required.

To determine how close the ultimate load predicted by a method to the most probable ultimate load was, the ratio L_R was calculated as

$$L_R = \frac{\textit{ultimate load predicted by the method}}{\textit{most probable ultimate load}} \times 100$$

It may be noted that the value of L_R could exceed 150 % or could be less than 50 %. The average of L_R for all piles was calculated for each method. A value close to 100 % indicate that the ultimate load predicted by the method was generally close to the most probable ultimate load. A value significantly less than 100 % indicated the method to be possibly conservative and would predict less than true ultimate load. The opposite was true when the value exceeded 100 % significantly.

In selecting the methods to predict the ultimate loads, unless it is stated that a method was applicable only for a particular situation it was deemed that the method was dependent only on the shape of the load-settlement curve and is applicable for the conventional shape of the curve.

At the most probable ultimate load, consideration to gross settlement at pile head and the rate of increase of settlement with increase in load was given by computing two ratios R_1 and R_2, respectively. R_1 and R_2 are defined as

$$R_1 = \frac{S_u}{D} \times 100$$

$$R_2 = \frac{\Delta S}{\Delta Q} \quad \text{mm/kN}$$

where S_u = gross pile head settlement at the most probable ultimate load

D = stem diameter of pile

ΔS = increase in settlement for an increase in load (ΔQ)

R_2 gives the slope of the load-settlement curve at the most probable ultimate load.

Figure 4.29 shows the range and average of L_R for different methods for driven straight shaft piles. Plots similar to Fig.4.29 showing the range and average of L_R for the different methods for bored straight shaft piles, driven expanded base piles, bored expanded base piles, H-section and I-section piles, and piles tested by CRP method are shown in Figs 4.30, 4.31, 4.32, 4.33 and 4.34 respectively.

Figures 4.29 to 4.34 give comparisons between ultimate loads obtained by various methods. Figure 4.29 makes possible an easy appreciation of several facts. For example, consider the Chin's method. The range of prediction by this method was very large and as indicated by the average mark, the method predicted a higher load than the probable ultimate load. The position of the average mark, however, indicates that there was only a few extreme values on the higher side.

TABLE 4.10

DETAILS OF PILES ANALYSED

Sl. No.	Pile Designation	Diameter (mm)	Method of installation	Base type	Soil type	Method of load test	Source
1	P1	500	d	N	NC	SML	(1)
2	P2	450	d	N	NC	SML	(1)
3	P3	500	d	N	NC	SML	(1)
4	P4	500	d	N	NC	SML	(1)
5	P5	400	d	N	NC	SML	(1)
6	P6	400	d	N	NC	SML	(1)

7	P7	400	d	N	NC	SML	(1)
8	P8	400	d	N	NC	SML	(1)
9	P9	500	d	N	C	SML	(1)
10	P10	450	d	N	NC	SML	(1)
11	P11	1070	b	N	NC	SML	(1)
12	P12	530	d	N	NC	SML	(1)
13	P13	530	d	N	C	SML	(1)
14	P14	450	d	N	NC	SML	(1)
15	P15	450	d	N	NC	SML	(1)
16	P16	530	d	N	C	SML	(1)
17	P17	1000	b	N	C	SML	(1)
18	P18	750	b	N	C	SML	(1)
19	P19	406	b	E	C	SML	(2)
20	P20	406	d	E	C	SML	(2)
21	P21	406	d	E	C	SML	(2)
22	P22	406	d	E	C	SML	(2)
23	P23	406	d	E	C	SML	(2)
24	P24	406	d	E	C	SML	(2)
25	P25	406	b	E	C	SML	(2)
26	P26	406	b	E	C	SML	(2)
27	P27	406	d	E	C	SML	(2)
28	P28	406	d	E	C	SML	(2)
29	P29	406	d	N	C	SML	(2)
30	P30	406	d	N	C	SML	(2)
31	P31	406	d	N	C	SML	(2)
32	P32	406	d	N	C	SML	(2)
33	P33	150	d	N	NC	SML	(3)
34	P34	400	d	N	C	CRP	(4)
35	P35	1220	d	N	C	CRP	(4)
36	P36	120	d	N	C	CRP	(4)
37	P37	400	d	E	C	CRP	(4)
38	P38	350	b	N	C	SML & QML	(5)
39	P39	425	b	N	C	SML & QML	(5)
40	P40	400	d	N	C	SML	(5)
41	P41	400	d	E	C	QML	(5)
42	P42	700	b	N	C	SML & QML	(5)
43	P43	917	b	N	C	SML & QML	(5)
44	P44	500	b	N	C	QML	(5)
45	P45	500	b	N	C	SML & QML	(5)
46	P46	127	d	N	C	SML & QML	(5)
47	P47	I	d	N	C	SML & QML	(5)
48	P48	320	b	N	C	SML & QML	(5)
49	P49	500	b	E	C	SML	(6)
50	P50	500	b	E	C	SML	(6)
51	P51	475	d	E	C	SML	(6)
52	P52	500	d	E	C	SML	(6)
53	P53	H	d	N	C	SML	(6)
54	P54	450	d	N	C	QML	(7)
55	P55	450	d	N	C	QML	(7)
56	P56	460	b	N	C	QML	(7)
57	P57	H	d	N	C	SML	(8)
58	P58	H	d	N	C	SML	(8)
59	P59	H	d	N	C	SML	(8)
60	P60		d	N	C	SML	(8)
61	P61		d	N	C	SML	(8)
62	P62		d	N	C	SML	(8)
63	P63	606	b	E	C	CRP	(9)
64	P64	600	d	N	C	CRP	(9)
65	P65	1021	b	N	C	QML	(10)
66	P66	808	b	N	C	CRP	(10)
67	P67	646	b	N	C	CRP	(10)
68	P68	786	b	N	C	CRP	(10)

