Stability of High Embankments Founded on Soft Soils

by

Rakshith Badarinath

Submitted in partial fulfilment of the requirements
for the degree of Master of Applied Science

at

Dalhousie University
Halifax, Nova Scotia
April 2021

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

With the rapid urbanization and development of infrastructure, construction of structures must be carried out on all soil types. When problematic soils like soft clayey expansive soils are encountered, sound knowledge of the geotechnical properties of the soft soil must be required to carry out the construction over such deposits. This thesis focuses on the stability of high embankments found on soft soil deposits like saturated Marine clays. Staged construction of embankments over Marine clay and the use of Prefabricated Vertical Drains (PVD's) as means of ground improvement are studied in detail. It is seen that staged construction of embankments improves the shear strength of foundation soils, and the PVD's shorten the drainage paths for the excess pore pressures, thus improving the global stability of the embankments significantly. The embankment stability is studied in terms of settlements, factor of safety, excess pore pressures and lateral displacements.

ACKNOWLEGEMENTS

I would like to thank my supervisor Dr. Hany El Naggar for giving me an opportunity to work on a very interesting problem. His advice on problem-solving and timely guidance made this research very enjoyable. His passion for teaching and solving problems was immensely admirable, and I was extremely grateful to learn from his expertise.

I would also like to thank my supervisor committee, Dr. Mysore Satish and Dr. George Jarjoura for their support and guidance in my work.

My thanks also go to my family and friends; without their constant motivation and engaging discussions, this thesis would not have been possible.

Finally, I would like to thank the entire faculty and the staff of the Civil & Resource engineering department for making my master's degree a smooth and memorable journey.

CHAPTER 1 INTRODUCTION

1.1 Statement of the Problem

Sensitive Marine clays cover large parts of Canada and with the steady increase in the population of Canada; there has been a significant increase in the infrastructure facilities. These include the construction of several residential communities, Highways, Pipelines, Railway embankments, etc. which had to be carried over problematic soils like saturated Marine clays. Therefore, a clear understanding about the behavior and Geotechnical properties of Marine clays is necessary in order to carry out safe and cost-effective design over problematic soils.

In order to carry out the construction of embankments on soft soils like clays, the hydraulic conductivity plays a crucial role in estimating the settlements and the rate of consolidation. This essentially means that lower the hydraulic conductivity of the soil, higher will be its settlements and lower is the rate of consolidation. Saturated Marine clays (like the one documented in this thesis from St. Stephens, NB, Canada) gives rise to excessive primary settlements and hence construction of embankments on such deposits must be carried out either by modifying the foundation soil properties or by the use of Lightweight fills which reduces the magnitude of effective stresses applied.

The failure of embankments founded on Marine clay is primarily due to the very poor undrained shear strength of such soils. This gives rise to a deep seated circular failure which is ideally caused due to the loss in shear strength of foundation soils. Due to the very high natural water content of Marine clays, it takes a lot of time to consolidate (>90%) and a characteristic feature of Marine clays is the gain in strength when it is completely consolidated. Thus, when carrying out the design of embankments on marine clays, the strength gain property is accounted by constructing the embankments in stages and allowing time for the consolidation at the end of each stage.

However, for some projects where time is a constraint, suitable modification methods like the use of Pre-fabricated Vertical Drains (PVD"s) to accelerate the rate of consolidation could be utilized to reduce the consolidation period at the end of each stage and the overall period of construction. The staged construction technique is often used in combination with PVD"s to provide the most economical and safe design of the embankments found on soft soils.

1.2 Background Overview

In order to study the effects of the embankments constructed on soft soils, numerical modelling is carried out by Geotechnical engineers. With the modernization of computers and its processors, Finite Element Analysis has been found to be one of the most prominent means to numerically model a problem under consideration. The softwares used in Geotechnical engineering for numerical modelling include PLAXIS 2D & 3D, Geostudio, FLAC, Rocscience, etc. The embankment problem considered in this thesis is modelled using PLAXIS 2D.

Numerical modelling using Finite element techniques can be really tricky particularly because of its GIGO (Garbage-In Garbage-Out) nature. Therefore, sufficient theoretical background in Soil mechanics is required in order to evaluate the post-processing results. Earlier, the problems were analyzed using Mohr-Coulomb concept where it assumes an Elastic, perfectly Plastic behavior of the soils. This is an idealization and not particularly an accurate representation of the soil at the site, as it does not take the Strain Hardening/Softening into account.

With time, there were several soil models developed (Like the Modified Cam-Clay model, Hardening soil model) to account for the non-linear behavior and the Strain Hardening/Softening of the soils numerically modelled for the problem under consideration. In this thesis, Soft Soil model and Hardening soil model are used to numerically model the construction of embankments on Marine clay soil.

Defining the key material parameters plays an important role in getting the most accurate model, replicating the soil behavior at the site. There are certain correlations between the standard tests done on the soil at the site/ Laboratory and the Geotechnical properties of the soil like the Undrained shear strength, Cohesion, Angle of Friction, Unit weight, Voids ratio, Coefficient of Compression & swelling for soft soils and Elastic parameters like Young‟s modulus and Poisson‟s ratio. Some of the tests performed include Standard Penetration test (SPT) where the correlations are linked to the “N” value obtained from the boreholes cast at the site and the Cone Penetration Test (CPT) where the tip resistance values could be used to obtain the key material properties required for numerical modelling.

Back analysis is a key concept in numerical modelling as it is used to replicate the failure of the problem under consideration and the properties of soils are varied until a similar failure is observed in the model along with the similarities in settlements and excess pore pressures which can be monitored using instruments at the site. On calibrating the model, different ground improvement and construction techniques could be used in order to avoid the failure and a rate analysis is performed for the different techniques used to find the most economical solution. The drains could be designed using PLAXIS 2D and it could also be used to study the effects of using drains as line elements which are pre-coded, on the stability of high embankments found on soft soils. There have been several studies done on the numerical modelling of Staged embankments and the use of PVD‟s as a ground improvement technique which have been covered in the technical papers section.

1.3 Thesis contributions and Objectives

The main objectives and contribution of this thesis is to study the effects of Staged construction, the construction pace and the effects of using PVD‟s on the overall stability of High Embankments which are found on soft soils like Marine clay. The objectives and contributions of this thesis are divided into 3 sections. The key objectives are as follows:

- To replicate the construction and the failure of the embankment from a well-documented case study by Back analysis and soil properties are obtained through correlations by using SPT values which are provided after the failure occurred at the site.
- To carry out Staged construction of Embankments by allowing sufficient time for Consolidation and construction pace by placing the fills in thicker layers. Reporting on the benefits of staged construction on the global stability of the embankments and how it could be used to mitigate the failure of embankments on soft soils.
- To study the effects of PVD‟s which are primarily used to accelerate the rate of consolidation which also plays a pivotal role in strength gain of soils like Marine clay. Reporting on the increased global stability of using PVD‟s in combination with staged construction and also on the construction pace with varying fill thickness and consolidation period.

The key contributions are as follows:

- Upon successful Calibration of the numerical model from replicating the failure as observed at the site, some insights on the use of staged construction technique as opposed to rapid construction is provided.

- Reporting the benefits of using the drains which are used to accelerate the rate of consolidation along with the combination of staged construction technique on the overall stability of high embankments.
- Some design considerations with respect to the fill thickness, consolidation period, along with the area covered by the drains underneath the embankments are provided for the most economical and safe design of embankments on soft soils.

1.4 Thesis Outline

The thesis consists of 5 Chapters including this chapter and it consists of two technical papers. It is outlined as follows:

- Chapter 2 provides some background information about the Marine clay distribution in Canada, the mineralogical characteristics and structure of Marine clays in Canada, Geotechnical characteristics of Marine clays, brief information on some of the damages caused by marine clays to structures/properties, and finally some case studies involving staged construction technique and the use of PVD"s for embankments built on soft soils.
- Chapter 3 consists of Technical Paper I which studies effects of Construction sequence and the Construction pace on the stability of High Embankments found on Marine Clays.
- Chapter 4 consists of Technical Paper II which studies the use of PVD"s as means of ground improvement along with the combination of Staged construction on the Global stability of High Embankments on soft soils.
- Chapter 5 consists of a brief summary of the thesis and the conclusions of both the technical papers are presented in this section.

CHAPTER 2 LITERATURE REVIEW

2.1 Marine Clay background in Canada

Marine clay in Canada is mainly a product of proglacial and post glacial sedimentation after the decline of Wisconsin Ice sheet (Figure 2.1). [1] The Wisconsin Glacial Episode, also called the Wisconsin glaciation, was the most recent glacial period of the North American ice sheet complex. This advance included the Cordilleran Ice Sheet, the Innuitian ice sheet, the Greenland ice sheet and the massive Laurentide Ice Sheet. [2] The oldest sedimentation occurred in the south and the youngest sedimentation occurred in the north, where even today, the glacial lakes present.

It is estimated that the Wisconsin ice sheet was about 5000m thick around 20,000 to 18,000 years before present. Hence, there was a land displacement of about 1000m as a result of the weight of the existing ice sheet. [3] The Wisconsin ice sheet started to retreat in random stages where glacial lakes and seas were formed at its front as a result of melting. However, these glacial lakes shrunk with the rise of the land. [4] The proglacial and post glacial lakes formed as a result of melting of the ice sheet, the rate of rebound after the retreat of ice sheet, and damming of drainage terraces from the seas and lakes. Figure 2.2 shows the distribution of glacial lakes and marine clay deposits.

Early deposition of clay took place in the ice front in Freshwater lakes like Lake Eerie in southern Ontario. The clay deposit in Lake Eerie is rich in Illite, chlorite, calcite, and dolomite because its basin is dominated by Paleozoic shales and carbonates from the glacial erosion that produced these clay minerals. [4]

Figure 2.1: The shaded region indicates the location of the Wisconsin Ice sheet (Weir 1882)

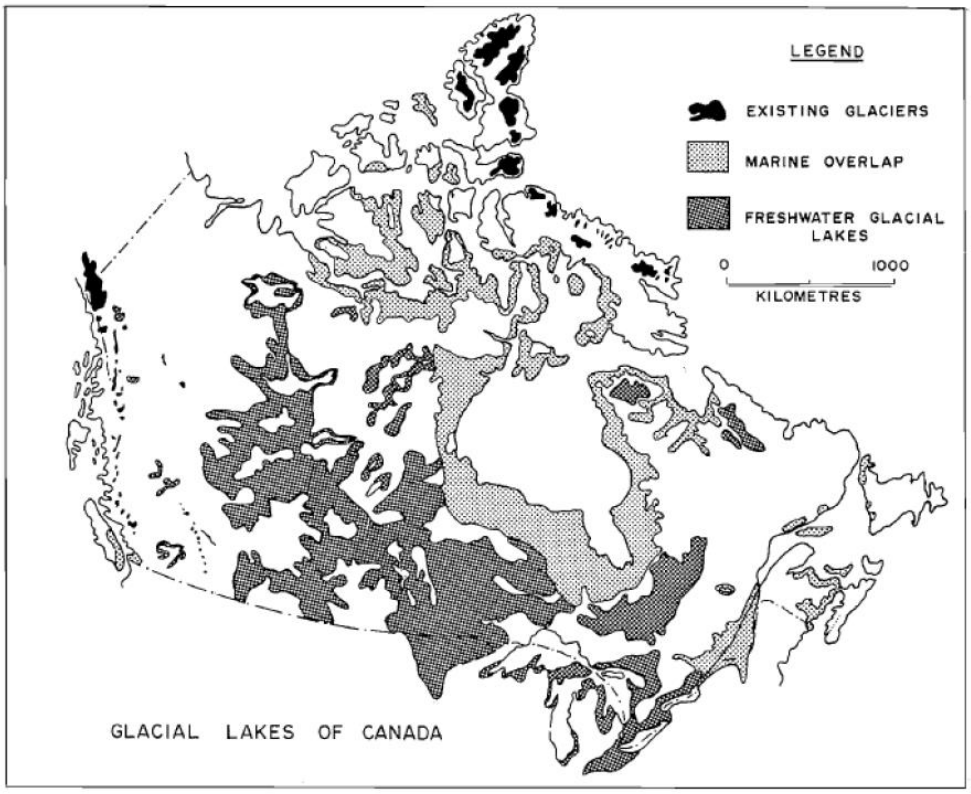


Figure 2.2: Glacial Lakes and Marine deposits in Canada (Quigley 1980)

Chlorite found in the clay deposits can be oxidized into Smectite through erosion and Smectite causes more activity in soil than Chlorite which results in active surface soil compared to deeper soils. [5] The Champlain sea (Figure 2.3) is believed to have invaded the lowlands surrounding the St. Lawrence River as a result of melting of an Ice dam near Quebec City and the Champlain Sea mixed with fresh water and Marine clays were deposited. In addition, an ice front was formed at the Northwest region of the Champlain Sea. This ice front experienced frequent freeze-thaw cycles, thus causing Marine clay deposits throughout this region. Several years later, the Champlain Sea was expelled and Laflamme Sea in the St. Jean region of Quebec was started by an ice front. Deep clay deposits in the region of North West Territories and the Prairies are formed as a result of this ice front. [4]

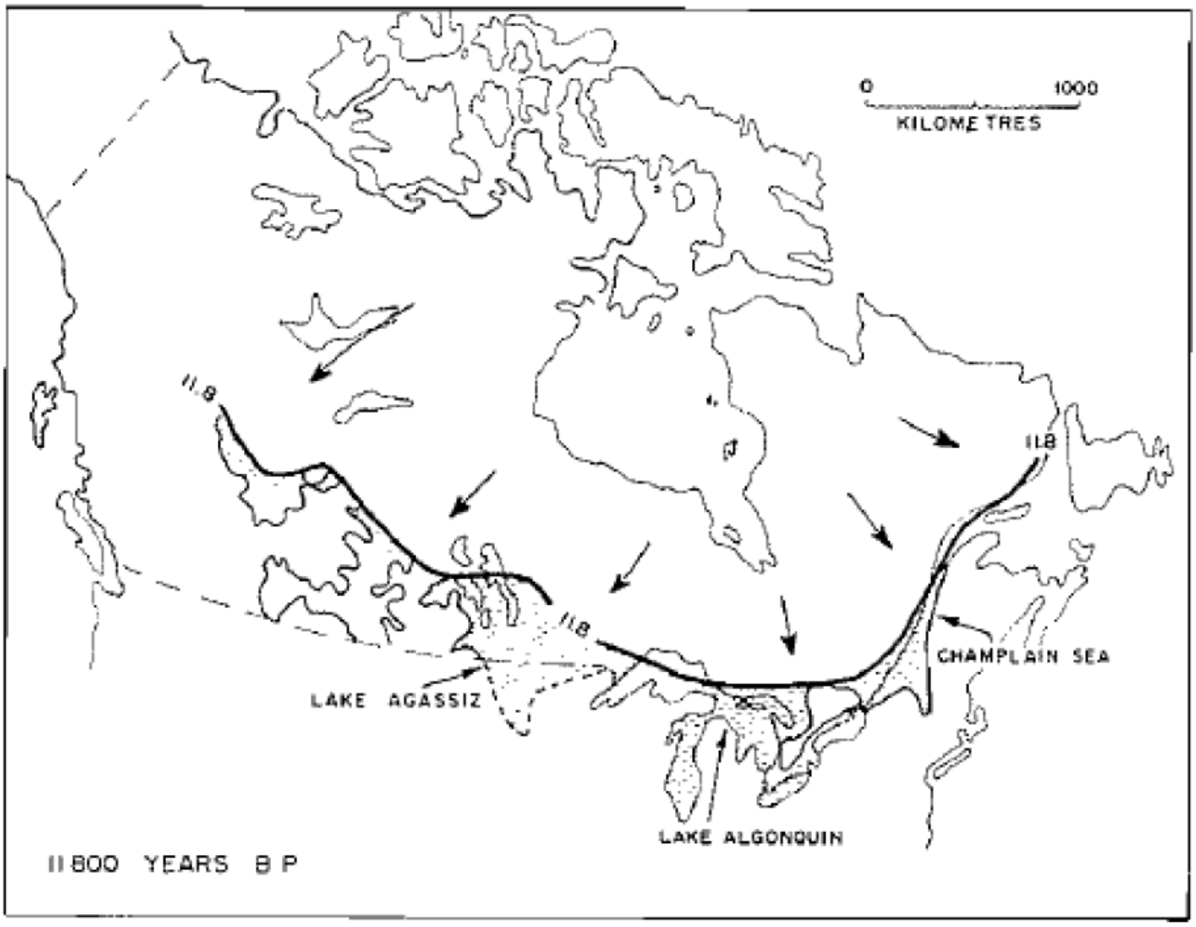


Figure 2.3: Various Ice fronts in Canada along with Champlain Sea (Quigley 1980)

Marine Clays in Canada are formed as a result of three main types of sedimentation process like Waterlaid tills, Lacustro tills, and Mudflows. Waterlaid till is a stratified variety of till that is deposited in water and usually overlain by a hard till layer. [7] Waterlaid till is lacustrine clay deposited below a shallow floating ice sheet and it may have varied depth from 1 m as in Alberta up to 30 m as in Sarnia, Ontario. [5] Lacrusto tills are formed in a lacustrine environment or they are deposited by a flow mechanism. These are submarine mudflows in glacial lakes and may include waterlaid tills as well. If the mudflow has enough momentum to travel under water, the Lacustro tills are deposited on the shores to form Turbidity current deposits. [8]

2.2 Mineralogical Characteristics & Structure of Marine Clay in Canada

2.2.1 Mineralogical Characteristics

The main minerals that are found in Marine clays in Canada include quartz, plagioclase, feldspar, amphibole, calcite, dolomite, phyllosilicates and amorphous matter. [9] It was found that minerals such as quartz, feldspar, and plagioclase, dominate the mineral content of the Champlain Sea clay. [10, 11, 12] Quartz, feldspar, and plagioclase minerals differ in quantities from one region to another. So, any one or two of them could be present in higher quantities than the others. For clay minerals in some sites such as St. Barnabe (Quebec) Illite is more abundant than Chlorite and Montmorillonite. It was seen that there is variation of clay minerals with depth and when the amount of Illite increases, the amounts of the other clay minerals decrease.

In other sites such as Henryville (Quebec), Illite and Chlorite are dominant in samples from all depths, but minerals like Montmorillonite are dominant over the first meter of the surface and also present at the base of the deposit soil. This indicates that the surface layer had been subjected to weathering and leaching. [4]

2.2.2 Structure of Marine Clay

Marine clay consists of negatively charged particles on their surface due to expansive minerals and these negative charges attract positively charged hydrogen ions of the water molecules. Thus, the bonding between clay particles and water molecules causes repulsion between the soil particles, and hence swelling of the soil occurs. A double layer concept was proposed to understand the structure of the clay particles. The concept stated that there is a negatively charged layer at the surface of the clay particles and this charge is expected to drop exponentially in moving away from the center of the soil particles. After the negatively charged layer, there is a stern layer and then a positively charged diffusion layer. The diffusion layer attracts negative electrons, such as water molecules (Cathodes). The negative particles in the absorbed water layer are called the immobile layer and this layer attracts positive electrons (Anodes).

The potential difference between the clay particles and the water is called the electrokinetic potential. If the electrolyte concentration of the water increases, the diffusion layer compresses and the repulsion between the soil particles decrease. [4] This essentially means that higher the electrokinetic potential, higher is the sensitivity of the Marine Clay. Figure 2.4 explains the double layer concept in detail.

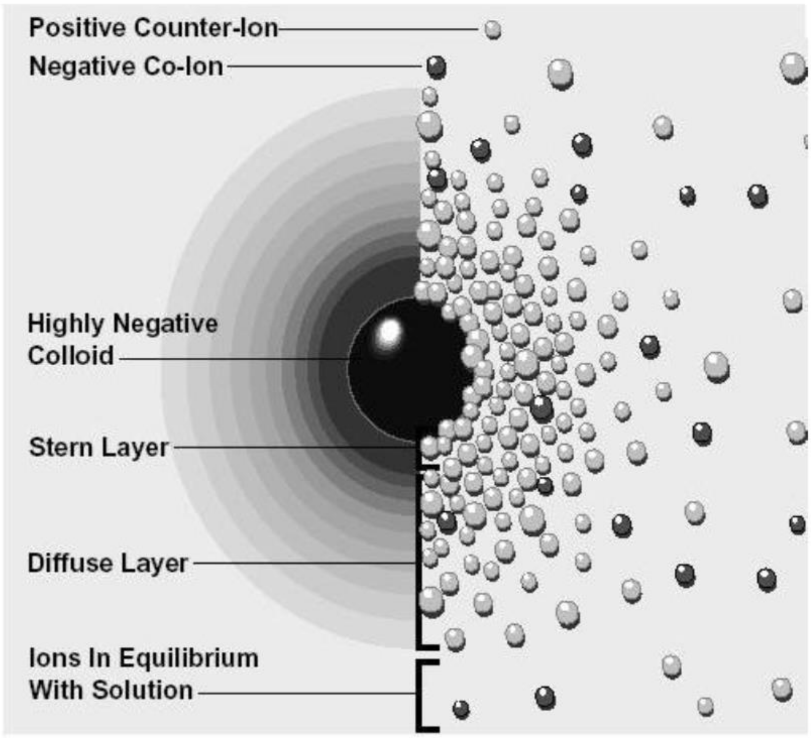


Figure 2.4: Concept of Double layer (Athir 2014)

2.3 Geotechnical properties of Marine Clay

The properties of Marine soils depends significantly on the initial conditions which maybe normally consolidated or over consolidated and its properties differs significantly from moist soil to dry soil. Marine clay is microcrystalline in nature and clay minerals like Chlorite, Kaolinite and Illite as mentioned earlier and non-clay minerals like quartz and Feldspar are also present in the soil. [14] Saturated Marine clays are extremely problematic as they have very high natural water content and consequently very less shear strength & bearing capacity.

Marine clay is made up of silt, clay and sand with low organic content. The percentage finer of silt and clay in marine clay constituents dominated about 78% to 88%. [15] The natural water content of most marine clays varies from 39% to 129% and the specific gravity of most marine clays is found to be around 2.55 to 2.65. The consistency indices for some typical marine clays ranges from Liquid limit (W_L) about 50% to 127%, Plastic limit (W_P) about

18% to 51%, and Plasticity index (I_P) was found to vary from 19% to 77%. The clay percentages measured from the soil samples varies from 22% to 66%, with most of the samples containing more than 40% of clay and the hydraulic conductivity of Marine clay varies typically from $1x10^{-9}$ to $2x10^{-9}$ m/s depending on the percentage of fines in the soil sample. The main compressibility parameter considered is compression index (c_c) and it typically varies from 0.21-2.6 which depends on the particle size distribution, the constituent mineral particles and the location of the soil. The typical initial void ratio ranges from 0.43 to 3.36 with most of the voids filled by water due to the high natural water content of such soils. The coefficient of consolidation, defined as the rate of consolidation, typically ranges from 0.15-3.7 m^2/year depending on the type of the marine deposit at a location. [16]

Marine clay can be often described as soft sensitive soil that poses to be problematic with high settlement and instability, poor soil properties that are not suitable for construction, very low shear strength, and low unconfined compressive strength of 25-50 kPa. Marine clay also exists at the bottom of water bodies and reservoirs such as rivers, lakes and streams. This kind of marine clay is called degraded marine clay and it is usually excavated for the purpose of enlarging the rivers, controlling flood, and water retaining structures owing to its very low hydraulic conductivity. It is extremely challenging for engineers to work with marine clay soil due to its expansive nature. It swells drastically when the moisture increased and shrinks when decreased. [17]

There is a significant difference between the saturated marine clay and any saturated soils in terms of engineering properties. Marine clays are extremely susceptible to changes in the moisture content (unlike other clay soils) and when they are the foundation soils for pavements/embankments or foundation of structures, they are unstable and their performance is often unpredictable. Therefore, marine clay is often treated to improve the properties of such soils before it is a suitable foundation material for construction of Building foundations or embankments which are found on soft marine clay deposits. It is seen that for most of the cases, high compressibility, low shear strength and the sensitivity (expansive nature) leads to the failure of foundation soils. When the Marine clay deposits are above water table or in dry condition, the marine clay is very strong but it loses its strength even when partially wetted, thus explaining its extremely sensitive nature. Cracks appear on its surface when dried and the cracks width is almost 250-500 mm with 1 m depth (Extremely brittle when dry). [17]

2.4 Damages caused by Marine Clay to properties

2.4.1 Damage to Foundation footings

The most common problem in Marine Clay areas is the settlement and heave of foundation footings. Foundation footings support the house and distribute the load to the underlying soils. The footings are located well below the ground surface and during dry periods, especially the summer months, the soil loses moisture. This causes the clays to shrink, leaving a void, or gap, under the footings and hence bearing support for the footings is lost. The buildings settle, usually in an uneven fashion, resulting in broken footings, cracked masonry walls, interior cracks in plaster, and warped door and window frames as shown in Figure 2.5. [18]

2.4.2 Damage to Foundation walls

Damage to foundation walls in Marine Clay soils often occurs when high shrink-swell clays are placed in the backfill against basement foundation walls and retaining walls. The potential swelling pressures of the Marine Clays far exceed the design strengths of typical house foundation walls and have resulted in failure of such structures. Damage from the pressure of swelling Marine Clays in backfill does not normally occur immediately after construction.

Figure 2.5: Cracks caused due to the settlement of Marine clay deposits (Fairfax County publication)

The clay must often go through several yearly cycles of shrinking and swelling before the detrimental effects of the soils are revealed through cracked or failed walls. The clays in the backfill gradually settle under their own weight as years pass, increasing in density and exerting more stresses against walls. In addition, soil particles fall into the surface cracks formed in the clay backfill during dry periods of the year and cause an increase in pressure against the wall when swelling again occurs. This is illustrated in Figure 2.6. [18]

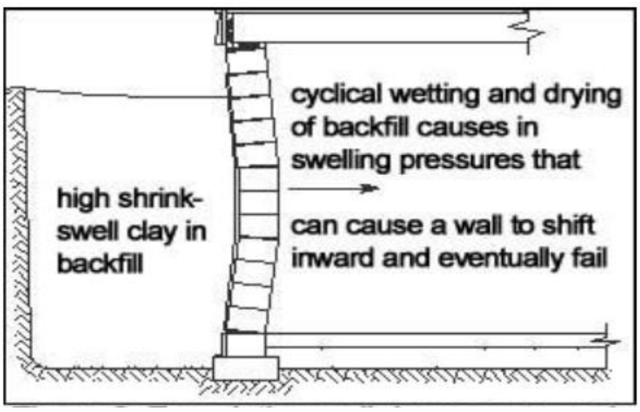


Figure 2.6: Shrink-swell cycle of Marine clays leading to failure of foundation walls(Fairfax County publication)

2.4.3 Damages caused to structures due to Landslides

Damage to structures from landslides in Marine Clay soils can be dramatic. Slope failures have jeopardized buildings or utilities and made parts of yards unusable. Landslides are more common in landscaped yards and undeveloped areas, especially if they are highly sloped. In landscaped areas, landslides often occur in clay fills that were placed at the time of original construction. Slope failures occur most commonly during wet periods of the year when the soil moisture is at a maximum. Where houses are involved in landslides, movement is usually very slow and can be corrected through routine inspections, although at a significant expense. [18]

2.4.4 Damages caused to Floor slabs and Roadways

Damage to floor slabs and roadways occurs when underlying clays shrink and swell. Road surfaces may deteriorate prematurely. Curbs and gutters may settle excessively. Swelling pressures under floor slabs can cause a damaging uplift of the floor, sometimes requiring replacement of the floor slab. Damage can be minimized by removing high shrink-swell clays under floor slabs and roadways to some depth, usually one to two feet, and replacing the clays with compacted non-swelling soils. [18]

2.4.5 Yard Drainage problems

Yard drainage problems occur in Marine Clay soils because water percolates very slowly through the clays and collects in flat yard areas. It is important to maintain positive drainage away from houses and throughout yard areas. Positive grades slope away from a house and help prevent water from soaking into the ground next to the house and causing potential foundation problems. Rainfall runoff may stand for long periods in areas that do not have at least a two percent slope. [18]

2.5 Problems posed by Marine Clay to the construction of Road Embankments

When Embankments are proposed to be constructed whose foundation soils are soft soils like Marine Clay, it becomes very problematic because of the high deformability and high sensitivity of such deposits. Since the construction of embankments takes a lot of time to complete, preference should be given to consolidation of foundation soils. One of the most important characteristics of Marine clay soils is the strength gain associated with it. This essentially means that as water is consolidated from such soils; it settles rapidly and as the pore pressures are dissipated there is an increase in Cohesion and consequently an increase in its shear strength. Thus, engineers need to take this into account for carrying out the design and construction of embankments on soft soils.

The embankments are often constructed in staged construction technique. Staged construction consists in the filling of an embankment at a controlled rate, so as not to cause failure but to permit an increase in shear strength due to consolidation (Figure 2.7). This is done to gradually increase shear strength while excess pore water pressure dissipates, void ratio decreases, and the embankment settles. Staged construction not only helps increasing the bearing capacity of the foundation soils but also reduces the post construction settlements. [19] The stability of the embankment needs to be checked for safety at each stage and the strength gain of the soil needs to be verified in order to continue the construction of the next stage. However, fill placement causes vertical compression and lateral deformations toward zones of lower confining pressure (Heaving). Since tall embankments are not fully constrained in lateral directions, the settlement problem is magnified. This thesis attempts to understand the engineering challenges posed by Lateral and Vertical deformations due to the applied embankment loads.

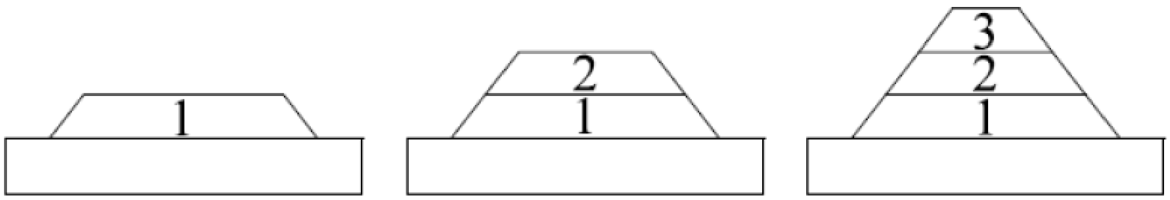


Figure 2.7: Staged Construction technique showing fills placed in 3 layers (Bozkurt 2014)

Staged construction uses controlled rates of loading to enable soil strengthening via consolidation in order to increase the foundation stability of dams, embankments, landfills, and tanks founded on soft cohesive soils. It is also used for the operation of many tailings waste storage dams.

When embankments are constructed using a staged construction technique, the soil properties must be determined accurately before construction and they should also be updated during the subsequent stages of the construction. The parameters of soft soils depend on the size of the loading area, load intensity, rate of construction, and consolidation period between each stage. Therefore, the best design and construction practices for embankments involve an evolutionary process that must be also flexible enough to accommodate modifications throughout the construction stages. With the development of new numerical analysis techniques over the last two decades, it is possible to more realistically model soil behavior and complex construction processes in advance by making use of appropriate soil models. Modern softwares like PLAXIS present solutions under Meshing, different boundary and initial conditions for complex geotechnical problems such as staged construction of an earth fill. [19]

2.6 Case studies of failure of Embankment constructed over soft soil

This section consists of previous failures of Embankment which were founded on soft soils like Marine Clay.

2.6.1 Failure of an interstate connecting-ramp

The proposed embankment was 91 m long which was a section of an interstate connecting-ramp (Ramp ES) between westbound Interstate-76 (I-76) to southbound Interstate 71 (I-71) in Medina County, Ohio. After placement of only 2.4 m of the embankment fill, or just over one-quarter of the long-term embankment height of 9.2 m at this location, tension cracks developed along the crest of the embankment. After the embankment height reached about 43% (4.0 m) of the long-term height (9.2 m), a 91 m long section of the embankment failed in 2007. The maximum embankment height during construction was to be 9.7 m to reflect a 0.6 m surcharge to preload the foundation soils that would be removed before pavement placement. Figure 2.8 shows the cross section of the soil profile at site. [20]

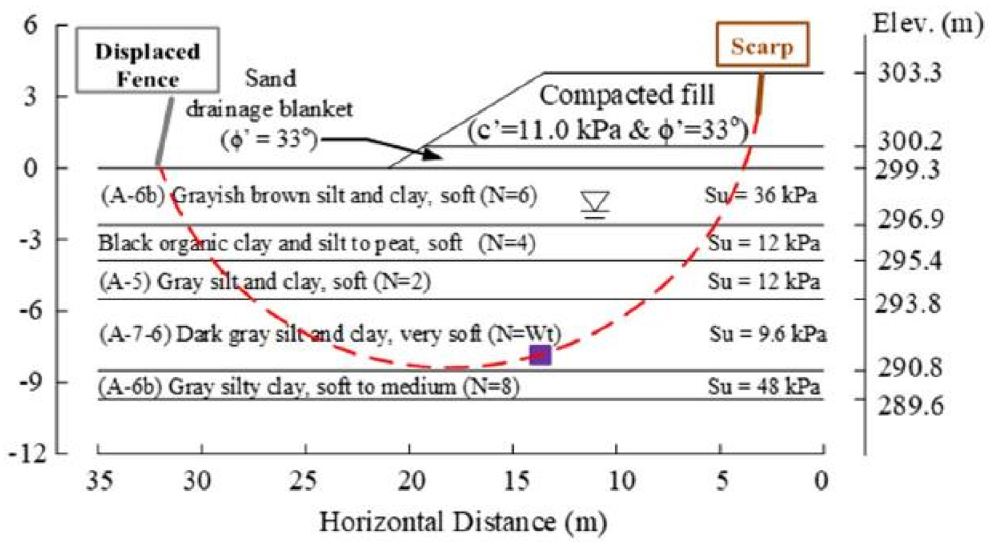


Figure 2.8: Sub soil profile for the failed embankment at the site and representing estimated undrained shear strength of the layered soils (Ghada 2020)

2.6.2 Failure of two Highway Bridge approaches

Two highway embankments constructed with mechanically stabilized earth (MSE) failed in 2006 near Kolkata, West Bengal state, India. The two sites of failures are represented as KM

18 and KM 26 which were founded on soft and compressible, fine-grained soils of the intertidal flats and back-swamps of the Ganges delta. KM 18 site remains waterlogged throughout the year and KM 26 site, also remains waterlogged over prolonged periods. Both the embankments were retained by MSE walls constructed with compacted hydraulic fills reinforced with galvanized steel reinforcements.

2.6.2.1 Failure at KM 26 site

The first event affected the highway interchange at site KM 26, which was under construction since July 15, 2003. The structure was retained by a MSE wall along the outer shoulder and with the fill slope of 2H: 1V along the inner shoulder. The nearly complete MSE wall underwent a deep seated failure in the early hours of a heavy downpour and an additional 0.4 mm of rainfall on the following day. The height of the affected embankment was between 8.9 and 9.8 m at the time of the incident. Post failure inspection indicated that the MSE wall failed due to external instability without significant internal distress. [21] This essentially means that the foundation soil failed resulting in the highway interchange causing significant earth pressures on the side of the wall leading to its failure. Figure 2.9 shows the sub soil profile at site prior to its failure.

