Study on the Field Performance of Granular Pile as a Ground Improvement Technique

by

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Declaration

This is to certify that this thesis work entitled "Study on the Field Performance of Granular Pile as a Ground Improvement Technique" has been carried out by Md. Ikramul Hoque in the Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna, Bangladesh. The above research work or any part of this work has not been submitted anywhere for the award of any degree or diploma.

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Approval

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Affectionately dedicated

To

My beloved mother, for her love

and

my wife and son, for their continuous

support and encouragement.

ABSTRACT

Ground improvement by granular piles is considered as one of the versatile and cost effective ground improvement method. Installation of granular piles transforms the soils into a stiffer composite mass with intervening native soil proving lower overall compressibility and higher shear strength. In the recent years this technique has also been adopted in Bangladesh in various projects to improve the marginal sites. The installation technique has the big influence on the performance of this ground improvement method. There is a record of successful application of rammed-displacement method in the installation of granular piles in the soft ground of Bangladesh.

This study concerns on the field performance of granular pile installed in soft ground of Bangladesh. For this investigation a typical soft ground site at KUET campus, Khulna is considered. The sub-soil of KUET campus consists of fine-grained soil of very soft to soft consistency up to the great depth. The more one sand pile installation in this study, sylhet sand. Granular piles of 300 mm diameter and 8.25 m length were installed in single. Load tests were conducted on 0.30m diameter plate resting at a depth of 0.91m from the existing ground surface on both the natural and improved ground. The plate load test were conducted on the natural ground and on the top of sand piles after one month and one year of installation of granular piles. Standard penetration tests were also conducted to observe the change of soil stiffness due to the installation of sand pile.

The test results reveal that the bearing capacity of the normal ground was increased by the installation of granular piles. Comparing to the natural ground, the bearing capacity of improved ground was increased by 200% to 250%. The cost of mat and pre-cast pile foundation increased approximately 114% and 154% respectively while compared with sand pile. Field investigation reveals that the granular pile made-up of locally available granular materials and employed installation technique can be used successfully as a suitable ground improvement method to improve the bearing capacity of such soft ground.

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Nomenclature

All the notation and symbols are defined where they first appear in the next or figures. For convenience, the more frequently used symbols and their meanings are listed below.

A	: area
a_s	: area replacement ratio
a_v	: coefficient of compressibility
B	: width
β	: inclination of the failure surface as given by equation
C_c	: compression index
C_t	: temperature correction
C_v	: coefficient of consolidation in the radial distance
D	: grain size, depth, diameter, distance
E	: modulus of elasticity of the clay
e	: void ratio
e_o	: initial void ratio
G_s	: specific gravity of soil particles
H	: height, thickness
H_1H_2	: height
k	: coefficient of passive resistance of granular material
L	: length, distance
W_1	: liquid limit
m_v	: coefficient of volume compressibility
OMC	: optimum moisture content
OC	: organic content
P	: pressure, load

W_p	: plastic limit
I_p	: unconfined compressive strength
q_u	: unconfined compressive strength
r	: radius
S_r	: degree of saturation
S_u	: undrained shear strength
T_v	: time factor
t	: time
u	: pore pressure
W	: weight of soil sample
W	: moisture content
W_d	: dry weight
z	: depth of the clay
γ	: bulk density, unit weight
γ_c	: saturated or wet unit weight of the cohesive soil
γ_d	: maximum dry density
Δ_σ	: deviator stress
σ_3	: average lateral confining pressure
σ_c	: stress in the surrounding cohesive soil
σ_r	: passive resistance of the soil
ϕ	: angle of shearing resistance
μ	: poisson's ratio
$\in_a$	: axial strain

CHAPTER ONE

INTRODUCTION

1.1 General

Construction of granular piles are considered as one of the most versatile and cost effective ground improvement technique compared to the other methods such as preloading, dredging and replacement, dynamic compaction etc. Granular piles, a fairly recent ground improvement technique, have been used successfully in several projects throughout the world. Granular piles are generally cylindrical in shape and composed of compacted gravel, crushed stone or sand. Various techniques of installation have been conceived depending on technical ability, efficiency and local condition. In Europe and USA, the vibro-floatation technique is widely used while in Japan, the vibro-compozer method is used. In India, granular piles are constructed by simple bored piling equipment.

In this study field investigation are carried out on the construction of granular piles by rammed-displacement and its various aspects. Granular piles with different types of granular materials were constructed in a typical soft ground of south western region of Bangladesh. Load tests were done over the constructed granular piles and the results show that the bearing capacity of the improved ground increased by 2.00 to 2.50 times than that of the natural ground.

1.2 Historical Background

One of the oldest historical examples of the use of granular piles (stone column) is found in the 1830's where French Military Engineers used it to support heavy foundations of iron works at the artillery arsenal in Bayoune (Hughes et al, 1975). The modern origin of the granular piles actually began 60 years ago in the 1930's in Germany by their Russian Émigré.

After the beginning of the modern phase of the use of granular piles, the theoretical background, analysis and design aspects, and installation techniques have been developed by various researchers and the practicing engineers. As a result in the recent years, this method of ground improvement has been proved to be a most popular versatile and cost effective technique.

The vibro-compaction method is used to improve the density of cohesionless soil using a vibroflot which sinks in the ground under its own weight and with the assistance of water and vibration (Baumann and Bauer 1974 and Engelhardt and Kirsch 1975). The vibro-replacement method is used to improve cohesive soils with more than 18% passing no. 200 U.S standard sieve. The vibroflot sinks into the ground under its own weight assisted by water or air jets as a flushing medium until it reaches the predetermined depth (Baumann and Baner 1974 and Engelhardt and Kirsch 1975). The vibro-compozer method is popularized in Japan and is used for stabilizing sofy clays in the presence of high ground water level (Aboshi et al 1979, Aboshi and Suematsu 1985, and Barksdale 1981). In cased borehole method, the piles are constructed by ramming granular materials in the prebored holes in stages using a heavy falling weight (usually of 15 to 20 KN) from a height of 1.0 to 1.5 m (Datye and Nagaraju 1975, Datye 1798, Datye and Nagaraju 1981 and Bergado et al. 1984).

A large number of laboratory and field tests have been conducted in order to quantity the applicability of this ground improvement technique to improve the behavior of soft ground. Hughes and Withers (1974) and Hughes et al. (1975) made a systematic study to identify the failure mechanism of single granular piles and suggested, bulging, as the mostly likely mode of failure. The findings of laboratory and field investigation on granular piles treated ground can be obtained from Madhav 1982, Charles and Watts 1983, Kimura et al. 1985, Mitchall and Huber 1985, Bergadeo and Lamb 1987, Juran and Guermazi 1988, Bergade et al. 1988, Leung and Tan 1993, Alamgir et al. 1995 and Fujimoto et al. 1995, In Bangladesh sand piles have been used successfully in some ground improvement projects. Alamgir and Zaher (1999a and 1999b) reported that a large number of sand piles were installed to soft cohesive soils in south-western region of Bangladesh which a six-vent regulator was constructed. Some research works were conducted in the department of civil engineering, Khulna University of Engineering & Technoloogy to see the response of soft ground improved by columnar inclusion as a part of post graduate research (Zaher 2000, Sobhan 2001, Hossain 2009).

1.3 Scope of this Study

Granular piles have become a common ground improvement technique for improving the marginal sites. This technique is suitable for very soft cohesive soil to loose deposits. Granular piles have already been installed in the soft soil regions of Bangladesh for the improvement of marginal sites. In Bangladesh dry-displacement method is employed, another practices of column installation i.e wet-replacement method with the help of locally available SPT arrangement can be used in Bangladesh. This method will be easier and cost effective than the dry-displacement counterpart. Once the field performance in increasing the bearing capacity of this technique is established, this ground improvement technique can be used for soft ground specially for small projects. Since this installation technique is simple, manual labour oriented and required instrument is available locally, the practicing engineers can suggest the client to adopt this ground improvement technique to improve the soft ground and construct the structure on it. This study will be very helpful to verify the applicability of this method in case of soft fine-grained deposits in Khulna regions.

1.4 Objectives of this Study

The main objectives of this research work can be outlined as:

i) To evaluate the effectiveness of locally available installation method for the condition of sand compaction pile in soft soil condition.

ii) To determine the degree of improvement of the bearing capacity of soil due to the installation of granular piles by comparing the load settlement response obtained in the natural and the treated grounds.

iii) To study the improvement of the ground along the depth by comparing strength profiles of the ground before and after improvement.

iv) To evaluate the response of ground improvement with elapsed period after the installation of sand pile.

1.5 Layout of this study

This study is composed of six chapters. The first chapter describes the introduction of the study. The historical background, scope and findings of this study are presented. A literature review of the study is summarized in chapter two. The soft ground, ground improvement technique, case studies are described in brief. Chapter three contains the statement of the problem, the development and fabrication of the equipment, site condition, selection of improvement technique, material of granular piles and lay-out of the granular piles. In chapter four, granular piles installation method and field test to measure the degree of improvement are depicted. Here installation technique, development of installation equipment, installation procedures, construction sequence and performed field tests are presented. Chapter five presents results obtained from field investigations and then the relevant discussion on this field investigations are also made. Summary and conclusion of this study are narrated in chapter six.

CHAPTER TWO
LITERATURE REVIEW

2.1 General

Granular piles such as stone columns, sand compaction piles etc, have been used as a ground improvement technique to increase the bearing capacity, reduce settlement, increase the rime of consolidation, improve stability and resistance to liquefaction of soft ground since 19^{th} century. However, the modern origin of this type of ground improvement technique truly began in 1930's in Germany and in 1950's in Japan. In the modern phase of the use of columnar inclusions, the theoretical background, analysis and design aspects and installation techniques have been developed by various researchers and practicing engineers and this method of ground improvement is being used extensively throughout the world for site improvement.

Amongst the various techniques for improving in situ soft ground conditions, granular piles are considered as one of the most versatile and cost effective ground improvement techniques. They are ideally suitable for the improvement of soft clays and silts and also for loose granular deposits. This chapter describes about the soft ground, ground improvement techniques, granular piles and the relevant topics. The present state of the art of the use of granular piles for the improvement of soft ground are specially described.

2.2 Soft Ground

The term "soft ground' has been used broadly, however, so far its meaning as engineering or technical term has not been defined clearly. In general, the following soil types are considered as the soft ground; (i) soft clayey soils, (ii) soils which have large fractions of particles as fine as silt, (iii) clayey soils which have high moisture content and (v) peat and sand deposit with a loose state under water table, Originally the concept of soft ground was mostly focused on the soils which are composed of clay deposit and have high moisture content, However, since the occurrence of the liquefaction phenomena in loose sand foundation during the earthquake, such deposit is now regarded as soft or problematic ground.

From a geological viewpoint, weak grounds which are accumulated naturally into alluvial layers in alluvial plants, swamps or man made lands which are reclaimed around the offshore areas, lakes and marshes are likely susceptible to formation as soft ground, The alluvial layers were accumulated in the latest geological and during these recent thousand years they were formed easily into soft ground, For artificial lands as observed from their geological age, they were formed during relatively recent years and mostly around the marine regions.

From a mechanical view point, soft grounds are soil deposits which have high compressibility but low strength. What regards as problems is due to what extents the weakness and deformation are. However, the determination of such parameters cannot be done clearly in the past since the soil responses are different for the applied methods and corresponding objectives. For example, small embankment or shallow excavation may be found on a ground, which does not cause any engineering problems. But if higher embankment or deeper excavation are to be executed on the same foundation, the excessive deformation may occur and cause a structural failure. In addition to this, it is certainly true that the limitations concerning ground characteristics are also significantly different depending on the allowable differential settlement and total deformation of the foundation for the structures. Furthermore, the ground may not cause any problem if the execution of an embankment is followed by other constructions in a slow process after the long span which allows the ground to become adequately stable. However, if the embankment has not to be constructed in a very short time, such problem as bearing capacity and consolidation at long-term will become serious problems. This along with the increasing trend to establish in large scale embankments using large equipment have made it necessary to treat the ground, even one which has favorable conditions, as soft ground.

2.3 Foundation Practice in Soft Ground

Foundation practice in soft soils depend on the index properties of the soil and the sub-soil report. Normally for soft soils raft or mat foundation, floating foundation, and transferring the load to the deeper hard strata by piles have been practiced for long time. These are all termed as conventional foundation system. For some better results soil may be replaced by good quality soil. Ground improvement techniques are adopted for soft soils for the construction of foundation at marginal projects. At present problem for soft soil foundation, stone columns and granular piles are widely used in many countries throughout the world.

2.4 Ground Improvement Techniques

To improve the physical and mechanical properties of the above mentioned soft ground, several ground improvement techniques have been and are being used since 19^{th} century. The different soil improvement methods can be classified into geometrical, mechanical, physical and chemical, and structural methods as follows depending on how the methods effect the stability or reduce the settlement (Broms 1987).

i. Geometrical methods: where the moment or force causing failure of excessive settlement is reduced; (a) Floating foundation and (b) Light weight fills.

ii. Mechanical methods: where the shear strength is increased or the compressibility reduced primarily by reducing the water content of the soil;

(a) Pre loading (often combined with vertical drains to increase the consolidation), (b) Lime piles and (c) Heating.

iii. Physical and Chemical methods: where the shear strength is increased and the compressibility of soft clay reduced by altering the clay-water system e.g. by freezing or by mixing the soil with lime, cement or other chemicals; (a) Lime or cement columns, (b) Electro-osmosis and (c) freezing.

iv. Structural methods: where structural elements such as geo-fabric, piles are used.

v. Sand, gravel or stone are used to reinforce the soil or to transfer the load to an underlying less compressible stratum or layer; (a) Geo-fabrics and geo-membranes, (b) Extension and replacement (c) Soil displacement, (d) Heavy tamping/Dynamic consolidation, dynamic replacement, and mixing, (e) Jet grouting, (f) Stone, gravel or sand columns, (g) Embankment piles and (h) Soil nailing.

From the beginning of the modern phase of ground improvement several techniques have been developed. Some commonly used ground improvement techniques are discussed in brief in the following sections.

2.4.1 Preloading

Preloading is a temporary loading applied at a construction site to improve subsurface soils. Preloading is sometimes called precompression or surcharge. Preloading increase the bearing capacity and reduces the compressibility of weak ground by forcing loose cohesionless soils to densify clayey, silty soils to consolidate. In the case of building, the preloading would normally be equivalent or higher than the expected bearing pressure.

2.4.2 Deep densification of cohesion less soils

In-situ deep densification of loose cohesion less soil layers is usually done by dynamic methods. In many methods, dynamic loading is accompanied by displacement in the form of the insertion of a probe or construction of a sand or gravel column in-situ methods used for the in-situ deep densification of cohesion less soils include blasting, vibro-compaction, heavy tamping. Vibro-compaction includes all those methods involving the insertion of vibrating probes in to the ground without the addition of a back fill material.

2.4.3 Densification of soft soils

Settlement resulting from the long-term consolidation of cohesion less soils create serious problems in foundation engineering. As the consolidation process is governed by the rate of excess pore-pressure dissipation, shortening the length of the pore water flow paths which greatly reduce the consolidation time. Vertical drains are artificially created drainage paths installed for the purpose of shortening drainage paths. Until a few years ago, vertical drains of sand were widely used. Present indications are that conventional sand drains is installed for the acceleration of consolidation may soon be things of the past as a variety of prefabricated drains are coming in to wide use.

2.4.4 Injection and grouting

Injection of material in to the ground has developed in to a widely used method for soil stabilization and ground improvement. More recently injections have been used for ground strengthening and ground improvement control. Three methods for injections are possible viz. permeation, displacement and encapsulation. Permeation grouts are of two types, particulate grouts and chemical grouts. Chemical grouts offer the advantages over particulate grouts that they can penetrate smaller pores, they have a lower viscosity and there is a better control of the settings time.

2.4.5 Soil reinforcement

Of the method of soil improvement and ground strengthening, none have been so intensively suited and advanced in application in the past several years, as has soil reinforcement. Basically there methods involves the in-situ inclusion of reinforcing element in the ground to improve its engineering characteristics or to carry the load to a competent material. The six mostly used types of in-situ reinforcement are stone columns, soil nailing, micro piles, jet grouting, permanent anchors and geo-textiles.

2.4.6 Stone columns

The concept involves replacement of 10 to 35 percent of the weak soil with stone or sometimes with sand in the form of columns. Holes are created in the ground and then back filled with stone compacted by impact and vibration. The soil is, thus, transformed in to a stiffer composite mass of granular cylinder with intervening native soil providing lower overall compressibility and higher shear strength. To-date, stone column or granular piles have been used mainly to improve the bearing and reduce the settlement of foundations or to improve the stability of embankment and slopes.

2.5 Method of Selection of Ground Improvement Techniques

There are several different ground improvement techniques as mentioned above, each has its own advantages, limitations, and special applications. Therefore, none can be considered suitable for solution of every problems in all kinds of soils. For soft and cohesive on subsiding environments, ground improvement by reinforcement (i.e. stone columns or sand compaction piles), by admixtures (i.e. by deep mixing method) and by dewatering (i.e. vertical drains) are applicable. Since the domain of ground improvement is indeed very vast, it is often a difficult task to select a particular type of ground improvement technique. The selection of the most suitable one in any case can only be made after evaluation of several factors specific to the problem at hand.

A flow chart for selection of ground improvement techniques is given below (Fig. 2.1)

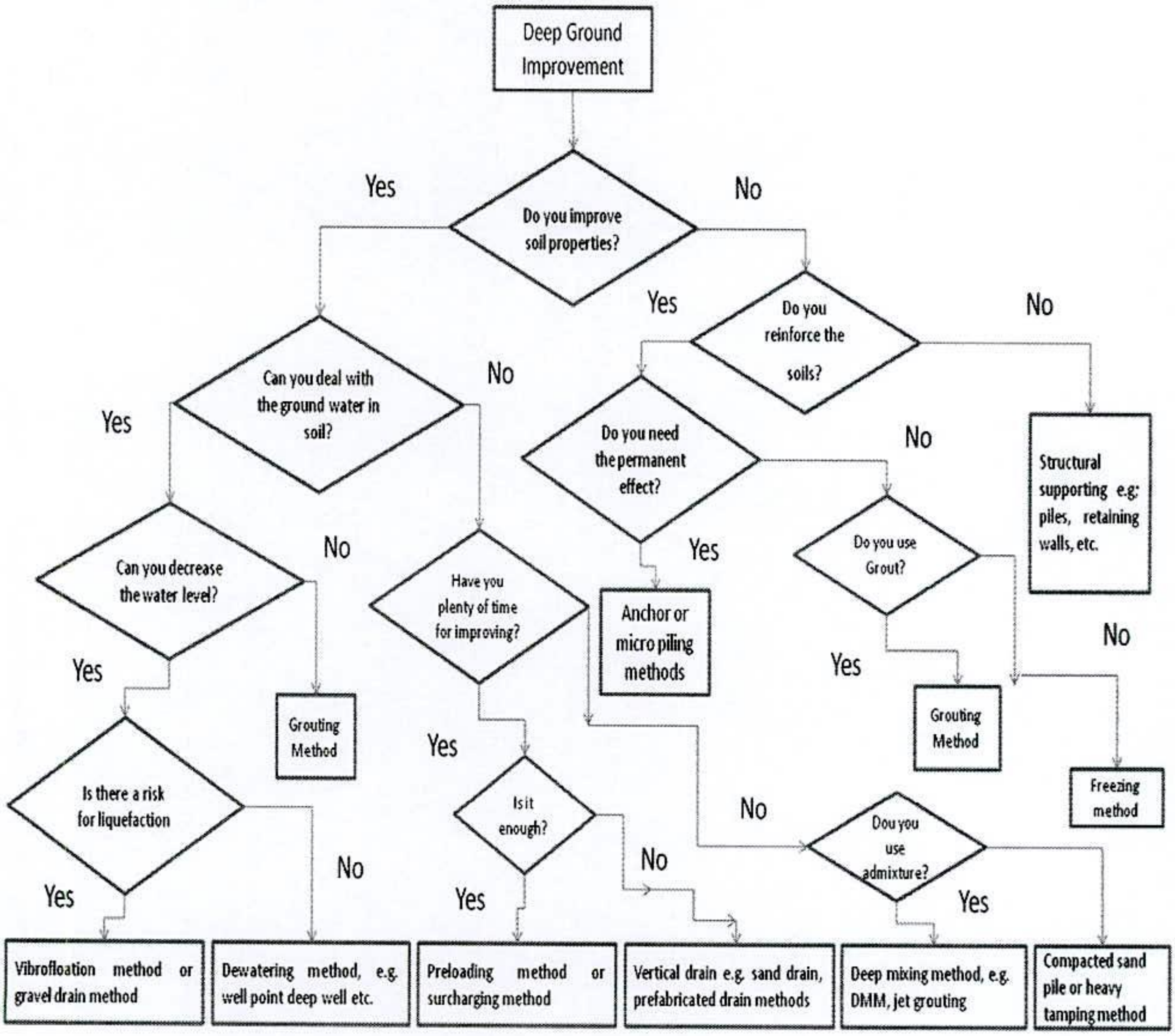


Fig. 2.1 Flow chart for selection of grond improvement techniques

(after Bergado and Miura 1994)

2.6 Columnar Inclusions

To utilize the marginal sites and to make the many problematic soils into useful construction sites, soil improvement has become a part of many present day civil engineering projects. The various techniques by which this improvement can be accomplished are discussed briefly in the previous sections. The flow chart to choose a ground improvement technique suitable for a particular project considering the associated situations are also discussed. Amongst the various ground improvement techniques for improving in-situ ground conditions, columnar inclusions is considered as one of the most versatile and cost effective ground improvement technique (Alamgir er al. 1995). The columnar inclusions can be of the form such as stone columns or granular piles, sand compaction piles, line or cement columns, etc., which are stiffer and stronger than the surrounding soil. The theoretical and constructional aspects of ground improvement by columnar inclusions have been developed intensively after the beginning of modern phase and have been used extensively throughout the world in the past several years. The horizon of applicability and the advantageous aspects of this method are wider than any other ground improvement technique. They are applicable for all types of soft soil ranging from soft clays to loose granular deposits. Installation techniques are ranging from mechanical equipment to sophisticated computerized one. As the other conventional ground improvement techniques such as preloading, dredging and soil displacement techniques can often no longer be used due to environmental restrictions and post construction maintenance expenses (Barksdale and Bachus 1983), the columnar inclusions can be treated as an ideal choice for today's soft ground improvement projects. The advantages of this method are (i) Moderate increase in load carrying capacity; (ii) Significant reduction of ground settlement; (iii) Granular columns being free draining, post-consolidation settlement will be small; (iv) Installation is relatively simple and low energy input or moderate labor; (v) Increase in resistance to liquefaction and (vi) Cost effective.

2.7 Sand Compaction Piles (SCP)

Sand Compaction Pile (SCP) is one of the in-situ soil improvement methods used in strengthening of soft clay. First pioneered around 1958, the technique has been much improved over the years by incorporating the latest technology for its installation and quality control. SCP is a system of large diameter (250 to 2000mm) well compacted sand piles driven into soft clay at regular intervals to improve its strength. The first project in Singapore to use this method was HDB's North East coast Reclamation Scheme Phase 3 in 1988. SCP was used again in much large scales in subsequent reclamation projects at Marina Bay and Tanjong Rhu. And, Most recently, it was used in part of the shoreline of the Taus Reclamation for the Malaysia-Singapore Second Crossing.

The depth, size and spacing of the sand piles have to he designed according to the soil and loading conditions. In the case of the North Eastern Coast project, the size of the pile was 0.8m at spacing varying from 1m to 1.5m to produce a area replacement ratio between 0.25 to 0.5. The depth of sand piles varied from 2m to 11m. For both the Marina Bay and Tanjong Rhu reclamation projects a much heavier system was used. Here 2m diameter piles at 2.1m spacing were driven to depth varying from 14.5m to 25.5m. Replacement ratio achieved was 0.7. In the Taus reclamation project again 2m diameter piles were driven but at a bigger spacing 3.2m to achieve a replacement ratio of 0.3. Average penetration depth was 4m.

2.8 Installation Techniques

Various installation techniques for installation of granular piles have been used all over the world depending on their proven applicability and availability of equipment in the locality. The following common installation techniques are used all over the world as common practice.

(i) Vibro-displacement method.
(ii) Vibro-replacement method
(iii) Vibro-compozer method
(iv) Cased-borehole method
(v) Rammed-displacement method

A brief account of this installation techniques are described below.

2.8.1 Vibro-displacement method

The vibro-displacement method is a dry process. The dry technique is situated for partially sutured soils saturated soils that can stand unsupported, specially those that will densify as a result of lateral vibration. In vibro-displacement method the vibrated hole must be able to stand open upon extraction of the probe. It is recommended that the vibro-displacement to be possible soils must exhibit untrained shear strength in excess of 40 to 60 kPa (850 to 125psf), with a relatively low ground water table being present at the site (Barksdale & Bachus 1983).

2.8.2 Vibro-replacement method

The vibro-replcement method is used to improving cohesive soils with the more than 18% passing no.200 U.S. standard sieve. The vibro float sinks into the ground under its own weight assisted by water or air jets as a flushing medium until it reaches the predetermined depth (Baumann and Bauer, 1974).

2.8.3 Vibro- compozer method

The method is popularized in Japan and is used for stabilizing soft clays in the presence of high ground water level (Aboshi et al. 1979, Aboshi and Seumatsul 1985, and Barksdale 1981). The resulting pile is usually termed as sand compaction pile. The sand compaction piles are constructed by driving the casing pipe to the desired depth using a heavy, vertical vibratory hammer located at the top of the pipe. The casing is filled with specified volume of sand and the casing is then repeatedly extracted and partially redriven using the vibratory hammer starting from the bottom. The process is repeated until a fully penetrating compacted granular piles is constructed.

2.8.4 Cased-borehole method

In this method, the piles are constructed by remaining granular materials in the prebored holes in stages using a heavy falling weight (usually of 15 to 20KN) from a height of 1.0 to 1.5m (Datye and Nagaraju 1975, Datye and Nagaraju 1981 and Bergado et al. 1984). The method is a good substitute for vibrator compaction considering its low cost. However, disturbance and subsequent remolding by the ramming operation may limit its applicability to sensitive soils. The method is useful in developing countries utilizing only an indigenous equipment in contrast to the methods described above which require special equipment and trained personnel (Ranjan and Rao 1983 and Ranjan 1989).

2.8.5 Rammed-displacement method

The rammed-displacement method is a dry process. The dry technique is situated for partially sutured soils saturated soils that can stand unsupported, specially those that will densify as a result of vertical ram hammering. The resulting pile is usually termed as sand compaction pile. The sand compaction piles are constructed by driving the casing pipe to the desired depth using a heavy, vertical rammed hammer located at the top of the pipe. The casing is filled with specified volume of sand and the casing is then repeatedly extracted and partially redriven using the rammed hammer starting from the bottom. The process is repeated until a fully penetrating compacted granular piles is constructed.

2.9 Sensitivity of soft soil

A cohesive soil in its natural state of occurrence has a certain structure when the structure is disturbed, the soil becomes remoulded, and its engineering properties change considerably. Sensitivity (S_t) of a soil indicates its weakening due to remoulding. It is defined as the ratio of the undisturbed strength to the remoulded strength, at the same water content,

$$S_t = \frac{(q_u)u}{(q_u)r}$$

Where $(q_u)u$= unconfined compressive strength of undisturbed clay

$(q_u)r$= unconfined compressive strength of remolded clay.

Depending upon sensitively, the soil can be classified into six types, as given in Table 2.1

Table 2.1 Classification of soils based on sensitivity.

Sl. no.	Sensitively	Soil type
1	<1.00	Insensitive
2	1.0-2.0	Little Sensitive
3	2.0-4.00	Moderately sensitive
4	4.0-8.00	Sensitive
5	8.4-16.0	Extra sensitive
6	>16	Quick

For most clays, sensitivity lies between 2 and 4, clays considered sensitive have S, values between 4 and 8, In case of sensitive clays, remolding causes a large reduction in strength. Quick clays are unstable. These turn into slurry when remolded.

