PERFORMANCE STUDY OF RAMMED AGGREGATE PIER AS A GROUND IMPROVEMENT TECHNIQUE IN SOFT GROUND

by

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A thesis submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering

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Declaration

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

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Md. Julfikar Hossain
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Affectionately dedicated

To

my beloved mother, for her love

and

my elder brother and younger sister, for their continuous support and encouragement.

ABSTRACT

The development of modern foundation practices, namely ground improvement techniques, has been proved to be viable both the technically and economically for the improvement of marginal sites and to overcome the limitations of conventional foundation systems. Amongst the various ground improvement techniques for improving soft ground conditions, Rammed Aggregate Pier (RAP) such as Geopier is considered as one of the most versatile and innovative ground improvement method than the other methods. The performance of Rammed Aggregate Pier has yet not been examined in soft ground condition of Bangladesh at field level.

This study is concerned with the performance of Rammed Aggregate Pier in soft ground at a selected site of South-West region of Bangladesh i.e. KUET (Khulna University of Engineering & Technology) campus. The ground at the site consists of soft fine-grained soil up to great depth with a layer of organic soils at 4.5 to 9m depth from the existing ground surface. The Rammed Aggregate Piers were installed by rammed method with locally fabricated equipments. This installation method is easier and cost effective than other counterpart. Rammed Aggregate Pier of cylindrical shape having 0.75m diameter and 3.4m length were installed manually in three arrangements as single, double and group. A uniform mixture of local sand and brick aggregates at the proportion of 1:2 was used as the granular materials maintaining saturated surface dry condition. The granular materials were poured into the excavated hole in layers and hence compacted adequately by using a hammer of 108kg and a free fall height of 600mm. Load tests on full-size isolated square footing of 1.68x1.68m resting at a depth of 0.75m from the existing ground surface were conducted on both the natural and improved ground by using the method similar to pile load test. The field measurement shows that the ultimate bearing capacity of footing resting on single, double and group RAP treated ground can be increased by 1.5, 1.8 and 1.96 times, respectively, comparing to that of natural ground. Field investigation reveals that the RAP made-up of locally available granular materials and the employed installation technique can be used successfully as a suitable ground improvement method to improve the bearing capacity of such soft ground.

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Nomenclature

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

A	:	area
a_s	:	area replacement ratio
a_v	:	coefficient of compressibility
B	:	width
β	:	inclination of the failure surface as given by equation
C_c	:	compression index
C_t	:	temperature correction
C_v	:	coefficient of consolidation in the radial distance
D	:	grain size, depth, diameter, distance
E	:	modulus of elasticity of the clay
e	:	void ratio
e_o	:	initial void ratio
G_s	:	specific gravity of soil particles
H	:	height, thickness
H_1, H_2	:	height
k	:	coefficient of permeability
k_{ps}	:	coefficient of passive resistance of granular material
L	:	length, distance
W_l	:	liquid limit
m_v	:	coefficient of volume compressibility
OMC	:	optimum moisture content
OC	:	organic content
p	:	pressure, load
Wp	:	plastic limit
Ip	:	plasticity index
q_u	:	unconfined compressive strength
r	:	radius

S_r	:	degree of saturation
s_u	:	undrained shear strength
T_v	:	time factor
t	:	time
u	:	pore pressure
W	:	weight of soil sample
w	:	moisture content
W_d	:	dry weight
z	:	depth of the clay
γ	:	bulk density, unit weight
γ_c	:	saturated or wet unit weight of the cohesive soil
γ_d	:	maximum dry density
$\Delta\sigma$	:	deviator stress
σ_3	:	average lateral confining pressure
σ_c	:	stress in the surrounding cohesive soil
σ_r	:	passive resistance of the soil
ϕ	:	angle of shearing resistance
μ	:	poisson's ratio
ε_a	:	axial strain

CHAPTER ONE

INTRODUCTION

1.1 General

The sub-soil of Khulna region consists of soft fine-grained soil. This region is situated at the south-western part of Bangladesh. The organic soil layer exists in most of the place within the depth of 10 to 25 ft below the existing ground surface of this sub-soil. Moreover, the nature, the organic contents and geotechnical properties of this soil vary from place to place. The soil is also erratic in nature both in the vertical and horizontal directions and the bearing capacity of fully decomposed organic soil is very low and always deals to adopt a costly foundation for the construction of structures.

The traditional practice of these regions is to transfer the structural load to the hard layer through conventional pile foundation or to use other deep foundation. But this foundation system is very costly for the construction of infrastructure in the projects at marginal site. The Geopier foundation system, a common type of Rammed Aggreate Pier, is one of the few soil improvement methods which are widely used for ground improvement techniques in several projects through the world. The soil improvement technique has proven record in solving such problems economically and technically. Rammed Aggregate Piers (RAPs) foundation can safely carry significant lateral and up lift forces. This system is an innovation ground improvement method. RAPs are installed creating cylindrical holes in the ground by auguring or excavation, and filling the cavities with highly densified granular material, which are compacted using high energy impact temper

Amongst the various ground improvement technique, construction of RAP is considered recently as one of the cost effective and versatile foundation solutions for its proven records of effectiveness in improving weak soil deposits. This Ground Improvement method was developed 1980 that has grown in the United States and more recently in Asia and Europe for supporting lightly to heavily loaded structures. For construction of low rise building, piling or other deep foundations are applied at very soft fine-grained soil which foundation cost is very high. In this case as an alternative solution, construction of RAP foundation is found most economic than other type of foundation.

This study concern about the Rammed Aggregate Pier constructed in very soft fine-grained soil as encountered in KUET campus situated at Khulna region. RAPs were installed in several conditions and then its effectiveness was investigated through Real Footing load test resting on the Rammed Aggregate Pier improved ground.

1. 2 Background of this Study

The Rammed Aggregate Pier system is an innovative ground improvement method developed in the 1980's that has grown in the United States and more recently in Asia and Europe, for supporting lightly to heavily loaded structures and highway and railroad embankments. The system is unique because it prestresses and prestrains the adjacent matrix soils during installation of rammed aggregate piers. It has been successfully used on hundreds of project sites to support building foundations, floor slabs storage tanks, and road way embankments founded on both poor and unsuitable soils as well as fair to good soils. There are many case histories available around the world about the specialized applications of Geopier such as, Wind tower projects in Germany, where the Rammed Aggregate Pier system provides high bearing capacity and overturning moment resistances to support the foundations in soft soils; and Rammed Aggregate Pier soil reinforcement support of foundations and large area floor slab system for a commercial warehouse facility in the Philippines .The new soil improvement system tailored to increase foundation bearing capacities for dynamic footing loadings and provide positive settlement control for wide area loads including floor slab .

In the past five years, a number of projects on peat and highly organic subsoil sites ranging from a four-story bank headquarters building in the Cayman Islands, warehouse project in Salem, Oregon which was a large, single story, warehouse facility with total footprint area of about 11,150 sqm (12,000 sft) and two to four story residential and commercial structures built in the United Stated have been successfully and economically supported on a RAP system. In this system, very stiff and highly densified, short aggregate pier elements are installed in cavities made within the very soft peat and highly organic soil layers. Although the peat soils are penetrated by the pier cavities, the pier bottoms often terminate on soft and compressible, underlying inorganic soils. The resulting composite bearing material of stiff piers and adjacent, soft matrix soils is substantially stiffer than the unimproved matrix soil. Generalized construction and design methods for RAPs systems are described by Lawton, et al (1994) and Wissmann & Fox (2000).

In Bangladesh many other ground improved technique have been used in ground improvement project Alamgir and Zaher (1999a and 1999b) reported that a large number of sand piles were installed to improve the soft cohesive soils in south-western region of Bangladesh in which a six-vent regulator was constructed. Soft fine-grained soil with significant organic content dominates the sub-soil of Khulna region, which often creates problem to the geotechnical engineers to select suitable economic foundations for structures due to low shear strength and high compressibility (Alamgir et al. 2001). Recently some ground improvement techniques including granular columns have been employed successfully in this region. The performance of geotextile-reinforced footing, sand compaction piles, stone columns and granular piles have also been studied in this region at field level (Haque 2000, Zaher 2000, Alamgir and Zaher 2001, Haque et al. 2001 and Sobhan 2001). But the Rammed Aggregate Pier system is an innovative ground improvement method which is most economy than other ground improvement techniques. This method is used most suitable for low to high rise building on very soft cohesive soil to loose deposits in Bangladesh.

1.3 Objective and Scope of this Study

To achieve the desired goal, the main objectives of this study can be listed as the following:

i. To identify the advantages and disadvantages of the installation of Rammed Aggregate Pier (RAP) system for the improvement of soft ground.

ii. To observe the load-settlement behavior of improved ground for the cause of group, double and single RAPs foundation system.

iii. To observe the group effect of RAPs treated ground comparing the load carrying capacity with that of single counterpart.

iv To determine the degree of improvement of the bearing capacity of soil due to the installation RAPs by comparing the load –settlement beheviour of footing resting on untreated and treated ground.

This ground improvement method can be used successfully using locally available granular materials and installation technique. Once the field performance shows the increasing trend of the bearing capacity of this soft ground due to the installation of RAP, this technique can be established. Since this installation technique is simple, manual labour oriented and required instrument is available locally, the practicing geotechnical engineers can take decision about the use of this ground improvement technique and can suggest the client to adopt this technique to improve the soft ground and construct the structure on it. This study will be very helpful to verify the applicability of this method in case of a typical soft fine-grained soil exists in Khulna regions.

1.4 Organization and Thesis Outline

The organization and outline of this works as appeared in this dissertation is illustrated in Figure 1.1

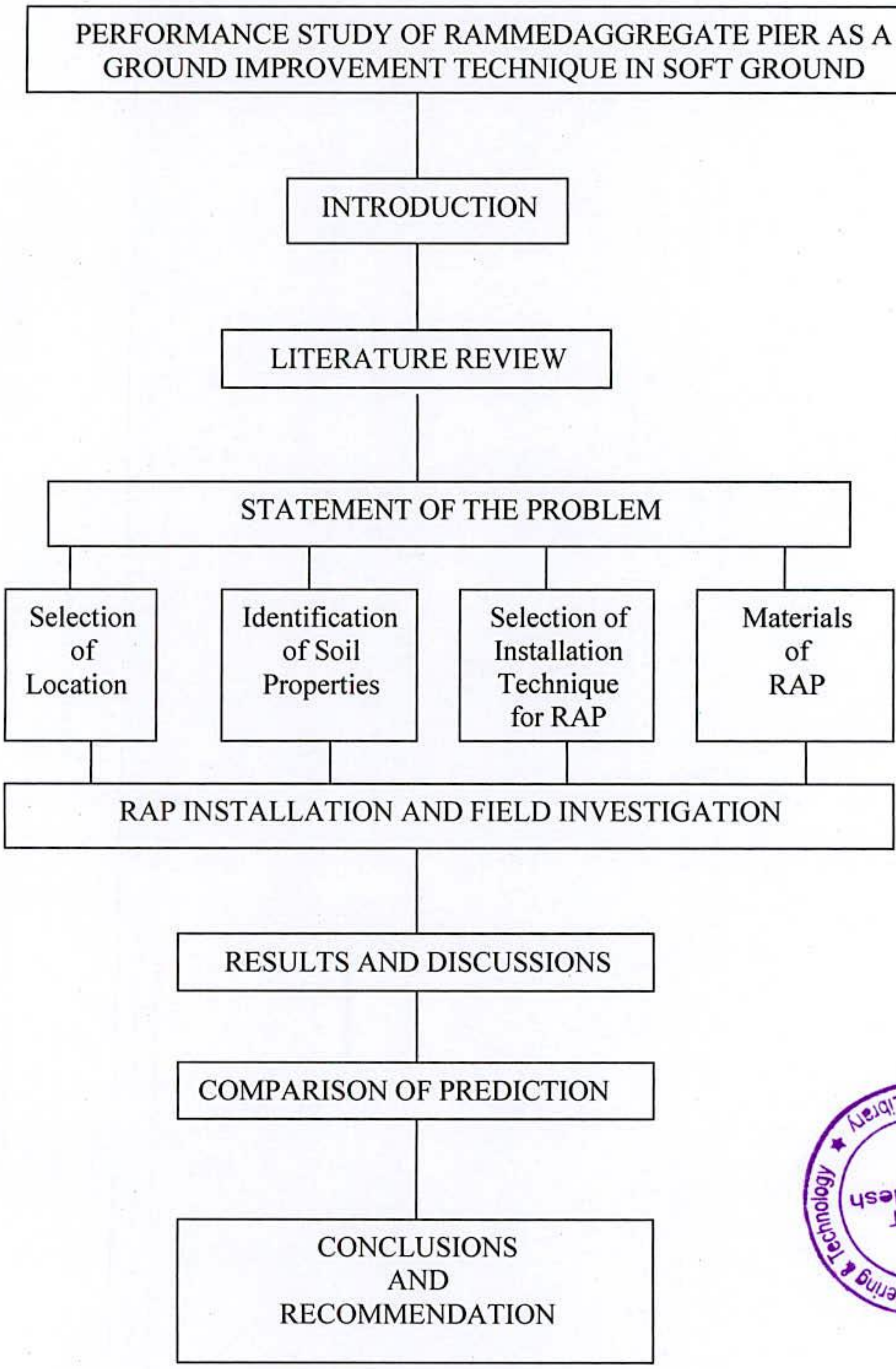


Figure 1.1 Diagram of the thesis outline

CHAPTER TWO

LITERATURE REVIEW

2.1 General

The Rammed Aggregate Pier (RAP) such as Geopier/ Stone Columns/ Granular Piles system consisting of granular materials compacted in cylindrical holes, have been used as a technique for improving the bearing capacity, reduce settlement, increase the time of consolidation, improve stability and resistance of liquefaction of soft ground since 1980's. A brief account of historical development has been presented previously in section 1.2. In the modern phase of the use of RAP and similar inclusions, the theoretical background, analysis and design aspects and installation techniques have been developed by various researchers and this method of ground improvement is being used extensively throughout the world for site improvement. Amongst the various techniques for improving in-situ soft ground conditions, rammed aggregate pier foundation are consider as one of the most versatile and cost effective in ground improvement techniques.

They are ideally suitable for the improvement of the soft clays, silts and also for loose soil deposits. This chapter describes about the soft ground, ground improvement techniques, rammed aggregate pier foundation and the relevant topics. This chapter also described the existing methods of evaluation of load carrying capacity on various types of RAP foundation system. The present state-of-the art of the use of rammed aggregate pier foundation for the improvement of soft ground are specially described.

2.2 Soft Ground

The term 'soft ground' has been used broadly, however, so far its meaning as engineering or technical term has not been defined clearly. In general, the following soil types are considered as the soft ground; (i) soft 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.

2.3 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 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 cohesionless 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 q_c (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) q_c 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			

2.4 Foundation Practice in Soft Ground

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

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 19^{th} 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 (Broms 1987):

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 freezing or by mixing the soil with lime, cement or other chemicals; (a) lime or cement columns, (b) Electro-osmosis and (c) freezing.

iv. Structural methods: where structural elements such as geofabric, piles ere used.

v. Sand, gravel or stone are used to reinforce the soil or to transfer the load to an underlying less compressible stratum or layer; (a) Geofabrics and geomembranes, (b) Excavation and replacement (c) Soil displacement, (d) Heavy tamping /Dynamic consolidation, dynamic replacement, and mixing, (e) Jet grouting, (f) Stone, gravel or sand columns, (g) Embankment piles, (h) Soil nailing and (i) Geopier Rammed Aggregate piers.

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

2.5.1 Preloading

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

2.5.2 Deep densification of cohesionless soils

In-situ deep densification of loose cohesionless soil layers is usually done by dynamic methods. In many methods, dynamic loading is accompanied by displacement in the form of the insertion of a probe and/or construction of a sand or gravel column in-situ methods used for the in-situ deep densification of cohesionless soils include blasting, vibro-compaction, heavy tamping. Vibro-compaction includes all those methods involving the insertion of vibrating probs into the ground with or without the addition of a backfill material.

2.5.3 Densification of soft soils

Settlement resulting from the long-term consolidation of cohessionless 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

Of the method of soil improvement and ground strengthening, none have been so intensively suited and advanced in application in the past several years, as has soil reinforcement. Basically 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.

2.5.6 Stone column

The concept involves replacement of 10 to 30 percent of the weak soil with stone or sometimes with sand in the form of columns. Holes are created in the ground and then back filled with stone compacted by impact and vibration. Soils are thus, transformed into a stiffer composite mass of granular cylinder with intervening native soil providing lower overall compressibility and higher shear strength. Today, stone column/ granular piles have been used mainly to improve the bearing capacity and reduce the settlement of foundations or to improve the stability of embankment and slopes.

2.5.7 Rammed Aggregate Pier

Rammed Aggregate Pier systems are the only intermediate foundation system in existence. They constitute an excellent alternative to piles, caissons, over excavation/replacement filling, surcharge, and other foundation support approaches. Rammed Aggregate Pier elements are densely compacted aggregate piers that improve the soils in which they are installed. The piers are constructed equipment in pre-drilled cavities (usually 30"diameter) and can resist both compression loading and uplift forces. Thousands of structures are currently supported by the rammed aggregate pier system – proven experience that ensures high levels of performance and reliability compared to traditional systems.

2.6 Methods 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 (i.e. stone columns, sand compaction piles or Rammed Aggregate Pier foundation), by admixtures (i.e. by deep mixing method) and by dewatering (i.e. vertical drains) 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. To install columnar inclusions (stone columns, granular piles, sand compaction piles, lime/cement column, rammed aggregate pier, etc.) 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 to ground improvement technique. The selection of the most suitable one in any case can only be made after evaluation of several factors specific to the problem at hand (Zaher 2000).

A flow chart for selection of ground improvement techniques are given in Figure 2.1.

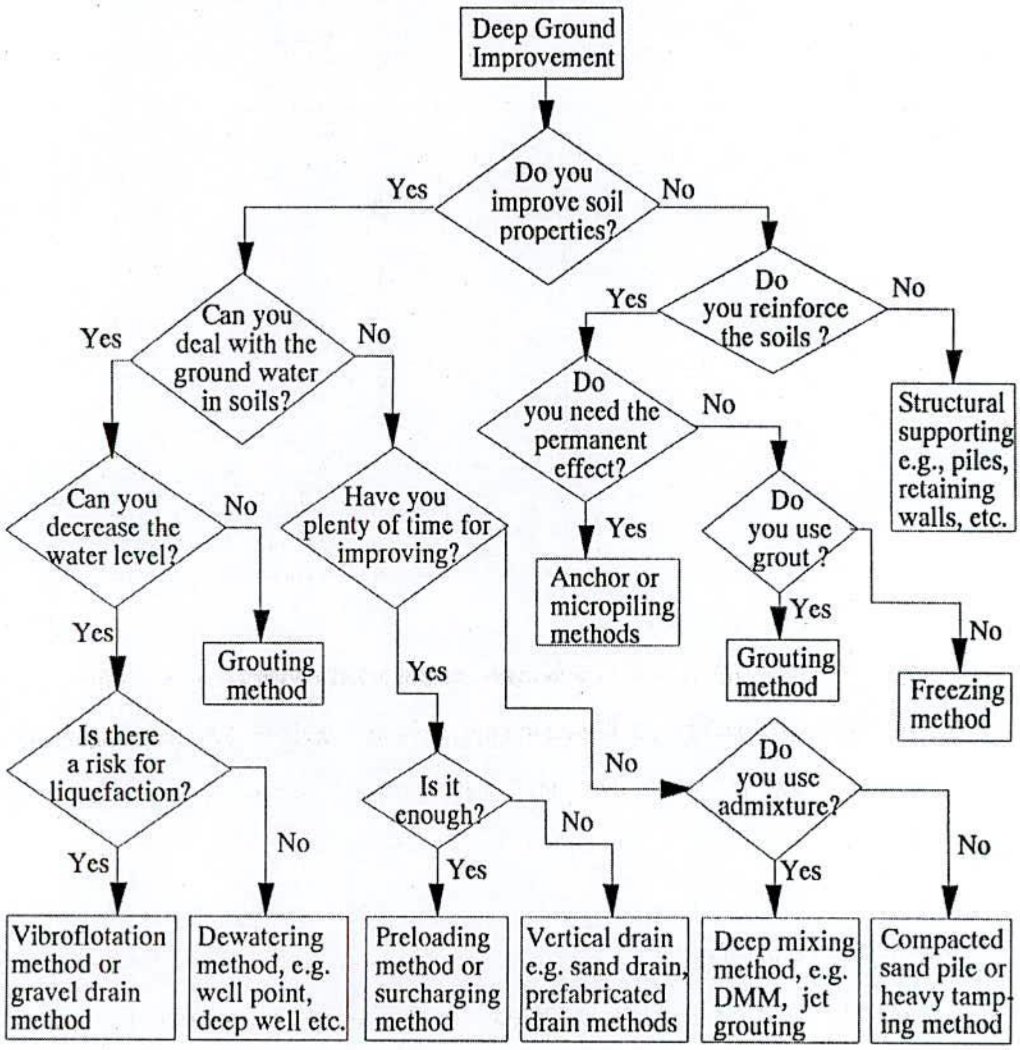


Fig. 2.1. Chart for selection of ground improvement techniques (After Bergado and Miura 1994)

2.7 Rammed Aggregate Pier Foundation

Rammed Aggregate Pier has become a common ground improvement technique for improving the marginal sites. Rammed aggregate pier methods have used successfully other country for some ground improvement projects. The performance of this technique is required to investigate further in details in local condition. The Rammed Aggregate Pier (RAP) system uses of reinforce good to poor soils, including soft to stiff clay and silt, loose to dense sand, organic silt and peat and variable, uncontrolled fill.

During earthquake loadings, RAP-supported foundation systems are designed to behave similar to shallow foundations but exhibit greater bearing capacities and greater resistance to lateral forces. When anchors are incorporated in to the rammed aggregate pier elements, uplift resistance is provided. Additionally, the installation of rammed aggregate pier elements should provide for a substantial reduction in the potential for liquefaction within the RAP - enhanced soil layer.

Rammed Aggregate Pier soil reinforcement is used for support of transportation structures including Mechanically Stabilized Earth (MSE) retaining walls and large embankment fills. The installation of stiff Rammed Aggregate Pier elements provides a significant increase in the composite stiffness of otherwise soft and compressible foundation soils. Rammed Aggregate Pier construction using open-graded stone affords radial drainage to the elements. The result of RAP installation is a significant decrease in both settlement magnitude and duration with the Rammed Aggregate Pier-reinforced zone.

Rammed Aggregate Pier construction is described in the Geopier Reference Manual(Fox and Cowell) and in the literature (Lawton and Fox 1994, Lawton et.al. 1994). The elements are constructed by drilling out a volume of compressible soil to create a cavity and then ramming select aggregate into the cavity in thin lifts using a patented beveled tamper. The ramming action causes the aggregate to compact vertically as well as to push laterally against the matrix soil, thereby increasing the horizontal stress in the matrix soil and reducing the compressibility of the matrix soil between the elements. Rammed

Aggregate Pier construction results in a very dense aggregate pier with a very high stiffness that yields a significantly increased composite stiffness within the RAP-reinforced zone. The use of open-graded stone during construction affords radial drainage of excess pore water pressure to the elements, which act as vertical drains to increase the time-rate of settlement. Figure 2.2 (a) and (b) shows geopier installation process and geopier installation below structure's foundation.

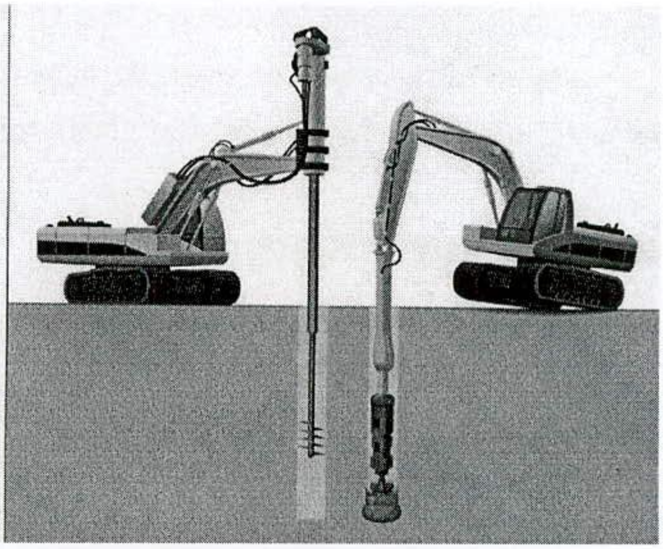


Fig.2.2 (a) Rammed Aggregate Pier installation process before structure's foundation (After Geopier 2005a)

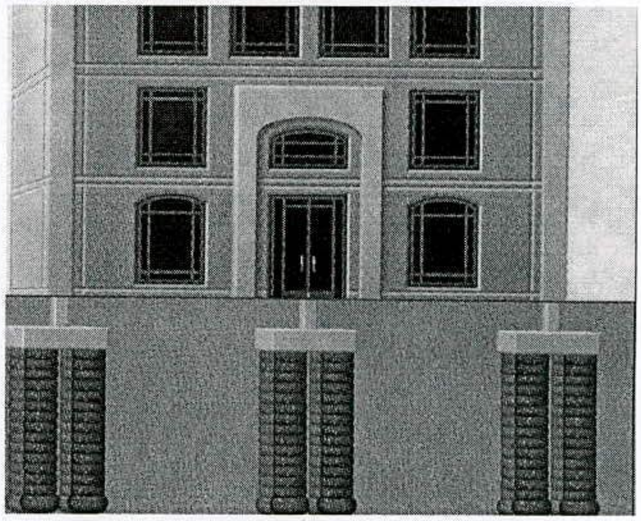


Fig.2.2 (b) Rammed Aggregate Pier installation below structure's foundation (After Geopier 2005a)

2.7.1 Type of Rammed Aggregate Pier foundation

Treatment of soft or weak compressible soils by Rammed Aggregate Pier involves providing at the ground surface a dense gravel bed as a working platform and a drainage layer acting as a stiff raft. The response of the system is shown to depend on the relative stiffness of the gravel bed. Consider this action RAPs are classified as two types (Fig.2.3).

1. RAP Single or Double pattern
2. RAP Group pattern

RAP Single or Double pattern: To improve poor soils, including soft to stiff clay and silt, loose to dense sand, organic silt and peat and variable, uncontrolled fill, where structural foundation were supported isolated is used Single or double Rammed Aggregate Pier. In this case, super structural load is considered to design and construction of single or double RAP foundation.

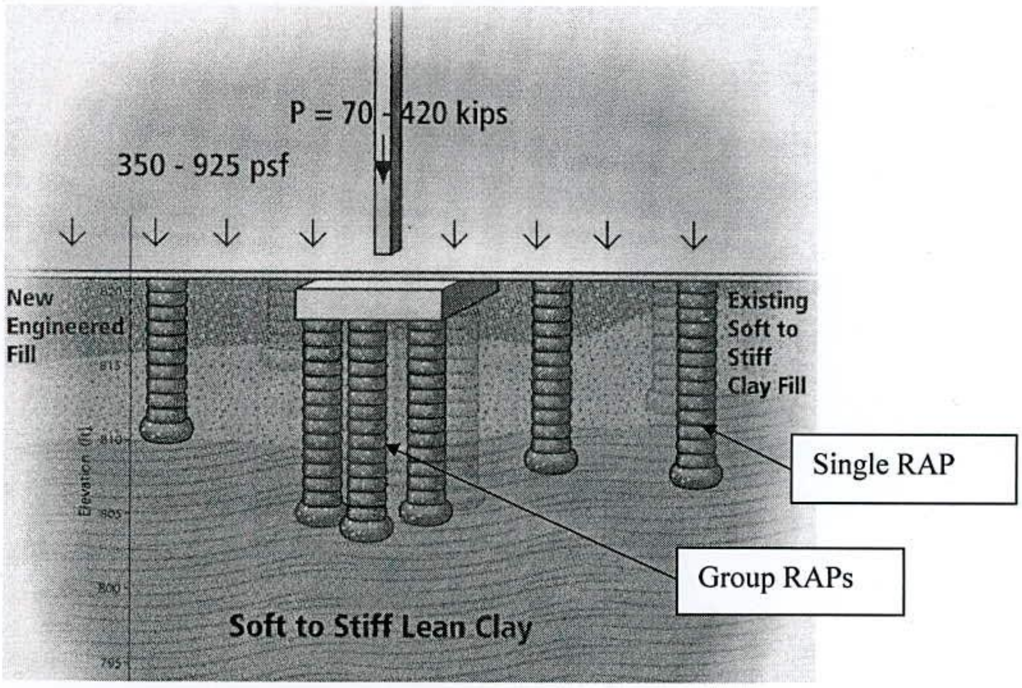


Fig.2.3 Section of single or double and group RAP (After Geopier 2006)

RAP Group pattern: To improve poor soils, including soft to stiff clay and silt, loose to dense sand, organic silt and peat and variable, uncontrolled fill, where structural foundation were supported isolated, raft, continuous footing and other structure is used group Rammed Aggregate Pier system. In this case, super structural load is considered to design and construction of group Geopier foundation. In generall, group rammed aggregate pier system is used as heavy loaded structural foundation.