69	P69	786	b	N	C	CRP	(10)
70	P70	521	b	N	C	CRP	(10)
71	P71	762	b	N	C	QML	(10)
72	P72	914	b	N	C	QML	(10)
73	P73	762	b	N	C	QML	(10)
74	P74	762	b	N	C	QML	(10)
75	P75	457	b	N	C	QML	(10)
76	P76	1198	b	N	C	QML	(10)
77	P77	1000	b	N	C	SML	(10)
78	P78	1000	b	N	C	QML	(10)
79	P79	853	b	N	C	QML	(10)
80	P80	762	b	N	C	QML	(10)
81	P81	914	b	N	C	QML	(10)
82	P82	762	b	N	C	QML	(10)
83	P83	610	b	N	C	QML	(10)
84	P84	786	b	E	C	QML	(10)
85	P85	634	b	N	C	QML	(10)
86	P86	628	b	E	C	QML	(10)
87	P87	774	b	N	C	QML	(10)
88	P88	802	b	N	C	QML	(10)
89	P89	774	b	E	C	QML	(10)
90	P90	774	b	E	C	QML	(10)
91	P91	939	b	N	C	QML	(10)
92	P92	939	b	E	C	QML	(10)
93	P93	939	b	E	C	QML	(10)

(1) - Chakraborty (1989) (2) - DeBeer et al. (1977)
(3) - Franx (1936) (4) - Boonstra (1936)
(5) - ABEF Research on Foundation Engineering (1989)
(6) - Joshi and Sharma (1987) (7) - Van Impe et al. (1988)
(8) - Fellinius (1989) (9) - DeBeer (1988)
(10)- Reese and O'Neill (1988)

b = Bored Pile C = Cohesive Soil
d = Driven Pile E = Expanded Base Pile
H = H - Section Pile I = I - Section Pile
N = Straight Shaft Pile NC = Non Cohesive Soil
CRP = Constant Rate of Penetration
QML = Quick Maintained Load Test
SML = Slow Maintained Load Test

Similar conclusions, but having a tendency to predict lower than the actual ultimate load, was made for the Davisson's method. The double tangent method had a narrow range and an average L_R value close to 100 %.