High sensitivity in clays is due to a well-developed flocculent structure, which is disturbed when the soil is remolded. High sensitivity may also be due to leaching of soft glacial clays deposited in slat water and subsequently uplifted.

Extra-sensitive clay, such as clays of Mexico city, are generally derived from the decomposition of volcanic ash.

2.10 Effect of Sample Disturbance During Installation Technique

Disturbance may be defined as a change in fabric and moisture content of a soil which alters its physical properties. The physical properties of cohesive soils which can be affected by disturbance are (i) compressive strength (drained and undrained), (ii) modulus 'E' from compression tests (tangent or secant), (iii) shear modulus (iv) strain at peak strength, and (v) C_c, C_v, σ_p, 'C' and φ'. Disturbance does not affect all of these properties to the same degree in the same manner and that the degree of disturbance various in various part of the sample (Osterberg and Murphy 1979). In case of silty sands, disturbance affects the dynamic shear strength, But the drained shear strength of sands does not appear to be sensitive disturbance of soil fabric if density is unchanged (Mori and Koreeda 1979).

2.11 Design of Granular Piles

The performance of improved ground is best to investigated in terms of ultimate bearing capacity, settlement and general stability, In the following sections, basic relationships of the improved ground as well as failure mechanisms of granular piles on homogenous soft clay are first described and the ultimate bearing capacity of single and group granular piles in the improved ground based on experimental sand analytical studies are then presented.

2.11.1 Unit Cell Concept

The tributary area of the soil surrounding each granular pile is closely approximated by an equivalent circular area. For an equilateral triangular pattern of granular piles, the equivalent circle has an effective diameter of:

$D_e = 1.05S$

While for a square pattern,

$D_e = 1.13S$

Where S is the spacing of granular piles. The equilateral triangular pattern gives the most dense packing of granular piles in a given area. The resulting cylinder of composite ground with diameter De enclosing the tributary soil and one granular pile is known as the unit cell.

2.11.2 Area replacement ratio

Figure 2.2 Illustrates the area replacement as well as the stress concentration in the granular pile, The area replacement ratio is defined as the ratio of the granular pile area over the whole area of the equivalent cylindrical unit cell and expressed as

$$a_s = \frac{A_c}{A_s + A_c}$$

Where As is the horizontal area of a granular pile and A_c is the horizontal area of clayey ground surrounding the pile. The area replacement ration can also be expressed in terms of the diameter (D) and spacing (S) of the granular pile as follows:

$a_s = C_1(D/S)^2$

Square pattern $C_1 = \pi/4$ and for equilateral triangular pattern $C_1 = \pi/(2\sqrt{3})$

Where C_1 is a constant depending upon the pattern of granular piles used.

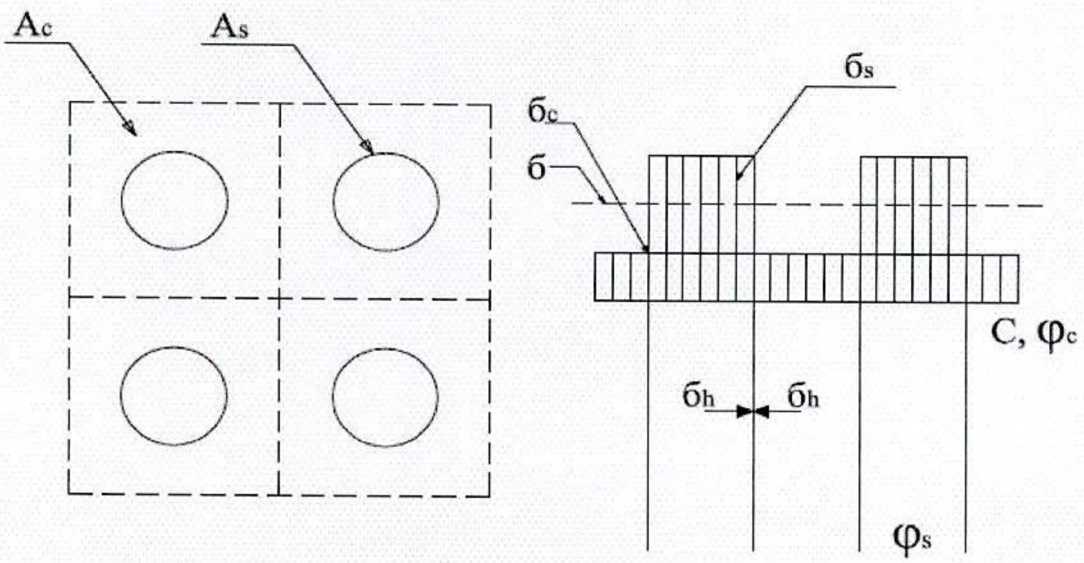


Fig. 2.2 Diagram of composit ground

When the composite ground is loaded, studies (e.g Greenwood, 1970; Aboshi et al. 1979; Goughnour and Bayuk, 1979, Balaam et al; 1977) indicated that concentration of stress occurs in the granular pile accompanied by the reduction in stress which occurs in the surrounding less stiff clayey soil (Fig. 2.2). This can be explained by the fact that, when loaded, the vertical settlement of the granular pile and the surrounding soil is approximately the same (Vautrain, 1977) causing the occurrence of stress concentration in the granular pile which is stiffer than the surrounding cohesive or loose cohesion less soil. The distribution of vertical stress within the unit cell can be expressed by a stress concentration factor defined as;

$\eta=\sigma_s/\sigma_c$

Where σ_s is the stress in the granular pile and σ_c is the stress in the surrounding cohesive soil. The magnitude of stress concentration also depend on the relative stiffness of the granular pile and the surrounding soil. The variation of stress concentration factor with area replacement ratio compiled by Bark less and Bachsu (1983), Vautarin (1977), Goughnour (1983) and Parsons and Co, Inc, (1980) ranged from 2 to 5. Meanwhile, Aboshi et al (1979) and Bergado et al. (1987) obtained higher stress concentration factor as much as 9. The higher value obtained by Bergador et al. (1987) was probably due to the high rigidity of the plates used during the load tests. From full scale test embankment observations on soft Bankok clay at low area replacement ratio of 0.06, the stress concentration factor of 2 was obtained and was found to decrease to 1.45 with the increasing applied load (Bergado et al, 1988). The average stress over the unit cell corresponding to a given area replacement ratio is expressed as;

$\sigma_s=\sigma_s a_s+\sigma_c(1-a_s)$

The stresses in the pile and the clay using the stress concentration factor are;

$\sigma_s=\eta\sigma/[1+(n-1)a_s]=\mu_s\sigma$

$\sigma_c=\sigma/[1+(n-1)a_s]=\mu_c\sigma$

Where μ_c and μ_s are the ratio of stress in the pile and clay, respectively, to the average stress over the unit cell area.

2.11.3 Failure mechanisms

In practice, granular piles are usually constructed fully penetration a soft soil layer overlying a firm stratum. It may be constructed also as floating piles with their tips embedded within the soft clay layer. Granular piles may fail individually or as a group. The failure mechanisms for a single pile are illustrated in Fig. 2.3 indicating the possible failure as: a) bulging, b) general shear, and c) sliding. For pile groups, additional failure mechanisms such as lateral spreading and shear failure across the granular pile cross-section may occur.

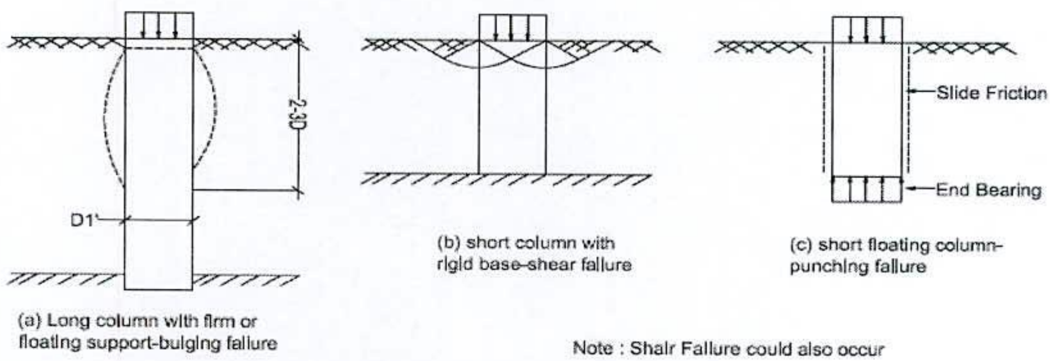


Fig. 2.3 Failure mechanism of a single granular pile in a homogeneous soft layer (after Barksdale and Bachus, 1983).

2.11.4 Ultimate bearing capacity of single and isolated granular pile

For single, isolated granular piles, the most probable failure mechanism is bulging failure. The mechanism develops whether the tip of the pile is floating in the soft or fully penetrating and bearing on a firm layer. The lateral confining stress which supports the granular pile is usually taken as the ultimate passive resistance which the surrounding soil can mobilize as the pile bulges outward. Most of the approaches in prediction the ultimate bearing capacity of a single, Isolated granular pile has been developed based on the above assumptions.

A relationship between ultimate bearing capacity and area replacement ratio is shown in Fig. 2.4

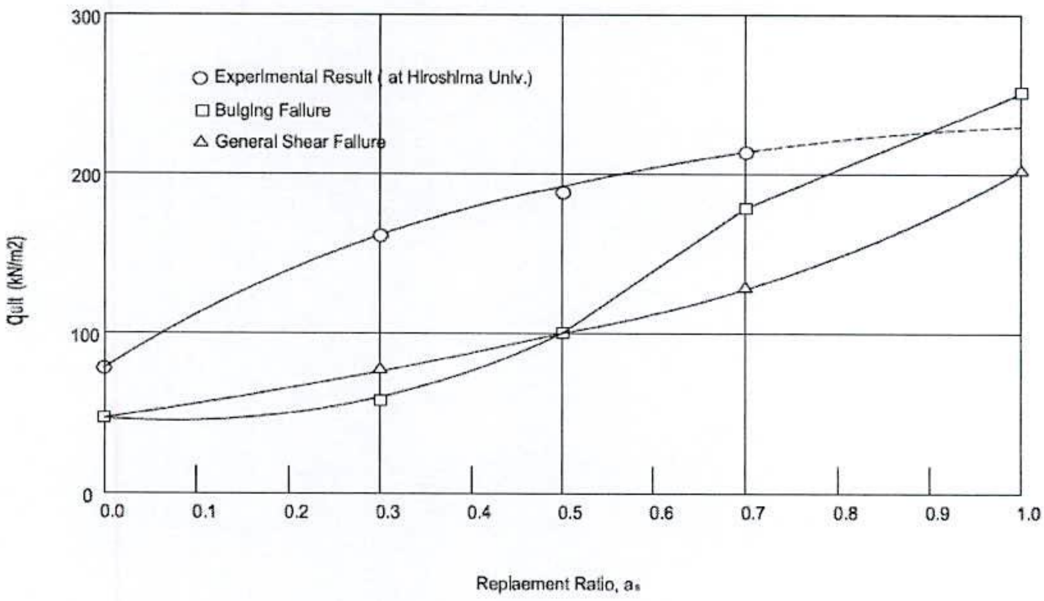


Fig.2.4 Relationship between ultimate bearing capacity and area replacement ratio (after Aboshi and Suemstsu, 1985).

$$\beta=45+\frac{\phi_{avg}}{2}$$

$$\phi_{avg} = \tan^{-1}(\mu_s a_s \tan\phi_s)$$

$$C_{avg}=(1-a_s)\ c$$

Where γ_c= saturated or wet unit weight of the cohesive soil; B= foundation width; β=failure surface inclination; c= undrained shear strength within the unreinforced cohesive soil; φ= angle of internal friction of the granular soil; ϕ_{avg}= composite angle of internal friction C_{avg}= composite cohesive on the shear surface.

The development of the above approach did not consider possibility of a local bulging failure of the individual pile. Hence, the approach is only applicable for firm and stronger cohesive soils having an undrained strengths greater than 30-40kN/m^2. However, it is useful for approximately deternining the relative effects on ultimate bearing capacity design variables such as pile diameter, spacing, gain in shear strength due to consolidation and angle of internal friction.

2.11.5 Ultimate bearing capacity of group granular pile

The common method estimating the ultimate bearing capacity granular pile groups assumed that the angle of internal friction in the surrounding cohesive soil and the cohesion in the granular pile are negligible. Furthermore, the full strength of both the granular pile and cohesive soil has been mobilized. The pile groups is also assumed to be loaded by rigid foundation The ultimate bearing capacity of granular pile groups as suggested by Barksdale and Bachus (1983) is determined by approximating the failure surface with two straight rupture lines as shown in Fig.2.5. Assuming the ultimate vertical stress, q_{ult} and the ultimate lateral stress, σ_3 to be principal stresses, then the equilibrium of the wedge requires:

$$Q_{ult}=\sigma_3 \tan^2\beta+2c_{avg} \tan\beta$$

Where

$$\sigma_3=\frac{\gamma_c \tan\beta}{2}+2c$$

For the case of the soft and very soft cohesive soils, the pile group capacity of a single, isolated pile located within a group and to be multiplied by the number for piles (Barksdale and Bachus, 1983). The ultimate bearing capacity for a single, isolated pile in this case is expressed as:

$$Q_{ult}= cN'_c$$

Where N'_c= composite bearing capacity factor for the granular pile which ranges from 18 to 22, For the soft Bangkok clay, N'_c ranges from 15 to 18 using an initial pile diameter of 25.4 cm with the gravel compacted by 0.16 ton hammer dropping 0.70m (Bergado and Lam, 1987)

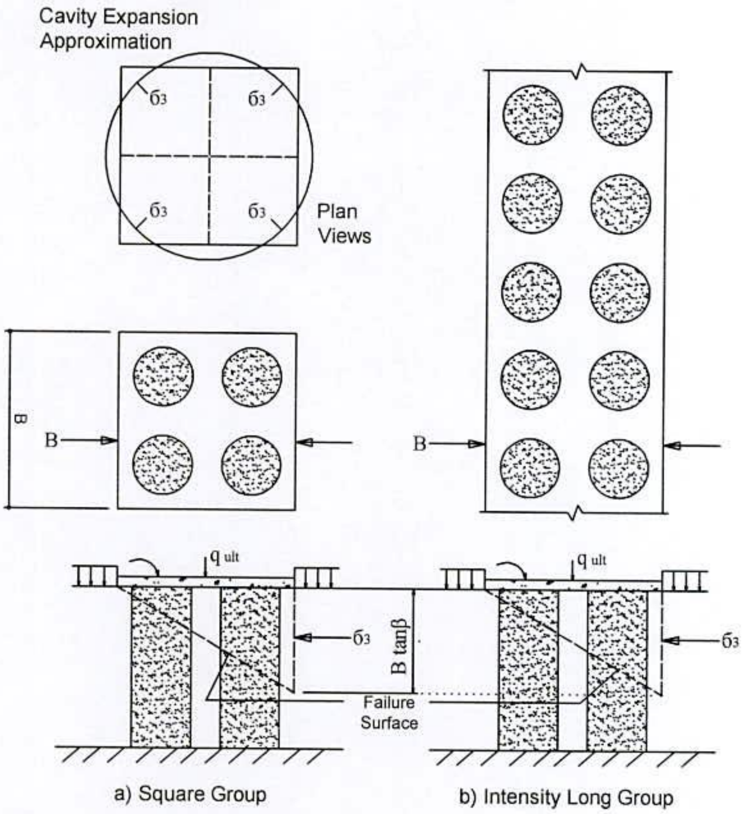


Fig. 2.5. Granular pile group analysis (after Barksdale and Bachus, 1983).

2.12 Experimental Investigations

Hughes et al. (1975) conducted a field test on a single stone column to investigate its performance and also to verify the theory proposed by Hughes and Withers (1974) on field scale. The column was constructed by vibro-replacement and, after the test, was excavated to check its dimensions. The cylindrical stone columns as installed were 10m long and 0.66m in diameter which was estimated on the basis of stone consumption. A standard site investigation supplemented by the Cambridge (Worth and Hughes 1973) and the Menard pressure meter tests, provided the basic soil parameters. The column was tested by loading a concentric circular plate of 0.66m diameter proved to be marginally smaller than the top of the column. The column improved substantially the bearing capacity of the natural soil. The method proposed by Hughes & Withers (1974) for calculating the ultimate load apparently under predicts by a surprisingly large amount. It was also observed that the prediction is excellent if allowance is made for transfer of load from column to clay through side shear and correct column size. They commented that the accurate estimate of the column diameter is the major factor influencing the calculation of ultimate load and the settlement characteristics.

McKenna et al. (1975) reported the lack of effectiveness of stone columns constructed by vibro-replacement technique, in reducing the settlement of a high trial embankments built on soft alluvium. The alluvium was 27.5m thick, the columns were 0.90m in diameter and 11.3m long, and they were constructed on a triangular grid at 2.4m centers. The embankment was built to a height of 7.9m. The instrumentation records showed that the columns had no apparent effect on the performance of the embankment. The reasons of no improvement are, as they stated, the grading of the granular materials was too coarse to act as a filter, and as a result, the void in the gravel backfill probably become filled with clay slurry which prevented them from acting as drains. In addition, the method of construction would probably have remolded the adjacent soft clays and damaged the natural drainage paths, nullifying any potential drainage provided by the stone columns. The backfill was so coarse that when the embankment load come on the column, the crushed stone forming each column was not restrained by the surrounding soft clay, and as columns expanded, the soft clay squeezed into voids.

Rao and Bhandari (1977) performed experimental investigation on single and group granular piles by skirting them at the top region to prevent the bulging and thus to increase the load carrying capacity. Therefore, bulging if at all possible, can occur below the depth of the skirt. From the results it was found that skirting the top of the piles up to a depth of 0.8m, prevented bulging of granular piles and increased the load carrying capacity by about 1.5 times compared with that of its unskirted counterpart.

Madhav (1982) presented two alternative approaches to prevent building in the top region of granular piles either by providing reinforcement in between the granular materials or replacing the top granular materials by the stiffer concrete plug. They prevent lateral strains and thus increasing the vertical load carrying capacity of the piles. The results of small scale model tests on reinforced granular piles indicate that large the number of reinforcement layers higher is the improvement in the load carrying capacity and the stiffness of the reinforced ground. Reinforcement ground. Reinforcement increased the load carrying capacity and the stiffness of the granular piles by about four times compared with its unreinforced counterparts. For the case of rigid plug, it was observed that if the top 15% to 30% of the length of pile is replaced, the load carrying capacity becomes 2to 4 times compared with that of the granular piles without rigid plug.

Instrumented large scale laboratory tests were performed by Charles and Watts (1983) to assess the effectiveness of granular columns in reducing the vertical compression of soft clay. The tests modeled the situation in which a soft-clay layer reinforced with fully penetrating columns is subjected to a widespread and relatively rigid load. Five tests were carried out to assess the effect of different column diameters on vertical compression. The details about the test conditions are given in Charles and Watts (1983) both columns and clay were instrumented so that stresses and strains could be monitored as the samples were loaded. The test results demonstrated the complexity of the soil behavior. It was found that the settlement reduction facto obtained using the approach of Balaam & Brooker (1981) differ significantly was in a state of failure, dilation took place and the principal stress ratio was at, or close to the peak value. With large diameter columns the behaviour of the gravel was quite different. There was a deduction in volume as the load was applied and the principal stress ratio was well below the peak value.

Kimura et al. (1985) conducted centrifuge tests to investigate the mechanical behavior of clay improved by sand compaction piles under inclined loading. Kawasaki clay was normally consolidated in a centrifuge and the model sand piles were installed. Loading tests were carried by applying lateral force to a model caisson placed on improved soil for four different combinations of lateral load to vertical load, The improvement by sand compaction piles increased the bearing capacity of clay by about 200 to 700% and it was extremely effective in reducing the lateral displacement of caisson. The postmortem studies of improved soil revealed that sand compaction piles were sheared when they were subjected to inclined load, It was found that for the improvement area ratio of 70% the caisson tended to move horizontally, while for the ratio of 33 to 55%, the caisson tilted even at the early stage.

Mitchell and Huber (1985) reported the performance of vibro-replacement stone columns used to support a large waste water treatment plant founded on up to 15m of estuarine deposits. Column spacing ranged from a 1.2 m x 1.5m pattern under the most heavily loaded areas, to a 2.1m × 2.1m pattern under lightly loaded areas. Twenty-eight single column load tests were done during the installation of 6,500 stone columns to evaluate load settlement behavior. The installation of stone columns lead to a reduction in settlement to about 30%-40% of the values to be expected on unimproved ground. Load test settlements calculated by the finite element method for the initial settlement conditions, using undrained clay properties and drained properties of sand and stone columns, are some what higher than the average settlements observed during actual field load tests conducted on similar stone column spacing patterns. However, the overall results obtained from the finite element analysis indicated reasonable agreement between the calculated and the observed settlement for the individual load tests. Settlement predications using several other, more simplified methods gave values that agreed reasonably well with both the finite element predictions and the measured values, This lends support to the use of the simple methods in practice.

Bergado and Lam (1987) investigated the behavior of granular piles on soft Bangkok clay with different densities and different proportions of gravel and sand. A total of 13 piles were installed with 0.30 m diameter and 8.0 m long using a non-displacement cased borehole method with 1.20m spacing in a triangular pattern. The completed diameter of the granular piles were 1.05to 1.35 times the initial diameter of the hole and varied progressively with depth. The piles were grouped into 5catagories. Group 1, 2 and 3 with 3 piles each. Were constructed using the sand compacted at 20, 15and 10 hammer blows per layer, respectively. Group 4 was made of gravel mixed with sand in the proportion of 1:0.30 by volume and group 5was constructed with gravel; both groups consisted of two piles and each was compacted at 15 blows per layer. The soil properties were investigated by the field vane and the pressure meter tests. The ultimate capacity of each granular pile was determined by using full scale plate loading tests. It was found that the ultimate bearing capacity increases with the density of column and the pure gravel column indicated higher capacity than that of the mixed counterparts. The pile made of mixed with 15 blows as layer (group 5) yielded the maximum ultimate pile capacity closely followed by the piles constructed out of sand with 20 blows per layer (Group 1). The deformed shape of the granular pile was found as of bulging type and the maximum bulge was observed to be at a depth of one pile diameter from the ground surface.

Bergado et al. (1988) reported the performance of full scale load test on a test embankment constructed on soft Bankok clay improvement by granular piles. The test embankment had a first stage height of 2.4m and subsequently raised to a second stage of 2.40m after 345 days. The case bore hole method and a hammer 1.6 KN of 0.60m falling height was used for the construction of piles. The friction angle and the compacted density of granular material varied from 39^0 to 45^0 and 17 to 8.1 $KN/m^{3,}$ respectively. Piles having diameter of 0.30m and length 8.0m, fully penetrating the soft clay layer, were arranged in a triangular pattern with a spacing of 1.5m. The granular materials consists of whitish-gray, poorly graded crushed lime stones with a maximum size of 20mm. A drainage blanket of 0.25m thick consisting of clean sand was laid on top of the compacted granular piles. It was observed that the granular piles increased the bearing capacity, reduced the settlement and increased the stability. The comparative study indicated that the embankment on granular on granular piles settled about 40% less than that constructed on vertical drains. These results indicated the effectiveness of granular piles over the vertical drains in improving the soft ground.

Leung and Tan (1993) implemented laboratory investigation of examine the load distribution and consolidation characteristics of a composite soil made up of a soft clay reinforced by a sand column. The marine clay used has a composition of 15% sand, 40% clay. The compacted sand used has a composition of 15% fine , 75% medium and 10% coarse and was compacted to a unit weight of 18 KN/m^3. The experiments were carried out in a circular steel tank of 1m diameter and 1m high, the column installed at the center rests on the rigid base and the loads were applied through a granular fill surcharge. Five tests were conducted with compacted sand columns having diameters of 100mm, 150, 200, 250 and 400mm, respectively, representing a range of replacement ratios from 1% to 16%. In general, the tests were conducted for a duration of about 30days. It was observed that the stress concentration ratio generally increased as consolidation of clay took place and reached a maximum value at the end of the primary consolidation. The maximum stress concentration ratio was found to increase approximately linearly with the replacement ratio of the sand column. Further tests revealed that the maximum stress concentration ratio appeared to be independent of the surcharge loading under working loading conditions, However, the stiffness of the sand column played a significant role on the magnitude of stress concentration on the column. The finding revealed the deficiencies in the assumptions made in conventional design procedures of sand compaction piles and sand drains.

A case study on improving of soft ground by installation of sand compaction piles were presented by Alamgir and Zaher (1999a and 1999b). The effectiveness of sand piles in improving a typical soft ground of south western region of Bangladesh to construction a water structure (6-vent regular) in a river, was examined in the papers. At the site, a soft alluvium fine-grained soil deposit exists up to 12m depth from the ground surface. The site was improved by total 765 number of sand piles, 0.20m in diameter and 8.80 to 9.40m long, installed in squire grid at 0.75m spacing installed by vibro-displacement method. Typical sand of Bangladesh, Sylhet sand, was used in the sand piles. Prior to the commencement of concreting for the floor construction of regular, sub-soil explorations were performed to examine the improvement. The investigation reveals that the sand piles improved substantially the bearing capacity of the natural ground. Therefore, the soft ground improvement using sand pile technique is revealed as fast, economical and an efficient method to improve work soil compared with other conventional ground improvement technique. The use of smaller diameter with closer spacing was found suitable in such soft soil deposits for the vibro-compozer type of sand pile construction while comparing the construction problem arises from the installation of large diameter due to the development of side friction. The simple construction procedure and the related equipment adopted in this project for the installation of the desired sand pile was found to provide high degree of effectiveness. Sub-soil investigation revealed that the sand piles improved substantially the bearing capacity of the natural soil and hence the concreting for floor construction of regular was done without any trouble. The monitoring system conduct in this project during the construction process and hence ensured by the Engineer-in-charge made a great contribution to the quality control of the sand piles.

The effectiveness of granular piles in improving a typical soft ground exist in the south-western region of Bangladesh was studied by Ikramul (2014). At a typical soft ground site in KUET campus, Khulna, sand piles of 300mm diameter and 8.5m long were installed by rammed-displacement method in a single and group pattern with rectangular arrangement at 900mm spacing. 1350 sand pile in group were installed were in the project site. The effectiveness was measure by the plate load test at natural ground and improved ground. The field measurement shows that the load carrying capacity of soft ground is increased significantly due to the installation of granular piles irrespective of the type of granular material. The plate load test one month after sand pile installation, the result reveals that the granular pile can carry about 2.0 times more load than that of natural soils. The plate load test one year after sand pile installation, the result reveals that the granular pile can carry about 2.5 times more load than that of natural soils. The investigations show that the piles spacing ratio is to be less than 2.5 to get group effect. The change of soil strength along the depth was also examined by conducting Standard Penetration Test (SPT) after one year of ground improvement. SPT result shows that the soil strength measured as N-values increases along the depth by 1.5 to 2 times, due to the installation of granular piles.

2.13 Discussions

Description of the present state-of-the art about the ground improvement technique, granular piles are presented in the foregoing sections, From the literature review, it is revealed that this ground improvement technique has been and are being used throughout the world successfully in improving poor soil conditions. In the recent years, this technique has also been adopted in Bangladesh in various projects to improve the marginal sites. However, the effectiveness of this method has not been well recorded. The performance was not mentioned in details and required filed investigation was not done. The effectiveness of granular piles in improving soft ground is largely depended on the method of installation of granular piles. At present there are various well installation techniques with the available field equipment and their effectiveness were proved through successful installation of granular piles. These have been practiced successfully in U.S.A Europe and Asian countries. In Bangladesh as per literature review indicates that rammed-displacement method was used successfully. For this installation method, the required equipment are not easily available. Therefore, there is a need of a simple installation technique and the equipment those can be fabricated locally. Wet-replacement method can be a such alternative. In this method, equipment can be fabricated locally with a slight modification of SPT test equipment. In this study this modification was done and the effectiveness of wet-replacement method in the installation of granular piles were examined in the field.