2.7.2 Rammed Aggregate Pier installation techniques

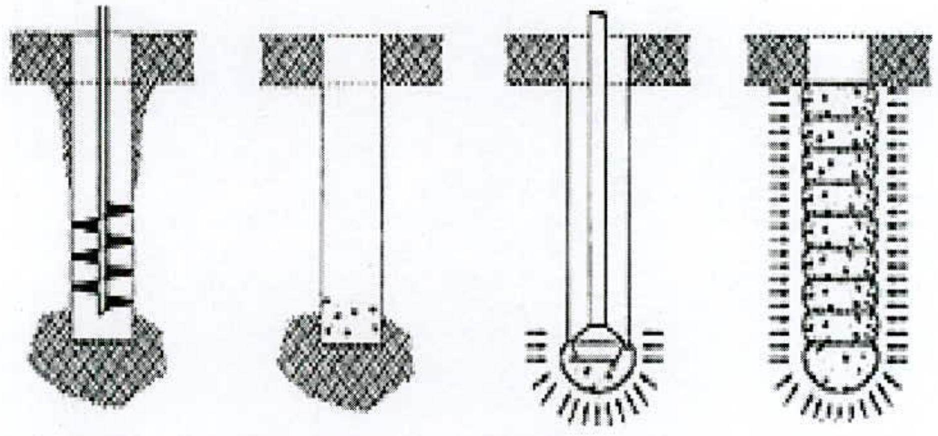
Replacement of Rammed Aggregate Piers (RAPs) for reinforcing good to poor soils, including soft to stiff clay and silt, loose to dense sand, organic silt, peat and variable, uncontrolled fill. The unique installation process utilizes pre-auguring and vertical impact ramming energy to construct RAPs, which exhibit unsurpassed strength and stiffness. RAP solutions are designed to provide superior total and differential settlement control and increased bearing support to meet project requirements. This system is also called three-step process.

1. Rammed Aggregate Piers are first involved drilling a cavity which drill diameter normally range from about 450mm to 900mm and drill depths range from 2 to 9m, depending on design requirements (Geopier 2003). Pier cavities are typically excavated by conventional drilling techniques, using either truck-mounted auguring equipment or "dangle drill" equipment mounted on an excavator or crane. Rammed Aggregate Pier elements can be constructed below ground water in all soils ranging from peat to loose clean sands to soft clays. Pre-drilling allows the physical investigation of soil between the borings, ensuring that the piers are engineered to reinforce the right soils.

2. Layers of aggregate are then introduced in to the drilled cavity in thin lift of 300 mm compacted thickness or to use follow with Standard Proctor Test method. Aggregate used for pier construction is typically high quality crushed rock, such as used for highway base course construction. For liquefaction mitigation, free-draining aggregate can be used so the Rammed Aggregate Pier element also functions as a drain to relieve excess pore water pressures.

3. A patented beveled tamper rams each layer of aggregate using vertical impact ramming energy, resulting in superior strength and stiffness. The tamper densifies aggregate vertically and forces aggregate laterally in to cavity sidewalls. Within 15 seconds of tamping, a lift can receive over two times the compactive energy that is put into the maximum density laboratory test (ASTM1557). This results in excellent coupling with surrounding soils and reliable settlement control.

Since Rammed Aggregate Pier elements are constructed in pre-excavated cavities, there is essentially no remolding of the surrounding soils, as occurs with other stone column techniques that involve complete soil displacement. Hence, with the Rammed Aggregate Pier technique the surrounding soils cannot experience strength loss due to the construction methods, but rather gain a significant increase in stiffness as each 12" thick lift of aggregate fill is rammed.



1. Make Cavity.	2. Placed washed stone at bottom of cavity.	3.Make a bottom bulb.	4. Make a complete Geopier and build up lateral soil pressures in matrix soil during shaft constructions.

Fig.2.4 Three step installation process of Geopier (After Geopier 2003)

By constructing Rammed Aggregate Pier elements in clusters spaced from about 1½ to 3 diameters apart, the Rammed Aggregate Pier-reinforced soil mass experiences significant permanent prestressing, which greatly improves its strength and consolidation characteristics (extending several feet beyond the outside piers). Hence, the so-called "group effect" is very desirable for the Rammed Aggregate Pier system because it improves performance (whereas, in the design of pile foundations the group effect is normally avoided because it tends to reduce individual pile capacities).

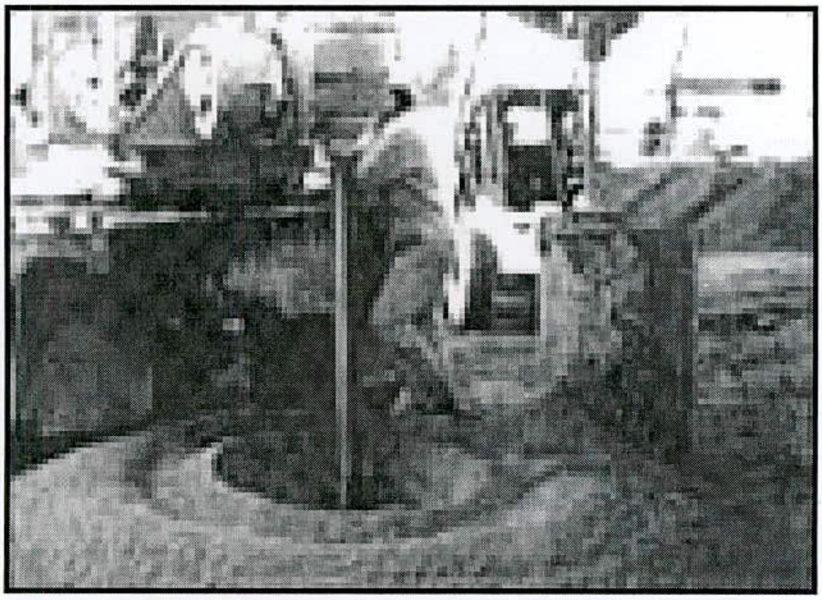


Fig. 2.5 Mechanically drilling process of Rammed Aggregate Pier (After Geopier 2003).

Rammed Aggregate Pier are used following installation, RAPs reinforce slopes and embankments, support shallow foundations, floor slabs and tank pads. The footing stresses are attracted to the stiff RAPs, resulting in engineered settlement control.

2.8 Modes of Failure

The Rammed Aggregate Pier system is as like Similarly of Granular piles which also may be called one kinds of stone column. The modes of failure of RAPs are same as stone column. This foundation system may suffer failure in a number of modes. They are described in the followings.

i) Bulging: This type of failure as shown in Fig.2.6a may be attributed to the plastic failure of an expanding cylindrical cavity.

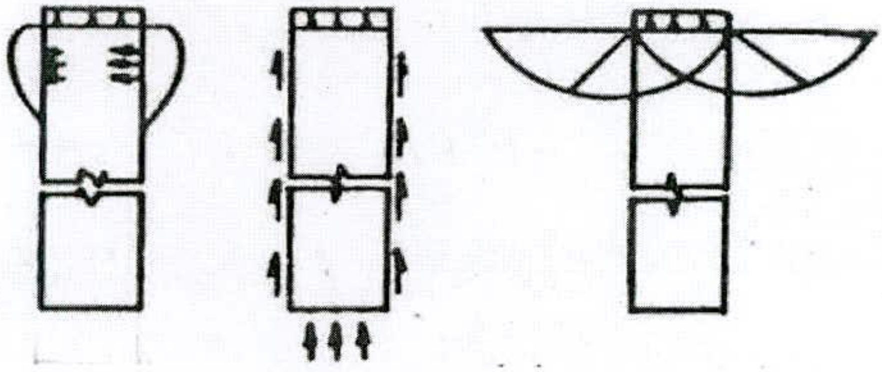
ii) Pile type failure: Failure may occur by shear failure in end bearing or in skin friction as in case of conventional piles as shown in Fig.2.6b.

iii) General shear failure: Failure may occure in a shallow footing with Stone column, Rammed Aggregate Pier and providing other additional support as shown Fig.2.6c.

Hughes et. al (1975) have showed that the first mode of failure is the most common one (Fig.2.7). The experimental results showed clearly that the ultimate strength of an isolated column loaded at its top only, is governed primarily by the maximum lateral reaction of the soil round the bulging zone and that the extent of vertical movement within the column is limited. Their experiments using radiographic method revealed that the bulging of the pile occurs near the top at a depth approximately equal to half to one diameter of the pile, as the lateral confinement is minimum there. The radial deformation decreases with depth and appears to the negligible beyond a depth greater than twice the diameter of the pile.

In a conventional pile failure, against by skin friction and/ or end bearing only the equilibrium of the vertical forces on the Rammed Aggregate Pier or column is considered. Clearly, if the vertical load exceeds, the shear resisting forces along the side of the column and the ultimate bearing pressure at the base, the Rammed Aggregate Pier or column will push through the soil. For simplicity the limiting value of the shear stresses along the side of the column are taken to be equal to the undrained shear strength of the soil.

Madhav and Vitkar (1978) proposed a general shear failure type mechanism for stone column. In practice, the failure of such category can be avoided by taking advantage of increase of soil stiffness and strength with depth and by replacing the surface layer of weak soil by a well compacted granular material or by covering the soft soil with a pad of granular material.



(a) Bulging (b) Pile action (c) General shear

Fig. 2.6 Modes of failure (after Hughes et al, 1975)

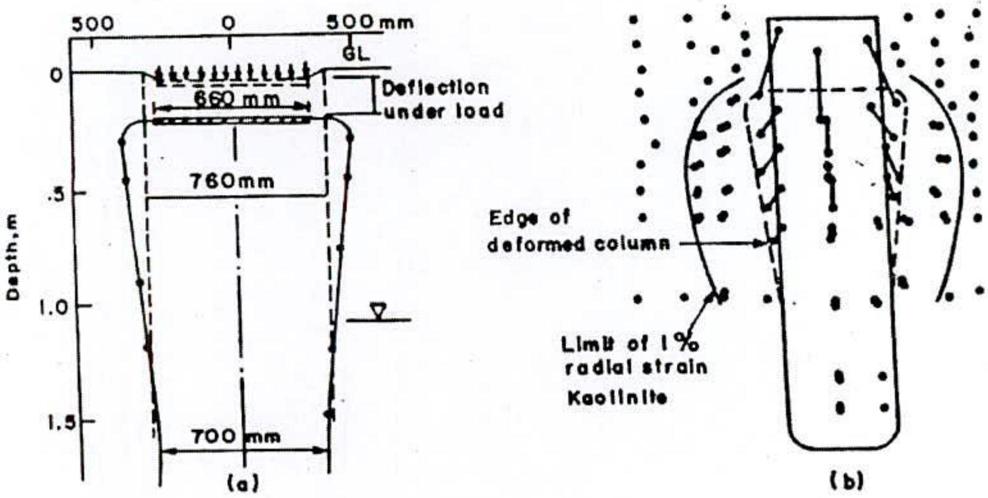


Fig. 2.7 Measured shape and deflections (after Hughes et al, 1975)

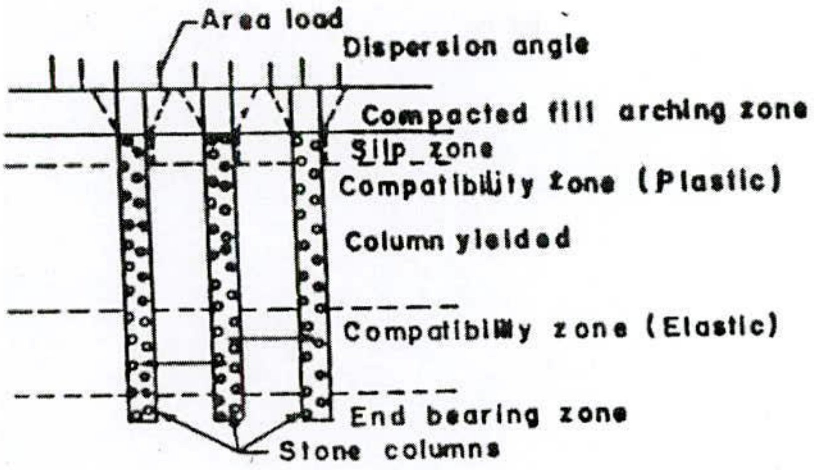


Fig.2.8 Zone categorization

In a bulging type failure when the stone column yields, four zones may develop (Datye, 1982). There are, (i) arching zone, (ii) slip zone, (iii) compatibility zone with plastic and elastic condition and (iv) load transfer zone.

(i) Arching zone: When load is applied on the stone column, load transfer takes place by arching of the soil. Depending on the strength of soil, diameter of the stone columns and the depth of the stiff soil layers, The dispersion angle will vary (Fig 2.8). During installation and settlement, tension cracks may develop in cohesive brittle strata. It is, therefore, advisable to rely only on the 'pad' of well compacted sandy or gravelly soil for load distribution.

(ii) Slip zone: This zone is comparatively of small depth. The slip may not occur, if top layer is of comparatively high strength due to desiccation or over consolidation.

(iii) Compatibility zone: This zone may consist of plastic and elastic sub zone. (a) Plastic sub zone: If the load transfers to the stone column exceed the yield limit, the region can be defined as plastic sub zone. (b) Elastic sub zone: Below plastic zone, all the settlements are elastic in nature and compatibility is achieved by the load redistribution which depends on the relative compressibility of stone column and soil.

(iv) Load transfer zone: If the lower layers are of stiffer material with sufficient shear strength, the settlements in this portion are elastic as the column does not yield.

2.9 Methods of Evaluation of Load Carrying Capacity on Natural Ground

One of the early sets of bearing capacity equations was proposed by Terzaghi (1943) as shown in equation 2.2 but these equations are similar to Eq.2.1. Terzaghi used shape factors noted when the limitations of the equations were discussed. Terzaghi's equations were produced from a slightly modified bearing-capacity theory developed from using the theory of plasticity to analyze the punching

$$q_{ult} = cN_c + \bar{q}N_q + \gamma BN_\gamma \qquad 2.1$$

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5BN_\gamma s_\gamma \qquad 2.2$$

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

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

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

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

Where, value of s_c and s_γ are:

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

Table 2.2 Bearing-capacity factors for the Terzaghi equations

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

*$N_c = 1.5\pi+1$. [See Terzaghi (1943),p. 127.]

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

2.10 Existing Methods of Evaluation of Load Carrying Capacity on RAP Treated Ground

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

2.10.1 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.

$$\sigma_R = \gamma\, z\, Kpc \;+2c\, \sqrt{Kps} \qquad 2.3$$

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

$$q_u = \sigma_R\, Kps \qquad 2.4$$

Where, $Kp_s = \tan^2 (45^0+\varphi'/2)$; φ' = angle of shearing resistance of the granular material

2.10.2 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.

$$\sigma'_R = \sigma_{Ro} + c_u\,(1+\log_e (E/2\,(1+\mu)\, c_u)\,) \qquad 2.5$$

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,

$$\sigma_R = \sigma'_{Ro} + 4c_u + u = \sigma_{Ro} + 4c_u \qquad 2.6$$

Ultimate stress, $q_u = k_{ps\,(}\sigma_{Ro} + 4c_{u)}$

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

2.10.3 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.

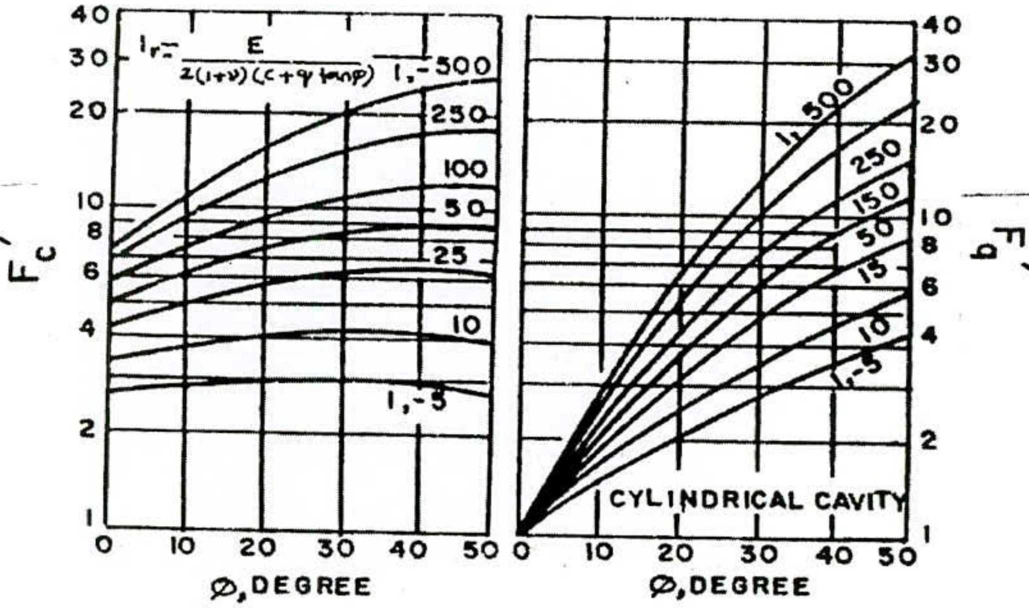


Fig.2.9 Cavity expansion parameters(After Vesic,1972)

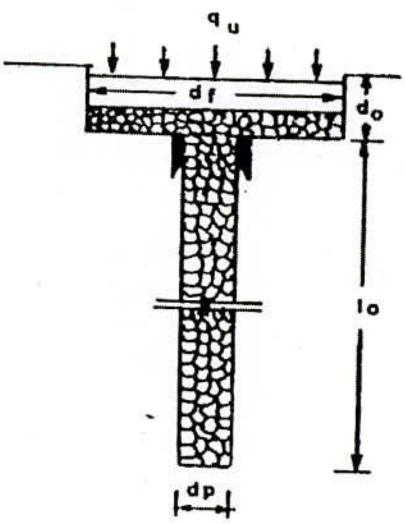


Fig.2.10 Definition sketch.

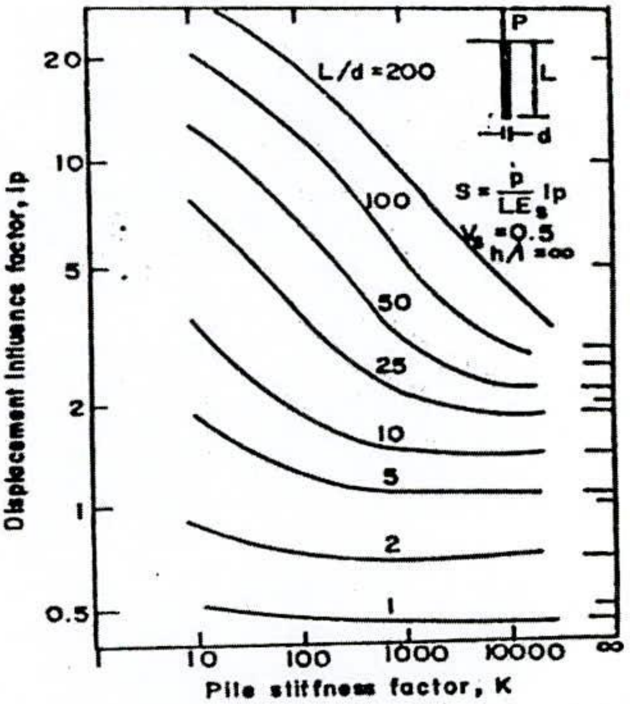


Fig.2.11 Displacement influence factors.

$$\sigma_R = Fc' \; c_u + Fq' \; q \qquad 2.7$$

$$\sigma_\theta = \sigma_R \; N\phi \qquad 2.8$$

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:

$$F'q = (1+\sin\phi \; (Irr \sec\phi)^{\frac{Sin\phi}{1+Sin\phi}} \qquad 2.9$$

$$F'c = (F'q - 1)\cot\phi \qquad 2.10$$

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.

2.10.4 Based on Pile Formula

The ultimate load carrying capacity of stone column or Geopier 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.

$$q_u = c_u\ (4\ (l/d\ ) + 9\ ) \qquad 2.11$$

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. 2.9, 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.

2.10.5 Based on general shear failure

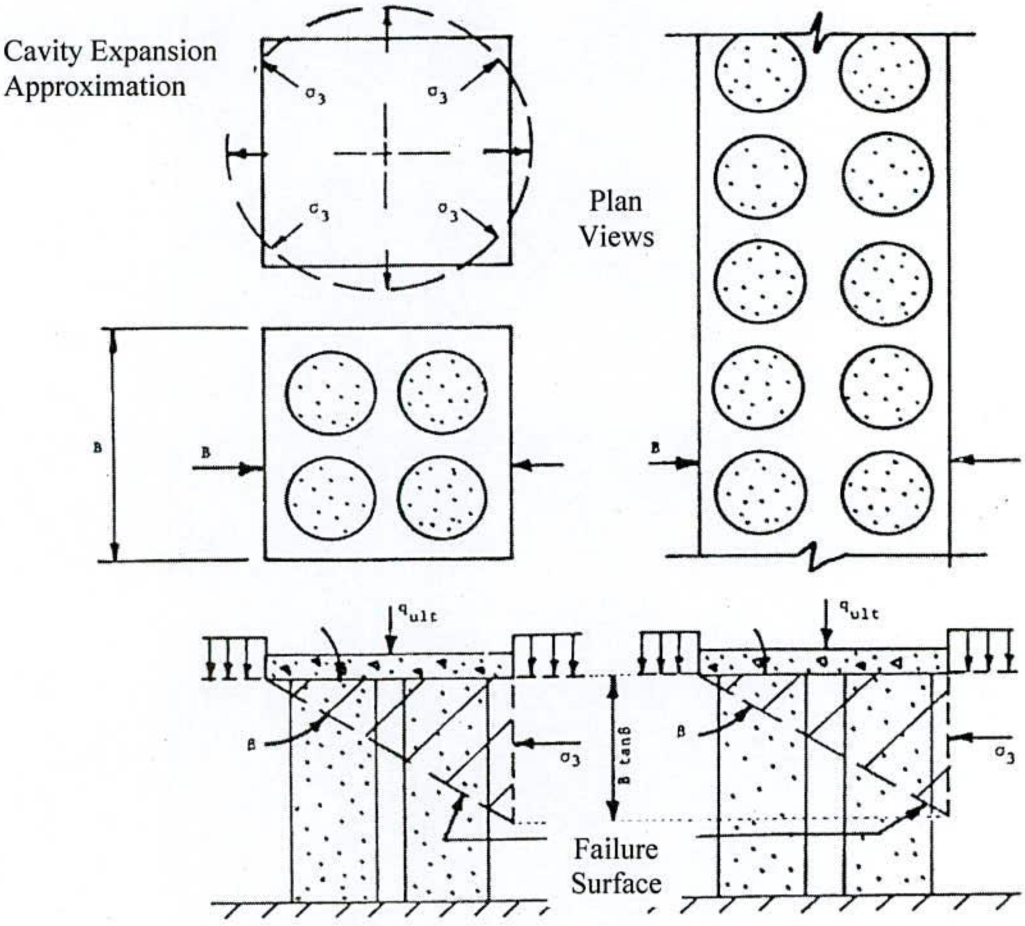
If the mechanical properties of the soil are such that the strain which proceeds the failure of the soil by plastic flow is very small, the footing does not sink into the ground untill the plastic equilibrium has been reached, this type of failure is called general shear failure, In case of stone column, Madhav and Vitkar (1978) proposed a general shear failure type mechanism. The equation given by them to calculate the ultimate load carrying capacity of a granular pile is similar to that of a shallow footing given by Terzaghi for ideal soil condition.

$$q_u = cNc^* + d_f\, Nq^* + 0.5\, BN_\gamma^* \qquad 2.11$$

Where , bearing capacity factors Nc^*, Nq^* and BN_γ^* depend on the frictional resistances of the granular and stabilized soils and the ratio d_f/dp, where, d_f = size of the footing ; dp = size of the stone column as shown in the Fig.2.10.

2.11 Ultimate Capacity of Stone Column and RAP Groups

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.2.12. 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.2.12 RAP or 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 2.12. 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

$$[\tan\phi]_{avg} = \mu_s \ a_s \tan\phi_s \qquad 2.12a$$

$$c_{avg} = (1 - a_s)c \qquad 2.12b$$

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, as defined in Eqs.2.15 and 2.20b, 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

$$\beta = 45 + \frac{\phi_{avg}}{2} \qquad 2.13$$

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

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

$$\sigma_3 = \frac{\gamma_c\ B\tan\beta}{2} + 2c \qquad 2.14$$

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 (2.13)

c = undrained shear strength within the unreinforced cohesive soil.

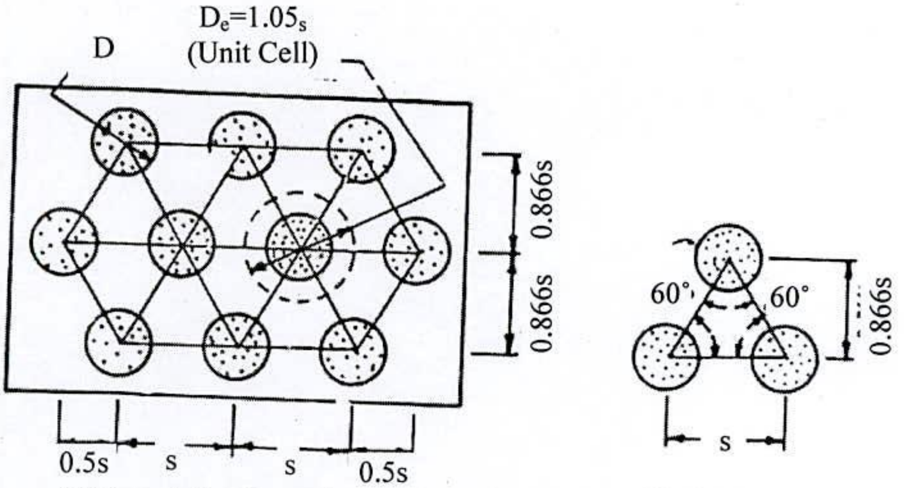
The lateral confining pressure for a square foundation can be determined using the Eq. 2.7 proposed by Vesic based on cavity expansion theory. The Vesic cylindrical expansion theory gives the ultimate stress that can be exerted on the failure block by the surrounding soil. The three-dimensional failure 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

$$q_{ult} = \sigma_3 \tan^2 \beta + 2\, c_{avg} \tan \beta \qquad 2.14$$

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:

$$a_s = A_s/A \qquad 2.15$$



(a) Foundation lay out

(b) Geometry

Fig. 2.13(i) Equilateral triangular pattern of stone column

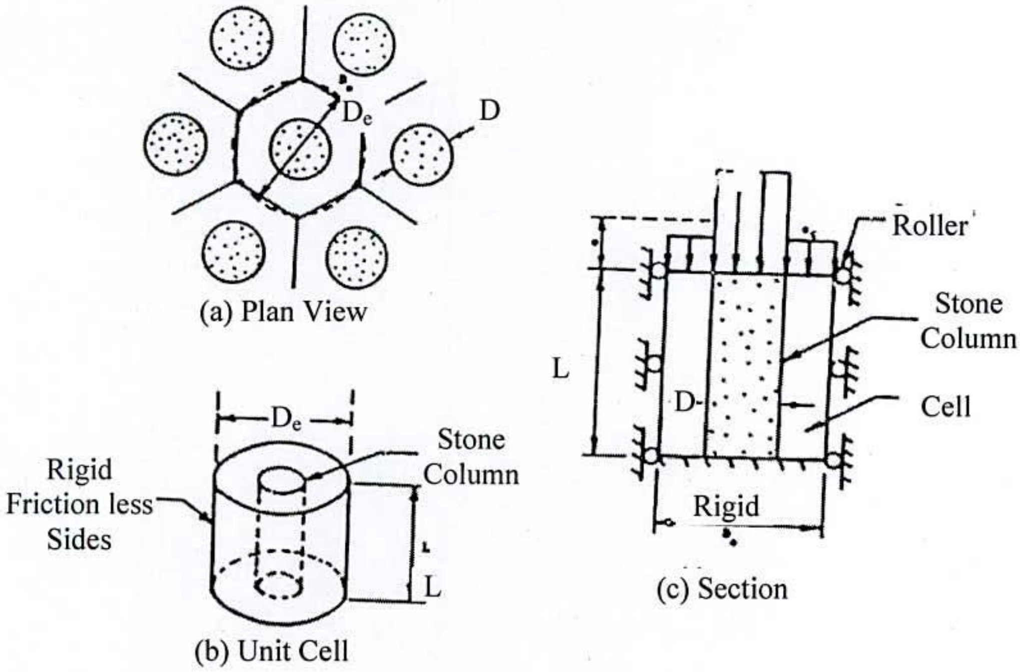


Fig. 2.13(ii) Unit cell idealization.

Where, A_s is the area of the stone column after compaction and A is the total area within the unit cell (Fig. 2.13.i.a). Further, the ratio of the area of the soil remaining, A_c, to the total area is then

$$a_c = A_c / A \qquad 2.16$$
$$= 1 - a_s$$

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

$$a_s = C_1 \left(\frac{D}{s}\right)^2 \qquad 2.17a$$

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,

$$a_s = 0.907 \left(\frac{D}{s}\right)^2 \qquad 2.17b$$

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. 2.13.ii.c) can be expressed by a stress concentration factor 'n' defined as

$$n = \sigma_s / \sigma_c \qquad 2.18$$

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

$$\sigma = \sigma_s . a_s + \sigma_c (1 - a_s) \qquad 2.19$$

Where, all the terms have been previously defined. Solving equation (2.19) for the stress in the clay and stone using the stress concentration factor n gives (Aboshi et-al. 1979 and Barksdale 1981).

$$\sigma_c = \sigma / [1 + (n-1)a_s] . = \pi_c \sigma \qquad 2.20a$$

$$\sigma_s = n \sigma / [1 + (n-1)a_s] . = \pi_s \sigma \qquad 2.20b$$

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.2.20a and 2.20b 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.

