# EFFECTIVENESS OF SAND COMPACTION PILE AND PREFABRICATED VERTICAL DRAINS IN IMPROVING SOFT SOIL FOR PAVEMENT SUBGRADE

TANIM SHAHRIAR (SN. 1016110011)

A Thesis Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

DEPARTMENT OF CIVIL ENGINEERING
MILITARY INSTITUTE OF SCIENCE AND TECHNOLOGY
DHAKA, BANGLADESH

JULY 2022

# DECLARATION

I hereby declare that this thesis is my original work and it has been written by me entirely. I have duly acknowledged all the sources of information which have been used in the thesis. The thesis (fully or partially) has not been submitted for any degree or diploma in any university or institute previously.

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# ACKNOWLEDGEMENTS

The author would like to express his sincere thanks to his supervisor, Dr. Shah Md. Muniruzzaman, whose encouragement and engagement in this study was invaluable. He provided the author keen guidance, thoughtful direction and support throughout the project. The author is also grateful to him for reviewing the manuscript time and again and providing valuable comments.

The author would also like to show his deep appreciation to Professor Dr. Md. Zoynul Abedin, who guided the author at the later part of this study especially with thesis writing and data analysis. The author is highly privileged for the meaningful discussions with him concerning the mechanical behavior of soil and ground improvement technics.

Special regards are due to the officials of 24 Engineer Construction Brigade, Bangladesh Army for the great collaboration during the study, especially to Resident Engineer Abu Bakar Siddique for assisting me in data collection.

The author is also indebted to the officials of Purbachol to Madani Avenue Link Road Project, including the team leader Sheikh Md Moktar Ali for their support. The efforts of resident engineer Md Abir is also gratefully acknowledged for his assistance.

Thanks are also due for Lt Col Asheque, Project Director, Madani Avenue to Balu River Road Construction Project and Major Shams, Project Officer Purbachol to Madani Avenue Link Road Project for their wholehearted support.

The author would like to express his sincere thanks to his beloved and admired wife, Nishat-E-Moyen (Jerry), for inspiring him throughout this research work.

The funding for my research was provided by 24 Engineer Construction Brigade of Bangladesh Army and MIST. Sincere thanks are also directed to them for providing the opportunity to conduct higher studies in the field of transportation and geotechnical engineering.

# ABSTRACT

## Effectiveness of Sand Compaction Pile and Prefabricated Vertical Drains in Improving Soft Soil for Pavement Subgrade

The present study examines the effectiveness of Sand Compaction Pile (SCP) and Prefabricated Vertical Drains (PVD) in improving properties of soft soil underlying road pavement. Subgrade improvement techniques are often used to reinforce the subsoil properties with regards to its bearing capacity, shear strength, settlement etc. At present various types of ground improvement technique aims to increase the bearing capacity of soil and reduce settlement. This research focuses on such two well-practiced subgrade improvement techniques for pavement.

The study area is located at reclaimed land of Purbachol known as Jolshiri Abason, in close vicinity of Dhaka city. For the purpose of investigating the effectiveness of SCP and PVD, two interconnecting roads Purbachal 300 feet Expressway to Madani Avenue (Road-1) and Madani Avenue extension up to Shittalakha River (Road-2) were selected as a representative of reclaimed land. Physical soil properties (soil profile, specific gravity, grain size analysis) were measured at four different locations, two locations each from both the roads. Subsoil condition was almost similar before installation of ground improvement in this area. Average SPT-N values were found around 3 to 4 before subgrade improvement in all four locations up to a significant depth. Basing on soil properties and SPT-N values, suitable dimensions and parameters of SCP and PVD were designed. Physical soil properties and ground water condition directed choice of method of subgrade improvement at study area. Accordingly, SCP was installed at Road-1 due to presence of more clayey soil and PVD was conducted at Road-2 due to higher ground water table as per design. SPT test was executed again and average SPT-N values were found around 14 to 18 in all four locations. Average SPT-N value was 18 after installation of SCP at Road-1 whereas average SPT-N value was identified around 14 after installation PVD at Road-2. Subsoil improvement through SCP took less time than PVD due to soil settlement time required for the later. After installation of SCP, it took around one month to improve subsoil condition whereas, PVD took seven months for soil settlement with necessary improvement. Dynamic Cone Penetration (DCP) test was performed to check the suitability between SCP and PVD. DCP index value and average CBR (%) was slightly higher where ground

improvement was conducted through PVD than that of SCP. DCP test results shows that PVD provides slightly more compacted ground surface than SCP at shallow depth. An economic analysis was also carried out by considering 100 square meter area to check the cost effectiveness between these two subgrade improvement techniques. It was found that, the advantage of SCP is due to sand used for construction is considerably cheaper when compared to PVD which needs geotextile and other imported materials. In comparison with SCP, PVD cost was found 1.37 times higher.

From the study it can be concluded that SCP and PVD are two technically viable and cost-effective solutions for soils of the study area which are weak in strength and needed treatment in order to make them suitable for construction of road pavement over them. The SCP and PVD both are suitable techniques for subsoil improvement with appropriate design. This study may guide to adopt a flexible approach for improving the poor soil conditions of any reclaimed area. However, the study was limited to two sites only and the generalization of observations needs further studies with data from various subsoil conditions.

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# CHAPTER 1
# INTRODUCTION

## 1.1 General

The performance of road pavement depends largely on its subgrade layer that provides support to a whole pavement system. As such, subgrade layer plays a key role in mitigating the detrimental effects of geotechnical, environmental and dynamic stresses generated by the traffic load, and building a stable subgrade is vital for constructing an effective and sustainable pavement system.

Bangladesh is a riverine country and it is mostly alluvial flood plains and in many occasions the depression areas of the flood plains are required to be raised with hydraulic fill of silty fine sand river bed soils with fines resulting in filled soft soil deposits. Because of scarcity of land, construction of infrastructure on these types of soft soil are very common. Most of these soft soils require improvement while constructing highways and other structures over them. The improvement techniques are mainly used to reinforce the soil properties with regards to its bearing capacity, shear strength and settlement. Many highways are being constructed on these reclaimed grounds consisting of soft alluvial soils, organic soils and so on. To solve these problems, appropriate ground modifying techniques are needed to strengthen the quality of reclaimed ground.

## 1.2 Background of the Present Research

Different types of subgrade improvement techniques have been developed: such as sand compaction pile (SCP), prefabricated vertical drain (PVD), vibratory surface and deep vibro-compaction, removal and replacement, preloading, dynamic replacement, stone columns, piled embankment and viaduct, transition structures etc. (Hossain, 2015). Out of these subgrade improvement techniques, some of them are widely practiced in our country. Among these, SCP is one of the potential methods for improving ground stability. This method was originally developed in order to increase the density of loose sandy ground and to increase the uniformity of sandy ground, to improve its stability or compressibility and to prevent liquefaction failure. It has also been applied to soft clay ground to assure stability and to reduce ground settlement.

The principal concept of the SCP method for application at sandy ground is to increase the ground density by placing a certain amount of granular material (usually sand) in the ground. Similarly, the principal concept of the SCP method for application at clayey ground is to strengthen the composite soil consisting of compacted sand piles and the surrounding clay. SCP method is different from sand drain method in which sand piles are constructed without any compaction principally for drainage function alone (Kitazume, 2005).

Excessive settlement is a common problem for road construction at compressible or saturated soils such as very loose silty sand, lean clay and fat clay etc. The subgrade improvement technique using prefabricated vertical drains (PVD) is another appropriate methods to control excessive soil settlement. Vertical drain system is used to shorten the drainage path of the pore water from a low permeable layer to free water surface or to pre-installed drainage layer, thereby accelerating the rate of primary consolidation or the process of settlement. Application of ground improvement method using PVD coupled with surcharge can significantly shorten the period of primary settlement (Hansbo, 1993).

Other than these, a large number of subsoil advancement methods that can be utilized to overcome poor soil conditions, some of which have been in practice for many decades. The recent developments in subgrade improvement methods, systems and engineering tools have been massive, resulting in a huge proficiency. The selection of the most suitable ground improvement technology is a complex undertaking that depends upon unification of available knowledge and site-specific factors. These factors are considered as the essential elements for success of a subgrade improvement research. In Bangladesh, subgrade improvement had been conducted through different methods especially SCP and PVD. However, comparative subgrade improvement research work and evaluation are limited in this regard. The outcome of this research work will assist further case studies regarding subsoil improvement including comparative analysis. Proper selection of subgrade improvement methods will also justify the requirement of originality of the research.

Pavement construction frequently experiences reclaimed land or very problematic soils like loose silty sand, lean clay and fat clay, which can constitute problems of strength, stability and liquefaction. To interpret these problems, different types of subgrade improvement techniques have been developed around the world. SCP method was enlarged for improving sandy and clay grounds in Japan. Subsequently, SCPs were taken in many countries in mostly of Asia to improve loose sandy and clayey soils (Samanta et al., 2010). Compacted sand piles were used at the site of a steam power plant in the southern part of Taiwan which had high liquefaction potential. The improvement was successful with 100% samples giving over the required 65% relative density and 92% samples with more than 75% relative density. In 2008, the effect of granular pile installation on the modifications induced in loose to medium dense granular deposits was also studied by Krishna and Madhav (Moh et al., 1981). Performance of ground was amplified by applying sand compaction piles as a ground improvement method. It was seen that installation of compaction piles densifies the ground and modifies the distortion properties of the soils. As soil is a regional material and the approach are different in various parts of the world, the advantage of SCP method will also differ in various parts of the world.

Vertical draining of fine-grained soils for ground improvement purposes was first advised and applied in the US in the late 1920's (Hansbo, 1993). In the early days of the method, there was some pioneer work done by a Swedish engineer, Walter Kjellman, who patented a type of prefabricated cardboard drain and equipment for its installation. This type of drain is the precursor of all the prefabricated band drains frequently used across the world today. Later on, methods for designing vertical drains were developed by Barron in 1948. However, the design of vertical drains today is based mainly on the subsequent and more practical, simplifications and additions to the theory (Hansbo, 1979).

In Bangladesh roads are generally flexible pavement type. To ensure expected performance of the pavement it is therefore important to have a sound knowledge on subgrade improvement. Subgrade improvement techniques are typically complex and also costly in most of the time. Structural capacity of flexible pavement depends on the characteristics of every single layer. On the other hand, the structural capacity of rigid pavement mostly dependent on the characteristics of concrete slab. Subgrade bearing capacity requirement of flexible pavement is substantially higher than rigid pavement. Bearing capacity of subgrade varies from 6 to 10 psi in flexible pavement whereas 1 to 3 psi in rigid pavement (AASHTO, 2020). SCP method of soil improvement has recently been used in selected locations of approach road of Padma Multipurpose Bridge Project for refining the subsoil conditions. It is notable that this type of highway project based on subgrade improvement techniques are expensive, but it's contribution in the long run are quite significant since it will reduce the problems of stability, deformation, excessive settlement and liquefaction.

In Bangladesh SCP and PVD have been used in few highway projects in subgrade improvement recently. In this research a venture is taken to look over the effect of subgrade improvement using SCP and PVD methods.

Selection of study area was the very first important step in research dissertation. Judicious selection of study area would help in many folds for correct findings of thesis. This study area had been selected primarily as a representative of reclaimed land which needs major ground improvement before highway construction over it. Another reason for selection of this area was two inter-connecting roads from Jolshiri to Shittalakha River and Jolshiri to Purbachal 300 feet Expressway would be constructed over this area. As subgrade condition is poor in this reclaimed area whish needs improvement therefore this area had been selected for this research work.

### 1.3 Scope and Objectives

Formed on the above raised background of the study, the principle objective of this research is to investigate and analyze the subgrade improvement through SCP and PVD in the study area. To satisfy the objective, SCP and PVD are installed respectively at the reclaimed land of ‘Purbachol 300' road to Madani Avenue link road’ and ‘Madani Avenue extension upto Shitalakkhya river’ in the study area with different depth and spacing. SPT-N value of this reclaimed land, governing contributor for determining the effectiveness of SCP and PVD are used as a significant parameter in this research. Based on the main purpose of this analysis, the specific objectives of this research are as follows:

(i) To assess the effectiveness of sand compaction pile (SCP) and prefabricated vertical drain (PVD) in improving the subgrade foundation soil.

(ii) To compare the cost of improvement through SCP and PVD.

It is expected that the investigation work will ease in identification and compare effectiveness SCP and PVD venture in the context of the reclaimed land of Bangladesh.

### 1.4 Outline of Methodology

This study on the effectiveness of SCP and PVD in improving soft soil for pavement subgrade will be conducted based on the field test and laboratory test results. The outline methodology of the research work is as follows:

(i) Conducting standard penetration test (SPT) in boreholes of four selected locations of the study area.

(ii) Collection of soil samples from the boreholes to conduct various laboratory tests for design soil parameters.

(iii) Design of sand compaction pile and prefabricated vertical drains for the site based on soil parameters obtained from field data and laboratory test results.

(iv) Installation of sand compaction pile in two selected locations, and also the prefabricated vertical drains in proximity locations.

(v) Performing standard penetration test (SPT) and dynamic cone penetration (DCP) tests as a measure of quality control.

(vi) Comparison of improved properties of soil, and cost for each of the methods.

### 1.5 Overview of the Thesis

The outcome of the study is presented in thesis comprising 5 (five) chapters. Chapter 1: Introduction is the introductory chapter that includes few general issues, background, objective and scope of the study. Chapter 2: Literature Review shows the review of the past researches on subgrade improvement in transportation engineering based on the SCP and PVD as well as their relevant analysis. This chapter includes history and chronological development works of SCP and PVD around the world. It also highlights few case studies of SCP and PVD which were performed under different geographical conditions by various researchers. Chapter 3: Test Program and Procedure describe the test program and overall experimental setup for this research work. It also includes the design of both SCP and PVD based on the obtained soil properties. The required diameter, spacing, length of the SCP and PVD were determined to suit the soil condition. Chapter 4: Test Results and Data Analysis describes the results and analyses in the survey area to investigate the effectiveness of the proposed ground improvement method. Detail location wise reading were tabulated in graphical presentation and result obtained are analyzed. This chapter also brought out findings of this research work through comparison of SCP and PVD basing on their effectiveness on ground. Economic analysis between these two methods were sorted out to compare their suitability in the study area. Chapter 5: Conclusions and Recommendations mainly summarizes all the chapter those are discussed above. It focuses on the findings of the research work and discusses the scopes for hereafter researches.

# CHAPTER 2
# LITERATURE REVIEW

## 2.1 General

The present study is focused to investigate the effectiveness of sand compaction pile (SCP) and prefabricated vertical drain (PVD) as an implement to upgrade reclaimed land. The soil in this area is very loose and soft for highway construction like most other part of Bangladesh. This chapter represents the basic idea of SCP and PVD, the soil parameters upon which they act and the available literature related to subgrade improvement techniques. These are mostly the former studies on SCP and PVD as an implement of pavement subgrade improvement.

This chapter also accords a synopsis of the techniques that are generally used in numerous locations of the world to enhance the performance of subgrade. In addition, conceptual ingredients of SCP and PVD methods including their expansion, related techniques of subgrade improvement, basis and various approaches are also discussed.

## 2.2 Background of Ground Improvement

Ground improvement methods have been used since ancient times. About 6000 years ago, in the neolithic age, the Banpo people in China used rammed columns to support wooden posts in the ground. They also used soil compaction methods using rammers. Different types of rammers were used, from stone rammers to iron rammers (about 1000 years ago). One type of rammer was operated by 8−12 people, each pulling a rope connected to the rammer to raise it and then letting it fall freely to pound the ground (Chen et al., 1995). About 3500 years ago, reeds in the form of bound cables (approximately 100mm in diameter) were used in Iraq as horizontal drains for dissipation of pore water pressure in soil mass in high earth structures (Mittal, 2012).

About 2000 years ago, the romans used lime for roadway construction. More than 1000 years ago in the Han dynasty, Chinese people-built earth retaining walls using local sand and weeds for border security and paths to the western world. About 500 years ago (Ming dynasty of China), lime was mixed with clayey soil in proportion (typically 3:7 or 4:6 in volume) to form compacted lime−soil foundations for load support (Chen et al., 1995).

Modern ground improvement methods were developed since the 1920s. For example, the use of vertical sand drains to accelerate consolidation of soft soil was first proposed in 1925 and then patented in 1926 by Daniel D. Moran in United States. Cotton fabric was used as reinforcement by South Carolina Highway Department in the USA for roadway construction in 1926. The vibro-flotation method was developed in Germany to densify loose cohesion less soil in 1937. The first type of prefabricated vertical drains was developed by Walter Kjellman in Sweden in 1947. Fernando Lizzi developed and patented the root pile method to underpin existing foundations in Italy in 1952. In the 1960s, there were several developments of ground improvement methods, including the steel reinforcement for retaining walls by Henri Vidal in France, dynamic compaction by Louis Menard in France, deep mixing in Japan and Sweden, and jet grouting in Japan. In 1986, J. P. Giroud acclaimed the development from geotextiles to geosynthetics is a revolution in geotechnical engineering (Giroud, 1986; Han, 2015).

## 2.3 Subgrade Improvement Techniques

Subgrade improvement is mainly required for reclaimed lands, loose and soft clay. These can cause serious problem of accelerated runoff, erosion and poor soil structure. The ground settling is caused by the immoderate withdrawal of ground water. To ease such difficulty, different subgrade improvement methods have been studied, namely: Sand Compaction Piles (SCP), Prefabricated Vertical Drains (PVD), Vibro-Compaction, Vacuum Consolidation, Preloading of Soil, Soil Nailing, Grouting and so on. In view of their proven performance, durability, constructability SCP and PVD are seem to be very suitable and favorable below ground subgrade improvement methods of reclaimed lands.

The SCP method was practically applied as a densification method in 1957 and as a replacement method in 1966 that was rather early among the improvement methods (Kitazume, 2005). In the 1920s, a technique for installing sand drains, a PVD predecessor, was patented in the U.S. The California Division of Highways, Materials and Research Department conducted laboratory and field tests on vertical sand drain performance beginning in 1933 (Holtz, 1987).

Improvement of the subgrade is integral with and dependent on the improvement of the underlying natural ground formation. Ground treatment is required at reclaimed areas and poor ground areas, as the naturally occurring sub-soils may be unable to support the highway, embankment and rail system.

### 2.4 Methods of Ground Treatment

Ground treatment methods for soft ground can be broadly categorized into the structural (rigid) solutions based on various considerations, which included the height of fill, thickness and compressibility of the soil as well as time and cost. Following methods of ground treatment can be adopted for various poor ground conditions.

(i) Vibratory surface compaction and Deep vibro-compaction

(ii) Removal and replacement of soft cohesive deposits of limited thickness

(iii) Preloading of existing soft and loose fill

(iv) Sand Compaction Piles

(v) Preloading with vertical drains

(vi) Dynamic Replacement

(vii) Stone Column

(viii) Piled Embankments in areas having soft soil to large depths

(ix) Viaduct for embankments having very deep soft soils with organic deposits (The Constructor, 2021).

The methods are illustrated in Fig. 2.1. The purpose of soil and ground Improvement is essentially to alter the natural properties of soil and control the behavior of a geotechnical feature or earthwork in order to improve the behavior and performance of a project. Among the properties that are usually targeted for improvement are:

(i) Reducing compressibility to avoid settlement

(ii) Increasing strength to improve stability, bearing capacity, or durability

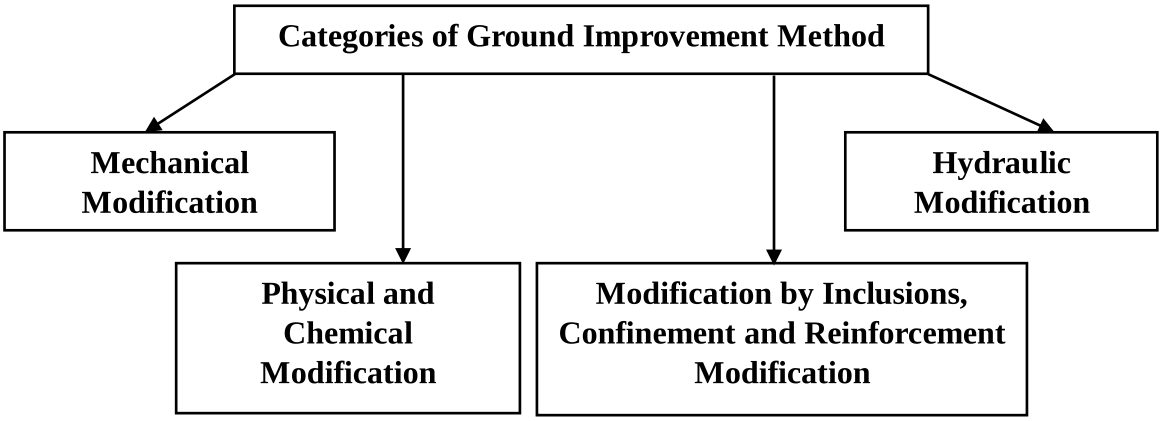
(iii) Reducing permeability to restrict groundwater flow

(iv) Increasing or decreasing permeability to allow drainage

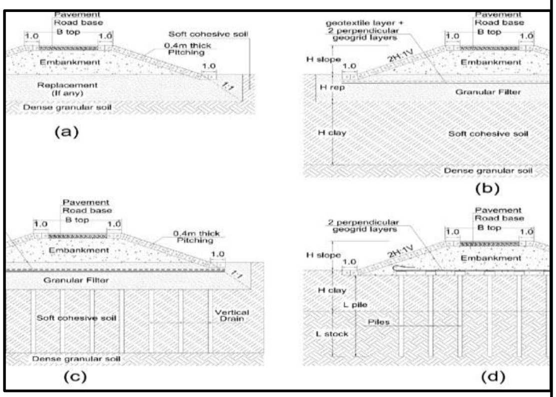
(v) Mitigating the potential for (earthquake-induced) liquefaction

## 2.5 Categories of Ground Improvement Method

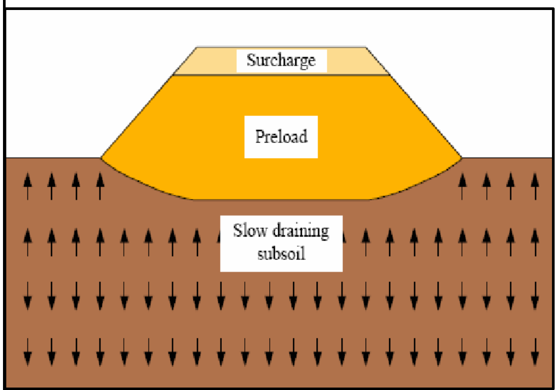
The approaches incorporating ground improvement processes can generally be divided into four categories grouped by the techniques or methods by which improvements are achieved, as shown in the following line diagram.




(a) Ground improvement using vibro-compaction method



(a) Removal and replacement of soft cohesive deposits of limited

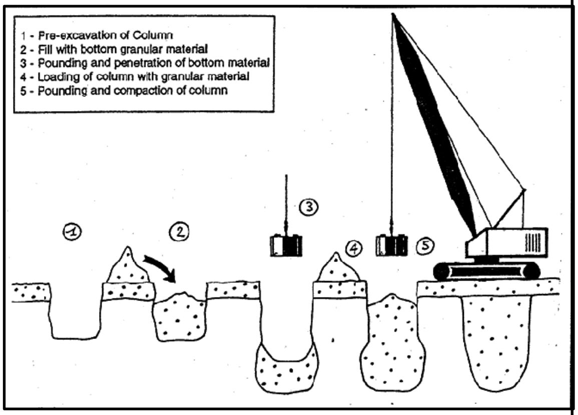



(c) Preloading of existing soft and loose fill

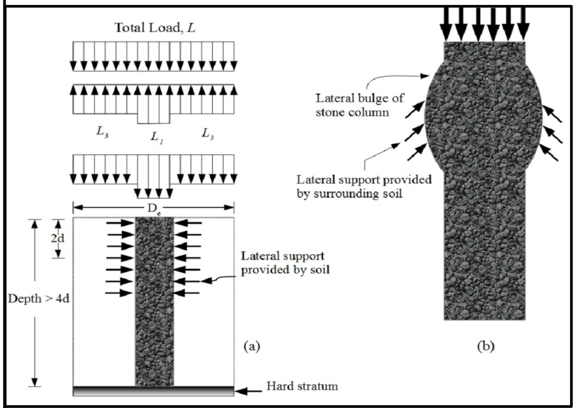
(d) Sand compaction pile

Fig. 2.1: Various ground improvement methods

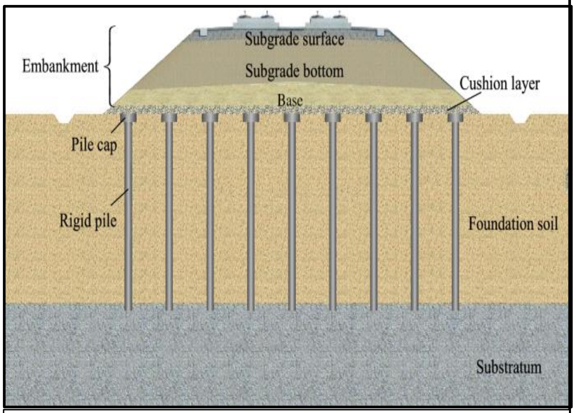
(e) Prefabricated vertical drains



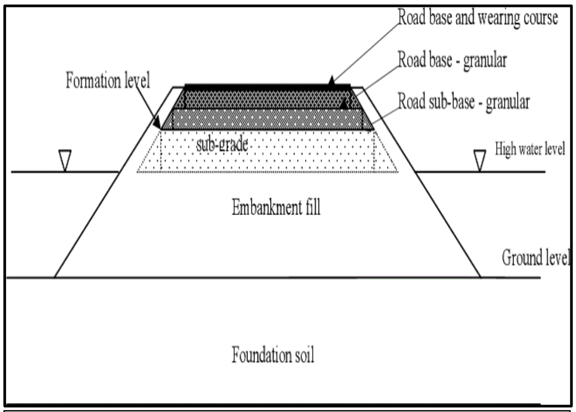

(f) Dynamic replacement installation




(g) Schematic of stone column with deformed shape




(h) Piled Embankments




(i) Viaduct for high embankments

Fig. 2.1 (contd.): Various ground improvement methods.

(a) Mechanical Modification: Includes physical manipulation of earth materials, which most commonly refers to controlled densification either by placement and compaction of soils as designed “engineered fills,” or “in-situ” (in place) methods of improvement for deeper applications. Many engineering properties and behaviors can be improved by controlled densification of soils by compaction methods. Other in situ methods of improvement may involve adding material to the ground as is the case for strengthening and reinforcing the ground with nonstructural members.

(b) Hydraulic Modification: Where flow, seepage, and drainage characteristics in the ground are altered. This includes lowering of the water table by drainage or dewatering wells, increasing or decreasing permeability of soils, forcing consolidation and pre-consolidation to minimize future settlements, reducing compressibility and increasing strength, filtering groundwater flow, controlling seepage gradients, and creating hydraulic barriers. Control of hydraulic characteristics may be attained through a variety of techniques that may incorporate improvement methods associated with other ground improvement categories.

(c) Physical and Chemical Modification: Stabilization of soils caused by a variety of physiochemical changes in the structure and/or chemical makeup of the soil materials or ground. Soil properties and/or behavior are modified with the addition of materials that alter basic soil properties through physical mixing processes or injection of materials (grouting), or by thermal treatments involving temperature extremes. The changes tend to be permanent (with the exception of ground freezing), resulting in a material that can have significantly improved characteristics. Recent work with bio stabilization, which would include adding or introducing microbial methods, may also be placed in this category.

