Study on a Combined Ground Improvement Technic to Facilitate Rapid Embankment Construction on Soft Soil Deposits

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Dedicated to

My parents & my brother

Abstract

Soft soils are characterized by their high compressibility, and low shear strength. Highway embankments proposed to construct on such soils will undergo excessive consolidation settlements. The failure of embankments on soft soils is attributed also to the undrained shear when the embankments are constructed in a short time. Hence, the construction of high embankments is taken up in stages by maintaining enough waiting period for consolidation between stages due to stage loading, which delays the whole construction period. Among the several ground improvement technics available to reduce the construction period, pre-consolidation of soft soils through the application of surcharge, use of vertical drains are common methods to achieve required degreed of consolidation (usually 90-95%). In any of these methods, time and cost of the project takes a major role in achieving the required degree of improvement. The stability of an embankment is an issue if the embankment is proposed to build in a very short time.

In this research, a combined ground improvement technic is proposed to simultaneously address the compressibility and shearing resistance issues of soft soils. Prefabricated vertical drains (PVD) along with in-situ deep soil mixing (DSM) columns are proposed to reduce the shear failure while improving the consolidation behavior of soft soil simultaneously. Stress analysis and deformation analysis have been performed to understand the respective behaviour at various locations in the foundations soil. Various area replacement ratios of DSM columns are considered in order to study the consolidation behaviour of the treated ground along with constant PVD spacing (1, 1.5, 2m). Stability analysis (Ø-c reduction analysis) for untreated, PVD treated and PVD-DSM treated ground considered in this research. The influence of construction time on settlement of soft soil was also addressed in this study. Numerical analysis of combined ground improvement technic is discussed. The model is validated with an embankment constructed at second Bangkok International Airport (SBIA).

Nomenclature

c	Cohesion
φ	Friction angle
ψ	Dilatancy angle
E	Young’s modulus
ν	Poisson’s ratio
γ	Density
σ	Stress
l_d	Length of the drain
k_h	Co efficient of Horizontal Permeability
k_v	Co efficient of Vertical Permeability
k_{ve}	Equivalent vertical permeability
c_c	Compression index
c_s	Swell index
e_0	Initial void ratio
p^{l}_{f}	Final stress (overburden + change in total stress)
p^{l}_{s}	Initial stress (initial overburden pressure)
A_r	Area replacement ratio
U_v	Consolidation in vertical direction
U_h	Consolidation in vertical direction
U_{vr}	Average degree of consolidation
c_v	Co efficient of vertical permeability
c_h	Co efficient of horizontal permeability
n	Spacing ratio
F (n)	Spacing influence factor
d_e	Diameter of the equivalent soil cylinder
d_w	Equivalent diameter of the drain
s	Centre to centre spacing
t	Time for consolidation
a	Width of the PVD
b	Thickness of PVD
H	Thickness of the foundation soil
T_v	Dimensionless time factor for vertical flow

T_h	Dimensionless time factor for horizontal flow
d	Diameter of the DSM columns
s_t	Settlement at any time
s_f	Ultimate settlement

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Chapter 1

Introduction

1.1 Introduction

The consolidation settlement of soft clay creates a lot of problems in foundation and infrastructure engineering. The inferior characteristics of soft clays such as high compressibility and low shear strength and low permeability may result in excessive settlements in the foundation soil as well as responsible for prolonged primary consolidation settlements [1].

To shorten this consolidation time, vertical drains are installed along with preloading by embankment and surcharge. Vertical drains are artificially created drainage paths which can be installed by one of several methods and which can have a variety of physical characteristics. Initially, sand drains have been used extensively as vertical drains. Recently, artificially manufactured vertical drains called prefabricated vertical drains (PVDs) came into existence [2]. When vertical drains are used, much of the water flow is horizontal, although the compression is vertical. In this method, pore water squeezed out during the consolidation of the clay. Thus, the installation of vertical drains in clay reduces the length of the drainage path, thereby, reducing the time to attain the desired amount of consolidation. Therefore, the purpose of vertical drains is to accelerate the consolidation process of the clay subsoil.

However, vertical drains can only accelerate the consolidation process and seldom improves the shearing resistance of the foundation soil. At times, speedy surcharge may cause shear failure. In these situations, the embankment/surcharge loads are applied in stages while maintaining a large consolidation period between consecutive stages to allow for dissipation of excess pore water pressures. Sand drains/PVDs together with preloading are considered as the most cost effective solution for the consolidation of saturated compressible soils [3]. Experimental and numerical modeling of consolidation by vertical drains supporting embankments were analysed by several researchers [2-4].

In recent years, the reinforcement of weak foundation soils with deep soil mixing (DSM) columns has expanded a great deal around the world, which allows increase in stability, reduction of settlements, greater speed of execution and reduced cost [5]. DSM is a ground improvement technic in which soft soils are strengthened by mixing them with the grout materials such as quicklime, cement, lime-cement or ashes in proper proportions forming in-situ soil cement columns [6]. The lateral pressure and shear stress can be exerted on surrounding clays during the installation of DSM columns [7]. Thus improving the load bearing capacity of the foundation soil. The application of DSM columns are wide including the retaining structures, column supported embankments, bridge abutments etc. [6, 8].

1.2 Combined Ground Improvement Technic

The issues with the rapid embankment construction on soft soils as discussed above would be the slower consolidation process and thus the gain in shear strength of the soil. This will lead to a failure of the structure. Figure 1.1 shows the typical highway embankment failures from West Bengal, near Kolkata during the construction of final loading stage and right after the embankment construction (before open to the traffic). This kind of failures can be avoided by careful understanding of the undrained behavior of saturated soft soils. In this study, an attempt has been made to combine two ground improvement technics those were discussed earlier. The combined PVD-DSM method can effectively utilize the improved lateral resistance and shear stress along with the radial consolidation. It effectively combines the two independent technics of DSM and vertical drain method into a new technic. A remarkable combined method of the dry jet mixing with vertical drains (DJM-PVD combined method) was successfully practiced in a highway project on very soft clay in Jiangsu, China. This combined method could reduce the project budget (about 35%) compared with the traditional DJM treated ground [9]. It was concluded that since the embankment was constructed in layers short stabilized columns were sufficient in satisfying the stability requirements under normal filling rate. Lately, in a limited field study, Ye et al., [10] have experimented with short DSM columns to improve shallow soft soils in Yancheng City, Jiangsu Province, China, while long prefabricated vertical drains (PVDs) were inserted between DSM columns to promote the consolidation of deep soft soils under the embankment load.

Finally in this technic both DSM columns and prefabricated vertical drains are introduces together to ensure the acceleration of the consolidation of normally consolidated soft clay layers as well as to increase the undrained shear strength of the foundation soil.

Figure 1.1: Failure during final stage of the embankment

1.3 Benefits of combined ground treatment with PVD-DSM

Due to installation of DSM columns, lateral pressure and shear stress will exert. The PVD-DSM method can effectively utilize the lateral pressure and shear stress. It effectively combines the two individual technics DSM and vertical drain methods into a new technic.

The PVD-DSM combined method utilizes the lateral pressure and shear stress to powerful way of accelerating the consolidation of surrounding clays through the vertical drain. It can effectively reduce the post construction settlement and increase the stability of embankment. Consequently, effectively increases the strength of surrounding clays.

1.4 Objective of the study & Definitive objectives

The main aim of this study is accelerate the construction of high embankments on soft soils for infrastructure development.

Definitive objectives

- Improvement of consolidation behaviour of the soft ground by introducing prefabricated vertical drains.
- Improvement of stiffness of the soft ground by introducing Deep Soil Mixing Columns.
- Thus, to attempt a combined PVD-DSM technic to simultaneously improve the soft clay characteristics.

1.5 Scope of the study

To achieve the definitive objectives outlined above the following studies are required to perform:

Theoretical design methodology to design PVDs and DSM configurations for a given soil condition individually and to obtain the combined PVD-DSM improved ground properties to estimate the efficacy of the combined method

Numerical simulations to validate the model with the field data and to perform stability analysis of the embankment treated with combined PVD-DSM method. Numerical study is also important to perform parametric study to verify the influence of individual PVD and DSM method's influence on the stability and consolidation behaviour of the system.

1.6 Thesis organization

Chapter 1 - (Introduction) provides a brief understanding of soft soils and consolidation behaviour of PVD treated soft ground and brief introduction into combined PVD-DSM treated ground.

Chapter 2 - (Literature Reviews) gives a summary of the background of various technics adopted to accelerate the consolidation behaviour of the soft ground. Methods to improve shear strength of the soft ground to withstand the rapid embankment construction.

Chapter 3 - (Design Methodology) presents the fundamental design of prefabricated vertical drains and deep soil mixing columns and combined PVD-DSM treated ground to achieve the average degree of consolidation at different time periods.

Chapter 4 - (Numerical Modeling) deals with the material models and their material parameters used in the numerical analyses to simulate the mechanical behaviour of the soil and method of numerical analysis. Numerical modeling of untreated, PVD treated and PVD-DSM embankment using PLAXIS and numerical design parameters have been presented to simulate one real embankment on Bangkok soft clay.

Chapter 5 - (Results and Discussion) presents the model validation of an embankment constructed at second Bangkok international airport area with the treatment of PVDs. Also presents the stability analysis of present model and parametric studies based on various DSM area replacement ratios with changing c/c PVD spacing. Parametric study results on the influence of construction time on settlement of soft soil have been presented.

Chapter 6 - Summary and Conclusions.

Chapter 2

Literature Review

2.1 Introduction

Developing infrastructure on soft soils is always a challenging task for Engineers in design stage as well as construction/execution stage. As discussed, soft soils are characterized by their high compressibility, low shear strength and low permeability, they offer poor support to the infrastructure. Several ground improvement technics are available in practice to improve the detriment effects of soft soils those include preloading, surcharge, vertical drains, vacuum consolidation, mass stabilization using chemicals, additives etc. and column supports. These treatment methods can be divided in to two categories to mainly improve either the consolidation properties or the shear strength properties or both.

In general, the embankment construction is taken up in stages. Ample time is left out between construction stages to avoid failures during the construction. The conveniently left time period between stages allow the excess pore water pressure to dissipate to a maximum extent and gain the undrained shear strength to some extent. Generally, the soft soil deposits have been treated by individual ground improvement methods noted above. Each method has its own merits and demerits. The most common method of ground improvement in soft soils would be vertical drains with preloading and combination of vacuum consolidation with vertical drains [3 &11]. Prefabricated vertical drains together with surcharge and preloading are considered to as the most cost and time constrain effective solution for the consolidation of saturated compressible soils [2]. However, not much information is available on combined ground improvement technics which can address the consolidation and shear strength properties of soft soils simultaneously in the literature. Hence there is a need to address this issue. Following sections clearly describe each method of improvement for soft soils.

2.2 Problems associated with soft soils

The construction industry is constantly facing challenges with soft soil deposits. Soft clay deposits have a very low bearing capacity, highly compressible and excessive settlement characteristics. The strength development of soft soil is time dependent. These clay deposits are commonly widespread in the coastal areas and major river valleys, of varying thickness, ranging from 5 m to 30 m. (Bujang B. K Huat) [12]. Surface loadings in the form of embankments inevitably results in large settlements.

One of the very good example showed in the below two Figures 2.1 & 2.2. Here two highway embankments named k26 and k18 located near the Hooghly River Kolkata, India failed recently. As the structures were founded on sensitive, soft and compressible, fine-grained soils [13], these two highway embankments appeared to have failed due to the main factor contributing to the incomplete consolidation of the foundation soil. In the below sections some of the methods have been discussed which will accelerates the consolidation process in the foundation soil.

Figure 2.1: Failure during final stage of the embankment

Figure 2.2: Failure of embankment right after construction

2.3 Methods to accelerate the consolidation of the soft soil

Some of the available methods to improve the consolidation behaviour of the soft ground have been in the following sections.

2.2.1 Preloading with surcharge

Preloading is the application of surcharge load on the site prior to construction of the permanent structure, until most of the primary settlement has occurred. Even with high surcharge load, the total consolidation time is very long due to the low permeability of the soft soil. Therefore, the application of preloading alone may not be possible with tight construction schedule. In the below Figure 2.3 illustrates the typical preloading criteria.

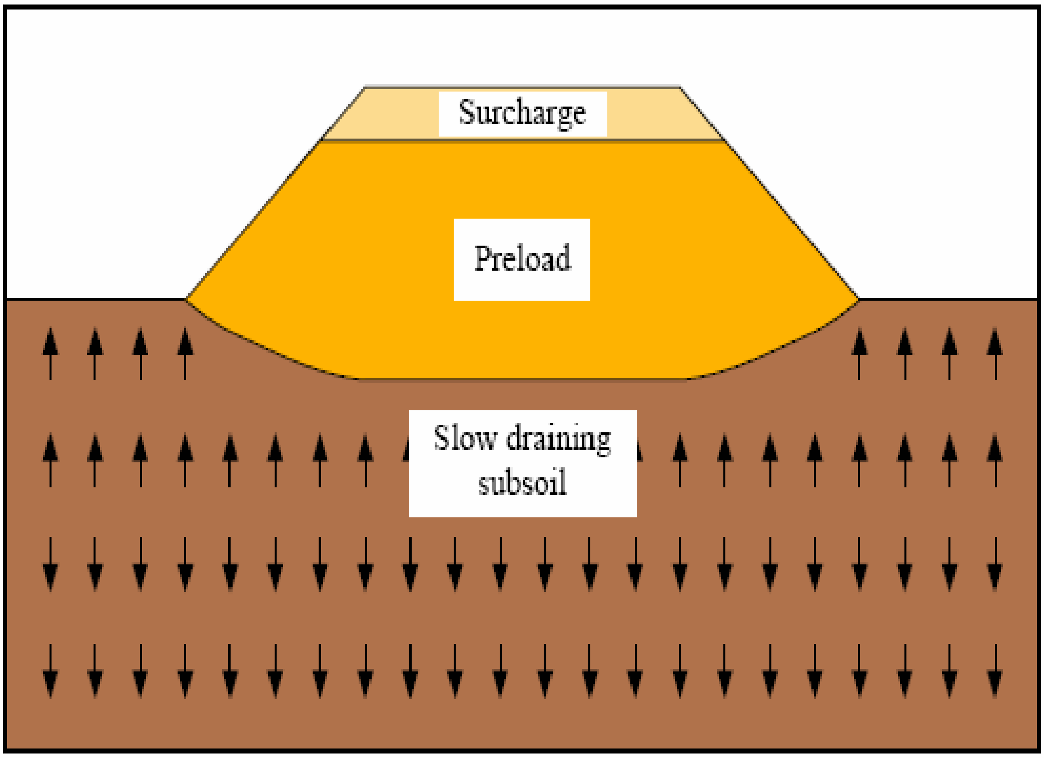

Figure 2.3: Preloading and surcharge to achieve the consolidation settlements

2.2.2 Vacuum consolidation

Vacuum preloading method was first introduced by Kjellman [14] to improve the strength of soft clays. Vacuum consolidation method is a technic of applying vacuum pressure to the soil mass, which will reduce the atmospheric pressure inside the soil; at the same time reducing the pore water pressure in the soil as a result the effective stress will increase.

When a vacuum load is applied, the negative pore water pressure in the soil generates. As the applied total stress is constant, the effective stress in the soil increases due to the suction generated. Gradually, the pore pressure decreases and the spring start to compress, hence, the soil skeleton gains in effective stress. (C. Rujikiatkamjorn et al.,) [11]. Using a vacuum pressure to consolidate a soil deposit has several advantages over embankment loading, e.g., no fill material is required, construction periods are generally shorter and there is no need for heavy machinery. However, there are still differing opinions regarding the important characteristics of vacuum consolidation. Vacuum consolidation can result in settlements nearly identical to those induced by a corresponding applied surcharge loading [15]. But in the case of huge area application of vacuum pressure is expensive. Figure 2 shows the typical operation of vacuum consolidation. In the below Figure 2.4 a typical vacuum consolidation process has been presented.

Figure 2.4: Vacuum consolidation to achieve the consolidation settlements

2.2.3 Vertical drains

2.2.3.1 Introduction

In general, sand drains and prefabricated vertical drains are used in the field to accelerate the consolidation settlement in soft normally consolidated clay layer(s), and to achieve the pre compression before the construction of a desired foundation.

Sand drains are constructed by drilling holes through the clay layer(s) in the field at regular intervals. The holes are then back filled by sand. After backfilling the drill hole with sand, a surcharge is applied at the ground surface. This surcharge will increase the pore water pressure in the clay. The excess pore water pressure in the clay will be dissipated by drainage both vertically and horizontally to the sand drains. Prefabricated vertical drains (PVDs), which also referred as wick drains. These drains are manufactured from synthetic polymers such as poly propylene and high density polyethylene. PVDs are normally manufactured with corrugated or channeled synthetic core enclosed by a Geosynthetics filter as shown schematically in the Fig. 2.5.

Therefore, the vertical drain installation reduces the length of the drainage path and, consequently, accelerates the consolidation process.

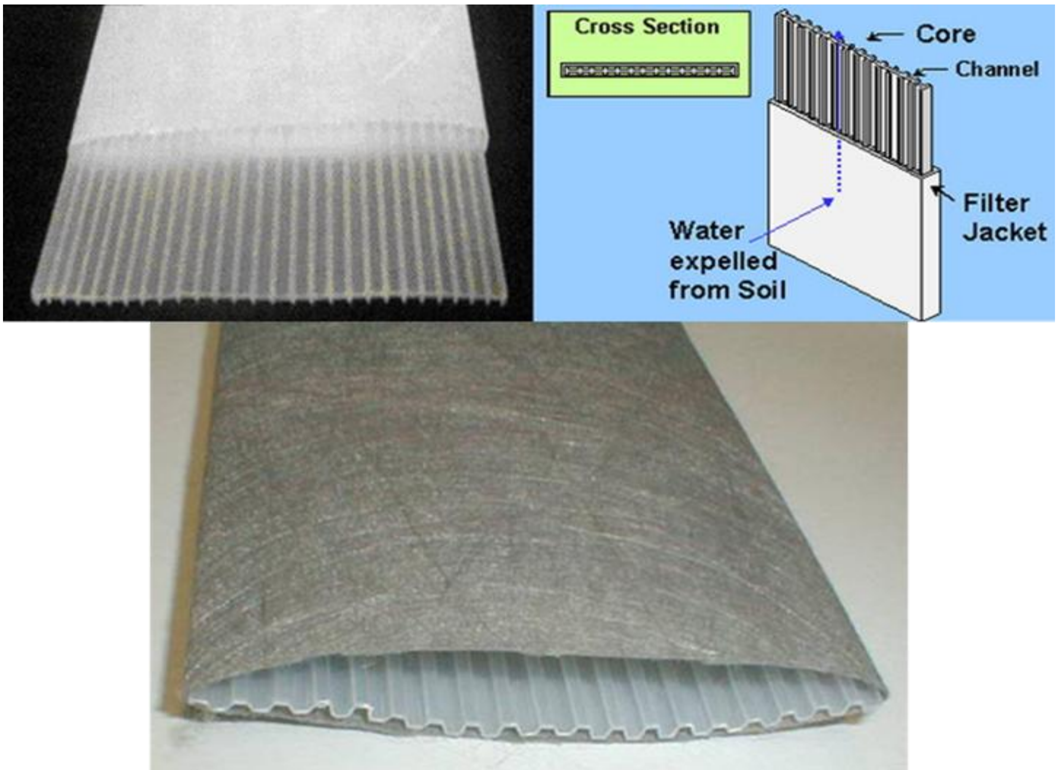

Figure 2.5: Typical picture of PVDs

Factors influencing the drain efficiency

2.2.3.2 Smear zone

Different relationships have been proposed to determine the size of the smear zone. For design purposes the diameter of the smear zone (d_s) and the cross sectional area of the mandrel can be related as, d_s = 5 to 6 times $d_m/2$ (Jamiolkowski and Lancellotta) [16]. Where (d_m) is the diameter of a circle with an area equal to the cross sectional area of the mandrel or the cross sectional area of the anchor at the tip whichever is greater. Based on laboratory investigations, the ratio of (d_s / d_m) to be four to five [17].