2.6.2.2 Failure at KM 18 site

The second event involved an MSE wall that runs along the western edge of the northbound lanes of a highway approaching a railway overpass. A 30-year old, 9-m high earth embankment with 3H to 1V side-slope along the eastern edge of the approach carries the southbound traffic.

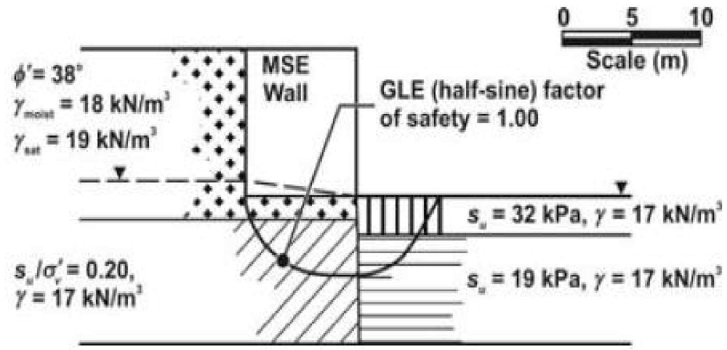


Figure 2.9: Deep seated failure observed at site KM 26 due to failure of foundation soil (Roy 2008)

The failure occurred immediately after midnight on February 9, 2006 about a month after the highway was opened for vehicular traffic. During the failure, a section of the newly constructed 2-lane approach vertically settled by about 3 m and laterally translated outward by about 1 m. As at KM 26 site, the MSE wall at KM 18 site appeared to have failed due to external instability without significant internal distress indicating the foundation soil underwent a deep seated failure. [21] The cross section of the soil profile at site KM 18 is shown in figure 2.10.

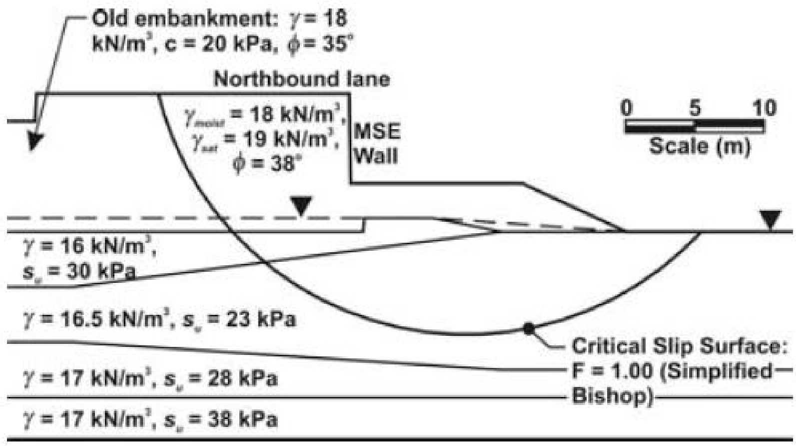


Figure 2.10: Deep seated failure observed at site KM 18 due to failure of foundation soil (Roy 2008)

2.6.3 Failure of a road Embankment

The Paphos district in Cyprus has a long history of slope instability particularly in areas of high elevation. Cyprus is known to be seismically active, particularly along the south coast with most of the rainfall in this region confined to the summer and spring months. Recently there has been a lot of development in this region consisting of roads which connects the remote villages. Thus, this led the construction of roads in Landslide prone areas. The road linking the villages of Nata–Pentalia-Panayia (along P453-P513) had already displayed signs of instability in January 2002 while the road was still under construction. The development of cracks in the road was assumed to be caused due to higher than usual rainfalls and snowfalls.

Despite the preliminary reparation measures which involved re-pavement of the asphalt following the construction of the road, more cracks continued to appear. Following the rainfalls of winter of 2004 preliminary investigations suggested that, although it was possible that the road traversed older and deep failures the most likely slip surface was in fact shallow. In 2005, there were still no major remedial measures implemented, apart from localized repaving of the road to fill cracks. By the summer of 2005 there were clear signs of settlement and the development of further cracks. Following the construction of a terrain model and further observations it was recognized that a slip surface, at a distance 80 m further than the first analysis, was deeper than originally assumed. [22] There were clear signs at the site that indicated the failure of the soft soil underneath and shear strength was significantly lost due to the accumulation of water following heavy rainfall. Figure 2.11 shows the cracks developed at the surface of the road embankment at the site.

Figure 2.11: Cracks developed at the Pentalia road in July 2005 (Hadjigeorgiou 2012)

2.6.4 Failure of an Expressway Embankment

The south extension line was constructed as a new part of the Xintai Expressway in Guangdong Province, China. The project required the construction of an embankment over soft soil with a thickness of up to approximately 14.0 m and the total length of the embankment section was around 5kms. After a rainstorm, the constructed embankment exhibited some failures in different sections. The two main sections which were analyzed were a cracking section and a sliding section along the length of the Embankment. The cracking section is located in the alluvial plain, and a water body is distributed on both sides of embankment. During the embankment unloading process, the embankment had evident macroscopic deformation, and several large cracks had appeared on both sides of the drainage ditches. The right drainage ditch was dislocated by 6–18 mm, cracked by 3–12 mm, and sunk by 5–11 mm. [23] This shows that the foundation soil had failed due to the accumulation of water and the consequent loss of shear strength.

The sliding section was located in the bridge approach, and the embankment was surrounded by farmland. After the embankment construction was completed, the right and left embankments slid successively. The length, width, fill height, maximum ground heaving height, and ground heaving width of the right sliding embankment are approximately 110 m, 10 m, 6.9 m, 2.177 m, and 17.3 m, respectively, whereas those of the left sliding embankment are 110 m, 8 m, 6.9 m, 2.021 m, and 26.2 m, respectively. [23] As there was heaving on both the sides of embankment, this was attributed to the bearing capacity failure of the foundation soft soil. Figure 2.12 shows the embankment failure in sliding observed at the site.

(a)

(b)

Figure 2.12: Sliding observed towards (a) The left of the embankment (b) The right of the embankment (Shaofu 2020)

2.7 Ground improvement using Pre-Fabricated Vertical Drains (PVD's)

There are various ground improvement techniques available to improve the properties of Marine clay and other techniques of construction through which structures can be constructed whose foundation soils are predominantly soft. Some of the common techniques include the use of drains to accelerate the rate of consolidation, use of stone columns to increase the bearing capacity of the soil, use of lightweight fill to decrease the effective stresses on the soft soil, staged construction technique as discussed earlier, helps in strength gain of soils like Marine Clay, etc. Since this thesis is done to study the effect of drains on the global stability of the embankment, the literature studied include only the use of PVD‟s to facilitate the construction of high embankments on soft marine clay.

Prefabricated Vertical drains generally have a rectangular cross section consisting of a filter fabric sleeve or a jacket surrounding the plastic core. The fabric sleeve acts as a physical barrier separating the core and the surrounding soil but permits pore water to enter the core through which it is dissipated. The various materials which are used to fabricate the drains include non-woven Polyester, polypropylene geotextile or synthetic paper. The plastic core has grooved channels which act as flow paths and this supports the filter sleeve. [24]

PVD‟s are usually installed by static or dynamic method. In the dynamic method, the mandrel is driven into the ground with a vibrating or drop hammer, but in the static method the mandrel is driven by means of a static load. The static method usually cause less ground disturbances and is preferred for sensitive soils. Although it might be faster, the dynamic method generates higher excess pore pressures and disturbs more soil around the mandrel. [24]. The practical use of drains which shortens the drain paths, is illustrated in figure 2.13.

Surcharging means to pre-load soft soils by applying a temporary 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 wick drains are used to shorten pore water travel distance, reducing the preloading time. The intent is to accelerate primary settlement. Pore water 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. [25]

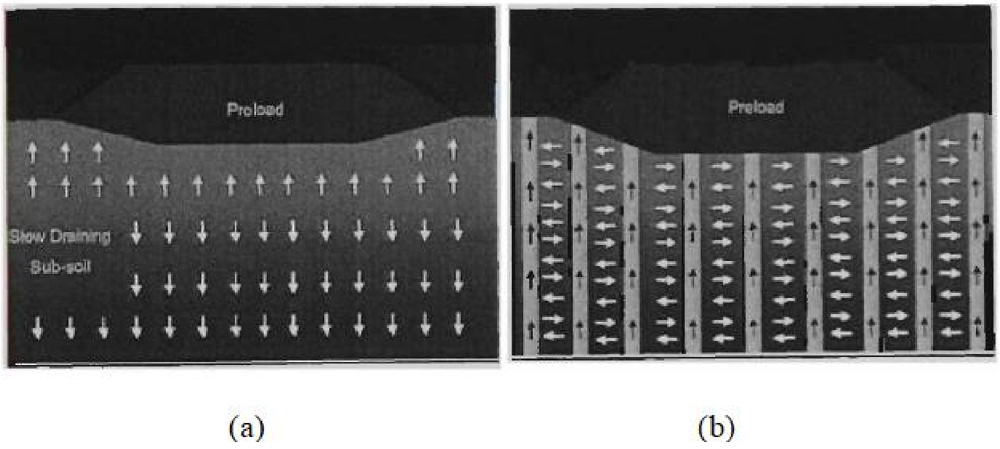


Figure 2.13: Drainage path of excess pore pressures (a) Without drains (b) With drains (Sathananthan 2005)

There are two main factors influencing the design of PVD‟s to modify the soft foundation soil. These are smear effects and well resistance. Smear effect has to be considered due to the disturbance of the soil surrounding the mandrel during its installation. Well resistance is the resistance to the flow of pore pressure inside the drains. During the installation of the drains it is possible that sometimes the soil can creep into the drain, thus clogging it. This affects the discharge capacity of the drains and hence, providing resistance to drain the excess pore pressures out of the foundation soil. The drains are usually installed in patterns which vary from rectangular to triangular depending on the purpose of using the drains and the area of the foundation soil which has to be consolidated.

With the development of more efficient methods and equipment for prefabricated vertical drain installation, and the increasing ability to work through difficult site and soil conditions, prefabricated vertical drains are often the most cost effective and expedient solution when combined with surcharge to achieve drainage and faster settlement and strength gain of soft soils. [26]

2.8 Case studies of Pre-fabricated Vertical Drains as means of Ground improvement

2.8.1 Craney Island Eastward Expansion, South and Division Cross Dikes Stage 1, Portsmouth, VA.

The site consisted of expanding over 200 hectares of open water to the east of the existing Craney Island confined disposal facility. The subsurface conditions consisted of 3 meters of water to the seabed (mostly sand) followed by 12 to 35 meters of soft normally consolidated Holocene clay. Due to the presence of thick soft clay deposits, the principal challenge was to maintain the stability of the dikes that would be constructed to confine the expansion. PVD"s were installed at the site to a maximum depth of about 46 meters below the seabed level. This was performed over an area of about 17 hectares with a drain spacing of 1.5 meters arranged in a triangular pattern. Approximately 3.6 million linear meters of prefabricated vertical drains were installed in 2011. This project site was unique as it involved the offshore installation of PVD"s. The drains on this project were required to function for over a decade because of the long period of time necessary for the staged construction of the dikes. [27] Figure 2.14 shows the installation of the PVD"s at the site.

2.8.2 Phase 1 Upland and Wall Fill, Charleston Naval Base Container Terminal, Charleston, SC.

The site consisted of 113 hectares which was primarily an inactive dredge spoil basin and filled tidal marsh. The container terminal site elevation was to be raised by an average of about 3 m, and hence the primary site stabilization challenges were large settlements and to control the post construction settlement.

Figure 2.14: PVD installation at the site (Goldberg 2013)

The project involved about 15 hectares of the drains installed at a 1.5-m triangular spacing and approximately 73,000 drains were installed to an average depth of 20.7 meters below the ground level. The subsurface conditions consisted of sandy soil from the ground surface to an average depth of 2.4 m and soft fine-grained deposits, generally consisting of compressible clay and silt, were encountered across the PVD area. The thickness of the soft normally consolidated deposits ranged from about 7.3 m to 22 m with an average of 13.7 m. [27] The drains were installed as part of a surcharge program design to reduce the post-construction settlement for the container yard and it resulted in 150 mm of post-construction settlement under a pressure of 12 kPa approximately which was uniformly loaded over the surface. [27] Figure 2.15 shows the installation of PVD‟s at the site.

2.8.3 Port of Gulfport Restoration West Pier Fill Phase 1, Gulfport, MS.

The site consisted of 28 Hectares which was previously an unpaved storage yard, with a portion being the open waters of the Mississippi sound. The primary site stabilization challenge was control of post-construction settlement. This project included about 18 hectares of drains installed at 0.9-m spacing in a triangular pattern and approximately 250,000 drains were designed to an average depth of 19 meters.

Figure 2.15: Installation of PVD at the container site (Goldberg 2013)

The subsurface soil conditions generally consisted of sandy fill from the ground surface to an average depth of 6 meters. Soft fine-grained deposits, generally consisting of compressible clay and silt were encountered across the vertical drain area. These normally consolidated soft marine deposits ranged in thickness from about 7.5 m to 15 m. The drains were designed to mitigate post-construction settlement for the container yard along with preloading the surface with a surcharge load of approximately 48 kPa to mimic container yard loading. [27] Figure 2.16 shows the drains installed at 0.9m spacing.

Since this thesis focuses on the use of PVD‟s for the construction of high embankments over soft soil, some literature on the uses of the drains for embankments is presented further.

Figure 2.16: PVD‟s installed at 0.9m at the Gulfport site (Goldberg 2013)

2.8.4 Construction of a Railway Embankment using PVD's

Bangladesh Railway had started construction work of a new 32.4-km railway embankment along Kashiani–Gopalganj section. The project site resting on soft soil and the soil had to be treated to facilitate the construction of the proposed railway embankment which proved to be challenging. The subsurface soil profile consisted of soft compressible layers like silty clay, clayey silt, and fine silty sand, which varied from 4.5 to 16.0 m. As the consolidation process was governed by the rate of excess pore pressure dissipation, shortening the length of pore water flow paths greatly reduces the consolidation time. PVD‟s were used to improve the ability of the soil to support construction and accelerate the rate of settlement. Most soil deposits had greater permeability in the horizontal direction than in the vertical direction. The total length of the track alignment was divided into 18 sections consisting of PVD‟s design & its spacing and staged construction of embankments. The project required the construction of embankment with an average height of 5 to 8 m along its total length. [28]

Since the underlying soil was found to be soft silty clay, ground improvement by installation of PVD‟s followed by preloading had been proposed. In the ground level of the railway embankment, a total number of 4,402,326 PVD‟s were used. Values of undrained shear strength of soft soil were assumed to vary with depth. [28] The drain spacing was assumed to vary between 1.0m and 1.5m c/c where lesser spacing was used for soil deposits with very fine grains whose permeability in the horizontal direction was more than the vertical direction. Figure 2.17 shows the sectional elevation of the subsurface soil improved by installing PVD‟s.

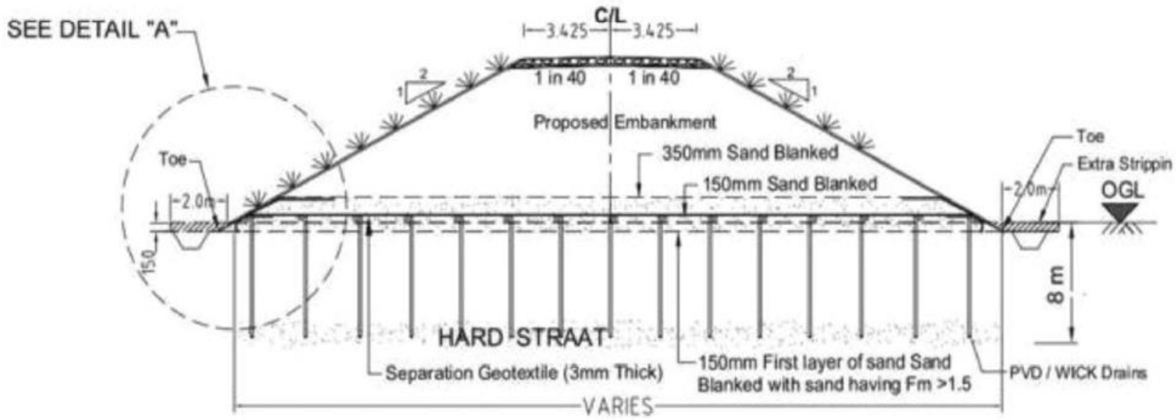


Figure 2.17: Sectional elevation of the embankment on soft soil improved by installing PVD‟s (Ripon 2020)

2.8.5 Reconstructing a failed embankment by improving the ground using PVD's

The site consisted of the failure of a 91m long section of a connecting ramp embankment (RampES) between westbound Interstate-76 (I-76) and southbound Interstate-71 (I-71) in Medina County, Ohio. A key feature in the design of Ramp ES design was the use of drains to accelerate consolidation and increase shear strength of the soft foundation soils under the ramp embankment. It was only after 2.4m of embankment fill placement, or just over one-quarter of the full embankment height, of 9.2m at this location, tension cracks began to appear along the crest of the embankment (Figure 2.18). After the embankment height, reached approximately 43% (4m) of the full height (9.2m), a 91-m-long section of the embankment failed in 2007. [29]

The subsoil profile consists of very soft to soft silty clays, sandy silts, and organic clays ranging from depths of 0.3 to 11.6m had organic contents ranging from 3 to 84% and depths of the groundwater ranged from 0.4 to 7.5m with the average depth being 3.2m. The PVD‟s were installed with a spacing of 1.8m in equilateral triangle spacing, with an effective drainage area of approximately 2.9 m^2. [29] A 0.9-m-thick sand drainage blanket was placed on the existing ground prior to PVD installation to facilitate drainage of water subsequently emerging from the PVD‟s due to consolidation. The material used for the drainage blanket was fine sand, and it was placed at a rate of 0.3m per day. The embankment fill material used for the construction of Ramp ES on top of the sand blanket consisted of approximately 90% fine-grained soil and 10% broken concrete pavement and a gravel base.

The PVD‟s were installed with an anchor plate to keep the drain at the required depth when the T-shaped mandrel was removed during installation. The PVD‟s were installed to depths that penetrated the weak foundation layers completely and it was continued into the underlying firm soil. [29] It was seen that there was a significant improvement in the performance of the embankment as the shear strength of the foundation soil was increased.

2.8.6 Use of PVD's in a Railway Embankment in Tamil Nadu, India

The additional land had to be reclaimed from the sea at Ennore, Tamil Nadu to construct new railway lines and the reclaimed land was approximately 45m wide & about 1000m long. The embankment had to be constructed over the reclaimed land to a height of 4m before the additional railway lines could be constructed. The subsoil profile consisted of very soft marine clay from the Ground level to depths varying from 10m to 15m. Below the very soft clay, stiff clay up to depth more than 20m was present. The very soft marine clay had very low shear strength (<20 kPa) which is classified as highly compressible soil with high moisture content. [30]

Figure 2.18: Tension cracks appeared on top of the embankment at site (Timothy 2017)

Since the Marine clay extended to large depths, it was not possible to construct the embankment within a reasonable period without ground improvement. The method finally chosen for the ground improvement was to install PVD‟s and to construct the embankment in two stages. PVD‟s were installed up to the full depth of the soil at a spacing 1.25m c/c over the entire area of reclamation to accelerate the consolidation of the soft marine clay under imposed load. The PVD was installed with a steel mandrel using constant rate of penetration and the maximum depth of installation was 17m below the ground level. [30]

A sand drainage blanket of approximately 300mm was placed on top of the installed PVD to allow free drainage of expelled pore water from the band drains. A non-woven geotextile filter fabric was provided over the sand drainage blanket to prevent any contamination of the sand drainage blanket from the earth fill during embankment construction. It was concluded from this case study that The construction of a 4m high railway embankment on land reclaimed over very soft clay 15m deep was achieved satisfactorily within a short period of time by installing PVD‟s and adopting a two stage construction procedure. [30]

It can be seen from the case studies reported that the use of PVD"s clearly increased the global stability of the embankments and in some cases, the embankments were reconstructed with the drains to accelerate the rate of consolidation. Thus, this thesis is aimed to study the effects of using drains along with the staged construction technique on the overall stability of the embankments built on soft soils. A lot of Numerical modelling has been done through studying the failed cases and using back analysis to replicate the scenario observed at the site. Technical Papers I & II cover the literatures on the numerical modelling of such embankments and to avoid the repetition of Literature, it has not been covered in this section.

2.9 References

1. Quigley, R. M. (1980) "Geology, mineralogy, and geochemistry of Canadian soft soils: a geotechnical perspective" Canadian Geotechnical Journal 17:261-285.
2. Ann G. Harris, Esther Tuttle and Sherwood D. Tuttle. 1997. Geology of National Parks: Fifth Edition, Iowa, Kendall/Hunt Publishing ISBN 0-7872-5353-7
3. Andrews, J. T. (1972) "The National Atlas of Canada. Canada Department of Energy, Mines and Resources"
4. Athir Nader (2014) "Engineering Characteristics of Sensitive Marine Clays - Examples of Clays in Eastern Canada" MASc thesis report, University of Ottawa
5. Quigley R.M., T. (1976) "Till geology, mineralogy and geotechnical behaviour, Sarnia, Ontario. Royal Society of Canada Special Publication" 12:336-345.
6. Weir, Wilbert Walter, 1882-. Philadelphia London, J. B. Lippincott company
7. Dreimanis, A. (1976) "Tills: Their origin and properties. Royal society of Canada Special Publication" 12:11-49.
8. Morgenstem, N. (1967) "Submarine slumping and the initiation of turbidity current" University of Illinois Press 189-220.
9. Leroueil, S. (1999) "Geotechnical characteristics of eastern Canada clays. Characteristics of Soft Marine Clays"
10. Locat, E. D. (1996) "On the development of microstructure in Genesis and Properties of Collapsible soils "93-128.
11. Torrance. (1988) "Mineralogy, pore-water chemistry, and. In The late Quaternary Development" 35:259-275.
12. Berry, R. W. (1988) "Mineralogy, grain size distribution and geotechnical behavior. The Canadian Mineralogist" 36:1625-1636.
13. Penner, E. (1965) "A study of the sensitivity of Leda clay. Canadian Journal of Earth Sciences 2.

14. Basack, S and Purkayastha, R. (2009) “Engineering Properties of Marine Clay from the Eastern Coast of India”, Journal of Engineering and Technology Research, 109-114.
15. N. R. Ahmad , Indra S. and H. Harahap (2016) “The Compression Behaviour of Marine Clays in Malaysia” International Conference on Ocean, Offshore and Arctic Engineering, Volume 1, ISBN: 978-0-7918-4992-7.
16. Oh, E. Y. N. and Chai G. W. K. (2006) ”Characterization of marine clay for road embankment design in coastal area” Proceedings of the sixteenth International Offshore and Polar Engineering Conference. 7–10.
17. Mohammed Ali, Mohammed Al-Bared and Aminaton Marto (2017) “A review on the geotechnical and engineering characteristics of marine clay and the modern methods of improvements” Malaysian Journal of Fundamental and Applied Sciences Vol. 13, No. 4 (2017) 825-831.
18. Fairfax County, Va., publication (2013) “Homeowner's guide to overcoming problems with marine clay-marumsco soils in Fairfax county”.
19. Bozkurt M. G. and Fratta, D. (2014) “Deformation Response of Soft Foundation Soils under Tall Embankments – A Numerical Analysis” GeoInstitute Congress. Atlanta, GA. February 2014.
20. Ghada S. Ellithy and Timothy D. Stark (2020) “Case Study: Unsaturated Embankment Failure on Soft Soils” J. Geotech. Geoenviron. Eng., 2020, 146(12): 05020011.
21. Roy, Debasis and Singh, Raghvendra (2008) "Failure of Two High Embankments at Soft Soil Sites" International Conference on Case Histories in Geotechnical Engineering. 2.
22. Hadjigeorgiou J, Kyriakou E and Papanastasiou P. (2006) “A Road Embankment Failure Near Pentalia in Southwest Cyprus” In: Int. Symp. Stab. Rock Slopes Open Pit Min. Civ. Eng.The South African Institute of Mining and Metallurgy, Cape Town, pp 343-352.
23. Shaofu Gu, Weizheng Liu and Mengyuan Ge (2020) “Failure and Remediation of an Embankment on Rigid Column-Improved Soft Soil: Case Study” Hindawi, Advances in Civil Engineering, Volume 2020, Article ID 2637651.
24. Sathananthan, Iyathurai, (2005) “ Modelling of Vertical drains with smear installed in soft clay” Doctor of Philosphy thesis, School of Civil, Mining and Environmental engineering, University of Woolongong, 2005.
25. https://www.geoengineer.org/education/web-class-projects/cee-542-soil-site-improve-winter-2014/assignments/prefabricated-vertical-drains
26. D. T. Bergado, M. C. Alfaro and A. S. Balasubramaniam (1992) “Improvement of Soft Bangkok Clay Using Vertical Drains” Geotextiles and Geomembranes 12 (1993) 615-663.
27. Goldberg A. D, M.A. Palmatier (2013) "Recent Advances in Prefabricated Vertical Drains." Ports 2013@ Success through Diversification. ASCE.
28. Ripon Hore, Sudipta Chakraborty and Mehedi A. Ansary (2016) “A Field Investigation to Improve Soft Soils Using Prefabricated Vertical Drains” Transportation Infrastructure Geotechnology (2020) 7:127–155.

29. Timothy D. Stark, Perry J. Ricciardi and Ryan D. Sisk (2017) "Case Study: Vertical Drain and Stability Analyses for a Compacted Embankment on Soft Soils" Journal of Geotechnical and Geoenvironmental Engineering, ASCE, ISSN 1090-0241.
30. Radhakrishnan R and Suriyanarayanan N.S, (2010) „„Embankment Construction over Reclaimed Land Using Pre-Fabricated Vertical Drains" IGC (2010), Mumbai.

CHAPTER 3 TECHNICAL PAPER I

Effects of Construction Sequence and Pace on the Stability of High Embankments Founded on Soft Marine Clay

Rakshith Badarinath [1] and Hany El Naggar [2]

[1] Graduate Student, Department of Civil and Resource Engineering, Dalhousie University, Nova Scotia, Canada B3H 4R2. Email: Rk822002@dal.ca

[2] Professor, Department of Civil and Resource Engineering, Dalhousie University, Nova Scotia, Canada B3H 4R2. Email: hany.elnaggar@dal.ca

Corresponding Author: Hany El Naggar, Ph.D., P.Eng.
Professor, Dalhousie University
Tel.: 1(902) 494 3904, email: hany.elnaggar@dal.ca

Abstract: The construction of high embankments in regions where the soils are predominantly soft and consist of marine clay deposits with low shear strength poses special challenges. Due to the high water content of marine clay deposits, an increase in applied stresses results in excess pore water pressures and a consequent reduction in shear strength. The sequence, pace, and duration of embankment construction play an important role in determining the stability of high embankments built on these soft soils. In this paper, the effects of these parameters on embankment performance are examined by utilizing a case study of an embankment failure which occurred in 2006 during the construction of a four-lane highway leading to the Canada-USA border crossing in St Stephen, New Brunswick, Canada. The St. Stephen highway embankment, with a design height of 14 m, failed at a height of approximately 12.3 m. The problem under consideration is modelled by using finite element analyses with the PLAXIS 2D software. Modelling is carried out with a staged construction technique, where the pace of embankment construction depends on the thickness of the fill layer added at each construction stage, and the time allowed for consolidation following the addition of each layer. Fill layer thicknesses ranging from 0.6 m to 7 m are used, in combination with consolidation times ranging from 1 to 64 days. The results obtained show that the stability of embankments founded on soft soil deposits increases as the pace of construction decreases and as the time allowed for consolidation increases.

Keywords: Consolidation, settlement, excess pore water pressure, numerical modelling, soil models, back analysis, marine clay.

3.1 Introduction

The construction of high embankments on fine-grained, soft soil deposits has always presented a challenge for geotechnical engineers, especially when the soil profile contains predominantly soft soils which consist of marine clays with low shear strength [1]. This is mainly due to the high water content and low permeability of marine clays. Thus, increased stresses in the foundation soils, induced by the placement of embankment fill causes an excessive increase in the pore water pressure of the saturated soils. Because these soils have a significant content of fines and low permeability, dissipation of the excess pore water pressures induced by fill placement is slow. As a result, fill placement during construction can lead to cumulative increases in the pore water pressure and a consequent reduction in shear strength. Under such stress conditions, saturated soils undergo constant-volume deformations, compressing vertically under the stress induced by the overlying embankment and expanding laterally toward zones of lower confining pressures close to and beneath the toe of the embankment. Beyond the toe, vertical confining stresses at shallow depths are even lower than the available horizontal confining stresses; hence, ground heave can occur. These changes in internal stresses cause increased shear stresses and strains within the soft soils, which can lead to failure. When shear stresses within the soils increase to the point of overcoming the soil shear resistance, a failure condition is reached, and no additional stresses can be resisted. Under such conditions, the shear strains can become substantial unless the propagation of displacement is resisted by adjacent soil elements that have not yet failed. When this scenario occurs in several soil elements along a continuous path, a slip failure surface develops, causing the embankment to collapse.

In coastal regions there are varying soil deposits, which consist of overconsolidated as well as soft normally consolidated clay deposits. The behaviour of soft marine clays has been extensively studied and reported in the literature by various researchers, since the early work on soft Bangkok clay carried out by Muktabhant et al. [2], Moh et al. [3], Eide [4 and 5], Bergado et al. [6] and Lei Ma et al. [7].

With the constantly expanding development of infrastructure, structures such as roads and river embankments are increasingly constructed over these soft clay soils. Highways in different parts of the world constructed over soft marine clay subsoils include the Thon Buri Pak Tho highway and the Bangna-Bangpakong highway in Thailand; the no. 54 state highway from Panvel to Uran in Mumbai, India; and the Wenzhou highway in China. Since soft soils have a very low shear strength, they cannot support the quick construction of embankments. In addition to staged construction, other techniques frequently used in the construction of embankments over soft soils include surcharge loading, and the use of stone columns, sand compaction piles, and geo-synthetic reinforcement to accelerate the rate of consolidation and the accompanying gain in shear strength of clay soils [13]. When these techniques cannot be employed due to budget, scheduling, or other construction constraints, staged construction may be the only practical construction alternative. In such cases, to facilitate the construction of high embankments, staged construction must be carried out with a suitable pace of construction, so that enough time is allowed for the dissipation of excess pore water pressures. This paper focuses on staged construction and does not discuss other remediation methods.

Because the construction of high embankments over soft soil deposits via a single-stage technique would require very gentle slopes, this would necessitate extensive earthworks, which would not be economically feasible. In the construction of embankments, settlement occurs during and after filling. Therefore, it is necessary to study the settlement behaviour of the subsoil. When staged construction is implemented, soft soils undergo consolidation, during which the soils gain strength as the settlement progresses. This is an essential aspect of working with soft soils such as marine clays, which have a very high natural water content and consequently low shear strength [8]. The extremely soft sensitive soil deposits formed after the Pleistocene period cover various areas of Europe, North America, and Asia, where construction over soft soils is now taking place due to rapid modernization. Other key features of these soft soils include changes in the isotropy of the soils in the course of their deposition, sedimentation and consolidation history, loss of cohesion due to changes in the soil particle orientation, and time-dependent stress-strain behaviour, which has a significant influence on the soil shear strength and pre-consolidation pressure [9].

Embankments with steep slopes can be constructed over soft soil deposits via a staged construction technique. This technique takes advantage of the increased soil strength resulting from consolidation and the accompanying progression of settlement.

To model the subsoil profile realistically, the nonlinearity of soft soils must be considered. Various soil models have been developed to achieve this, including the Cam-clay model, the modified Cam-clay model, the hardening soil model, the soft soil model, etc. [10]. As described above, the design of high embankments on normally consolidated soft soils can be challenging. Although stability is the most important factor to be considered in the design of embankments over these soils, other factors such as the magnitude of the applied stress and the direction of the settlement generated need to be considered as well, since these aspects may also affect stability. Excessive deformation and lateral displacement beneath embankments can result in the initiation and formation of cracks and fractures or eventual collapse. To increase overall stability, staged construction should therefore be carried out with a suitable pace and duration of construction [11].

In recent years there have been reports of several failures of high embankments, which resulted from insufficient subsoil strength. This points the need to recognize the importance of developing increased strength in soils such as marine clay through the process of consolidation [14].

This paper therefore aims to study the effects of staged construction and the sequence and pace of construction on the stability of embankments built on soft soils. This investigation considers a case study of the St. Stephen embankment which failed in 2006, during the construction of a four-lane highway leading to the Canada-USA border crossing in St Stephen, New Brunswick, Canada. The highway embankment failed at a height of approximately 12.3 m, only a little below the design height of 14 m [16]. In the present research, a two-dimensional finite element model of the embankment up to the time of failure was developed with the PLAXIS 2D software, and the predictions of the model were validated against the failure details available for the embankment. In addition, after the failure observed at the site was replicated in the numerical model, a parametric study was conducted to investigate the effect of the pace of construction on the overall performance of the embankment during and after construction. The factors considered were the thickness of the fill layer placed on the embankment at each stage of construction, and the consolidation time allowed between each construction stage.

3.2 The St. Stephen Case Study

The problem considered in this paper was modelled numerically by using the geometry and details of the case study of the St. Stephen, New Brunswick, embankment failure which occurred in July 2006. The embankment was located on route 1 in a section of divided four-lane highway leading to the Canada-USA border crossing (see Figure 3.2a). The embankment failed at a height of 12.3 m, only 1.7 m below the design height of 14 m. The failure was attributed to the quick rate of construction and the intensity of loading on the low strength foundation soil, which consisted of up to 15 m of marine clay. The failure was described as a deep-seated circular slip failure with some lateral spreading of the embankment toe. In the plan view, the failure encompassed an area measuring approximately 60 m in cross-section and 130 m in the longitudinal direction [16]. The construction was done in stages, with the layers being compacted in 0.6 m lifts. The embankment construction sequence and its implementation in the numerical model are explained in more detail in Section 3.3.4. The failure occurred during the construction phase, as reported in [16]. The marine clay soil was normally consolidated (NC) clay, with an over-consolidation ratio (OCR) of 1.

