Study on the Settlement Response of Soft Ground Improved by Granular Columns

by

Md. Abdullah Al Mahamud

A thesis submitted in partial fulfillment of the requirements for the Degree of

Master of Science in Civil Engineering

Khulna University of Engineering & Technology
Khulna-9203, Bangladesh

March 2008

Declaration

This is to certify that this thesis work entitled "*Study on the Settlement Response of Soft Ground Improved by Granular Columns*" has been carried out by Md. Abdullah Al Mahamud in the Department of Civil Engineering, Khulna University of Engineering & Technology, Khulna-9203, Bangladesh. The above research work or any part of this work has not been submitted anywhere for the award of any degree or diploma.

Professor Dr. Muhammed Alamgir	Md. Abdullah Al Mahamud
Supervisor	Roll No. 0501502

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This is to certify that this thesis work submitted by Md. Abdullah Al Mahamud entitled "*Study on the Settlement Response of Soft Ground Improved by Granular Columns*" has been approved by the Board of Examiners for the partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering, Khulna University of Engineering & Technology (KUET), Khulna-9203, Bangladesh in March 2008.

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Acknowledgement

The author would like to pay his heartiest gratitude to his supervisor Professor Dr. Muhammed Alamgir, Dean, Faculty of Civil Engineering, Khulna University of Engineering &Technology (KUET) Khulna-9203, for his fruitful guidance, constant supervision, energetic encouragement, constructive and helpful criticism and suggestions at every stage of this dissertation. It was a valuable experience to work under the supervision of Professor Alamgir.

The author also express profound gratitude to Professor Dr. Md. Abul Bashar, Head, Department of Civil Engineering, KUET, for all his valuable suggestions, supports and cooperation and for providing laboratory facilities during the period of this study.

The author wishes to convey honest gratefulness to Professor Dr. Md. Keramat Ali Molla, Professor Dr. Quazi Hamidul Bari, Professor Dr. Monjur Hossain and Assistant Professor K. A. B. M. Mohiuddin, Department of Civil Engineering, KUET for their unremitting encouragement and cooperation at every stage of this dissertation.

Finally, the author likes to offer special thanks and gratefulness to Technician Md. Monirul Islam, Geotechnical Engineering Laboratory, KUET and also Technician Md. Habibur Rahman, Engineering Material Laboratory, KUET for their cordial assistance during laboratory experimental works.

Md. Abdullah Al Mahamud

March, 2008

Affectionately dedicated

to

my mother who taught me

that Allah only knows where is your betterment

and

all of my colleagues who promote me

for higher studies

ABSTRACT

Organic soil is generally encountered at a depth of 10 to 25 ft from the existing ground surface in the Khulna region, the South-west part of Bangladesh. Due to inherent geotechnical properties of this soil, i.e. high moisture content, high compressibility and low shear strength; conventional foundation systems are not suitable for this ground. To overcome such problems, soil improvement techniques are to be adopted to provide safe and economical foundation systems. In many ground improvement techniques, geotextile is used to enhance the performance of the adopted foundation systems. Granular columns overlain by compacted sand bed with geotextile is one of such systems, the behaviour of which needs to be examined experimentally.

This dissertation describes the load-settlement behaviour of treated and untreated reconstituted organic grounds constructed in the laboratory. The reconstituted organic grounds or the test grounds were prepared in a cylindrical tank based on "unit cell" concept. There were mainly two layers in the test grounds. The bottom layer was compacted sand bed performed as a good filter media and the top layer was reconstituted organic soil represented the problematic ground of Khulna region. The reconstituted organic grounds were prepared under pre-set pre-consolidation pressure and different ground improvement techniques were applied on it to find out effective solution for organic soil. The test grounds were treated by four conditions - compacted sand column, compacted sand bed with and without geotextile and compacted sand bed with geotextile in conjunction of compacted sand column. The load-settlement responses of the treated and untreated test grounds were determined by footing load test.

Laboratory investigation reveals that the compacted sand bed with and without geotextile improve the bearing capacity of organic grounds 1.5 times than untreated grounds. The results also showed that degree of improvement increases significantly due to the installation of compacted sand column with compacted sand bed with geotextile over it. This system improved the bearing capacity of footing more than two times than that of placed on untreated ground. The use of geotextile not only improves the bearing capacity but also ensures flatter settelement pattern against the increase of load intensity. The study reveals that the available empirical equations are suitable for the prediction of bearing carrying capacity of footing resting on the sand column and compacted sand bed treated grounds but highly over predicts the same in case of grounds treated by compacted sand bed with geotextile and compacted sand bed with geotextile in conjunction of compacted sand column.

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CHAPTER ONE

INTRODUCTION

1.1 General

Soft grounds create problems for the construction of massive structures such as buildings, roads and highways, railways, airfields, embankments, dams, storage tanks, car parks, and temporary working platforms due to its low shear strength and high compressibility. The valuable structures are sometimes collapsed due to excessive total and differential settlements while constructed on the soft ground without adopting proper foundation system.

In Khulna region i.e. the south-west part of Bangladesh, the soft deposits extend up to a considerable depth, as a result the recent alluvial deposits with significant amount of organic composition offer a big challenge to the Geotechnical Engineers in designing economic foundation systems for the construction of necessary infrastructures (Alamgir et al., 2001). Due to the presence of organic soil layers, the Civil Engineering constructions in such sites require special attention to overcome the possible adverse consequences. In the recent years, ground improvement techniques have been considered as one of the most versatile solutions to tackle such adverse consequences.

In this study, the effectiveness of compacted sand bed with and without geosynthetic and granular column in improving the bearing capacity of soft organic soils were studied through laboratory investigation. The test grounds, made up of reconstituted organic soil, of four different improvement conditions were investigated by footing load test applied through a circular footing resting on the ground surface. The results were compared with that of the untreated ground. The outcome of the study can be used as a guideline for the selection of appropriate soil improvement method for the construction of safe and economical foundation systems for massive infrastructures in such soft grounds existing in the soft soil regions of Bangladesh, e.g. Khulna region.

1.2 Background of this Study

Many researches have been conducted to develop a reliable and cost effective solution for the construction of structures on soft ground conditions. Considering the inherent limitations of conventional foundation systems, ground improvement methods have been practiced for a long time. Amongst them columnar inclusions for earth reinforcement was first introduced in the traditional foundation by French military army in 1830's to support heavy foundations of iron works at the artillery arsenal in Bayoune (Hubhes & Withers, 1974). Many successful applications proved that the granular pile is a valuable addition to the special foundation systems due to its versatile application. Meanwhile, the geosynthetic reinforcement was first proposed by Casagrande who idealized the problem in the form of a weak soil reinforced with high-strength membranes laid horizontally in layers (Westgaard, 1938) and some researches were conducted successfully to improve soft soil conditions. Both of the mentioned treatment techniques for soft soil are world wide renowned and more investigation is required for required refinement and/or endorsement of the previous findings.

The geosynthetic reinforced granular fill soft soil systems are now being used very frequently as foundations for shallow footings, unpaved roads, low embankments, oil drilling platforms, heavy industrial equipment, car parks and closure covers for tailing dam etc. Such reinforced soil systems improve the bearing capacity and reduce the settlement by distributing the imposed loads over a wider area of weak subsoil. In conventional construction technique, a thick layer of granular fill is needed which may be costly or may not be possible, especially in the sites of limited availability of granular materials of good-quality. On the other hand the columnar inclusions are used for improving slope stability of embankments and natural slopes, increasing bearing capacity, reducing total and differential settlements, reducing the liquefaction potential of sands and increasing the time rate of settlement.

Among the various ground improvement methods, columnar inclusions has been considered as one of the most versatile and cost effective deep ground improvement technique (Alamgir, 1996 and Alamgir & Miura, 1999). In addition to the installation of granular columns in soft ground, the capacity of the column improved ground can also be enhanced through the placing of a compacted granular fill with or without reinforcement i.e. Geosynthetics (Alamgir et al., 1996 and Shukla, 1995). Settlement of ground improved by granular piles is governed by the compressibility of the soil and the deformability of the granular piles (Mitchell, 1981). The geosynthetic-reinforced granular fill system provides the interaction between the soil and the reinforcing members slowly by friction generated by gravity (Vidal, 1960). The total and differential settlement of the soft soil reduces significantly due to the use of geosynthetic reinforced granular fill (Shukla, S. K., 1995).

Soft soil deposits exist up to a great depth in the Khulna region, the south-west part of Bangladesh. In this region, a thick organic soil layer generally exists at 10 to 25 feet depth measured from the existing ground surface (Islam, 2006). Due to the expensive and time consuming nature of the conventional foundation system, the practicing engineers usually choose granular columns for the improvement of soft ground. Some researches have already been conducted in this field to examine the effectiveness of such ground improvement techniques in Khulna region (Zaher, 2000, Sobhan, 2001 and Hossain, 2007). However, detailed experimental study in the laboratory has not yet been conducted to investigate the effect of different relevant parameters on the load-settlement response of the improved ground. As a follow-up of ongoing research on soft grounds in the Department of Civil Engineering, Khulna University of Engineering & Technology, Bangladesh, this research has been conducted. Hence, a reconstituted organic soil media has been prepared to represent the field condition. Over which compacted sand bed was constructed with or without Geotextile and the grounds also were improved by the installation of compacted sand column.

1.3 Objectives of the Study

The objectives of this research work can be outlined as:

(i) To examine the settlement behaviour of treated grounds for various conditions:

- Inclusion of compacted sand column
- Improvement by compacted sand bed with and without geotextile.
- Improvement by compacted sand bed with the inclusion of geotextile layer in conjunction of compacted sand column.

(ii) To establish comparisons on the degree of improvement among the treated grounds with respect to untreated ground based on the experimental results and predictions using the available empirical equations.

1.4 The Research Scheme

Organic soil both the undisturbed and disturbed conditions were collected to conduct the research to find out the effective ground improvement solution for soft ground conditions such as Khulna region. The undisturbed soil samples were collected for the characterization of organic soil, which was used for the preparation of reconstituted grounds. The organic soil was remolded using standard procedures and kept under pre-determined pre-consolidation pressure to prepare grounds of same, as close as possible, pre-consolidation pressure. Then the experimental grounds were prepared applying different ground improvement techniques and the load-settlement behavior of both the treated and untreated grounds were examined by footing load test. The results were analyzed to develop a useful comparison and to find out an effective ground improvement solution for Khulna region or the region having similar sub-soil condition. In order to attain the objectives, the whole experimental research scheme was carried out according to the following phases:

Phase 1: Determination of index and engineering properties of organic soils collected from Khulna region using conventional methods (ASTM, 2004).

Phase 2: Preparation of reconstituted soft organic grounds having the similar nature of organic soil available in Khulna region.

Phase 3: The reconstituted organic grounds are improved by different ground improvement conditions to establish a comparison and find an effective solution.

Phase 4: Determination of strength and compressibility characteristics of reconstituted organic grounds using standard laboratory tests (ASTM, 2004).

Phase 5: Determination of load-settlement behavior and bearing capacity of the footing resting on the ground from footing load test.

Phase 6: Establishment of comparisons among the treated and untreated grounds based on test results and predictions using available empirical equations.

1.5 Organization of the Thesis

This dissertation is written through six chapters in the following sequence. The outline and relations between these six chapters are shown in Figure 1.1. The background and objectives as well as the research scheme are presented in a comprehensive style in Chapter One.

Chapter Two deals with the literature review of this dissertation. This chapter describes the properties of organic soil, complexity of organic soil, different soil improvement techniques, the history of soil improvement techniques with case studies related to the present research.

Chapter Three represents the laboratory investigation of this study. This chapter deals with the index and engineering properties of organic soil collected from Khulna region that was also used for preparation of reconstituted organic grounds. The laboratory investigations were made on the experimental grounds described in details in this chapter. A Flow chart of laboratory investigation is presented in Figure 3.1 in a flow chart.

Chapter Four represents the results of this study with discussions elaborately. This chapter deals with the load-settlement behavior of experimental reconstituted organic grounds.

Chapter Five describes the comparisons of the experimental findings with the predicted values using available empirical equations applicable for the considered ground improvement conditions.

Finally, the conclusions of the study, limitations of the study and the recommendations for future studies are described in Chapter Six.

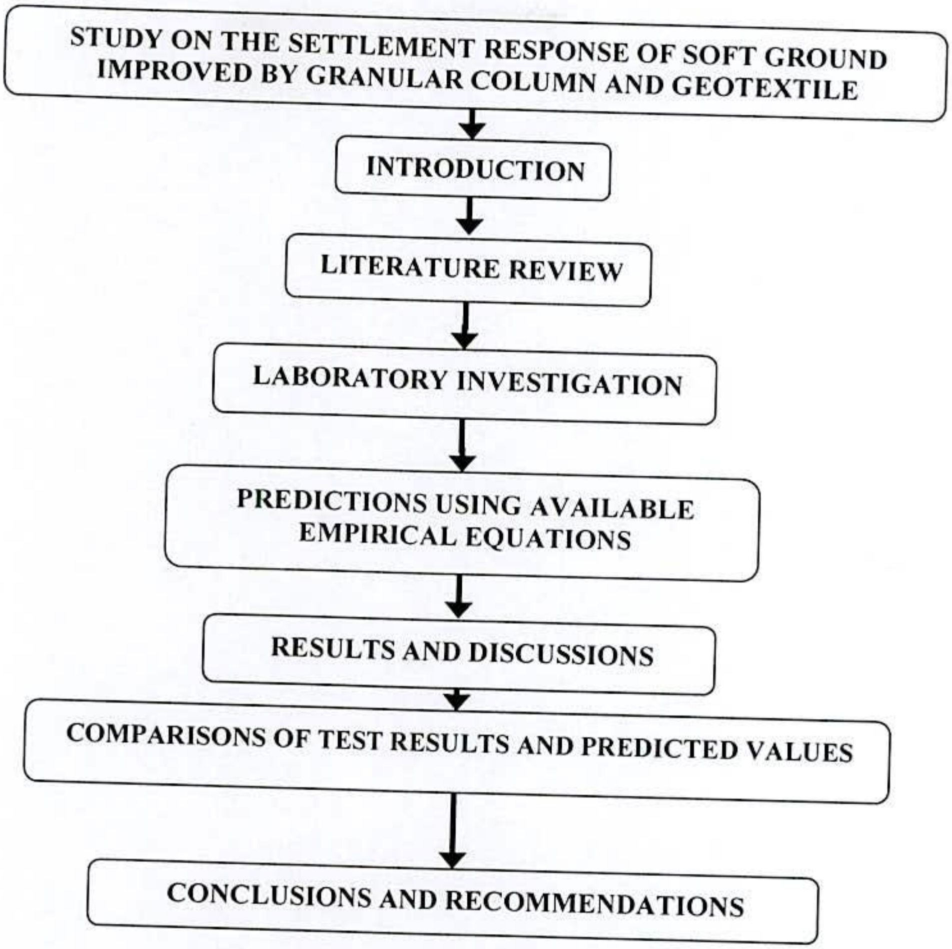


Fig. 1.1 Diagram of the thesis outline.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

The present study describes the effectiveness of different soil improvement techniques in improving reconstituted organic ground. This chapter describes the nature of soft ground and application of various ground improvement techniques in improving such ground conditions. The theoretical approach available in the literature to predict the behaviour of such improved ground is described here briefly. Case studies on the application of such ground improvement techniques considered in this study are also illustrated here.

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 clay soils, (ii) soils which have large fractions of particles as fine as silt, (iii) clayey soils which have high moisture content and (iv) 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 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 plains, 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. The determination of strength and compressibility parameters of soft ground 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 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.

2.2.1 General properties of soft ground

In general, the term soft ground includes such soft clay soils, soils with large fraction of fineness such as silts, clayey soils which have high moisture content, peat foundations, and loose sand deposits just above or under water table (Kamon & Bergado, 1991). Table 2.1 represents an outline for the identification of soft ground according to the types of structures. It may be noted that the criteria are different and depend on the structures constructed. The general ranges of N-values (STP), unconfined compressive strength (q_u), cone penetration resistance (q_c), and the water content of these soft ground are also stated in the Table 2.1.From relationship between relative density, penetration resistance, and angle of friction of cohesion less soils, which was represented as a soft ground and very loose soil condition its Relative Density value is less than 0.2, Standard Penetration resistance N (blows/ft) less than 4, Static cone resistance qc (ton/ft^2) less than 20 and angle of friction ϕ (deg) less than 30. In loose condition, its Relative Density value is 0.2 to 0.4, Standard Penetration resistance N (blows/ft) is 4 to 10, Static cone resistance (tsf) qc is 20 to 40 and angle of friction ϕ (deg) is 30 to 35. Unconfined compression strength for fine particle clay, the value of consistency (q_u) of very soft clay is 0 to 0.25 tsf or 0 to 24 kPa and the value of consistency (q_u) of soft clay is 0.25 to 0.5 tsf or 24 to 48 kPa.

Table 2.1 Outline for identification soft ground (after Kamon & Bergado, 1991)

Structures	Soil conditions	N-values (SPT)	q_u (kPa)	q_c (kPa)	Water content (%)
Road	A: Very soft B: Soft C: Moderate	Less than 2 2 to 4 4 to 8	Less than 25 25 to 50 50 to 100	Less than125 125 to 250 250 to 500	
Express Highway	A: Peat soil B: Clayey soil C: Sandy soil	Less than 4 Less than 4 Less than 10	Less than 50 Less than 50		More than 100 More than 50 More than 30
Railway	(Thickness of layers) More than 2m More than 5m More than10m	0 Less than 2 Less than 4			
Bullet train	A B	Less than 2 2 to 5		Less than 200 200 to 500	
River dike	A: Clayey soil B: Sandy soil	Less than 3 Less than 10	Less than 60		More than 40
Fill dam		Less than 20			

2.2.2 Geotechnical aspects of soft soils in Bangladesh

The major part of Bangladesh is on the DELTA formed by the three mighty rivers, namely, BRAHMAPUTRA, GANGES and MEGHNA. These rivers and many of the country's other minor rivers originate outside the national boundary of the country and make up the Ganges-Brahmaputra-Meghna river system.

In many areas, the soil surveys recognized active, young, and old floodplain landscapes. Active floodplains occupy land within and adjacent to the main rivers where shifting channels deposit and erode new sediments during the annual floods. newly deposited alluvium within this flood plain is stratified in different layers. Usually, silty and clay deposits are finely stratified, and sandy deposits, as well as mixed sandy and silty deposits are coarsely stratified. This is a state from where the soil forming factors are yet to activate the soil forming processes.

The young and old flood plains are virtually stable land that the main river channel has moved away, but they are criss-crossed by tributary or distributary's channels that vary from active to moribund delta. On these flood plains, the process of soil formation dominates over sediment deposition, as evidenced by soil characteristics i.e. the original alluvial stratification has been broken up by biological mixing; the subsoil has developed structure and oxidized mottles; and, in older soils, the topsoil has become acid.

Hill Soil forming processes are active on the hills for a significant period. Due to erosion on steep slopes of high hills, the weathered material on the hills is constantly removed and thus keeps the soils young on the high hills. The soils on the low hills are older as erosion is less severe and allows soil material to accumulate. The sedimentary rocks underlying steep to very steep high hill ranges are usually hard and relatively rich in weatherable minerals and lower hills are unconsolidated and poor in weatherable minerals. Soils have been developed from these minerals through prolonged weathering under well to excessively drained conditions, leaching, acidification and removal of surface material by erosion. There is an increase of clay content from surface to subsoil indicating clay illuviation in most of the soils formed in the low hills.

Safiullah (1991) reported that geotechnical aspects of soils of Bangladesh are complex and are usually heterogeneous, both in vertical and horizontal directions. Soils consist of wide varieties of materials ranging from gravel, poorly graded sand to silt and clay. In general, there is a predominance of silt sized particles. Majority of the soils of Bangladesh falls in two types of deposits, namely terrace deposits and as recent deposits. Finer materials at the surface underlain by coarser materials characterize recent deposits, which consist more than eighty percent of land surface of Bangladesh. The nature of organic soil profile in Bangladesh is highly stratified, complex and discontinuous in each direction (Safiullah, 1991).

2.2.3 The nature of organic soil

The different definitions are applied in the fields of geology, pedology, and agriculture, and so forth, but in geotechnology the significant features of organic soils are particular nature, and the porous structure, which the particles form (Head, 1980). The relatively thin layer of topsoil, which supports vegetation, is usually unimportant unless it extends to exceptionally great depths. Most natural soils comprise particles known as boulders, cobbles, gravels, sands, silts and clays. Extensive deposits of organic matter, notably peat, though usually of fibrous rather than particular nature, are included within the definition of soils. Natural materials which have been disturbed by man, such as clay or gravel placed in a embankment, or colliery and quarry waste, are also included. To these may be added man made materials such as furnace slag; pulverized fuel ash (PFA) from power station; builders, rubble; domestic refuse.

2.2.4 The complexity of organic soil nature

To select representative soil parameters for natural soil deposits, unexpected changes take place in soils when certain environmental changes occur (Head, 1980). For instance, vibrations can alter the state of a sand deposit from loose to dense. Some clay soils, which are extremely hard when dry, can turn into slush having very little shearing strength, when their water content becomes high. Indeed, water is by far the most important variable controlling the bahaviour of fine-grained soils. Natural soil deposits are complex to deal with, because of

(i) The stress-strain relationship for a soil deposit is non-linear; hence the difficulty in using easily determinable parameters to describe its behavior.
(ii) Soil deposit has a memory for stress undergone in their geological history. Their behavior is vastly influenced by their stress history; time and environment are other factors which may alter their behaviour.

(iii) Soil deposit being far from homogeneous, exhibit properties which vary from location to location.

(iv) As soil layers are buried and hidden from view, one has to rely on tests carried out on small samples obtained from selected depths and locations. Since there is a constraint on the number of samples that can be taken, there is no guarantee that the soil parameters are truly representative of the field strata.

(v) No sample is truly undisturbed. In a soil, which is sensitive to disturbance, the behavior surmised from the laboratory, tests might not reflect the likely bahaviour of the field stratum.

From the above discussion it is clear that it is not sufficient enough to a soil engineer in search of practical solutions, to possess the knowledge of the principles of soil mechanics. Knowledge of the various processes that have gone into the composition of the natural soil mass is important since these have a direct relation to the soil behaviour.

2.3 General Properties of Organic Soils

Organic deposits are due to the decomposition of organic matters and found usually in topsoil and marshy place. A soil deposit of organic origin is said to peat if it is at the higher end of the organic content scale (75% or more according to some authors), organic soil at the low end, and muck in between. Peat soil deposit is usually formed of fossilized plant materials and characterized by fibre content and lower decomposition or humification. However, there are many criteria existed to classify the organic deposits and it remains still as a controversial issue with numerous approaches available for varying purposes of classification. Soil from organic deposit refers to a distinct mode of behaviour different than traditional soil mechanics in certain respects. A possible approach is being considered by the America Society for Testing and Materials (ASTM) for classifying soils having organic contents (OC) which may stated as follows (Edil, 1997).

(i) OC< 5%; little effect on behaviour, considered inorganic soil.

(ii) OC in between 6-20%; effects properties but behaviour is still like mineral soils, organic silts and clays.

(iii) OC in between 21- 74%; organic matter governs properties; traditional soil mechanics may be applicable; silty or clayey organic soils.

(iv) OC> 75%; displays behaviour distinct from traditional soil mechanics especially at low stresses; peats.

Peats have certain characteristics that set them apart from most mineral soils and require special considerations for construction over them. These special characteristics include:

(i) High natural moisture content (up to 1500%).

(ii) High compressibility including significant secondary and even tertiary compression.

(iii) Low strength in natural conditions.

Any soil containing a significant amount of organic matter to influence its engineering properties is called an organic soil. The amount of organic matter is expressed in terms of organic content, which is the ratio between the weight of organic matter and the oven dried weight of sample. The weight of organic matter can be determined by heating the sample to ignite the organic substances. The presence of organic matter in soils is often ignored, although it influences some important properties. Organic matter, although relatively small in volumetric proportion, significantly affects the water absorbing capacity of the soils. In terms of mechanical properties of soils, the organic matter is known to reduce the maximum dry unit weight and cohesion of soils.

Organic matter in soil may be responsible for high plasticity, high shrinkage, high compressibility, low hydraulic conductivity, and low strength. Soil with organic matter is complex both chemically and physically, and a variety of reactions and interactions between the soil and the organic matter is possible (Oades, 1989). It may occur in any of five groups: carbohydrates; proteins; fats; resins, and waxes; hydrocarbons; and constituent of soil. In residual soils organic matter is most abundant in the surface horizons. The humic fraction is gel-like in properties and negatively charged (Marshall, 1964). Organic particles may be strongly absorbed on mineral surfaces, and this adsorption modifies both the properties of the minerals and organic materials itself.
Based on organic soils, the soils are categorized in the following groups and it is listed in Table 2.2.

Table 2.2 Classification of organic soils (after Islam, 2006)

Category	Name	Organic content (%)	Group symbols	Distinguishing characteristics for visual identification	Range of Laboratory test values
Organic matter	Fibrous peat (woody, mats etc.)	75 to 100% organic either visible or inferred	Pt	Light weight, spongy and often elastic at shrinkage considerably on air-drying. Much water squeezes from sample.	w: 500 to 1200% γ= 60 to 70 pcf G= 1.2 to 1.8 $Cc/(1+e_o)$=0.4+
	Fine grained peat (amorphous)			Light weight, spongy but not often elastic at shrinkage considerably on air-drying. Much water squeezes from sample	w: 400 to 800% LL: 400 to 900% PI: 200 to 500 γ= 60to 70 pcf G= 1.2 to 1.8 $Cc/(1+e_o)$=0.35 to 0.4+
Highly organic soils	Silty peat	30 to 75% organic either visible or inferred	Pt	Relatively light weight, spongy. Thread usually weak spongy near PL shrinks on air drying; medium dry strength. Usually can squeeze water from sample readily slow dilatency.	w: 250 to 500% LL: 250 to 600% PI: 150 to 350 γ= 65 to 90 pcf G= 1.8 to 2.3 $Cc/(1+e_o)$=0.30 to 0.4
	Sandy peat			Sand fraction visible. Thread weak and friable near PL shrinks on air-drying; low dry strength. Usually can squeeze water from sample readily-high dilatency-"gritty"	w: 100 to 400% LL: 150 to 300% PI: 50 to 150 γ= 70 to 100 pcf G= 1.8 to 2.4 $Cc/(1+e_o)$=0.20 to 0.30
Organic soil	Clayey organic silt	5 to 30% organic either visible or inferred	OH	Often has strong H_2s odor. Thread may be tough depending on clay fraction. Medium dry strength, slow dilatency.	w: 65 to200% LL: 65 to 150% PI: 50 to 150 γ= 70 to 100 pcf G= 2.3 to 2.6 $Cc/(1+e_o)$=0.20-0.35
	Organic silt or sand		OL	Threads weak and friable near PL or may not roll at all. Low dry strength; medium to high dilatency.	w: 30 to125% LL: 30 to 100% PI: non-plastic to 40 γ= 90 to 110 pcf G= 2.4 to 2.6 $Cc/(1+e_o)$=0.10-0.25
Slightly organic soils	Soil fraction add slightly organic	Less than 5% organic combined visible and inferred	Depend upon inorganic fraction	Depend upon the characteristics of the inorganic fraction.	Depend upon the inorganic fraction

Notation: Pt-Peat, mulch, and other highly organic soils: OL- Organic silts, organic silty clays (low plasticity, OH- Organic clays (medium to high plasticity), organic silts. (source: Design Manual 7.1 by scientific publisher ,India)

2.4 Preparation of Reconstituted Organic Ground

Reconstituted organic soils are those that are prepared by breaking down natural soils, mixing them as slurry and reconsolidating. The reconstituted soils are distinguished from both remoulded and resedimented soils, which are mixed as a suspension and allowed to settle from that state.

2.4.1 Preparation of soil slurry

Soil was air-dried firstly and then powdered. The powered soil was then sieved by No 16 sieve. Generally No. 40 sieve is used for making reconstituted inorganic soils by various researchers, such as Bashar (2002), however, in this study, realizing the inherent physical conditions of organic soils, a standard sieve of larger opening size i.e. No. 16 sieve was used to obtain a better representative reconstituted organic soils. Then the sieved soil was mixed with water that was found as sufficient to yield uniform and homogeneous slurry. The soil and water was then thoroughly mixed by mixture machine to form slurry to ensure full saturation.

According to Burland, 1990, a reconstituted soil has been defined as one that has been thoroughly mixed at moisture content equal to or greater than liquid limit (w_L). The termed "intrinsic" has been used to describe the properties of clays which have been reconstituted at a water content of between w_L and 1.5 w_L (preferably 1.25 w_L) and then consolidated under one-dimensional condition. Soil slurry with initial water content well beyond the liquid limit has been commonly used as an initial state for sample preparation. However, higher initial water contents provide higher degree of saturation and higher freedom of particle orientation but require larger initial volumes and longer consolidation periods. Since large volumes of soil was required for preparing enough soil and also in order to reduce the consolidation time, it is essential to use initial water content which is sufficient to yield a uniform and homogeneous slurry.

2.4.2 Consolidation of Slurry

The slurry was consolidated to form a uniform reconstituted soil in a cylindrical consolidation Tank of 0.55 m diameter and 1.0 m in height. A 175 mm thick compacted sand bed with geotextile to permit uniform drainage was placed at the bottom of the mold. The inner surface of the cylindrical consolidation mold was coated with thin layer silicon grease to minimize side friction. The slurry was then poured into the mold and stirred with steel rod to remove the entrapped air from the slurry. After removing air bubble, the top surface of the soil was levelled properly. At the top of the slurry, a 150 mm thick perforated RCC slab with a geotextile was placed so that water could go out towards upper direction also.

The required axial load of 100 kPa was gradually applied to the sample using a loading frame with providing constant reading of compression dial gauge. Initially the slurry was allowed to consolidate by the self-weight and the weight of the porous RCC slab for about 24 hours. Then a small pressure of 5 kPa was applied to the sample for next 24 hours. Similarly, the pressure was increased by about 10 kPa and ultimately to the final value of 100 kPa. This pressure of $100 kN/m^2$ was maintained until the end of primary consolidation, which was indicated by the constant reading of compression dial gauge. The consolidation process required about seven or eight days for the completion of primary consolidation. The rate of compression was very fast at its initial stage of consolidation and then it gradually decreased with time.

2.4.3 Selection of Overburden Pressure

An external pressure i.e. pre-consolidation pressure is required in which the slurry has been subjected to obtain reconstituted soil samples, justification the completion of the primary consolidation at this applied pressure. Earlier, it was considered that a pre-consolidation pressure of 276 kPa was about the minimum value which could make the clay soil just stiff enough to allow setting up specimens (Kirkpatrick and Khan, 1984). Latter as the skill in testing has improved and it is found that the sample is possible to remove at 150 kPa (Kirkpatrick and Khan, 1984). Singh (1992) suggested that soil containing high organic matter shows large volume changes on loading and expulsion of water, low shear strength and low dry density. In addition, the reconstituted organic soil is fully decomposed with normally loaded state and shows highly compressible phenomena. The intrinsic compressibility characteristics are well defined for the reconstituted soil prepared at pre-consolidation pressure equal to or greater than 100 kPa. So, in this study the reconstituted organic grounds were prepared in the laboratory in K_o-

consolidation cell by consolidation pressure is 70 kPa instead of 100 kPa considering available laboratory load applying facilities.

2.4.4 Advantages of reconstituted ground

Reconstituted organic soil enables a general pattern of behavior to be established and comparisons with the response of intact/organic soil may be used to identify any special features associated with fabric, stress history or bonding. It was found that the most comprehensive studies invariably employed on reconstituted soil. Jardine (1985) discussed the difficulties of implementing detailed investigations of strength and compressibility properties of organic soils. The major advantages of using data from reconstituted soils are that the ambiguous and substantial effects of sampling of natural soils and inhomogeneity can be eliminated, while the essential history and composition of in-situ soils can be represented.

2.5 Ground Improvement Techniques

To improve the physical and mechanical properties of the soft ground, several ground improvement techniques have been and are being used since the 19th century. The different soil improvement methods can be classified into geometrical, physical and chemical, and structural methods as follows depending on how the methods affect the stability or reduce the settlement:

I. Geometrical methods: where the moment or force causing failure or 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) preloading (often combined with vertical drains to increase the consolidation rate), (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 alternating the clay-water system e.g. by

2.5.3 Consolidation of soft soil

Settlement resulting from the long-term consolidation of cohesive soils creates 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 reduces 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 into wide use.

2.5.4 Injection and grouting

Injection of material into the ground has developed into 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 of injections are possible viz, permeation, displacement and encapsulation. Permeation grouts are two types, particular grouts and chemical grouts. Chemical grouts offer the advantages over particular grouts that they can penetrate smaller pores, the have a lower viscosity and there is a better control of the setting time.

2.5.5 Soil reinforcement

Basically this method involves the in-situ inclusion of a reinforcing element in the ground to improve its engineering characteristics or to carry the load to a competent material. The six types mostly used of in-situ reinforcement are stone columns, soil nailing, micro piles, jet grouting, permanent anchors and geotextiles.

Much of the use of geotextiles is involved with soil protection or reinforcement. The former involves control of erosion but may also entail isolating a soil mass from water. A particular installation may include excavating 0.5 to 1.5 m of soil that is susceptible to volume change, installing a plastic film, then carefully backfilling. Subsurface water migrating to the surface is blocked by the film so that the upper soil does not become saturated and undergo volume change. Obviously, careful site grading and protection

2.6 Method of Selection of Ground Improvement Technique

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 all problems in all soils. For soft and cohesive soils in subsiding environments, ground improvement by reinforcement, by admixtures and by dewatering are applicable.