(d) Modification by Inclusions, Confinement, and Reinforcement: Includes use of structural members or other manufactured materials integrated with the ground. These may consist of reinforcement with tensile elements; soil anchors and “nails”; reinforcing geo-synthetics; confinement of materials with cribs, gabions, and “webs”; and use of lightweight materials such as polystyrene foam or other lightweight fills. In general, this type of ground improvement is purely physical through the use of structural components. Reinforcing soil by vegetating the ground surface could also fall into this category.

Table 2.1: Ground improvement methods (After Kitazume, 2005)

| Improvement principle | | Engineering method | Work examples | Period practical application introduced |
|---|---|---|---|---|
| Replacement | | Excavation Replacement | Dredging replacement method | → |
| | | Forced Replacement | Sand compaction pile method | 1966 → |
| Consolidation | | Preloading | Preloading method | 1928 → |
| | | Preloading with vertical drain | Sand drain method | 1952 → |
| | | | Packed sand drain method | 1967 → |
| | | | Board drain method | 1963 → |
| | | Dewatering | Deep well method | 1944 → |
| | | | Well point method | 1953 → |
| | | | Vacuum consolidation method | 1971 → |
| | | Chemical dewatering | Quick lime pile method | 1963 → |
| Densification | Dewatering compaction | Compaction by displacement and vibration | Sand compaction pile method | 1957 → |
| | | | Grand compaction pile method | 1965 → |
| | compaction | Vibration compaction | Vibro-floatation method | 1955 → |
| | | Impact compaction | Dynamic consolidation method | 1973 → |
| Solidification (Admixture stabilization) | | Agitation mixing | Shallow mixing method | 1972 → |
| | | | Deep mixing method | 1974 → |
| | | Jet mixing | Jet mixing method | 1981 → |
| Contact pressure reduction | | Load Distribution | Fascine mattress | → |
| | | | Sheet net method | 1962 → |
| | | | Sand net method | → |
| | | | Surface solidification method | 1970 → |
| | | Balancing loads | | → |

## 2.6 Factors Affecting Ground Improvement Method

Selection of ground improvement methods are based on certain conditions. It must be remembered that, the interplay of geotechnical parameters and ground improvement is complex. This has resulted in a gap between the understanding of geotechnical properties of subgrades based on research findings, and the design and construction practices for these elements. Following key conditions need to be understood before selecting any particular ground improvement methods:

(a) Structural Conditions: The structural conditions may include type, shape, width and dimension of pavement, magnitude and distribution of loads and overall performance requirements.

(b) Geotechnical Conditions: The geotechnical conditions may include geographic landscape, geologic formations, type, location, and thickness of problematic geomaterial, distribution of fill and groundwater table. Soil type and particle size distribution are essential for preliminary selection of ground improvement methods. This guideline is particularly suitable for ground improvement methods for subgrade. The thickness and location of problematic geomaterial are also important for the selection of ground improvement methods. When a thin problematic geomaterial layer exists at a shallow depth, the over excavation and replacement method is one of the most suitable and economic method. When a relatively thick loose cohesionless geomaterial layer exists near ground surface, dynamic compaction and vibro-compaction methods are suitable ground improvement methods. When a relatively thick soft cohesive geomaterial layer exists near ground surface, preloading and deep mixing methods may be used. The level of groundwater table often affects the selection of ground improvement methods. Deep excavation in a ground with a high groundwater table seriously hampers the construction work and need to replace by some other suitable method.

(c) Environmental Constraints: The environmental constraints may include limited vibration, noise, traffic, water pollution, spoil, and headspace. For example, dynamic compaction induces vibration and noise, which may not be suitable in a residential area. The wet method to construct stone columns by water jetting produces spoil on site area, which may be troublesome for a site with limited space. Under such a condition, the dry method may be used instead. Preloading induces settlements at nearby areas, which may be detrimental to existing structures.

(d) Construction Conditions: The selection of a ground improvement method should consider the following construction conditions:

   (i) Site area condition

   (ii) Allowed construction time

   (iii) Availability of construction material

   (iv) Availability of construction equipment

   (v) Construction cost

The selection of a ground improvement method must consider whether the site is accessible to its associated construction equipment. Construction time is one of the most important factors for the selection of a ground improvement method. For example, preloading is a cost-effective ground improvement method to improve soft soil; however, it takes time for the soil to consolidate. The use of prefabricated vertical drains can accelerate the rate of consolidation, but sometimes it still may not meet time requirement.

(e) Reliability and Durability. Reliability of a ground improvement method depends on several factors, such as variability of geotechnical and structural conditions, variability of construction material, quality of the Contractor, quality of installation and quality control. For example, geo-synthetics have creep behavior. The corrosion of steel reinforcement with time reduces its thickness. The strength of cement-stabilized soil in seawater degrades with time (Han, 2015).

## 2.7 Sand Compaction Pile Method of Ground Improvement

To solve the problems of stability, excessive settlement and liquefaction due to very soft/loose soil deposits encountered by the construction projects, a variety of ground improvement techniques have been evolved around the world. One of them, the Sand Compaction Pile (SCP) method has been developed and frequently adopted for many construction projects specially in Asia for improving sandy and clay grounds in which sand is fed into a ground through a casing pipe and is compacted by either vibration, dynamic impact or static excitation to construct a compacted sand pile in a soft/loose soil ground (Samanta et al., 2010; Unnikrishnan and Johnson, 2009; Kitazume, 2005; Kempfert, 2003; Solymar et al, 1986; Moh et al., 1981).

### 2.7.1 General Principle and Purpose of SCP

Developed in Japan, the sand compaction pile (SCP) method is used to strengthen soft ground by installing sand or a similar material into the soft ground via a casing pipe and vibrating the sand to produce firmly compacted sand piles in the ground. According Harada and Ohbayashi (2017), SCP can be to all three representative soil types, i.e., sandy grounds, clayey grounds and soft clay deposits for various reasons and they explained the reasons of its versatile use that are stated below.

The principle of the SCP method for clayey grounds is based on the theory for composite grounds proposed by Murayama (1957). Composite grounds consist of soft cohesive grounds and compacted sand piles formed therein; the composite ground formed has high shear strength and drainage capability owing to the presence of the sand piles. Through the formation of these compacted sand piles, the bearing capacity of the ground can be increased due to replacement effect and stress concentration effect. Stress concentration means that external load is concentrated mainly on the sand piles, as shown in Fig. 2.2(a). Furthermore, by including drainage effect an increase in the stiffness of the whole ground as well as a decrease in lateral spreading and in consolidation settlement can be expected.

The principle of the SCP method for sandy grounds is primarily to decrease the void ratio and to densify the ground as a result of the sand pile installation, as shown in Fig. 2.2(b). Accordingly, the purpose of the SCP method is to increase the bearing capacity, to decrease the compression settlement, to prevent the occurrence of liquefaction, and to increase horizontal resistance. For sandy grounds, Ogawa and Ishido (1965) suggested a practical design procedure related to the increase in density due to the installation of sand piles.

Conversely, for soft clay deposits which are typically encountered in offshore works, thicker sand piles are installed into the clay at the sea bottom, as shown in Fig. 2.2(c). Forced replacement is the major principle for the improvement of offshore works, rather than the formation of composite ground where the sand piles replace the cohesive soils. In such cases, the objectives of the improvement are to increase the bearing capacity, to reduce the consolidation settlement, and to increase the horizontal resistance.

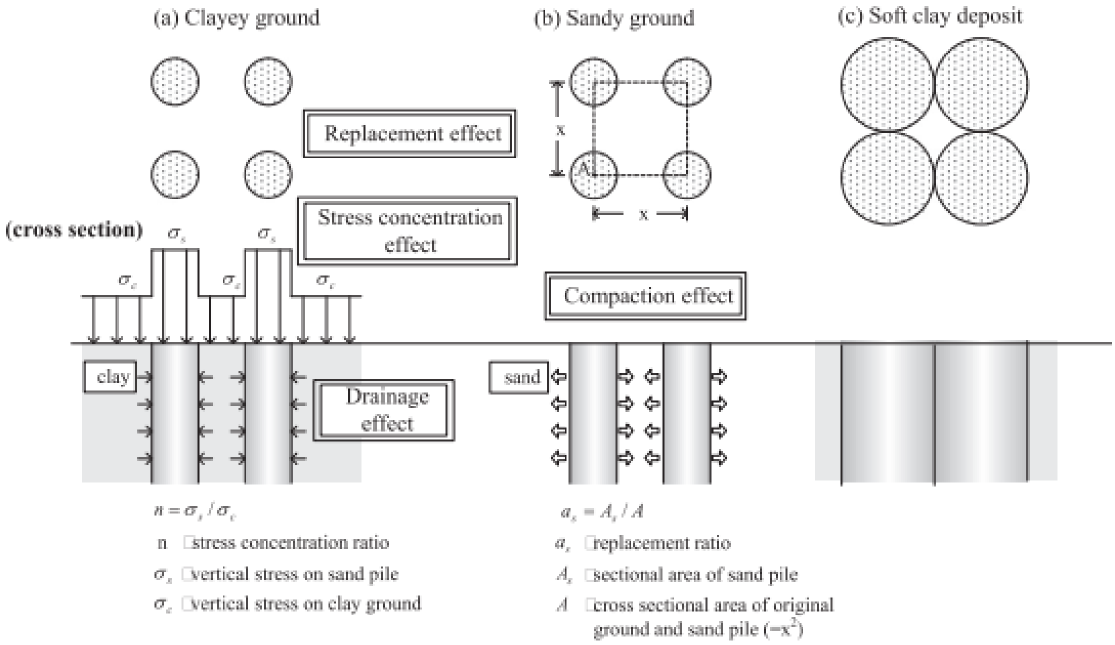



Fig. 2.2: Concept of working of sand compaction piles (after Harada and Ohbayashi, 2017).

### 2.7.2 Historical Background of SCP

A comprehensive historical background of SCP is presented by Ezoe, Harada and Otani (2019) that may be reproduced as under.

The history of the SCP method began in 1956 when the onshore vibratory SCP method was developed as a ground improvement method for cohesive soil ground. The background to this development was that the major geotechnical focuses at that time were stability and settlement of cohesive soil ground. SCP was then applied to sandy soil in 1961 as a field test to enhance the ground's bearing capacity for spread foundation of residential housing complexes and for liquefaction remediation. Wider use as liquefaction countermeasure for various structures began after the Niigata Earthquake in 1964. Gradually, the method then accumulated successful results as liquefaction countermeasure. In 1965, the method was fully applied to offshore construction to improve the foundation of port structures.

The SCP method's effectiveness as liquefaction countermeasure was first confirmed when a major earthquake occurred of the coast of Miyagi Prefecture, Japan in 1978 when the method successfully prevented the liquefaction of the foundation of a tank (Ishihara, Kawase and Nakajima, 1980). Detailed academic verification followed to examine why the method prevented liquefaction. Many earthquakes occurred thereafter, including the Nihonkai–Chubu Earthquake in 1983. A follow-up survey on projects which employed the

SCP method indicated that the earthquakes did not damage the ground improved by this method. As a result, it became generally understood that the SCP method was an effective countermeasure against liquefaction. However, since the vibratory SCP method employs a vibro-hammer, which means it generates vibration and noise during implementation, it was sometimes difficult to use the method in urban areas and at sites adjacent to other buildings or structures. Then, to resolve the above-mentioned environmental issues, a non-vibratory SCP method with the following characteristics was developed in 1995, which employed static compaction, provided with a forced lifting/ driving device and rotating drive unit for reducing vibration and noise (Harada et al., 2004). In addition, an offshore machine dedicated to implementing this method was put into practical use in 2002.

The non-vibratory SCP method has been widely applied to locations where vibration and noise should be avoided, such as urban areas and sites adjacent to structures. However, as large machines become necessary for some versions of this method, it is not used at sites with constricted space because it is very difficult to work at such areas, and the number of cases involving narrow areas has slowly increased. Moreover, when compared with conventional methods that use compact machines (such as jet grouting or chemical grouting), there were also cases in which cost reduction or use of materials that can reduce the load to the environment (such as sand) is required. In such circumstances, the injection-type SCP method with further downsizing of execution machine was developed and put into practical use in 2008 (Imai et al, 2009; cited by Ezoe1, Harada and Otani, 2019). The history of the development of vibratory SCP, non-vibratory SCP, and injection-type SCP methods, as well as their accumulated construction lengths, is shown in Fig. 2.3.

### 2.7.3 Basic Installation Principles of SCP

Sand compaction pile (SCP) method involves driving a hollow steel pipe into the ground. The bottom is closed with a collapsible plate down to the required depth and then pipe is filled with sand. The pipe is withdrawn while the air pressure is directed against the sand inside it. The bottom plate opens during withdrawal and the sand backfills the voids created earlier during the driving of the pipe. The sand backfill prevents the soil surrounding the compaction pipe from collapsing as the pipe is withdrawn. Thus, the soil gets densified. (Kitazume, 2005).

| Classification | Application | | | | | | |
|---|---|---|---|---|---|---|---|
| | 1950's | 1960's | 1970's | 1980's | 1990's | 2000's | 2010's |
| Vibratory SCP | '56 on shore | '65 off shore | | | | | |
| Accumulated construction length (thousand km) | | 12 | 80 | 180 | 330 | 380 | 400 |
| Non-Vibratory SCP | | | | | '95 on shore | '02 off shore | |
| Accumulated construction length (thousand km) | | | | | 1.9 | 6.8 | 13.0 |
| Injection type SCP | | | | | | | '08 on shore |
| Accumulated construction length (thousand km) | | | | | | | 0.2 |

Fig. 2.3: Chart showing history and accumulated construction length of SCP method (after Ezoe, Harada and Otani; 2019)

### 2.7.4 Construction Method and Materials Used

There are basically three types of mechanized SCP installation methods depending on the system deployed. First one is vibratory system with vibro-hammer, the second one has a non-vibratory system with forced lifting or driving device and the third one is the recently developed (in 2008) injection-type SCP method with downsizing of the execution machine. Their working principles and construction sequences along with their pictures are illustrated in Figs. 2.4, 2.5 and 2.6 respectively. The marks 1, 2, 3 and so on in the Figure indicate the construction sequence.

In Bangladesh, use of these sophisticated mechanized method of SCP construction like vibro-composer or silent non-vibro-composer and injection type are very limited because of the availability of these heavy equipment. However, vibro-composer SCP has been used in few projects, for example, Padma Multipurpose bridge project. Some manual method called cased borehole method as reported by Juneja and Mir (2012) is common in Bangladesh that basically uses a heavy cylindrical bailer dropped from a height on to the ground to create a displacement type hole in the ground. The hole is then filled up with sand and compacted again with dropping the bailer thus forming a SCP type sand column. The construction sequences with marks 1, 2, 3 etc. are illustrated in Fig. 2.7. Though their use is very common, yet their performance is awaiting to be reported by investigators in the form of authentic documents especially for Bangladesh soil.

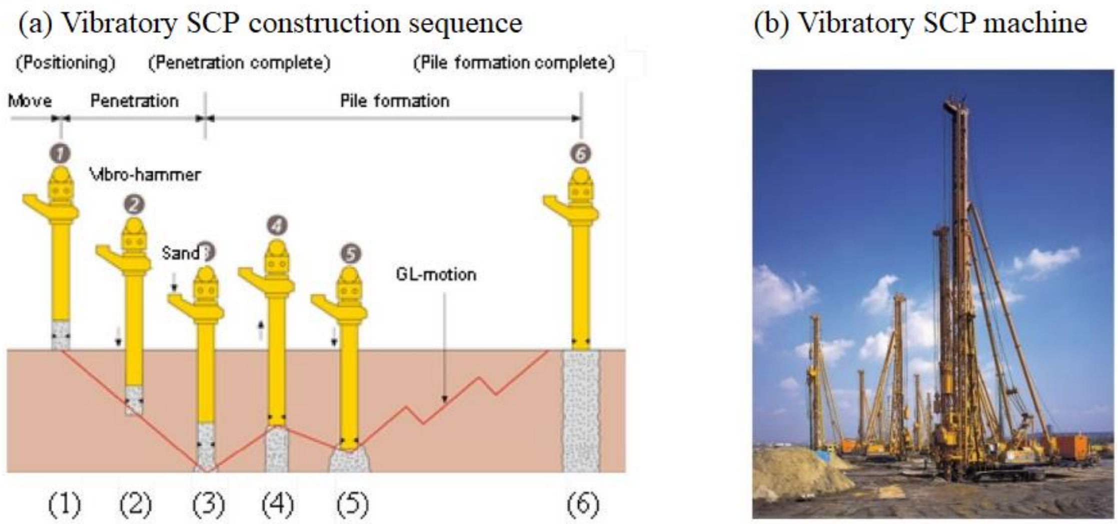



Fig. 2.4: Construction sequence and picture for vibratory SCP (after Ezoe, Harada and Otani, 2019).

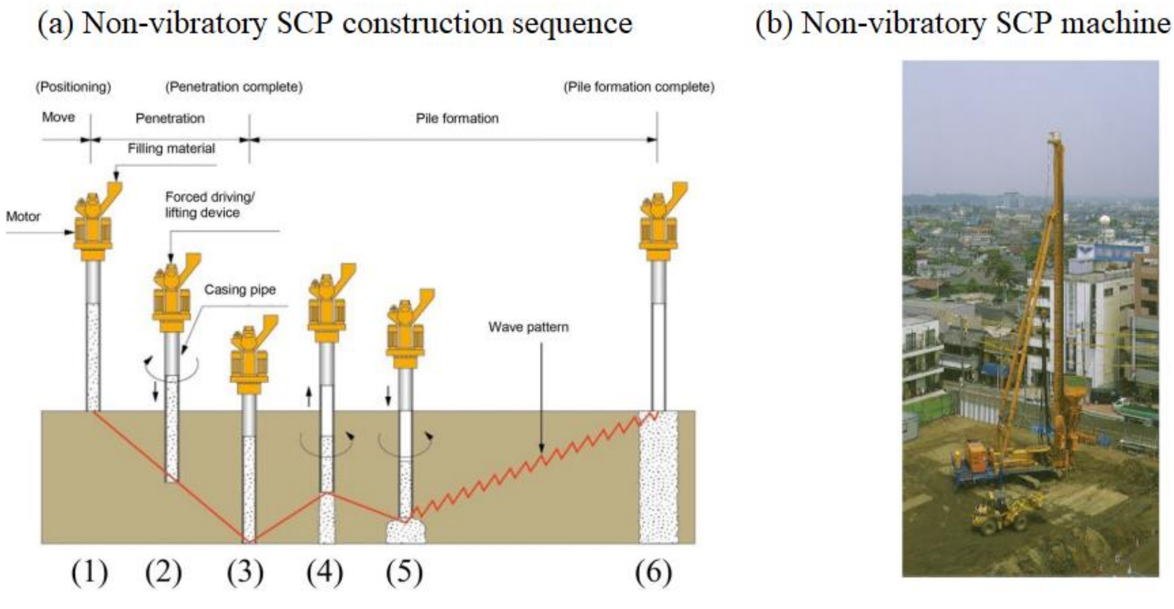



Fig. 2.5: Construction sequence and picture for non-vibratory SCP (after Ezoe, Harada and Otani, 2019)

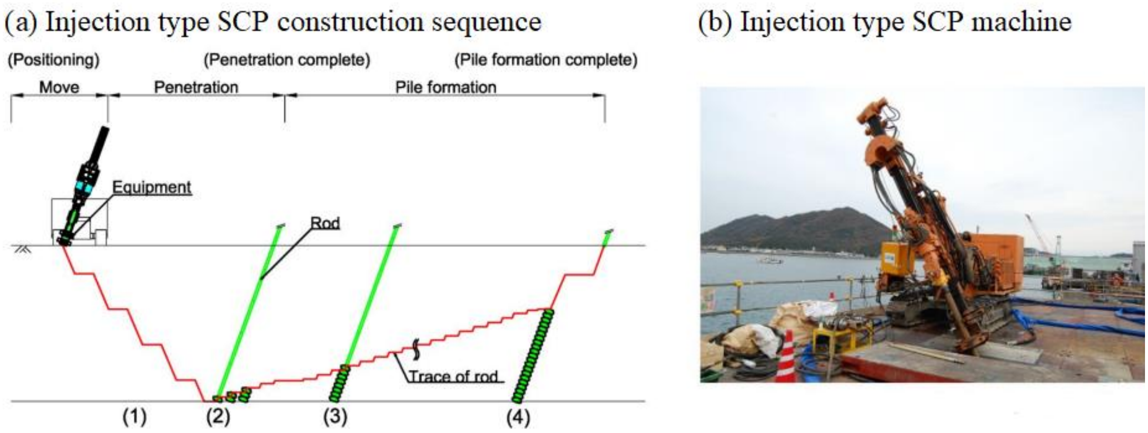



Fig. 2.6: Construction sequence and picture for injection type SCP (after Ezoe, Harada and Otani; 2019).

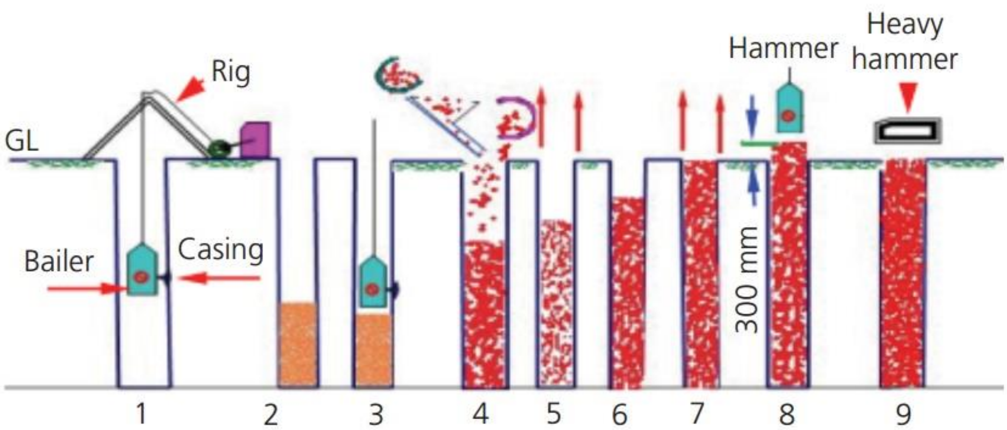



Fig. 2.7: Construction sequence of cased borehole type SCP (after Juneja and Mir; 2012).

### 2.7.5 Construction Method and Materials Used

According to Ezoe1, Harada and Otani (2019), the sand compaction pile (SCP) method is used to strengthen soft ground by installing sand or a similar material into the soft ground via a casing pipe and vibrating the sand to produce firmly compacted sand piles in the ground. The method's basic improvement principles are "compaction" and "drainage". Therefore, the SCP method has been applied in Japan to construct foundations for various structures, including roads, ports, and buildings, as a soft ground improvement technique applicable to various types of ground, such as sandy soil or cohesive soil. It requires only a single machine and its performance has been successfully demonstrated in many projects. In particular, when applying SCP to improve sandy soil, it is also used to prevent the occurrence of liquefaction in many cases. Good performance of many SCP-improved grounds in past large earthquakes proved the method's effectiveness. Thus, it is considered to be the most reliable method of liquefaction countermeasure in Japan.

According to Ezoe1, Harada and Otani (2019), when applying the SCP method to cohesive soil to create a composite ground, which consists of compacted sand piles and the surrounding in situ soil, the method's principle demonstrates a composite effect that combines sand piles with high shear strength (replacement effect and stress concentration effect) and the effect of dehydration from surrounding cohesive soil by the sand piles (draining effect), as shown in Fig. 2.2(a). Meanwhile, when applying the method to sandy soil, compacted sand piles are compacted to reduce the void ratio of the sandy soil adjacent to the said sand piles and create soil with high density, high bearing capacity, and high resistance to liquefaction, Fig. 2.2(b).

This method was originally developed in order to increase the density of loose sandy ground and to increase the uniformity of sandy ground, to improve its stability or compressibility and to prevent liquefaction failure, but now it has also been applied to soft clay ground to assure stability and to reduce ground settlement (Kitazume, 2005).

The principle concept of the SCP method for application to sandy ground is to increase the ground density by placing a certain amount of granular material (usually sand) in the ground. The principle concept for application to clay ground on the other hand, is reinforcement of composite ground consisting of compacted sand piles and surrounding clay, which is different from sand drain method in which sand piles without any compaction are constructed principally for drainage function alone (Kitazume, 2005).

The sand compaction pile (SCP) method has been applied to improve soft clays, organic soils and loose sandy soils for various purposes and in various ground conditions. Table 2.2 shows typical improvement purposes of the SCP. Table 2.3 describes the purpose of SCP under various structures constructed on land sites (Hossain, 2015).

Table 2.2: Typical improvement purposes of SCP method (After Kitazume, 2005)

| Application of SCP | | |
|---|---|---|
| Sl. No. | Clay soil | Sandy soil |
| 1 | Increasing bearing capacity and passive earth | Increasing bearing capacity |
| 2 | Reducing settlement and active earth | Reducing settlement |
| 3 | Increasing horizontal resistance to pile & sheet | Preventing Liquefaction |

Table 2.3: SCP applications for on-land construction (After Kitazume, 2005)

| Sl. No | Structure | Purpose of using SCP |
|---|---|---|
| 1 | Embankments for road & highway | Prevention of sliding failure of bearing capacity, reduction of settlement |
| 2 | Filling behind bridge foundation | Prevention of sliding failure of bearing capacity, reduction of settlement, prevention of liquefaction |
| 3 | Storage yard for power station | Prevention of sliding failure of bearing capacity, reduction of settlement |
| 4 | River embankment | Prevention of sliding failure of bearing capacity, reduction of settlement, prevention of liquefaction |
| 5 | Foundation of building and factory | Prevention of sliding failure of bearing capacity, reduction of settlement |
| 6 | Underground structure | Increase of bearing capacity, prevention of liquefaction, reduction of earth pressure, increase of $K_o$-value |
| 7 | Foundation of tank and retaining wall | Prevention of sliding failure, increase of bearing capacity, reduction of settlement, prevention of liquefaction, increase of $K_o$-value |

The primary advantage of sand piles is that the sand used is often considerably cheaper when compared to other similar ground improvement techniques like stone columns. Construction of the sand columns is extremely fast. After creating the hole, it’s fully supported by casing during construction that prevents the possibility of collapse. Some advantages of SCP can be summarized as below:

(i). Provision of sand piles allows drainage of pore water in radial direction in addition to the drainage in vertical direction.

(ii). Since the permeability of soil in horizontal direction is usually several times larger than that in vertical direction, the rate of consolidation becomes considerably faster compared to conventional soil system.

(iii). With faster consolidation, the soil gains shear strength rapidly, allowing faster pace of construction and thus reducing the project cost.

(iv). The long-term stability of the structure is also increased significantly as the potential settlements are completed mostly before or during the construction.

(v). Sand drains avoid potential problems during construction in soft soils.

Sand compaction piles have a low stiffness when compared to other methods. Hence larger percentage replacement of weak soil is required. These piles do not have sufficiently high permeability to function as effective vertical drains during earthquakes. Some disadvantages of SCP can be summarized as below:

(i) Installation of sand piles by driving down hollow mandrels causes a disturbance of the soil surrounding each pile. This may reduce the flow of water to the drain.

(ii) To receive adequate drainage properties, sand has to be carefully chosen which might seldom be found close to the construction site.

(iii) Drains might become discontinuous because of careless installation or horizontal soil displacement during the consolidation process.

(iv) During filling bulking of the sand might appear which could lead to collapse due to flooding.

(v) Construction problems or budget may arise due to large diameter of sand piles.

(vi) The reinforcing effect may reduce the effectiveness of preloading the subsoil.