CHAPTER THREE

STATEMENT OF THE PROBLEM

3.1 General

This study deals with a particular improvement technique, namely granular piles, used to improve the soft ground condition. Amongst the various ground improvement techniques, construction of granular piles is considered as one of the foundation solution for its proven records of effectiveness in improving soft soil deposits. At the present time, more granular pile projects in the U.S.A have been constructed in silty sands rather than cohesive soils. World wide the reverse is true. Improvement of soft cohesive soils for construction purposes by means of vibro-displacement i.e. granular piles have been established for the last few decades (Engelhardt and Golding 1975, Barksdale and Bachus 1983, Shin et al. 1991, Okiawa et al. 1992, Bergado and Miruia 1994 and Alamgir 1996). Many successful applications have prove that the method is a valuable addition to the field of special foundation system.

In the recent years, This method of ground improvement has been practicing in Bangladesh at different projects. The installation technique has big influence about the performance of this technique. This study concern about the granular piles in very soft-grained soil consisting organic matter by rammed-displacement method in dry process. The dry technique is suited for partially saturated soils that can stand unsupported, specially those that will densify as a result of vertical displacement. In rammed-displacement method the rammed hole must be able to stand open upon extraction of the probe.

3.2 Site Condition and Sub-Soil Properties

For this field investigation a typical soft ground site at KUET campus Khulna, is considerd. Field investigation about the effectiveness of granular piles, such as sand piles in improving such soft soil deposits is performed here. The location of the investigated site and the sub-soil profiles describing the soil conditions are discussed in the following sections.

3.2.1 Location

For this study, the project site for the field investigation is selected within KUET campus, Khulna. The KUET campus Khulna is located in the south part of the country. The investigated region for the present study is shown in fig. 3.1. The location map of the investigated site in KUET campus is also shown in fig. 3.2

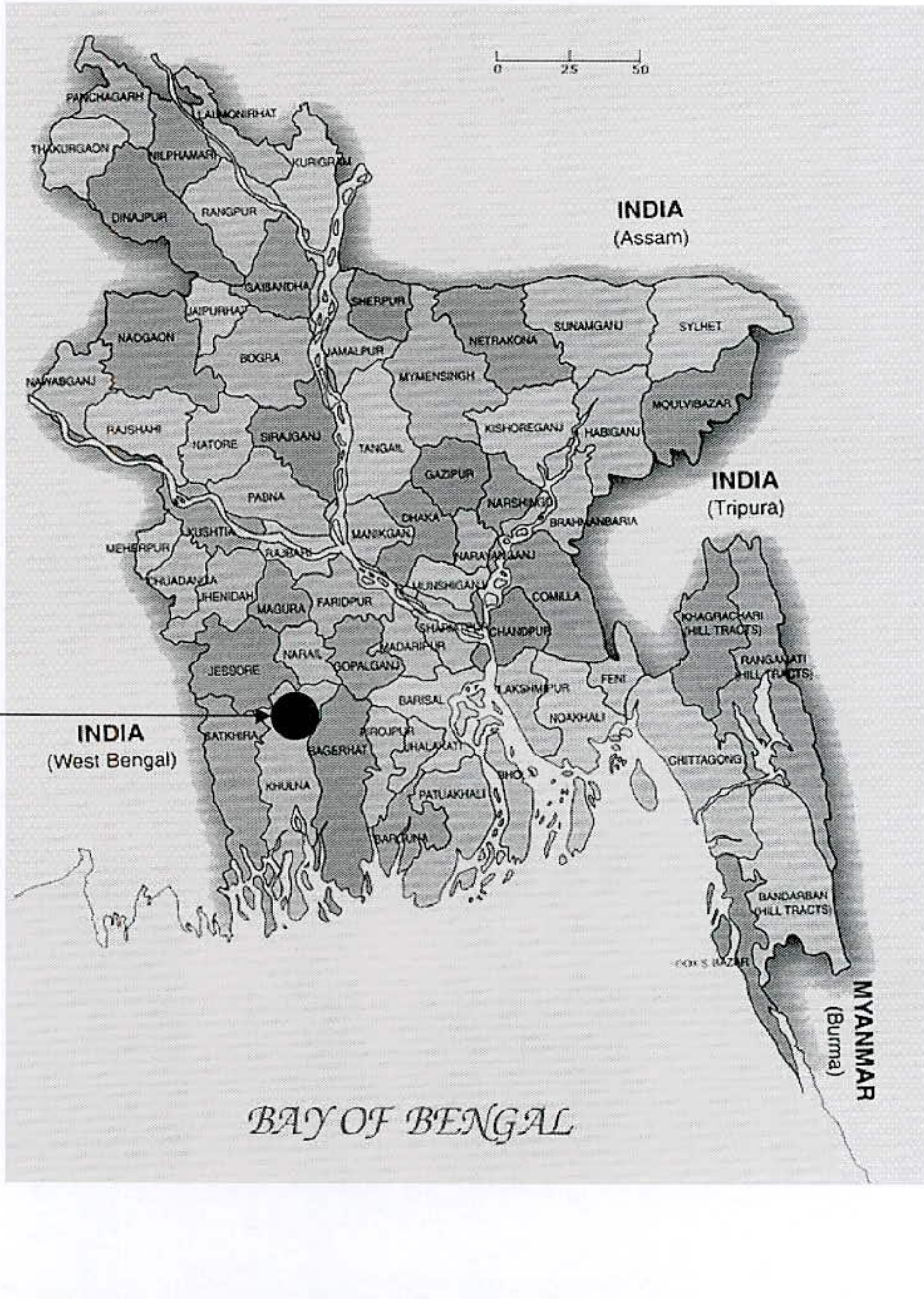


Fig. 3.1 Map of Bangladesh showing the location of Khulna.

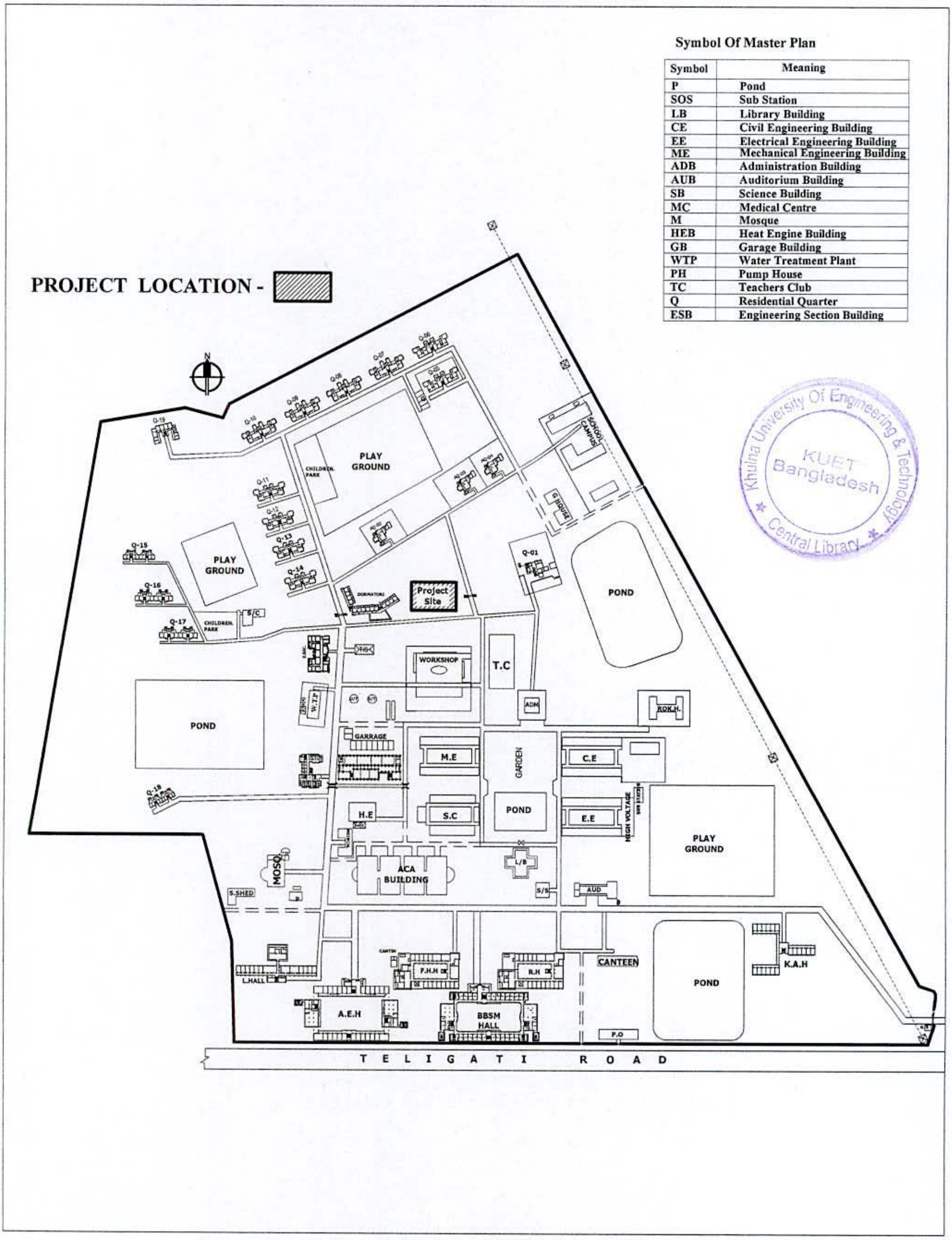


Fig. 3.2 Map of KUET campus and location of project site.

3.2.2 Sub-soil profiles

Bangladesh is apart of Bengal Basin at the lower reaches of the three mighty rivers the Ganges, the Brahmaputra and the Meghna and their associated tributaries. In the upper horizons, the sub-soil of vast areas of Bangladesh is composed of very soft fine grained soil deposits of recent origin. In the south-western coastal districts, fine grained soil deposits where predominantly peat and muck are abundant and undergoing continuous subsidence. In this regions peat deposits are encountered due to the presence of world's biggest mangrove forest, the Sundarbans of 577285 hectares as its present area. In the past, the Sundarban was extended in these regions. For the last few centuries it was double spreading over the present area. By pollen analytical studies and studying the peat soil it was claimed that the present metropolies of Calcutta city was under the mangrove swamps of Sundarbans only 5000 years back (Ghosh, 1941; Chandan and Mukherjee,1972). During the geological changes in the past, some part of the Sundarbans submerged by the weathered and sedimented deposits resulting in the present peat deposits in these regions. The peat deposits extended south-western coastal districts through Sathkhira to Patuakhali of Bangladesh. Practicing Engineers are facing many difficulties in these regions to solve the several geotechnical engineering problems such as very large total and differential settlement, bearing capacity failure and slope stability problems. However, the failure of structures and the related problems, due to the extensive present of peat deposits, were not recorded properly which the practicing engineers can use as a reference. Several structures have been and are being constructed in typical peat deposits of around 20 ft.

Most of the past records, it is found that the KUET campus consisting of soft soil layers containing organic. Sub-soil investigation is done and index properties of soil is determined at different layers. The details of sub-soil condition and soil properties are given in Table 3.1. For determining the sub-soil properties Standard Penetration Test (SPT) is performed at selected site which is situated the place of KUET guest house cum club building. The sub-soil stratification and the soil properties were investigated by means of Standard Penetration Test. The Standard Penetration Test (SPT) developed in 1927, is currently the most popular and economical means to obtain the sub-surface information. It is estimated that 85% to 90% of conventional foundation design in North and South America is made using the sub-soil condition determined by SPT (Bowles 1988). In sensitive silty clay and clayey silts, it is often difficult to determine the true soil properties by conventional field investigation methods. In such cases in-situ methods such as CPT sounding, dilatometer or pressure meter tests often give the most reliable results (Ekstrom et al. 1994). However, SPT is employed here due to not availability of appropriate method for soft soil investigation such as CPT test, dilatometer test and field vane shear test etc. It can be noted here that the SPT test still the most popular field test in Bangladesh to determine the sub-soil profile. The location of bore holes for SPT test and the respective site condition obtained from sub-soil investigation is given in fig. 3.3. Fig 3.3 shows that the N-values are very low up to a depth of 25 ft. from the natural ground surface, which is 2 to 4 and a layer of 15 ft. to 25 ft. containing organic clays. The layers from 25 ft. to 45 ft. depth are grey clay and the layers from 45 ft. to 60 ft. containing clay and silty clay where the N-values are larger than the top layer. In this layer the N-value ranges from 7 to 11.

Date	Number of sample	Depth in m	Thickness in m	Type of sample	Description of Materials	Log	Ø in Degree	F (psi)	Blows on Spoon per 6 Inch Penetration			N Measured	N Corrected	Standard Penetration Resistance: Blows Per 12 inch of Penetration (2 4 6 8 10 12)
									6 Inch	6 inch	6 Inch			
	1	0-1.5	1.5	Gray	Silty Sand				2	3	4	7		
	2	1.5-3.0	1.5	Dark Gray	Clayey Silty				2	2	2	4		
	3	3.0-4.5	1.5	Dark Gray	Clayey Silty				1	1	2	3		
	4	4.5-6.0	1.5	Black	Organic Clay				2	2	2	4		
	5	6.0-7.5	1.5	Black	Organic Clay				2	2	3	5		
	6	7.5-9.0	1.5	Dark Gray	Clayey Silty				3	3	6	7		
	7	9.0-10.5	1.5	Dark Gray	Clayey Silty				1	1	2	3		
	8	10.5-12.0	1.5	Dark Gray	Clayey Silty				2	2	2	4		
	9	12.0-13.5	1.5	Dark Gray	Clayey Silty				4	3	3	6		
	10	13.5-15.0	1.5	Dark Gray	Clayey Silty				4	3	3	6		
	11	15.0-16.5	1.5	Dark Gray	Clayey Silty				2	2	3	5		
	12	16.5-18.0	1.5	Dark Gray	Clayey Silty				3	3	3	6		

Fig. 3.3 Sub-soil stratification and bore log

Table 3.1 Geotechnical engineering properties of the site at KUET campus

Depth (m)	Soil Stratification	W(%)	Physical Properties						Compressibility properties		Shear strength properties	
			W_l(%)	W_p(%)	I_p	γ kN/m^3	G_s	Organic contents (%)	e_o	C_c	S_u(kPa)	N Value
0-1.5	Silty Sand	-	-	-	-	-	-	1.49	-	-	-	7
1.5-3		-	-	-	-	-	-	3.44	-	-	-	4
3-4.5	Clay Silt	48.20	53.20	21.21	31.99	16.92	2.78	10.01	1.15	0.66	20	3
4.5-6	Organic clay	74.65	81.50	47.27	34.23	13.88	2.59	12.70	5.22	1.80	28	4
6-7.5		167.56	255.0	177.42	77.58	10.59	2.10	31.75	1.35	0.65	30	5
7.5-9		63.63	44.80	34.41	10.39	13.42	2.68	7.76	1.73	0.55	35	7
9-10.5	Clay Silty	50.12	39.10	28.07	11.03	17.10	2.75	6.46	1.06	0.35	18	3
10.5-12		44.72	40.00	32.71	7.29	16.85	2.52	7.22	1.36	0.37	11	4
12-13.5		55.58	32.50	31.30	1.2	16.12	2.15	5.59	0.76	0.25	9	6
13.5-15		51.52	36.40	32.46	3.94	-	-	6.49	-	-	-	6
15-16.5		54.32	39.00	35.29	3.71	-	-	4.89	-	-	-	5
16.5-18		55.21	32.30	31.11	1.19	-	-	3.52	-	-	-	6

Note: w= Water content, W_l= Liquid limit, W_p= Plastic limit, γ= Unit weight, G_s= Specific gravity, e_o= Initial void ratio, C_c= Compression index, s_u= Undrained shear strength. Average values are provided here of various parameters.

3.3 Selection of Improvement Technique

During the planning stage of any construction project, it is needed to establish whether any improvement of the soil is required or the construction could proceed without any improvement. Only then the basic design and its execution conditions are decided. For soft ground, countermeasures are actually required in most cases. The methods suitable for countermeasures application are decided according to various conditions such as structural conditions following with ground or soil conditions, construction site conditions, economical feasibility, and execution condition. In this study granular pile is considered as the ground improvement technique to increase the bearing capacity of the soft compressible ground.

3.3.1 Granular Piles

The following types of granular piles are considered for the conducted field investigations.

i. Sand Piles

Table 3.2 Dimension of granular piles

Types of granular piles	Diameter	Length
Sand piles	300 mm	8.25 m

The dimension of the granular piles considered for the present study are given in Table 3.2. Their schematic diagram of granular piles are shown in Fig. 3.4 for single pattern, respectively.

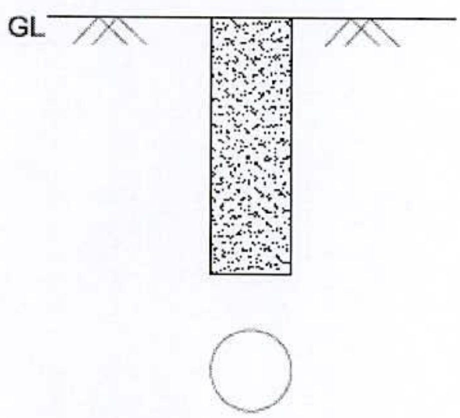


Fig. 3.4 Schematic diagram of granular piles

3.3.2 Installation technique

There are various types of effective techniques for the Installation of granular piles such as (i) Vibro-displacement method (ii) Vibro-replacement method (iii) Vibro-compozer method (iv) Cased-borehole method (v) Rammed-displacement method as described in the previous chapter. Considering the advantages of Rammed-displacement installation technique over the other, it is considered for the present study. Rammed-displacement is a dry process and it is suitably used in several marginal site improvement projects. The equipments for Rammed-displacement method is locally available.

3.4 Materials of Granular Piles

In this study, types of granular materials namely, sands were considered for the construction of granular piles. These types are selected since they are readily available in Bangladesh and also to compare their capacities with each other. The locally available sand mixed with the sylhet sands for well gradation made by trial and error. The properties of granular materials are described in the following sections.

3.4.1 Physical properties of sand

Types of sand grain are used in this study are Sylhet sand

(a) Sylhet Sand

Sylhet sand is considered as one of the best quality sand of Bangladesh for civil engineering construction. Sylhet sand is a yellowish-brown color river sand. The Physical properties of Sylhet sand can be described as FM=2.58, D_{10} = 0.22; D_{30}=0.38; D_{60}=0.80; C_u=3.64; and C_c=0.82. The grain size distribution of Sylhet sand is shown in Fig.3.5.

Where, FM= Fineness modulus, D_{10}=Effective diameter of particle size of which 10% sample is smaller. D_{30}=Diameter of particle size of which 30% Sample is smaller. D_{60}=Diameter of particle size of which 60% sample is smaller. C_u = Co-efficient of uniformity C_c= Co-efficient of curvature.

(b) Local sand

This sand is one of the locally available sand in the South western region of Bangladesh, the location of present study. Locally available sand are river sand and of light grey in color. The physical properties of local sand can be described as FM=0.13 D_{10}=0.15; D_{30}=0.22; D_{60}=0.28; C_u=1.86 ; and C_c=1.15. The grain size distribution of local sand is shown in Fig.3.6.

Table 3.3 Calculate the F.M of local sand

Sieve analysis for fine aggregate (local sand).

Total weight of sample (gm) = 500 gm

Grain Diameter in mm	Sieve#	Wt Retained (gm)	Cum wt. retained (gm)	Cum wt. retained (%)	% Finer	FM
4.75	#4	0.00	0.00	0.00	100.00	**1.30**
2.38	#8	0.00	0.00	0.00	100.00	
1.19	#16	1.10	1.10	0.22	99.78	
0.6	#30	20.10	21.20	4.24	95.76	
0.3	#50	144.40	165.60	33.12	66.88	
0.15	#100	295.00	460.60	92.12	7.88	
Sum of cumulative weight retained (%) =				129.70		

Table 3.4. Calculate the F.M of sylhet sand

Sieve analysis for fine aggregate (sylhet sand).

Total weight of sample (gm) = 500 gm

Grain Diameter in mm	Sieve#	Wt Retained (gm)	Cum wt. retained (gm)	Cum wt. retained (%)	% Finer	FM
						2.58
4.75	#4	0.00	0.00	0.00	100.00	
2.38	#8	21.80	21.80	4.36	95.64	
1.19	#16	97.20	119.00	23.80	76.20	
0.6	#30	133.30	252.30	50.46	49.54	
0.3	#50	147.40	399.70	79.94	20.06	
0.15	#100	97.10	496.80	99.36	0.64	
Sum of cumulative weight retained (%) =				257.92		

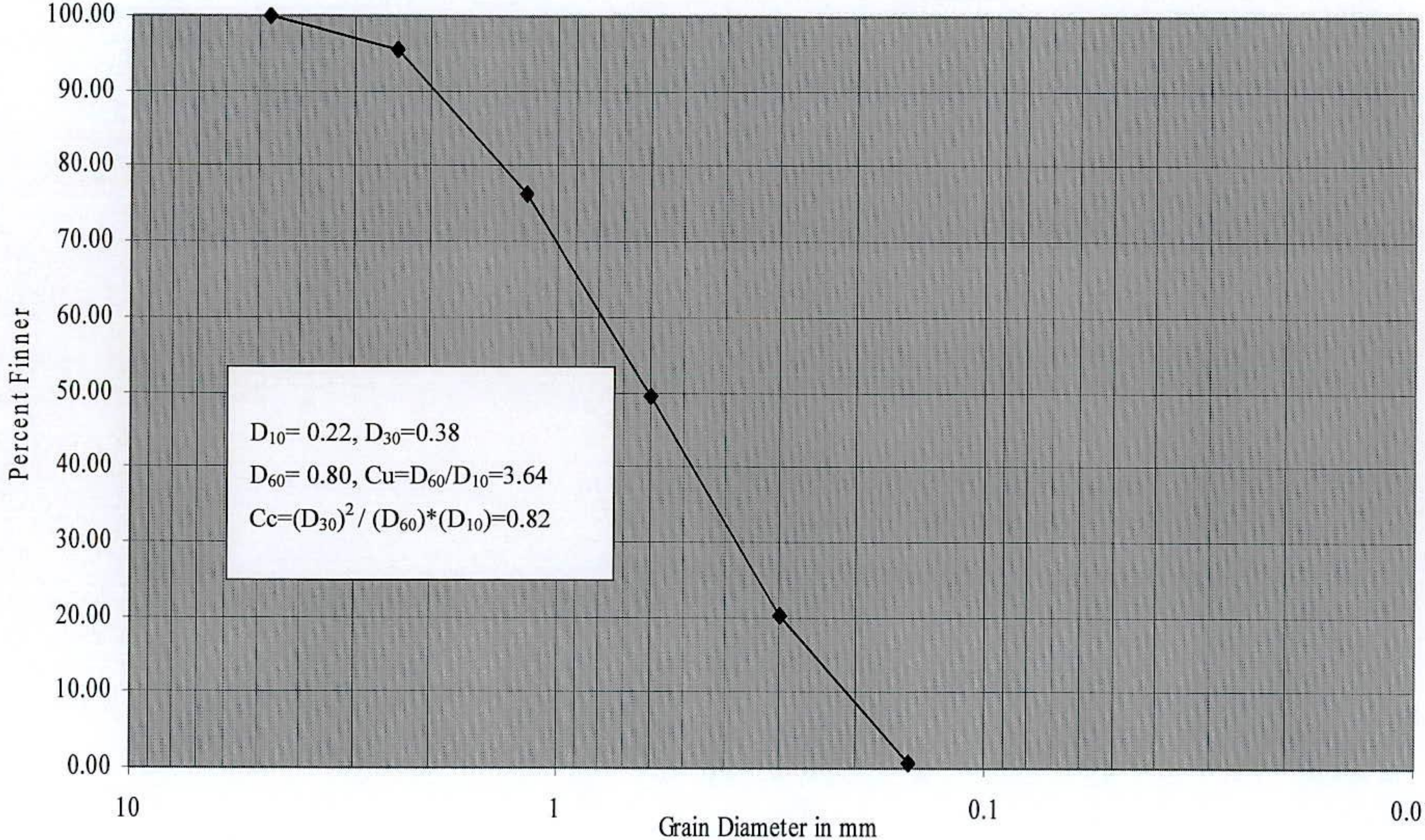


Fig. 3.5 Grain Size Distribution Curve (Sylhet Sand)

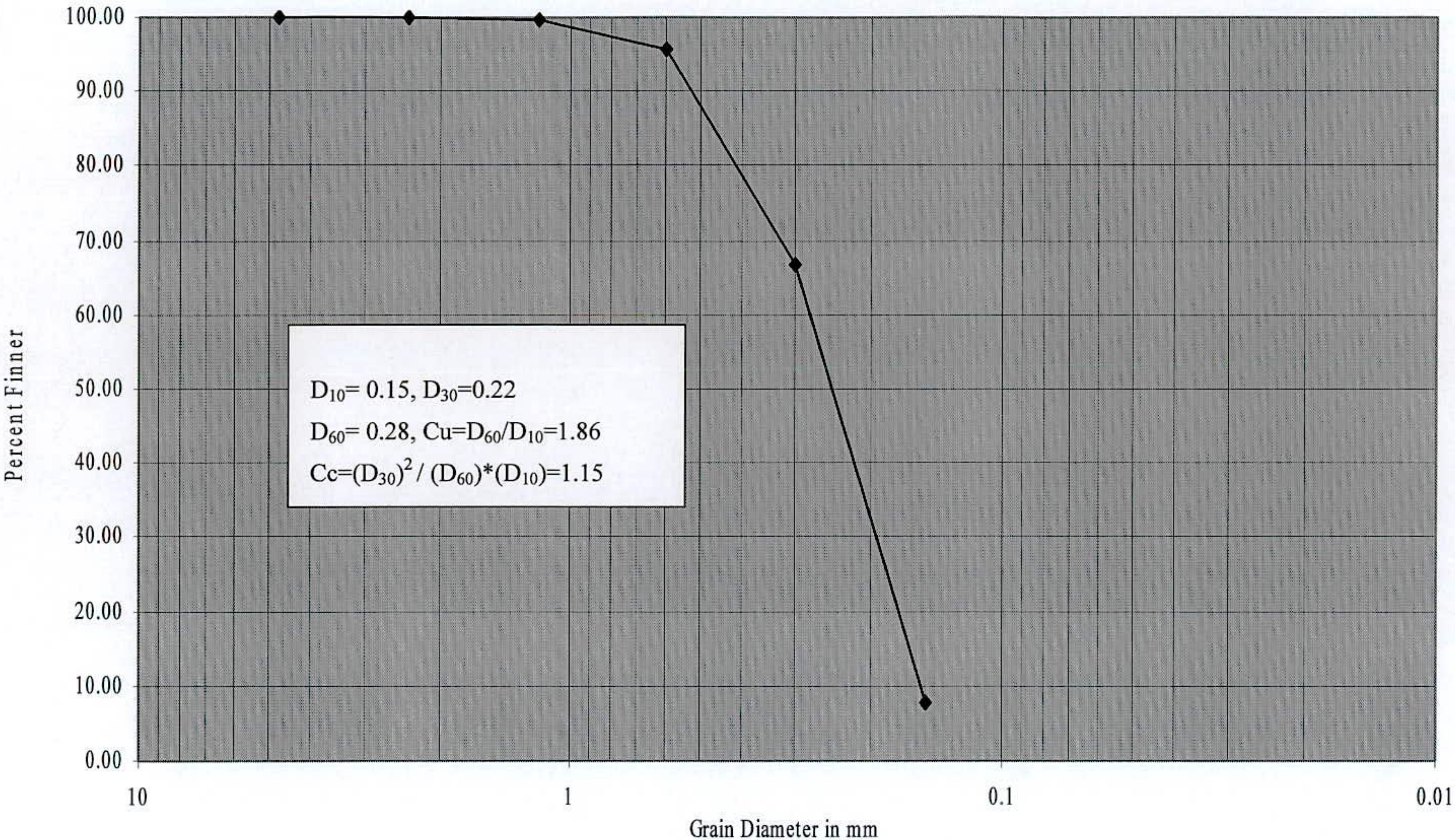


Fig. 3.6 Grain Size Distribution Curve (Local Sand)

CHAPTER FOUR

GRANULAR PILE INSTALLATION AND FIELD INVESTIGATION

4.1 General

The installation of granular piles in the selected location and the subsequent field investigations of improved ground are described in this chapter. Two types of granular piles, sand pile and stone columns were installed by rammed-displacement method in single and group pattern. Plate load test were performed to observe the load settlement response and the Standard Penetration Test was conducted to examine the change of penetration resistance along the depth of sub-soil after improvement.