2.12 Limitations of Existing Theories

The proposed method for estimating the ultimate capacity of stone column, granular piles or Rammed Aggregate Pier considers (1) foundation shape, (2) foundation size, (3) the angle of internal friction of the RAP materials, (4) composite shear strength of the stone column reinforced soil, (5) the shear strength and overburden pressure in the soil surrounding the foundation, and (6) the compressibility of the surrounding soil as defined by the Rigidity Index. In applying this approach it must be remembered that the composite strength of the stone column reinforced soil below the foundation is considered to be mobilized; therefore in soft soils use of a composite strength which is less than the combined individual strengths of the two materials at failure is required to reflect the actual shear resistance mobilized along the failure wedge.

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.

2.12.1 Based on passive pressure condition

i)In this method, it is assumed that passive limit state is developed in the constituent materials simultaneously but this is not possible unless they have same mechanical properties.

ii)Ultimate load carrying capacity (q_u) of the stone column is calculated using the relarionship $q_u = \sigma_r k_{ps}$, which is based on the yield strength of granular material and surrounding clay media, where, σ_r = passive resistance of the soil, k_{ps}= coefficient of passive resistance of granular material and Compatibility of deformation of the constituent materials are not considered here, which will lead to considerable error.

2.12.2 Based on pile formula

i) In this method it is assumed that stone column behaves like a pile in transferring load to the soil. However, it is not correct because the constituting material such as stone aggregates play an important role in case of stone column for transferring load in the surrounding clay media.

ii) In case of stone column, the foundation system is considered as a composite system of aggregates and surrounding clay media. This is not considered for the design of stone column, based on pile formula.

iii) The radial deformation of columns governed the load carrying capacity of stone columns, which is almost negligible in case of pile.

iv) Here it is assumed that shaft friction is constant throughout the length of column and is equal to undrained shear strength of clay. However, skin friction is not constant; rather it depends on the vertical and radial displacement of column and related to undrained shear strength by the adhesion factor.

2.12.3 Based on cavity expansion theory

i) Vesic's cavity expansion theory appears to be applicable for recompressed soil only i.e. to a soil which has been first subjected to a very high hydrostatic pressure and than unloaded.

ii) It is difficult to estimate in-situ undrained shear strength of clay in the vicinity of stone column due to influence of installation procedure, which is required to determine the cavity expansion factor F_c' and F_q'.

iii) Since strength characteristics vary in the annuals around the stone column, it is difficult to assign a representative value of rigidity index which is essential for estimating the cavity expansion factors Fc′ and Fq′.

iv) Here load carrying capacity of stone column is based on the ultimate strength of aggregates and clay. Compatibility of deformation of the constituent materials is not taken into account, which is the major drawback of this method. At a given load the same deformation will never produce into the constituent materials unless they have identical mechanical properties.

v) The cavity expansion theory can not be used directly to estimate the value σ_v because of the above mentioned uncertainties.

2.13 Experimental Investigation

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

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

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

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

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

Mitchell and Huber (1985) reported the performance of vibro- replacement stone columns used to support a large waste water treatment plant founded on up to 15m of soft estuarine deposits. Column spacing is ranged from a 1.2mX1.5m pattern under the most heavily loaded areas, to a 2.1mX2.1m pattern under lightly loaded areas. Twentyeight single column load tests were done during the installation of 6,500 stone columns to evaluate load settlement behavior. The installation of stone columns leads to a reduction in settlement to about 30 - 40% of the values to be expected on unimproved ground. Load test settlement calculated by the finite element method for the initial settlement conditions, using undrained clay properties and drained properties of sand and stone columns, are some what higher than the average settlements observed during actual field load tests conducted on similar stone column spacing patterns. However, the overall results obtained from the finite element analysis indicated reasonable agreement between the calculated and the observed settlements for the idealized load tests. Settlement predictions using several other, more simplified methods gave values that agreed reasonably well with both the finite element predictions and the measured values. This lends support to the use of the simple methods in practice.

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

2.14 Case Studies

A considerable amount of work has been performed successfully throughout the world to improve poor ground by Granular pile/stone column/Geopier foundation system. This foundation system gives better result both in load carrying capacity and limiting settlement. A few case studies are presented here to show the applicability of such foundation system

2.14.1 Werehouse and Machine shop at Kandla, India

Rammed type stone columns having 750mm diameter and 10m long were installed in warehouse and machine shop area in 1974. Typically two stone columns were placed under each column footing. In the rest of the area, 400mm diameter sand drains were installed. The area was preloaded to general plinth load intensity. Inspite of the large difference in the load intensities on the floor and the column footings, no visible differential settlement or cracking has been observed over a period of seven years and crane rails perform perfectly. Stone columns with sand drains helped to bring down the preload intensity for the footings, reduced cost of preload and made the preloading operation simple. Stone columns helped to mobilize the drag and accelerate consolidation.

2.14.2 Simulated seismic test at South Tampale Bridge on interstate 15 at Salt Lake City, Utah

Evert C. Lawton was researched that was conducted at the I-15 Bridge over South Tample site in Salt Lake City, Utah (Lawton, 1999). The primary purpose of the geotechnical research were to perform full-scale simulated seismic tests on existing bridge bents in which the behavior of pile foundations supporting the existing bridge, as well as Geopier foundations support the structural reaction frame, were studied. During May and June of 1998, geotechnical testing was conducted in conjunction with structural testing on a section of the existing northbound bridge had been taken out of service. The section of the bridge that was tested consisted of two bents and the deck and girders spanning the two bents. Cyclic lateral loads were applied to the bent caps to simulate seismic shaking during an earthquake.

Three separated test were conducted. The anticipated maximum load to be applied to the bent caps during testing was 400 kips. During the actual testing a maximum lateral load of 490 kips was applied. The cyclic lateral loads were applied to the bent caps using a hydraulic actuator attached to a steel reaction frame. The reaction frame was found on two reinforced concrete footings, which were newly constructed for this research project. Each footing was 24.5 ft long, 8.25ft wide and 3.75 ft thick and was supported by a Geopier foundation system consisting of 10 uplift piers.

The forces generated by the reaction frame on top of the supporting footings during the cyclic pushing and pulling on a bent cap are illustrated for the anticipated maximum lateral pushing /pulling force was produced on the exterior footing with a 500 kips upliet forces prodused on the interior footing . During pulling the same magnitude of vertical forces were produced, with up lift on the exterior footing and compression on the interior footing. The 400 kips horizontal force was carried by the two footings as a unit because the footings were tied together by the rigid reaction frame. During the test the two reaction frame footings were found on the ground surface, so the resistance to the lateral load was produced by shearing along the footing-soil interface, as well as by pushing of the uplift bars on the Geopiers. This horizontal force couple produced overturning moment on each footing, with a moment arm equal to or greater than the thickness of the footing.

Rammed Aggregate pier systems have been successfully installed on numerous major project sites within the United States within a wide variety of soil conditions of exclusive of peat soil, over a time span of over ten years. In 1997, a three story wood framed Assisted Living facility structure was planned for construction in the city of summer, Washington (United States) on a site containing peat soils. Cone Penetration Test (CPT's) taken to depths of 18.3m (60ft) located on reliable strata capable of supporting deep foundations to those depths. Driven piles would have to extend to depths greater than 18.3m (60ft). Alternative support methods of over-excavation and replacement of the soft organic silts and peat, and of traditional vibro-replacement stone columns, were rejected because of ground water problems and anticipated poor reinforcement, respectively.

A geotechnical engineer and principal with Geopier Foundation Company, Northwest proposed a Value Engineer (VE) alternative , using the patented (US and European patents), Geopier Rammed Aggregate Pier method reinforce the soils to make a stiffer, composite, pier-matrix soil bearing support zone. This composite material would support high bearing pressure, shallow spread footings. The proposal included design of footing to control settlements to less than 25mm (1in), and the performance of a full-scale, modulus load test to verify assumptions made regarding Geopier element stiffness modulus.

Shallow foundations designed with a maximum allowable bearing pressure of 216kPa (4,500 psf), and supported by a system of Geopier Rammed Aggregate Pier elements, were subsequently design and constructed. Observation indicates that total settlement of the structure has been less than the design settlement of 25mm (1in).

2.14.3 Mt. high school field at Snolquamie, WA

"Leveling the playing field" recently took on new meaning at Snolquamic, Washington's Mount Si High School (Geopier 2005b). The school district's athletic facility improvements program included the construction of a 230 X 400 ft artificial turf football field. Since the site was located in a sensitive floodway area, the field itself had to be constructed at a higher grade. Filling the area to achieve this was not possible as this would create higher flood levels in the adjacent community. The field (platform) was therefore design as a pre-cast concrete deck established at approximately six feet above existing site grade and supported on elevated beams. The beams were, in turn, supported on columns bearing on isolated footings spaced on a grid pattern of 20 X 40 ft. Site soils consisted of very moist to saturated, interceded layers of soft silt, sandy silt and loose silty sands to depths of approximately 75 feet. Field and slab loading was estimated to be on the order of 220 psf. Under these conditions, Associated Earth Science Inc. (AES) of Kirkland, WA, which provided the geotechnical services, determined that conventional shallow foundations were not viable.

"Auger-cast pilling had been a preliminary consideration for the foundation support," said AES principal engineer Kurt Merriman, p.e. "However, we have used the geopier system for years in the Puget Sound area and have found it to be very flexible in solving challenging issues on construction projects. We recommended the Geopier option based on previous successful work in these types of soil and on cost." "The auger-cast pile design was not applicable from both a layout and cost perspective," agreed the owner's representative Clint Marsh of KJM Associates, Bellevue, WA.

2.14.4 Windpark at Guntersblum, Germany

71 m high wind towers at the wind energy station of Guntersblum, Germany were planned to be supported by shallow foundations. The circular foundations had a diameter

of 12.5m with maximum design edge pressure of 306kN/m^2. Additional, the Rammed Aggregate Pier system had to be designed to provide stiffness modules of 300 MN/m^2 and a rotational spring stiffness constant of 30.000 MN/m (Wissmann, K.J. and N.S. Fox.2000).

Subsurface conditions

Subsurface exploration at the site exhibited soft, sandy and clayey silts with STP- N blow counts of 2 to 5inch from the upper 4m. The soft soils were underlain by medium stiff, loessial deposits to boring termination. Stiff soils with SPT-N values exceeding 12 were encountered at depths from 9m below ground surface.

Geopier Design

Based on the results of the geotechnical exploration, 4m long Geopier elements were designed to be arranged in three to five concentric circles below the circular foundation. Most of the Rammed Aggregate Piers were located near the perimeter of the foundation to provide edge pressure resistance. The elements were designed with cell capacities ranging from 311 to 378 kN which is presented in Table 2.3.

Table 2.3. Geopier Design Parameter Example(After S.Fox et.al. 2004)

WKA No.	Foundation Area (m^2)	Geopier Shaft Length(m)	Geopier Cell Capacity Q_{qp} (kN)	No. Geopier Elements	Geopier Stiffness Modulus k_{gp}(MN/m^3)
1.0	120.8	4.0	378	74	47.5

Modulus Load Test

A modulus load test was installed at the area of the site that exhibited the most unfavorable soil condition at the design stress of 705 kN/m^2 was measured to be 8.2mm, resulting in a stiffness modulus value of 82 MN/m^3 can be obtained in Fox et al. (2004).

2.14.5 Pricesmart Superstore, Philippines

The Pricesmart Superstore project constructed in 2001 was the first Geopier application in the Philippines. Subsurface constructions are characterized by soft soil extending to 18 meters below ground. The original design called 6,500 square meters of suspended structural floor slab to be supported by drilled shaft foundations. Driven piles were ruled out because of potential damage to surrounding residential areas from excessive vibrations induced within the very poor sub-soils. By adopting a Geopier floating foundation system, costly bored piling and suspended floor slabs were each eliminated. This allowed the heavily loaded floor slabs to be supported by the Geopier soil reinforcement and designed as a slab on grade system. This floating foundation system was designed to control the foundation and floor slab total and differential settlements to meet the project design criteria. A total of 1,900 Geopier elements with lengths of 3 to 3.5meters were installed in 60 working days reducing the project completion schedule by 60 days (Fox et.al.2004)..

A modulus test performed on-site produced a Geopier stiffness modulus value of 83MN/m^3, which was greater than the 35MN/m^3 used in the design. The Geopier reinforced upper zone settlements were estimated to range from 10 to 15 mm. The Geopier construction saved more than 50% of foundation cost compared to alternative solutions. The soil profiles at that place were at 0 to 5m-very soft to medium clay, SPT-N=2 to 9 ; 5 to 8m – very loose to medium dense silty sand, SPT-N value=2 to 11 ; 8 to 15m – very soft to soft silty clay, SPT-N=2 to 4 and ground water table at 1.2m depth from the ground surface.

2.14.6 Regulator at Passur river, Khulna, Bangladesh

A case study on improving of soft ground by installation of sand compaction piles was presented Alamgir and Zaher (1999a and 1999b). The effectiveness of sand piles in improving a typical soft ground at south western region Bangladesh to construct a water control structure (6-vent regulator) in a river was examined in the papers. At the site, a soft alluvium fine grained soil deposit exists up to 12m depth from the ground surface.

The site was improved by total 765 numbers of sand piles, 0.20 m in diameter and 8.80 to 9.40m long, installed in square grid at 0.75m spacing, by vibro- displacement method using the simple technique. Typical sand of Bangladesh, Sylhet sand, is used in the sand pile. Prior to the commencement of concreting for floor construction of regulator, sub-soil explorations were performed to examine the improvement. The investigation reveals that the sand piles improved substantially the bearing capacity of the natural ground. Therefore, the soft ground improvement using sand pile technique is revealed as fast, economical and an efficient method to improve weak soil compared with other conventional ground improvement technique. The use of smaller diameter with closer spacing was found suitable in such soft soil deposits for the vibro-displacement type of sand pile construction while comparing the construction problem arises from the installation of large diameter due to the development of side friction. The simple construction procedures and the related equipment adopted in this project for the installation of the desired sand piles were found to provide high degree of effectiveness. Sub-soil investigations revealed that the sand piles improved substantially the bearing capacity of the natural soil and hence the concreting for floor construction of regulator was done without any trouble. The monitoring system conducted in this project during the construction process and hence ensured by the Engineer-in-charge made a great contribution to the quality control of the sand piles.

2.15 Concluding Remarks

For soft soil, it is very significant process to install Rammed Aggregate Pier for low rise or high rise buildings or structures. From few case studies it is depicted that RAP is an innovative ground improvement method which is very effective foundation technique for soft soils. The sub-soil of Bangladesh in general, is alluvial deposit of recent origin. Moreover soil condition at south–western part in Bangladesh is soft up to great depth with mixed organic. So use of RAP foundation is activated more effective for soft soil in Bangladesh. KUET campus is situated south-western region in Bangladesh in which the ground condition is very weak and predominant by fine-grained soil mixed with organic. In this respect the suitability and effectiveness of Rammed Aggregate Pier foundation system should be investigated in this area.

CHAPTER THREE

STATEMENT OF THE PROBLEM

3.1 General

This study deals with the improvement of soft soils by the installation of Rammed Aggregate Piers (RAPs). Amongst the various ground improvement techniques, columnar inclusion such as stone columns, granular piles, sand compaction piles, etc is the most versatile and cost effective option (Alamgir 1996). In the recent years, Rammed Aggregate Pier, a type of Columnar Inclusion has been used as the foundation solution in marginal sites for its proven records of effectiveness in improving soft fine-grained soil deposits. Geopier a type of Rammed Aggregate Pier, had been developed in the 1980's in the United States and more recently in Asia and Europe, for supporting of lightly to heavily loaded structures, highway and railway embankments (Lawton and Fox 1994 and Lawton et. al. 1994). This improvement method can safely carry significant lateral and up lift forces and has been successfully used on hundreds project sites both the poor and unsuitable soils as well as fair to good soils, which are compacted using high energy impact temper (Lawton 1999). Some practical application of Rammed Aggregate Pier can be obtained in Wissmann and Fox (2000) and Wissmann et al. (2000).

3.2 Statement of the Problem

Soft fine-grained soil with significant organic content dominates the sub-soil of Khulna region, which often creates problem to the geotechnical engineers to select suitable economic foundations for structures due to low shear strength and high compressibility (Alamgir et al. 2001). Recently some ground improvement techniques including granular columns have been employed successfully in this region. The performance of geotextile-reinforced footing, sand compaction piles, stone columns and granular piles have also been studied in this region at field level (Haque 2000, Zaher 2000, Alamgir and Zaher 2001, Haque et al. 2001 and Sobhan 2001). This study has been undertaken to depict the applicability of Rammed Aggregate Pier in such sub-soil conditions. However, acknowledging the reality, instead of standard practices, locally available granular materials and installation technique have been used for the construction of geopier. For this field investigation a typical soft ground site at BIT campus, Khulna is considered, in which Rammed Aggregate Pier were installed in Single, Double and Group pattern using locally available granular materials and installation techniques. The effectiveness of Rammed Aggregate Pier in improving ground conditions where measured by conducting real footing load tests on the improved ground.

3.3 Site-Condition and Sub-soil Strata

For this field investigation a typically soft ground site located at the campus of Khulna University of Engineering and Technology (KUET), Khulna is considered. Field investigation about the effectiveness of Rammed Aggregate Pier, is conducted here. The location of the investigated site and the sub-soil profiles describing the soil conditions are described in the following sections.

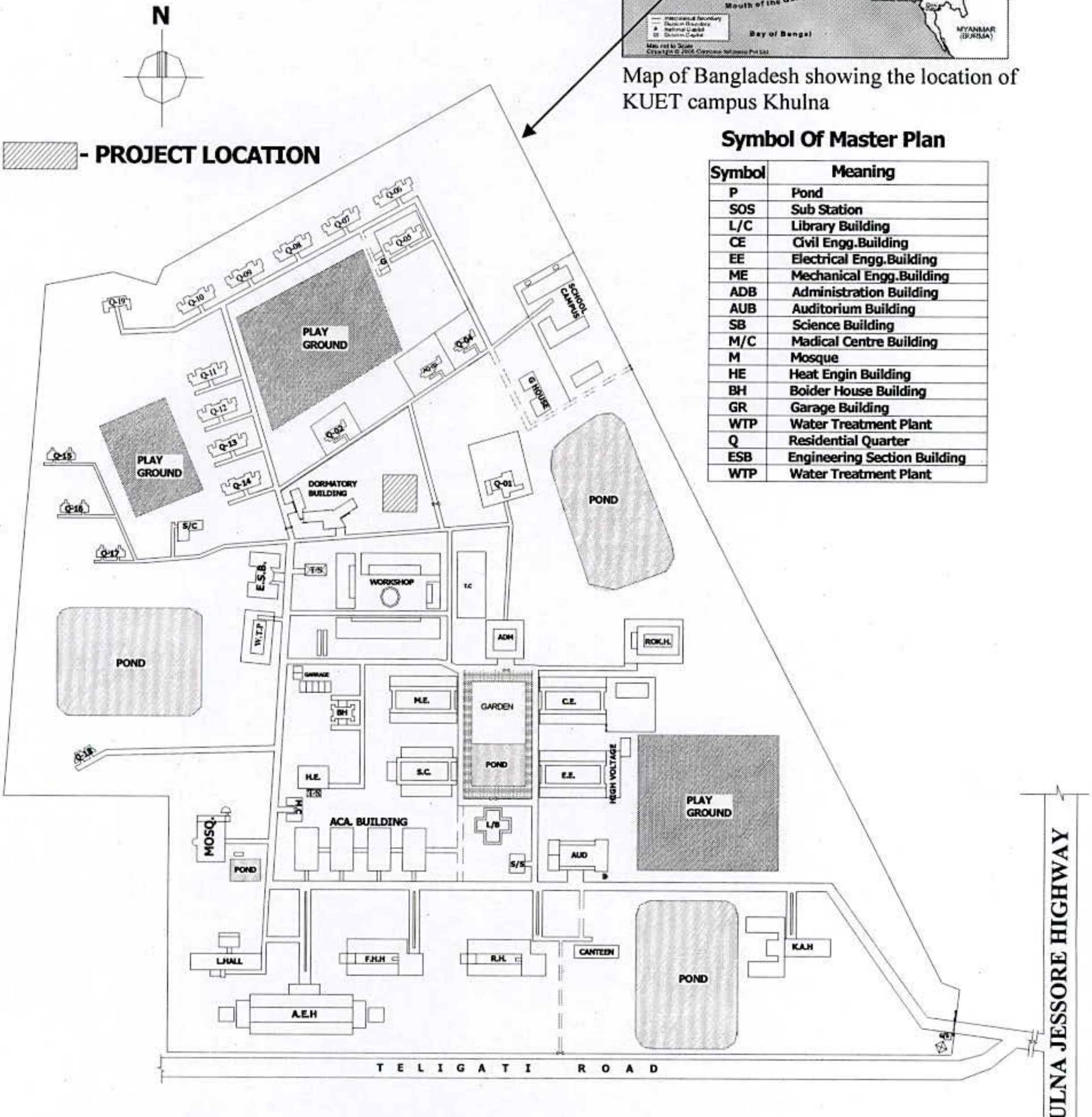
3.3.1 Location

For this study, the project site for the field investigation is selected within KUET campus, Khulna. The KUET campus Khulna is located in the South-Western part of the country. The investigated region for the present study and it's location map of the investigated site in KUET campus is also shown in Fig.3.1. Alamgir and Zaher (2001) revealed almost similar sub-soil profile in another location about 250m apart from the present site in the KUET campus, where field investigations were conducted to established the performance of stone columns and sand compaction piles as installed using both the dry-displacement (Zaher 2000) and wet-replacement (Sobhan 2001) methods.

Location of site

KUET Campus, Khulna,

Map of Bangladesh showing the location of KUET campus Khulna



Symbol Of Master Plan

Symbol	Meaning
P	Pond
SOS	Sub Station
L/C	Library Building
CE	Civil Engg.Building
EE	Electrical Engg.Building
ME	Mechanical Engg.Building
ADB	Administration Building
AUB	Auditorium Building
SB	Science Building
M/C	Madical Centre Building
M	Mosque
HE	Heat Engin Building
BH	Boider House Building
GR	Garage Building
WTP	Water Treatment Plant
Q	Residential Quarter
ESB	Engineering Section Building
WTP	Water Treatment Plant

Figure 3.1 Map of KUET Khulna campus and location of project site.

3.3.2 Sub-soil profiles

Bangladesh is a part of Bangla Basin at the lower reaches of the three mighty rivers the Ganges, the Brahmaputra and the Meghna and their associated tributaries. In the upper horizons, the sub-soil of vast areas of Bangladesh is composed of very soft fine-grained soil deposits of recent origin. In the south-western coastal districts, fine grained soil deposits, predominantly peat and muck are abundant. In this regions peat deposits are encountered due to the presence of world's biggest mangrove forest, the Sundarbans of 5,77,285 hectares, as its present area (Zaher 2000). In the past, the Sundarbans was extended in this region. For the last few centuries it was double spreading over the present area. During the geological changes in the past, some part of the Sundarbans was submerged by the weathered and sedimented deposits resulting in the present peat deposits in these regions. The peat deposits are extended to the south-western coastal districts through Satkhira to Potuakhali. Practicing engineers are facing many difficulties in these regions to solve the several geotechnical engineering problems such as very large total and differential settlements, bearing capacity failure and slope stability problems. However, the failure of structures and the related problems, due to the extensive presence of peat deposits, were not recorded properly which the practicing engineers can use as a reference. Several structures have been and are being constructed in typical peat deposits exist a depth of around 20ft. Most of the past records show that the KUET campus consisting of soft soil layer contains organic. Sub-soil investigation is done and the index properties of soil are determined at different layers. The details of subsoil condition and soil properties are given in Table.3.1. For determining the sub-soil properties Standard Penetration Test (STP) was performed at the selected site which is situated in the same premise of newly constructed four storied building for Teachers Dormitory at KUET. The Standard Penetration Test (SPT) developed in 1927, is currently the most popular and economical means to obtain the sub-surface information. It is estimated that 85% to 90% of conventional foundation design in North and South America is made using the sub-soil condition determined by SPT (Bowles 1988). In sensitive silty clay and clayey silts, it is often difficult to determine the true soil properties by conventional field investigation methods. In such cases in-situ methods such as CPT sounding, dilatometer or pressure meter test often given the most reliable results (Ekstrom et al.1994). However, SPT is employed here due to not availability of appropriate method for soft soil investigation such as CPT test, Dilatometer test and field vane shear test etc. It can be noted here that the SPT test still the most popular field test in Bangladesh to determine the sub-soil profile. The N-values are very low up to the depth of 18m from the natural ground surface, which is 2 to 5 and a layer of 4.5m to 9m containing organic clays is encountered as shown in Fig.3.2. The layers from 9m to 19.5m depth and 1 to 3m are containing clay and silty sand where the N-values are larger than the top layer. In this layer the N-value ranges from 5 to 8 as shown in Table.3.1.

Number of sample	Depth in m	Thickness in m	Type of sample	Description of materials	Log
1	1-1.5	1.5	Brown	Silty sand	
2	1.5-3	1.5	Gray		
3	3-4.5	1.5	Dark gray	Clay	
4	4.5-6	1.5	Dark gray	Organic clay	
5	6-7.5	1.5			
6	7.5-9	1.5			
7	9-10.5	1.5	Dark gray	Silty clay	
8	10.5-12	1.5			
9	12-13.5	1.5	Gray		
10	13.5-15	1.5			
11	15-16.5	1.5	Dark gray		
12	16.5-18	1.5			
13	18-19.5	1.5	Gray		

Figure 3.2 Sub-soil stratification and bore log.

3.4 Geotechnical Properties of Sub-soils

The engineering properties of the sub-soil particularly index properties, organic contents, shear strength and compressibility have been evaluated by performing different conventional laboratory test on soil samples. Ranges of different engineering properties of the soil samples as obtained from the tests on samples collected from the site are given below.

The value of Natural Moisture Content varies from 44.72 to 167.56 percent, Liquid Limit varies from 32.30 to 255.00 percent, Plastic Limit varies from 21.21 to 177.42 percent and the value of Plasticity Index varies from 1.2 to 77.58 percent.

Unit weight was measured, which minimum and maximum value presented respectively 10.59 and 17.10. The value of Specific Gravity varies from 2.10 to 2.78. While the maximum and minimum initial void ratio value were determined respectively 0.76 at depth 12m and 5.22 at depth 4.5m, Cc varies from 0.25 to 1.80.

Organic contents: The percentage of organic contents varies from 1.49% to 31.75%.

Unconfined Compressive Strength: The value of Unconfined Compressive Strength as reported by unconfined compression test varies from18.00 to 70.00 kPa.

Penetration Resistance: The N-value obtained from SPT test in the field which minimum and maximum value respectively 3 and 9.

Compressibility: The compression index C_c, of the soil varies from 0.25 to 1.80.

Detailed test results of engineering properties of soil samples collected from the boreholes are shown in Table.3.1. Some typical curves of sieve and hydrometer analysis at depth of upper sub-soil in field where RAPs were installed are shown in Figs. 3.3 to 3.7. From this figures, soils are categorized into three groups as 4.75 to 0.076mm, 0.076 to 0.002mm and <0.002mm as shown in Table 3.2

Table 3.1. Geotechnical engineering properties of the site at KUET campus

Depth (m)	Soil stratification	w (%)	Physical properties						Compressibility properties		Shear strength properties	
			W_l (%)	Wp (%)	Ip	γ kN/m^3	G_s	Organic contents (%)	e_0	Cc	s_u (kPa)	N Value
1-1.5	Silty sand	-	-	-	-	-	-	1.49	-	-	-	8
1.5-3		-	-	-	-	-	-	3.44	-	-	-	5
3-4.5	Clay	48.20	53.20	21.21	31.99	16.92	2.78	10.01	1.15	0.66	20	5
4.5-6	Organic clay	74.65	81.50	47.27	34.23	13.88	2.59	12.70	5.22	1.80	28	3
6-7.5		167.56	255.0	177.42	77.58	10.59	2.10	31.75	1.35	0.65	30	9
7.5-9		63.63	44.80	34.41	10.39	13.42	2.68	7.76	1.73	0.55	35	4
9-10.5	Silty clay	50.12	39.10	28.07	11.03	17.10	2.75	6.46	1.06	0.35	18	4
10.5-12		44.72	40.00	32.71	7.29	16.85	2.52	7.22	1.36	0.37	11	4
12-13.5		55.58	32.50	31.30	1.2	16.12	2.15	5.59	0.76	0.25	9	5
13.5-15		51.52	36.40	32.46	3.94	-	-	6.49	-	-	-	5
15-16.5		54.32	39.00	35.29	3.71	-	-	4.89	-	-	-	6
16.5-18		55.21	32.30	31.11	1.19	-	-	3.52	-	-	-	5
18-19.5		53.25	37.00	30.34	6.66	-	-	3.74	-	-	-	8

Note: w=Water content, W_l= Liquid limit, Wp=Plastic limit, γ=Unit weight, G_s=Specific gravity, e_0=Initial void ratio, C_c=Compression index, s_u=Undrained shear strength. Average values are provided here of various parameters.

