PERFORMANCE OF AXIALLY LOADED SMALL SIZE PRESTRESSED CONCRETE PILES

A Thesis

By

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ABSTRACT

The thesis deals with the performance of small size (175mmX175mm) prestressed pile. Maintained load test were conducted to investigate the load carrying capacity and settlement of the piles. Four sites within Dhaka City were selected for the purpose.

The measured capacity determined from pile load tests was compared with the predicted capacity using static methods & dynamic methods.The measured ultimate capacity of piles driven through Dhaka Clay and resting on Dhaka Clay is in close agreement with the predicted values using λ method.The skin friction of piles driven through Dhaka Clay predicted by α_2 method is slightly smaller than the estimated value from pile load test. This method can safely be used for predicting the ultimate skin friction of Dhaka Clay. It is observed that API method grossly underestimate the skin friction of piles in Dhaka Clay.

The ultimate capacity of piles driven through Dhaka Clay and resting on medium dense sand can be predicted using the combinations of λ method and Meyerhof's empirical method, λ method and Hansen's method,α method and Meyerhof's empirical method, α method and Hansen's method, α_2 method and Meyerhof's empirical method, α_2 method and Hansen's method. The value of ultimate capacity predicted by API method and Indian Standards method is about half of the measured ultimate pile capacity determined from pile load test.

It is also observed that the ultimate capacity predicted by pile driving formulae such as Engineering News Records formula, Janbu formula and Hiley formula, in general, overestimate the measured ultimate capacity. However these formulae underestimate the allowable capacity when used with the recommended factors of safety

LIST OF NOTATIONS

A = pile cross-sectional area

a = coefficient used in Gates formula

A_{tip}= area of pile tip

A_s = area of pile shaft

A_r = ram x-sectional area = cross-sectional area of column

B = width (least dimension) of pile

b = coefficient used in Gates formula

c = undrained cohesion of the soil

D = depth of embedment of pile (depth of foundation)

D_c = Critical depth

d_r = relative density of soil

d_q = depth factor

C_a= adhesion per unit area of pile

C_d = coefficient in Janbu method

C_1, C_2, C_3 = coefficient used in Canadian National Building Code formula/PCUBC formula for Dynamic analysis of pile capacity

E = modulus of elasticity

e_h= hammer efficiency

E_h = manufacturer's hammer energy rating

f'_c = compressive strength of concrete

f_c = resistance measured by friction jacket

f_s = allowable stress in column vertical reinforcement/average ultimate unit skin friction

F_w = correction factor for tapered pile

h = height of fall of ram

I_{rr} = reduced rigidity index

k = coefficient used in PCUBC formula/coefficient of lateral earth pressure/foundation modulus

k_s = coefficient of earth pressure at rest

k_1 =elastic compression of cap block

k_2 = elastic compression of pile

k_3= elastic compression of soil

k_u = coefficient used in Janbu formula

L = pile length

L_b = pile penetration depth into point bearing stratum

LL = liquid limit

$M = \tan\delta|/\tan\phi$

N_t = bearing capacity factor for pile foundation in the method recommended by Canadian Geotechnical Society .$N_t > N_q$

N_p = standard penetration blow no at pile base

n = co-efficient of restitution/layers of soil in which pile is installed

N =average value of N along pile shaft

N = number of blows per 0.3m penetration in standard penetration test

N' = SPT value found in the field

N_c, N_q, N_λ = bearing capacity factor for deep foundation

p = steam or air pressure

PL = plastic limit

P_{Di} = effective overburden pressure for the ith layer where i varies from 1 to n

Q = total ultimate pile capacity

Q_s = total skin friction

Q_{tip} = total end bearing

q = average effective overburden pressure over the embedment depth of pile

q_u = unconfined compressive strength/unit end bearing

q = effective overburden pressure

q_c = cone resistance

s = amount of point penetration per blow

W = weight of ram + weight of casing

W_r = weight of ram

W_p = weight of pile

Z_c = critical depth of embedment of pile

α = adhesion factor in α method of Tomlinson

α_2 = reduction factor in α_2 method of Peck et al

γ= unit weight of soil

ϕ = angle of internal friction

ϕ_t =friction angle determined from triaxial test

δ=angle of wall friction

6_1 =major principal stress

6_3 = minor principal stress

6_v = effective vertical stress at the level of pile

$6'_D$ =unit effective vertical pressure at pile toe

β= coefficient in β method

λ = coefficient in λ method/a coefficient in Janbu method

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 GENERAL

With the increase of population and development activities more land are required for construction. As our ancestors used high and firm lands for the construction of buildings and other structures, there is scarcity of suitable lands for new construction in all the urban areas. Natural subsoil condition in many areas does not allow to build heavy structures due to these reasons. Therefore it becomes necessary to transfer the superstructural load to deeper strata through piles.

Piles are normally called point bearing when load is carried mainly by tip and they are called friction piles when they are supported mainly by the adhesion or frictional resistance of the soil along the shaft of the pile. Piles may be classified on the basis of materials of which they are made of and on the basis of manner of installation. On the basis of materials of which they are made piles may be classified as: timber piles, concrete piles and steel piles. Depending on the method of installation piles may be classified as: driven piles and bored piles. Driven piles may further be classified into precast piles, prestressed piles, driven and cast in situ piles , composite piles and steel piles.

There are areas where the top soil is poor and not suitable for shallow foundation. Conventional RCC piles are costly for light structures in such subsoil condition.Matured timber piles are also costly. Above all they are subject to decay under fluctuating water table zone.In such a circumstance small size prestressed pile are used as a substitute of timber pile.The piles are of 175 mm x 175 mm in cross section and the length varies from 5.0m to 7.5 m. These piles have been installed and used in many areas of Dhaka Metropolitan City and elsewhere in Bangladesh.

Although the size of small size prestressed pile does not conform to any standard they may be compared with timber piles, short precast piles used in San Antonio, Texas and Pedestal piles used in India. In San Antonio, Taxas, USA, precast piles of 225mm diameter and length 3 to 4.8m are used for light structures to avoid heaving. In India pedestal piles of section 10 cm x 10 cm and length upto 3 m are in use specially in black cotton soil. Details of their construction are available in CBRI Building Research Note No. 29 (1986)*.

1.2 AREA OF RESEARCH

In Bangladesh the use of small size prestressed pile is increasing day by day. However little information is available about the performance of these piles.It is necessary to carry out a detail investigation on different aspects of these piles. The capacity of pile may be obtained by a full scale load test. The pile capacity determined from pile load test may be compared with the capacity predicted by static methods and dynamic methods. In view of this, the main objectives of the present study are as follows:

(i) To investigate the method of construction and installation of small size prestressed pile

(ii) To drive and carry out pile load test at 4 different locations

(iii) To compare the capacity of piles determined from load test with the predicted values using other methods .

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

Foundation is the part of the structure which remains in direct contact with the ground and which transmits the load of the structure to the ground. Shallow foundation (such as isolated column footing and strip footing) are used where the bearing capacity of the top soil is good enough to bear the load of the structure without detrimental settlement. .

Pile foundation are used in situations where the soil at shallow depth can not support the imposed load safely.Piles are used to transmit this load to deep soil strata. Piled foundations are also used for supporting structures built over water or where uplift loads are to be resisted. Generally, a pile foundation is more expensive than a shallow foundation.

2.2 TYPES OF PILE

Piles are columnar elements in a foundation which transfer load from superstructure through weak compressible strata or through water, on to stiffer or more compact and less compressible soils or onto rock. They resist uplift loads when used to support tall structures subjected to overturning forces from winds,earthquake or waves. Piles used in marine structures are subjected to lateral loads from the impact of berthing ships and from waves.Combinations of vertical and horizontal loads are carried where piles are used for retaining walls, bridge piers and abutments, and machinery foundations.

Piles are classified in different ways. They may be classified according to:

(i) Material of the pile
(ii) Method of installation
(iii) Load carrying mechanism

According to the materials of which they are made, piles may be classified as:

(i) Timber piles
(ii) Concrete piles and
(iii) Steel piles

Depending on the methods of installation, piles may be classified as:

(i) Driven piles
(ii) Driven and cast- in -situ piles
(iii) Bored and cast- in-situ piles
(iv) Screw piles

Depending on the load carrying mechanism, piles may be classified as:

(i) Friction piles
(ii) Point bearing piles
(iii) Compaction piles
(iv) Uplift piles

Timber Pile

Timber piles are made of tree trunks with branches trimmed off usually treated with preservative and driven with small end as a point. The tip may be provided with a metal driving shoe when the pile is to penetrate hard soil; otherwise it may be cut either square or with some point.

Untreated timber piles that are fully embedded in soil below the permanent fresh ground water level may last for many years. Where pile remains above ground water level, they may be subject to decay. Preservative treatment is effective in preventing decay.

The main advantage is that the timber piles are relatively inexpensive and comparatively light.They are very easy to transport. This type of piles may be installed with the traditional drop hammers. Main disadvantage of timber piles are that they are damaged during hard driving and are subjected to decay. Timber piles lose strength when subjected to prolonged high temperatures and cannot be used under such structures as blast furnaces and chemical reaction units. Teng (1962) suggests that the design capacity of timber pile should be limited to 25 tons (empirically) to avoid hard driving .

There are limitations on the size of the tip and butt as well as misalignment that can be tolerated. Different codes have different requirements. According to ASTM D - 25[*], minimum tip diameter should be 125mm.The Chicago Building Code[**] requires minimum tip diameter 150 mm and butt diameter 250 mm if the pile length is under 7.6 m and butt diameter 300 mm if pile length is more than 7.6m. The alignment requirement is that a straight line from the centre of butt to the centre of tip lie within pile shaft.

The New York Building Code[**] limits timber pile with a uniform shaft taper to a tip diameter of 150 mm for loads under 220 kN and minimum tip diameter of

* Cited by ASCE Deep Foundation Committee (June 1984)
** Cited by Bowles (1988)

200 mm for larger loads.The specification of a timber pile recommended by ASTM D 25* is shown in Fig. 2.1

Timber piles are widely used in Bangladesh.Teak,Sal,Gajari,Sundari types of timber are used as piles.There is no specification of timber piles in Bangladesh National Building Code (1993) but they are extensively used throughout the country. A large number of low to medium rise buildings have been built over timber piles. However, severe damage or failure of any of such buildings are not yet known. The Sal, Gajari & Sundari are available as round timbers with bark stipped. To facilitate driving, the bottom ends of piles are made pointed & some times fitted with steel shoe. The butt diameter of timber piles (round) of Bangladesh varies from 150mm to 300mm. However, tip diameter of timbers available in the open market varies from 75mm to 150 mm.

Steel Piles

Piles in this category include those for which the load-bearing material is exclusively steel. These piles can develop high capacities but must be of adequate cross-sectional area to withstand driving stresses and to provide the necessary stiffness or drivability characteristics to achieve proper penetration and bearing capacity. Steel piles come in several cross-sectional shapes, including H, round pipe (open or closed-ended), rail, and box sections. They can be readily spliced by welding to provide any required length.

The long-term structural capacity of these piles may be affected by environmental conditions that could cause loss of metal from such actions as oxidation, corrosion or electrolysis. Under such conditions all exposed steel surfaces should be protected by a suitable coating or the pile should be designed with an adequate sacrificial "skin". [ASCE Deep Foundation Committee (1984)] stated that no significant corrosion due to oxidation occurs for piles

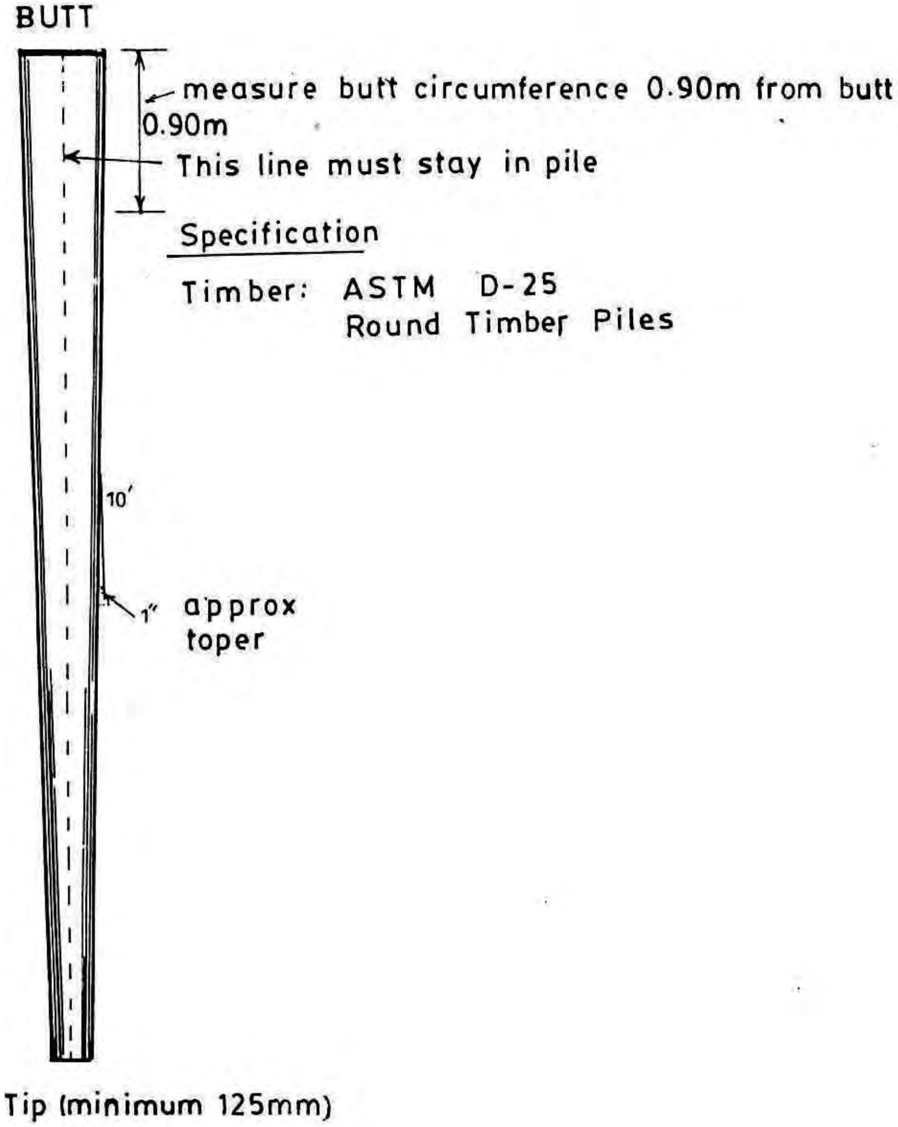


Figure 2.1 Typical timber pile

embedded in undisturbed soil. However, the most vulnerable portion of the pile is that portion in disturbed soils, especially directly under a pile cap, or in free water near the air water interface (splash zone).

The material cost for steel piles is high relative to other types of piles.

Concrete Piles

Concrete piles are the widely used piles in construction practice with or without prestressing. The broad categories of concrete piles are precast piles and cast-in-place piles. Although concrete piles could be attacked by various chemicals, including those in sea water, these piles are generally not susceptible to environmental deterioration if they are made of high quality dense concrete and if reasonable precautions are taken to ensure that the concrete is undamaged after installation. For concrete exposed to high sulphate concentrations, a sulphate resistant cement with or without a pozzolan should be used. For piles exposed to a freezethaw condition, the use of an air-entraining admixture is recommended. A dense impermeable concrete will prevent the chlorides normally found in sea water from attacking the reinforcing steel. Under certain exposure conditions, a special protective coating may be required.

Precast Concrete Piles

Precast concrete piles are either conventionally reinforced or prestressed. They are commonly manufactured in square, octagonal, or round configurations and may be solid or contain a hollow central core.

Precast concrete piles theoretically can be manufactured in any size or length to meet the design conditions, but there may be practical constraints dictated by the limitations of the handling equipment, transport facilities or pile driving equipment. The use of sectional precast piles that will be joined together during driving may alleviate this problem but create others, such as increased costs, delays caused by splicing, inability to achieve further pile penetration after the splicing time delay (due to soil freeze), and loss of pile capacity as a result of the joint's breaking or separating due to improper construction, installation, or pile driving operations.

This type of pile is generally ordered in predetermined lengths. The cut off and waste cost could be relatively high where accurate pile lengths have not been determined by the designer or under variable soil conditions not revealed by a site investigation.

Inspection during the manufacture of precast concrete piles can help to ensure adherence to specifications and provide initial quality control. After manufacture, these piles can also be inspected for such properties as straightness dimensions, cracks, surface defects and the squareness of the butt end with the longitudinal axis.

Precast concrete piles must be properly designed, and must be handled and installed with care to avoid damage such as cracking, crushing or spalling.Damage that may occur below the ground surface during driving can not be detected by normal inspection. A careful review of driving logs, as well as the use of special integrity testing techniques, such as sonic or impact methods, can often reveal large cracks or breaks in the pile.

Precast piles, in general, are ideally suited for marine and trestle-type structures and are also used as foundation piles. If adequately designed and constructed, these piles can carry high compressive loads and bending moments when proper installation methods and techniques are used.

These piles are constructed of conventional reinforced concrete with internal reinforcement consisting of a cage made of four or more longitudinal bars and lateral or tie steel in the form of individual hoops or a spiral. Longitudinal steel arranged in symmetrical circular pattern is more effective than bars arranged in a square pattern, especially under seismic loading when ductility and core confinement become important factors. Tie steel should be closely spaced at the ends of the pile to help resist driving forces.

Reinforced precast concrete piles must be designed, manufactured, stored, handled and driven with care to avoid serious cracking. Minor cracking is virtually impossible to prevent. Cracks up to 0.15 mm in width are normally considered acceptable. If severe cracking or spalling occurs, or if the pile is made of poor quality concrete, the pile could deteriorate quite rapidly under adverse environmental factors such as a marine environment or freeze- thaw action.

Prestressed Concrete Piles

Prestressed concrete piles are constructed using steel rods, strands or wires under tension to replace the conventional longitudinal steel reinforcement used in the construction of reinforced precast concrete piles. The prestressing steel is tensioned either before (pretensioned) or after (post-tensioned) the concrete pile is cast.

The concrete is put into compression by the tensioned steel, which increases the ability of the concrete to withstand handling and driving stresses. Careful measurements of concrete strength and prestress force should be made during construction, as these factors influence structural pile behaviour significantly. Since the concrete is under continuous compression, hairline cracks are kept tightly closed, thus prestressed piles are more durable than reinforced precast piles. Tensile stresses that develop during driving are reduced because of the effective compressive prestress; however, the allowable compressive stress due to externally applied forces and moments will be reduced by the amount of effective prestress.

Prestressed concrete piles are capable of withstanding prolonged hard driving, and they may be handled without damage. However, spalling, cracking and breaking can occur if precautions are not taken. Prestressed piles are generally less permeable than reinforced precast piles and exhibit superior performance in a marine environment. Prestressed piles can be cut off to the required grade without losing the effective prestress except when unbonded strands or tendons are used.

Pretensioned prestressed concrete piles are generally manufactured in a permanently established plant. The stressing steel is placed and tensioned before piles are cast, which requires adequately constructed stressing beds to maintain the stressing forces and pile alignment as the concrete cures.

Stressing steel is usually enclosed in a steel spiral, having a maximum 150 mm pitch throughout the central third of the pile, with closer spacing over the remaining two-thirds of the pile. Several tight turns are used at each end to help withstand driving forces . ACI-318-95(1995) recommended clear cover of 25mm and 37.5mm for structures exposed to earth However, ASCE Deep Foundation Committee (June,1984) recommended a clear cover of 50 mm in typical design of a prestressed pile as shown in Fig.2.2.

Most commonly used sizes of pretensioned prestressed piles range from 250 mm square to 600 mm square or octagonal. Pretensioned hollow-core cylinder piles have been produced in diameters up to 1676mm. However, smaller sizes have also been used.

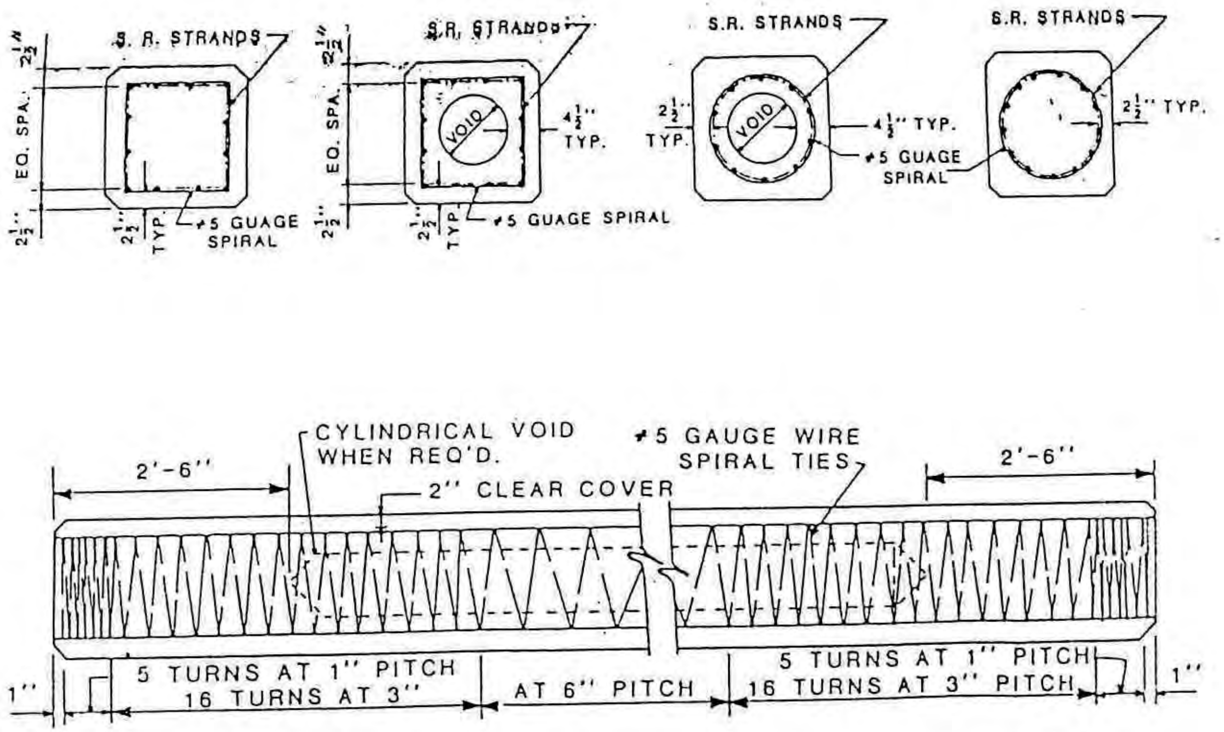


Figure 2.2 Typical prestressed concrete pile according to ASCE Deep Foundation Committee (June , 1984)

Sunway P M I-Pile Construction Sdn Bhd of Malaysia (a member of SungeiWay group) introduced a new piling system called Precast Micro Injection Pile System[PMI-Pile (1986)] which is noise free,vibration free and pollution free. Piles are cast using a low water-cement ratio designed mix with a cube strength of minimum 50 Mpa and are prestressed by high tensile strength wire of diameter 5.0mm. While prestress in the concrete pile takes care of all handling stress the axial load capacity of the piles is derived mainly from the concrete strength.These piles are driven by the technologically advance Injection method of pile driving and hence no transverse reinforcements are necessary which is not the case in all other hammer driven piles. These piles are manufactured in segments and can be manually lifted. The piles are joined by means of steel sleeve and high strength epoxy. The joints are found to be stronger than the normally used timber joints.

Common sizes of PMI piles are:

(i) the 100mm x 100mm size piles which comes in 3.0m lengths

(ii) the 125mm x 125mm size piles which comes in lengths of 3m,4.5m and 6m.

(iii) the 150mm x 150mm size piles which come in standard length of 3m and 6m.

The pile has clear cover of 27.5 mm

The 100mm×100mm size pile of length 3.0m was test loaded to 24 tons while driven by injection system.The 125mm×125mm size pile of length 4.5m and the 150mm x 150mm size pile of length 6.0m was test loaded to 40 tons and 52 tons respectively while they were driven by injection system.Typical PMI pile is shown in Fig.2.3.Details may be available from PMI-Pile (1986).

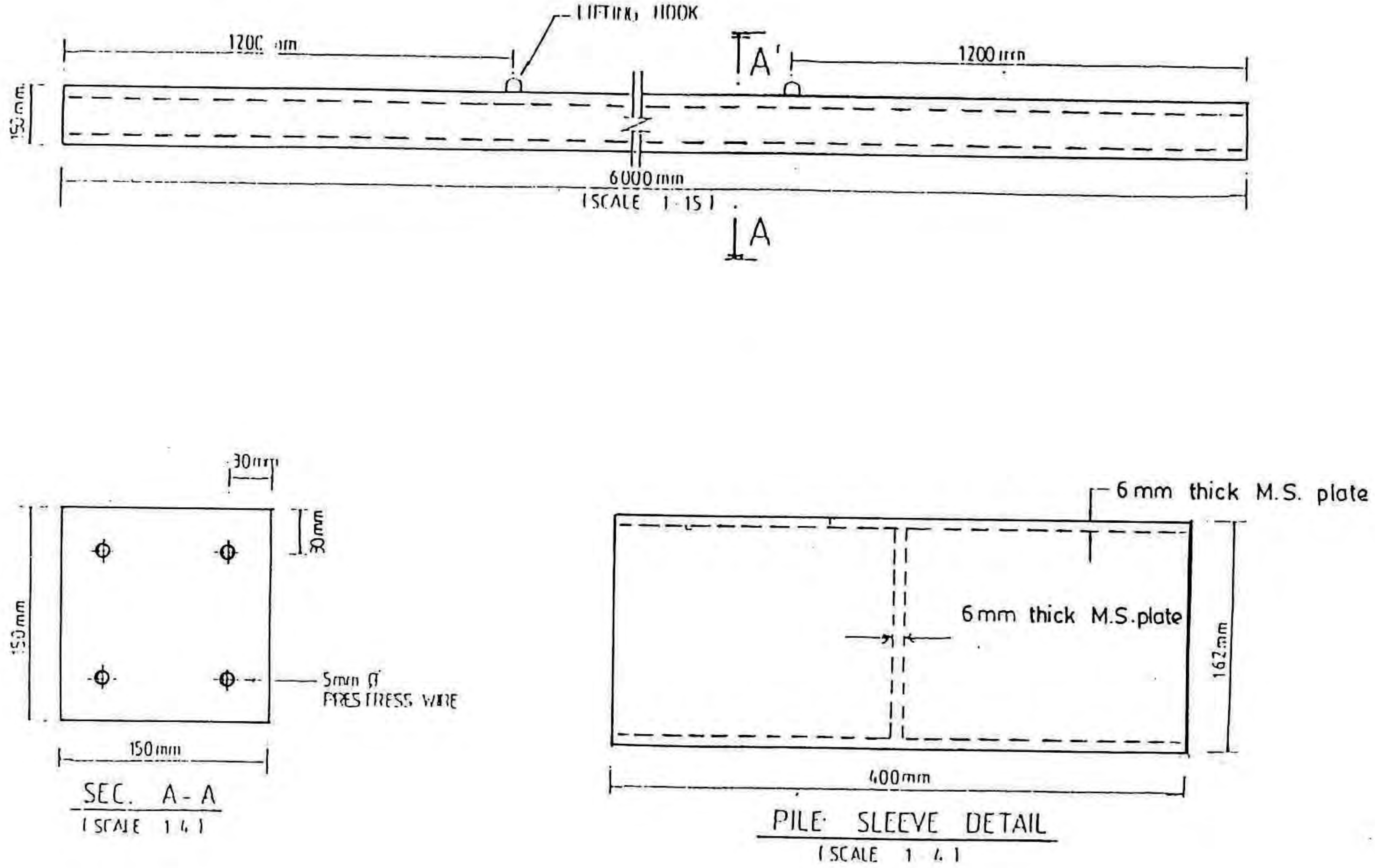


Figure 2.3 Prestressed Micro Injection pile. [(PMI Pile(1986)]

In Bangladesh, Housing and Building Research Institute (HBRI) and some private enterprises are making prestressed piles of size 175mm square and 150 mm square and upto the maximum length 7.5m. In these piles, there are no arrangement for pile splicing. The piles are constructed with clear cover of 25mm

These piles are lifted and shifted manually. The traditional timber pile driving equipment are used for driving this type of piles. These piles are used as substitute of timber piles. In Dhaka Metropolitan City and elsewhere in Bangladesh several projects have been successfully completed using these piles.The typical section of a small size prestressed pile is shown in Fig. 2.4. Codes have not mentioned about the minimum size of these piles . However, these piles may be compared with timber piles.

Bored And Cast In Situ Piles

This type of pile, known also as a caisson pile, bored pile, drilled shaft, or drilled pier, is installed by drilling a hole in the ground to the required depth and filling the hole with reinforced or plain concrete.Sometimes the pile shaft is socketed into rock or underreamed to form an enlarged base (bell) in the soil to increase the bearing area. lengths exceeding 30m are possible in favourable (stiff or dense) soils. The principal advantages of the this type of pile are cost and minimum vibration during installation. When bored pile is installed in caving or pervious waterbearing soils, it is usually necessary to drill the hole under a head of water or bentonite slurry and/or to use a temporary or permanent steel liner. When slurry is used it is usually displaced by tremie-placed, high slump concrete.

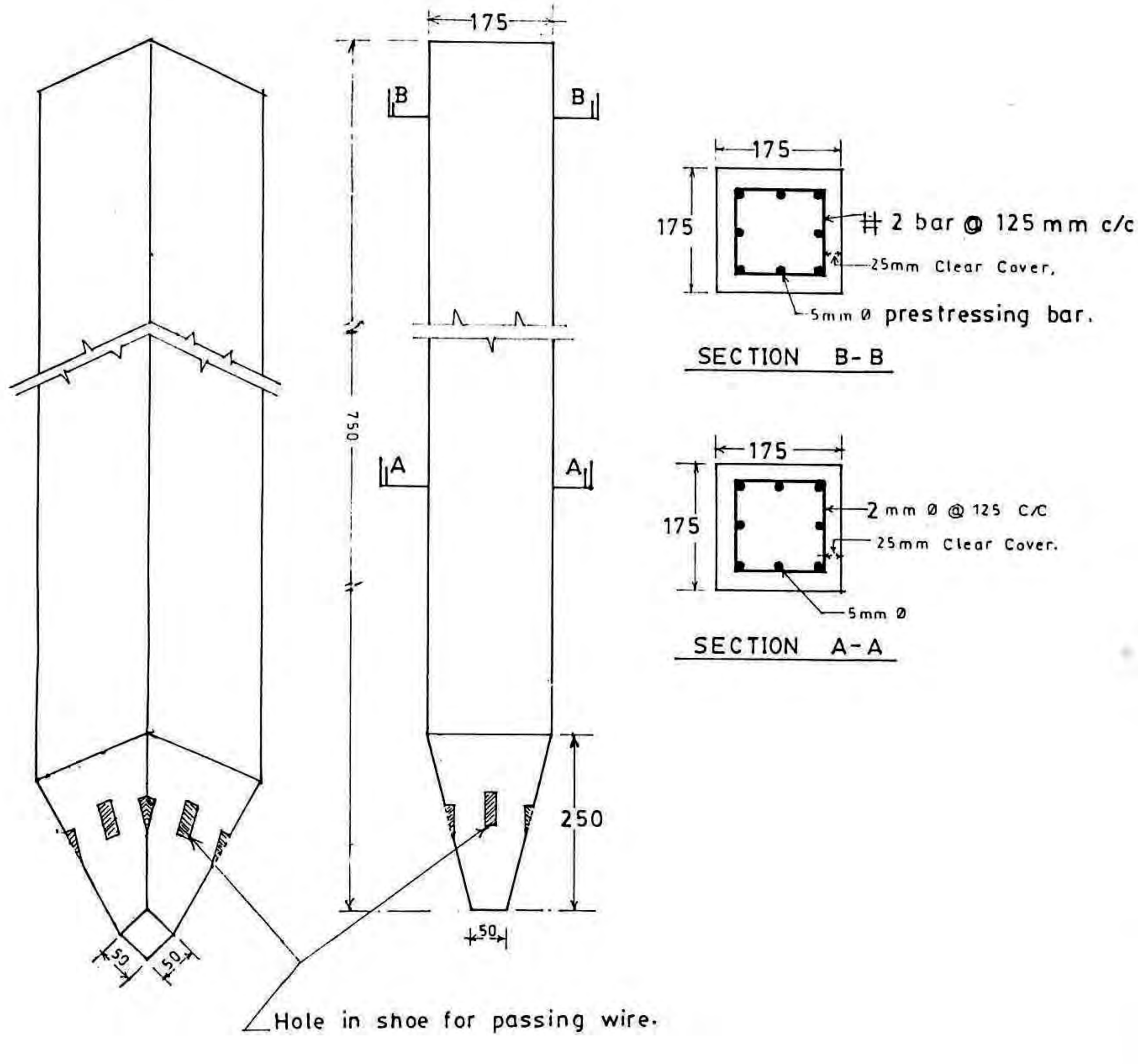


Figure 2.4 Typical section of 175 x 175 size prestressed pile.

Construction of enlarged bases is usually carried out with underreaming tools.These tools are not, in general,effective in removing cuttings from the base of the bell when underreaming is attempted under water or slurry.

Bored piles are generally reinforced with a cage of longitudinal and spiral steel extending below any zone of significant bending moments or deformations in the soil. If piles are to be installed with the hole full of water or bentonite slurry, the spacing of the spiral reinforcement must be constructed to prevent trapping of clay balls or cuttings within the concrete. When temporary liners are used it is desirable to weld the reinforcing cages, especially if batter piles are used, to prevent unraveling of the steel caused by contact with the liner when it is removed.Construction procedures are critical to the quality of the bored pile, and very careful inspection is required.

Piles with enlarged bases are generally designed as end-bearing units. Straight shaft drilled hole piles may derive none, part or all of their bearing capacity from skin friction, depending upon the stiffness of the soil at the base of the shaft.

This type of pile generally should not be installed with diameters less than about 400mm. Successful completion of drilled hole piles through very soft or very loose soils is often difficult unless liners are left permanently in place. If slurry displacement is employed, the concrete should be placed upto the working surface such that all slurry and slurry-contaminated concrete is displaced from the hole. This makes it relatively expensive to construct such piles where cut-off elevations are significantly below the working surface.

Micro-Piles

A micro-pile is a small-diameter pile constructed by forming a borehole and sealing into it a steel tube with a high strength grout injected under pressure through the tube. The grout injection forms bulbs along the shaft of the pile, through which the pile derives added frictional resistance. The method of execution has been developed and perfected to give a high quality pile.

A micro-pile is constructed by a small rotary-cum-percussion rig which can work in low headroom and small working space, using bentonite mud to retain the sides of the hole. Due to the specialised installation process, the vibrations and disturbances to the surrounding soil and structures are minimised. These piles are ideally suited for piling in restricted area and/or low headroom and close to the existing structures, such as in case of underpinning. The piling can also be used for dock floors, where tensile forces are to be carried. More feedback studies and case records describing application of micro-piling are however necessary [Mohan(1988)]. Micro-piles are small diameter bored piles with steel tube as reinforcement [Sabani and Sapio (1981)].

Pedestal Piles

Pedestal piles have been introduced by CBRI (India)* as an economical substitute of underreamed piles, for light structures and buildings. They are specially recommended for foundation on expansive clays. According to CBRI Building research Note No. 29 (1986) * they consist of precast reinforced concrete stem of 10 cm x 10 cm section with 30 cm high concrete pedestal cast in 30 cm diameter auger holes. Field tests carried out on these piles in loose saturated sand have revealed that the piles can take about 22 kN in compression and 8 kN in uplift. Studies, extended to Indian black cotton soil (highly

* Cited by Mohan (1988)

expansive clays) revealed safe loads in compression and uplift of 35 kN and 17.5 kN respectively, for a 3 m long pile.

End Bearing and Friction Piles

All the piles again may be classified as friction piles & end bearing piles depending on load carrying mechanism. A pile obtains its support from both the frictional forces on the surface of its shaft and from direct bearing on its base or point. However, generally one of these components predominates and the division into end bearing and friction pile is a convenient terminology.

If the bearing stratum for foundation piles is hard and relatively impenetrable material such as rock or a very dense sand and gravel, the piles derive most of their carrying capacity from the resistance of the stratum at the tip of the piles. In such conditions they are called end bearing or point bearing piles (Fig.2.5).

If the piles do not reach an impenetrable stratum but are driven for some distance into a penetrable soil their carrying capacity is derived partly from end bearing and partly from the skin friction between the embedded surface of the pile surrounding the soil. Piles which obtain greater parts of their capacity by skin friction or adhesion are called friction piles (Fig 2.6). Friction piles are used when hard stratum or bed rock is deep which would require very long end bearing pile.

2.3 DETERMINATION OF PILE CAPACITY

The ultimate capacity of a pile is determined on the basis of two main considerations:

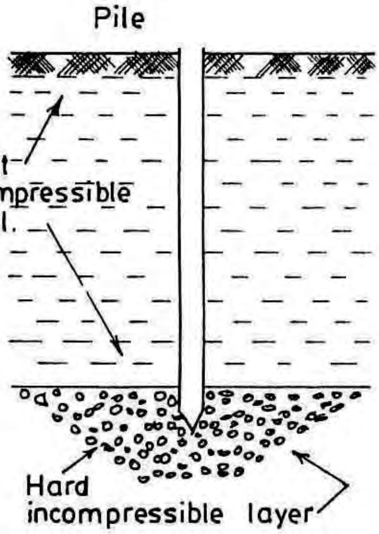


Figure 2.5 End bearing pile

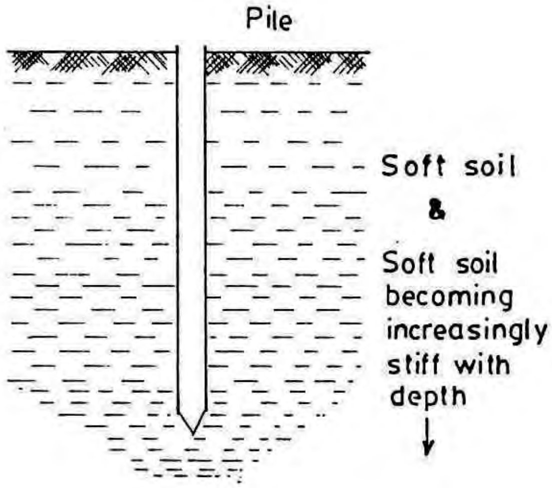


Figure 2.6 Friction pile

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

ii) The capacity of the soil to support the load transmitted from the pile.

The pile design load is smaller of the two divided by a suitable factor of safety.

2.3.1 Structural Capacity

There are two basic approaches in use for determining the structural strength of a pile. They are:

(i) Fixed values of allowable unit stress

(ii) Capacity of pile as a column

Fixed values of allowable unit stress in pile vary widely, depending on codes ,even for exactly the same materials in the same type of pile.Typical values of allowable unit stresses for design of precast concrete piles are shown in Table 2.1

Determination of the capacity of pile considering pile as a column is a rational design approach. The design of a column is a function of the following basic parameters.

(a) Strength of the material

(b) The stiffness of the materials

(c) The equivalent unbraced length

(d) The cross-sectional area of the column.

(e) Soil parameters surrounding the pile.

Table 2.1 Typical values of allowable unit stress for design of precast concrete piling*

Source	Allowable Unit Stress, psi — Reinforcing steel	Allowable Unit Stress, psi — Concrete	Remarks
National standards			
1. National Research Council Survey (1962) [3]	Varies: 3,500-24,000 psi. Also referenced as $0.35F_y$	Varies: 460-3,000 psi. Also expressed as function of f_c' with values varying from $0.225f_c'$ - $0.45f_c'$	
2. "Pile Foundations" (1963)	$0.40F_y$	$0.225f_c'$	
3. National Building Code (1967)		$0.225f_c'$	
4. BOCA (1963)	$0.34F_y$	$0.225f_c'$	Designed as short column. Indicated values are for tied columns (ACI-318) [4].
5. Southern Standard Building Code (1961)			Ostensibly piles are designed as short columns. Maximum load value is stipulated, however as: load (in tons) = 2.2 x side dimension (in inches).
6. Uniform Building Code (1961)	$0.34F_y$	$0.225f_c'$	
7. Building Code of City of Boston (1962)			Maximum allowable load of 50 tons on 12 x 12 pile of f_c' = 4000 psi. Increase proportionately to area to a maximum 90-ton load.
8. Building Code of City of St. Louis (1961)	$0.34F_y$	$0.225f_c'$	Designed as short column. Indicated values are for tied columns (ACI - 318) [4].
9. Building Code of City of Baltimore (1955)	$0.40F_y$	$0.225f_c'$	Designed as reinforced concrete columns having L/d = 10.
10. Los Angeles Building Code (1958)	$0.34F_y$	$0.225f_c'$	
11. Building Code of City and County of Denver (1962)	$0.40F_y$	$0.225f_c'$	
12. Building Code of City of Chicago (1963)			Designed as column. Assumed unbraced length not specified.
13. Building Code of City of Buffalo (1965)		$0.25f_c'$	

* Cited by Johnson and Kavanagh (1968)

Evaluation of pile soil interaction is a complex problem. There are expressions for stress and equivalent unbraced length which are complex. However, Johnson and Kavanagh (1968) summarized the results in simplified forms which are shown in Fig.2.7.

It has been reported by Whitaker (1976) and Mohan (1988) that piles of normal dimensions driven through soft soil to end bearing on some strong underlying stratum do not buckle under load, provided the soil has some shear strength and is not merely liquid mud. Bjerrum (1957)* reported that 30.5 m long steel piles made from flat bottom rail 12 cm wide and 11.6 cm high were installed through soft clay of shear strength ranging from 14.3 kPa to 33.5 kPa which did not buckle under load. As such, generally piles embedded in soil are designed as short columns. Normally, structural capacity does not govern in the case of piles. The capacity of pile considering soil condition is generally smaller than the structural capacity.

2.3.2 Capacity of Pile as Determined by the Supporting Strength of Soil

The bearing capacity of a pile,considering the strength of surrounding and underlying soil can be estimated by number of methods. They are :

(i) Static methods using soil parameters

(ii) Empirical methods using static field penetration tests

(iii) Dynamic formulae which estimates the load capacity of driven piles from the pile driving records.

(iv) Wave equation.

(v) Pile load test.

*Cited by Whitaker (1976)

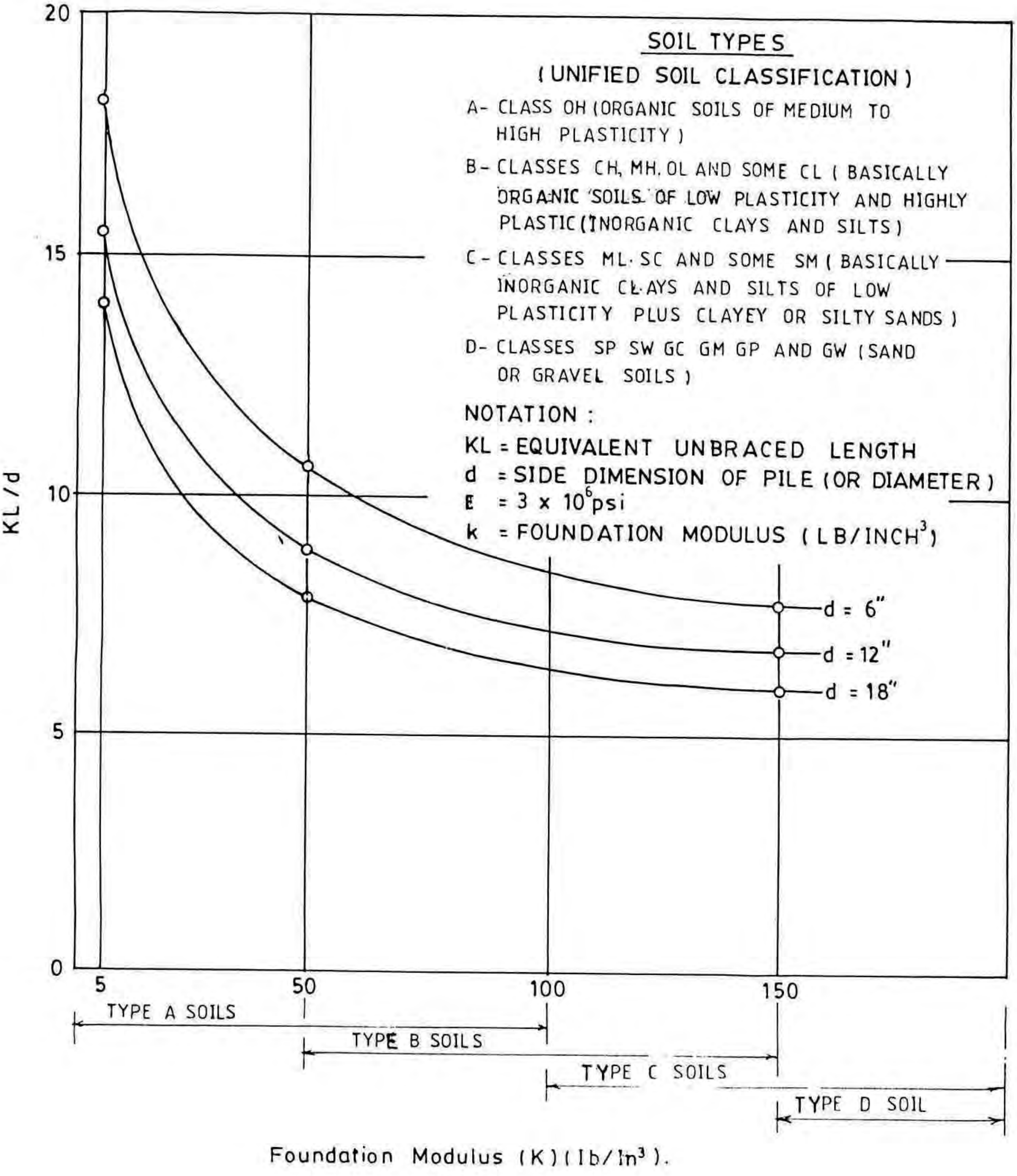


Figure 2.7 Pile column Equivalent Unbraced length (Concrete piles)*

*Cited by Johnson & Kavanagh (1968).

2.3.2.1 Determination of Pile Capacity by Static Methods

For a pile having adequate structural strength the total downward capacity Q is based on soil condition. The ultimate capacity of a pile due to soil resistance developed by friction between the soil and pile shaft and end bearing at the tip of pile is

$$Q_u = Q_s + Q_{tip} \qquad (2.1)$$

or,

$$Q_u = f_s A_s + q_u A_{tip} \qquad (2.2)$$

Where Q = total ultimate capacity of the pile

Q_s = total ultimate skin friction

Q_t = total ultimate end bearing

A_s = total surface area of the pile in contact with soil along the embedded shaft length.

f_s= ultimate unit skin friction between soil and pile surface

A_{tip} = pile tip bearing area

q_u = ultimate unit end bearing capacity of soil at pile tip

There are several methods to compute skin friction (f_s) and end bearing (q_u). The analysis differs for clay soil and for sandy soil. Where strata of soils possessing different properties are penetrated or pile cross-section and surface area vary along its length,the skin friction can be calculated by using segments of pile length and the appropriate soil parameters and pile area.