2.2.3.3 Size and shape of mandrel

In general, the disturbances increase with increasing cross sectional area of the mandrel. Therefore, in order to reduce disturbances, the mandrel size should be as close as possible to that of the drain some researchers reported from a case study where the installation of drains was carried out using a small mandrel in one half of the site and a large mandrel in the other half [18]. The results indicated a faster settlement rate and a slightly higher compression in the small mandrel area. That would verify that a smaller smear zone was developed in the vicinity of the smaller mandrel.

2.2.3.4 Influence zone of vertical drains

Vertical drains are commonly installed in square or triangular patterns as illustrated in Figure blow. The influence zone of the drain (R) is a controlled variable, since it is a function of drain spacing (S) as given by:

R =0.546*S (for drains installed in a square pattern)

R =0.525 *S (for drains installed in a triangular pattern)

The square pattern is more convenient to layout and to control in the field. However, a triangular pattern is usually preferred since it provides a more uniform consolidation between drains than the square pattern [19]. The details of drain pattern and zone of influence can be seen in Figure 2.6.

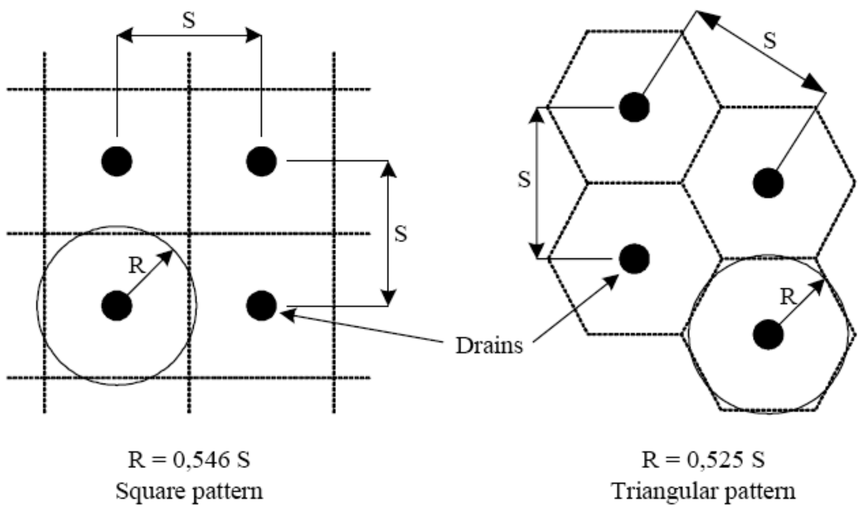

Figure 2.6: Plan of drain pattern and zone of influence

2.2.4 Equivalent vertical permeability of the foundation soil

Vertical drains increase the mass permeability in vertical direction. Therefore, it is possible to establish an equivalent vertical permeability, k_{ve}, approximately represents the effect of both the vertical permeability of natural subsoil and radial consolidation by vertical drain. Finally, the equivalent vertical permeability, k_{ve}, proposed by Chai and Miura (2001) [20] can be expressed as:

$$k_{ve} = 1 + \frac{2.26 l_d^2}{\mu d_e^2} \frac{k_h}{k_v} k_v \quad \text{and} \qquad \mu = \ln \frac{n}{s} + \frac{k_h}{k_s} \ln s \quad \frac{3}{4} + \prod \frac{2 l_d^2 k_h}{3 q_w}$$

Where,

d_e= diameter of the influence zone of PVD.

d_w = [2(a+b)/π] = equivalent diameter of PVD.

where, a & b are Thickness and width of the PVD.

$n = d_e/d_w$ = spacing influence factor (or spacing ratio) of PVD.

$s = d_s/d_w$ = smear disturbance ratio of PVD.

l_d= length of the PVD.

k_h & k_v = horizontal and vertical permeability.

k_s= smear zone permeability.

q_w= discharge capacity of PVD.

2.2.5 PVD design steps

The design of PVDs for a given soft soil condition can be done using a trial and error method. The design steps are briefly given below.

1. Calculate T_v; for a given c_v, H of the soil strata, and time, t required for complete consolidation
2. Assume an average degree of consolidation due to radial and vertical drainage, U_{vh} = 0.95 or 0.99
3. Find U_h from steps 1 & 2. Use $U_{v,r} = 1-(1-U_h)(1-U_v)$
4. Assume some arbitrary spacing s and calculate d_e, n, F(n) and T_h (use $T_h = c_h t/d_e^2$)
5. Then, find U_h from the equation given by Hansbo [21], $U_h = 1-\exp(-8T_h/F(n))$
6. Compare U_h from step 5 with step 3.

7. If they are not equal, change the spacing and repeat step 5. When U_h matches with that calculated in step 3, then that is the design spacing.

2.3 Methods to improve the shear strength of soft soil

2.3.1 Piling methods

Pile foundations are adopted generally in the following situations:

- Low Bearing Capacity of soil
- Non availability of proper bearing stratum at shallow depths.
- Heavy loads from the super structure for which shallow foundation may not be feasible.

Classification of piles

Classification of piles is based on the material type, method of construction and load transfer mechanicsm as listed below. Description of these methods is not detailed here as the discussion is out of the scope of the study.

Based on material,

- Concrete piles
- Steel piles
- Timber piles

Based on method of construction/installation

- Driven /Displacement Pre cast Piles.
- Driven/Displacement Cast in Situ Piles.
- Bored/ Replacement Pre cast piles.
- Bored/ Replacement Cast in situ piles.

Based on load transfer mechanism

- End bearing piles
- Friction/Floating piles
- Bearing cum Friction piles

2.3.2 Stone columns

Stone columns are used to improve the bearing capacity of soft soils. The construction of stone column is carried out either by Replacement Method or by Displacement Method (also known as wet method and dry method) [22]. The mode of stone column and relative theories of failure are well documented by Greenwood (1970), Madhav and Vitkar [23, 24]. A stone column may fail due to (a) Shallow shear failure (b) Bulging – Plastic failure and (c) Shear failure in end bearing or skin friction. In case of overload, columns automatically relieve the stress as it deforms. A typical stone column tends to perform the following function [25];

- Reduce settlement by reinforcement soil.
- Mobilizing the drag forces during initial stage.
- Accelerating consolidation process.

2.3.3 Deep soil mixing columns (DSM)

2.3.3.1 Characteristics of DSM columns

The Deep Soil Mixing is a process to improve soil by injecting grout through augers that mix with the soil, forming in-place soil-cement columns as shown in Figure 2.7.

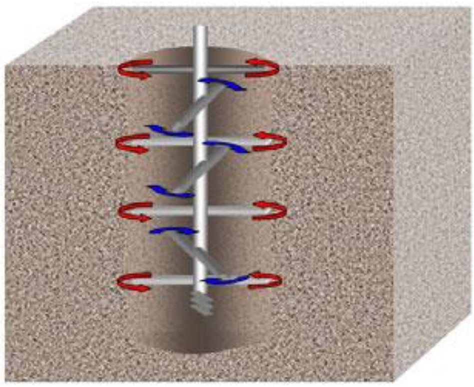

Figure 2.7: Deep soil mixing (DSM) operation [6]

In DSM technic soft soils are strengthened by mixing them with the grout materials such as quicklime, cement, lime-cement or ashes in proper proportions forming in-situ soil cement columns [6]. These columns act as reinforcement to the weak soil stratum and absorb major portion of the load coming on to it. Typical DSM installation has been showed in the below Figure 2.8.

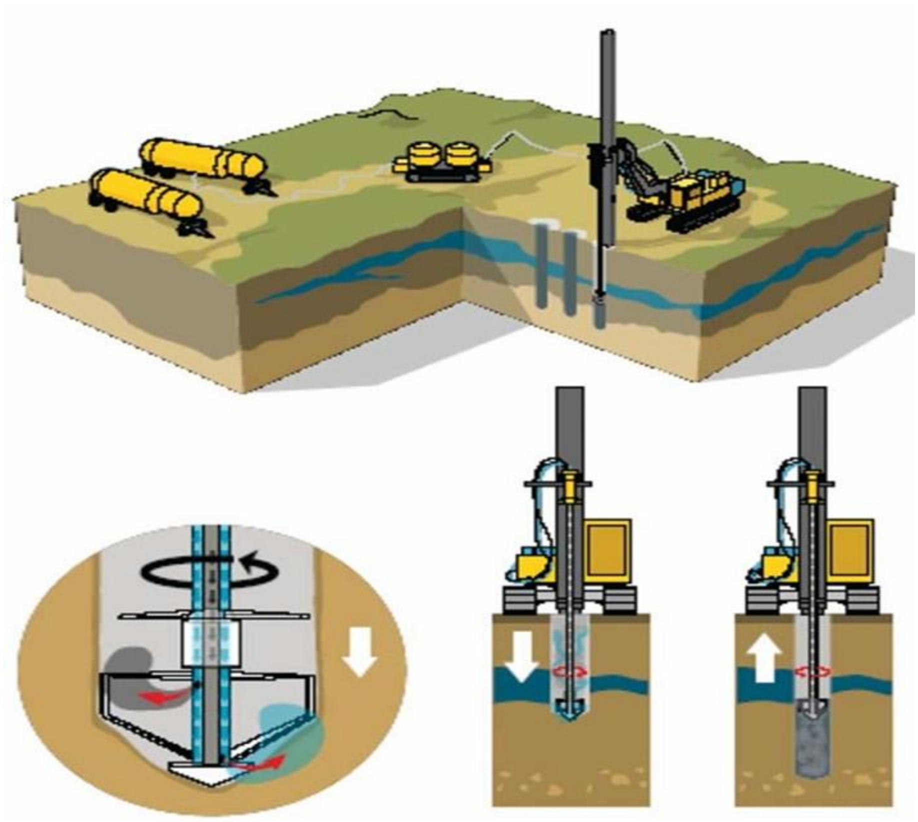

Figure 2.8: Typical Installation process of the deep mixing method

2.3.3.2 Applications of DSM Columns

1. Increasing bearing capacity of sub-grade for structures.
2. Controlling heave in soft clays.
3. Prevent soil liquefaction during earthquakes.
4. Excavation support / installation of temporary or permanent retaining walls.

2.3.3.3 Advantages of DSM Columns

1. Reduced vibration – Method induces very low vibrations, which reduces the potential impact to nearby utilities.
2. Time saver – process is quick.
3. Good amount of strength can be achieved in difficult soil conditions.

2.3.3.4 Design methodology of DSM columns

The design of deep soil mixing columns is based on area replacement ratio (A_r, %) and center to center spacing of the DSM columns. The following set of equations can be used to calculate area replacement ratio and spacing between columns based on the pattern chosen.

$$A_r = \frac{A_{col}}{A_{soil} + A_{col}}$$

$$\frac{Area\ \ of\ column}{Area\ of\ \ square} = \frac{\Pi d^2 / 4}{s \times s}$$

$$s = \sqrt{\frac{\Pi}{4A_r}} d_{col}$$

2.3.4 Benefits of column supported embankment

Abusharar et al., [27] proposed that Multi-column support allows for a faster rate of consolidation and significantly increases embankment stability. Often, due to time constraints involved in construction and uncertainty of underlying soil conditions, the use of pile supported embankment is regarded as the most practical and economic option [28]. Multi-column support allows for a faster rate of consolidation and significantly increases embankment stability. Multi-column ground treatment can significantly reduce total and differential settlements and restrict the lateral movement of the embankment; as a result, the stability of the embankment can be improved [27].

2.3.5 Combined Ground Improvement Technic

As discussed, not many combined technics are available for simultaneously address the consolidation and shear strength issues of soft soils. Few studies are available using combined ground improvement technics for soft soils which include vertical drains in combination with surcharge loading, vacuum consolidation in combination of vertical drains [29]. Both these methods are designed to address the consolidation behavior of the soft soils and to accelerate the consolidation settlements. Both the methods have their own merits and demerits. Very few studies are available addressing the consolidation and shear strength issues simultaneously. Few studies available in this connection are addressed below.

For treatment of thick soft sub soil, Ye & Xu [10] suggested a combined method of DSM column and preloading with prefabricated vertical drains (PVDs). With this method, a short cement column is used to stabilize the upper soft soil, and long prefabricated vertical drains penetrate into the deep soft subsoil. In the process of embankment construction and preloading period, the deep soft soil can be consolidated under the embankment.

During the installation of DSM columns, the lateral pressure and shear stress can be exerted on surrounding clays. The DSM-PVD combined method can effectively utilize this lateral pressure and shear stress. It effectively combines the two independent technics of DSM and vertical drain method into a new technic.

The DSM-PVD combined method utilize the lateral pressure and shear stress as a powerful way of accelerating the consolidation of surrounding clays through the vertical drain, consequently effectively increasing the strength of surrounding clays. A remarkable combined method of the dry jet mixing with vertical drains (DJM-PVD combined method) is innovated and successfully practiced in a highway project on very soft clay in Jiangsu, China. This combined method can reduce the project budget (about 35%) compared with the traditional DJM treated ground [9]. Lately, in a limited field study, Ye et al., [30] have experimented with short DSM columns to improve shallow soft soils in Yancheng City, Jiangsu Province, China, while long PVDs were inserted between DSM columns to promote the consolidation of deep soft soils under the embankment load.

2.3.6 Summary

The literature review reveals that a limited knowledge is available on combined ground improvement technics those can address the consolidation and shear strength issues of soft soils. Review also shows that there is a pressing need to systematically study the combined ground improvement technic to be adopted in difficult soft soil conditions.

Chapter 3

Design Methodology

3.1 Introduction

In this chapter the design of prefabricated vertical drains and deep soil mixing columns are discussed. The design of combined PVD-DSM technic is also discussed. The data obtained from this chapter is important in numerical simulations and theoretical prediction performed in the coming chapters. Following sections describe the design aspect of PVD and DSM columns.

3.2 Design of Prefabricated Vertical Drains

The problem of designing a vertical drain scheme is to determine the drain spacing which will give the required degree of consolidation in a specified time for any given drain type and size. The design procedure would consist of the following steps.

1. Calculate time factor (T_v) for a given coefficient of vertical permeability (c_v), height of the clay layer (H), and time (t).

2. Assume an average degree of consolidation due to radial and vertical drainage, U_{vh} = 0.95 or 0.99.

3. Find U_h from steps 1 & 2. Use U_{vr} = 1-(1-U_h)*(1-U_v).

4. Assume spacing s, calculate d_e, n, F (n) and T_h (use c_h*t/de^2).

   Where,

   Influence zone diameter (de) = 1.13s (for square pattern of drains).

   = 1.05s (for triangle pattern of drains).

   F (n) = ln (n) – 0.75 (Spacing influence factor).

Spacing influence factor or spacing ratio (n) = d_e / d_w.

The equivalent diameter of PVD, d_w can be obtained from equation proposed by Hansbo [22]:

$d_w = [2(a+b)/\pi]$.

Where, a = width of the PVD and b = thickness of the PVD

5. Then, find U_h from the equation, $U_h = 1\text{-exp}(-8T_h/F(n))$.

6. Compare U_h from step 5 with step 3.

7. If they are not equal, change the spacing and repeat step 5 until the value U_h calculated in Step 5 matches with the calculated U_h from Step 3. This spacing is adopted as the design spacing of the PVD.

Where, Tv = Time factor; c_v = degree of consolidation; H = height of the clay layer; t = time required to achieve given degree of consolidation; U_v, U_h = degree of consolidation in vertical and horizontal directions respectively; s = c/c spacing of PVDs.

3.3 Design of DSM Columns

The design of deep soil mixing columns is based on the area replacement ratio (A_r, %) and center to center spacing of the DSM columns. The following set of equations can be used to calculate the area replacement ratios and spacing between columns based on the pattern chosen as shown in the Figure 3.1.

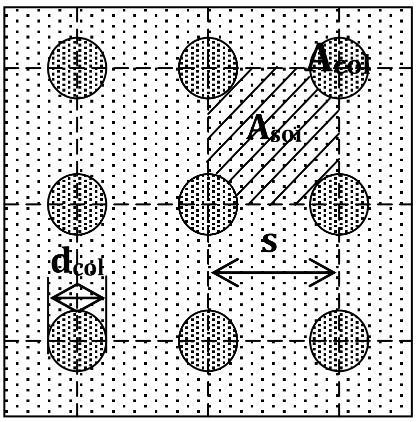

Figure 3.1 Rectangular Pattern of DSM Installation

$$A_r = \frac{A_{col}}{A_{soil} + A_{col}}$$

$$\frac{Area\ \ of\ column}{Area\ of\ \ square} = \frac{\Pi d^2 / 4}{s \times s}$$

$$s = \sqrt{\frac{\Pi}{4A_r}} d_{col}$$

Based on the area replacement ratio ($A_{r,}$ %) derived by the above equation and for various diameters of the DSM columns, the optimum spacing of DSM columns can be determined. The area replacement ratios of DSM columns were varied from 1 to 10% in this study. However, the area replacement ratio of 40 % has been shown to understand the behavior of the heavily occupied DSM columns in a soft ground. It is postulated that the area replacement ratios more than 10% is not amicable in terms of cost of the project and to allow the soft soil to undergo consolidation. For all practical purposes the diameter of DSM columns is varied between 0.3 to1.2 m. Figure 3.2 shows the variation of design spacing of DSM columns and the area replacement ratios for different sizes of DSM columns.

