ANALYSIS AND DESIGN OF SAND DRAINS AND PREFABRICATED VERTICAL DRAINS FOR THE RUNWAY OF KHAN JAHAN ALI AIRPORT AT BAGERHAT

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

by

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DEDICATIONS

This thesis is dedicated to my parents who have given me the opportunity of education from the best institutions and the support throughout my life. I am thankful to the Almighty who had blessed me with my parents.

DECLARATION

It is declared that this thesis or any part of it has not submitted elsewhere for the award of any degree or diploma, except for publication.

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Signature

Pradipta Banik Dip

ACKNOWLEDGEMENT

Above all, I am grateful to the Almighty that I have completed my thesis. I wish to express my sincere gratitude and heartiest felicitation to my respected supervisor Dr. Abu Siddique, Professor, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka for his valiant help and meticulous supervision. He taught me how to pursue a research work with endurance. Many terms on different problems very often seemed vague to me and obstacles at the time of thinking were cleared by his deliberate direction and pragmatic discussion. I am also greatly indebted to him for continuous support, valuable suggestion and enthusiastic encouragement throughout the progress of my thesis. Without his bountiful support and spontaneous assistance both in academic and personal concerns, it would have been impossible to go smoothly with my thesis. At home my parents always geared up me till the completion of the thesis. My heart-felt gratitude goes to them for their roles like mentors.

ABSTRACT

The thesis contains the design of sand drain and prefabricated vertical drain for the proposed Khan Jahan Ali Airport Runway. After the designing, suitable spacing of drain, @ 1.25-1.5 c/c of equivalent diameter of drain for PVD in square pattern has been recommended according to the soil condition of the project area, cost and time of the project.

The thesis also concerns with the parametric studies of sand drain and prefabricated vertical drain. Effects of parameters such as spacing of drain (1, 1.25, 1.5, 1.75, 2 m), co-efficient of consolidation for horizontal flow (5, 10, 15, 20, 30 m^2/yr), degree of consolidation (50, 70, 90, 92 %), soft soil layer thickness (2, 4, 6, 8 m) and diameter of drain (0.15, 0.2, 0.25, 0.3 m) on consolidation time have been presented. For both sand drain and PVD, consolidation time increases with the increase of spacing of drains, soft soil layer thickness and degree of consolidation, keeping other parameters constant. Consolidation time decreases with the increase of co-efficient of consolidation for horizontal flow and diameter of drain, other parameter keeping constant. However, sand drain of any specific diameter require less consolidation time than PVD keeping degree of consolidation and co-efficient of consolidation for horizontal flow and soft soil layer thickness constant. Keeping other parameters constant, sand drain or PVD in triangular pattern requires less consolidation time than square pattern. Also bilateral flow requires less time than unilateral flow.

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List of Symbols

Symbols	Meaning
C_h	Co-efficient of consolidation for horizontal flow
C_v	Co-efficient of consolidation for vertical flow
D	Diameter of sphere of influence of drained soil
d	Diameter of drain
d_e	Diameter of equivalent cylinder of drained soil
d_w	Diameter of drain well
n	A ratio = d_c/d_w
q_w	Drainage capacity of the drained well
H	Thickness of consolidating layer
t	Consolidation time
S	Final settlement due to surcharge
s	Spacing of drain
U_h	Average horizontal consolidation degree
T_h	Time factor in radial consolidation
I	Drain length at unilateral flow
S(b)	Square pattern of bilateral flow
S(u)	Square pattern of unilateral flow
T(b)	Triangular pattern of bilateral flow
T(u)	Triangular pattern of unilateral flow
k_c	Permeability of soil
ΔC_U	Increase in undrained shear strength
I_z	Influence factor for centre of embankment
I_{z1}, I_{z2}	Influence factors for toe of embankment
ϕ'	Effective angle of internal friction
σ	Vertical stress at the base of the embankment
C_m	Undrained Shear Strength of Fill
φ_m	Undrained angle of internal friction

Chapter 1

Introduction

1.1 General

The main goal of most soil improvement techniques used for reducing liquefaction hazards is to avoid large increases in pore water pressure during earthquake shaking. This can be achieved by densification of the soil or improvement its drainage capacity. In Bangladesh, there has been a growing demand for construction for on sites underlain by a thick layer of soft soil. Circumstances require that, the soft soil must be properly treated before constructing a structure on such a soil. Otherwise the structure may damage due to shear failure and differential settlement of the underlying soil.

Soft soils in Bangladesh are mainly available in the alluvial flood plain deposits, depression deposits and estuarine and tidal plain deposits. The soft soil, mainly comprise under-consolidated to normally consolidated clays and silts often containing organic materials. So soil improvement is essential before constructing a structure. Sand drain and prefabricated vertical drain can be a feasible solution as soil improvement techniques for such soft soils in Bangladesh.

1.2 Objectives of the Study

1) Designing of sand drain and prefabricated vertical drain are to be used in the Runway of Khan Jahan Ali Airport with the help of excel generated worksheet according to the soil condition of the project area.

2) Analyzing the parametric studies of sand drain and PVD like effect on consolidation time varying various parameters such as spacing of drain, co-efficient of consolidation for horizontal flow, degree of consolidation, diameter of drain, soft soil layer thickness.

1.3 Layout of the Thesis

The investigation is presented in five chapters including chapter 1 is introduction, chapter 2 is literature review, chapter 3 is design methodology for sand drains and PVDs, chapter 4 presents the analysis and design of sand drain and PVD, finally chapter 6 describes the conclusions and recommendations for future study. Excel generated worksheets are included in Appendix A, Appendix B, Appendix C.

1.4 Problem with Soft Soils in Bangladesh

Ansary et al. (2000) reported that the principal foundation problems in Bangladesh are related to the low shearing resistance of the underlying soil. Soils with low shear strength are not strong enough to support the most common structures with conventional shallow foundation systems and therefore pose a serious problem for the entire region. In practice, various types of shallow foundations such as pad, strip and compensating types are used for light structures with pressures ranging from 25-40kPa at depth between 1.5-3m.

Settlement plays another major foundational problem in Bangladesh related to the loose and compressible nature of the subsoil. Excessive settlement is observed with many structures even with portal frames and boundary walls. The most extreme settlement can be seen in rural roads and also in major roads and also in regional roads connecting the districts. Several segments of these roads are built on 1.5-3m high embankments where settlements up to 40cm were recorded (Mollah 1993). It is expected that these settlement are the results of consolidation of both fill material and compressible soft soil occurring near the surface. Numerous settlements on a large number of road segments have made the bituminous running surface uneven, causing severe cracking. Embankments, requiring extensive filling work, are constructed throughout the plains of Bangladesh for flood protection, irrigation and the development of a road network. In general, the natural state of the local soil is not suitable for embankment construction and maintenance. The most common problem with embankment construction is the generally soft nature of the top soil as well as the foundation soil.

The position of ground water table in Bangladesh lies between 1-5m. During the rainy season the water table is higher than 1m and the land is often flooded. Construction activities, therefore often involve excavation of considerable depth underwater. The excavations lead to unstable situations because the work is made either in highly permeable loose sandy soils or in soft plastic clays.

Liquefaction is the phenomenon by which saturated soils, essentially loose, are temporarily transformed into a liquefied state. In the process, the soil undergoes transient loss of shear strength which commonly allows ground improvement displacement or ground failure. Liquefaction characteristics of a soil depend on several factors, among others, position of ground water table, grain size distribution, soil density, ground acceleration, sedimentation history, thickness of the deposit, location of drainage, magnitude and nature of superimposed loads (Seed & Idriss 1967, Casagrandre 1976, Seed 1979). In Bangladesh, soils are typically young, comprising sandy material and occurring in a saturated condition. In general, the soil has a density less than the critical density. Therefore the particular characteristics of the soil in the top 15-20m together with the seismic history of the region render these soils sensitive to liquefaction. Extensive field and laboratory tests conducted during the feasibility study for Bangabandhu Jamuna Multipurpose Bridge gave the same conclusion (Heiznen 1988).

Chapter 2

Literature Review

2.1 General

There are several advanced technologies of soil improvement available now-a-days. Based on soil condition and cost analysis one of the improvement techniques is chosen. Civil engineers are solely responsible for selecting the appropriate technology with the judgment and experience they have got according to the soil conditions and characteristics.

Definition of soft soil and ground

1) Loose sandy soil N≤10
2) Soft clay and silt N≤4
3) Organic soil N≤4

Depends on Cohesion value

1) Soft soil C<25
2) Medium stiff C=25-50kPa
3) Stiff soil C=50-100kPa
4) Very stiff soil C>100kPa

Difficulties of soft soil and ground

1) Liquefaction earthquake
2) Consolidation settlement
3) Stability or shear strength

2.2 Methods of Subsoil Improvement

1) Removal and Replacement
2) Preloading
3) In-situ densification
4) Dynamic compaction
5) Blast densification
6) Vibroflotation
7) Grouting
8) Stabilization using admixtures
9) Vacuum consolidation
10) Sand drain
11) Prefabricated vertical drain
12) Reinforced earth method
13) Soil nailing
14) Stone column
15) Sand compaction pile

2.2.1 Removal and Replacement

It is one of the oldest and simplest soil improvement methods. For localized areas with soft soil of limited depth and thickness, removal of unsuitable material and replacement with suitable fill may be carried out. These unsuitable materials were encountered in valleys and low-lying areas and may be replaced with well-compacted suitable fill. Excavation and replacement could be carried out up to 5-6m. The removal and replacement may be required to be carried out even in cutting areas where the naturally occurring soils were found to be of a low shear strength and high moisture content. Figure 2.1 shows the removal and replacement procedure.

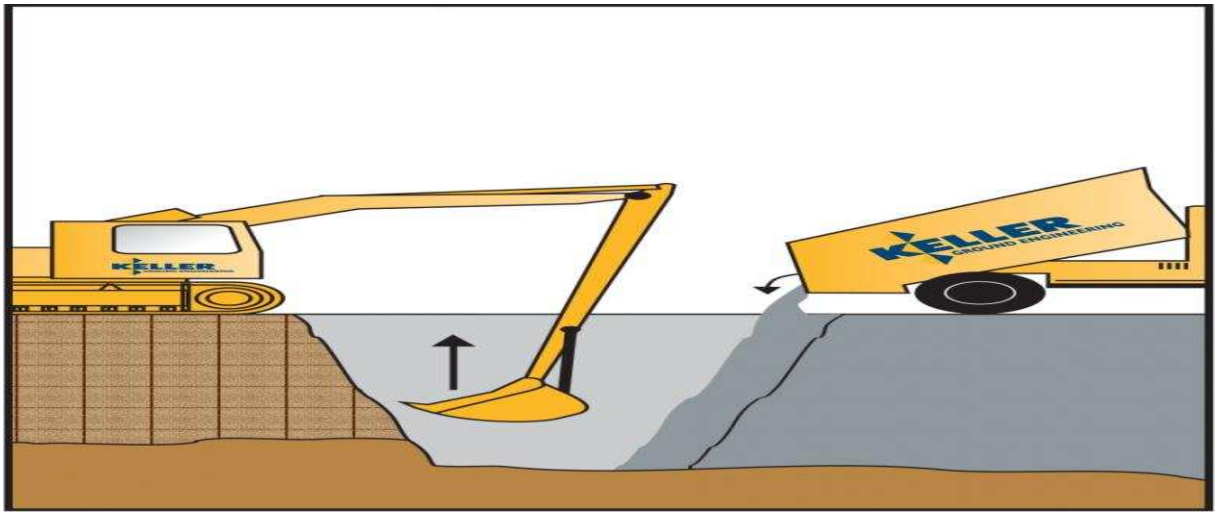


Figure 2.1: Removal using Excavator and Replacement using Dump Vehicle

2.2.2 Preloading

It is quite common for designers of embankments on soft ground to specify surcharge preloading to compensate or eliminate post-construction settlements. Properly designed and executed, the method can be a powerful and economical way to build high embankments on soft ground. It certainly represents a much cheaper alternative to solutions that involve constructing a rigid foundation such as a piled slab or stone columns beneath the embankment. To build an embankment to a final height, H(f) that would not settle or would settle very little after it has been built, the embankment is first built to a height H(s) + H(f) that is higher than the desired final height and is left to settle for a period T(p) under the load intensity p(f)_+ p(s) due to this extra height of fill. At the end of the preloading period T(p), the surcharge fill of height H(s) is removed, causing the soft soil to be unloaded, resulting in elimination or a huge reduction in post-construction settlements under the final height of embankment H(f). Figure 2.2 illustrates the key elements of the concept of surcharge preloading to compensate for primary and secondary settlements. Usually, the aim is to eliminate 100% of primary consolidation settlement and enough secondary settlement such that the residual settlement is within acceptable performance limits.

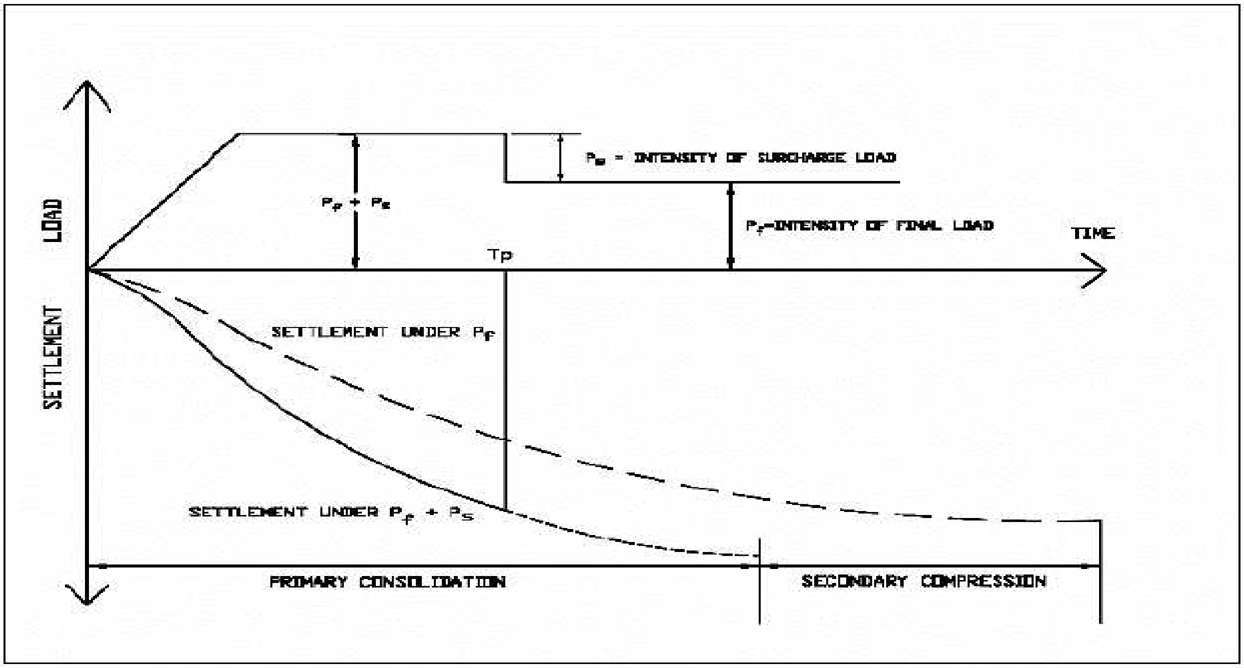


Figure 2.2: Concept of Surcharge Preloading to Compensate for Primary Consolidation Settlement and Secondary Compression

2.2.2.1 Advantages of Preloading

1) Requires only conventional earthmoving equipment.
2) Any grading contractor can perform the work.
3) Long track record of success.

2.2.2.2 Disadvantages of Preloading

1) Surcharge fill must extend horizontally at least 10m beyond the perimeter of the planned construction, which may not be possible at confined sites.
2) Transport of large quantities of soil required.
3) Surcharge must remain in place for months or years, thus delaying construction.

In the figure 2.3 it is pointed out the preloading using surcharge.

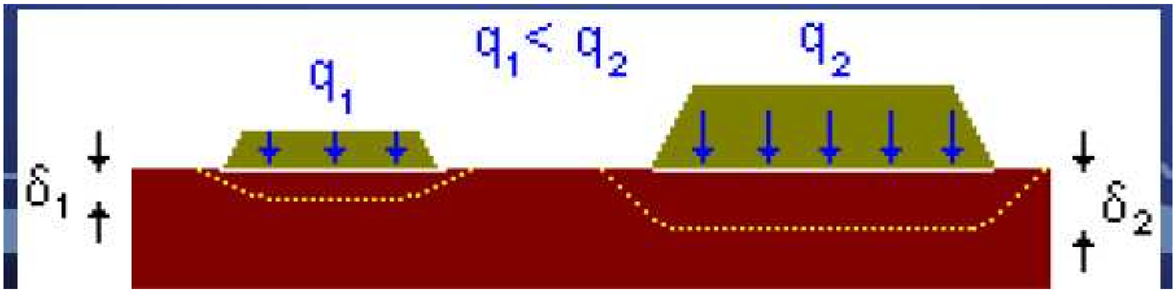


Figure 2.3: Preloading using Surcharge

2.2.3 In-situ Densification

1) Most effective in sands.
2) Methods used in conventional earthwork are only effective to about 2m below the surface.
3) In-situ methods like dynamic deep compaction are for soils deeper than can be compacted from the surface.

2.2.4 Dynamic Compaction

Dynamic compaction is a method that is used to increase the density of the soil when certain subsurface constraints make other methods inappropriate. The process involves of dropping a heavy weight repeatedly on the ground at regular spaced intervals. The weight and the height determine the amount of compaction that would occur. The weight that is used depends on the degree of compaction desired 8-36ton. The height varies from 1m to 30m. The impact of the free fall creates stress waves that help in the densification of the soil. These stress waves can penetrate up to 10m. In cohesion less soils, these waves create liquefaction that is followed by the compaction of the soil and in cohesive soils they create an increased amount of pore water pressure that is followed by the compaction of the soil. Pore water pressure is the pressure of water that is trapped within the particles of rocks and soils. The degree of compaction depends on the weight of the hammer, the height from which the hammer is dropped and the spacing of the locations at which the hammer is dropped. The initial weight dropping has the most impact and penetrates up to a greater depth. The following drops if spaced closer to one another, compact the shallower layers and the process is completed by compacting the soil at the surface.

Most soils can be improved with dynamic compaction. Figure 2.4 points about the dynamic compaction.

1) Uses a special crane to lift 5-30 tons to heights of 40 to 100 feet then drop these weights onto the ground.

2) Cost effective method of densifying loose sands and silty soils up to 15 to 30 feet deep.

Figure 2.4: Dynamic Compaction

2.2.5 Blast Densification

Another method of in-situ densification is blast densification. This method consists of drilling a series of boring and using them to place explosives underground. These explosives are then detonated and the resulting shock waver densify the surrounding soils. Blast densification has been used successfully on many projects and is most effective in clean sands. Figure 2.5 indicates the blast densification process.

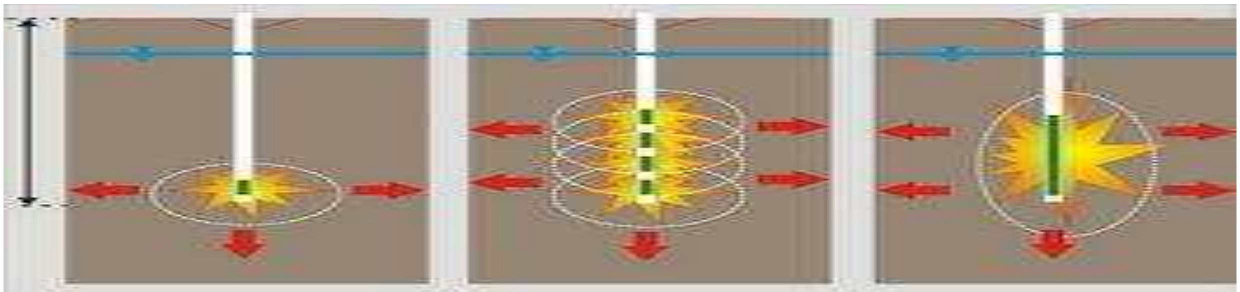


Figure 2.5: Blast Densification

2.2.6 Vibroflotation

Vibroflotation is a ground improvement technique used at a considerable depth that by using a powered electrically or hydraulically probe, it strengthens the soil. The vibroflotation will compact the soil making it suitable to support design loads. It involves the introduction of granular soil to form interlocking columns with surrounding soil. The technique is used to improve bearing capacity and reduce the possibility of differential settlements that might be allowed for the proposed loads. Sometimes it is also referred as Vibro-compaction and the ultimate concept is to repack soil particles by joining them together improving soil's bearing capacity. The compaction of soil can be obtained in soils as deep as 200 feet. The risk of liquefaction an earthquake prone area is also drastically reduced.

2.2.6.1 Vibroflotataion Techniques

Vibroflotation can be obtained by using three different techniques:

1) Vibro Compaction method- This method allows granular soils to be compacted. This method is only used to compact sandy soils.

2) Vibro Replacement method- The technique is used to replace poor or inadequate soil material by flushing out the soil with air or water and replacing it with granular soil. This can be used in various soil types such as clay and sandy soils.

3) Vibro Displacement method- This procedure is used with no or small amounts of water used during the technique. The probe in inserted into the soil and it will displace it laterally as the new soil column is being formed and compacted.

2.2.6.2 Advantages of Vibroflotation

Vibroflotation is one affordable way to improve ground conditions when a deep layer of inadequate soil is found. The technique is so simple that will not require the delivery of additional materials or additional equipment other than the probe and the equipment that has it installed. The vibroflotation process can offer the following benefits:

1) When the process is done properly, it will reduce the possibility of differential settlements that will improve the foundation condition of the proposed structure.

2) It is the fastest and easiest way to improve soil when bottom layers of soil will not provide good load bearing capacity.

3) It is a great technology to improve harbor bottoms.

4) On a cost-related standpoint, it helps improve thousands of cubic meters per day. It is faster than piling.

5) It can be done around existing structures without damaging them.

6) It does not harm the environment.

7) It improves the soil strata using its own characteristic.

8) No excavations are needed, reducing the hazards, contamination of soils and hauling material out from the site.

9) No need to manage table water issues, neither the permits required to manage water discharge and dewatering issues.

10) The technique of vibroflotation can be adapted to each scenario and site.

11) When vibroflotation is performed at a site, it will reduce the possibility of liquefaction during an earthquake.

2.2.6.3 Process of Vibroflotation

The process of vibroflotation is really simple as you will see in the following short description. The depth probe is located over the compaction point. Flushing water or air is expelled trough jets in the tip of the probe. These induced injected vibrations will liquefy the soil temporarily allowing the probe a continuous penetration under its own weight. Once the probe has reached the strata or poor soil, the water and air injections is stopped. At this point the soil is densified by the probe vibrations causing a crater around the vibrator that should be backfilled with granular material. Once the process has been completed, the probe is slowly withdrawn usually in stages of 12 inched. A cylindrical compaction zone is formed around the probe, and the achieved degree of compaction is indicated by an increase in oil pressure. The area around the probe is backfilled with granular material that will auto-consolidate, as the probe is begin brought up. The material used to backfill should be free of silt, gravel or crushed stone. Figure 2.6 indicates about vibroflotation installation.