## 2.8 Case Studies Regarding Sand Compaction Pile

Some case studies are discussed here concerning installation of SCP in different situations. SCP is mainly concerned in improving and modifying ground or subgrade condition for highway construction or structural purpose. Through installation, consolidation process remains faster which empowering faster construction and reducing construction cost. Here in the following case studies are considered in improving alluvial soil, clayey soils and their effects through improvement by SCP. Studies are as follows:

(i). Effects of Installation Method on Sand Compaction Piles in Clay in the Centrifuge

(ii). Effectiveness of Sand Compaction Pile (SCP) in improving the density of alluvial soil deposits of Bangladesh

(iii). Effects of Sand Compaction Pile Installation in Model Clay Beds.

### 2.8.1 Effects of Installation Method on Sand Compaction Piles

Lee et al. (2001) investigated the effect of installation method on the soil properties due to sand compaction piles. In this study the effects of the method of installation of centrifuge model sand compaction piles (SCPs) in soft clay were deliberated. A comparative study among the frozen pile method, a 1-g displacement method and a high-g displacement method was discussed in this research. The results showed that, although all the SCP-improved models exhibit higher strength compared to the unimproved models, both displacement methods confer additional enhancement in strength to ground improvement, which was not present in the frozen pile models. In addition, the frozen pile models observed wavy settlement patterns. The suggested differences in stress states of the improved models explained the observed differences. In the case of the frozen pile models, thawing during reconsolidation was advanced to lead to a reduction in effective lateral stress, resulting in further softening of the clay. In this study, the cavity expansion effect caused by the displacement methods was proposed to lead to a set-up in the strength of the clay, resulting in better cohesion in the feedback of the improved ground to loading.

The findings can be summarized that the displacement SCP installation methods consult higher rigidity and toughness to the improved ground, collated to the frozen pile methods. This study also revealed that the lower rigidity and strength of the frozen pile improved ground can be assigned to moderating of the soft clay as the sand piles reduce upon thawing, thus allowing the softened clay to flow around the SCPs under loading. In contrast, the cavity expansion enforced by the displacement methods induces a set-up effect, leading to increased strength in the soft clay, thereby obstructing such local soil flow.

### 2.8.2 Sand Compaction Piles in Improving Soil Bed Formed of Alluvial Deposits

Hossain (2015) carried out a laboratory investigation on sand compaction pile preparing a sand bed of alluvial soil of Bangladesh. Small sand compaction pile device, miniature dynamic cone penetrometer (DCP), soil tank and sand shower bowl were planned and manufactured for the purpose in his study. Different alluvial sandy soil samples were accumulated from two selected locations of Bangladesh. Soil beds were structured in the soil tank by flowing sand shower from different heights using the specially prepared sand shower bowl so as to attain sand beds of various densities. The density of soil bed, thus developed, was calculated using density pots and dynamic cone penetration readings were taken to scale the soil bed density against cone penetration in this research.

For the purpose of this study, the effectiveness of sand compaction piles was scrutinized in improving the density of loose soil deposits, a sand bed of loose density was formed by sand raining from pre-calibrated height. Initial density of the soil bed was measured using the miniature dynamic cone penetrometer. Sand compaction piles were placed in the soil bed using the small sand compaction pile device where a hole was arranged in the soil bed by displacing the soil in the lateral direction and pouring sand in the holes and compacted. Square and triangular arrangements of sand compaction piles were operated with various spacing. The density of the sand bed with installed sand compaction pile was measured at locations in between the sand piles using the miniature. A term penetration index for the DCP test value was introduced to indicate the density of soil bed.

In this investigation, DCP value and field density were analyzed to derive correlation parameters between dynamic cone resistance (Penetration Index) and relative density of sand. This correlation was used to determine the relative density of improved soil bed due to sand compaction piles of various spacing and arrangements. Results indicated that a triangular arrangement of SCP with a spacing of 2.5 times the diameter of the pile would the most systematic arrangement for development of soil bed formed of alluvial soil of Bangladesh. The study yielded useful correlation equations to estimate density from DCP values, and also between SCP spacing and density. In this research it revealed that soil improvement due to SCP is not only the function of replacement ration, but also a function of SCP arrangements. This study also suggested that penetration index is not uniquely related to density of soil, rather it is a function of grain characteristics of soil that needed to be investigated.

### 2.8.3 Effects of Sand Compaction Pile Installation in Model Clay Beds

Juneja et al. (2011) carried out research on the effect on SCP installation on the properties of clay bed using centrifuge. The results of frozen pile method of installation at 1g to the in-flight method of SCP installation at high-g using the centrifuge were reported. Pore pressure replaces were recorded during the entire installation procedure. Stress set up was not observed in 1g tests. However, during the sand injection stage, stress relaxation did not occur in high-g tests. The centrifuge test results were then compared to plane strain cavity expansion theory (CET) in this study. The findings of this research appeared to show that the CET gave a soundly good evaluate at large depth for the entire installation process but not for the remaining stress after the casing jack-in during the first stage. These findings suggested that in order to prepare significant set-up of stress in the upgraded ground, there must be considerable further hollow expansion during the sand injection stage of SCP.

In his study the procedure of sand compaction pile insertion, which surfaced the way towards a more description of the state and by such means the performance of the improved ground after installation of sand compaction piles. In order to sort out remarkable set-up of stress in the improved ground, there must be substantial further cavity expansion during the sand injection stage of sand pile installation. It means that, for the same sand pile diameter, a miniature casing is likely to be able to initiate larger set-up than an enormous casing.

### 2.8.4 Design Considerations of Sand Compaction Pile

In general, granular column-reinforced foundations are designed as composite foundations while concrete columns are designed as piles. In the composite foundation design, a unit cell concept is often used for simplification. This section addresses the following design issues: (a) general rules (such as backfill material, area of improvement, pattern, area replacement ratio, and depth of improvement, and diameter of column), (b) bearing capacity, (c) settlement, (d) consolidation, (e) stability, and (f) liquefaction.

#### 2.8.4.1 Backfill Material

A rating system given by Eq. (2.1) has been developed by Brown (1977) to judge the suitability of backfill material for vibro-replacement based on the settling rate of the backfill in water and project experience using the suitability number Table 2.4.

$$S_N = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}}$$

Where, $D_{50}$, $D_{20}$, and $D_{10}$ are particle sizes of 50%, 20%, and 10% finer, respectively, in a unit of mm.

Table 2.4: Suitability rating of backfill material (after Brown, 1977)

| Suitability number | 0 - 10 | 10 - 20 | 20 - 30 | 30 - 40 | > 50 |
|---|---|---|---|---|---|
| Rating | Excellent | Good | Fair | Poor | Unsuitable |

#### 2.8.4.2 Patterns of SCP

Two patterns of columns as shown in Fig. 2.8 have been commonly used in practice. When the center-to-center spacing ($s_1$) is equal to $s_2$ in Fig. 2.8(a), it becomes a square pattern. When $s_1$ is equal to $s_2$ in Fig. 2.8(b), it becomes an equilateral triangular pattern.

Rectangular and triangular patterns are commonly used for most foundations while the radial pattern is most suitable for circular or ring foundations (e.g., tank foundations).

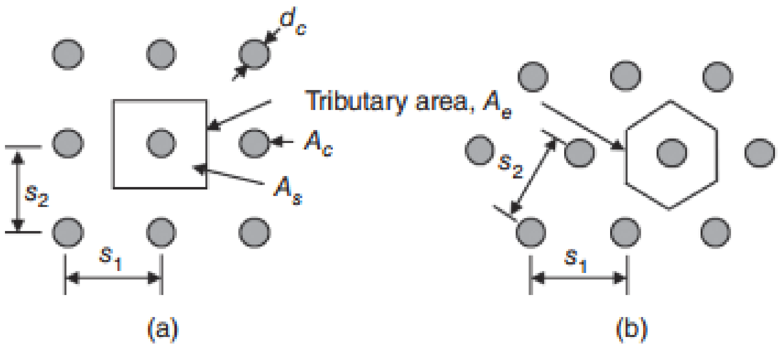



Fig. 2.8: Typical patterns of sand compaction piles: (a) rectangular and (b) triangular.

#### 2.8.4.3 Diameter of SCP Column

The diameter of sand compaction pile columns depends on the equipment used to install the columns. Typical column diameters used in practice varies from 300 mm to 800 mm.

#### 2.8.4.4 Area Ratio of SCP Column

When columns are installed, the area replacement ratio is defined as the ratio of the cross-sectional area of a column to the tributary area of the column, as shown in Fig. 2.9, that is,

$$a_s = \frac{A_c}{A_e} = C\left(\frac{d_c}{s}\right)^2$$

Where, $a_s$ = area replacement ratio

$A_c$ = Cross-sectional area of the column

$A_e$ = Tributary area of the column

$d_c$ = Diameter of the column

$s$ = Center-to-center spacing between columns in a square or equilateral triangular pattern

$C$ = Constant (π/4 or 0.785 for a square pattern or $\pi/\langle 2|3\rangle$ or 0.907 for an equilateral triangular pattern)

Area replacement ratios for granular columns without geosynthetic encasement typically range from 0.1 to 0.4. Larger ratios are used for very soft or loose soil. Geosynthetic encased granular columns are typically designed with area replacement ratios from 0.1 to

0.2 (Alexiew and Thomson, 2013). Concrete columns are typically installed with area replacement ratios of 0.05–0.15, based on the cross-sectional area of column shafts.

#### 2.8.4.5 Depth of Improvement

Depth of improvement should be determined based on-site conditions, soil properties, and performance requirements. According to Han (2015), the following rules may be followed:

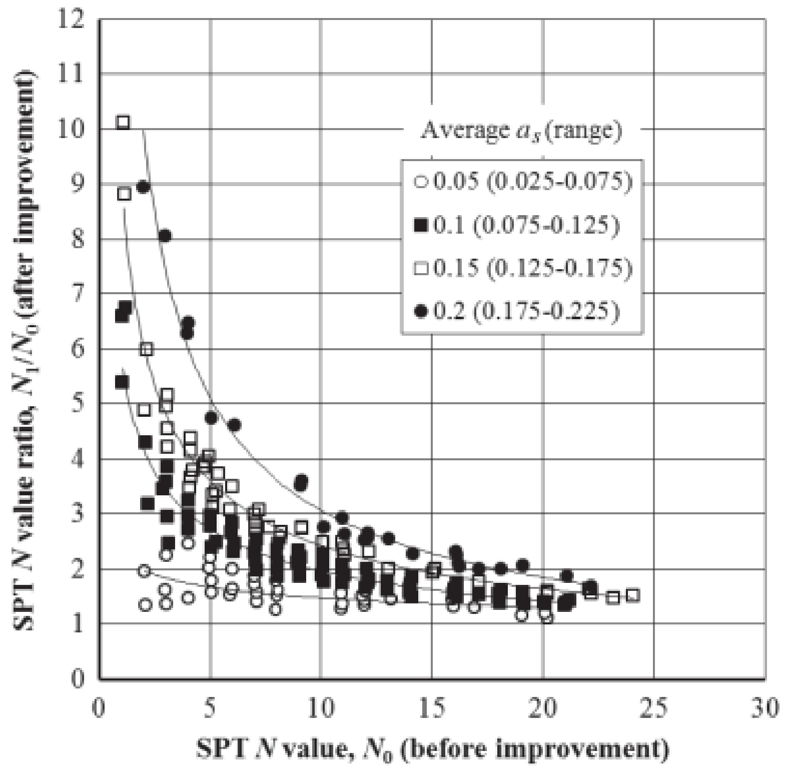
- When a firm stratum exists at a relatively shallow depth, the depth of improvement should reach this stratum.
- When a firm stratum exists at a relatively deep depth, the depth of improvement should be determined to meet performance requirements, such as bearing capacity, settlement, slope stability, and liquefaction.
- Typical depth of improvement ranges from 5 to 15 m.

#### 2.8.4.6 Area of Improvement

Han (2015) suggested that the area of improvement should be determined based on-site conditions and importance of superstructures. In general, the area of improvement should be larger than footprints of footings. Under a general condition, one to two rows of columns may be installed outside of a footing. On a liquefiable soil site, two to four rows of columns may be installed outside of a footing.

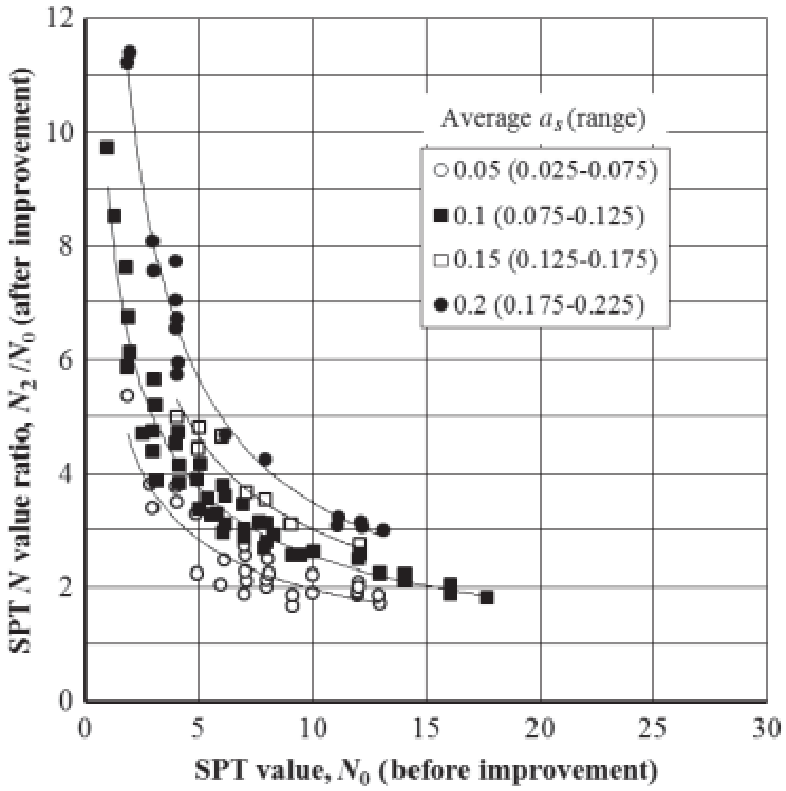
### 2.8.5 Densification Effect due to SCP in Granular Soil

The method for volume change by vibro-compaction with backfill can be used to analyze the densification effect on the surrounding soil. Kitazume (2005) suggested the use of Fig. 2.9 to estimate the SPT N values midway between sand compaction columns ($N_1$) and in the center of sand compaction columns ($N_2$). The SPT N values depend on the initial SPT N values ($N_o$), the area replacement ratio ($a_s$), and the location (between or in the center of columns). An increase of the initial SPT N value ($N_o$) or the area replacement ratio ($a_s$) increases the SPT N value after installation ($N_1$ or $N_2$). And, the SPT N value at the center of the columns ($N_2$) is higher than that midway between columns ($N_1$).



(a)

Fig. 2.9(a): SPT N values in midway between columns after installing SCP



(b)

Fig. 2.9(b): SPT N values at centre of columns after installing SCP

Kitazume (2005) suggested the average weighted SPT N value including the sand compaction column and the surrounding soil as follows:

$$N_{eq} = a_s N_2 + (1 - a_s) N_1$$

Where, $N_{eq}$ = average weighted (equivalent) SPT N value

$N_1 =$ SPT N value in the surrounding soil

$N_2 =$ SPT N value in the sand compaction column

### 2.8.6 Bearing Capacity due to SCP in Clay Soil

Brauns (1978; cited by Han, 2015) proposed a simplified method to estimate the ultimate bearing capacity of an individual stone column in saturated soft soil under an undrained condition. Since granular columns and the surrounding soil mobilize their strengths at a similar strain level, the ultimate bearing capacity ($q_{ult}$) of a granular column-reinforced composite foundation can be estimated as follows:

$$q_{ult} = q_{(ult,c)} a_s + q_{(ult,s)} (1 - a_s)$$

Where, $q_{(ult,s)}$ is the ultimate bearing capacity of the surrounding soil, which can be estimated as $5c_u$ for clayey soil as suggested by Barksdale (1987). It is recommended that the following formula be used to approximately estimate the ultimate bearing capacity of an individual sand column:

$$q_{(ult,c)} = 25c_u$$

## 2.9 Development of PVD Method

Prefabricated vertical drains (PVDs) are composed of a plastic core encased by a geotextile for the purpose of expediting consolidation of slow draining soils. They are typically coupled with surcharging to expedite preconstruction soil consolidation. Surcharging means to pre-load soft soils by applying a temporary or permanent load to the ground that exerts stress of usually equivalent or greater magnitude than the anticipated design stresses. The surcharge will increase pore water pressures initially, but with time the water will drain away and the soil voids will compress. These prefabricated drains are used to shorten pore water travel distance, reducing the preloading time. The intent is to accelerate primary settlement. Porewater will flow laterally to the nearest drain, as opposed to vertical flow to an underlying or overlying drainage layer. The drain flow is a result from the pressures generated in the pore water. Typical PVD installation arrangement is shown in Fig. 2.10.

In the 1920s, a technique for installing sand drains, a PVD predecessor, was patented in the U.S. The California Division of Highways, Materials and Research Department conducted laboratory and field tests on vertical sand drain performance beginning in 1933. Within the decade Walter Kjellman, then Director of the Swedish Geotechnical Institute, developed a prefabricated band-shaped vertical drain made of cardboard core and paper filter jacket which was installed into the ground with mechanical equipment (Holtz, 1987). Cardboard wick drains, and subsequently paper-wrapped plastic drains, were installed outside of the U.S. though the 1970s. A decade after that, entirely plastic PVDs were introduced as a more durable, reliable, and inexpensive option over the sand drains. Because these plastic drains could be installed very quickly as compared to sand drains, by the late 1980s, they largely replaced sand drains (Martin, 2014).

Fig. 2.10: Typical prefabricated vertical drains (PVD) installation arrangements

A center to center spacing for the wick drains of 5 feet was computed by the method outlined in (Hansbo, 1979) based on the requirement that 90% consolidation of the soft sediments occur within the above mentioned 6 months. One foot of fill was placed every 2 days so that there were 85 1-foot increments in 6 months. The use of wick drains indicated that primary consolidation settlement would be accelerated by a factor of about 25, and the secondary compression to not be affected. There were 12 feet of fill placed before installation of the wick drains, and a subsequent 1.5 foot-thick drainage blanket placed on top of the fill.

Surface settlement markers and deep settlement gauges were installed throughout the floodplain to provide settlement data before, during, and after the fill embankment construction, enabling ongoing evaluations of the wick drain performance. Early readings observed an immediate response to the installation of the wick drains. The last reading was taken in July of 1982 and the maximum settlement occurred was 7 feet. It was assumed this represented 90% of the primary consolidation and that total primary consolidation settlement would be 7.8 feet. This was in good agreement with the predicted maximum primary consolidation of 8.3 feet. The installation of wick drains in the soft floodplain soils allowed construction of the fill embankment to proceed on schedule and brought about the desired results, increasing rate of consolidation by a factor of 25 (Geo Engineer, 2021).

## 2.10 Case Studies Concerning PVD

Prefabricated Vertical Drains are typically operated in soft ground, soaked fine grained soils such as silt, lean clay, fat clay, peat with big pore capacity and generally filled with water. The main reason in the execution of PVDs are that it decreases the amount of surcharge required to achieve the desired amount of pre compression in the given time and increases the rate of strength gain due to consolidation of soft soils. In this column, few cases regarding subgrade improvement through installation of PVD and studies are mentioned as follows:

(a) Ground Engineering Using Prefabricated Vertical Drain

(b) Application of Prefabricated Vertical Drains in Soft Clay Improvement

(c) Ground Improvement using Pre-loading with Prefabricated Vertical Drains

### 2.10.1 Ground Engineering Using Prefabricated Vertical Drain

Subgrade refinement by preloading with prefabricated vertical drains (PVDs) is a frequent practice in the field of ground engineering. PVDs advance the consolidation process of soft soils by providing a shorter drainage path for the pore water and enlarge the strength and rigidity of soft soils over time. Conventional PVDs without the use of vacuum, thermal and electro-osmosis techniques was mainly focused in this study. Summary tables, which provide quick and easy access to the latest instruction from various research efforts, have been prepared and discussed.

Soft soils, such as soft estuarine and marine clays, peats, and marshy soils, encountered commonly along deltaic and coastal regions throughout the world, are highly compressible in nature and possess undesirable geotechnical properties. Therefore, structures constructed on these soils face problems of stability and serviceability if measures are not taken to improve them. Although pile foundations may be adopted in some situations to overcome these problems, they may be too expensive, especially for supporting embankments and low-to-medium-rise buildings. In such cases, the soil within the load transfer zone of the structure needs to be improved to make the ground suitable to support the applied load. Ground improvement essentially means increasing the shear strength and reducing the compressibility of the soil. Several soft ground engineering techniques, such as preloading alone, preloading with vertical drains, vacuum consolidation, stone columns, and deep soil mixing, have been used throughout the world.

Among all these techniques, preloading is the simplest and most economical method of inducing settlement so that a structure constructed on improved ground does not settle excessively. Preloading is achieved by placing a temporary surcharge, such as earth fill or sand bags, over soft ground prior to the construction of the proposed structure (Fig. 2.11). The magnitude of the surcharge is generally higher than the pre-consolidation pressure of soft soil so that it is forced to consolidate along the normal consolidation line. The soil gradually gets strength and stiffness over time. However, a major limitation of preloading is the time needed to achieve the required degree of consolidation, which is often so large (typically decades) that no construction project has the luxury of waiting that long.

Provision of vertical drains, as shown in Fig. 2.14, reduces the time required for consolidation of soft soil, and thus the two techniques combined, preloading with vertical drains, is one of the most preferred methods for improvement of soft ground. Preloading with vertical drains accelerates the primary consolidation of soft soil due to two mechanisms.

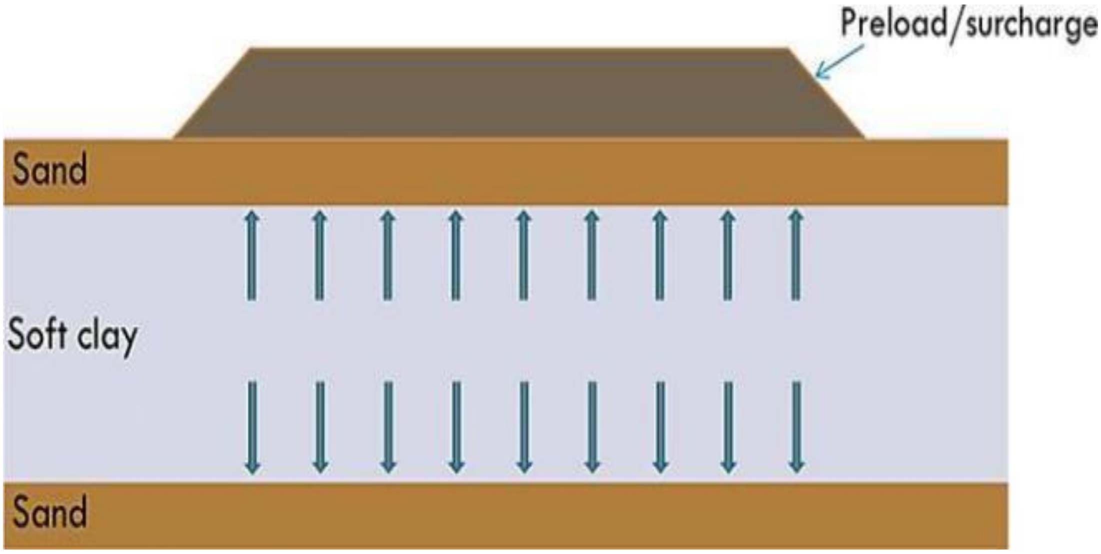



Fig. 2.11: Principles of preloading.

Firstly, the drains are often spaced closely and thus the maximum length of the pore water drainage path reduces to about half of the PVD spacing, which was usually a small fraction of the thickness of the soil layer. Secondly, the direction of flow of pore water changes from vertical (for preloading alone; Figure 2.11) to horizontal (for preloading with vertical drains; Figure 2.12). Most sedimentary deposits exhibit anisotropy with respect to the hydraulic conductivity in such a way that the horizontal component is at least twice that of the vertical component. Therefore, the coefficient of consolidation for flow of pore water in the horizontal direction is higher than that corresponding to flow in the vertical direction. Because of these two effects, the time needed to achieve the required degree of consolidation decreases to a few months instead of decades in the case of preloading alone. Basic principles of working of PVD and installation procedures of PVD are illustrated in Figs. 2.12 and 2.13 respectively.

Some practical considerations concerning the discharge capacity of PVDs and the apparent opening size and cross-plane hydraulic conductivity of the geotextile filter sleeve were discussed in this research. Equations proposed by various researchers for estimation of the required discharge capacity of PVDs were organized. In the end, the review was complemented by two case histories of ground improvement using preloading with PVDs, one in Thailand and the other in China. Both case histories clearly highlight the main advantage of PVDs, which was to quicken the consolidation of soft soils so that construction time can be reduced crucially (Sakleshpur et al., 2018).

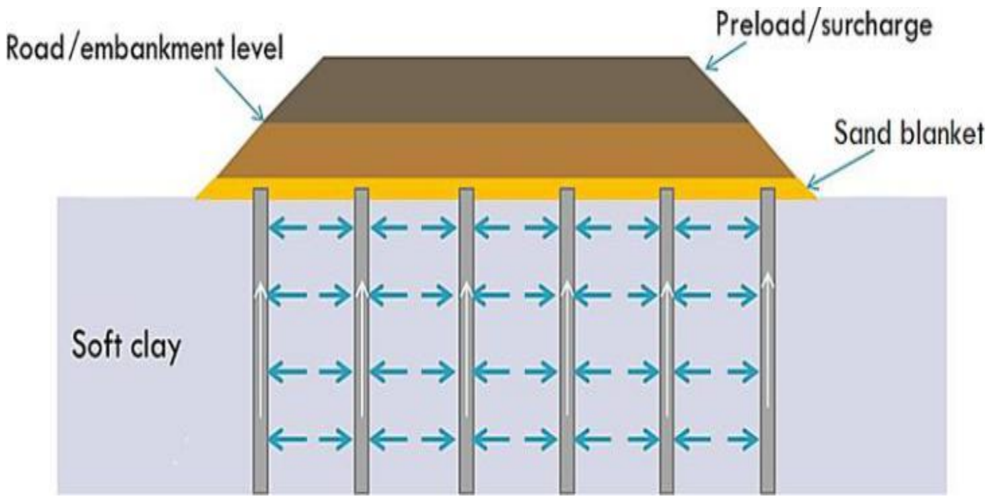



Fig. 2.12: Preloading with vertical drains.

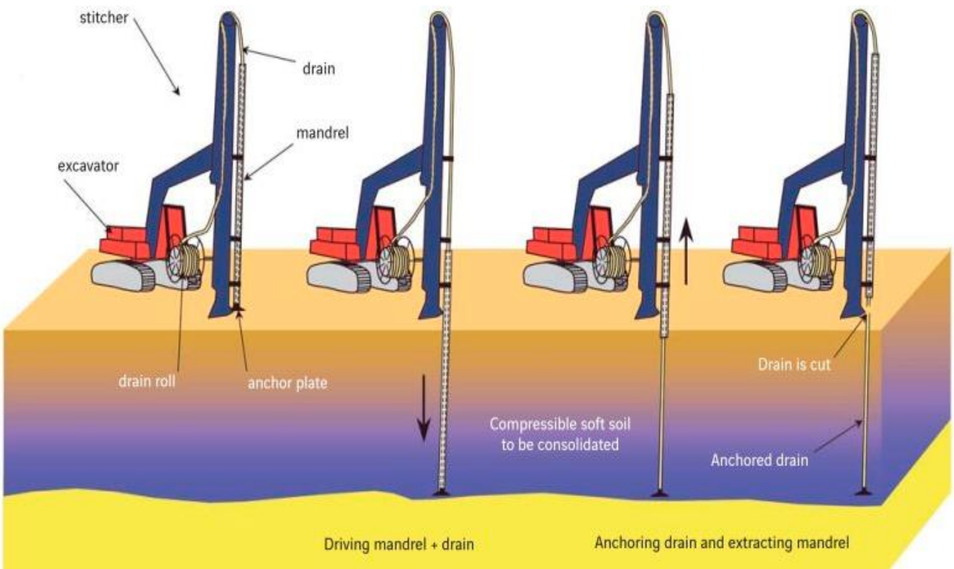



Fig. 2.13: Driving of PVD.