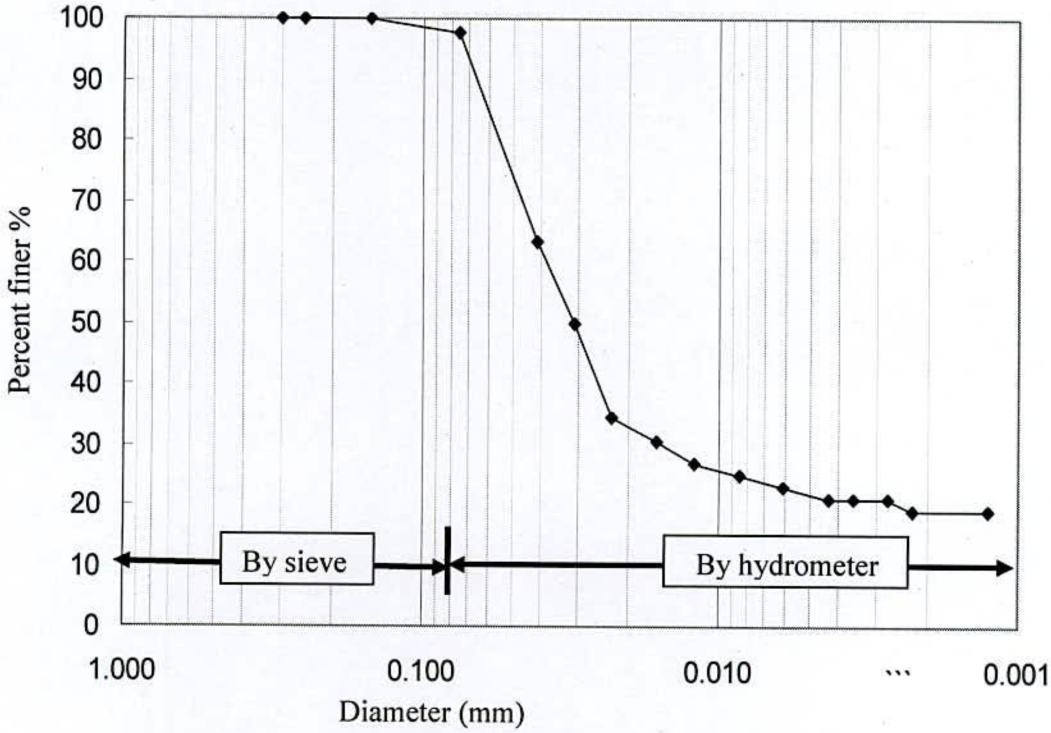


Figure 3.3 Grain size distribution of soil sample at depth 1m-1.5m in the site.

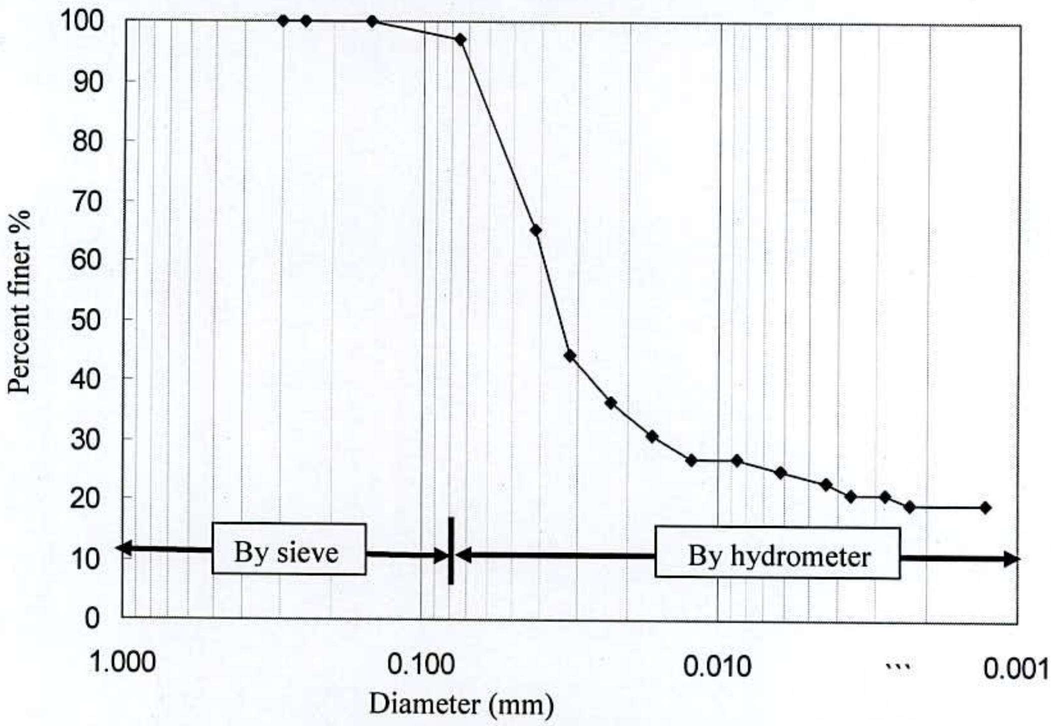


Figure 3.4 Grain size distribution of soil sample at depth 1.5m-3m in the site.

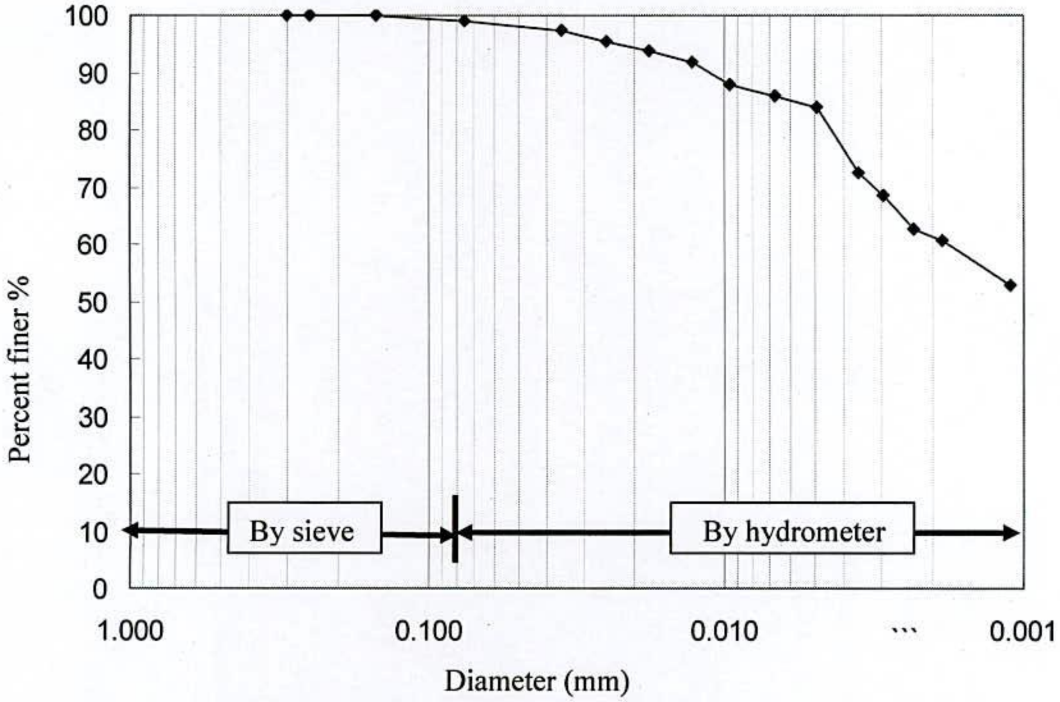


Figure 3.5 Grain size distribution of soil sample at depth 3m-4.5m in the site.

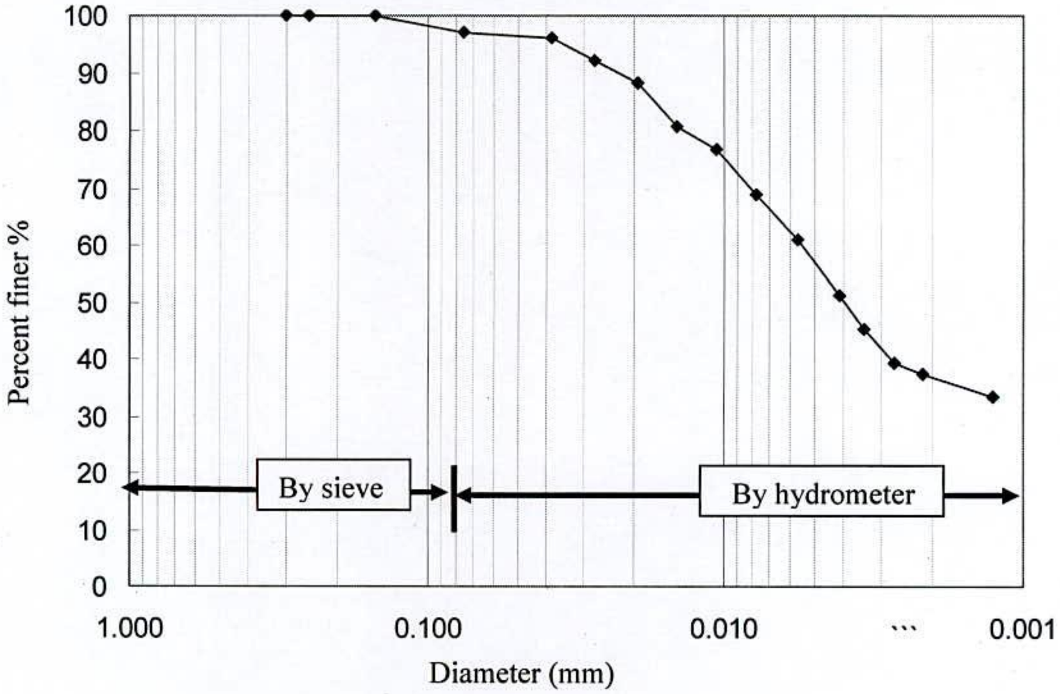


Figure 3.6 Grain size distribution of soil sample at depth 4.5m-6m in the site.

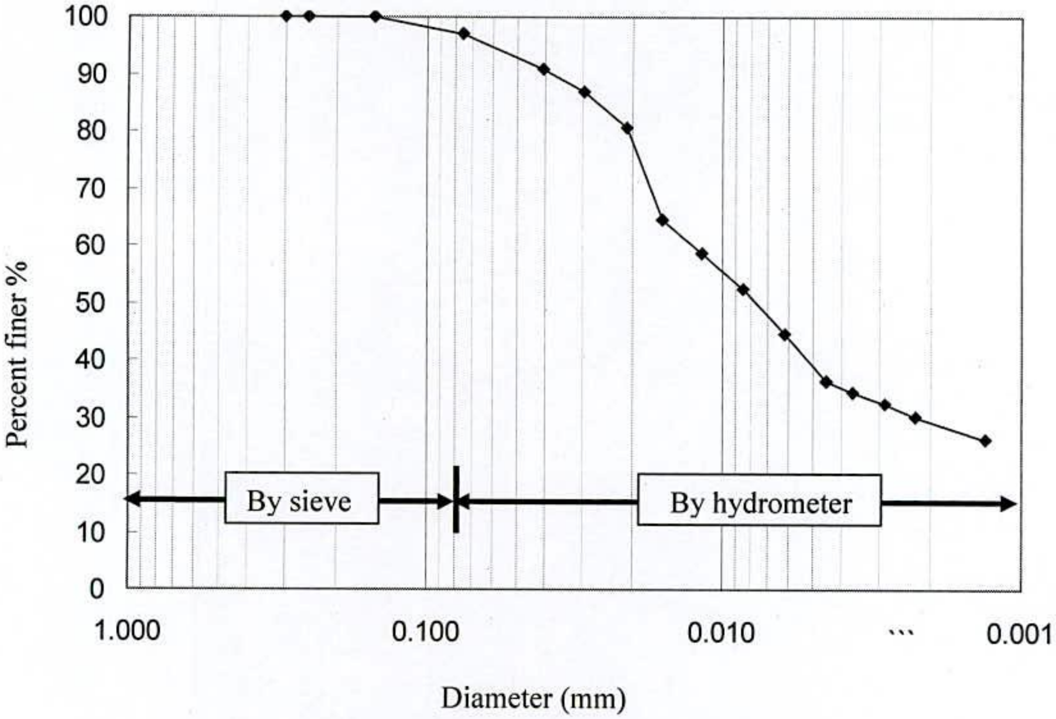


Figure 3.7 Grain size distribution of soil sample at depth 6m-7.5m in the site.

Table 3.2 Properties of Soil Particles from Sieve and Hydrometer Test

Different Depth(m)	Percentage of Soil Particle Size		
	4.75-0.076mm	0.076-0.002mm	<0.002mm
1-1.5	2.90	78.09	19.01
1.5-3	3.00	77.79	19.21
3-4.5	1.10	37.93	60.97
4.5-6	2.80	63.99	33.21
6-7.5	2.90	68.6	28.50

3.5 Granular Materials of Rammed Aggregate Piers

In this study, two types of granular materials, namely sand and brick aggregates were considered for the construction of Rammed Aggregate Piers. These two types of materials are selected since they are commonly used as construction material and also readily available in Bangladesh. These two type materials are also used to judge their suitability as a granular material for the construction of Rammed Aggregate Piers. The locally available sand is mixed with the brick aggregates in a specific ratio to use RAP materials as shown in Fig.3.8. The proportion of Local sand and Brick aggregates for well gradation was identified by trial and error. Finally, one-third of local sand and two-thirds of brick aggregates are mixed together to have a well graded material. The properties of granular materials are described in the following sections.

Figure 3.8 Granular materials used in Rammed Aggregate Pier construction.

3.5.1 Properties of sand

It is one of the locally available sand in the south-western region of Bangladesh. It is a river sand and of light gray in colour. The physical properties of this local sand can be described as FM=1.26, D_{10} = 0.17, D_{30} =0.2, D_{60} =0.30, C_u=1.76, C_c=0.78 and 1.62% passing #200 Sieve. Where, FM= Fineness modulus, D_{10} = Effective diameter of particle size of which 10% sample is smaller, D_{30} = Effective diameter of particle size of which

30% sample is smaller, D_{60} = Effective diameter of particle size of which 60% sample is smaller, C_u= Co-efficient of uniformity and C_c= Co-efficient of curvature. The grain size distribution of local sand is shown in Fig.3.9.

To increase the cost effectiveness, and availability local sand is prepared to use in geopier construction. Mixing of one-thirds of Sylhet sand with two-thirds of local sand provides an effective and cheap combination of sand used for the construction of sand piles in improving marginal sites (Alamgir & Zaher 1999a and 1999b).

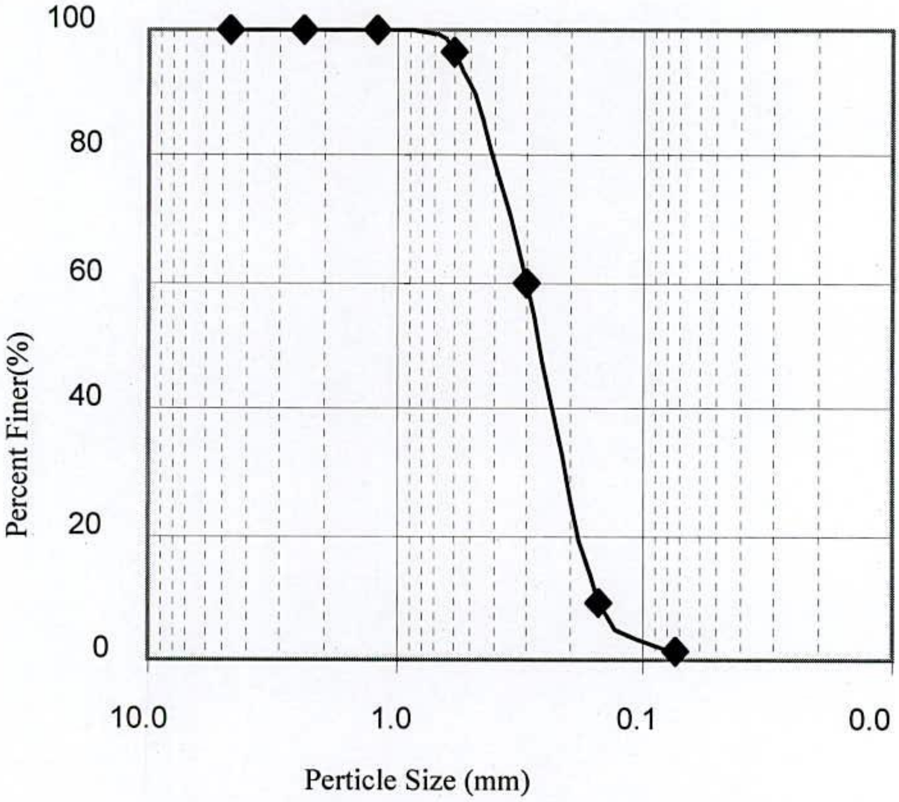


Figure 3.9 Grain size distribution of local sand.

3.5.2 Properties of brick aggregate

Brick chips of 38mm down well graded is considered for the present study. It is originated from class one over burned bricks. The grain size distribution of brick aggregates is shown in Fig.3.10. From this figure, the physical properties of this brick aggregate are obtained as D_{10}=7.5, D_{30}= 15.5, D_{60}=21.5, Cu=2.86, Cc=1.49.

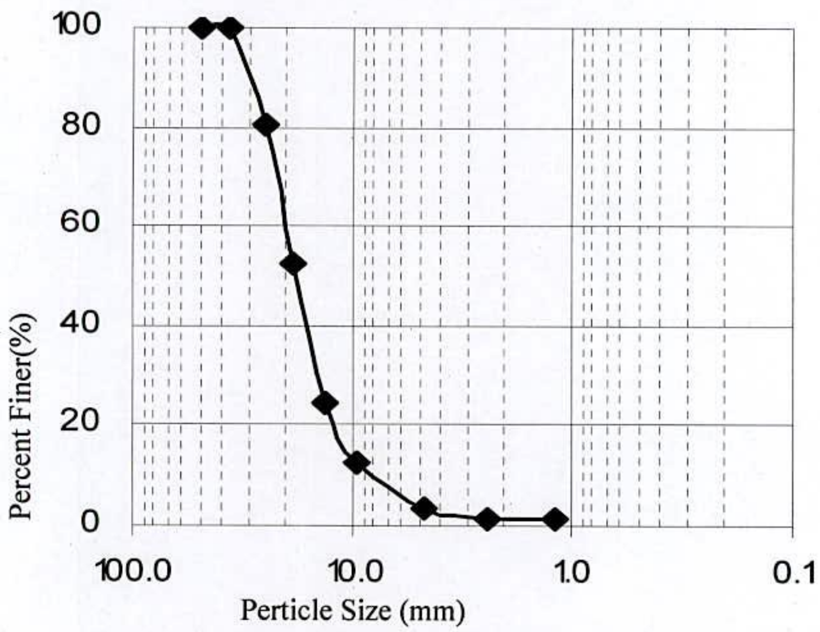


Figure 3.10 Grain size distribution curve of brick aggregates.

3.5.3 Properties of used granular material

Granular materials of Rammed aggregate pier which is consisting of 2:1 mixture of brick aggregates and local sand The physical properties of this mixture can be described as D_{10} =0.9, D_{30} =1.4, D_{60} =10.7, Cu=11.89, Cc= 0.20 . The grain size distribution of mixture brick aggregates and sand is shown in Fig.3.11.

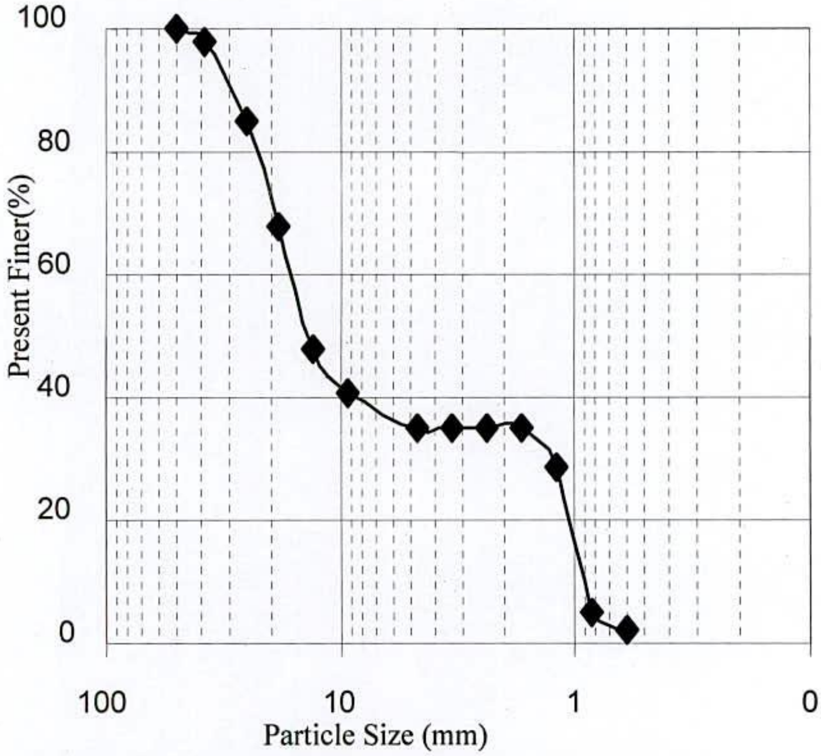


Figure 3.11 Grain size distribution curve of mixed granular material prepared through the mixing of sand and brick aggregates.

3.6 Configuration of Rammed Aggregate Piers

The physical configuration of the installed Rammed Aggregate Pier and their arrangements as considered in this study are described here. As the most common shape of columnar inclusions is cylindrical, Rammed Aggregate Piers of cylindrical shape are considered in this study for installation.

The dimension of Rammed Aggregate Piers was considered as 0.75m diameter and 3.4m long. Irrespective of Sub-Soil conditions the length of geopier were decided based on the installation technique. The diameter of the RAP was set as the minimum diameter required for the excavation of borehole manually. The schematic diagram of the installed RAP is shown in Fig.3.12 for single Rammed Aggregate Piers. To obtain a vivid picture about the effectiveness of RAP, three different arrangements of Rammed aggregate piers of same dimensions are installed in the same location.

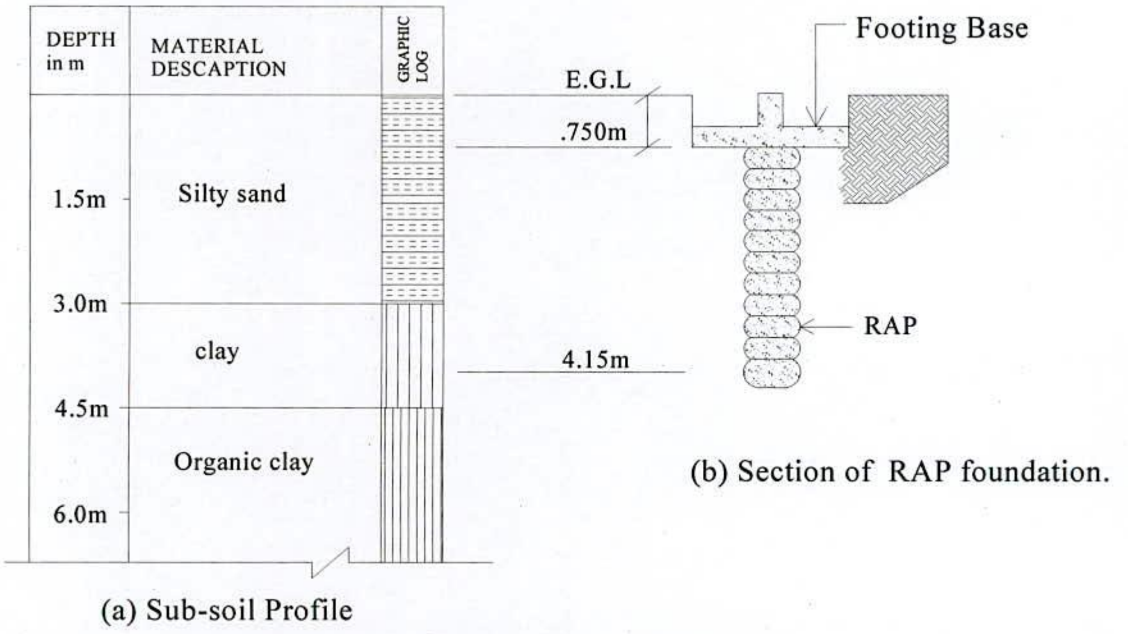


Figure 3.12 Schematic diagram of RAP under footing.

The arrangements are categorized: (i) Single, (ii) Double and (iii) Group are shown in Fig.3.13. Total three numbers of RAPs were constructed in a group at triangular pattern. Each RAP was constructed with almost same spacing for the double and group RAP as 1150mm center to center.

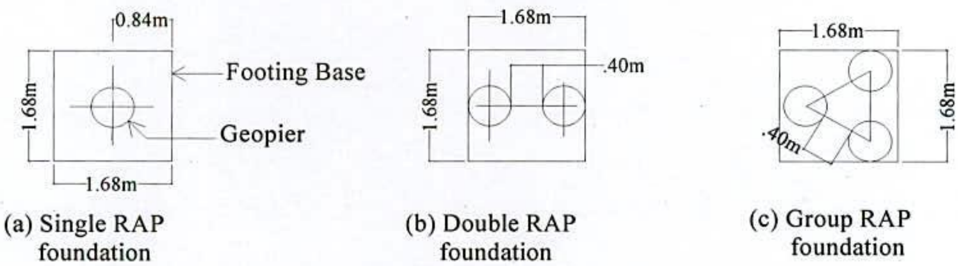


Figure 3.13 Layout plan of RAPs under footing.

3.7 Rammed Aggregate Piers Installation Techniques

There are various types of techniques for the installation of granular piles, sand compaction piles, Stone Columns and RAP. These are (i) Vibro-Displacement method (ii) Vibro-Replacement method, (iii) Vibro-Compozer method, (iv) Cased-Borehole method and (v) Rammed Aggregate Piers method. In this investigation, considering the practical situation of availability of construction techniques and equipments in Bangladesh, RAPs were installed completely manually. The equipment for this method is locally available and can be fabricated easily if required. Rammed Aggregate Piers installation consists of following five steps: (i) boring, (ii) casing, (iii) bottom plugging, (iv) pouring, and (v) compacting.

3.8 Methods to Investigate Rammed Aggregate Piers Performance

The methods suitable for countermeasures application are decided according to various conditions such as structural conditions following with ground or soil conditions, construction site conditions, economical feasibility, and execution condition. Most reliable method of obtaining ultimate bearing capacity at a site is to perform a load test on a full-size footing, which is not usually done since an enormous load would have to be applied, which eventually leads to high cost (Bowles 1997). The usual practice is to perform plate load test to avoid cost and related involvements. In some instance, penetration tests are also performed to depict the improvement of the strength of ambient sub-soil after the installation of columnar inclusions. Despite the problems with full-size footing load test, in this study, the bearing capacity of the improved ground was measured through load tests on full-size square footing of 1.68mx1.68m dimension placing on the both of natural and improved grounds at a depth of 750 mm measured from the existing ground surface. Sufficient dead load, more than the estimated capacity of footing based on the ground conditions, was placed on the ground and hence transfers through hydraulic jack to the top of column(300mmx300mm) sectioned at middle of the footing. Full-size footing is used as a Rammed Aggregate Pier's cape after completion of Rammed Aggregate Piers.

CHAPTER FOUR

RAMMED AGGREGATE PIER INSTALLATION AND FIELD INVESTIGATION

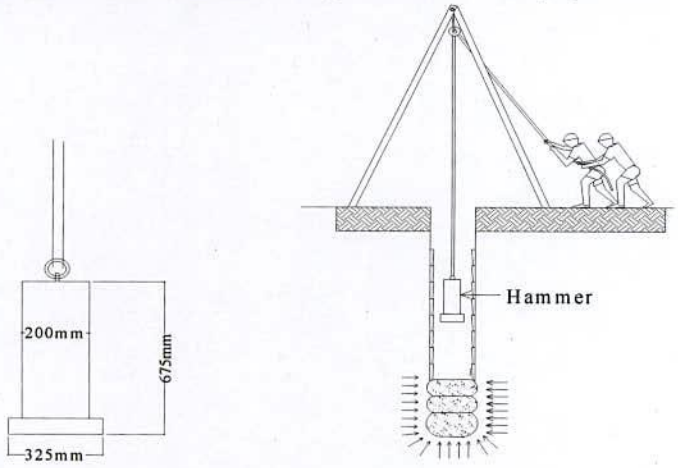
4.1 General

Rammed Aggregate Pier installation in the selected location and the field investigations of improved ground are described in this chapter. To install columnar inclusions (stone columns, granular piles, sand compaction piles, cement column, rammed aggregate pier, etc.) several methods ranging from conventional labor intensive to well-equipped have been practiced throughout the world. In Bangladesh, no well-equipments and techniques are readily available and hence practiced. For sand compaction piles installation, generally, manually operated dry displacement method has been practiced. In this investigation, considering the practical situation of the availability of construction techniques in Bangladesh, RAPs were installed completely manually using rammed aggregate method. The installation equipment was fabricated locally. Full-scale footing load tests were performed over the treated ground to observe the performance of RAP in improving soft ground.