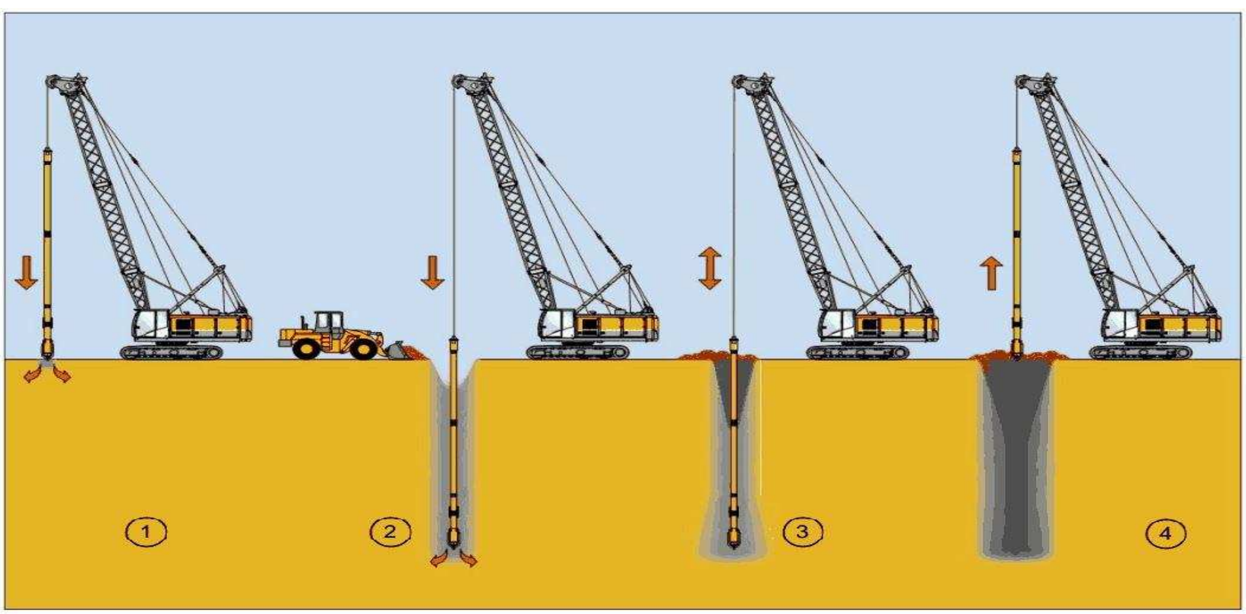


Figure 2.6: Vibroflotation

2.2.7 Grouting

It is defined as the injection of a special liquid or slurry material called grout into the ground for the purpose of improving the soil or rock. Figure 2.7 indicates about the grouting procedure.

2.2.7.1 Types of Grouts

1) Cementitious grouts

2) Chemical grouts

2.2.7.2 Grouting Methods

Intrusion grouting

1) Consists of filling joints or fractures with grout.

2) Primary benefit is reduction in hydraulic conductivity.

3) Used to prepare foundation and abutments for dams.

4) Usually done using cementitious grouts.

Permeation grouting

1) Injection of thin grouts into the soil.

2) Once the soil cures, becomes a solid mass.

3) Done using chemical grouts.

4) Used for creating groundwater barriers or preparing ground before tunneling.

Compaction grouting

1) When low-slump compaction grout is injected into granular soils, grout bulbs are formed that displace and densify surrounding loose soils.

2) Used to repair structures that have excessive settlement.

Jet grouting

1) Developed in Japan.

2) Uses a special pipe with horizontal jets that inject grout into the ground at high pressures.

3) Jet grouting is an erosion/replacement system that creates an engineered, in situ soil/cement product known as Soil cretesm. Effective across the widest range of soil types, and capable of being performed around subsurface obstructions and in confined spaces, jet grouting is a versatile and valuable tool for soft soil stabilization, underpinning, excavation support and groundwater control.

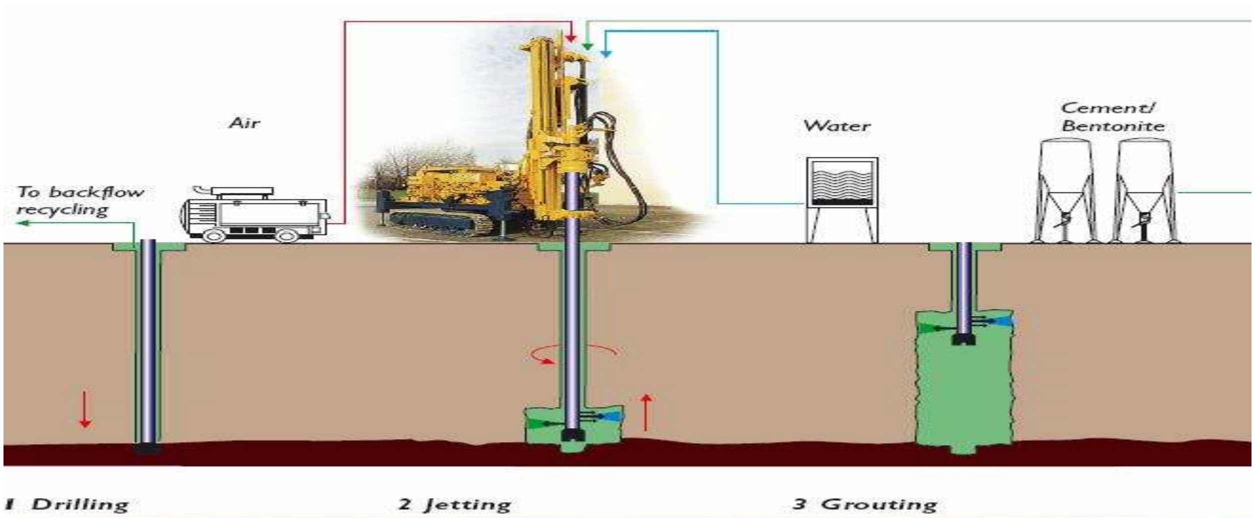


Figure 2.7: Grouting Procedure

2.2.8 Stabilization using Admixtures

Most common admixture is Portland Cement

1) When mixed with soil, forms soil-cement which is comparable to a weak concrete.
2) Other admixtures include lime and asphalt.
3) Objective is to provide artificial cementation, thus increasing strength and reducing both compressibility and hydraulic conductivity.
4) Used to reduce expansion potential of clays.
5) Used in surface mixing applications.

2.2.9 Vacuum Consolidation

The Menard Vacuum Consolidation method is an atmospheric consolidation system used for preloading soft saturated fine grained soils such as clays, silts or peat. This innovative procedure consists of installing a vertical and horizontal draining and vacuum pumping system under an airtight impervious membrane. The treatment area is the sealed by sealing the membrane into a network of peripheral trenches. These trenches are continuously recharged and filled with water to maintain full saturation of the soils and to avoid a general lowering of the ground water table within the treatment area. Figure 2.8 points the mechanism of vacuum consolidation.

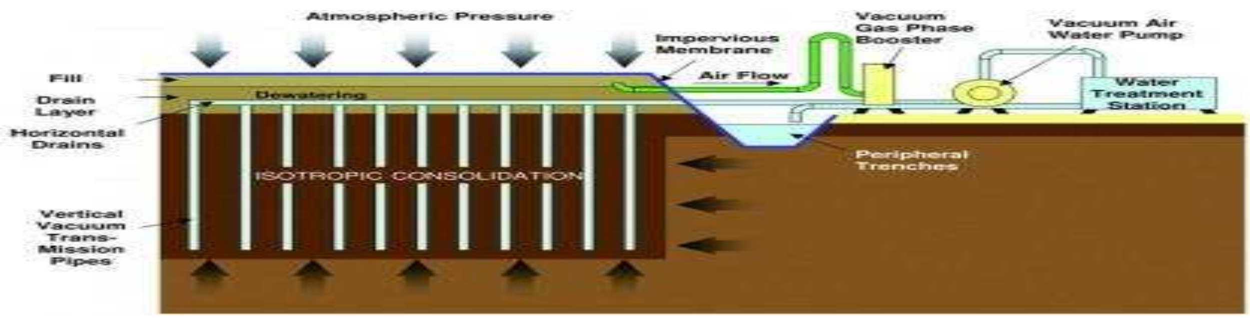


Figure 2.8: Vacuum Consolidation

2.2.10 Sand Drain

1) Vertical drains are installed under a surcharge load to accelerate the drainage of impervious soils and thus speed up consolidation.

2) These drains provide a shorter path for the water to flow through to get away from the soil.

3) Time to drain clay layers can be reduced from years to a couple of months.

Figure 2.9 shows the typical sand drain installation.

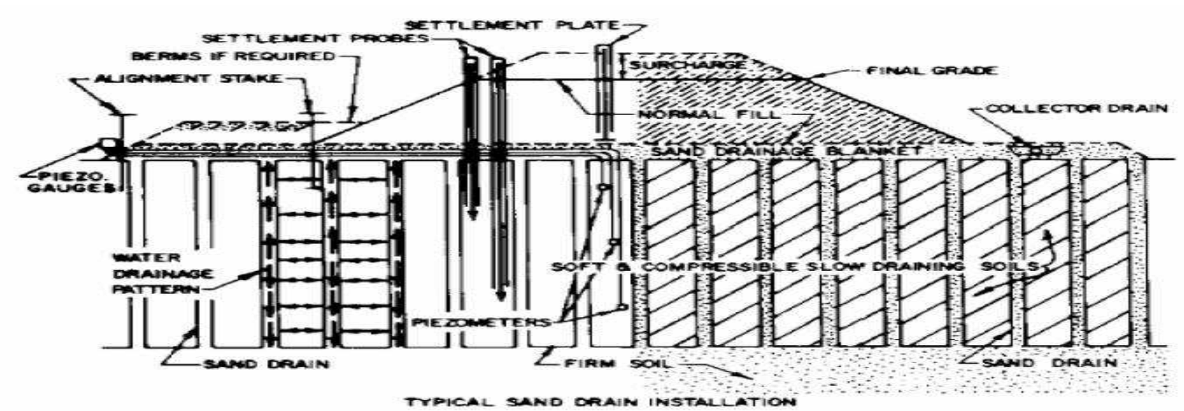


Figure 2.9: Sand Drain

2.2.11 Prefabricated Vertical Drain

Geo-synthetics are used as a substitute to sand columns. In figure 2.10, wick drain and PVD mechanism are found.

1) Installed by being pushed or vibrated into the ground.

2) Most are about 100 mm wide and 5 mm thick.

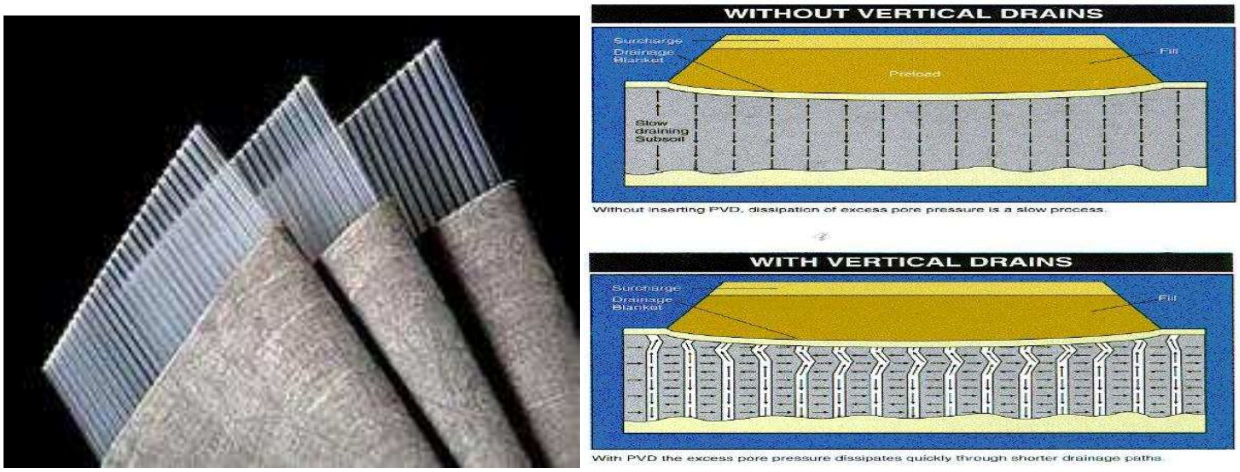


Figure 2.10: Wick Drain and PVD Mechanism

2.2.12 Reinforced Earth Method

Reinforced earth method is a composite material that is made internally stable by the interaction between soil which for certain gradations and compaction conditions is strong in compression, shear and reinforcement which are strong in tension. Figure 2.11 shows the detailing of reinforced earth method.

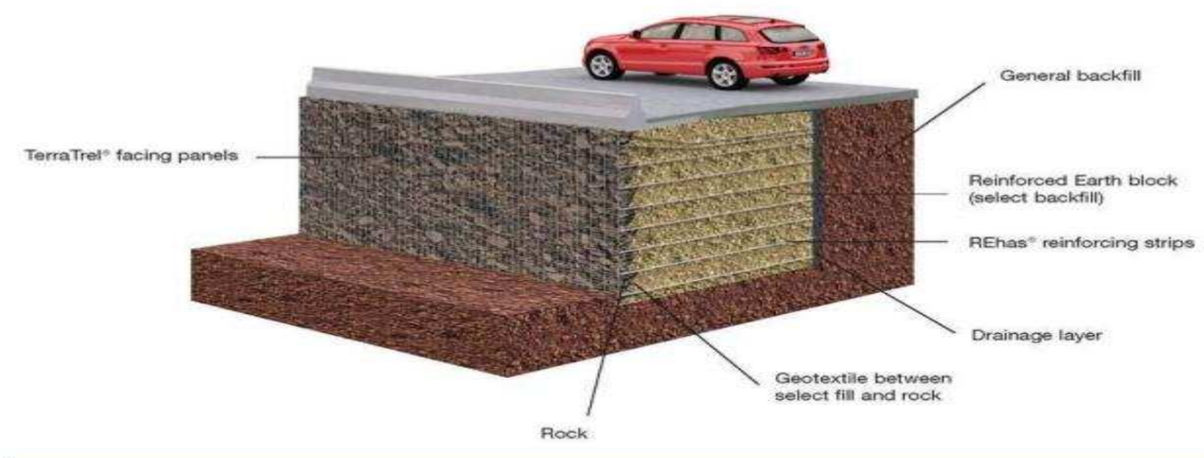


Figure 2.11: Reinforced Earth

2.2.13 Soil Nailing

The fundamental concept of soil nailing consists of reinforcing the ground by passive inclusions, closely spaced, to create in-situ coherent gravity structure and thereby to increase the overall shear strength of the in-situ soil and restrain its displacements. Figure 2.12 shows the picture of soil nailing.

Figure 2.12: Soil Nailing

2.2.14 Stone Columns

Stone Columns are designed to improve the load bearing capacity of in-situ soils and fills and to reduce differential settlements of non-homogeneous and compressible soils, allowing the use of shallow footings and thinner base slabs. Stone Columns are formed by inserting a vibrating probe to incorporate granular aggregate into the ground via the resulting void. This is followed by the re-compaction of granular aggregate. Both Top and Bottom feed techniques are available, depending on the stability of the in-situ soils and water level. The Stone Columns are typically installed under uniformly loaded structures, such a building slabs and embankments, on a regular grid spacing. A load transfer platform can then be designed to spread the load from the structure to the improved ground. This technology is well suited for the improvement of soft soils such as silty sand, silts, clays and non homogeneous fills. In figure 2.13 the mechanism of stone columns is found.

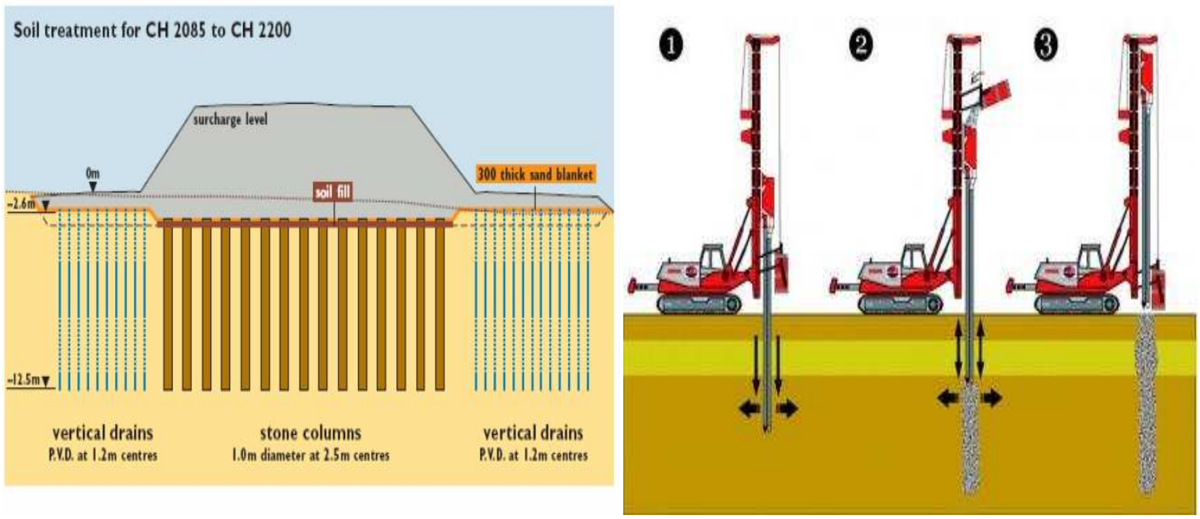


Figure 2.13: Stone Columns

2.2.15 Sand Compaction Pile

It is an improvement method whereby strong compacted sand piles are created in ground to form composite ground at cohesive soil. This increases the shear resistant strength, and at the same time, it stabilizes early settlement to reduce the consolidation settlement rate. For sandy ground, it increases the relative density, thus enhances the shear strength. By using vibro-hammer to repeatedly driving in & withdraw the casing pipe, then again re-drive in the casing to forcefully create the compacted sand piles into the ground. In sandy ground, due to the insertive effect of the sand piles, relative density of the entire ground is being increased, the shear strength is enhanced. In cohesive ground, the composite ground is formed by the mixture of sand piles & cohesive soil, thus they strengthen the ground. A static SCP method which consolidates the expanded sand pile by generating the injecting force through the inner screw and the twisting shear occurs at the bottoms side of screw. The strength & the improvement effect of the bottom side created Sand Compaction Pile is nearly the same as Land SCP. In figure 2.15, Sand compaction pile procedure is shown.

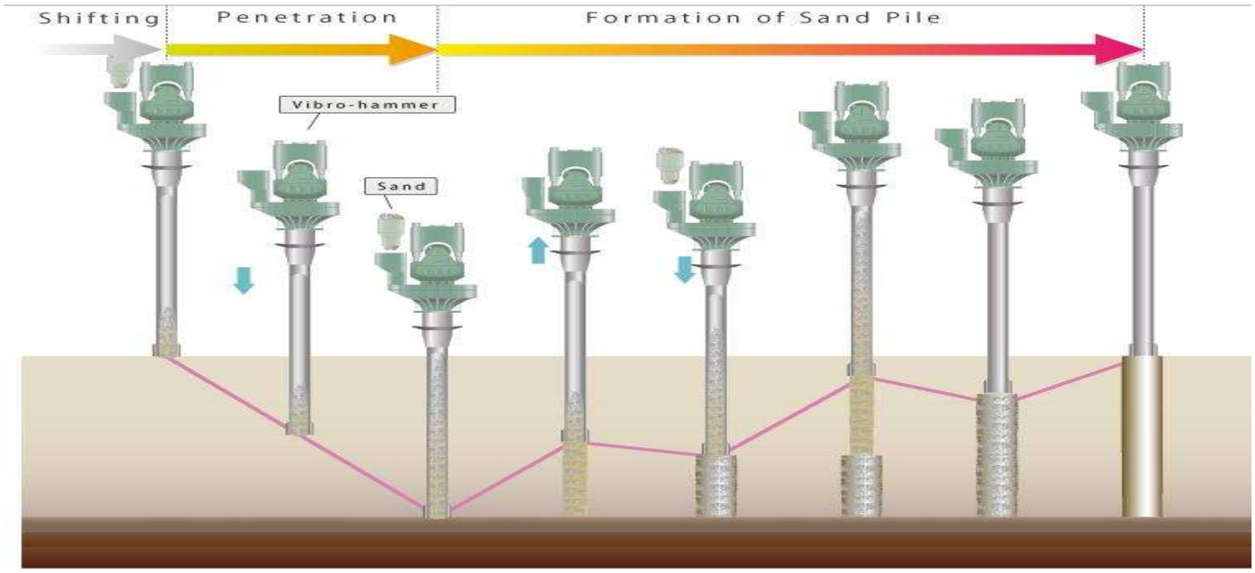


Figure 2.14: Sand Compaction Pile

2.3 Case Histories of Preloading with Prefabricated Vertical Drains in Road and Railway Projects

2.3.1 Dhaka Integrated Flood Protection Project

Siddique et al. (2000) reported about the information of the wake of the floods of the 1987 and 1988, the Government of Bangladesh established a committee for flood control and drainage of greater Dhaka in October, 1988. The primary objective of the committee was the preparation of a flood control plan for the greater Dhaka area. In January 1989, the committee submitted a detailed plan phased investments in flood protection and drainage for Dhaka and other cities which was approved by the government of Bangladesh in March, 1989. In view of the high priority assigned to the Dhaka flood protection project, the Government of Bangladesh immediately initiated Phase 1 of the recommended works on a crash program basis using their own resources. These works which were designed to provide protection for about 137 sq. km in the urbanized western part of the city of Dhaka, included among others, the construction of: (i) a 29.2 km of embankment along the west side of the city and (ii) an 8.5 km of reinforced concrete wall bordering the densely populated south western side of the city. Ansary et al. (1998) reported the results of recent soil investigations carried out at the Dhaka Integrated Flood Protection Embankment site. The sub grade soil profile beneath the western embankment was found to be fairly consistent along the embankment alignment. The sub grade soils consisted of an upper 1m to 30m layer of soft clayey silt with high plasticity or non-plastic clay with silt. This layer was underlain by medium dense silty sand or sand at depth. The clayey silt or silt lyer encountered below the embankment varied in thickness from about 1m to 30m. Undrained shear strength varied between 25kPa to 50kPa. Initial void ratio, compression index and co-efficient of consolidation (C_v) varied from 0.78 to 0.89, 0.06 to 0.09 and 0.001 to 0.008 cm^2/s respectively. The upper layer of clayey silt to silt soils were interbedded with very soft, high plasticity organic clays or silts at several locations along the embankment alignment. The soil layers were typically less than 3m in thickness. The organic clay and silt layers were of high plasticity, very weak and highly compressible. The upper layer of clayey silt and soils are underlain at depth by silty sand layer.