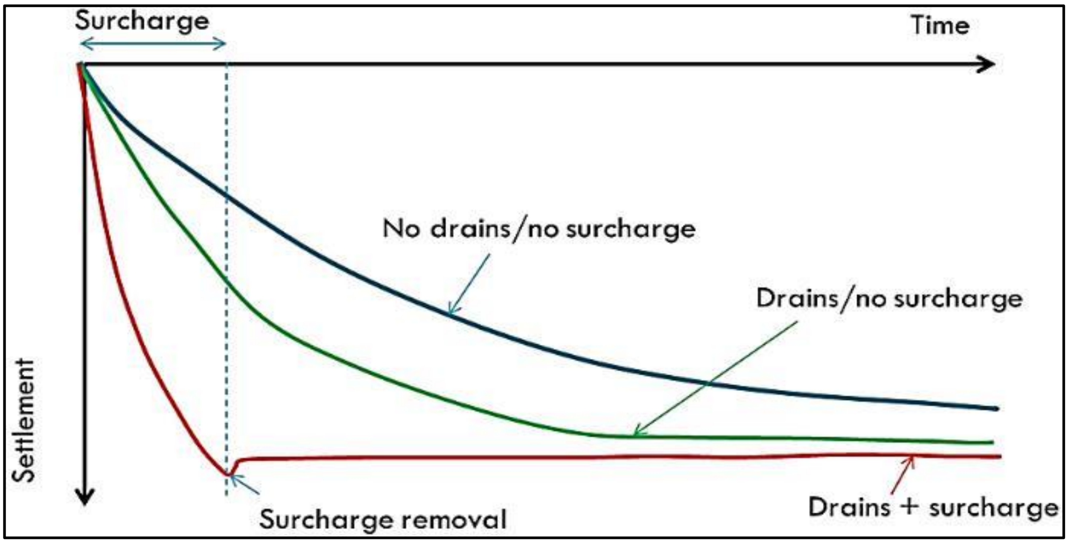



Fig. 2.14: Typical time settlement curves for different combinations of ground improvement.

### 2.10.2 Application of Prefabricated Vertical Drains in Soft Clay Improvement

The behavior of embankments built on soft soil deposits improved by the combination of preloading and prefabricated vertical drains was scrutinized. Different constitutive models were taken and submitted that the soft soil creep model (SSC) was most accurate to model the behavior of the soft clay deposits. A parametric study was accomplished to scrutinize the various factors affecting the required development duration and the expected residual settlement. Tts result obtained the significant influence of the variation of the PVDs length ratio and spacing on both the duration of the consolidation process and the excess settlement, while the PVDs configuration was found to be of minimum significance.

Numerical analyses can be performed to analyze the behavior of embankments built on soft soil deposits improved by the preloading process, accompanied by the use of prefabricated vertical drains. Three-dimensional finite element code PLAXIS 3D 2019 was used in the back analyses of embankments in the Changi East reclamation project in Singapore with and without PVD. Using the soft soil creep model (SSC) for simulating the soft soil deposits resulted in good agreement between the predicted and measured settlement values. On the other hand, the soft soil model (SS) and hardening soil model (HS) were utilized in the numerical modeling resulted in less predicted settlement values compared to the field measurements and the predicted values using the (SSC) model, as they do not take the secondary consolidation into account. However, the difference between the results with or without considering creep was not highly important in the studied case. The creep effect was ignored and the results applying either the soft soil or the hardening soil models were comparable to each other (Hammad et al., 2019).

### 2.10.3 Ground Improvement using Pre-loading with Prefabricated Vertical Drains

Ground improvement using pre-loading with prefabricated vertical drains was undertaken to pre-consolidate the compressible sub-soils, which was followed by field monitoring. It revealed that the classical theories can effectively be used in calculating the consolidation settlement and the time for consolidation. Predicted settlements and the consolidation time matched reasonably with the measured values. The coefficients of consolidation and permeability were taken as those for vertical flow. Predictions with slander diameter equal to two times the equivalent drain diameter provided an upper bound of the consolidation time while prediction without consideration for smear effects provided a lower bound of the consolidation time for the container yard project.

A detailed laboratory investigation was useful for regulating the geotechnical design parameters for scrutiny of consolidation with prefabricated vertical drains. Classical theories of consolidation with the parameters from laboratory tests resulted in estimates of the ground settlements and the consolidation time that were similar to those observed during field monitoring. The effect of drainage congestion can generally be neglected in most prefabricated vertical drain with sufficient discharge capacity. Installation of the vertical drains reduced pre-consolidation time significantly from 1 to 5 years without vertical drain to about 50 days with PVDs (Dhar et al., 2011).

## 2.11 Dynamic Cone Penetration (DCP)

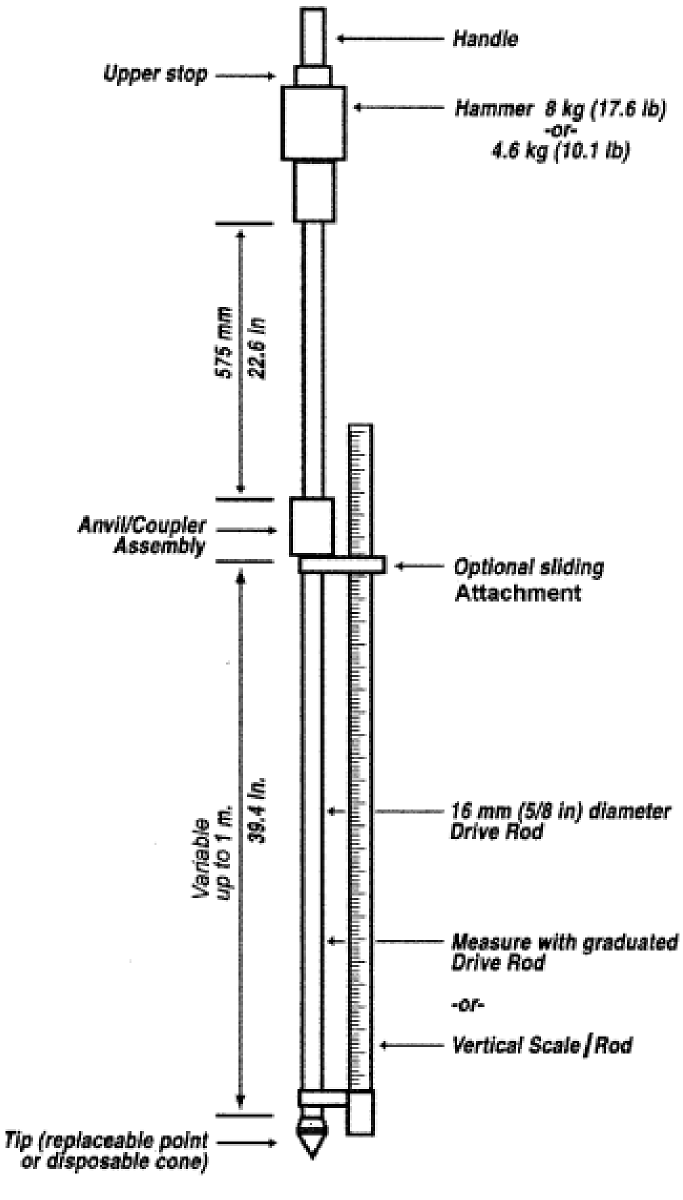
The dynamic cone penetration test (DCPT) was originally developed as an alternative for evaluating the properties of flexible pavement or subgrade soils. The conventional approach to evaluate strength and stiffness properties of asphalt and subgrade soils involves a core sampling procedure and a complicated laboratory testing program such as resilient modulus, Marshall tests and others (Livneh et al., 1994). Due to its economy and simplicity, better understanding of the DCPT results can reduce significantly the effort and cost involved in the evaluation of pavement and subgrade soils.

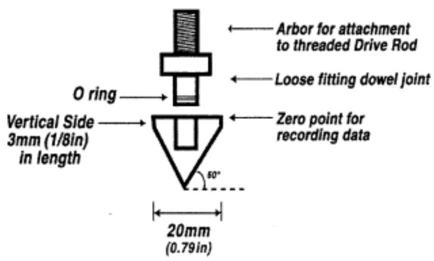
The DCPT is a test carried out to find the resistance value of the cone against the soil that helps to determine different mechanical properties of soil such as strength, bearing capacity and so on. It also assists to monitor the condition of granular layers and subgrade soils in the pavement section over time.

A standard test procedure is suggested by ASTM D-6951-03(2003). This test method covers the measurement of the penetration rate of the dynamic cone penetrometer with an 8-kg hammer (8-kg DCP) through undisturbed soil and/or compacted materials. The penetration rate may be related to in situ strength such as an estimated in situ CBR (California Bearing Ratio). A soil density may be estimated if the soil type and moisture content are known. The DCP described in this test method is typically used for pavement applications.The test method provides for an optional 4.6-kg sliding hammer when the use of the 8-kg sliding mass produces excessive penetration in soft ground conditions.

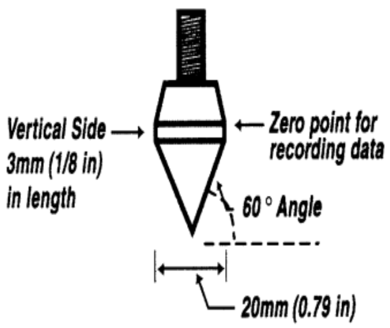
The operator drives the DCP tip into soil by lifting the sliding hammer to the handle then releasing it. The total penetration for a given number of blows is measured and recorded in mm/blow, which is then used to describe stiffness, estimate an in situ CBR strength from an appropriate correlation chart, or other material charcharacteristics.

The 8-kg DCP is shown schematically in Fig. 2.15(a). It consists of the following components: a 15.8-mm (5⁄8-in.) diameter steel drive rod with a replaceable point or disposable cone tip, an 8-kg (17.6-lb) hammer which is dropped a fixed height of 575-mm (22.6-in.), a coupler assembly, and a handle. The tip has an included angle of 60 degrees and a diameter at the base of 20-mm (0.79-in.), Fig. 2.15(b).




(b) Replaceable point tip




(a) Schematic of DCP device

(c) Disposable point tip

Fig 2.15: Schematic diagram of dynamic cone penetration test arrangements (ASTM D 6951-03, 2003)

The apparatus is typically constructed of stainless steel, with the exception of the replacement point tip, which may be constructed from hardened tool steel or a similar material resistant to wear. A disposable cone tip may also be used. The deposable cone tip is held in place with an o-ring, which allows the cone tip to be easily detached when the drive rod is pulled upward after completion of the test. The disposable cone tip is shown schematically in Fig. 2.15(c).

The significance and use of DCP as per ASTM D6951-03 (2003) can be reproduced as follows.

(i) This test method is used to assess in situ strength of undisturbed soil and/or compacted materials. The penetration rate of the 8-kg DCP can be used to estimate in-situ CBR (California Bearing Ratio), to identify strata thickness, shear strength of strata, and other material characteristics.

(ii) Other test methods exist for DCPs with different hammer weights and cone tip sizes, which have correlations that are unique to the instrument.

(iii) The 8-kg DCP is held vertically and therefore is typically used in horizontal construction applications, such as pavements and floor slabs.

(iv) This instrument is typically used to assess material properties down to a depth of 1000-mm (39-in.) below the surface. The penetration depth can be increased using drive rod extensions. However, if drive rod extensions are used, care should be taken when using correlations to estimate other parameters since these correlations are only appropriate for specific DCP configurations. The mass and inertia of the device will change and skin friction along drive rod extensions will occur.

(v) The 8-kg DCP can be used to estimate the strength characteristics of fine- and coarse-grained soils, granular construction materials and weak stabilized or modified materials. The 8-kg DCP cannot be used in highly stabilized or cemented materials or for granular materials containing a large percentage of aggregates greater than 50-mm (2-in.).

(vi) The 8-kg DCP can be used to estimate the strength of in situ materials underlying a bound or highly stabilized layer by first drilling or coring an access hole. The DCP may be used to assess the density of a fairly uniform material by relating density to penetration rate on the same material. In this way undercompacted or “soft spots” can be identified, even though the DCP does not measure density directly.

(vii) A field DCP measurement results in a field or in situ CBR and will not normally correlate with the laboratory or soaked CBR of the same material. The test is thus intended to evaluate the in situ strength of a material under existing field conditions.

### 2.11.1 California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) is performed to determine the strength of soil subgrades and base course materials. Correlations have been established between measurements with Dynamic Cone Penetration (DCP) and California Bearing Ratio (CBR) so that results can be interpreted and compared with CBR specifications for pavement design. This value is given as a percentage as compared to a standardized material where a low average CBR percentage corresponds to weak fill and a high CBR value to strong filling.

## 2.12 Concluding Remarks

Ground improvement has become an important part of geotechnical practice. Different terminologies have been used in the literature for ground improvement, such as soil improvement, soil stabilization, ground treatment, and ground modification. (Han, 2015) Literature review reveals that the experimental designs of subgrade improvement through SCP and PVD were widely varied according to soil parameters and characteristics. SCP and PVD are effective tools of ground improvement especially on soft and loose soil. It is quite evident that lots of research were carried out on SCP and PVD for their performance evaluation on ground improvement. There are well established technics and procedures by which SCP and PVD were applied on various types of ground. But still there remains a research gap of performance of these two ground improvement methods at similar soil condition. As such, it is felt necessary to carry out thorough research for comparing the effectiveness of SCP and PVD at similar area to understand their suitability.

# CHAPTER 3
# TEST PROGRAM AND PROCEDURE

## 3.1 General

The presence of thick soft clay in soil layers makes the soil prone to consolidation settlement after loading. Due to vehicular movement on the road surface, stress is generated and it creates detrimental effects on the soil beneath. Selection of correct ground improvement technique is very important to evaluate the cost of each particular method and expected soil improvement, which are the decisive factors for appropriate method. It was determined to design suitable sand compaction pile (SCP) and prefabricated vertical drain (PVD) for improving the pavement subgrade strength in the study area. Main focus of the thesis was on analyzing the effectiveness SCP and PVD for pavement subgrade improvement in highway construction. This chapter presents description of the study area, test programme, experimental setup and test. The outline of the test programme and research method can be summarized as presented in chart of Fig. 3.1.

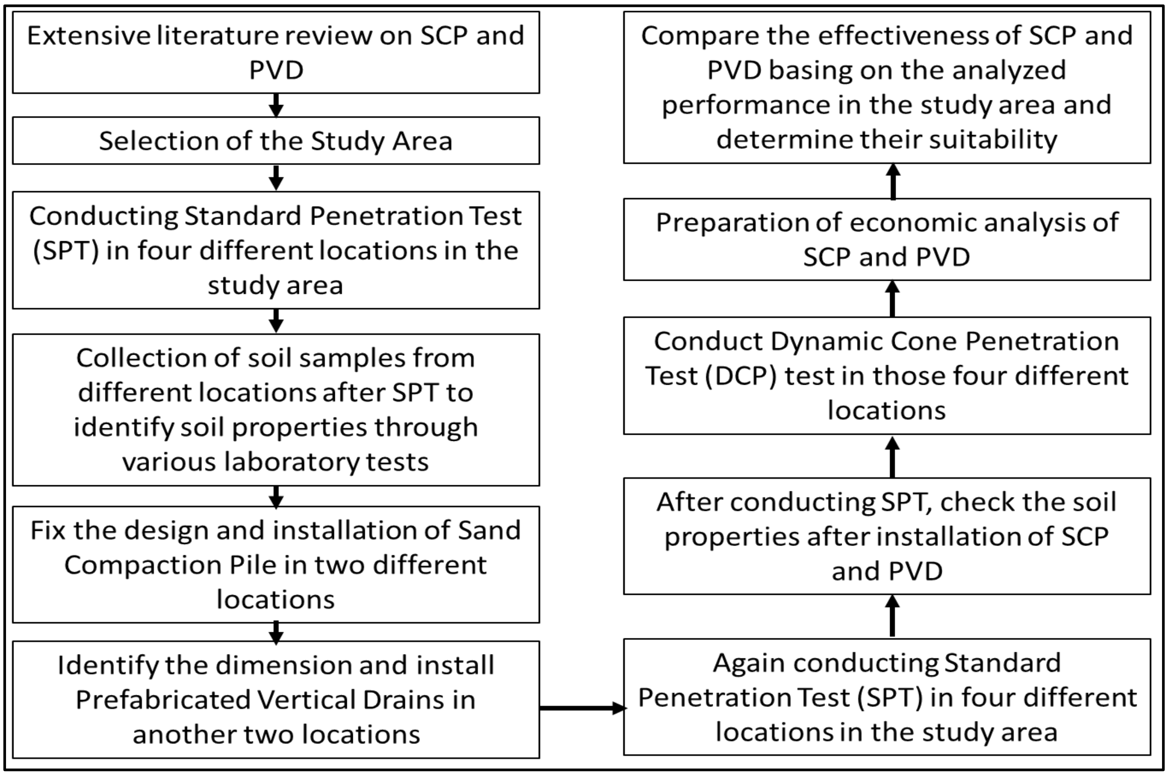



Fig 3.1: Outline of test programme and research method.

### 3.2 Description of the Study Area

Dhaka, one of the fastest growing megacities of the world experiences huge pressure of mass population and traffic congestion. To modernize the life style of city dwellers, numbers of housing projects like Purbachal new city, Jolshiri Abashon, Bashundhara residential area etc are emerging at the eastern fringe of metropolitan area. Two east bound roads Purbachal Expressway and Madani Avenue traversed to connect Dhaka city with its Eastern region. Considering inter connectivity, it had been decided to construct two roads of 3.25 km (Purbachal 300 feet Expressway to Madani Avenue) and 3.79 km (Madani Avenue extension up to Shittalakha River) in this area. But both these future roads situated on the reclaimed area with soft clay soil underground. To increase the interconnectivity in Purbachal areas, these two roads would require substantial subgrade improvement to withstand future designed traffic load. The study area was located at this reclaimed land of Purbachal commonly known as Jolshiri Abashon. The area was basically a low-lying paddy field which was later sand filled for housing project about 10 years ago. The surface of the ground seems to be settled and hard enough. But underlying ground were still comprising very loose and soft clay or sandy layer of soil. The main features of this area are two inter-connecting under construction roads from Jolshiri to Shittalakha River and Jolshiri to Purbachal 300 feet Expressway. Locations of project site are indicated on the picture of google site map as presented in Fig. 3.2.

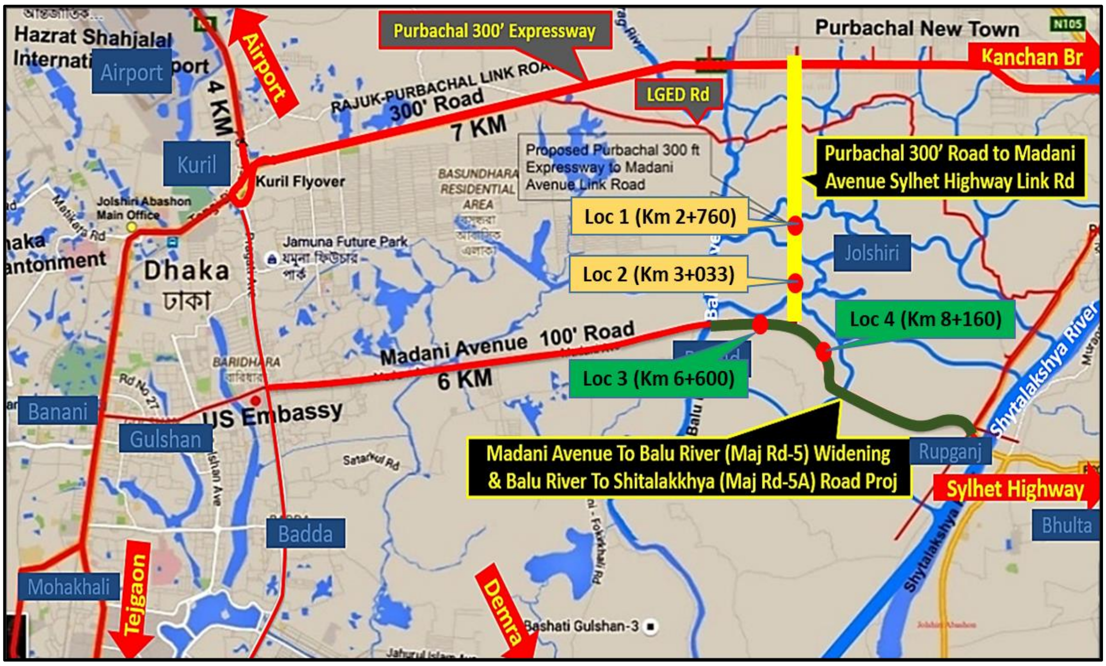



Fig. 3.2: Locations of study area (marked as dots).

### 3.3 Test Programme

In order to determine the effectiveness of subgrade improvement, SCP and PVD had been used to improve subgrade condition. The SCP method was conducted at 3.25 km road from Purbachal 300 feet Expressway to Madani Avenue. Similarly PVD was applied at 3.79 km road from Madani Avenue extension up to Shittalakha River. Before design and installation of SCP and PVD, soil conditions were examined through different experiments. In order to investigate the geotechnical conditions, the results of soil investigations from two subsoil investigation schemes were evaluated: one before subgrade improvement and the other one after subgrade improvement. Four locations were selected to investigate the effectiveness of SCP and PVD in enhancing pavement subgrade as indicated in Table 3.1.

Table 3.1: SCP and PVD test locations

| Location No. | Chainage (km) | Description of location |
|---|---|---|
| 1 | 2+760 | 300 feet expressway to Madani Avenue link road |
| 2 | 3+033 | 300 feet expressway to Madani Avenue link road |
| 3 | 6+600 | Madani Avenue to Shitalakkhya river road |
| 4 | 8+160 | Madani Avenue to Shitalakkhya river road |

Soil boring was done at all four locations. Field tests were conducted at boreholes and laboratory tests were performed on the collected soil samples to assess the subsoil conditions of the sites before going for any improvements. Similar soft subsoil conditions were observed at shallow locations of all the locations. As such, it was decided to install sand compaction pile (SCP) at locations 1 and 2 and PVD at locations 3 and 4 for comparison purposes. The present study was mainly concerned with the effectiveness of sand compaction pile (SCP) and prefabricated vertical drain (PVD) in improving properties of soft soil underlying road pavement.

### 3.4 Field and Laboratory Test to Assess the Site Conditions

Boreholes were drilled at the site at selected four location using wash boring method. The standard penetration and dynamic cone penetration tests were conducted to assess the in-situ properties of subsoil of sites before and after applying the soil improvement measures. The standard penetration test (SPT) was conducted at 1.5 m intervals at the boreholes to determine the stratification and stiffness of the subsoil conditions. Dynamic cone penetration tests were also performed at the proximity locations of SPT for additional checking of the subsoil properties. Disturbed and undisturbed soil samples were collected from boreholes for laboratory testing. Following tests, as mentioned in Table 3.2, were performed in this research work, following the standard procedures. The tests are briefly described sub-sections follow.

Table 3.2: Number of tests performed in the present study

| Description of test/ experiment | | Number of test |
|---|---|---|
| Field Tests | | |
| Standard Penetration Test | | 14 |
| Dynamic Cone Penetration | | 4 |
| Total Field Tests Conducted | | 18 |
| Laboratory Tests | | |
| Grain Size Analysis | Sieve Analysis | 23 |
| | Hydrometer Analysis | 12 |
| Atterberg Limit | Liquid Limit (LL) | 22 |
| | Plastic Limit (PL) | 20 |
| Specific Gravity ($G_s$) | | 26 |
| Total Laboratory Tests Conducted | | 103 |

### 3.4.1 Standard Penetration Test (SPT)

At first, borehole was drilled to the desired sampling depth for this test. The split-spoon sampler that was attached to the drill rod was placed at the testing point. The SPT equipment (ASTM D1586-11) comprised a split tube sampler with a driving head and other attachments were used to recover disturbed soil samples.

The head of the tube was threaded for connection (via a series of drive rods) to a hammer. The device was driven into the ground at the base of the borehole with a 140 lbs (75 kg) hammer dropping vertically as freely as possible through 30" (0.76 m) before hitting the anvil. The SPT test involved driving the split spoon sampler into the bottom of a borehole. The total blows required from a hammer, over the interval 150 to 450 mm (6 to 18 inches) were summed to give the blow count. Typical components of a SPT set up used in this study are shown in the picture of Fig. 3.3. A split spoon sampler with two halves opened showing the soil samples obtained is shown in Fig. 3.4.

Fig 3.3: Picture showing SPT setup used in the study.

Fig 3.4: Picture of split spoon sampler with soil samples.

### 3.4.2 Dynamic Cone Penetration (DCP) Test

The dynamic cone penetrometer (DCP) test was developed by Transport and Road Research Laboratory (TRRL), England. The DCP is an instrument designed for the rapid in-situ measurement of the structural properties of existing road pavements constructed with unbound materials. It is also used for determining the in-situ CBR value of compacted soil sub-grade beneath the existing road pavement. Continuous measurements can be made down to a depth of 800 mm or, when an extension rod is fitted, to a depth of 1200 mm. Where pavement layers have different strengths, the boundaries can be identified and the thickness of the layers determined. Correlations have been established between measurements with DCP and California Bearing Ratio (CBR) so that results can be interpreted and compared with CBR specifications for pavement design. Agreement is generally good over most of the range but differences are apparent at low values of CBR, especially for fine-grained materials. A typical test takes only a few minutes and therefore the instrument provides a very efficient method of obtaining information which would normally require the digging of test pits.

After assembly, the zero reading of the apparatus was recorded. This was done by standing the DCP on a hard surface, such as concrete, checking that it was vertical and then entering the zero reading in the appropriate place on the data sheet. The instrument was held vertical and the weight carefully raised to the handle. It was carefully ensured that the weight was touching the handle, but not lifting the instrument, before it was allowed to drop and that the operator let it fall freely and did not lower it with his hands. However, it is usually easier to take a scale reading after a set number of blows. Therefore, it was necessary to change the number of blows between readings according to the strength of the layer being penetrated. For good quality granular bases readings every 5 or 10 blows are normally satisfactory but for weaker sub-base layers and sub-grades readings every 1 or 2 blows may be appropriate. After completing the test, the DCP was removed by gently tapping the weight upwards against the handle. A picture showing the DCP test arrangements is presented in Fig. 3.5. Relationship between the DCP readings and CBR can be obtained by the following equation: (STP-RHD, 2001)

DCP-CBR percent = $\frac{3700}{(Pen)^{1.5}}$

### 3.4.3 Grain Size Analysis

The soil samples collected Grain Size Analysis is a particular laboratory test conducted in the soil mechanics field. In this research, Grain size Analysis was conducted by considering both Sieve Analysis and Hydrometer test. Sieve Analysis was done following the standard tests procedure of AASHTO T27 and hydrometer analysis was performed by following the standard procedure of AASHTO T88 which is shown in Fig. 3.6.