4.2 Installation of Granular Piles

The granular piles were constructed here by rammed-displacement method in dry process. The dry method is frequently used to construct columnar inclusions through weak soils in developed areas because of the problems associated with the acquisition, retention and disposal of significant amount of water. The dry technique is suited for partially saturated soils that can stand unsupported, especially those that will density as a result of lateral displacement. In rammed-displacement method the hole must be able to stand open upon extraction of the probe. It is recommended that the rammed-displacement to be possible soils must exhibit undrained shear strengths in excess of 40 kPa to 60 kPa (850 psf to 1250 psf), with a relatively low ground water table being present at the site (Barksdale and Bachus, 1983). The construction process must capable to stabilize of the sites underlain by soft soils and high ground water using the dry process. The rammed-displacement type construction method employed in this project using simple technique was found suitable for this site consists of soft fine grained soil deposits having high water table. A brief description of the equipment and method of installation are described in the following sections.

4.2.1 Description of equipment

A 1500rpm traditional two rig machine and a two end open casing pipe 8mm thickness and 300mm in diameter and 7m long with a rammed-hammer of weight 1200kg. The rammed-hammer was 250mm in diameter and 3.00 m long. The equipment is shown in Fig. 4.2 and 4.3

4.2.2 Installation procedures

The granular piles were installed here by rammed-displacement method. The construction sequences are described in the following statements. The schematic diagram is shown in Fig. 4.1. The installation process are also shown in Fig. 4.4 and 4.5.

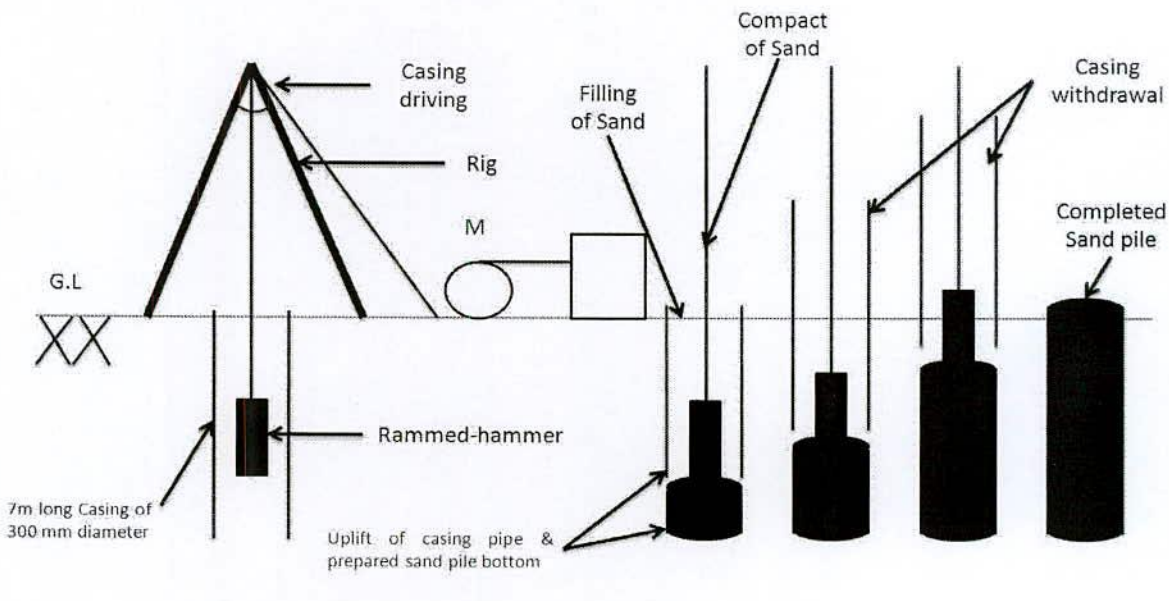


Figure 4.1 Schematic diagram of installation procedures of granular piles

i. A two end open casing pipe, 300mm in diameter and 7m long was placed vertically at the designed point on the natural ground surface for sand pile construction.

ii. The casing pipe was then inserted vertically into the ground about 300mm to 450mm depth at its own weight just by applying some pressure manually. At first a plug is made by the designated sand up to 750mm of casing pipe at bottom level.

iii. The rammed-hammer 250mm in diameter and 3.0m long, weighting 1000kg was placed inside the casing pipe. The rammed-hammer displaced the soil from beneath the casing pipe hence the casing pipe was driven by its own weight till reached the designated position (depth) into the ground. Here one casing pipe of 7m long was driven inside the ground.

iv. After reaching the designated depth, the sand plug is broken by providing excess energy then the rammed-hammer is withdrawn from the casing pipe.

v. Casing pipe was then lifted up by about 1m from its original bottom position. The designated granular materials were poured into the hole about 1m layer thickness measured from the bottom. The poured granular materials was then densified by rammed-hammer till the required compactness achieved.

vi. Casing pipe was then withdrawn from inside the ground that left the bottom portion of the hole unsupported and the top portion supported by the casing pipe. It was observed that the bottom portion of the hole standing safely without any lateral support.

vii. Then hole was poured by the selected granular materials in layers and hence 10 to 15 drops compacted each layer was densified by rammed-hammer till the designated compactness was reached. In general, the thickness of each layer was about 1.0m and 0.65m to 0.75m before and after densification respectively and the free fall height of the rammed-hammer was 0.75m to 1.0m.

viii. After the top of granular piles were reached about 1.0m to 1.5m below the ground surface the casing pipe was withdrawn and left the remaining hole unsupported.

ix. Then step five (v) was continued until the granular piles were constructed up to the ground level.

Fig. 4.2 Investigation field during construction of granular piles

Fig. 4.3 Preparation of 10 meter mounted tripod stand and 7m long casing pipe

Fig. 4.4 Lifting of 300mm diameter steel pipe

Fig. 4.5 Pouring the granular material into the created hole.

Fig. 4.6 Compacting the granular materials inside the ground.

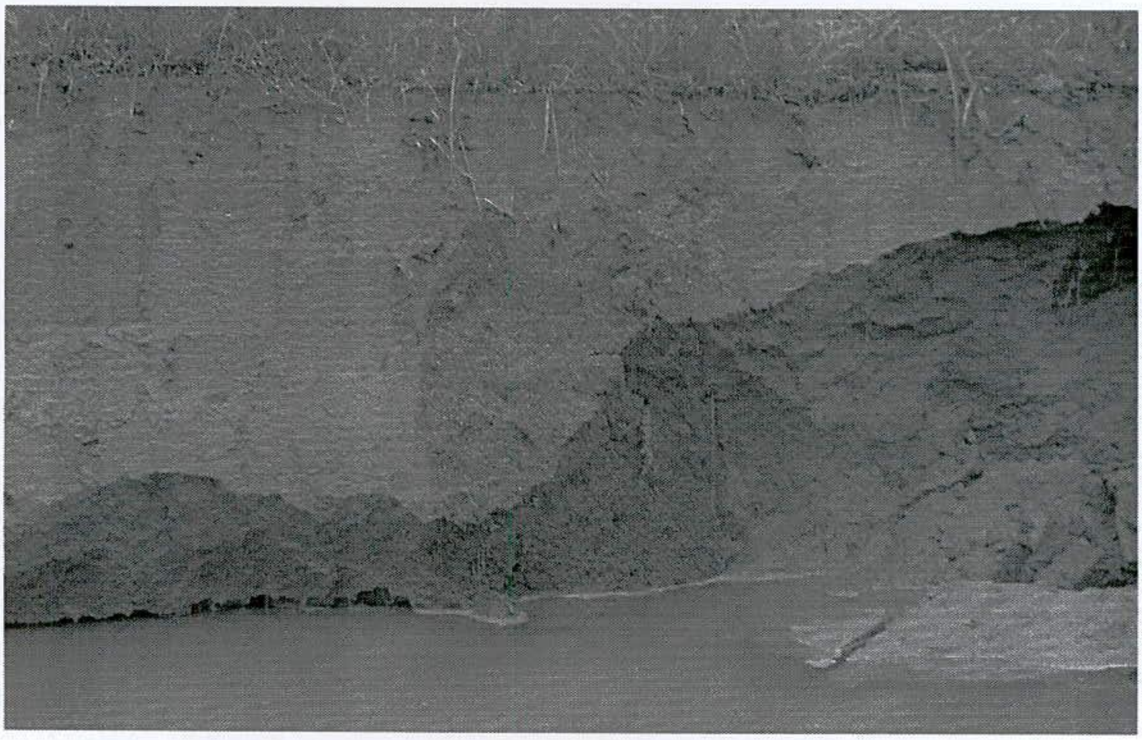


Fig. 4.7 As built cross section of sand pile (4ft from the existing ground level)

4.2.3 Monitoring of construction process.

The progress of construction granular piles were carefully observed and recorded at the construction site to ensure the quality. In this regard the following items were checked and noted during the installation of each granular pile.

i. The granular pile number, the date and time of installation begins.

ii. The time required for driving the casing pipe to create the hole till the designated depth reached.

iii. To ensure the quality of the granular materials according to specification.

iv. The drop of hammer both the number of drop and free fall length, observed carefully to ensure the required compaction of the granular materials for each granular pile.

v. The granular pile length, bottom tip elevation and the diameter after construction.

vi. The total time required for construction of each granular pile.

vii. The total quality of granular materials used in the granular piles.

viii. The amount of granular materials poured in each layer.

ix. The total quantity of granular material required for each granular pile.

These observations and recording system made a great contribution to ensure the quality control of the constructed granular piles. The careful site observation is the main requirement to construct the granular pile as per desire.

4.2.4 Installation sequences

The construction sequence of granular piles according to layout pattern are given below. The diameter and center to center distance shown in granular piles in Fig. 4.8 indicates order of construction of group sand piles. The single sand pile was constructed individually. The layout of group sand piles is also shown in Fig. 4.9.

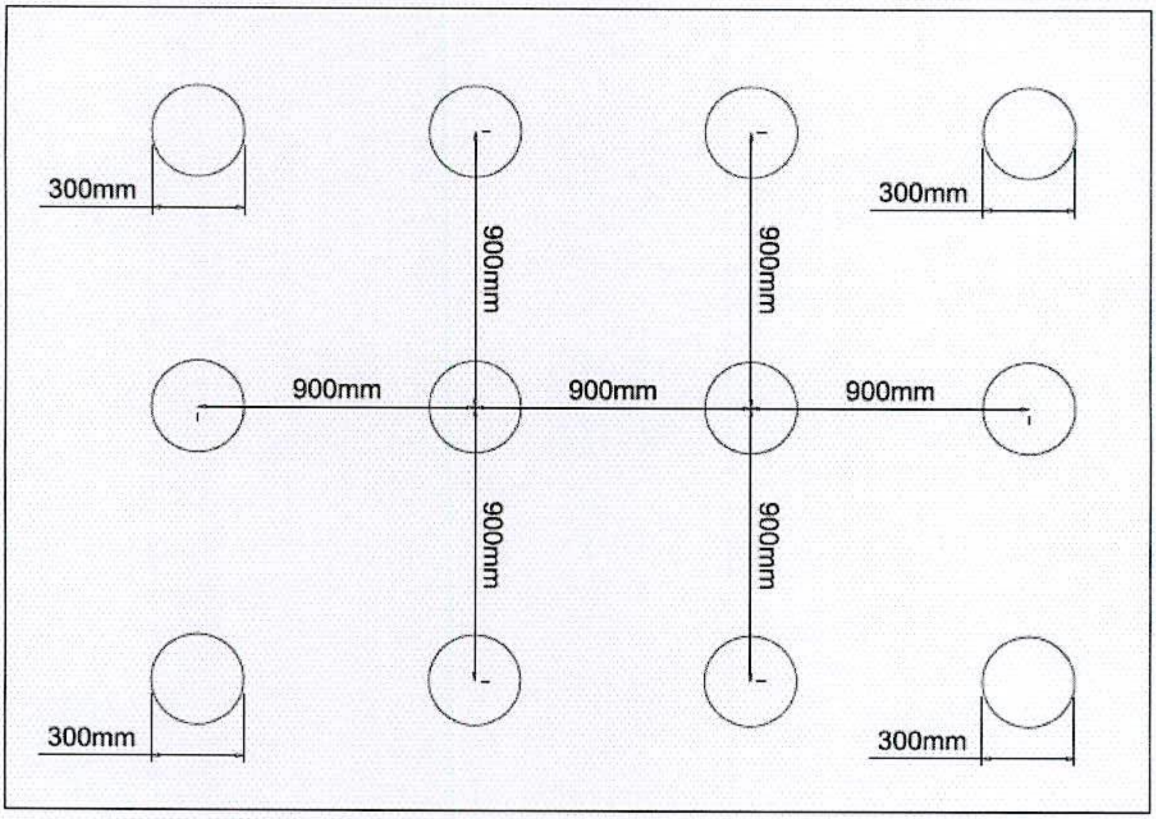


Fig. 4.8 The installation sequences of sand pile

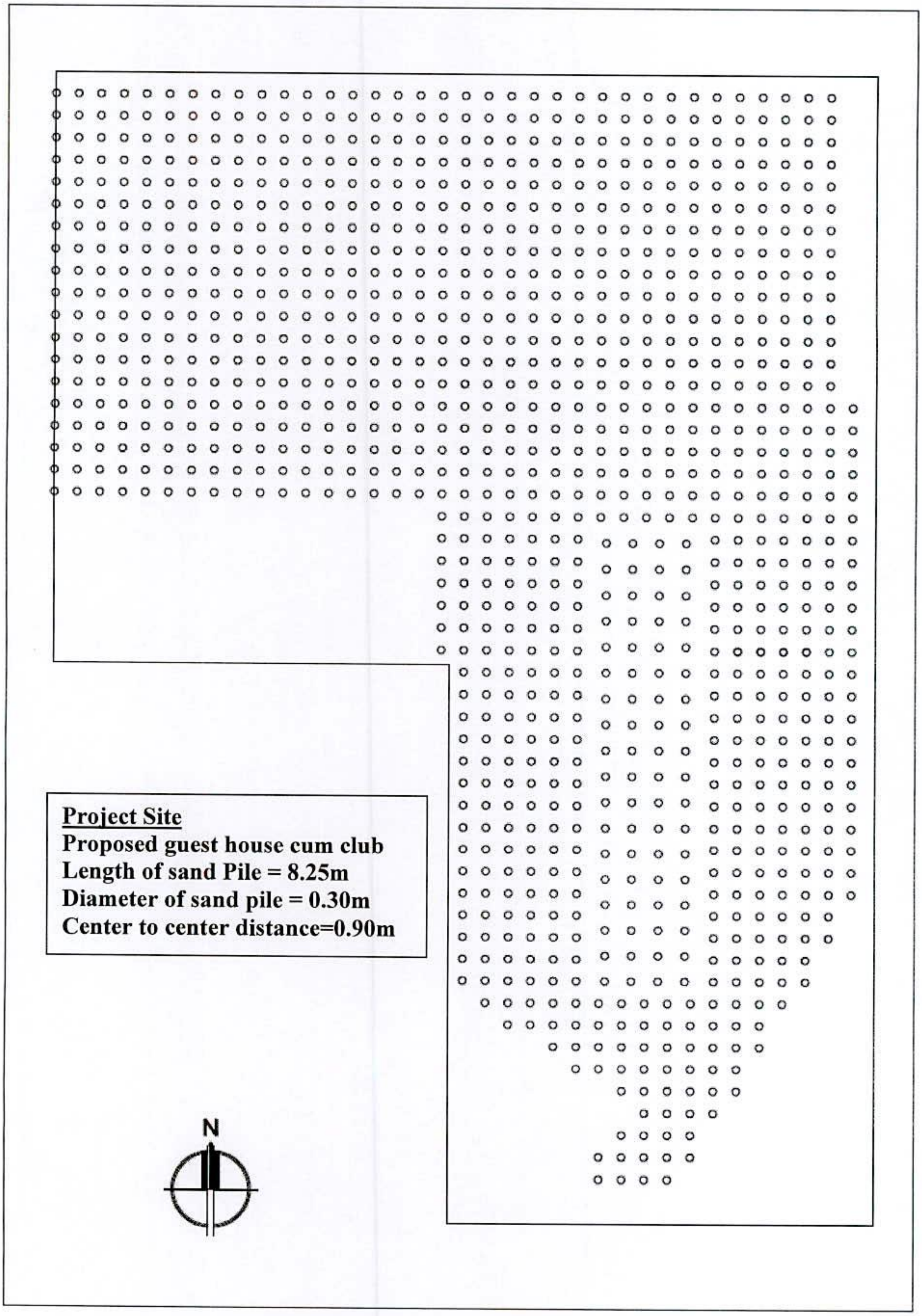


Fig. 4.9 The layout of group sand pile

4.3 Field Investigations

Plate load test was performed for single and group granular piles installed in the natural ground. The granular piles were constructed by using sand for the single and group cases. In this study total one thousand and three hundreds numbers of granular piles were installed their lay-out pattern and detail description are given in chapter three. Single and group granular piles are considered in this study to establish the effect of the type of granular materials and also to know the behavior of group and granular piles.

4.3.1 Methods of investigation

Obviously the most realized method to determine the bearing capacity at a site is to perform a load test. This would directly give the bearing capacity if the load test is performed on a full size footing. Due to inherent constraints of full scale load test, the usual practice is to perform the load test on a small steel plate of diameter 12 inch (300mm) or square of side 12inch x 12inch (300mm x 300mm). The schematic diagram of a typical plate load test is shown in Fig. 4.9. In the present study a circular plate of diameter 300mm was used. The plate load tests conducted in the present investigation are shown in Fig. 4.10 and

The procedure has been standardized as ASTM D1194, which is essentially as follows:

i. Decide on the type of load application, and if it is to be a reaction against piles, they should be driven first to avoid excessive vibration and loosening of the soil in the excavation where the load test is performed.

ii. A pit was excavated to the depth at which was test performed. The test pit should be at least four times as wide as the plate and to the depth the foundation was to be placed.

iii. A load was placed on the plate and settlement were recorded from two dial gauges accurate to 0.25mm. Observation on a load increment should be taken until the rate of settlement was beyond the capacity of dial gauge. Load increments should be approximately one-fifth of the estimated bearing capacity of the soil. Time intervals should not be more than one hour and should be same duration for all the load increments.

iv. The test was continued till a total settlement of 18mm was obtained. After the load test was released the elastic rebound of the soil was recorded for a period of time at equal to the time duration of a load increment.

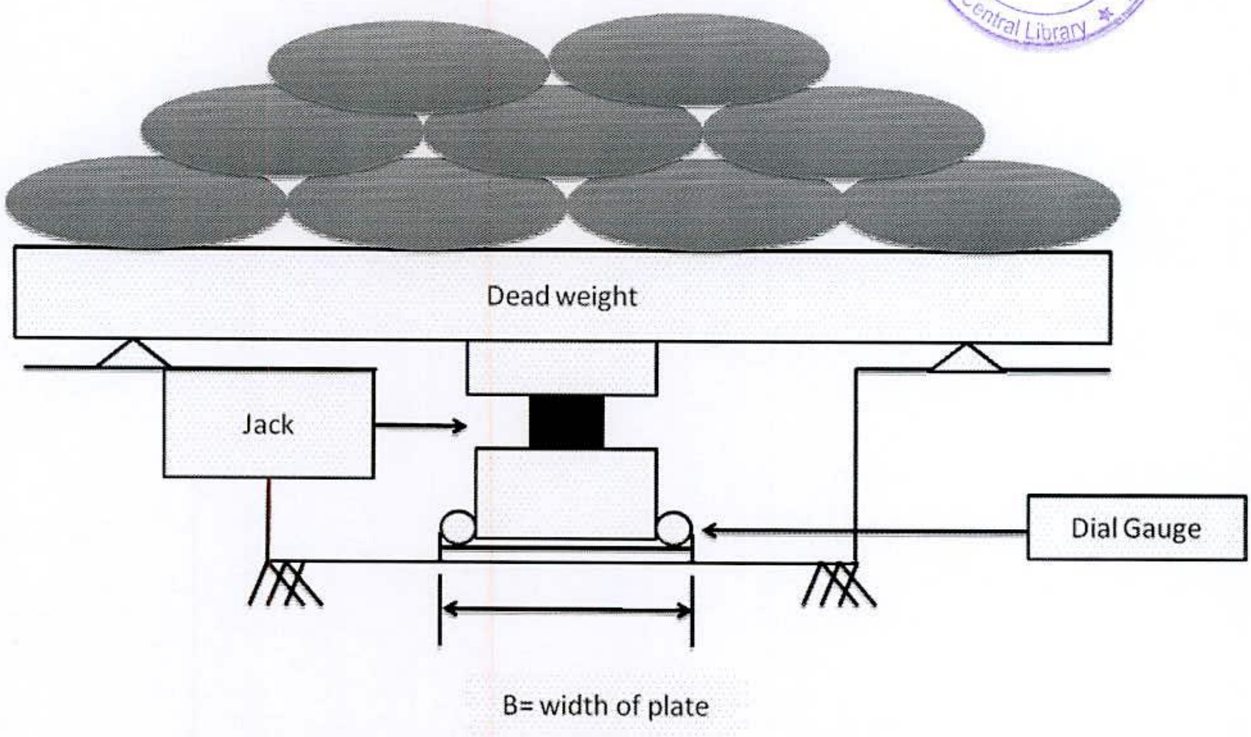


Fig. 4.10 Schematic diagram of a typical plate load test

4.3.2 Constraints of plate load test

Plate load test is an alternative field load test conduct to avoid the huge cost and to save the time of full scale load test. The inherent constraints of plate load test are described in the followings:

i. Size effect: The results of the plate load test reflect the strength and the settlement characteristics of the soil within the pressure bulbs. As the pressure bulb depends upon the size of the loaded area, it is much deeper for the actual foundation. The plate load test does not truly represents the actual conditions if the soil is not homogeneous and isotropic to a large depth.

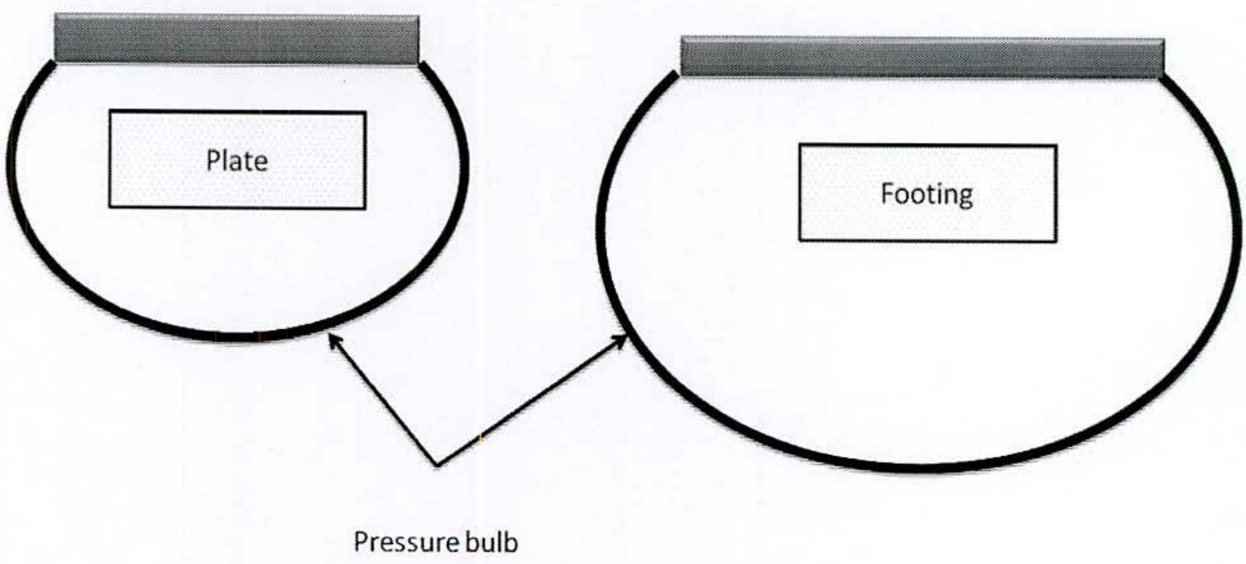


Fig. 4.11 Constraints of plate load test

ii. Scale Effect: The ultimate bearing capacity of saturated clays is independent of the size of the plate but for cohesion less soils, it increases with the size of the plate.

Though the plate load test has several short-comings and limitations but this testis used extensively because of its simplicity and availability of the testing equipment.

4.4 Plate Load Test on Natural Ground

The plate load test was done on the natural ground at the project site to determine the load settlement response of the untreated ground. The test plate was placed at a depth of 1.5m from the natural ground surface. The load intensity was increased from 67.17 kN/m^2 to 671.10 kN/m^2 at an equal interval of 67.17 kN/m^2. In each load, the settlement were measured till the rate of deformation less than 1.5 mm/hr. The settlement values with time for each increment of vertical load and the time-settlement curves are given in Table 4.1 and Fig, 4.13 respectively. From this figure, it can be seen, that maximum settlement observed as 18.00 mm at a load intensity of 671.10 kN/m^2. The total time required for this observation was 650 minutes.

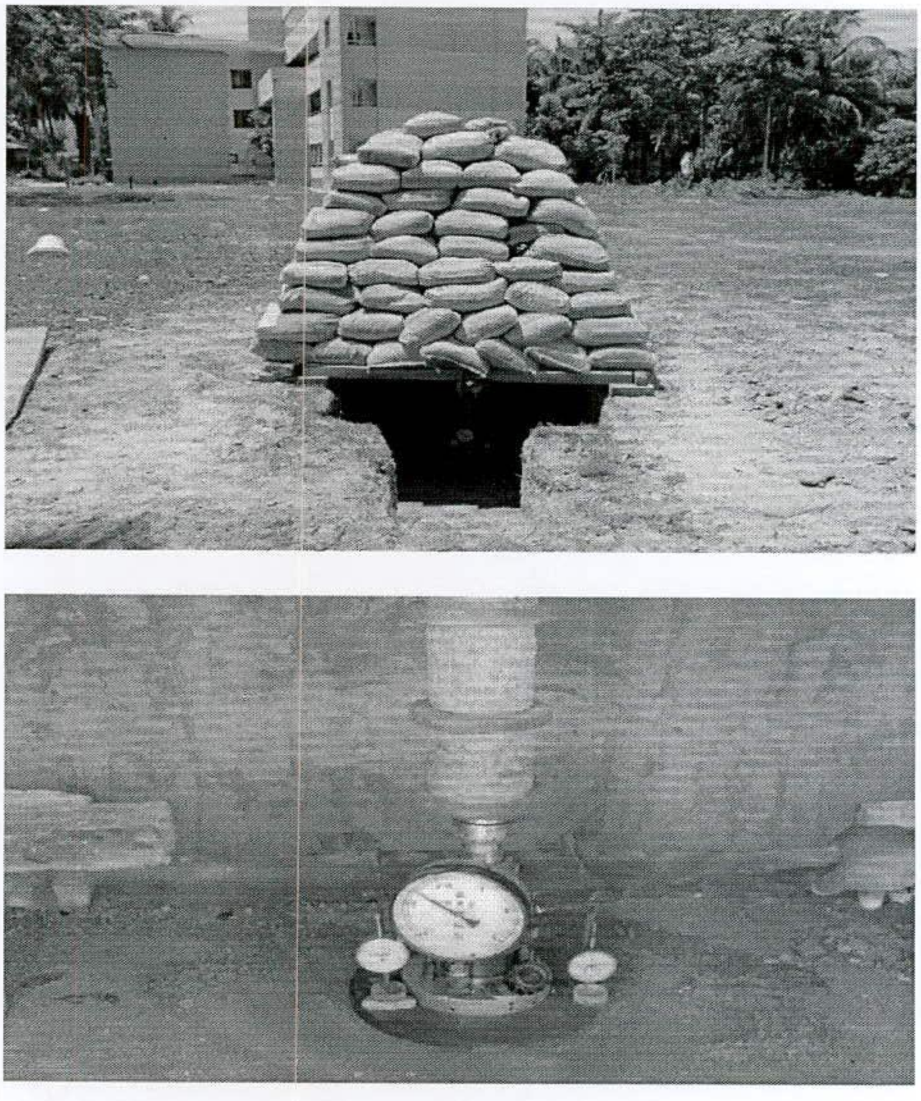


Fig. 4.12 Plate load test on natural ground

Table 4.1 Measured settlement values with time for natural ground.