During the Planning stage of any construction projects, 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. Several methods ranging from conventional labor intensive to proper-equipped techniques have been practiced throughout the world. The choice of installation techniques primarily depends on the sub-soil condition, required degree of improvement, availability of installation equipments and finally cost involvement. In Bangladesh, no well-equipments are readily available and hence practiced. 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 most appropriate one in any case can only be made after evaluation of several factors specific to the problem at hand (Zaher 2000). A flow chart for selection of ground improvement techniques are given in Figure 2.6

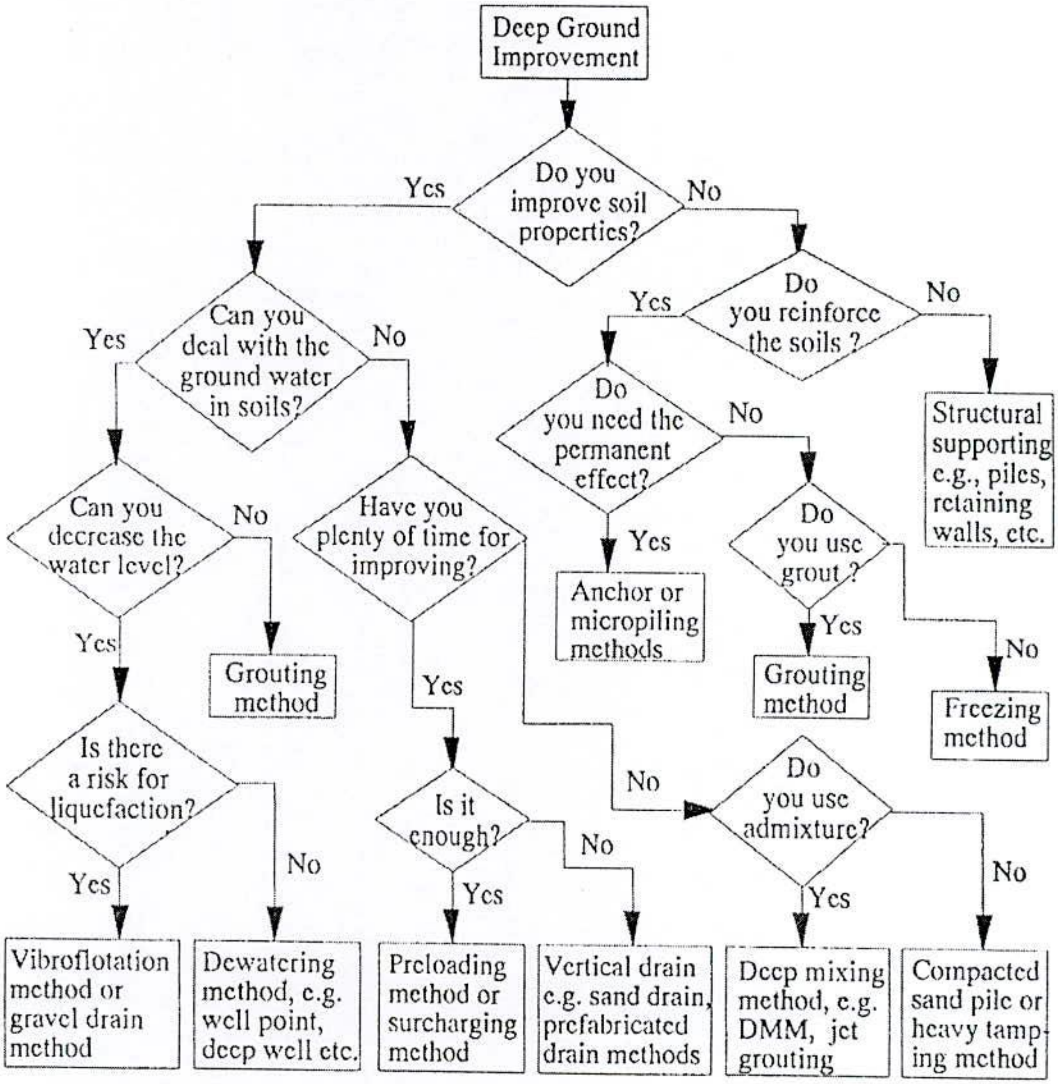


Fig. 2.1 Chart for selection of ground improvement techniques
(After Bergado and Miura 1994)

2.7 Foundation Practice on Soft Ground of Khulna Region

Foundation practice in soft soils depends on the index and engineering properties of the sub-soil. Normally for soft grounds raft or mat foundation, floating foundation, and transferring the load to the deeper hard strata by piles have been practiced for long time and for soft soil foundation, stone columns and granular piles are also used in few projects. For some better results soft soil may be replaced by soil of good quality. Ground improvement techniques are adopted for soft soils for the construction of foundation at marginal projects. At present study for improvement reconstituted organic soil, collected from a selected location of Khulna region, was improved by compacted sand column, compacted sand bed with and without geotextile and also further enhancing with compacted sand column.

2.8 Determination of Bearing Capacity by Established Method

One of the early sets of bearing capacity equations was proposed by Terzaghi and then it was modified by Meyerhof, Hansen and Vesic considering different important parameter like size, shape and inclination. The well established equations are discussed in the following consecutive articles.

2.8.1 Terzaghi's bearing capacity equation

Terzaghi's equations were produced from a slightly modified bearing-capacity theory developed from using the theory of plasticity

$$q_{ult} = cN_c + \bar{q}N_q + \gamma BN_\gamma \tag{1}$$

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5BN_\gamma s_\gamma \tag{2}$$

$$N_q = \frac{a^2}{a\cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{py}}{\cos^2\phi} - 1\right)$$

Where, value of s_c and s_γ are:

For :	Strip	Round	Square
s_c =	1.0	1.3	1.3
s_γ =	1.0	0.6	0.8

Table 2.3 Bearing-capacity factors for the Terzaghi's equation

ϕ deg	N_c	N_q	N_γ	$k_{p\gamma}$
0	5.7*	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	--
35	57.8	41.4	42.4	82.0
40	59.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	--
50	347.5	415.1	1153.2	800.0

*$N_c=1.5\pi+1$.

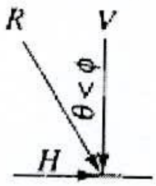
Values of N_γ for ϕ of 0°, 34°, and 48° are original Terzaghi Values and used to back-compute $k_{p\gamma}$

2.8.2 Meyerhof's bearing capacity equation

Vertical load: $q_{ult} = cN_c s_c d_c + \bar{q}\, N_q s_q d_q + 0.5\,\gamma\, B'N_\gamma\, S_\gamma\, d_\gamma$ (3)

Inclined load: $q_{ult} = cN_c d_c i_c + \bar{q}\, N_q d_q i_q + 0.5\,\gamma\, B'N_\gamma\, d_\gamma\, i_\gamma$ (4)

Table 2.4 Shape, depth, and inclination factors for the Meyerhof's bearing-capacity equations

Factors	Value	For
Shape:	$s_c = 1 + 0.2K_p\dfrac{B}{L}$	Any ϕ
	$s_q = s_\gamma = 1 + 0.1K_p\dfrac{B}{L}$	$\phi > 10°$
	$s_q = s_\gamma = 1$	$\phi = 0$
Depth:	$d_c = 1 + 0.2\sqrt{K_p}\dfrac{D}{B}$	Any ϕ
	$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\dfrac{D}{B}$	$\phi > 10$
	$d_q = d_\gamma = 1$	$\phi = 0$
Inclination: 	$i_c = i_q = \left(1 - \dfrac{\theta°}{90°}\right)^2$	Any ϕ
	$i_\gamma = \left(1 - \dfrac{\theta°}{\phi°}\right)^2$	$\phi > 0$
	$i_\gamma = 0 \text{ for } \theta > 0$	$\phi = 0$

Where $K_p = \tan^2(45 + \phi/2)$ as in Fig. 4-2

θ = angle of resultant R measured from vertical without a sign; if $\theta = 0$ all $i_i = 1.0$.

B, L, D = previously defined

2.8.3 Hansen's bearing capacity equation

General: $q_{ult} = cN_cs_cd_ci_cg_cb_c + \bar{q}N_qs_qd_qi_qg_qb_q + 0.5\gamma B'N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$ (5)

When $\phi = 0$

Use $q_{ult} = 5.14s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$ (6)

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 1.5(N_q - 1)\tan\phi$

2.8.4 Vesic's bearing capacity equation

Use Hansen's equations above.

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 2(N_q + 1)\tan\phi$

Table 2.5 Bearing-capacity factors for the Meyerhof's, Hansen's, and Vesic's bearing capacity equation

Note that N_c and N_q are the same for all three methods; subscripts identify author for N_γ

ϕ	N_c	N_q	$N_{\gamma(H)}$	$N_{\gamma(M)}$	$N_{\gamma(V)}$	N_q/N_c	$2\tan\phi(1-\sin\phi)^2$
0	5.14*	1.0	0.0	0.0	0.0	0.195	0.000
5	6.49	1.6	0.1	0.1	0.4	0.242	0.146
10	8.34	2.5	0.4	0.4	1.2	0.296	0.241
15	10.97	3.9	1.2	1.1	2.6	0.359	0.294
20	14.83	6.4	2.9	2.9	5.4	0.431	0.315
25	20.71	10.7	6.8	6.8	10.9	0.514	0.311
26	22.25	11.8	7.9	8.0	12.5	0.533	0.308
28	25.79	14.7	10.9	11.2	16.7	0.570	0.299
30	30.13	18.4	15.1	15.7	22.4	0.610	0.289
32	35.47	23.2	20.8	22.0	30.2	0.653	0.276
34	42.14	29.4	28.7	31.1	41.0	0.698	0.262
36	50.55	37.7	40.0	44.4	56.2	0.746	0.247
38	61.31	48.9	56.1	64.0	77.9	0.797	0.231
40	75.25	64.1	79.4	93.6	109.3	0.852	0.214
45	133.73	134.7	200.5	262.3	271.3	1.007	0.172
50	266.50	318.5	567.4	871.7	761.3	1.195	0.131

* $= \pi + 2$ as limit when $\phi \rightarrow 0°$.

Slight differences in above table can be obtained using program BEARING.EXE on diskette depending on computer used and whether or not it has floating point.

Table 2.6 Shape and depth factors for use in either the Hansen's or Vesic's bearing-capacity equations. Use s'_c & d'_c when $\phi = 0$ only for Hansen equations. Subscripts H, V for Hansen, Vesic, respectively.

Shape factors	Depth factors
$s'_{c(H)} = 0.2\dfrac{B'}{L'}$ $(\phi = 0°)$ $s_{c(H)} = 1.0 + \dfrac{N_q}{N_c} \cdot \dfrac{B'}{L'}$ $s_{c(V)} = 1.0 + \dfrac{N_q}{N_c} \cdot \dfrac{B}{L}$ $s_c = 1.0$ for strip	$d'_c = 0.4k$ $(\phi = 0°)$ $d_c = 1.0 + 0.4k$ $k = D/B$ for $D/B \le 1$ $k = \tan^{-1}(D/B)$ for $D/B > 1$ k in radians
$s_{q(H)} = 1.0 + \dfrac{B'}{L'}\sin\phi$ $s_{q(V)} = 1.0 + \dfrac{B}{L}\tan\phi$ for all ϕ	$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2 k$ k defined above
$s_{\gamma(H)} = 1.0 - 0.4\dfrac{B'}{L'} \ge 0.6$ $s_{\gamma(V)} = 1.0 - 0.4\dfrac{B}{L} \ge 0.6$	$d_\gamma = 1.00$ for all ϕ

Notes:

1. Note use of "effective" base dimensions B', L' by Hansen but not by Vesić.
2. The values above are consistent with either a vertical load or a vertical load accompanied by a horizontal load H_B.
3. With a vertical load and a load H_L (and either $H_B = 0$ or $H_B > 0$) you may have to compute two sets of shape s_i and d_i as $s_{i,B}$, $s_{i,L}$ and $d_{i,B}$, $d_{i,L}$. For i, L subscripts of Eq. (4-2), presented in Sec. 4-6, use ratio L'/B' or D/L'.

2.8.5 Skempton's bearing capacity equation

Skempton's equation is widely used for undrained clay soils:

$q_f = s_u . N_{cu} + q_o$ (7)

where N_{cu} = Skempton's bearing capacity factor, which can be obtained from a chart 2.2 or by using the following expression:

$N_{cu} = N_c . s_c . d_c$

where s_c is a shape factor and dc is a depth factor.

$N_q = 1$, $N_g = 0$, $N_c = 5.14$

$s_c = 1 + 0.2\ (B/L)$ for B<=L

$d_c = 1+$ Ö$(0.053\ D/B)$ for $D/B < 4$

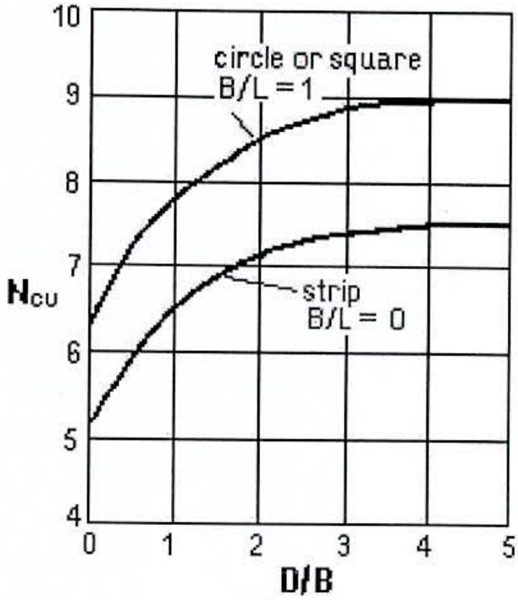


Fig. 2.2 Skempton's bearing capacity factor

2.8.6 Bearing capacity of footing on layered soils

It is necessary to place footing on stratified deposits where the thickness of the top stratum from the base of the footing d_1 is less than the H distance computed as in Fig. 4-2. In this case the rupture zone will extend into the lower layer(s) depending on their thickness and require some modification of q_{ult}. There are three general cases of the footing on a layered soil as follows:

Case 1. Footing on layered clays (all $\phi = 0$) as in Fig. 22.

a. Top layer weaker than lower layer ($c_1 < c_2$)

b. Top layer stronger than lower layer ($c_1 > c_2$)

Case 2. Footing on layered 0-c soils with a, b same as case 1.

Case 3. Footing on layered sand and clay soils as in Fig. 23.

a. Sand overlying clay

b. Clay overlying sand

Experimental work to establish methods to obtain qu\t for these three cases seems to be based mostly on models—often with B < 75 mm. Several analytical methods exist as well, and apparently the first was that of Button (1953), who used a circular arc to search for an approximate minimum, which was found (for the trial circles all in the top layer) to give $N_c = 5.5 < 2\pi$

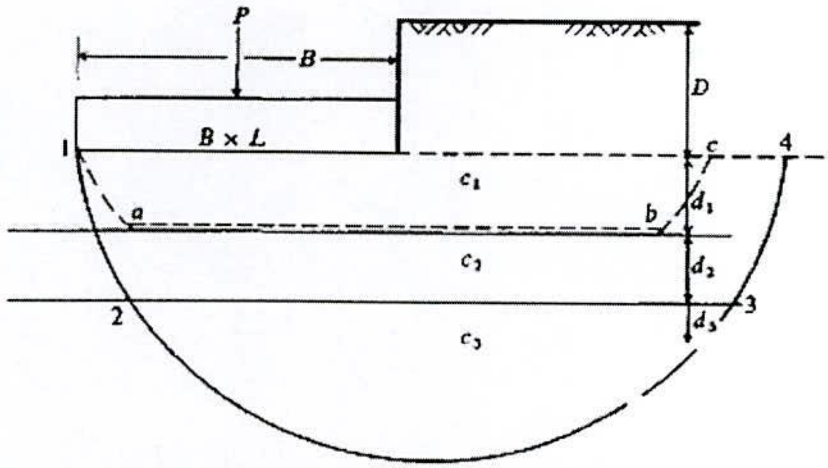


Fig. 2.3 Footing on layered clay soil. For very soft C_1 failure may occur along sliding block 1abc and not a circular arc and reduce N_c to a value less than 5.14.

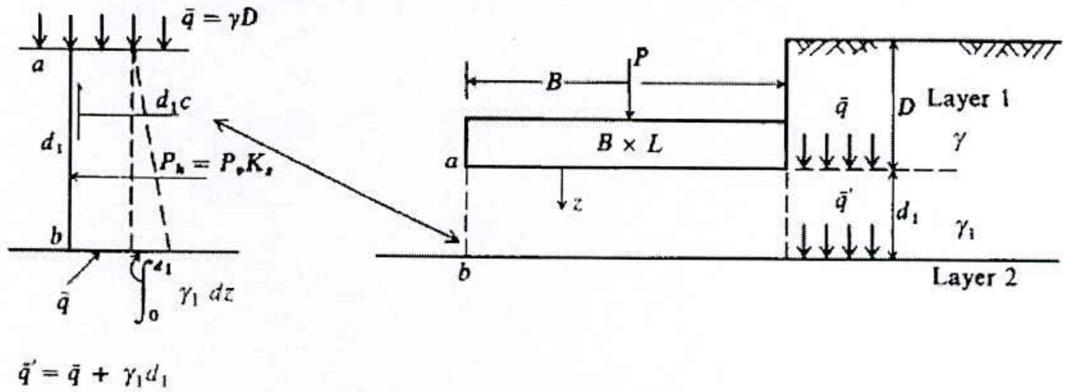


Fig. 2.4 Footings on layered soil.

The use of trial circular arcs can be readily programmed for a computer for two or three layers using s_u for the layers. Note that in most cases the layer s_u will be determined form q_u tests, so the circle method will give reasonably reliable results. It is suggested that circular arcs be limited to cases where the strength ratio $C_R = c_2 / c_1$ of the top two layers is on the order of

$$0.6 < CR \leq 1.3$$

Where C_R is much out of this range there is a large difference in the shear strengths of the two layers, and one might obtain N_c using a method given by Brown and Meyerhof (1969) based on model tests as follows:

For $C_R > 1$

$$N_{c,s} = \frac{1.5 d_1}{B} + 5.14 C_R \leq 5.14 \quad \text{For strip footing} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For a circular base with B = diameter

$$N_{c,r} = \frac{3.0d_1}{B} + 6.05C_R \qquad \leq 6.05 \qquad \text{For round base} \qquad \ldots\ldots\ldots\ldots\ldots\ldots(9)$$

When CR > 0.7 reduce the foregoing N_{cj} by 10 percent.

For C_R >1

$$N_{1,s} = 4.14 + \frac{0.5B}{d_1} \qquad \text{(strip)} \qquad (10)$$

$$N_{2,s} = 4.14 + \frac{1.1B}{d_1} \qquad (11)$$

$$N_{1,r} = 5.05 + \frac{0.33B}{d_1} \qquad \text{(round base)} \qquad (12)$$

$$N_{2,r} = 5.05 + \frac{0.66B}{d_1} \qquad (13)$$

In the case of CR > 1 we compute both $N_{1.i}$ and $N_{2.i}$ depending on whether the base is rectangular or round and then compute an averaged value of N_{ci} as

$$N_{c,i} = \frac{N_{1,i} \cdot N'_{2,i}}{N_{1,i} + N_{2,i}} \cdot 2 \qquad (14)$$

The preceding equations give the following typical values of $N_{c.i}$ which are used in the bearing-capacity equations of Table 2.3 for N_c.

Table 2.7 the value for C_R

	C_R = 0.4			2.0	
d_1/B	Strip	Round	$N_{1,s}$	$N_{2,s}$	$N_{c,s}$
0.3	2.50	3.32	5.81	7.81	6.66
0.7	3.10	4.52	4.85	5.71	5.13
1.0	3.55	5.42	4.64	5.24	4.92

When the top layer is very soft with a small d_1/B ratio, one should give consideration either to placing the footing deeper onto the stiff clay or to using some kind of soil improvement method. Model tests indicate that when the top layer is very soft it tends to squeeze out from beneath the base and when it is stiff it tends to "punch" into the lower softer layer. If $q_{ult} > 4c_1 + \bar{q}$, the soil may squeeze from beneath the footing.

Purushothamaraj et al. (1974) claim a solution for a two-layer system with ϕ-c soils and give a number of charts for N_c factors; however, their values do not differ significantly from N_c in Table 2.5. From this observation it is suggested for ϕ-c soils to obtain modified ϕ and c values as follows:

$$\phi' = \frac{d_1\phi_1 + (H - d_1)\phi_2}{H} \tag{15}$$

1. Compute the depth H = 0.5Btan (45 + ϕ/2) using ϕ for the top layer.
2. If H > d_1 compute the modified value of φ for use as
3. Make a similar computation to obtain c'.
4. Use the bearing-capacity equation (1) to (6) for q_{ult} with ϕ and c'.

For bases on sand overlying clay or clay overlying sand, first check if the distance H will penetrate into the lower stratum. If H > d_1 (refer to Fig. 2.3 & 2.4) you might estimate q_{ult} as follows:

1. Find q_{ult} based on top-stratum soil parameters using an equation of bearing capacity.
2. Assume a punching failure bounded by the base perimeter of dimensions B x L. Here include the q contribution from d_1, and compute q'_{ult} of the lower stratum using the base dimension B. You may increase q'_{ult} by a fraction k of the shear resistance on the punch perimeter (2B + 2L) x ksu if desired.
3. Compare q_{ult} to q'_{ult} and use the smaller value.

In equation form the preceding steps give the controlling q'_{ult} as

$$q'_{ult} = q''_{ult} + \frac{pP_v K_s \tan\phi}{A_f} + \frac{pd_1 c}{A_f} \qquad \leq q_{ult} \tag{16}$$

Where q_{ult} = bearing capacity of top layer from equations (1) to (6)

q''_{ult} = bearing capacity of lower layer computed as for q_{ult} but also using B = footing dimension, $\bar{q} = \gamma d_1$; c, ϕ of lower layer

p = total perimeter for punching [may use 2(B + L) or π x diameter]

P_v = total vertical pressure from footing base to lower soil computed as $\int_0^{d_1} \gamma h dh + \bar{q} d_1$

K_s = lateral earth pressure coefficient, which may range from $\tan^2(45 \pm \phi/2)$ or use K_o

$\tan\phi$ = coefficient of friction between $P_V K_S$ and perimeter shear zone wall

pd_1c = cohesion on perimeter as a force

A_f = area of footing (converts perimeter shear forces to a stress)

$$c_{av} = \frac{c_1H_1 + c_2H_2 + c_3H_3 + \cdots + c_nH_n}{\sum H_i} \tag{17}$$

$$\phi_{av} = \tan^{-1}\frac{H_1\tan\phi_1 + H_2\tan\phi_2 + \cdots + H_n\tan\phi_n}{\sum H_i} \tag{18}$$

2.8.7 The Sellmeijer's bearing capacity method

This calculation method is used to calculate the effect of membrane reinforcing provided by a geotextile inserted between a poor load-bearing sub grade and the aggregate layer of unpaved road of a road construction which still has to be paved. The sub grade and geotextile will deform under load. As a result the geotextile is strained and come into tension. The load in the zone under load is thus relieved by the geotextile which, in turn loads the adjacent zone.

In the rigid plastic phase the sub grade behaviour can be described using the Brinch-Hasen bearing capacity formula:

$$q_1 = N_c\,(c + \frac{3}{4} B \gamma^1 \tan^2\phi) \tag{19}$$

Where

c = Subsoil cohesion

ϕ = angle of internal friction of sub grade

γ^1 = effect of unit weight of sub grade

N_c= bearing capacity factor

B = footing width

2.9 Determination Bearing Capacity of Ground Treated by Granular Column

A number of methods for evaluation of load carrying capacity of stone columns/or Rammed Aggregate Pier are available. These include methods by different researchers developed using various approaches, which are described in the following sections

2.9.1 Passive pressure condition

Greenwood (1970) proposed that the surrounding clay media of the stone column can be expected to mobilize passive pressure conditions during failure. The stone column materials get compressed axially and expand laterally. He proposed the following equation for evaluation of the lateral stress.

$$\sigma_R = \gamma\ z\ Kpc\ +2c \sqrt{Kps} \tag{20}$$

Where, σ_R = passive resistance of the soil; γ = unit weight of the clay; c = cohesion of the clay; z = depth of the clay; K_{pc} = Ranking coefficient of passive resistance. The ultimate stress, q_u carried by the stone column will be

$$q_u = \sigma_R K_{ps} \tag{21}$$

Where, $K_{ps} = \tan^2 (45^0+\phi'/2)$; ϕ' = angle of shearing resistance of the granular material

2.9.2 Based on expansion of a cylinder

Gibson and Anderson (1961) proposed the following equation to evaluate the limiting stress in a cylindrical cavity. They assumed that the surrounding clay media of the stone column will behave like an ideal elasto-plastic material.

$$\sigma'_R = \sigma_{Ro} + c_u \left(1+\log_e \left(E/2\ (1+\mu)\ c_u\right)\right) \tag{22}$$

Where, σ_{Ro} = Total in situ radial stress; E = modulus of elasticity of the clay; μ = Poisson's ratio of the clay ; c_u = undrained cohesion of clay.

In other words the stone column can be thought of as being confined in a triaxial stress system where the cell pressure is limited. Therefore, there is an ultimate load that the column can carry. From a detailed examination of many field records of quick expansion pressuremeter test, Eq.2.3 is modified as suggested by Hughes and Withers (1974) for the normal range of E/c_u. This may be expressed as,

$$\sigma_R = \sigma'_{Ro} + 4c_u + u = \sigma_{Ro} + 4c_u \tag{23}$$

$$\text{Ultimate stress,} \quad q_u = k_{ps}(\sigma_{Ro} + 4c_u) \tag{24}$$

Where, σ'_{Ro} = effective insitu radial stress; u = pore pressure.

2.9.3 Based on cavity expansion theory

Vesic (1972) proposed the cavity expansion theory which constitutes the main theoretical basis of estimation of the yield stress or the maximum vertical stress in a stone column, beyond which excessive deformations would occur. The cavity expansion theory can be applied to evaluate the vertical yield stress according to the following equation.

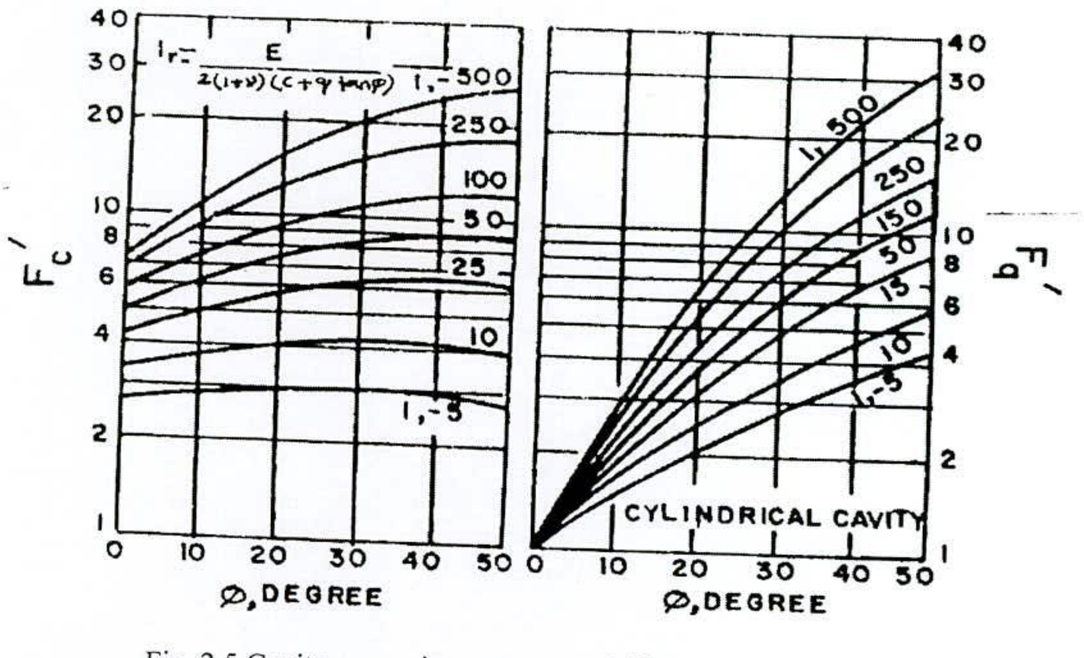


Fig. 2.5 Cavity expansion parameters (After Vesic, 1972)

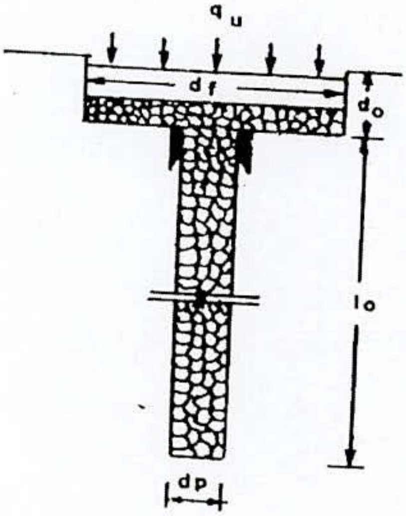


Fig. 2.6 Definition sketch.

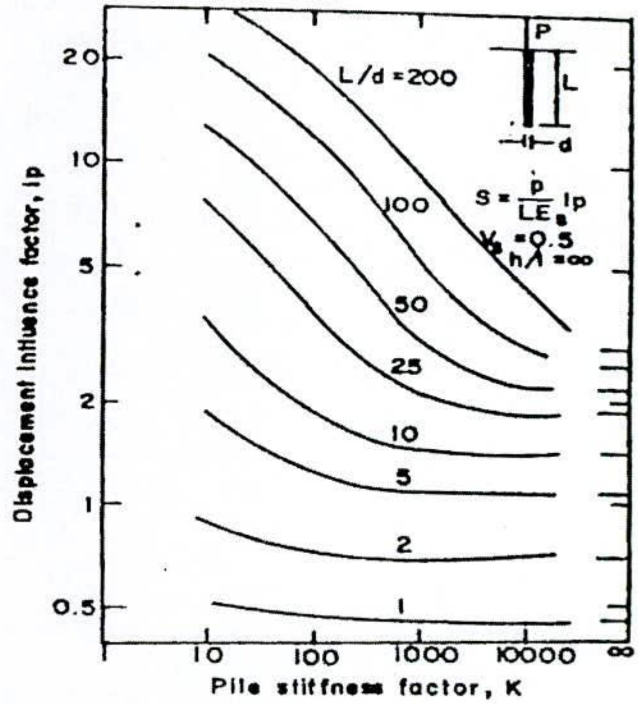


Fig. 2.7 Displacement influence factors.

$$\sigma_R = Fc' \ c_u + Fq' \ q \tag{25}$$

$$\sigma_\theta = \sigma_R \ N\phi \tag{26}$$

where , $N\phi = (1+\sin\phi)/(1-\sin\phi)$; c_u = undrained shear strength of clay ; q= effective mean normal stress ; ϕ = angle of shearing resistance ; σ_R = principal stress in the radial direction ; σ_θ = principal stress in the circumferential direction ; F'c, F'q = the cavity expansion factors for cylindrical cavity. These two parameters (F'c, F'q) depend on angle of shearing resistance of soil (ϕ) and the Relative Rigidity Index of the soil (Irr) as shown in Fig.2.9, which is a function of Rigidity Index (Ir) and volumetric strain in the plastic region (Δ). The Rigidity Index (Ir) is a function of modulus of elasticity (E), Poisson's ratio (υ) , cohesion (c) , angle of shearing resistance (ϕ) and effective mean normal stress (q) of the soil. The suggested relations by Vasic are:

$$F'q = (1+\sin\phi \ (Irr \sec\phi)^{\frac{Sin\phi}{1+Sin\phi}} \tag{27}$$

$$F'c = (F'q - 1)\cot\phi \tag{28}$$

Where, $Irr = Ir/ (1+Ir \Delta \sec\phi) = \xi 'v \; Ir$; $\xi 'v$ = volume change factor for a cylindrical cavity ; $Ir = E/ (2 (1+ \upsilon) (c+ q \tan\phi))$

For $\phi = 0$, and for incompressible soil ($\Delta = 0$)

$$F 'c = \ln Ir +1$$

This is identical with the value found by Gibson and Anderson for frictionless soil. The cavity expansion theory is a very useful tool for understanding the factors influencing the yield values of the vertical stress in the stone column and for interpreting the load test data so that the test results can be used for evaluating design parameters.

2.9.4 Based on pile formula

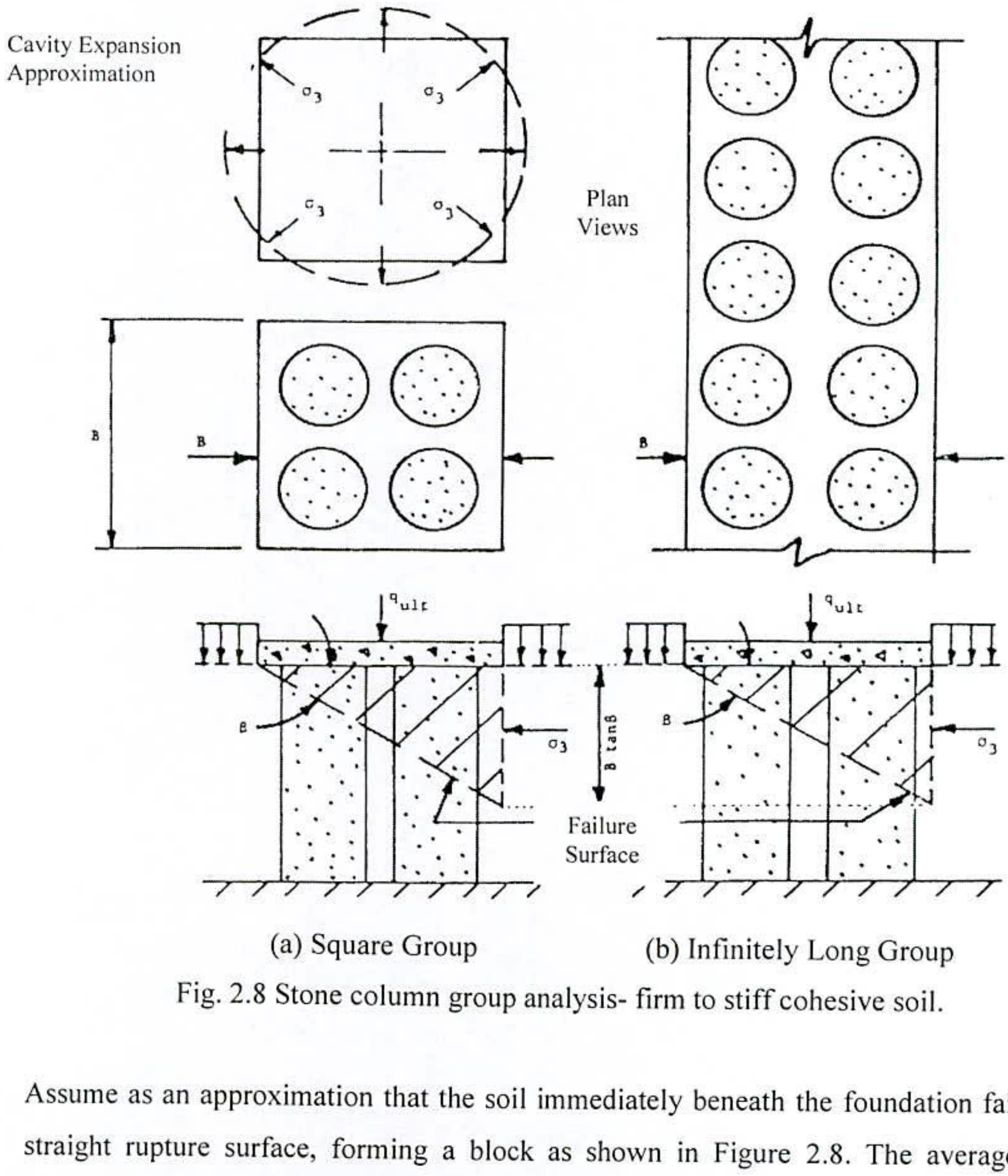
The ultimate load carrying capacity of stone column can be estimated using the conventional formulas which are used to evaluate the load carrying capacity of piles. In this case, total vertical load is carried by the skin friction which is developed between the pile and clay interface due to the movement of pile and end bearing, which is developed at the base of pile. The vertical load carried by the stone column is calculated by the following equation.

$$q_u = c_u (4 (l/d) + 9) \qquad (29)$$

Where, q_u = ultimate stress carried by the stone column; c_u = undrained shear strength of clay; d = diameter of stone column; l = length of stone column. In Eq. 26, it is assumed that the shaft friction is equal to the undrained shear strength (c_u) of clay. It is also assumed that the frictional resistance is constant throughout the length of stone column. The bearing capacity factor for deep foundation is taken as 9.