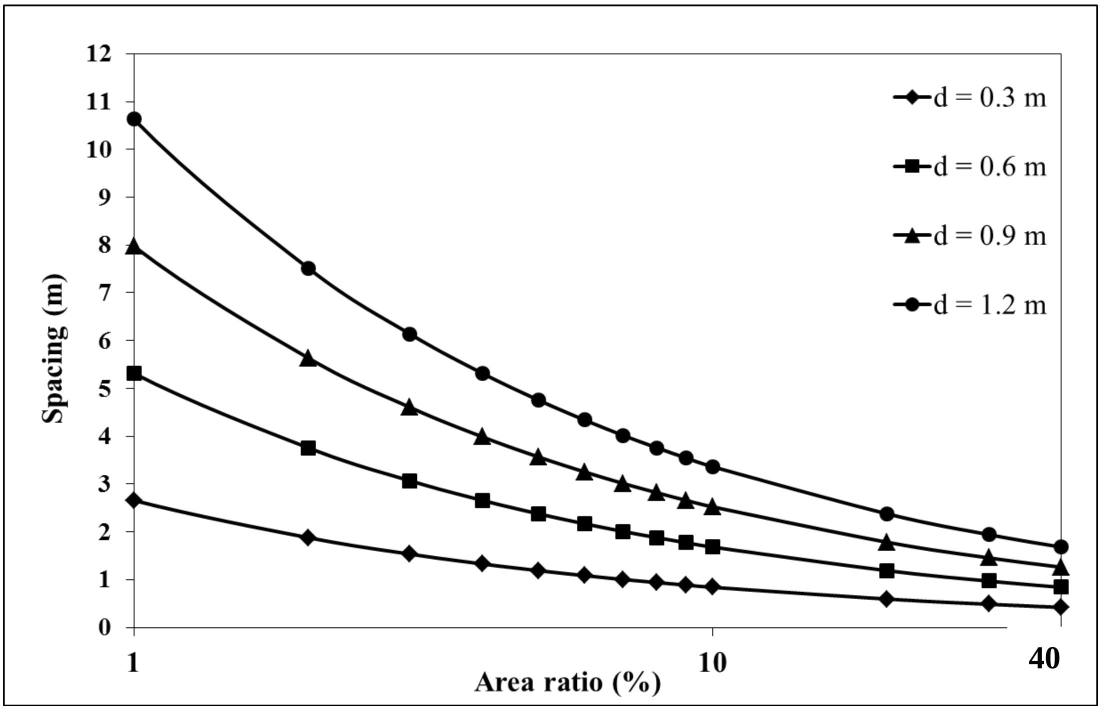

Figure 3.2: Variation of area replacement ratio with Spacing of DSM columns under typical range of column diameters

3.4 Design of Combined PVD-DSM Treatment

It is proposed to first simulate the foundation soil with the equivalent vertical permeability to obtain the time –settlement patterns. Similarly, the combined DSM-PVD treated ground properties can be calculated to obtain the combined time-settlement plots.

The ultimate consolidation settlement of a composite ground can be determined from the Fredlund and Rahardjo [31] work:

$$\Delta h = \frac{C_{c,composite}}{1 + e_{0,composite}}(h)\log\frac{p'_f}{p'_{c,composite}}$$

Where, $c_{c,\ composite}$, $e_{0,\ composite}$, p_f, p_c, and h are compression index of the composite ground, initial void ratio of the composite ground, final stress (overburden ± any changes in total stress), initial stress (overburden pressure), and thickness of the clay layer.

To determine the unknown parameters in the above equation, the following set of equations developed in the next set of sections can be used.

3.4.1 Finding the Combined Equivalent Parameters for Composite Ground

The combined equivalent parameters for the composite ground have been presented below. These combined equivalent parameters can directly be incorporated in to the equations proposed by Fredlund and Rahardjo, [31] to obtain the settlement of the composite ground.

$c_{c,\ composite} = c_{s,\ column} * a_r + c_{c,\ soil} * (1- a_r)$

$e_{0,\ composite} = e_{0,\ column} * a_r + e_{0,\ soil} * (1- a_r)$

3.4.2 Finding the Individual Stresses on Soil and DSM Columns

By using equilibrium equations and compatibility conditions one can determine the unknown values of stresses acting on soft soil and DSM columns due to the embankment loading.

3.4.2.1 Equilibrium equations

$Area_{soil} * \sigma_0 = Area_{col} * \sigma_{col} + \sigma_{soil} (Area_{soil} – Area_{col})$ ------------------------------------- (1)

Where, σ_0 is the stress coming from the embankment loading

3.4.2.2 Compatibility conditions

Settlement in DSM columns (S_{col}) = Settlement in clay (S_{soil})

$\sigma_{col}/ E_{DSM} = \sigma_{clay}/ E_{clay}$

$\therefore \sigma_{col} = \sigma_{clay} (E_{DSM}/ E_{clay})$ --- (2)

From the above relations, one can find the two unknown stresses of DSM column (σ_{col}) and the soft clay (σ_{soil}).

By incorporating all the composite parameters and the stress acting on the soft clay, one can come up with the ultimate settlement of the composite ground. From this known value it is possible to draw the graph between design time versus the degree of consolidation.

Degree of consolidation (U %)

U % = [Settlement at any time (S_t) / Ultimate settlement (S_u)] * 100

$\therefore S_t = S_u * U$.

3.5 Design parameters for combined PVD-DSM treatment for SBIA case study

3.5.1 Design of c/c PVD spacing

The soil properties available from the literature [32] for Second Bangkok International Airport are:

Height of the clay layer H = 10m.

Time t = 1.5 years.

Average degree of consolidation U_{av} = 99%.

Size of the PVD = 100X4 mm.

Equivalent diameter of PVD = a+b/π = 100+4/2 = 0.051m.

Further the required parameters and properties of the soil are calculated here:

The coefficient of consolidation, c_v of this soil was back calculated and found to be 3.9 m^2/day

Time factor can be calculated as, $Tv = C_v * t/ H^2$

Where,

H is the thickness of clay layer.

Therefore $T_v = (3.9*0.5)/ (10)^2$

T_v= 0.0585

From the T_v vs U_v graph U_v = 27% i.e., 0.27

To design of the c/c spacing between two vertical drains,

$U_{av} = 1- (1-U_V) (1-U_h)$

$U_h = 1- (1-U_{av})/ (1-U_V)$

$\therefore U_h = 98.6$ %

Based on spacing value the formula for U_h is $U_h = 1- \exp [-8T_h/F (n)]$

$T_h = C_{h*}t/ d_e^2$

$d_e = 1.13S$ (for square pattern of drains)

$d_e = 1.13$ (1m) = 1.13 m.

$d_e^2 = (1.13*1)^2$

$d_e^2 = 1.27\ m^2$

$c_h = c_v$ (assumed as discussed by (Rixner et al.) [33])

$T_h = C_{h*}t/ d_e^2$

$T_h = 4.6$

and $n = d_e/d_w$

n = 1.13/0.052 = 21.7

n= 21.7

F (n) = ln (n) - 0.75 = 2.32

From these known values of T_h and F (n) one can come up with the U_h value based on assumed spacing,

$U_h = 1- \exp [-8T_h / F (n)]$

$U_h = 1 - \exp [-8(4.6)/2.32]$

Uh = 99% $\longrightarrow$ (ii)

Equations (i) and (ii) are equal. So our assumption is correct.

$\therefore$ Design spacing s = 1m c/c.

Design chart to find U_v and U_h values based on T_v, T_h has been presented below Figure 3.3.

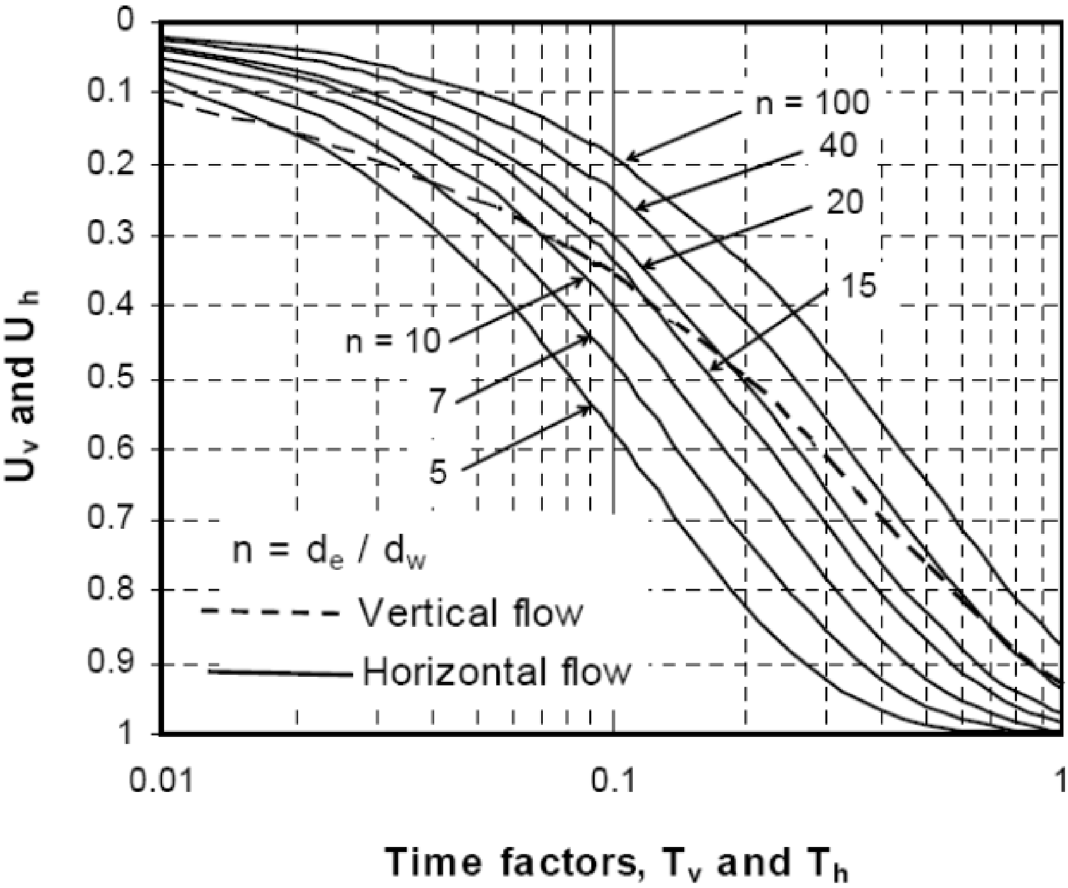

Figure 3.3: Variation of U_v and U_h with Time factors (T_v and T_h)

3.5.2 Design of DSM columns

The design of deep soil mixing columns is based on area replacement ratio (A_r %) and center to center spacing of the DSM columns.

$$A_r = \frac{A_{col}}{A_{soil} + A_{col}}$$

$$\frac{Area\ \ of\ column}{Area\ of\ \ square} = \frac{\Pi d^2 / 4}{s \times s}$$

$$s = \sqrt{\frac{\Pi}{4A_r}} d_{col}$$

Assume A_r = 10 % and column diameter = 0.6 m.

$$s = \sqrt{\frac{\Pi}{4(0.1)}}(0.6)$$

∴ c/c DSM column spacing = 1.1 m

3.5.3 Combined equivalent parameters for PVD-DSM improved ground to achieve ultimate consolidation settlement

3.5.3.1 Equilibrium equations

$A_{soil} * \sigma_0 = A_{col} * \sigma_{col} + \sigma_{soil} (A_{soil} - A_{col})$

Where, σ_0 is the stress coming from the embankment

$0.773 * 79.45 = 0.282 * \sigma_{col} + \sigma_{soil} (0.773\text{-}0.282)$

$$61.41 = 0.282\ \sigma_{col} + 0.490\ \sigma_{soil} \longrightarrow (1)$$

3.4.3.2 Compatibility conditions

Settlement in DSM columns = Settlement in clay

$\sigma_{col}/ E_{DSM} = \sigma_{clay}/ E_{clay}$

$\sigma_{col} = \sigma_{soil} (E_{DSM}/ E_{clay})$

E_{DSM} = 100 MPa

E_{clay} = 3.44 MPa

Therefore $\sigma_{col} = 29.06\ \sigma_{soil} \longrightarrow (2)$

By substituting Equation 2 in Equation 1 we will get

From Equations 1 and 2

σ_{soil} = 10.70 kPa

From (1) and (2)

σ_{col} = 29.06 *18.61 = 311 kPa

Ultimate consolidation settlement due to combined equivalent PVD – DSM is,

$$\Delta h = \frac{C_{c,composite}}{1+e_{0,composite}}(h)\log\frac{p'_f}{p'_{c,composite}}$$

p^{l}_{f} = Stress acting on soil due to embankment loading + over burden pressure due to foundation soil.

Overburden pressure at middle of the clay layer = γ_{sub} *H/2 = (13.73-9.81)* (10/2)

= 18.65 kPa

S_f = 1.48 / (1+2.5) * (10) log {(18.65 + 10.70) /18.65}

Ultimate settlement of composite ground = 0.82 m.

3.5 Summary

Design of PVDs for a given soft soil condition is discussed. PVD design is influenced by their spacing rather than the size of the PVD. Design of DSM columns is also discussed in which case the area replacement ratio plays a major role in improving the bearing capacity of the adapted soil. A combined PVD-DSM design is also discussed and typical values required for the numerical analysis and theoretical prediction of ultimate settlement of combined PVD-DSM treated ground is established.

Generalized design charts for the combined ground improvement technic is not possible as there are many variables which makes it difficult to draw a generalized format.

Chapter 4

Numerical Model Development

4.1 Introduction

Series of numerical simulations are required to perform to understand the behavior of the combined ground improvement technic proposed in this study including the stability of the embankment structure. It is difficult to develop analytical model combining both PVD and DSM column effects in a soil model. Hence, numerical study is undertaken to model the combined ground improvement technic. The developed model needs to be validated first before perform a parametric study. Hence, an attempt is made to validate the model with an embankment data at the second Bangkok International Airport (SBIA).

Several iterations were made to validate the best available model to simulate the consolidation behavior of the soft soil using finite element software PLAXIS Version 11. The material models adapted to soil layers based on their stress strain behaviors to carry out the consolidation analysis include Cam Clay (Soft soil model), Mohr Coulomb model, and Linear Elastic and Hardening Soil models. The details of material properties are given in Table 4.1 and methods to solve the numerical analysis has been discussed right after the material models which includes boundary conditions, meshing, water pressure generation, initial stresses generation and consolidation calculation method. To compute the factor of safety, safety analysis (ϕ-c reduction method) is available in the PLAXISPLAXIS analysis.

Table 4.1: Material properties used in the numerical analysis

	Unit	Crushed rock	Soft clay	Medium stiff clay	DSM columns
Model Type		Linear elastic	Soft soil/Cam Clay	Mohr-Coulomb	Hardening Soil Model
Moist Unit Weight	kN/m^3	21	11.50	16	17.66
Saturated Unit Weight	kN/m^3	23	13.73	18	19.54
Poisson's Ratio, ν	-	0.28	-	0.3	0.28
Cohesion, c	kPa	2	6	20	150
Friction Angle	°	40	3	30	40
Permeability, k_x & k_y	m/day	0.1	6.90E-4 & 4.23E-4	2.16E-4 & 1.32E-4	0.864E-3 & 0.864E-3
Compression Index, C_c	-	-	1.64	-	.26
Recompression Index, C_r	-	-	0.29	-	0.039
Initial Void Ratio, e_o	-	-	2.72	-	0.8
Young's Modules, E	MPa	60	-	17.2	-

4.2 Material models used in the numerical analysis

Below mentioned material models have been used in the numerical analysis to represent the mechanical behaviour of the soil.

4.2.1 Mohr-Coulomb model

The Mohr-Coulomb model is linear elastic perfectly-plastic behaviour. Five basic input parameters are involved in this model; those are E and ν for soil elasticity; ϕ and c for soil plasticity and ψ as an angle of dilatancy.

Material parameters in the Mohr-Coulomb model

4.2.1.1 Young’s modulus (E)

Young's modulus works as the basic stiffness modulus in PLAXIS analyses. Young's modulus is also known as the tensile modulus. It is a measure of the stiffness of an elastic material.

It is defined as the ratio of the uniaxial stress over the uniaxial strain in the range of stress in which Hooke's Law holds. The slope of the stress-strain curve at any point is called the tangent modulus. It has the dimension of stress.

4.2.1.2 Poisson’s ratio (ν)

Poisson’s ratio is defined as the ratio of axial compression to lateral expansion. In most of the cases, the value of Poison’s ratio is considered between 0.3 to 0.4.

4.2.1.3 Cohesion (c)

Cohesion is defined as attractive force between two similar soil bodies. The cohesive strength has the dimension of stress. In Mohr-Coulomb model, effective cohesion (c') also can be modeled in combination with effective friction angle (ϕ'). Analysis can be performed for both drained and undrained soil behaviour. PLAXIS can handle both cohesion less soils and cohesive soils. But minimum cohesion value has to mention in the analysis.

4.2.1.4 Friction angle (ϕ)

The friction angle largely determines the shear strength by Mohr’s stress circles. The computational time increases more or less exponentially with increase in friction angle (PLAXIS, 2011 Reference Manual).

4.2.1.5 Dilatancy angle (ψ)

Dilatancy occurs because the grains in a compacted state are interlocking and therefore do not have the chance to move around one another, which produces a bulk expansion of the material. The dilatancy angle, (ψ), is specified in degrees. In general the dilatancy angle of soils is much smaller than the friction angle. Sandy soils shows dilatancy whereas clay soils tends to show no or negligible magnitude.

4.2.2 Soft Soil Model (Cam-Clay Model)

The Soft Soil model is a Cam-Clay type model especially meant for primary compression of near normally consolidated clay-type soils.

Material parameters in the soft soil model

4.2.2.1 Compression Index and Swell Index (c_c and c_s)

These parameters can be obtained from one dimensional compression test including isotropic loading. The slope of the primary loading line will gives the compression index and the slope of the unloading line will gives the swell index. These parameters can be obtained from the one dimensional compression test.

4.2.2.2 Initial void ratio (e_0)

The initial void ratio is the in situ void ratio of the soil mass.

4.2.3 Hardening soil model

The soil stiffness is described much more accurately in this model. In contrast to cam-clay model, the hardening soil model contains the additional parameters like triaxial stiffness E_{50}, triaxial unloading stiffness E_{ur} and the oedometer loading stiffness E_{oed} in the advanced option.

4.2.4 Linear elastic model

Soil behaviour is highly non-linear and irreversible. The linear elastic model is in sufficient to capture the essential features of soil. The use of linear elastic model may be considered to model the massive structures in the soil or bed rocks.

In the linear elastic material model the input parameters are E,ν, c, ϕ which are already discussed in the Mohr-Coulomb model.

4.3 Method of analysis

This analysis includes boundary conditions, mesh generation, water pressure, initial stresses generation and consolidation analysis are required to solve the whole problem and those are discussed as follows.

4.3.1 Boundary conditions

The left and right boundaries are horizontally restrained and vertically released. Both vertical and horizontal movements of the bottom boundary are fixed. The left and right vertical boundaries should be closed for free out flow at these boundaries because of the symmetrical boundary and extreme boundary respectively. Bottom boundary of the clay layer is also a close one because it is an impermeable layer. A closed consolidation boundary needs to be included to facilitate the consolidation behavior of the soft clay layer. This boundary excludes the embankment portion. Below figure represents the boundary conditions adapted for the embankment. Typical embankment geometry with boundary conditions is shown in the the Figure 4.1.