Load kN/m^2	Time (minutes)	Settlement (mm)		Load kN/m^2	Time (minutes)	Settlement (mm)
67.17	0	1.15		403.02	325	8.88
	5	1.23			330	9.14
	15	1.24			340	9.90
	35	1.28			360	10.52
	65	1.31			390	10.60
134.34	65	1.73		470.19	390	11.63
	70	1.90			395	12.19
	80	2.13			405	12.32
	100	2.39			425	12.47
	130	2.57			455	12.52
201.51	130	3.16		537.36	455	12.87
	135	3.35			460	13.28
	145	3.64			470	13.46
	165	4.10			490	13.53
	195	4.38			520	13.70
268.68	195	4.70		604.53	520	14.63
	200	4.96			525	15.73
	210	5.45			535	15.88
	230	6.13			555	16.03
	260	6.57			585	16.27
335.85	260	7.05		671.70	585	16.60
	265	7.40			590	17.52
	275	7.47			600	17.75
	295	7.51			620	17.90
	325	8.16			650	18.00

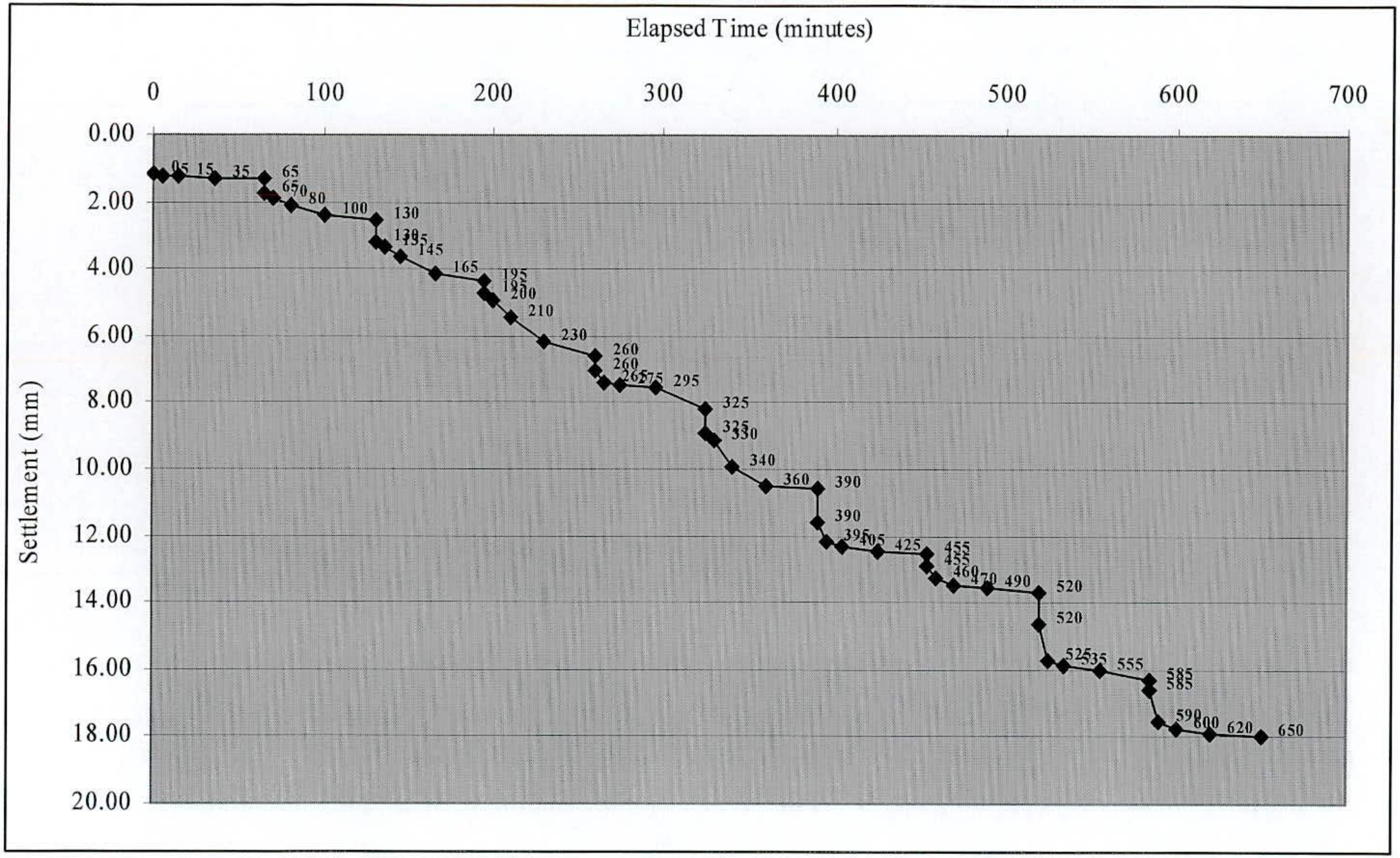


Fig. 4.13 Time versus settlement obtained from load test on natural ground

4.5 Plate Load Test on Improved Ground

Plate load test were performed on the improved ground for different conditions. There are (i) The plate load test immediately after sand pile installation and (ii) The plate load test one year after sand pile installation. The plate was placed exactly on the top of the single piles. In case of group, plate was placed on the top of middle sand pile. The location of plate for group piles are shown in Fig. 4.12. The plate load test results are given in the following sections for different improved conditions.

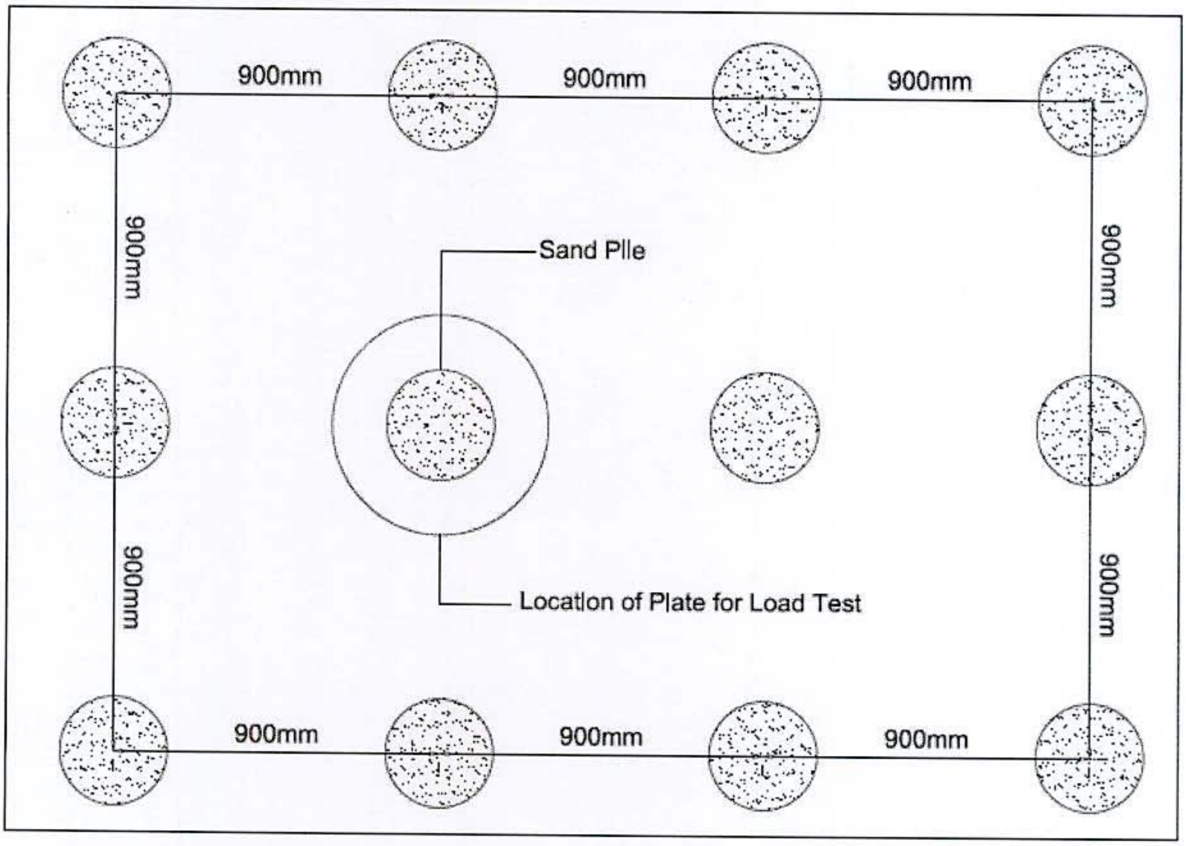


Fig. 4.14 Location of plate in plate load test on group sand piles.

4.5.1 One month after sand pile installation

The plate load test was done on the natural ground at the project site to determine the load settlement response of the untreated ground. The test plate was placed at a depth of 1.5m from the natural ground surface. The load intensity was increased from 67.17 kN/m^2 to 671.10 kN/m^2 at an equal interval of 67.17 kN/m^2. In each load, the settlement were measured till the rate of deformation less than 1.5mm/hr. The settlement values with time for each increment of vertical load and the time-settlement curves are given in Table 4.2 and Fig. 4.16 respectively. From this figure, it can be seen, that maximum settlement observed as 8.20mm at a load intensity of 671.10 kN/m^2. The total time required for this observation was 650 minutes.

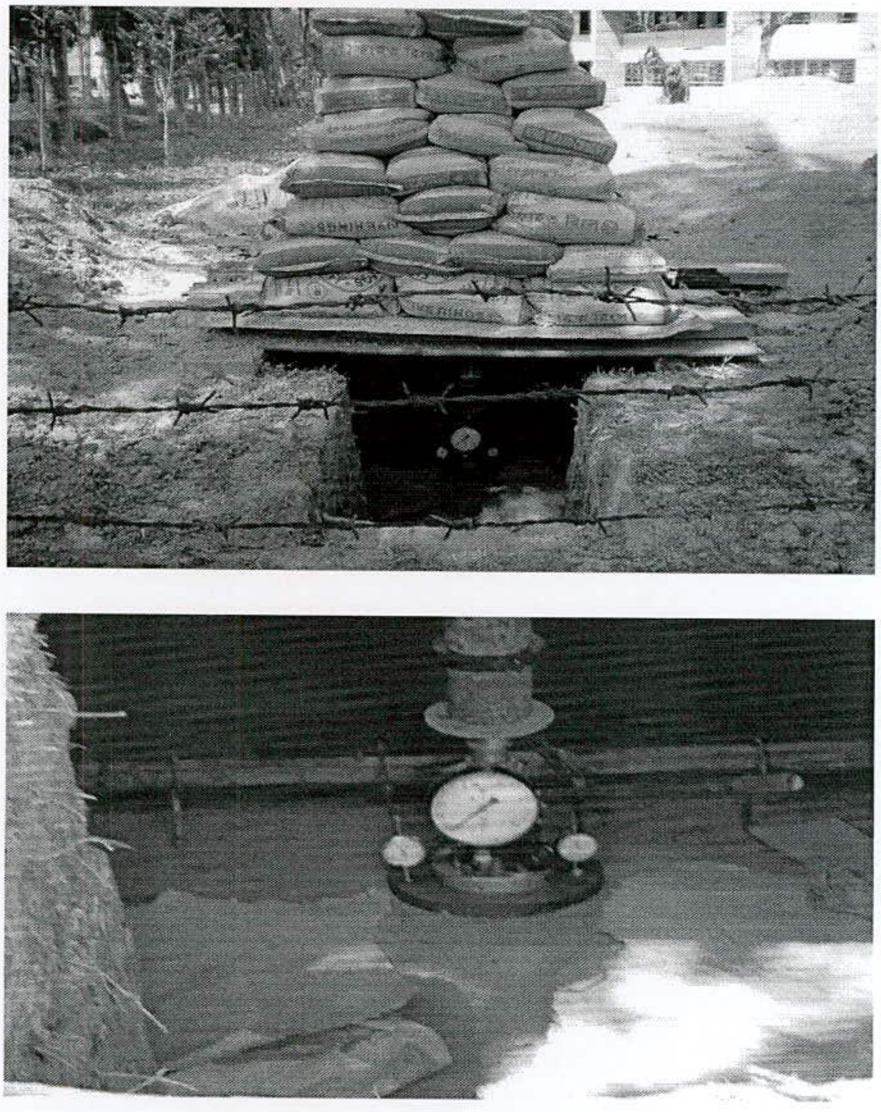


Fig. 4.15 Plate load test on sand pile (One month after sand pile installation)

Table 4.2 Measured settlement values with time for sand pile (One month after sand pile installation)

Load kN/m^2	Time (minutes)	Settlement (mm)		Load kN/m^2	Time (minutes)	Settlement (mm)
67.17	0	0.04		403.02	325	2.83
	5	0.08			330	2.95
	15	0.09			340	3.14
	35	0.10			360	3.22
	65	0.10			390	3.32
134.34	65	0.29		470.19	390	3.65
	70	0.30			395	3.89
	80	0.33			405	3.97
	100	0.36			425	4.08
	130	0.39			455	4.21
201.51	130	1.03		537.36	455	4.62
	135	1.10			460	4.91
	145	1.11			470	5.06
	165	1.13			490	5.18
	195	1.16			520	5.33
268.68	195	1.48		604.53	520	5.65
	200	1.56			525	6.08
	210	1.61			535	6.19
	230	1.63			555	6.44
	260	1.65			585	6.53
335.85	260	2.10		671.70	585	6.94
	265	2.28			590	7.32
	275	2.35			600	7.38
	295	2.43			620	7.87
	325	2.55			650	8.20

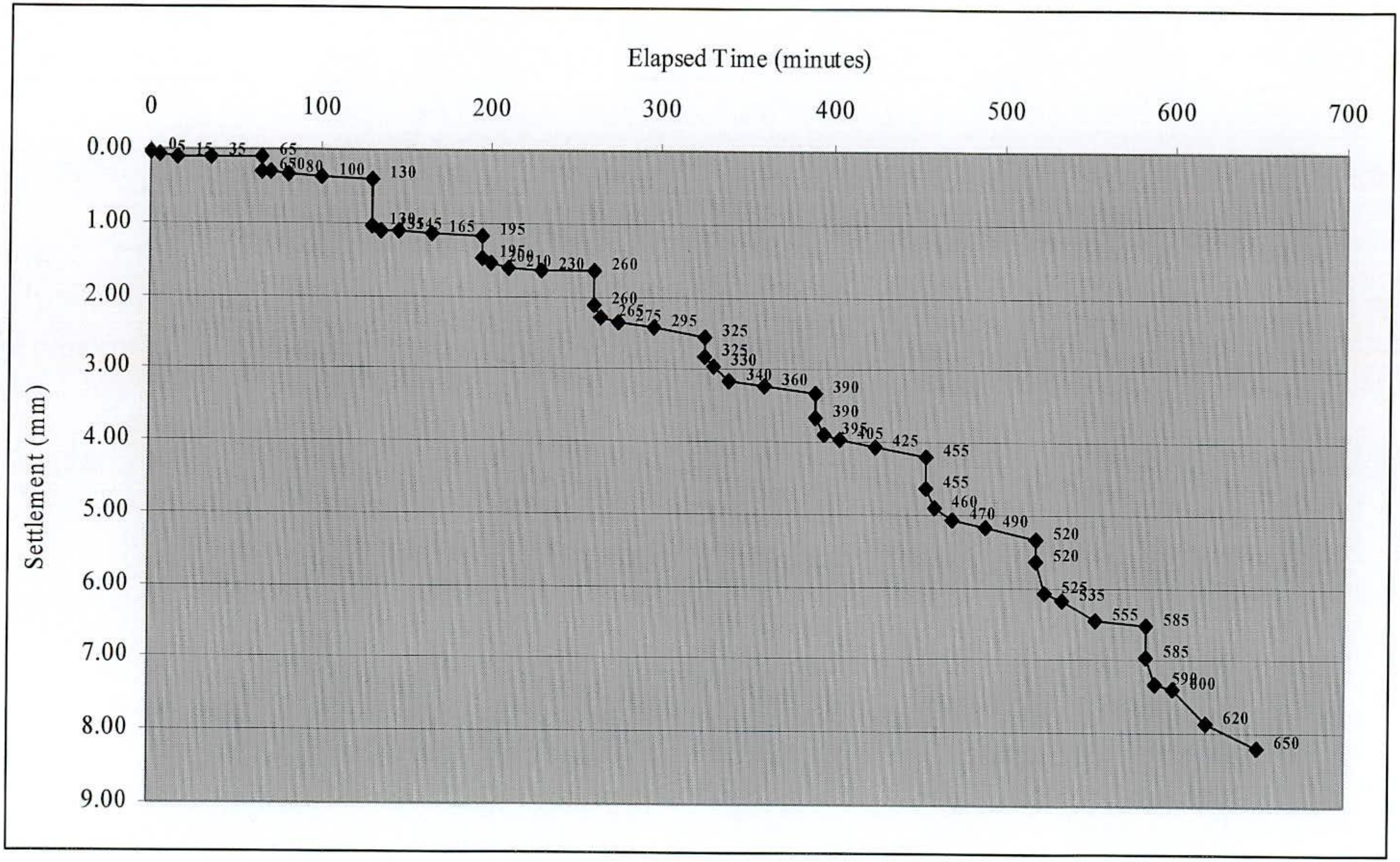


Fig. 4.16 Time versus settlement obtained from load test on improved ground (One month after sand pile installation)

4.5.2 One year after sand pile installation

The plate load test was done on the natural ground at the project site to determine the load settlement response of the untreated ground. The test plate was placed at a depth of 1.5m from the natural ground surface. The load intensity was increased from 67.17 kN/m^2 to 671.10 kN/m^2 at an equal interval of 67.17 kN/m^2. In each load, the settlement were measured till the rate of deformation less than 1.5mm/hr. The settlement values with time for each increment of vertical load and the time-settlement curves are given in Table 4.3 and Fig. 4.18 respectively. From this figure, it can be seen, that maximum settlement observed as 7.00mm at a load intensity of 671.10 kN/m^2. The total time required for this observation was 650 minutes.

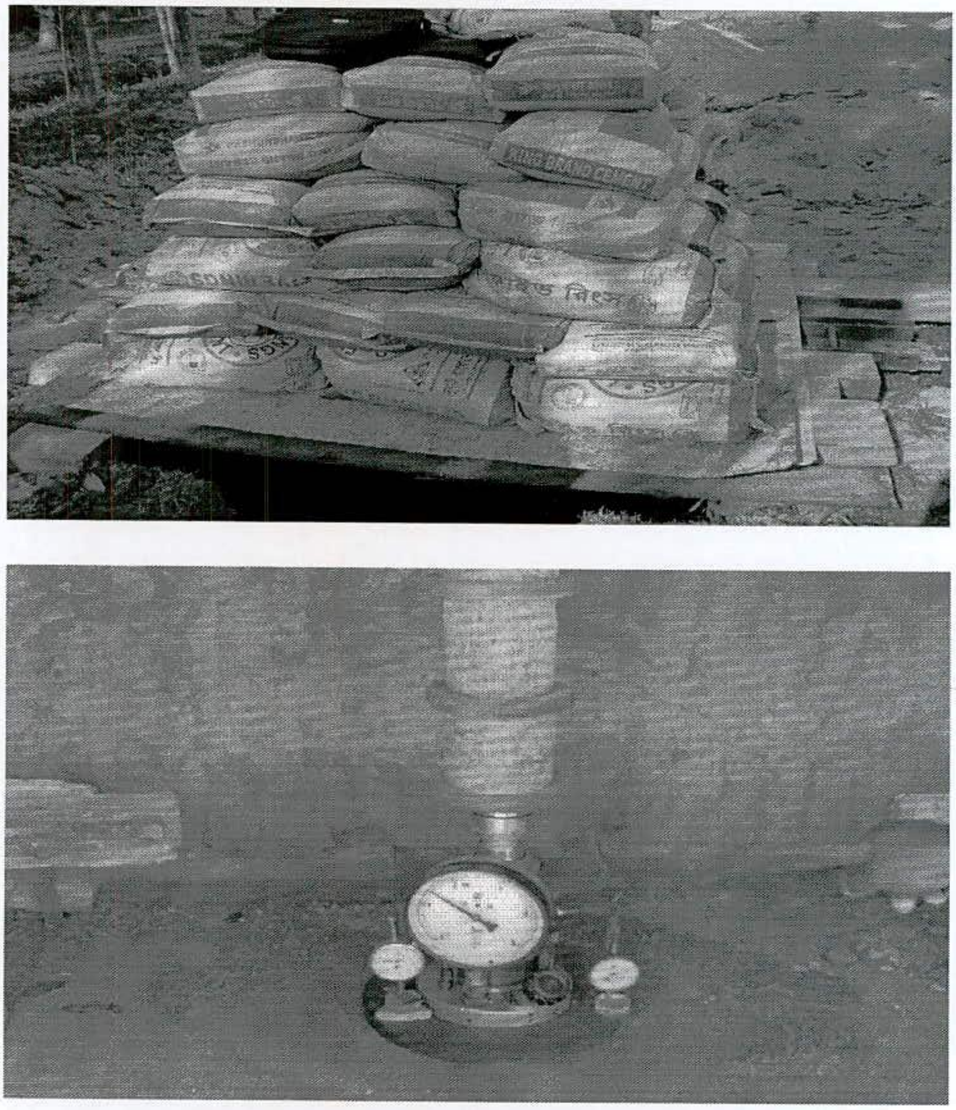


Fig. 4.17 Plate load test on sand pile (One year after sand pile installation)

Table 4.3 Measured settlement values with time for sand pile (One year after sand pile installation)

Load kN/m^2	Time (minutes)	Settlement (mm)		Load kN/m^2	Time (minutes)	Settlement (mm)
67.17	0	0.54		403.02	325	3.87
	5	0.59			330	4.05
	15	0.60			340	4.17
	35	0.61			360	4.22
	65	0.61			390	4.34
134.34	65	1.26		470.19	390	4.71
	70	1.29			395	4.87
	80	1.31			405	4.91
	100	1.31			425	4.93
	130	1.33			455	4.95
201.51	130	1.87		537.36	455	5.49
	135	1.93			460	5.65
	145	1.97			470	5.75
	165	1.99			490	5.80
	195	2.01			520	5.87
268.68	195	2.42		604.53	520	6.11
	200	2.59			525	6.25
	210	2.67			535	6.45
	230	2.69			555	6.57
	260	2.71			585	6.57
335.85	260	3.07		671.70	585	6.77
	265	3.29			590	6.84
	275	3.37			600	6.91
	295	3.43			620	6.98
	325	3.47			650	7.00

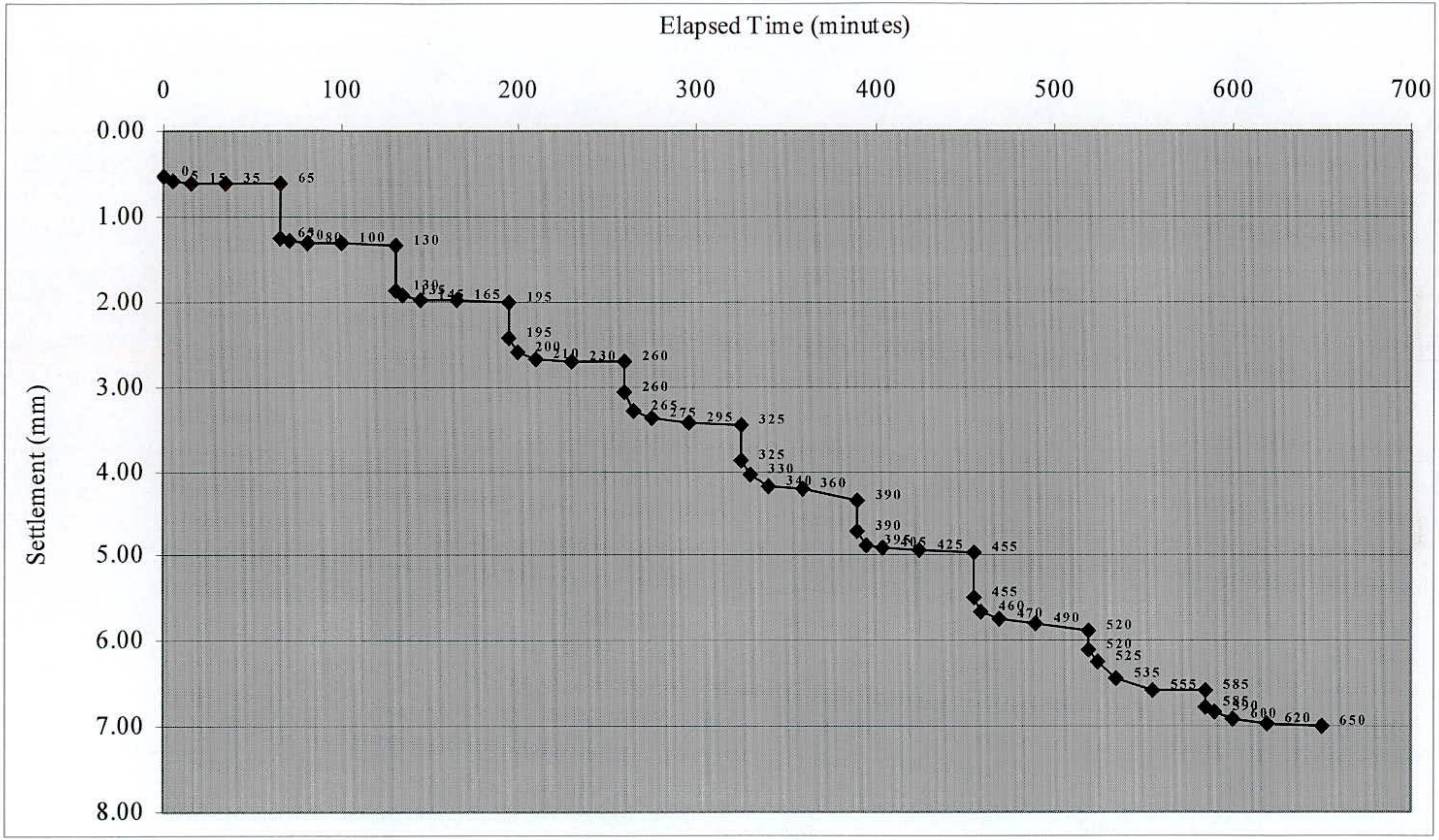


Fig. 4.18 Time versus settlement obtained from load test on improved ground (One year after sand pile installation)

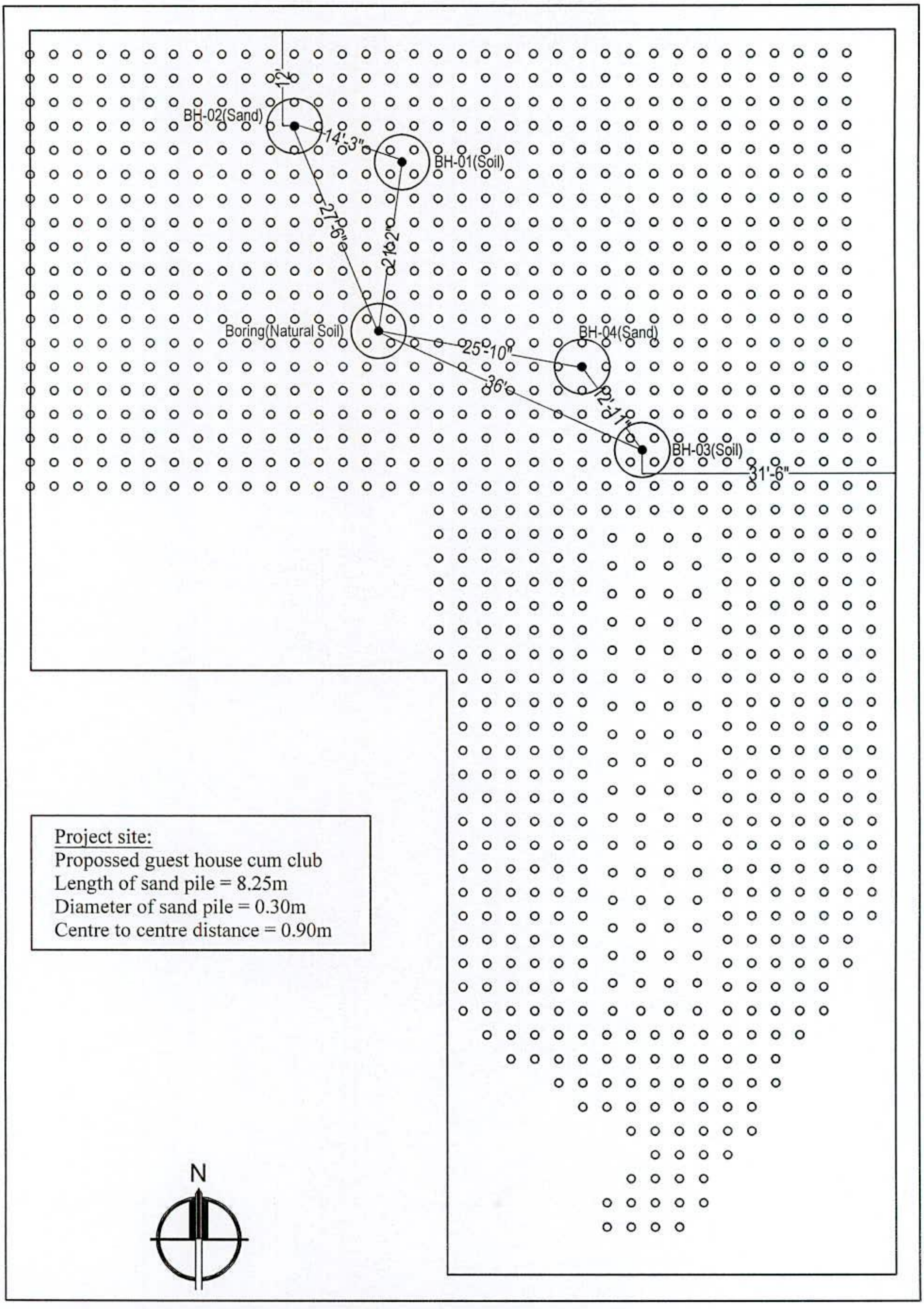


Fig. 4.19 The layout of group sand pile and location of bore holes

4.6 Standard Penetration Test on Improve Ground

The ground improved by granular piles was also investigated by performing Standard Penetration Test. The effectiveness of granular piles in improving soft ground is justified by comparing N-values obtained for natural ground and improved ground. SPT was performed in two location up to a depth 60 ft. from the ground surface, where sand piles are constructed in group. The location of the bore holes are shown in Fig. 4.14. The N-values obtained at the two locations in between the granular piles and center of the granular pile are presenting in Fig. 4.19. The tests were performed around one month and one year after the installation of granular piles.