2.9.5 Ultimate capacity of stone column

The ultimate strength of either a square or infinitely long, rigid concrete footing resting on the surface of a cohesive soil reinforced with stone columns as illustrated in Fig.2.8. Assume the foundation is loaded quickly so that the undrained shear strength is developed in the cohesive soil, with the angle of internal friction being negligible. Also neglect cohesion in the stone column. Finally, assume, for now, the full shear strength of both the stone column and cohesive soil is mobilized. The ultimate bearing capacity of the group can be determined by approximating the failure surface by two straight rupture lines. Such a theory was first developed for homogeneous soils, by Bell and modified by Terzaghi and Sowers (1979). For homogeneous soils, this theory compares favorably with the bell bearing capacity theory and gives results reasonably close to the Terzaghi's local bearing failure theory.



(a) Square Group (b) Infinitely Long Group

Fig. 2.8 Stone column group analysis- firm to stiff cohesive soil.

Assume as an approximation that the soil immediately beneath the foundation fails on a straight rupture surface, forming a block as shown in Figure 2.8. The average shear resistance of the composite soil would be developed on the failure surface. The ultimate stress (q_{ult}) of the composite soil with stand depends upon the lateral ultimate resistance (σ_3) of the block movement and the composite shear resistance developed along the inclined shear surface. From a consideration of equilibrium of the block, the average shear strength parameters within the block are

$$[\tan\phi]_{avg} = \mu_s \; a_s \tan\phi_s \tag{30}$$

$$c_{avg} = (1 - a_s)c \tag{31}$$

Where $[\tan\phi]_{avg}$ the tangent of the composite angle of internal friction and c_{avg} is the composite cohesion on the shear surface beneath the foundation; a_s is the area replacement ratio and μ_s is the stress concentration factor for the stone, respectively. As mentioned in a statement, the strength components due to cohesion of the granular material and friction of the clay are neglected in this derivation. The failure surface makes an angle β with the foundation, where β for the composite soil is

$$\beta = 45 + \frac{\phi_{avg}}{2} \tag{32}$$

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

To calculate the ultimate capacity for a group first determined the ultimate lateral pressure σ_3. For an infinitely long footing from classical earth pressure theory for saturated clay having only cohesion c is

$$\sigma_3 = \frac{\gamma_c \; B\tan\beta}{2} + 2c \tag{33}$$

Where: σ_3 = average lateral confining pressure

γ_c = saturated or wet unit weight of the cohesive soil

B = foundation width

β = inclination of the failure surface as given by equation (29)

c = undrained shear strength within the unreinforced cohesive soil.

The lateral confining pressure for a square foundation can be determined using the Eq. 22 proposed by Vesic based on cavity expansion theory. The Vesic's cylindrical expansion theory gives the ultimate stress that can be exerted on the failure block by the surrounding soil. The three-dimensional failure on a cylindrical surface should give a satisfactory approximation of the three-dimensional failure of a square foundation.

Assuming the ultimate vertical stress q_{ult} (which is also assumed to be σ_1) and ultimate lateral stress σ_3 to be principal stresses, equilibrium of the wedge requires

$$q_{ult} = \sigma_3 \tan^2 \beta + 2\, c_{avg} \tan \beta \tag{34}$$

Area Replacement Ratio: The volume of soil replaced by stone columns or Rammed Aggregate Pier has an important effect upon the performance of the improve ground. To quantify the amount of soil replacement, defined the Area Replacement Ratio, a_s, as the fraction of soil tributary to the stone column replaced by the stone:

$$a_s = A_s/A \tag{35}$$

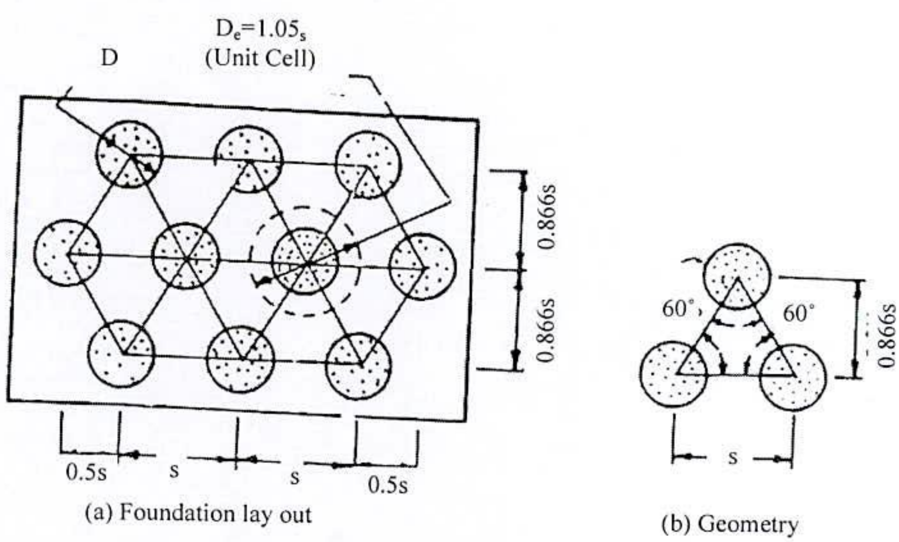


Fig. 2.9 (i) Equilateral triangular pattern of stone column.

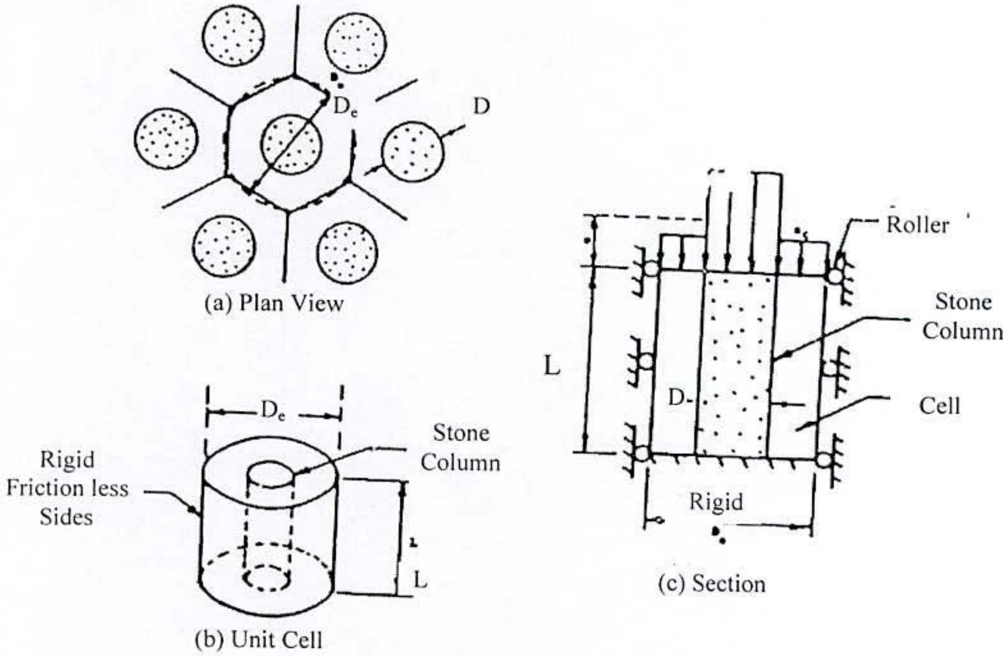


Fig. 2.9(ii) Unit cell idealization.

Where, A_s is the area of the stone column after compaction and A is the total area within the unit cell (Fig. 2.9.i.a). Further, the ratio of the area of the soil remaining, A_c, to the total area is then

$$a_c = A_c / A \qquad (36)$$

$$= 1 - a_s$$

The area replacement ratio, a_s, can be expressed in terms of diameter and spacing of the stone columns as follows:

$$a_s = C_1 \left(\frac{D}{s}\right)^2 \qquad (37)$$

Where, D = diameter of the compacted stone column

s = center- to- center spacing of the stone columns

C_1 = a constant dependent upon the pattern of stone columns used; for a square pattern

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

For equilibrium triangular pattern of stone columns and Rammed Aggregate Pier the area replacement ratio is then expressed as,

$$a_s = 0.907 \left(\frac{D}{s}\right)^2 \tag{38}$$

In working with ground improvement using stone columns, it is important to think in terms of the area replacement ratio, a_s.

Stress Concentration: Stress concentration occurs in the stone column since it is stiffer than the ambient cohesive or loose cohesionless soil. Now consider the conditions for which the 'unit cell' concept is valid such as a reasonably wide, relatively uniform loading applied to a group of stone columns having either a square or equilateral triangular pattern. The distribution of vertical stress within a 'unit cell' (Fig. 2.9.ii.c) can be expressed by a stress concentration factor 'n' defined as

$$n = \sigma_s / \sigma_c \tag{39}$$

Where: σ_s = stress in the stone column

σ_c = stress in the surrounding cohesive soil

The average stress σ which must exist over the unit cell area at a given depth must, for equilibrium of vertical forces to exist within the unit cell, equal for a given area replacement ratio, a_s

$$\sigma = \sigma_s . a_s + \sigma_c (1 - a_s) \tag{40}$$

Where, all the terms have been previously defined. Solving equation (40) for the stress in the clay and stone using the stress concentration factor n gives (Aboshi et-al. 1979 and Barksdale 1981).

$$\sigma_c = \sigma / [1 + (n-1)a_s] . = \pi_c \sigma \tag{41}$$

$$\sigma_s = n \sigma / [1 + (n-1)a_s] . = \pi_s \sigma \tag{42}$$

Here π_c and π_s are the ratio of stresses in the clay and stone, respectively, to the average stress σ over the tributary area. For a given set of field conditions, the stress in the stone and clay can be readily determined using Eqs.41 and 42 if a reasonable value of the stress concentration factor is assumed based on previous measurements. The above σ, σ_c and σ_s stresses are due to the applied loading.

Almost all the existing theories for design of stone column are based on liner material behaviour and limit state analysis. In these methods material behaviour are characterized by single parameter representation which in most of the cases will fail to predict realistic behaviour. The limitations of the existing theories are discussed in the following sections.

2.9.6 Hughes et al. (1975) formula

An approximate formula for the allowable bearing pressure q_a of stone columns is given by Hughes et al. (1975)

$$q_a = \frac{K_p}{\mathrm{SF}}(4c + \sigma_r') \qquad (43)$$

where $K_p = \tan^2(45° + \phi/2)$

ϕ' = drained angle of internal friction of stone

c = either drained cohesion (suggested for small column spacings) or the undrained shear strength su when the column spacing is over about 2 m

σ'_r = effective radial stress as measured by a pressuremeter (but may use 2c if pressure meter data are not available)

SF = safety factor—use about 1.5 to 2 since Eq. (6-5) is fairly conservative

The allowable load Pa on the stone column of average cross-sectional area Ac = $0.7854D^2_{col}$ is

$$P_a = q_a A_c \qquad (44)$$

where q_a = allowable bearing pressure from Eq. (6-5)

We can also write the general case of the allowable column load P_a as

$$P_a = (c_s A_s + A_c c_p N_c) \cdot \frac{1}{\mathrm{SF}} \qquad (45)$$

where c_s = side cohesion in clay—generally use a "drained" value if available;

c_s is the side resistance (γ zK $\tan\delta$) in sand

c_p = soil cohesion at base or point of stone column

A_s = average stone column perimeter area

To compute A_s, use the in-place volume of stone Vc and initial column depth Lc as follows:

$$A_c L_c = 0.7854 D_{\text{col}}^2 L_c = V_c \qquad \text{and} \qquad D_{\text{col}} = \sqrt{\frac{V_c}{0.7854 L_c}}$$

$$A_s = \pi D_{\text{col}} L_c$$

Observe that, by using the volume of stone V_c, the diameter D_{coi} computed here is the nominal value. N_c = bearing capacity factor, but use 9 for clay soils if $L_C/D_{Col} > 3$ (value between 5.14 and 9 for smaller L/D)

The allowable total foundation load is the sum of the several stone column contributions beneath the foundation area. Stone columns should extend through soft clay to firm strata to control settlements. If the end-bearing term ($A_cC_pN_c$) of Eq. 45 is included when the column base is on firm strata, a lateral bulging failure along the shaft may result. The bulge failure can develop from using a column load that is too large unless the confinement pressure from the soil surrounding the column is adequate. The failure is avoided by load testing a stone column to failure to obtain a P_{ult} from which the design load is obtained as P_{ult}/SF or by using a large SF in Eq. 45 or by not including the end-bearing term.

Taking this factor into consideration gives a limiting column length L_c (in clay based on ultimate resistance) of

$$P_{\text{ult}} \leq \pi D_{\text{col}} L_c c_s + 9 c_p A_c \qquad A_c = 0.7854 D_{\text{col}}^2$$

$$L_c \geq \frac{P_{\text{ult}} - 7.07 d_p D_{\text{col}}^2}{\pi D_{\text{col}} c_s} \tag{46}$$

Settlement is generally the principal concern with stone columns since their bearing capacity is usually quite adequate. No method is currently available to compute settlement on a theoretical basis. Settlements are estimated on the basis of empirical methods. From this figure we see that stone columns can reduce the settlement to nearly zero depending on column area, spacing, and initial soil strength.

2.10 Failure Patterns of Grounds

The application of a load on a foundation causes some settlement. The three main stages of the load-settlement curve are:

(i) In (O-A) portion of the curve, the load-settlement curve is almost straight due to relatively elastic vertical compression found.

(ii) At point (B), local yielding caused some upward and outward movement of the soil and results in slight surface heave is termed as Local shear failure.

(iii)At point (C), large settlements are produced as plastic yielding is fully developed within the soil is termed as General shear failure.

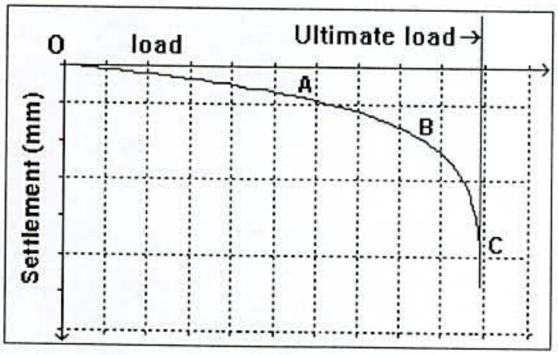


Fig. 2.10 Typical load-settlement curve of ground.

From the description of above curve, it can be said that bearing capacity failure can occur in three different modes: general shear failure, local shear failure, or punching shear failure. Local or punching shear are characterized by relatively large settlements and the ultimate bearing capacity is not clearly defined. In these cases settlement is the major factor in the foundation design.

2.10.1 General shear failure

When a load (Q) is gradually applied on a foundation, settlement occurs which is almost elastic to begin with. At the ultimate load, **general shear failure** occurs when a plastic yield surface develops under the footing, extending outward and upward to the ground surface, and catastrophic settlement and/or rotation of the foundation occurs. The load per unit area at this point is called the **ultimate bearing capacity** (q_f) of the foundation.

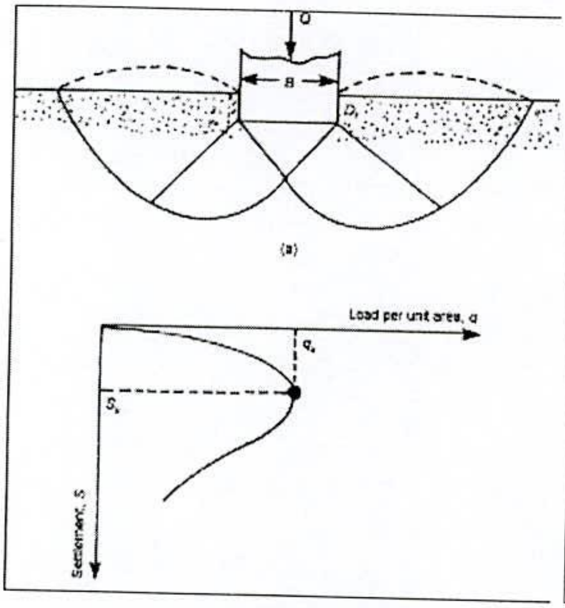


Fig. 2.11 General shear failure.

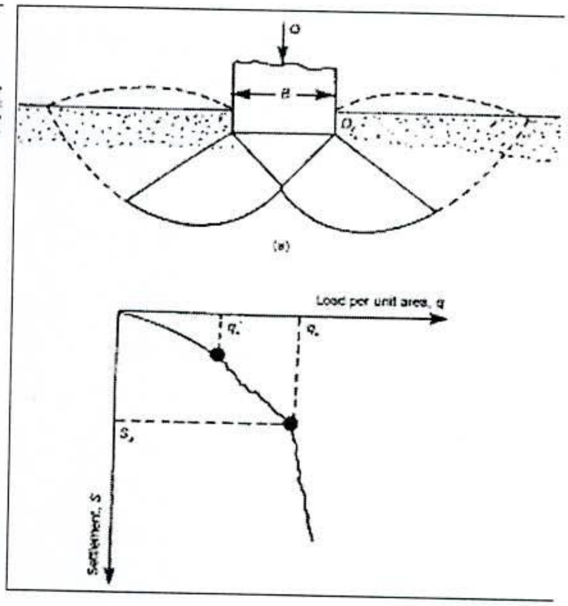


Fig. 2.12 Local shear failure.

2.10.2 Local shear failure

In moderately compressible soils, and soils of medium relative density, significant vertical settlement may take place due to **local shear failure**, i.e. yielding close to the lower edges of the footing. The yield surfaces often do not reach the surface. Several yield developments may occur accompanied by settlement in a series of jerks. The bearing pressure at which the first yield takes place is referred to as the **first-failure pressure ($q_{f(1)}$)** - the term first-failure load ($Q_{f(1)}$) is also used.

2.10.3 Punching shear failure

In weak compressible soils, and soils of low relative density, considerable vertical settlement may take place with the yield surfaces restricted to vertical planes immediately adjacent to the sides of the foundation; the ground surface may be dragged down. After the first yield has occurred the load-settlement curve will steeper slightly, but remain fairly flat. This is referred to as a **punching shear failure.**

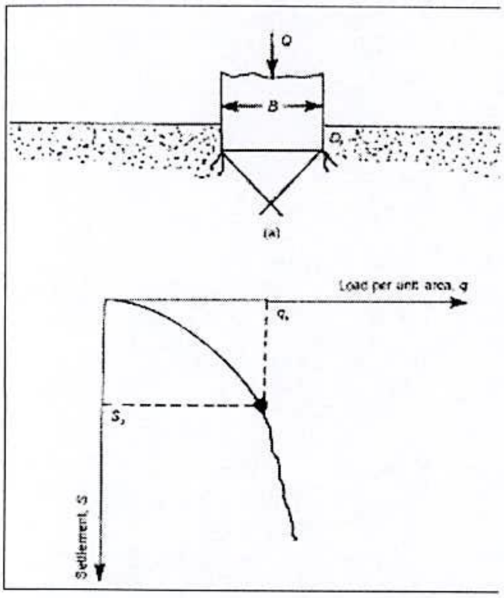


Fig. 2.13 Punching shear failure.

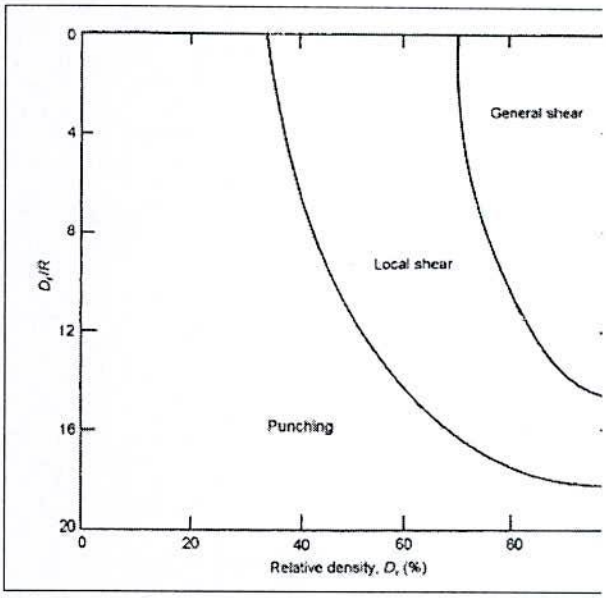


Fig. 2.14 Factor affecting modes of failure.

2.10.4 Factors affecting modes of failure

According to experimental results from foundations resting on sands (Vesic, 1973), the mode of failure likely to occur in any situation depends on the size of the foundation and the relative density of the soil.

The other factors might be affecting the modes of failure as follows:

Permeability: relating to drained/undrained behaviour

Compressibility: similar to RD

Shape: e.g. strips can only rotate one way

Interaction between adjacent foundations and other structures

Relative stiffnesses of soil and footing/structure

Incidence and relative magnitude of horizontal loadings or moments

Presence of stiffer or weaker underlying layers.

2.11 History of Experimental Investigation

Granular columns are used extensively throughout the world as a ground improvement technique due to its proven versatile applicability. Granular columns in end bearing conditions arc cost effective and can installed rapidly using vibroreplacement, compozer, rammed stone column techniques (Madhav 1982) and even by heavy tamping (Van Impe & De Beer 1983). In Europe and U.S.A., the vibroflotation technique is widely used in granular column installation while in Japan; the composer method is widely used (Aboshi, et al. 1979). Most of the literature on the sand compacted pile method has actually reported the degree of vertical stress concentration on sand piles either observed or predicted.

The sand compaction pile (SCP) method, which is soil reinforcing technique, is frequently implemented for improving soft clay and silt and also for loose granular deposit. Granular column offers a valuable technique for (i) increasing bearing capacity and slope stability; (ii) reducing total and differential settlements; (iii) increasing the time rate of consolidation and (iv) reduction of liquefaction potential (Barksdale & Bachus, 1983). Since a very high replacement area ratio of sand piles has been considered in engineering practice, most of the discussion on this soil improvement technique has been concentrated on the bearing capacity (e.g., Ogawa and Ichimoto, 1963; Baumann and Bauer, 1974; Hughes et al., 1975; Takemura et al., 1991), in which the overall bearing capacity increase in the composite ground is interpreted as due to the increase of strength of the sand piles.The soil-column interaction depends on several parameters, including the loading process and the loading rate, the replacement factor, the group effect, and the partial consolidation of the soft soil due to radial drainage through the column (Juran and Guermazi, 1988).

The analytical approach to predict the effect of mixed zone on the settlement response of granular column reinforced clayey soils evaluate the reinforced grounds varying the values of spacing of column, thickness of mixed zone and the relative stiffness. The results lead to the following conclusions: (i) The settlement of the column reinforced ground reduces significantly as the relative stiffness and the thickness of the mixed zone increase; (ii) The time rate of consolidation decreases with the increase of mixed zone thickness and the column spacing; (iii) There is a strong need to determine the actual thickness and stiffness of mixed zone after the installation of column for the design of granular column reinforced clayey soils. Date and Nagaraju (1981) found that the material consumption varies progressively with depth. From field tests, Bergado and Lam (1987) found that the completed diameter of granular column is around 1.05 to 1.35 times of the initial diameter of the hole and it varies progressively with depth. Hughes et al. (1975) observed that the prediction is excellent if allowance is made for transfer of load from column to soil and correct column size.

A laboratory study was conducted to investigate the effect of various parameters on the settlement response of a soft foundation soil reinforced by compacted-sand columns. Triaxial compression tests under different boundary conditions were performed on composite soil specimens made of annular silty soil samples with a central, compacted river-sand column. These tests, performed in a specially modified triaxial cell, show that the group effect, the replacement factor, and the partial consolidation of the soft soil during loading have a significant effect on the vertical stress concentration on the column and on the settlement reduction of the foundation soil. Analysis of the test results indicates that the elastic solution proposed by Balaam and Booker (1981) for drained soil conditions provides a reasonable estimation of the load-transfer mechanism developing in the reinforced soil "unit cell" and of the settlement-reduction ratio.

In recent years major research effort has been applied on use of geotextiles and geogrids soil improvements. Hence it appears possible to use semi-flexible non-horizontal reinforcement in soil to improve its load-bearing capacity for supporting shallow foundation. Vertical reinforcement may be easier to install than the horizontal reinforcement since no soil excavation or recompaction may be needed. A preliminary study for determination of the beneficial effects of vertical reinforcement on the load-bearing capacity of a model footing resting on the surface of a sand layer reinforced with vertical semi-flexible reinforcement (i.e., metal rods) has been reported by Verma and Char (1988). The length of the reinforcement used should be at least equal to the width of the footing and preferably should be 1.5 times the footing width. Reinforcing elements longer than about 1.5 times the footing width may pose problems during placement and may not enhance the bearing capacity further (Puri et al., 2005).

Some experimental studies have been carried out in triaxial apparatus on sand samples reinforced by horizontal thin aluminium plates (Schlosser and Long, 1972), and by horizontal nets of fibre glass (Yang, 1972) uniformly spaced. Failure occurred either by excessive lateral deformations due to sliding of the sand on the reinforcements or by the breakage of the reinforcements. Schlosser and Long (1972) showed that the reinforcements give to the sand an anisotropic cohesion directly proportional to their resistance to tension. Accordingly they interpreted the strength envelope for reinforced sand at failure by breakage of the reinforcements, as that of a cohesive frictional Mohr-Coulomb material. The vertical stresses increase, the horizontal stresses increase in direct proportion. Yang (1972) hypothesized that the tensile stresses built up in the reinforcements were transferred to the soil through sliding friction and caused an increase in the confining pressure. Both, the anisotropic or apparent cohesion approach (Schlosser and Long, 1972) and confining pressure approach (Yang, 1972) are related to each other and either one can be used to analyse failure by reinforcement breakage.

The induced deformations concept was presented by Basset and Last (1978). This concept considers that the mechanism of tensile reinforcement involves anisotropic restraint of the soil deformations in the direction of the reinforcements. This effect results in a rotation of the principal directions of the deformations tensor. Basset and Last (1978) suggested that more can be learnt by analysis of the modifications to strain fields caused by reinforcements than by the study of forces and stresses. Analysis of strain fields also suggests the ideal reinforcing pattern below a shallow footing. The ideal pattern has reinforcement placed horizontally below the footing, which becomes progressively more vertical farther from the footing, i.e., reinforcement should be placed in the direction of major principal strain. This fact was stated by Hausmann (1990) in terms of stress. He stated that the tensile reinforcement is most effective if placed in the major principal plane, in the direction of the minor principal stress, which in many practical geotechnical problems is horizontal. If relatively extensible reinforcements, such as geosynthetics, are used in the same manner as the inextensible reinforcements, they will also inhibit the development of internal tensile strains in the soil and develop tensile stresses. However, the differences between the influence of relatively inextensible and extensible inclusions exist and are significant in terms of the load-settlement behaviour of the reinforced soil system (McGown et al., 1978). It was suggested that the behaviour of the reinforced soil system using extensible reinforcements does not fall within the concepts presented by

Vidal (1969) for reinforced earth and therefore, was termed as ply-soil by McGown and Andrawes (1977). The ply-soil, i.e. the geosynthetic-reinforced soil has greater extensibility and smaller losses of post peak strength compared to sand alone or reinforced earth.

At present, the role of geosynthetic reinforcement in improving the load-carrying capacity and settlement characteristics of the geosynthetic-reinforced foundation soils is regarded in five different ways. The first is the increase of the subgrade bearing capacity changing the failure mode, i.e. geosynthetics tend to force a general, rather than a local failure. The second is the reduction of the maximum applied stress due to a distribution of the applied surface load below the geosynthetics by providing restraint of he granular fill if embedded in it or by providing restraint of the granular fill and the soft foundation soil, if placed at their interface (Giroud et al., 1986; Madhav and Poorooshasb, 1989; Sellmeijer, 1990; Hausmann, 1990). Geosynthetics improve the performance by acting as a separator between the soft soil and the granular fill. This influence is known as separation effect of reinforcement. Use of geogrids has another benefit owing to the interlocking of the soil through the apertures of the grid membrane known as anchoring effect (Guido et al., 1986).

Among the various ground improvement methods by columnar inclusions such as stone columns/ granular piles/ sand compaction piles/ cement columns has been considered as one of the most versatile and cost effective deep ground improvement techniques (Alamgir, 1996 and Alamgir & Miura, 1999). In addition to the installation of granular columns in soft ground, the capacity of the column improved ground can also be enhanced through the placing of a compacted granular fill with or without reinforcement i.e. Geosynthetics (Alamgir et al., 1996 and Shukla, 1995). Settlement of ground improved by granular piles is governed by the compressibility of the soil and the deformability of the granular piles (Mitchell, 1981). The geosynthetic-reinforced granular fill system provides the interaction between the soil and the reinforcing members being slowly by friction generated by gravity (Vidal, 1969). The total and differential settlement of the soft soil is reducing significantly (Shukla, 1995) due to use of geosynthetic - reinforced granular fill.

But till to date, significant research has not been performed on columnar inclusions combined with geosynthetic-reinforcement for improvement of organic ground. Hence, this study has been conducted on the soft organic ground improved compacted sand bed with geotextile and compacted sand column. Some research works have already conducted in the field level to examine the effectiveness of such ground improvement techniques in Khulna region (Zaher, 2000, Sobhan, 2001 and Hossain, 2007). As a follow-up of ongoing research on soft ground in the Department of Civil Engineering, Khulna University of Engineering & Technology, Bangladesh, this research works have been conducting. Hence, reconstituted organic soil grounds have been prepared the laboratory to represent the field condition of this region and improve the grounds by different ground improvement techniques to find out effective solution of the region.

2.12 Case Study

A numerous works have been performed successfully throughout the world to improve soft grounds by compacted sand column, compacted sand bed with and without geotextile and also associated with compacted sand column. These improvement methods give better out put both in load bearing capacity and reduction of settlement. A few case studies are presented here to show the applicability of foundation system.

2.12.1 Load distribution in improved clay by Leung and Tan

A laboratory study has been implemented to examine the load distribution and consolidation characteristics of soft soils reinforced by compacted sand columns. Various factors such as the size and stiffness of the sand column, and the magnitude of the loading pressure were examined.

Under a constant applied pressure, the sand column progressively carried more loads and reached a maximum stress concentration factor after consolidation of clay has been completed. The maximum stress concentration ratio increased with increasing replacement ratio of the sand column. In addition, the maximum stress concentration ratio of the sand columns appeared to be independent of the surcharge loading columns.

2.12.2 Settlement response test by Juran

A laboratory study was conducted to investigate the effect of various parameters on the settlement response of a soft foundation soil reinforced by compacted-sand columns. Triaxial compression tests under different boundary conditions were performed on composite soil specimens made of annular silty soil samples with a central, compacted river-sand column. These tests, performed in a specially modified triaxial cell, showed that the group effect, the replacement factor, and the partial consolidation of the soft soil during loading have a significant effect on the vertical stress concentration on the column and on the settlement reduction of the foundation soil.

This experimental study demonstrates the significant influence of the drainage of the column, the consolidation of the soil, the replacement factor, and the group effect on the settlement response of soft foundation soils reinforced by compacted sand columns. The consolidation and partial drainage of the soil during loading have an important effect on the load-transfer mechanism and should be considered in the analysis/design procedure. The rate of generation and dissipation of the excess pore-water pressures depends primarily upon the replacement factor and the ratio of the loading rate to the soil permeability. The test results show that due to the partial drainage of the soil, reinforcement with a replacement factor significantly smaller than that generally used in practice can efficiently contribute to reduce settlement. The group effect is a fundamental design parameter and prevents the plastic yielding of the column and of the soft soil and, consequently, significantly decreases settlements. In cases of relatively small values of replacement factor, effective mobilization of the group effect requires larger radial strains of the column. Therefore, it is mobilized only when the column reaches a state of plastic yielding, restraining the lateral expansion. Analysis of the loading tests indicates that unit cell elastic solutions provide a reasonable estimation of the reinforcing effect on the settlement response of soft foundation soils.

2.12.3 Benefits due to the use of reinforcement by Rowe and Soderman

The benefits of reinforcement in increasing embankment stability were clearly demonstrated by the Almere test embankment, where unreinforced and reinforced granular fill sections were constructed on a 3.3 m thick clay foundation with undrained shear strength of 8 kPa. The reinforced embankment experienced a relatively ductile failure at a height of 2.75 m, which is in remarkable contrast to the rapid failure of the unreinforced section at 1.75 m thickness. It has been shown that the tensile force mobilized in reinforcement increased the embankment stability after the foundation became plastic and reduced the lateral deformation of the foundation. The stiffer the reinforcement, the higher the embankment could be constructed.

It is evident from the field cases cited that the use of basal geosynthetic reinforcement can increase embankment stability and reduce deformations. The benefits arising from the use of geosynthetic reinforcement include the improvement of embankment behaviour, cost savings, an increase in the feasibility of embankment construction, and the elimination of stage construction in some cases.

2.12.4 Use of reinforcement and prefabricated vertical drains by Lau and Cowland

Lau and Cowland (2000) reported a case where neither reinforcement nor PVDs alone would have been sufficient to allow safe embankment construction to the design height. The combined use of both reinforcement and PVDs increased the short-term stability and made it feasible to construct this 4 m high embankment. The use of PVDs and control of the construction rate effectively reduced the excess pore pressures during construction and accelerated the dissipation of pore pressure after construction. The rate of strength gain of the soft clays due to partial consolidation arising from the presence of the prefabricated vertical drains was rapid and significant.

In this case, the construction of a relatively high embankment over a foundation having extremely low undrained shear strength was achieved by the combined use of reinforcement and PVDs. These cases illustrate the importance of strength gain in the foundation due to partial consolidation during embankment construction where PVDs are used, and highlight the benefit that can arise from considering partial consolidation in design.

2.12.5 Performance of geosynthetic reinforcement materials by S. K. Dash

A laboratory model tests carried out to study the relative performance of different forms of reinforcement in sand beds under strip loading. The results demonstrate that geocell reinforcement is the most advantageous soil reinforcement technique of those investigated. With the provision of geocell reinforcement, failure was not observed even at a settlement equal to about 45% of the footing width and a load as high as eight times the ultimate capacity of the unreinforced soil, whereas, with planar reinforcement, failure took place at a settlement of about 15% of the footing width and a load of about four times the ultimate capacity of the unreinforced soil. For the case with randomly distributed reinforcement mesh, failure was recorded at a load of about 1.8 times the ultimate capacity of the unreinforced soil and at a settlement of about 10% of the footing width (Dash et al., 2004).

2.13 Remarks

Soft ground is susceptible to severe problem for the construction of massive structures due to its low shear strength and high compressibility. If the thickness of poor ground layer is low, it is easy to transfer load to the hard strata by compacted sand column. Compacted sand column is widely used as a soil improvement technique. If the hard strata are not available within shallow depth, compacted sand bed can be used to reduce settlement within permissible limit and also reduce the risk of differential settlement of the structure. Geosynthetics are used in addition of compacted sand bed to increase the shear strength of soil and also reduction of settlement. In this laboratory investigation organic soft ground is improved by geotextile sandwich compacted sand bed with compacted sand column to investigate improvement in comparison with that of other soil improvement techniques like compacted sand column, compacted sand bed with and without geotextile. So far, no report has been obtained in the available literature on both the theoretical and experimental (both field and laboratory) on the performance of a soft ground improved compacted sand with geotextile layer in conjunction of compacted sand column. Therefore, such research works can be an interesting and worthy reference for both the researchers and practicing engineers.

CHAPTER THREE

STATEMENT OF THE PROBLEM AND LABORATORY INVESTIGATION

3.1 General

The laboratory investigation conducted on the reconstituted organic grounds is described in this chapter. The index and engineering properties of the collected organic soil and the reconstituted organic grounds are also presented here. Experimental grounds were prepared using four different ground improvement techniques in the reconstituted organic soils. Load-settlement test was conducted on these experimental grounds as well as untreated grounds.