4.2 Installation Methods and Equipment

For the installation of Rammed Aggregate Pier, Locally available method was employed. The methods consists three operational steps and the associated techniques. These are boring, pouring of granular materials and compaction. The boreholes of 0.70m diameter and 3.4m long were excavated manually using local earth digging tools. A suitable hammer is also required to compact the granular materials poured in the cylindrical hole.

In this study a hammer of 200mm diameter, 650mm length and 108kg weight made by iron is used for the densification of granular materials. The configuration of hammer is decided inconsistence with the suitability for compaction of granular materials. The weight of hammer is kept for such an amount suitable for manual operation. For having a wider area the hammer has an enlarged head of 325mm diameter. The tripod stand with rope-pulley system locally made for the operation of Standard Penetration Test (SPT), was used here for the free fall of hammer. Fig.4.1 shows the equipments.



(a) Hammer (b) Compaction by hammer

Figure 4.1 Schematic diagram of used hammer.

4.3 Installation Procedures

The RAPs were installed here completely manually using by rammed aggregate method. The boreholes of 0.70m diameter and 3.4m long measured from the existing ground surface were excavated manually, using local earth digging tools and the side of excavated boreholes were retained by using locally made burned clay ring of 650mm diameter, 150mm length and 10mm thickness, which were placed as the excavation proceeds. After the completion of boreholes, designated granular materials were placed in layer and hence compacted to get the required density. Total 14 layers are constructed in each RAP. The installation procedures of Rammed Aggregate Piers are described in the followings sections.

4.3.1 Boring

As the equipments for the excavation of large diameter cylindrical borehole are not readily available, locally available manual labor intensive technology for making borehole was adopted. Such technology were used long ago for the creation of well to get the source of drinking water. Skilled workers for making boreholes are also available. A borehole of 700mm diameter till the depth of 4.15m measured from the existing ground surface was excavated manually using locally available earth digging tools shown in Fig. 4.3. Bucket was used to carry up of excavated soil from borehole at deep depth. Every borehole is excavated to follow same vertical aliment and same diameter in all height at borehole. Rammed Aggregate Pier under footing was placed at various arrangements of single, double and group pattern. Every RAPs were excavated of same properties in terms of diameter and length. Same spacing was used for the double and group RAPs as 1110mm center to center.

Figure 4.2 Excavation of hole to install RAPs.

4.3.2 Retaining Borehole

During excavation of borehole, it is realized that excavated borehole sides are to be retained to avoid possible collapse and caving during the construction of Rammed Aggregate Pier. Since the casing required to support the sidewall will not be removed as the pouring of granular materials proceeds. It is decided to use locally available burned clay ring, which will remain in place even after the completion of column. Infact during the compaction of granular materials, the ring was broken into pieces and mixed with the granular materials. Burned clay ring of about 650mm diameter, 150mm length and 10mm thick were placed to protect the borehole side from collapse as the excavation proceeds. The dimension of casing and its placement in the hole as excavation proceeds are as shown in Figs.4.3 and Fig.4.4. Burned clay ring is placed one after another with consider ring's top and bottom patterns and groove. Earth excavation and placing of clay ring both are done parallel till required excavation depth.

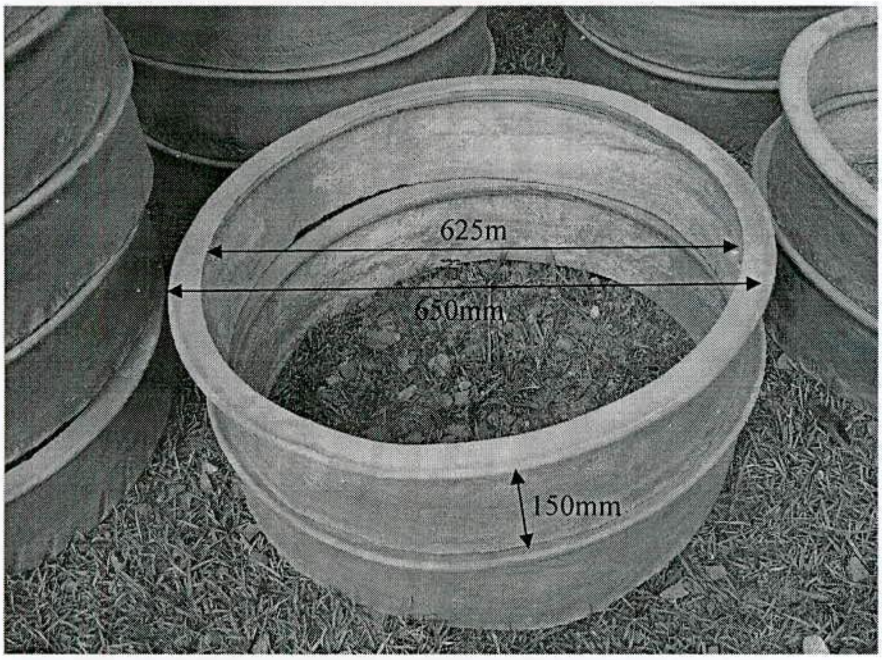


Figure 4.3 Dimension of burned clay ring used as casing.

Figure 4.4 Placement of casing during the excavation hole for RAP construction.

4.3.3 Bottom plugging

After the completion of borehole the properly mixed granular materials at designated proportion, were placed at the bottom and sufficiently compacted with a hammer of 200mm diameter, 650mm length and 108kg weight to make a bottom plug. Large amount of granular materials are placed at the bottom at first to make a stiffer layer, is called as bottom plug. Thickness of bottom plug is larger than the ordinary layer of granular material of RAP as shown in Fig. 4.7. The diameter of the bottom plug is also larger than the diameter of the shape.

3.3.4 Pouring of Granular Materials

After the construction of bottom plug, granular materials were placed in layers having initial thickness of around 350mm and hence compacted properly by dropping the designated hammer. Pre-set type of granular materials prepared by mixing of brick aggregate and local sand is placed as shown in Fig.4.5. After completion of first layer, granular materials are poured on it to prepare the next layer. Amount of granular material in each layer is maintained as same amount. Granular materials were used approximately same quantity in each layer and were used approximately same quantity in each complete Rammed Aggregate Pier. In each layer the amount of granular materials was 6 to 7 Juri (bamboo made basket) or 4.2 to 4.9cft (1Juri = 0.7cft) and in each completed RAP contained the granular material of 72 to78 Juri or 50.4 to 54.6 cft.

Figure 4.5 Pouring of mixed granular materials.

4.3.5 Compaction

A hammer of 200mm diameter, 650mm length and 108kg weight is used to compact the granular materials of each layer and also to construct the bottom plug. The granular materials were compacted by used method of standard proctor test (ASTM Designation D-1577). The compaction effort in the standard Proctor test is equal to

$$[(5.5 \text{ lb/blow}) (3 \text{ layers}) (25\text{blows/layer}) (1\text{-ft drop})]/ (1/30) \text{ ft}^3 = 12{,}375 \text{ ft-lb/ft}^3$$

$$(=593 \text{ kj/m}^3)$$

By flow of this method the freefall of hammer (about 750mm) was maintained through the compaction period. Total 45 number of hammer drops were provided in each layer and hence obtained a compacted layer of around 225mm thickness. The tripod stand with rope-pulley system, which is locally made for conducting Standard Penetration Test, was used here for the free fall of hammer as shown in Fig.4.6. Pouring and compacting were then repeated and continued till the Rammed Aggregate Pier reached the ground surface to have a compacted and completed Rammed Aggregate Pier. The schematic diagram of this completed RAP with the followed installation sequence is shown in Fig.4.7. The RAPs were installed in three patterns; namely, Single, Double, and Group, are shown in the Fig.4.8, with the dimension. In case of double and group RAPs, borehole excavation and installation was done sequentially after the completion of first one.

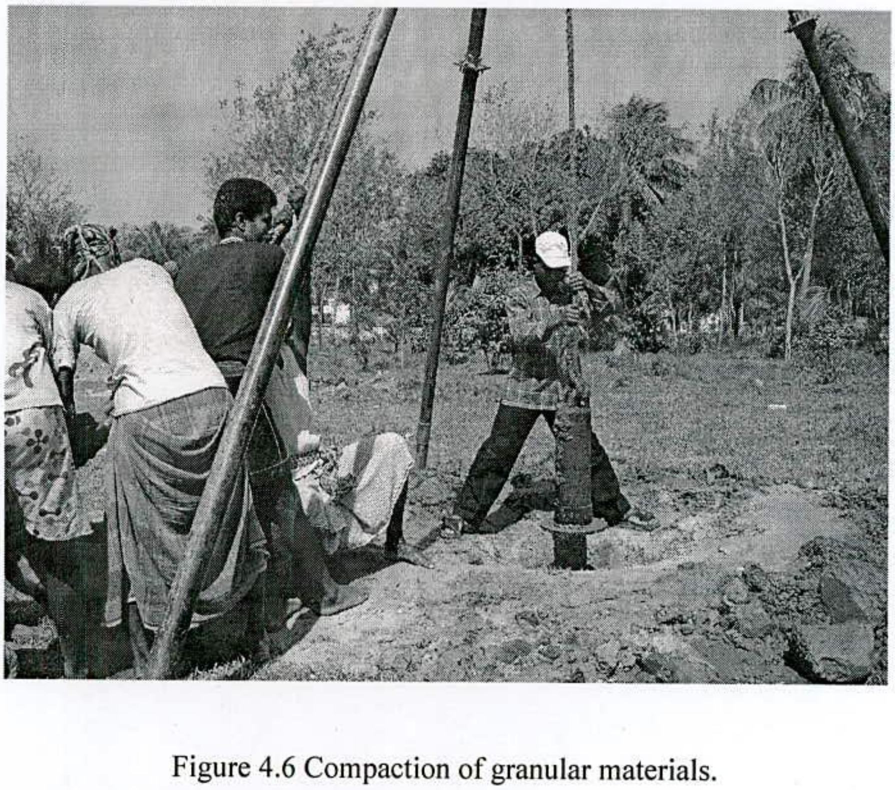


Figure 4.6 Compaction of granular materials.

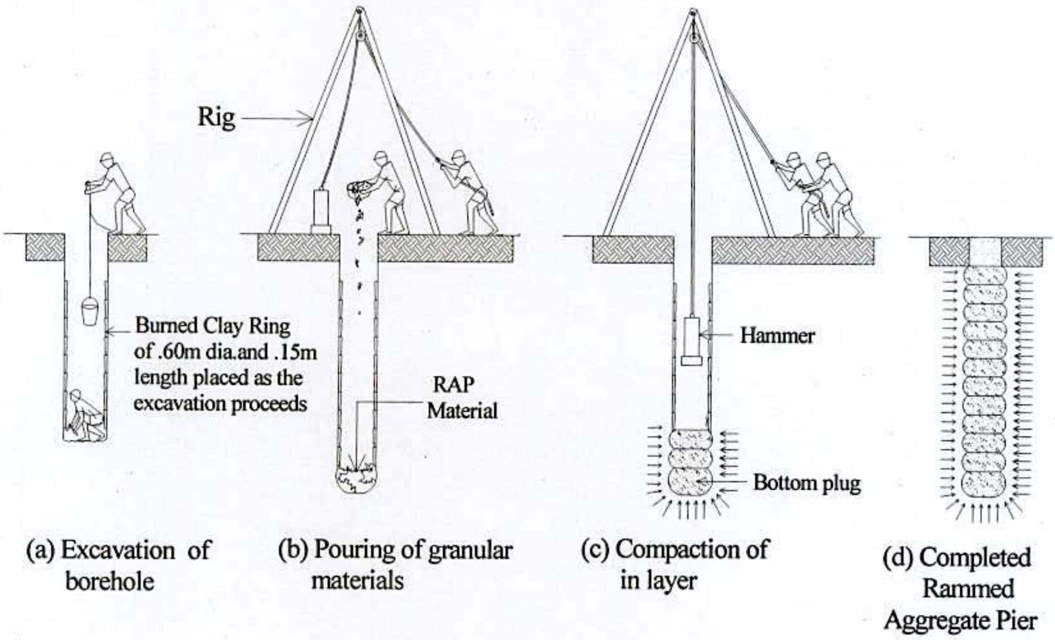


Figure 4.7 Schematic diagram of installation process of RAP used in this study.

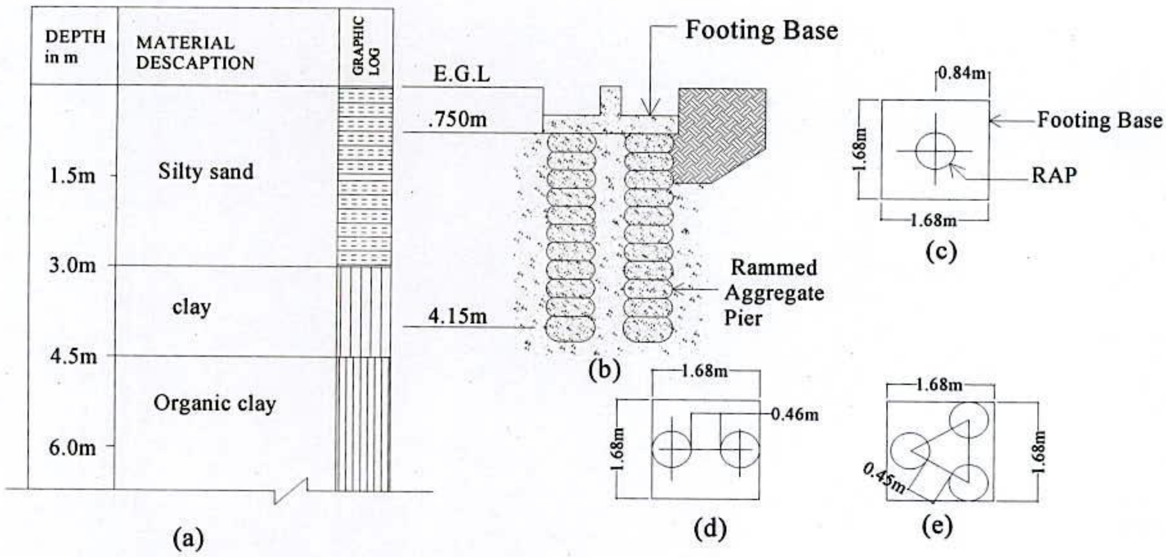


Figure 4.8 Different arrangements of RAP: (a) sub-soil profile, (b) section of installed RAP under footing, (c) plan of single RAP (d) plan of double RAP (d) plan of group RAP.

4.4 Monitoring of the Installation Process

The progress of the installation of RAPs were carefully observed and recorded at the site to ensure the desired quality. In this regard, the following items were checked and noted during the construction of each RAP.

4.4.1 Dimension of bore hole

The proper dimension of Rammed Aggregate Pier (RAP) such as diameter and length mainly depends on the size and shape which is excavated manually by earth digging tools of bore hole. It is very important to make the uniform its of the diameter of borehole along the depth. To ensure the designated RAP length, bottom tip elevation, and the diameter of RAP before and after constructions continuous physical inspections was done during the excavation of borehole. In case of double and group RAP to ensure the designated center to center distance within the RAP and the uniformly of the dimension, close monitoring is required since 2nd borehole was excavated after the completion of the construction of first RAP. Surrounding soil condition of RAP and time of installation begins where observed carefully. The time required to create the cylindrical hole till the designated depth is also recorded.

4.4.2 Protection against caving

Caving in to the ambient soil of the RAP is an inevitable consequence while excavate a borehole for the installation of RAP. Even during the densification of granular materials of RAP, possibility of caving is very high. Special attentions were given as the placement of casing while excavation proceeds and during the compaction of granular materials. To avoid caving, the withdrawal of incoming water into the borehole was done carefully and also excessive withdrawal was avoided. Proper alignment of the hammer during tamping was ensured so that the hammer does not hit the side, while might creates caving.

4.4.3 Bottom plug

After the completion of borehole, appropriate measure is required to seal the bottom of the borehole by strong bottom layer of granular materials. The success of RAP largely depends on the quality of the bottom seal, here known as bottom plug. To ensure the quality, special care for compaction of bottom plug was taken and the granular material having particle size was used. To determine the actual length, the amount of material required was determined. Required attentions were given at the field while constructions bottom plug to ensure the similarity of stiffness and dimension of all RAP.

4.4.4 Quality and quantity of the granular materials

Quality of the mixed granular materials according to the designated specification as its mixture proportion of sand and brick aggregates is required to ensure to these observations and recording system achieve the good quality Rammed Aggregate Piers were made to ensure the same quality of the all constructed RAPs. With great care through continuous monitoring carefully determined the volume of the granular material required for each layer and ultimately for the construction of a complete RAP. Same amount of granular materials were maintained to pour in all the construction RAP having same length and diameter.

4.4.5 Compaction efforts

Compaction effects are one of the main factors that ensure the designated quality of Rammed Aggregate Pier. Here, the number of blows in each layer and the height of free fall were maintained with close monitoring in order ensure the similarity of all the constructed RAP, since the compaction was done manually. Another important aspects were considered during compaction is that retain of the side of excavated borehole against possible damage due to wrong placement of hammer during dropping. the same compaction rate interns of time was also maintained by ensuring the equal time required for the compaction of each layer.

4.4.6 Consumption of granular materials

Total consumption amount of granular material for each boreholes are calculated which are described briefly in Table 4.1. From this Table it is shown that the maximum and minimum consumption quantity of boreholes varies from 75.00cft to 84.00cft.

Table 4.1 Consumption of granular materials in borehole

SI. No.	Description of borehole	No. of borehole	Consumption material unit	Consumption quantity of granular materials (Brick agg.: sand = 2:1)
1	Borehole of Single RAP	1	cft	80.00
2	Boreholes of Double RAPs	2	cft	82.00
		3	cft	84.00
3	Boreholes of Group RAPs	4	cft	82.00
		5	cft	75.00
		6	cft	81.00

4.5 Construction of Footing

The performance of Rammed Aggregate Piers was investigated using the load test of real size footing in the field. After the completion of Rammed Aggregate Piers installation, a Full-size square footing of 1.68mx1.68m having a column of 0.30mx0.30m size at the center was constructed on the both of natural and improved grounds at a depth of 750 mm from the existing ground surface. After completion of Rammed Aggregate Piers (Fig.4.9) the same size footings were constructed on natural ground and the single, double and group Rammed Aggregate Piers treated ground. Structural design of each type of footing is considered to its loading capacity which load is transferred by column footing on below footing ground as shown in Fig. 4.10 and reinforcement design of each type of footing are shown in Appendix – B.

Figure 4.9 Completed Rammed Aggregate Piers in the ground.

Figure 4.10 Construction of footing on Rammed Aggregate Piers treated ground.

4.6 Field Load Test

Most reliable method of obtaining ultimate bearing capacity at a site is to perform a load test on a full-size footing, which is not usually done since an enormous load would have to be applied, which eventually leads to high cost (Bowles 1997). The usual practice is to perform plate load test to avoid cost and related involvements. Despite the problems with full-size footing load test, in this study, the bearing capacity of the improved ground was measured through load tests on full-size footing on the both of natural and improved ground. The employed method for load test and the related aspects are described in the following sections.

4.6.1 Method of load test

In the present study, the degree of improvement of the treated ground was measured through load tests on a square full-size footing of 1.68mx1.68m placing on both the natural and improved grounds. Sufficient dead load, more than the estimated capacity of footing based on the ground conditions, was placed on the platform and hence transfers through ultimate the hydraulic jack to the top of column of 300x300mm at positioned middle of the footing. The schematic diagram of a typical load test is shown in Fig. 4.11. The procedure has been standardizing as ASTMD1194, which is essentially as follows;

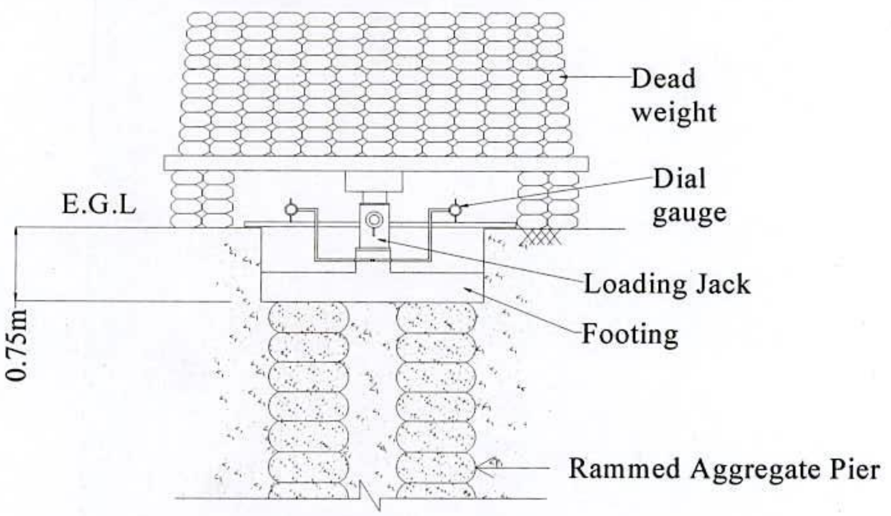


Figure 4.11 Schematic diagram of typical load test on footing.

The procedure has been standardizing as ASTMD 1194, which is essentially as follows;

i. A load arrangement is placed with sufficient height to the column footing at which centered of load is acted on footing column. Typical load arrangement and set-up for full scale footing load test are shown in Figs. 4.12 and 4.13.

ii. The jack is placed in the central column and the load is applied by means of a hydraulic jack which is supported by reaction beam.

iii. The reaction to the jack is provided by means of a loaded platform.

iv. The load is applied in equal increment of about one-eights of the estimated allowable load capacity. The settlement is recorded with the help of dial gauges of sensitivity. 01mm fixed to an independent datum bar.

v. Load increments of each test were continued till the settlement was reached to 25mm. At each level of load increment, the readings were continued till the rate of settlement less than 0.25mm per hour and rebound readings were also recorded at four steps. The settlement is by two deformation dial gauge mounted from a position not effected by the settlement of the footing. (In this project, the test was continued till a total settlement of 25mm recorded.)

vi. The results are plotted in the form of load settlement curve.

Figure 4.12 Set up of load arrangement for footing load test.

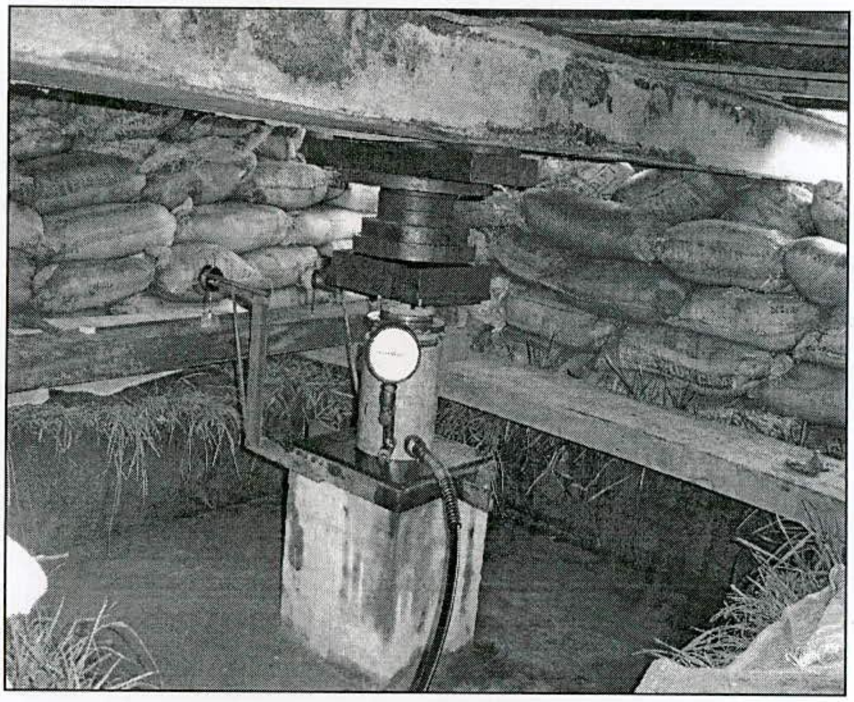


Figure 4.13 Full scale footing load test arrangement

4.6.2 Constraints of load test

The usual practice is to perform plate load test to avoid cost and related involvements. From this field load test method we are obtained most accurate result than plate load test method. The inherent constraints of plate load test are described in the followings:

i. Effect of load test equipment: Though the full-size footing load test is most reliable and accurate method to determine the bearing capacity of soil but this test is used high loads, which equip mental arrangement is very costly. The costing amount of the full-size footing test is comparatively very high than that of plate load test.

ii. Scale effect: The ultimate bearing capacity of saturated clays is independent of the size of the footing but for cohesion less soil, it increases with the size of the footing.

Though load test of the full-size footing has several short-comings and limitations but this test is used extensively because of its result value is more accurate than other method as like plate load test.

4.7 Set-up of Field Load Test

At field load test, it is very essential to ensure accuracy of load arrangement and load measuring jack. Bottom support and bottom platform is placed carefully and measured accurately their load sustainability which is made by sand bag, wood and cast iron girder. Sand bag is used and placed in binding pattern on load platform. Loading jack which is used for test was checked through celebration after fill up its oil chamber by the respective oil.

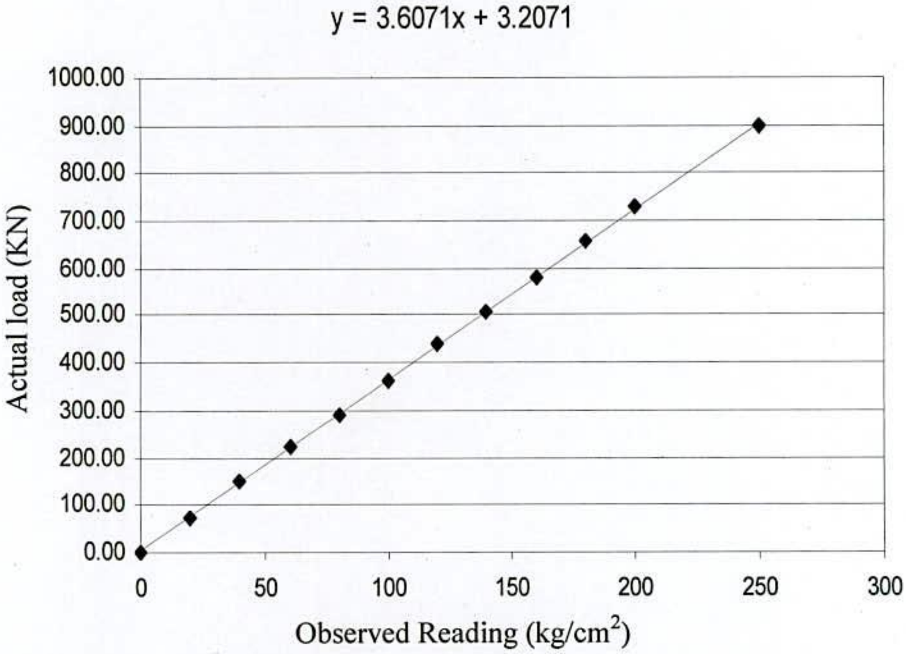


Figure 4.14 Calibration of hydraulic jack used in footing load test.

Figure 4.14 shows the calibration of loading jack, which is used to measure load carrying capacity of the footing. From this graph it can be seen that the observed and actual load of test have followed a definite relationship. Observed reading is presented on loading gage and actual load is measured from observed reading by use of equation Y=3.6071X +

3.2071. Here Y represent actual load and X represent observed reading. Load measured capacity of this machine is varied from 51kN to 1000kN, with an accurately recorded interval of 1kN

4.8 Execution of Field Load Test

The bearing capacity of the natural and treated ground was measured through load tests at field on full-size footing placed on natural and treated grounds. Sufficient quantity of dead load, more than the estimated capacity of footing based on the ground conditions, was placed on the ground at a sufficiently stable platform and hence transfers through hydraulic jack to the top of column sectioned positioned at the middle of the footing. The loading arrangement is similar as of followed for pile load test. The settlement is measured by two deformation dial gauge place at left and right position and mounted at a safe distance not affected by the settlement of the footing. Typical load arrangement and transfer of load is shown in Figure 4.10. Each load increment is one-eights of the estimated bearing capacity of the footing. However, load increments were continued till the settlement was reached to 25mm for each test. For each load increment settlement measurements were taken till the rate of settlement falls to the acceptable value. After the completion of loading, rebound readings were recorded for the unloading at four steps till the total load removal. The settlement records during loading and unloading, and the time intervals were followed as the standard pile load test method (ASTM D1194). Four number of load tests are completed which features and results are described below.