2.3.2 Jamuna Bridge Railway Link Project

Siddique at al. (2000) reported about the information that the embankment for the Jamuna Bridge Railway Link Project at Kaliakoir, Gazipur, runs through 8 zones of soft ground. The total length of the soft ground zones in the Jamuna Bridge Railway Link Project is 4.05 km. The maximum height of the embankment to be constructed is 6 m. Some sections of the embankment have been designed with lateral berm. The soft ground consists of very soft organic clay. The thickness of the soft organic clay layers varied from 3.5 m to 14 m in the 8 zones. The N-value from SPT in these soft layers varied from 0 to 3. The values of compression index, initial void ratio in the compressible layers varied from 0.4 to 0.65 and 1.2 to 1.6 respectively. Primary consolidation settlements and consolidation time of organic soft ground caused by loads transmitted with the proposed embankment heights were estimated. The estimated primary settlements for the 8 zones varied between 33 cm and 148 cm depending on the thickness of the compressible layer. It was found that without installation of any drain 80% consolidation settlements will reach within 5 to 70 years, depending on the thickness of the compressible layer.

The co-efficient of consolidation for vertical drainage and radial drainage were taken as $2.5x10^{-4}$ cm^2/s and $5x10^{-4}$ cm^2/s respectively.

Due to bad soil quality and poor mechanical properties, it has been recommended to construct the embankment in 2 stages. The critical height of the first stage construction should provide a factor of safety at least 1.3 by short time. The vertical drain spacing for 8 soft ground zones has been calculated to obtain at least 80% consolidation in 4 months. This time has been considered to be sufficient to improve enough supporting ground and construction of the next stage. The recommended vertical drain spacing for square and triangular pattern varied from 1m to 2m and 1.07 m to 2.15 m respectively. Close monitoring of settlement and pore pressure has been recommended. Settlement will be measured with gauge placed at the surface of natural ground. Pore water pressur will be measure with gauge drilled in the compressible layers at various levels. These apparatus are estimated to check the soil behavior and to prevent failure in case of stage construction. Each measurement profile will be composed of 2 settlement gauges and to water pressure gauges in the centerline and drilled at 2.5 m and 5 m depth.

2.3.3 Case History using Sand Drains

Prior to 1959, the chemical industry used sugar cane (mainly from Cuba) for its source of the carbohydrate sucrose, used mainly to produce ethyl alcohol in the manufacture of munitions and alcohol. With Fidel Castro coming to power in 1960 this nearby source was abruptly eliminated and the alternate was molasses. Of course, storage of molasses (a high viscosity liquid of specific gravity 1.5) required storage tanks for the unloading of tanker ships until such time as usage required. Two large 37 m diameter side-by-side tanks were designed for a site at the Wilmington Port Authority ("property owner") which is at the confluence of the Delaware and Christiana Rivers in Wilmington, Delaware; see Figure 4. The two tanks were each to be 12.1 m high and with molasses at a specific gravity of 1.5 this is equivalent to a ground surface loading of 180 kPa. The site, however, consisted of approximately 20 m of saturated organic clay. Three soil borings (center and at opposite ends of the site) were taken along with numerous undisturbed soil samples. Blow counts were extremely low (0 to 5 blows/300 mm) for 20-m and then a firm granular soil layer was encountered. There was some visual evidence of horizontal stratification within the organic clay, but it was inconclusive. The water table was at the slag covered ground surface and site flooding was not unusual. The undisturbed samples were sent to the consultant's in-house soil testing laboratory and revealed the following average properties for use in the sand drain design.

void ratio, e = 1.72

coefficient of (vertical) hydraulic conductivity, k_v = 0.508 × 10-8 cm/sec

coefficient of (vertical) consolidation, c_v = 0.0405 cm^2/min

"assumed" coefficient of (horizontal) consolidation, c_h= 0.081 cm^2/min; i.e., $c_h = 2\ c_v$

2.3.4 Random Case Histories throughout the Entire World

1) Broid and Melnik discuss the effectiveness of jet grouting in the densification of voids and loose sand backfill behind a sheet pile wharf in their paper "Use of Jet Grouting Method for Elimination. The phenomena of suffusional destruction is described as the progressive loosening of soil Is and the formation of voids within sheet pile backfill. It is an occurrence similar to that of piping through a dam or raveling of soils into bedrock openings in karst geology.

2) "Ground Improvement at the Queensway Bay Downtown Harbor, Long Beach. California ", Somasundaram, Weeratunga and Khilnani discuss the use of stone columns for a harbor/marina project in Souuthern California, USA. The development area is underlain by near surface dense sand fills. loose to medium dense hydraulic fills and sea floor sediments with interbedded low plasticity silt and clay layers.

3) "A Non-Destructive Testing Program for a Group of Jet Grouting Columns", Varosio and 0' Appolonia report the effectiveness of different non-destructive test methods in evaluating the as-built characteristics of jet-grouted columns.

4) Gupta, Kumar and Tolia in their paper Lime Slurry Injection, Lime Piles and Stone Columns for Improvement of Soft Soils - Field Trials". In the first case history, stone columns are used to stabilize soft soils beneath a iron ore loading dock in southeast India. Stockpiling of iron ore up to 9 m in height resulted in large settlements and heave of the underlying soft clay soils.

5) '"Field Experiments on Jute Soil Stabilizers", Gupta, Yadav and Bhagwan describe seven experimental projects across India where jute geotextiles are being used. Jute based geotextiles are touted as a low cost alternative to synthetic geotextiles and can be used in a number of applications including slope stabilization, filtration, ground improvement and erosion control. Due to the fact that jute is biodegradable, primarily temporary applications should be considered, though permanent filtration and separation are feasible with jute.

6) '"Grouting Evaluation Program of the Rest Aiethods for Use of Microfine and Portland Cements During Treatment of the Rock Foundation at the Portugues Dam", Conway and Novak investigate the results of a test grouting program for the rock foundation of a large concrete dam structure in Puerto Rico.

7) "Performance Prediction and Uses of PV Band Drains Under the Embankments on Soft Marine Clays of Bangkok", Mukherjee compares the settlements and excess pore pressures recorded at a test embankment on soft clays to the results of a numerical modeling analysis. The author dedicates much of his paper to an excellent and detailed discussion of the historical development of vertical drain theories and a review of the more recent theories. The experimental test site is covered by up to 8 m of highly compressible soft clay deposits underlain by stiff clay and dense sand layers.

Chapter 3

Design Methodology for Sand Drains and Prefabricated Vertical Drains

3.1 Sand Drain Introduction

Sand drain is a process of radial consolidation which increase rate of drainage in the rate of drainage in the embankment by driving a casing into the embankment and making vertical bore holes. These holes are back filled with suitable grade of sand.

3.2 Methodologies for Sand Drains

The basic concept of vertical drain is installation of additional drainage channels in low permeability soil to reduce for dissipation of excess pre-pressure, Barren in 1947 proposed an equation for consolidation of a cylinder containing a central drain with assumption of equal vertical strain.

$$U_h=1-\exp(-8^{Th}/F(n))$$

T_h= Non-dimensional time factor= $C_h t/d_e^2$

C_h= Co-efficient of permeability for horizontal flow

t = Time after an instantaneous increase of the total vertical stress

d_e= Diameter of the drained soil cylinder subjected to radial flow

$F(n)= (n^2/n^2-1)\ln(n)-(3n^2-1/4n^2)$

$N= d_e/d_w$= ratio of equivalent diameter of influence zone to drain diameter

The r_e expression for a triangular pattern is

$$R_e= S*[3^{1/2}/2\pi]^{1/2}= 0.525S; \text{ where } S= \text{spacing}$$

For a square spacing,

$$R_e= S/(\pi)^{1/2}= 0.564S$$

Another important practical encounter is the effect of smear. This is relatively thin layer of soil remolded during installation of the pile. Let the horizontal permeability of the smeared zone of outer radius r_s be K_s. For the equal strain case, the solution for excess pore water is

$$U= u/v*[\log_e(r/r_s)-(r^2-r_s^2)/2r_e^2+K_h/K_s*(n^2-s^2/n^2)\log_e(s)]$$

Where the average excess pore pressure u is given by

$$u= u_0\exp(-8T_r/v)$$

and hence giving the data for the degree of settlement time factor curve where

$$v=n^2/(n^2-s^2)* \log_e(n/s)-3/4+s^2/4n^2+k_h/k_s*(n^2-s^2/n^2) \log_e(s)$$

and $S= r_s/r_w$

Scott (1963) recognized the expression and presented the solution. The expression for the degree of settlement is

$$U= 1-\exp(-8T_r/m)$$

Where

$$m= [n^2/(n^2-s^2)\log_e n-(3n^2-1)/4n^2+k_h/(r_w*k)*(n^2-1)/n^2]$$

3.2.1 Process of Construction of Sand Drains

The driven casing is withdrawn after the sand has been filled. A sand blanket is placed over the top of the sand drains to connect all the sand drains. To accelerate the drainage, a surcharge load is placed on the sand blanket. The surcharge is usually in the form of dumped soil.

3.2.2 Mechanism of Consolidation

Pore water is increased by the applied surcharge load in the embankment. The drainage occurs in the vertical and horizontal directions. The horizontal drainage occurs because of sand drains. The sand drains accelerate the process of dissipation of excess pore water created by the surcharge. The drains are generally laid either in a square pattern or a triangular pattern. The spacing (s) of the drains is kept smaller than the thickness of the embankment (2H) in order to reduce the length of the radial drainage path. In the figure 3.1 spacing of drains is indicated and figure 3.2 it is pointed about the typical sand drain installation.

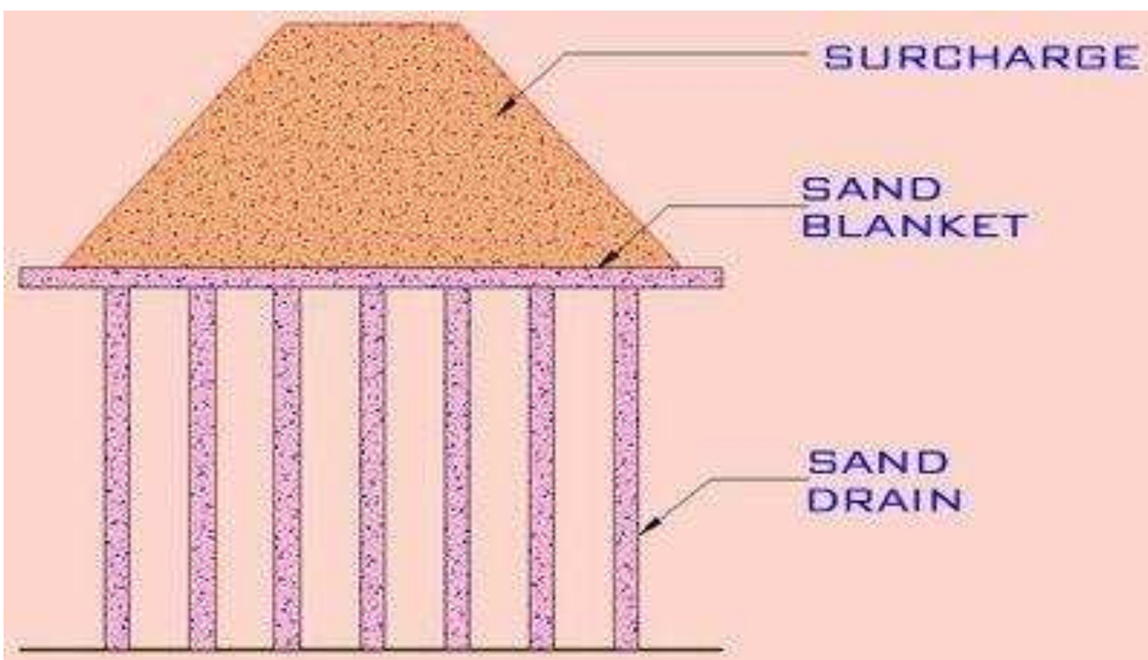


Figure 3.1: Spacing of Drains

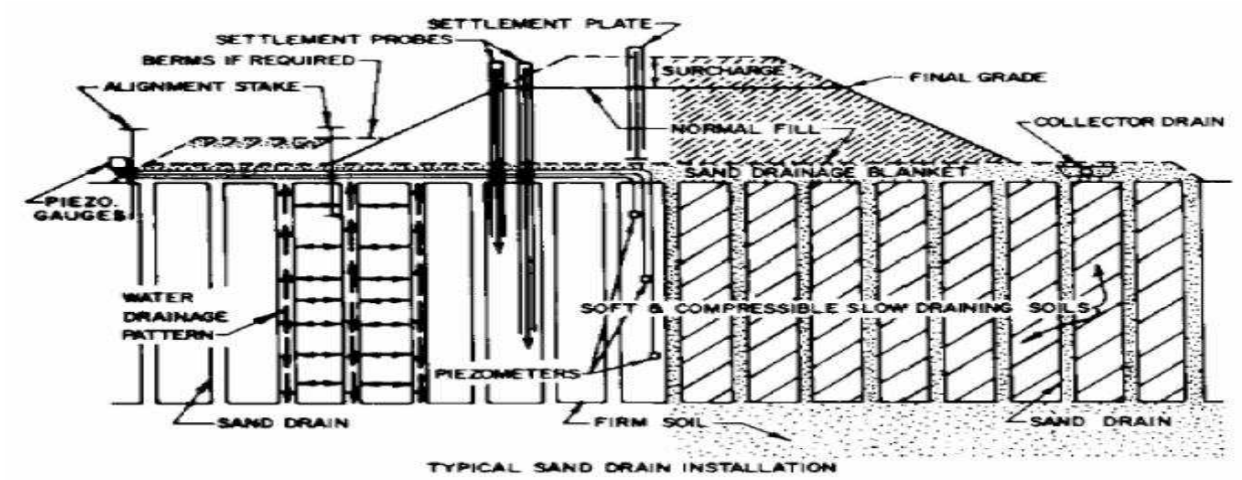


Figure 3.2: Typical Sand Drain Installation

3.2.3 Zone of Influence

The zone of influence of each drain in a triangular pattern is hexagonal in plan, which can be approximated by an equivalent circle of radius R, where R= 0.525S (in figure 3.3). In case of a square pattern, the radius of circle of influence R= 0.564S (in figure 3.3). The radius of the sand drain is presented by r_w.

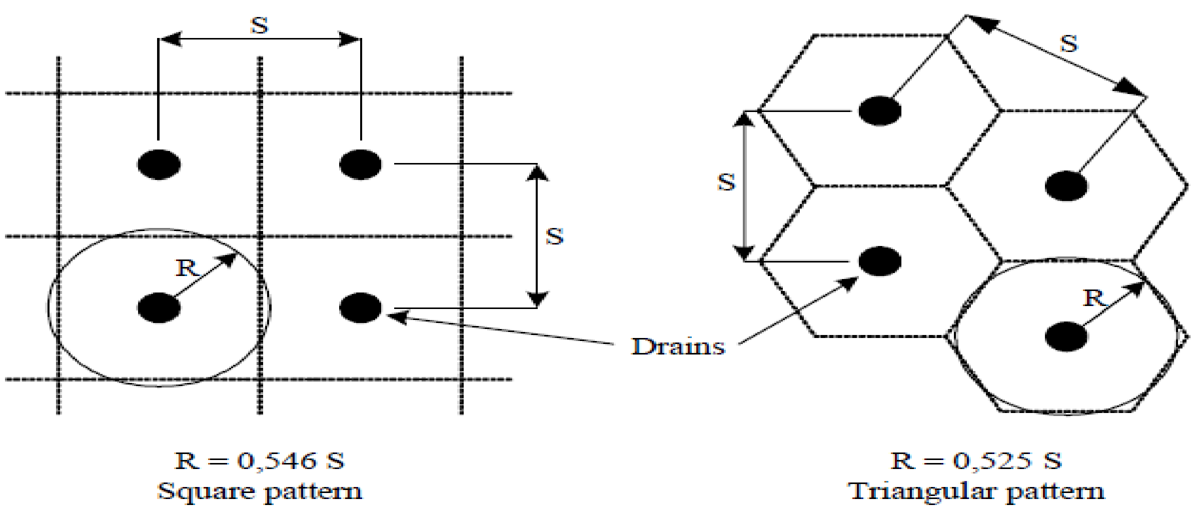


Figure 3.3: Sand Drain of Square and Triangular Pattern

3.2.4 Smear Effect

The installation of vertical drains by means of a mandrel causes significant remolding of the subsoil, especially adjacent to the mandrel. Hence, a zone of smear will be developed with reduced permeability and increased compressibility. In varved soils the finer and more impervious layers will be dragged down and smeared over the more pervious layers (Barron, 1948). The smear zone creates additional resistance which must be overcome by the excess water. This, in turn, will retard the rate of consolidation. The behavior of permeability and compressibility within the smear zone is different than the behavior of the undisturbed soil, hence, the behavior of soil stabilized with vertical drains cannot be predicted accurately if the effect of smear is ignored. Both Barron (1948) and Hansbo (1981) modeled the smear zone by dividing the soil cylinder dewatered by the central drain into two zones. The smear zone is the zone in the immediate vicinity of the drain and the other is the undisturbed zone. In figure 3.4 smear effect is shown.

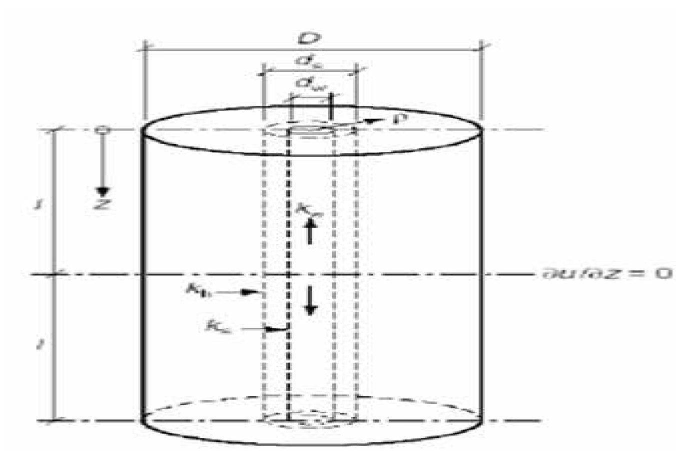


Figure 3.4: Smear Effect

3.2.5 Theory of Sand Drains

The basic theory of radial consolidation around a vertical drain system is an extension of the classical one-dimensional consolidation theory. Barron (1948) studied the two extreme cases of free strain and equal strain and showed that the average consolidation obtained in both cases are nearly the same. The “free strain hypothesis” assumes that the load is uniform over a circular zone of influence for each vertical drain, and that the differential settlements occurring over this zone have no effect on the redistribution of stresses by arching of the fill load. The “equal strain hypothesis” on the other hand assumes that the load applied is rigid and equal vertical displacement in enforced at the surface, i.e. horizontal sections remain horizontal. The solution for the second case is considerably simpler (Barron 1948).

3.2.6 Equal Vertical Strain Hypothesis (Barron, 1948)

Barron developed a solution of the horizontal consolidation under ideal conditions using an asymmetric unit cell model. The solution is based on the following assumptions:

1) All vertical load are initially carried by excess pore pressure, thus the soil is saturated.
2) The applied load is assumed to be uniformly distributed and all strains occur in vertical direction.
3) The zone of influence of the drain is assumed to be circular and axisymmetric.
4) The permeability of the drain is infinite in comparison with that of the soil.
5) Darcy’s law is valid.

3.2.7 Limitations of Sand Drain Application

Following considerations are not included in design of sand drains.

i. Secondary consolidation is not taken into account in the design of sand drains. In fact, the sand drains are ineffective in controlling the secondary consolidation for highly plastic and organic soils.
ii. In case of deriving equation for effectiveness of sand drains, it is not considered that that excess pore water pressure developed, actually in soil where sand drains are exist, is generally less than that of the case having no sand drains. Sand drains tend to act as weak piles and reduce the stresses in the clay.
iii. The typical design parameter for sand drain may vary as below:

1) Spacing of sand drains, S = (2-5) m
2) Depth of sand drains, 2H = (3-35) m
3) Radius of sand drains well, r_w = (0.2-0.3) m
4) Thickness of sand blanket = (0.6-1) m

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

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

vi. During filling bulking of the sand might appear which could lead to cavities and subsequently to collapse due to flooding.

vii. Construction problems and/or budgetary burdens might arise due to the large diameter of sand drains. The disturbance of the soil surrounding each drain caused by installation may reduce the permeability, the flow of water of water to the drain and thus the efficiency of the system.

viii. The reinforcing effect of sand drains may reduce the effectiveness of preloading the subsoil.

3.3 Design Methodologies for Prefabricated Vertical Drains

3.3.1 Overview

Prefabricated vertical drains (PVDs), also commonly called wick drains, are used to accelerate the consolidation and strength gain of compressible soils with low or restricted permeability. The use of the term wick drains is a misnomer since water is not wicked out of the ground by the drains under capillary tension, but rather water flows into the vertical drains from a compressible soil layer experiencing a temporary water pressure gradient induced by placement of permanent fill and or a temporary surcharge fill. In the figure 3.5 application of pvd with surcharge loading for ground improvement and in the figure 3.6 consolidation with and without pvd have been showed.

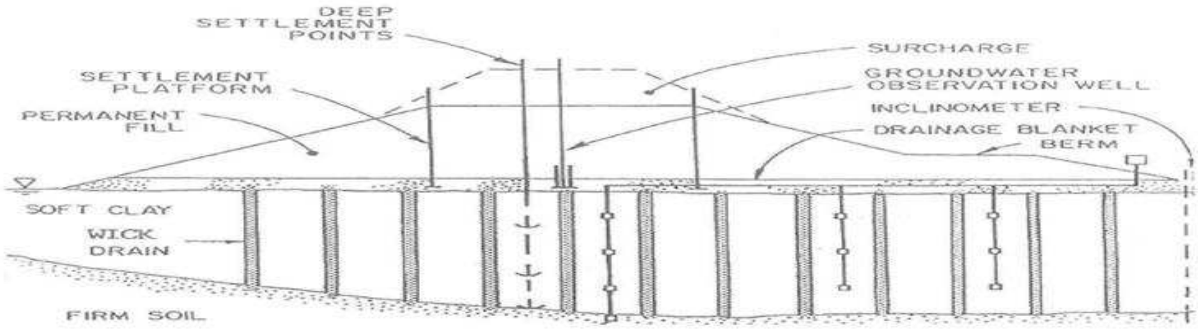


Figure 3.5: Application of PVD with Surcharge Loading for Ground Improvement

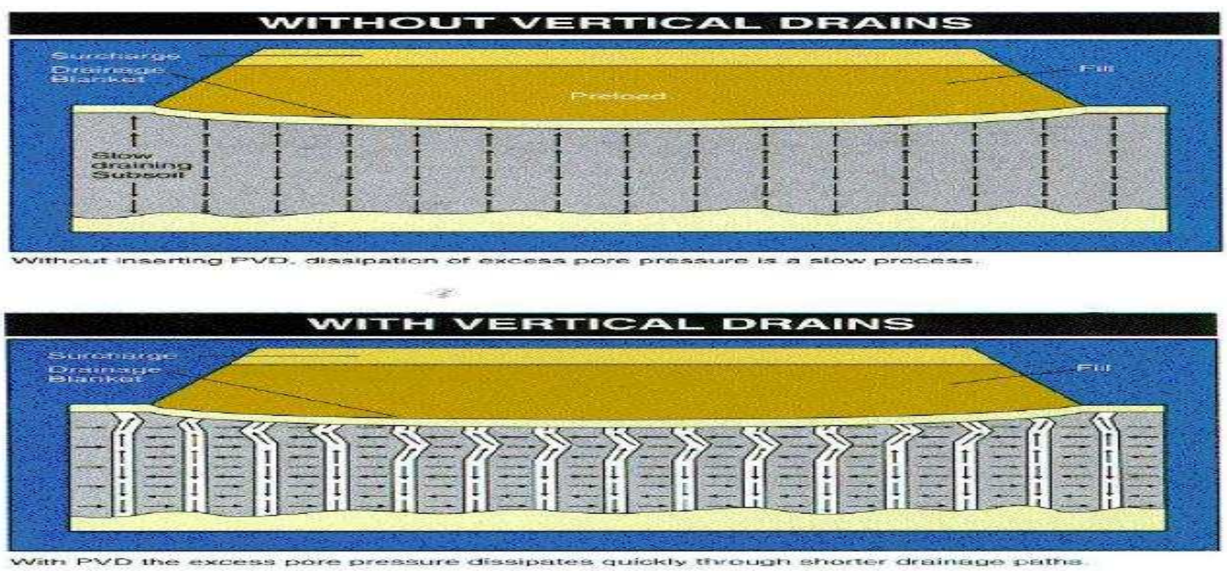


Figure 3.6: Consolidation with and without PVD

3.3.2 Characteristics of PVD

PVD is a prefabricated material consisting of a plastic core covered by synthetic “filter jacket” (Typically 95-100 mm wide by 3-5 mm thick).