3.2 Statement of the Problem

The sub-soil of Khulna region, the south-west part of Bangladesh, contains fine-grained soils with significant amount of organic content which often creates problem to the geotechnical engineers to select an economic foundation for the construction of massive structures due to low shear strength and high compressibility (Alamgir et al. 2001). Recently the performance of geotextile reinforced footing, sand compaction piles, stone columns and granular piles have also been studied in this region at field level and acceptable results were reported (Haque, 2000; Zaher, 2000; Alamgir and Zaher, 2001; Haque et al., 2001, Sobhan, 2001 and Hossain, 2007). As the performance of ground improvement techniques depends significantly on the properties of soil, it is necessary to know the effect of ground improvement techniques on the organic soil collected from Khuna region. To this endeavor, laboratory investigation was conducted adopting four different ground improvement techniques on the same reconstituted soft organic grounds. The considered ground improvement techniques are compacted sand column, compacted sand bed with and without geotextile, and compacted sand bed with geotextile in conjunction of compacted sand column.

3.3 Work Plan

The methodology of this research was accomplished by the work plan as described below and also shown in Figure 3.1.

(i) Collection of undisturbed samples for characterization and disturbed organic soil from a selected Khulna region to prepare reconstituted organic grounds for application of different ground improvement techniques.

(ii) Determination of physical, index and engineering properties of collected organic soil used for the construction of reconstituted organic grounds, using conventional laboratory test methods.

(iii) Adding water equal to 1.25 times of liquid limits to the organic soil and mixed thoroughly by mixture machine to obtain homogeneous slurry.

(iv) Pouring organic soil slurry on the constructed sand bed at the bottom of a cylindrical tank of 0.55m diameter and 1m height and kept the slurry under pre-designated pre-consolidation pressure to prepare the test grounds.

(v) After the completion of test ground preparation, samples were collected to determine properties of the grounds by routine tests.

(vi) Then prepared test grounds were treated by four different ways: (a) installation of compacted sand column in the test ground, (b) placing of compacted sand bed over the reconstituted layer, (c) placing of compacted sand bed with geotextiles over the reconstituted layer, and (d) placing of compacted sand bed with geotextiles over the ground in conjunction of compacted sand column.

(vii) The settlement response of untreated and treated grounds was measured by footing load test and determined the improvement of load carrying capacities of the grounds.

(viii) Finally the bearing capacity of treated and untreated grounds was determined by available equations and compared predicted and measured bearing capacity of the footing to justify the effectiveness of different ground improvement techniques applied in the soft organic soil.

3.4 Collection of Organic Soil for the Preparation of Reconstituted grounds

The reconstituted grounds were prepared by the organic soil collected from Mohersharpasa, Khan Jahan Ali thana, Khulna, Bangladesh. This site is about 2 km away from KUET campus as shown in Figure 3.2. In the present study undisturbed and disturbed organic soil was collected from the same location at a depth of about 10 feet from the existing ground surface. The organic soil was collected by the block sampling method. Undisturbed organic samples were taken and transported to the Geotechnical Engineering Laboratory of the Department of Civil Engineering, Khulna University of Engineering & Technology (KUET), Khulna for characterization using proper sample handing and preservation techniques. The disturbed organic soil was used for the preparation of reconstituted organic grounds.

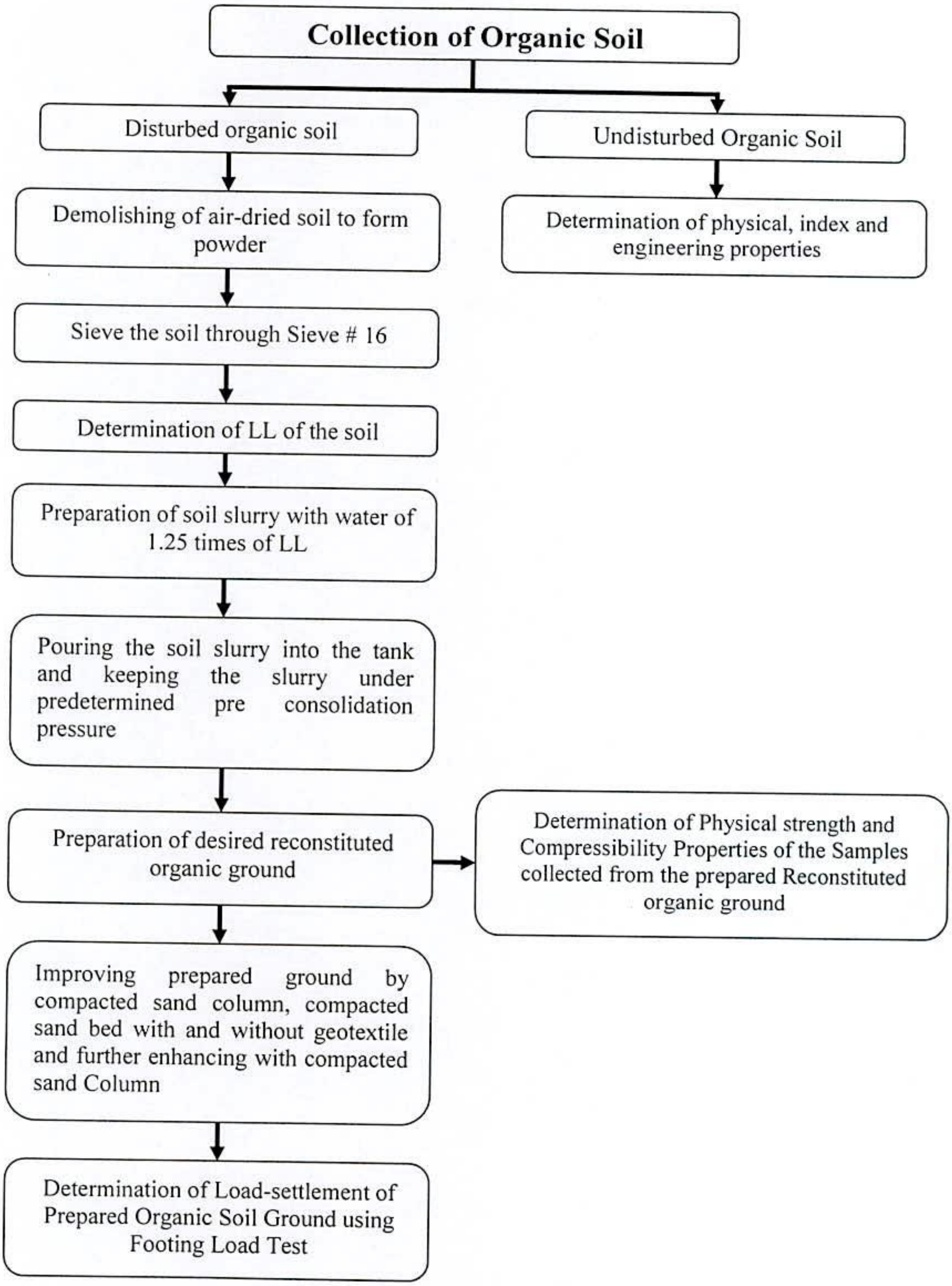


Fig. 3.1 Flow chart of the laboratory investigation.

3.4.1 Preservation of soil samples for the testing programme in the laboratory

In the laboratory, the identification of samples, samples removal, storage and sealing were done properly. However, the inherent problems associated with the handling of soft soil reasonable degree of sample disturbance were observed. This disturbance might be significant and laboratory results may differ markedly from its in-situ behavior. Sample disturbance is often regarded as a significant problem because it is thought to prevent acquisition of realistic soil parameters. To avoid this sample disturbance the following guidelines were considered carefully in the laboratory during this research.

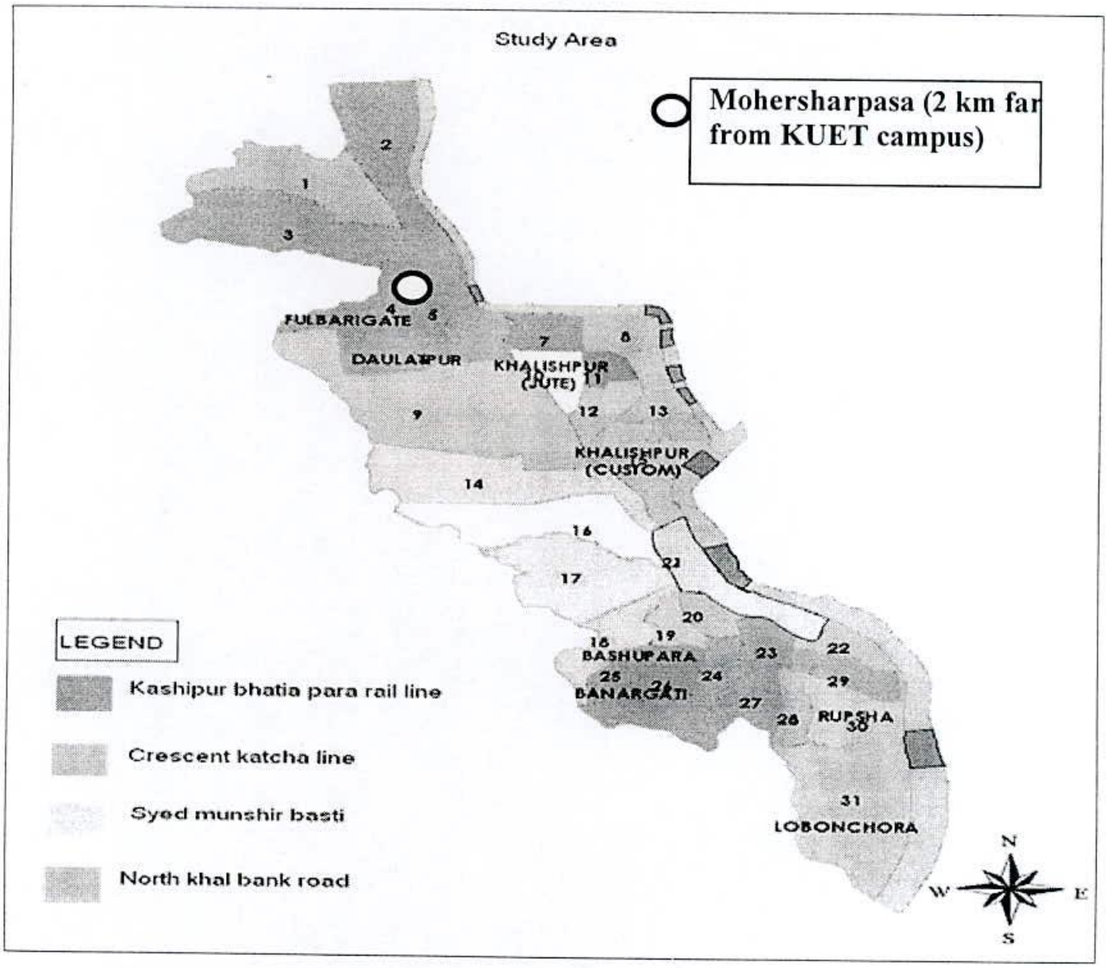


Fig. 3.2 The location for the collection of organic soil.

3.4.2 Storage of organic soil

The collected bulk amount of disturbed soil for the preparation of reconstituted grounds was covered with polythene and stored into the plastic container in the laboratory at room temperature around 25°C. The undisturbed samples were conveniently stored in desiccators.

3.5 Properties of the Used Organic Soil and Sand

The physical, index and engineering properties were determined in order to classify the collected organic soil. The undisturbed samples were used for laboratory test to determine the properties and the collected disturbed soil was mainly used for the preparation of reconstituted organic grounds. These samples were then air-dried and the soil lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particles. The following conventional tests were performed using standard procedures to determine the physical, index and engineering properties of soil.

(i) Moisture content
(ii) Specific gravity
(iii) Organic content
(iv) Atterberg limits
(v) Grain size distribution
(vi) Unconfined compression test
(vii) Consolidation test
(viii) Compaction test

3.5.1 Determination of physical properties

To observe the physical state of collected organic soil, the following series of ASTM (2004) methods were adopted in the Geotechnical Engineering Laboratory at KUET.

(i) Determination of moisture content

The natural moisture content of collected organic soil and water content of reconstituted soils were determined by electric drying oven method and the testing standard is ASTM D2974.

(ii) Determination of specific gravity

The specific gravity of the collected and reconstituted organic soil was determined based on Pycnometer Method following the testing standard ASTM D854.

(iii) Determination of organic content

The "Loss and Ignition" method was used for the determination of organic content of those materials identified as peat, organic mucks and soil containing relatively vegetable matter of fresh plant materials. In this study, for the determination of organic contents "Loss and Ignition" method was used the following standard ASTM D2974. Burning the soil samples in the oven for 5.5 hours at temperature 550°C, the organic content was determined in the laboratory based on the loss of weight of the oven-dried (at 105°C) sample.

3.5.2 Determination of index properties

(i) The liquid limits and plastic limits of collected organic soil were determined based on Atterberg Limits test following the testing standard ASTM D4318.
(ii) The bulk unit weight of collected organic soil was determined based on weight-volume method following the testing standard ASTM D2967.
(iii) The percentages of different sized soil particle of collected organic soil were determined based on grain size analysis following the testing standard ASTM D422.

3.5.3 Determination of strength properties

The strength properties of collected organic soil and sample of reconstituted organic grounds were determined in the laboratory by performing unconfined compression test following the testing standard ASTM D2166.

3.5.4 Determination of compressibility characteristics

The compressibility characteristics of collected organic soil and reconstituted organic soils were determined in the laboratory by performing K_o-consolidation test following the testing standard ASTM D2435.

3.5.5 Determination of compaction properties

The maximum dry density and optimum moisture content of sand were determined in the laboratory by standard proctor test (ASTM D558). In the study local sand was used as the granular materials for the construction of compacted sand column and compacted sand bed.

Table 3.1 Index properties of the collected organic soil

w (%)	**G_s**	**LL (%)**	**PL (%)**	**OC (%)**	**γ kN/m^3**	**Percentages of Constituted Soil Particles (ASTM method)**			**USCS Symbol**
						4.75-.076 mm	**0.076-.002 mm**	**<.002 mm**	
277	2.02	350	208	51	10.89	2	61	37	OH

Table 3.2 Engineering properties of collected organic soil

Unconfined Compressive Strength (kPa)	**Compressibility properties**	
	compression index, C_c	**initial void ratio, e_o**
54.5	2.43	5.77

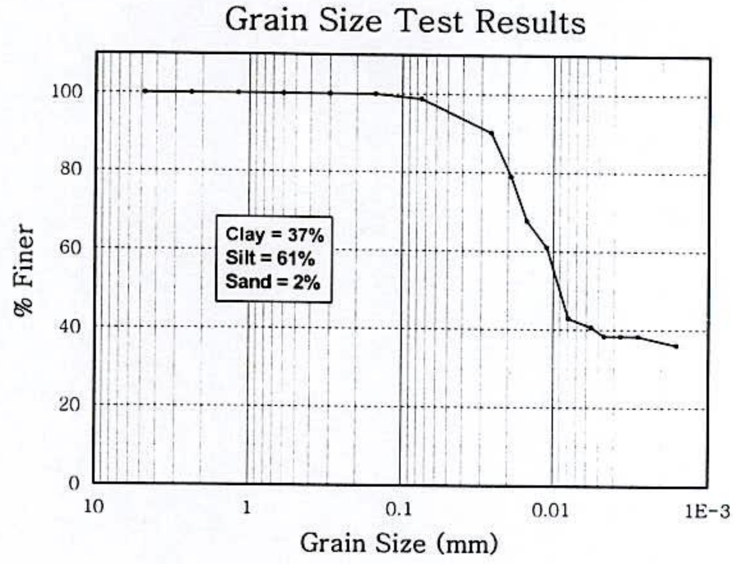


Fig. 3.3 Grain size distribution diagram of collected organic soil.

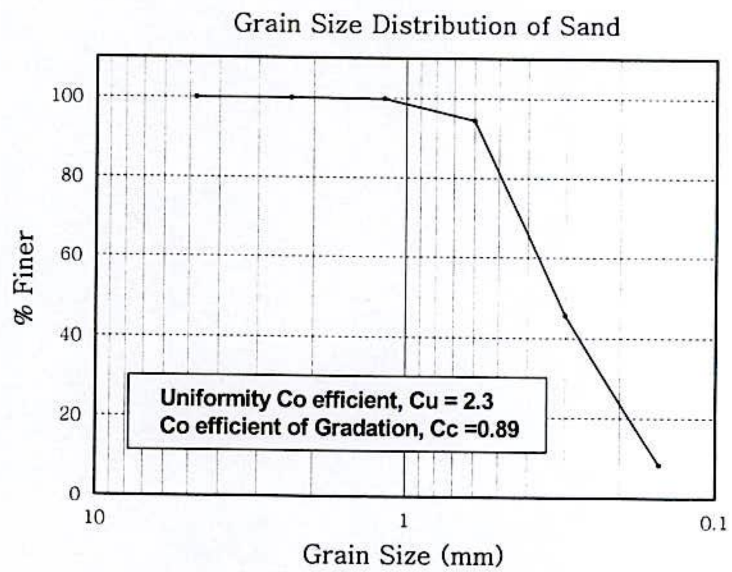


Fig. 3.4 Grain size distribution of sand.

Table 3.3 Properties of the sand used for the ground improvement

Properties	Value
Coefficient of uniformity	2.3
Coefficient of gradation	0.89
Optimum Moisture Content (%)	14.42
Maximum Dry Density (kN/m^3)	18.33
Cohesion (kPa)	7.64
Angle of internal friction (°)	34

3.6 Preparation of Reconstituted Organic Grounds

The 'Unit Cell' concept (Barksdale and Bachus, 1983) was used for the preparation of experimental grounds representing problematic soft ground condition. The experimental grounds were constructed in cylindrical tanks of 0.55 m diameter and 1 m height as shown in Figure 3.5. The tank is made of tin which can be considered as well enough to resist the lateral budging during preparation of test ground and footing load test. The bottom of the tank was made sufficiently porous to ensure free draining of water during the preparation of reconstituted organic soils.

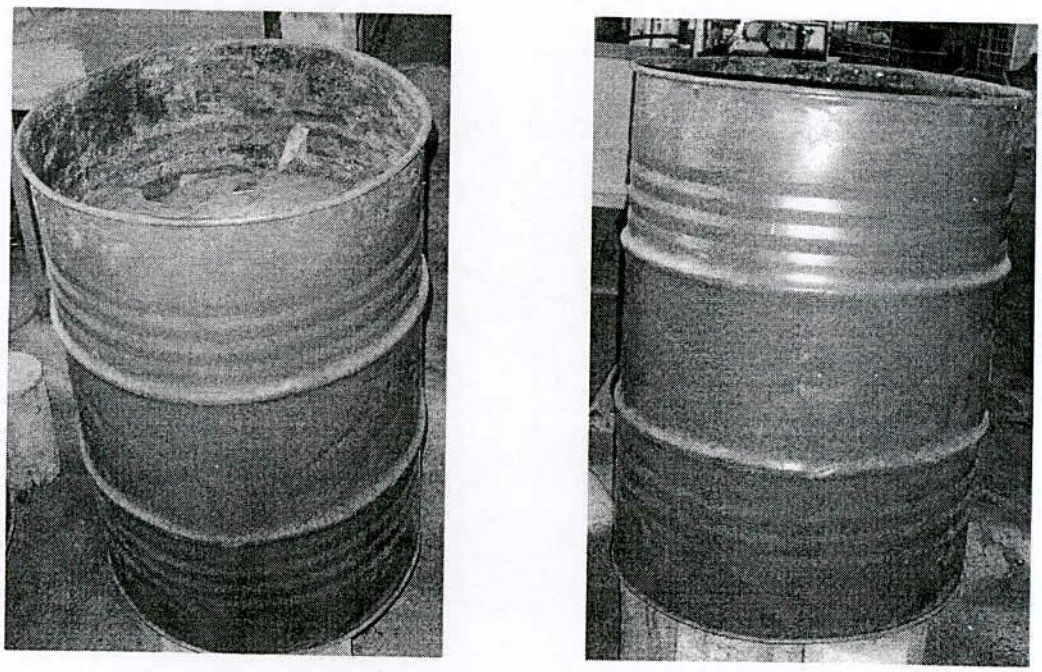


Fig. 3.5 Cylindrical tank for preparation of experimental grounds.

3.6.1 Preparation of reconstituted organic layer

The reconstituted organic grounds were prepared on the compacted sand layer of 175mm thick as shown in Figures 3.10 to 3.14. To prepare reconstituted organic grounds, firstly organic soil was broken into powder and screened with sieve # 16 to obtain the organic soil that could be easily formed into slurry for the preparation of reconstituted organic grounds. Then soil powder was mixed with water, 1.25 times of liquid limit, thoroughly in the mixture machine to get homogeneous slurry. The mixture machine used for this work has capacity of 10 litres and planetary and spindle speeds are 50-150 rpm and 180-540 rpm, respectively. The used mixture machine, CONTROL Mod. 16-B0072, /Y/Z, is shown in Figure 3.6.

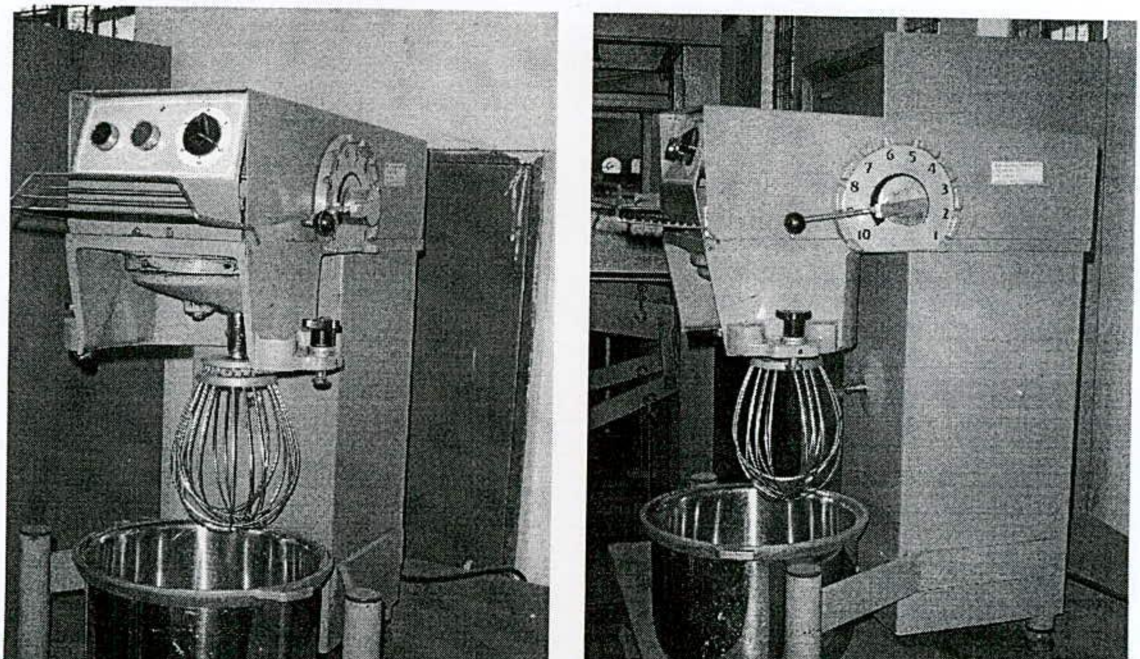


Fig. 3.6 Mixture machine.

The organic soil slurry was poured into the designed cylindrical tank and a RCC slab as shown in Fig. 3.15 was placed on the organic soil slurry. A hydraulic jack was then placed on the RCC slab to apply pre-selected pre-consolidation i.e. 70kN/m^2 for the preparation of reconstituted organic grounds. As the soil slurry was very weak to carry the predetermined pre-consolidation pressure fully from the very beginning, the pressure was applied from very small value, e.g. 10 kPa and the dissipation of pore water was allowed through the bottom of sand layer. When the generated water pressure was dissipated completely, the load indicator on the dial of hydraulic jack was reduced to zero. Again same amount of load was applied manually by hydraulic jack and waited till the full dissipation of generated pore water pressure. When the soil slurry was capable of carrying the applied load, the load was increased by 10 kPa and maintained same procedure of the previous loading case to increase load carrying capacity of the soil slurry. After required repetitions and increment of applied loading (10kPa), the designated pre-consolidation pressure was applied when it appeared that slurry was well stable to bear the full pre-consolidation pressure (70kPa). To achieve this level of ground condition for the pre-determined pre-consolidation pressure of 70 kPa, around 30 days are required to end of full dissipation of generated pore water pressure. The resultant reconstituted ground was considered as the prepared ground for application of ground improvement techniques. The soil samples were then collected from the prepared ground by shellby tube to determine its strength and compressibility properties as shown in Tables 3.5.

3.6.2 Placement of bottom sand layer

A bottom sand layer of 175 mm thickness was provided at the bottom of every reconstituted ground in this study as shown in Figures 3.10 to 3.14. The sand layer was normally compacted to function as a base layer of reasonably good strength and free-draining properties. To avoid clogging of sand bed, geotextile was used in the interface of sand bed and organic soil slurry.

3.6.3 Preparation of compacted sand bed over the reconstituted organic ground

A compacted sand bed was provided on the reconstituted organic soil ground as a part of implementation soil improvement technique as shown in Figures 3.12 to 3.14. Before using sand in compacted sand bed, the properties of sand were determined in the laboratory. The Optimum Moisture Content and Maximum Dry Density of the sand was found 14.42 % and 18.33 kN/m^3, respectively, determined by Standard Proctor Test. The sand was mixed with water of optimum moisture content before placing on the reconstituted organic grounds. The sand bed was constructed by layer of 25 mm thickness and compaction was done manually by using brick as hammer. Finally the degree of compaction of the provided sand compacted bed was determined as presented in Table 3.5. The compacted sand was further reinforced with geotextile.

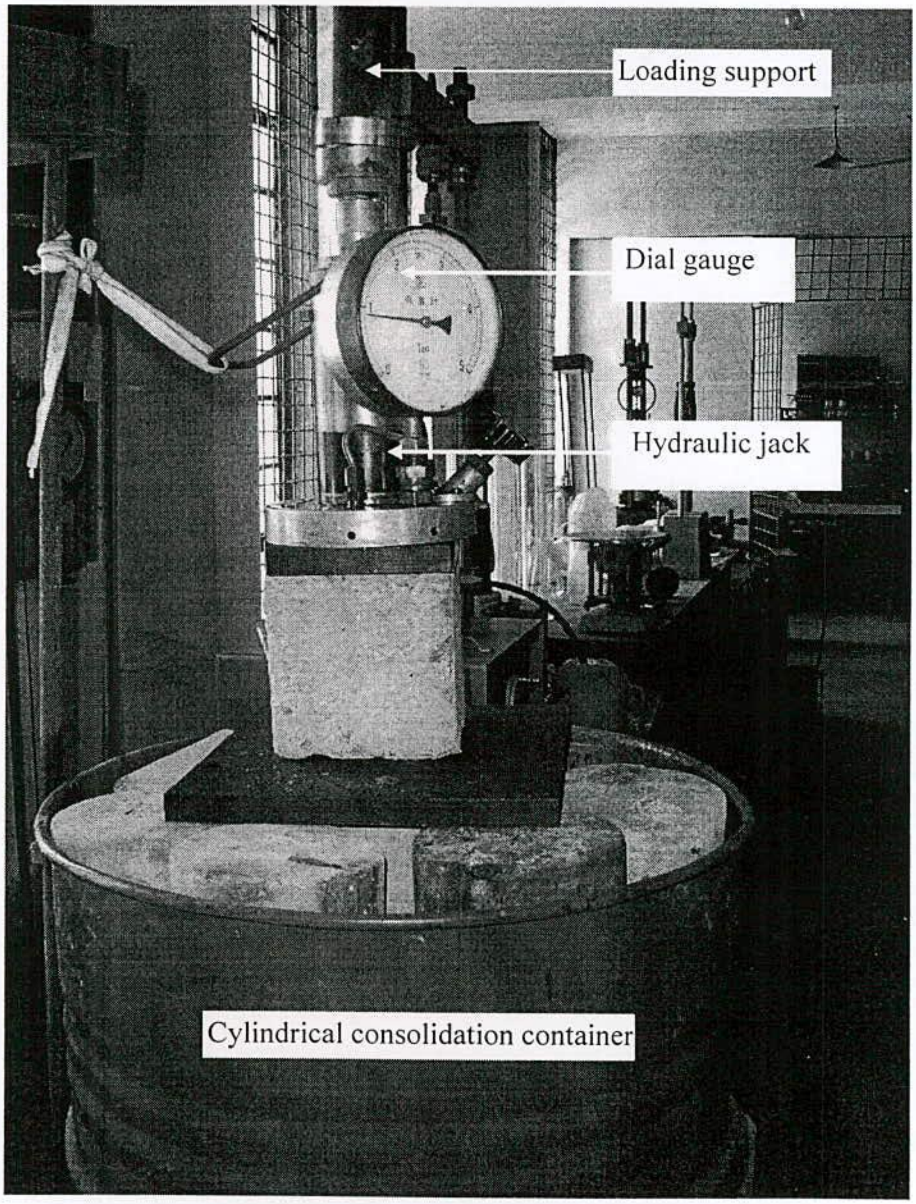


Fig. 3.7 Loading arrangement for the preparation of reconstitute organic ground.

3.6.4 Installation of sand column in the reconstituted organic ground

The sand column was installed in the reconstituted organic ground at the centre of the cylindrical test mold by replacement method as shown in Figure 3.11. Local sand was used for the construction of sand column.

A hollow steel pipe of 75 mm diameter was inserted into the test ground till the final depth of reconstituted clay media by hammering. After full insertion, the pipe was pulled out with the soil stacked inside; as a result a cylindrical hole was formed for the construction of sand column. The sand column was then constructed by pouring sand into the hole in layers of 150 mm thickness. Each layer was compacted by using hammer of 50 mm diameter and 5.5 lb weight with 25 number of blow per layer. The height of free drop of hammer was maintained as 300 mm to produce the compaction energy as 6.05×10^6 m-kg/m^3. Before pouring of sand into the hole, it was mixed with water to ensure the level of optimum moisture content. It was observed that such compaction energy and the method of construction leads to have a reasonably compacted sand column.

3.6.5 Placement of geotextile

Geotextile was used to reinforce the compacted sand on the soft organic grounds to impart stiffness and increase bearing capacity as shown in Figure 3.8. To serve the purpose, geotextile should place on the soft ground in such a way that it must remain in tension after buried by sand bed. So during the placement of geotextile, extra necessary care was taken.

Table 3.4 Properties of Geotextile used in this study

Manufacturer Product	**: Polyfelt TS 650**
Type of Geotextile	: Lightweight nonwoven geotextile
Composition	: Needle-punched nonwoven polypropylene
Mass (g/m^2)	: 235
Thickness (@ 2 kPa)	: 2.3
Grab Tensile Strength (N)	: 755

In this study the Geotextile was used as reinforcement whose properties are presented in Table 3.4. Prior to the placement of geotextile, trench of about 30 mm deep and 25 mm wide was cut at the periphery of the ground/tank as shown in Figure 3.9. Moreover, geotextile would be placed on the soft ground in such a way that it remains in tension. After placement, geotextile was buried by designated sand. The surrounding trench was acted as an anchor and kept the geotextile in tension after construction.

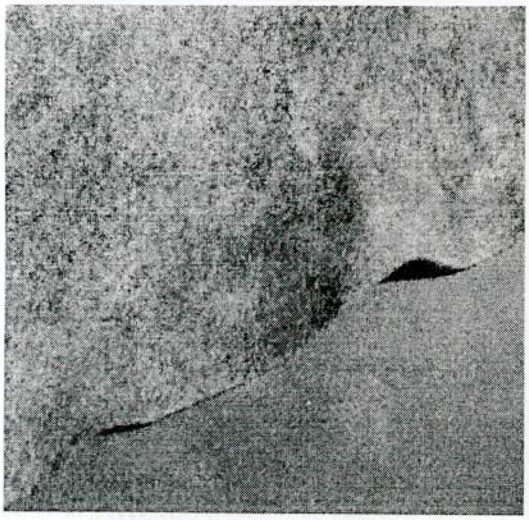


Fig. 3.8 Sample of Geotextile used in this study.

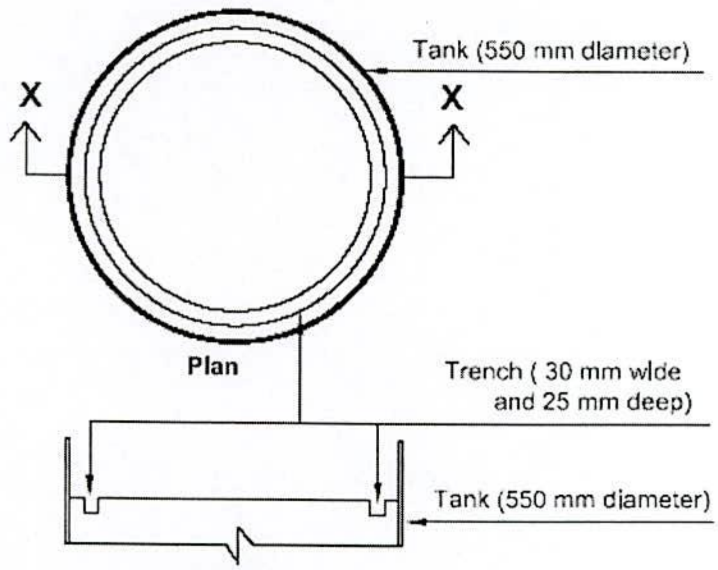


Fig. 3.9 Placement mechanism of geotextile on soft ground.

3.6.6 Summary of Test Grounds

The prepared test grounds both the untreated and treated are shown in the Figures 3.10 to 3.14. The placement of footing and the loading conditions are also shown in this figure. The properties both the reconstituted organic soils and used are also presented in the Table 3.5. The test grounds are designated here as G-1 to G-5 as listed below for the convenience of description.

G-1 : Untreated reconstituted organic ground

G-2 : Reconstituted organic ground treated by Compacted Sand Column

G-3 : Reconstituted organic ground treated by Compaction Sand Bed

G-4 : Reconstituted organic ground treated by Compaction Sand Bed with Geotextitle

G-5 : Reconstituted organic ground treated by Compaction Sand Bed with Geotextitle in conjunction of Compacted Sand Column

For each case, two similar reconstituted grounds were prepared, which designated as ground 1 and 2, e.g. G-1.1 and G-1.2.