4.8.1 Natural ground

Load test was performed on full-size footing of 1.68x1.68m placed on natural ground at a depth of 750 mm measured from the existing ground surface to determine the load-settlement response. The bearing capacity of the footing was estimated for this ground condition using Terzaghi's equation. Sufficient dead load, more than the estimated capacity was placed on the prepared platform at ground and hence transferred through hydraulic jack to the top of column (300x300mm) positioned at middle of the footing. The settlement is measured by two deformation dial gauge positioned at left and right side and mounted from a position not effected by the settlement of the footing. The load increment amount was considered as 10.50 kN/m^2, which is equivalent of 3.02 tons. The loadings were continued to apply up to ten increments, i.e. 141.33kN/m^2 (40.6tons), which resulting a settlement of 25mm. Total loading time was 18 hours. Unloading were done in four steps and kept it for 6 hours after the total removal of load. Total time required for unloading was 8 hours. At each level of load increment, the readings were continued till the rate of settlement less than 0.25mm per hour and rebound readings were also recorded at four steps. The load-settlement-time diagram is shown in Fig. 4.15. The applied loading and unloading steps with elapsed time and the settlement of footing for the same elapsed time for the loading and unloading paths are also shown in Fig.4.15. From this figure, it can be seen, that the maximum settlement observed as 25mm at the load intensity on the footing as 141.33kN/m^2.

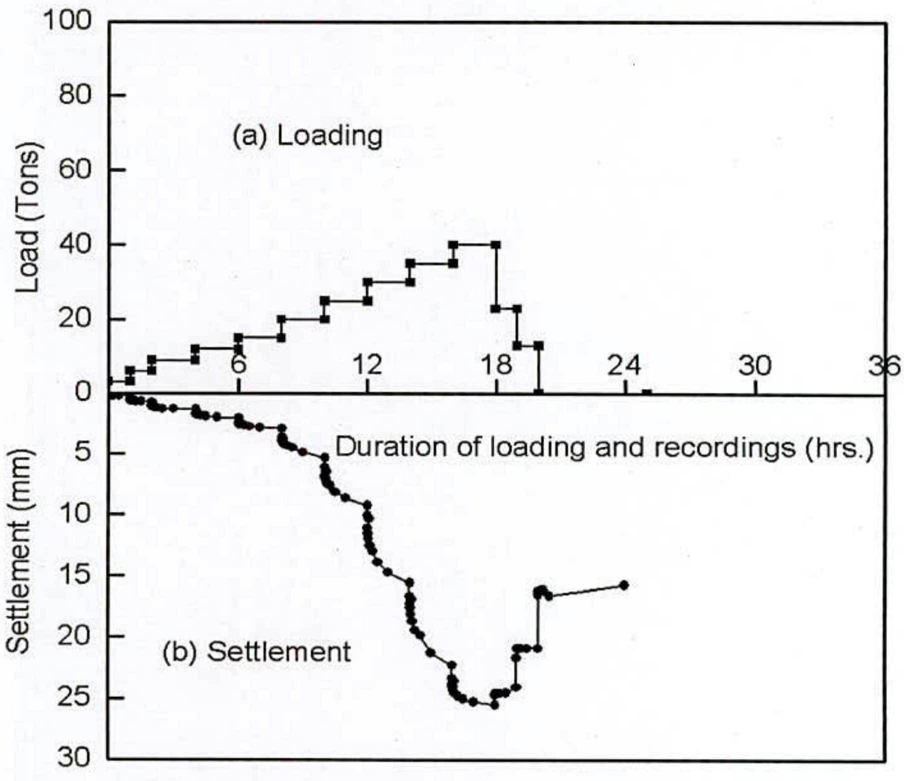


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Figure 4.15 Load-settlement-time response of natural ground obtained from full-scale footing load test.

4.8.2 Single Rammed Aggregate Pier treated ground

Load test was performed on full-size footing of 1.68mx1.68m placed on single Rammed Aggregate Pier treated ground at a depth of 750mm measured from the existing ground surface to determine the load-settlement response. The bearing capacity of the footing was estimated for this ground condition using Terzaghi's equation. Similar as untreated ground sufficient dead load, more than the estimated capacity was placed on the prepared platform at ground and hence transfer through hydraulic jack to the top of column (300mmx300mm) positioned at middle of the footing. The settlement is measured by two deformation dial gauge positioned at left and right side and mounted from a position to minimize the effect due to the settlement of the footing. The load increment amount was considered as 20.50kN/m^2, which is equivalent of 5.9tons. The loadings were continued to apply up to ten increments, i.e.177.18kN/m^2 (50.9 tons), which resulting a settlement of 25mm. Total loading time was 18 hours.

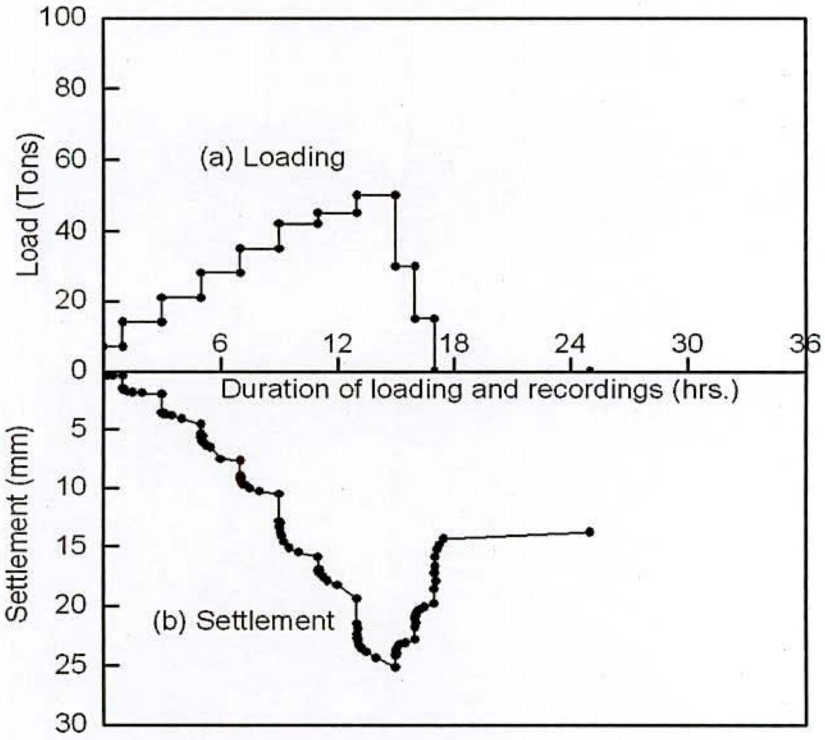

Figure 4.16 Load-settlement-time response of single RAP treated ground obtained from full-scale footing load test.

Unloading were done in four steps and kept it for 6 hours after the total removal of load. Total time required for unloading was 8 hours. Each level of load increment is one-eights of the estimated capacity. The load settlement response was measured in the similar way to that of the untreated ground. However, load increments were continued till the settlement was reached to 25mm. At each level of load increment, the readings were continued till the rate of settlement less than 0.25mm per hour. After completion of loading till the designated level, rebound readings were also recorded at four steps till the load reached to zero. Typical load-settlement-time diagram is shown in Fig.4.16. From this figure, it can be seen, that the maximum load intensity on the footing is 177.18kN/m^2 corresponding to the settlement of 25 mm.

4.8.3 Double Rammed Aggregate Pier treated ground

The same load test system is also followed to observe the load-settlement response of the ground improved by the installation double RAPs. Load test was performed on full-size footing of 1.68mx1.68m placed on double Rammed Aggregate Pier treated ground at a depth of 750 mm measured from the existing ground surface to determine the load-settlement response. The bearing capacity of the footing was estimated for this ground condition using Terzaghi's equation. Similar to the loading test performed on the untreated ground, sufficient dead load which is more than the estimated capacity was placed on the prepared platform at ground and hence transferred through hydraulic jack to the top of column (300x300mm) positioned at middle of the footing. The settlement is measured by two deformation dial gauge positioned similar as previous test. The load increment amount was considered approximately as 30.0kN/m^2, which is equivalent of 8.6 tons. The loadings were continued to apply till the settlement reach to 25mm, which was occurred at the applied load intensity of 254.25kN/m^2 (73.1 tons).

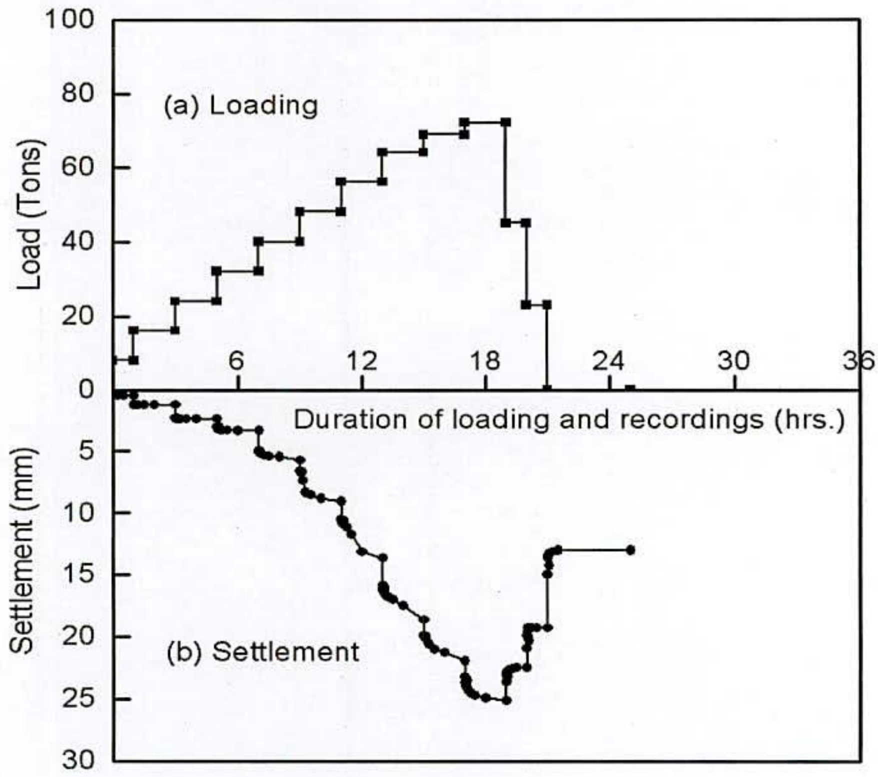

Figure 4.17 Load-settlement-time response of double RAP treated ground obtained from full-scale footing load test.

Total loading time was recorded as 18 hours. Unloading was done in four steps and continued to recording the settlement till the 6 hours after the total removal of applied load. Total time required for unloading was recorded as 8 hours. The interval of load increment is one-eights of the estimated capacity. The result of load settlement response was measured as like as untreated ground. However, load increments were continued till the settlement was reached to 25mm. At each level of load increment, the readings were continued till the rate of settlement less than 0.25mm per hour and rebound readings were also recorded at four steps. Typical load-settlement-time diagram is shown in Fig.4.17. From this figure, it can be seen, that the maximum settlement observed as 25 mm at the load intensity on the footing as 254.25kN/m^2.

4.8.4 Group Rammed Aggregate Pier treated ground

Load test was performed on full-size footing of 1.68mx1.68m placed on group Rammed Aggregate Pier treated ground at a depth of 750 mm measured from the existing ground surface to determine the load-settlement response. The bearing capacity of the footing was estimated for this ground condition using Terzaghi's equation. Similar as untreated ground sufficient dead load, more than the estimated capacity was placed on the prepared platform at ground and hence transfer through hydraulic jack to the top of column (300x300mm) positioned at middle of the footing. The settlement was measured in the same way as described earlier. The load increment amount was considered as 35.0kN/m^2, which is equivalent of 10.0tons. The loadings were continued to apply up to ten increments, i.e. 277.29kN/m^2 (79.7tons), which resulting a settlement of 25mm. Total loading time was 18 hours. Unloading were done in four steps and kept it for 6 hours after the total removal of load.

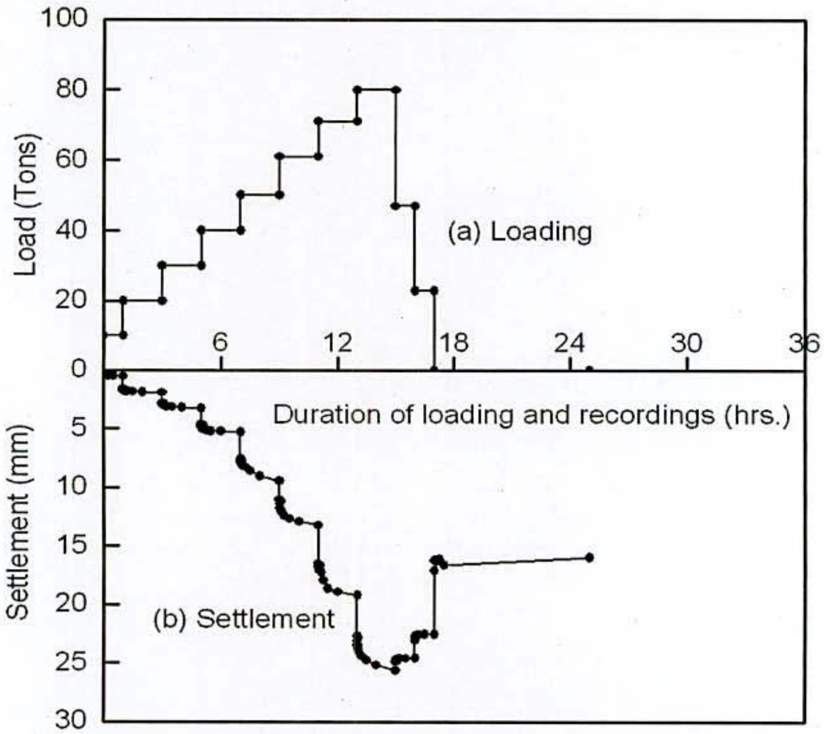

Figure 4.18 Load-settlement-time response of group RAPs treated ground obtained from full-scale footing load test.

Total time required for unloading was 8 hours. Typical load arrangement and transfer of load is shown in Fig. 4.11. Each level of load increment is one-eights of the estimated capacity. The result of load settlement response was measured as like as untreated ground. However, load increments were continued till the settlement was reached to 25mm. At each level of load increment, the readings were continued till the rate of settlement less than 0.25mm per hour and rebound readings were also recorded at four steps.Typical load-settlement-time diagram is shown in Fig. 4.18. From this figure, it can be seen, that the maximum load carrying capacity of the footing on the group RAPs is 277.29kN/m^2 resulting a settlement of 25 mm.

4.9 Cost Analysis

The total estimate cost of Rammed Aggregate Pier is calculated for several factors, cost of granular materials, labour cost, Installation equipment cost, etc. The construction cost of single Rammed Aggregate Pier is calculated take 4874.00 which material cost is Taka 2594.00 is shown in Table C. 2 and labour cost is Taka 2280.00 is shown in Table C.1. In this study total six number of RAPs are constructed by three types of pattern, single, double and group RAPs, which total construction costing value was taka 29244.00. Their details estimates are described in Appendix – C.

In the present study, the degree of improvement of the treated ground was measured through load tests on a square full-size footing of 1.68mx1.68m placing on both the natural and improved grounds. Four types of full-size footing load test are completed in this study. So, construction cost of footing and cost of load test are considered to estimate total cost of load test. Total cost for four types load test of Full-size footing on natural ground, single RAP treated ground, double RAPs treated ground and group RAPs treated ground are estimated respectively Take 17854.60, 19133.80, 21764.00 and 23628.20, which details are described in Appendix – D.

CHAPTER FIVE

RESULTS AND DISCUSSIONS

5.1 General

The results obtained from field investigations are presented and hence discussed in this chapter. Load tests were performed on improved ground for different number of Rammed Aggregate Pier installation and also on natural ground. The load-settlement curves as measured from full-size footing load tests on natural and improved ground of single, double and group Rammed Aggregate Pier. Both the loading and unloading responses are illustrated as measured from loading tests.

5.2 Load Settlement Response of Natural Ground

The load settlement response of natural ground obtained from full-size footing load tests is shown in Fig.5.1 and Table 5.1. From this figure it can be seen that settlement increases with the application of load. The full-size footing moved downward without moving any resistance after the application of the load beyond 141 kPa. At this level of load intensity, the settlement is more than 25mm and plastic deformation of soil occurs. The figure show that the mode of failure under the footing is combined of general and punching shear type failure. At the lower level of load is up to50 kN/m^2, the response is general shear type, beyond this load, the mode of failure is punching type.

The loading was continued till the 140 kN/m^2, which yield the settlement more than 25mm. The rebound curve shows that the significant amount of plastic deformation occurred only 10mm settlement rebounded due to the total removal of load.

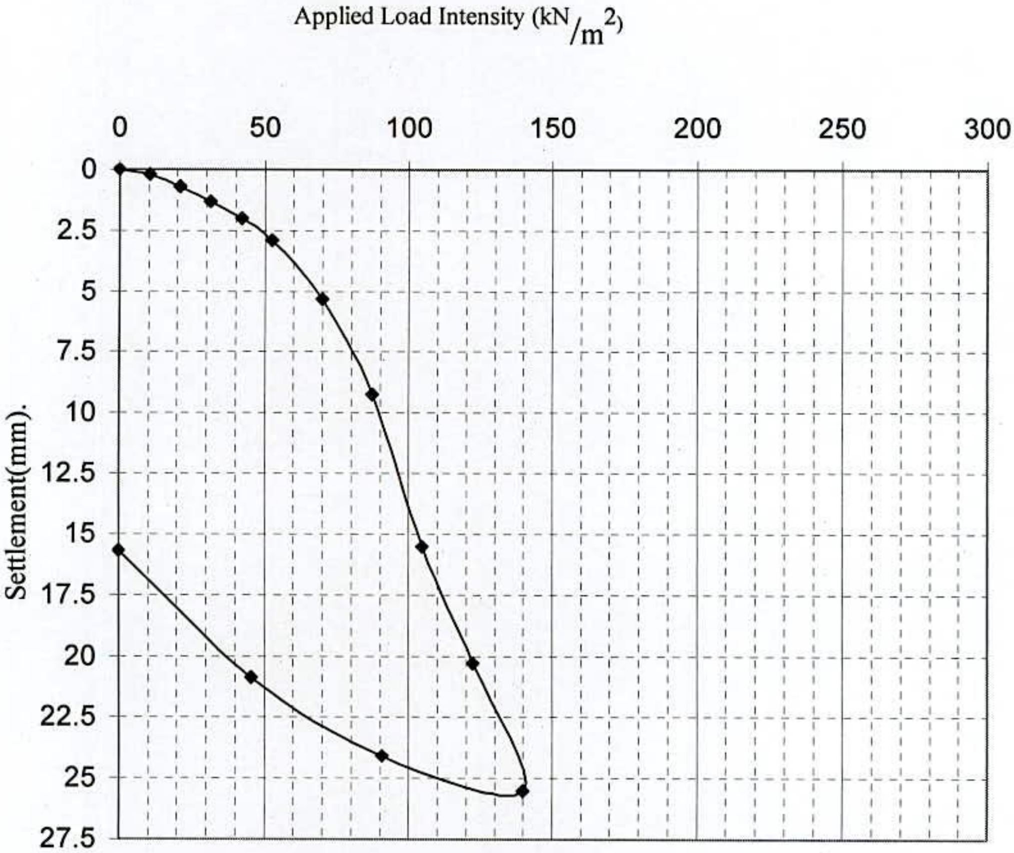

Figure 5.1 Applied load intensity versus settlement curve on the natural ground.

Table: 5.1 The measured value of settlement of natural ground with applied load.

Applied Load Intensity (kN/m^2)	0	10.49	20.98	31.46	41.95	52.44	69.9	87.4	104.88	122.4	141.33	90.89	45.45	0
Settlement (mm)	0	0.2	0.7	1.31	2.02	2.9	5.33	9.25	15.54	20.3	25.51	24.1	20.89	15.7

5.3 Load Settlement Response of Rammed Aggregate Pier Treated Ground

The load settlement response of treated ground obtained from full-size footing load tests is described here. The single, double and group Rammed Aggregate Pier improved ground were investigated and hence described here. In all these case both the loading and unloading responses are illustrated as measured from field loading tests. The figures are described in the following sections.

5.3.1 Single Rammed Aggregate Pier treated ground

The load test was performed to determine the load carrying capacity of single Rammed Aggregate Pier treated ground. The load settlement response of treated ground obtained from full-size footing resting on single Rammed Aggregate Pier treated ground is shown in Fig.5.2 and Table 5.2. From this figure it can be seen that the settlement increases with the application of load. The full-size footing moved downward without moving any resistance after the application of the load beyond 177kPa. At this level of load intensity, the settlement is 25.11mm and plastic deformation of soil occurs. The figure shows that the mode of failure under the footing is general shear type.

The loading was continued till the applied load of 180 kN/m^2, which yield the settlement more than 25mm. The rebound curve shows that the significant amount of plastic deformation occurred only 10mm settlement rebounded due to the total removal of load.

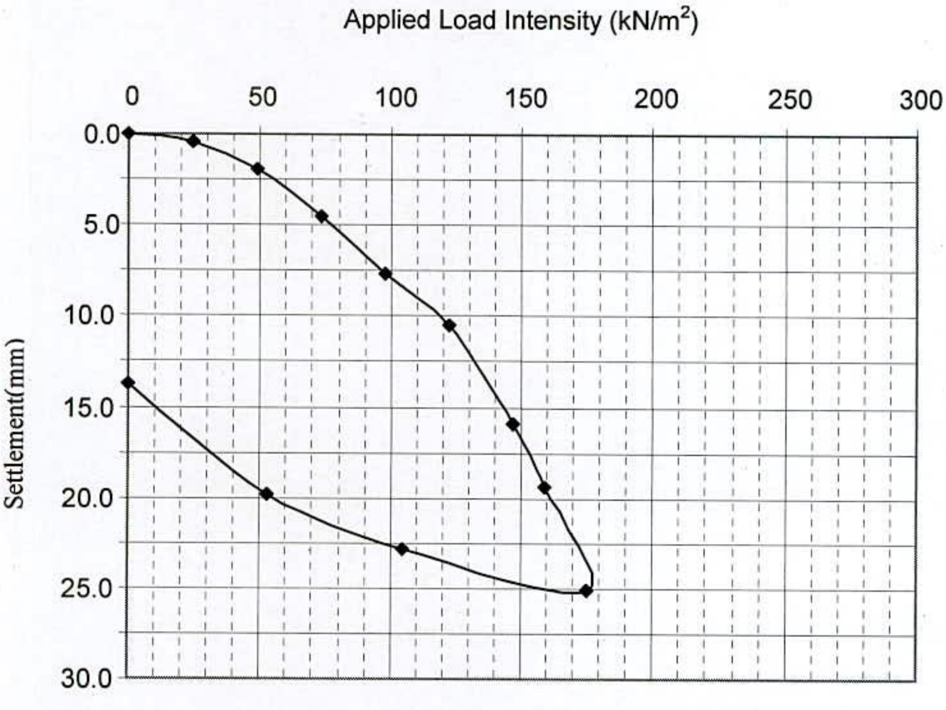

Figure 5.2 Load versus settlement curve of the footing resting on the single RAP treated ground.

Table 5.2 The measured value of settlement of single RAP treated ground with applied load.

Applied Load Intensity (kN/m2)	0	24.47	48.94	73.41	97.89	122.36	146.83	159.32	177.18	104.88	52.44	0
Settlement (mm)	0	0.39	1.96	4.56	7.68	10.49	15.91	19.38	25.11	22.82	19.79	10.76

5.3.2 Double Rammed Aggregate Pier treated ground

The load settlement response of treated ground obtained from full-size footing resting on double Rammed Aggregate Pier treated ground is shown in Fig.5.3 and Table 5.3. From this figure it can be seen that settlement increases with the application of load. The full-size footing moved downward without moving any resistance after the application of the load beyond 254kPa. At this level of load intensity the settlement is 25.12mm and plastic deformation of soil occurs. The figure shows that the mode of failure under the footing is general shear type. The loading was continued till the 254 kN/m^2 which yield the settlement more than 25mm. The rebound curve shows that the significant amount of plastic deformation occurred only 12.5mm settlement rebounded due to the total removal of load.

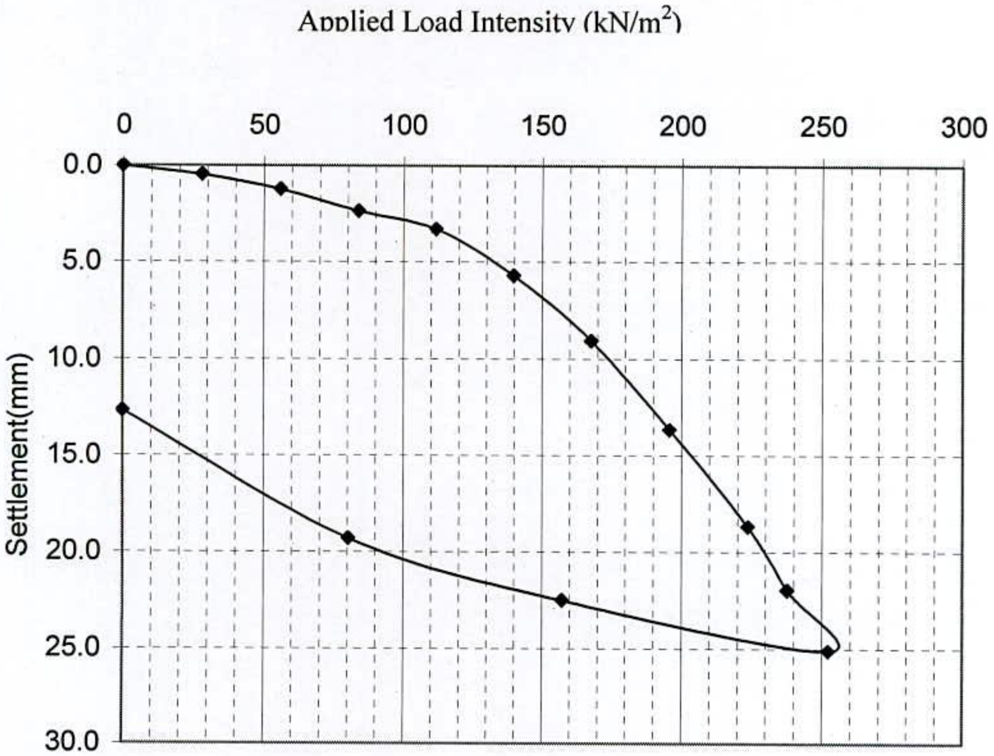

Figure 5.3 Load versus settlement curve of the footing resting on the double RAP treated ground.

Table 5.3 The measured value of settlement of double RAP treated ground with applied load.

Applied Load Intensity (kN/m2)	0	27.97	55.93	83.9	111.9	139.84	167.8	195.77	223.74	237.72	254.25	157.32	80.41	0
Settlement (mm)	0	0.47	1.25	2.39	3.32	5.73	9.07	13.66	18.68	21.98	25.12	22.5	19.32	12.68

5.3.3 Group Rammed Aggregate Pier treated ground

To determine the load carrying capacity of treated ground obtained from full-size footing resting on group Rammed Aggregate Pier treated ground are shown in Fig.5.4 and Table.5.4. From this figure it can be seen that settlement increases with the application of load. The full-size footing moved downward without moving any resistance after the application of the load beyond 277kPa. At this level of load intensity, the settlement is 25.62mm and plastic deformation of soil occurs. The figure shows that the mode of failure under the footing is general shear type. The loading was continued till the 277kN/m^2 which yields the settlement more than 25mm. The rebound curve shows that the significant amount of plastic deformation occurred only 10mm settlement rebounded due to the total removal of load.

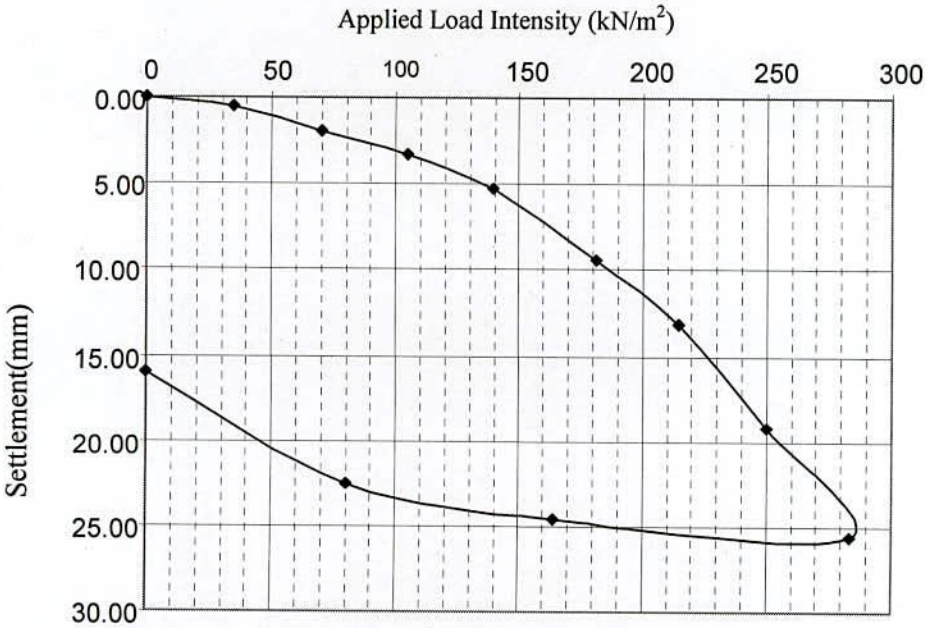

Figure5.4 Load versus settlement curve of the footing resting on the group RAP treated ground.

Table : 5.4 The measured value of settlement of group RAPs treated ground with applied load.