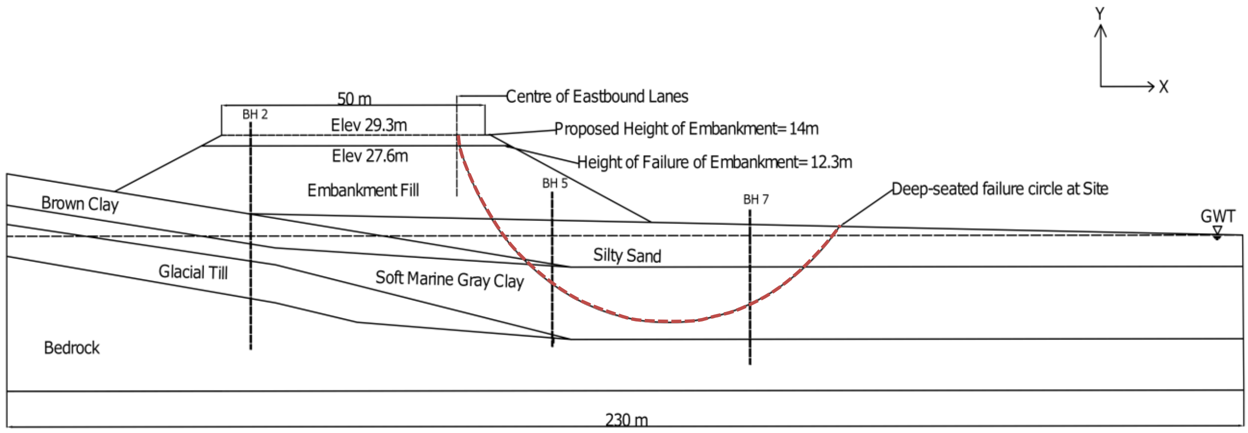


Figure 3.1: Site cross-section showing the soil profile, position of boreholes (BH), and location of the deep-seated circular slip failure (indicated in red)

3.2.1 Site conditions and Geology

The subsurface profile of the site and its geology were studied after the occurrence of the embankment failure. Standard penetration tests (SPT) were conducted after the failure, just before reconstruction of the embankment. Based on the SPT corrected values, a generalized soil profile, shown in Figure 3.1, was developed via back analysis. SPT tests were conducted at seven boreholes on the site. The locations of boreholes 2, 5, and 7 are indicated in Figure 3.1.

At borehole 2 (BH 2), a layer of fill 8 m thick was underlain by a very thin layer of topsoil (approximately 0.1 m), resting on a very soft gray silty lean clay (CL) layer. Under this was a layer of dense gray silty sand (SM) 0.7 m thick, underlain by a layer of till 4.6 m thick. Bedrock was encountered at about 19 m below ground level. At borehole 5 (BH 5), a layer of fill 3.5 m thick was underlain by 4.1 m of compact to dense brown silty sand with gravel

(SM), which rested on a very soft gray lean clay (CL) layer. Bedrock was encountered at approximately 18.5 m below ground level. At borehole 7 (BH 7), 3.7 m of very loose to compact brown clayey sandy silt (ML) with gravel was underlain by a thin layer (0.6 m) of soft sandy clay (CL), resting on a layer of very soft gray clay (CL) 10.6 m thick. This was underlain by another thin layer (0.7 m) of compacted silty clayey sand with gravel, with bedrock located at a depth of 15.6 m beneath the ground level. The water content, W_c, of the soft clay layer ranged from 30% to 35%; with a plastic limit, W_P, of 22% and a liquid limit, W_L, of 45%. The vertical coefficient of consolidation, C_v, was estimated to be 0.010 cm^2/min, and the horizontal coefficient of consolidation, C_h, was estimated to be 0.012 cm^2/min [17].

3.2.2 Soil Properties

Based on the SPT values, empirical relations were used to determine the strength parameters of the different types of soil at the site. Empirical relations [18] were used to calculate the undrained cohesion (S_u) of the soft gray clay layer. From the water content of 30% to 35% and the plasticity index (PI) of 22%, other parameters of the soft soil such as C_c and C_s, which determine the stiffness during loading and unloading, were obtained. The stiffness of the other soils was determined by using relations based on the average SPT value. The marine clay layer was modelled by using the soft soil (SS) model in PLAXIS 2D, and the other soils were modelled by using the hardening soil (HS) model. A detailed description of the soil models considered is provided in the following section. Table 3.1 summarizes the soil properties used in the numerical model, which were determined from the average SPT values, based on empirical relations. The hydraulic conductivities were determined by using the estimates of C_v and C_h obtained from the site.

All the soil properties presented in Table 3.1 are given in SI units. For example, the unit weight, γ, is measured in kN/m^3; the stiffness in kPa; the cohesion in kPa; and the hydraulic conductivity, *k*, in m/day. The three stiffness parameters used in the hardening soil model are explained in Section 3.3.3. The marine clay strength parameters for the given water content and plasticity index are estimated in accordance with Zhongkung and Paul [20] and Myint [21].

(a) (b)

Figure 3.2: (a) Section of route 1 divided highway leading to the Canada-USA border crossing at St. Stephen, New Brunswick; (b) Failure observed at the site (west view) (Bernie 2010)

Table 3.1: Summary of soil properties used for the numerical model

Soil Layers	Soil Model	Unit Weight (kN/m^3)		Stiffness Parameters (MPa)			Poisson's Ratio	Cohesion (kPa)	Friction Angle (°)	Dilation Angle (°)	Hydraulic Conductivity (m/day)
		γ_{dry}	γ_{sat}	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}					
Fill	HS	20	22	50	40	150	0.2	0	38	8	8.65×10^{-1}
Soft grey marine clay	SS	13.8	16.1	C_c= 0.35* C_s= 0.035*			0.495	0	28	0	8.65×10^{-5}
Brown clayey silty sand	HS	16	20	30	24	90	0.2	5	29	0	8.65×10^{-4}
Silty sand	HS	16.5	20	40	32	120	0.2	0	31	1	8.65×10^{-3}
Glacial till	HS	17	20	40	32	120	0.2	0	34	4	8.65×10^{-2}
Bedrock	Linear non-porous	28	-	E= 6200			0.15	-	-	-	-

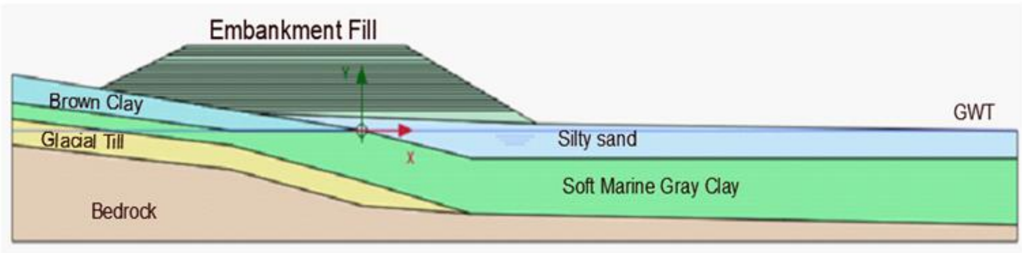
*unitless

3.3 Numerical Modelling

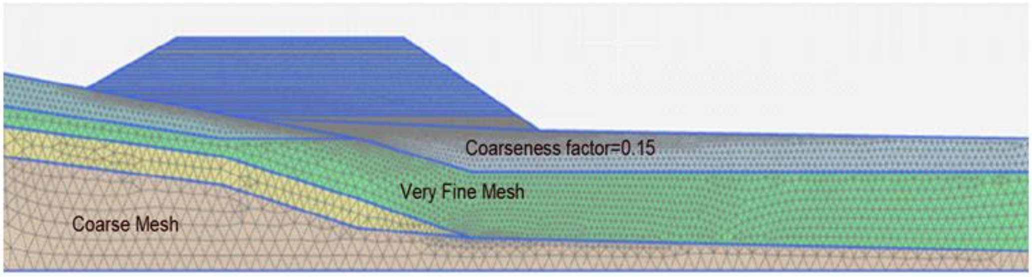
Finite element analyses were carried out by using PLAXIS 2D for the assumed plain strain problem. The model geometry was constructed from the cross-section of the soil profile shown in Figure 3.1. The soil profile consists of a brown clayey silty sand layer and a silty sand layer resting on a marine clay layer 15 m thick, underlain by glacial till and bedrock. The groundwater table is assumed to be at the ground level and is assumed to remain constant in the upward sloping soil profile. Since the soil profile is not symmetric, the entire profile was modelled and analyzed. The control case models the actual construction of the embankment at the site, with fill layers 0.6 m thick, and replicates the consolidation periods that occurred at the site. In the parametric study, staged construction is used, with fill layers ranging in thickness from 0.6 m to 7 m, and consolidation periods ranging from 1 day to 64 days are implemented between the construction stages. The following five models are used in the parametric study: 1) Normal construction, with fill layers 0.6 m thick, 2) fast construction, with fill layers 1.2 m thick, 3) very fast construction, with fill layers 1.8 m thick, 4) rapid construction, with fill layers 3.6 m thick, and 5) two-stage construction, with fill layers 7 m thick. A more detailed explanation of the parametric study is given in Section 3.3.4.2.

3.3.1 Geometry

Figure 3.3(a) shows the geometry adopted for the problem. To represent the deep-seated circular slip failure more accurately, the geometry is modelled assuming that the soil profile on the right-hand side of the embankment, as depicted in Figure 3.3(a), extends for an additional 70 m from the toe, to a total of 150 m from the origin. The side slope for the profile on the right-hand side is assumed to be 1V:2.14H and the side slope for the profile on the left-hand side is assumed to be 1V:1.43H, in accordance with the dimensions observed in the case study. The total width of the crest of the embankment is 50 m since the site is a section of a divided highway. In Figure 3.3(a), the marine clay layer represented in green is underlain by glacial till on the left-hand side and by bedrock on the right-hand side of the embankment. The soil profile in the numerical model is assumed to be isotropic. The marine clay is modelled in an undrained condition by the soft soil model, and the brown clayey sandy silt and the other soils (except for the bedrock) are modelled in a drained condition by the hardening soil model. In the mesh, 15-noded triangular elements are used, constituting 3 degrees of freedom for each node [8].



(a)



(b)

Figure 3.3: (a) Cross-section of the embankment modelled by using PLAXIS 2D; (b) Meshing used in the model geometry

3.3.2 Meshing and Boundary conditions

Figure 3.3(b) illustrates the meshing used to model the embankment. A very fine mesh with a coarseness factor of 0.15 is used at the boundaries of the embankment fill and the subsoil profile, to prevent errors in the calculations due to small element meshing.

It should also be noted that approximately 42,000 soil elements are used in the control and normal construction models, 40,000 are used in the fast construction model, 38,500 are used in the very fast construction model, 38,000 are used in the rapid construction model, and 33,000 are used in the two stage construction model. Since the geometry adopted for the problem does not involve symmetry and the soil profile is not horizontal, meshing is a key aspect which helps to ensure accurate results for the parameters assumed in the model.

In the models, it is assumed that the groundwater can drain freely in all directions except at the bottom, where it is fixed, since the bedrock is non-porous. For the deformation boundary conditions, a normally fixed position is assumed for the sides of the model, the bottom is assumed to be fully fixed, and the top is assumed to be free.

3.3.3 Soil models used in modelling

In this study, the soil models used for numerical modelling are the soft soil model (SSM) and the hardening soil model (HSM), described below.

3.3.3.1 Soft Soil Model

To model soft soils such as normally consolidated clay, peat, and clayey silts, the nonlinearity of these soils must be considered. The Mohr-Coulomb model assumes a constant stiffness and linear behaviour throughout the entire depth of a particular soil profile; however, these conditions generally do not hold true in the field. Different soil models have therefore been developed to account for the nonlinear behaviour of soils. The soft soil model accounts for soil non-linearity via a linear stress dependency of stiffness. The modified compression index (λ^*) and modified swelling index (k^*) are key model inputs which define the compressibility of the soil in isotropic loading, and in unloading and subsequent reloading, respectively. Alternatively, if C_c and C_s are defined, the software can use these parameters to calculate λ^* and k^*. The yield function of the soft soil model is based on the yield failure criterion of the Mohr-Coulomb theory, except that the yield function defines an ellipse, where M is the height of the ellipse, and the isotropic pre-consolidation pressure, P_p, defines the length of the ellipse in the p' plane. The height of the ellipse, M, also represents the ratio of horizontal to vertical stresses in primary loading, and is therefore used to determine K_o^{nc}, where K_o^{nc} represents the earth pressure at rest. The top of all the ellipses coincides with the M line, and the M line has a slope greater than the line defined by Mohr-Coulomb yield line. Thus, the M line is referred to as the critical line. The pre-consolidation pressure decreases with compaction. It is assumed that at the initial stage there is no plastic volumetric strain, which corresponds to P_{po}, where P_{po} is the pre-consolidation pressure during the initial stage.

The total strain has components of elastic and plastic strain, as shown in Eq (1):

$$\varepsilon_T = \varepsilon^e + \varepsilon^p \quad (1)$$ [15]

where ε^e is the elastic component, ε^p is the plastic component, and ε_T is the total strain.

3.3.3.2 Hardening Soil Model

The hardening soil model accounts for soil nonlinearity via a stress dependency of stiffness but does not account for viscous effects such as creep and stress relaxation. It employs the plasticity theory instead of the theory of elasticity used in the Mohr-Coulomb model. The hardening soil model accounts for both shear hardening and compression hardening in the soil. Unlike the soft soil model, which can be used to model only soft soils, the hardening soil model can be used to model different types of soil, including granular soils and soft soils. The stress dependency of soil stiffness is defined by the power m, where m = 1 is typical for soft soils. The model requires the input of three stiffness parameters at a chosen reference pressure (P_{ref}): E_{50}^{ref} (secant modulus/triaxial modulus), E_{oed}^{ref} (oedometric modulus) and E_{ur}^{ref} (unloading-reloading modulus). The yield surface consists of a parabolic curve, and soil dilatancy and a yield cap are considered. The position of the shear hardening yield surface is determined mainly by the triaxial modulus, while the position of the yield cap (compression hardening) is primarily determined by the oedometric modulus [15].

For soft soils (where m = 1), the stiffness moduli can alternatively be entered in terms of the compression (C_c) and swelling (C_s) indices, which are related to the stiffness moduli as shown in Eq (2, 3 and 4), where γ is Poisson‟s ratio; e_o is the initial void ratio; and P_{ref} is the reference pressure [15].

$$E_{oed}{}^{ref} = 2.3\,(1 + e_o)\frac{P_{ref}}{C_c} \quad (2)$$

$$E_{ur}{}^{ref} = 2.3\,(1 + e_o)\frac{(1+\gamma)(1-2\gamma)P_{ref}}{(1-\gamma)C_c} \quad (3)$$

$$E_{50}{}^{ref} = 1.25\,E_{oed}{}^{ref} \quad (4)$$

where $E_{50}{}^{ref}$ is the stiffness modulus for primary loading in the drained triaxial shear test, $E_{ur}{}^{ref}$ is the stiffness modulus for unloading and reloading in drained triaxial shear test, and $E_{oed}{}^{ref}$ is the stiffness modulus for primary loading in the oedometer test. In the present study, it is assumed that $E_{oed}{}^{ref} = 0.8\,E_{50}{}^{ref}$.

For a more detailed explanation of the soil models, please refer to the PLAXIS material models manual [19].

3.3.4 Development of the finite element model and objectives of the parametric study

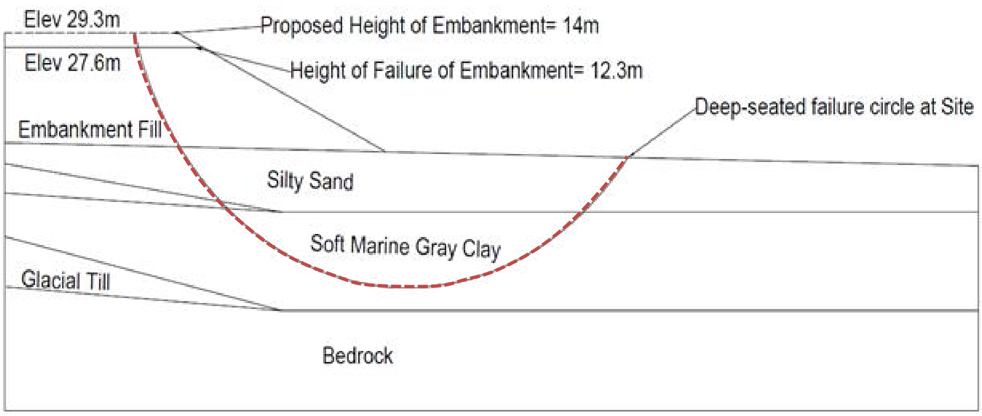
3.3.4.1 Development of the finite element model

The construction sequence of the embankment built at St. Stephens is modelled as a staged construction procedure, which is analyzed in PLAXIS. Below are details of the construction, which was begun in 2005 and proceeded until failure of the embankment in July 2006:

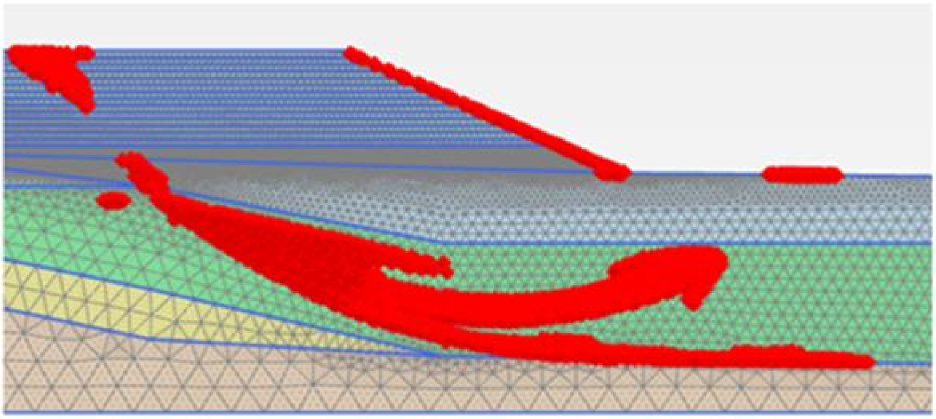
- In 2005, the embankment was constructed to a height of 5 m in 90 days. In the model, the first layer of fill is assumed to have a thickness of 2.4 m, with a construction time of 4 days, followed by a consolidation period of 15 days. Four subsequent layers of fill are assumed, each with a thickness of 0.6 m, a construction time of one day and a consolidation period of 15 days. This amounts to a total fill height of 5 m and a total time of approximately 90 days.

- After reaching a height of 5 m, the embankment was allowed to consolidate for 270 days, when construction stopped for the winter. Therefore, an additional consolidation phase, for a period of 270 days, was defined in the model following the previous consolidation phase.
- Construction of the remaining portion of the embankment resumed on June 11, 2006, and continued until July 4, 2006, when the embankment failed at a height of 12.3 m. In the model, this is estimated as roughly one month, and is modelled as 15 one-day construction stages, each followed by a one-day consolidation period, for a total of 30 days.
- For the initial phase, a K_o procedure was used to calculate the initial in-situ stresses, with pore water pressures being calculated for the phreatic level. All construction phases were defined as plastic phases, and consolidation phases were defined as consolidation.
- The soil parameters of the marine clay layer were varied until a deep-seated circular slip failure was replicated at the required embankment height of 12.3 m, following the construction sequence described above. The soil properties are presented in Table 1.

Figure 3.4 shows that the finite element model developed here succeeded in replicating deep-seated failure, mimicking the failure surface that occurred in the case study, as reported by Mills et al. [17]. The extent of lateral spreading of the embankment at failure was also similar to that observed in the field and reported by Mills and McGinn [16] and Mills et al. [17].



(a)



(b)

Figure 3.4: (a) Deep-seated circular slip failure at the case study site (shown in red); (b) Deep-seated circular slip failure developed in the finite element model at the embankment height at which failure occurred in the case study

3.3.4.2 Objectives of Parametric study

Following replication of the embankment failure by the control model (see Figure 3.4(b)); a parametric study was conducted by varying the fill layer thickness and consolidation period implemented at each construction stage. The aim was to investigate the reason for failure at the case study site, and to determine whether the failure could have been avoided by following a different construction schedule. It was suspected that one of the main reasons for failure was quick construction on a soft foundation soil. By varying the consolidation periods implemented for each phase, the parametric study described here demonstrates the importance of allowing time for consolidation when building on soft marine clays. The thickness of the fill layer added at each stage was also varied, to study the effect of using thicker fills on soft soils with the same range of consolidation periods. The best combination is selected from this study, which suggests a suitable construction sequence to follow, without any ground modifications or improvements, when high embankments are constructed on soft soil deposits such as marine clays. In some instances, this could help to realize substantial savings, if the project specifics allow for more extended construction periods.

The models for the parametric study are designed as follows:

- In the normal construction model, the thickness of each fill layer is 0.6 m and the construction time for each layer is in one day. Consolidation periods of 1, 2, 4, 8, 16, 32, and 64 days between the construction stages are modelled, to investigate the effect of the consolidation period on the embankment performance.
- In the fast construction model, the thickness of each fill layer is 1.2 m, and the consolidation periods are varied as in the normal construction model.
- In the very fast construction model, the thickness of each fill layer is 1.8 m, and the consolidation periods are varied as in the normal construction model.
- In the rapid construction model, the thickness of each fill layer is 3.6 m, and the consolidation periods are varied as in the normal construction model.
- In the two-stage construction model, the thickness of each fill layer is 7 m, and the consolidation periods are varied as in the normal construction model.

3.4 Results and Discussions

This paper focuses primarily on the embankment design parameters of factor of safety (FS), settlement, percentage increase in excess pore water pressure, and extent of horizontal and vertical lateral displacement. Since a crucial aspect of embankment design is stability, to prevent failure and excessive deformation, results for these parameters obtained from the models are used to study the effects of the sequence and pace of construction on the stability of high embankments built on soft soils. Because it was necessary to calibrate the numerical models to replicate the failure observed in the field, the control model that mimics the construction sequence used in the St. Stephens case study is considered first.

3.4.1 Results of the control model

In Figure 3.5, it can be seen that with a consolidation period of one day following each construction stage, the factor of safety (FS) decreases as the embankment height increases. Figure 3.5 presents the results for embankment heights of 7.2 m and 9.6 m, and for the embankment height just before failure. In the control model, failure occurs when the height of the embankment is 12.3 m above the ground level, as was observed in the field in the case study. The FS at an embankment height of 7.2 m is 1.681, and the FS just before failure is 1.034. Thus, with a consolidation period of one day following each construction stage, the stability of the embankment decreases as the height of the embankment increases, and it is not possible to complete the construction of the embankment to the full design height.

Figure 3.5 also shows the settlement of the embankment as the construction progresses. At an embankment height of 7.2 m the settlement is 267.3 mm, and just before failure the settlement is 427.1 mm. The analysis of settlement is one of the most important criteria to consider when designing embankments. The greater the settlement, the more strength is gained by the soft soil during the consolidation process. Thus, settlement helps to increase the overall stability of the construction. In the control case, there was an extended consolidation period of nine months following the construction of the first 5 m of the embankment. This resulted in sufficient consolidation and increased soil strength. However, in the construction phase which began in the spring of 2006, the construction process accelerated, with a consolidation period of only one day following the placement of each fill layer. This resulted in less settlement and a reduced gain in soil strength, eventually leading to failure of the embankment at a height of 12.3 m, due to shear strength failure. Thus, it is very important to consider the time allowed for consolidation in the case of embankments constructed on soft soil deposits such as marine clay.

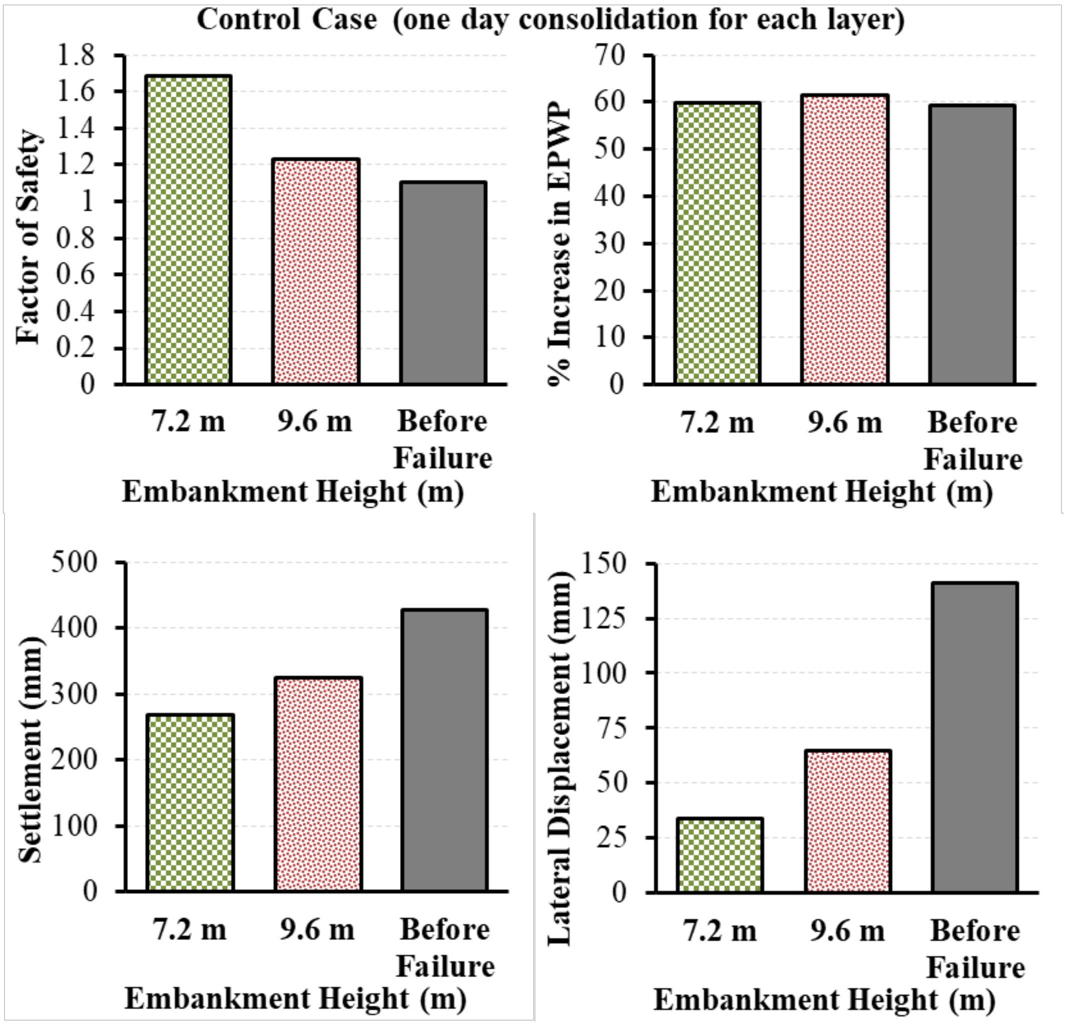


Figure 3.5: Summary of results obtained from the control case model

Figure 3.5 also shows that, as expected, there is a decrease in the percentage of excess pore water pressure during consolidation. This parameter represents the increase in excess pore water pressure in comparison to the hydrostatic pore water pressure at a reference point, due to the construction of the embankment. The reference point is considered to be around the mid-point of the marine clay layer directly beneath the embankment at 6.675 m, where there was maximum pore water pressure. The hydrostatic pressure at this point was 65.48 kPa, and the increase in excess pore water pressure at the embankment height under consideration is expressed as a percentage. In the control case model, mid-way through the embankment construction, the percentage increase in excess pore water pressure is 62.6%, and just before failure it is 59.3%. Since the consolidation period between construction stages is only one day, there is little dissipation of the excess pore water pressure. Hence, there is minimal variation in the excess pore water pressure during the consolidation period. Since there is an accumulation of excess pore water pressure, stability is reduced significantly, as the effective shear strength decreases. Hence, the time allowed for consolidation must be increased to prevent the development of excess pore water pressure.

The lower right-hand graph in Figure 3.5 shows the extent of horizontal lateral displacement. It can be seen that lateral displacement increases as the height of the embankment increases. Lateral displacement is another key parameter to be studied to check the stability of an embankment founded on soft highly compressible soils. Lateral spread is defined as the lateral displacement of the embankment toe under the application of stress or self-weight of the structure. Since failure at the study site was also attributed to displacement of the toe of the embankment, this parameter represents another important consideration in the design of embankments. During the consolidation periods and construction stages, a certain amount of settlement can be expected, depending on the applied stresses and the type soil underneath. Thus, there is always the possibility that lateral displacement of the embankment toe can occur, due to the compressibility of the soft soil beneath. The extent to which lateral displacement can be used directly to estimate embankment stability has not been extensively studied in the literature. This paper therefore attempts to examine the effect of lateral displacement on the stability of the embankment.

Based on the successful replication of the case study scenario in the control model, a parametric study was conducted by varying the consolidation period and the fill layer thickness to determine the effects of the construction sequence and pace on the stability of embankments founded on soft marine clay. The following graphs show the results of the parametric study.

3.4.2 Results of the parametric study

In the normal construction model, where the fill layer thickness is 0.6 m, when consolidation periods of 32 days and 64 days are implemented after the placement of each fill layer, the embankment does not fail and reaches the design height of 14 m. However, in the normal construction model with consolidation periods of 1 to 16 days, the embankment fails at the height of 10.8 m. In the fast construction model, where the fill layer thickness is 1.2 m, when a consolidation period of 64 days is implemented after the placement of each fill layer, the embankment does not fail. However, in the fast construction model the embankment fails at a height of 12 m with a consolidation period of 32 days, and it fails at a height of 10.8 m with consolidation periods of 1 to 16 days. In the very fast construction model, where the fill layer thickness is 1.8 m, the embankment fails at a height of 12.6 m with a consolidation period of 64 days, and it fails at a height of 10.8 m with shorter consolidation periods. In the rapid construction model, where the thickness of each fill layer is 3.6 m, the embankment fails at a height of 10.8 m with all the consolidation periods considered, from 1 day to 64 days. Similarly, in the two-stage construction model, where the fill layer thickness is 7 m, the embankment fails at a height of 10.8 m with all the consolidation periods considered, from 1 day to 64 days.

Figures 3.6 to 3.10 present the results of the parametric study. In the graphs, the design parameters on the *y*-axis are plotted against the consolidation period in days on the *X*-axis. As explained in the following sections, the design parameters considered are the factor of safety, settlement, percentage increase in excess pore water pressure, and horizontal and vertical lateral displacement.

3.4.3 Factor of safety

Figure 3.6 shows that as the time allowed for consolidation increases from 1 day to 64 days, the factor of safety increases, resulting in increased stability. In contrast, as the fill layer thickness increases from 0.6 m to 7 m, the factor of safety decreases. This is because thicker fill layers apply greater stress to underlying soils within a shorter period of time, which results in less stability and hence a lower factor of safety. The lowest factor of safety occurs with a fill layer thickness of 7 m and a consolidation period of 1 day, and the highest factor of safety occurs with a fill layer thickness of 0.6 m and a consolidation period of 64 days. The graphs for fill layer thicknesses of 3.6 m (rapid) and 7 m (two stage construction) each show only the curve corresponding to an embankment height of 7.2 m. This is because in these models, embankment failure occurs at 10.8 m, before the addition of subsequent fill layers can be completed.

3.4.4 Settlement

Figure 3.7 shows that as the time allowed for consolidation increases, the settlement increases, causing the clay soil to gain in strength. With increased fill thickness, the settlement increases because of the compressibility of the soft soil; however, this does not necessarily result in a gain in soil strength, because it means that greater stress is applied to the soil in a shorter period of time. Due to the high water content of the marine clay, excess pore water pressures develop to a much greater extent with thicker layers of fill. In Figure 3.7, it can be seen that the settlement is much less in cases where the embankment fails. This is because the lack of sufficient settlement significantly reduces the shear strength of the soil. Here it can be seen that the least settlement occurs with a fill layer thickness of 7 m and a consolidation period of 1 day; and the most settlement occurs in a situation where the embankment does not fail, with a fill layer thickness of 0.6 m and a consolidation period of 64 days.

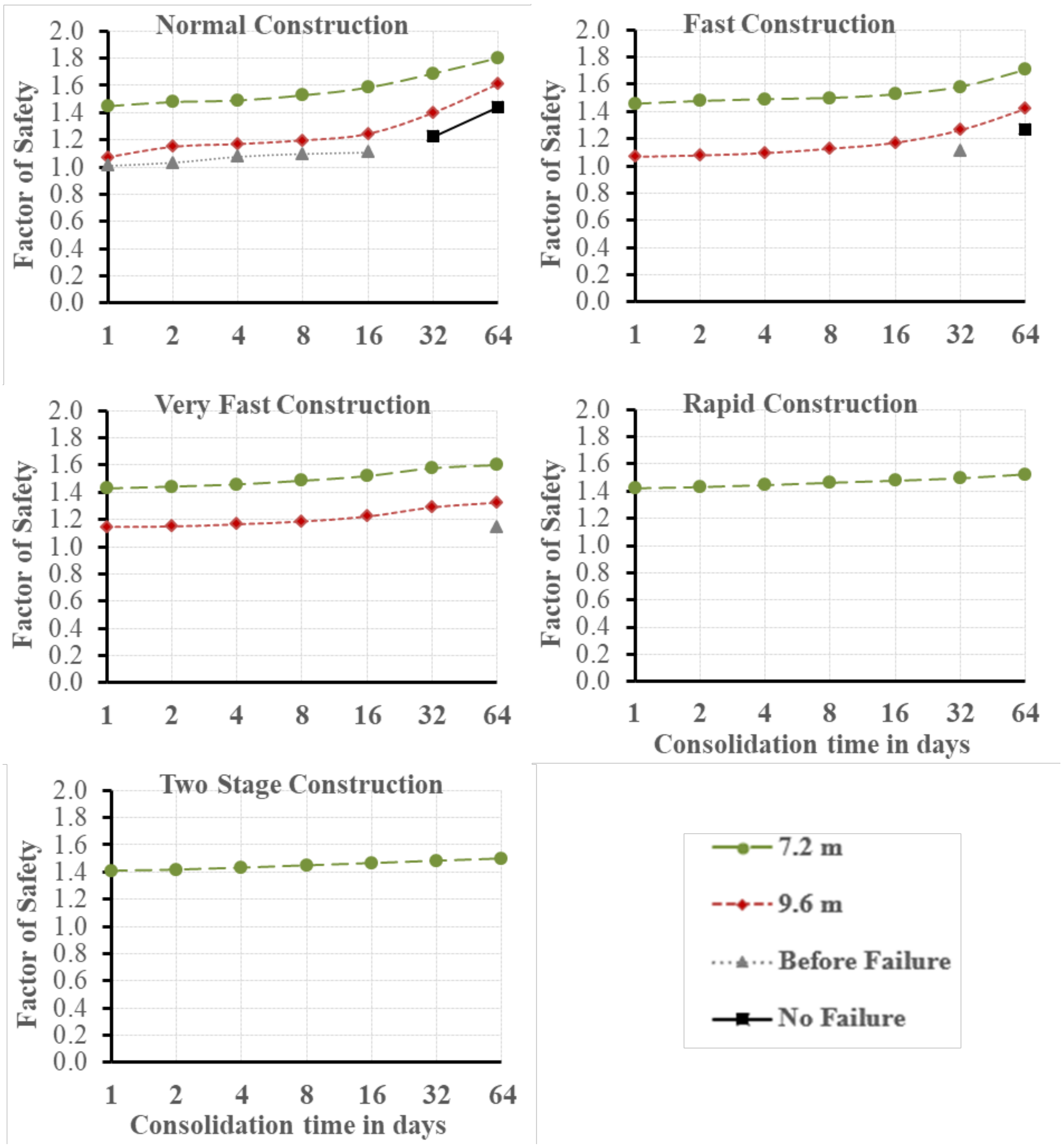


Figure 3.6: Factor of safety plotted against consolidation time, showing parametric study results for fill layer thicknesses of 0.6 m (normal), 1.2 m (fast), 1.8 m (very fast), 3.6 m (rapid), and 7 m (two stage construction)

3.4.5 Percentage increase in excess pore water pressure

Figure 3.8 shows that the percentage increase in excess pore water pressure declines as the consolidation time increases, and becomes greater as the as the fill layer thickness increases. This means that thicker fill layers result in greater excess pore water pressure; however, if more time is allowed time for consolidation, the excess pore water pressure dissipates, causing the percentage increase in excess pore water pressure to decline. If a project is subject to time constraints, ground improvement measures such as wick drains can be implemented, to improve drainage and thus accelerate the dissipation of excess pore water pressures. In this study, the greatest percentage increase in excess pore water pressure occurs with a fill layer thickness of 7 m and a consolidation period of 1 day, and the lowest percentage increase in excess pore water pressure occurs with a fill layer thickness of 0.6 m and a consolidation period of 64 days.