Table 3.5 Properties of the test grounds

Properties	Parameters	G-1		G-2		G-3		G-4		G-5	
		G-1.1	G-1.2	G-2.1	G-2.2	G-3.1	G-3.2	G-4.1	G-4.2	G-5.1	G-5.2
Reconstituted Organic Grounds	Water Content (%)	161.5	155	149	153	136	136	153	144	159	145
	Organic content (%)	51	61	67	73	46	45	69	56	66	57
	Reconstituted Organic Ground Thickness (mm)	290	405	365	450	345	480	480	375	520	420
	Unconfined Compressive Strength (kPa)	51.7	45	45	60	39	51	47	53	43	69
	compression index Cc	1.37	1.03	1.3	1.2	1.7	1.18	1.13	0.93	1.24	1.2
	initial void ratio, eo	3.71	3.39	3.56	3.78	3.78	3.06	3.78	3.5	3	3.5
Sand	Bulk Density of Sand Bed (kN/m3)	--	--	--	--	14.72	14.72	14.72	14.72	14.72	14.72
	Wet Density of Sand Bed (kN/m3)	--	--	--	--	19.33	19.33	19.33	19.33	19.33	19.33
	Dry Density of Sand Bed (kN/m3)	--	--	--	--	17.45	17.45	17.45	17.45	17.45	17.45
	Moisture Content (%)	--	--	10.5	11	10.5	10.5	10.7	8.8	10.6	10
	Degree of Compaction (%)	--	--	--	--	95	95	95	97	95	96

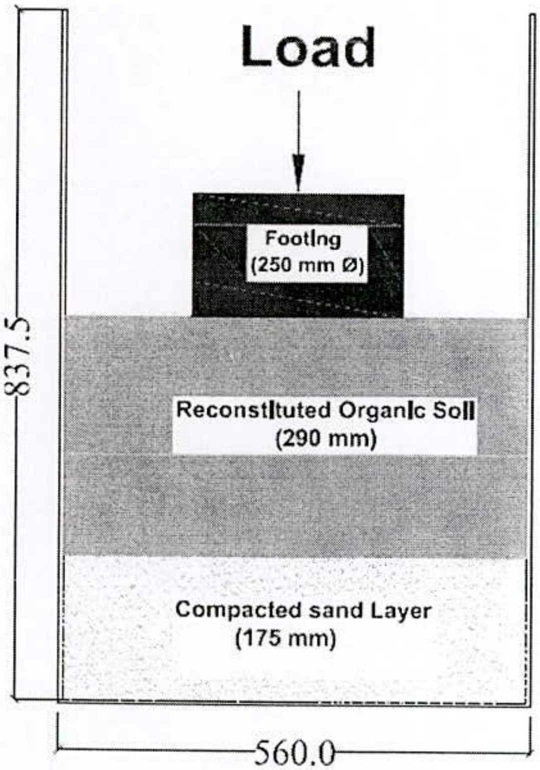


Fig. 3.10 Untreated ground (G-1.1)

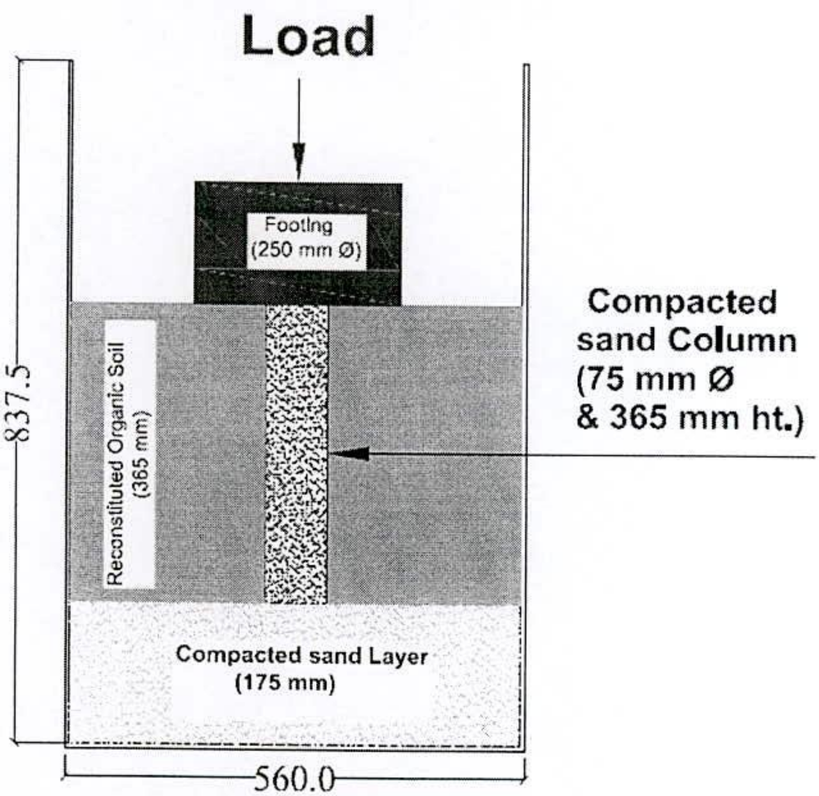


Fig. 3.11 Ground treated with compacted sand column (G-2.1)

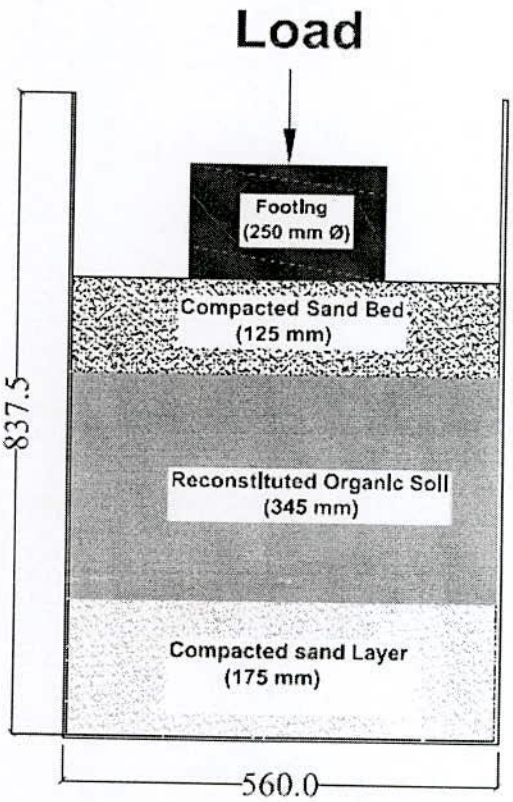


Fig. 3.12 Ground treated with compacted sand bed (G-3.1)

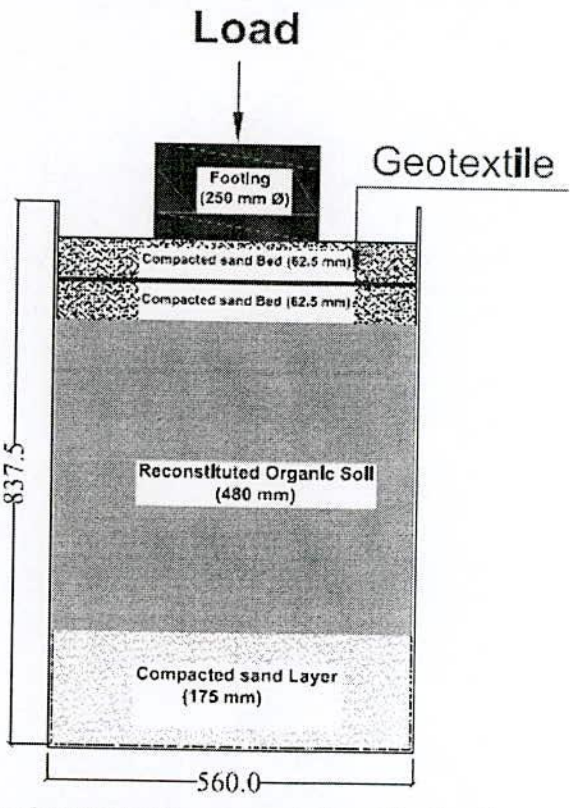


Fig. 3.13 Ground treated with compacted sand bed and geotextile (G-4.1)

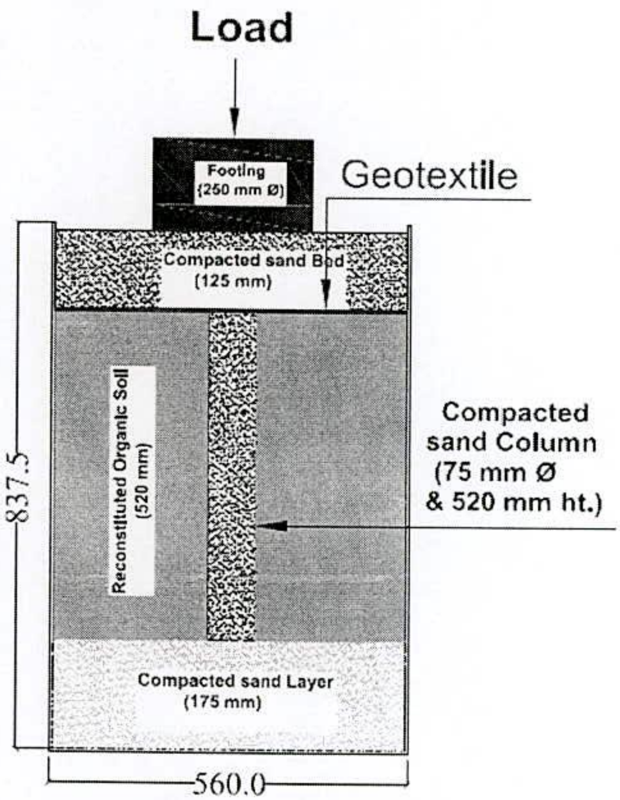


Fig. 3.14 Ground treated with compacted sand bed- geotextile and sand column (G-5.1)

3.7 Footing Load Test

The performance of improved grounds was determined conducting footing load test. The load-settlement behaviour of experimental grounds was evaluated based on the plate load test concept. This test involving static loading of a concrete footing as used to determine: (i) ultimate and safe bearing capacities, (ii) bearing pressure for an allowable settlement, (iii) probable settlement of model foundation, (iv) undrained shear strength. The footing load test procedure can be found in ASTM D-1194, D-1195, and D-1196

3.7.1 RCC footing and slab

A circular RCC footing and RCC slab was used in the experiment to apply load to the experimental grounds. The diameter and thickness of the footing were 250 mm and 150 mm, respectively. The footing was cast in the laboratory with Portland cement and the ratio of the concrete mixing was 1:1.5:3. M. S. steel of 10 mm diameter was used as reinforcement at the spacing of 100 mm centre to centre. After casting, the reinforced concrete footing, as shown in Figures 3.15 and 3.16, was kept under curing for 28 days for proper strengthening. Then the footing was considered suitable for using as a base for load application. The compressive strength of the footing was found as more than 3000 psi. The weight of the footing was measured as 5.5 kg.

A slab of 550 mm diameter and 100 mm thick, was cast with cement concrete of ratio 1:1.5:3 and reinforced with 10 mm dia. bar @ 125 mm centre and centre. The RCC slab was used as base on the reconstituted organic grounds to apply pre-consolidation for the preparation of experimental grounds. The compressive strength of the slab was 3000 psi. There were two handles on the slab to ease movement as shown in Figure 3.15.

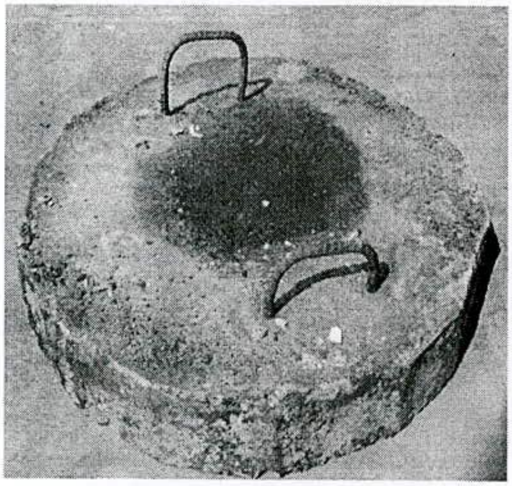


Fig. 3.15 RCC slab

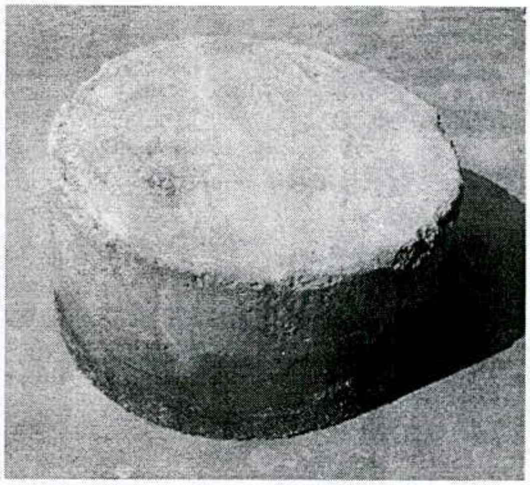


Fig. 3.16 RCC footing

3.7.2 Hydraulic jack

Hydraulic jack was used to apply pre-set pressure to the reconstituted organic slurry for the preparation of reconstituted grounds and also to apply load in the footing during load test. The capacity of hydraulic jack used for this laboratory investigation is 5 tons. The identity mentioned in the jack is MARUTO, Testing Machine Company, Made in Japan, ID: S05L.12.5. The identity mentioned in the dial is NKS, NAGANO, 440, ID: 2393504 shown in Figure 3.17.

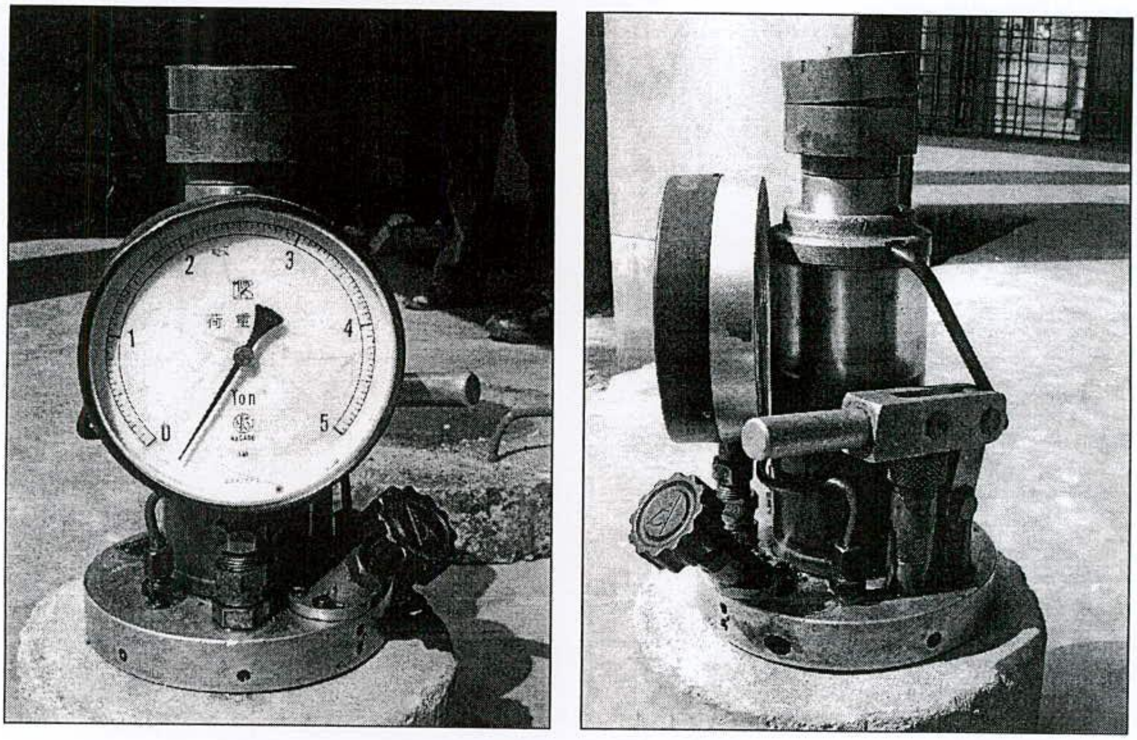


Fig. 3.17 Hydraulic Jack

3.7.3 Dial gauge

In footing load test, the settlements with respect to load were measured by deformation dial gauge as shown in Fig. 3.18. The dial gauge used for measuring deformation has the capacity of 25 mm. The identity mentioned in the dial gauge is Mitutoyo, No. 2050-08, made in Japan. The dial gauge was set on a platform to ease the setting of dial and get accurate measurement of settlement. The accuracy of the dial gauge used in the work is 0.001 mm.

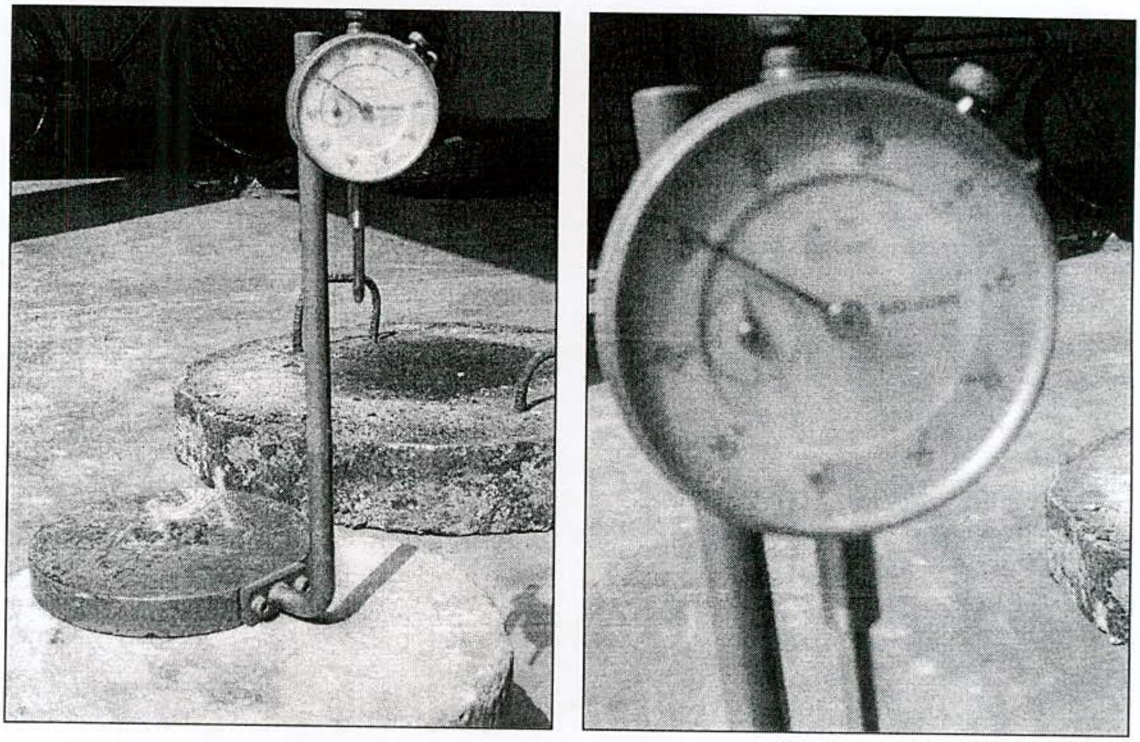


Fig. 3.18 Dial gauge

3.7.4 Experimental set-up

A circular concrete block of 250 mm diameter and 150mm thickness was used as footing to apply load to the experimental grounds. In this study circular shape was preferable as loading base to eliminate secondary dimensional effects, such as corner losses caused by yielding of soil at such points of high concentration of stress. The circular footing was placed at the centre of the experimental grounds. Then a hydraulic jack was placed on the footing to apply load to the experimental grounds. The detailed set up of loading is shown in Figure 3.19.

The settlement of the footing was measured by at least two dial gauges with reference to an independent datum. Two dial gages were set at positions having an included angle of 180°. As the area within the test pit around the footing likely to be disturbed, the datum was fixed at the ground surface.

3.7.5 Load-settlement test

The footing load test was conducted on the untreated and treated grounds by the following steps:

(i) A hydraulic jack and two deformation dial gauges were positioned as described earlier. The load was then applied by hydraulic jack and the settlement was recorded by the dial gages.

(ii) The load was applied to the soil in cumulative equal increments of not more than 98.1 kN/m^2 or not more than one-fifth of the estimated allowable bearing pressure. On the clay soil, keep the load constant until the settlement, as indicated by the pressure/settlement curve, reaches about 75% of the probable ultimate value at that stage, or until 24 hours as the method described in IS: 1888-1971.

(iii) For an applied load, reading for settlement was taken from deformation dial gauge at the elapsed time of 0, 1, 2, 4, 8, 16, 30, 60 minute measured from the starting of loading. If the settlement was more than 0.25mm per hour, then the reading for the settlement was taken at the time of 120 minute.

(iv) Then loading was increased to next and readings were taken at aforementioned durations (iii). The test should be run to the maximum applied load i.e. 1.5 times of the estimated ultimate load or 3 times the proposed allowable bearing pressure or to more than 25 mm settlement of footing.

(v) With the help of the load and settlement data, load-settlement curve for the grounds would be drawn for five conditions.

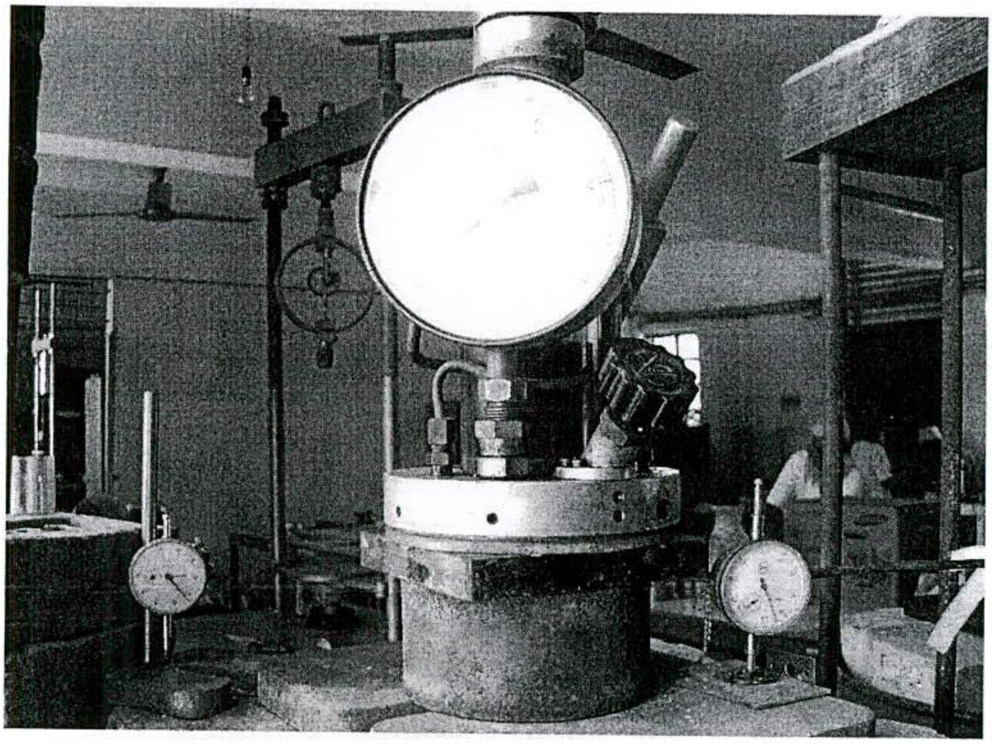


Fig. 3.19 Detailed arrangement of footing load test

3.7.6 Method of determining bearing capacity from test data

A "net load" versus "final settlement" curve was plotted on linear scale. An excessive preliminary settlement might result under the first load due to improper seating. A correction was applied in such cases, by extending the straight portion of the curve, above the point of slope change, backwards to intersect the settlement axis and gave the corrected zero settlement point. If the test had been carried to failure, the curves for cohesive soils and for dense cohesionless soils usually indicate definite yield points. Partially cohesive, soils and loose to medium cohesionless soils did not normally develop definite yield points. A log-log plot is often able to give the failure point even for soils for which a linear plot is not helpful. Caution is, however, necessary in interpreting a log-log plot as in a few cases the failure may he indicated even within an insignificant range of initial settlement, say 1 to 2 mm, when the plate is still in the process of proper seating that is neglected.

The load intensity and settlement observation of the footing load test is plotted in the similar way as shown in Fig. 3.20 (a). Curve I corresponds to general shear failure and II corresponds to the local shear failure. Curve III is a typical one usually obtained for dense cohesionless soils which does not show any marked sign of shear failure under the loading intensities of the test. IS: 1888-1962 recommends a log-log plot giving two straight lines, the intersection of which is considered the yield value of the soil as shown in Fig. 3.20 (b). When a load settlement curve does not indicate any marked breaking point, failure may alternatively be assumed corresponding to a settlement equal to the settlement of 25 mm as shown in Fig. 3.20 (a).

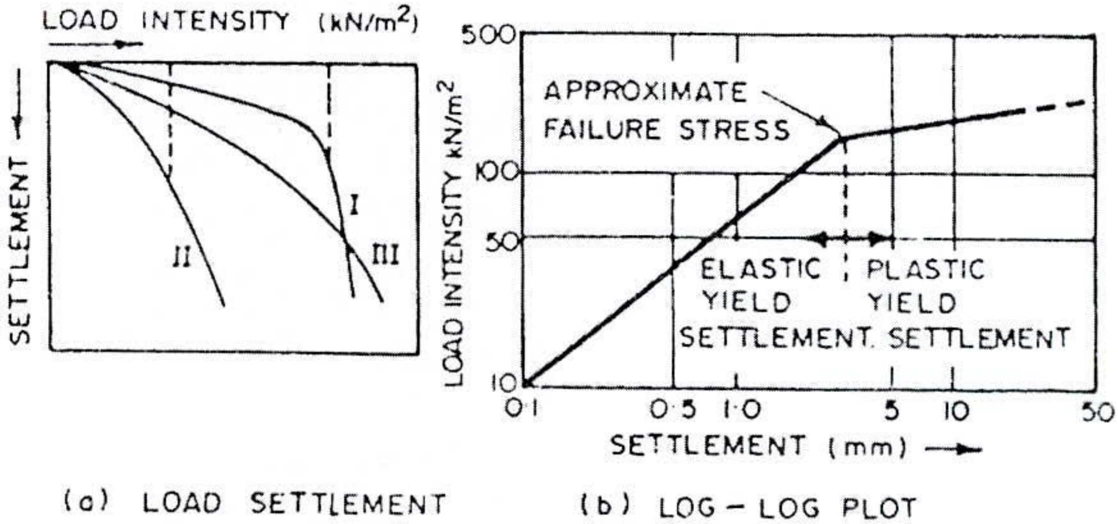


Fig.3.20 Determination of bearing capacity from load intensity and settlement.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 General

The results obtained from the laboratory investigation are presented and discussed in this chapter. The results obtained from footing load tests on the treated reconstituted organic grounds for four different ground improvement conditions are compared with that of untreated reconstituted organic ground.

4.2 Load-settlement Results and Failure Pattern of Grounds

The load-settlement behaviour of the untreated and treated reconstituted organic grounds was determined by footing load test method. The load-settlement curves of the experimental grounds are described in following articles.

4.2.1 Load-settlement behaviour and failure pattern of untreated grounds

The load test through individual circular footing was performed on prepared grounds to find out settlement responses. From Figures 4.1 to 4.3, the rate of settlement of organic grounds is found very high. After reaching the settlement of 12 mm, the rate of settlement due to loading has increased sharply. The point of failure can be identified easily by the settlement rate method. The load-settlement curve obtained for the footing on organic ground indicates that the organic ground failed when the amount of settlement has reached to 12 mm. The failure pattern of untreated ground has shown in Figure 4.17 and it is clear that the punching shear failure occurred.

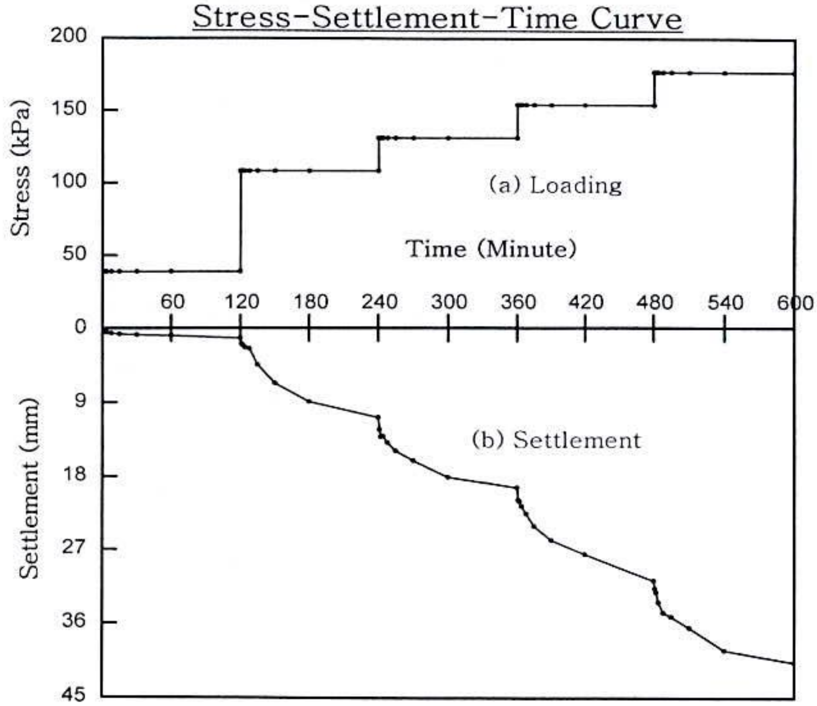


Fig. 4.1 Untreated ground (G-1.1)

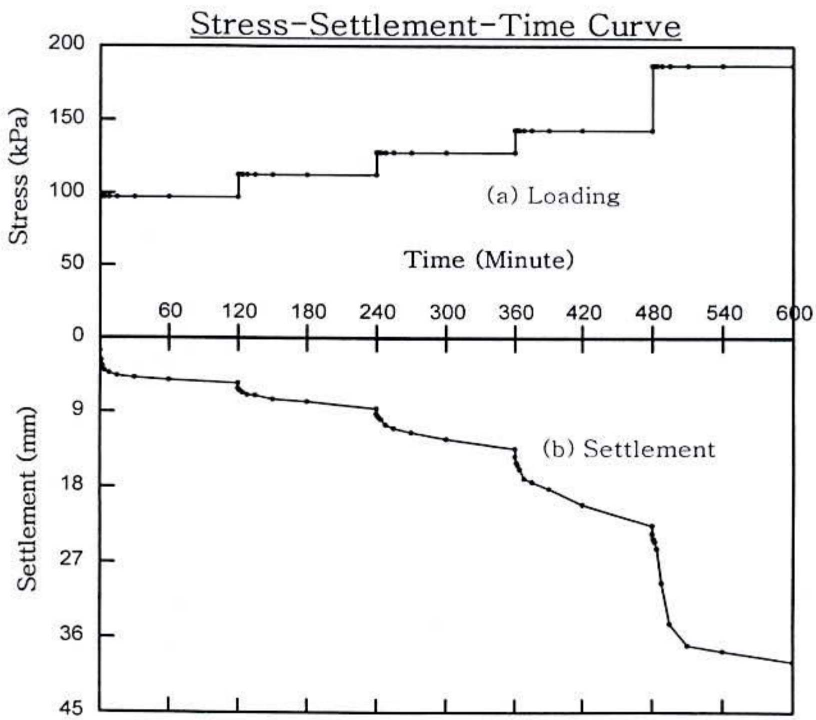


Fig. 4.2 Untreated ground (G-1.2)

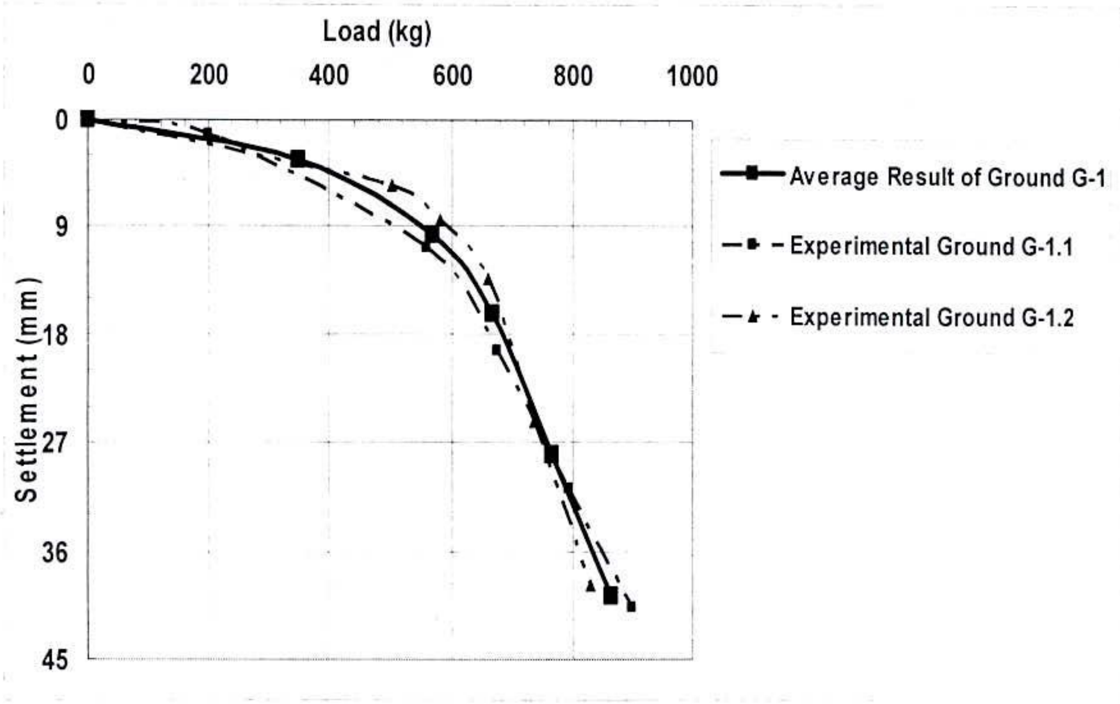


Fig. 4.3 Untreated ground (G-1)

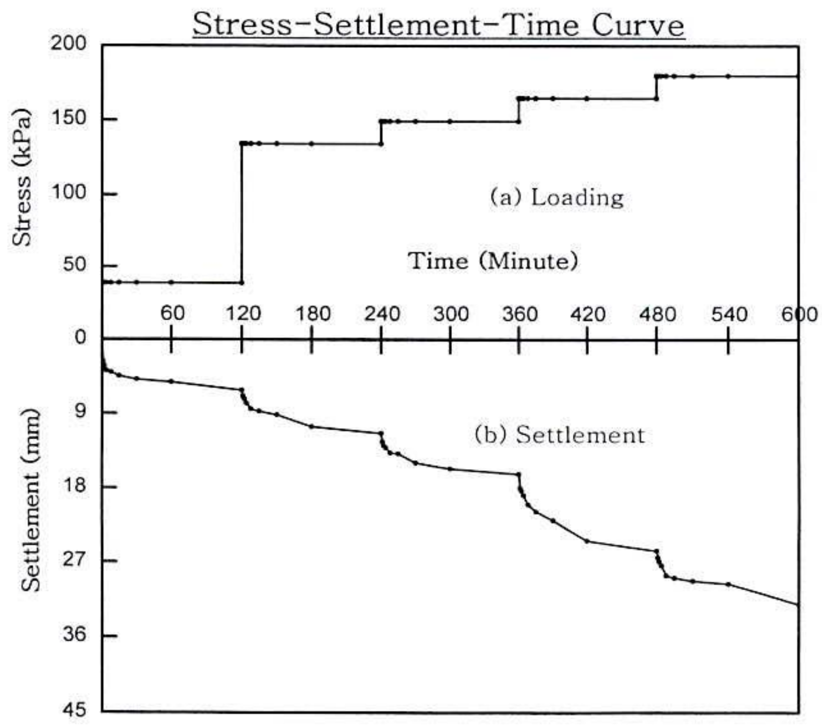


Fig. 4.4 Ground treated with compacted sand column (G-2.1)