From an observation of Figs 4.29 to 4.34 it was concluded that the Chin's method

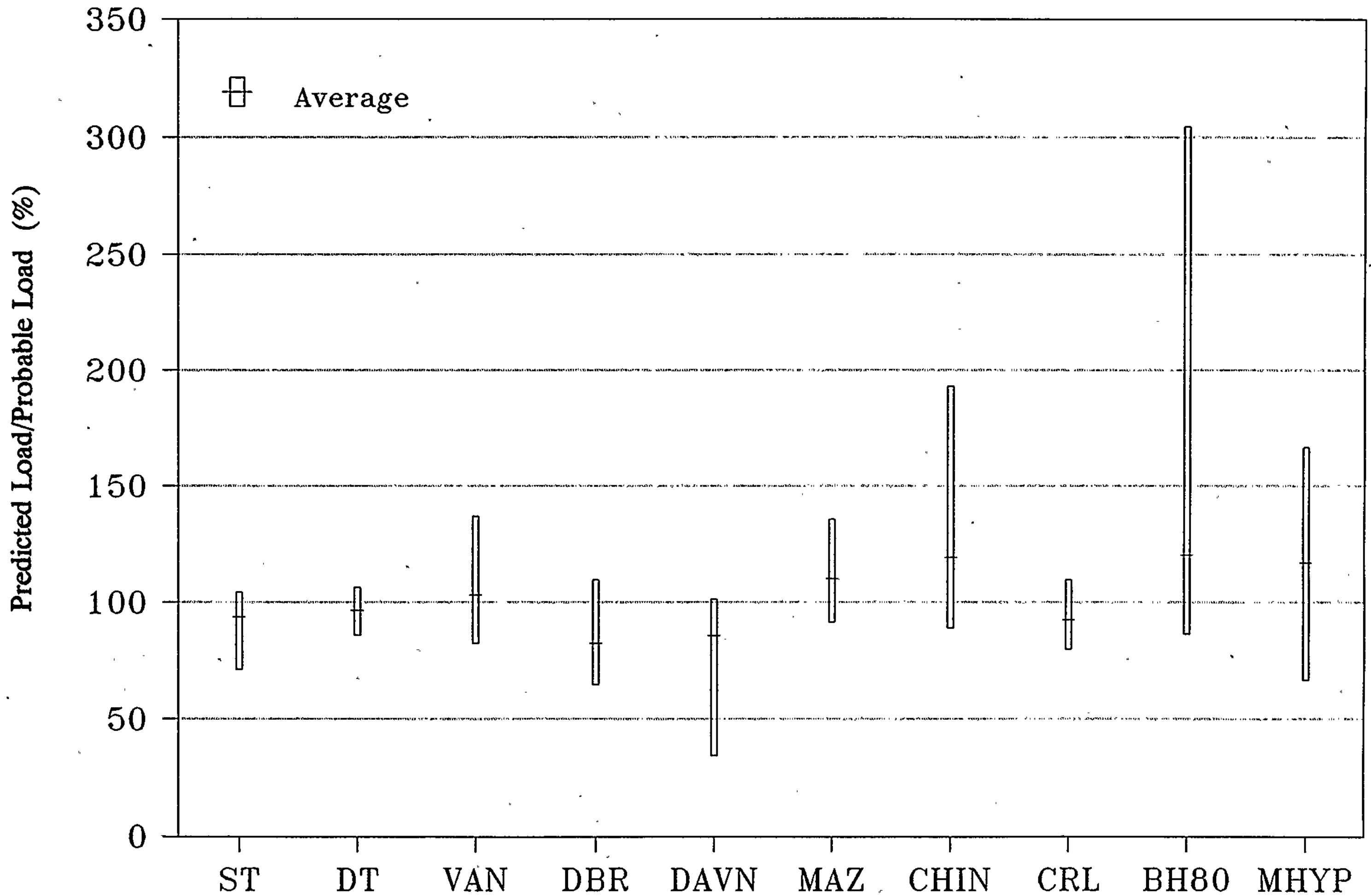


Figure 4.29 Range of Prediction of the Methods for Driven Straight Shaft Piles

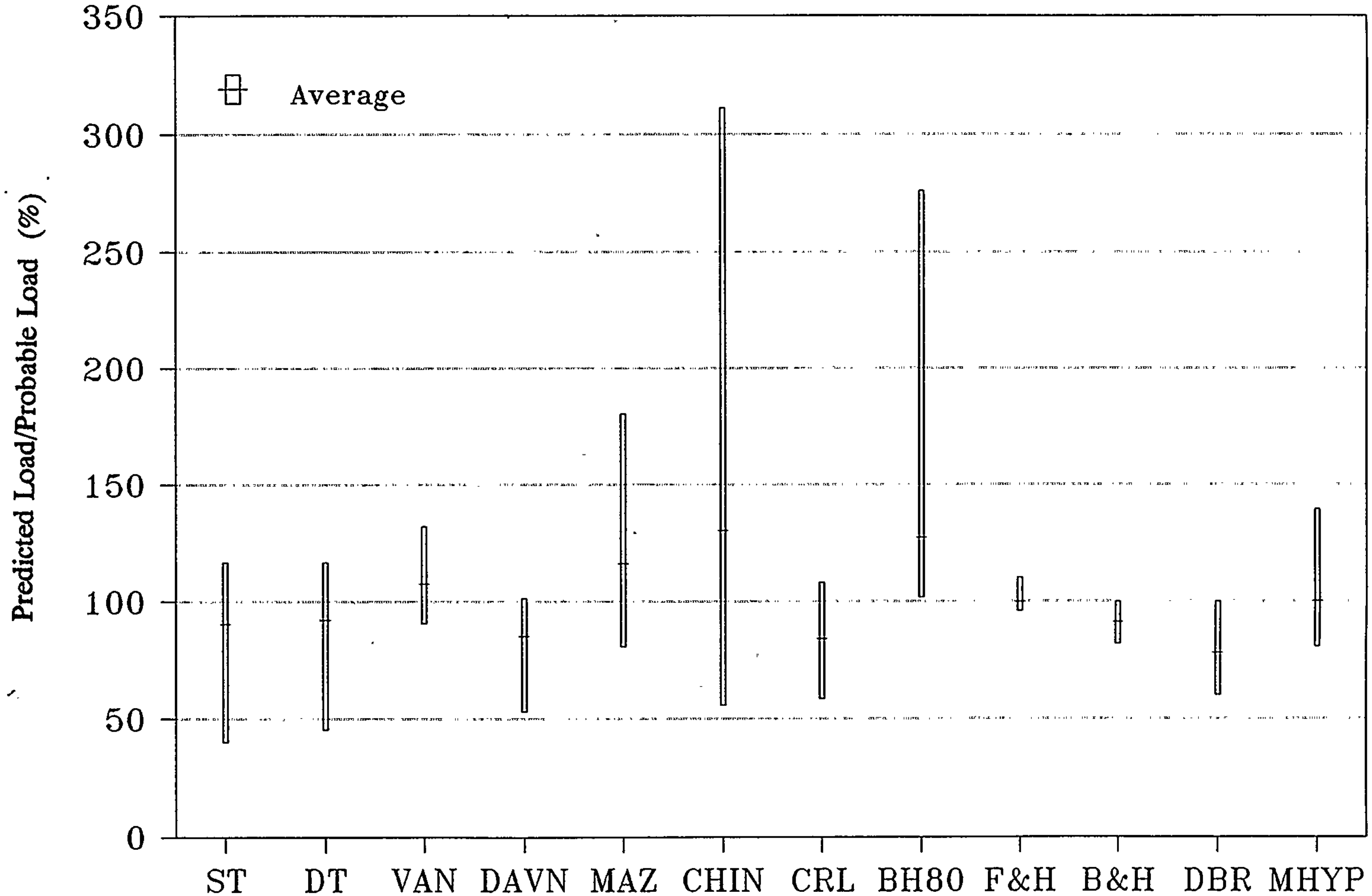


Figure 4.30 **Range of Prediction of the Methods for Bored Straight Shaft Piles**