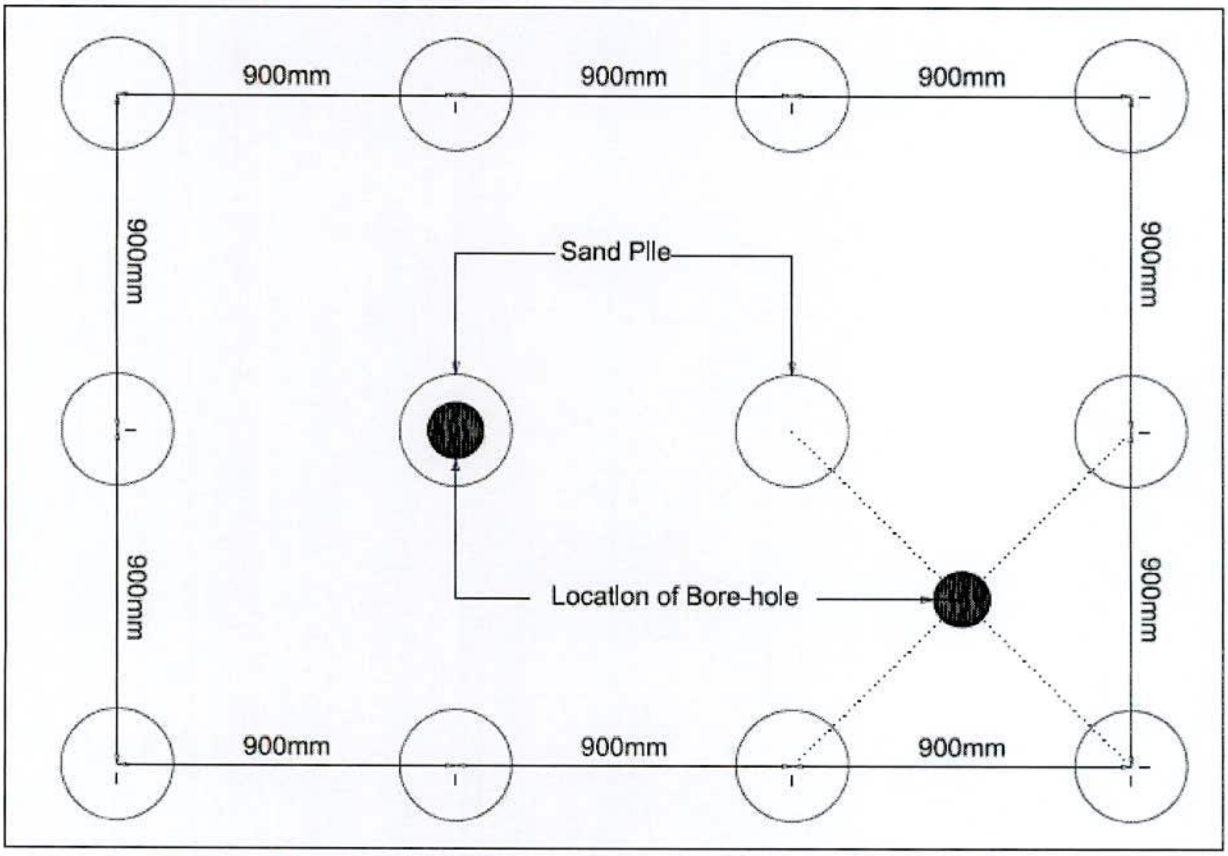


Fig. 4.20 Location of bore holes for Standard Penetration Test

CHAPTER FIVE

RESULT AND DISCUSSION

5.1 General

The result obtained from field investigation are presented and hence discussed in this chapter. The results obtained from plate load test on improved ground for different condition are compared with that of natural ground. Comparison is also made for penetration resistance obtained in the natural and improved ground. Investigation results are also compared with those of the predicted values as well as measured value of other cases.

5.2 Load-Settlement Response of Natural ground

The load-settlement response of natural ground are shown in Fig.5.1. From this figure it can be seen that the settlement increases with the application of load increment. From the plate load on the ground, it was observed that the plate moved downward without giving any resistance after the application of load beyond 671.70kN/m^2. At this load intensity the settlement is 18.00mm

5.3 Load Carrying Capacity of Single Sand Pile

Plate load test were performed on the improved ground for different conditions. There are (i) The plate load test immediately after sand pile installation and (ii) The plate load test one year after sand pile installation. The plate load test results are given in the following sections for different improved conditions.

5.3.1 One month after sand pile installation

Plate load test on the single pile immediately after sand pile installation. To determine the load carrying capacity of single sand pile plate load test was done. The load carrying capacity of the single sand pile is 671.70kN/m^2 for the settlement of 8.20 mm. The settlement profile of the single sand pile reinforced ground is shown in Fig 5.2 for the verification of load intensity. This figure shows that the settlement of the improved ground increase ad the load intensity increases. The field test result shows that the load carrying capacity of the single sand pile is 671.70 kN/m^2 which is about 2.0 times less than that of natural ground.

5.3.2 One year after sand pile installation

Plate load test on the single pile one year after sand pile installation. To determine the load carrying capacity of single sand pile plate load test was done. The load carrying capacity of the single sand pile is 671.70kN/m^2 for the settlement of 7.00 mm. The settlement profile of the single sand pile reinforced ground is shown in Fig 5.2 for the verification of load intensity. This figure shows that the settlement of the improved ground increase ad the load intensity increases. The field test result shows that the load carrying capacity of the single sand pile is 671.70 kN/m^2 which is about 2.50 times less than that of natural ground.

5.4 Comparison of Plate Load Test Results

The test results obtained in the field by plate load test are compared in this section. The improvement of load carrying capacity due to the installation of granular piles in different conditions are compared with that of natural ground. Comparison is also made among the different improvement conditions.

5.4.1 One month after sand pile installation

The plate load test on the sand pile immediately i.e one month after sand pile installation. The improvement of natural ground due to the installation of single granular piles are shown in Fig.5.4 in a load intensity versus settlement diagram. This figure shows that the different settlement at same carrying load (671.70kN/m^2), The load carrying capacity for the natural ground is 671.70kN/m^2 and settlement is 18.00mm, while it is 671.70kN/m^2 for single sand pile and settlement is 8.20mm. The result shows that the installation of granular piles increased the bearing capacity of natural ground significantly irrespective of the materials. The plate load test after one month of installation sand pile, the load carrying capacity is 2.0 times higher for sand piles comparing the natural ground for the allowable settlement. The results also reveals that the bearing capacity increases due to the installation of sand pile is more than two times higher than that of natural ground.

5.4.2 One year after sand pile installation

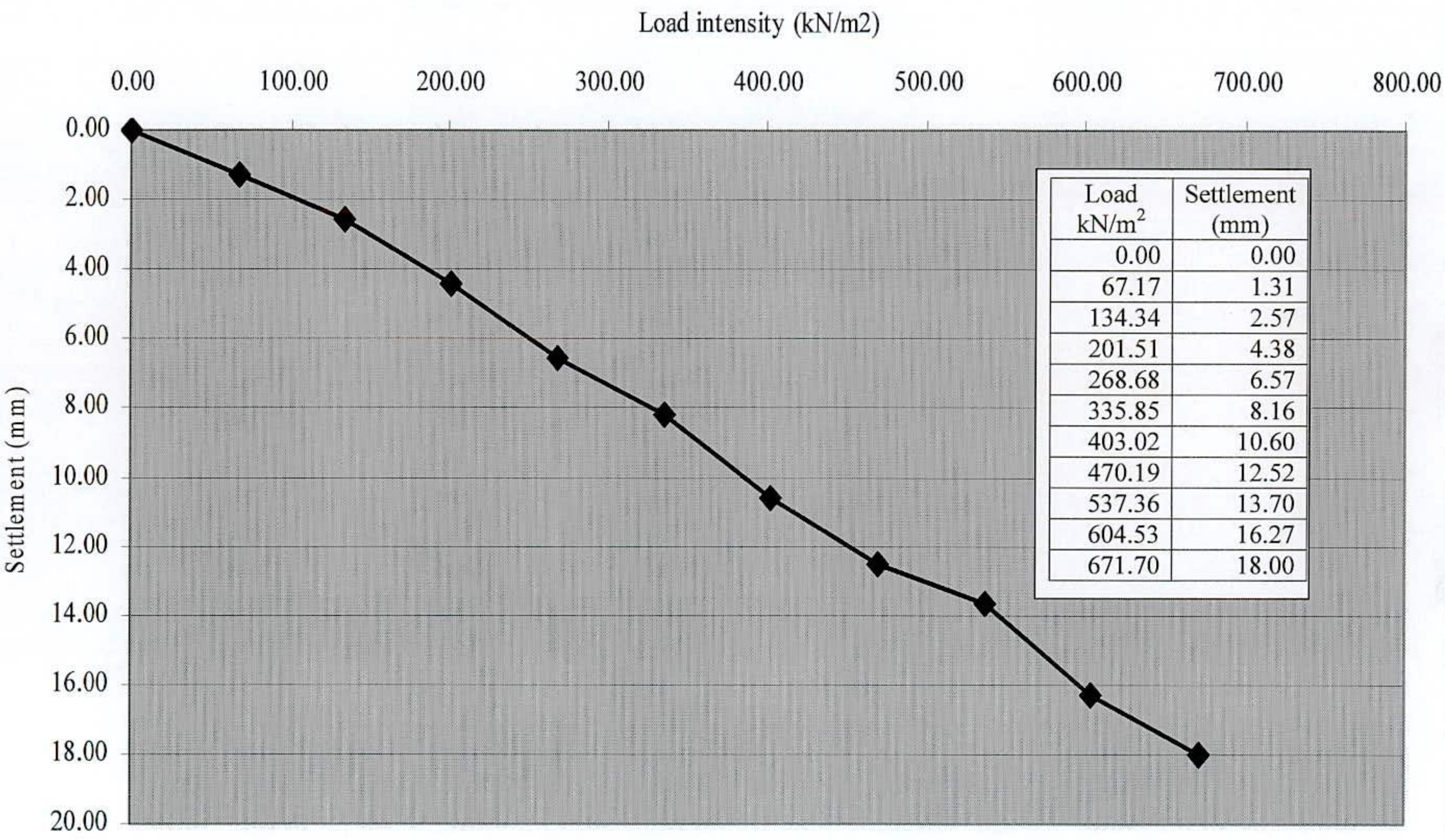
The plate load test on the sand pile one year after sand pile installation. The improvement of natural ground due to the installation of single granular piles are shown in Fig.5.4 in a load intensity versus settlement diagram. This figure shows that the different settlement at same carrying load (671.70kN/m2), The load carrying capacity for the natural ground is 671.70kN/m^2 and settlement is 18.00mm, while it is 671.70kN/m^2 for single sand pile and settlement is 7.00mm. The result shows that the installation of granular piles increased the bearing capacity of natural ground significantly irrespective of the materials. This capacity is due to the spacing of sand piles used in equilateral rectangular arrangement which is 900mm. The plate load test after one year of compaction sand pile, the load carrying capacity is 2.50 times higher for sand piles comparing the natural ground for the allowable settlement. The results also reveals that the bearing capacity increases due to the installation of sand pile is more than two times higher than that of natural ground.

5.4.3 Summary of the plate load test result

The plate load test reveal that the bearing capacity of the improved ground increased significantly due to the installation of granular piles. The arrangement of granular piles, installation pattern and the ratio of bearing capacity of treated (q_t) and natural ground (q_n) are shown in Table 5.1. In this table q_t and q_n represent the load intensity that the treated and natural ground can carry, respectively, during the plate load test for the different settlement at same carrying load. The measure results also show that the installation sand pile and load test immediately after sand pile installation increases the bearing capacity by 2.00 times than that of natural ground corresponding to 7mm settlement. The measure results also show that the installation sand pile and load test one year after sand pile installation increases the bearing capacity by 2.50 times than that of natural ground corresponding to 7mm settlement.

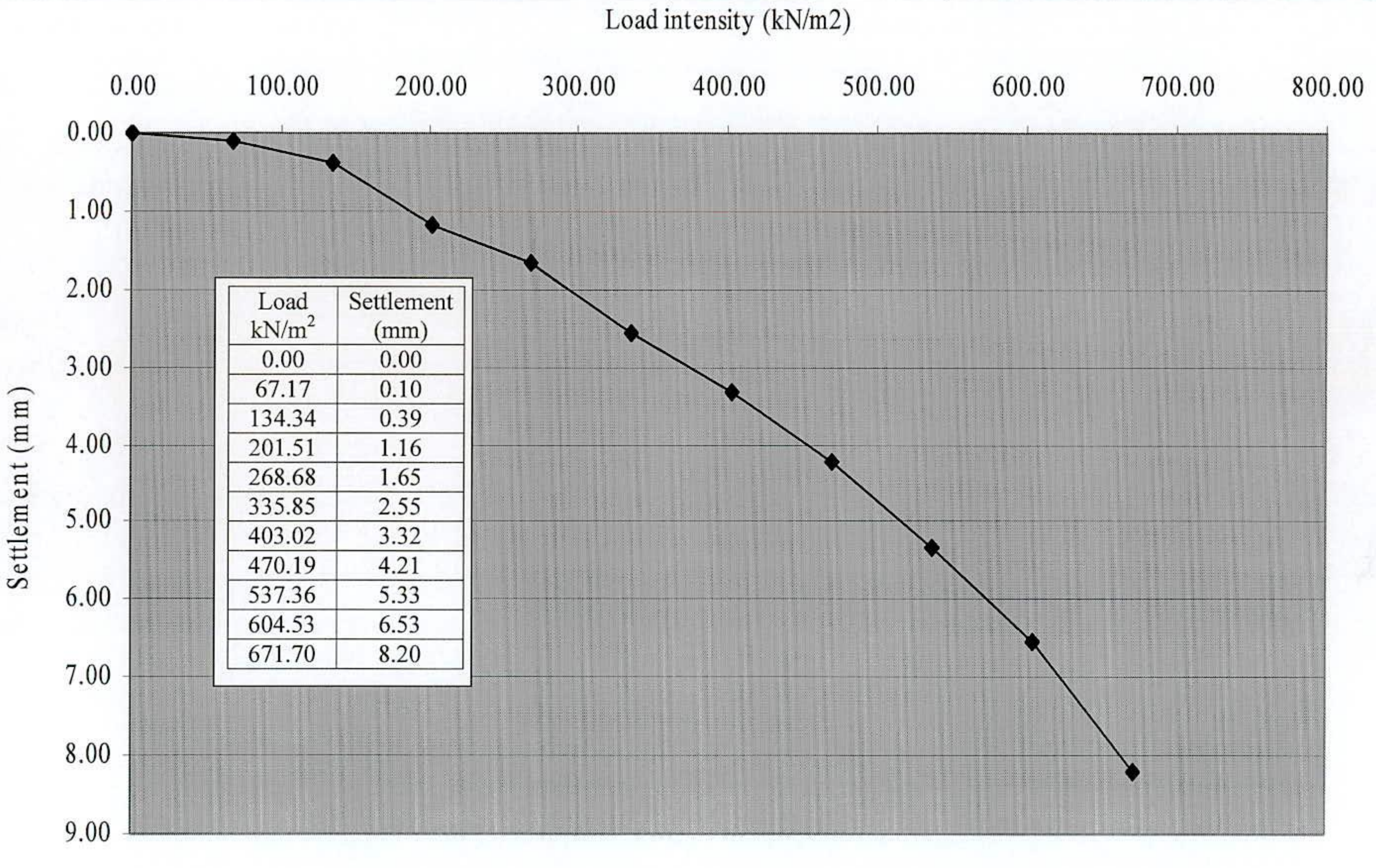
Table 5.1 Measured load carrying capacity of sand pile yields the largest increment of load carrying capacity which is 2.50 times than that of natural ground.

No. of Test	Description of granular piles	Load test after sand pile installation	Location of plate	q_t/q_n (Corresponding to 7.00 mm settlement)
1	Sand piles	After one month	Top of pile	2.00
2	Sand piles	After one year	Top of pile	2.50



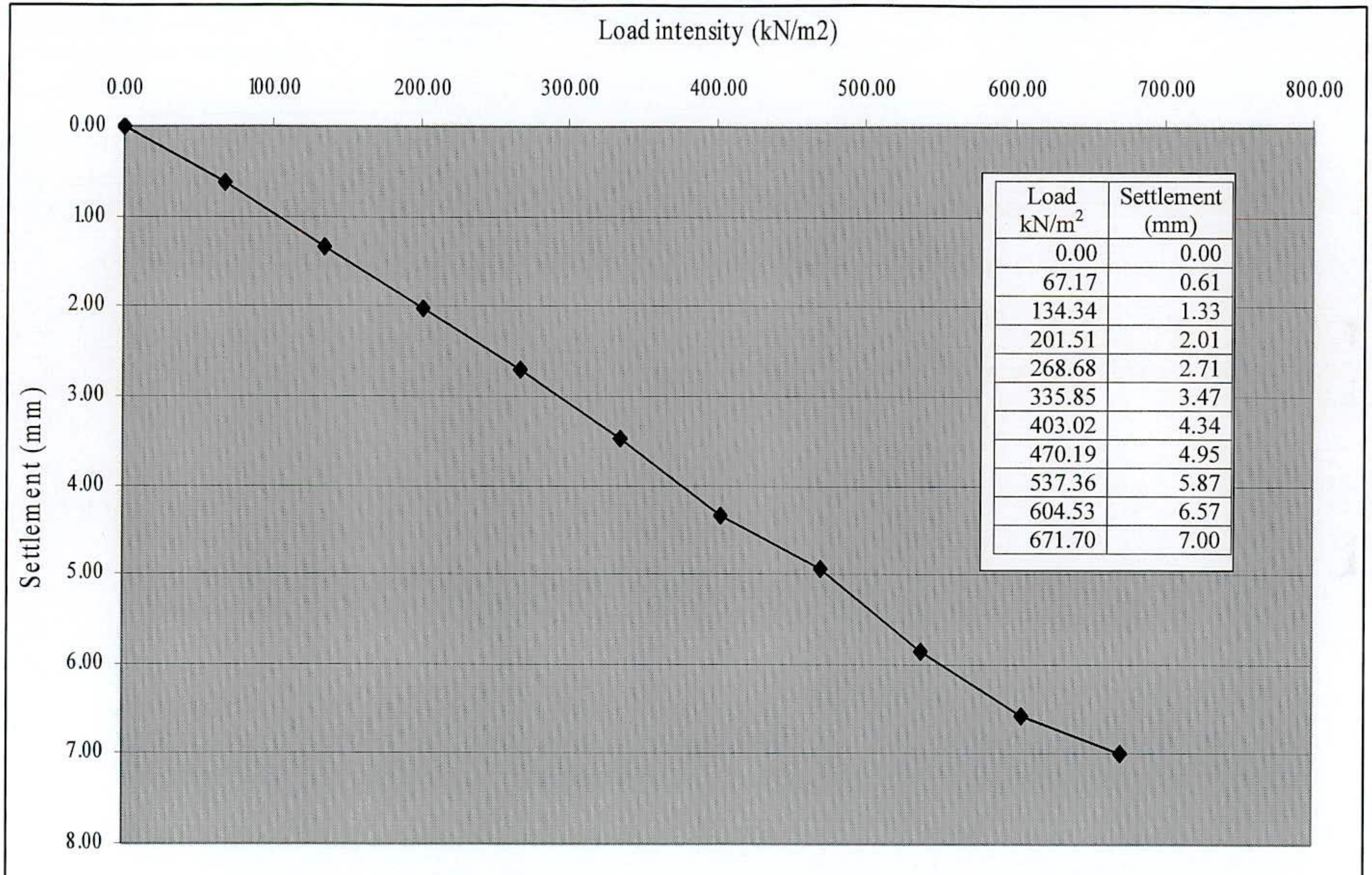
Load kN/m^2	Settlement (mm)
0.00	0.00
67.17	1.31
134.34	2.57
201.51	4.38
268.68	6.57
335.85	8.16
403.02	10.60
470.19	12.52
537.36	13.70
604.53	16.27
671.70	18.00

Fig. 5.1 Load intensity versus settlement for natural ground



Load kN/m^2	Settlement (mm)
0.00	0.00
67.17	0.10
134.34	0.39
201.51	1.16
268.68	1.65
335.85	2.55
403.02	3.32
470.19	4.21
537.36	5.33
604.53	6.53
671.70	8.20

Fig. 5.2 Load intensity versus settlement for sand pile (One month after sand pile installation)



Load kN/m^2	Settlement (mm)
0.00	0.00
67.17	0.61
134.34	1.33
201.51	2.01
268.68	2.71
335.85	3.47
403.02	4.34
470.19	4.95
537.36	5.87
604.53	6.57
671.70	7.00

Fig. 5.3 Load intensity versus settlement for sand pile (One year after sand pile installation)

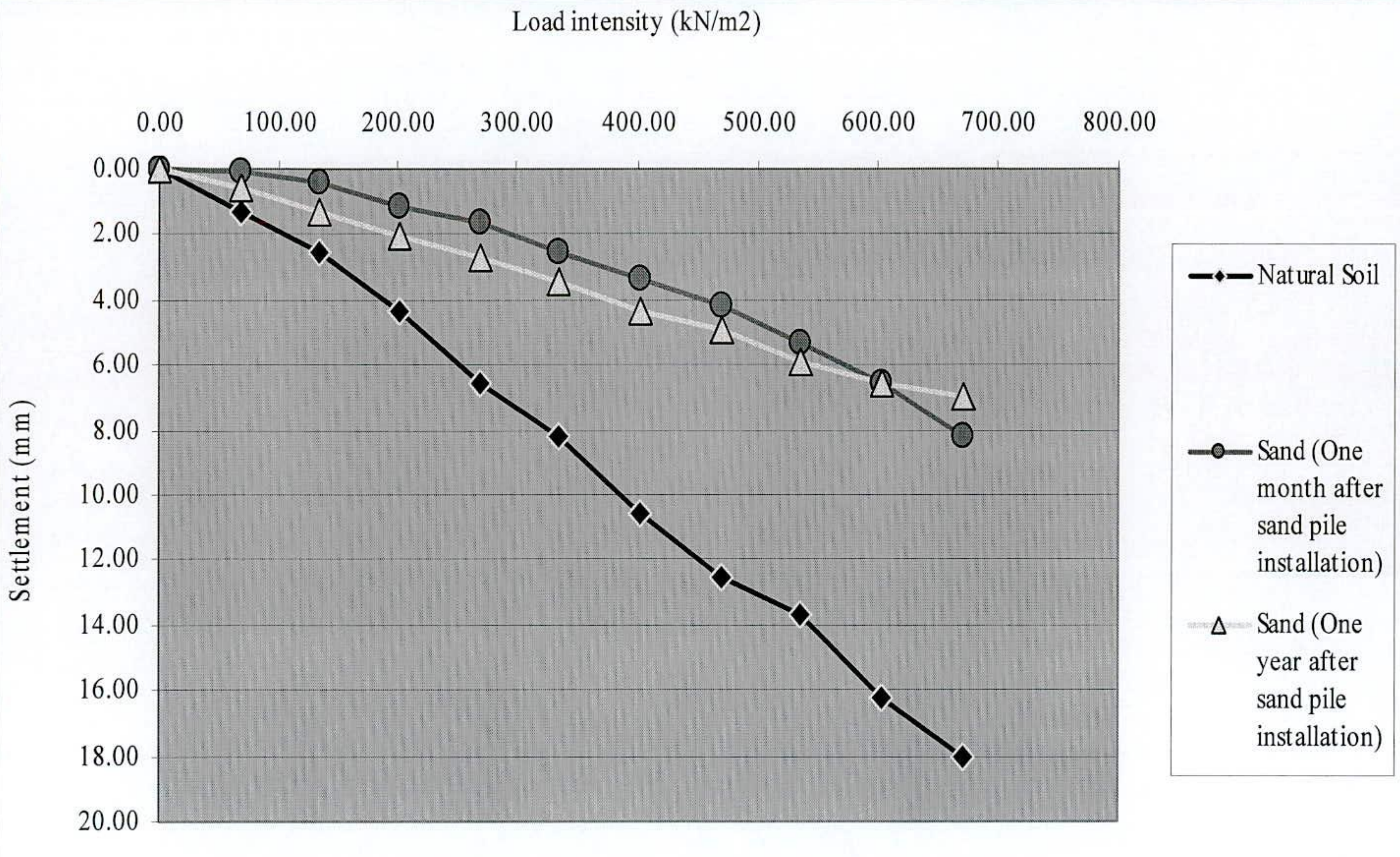


Fig. 5.4 Load intensity versus settlement

Table 5.2 Comparison of SPT result, treated and natural ground. (One month after sand pile installation)

Depth of soil strata (m)	Standard Penetration Test Results (N-Values)		
	Natural Ground (Before installation of sand Pile)	Ground improved by granular piles	
		BH-01 (Soil)	BH-02 (Sand)
0-1.5	7	15	24
1.5-3	4	5	17
3-4.5	3	4	15
4.5-6	4	5	12
6-7.5	5	6	10
7.5-9	7	7	12
9-10.5	3	6	11
10.5-12	4	5	6
12-13.5	6	5	5
13.5-15	6	5	6
15-16.5	5	7	8
16.5-18	6	9	8

5.5 Comparison of Result Obtained from SPT

Standard Penetration Test were performed on the improved ground for different conditions. There are (i) The boring immediately after sand pile installation and (ii) The boring one year after sand pile installation. The Standard Penetration Test results are given in the following sections for different improved conditions.