End Bearing of Pile

Various theoretical solutions are proposed for the problems of bearing capacity from about 1934. Terzaghi (1943) proposed the following general equation for calculating the ultimate bearing capacity:

$$q_u = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma \qquad (2.3)$$

Where c = undrained cohesion of soil

B = width of foundation

γ = density of soil

D = depth of foundation

N_c, N_q & N_γ are the bearing capacity factors which depends on the angle of internal friction, ϕ. The values of N_c,N_q&N_γ proposed by Terzaghi are shown in Fig .2.8. Since then various researchers have worked on this subject. They used the basic Terzaghi equation but suggested different values for bearing capacity factors and introduced other factors such as shape factor,depth factor,inclination factor.

End Bearing of Piles in Cohesionless Soil

Important methods for calculating the end bearing of piles with its base in cohesionless soil are as follows :

Terzaghi (1943) and Terzaghi & Peck (1967) Method

Terzaghi & Peck (1967) suggest that for piles driven through compressible soil to a firm base the end bearing of a pile can be calculated from Terzaghi's (1943) general bearing capacity equation.(equation 2.3).

For pile with its base in sand, c = 0 and since, B is small compared to D, the terms containing N_γ may be neglected. Therefore, Terzaghi's general bearing capacity equation reduces to:

$$q_u = \gamma D N_q \qquad (2.4)$$

Where γ = Density of soil

D = Depth of foundation

N_q= Bearing capacity factors which depend on ϕ

Terzaghi's bearing capacity factor N_q may be determined from Fig. 2.8.

Meyerhof's Method (1956)

According to Meyerhof (1956)* the ultimate end bearing capacity of pile in cohesionless soil is given by the following equation :

$$q_u = \gamma D N_q \qquad (2.5)$$

Where γ = Density of soil

D = Depth of foundation

N_q = bearing capacity factor which depend on ϕ .

The values of Nq may be found from Fig-2.9. Meyerhof** assumed the failure pattern as shown in Fig.2.10.

* Cited by Whitaker (1976)

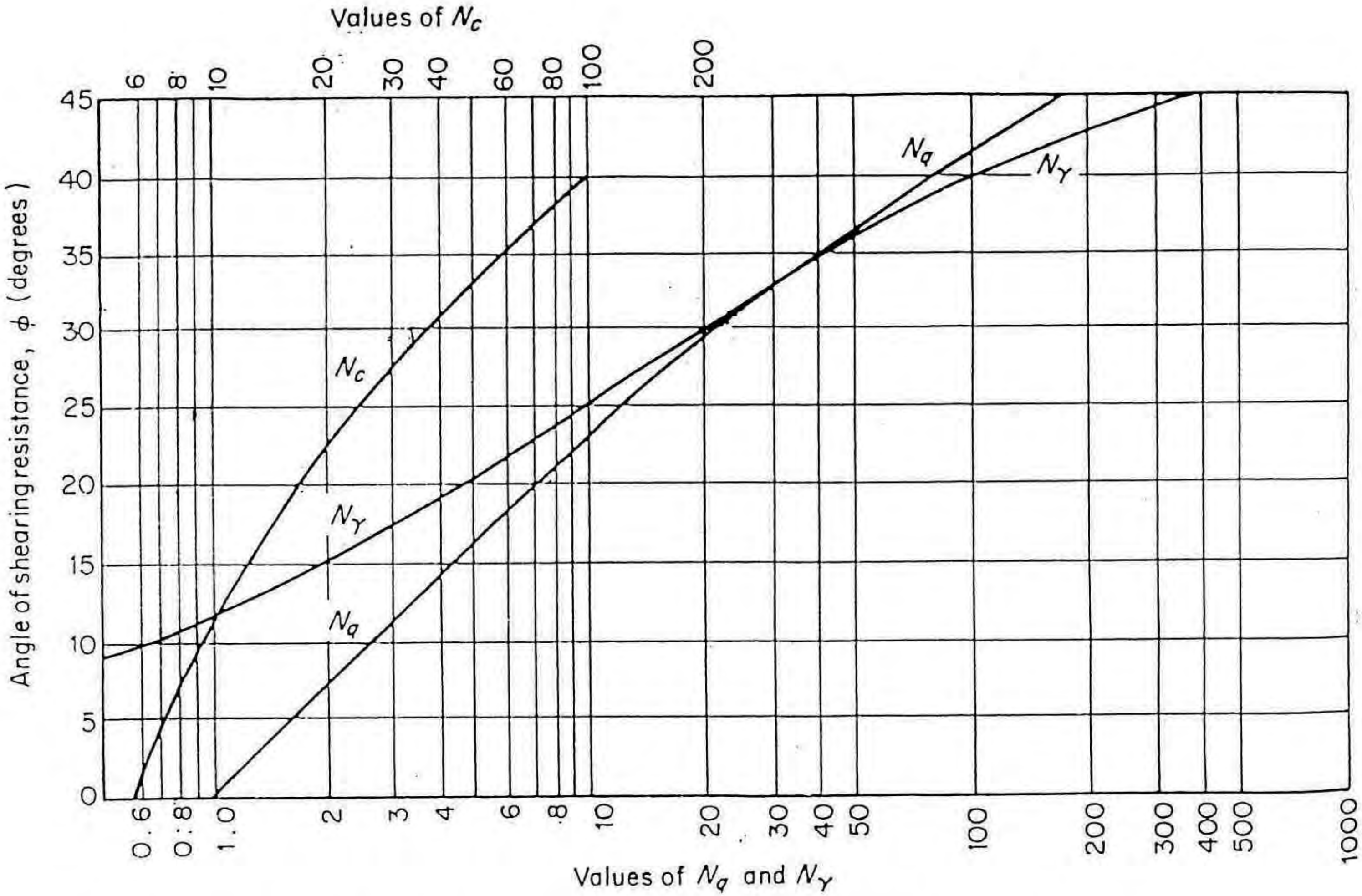


Figure 2.8 Terzaghi's bearing capacity factors for shallow foundations ($D \ngtr B$).*

*Cited by Tomlinson (1986)

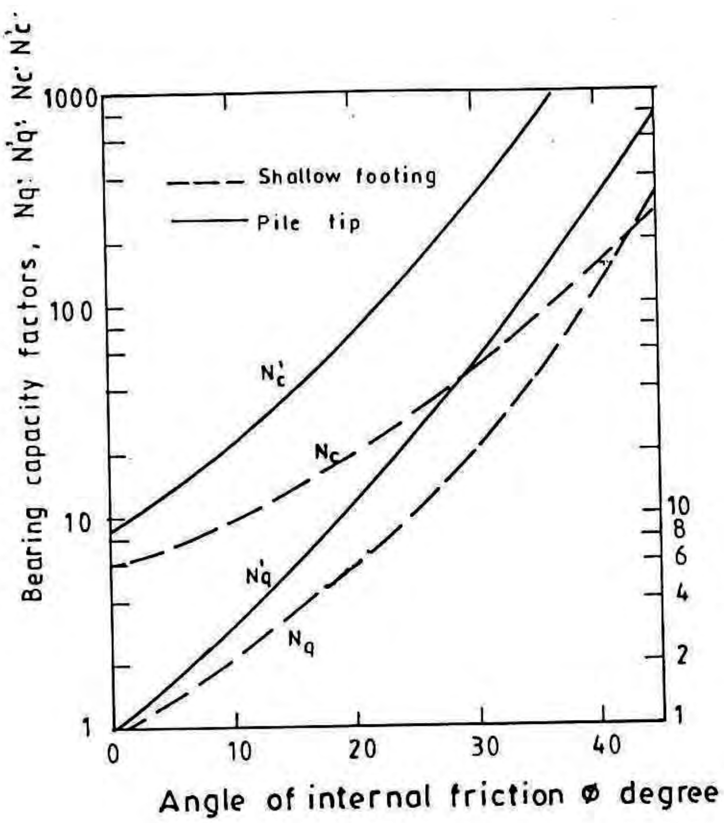


Figure 2.9 Bearing capacity factors according to the formula furnished by Meyerhof *

*Cited by Winterkorn & Fang (1979).

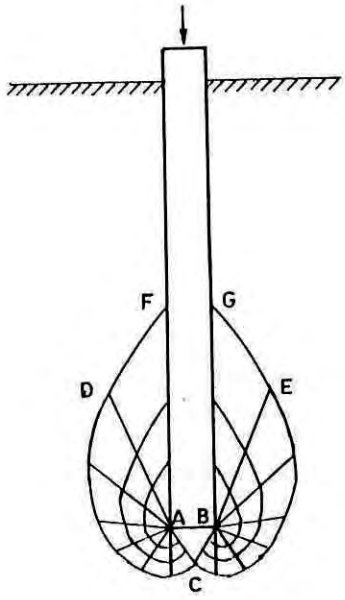


Figure 2.10 The Zones of Shear around the base of a pile according to Meyerhof (1951) *

* Cited by Whitaker (1976)

Brezantzev et al's Method (1961).

Berezantzev et al (1961)* proposed the following equation for finding ultimate end bearing capacity of pile in a granular soil.

$$q_u = \gamma D N_q \qquad (2.6)$$

Where q_u = ultimate end bearing capacity of pile

N_q = Berezantzev et al's bearing capacity factor which depends on ϕ & D/B ratio

N_q may be found from Fig. 2.11.

Hansen's Method(1970)

Hansen (1970)** proposed the general bearing capacity equation which includes the shape,depth and inclination factors.According to Coyle and Castello (981)** this equation is equally applicable to pile. For pile with its base in sand, c = 0 and since, B is small compared to D, the terms containing N_γ may be neglected.Therefore,the general equation of Hansen (1970) reduces to

$$q_u=\gamma D\, s_q\, d_q\, N_q \qquad (2.7)$$

Where N_q = Hansen's bearing capacity factor which may be obtained from Fig. 2.12.

$d_q = 1+2 \tan\phi \,(1-\sin \phi)^2 \tan\text{-}1\ D/B$

ϕ = angle of internal friction

γ = density of soil

$s_q = 1 + B/L \tan\phi$

* Cited by Tomlinson (1986)
** Cited by Bowles (1988)

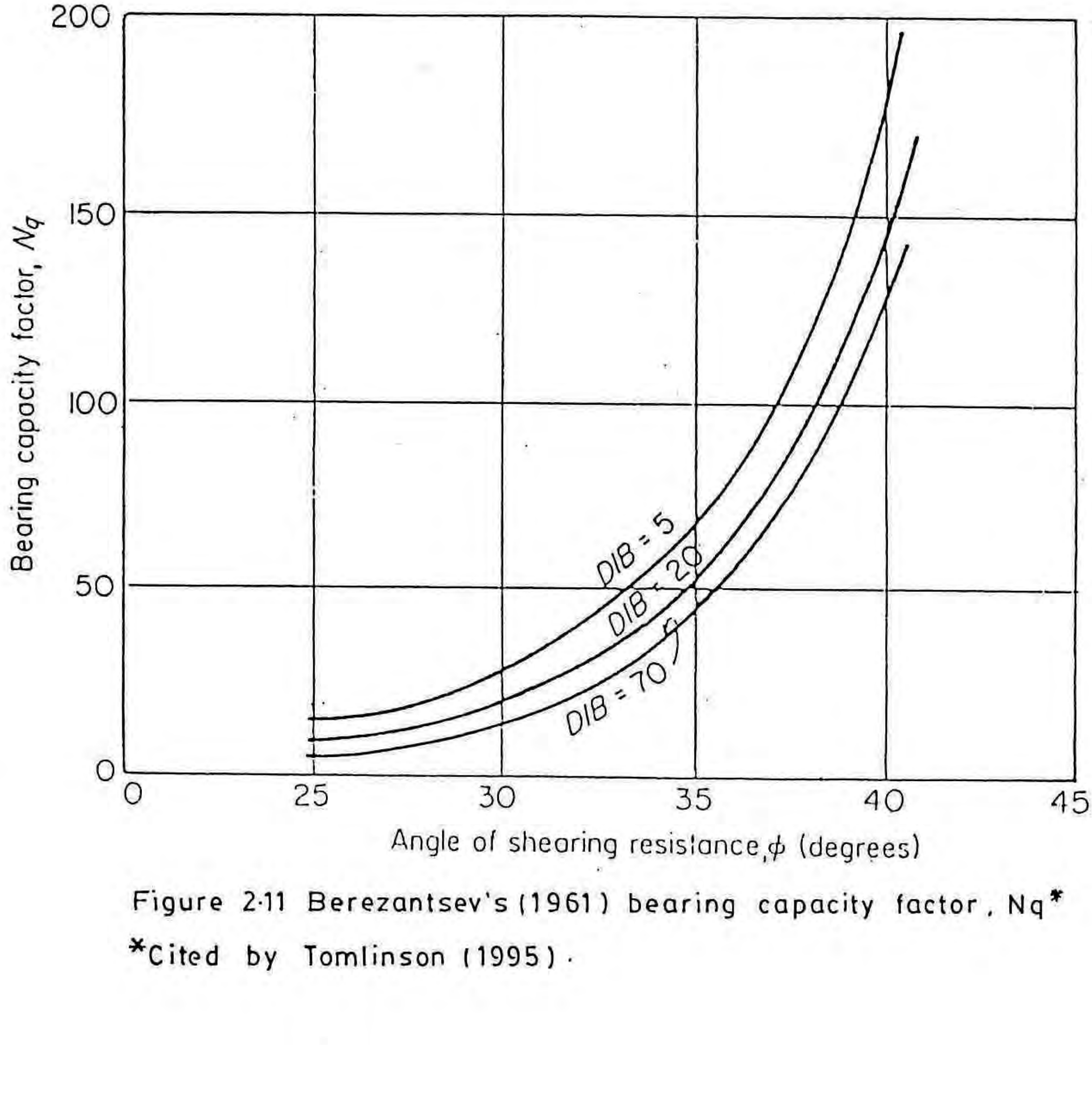


Figure 2·11 Berezantsev's (1961) bearing capacity factor, Nq*

*Cited by Tomlinson (1995).

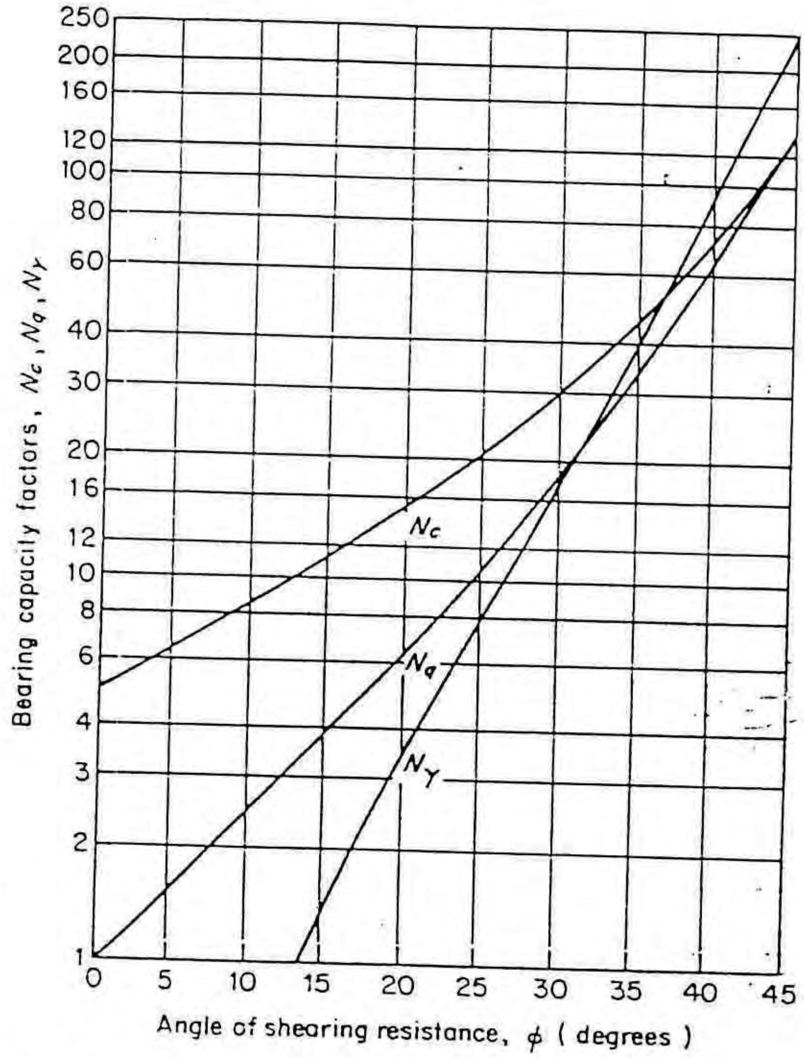


Fig. 2.12 Bearing capacity factors N_c, N_q, and N_γ [(after Brinch Hansen) (1961)]

*Cited by Tomlinson(1995).

It can be observed from the expression of depth factor that end bearing increases with depth and reaches a limiting value.

Vesic's Method (1977) :

Vesic (1977)* proposed a method of estimating the end bearing capacity of pile. According to Vesic(1977)*

$$q_u = 6_v N_q \qquad (2.8)$$

Where 6_v = mean normal effective stress at the level of pile tip

N_q =f (I_{rr})

I_{rr} = reduced rigidity index

Details may be available in Das (1984)

McCarthy's Method (1977)

McCarthy (1977) recommended that the end bearing of a pile driven in sand can be evaluated using the following equation:

$$q_u = 6_v N_q \qquad (2.9)$$

Where q_u = ultimate unit end bearing at pile tip

N_q = bearing capacity factor for deep foundations

6_v = effective overburden pressure acting at pile tip depth

McCarthy (1977) recommends that the value of N_q may be obtained from curves of Berezantzev et al (1961).The value of effective overburden pressure

* Cited by Das (1984)

6_v which develops at the pile tip is limited below the critical depth. According to McCarthy (1977) for design purpose it should be be assumed that the value of 6_v in the above equation is equal to the effective overburden pressure at the critical depth.

In the case of driven piles it has been found that the effective overburden stress of soil adjacent the pile does not continue to increase without limit, as implied by the above equations .Adjacent to a pile, the effective vertical stress increases only until a certain distance of penetration, termed the critical depth D_c is reached. Below this depth, the effective vertical pressure remains essentially constant or changes at a low rate. The point where the critical depth is reached is influenced by the initial condition of the sand (loose or compact) and the dimension of the pile. Field and model test indicate that the critical depth ranges from about ten pile diameters for loose sands to about twenty pile diameters for dense compact sand [McCarthy(1977)]. Fig. 2.13 provides information on critical depth for different conditions to use in pile design.

Method recommended by Indian Standards Institution (1979)

According to IS: 2911 - 1979, the end bearing of piles in granular soil is given by the following relation :

$$q_u = qN_q \qquad (2.10)$$

Where q = effective overburden pressure $=\gamma D$

N_q = bearing capacity factors depending on the angle of internal friction at pile toe.

γ= unit weight of soil

D=Depth of foundation

The N_q factors may be found from Fig.2.14.

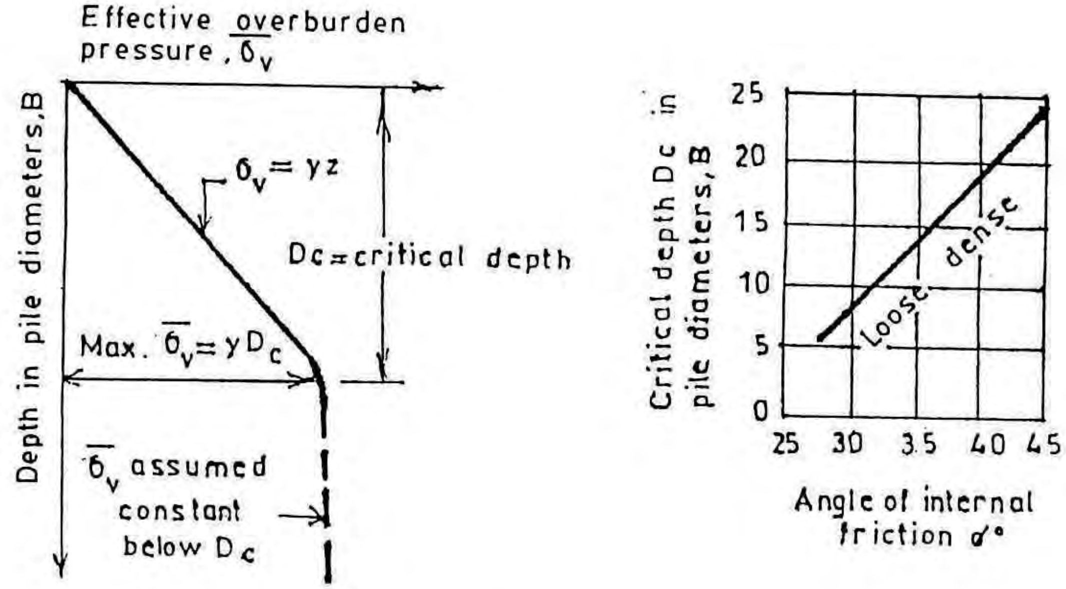


Figure: 2·13 Variation of effective overburden stress in sands adiacent to driven straight-sided piles [Mc Carthy (1977)] .

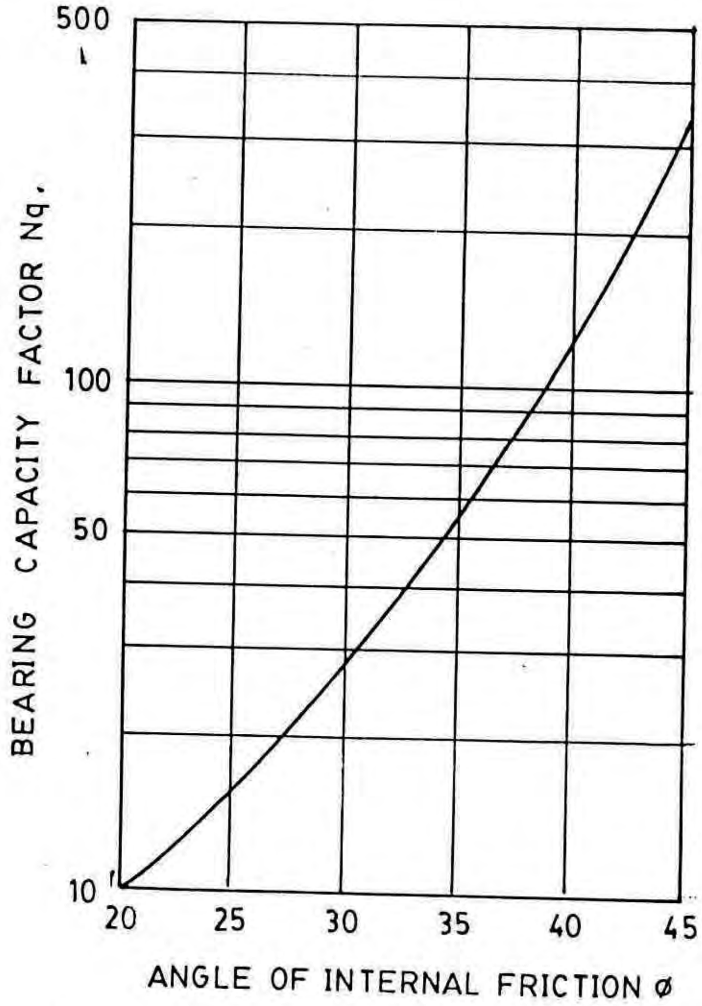


Fig 2.14 Bearing Capacity Factors Nq for Driven Piles (IS:2911 - 1979).

This method suggests that when pile is longer than 15 to 20 pile diameters, maximum effective overburden at tip should correspond to pile length equal to 15 to 20 diameter.

Method recommended by Canadian Geotechnical Society (1985)

To calculate the end bearing capacity of pile in sand Canadian Geotechnical Society (1985) proposed to use the following relation.

$$q_u = 6'_D N_t \qquad (2.11)$$

Where, q_u = ultimate unit end bearing of pile

$6'_D$ = unit effective vetical pressure at pile toe (below critical = D_c, effective pressure at critical depth is proposed to be used)

D = embedment length of pile in soil.

N_t = bearing capacity factor. Nt> Nq. usually, $N_t = 3N_q$. N_q is the bearing capacity factor for shallow foundation. Nq may be obtained from Fig. 2.15

API (1987) Method :

API (1987) recommended that the ultimate end bearing capacity of pile in cohesionless soil may be calculated by the following equation :

$$q_u = q\, N_q \qquad (2.12)$$

Where, q_u = ultimate end bearing capacity of the pile in cohesionless soil

q = effective overburden pressure at the pile tip

N_q = bearing capacity factor

The values of N_q recommended by API are shown in Table-2.2.

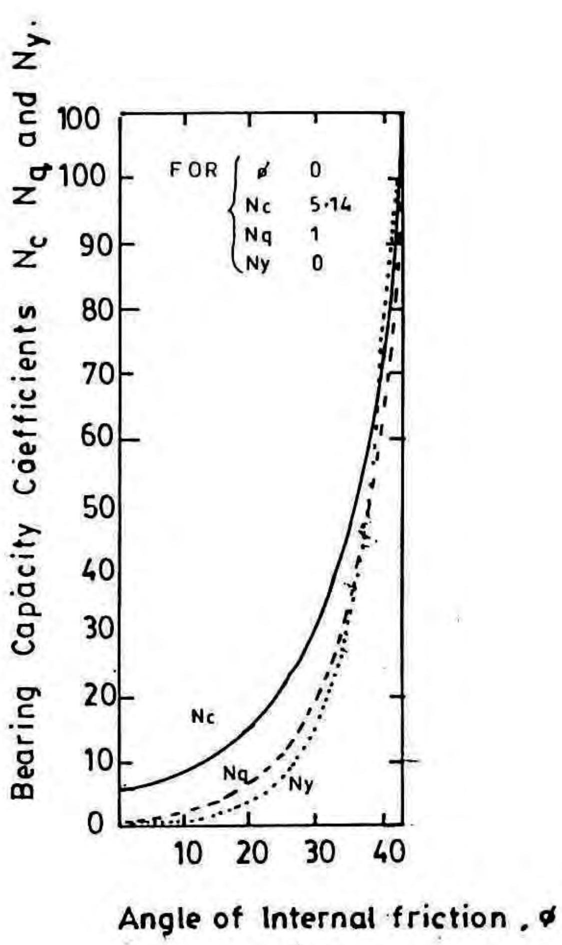


Figure 2.15 Bearing capacity coefficients for Shallow foundation [(Canadian Geotechnical Society (1985)]

Table 2.2 Design parameters for Cohesionless Siliceous Soil [API(1987)]

Density	Soil Description	Soil-Pile Friction Angle. δ Degrees	Limiting Skin Friction Values kips/ft² (kPa)	Nq	Limiting Unit End Bearing Values kips/ft² (MPa)
Very Loose Loose Medium	Sand Sand-Silt Silt	15	1.0 (47.8)	8	40 (1.9)
Loose Medium Dense	Sand Sand-Silt Silt	20	1.4 (67.0)	12	60 (2.9)
Medium Dense	Sand Sand-Silt	25	1.7 (81.3)	20	100 (4.8)
Dense Very Dense	Sand Sand-Silt	30	2.0 (95.7)	40	200 (9.6)
Dense Very Dense	Gravel Sand	35	2.4 (114.8)	50	250 (12.0)

Determination of Angle of Internal Friction

The end bearing of a pile resting on sand depends on the value of N_q .Again the value of N_q depends on the angle of internal friction, ϕ (which depends on the relative density of the soil). The value of N_q is very sensitive to the variation of ϕ (Fig .2.8).

The angle of internal friction, ϕ may be determined from triaxial test. Hansen (1970) recommended $\phi = 1.1\ \phi_t$ to be used in the bearing capacity equation. (2.7).

On the basis of further information Hansen (1979) recommended

$$\phi = \sin^{-1} \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$$

determined from triaxial test to be used in bearing capacity equation .

where σ_1 = major principal stress

σ_3 = minor principal stress

Das (1983) also pointed out that the value of ϕ determined from triaxial test is widely used for the design of structures.

Bowles (1996) pointed out that ϕ is pressure dependent and laboratory values of ϕ in the common range of triaxial cell pressures of 70 to 150 kPa may be several degrees larger than field values at the pile point, which may be 20m or 30 m down where there is substantially larger effective normal stress.

During installation of piles the soil surrounding the pile shaft and underlying the pile tip is disturbed.It is also difficult to obtain sand in the undisturbed state.For this reason, several approximate correlations have been developed for the determination of angle of internal friction. The correlations have been widely used for determining ϕ.

Meyerhof (1956)* based on the observations of several field explorations, provided relationship between angle of internal friction ϕ, standard penetration resistance, N and static cone penetration resistance (Table 2.3). Peck et al (1974) proposed a correlation of ϕ with the standard penetration resistance N (Fig.2.16). Whitaker (1976), Mohan (1988) pointed out about the correlation proposed by Peck et al (1974). The correlation proposed by Peck et al (1974) is widely used for the determination of ϕ.

End Bearing of Piles in Cohesive Soil

For pile in clay, the undrained load capacity is generally taken to be the critical. If clay is saturated the undrained angle of friction ϕ = zero, $N_q = 1$ & $N_\gamma = 0$ for $\phi = 0$

So ,Terzaghi's general equation reduces to

$$q_u = c\,N_c + \gamma\,D \qquad (2.13)$$

Considering weight of the pile = weight of soil displaced, the net end bearing of pile in cohesive soil is:

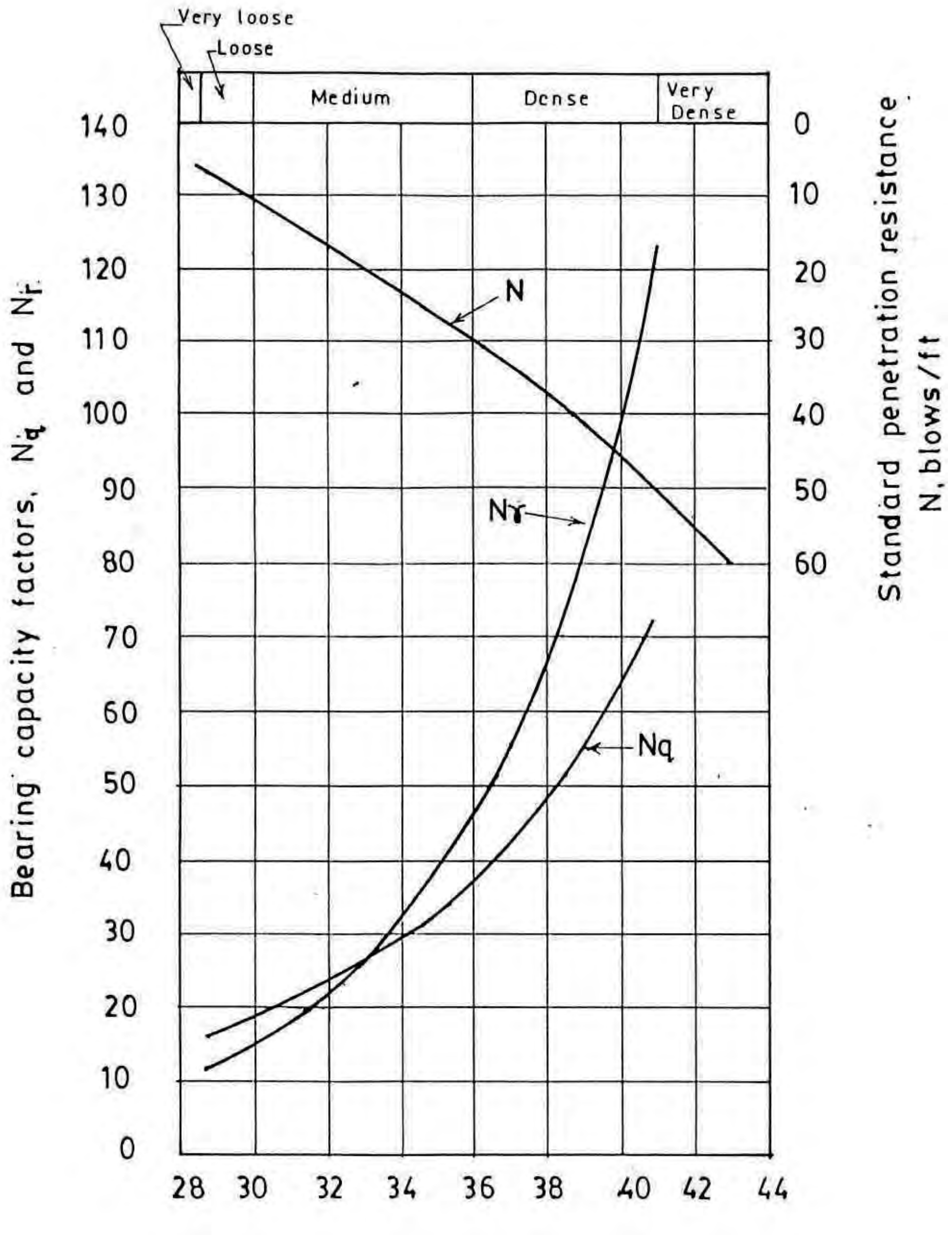
$$q_u = cN_c \qquad (2.14)$$

* Cited by Das (1985)

Table 2·3 Relationship between relative density, pentration resistance, and angle of friction of cohesionless soils [After Meyerhof (1956)]*

State of packing	Relative density	Standard penetration resistance N, blows/ft	Static cone resistance q_c ton / ft^2	Angle of friction ø deg
Very Loose	< 0·2	< 4	< 20	< 30
Loose	0·2 to 0·4	4 to 10	20 to 40	30 to 35
Compact	0·4 to 0·6	10 to 30	40 to 120	35 to 40
Dense	0·6 to 0·8	30 to 50	120 to 200	40 to 45
Very Dense	> 0·8	> 50	> 200	> 45

*Cited by Das (1985)



Angle of internal friction, Ø, degrees

Fig: 2.16 Correlation of the standard Penetration test N values with Nq, Ny and Ø. [Peck et al (1974)]

According to Meyerhof : N_c = 9.3 to 9.8 depending on whether the base is smooth or rough

According to Skempton : $N_c = 9$

According to Sewers

for model piles : $5 < Nc < 8$

The value of N_c proposed by Skempton (1951)** is shown in Fig=2.17.

Skin Friction of Piles

Skin friction is the resistance between the pile surface and the soil. The procedure for computing skin friction of piles in cohesionless soil and skin friction in cohesive soil are different.

Skin Friction of Pile In Cohesionless Soil

There are several methods for computing skin friction of pile in cohesionless soil. Important methods are discussed below :

Meyerhof's Method (1953)

Meyerhof (1953)* expressed the skin friction per unit area as

$$fs = ks\ q \tan\delta = Ks\ \gamma\ D \tan\delta \qquad (2.15)$$

Where, δ = angle of friction between soil & pile

K_s = coeff of earth pressure at rest

* Cited by Whitaker (1976)

** Cited by Poulos & Davis (1980)

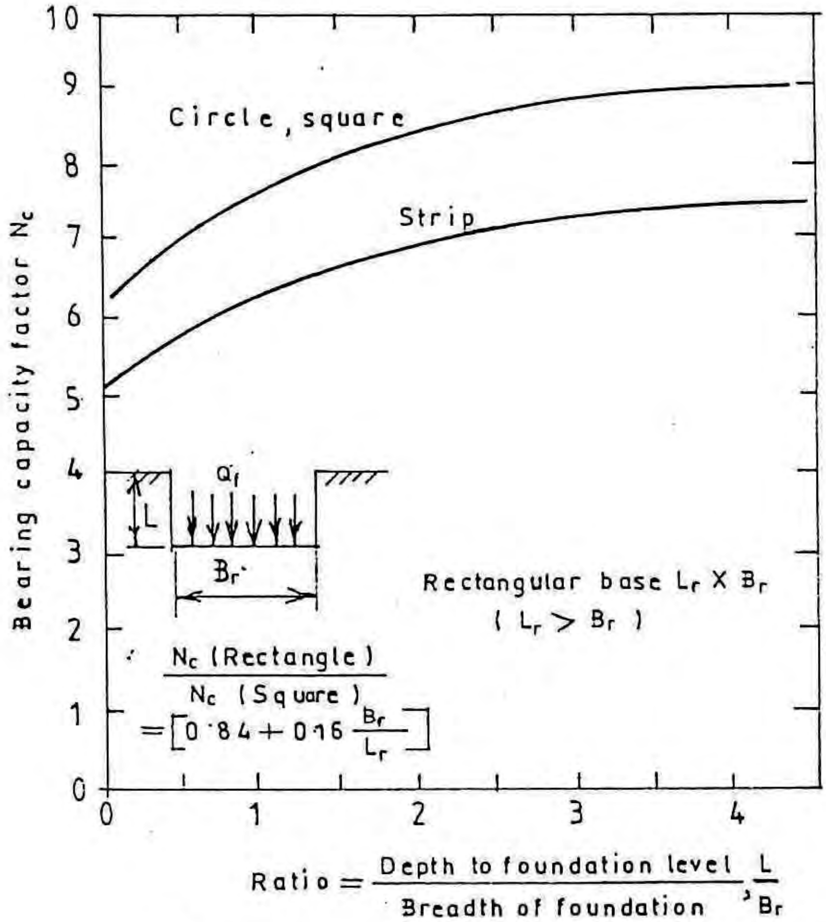


Figure 2.17 Bearing capacity factors for foundations in clay ($\phi = 0$) Skempton (1951) *

* Cited by Poulos & Davis (1980)

Method Recommended by Indian Standards Institution (1979)

According to IS : 2911-1979, the unit skin friction of pile in a granular soil is given by

$$q_u = K\,P_{Di} \tan\delta \qquad (2.16)$$

Where, K = co-efficient of earth pressure: for loose to medium sands, K values of 1 to 3 should be used.

P_{Di} = effective overburden pressure for the ith layer where i varies from 1 to n

n = layers of soil in which pile is installed.

δ = angle of friction between pile & soil,in degress (may be taken equal to ϕ)

Method Recommended by Canadian Geotechnical Society (1985)

According to Canadian Geotechnical Society (1985), skin friction of pile in sand is given by the following equation :

$$f_s = \sum_{Z=0}^{D} M\,K_s \tan\phi\ 6'_z \qquad (2.17)$$

Where $6'_z$ = effective vertical stress at depth z below critical depth

$Z = Dc$, use $6'_{Dc}$

K_s = ratio between horizontal effective soil stress to the vertical effective soil stress at pile shaft.

$M = \tan\delta / \tan\phi$: $\tan\phi$ = soil friction , $\tan\delta$ = soil pile friction.

M ranges from 0.7 to 1.0 depending on the material of pile & method of installation.

γ = density of soil

Tomlinson's method(1986) :

Tomlinson (1986) proposed the following relation for unit skin friction in cohesionless soil:

$$f_s = k_s \ q \tan\delta \qquad (2.18)$$

Where f_s = Unit skin friction

k_s = earth pressure co-efficient

δ = angle of wall friction

q = average effective overburden pressure over embedded depth of pile

Tomlinson recommended to use the values k_s and δ which have been proposed by Broms (1966)*. Broms related k_s & δ to the effective angle of shearing resistance of cohesionless soils for various pile materials and relative densities of soil (Table 2.4).

Borms used the effective angle instead of undrained angle.Tomlinson (1986) stated that for practical purposes ϕ can be used as obtained from standard penetration tests. It may be mentioned here that this method also indirectly depends on standard penetration test values.

API (1987) Method

According to API (1987), the skin friction over the shaft of the pile in cohesionless soil may be calculated by the following relation :

$$f_s = k \ q_o \tan\delta \qquad (2.19)$$

Where, k = earth pressure coefficient

* Cited by Tomlinson (1986)

Table 2.4 Values of K_s and δ [After Broms (1966)]*

Pile material	δ	Value of K_s	
		Low relative density	High relative density
Steel	20°	0.5	1.0
Concrete	$\frac{3}{4}\phi$	1.0	2.0
Wood	$\frac{2}{3}\phi$	1.5	4.0

* Cited by Tomlinson (1986)

q_o= effective overburden pressure at the point in question

δ =angle of wall friction between the soil and the pile wall (to be taken from Table 2.2)

Skin Friction of Pile in Cohesive Soil

There are several methods of computing skin friction in cohesive soil. Important methods are :

(i) α method

(ii) λ method

(iii) β method

(iv) α_2 method

(iv) Method recommended by Indian Standards Institution

(v) API method

(vi) Method recommended by Canadian Geotechnical Society

α method [Tomlinson (1971)]

To calculate the skin friction of pile in cohesive soil, Tomlinson (1971)* proposed this method. According to this method skin friction is computed as :

$$f_s = \alpha c \qquad (2.20)$$

Where, α = coefficient to be obtained from design curve of Fig 2.18

c = average cohesion (or su) for the soil stratum of interest

* Cited by Bowles (1988)

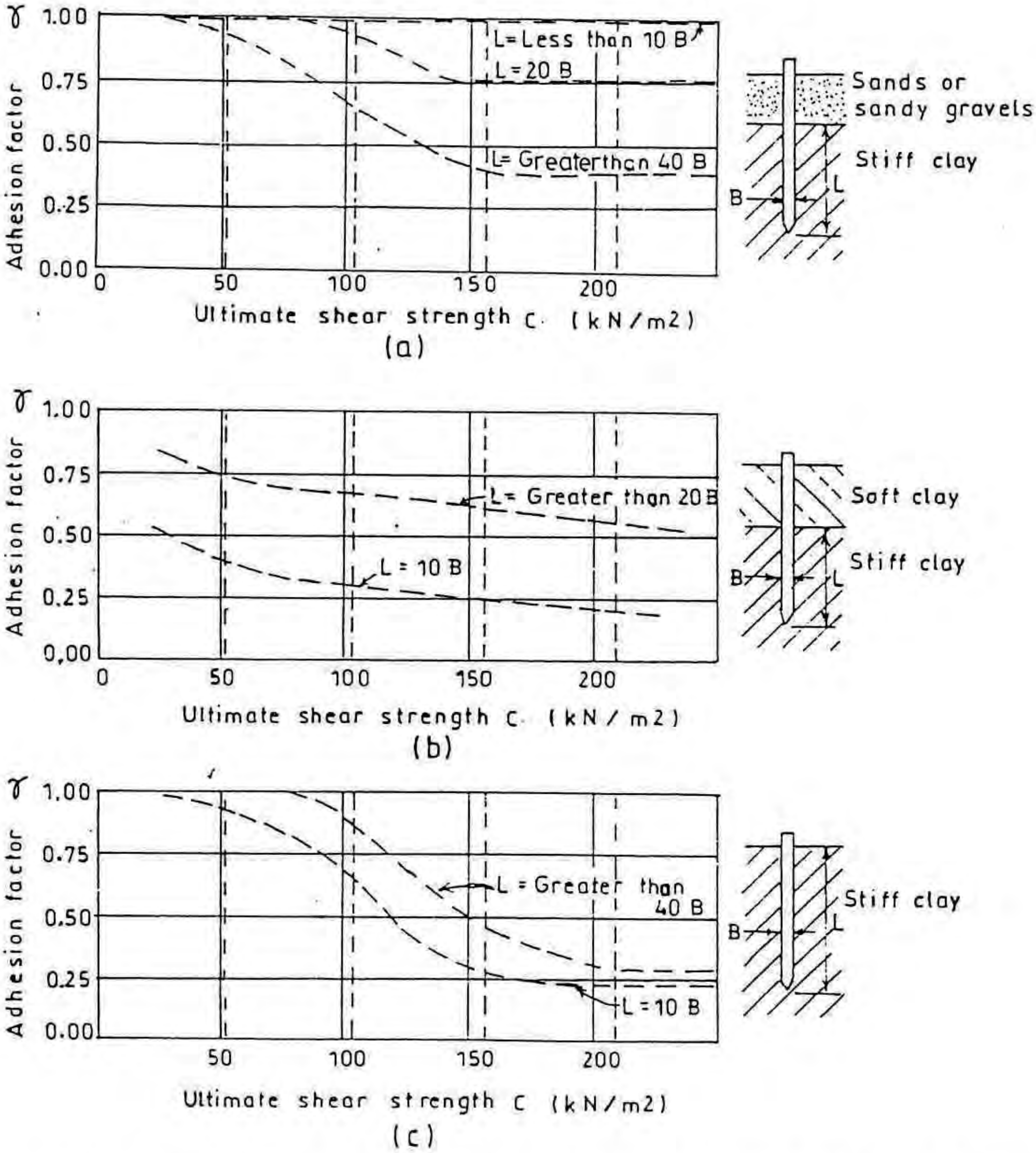


Figure 2.18 Adhesion factors for driven piles in clay. (a) Case 1: piles driven through overlying sands or sandy gravels. (b) Case 2: piles driven through overlying weak clay. (c) Case 3: piles without different overlying strata. [Tomlinson (1971)]*

Cited by Tomlinson (1995)

λ Method [Vijavergiya and Focht (1972)]

Vijayvergiya and Focht (1972)* presented a method of obtaining skin resistance of a pile in clay. This method is λ method. The relation was expressed in the following way ;

$$f_s = \lambda\,(q + 2\,c_u) \qquad (2.21)$$

Where f_s = unit skin friction

q = effective overburden pressure

c_u = cohesion for the soil stratum of interest

λ = coefficient which can be obtained from Fg.2.19.

For shorter piles, λ values are larger mostly because the shorter piles are in stiffer clay or clay with stiff upper crust (OCR>1). Where long piles penetrate into into soft clay, the values reflect both averaging for a single value and development of a limited skin resistance since q does not increase pile capacity without bound.

β Method [Burland (1973)]

Burland (1973)* developed this method of obtaining skin friction from effective stress on the shaft of pile. Following assumptions were made in the derivation.:

(i) Before loading the excess pore pressures set up during installation are completely dissipated.

(ii) Loading takes place under drained condition because the zone of major distortion around the shaft is relatively thin.

* Cited by Bowles (1988)

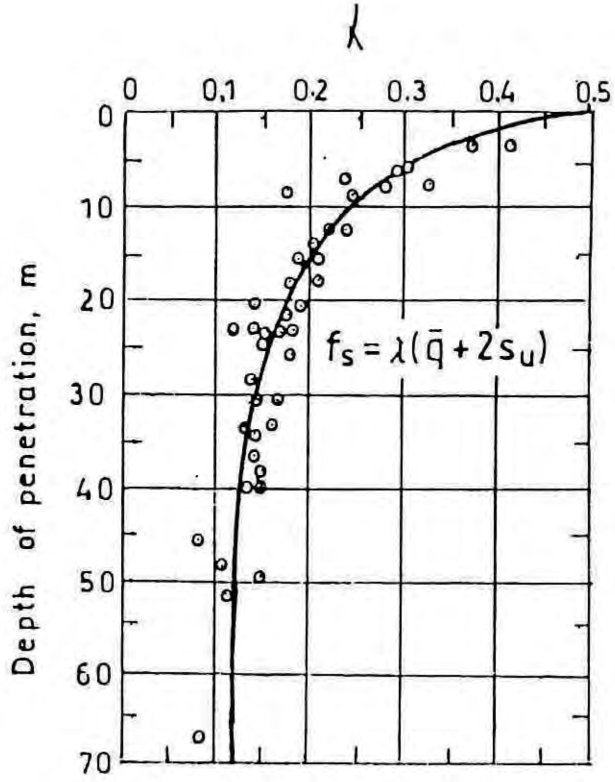


Figure 2.19 λ Coefficients depending on pile penetration. Data replotted and depths converted to meters by Bowles (1996) from Vijayvergia and Focht (1972)*

Cited by Bowles (1996)

(iii) As a result of remoulding during installation the soil has no effective cohesion.