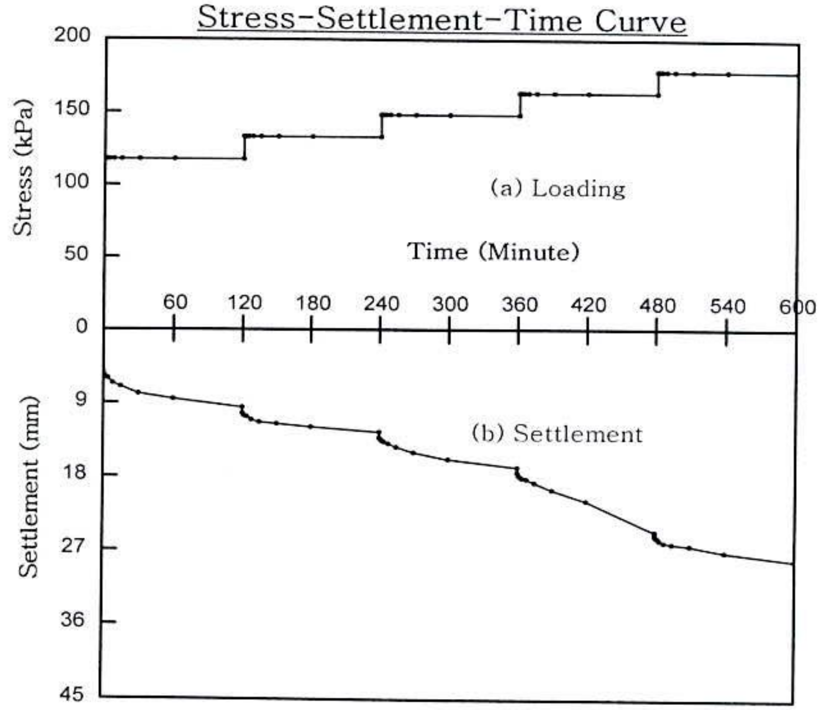


Fig. 4.5 Ground treated with sand column (G-2.2)

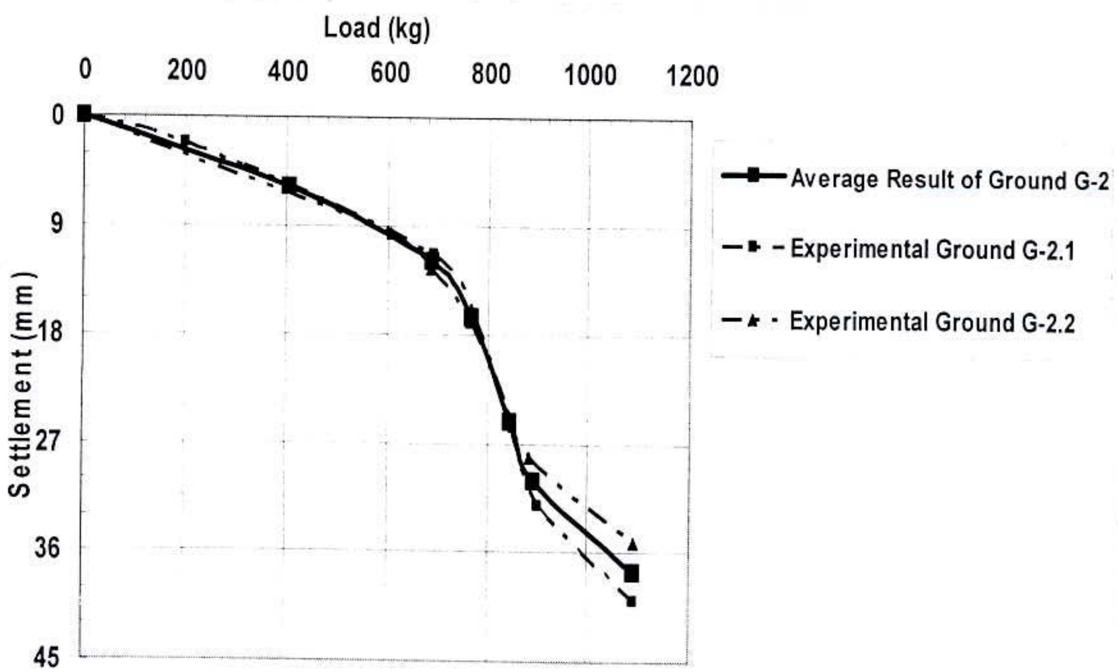


Fig. 4.6 Ground treated with sand column (G-2)

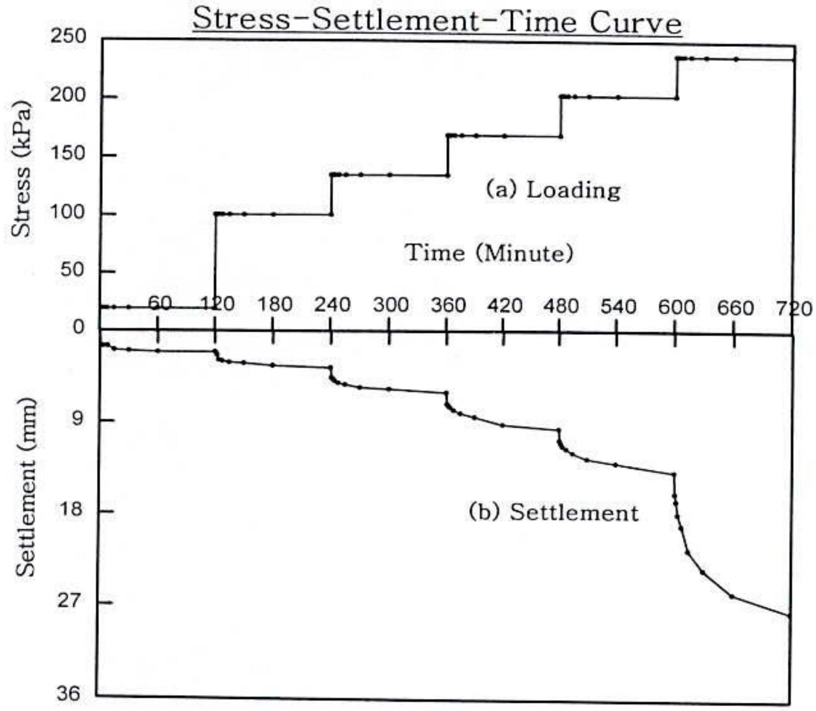


Fig. 4.7 Ground treated with compacted sand bed (G-3.1)

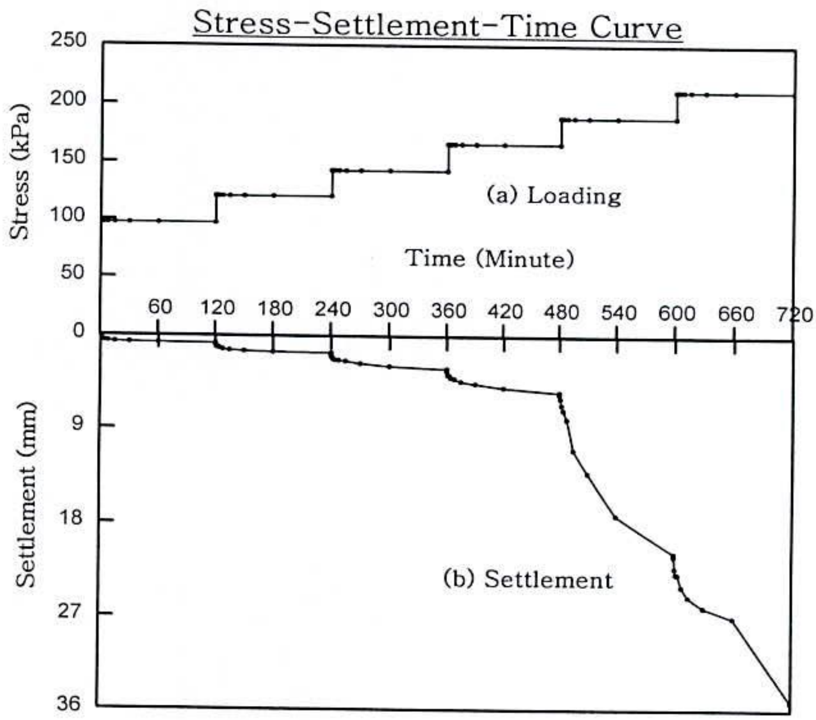


Fig. 4.8 Ground treated with compacted sand bed (G-3.2)

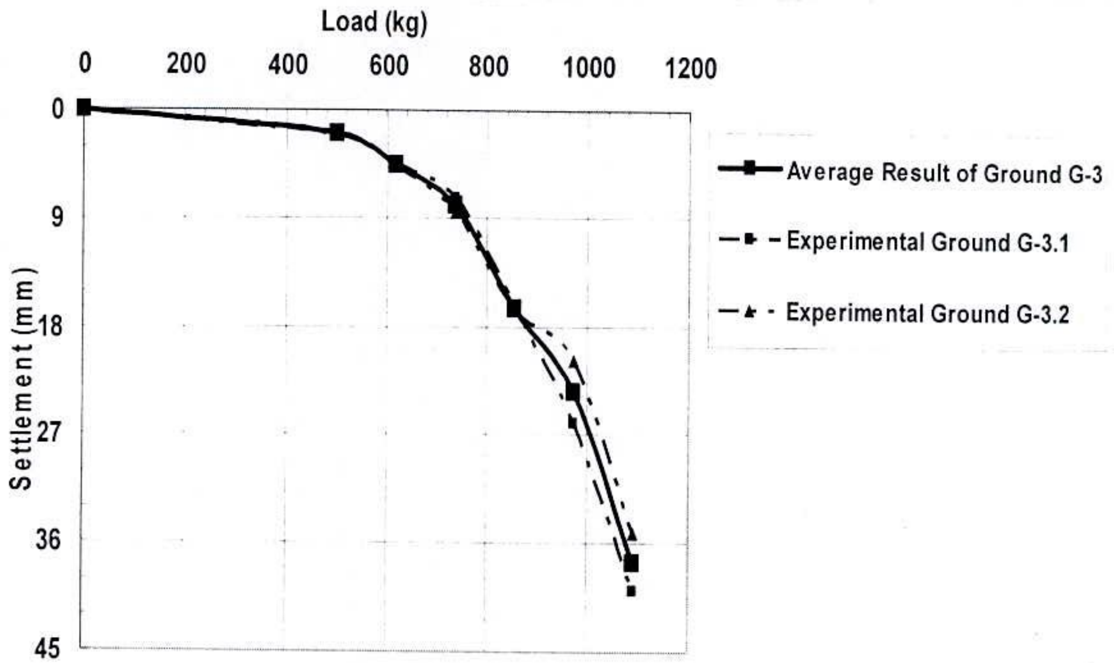


Fig. 4.9 Ground treated with compacted sand bed (G-3)

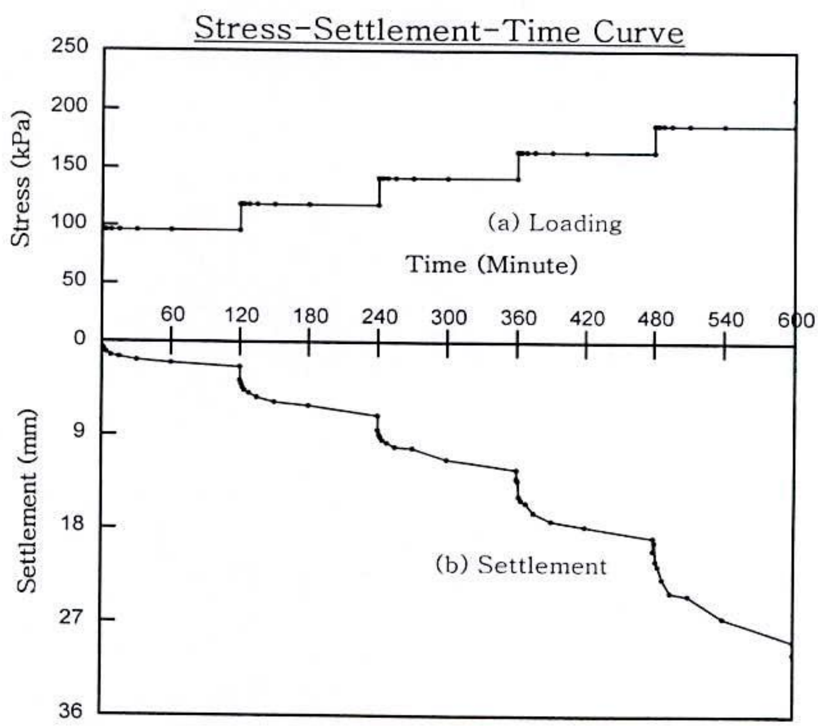


Fig. 4.10 Ground treated with compacted sand bed and geotextile (G-4.1)

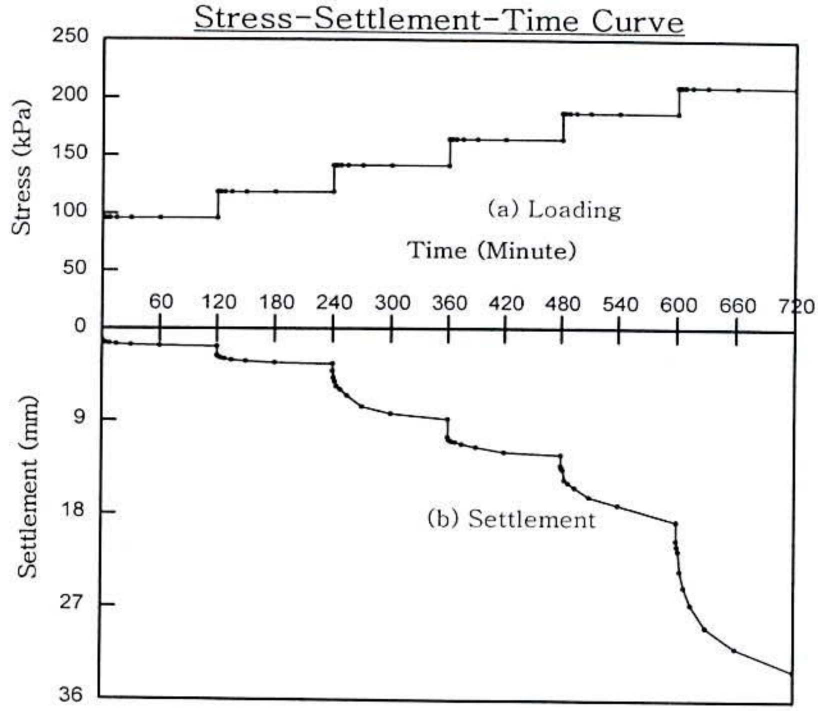


Fig. 4.11 Ground treated with compacted sand bed and geotextile (G-4.2)

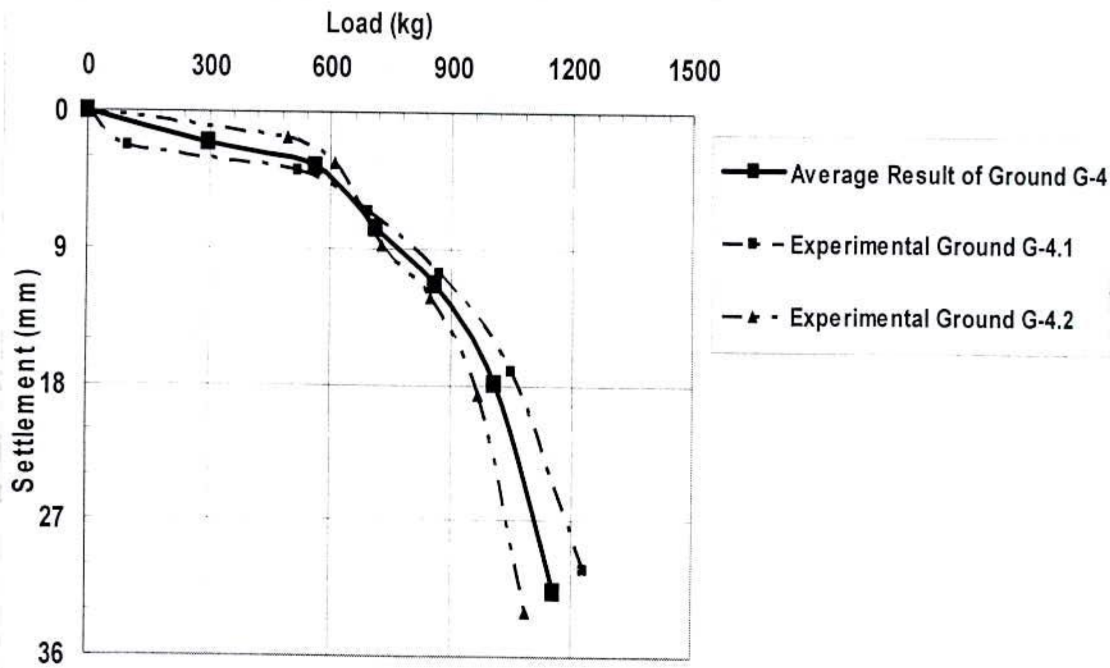


Fig. 4.12 Ground treated with compacted sand bed and geotextile (G-4)

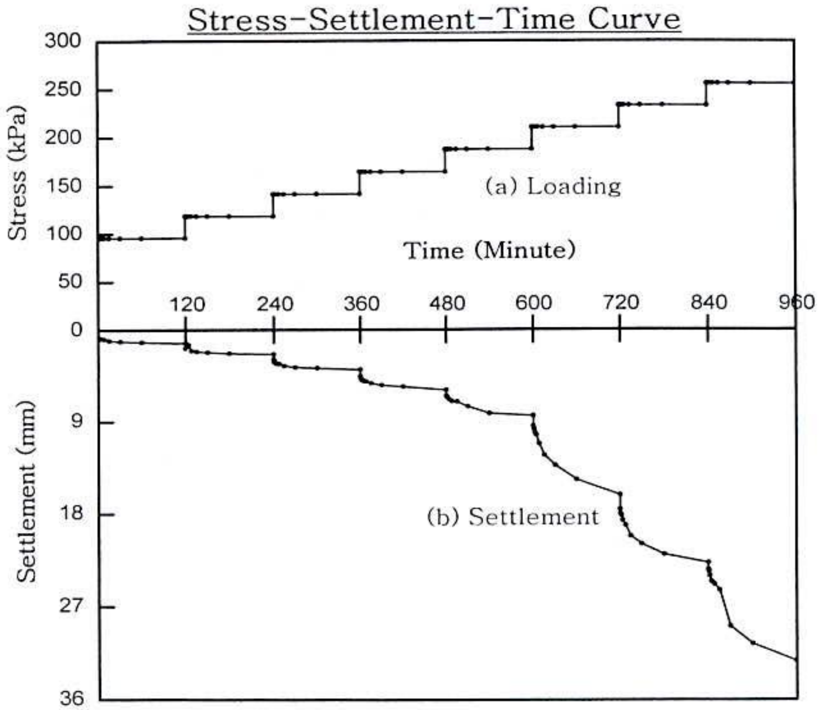


Fig. 4.13 Ground treated with compacted sand bed with geotextile and sand column (G-5.1)

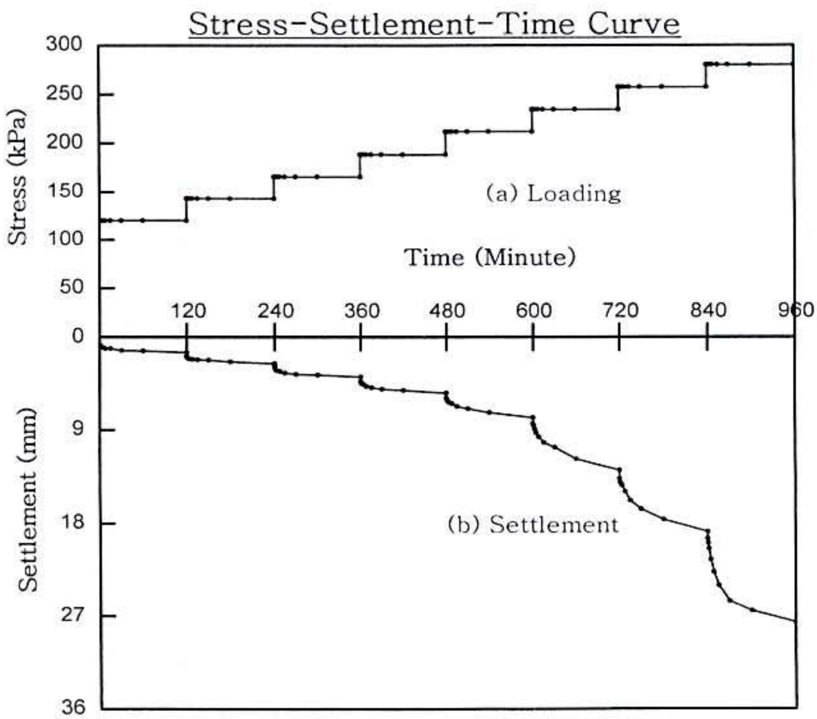


Fig. 4.14 Ground treated with compacted sand bed with geotextile and sand column (G-5.2)

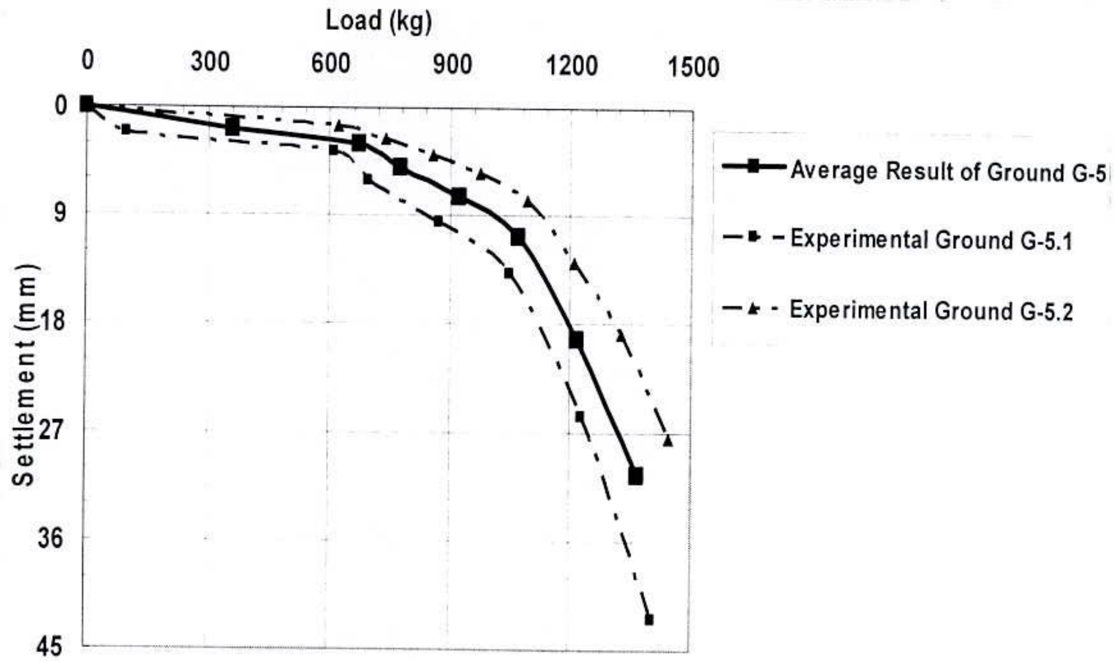


Fig. 4.15 Ground treated with compacted sand bed-geotextile and sand column (G-5)

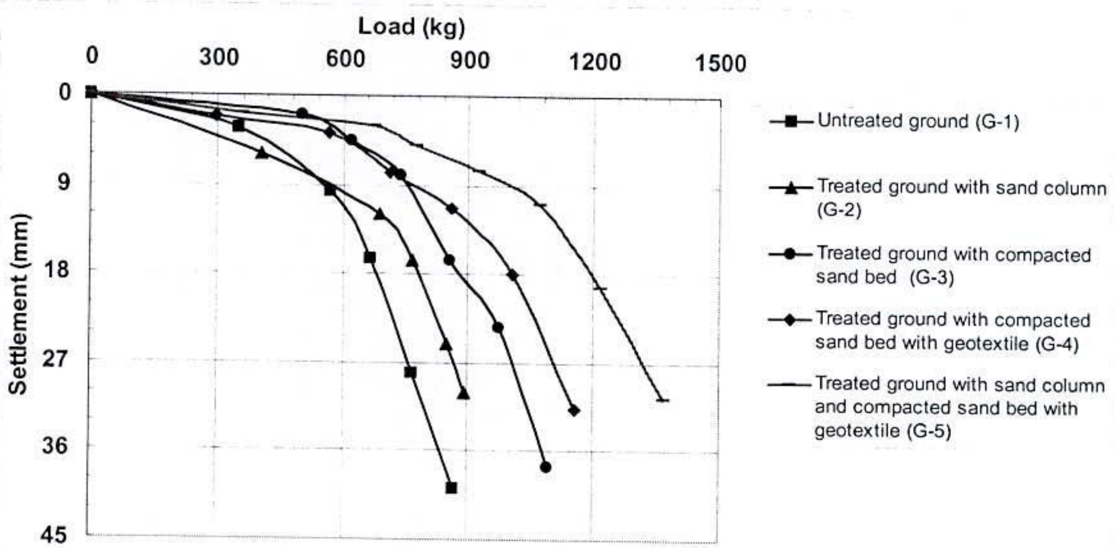


Fig. 4.16 Load-settlement curve of untreated and treated grounds

4.2.2 Load-settlement behaviour and failure patterns of treated grounds

Soil improvement techniques were applied to find out the effectiveness of different soil improvement. Compacted sand column was used as an application of one of the four ground improvement conditions. The load-settlement curves for treated grounds by sand column are shown in Figures 4.4 to 4.6. The load- settlement curve of the soft ground improved by single compacted sand column did not show any significant improvement. The settlement rate has increased sharply after reaching value of settlement as 15 mm. The bearing capacity of soft organic ground improved by sand column was not significant.

But it was found in the experiments that improvement by compacted sand bed without and with geotextile and also in conjunction of compacted sand column have played a vital role in improving load-settlement behaviour of soft organic ground as shown in Figures 4.7 and 4.9. With these ground improvement techniques, the soft grounds not only improved in context of settlement but also the settlement rate become slower and uniform. Due to the use of compacted sand bed on reconstituted organic ground, the ground can be able to carry huge load against small amount of settlement. In the laboratory investigation, it is found that the ground treated by compacted sand bed can be able to carry 840 kg of load for the settlement of 6 mm. After the settlement of 6 mm, the load-settlement curve has changed sharply and large amount of settlement occurred. So compacted sand bed is an effective solution for improving the organic soil ground if there is a possibility of differential settlement or having very low bearing capacity than the requirement. To overcome the problem of the compacted sand bed, the use of geotextile could be an alternative solution. The use of geotextile reduces the possibility of sudden occurrence of large settlement.

In addition to this ground improvement technique, when a compacted sand column was inserted with compacted sand bed having geotextile layer, the bearing capacity of organic ground was improved significantly as shown in Figures 4.10 to 4.12. The improved ground can able to carry huge amount load of massive structures. The geotextile sandwich compacted sand bed with compacted sand column not only reduces the excessive settlement but also bring uniformity in settling under loading i.e. below the base of the footing. This improved behaviour of soft ground can reduce the risk of sudden failure of foundation due to excessive total and differential settlements. The failure pattern of treated grounds and also geotextile are shown in Figures 4.18 to 4.22.

Fig. 4.17 Failure pattern of untreated ground.

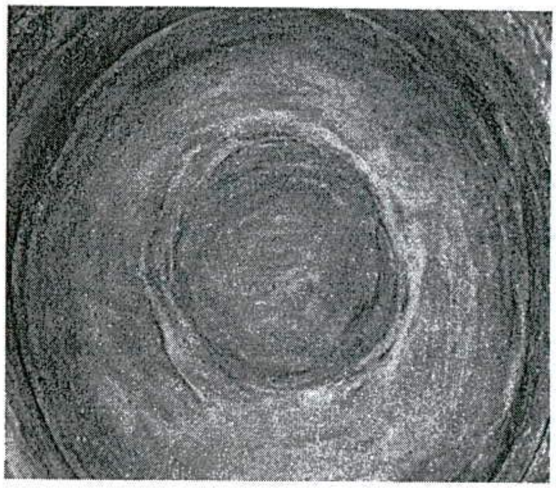


Fig. 4.18 Failure pattern of ground treated by compacted sand bed.

Fig. 4.19 Failure pattern of ground treated by compacted sand bed with geotextile.

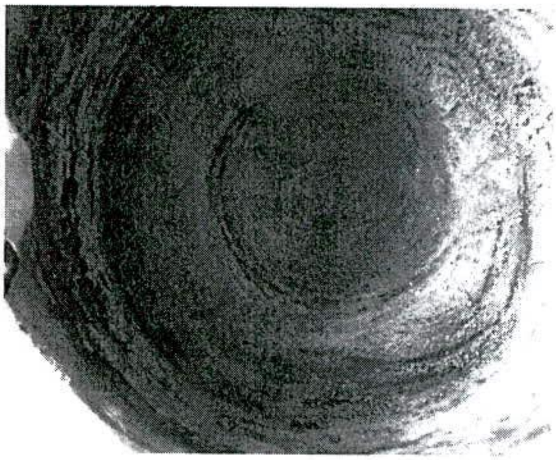


Fig. 4.20 Failure pattern of ground treated by sand bed with geotextile and sand column.

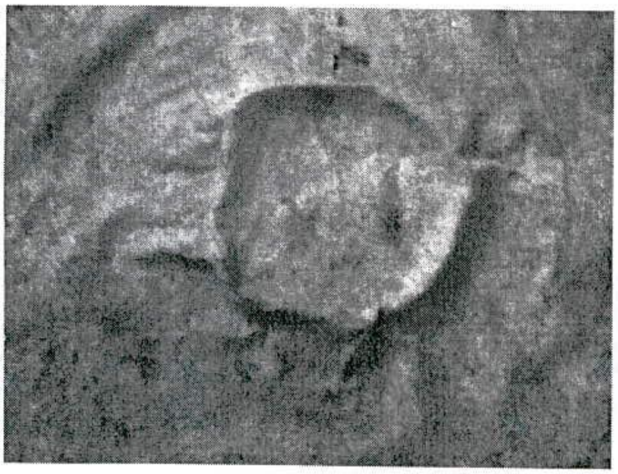


Fig. 4.21 Failure pattern of geotextile.

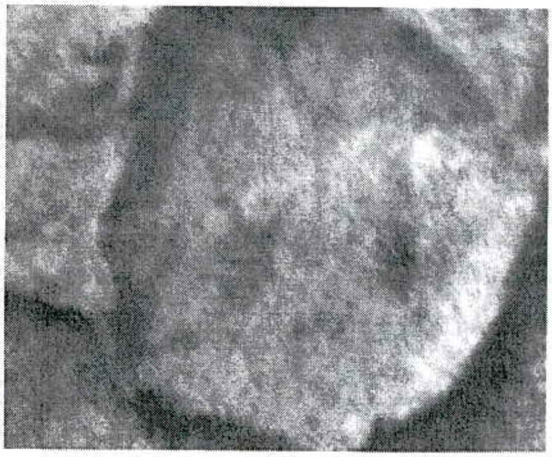


Fig. 4.22 Failure pattern of geotextile.

4.3 Evaluation of Bearing Capacity of Untreated and Treated Grounds

The bearing capacity of the untreated and treated reconstituted organic grounds was determined by footing load test method from the measured load-settlement curve. By plotting load versus settlement in both normal graph and log-log graph, the failure point of the load-settlement curve was determined and with respect to the failure point the bearing capacity of the ground was evaluated. The determination of bearing capacity of reconstituted organic ground treated by compacted sand column (G-2.1) has been described in Figures 4.23-4.24 and for other cases as shown in Appendix-B. If the failure point was found out after settlement of 25 mm, then load for 25 mm settlement was considered as bearing capacity of the footing resting on that ground. The bearing capacity obtained in the untreated and treated grounds determined from the load –settlement curve as shown in Table 4.1.

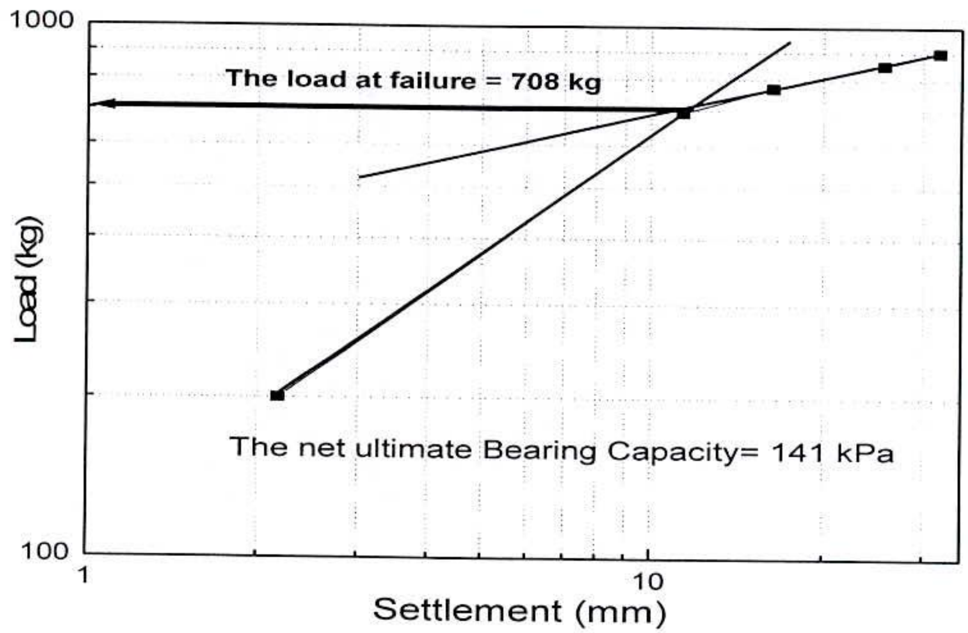


Fig. 4.23 Determination of bearing capacity from load-settlement curve (log-log graph) for ground treated by compacted sand column (G-2.1).

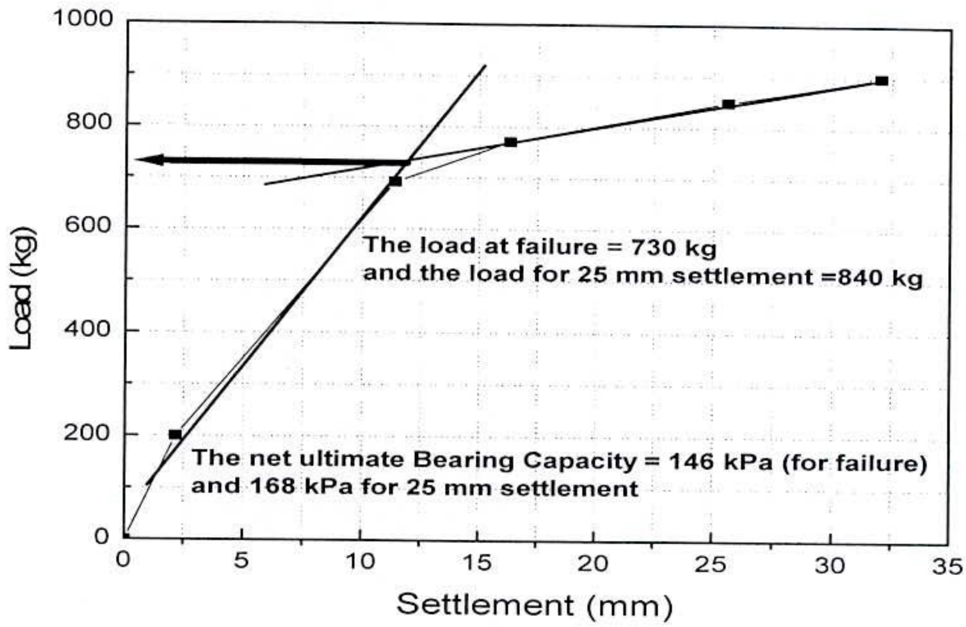


Fig. 4.24 Determination of bearing capacity from load-settlement curve (normal graph) for ground treated by compacted sand column (G-2.1).