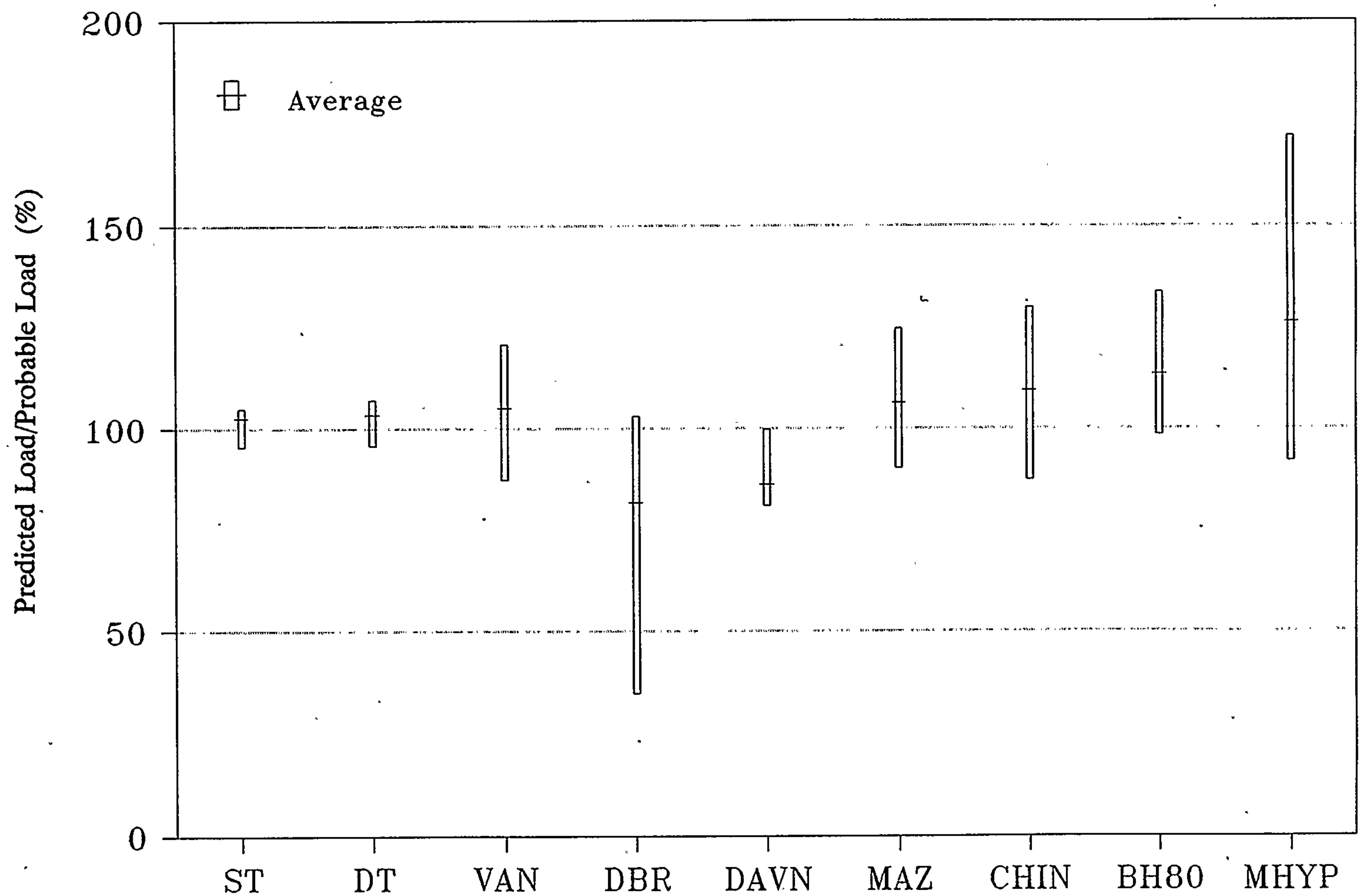


Figure 4.31 Range of Prediction of the Methods for Driven Expanded Base Piles

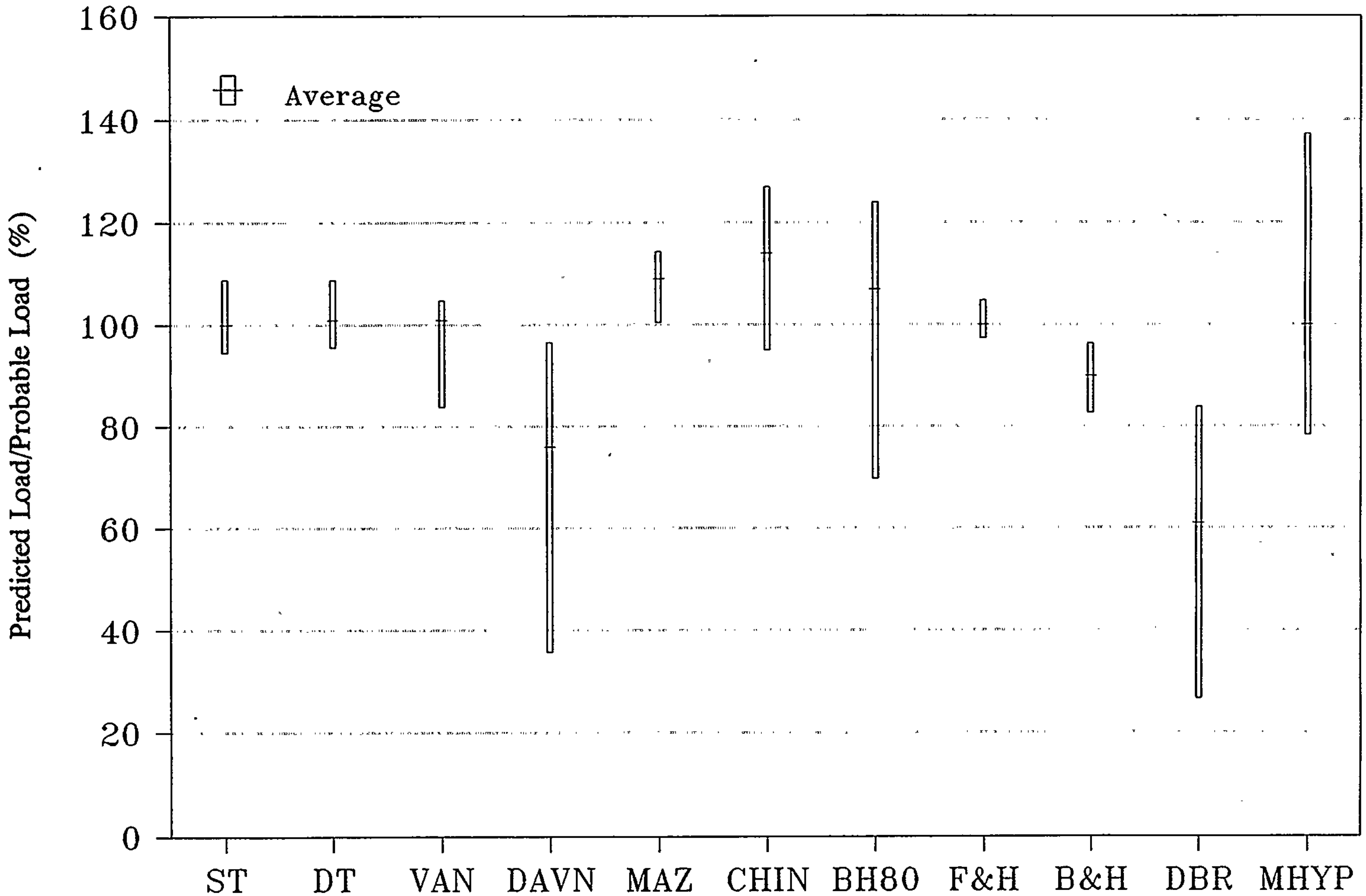


Figure 4.32 Range of Prediction of the Methods for Bored Expanded Base Piles

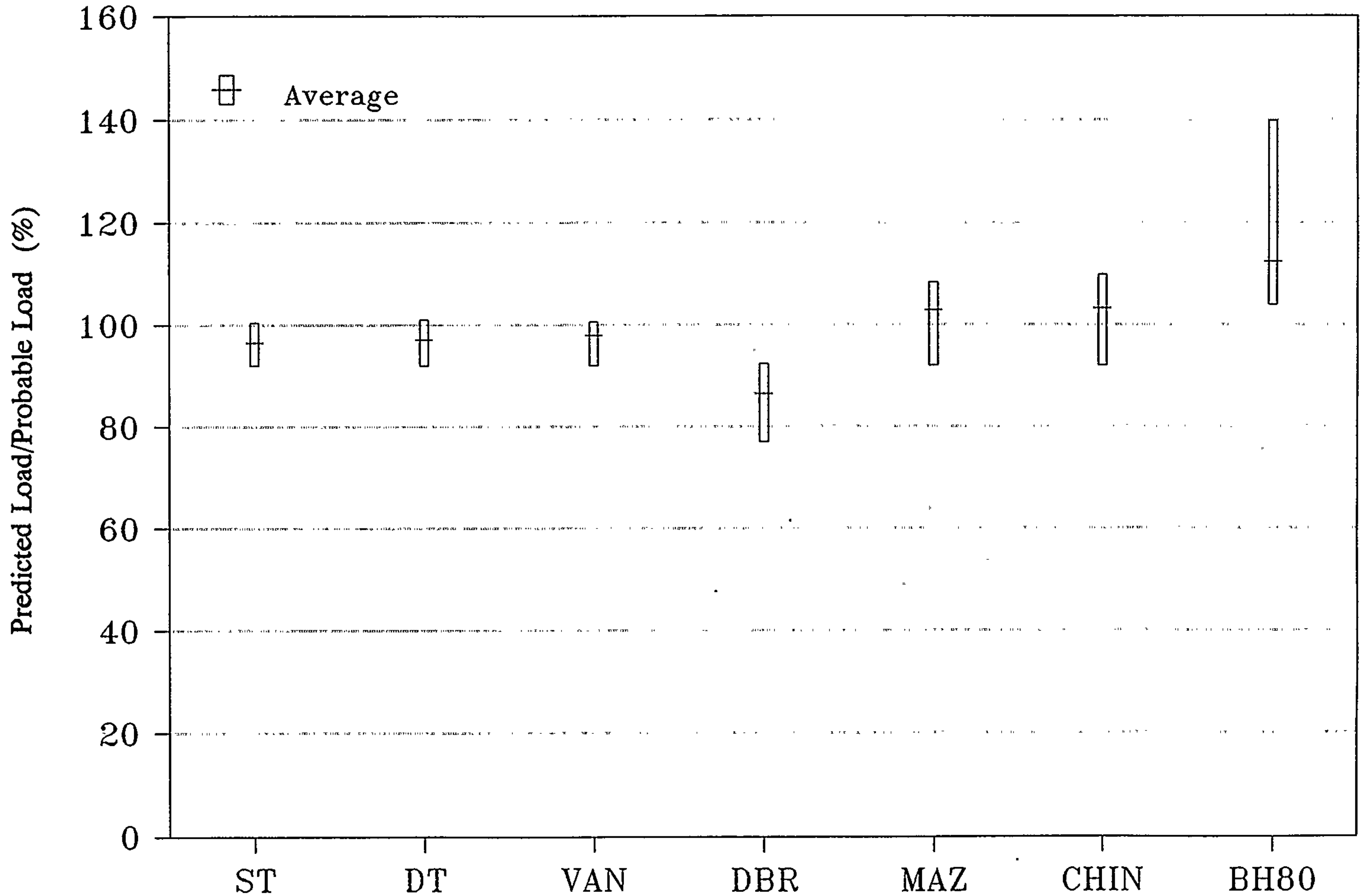


Figure 4.33 Range of Prediction of the Methods for I-section and H-section Piles