Fig. 3.5: Picture showing dynamic cone penetration test arrangements.

Fig. 3.6: Hydrometer test.

### 3.4.4 Atterberg Limit

The Atterberg limit brings up to the liquid limit and plastic limit of soil. These two limits are operated for soil identification, classification, and strength correlations. In this study, Liquid limit and Plastic limit were carried out by following the Standard Procedure of AASHTO T89 and AASHTO T90. A picture showing the Atterberg limit Test is presented in Fig. 3.7.

Fig. 3.7: Atterberg limit test.

### 3.4.5 Specific Gravity

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. This test was also conducted in this research to find out the specific gravity following the Standard Procedure of ASTM D854-02.

## 3.5 Sand Compaction Pile (SCP)

Sand compaction piles are one of the potential methods for improving ground stability. It prevents liquefaction, reduces settlement and performs similar applications.

The granular piles were constructed here by cased borehole type of SCP. This method is frequently used to construct columnar inclusions through weak soils in developed areas because of the problems associated with the acquisition, retention and disposal of significant amount of water. The dry technique is suited for partially saturated soils that can stand unsupported, especially those that will density as a result of lateral vibration.

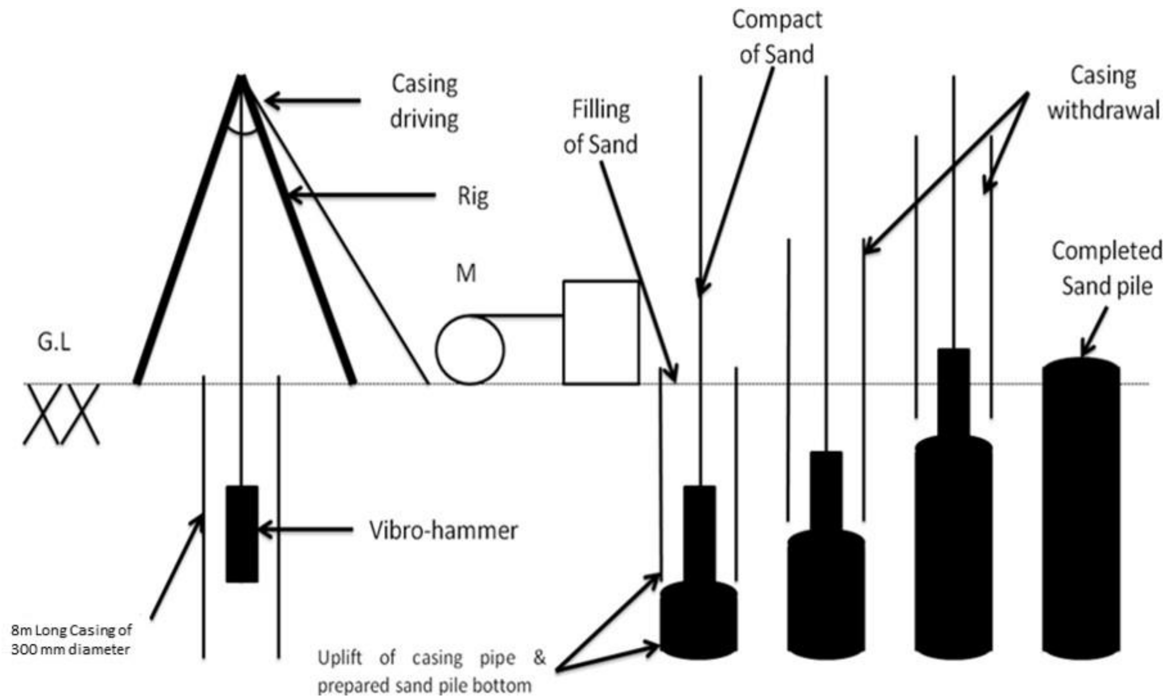



Fig. 3.8: Schematic diagram of installation procedures of granular piles.

A 1500 rpm traditional rig machine and a two-end open casing pipe 8 mm thickness and 300 mm in diameter and 8 m long with a hammer of weight 1000 kg. The hammer was 250 mm in diameter and 3.00 m long. The construction sequences are described in the following statements. The schematic diagram is shown in Fig. 3.8.

A two-end open casing pipe, 300 mm in diameter and 8 m long was placed vertically at the designed point on the natural ground surface for sand pile construction. The casing Pipe was then inserted vertically into the ground about 300 mm to 450 mm depth at its own weight just by applying some pressure manually.

At first a plug is made by the designated sand up to 750 mm of casing pipe at bottom level. The hammer 250 mm in diameter and 3.0 m long, weighting 1000 kg was placed inside the casing pipe. The hammer displaced the soil from beneath the casing pipe hence the casing pipe was driven by its own weight till reached the designated position (depth) into the ground. Here one casing pipe of 7 m long was driven inside the ground.

After reaching the designated depth, the sand plug is broken by providing excess energy then the hammer is withdrawn from the casing pipe. Casing pipe was then lifted up by about 1m from its original bottom position. The designated granular materials were poured into the hole about 1m layer thickness measured from the bottom. The poured granular materials were then densified by hammer till the required compactness achieved. Casing pipe was then withdrawn from inside the ground that left the bottom portion of hole unsupported and the top portion supported by the casing pipe. It was observed that the bottom portion of the hole standing safely without any lateral support. Then hole was poured by the selected granular materials in layers and hence 10 to 15 drops compacted each layer was densified by hammer till the designated compactness was reached. After the top of granular piles were reached about 1.0 m to 1.5 m below the ground surface the casing pipe was withdrawn and left the remaining hole unsupported. The process was continued until the granular piles were constructed up to the ground level. Time is also a very important factor for construction of SCP. Though a single SCP can be constructed within few hours but it would normally gain required strength after one month of installation of SCP (Hoque and Alamgir, 2014). Figure 3.9 represents the installation of sand compaction pile in the study area.

Fig. 3.9: Installation of sand compaction pile in the study area.

### 3.6 Prefabricated Vertical Drains

Prefabricated vertical drains (PVDs) are commonly used when an accelerated rate of consolidation of a clay layer is desired. The installation of PVDs is achieved by pushing a steel mandrel into the clay layer to the desired depth, and this result in significant disturbance of the clay layer surrounding the drain, resulting in a "smear" zone. As a consequence, horizontal permeability of the clay stratum gets significantly reduced. The installation of vertical drains into a relatively thick stratum of clay before the application of a load increases the rate of consolidation of the clay by shortening the drainage path. In addition, in non-uniform soils the horizontal permeability may be greater than the vertical permeability; this anisotropy confers an additional advantage on the use of drains.

### 3.6.1 Depth of Installation

Drains are not likely to accelerate consolidation if induced effective stress is not greater than the pre-consolidation stress. The optimum depth of the PV drains lies within the pre-consolidation stress margin as the stress from the surcharge diminishes with depth. However, if there is a pervious soil layer below the pre-consolidation margin, the PV drain should be extended into that soil layer. This will aid in assuring the discharge of the water.

### 3.6.2 Spacing and Width of Installation

The spacing and pattern of the drains are now fairly well standardized. In most situations, triangular or square grid spacing of 1-4 m is used with 1.5-2.5 m being most commonly adopted. Drain spacing may vary according to ground requirement. Soil strata are not defined as entirely uniform layers, therefore there may not be equal volumes of water to be drained. If some portions of a layer have a greater amount of drainage, the soil will settle to fill those voids. This leads to differential settlements and could prolong the consolidation time. To help avoid this issue, PVD should be distributed across the entire footprint of an embankment and a small distance beyond. It is advised to place the outermost rows of drains between one third and one half of the proposed embankment's height beyond the embankment. However, when designing the PV drain’s layout, homogeneous soil can be assumed for simplicity. With the use of PVD, degree of consolidation can be reached in a very short amount of time. As a result, total settlement can be reduced and further filled. Then the desired construction can be started on that site (Geo Engineer, 2021).

### 3.6.3 PVD Installation

For chronological installation of PVD, number of sections were defined and each section was indicated by a letter code. For each section a predefined installation depth for the drains was identified. A section was set out based on the information from the detail survey as per ground condition. The four corners of each section were marked with pegs. The grid, which defined the actual positions of the drains, had square spacing as per design. The individual drain position was marked by pulling a nylon string, marked with the required drain spacing along the alignment of the drain positions. The anchor plate was used to mark the position of the drain.

The drilling rig was then aligned with the leader on top the drain location. The drain was wrapped around the fixture on the anchor plate (dimensions: 140x80x1 mm) and the folded end of the wrapped drain was pulled back into the mandrel, until the plate rested against the base of the mandrel. The hydraulic motors of the machine pushed the mandrel to the design depth. The drains were installed to the depth as defined above. This actual depth may vary due to irregularities in the layer profile; which had to be taken into account and actual depth need to be finalized.

On reaching the depth of the drains, the operation was reversed and the mandrel was withdrawn from the PVD. The anchor plate locked itself at the driven depth such that the drain was fixed as the mandrel rises. The drain roll was mounted on the side of leader allowing the drain to be fed into the mandrel through a series of rollers, which prevent damage and minimize friction. Once the mandrel cleared the ground surface, the drain was cut off approximately 250 mm above ground level and the drain was installed. The leader was aligned onto the next drain position and the above procedure was repeated. Installation of prefabricated vertical drains in the study area is presented in Fig. 3.10.


Fig. 3.10: Installation of prefabricated vertical drains in the study area.

# CHAPTER 4
# TEST RESULTS AND DATA ANALYSIS

## 4.1 General

The study was mainly concerned with investigating the effectiveness of sand compaction pile (SCP) and polyvinyl vertical drain (PVD) in improving pavement subgrade. The SPT test were conducted before execution of subgrade improvement and accordingly models of SCP and PVD were designed. After implementation of subgrade improvement by these two methods again SPT test were conducted in order to compare the increased strength of subgrade. Correspondingly, DCP test were also conducted at shallow depth after execution of SCP and PVD. All these data were recorded and they were analyzed, and the results are discussed in the following sections.

## 4.2 Subgrade Investigation Schemes

In order to investigate the geotechnical conditions, the results of soil investigations from two subsoil investigation schemes were evaluated: one before subgrade improvement and the other one after subgrade improvement. There were 10 boreholes done in total, out of which 6 boreholes on the Purbachal 300 feet Expressway to Madani Avenue Link Road (Location 1 and Location 2) and 4 boreholes on the Madani Avenue extension to Shitalakshya river Road (Location 3 and Location 4). The maximum depth of boring was 30 meter before subgrade improvement and 10 meter after subgrade improvement.

As mentioned earlier, subgrade improvement investigation through SCP and PVD in four different locations were executed. Subgrade improvement was designed by SCP at location 1, location 2 and designed by PVD at location 3, location 4. These boreholes were done at four different locations with an approximate interval. Before conducting subgrade improvement, SPT-N values at different depths suggest that the underlying soil at the site consists of a soft clayey layer having varying properties at different sections. In this research work, following tests were performed which are presented in Table 4.1. Details were found in AASHTO T27, T88, T89, T90, and ASTM D854-02.

Table 4.1: List of tests conducted

| Description of test/ experiment | | Location 1 | Location 2 | Location 3 | Location 4 | Total |
|---|---|---|---|---|---|---|
| Standard Penetration Test | | 4 | 4 | 3 | 3 | 14 |
| Dynamic Cone Penetration | | 1 | 1 | 1 | 1 | 4 |
| Grain Size Analysis | Sieve Analysis | 7 | 4 | 6 | 6 | 23 |
| | Hydrometer Analysis | 3 | 3 | 3 | 3 | 12 |
| Atterberg Limit | Liquid Limit (LL) | 5 | 5 | 6 | 6 | 22 |
| | Plastic Limit (PL) | 5 | 5 | 5 | 5 | 20 |
| Specific Gravity ($G_s$) | | 6 | 8 | 7 | 5 | 26 |

### 4.2.1 Soil Investigation Data and Soil Properties at Location 1

There exists a thick (around 7.5 meter) soft clay layer underlying a loose silty sand layer in the road widening area in location 1. Thickness of the top loose silty sand layer is up to 7.5 meter. Water Table was found at 2 meter below existing ground level. Table 4.2 shows typical soil profile data of location 1. The soft soil layers, in general, were underlain by a medium dense to dense sandy layer. The soil properties are presented in Table 4.3. It is well understood that the soft layer needs to be improved before going for construction of highway, using a suitable method of ground improvement like SCP or PVD. The method of subgrade improvement is in fact dependent on the physical, index and engineering properties of soil encountered. Reports of borehole test and soil properties at location 1 are presented in Section Anx.1 of ANNEXURE.

Table 4.2: Borehole test data at location 1

| Depth (m) | Soil strata | SPT-N value |
|---|---|---|
| 0.0 | Gray, very loose to loose silty sand | 0 |
| 1.5 | | 4 |
| 3.0 | | 4 |
| 4.5 | | 3 |
| 6.0 | | 5 |
| 7.5 | Gray, medium stiff fat clay | 5 |
| 9.0 | Gray, soft to stiff lean clay | 4 |

Table 4.3: Soil properties at location 1 (before SCP installation)

| Soil layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
|---|---|---|---|---|---|
| Gray, very loose to loose Silty Sand<br>USCS classification: SM | 0-7.5 | 2.67 | 50 | 22 | 28 |
| Gray, medium stiff Fat Clay<br>USCS classification: CH | 7.5-9 | 2.74 | 46 | 23 | 23 |
| Gray, soft to stiff Lean Clay<br>USCS classification: CL | 9-10 | 2.74 | 48 | 23 | 25 |

### 4.2.2 Soil Investigation Data and Soil Properties at Location 2

Table 4.4 shows soil profile data of location 2 and soil properties are indicated in Table 4.5. It is seen that there exists a thick (approximately 6 meter) soft clay layer underlying a loose silty sand layer in the road widening area. Thickness of the top very loose silty sand layer is up to 6 meter. Water Table was noted 7 meter below existing ground level.

Table 4.4: Borehole test data at location 2

| Depth (m) | Soil strata | SPT-N value |
|---|---|---|
| 0.0 | Gray, very loose to loose Silty Sand | 0 |
| 1.5 | | 7 |
| 3.0 | | 4 |
| 4.5 | | 2 |
| 6.0 | Gray, medium stiff Fat Clay | 1 |
| 7.5 | | 2 |
| 9.0 | Gray, soft to stiff Lean Clay | 1 |

Table 4.5: Soil properties at location 2 (before SCP installation)

| Soil layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
|---|---|---|---|---|---|
| Gray, very loose to loose Silty Sand<br>USCS Classification: SM | 0-6 | 2.66 | 78 | 24 | 54 |
| Gray, medium stiff Fat Clay<br>USCS Classification: CH | 6-10 | 2.72 | 62 | 26 | 36 |

### 4.2.3 Soil Investigation Data and Soil Properties at Location 3

Tables 4.6 and 4.7 show typical soil profile and soil properties respectively of location 3. It is seen that there exists a thick (around 6 meter) soft clay layer underlying a loose silt layer in the road widening area. Thickness of the top very loose silt and fat clay is up to 6 meter. Water Table was found 1.5 meter below existing ground level. Reports of Borehole test and soil properties at location 3 are given in Section Anx.3 of ANNEXURE.

Table 4.6: Borehole test data at location 3

| Depth (m) | Soil strata | SPT-N value |
|---|---|---|
| 1.5 | Gray, very loose to loose Silty Sand | 3 |
| 3.0 | Gray, loose Silt | 4 |
| 4.5 | Yellowish Gray, soft Fat Clay | 4 |
| 6.0 | Yellowish gray, soft to hard Lean Clay | 3 |
| 7.5 | | 3 |
| 9.0 | | 4 |

Table 4.7: Soil properties at location 3 (before PVD installation)

| Soil layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
|---|---|---|---|---|---|
| Gray, very loose to loose Silty Sand<br>USCS Classification: SM | 0-3.0 | 2.66 | 35 | 28 | 07 |
| Gray, loose Silt<br>USCS Classification: ML | 3.0-4.5 | 2.66 | 35 | 28 | 07 |
| Yellowish gray, soft Fat Clay<br>USCS Classification: CH | 4.5-6.0 | 2.66 | 35 | 28 | 07 |
| Yellowish gray, soft to hard Lean Clay<br>USCS Classification: CL | 6.0-10.0 | 2.75 | 49 | 18 | 31 |

### 4.2.4 Soil Investigation Data and Soil Properties at Location 4

Tables 4.8 and 4.9 show soil profile and properties respectively of location 4. It is seen that there exists a thick (around 10 meter) soft clay layer underlying a loose silt layer in the road widening area. Thickness of the top very loose silt and fat clay is up to 6 meter. Water Table was found 6 meter below existing ground level.

Table 4.8: Borehole test data at location 4

| Depth (m) | Soil strata | SPT-N value |
|---|---|---|
| 1.5 | Gray, very loose to loose Silt | 2 |
| 3 | | 4 |
| 4.5 | | 5 |
| 6 | Gray, loose Silty Sand | 4 |
| 7.5 | Gray, very soft to soft Fat Clay | 2 |
| 9 | | 4 |

Table 4.9: Soil properties at location 4 (before PVD installation)

| Soil layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
|---|---|---|---|---|---|
| Gray, very loose to loose Silt<br>USCS Classification: ML | 0-6 | 2.68 | NP | NP | NP |
| Gray, loose Silty Sand<br>USCS Classification: SM | 6-7.5 | 2.68 | NP | NP | NP |
| Gray, very soft to soft Fat Clay<br>USCS Classification: CH | 7.5-10 | 2.74 | 71 | 30 | 41 |

## 4.3 Necessity of Ground Treatment

As the subgrade of the road consists of loose sand and soft clay, it has the potential of differential settlement due to the construction of the road as well as heavy traffic load in near future. Besides, excess pore water pressure would be generated in the underlying soft clay layer due to any form of loads (surcharge of the soil fill, pavement, construction equipment, vehicles etc.) during road construction. In most places, Liquid Limit (LL) and Plasticity Index (PI) of the existing soft clay are about 35 to 65 and 15 to 30 respectively, which indicate the coefficient of consolidation is low (Munthe et al., 2018). Therefore, a long-term settlement would be associated due to the excess pore pressure generation as mentioned above. For the dissipation of the excess pore water during the construction period ground treatment needed. If ground treatment is not done, there could be crack in the pavement for any differential settlement in the subgrade. In addition, ground subsidence may take place in any section of the road after construction. Sand compaction pile (SCP) and prefabricated vertical drain (PVD) are effective ground treatment methods for such soft clay and loose sandy ground. Therefore, SCPs and PVDs are suitably designed according to examined soil condition in two locations.

## 4.4 Ground Improvement after Installation of SCP

The ground improvement after installation of SCP was assessed by performing SPT at locations of center of SCP and in between the SCP holes after one month of construction. The data obtained for both the locations (Location 1 and 2) of SCP are presented in Tables 4.10 and 4.11.

**Soil profile after installation of SCP at location 1**

The data of SCP location 1 were analyzed and the results are presented in Tables 4.12, 4.14 and 4.15. The results are also presented in Fig. 4.2. Reports of Borehole data and soil properties at location 1 are listed in Section Anx.2 of ANNEXURE.

Table 4.10: Borehole data in the center of sand column at location 1

| Depth | | Blow count | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 1 | 1.45 | 6 | 8 | 11 | 19 |
| 2 | 2.45 | 6 | 7 | 10 | 17 |
| 3 | 3.45 | 9 | 11 | 11 | 22 |
| 4 | 4.45 | 8 | 9 | 12 | 21 |
| 5 | 5.45 | 7 | 10 | 11 | 21 |
| 6 | 6.45 | 9 | 11 | 12 | 23 |
| 7 | 7.45 | 8 | 11 | 14 | 25 |
| 8 | 8.45 | 4 | 10 | 13 | 23 |
| 9 | 9.45 | 4 | 4 | 5 | 9 |
| 10 | 10.45 | 3 | 3 | 5 | 8 |

Table 4.11: Borehole results at the center of three SCPs at location 1

| Depth | | Blow count | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 1 | 1.45 | 7 | 7 | 11 | 18 |
| 2 | 2.45 | 8 | 9 | 9 | 18 |
| 3 | 3.45 | 8 | 9 | 12 | 21 |

| Depth | | Blow count | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 4 | 4.45 | 7 | 8 | 11 | 19 |
| 5 | 5.45 | 9 | 8 | 9 | 17 |
| 6 | 6.45 | 8 | 8 | 1 | 18 |
| 7 | 7.45 | 9 | 9 | 10 | 19 |
| 8 | 8.45 | 8 | 8 | 8 | 16 |
| 9 | 9.45 | 4 | 3 | 5 | 8 |
| 10 | 10.45 | 3 | 4 | 4 | 8 |

Then columns were installed, the area replacement ratio is defined as,

$$a_s = C \times (d_c \div s)^2 \tag{4.1}$$

Where, $a_s$ = area replacement ratio

$d_c$ = diameter of the column

$s$ = center to center spacing of columns in a square or equilateral triangular pattern

$C$ = constant (0.907 for an equilateral triangular pattern)

Kensetsu Kikai Chosa (Han, 2015) suggested the average weighted SPT-N value including the sand compaction column and the surrounding soil as follows:

$$N_{eq} = a_s \times N_2 + (1 - a_s) \times N_1 \tag{4.2}$$

Where,

$N_{eq}$ = average weighted (equivalent) SPT N value

$N_1$ = SPT N value in the surrounding soil

$N_2$ = SPT N value in the sand compaction column.

For 1st Location: SPT-N value at 3-meter depth;

SPT-N Value midway between sand compaction columns, $N_1$ = 21

SPT-N Value in the center of sand compaction columns, $N_2$ = 22

Area Replacement Ratio, $a_s = C \times (d_c \div s)^2 = 0.907 \times (0.3 \div 1.4)^2 = 0.04165$

Now for example at 3 meter depth, the average weighted SPT-N value including the sand compaction column and the surrounding soil,

$$N_{eq} = a_s \times N_2 + (1 - a_s) \times N_1 = 0.04165 \times 22 + (1 - 0.04165) \times 21 = 21$$

Table 4.12: Average weighted SPT-N value for location 1 (up to 10 meter)

| Depth (m) | SPT-N values midway between SCP columns ($N_1$) | SPT-N values at the center of SCP columns ($N_2$) | Area replacement ratio, $a_s = C\times(d_c/s)^2$ | Average weighted SPT-N value, $N_{eq} = a_s\times N_2 + (1-a_s)\times N_1$ |
|---|---|---|---|---|
| 1 | 18 | 19 | 0.04165 | 18 |
| 2 | 18 | 17 | | 18 |
| 3 | 21 | 22 | | 21 |
| 4 | 19 | 21 | | 19 |
| 5 | 17 | 21 | | 17 |
| 6 | 18 | 23 | | 18 |
| 7 | 19 | 25 | | 19 |
| 8 | 16 | 23 | | 16 |
| 9 | 8 | 9 | | 8 |
| 10 | 8 | 8 | | 8 |

Grain Size Analysis: Sieve analysis tests were conducted according to ASTM D2487. For oven-dry materials, sieving were carried out for particles retained on a 0.075 mm sieve. In sieve analysis, the mass of soil retained on each sieve is determined and expressed as a percentage of the total mass of the sample. The particle size is plotted on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the distribution plot. Hydrometer analysis was conducted for fine materials. This test is based on the principle of sedimentation of soil grains in water. When a soil specimen is dispersed in water, the particles settle at different velocities, depending on their shape, size, and weight. For simplicity, it is assumed that soil particles are spheres and the velocity of soil particles can be express by Stokes' law. Table 4.13 and Fig. 4.1 shows a sample of sieve analysis test report at location 1.

Table. 4.13: Sieve Analysis at the center of the SCP (4 m depth), location 1

| SIEVE ANALYSIS (Method-AASHTO T27) | | | | | |
|---|---|---|---|---|---|
| Weight of total dry sample 104.1 gm | | | | | |
| Material finer No. 200 Sieve (By Wash Pass Method- AASHTO T 11) Depth (m) | | Sieve Size (mm) | Percent Passing (%) | Size Fraction | (%) |
| Dry wt of Sample | 104.1 | 9.5 (3/8”) | 100.0 | Gravel | 0.0% |
| | | 4.75 (#4) | 98.0 | Coarse Sand | 7.7% |
| Dry wt of Sample After Washing | 99.9 | 2.36 (#8) | 94.4 | Medium Sand | 49.8% |
| | | 1.18 (#16) | 85.4 | Fine Sand | 36.5% |
| | | 0.6 (#30) | 62.4 | Silt | 4% |
| | | 0.3 (#50) | 25.0 | Clay | |
| % of material finer than 0.075 mm | 4.0 | 0.15 (#100) | 10.8 | Colloid | |
| | | 0.075 (#200) | 4.0 | For Materials finer than 0.075 mm, Hydrometer test was carried out for soil (Silt + Clay + Colloid) classification | |

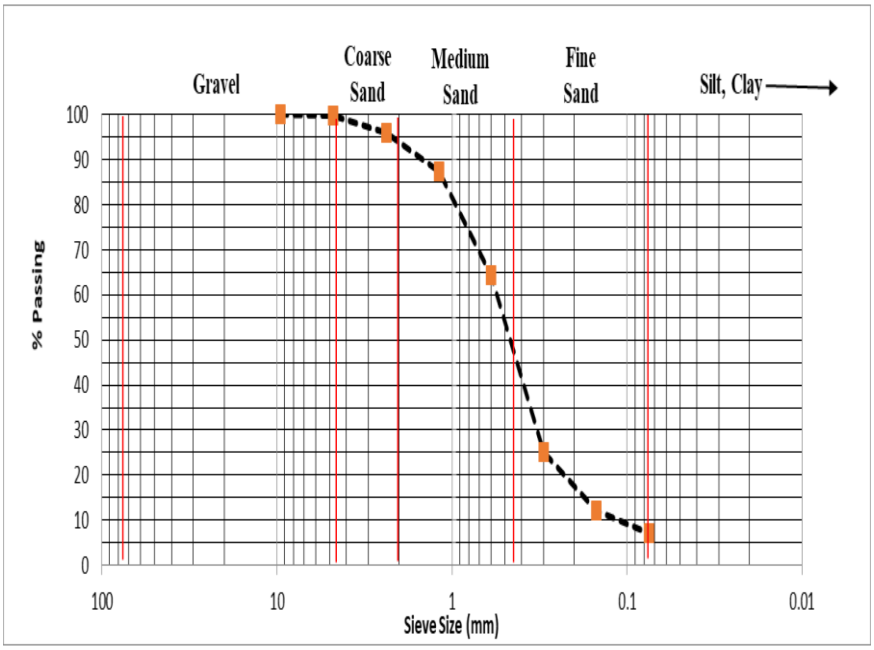



Fig. 4.1: Graphical presentation of sieve analysis after installation of SCP in 1st location.