Table 4.1 The bearing capacity of footing at different soil improvement conditions

Type of Ground	The net ultimate bearing capacity by the settlement rate method (kPa)								
	I				II				Average bearing capacity
	Normal graph	Log-log graph	25mm settlement	Selected bearing capacity	Normal graph	Log-log graph	25mm settlement	Selected bearing capacity	
G-1	--	112	147	**112**	--	130	148	**130**	**121**
G-2	146	141	168	**141**	141	153	170	**141**	**141**
G-3	--	171	193	**171**	--	171	202	**171**	**171**
G-4	--	--	204	**204**		--	204	**204**	**204**
G-5			239	**239**			283	**283**	**261**

4.4 Degree of Improvement

The net ultimate bearing capacity of the untreated ground was found as 118 kPa by the settlement rate method. The degree of improvement for ground treated by compacted sand column is found as 1.18., the net ultimate bearing capacity is improved by 1.45 times of the untreated ground by improving with compacted sand bed. Then the net ultimate bearing capacities of grounds treated with geotextiles sandwiched sand compacted bed was obtained as 1.67 times than that of untreated ground. This improved value was increased to 2.20 times while the improvement associated with geotextiles-sand bed and sand column. The degree of improvement of treated grounds with respect to untreated ground are shown in Table 4.2

Table 4.2 The degree of improvement for different soil improvement conditions

Type of Ground	Net ultimate bearing capacity (kPa)	Degree of improvement
G-1	121	1.00
G-2	141	1.17
G-3	171	1.41
G-4	204	1.69
G-5	261	2.16

The contribution of sand and geotextile in improving organic soils while using in compacted sand column and compacted sand bed are shown in Table 4.3. It can be found from the laboratory investigation that sand provides strength effectively in compacted sand bed and it is about 45% and geotextile improve bearing capacity of organic ground about 22%.

Table 4.3 Contribution of improvement techniques in ground improvement conditions

Type of Ground	Net ultimate bearing capacity (kPa)	Degree of improvement	Compacted sand column (%)	Compacted sand bed (%)	Geotextile as reinforcement (%)	Sand column in geotextile reinforced compacted sand bed (%)
G-1	121	1.00				
G-2	141	1.17	17			
G-3	171	1.41		41		
G-4	204	1.69			28	
G-5	261	2.16				47

CHAPTER FIVE

COMPARISION BETWEEN TEST RESULT AND THEORETICAL SOLUTION

5.1 General

This chapter describes the comparison on load-settlement behaviour between measured and predicted values obtained in reconstituted organic grounds for five conditions. For appropriate comparison, the reconstituted organic grounds were constructed by using common ground preparation technique, loading condition, loading duration and environment. Although some variations were found in the strength and compressibility parameters in the prepared grounds but in geotechnical point of view these variations were negligible. The net bearing capacity of the untreated and treated grounds was evaluated not only using the measured load-settlement curves but also based on the prediction evaluated from the available bearing capacity equations. Based on this comparative study the applicability of the available empirical equations is also commented here.

5.2 Prediction of Bearing Capacity of Reconstituted Organic Grounds

The bearing capacity of the reconstituted organic grounds was determined using some established equations besides the footing load test. Despite some limitations of using the bearing capacity equations in case of organic grounds, the equations were used to determine the bearing capacity of the footing resting on these experimental grounds. The considered values of some used parameters are not the exact one appropriate for organic soils. In the following articles, the bearing capacities of the footing resting on test grounds determined by available equations are described.

5.2.1 Bearing capacity of untreated grounds

The bearing capacity equations proposed by Terzaghi (1943), Meyerhof (1963), Hansen (1970), Vesic (1973 and 1975) were used to determine the bearing capacity of reconstituted organic grounds. In all equations the value of cohesion was used as undrained shear strength of soil i.e. half of unconfined compressive strength of soil. As the RCC footing was placed on surface of reconstituted organic grounds, the depth factor was considered as zero. Some factors of bearing capacity equations were calculated by the recommended equations and some were taken from the recommended tables as described in the chapter of literature review. The bearing capacity obtained in the two grounds were predicted as 193 and 167 kPa by Terzaghi's and Meyerhof's equations, respectively, and 214 and 185 kPa obtained by Hansen and Vesic equation, respectively.

5.2.2 Bearing capacity of treated grounds

In this study the reconstituted organic grounds were treated with compacted sand column, compacted sand bed with and without geotextile and compacted sand bed with geotextile in conjunction of compacted sand column.

Firstly, two reconstituted organic grounds were treated with single compacted sand column and the bearing capacity of the treated grounds was determined using five available empirical equations already discussed in the chapter of literature review. These are (i) Passive pressure condition, (ii) Based on expansion of a cylinder, (iii) Based on cavity expansion theory, (iv) Based on pile formula and (v) Hughes et al. (1975). The bearing capacity of the footing on the two reconstituted organic grounds treated by compacted sand column was presented in the Table 5.1 and among the obtained values the lowest was obtained as 237 and 282 kPa, while using the equation of cavity expansion theory.

Table 5.1 Measured and Predicted bearing capacity of footing resting on untreated and treated grounds

	G-1		G-2		G-3		G-4		G-5	
	G-1.1	G-1.2	G-2.1	G-2.2	G-3.1	G-3.2	G-4.1	G-4.2	G-5.1	G-5.2
Unconfined compressive strength (kPa)	52	45	45	60	46	51	46	51	47	53
$c = q_u/2$ (kPa)	26	22.5	22.5	30	23	25.5	23	25.5	23.5	26.5
Bearing capacity obtained in the untreated grounds in kPa										
Terzaghi (1943)	193	167	167	222	170	189	174	196	159	256
Meyerhof (1963)	193	167	167	222	170	189	174	196	159	256
Hansen (1970), Vesic (1973& 1975)	214	185	185	247	189	210	193	218	177	284
Bearing capacity obtained in the treated grounds in kPa										
Passive pressure condition (Greenwood, 1970)			349	460						
Based on expansion of a cylinder (Gibson & Anderson, 1961)			332	438						
Based on cavity expansion theory (Vesic 1972)			237	282						
Based on conventional pile formula			641	990						
Hughes et al. (1975) formula			478	637						
General Bearing Capacity Equation of footing for layered soil					240	265	814	814	1060	1106
Measured bearing capacity obtained in untreated and treated grounds in kPa										
Footing load test result	112	130	141	141	171	171	204	204	239	283

Then the bearing capacity of reconstituted organic grounds treated with compacted sand bed with and without geotextile was determined by considering footing on layered soils. In this case, Hansen's bearing capacity equations were used for the determination of bearing capacity of footing in the treated grounds. The values of bearing capacity were found as 240 and 265 kPa for ground treatment with compacted sand bed and 814 kPa for the ground treated with compacted sand bed with geotextile.

Finally, reconstituted organic grounds were treated with compacted sand bed with geotextile in conjunction of compacted sand column. The bearing capacity of the treated grounds was determined by considering the concept of layered soil and reinforcing with compacted sand column. The bearing capacity of the treated grounds was calculated as arithmetic sum of obtained values from both the concepts. To determine the contribution of compacted sand column and geotextile sandwich compacted sand bed in improving soft grounds were determined by cavity expansion theory and Brinch- Hansen formula. The values of bearing capacity of treated grounds were found from the calculation as 1060 and 1106 kPa, respectively.

5.3 Comparison of the Predicted and Measured Values of Bearing Capacity

In this study the bearing capacity of treated and untreated reconstituted organic grounds were determined not only by footing load test but also by available equations as shown in the Table 5.1. The bearing capacity obtained from the established equations was significantly higher than that of the experimental result. Despite such noticeable discrepancy, these equations were used in this study to develop a comparison among the ground improvement conditions.

The bearing capacity in the untreated grounds were obtained as 112 and 130 kPa from the footing load test and 193 and 167 kPa from available equations. The variation between test and theoretical results is clearly significant. So it can be said that the available equations for the determination of the bearing capacity of footing in such soft organic grounds is not suitable since it highly over predicted.

In case of improvement of reconstituted organic grounds by compacted sand column, theoretically the degree of improvement was found as 1.5 times than that of untreated grounds. But experimentally the improvement of grounds by compacted sand column were not significant than that of untreated grounds that was about 1.2 times. Again unacceptable differences were found in the results of bearing capacity obtained from the available equations and footing load tests.

The bearing capacity of reconstituted organic grounds treated by compacted sand bed was determined by the equation of layer soil that was about 1.4 times than that of untreated grounds. After that when geotextile was introduced in the compacted sand bed, experimentally this improvement techniques improved the organic grounds more than 1.5 times than that of untreated ground but theoretically the degree of improvement was found as four times than that of untreated grounds. In this case significant variations were observed in the theoretical and experimental results.

Finally the reconstituted organic grounds were treated by compacted sand bed with geotextile and compacted sand column. Both experimentally and theoretically this improvement condition improved the organic grounds significantly. Theoretically, this improvement condition improved the reconstituted organic grounds about six times than that of untreated grounds, while the experimental result showed that this improvement is slightly more than two times.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusion

Based on this experimental study the following conclusions can be made:

(i) The test results reveal that the load carrying capacity of soft organic ground can be enhanced significantly by adopting one of the studied ground improvement techniques.

(ii) The rate of settlement of the test footing resting on untreated organic ground has increased suddenly after the settlement of the ground exceeds 12 mm which is 3% of the thickness of compressible soil media.

(iii) It is observed that the inclusion of compacted sand bed with and without geotextile on the soft ground is more effective for improving bearing capacity of organic soils in compare to that of the compacted sand column.

(iv) The soft ground treated by compacted sand bed with and without geotextile shows flatter load-settlement curve which represents slower settlement rate with respect to untreated ground and improved the bearing capacity of reconstituted organic ground about 1.5 times than that of untreated reconstituted organic ground.

(v) The soft ground treated by compacted sand bed using geotextile in conjunction with compacted sand column also shows flatter load-settlement curve which represents slower settlement rate and this system increases the load bearing capacity of reconstituted organic ground about two times than that of the untreated reconstituted organic ground.

(vi) The results reveal that the ground improvement system, compacted sand bed with geotextile in conjunction of compacted sand column is more suitable for the improvement of soft ground considering the degree of improvement.

(vii) The comparison of predicted and measured load carrying capacity of the improved grounds reveals that the available empirical equations are suitable for the prediction of bearing carrying capacity of footing resting on the sand column and compacted sand bed treated grounds but highly over predicts the same in case of grounds treated by compacted sand bed with geotextile and compacted sand bed with geotextile in conjunction of compacted sand column.

6.2 Limitations of the Study

This investigation was performed on grounds prepared in the laboratory using reconstituted organic soils collected from selected site of Khulna region. This is a comparative study to find out degree of effectiveness of different ground improvement techniques in reconstituted organic soils. So the grounds should be same in the context of geotechnical properties. Although some variations were observed in the geotechnical properties of grounds, however, such differences can be neglected from the geotechnical point of view. The prepared grounds were treated as the similar to the problematic ground of Khulna region; however, this similarity does not exist while comparing with the field condition due to the large difference of overburden pressure. As all the prepared grounds were taken as experimental grounds having very close values of different properties, so a comparison can be established on the improvement capacity for different ground improvement conditions.

Besides laboratory investigation, the bearing capacities of reconstituted organic grounds were determined by using available equations. While using these equations, the values of parameters were assumed or taken considering soft clay. Some equations are not fully appropriate to use in case of organic soils. As the bearing capacity of reconstituted organic grounds was determined with these parameters, the predicted values are not appropriate for practical application, but for comparative study, these are reasonably acceptable.

6.3 Recommendations for Future Studies

Based on the present study the following recommendation for future research can be made:

(i) The reconstituted organic ground can be improved using compacted sand bed with two layers of geotextile to examine another soil improvement condition.

(ii) Geotextile should be used at bottom and top positions of the reconstituted organic ground as good filter media to avoid the possibility of clogging of sand bed.

(iii) Woven geotextile can be used as reinforcement instead of non-woven goetextile.

(iv) Pore pressure transducer should be used for measuring pore pressure of the grounds to obtained grounds of same conditions to compare appropriate the behaviour of the test grounds.

(v) Data logger, load cell and deformation transducer should be used for preparation of grounds and also for determination of appropriate load-settlement nature of test grounds.

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APPENDIX-A

Load-settlement curve for test grounds

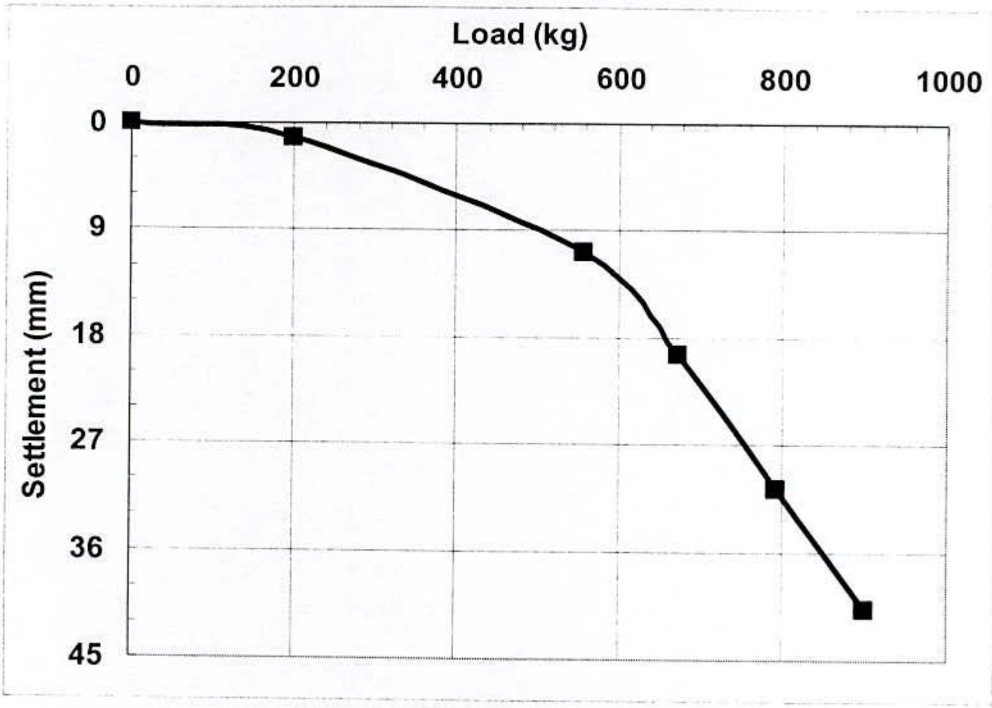

Fig. A-1 Untreated ground (G-1.1)

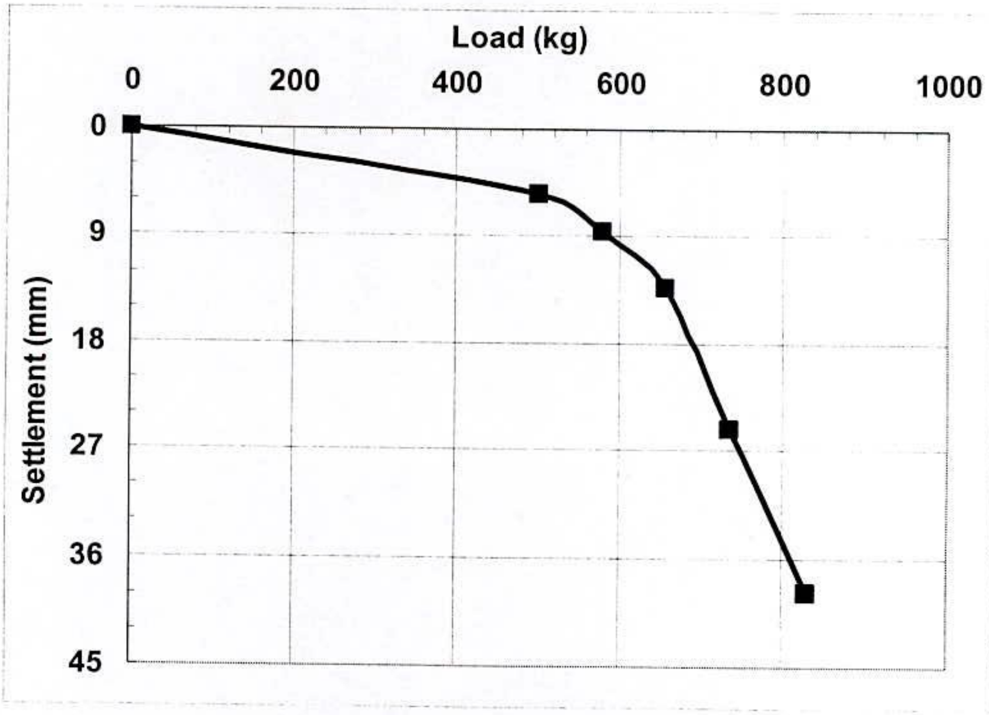

Fig. A-2 Untreated ground (G-1.2)

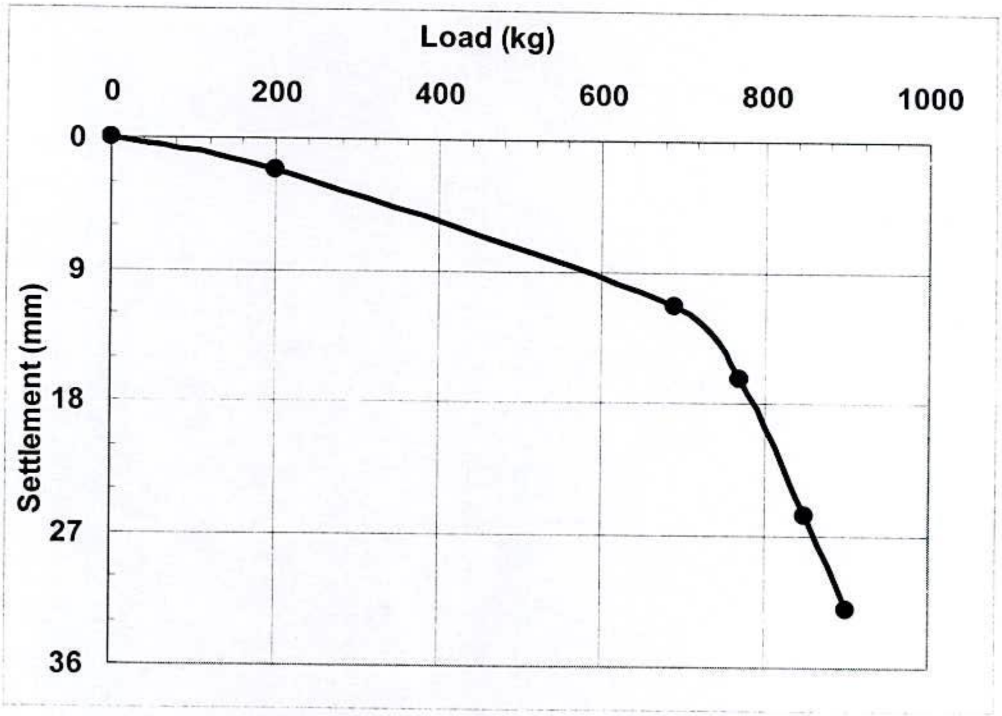

Fig. A-3 Ground treated with sand column (G-2.1)

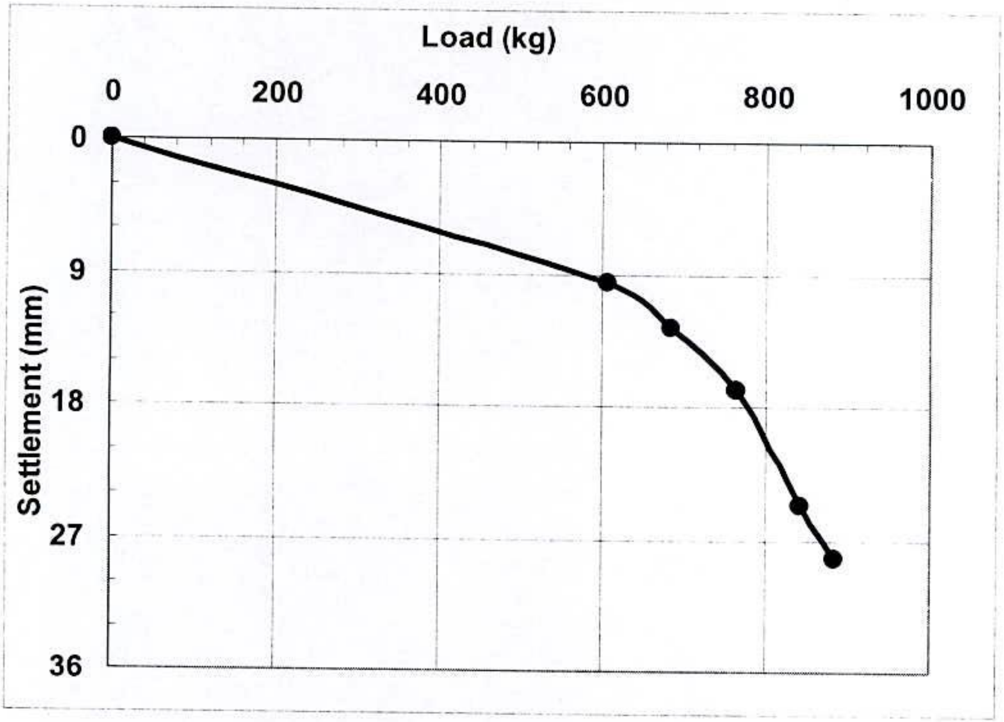

Fig. A-4 Ground treated with sand column (G-2.2)

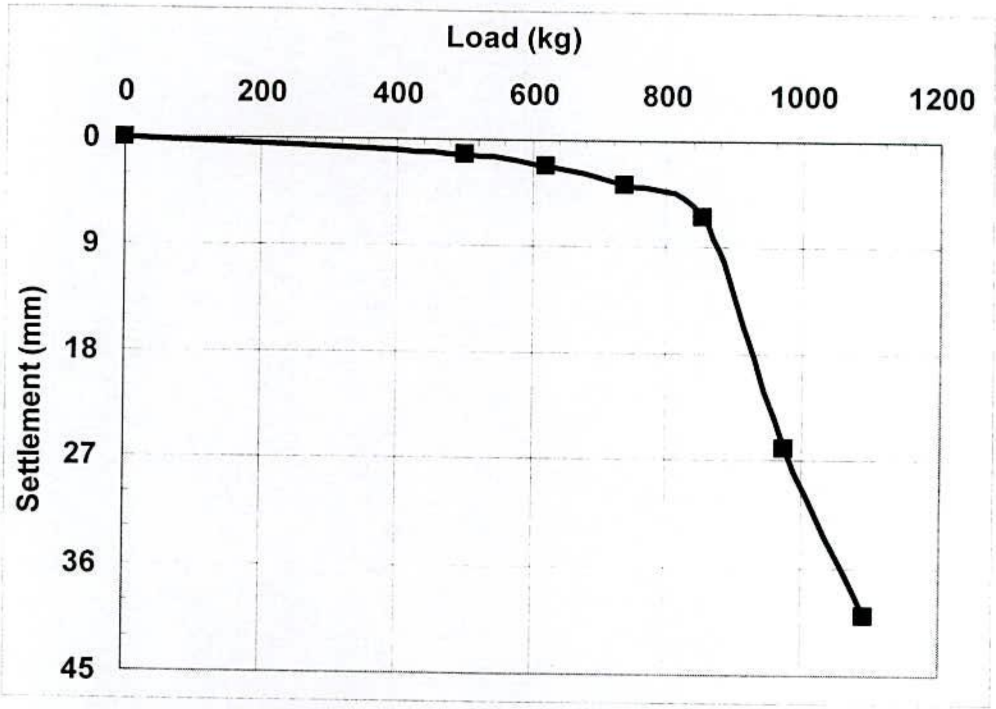

Fig. A-5 Ground treated with compacted sand bed (G-3.1)

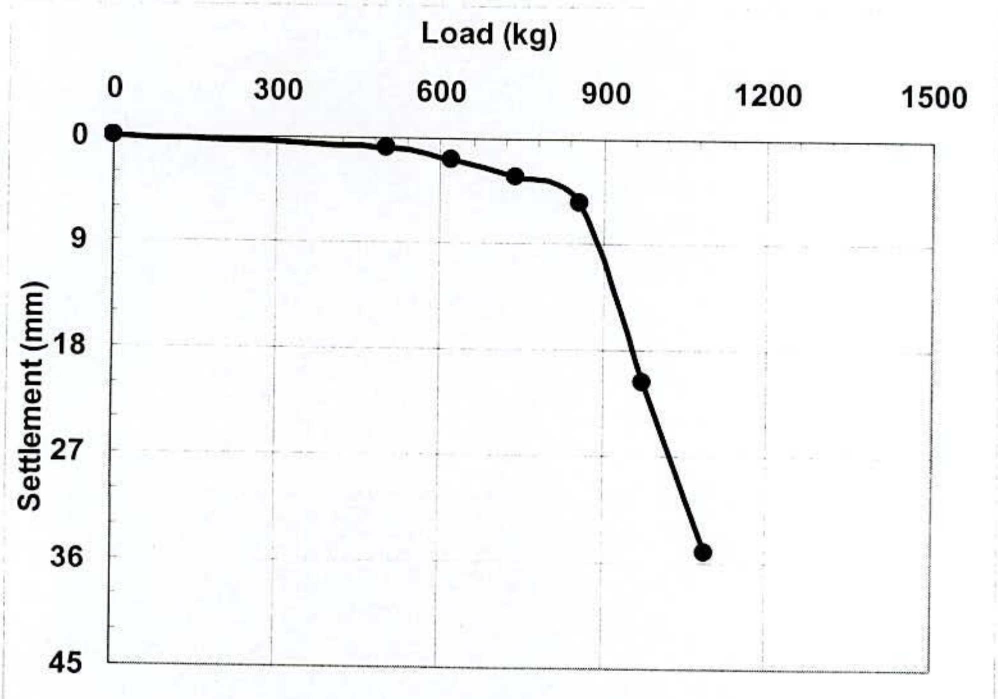

Fig. A-6 Ground treated with compacted sand bed (G-3.2)

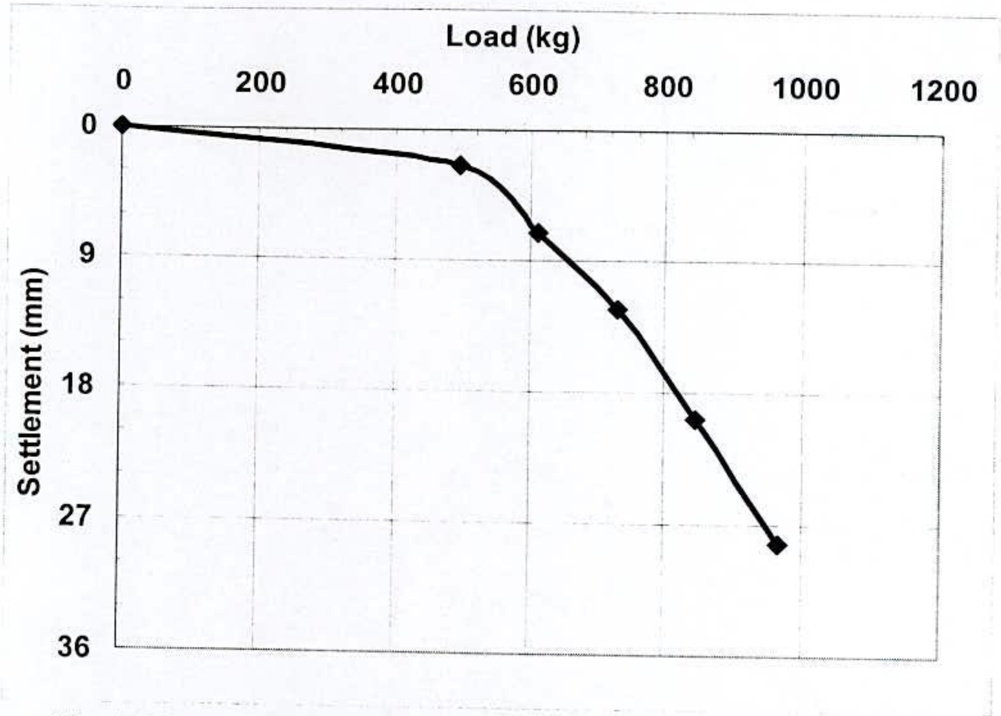

Fig. A-7 Ground treated with compacted sand bed and Geotextile (G-4.1)

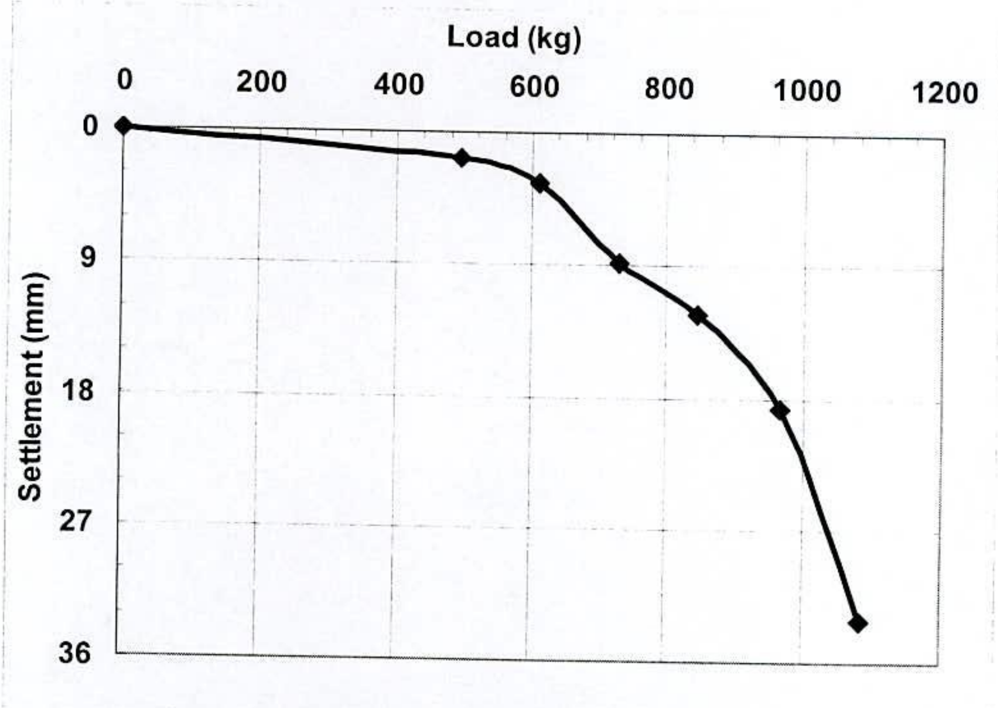

Fig. A-8 Ground treated with compacted sand bed and Geotextile (G-4.2)

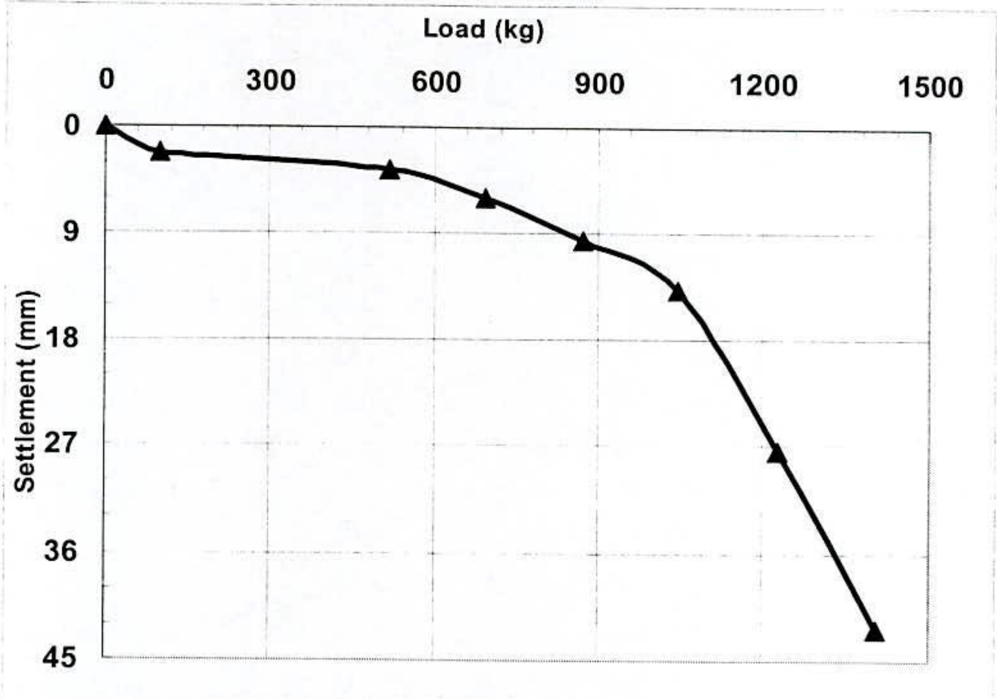

Fig A-9 Ground treated with compacted sand bed-geotextile and sand column (G-5.1)

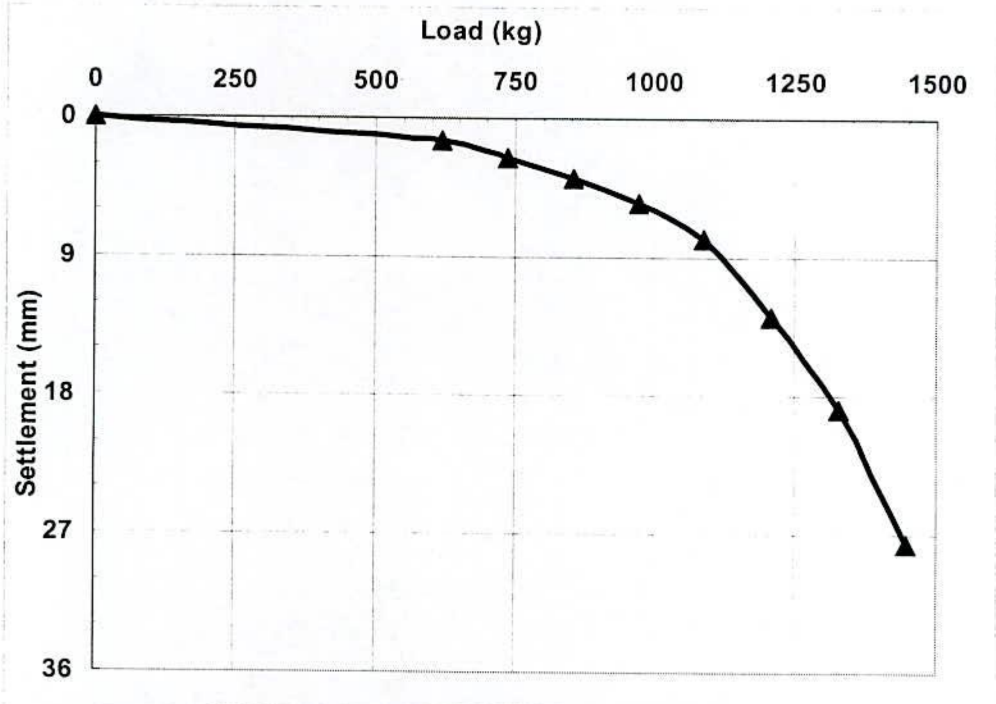

Fig. A-10 Ground treated with compacted sand bed-geotextile and sand column (G-5.2)