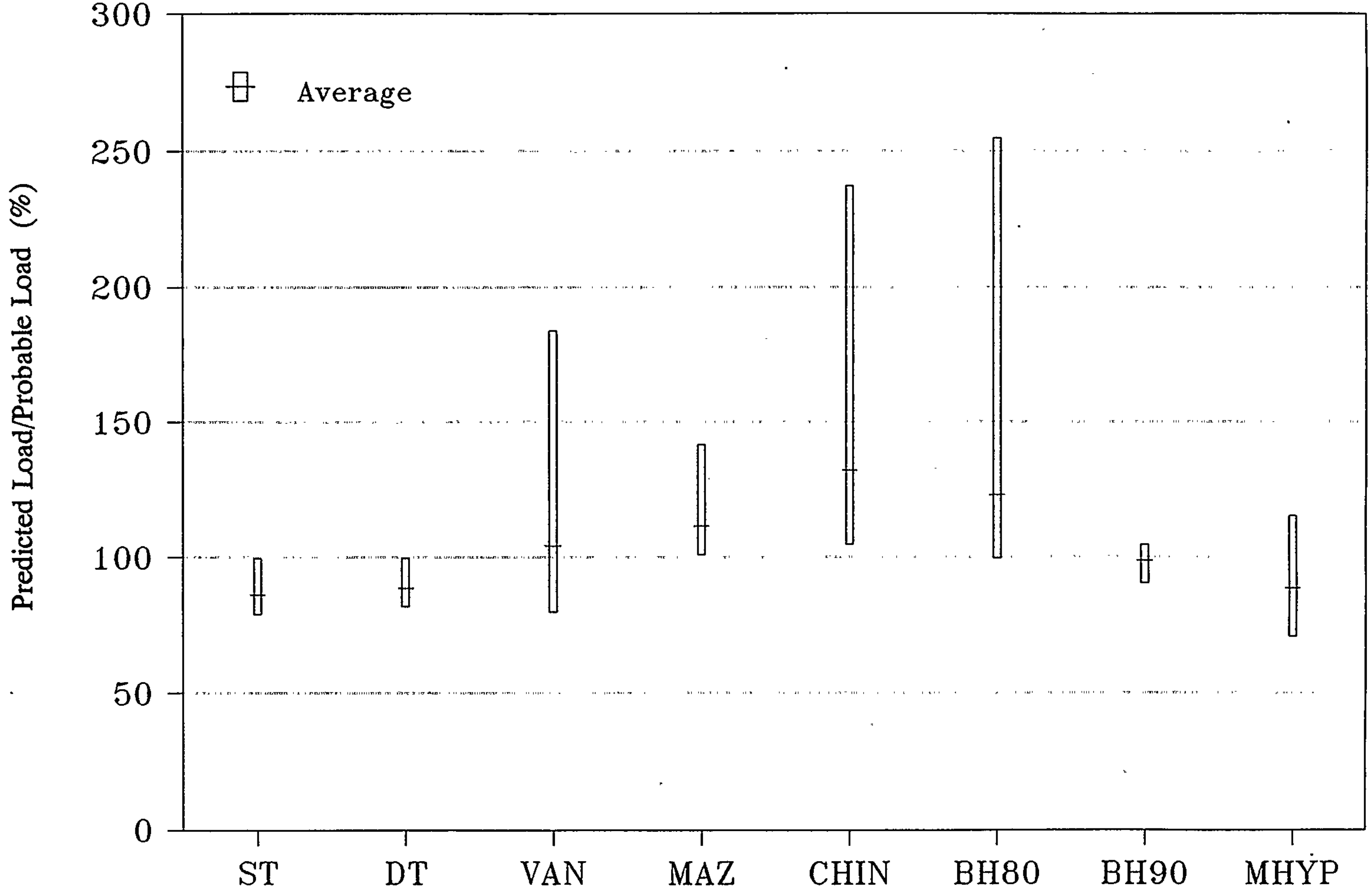


Figure 4.34 Range of Prediction of the Methods for Piles tested by CRP Method

and the Brinch Hansen's 80 % criterion generally tend to predict higher than the most probable values of ultimate load. The wide range of L_R value for these methods, above 100 %, suggested that the predicted ultimate loads could be sometimes unsafe.