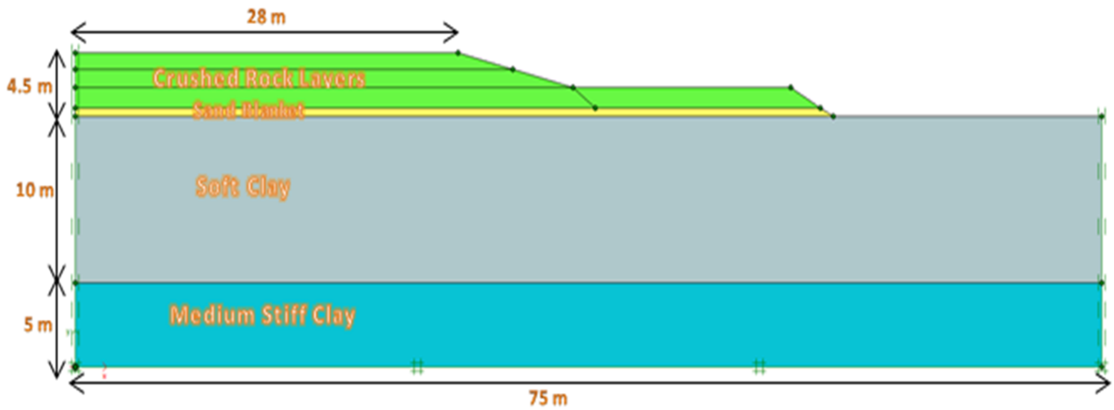

Figure 4.1: A typical embankment geometry of SBIA with dimensions and boundary conditions

4.3.2 Meshing

A composition of interconnected finite elements is called mesh. The geometry has to be divided into finite elements in order to perform finite element analyses. A 15-node triangular mesh element is considered in the present modeling. Coarse, medium, fine and very fine meshes are available in PLAXIS. Model with triangular mesh is presented in the below Figure 4.2.

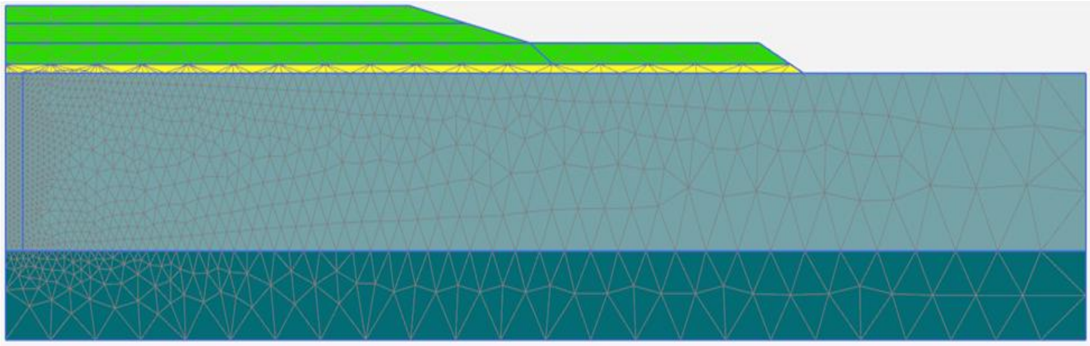

Figure 4.2. Finite element mesh by using PLAXIS-2D software

4.3.3 Water Pressure Generation

Ground water table is considered at the ground surface. Water pressure in PLAXIS can be generated by phreatic level and in present model the water pressure is generated by phreatic level. Model with water pressure generation presented in the below figure 4.3.

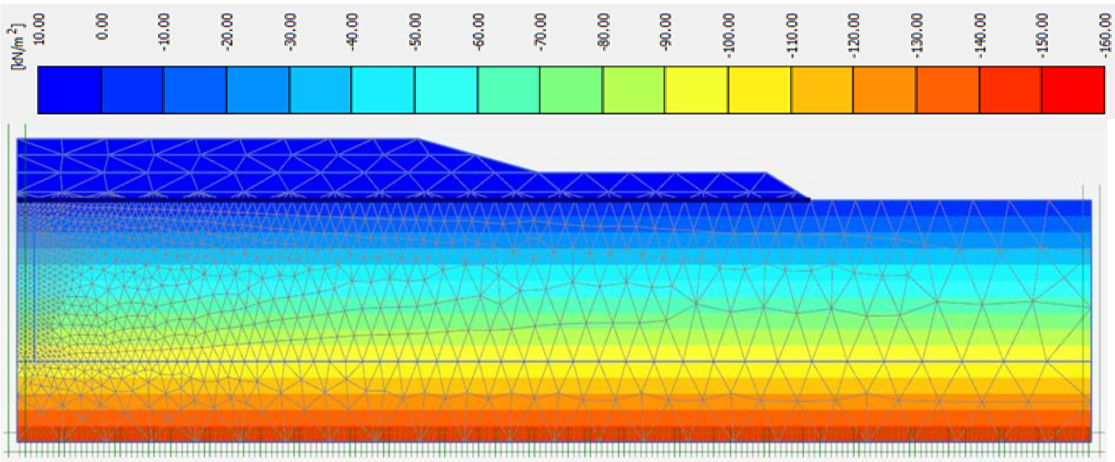

Figure 4.3. Generation of water pressures by using PLAXIS-2D Software

4.3.4 Initial stresses generation

Here in this case the soil stratum is in in pressures are generated based on the initial condition of the soil stratum. The history of soil formation and the weight of the materials influence the initial stress in the soil body. While generating the initial stress in the foundation soil we have to remove the embankment loading on top of it. Below Figure 4.4 illustrates the generation of initial stress in the foundation soil. In the same Figure, it can be noticed that, the extreme effective stress is -76.87 kN/m^2. Which is equals to submerged unit weight of soil layers times the height of the clay layers (≈ (13.73-10*10) + (18-10*5) = 77.3 kPa).

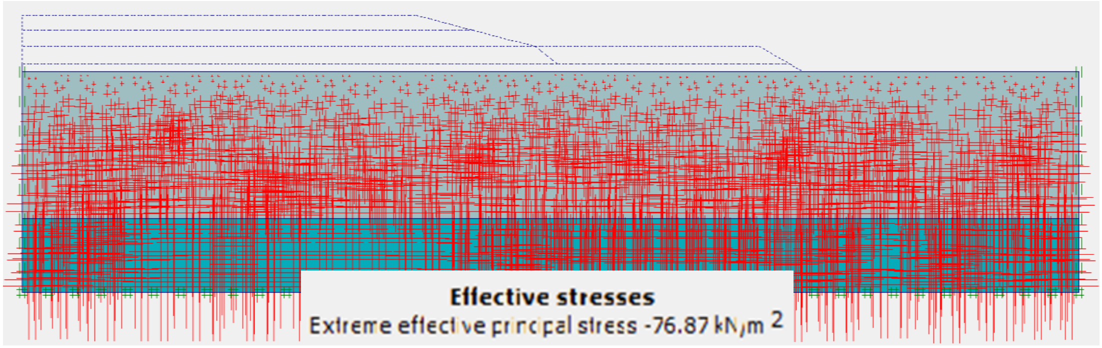

Figure 4.4: A typical figure of initial stresses generation in the PLAXIS-2D Software

4.3.5 Consolidation analysis

Consolidation analysis should be selected when it is necessary to analyses the development or dissipation of excess pore water pressure in saturated soil mass as a function of time. In PLAXIS it is also allows possible to apply loads during the consolidation analysis.

4.3.6 Stability analysis (ϕ-c reduction analysis)

ϕ-c reduction analysis is a separate calculation type available in PLAXIS to compute the factor of safety. In phi-c reductions approach the strength parameters c and ϕ of the soil are reduced until failure occurs. It must always be checked whether the final step has resulted in a fully developed failure mechanism. If that is the case the factor of safety is as follows:

Factor of safety = Available strength / Strength at failure

If the failure mechanism is not developed, then the calculation must be repeated with a larger number of additional steps. (PLAXIS reference manual, 2011)

4.4 Model with DSM columns

A model is presented below Figure 4.5 to understand the type of typical DSM columns installation with 5% area replacement ratio.

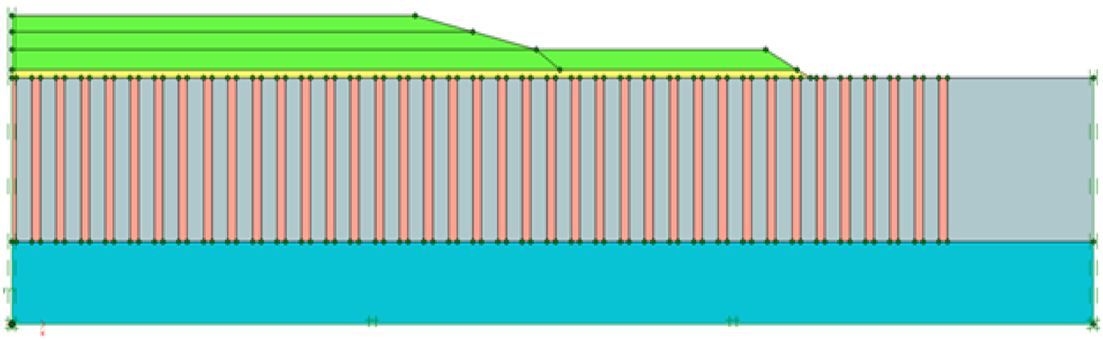

Figure 4.5: A typical picture of DSM columns installation modeled in PLAXIS-2D software

4.5 Numerical simulations to verify the influence of PVD spacing and DSM A_r, % on degree of consolidation

A series of numerical simulations have been conducted to verify the influence of the area replacement ratios of the DSM column, spacing of PVDs, rapid embankment construction etc. Table 4.2 describes the list of simulations performed with variable parameters. The corresponding results have been presented in the results and discussion chapter.

Table 4.2: Parameters considered studying the effect of degree of consolidation

PVD c/c Spacing (m)	DSM area replacement ratio (A_r %)
1	1
	3
	5
	7
	10
1.5	1
	3
	5
	7
	10
2	1
	3
	5
	7
	10

4.6 Validation with embankment constructed at SBIA

In the below Figure 4.6 one can be observe the variation of foundation settlement with respect to the time (in years) for treated (with PVD) and untreated ground. The classical one-dimensional consolidation theory of Terzaghi [35] was used for theoretical calculation of the consolidation settlements due to full design load (i.e. 80 kPa) and the time for consolidation. PLAXIS-2D finite element simulations have been performed to validate the present model, it showed a very good agreement with theoretical solutions as well as Lin et al., 2000, same thing can be observed in the below Figure 4.6

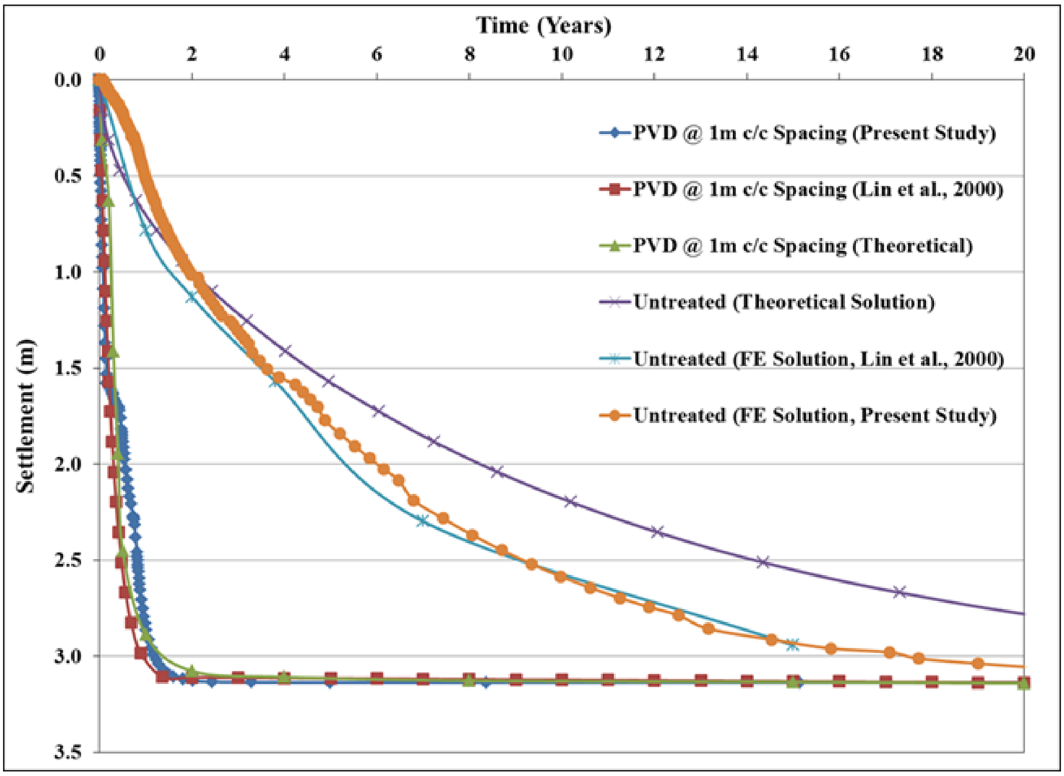

Figure 4.6: Model validation in FE software PLAXIS-2D

4.7 Summary

All the material models used in this study worked successfully by validating the present study with embankment constructed at SBIA area with different cases.

Chapter 5

Results and Discussion

5.1 Introduction

The application of prefabricated vertical drains and deep soil mixing columns together could be an effective solution for the rapid embankment construction on top of the highly compressible soils. Installation of prefabricated vertical drains (PVDs) accelerates the consolidation by reducing the drainage path. Installation of deep soil mixing columns yields enough bearing capacity to the foundation and the area replacement ratio of DSM columns reduces the pressure coming from the embankment on to the soil. The combined method may have dual positive affect on the soft soil. Hence, a numerical model was developed simulating both PVD-DSM treatments together. First, the model is validated with an embankment constructed at Second Bangkok International Airport (SBIA). In this chapter SBIA model validation, the variation of degree of consolidation with time for untreated and PVD treated foundation soil have been observed. A series of numerical studies were performed to verify the stability (ϕ-c reduction analysis) of PVD-DSM treated soft ground with decreased total construction time (1.5, 1.0, 0.5, and 0.12 years). Further a series of parametric studies have been conducted in order to analyze the variation of degree of consolidation of PVD-DSM treated ground. The stress concentration in the combined treated ground and displacement patterns are also discussed with respect to the configuration of the treated and untreated ground.

5.2 Analysis of Results

5.2.1 Validation of the model with embankment constructed at SBIA

Initially, numerical simulations are performed to validate the model with the theoretical and SBIA embankment field data as discussed in Section 4.6. Model validation part has been discussed here as well for continuity.

5.2.1.1 Variation of degree of consolidation

Figure 5.1 shows the variation of degree of consolidation with time for untreated and PVD treated foundation soil. It can be seen that the numerical results are in good agreement with the theoretical (Hansbo et al., 1982) as well as predicted (Lin et al. 2000) solutions for untreated and PVD treated ground.

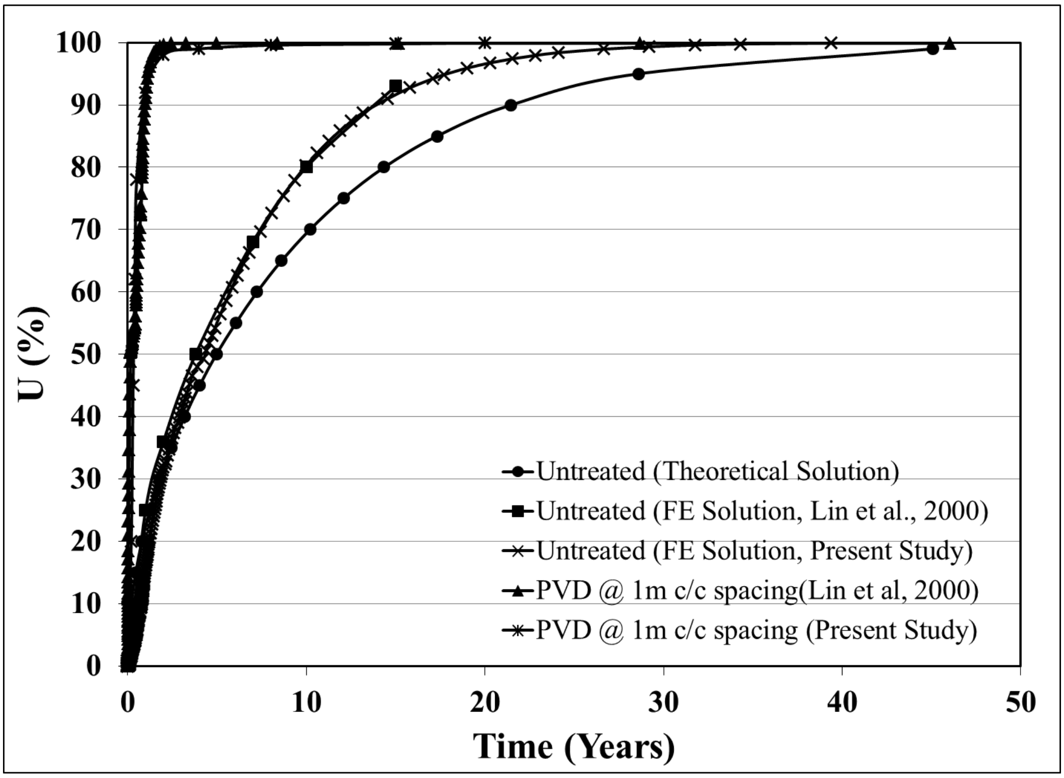

Figure 5.1: Variation of degree of consolidation with time for 1m c/c PVD spacing

It can be inferred from the Figure 5.1 that the installation of PVDs exhibited maximum reduction in time as compared to the untreated ground. It is calculated that the untreated ground took 45 years' to reach the complete consolidation, which can be observed in the lower three curves. Extreme bottom curve is for theoretical solution and middle ones are observed from finite element solutions developed by Lin et al., and the present numerical study. Extreme top curves depict the results for PVD improved ground which shows the drastic reduction in consolidation time from 45 years to about two years. It can be seen that there is a very good agreement among the theoretical and numerical studies.

5.2.1.2 Variation of excess pore water pressure

The indication of gaining shear strength and increased consolidation settlements are due to expulsion of excess pore water pressure. More the expulsion of pore water pressure generated during the construction of embankment implies excessive settlements in a given period of time. The typical variations of excess pore water pressure with time for different soil conditions (Untreated, PVD treated) have been observed. Figure 5.2 shows the change in excess pore water pressures due to staged construction loading.

High excess pore water pressures are noticed in untreated and comparatively lesser in PVD improved ground at different stages. With increase in excess pore water pressure; there is a reduction in the undrained shear strength of the soft ground which in turn brings down the stability of the system. Hence, it is required to further increase the consolidation time to make the excess pore water pressures to zero which leads to an uneconomical solution.