3.4.6 Lateral displacement

Figures 3.9 shows the horizontal lateral displacement of the embankment toe and Figure 3.10 shows its vertical displacement, plotted against the time allowed for consolidation following each construction stage. It can be seen that the lateral displacement decreases as the time allowed for consolidation increases, and increases as the as the fill layer thickness increases. As the fill layer thickness increases, more stress is applied in a shorter period of time, and hence there is greater horizontal lateral displacement. Similarly, as the fill layer thickness increases, vertical displacement also increases. For example, fill layers 3.6 m thick result in greater vertical displacement than is the case with fill layers 0.6 m thick. This is because when thinner fill layers are used, the application of stress when fill layers are added is spread out over a longer period of time, since more layers are required to reach the design embankment height of 14 m.

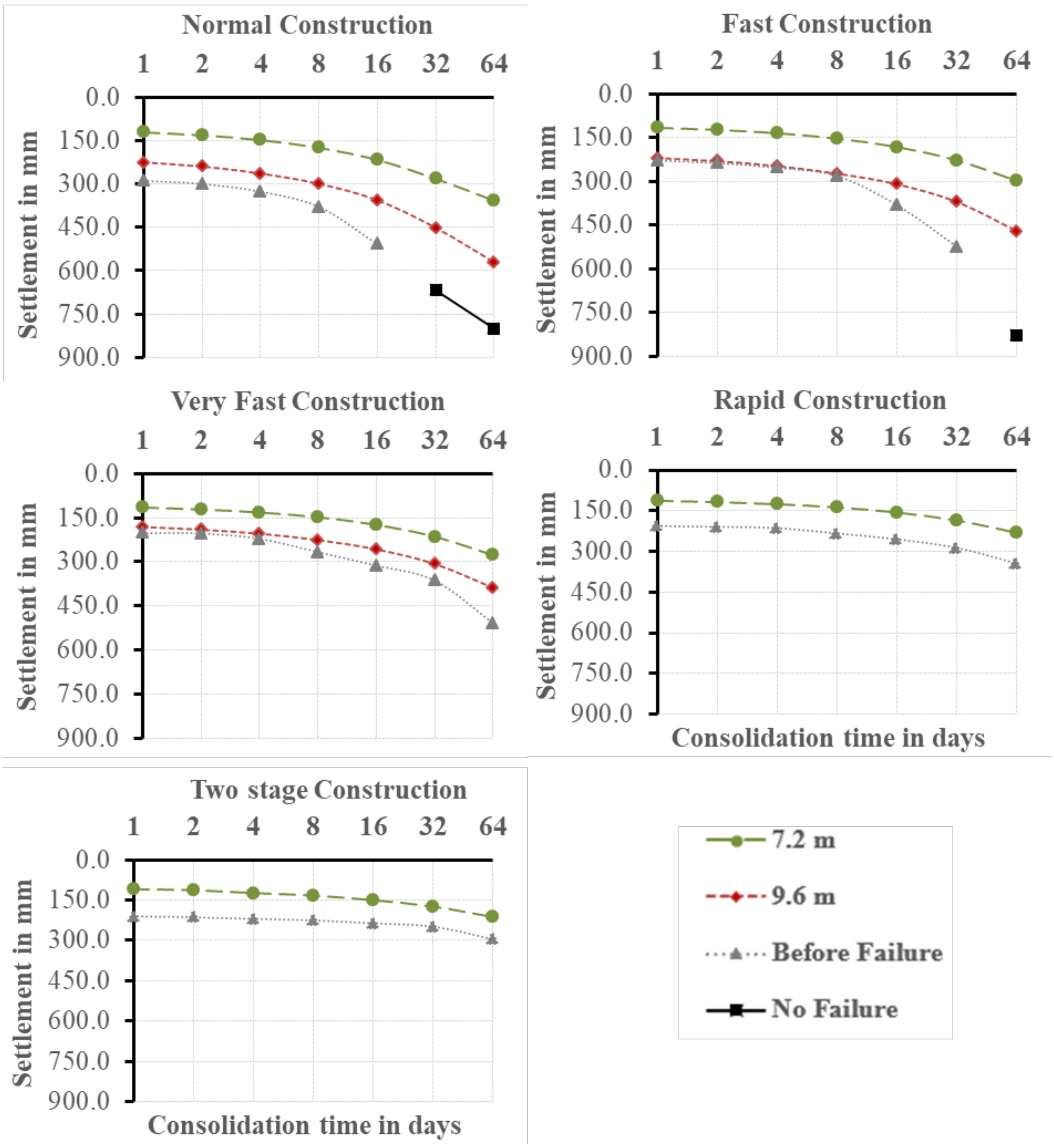


Figure 3.7: Settlement plotted against consolidation time, showing parametric study results for fill layer thicknesses of 0.6 m (normal), 1.2 m (fast), 1.8 m (very fast), 3.6 m (rapid), and 7 m (two stage construction)

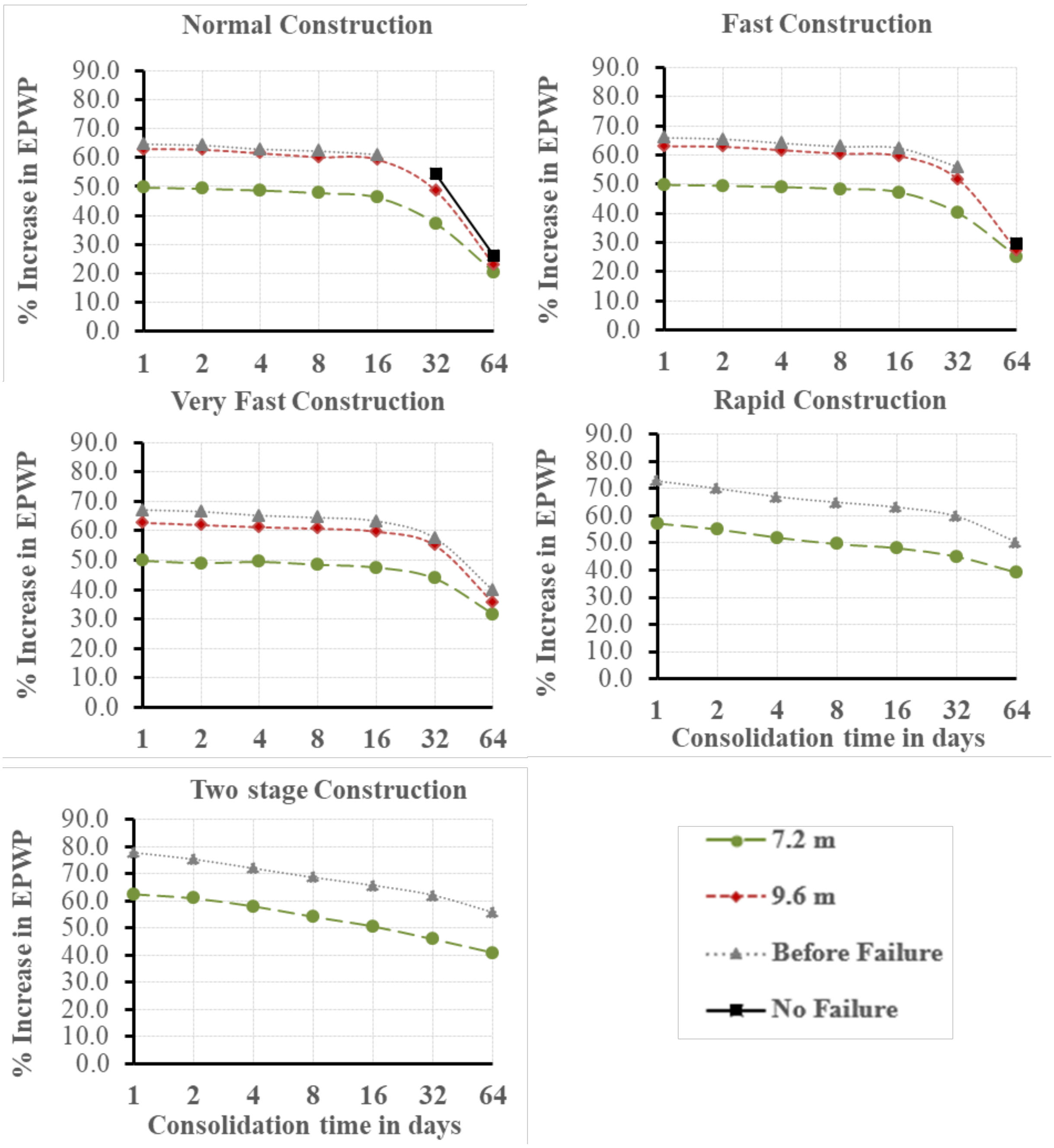


Figure 3.8: Percentage increase in excess pore water pressure plotted against consolidation time, showing parametric study results for fill layer thicknesses of 0.6 m (normal), 1.2 m (fast), 1.8 m (very fast), 3.6 m (rapid), and 7 m (two stage construction)

Thus, with thinner fill layers, the soil is subjected to less additional compression during a given period of time. Increasing the time allowed for consolidation permits excess pore water pressures to dissipate, so that the soil gains strength, and horizontal and vertical lateral displacements are reduced.

From the parametric study results presented above, it may be concluded that if the construction schedule in the St. Stephens case study had allowed a longer consolidation period of 64 days following the placement of every fill layer, the embankment failure could have been avoided. On the other hand, in the case of a tight construction schedule with imposed time constraints, ground improvement measures should have been implemented to accelerate drainage and thus improve the strength of the foundation soil.

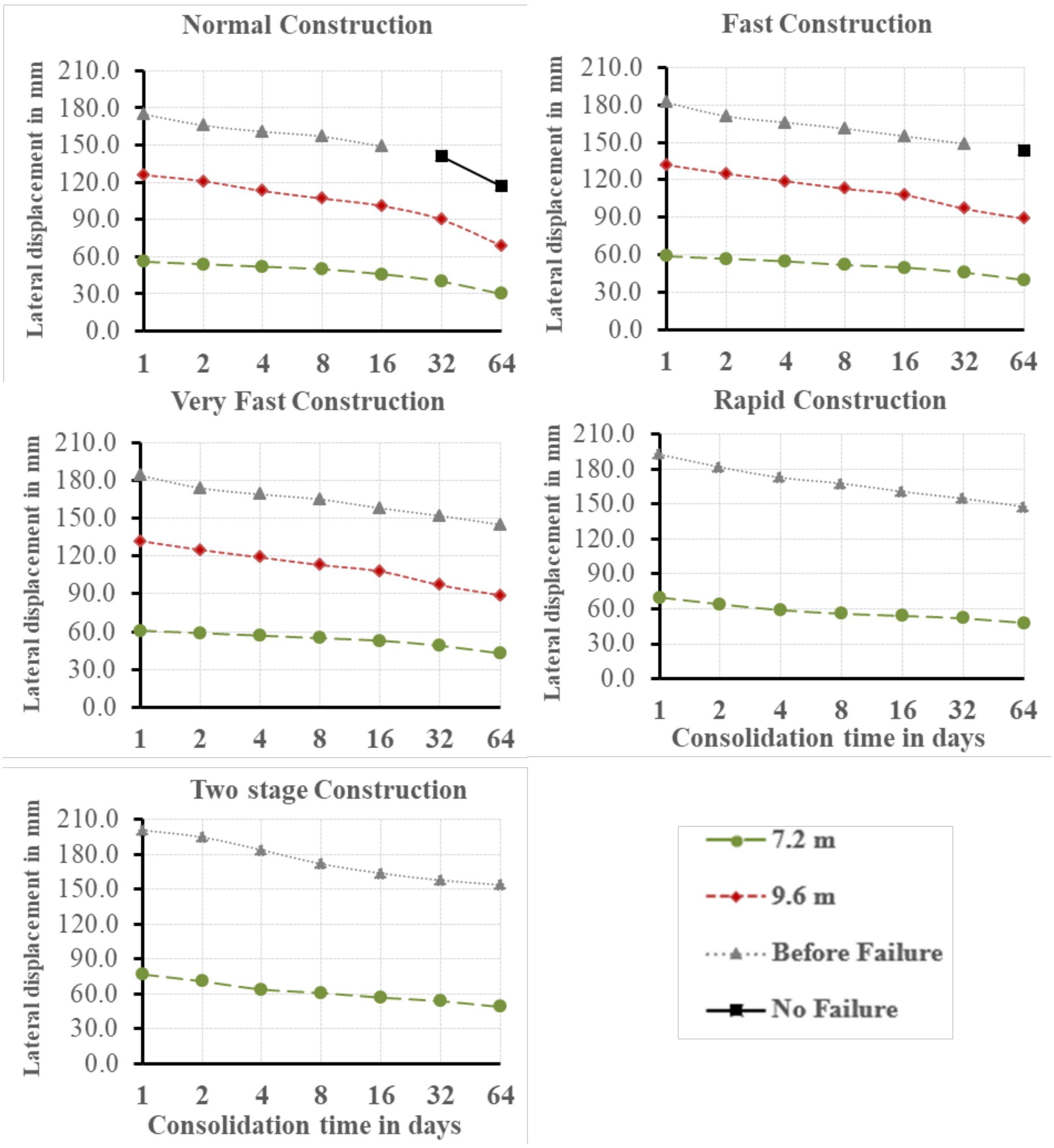


Figure 3.9: Horizontal lateral displacement plotted against consolidation time, showing parametric study results for fill layer thicknesses of 0.6 m (normal), 1.2 m (fast), 1.8 m (very fast), 3.6 m (rapid), and 7 m (two stage construction)

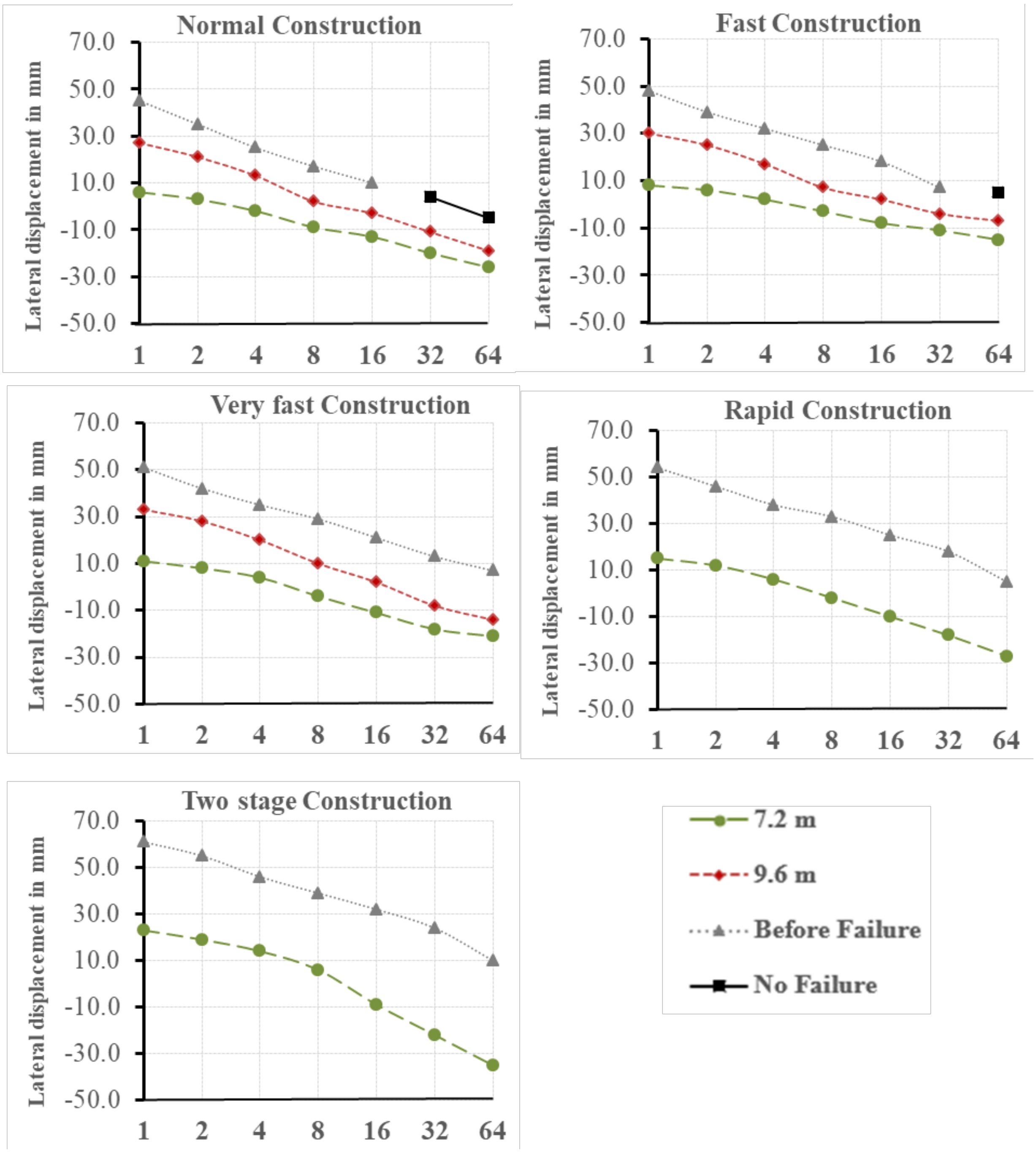


Figure 3.10: Vertical lateral displacement plotted against consolidation time, showing parametric study results for fill layer thicknesses of 0.6 m (normal), 1.2 m (fast), 1.8 m (very fast), 3.6 m (rapid), and 7 m (two stage construction); positive vertical displacement represents ground heave, and negative vertical displacement indicates settlement

3.5 Conclusions

From the parametric study presented in this paper, it can be concluded that:

- Increasing the time allowed for consolidation following the addition of each fill layer increases the stability of the embankment due to an increase in the shear strength and increasing the pace of construction decreases the stability of the embankment due of the loss in shear strength of soft foundation soils.
- The parametric study shows that the design height can be reached in the case of an embankment constructed with a fill layer thickness of 0.6 m and a consolidation period of 32 days or more following the placement of each fill layer. Moreover, the study shows that with a fill layer thickness of 1.2 m and a consolidation period of 64 days, the design height of the embankment can be reached with a factor of safety of 1.27.
- The parametric study shows that with fill layer thicknesses of 1.8 m and 3.6 m, in most cases failure occurs at an embankment height of 10.8 m, since the addition of thicker fill layers results in greater applied stresses and a greater percentage increase in excess pore water pressure within a shorter period of time.
- The settlement increases as the time allowed for consolidation increases. Greater settlement results in greater strength gained by the soil, and lower excess pore water pressures. Thus, with a settlement of more than 700 mm at the end of construction and with an excess pore water pressure of around 19%, no embankment failure occurs. However, long periods of time are required to achieve this level of excess pore water pressure dissipation.
- With thicker fill layers, the pace of construction is accelerated, and there is less time for settlement to occur. Thus, settlement is reduced when the layers are thicker and

there is a smaller number of layers. This reduces the gain in soil strength and can result in failure of the embankment.

- The percentage increase in excess pore water pressure becomes greater as the fill layer thickness increases and declines as the time allowed for consolidation increases. Therefore, increasing the thickness of the fill layers reduces the stability of the embankment, whereas increasing the consolidation period gives excess pore water pressures time to dissipate, resulting in increased embankment stability.
- In cases where there is no embankment failure and the design height of 14 m is achieved, the percentage increase in excess pore water pressure is around 22% to 28%. In contrast, in cases where embankment failure occurs at a height lower than 14 m, the percentage increase in excess pore water pressure is around 55% to 65%. This underlines the importance of allowing sufficient time for the consolidation of soft soils.
- The horizontal lateral displacement of the embankment toe increases as the fill layer thickness increases and decreases as the time allowed for consolidation following the placement of each fill layer increases. This essentially means that failure due to lateral displacement occurs when the embankment is constructed quickly with thick fill layers, and insufficient time is allowed for consolidation.
- Vertical lateral displacement with movement in an upward direction, in the form of ground heave, increases when the fill layer thickness increases. The magnitude of vertical lateral displacement in the form of downward movement increases as the time allowed for consolidation following the placement of each fill layer increases. Downward displacement indicates settlement and thus a gain in soil strength and stability. In contrast, upward displacement means that stability is reduced, with a loss of soil strength.

DATA AVAILABILITY

All data, models, and code generated or used during the study appear in the submitted article.

ACKNOWLEDGEMENTS

The authors acknowledge funding provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) to the second author for this research project.

3.6 References

1. Jose Leitao Borges (2004) “Three-dimensional analysis of embankments on soft soils incorporating vertical drains by finite element method” Computers and Geotechnics 31 (2004) 665–676.
2. Muktabhant, C., Teerawong, P., and Tengamnuay, V., “Engineering properties of Bangkok subsoils”, Proc. of Southeast Asian Conference on Soil Engineering, Bangkok, 1, 1967, pp1-7.
3. Moh, Z. C., Nelson, J. D., and Brand, E. W., “Strength and deformation behaviour of Bangkok clay”, Proc. of 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico, 1, 1969, pp287-295.
4. Eide, O., “Geotechnical engineering problems with soft Bangkok clay on the Nakhon Sawan highway project”, Pub. No 78, Norwegian Geotechnical Institute, Norway, 1968.
5. Eide, O., “Exploration, sampling and in-situ testing of soft clay in the Bangkok area”, Proc. of International Symposium on Soft Clay, Thailand, 1977, pp122-137.
6. Bergado, D.T., Anderson, L.R., Miura, N., and Balasubramaniam, AS, "Soft Ground Improvement", American Society of Civil Engineers, 1996, 427 pages
7. Lei Ma, Shui-Long Shen , Chun-Yong Luo & Ye-Shuang Xu (2011) “Field Evaluation on the Strength Increase of Marine Clay under Staged Construction of Embankment” , Marine Georesources & Geotechnology, 29:4, 317-332.
8. Maryam Gaber, Anuar Kasa, Norinah Abdul-Rahman, Jamal Alsharef (2019) “Simulation of Sequential Construction of Embankments on Reinforced Soft Clay Foundation” Indian Geotech J (April 2019) 49(2):224–231.
9. Zhen-Yu Yin, Minna Karstunen, Ching S. Chang, Mirva Koskinen and Matti Lojander (2011) “Modeling Time-Dependent Behavior of Soft Sensitive Clay” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 137, No. 11, ISSN 1090-0241.
10. A. Burman, S. Kumar, N. Himanshu and D. Maity (2015) “Simulation of sequential construction of embankment by finite element method” Asian journal of Civil Engineering (BHRC) vol. 16, no. 3 (2015), pages 347-367.
11. Sukru Ozcoban, Mehmet M. Berilgen, Havvanur Kilic, Tuncer B. Edil, and I. Kutay Ozaydin (2007) “Staged Construction and Settlement of a Dam Founded on Soft Clay”

Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 8, ASCE, ISSN 1090-0241/2007/8-1003–1016.
12. Madhat Shakir Al-Soud (2016) " Numerical analysis of prefabricated vertical drains improved soft soil beneath an embankment during staged construction" Journal of Engineering and Development Vol. 20, No.1, January 2016, ISSN 1813-7822.
13. AK M. Zein and L. E. Elgasim (2014) "Study of various techniques for improving weak and compressible clay soil under a high earth embankment" MATEC Web of Conferences 11, 03006 (2014), ISSN 10.1051/ matecconf/20141103006.
14. Huai-Na Wu, Shui-Long Shen, Lei Ma, Zhen-Yu Yin & Suksun Horpibulsuk (2015) "Evaluation of the Strength Increase of Marine Clay under Staged Embankment Loading: A Case Study", Marine Georesources & Geotechnology, 33:6, 532-541.
15. B. G. Sexton, B. A. McCabe (2013) "Numerical modelling of the improvements to primary and creep settlements offered by granular columns" Acta Geotechnica (2013) 8:447–464.
16. Bernie Mills and Jared McGinn (2010) "Design, Construction, and Performance of a Highway Embankment Failure Repaired with Tire-Derived Aggregate" Transportation Research Record: Journal of the Transportation Research Board, No. 2170, Transportation Research Board of the National Academies, Washington, DC, 2010, pp. 90–99.
17. Bernie Mills, Hany El Naggar, Arun Valsangkar (2015) "North American Overview and a Canadian Perspective on the Use of Tire-Derived Aggregate in Highway Embankment Construction" Chapter 22, Ground Improvement Case Histories, Volume 2, Elsevier Ltd.
18. Manual on estimating soil properties for foundation design (1990), prepared by Cornell University, Ithaca, New York, EPRI EL-6800.
19. PLAXIS material models 2019
20. Zhongkun Ouyang and Paul W. Mayne (2018) "Effective friction angle of clays and silts from piezocone penetration test" Can. Geotech. J. 55: 1230–1247 (2018)
21. Myint Win Bo, Arul Arulrajah, Patimapon Sukmak, Suksun Horpibulsuk (2014) "Mineralogy and geotechnical properties of Singapore marine clay at Changi" Japanese Geotechnical Society, Soils and Foundations 2015; 55 (3):600–613, Elsevier.

CHAPTER 4 TECHNICAL PAPER II

Improving the Stability of High Embankments Founded on Soft Marine Clay by Utilising Prefabricated Vertical Drains and Controlling the Pace of Construction

Rakshith Badarinath [1] and Hany El Naggar [2]

[1] Graduate Student, Department of Civil and Resource Engineering, Dalhousie University, Nova Scotia, Canada B3H 4R2. Email: Rk822002@dal.ca

[2] Professor, Department of Civil and Resource Engineering, Dalhousie University, Nova Scotia, Canada B3H 4R2. Email: hany.elnaggar@dal.ca

Corresponding Author: Hany El Naggar, Ph.D., P.Eng.
Professor, Dalhousie University
Tel.: 1(902) 494 3904, email: hany.elnaggar@dal.ca

Abstract: In many cities worldwide, the development of sites underlain by thick deposits of soft marine clay has become more common, to meet the demand for essential transportation infrastructure arising from the current unprecedented urban population growth. Staged construction and ground improvement techniques are two approaches frequently used in constructing high embankments over soft soils. These approaches improve stability, ensuring that the pace of construction does not cause a buildup of excess pore water pressures and an associated reduction in shear strength, which could lead to progressive failure. This study examines the use of prefabricated vertical drains (PVD‟s) and the pace and sequence of construction, to determine the most suitable method for constructing high embankments on soft soil deposits. The problem under consideration is modelled by using finite element analyses with the PLAXIS 2D software, where the PVD‟s are modelled via pre-coded drainage line elements. The modelling was carried out with a staged construction technique. The analyses considered PVD spacing ranging from 1 m to 2.5 m, with PVD‟s extending to the crest, to the toe, or beyond the toe of the embankment. The fill layer thickness and the consolidation time following the placement of each layer were also varied to study the effect of these parameters on overall stability. The findings obtained from this study show that PVD‟s can significantly improve the stability of embankments when combined with an appropriate consolidation time for a faster construction pace.

Keywords: Consolidation, Marine clay, Excess pore water pressure, Prefabricated vertical drains, Settlement, Plain strain model.

4.1 Introduction

Studying the behaviour of high embankments founded on soft marine soils is a challenging area of soil mechanics, due to the complex problems posed by the weak geotechnical characteristics of the foundation soils. The low strength of marine clays significantly limits the load (and embankment height) that can be applied while maintaining adequate safety for short-term stability. In addition, the high deformability and low permeability of these soils result in excessive settlement and very slow dissipation of excess pore water pressures, which in turn significantly reduces the shear strength. Various constructive techniques are available for designing high embankments on soft soils. Altering the properties of the foundation soil or the fill layers can increase the global stability, accelerate consolidation, and reduce creep settlement. However, the most commonly applied technique for accelerating the rate of consolidation and reducing the consolidation time is the use of prefabricated vertical drains (PVD‟s) in the foundation soils. [1] Early work on soft Bangkok clay was carried out by Muktabhant et al. [2], Moh et al. [3], Eide [4 and 5], and Bergado et al. [6]. Since saturated marine clays have very low shear strength, rapid construction of embankments cannot be carried over such soils. To increase the overall stability, embankments are built using a staged construction technique, where each stage of the fill is allowed to consolidate after the construction stage. PVD‟s have been known since decades to be an efficient means of accelerating the rate of consolidation. In combination with the use of PVD‟s, staged construction has become the most appropriate technique in the construction of embankments. [7]

With rapid urbanization, construction on soft soils is constantly increasing, especially in the coastal regions of many developing countries. Therefore, it is essential to stabilize the existing soft soils prior to construction, in order to avoid excessive primary or creep settlement. Although a variety of soil improvement techniques are available, the application of PVD"s is still the most economical solution. [8] The application of PVD"s provides an artificially created drainage by shortening the drainage paths, so that excess pore water pressures can dissipate radially rather than vertically. A key characteristic of PVD"s are that they increase the soil shear strength by increasing the rate of primary consolidation, thereby resulting in the strength gain of the soft soils. [9] From the various vertical drains used for rapid dissipation of excess pore pressures, PVD"s are the most economical. In combination with the use of surcharge fill, the radial drainage paths of excess pore pressures created by PVD"s stabilize the soft ground by increasing the soil shear strength and hence reducing the primary settlements. [10]

In 1940, prefabricated band-shaped drains and Kjellman cardboard wick drains were introduced in the field of ground improvement. Several other types of PVD"s have been developed since then, such as the Geodrain (in Sweden), Alidrain (in England), and Mebradrain (in the Netherlands). PVD"s consist of a perforated plastic core functioning as a drain, and a protective sleeve of fibrous material acting as a filter around the core. PVD"s are usually installed by using either a dynamic or a static method. In the dynamic method, a steel mandrel is driven into the ground by using either a vibrating hammer or a conventional drop hammer. In the static method, a mandrel is pushed into the soil by means of a static force. Although the dynamic method is quicker, it disturbs and hence distorts the structure of the surrounding soil during the installation. This leads to an increase in shear strain, accompanied by an increase in the total stress and pore water pressure, in addition to the displacement of the surrounding soil. [11]

The effectiveness of PVD"s are related to the drain characteristics, including the discharge capacity. A decrease in discharge capacity referred to as well resistance, retards the dissipation of excess pore pressures and the primary settlement. The main causes of well resistance are deterioration of the drain filter, silt intrusion into the filter (thus reducing the

Vertical Hydraulic conductivity of the drain) and folding of the drain due to lateral movement. [12] The finite element method (FEM) has been generally adopted to analyze the behaviour of PVD-improved soft foundation soils for construction of embankments. There are three existing approaches for numerically modelling the embankments over PVD-modified soft soils. The first method employs a one-dimensional (1D) drainage element. The second method uses a macro-element in the FEM program to consider the drainage behaviour of PVD"s. The third method is an approximate approach, which estimates an equivalent value for the vertical hydraulic conductivity of PVD-improved soft soil deposit. [13]

This study makes use of drainage line elements that are pre-coded in PLAXIS 2D. Hence, in the numerical modelling, drain design parameters such as the smear effect, well resistance, and discharge capacity are ignored. A drain line element acts as a macro-element which examines the vertical hydraulic conductivity of the drains by considering all the pore water pressures in the drain nodes to be equal to zero. Thus, the excess pore water pressures are dissipated via the drain element to the top of the soil layer through the difference in pore water pressure between the soil and the drain element.

The numerical models described in this paper are based on the embankment design and soil profile of a case study of an embankment in St. Stephen, New Brunswick, Canada, which failed in 2006 during the construction of a four-lane highway leading to the Canada-USA border crossing in St Stephen. [14] In the present research, a two-dimensional finite element model of the embankment was prepared by estimating the soil properties via back analysis from the failure results obtained from the case study. After a failure similar to that observed at the site was established in the numerical model, drain line elements were added as a ground improvement technique, to study their effect on the stability of high embankments constructed over soft soil deposits. A parametric study was conducted to investigate the effect of the spacing and extent of coverage of the PVD"s; the effect of different fill layer thicknesses and extent of PVD coverage with constant drain spacing; and the effect of different consolidation times for thicker fill layers, with constant drain spacing and extent of coverage of the PVD‟s.

4.2 Methodology

First the soil profile and embankment were modelled in accordance with the cross-section of the embankment from the case study (see Figure 1). Then drainage line elements were added to model PVD‟s in the parametric study. This was done to examine the effects of drain spacing and the extent of coverage of the PVD‟s on the stability of the embankment.

In PLAXIS 2D, when 15-node (fourth-order) soil elements are employed, each drainage line element is defined by five nodes. However, when 6-node (quadratic) soil elements are used, each drainage line element is defined by three nodes. [15] This study makes use of 15-node soil elements. Relationships between the axial force and displacement and the axial flow rate and hydraulic gradient are established in the element stiffness matrix. In the consolidation analyses, the drainage line elements are assumed to have a negligible area cross-section. [15]

The drainage line elements in PLAXIS 2D are handled as seepage boundaries and are located inside the domain. At atmospheric pressure, the drains cannot work perfectly and do not permit the discharge of water leaving the domain. Hence, a prescribed head, $\emptyset^*$, should be considered for drains below the water level. Thus, the conditions are:

- $\emptyset = \emptyset^*$; in the case of outflow
- $q.n = q_x n_x + q_y n_y + q_z n_z = 0$; in the case of suction

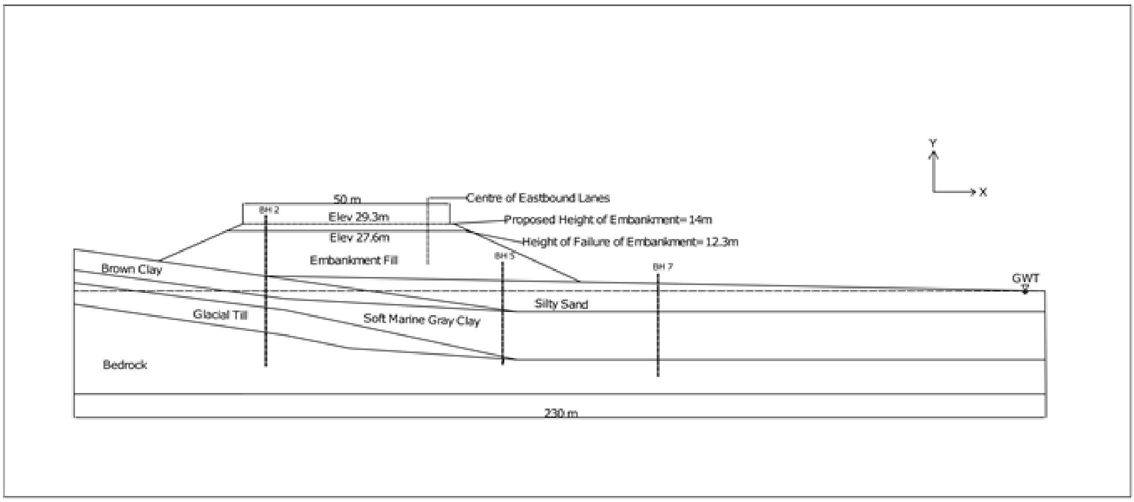
where n_x, n_y and n_z are the outward-pointing fast vector components on the boundary, and q is the discharge. Here the drain itself does not generate a resistance against flow. [15]

The drawback of using drainage line elements is that they do not simulate the exact behaviour of drains which would be used at the site, since they do not account for the well resistance, discharge capacity, smear zone effects, or clogging of the drains due to the migration of soil particles during installation. However, from work done by Wong et al. in 2013, it can be concluded that the drainage line elements, although simplistic, can be used to study the effects of using drains on the stability of embankments on soft soils. However, they should not be used to design or model drains that are to be implemented in a specific project.