On the other hand, the DeBeer's method and the Davisson's method were generally conservative. The predicted ultimate load by these methods was less than the most probable ultimate load.

For driven piles, the predicted ultimate load by the modified hyperbolic method was more than the probable ultimate load, 16 % more in the case of straight shaft piles and 26 % more in the case of expanded base piles. For bored piles the ultimate load predicted by the modified hyperbolic method was almost equal to the most probable ultimate load. In the case of piles tested by constant rate of penetration method, modified hyperbolic method gaves conservative results. The predicted ultimate load was about 10 to 12 % less than the most probable load. Considering the fact that the method makes use of only the initial portion of the load-settlement curve, the modified hyperbolic method made a good prediction of the ultimate load even when the pile had not been tested to failure.

Simple methods such as single tangent method and double tangent method gave very good estimates of ultimate load. Additionally, the narrow range of L_R for these methods, indicates that error associated with these methods is indeed very small.

4.2.1 Comparison of SML and QML Test Results

Table 4.11 gives the results for eight piles which have been tested both by slow and quick maintained load test methods. It was evident that the ultimate load depends on the method of testing.

The quick maintained load test tend to give a higher ultimate load than the slow maintained load test. For the six bored piles, P38 to P46, the ultimate load obtained by the QML test was, on the average, 61% more than the value obtained by the SML test.

TABLE 4.11

COMPARISON OF SML AND QML TESTS

Pile Designation	Q_{max} (kN)		$Q_{max\text{-}QML}/Q_{max\text{-}SML}$
	SML	QML	
P38	445	576.4	1.30
P39	672	744.5	1.11
P42	904	1273.8	1.41
P43	1745	4189.9	2.40
P45	1904.3	3868.8	2.03
P46	547	779.1	1.42
P47	261	273.6	1.05
P48	677.9	730	1.08

For the two driven piles, P47 and P48, the ultimate load obtained by the QML test was only marginally higher than that obtained by the SML test. However, it might be noted that all the eight piles were in cohesive soil medium. In a granular soil medium, the method of conducting the test may not influence the ultimate load significantly (Sparrow 1988).

4.2.2 Comparison of the Values of Criteria based on Settlement

From an inspection of the calculated values of R_1 and R_2 the range in which most values lie was identified for the different types of piles. The average values of R_1 and R_2 were calculated for all piles, discarding those values which were more than 1.5 times the higher value of the range or less than 0.5 times the lower value of the range.

Figure 4.35 shows the range and average of the R_1 values for the different pile types. A common practice in the case of straight shaft piles was to assume the settlement at the ultimate load to be 10 % of the pile diameter. That is R_1 equal to 10 %. However, the calculated values at the most probable ultimate load were much smaller than 10 %, as shown in Fig. 4.35, the average values of R_1 being 4.6 % and 3.5 % for driven straight shaft and bored straight shaft piles, respectively. Probably the definition adopted for most probable ultimate load gave conservative results. If it were true then the Chin's method and the Brinch Hansen's method would tend to give better estimates of ultimate load. However, the range of L_R for these methods will still remain wide. On the other hand, the DeBeer's method and the Davisson's method would become still more conservative. The modified hyperbolic method would give better results for driven piles and conservative results for constant rate of penetration method.

It may be, however, stated that the criterion of settlement equal to 10 % of pile diameter at ultimate load is not realistic at all situations. In friction piles this value would be less and would depend on the length of the pile. In end bearing bored piles, this value can exceed 10 %.