Applied Load Intensity (kN/m^2)	0	34.96	69.92	104.88	139.84	181.8	209.75	244.71	277.29	164.31	80.41	0
Settlement (mm)	0	0.54	1.94	3.29	5.29	9.45	13.2	19.2	25.62	24.56	22.51	14.94

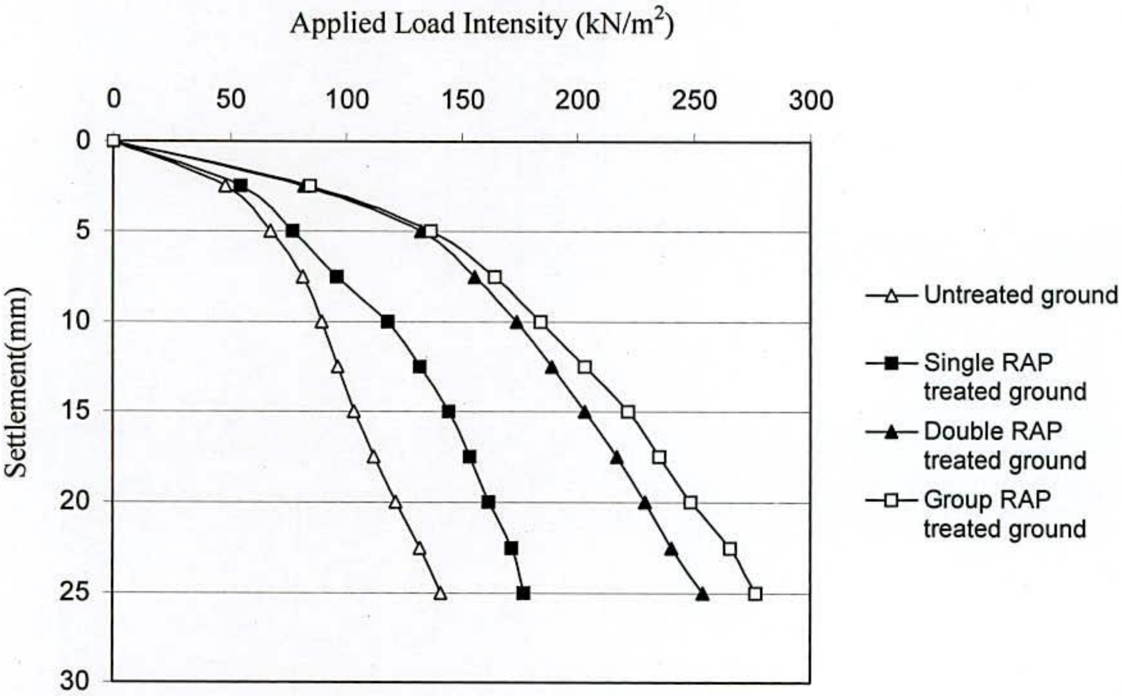

Figure 5.5 Load-settlement response of footing resting on both the natural and improved ground

Table 5.5 The measured value of settlement of treated and untreated ground with applied load.

Settlement(mm)		0	2.5	5	7.5	10	12.5	15	17.5	20	22.5	25
Applied load (KN/m^2)	Untreated ground	0	48.2	67.5	81.5	89.7	96.5	103.5	112	121.6	132	141
	Single RAP treated ground	0	54.5	77	96	118	132	144.5	153.5	162	172	177
	Double RAP treated ground	0	82	132.5	156	174	189	203.5	217.5	229.5	241	254
	Group RAP treated ground	0	84.4	136.7	164.5	184.5	203.5	222	235.5	249	266	276.5

Table 5.6 Degree of improvement of Rammed Aggregate Pier improved ground.

Ground condition	Ultimate bearing capacity, q_{ult} (kN/m^2)	Degree of improvement (ratio of ultimate bearing capacity of improved ground and natural ground)
Natural ground	141	1
Single RAP improved ground	177	1.25
Double RAP improved ground	254	1.8
Group RAP improved ground	277	1.96

5.4 Degree of Improvement

The result shows that the bearing capacity of single, double and group Rammed Aggregate Pier treated ground have increased to 1.5, 1.8 and 1.96 times compared to that of untreated ground, which are shown in Table 5.6. Load settlement comparison result of natural ground and treated ground of single, double and group Rammed Aggregate Piers are expressed in Figure 5.5. The load intensity of double and group Rammed Aggregate Pier treated ground is 1.48 and 1.56 times higher than that of single Rammed Aggregate Pier treated ground, respectively. The result also reveals that the group Rammed Aggregate Pier improved ground yields nearly equal increase of ultimate bearing capacity with that of double- Rammed Aggregate Pier improved ground having same spacing of RAP. This finding depicts under a square footing installation of double Rammed Aggregate Pier yields better results than its other two counterparts have been investigated in this study.

CHAPTER SIX

COMPARISON OF PREDICTION AND MEASUREMENT

6.1 General

Ultimate bearing capacity of Rammed Aggregate Pier improved ground is measured by various methods available in the literature and hence compared with the measured value obtained through full-scale footing load test in the field as presented in the chapter. The prediction methods to determine ultimate bearing capacity of the footings resting on both the natural ground and improved are based on passive pressure condition, cavity expansion theory, pile formula and general shear failure, as applicable.

6.2 Prediction of the Load Carrying Capacity

Load carrying capacity of natural and Rammed Aggregate Pier treated ground is measured here by some available methods, in which the required prediction parameters depend on soil profile, Rammed Aggregate Pier configuration, Rammed Aggregate Pier arrangement and properties of granular materials of Rammed Aggregate Pier and finally the stiffness of Rammed Aggregate Pier. The methods are Passive Pressure condition, Cavity Expansion theory, Pile formula, General Shear Failure and Ultimate Capacity of composite soil. Predicted value of bearing capacity of natural and treated ground is measured with the consideration of these above condition but it is difficult to determine most accurate value because accurate determination of shearing resistance parameters in term of angle of internal friction of the Rammed Aggregate Pier material is not possible. In this case most probable range of angle of internal friction of the RAP materials is considered with respect to physical properties of materials from the desired prediction with the consideration of soil parameters at site where unit weight, $\gamma = 16.92$ kN/m^3 and undrained shear strength, $s_u = 20$ kPa which are presented in Table 3.1.

6.2.1 Natural ground

Load carrying capacity of a square footing resting on natural ground is measured by using the famous and mostly used Terzaghi's Bearing Capacity Equation. In this equation Terzaghi used shape factor and the value of undrained shear strength. In the investigated site, for the considered isolated square footing of 1.68mx1.68m, resting at a depth of 0.75m, the predicted value of the bearing capacity of natural ground is 192 kPa by using the Eq.6.1, which is described in detail at Chapter 2. Physical stratification of natural ground and the schematic diagram of the square of footing are placed on natural ground as shown in Fig.6.1.

$$q_{ult} = cN_c + \bar{q}N_q + \gamma BN_\gamma \qquad 6.1$$

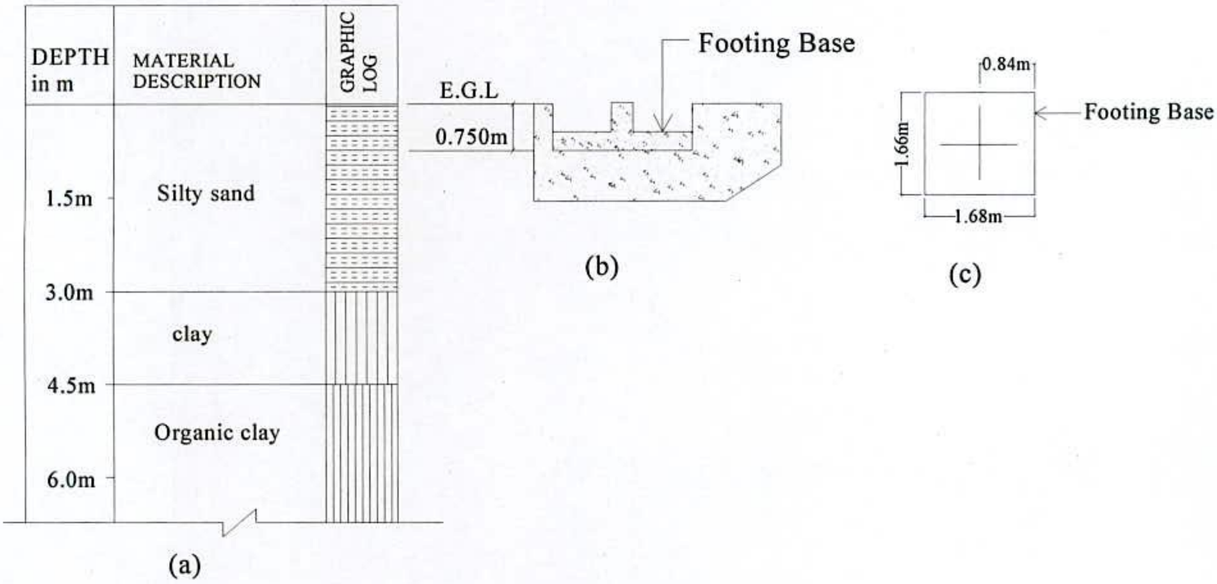

Figure 6.1 Footing placed on natural ground: (a) Sub-soil profile, (b) Section of footing and (c) Plan of footing

6.2.2 Single Rammed Aggregate Pier treated ground

The predicted value of load carrying capacity of single Rammed Aggregate Pier treated ground is measured by various methods. The methods are (i) Passive Pressure Condition, (ii) Cavity Expansion theory, (iii) Based on Pile formula, (iv) Based on General Shear Failure and (v) Ultimate Capacity as a composite soil. The schematic diagram of the completed Rammed Aggregate Pier is shown in Fig.6.2.

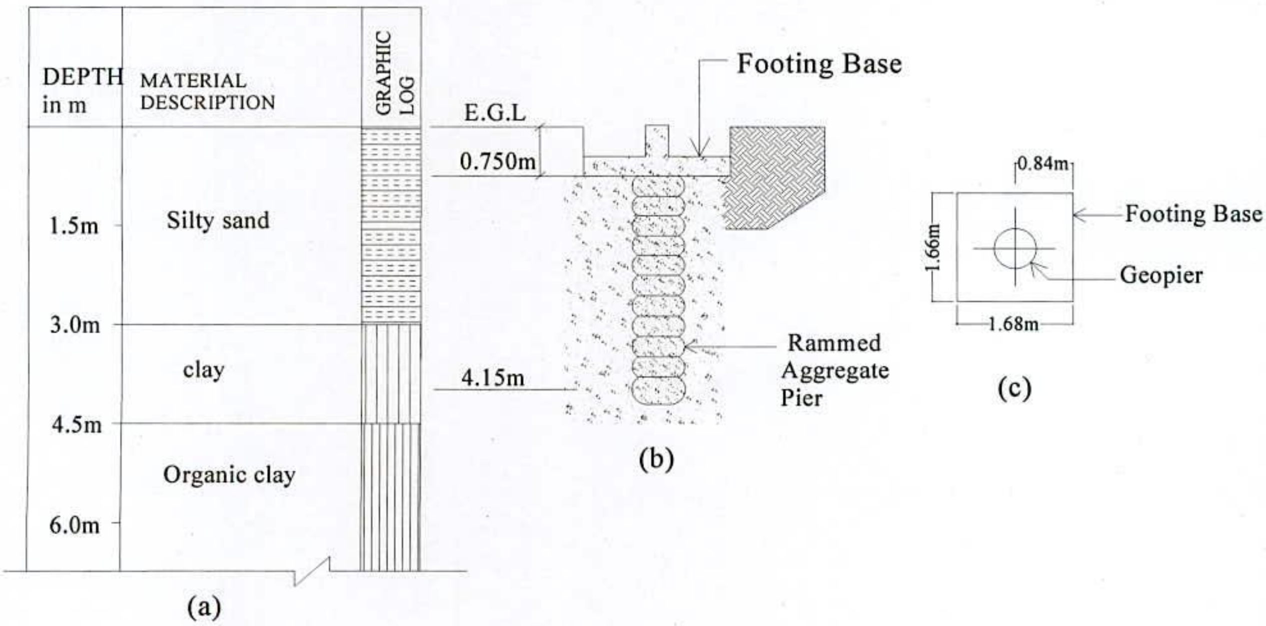

Figure 6.2 Footing placed on RAP treated ground: (a) Sub-soil profile, (b) Section of installed RAP under footing and (c) Plan of single RAP

Here, ϕ can be estimated at a reasonable range for the experimental angle of shearing resistances value from 25° to 50°. This figure shows that the ultimate bearing capacity is increased due to the increase of angle of shearing resistance. Similarly, Cavity Expansion theory, Based on General Shear Failure and Ultimate Capacity as a composite soil is used to determined ultimate bearing capacity of single RAP treated ground as shown in

Fig.6.3. By using these methods, the ultimate bearing capacity are also measured with respect to various angle of shearing resistance. In all above methods angle of shearing resistance is considered as a reasonable range for the experimental value as shown in Fig. 6.3 by using the Equations 6.3 and 6.4 of Passive pressure condition method.

$$q_u = \sigma_R \, Kps \tag{6.3}$$

$$\sigma_R = \gamma \, z \, Kpc \; + 2c \sqrt{Kps} \tag{6.4}$$

The ultimate bearing capacity of single RAP treated ground was determined by using method of Cavity Expansion Theory. The bearing capacity of single RAP treated ground with respect of various angle of shearing resistance is also measured by using Eq.6.5. Similarly, Eq.6.6 is used at the method of General Shear Failure and Eq.6.7 is used at the method of Ultimate Capacity as a composite soil to determine the ultimate bearing capacity of single RAP treated ground as shown in Fig.6.3. These equations are also described briefly in Chapter: 2.

$$\sigma_\theta \; / \; q_{ult} = \sigma_R \; N\phi \tag{6.5}$$

Where, $N\phi = (1+\sin\phi)/(1-\sin\phi)$; $\sigma_R = Fc' \; c_u + Fq' \, q$

$$q_u = cNc^* + d_f \, Nq^* + 0.5 \, BN_\gamma^* \tag{6.6}$$

$$q_{ult} = \sigma_3 \tan^2\beta + 2 \, c_{avg} \, \tan\beta \tag{6.7}$$

$$\text{Where, } \sigma_3 = \frac{\gamma_c \; B\tan\beta}{2} + 2c$$

This figure shows that the ultimate bearing capacity in all methods is increased with respect to the increment of ϕ value.

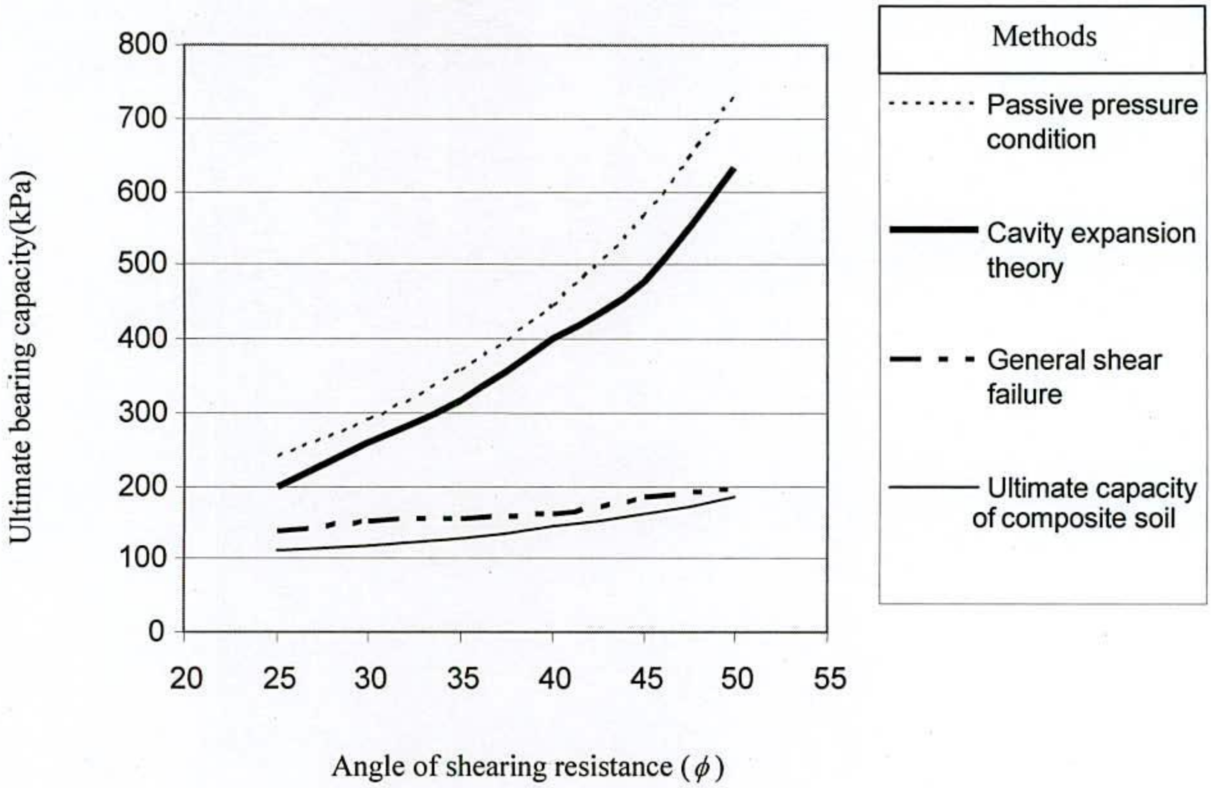

Figure 6.3 Predicted ultimate bearing capacity versus angle of shearing resistance curve of single RAP treated ground in different methods

6.2.3 Double Rammed Aggregate Pier treated ground

The load carrying capacity of double Rammed Aggregate Pier treated ground is predicted considering the treated ground as a composite soil. From this method, the ultimate bearing capacity was calculated with a wide arrange of angle of shearing resistance as shown in Fig. 6.5. Here, ϕ can be estimated also a reasonable range for the experimental value. This figure shows that the ultimate bearing capacity is increased due to increase of angle of shearing resistance, which is obvious. This figure shows that the minimum value of ultimate bearing capacity is 118.5 kPa, in which ϕ value of RAP material is represented as 25° and the maximum value of ultimate bearing capacity is predicted as 259 kPa, in which ϕ value of RAP material is represented as 50°. The schematic diagram of the soil profile and completed RAPs are shown in Fig.6.4.

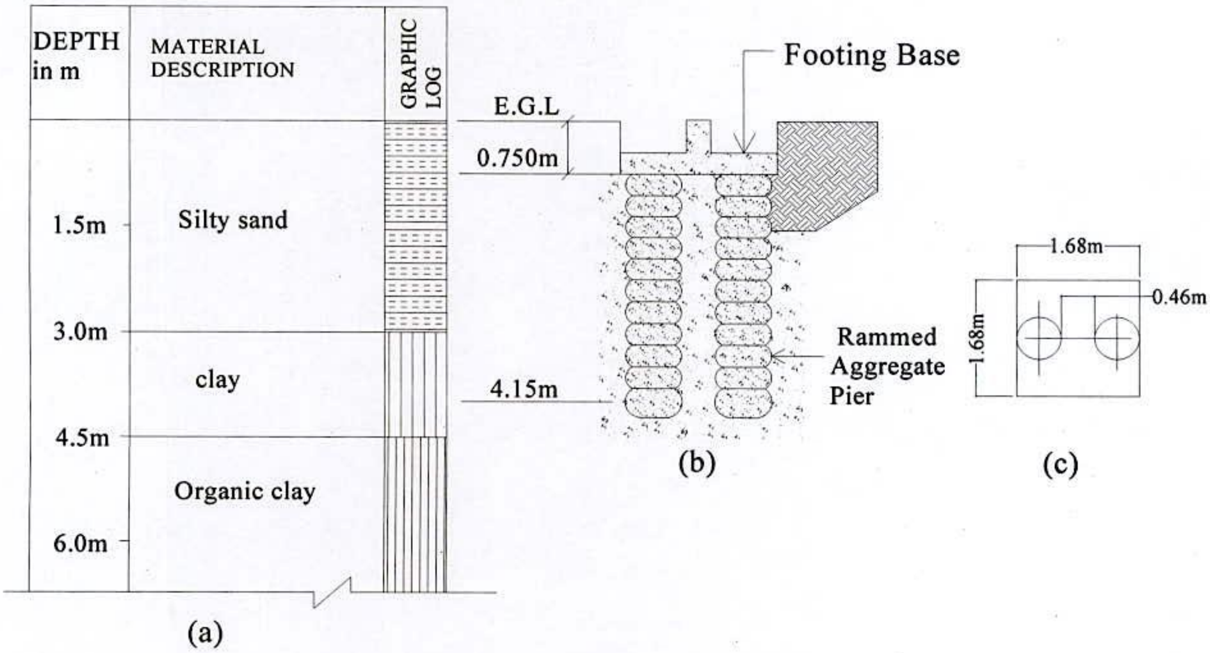

Figure 6.4 Footing placed on double RAP treated ground: (a) Sub-soil profile, (b) Section of installed double RAP under footing and (c) Plan of double RAP.

Determined ultimate bearing capacity of double RAP treated ground by using method of Ultimate Capacity on RAP ground as a composite soil. To use Eq.6.7 are also measured bearing capacity of double RAP treated ground with respect of various angle of shearing resistance.

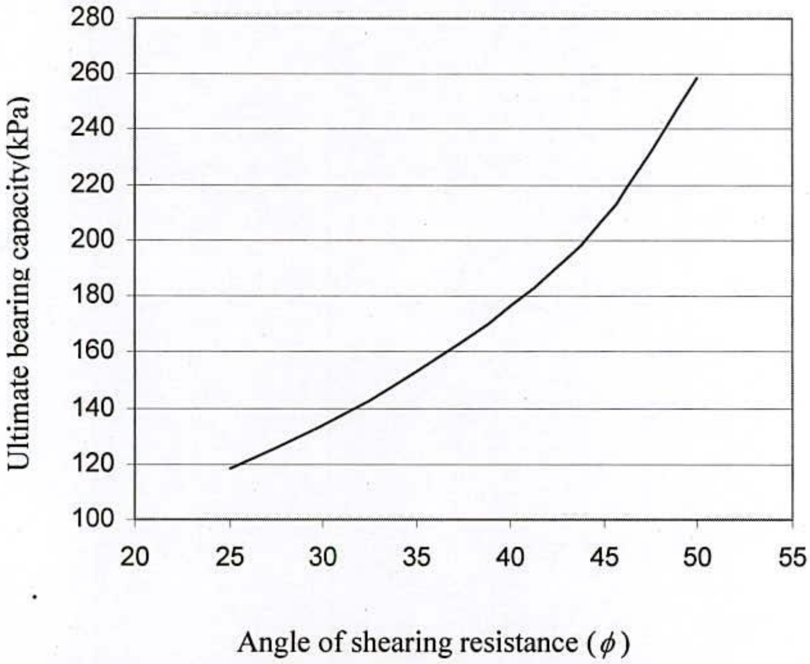

Figure 6.5 Predicted ultimate bearing capacity of double RAP treated ground for the possible range of angle of shearing resistance.

6.2.4 Group Rammed Aggregate Pier treated ground

Load carrying capacity of group Rammed Aggregate Pier treated ground is predicted considering the treated ground as a composite soil mass. In this concept, the ultimate bearing capacity was calculated by using Eq.6.7 with the wide range of angle of shearing resistance as shown in Fig. 6.7. Here, for the prediction, the ϕ value is estimated from 25° to 50°. This figure shows that the ultimate bearing capacity is increased due to increase of angle of shearing resistance. This figure shows that the minimum value of ultimate bearing capacity of the footing resting on group RAP treated ground is 123 kPa, for the value of angle of shearing resistance (ϕ) of RAP material as 25° and the maximum value of ultimate bearing capacity is predicted as 302 kPa, for the angle of shearing resistance value of RAP material as 50°. The predicted value of ultimate bearing capacity of single, double & group RAP treated ground with respect of the value of angle of shearing resistance (ϕ) are shown in Fig.6.8.

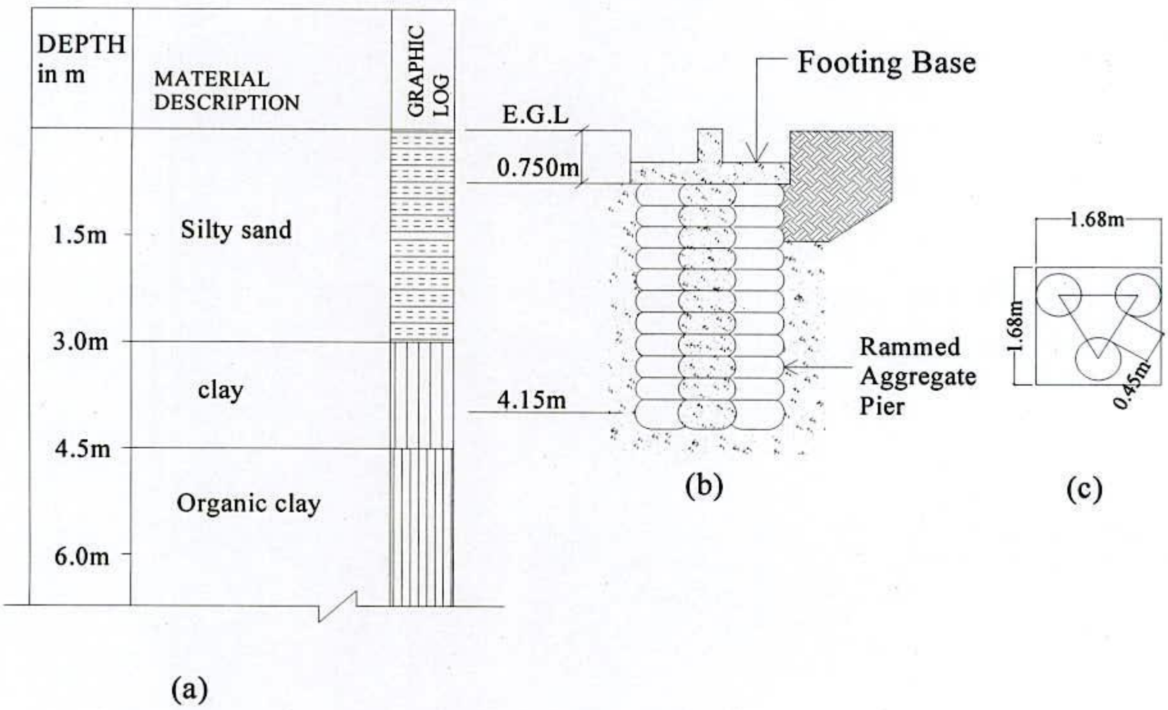

Figure 6.6 Footing placed on group RAP treated ground: (a) Sub-soil profile, (b) Section of installed group RAP under footing and (c) Plan of group RAP

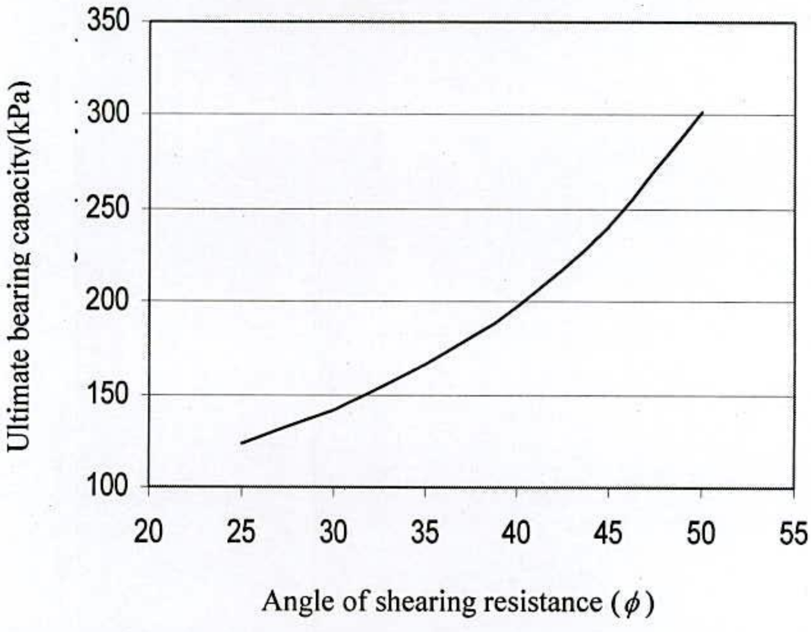

Figure 6.7 Predicted ultimate bearing capacity of group RAP treated ground for a range of angle of shearing resistance.