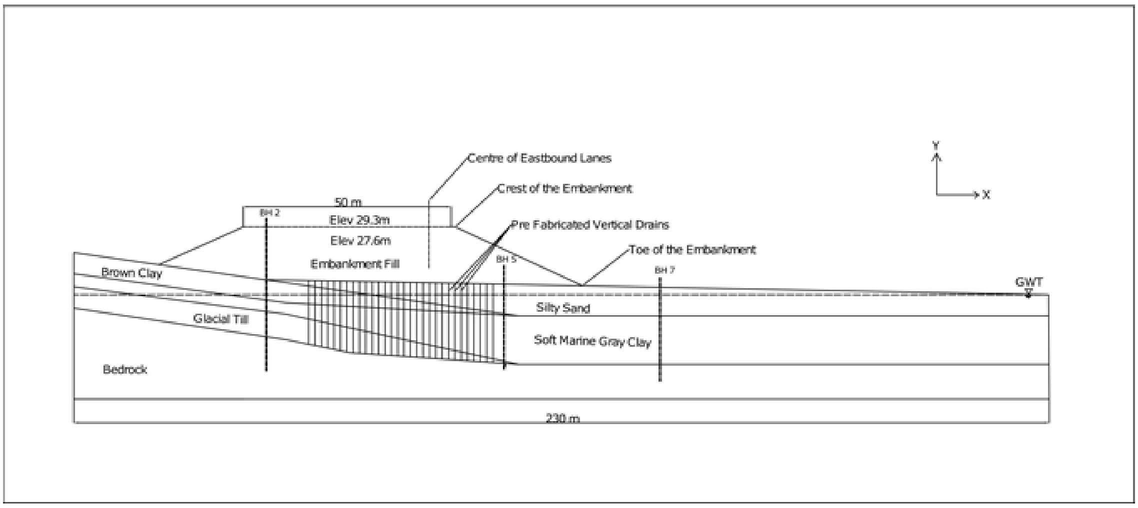
4.3 Site Conditions and Geology

The subsurface profile and geology of the site were studied following the embankment failure. Standard penetration tests (SPTs) were conducted after the failure, immediately prior to reconstruction of the embankment. Based on the SPT corrected values, a generalized soil profile, shown in Figure 4.1, was developed via back analysis. SPT tests were conducted at seven boreholes on the site. The locations of boreholes 2, 5, and 7 are indicated in Figure 4.1.

At borehole (BH) 2, a layer of fill 8 m thick was underlain by a very thin layer of topsoil (approximately 0.1 m), resting on a very soft gray silty lean clay layer. Under this was a layer of dense gray silty sand 0.7 m thick, underlain by a layer of glacial till of thickness 4.6 m. Bedrock was encountered at about 19 m below ground level. At borehole 5, a layer of fill 3.5 m thick was underlain by 4.1 m of compact to dense brown silty sand with gravel, which rested on a very soft gray lean clay layer. Bedrock was encountered at approximately 18.5 m below ground level. At borehole 7, 3.7 m of very loose to compact brown clayey sandy silt with gravel was underlain by a thin layer (0.6 m) of soft sandy clay, resting on a layer of very soft gray clay 10.6 m thick. This was underlain by another thin layer (0.7 m) of compacted silty clayey sand with gravel, with bedrock located at a depth of 15.6 m beneath the ground level.



(a)



(b)

Figure 4.1: Site soil profile cross-section with boreholes (BH), showing (a) the proposed height of the embankment and the height at failure, and (b) PVD‟s with a spacing of 1.5 m c/c

The water content, W_C, of the soft clay layer ranged from 30% to 35%; with a plastic limit, W_P, of 22% and a liquid limit, W_L, of 45%. The vertical coefficient of consolidation, C_V, was estimated to be 0.010 cm^2/min, and the horizontal coefficient of consolidation, C_H, was estimated to be 0.012 cm^2/min [16]. The mechanical properties of the PVD‟s used as a ground improvement measure (with 1.5 m spacing in a triangular pattern) are shown in Table 4.2, as reported by Bernie et al. [16]

4.4 Soil Properties

Based on the SPT values, empirical relations were used to determine the strength parameters of the different types of soil at the site. Empirical relations [17] were used to calculate the undrained cohesion (S_U) of the soft gray clay layer. From the water content of 30% to 35% and the plasticity index (*PI*) of 22%, other parameters of the soft soil, such as C_C and C_S, which determine the stiffness during loading and unloading, were obtained. The stiffness of the other soils was determined by using relations based on the average SPT values. The marine clay layer was modelled by using the soft soil (SS) model in PLAXIS 2D, and the other soils were modelled by using the hardening soil (HS) model. A detailed description of the soil models considered is provided in the following section. Table 4.1 summarizes the soil properties used in the numerical model, which were determined from the average SPT values, based on empirical relations. The hydraulic conductivities were determined by using estimates of C_V and C_H obtained from the site. The three stiffness parameters used in the hardening soil model are explained in sections *4.5.3.1* and *4.5.3.2*. The marine clay strength parameters for the given water content and plasticity index are estimated in accordance with Zhongkung et al. [18] and Myint et al. [19]

(a) (b)

Figure 4.2: (a) Section of route 1 divided highway leading to the Canada-USA border crossing at St. Stephen, New Brunswick, and (b) embankment failure at the site (west view) (Bernie 2010)

Table 4.1: Summary of soil properties used in the numerical model

Soil layers	**Soil model**	**Unit weight (kN/m^3)**		**Stiffness parameters (MPa)**			**Poisson's ratio**	**Cohesion (kPa)**	**Friction angle (°)**	**Dilation angle (°)**	**Hydraulic conductivity (m/day)**
		γ_{dry}	γ_{sat}	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}					
Fill	HS	19.5	22	50	40	150	0.2	1	38	8	8.65×10^{-1}
Soft grey marine clay	SS	13.8	16.1	C_c= 0.35* C_s= 0.035*			0.495	0	28	0	8.65×10^{-5}
Brown clayey silty sand	HS	16	20	30	24	90	0.2	5	29	0	8.65×10^{-4}
Silty sand	HS	16.5	20	40	32	120	0.2	0	31	1	8.65×10^{-3}
Glacial till	HS	17	20	40	32	120	0.2	0	34	4	8.65×10^{-2}
Bedrock	Linear non-porous	28	-	E= 6200			0.15	-	-	-	-

*unitless

Table 4.2: Summary of properties of PVD‟s used at the site

Properties	**Test method**	**Units**	**Value**
Weight	ASTM D1777	g/m	75
Width	-	mm	95
Thickness	ASTM D5199	mm	3.0
Mass of filter	ASTM D1777	g/m^2	140
Equivalent diameter of core	-	mm	65
Discharge capacity: at 10 kPa at 300 kPa	ASTM D4716	m^3/s	100x10^{-6} 50x10^{-6}
Pore size opening	ASTM D4751	mm	0.075

Table 4.2: Summary of properties of PVD‟s used at the site

Permeability	ASTM D4491	cm/s	0.02
Permittivity	ASTM D4491	s^{-1}	0.3

4.5 Numerical Modelling

Finite element analyses were carried out via PLAXIS 2D for the assumed plain strain problem. The model geometry was constructed from the cross-section of the soil profile shown in Figure 4.1. The groundwater table is assumed to be at the ground level and is assumed to remain constant in the upward sloping soil profile. Since the soil profile is not symmetric, the entire profile was modelled and analyzed. Each model consists of a construction phase modelled as a plastic calculation, and a consolidation phase followed by a construction phase modelled as a consolidation type calculation in PLAXIS 2D. The initial models involved modelling the fill being placed in layers with a thickness of 0.6 m, with PVD‟s spacing of 1 m, 1.5 m, 2 m, and 2.5 m, and with PVD‟s extending to the crest, to the toe, or beyond the toe of the embankment. In subsequent models, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m were used with a constant PVD spacing, and the time allowed for consolidation after the placement of each fill layer was set at 5 days, 10 days, or 15 days, in order to study the effect of the pace of construction on the stability of the embankment. A detailed explanation of the numerical models is given in section 4.5.4.

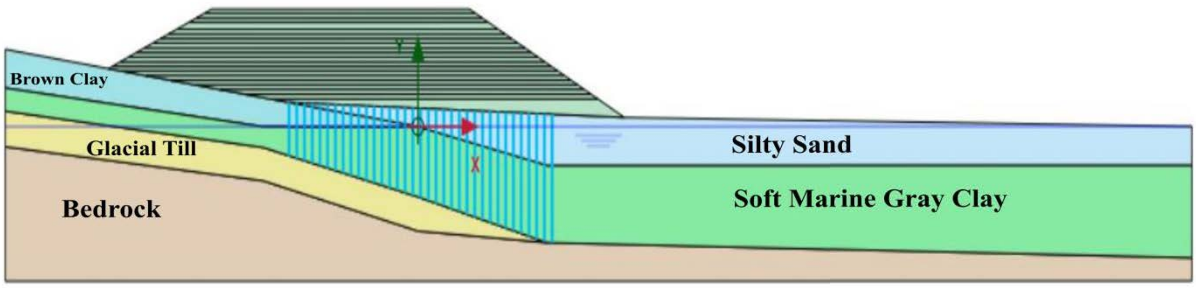
4.5.1 Geometry

Figure 4.3 shows the geometry adopted for the problem. The soil profile consists of a brown clayey silty sand layer and a silty sand layer resting on a marine clay layer 15 m thick, underlain by glacial till and bedrock. The side slope of the profile shown on the right-hand side of Figure 3 is assumed to be 1V:2.14H and the side slope of the profile shown on the left-hand side of Figure 3 is assumed to be 1V:1.43H, in accordance with the dimensions observed in the case study. The width of the crest of the embankment is 50 m, and it is assumed that the soil profile extends an additional 70 m on the right-hand side of Figure 4.3; representing a total width of 150 m. Figure 4.3 also shows the various soil layers in the soil profile at the site. The soil profile in the numerical model is assumed to be isotropic. The marine clay is modelled in an undrained condition by the soft soil model, and the brown clayey sandy silt and other soils (except for the bedrock) are modelled in a drained condition by the hardening soil model. In the mesh, 15-noded triangular elements are used, with 3 degrees of freedom for each node. [20] In the numerical models, the PVD‛‛s are considered with various configurations of drain spacing and extent of coverage of the PVD‛‛s. Figure 4.3 illustrates three different extents of PVD coverage that are considered in this study.

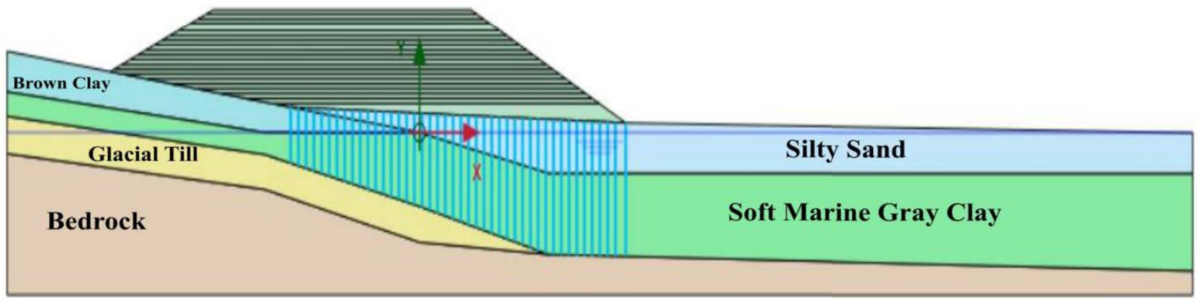
4.5.2 Meshing and Boundary Conditions

A very fine mesh is used for all the numerical models in this study. A coarseness factor of 0.045 is used in the soil region between the drains and the drain line elements. A coarseness factor of 0.1 is used at the interface of the first fill layer and the upper soil profile in order to avoid any errors resulting from the meshing of small soil elements.

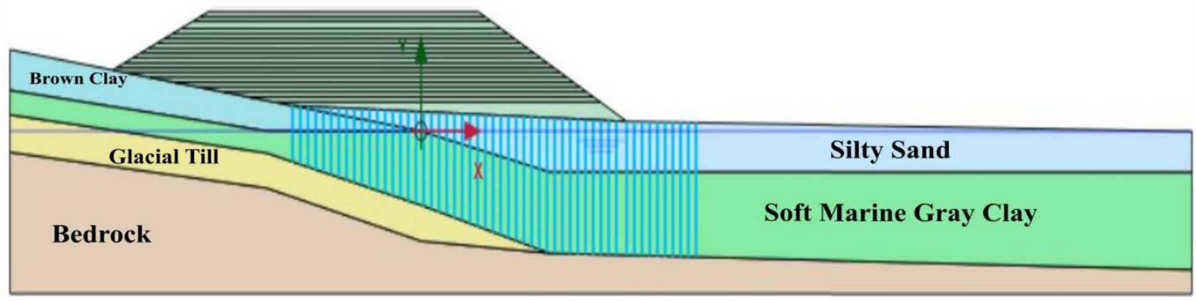
Figure 4.4 illustrates the different spacing of the PVD‛‛s, with PVD‛‛s extending to the crest of the embankment. With a drain spacing of 1 m center-to-center (c/c) there were 45,536 elements, with a spacing of 1.5 m c/c there were 47,846 elements, with a spacing of 2 m c/c there were 49,636 elements, and with a spacing of 2.5 m c/c there were 51,620 elements. With PVD‛‛s extending only to the crest of the embankment, for a drain spacing of 1 m, the number of elements was around 45,000; while with PVD‛‛s extending beyond the toe of the embankment, for a drain spacing of 2.5 mm, the number of elements was approximately 67,000.



(a)



(b)



(c)

Figure 4.3: Models with PVD‟s extending (a) to the crest of the embankment, (b) to the toe of the embankment, and (c) beyond the toe of embankment

In the models, it is assumed that the groundwater can drain freely in all directions except at the bottom, which is fixed, since the bedrock is non-porous. For the deformation boundary conditions, a normally fixed position is assumed for the sides of the model, the bottom is assumed to be fully fixed, and the top is assumed to be free. For the consolidation phase, a fully fixed position is assumed for all the boundaries except the top boundary, where deformation is allowed to occur freely.

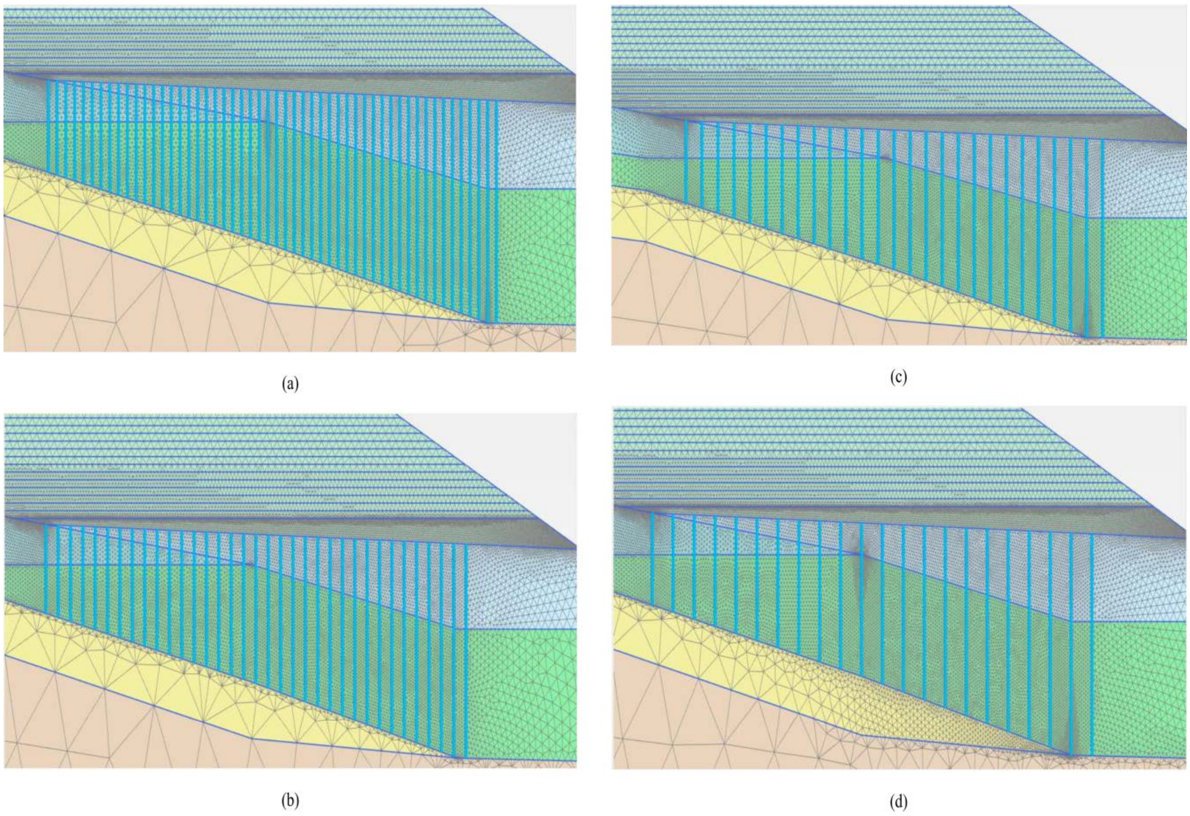


Figure 4.4: Mesh with a PVD spacing of (a) 1 m, (b) 1.5 m, (c) 2.0 m, and (d) 2.5 m

4.5.3 Soil Models used in Numerical Modelling

In this study, the soil models used for numerical modelling are the soft soil model (SSM) and the hardening soil model (HSM), described below.

4.5.3.1 Soft Soil Model

To model soft soils such as rapidly consolidated clay, peat, and clayey silts, the non-linearity of these soils must be considered. It is considered via stress-dependent stiffness. In the soft soil model, as shown in Eq. (1), the total strain (ε_T) is composed of an elastic component (ε^e) and a viscoplastic component (ε^p), and is based on the Mohr–Coulomb failure criterion. The modified compression index (λ*) and the modified swelling index (k*) are key model inputs, which define the compressibility of the soil in isotropic loading, and in unloading and subsequent reloading, respectively. Alternatively, if C_c and C_s are defined, the software can use these parameters to calculate λ* and k*. The yield surface of the soft soil model is similar to that of the Mohr-Coulomb model, except that in the soft soil model the yield function defines an ellipse, where M is the height of the ellipse, and the isotropic pre-consolidation pressure, P_p, defines the length of the ellipse in the p' plane. Stress changes below the yield cap induce the development of elastic strains. Once the yield stress is exceeded, plastic strains are developed. The mean yield stress is updated based on the amount of accumulated plastic strain. [22] The height of the ellipse, M, also represents the ratio of horizontal to vertical stresses in primary loading, and is therefore used to determine K_o^{nc}, where K_o^{nc} represents the earth pressure at rest. The top of all the ellipses coincides with the M line, and the M line has a slope greater than the line defined by Mohr-Coulomb yield line. Thus, the M line is referred to as the critical line. The pre-consolidation pressure decreases with compaction. It is assumed that at the initial stage there is no plastic volumetric strain, corresponding to P_{po}, where P_{po} is the pre-consolidation pressure during the initial stage.

The total strain has components of elastic and plastic strain, as shown in Eq (1):

$$\varepsilon_T = \varepsilon^e + \varepsilon^p \quad (1)$$ [15]

where ε^e is the elastic component, ε^p is the plastic component, and ε_T is the total strain.

4.5.3.2 Hardening Soil Model

The hardening soil model accounts for soil nonlinearity via the stress-dependency of stiffness but does not account for long-term conditions such as creep and stress relaxation. The hardening soil model employs the theory of plasticity rather than the elastic perfectly plastic theory used in the Mohr-Coulomb model, and hence accounts for stress hardening of the soil. Unlike the soft soil model, which can be used to model only soft soils, the hardening soil model can be used to model different types of soil, including soft soils and granular soils. The stress-dependency of soil stiffness is defined by the power m, where $m = 1$ is typical for soft soils. The model requires the input of three stiffness parameters at a chosen reference pressure (P_{ref}): E_{50}^{ref} (the secant modulus/triaxial modulus), E_{oed}^{ref} (the oedometric modulus), and E_{ur}^{ref} (the unloading-reloading modulus). The yield surface consists of a parabolic curve, and soil dilatancy and a yield cap are considered. The position of the shear hardening yield surface is determined mainly by the triaxial modulus, while the position of the yield cap (associated with compression hardening) is primarily determined by the oedometric modulus [22].

For soft soils (where $m = 1$), the stiffness moduli can alternatively be entered in terms of the compression (C_c) and swelling (C_s) indices, which are related to the stiffness moduli as shown in Equations (2), (3), and (4), where γ is Poisson‟s ratio, e_o is the initial void ratio, and P_{ref} is the reference pressure. [22]

$$E_{oed}{}^{ref} = 2.3\,(1 + e_o)\frac{P_{ref}}{C_c} \qquad (2)$$

$$E_{ur}{}^{ref} = 2.3\,(1 + e_o)\frac{(1+\gamma)(1-2\gamma)P_{ref}}{(1-\gamma)C_c} \qquad (3)$$

$$E_{50}{}^{ref} = 1.25\,E_{oed}{}^{ref} \qquad (4)$$

where $E_{50}{}^{ref}$ is the stiffness modulus for primary loading in the drained triaxial shear test, $E_{ur}{}^{ref}$ is the stiffness modulus for unloading and reloading in drained triaxial shear test, and

${E_{oed}}^{ref}$ is the stiffness modulus for primary loading in the oedometer test. In the present study, it is assumed that ${E_{oed}}^{ref}$ = 0.8 ${E_{50}}^{ref}$.

For a more detailed explanation of the soil models, please refer to the PLAXIS material models manual. [21]

4.5.4 Development of the finite element model and objectives of the parametric study

4.5.4.1 Development of the finite element model

The construction sequence of the embankment built at St. Stephens is modelled as a staged construction procedure, which is analyzed in PLAXIS 2D. The construction of the embankment was begun in 2005 and preceded until failure of the embankment in July 2006. In 2005, the embankment was constructed to a height of 5 m in 90 days. Construction then stopped for the winter, which allowed the embankment to consolidate for 270 days. Construction of the embankment resumed on June 11, 2006, and continued until July 4, 2006, when the embankment failed at a height of 12.3 m. In this study, the soil parameters of the underlying marine clay layer were varied in the model, until a deep-seated circular slip failure was replicated at an embankment height of 12.3 m, as was observed at the St. Stephens site.

- For the control case, first the soil profile was modelled, and then the embankment construction sequence was modelled by using a staged construction technique with a fill layer thickness of 0.6 m. A consolidation period of 15 days was allowed after the placement of each fill layer until the embankment reached a height of 5 m. For the subsequent construction, the consolidation period allowed after the placement of each fill layer was 1 day until the embankment reached a height of 12.3 m, which was when failure occurred. The failure was replicated as observed at the site. In the control case, no drains were used, since the purpose of the control case was primarily to calibrate the model in accordance with the failure that was observed at the site.

- For the case with a normal pace of construction, the thickness of the fill layer was set to 0.6 m. The soil profile was enhanced with PVD‟s modelled via drainage line elements in PLAXIS 2D, with variations in the drain spacing and the extent of PVD coverage. Drain spacing of 1 m, 1.5 m, 2 m, and 2.5 m were used, with PVD‟s extending to the crest, to the toe, or 14 m beyond the toe of the embankment. The distance of 14 m beyond the toe of the embankment was chosen, because the distance between the crest and the toe of the embankment was also roughly equal to 14 m. The consolidation period following the placement of each fill layer was considered to be 1 day for the purpose of this study.
- For the case with a fast pace of construction, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m were used, with a constant drain spacing of 1.5 m. The PVD‟s extended to the crest, to the toe, or to 14 m beyond the toe of the embankment. A drain spacing of 1.5 m was selected, because findings from the case with a normal pace of construction indicated that in terms of economy and safety, this spacing was the most suitable (see section 4.6.2). The consolidation time following the placement of each fill layer was 1 day, for the purpose of this study. This case was designed to investigate the effect of different fill layer thicknesses combined with the use of PVD‟s, as well as effect of varying the extent of PVD coverage.
- For the case with construction allowing more time for consolidation, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m were used, with a constant drain spacing of 1.5 m. In this case, the PVD‟s extended to the toe of the embankment. This extent of PVD coverage was selected, because findings from the case with a fast pace of construction indicated that in terms of economy and safety, this extent of coverage was the most suitable (see section 4.6.3). Consolidation times of 5 days, 10 days, and 15 days were used. This case was designed to investigate the effect of different consolidation times

on the stability of embankments constructed over soft soils, with a constant drain spacing and constant extent of coverage by the PVD"s.

4.5.4.2 Objectives of the parametric study

A parametric study was conducted which varied the drain spacing, the extent of the area covered by the PVD"s, the fill layer thickness, and the consolidation time. There are few reports in the literature showing the effects of these parameters, although these are some of the essential factors that need to be prioritised to optimise safety, costs, and construction time, in the design of embankments constructed over soft soils. The three cases considered in the parametric study can be summarized as follows.

- Case with normal pace of construction: Fill layer thickness of 0.6 m; drain spacing of 1.0 m, 1.5 m, 2.0 m, and 2.5 m; PVD"s extending to the crest, to the toe, and beyond the toe of the embankment; and consolidation time of 1 day.
- Case with fast pace of construction: Fill layer thicknesses of 2.0 m, 3.5 m, and 4.5 m; constant drain spacing of 1.5 m; PVD"s extending to the crest, to the toe, and beyond the toe of the embankment; and consolidation time of 1 day.
- Case with construction allowing more time for consolidation: Fill layer thicknesses of 2.0 m, 3.5 m, and 4.5 m; constant drain spacing of 1.5 m; PVD"s extending to the toe of the embankment; and consolidation times of 5 days, 10 days, and 15 days.

4.6 Results and Discussions

This paper focuses primarily on the embankment design parameters of factor of safety (FS), settlement, the excess pore water pressures generated, and the extent of lateral and vertical deformation. A crucial aspect of embankment design is global stability, to prevent failure and excessive deformation. Thus, the results obtained from the models in the parametric study are used to study the effects of using PVD"s, the drain spacing, the extent of the area covered by the PVD"s, and the pace of construction on the stability of high embankments built on soft soils. Because it was necessary to calibrate the numerical models to replicate the failure observed in the field, the control model calibrated from the construction sequence used in the St. Stephens case study is considered first, as the control case. Gravity loading is used for the initial calculation in all the numerical models, because the layers of soil are not horizontal. Each construction phase, involving the placement of a fill layer, is assumed to have duration of 1 day as a plastic calculation phase. The installation of PVD"s is assumed to have duration of 2 days as a plastic calculation phase. The results are discussed for the control case, the case with a normal pace of construction, the case with a fast pace of construction, and the case allowing more time for consolidation.

4.6.1 Control Case

From Figure 4.5, it can be seen that with a consolidation period of one day following the placement of each fill layer, the factor of safety (FS) decreases as the embankment height increases. The results are presented for embankment heights of 7.2 m, 9.6 m, and 12.3 m above ground level. The embankment height of 12.3 m is the height immediately before failure. The FS at an embankment height of 7.2 m is 1.681, and the FS immediately before failure is 1.034. Thus, the stability of the embankment decreased as the height of the embankment increased, and it was not possible to complete construction of the embankment to the full design height, due to the loss of shear strength in the foundation soil.

In Figure 4.5, the lower graph shows the change in settlement of the embankment as the embankment height increases. At an embankment height of 7.2 m, the settlement is 267.3 mm, and immediately before failure the settlement is 427.1 mm. Settlement is one of the most important parameters to be considered in the design of embankments. The greater the settlement during the construction period, the more strength is gained by the soft soil during the consolidation process, since faster settlement in the initial period indicates a rapid dissipation of excess pore water pressures. In the control case, there was a 9-month break in construction during the winter. The subsequent construction phase that began in the spring of 2006 was accelerated, with a consolidation period of only one day following the placement of each fill layer. This resulted in insufficient settlement and a reduced gain in soil strength, with little dissipation of excess pore water pressures. This eventually led to failure of the embankment at a height of 12.3 m, due to shear strength failure. Thus, the consolidation time plays a key role in determining the overall stability of embankments constructed over soft soils.

In Figure 4.5, the upper right-hand graph shows that lateral displacement increases as the height of the embankment increases. It was estimated that one of the causes of failure at the study site was the lateral deformation of the toe of the embankment. Hence, this is also an important parameter to be considered in the design of embankments. During the construction and consolidation phases, deformation of the toe of the embankment occurs due to the high deformability of the soft soils, which means that this factor must always be taken into consideration in the design of such embankments. Because the effects of lateral deformation have not been extensively investigated in the literature, this paper attempts to relate the lateral deformation to the stability of the embankment.

4.6.2 Case with Normal Pace of Construction

This case studies the effect of PVD spacing and the extent of the area covered by the PVD"s on the stability of the embankment, for the particular problem considered in this paper. Since the focus is primarily on the spacing and extent of coverage of the PVD"s, this case does not include the situation with no drains.

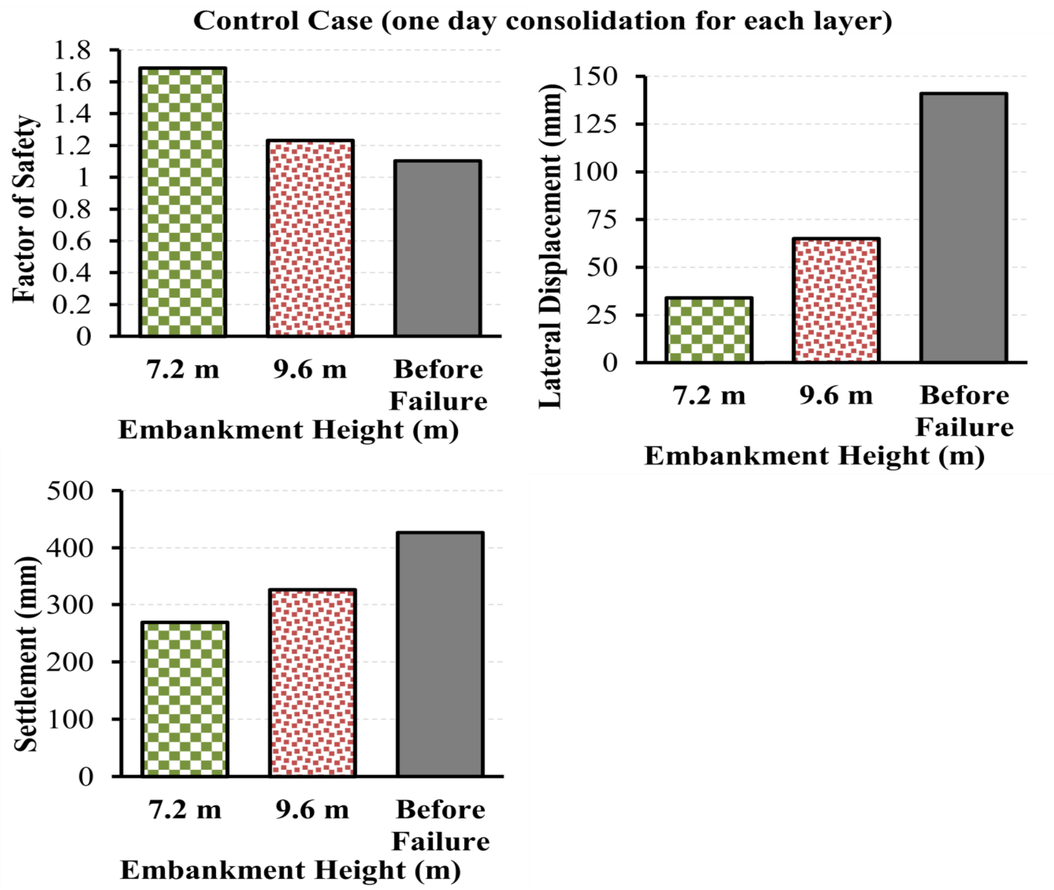


Figure 4.5: Factor of safety, lateral displacement, and settlement values in the control case, at embankment heights of 7.2 m, 9.6 m, and 12.3 m (immediately before failure)

The situation with no drains is considered in the following case. This case examines the effect of drain spacing on the stability of the embankment when the PVD coverage extends to the crest, to the toe, or beyond the toe of the embankment. The consolidation time following the placement of each fill layer is assumed to have duration of 1 day.

4.6.2.1 Factor of safety

Figure 4.6 shows that the factor of safety increases as the area covered by the drains increases, and decreases as the spacing between the drains increases. When the area covered by the drains extends to the crest of the embankment, with a drain spacing of 1 m there is a maximum FS of 1.897, and with a drain spacing of 2.5 m the embankment is found to fail at a height of 11.9 m. When the area covered by the drains extends to or beyond the toe of the embankment, with a drain spacing of 2.5 m the embankment is found to fail at a height of 12.8 m. When the area covered by the drains extends to the toe of the embankment, a maximum FS of 1.953 is found for a drain spacing of 1 m. Thus, extending the area covered by the drains beyond the toe of the embankment with a drain spacing of 1 m yields the highest factor of safety, but may not be the most economical solution. Because failure occurs with a drain spacing of 2.5 m, it can be seen that a drain spacing of 1.5 m is the most suitable for the problem under consideration.