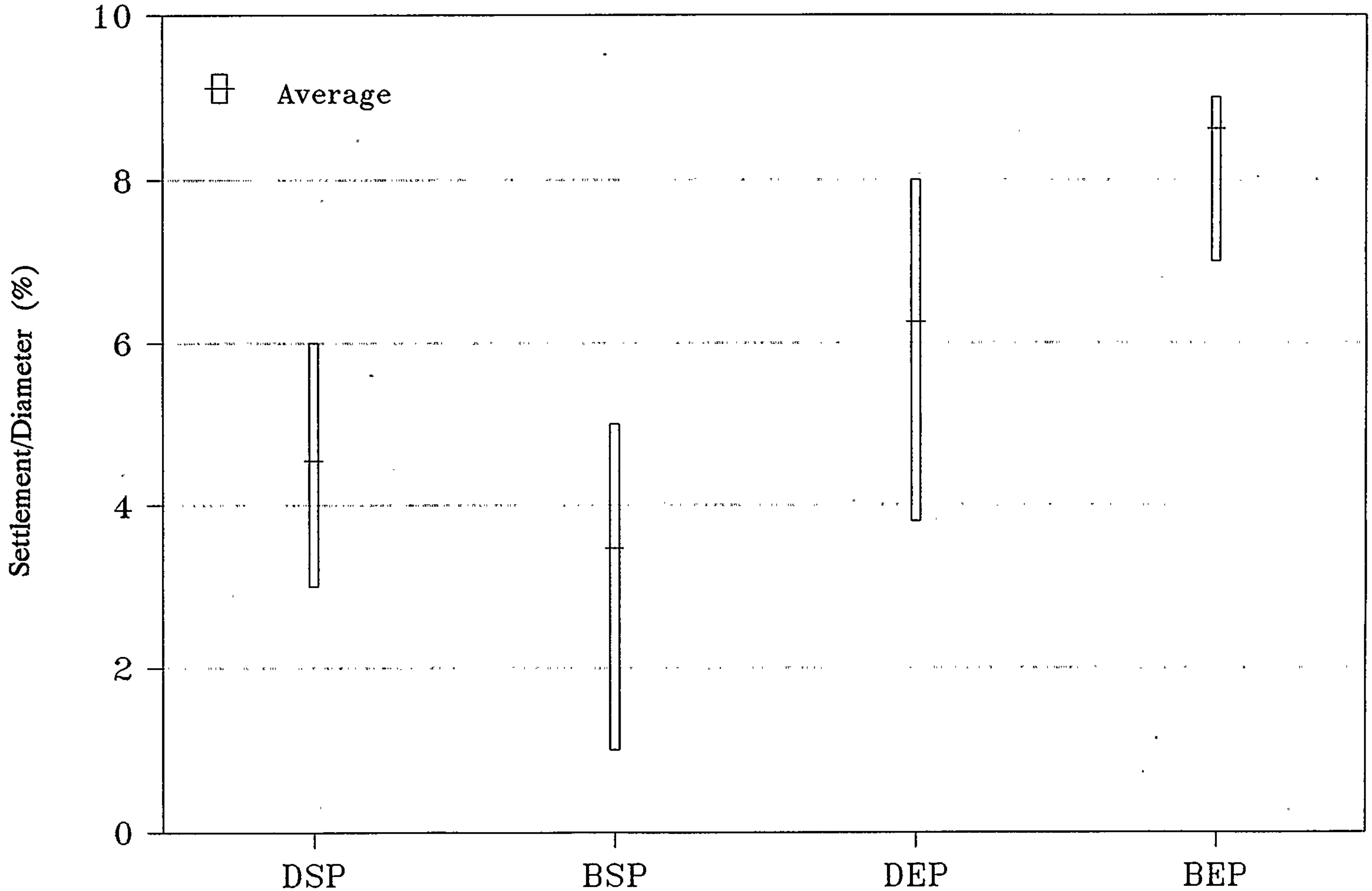


Figure 4.35 Range of Settlement to Pile Diameter Ratio at Ultimate Load

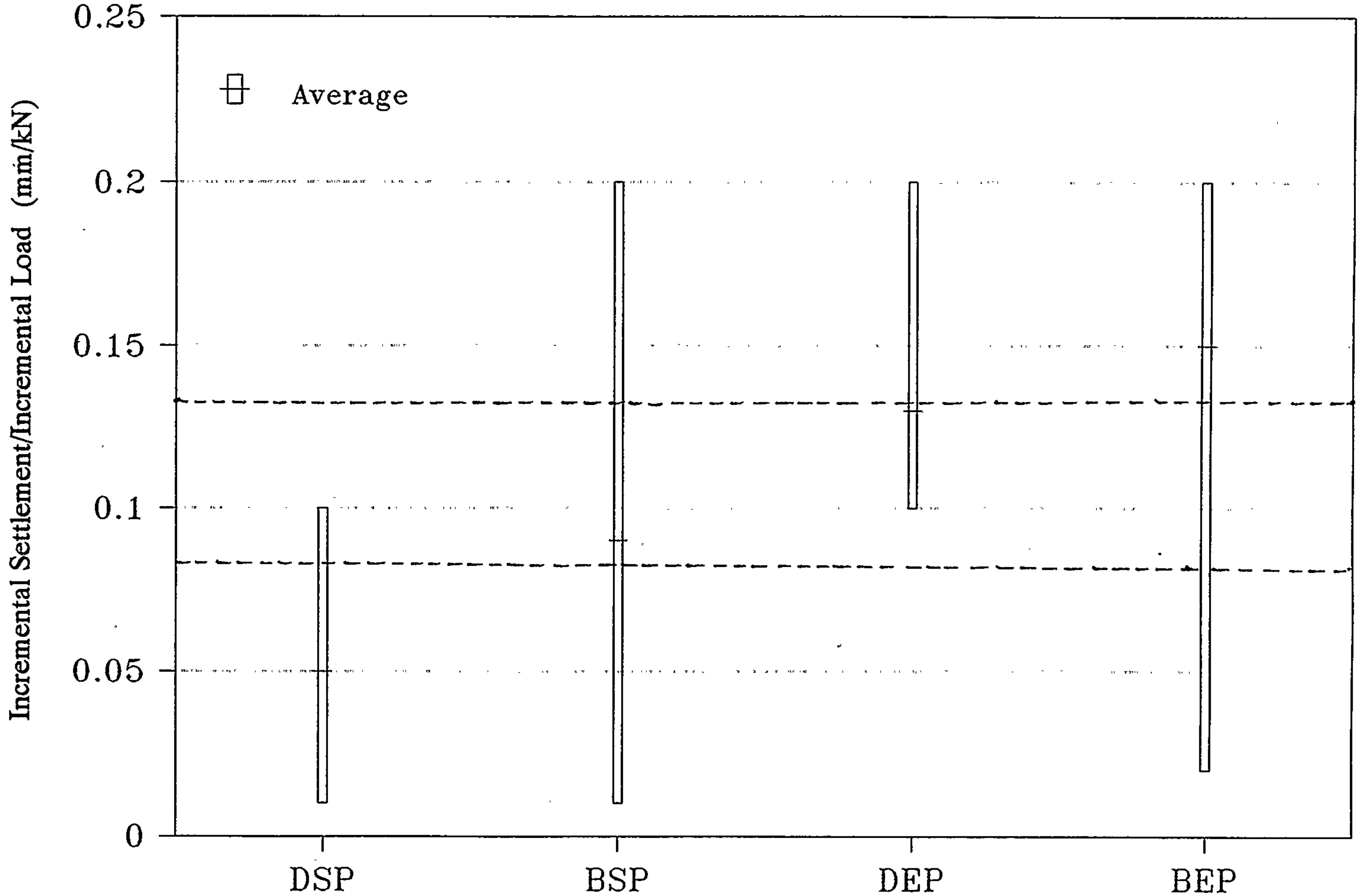


Figure 4.36 Range of Ratio of Incremental Settlement to Incremental Load at Ultimate Load

According to Broms et al. (1988) the limit load corresponds to the ultimate load determined by the double tangent method. However, the reported ratios of the settlement at limit load to pile diameter are in the range of 0.4 to 1.3 % (Chin 1982; Chang and Goh 1988). This is a much lower range than those determined in the present study at the most probable ultimate load and it might be recalled that the double tangent method gave ultimate load close to the most probable ultimate load. Fukuoka (1988) reported a settlement to diameter ratio of about 5.4 % at the ultimate load.

In this study it was observed that the average pile settlement to diameter ratio ranged between 3 % to 7 % for all types of piles. Only in the case of bored expanded base piles this ratio was observed to be more than 8 %. Since, the data base here is large and piles have been tested in widely varied geographical locations, it can be safely concluded that failure of piles occur at a settlement to diameter ratio of 3 % to 7 %. This ratio is applicable for field test results. For laboratory test results it was observed that failure occurs at a higher settlement to diameter ratio.