5.5.1 One month after sand pile installation

Standard Penetration Test was performed in two bore hole of improved ground to depict the improvement of soft ground along the depth due to the penetration installation of granular piles. The penetration resistance i.e. N-values obtained in two boreholes are compared with those of natural ground before granular piles installation. The comparison is shown in Fig. 5.5. This Figure shows that N-values ranges 3 to 9. for the natural ground, while the values increases to 4 to 15 and 5 to 24 for the bore holes one and two respectively, in case of improved ground. The N-values of improved ground of depth 12 m to 15 m decreased slightly from the N-values of natural soil which is an unusual phenomenon of this study. During the installation of sand pile, the soil surrounding the pile is disturbed due to continuous hammering and then compressible soil is lost their capacity. The inadequate pouring of sand, lack of compaction as well as dispersion of sand in the surrounding soil during hammering are the main causes of producing lower N-values on the sand pile of improved ground when compared to the untreated one.

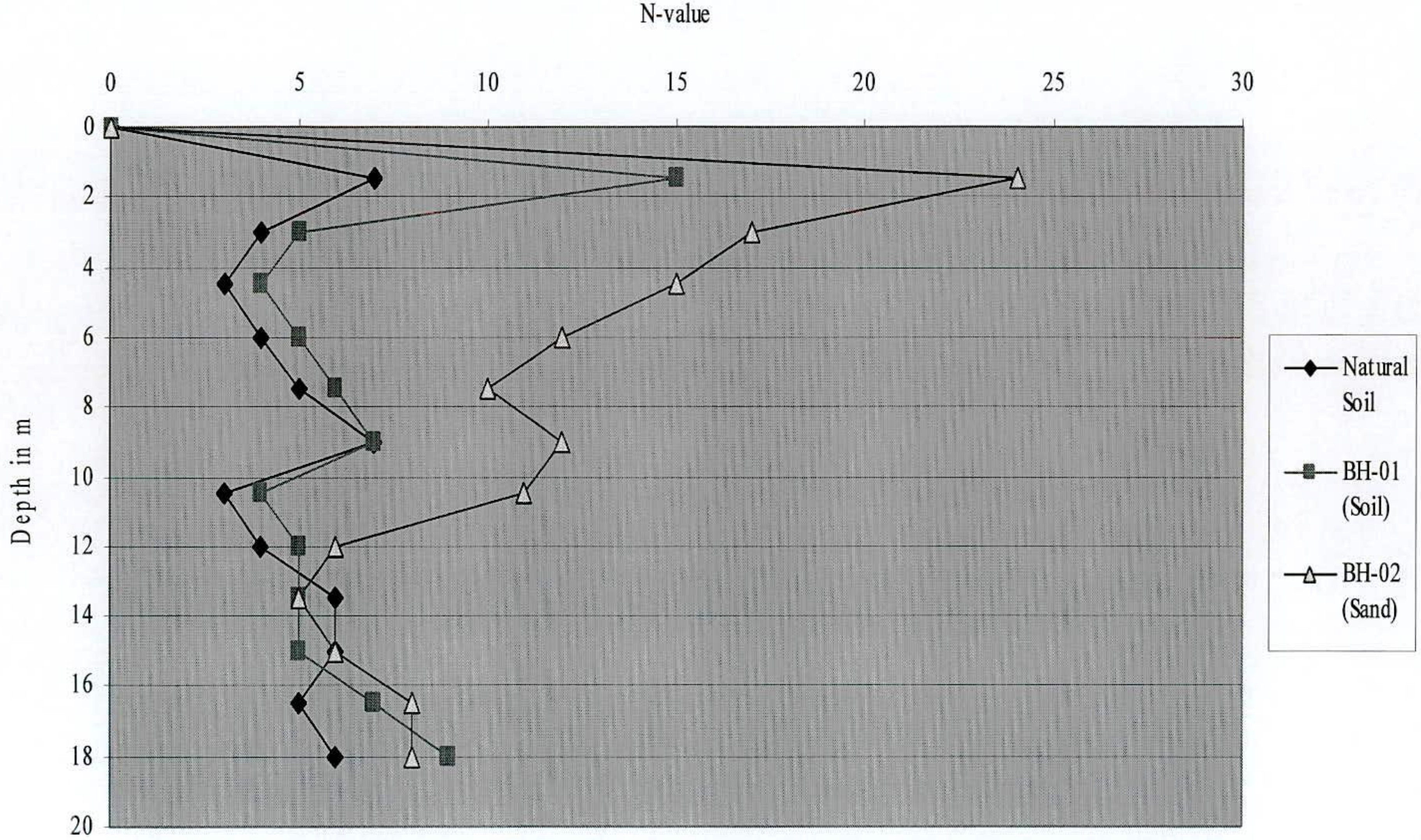


Fig. 5.5 Comparision of SPT result, treated and natural ground (One month after sand pile installation)

Table 5.3 Comparison of SPT result, treated and natural ground. (One year after sand pile installation)

Depth of soil strata (m)	Standard Penetration Test Results (N-Values)		
	Natural Ground (Before installation of sand Pile)	Ground improved by granular piles	
		BH-03 (Soil)	BH-04 (Sand)
0-1.5	7	12	19
1.5-3	4	4	14
3-4.5	3	6	13
4.5-6	4	4	12
6-7.5	5	10	10
7.5-9	7	8	9
9-10.5	3	9	9
10.5-12	4	5	6
12-13.5	6	5	5
13.5-15	6	5	6
15-16.5	5	7	8
16.5-18	6	9	8

5.5.2 One year after sand pile installation

Standard Penetration Test was performed in two bore hole of improved ground to depict the improvement of soft ground along the depth due to the penetration installation of granular piles. The penetration resistance i.e. N-values obtained in two boreholes are compared with those of natural ground before granular piles installation. The comparison is shown in Fig. 5.6. This figure shows that N-values ranges 3 to 9. for the natural ground, while the values increases to 4 to 12 and 5 to 19 for the bore holes three and four respectively, in case of improved ground. The N-values of improved ground of depth 12 m to 15 m decreased slightly from the N-values of natural soil which is an unusual phenomenon of this study. During the installation of sand pile, the soil surrounding the pile is disturbed due to continuous hammering and then compressible soil is lost their capacity. The inadequate pouring of sand, lack of compaction as well as dispersion of sand in the surrounding soil during hammering are the main causes of producing lower N-values on the sand pile of improved ground when compared to the untreated one.

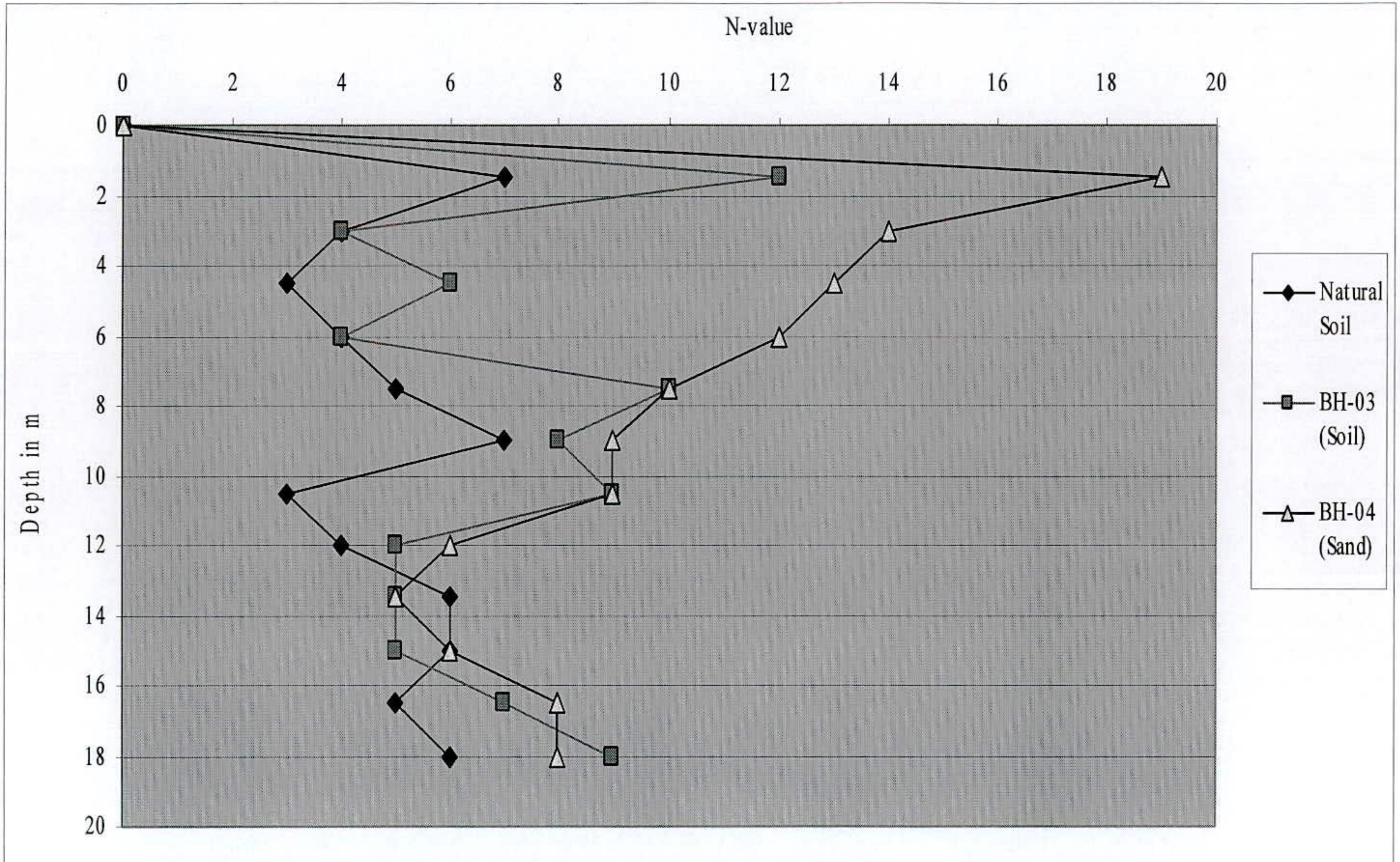


Fig. 5.6 Comparision of SPT result, treated and natural ground (One year after sand pile installation)

5.6 Prediction of Ultimate Bearing Capacity of Granular Piles

In general the failure nature of isolated granular pile embedded in the soft soil is found as building as shown in Fig.2.3. Here the ultimate capacity of a single granular pile is predicted considering typical condition. According to unit cell concept and given formulas in Article 2.11 at chapter two, a single sand pile is considered among the group sand piles for prediction of ultimate bearing capacity. Here the ultimate bearing capacity of sand pile is predicted incorporated with the following formulas.

$a_s = c_1 (D/S)^2$

Where

a_s= Area replacement ratio

$c_1 = \pi/4$

for square pattern

D= Diameter of pile =300mm

S= Center to center spacing of granular pile=900mm

$a_s = 0.10$

$q_{ult} = \sigma_3 \tan^2\beta + 2c_{avg} \tan\beta$

Where,

q_{ult} = Ultimate bearing capacity of single sand pile

σ_3= Principal stress

β= Failure surface inclination

c = Undrained shear strength= 25.86kN/m^2

Where

$\sigma_3 = \frac{\gamma_c}{2} \tan 45 + 2c$

$\beta = 45 + \frac{\phi_{avg}}{2}$

Considering the geotechnical properties of soil of the site as given in Table 3.1 and the values of above perimeters are predicted as follows and the details of each notations are given in chapter two in article 2.11.4.

$\phi_{avg} = \tan^{-1}(\mu_s a_s \tan\phi_s)$

$\mu_s = \frac{\sigma_s}{\sigma} = 1.8$

$\tan\phi_s = 0.781$ [for sand]

$\gamma_c = 19.12$

$C_{avg} = (1-a_s)c = 23.27 kN/m^2$

$\beta = 49.00$

$\sigma_3 = 61.28\ kN/m^2$

Considering the above values, the ultimate bearing capacity of the sand pile is calculated as,

$q_{ult} = 134.63 kN/m^2$

From the predicted and measured values, it can be seen that the measured load carrying capacity of the sand piles obtained from plate load test (268.68 kN/m^2 to 671.70 kN/m^2) is higher than that of prediction (134.63 kN/m^2).

5.7 Comparison of Predicated Values and Field Results

The predicted and measured values of ultimate bearing capacity of treated ground for different configuration are described here. In this respect the ultimate bearing capacity of treated ground is predicted with respect of various angle of shearing resistance (Ø) and specific used the method of ultimate capacity of composite soil, but at field investigation it is shown that the measured value of bearing capacity is more accurate than predicted value and it is occurred due to respect of soil profile and physical properties of sand pile granular materials such as sand pile dimension, sand pile arrangement and stiffness of sand pile materials (Ø) which depends on materials properties, overburden pressure, compaction, lateral pressure etc. From this study it is revealed that the field measured value of ultimate bearing capacity of untreated and treated ground such as immediately and one year after sand pile installation whose measured value is gradually increased from value of untreated soil. Also the field measured value of ultimate bearing capacity of treated and untreated soil is higher than that of the predicted value of ultimate bearing capacity.

Table 5.4 Comparison of prediction and measured value

Ultimate bearing capacity (kPa) corresponding to 7mm settlement		
Untreated ground	Predicted value (q_{ult})	134.63
Natural ground	Field value (q_{ult})	268.68
One month after sand pile installation	Field value (q_{ult})	604.53
One year after sand pile installation	Field value (q_{ult})	671.70

5.8 Comparison of Field Test Results with other Published Results

A series of test data available in literature are compared with the proposed model. As all the relevant data are not well documented, the average values of the parameters are considered for the predictions. For the predictions of settlement of reinforced ground by the proposed model the values of the parameters are taken as p_0/E_s=0.10, E_c/E_s=5 to 30 and V_s=0.40. It is considered that the reinforced ground is covered by a layer of well compacted granular fill having reasonably high value of stiffness. This considerations ensures uniform settlement over the entire composite ground. The results are presented in Fig 5.7 in the form of settlement reduction ratio versus spacing ratio of columns. The data points are plotted for the measured values reported in the literature while the predictions by the proposed model are indicated as full lines for the variation of E_c/E_s form 5 to 30. The variation of settlement reduction ratio with spacing for the data is considerable. These variations may be due to the differences in site conditions and methods of granular column installation. While the results are not conclusive, since not enough well documented data are available from the field, these figures support the validity of proposed model in the field scale.

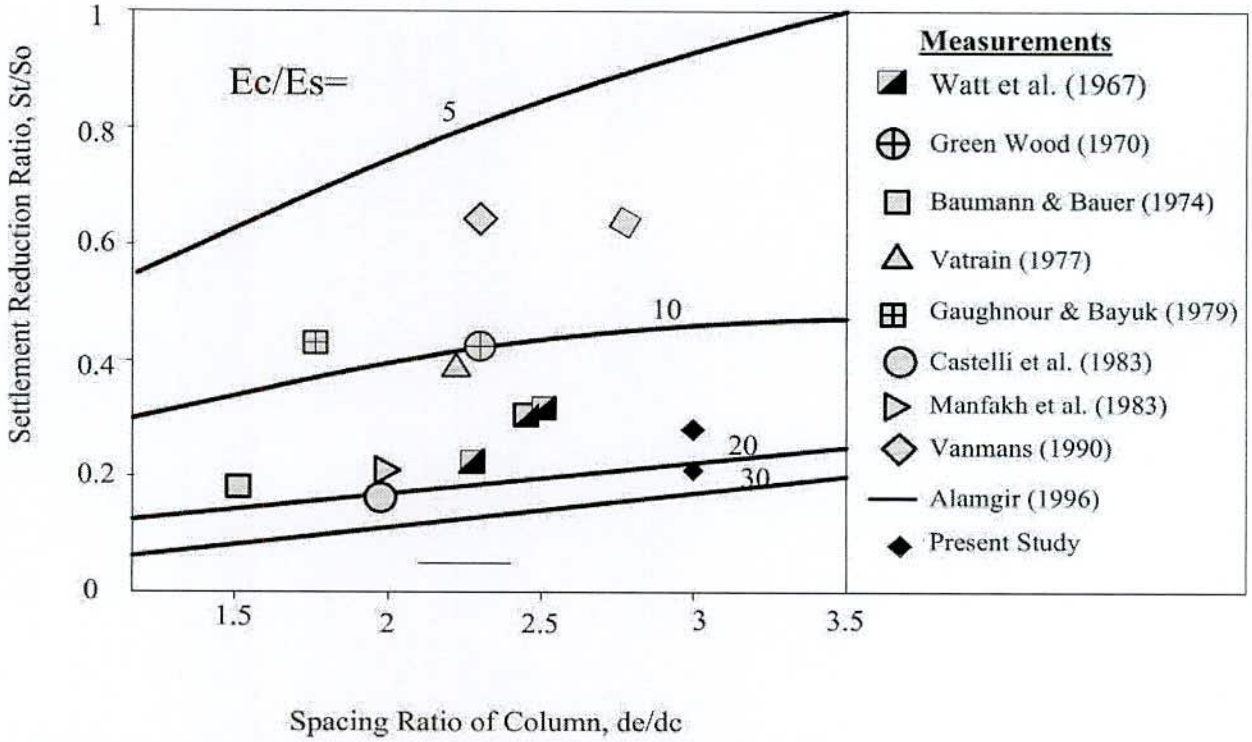


Fig. 5.7 Validation of the proposed model for predicting the settlement reduction ratio.
(Alamgir, M. 1996.)

5.9 Comparison of Cost for Sand Pile Solution with Conventional Foundation

The summary of estimated quantity of materials with their expected total cost of three different types of footing has given in the Table 5.5. The total cost of mat foundation has dominated by the cost of reinforcing steel which is approximately 160% and 72.8% more the cost of reinforcing steel than the continuous footing with sand pile and pre-cast pile foundation, respectively. On the other hand, the total estimated construction and installation cost for mat foundation and continuous footing decreased significantly about 82% and 26%, respectively when compared with pre-cast pile foundation. However, the total cost of mat and pre-cast pile foundation increased approximately 114% and 154% respectively while compared with their other counterpart. From the analysis, it is clear that the sand pile with continuous footing has given more economical solution in this case, however, after installation of sand pile, the bearing capacity of soil should be justified before adopting continuous footing. The details structural design of continuous footing, mat foundation and pre-cast pile foundation has given in the appendix D, E and F respectively.

Table 5.5 Cost analysis of using sand pile with respect to mat and pre-cast pile

Details Estimate of Sand Pile with Continuous Footing						
Name of Item	Description of Item	Unit	Quantity	Rate	No of Item	Taka
Sand Pile	Sand	cft	32.00	25.00	1350.00	1080000.00
	Installation Cost	rft	27.00	50.00	1350.00	1822500.00
Continous Footing	Cement	Bag	1200.00	425.00	1.00	510000.00
	Sand	cft	2250.00	25.00	1.00	56250.00
	Stone Chips	cft	4500.00	125.00	1.00	562500.00
	Steel	ton	25.00	60000.00	1.00	1500000.00
	Construction Cost	sft	5000.00	50.00	1.00	250000.00
Total Taka						57,81,250.00
Details Estimate of Mat Foundation						
Name of Item	Description of Item	Unit	Quantity	Rate	No of Item	Taka
Mat Foundation	Cement	Bag	2500.00	425.00	1.00	1062500.00
	Sand	cft	4000.00	25.00	1.00	100000.00
	Stone Chips	cft	8000.00	125.00	1.00	1000000.00
	Steel	ton	65.00	60000.00	1.00	3900000.00
	Construction Cost	sft	10000.00	50.00	1.00	500000.00
Total Taka						65,62,500.00
Details Estimate of Pre-cast Pile						
Name of Item	Description of Item	Unit	Quantity	Rate	No of Item	Taka
Pile	Cement	Bag	38.00	425.00	80.00	1292000.00
	Sand	cft	72.00	25.00	80.00	144000.00
	Stone Chips	cft	145.00	125.00	80.00	1450000.00
	Steel	ton	0.36	60000.00	80.00	1728000.00
	Installation Cost	rft	100.00	300.00	80.00	2400000.00
Pile Cap	Cement	Bag	9.00	425.00	80.00	306000.00
	Sand	cft	16.00	25.00	80.00	32000.00
	Stone Chips	cft	33.00	125.00	80.00	330000.00
	Steel	ton	0.18	60000.00	80.00	864000.00
	Construction Cost	sft	16.00	300.00	80.00	384000.00
Total Taka						89,30,000.00

Table 5.6 Percent of cost increase of sub structure with respect to sand pile

Type of Foundation	% of cost increase
Sand Pile	100
Mat	114
Pre-cast pile	154

CHAPTER SIX

SUMMARY AND CONCLUSIONS

6.1 Summary

The development of modern foundation practices, ground improvement techniques, to overcome the limitations of the conventional foundation system has been proved to be a viable option both technically and economically for the improvement of the marginal sites. Amongst the various ground improvement techniques for improving soft ground conditions, granular piles are considered as one of the most versatile and cost effective method. The granular piles can be of the form such as stone column, sand compaction piles, lime or cement column etc., which are stiffer than the surrounding soil. This ground improvement techniques have been used in many difficult foundation sites throughout the world to increase the bearing capacity, reduce settlement, increase the rate of consolidation, improve embankment stability and resistance to liquefaction.

This study is concerned with the performance of granular piles i.e. sand piles installed in soft ground at a selected site of KUET campus, KUET, Khulna. The soil at the site consists of very soft to soft clayey soils. The granular piles were installed by rammed-displacement method. Here a rig used for the installation of conventional pile foundation is for the installation of granular piles by rammed-displacement method in dry condition. Rammed-displacement is a dry method for the installation of sand piles by hammering a weight of 9.801KN from a height of 750mm to 1000mm. Individual granular pile diameter of 300mm and were installed in single and a group pattern of equilateral rectangle at a spacing of 900mm.

The sand piles were constructed of sylhet sand and the Fineness Modulus (F.M) of sylhet sand was 2.58. The performance study was limited to the improvement of bearing capacity and settlement behavior of the soft soils. After one and twelve months of installation of sand piles, the plate load tests were carried out. Total three load tests were performed. One over normal ground and others two on the sand piles. From test results the load-settlement response of the improved ground is obtained. The results reveal that the bearing capacity of the normal ground was increased significantly by the installation of granular piles significantly.

Comparing the natural ground, the bearing capacity of improved ground was increased by 200% and 250% for sand piles, respectively. After one and twelve months of the installation of granular piles the improvement of the site is also examined by performing standard penetration test (SPT) till the depth of 18m in four specified locations. The sub soil profile shows that the N-values increased by 2-3 fold than that of natural ground.

6.2 Conclusions

Based on the construction of granular piles and the related field investigations reported in this study, the following conclusions can be made:

i. The soft ground improvement using granular piles technique is revealed as fast, economical and an efficient method to improve weak soil.

ii. The simple construction procedures and the related equipment adopted in this project for the installation of the desired granular piles were found to provide high degree of effectiveness.

iii. Comparing to the natural ground, the bearing capacity of improved ground was increased by 200% to 250%.

iv. The field investigation on the improved ground by plate-load test revealed that the granular piles improved substantially the bearing capacity of the natural soil.

v. The field experience during the installation of granular pile depicts that such installation technique required very close monitoring and precaution in case of group pile arrangement.

vi. Proper FM of sand and pouring of granular materials are required to ensure the stiffness of granular pile. The layer thickness of granular material, dropping height, number and placement of hammer are to be maintained properly through close field monitoring to achieve the designed stiffness of the granular pile.

vii. The better increment of bearing capacity by granular pile are observed than that of natural soil.

viii. Standard Penetration Test (SPT) results also reveal that the significant improvement of the ground can be achieved along the depth due to the installation of granular piles.

ix. The predicated ultimate load carrying capacity of single sand pile is less than that of measured capacity of single sand pile obtained from plate load test in the field.

x. The cost of mat and pre-cast pile foundation increased approximately 114% and 154% respectively while compared with sand pile

xi. The practicing engineer can get help from this study and experience to improve the soft ground by the installation of granular piles.

RECOMMENDATIONS FOR FUTURE STUDY

Based on the present study the following recommendations for future research can be made.

i. To establish the relation between the number of blows at different layer of granular columns by using sand and stone chips as column material to the bearing capacity of granular piles improved ground.

ii. A field study on load carrying capacity of geo-textile jacketed granular piles having lateral reinforcement within the granular piles can be an interesting future research works.

iii. To establish the effectiveness of the adopted installation technique, various type of installation technique can be used for the construction of granular pile in single and croup.

iv. To establish the load carrying capacity of granular piles using different types of material proportion can be a future research works.

v. Long term settlement behavior of the improvement ground can be investigated in future.

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APPENDIX-A

DEPARTMENT OF CIVIL ENGINEERING

Khulna University of Engineering & Technology

Khulna-9203.

FIELD RECORD OF PLATE LOAD TEST

ASTM D 1134

TEST NO.:

Name of the Project : Thesis

Location Site : Guest House Cum Club , KUET.

Design Pile Load : 1250 kg Ton's Total Test Load : 5000 kg Ton's Ram Dim: 1ft

Specification of Pile :

Type of Sub- Soil : Natural Ground

Type of Pile : Natural Ground

Date of Pile Driving/ Construciton :

Pile Diamenter : 1 ft Length: 27 ft

Date of Testing : 04.03.2012

Date	Load (Ton's)	Clock Time	Interval for Recording (Min's)	Elapsed Time (hrs.)	Dial Reading (Left)	Dial Reading (Right)	Average Reading	Displacement (mm)
	500 kg	8.00	0	0	109.00	120.00	114.50	1.15
		8.05	5	5	117.00	129.00	123.00	1.23
		8.15	10	15	118.00	130.00	124.00	1.24
		8.35	20	35	122.00	133.00	127.50	1.28
		9.05	30	65	125.00	136.00	130.50	1.31
	1000 kg	9.05	0	65	163.00	182.00	172.50	1.73
		9.10	5	70	165.00	214.00	189.50	1.90
		9.20	10	80	181.00	245.00	213.00	2.13
		9.40	20	100	193.00	285.00	239.00	2.39
		10.10	30	130	197.00	317.00	257.00	2.57
	1500 kg	10.10	0	130	245.00	387.00	316.00	3.16
		10.15	5	135	262.00	407.00	334.50	3.35
		10.25	10	145	285.00	442.00	363.50	3.64
		10.45	20	165	330.00	490.00	410.00	4.10
		11.15	30	195	358.00	518.00	438.00	4.38
	2000 kg	11.15	0	195	390.00	550.00	470.00	4.70
		11.20	5	200	418.00	574.00	496.00	4.96
		11.30	10	210	465.00	625.00	545.00	5.45
		11.50	20	230	525.00	700.00	612.50	6.13

Date	Load (Ton's)	Clock Time	Interval for Recording (Min's)	Elapsed Time (hrs.)	Dial Reading (Left)	Dial Reading (Right)	Average Reading	Displacement (mm)
		12.20	30	260	571.00	743.00	657.00	6.57
	2500 kg	12.20	0	260	620.00	790.00	705.00	7.05
		12.25	5	265	656.00	824.00	740.00	7.40
		12.35	10	275	662.00	832.00	747.00	7.47
		12.55	20	295	668.00	834.00	751.00	7.51
		1.25	30	325	780.00	852.00	816.00	8.16
	3000 kg	1.25	0	325	805.00	971.00	888.00	8.88
		1.30	5	330	831.00	997.00	914.00	9.14
		1.40	10	340	900.00	1080.00	990.00	9.90
		2.00	20	360	972.00	1132.00	1052.00	10.52
		2.30	30	390	981.00	1138.00	1059.50	10.60
	3500 kg	2.30	0	390	1080.00	1245.00	1162.50	11.63
		2.35	5	395	1142.00	1295.00	1218.50	12.19
		2.45	10	405	1152.00	1311.00	1231.50	12.32
		3.05	20	425	1172.00	1322.00	1247.00	12.47
		3.35	30	455	1181.00	1323.00	1252.00	12.52
	4000 kg	3.35	0	455	1223.00	1350.00	1286.50	12.87
		3.40	5	460	1250.00	1405.00	1327.50	13.28
		3.50	10	470	1274.00	1417.00	1345.50	13.46
		4.10	20	490	1280.00	1425.00	1352.50	13.53
		4.40	30	520	1285.00	1455.00	1370.00	13.70
	4500 kg	4.40	0	520	1405.00	1520.00	1462.50	14.63
		4.45	5	525	1505.00	1641.00	1573.00	15.73
		4.55	10	535	1519.00	1656.00	1587.50	15.88
		5.15	20	555	1537.00	1669.00	1603.00	16.03
		5.45	30	585	1585.00	1669.00	1627.00	16.27
	5000 kg	5.45	0	585	1608.00	1712.00	1660.00	16.60
		5.50	5	590	1693.00	1810.00	1751.50	17.52
		6.00	10	600	1719.00	1831.00	1775.00	17.75
		6.20	20	620	1735.00	1845.00	1790.00	17.90
		6.50	30	650	1750.00	1850.00	1800.00	18.00

APPENDIX-B

DEPARTMENT OF CIVIL ENGINEERING
Khulna University of Engineering & Technology
Khulna-9203.