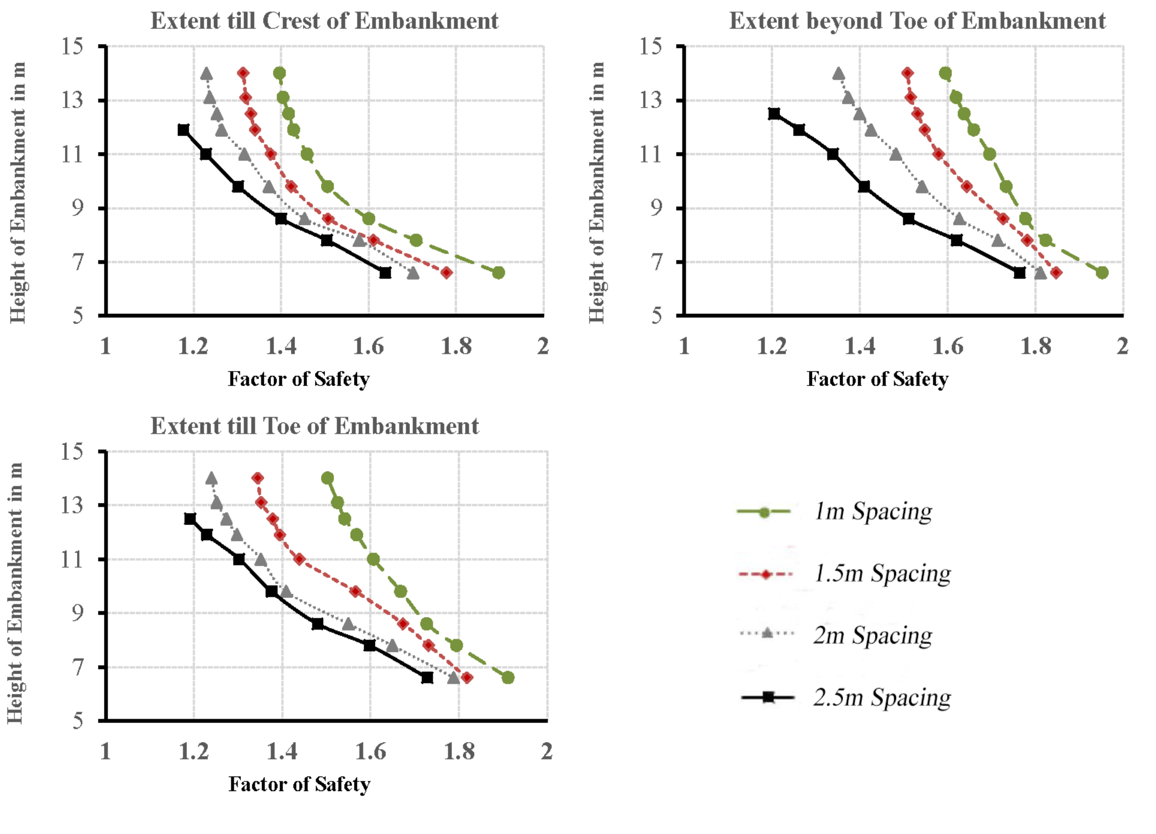


Figure 4.6: Factor of safety results for normal pace of construction, with 0.6 m fill layer thickness and PVD‟s extending to the crest, to the toe, and beyond the toe of the embankment

4.6.2.2 Settlement

Figure 4.7 shows that as the extent of the area covered by the drains increases, settlement within a given time increases, but not by a significant amount. When the area covered by the drains extends only to the crest of the embankment, settlement is less than when the drains extend beyond the toe of the embankment. This essentially means that final settlement occurs more quickly when the drains extend to or beyond the toe of the embankment than when the drains extend only to the crest. This is due to the fact that when the drains extend beyond the toe of the embankment, there is a faster rate of consolidation and hence greater settlement. It can be seen from Figure 4.7 that for a 40-day period, the settlement ranges from 800 mm for a drain spacing of 1 m, to 600 mm for a drain spacing of 2.5 m. It can also be seen that as the spacing of the drains increases, the rate of initial settlement decreases. This means that as the drain spacing increases from 1 m to 2.5 m, the dissipation of excess pore water pressure decreases and hence the settlement is reduced, causing the soil to lack shear strength, which eventually leads to failure. The models are also analysed for 95%, 98%, and 100% consolidation, as shown in the graphs on the right-hand side of Figure 4.7. It can be seen that when the area covered by the drains is expanded from the crest to the toe of the embankment, it takes less time to reach 95% consolidation, since many more drains are available to help dissipate the excess power water pressures and consolidate the soil. With a drain spacing of 2.5 m, failure occurs after 37 days, at a settlement value of 540 mm, when the PVD‟s extend only to the crest of the embankment. For the same drain spacing, with PVD‟s extending beyond the toe of the embankment, failure occurs after 45 days, at a settlement value of 640 mm.

4.6.2.3 Excess Pore Water Pressure

Since this case primarily focuses on studying the effect of PVD spacing on the stability of the embankment, Figures 4.8 and 4.9 compare the effect of different PVD"s spacing. Figure 4.8 shows excess pore water pressure distributions at an embankment height of 7 m (mid-height) and Figure 4.9 shows excess pore water pressure distributions immediately prior to failure. It can be clearly seen from the two figures that as the drain spacing increases, the dissipation of excess pore water pressure decreases. The slower dissipation of excess pore water pressures significantly reduces the shear strength of the soil, which increases effective stresses and eventually leads to failure. The maximum excess pore water pressure at mid-height is around 140 kPa and the maximum excess pore water pressure immediately prior to failure is around 260 kPa.

4.6.2.4 Lateral and Vertical Deformations

The graphs on the left-hand side of Figure 4.10 show that as the extent of the area covered by the PVD"s expands from the crest to beyond the toe of the embankment, the lateral displacement decreases. With a larger area covered by the PVD"s, the dissipation of excess pore water pressure increases, resulting in greater vertical deformation due to increased settlement, rather than lateral deformation. It can also be seen that as the drain spacing increases, the lateral deformation increases. This is due to a reduced dissipation of excess pore water pressures as the drain spacing is increased, leading to less settlement and more lateral deformation.

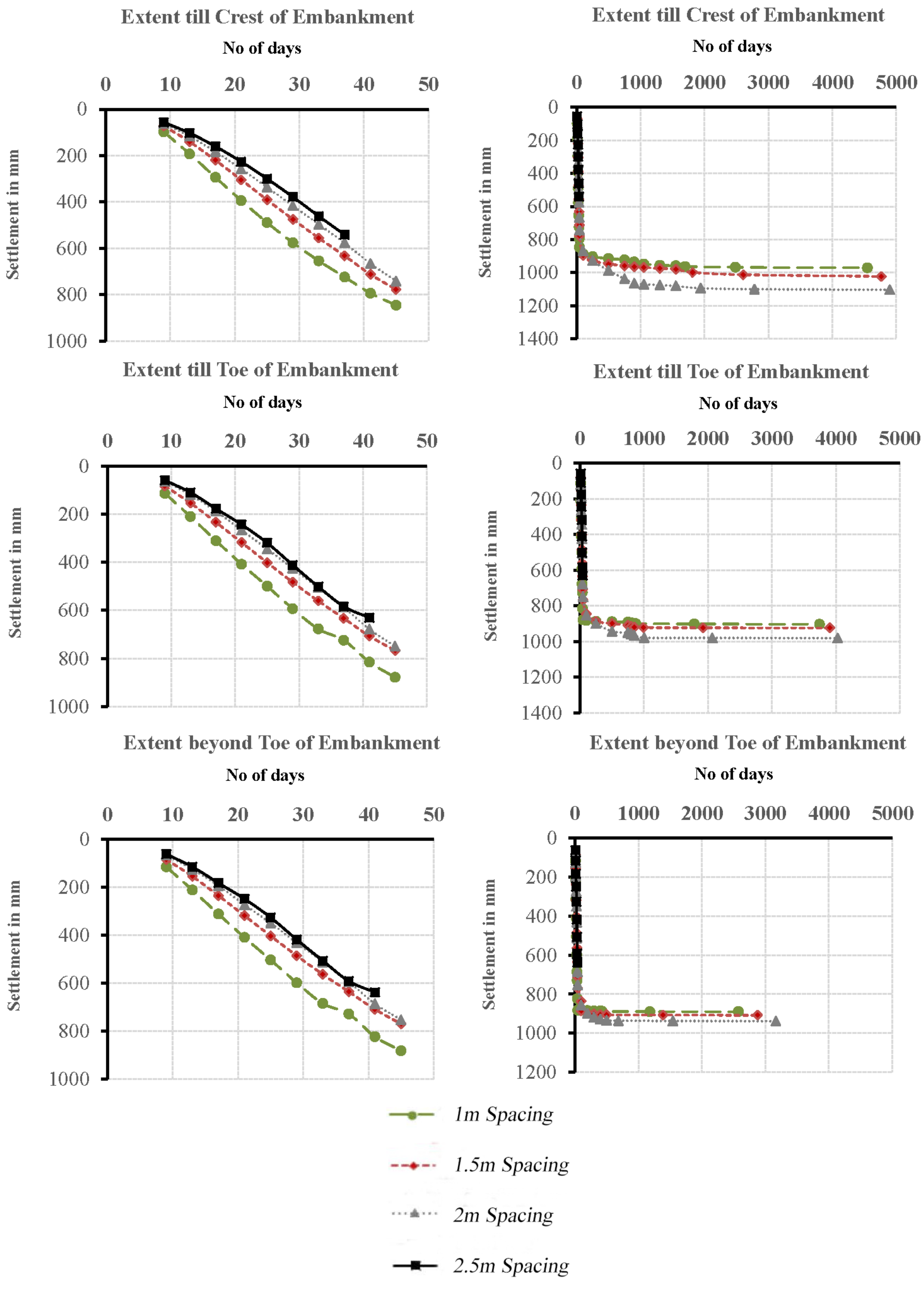


Figure 4.7: Settlement results for normal pace of construction, with 0.6 m fill layer thickness and PVD‟s extending to the crest, to the toe, and beyond the toe of the embankment

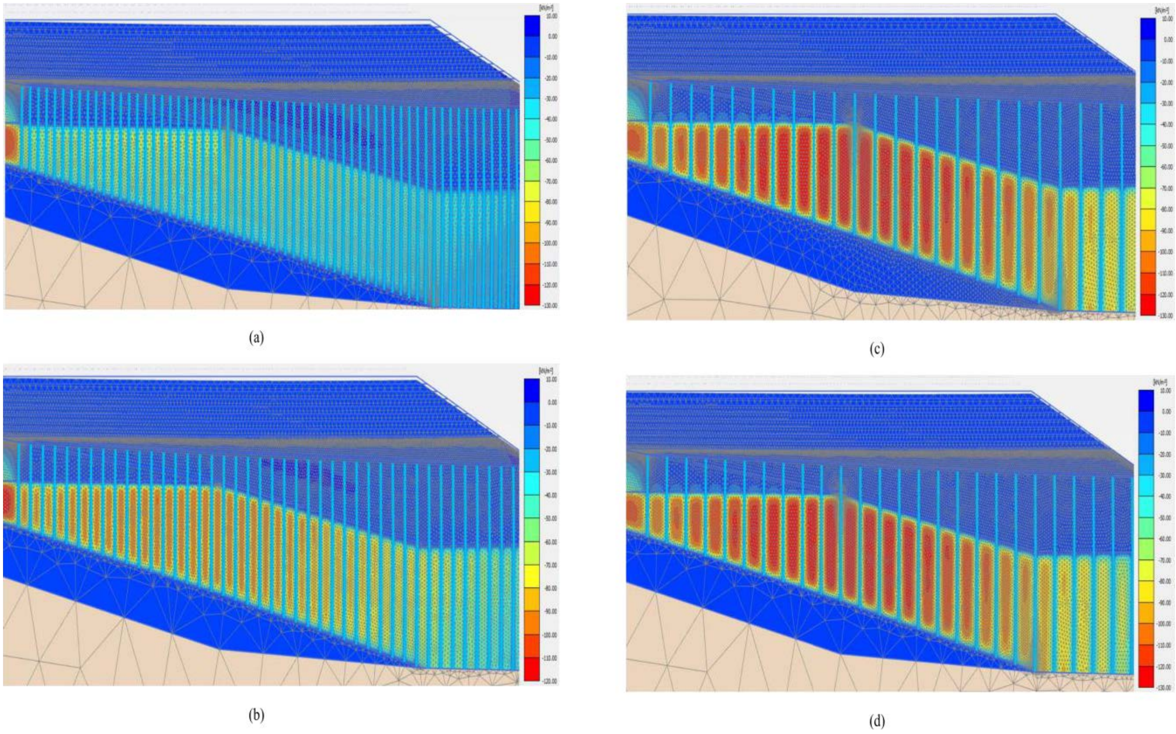


Figure 4.8: For an embankment height of 7 m, distribution of excess pore water pressures with drain spacing of (a) 1 m, (b) 1.5 m, (c) 2 m, and (d) 2.5 m

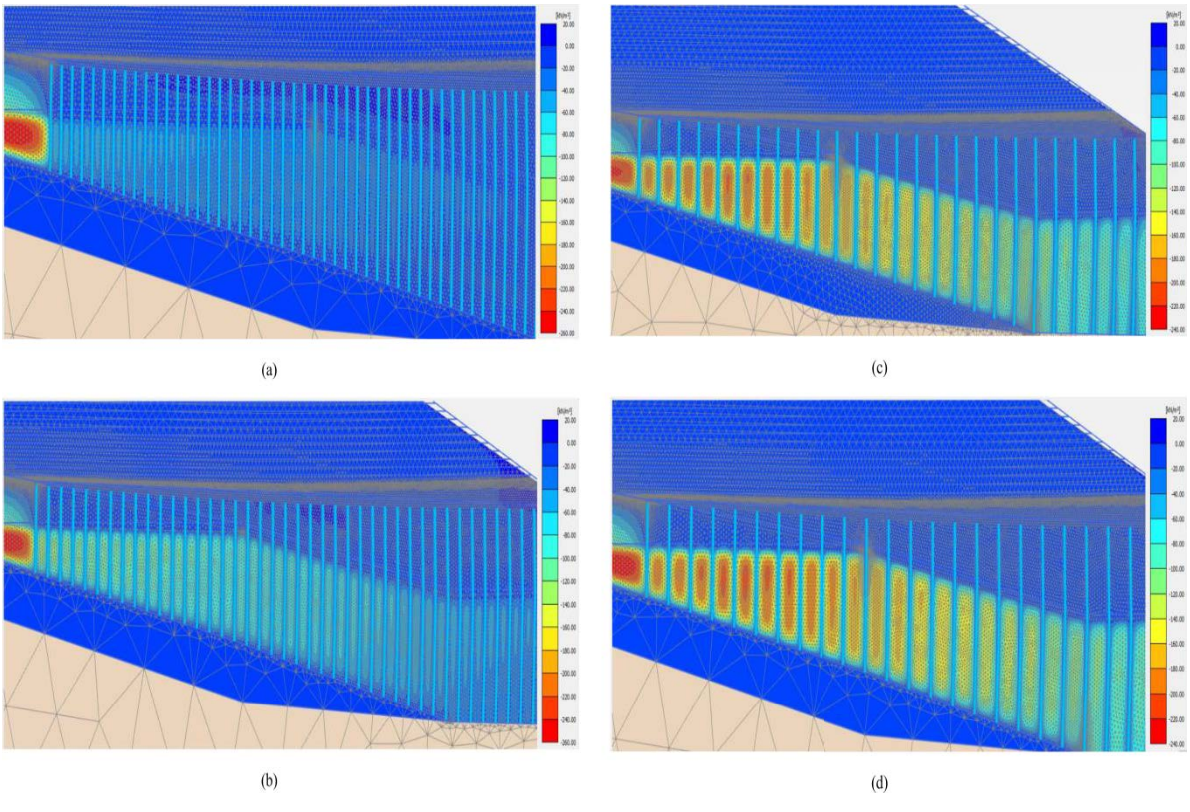


Figure 4.9: For embankment height immediately prior to failure, distribution of excess pore water pressures with drain spacing of (a) 1 m, (b) 1.5 m, (c) 2 m, and (d) 2.5 m

The graphs on the right-hand side of Figure 4.10 show that the vertical deformation increases as the extent of the area covered by the drains increases. However, there is no significant difference in the result when the PVD coverage is expanded from the toe to beyond the toe of the embankment. In the upper right-hand graph of Figure 4.10, it can be seen that when the PVD‟s extend only to the crest of the embankment, the vertical deformation is above ground level (GL), where the *X*-axis represents ground level. Vertical deformation above the ground level is due to heaving of the soil, resulting from loss of shear strength of the foundation soil. Thus, to strengthen the foundation soil, it is recommended that the drains should extend to the toe of the embankment, at least for the problem under consideration.

4.6.3 Case with Fast Pace of Construction

For the particular problem considered in this paper, this case studies the effect of the extent of the area covered by PVD‟s and the effect of thicker fill layers on the stability of the embankment. Since the focus is primarily on the extent of PVD coverage, this case includes a model with no drains, for the purpose of comparison. The extent of the area covered by PVD‟s is considered with a constant drain spacing of 1.5 m. This drain spacing was selected based on the results presented in section 4.6.1. In this case, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m are considered. The consolidation time following the placement of each fill layer is assumed to be 1 day.

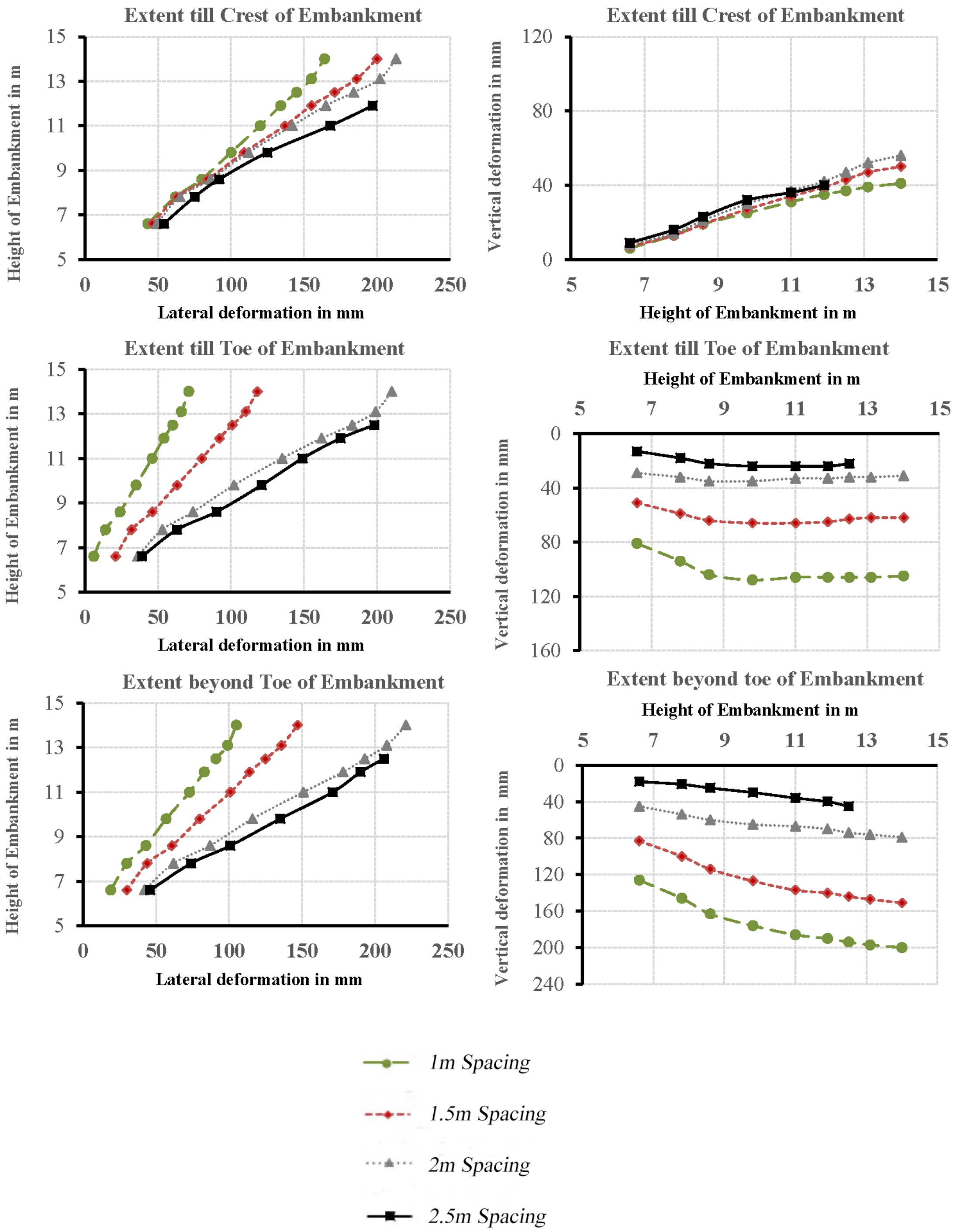


Figure 4.10: Lateral and vertical deformation for normal pace of construction, with 0.6 m fill layer thickness and PVD“s extending to the crest, to the toe, and beyond the toe of the embankment

4.6.3.1 Factor of safety

Figure 4.11 shows that as the extent of the area covered by drains increases, the factor of safety increases. However, there is no significant difference in the result when the PVD coverage is expanded from the toe to beyond the toe of the embankment. The FS decreases as the fill layer thickness increases, due to greater effective stresses acting on the soil over a shorter time when thicker fill layers are used. When drains extend beyond the toe of the embankment, there is a maximum factor of safety of 1.794 for a fill layer thickness of 2 m; however, with fill layer thicknesses of 3.5 m and 4.5, the embankment fails at heights of 10.5 m and 9 m, respectively. Thus, for the problem under consideration, a fill layer thickness of 2 m, with PVD"s extending to the toe of the embankment is considered the most suitable in terms of the safety and cost of the project. It can also be seen the use of drains results in significant improvement in the factor of safety. In the model with no PVD"s, failure occurs at embankment heights of 10.5 m and 9 m, for fill layer thicknesses of 2 m and 4.5 m, respectively.

4.6.3.2 Settlement

Figure 4.12 shows that as the extent of the area covered by the PVD"s increases, the rate of initial settlement increases. However, the settlement curves are similar when PVD"s extend to the toe of the embankment, and when they extend beyond the toe. A greater number of drains cause the settlement to increase, due to more rapid dissipation of excess pore water pressures. It can also be seen that as the fill layer thickness increases, the settlement increases, because of a more rapid rate of application of effective stresses. However, this sudden increase in settlement over a short period of time gives rise to excess pore water pressures, thus reducing the shear strength of the soil, which leads to failure of the foundation soil.

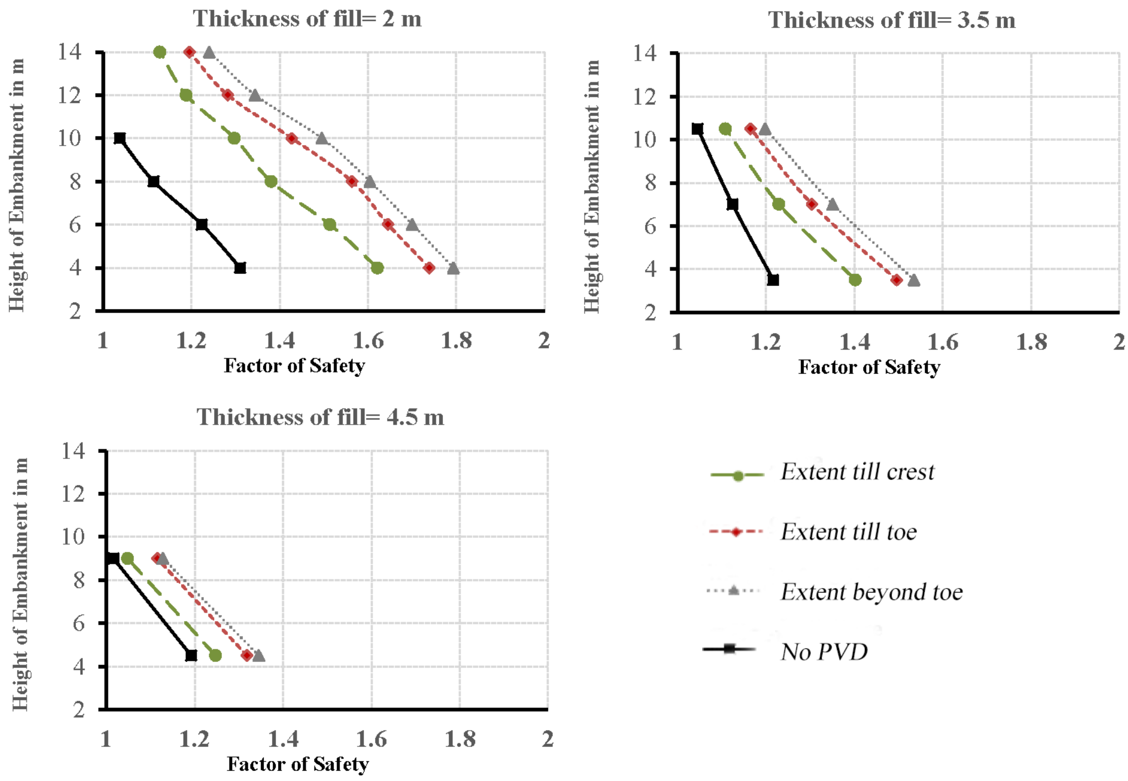


Figure 4.11: Factor of safety results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of 2 m, 3.5 m, and 4.5 m

For a fill layer thickness of 3.5 m, in days 8 to 9, the settlement ranges from 440 mm with PVD"s extending to the crest, to 580 mm with PVD"s extending to the toe of the embankment. With a fill layer thickness of 4.5 m, in days 6 to 7, the settlement ranges from 350 mm with PVD"s extending to the crest, to 470 mm with PVD"s extending to the toe of the embankment. For the model where no drains were used, the final settlement ranges from 230 mm with a fill layer thickness of 2 m, to 166 mm with a fill layer thickness of 4.5 m. The PVD"s thus help to dissipate excess pore water pressures, resulting in gradual settlement over time. The settlement results for 95%, 98%, and 100% consolidation are shown in the graph on the right-hand side of Figure 4.12.

It can be clearly seen that with a fill layer thickness of 2 m, 95% consolidation is achieved in the shortest time with PVD"s extending to the toe or beyond the toe of the embankment.

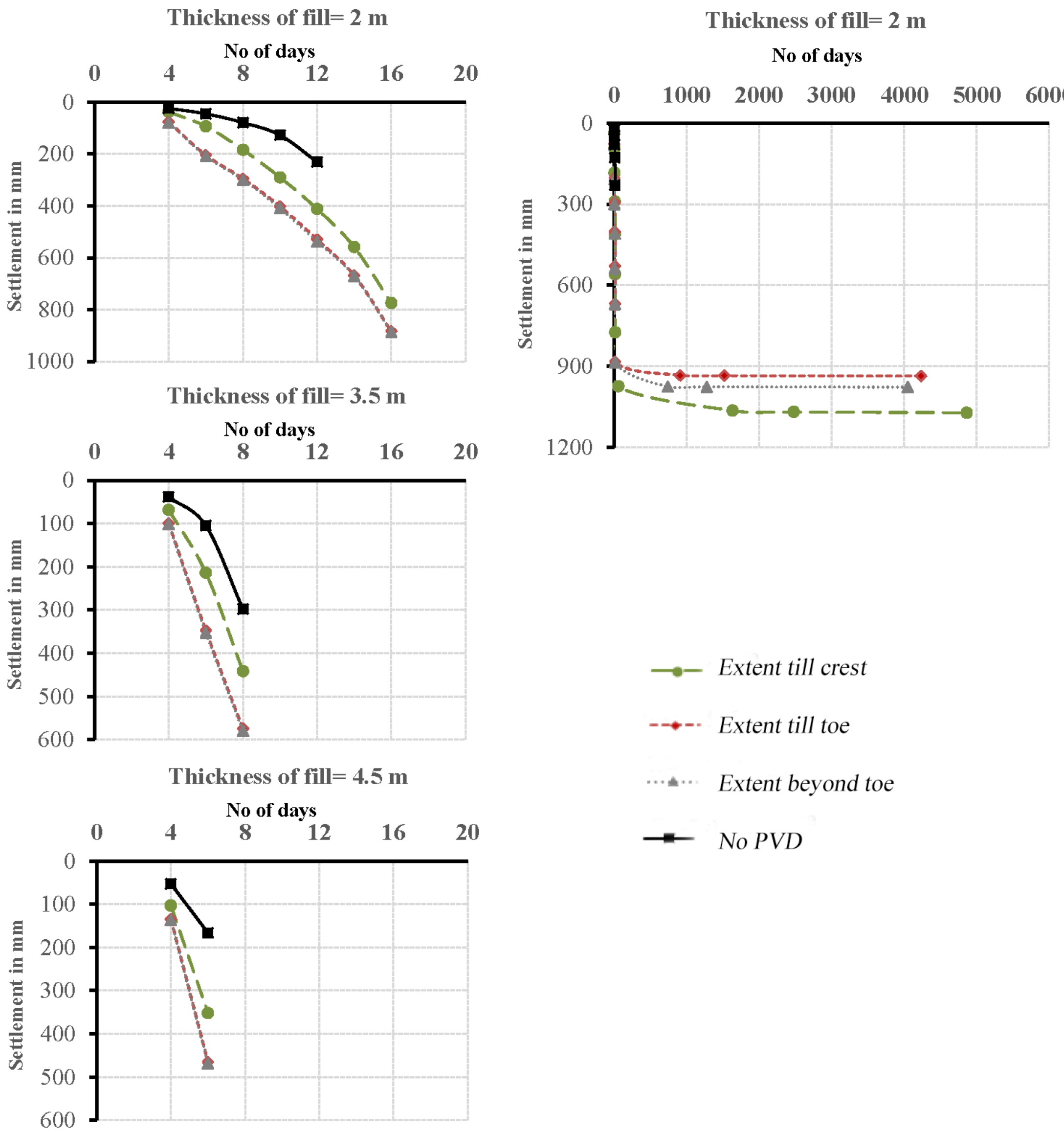


Figure 4.12: Settlement results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of 2 m, 3.5 m, and 4.5 m

4.6.3.3 Excess pore water pressure

Figures 4.13 and 4.14 show excess pore water pressure distributions for an embankment height of 7 m (mid-height) and the final height, respectively. A fill layer thickness of 2 m is used. The excess pore water pressures range from about 130 kPa at mid-height to 250 kPa at the final embankment height. It can be seen that the dissipation of excess pore water pressure increases as the extent of the area covered by PVD‟s increases. For the model with no PVD‟s, there is an accumulation of excess pore water pressures directly beneath the embankment, which significantly reduces the shear strength of the soil in that zone.

Figure 4.15 shows excess pore water pressure distributions for a fill layer thickness of 3.5 m. It can be seen that with thicker fill layers, the dissipation of excess pore water pressure is reduced, because thicker fill layers mean a shorter construction time, with less time for consolidation of the soil. Thus, with PVD‟s extending only to the crest of the embankment, failure occurs before the proposed embankment height is reached, with an effective stress of around 260 kPa. However, increasing the area covered by the PVD‟s results in greater dissipation of excess pore water pressures.

Figure 4.16 shows excess pore water pressure distributions for a fill layer thickness of 4.5 m. The results follow the same pattern as seen in Figure 4.15. However, the dissipation of excess pore water pressure is further reduced, and with PVD‟s extending only to the crest of the embankment, the embankment fails at a height even lower than is the case with a fill layer thickness of 3.5 m. As shown in Figures 4.15 and 4.16, with no PVD‟s, excess pore water pressures are concentrated beneath the embankment in the marine clay layer, thus significantly reducing the shear strength of this layer.