Figure 4.36 shows the range and average of R_2 for the different pile types. Also shown in the figure are the two values of R_2, 0.082 mm/kN and 0.133 mm/kN, mentioned by Vesic (1977). The Butler and Hoy's method for QML tests is also a specification of a value of 0.129 mm/kN for R_2. It was evident from Fig. 4.36 that for the driven straight shaft piles and bored straight shaft piles the R_2 values were generally smaller than one or both the values mentioned above. This raised a doubt whether the calculated values were conservative. However, if the load-settlement curve is nearly linear before becoming nonlinear near the ultimate load, the slope of the curve remains almost constant till close to the ultimate load and then increases rapidly. Thus a small value of R_2 need not necessarily indicate that the calculated ultimate loads are very conservative.

Hence, from all the above discussions it was concluded that the definition of the most probable ultimate load adopted in the present study is realistic. The error might have given slightly conservative ultimate loads.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 **Conclusions**

Based on the test results and discussion presented in Chapter 4 the following conclusions were drawn:

5.1.1 Loading History

1. Pile loading history has a pronounced effect on the ultimate failure load of a pile. The ultimate failure load, for piles having a loading history is significantly lower than piles which have not been subjected to prior loading.

2. The first compression test, after tension failure of the pile, shows lower pile capacity than the second compression after tension failure.

3. The tip capacity for piles which are loaded in compression after a tension failure, is mobilized only after a pile movement of around 5 mm, for dense sands. Whereas, the tip capacity is mobilized almost immediately for medium dense sands.

4. The pile shaft capacity is affected more than the tip capacity as a result of prior loading on piles.

5. The effect of loading history on the shaft capacity of piles is more in tension as compared to piles in compression.

6. Pile tests show that pile failure load occurs at a settlement of 15 - 20 % of the

pile diameter, in dense sands and more than 20 % of the pile diameter for medium dense sands.

7. Prediction of the pile tip capacity by Poulos and Davis (1980) are very close to the measured values.

8. The shaft capacity in compression, is predicted well by Meyerhof's (1976) method, though a slightly lower value was observed at higher L/D ratios.

9. The shaft capacity of piles in tension are predicted well by Coyle and Sulaiman's (1967) method. Predictions by Das (1984) and Tejchman (1971) are lower than actual values.

9. The ultimate shaft capacity in tension is significantly lower than the ultimate shaft capacity mobilized in compression. The ratio of the ultimate shaft capacity for piles in tension to that of piles in compression varies between 51 % for L/D ratio of 20 to 64.5 % for piles having L/D ratio of 33.

5.1.2 <u>Repeated Loading</u>

1. Piles in dense dry sand have negligible movement if repeated tensile loads were applied upto a maximum of 10 % of the tensile failure load. For higher load ranges the pile might experience a steady pull out.

2. The L/D ratio of piles has a significant effect on the behaviour of piles subjected to repeated loads at lower load ranges. At higher load ranges the pile behaviour became independent of the L/D ratio of the pile.

3. At higher load ranges the effect of frequency is negligible.

5.1.3 Surcharge Loading

Surcharge loading of soil around the pile has the following conclusions :

1. At low surcharge pressures of 34.5 kPa (5 psi), higher ultimate load is registered. A significant decrease of the ultimate load occurs at a surcharge pressure of 69 kPa (10 psi). Beyond this pressure, the total capacity of a pile increases steadily.

2. The tip load for a surcharge pressure of 34.5 kPa (5 psi) is lower than that for a pile with no surcharge. Beyond a pressure of 34.5 kPa (5 psi) the tip capacity of the pile decreases and remains almost constant for all pressures of 69 kPa (10 psi), 103.5 kPa (15 psi) and 138 kPa (20 psi). This may be due to the formation of a relatively loose pocket of sand below the tip as a result of densification of sand further away from the tip.

3. The shaft capacity of the pile increases steadily with an increase in the surcharge pressure both in compression and tensile tests.

4. Plotted against a depth equivalent of the surcharge pressure, it is observed that the shaft resistance increases, though in a decreasing manner with increasing depth.

5. The value of $K^*\tan\delta$ in shaft resistance was found to decrease with an increase in overburden pressure.

5.1.4. Ultimate Load Analysis of Piles

1. Comparison of the ratios of total settlement to pile diameter and incremental

settlement to incremental load, at the ultimate load, with those suggested in literature indicates that the definition for the most probable ultimate load adopted, for comparison of failure loads by different methods, in the present study was satisfactory.

2. Pile failure occurs at a settlement to diameter ratio between 3 % to 7 %. However, for bored expanded base piles the ratio is more than 8 %.

3. Simple methods such as the single tangent method and the double tangent method gives very good and reliable estimates of ultimate load.

4. The ultimate load predicted by the Chin's method and the Brinch Hansen's 80% criterion tends to be higher than the most probable ultimate load.

5. The DeBeer's method and the Davisson's method are generally conservative and the predicted ultimate load by these methods is less than the most probable ultimate load.

6. Even though the modified hyperbolic method makes use of only the initial portion of the load-settlement curve it is still able to predict the ultimate load satisfactorily. The predicted ultimate load is slightly more than the most probable ultimate load in the case of driven piles, and slightly less in the case of piles tested by constant rate of penetration method. For bored piles, the method gives almost the same ultimate load as the most probable ultimate load.

7. In cohesive soils, quick maintained load test gives a higher ultimate load than the slow maintained load test. The differences in the ultimate loads are significant for bored piles, however, marginal for driven piles.

5.2 **Recommendations**

Some of the recommendations that can be made for future research are :

1. The effect of boundary conditions on the pile performance need to be studied. Preferably, the test tank sides should be made flexible in order to study the effect of lateral stress on pile performance.

2. The effect of loading history should be studied very thoroughly using various kinds of prior loading like accidental impact loading, maintained loading which do not cause failure etc.

3. The effect of surcharging the soil is not an exact simulation of overburden pressure. Yet, since it is probably the best way of simulating the overburden pressure in the laboratory, more detailed study as to the way the pressure transmits downward, effect of time etc. should be studied.

4. Sand crushing and variation in grain size distribution due to driving and surcharge loading should be critically analysed.

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