FIELD RECORD OF PLATE LOAD TEST

ASTM D 1134

TEST NO.:

Name of the Project : Thesis

Location Site : Guest House Cum Club , KUET.

Design Pile Load : 1250kg Ton's Total Test Load : 5000kg Ton's Ram Dim: 1ft

Specification of Pile :

Type of Sub- Soil :

Type of Pile : Sand Pile

Date of Pile Driving/ Construciton : 19.04.12

Pile Diamenter : 1 ft Length: 27 ft

Date of Testing : 18.11.2012

Date	Load (Ton's)	Clock Time	Interval for Recording (Min's)	Elapsed Time (hrs.)	Dial Reading (Left)	Dial Reading (Right)	Average Reading	Displaceme nt (mm)
	500 kg	8.00	0	0	4.00	4.00	4.00	0.04
		8.05	5	5	7.00	8.00	7.50	0.08
		8.15	10	15	8.00	8.00	8.00	0.08
		8.35	20	35	8.00	9.00	8.50	0.09
		9.05	30	65	8.00	9.00	8.50	0.09
	1000 kg	9.05	0	65	30.00	28.00	29.00	0.29
		9.10	5	70	31.00	28.00	29.50	0.30
		9.20	10	80	35.00	30.00	32.50	0.33
		9.40	20	100	37.00	34.00	35.50	0.36
		10.10	30	130	40.00	38.00	39.00	0.39
	1500 kg	10.10	0	130	109.00	97.00	103.00	1.03
		10.15	5	135	116.00	103.00	109.50	1.10
		10.25	10	145	118.00	104.00	111.00	1.11
		10.45	20	165	120.00	106.00	113.00	1.13
		11.15	30	195	123.00	109.00	116.00	1.16

Date	Load (Ton's)	Clock Time	Interval for Recording (Min's)	Elapsed Time (hrs.)	Dial Reading (Left)	Dial Reading (Right)	Average Reading	Displacement (mm)
	2000 kg	11.15	0	195	155.00	141.00	148.00	1.48
		11.20	5	200	164.00	148.00	156.00	1.56
		11.30	10	210	169.00	152.00	160.50	1.61
		11.50	20	230	173.00	153.00	163.00	1.63
		12.20	30	260	176.00	154.00	165.00	1.65
	2500 kg	12.20	0	260	221.00	199.00	210.00	2.10
		12.25	5	265	240.00	215.00	227.50	2.28
		12.35	10	275	248.00	222.00	235.00	2.35
		12.55	20	295	256.00	230.00	243.00	2.43
		1.25	30	325	269.00	240.00	254.50	2.55
	3000 kg	1.25	0	325	300.00	265.00	282.50	2.83
		1.30	5	330	313.00	276.00	294.50	2.95
		1.40	10	340	334.00	294.00	314.00	3.14
		2.00	20	360	342.00	301.00	321.50	3.22
		2.30	30	390	354.00	310.00	332.00	3.32
	3500 kg	2.30	0	390	387.00	343.00	365.00	3.65
		2.35	5	395	415.00	362.00	388.50	3.89
		2.45	10	405	425.00	369.00	397.00	3.97
		3.05	20	425	436.00	379.00	407.50	4.08
		3.35	30	455	455.00	386.00	420.50	4.21
	4000 kg	3.35	0	455	491.00	432.00	461.50	4.62
		3.40	5	460	528.00	454.00	491.00	4.91
		3.50	10	470	540.00	472.00	506.00	5.06
		4.10	20	490	552.00	483.00	517.50	5.18
		4.40	30	520	569.00	497.00	533.00	5.33
	4500 kg	4.40	0	520	598.00	532.00	565.00	5.65
		4.45	5	525	648.00	567.00	607.50	6.08
		4.55	10	535	655.00	582.00	618.50	6.19
		5.15	20	555	687.00	601.00	644.00	6.44
		5.45	30	585	696.00	609.00	652.50	6.53
	5000 kg	5.45	0	585	738.00	650.00	694.00	6.94
		5.50	5	590	780.00	684.00	732.00	7.32
		6.00	10	600	788.00	687.00	737.50	7.38
		6.20	20	620	838.00	735.00	786.50	7.87
		6.50	30	650	874.00	765.00	819.50	8.20

APPENDIX-C

DEPARTMENT OF CIVIL ENGINEERING
Khulna University of Engineering & Technology
Khulna-9203.

ASTM D 1134

FIELD RECORD OF PLATE LOAD TEST

TEST NO.:

Name of the Project : Thesis

Location Site : Guest House Cum Club , KUET.

Design Pile Load : 1250 kg Ton's Total Test Load 5000 kg Ton's Ram Dim: 1ft

Specification of Pile :

Type of Sub- Soil :

Type of Pile : Sand Pile

Date of Pile Driving/ Construciton : 26.04.12

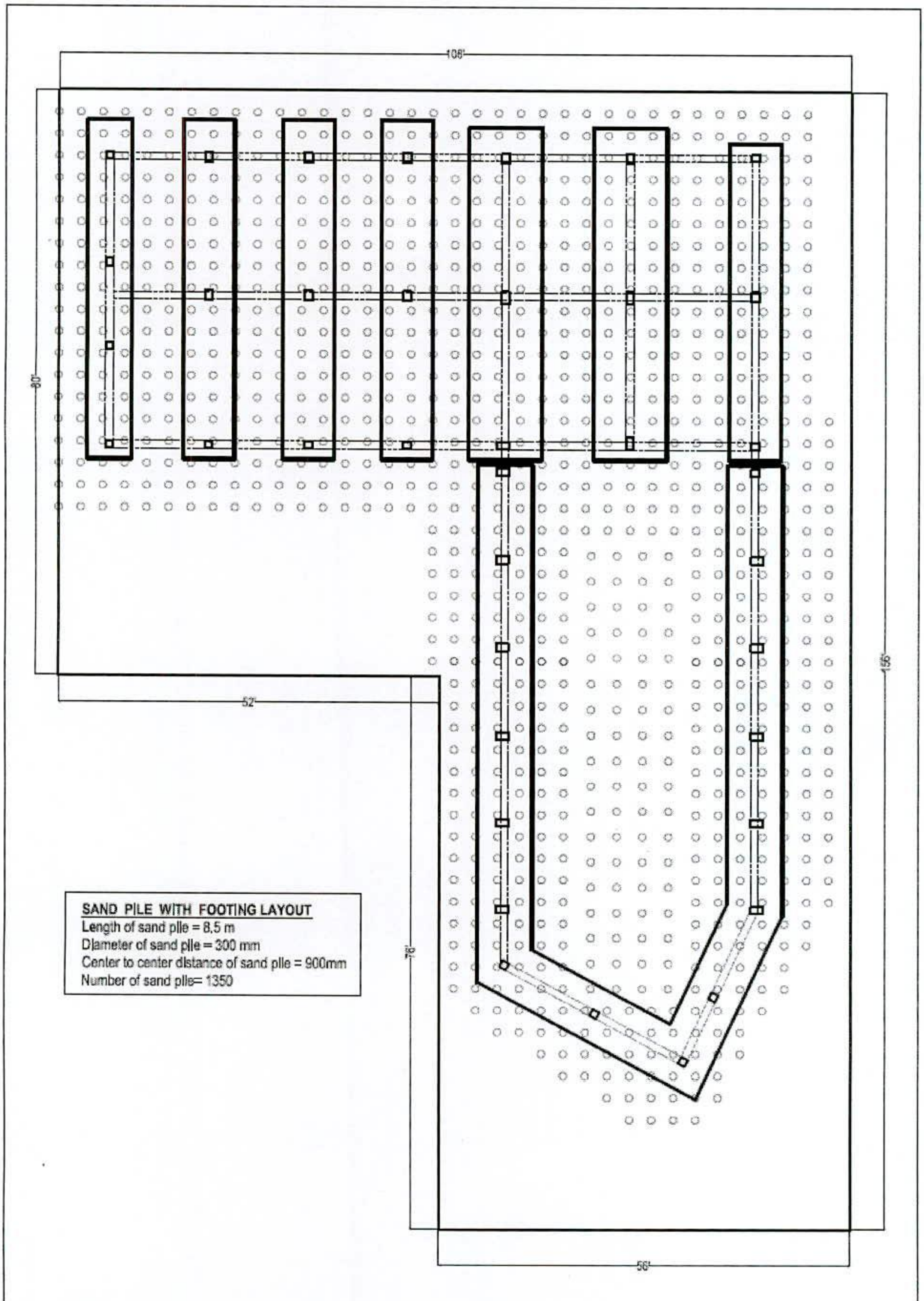
Pile Diamenter : 1 ft Length: 27 ft

Date of Testing : 19.04.2013

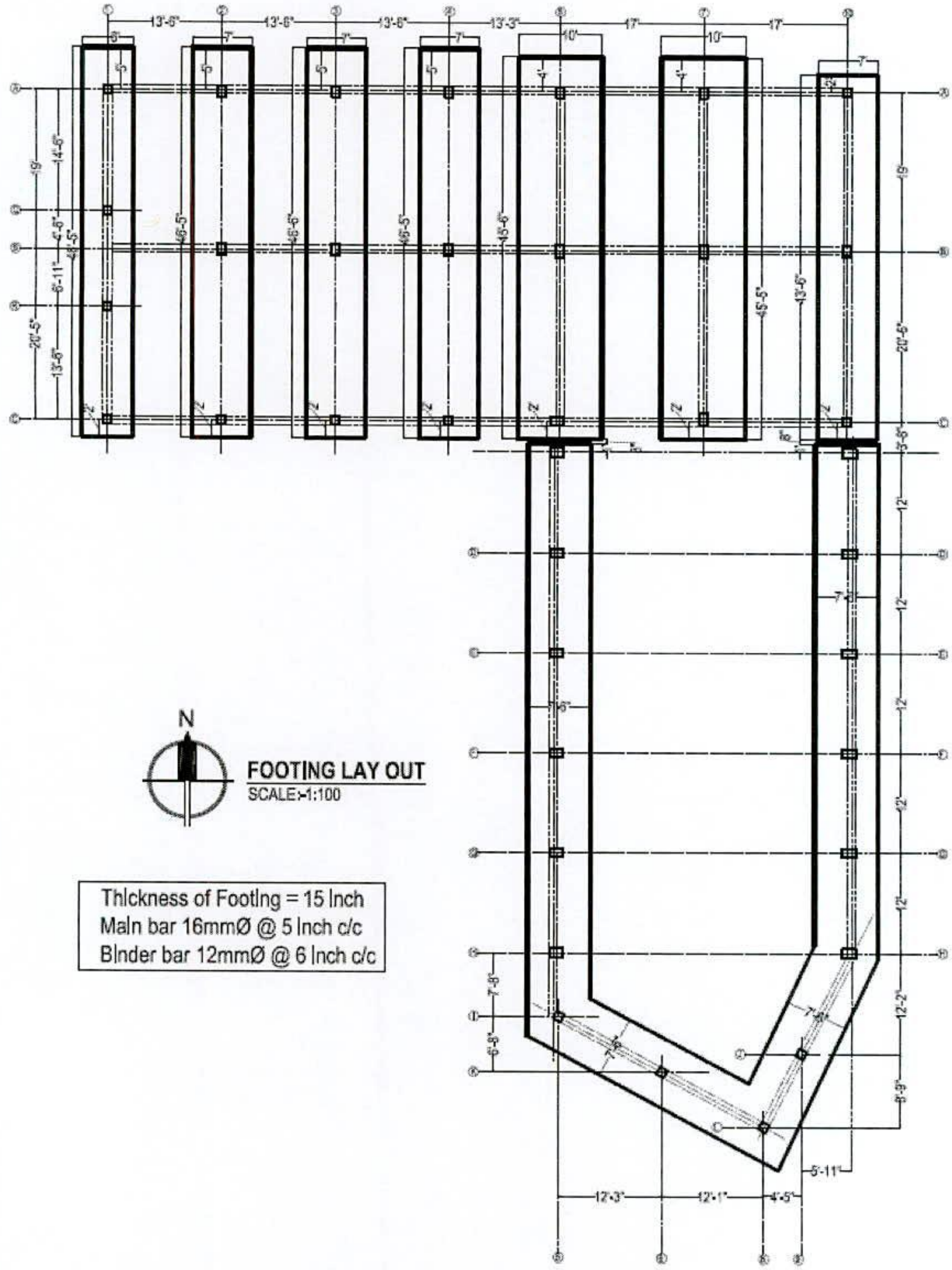
Date	Load (Ton's)	Clock Time	Interval for Recording (Min's)	Elapsed Time (hrs.)	Dial Reading (Left)	Dial Reading (Right)	Average Reading	Displacement (mm)
	500 kg	8.00	0	0	53.00	54.00	53.50	0.54
		8.05	5	5	59.00	59.00	59.00	0.59
		8.15	10	15	60.00	60.00	60.00	0.60
		8.35	20	35	61.00	61.00	61.00	0.61
		9.05	30	65	61.00	61.00	61.00	0.61
	1000 kg	9.05	0	65	136.00	115.00	125.50	1.26
		9.10	5	70	140.00	118.00	129.00	1.29
		9.20	10	80	142.00	119.00	130.50	1.31
		9.40	20	100	142.00	119.00	130.50	1.31
		10.10	30	130	144.00	121.00	132.50	1.33
	1500 kg	10.10	0	130	208.00	166.00	187.00	1.87
		10.15	5	135	216.00	170.00	193.00	1.93
		10.25	10	145	220.00	173.00	196.50	1.97
		10.45	20	165	223.00	174.00	198.50	1.99
		11.15	30	195	226.00	176.00	201.00	2.01

Date	Load (Ton's)	Clock Time	Interval for Recording (Min's)	Elapsed Time (hrs.)	Dial Reading (Left)	Dial Reading (Right)	Average Reading	Displacement (mm)
	2000 kg	11.15	0	195	268.00	216.00	242.00	2.42
		11.20	5	200	289.00	229.00	259.00	2.59
		11.30	10	210	300.00	234.00	267.00	2.67
		11.50	20	230	302.00	235.00	268.50	2.69
		12.20	30	260	304.00	237.00	270.50	2.71
	2500 kg	12.20	0	260	341.00	273.00	307.00	3.07
		12.25	5	265	365.00	292.00	328.50	3.29
		12.35	10	275	377.00	296.00	336.50	3.37
		12.55	20	295	383.00	302.00	342.50	3.43
		1.25	30	325	387.00	306.00	346.50	3.47
	3000 kg	1.25	0	325	425.00	348.00	386.50	3.87
		1.30	5	330	448.00	362.00	405.00	4.05
		1.40	10	340	459.00	374.00	416.50	4.17
		2.00	20	360	464.00	380.00	422.00	4.22
		2.30	30	390	471.00	396.00	433.50	4.34
	3500 kg	2.30	0	390	511.00	431.00	471.00	4.71
		2.35	5	395	527.00	446.00	486.50	4.87
		2.45	10	405	532.00	449.00	490.50	4.91
		3.05	20	425	534.00	451.00	492.50	4.93
		3.35	30	455	538.00	452.00	495.00	4.95
	4000 kg	3.35	0	455	589.00	509.00	549.00	5.49
		3.40	5	460	605.00	524.00	564.50	5.65
		3.50	10	470	616.00	534.00	575.00	5.75
		4.10	20	490	620.00	539.00	579.50	5.80
		4.40	30	520	627.00	546.00	586.50	5.87
	4500 kg	4.40	0	520	646.00	575.00	610.50	6.11
		4.45	5	525	655.00	595.00	625.00	6.25
		4.55	10	535	685.00	605.00	645.00	6.45
		5.15	20	555	696.00	617.00	656.50	6.57
		5.45	30	585	696.00	617.00	656.50	6.57
	5000 kg	5.45	0	585	708.00	645.00	676.50	6.77
		5.50	5	590	715.00	653.00	684.00	6.84
		6.00	10	600	720.00	661.00	690.50	6.91
		6.20	20	620	727.00	668.00	697.50	6.98
		6.50	30	650	730.00	670.00	700.00	7.00

APPENDIX-D

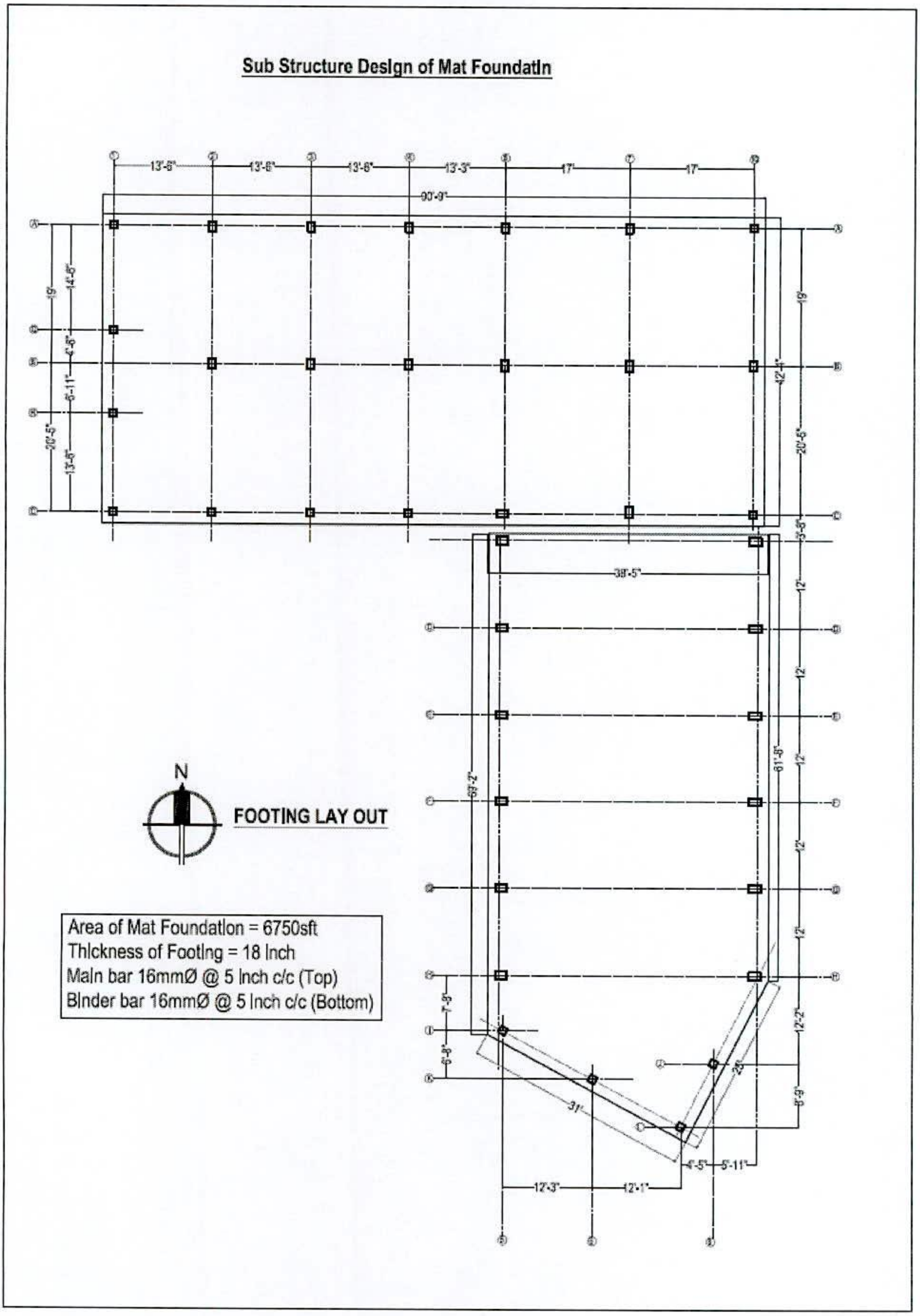

108'
80'
52'
76'
156'
56'
SAND PILE WITH FOOTING LAYOUT
Length of sand pile = 8.5 m
Diameter of sand pile = 300 mm
Center to center distance of sand pile = 900mm
Number of sand pile= 1350

Sub Structure Design of Continous Footing



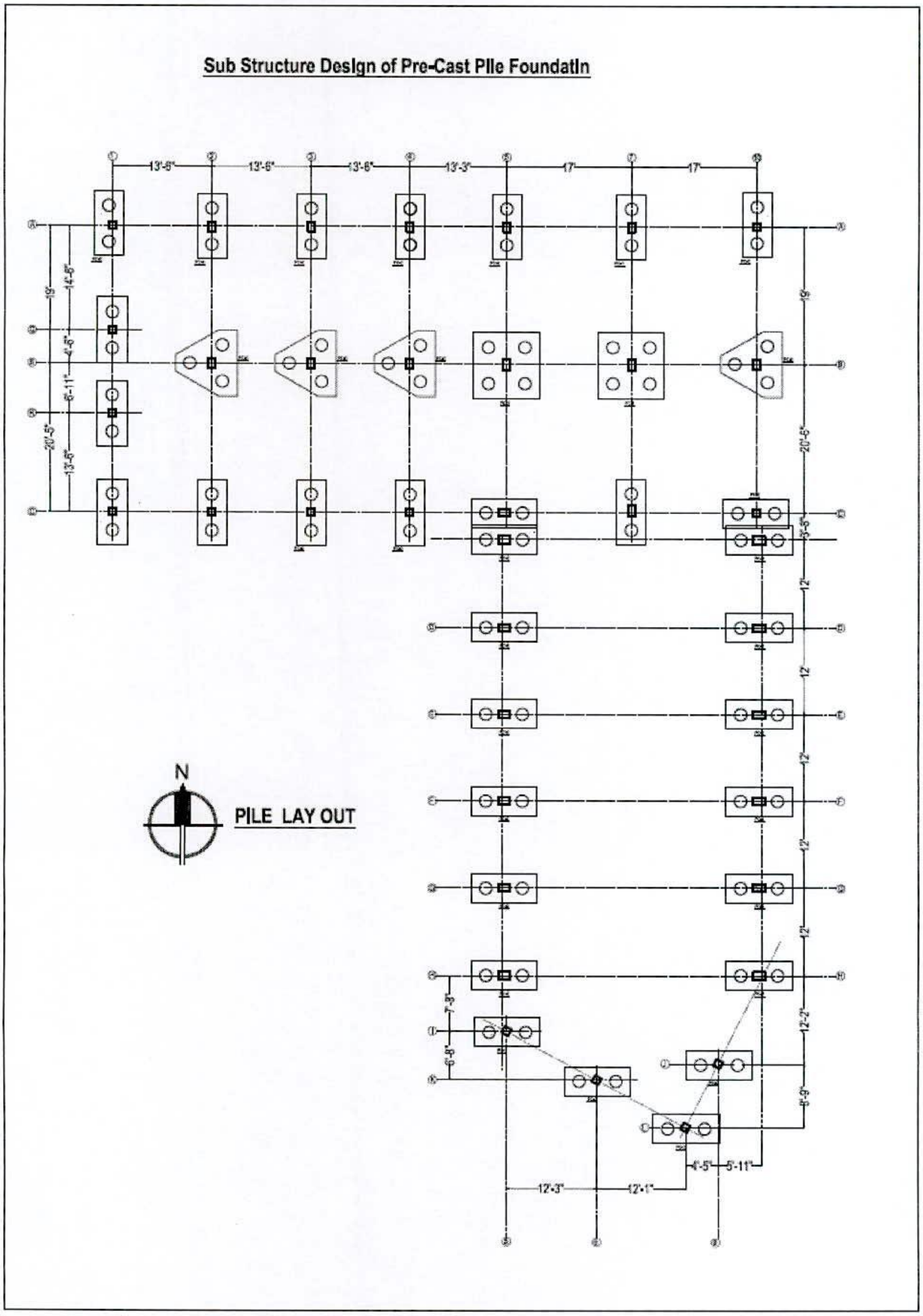
Thickness of Footing = 15 Inch
Main bar 16mmØ @ 5 Inch c/c
Binder bar 12mmØ @ 6 Inch c/c

APPENDIX-E



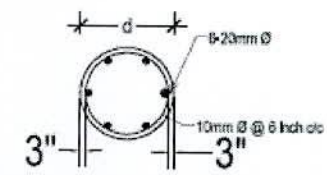
Sub Structure Design of Mat Foundatin
FOOTING LAY OUT
N
90'-9"
38'-5"
31'
25'
Area of Mat Foundation = 6750sft
Thickness of Footing = 18 Inch
Main bar 16mmØ @ 5 Inch c/c (Top)
Binder bar 16mmØ @ 5 Inch c/c (Bottom)

APPENDIX-F

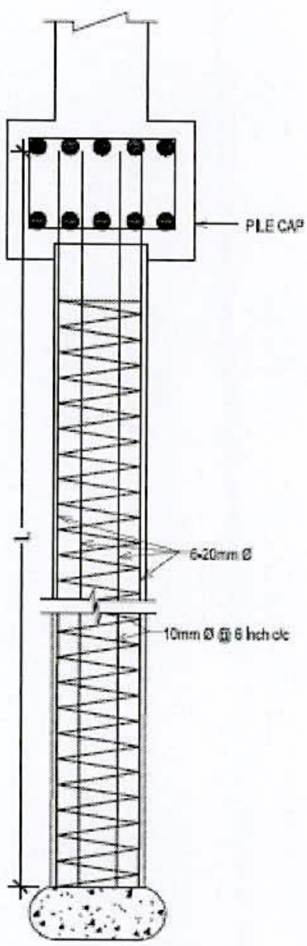

Sub Structure Design of Pre-Cast Pile Foundatin
13'-6"
13'-6"
13'-6"
13'-3"
17'
17'
19'
14'-6"
4'-6"
6'-11"
20'-5"
13'-6"
19'
20'-6"
12'
12'
12'
12'
12'
12'-2"
7'-8"
6'-8"
6'-9"
4'-5"
5'-11"
12'-3"
12'-1"
N
PILE LAY OUT

SUMMARY OF PILE FOUNDATION

TYPE	PILE					PILE CAP			
	NO. OF PILE	LENGTH	DIAMETER	REINFORCEMENT		SIZE	THICKNESS	REINFORCEMENT	
				MAIN	TIE			BOTTOM LAYER	TOP LAYER
PC2	2	100 ft	20 Inch	6-20mmØ bar	10mmØ 6 Inch c/c	9 ft X 4 ft	32 Inch	20mmØ 6 Inch c/c long direction 12mmØ 6 Inch c/c short direction	12mmØ 5 Inch c/c both direction
PC3	3	100 ft	20 Inch	6-20mmØ bar	10mmØ 6 Inch c/c	9 ft X 8.5 ft	36 Inch	20mmØ 5 Inch c/c both direction	12mmØ 5 Inch c/c both direction
PC4	4	100ft	20 Inch	6-20mmØ bar	10mmØ 6 Inch c/c	9 ft X 9 ft	32 Inch	20mmØ 5 Inch c/c both direction	12mmØ 5 Inch c/c both direction



CROSS SECTION OF PILE



LONG SECTION OF PILE