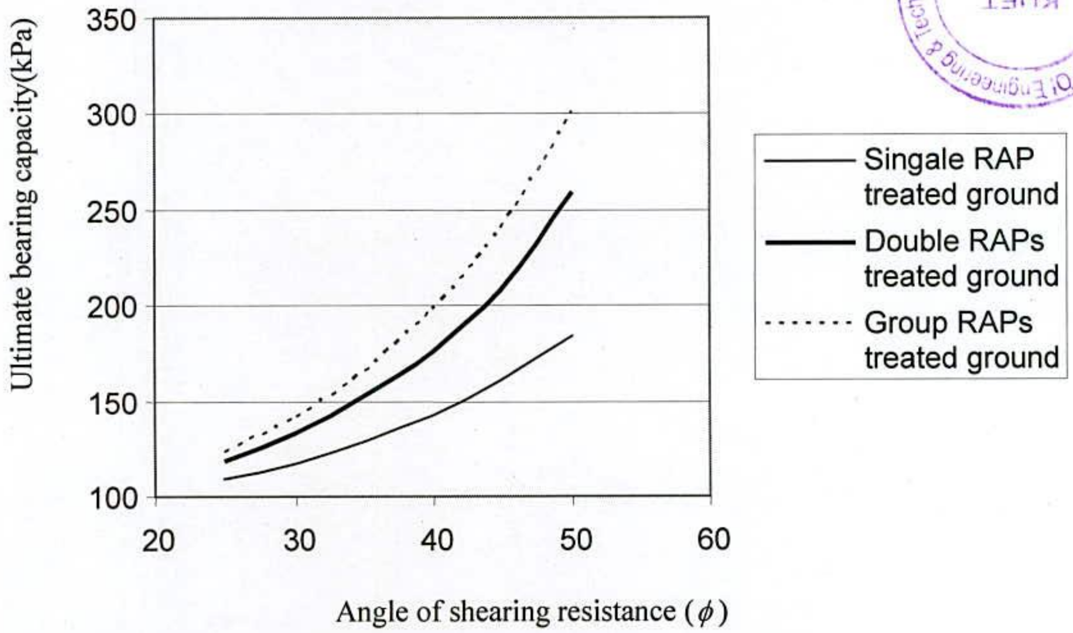

Figure 6.8 Predicted ultimate bearing capacity for single, double & group RAP treated ground consider the RAP installed ground as the composite ground for a range of angle shearing resistance of the granular materials.

6.3 Comparison between Predicted and Measured Values

Field investigation results show that the field measured value of ultimate bearing capacity of the improved ground is comparatively less than that of predicted value. At field investigation, the value of bearing capacity is measured through field load test. Four load tests had been conducted successfully for four types of ground by using full-size footing on natural ground and three types of Rammed Aggregate Pier treated ground. The bearing capacity of predicted value of Rammed Aggregate Pier treated ground were measured using various methods which result depends on the types of Rammed Aggregate Pier treated ground.

6.3.1 Natural ground

Ultimate bearing capacity of untreated ground is predicted using Terzaghi's Bearing Capacity equation. From field investigation, the bearing capacity of natural ground was measured as 141 kPa. The ultimate bearing capacity was predicted as 192 kPa. The results show that the predicted value of bearing capacity is grater than that of the measured value.

6.3.2 Single Rammed Aggregate Pier treated ground

Predicted value of bearing capacity of single Rammed Aggregate Pier treated ground is measured in various methods using the various properties of Rammed Aggregate Pier as its stiffness which depends on the granular materials types and compactness. The methods of Passive pressure condition, Cavity Expansion Theory, General Shear failure, Ultimate capacity of footing on Rammed Aggregate Pier treated ground are used to predict the value of the bearing capacity of single RAP treated ground which is shown in Fig. 6.9. Single RAP treated ground at same condition yielded as field value is 177kPa obtained from full-scale field load test. From the comparison between predicted and measured value, it is depicted that the predicted value for all methods is higher than that of the field measurement.

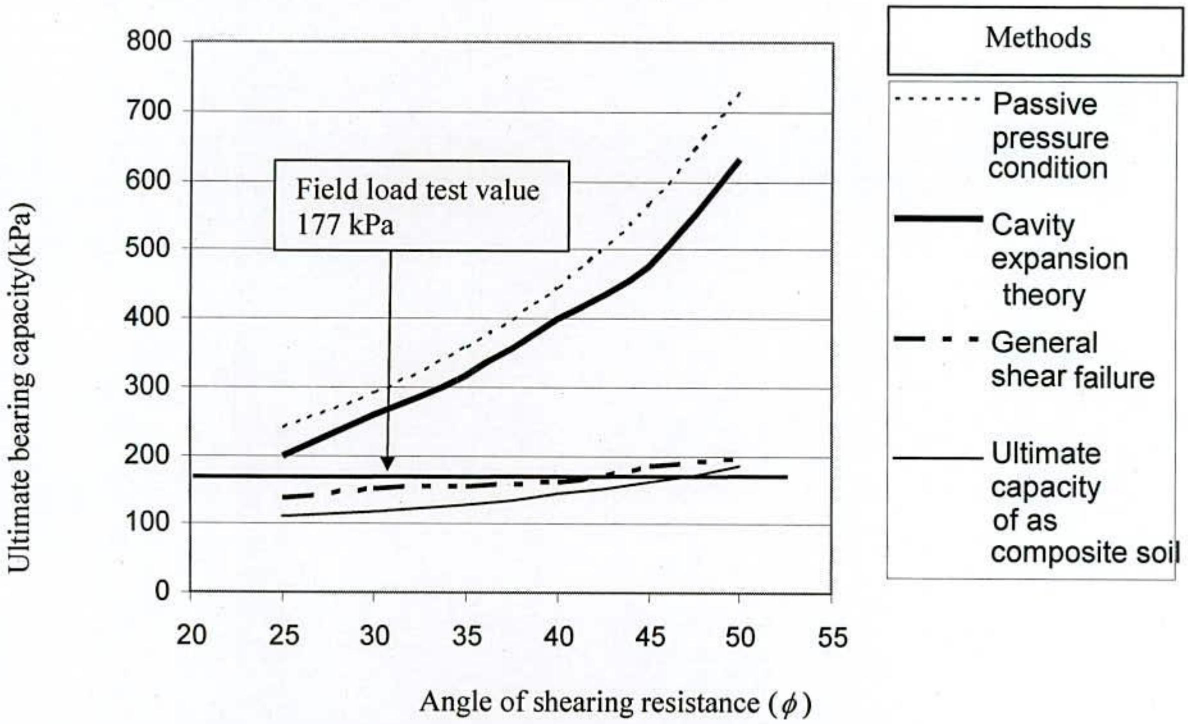

Figure 6.9 Comparison between predicted and measured ultimate bearing capacity of single Rammed Aggregate Pier treated ground.

The prediction method in which the ground is considered as composite soil, provides very close value of ultimate bearing capacity of the footing than that of the field measurement.

However, all other three methods give higher value of ultimate bearing capacity than the measured value. Based on the pile formula, which is not shown in the figure, the predicted value of bearing capacity was obtained as 539kPa, which is also much higher than the field value.

6.3.3 Double Rammed Aggregate Pier treated ground

The method named as "Ultimate Bearing Capacity of footing on composite soil" is used to predict the ultimate bearing capacity of double Rammed Aggregate Pier treated ground for a range of the stiffness of the RAP. Other methods such as Passive Pressure Condition, Cavity Expansion Theory, Pile Formula, and General Shear Failure are not used to predict value of ultimate bearing capacity as they can not be used for proper comparison in this case. From field investigation, the measured value of ultimate bearing capacity of double Rammed Aggregate Pier treated ground is obtained as 254 kPa as shown in Fig.6.10.

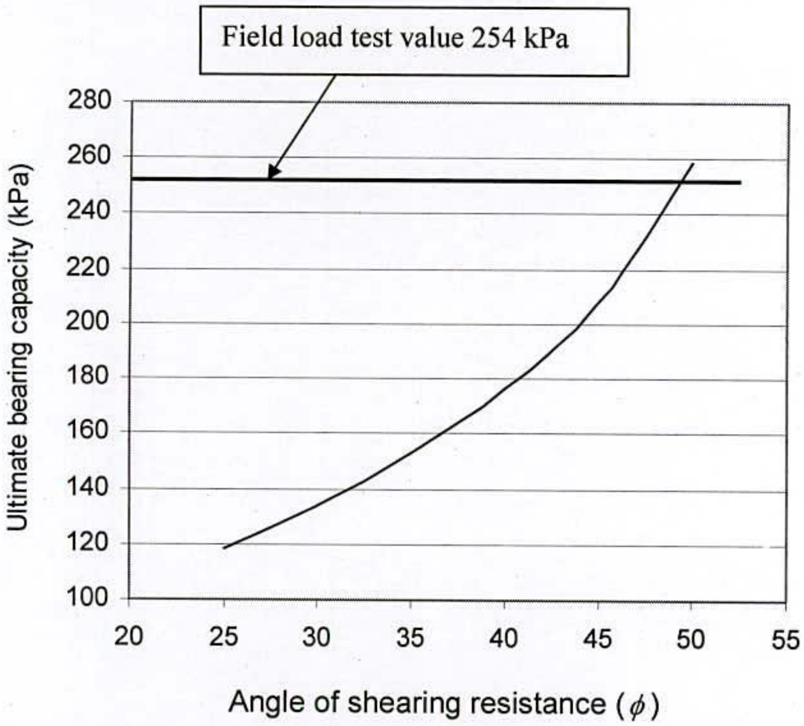

Figure 6.10 Comparison between predicted and measured ultimate bearing capacity of double Rammed Aggregate Pier treated ground.

This figure shows that the predicted value of ultimate bearing capacity is gradually increased with the gradual increase of angle of sharing resistance. From field measurement, it was found that the ultimate bearing capacity is 254 kPa, which is same as the predicted value when the angle of shearing resistances was considered as 48°.

6.3.4 Group Rammed Aggregate Pier treated ground

The ultimate bearing capacity of group Rammed Aggregate Pier treated ground presented here in Fig.6.11 is predicted using of method of "Ultimate Bearing Capacity of footing on Rammed Aggregate Pier treated ground", in which the predicted value also depends on the stiffness of RAP. Other methods such as Passive Pressure Condition, Cavity Expansion Theory, Pile Formula, General Shear Failure are not used to predict value of bearing capacity. The predicted value of ultimate bearing capacity is increased due to increase of angle of sharing resistance. From field investigation the bearing capacity of group Rammed Aggregate Pier treated ground is measured 276.5 kPa. as shown in Fig.6.11.

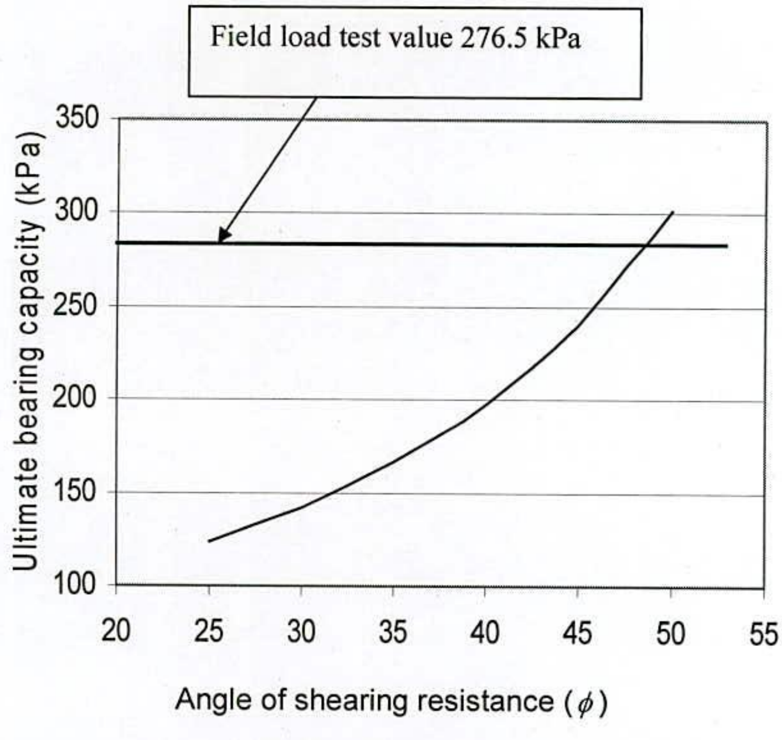

Figure 6.11 Comparison between predicted and measured ultimate bearing capacity of group Rammed Aggregate Pier treated ground

This figures show the predicted value of bearing capacity is gradually increased with the gradual increase of angle of sharing resistance. From the field measurement it was found that the ultimate bearing capacity was 254 kPa with same predicted value for the angle of shearing resistance of 47°.

6.4 Discussion on the Comparison

The predicted and measured values of ultimate bearing capacity of RAP treated ground for different configuration are described here. Prediction of accurate angle of shearing resistances for the granular materials of Rammed Aggregate Pier are not possible, for this reason it is very difficult to predict the actual ultimate bearing capacity of Rammed Aggregate Pier treated ground. Field measured and predicted measured value of ultimate bearing capacity for the angle of shearing resistance (ϕ) as 25°, 35° and 50° and their comparison is shown in Table 6.1. In this respect the ultimate bearing capacity of RAP treated ground is predicted with respect of various angle of shearing resistance (ϕ) and specific used the method of Ultimate capacity of composite soil, but at field investigation it is shown that the measured value of bearing capacity is more accurate than predicted value and it is occurred due to respect of soil profile, and physical properties of Rammed Aggregate Pier granular materials such as RAP dimension, RAP arrangement, and stiffness of Rammed Aggregate Pier materials (ϕ) which depends on materials properties, overburden pressure, compaction, lateral pressure etc. From this study it is revealed that the field measured value of ultimate bearing capacity of untreated and treated ground such as single, double and group Rammed Aggregate Pier whose measured value is gradually increased from value of untreated soil. Similarly, the predicted value of ultimate bearing capacity of untreated and treated ground such as single, double and group Rammed Aggregate Pier was gradually increased from value of untreated soil.

Table 6.1 Comparison of prediction and measured values

		Ultimate bearing capacity (kPa) corresponding to 25mm settlement		
Angle of shearing resistance (ϕ)		ϕ=25°	ϕ=35°	ϕ=50°
Untreated ground	Predicted value (q_u)	192		
	Measured value (q_u)	141		
Single Rammed Aggregate Pier treated ground	Predicted value (q_u)	109	129	184
	Measured value (q_u)	177		
Double Rammed Aggregate Pier treated ground	Predicted value (q_u)	118	153	259
	Measured value (q_u)	254		
Group Rammed Aggregate Pier treated ground	Predicted value (q_u)	123	166	240
	Measured value (q_u)	276.5		

CHAPTER SEVEN

CONCLUSION AND RECOMMENDATIONS

7.1 Conclusion

Based on this study the following conclusions can be made:

(i) Rammed Aggregate Piers have been practiced successfully in several important projects throughout the world to improve the bearing capacity of ground for constructing the infrastructures.

(ii) Modern installation equipments are available for the successful installation of Rammed Aggregate Pier.

(iii) The installation of Rammed Aggregate Pier in field level reveals that the locally fabricated equipment can be used successfully for the installation of Rammed Aggregate pier in soft ground condition by replacement method. However, the constraint about the dimension is evident and the placement of casing is required.

(iv) The field experience during the installation of Rammed Aggregate Pier depicts that such installation technique required very close monitoring and precaution in case of double and group Rammed Aggregate Pier arrangements.

(v) Proper mixing, moisture content and pouring of granular materials are required to ensure the stiffness of Rammed Aggregate Pier. The layer thickness of granular material, dropping height, number and placement of hammer are to be maintained properly through close field monitoring to achieve the designated stiffness of the Rammed Aggregate Pier.

(vi) The results reveals that the bearing capacity of footing resting on Rammed Aggregate Pier improved ground has increased significantly than that of resting on the natural ground.

(vii) The degree of improvement has increased with the number of Rammed Aggregate Pier under the footing. However, it is observed the difference of improvement between double and group (three) Rammed Aggregate Pier is insignificant.

(viii) The comparison shows that the existing theoretical approaches can be used successfully for the prediction of ultimate bearing capacity of RAP treated ground if the properties of the RAP and the surrounding soils can be estimated accurately.

(ix) The field experiences during Rammed Aggregate Pier installation and the degree of improvements as measured depict that the standard methods supported by appropriate equipments are needed for the installation of Rammed Aggregate Pier to get better results.

7.2 Recommendation for Future Study

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

i. Full length casing pipe is to be used during the creation of the hole by wash boring method, while the casing will be withdrawn during the pouring and densification of granular materials are done in layer.

ii. To establish the effectiveness of the adopted installation technique, various types of installation technique can be used for the construction of Rammed Aggregate Piers in single and group.

iii. Future research work can be conducted to establish the relation between the number, dimension and spacing of Rammed Aggregate Pier on the bearing capacity of the improved ground. However such study can be conducted in the laboratory due to the high cost involvement in field test.

iv. A field study on load carrying capacity of Rammed Aggregate Piers which is made of different types of granular materials can be a useful research works in future.

v. A field study on load carrying capacity of Rammed Aggregate Piers having lateral reinforcement with the use of geotextiles can be an interesting future research works.

vi. Long term settlement behavior of the improved ground can be investigated in future.

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APPENDIX

Appendix - A

Evaluation of vertical loads of Rammed Aggregate Pier by the existing methods:

1. RAPs dimensions:

   Length of RAP, L = 4.2 m

   Diameter of RAP d = 750 mm

   Depth of top of RAP from ground surface, $Df = 0.75$ m

   RAP depth at middle from ground surface, Z = 2.1 m

2. Aggregates properties:

   Aggregates type: Brick aggregates

   Aggregates size: 12 mm to 35 mm

   Angle of internal friction, $\phi = 40^0$

3. Clay properties:

   Type: Soft clay

   Location: South-east region of Bangladesh

   Unit weight, $\gamma = 16.92 kN/m^3$

   Possion's ratio, $\nu = 0.40$

   Undrained shear strength, Su = 20 kPa

   Modulus of elasticity, Es = 300 kPa

A. Existing method based on cavity expansion theory:

$q = \gamma Z = 16.92 \times 2.1$

$=$ 35.53 kPa

$N_\phi = \tan^2(45^0 + \phi'/2)$

$= 6.49$

$$Ir = E/(2(1+\nu)(cu + q\tan\phi)$$

$$= 11.04$$

Therefore, $Fc' = 6.0$

$Fq' = 1.0$

$= Fc'\ Cu + Fq'\ q$

$= 141.75$ kpa

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

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

$= 400$ kpa

B. Existing method based on passive pressure theory:

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

$$q_u = \sigma_R Kps$$

$$= (Kpc = 1.0)$$

$$= 444.6 \text{ kPa}$$

C. Existing method based on pile formula

$$q_u = c_u\,(4\,(1/d) + 9)$$

$$= 539 \text{ kPa}$$

Therefore, vertical load carrying capacity of full-size footing on single RAP treated ground

$$P_v = 153 \text{ ton}$$

D. Based on general shear failure

Mitchell and katti (1981) suggested from their experience that the load carrying capacity of RAP is 25 Cu.

Therefore, $q_u = 25\ Cu$

$$q_u = cNc^* + d_f\ Nq^* + 0.5\ BN_\gamma^*$$

Where, d_f = 1.68m (size of the footing) ; dp = .76m (size of the stone column or RAP) as shown in the Fig.2.10.

$$q_u = 154.81\ \text{kPa}$$

Therefore, vertical load carrying capacity of full-size footing on single Geopier treated ground.

$$P_v = 44\text{ton}$$

E. Ultimate Capacity of Stone Column and RAPs Groups as Composite soil

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

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

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

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

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

$$a_c = A_c / A$$

$$= 1 - a_s$$

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

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

Where, D = diameter of the compacted stone column

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

$a_s = A_s/A \qquad a_s = 0.46/2.8=0.164$

$n = \sigma_s / \sigma_c \qquad n = 4$

σ_s = stress in the stone column

σ_c = stress in the surrounding cohesive soil

$c_{avg} = 16.72$

$\phi_{avg} = 14.23$

$\beta = 52.12$

$\sigma_3 = 45.24$

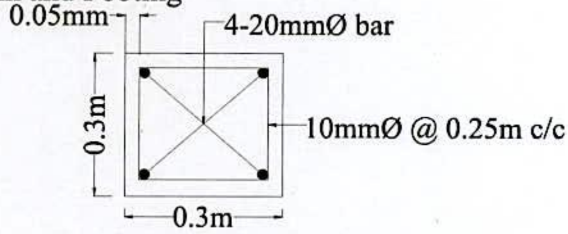
Assuming the ultimate vertical stress q_{ult} (which is also assumed to be σ_1) and ultimate lateral stress σ_3 to be principal stresses, equilibrium of the wedge requires

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

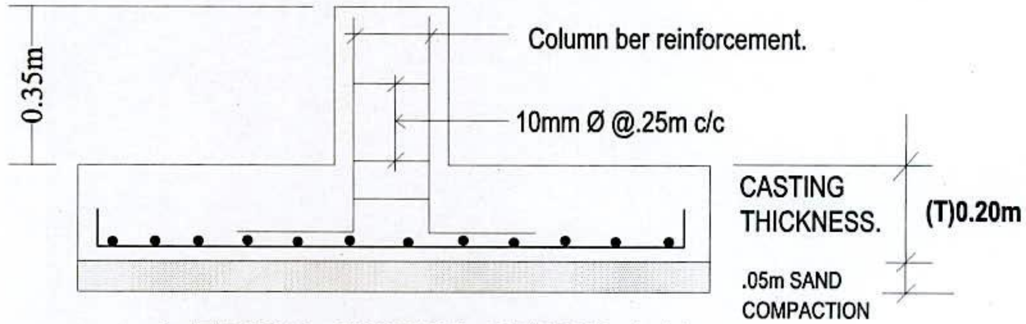
$$q_{ult} = 117.74 \text{ kPa}$$

Appendix - B

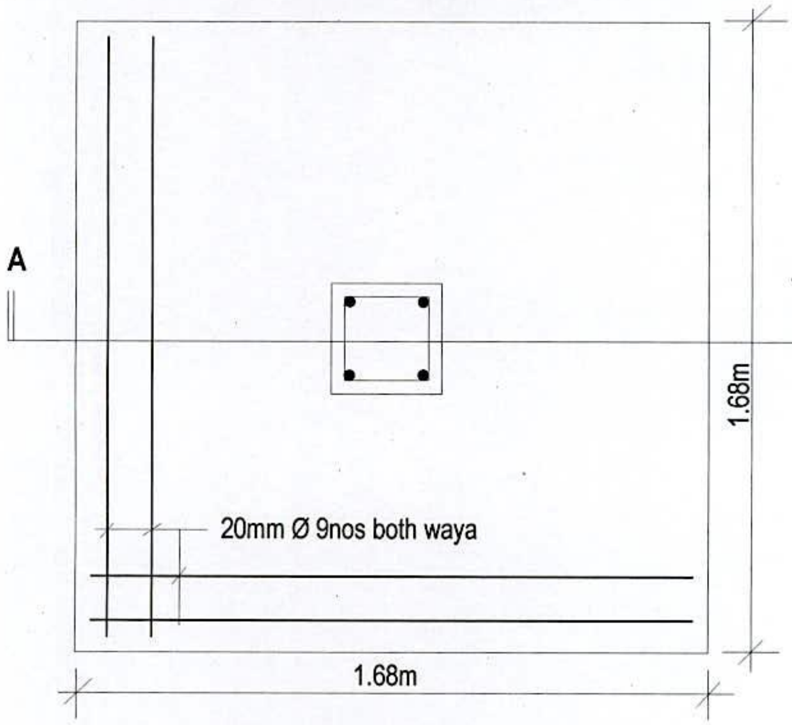
B1.Construction of Column and Footing



c. TYPICAL COLUMN DESIGN.



b. TYPICAL FOOTING SECTION- A-A'.



a. TYPICAL FOOTING PLAN.

Figure B.1 Reinforcement detailing for construction of footing and column

Table B.1 Schedule of Reinforcement for Footing Construction

Footing placed on	Footing size	Casting thickness (T)	Footing Reinforcement
Untreated ground	1.68mX1.68m	0.20m	20mm dia bar, 9nos place in both way.
Single RAP treated ground	1.68mX1.68m	0.225m	20mm dia bar, 10nos place in both way
Double RAP treated ground	1.68mX1.68m	0.250m	20mm dia bar, 10nos place in both way
Group RAP treated ground	1.68mX1.68m	0.275m	20mm dia bar, 11nos place in both way

Appendix - C

Cost Estimate for Single Rammed Aggregate Pier:

Table C. 1. Labour cost for Installation:

Si. No.	Description of item	Quantity	Unit	Rate	Taka
01	Earth work in excavation of borehole	$\{(\pi \times 2.5'^2)/4\} \times 13'=$ 64cft	cft	5.00	320.00
02	Installed of clay burn ring in borehole	16	nos	10.00	160.00
03	Mixing, Pouring and compaction of granular materials with including necessary works	60cft	cft	30.00	1800.00
Total taka					2280.00

Table C. 2. Material cost:

Si. No.	Description of item	Quantity	Unit	Rate	Taka
01	Clay burn ring Size: 650mm dia.X150mm height X10mm thickness.	16	nos	21.50	344.00
02	Brick Aggregate 38mm down size.	60	cft	35.00	2100.00
03	Local Sand 1.2F.M	30	cft	5.00	150.00
Total taka					2594.00

Total taka (a+b) 4874.00

Total installation cost for construction of six no Rammed Aggregate Pier in this study

= 6X 4874.00=29244.00

Appendix - D

Cost Estimate for Construction of Footing, and Load Test of Full-size Footing Resting on Natural & RAP Treated Ground.

1. Load Test of Full-size Footing on Natural Ground.

Table D. 1.a Construction cost of full-size footing on natural ground.

SI. NO.	Description of item	Quantity	Unit	Rate	Taka
01	Earth work in excavation for footing	75.5	cft	2.00	151.00
02	Reinforced cement concrete works (1:1.5:3) of specific compressive strength -25 Mpa, use stone braking chips with including all necessary works.	0.61	cum	5860.00	3574.60
03	Supplying, fabrication and fixing of Reinforcement in concrete fy=276Mpa	0.73cft	quintal	4800.00	3504.00
Total taka					7229.6

Table D. 1.b. Cost for load test of full-size footing on natural ground.

SI. No.	Description of item	Quantity	Unit	Rate	Taka
01	Higher cost for equipment (guarder, main guarder Rocker beam, loading jack and other accessories.	L/S		Per test	5000.00
02	Carrying charge for loading sand with including sand bag rent, and including its loading and binding cost.	1125	Per bags	2.00	2250.00
03	Labour cost for loading and unloading sand bag	1125	Per bags	3.00	3375.00
Total taka					10625.00
Total taka (a+b)					17854.6

2. Load Test of Full-size Footing on Single RAP Treated Ground.

Table D. 2.a. Construction cost of full-size footing on single RAP treated ground.

SI. NO.	Description of item	Quantity	Unit	Rate	Taka
01	Earth work in excavation for footing	75.5	cft	2.00	151.00
02	Reinforced cement concrete works (1:1.5:3) of specific compressive strength -25 Mpa, use stone braking chips with including all necessary works.	0.68	cum	5860.00	3984.80
03	Supplying, fabrication and fixing of Reinforcement in concrete fy=276Mpa	0.81	quintal	4800.00	3888.00
Total taka					8023.80

Table D. 2.b. Cost for load test of full-size footing on single RAP treated ground

Si. No.	Description of item	Quantity	Unit	Rate	Taka
01	Higher cost for equipment (guarder, main guarder Rocker beam, loading jack and other accessories.	L/S		Per test	5000.00
02	Carrying charge for loading sand with including sand bag rent, and including its loading and binding cost.	1222	Per bags	2.00	2444.00
03	Labour cost for loading and unloading sand bag	1222	Per bags	3.00	3666.00
Total taka					11110.00
Total taka (a+b)					19133.8

3. Load Test of Full-size Footing on Double RAP Treated Ground.

Table D.3.a Construction cost of full-size footing on double RAP treated ground.

SI. NO.	Description of item	Quantity	Unit	Rate	Taka
01	Earth work in excavation for footing	75.5	cft	2.00	151.00
02	Reinforced cement concrete works (1:1.5:3) of specific compressive strength -25 Mpa, use stone braking chips with including all necessary works.	0.75	cum	5860.00	4395.00
03	Supplying, fabrication and fixing of Reinforcement in concrete fy=276Mpa	0.81	quintal	4800.00	3888.00
Total taka					8434.00

Table D.3. b. Cost for load test of full-size footing on double RAP treated ground

Si. No.	Description of item	Quantity	Unit	Rate	Taka
01	Higher cost for equipment (guarder, main guarder Rocker beam, loading jack and other accessories.	L/S		Per test	5000.00
02	Carrying charge for loading sand with including sand bag rent, and including its loading and binding cost.	1666	Per bags	2.00	3332.00
03	Labour cost for loading and unloading sand bag	1666	Per bags	3.00	4998.00
Total taka					13330
Total taka (a+b)					21764.00

4. Load Test of Full-size Footing on Group RAP(3) Treated Ground.

Table D.4.a Construction cost of full-size footing on group RAP treated ground.

SI. NO.	Description of item	Quantity	Unit	Rate	Taka
01	Earth work in excavation for footing	75.5	cft	2.00	151.00
02	Reinforced cement concrete works (1:1.5:3) of specific compressive strength -25 Mpa, use stone braking chips with including all necessary works.	0.82	cum	5860.00	4805.20
03	Supplying, fabrication and fixing of Reinforcement in concrete fy=276Mpa	0.89	quintal	4800.00	4272.00
Total taka					9228.20

Table D.4. b Cost for load test of full-size footing on group RAP treated ground.

Si. No.	Description of item	Quantity	Unit	Rate	Taka
01	Higher cost for equipment (guarder, main guarder Rocker beam, loading jack and other accessories.	L/S		Per test	5000.00
02	Carrying charge for loading sand with including sand bag rent, and including its loading and binding cost.	1880	Per bags	2.00	3760
03	Labour cost for loading and unloading sand bag	1880	Per bags	3.00	5640
Total taka					14400.00
Total taka (a+b)					23628.20