Table 4.14: Soil properties after installation of SCP at location 1

<table>
<tr><th colspan="6">Location 1 (in between three sand column)</th></tr>
<tr><th>Soil layer</th><th>Depth (m)</th><th>Specific gravity</th><th>Liquid limit</th><th>Plastic limit</th><th>Plasticity index</th></tr>
<tr><td rowspan="2">Silty Sand<br>USCS Classification: SM</td><td>2</td><td>2.56</td><td>NP</td><td>NP</td><td>NP</td></tr>
<tr><td>4</td><td>2.37</td><td>NP</td><td>NP</td><td>NP</td></tr>
<tr><td>Lean Clay<br>USCS Classification: CL</td><td>9</td><td>2.31</td><td>33</td><td>21</td><td>12</td></tr>
<tr><td>Silt<br>USCS Classification: ML</td><td>10</td><td>2.40</td><td>38</td><td>25</td><td>13</td></tr>
</table>

Table 4.15: Comparison between SPT-N values before and after execution of SCP at location 1 (From Table 4.2 and Table 4.12)

<table>
<tr><th>Depth</th><th>SPT-N (before SCP)</th><th>Average SPT-N (value before SCP)</th><th>SPT-N (after SCP)</th><th>Average SPT-N (value after SCP)</th></tr>
<tr><td>1</td><td>-</td><td rowspan="13">4.20 ≈ 4</td><td>18</td><td rowspan="13">18.25 ≈ 18</td></tr>
<tr><td>1.5</td><td>4</td><td>-</td></tr>
<tr><td>2</td><td>-</td><td>18</td></tr>
<tr><td>3</td><td>4</td><td>21</td></tr>
<tr><td>4</td><td>-</td><td>19</td></tr>
<tr><td>4.5</td><td>3</td><td>-</td></tr>
<tr><td>5</td><td>-</td><td>17</td></tr>
<tr><td>6</td><td>5</td><td>18</td></tr>
<tr><td>7</td><td>-</td><td>19</td></tr>
<tr><td>7.5</td><td>5</td><td>-</td></tr>
<tr><td>8</td><td>-</td><td>16</td></tr>
<tr><td>9</td><td>4</td><td>8</td></tr>
<tr><td>10</td><td>-</td><td>8</td></tr>
</table>

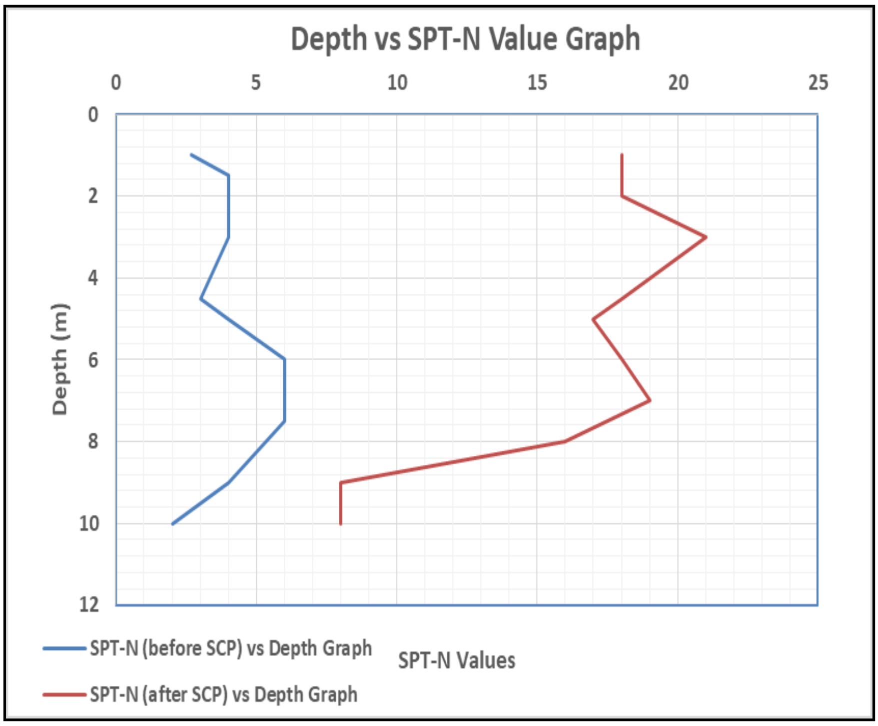



Fig. 4.2: Increasing SPT-N values after installation of SCP in 1st location.

From Fig 4.2, it can be seen that at 3 meter depth before installation of sand column the SPT-N value was very low and the value was 4. The soil condition was not suitable for highway construction and soils were mostly loose and very soft fat clay. Average SPT-N value was identified around 4 up to 10 meter depth. After installation of sand column, subgrade conditions improved and become silty fine sand. SPT-N values were also increased after execution of SCP and average SPT-N value was found 18 up to 10 meter depth.

**Soil Profile after Installation of SCP at Location 2**

The data of SCP location 2 are analyzed and the results are presented in Tables 4.16 through 4.20. The results are also presented in Fig. 4.3.

Table 4.16: Borehole test results at the center of sand column at location 2

| Depth | | SPT-N value | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 1 | 1.45 | 9 | 10 | 13 | 23 |
| 2 | 2.45 | 8 | 9 | 11 | 20 |
| 3 | 3.45 | 9 | 9 | 9 | 18 |
| 4 | 4.45 | 10 | 10 | 11 | 21 |

| | | | | | |
|---|---|---|---|---|---|
| 5 | 5.45 | 11 | 11 | 12 | 23 |
| 6 | 6.45 | 10 | 11 | 14 | 25 |
| 7 | 7.45 | 12 | 10 | 13 | 23 |
| 8 | 8.45 | 11 | 10 | 11 | 21 |
| 9 | 9.45 | 3 | 4 | 5 | 9 |
| 10 | 10.45 | 4 | 3 | 3 | 6 |

Table 4.17: Borehole test results at the center of three SCPs at location 2

| Depth | | Blow count | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 1 | 1.45 | 8 | 9 | 9 | 18 |
| 2 | 2.45 | 9 | 7 | 9 | 16 |
| 3 | 3.45 | 7 | 7 | 10 | 17 |
| 4 | 4.45 | 6 | 9 | 10 | 19 |
| 5 | 5.45 | 6 | 8 | 9 | 17 |
| 6 | 6.45 | 8 | 9 | 10 | 19 |
| 7 | 7.45 | 7 | 10 | 11 | 21 |
| 8 | 8.45 | 5 | 7 | 10 | 17 |
| 9 | 9.45 | 3 | 3 | 4 | 7 |
| 10 | 10.45 | 4 | 3 | 5 | 8 |

SPT-N Value midway between sand compaction columns, $N_1$ = 17

SPT-N Value in the center of sand compaction columns, $N_2$ = 18

Area Replacement Ratio, $a_s = C \times (d_c \div s)^2 = 0.907 \times (0.3 \div 1.4)^2 = 0.04165$

At 3 meter depth, the average weighted SPT-N value including the sand compaction column and the surrounding soil,

$$N_{eq} = a_s \times N_2 + (1 - a_s) \times N_1 = 0.04165 \times 18 + (1 - 0.04165) \times 17 = 17$$

Table 4.18: Average weighted SPT-N value for Location 2 (up to 10 meter)

| Depth (m) | SPT-N values midway between SCP ($N_1$) | SPT-N values at the center of SCP ($N_2$) | Area replacement ratio, $a_s = C \times (d_c/s)^2$ | Weighted average SPT-N value, $N_{eq} = a_s \times N_2 + (1-a_s) \times N_1$ |
|---|---|---|---|---|
| 1 | 18 | 23 | 0.04165 | 18 |
| 2 | 16 | 20 | | 16 |
| 3 | 17 | 18 | | 17 |

| | | | | |
|---|---|---|---|---|
| 4 | 19 | 21 | | 19 |
| 5 | 17 | 23 | | 17 |
| 6 | 19 | 25 | | 19 |
| 7 | 21 | 23 | | 21 |
| 8 | 17 | 21 | | 17 |
| 9 | 7 | 9 | | 7 |
| 10 | 8 | 6 | | 8 |

Table 4.19: Soil properties after installation of SCP at $2^{nd}$ location

| Location 2 (in between three sand columns) | | | | | |
|---|---|---|---|---|---|
| Soil Layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
| Silty Sand<br>USCS Classification: SM | 3 | 2.68 | NP | NP | NP |
| | 6 | 2.63 | NP | NP | NP |
| Lean Clay<br>USCS Classification: CL | 9 | 2.33 | 33 | 22 | 11 |
| Silt: USCS Classification: ML | 10 | 2.20 | 47 | 32 | 15 |

Table 4.20: Comparison between SPT-N values before and after execution of SCP at location 2 (From Table 4.4 and Table 4.18)

| Depth | SPT-N (before SCP) | Average SPT-N (value before SCP) | SPT-N (after SCP) | Average SPT-N (value after SCP) |
|---|---|---|---|---|
| 1 | - | | 18 | |
| 1.5 | 7 | | - | |
| 2 | - | | 16 | |
| 3 | 4 | | 17 | |
| 4 | - | | 19 | |
| 4.5 | 2 | | - | |
| 5 | - | | 17 | |
| 6 | 1 | $3.20 \approx 3$ | 19 | 18 |
| 7 | - | | 21 | |
| 7.5 | 2 | | - | |
| 8 | - | | 17 | |
| 9 | 1 | | 7 | |
| 10 | - | | 8 | |

In Fig. 4.3, it is observed that at 6 meter depth before installation of sand compaction pile SPT-N value was as 1. Average SPT-N value was obtained 3 up to 10 meter depth. After installation of SCP, Subgrade conditions become Silty Fine Sand. SPT-N values were also increased after execution of SCP and average SPT-N value was found around 18 up to 10 meter depth.

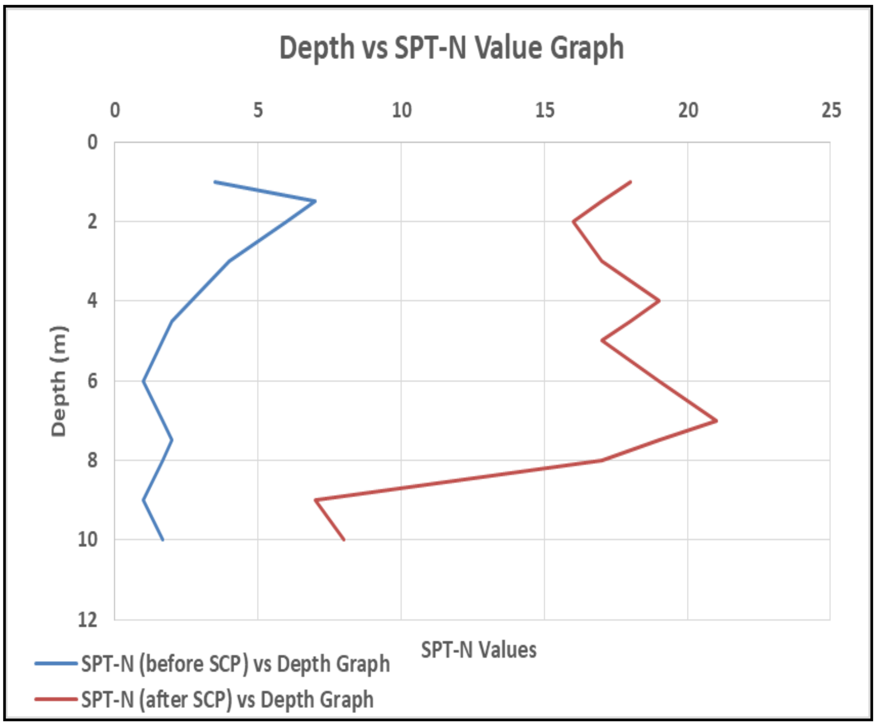



Fig. 4.3: Increasing SPT-N values after installation of SCP location 2.

## 4.6 Ground Improvement after Installation of PVD

The ground improvement after installation of PVD was also examined by performing SPT at random locations of PVD. The data obtained for both locations (Locations 3 and 4) undergone PVD are presented in Tables 4.21 and 4.26.

### 4.6.1 Soil Profile after Installation of PVD at Location 3

The data of PVD location 3 were analyzed and the results are presented in Tables 4.21 through 4.23. The results of SPT values before and after installation of PVD are presented in Table 4.24. The results are also presented in Fig. 4.5. Reports of Borehole data and soil properties at location 3 are presented in Section Anx.4 of ANNEXURE.

Table 4.21: Borehole test data after PVD installation at location 3

| Depth | | Blow count | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 1 | 1.45 | 4 | 5 | 8 | 13 |
| 2 | 2.45 | 5 | 6 | 8 | 14 |
| 3 | 3.45 | 6 | 8 | 10 | 18 |
| 4 | 4.45 | 7 | 6 | 8 | 14 |
| 5 | 5.45 | 7 | 7 | 6 | 13 |
| 6 | 6.45 | 6 | 7 | 5 | 12 |
| 7 | 7.45 | 3 | 8 | 8 | 16 |
| 8 | 8.45 | 2 | 5 | 6 | 11 |
| 9 | 9.45 | 3 | 5 | 7 | 12 |
| 10 | 10.45 | 3 | 4 | 6 | 10 |

Grain Size Analysis: Sieve analysis were performed in keeping with ASTM D2487. For oven-dry materials, sieving were finished for debris retained on a zero. Half mm sieve. In sieve analysis, the mass of soil retained on each sieve is decided and expressed as a percent of the entire mass of the pattern. The particle length is plotted on a logarithmic scale so that soils having the same diploma of uniformity are represented with the aid of curves of the distribution plot. Hydrometer analysis turned into performed for first-rate substances. This test is based totally at the precept of sedimentation of soil grains in water. When a soil specimen is dispersed in water, the particles settle at special velocities, relying on their form, length, and weight. For simplicity, its miles assumed that soil particles are spheres and the speed of soil debris may be specific by way of Stokes' regulation. Table 4.22 and Fig. 4.4 shows a sample of hydrometer analysis test report at location 3.

Table. 4.22: Hydrometer Analysis after installation of PVD (10 m depth), location 3

| HYDROMETER ANALYSIS (Method-AASHTO T88) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Weight of total dry sample: 50.0 gm | | | | | | | | | |
| Hydrometer Data | | | Percent Finer (%) | Diameter (mm) | Soil Classification | | | | |
| Hydrometer Type | ASTM 152H | | 100.0 | 4.75 | Gravel | | | | |
| Zero Correction | 5.0 | | 100.0 | 2.36 | Coarse Sand | | | | |
| Meniscus | 1.0 | | 100.0 | 1.18 | Medium Sand | | | | |
| Specific Gravity of Soil, $G_s$ | 2.395 | | 100.0 | 0.6 | Fine Sand | | | | |
| Dry wt of soil | gm | 50 | 100.0 | 0.3 | Silt | | | | |
| | | | 100.0 | 0.15 | | | | | |
| a | | 0.64 | 100.0 | 0.075 | Clay | | | | |
| | | | 53.77 | 0.04033 | | | | | |
| D10 | mm | 0.0035 | 47.36 | 0.02978 | Colloid | | | | |
| | | | 41.48 | 0.02167 | | | | | |
| D30 | mm | 0.012 | 35.06 | 0.02010 | $C_u$ | : | 13 | USCS | ML (Silt) |
| | | | 30.79 | 0.01186 | | | | | |
| D50 | mm | 0.036 | 26.51 | 0.00856 | $C_c$ | : | 1 | | |
| | | | 19.56 | 0.00632 | | | | | |
| D60 | mm | 0.05 | 13.15 | 0.00325 | LL | : | 38 | | |
| | | | 8.87 | 0.00324 | | | | | |
| D95 | mm | 0.075 | 4.60 | 0.00138 | PI | : | 13 | | |

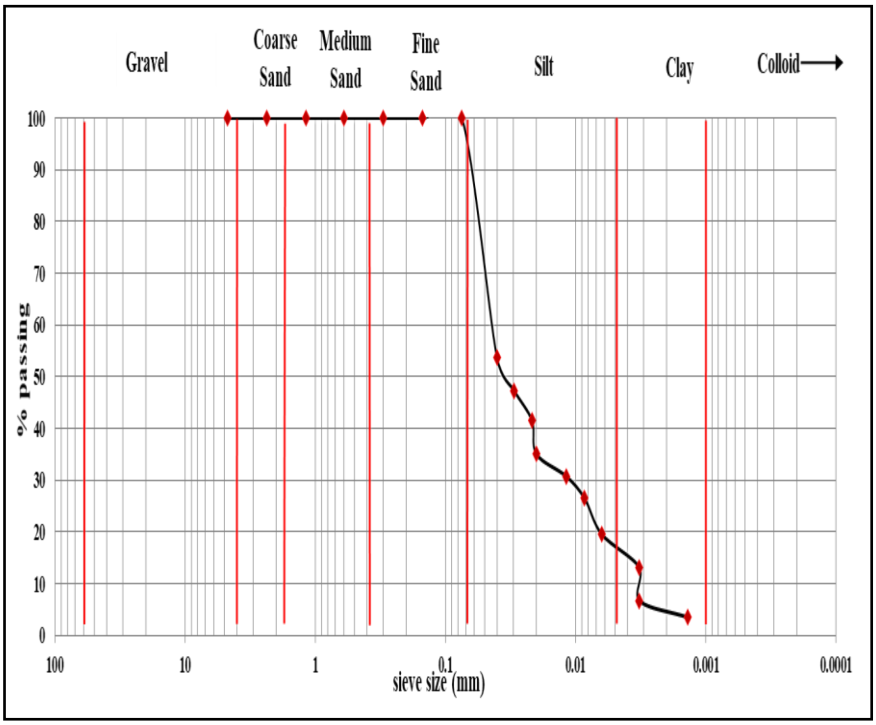



Fig. 4.4: Graphical presentation of Hydrometer test after installation of PVD in 3rd location.

Table 4.23: Soil properties after installation of PVD at location 3

| Soil Layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
|---|---|---|---|---|---|
| Silty Sand<br>USCS Classification: SM | 3 | 2.69 | NP | NP | NP |
| Silt<br>USCS Classification: ML | 9 | 2.69 | 46 | 29 | 17 |
| Poorly Graded Silty Sand<br>USCS Classification: SP-SM | 10 | 2.69 | 46 | 29 | 17 |

Table 4.24: Comparison between SPT-N values before and after execution of PVD at location 3 (from Table 4.6 and Table 4.21)

| Depth | SPT-N before PVD | Average SPT-N before PVD | SPT-N after PVD | Average SPT-N after PVD |
|---|---|---|---|---|
| 1.0 | - | | 13 | |
| 1.5 | 3 | | - | |
| 2.0 | - | | 14 | |
| 3.0 | 4 | | 18 | |

| Depth | SPT-N before PVD | Average SPT-N before PVD | SPT-N after PVD | Average SPT-N after PVD |
|---|---|---|---|---|
| 4.0 | - | 3.40 ≈ 3 | 14 | 13.87 ≈ 14 |
| 4.5 | 4 | | - | |
| 5.0 | - | | 13 | |
| 6.0 | 3 | | 12 | |
| 7.0 | - | | 16 | |
| 7.5 | 3 | | - | |
| 8.0 | - | | 11 | |
| 9.0 | 4 | | 12 | |
| 10.0 | - | | 10 | |

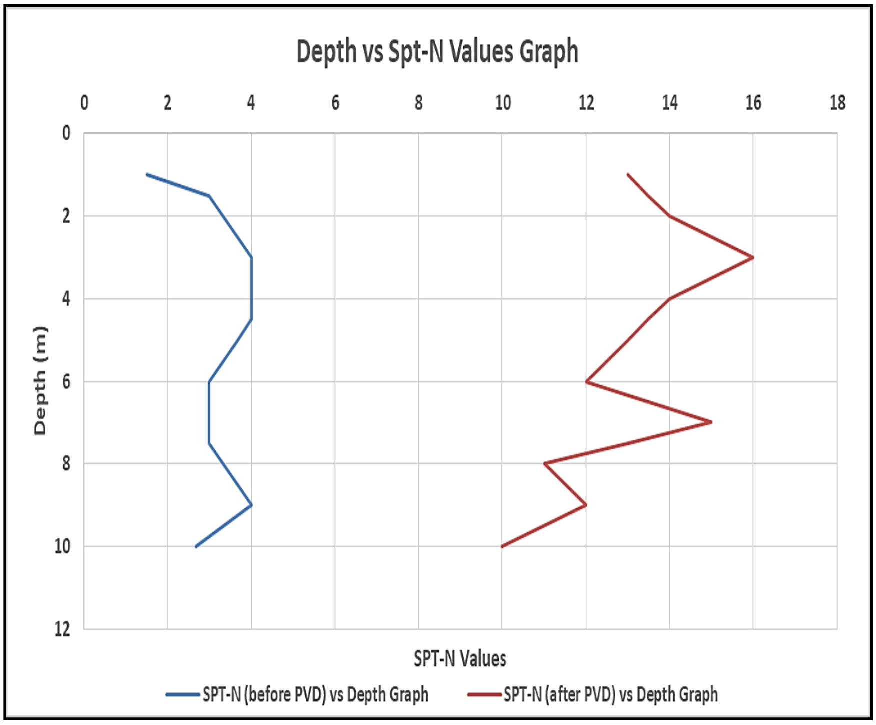



Fig. 4.5: Increasing SPT-N values after installation of PVD at location 3.

From Fig. 4.5, it is evident that before installation of PVD at 3 meter depth SPT-N value was 4 indicating that the soil was not suitable for highway construction, and soils were mostly loose and very soft fat clay and lean clay. Average SPT-N value was obtained around 3.5 up to 10 meter depth. After installation of PVD, subgrade conditions become suitable for highway construction and become mostly Sand. SPT-N values were also increased and average SPT-N value was found 14 up to 10-meter depth.

### 4.6.2 Settlement after Installation of PVD at Location 3

The settlement readings after installation of PVD at location 3 were observed and they are presented in Table 4.25 and Fig. 4.6. They show that at location 3, initial reduced level (RL) before installation PVD was 7.770 m on 22 June 2020. On 21 Jan 2021, reduced level (RL) was 7.578 m. Thus, the soil settlement was 192 mm (7.56 inch).

Table 4.25: Settlement plate monitoring data after installation of PVD at location 3

| Settlement plate monitoring field data | |
|---|---|
| Location | 3 |
| Installation Date | 22 June 2020 |
| Installation RL | 7.770 |

| Date | Plate pipe top RL (m) | Settlement (mm) | Cumulative settlement (mm) |
|---|---|---|---|
| 22-Jun-20 | 7.770 | 0 | 0 |
| 23-Jun-20 | 7.752 | 18 | -18 |
| 24-Jun-20 | 7.750 | 2 | -20 |
| 25-Jun-20 | 7.745 | 5 | -25 |
| 27-Jun-20 | 7.740 | 5 | -30 |
| 28-Jun-20 | 7.740 | 0 | -30 |
| 29-Jun-20 | 7.740 | 0 | -30 |
| 30-Jun-20 | 7.740 | 0 | -30 |
| 1-Jul-20 | 7.739 | 1 | -31 |
| 2-Jul-20 | 7.738 | 1 | -32 |
| 3-Jul-20 | 7.731 | 7 | -39 |
| 4-Jul-20 | 7.730 | 1 | -40 |
| 5-Jul-20 | 7.730 | 0 | -40 |
| 6-Jul-20 | 7.730 | 0 | -40 |
| 7-Jul-20 | 7.729 | 1 | -41 |
| 8-Jul-20 | 7.729 | 0 | -41 |
| 9-Jul-20 | 7.728 | 1 | -42 |
| 10-Jul-20 | 7.717 | 11 | -53 |
| 11-Jul-20 | 7.715 | 2 | -55 |

| Date | Plate pipe top RL (m) | Settlement (mm) | Cumulative settlement (mm) |
|---|---|---|---|
| 12-Jul-20 | 7.715 | 0 | -55 |
| 13-Jul-20 | 7.715 | 0 | -55 |
| 14-Jul-20 | 7.713 | 2 | -57 |
| 15-Jul-20 | 7.713 | 0 | -57 |
| 16-Jul-20 | 7.713 | 0 | -57 |
| 17-Jul-20 | 7.713 | 0 | -57 |
| 18-Jul-20 | 7.713 | 0 | -57 |
| 19-Jul-20 | 7.713 | 0 | -57 |
| 23-Jul-20 | 7.700 | 13 | -70 |
| 25-Jul-20 | 7.682 | 18 | -88 |
| 27-Jul-20 | 7.675 | 7 | -95 |
| 28-Jul-20 | 7.661 | 14 | -109 |
| 29-Jul-20 | 7.660 | 1 | -110 |
| 9-Aug-20 | 7.647 | 13 | -123 |
| 11-Aug-20 | 7.647 | 0 | -123 |
| 13-Aug-20 | 7.624 | 23 | -146 |
| 15-Aug-20 | 7.623 | 1 | -147 |
| 18-Aug-20 | 7.620 | 3 | -150 |
| 21-Aug-20 | 7.616 | 4 | -154 |
| 23-Aug-20 | 7.616 | 0 | -154 |
| 25-Aug-20 | 7.616 | 0 | -154 |
| 29-Aug-20 | 7.615 | 1 | -155 |
| 1-Sep-20 | 7.615 | 0 | -155 |
| 6-Sep-20 | 7.615 | 0 | -155 |
| 14-Sep-20 | 7.605 | 10 | -165 |
| 22-Sep-20 | 7.605 | 0 | -165 |
| 30-Sep-20 | 7.605 | 0 | -165 |
| 6-Oct-20 | 7.605 | 0 | -165 |
| 13-Oct-20 | 7.605 | 0 | -165 |
| 20-Oct-20 | 7.605 | 0 | -165 |
| 27-Oct-20 | 7.602 | 3 | -168 |
| 3-Nov-20 | 7.602 | 0 | -168 |

| Date | Plate pipe top RL (m) | Settlement (mm) | Cumulative settlement (mm) |
|---|---|---|---|
| 10-Nov-20 | 7.600 | 2 | -170 |
| 17-Nov-20 | 7.600 | 0 | -170 |
| 22-Nov-20 | 7.594 | 6 | -176 |
| 30-Nov-20 | 7.594 | 0 | -176 |
| 4-Dec-20 | 7.594 | 0 | -176 |
| 5-Dec-20 | 7.594 | 0 | -176 |
| 12-Dec-20 | 7.591 | 3 | -179 |
| 17-Dec-20 | 7.589 | 2 | -181 |
| 24-Dec-20 | 7.588 | 1 | -182 |
| 31-Dec-2020 | 7.585 | 3 | -185 |
| 7-Jan-2021 | 7.580 | 5 | -190 |
| 14-Jan-2021 | 7.580 | 0 | -190 |
| 21-Jan-2021 | 7.578 | 2 | -192 |

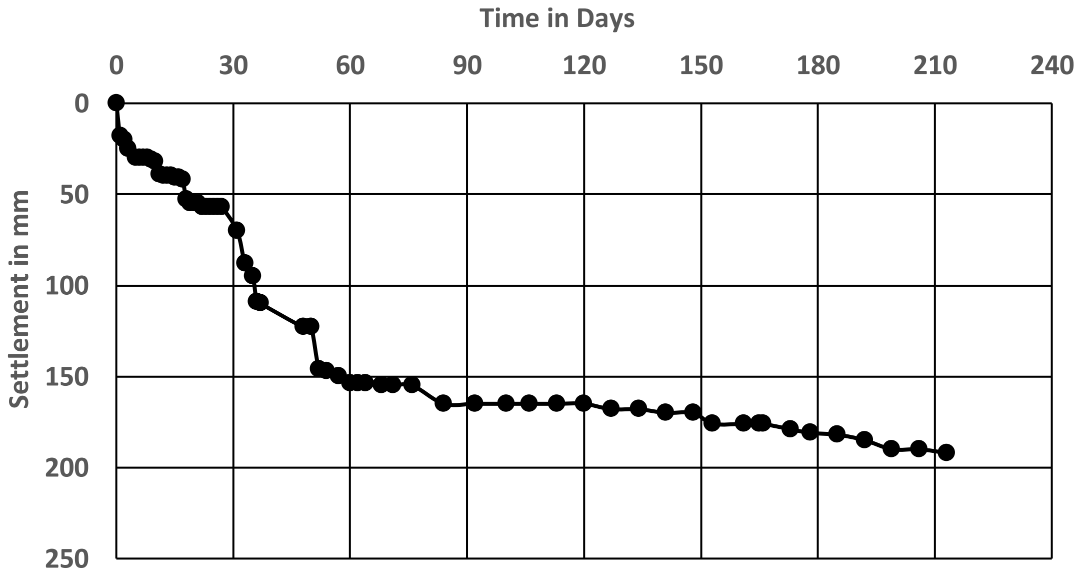



Fig. 4.6: Time-settlement curve for PVD at location 3.