APPENDIX-B

Determination of bearing capacity of untreated and treated grounds from load-settlement curve

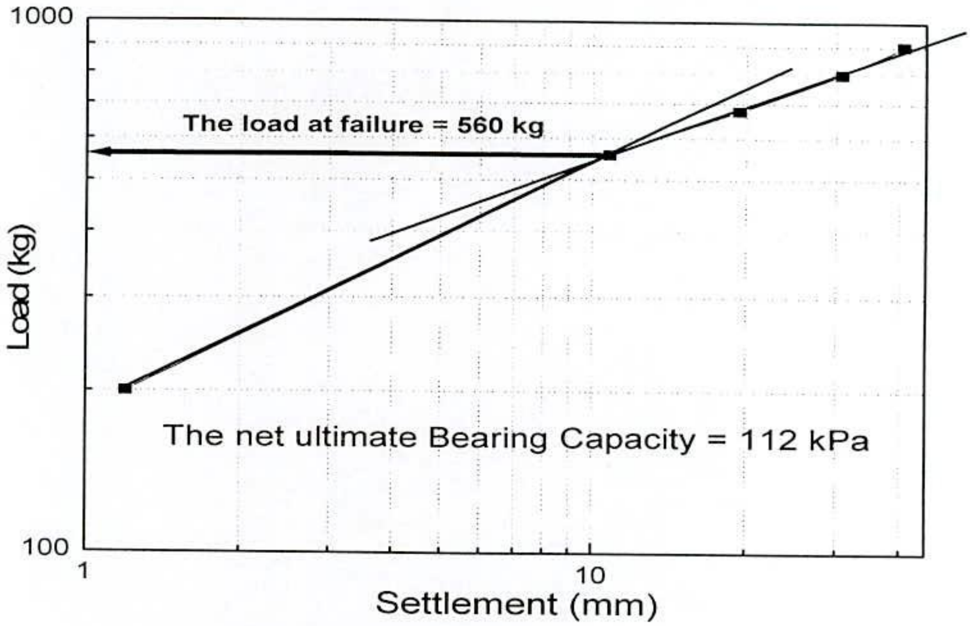

Fig. B-1 Determination of bearing capacity of untreated ground (G-1.1)

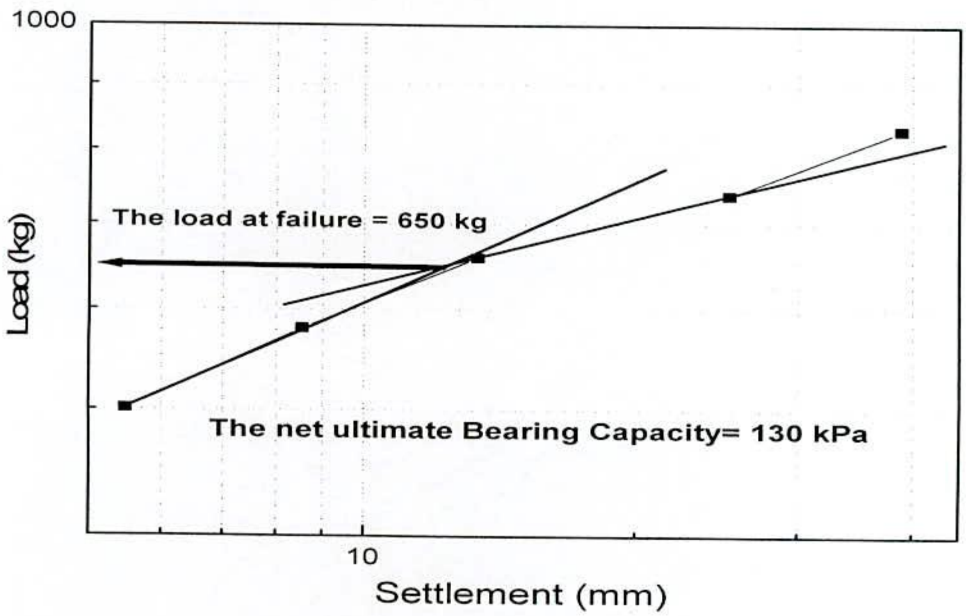

Fig. B-2 Determination of bearing capacity of untreated ground (G-1.2)

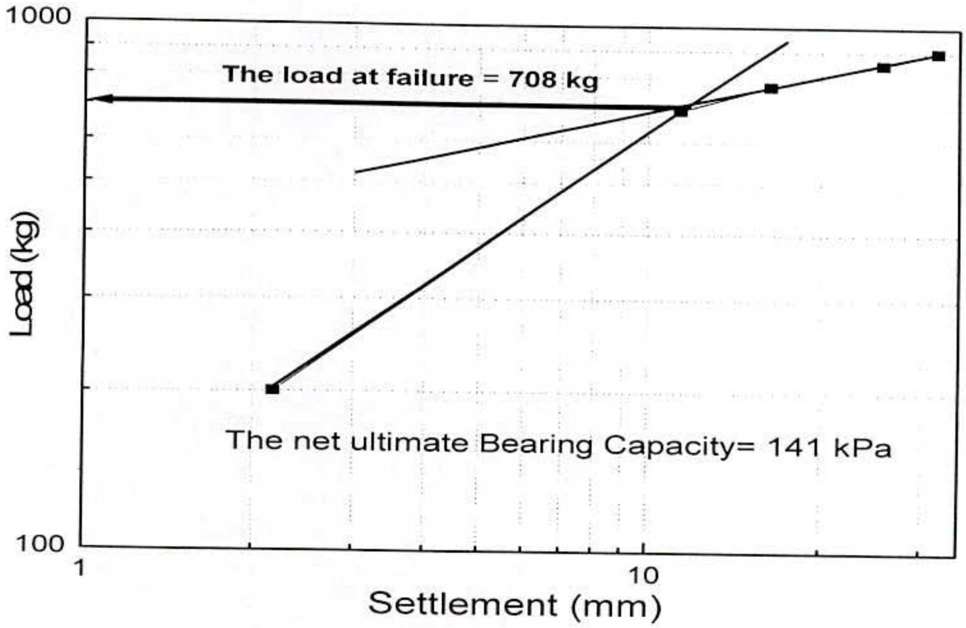

Fig. B-3 Determination of bearing capacity of ground treated by compacted sand column (G-2.1)

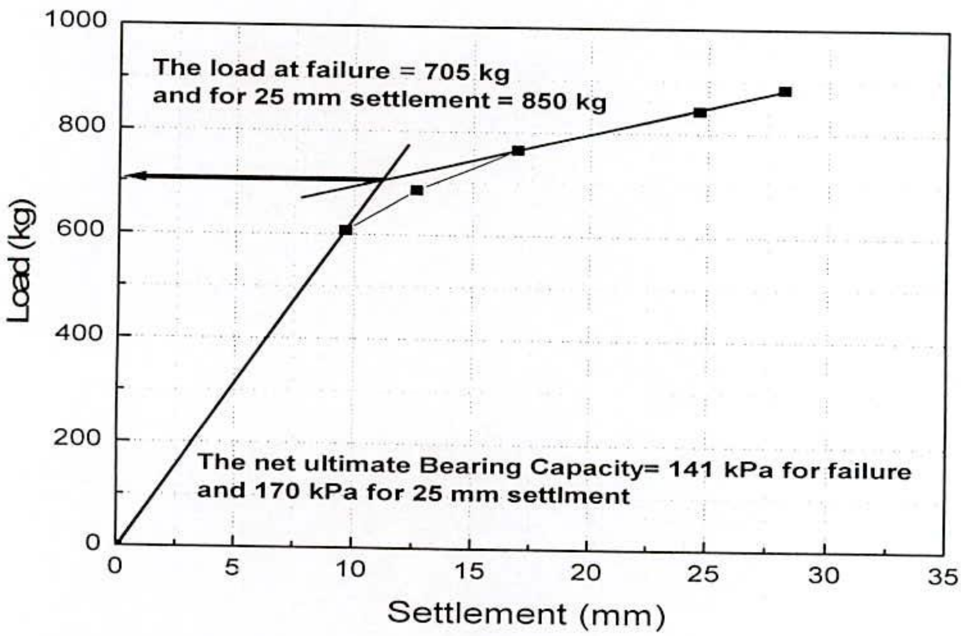

Fig. B-4 Determination of bearing capacity of ground treated by compacted sand column (G-2.2)

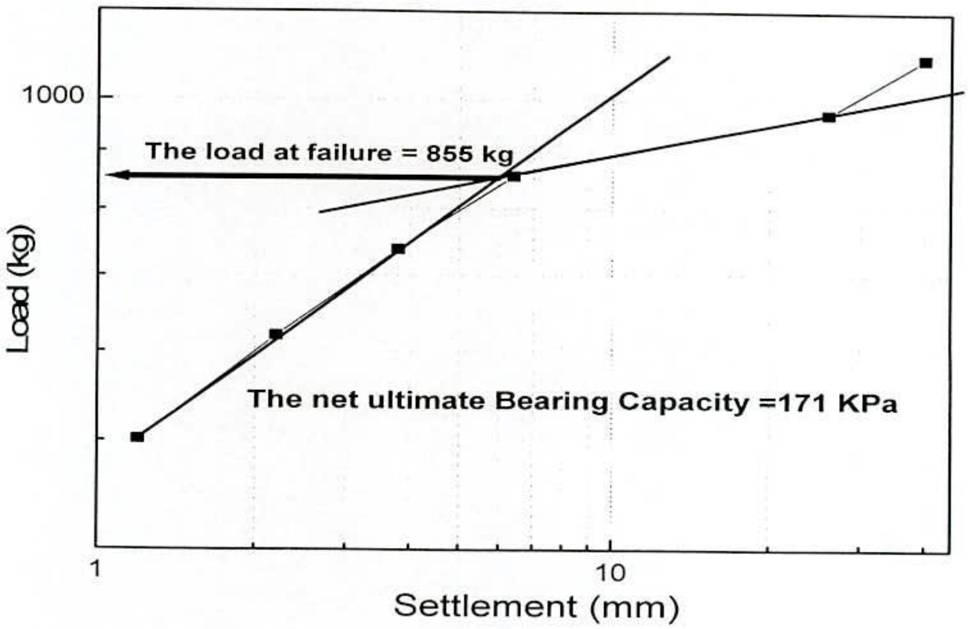

Fig. B-5 Determination of bearing capacity of ground treated by compacted sand bed (G-3.1)

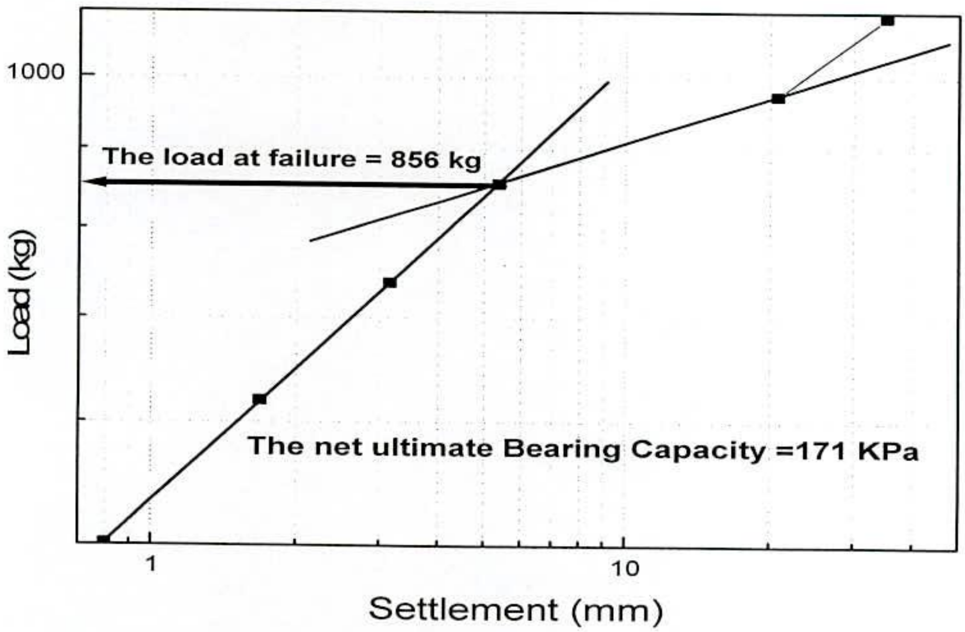

Fig. B-6 Determination of bearing capacity of ground treated by compacted sand bed (G-3.2)

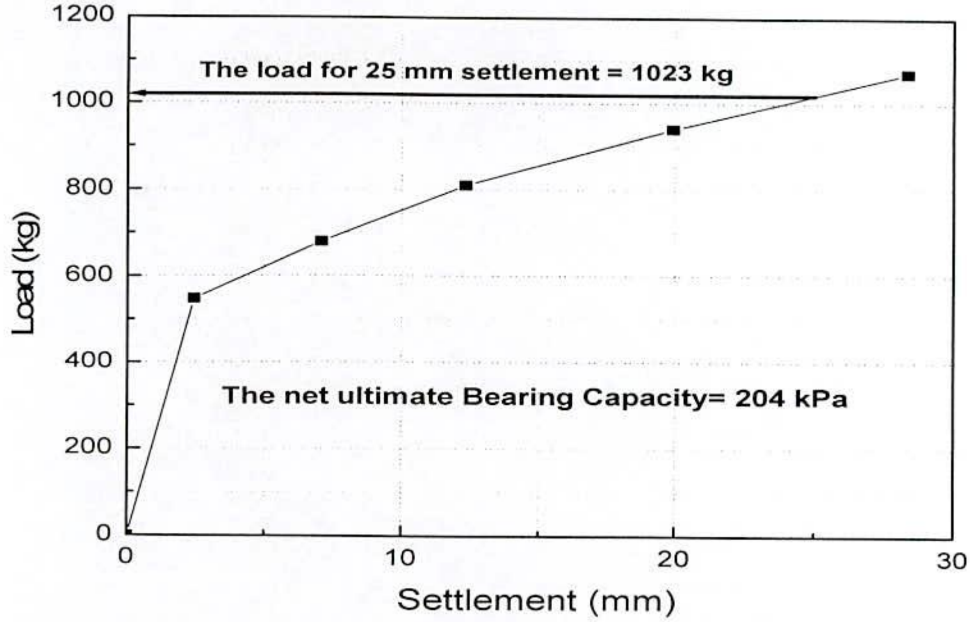

Fig. B-7 Determination of bearing capacity of ground treated by compacted sand bed with geotextile (G-4.1)

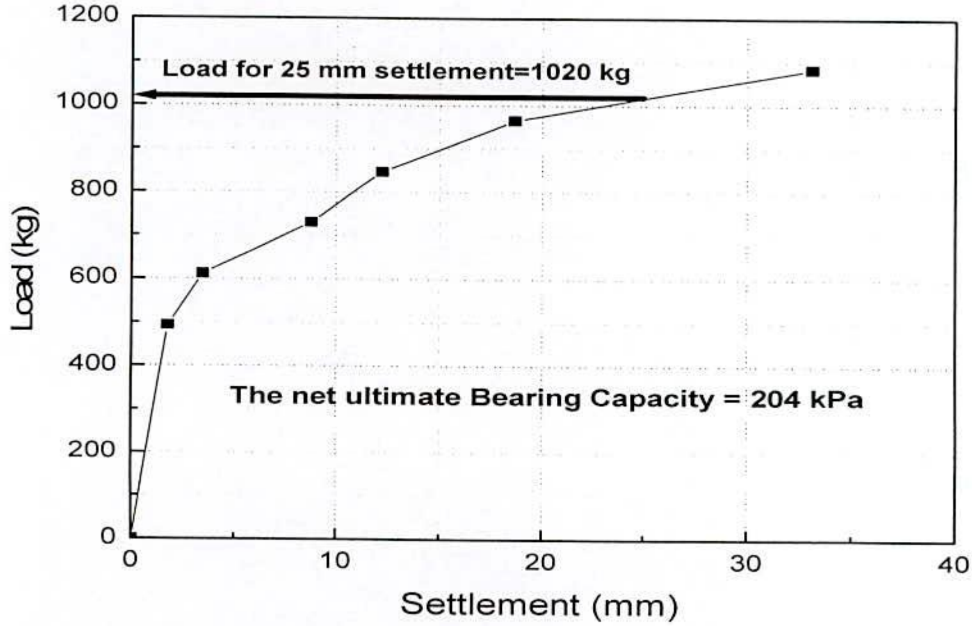

Fig. B-8 Determination of bearing capacity of ground treated by compacted sand bed with geotextile (G-4.2)

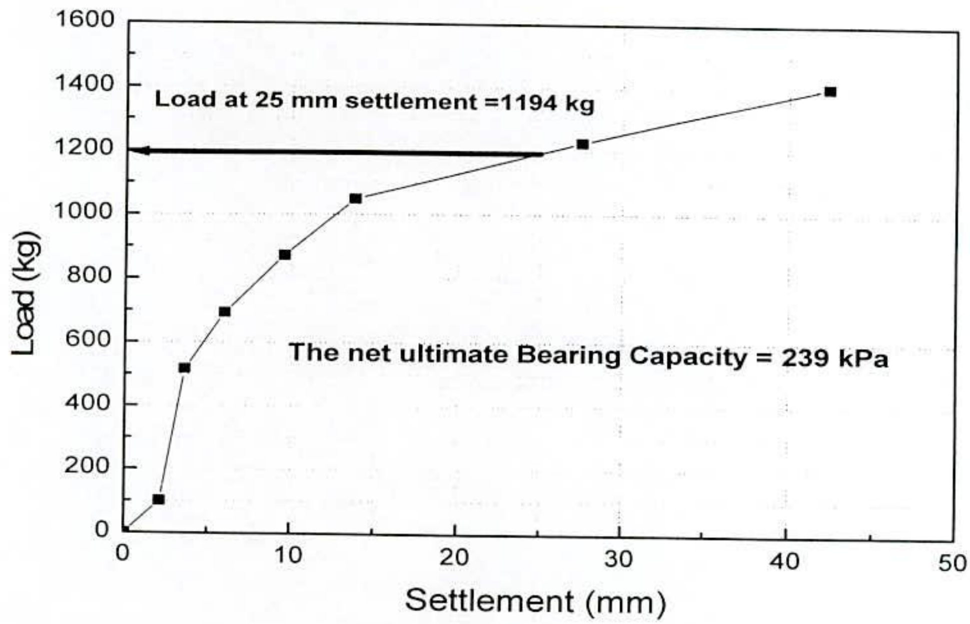

Fig. B-9 Determination of bearing capacity of ground treated by compacted sand bed with geotextile and compacted sand column (G-5.1)

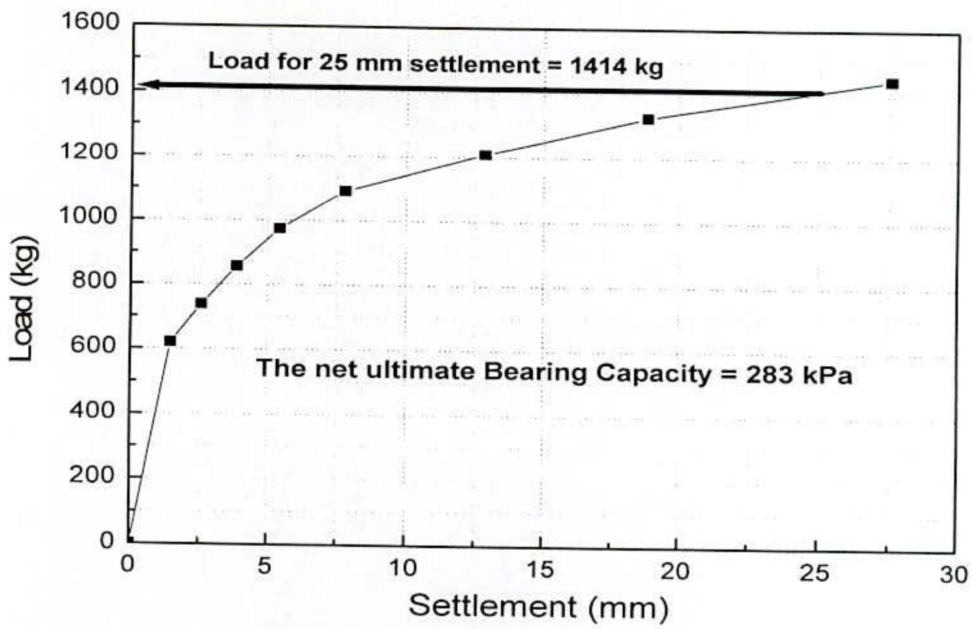

Fig. B-10 Determination of bearing capacity of ground treated by compacted sand bed with geotextile and compacted sand column (G-5.2)

APPENDIX-C

Determination of bearing capacity of untreated and treated grounds by available equations (sample calculation)

C-1: Bearing capacity of untreated ground (G-1.1)

(i) Terzaghi's bearing capacity equation

$q_{ult} = cN_cs_c + \bar{q}\,N_q + 0.5BN_\gamma s_\gamma$

Data:

$c = \frac{1}{2}\,q_u = 0.5 \text{ x } 52 = 26 \text{ kPa}$

$N_c = 5.7$	[ϕ=0°]
$s_c = 1.3$	[Round shaped footing]
$\bar{q} = \gamma h = 0$	[h=0]
$N_q = 1.0$	[ϕ=0°]
B = 0.25 m	[B= Width of footing]
$N_\gamma = 0.0$	[ϕ=0°]
$s_\gamma = 0.6$	[Round shaped footing]

$\mathbf{q_{ult}} = cN_cs_c + \bar{q}\,N_q + 0.5BN_\gamma s_\gamma == 26x5.7x1.3 + 0 + 0 = \mathbf{193\ kPa}$.

(ii) Meyerhof's bearing capacity equation

$q_{ult} = cN_cs_cd_c + \bar{q}\,N_qs_qd_q + 0.5\,\gamma\,B'N\,\gamma\,S\,\gamma\,d\,\gamma$

Data:

$c = \frac{1}{2}\,q_u = 0.5 \text{ x } 52 = 26 \text{ kPa}$

$N_c = 5.7$ [ϕ=0°]

$s_c = 1.3$ [Round shaped footing]

$d_c = 1 + 0.1 \times \tan(45 + \frac{\phi}{2}) \times \frac{D}{B} = 1 + 0.1 \times \tan(45 + \frac{0}{2}) \times \frac{0}{.25} = 1.0$

$\bar{q} = \gamma h = 0$ [h=0]

$N_q = 1.0$ [ϕ=0°]

$B = 0.25$ m [Width of footing]

$N_\gamma = 0.0$ [ϕ=0°]

$s_\gamma = 0.6$ [Round shaped footing]

$q_{ult} = cN_cs_cd_c + \bar{q}N_qs_qd_q + 0.5\gamma B'N\gamma S\gamma d\gamma == 26 \text{x} 5.7 \text{x} 1.3 \times 1 + 0 + 0 =$ **193 kPa**.

(iii) Hansen's & Vesic's bearing capacity equation

$q_{ult} = 5.14s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$ [for $\phi = 0$]

Data:

$s_u = 0.5 \times q_u = 0.5 \times 52 = 26$ kPa

$s'_c = 0.2 \times \frac{B'}{L'} = 0.2 \times 1 = 0.2$

$d'_c = 0.4 \times \frac{D}{B} = 0.4 \times \frac{0.25}{0.25} = 0.4$ [ϕ=0°]

$i'_c = 0$

$b'_c = 0$

$g'_c = 0$

$\bar{q} = \gamma h = 0$ [h=0]

$q_{ult} = 5.14s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

$q_{ult} = 5.14 \times 26 \times (1 + 0.2 + 0.4 - 0 - 0 - 0) + 0 =$ **214 kPa**

(iii) Skempton's bearing capacity equation

$q_f = s_u . N_{cu} + q_o$

Data:

$s_u = 0.5 \times q_u = 0.5 \times 52 = 26$ kpa

$N_c = 5.14$

$s_c = 1 + 0.2\,(B/L) = 1 + 0.2 \times (0.25/0.25) = 1.2$

$d_c = 1 + \text{Ö}(0.053\ D/B = 1 + 0 = 1.0$

$N_{cu} = N_c.s_c.d_c = 5.14 \times 1.2 \times 1.0 = 6.168$

$q_o = \gamma h = 0$ [h=0]

$q_f = s_u\,.N_{cu} + q_o = 26 \times 6.168 + 0 =$ **160 kPa**

C-2: Bearing capacity of ground treated by compacted sand column (G-2.1)

(i) Passive pressure condition

$q_u = \sigma_R K_{ps}$

$\sigma_R = \gamma z K_{pc} + 2c\sqrt{}\,Kps$

Data:

$K_{ps} = \tan^2 (45^0 + \phi'/2) = \tan^2 (45 + 34/2) = 3.537$ [$\phi = 34°$]

$K_{pc} = 3.5371$ [Bowles, p-603]

$\gamma = 10.89\ kN/m^3$

$z = 0.365$ m

$c = \frac{1}{2}\ q_u = 0.5 \times 45 = 22.5$ kPa

$q_u = K_{ps} \times [\gamma z K_{pc} + 2c\sqrt{}\,Kps]$

$\mathbf{q_u} = 3.537 \times [10.89 \times .365 \times 3.5371 + 2 \times 22.5 \times \sqrt{3.5371}\,] =$ **349 kPa**

(ii) Based on expansion of a cylinder

$q_u = k_{ps}\,(\sigma_{Ro} + 4c_u)$

$\sigma_R = \sigma'_{Ro} + 4c_u + u = \sigma_{Ro} + 4c_u$

$\sigma'_R = \sigma_{Ro} + c_u\,(1 + \log_e (E/2\,(1+\mu)\,c_u)\,)$

Data:

$K_{ps} = \tan^2 (45^0 + \phi'/2) = \tan^2 (45 + 34/2) = 3.537$ $\quad [\phi = 34°]$

$\sigma_{Ro} = \gamma \times z = 10.89 \times .365 = 3.975$ kPa

$c_u = \frac{1}{2} q_u = 0.5 \times 45 = 22.5$ kPa

$\mathbf{q_u} = k_{ps} (\sigma_{Ro} + 4c_u) = 3.537 \times [3.975 + 4 \times 22.5] = \mathbf{332\ kPa}$

(iii) Based on cavity expansion theory

$\sigma_\theta = \sigma_{ult} = \sigma_R\ N\phi$

$\sigma_R = Fc'\ c_u + Fq'\ q$

Data:

$E = 300$ Kpa [Kaniraj, 152]

$\nu = 0.40$ [Kaniraj, 152]

$Fc' = 3.0$ [From Figure 2.5]

$c_u = \frac{1}{2} q_u = 0.5 \times 45 = 22.5$ kPa

$Fq' = 3.0$ [From Figure 2.5]

$q = \gamma \times z = 10.89 \times .365 = 3.975$ kPa

$N\phi = (1+\sin\phi)/(1-\sin\phi) = (1+\sin 34°)/(1-\sin 34°) = 3.5371$

$Ir = E/ (2\ (1+\upsilon)\ (c + q \tan\phi)) = 300/ (2 \times (1 + 0.40)x(22.5 + 3.975x0.675)$

$Ir = 4.25$

$F'c = \ln Ir + 1 = \ln 4.25 + 1 = 2.45$ [For $\phi = 0$]

$\sigma_R = Fc'\ c_u + Fq'\ q = 2.45 \times 22.5 + 3.0 \times 3.975 = 67.05$ kPa

$\boldsymbol{\sigma_{ult}} = \sigma_\theta = \sigma_R\ N\phi = 67.05 \times 3.5371 = \mathbf{237\ kPa}$

(iv) Based on pile formula

$$q_u = c_u\,(4\,(l/d) + 9)$$

Data:

$c_u = \frac{1}{2}\, q_u = 0.5 \times 45 = 22.5$ kPa

$l = 0.365$ m

$d = 0.75$ m

$\mathbf{q_u} = c_u\,(4\,(l/d) + 9) = 22.5 \times [4 \times (0.365/.075) + 9] =$ **641 kPa**

(v) Hughes et al. (1975) formula

$q_u = K_p\,(4c + \sigma'_r)$

$K_p = \tan^2(45° + \phi/2)$

$\sigma'_r = 2c$

Data:

$c = \frac{1}{2}\, q_u = 0.5 \times 45 = 22.5$ kPa

$K_p = \tan^2(45° + \phi/2) = \tan^2(45° + 34/2) = 3.5371$

$\mathbf{q_u} = K_p\,(4c + \sigma'_r) = K_p\,(4c + 2c) = K_p \times 6c = 3.5371 \times 6 \times 22.5 =$ **478 kPa**

C-3: Bearing capacity of ground treated by compacted sand bed (G-3.1)

<u>Bearing capacity of top layer:</u>

$q_{ult} = cN_cs_cd_c + \bar{q}\,N_qs_qd_q + 0.5\,\gamma\,B'N_\gamma S_\gamma d_\gamma$

$q_{ult} = 7.64 \times 42.14 \times 1.71 \times 1.38 + 0 + 0.5 \times 12 \times 0.25 \times 36 \times 1 \times 1 = 814$ kPa

Bearing capacity of bottom layer:

$q_{ult}'' = cN_c\,(1 + s'_c + d'_c) + \bar{q}$

$q_{ult}'' = 5.14 \times 23\ (1 + 0.2 + 0.4) + 12 \times 0.125 = 191$ kPa

$P_v = \bar{q}\,d_1 + \int_0^{d_1} \gamma h\,dh = 0 + 12 \times 0.125 \times 0.6 = 0.9$ kN/m

$K_s = 1 - \sin\phi = 1 - \sin 34° = 0.44$

$q'_{ult} = q_{ult}'' + \dfrac{pP_vK_s\tan\phi}{A_f} + \dfrac{pd_1c}{A_f}$

$= 191 + (0.785 \times 0.90 \times 0.44 \times 0.675)/0.05 + (0.785 \times 0.125 \times 23)/0.05$

$q'_{ult} = 240$ kPa $< q_{ult}$

Taken **q_{ult} =240 kpa**

C-4: Bearing capacity of ground treated by compacted sand bed with geotextile (G-4.1)

Bearing capacity of top layer:

$q_{ult} = cN_cs_cd_c + \bar{q}\,N_qs_qd_q + 0.5\gamma B'N_\gamma S_\gamma d_\gamma$

$q_{ult} = 7.64 \times 42.14 \times 1.71 \times 1.38 + 0 + 0.5 \times 12 \times 0.25 \times 36 \times 1 \times 1 = 814$ kPa

Bearing capacity of bottom layer:

$q'' = N_c\,(c + \frac{3}{4} B\gamma\tan^2\phi) + \gamma d_1$

$q'' = 42.14 \times (23.5 + \frac{3}{4} \times 0.25 \times 10.89 \times 0.455) + 12 \times 0.125 = 1031$ kPa

$P_v = \bar{q}\,d_1 + \int_0^{d_1} \gamma h\,dh = 0 + 12 \times 0.125 \times 0.6 = 0.9$ kN/m

$K_s = 1 - \sin\phi = 1 - \sin 34° = 0.44$

$$q'_{ult} = q_{ult}'' + \frac{pP_vK_s\tan\phi}{A_f} + \frac{pd_1c}{A_f}$$

$$= 1031 + (0.785x0.90x0.44x0.675)/0.05 + (0.785x0.125x23.5)/0.05$$

$q'_{ult} = 1081$ kPa $> q_{ult}$

Taken **q_{ult} =814 kpa**

C-5: Bearing capacity of ground treated by compacted sand bed with geotextile and compacted sand bed (G-5.1)

The contribution of compacted sand bed with geotextile in improving reconstituted organic ground which treated with compacted sand bed with geotextile and sand column was determined by considering as layered soils:

Bearing capacity of top layer:

$$q_{ult} = cN_cs_cd_c + \bar{q}N_qs_qd_q + 0.5\gamma B'N_\gamma S_\gamma d_\gamma$$

$$q_{ult} = 7.64x42.14x1.71x1.38 + 0 + 0.5x12x0.25x36x1x1 = 814 \text{ kPa}$$

Bearing capacity of bottom layer:

$$q'' = N_c\left(c + \frac{3}{4}B\gamma\tan^2\phi\right) + \gamma d_1$$

$$q'' = 42.14x\left(21.5 + \frac{3}{4}x0.25 \times 10.89 \times 0.455\right) + 12 \times 0.125 = 947 \text{ kPa}$$

$$P_v = \bar{q}d_1 + \int_0^{d_1}\gamma h dh = 0 + 12 \times 0.125 \times 0.6 = 0.9 \text{ kN/m}$$

$$K_s = 1 - \sin\phi = 1\text{-}\sin 34° = 0.44$$

$$q'_{ult} = q_{ult}'' + \frac{pP_vK_s\tan\phi}{A_f} + \frac{pd_1c}{A_f}$$

$$= 947 + (0.785x0.90x0.44x0.675)/0.05 + (0.785x0.125x21.5)/0.05$$

$q'_{ult} = 993$ kPa $> q_{ult}$

Taken **q_{ult} =814 kpa**

The contribution of compacted sand column in improving reconstituted organic ground which was treated with compacted sand bed with geotextile and sand column was determined by using cavity expansion theory:-

$$\sigma_\theta = \sigma_{ult} = \sigma_R \ N\phi$$

$$\sigma_R = Fc' \ c_u + Fq' \ q$$

Data:

$E = 300$ Kpa [Kaniraj, 152]

$\nu = 0.40$ [Kaniraj, 152]

$Fc' = 3.0$ [From Figure 2.5]

$$c_u = \frac{1}{2} q_u = 0.5 \times 43 = 21.5 \text{ kPa}$$

$Fq' = 3.0$ [From Figure 2.5]

$q = \gamma \times z = 10.89 \times .52 = 5.66$ kPa

$N\phi = (1+\sin\phi)/(1-\sin\phi) = (1+\sin 34°)/(1-\sin 34°) = 3.5371$

$Ir = E/(2\ (1+\upsilon)\ (c + q \tan\phi)) = 300/\ (2 \times (1 + 0.40) \times (21.5 + 5.66 \times 0.675)$

$Ir = 4.23$

$F'c = \ln Ir + 1 = \ln 4.25 + 1 = 2.44$ [For $\phi = 0$]

$\sigma_R = Fc' \ c_u + Fq' \ q = 2.44 \times 21.5 + 3.0 \times 5.66 = 69.44$ kPa

$\boldsymbol{\sigma_{ult}} = \sigma_\theta = \sigma_R \ N\phi = 67.05 \times 3.5371 =$ **246 kPa**

So, the bearing capacity of the reconstituted organic ground treated by compacted sand bed with geotextile and compacted sand column = 814+246 =**1060 kPa**