Two main components of PVD serve the following functions;

1) Core serves as a longitudinal flow path along the drain.
2) Filter jacket allows water to pass into the core while restricting intrusion of soil particles.

3.3.3 Discharge Capacity

i. Maximum flow observed from PVD= $5x10^{-6}$ m^3/s=158 m^3/year. Hydraulic gradient approximately 0.1

ii. Reduction in discharge capacity from :

1) Deformation and creep of filter into core.
2) Permeability reduction due to clogging of filter and core.
3) Bending and kinking of PVD during settlement.
4) Pressure on PVD.

iii. q_w= Q / i Darcy's Law (Valid for laminar flow only)

Where $q_w \geq 140 \text{ X } 10^{-6}$ m^3/s from test ASTM D4716

3.3.4 Kjellman Formula

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

Where t= consolidation period

D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain

C_h= horizontal consolidation co-efficient (m^2/year)

U= average horizontal consolidation degree

d= diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness

z= distance to the flow point (m)

I= drain length at unilateral flow (m) (half length for bilateral flow)

k_c=permeability of the soil (m/s) q_w=discharge capacity of the drain

3.3.5 Influence Zone

PVDs are installed in either square or triangular patters. A square pattern is simpler fro setting out in the field. Triangular pattern is however provides more uniform consolidation between drains. Relationship of drain influence zone (D) to drain spacing (S) can be expressed by

D=1.13S (square) (in figure 3.7)

D=1.05S (triangular) (in figure 3.7)

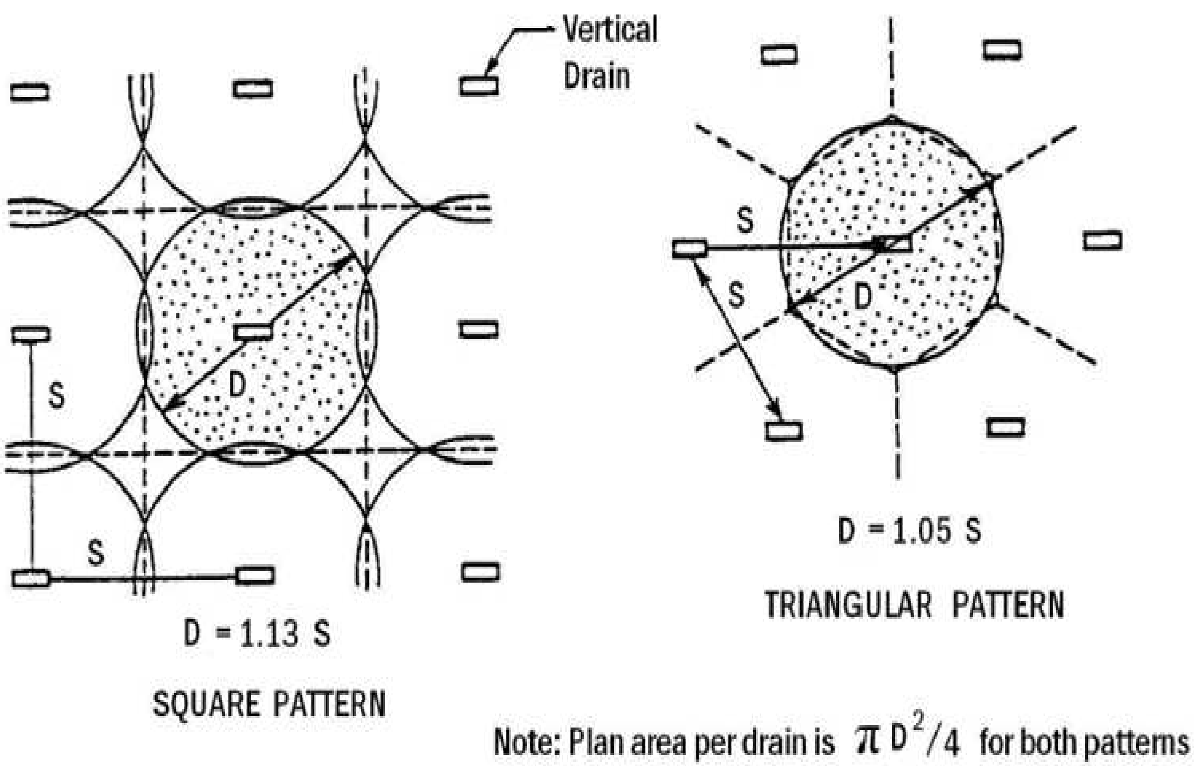


Figure 3.7: Typical PVD Layouts and Drainage Influence

3.3.6 Sample types of Vertical Drains

Figure 3.8 shows the vertical drain configuration to install sand drain and pvd.

Drain type	Installation method	Drain diameter [m]	Typical spacing [m]	Maximum length [m]
Sand drain	Driven or vibratory closed-end mandrel (displacement type)	0.15 - 0.6	1 - 5	≤ 30
Sand drain	Hollow stem continuous-flight auger (low displacement)	0.3 - 0.5	2 - 5	≤ 35
Sand drain	Jetted (non-displacement)	0.2 - 0.3	2 - 5	≤ 30
Prefabricated sand drains ("sandwicks")	Driven or vibratory closed-end mandrel; flight auger; rotary wash boring (displacement or non-displacement)	0.06 - 0.15	1.2 - 4	≤ 30
Prefabricated band-shaped drains	Driven or vibratory closed-end mandrel (displacement or low displacement)	0.05 - 0.1 (equivalent diameter)	1.2 - 3.5	≤ 60

Figure 3.8: Vertical Drain Configurations

3.3.7 Wick Drain Installation

As noted in the Holtz, et al (1991) book, the initial attempts at wick drains were first described in the literature by Kjellman (1948) and were actually known as "Kjellman Wick Drains". They are referred to as "wick drains" in this paper but it should be mentioned that they are called PVDs in most countries except in North America where they continue to be called wick drains. They were originally made from "non treated, then treated" corrugated cardboard, however, both types rapidly deteriorated when in the saturated soil. Wager (1971) was the first to use a grooved plastic core encased in a paper filter. Subsequently, nonwoven heat-bonded geotextiles encasing plastic cores of many different shapes and configurations are available. All, however, are approximately 100 mm wide with thicknesses ranging according to the type of core (8-15 mm). Wick drains arrive at a site in rolls and are placed on the relatively light installation rig in dispensers like a huge roll of toilet paper. The end is threaded down inside a hollow steel lance, which must be as long as the depth to which the wick drains are to be installed. As it emerges from the bottom of the lance, the wick drain is folded around a steel bar or base plate. The base plate is preferred so as to keep the wick drain down at the bottom of the lance and at the same time to keep the soft soil through which it will be installed out of the lower portion of the lance so that the drain properly releases when the lance is withdrawn. The entire assembly (lance, base plate, and wick drain) is now pressed into the ground to the desired depth. If a hard crust of soil or a high strength geotextile or geogrid is at the original ground surface, it must be pre-augered or suitably pierced beforehand. When it reaches the desired depth, the lance is withdrawn, leaving the base plate and wick drain behind. The rig moves and the process is repeated at the next location. It is a very rapid construction cycle (approximately 1 min), requiring no other materials than the wick drains and base plates. At the ground surface the ends of the wick drains (typically at 1 to 2 m spacing) are interconnected by a granular soil drainage blanket or a geo composite sheet drain layer. There are a number of commercially available wick drain manufacturers and installation contractors who readily provide information on the current products, styles, properties, and estimated costs. Concerning the design method for determining wick drain spacings, the initial focal point is on the time for consolidation of the subsoil to occur. Generally, the time for 90% consolidation is desired, but other values might also be of interest. Two approaches to such a design are possible. The first is an equivalent sand drain approach that uses the wick drain to estimate an equivalent sand drain diameter and then proceeds with design in the manner of sand drains. This is done by taking the actual cross-sectional area of the candidate wick drain and making it into an open void circle. This open void circle is then increased using the estimated porosity of sand to obtain the equivalent sand drain diameter.

Following the initial set up and feeding of the wick drain material, the drains are installed by pushing a hollow, steel mandrel, typically rectangular or rhombic in section into the ground. The mandrel houses the wick material and protects it from damage as the mandrel is inserted into the ground to the termination depth. At the base of the mandrel, the wick material is looped through an anchor which holds the drain securely in place as the mandrel is extracted. Once the mandrel has been extracted from the ground, the wick drain is cut and the next drain is installed. In some sites, production rates greater than one drain per minute can be achieved.

Wick drain installation units are typically powered and supported by large crawler excavators or by cranes, depending on the depth of the drain and weight of the installation unit. Pull down is typically accomplished by heavy chain, cable, or gear systems. Depending on the subsurface conditions, the mandrel may be vibrated or stoically pushed into the ground. The mandrel cross-sectional area is typically 10-12 square inches. Wick drain center-to-center spacing is usually in the range of 3-8 feet. The drains are typically laid out in a triangular grid pattern, less commonly in square grids and occasionally in rectangular or irregular grid patterns. The majority of applications are between 15 and 120 feet deep; however, drains have been installed to depths over 150 feet. In the figure 3.9 it is pointed about the wick drain, figure 3.10 shows about installation process of wick drain and figure 3.11 shows about installation of pvd in practical sites.

Figure 3.9: Wick Drain

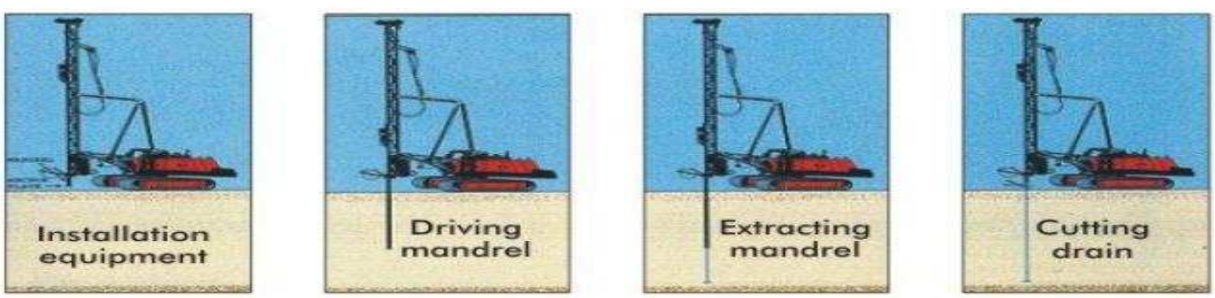


Figure 3.10: Installation Process of Wick Drain

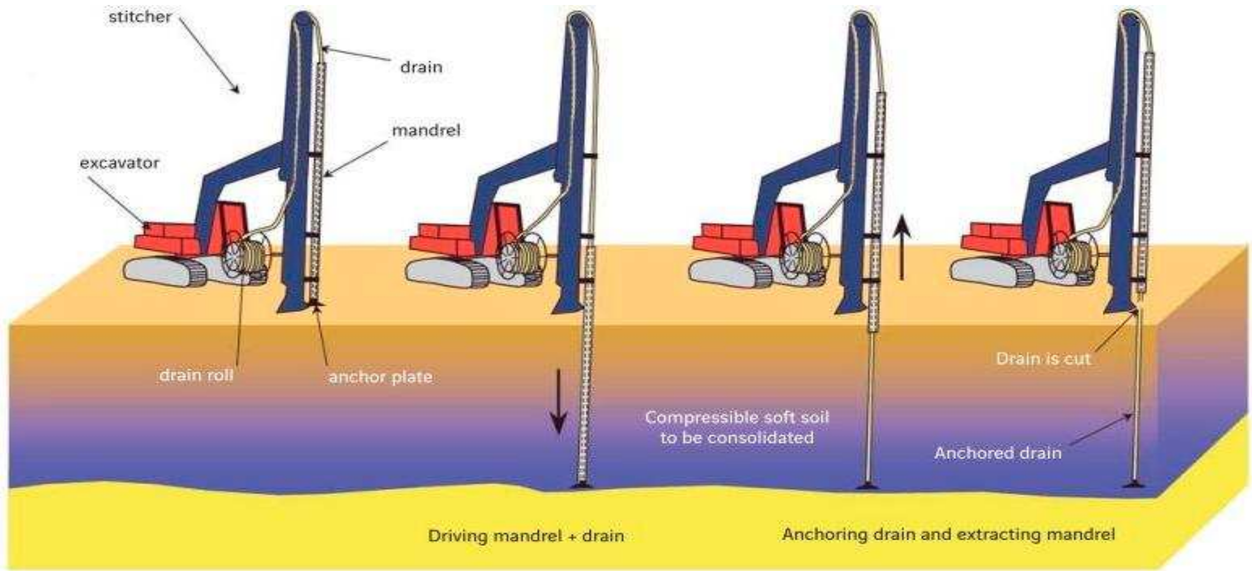


Figure 3.11: Installation of PVD

3.3.8 Applications of PVD

1) Construction of road, railway, embankment, airport and ports
2) Industrial projects
3) Land reclamation projects

3.3.9 Advantages of PVD

1) It decreases the overall time required for completion of primary consolidation.
2) It decreases the amount of surcharge load required to achieve the same desired amount of consolidation over a given period of time,
3) It increases the rate of strength gain due to consolidation of soft soils when short-term stability is of concern.

3.3.10 Advantages and Disadvantages of PVD versus Sand Drains

Advantages of PVD versus sand drains:

1) Fewer environmental problems concerning disposal of spoil materials.
2) Eliminates high cost of sand backfill of drains and material quality control problems.
3) Inspection requirements are reduced due to simplicity of installation procedures.
4) There is greater assurance of a continuous vertical drainage path (no collapsed holes).
5) PV drains can withstand considerable lateral displacement or buckling under vertical or horizontal soil movements.
6) Faster rate of installation.

Disadvantages of PVD versus sand drains:

1) Greater number of wick drains required to achieve same rate of drainage.
2) Do not provide short term improvement of target soil's compressive strength
3) Headroom limitations (typical equipment is 10 ft taller than wick drain depth).

4) Wick drains must be protected from sun light and large tears. Wick drains should not be used for long term artesian flows.
5) Equipment mobilization costs may be very high.
6) Wick drains over 60 feet deep require very tall or specialized equipment.
7) Sand drains can be designed to accommodate long term artesian conditions and therefore function as pressure relief wells.

3.3.11 Feasibility of Prefabricated Vertical Drain

The site conditions pertaining to the soil layer to be treated must be evaluated to determine the feasibility for using PVD's. The following factors, relating to the target soil layer or ground surface, are favorable for their use:

1) Moderate to high compressibility potential.

2) Low or restricted permeability.

3) The time to achieve at least 85% of primary settlement, without use of vertical drains, will result in excessive construction delays.

4) Full saturation of target soil layer.

5) Final embankment or temporary surcharge load increase, at the depth of the target soil layer, must exceed the maximum pre-consolidation stress (σ'p or p'c).

6) Secondary compression will not be significant.

7) Initial Low-to-moderate shear strength.

8) Soils normally to slightly over-consolidated ($OCR < 1.5$).

9) Very minimal obstructions (cobbles, boulders, etc) within or above target soil layer

10) Site allows relatively flat working surface with at least a 25 foot bench width.

11) Long term artesian conditions are not likely.

3.3.12 Equipment

PVD installation equipment can be developed to suit the soil condition, installation depth, specified scope of work and required production rate. Technical data of typical medium-sized PVD installation equipment and accessories are shown below.

3.3.12.1 Installation Rig

Types of Base machine : Excavator of suitable model; CAT EL200B or larger model

Size (CAT EL200B) : 3.18 x 4.45 m

Weight (CAT EL200B) : 20 ton

Pushing Force : 5.5-20 ton

Mandrel Lifting and Pushing : Hydraulic gear drive

Mast Height : 8 m

3.3.12.2 Typical Mandrel Dimensions

Weight of Guide and Mandrel : 1.5-4 ton

Length of Mandrel : 12-20 m

Cross-sectional area of Mandrel : 60-70 cm^2

Maximum Installation Depth : 11-19 m

Installation Rig, Drain Delivery Arrangement and Cross-section of Mandrel and Drain are shown in figure 3.12.

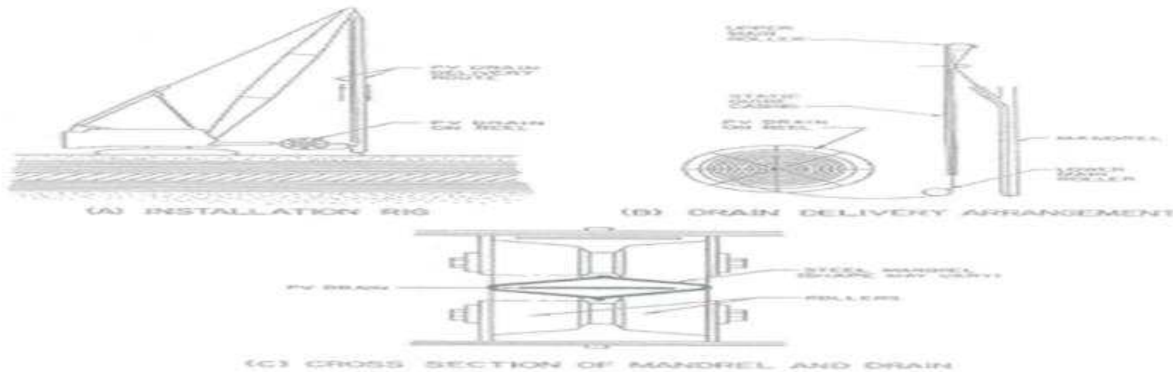


Figure 3.12: Installation Rig, Drain Delivery Arrangement and Cross-section of Mandrel and Drain

3.4 Stability Analysis of Embankment

Before the design of vertical drain, stability analysis of the embankment on unimproved soft ground have been carried out using the method proposed by Low (1989). Embankments constructed on soft clay foundations typically have potential failure mode in the form of an approximately circular slip surface extending into the soft foundation. An infinite number of slip circles all tangential to a given trial limiting tangent can be drawn. In another trial limiting tangent, again there will be another infinite number of possible slip circles. Among all these possible slip circles passing through the soft foundation, the critical circle has the lowest factor of safety. Failure, if it occurs, will follow the path along which the factor of safety is the minimum. This path is represented by the critical slip circle. In the figure 3.13 it is indicated about the stability analysis of embankment by Low's method.

The method proposed by Low (1989) is simple and convenient semi-analytical procedure to calculate the factor of safety of embankments constructed on soft clay. Stability numbers N_1 and N_2 are developed for the normalized foundation strength and normalized embankment strength respectively. The factor of safety has been computed as the sum of two components, each equal to the product of the respective stability number and the corresponding normalized strength. This method assumes short-term undrained response of the soft clay foundation. It can deal with cases where the undrained shear strength of the soft clay varies with depth, and where the embankment possesses both cohesion and internal friction.

It has been assumed in all the stability analyses that the computed factor of safety at any depth within the soft soil layer should be at least 1.2 to ensure adequate safety of the whole soft soil layer against bearing capacity failure. Chowdhury et al. (1978) reports following acceptable values of factor of safety for all situations.

1) For 100% probability of failure , F.S. =0.8

2) For 50% probability of failure , F.S. =1

3) For 10% probability of failure , F.S. =1.2

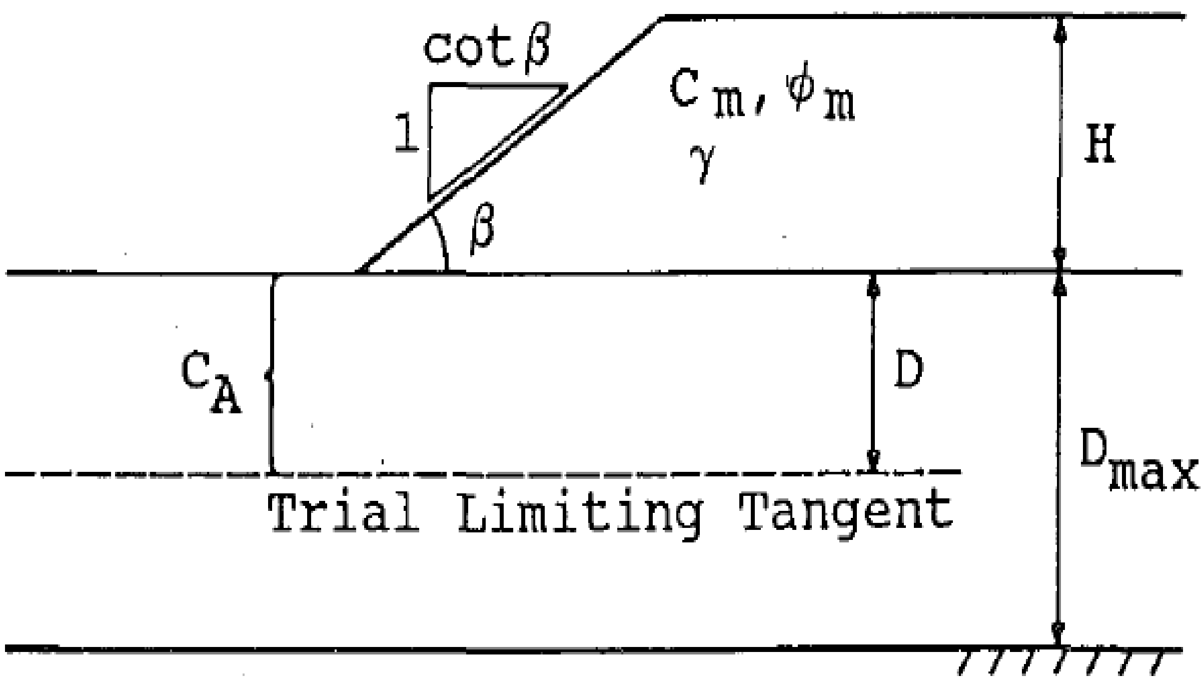
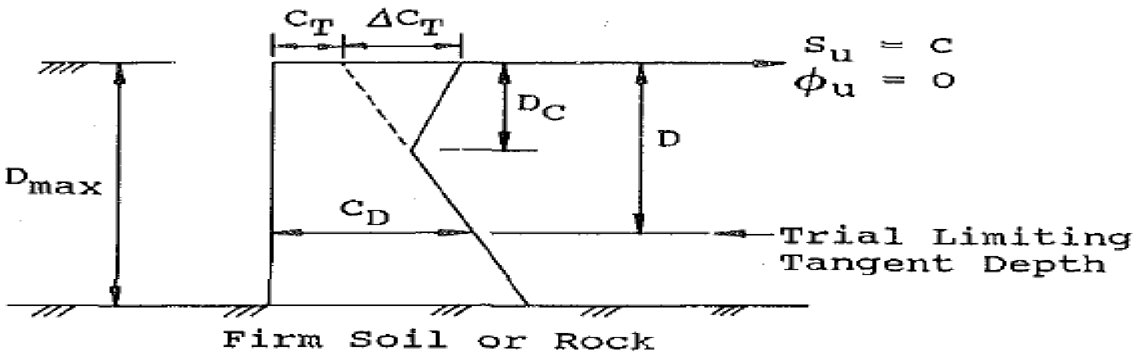


Figure 3.13 Stability Analysis of Embankment by Low’s Formula



C_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$C_A = 0.35\ C_T + 0.65\ C_D$, for $\Delta C_T = 0$.