### 4.6.3 Soil Profile after Installation of PVD at Location 4

The data of PVD location 4 were analyzed and the results are presented in Tables 4.26 through 4.28. The results of SPT values before and after installation of PVD are presented in Table 4.28. The results are also presented in Fig. 4.7.

Table 4.26: Borehole test data after PVD installation at location 4

| Depth | | Blow count | | | |
|---|---|---|---|---|---|
| From (meter) | To (meter) | 150 (mm) | 300 (mm) | 450 (mm) | N-Value |
| 1 | 1.45 | 6 | 5 | 7 | 12 |
| 2 | 2.45 | 6 | 7 | 9 | 16 |
| 3 | 3.45 | 7 | 6 | 6 | 12 |
| 4 | 4.45 | 6 | 5 | 8 | 13 |
| 5 | 5.45 | 6 | 8 | 11 | 19 |
| 6 | 6.45 | 5 | 6 | 9 | 15 |
| 7 | 7.45 | 6 | 6 | 7 | 13 |
| 8 | 8.45 | 4 | 5 | 5 | 10 |
| 9 | 9.45 | 2 | 4 | 6 | 10 |
| 10 | 10.45 | 3 | 5 | 6 | 11 |

Table 4.27: Soil properties after installation of PVD at location 4

| Location 4 | | | | | |
|---|---|---|---|---|---|
| Soil layer | Depth (m) | Specific gravity | Liquid limit | Plastic limit | Plasticity index |
| Silty Sand<br>USCS Classification: SM | 4.5 | 2.67 | NP | NP | NP |
| Lean Clay<br>USCS Classification: CL | 9 | 2.67 | 33 | 21 | 12 |
| Silty Sand<br>USCS Classification: SM | 10 | 2.67 | NP | NP | NP |

Table 4.28: Comparison between SPT-N values before and after execution of PVD at 4$^{th}$ location: (from Table 4.8 and Fig. 4.26)

| Depth | SPT-N (before SCP) | Average SPT-N (Value before SCP) | SPT-N (after SCP) | Average SPT-N (value after SCP) |
|---|---|---|---|---|
| 1 | - | | 12 | |
| 1.5 | 2 | | - | |
| 2 | - | | 16 | |
| 3 | 4 | | 12 | |

| Depth | SPT-N (before SCP) | Average SPT-N (Value before SCP) | SPT-N (after SCP) | Average SPT-N (value after SCP) |
|---|---|---|---|---|
| 4 | - | 3.4 ≈ 3 | 13 | 13.75 ≈ 14 |
| 4.5 | 5 | | - | |
| 5 | - | | 19 | |
| 6 | 4 | | 15 | |
| 7 | - | | 13 | |
| 7.5 | 2 | | - | |
| 8 | - | | 10 | |
| 9 | 4 | | 10 | |
| 10 | - | | 11 | |

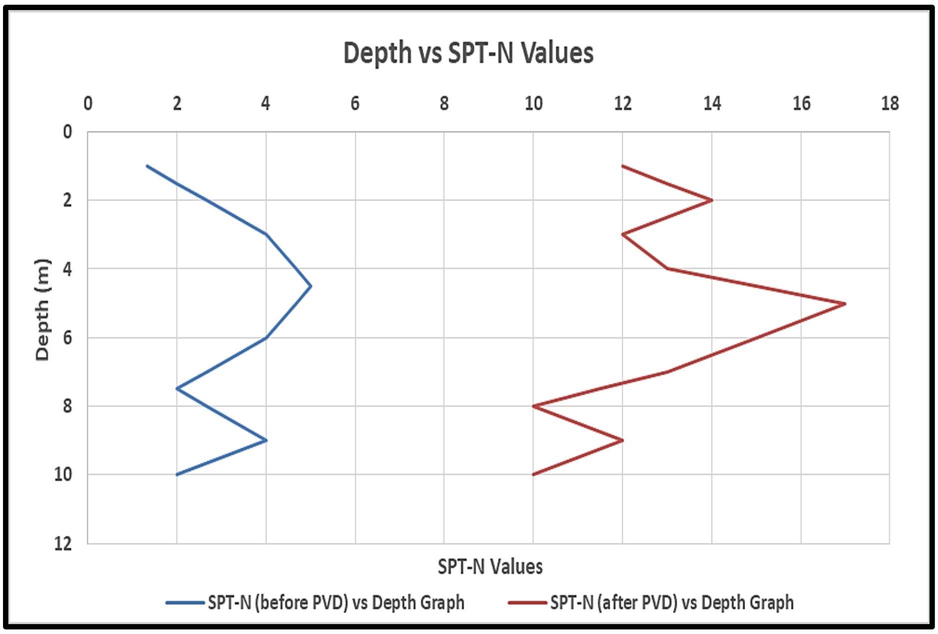



Fig. 4.7: Increasing SPT-N values after installation of PVD at location 4.

From Fig 4.7, before installation of PVD at 3 meter depth, the SPT-N value was found as low as 4. Soil condition was not suitable for highway construction and soils were mostly loose and very soft fat clay and Lean Clay. Average SPT-N value was identified around 3.5 up to 10 meter depth. After installation of PVD, subgrade conditions become suitable for highway construction and become mostly Sand. SPT-N values were also increased and average SPT-N was found around 14 up to 10 meter depth.

### 4.6.4 Settlement after Installation of PVD at Location 4

The settlement readings after installation of PVD at location 4 were observed and they are presented in Table 4.29 and Fig. 4.8. They show that at location 3, initial reduced level (RL) before installation PVD was 7.759 m on 22 June 2020. On 21 Jan 2021, reduced level (RL) was 7.595 m. Thus, the soil settlement was 164 mm (6.45 inch).

Table 4.29: Settlement plate monitoring data after installation of PVD at location 4

| Settlement Plate Monitoring Field Data | |
|---|---|
| Location | 4 |
| Installation Date | 22 June 2020 |
| Installation RL | 7.759 |

| Date | Plate pipe top RL (m) | Settlement (mm) | Cumulative settlement (mm) |
|---|---|---|---|
| 22-Jun-20 | 7.759 | 0 | 0 |
| 23-Jun-20 | 7.741 | 18 | -18 |
| 24-Jun-20 | 7.741 | 0 | -18 |
| 25-Jun-20 | 7.736 | 5 | -23 |
| 27-Jun-20 | 7.727 | 9 | -32 |
| 28-Jun-20 | 7.726 | 1 | -33 |
| 29-Jun-20 | 7.721 | 5 | -38 |
| 30-Jun-20 | 7.720 | 1 | -39 |
| 1-Jul-20 | 7.719 | 1 | -40 |
| 2-Jul-20 | 7.714 | 5 | -45 |
| 3-Jul-20 | 7.711 | 3 | -48 |
| 5-Jul-20 | 7.708 | 3 | -51 |
| 6-Jul-20 | 7.706 | 2 | -53 |
| 7-Jul-20 | 7.704 | 2 | -55 |
| 8-Jul-20 | 7.703 | 1 | -56 |
| 10-Jul-20 | 7.700 | 3 | -59 |
| 11-Jul-20 | 7.697 | 3 | -62 |
| 14-Jul-20 | 7.695 | 2 | -64 |
| 19-Jul-20 | 7.690 | 5 | -69 |
| 23-Jul-20 | 7.680 | 10 | -79 |
| 25-Jul-20 | 7.675 | 5 | -84 |

| Date | Plate pipe top RL (m) | Settlement (mm) | Cumulative settlement (mm) |
|---|---|---|---|
| 27-Jul-20 | 7.670 | 5 | -89 |
| 28-Jul-20 | 7.665 | 5 | -94 |
| 29-Jul-20 | 7.662 | 3 | -97 |
| 9-Aug-20 | 7.662 | 0 | -97 |
| 11-Aug-20 | 7.660 | 2 | -99 |
| 18-Aug-20 | 7.660 | 0 | -99 |
| 21-Aug-20 | 7.654 | 6 | -105 |
| 23-Aug-20 | 7.654 | 0 | -105 |
| 29-Aug-20 | 7.654 | 0 | -105 |
| 4-Sep-20 | 7.654 | 0 | -105 |
| 7-Sep-20 | 7.650 | 4 | -109 |
| 10-Sep-20 | 7.650 | 0 | -109 |
| 14-Sep-20 | 7.645 | 5 | -114 |
| 18-Sep-20 | 7.645 | 0 | -114 |
| 22-Sep-20 | 7.645 | 0 | -114 |
| 30-Sep-20 | 7.644 | 1 | -115 |
| 20-Oct-20 | 7.644 | 0 | -115 |
| 27-Oct-20 | 7.639 | 5 | -120 |
| 3-Nov-20 | 7.639 | 0 | -120 |
| 10-Nov-20 | 7.636 | 3 | -123 |
| 17-Nov-20 | 7.636 | 0 | -123 |
| 22-Nov-20 | 7.615 | 21 | -144 |
| 30-Nov-20 | 7.615 | 0 | -144 |
| 4-Dec-20 | 7.610 | 5 | -149 |
| 5-Dec-20 | 7.610 | 0 | -149 |
| 12-Dec-20 | 7.606 | 4 | -153 |
| 17-Dec-20 | 7.602 | 4 | -157 |
| 24-Dec-20 | 7.600 | 2 | -159 |
| 31-Dec-20 | 7.600 | 0 | -159 |
| 07-Jan-21 | 7.599 | 1 | -160 |
| 14-Jan-21 | 7.595 | 4 | -164 |
| 21-Jan-21 | 7.595 | 0 | -164 |

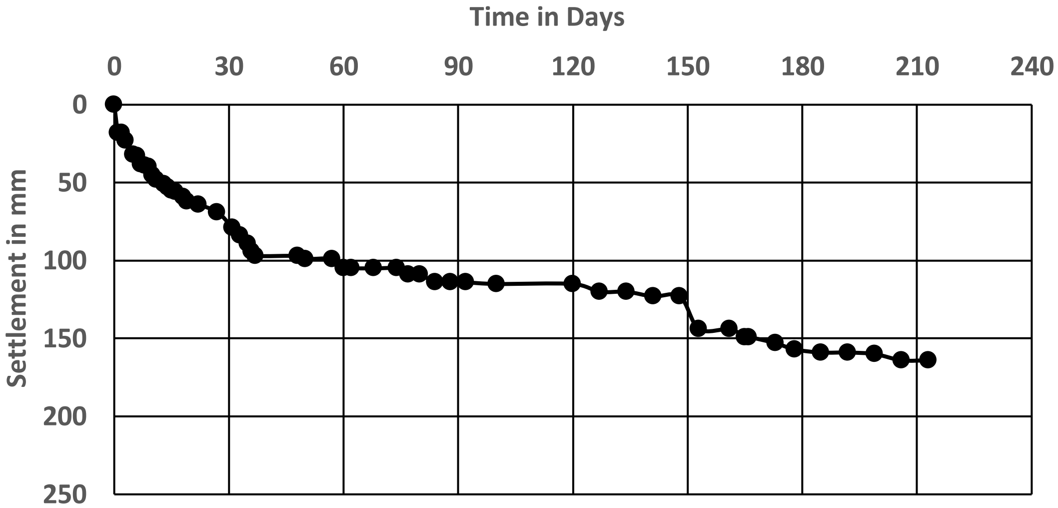



Fig. 4.8: Time-settlement curve for PVD at location 4.

## 4.7 Dynamic Cone Penetration (DCP) Results

The DCP tests were done at four different locations where SCP and PVD were done earlier at shallow depths up to 0.4 meter. The purpose of this test was to show the comparison of average CBR value between SCP and PVD. The result of this test ultimately indicated suitability of subgrade improvement between these two methods.

### 4.7.1 DCP Test Results at Location 1

The CBR values at location 1 where SCPs were installed are calculated as per ASTM D6951 (2018) and the results are presented in Table 4.30, along with the results of the DCP test. The variations of Cumulative Penetration and CBR values are also shown in Fig. 4.9 and Fig. 4.10 respectively. Reports of DCP Test at location 1 are presented in Section Anx.5 of ANNEXURE.

After installation of SCP, the penetration reading were taken at 90 mm to 390 mm (total depth 300 mm) and 28 blows were counted. Based on the wide application of dynamic cone penetrometer (DCP) in detecting engineering for subgrade, DCP Index (penetration/blow) was found 20 mm after 1$^{st}$ blow. In the 2$^{nd}$ blow, DCP Index was found 30 mm. After 20 blows, the DCP index was found 240 mm. The 1$^{st}$ Layer thickness was 135 mm (8 no blows) and the average CBR percentage was found 14.94. Next layer thickness was 165 mm (20 no blows). Here CBR percentage was found 31.3. DCP termination depth was taken -0.3 meter. Average CBR percentage for the full depth can be taken 23%.

Table 4.30: Average CBR percentage after installation of SCP at location 1

| No. of blows | Penetration reading (mm) | DCP Index (mm) | Cumulative penetration (mm) | CBR Value (%) = 3700/ $(Pen)^{1.3}$ | Average CBR (%) |
|---|---|---|---|---|---|
| 0 | 90 | 0 | 0 | 0 | 14.94 |
| 1 | 110 | 20 | 20 | 10.2 | |
| 2 | 140 | 30 | 50 | 6.5 | |
| 3 | 150 | 10 | 60 | 22.2 | |
| 4 | 165 | 15 | 75 | 14.1 | |
| 5 | 180 | 15 | 90 | 14.1 | |
| 6 | 190 | 10 | 100 | 22.2 | |
| 7 | 200 | 10 | 110 | 22.2 | |
| 8 | 225 | 25 | 135 | 8.0 | |
| 9 | 235 | 10 | 145 | 22.2 | 31.3 |
| 10 | 245 | 10 | 155 | 22.2 | |
| 11 | 255 | 10 | 165 | 22.2 | |
| 12 | 265 | 10 | 175 | 22.2 | |
| 13 | 270 | 5 | 180 | 48.2 | |
| 14 | 280 | 10 | 190 | 22.2 | |
| 15 | 290 | 10 | 200 | 22.2 | |
| 16 | 300 | 10 | 210 | 22.2 | |
| 17 | 310 | 10 | 220 | 22.2 | |
| 18 | 320 | 10 | 230 | 22.2 | |
| 19 | 325 | 5 | 235 | 48.2 | |
| 20 | 330 | 5 | 240 | 48.2 | |
| 21 | 340 | 10 | 250 | 22.2 | |
| 22 | 350 | 10 | 260 | 22.2 | |
| 23 | 355 | 5 | 265 | 48.2 | |
| 24 | 360 | 5 | 270 | 48.2 | |
| 25 | 370 | 10 | 280 | 22.2 | |
| 26 | 375 | 5 | 285 | 48.2 | |
| 27 | 380 | 5 | 290 | 48.2 | |
| 28 | 390 | 10 | 300 | 22.2 | |

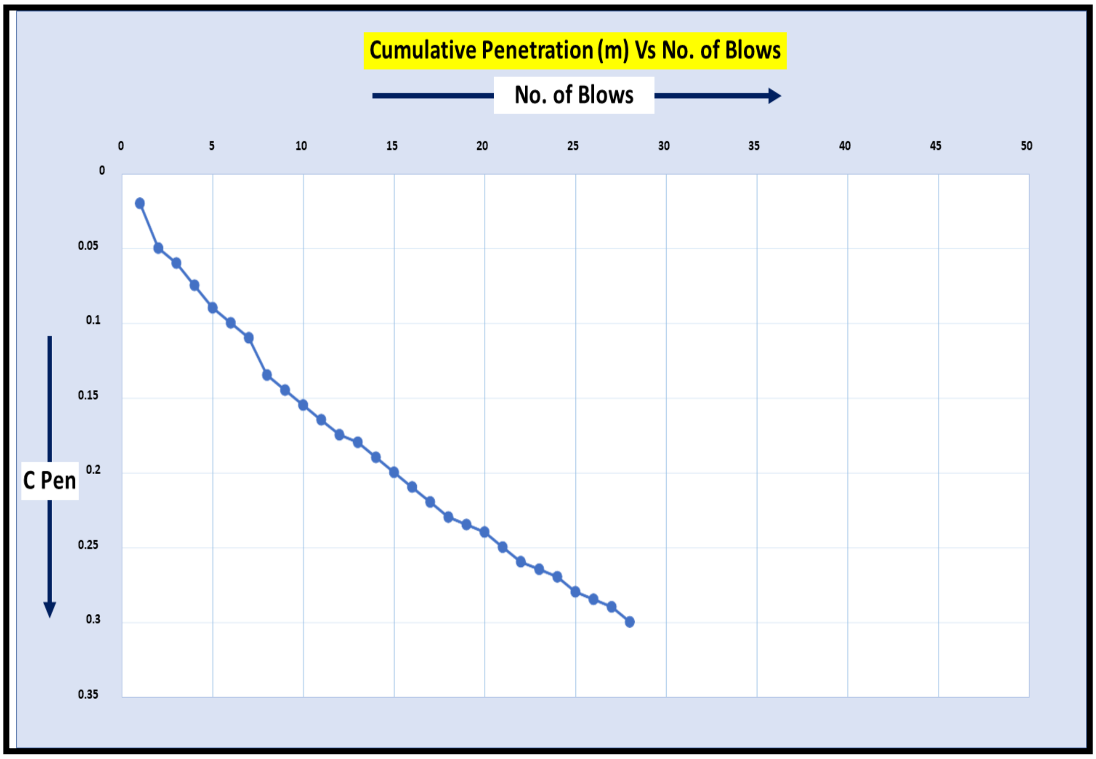



Fig. 4.9: Cumulative Penetration (mm) values with number of blows at location 1.

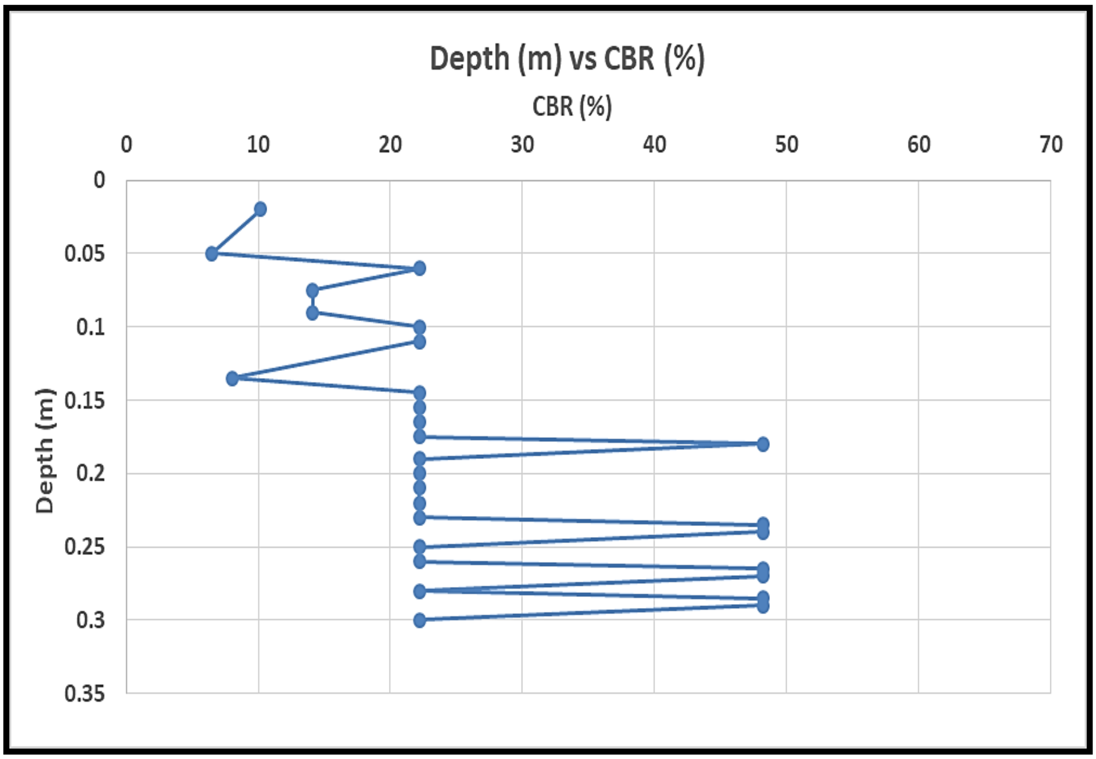



Fig. 4.10: CBR (%) values with depth (m) at location 1.

### 4.7.2 DCP Test Results at Location 2

The measured CBR values for the SCPs installed at Location 2 as per ASTM D6951 (2018) are presented in Table 4.31, along with the results of the DCP test. The variations of Cumulative Penetration and CBR values are also shown in Fig. 4.11 and Fig. 4.12 respectively. Reports of DCP Test at location 2 are given in Section Anx.5 of ANNEXURE.

After installation of SCP penetration reading of DCP was taken from 100 mm to 415 mm (total depth 315 mm). Dynamic cone penetrometer (DCP) test provides a reliable DCP index for the engineering properties of soils. Here, DCP Index (penetration/blow) was found 40 mm after 1st blow. In the 2nd blow, DCP Index was found 25 mm. After 20 blows, the DCP index was found 305 mm. The 1st layer thickness was 110 mm (4 no blows) and the average CBR percentage was found 7.73. Next layer thickness was 205 mm (17 no blows) and CBR percentage was found 19.11. The DCP termination depth was taken 0.315 meter. Average CBR percentage for the full depth can be taken 13%.

Table 4.31: Average CBR percentage after installation of SCP at location

| No. of blows | Penetration reading (mm) | DCP Index (mm) | Cumulative penetration (mm) | CBR Value (%) = 3700/ $(Pen)^{1.3}$ | Average CBR (%) |
|---|---|---|---|---|---|
| 0 | 100 | 0 | 0 | -- | 7.33 |
| 1 | 140 | 40 | 40 | 4.7 | |
| 2 | 165 | 25 | 65 | 8.0 | |
| 3 | 185 | 20 | 85 | 10.2 | |
| 4 | 210 | 25 | 110 | 8.0 | |
| 5 | 220 | 10 | 120 | 22.2 | |
| 6 | 235 | 15 | 135 | 14.1 | |
| 7 | 250 | 15 | 150 | 14.1 | |
| 8 | 260 | 10 | 160 | 22.2 | |
| 9 | 280 | 20 | 180 | 10.2 | 19.11 |
| 10 | 290 | 10 | 190 | 22.2 | |
| 11 | 305 | 15 | 205 | 14.1 | |
| 12 | 320 | 15 | 220 | 14.1 | |
| 13 | 330 | 10 | 230 | 22.2 | |
| 14 | 340 | 10 | 240 | 22.2 | |

| No. of blows | Penetration reading (mm) | DCP Index (mm) | Cumulative penetration (mm) | CBR Value (%) = 3700/ (Pen)$^{1.3}$ | Average CBR (%) |
|---|---|---|---|---|---|
| 15 | 350 | 10 | 250 | 22.2 | |
| 16 | 360 | 10 | 260 | 22.2 | |
| 17 | 370 | 10 | 270 | 22.2 | |
| 18 | 380 | 10 | 280 | 22.2 | |
| 19 | 390 | 10 | 290 | 22.2 | |
| 20 | 405 | 15 | 305 | 14.1 | |
| 21 | 415 | 10 | 315 | 22.2 | |

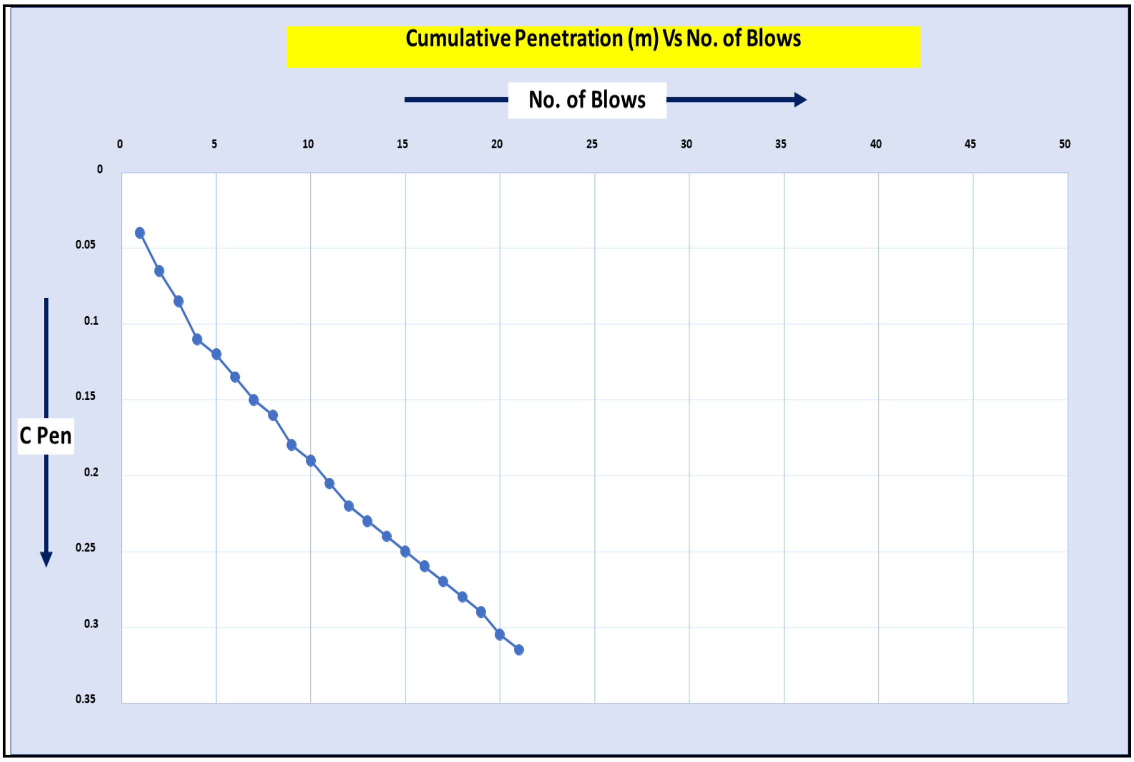



Fig. 4.11: Cumulative Penetration (mm) values with number of blows at location 2.