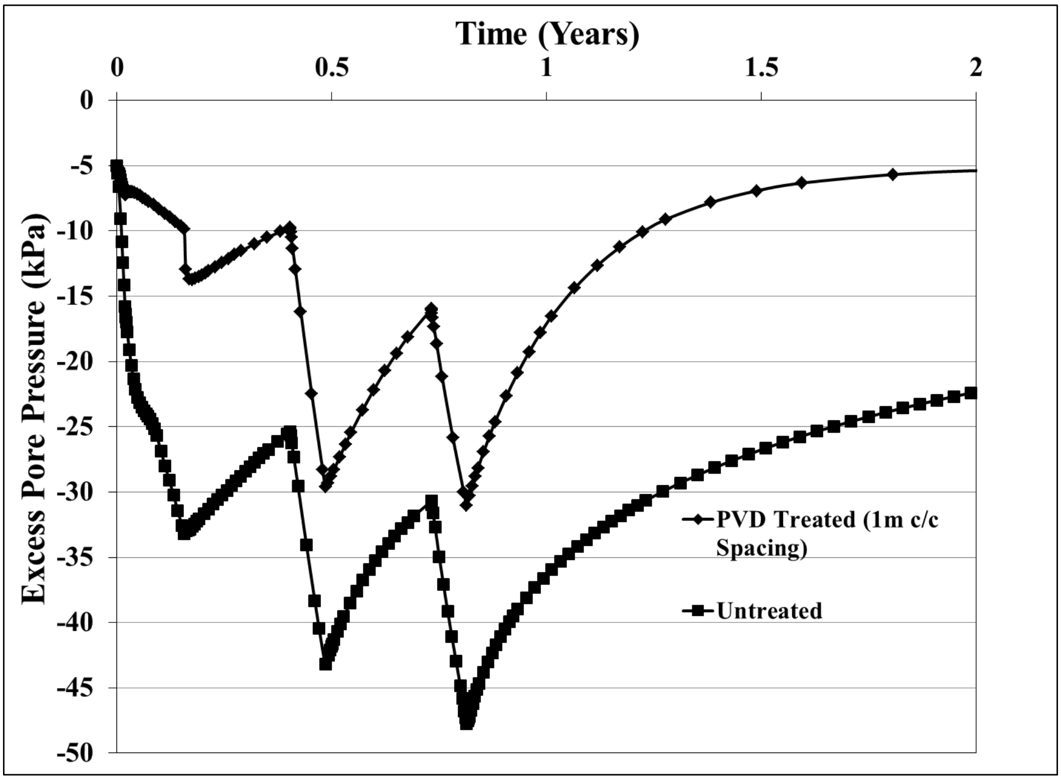

Figure 5.2: Variation of Excess pore water pressure for Untreated, PVD treated ground with time

5.2.2 Combined ground treatment method

In FEM simulation the embankment loading was considered as three steps. First 1m of embankment loading over 0.5 m sand blanket was placed within 57 days and 90 days left for the consolidation. Second 1.1 m of embankment stage placed within 30 days and left 90 days' time for the consolidation. In third step, the final stage of the embankment was constructed in 30 days and left 250 days for complete consolidation.

It is postulated that the embankment construction can be accelerated in lieu of DSM treatment. DSM columns were introduced at different DSM column area replacement ratios (Ar = 1%, 3%, 5 %, 7% and 10%) with a column diameters varied between 0.6m and 1.2m to improve the shear strength of the soft ground.

5.2.2.1 Influence on degree of consolidation

Figure 5.3 exhibits the variation of degree of consolidation of PVD treated and PVD-DSM treated grounds with time. In this series, only a DSM column of 0.6m diameter were used as a minimum reinforcement at different area replacement ratios. It can be deduced that the 100 % degree of consolidation could be achieved much earlier with combined PVD-DSM treatment compared to the PVD treated ground. The required consolidation period is observed to be much lesser with increase in area replacement ratio of DSM columns for any stage of construction. This observation clearly demonstrates the influence of combined PVD-DSM treatment in drastically reducing the construction time without compromising the stability of the structure. The results are further analyzed with the expulsion of excess pore water pressure.

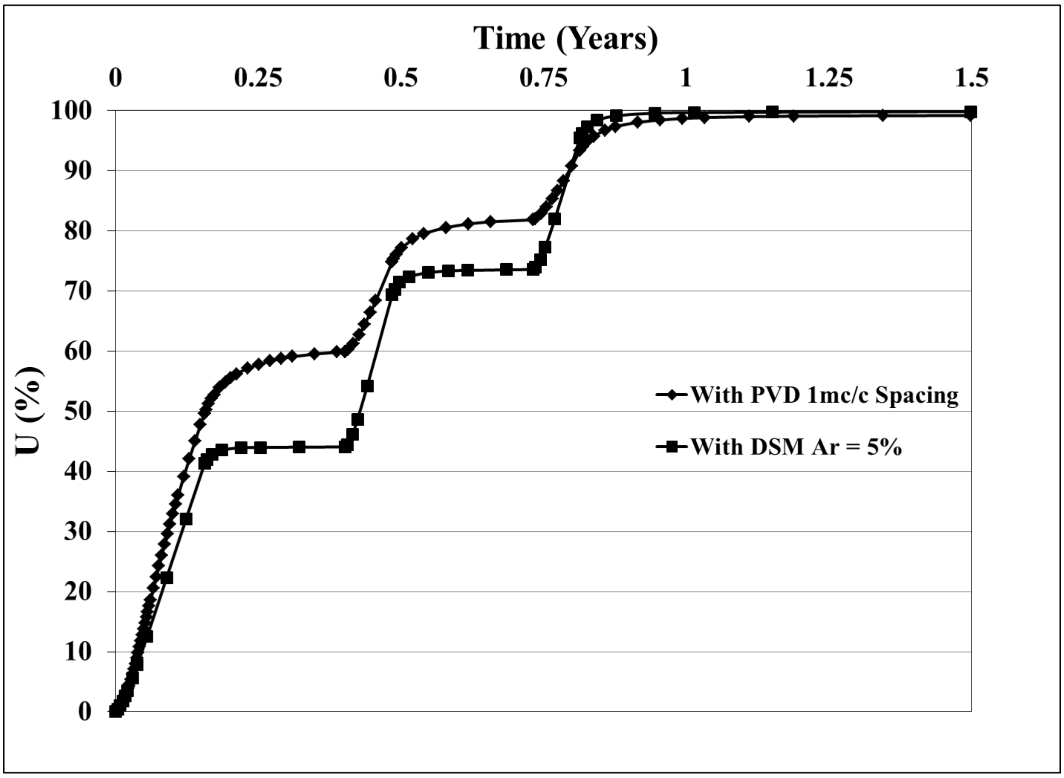

Figure 5.3 Variation of degree of consolidation with time for PVD and PVD-DSM treated ground

5.2.2.2 Influence on excess pore water pressure

Figure 5.4 shows the variation of excess pore water pressure with time for different area replacement ratios of DSM columns. It can be noticed that the excess pore water pressures in the soft ground have come down with increase in area replacement ratios of DSM columns. This observation confirms that the majority of the embankment load has been transferred to the DSM columns with increase in their area replacement ratio. Since, relatively low pressure is transferred to the foundation soil leads to a generation of low excess pore water pressures which dissipates quickly. It can also be understood that the increase in area replacement ratio on the dissipation of excess pore water pressure is almost negligible. It is observed that there is hardly a 5 to 8% variation in excess pore water pressure with area replacement ratios. It is inferred from this observation that a minimum amount of area replacement ratio would be satisfying the shear strength criteria. Optimum area replacement ratio for a given soil condition can be deduced based on further analysis.

Figures 5.2 and 5.4 clearly depict the staged construction of embankment in terms of rapid increase in pore water pressure due to the loading stages. There is a dissipation of excess pore water pressure takes place during the consolidation period. Though the excess pore water pressure did not become completely zero during intermediate stages, sufficient undrained strength of foundation soils was gained to support the subsequent embankment loading. The excess pore pressures would become negligible with increase in the consolidation period, which further delays the total construction. A similar phenomenon has taken place in every stage. After the consolidation of final stage, excess pore pressure in the foundation soil became zero in the case of PVD and PVD-DSM treated grounds. It can be noted that for PVD-DSM treated ground the time taken for complete dissipation of excess pore water pressure took very short period of time against the PVD alone treated and untreated ground. This implies the total stress equals to the effective stress in a shorter period of time, thereby increasing the undrained shear strength of the soft soil.

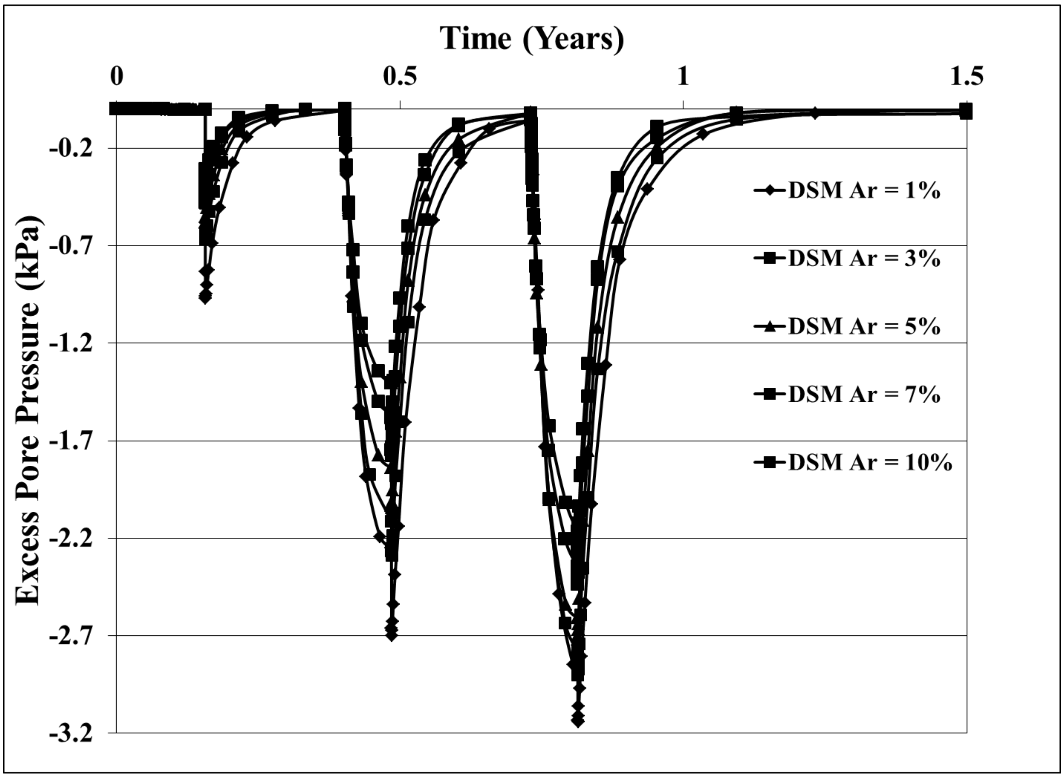

Figure 5.4: Variation of Excess pore water pressure for PVD-DSM treated ground with time

5.2.3 Stability Analysis

A series of numerical simulations were performed to verify the stability (using ϕ-c reduction analysis) of PVD-DSM treated soft ground with decreased total construction time (1.5, 1.0, 0.5, and 0.12 years). For this series, the DSM area replacement ratio was maintained at 5% and PVD spacing at 1m c/c. The consolidation period between construction-stages were maintained uniform percentage of the total construction period in all the cases. That means a 90 day consolidation period between stages for 550 days of total construction period is equal to about 16.4%. Figures 5.5 and 5.6 present the results from the stability analysis. Figure 5.5 describes the variation of factor of safety of PVD-DSM treated soft ground with construction time. It can be noted that the safety factors decreased with the number of stages. Besides, the safety factors are almost constant even with decrease in the total construction time. As per Ramiah and Chickangappa; Bowles [34, 35] the minimum safety factor against bearing capacity failure is 2. However, the factor of safety was only 1.6 for the last stage even with PVD treatment with 1.5 year construction period as shown in Figure 5.5, which leaves the structure in a critical position.

The minimum factor of safety for PVD-DSM treated case was observed to be around 3.8 for the last loading stage, which is considered to be very high for the stability of the embankment. It is interesting to note that the factor of safeties of each loading stage for different construction periods remain almost constant. This observation leads to the conclusion that the influence of the time of construction on factor of safety is negligible as long as a minimum required area replacement ratio of DSM is maintained. The composite shear strength depends on the shear strength of the soil, the shear strength of the columns, and the area replacement ratio.

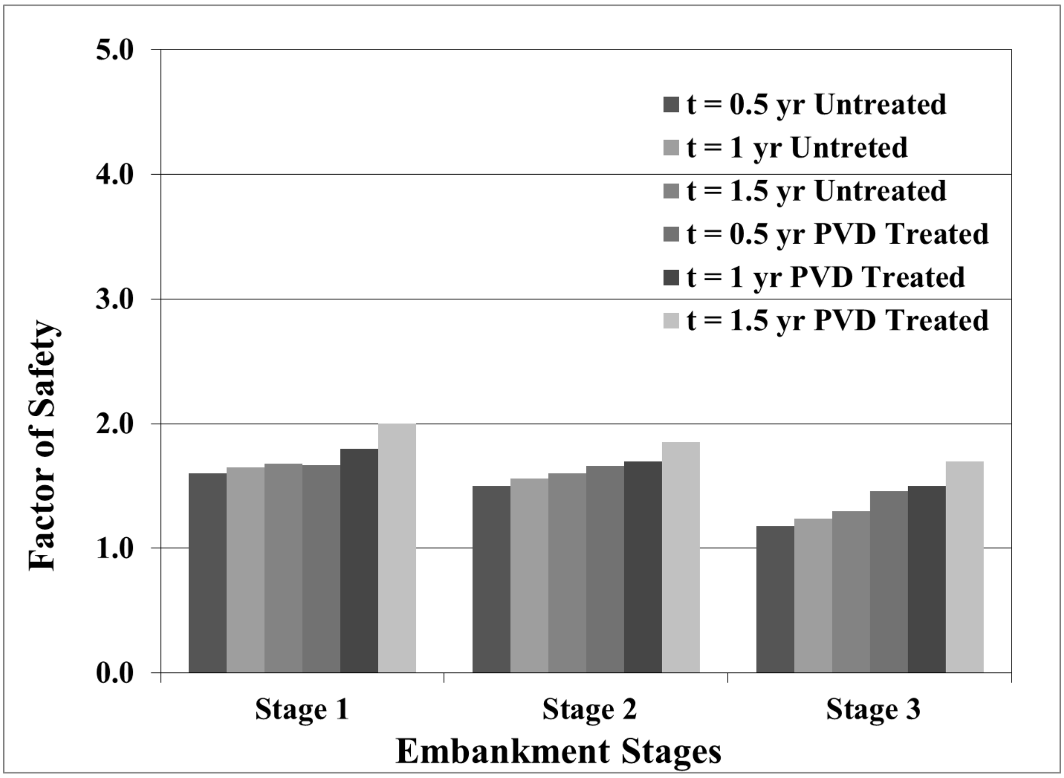

Figure 5.5: Variation of factory of safety of untreated and PVD treated ground

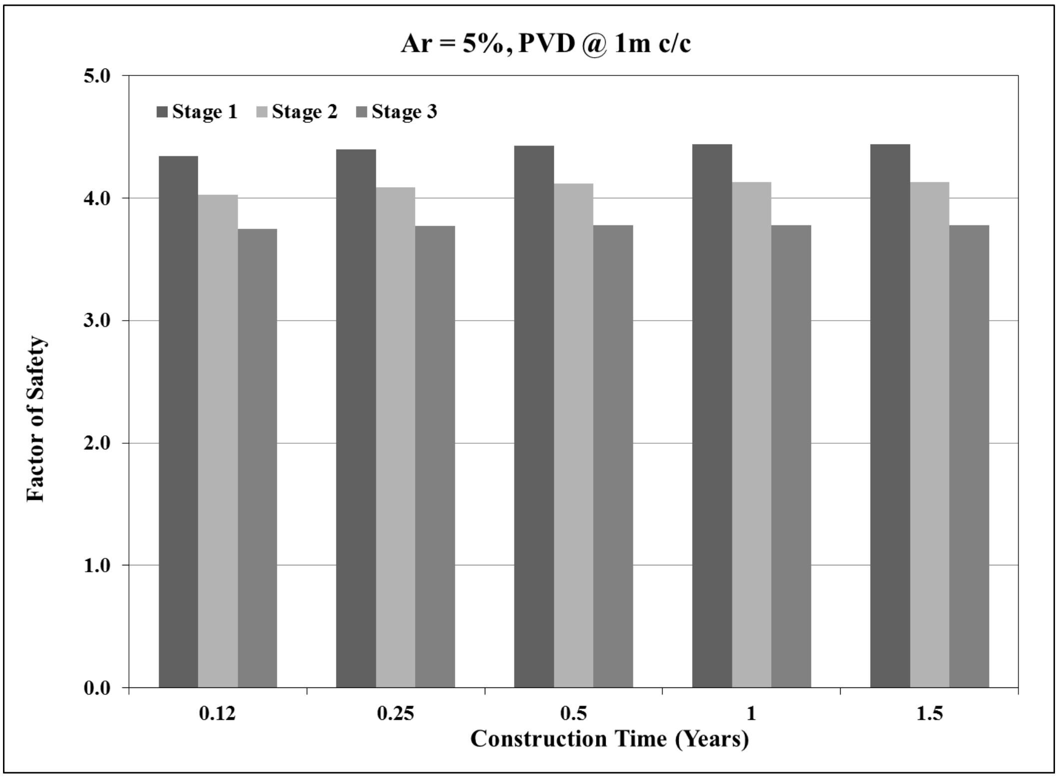

Figure 5.6: Variation of factory of safety of PVD-DSM treated ground

5.2.4 Stress Variations

A stress analysis has been performed to understand the variation of effective stresses with respect to time upon staged embankment construction. Typical effective stress variation with respect to construction time for the untreated, PVD treated and PVD-DSM treated ground have been considered and respective plots are depicted in Figures 5.7, 5.8, 5.9 and 5.10. Results show that, in the case of untreated ground the enhancement of effective stresses were not taking place. The stresses are decreased drastically in the middle of the clay layer due to the application of load with respect to the time. Contrary to untreated case, the PVD and PVD-DSM treated cases showed considerable enhancements in effective stresses with respect to depth of the clay layer. Similar trends were noticed below the embankment slope, the effective stresses are lesser than the stresses developed below the embankment centre line but stresses are increasing with depth. From the above discussion and results, the PVD-DSM treated case gained more strength than any other case which is clear from Figure 5.9 and Figure 5.10. The main reason is area replacement ratio of DSM columns which plays a major role for the improved strength. Higher area replacement ratio treated area contains less pore water pressure as per the effective stress principle. The effect of pore water pressure and factor of safety for untreated, PVD treated and PVD-DSM treated cases analysed and discussed individually in the above sections 5.2.2.2 and 5.2.3. These results suggest that with a decrease in excess pore water pressure, the effective stress was increased indicating more factor of safety in the soil that did not result in the untreated section and less resulted in the PVD treated section, which is also clear from the Figures 5.2, 5.4 and 5.5. Effective stress distribution patterns have been presented individually for the above mentioned cases in the Figures 5.11, 5.12 and 5.13 respectively.