In this method, then skin friction fs is given by the following equation :

$$f_s = 6'_h \tan\delta \qquad (2.22)$$

Where $6'_h$ = horizontal effective stress acting on the pile

δ = effective angle of friction between the clay and the pile shaft

Further simplifying assumption is made that the $6'_h$ is proportional to the vertical effective overburden pressure 6_v so that $6'_h = k\, 6_v$

Therefore, $f_s = K6_v \tan\delta$

If $K \tan\delta = \beta$, the equation reduces to

$$f_s = \beta\, 6_v \qquad (2.23)$$

Attractive feathure of the β method is that if we use, k_o for k and $\delta = \phi$ the product $k_o \tan\delta = \beta$ ranges from about 0.,25 to 0.40 for normal range of ϕ = 20° to 30° (Fig. 2.20). The upper values apply to short piles (length less than 15m) and lower values apply to long piles.

α_2 *Method[Peck et al (1974)]*

Peck, Hanson and Thornburn (1974) suggested the following relation for calculating the skin friction of pile in cohesive soil :

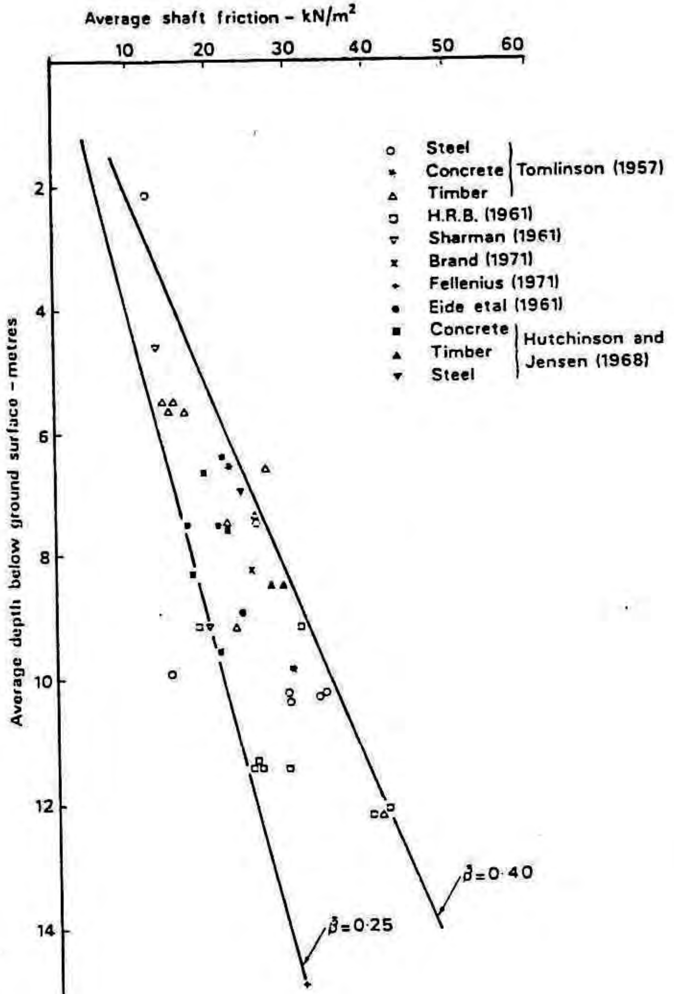


Figure 2.20 Average shaft friction versus average depth for driven piles in soft clays, with lines representing the values $\bar{\beta}$ = 0·25 and 0·40 *

Cited by Whitaker (1976)

$$f_s = \alpha_2 \, c_u \qquad (2.24)$$

Where, f_s = unit skin friction

c_u = undrained cohesion of soil

α_2 = reduction coefficient which may be obtained from Fig.2.21.

Unlike others, the curves of Fig. 2.21 shows a wide range of values of α_2 .Peck et al suggest that average values may be used for practical purpose.

Method Recommended by Indian Standards Institution (IS: 2911-1979)

IS : 2911-1979 recommends to calculate the skin friction of pile in cohesive soil using the following relation :

$$f_s = \alpha \, c \qquad (2.25)$$

Where c = average cohesion throughout the length of the pile

α = reduction factor

The suggested values of α are shown in Table 2 .5

API (1984) Method :

API (1984) also suggest to use α method with factors as shown in Fig. 2.22 for normally consolidated clay. However, API recommended that the value of f_s should not be more than 50 kPa for OCR > 1 or large L/B ratio

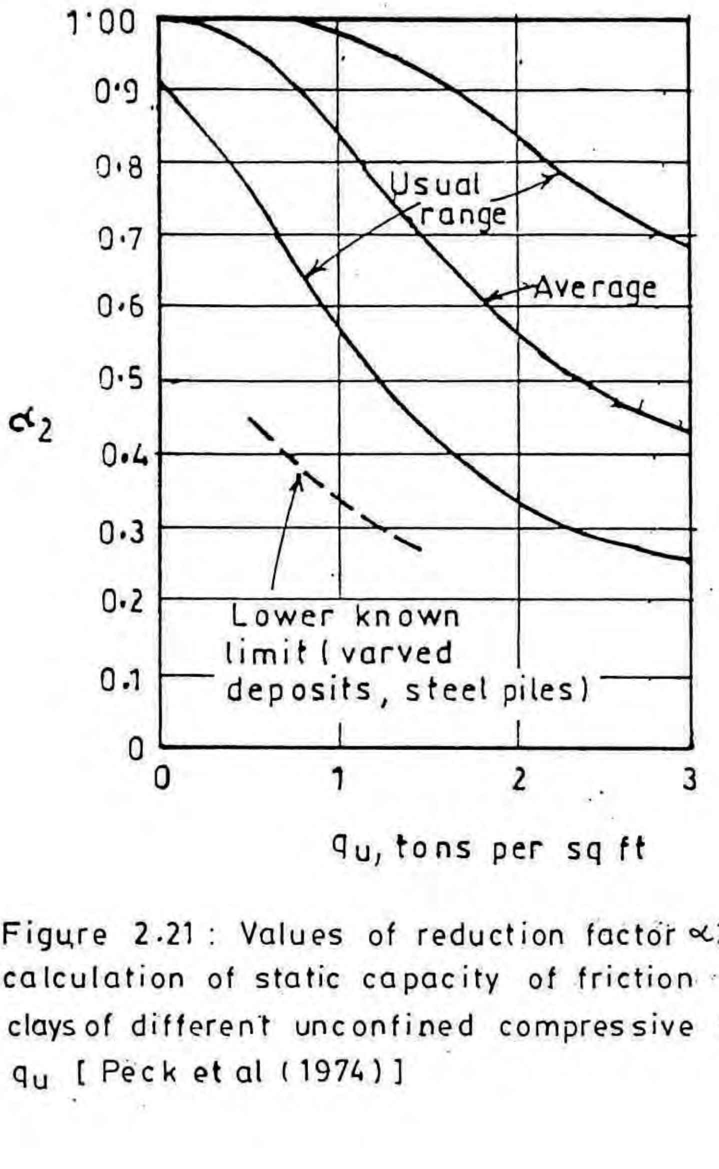


Figure 2.21 : Values of reduction factor $\alpha 2$ for calculation of static capacity of friction pile in clays of different unconfined compressive strengths q_u [Peck et al (1974)]

Table 2.5 Values of α recomnended by Indian Standards Institution (IS : 2911-1979)

Consistency of soil	N Value	Value of α	
		Bored pile	Driven pile
Very soft to soft	<4	0·7	1
Medium	4 to 8	0·5	0·7
Stiff	8 to 15	0·4	0·4
Stiff to hard	>15	0·3	0·3

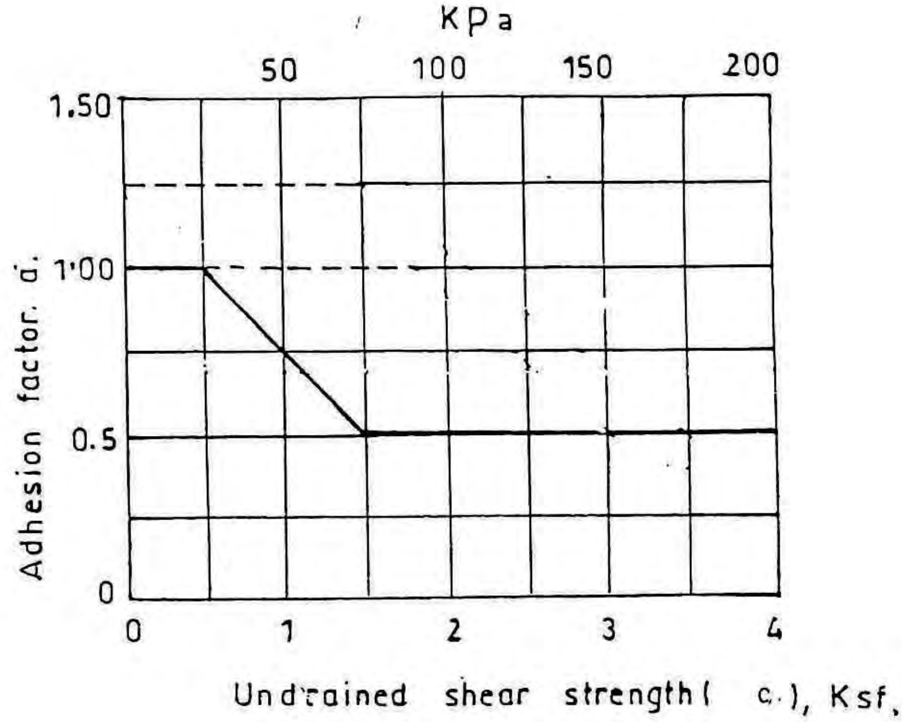


Figure 2.22 Relationship between soil and adhesion factor [A PI (1984)] *
*Cited by Bowles (1988)

Method suggested byCanadian Geotechnical Society (1985)

Canadian Geotechnical Society (1985) proposed to calculate the skin friction over the shaft of the pile using the following relation:

$$f_s = \alpha \tau_u \tag{2.26}$$

Where f_s = unit skin friction

α = reduction factor

τ_u = undrained shear strength of the soil

However, in this method skin friction is shown as a function of undrained shear strength of soil (Fig.2 .23)

Therefore,corresponding skin friction may be obtained from Fig.2.23

2.3.2.2 Determination of Pile Capacity by Empirical Methods

Different investigators have established correlation bearing relation between static field penetration tests such as (i) Standard Penetration Test and (ii) Cone Penetration Tests with end bearing of pile and shaft resistance.

Standard Penetration Test

Meyerhof (1956,1976)* has correlated the unit skin friction and unit end bearing of a pile with the result of standard penetration test. For displacement piles in saturated sand the unit end bearing is given by:

$$q_u = 40N_p \frac{L_b}{B} \leq 400N_p \text{ kPa} \tag{2.27}$$

and unit skin friction is given by:

$$f_s = X_n N \text{ kPa} \tag{2.28}$$

* Cited by Bowles (1988)

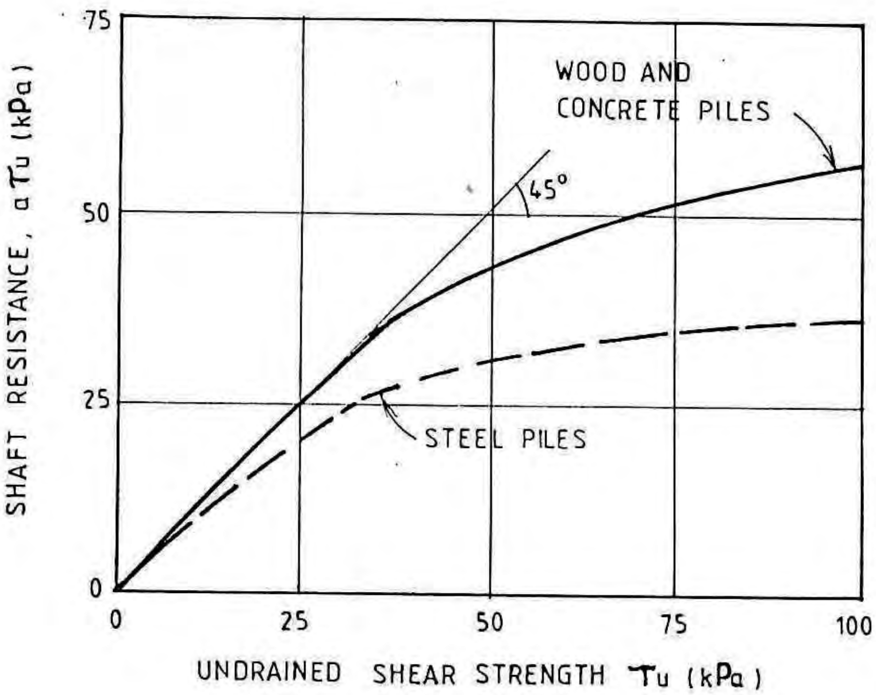


Figure 2.23 Shaft resistance in clay as function of undrained shear strength [Canadian Geotechnical Society (1985)] .

Where, N_p= statistical average of SPT in a zone of about 8B above to 3B below pile tip.

$\bar{N}$= average value of N along pile shaft

q_u = ultimate unit end bearing

f_s = ultimate unit skin friction

X_n= 2 for piles of large displacement and = 1 for small displacement

B = width or diameter of pile point

L_b = pile penetration depth into point bearing stratum

L_b /B = average depth ratio of point into point bearing stratum

Static Cone Penetration

The basis of the test is the measurement of the resistance to penetration of a 60 ° cone with a base area of 10 square cm. .Two types of cone are commonly used : the standard point, with which only point resistance can be measured ; and the friction-jacket point, which allows both point resistance and local skin resistance to be measured (Begemann, 1953 and 1965)*.

Van der Veen (1957)* suggested that the ultimate unit end bearing of a pile may be taken as that of the cone resistance. Hence, according to Van der Veen

$$q_u = q_c \qquad (2.29)$$

where, q_u = ultimate unit end bearing of pile

q_c = cone point resistance for zone of about 8B above and 3B below pile point

According to Begemann (1965) * the adhesion Measured by friction jacket may be safely taken as unit skin friction for driven piles in clay.

* Cited by Poulos and Davis (1980)

Hence, $f_s = f_c$ (2.30)

Where f_s = ultimate unit skin friction

f_c = adhesion measured from friction jacket.

Full scale tests carried by Vesic (1967)[*] showed that the unit end bearing is comparable with cone resistance of penetrometer but the unit skin friction is double that measured by penetrometer . Hence, according to Vesic (1967)[*]

$$q_u = q_c \tag{2.31}$$

$$f_s = 2f_c \tag{2.32}$$

According to Meyerhof (1956)[**] for cases where separate measurements of friction jacket resistance are not made, the ultimate skin friction may be measured by the following relation :

$$f_s = 0.005q_c \tag{2.33}$$

Where separate side friction is measured Meyerhof (1956)[**] suggested the following relation:

$f_s = f_c$ (small volume displacement pile) (2.34)

$f_s =$ 1.5 to $2f_c$ (large volume displacement pile) (2.35)

* Cited by Poulos & Davis (1980)

** Cited by Bowles (1988)

2.3.2.3 Determination of Pile Capacity by Dynamic Methods

It is natural for any one driving a stake into ground to assume that the effort needed depends on the resistance of the ground. For nearly two centuries engineers have applied this idea to pile driving and many mathematical formulae or dynamic formulae have been devised for calculating the resistance.

All dynamic pile formulae are based on the principle that resistance of piles to penetration under the working load has a direct relationship to their resistance to the impact of the hammer when they are being driven. Dynamic formulae considers the weight and height of drop of the hammer, the weight of pile, the penetration of the pile under each blow. Refined formulae also take into account losses of energy due to elastic compression of the pile, the helmet, the packing, ground surrounding the pile and losses due to inertia of the pile.

There are a large number of dynamic formulae. Some of the important methods are discussed below :

(i) Engineering News Record Formula:

This formula was published by Wellington in Engineering News in 1888 and hence it is usually called Engineering News Record formula(ENR).

The formula is expressed as follows :

For drop hammers, $$Q = \frac{e_h W_r h}{s + 25} \qquad (2.36)$$

For Steam Hammers: $$Q = \frac{e_h W_r h}{s + 2.54} \qquad (2.37)$$

Where, Q = ultimate pile capacity

h = height of fall of ram

W_r = weight of ram

s = amount of point penetration per blow

In this formula, the following assumptions were made:

a) Hammer and pile may be treated as impinging particles

b) Hammer gives up its entire energy on impact.

c) On impact the resistance increases in an elastic manner as the pile is displaced ,remains constant for further displacement and then falls to zero in an elastic manner as the pile rebounds.

(ii) Hiley Formula

This formula was proposed by Hiley in 1925 and may be expressed in the following form:

$$Q = \frac{e_h W_r h}{s + \frac{1}{2}(k_1 + k_2 + k_3)} \frac{W_r + n^2 W_p}{W_r + W_p} \qquad (2.38)$$

However, for double acting or differential Hammers ,Chellis (1961) suggested the following form of the Hiley equation:

$$Q = \frac{e_h E_h}{s + \frac{1}{2}(k_1 + k_2 + k_3)} \frac{W + n^2 W_p}{W + W_p} \qquad (2.39)$$

According to Chellis, the manufacturers energy rating of E_h is based on an equivalent weight term W and height of ram h as follows:

$E_h = Wh = (W_r + \text{Weight of casing})h$

where, Q = ultimate pile capacity

E_h = manufacturers' hammer energy rating

e_h = hammer efficiency

k_1 = elastic compression of cap block

k_2 = elastic compression of pile

k_3 = elastic compression of soil

W_p = weight of pile

s = amount of point penetration per blow

W_r = Weight of ram

n = co-efficient of restitution

It is assumed that there are losses of energy

a) in the hammer system

b) due to impact

c) due to elastic compression of the pile

d) due to elastic compression of the head assembly comprising the dolly, helmet and packing

e) due to elastic compression of the ground.

A factor of safety =3 to 6 is proposed

ii(a) ***Eytelwein Formula [Chellis (1961)]***

The Eytelwein formula as mentioned by Chellis in 1961 may be expressed as:

$$Q = \frac{e_h E_h}{s + C(W_p / W_r)} \qquad (2.40)$$

Where, Q = ultimate pile capacity

E_h = manufacturers' hammer energy rating

e_h = hammer efficiency

W_p = weight of pile

W_r = Weight of ram

s = amount of point penetration per blow

C=2.54 mm=0.1 in

It is based on the following assumptions:

(a) The hammer and pile may be treated as impinging particles having a co-efficient of restitution and that Newton's law of impact apply.

(b) An energy equation applies to the driving of a pile where energy supplied by hammer is not usefully absorbed in advancing pile.

(c) The only energy lost is due to impact

A factor of safety 6 is proposed for this formula.

(iii) Danish Formula [Olson & Flaate (1967*)]*

The Danish formula is expressed as :

$$Q = \frac{e_h E_h}{s + C_1} \qquad (2.41)$$

$$C_1 = \sqrt{\frac{e_h E_h L}{2AE}}$$

Where, Q = ultimate pile capacity

* Cited by Bowles (1988)

E_h = manufacturers' hammer energy rating

e_h = hammer efficiency

s = amount of point penetration per blow

W_p = weight of pile

W_r = weight of ram

A =pile cross sectional area

E = modulus of Elasticity

L = pile length

In the derivation of this formula also some assumptions were made which are :

(a) There is frictional loss in the hammer system.

(b) The elastic compression of the pile is that which would occur if all the available energy were used in causing the compression.

(c) There is loss due to impact.

Safety factor of 3 to 6 is proposed for this formula.

(iv) Janbu Formula [Olson & Flaate (1967) Mansur & Hunter (1970)]*

The Janbu formula as mentioned by Olson and Flaate (1967)* & Mansur & Hunter (1970)* and may be expressed in the following form:

$$Q = \frac{e_h E_h}{k_u s} \qquad \lambda = \frac{e_h E_h L}{AE_s^2}$$

$$C_d = 0.75 + 0.15\frac{W_p}{W_r}$$

$$k_u = C_d\left(1+\sqrt{1+\frac{\lambda}{C_d}}\right) \qquad (2.42)$$

Where Q = ultimate capacity of pile

E_h = manufacturers' energy rating

e_h = hammer efficiency

A = pile cross- sectional area

E =modulus of Elasticity

L = Pile length

s = penetration per blow

W_p = weight of pile

W_r = weight of ram

It also based on some assumptions such as:

(a) There is frictional or other loss in the hammer system so that energy actually applied at impact is less than energy delivered.

(b) There is loss due to elastic compression of the pile.

(c) There is loss due to impact.

Recommended factor of safety is 3 to 6.

(v) Gates Formula[Gates (1957)]*

This formula was proposed by Gates (1957)*

$$Q = a\sqrt{e_h E_h}(b - \log s) \tag{2.43}$$

Where, Q = ultimate of capacity of pile

E_h = manufacturers' hammer energy rating

e_h = hammer efficiency

In FPS , Q=kips, E_h=kips.fts, s =in,a = 27,b = 1

In Sl units, Q = kN, E_h = kNm, s = mm, a = 104.5 and b = 2.4

e_h = 0.75 for drop hammers and o.85 for all other hammers.

A factor of safety = 3 is proposed in this case.

(vi) Canadian National Building Code Formula*

According to this formula:

$$Q = \frac{e_h E_h C_1}{s + C_2 C_3} \tag{2.44}$$

$$C_1 = \frac{W_r + n^2(0.5W_p)}{W_r + W_p}$$

* Cited by Bowles (1988)

$$C_2 = \frac{3Q}{2A}$$

$$C_3 = \frac{L}{E} + C_4$$

C_4 =0.0001 in.^{3}/k (Fps)

=3.7 $\times 10^{-10}$ m^3/kN (SI)

Where, Q= ultimate capacity of pile

E_h= manufacturers' hammer energy rating

e_h=hammer efficiency

W_p= weight of pile

W_r= weight of ram

n = co-efficient of restitution.

L=length of pile

E= modulus of elasticity

C_d=0.0001 in/k (FPS)

=3.7$\times 10^{-10}$ m^3/kN (SI)

Factor of safety = 3 has been proposed in this case.

(vii) Pacific Coast Uniform Building Code (PCUBC)* formula

The equation is expressed in the following form :

$$Q = \frac{e_h E_h C_1}{s + C_2} \tag{2.45}$$

$$C_1 = \frac{W_r + kW_p}{W_r + W_p}$$

* Cited by Bowles (1988)

$$C_2 = \frac{QL}{AE}$$

Where Q=ultimate capacity of pile

E_h=manufacturer's energy rating

e_h = hammer efficiency

s = pile penetration per blow

W_p= weight of pile

W_r= weight of ram

A = cross sectional area of pile

L= length of pile

E= modulus of elasticity

k = 0.25 for steel piles

= 0.10 for all other piles

The solution of the equation is a trial and error approach. Factor of safety = 4 is proposed for this formula.

(viii) Navy- Mckay formula*

Navy- McKay formula is expressed in the following form :

$$Q = \frac{e_h E_h}{s(1 + 0.3C_1)} \tag{2.46}$$

$$C_1 = \frac{W_p}{W_r}$$

* Cited by Bowles (1988)

Where, Q = ultimate pile capacity

E_h= manufacturers' hammer energy rating

e_h = hammer efficiency

W_p = weight of pile

W_r = weight of ram

s = amount of point penetration per blow

A factor of safety = 6 is proposed in this case.

(ix) AASHTO (1990) Formula*

This formula was primarily developed for timber piles. The formula is expressed in the following way :

$$Q = \frac{2h(W_r + A_r p)}{s + C} \qquad (2.47)$$

C=2.5 mm=0.1 in

For double acting steam hammer it is proposed to use A_r = ram cross-sectional area, p = steam or air pressure; for single acting and gravity A_r p = 0.

It is proposed to take e_h = 1.0. suggested factor of safety is 6.

* Cited by Bowles (1988)

*(x) Modified ENR [(ENR(1965)] formula**

The Modified Engineering News Record formula is expressed as

$$Q = \frac{1.25 e_h E_h}{s + C} \frac{W_r + n^2 W_p}{W_r + W_p} \tag{2.48}$$

Where Q = ultimate pile capacity

E_h = manufacturers' hammer energy rating

e_h = hammer efficiency

W_p = Weight of pile

W_r = weight of ram

s = amount of point penetration per blow

n = co-efficient of restitution

C=2.5 mm=0.1 in

Applicability of the driving Formulae

It is assumed that the driving formula for a drop hammer may be applied to a single acting steam or compressed air hammers. Experience shows that in the case of double acting hammers, which deliver blows in rapid succession penetration per blow depends on the number of blows per minute. Therefore, manufacturer's rated energy per blow at the speed of operation should be used when taking the set for using in driving formula. In the case of diesel hammers manufacturer's rated energy may be used when taking set for using in energy equation.

* Cited by Bowles (1988)

Limitations of the driving Formulae

Although dynamic formulae have been used extensively to predict pile capacity none of them have been found consistently reliable or reliable over an extended range of pile capacity. There are several reasons . Each formula has been developed on the basis of one or more assumptions. Firstly, the Newtonian laws in a simple manner is not applicable to driving formula. Also it is over simplification in assuming that the dynamic resistance can be appropriately expressed by a single force in a simple energy formula and that the calculation of an energy correction for the elastic compression of the pile can be made as if the loading were static. Above all ,there is no basis in the assumption that the dynamic energy is equal to the static load bearing capacity.

The acceptability of any formula can be best examined by comparing the ultimate carrying capacities delivered by it with those obtained by pile load tests. Terzaghi (1942)* performed this task with data from 39 timber, concrete and steel piles using 7 different formulae. He observed that the ratio of the real load to computed load covered the range 0.25 to 4.0, that the range varied for different formulae applied to the same data and that the same formula was not necessarily good for timber, concrete and steel piles. Others made similar collections of data and have assessed the relative merits of different formulae by statistical analysis. Mention may be made of Sorensen, Hansen (1957)*, Agerschou (1962)*. Their study showed that Eytelwein & Engineering News Record formulae have poor reliability in comparison to others.

The most dangerous misinterpretation of the driving formulae is that they do not consider the soil conditions which affect the long term carrying capacity and settlement of piles and effect of remoulding and reconsolidation negative skin friction and group action.

* Cited by Whitaker (1976)

Though it is found that most of the driving formulae pay no attention to the nature of soil yet it is well known that no formula can be used with uniformly satisfactory results. However, better agreement with test loads is obtained in the case of end bearing piles in sand or gravel than for friction piles in clay.

Engineers with experience of pile driving in a particular area may make modifications to formulae to obtain greater reliability.

A driving formula is of practical importance to use as a control in places where ground conditions are substantially uniform to ensure that the piles of one kind are driven to approximately the same resistance within the limits of length decided from the site investigation results. A simple record of blow count per foot of penetration is generally valuable for indicating contact with a resistant bed.

In view of the above facts Whitaker (1976) proposed some general principles to be observed while using driving formulae:

(a) If possible use of a driving formula should be avoided except for end bearing piles in sand or gravel.

(b) A formula giving small scatter of the real ultimate capacity to computed ultimate capacity should be used for a particular type of pile.

(c) The simplest formulae that meets (b) above should be used. There is no merits in complications specially if it does not produce reliable results.

* Cited by Whitaker (1976)

2.3.2.4 Determination of Pile Capacity by Wave Equation

Wave equation method applies wave transmission theory to determine the carrying capacity developed by a pile and the maximum stresses that result within the pile during driving.

The method assumes that the pile and its behavior when embedded in soil can be represented by a series of individual spring connected weights and spring damping resistances (Fig.2.24). The various weight values W correspond to the weight of incremental sections of pile. The spring constant K relate to the elasticity of the pile. The spring damping R represents the frictional resistance of soil surrounding the shaft of the pile and soil resistance at the tip of pile.The spring damping along the shaft of the pile accounts for a gradual diminishing of the longitudinal force (from the hammer blow) which travels along the length of the pile. Spring damping at the pile tip is necessary to account for the force which remains within the pile to be transmitted at the tip.

To solve the wave equation, it is necessary to know approximate pile length, the weight of pile, cross-section of pile, elastic properties of pile, pile hammer characteristics, including efficiency, ram, weight, impact velocity to have data on the pile cap and capblock, and to assign values for soil damping and the spring constants. Determination of the effect of a stress wave travelling through the pile is a dynamics problem. However, if the effect of a pile hammer blow at one particular instant of time is selected (the reaction of each weight and spring to the forces acting on overlying weights is determined), the analysis can be handled as a statics problem.

By analyzing changing conditions for successive small increments of time the effects of the force wave travelling through the pile to the tip will be simulated. This analysis requires a numerical integration, a task conveniently undertaken

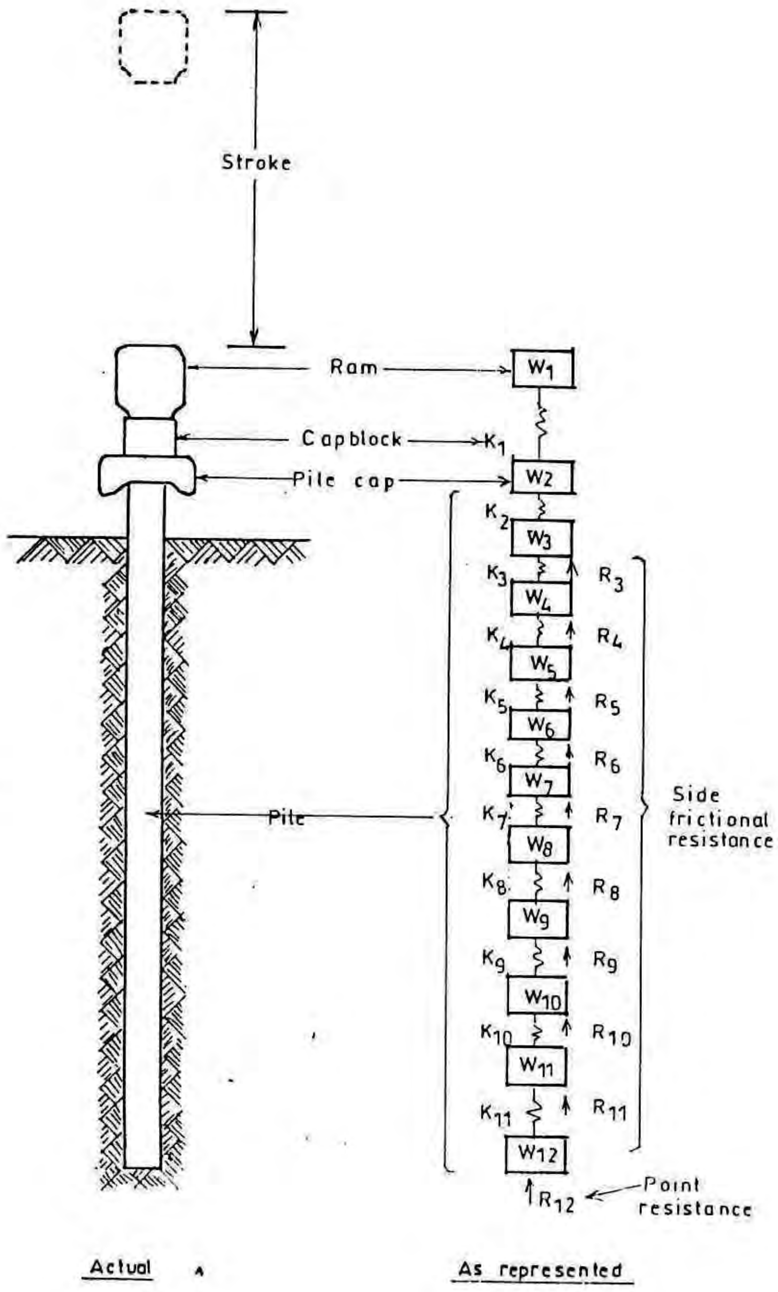


Figure 2.24 Method of representing pile for wave equation analysis.*

*Cited by Mc Carthy (1977)

by computer. The results obtained would be only for a particular pile driven by a specified pile hammer. Separate analysis are required for different conditions.

2.3.2.5 Determination of Pile Capacity by Pile Load Test

Pile load test is the most reliable method of determining carrying capacity of pile . The pile load test consists of driving the pile to the required design depth and applying a series of loads by some means. In the field two types of tests are generally performed. One is for the determination of ultimate load by applying load upto failure of the pile and other is for checking the design load i.e. to load upto 1.5 to 2 times the design load. Again, on the basis of method of loading, pile load tests are termed as :

(a) Maintained load test

(b) Constant Rate of Penetration Test and

(c) Equilibrium method of Test

Pile Capacity Determined By Maintained Load Test

Maintained load test is by far the most usual one in practice. The general procedure is to apply static loads in increments of 25% of the anticipated working load. Each load is maintained until settlement ceases or diminished to an acceptable rate or until a certain time period has elapsed. According to ASTM Standards (D-1143) each load increment is maintained until the rate of settlement is not greater than 0.25 mm in one hour but not greater than two hours. After the completion of the loading if the test pile has not failed the total test load is removed any time after 12 hours if the butt settlement over one hour period is not greater than 0.25mm otherwise the total load is allowed to remain on the pile for 24 hours. After the required holding time the test load is removed in decrement of 25% of the total test load with 1 hour between decrement. If failure occurs, jacking the pile is continued until settlement equals 15% of the pile diameter or diagonal dimension. (Details may be obtained in ASTM Standards).

Pile Capacity Determined by Constant Rate of Penetration Test (CRP)

The Constant Rate of Penetration test was developed by Whitaker (1953)*. He proposed that the pile could be treated as a probe. In the CRP test continuous loading is given to the pile so that the penetration of the pile remains at a constant rate. The rate of penetration selected usually corresponds to that of shearing soil samples in unconfined compression test. However rate does not affect the results significantly . Whitaker also states that very near to ultimate load very little increase in load is required to maintain a constant rate of penetration and the ultimate carrying capacity is reached when continuous vertical movement result in no increase in the penetration resistance. A penetration rate of 0.75 mm/minute is suitable for friction piles in clay and penetration rate of 1.5 mm/minute is suitable for end bearing piles in sand or gravel.

The CRP test has the advantage that it can be performed rapidly and hence is suitable for preliminary test piling when failure load is unknown and when design is based on a factor of safely against ultimate failure & it is desirable to know the real factor of safety.

However, this method has the disadvantage that it does not give elastic settlement under the working load which is of significance in determining

* Cited By Whitaker (1976)

whether or not there has been plastic yield of the soil at the working load. It requires heavy kentledge loads or high capacity anchors where large diameter piles are loaded to failure which is also a drawback. Therefore, according to Tomlinson (1995) it is suitable for research investigations where fundamental pile behavior is being studied.

Pile Capacity Determined By Equilibrium Method [Mohan et al (1967)]

This is another procedure of compression test called method of Equilibrium in which the main principle is to apply to the pile at each stages of the test a load slightly higher than the required load and then to decrease the load to the desired value. In this way the rate of settlement diminishes much more rapidly than with the maintained load and equilibrium is reached much earlier. This method was described by Mohan et al (1967)*.

The procedure as suggested by Mohan,Jain and Jain is first to apply about one tenth of the estimated ultimate load by hydraulic jack in a period of three to five minutes. It is maintained for about five minutes and then allowed to reduce itself due to downward movement of the pile. Within a few minutes a state of equilibrium is usually established. The next higher load is then applied and the process is repeated till the final load is reached. For higher loads,it is desirable to maintain the initial load for a period of 10-15 minutes before it is allowed to diminish. It is claimed that since the load at which equilibrium is always lower than the maximum in a particular loading cycle, it provides a better indication of load settlement than obtained in a maintained load test. The total time of test required by this method is generally reduced to about one-third of that required in normal maintained load test.

* Cited by Mohan (1988)

Determination of Ultimate Capacity from the Load test Results

The ultimate capacity can be determined only if the load test is carried to actual failure (a rapid, disproportionately large increase in settlement corresponding to a fixed increment of load). Determination of precise failure load is a matter of judgment. The relationship between settlement and load, generally is one of gradually increasing steepness with no well defined break to establish failure condition. In some pile load tests, the plot of load vs. settlement shows a sharp break so that a clearly defined failure load is indicated. Moreoften, the slope changes so gradually that the failure load is not clearly defined.

There are two common definitions of ultimate capacity:

(I) The load that causes a settlement equal to 10% of pile diameter (Terzaghi 1942)*

(ii) The load at which the rate of settlement continues undiminished without further increment of load unless this rate is so slow as to indicate that the settlement may be result of consolidation of soil.

Methods Recommended By Various Codes To Determine Ultimate Capacity From Pile Load Test:

(i) IS : 2911 (Part-IV)-1979

According to IS : 2911 (Part-IV)-1979 ultimate capacity is smaller of the following two:

(a) Load corresponding to a settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in the case of under-reamed or large diameter bored pile.

* Cited by Whitaker (1976)

(b) Load corresponding to a settlement of 12mm.

(ii) BS 8004 :1986

BS 8004 :1986 recommends that the ultimate capacity should be that which causes the pile to settle a depth of 10% of pile width or diameter.

iii) AASHO*

AASHO recommends that the ultimate capacity should be that which causes a net settlement (gross settlement less rebound) of ¼ in.

(iv) International Conference of Building Officials*

International Conference of Building Officials recommends that the ultimate capacity is the maximum load at which total settlement, including elastic deformation of pile, is not over 0.01 in per ton of test load and at which no settlement has occurred for 24 hours.

(v) New York City Building Code and Uniform Building Code*

New York City Building Code and Uniform Building Code suggest that the ultimate capacity is the maximum load which causes a net settlement (gross settlement minus rebound) not exceeding 0.01 in per ton of test load.

. (vi) Uniform Building code*

Uniform Building code recommends that the ultimate load is determined by the point on the load settlement curve at which an increase in load produces a disproportionate increase in Settlement.

* Cited by Jonson and Kavanagh (1968)

Besides the codes, the following definitions of ultimate capacity provided by individual authorities may be mentioned :

(i) W.H.Rabe, Bureau of Bridges, state of Ohio*

W.H.Rabe, Bureau of Bridges, state of Ohio recommends that the ultimate capacity is that at which the gross settlement exceeds 0.03 in per ton of additional load

(ii) Point at which slope of gross-settlement curve is four times the slope of the elastic deformation of the pile [Johnson and Kavanagh (1968)].

.(iii) D.R.L. Nordlung of Raymond Concrete Pile Company*

D.R.L. Nordlung of Raymond Concrete Pile Company suggests that the ultimate capacity is that load at which the gross settlement exceeds 0.05 in per ton of additional load or at which the plastic settlement (as determined from cyclical loading) exceeds 0.75mm per ton of additional load.

(iv) Point on load-settlement curve where penetration no longer is proportional to load [Johnson and Kavanagh (1968)].

(v) Davisson's Method (1973)**

Davission (1973) ** developed a method of determining ultimate capacity of pile for cases where the load settlement curves show no well defined breaks. The determination of ultimate capacity, in such cases, is a matter of interpretation. According to this method the elastic deflection of the pile is computed by means of the expression PL/AE and plotted on the load settlement curve as line 00'; for the

* Cited by Jonson and Kavanagh (1968)

** Cited by Peck et al (1974)

best interpretation the scales of the curve should be chosen so that the slope of 00'; is about 20° .The dashed line CC' is drawn parallel to 00' with an intercept on the settlement axis equal to (0.15 + 0.1d) in. where d is the diameter of the pile in feet. The intercept is a measure of the tip settlement required to develop the capacity. The ultimate capacity is defined as the load at which the line CC' intersects the load settlement curve (Fig. 2.25)

Allowable Capacity From Load Test as Suggested by Different Code of Practice:

To determine allowable load from pile load test results,different codes have suggested different methods. Important methods are:

(i) <u>Bangladesh National Building Code (1993):</u>

Bangladesh National Building Code recommends that the allowable capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm.

ii) <u>Indian Standard Code of Practice (I S 2911-1979)</u>

According to Indian Standard Code of Practice IS.:2911-1979, allowable pile capacity is smaller of the following:

(a) Two thirds of the final load at which the total settlement attains a value of 12 mm.

(b) Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of under -reamed pile.

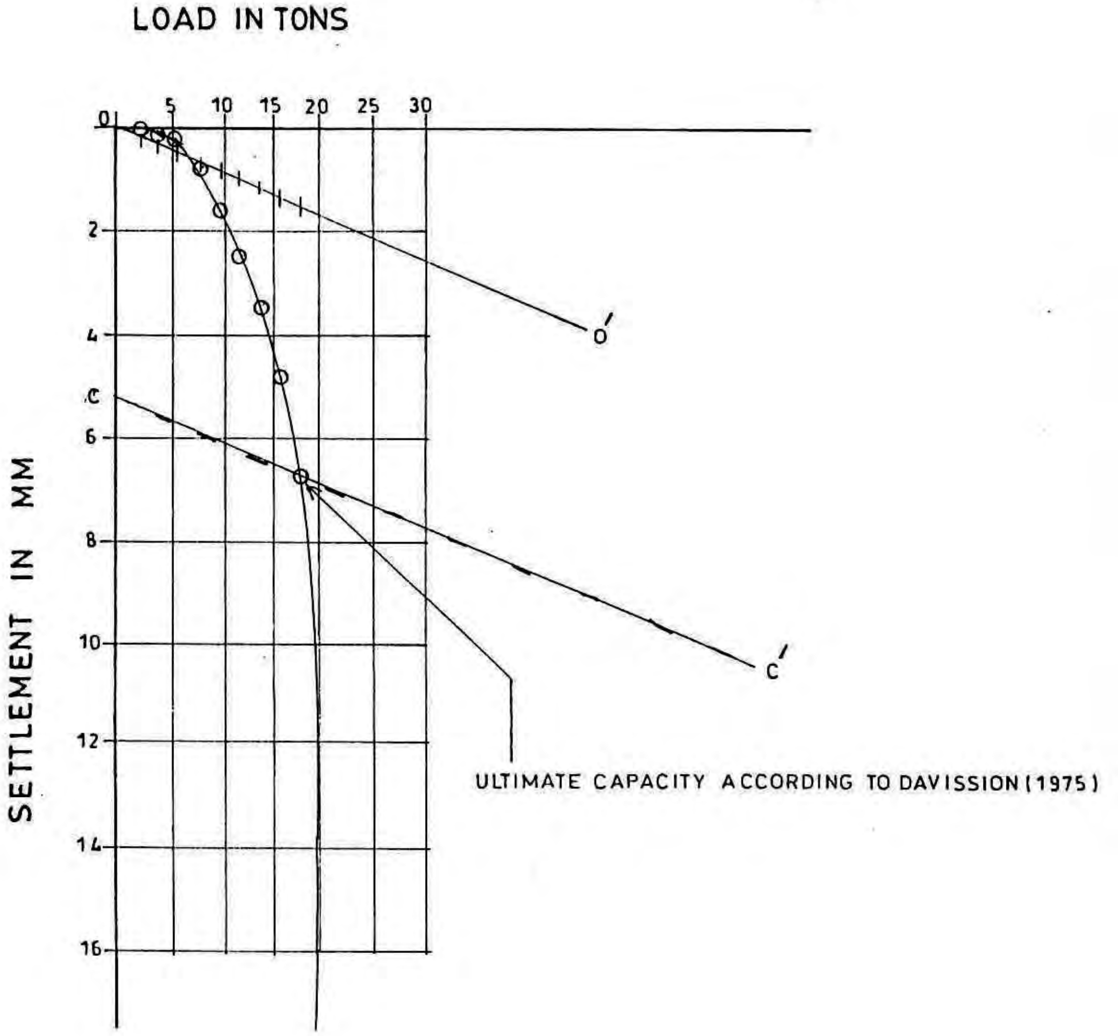


FIGURE 2.25 DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSION (1975)

(iii) British Standard Code of Practice (BSI CP-2004-1972)

According to BSI CP-2004-1972, the allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter.

There are a number of methods to predict pile capacity from static and dynamic methods.The angle of internal friction is difficult to determine.Again, different authorities proposed different value of bearing capacity factors. Also there are a number of methods to determine ultimate capacity from pile load test.

Therefore as so in the case of geotechnical engineering ,judgement still remains a decisive ingredient.

CHAPTER 3

EXPERIMENTAL WORK

3.1 GENERAL

The main objectives of this study are: to investigate the methods of construction of small size prestressed pile , to install and carry out load test at four different locations within Dhaka Metropolitan city and to compare the pile capacity obtained from load test with that from other methods. In order to attain the objectives detail experimental program was worked out. This experimental programme consists of the following works:

i) Site selection for pile load test

ii) Subsoil investigation

iii) Construction of prestressed piles

iv) Installation of the piles

v) Pile load tests

3.2 SITE SELECTION

Four sites within the Dhaka metropolitan city were selected for this purpose. All the four sites were selected on the basis of one or both of the following criteria :

a) Top soil upto the depth of 3m to 4m is not suitable for shallow foundation.

b) The underlying soil is either medium stiff to stiff cohesive soil or medium dense cohesionless soil.