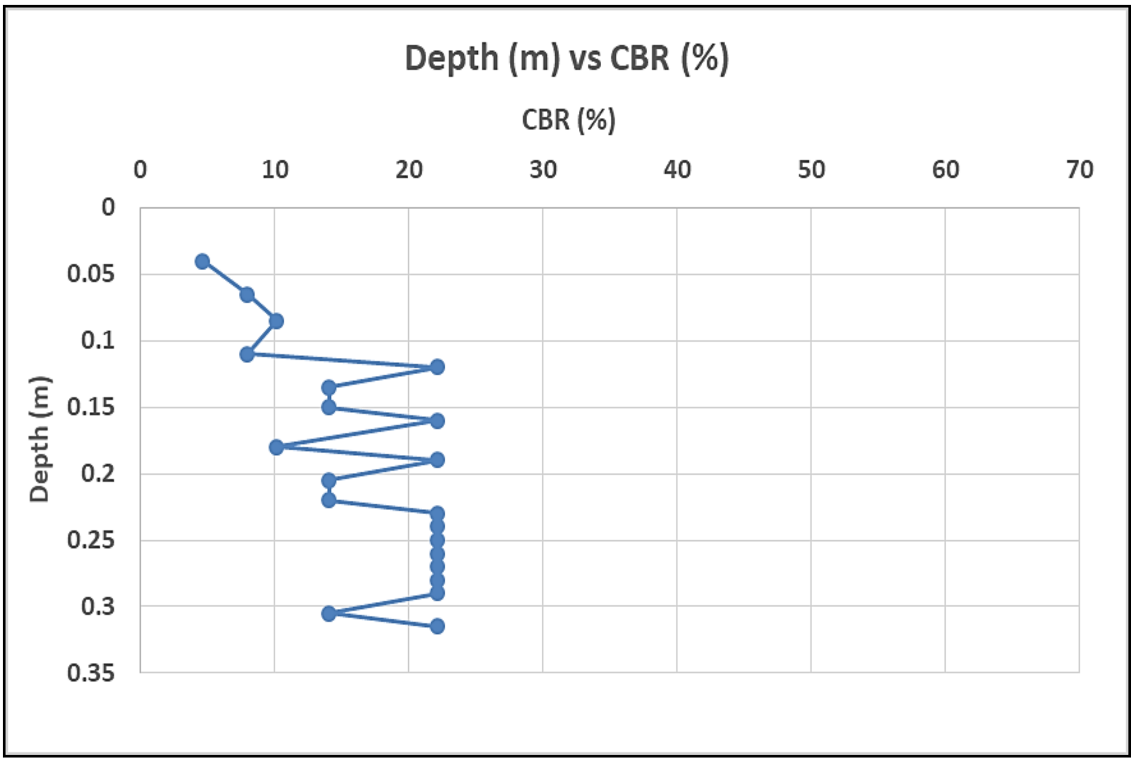



Fig. 4.12: CBR (%) values with depth at location 2.

### 4.7.3 DCP Test Results at Location 3

The measured CBR values for the PVDs installed at Location 3 as per ASTM D6951 (2018) are presented in Table 4.32, along with the results of the DCP test. The variations of Cumulative Penetration and CBR values are also shown in Fig. 4.13 and Fig. 4.14 respectively. Reports of DCP Test at location 3 are listed in Section Anx.5 of ANNEXURE.

The DCP penetration reading were taken from 140 mm to 365 mm (total depth 225 mm). DCP Index (penetration/blow) was found 45 mm after 1$^{st}$ blow. In the 2$^{nd}$ blow, DCP Index was found 25 mm. After 20 blows, the DCP index was found 225 mm. The 1$^{st}$ layer thickness was 95 mm up to 235 mm depth (3 no blows) and the average CBR percentage was found 6.73. The 2$^{nd}$ layer thickness was 45 mm (4 no blows) and CBR percentage was found 20.18. Next layer thickness was 85 mm (13 no blows) and CBR percentage was found 40.2. The DCP termination depth was taken as 0.225 meter. Average CBR percentage for the full depth can be taken 22%.

Table 4.32: Average CBR percentage after installation of PVD at location 3

| No. of blows | Penetration reading (mm) | DCP Index (mm) | Cumulative penetration (mm) | CBR Value (%) = 3700/ (Pen)$^{1.3}$ | Average CBR (%) |
|---|---|---|---|---|---|
| 0 | 140 | 0 | 0 | - | 6.73 |
| 1 | 185 | 45 | 45 | 4.2 | |
| 2 | 210 | 25 | 70 | 8.0 | |
| 3 | 235 | 25 | 95 | 8.0 | |
| 4 | 250 | 15 | 110 | 14.1 | 20.18 |
| 5 | 260 | 10 | 120 | 22.2 | |
| 6 | 270 | 10 | 130 | 22.2 | |
| 7 | 280 | 10 | 140 | 22.2 | |
| 8 | 285 | 5 | 145 | 48.2 | 40.2 |
| 9 | 295 | 10 | 155 | 22.2 | |
| 10 | 300 | 5 | 160 | 48.2 | |
| 11 | 305 | 5 | 165 | 48.2 | |
| 12 | 310 | 5 | 170 | 48.2 | |
| 13 | 320 | 10 | 180 | 22.2 | |
| 14 | 325 | 5 | 185 | 48.2 | |
| 15 | 330 | 5 | 190 | 48.2 | |
| 16 | 340 | 10 | 200 | 22.2 | |
| 17 | 350 | 10 | 210 | 22.2 | |
| 18 | 355 | 5 | 215 | 48.2 | |
| 19 | 360 | 5 | 220 | 48.2 | |
| 20 | 365 | 5 | 225 | 48.2 | |

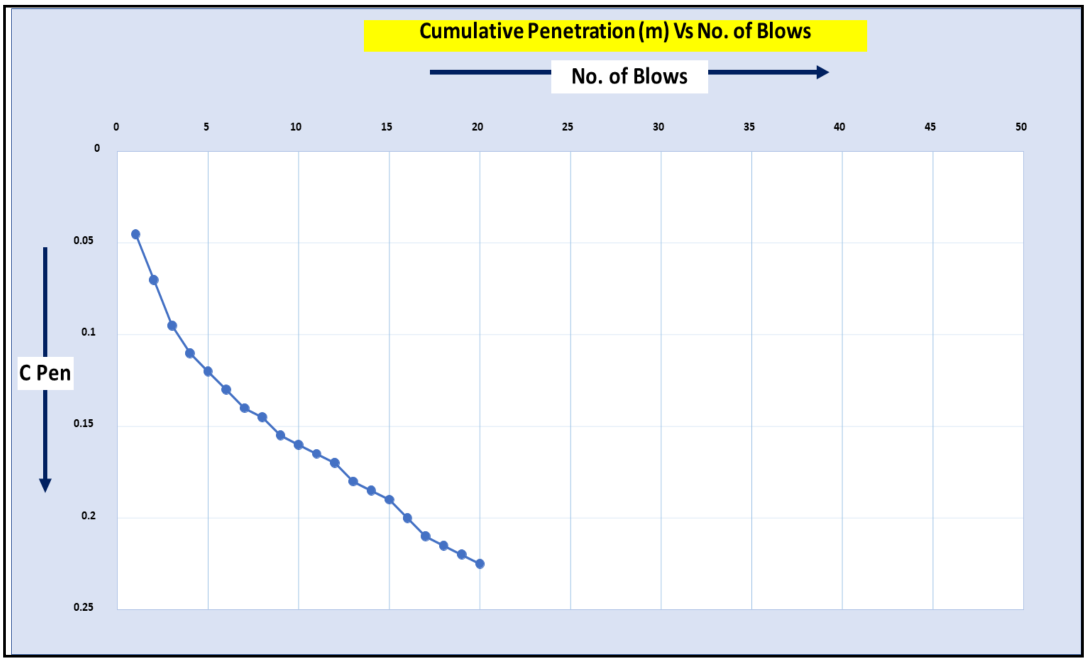



Fig. 4.13: Cumulative Penetration (mm) values with number of blows at location 3.

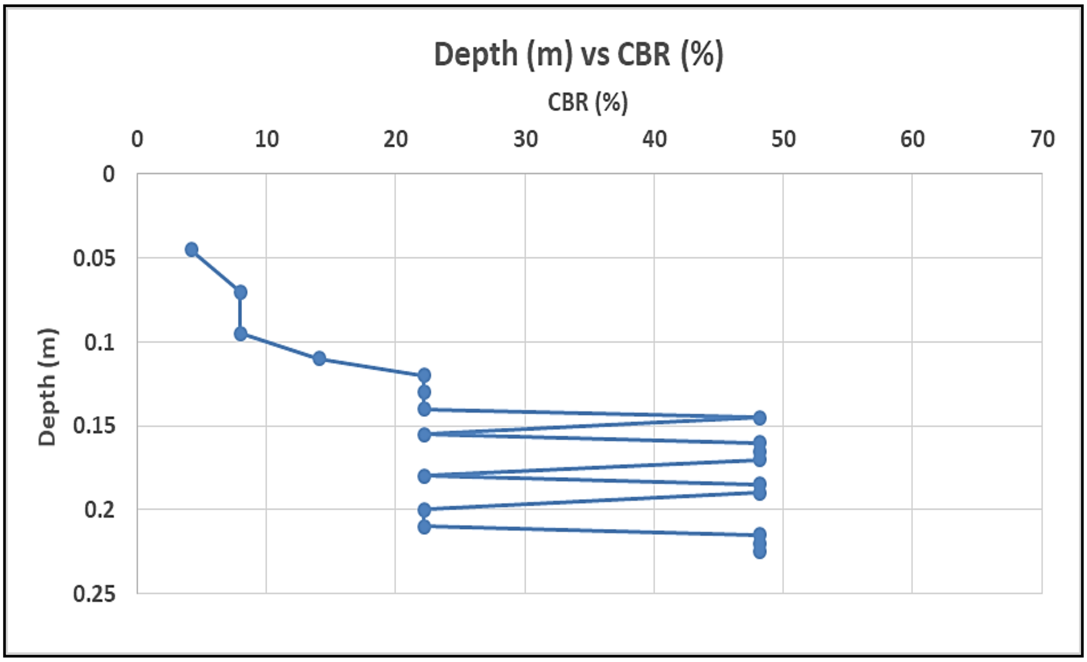



Fig. 4.14: CBR (%) values with depth at location 3.

### 4.7.4 DCP Test Results at Location 4

The measured CBR values for the PVDs installed at Location 4 as per ASTM D6951 (2018) are presented in Table 4.33, along with the results of the DCP test. The variations of Cumulative Penetration and CBR values are also shown in Fig. 4.15 and Fig. 4.16 respectively. Reports of DCP Test at location 4 are presented in Section Anx.5 of ANNEXURE.

The DCP penetration reading was taken from 125 mm to 420 mm (total depth 295 mm). 26 number of blows were counted. The layer thickness was 295 mm (26 no blows). DCP Index (penetration/blow) was found 5 mm after 1st blow. In the 2nd blow, DCP Index was found 20 mm. After 20 blows, the DCP index was found 225 mm. Average CBR percentage was 20.87. The DCP termination depth was taken as 0.295 meter. The average CBR percentage for the full depth can be taken 21%.

Table 4.33: Average CBR percentage after installation of PVD at Location 4

| No. of blows | Penetration reading (mm) | DCP Index (mm) | Cumulative penetration (mm) | CBR value (%) = 3700/ $(Pen)^{1.3}$ | Average CBR (%) |
|---|---|---|---|---|---|
| 0 | 125 | 0 | -- | -- | 20.87 |
| 1 | 130 | 5 | 5 | 48.2 | |
| 2 | 150 | 20 | 25 | 10.2 | |
| 3 | 160 | 10 | 35 | 22.2 | |
| 4 | 175 | 15 | 50 | 14.1 | |
| 5 | 190 | 15 | 65 | 14.1 | |
| 6 | 200 | 10 | 75 | 22.2 | |
| 7 | 210 | 10 | 85 | 22.2 | |
| 8 | 220 | 10 | 95 | 22.2 | |
| 9 | 230 | 10 | 105 | 22.2 | |
| 10 | 240 | 10 | 115 | 22.2 | |
| 11 | 255 | 15 | 130 | 14.1 | |
| 12 | 270 | 15 | 145 | 14.1 | |
| 13 | 280 | 10 | 155 | 22.2 | |
| 14 | 290 | 10 | 165 | 22.2 | |
| 15 | 300 | 10 | 175 | 22.2 | |
| 16 | 310 | 10 | 185 | 22.2 | |
| 17 | 320 | 10 | 195 | 22.2 | |
| 18 | 330 | 10 | 205 | 22.2 | |
| 19 | 340 | 10 | 215 | 22.2 | |
| 20 | 350 | 10 | 225 | 22.2 | |
| 21 | 360 | 10 | 235 | 22.2 | |
| 22 | 370 | 10 | 245 | 22.2 | |
| 23 | 380 | 10 | 255 | 22.2 | |

| No. of blows | Penetration reading (mm) | DCP Index (mm) | Cumulative penetration (mm) | CBR value (%) = $3700/\,(Pen)^{1.3}$ | Average CBR (%) |
|---|---|---|---|---|---|
| 24 | 395 | 15 | 270 | 14.1 | |
| 25 | 410 | 15 | 285 | 14.1 | |
| 26 | 420 | 10 | 295 | 22.2 | |

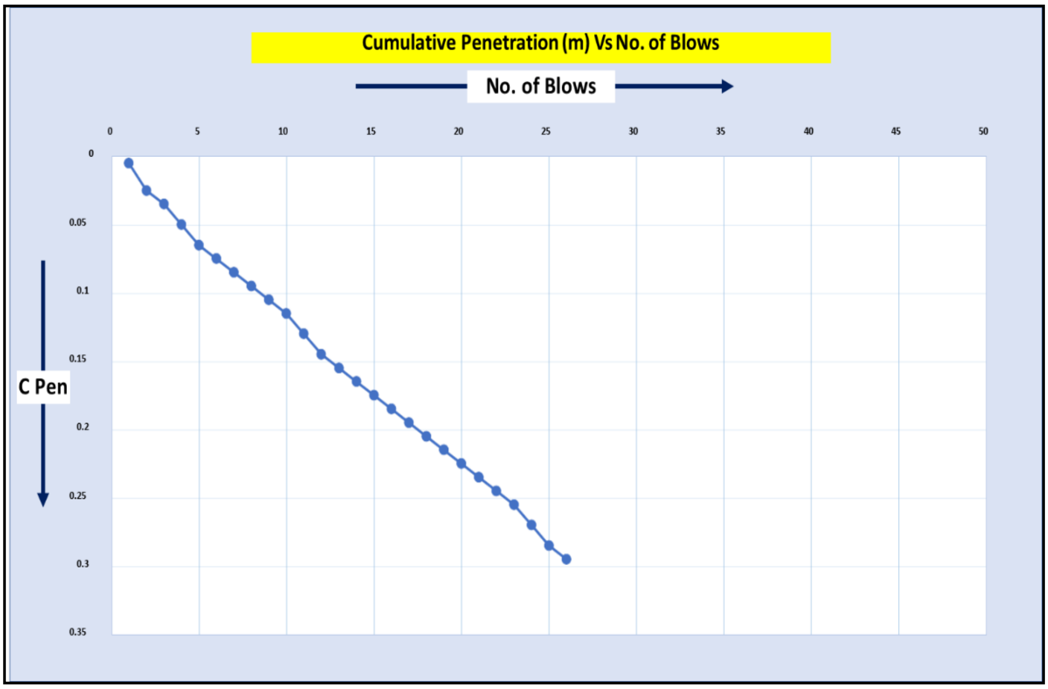



Fig. 4.15: Cumulative Penetration (mm) values with number of blows at location 4.

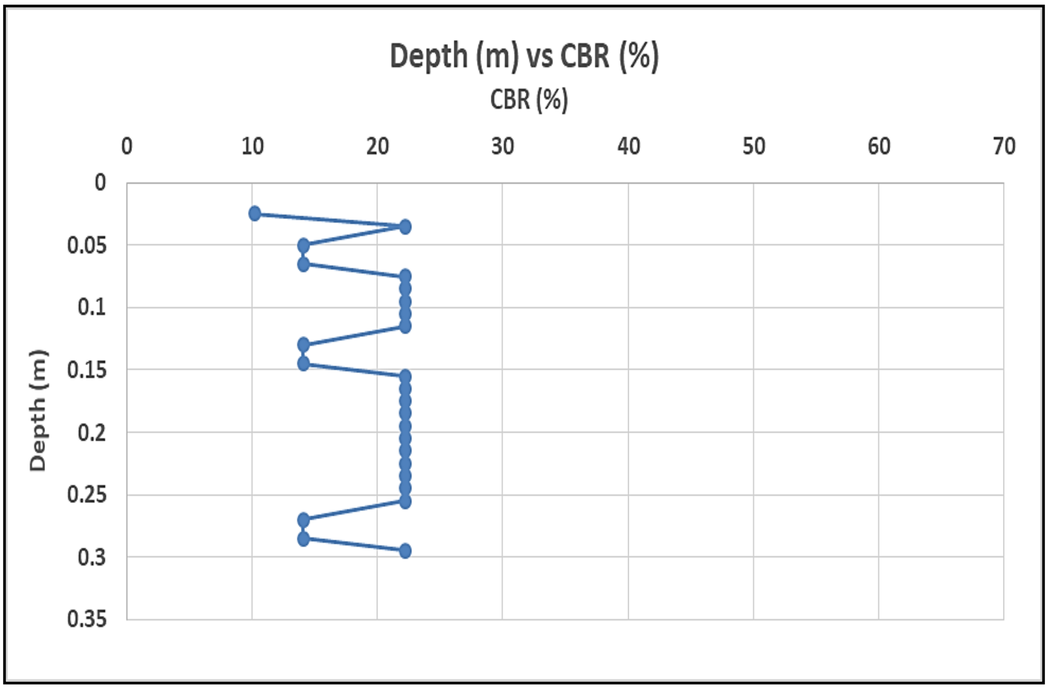



Fig. 4.16: CBR (%) values with depth at location 4.

### 4.7.5 Analysis of DCP Test Results

The results of DCP test were used to estimate the CBR of subgrade soil of both types of sites (improved by SCP and PVD) using the procedures suggested by ASTM D6951 (2018). The following observations obtained from DCP test are given below:

a. The DCP test was performed in shallow depth (only up to 0.4 meter) to assess the strength of subgrade at top surface. The DCP test was conducted after ground improvement only to compare effectiveness of SCP and PVD.

b. From Tables 4.30 to 4.33, DCP Index was found 240 mm, 305 mm, 225 mm and 225 mm after 20 no of blows at location 1, location 2, location 3 and location 4 respectively. Where ground improvement measures were undertaken using PVD, soil condition was marginally good at shallow depth than those where SCP was installed. From Figs. 4.10, 4.12, 4.14 and 4.16 the average CBR (%) was also found 18.5% and 21.5% respectively subgrade improvement done by SCP and PVD respectively. The average CBR as such was slightly higher where ground improvement was conducted through PVD than that of SCP. Therefore, it is understood that SCP and PVD provide comparable CBR at shallow depth.

c. One of the probable causes of higher CBR due to PVD might be that DCP test was done at shallow depth after the removal of surcharge and thus soil was under compression.

## 4.8 Cost Analysis of SCP and PVD

Cost analysis of SCP with different properties, i.e. diameter of the pile, spacing, length of pile, fineness modulus (FM) are estimated in subgrade improvement. On the contrary, the cost for PVD depends upon composite drain properties, filter fabric properties, average spacing, length, sand blanket height, surcharge height etc. This cost analysis is site specific and applicable to this particular design of SCP and PVD. The cost estimate was made as per the schedule of rates of LGED (2019-20). Detailed cost estimation for SCP and PVD are shown below.

**Cost for Sand Compaction Pile**

Diameter of pile : 300 mm

Length of pile : 8 Meter

Spacing : 1.4 Meter

Volume of sand : 0.25 cum/meter

Type of filled sand : Sylhet sand, a yellowish-brown color river sand is used as the granular materials. The Physical properties of Sylhet sand can be described as Suitability number ($S_n$)=10, FM= 2.5, $D_{10}$= 0.22, $D_{30}$=0.3, $D_{60}$=0.80, $C_u$=3.64 and $C_c$=0.82.

Considering 100 square meter (sqm) area:

Quantity of SCP = Total no. of boreholes × length of pile = 60 × 8 = 480 rm

Rate of SCP per rm = 930.49 BDT as per LGED (2019-20).

Total cost of SCP = 480 × 930.49 = 4, 46, 635 BDT

**<u>Cost for Prefabricated Vertical Drain</u>**

Drain consisted of a continuous plastic drainage core wrapped in a non-woven polypropylene/polyester geotextile material having discharge capacity at 200 kpa and Hydraulic Gradient of 1 with the following specifications:

Average spacing = 1.5 meter

Average length = 15.0 meter

Sand blanket height = 0.6 meter (FM >2.5)

Surcharge height = as per filling height, 2 meter (FM >1.0)

Considering 100 square meter (sqm) area:

Number of PVD in 100 sqm area = 42

Quantity of PVD required = No. of PVD × Length of PVD = 42 × 15 = 630 rm

Rate of PVD per rm = 268.56 BDT.

Soil Settlement required in 100 sqm area = 100 × 0.18 = 18 cum

Rate of Soil Settlement per cum = 181.44 BDT.

Sand Blanket required in 100 sqm area = 100 × 0.6 = 60 cum

Rate of Sand Blanket per cum = 2735.12 BDT.

Surcharge required in 100 sqm area = 100 × 2 = 200 cum.

Rate of Surcharge per cum = 1065.58 BDT.

Removal of Surcharge per cum = 304.80 BDT.

Total cost for PVD = [(630×268.56) + (18×181.44) + (60×2735.12) + (200×1065.58) + 200×304.80)]

= [1, 69, 192.80 + 3,265.92+ 1, 64, 107.20 + 2, 13, 116 + 60,960] BDT= 6, 10, 641.92 BDT.

**<u>Cost Comparison between SCP and PVD</u>**

Cost of SCP and PVD has been estimated for 100 sqm area for the sake of easy comparison.

Total cost of PVD = 6, 10, 641.92 BDT.

Total cost of SCP = 4, 46, 635.20 BDT.

Difference of cost = (6, 10, 641.92 - 4, 46, 635.20) BDT = 1, 64, 006.72 BDT.

The result evidently shows that PVD cost is (6, 10, 641.92/4, 46, 635.20) = 1.37 times higher cost as compared to that of SCP.

### 4.9 Suitability Comparison between SCP and PVD

Both SCP and PVD are suitable for improving subgrade strength at soft soil. However, when both these two methods were applied on ground some advantages and disadvantages were observed between these two. According to survey data analysis and observation, following comparison between SCP and PVD may be brought out:

a. With the installation of SCP at location 1 and 2, subgrade soil condition was improved and SPT-N values also increased significantly. The SPT-N value raised around 18 from 4 at location 1 and raised around 18 from 3 at location 2 up to the depth (8 meter) where sand piles are driven. (Table 4.15 & 4.20).

b. With the installation of PVD at location 3 and 4, subgrade soil condition was improved and SPT-N values also increased sharply. The SPT-N value raised around

14 from 3 at location 3 and raised around 14 from 3 at location 4 up to the depth examined (8 meter). (Table 4.24 & 4.28).

c. SPT-N values has increased significantly in both cases of SCP and PVD. Average SPT-N value was found around 18 after installation of SCP whereas average SPT-N value was identified around 14 after installation PVD. (Table 4.15, 4.20, 4.24 & 4.28).

d. DCP Index was found to be 240 mm, 305 mm, 225 mm, and 225 mm after 20 blows at location 1, location 2, location 3 and location 4 respectively. Ground improvement measures using PVD resulted in marginally better soil conditions at shallow depth than those achieved using SCP. Average CBR (%) was found around 18.5% where subgrade improvement has been done through SCP. Average CBR (%) was found around 21.5% where subgrade improvement was conducted through PVD. Average CBR (%) is slightly higher where ground improvement was conducted through PVD than that of SCP. Therefore, it is understood that PVD provide slightly more compacted ground surface than SCP at shallow depth. (Para. 4.7.5. iii).

e. The primary advantage of SCP is that the sand used is considerably cheaper when compared to PVD which needs geotextile and other imported materials. In comparison to Prefabricated Vertical Drains, SCP is more economic. In this case, PVD cost is 1.37 times (37%) higher/costly than SCP cost. (Para. 4.8.3).

a. SCP takes less time than PVD. Construction of the sand columns is extremely fast. Several SCP can be constructed very easily within a day and takes around one month for soil settlement whereas PVD takes around seven months due to soil settlement. The process of draining out ground water, soil settlement process, need for surcharge and removal of surcharge cost more time for PVD.

## CHAPTER 5
## CONCLUSION AND RECOMMENDATIONS

## 5.1 General

Rapid growth of population, fast urbanization and large-scale development of infrastructures like buildings, highways, railways and other structures in recent past years has resulted non- availability of good quality land in Dhaka city. Therefore, Engineers had to use soft and weak soil surface by improving their strength through suitable modern ground improvement techniques for construction activities. Various types of ground improvement technique aims to increase the bearing capacity of soil and reduce settlement. The present study scrutinized the effectiveness of such two widely used and popular ground improvement techniques in improving soft soil of pavement subgrade. This was mostly a field study on soft soils at four locations with similar type of soil conditions with different ground treatment in the form of sand compaction pile (SCP) and prefabricated vertical drain (PVD). Before design and construction of SCP and PVD, the SPT tests were conducted at several boreholes to assess the in-situ soil characteristics. After getting initial soil properties, SCPs were installed in two locations in triangular patterns. Similarly, PVDs were also installed in another two locations. Allowing at least 6 months' time after installation of both SCP and PVD, the SPT-N values at different depths were again measured to compare their effectiveness. Then results of four subsoil investigation schemes were evaluated and analyzed. Average CBR percentage was also determined performing dynamic cone penetration (DCP) tests at shallow depths. Test data were analyzed for different soil conditions and cost analyses were done. The following can be drawn from the present study.

### 5.2 Conclusions

The present study shows the following important findings:

a. SPT-N values has increased significantly in both cases of SCP and PVD. That depicts both of ground improvement method applied on ground worked correctly as per site investigation and design. Average SPT-N value was found around 18 after installation of SCP whereas average SPT-N value was identified around 14 after installation PVD. In situations where soil must remain undisturbed and stable to ensure safety, it is not acceptable to disturb the soil through the installation of drains. A type of ground improvement SCP, results in less disturbance during to ground improvement than does PVD. This illustrates that SCP provided better subgrade improvement than that of PVD.

b. Average CBR (%) was found around 18.5% where subgrade improvement has been done through SCP. Average CBR (%) was found around 21.5% where subgrade improvement was conducted through PVD. Thus, it is understood that, average CBR (%) is slightly higher where ground improvement was conducted through PVD than that of SCP. PVD provides slightly more compacted ground surface than SCP at shallow depth.

c. DCP Index was found to be 240 mm, 305 mm, 225 mm, and 225 mm after 20 blows at location 1, location 2, location 3 and location 4 respectively. Ground improvement measures using PVD resulted in marginally better soil conditions at shallow depth than those achieved using SCP.

d. Soil settlement was obtained after installation of PVD. At location 3 and location 4 soil settlement was found 192 mm (7.56 inch) and 164 mm (6.45 inch) respectively after seven months. As PVD drained out trapped water from the ground below, significant soil settlement was obtained.

e. After conducting both SCP and PVD, subgrade conditions improved and become silty fine sand. However, SCP took around one month in improving pavement subgrade. PVD took around seven months in improving subgrade condition according to this research. The process of draining out ground water, soil settlement process, need for surcharge and removal of surcharge cost more time for PVD.

f. In cost comparison the PVD was found 1.37 times higher as compared to SCP. The method of PVD required imported materials for construction in addition to cost of soil settlement, surcharge, removal of surcharge, sand blanket etc. whereas SCP required only cased borehole method of construction and locally available sand. Thus, SCP proved to be cheaper in overall construction process. However, this cost comparison is site specific of the study area and particular to this design of SCP and PVD.

### 5.3 Recommendation for Further Study

The present study was limited to small number of soil samples and study area. To generalize the findings, the following are some of the scopes of further study.

a. The research was limited to a smaller study area just at the outskirt of the city. The study may be expanded taking larger area outside of Dhaka.

b. The SCP was conducted using triangular arrangements in this research with a specific spacing of borehole. Further study should be done using other arrangements of SCP like square, hexagonal at varying spacing.

c. The DCP was conducted in shallow depth in this research. DCP should be done in depth thus comparison between SCP and PVD may be presented well.

d. PVD was done following modern technic of ground improvement. But SCP was conducted in this research in a manual procedure. Further study should be done using modern non-vibratory (silent) SCP method which causes less disturbance on subgrade improvement.

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