$C_A = 0.35\ C_T + 0.65\ C_D + 0.35\ \left(\frac{D_C}{D}\right)^{1.1} \Delta C_T$, for $D > D_C$

$C_A = 0.35\ (C_T + \Delta C_T) + 0.65\ C_D$, for $D \leq D_C$

Both C_T and ΔC_T can be positive (as shown), or negative.

$$(F_s)_D = N_1 \frac{C_A}{\gamma H} + N_2 \cdot \left(\frac{C_m}{\gamma H} + \lambda \tan \phi_m\right)$$

where $N_1 = 3.06 \left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 \left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53} \right] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 \left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303 \frac{\cot^2\beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 \left(\frac{D}{H}+\frac{1}{2}\right) - \frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2 - \frac{1}{24}(\cot^2\beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02 \cot \beta}{D/H}, \text{ or Fig. 4}$$

(For $D/H \geq 0.5$)

3.4.1 Increase in Shear Strength due to Embankment Loading

The increase in undrained shear strength (ΔC_U) due to stage loading of the embankment has been calculated using the following equation:

$\Delta C_U = \tan \phi' \, U \, \sigma$

Where ϕ' = effective angle of internal friction (In figure 3.14 effective angle of internal friction vs Plasticity index for Normally Consolidated Clays is shown)

σ = vertical stress at the base of the embankment

U = percent consolidation

$\sigma = \gamma \, h \, (B/B')$

where, γ = unit weight of embankment soil

h= height of embankment

(B/B')= ratio of width of embankment at crest and at base

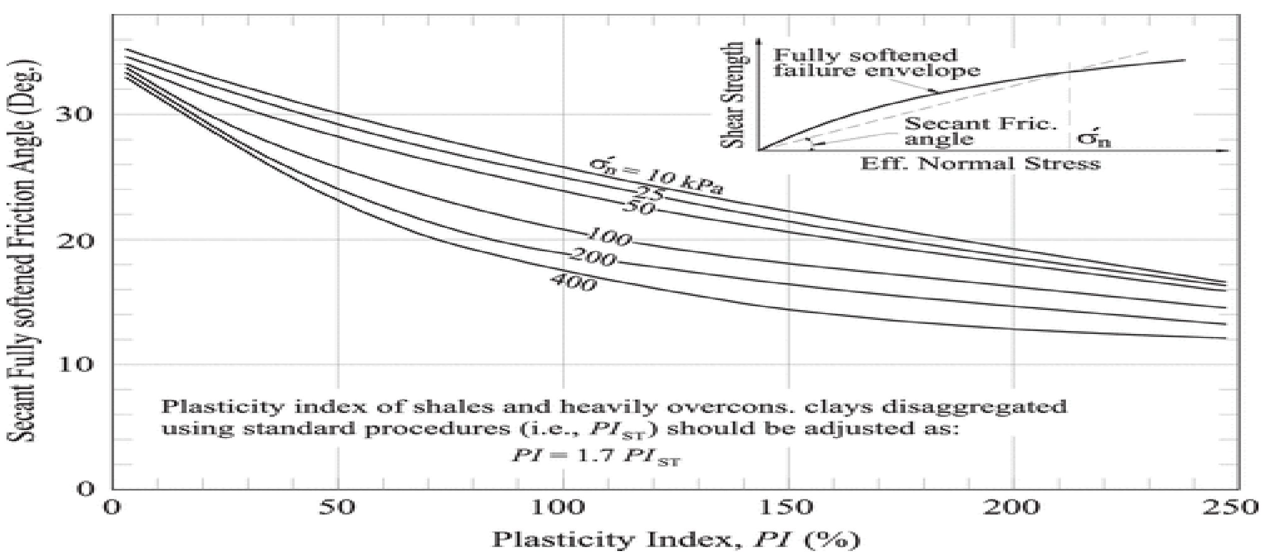


Figure 3.14: Effective Angle of Internal Friction vs Plasticity Index for Normally Consolidated Clays

3.5 Amount of Consolidation Settlement at Centre and Toe of Embankment and Time for Consolidation Settlement

Consolidation settlement always occurs in fine-grained soil, e.g. clays and silts. Embankment loading when applied on a layer of soft soil, the compression causes the soil to reduce in volume by expelling water from the pore spaces of soil. This process is very much dependent on the values of co-efficient of consolidation for vertical flow (C_v) and co-efficient of permeability (k). The consolidation settlement (S_t) of soft soil layers below the centre of embankment fill has been calculated using the following equations:

$S_c = \Delta eH$

$\Delta e= C_c / (1+e_0) \log (\Delta p+p_0 ')/p_0'$

$\Delta p= 2I_zq$

$p_0'= \gamma' h$

$I_z = 1/\pi [\tan^{-1} (b/z) + (1+b/a) \{ \tan^{-1} (a/z+b/z)- \tan^{-1}(b/z)\}]$

For toe of embankment, $I_z = I_{z1} + I_{z2}$

$I_{z1} = [\tan^{-1} (a/z+b/z)- \tan^{-1}(a/z) +z(a+b)/(z^2+(a+b)^2]/\pi$

$I_{z2} = [\tan^{-1} (a/z+2b/z)- \tan^{-1} (a/z+b/z) + 2((a+b)/a) * \{ \tan^{-1} 2(a/z+b/z)- \tan^{-1} (a/z+2b/z)-(a+b)/ (z^2+(a+b)^2)]/\pi$

Where, Δe= change in void ratio due to change in Δp

H= thickness of soil layer

Δp= change in total vertical stress at the centre of layer

I_z = Influence factor for centre of embankment

I_{z1},I_{z2} = Influence factors for toe of embankment q= increase in vertical stress at the base of embankment

p_0'= effective vertical stress at the centre of layer

Time for consolidation settlement of soft soil layer, t= $T_v H^2 / C_V$

Where, T_v = time factor

H = length of drainage path

C_V = co-efficient of consolidation for vertical flow

Dimensions of Embankment used for Computations of Influence Factor below the Centre and Toe of Embankment is shown in the figure 3.15.

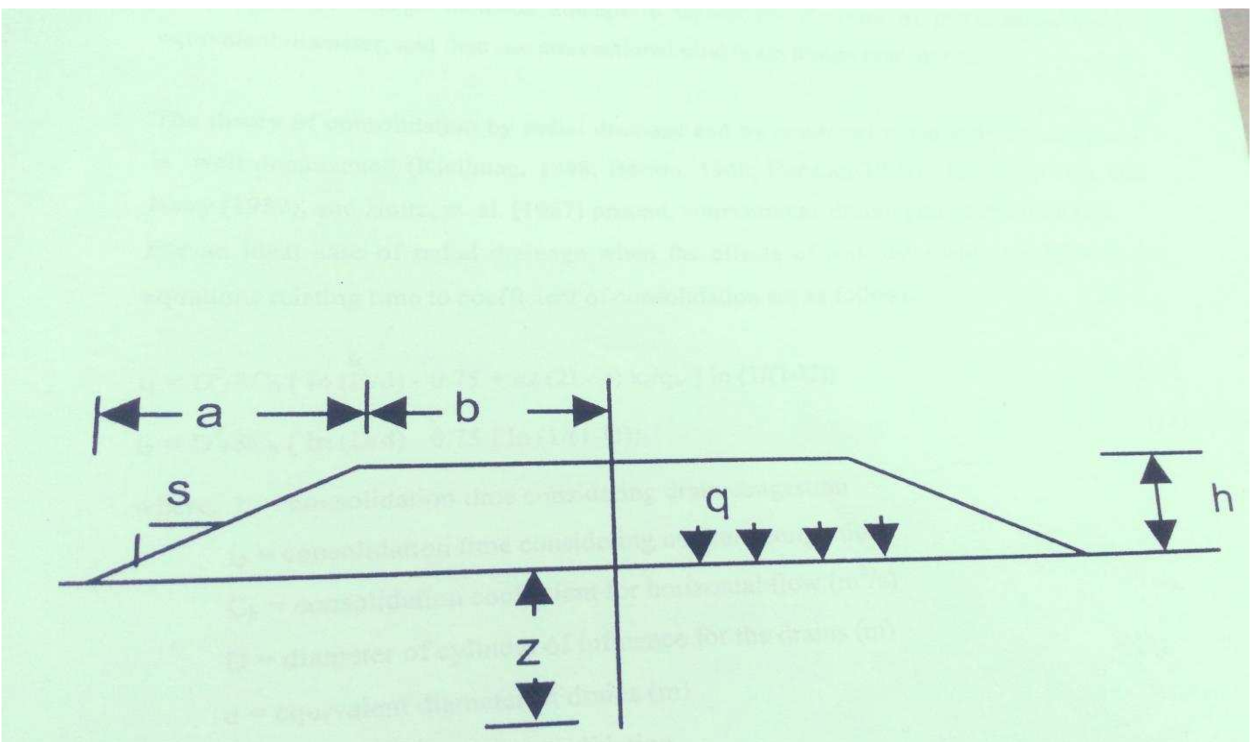


Figure 3.15: Dimensions of Embankment used for Computations of Influence Factor below the Centre and Toe of Embankment

Chapter 4

Analysis and Design of Sand Drains and Prefabricated Vertical Drains

4.1 Summary of Geotechnical Investigations

A comprehensive field investigation was carried out at the proposed Khan Jahan Ali Airport site at Bagerhat. So from the investigation it can be summarized as

1) The subsoil of 4000 ft length of runway from the north end consists of soft silt clay and clayey silt and loose clayey silt up to 7-8 m below the existing ground level.
2) From the runway areas, C_c has been found in the range of 0.19-0.22, e_0 has been found in the range of 0.95-1.03, C_v has been found in the range of 12-35 m^2/yr , C_h has been found in the range of 14-69 m^2/yr and k_v, k_c have to be found very similar, around 3.5×10^{-10} – 1.3×10^{-8} m/sec.
3) The samples obtained from the boreholes drilled in the runway are typically soft and soft to firm in consistency.
4) Depending on the thickness of soft layers, consolidation settlement times corresponding to 70% consolidation have been found 61-157 days and corresponding to 90% consolidation have been found 129-330 days. (Appendix A)
5) The positions of ground water table vary between 0.3 m and 1.5 m below EGL.
6) Specific gravity of subsoil have been found between 2.69 and 2.80.
7) Values of unconfined compressive strength of the samples obtained from the boreholes drilled in the runway area varied between 47.6-97.4 kN/m^2.
8) The LL and PI have been found successively between 37 and 53 and between 8 and 25.
9) The values of C_u and φ_u of the sample obtained from consolidated undrained triaxial compression test have been found to be 5 kN/m^2 and 12°.
10) The values of C_u of the samples obtained from consolidated undrained direct shear tests varied between 15 kN/m^2 and 20 kN/m^2 and φ_u was 30°.

4.2 Soil Improvement of the Runway Pavement

As the airport will be a domestic airport, 100kPa of surcharge is assumed on the runway, so height of embankment is needed about 6 m.

The increase in undrained shear strength (ΔC_U) due to stage loading of the embankment has been calculated using the following equation (Appendix B):

$\sigma = \gamma\ h\ (B/B')$

where γ = unit weight of embankment soil= 18 kN/m^3

h= height of embankment= 6 m

(B/B')= ratio of width of embankment at crest and at base= 45 m/ 225m (according to code of airport pavement= 0.2

$\Delta C_U = \tan \phi'\ U\ \sigma = 9.07$

Where ϕ' = effective angle of internal friction=25°

σ = vertical stress at the base of the embankment

U = percent consolidation = 90%

4.2.1 Properties of Embankment Fill

Height of embankment= 6 m

Undrained Shear Strength of Fill, C_m =30 kN/m^2

Undrained angle of internal friction, φ_m = 10°

Unit weight of fill, Υ_m = 18 kN/m^2

4.2.2 Properties of Soft Soil Layer

Maximum thickness= 8 m

Undrained Shear Strength= 10 kN/m^2 (at 1 m depth)

Undrained Shear Strength= 15 kN/m^2 (at 8 m depth)

Saturated Unit Weight of soil= 18 kN/m^3

Submerged Unit Weight of soil= 8 kN/m^3

Initial void ratio= 1

Compression index= 0.2

Co-efficient of consolidation for vertical flow= 15 kN/m^3

Co-efficient of consolidation for horizontal flow= 30 kN/m^3

4.2.3 Stability of Soft Soil on Unimproved Ground

Initially stability of the underlying soft soil layer of thickness 8 m was evaluated for a typical embankment section within this stretch consisting a fill height of 6 m. Details of calculation are presented on Appendix B. It has been recommended by the researchers that factor of safety should be at least 1.2. But if we consider the process only one stage, then factor of safety becomes objectionable from the layer 2-8 m except on 1 m. So we have to consider several stage of loading of fill and in Appendix B, it is determined that two stage of loading should be adequate considering the initial fill height of 3.5 m and the rest is done after. For the 1st stage of loading of fill with initial fill height of 3.5 m presents the suitable factor of safety in all layers of soft soil thickness, about 1.36-2 from D= from 8 to 1m depth.

4.2.4 Amount of Consolidation Settlement and Time for Consolidation Settlement for 1st stage fill

Details of consolidation settlement calculations at mid-depth of the soft soil layer under the centre and toe of the embankment for fill height of 3.5 m are shown in Appendix B, the maximum settlement under the centre and toe of the embankment (S_t) are found respectively 462.55 mm and 121.42 mm. Consolidation time for 70% and 90% consolidation was found respectively 10-627 days and 21-1321 days. In order to accelerate the process it is considered 90% consolidation.

4.2.5 Stability Check of 2nd stage Fill after the Completion of Consolidation of 1st stage fill

The stability of the underlying soft ground during placement of 2nd stage fill after the completion of consolidation of the 1st stage fill has been assessed. In Appendix B it presents the suitable factor of safety 1.32-1.8 from D= 8 to 1 m depth with 90% consolidation. For the procedure, the values of the increase in undrained shear strength of the soft soil at different layers due to consolidation of the 1st stage fill were estimated for a full fill height of 6 m.

4.2.6 Amount of Consolidation Settlement and Time for Consolidation Settlement for 2nd stage fill

Details of consolidation settlement calculations at mid-depth of the soft soil layer under the centre and toe of the embankment for fill height of 6 m are shown in Appendix B, the maximum settlement under the centre and toe of the embankment (S_t) are found respectively 600.8014 mm and 187.66 mm. Consolidation time for 70% and 90% consolidation was found respectively 10-627 days and 21-1321 days. In order to accelerate the process it is considered 90% consolidation.

Minimum degree of consolidation required= (600.8014-50)/600.8014 x 100 = 92% (Appendix B)

4.3 Design of Sand Drains and Prefabricated Vertical Drains

4.3.1 Design of Sand Drains

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m (for bilateral flow)

U= 0.92

In the figure 4.1 time vs drainage spacing of sand drain (triangular) is shown.

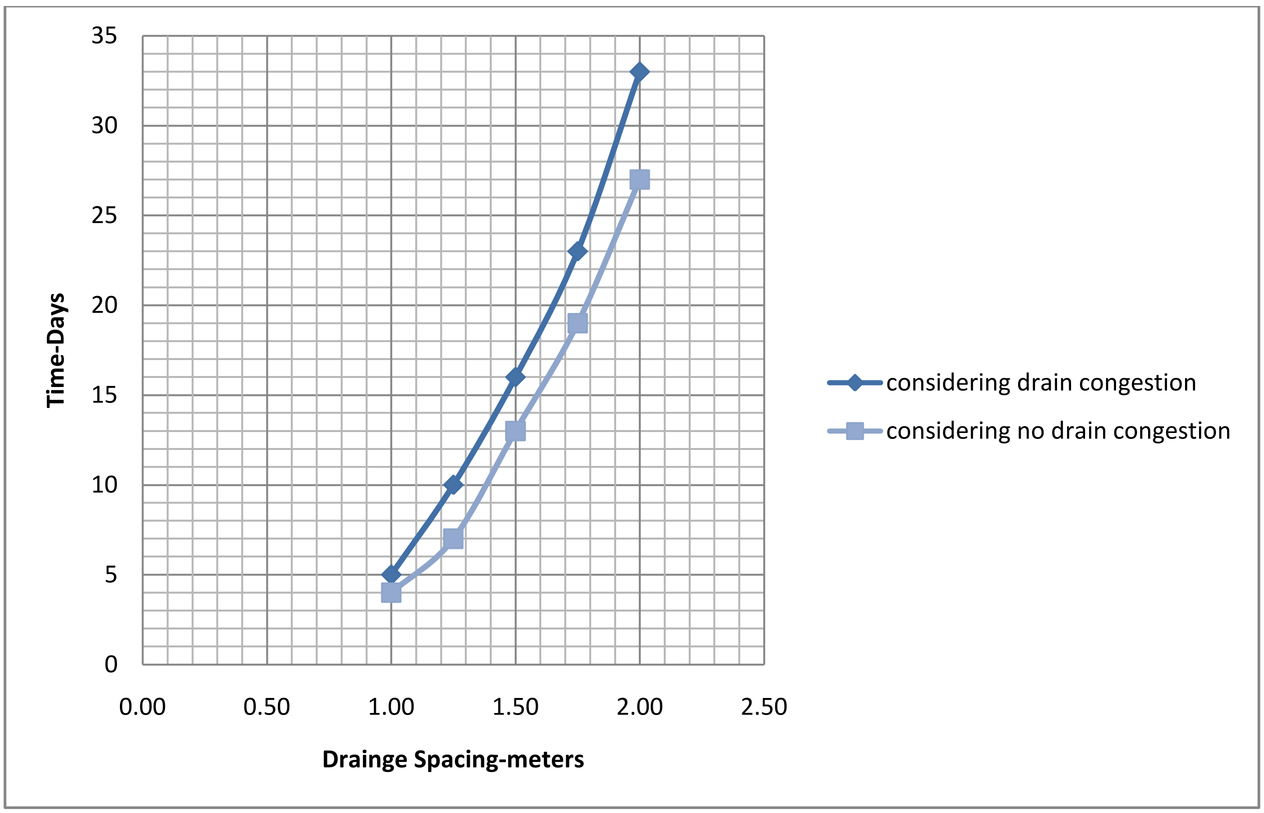


Figure 4.1 Time vs. Spacing of Sand Drains in Triangular Pattern

Details of calculation for designing sand drains are shown in Appendix B; from the graph, it is observed that

a) 200 mm sand @ 1.00 m c/c in a triangular pattern will require 5 days.
b) 200 mm sand @ 1.25 m c/c in a triangular pattern will require 10 days.
c) 200 mm sand @ 1.50 m c/c in a triangular pattern will require 16 days.
d) 200 mm sand @ 1.75 m c/c in a triangular pattern will require 23 days.
e) 200 mm sand @ 2.00 m c/c in a triangular pattern will require 33 days.

Consolidation time increases with increasing spacing of sand drains in triangular pattern based on the above design considerations for sand drain (Figure 4.1)

Figure 4.2 shows time vs drainage spacing of square pattern for sand drains

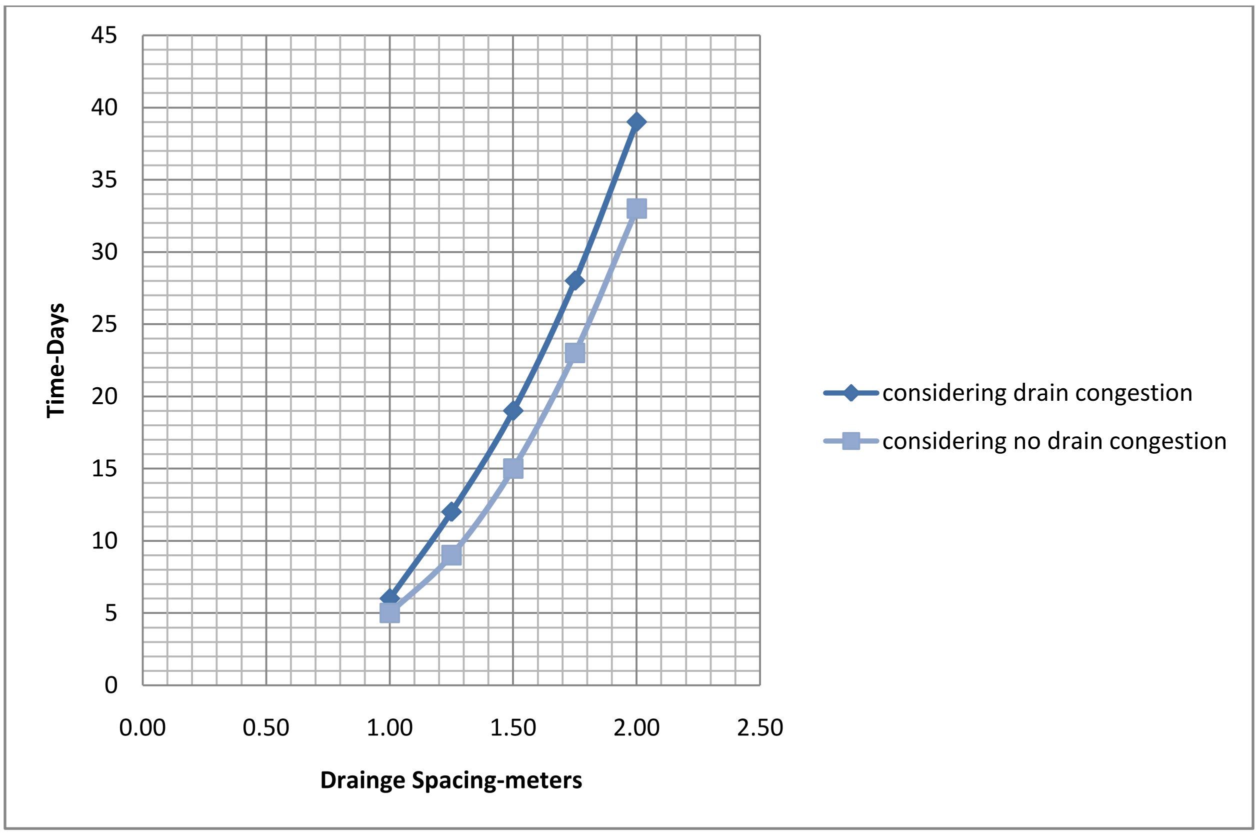


Figure 4.2 Time vs. Spacing of Sand Drains in Square Pattern

Details of calculation for designing sand drains are shown in Appendix B; from the graph, it is observed that

a) 200 mm sand @ 1.00 m c/c in a square pattern will require 6 days.
b) 200 mm sand @ 1.25 m c/c in a square pattern will require 12 days.
c) 200 mm sand @ 1.50 m c/c in a square pattern will require 19 days.
d) 200 mm sand @ 1.75 m c/c in a square pattern will require 28 days.
e) 200 mm sand @ 2.00 m c/c in a square pattern will require 39 days.