3.3 SUBSOIL INVESTIGATION

To calculate pile capacity by static methods soil properties surrounding the pile is required. Therefore, subsoil investigation was performed at four selected sites. Wash boring system was used in each case. The size of borehole was 100mm. In this method the drilling bit used for cutting soil is connected to a drill rod [50mm outer diameter and 35mm inner diameter] through which drilling is pumped. The pumped drilling mud carries fragments of the soil, cut by chopping bit, to the surface. The stratification of the subsoil is recorded with the advancement of the boreholes. Details of the investigation are given below :

Bonosree Project Site

In this site 2 borehole, BH1 & BH2 were drilled. From the location of BH1 and BH2 disturbed samples were collected and SPT were executed at a depth interval of 1.0 upto 3 m depth and at a depth of 1.5 m interval for the remaining depth of drilling in each case. Samples were preserved in watertight polythene bags after visual observation. Undisturbed samples were collected from the cohesive soil layers from both the boreholes. Laboratory tests such as unconfined compression test,liquid limit and plastic limit,natural moisture content, dry & wet density and grain size analysis tests were performed. The lithology and test results have been shown in Fig. A-1 and A-2

Novodoy Housing Site

In this site also 2 boreholes, BH3 & BH4, each upto the depth of 15.0 m were drilled. Standard penetration tests were carried out at the interval of 1.0 m upto the depth of 3.0 m. Thereafter, standard penetration tests were carried out at the interval of 1.5 m upto the final depth of drilling. Disturbed samples were collected along with SPT (at the same depth of SPT). Fills exists upto 1.50m depth followed by soft to medium stiff clayey silt upto 7.0m below existing ground level. The underlying layer consists of nonplastic silts and sandy silts. Undisturbed samples were collected using 75mm diameter thin walled shelby tubes from the cohesive soil layers.These samples were preserved for laboratory tests after visual identification.Unconfined compression test,liquid limit and plastic limit,natural moisture content, dry & wet density, grain size analysis tests were performed in the laboratory. The results are shown in Fig. A-3 and A-4

Ahmed Bagh Site

In this site two boreholes,BH5 & BH6, each upto 15.0 m below existing ground level were drilled.Standard penetration test was conducted at 1.0 m depth interval upto 3.0m depth. Below 3.0m depth SPT was was conducted at 1.5m interval .Disturbed samples were collected during standard penetration test.Undisturbed samples were collected from the cohesive soil layers only from both the boreholes. Laboratory tests such as unconfined compression test, unit weight, natural moisture content,liquid limit and plastic limit test,grain size analysis were performed. The results have been provided in Fig. A-5 and A-6

Moghbazar Site

In this site two boreholes,BH7 & BH8, each upto the depth of 16.0 m below existing ground level were drilled. Standard penetration tests were carried out at the interval of 1.0 m upto the depth of 5.0 m each case. Thereafter, standard penetration tests were carried out at the interval of 1.5 m upto the final depth of drilling Disturbed samples were collected along with SPT. Undisturbed samples were collected from the cohesive soil layers only. Laboratory tests such as unconfined compression tests, natural moisture content, liquid limit and plastic limit,unit weight & grain size analysis tests were performed in the geotechnical laboratory. For sandy soil only sieve analysis were carried out. The results have been given in Fig. A-7 and A-8

3.4 CONSTRUCTION OF PILES

For the purpose of pile load test, eight small size prestressed piles were required. These piles were constructed in Housing and Building Research Institute. The cross section of the pile is 175 mm x 175 mm square.. A shoe made of 18 gauge sheet is cast with concrete as shown in Fig.3.1(A). The concreting of the prestressed piles were performed in the casting yard of Housing and Building research Institute. In the construction concrete of proportion 1:1.5:3 with water cement ratio 0.46 was

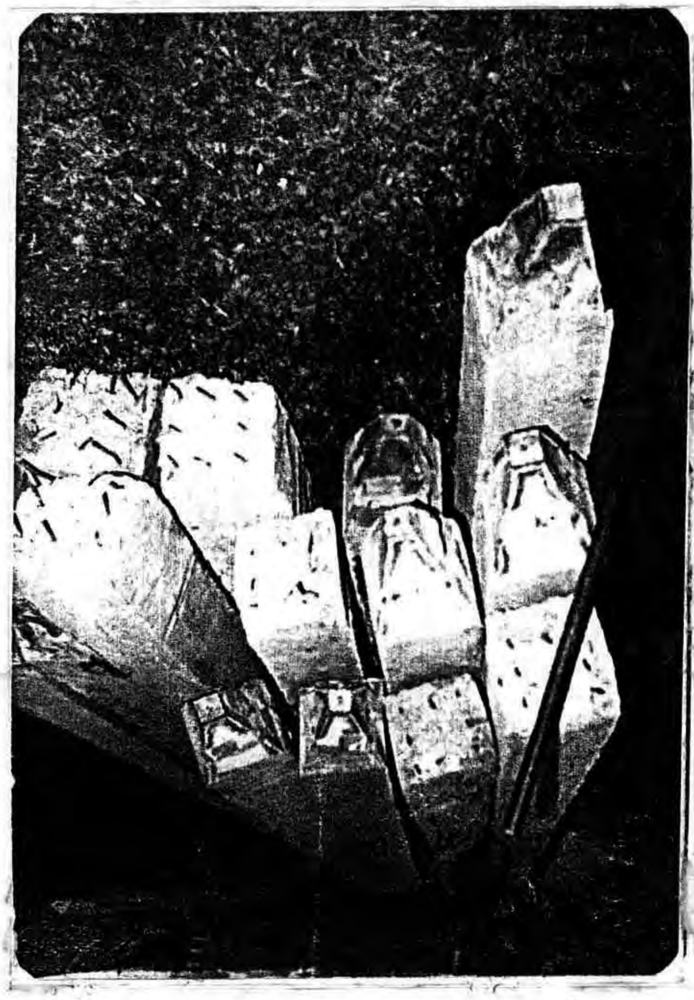


Figure 3.1 (A) Photographs of Small Size Prestressed Pile

used. Stone chips (12 mm down graded) was used as coarse aggregate while Sylhet sand having F.M. = 2.5 was used as fine aggregate. Aggregates were in saturated surface dry condition. Ordinary Portland cement was used.

Admixture named Febflow standard was used in the concrete for early strength gain. During mixing of cubes, slump test was performed and it was kept within 50 mm to 60 mm. 2 sets of 3 cubes were cast for compressive strength test. The results of 28 days compressive strength are presented in the Table 3.1.

Reinforcement

8 numbers 5 mm diameter prestressing bar were used as longitudinal main reinforcement.Ultimate strength of prestressing cable is 1,20,000 psi.The cable were prestressed to 1,08,000 psi.

For a distance of 1875mm from butt 4 numbers 10 mm diameter deformed bar was provided to take care of driving impact. 1/4 in. ϕ bar @ 125 mm c/c was used as tie for 25% of pile length from both ends. For the remaining portion of pile, 2 mm ϕ plain bar @ 125 mm c/c was provided as tie as shown in Fig.3.1(B).

Curing was done for 28 days as of ordinary concrete elements. Hessian was used to cover the piles during curing.When concrete reached 3 days age, prestressing cables were cut and the piles were shifted to stacking yard for the remaining curing period.

Table 3.1 Results of 28 Days' Compressive Strength of cubes for casting piles.

SL.No.	Cube. No.	28 Days' compressive strength. PSI	Averege compressive strength. PSI
1	C1	5360	5550
2	C2	5390	
3	C3	5890	
1	C4	5560	5230
2	C5	5100	
3	C6	5030	

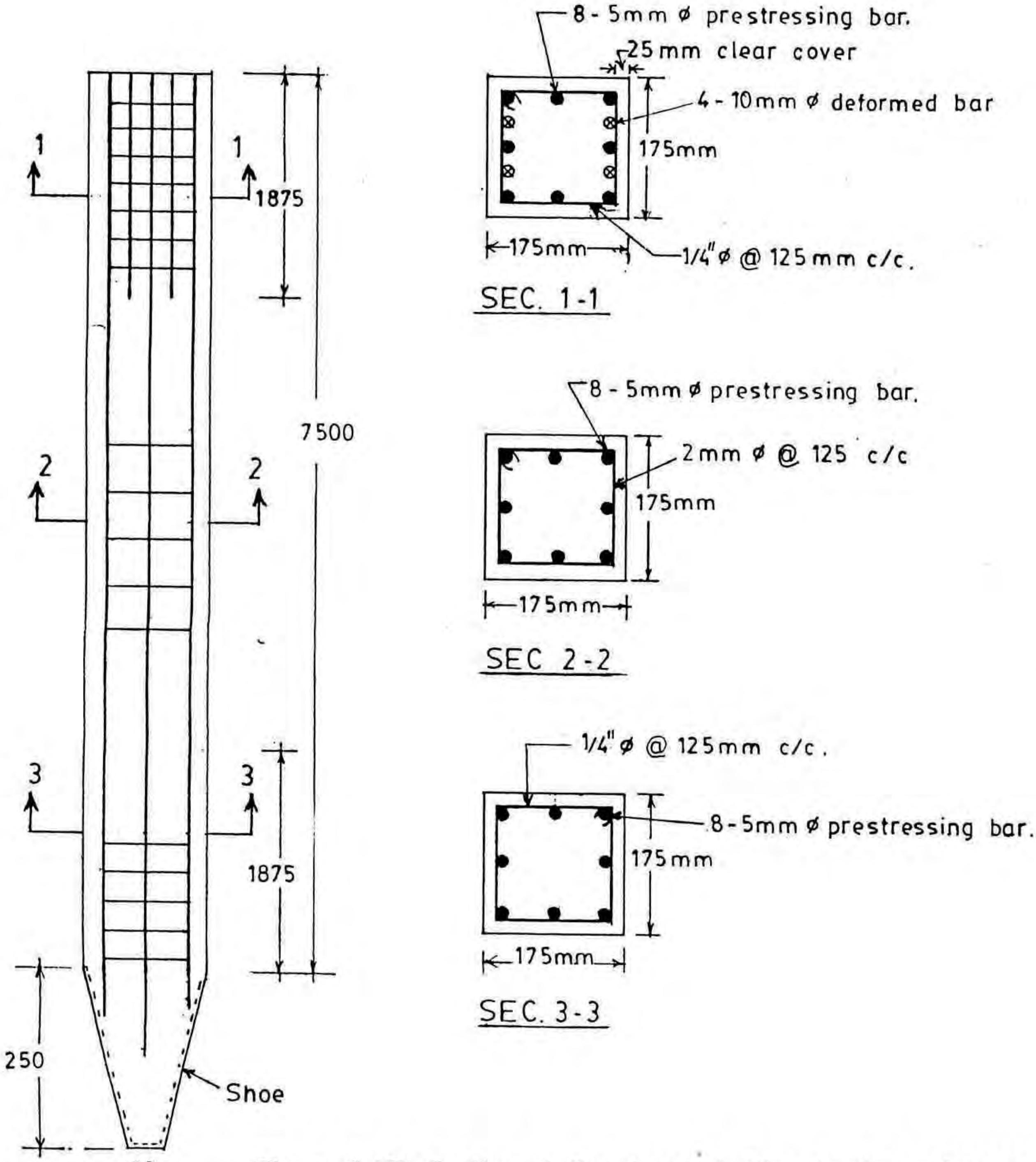


Figure 3.1(B) Section of Prestressed pile showing reinforcement

3.5 INSTALLATION OF PILES

Two small size piles were installed at each site. The piles were driven with the help of drop hammers. Hammer weighing 500 kg was used for piles at Novody Housing site. 336 kg weight hammer was used for piles in Bonosree & Ahmedbagh site.However, the weight of hammer was 272 kg in the case of Moghbazar site. The frame for driving pile was made of timber and bamboo as in Fig-3.2. Initially, height of drop and corresponding blows for 0.3m penetration were very small. In both cases a helmet made of 16 G sheet was used over the top of the pile. In between the helmet and pile, packing materials such as jute bags, coconut matting, hessian packing etc, were used. Helmet for this type of particular pile are specially shaped and are fitted with a recess of a hardwood and with steel wedges to keep the helmet tight on the pile. If the helmet is allowed to work loosely it would damage the pile head.

3.5.1 Handling of Small Size Prestressed Pile

The design of precast pile is governed by bending stress due to transportation & lifting. For shorter piles, one point lifting is used. For long piles 2 or 3 point lifting may be used to reduce bending stress. However, two or three point lifting are difficult to perform. One point lifting was used for small size prestressed piles. The lifting point was 2.50 m from end (Fig-3.3). From stock yard to site, the piles were transported with the help of push cart. They were lifted to cart manually and also

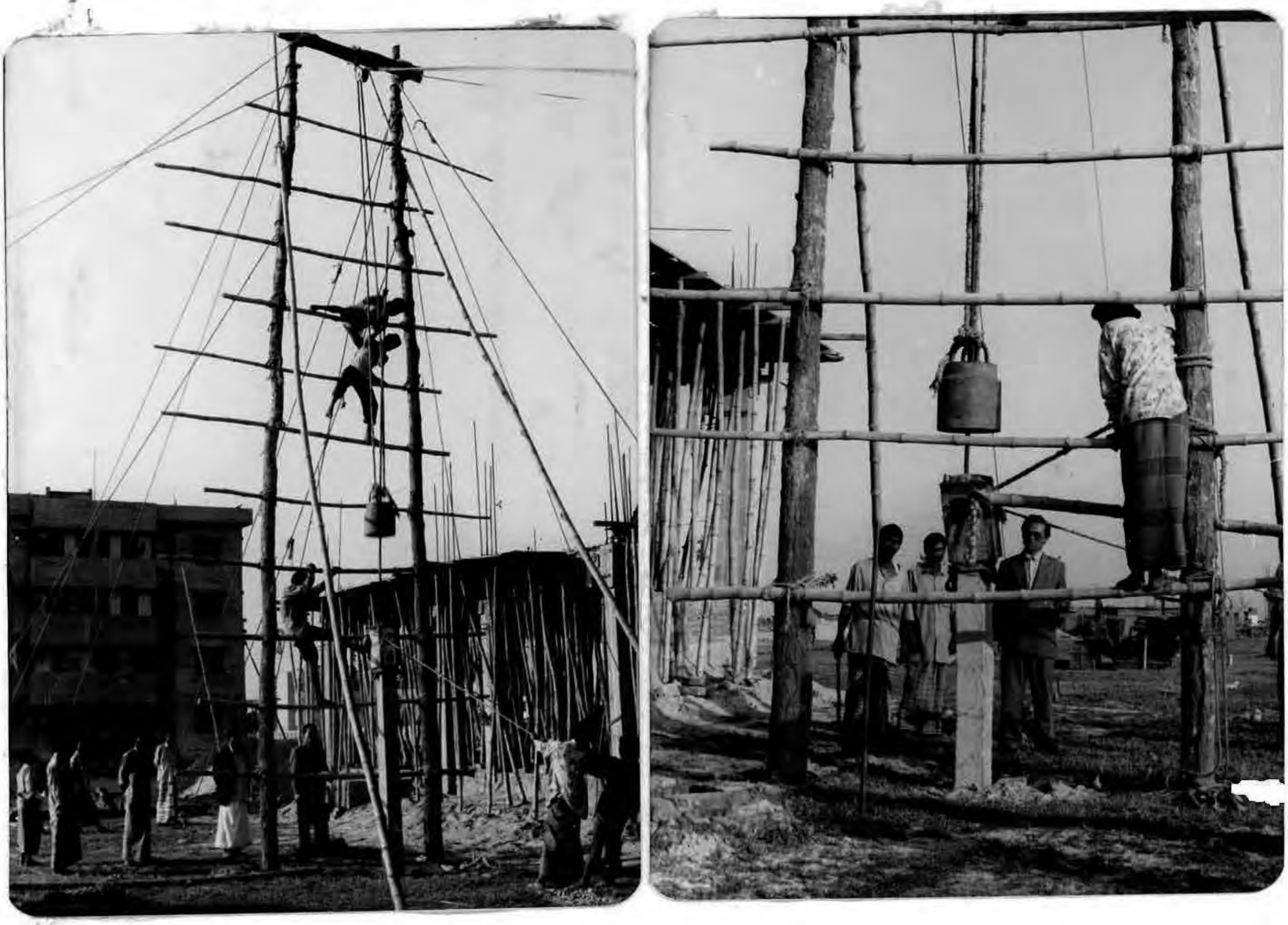


Figure 3.2 Photographs of Pile Driving Activity and Pile Driving Frame

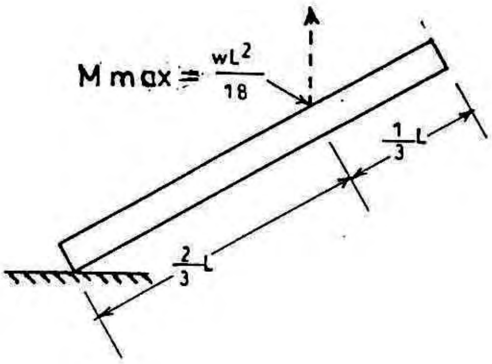


Figure 3.3 Location of lifting point and resulting bending moment.

unloaded manually. The piles were handled carefully to avoid dropping or severe jarring while in a horizontal position.

3.5 PILE LOAD TEST

Maintained load test was carried out according to ASTM D 1143. Since, the failure occurred, jacking the pile was continued until settlement equaled 15% of the diagonal dimension of the pile.As per ASTM Standard in the maintained load test (ML), the load is applied in increments of 25% of anticipated working load. Each load increment was maintained until the rate of settlement is not greater than 0.25mm /hour but not longer than 2 hours. The pile load tests were started nearly after 1 month of pile driving.in each case.

Testing Arrangements

For loading kentledge reaction system [Fig.3.4(A))] was used.The head of the pile was chipped, finished and levelled perfectly to transfer uniform load from the hydraulic jack to the pile. Two numbers of `U' channel(reference beam) was installed on two sides of the pile parallel to the pile and was anchored by bolt at 2.5 m away from the center of the pile which is under load test inside the pit. The pile is then clamped with M.S. clamps with arms inverted and a free space of 0.20 m is kept between the inverted clamp arms and the rigid concrete base to accommodate the dial gauges and the glass sheets. These glass sheets were provided for adjustment of upward or downward movement of the pile under test load if the strain accommodation of the dial gauge pins exceeds their limit.

The lead plate was placed on the leveled and finished pile head for uniform distribution of load on the pile. Then the hydraulic jack was placed on the lead plate and the rocker beam was placed on the ram of the hydraulic jack to transfer load of the platform to the hydraulic jack. The loading platform is made on the pit of the test pile with the help of the M.S. joists and a 6 cm clearance was kept between the rocker beam and the bottom of the M.S. cross joists so that the platform load can be transferred to or released from the pile head by upward or downward movement of the pile ram by pumping or releasing the hydraulic jack.

Two number of dial gauges with 50mm travel were fitted between the rigid concrete base and the clamp arm attachment so that any movement of the pile, whether downward or upward can be read at once in the dial gauge with an accuracy upto 0.01 mm. Then the platform is loaded with gunny bags filled with sand and stacked uniformly on the platform. The gross weight of the "KENTLEDGE" was about 1.7 times higher than the expected ultimate load. The arrangement is now ready for starting the test. For detail of the test procedure reference is made to ASTM D-1143.

***Loading and Data* Recording:**

The hydraulic pump was pumped with the handle attached and the jack with the rocker beam moves upward and touches the bottom of the cross joists to transfer the load to the pile [Figures 3.4(A) to 3.4(D)]. The amount of load transferred can be calculated from the reading of the dial of the pressure gauge. The jack and the dial gauges were calibrated in the material testing laboratory of BUET. After each load increment settlement was measured with the help of dial gauges G1 and G2, the average of which was be taken as mean settlement of the pile for that corresponding load imposed load imposed.

Figure 3.4 (A) Photographs of Pile Load Test at Novodoy Housing Site

Figure 3.4 (B) Photographs of Pile Load Test at Ahmedbagh Site

Figure 3.4 (C) Photographs of Pile Load Test at Moghbazar Site

Figure 3.4 (D) Photographs of Pile Load Test at Moghbazar Site

CHAPTER 4

EXPERIMENTAL RESULTS

4.1 GENERAL

The main objectives of this study are: to investigate the methods of construction of small size prestressed pile, to drive and carry out load test at four different locations within Dhaka metropolitan city and to compare the measured pile capacity obtained from load test with the predicted values by other methods

To fulfill the objectives, subsoil investigation was made at four sites within Dhaka city.Two piles at each site were driven and pile load test was performed.

The experimental results of this study is arranged in the following way:

(i) Results of subsoil investigation

(ii) Pile driving records

(iii) Pile load test results

4.2 RESULTS OF SUBSOIL INVESTIGATION

Borelogs of BH1 to BH8 were prepared wherein there is lithological description of the soil strata, SPT blow counts per 0.3 m penetration, depths of disturbed and undisturbed soil samples collected,and laboratory test results. Apart from these, the pile is also shown with levels of tip and butt. To calculate the capacity of pile using static methods,the soil parameters were used . The results of the subsoil investigation are shown in Fig.A-1 to Fig. A-8 in Appendix-A

4.3 PILE DRIVING RECORDS

The piles were marked at 0.30m intervals to record the blow counts for each 0.30m penetration . Driving records of all the 8 piles have been presented in Table B-1 to Table B-4 in Appendix-B

4.4 PILE LOAD TEST RESULTS

Eight full scale load tests were carried out at four sites. In each case piles were loaded to failure.Pile load test data are presented in Table C-1 to Table C-8 in Appendix-C. The results of these tests have been presented in the form of :

(i) Load-Settlement curve

(ii) Time-Settlement curve and

(iii) Time-Load curve

In these presentation gross settlement corresponding to each load increment have been used.The Load-Settlement curve, Time-Settlement curve and Time-Load curves are shown in Fig. D-1 to Fig. D-8 in Appendix -D

CHAFTER-5

DISCUSSION

5.1 GENERAL

Maintained load tests on eight piles were performed at four locations of Dhaka city. All the pile load test were performed in the soil condition where top soil upto 3m to 4m is not suitable for shallow foundation .The top 3 m to 4 m soil was very weak.The ultimate capacity of the piles were found to depend on the soil condition surrounding the pile.To analyze the results of the load tests the piles were divided into three groups depending on the soil condition surrounding and underlying the pile. The groups are:

.(i) Pile driven through clay and resting on clay

(ii) Pile driven through clay and resting on sand and

(iii) Pile driven through plastic silt and resting on nonplastic silt.

5.2 CAPACITY OF PILE DRIVEN THROUGH CLAY AND RESTING ON CLAY

Load tests on two piles were performed at Bonosree site. Soil conditions surrounding the piles are shown in Fig. A-1 and A-2. In this site fills consisting of fine sand in a very loose state exists upto the depth of 4.0m. The underlying soil is clay. Hence, both of the skin friction and end bearing of the pile is contributed by the cohesive soil.

5.2.1 Ultimate Capacity

Ultimate Capacity from Load Test

The pile load test was performed according to ASTM D-1143 .The load was applied in increment of 2 tons. Pile load test data are shown in Table C-1 and C-2. At the pick load the rate of settlement continued undiminished without further increment of load .

The load settlement curves of the piles are drawn in Fig. D-1 and D-2. The ultimate capacity from the pile load test was evaluated using

(i) Terzaghi's method (1942),

(ii) IS : 2911-1979 [Indian Standards Institution(1979)]

(iii) BS :8004-1986 and

(iv) Davission's method [Davisson(1973)]

The procedure of determining ultimate load according to Davission (1975) is shown in Fig. 5.1 (A) and 5.1 (B)

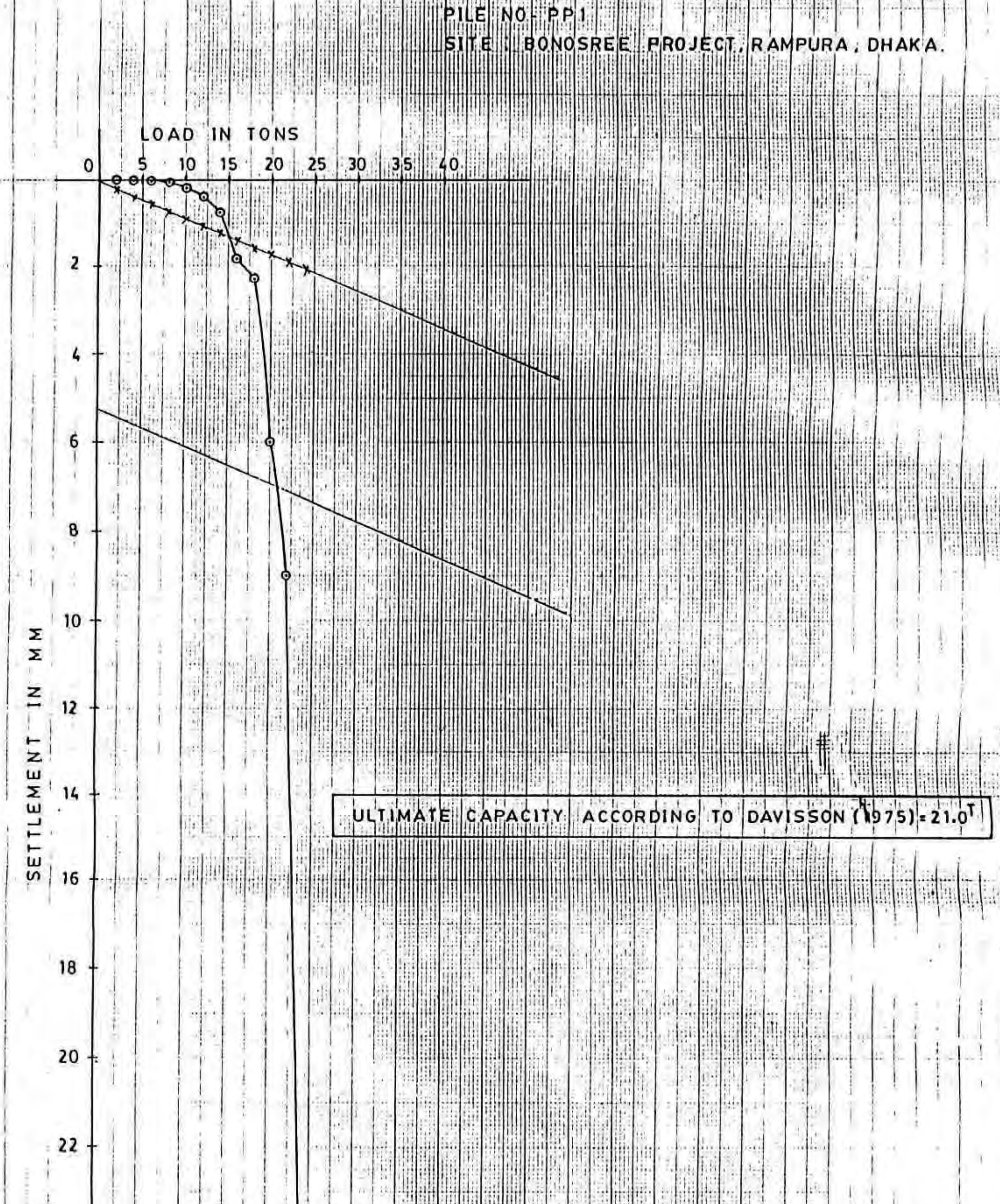


FIGURE 5·1(A) LOAD SETTLEMENT CURVES FOR PILE PP1 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSON'S METHOD (1975)

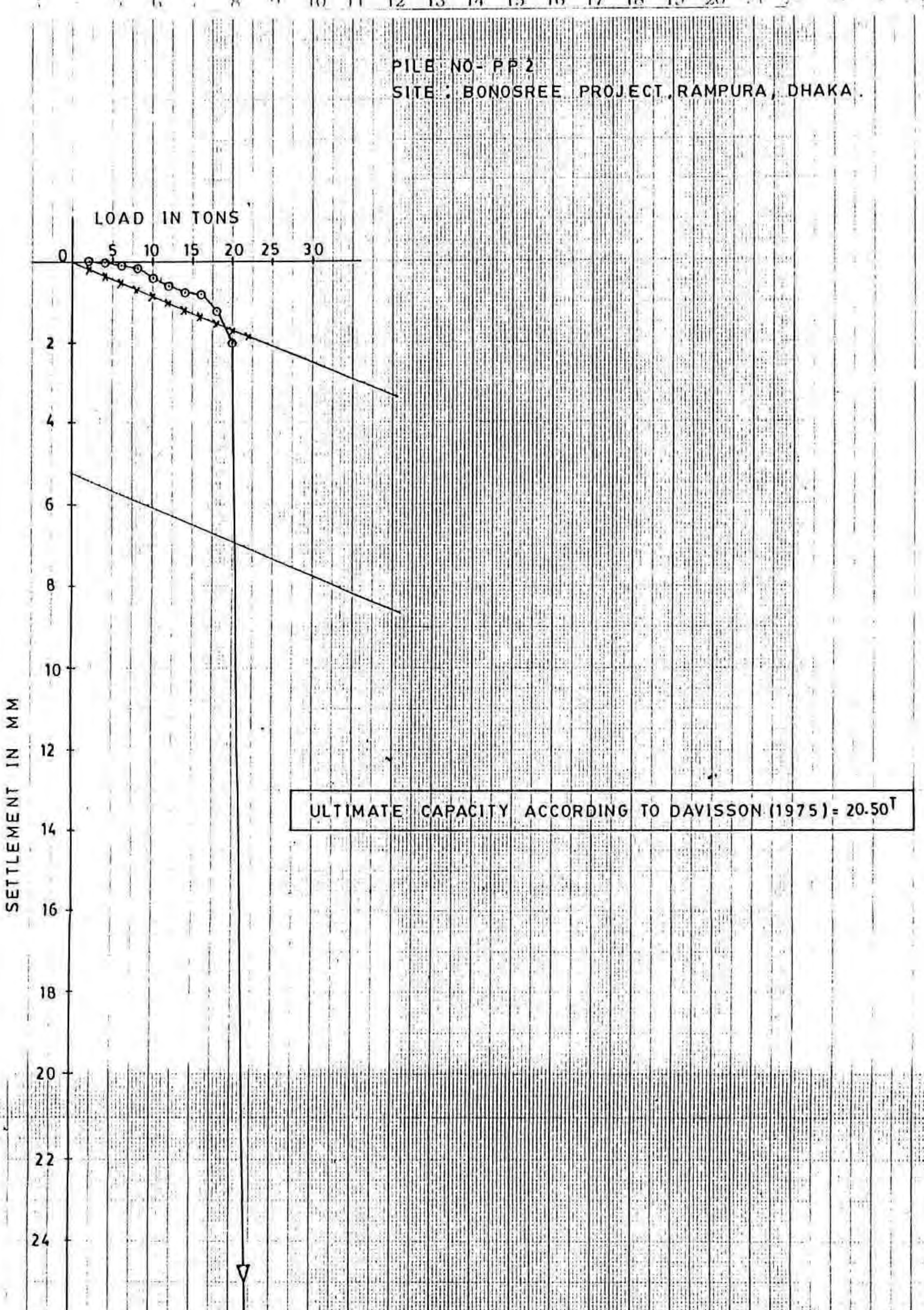


FIGURE 5.1(B) LOAD SETTLEMENT CURVE FOR PILE PP2 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSON'S METHOD (1975)

The evaluated ultimate loads are presented in Table-5.1 .The ultimate capacity evaluated by Terzaghi's method (1942), IS : 2911-1979 and BS :8004-1986 are equal. The ultimate capacity determined by Davisson's method [Davission (1973)] is slightly lower than that by other three methods. The ultimate load of the two piles tested are almost equal.

Ultimate Capacity From Static Methods

The ultimate capacity of the piles were predicted using static methods based on laboratory test data of the soil.Skin friction was calculated using

(i) α method [Tomlinson (1971)]

(ii) λ method [Vijayvergiya & Focht (1972)]

(iii) α_2 method [Peck et al (1974)]

(iv) API method [API (1984)]

(v) method recommended by Canadian Geotechnical Society (1985) and

(vi) method recommended by Indian Standards Institution (IS:2911-1979 }.

The end bearing was calculated using equation $q_u = cN_c$ (N_c =9).

Table 5.1 Measured Ultimate Capacity from Pile Load Test using Different Methods (Bonosree Site)

Site	Pile No.	Method	Criteria for determining ultimate capacity from pile load test	Ultimate capacity (Ton)
Bonosree (Piles in Clay)	PP1	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	22.80
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile b) Load corresponding to settlement 12mm.	22.80
		BS: 8004-1986	Load corresponding to settlement 10% of pile diameter	22.80
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	21
	PP2	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	21.5
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile b) Load corresponding to settlement 12mm.	21.5
		BS: 8004-1986	a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile	21.5
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	20.2

The ultimate static capacity was taken as sum of the skin friction & end bearing.The ultimate capacity evaluated from static methods are presented in Table 5.2 The measured values of ultimate capacity is compared with the predicted values of ultimate capacity in Table-5.3. The ratio between the predicted ultimate capacity using static method and the measured ultimate capacity from load test is also shown in Table 5.3 (A).

It is observed that the ultimate capacity predicted using λ method [Vijayvergiya & Focht (1972)] is very close to the ultimate capacity from load test.The ultimate capacity predicted using α method [Tomlinson (1971)] is reasonably close to the measured ultimate load from load test. α_2 method [Peck et al (1974)] ,API method [API (1984)] , methods recommended by Canadian Geotechnical Society (1985) and Indian Standards Institution(IS:2911-1979) underestimate the ultimate capacity.

Ultimate Capacity From Dynamic Formulae

The driving records of the piles are produced in Table B-1 in Appendix-B.The ultimate capacity of piles were predicted from Engineering News Records formula, Janbu formula and Hiley formula. The predicted values are presented in Table 5.4. The ultimate capacity from dynamic formulae are also compared with the ultimate capacity from load test in Table 5.4.

Table 5.2 Predicted Ultimate Capacity using Static Methods (Bonosree Site)

Site	Pile No.	Method of prediction (Static Methods)	Predicted ultimate capacity (Tons)		
			Skin friction	End bearing	Total capacity
Bonosree (Piles in Clay)	PP1	α method [Tomlinson (1971)] for skin friction & equation $q_u=cN_c$ for end bearing	17.17	2.50	19.67
		λ method[Vijayvergiya & Focht (1972)] for skin friction & equation $q_u=cN_c$ for end bearing	18.76	2.50	21.26
		α_2 method [Peck et al (1974)] for skin friction & equation $q_u=cN_c$ for end bearing	12.83	2.50	15.33
		API(1984) method	10.85	2.50	13.35
		Method recommended by Canadian Geotechnical Society(1985)	14.28	2.50	16.78
		Method recommended by Indian Standards Institution (IS:2911-1979)	7.89	2.50	10.39
	PP2	α method [Tomlinson (1971)] for skin friction & equation $q_u=cN_c$ for end bearing	17.58	2.48	20.06
		λ method [Vijayvergiya & Focht (1972)] for skin friction & equation $q_u=cN_c$ for end bearing	18.57	2.48	21.05
		α_2 method [Peck et al (1974)] for skin friction & equation $q_u=cN_c$ for end bearing	13.10	2.48	15.58
		API(1984) method	10.10	2.48	12.58
		Method recommended by Canadian Geotechnical Society(1985)	13.23	2.48	15.71
		Method recommended by Indian Standards Institution (IS:2911-1979)	8.08	2.48	10.56

Table 5.3 Comparison of Predicted Ultimate Capacity from Static Methods with Ultimate Capacity from Load Tests (Bonosree Site)

Site	Pile No.	Method of prediction (Static methods)	Predicted ultimate capacity (Tons)			Measured ultimate capacity [BS:8004-1986) (Tons)
			Skin friction	End bearing	Total capacity	
Bonosree (Piles in Clay)	PP1	α method [Tomlinson (1971)] for skin friction & equation q_u=cN_c for end bearing	17.17	2.50	19.67	22.8
		λ method[Vijayvergiya & Focht (1972)] for skin friction & equation q_u=cN_c for end bearing	18.76	2.50	21.26	
		α_2 method [Peck et al (1974)] for skin friction & equation q_u=cN_c for end bearing	12.83	2.50	15.33	
		API(1984) method	10.85	2.50	13.35	
		Method recommended by Canadian Geotechnical Society(1985)	14.28	2.50	16.78	
		Method recommended by Indian StandardsInstitution (IS:2911-1979)	7.89	2.50	10.39	
	PP2	α method [Tomlinson (1971)] for skin friction & equation q_u=cN_c for end bearing	17.58	2.48	20.06	21.5
		λ method [Vijayvergiya & Focht(1972)] for skin friction & equation q_u=cN_c for end ring	18.57	2.48	21.05	
		α_2 method [Peck et al (1974)] for skin friction & equation q_u=cN_c for end bearing	13.10	2.48	15.58	
		API(1984) method	10.10	2.48	12.58	
		Method recommended by Canadian Geotechnical Society(1985)	13.23	2.48	15.71	
		Method recommended by Indian Standards Institution (IS:2911-1979)	8.08	2.48	10.56	

Table 5.3 (A) Ratio between Predicted Ultimate Capacity and Measured Ultimate Capacity from Load Test (Bonosree Site)

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity from static methods		Predicted ultimate capacity / Measured ultimate capacity
			Method of prediction (Static Methods)	Predicted ultimate capacity (Tons)	
Bonosree (Piles in Clay)	PP1	22.8	α method [Tomlinson (1971)] for skin friction and equation q_u=cNc for end bearing	19.67	0.86
			λ method[Vijayvergiya & Focht (1972)] for skin friction & equation q_u=cNc for end bearing	21.26	0.93
			α_2 method [Peck et al (1974)] for skin friction & equation q_u=cNc for end bearing	15.33	0.67
			API(1984) method	13.35	0.58
			Method recommended by Canadian Geotechnical Society(1985)	16.78	0.73
			Method recommended by Indian StandardsInstitution (IS:2911-1979)	10.39	0.45
	PP2	21.5	α method [Tomlinson (1971)] for skin friction & equation q_u=cNc for end bearing	20.06	0.93
			λ method [Vijayvergiya & Focht(1972)] for skin friction & equation q_u=cNc for end bearing	21.05	0.97
			α_2 method [Peck et al (1974)] for skin friction & equation q_u=cNc for end bearing	15.38	0.71
			API(1984) method	12.58	0.58
			Method recommended by Canadian Geotechnical Society(1985)	15.71	0.73
			Method recommended by Indian Standards Institution (IS:2911-1979)	10.56	0.49

Table 5.4 Comparison of Predicted Ultimate Capacity using Dynamic Formulae with the Ultimate Capacity from Load Tests (Bonosree site)

Site	Pile No.	Predicted ultimate capacity using dynamic formulae (Tons)			Measured ultimate capacity from load test (Tons)
		Engg. News Records formula [Wellington (1888)]	Janbo formula (1953)	Hiley Formula (1925)	
Bonosree (Piles in Clay)	PP1	32	47.5	22.7	22.8
	PP2	34	55	22.7	21.5

It can be observed that the ultimate capacity predicted by Hiley formula is close to the measured ultimate capacity. Jabu & ENR formulae overestimate the ultimate capacity.

5.2.2 Allowable Capacity

Allowable* Capacity *From Load Test

Allowable capacity was determined from pile load test using:

(i) Bangladesh National Building Code (1993)

According to BNBC (1993) the allowable capacity of a pile is half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm

ii) Indian Standards Institution (I S 2911-1979)

According to I S: 2911-1979 the allowable capacity of a pile is least of the following:

(a) Two thirds of the final load at which the total settlement attains a value of 12 mm

(b) Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of underreamed pile.

(iii) BSI-CP-2004-1972

According to BSI CP-2004-1972, the allowable capacity of a pile should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter.

The evaluated allowable capacity determined from pile load test are presented in Table 5.5.

It may be noted that the criteria for determining allowable capacity from load test is different for Bangladesh National Building Code (1993), IS : 2911-1979 and BSI-CP.-2004 : 1972. However, the allowable capacity determined from load test using these methods are equal.The above mentioned codes recommend settlement criteria. to determine the allowable capacity .

Allowable Capacity From Static Methods

Allowable capacity from the static methods were determined using recommended factors of safety .The values of predicted allowable capacity are presented in Table 5.6 The predicted allowable capacity from static methods are also compared with the allowable capacity from load test in Table 5.6.It is observed that the values of

Table 5.5 Allowable Capacity from Pile Load Test using Various Codes and Corresponding Settlement (Bonosree Site)

Site	Pile No.	Method	Criteria for determining allowable capacity from pile load test	Allowable capacity (Tons)	Settlement at allowable capacity
Bonosree (Piles in Clay)	PP1	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than .00028 mm/kg of test load or 20 mm.	11.4	0.3mm
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of underreamed pile.	11.4	0.3mm
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	11.4	0.3mm
	PP2	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than .00028 mm/kg of test load or 20 mm.	10.75	0.5mm
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of underreamed pile.	10.75	0.5mm
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	10.75	0.5mm

Table 5.6 Comparison of Predicted Allowable Capacity using Static Methods with the Allowable Capacity from Load Tests

Site	Pile No.	Methods of prediction(static methods)	Predicted allowable capacity (Tons)	FS	Measured allowable capacity from load test
Bonosree (Piles in Clay)	PP1	α method [Tomlinson (1971)] for skin friction & equation q_u=cN_c for end bearing	7.86	2.5	11.4
		λ method[Vijayvergiya & Focht (1972)] for skin friction & equation q_u=cN_c for end bearing	8.50	2.5	
		α_2 method [Peck et al (1974)] for skin friction & equation q_u=cN_c for end bearing	6.15	2.5	
		API(1984) method	5.34	2.5	
		Method recommended by Canadian Geotechnical Society(1985)	5.59	3	
		Method recommended by Indian StandardsInstitution (IS:2911-1979)	4.15	2.5	
	PP2	α method [Tomlinson (1971)] for skin friction & equation q_u=cN_c for end bearing	8.02	2.5	10.75
		λ method[Vijayvergiya & Focht (1972)] for skin friction & equation q_u=cN_c for end bearing	8.42	2.5	
		α_2 method [Peck et al (1974)] for skin friction & equation q_u=cN_c for end bearing	6.23	2.5	
		API(1984) method	5.03	2.5	
		Method recommended by Canadian Geotechnical Society(1985)	5.23	3	
		Method recommended by Indian Standards Institution (IS:2911-1979)	4.22	2.5	

allowable capacity predicted using static methods are smaller than the allowable capacity determined from load test.It may be noted that the allowable capacity from pile load test is based on settlement criteria and the allowable capacity from static methods is determined using recommended factors of safety on ultimate bearing capacity.

Allowable load from dynamic formulae

Allowable capacity from the dynamic formulae were predicted from Engineering News Records formula, Janbu formula and Hiley formula using the recommended factors of safety.For determining the allowable capacity from dynamic formulae the recommended factor of safety is higher than the recommended factor of safety in static methods. The recommended factor of safety for ENR formula,Janbu formula and Hiley formula are 6, 3 to 6 and 4 respectively. The predicted allowable capacity are presented in Table 5.7.The predicted allowable capacity from dynamic formula are also compared with the allowable capacity from load test in Table 5.7. It is observed that the allowable capacity by Janbu formula is close to the measured allowable capacity. The predicted allowable capacity using ENR formula and Hiley formula is about half of the allowable capacity determined from pile load test.

Table 5.7 Comparison of Predicted Allowable Capacity using Dynamic Formulae with Allowable Capacity from Load Tests (Bonosree Site)

Site	Pile No.	Predicted allowable capacity using dynamic formulae (Tons)			Allowable capacity from load tests [BNBC(1993)] (Tons)
		Engg. News Records formula [Wellington (1888)] (FS=6)	Janbo formula (1953) (FS=4.5)	Hiley formula (1925) (FS=4)	
Bonosree (Piles in Clay)	PP1	5.33	10.55	5.67	11.4
	PP2	5.66	12.22	5.67	10.75

//

5.2.3 Settlement At AllowableCapacity

From the load settlement curves of Fig. D-1 and D-2 (Appendix-D) it is observed that the settlement at the allowable capacity is 0.3 mm for pile PP1 and 0.5 mm for pile PP2. At the allowable load, the settlement is very small. Settlements at the allowable loads are shown in Table 5.5

5.2.4 Unit Skin Friction Of Dhaka Clay

The load-settlement curves of the piles at Bonosree site are presented in Fig. D-1 and D-2 (Appendix-D). The ultimate capacity measured from from the tests are 22.8 tons for PP1 and 21.5 tons for PP2.

From the borelogs it can be observed that the pile derives its capacity mainly from skin friction of the clay layer extending from 4.0m to 8.0m depth. This clay layer represents typical Dhaka Red Clay . The SPT blow count of this clay layer varies from 9 to 16.The natural moisture content ranges from 22% to 24%.The unconfined compressive strength of this clay fall in range of 15 ton/m^2 to18 ton/m^2 .The liquid limit varies from 48%to 50% and plastic limit varies from 20% to 21% .The end bearing capacity calculated using the equation $q_u = cN_c$ ($N_c = 9$) is only 2.5 tons. Subtracting the end bearing from the measured capacity of the pile the total measured value of the skin friction can be calculated.Considering the top soil upto

4.0m very loose and assuming that it contributes very negligible skin friction, the total skin friction of the Dhaka clay layer in this case is 22.15-2.5 = 19.65 tons. Hence, the unit skin friction is 7.0 ton/m^2 [70 kPa.] .The ultimate skin friction predicted by static methods is compared with the estimated ultimate skin friction from pile load test in Table 5.8.It can be observed that in this type of soil the ultimate predicted skin friction using α method and λ method are close to the estimated ultimate skin friction from load test. API method [API (1984)], method recommended by Canadian Geotechnical Society (1985) and method recommended by Indian Standards Institution (IS:2911-1979) grossly underestimate the skin friction friction of Dhaka Clay

The recommended value of reduction factor α_2 [Peck et al (1974)] and the value of reduction factor α_2 determined from pile load test is shown in Fig. 5.2 It can be observed from the figure that the value of α_2 for Dhaka Clay is higher than the average value.However the value lies within the upper limit.

5.3 CAPACITY OF PILES DRIVEN THROUGH CLAY AND RESTING ON SAND

The piles of Ahmedbagh and Moghbazar sites are driven through clay and rest on sand.

Load tests on two piles were performed at Ahmedbagh site . Soil condition surrounding the piles are shown in Fig.A-5 and A-6 . In this site recent fills exists upto 3.0m depth underlain by stiff Dhaka Clay from 3m to 7m. There is a sand

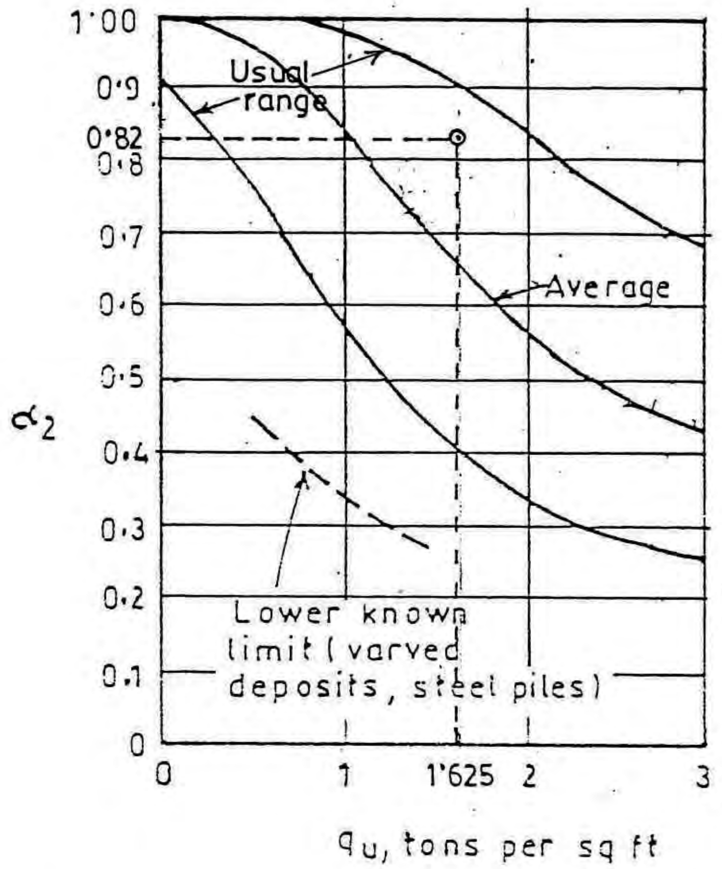


Figure 5.2 Values of reduction factor $\alpha 2$ for calculation of static capacity of friction pile in clays of different unconfined compressive strengths q_u. [Pecket al (1974)]

Table 5. 8 Comparison of Predicted Values of Skin Friction with Measured Values of Skin Friction (Bonosree Site)

Site	Pile No.	Estimated values of skin friction (Tons)	Predicted values of skin friction from static methods (Tons)					
			α method [Tomlinson (1971)]	λ method [Vijayvergiya & Focht (1972)]	α_2 method [Peck et al (1974)]	API (1984) method	Method recommended by Canadian Geotechnical Society	Method recommended by Indian Standards Institution (IS:2911-1979)
Bonosree	PP1	20.3	17.17	18.76	12.83	10.85	14.28	7.89
(Piles in Clay)	PP2	19.02	17.58	18.52	13.10	10.10	13.23	8.08

layer below this strata. The pile at this site derives its capacity from both of skin friction and end bearing.

At Moghbazar site also two pile load tests were performed . Soil condition surrounding the piles are shown in Fig.A-7 & A-8 . In this site recent fills also exist upto the depth of 3.0m at the location of pile PP-7 & upto 3.50m at the location of pile PP8. Underlying this layer there is a clay layer from 3m to 6m at location of PP7 & 3.5m to 7.0m at the location of PP8. Similar to Ahmedbagh site there is a sand layer below this red clay layer and the piles derive capacity from skin friction and end bearing.