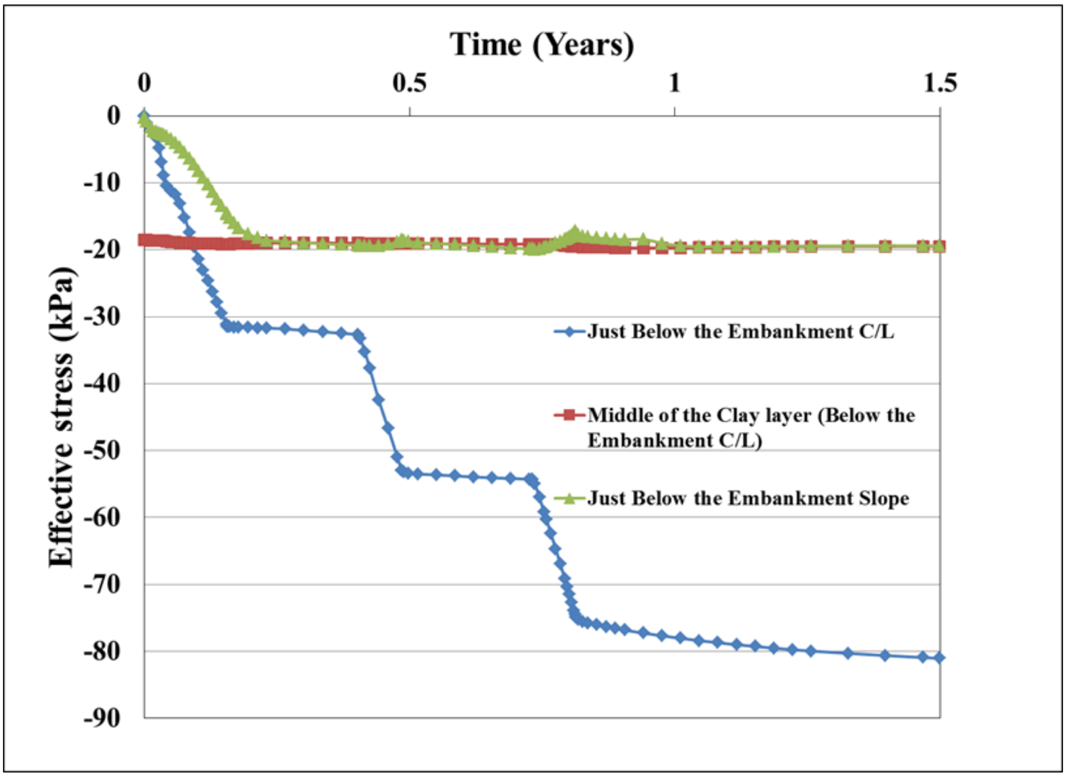

Figure 5.7: Effective stress variations with respect to time for untreated ground

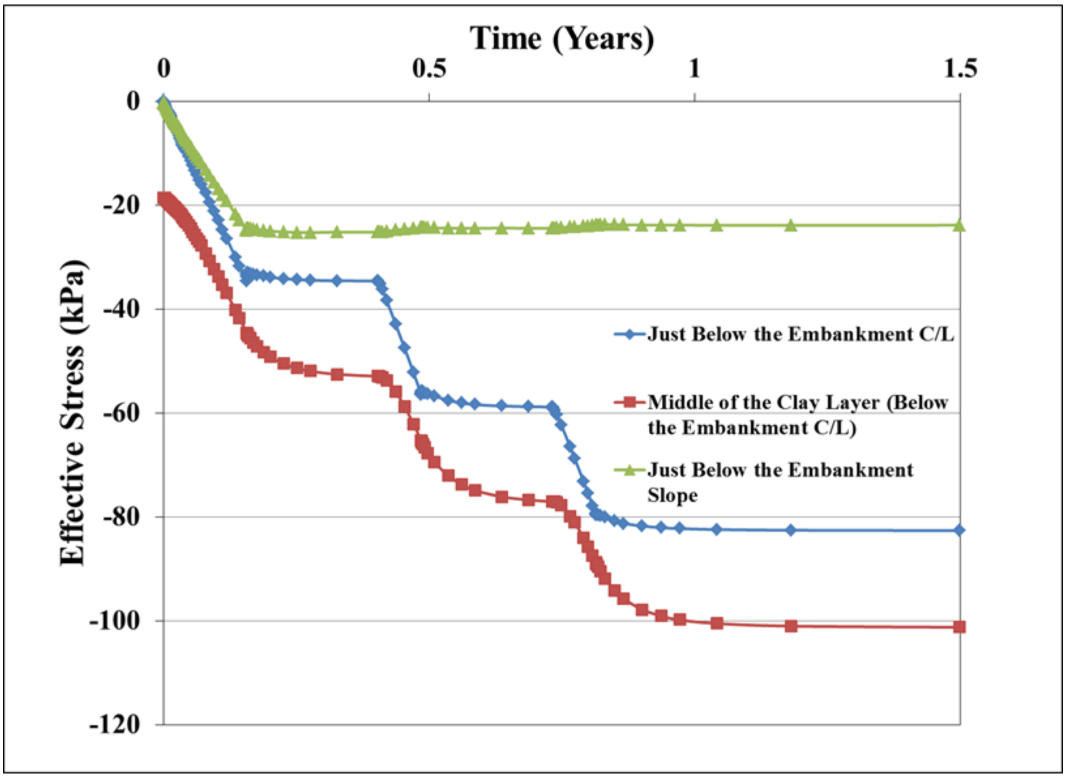

Figure 5.8: Effective stress variations with respect to time for PVD treated ground

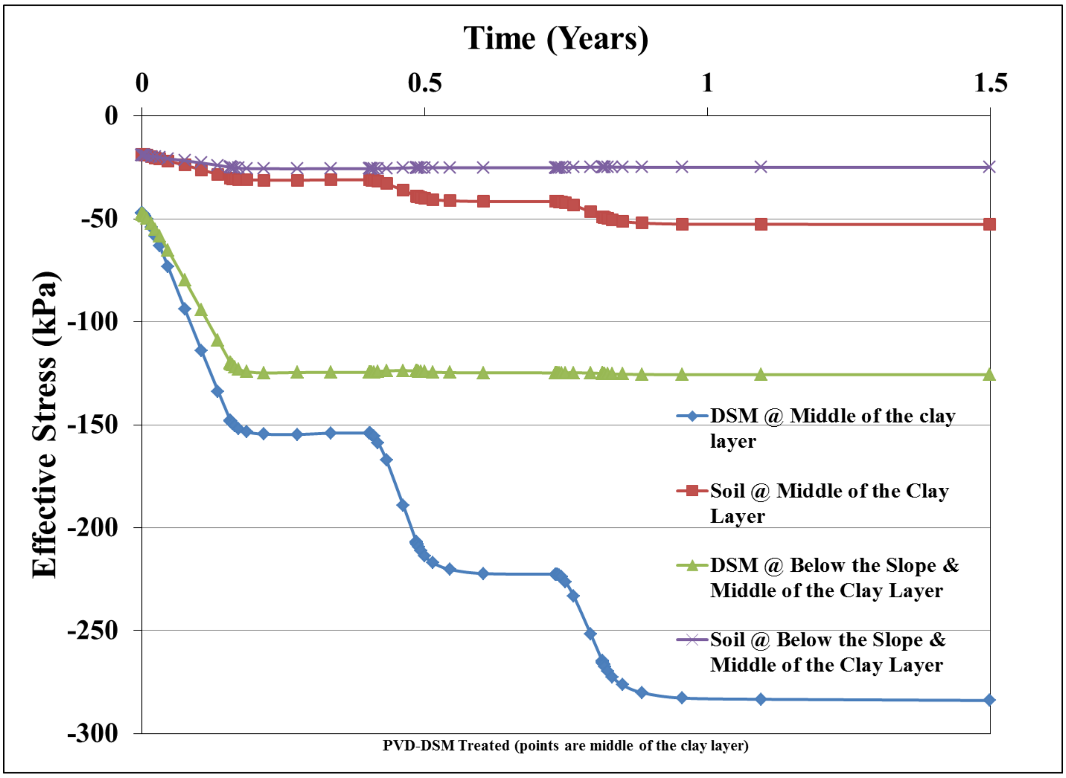

Figure 5.9: Effective stress variations with respect to time for PVD-DSM treated ground

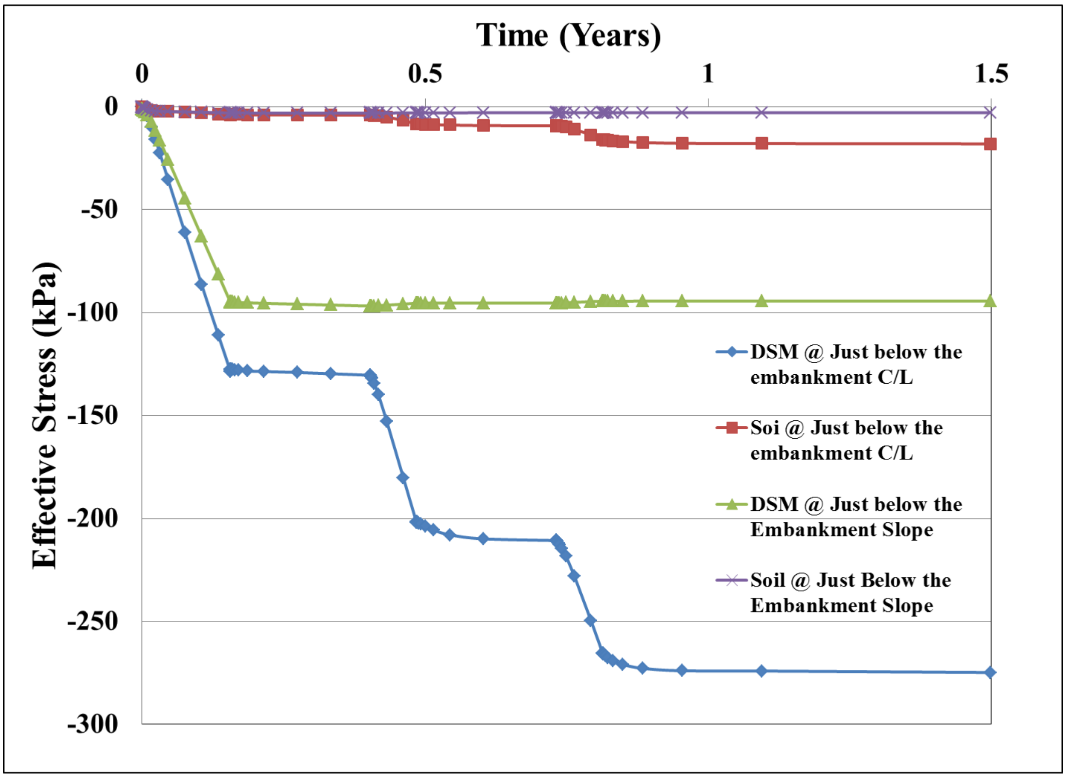

Figure 5.10: Effective stress variations with respect to time for untreated (Points taken at middle of the clay layer)

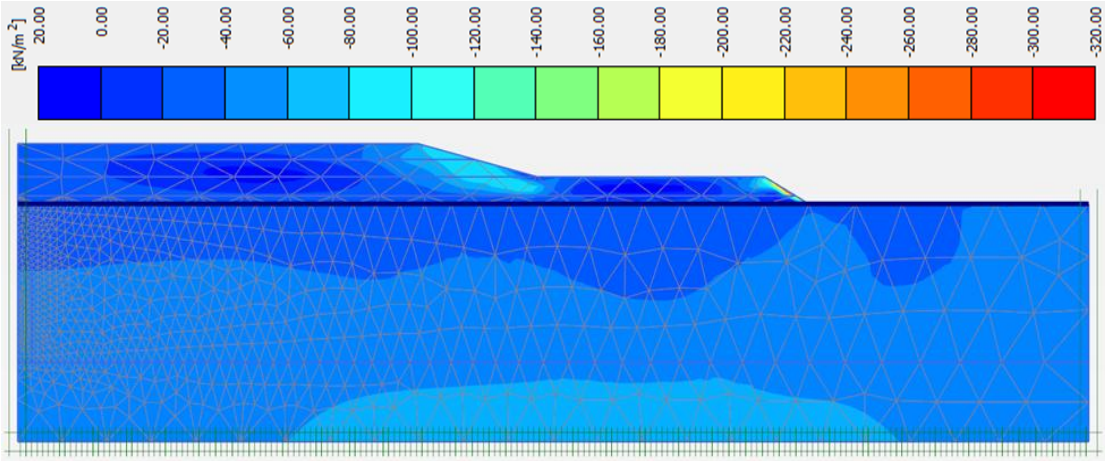

Figure 5.11: Effective stress generation in untreated ground

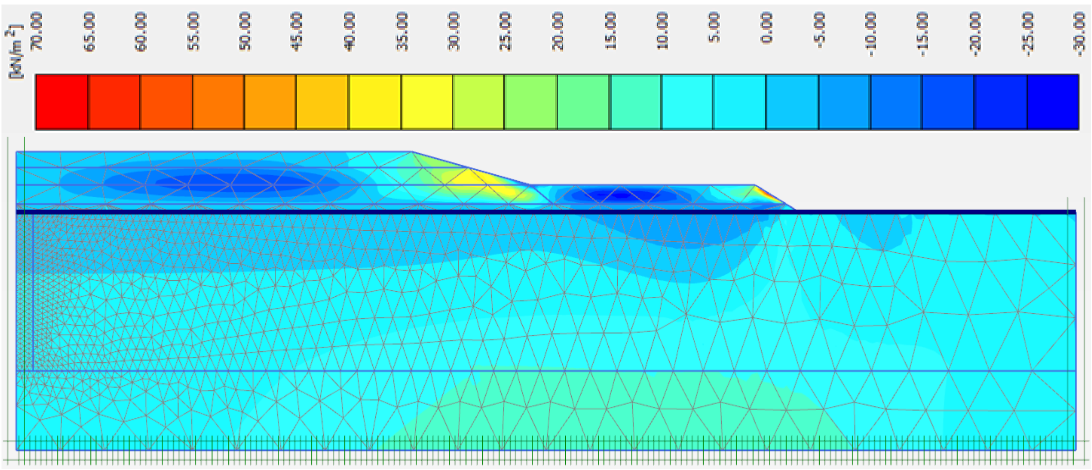

Figure 5.12: Effective stress generation in PVD treated ground

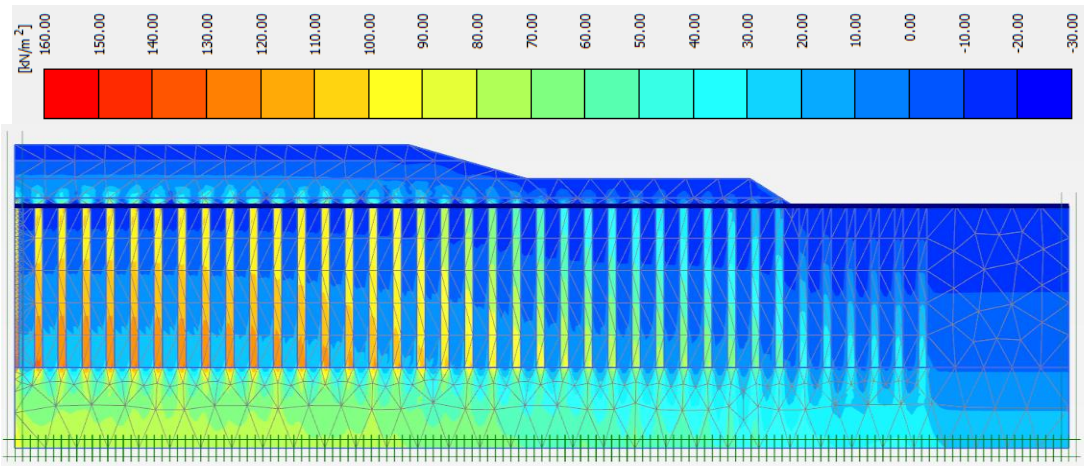

Figure 5.13: Effective stresses in PVD-DSM treated ground

5.2.5 Deformation patterns

A deformation analysis for embankment construction has been performed in terms of staged embankment construction. Deformation of the individual embankment stage in one and half year time period have been presented in the Figure 5.14 for untreated, PVD treated and PVD-DSM treated cases. In this figure it can be observed that the untreated ground has not reached to its ultimate settlement whereas PVD treated ground reaches within 1.5 year time period. PVDs accelerate the flow of water from the soils; hence higher settlement rate can be achieved. In the case of PVD-DSM the foundation ground reaches its ultimate settlement less than one year. Installation of DSM columns reduces the quantity of excess pore water in the foundation soil. This is the reason the dissipation process won't take much time.

Deformation patterns have been presented for above mentioned cases in the figure 5.15, 5.16 and 5.17 respectively. Untreated ground shows much deformation at the slope of the embankment due to rapid construction and PVD treated ground also exhibit the same thing but not in extreme manner. Whereas PVD-DSM treated ground is not exhibiting any excessive deformations in the ground.