Consolidation time increases with increasing spacing of sand drains in square pattern based on the above design considerations for sand drain. (Figure 4.2)

4.3.2 Design of PVDs

d= 66 mm ; a= 100 mm ; b= 4mm; C_h= 30 m^2/year ; k_c/q_w= 0.01 m^{-2} ; I= 4 m (for bilateral flow); U= 0.92

Figure 4.3 shows time vs drainage spacing for pvd of triangular pattern

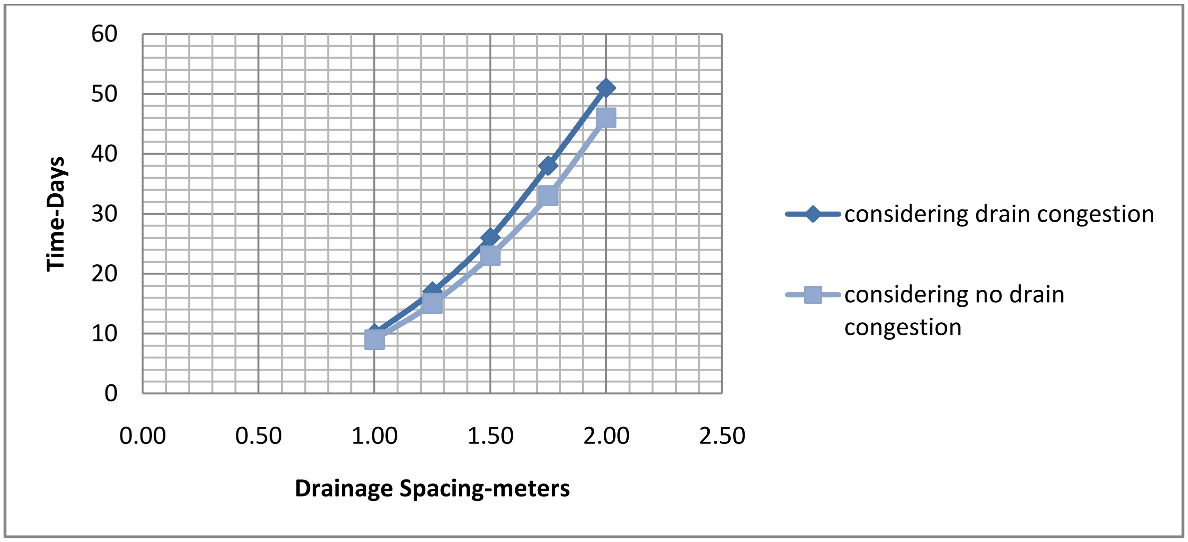


Figure 4.3 : Time vs. Spacing of PVD in Triangular Pattern

Details of calculation for designing PVDs are shown in Appendix B; from the graph, it is observed that

a) PVDs (width=100 mm, t= 4mm) @ 1.00 m c/c in a triangular pattern will require 10 days.
b) PVDs (width=100 mm, t= 4mm) @ 1.25 m c/c in a triangular pattern will require 17 days.
c) PVDs (width=100 mm, t= 4mm) @ 1.50 m c/c in a triangular pattern will require 26 days.
d) PVDs (width=100 mm, t= 4mm) @ 1.75 m c/c in a triangular pattern will require 38 days.
e) PVDs (width=100 mm, t= 4mm) @ 2.00 m c/c in a triangular pattern will require 51 days.

Consolidation time increases with increasing spacing of PVDs in triangular pattern based on the above design considerations for PVD. (Figure 4.3)

Figure 4.4 shows time vs drainage spacing for pvd of square pattern

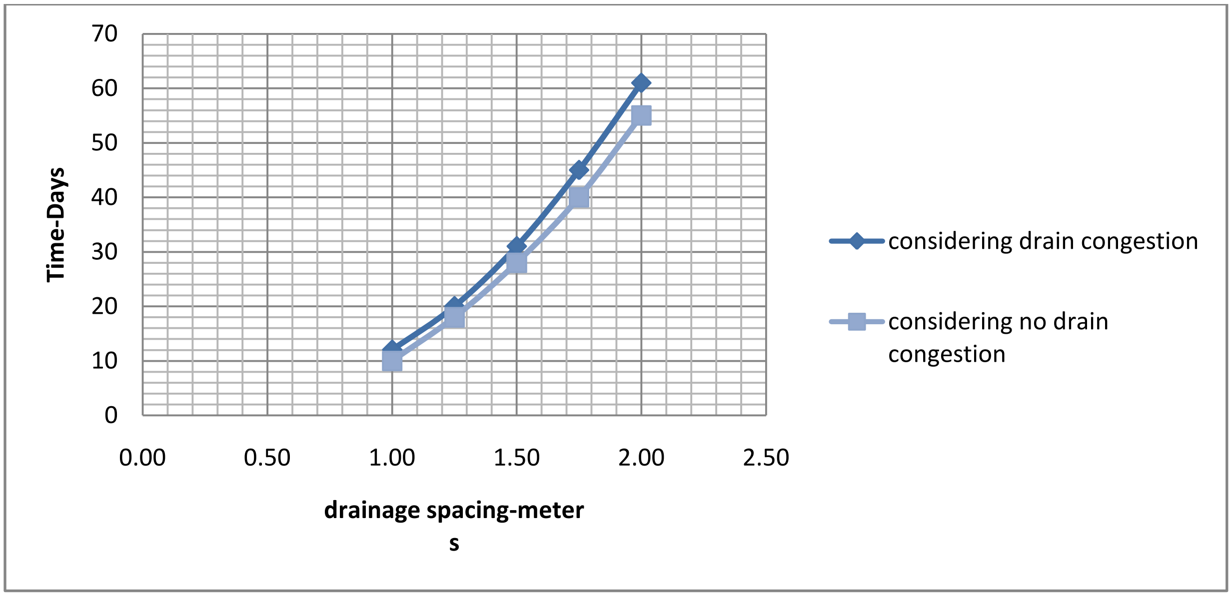


Figure 4.4 : Time vs. Spacing of PVD in Square Pattern

Details of calculation for designing PVDs are shown in Appendix B; from the graph, it is observed that

a) PVDs (width=100 mm, t= 4mm) @ 1.00 m c/c in a square pattern will require 12 days.
b) PVDs (width=100 mm, t= 4mm) @ 1.25 m c/c in a square pattern will require 20 days.
c) PVDs (width=100 mm, t= 4mm) @ 1.50 m c/c in a square pattern will require 31 days.
d) PVDs (width=100 mm, t= 4mm) @ 1.75 m c/c in a square pattern will require 45 days.
e) PVDs (width=100 mm, t= 4mm) @ 2.00 m c/c in a square pattern will require 61 days.

Consolidation time increases with increasing spacing of PVDs in square pattern based on the above design considerations for PVD. (Figure 4.4)

Considering about cost, time frame of the project and efficiency of drain, PVDs @ 1.5 m c/c may be selected. Though triangular pattern requires less time than square pattern, it is easier to install PVDs in square pattern.

4.4 Parametric Studies

Consolidation time depends on parameters such as spacing of drain, co-efficient of consolidation for horizontal flow, degree of consolidation, soft soil layer thickness, diameter of drain. (Appendix C)

4.4.1 Effect of Spacing of Drain on Consolidation Time for Sand Drain and PVD

Effect of spacing of drain on consolidation time has been examined by taking various spacing of drain like 1, 1.25, 1.5, 1.75, 2 m for sand drain and PVD.

Table 4.1 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d=0.2 m for sand drain (triangular) for both unilateral and bilateral flow. Figure 4.5 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d=0.2 m for sand drain (triangular) for both unilateral and bilateral flow.

Table 4.1: Time vs. Spacing of Drain when C_h=30 m^2/yr, U=0.92, d= 0.2 m; Sand drain (triangular)

Spacing of drain (m)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
1.00	9	1	5	1
1.25	16	1.78	10	2
1.50	25	2.78	16	3.2
1.75	36	4	23	4.6
2.00	49	5.44	33	6.6

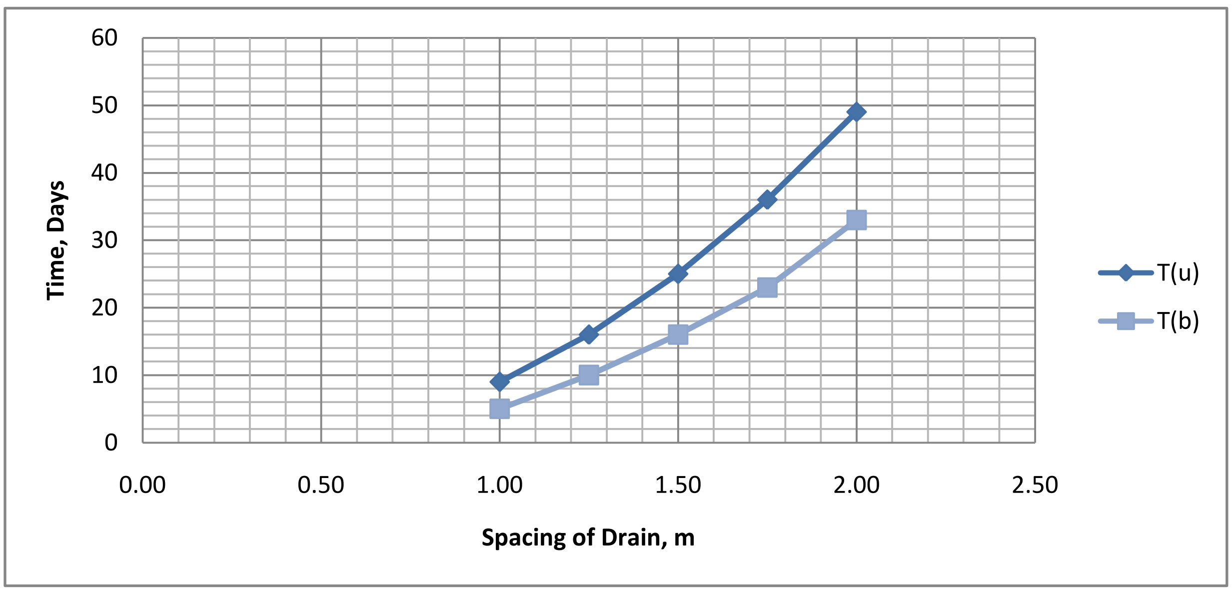


Figure 4.5: Time vs. Spacing of Drain when C_h=30 m^2/yr, U=0.92, d=0.2 m; Sand drain (triangular)

Table 4.2 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d=0.2 m for sand drain (square) for both unilateral and bilateral flow. Figure 4.6 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d=0.2 m for sand drain (square) for both unilateral and bilateral flow.

Table 4.2: Time vs. Spacing of Drain when C_h=30 m^2/yr, U=0.92, d=0.2 m; Sand drain (square)

Spacing of drain (m)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
1.00	11	1	5	1
1.25	19	1.73	9	1.8
1.50	30	2.73	15	3
1.75	42	3.82	23	4.6
2.00	58	5.27	33	6.6

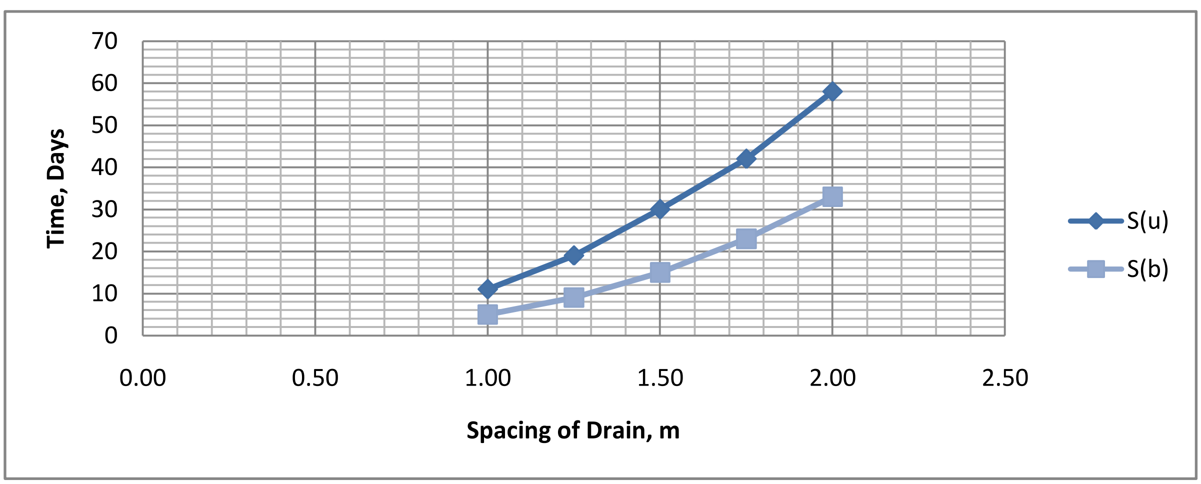


Figure 4.6: Time vs. Spacing of Drain when C_h=30 m^2/yr, U=0.92, d= 0.2 m; Sand drain (square)

Table 4.3 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d= 66 mm for PVD (triangular) for both unilateral and bilateral flow. Figure 4.7 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d= 66 mm for PVD (triangular) for both unilateral and bilateral flow.

Table 4.3: Time vs. Spacing of Drain, C_h=30 m^2/yr, U=0.92, d=66 mm; PVD (triangular)

Spacing of drain (m)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
1.00	14	1	10	1
1.25	23	1.64	17	1.7
1.50	35	2.5	26	2.6
1.75	50	3.57	38	3.8
2.00	68	4.86	51	5.1

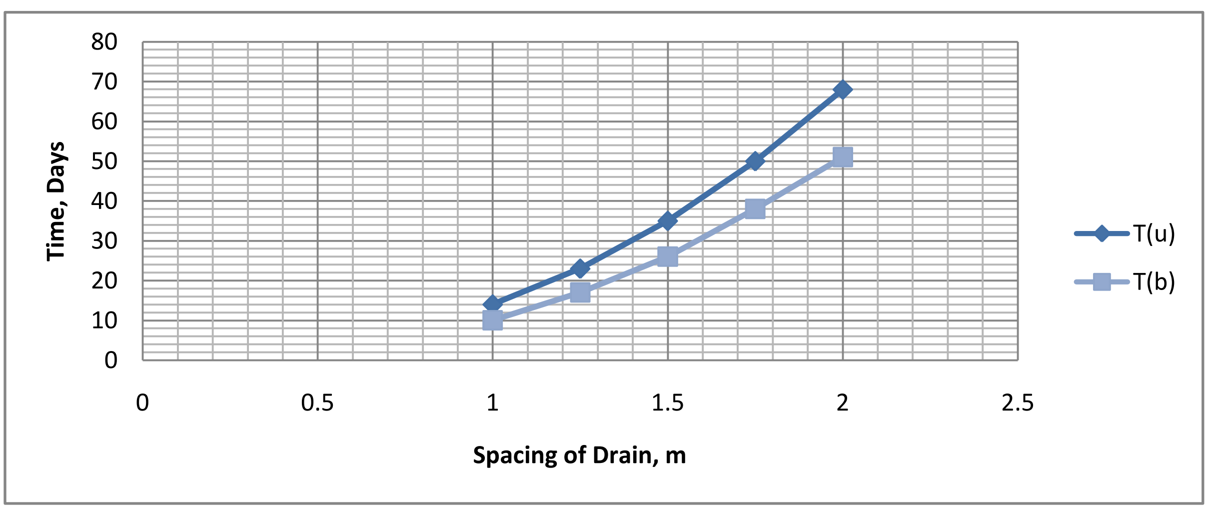


Figure 4.7: Time vs. Spacing of Drain when C_h=30 m^2/yr, U=0.92, d= 66 mm; PVD (triangular)

Table 4.4 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d=66 mm for PVD (square) for both unilateral and bilateral flow. Figure 4.8 shows the time vs spacing of drain when C_h=30 m^2/yr, U=0.92, d=66 mm for PVD (square) for both unilateral and bilateral flow.

Table 4.4: Time vs. Spacing of Drain, C_h=30 m^2/yr, U=0.92, d= 66 mm; PVD (square)

Spacing of drain (m)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
1.00	17	1	12	1
1.25	28	1.65	20	1.67
1.50	42	2.47	31	2.58
1.75	59	3.47	45	3.75
2.00	80	4.71	61	5.08

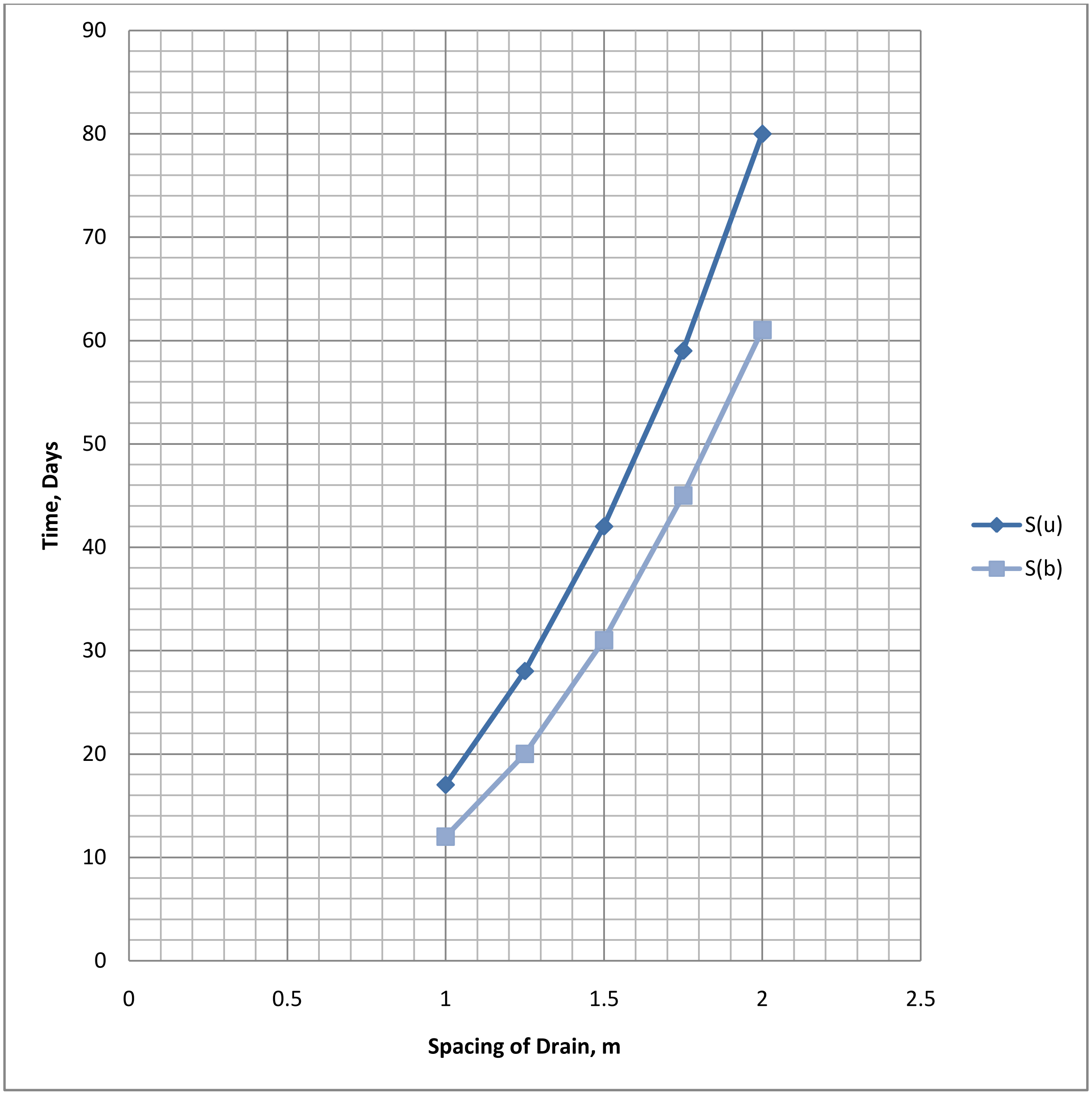


Figure 4.8: Time vs. Spacing of Drain when C_h=30 m^2/yr, U=0.92, d= 66 mm; PVD (Square)

For Sand drain and PVD, whether it is triangular or square pattern, it has been found that consolidation time increase with increasing spacing of drain keeping other parameters constant.

4.4.2 Effect of Co-efficient of Consolidation for Horizontal Flow on Consolidation Time for Sand Drain and PVD

Effects of Co-efficient of Consolidation for Horizontal Flow on Consolidation time has been examined by taking various C_h like 5, 10, 15, 20, 30 m^2/yr for both sand drain and PVD.

Table 4.5 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.2 m; Sand drain (triangular) for both unilateral and bilateral flow and figure 4.9 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.2 m; Sand drain (triangular) for both unilateral and bilateral flow.