5.3.1 Ultimate Capacity

Ultimate **Capacity** ***From Load Test***

Maintained pile load test to failure was performed according to ASTM D-1143 .The load was applied in increment of 2.5 tons.The pile load test data are shown in Table C-5 and Table C-6 and ,Table C-7 and Table C-8. At the pick load the rate of settlement continued undiminished without further increment of load .

The load settlement curves of the piles are drawn in Fig. D-5 and D-6 and, D-7 and D-8.The ultimate capacity was evaluated by :

(i) Terzaghi's method (1942),

(ii) IS : 2911-1979

(iii) BS 8004 : 1986. and

(iv) Davission's method (1973)

The procedure of determining ultimate capacity according to Davisson (1973) is shown in Fig. 5.3 (A), 5.3 (B) and Fig. 5.4 (A) and 5.4 (B).

The ultimate capacities are presented in Table 5.9. The ultimate capacity evaluated by Terzaghi's method (1943), IS : 2911-1979 [Indian Standards Institution(1979)] and BS :8004-1986 are equal. The ultimate capacity determined by Davisson's method [Davisson(1973)] is negligibly lower than that by other three methods. The ultimate capacity of the two piles of each site are almost equal. The ultimate capacity determined from load tests are 37.0 tons for pile PP5 and 39.5 tons for pile PP6, and 34.5 tons for pile PP7 and 36.9 tons for pile PP8

Ultimate **Capacity** *From Static Methods*

Ultimate capacity was predicted based on soil parameters obtained from geotechnical investigation. Skin friction was calculated using :

(I) α method [Tomlinson (1971)]

(ii) λ method [Vijayvergiya & Focht (1972)]

(iii) α_2 method [Peck et at (1974)]

(iv) API method [API (1984)]

(v) method recommended by Canadian Geotechnical Society (1985) and

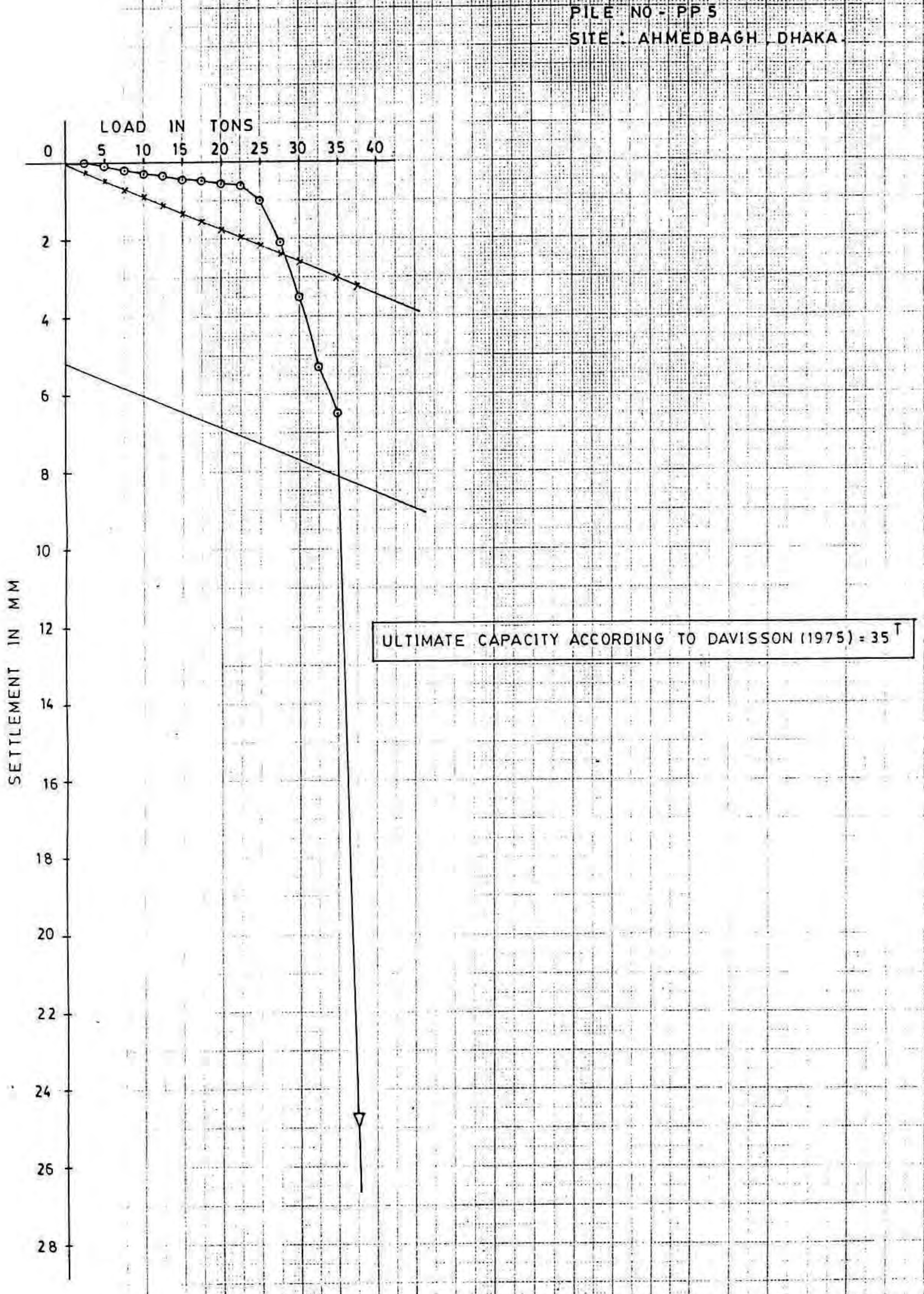


FIGURE 5.3(A) LOAD SETTLEMENT CURVE FOR PILE PP 5 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSON'S METHOD (1975)

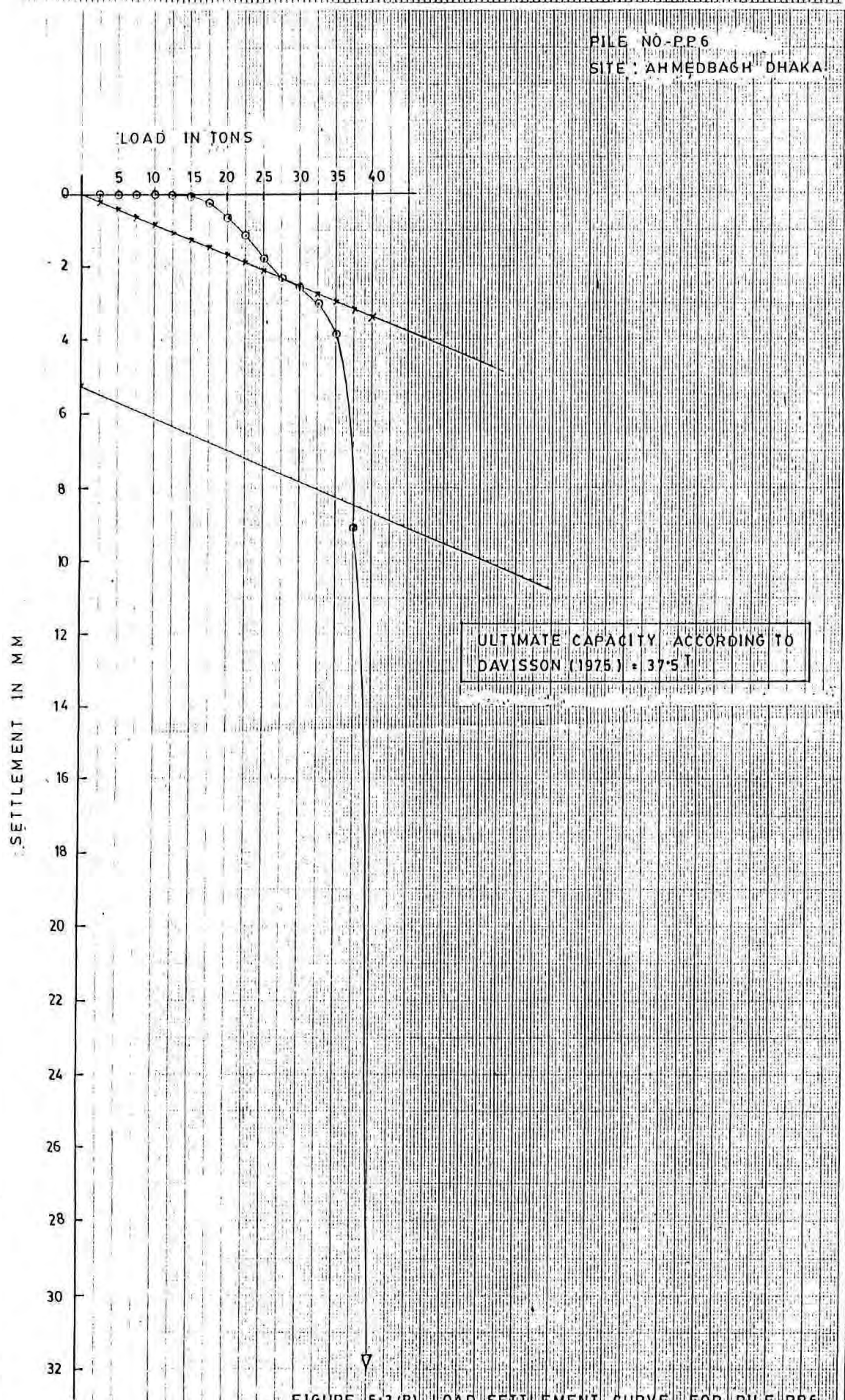


FIGURE 5·3 (B) LOAD SETTLEMENT CURVE FOR PILE PP6 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSON'S METHOD (1975)

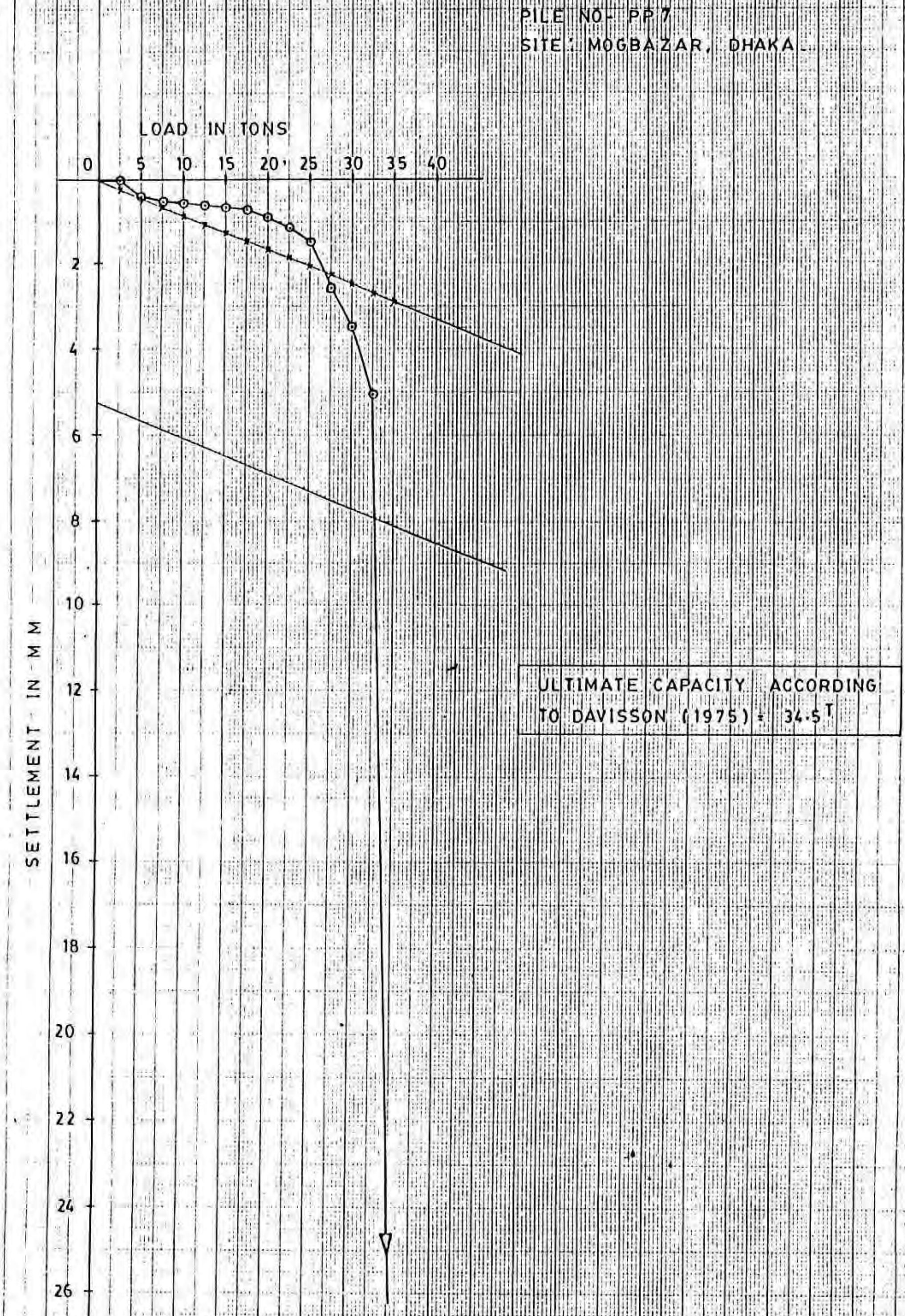


FIGURE 5.4 (A) LOAD SETTLEMENT CURVES FOR PILE PP 7 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSON'S METHOD (1975)

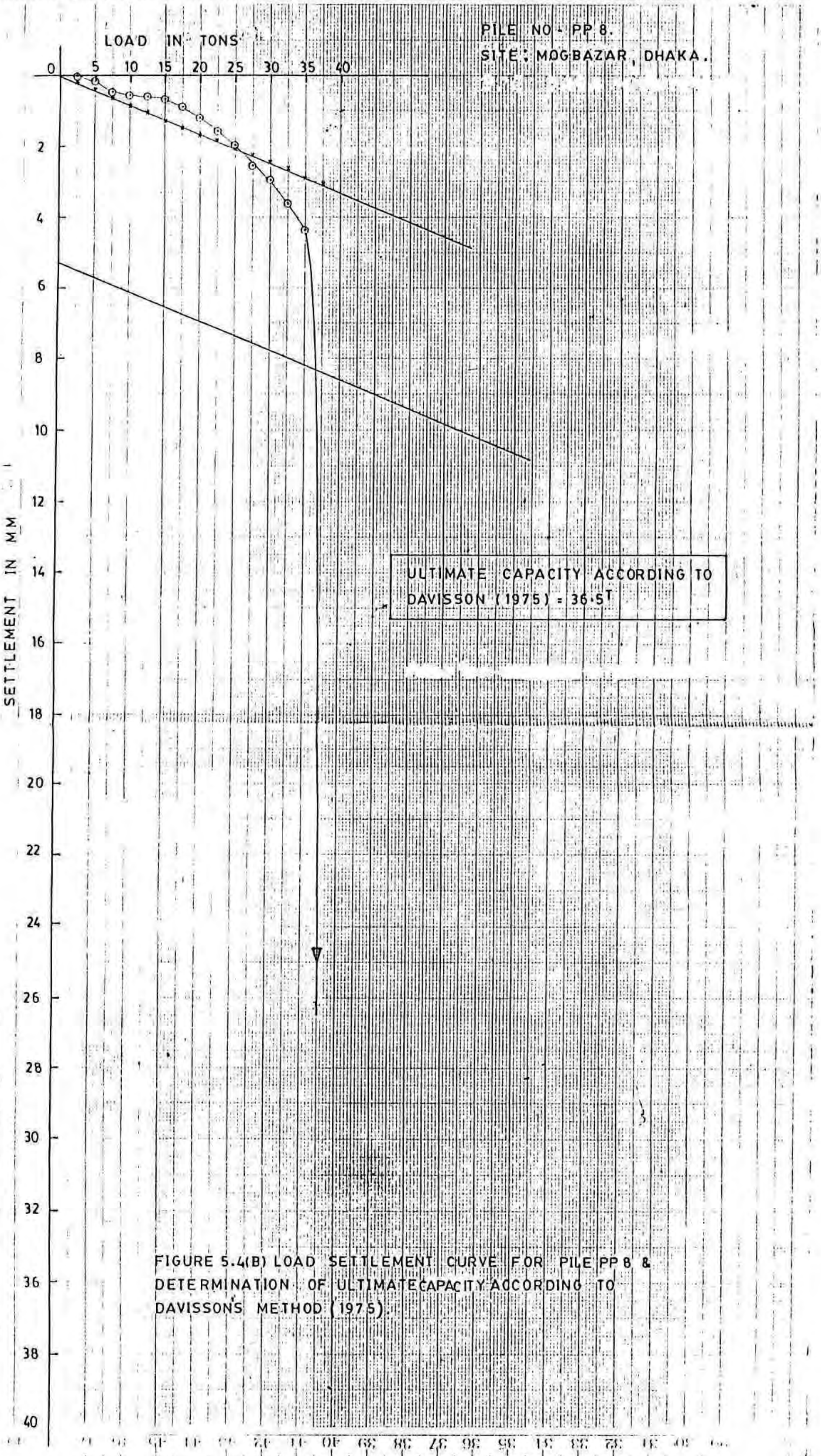


FIGURE 5.4(B) LOAD SETTLEMENT CURVE FOR PILE PP 8 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSONS METHOD (1975).

Table 5.9 Measured Ultimate Capacity from Pile Load Test using Different Methods (Ahmedbagh Site and Moghbazar Site)

Site	Pile No.	Method	Criteria for determining ultimate capacity from pile load test	Measured ultimate capacity from pile load test (Tons)
Ahmedbagh (Piles through Clay and resting on Sand)	PP5	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	37
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile b) Load corresponding to settlement 12mm.	37
		BS: 8004-1986	Load corresponding to settlement 10% of pile diameter	37
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	36
	PP6	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	39.5
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile b) Load corresponding to settlement 12mm.	39.5
		BS: 8004-1986	Load corresponding to settlement 10% of pile diameter	39.5
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	37.5
Moghbazar	PP7	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	34.5
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile b) Load corresponding to settlement 12mm.	34.5
		BS: 8004-1986	Load corresponding to settlement 10% of pile diameter	34.5
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	34.5
	PP8	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	36.9
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base diameter for underreamed pile b) Load corresponding to settlement 12mm.	36.9
		BS: 8004-1986	a) Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base dia for underreamed pile	36.9
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	36.5

(vi) method recommended by Indian Standards Institution (IS:2911-1979).

End bearing was calculated using :

(I) Terzaghi's Method [Terzaghi (1943), Terzaghi and Peck(1967)]

(ii) Meyerhof's empirical method [Meyerhof (1956, 1976)]

(iii) Hansen's method [Hansen (1970)]

(iv) API method (1984, 1987)

(v) method recommended by Canadian Geotechnical Society (1985)

(vi) method recommended by Indian Standards Institution (IS : 2911-1979).

The value of angle of internal friction ϕ required for the determination of N_q was determined from the correlation of ϕ and SPT blow count N given by Peck et al (1974) shown in Fig. 2.16 .

It may be mentioned that the skin friction of the thin layer of sand near the bottom of pile was determined using Tomlinson's method (1986)

The total ultimate predicted capacity of a pile is the sum of the skin friction along the shaft of the pile and the end bearing at the pile tip .To predict the total ultimate capacities of the piles the following combinations of skin friction and end bearing were used :

Combination-1:	α method [Tomlinson (1971)] for skin friction + Terzaghi's Method [Terzaghi (1943),Terzaghi and Peck(1967)] for end bearing.
Combination -2 :	α method [Tomlinson (1971)] for skin friction + Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing
Combination -3 :	α method [Tomlinson (1971)] for skin friction + Hansen's method [Hansen (1970)] for end bearing
Combination -4 :	λ method [Vijayvergiya & Focht (1972)] for skin friction + Terzaghi's method [Terzaghi (1943),Terzaghi and Peck (1967)] for end bearing
Combination -5 :	λ method [Vijayvergiya & Focht (1972)] for skin friction + Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing
Combination -6 :	λ method [Vijayvergiya & Focht (1972)] for skin friction + Hansen's method [Hansen (1970)] for end bearing
Combination -7 :	α_2 method [Peck et al (1976)] for skin friction + Terzaghi's method [Terzaghi (1943),Terzaghi and Peck(1967)] for end bearing
Combination -8 :	α_2 method [Peck et al (1976)] for skin friction + Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing

Combination -9 : α_2 method [Peck et al (1976)] for skin friction + Hansen's method [Hansen (1970) for end bearing

The ultimate capacity of piles were also predicted using the recommendations of the following methods:

Method-1 : API method (1984,1987)

Method-2 : method recommended by Canadian Geotechnical Society(1985)

Method-3 : method recommended by Indian Standards Institution (IS : 2911-1979).

The predicted values of ultimate capacity are presented in Table 5.10

The measured ultimate capacity determined from pile load tests are compared with the ultimate capacities predicted by using static methods in Table-5.11 The ratio between the predicted ultimate capacity using static methods and the measured ultimate capacity from load test is also shown in Table 5.11(A).

It is observed that the ultimate capacity obtained from the combination of λ method [Vijayvergiya & Focht (1972)] and Meyerhof's empirical method [Meyerhof (1956,1976)] or λ method [Vijayvergiya & Focht (1972)] and Hansen's method

Table 5.10 Predicted Ultimate Capacity using Static Methods (Ahmedbagh and Moghbazar Site)

Site	Pile No.	Predicted Ultimate Capacity using Static Methods (Tons)								
		Combination -1			Combination-2			Combination --3		
		Skin friction α method [Tomlinson (1971)]	End bearing [Terzaghi(1943)]	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing [(Meyerhof's emperical method (1976)]	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing [Hansen's method (1970)]	Total capacity
Ahmedbagh	PP5	18.41	5.65	24.06	18.41	12.24	30.65	18.41	7.93	26.34
	PP6	22.00	7.08	29.08	22.00	13.46	35.46	22.00	10.01	32.01
Moghbazar	PP7	17.66	7.13	24.79	17.66	14.6	32.26	17.66	10.15	27.81
(Piles Through Clay And Resting On Sand)	PP8	14.22	8.39	22.61	14.22	15.9	30.12	14.22	11.81	26.03

Table 5.10 Contd..

Site	Pile No.	Predicted Ultimate Capacity using Static Methods (Tons)								
		Combination -4			Combination-5			Combination --6		
		Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing [Terzaghi(1943)]	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing [Meyerhof's emperical method (1976)]	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing Hansen's method Hansen(1970)	Total capacity
Ahmedbagh	PP5	23.81	5.65	29.46	23.81	12.24	36.05	23.81	7.93	31.74
	PP6	28.69	7.08	35.77	28.69	13.46	42.15	28.69	10.01	38.7
Moghbazar	PP7	19.73	7.13	26.86	19.73	14.60	34.33	19.73	10.5	29.91
	PP8	19.05	8.39	27.41	19.05	15.9	34.95	19.05	11.81	30.86

Table 5.10 Contd...

Site	Pile No.	Predicted Ultimate Capacity using Static Methods (Tons)								
		Combination -7			Combination-8			Combination --9		
		Skin friction α_2 method [Peck et al (1974)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction α_2 method [Peck et al (1974)]	End bearing [Meyerhof"s emperical method Meyerhof(1976)]	Total capacity	Skin friction α_2 method [Peck et al (1974)]	End bearing [Hansen's method Hansen(1970)]	Total capacity
Ahmedbagh	PP5	18.44	5.65	24.09	18.44	12.24	30.68	18.44	7.93	26.37
	PP6	20.18	7.08	27.26	20.18	13.46	33.64	20.18	10.01	30.18
Moghbazar	PP7	17.7	7.13	24.83	17.7	14.67	32.30	17.7	10.15	27.85
	PP8	16.17	8.39	24.56	16.17	15.90	32.07	16.17	11.81	27.98

Table 5.10 Cont.d...

Site	Pile No.	Predicted Ultimate Capacity using Static Methods (Tons)								
		Method-1 API(1984,1987)			Method-2[Method recommended by Canadian Geotechnical Society(1985)]			Method-3[Method recommended by Indian Standards Institution(IS:2911-1979)]		
		Skin friction	End bearing	Total capacity(tons)	Skin friction	End bearing	Total capacity(tons)	Skin friction	End bearing	Total capacity(tons)
Ahmedbagh	PP5	16.10	3.52	19.62	17.97	9.88	27.85	13.41	3.85	17.26
	PP6	9.61	4.25	23.86	20.46	10.74	31.2	16.49	4.13	20.62
Moghbazar	PP7	13.2	4.0	17.21	14.91	12.35	27.26	10.29	6.27	16.56
		12.0	4.28	16.28	15.62	13.27	28.89	10.60	7.50	18.10

Table 5.11 Comparison of Predicted Ultimate Capacity using Static Methods with Measured Ultimate Capacity from load Tests

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Combination -1			Combination-2			Combination --3		
			Skin friction α method [Tomlinson (1971)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing (Meyerhof's emperical method) Meyerhof(1976)	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing Hansen's method Ha-nsen(1970)	Total capacity
Ahmedbagh	PP5	37	18.41	5.65	24.06	18.41	12.24	30.65	18.41	7.93	26.34
	PP6	39.5	22.00	7.08	29.08	22.00	13.46	35.46	22.00	10.01	32.01
Moghbazar	PP7	34.5	17.66	7.13	24.79	17.66	14.6	32.26	17.66	10.15	27.81
	PP8	36.9	14.22	8.39	22.61	14.22	15.9	30.12	14.22	11.81	26.03

Table 5.11 Cont.

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Combination -4			Combination-5			Combination --6		
			Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing Meyerhof's emperical method Meyerhof(1976)	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing Hansen's method Hansen (1970)	Total capacity
Ahmedbagh	PP5	37	23.81	5.65	29.46	23.81	12.24	36.05	23.81	7.93	31.74
	PP6	39.5	28.69	7.08	35.77	28.69	13.46	42.15	28.69	10.01	38.7
Moghbazar	PP7	34.5	19.73	7.13	26.86	19.73	14.60	34.33	19.73	10.5	29.91
	PP8	36.9	19.05	8.39	27.41	19.05	15.9	34.95	19.05	11.81	30.86

Table 5.11 Contd...

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Combination -7			Combination-8			Combination --9		
			Skin friction α_2 method [Peck et al (1974)]	End bearing [Terzaghi (1943)]	Total capacity(tons)	Skin friction α_2 method [Peck et al (1974)])	End bearing Meyerhof's emperical method Meyerhof(1976)	Total capacity (tons)	Skin friction α_2 method [Peck et al (1974)].	End bearing Hansen's method Hansen(1970)	Total capacity(tons)
Ahmedbagh	PP5	37	18.44	5.65	24.09	18.44	12.24	30.68	18.44	7.93	26.37
	PP6	39.5	20.18	7.08	27.26	20.18	13.46	33.64	20.18	10.01	30.18
Moghbazar	PP7	34.5	17.7	7.13	24.83	17.7	14.67	32.30	17.7	10.15	27.85
	PP8	36.9	16.17	8.39	24.56	16.17	15.90	32.07	16.17	11.81	27.98

Table 5.11 Cont.

Site	Pile No	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Method-1 API(1984,1987)			Method-2[Method recommended by Canadian Geotechnical Society(1985)]			Method-3[Method recommended by Indian Standards Institution(IS:2911-1979)]		
			Skin friction	End bearing	Total capacity	Skin friction	End bearing	Total capacity	Skin friction	End bearing	Total capacity
Ahmedbagh	PP5	37	16.10	3.52	19.62	17.97	9.88	27.85	13.41	3.85	17.26
	PP6	39.5	19.61	4.25	23.86	20.46	10.74	31.2	16.49	4.13	20.62
Moghbazar	PP7	34.5	13.2	4.0	17.21	14.91	12.35	27.26	10.29	6.27	16.56
	PP8	36.9	12.0	4.28	16.28	15.62	13.27	28.89	10.60	7.50	18.10

Table 5.11 (A) Ratio between Predicted Ultimate Capacity and Measured Ultimate Capacity from Load Test (Ahmedbagh Site And Moghbazar Site)

Site	Pile No.	Measured ultimate capacity from pile load test (Tons)	Predicted ultimate capacity from static methods		Predicted ultimate capacity- Measured ultimate capacity
			Method of prediction (static methods)	Predicted ultimate capacity (Tons)	
			α method [Tomlinson (1971)] for skin friction & Terzaghi's Method [Terzaghi (1943)] method for end bearing	24	0.64
			α method [Tomlinson (1971)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	30.65	0.82
			α method [Tomlinson (1971)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	26.34	0.71
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Terzaghi's Method [Terzaghi (1943) for end bearing	29.46	0.79
Ahmedbagh	PP 5	37	λ method [Vijayvergiya & Focht(1972)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	36.05	0.97
(Piles through Clay and resting on Sand)			λ method [Vijayvergiya & Focht(1972)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	31.74	0.85
			α_2 method [Peck et al (1974)] for skin friction & Terzaghi's Method [Terzaghi (1943)] for end bearing	24.09	0.65
			α_2 method [Peck et at (1974)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	30.68	0.82
			α_2 method [Peck et al (1974)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	26.37	0.71
			API (1984) method	19.62	0.53
			Method recommended by Canadian Geotechnical Society(1985)	27.85	0.75
			Method recommended by Indian Standards Institution(IS:2911-1979)	17.26	0.46

Table 11 (A) Cont.

Site	Pile No.	Measured utimate capacity from load test (Tons)	Predicted ultimate capacity from static methods		Predicted ultimate capacity / Measured ultimate capacity
			Method of prediction (static methods)	Predicted ultimate capacity (Tons)	
Ahmedbagh (Piles driven through Clay and resting on sand)	PP6	39.5	α method [Tomlinson (1971)] for skin friction & Terzaghi's method [Terzaghi (1943)] method for end bearing	29	0.73
			α method [Tomlinson (1971)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	35.4	0.89
			α method [Tomlinson (1971)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	32	0.81
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Terzaghi's Method [Terzaghi (1943) for end bearing	35.7	0.90
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	42.1	1.06
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	38.7	0.97
			α_2 method [Peck et al (1974)] for skin friction & Terzaghi's Method [Terzaghi (1943)] for end bearing	27	0.68
			α_2 method [Peck et al (1974)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	33.6	0.85
			α_2 method [Peck et al (1974)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	30.1	0.76
			API (1984) method	23.8	0.60
			Method recommended by Canadian Geotechnical Society(1985)	31	0.78
			Method recommended by Indian Standards Institution(IS:2911-1979)	20.62	0.52

Table 5.11 (A) Cont.

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity from static methods		Predicted ultimate capacity / Measured ultimate capacity
			Method of prediction (Static methods)	Predicted ultimate capacity (Tons)	
Moghbazar (Piles driven throughClay and resting on sand)	PP7	34.5	α method [Tomlinson (1971)] for skin friction & Terzaghi's Method [Terzaghi (1943)] method for end bearing	24.79	0.71
			α method [Tomlinson (1971)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	32.26	0.92
			α method [Tomlinson (1971)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	27.81	0.80
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Terzaghi's Method [Terzaghi (1943) for end bearing	26.86	0.77
			λ method [Vijayvergiya & Focht 1972)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	34.33	0.99
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	29.91	0.86
			α_2 method [Peck et al (1974)] for skin friction & Terzaghi's Method [Terzaghi (1943)] for end bearing	24.83	0.72
			α_2 method [Peck et al (1974)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	32.30	0.93
			α_2 method [Peck et a (1974)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	27.85	0.81
			API (1984) method	17.21	0.50
			Method recommended by Canadian Geotechnical Society(1985)	27.26	0.79
			Method recommended by Indian Standards Institution(IS:2911-1979)	16.56	0.48

Table 5.11 (A) Cont.

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity from static methods		Predicted ultimate capacity / Measured ultimate capacity
			Method of prediction (Static methods)	Predicted ultimate capacity (Tons)	
Moghbazar (Piles driven through clay and resting on sand)	PP8	36.9	α method [Tomlinson (1971)] for skin friction & Terzaghi's Method [Terzaghi (1943)] method for end bearing	22.6	0.61
			α method [Tomlinson (1971)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	30.12	0.81
			α method [Tomlinson (1971)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	26.03	0.70
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Terzaghi's Method [Terzaghi (1943) for end bearing	27.44	0.74
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	34.93	0.94
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	30.86	0.83
			α_2 method [Peck et al (1974)] for skin friction & Terzaghi's Method [Terzaghi (1943)] for end bearing	24.56	0.66
			α_2 method [Peck et al (1974)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	32.07	0.86
			α_2 method [Peck et al (1974)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	27.98	0.75
			API (1984) method	16.28	0.44
			Method recommended by Canadian Geotechnical Society(1985)	28.89	0.78
			Method recommended by Indian Standard Institution(IS:2911-1979)	18.10	.49

[Hansen (1970) is close to the ultimate capacity from load tests. α method [Tomlinson (1971)], α_2 method [Peck et al (1976)] in combinations with + Terzaghi's method [Terzaghi (1943),Terzaghi and Peck(1967)], Meyerhof's empirical method [Meyerhof (1956,1976)] or Hansen's method [Hansen (1970) and methods suggested by Canadian Geotechnical Society(1985) and Indian Standards Institution(IS : 2911-1979) and, API (1984,1987) method underestimate the ultimate pile capacity.

Ultimate **Capacity** *From Dynamic Methods*

The driving records of the piles are produced in Table B-3 to B-4 in Appendix-B. The ultimate capacity of the piles was predicted using :

(i) Engineering News Records formula (ENR) [Wellington (1888)]

(ii) Hiley formula. (1925) and

(iii) Janbu formula (1953)

The ultimate capacity predicted by dynamic formulae are presented in Table 5.12 These values are also compared with the ultimate capacity of piles determined from load test in Table-5.12

It is observed that ultimate capacity predicted by ENR is close to the measured ultimate capacity from load test .The predicted ultimate capacity by Janbu formula

Table 5.12 Comparison of Predicted Ultimate Capacity using Dynamic Formulae with the Ultimate Capacity from Load Tests (Ahmedbagh Site and Moghbazar Site)

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity using dynamic formulae (Tons)		
			Engg. News Records formula [Wellington(1888)]	Janbo formula (1953))	Hiley formula (1925)
Ahmedbagh	PP5	37	35	63	26
	PP6	39.5	42.5	73	25
Moghbazar	PP7	34.5	29	57	34
	PP8	36.9	30	39	26

overestimates the measured ultimate capacity and the Hiley formula underestimate the measured ultimate capacity.

5.3.2 Allowable capacity

Allowable* Capacity *From Load Test

Allowable capacity was determined from pile load test using:

(i) Bangladesh National Building Code (1993)

According to BNBC (1993) the allowable capacity of a pile is half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm

ii) Indian Standard Code of Practice (I S 2911-1979)

According to I S 2911-1979 the allowable capacity of a pile is least of the following:

(a) Two thirds of the final load at which the total settlement attains a value of 12 mm

(b) Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of under -reamed pile.

Table 5.13 Allowable Capacity from Pile Load Test using Various Codes and Corresponding Settlement (Ahmedbagh Site and Moghbazar Site)

Site	Pile No.	Method	Criteria for determining allowable capacity from pile load test	allowable capacity (Tons)	Settlement (mm)
Ahmedbagh (Piles through Clay and resting on Sand)	PP5	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm.	18.5	0.50
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of underreamed pile.	18.5	0.50
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	18.5	0.50
	PP6	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm.	19.75	0.60
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of underreamed pile.	19.75	0.60
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	19.75	0.60
Moghbazar (Piles through Clay and resting on Sand)	PP7	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm.	17.25	0.70
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of under -reamed pile.	17.25	0.70
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	17.25	0.70
	PP8	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm.	18.45	1.00
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of under -reamed pile.	18.45	1.00
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	18.45	1.00

(iii) <u>BSI-CP-2004-1972</u>

According to BSI CP-2004-1972, the allowable capacity of a pile should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter.

The evaluated allowable capacity determined from pile load test are presented in Table 5.13.

It may be noted that the criteria for determining allowable capacity from load test is different for Bangladesh National Building Code (1993), IS : 2911-1979 and BSI-CP.-2004 : 1972. However, the allowable capacity determined from load test are equal.The recommended allowable capacity of a pile from pile load test by the above mentioned methods are based on the settlement criteria.

Allowable **Capacity** *From Static Methods*

The allowable capacity of the piles were calculated from static methods using the recommended factors of safety.The allowable capacities determined from static methods are presented in Table 5.14 .The allowable capacity from static methods are also compared with the allowable capacity from load tests in Table.5.14

Table 5.14 Comparison of Allowable Capacity from Load Tests with Allowable capacity from Static Methods

Site	Pile No.	Allowable capacity from load test (Tons)	Allowable capacity using static methods (Tons)					
			Combination -1 (Skin friction by α method & end bearing by Terzaghi method)	Combination -2 (Skin friction by α method & end bearing by Meyerhof's emperical method)	Combination 3 (Skin friction by α method & end bearing by Hansen's method)	Combination -4 (Skin friction by λ method & end bearing by Terzaghi method)	Combination -5 (Skin friction by λ method & end bearing by Meyerhof's Emperical method)	Combination -6 (Skin friction by λmethod & end bearing by Hansen's method)
Ahmedbagh	PP5	18.56	9.62	12.26	10.53	11.78	14.42	12.69
	PP6	19.56	11.63	14.18	12.80	14.30	16.86	15.48
Moghbazar	PP7	17.5	9.91	12.9	11.12	10.74	13.73	11.96
	PP8	18.75	9.04	12.05	10.41	10.97	13.98	12.34

Table 5.14 Contd...

Site	Pile No.	Allowable capacity from load test (Tons)	Allowable capacity using static methods (Tons)					
			Combination-7 (Skin friction by α_2 method & end bearing by Terzaghi method)	Combination -8 (Skin friction by α_2 method & end bearing by Meyerhof's emperical method)	Combination-9 (Skin friction by α_2 method & end bearing by Hansen's method)	Method-1 [API method (1984,1987)]	Method-2: [Method recommended by Canadian Geotechnical Society(1985)]	Method-3 [Method rcommended by Indian Standards Institution(IS:2911-1979)]
Ahmedbagh	PP5	18.56	9.63	12.27	10.54	7.84	11.14	6.90
	PP6	19.56	10.9	13.45	12.07	9.53	12.48	8.24
Moghbazar	PP7	17.5	9.93	12.92	11.14	6.8	10.9	6.62
	PP8	18.75	9.92	12.82	11.19	6.51	11.55	7.22

It is observed that the allowable capacity predicted from the combination of λ method [Vijayvergiya & Focht (1972)] and Meyerhof's empirical method [Meyerhof (1956,1976)] or λ method [Vijayvergiya & Focht (1972)] and Hansen's method [Hansen (1970) underestimate the allowable capacity.However, the allowable capacity predicted by α method [Tomlinson (1971)] or α_2 method [Peck et at (1976)] in combinations with Terzaghi's method [Terzaghi (1943),Terzaghi and Peck(1967)], Meyerhof's empirical method [Meyerhof (1956,1976)] or Hansen's method [Hansen (1970) and method recommended by Canadian Geotechnical Society (1985) underestimate the allowable pile capacity by a large amount.The allowable capacity predicted by API (1984,1987) method and method recommended by Indian Standards Institution (IS : 2911-1979) is less than half of the allowable capacity determined from pile load test.

Allowable Capacity from dynamic formulae

Allowable loads predicted by dynamic analysis were also calculated using recommended factors of safety. The allowable capacity from dynamic formulae along with the recommended factors of safety are presented in Table 5.15.The allowable capacity from dynamic formulae are compared with allowable capacity from load tests in Table 5.15.

Allowable capacity from the dynamic formulae were predicted from Engineering News Records formula, Janbu formula and Hiley formula using the recommended

Table 5.15 Comparison of Predicted Allowable Capacity using Dynamic Formulae with Allowable Capacity from Load Tests(Ahmedbagh Site and Moghbazar Site)

Site	Pile No.	Measured allowable capacity [BNBC(1993)] (Tons)	Predicted allowable capacity using dynamic formulae (Tons)		
			Engg. News Records formula [Wellington (1888)] (FS=6)	Janbo formula [(1953)] (FS=4.5)	Hiley formula(1925) (FS=4)
Ahmedbagh	PP5	18.5	5.88	14	6.5
	PP6	19.75	7.08	16.22	6.25
MoghBazar	PP7	17.25	4.88	12.66	8.5
	PP8	18.45	5	8.66	6.5

factors of safety.For determining the allowable capacity from dynamic formulae the recommended factor of safety is higher than the recommended factor of safety in static methods. The recommended factor of safety for ENR formula,Janbu formula and Hiley formula are 6, 3 to 6 and 4 respectively. The predicted allowable capacity are presented in Table 5.15.The predicted allowable capacity from dynamic formula are also compared with the allowable capacity from load test in Table 5.15.It is observed that the allowable capacity predicted by ENR formula,Janbu formula and Hiley formulae using recommended factors of safety underestimate the allowable capacity of the pile The allowable capacity by Janbu formula is reasonably less than the allowable capacity determined from load test . The predicted allowable capacity using ENR formula and Hiley formula is less than about half of the allowable capacity determined from pile load test.

5.3.3 Settlement At Allowable Capacity

Settlements at the allowable loads from pile load test are shown in Table 5.13 along with the allowable loads.The settlements at the allowable loads vary from 0.4mm to 1.00mm. for pile PP5 and PP6 and 0.5mm to 0.70mm for pile PP7 and PP8. The observed settlements at allowable capacity are very small for all piles. Normally shallow foundations are designed for an allowable settlement of 25mm.

5.4 CAPACITY OF PILE DRIVEN THROUGH PLASTIC SILT AND RESTING ON NONPLASTIC SILT

Two pile load tests were performed at Novody site. Soil condition surrounding the piles are shown in Fig.A-3 and A-4. In this site soft fills exists upto the depth of 1.50m underlain by soft to medium stiff plastic silt. The underlying layer consists of nonplastic silt. Hence, pile capacity is attained from skin friction as well as end bearing.

5.4.1 Ultimate Capacity

Ultimate Capacity From Load Test

Maintained pile load test to failure was performed according to ASTM D-1143 .The load was applied in increment of 2 tons.The pile load test data are shown in Table C-3 and Table C-4.At the pick load the rate of settlement continued undiminished without further increment of load .

The load settlement curves of the piles are drawn in Fig. D-3 and D-4

The ultimate capacity was evaluated by :

(I) Terzaghi's method (1942),

(ii) IS : 2911-1979

(iii) BS 8004 : 1986. and

(iv) Davisson's method (1973)

The procedure of determining ultimate load according to Davisson (1973) is shown in Fig 5.5 (A) and 5.5 (B)

The ultimate capacities are presented in Table.5.16. The ultimate capacity evaluated by Terzaghi's method (1943), IS : 2911-1979 [Indian Standards Institution(1979)] and BS :8004-1986 are equal. The ultimate capacity determined by Davisson's method [Davisson (1973)] is lower than that by other three methods. The ultimate capacity of the two piles of each site are almost equal.

Ultimate* Capacity *From Static Methods

Ultimate capacity was predicted based on soil parameters obtained from geotechnical investigation. Skin friction was calculated using :

(i) α method [Tomlinson (1971)],

(ii) λ method [Vijayvergiya & Focht (1972)]

(iii) α_2 method [Peck et al (1976)],

(iv) API method [API (1984)]

(v) method recommended by Canadian Geotechnical Society (1985) and

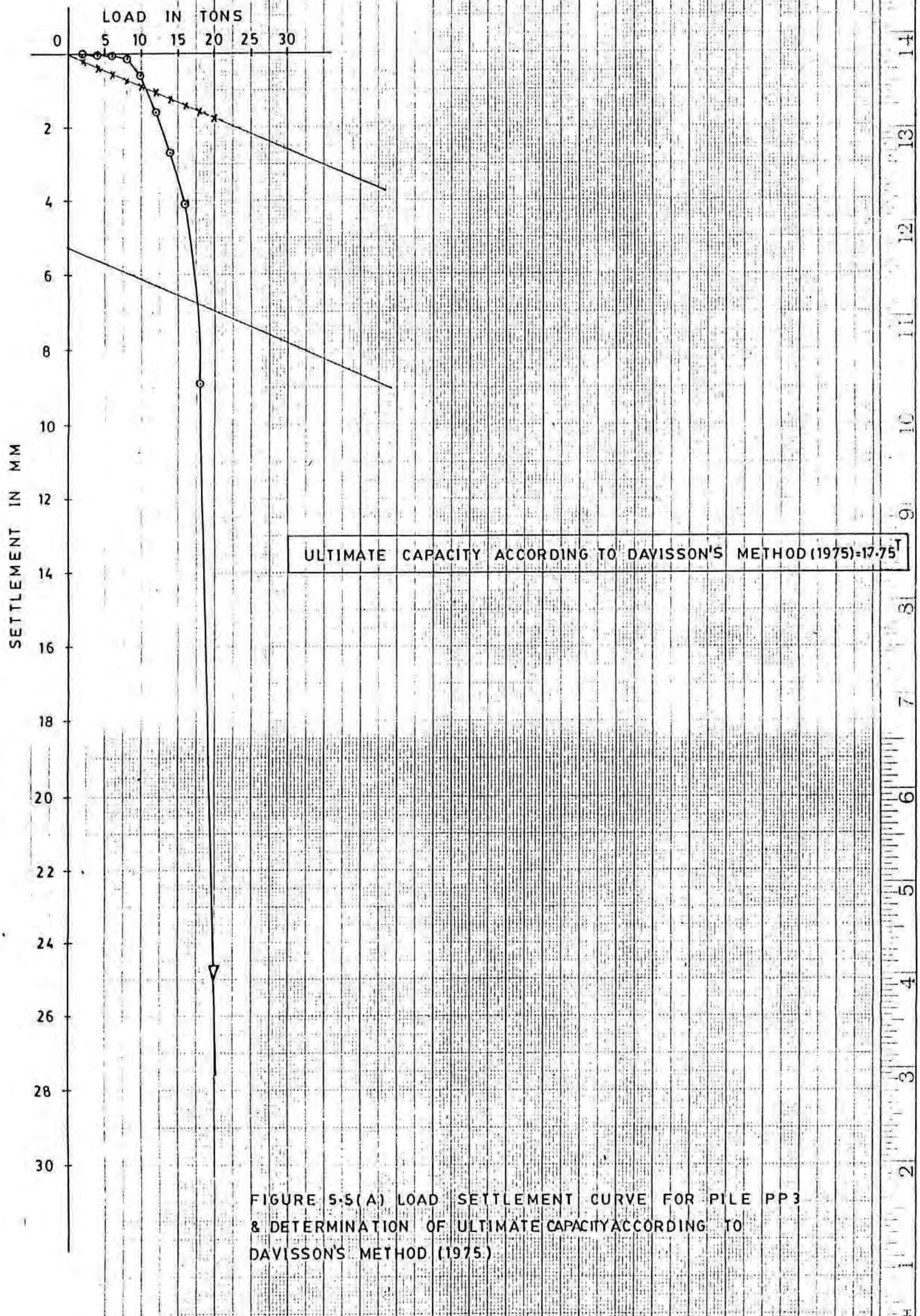


FIGURE 5.5(A) LOAD SETTLEMENT CURVE FOR PILE PP3 & DETERMINATION OF ULTIMATE CAPACITY ACCORDING TO DAVISSON'S METHOD (1975)

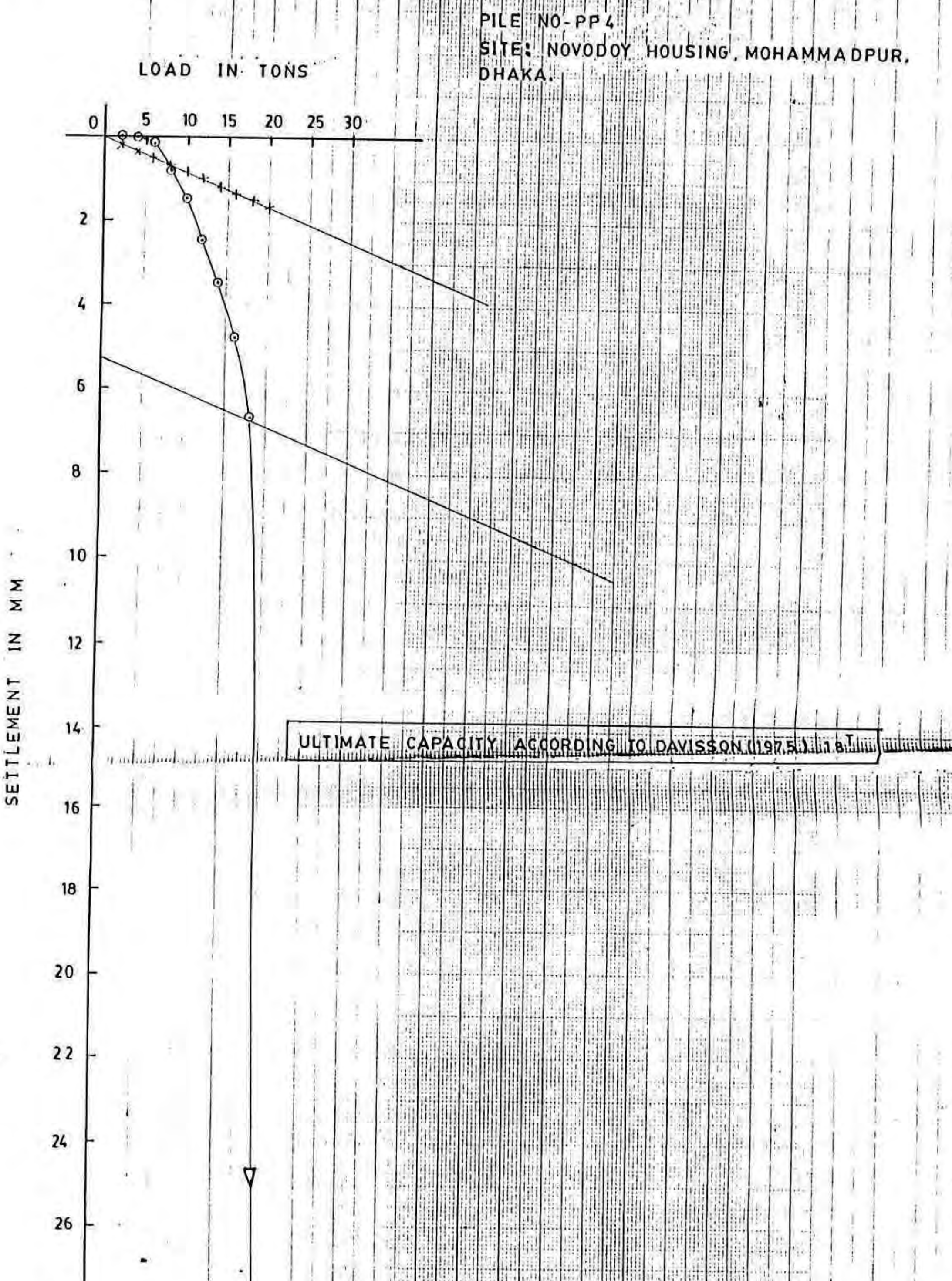


FIGURE 5.5(B) LOAD SETTLEMENT CURVES FOR PILE PP4 & DETERMINATION OF ULTIMATE CAPACITY OF PILE ACCORDING TO DAVISSON'S METHOD (1975).