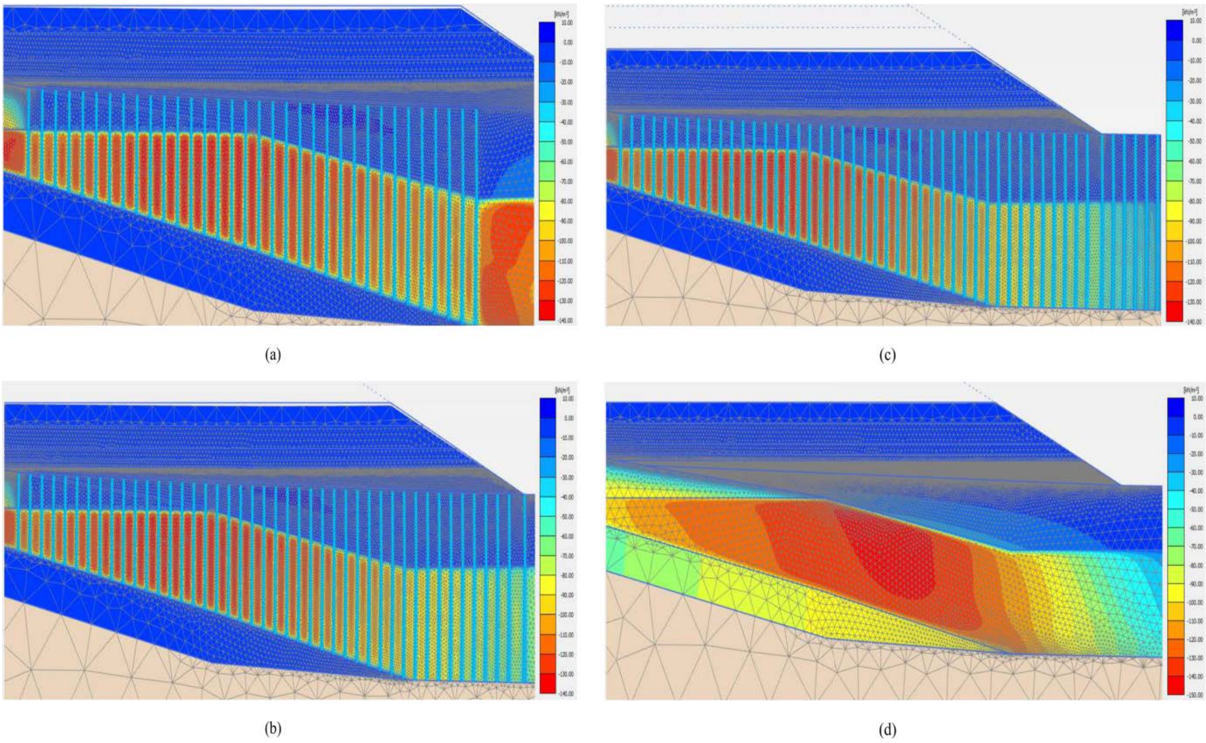


Figure 4.13: For an embankment height of 7 m and fill layers 2.0 m thick, distribution of excess pore water pressures with PVD‟s extending (a) to the crest, (b) to the toe, and (c) beyond the toe of the embankment; and (d) with no PVD‟s

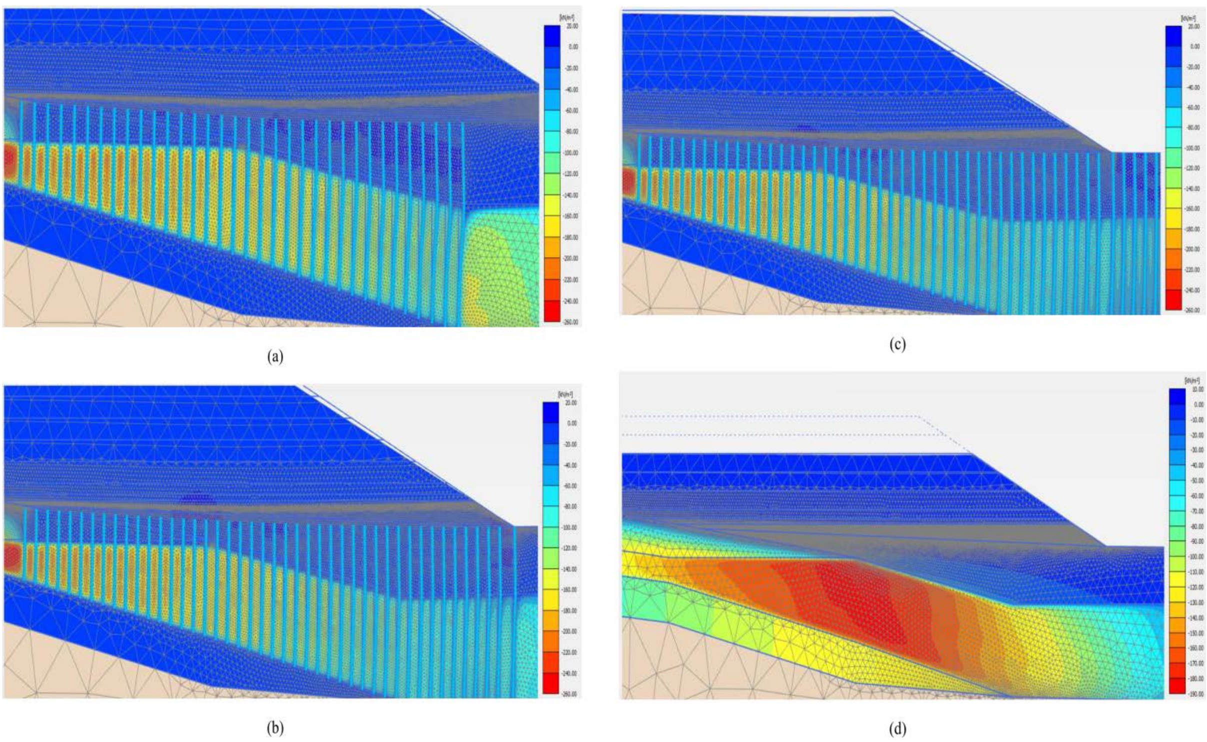


Figure 4.14: For embankment at final height and fill layers 2.0 m thick, distribution of excess pore water pressures with PVD‟s extending (a) to the crest, (b) to the toe, and (c) beyond the toe of the embankment; and (d) with no PVD‟s

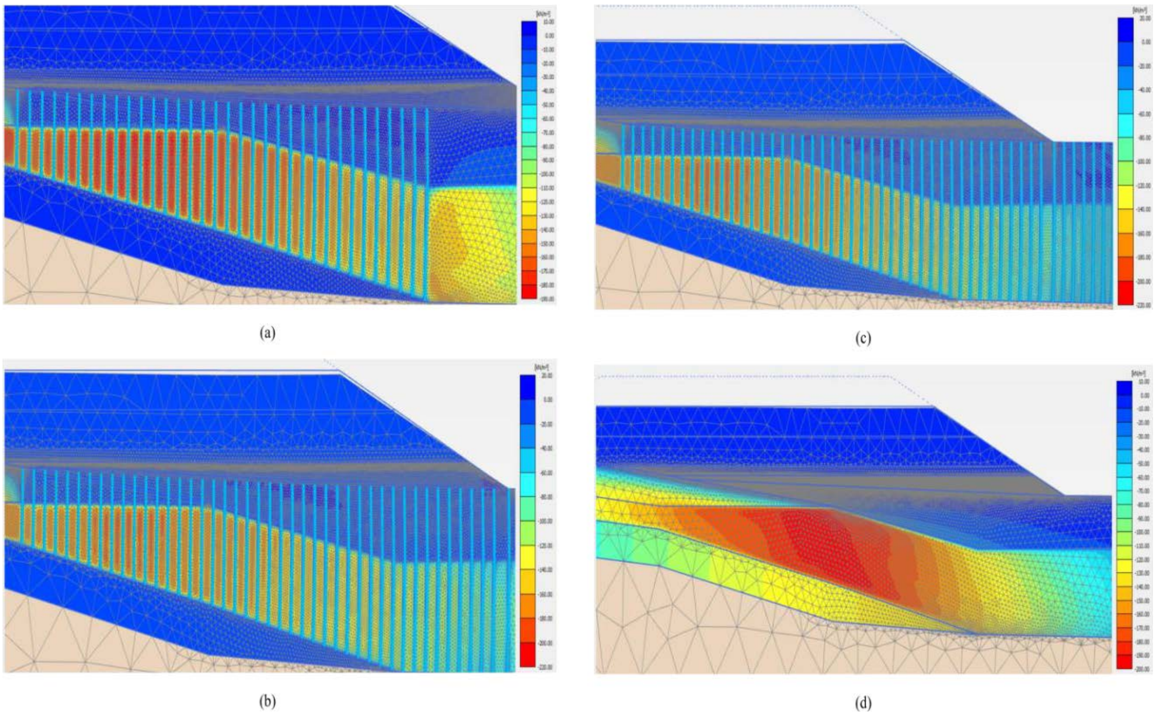


Figure 4.15: For fill layers 3.5 m thick, distribution of excess pore water pressures with PVD‟s extending (a) to the crest, (b) to the toe, and (c) beyond the toe of the embankment; and (d) with no PVD‟s

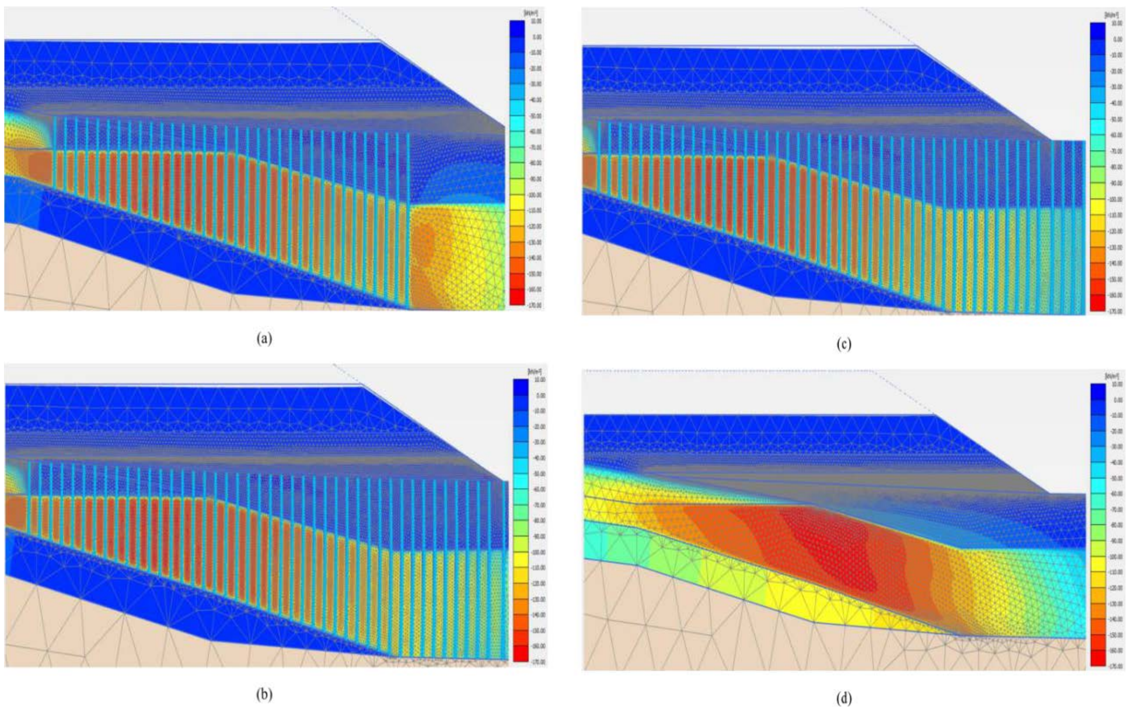


Figure 4.16: For fill layers 4.5 m thick, distribution of excess pore water pressure with PVD‟s extending (a) to the crest, (b) to the toe, and (c) beyond the toe of the embankment; and (d) with no PVD‟s

4.6.3.4 Lateral and vertical deformation

Figure 4.17 shows that the lateral deformation increases as the fill layer thickness increases. This is because there is less dissipation of excess pore water pressures when there is a significantly faster rate of application of effective stresses. This results in a slower rate of consolidation, leading to increased lateral deformation. It can be seen that for a given embankment height, the lateral deformation is greater for fill layer thicknesses of 3.5 m and 4.5 than for a fill layer thickness of 2 m. It should be noted that with fill layers 3.5 m and 4.5 m thick, the embankment fails at heights of 10.5 m and 9 m, respectively. It can also be seen that as the area covered by PVD‟s increases, the lateral deformation decreases. This is due to more rapid dissipation of excess pore water pressures, leading to a higher rate of consolidation, which results in less lateral deformation and greater vertical deformation.

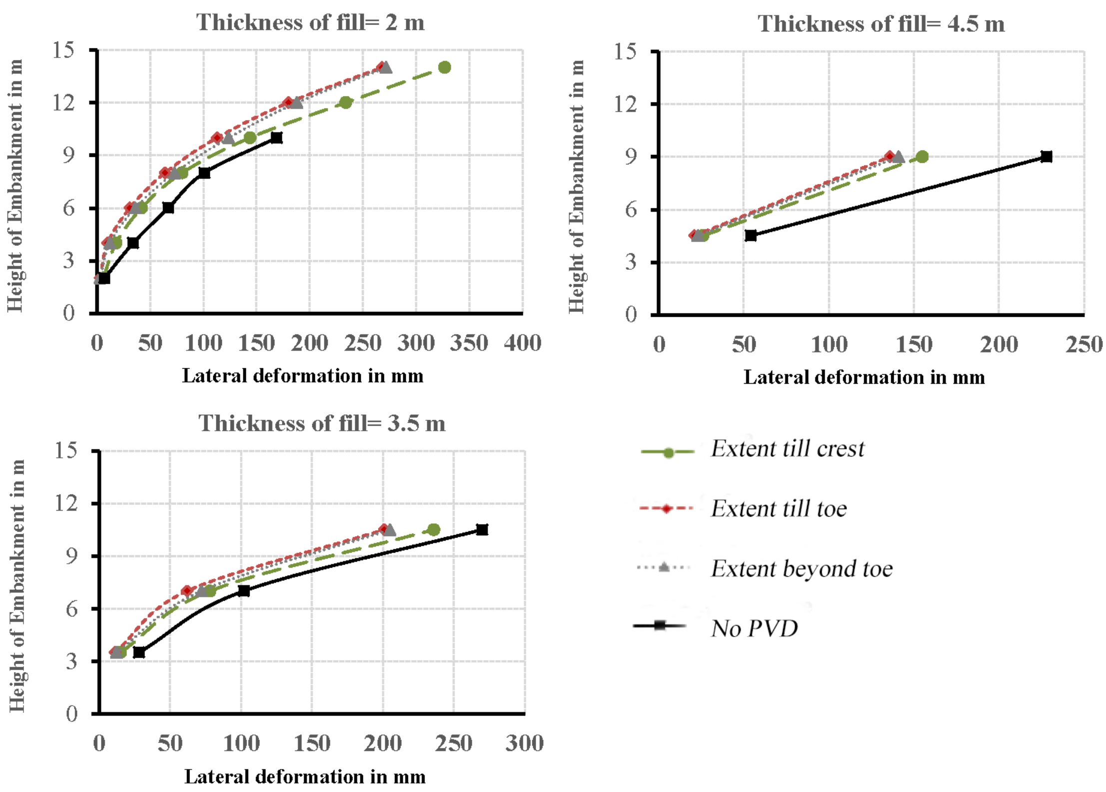


Figure 4.17: Lateral deformation results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of 2 m, 3.5 m, and 4.5 m

Figure 4.18 shows that as the fill layer thickness increases, the vertical deformation also increases. Here the *X*-axis represents the ground level. As shown on the left-hand side of Figure 18, when PVD‟s extend only to the crest of the embankment, or when there are no PVD‟s, there is heaving of the soil, with vertical deformations above the ground level. This is because the slow rate of consolidation results in a rapid loss of shear strength, causing the soil to heave above the ground level. As shown on the right-hand side of Figure 18, when PVD‟s extend to or beyond the toe of the embankment, there is an increased rate of consolidation, with the result that vertical deformations remain below the ground level. It can be seen that when a larger area is covered by PVD‟s, greater vertical deformation results, and due to increased consolidation.

4.6.4 Case with construction allowing more time for consolidation

For the particular problem considered in this paper, this case studies the effect of increasing the consolidation time and the effect of thicker fill layers on the stability of the embankment. Since the focus is primarily on the pace of construction, in this case the use of PVD‟s is kept constant, with PVD‟s extending to the toe of the embankment (based on the findings in section 4.6.3) and with a drain spacing of 1.5 m (based on the findings in section 4.6.2). In this case, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m are considered, and consolidation times of 5 days, 10 days, and 15 days are used following the placement of each fill layer.

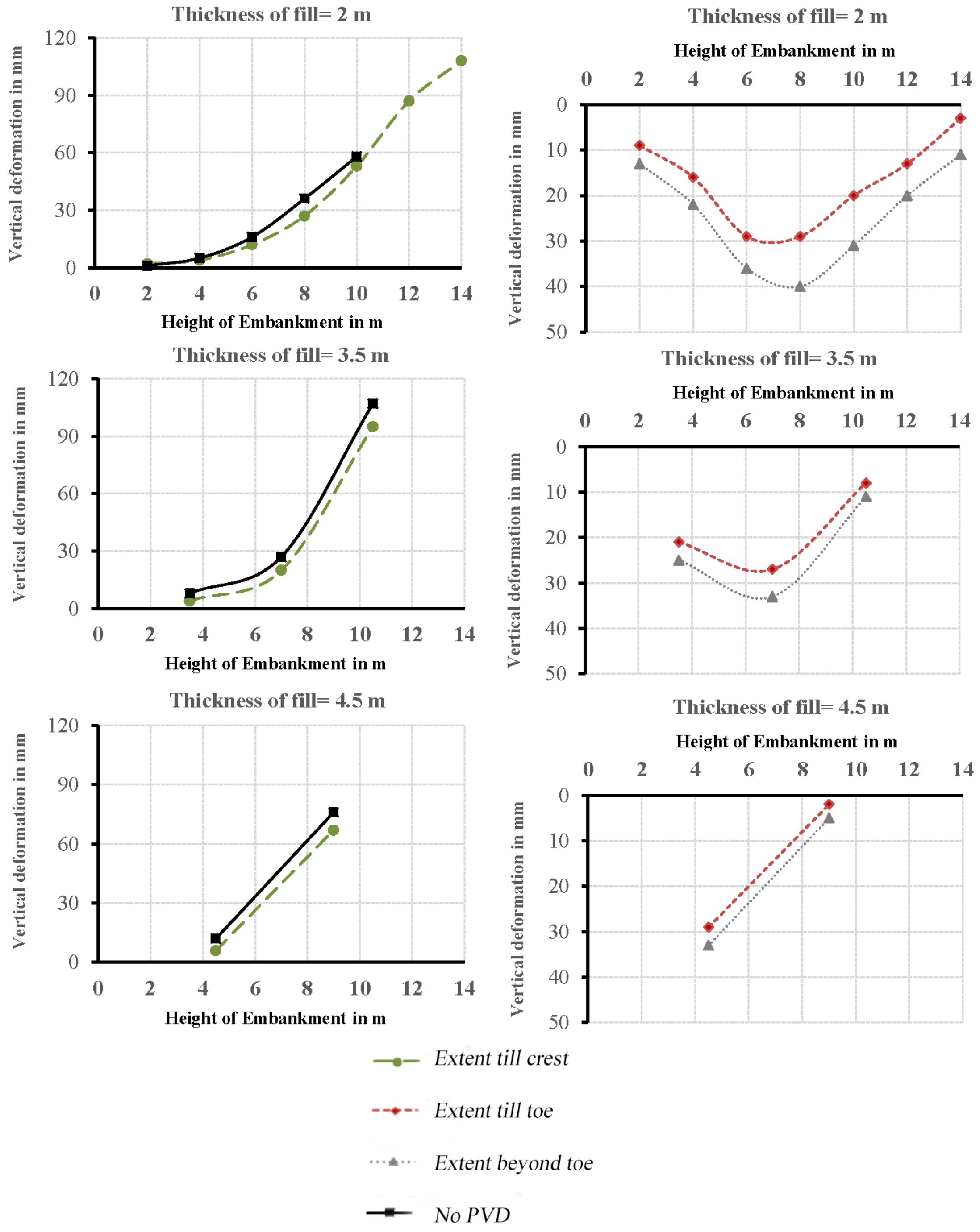


Figure 4.18: Vertical deformation results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of 2 m, 3.5 m, and 4.5 m

4.6.4.1 Factor of safety

Figure 4.19 shows that as the consolidation time increases, the factor of safety increases. This is because excess pore water pressures can dissipate rapidly due to the presence of drains, and the longer consolidation times permit increased consolidation. With greater consolidation, the soft soils gain strength due to the decrease in shear stresses, and the stability of the embankment therefore increases. As shown in Figure 4.19, at an embankment height of 14 m, the highest factor of safety obtained is 1.769, for fill layers 2 m thick, consolidated for 15 days after the placement of each layer; and the lowest factor of safety obtained is 1.442, for fill layers 4.5 m thick, consolidated for 5 days after the placement of each layer. Thus, the embankment stability increases significantly with an increase in consolidation time. However, when the costs and total duration of the construction are also taken into consideration, it may be concluded that consolidation periods of 5 to 10 days, with fill layer thicknesses of 2 m to 3 m, may provide the most suitable design solution for this particular problem.

4.6.4.2 Settlement

Figure 4.20 shows that as the consolidation time following the placement of each fill layer increases, the settlement becomes more gradual. The graph at the top left of Figure 4.12 shows that after 40 days, for fill layers with a thickness of 2 m, settlement is around 380 mm for consolidation times of 15 days, and is approximately 780 mm for consolidation times of 5 days. On the left-hand side of Figure 4.20, it can be seen that the rate of settlement increases sharply with an increase in fill layer thickness, resulting in a dramatic loss of shear strength in the foundation soil.

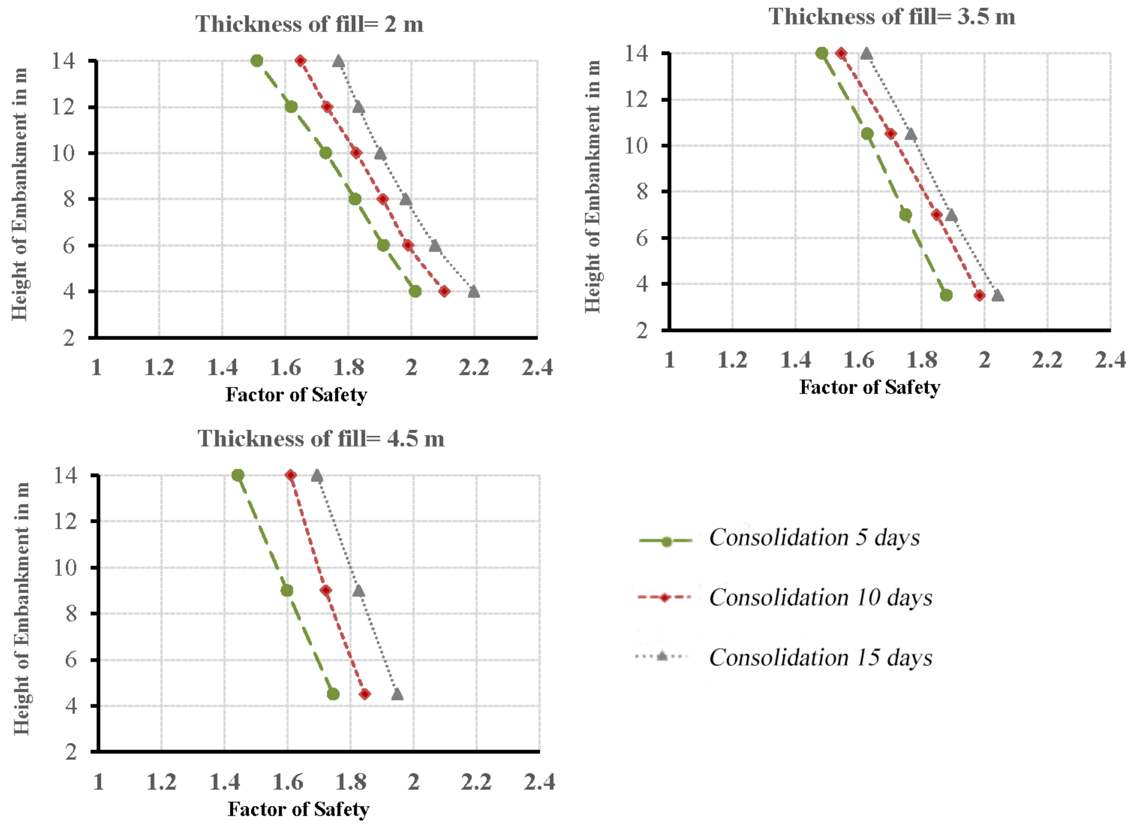


Figure 4.19: Factor of safety results with 5, 10, and 15 days allowed for consolidation, with 1.5 m drain spacing, PVD‟s extending to toe of embankment, and 2 m, 3.5 m, and 4.5 m fill layers

After 40 days, for consolidation times of 15 days, the settlement is around 380 mm for fill layers 2 m thick, around 610 mm for fill layers 3.5 m thick, and around 760 mm for fill layers 4.5 m thick. With longer consolidation times, the settlement becomes more gradual, and greater strength is gained by saturated soft soils such as marine clays. For the particular problem under consideration, consolidation times of 10 to 15 days are recommended.

On the right-hand side of Figure 4.20, it can be seen that with consolidation times of 15 days and a fill layer thickness of 2 m, 95% consolidation is achieved in around 1019 days, with a settlement of about 846 mm. In contrast, with consolidation times of 5 days and a fill layer thickness of 4.5 m, 95% consolidation is achieved in around 869 days, with a settlement of about 952 mm.

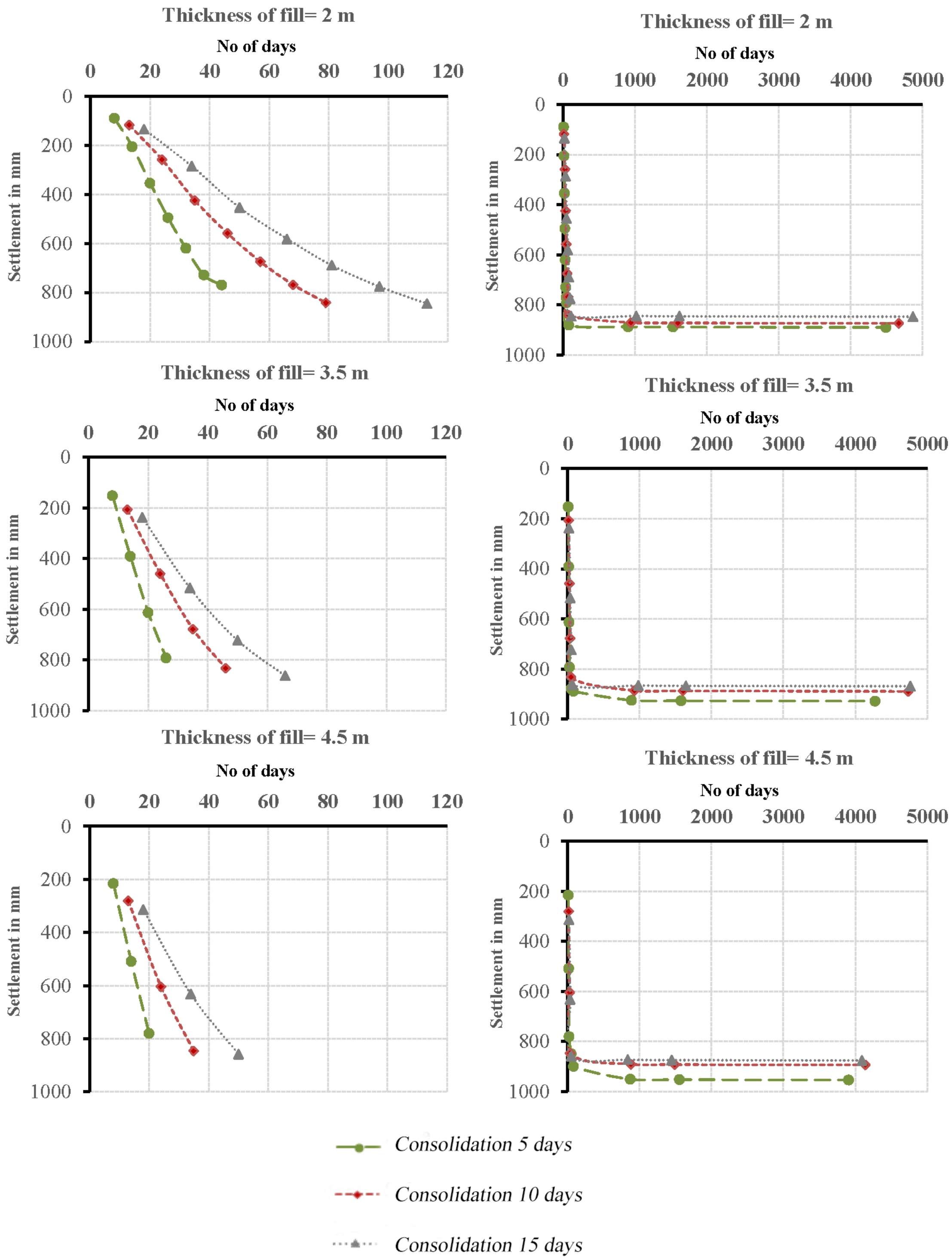


Figure 4.20: Settlement results with 5, 10, and 15 days allowed for consolidation, with 1.5 m drain spacing, PVD‟s extending to toe of embankment, and 2 m, 3.5 m, and 4.5 m fill layers

4.6.4.3 Excess pore water pressure

Figure 4.21 shows excess pore water pressure distributions for a fill layer thickness of 2 m. It can be seen that as the consolidation time increases, the dissipation of excess pore water pressures increases, due to greater consolidation. The excess pore water pressure ranges from 260 kPa for a consolidation period of 5 days to 110 kPa for a consolidation period of 15 days.

Figure 4.22 shows excess pore water pressure distributions for a fill layer thickness of 3.5 m, where the excess pore water pressure ranges from 260 kPa for a consolidation period of 5 days to 170 kPa for a consolidation period of 15 days. Figure 4.23 shows excess pore water pressure distributions for a fill layer thickness of 4.5 m, where the excess pore water pressure ranges from 260 kPa for a consolidation period of 5 days to 190 kPa for a consolidation period of 15 days. Thus, for a consolidation period of 15 days, upon completion of construction of the embankment, the excess pore water pressure ranges from around 110 kPa for fill layers 2 m thick to 190 kPa for fill layers 4.5 m thick, due to the more rapid increase in effective stresses with the addition of thicker fill layers.

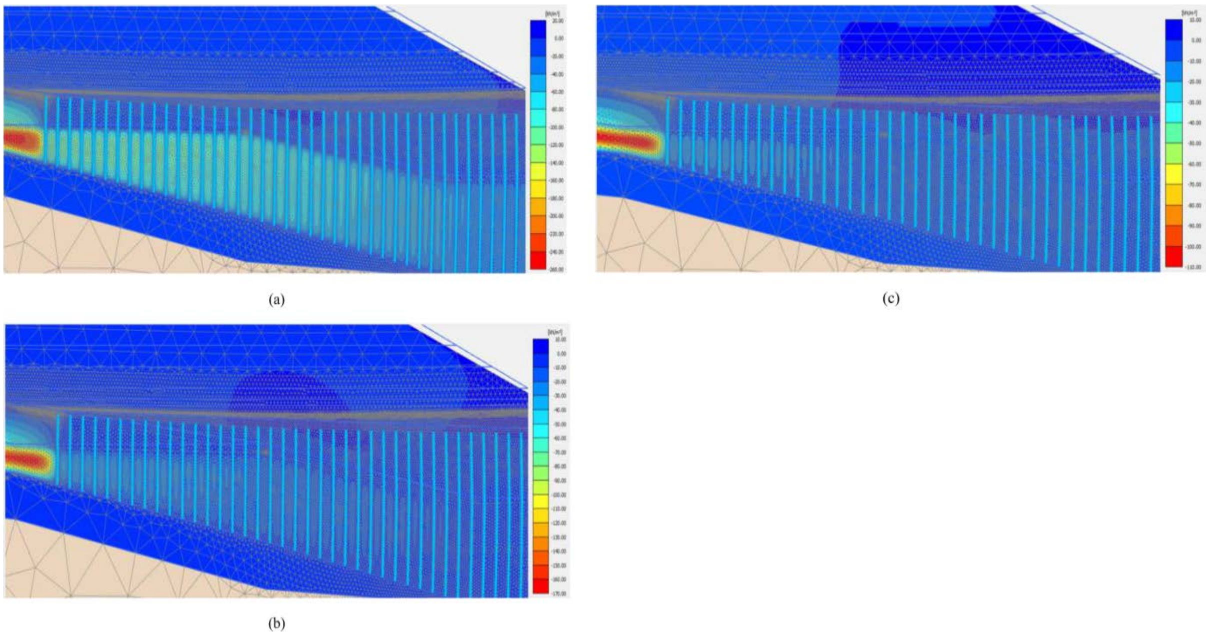


Figure 4.21: For a fill layer thickness of 2 m, distribution of excess pore water pressures with consolidation times of (a) 5 days, (b) 10 days, and (c) 15 days

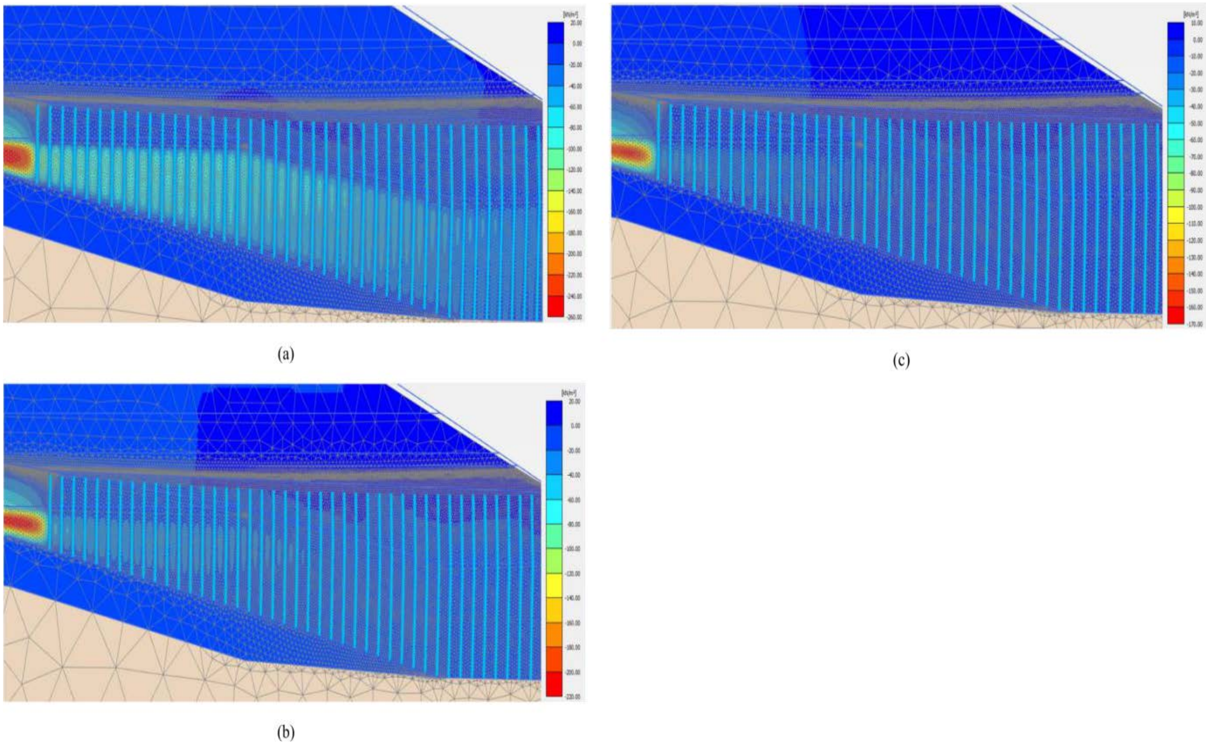


Figure 4.22: For a fill layer thickness of 3.5 m, distribution of excess pore water pressures with consolidation times of (a) 5 days, (b) 10 days, and (c) 15 days

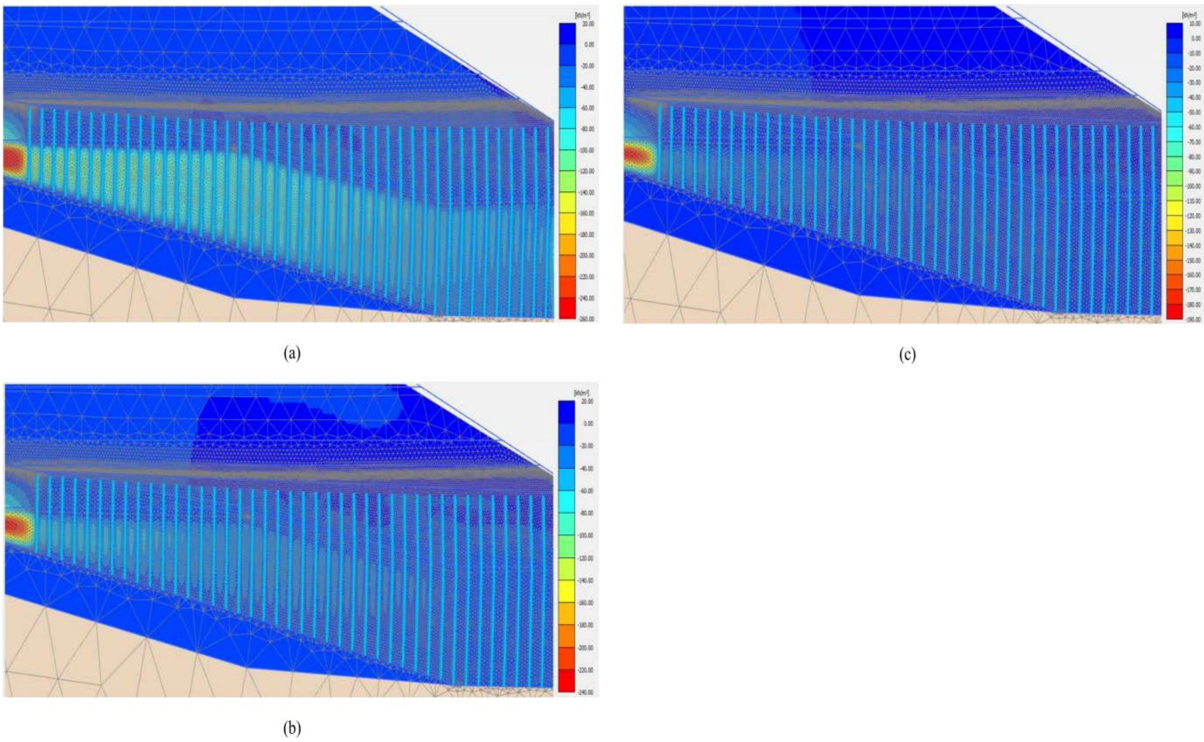


Figure 4.23: For a fill layer thickness of 4.5 m, distribution of excess pore water pressures with consolidation times of (a) 5 days, (b) 10 days, and (c) 15 days

4.6.4.4 Lateral and vertical deformation

The graphs on the left-hand side of Figure 4.24 show that as the fill layer thickness increases, the lateral deformation increases. However, as the time allowed for consolidation increases, the lateral deformation increases only slightly and hence the effect of the consolidation time on lateral deformation can be ignored. For a fill layer thickness of 2 m, the lateral deformation increases in a linear elastic manner up to an embankment height of 6 m, and then increases in a linear plastic manner up to an embankment height of 14 m. This is due to a change in the orientation of the soil particles, which gives rise to large lateral deformations, even with small changes in the effective pressure. For a fill layer thickness of 3.5 m, the lateral deformation increases in a linear elastic manner almost throughout, and for a fill layer thickness of 4.5 m the increase is completely linear elastic. This is because the embankment height of 14 m is reached in only 3 or 4 construction phases, for fill layers with a thickness of 4.5 m and 3.5 m, respectively. Thus, with thicker fill layers, there are fewer points on the graph to interpolate, which results in a more linear curve in the graphical representation.