Table 4.5: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.2 m; Sand drain (triangular)

Consolidation co-efficient for horizontal flow, C_h (m^2/yr)	Consolidation time, Days (unilateral)	Consolidation time decreases about (%)	Consolidation time, Days (bilateral)	Consolidation time decreases about (%)
30	49	83%	33	83%
20	73	75%	49	74.87%
15	98	67%	65	67%
10	147	50%	98	49.74%
5	294	0%	195	0%

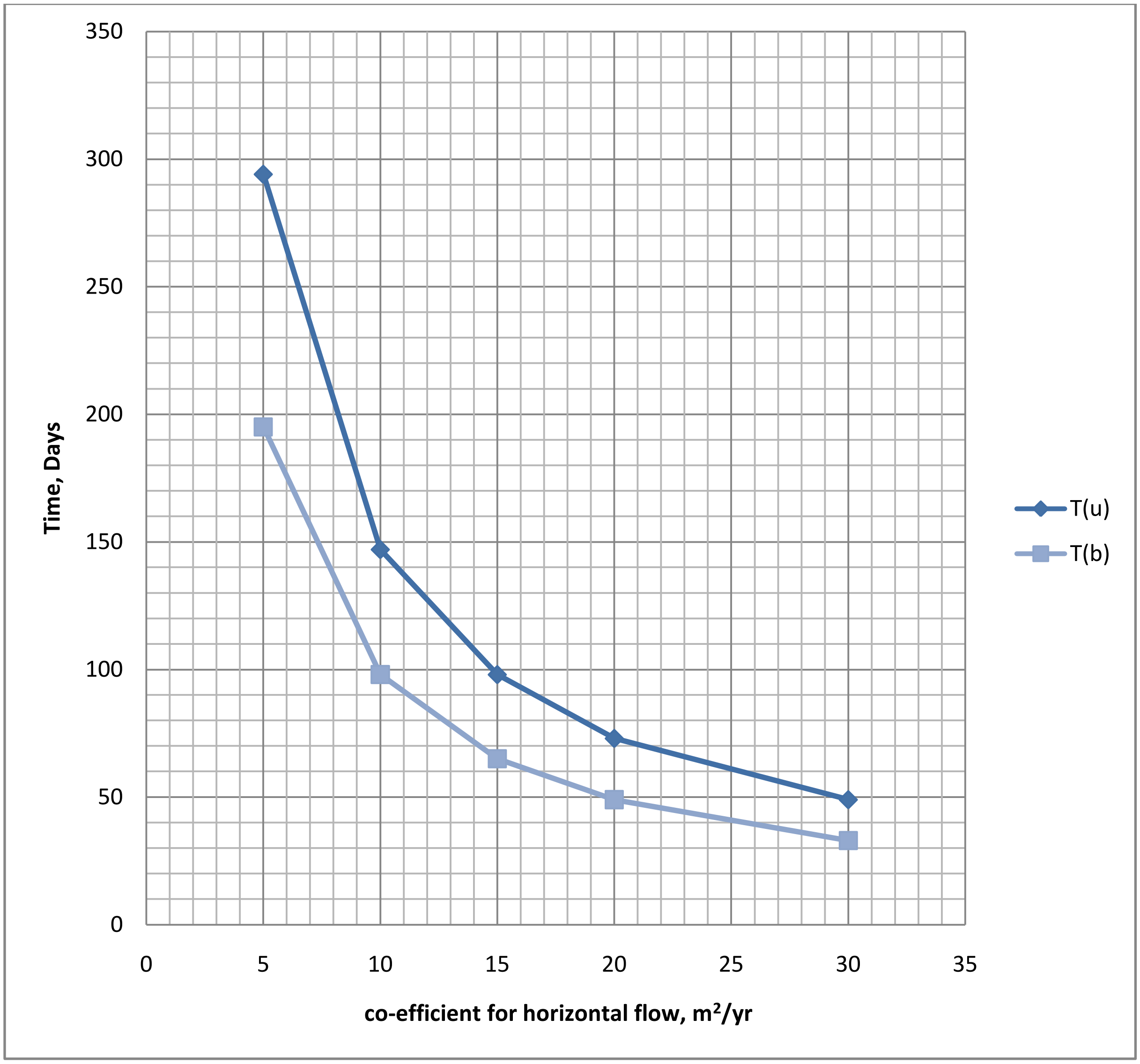


Figure 4.9: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.2 m; Sand Drain (Triangular)

Table 4.6 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.2 m; Sand drain (square) for both unilateral and bilateral flow and figure 4.10 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.2 m; Sand drain (square) for both unilateral and bilateral flow.

Table 4.6: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.2 m; Sand drain (square)

Consolidation co-efficient for horizontal flow, C_h (m^2/yr)	Consolidation time, Days (unilateral)	Consolidation time decreases about (%)	Consolidation time, Days (bilateral)	Consolidation time decreases about (%)
30	58	83.38%	39	83.40%
20	87	75.07%	59	74.89%
15	116	66.76%	78	66.81%
10	174	50.14%	118	49.79%
5	349	0%	235	0%

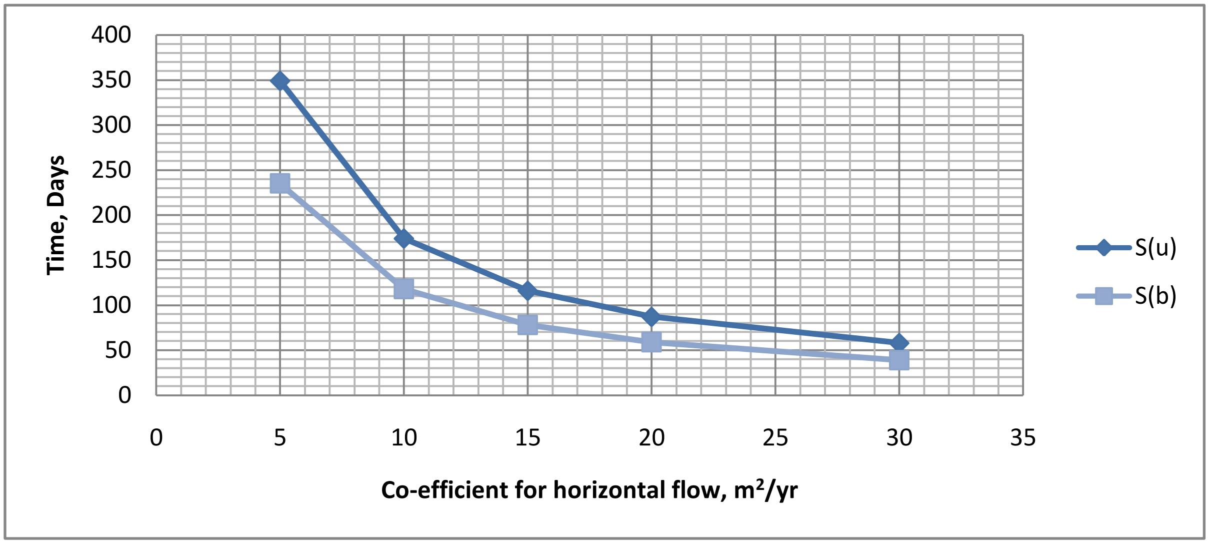


Figure 4.10: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.2 m; Sand Drain (Square)

Table 4.7 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.066 m; PVD (triangular) for both unilateral and bilateral flow and figure 4.11 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.066 m; PVD (triangular) for both unilateral and bilateral flow.

Table 4.7: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.066 m; PVD (triangular)

Consolidation co-efficient for horizontal flow, C_h (m^2/yr)	Consolidation time, Days (unilateral)	Consolidation time decreases about (%)	Consolidation time, Days (bilateral)	Consolidation time decreases about (%)
30	68	83.25%	51	83.44%
20	101	75.12%	77	75%
15	135	66.75%	103	66.56
10	203	50%	154	50%
5	406	0%	308	0%

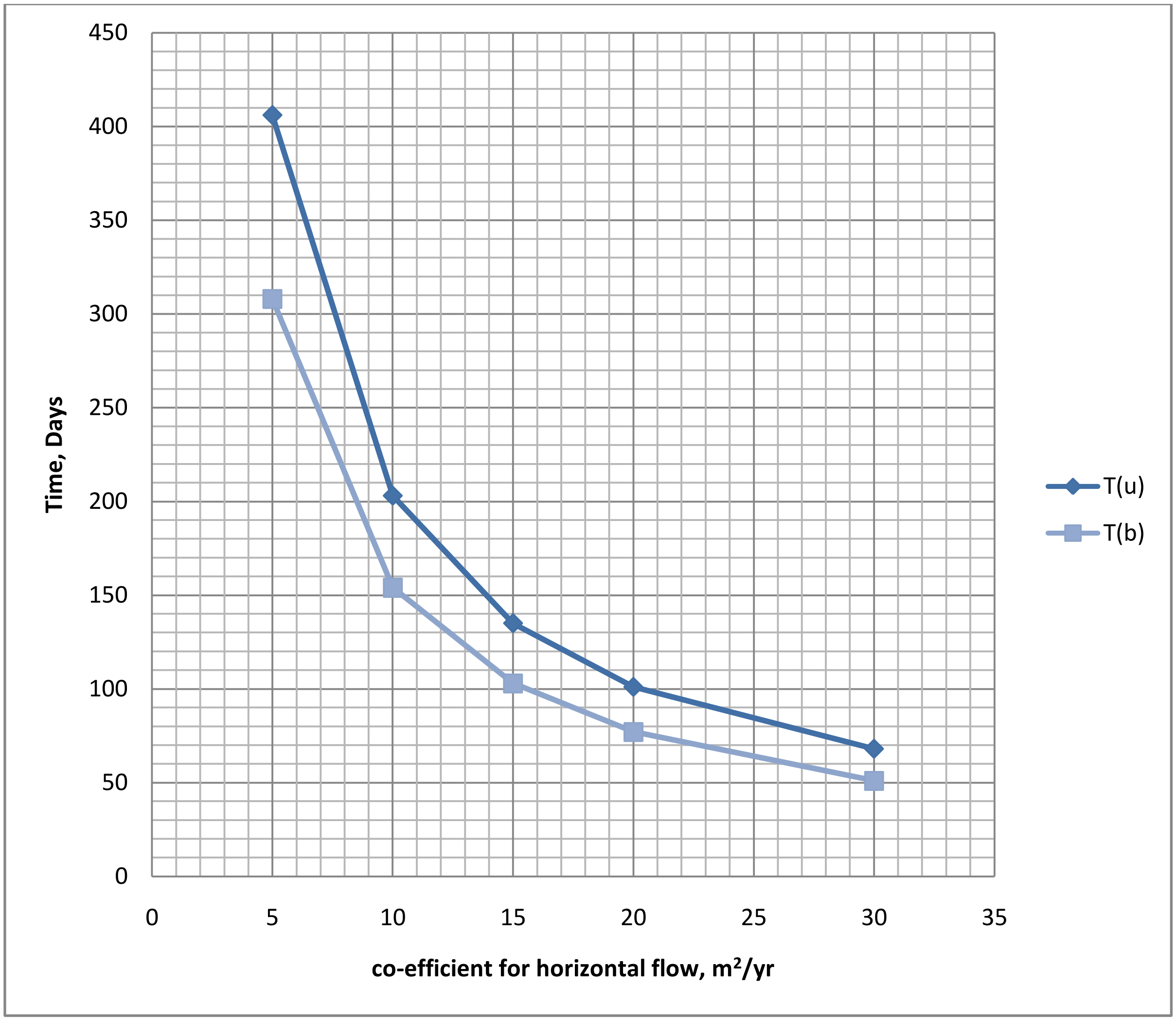


Figure 4.11: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.066 m; PVD (triangular)

Table 4.8 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.066 m; PVD (square) for both unilateral and bilateral flow and figure 4.12 shows time vs co-efficient for horizontal flow, s=2 m, U=92 %, d=0.066 m; PVD (square) for both unilateral and bilateral flow.

Table 4.8: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.066 m; PVD (square)

Consolidation co-efficient for horizontal flow, C_h (m^2/yr)	Consolidation time, Days (unilateral)	Consolidation time decreases about (%)	Consolidation time, Days (bilateral)	Consolidation time decreases about (%)
30	80	83.30%	61	83.29%
20	120	74.95%	91	75.07%
15	160	66.60%	122	66.58%
10	239	50.10%	183	49.86%
5	479	0%	365	0%

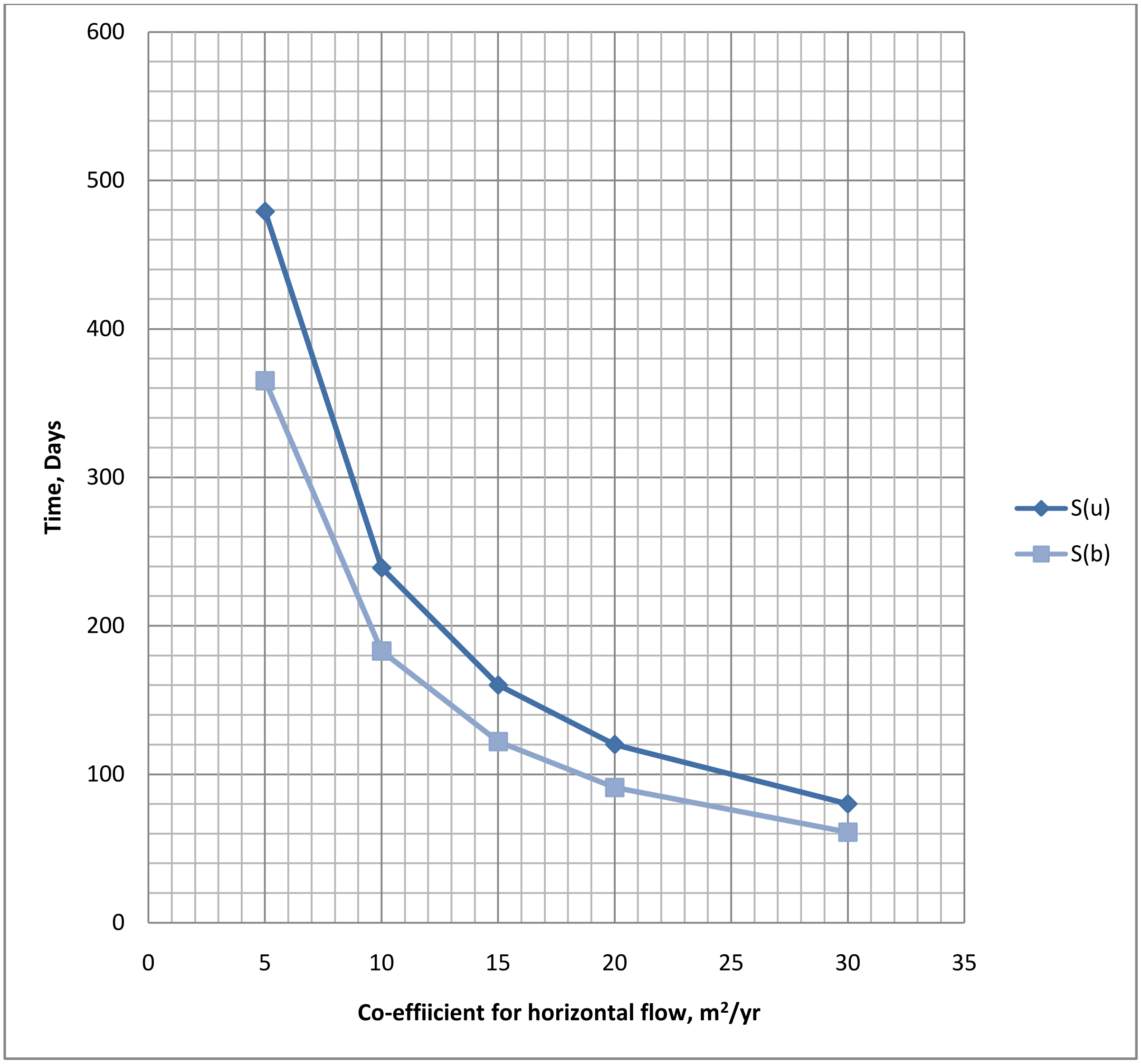


Figure 4.12: Time vs. Co-efficient for horizontal flow, s=2 m, U=92%, d=0.066 m; PVD (square)

For Sand drain and PVD, whether it is triangular or square pattern, consolidation time decrease with increasing co-efficient for horizontal flow keeping other parameters constant.

4.4.3 Effect of Diameter of Drain on Consolidation Time for Sand Drain

Effects of Diameter of Drain on Consolidation time has been examined by taking various diameters like 0.15, 0.2, 0.25, 0.3 m for both sand drain and PVD.

Table 4.9 shows time vs diameter of drain, s=2 m, U=92 %, C_h= 30 m^2/yr ; Sand drain (triangular) for both unilateral and bilateral flow and figure 4.13 shows time vs diameter of drain, s=2 m, U=92 %, C_h= 30 m^2/yr ; Sand drain (triangular) for both unilateral and bilateral flow.

Table 4.9: Time vs. Diameter of Drain; C_h=30 m^2/yr, s=2 m, U=92% (triangular)

Diameter of Drain, d (m)	Consolidation time, Days (unilateral)	Consolidation time decreases about (%)	Consolidation time, Days (bilateral)	Consolidation time decreases about (%)
0.15	54	0.00%	37	0%
0.2	49	9.26%	33	10.81%
0.25	45	16.67%	29	21.62%
0.3	42	22.22%	26	29.73%

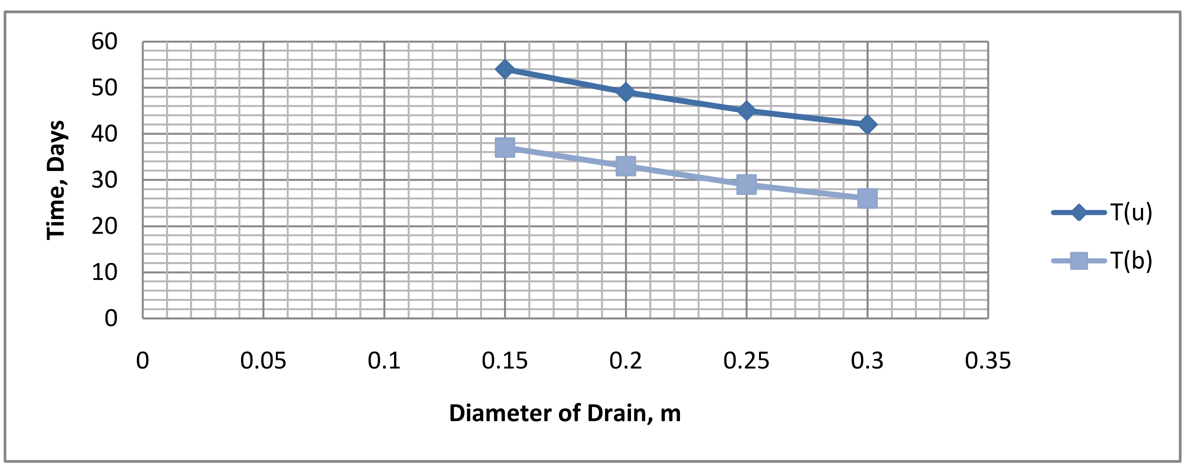


Figure 4.13: Time vs. Diameter of Drain; C_h=30 m^2/yr, s=2 m, U=92% (triangular)

Table 4.10 shows time vs diameter of drain, s=2 m, U=92 %, C_h= 30 m^2/yr ; Sand drain (square) for both unilateral and bilateral flow and figure 4.14 shows time vs diameter of drain, s=2 m, U=92 %, C_h= 30 m^2/yr ; Sand drain (square) for both unilateral and bilateral flow.

Table 4.10: Time vs. Diameter of Drain; C_h=30 m^2/yr, s=2 m, U=92% (square)

Diameter of Drain, d (m)	Consolidation time, Days (unilateral)	Consolidation time decreases about (%)	Consolidation time, Days (bilateral)	Consolidation time decreases about (%)
0.15	64	0%	45	0%
0.2	58	9.38%	39	13.33%
0.25	54	15.63%	35	22.22%
0.3	50	21.88%	31	31.11%

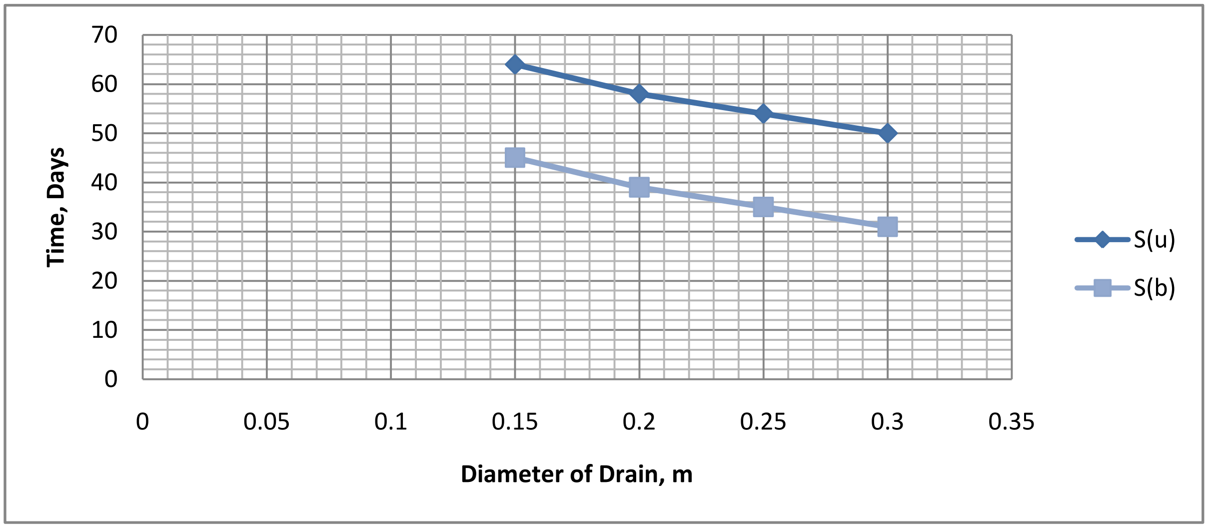


Figure 4.14: Time vs. Diameter of Drain; C_h=30 m^2/yr, s=2 m, U=92% (square)

For Sand drain, whether it is triangular or square pattern, consolidation time decrease with increasing diameter of drain keeping other parameters constant.

4.4.4 Effect of Degree of Consolidation on Consolidation Time for Sand Drain and PVD

Effects of Degree of Consolidation on Consolidation time has been examined by taking various degree of consolidation like 50, 70, 90, 92 % for both sand drain and PVD.

Table 4.11 shows time vs degree of consolidation, s=2 m, d=0.2 m, C_h= 30 m^2/yr ; Sand drain (triangular) for both unilateral and bilateral flow and figure 4.15 shows time vs degree of consolidation, s=2 m, d=0.2 m, C_h= 30 m^2/yr ; Sand drain (triangular) for both unilateral and bilateral flow.