Table 5.16 Measured Ultimate Capacity from Pile Load Test using Different Methods (Novodoy Housing Site)

Site	Pile No.	Method	Criteria for determining ultimate capacity from pile load test	Measured ultimate capacity (Tons)
Novodoy Housing (Piles through plastic silt and resting on nonplastic silt)	PP3	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	18.75
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base dia for underreamed pile b) Load corresponding to settlement 12mm.	18.75
		BS: 8004-1986	Load corresponding to settlement 10% of pile diameter	18.75
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	17.5
	PP4	Terzaghi's method[Terzaghi (1942)]	Load at which settlement is 10% of pile diameter	19.5
		IS:2911-1979	Smaller of the two : a)Load corresponding to 10% of pile diameter for normal uniform pile or 7.5% of base dia for underreamed pile b) Load corresponding to settlement 12mm.	19.5
		BS: 8004-1986	Load corresponding to settlement 10% of pile diamete	19.5
		Davission's method (1973)	A line drawn parallel to the elastic deflection line cuts the load settlement curve to give ultimate capacity	18

(vi) method recommended by Indian Standards Institution (IS:2911-1979).

End bearing was calculated using :

(I) Terzaghi's Method [Terzaghi (1943),Terzaghi and Peck(1967)]

(ii) Meyerhof's empirical method [Meyerhof (1956,1976)]

(iii) Hansen's method [Hansen (1970)]

(iv) API method (1984,1987)

(v) method recommended by Canadian Geotechnical Society(1985)

(vi) method recommended by Indian Standards Institution (IS : 2911-1979)

The value of angle of internal friction ϕ required for the determination of N_q was determined from the correlation of ϕ and SPT blow count N given by Peck et al (1974) shown in Fig. 2.16 .

It may be mentioned that the skin friction of the thin layer of nonplastic silt near the bottom of pile was determined using Tomlinson's method (1986)

The total ultimate predicted capacity of a pile is the sum of the skin friction along the shaft of the pile and the end bearing at the pile tip .To predict the total ultimate capacities of the piles the following combinations of skin friction and end bearing were used :

Combination-1 : α method [Tomlinson (1971)] for skin friction + Terzaghi's Method [Terzaghi (1943),Terzaghi and Peck(1967)] for end bearing

Combination -2 : α method [Tomlinson (1971)] for skin friction + Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing

Combination -3 : α method [Tomlinson (1971)] for skin friction + Hansen's method [Hansen (1970)] for end bearing

Combination -4 : λ method [Vijayvergiya & Focht (1972)] for skin friction + Terzaghi's method [Terzaghi (1943),Terzaghi and Peck(1967)] for end bearing

Combination -5 : λ method [Vijayvergiya & Focht (1972)] for skin friction + Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing

Combination -6 : λ method [Vijayvergiya & Focht (1972)] for skin friction + Hansen's method [Hansen (1970)] for end bearing

Combination -7 : α_2 method [Peck et al (1976)] for skin friction + Terzaghi's Method [Terzaghi (1943),Terzaghi and Peck(1967)] for end bearing

Combination -8 : α_2 method [Peck et al (1976)] for skin friction + Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing

Combination -9 : α_2 method [Peck et al (1976)] for skin friction + Hansen's method [Hansen (1970)] for end bearing

The ultimate capacity of piles were also predicted using the recommendations of the following methods:

Method-1 : API method (1984,1987)

Method-2 : method recommended by Canadian Geotechnical Society (1985)

Method-3 : method recommended by Indian Standards Institution (IS : 2911-1979).

The ultimate capacity evaluated by static methods are presented in Table 5.17

The predicted values of ultimate capacity are compared with the measured values of ultimate capacity in Table-5.18. The ratio between the predicted ultimate capacity using static methods and measured ultimate capacity from load test is also shown inTable 5.18 (A)

It is observed that the ultimate capacity obtained from the combination of λ method [Vijayvergiya & Focht (1972)] and Meyerhof's empirical method [Meyerhof (1956,1976)] or λ method [Vijayvergiya & Focht (1972)] and Hansen's method [Hansen (1970)] overestimate the ultimate capacity from load tests by a large amount.. α method [Tomlinson (1971)], α_2 method [Peck et al (1976)] in combinations with Meyerhof's empirical method [Meyerhof (1956,1976)] or Hansen's method [Hansen (1970) also overestimate the measured ultimate capacity determined from load tests. The method recommended by Canadian

Table 5.17 Predicted Ultimate Capacity using Static Methods (Piles ThroughPlastic Silt and resting on nonplastic Silt)

Site	Pile No.	Predicted ultimate capacity using static methods (Tons)								
		Combination -1			Combination-2			Combination --3		
		Skin friction α method [Tomlinson (1971)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing Meyerhof's emperical method Meyerhof(1976)	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing Hansen's method Hansen(1970)	Total capacity
Novodoy	PP3	11.87	5.92	17.79	11.87	8.26	20.13	11.87	8.82	20.69
Housing	PP4	13.87	5.92	19.79	13.87	8.26	22.13	13.87	8.82	22.69

Table 5.17 Cont

Site	Pile No.	Predicted ultimate capacity using static methods(Tons)								
		Combination -4			Combination-5			Combination --6		
		Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing Meyerhof's emperical method Meyerhof(1976)	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing Hansen's method Hansen (1970)	Total capacity
Novodoy	PP3	17.01	5.92	22.93	17.01	8.26	25.27	17.01	8.82	25.83
Housing	PP4	18.55	5.92	24.47	18.55	8.26	26.81	18.55	8.82	27.37

Table 5.17 Cont.

Site	Pile No.	Predicted ultimate capacity using static methods (Tons)								
		Combination -7			Combination-8			Combination --9		
		Skin friction α_2 method [Peck et al (1974)]	End bearing [Terzaghi(1943)]	Total capacity	Skin friction α_2 method [Peck et al (1974)]	End bearing [Meyerhof's emperical method Meyerhof(1976)]	Total capacity	Skin friction α_2 method [Peck et al (1974)]	End bearing [Hansen's method Hansen (1970)]	Total capacity
Novodoy	PP3	13.72	5.92	19.64	13.72	8.26	21.98	13.72	8.82	22.54
Housing	PP4	15.7	5.92	21.62	15.7	8.26	23.96	15.7	8.82	24.52

Table 5.17 Cont.

Site	Pile No.	Predicted ultimate capacity using static methods (Tons)								
		Method-1 API(1984,1987)			Method-2[Method recommended by Canadian Geotechnical Society(1985)]			Method-3[Method recommended by Indian Standards Institution(IS:2911-1979)]		
		Skin friction	End bearing	Total capacity	Skin friction	End bearing	Total capacity	Skin friction	End bearing	Total capacity
Novodoy	PP3	13.68	4.18	17.86	14.42	9.24	23.66	12.30	4.89	17.19
Housing	PP4	14.31	4.18	18.49	16.62	9.24	25.86	13.11	4.89	18

Table 5.18 Comparison of Predicted Ultimate Capacity using Static Methods with Measured Ultimate Capacity from Load Test

Site	Pile No.	Measured ultimate capacity (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Combination -1			Combination-2			Combination --3		
			Skin friction α method [Tomlinson (1971)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing [Meyerhof's emperical method Meyerhof(1976)]	Total capacity	Skin friction α method [Tomlinson (1971)]	End bearing [Hansen's method Hansen (1970)]	Total capacity
Novodoy Housing	PP3	18.75	11.87	5.92	17.79	11.87	8.26	20.13	11.87	8.82	20.69
	PP4	19.5	13.87	5.92	19.79	13.87	8.26	22.13	13.87	8.82	22.69

Table 5.18 Contd..

Site	Pile No.	Measured ultimate capacity (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Combination -4			Combination-5			Combination --6		
			Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing [Meyerhof's emperical method Meyerhof(1976)]	Total capacity	Skin friction λ method [Vijayvergiya & Focht (1972)]	End bearing Hansen's method Hansen (1970)	Total capacity
Novodoy Housing	PP3	18.75	17.01	5.92	22.93	17.01	8.26	25.27	17.01	8.82	25.83
	PP4	19.5	18.55	5.92	24.47	18.55	8.26	26.81	18.55	8.82	27.37

Table 5.18 Cont.

Site	Pile No.	Measured ultimate capacity (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Combination -7			Combination-8			Combination --9		
			Skin friction α_2 method [Peck et al (1974)]	End bearing [Terzaghi (1943)]	Total capacity	Skin friction α_2 method [Peck et al (1974)]	End bearing [Meyerhof's emperical method Meyerhof(1976)]	Total capacity	Skin friction α_2 method [Peck et al (1974)]	End bearing [Hansen's method Hansen (1970)]	Total capacity
Novodoy Housing	PP1	18.75	13.72	5.92	19.64	13.72	8.26	21.98	13.72	8.82	22.54
	PP2	19.5	15.7	5.92	21.62	15.7	8.26	23.96	15.7	8.82	24.52

Table 5.18 Cont.

Site	Pile No.	Measured ultimate capacity (Tons)	Predicted ultimate capacity using static methods (Tons)								
			Method-1 API(1984,1987)			Method-2[Method recommended by Canadian Geotechnical Society(1985)]			Method-3[Method recommended by Indian Standards Institution(IS:2911-1979)]		
			Skin friction	End bearing	Total capacity	Skin friction	End bearing	Total capacity	Skin friction	End bearing	Total capacity
Novodoy	PP3	18.75	13.68	4.18	17.86	14.42	9.24	23.66	12.3	4.89	17.19
	PP4	19.5	14.31	4.18	18.49	16.62	9.24	25.86	13.11	4.89	18

Table 5.18 (A) Ratio between Predicted Ultimate Capacity and Measured Ultimate Capacity from Load Test (Novodoy Housing Site)

Site	Pile No.	Measured ultimate capacity from load test (Tons)	Predicted ultimate capacity from static methods		Predicted ultimate capacity / Measured ultimate capacity
			Method of prediction (static methods)	Predicted ultimate capacity (Tons)	
Novodoy Housing (Piles through plastic Silt and resting on nonplastic Silt)	PP 3	18.75	α method [Tomlinson (1971)] for skin friction & Terzaghi's Method [Terzaghi (1943)] method for end bearing	17.79	0.95
			α method [Tomlinson (1971)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	20.13	1.o7
			α method [Tomlinson (1971)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	20.69	1.10
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Terzaghi's Method [Terzaghi (1943) for end bearing	22.93	1.22
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	25.27	1.34
			λ method [Vijayvergiya & Focht(1972)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	25.83	1.37
			α_2 method [Peck et al (1974)] for skin friction & Terzaghi's Method [Terzaghi (1943)] for end bearing	19.64	1.04
			α_2 method [Peck et al (1974)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	21.98	1.17
			α_2 method [Peck et al (1974)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	22.54	1.20
			API (1984) method	17.86	0.95
			Method recommended by Canadian Geotechnical Society(1985)	23.66	1.26
			Method recommended by Indian Standards Institution(IS:2911-1979)	17.19	0.91

Table 5.18 (A) Cont.

Site	Pile No.	Measured ultimate capacity from load test(Tons)	Predicted ultimate capacity from static methods,(Tons)		Predicted ultimate capacity / Measured ultimate capacity
			Method of Prediction (Static Methods)	Predicted Ultimate Capacity	
Novodoy Housing (Piles throughPlastic Silt and resting on nonplastic Silt)	PP4	19.5	α method [Tomlinson (1971)] for skin friction & Terzaghi's Method [Terzaghi (1943)] method for end bearing	19.79	1.01
			α method [Tomlinson (1971)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	22.13	1.13
			α method [Tomlinson (1971)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	22.69	1.16
			λ method [Vijayvergiya & Focht (1972)] for skin friction & Terzaghi's Method [Terzaghi (1943) for end bearing	24.47	1.25
			λ method [Vijayvergiya & Focht (1972)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	26.81	1.37
			λ method [Vijayvergiya & Focht (1972)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	27.37	1.40
			α_2 method [Peck et at (1974)] for skin friction & Terzaghi's Method [Terzaghi (1943)] for end bearing	21.62	1.10
			α_2 method [Peck et al (1974)] for skin friction & Meyerhof's empirical method [Meyerhof (1956,1976)] for end bearing	23.96	1.22
			α_2 method [Peck et at (1974)] for skin friction & Hansen's method [Hansen (1970)] for end bearing	24.52	1.25
			API (1984) method	18.49	0.94
			Method recommended by Canadian Geotechnical Society (1985)	25.86	1.32
			Method recommended by Indian Standard Institution(IS:2911-1979)	18	0.92

Geotechnical Society (1985) also overestimate the measured ultimate capacity determined from load tests. The ultimate capacity obtained from the combination of α method [Tomlinson (1971)] or α_2 method [Peck et alt (1976)] with Terzaghi's method [Terzaghi (1943),Terzaghi and Peck (1967)] and, method recommended by Indian Standards Institution (IS : 2911-1979) and API (1984,1987) method is close to the ultimate capacity determined from load test.

Ultimate* Capacity *From Dynamic Methods

The driving records of the piles are produced in Table B-2 in Appendix-B.

The ultimate capacity of the piles was predicted using :

(i) Engineering News Records formula (ENR) [Wellington (1888)]

(ii) Hiley formula. (1925) and

(iii) Janbu formula (1953)

The ultimate capacity predicted by dynamic formulae are presented in Table 5.19.These values are also compared with the ultimate capacity of piles determined from load test in Table-5.19

It is observed that ultimate capacity predicted by ENR [Wellington (1888)], Hiley formula. (1925) and Janbu formula (1953) is close to the measured ultimate capacity from load test.

Table 5.19 Comparison Of Predicted Ultimate Capacity using Dynamic Formulae with the Ultimate Capacity from Load Tests (Novodoy Housing Site)

Site	Pile No.	Predicted ultimate capacity using dynamic formulae (Tons)			Measured ultimate capacity from load test (Tons)
		Engg. News Records formula [Wellington (1888)]	Janbo formula (1953)	Hiley Formula (1925)	
Novodoy Housing	PP3	17	22	19	18.75
	PP4	19	22	16	19.5

5.4.2 Allowable capacity

Allowable* Capacity *From Load Test

Allowable capacity was determined from pile load test using:

(I) Bangladesh National Building Code (1993)

According to BNBC (1993) the allowable capacity of a pile is half of that test load which produces a permanent net settlement of not more than 0.00028 mm/kg of test load or 20 mm

ii) Indian Standard Code of Practice (I S 2911-1979)

According to I S 2911-1979 the allowable capacity of a pile is least of the following:

(a) Two thirds of the final load at which the total settlement attains a alue of 12 mm

(b) Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameterin the case of under -reamed pile.

(iii) BSI-CP-2004-1972

According to BSI CP-2004-1972, the allowable capacity of a pile should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter.

The evaluated allowable capacity determined from pile load test are presented in Table 5.20.

It may be noted that the criteria for determining allowable capacity from load test is different for Bangladesh National Building Code (1993), IS : 2911-1979 and BSI-CP.-2004 : 1972. However, the allowable capacity determined from load test are equal.The recommended allowable capacity of a pile from pile load test by the above mentioned methods are based on the settlement criteria.

Allowable* Capacity *From Static Analysis

The allowable capacity of the piles were calculated from static methods using the recommended factors of safety.The allowable capacities determined from static methods are presented in Table 5.21 The allowable capacity from static methods are also compared with the allowable capacity from load tests in Table 5.21

Table 5.20 Allowable Capacity from Pile Load Test using Various Codes and Corresponding Settlement (Novodoy Housing Site)

Site	Pile No.	Method	Criteria for determining allowable capacity from pile load test	Allowable capacity (Tons)	Settlement at allowable capacity
Novodoy	PP3	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.0028 mm/kg of test load or 20 mm.	9.34	0.4mm
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of under -reamed pile.	9.34	0.4mm
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	9.34	0.4mm
	PP4	BNBC (1993)	The allowable pile capacity shall not be more than one half of that test load which produces a permanent net settlement of not more than 0.0028 mm/kg of test load or 20 mm.	9.75	1.5mm
		IS: 2911-1979	The allowable capacity is least of the following: (a)Two thirds of the final load at which the total settlement attains a value of 12 mm. (b)Half of the final load at which total settlement equals to 10% of the pile diameter in the case of normal uniform diameter pile and 7.5% of base diameter in the case of under -reamed pile.	9.75	1.5mm
		BSI: CP 2004-1972	The allowable pile capacity should be 50% of the final load which causes the pile to settle a depth of 10% of pile width or diameter	9.75	1.5mm

Table 5.21 Comparison of Measured Allowable Capacity Load from Load Tests with Predicted Allowable Capacity from Static Analysis (Novodoy Housing Site)

Site	Pile No.	Measured allowable capacity from load test (Tons)	Predicted allowable capacity using static methods (Tons)					
			Combination-1 (Skin friction by α method & end bearing by Terzaghi method)	Combination -2 (Skin friction by α method & end bearing by Meherhof's emperical method)	Combination -3 (Skin friction by α method & end bearing by Hansen's method)	Combination 4 (Skin friction by λ method & end bearing by Terzaghi method)	Combination -5 (Skin friction by λ method & end bearing by Meyerhof's emperical method)	Combination 6 (Skin friction by λ method & end bearing by Hansen's method)
Novodoy	PP3	9.37	7.11	8.05	8.27	9.17	10.10	10.33
Housing	PP4	9.75	7.91	8.85	9.07	9.78	10.72	10.94

Table 5.21 Cont

Site	Pile No.	Measured allowable capacity from load test (Tons)	Predicted allowable capacity using static methods (Tons)					
			Combination 7 (Skin friction by α_2 method & end bearing by Terzaghi method)	Combination -8 (Skin friction by α_2 method & end bearing by Meyerhof's emperical method)	Combination -9 (Skin friction by α_2 method & end bearing by Hansen's method)	Method-1 (API method)	Method-2 (Method recommended by Canadian Geotechnical Society)	Method-3 (Method rcommended by Indian Standards Institution
Novodoy	PP3	9.37	7.85	8.79	9.01	7.14	9.46	6.87
Housing	PP4	9.75	8.64	9.58	9.80	7.39	10.34	7.2

It is observed that the allowable capacity predicted from the combination of λ method [Vijayvergiya & Focht (1972)] and Meyerhof's empirical method [Meyerhof (1956,1976)] or λ method [Vijayvergiya & Focht (1972)] and Hansen's method [Hansen (1970) slightly overestimate the allowable capacity determined from load test. Method recommended by Canadian Geotechnical Society (1985) also overestimate the allowable capacity from load test However,the allowable capacity predicted from α method [Tomlinson (1971)] α_2 method [Peck et al (1976)] in combinations with Terzaghi's method [Terzaghi (1943),Terzaghi and Peck(1967)], Meyerhof's empirical method [Meyerhof (1956,1976)] or Hansen's method [Hansen (1970) slightly underestimate the allowable pile capacity determined from load test. Method suggested by Indian Standards Institution (IS:2911-1979) and API (1984,1987) method underestimate the allowable pile capacity.

Allowable Capacity from dynamic formulae

Allowable loads from the dynamic formulae were predicted by ENR(Wellington (1888), Hiley formula. (1925) and Janbu formula (1953) using the recommended factors of safety.For determining the allowable capacity from dynamic formulae the recommended factor of safety is higher than the recommended factor of safety in static methods. The recommended factor of safety for ENR formula,Janbu formula and Hiley formula are 6, 3 to 6 and 4 respectively. The predicted allowable capacity are presented in Table 5.22.The predicted allowable capacity from dynamic formula are also compared with the allowable capacity from load test in

Table 5.22 Comparison of Predicted Allowable Capacity using Dynamic Formulae with Allowable Capacity from Load Tests (Novodoy Housing Site)

Site	Pile No.	Predicted Allowable Capacity Using Dynamic Formulae (Tons)			Allowable capacity from Load Tests [BNBC(1993)] (Tons)
		Engg. News Records formula [Wellington (1888) (FS=6)	Janbo formula [(1953)] (FS=4.5)	Hiley formula (1925) (FS=4)	
Novodoy Housing	PP3	5.33	10.55	5.67	9.37
	PP4	5.66	12.22	5.67	9.75

Table-5.22. It is observed that the allowable capacity predicted by Janbu formula, ENR formula and Hiley formula is less than about half of the allowable capacity determined from pile load test.

5.4.3 Settlements At Allowable Capacity

From the load settlement curves it is observed that the settlement at the allowable load is 0.40 mm in the case of PP3 and 1.50mm in the case of PP4. Thus,at the allowable load, settlement is very small. Settlements at the allowable loads are shown in Table 5.20.

5.5 INTERPRETATION OF RESULTS

It is observed that the measured ultimate capacity from load test, in general, is greater than the predicted values from static analysis.However the pile load test is the most reliable method of determining carrying capacity of piles. Reasons for this variation may be due to compaction of soil surrounding and underlying the pile tip during driving of the pile and error in determining the ground water table.

Most of theories do not take into account the compaction of soil resulting from pile driving.The value of N_q in the equation of end bearing is determined from the estimated values of ϕ of the soil in its unaffected state prior to piling. Due to conmpaction of the soil the ϕ value of the soil would increase. Consequently pile capacity would also increase

Ground water level was measured 24 hours after the completion of boring work and was found at shallow depths. In the calculation of effective overburden pressure in static analysis, submerged unit weight of the soil was used below this ground water level. However, the actual ground water table in Dhaka city is much lower than that shown in borelogs. pressure The pile capacity.was smaller due to low value of the effective overburden pressure If actual ground water table were available the effective overburden pressure would have been more and, as such ,pile capacity would have been increased.The actual ground water level may be obtained by installing piezometers and observing the water level in frequent intervals of time for the whole year.

CHAPTER 6
CONCLUSIONS

Maintained load tests on eight piles of 175 mm square and length 7.5 m were performed at four locations of Dhaka city. All the load tests on pile were performed at sites where top soil upto 3m to 4.0 m was very weak and unsuitable for shallow foundation .The ultimate capacity of the pile depends on the soil condition surrounding the pile.The ultimate capacity of the piles were predicted using static methods and dynamic methods.The predicted ultimate capacity using static methods and the dynamic methods were compared with the measured ultimate capacity from pile load tests.

The following conclusions are drawn from this study:

i) The ultimate capacity of pile determined from load test using the criteria of Terzaghi's method (1942), IS : 2911-1979 , BS 8004 : 1986. and Davisson's method (1973) are almost equal.

ii) The measured ultimate capacity of piles driven through Dhaka Clay is in close agreement with the predicted values using λ method [Vijayvergiya & Focht (1972)] . The predicted ultimate capacity of pile using α method

[Tomlinson (1971)] and α_2 method [Peck et al (1974)] is slightly smaller than the measured value. α method [Tomlinson (1971)] and α_2 method [Peck et al (1974)] can safely be used for predicting the ultimate skin friction of Dhaka Clay.

iii) For friction pile in Dhaka Clay the reduction factor α_2 determined from pile load test is slightly higher than the average value recommended by Peck et al (1974).The reduction factor recommended by Peck et al (1974) can safely be used for Dhaka Clay.

iv) The ultimate unit skin friction of typical Dhaka Clay [LL=48% to 50 %, PL=20% to 21%, W=22% to 24%, SPT=9 to 16 and q_u =15 to 18 ton/m^2] estimated from pile load test is about 7.0 ton /m^2

v) The measured ultimate capacity of piles driven through Dhaka Clay and resting on medium dense sand is in close agreement with the predicted values using the combinations of λ method [Vijayvergiya & Focht (1972)] and Meyerhof's empirical method [Meyerhof (1956,1976)] , λ method [Vijayvergiya & Focht (1972)] and Hansen's method [Hansen(1970)]. The values of ultimate capacity predicted by the combinations of α method [Tomlinson (1971)] and Meyerhof's empirical method [Meyerhof (1956,1976)] ,α method [Tomlinson (1971)] and Hansen's method [Hansen(1970) , α_2 method [Peck et al (1974)] and Meyerhof's empirical method [Meyerhof (1956,1976)] ,α_2 method [Peck et al (1974)] and Hansen's method [Hansen(1970)] underestimate the measured value by a small margin .The ultimate predicted values using α method [Tomlinson (1971)] or α_2 method [Peck et al (1974)] with Terzaghi's method [Terzaghi (1943)] ,and method recommended by Canadian Geotechnical Society(1985) underestimate the ultimate pile capacity by a large extent. The value of ultimate capacity predicted by API(1984) method and IS method (IS : 2911-1979) is about half of the measured ultimate pile capacity determined from pile load test.

vi) The ultimate pile capacity predicted by using driving formulae such as Engineering News Records formula [Wellington (1888)], Janbu formula (1953) and Hiley formula. (1925), in general, overestimate the measured ultimate capacity determined from pile load test.However these formulae underestimate the allowable capacity when used with the recommended factors of safety. The recommended values of factors of safety for these formulae are large .Hence the driving formulae should be used with caution.

(viii) The settlement of small size piles driven in Dhaka city,at allowable load, is very small ranging from 0.3mm to 1.50 mm for these particular pile under investigation..

RECOMMENDATIONS FOR FUTURE STUDY

The present research has covered the different aspects of the axially loaded small size prestressed pile.It is recommended to extend this research in order to establish a complete picture of the behavior of this type of pile.This can be achieved by :

i) Studying the long term settlement behavior of pile resting on clay at working load
ii) Studying the group action of small size prestressed pile
iii) Studying the lateral capacity of small size prestressed pile
iv) Investigating the ultimate capacity of piles at other locations

APPENDICES

APPENDIX- A

BORELOGS

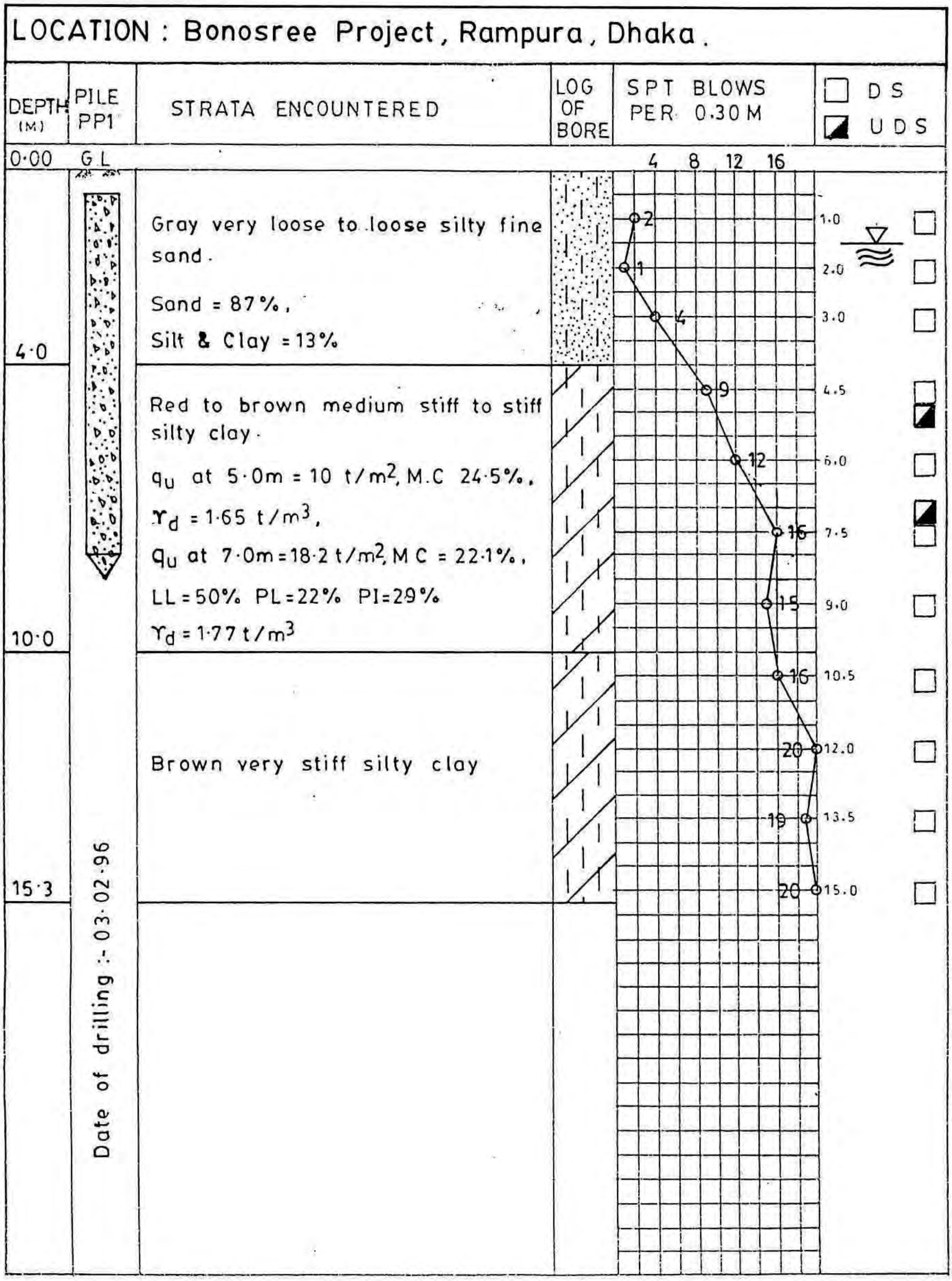


Figure A-1 Borelog of BH 1 at the location of pile PP1

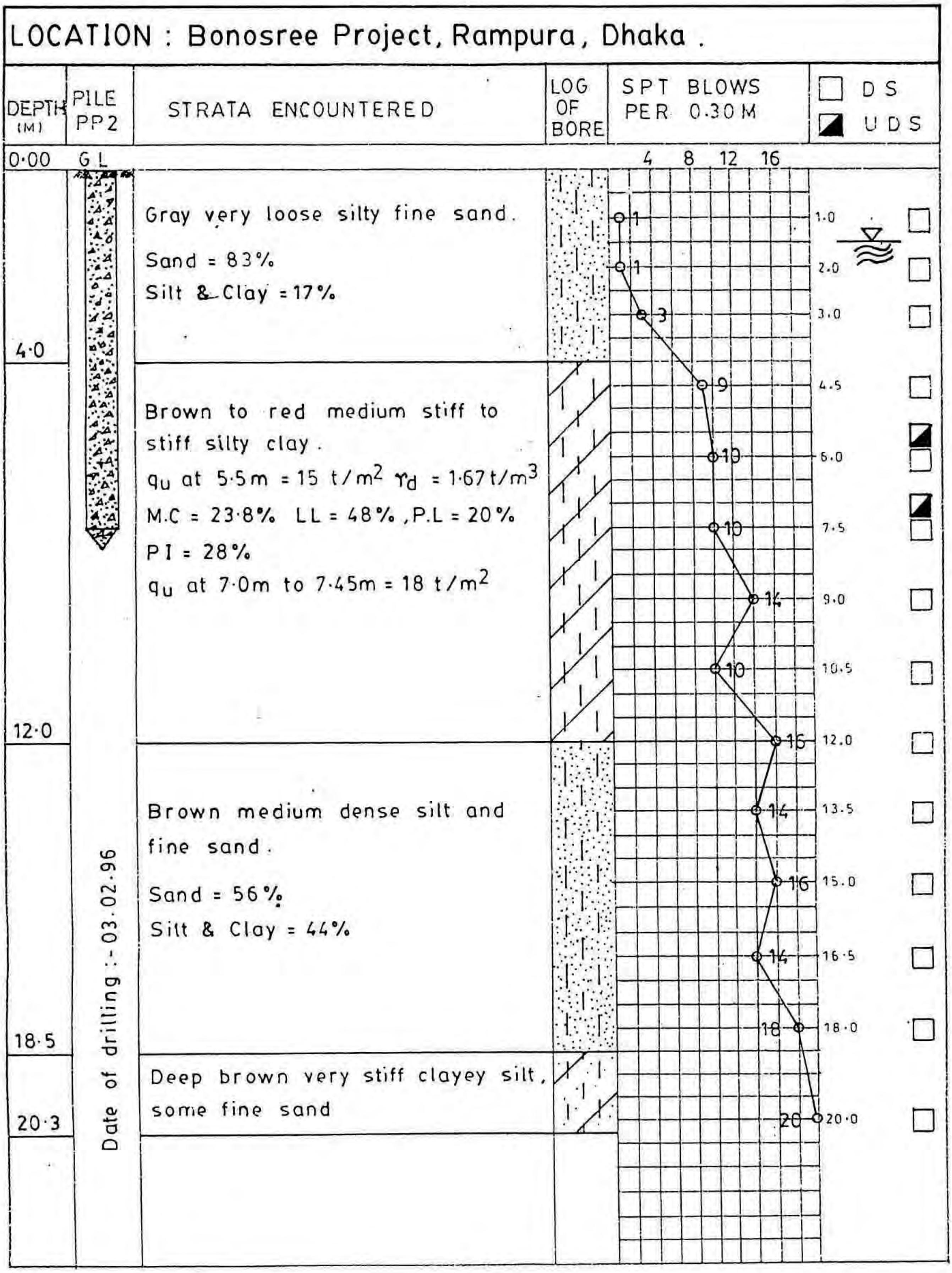


Figure A-2 Borelog of BH 2 at the location of pile PP2

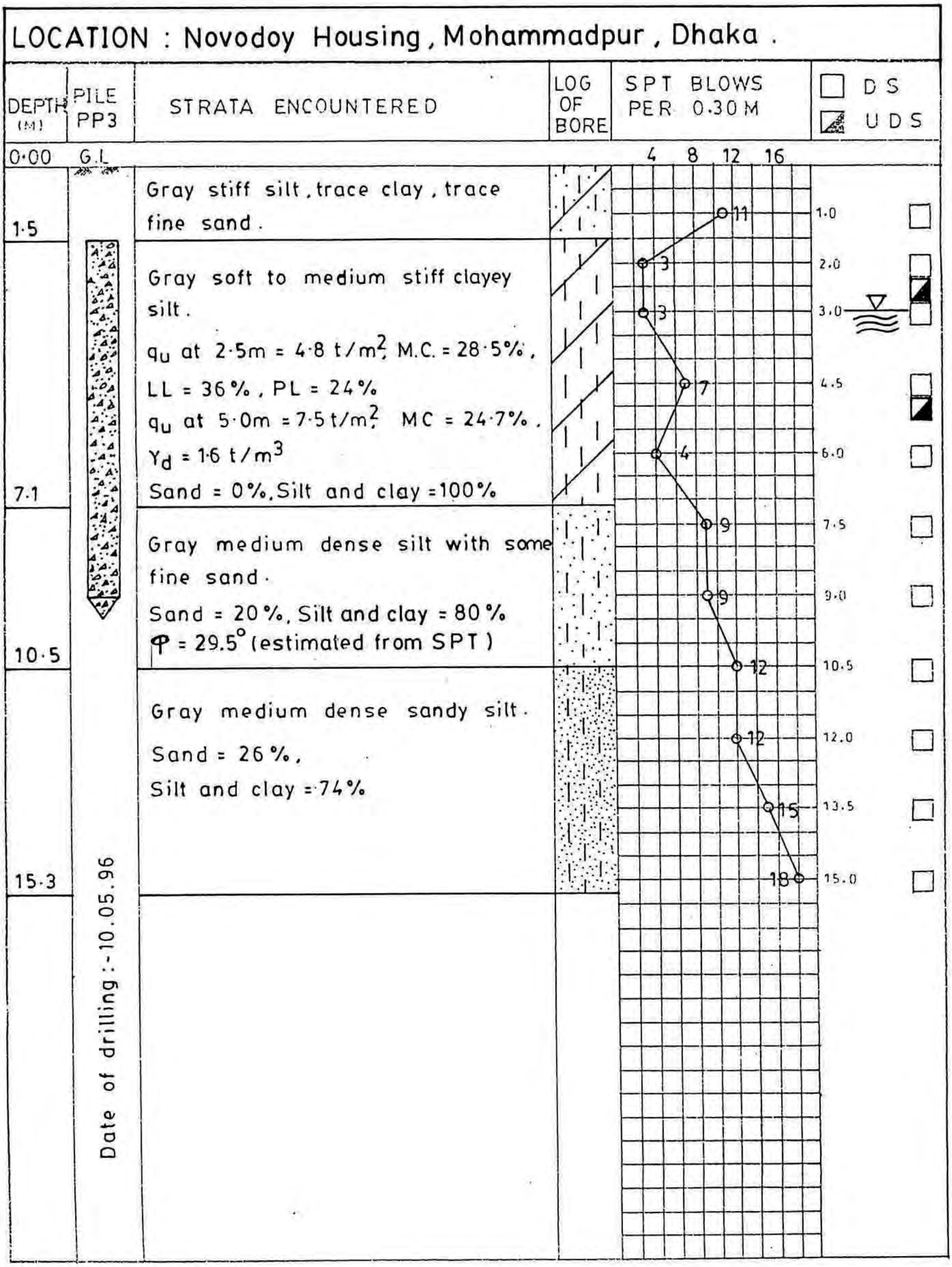


Figure A-3 Borelog of BH 3 at the location of pile PP3

LOCATION : Novodoy Housing, Mohammadpur, Dhaka.