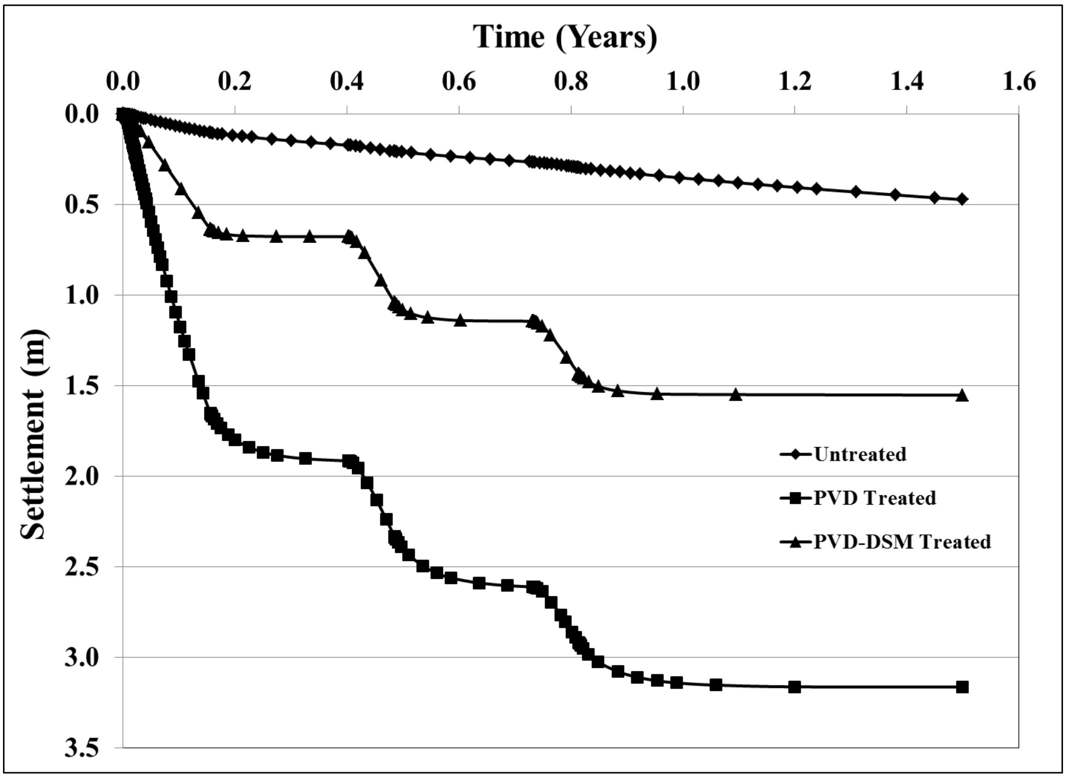

Figure 5.14: Typical variation of deformations with timing

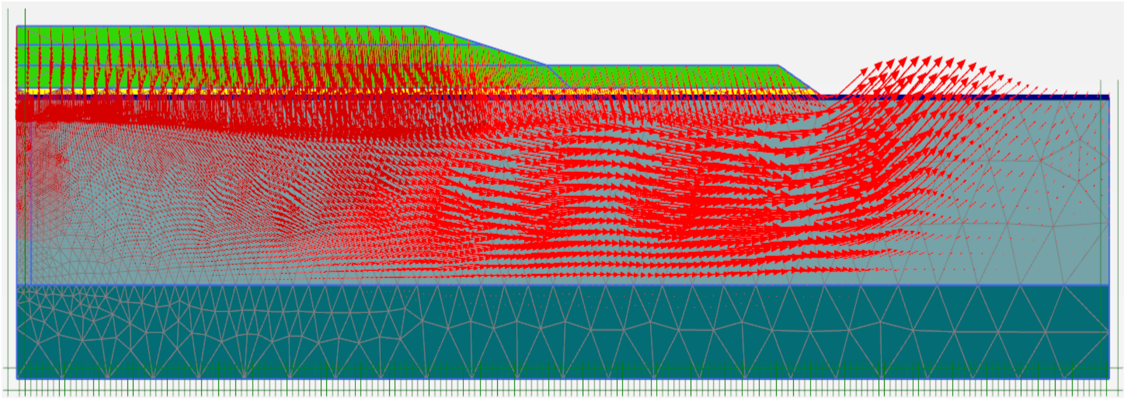

Figure 5.15: Total displacements in untreated ground

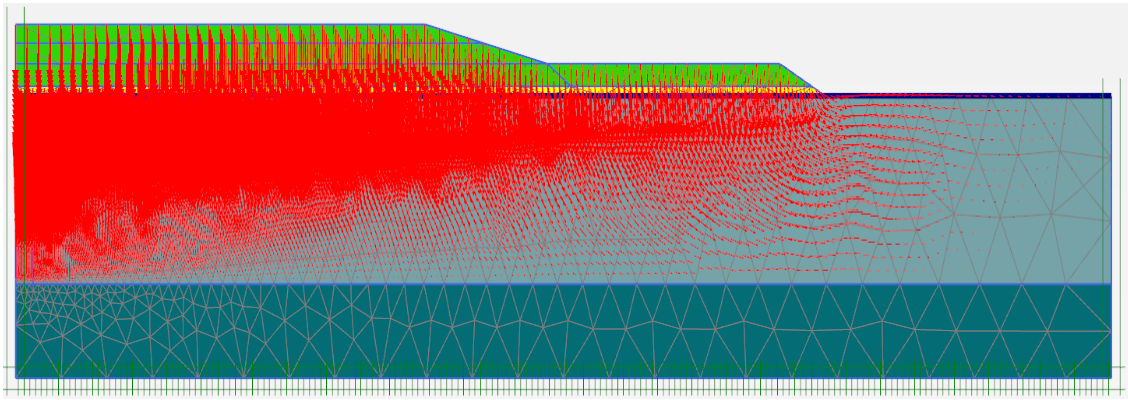

Figure 5.16: Total displacements in PVD treated ground

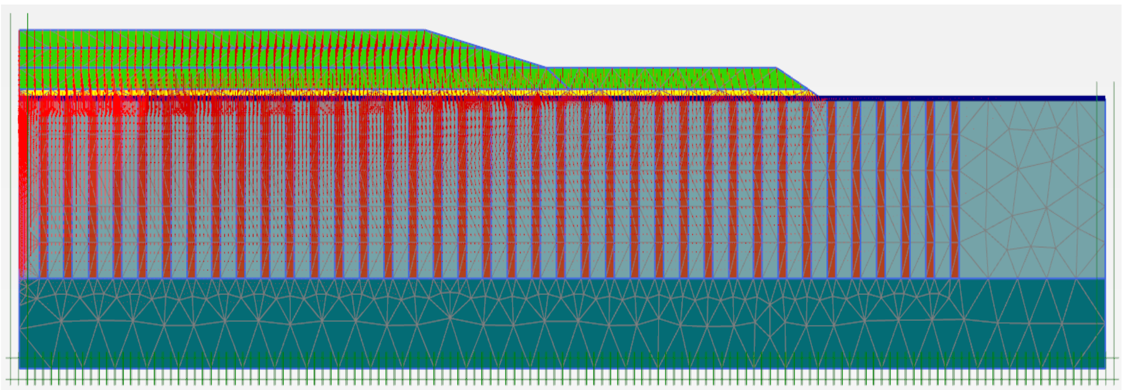

Figure 5.17: Total displacements in PVD-DSM treated ground generated in PLAXIS-2D

5.2.6 Parametric study

5.2.6.1 Introduction

A series of parametric studies have been conducted in order to analyze the influence of DSM columns area replacement ratio with various PVD spacing on the degree of consolidation of the treated soft ground.

5.2.6.2 Effect of PVD spacing on degree of consolidation with various DSM area replacement ratio

Prefabricated vertical drains have been introduced at different spacing 1, 1.5, and 2m c/c spacing and DSM columns were introduced at different DSM column area replacement ratios 1%, 3%, 5 %, 7% and 10%. For each PVD design spacing all DSM area replacement ratios have been analyzed for variation of the degree of consolidation. The variation of degree of consolidation with time for different DSM area replacement ratios are shown for PVD spacing of 1m c/c, 1.5m c/c and 2m c/c in Figures 5.18, 5.19 and 5.20 respectively. Results show that the ground with closer PVD spacing giving more degree of consolidation at any particular time. It is also noticed that with less DSM area replacement ratio treated ground has consolidated up to 90% degree of consolidation. Reverse trend was noticed thereafter. The reason being higher area replacement ratio DSM treated ground contains less pore water pressure to dissipate as the soil is replaced largely by the DSM columns and DSM columns carry a major portion of the embankment load.

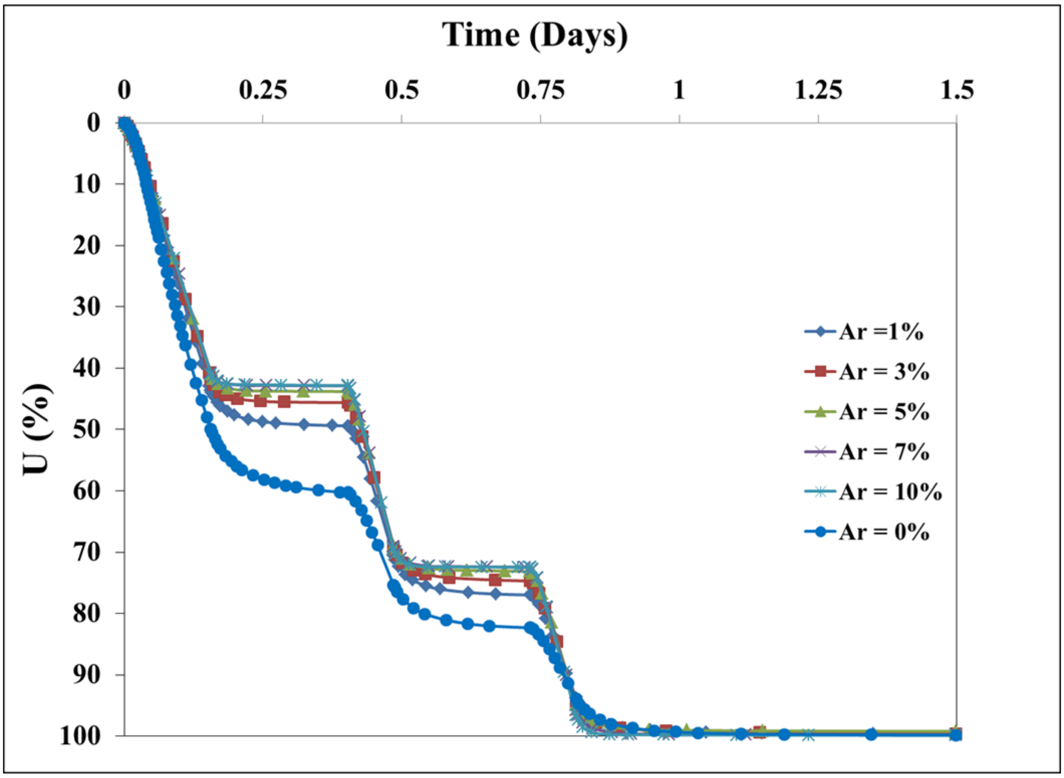

Figure 5.18: Variation of Degree of Consolidation with Time for 1m c/c PVD Spacing

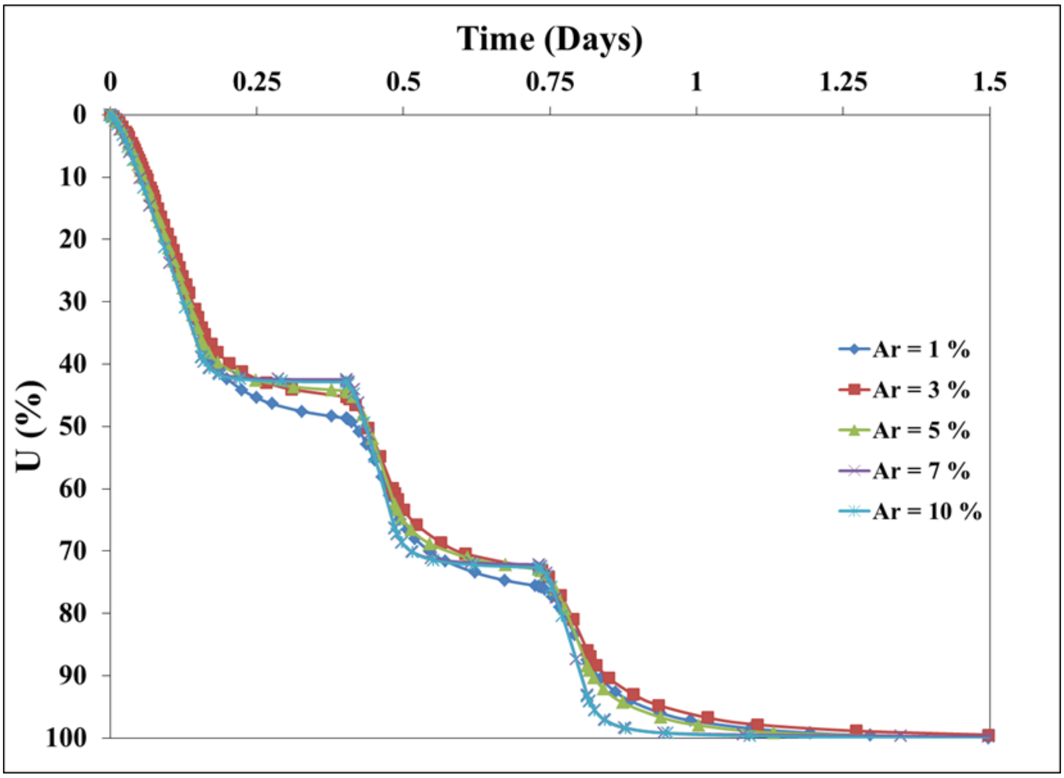

Figure 5.19: Variation of Degree of Consolidation with Time for 1.5m c/c PVD Spacing

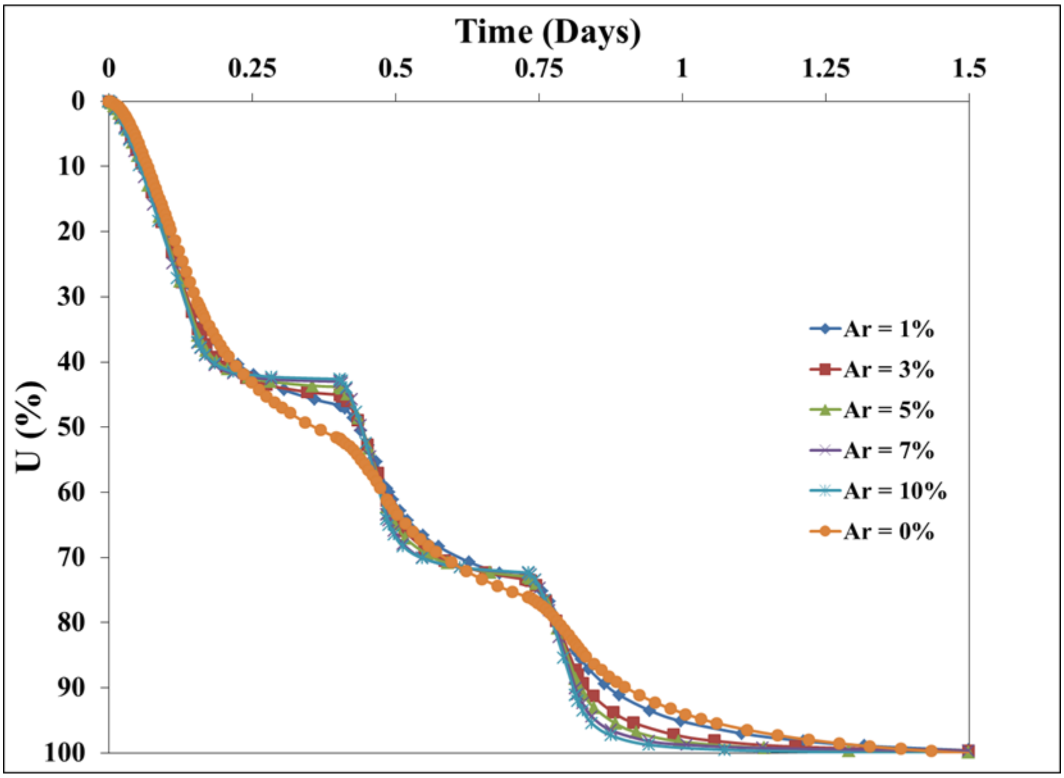

Figure 5.20: Variation of Degree of Consolidation with Time for 2m c/c PVD Spacing

5.2.6.3 Influence of Construction Time on Settlement with Various DSM Area replacement ratios

A series of parametric studies have been conducted to understand the settlement behaviour of PVD-DSM treated ground with various DSM area replacement ratios (1, 3, 5, 7, and 10 % respectively). Figure 5.21 visualizes the consolidation settlements for different construction periods for different DSM area replacement ratios. It can be noticed that low consolidation settlements are attributed to the higher area replacement ratios of DSM columns. It can be visualized that the settlements of soft soil with DSM columns of any area replacement ratio become constant for the construction time equal to or higher than 0.5 years (6 months). It can be concluded that the total embankment construction can be finished in 6 months without compromising on the safety and the consolidation settlements.

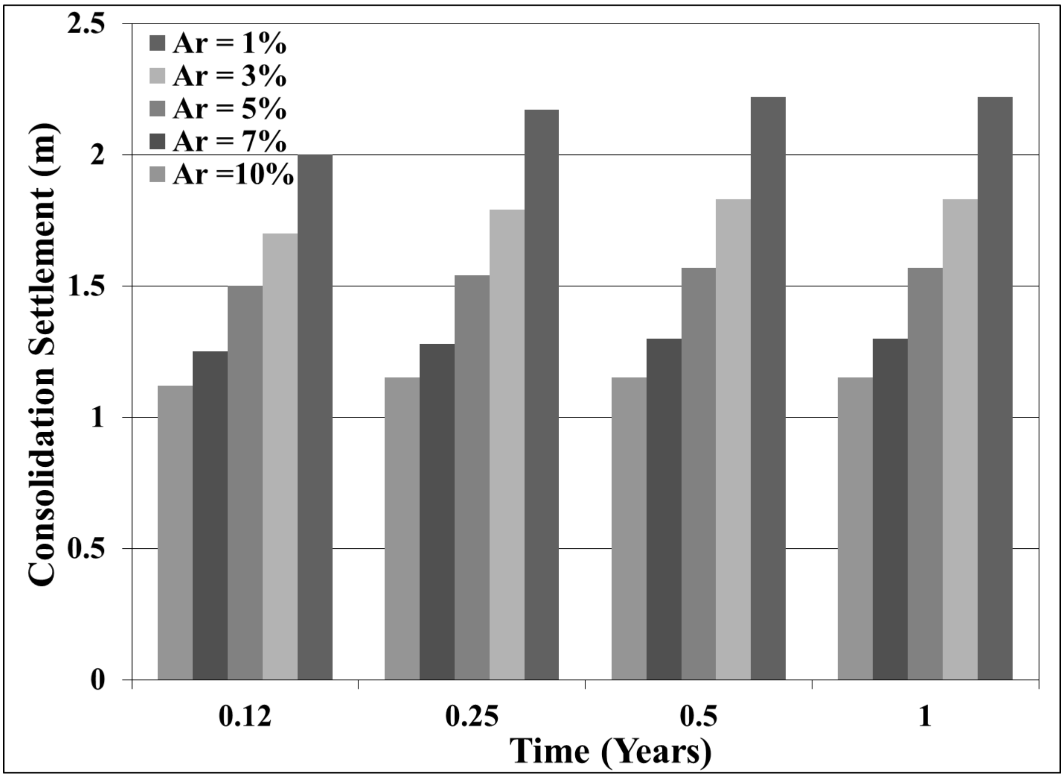

Figure 5.21: Variation of consolidation settlements of different PVD-DSM treated grounds with embankment construction time

5.3 Summary

1. The observed ultimate settlement for PVD-DSM treated ground with various DSM area replacement ratios were similar even though with the increased construction time periods. It can be concluded that total construction can be finished within six months.
2. Excess Pore pressures generated due to rapid construction of high embankments can greatly be reduced by introducing PVDs and DSM columns together.
3. The total amount of construction time of high embankments can greatly be reduced by introducing PVDs and DSM columns together.
4. DSM columns with minimal area replacement ratios are worked successfully by producing higher safety factors in the rapid embankment construction.
5. PVD spacing has a great influence on the combined ground improvement technic.
6. PVD and PVD-DSM treated cases showed considerable enhancements in effective stresses with respect to depth of the clay layer.
7. PVD-DSM improvement reduces the stress acting in the soil by arching effect. Thus by reducing the ultimate settlement in the foundation soil.

Chapter 6

Summary, Conclusions and Recommendations

6.1 Summary and Conclusions

A combined PVD-DSM column treatment has been attempted to simultaneously address the consolidation and shear strength characteristics of soft soils. A finite element numerical scheme has been developed to model an embankment construction on very soft foundation soil. The model has been validated with the field and numerical data reported by Lin et al. (2000) for embankment constructed at Second Bangkok International Airport, Bangkok, Thailand. Numerical simulations were performed to evaluate the effectiveness of prefabricated vertical drains in soft soil. Consolidation behaviour has been observed by using the two dimensional Finite Element program PLAXIS (Version 2011). Further, the embankment behavior was studied with deep soil mixing columns (DSM) along with prefabricated vertical drains to take care of consolidation as well as embankment stability in the case of rapid embankment construction. Stress analysis and deformation analysis have been performed to understand the effective stress distribution and its variation with depth of the soft clay layer. . Various area replacement ratios of DSM columns are considered in order to study the stiffness improvement and consolidation behaviour of the treated ground along with constant PVD spacing (1, 1.5, 2m c/c). Further, stability analysis (Ø-c reduction analysis) for untreated, PVD treated and PVD-DSM treated ground has been considered in the analysis. The influence of construction time on settlement of soft soil was also addressed in the present analysis.