The graphs on the right-hand side of Figure 4.24 show that vertical deformation increases as the consolidation time increases, and as the fill layer thickness increases. Here the *X*-axis represents the ground level. As the time allowed for consolidation increases, there is greater consolidation, resulting in more settlement, with the dissipation of excess pore water pressures. Thus, increased consolidation times result in increased settlement and vertical deformation. As the fill layer thickness increases, there is a sharp increase in the settlement, which reduces the shear strength of the soil. However, increased consolidation times allow the resulting excess pore water pressures to dissipate, and hence failure of the embankment does not occur even with greater fill layer thicknesses. Thus, it is evident that increasing the consolidation time significantly improves the stability of the embankment.

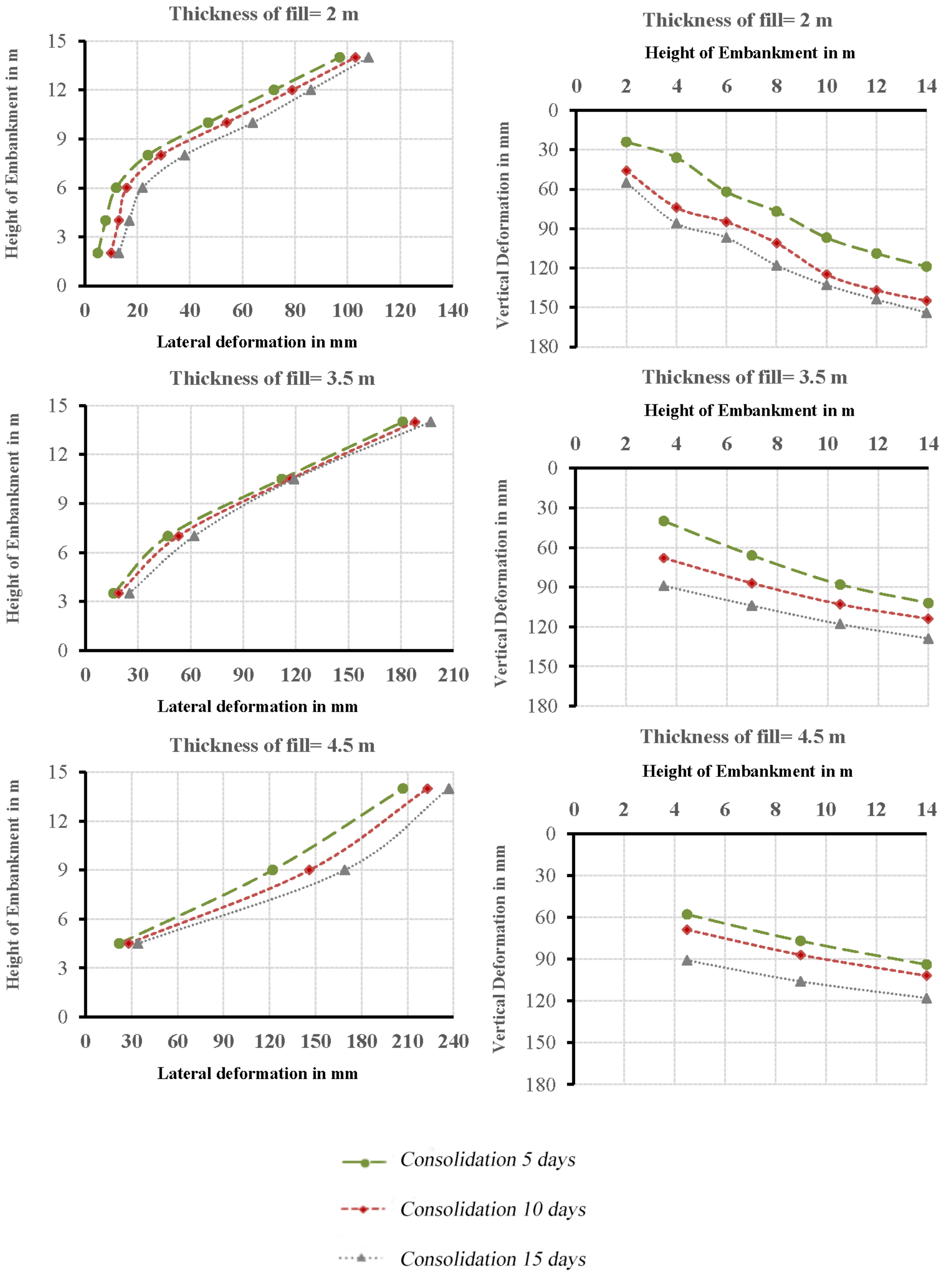


Figure 4.24: Lateral and vertical deformation results with 5, 10, and 15 days allowed for consolidation, with 1.5 m drain spacing and PVD"s extending to toe of embankment

4.7 Conclusion

From the parametric study presented in this paper, it can be concluded that:

- The use of ground improvement techniques such as prefabricated vertical drains improves embankment stability significantly, while reducing the duration of construction and project costs.
- Drain spacing is one of the key parameters that influence embankment stability. It could be concluded that a drain spacing of 1.5 m c/c provides a safe, economical solution that is the most suitable for the problem under consideration. A rough estimate of 1.5 m spacing could also be considered for various projects aiming to implement the use of drains for embankments founded on soft saturated soil deposits.
- Another important parameter apparent from this study is the extent of the area covered by the drains. The soil profile region where excess pore water pressures are concentrated must first be estimated or modelled without the use of drains. Based on this, the drains must be constructed so that they extend to the toe or beyond the toe of the embankment, depending on the stability and cost requirements of the project. For the problem under consideration, the extent of coverage found to be most suitable was for the drains to extend to the toe of the embankment.
- The pace of construction was studied by using thicker fill layers to reduce the construction time. It can be concluded that the embankment can be constructed by placing compact fill in layers 2 m to 2.5 m thick. This enables the construction to proceed at a faster pace but reduces the stability of the embankment. Thus, there is a trade-off between stability and reducing the construction time and cost of the project.
- The pace of construction was also studied by combining the use of drains with varying consolidation times. It was found that even with a fill layer thickness of 4.5 m, a consolidation time of 5 days following the placement of each fill layer resulted in

a relatively stable embankment, in comparison to the situation with a consolidation time of only 1 day, which caused the embankment to fail. Hence, allowing sufficient time for consolidation is one of the most important parameters influencing the pace of construction, in order to stabilise embankments founded on soft soil deposits. However, to minimise construction costs, using fill layers 2 to 2.5 m thick and allowing a consolidation time of 10 to 15 days following the placement of each fill layer would be the optimal design solution for the problem under consideration.

DATA AVAILABILITY

All data, models, and code generated or used during the study appear in the submitted article.

ACKNOWLEDGEMENTS

The authors acknowledge funding provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) to the second author for this research project.

4.8 References

1. Jose Leitao Borges (2004) “Three-dimensional analysis of embankments on soft soils incorporating vertical drains by finite element method” Computers and Geotechnics 31 (2004) 665–676.
2. Muktabhant, C., Teerawong, P., and Tengamnuay, V., “Engineering properties of Bangkok subsoils”, Proc. of Southeast Asian Conference on Soil Engineering, Bangkok, 1, 1967.
3. Moh, Z. C., Nelson, J. D., and Brand, E. W., “Strength and deformation behaviour of Bangkok clay”, Proc. of 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico, 1, 1969, pp287-295.
4. Eide, O., “Geotechnical engineering problems with soft Bangkok clay on the Nakhon Sawan highway project”, Pub. No 78, Norwegian Geotechnical Institute, Norway, 1968.
5. Eide, O., “Exploration, sampling and in-situ testing of soft clay in the Bangkok area”, Proc. of International Symposium on Soft Clay, Thailand, 1977, pp122-137.
6. Bergado, D.T., Anderson, L.R., Miura, N., and Balasubramaniam, AS, "Soft Ground Improvement", American Society of Civil Engineers, 1996, 427 pages
7. Lei Ma, Shui-Long Shen , Chun-Yong Luo & Ye-Shuang Xu (2011) “Field Evaluation on the Strength Increase of Marine Clay under Staged Construction of Embankment” , Marine Georesources & Geotechnology, 29:4, 317-332.

8. B. Indraratna, I. Sathananthan, C. Rujikiatkamjorn and A. S. Balasubramaniam (2005) "Analytical and Numerical Modeling of Soft Soil Stabilized by Prefabricated Vertical Drains Incorporating Vacuum Preloading" International Journal of Gemechnics, Vol 5, No.2, ISSN 1532-3641/2005/2-114–124.
9. Sathananthan Iyathurai (2005) "Modelling of vertical drains with smear installed in soft clay", Doctor of Philosophy thesis, School of Civil, Mining and Environmental Engineering, University of Wollongong, 2005. https://ro.uow.edu.au/theses/1924
10. Buddhima Indraratna, Rui Zhong, and Cholachat Rujikiatkamjorn (2016) "An Analytical Model of PVD-assisted Soft Ground Consolidation" Advances in Transportation Geotechnics 3, Volume 143 2016, Pages 1376-1383. doi: 10.1016/j.proeng.2016.06.162.
11. Indraratna, B, Bamunawita, C, Redana, I, and McIntosh, G (2003) "Modelling of prefabricated vertical drains in soft clay and evaluation of their effectiveness in practice". http://ro.uow.edu.au/engpapers/394
12. Lin, D. G. & Chang, K. T. (2009) "Three-dimensional numerical modelling of soft ground improved by prefabricated vertical drains" Geosynthetics International, 16, No. 5, 339–353. doi: 10.1680/gein.2009.16.5.339.
13. Shui-Long Shen, Jin-Chun Chai, Zhen-Shun Hong, Feng-Xi Cai (2005) "Analysis of field performance of embankments on soft clay deposit with and without PVD improvement" Geotextiles and Geomembranes 23 (2005) 463–485.
14. Bernie Mills and Jared McGinn (2010) "Design, Construction, and Performance of a Highway Embankment Failure Repaired with Tire-Derived Aggregate" Transportation Research Record: Journal of the Transportation Research Board, No. 2170, Transportation Research Board of the National Academies, Washington, DC, 2010, pp. 90–99.
15. Y.K. Wong, Joseph (2013) "Comparison of Drainage Line Elements in PLAXIS 2D and 3D applied in Consolidating Marine Clay Deposits" SEIC 2013, IP 6-63.
16. Bernie Mills, Hany El Naggar, Arun Valsangkar (2015) "North American Overview and a Canadian Perspective on the Use of Tire-Derived Aggregate in Highway Embankment Construction" Chapter 22, Ground Improvement Case Histories, Volume 2, Elsevier Ltd.
17. Manual on estimating soil properties for foundation design (1990), prepared by Cornell University, Ithaca, New York, EPRI EL-6800.
18. Zhongkun Ouyang and Paul W. Mayne (2018) "Effective friction angle of clays and silts from piezocone penetration test" Can. Geotech. J. 55: 1230–1247 (2018)
19. Myint Win Bo, Arul Arulrajah, Patimapon Sukmak, Suksun Horpibulsuk (2014) "Mineralogy and geotechnical properties of Singapore marine clay at Changi" Japanese Geotechnical Society, Soils and Foundations 2015; 55 (3):600–613, Elsevier.
20. Maryam Gaber, Anuar Kasa, Norinah Abdul-Rahman, Jamal Alsharef (2019) "Simulation of Sequential Construction of Embankments on Reinforced Soft Clay Foundation" Indian Geotech J (April 2019) 49(2):224–231.
21. PLAXIS material models 2019
22. B. G. Sexton, B. A. McCabe (2013) "Numerical modelling of the improvements to primary and creep settlements offered by granular columns" Acta Geotechnica (2013) 8:447–464.

CHAPTER 5 SUMMARY AND CONCLUSIONS

5.1 Summary

In this thesis, a brief background about the distribution of Marine clays in Canada is provided along with important Geotechnical characteristics of Marine clays. This thesis focuses on the strength gain properties of Marine clays which form the basis in design of structures where the foundation soils are predominantly soft. Construction of embankments on Marine clay soils deposited around the Bay of Fundy region in St. Stephens, NB is studied in detail which consists of replicating construction of the embankment and the failure observed at the site. Furthermore, an attempt at providing safe and economical design of High Embankments founded on soft soils is done.

Staged construction of embankments is a technique which is followed when the construction is carried out on very soft soils like Marine clay which have very high natural water content. Following the calibration of the numerical model, the effects of staged construction and the construction pace on the stability of such embankments are studied in detail which is covered Chapter 3. The effects on the use of Pre-fabricated Vertical Drains (PVD"s) which are primarily used to accelerate the rate of consolidation are then studied in detail in Chapter 4. This thesis attempts to focus on the parametric studies to give reasonable design considerations for high embankments on soft soils and hence the design of drains is not provided, rather the Line element as a drain feature from PLAXIS 2D is implemented.

5.2 Conclusions

The most parameters which are considered for the design of embankments in this thesis include Factor of safety (FS), Settlements, % increase in excess pore pressures, excess pore pressure distribution in the numerical models, Lateral & Vertical deformations of the toe of the embankments. Conclusions from the two technical papers presented are as follows:

- Increasing the consolidation period after each construction stage increases the stability of the embankment and constructing at a rapid pace decreases the stability of embankments as there is an increase in the excess pore pressures which eventually leads to a loss in Shear strength of soft foundation soils.
- The parametric study carried out shows that the FS decreases with the increase in the thickness of the fills and the increase in the construction pace of the embankments. However, the FS increases with the increase in time allotted for the consolidation period which translates to increase in the construction time, allowing for sufficient consolidation. The FS also increases when the fills are placed in smaller layers, but this constraints the economical design of embankments. Thus, ground improvements like the use of PVD‟s are used to accelerate the rate of consolidation, so thicker fills can be placed to reduce the overall construction time.
- With the use of PVD‟s it can be concluded that the drains significantly increase the Factor of safety of the embankments. From the parametric study carried out, it is seen that the FS reduces with an increase in the c/c spacing of the PVD‟s and conversely, the FS increases as the spacing reduces. However, it is seen that an ideal spacing of the drains is around 1m to 1.5m c/c for the considered problem. For the area covered by PVD‟s, the FS increased when the area covered by the drains increased beyond the toe of the embankment. From an economical perspective, it can be concluded that an

ideal coverage of area for the drains would be from the region with most excess pore pressures till the toe of the embankment.

- Settlement, being one of the most important factors influencing the design of embankments, was studied in detail in both the technical papers. It was seen that the primary settlements increases with the increase in the construction pace and thicker fills being placed. The increase in primary settlements is due to the self-weight of the embankment applied without much consolidation. This leads to a rapid accumulation of excess pore pressures, thus resulting in the failure of the embankments. It can also be observed that the settlements increased with the increase in consolidation period because of the dissipation of excess pore pressures and the self-weight of the embankment. Settlements associated with the dissipation of excess pore pressures leads to a gain in strength of the soft foundation soils and a faster consolidation rate means, maximum settlements are achieved over a short period of time.
- Hence, PVD‟s are used for accelerating the rate of consolidation. This leads to reduced long term settlements and at the same time increasing the shear strength of the foundation soils. It is seen that as the spacing of PVD‟s increases, the settlement decreases as there is a reduction in the rate of consolidation. Hence, lesser spacing leads to reduced long term settlements and an early strength gain of the soft foundation soils. It can also be observed that with an increase in the area covered by PVD‟s, the settlement increases. This is because of the increased rate of consolidation and hence an increase in the dissipation of excess pore pressures which leads to a gain in strength and reducing the long term settlements.
- The % increase in excess pore pressure is also considered to be important as this parameter underlines the importance of increasing the time allotted for consolidation of foundation soft soils. It can be seen that the percentage of excess pore pressure

increases with the increase in construction time and the thickness of the fills. This is because of the rapid accumulation of excess pore pressures and an increase in the percent excess pore pressure translates to a gradual loss in shear strength, eventually leading to failure of the soft soils. It can be observed that as the duration for consolidation increases, the percent of accumulated excess pore pressure reduces as a result of dissipation of excess pore pressures and a consequent gain in shear strength with an increased rate of consolidation.

- When the ground is improved by using PVD"s to accelerate the rate of consolidation, the accumulated excess pore pressures underneath the embankment reduces significantly. The drains act as paths for the excess pore pressures to be drained out of the foundation soil through the base layer of the embankment. It can be seen that as the spacing of the PVD"s decreases, there is a decrease in the excess pore pressures as a result of an increase in the rate of consolidation. When the area covered by the drains increases, the excess pore pressures are reduced because of an increased dissipation of excess pore pressures. Thus, it can be concluded that the PVD"s can be used to increase the rate of consolidation and increase the global stability of the embankment significantly by increasing the shear strength of the foundation soils.
- Lateral displacements of the toe of the embankment was seen to increase with the increase in the thickness of the fills as well and a rapid construction pace. This is because of the increased primary settlements and a reduced rate of consolidation and hence a consequent loss in the shear strength of soft soils which eventually leads to heaving of foundation soils. It can also be seen that as the duration for consolidation period increases, the Lateral displacements decrease because of the increase in shear strength of the foundation soils as a result of an increased rate of consolidation.

- The vertical displacements were found to be above the ground level as there were thicker fills in place and when the construction pace was increased and hence heaving of the foundation soils is observed. The displacements were predominantly below the ground when the consolidation time increased and thus reducing the heaving of foundation soils due to an increase in its shear strength.
- When PVD‟s are used as a ground improvement technique, it reduces the heaving of the foundation soft soils significantly due to the increased shear strength provided by the rapid dissipation of excess pore pressures. It can be observed that as the spacing of the drains increases, the Lateral deformations increase as a result of reduced rate of consolidation and hence a gradual loss of shear strength. The Lateral deformations decrease with the extent in the spread of the drains because of the increased rate of consolidation and conversely an increase in the shear strength.
- When PVD‟s are used, the Vertical deformations increase with the increase in spacing of the drains and there is heaving observed only when the area covered by the drains is till the crest of the embankment for the problem considered in Chapter 4. When the area covered by the drains is till the toe or beyond the toe, it can be seen that the Vertical deformations are below the ground. This is because of the increased rate of consolidation with the area of the drains covered beyond the toe, and hence a consequent gain in shear strength and foundation soils do not experience heaving with increased rate of consolidation.

CHAPTER 6 REFERENCES

1. Quigley, R. M. (1980) “Geology, mineralogy, and geochemistry of Canadian soft soils: a geotechnical perspective” Canadian Geotechnical Journal 17:261-285.
2. Ann G. Harris, Esther Tuttle and Sherwood D. Tuttle. 1997. Geology of National Parks: Fifth Edition, Iowa, Kendall/Hunt Publishing ISBN 0-7872-5353-7
3. Andrews, J. T. (1972) “The National Atlas of Canada. Canada Department of Energy, Mines and Resources”
4. Athir Nader (2014) “Engineering Characteristics of Sensitive Marine Clays - Examples of Clays in Eastern Canada” MASc thesis report, University of Ottawa
5. Quigley R.M., T. (1976) “Till geology, mineralogy and geotechnical behaviour, Sarnia, Ontario. Royal Society of Canada Special Publication” 12:336-345.
6. Weir, Wilbert Walter, 1882-. Philadelphia London, J. B. Lippincott company
7. Dreimanis, A. (1976) “Tills: Their origin and properties. Royal society of Canada Special Publication” 12:11-49.
8. Morgenstem, N. (1967) “Submarine slumping and the initiation of turbidity current” University of Illinois Press 189-220.
9. Leroueil, S. (1999) “Geotechnical characteristics of eastern Canada clays. Characteristics of Soft Marine Clays”
10. Locat, E. D. (1996) “On the development of microstructure in Genesis and Properties of Collapsible soils “93-128.
11. Torrance. (1988) “Mineralogy, pore-water chemistry, and. In The late Quaternary Development” 35:259-275.
12. Berry, R. W. (1988) “Mineralogy, grain size distribution and geotechnical behavior. The Canadian Mineralogist” 36:1625-1636.
13. Penner, E. (1965) “A study of the sensitivity of Leda clay. Canadian Journal of Earth Sciences 2.
14. Basack, S and Purkayastha, R. (2009) “Engineering Properties of Marine Clay from the Eastern Coast of India”, Journal of Engineering and Technology Research, 109-114.
15. N. R. Ahmad , Indra S. and H. Harahap (2016) “The Compression Behaviour of Marine Clays in Malaysia” International Conference on Ocean, Offshore and Arctic Engineering, Volume 1, ISBN: 978-0-7918-4992-7.
16. Oh, E. Y. N. and Chai G. W. K. (2006) ”Characterization of marine clay for road embankment design in coastal area” Proceedings of the sixteenth International Offshore and Polar Engineering Conference. 7–10.
17. Mohammed Ali, Mohammed Al-Bared and Aminaton Marto (2017) “A review on the geotechnical and engineering characteristics of marine clay and the modern methods of improvements” Malaysian Journal of Fundamental and Applied Sciences Vol. 13, No. 4 (2017) 825-831.
18. Fairfax County, Va., publication (2013) “Homeowner's guide to overcoming problems with marine clay-marumsco soils in Fairfax county”.

19. Bozkurt M. G. and Fratta, D. (2014) "Deformation Response of Soft Foundation Soils under Tall Embankments – A Numerical Analysis" GeoInstitute Congress. Atlanta, GA. February 2014.
20. Ghada S. Ellithy and Timothy D. Stark (2020) "Case Study: Unsaturated Embankment Failure on Soft Soils" J. Geotech. Geoenviron. Eng., 2020, 146(12): 05020011.
21. Roy, Debasis and Singh, Raghvendra (2008) "Failure of Two High Embankments at Soft Soil Sites" International Conference on Case Histories in Geotechnical Engineering. 2.
22. Hadjigeorgiou J, Kyriakou E and Papanastasiou P. (2006) "A Road Embankment Failure Near Pentalia in Southwest Cyprus" In: Int. Symp. Stab. Rock Slopes Open Pit Min. Civ. Eng.The South African Institute of Mining and Metallurgy, Cape Town, pp 343-352.
23. Shaofu Gu, Weizheng Liu and Mengyuan Ge (2020) "Failure and Remediation of an Embankment on Rigid Column-Improved Soft Soil: Case Study" Hindawi, Advances in Civil Engineering, Volume 2020, Article ID 2637651.
24. Sathananthan, Iyathurai, (2005) " Modelling of Vertical drains with smear installed in soft clay" Doctor of Philosphy thesis, School of Civil, Mining and Environmental engineering, University of Woolongong, 2005.
25. https://www.geoengineer.org/education/web-class-projects/cee-542-soil-site-improve-winter-2014/assignments/prefabricated-vertical-drains
26. D. T. Bergado, M. C. Alfaro and A. S. Balasubramaniam (1992) "Improvement of Soft Bangkok Clay Using Vertical Drains" Geotextiles and Geomembranes 12 (1993) 615-663.
27. Goldberg A. D, M.A. Palmatier (2013) "Recent Advances in Prefabricated Vertical Drains." Ports 2013@ Success through Diversification. ASCE.
28. Ripon Hore, Sudipta Chakraborty and Mehedi A. Ansary (2016) "A Field Investigation to Improve Soft Soils Using Prefabricated Vertical Drains" Transportation Infrastructure Geotechnology (2020) 7:127–155.
29. Timothy D. Stark, Perry J. Ricciardi and Ryan D. Sisk (2017) "Case Study: Vertical Drain and Stability Analyses for a Compacted Embankment on Soft Soils" Journal of Geotechnical and Geoenvironmental Engineering, ASCE, ISSN 1090-0241.
30. Radhakrishnan R and Suriyanarayanan N.S, (2010) „„Embankment Construction over Reclaimed Land Using Pre-Fabricated Vertical Drains" IGC (2010), Mumbai.
31. Jose Leitao Borges (2004) "Three-dimensional analysis of embankments on soft soils incorporating vertical drains by finite element method" Computers and Geotechnics 31 (2004) 665–676.
32. Muktabhant, C., Teerawong, P., and Tengamnuay, V., "Engineering properties of Bangkok subsoils", Proc. of Southeast Asian Conference on Soil Engineering, Bangkok, 1, 1967, pp1-7.
33. Moh, Z. C., Nelson, J. D., and Brand, E. W., "Strength and deformation behaviour of Bangkok clay", Proc. of 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico, 1, 1969, pp287-295.

34. Eide, O., “Geotechnical engineering problems with soft Bangkok clay on the Nakhon Sawan highway project”, Pub. No 78, Norwegian Geotechnical Institute, Norway, 1968.
35. Eide, O., “Exploration, sampling and in-situ testing of soft clay in the Bangkok area”, Proc. of International Symposium on Soft Clay, Thailand, 1977, pp122-137.
36. Bergado, D.T., Anderson, L.R., Miura, N., and Balasubramaniam, AS, "Soft Ground Improvement", American Society of Civil Engineers, 1996, 427 pages
37. Lei Ma, Shui-Long Shen , Chun-Yong Luo & Ye-Shuang Xu (2011) “Field Evaluation on the Strength Increase of Marine Clay under Staged Construction of Embankment” , Marine Georesources & Geotechnology, 29:4, 317-332.
38. Maryam Gaber, Anuar Kasa, Norinah Abdul-Rahman, Jamal Alsharef (2019) “Simulation of Sequential Construction of Embankments on Reinforced Soft Clay Foundation” Indian Geotech J (April 2019) 49(2):224–231.
39. Zhen-Yu Yin, Minna Karstunen, Ching S. Chang, Mirva Koskinen and Matti Lojander (2011) “Modeling Time-Dependent Behavior of Soft Sensitive Clay” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 137, No. 11, ISSN 1090-0241.
40. A. Burman, S. Kumar, N. Himanshu and D. Maity (2015) “Simulation of sequential construction of embankment by finite element method” Asian journal of Civil Engineering (BHRC) vol. 16, no. 3 (2015), pages 347-367.
41. Sukru Ozcoban, Mehmet M. Berilgen, Havvanur Kilic, Tuncer B. Edil, and I. Kutay Ozaydin (2007) “Staged Construction and Settlement of a Dam Founded on Soft Clay” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 8, ASCE, ISSN 1090-0241/2007/8-1003–1016.
42. Madhat Shakir Al-Soud (2016) “ Numerical analysis of prefabricated vertical drains improved soft soil beneath an embankment during staged construction” Journal of Engineering and Development Vol. 20, No.1, January 2016, ISSN 1813-7822.
43. AK M. Zein and L. E. Elgasim (2014) “Study of various techniques for improving weak and compressible clay soil under a high earth embankment” MATEC Web of Conferences 11, 03006 (2014), ISSN 10.1051/ matecconf/20141103006.
44. Huai-Na Wu, Shui-Long Shen, Lei Ma, Zhen-Yu Yin & Suksun Horpibulsuk (2015) “Evaluation of the Strength Increase of Marine Clay under Staged Embankment Loading: A Case Study”, Marine Georesources & Geotechnology, 33:6, 532-541.
45. B. G. Sexton, B. A. McCabe (2013) “Numerical modelling of the improvements to primary and creep settlements offered by granular columns” Acta Geotechnica (2013) 8:447–464.
46. Bernie Mills and Jared McGinn (2010) “Design, Construction, and Performance of a Highway Embankment Failure Repaired with Tire-Derived Aggregate” Transportation Research Record: Journal of the Transportation Research Board, No. 2170, Transportation Research Board of the National Academies, Washington, DC, 2010, pp. 90–99.
47. Bernie Mills, Hany El Naggar, Arun Valsangkar (2015) “North American Overview and a Canadian Perspective on the Use of Tire-Derived Aggregate in Highway Embankment Construction” Chapter 22, Ground Improvement Case Histories, Volume 2, Elsevier Ltd.
48. Manual on estimating soil properties for foundation design (1990), prepared by Cornell University, Ithaca, New York, EPRI EL-6800.
49. PLAXIS material models 2019
50. Zhongkun Ouyang and Paul W. Mayne (2018) “Effective friction angle of clays and silts from piezocone penetration test” Can. Geotech. J. 55: 1230–1247 (2018)

51. Myint Win Bo, Arul Arulrajah, Patimapon Sukmak, Suksun Horpibulsuk (2014) “Mineralogy and geotechnical properties of Singapore marine clay at Changi” Japanese Geotechnical Society, Soils and Foundations 2015; 55 (3):600–613, Elsevier.
52. Jose Leitao Borges (2004) “Three-dimensional analysis of embankments on soft soils incorporating vertical drains by finite element method” Computers and Geotechnics 31 (2004) 665–676.
53. 2. Muktabhant, C., Teerawong, P., and Tengamnuay, V., “Engineering properties of Bangkok subsoils”, Proc. of Southeast Asian Conference on Soil Engineering, Bangkok, 1, 1967.
54. 3. Moh, Z. C., Nelson, J. D., and Brand, E. W., “Strength and deformation behaviour of Bangkok clay”, Proc. of 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico, 1, 1969, pp287-295.
55. 4. Eide, O., “Geotechnical engineering problems with soft Bangkok clay on the Nakhon Sawan highway project”, Pub. No 78, Norwegian Geotechnical Institute, Norway, 1968.
56. 5. Eide, O., “Exploration, sampling and in-situ testing of soft clay in the Bangkok area”, Proc. of International Symposium on Soft Clay, Thailand, 1977, pp122-137.
57. 6. Bergado, D.T., Anderson, L.R., Miura, N., and Balasubramaniam, AS, "Soft Ground Improvement", American Society of Civil Engineers, 1996, 427 pages
58. 7. Lei Ma, Shui-Long Shen , Chun-Yong Luo & Ye-Shuang Xu (2011) “Field Evaluation on the Strength Increase of Marine Clay under Staged Construction of Embankment” , Marine Georesources & Geotechnology, 29:4, 317-332.
59. 8. B. Indraratna, I. Sathananthan, C. Rujikiatkamjorn and A. S. Balasubramaniam (2005) “Analytical and Numerical Modeling of Soft Soil Stabilized by Prefabricated Vertical Drains Incorporating Vacuum Preloading” International Journal of Gemechnics, Vol 5, No.2, ISSN 1532-3641/2005/2-114–124.
60. 9. Sathananthan Iyathurai (2005) “Modelling of vertical drains with smear installed in soft clay”, Doctor of Philosophy thesis, School of Civil, Mining and Environmental Engineering, University of Wollongong, 2005. https://ro.uow.edu.au/theses/1924
61. 10. Buddhima Indraratna, Rui Zhong, and Cholachat Rujikiatkamjorn (2016) “An Analytical Model of PVD-assisted Soft Ground Consolidation” Advances in Transportation Geotechnics 3, Volume 143 2016, Pages 1376-1383. doi: 10.1016/j.proeng.2016.06.162.
62. 11. Indraratna, B, Bamunawita, C, Redana, I, and McIntosh, G (2003) “Modelling of prefabricated vertical drains in soft clay and evaluation of their effectiveness in practice”. http://ro.uow.edu.au/engpapers/394
63. 12. Lin, D. G. & Chang, K. T. (2009) “Three-dimensional numerical modelling of soft ground improved by prefabricated vertical drains” Geosynthetics International, 16, No. 5, 339–353. doi: 10.1680/gein.2009.16.5.339.
64. 13. Shui-Long Shen, Jin-Chun Chai, Zhen-Shun Hong, Feng-Xi Cai (2005) “Analysis of field performance of embankments on soft clay deposit with and without PVD improvement” Geotextiles and Geomembranes 23 (2005) 463–485.
65. 14. Bernie Mills and Jared McGinn (2010) “Design, Construction, and Performance of a Highway Embankment Failure Repaired with Tire-Derived Aggregate” Transportation Research Record: Journal of the Transportation Research Board, No. 2170, Transportation Research Board of the National Academies, Washington, DC, 2010, pp. 90–99.
66. 15. Y.K. Wong, Joseph (2013) “Comparison of Drainage Line Elements in PLAXIS 2D and 3D applied in Consolidating Marine Clay Deposits” SEIC 2013, IP 6-63.

67. 16. Bernie Mills, Hany El Naggar, Arun Valsangkar (2015) “North American Overview and a Canadian Perspective on the Use of Tire-Derived Aggregate in Highway Embankment Construction” Chapter 22, Ground Improvement Case Histories, Volume 2, Elsevier Ltd.
68. 17. Manual on estimating soil properties for foundation design (1990), prepared by Cornell University, Ithaca, New York, EPRI EL-6800.
69. 18. Zhongkun Ouyang and Paul W. Mayne (2018) “Effective friction angle of clays and silts from piezocone penetration test” Can. Geotech. J. 55: 1230–1247 (2018)
70. 19. Myint Win Bo, Arul Arulrajah, Patimapon Sukmak, Suksun Horpibulsuk (2014) “Mineralogy and geotechnical properties of Singapore marine clay at Changi” Japanese Geotechnical Society, Soils and Foundations 2015; 55 (3):600–613, Elsevier.
71. 20. Maryam Gaber, Anuar Kasa, Norinah Abdul-Rahman, Jamal Alsharef (2019) “Simulation of Sequential Construction of Embankments on Reinforced Soft Clay Foundation” Indian Geotech J (April 2019) 49(2):224–231.
72. 21. PLAXIS material models 2019
73. 22. B. G. Sexton, B. A. McCabe (2013) “Numerical modelling of the improvements to primary and creep settlements offered by granular columns” Acta Geotechnica (2013) 8:447–464.