DEPTH (M)	PILE PP4	STRATA ENCOUNTERED	SPT BLOWS PER 0.30 M
0.00	G.L		
2.0		Gray medium stiff silt, trace clay	1.0: 5; 2.0: 4
7.0		Gray medium stiff clayey silt. q_u at 2.5m = 7 t/m^2, NMC = 25.5%, γ_d = 1.59 t/m^3, LL = 40%, PL = 21%, PI = 19%, q_u at 5.0m = 8.3 t/m^2, M.C = 24.35%, γ_d = 1.62 t/m^3, LL = 38.5%, PL = 25%, PI = 13.5%.	3.0: 7; 4.5: 6; 6.0: 5
11.5		Gray medium dense silt with some fine sand, trace clay. Sand = 18%, Silt and clay = 82%, φ = 29.5° (estimated from SPT)	7.5: 7; 9.0: 9; 10.5: 9
15.3		Gray medium dense sandy silt. Sand = 28%, Silt and clay = 72%	12.0: 10; 13.5: 20; 15.0: 18

Legend: □ DS, ◪ UDS

Date of drilling :- 10 05 96

Figure A-4 Borelog of BH-4 at the location of pile PP4

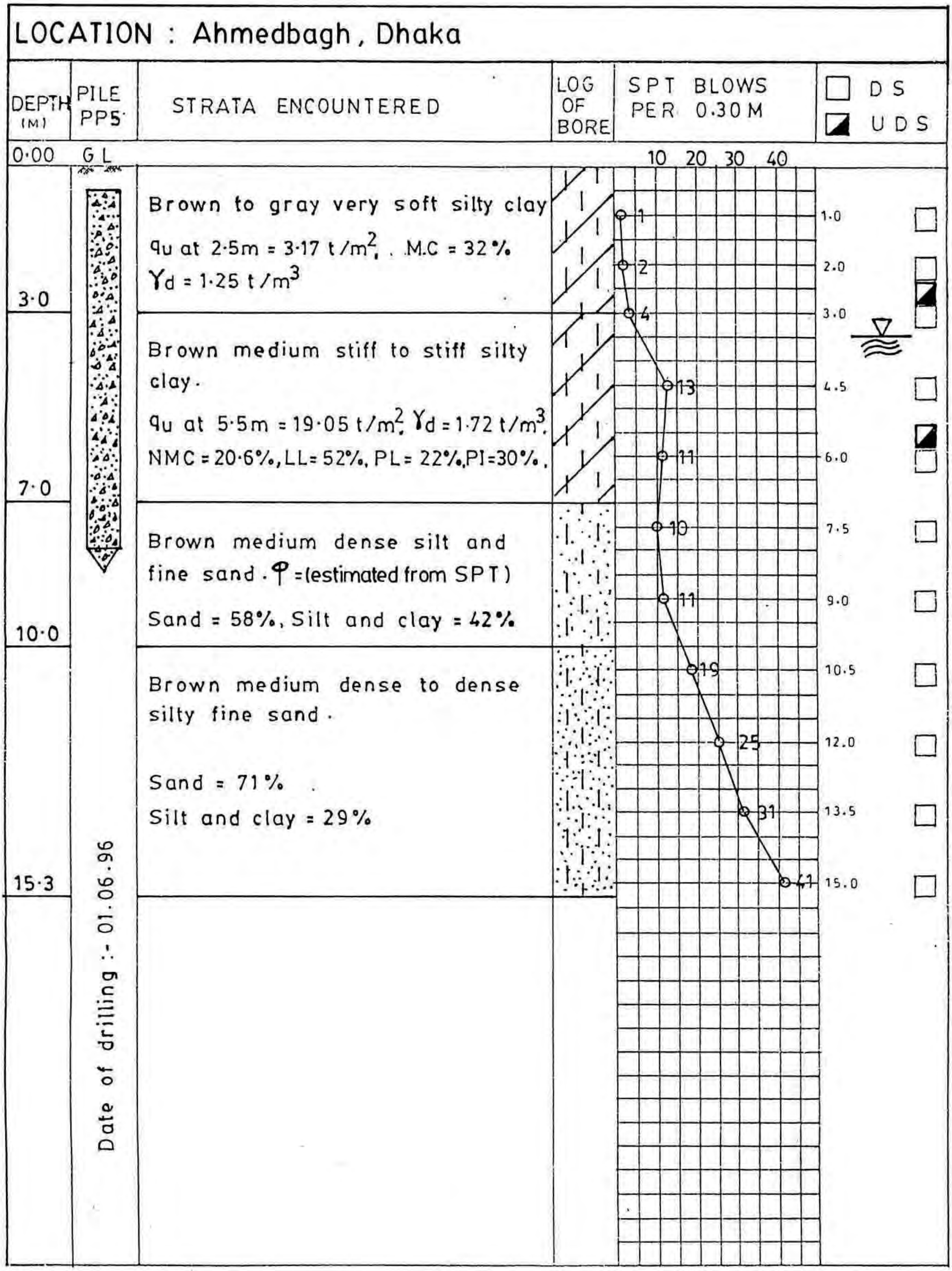


Figure A-5 Borelog of BH 5 at the location of pile PP5

LOCATION : Ahmedbagh , Dhaka ,

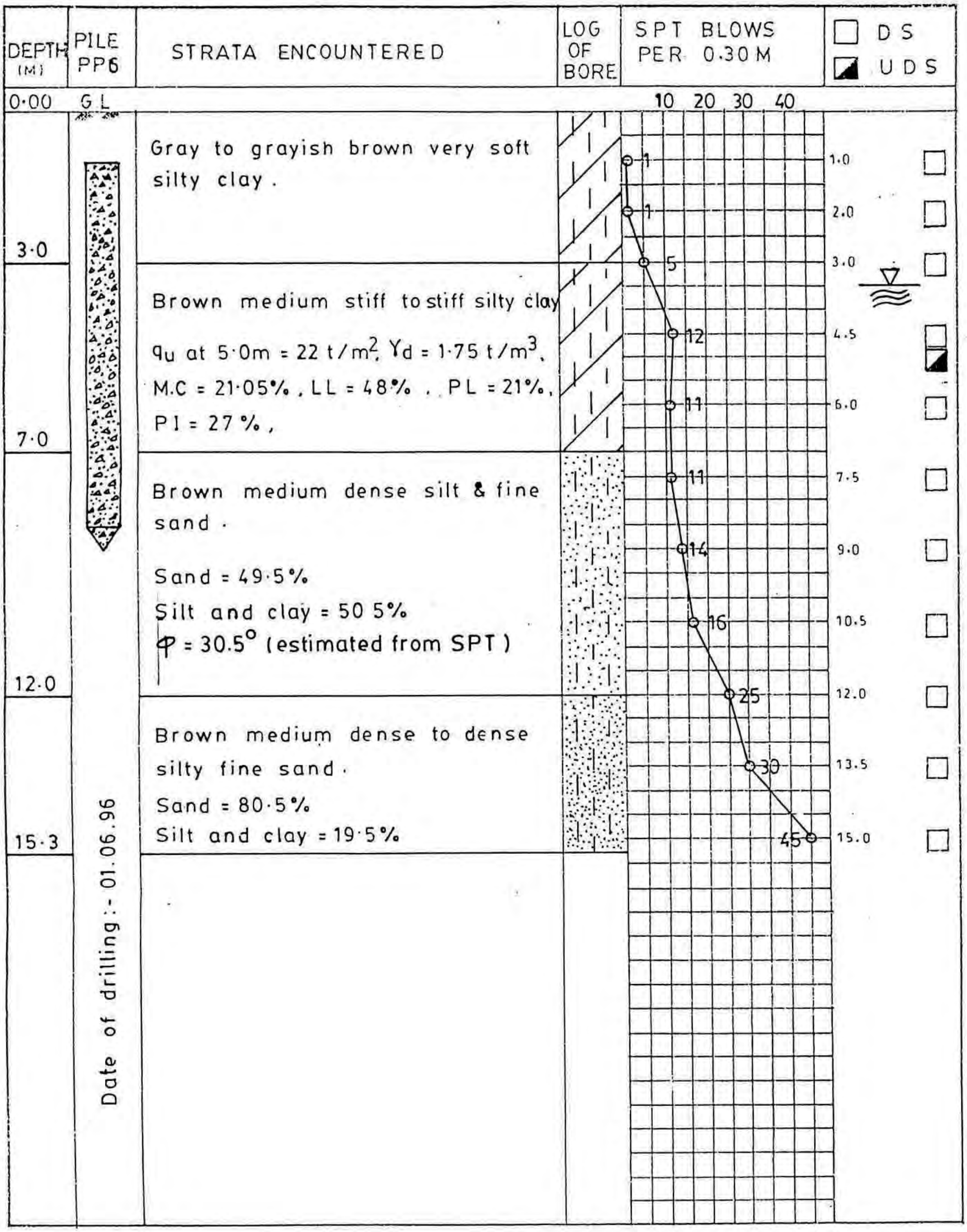


Figure A-6 Borelog of BH 6 at the location of pile PP6

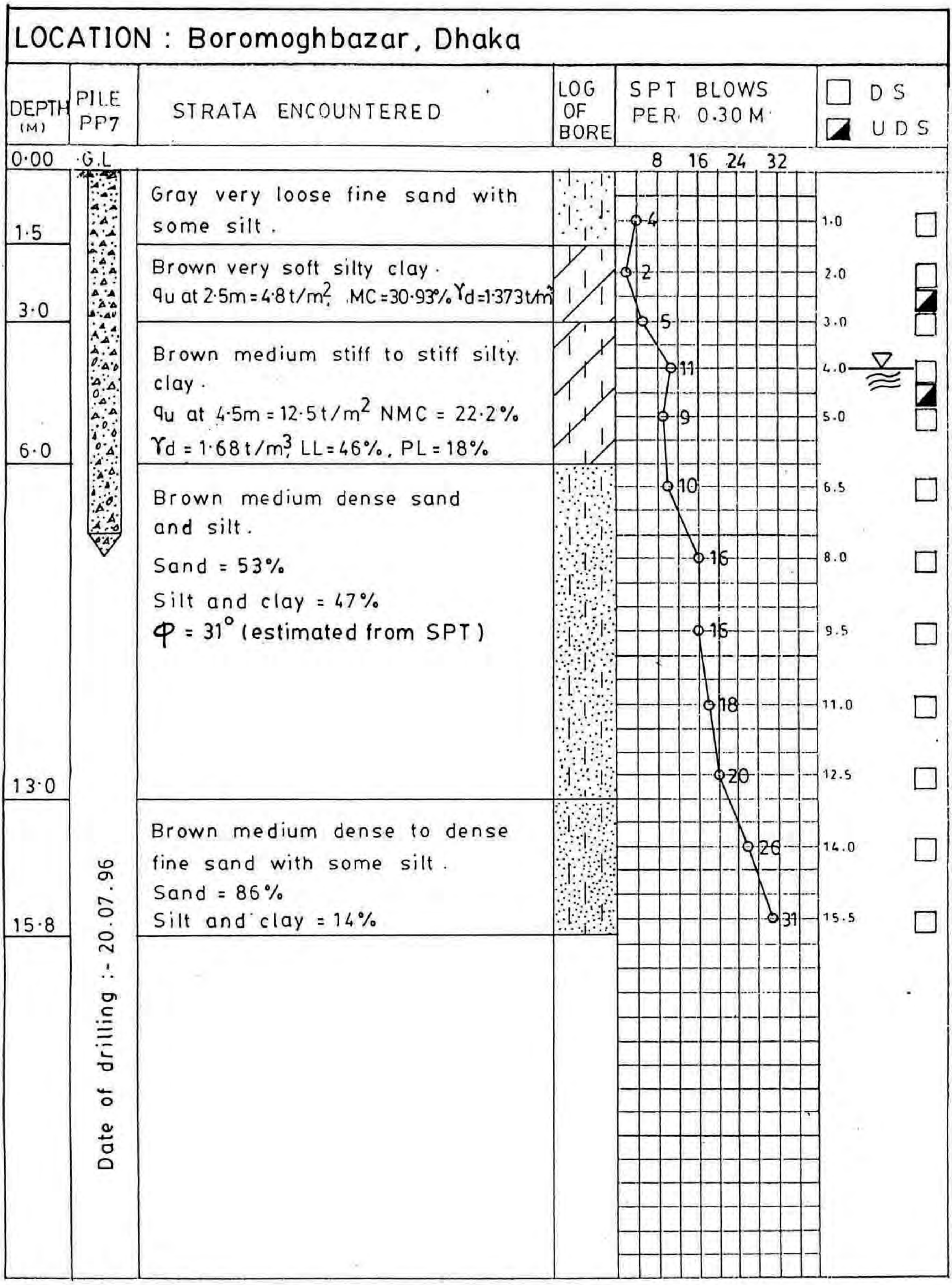


Figure A-7 Borelog of BH-7 at the location of pile PP7

LOCATION : Boromoghbazar, Dhaka

PILE PP8 · Log of bore · SPT blows per 0.30 M (scale 8, 16, 24, 32) · □ DS · ◩ UDS

DEPTH (M)	STRATA ENCOUNTERED	SPT BLOWS PER 0.30 M (depth, m : N)
0.00 G.L		
1.5	Gray very loose fine sand with some silt.	1.0 : 3
3.5	Gray to brown very soft to soft silty clay.	2.0 : 1; 3.0 : 4
6.87	Brown stiff silty clay. q_u at 5.5m = 11.8 t/m², MC = 22.8%, γ_d = 1.701 t/m³, LL = 49%, PL = 23%	4.0 : 9; 5.0 : 9; 6.5 : 9
12.5	Brown medium dense sand and silt. Sand = 51.5%, Silt and clay = 48.5%, φ = 32° (estimated from SPT)	8.0 : 16; 9.5 : 22; 11.0 : 26; 12.5 : 35
15.8	Light brown dense fine sand with some silt. Sand = 88%, Silt and clay = 12%	14.0 : 39; 15.5 : 40

Date of drilling :- 20.07.96

Figure A-8 Borelog of BH-8 at the location of pile PP8

APPENDIX- B

PILE DRIVING RECORDS

Table B-1 Pile Driving Records of PP1and PP2 (Bonosree Project Site)

Weight of Hammer	Pile Segment	Pile No. PP1		Pile No. PP2	
		Blows per 0.3m	Height of Drop,M	Blows per 0.3M	Height of Drop,M
	0.0-0.30m	-	0.3	-	0.3
	0.30-.60m	14	0.3	10	0.3
	0.60-0.90m	7	0.3	5	0.3
	0.90-1.20m	6	0.3	7	0.3
	1.20-1.50m	5	0.3	7	0.3
	1.50-1.80m	5	0.3	8	0.3
	1.80-2.10m	9	0.3	10	0.3
	2.10-2.40m	10	0.6	9	0.6
	2.40-2.70m	14	0.6	12	0.6
	2.70-3.00m	27	0.6	16	0.6
	3.00-3.30m	30	0.6	26	0.6
	3.30-3.60m	30	1.05	25	1.05
	3.60-3.90m	32	1.05	28	1.05
	3.90-4.20m	39	1.05	29	1.05
	4.20-4.50m	40	1.8	29	1.8
	4.50-4.80m	42	1.8	27	1.8
	4.80-5.10m	47	1.8	30	1.8
335 Kg	5.10-5.40m	40	2.5	30	2.5
	5.40-5.70m	50	2.5	26	2.5
	5.70-6.00m	52	2.5	28	2.5
	6.00-6.30m	54	2.5	37	2.5
	6.30-6.60m	60	2.89	51	3
	6.60-6.90m	60	2.89	63	3
	6.90-7.20m	62	2.89	64	3
	7.20-7.50m				

Table B-2 Pile Driving Records of PP3and PP4(Novodoy Housing Site)

Weight of Hammer	Pile Segment	Pile No. PP3		Pile No. PP4	
		Blows per 0.3m	Height of Drop,M	Blows per 0.3M	Height of Drop,M
	0.0-0.30m	-	-	-	-
	0.30-60m	-	0.3	-	-
	0.60-0.90m	4	0.3	4	0.3
	0.90-1.20m	6	0.45	3	0.3
	1.20-1.50m	5	0.45	3	0.45
	1.50-1.80m	7	0.45	7	0.45
	1.80-210m	9	0.75	10	0.45
	2.10-2.40m	10	0.75	15	0.45
	2.40-2.70m	12	0.75	11	0.45
	2.70-3.00m	12	0.75	12	0.45
	3.00-3.30m	14	0.75	16	0.45
	3.30-3.60m	17	1.0	13	0.75
500Kg	3.60-3.90m	18	1.0	17	0.75
	3.90-4.20m	14	1.0	20	0.75
	4.20-4.50m	17	1.2	21	0.75
	4.50-4.80m	23	1.2	15	1.0
	4.80-5.10m	20	1.2	18	1.0
	5.10-5.40m	21	1.2	22	1.0
	5.40-5.70m	23	1.2	23	1.0
	5.70-6.00m	18	1.2	19	1.35
	6.00-6.30m	20	1.2	22	1.35
	6.30-6.60m	25	1.2	22	1.35
	6.60-6.90m	27	1.2	23	1.35
	6.90-7.20m	29	1.2	25	1.35
	7.20-7.50m				

Table B-3 Pile Driving Records of PP5 and PP6 (Ahmedbagh Site)

Weight of Hammer	Pile Segment	Pile No. PP5		Pile No. PP6	
		Blows per 0.3m	Height of Drop,M	Blows per 0.3M	Height of Drop,M
	0.0-0.30m	-	-	8	0.3
	0.30-.60m	10	0.4	10	0.3
	0.60-0.90m	12	0.4	11	0.3
	0.90-1.20m	12	0.4	13	0.6
	1.20-1.50m	11	1.05	16	0.6
	1.50-1.80m	9	1.05	18	0.6
	1.80-2.10m	12	1.05	22	0.6
	2.10-2.40m	12	1.05	10	0.75
	2.40-2.70m	10	1.5	12	0.75
	2.70-3.00m	11	1.5	14	0.75
335Kg	3.00-3.30m	20	1.5	10	1.5
	3.30-3.60m	33	1.8	18	1.5
	3.60-3.90m	40	1.8	26	1.5
	3.90-4.20m	45	1.8	30	2.3
	4.20-4.50m	49	2.3	39	2.3
	4.50-4.80m	57	2.3	48	2.3
	4.80-5.10m	64	2.3	58	2.6
	5.10-5.40m	73	2.3	58	2.6
	5.40-5.70m	65	2.7	67	2.6
	5.70-6.00m	77	2.7	84	3
	6.00-6.30m	82	2.7	80	3
	6.30-6.60m	75	3	81	3
	6.60-6.90m	80	3	84	3.6
	6.90-7.20m	83	3	88	3.6
	7.20-7.50m				

Table B-4 Pile Driving Records of PP7 and PP8 (Moghbazar Site)

Weight of Hammer	Pile Segment	Pile No. PP7		Pile No. PP8	
		Blows per 0.3m	Height of Drop,M	Blows per 0.3M	Height of Drop,M
272 Kg	0.0-0.30m	-	0.3	-	0.3
	0.30-.60m	16	0.3	12	0.3
	0.60-0.90m	42	0.3	20	0.3
	0.90-1.20m	43	0.3	30	0.3
	1.20-1.50m	40	0.45	32	0.3
	1.50-1.80m	41	0.45	28	0.6
	1.80-2.10m	15	1.05	28	0.6
	2.10-2.40m	12	1.05	30	0.6
	2.40-2.70m	13	1.05	20	1.35
	2.70-3.00m	18	1.05	29	1.35
	3.00-3.30m	20	1.8	29	1.35
	3.30-3.60m	23	1.8	33	1.35
	3.60-3.90m	28	1.8	33	1.8
	3.90-4.20m	28	2.1	31	1.8
	4.20-4.50m	35	2.1	27	1.8
	4.50-4.80m	42	2.1	23	1.8
	4.80-5.10m	76	2.4	36	2.5
	5.10-5.40m	92	2.4	31	2.5
	5.40-5.70m	116	3	36	2.5
	5.70-6.00m	104	3	41	2.5
	6.00-6.30m	102	3	50	3
	6.30-6.60m	104	3	55	3
	6.60-6.90m	105	3	40	3
	6.90-7.20m	105	3	43	3.6
	7.20-7.50m				

APPENDIX- C

LOAD TIME SETTLEMENT RECORDS OF PILE LOAD TEST

Table C-1 Load Time Settlement Records of Pile Load test on Pile PP1

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
7/3/96	80	0/2	22.10	0	1680	0.00	1700	0.00	Loading
			22.15	5	1680	0.00	1700	0.00	
			22.20	10	1680	0.00	1700	0.00	
			22.30	20	1680	0.00	1700	0.00	
			22.40	30	1680	0.00	1700	0.00	
			22.50	40	1680	0.00	1700	0.00	
			23.00.	50	1680	0.00	1700	0.00	
			23.20	80	1680	0.00	1700	0.00	
	160	2/4	0.00	110/0	1680	0.00	1700	0.00	Loading
			0.05	5	1680	0.00	1700	0.00	
			0.10	10	1680	0.00	1700	0.00	
			0.20	20	1680	0.00	1700	0.00	
			0.30	30	1680	0.00	1700	0.00	
			0.40	40	1680	0.00	1700	0.00	
			1.00.	60	1680	0.00	1700	0.00	
			1.20	80	1680	0.00	1700	0.00	
	240	4/6	2.00	120/0	1680	0.00	1700	0.00	Loading
			2.05	5	1680	0.00	1700	0.00	
			2.10	10	1680	0.00	1700	0.00	
			2.20	20	1680	0.00	1700	0.00	
			2.30	30	1680	0.00	1700	0.00	
			2.40	40	1680	0.00	1700	0.00	
			3.00.	60	1680	0.00	1700	0.00	
			3.20	80	1680	0.00	1700	0.00	
	320	6/8	4.00	120/0	1680	0.03	1700	0.00	Loading
			4.05	5	1679	0.03	1697	0.08	
			4.10	10	1679	0.03	1697	0.08	
			4.20	20	1679	0.03	1697	0.08	
			4.30	30	1679	0.03	1697	0.08	
			4.40	40	1679	0.03	1697	0.08	
			5.00.	60	1679	0.03	1697	0.08	
			5.20	80	1679	0.03	1697	0.08	
	400	8/10	6.00	120/0	1679	0.03	1697	0.08	Loading
			6.05	5	1675	0.15	1692	0.20	
			6.10	10	1674	0.15	1690	0.25	
			6.20	20	1674	0.15	1690	0.25	
			6.30	30	1674	0.15	1690	0.25	
			6.40	40	1674	0.15	1690	0.25	
			7.00.	60	1674	0.15	1690	0.25	
			7.20	80	1674	0.15	1690	0.25	
			8.00	120/0	1674	0.15	1690	0.25	Loading

Table C-1 Contd..

	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
			8.00	0	1674	0.15	1690	0.25	Loading
	480	10/12	8.05	5	1670	0.25	1687	0.33	
			8.10	10	1668	0.30	1685	0.38	
			8.20	20	1666	0.35	1683	0.43	
			8.30	30	1666	0.35	1682	0.45	
			8.40	40	1666	0.35	1682	0.45	
			9.00.	60	1666	0.35	1682	0.45	
			9.20	80	1666	0.35	1682	0.45	
			10.00	120/0	1666	0.35	1682	0.45	Loading
	560	12/14	10.05	5	1660	0.50	1669	0.78	
			10.10	10	1655	0.63	1665	0.88	
			10.20	20	1652	0.70	1664	0.90	
			10.30	30	1652	0.70	1664	0.90	
			10.40	40	1652	0.70	1664	0.00	
			11.00.	60	1652	0.70	1664	0.90	
			11.20	80	1652	0.70	1664	0.90	
			12.00	120/0	1652	0.70	1664	0.90	Loading
	640	14/16	12.05	5	1631	1.23	1653	1.18	
			12.10	10	1620	1.50	1640	1.50	
			12.20	20	1614	1.65	1628	1.80	
			12.30	30	1614	1.65	1614	2.15	
			12.40	40	1614	1.65	1614	2.15	
			13.00.	60	1614	1.65	1614	2.15	
			13.20	80	1614	1.65	1614	2.15	
			14.00	120/0	1614	1.65	1614	2.15	Loading
	720	16/18	14.05	5	1610	1.75	1604	2.40	
			14.10	10	1604	1.90	1601	2.48	
			14.20	20	1600	2.00	1598	2.60	
			14.30	30	1600	2.00	1596	2.55	
			14.40	40	1600	2.00	1596	2.60	
			15.00.	60	1600	2.00	1596	2.60	
			15.20	80	1600	2.00	1596	2.60	
			16.00	120/0	1600	2.00	1596	2.60	Loading
	800	18/20	16.05	5	1554	3.15	1536	4.10	
			16.10	10	1506	4..35	1502	4.95	
			16.20	20	1483	4.93	1472	5,70	
			16.30	30	1470	5.25	1450	6.25	
			16.40	40	1465	5.38	1443	6.43	
			17.00.	60	1460	5.50	1440	6.50	
			17.20	80	1460	5.50	1440	6.50	
			18.00	120/0	1460	5.50	1440	6.50	Loading

Table C-1 **Contd...**

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			18.00	0	1460	5.50	1440	6.50	Loading
	880	20/22	18.05	5	1407	6.83	1392	7.70	
			18.10	10	1369	7.78	1343	8.93	
			18.20	20	1355	8.13	1331	9.23	
			18.30	30	1350	8.25	1325	9.38	
			18.40	40	1340	8.50	1320	9.50	
			19.00.	60	1340	8.50	1320	9.50	
			19.20	80	1340	8.50	1320	9.50	
			20.00	120/0	1340	8.50	1320	9.50	Loading
	960	22/24	20.03	3	963/ 1650	17.93	890/ 1674	20.25	Reset to 1650 (G1) 1674 (G2)
			20.08	8	1252	27.88	1306	29.5	
			20.15	15	1107	31.05	1172	32.85	
			20.30						Unloading
									records not maintained

Table C-2 Load Time Settlement Records of Pile Load test on Pile PP2

Date	Load		Time	Elapsed time	Settlement records				Remarks
	Jack pressure	Load in tons		Minutes	Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
12/3/96	80	0/2	22.15	0	1881	0.00	1860	0.00	Loading
			22.20	5	1881	0.00	1860	0.00	
			22.25	10	1881	0.00	1860	0.00	
			22.35	20	1881	0.00	1860	0.00	
			22.45	30	1881	0.00	1860	0.00	
			22.55	40	1881	0.00	1860	0.00	
			23.15.	60	1881	0.00	1860	0.00	
			23.35	80	1881	0.00	1860	0.00	
	160	2/4	0.00	105/0	1881	0.00	1860	0.00	Loading
			0.05	5	1881	0.00	1860	0.00	
			0.10	10	1881	0.00	1860	0.00	
			0.20	20	1881	0.00	1860	0.00	
			0.30	30	1881	0.00	1860	0.00	
			0.40	40	1881	0.00	1860	0.00	
			1.00.	60	1881	0.00	1860	0.00	
			1.20	80	1881	0.00	1860	0.00	
	240	4/6	2.00	120/0	1881	0.00	1860	0.00	Loading
			2.05	5	1876	0.13	1857	0.08	
			2.10	10	1876	0.13	1857	0.08	
			2.20	20	1876	0.13	1857	0.08	
			2.30	30	1876	0.13	1857	0.08	
			2.40	40	1876	0.13	1857	0.08	
			3.00.	60	1876	0.13	1857	0.08	
			3.20	80	1876	0.13	1857	0.08	
	320	6/8	4.00	120/0	1876	0.13	1857	0.08	Loading
			4.05	5	1873	0.20	1856	0.10	
			4.10	10	1873	0.20	1856	0.10	
			4.20	20	1873	0.20	1856	0.10	
			4.30	30	1873	0.20	1856	0.10	
			4.40	40	1873	0.20	1856	0.10	
			5.00.	60	1873	0.20	1856	0.10	
			5.20	80	1873	0.20	1856	0.10	
	400	8/10	6.00	120/0	1873	0.20	1856	0.10	Loading
			6.05	5	1870	0.28	1850	0.25	
			6.10	10	1862	0.48	1848	0.30	
			6.20	20	1862	0.48	1847	0.33	
			6.30	30	1862	0.48	1847	0.33	
			6.40	40	1862	0.48	1847	0.33	
			7.00.	60	1862	0.48	1847	0.33	
			7.20	80	1862	0.48	1847	0.33	
			8.00	120/0	1862	0.48	1847	0.33	Loading

Table C - 2 Contd..

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
	480	10/12	8.00	0	1862	0.48	1847	0.33	Loading
			8.05	5	1855	0.65	1843	0.43	
			8.10	10	1853	0.70	1840	0.50	
			8.20	20	1853	0.70	1840	0.50	
			8.30	30	1853	0.70	1840	0.50	
			8.40	40	1853	0.70	1840	0.50	
			9.00.	60	1853	0.70	1840	0.50	
			9.20	80	1853	0.70	1840	0.50	
	560	12/14	10.00	120/0	1853	0.70	1840	0.50	Loading
			10.05	5	1850	0.78	1837	0.58	
			10.10	10	1848	0.83	1834	0.65	
			10.20	20	1848	0.83	1833	0.68	
			10.30	30	1848	0.83	1833	0.68	
			10.40	40	1848	0.83	1833	0.68	
			11.00.	60	1848	0.83	1833	0.68	
			11.20	80	1848	0.83	1833	0.68	
	640	14/16	12.00	120/0	1848	0.83	1833	0.68	Loading
			12.05	5	1846	0.88	1831	0.78	
			12.10	10	1846	0.88	1831	0.73	
			12.20	20	1846	0.88	1831	0.73	
			12.30	30	1846	0.88	1831	0.73	
			12.40	40	1846	0.88	1831	0.73	
			13.00.	60	1846	0.88	1831	0.73	
			13.20	80	1846	0.88	1831	0.73	
	720	16/18	14.00	120/0	1846	0.88	1831	0.73	Loading
			14.05	5	1840	1.13	1824	0.90	
			14.10	10	1833	1.20	1820	1.00	
			14.20	20	1830	1.28	1820	1.00	
			14.30	30	1825	1.40	1820	1.00	
			14.40	40	1825	1.40	1820	1.00	
			15.00.	60	1825	1.40	1820	1.00	
			15.20	80	1825	1.40	1820	1.00	
	800	18/20	16.00	120/0	1825	1.40	1820	1.00	Loading
			16.05	5	1800	2.03	1805	1.38	
			16.10	10	1797	2.10	1800	1.50	
			16.20	20	1795	2.15	1795	1.63	
			16.30	30	1790	2.28	1794	1.65	
			16.40	40	1790	2.28	1791	1.73	
			17.00.	60	1790	2.28	1791	1.73	
			17.20	80	1790	2.28	1791	1.73	
			18.00	120/0	1790	2.28	1791	1.73	Loading

Table C - 2 **Contd..**

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
			18.00	0	1790	2.28	1791	11·98	Loading
	880	20/22	18.03	3	1415	11.65	1381	7·70	
			18.08	8	1010/ 1550	21·80	1032/ 1674	20.70	Reset to 1650 (G1) 1674 (G2)
			18.13	13	1289	30.81	1341	29·15	
			18.23	23	1009	37.81	1116	34.65	
			18.40	40					Unloading
									records not maintained

Table C-3 Load Time Settlement Records of Pile Load test on Pile PP3

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
21/6/96	80	0/2	22.00	0	1640	0.00	1635	0.00	Loading
			22.05	5	1640	0.00	1635	0.00	
			22.10	10	1640	0.00	1635	0.00	
			22.20	20	1640	0.00	1635	0.00	
			22.30	30	1640	0.00	1635	0.00	
			22.40	40	1640	0.00	1635	0.00	
			23.00.	60	1640	0.00	1635	0.00	
			23.20	80	1640	0.00	1635	0.00	
	160	2/4	0.00	120/0	1640	0.00	1635	0.00	Loading
			0.05	5	1637	0.08	1634	0.03	
			0.10	10	1637	0.08	1634	0.03	
			0.20	20	1637	0.08	1634	0.03	
			0.30	30	1637	0.08	1634	0.03	
			0.40	40	1637	0.08	1634	0.03	
			1.00.	60	1637	0.08	1634	0.03	
			1.20	80	1637	0.08	1634	0.03	
	240	4/6	2.00	120/0	1637	0.08	1634	0.03	Loading
			2.05	5	1636	0.10	1633	0.05	
			2.10	10	1635	0.13	1633	0.05	
			2.20	20	1634	0.15	1633	0.05	
			2.30	30	1634	0.15	1633	0.05	
			2.40	40	1634	0.15	1633	0.05	
			3.00.	60	1634	0.15	1633	0.05	
			3.20	80	1634	0.15	1633	0.05	
	320	6/8	4.00	120/0	1634	0.15	1633	0.05	Loading
			4.05	5	1633	0.18	1632	0.08	
			4.10	10	1633	0.18	1632	0.08	
			4.20	20	1633	0.18	1631	0.10	
			4.30	30	1633	0.18	1631	0.10	
			4.40	40	1633	0.18	1630	0.13	
			5.00.	60	1633	0.18	1630	0.13	
			5.20	80	1633	0.18	1630	0.13	
	400	8/10	6.00	120/0	1633	0.18	1630	0.13	Loading
			6.05	5	1620	0.50	1618	0.43	
			6.10	10	1617	0.58	1616	0.48	
			6.20	20	1615	0.63	1615	0.50	
			6.30	30	1613	0.68	1615	0.50	
			6.40	40	1612	0.70	1615	0.50	
			7.00.	60	1612	0.70	1615	0.50	
			7.20	80	1612	0.70	1615	0.50	
			8.00	120/0	1612	0.70	1615	0.50	Loading

Table C-3 Contd..

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			8.00	0	1612	0.70	1615	0.50	Loading
	480	10/12	8.05	5	1592	1.20	1590	1.13	
			8.10	10	1583	1.43	1583	1.30	
			8.20	20	1580	1.50	1581	1.35	
			8.30	30	1575	1.63	1578	1.43	
			8.40	40	1572	1.70	1577	1.45	
			9.00.	60	1570	1.75	1577	1.45	
			9.20	80	1570	1.75	1577	1.45	
			10.00	120/0	1570	1.75	1577	1.45	Loading
	560	12/14	10.05	5	1530	2.75	1552	2.08	
			10.10	10	1525	2.88	1547	2.20	
			10.20	20	1522	2.95	1542	2.33	
			10.30	30	1520	3.00	1540	2.38	
			10.40	40	1520	3.00	1539	2.40	
			11.00.	60	1520	3.00	1539	2.40	
			11.20	80	1520	3.00	1539	2.40	
			12.00	120/0	1520	3.00	1539	2.40	Loading
	640	14/16	12.05	5	1480	4.00	1509	3.15	
			12.10	10	1475	4.13	1502	3.33	
			12.20	20	1471	4.23	1492	3.58	
			12.30	30	1470	4.25	1485	3.75	
			12.40	40	1468	4.30	1482	3.83	
			13.00.	60	1465	4.38	1482	3.83	
			13.20	80	1465	4.38	1482	3.83	
			14.00	120/0	1465	4.38	1482	3.83	Loading
	720	16/18	14.05	5	1385	6.38	1397	5.95	
			14.10	10	1360	7.00	1377	6.45	
			14.20	20	1342	7.45	1360	6.88	
			14.30	30	1303	8.43	1342	7.33	
			14.40	40	1282	8.95	1330	7.63	
			15.00	60	1276	9.10	1322	7.83	
			15.20	80	1264	9.40	1316	7.98	
			16.00	120/0	1260	9.50	1303	8.30	Loading
	800	18/20	16.02	2	760/ 1702	22.0	823/ 1506	20.30	Reset to 1702 (G1) 1506 (G2)
			16.07	7	1506	26.90	1321	24.95	
			16.17	17	1392	29.75	1207	27.80	
			16.30	30					Unloading

Table C-4 Load Time Settlement Records of Pile Load test on Pile PP4

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			22.00	0	1705	0.00	1700	0.00	Loading
30/6/96	80	0/2	22.05	5	1705	0.00	1700	0.00	
			22.10	10	1705	0.00	1700	0.00	
			22.20	20	1705	0.00	1700	0.00	
			22.30	30	1705	0.00	1700	0.00	
			22.40	40	1705	0.00	1700	0.00	
			23.00.	60	1705	0.00	1700	0.00	
			23.20	80	1705	0.00	1700	0.00	
			0.00	120/0	1705	0.00	1700	0.00	Loading
	160	2/4	0.05	5	1705	0.00	1700	0.00	
			0.10	10	1705	0.00	1700	0.00	
			0.20	20	1705	0.00	1700	0.00	
			0.30	30	1705	0.00	1700	0.00	
			0.40	40	1705	0.00	1700	0.00	
			1.00.	60	1705	0.00	1700	0.00	
			1.20	80	1705	0.00	1700	0.00	
			2.00	120/0	1705	0.00	1700	0.00	Loading
	240	4/6	2.05	5	1702	0.08	1697	0.08	
			2.10	10	1701	0.10	1693	0.18	
			2.20	20	1700	0.13	1693	0.18	
			2.30	30	1700	0.13	1693	0.18	
			2.40	40	1700	0.13	1693	0.18	
			3.00.	60	1700	0.13	1693	0.18	
			3.20	80	1700	0.13	1693	0.18	
			4.00	120/0	1700	0.13	1693	0.18	Loading
	320	6/8	4.05	5	1680	0.63	1670	0.75	
			4.10	10	1678	0.68	1668	080	
			4.20	20	1677	0.70	1665	0.88	
			4.30	30	1677	0.70	1664	0.90	
			4.40	40	1677	0.70	1664	0.90	
			5.00.	60	1677	0.70	1664	0.90	
			5.20	80	1677	0.70	1664	0.90	
			6.00	120/0	1677	0.70	1664	0.90	Loading
	400	8/10	6.05	5	1660	1.13	1651	1.23	
			6.10	10	1653	1.30	1640	1.50	
			6.20	20	1649	1.40	1638	1.55	
			6.30	30	1649	1.40	1636	1.60	
			6.40	40	1649	1.40	1636	1.60	
			7.00.	60	1649	1.40	1636	1.60	
			7.20	80	1649	1.40	1636	1.60	
			8.00	120/0	1649	1.40	1636	1.60	Loading

Table C-4 Contd..

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
	480	10/12	8.00	0	1649	1.40	1636	1.60	Loading
			8.05	5	1628	1.93	1611	2.23	
			8.10	10	1620	2.13	1602	2.45	
			8.20	20	1615	2.25	1596	2.60	
			8.30	30	1615	2.25	1593	2.68	
			8.40	40	1615	2.25	1590	2.75	
			9.00.	60	1615	2.25	1590	2.75	
			9.20	80	1615	2.25	1590	2.75	
	560	12/14	10.00	120/0	1615	2.25	1590	2.75	Loading
			10.05	5	1595	2.75	1568	3.30	
			10.10	10	1582	3.08	1552	3.70	
			10.20	20	1577	3.20	1550	3.75	
			10.30	30	1577	3.20	1548	3.80	
			10.40	40	1577	3.20	1548	3.80	
			11.00.	60	1577	3.20	1548	3.80	
			11.20	80	1577	3.20	1548	3.80	
	640	14/16	12.00	120/0	1577	3.20	1548	3.80	Loading
			12.05	5	1547	3.95	1548	3.80	
			12.10	10	1540	4.13	1516	4.60	
			12.20	20	1532	4.33	1500	5.00	
			12.30	30	1527	4.45	1498	5.05	
			12.40	40	1525	4.50	1496	5.10	
			13.00.	60	1525	4.50	1496	5.10	
			13.20	80	1525	4.50	1496	5.10	
	720	16/18	14.00	120/0	1525	4.50	1496	5.10	Loading
			14.05	5	1497	5.20	1469	5.78	
			14.10	10	1483	5.55	1450	6.25	
			14.20	20	1471	5.85	1439	6.53	
			14.30	30	1465	6.00	1430	6.75	
			14.40	40	1457	6.20	1421	6.98	
			15.00.	60	1454	6.28	1415	7.13	
			15.20	80	1454	6.28	1415	7.13	
	800	18/20	16.00	120/0	1454	6.28	1415	7.13	Loading
			16.04	4	1051/ 1850	16.35	1004/ 1780	17.40	Reset to 1850 (G1) 1780 (G2)
			16.7	7	1409	27.38	1315	29.03	
			16.10	10	1305	29.97	1165	32.78	
			16.38	38					Unloading ,

Table C - 5 Load Time Settlement Records of Pile Load test on Pile PP5

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			8.00	0	1790	0.00	1806	0.00	Loading
24/7/ 96	100	0/2.5	8.05	5	1790	0.00	1806	0.00	
			8.10	10	1790	0.00	1806	0.00	
			8.20	20	1790	0.00	1806	0.00	
			8.30	30	1790	0.00	1806	0.00	
			8.40	40	1790	0.00	1806	0.00	
			9.00.	60	1790	0.00	1806	0.00	
			9.20	80	1790	0.00	1806	0.00	
			10.00	120/0	1790	0.00	1806	0.00	Loading
	200	2.5/5	10.05	5	1784	0.15	1804	0.05	
			10.10	10	1784	0.15	1804	0.05	
			10.20	20	1784	0.15	1804	0.05	
			10.30	30	1784	0.15	1804	0.05	
			10.40	40	1784	0.15	1804	0.05	
			11.00.	60	1784	0.15	1804	0.05	
			11.20	80	1784	0.15	1804	0.05	
			12.00	120/0	1784	0.15	1804	0.05	Loading
	300	5/7.5	12.05	5	1780	0.25	1800	0.15	
			12.10	10	1780	0.25	1800	0.15	
			12.20	20	1780	0.25	1800	0.15	
			12.30	30	1780	0.25	1800	0.15	
			12.40	40	1780	0.25	1800	0.15	
			13.00.	60	1780	0.25	1800	0.15	
			13.20	80	1780	0.25	1800	0.15	
			14.00	120/0	1780	0.25	1800	0.15	Loading
	400	7.5/10	14.05	5	1776	0.45	1798	0.20	
			14.10	10	1772	0.45	1798	0.20	
			14.20	20	1772	0.45	1798	0.20	
			14.30	30	1772	0.45	1798	0.20	
			14.40	40	1772	0.45	1798	0.20	
			15.00.	60	1772	0.45	1798	0.20	
			15.20	80	1772	0.45	1798	0.20	
			16.00	120/0	1772	0.45	1798	0.20	Loading
	500	10/12.5	16.05	5	1770	0.50	1796	0.25	
			16.10	10	1770	0.50	1796	0.25	
			16.20	20	1770	0.50	1796	0.25	
			16.30	30	1770	0.50	1796	0.25	
			16.40	40	1770	0.50	1796	0.25	
			17.00.	60	1770	0.50	1796	0.25	
			17.20	80	1770	0.50	1796	0.25	
			18.00	120/0	1770	0.50	1796	0.25	Loading

Table C-5 Contd..

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			18.00	0	1770	0.50	1796	0.25	Loading
	600	12.5/15	18.05	5	1768	0.55	1794	0.30	
			18.10	10	1768	0.55	1794	0.30	
			18.20	20	1768	0.55	1794	0.30	
			18.30	30	1768	0.55	1794	0.30	
			18.40	40	1768	0.55	1794	0.30	
			19.00	60	1768	0.55	1794	0.30	
			19.20	80	1768	0.55	1794	0.30	
			20.00	120/0	1768	0.55	1794	0.30	Loading
	700	15/17.5	20.05	5	1767	0.58	1791	0.38	
			20.10	10	1767	0.58	1791	0.38	
			20.20	20	1767	0.58	1791	0.38	
			20.30	30	1767	0.53	1791	0.38	
			20.40	40	1767	0.58	1791	0.38	
			21.00	60	1767	0.58	1791	0.38	
			21.20	80	1767	0.58	1791	0.38	
			22.00	120/0	1767	0.58	1791	0.38	Loading
	800	17.5/20	22.05	5	1758	0.80	1789	0.43	
			22.10	10	1757	0.83	1789	0.43	
			22.20	20	1757	0.83	1789	0.43	
			22.30	30	1757	0.83	1789	0.43	
			22.40	40	1757	0.83	1789	0.43	
			23.00	60	1757	0.83	1789	0.43	
			23.20	80	1757	0.83	1789	0.43	
			0.00	120/0	1757	0.83	1789	0.43	Loading
	900	20/22.5	0.05	5	1754	0.90	1785	0.53	
			0.10	10	1752	0.95	1783	0.58	
			0.20	20	1752	0.95	1781	0.63	
			0.30	30	1752	0.95	1781	0.63	
			0.40	40	1752	0.95	1781	0.63	
			10.00	60	1752	0.95	1781	0.63	
			10.20	80	1752	0.95	1781	0.63	
			2.00	120/0	1752	0.95	1781	0.63	Loading
	1000	22.5/25	2.05	5	1748	1.05	1777	0.73	
			2.10	10	1745	1.13	1773	0.83	
			2.20	20	1741	1.23	1773	0.83	
			2.30	30	1741	1.23	1773	0.83	
			2.40	40	1741	1.23	1773	0.83	
			3.00	60	1741	1.23	1773	0.83	
			3.20	80	1741	1.23	1773	0.83	
			4.00	120/0	1741	1.23	1773	0.83	Loading

Table C - 5 Cont

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
			4.00	0	1741	1.23	1773	0.83	Loading
	1100	25/27.5	4.05	5	1698	2.30	1753	1.33	
			4.10	10	1696	2.35	1742	1.60	
			4.20	20	1694	2.40	1738	1.70	
			4.30	30	1694	2.40	1735	1.78	
			4.40	40	1694	2.40	1732	1.85	
			5.00	60	1694	2.40	1732	1.85	
			5.20	80	1694	2.40	1732	1.85	
			6.00	120/0	1694	2.40	1732	1.85	Loading
	1200	27.5/30	6.05	5	1646	3.60	1699	2,68	
			7.10	10	1635	3.88	1690	2.90	
			7.20	20	1628	4.05	1687	2.98	
			7.30	30	1627	4.08	1687	2.98	
			7.40	40	1627	4.08	1687	2.98	
			8.00	60	1627	4.08	1687	2.98	
			8.20	80	1627	4.08	1687	2.98	
			9.00	120/0	1627	4.08	1687	2.98	Loading
			9.05	5	1586	5.10	1655	3.78	
	1300	30/32.5	9.10	10	1569	5.53	1642	4.10	
			9.20	20	1560	5.75	1631	4.38	
			9.30	30	1551	5.98	1628	4.45	
			9.40	40	1547	6.08	1625	4.53	
			10.00	60	1547	6.08	1623	4.58	
			10.20	80	1547	6.08	1623	4.58	
			11.00	120/0	1547	6.08	1623	4.58	Loading
			11.05	5	1519	6.78	1600	5.15	
	1400	32.5/35	11.10	10	1513	6.93	1579	5.68	
			11.20	20	1509	7.03	1569	5.93	
			11.30	30	1507	7.03	1568	5.95	
			11.40	40	1507	7.03	1567	5.98	
			12.00	60	1507	7.03	1567	5.98	
			12.20	80	1507	7.03	1567	5.98	
			12.40	100	1507	7.03	1567	5.98	
			13.00	120/0	1507	7.03	1567	5.98	
	1500	35/37.5	13.03	3	997	19.83	1089	17.93	
			13.07	7	752/ 1620	25.95	709/ 1605	27.43	Reset 1620(G1) 1605 (G2)
			13.15	15	1411	31.18	1389	32.83	
			13.40						Unloading records not maintained

Table C-6 Load Time Settlement Records of Pile Load test on Pile PP6

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment	Reading	S.ment	
29/7/96	100	0/2.5	9.00	0	1603	0.00	1592	0.00	Loading
			9.05	5	1603	0.00	1592	0.00	
			9.10	10	1603	0.00	1592	0.00	
			9.20	20	1603	0.00	1592	0.00	
			9.30	30	1603	0.00	1592	0.00	
			9.40	40	1603	0.00	1592	0.00	
			10.00	60	1603	0.00	1592	0.00	
			10.20	80	1603	0.00	1592	0.00	
	200	2.5/5	11.00	120/0	1603	0.00	1592	0.00	Loading
			11.05	5	1603	0.00	1592	0.00	
			11.10	10	1603	0.00	1592	0.00	
			11.20	20	1603	0.00	1592	0.00	
			11.30	30	1603	0.00	1592	0.00	
			11.40	40	1603	0.00	1592	0.00	
			1.2.00	60	1603	0.00	1592	0.00	
			11.20	80	1603	0.00	1592	0.00	
	300	5/7.5	13.00	120/0	1603	0.00	1592	0.00	Loading
			13.05	5	1603	0.00	1592	0.00	
			13.10	10	1603	0.00	1592	0.00	
			13.20	20	1603	0.00	1592	0.00	
			13.30	30	1603	0.00	1592	0.00	
			13.40	40	1603	0.00	1592	0.00	
			14.00	60	1603	0.00	1592	0.00	
			14.20	80	1603	0.00	1592	0.00	
	400	7.5/10	15.00	120/0	1603	0.00	1592	0.00	Loading
			15.05	5	1603	0.00	1592	0.00	
			15.10	10	1603	0.00	1592	0.00	
			15.20	20	1603	0.00	1592	0.00	
			15.30	30	1603	0.00	1592	0.00	
			15.40	40	1603	0.00	1592	0.00	
			16.00	60	1603	0.00	1592	0.00	
			16.20	80	1603	0.00	1592	0.00	
	500	10/12.5	17.00	120/0	1603	0.00	1592	0.00	Loading
			17.05	5	1603	0.00	1592	0.00	
			17.10	10	1603	0.00	1592	0.00	
			17.20	20	1603	0.00	1592	0.00	
			17.30	30	1603	0.00	1592	0.00	
			17.40	40	1603	0.00	1592	0.00	
			18.00	60	1603	0.00	1592	0.00	
			18.20	80	1603	0.00	1592	0.00	
			19.00	120/0	1603	0.00	1592	0.00	Loading

Table C-6 Contd..

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			19.00	0	1603	0.00	1592	0.00	Loading
	600	12.5/15	19.05	5	1601	0.05	1590	0.50	
			19.10	10	1601	0.05	1590	0.50	
			19.20	20	1601	0.05	1590	0.50	
			19.30	30	1601	0.05	1590	0.50	
			19.40	40	1601	0.05	1590	0.50	
			20.00	60	1601	0.05	1590	0.50	
			20.20	80	1601	0.05	1590	0.50	
			21.00	120/0	1601	0.05	1590	0.50	Loading
	700	15/17.5	21.05	5	1598	0.13	1585	0.18	
			21.10	10	1596	0.18	1584	0.20	
			21.20	20	1596	0.18	1583	0.23	
			21.30	30	1596	0.18	1583	0.23	
			21.40	40	1596	0.18	1583	0.23	
			22.00.	60	1596	0.18	1583	0.23	
			22.20	80	1596	0.18	1583	0.23	
			23.00	120/0	1596	0.18	1583	0.23	Loading
	800	17.5/20	23.05	5	1590	0.33	1570	0.55	
			23.10	10	1585	0.45	1564	0.70	
			23.20	20	15 83	0.50	1562	0.75	
			23.30	30	1581	0.55	1562	0.75	
			23.40	40	1581	0.55	1562	0.75	
			0.00	60	1581	0.55	1562	0.75	
			0.20	80	1581	0.55	1562	0.75	
			1.00	120/0	1581	0.55	1562	0.75	Loading
	900	20/22.5	1..05	5	1570	0.83	1541	1.28	
			1.10	10	1563	1.00	1538	1.35	
			1.20	20	1563	1.00	1536	1.40	
			1.30	30	1563	1.00	1536	1.40	
			1.40	40	1563	1.00	1536	1.40	
			2.00	60	1563	1.00	1536	1.40	
			2.20	80	1563	1.00	1536	1.40	
			3.00	120/0	1563	1.00	1536	1.40	Loading
	1000	22.5/25	3.05	5	1550	1.33	1530	1.55	
			3.10	10	1546	1.43	1523	1.73	
			3.20	20	1538	1.63	1515	1.93	
			3.30	30	1538	1.63	1513	1.98	
			3.40	40	1538	1.63	1513	1.98	
			4..00	60	1538	1.63	1513	1.98	
			4.20	80	1538	1.63	1513	1.98	
			5.00	120/0	1538	1.63	1513	1.98	Loading

Table C-6 Contd...

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
	1100	25/27.5	5.00	0	1538	1.63	1513	1.98	Loading
			5.05	5	1531	1.80	1501	2.28	
			5.10	10	1536	1·68	1491	2.53	
			5.20	20	1521	2.05	1484	2.70	
			5.30	30	1521	2.05	1482	2.75	
			5.40	40	1521	2.05	1482	2.75	
			6.00	60	1521	2.05	1482	2.75	
			6.20	80	1521	2.05	1482	2.75	
	1200	27.5/30	7.00	120/0	1521	2.05	1482	2.75	Loading
			7.05	5	1518	2.13	1480	2.80	
			7.10	10	1513	2.25	1475	2.93	
			7.20	20	1512	2.28	1475	2.93	
			7.30	30	1512	2.28	1475	2.93	
			7.40	40	1512	2.28	1475	2.93	
			8.00	60	1512	2.28	1475	2.93	
			8.20	80	1512	2.28	1475	2.93	
	1300	30/32.5	9.00	120/0	1512	2.28	1475	2.93	Loading
			9.05	5	1507	2.40	1464	3.20	
			9.10	10	1504	2.48	1460	3.30	
			9.20	20	1504	2.48	1457	3.38	
			9.30	30	1504	2.48	1451	3.53	
			9.40	40	1504	2.48	1451	3.53	
			10.00	60	1504	2.48	1451	3.53	
			10.20	80	1504	2.48	1451	3.53	
	1400	32.5/35	11.00	120/0	1504	2.48	1451	3.53	Loading
			11.05	5	1481	3.05	1433	3·98	
			11.10	10	1476	3.18	1421	4.28	
			11.20	20	1472	3.28	1421	4·28	
			11.30	30	1472	3.28	1419	4.33	
			11.40	40	1472	3.28	1419	4.33	
			11.50	50	1472	3.28	1419	4.33	
			12.00	60	1472	3.28	1419	4.33	
	1500	35/37.5	12.20	80	1472	3.28	1419	4.33	
			13.00	120/0	1472	3.28	1419	4.33	
			13.05	5	1416	4.68	1342	6.25	Reset 1620(G1) 1605 (G2)
			13.10	10	1400	5.08	13.18	6.85	
			13.20	20	1385	5.45	1270	8.05	
			13.30	30	1361	6.05	1262	8.25	
			13.40	40	1341	6.55	1249	8.58	
			14.00	60	1322	7.03	1242	8.75	
			14.20	80	1315	7.20	1226	9.15	
			15.00	120/0	1315	7.20	1226	9.15	

Table C-6 Contd..