The following conclusions were drawn from the current study:

- Numerical model was successfully validated with the field data of an embankment constructed on Bangkok clay.

- Consolidation time greatly came down from 45 years to two years' by installing 1m c/c PVD spacing into the soft foundation soil.

- Excess Pore pressures generated due to rapid embankment construction can greatly be reduced by introducing PVDs to a maximum extent and to some extent through DSM columns together. While constructing the embankment in stages, the maximum pressure that exerted by the pore water inside the soil mass have been observed to be 48 kPa in the case of untreated ground. Whereas in the PVD improved ground it was observed as 32 kPa. But in the case of PVD-DSM treated ground the excess pore water pressure drastically reduced to about 3.2 kPa (reduced ten times compared to PVD treated ground). The extensive reduction in excess pore water pressure can be attributed to the amount of stress distributed to the soft soil, is much less, in the presence of DSM columns.

  The effective stress distribution with in the foundation soil for different cases depicted that there is a slight reduction in effective stress in untreated ground during the last stage of embankment loading. However, with PVDs and PVD-DSM treated cases there is a considerable enhancement in the effective stress. It is found that the increase in effective stress with time was noticed high towards the centre line of the embankment than the outer slope of the embankment.
- The total stress distribution, after the consolidation period, on the DSM and soft soil shows that a major portion of stress is concentrated on the DSM columns (in the order of 280 kPa) against soft soil columns in between DSM columns.

- The deformation pattern with in the foundation soil clearly describes the failure in untreated soft soil during the final stage of construction. The total deformations are minimal in the case of combined PVD-DSM treated ground.

- Based on the safety analysis, it is noticed that the factor of safety value of the untreated ground even with PVD treated ground were very less (about 1.5) in the event of rapid embankment construction which is barely sufficient for the stable structure. But in the case of PVD-DSM treated ground the factor of safety can be drastically improved to the order of about 4 with a minimal area replacement ratio of DSM columns.

- PVD spacing has a great influence on the combined ground improvement technic. Closer PVD spacing with more DSM area replacement ratio treated ground allows faster rate of consolidation and significantly increases the embankment stability.

- The observed ultimate settlement for PVD-DSM treated ground with various DSM area replacement ratios were similar even though with the increased construction time periods. It can be concluded that total construction can be finished within six months with minimal area replacement ratio.

- In summary, a combined PVD-DSM column ground improvement technic can be adopted to simultaneously improve the consolidation behavior and shear strength of soft foundation soils. It can also be concluded that rapid embankment construction can be adopted with the proposed combined ground improvement. The required area replacement ratio of DSM columns for this purpose is much lower in the order of 5% or lower. A design example is presented at the end clearly explains the design steps followed in the design of combined ground improvement technic.

6.2 Future Scope of the Work:

A theoretical model can be developed to address and validate the numerical models presented in this study using a unit cell approach.

Improved material models can be adapted to model with further accuracy.

Design charts can be developed for the combined PVD-DSM treated ground for an easy design.

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Appendix-1

Design Example

Illustrative Design Example:

Construction of a 4 m height embankment with side slopes 2:1 (H/V) on a very soft foundation soil of 20 m. thick normally consolidated soft clay. Embankment crest width is 8 m. The ground water table is assumed to be at the ground surface. The embankment construction was taken up in three stages with a 90 day waiting period between stages for consolidation. Design PVD for 180 day time period with 99% degree of consolidation and design DSM columns in the same ground with appropriate area replacement ratio to support the rapid embankment construction.

PVD Design Steps:

1. The design of PVDs for a given soft soil condition can be done using a trial and error method. The design steps are briefly given below.
2. Calculate Tv; for a given cv, H of the soil strata, and time, t required for complete consolidation
3. Assume an average degree of consolidation due to radial and vertical drainage, U_{vh} = 0.95 or 0.99
4. Find Uh from steps 1 & 2. Use $U_{vh} = 1-(1-U_h) *(1-U_v)$
5. Assume some arbitrary spacing s and calculate de, n, F(n) and Th (use Th = cht/de2)
6. Then, find Uh from the equation given by Hansbo (1979), $U_h = 1-\exp(-8T_h/F(n))$
7. Compare Uh from step 5 with step 3.
8. If they are not equal, change the spacing and repeat step 5. When U_h matches with that calculated in step 3, then that is the design spacing.

Design of c/c spacing between two PVDs:

$c_v = 4.09\ m^2$/year (back calculated)

Time = 6 months = 0.5 years

Size of the PVD = 100X4 mm.

Equivalent diameter of PVD = a+b/π = 100+4/2 = 0.051m.

Height of the clay layer H = 20m.

Average degree of consolidation U_{av} = 99%.

Time factor $T_v = C_{v*}t/ H^2$

Where,

H is thickness of the clay layer.

Therefore time factor $T_v = (4.09*0.5)/ (20)^2$

T_v= 0.00511

From the T_v vs U_v graph, U_v is 8% i.e., 0.08

To design of the c/c spacing between two vertical drains,

$U_{av} = 1- (1-U_V) (1-U_h)$

$U_h = 1- (1-U_{av})/ (1-U_V)$

$U_h = 1- (1- 0.99)/ (1-0.08)$

Therefore U_h= 98.9 % $\longrightarrow$ (i)

Based on spacing value the formula for U_h is $U_h = 1- \exp [-8T_h/F (n)]$

$T_h = C_{h*}t/ d_e^2$

d_e = 1.13s (for square pattern of drains)

assume PVD c/c spacing = 1m

d_e = 1.13 (1) = 1.13 m.

$d_e^2 = (1.13*1)^2$

$d_e^2 = 1.27\ m^2$

$c_h = c_v$ (assumed as discussed by (Rixner et al.) [33]

From this known values of c_h, t and d_e^2, T_h = 1.60

[$c_v = 4.09\ m^2$/year (from back calculations)]

and $n = d_e/d_w$

n = 1.13 (1) /0.051 = 22.15

F (n) = ln (n) - 0.75 = 2.34

Therefore, $U_h = 1- \exp [-8T_h / F (n)]$

U_h = 1 –exp [-8(1)/2.34]

Uh = 99 % $\longrightarrow$ (ii)

So after doing the calculations the appropriate spacing for equalize the equations (i) and (ii) are equal. So our assumption is correct. The design spacing is s = 1 m c/c.

Equivalent vertical permeability of PVD

$$k_{ve} = \; 1 + \frac{2.26 l_d^2}{\mu d_e^2} \frac{k_h}{k_v} \; k_v \qquad \text{and} \qquad \mu = \ln\frac{n}{s} + \frac{k_h}{k_s}\ln s \quad \frac{3}{4} + \Pi \frac{2 l_d^2 k_h}{3 q_w}$$

Where,

d_e= diameter of the influence zone of PVD.

d_w = [(a+b)/2] = equivalent diameter of PVD .

Where,

a & b are thickness and width of the PVD.

$n=d_e/d_w$ = spacing influence factor (or spacing ratio) of PVD.

$s=d_s/d_w$ = smear disturbance ratio of PVD.

l_d= length of the PVD.

k_h & k_v = horizontal and vertical permeability.

k_s= smear zone permeability.

q_w= discharge capacity of PVD

By substituting all the known parameters, equivalent vertical permeability k_{ve} = 0.07 m/day

Design of DSM Columns

The design of deep soil mixing columns is based on area replacement ratio (A_r %) and center to center spacing of the DSM columns.

$$A_r = \frac{A_{col}}{A_{soil} + A_{col}}$$

$$\frac{Area\ \ of\ column}{Area\ of\ \ square} = \frac{\Pi d^2 / 4}{s \times s}$$

$$s = \sqrt{\frac{\Pi}{4A_r}} d_{col}$$

Assume A_r = 10 % and column diameter = 0.6 m

$$s = \sqrt{\frac{\Pi}{4(0.4)}}(0.6)$$

$\therefore$ c/c DSM column spacing = 0.84 m

Combined equivalent parameters for PVD-DSM improved ground to achieve ultimate consolidation settlement:

Ultimate consolidation settlement equation for PVD-DSM is,

$S_f = c_{c,\ composite} / (1+e_{0,\ eqlnt}) * H \log_{10} (p_f^l / p_{s,\ composite}^l)$

Where,

c_c = compression index

e_0= initial void ratio

H = height of the clay layer

p_f^l = Final stress (overburden + change in total stress)

$p_{s,\ composite}^l$ = Initial stress (initial overburden pressure)

Finding the combined equivalent parameters:

$c_{c,\ composite} = c_{s,\ column} * a_r + c_{c,\ soil} * (1- a_r)$

$= 0.04 * 0.1 + 4.6 * (1 - 0.1)$

$c_{c,\ composite} = 4.14$

$e_{0,\ eqlnt} = e_{0,\ column} * a_r + e_{0,\ soil} * (1- a_r)$

$= 0.8 * (0.1) + 2.3 * (1-0.1)$

$e_{0,\ eqlnt} = 2.15$

By using equilibrium equations and compatibility conditions one can find the unknown values of stress in DSM columns and soft clay.

Equilibrium equations:

$A_{soil} * \sigma_0 = A_{col} * \sigma_{col} + \sigma_{soil} (A_{soil} – A_{col})$

$0.554 * 71 = 0.28 * \sigma_{col} + \sigma_{soil} (0.554 - 0.28)$

$39.33 = 0.28\ \sigma_{col} + 0.274\ \sigma_{soil} \longrightarrow (1)$

Compatibility Conditions:

Settlement in DSM columns = Settlement in clay

$\sigma_{col} / E_{DSM} = \sigma_{clay} / E_{soil}$

$\sigma_{col} = \sigma_{clay} (E_{DSM} / E_{soil})$

E_{DSM} = 100 MPa

E_{clay} = 3.5 MPa

$\therefore \sigma_{col} = 28.57\ \sigma_{soil}$

$\therefore \sigma_{soil} = 4.76$ kPa (from equation 1) $\longrightarrow$ (2)

From (1) and (2)

$\sigma_{col} = 28.57 * 4.76 = 136.13$ kPa

Ultimate consolidation settlement due to combined equivalent PVD – DSM is,

$S_f = c_{c,\ composite} / (1+e_{0,\ eqlnt}) * H \log_{10} (p_f^l / p_{s,\ composite}^l)$

$S_f = 4.15 / (1+2.15) * (20) * \log \{(40 + 4.76)/40\}$

Ultimate settlement due to combined PVD-DSM = 1.27 m.

Finding settlement at any time (S_t):

U % = [Settlement at any time (S_t) / Ultimate settlement (S_u)] * 100

$\therefore S_t = S_u * U$.

By incorporating all the composite parameters and the stress acting on the soft clay, one can come up with the ultimate settlement of the composite ground which was showed in the above example. From this known value it is possible to draw the graph between design time versus degree of consolidation. Likewise design plots have been developed and presented in the below Figure AI.1 for this design problem to study the consolidation behaviour of the soft ground with various DSM Ar (%) and design PVD spacing. From this Figure AI., it can be observed that 40 % of DSM replacement has very good improvement in consolidation.

Design charts for PVD and DSM c/c spacing from theoretical analysis

Individual design charts have been produced for PVDs and DSM columns in the below Figures AI.2 and AI.3 for c/c spacing. Same calculations have been done in the design example.

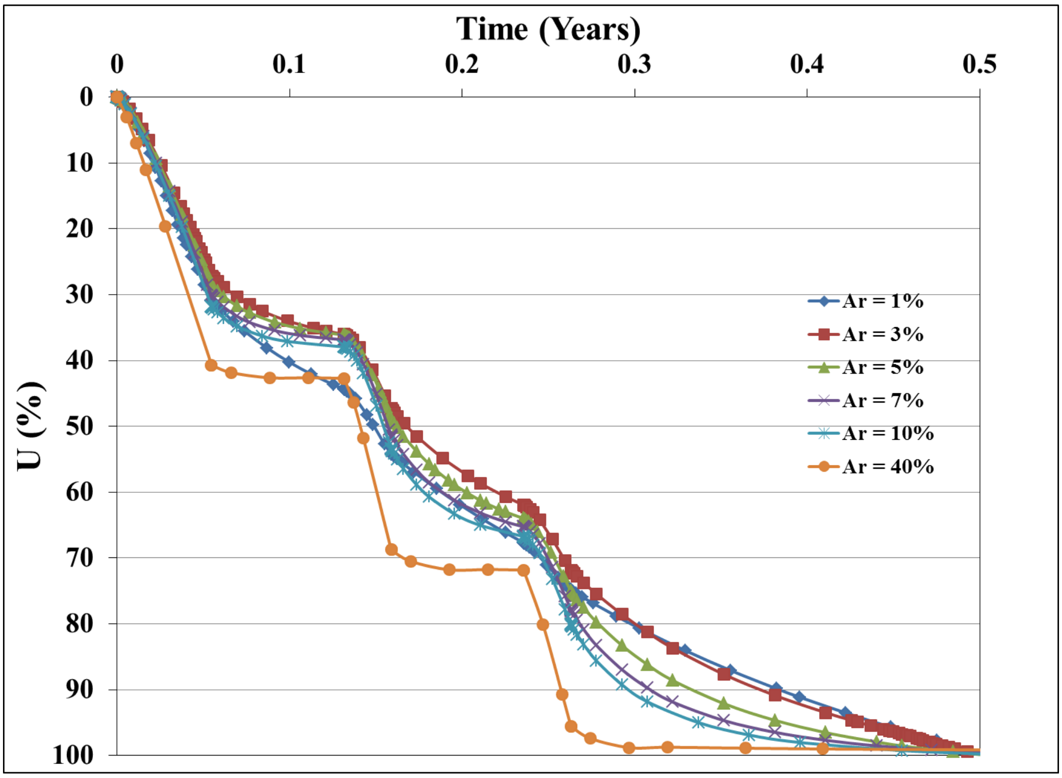

Figure AI.1: Variation of degree of consolidation with time

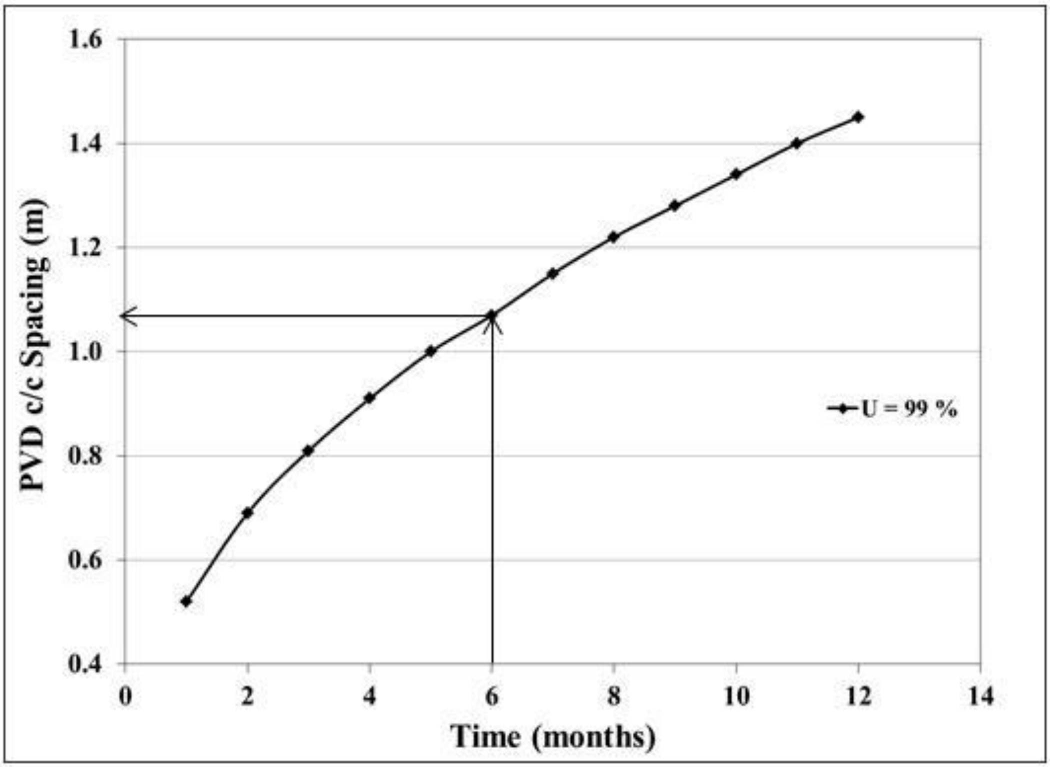

Figure AI.2: Typical time versus c/c spacing of PVDs in the present study

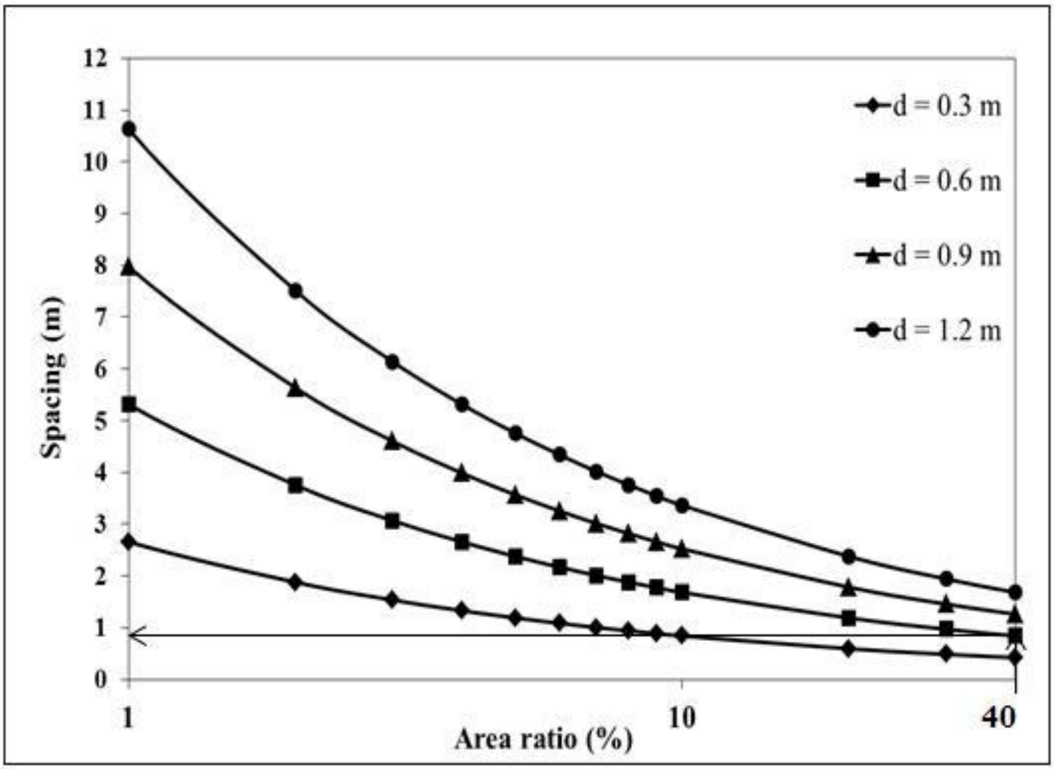

Figure AI.3: Representation of DSM area ratio versus spacing present study

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