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
			15.00	120/0	1315	7.25	1226	9.15	Loading
			15.02	2	974/ 1620	15.73	906/ 1605	17.15	
	1600	37.5/40	15.05	5	906	33.58	912	34.47	
			15.10	10	726	38.08	809	37.04	
			15.20	20					Unloading
									not
									recorded

Table C-7 Load Time Settlement Records of Pile Load test on Pile PP7

Date	Load		Time	Elapsed time	Settlement records				Remarks
	Jack pressure	Load in tons		Minutes	Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
20/8/96	100	0/2.5	10.12	0	1685	0.00	1706	0.00	Loading
			10.17	5	1682	0.08	1705	0.03	
			10.22	10	1682	0.08	1705	0.03	
			10.32	20	1682	0.08	1705	0.03	
			10.42	30	1682	0.08	1705	0.03	
			10.52	40	1682	0.08	1705	0.035	
			11.12	60	1682	0.08	1705	0.035	
			12.32	80	1682	0.08	1705	0.03	
	200	2.5/5	12.00	108/0	1682	0.08	1705	0.03	Loading
			12.05	5	1676	0.23	1700	0.15	
			12.10	10	1668	0.43	1694	0.30	
			12.20	20	1667	0.45	1693	0.33	
			12.30	30	1667	0.45	1692	0.35	
			12.40	40	1667	0.45	1692	0.35	
			13.00	60	1667	0.45	1692	0.35	
			13.20	80	1667	0.45	1692	0.35	
	300	5/7.5	14.00	120/0	1667	0.45	1692	0.35	Loading
			14.05	5	1660	0.63	1691	0.38	
			14.10	10	1660	0.63	1691	0.38	
			14.20	20	1660	0.63	1691	0.38	
			14.30	30	1660	0.63	1691	0.38	
			14.40	40	1660	0.63	1691	0.38	
			15.00	60	1660	0.63	1691	0.38	
			15.20	80	1660	0.63	1691	0.38	
	400	7.5/10	16.00	120/0	1660	0.63	1691	0.38	Loading
			16.05	5	1657	0.70	1690	0.40	
			16.10	10	1657	0.70	1690	0.40	
			16.20	20	1657	0.70	1690	0.40	
			13.30	30	1657	0.70	1690	0.40	
			16.40	40	1657	0.70	1690	0.40	
			17.00	60	1657	0.70	1690	0.40	
			17.20	80	1657	0.70	1690	0.40	
	500	10/12.5	18.00	120/0	1657	0.70	1690	0.40	Loading
			18.05	5	1655	0.75	1689	0.43	
			18.10	10	1655	0.75	1688	0.45	
			18.20	20	1655	0.75	1688	0.45	
			18.30	30	1655	0.75	1688	0.45	
			18.40	40	1655	0.75	1688	0.45	
			19.00	60	1655	0.75	1688	0.45	
			19.20	80	1655	0.75	1688	0.45	
			20.00	120/0	1655	0.75	1688	0.45	Loading

Table C-7 Contd..

Date	Load: Jack pressure	Load: Load in tons	Time	Elapsed time Minutes	Gauge G1 Reading	Gauge G1 S.ment (mm)	Gauge G2 Reading	Gauge G2 S.ment (mm)	Remarks
			20.00	0	1655	0.75	1688	0.45	Loading
	600	12.5/15	20.05	5	1654	0.78	1687	0.47	
			20.10	10	1654	0.78	1685	0.53	
			20.20	20	1654	0.78	1685	0.53	
			20.30	30	1654	0.78	1685	0.53	
			20.40	40	1654	0.78	1685	0.53	
			21.00	60	1654	0.78	1685	0.53	
			21.20	80	1654	0.78	1685	0.53	
			22.00	120/0	1654	0.78	1685	0.53	Loading
	700	15/17.5	22.05	5	1653	0.80	1684	0.55	
			22.10	10	1653	0.80	1682	0.60	
			22.20	20	1653	0.80	1682	0.60	
			22.30	30	1653	0.80	1682	0.60	
			22.40	40	1653	0.80	1682	0.60	
			23.00	60	1653	0.80	1682	0.60	
			23.20	80	1653	0.80	1682	0.60	
			24.00	120/0	1653	0.80	1682	0.60	Loading
	800	17.5/20	0.05	5	1648	0.93	1680	0.65	
			0.10	10	1646	0.98	1675	0.78	
			0.20	20	1644	1.03	1675	0.78	
			0.30	30	1644	1.03	1675	0.78	
			0.40	40	1644	1.03	1675	0.78	
			1.00	60	1644	1.03	1675	0.78	
			1.20	80	1644	1.03	1675	0.78	
			2.00	120/0	1644	1.03	1675	0.78	Loading
	900	20/22.5	2.05	5	1641	1.10	1672	0.85	
			2.10	10	1638	1.18	1672	0.85	
			2.20	20	1635	1.25	1672	0.85	
			2.30	30	1635	1.25	1672	0.85	
			2.40	40	1635	1.25	1672	0.85	
			3.00	60	1635	1.25	1672	0.85	
			3.20	80	1635	1.25	1672	0.85	
			4.00	120/0	1635	1.25	1672	0.85	Loading
	1000	22.5/25	4.05	5	1626	1.48	1668	0.95	
			4.10	10	1616	1.73	1663	1.08	
			4.20	20	1612	1.83	1661	1.13	
			4.30	30	1610	1.88	1661	1.13	
			4.40	40	1610	1.88	1661	1.13	
			5.00	60	1610	1.88	1661	1.13	
			5.20	80	1610	1.88	1661	1.13	
			6.00	120/0	1610	1.88	1661	1.13	Loading

Table C-7 Contd.

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			6.00	0	1610	1.88	1661	1.13	Loading
	1100	25/27.5	6.05	5	1592	2.33	1641	1.63	
			6.10	10	1581	2.60	1633	1.83	
			6.20	20	1577	2.70	1622	2.10	
			6.30	30	1573	2.80	1615	2.28	
			6.40	40	1573	2.80	1610	2.40	
			7.00	60	1573	2.80	1610	2.40	
			7.20	80	1573	2.80	1610	2.40	
			8.00	120/0	1573	2.80	1610	2.40	Loading
	1200	27.5/30	8.05	5	1562	3.02	1601	2.63	
			8.10	10	1553	3.30	1596	2.75	
			8.20	20	1540	3.63	1592	2.85	
			8.30	30	1532	3.83	1585	3.03	
			8.40	40	1530	3.88	1581	3.13	
			9.00	60	1530	3.88	1581	3.13	
			9.20	80	1530	3.88	1581	3.13	
			10.00	120/0	1530	3.88	1581	3.13	Loading
			10.05	5	1511	4.35	1551	3.88	
	1300	30/32.5	10.10	10	1482	5.08	1545	4.03	
			10.20	20	1466	5.48	1533	4.33	
			10.30	30	1460	5.63	1527	4.48	
			10.40	40	1460	5.63	1527	4.48	
			11.00	60	1460	5.63	1527	4.48	
			11.20	80	1460	5.63	1527	4.48	
			12.00	120/0	1460	5.63	1527	4.48	Loading
			12.05	5	949/ 1525	18.40	945/ 1581	19.03	Reset 1525 (G1) 1581 (G2)
	1400	32.5/35	12.07	7	765	37.4	893	36.23	
			12.35						Unloading
									Records not maintained

Table C-8 Load Time Settlement Records of Pile Load test on Pile PP8

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			10.35	0	1506	0.00	1525	0.00	Loading
29/8/96	100	0/2.5	10.40	5	1506	0.00	1525	0.00	
			10.45	10	1506	0.00	1525	0.00	
			10.55	20	1506	0.00	1525	0.00	
			11.05	30	1506	0.00	1525	0.00	
			11.15	40	1506	0.00	1525	0.00	
			11.35	60	1506	0.00	1525	0.00	
			11.55	80	1506	0.00	1525	0.00	
			12.00	120/0	1506	0.00	1525	0.00	Loading
	200	2.5/5	12.05	5	1500	0.15	1520	0.13	
			12.10	10	1498	0. 20	1518	0.18	
			12.20	20	1497	0.23	1518	0.18	
			12.30	30	1497	0.23	1518	0.18	
			12.40	40	1497	0.23	1518	0.18	
			13.00	60	1497	0.23	1518	0.19	
			13.20	80	1497	0.23	1518	0.18	
			14.00	120/0	1497	0.23	1518	0.18	Loading
	300	5/7.5	14.05	5	1490	0.40	1516	0.23	
			14.10	10	1485	0.53	1515	0.25	
			14.20	20	1485	0.53	1514	0.28	
			14.30	30	1485	0..53	1514	0.28	
			14.40	40	1485	0.53	1514	0.28	
			15.00	60	1485	0.53	1514	0.28	
			15.20	80	1485	0.53	1514	0.28	
			16.00	120/0	1485	0.53	1514	0.28	Loading
	400	7.5/10	16.05	5	1481	0.63	1512	0.33	
			16.10	10	1481	0.63	1510	0.38	
			16.20	20	1481	0.63	1510	0.38	
			16.30	30	1481	0.63	1510	0.38	
			16.40	40	1481	0.63	1510	0.38	
			17.00	60	1481	0.63	1510	0.38	
			17.20	80	1481	0.63	1510	0.38	
			18.00	120/0	1481	0.63	1510	0.38	Loading
	500	10/12.5	18.05	5	1478	0.70	1505	0.50	
			18.10	10	1476	0.75	1504	0.53	
			18.20	20	1476	0.75	1503	0.55	
			18.30	30	1476	0.75	1503	0.55	
			18.40	40	1476	0.75	1503	0.55	
			19.00	60	1476	0.75	1503	0.55	
			19.20	80	1476	0.75	1503	0.55	
			20.00	120/0	1476	0.75	1503	0.55	Loading

Table C-8 Cont

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
	Jack pressure	Load in tons			Gauge G1		Gauge G2		
					Reading	S.ment (mm)	Reading	S.ment (mm)	
			20.00	0	1476	0.75	1503	0.55	Loading
	600	12.5/15	20.05	5	1474	0.80	1500	0.63	
			20.10	10	1474	0.80	1497	0.70	
			20.20	20	1474	0.80	1497	0.70	
			20.30	30	1474	0.80	1497	0.70	
			20.40	40	1474	0.80	1497	0.70	
			21.00	60	1474	0.80	1497	0.70	
			21.20	80	1474	0.80	1497	0.70	
			22.00	120/0	1474	0.80	1497	0.70	Loading
	700	15/17.5	22.05	5	1470	0.90	1495	0.75	
			22.10	10	1468	0.95	1493	0.80	
			22.20	20	1466	1.00	1493	0.80	
			22.30	30	1466	1.00	1493	0.80	
			22.40	40	1466	1.00	1493	0.80	
			23.00	60	1466	1.00	1493	0.80	
			23.20	80	1466	1.00	1493	0.80	
			24.00	120/0	1466	1.00	1493	0.80	Loading
	800	17.5/20	24.05	5	1460	1.15	1490	0.88	
			24.10	10	1452	1.35	1488	0.93	
			24.20	20	1450	1.40	1487	0.95	
			24.30	30	1450	1.40	1485	1.00	
			24.40	40	1450	1.40	1485	1.00	
			1.00	60	1450	1.40	1485	1.00	
			1.20	80	1450	1.40	1485	1.00	
			2.00	120/0	1450	1.40	1485	1.00	Loading
	900	20/22.5	2.05	5	1435	1.78	1480	1.13	
			2.10	10	1431	1.88	1476	1.23	
			2.20	20	1428	1.95	1476	1.23	
			2.30	30	1427	1.98	1476	1.23	
			2.40	40	1427	1.98	1476	1.23	
			3.00	60	1427	1.98	1476	1.23	
			3.20	80	1427	1.98	1476	1.23	
			4.00	120/0	1427	1.98	1476	1.23	Loading
	1000	22.5/25	4.05	5	1420	2.15	1461	1.60	
			4.10	10	1413	2.33	1460	1.63	
			4.20	20	1413	2.33	1458	1.68	
			4.30	30	1413	2.33	1456	1.73	
			4.40	40	1413	2.33	1456	1.73	
			5.00	60	1413	2.33	1456	1.73	
			5.20	80	1413	2.33	1456	1.73	
			600	120/0	1413	2.33	1456	1.73	Loading

Table C-8 **Cont**

Date	Load		Time	Elapsed time Minutes	Settlement records				Remarks
					Gauge G1		Gauge G2		
	Jack pressure	Load in tons			Reading	S.ment (mm)	Reading	S.ment (mm)	
			6.00	0	1413	2.33	1456	1.73	Loading
	1100	25/27.5	6.05	5	1401	2.63	1450	1.88	
			6.10	10	1396	2.75	1446	1.98	
			6.20	20	1390	2.90	1444	2.03	
			6.30	30	1387	2.98	1444	2.03	
			6.40	40	1387	2.98	1444	2.03	
			7.00	60	1387	2.98	1444	2.03	
			7.20	80	1387	2.98	1444	2.03	
			8.00	120/0	1387	2.98	1444	2.03	Loading
	1200	27.5/30	8.05	5	1381	3.13	1428	2.43	
			8.10	10	1370	3.40	1419	2.65	
			8.20	20	1368	3.45	1419	2.65	
			8.30	30	1368	3.45	1419	2.65	
			8.40	40	1368	3.45	1419	2.65	
			9.00	60	1368	3.45	1419	2.65	
			9.20	80	1368	3.45	1419	2.65	
			10.00	120/0	1368	3.45	1419	2.65	Loading
			10.05	5	1351	3.88	1403	3.05	
	1300	30/32.5	10.10	10	1348	3.95	1401	3.10	
			10.20	20	1348	3.95	1395	3.25	
			10.30	30	1348	3.95	1395	3.25	
			10.40	40	1348	3.95	1395	3.13	
			11.00	60	1348	3.95	1395	3.25	
			11.20	80	1348	3.95	1395	3.25	
			12.00	120/0	1348	3.95	1395	3.25	Loading
			12.05	5	1340	4.15	1374	3.78	
	1400	32.5/35	12.10	10	1328	4.45	1370	3.88	
			12.20	20	1324	4.55	1366	3.98	
			12.30	30	1324	4.55	1363	4.05	
			12.40	40	1324	4.55	1363	4.05	
			13.00	60	1324	4.55	1363	4.05	
			13.20	80	1324	4.55	1363	4.05	
			14.00	120/0	1324	4.55	1363	4.05	
			14. 02	2	1025	12.03	1053	11.80	
	1500	35/37.5	14.05	5	614/ 1611	22.30	580/ 1703	23.62	Reset 1611 (G1) 1703 (G2)
			14.10	10	1393	27.75	1452	29.80	
			14.20	20	987	37.90	1004	41.00	
			14.43	43					Unloading

APPENDIX- D

LOAD SETTLEMENT, TIME SETTLEMENT AND TIME LOAD CURVES OF PILE LOAD TEST

PILE LOAD TEST FOR AXIAL COMPRESSION
LOCATION: BONOSREE PROJECT, RAMPURA, DHAKA.
PILE NO - PP1, PILE TYPE: PRESTRESSED
PILE LENGTH: 7.5 M, PILE SECTION: 175 MM x 175 MM
DATE OF CASTING : 2/1/96.
DATE OF DRIVING : 5/2/96.
DATE OF TEST : 7/3/96.
TEST TYPE : ML
MAXIMUM APPLIED LOAD: 24 TONS
TEST METHOD : ASTM D-1143

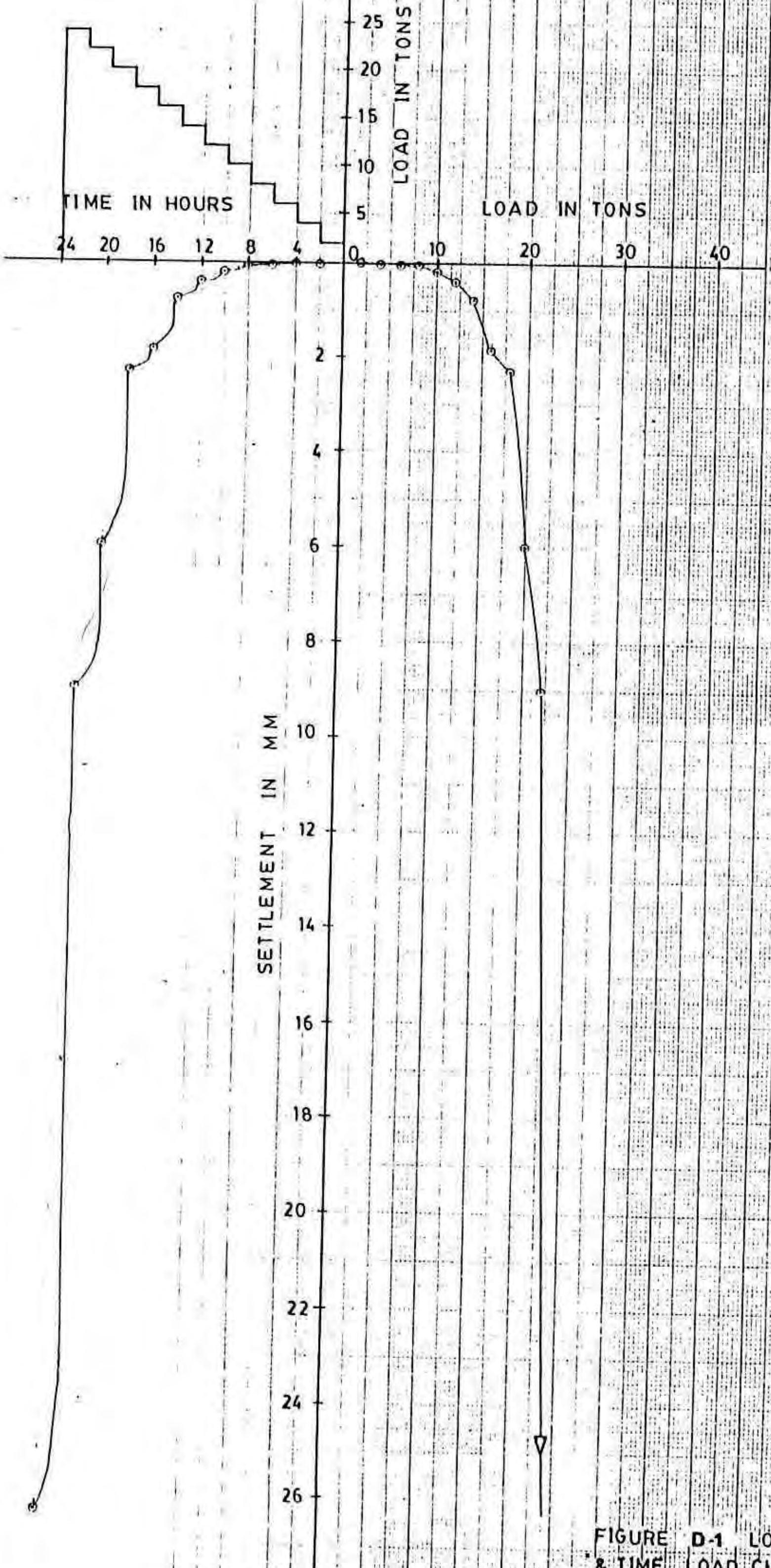


FIGURE D-1 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES OF PILE PP1.

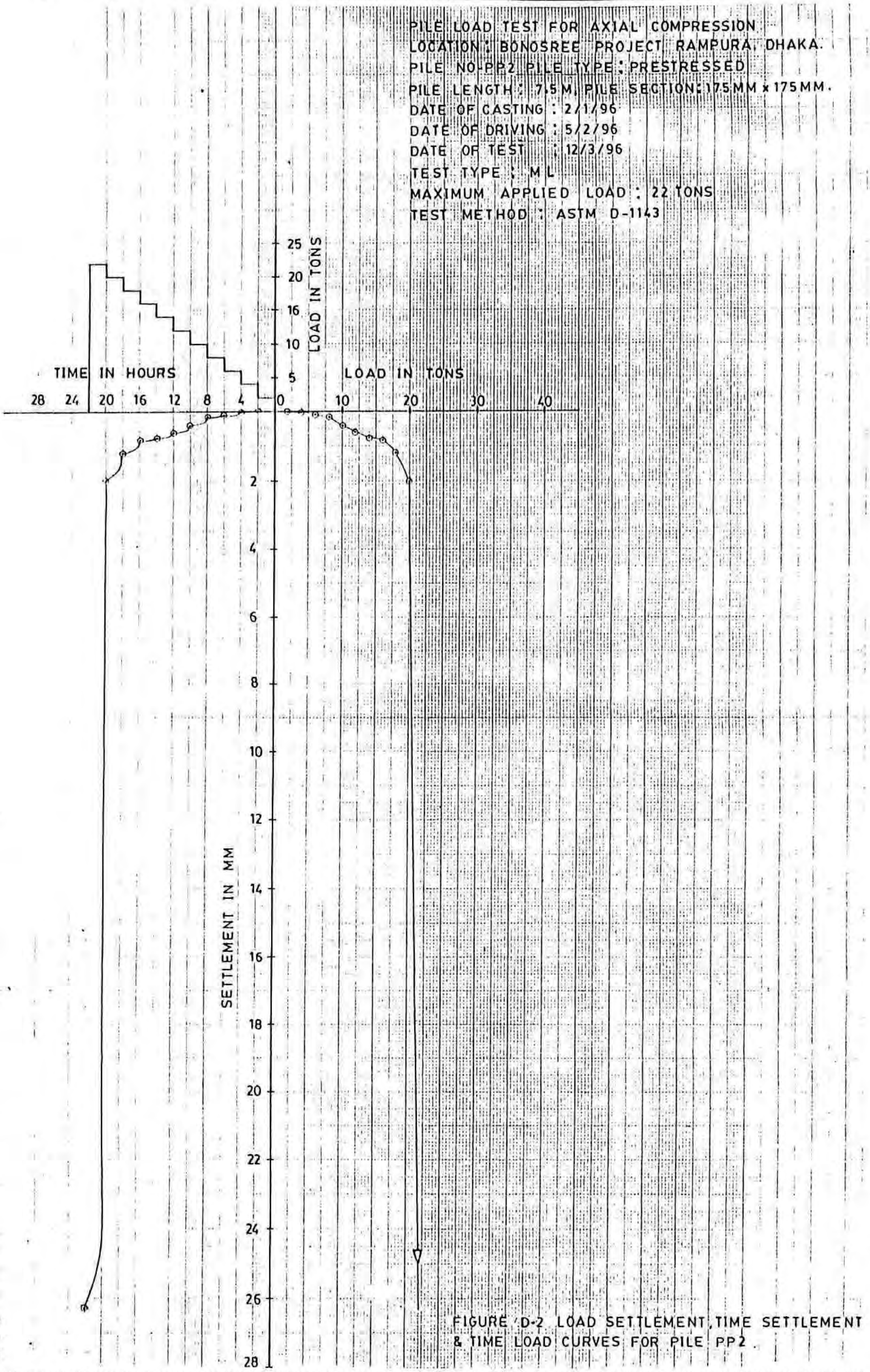


FIGURE D-2 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP2.

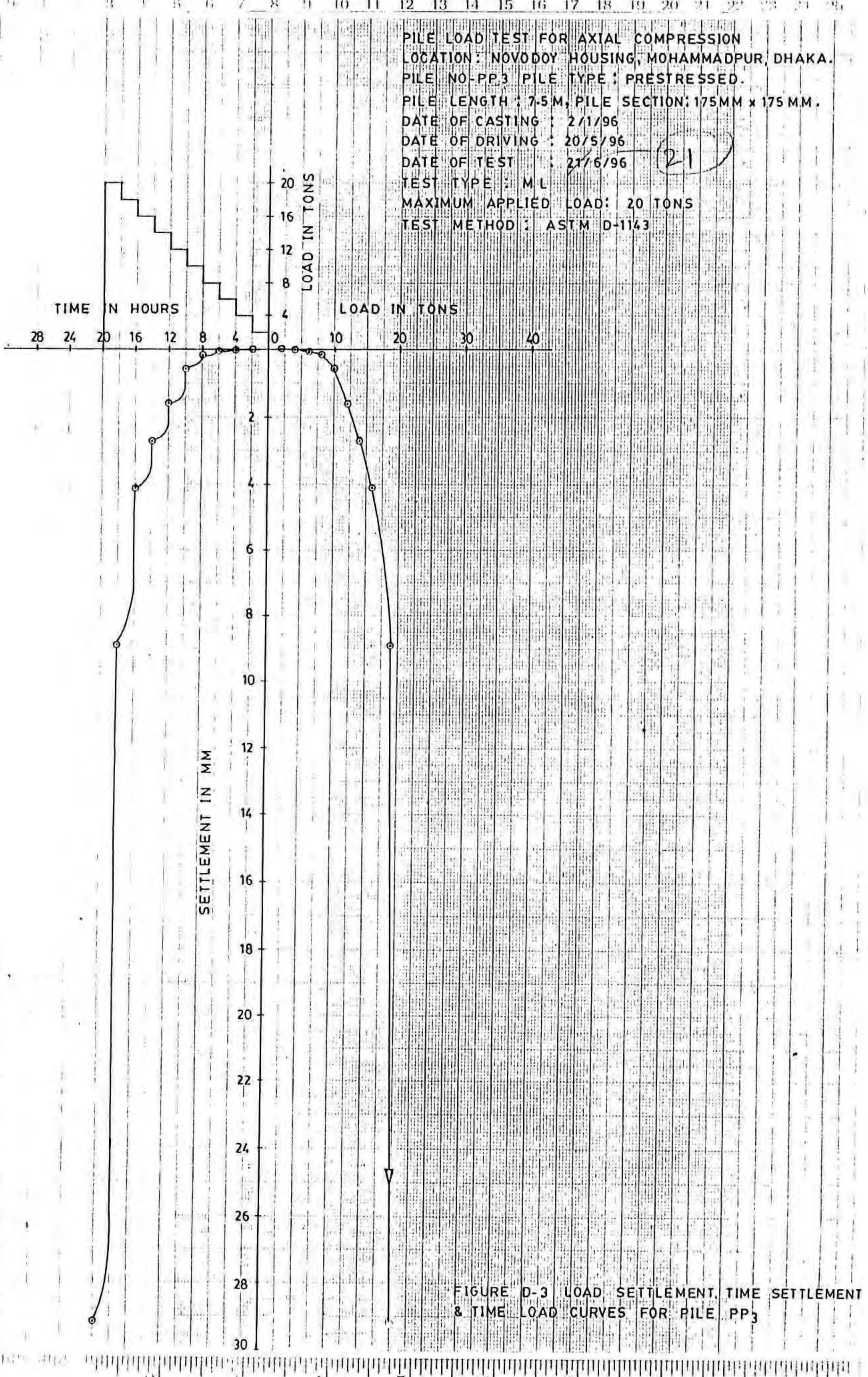


FIGURE D-3 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP_3

PILE LOAD TEST FOR AXIAL COMPRESSION
LOCATION : NOVODOY HOUSING, MOHAMMADPUR, DHAKA.
PILE NO- PP 4, PILE TYPE : PRESTRESSED.
PILE LENGTH : 7.5 M. PILE SECTION : 175 MM x 175 MM
DATE OF CASTING : 2/1/96
DATE OF DRIVING : 20/5/96
DATE OF TEST : 30/6/96
TEST TYPE : ML
MAXIMUM APPLIED LOAD : 20 TONS
TEST METHOD : ASTM D-1143.

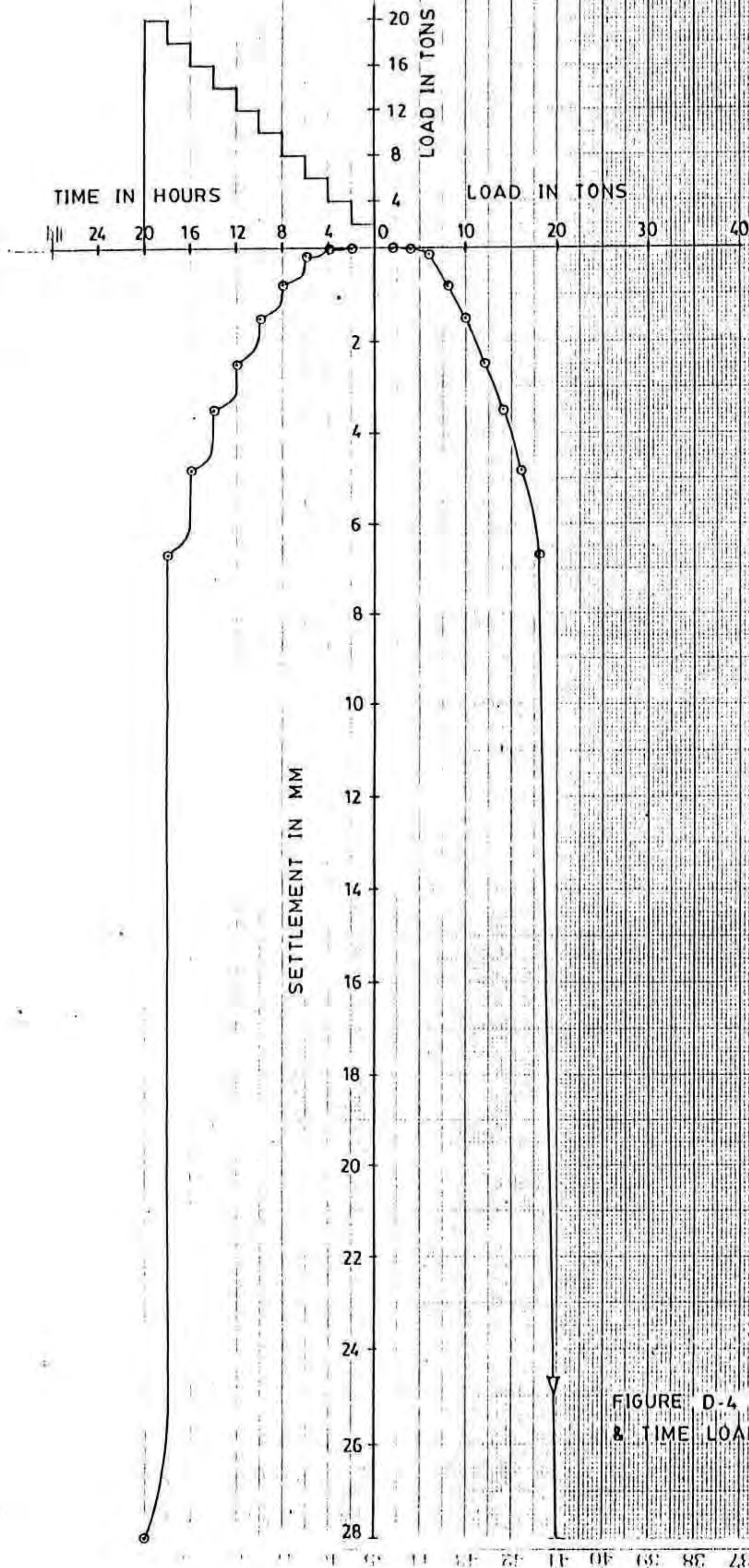


FIGURE D-4 LOAD SETTLEMENT TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP 4.

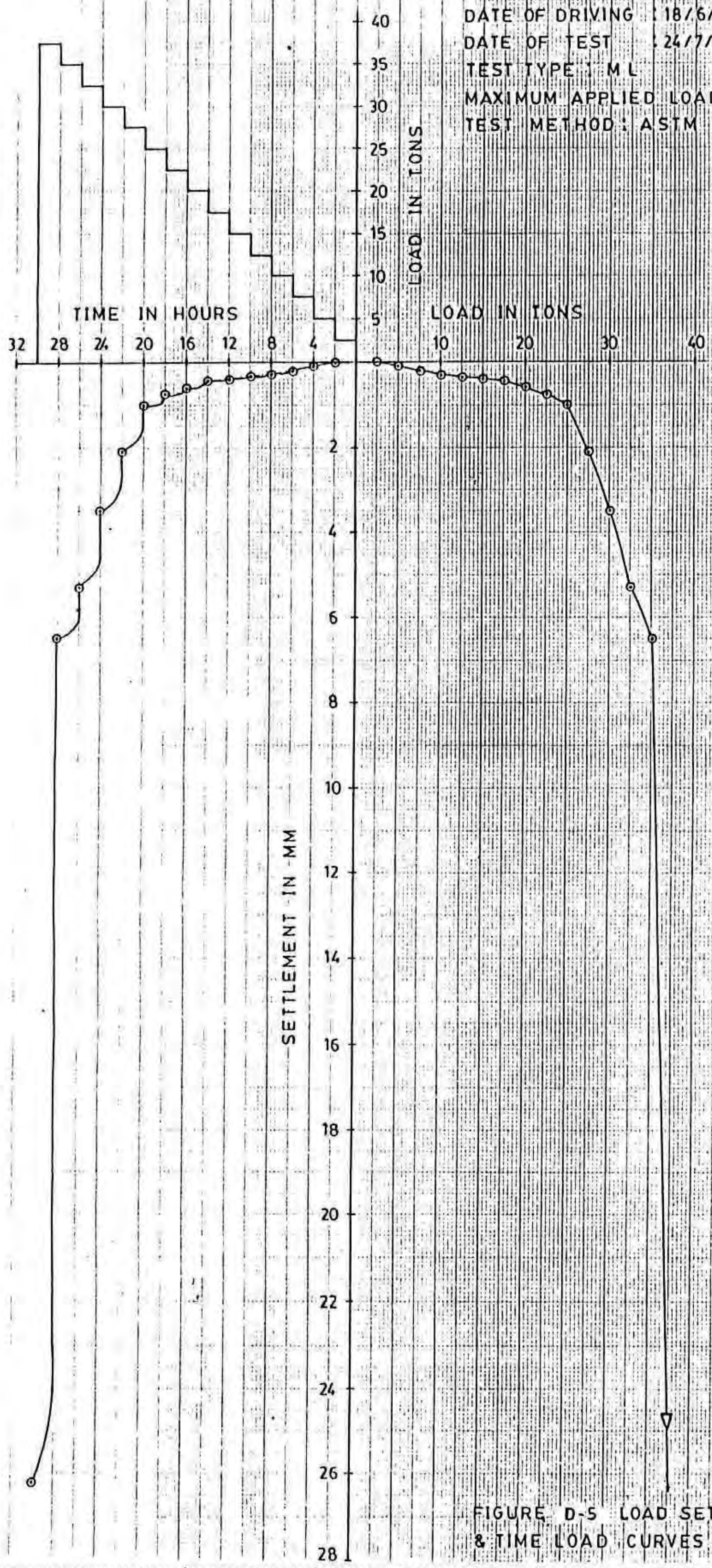


FIGURE D-5 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP 5.

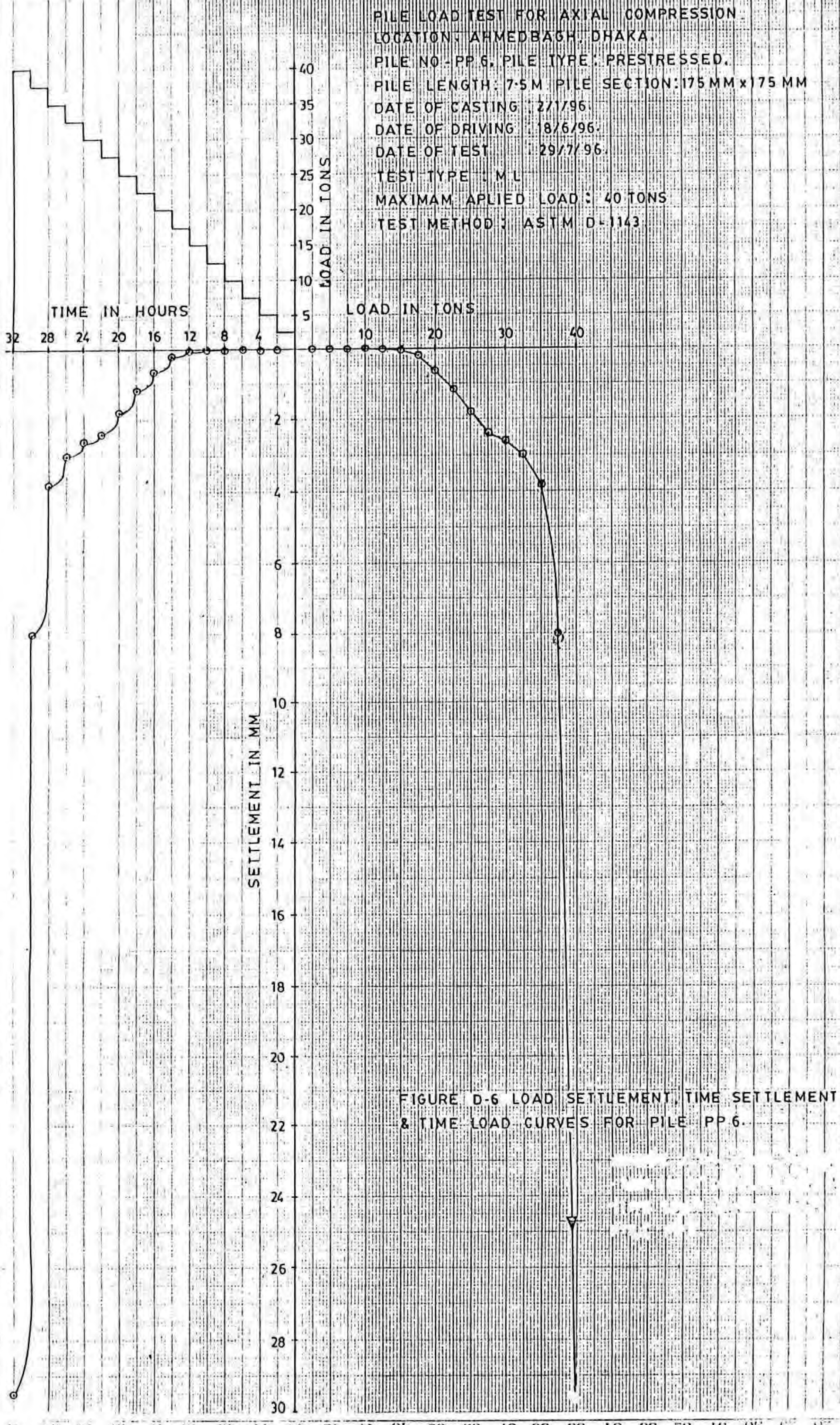


FIGURE D-6 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP 6.

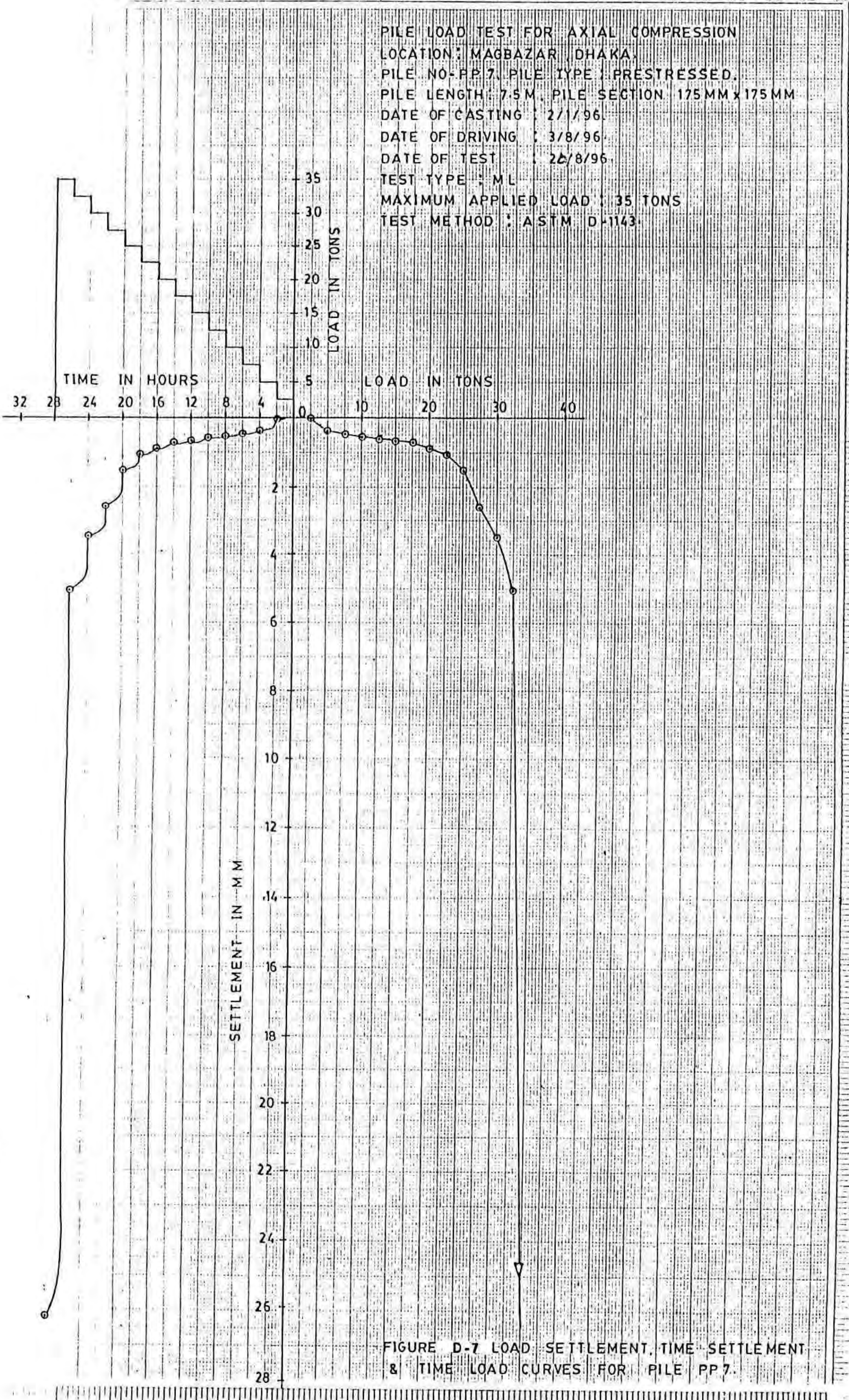


FIGURE D-7 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP 7.

PILE LOAD TEST FOR AXIAL COMPRESSION
LOCATION : MOGHBAZAR, DHAKA.
PILE NO- PP8, PILE TYPE: PRESTRESSED.
PILE LENGTH : 7.5M, PILE SECTION :175MM x 175MM
DATE OF CASTING : 2/1/96.
DATE OF DRIVING : 3/8/96.
DATE OF TEST : 29/8/96.
TEST TYPY : ML
MAXIMUM APPLIED LOAD : 37.5 TONS
TEST METHOD : ASTM D-1143.

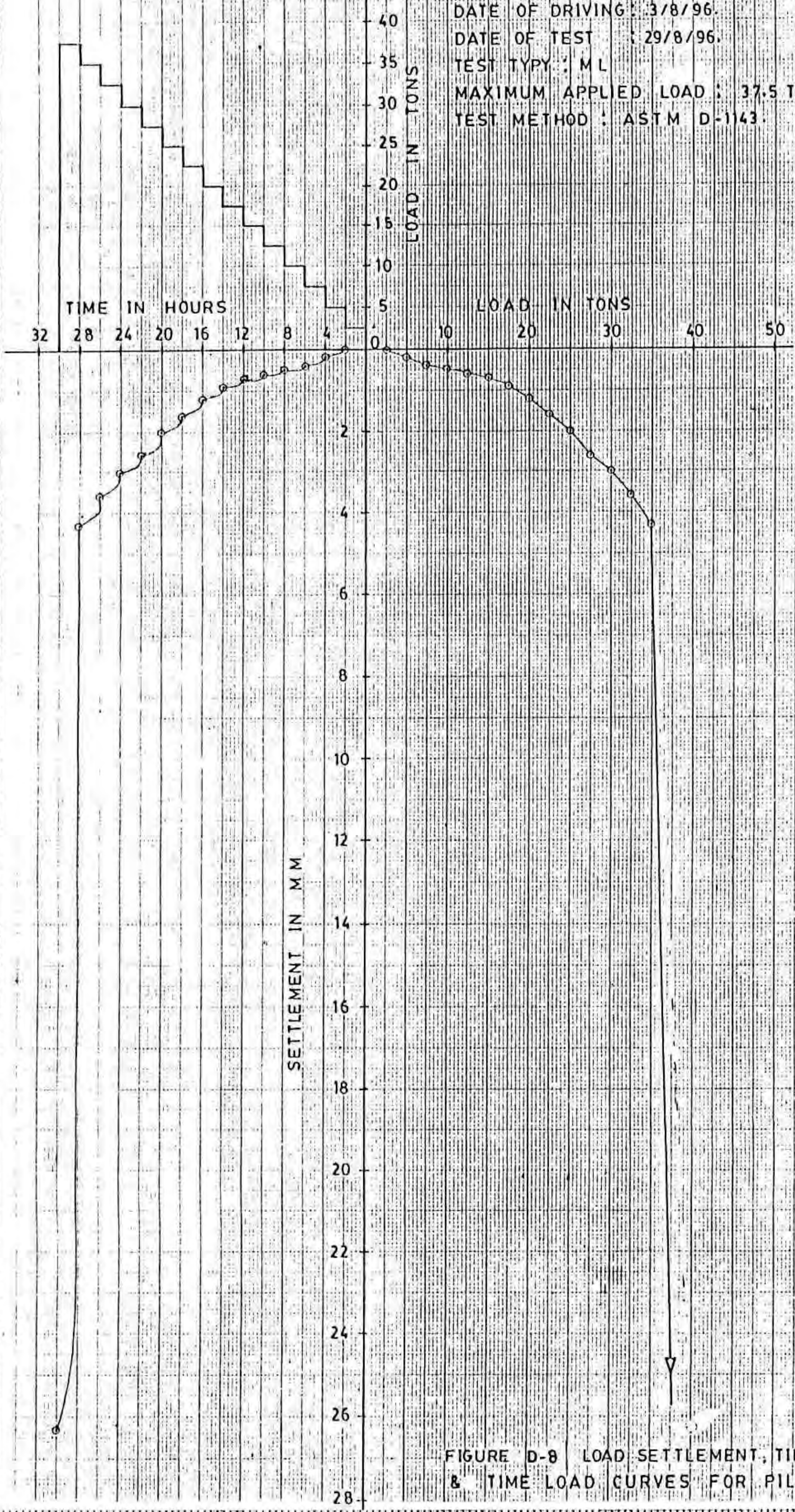


FIGURE D-8 LOAD SETTLEMENT, TIME SETTLEMENT & TIME LOAD CURVES FOR PILE PP8.

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