Table 4.11: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.2 m; Sand drain (triangular)

Degree of Consolidation, U(%)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
0.92	49	3.77	33	3.67
0.9	45	3.46	30	3.33
0.7	23	1.77	16	1.78
0.5	13	1	9	1

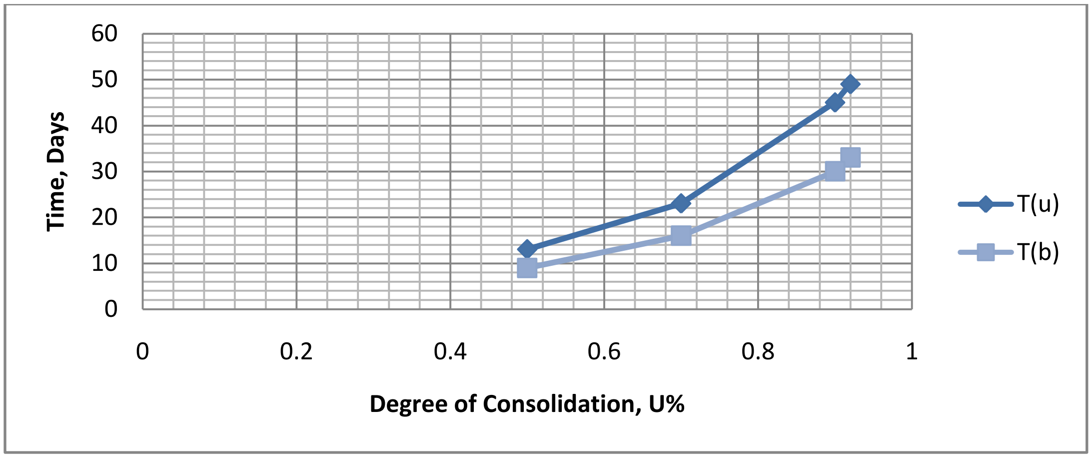


Figure 4.15: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.2 m; Sand drain (triangular)

Table 4.12 shows time vs degree of consolidation, s=2 m, d=0.2 m, C_h= 30 m^2/yr ; Sand drain (square) for both unilateral and bilateral flow and figure 4.16 shows time vs degree of consolidation, s=2 m, d=0.2 m, C_h= 30 m^2/yr ; Sand drain (square) for both unilateral and bilateral flow.

Table 4.12: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.2 m; Sand drain (square)

Degree of Consolidation, U(%)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
0.92	58	3.63	39	3.55
0.9	53	3.31	36	3.27
0.7	28	1.75	19	1.73
0.5	16	1	11	1

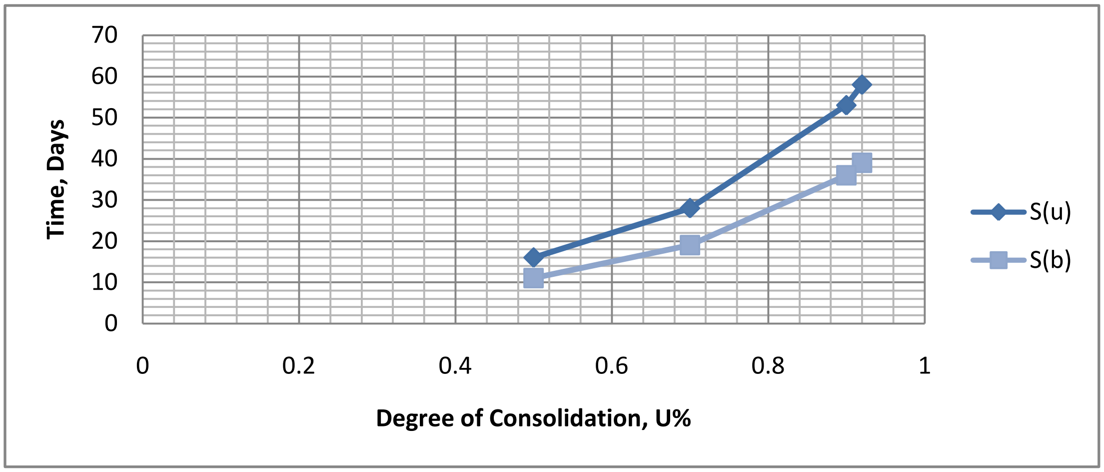


Figure 4.16: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.2 m; Sand drain (square)

Table 4.13 shows time vs degree of consolidation, s=2 m, d=0.066 m, C_h= 30 m^2/yr ; PVD (triangular) for both unilateral and bilateral flow and figure 4.17 shows time vs degree of consolidation, s=2 m, d=0.066 m, C_h= 30 m^2/yr ; PVD (triangular) for both unilateral and bilateral flow.

Table 4.13: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.066 m; PVD (triangular)

Degree of Consolidation, U(%)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
0.92	68	3.58	51	3.64
0.9	62	3.26	47	3.36
0.7	32	1.68	24	1.71
0.5	19	1	14	1

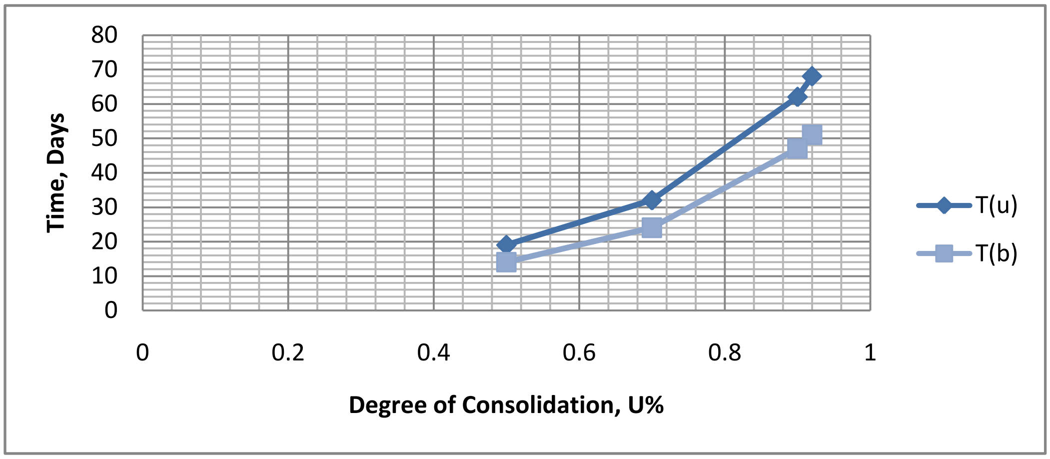


Figure 4.17: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.066 m; PVD (triangular)

Table 4.14 shows time vs degree of consolidation, s=2 m, d=66 mm, C_h= 30 m^2/yr ; PVD (square) for both unilateral and bilateral flow and figure 4.18 shows time vs degree of consolidation, s=2 m, d=66 mm, C_h= 30 m^2/yr ; PVD (square) for both unilateral and bilateral flow.

Table 4.14: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.066 m; PVD (square)

Degree of Consolidation, U(%)	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
0.92	80	3.64	61	3.59
0.9	73	3.32	55	3.24
0.7	38	1.73	29	1.71
0.5	22	1	17	1

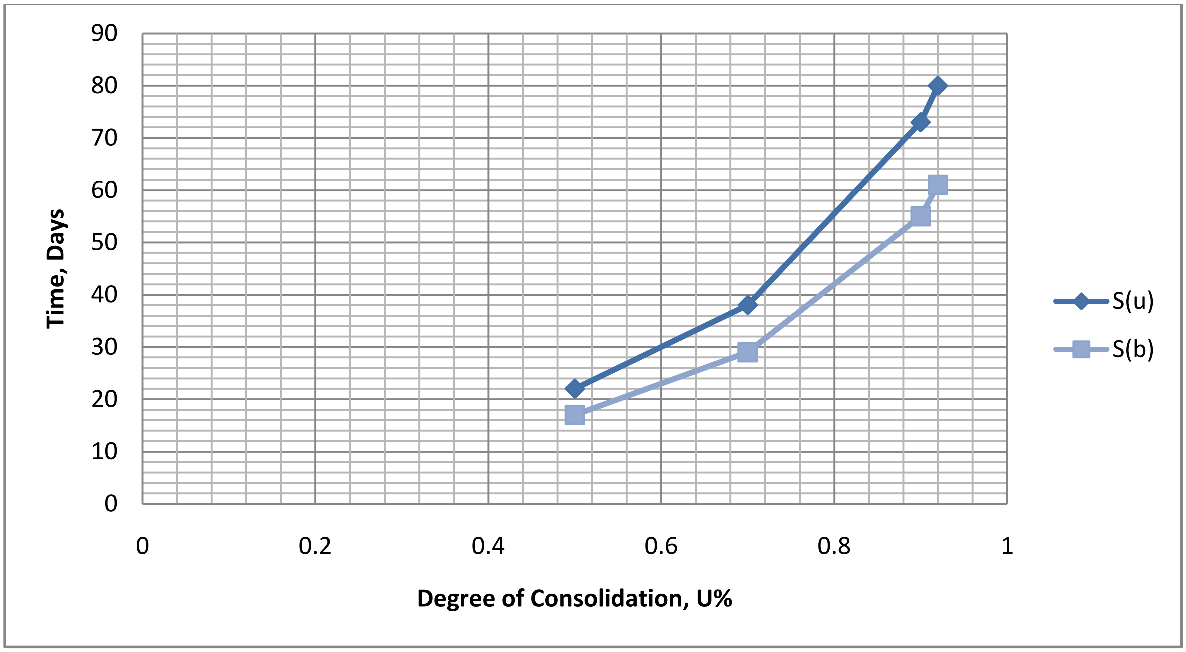


Figure 4.18: Time vs. Degree of Consolidation; C_h=30 m^2/yr, s=2 m, d=0.066 m; PVD (square)

For Sand drain and PVD, whether it is triangular or square pattern, consolidation time increase with increasing degree of consolidation keeping other parameters constant.

4.4.5 Effect of Soft Soil Layer Thickness on Consolidation Time for Sand Drain and PVD

Effects of Soft Soil Layer Thickness on Consolidation time has been examined by taking various soft soil layer thickness like 2, 4, 6, 8 m for both sand drain and PVD.

Table 4.15 shows time vs soft soil layer thickness, s=2 m, d=0.2 m, C_h= 30 m^2/yr, d= 0.2 m; Sand drain (triangular) for both unilateral and bilateral flow and figure 4.19 shows time vs soft soil layer thickness, s=2 m, d=0.2 m, C_h= 30 m^2/yr ; Sand drain (triangular) for both unilateral and bilateral flow.

Table 4.15: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.2 m, U=92%; Sand Drain (triangular)

Soft Soil Layer Thickness, m	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
8	49	1.75	33	1.22
6	39	1.39	30	1.11
4	33	1.18	28	1.04
2	28	1	27	1

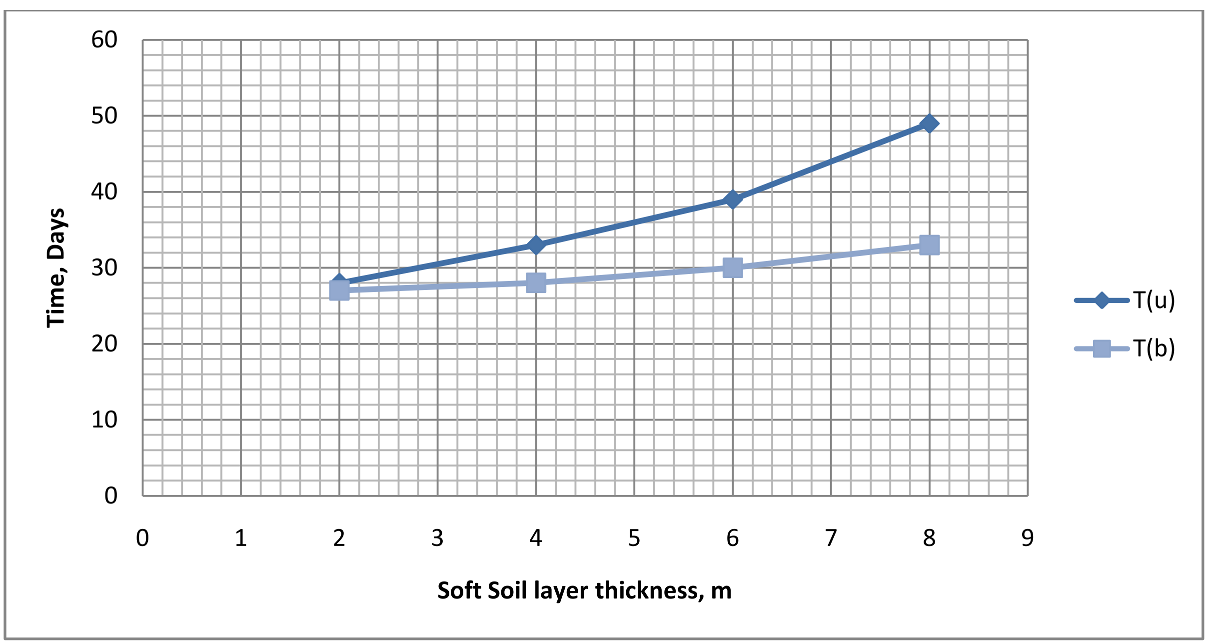


Figure 4.19: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.2 m, U=92%; Sand Drain (triangular)

Table 4.16 shows time vs soft soil layer thickness, s=2 m, d=0.2 m, C_h= 30 m^2/yr, d= 0.2 m; Sand drain (triangular) for both unilateral and bilateral flow and figure 4.20 shows time vs soft soil layer thickness, s=2 m, d=0.2 m, C_h= 30 m^2/yr ; Sand drain (triangular) for both unilateral and bilateral flow.

Table 4.16: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.2 m, U=92%; Sand Drain (square)

Soft Soil Layer Thickness, m	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
8	58	1.71	39	1.18
6	48	1.41	36	1.09
4	39	1.15	34	1.03
2	34	1	33	1

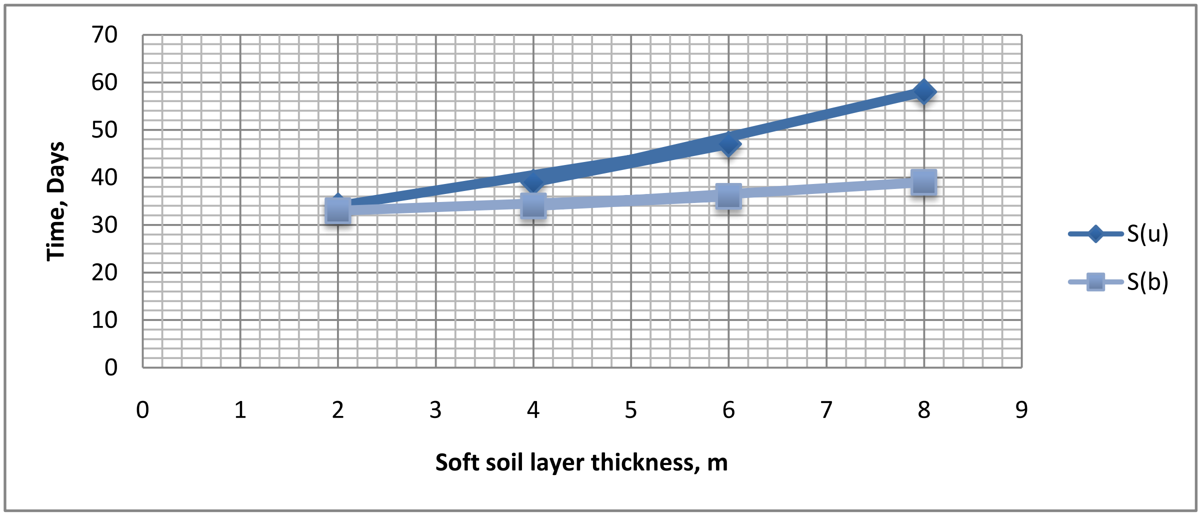


Figure 4.20: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.2 m, U=92%; Sand Drain (square)

Table 4.17 shows time vs soft soil layer thickness, s=2 m, d=0.066 m, C_h= 30 m^2/yr; PVD (triangular) for both unilateral and bilateral flow and figure 4.21 shows time vs soft soil layer thickness, s=2 m, d=0.066 m, C_h= 30 m^2/yr ; PVD (triangular) for both unilateral and bilateral flow.

Table 4.17: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.066 m, U=92%; PVD (triangular)

Soft Soil Layer Thickness, m	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
8	68	1.45	51	1.11
6	58	1.23	49	1.07
4	51	1.09	47	1.02
2	47	1	46	1

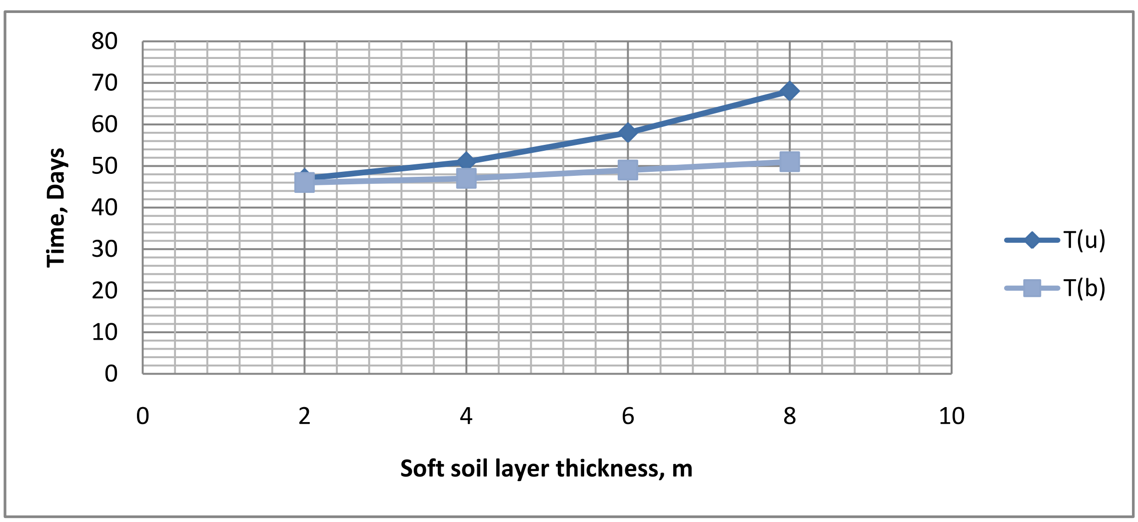


Figure 4.21: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.066 m, U=92%; PVD (triangular)

Table 4.18 shows time vs soft soil layer thickness, s=2 m, d=0.066 m, C_h= 30 m^2/yr; PVD (square) for both unilateral and bilateral flow and figure 4.22 shows time vs soft soil layer thickness, s=2 m, d=0.066 m, C_h= 30 m^2/yr ; PVD (square) for both unilateral and bilateral flow.

Table 4.18: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.066 m, U=92%; PVD (square)

Soft Soil Layer Thickness, m	Consolidation time, Days (unilateral)	Consolidation time increases about (times)	Consolidation time, Days (bilateral)	Consolidation time increases about (times)
8	80	1.43	61	1.11
6	69	1.23	58	1.05
4	61	1.09	56	1.02
2	56	1	55	1

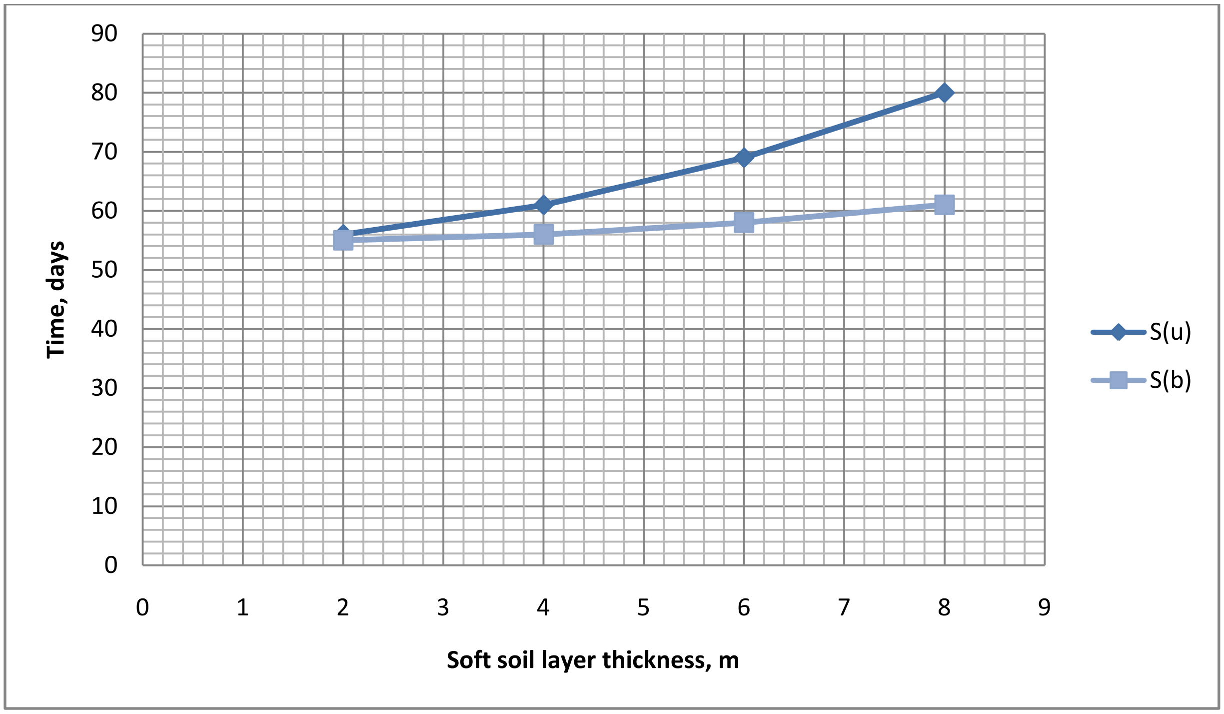


Figure 4.22: Time vs. Soft soil layer thickness; C_h=30 m^2/yr, s=2 m, d=0.066 m, U=92%; PVD (square)

For Sand drain and PVD, whether it is triangular or square pattern, consolidation time increase with increasing soft soil layer thickness keeping other parameters constant.

4.5 Summary of Parametric Studies

1) Consolidation time increases with increasing spacing of drain, for a certain value of co-efficient of consolidation for horizontal flow, degree of consolidation, soft soil layer thickness and for specific equivalent diameter of sand drain.

2) Consolidation time decreases with increasing co-efficient of consolidation for horizontal flow, for a certain value of spacing of drain, degree of consolidation, soft soil layer thickness and for specific equivalent diameter of sand drain.

3) Consolidation time increases with increasing degree of consolidation, for a certain value of co-efficient of consolidation for horizontal flow, spacing of drain, soft soil layer thickness and for specific equivalent diameter of sand drain.

4) Consolidation time increases with increasing soft soil layer thickness, for a certain value of co-efficient of consolidation for horizontal flow, degree of consolidation, spacing of drain and for specific equivalent diameter of sand drain.

5) Consolidation time decreases with increasing diameter of sand drain, for a certain value of co-efficient of consolidation for horizontal flow, degree of consolidation, soft soil layer thickness and for spacing of drain.

Chapter 5

Conclusions and Recommendations

5.1 General

According to the soil condition of the project area, designing of sand drain and prefabricated vertical drain have been done for the proposed Khan Jahan Ali Airport Runway. Based on project cost and time, suitable recommendations have been given. There is also a parametric study of sand drain and prefabricated vertical drain for this particular project. Recommendations for the future study will help to work the soil improvement process smoothly.

5.2 Recommendations about choosing Suitable Sand Drains and PVD

Checking with the minimum settlement, if less consolidation time is preferable,

1) Sand drains having 200 mm diameter @ 1.25 m c/c either in square or triangular pattern can be preferable, cost effective.
2) PVDs (width= 100 mm and thickness= 4 mm) @ 1.25 m c/c either in square or triangular pattern can be preferable, cost effective.

Checking with the minimum settlement, if comparatively more consolidation time is preferable,

1) Sand drains having 200 mm diameter @ 1.75 m c/c either in square or @ 2.00 m c/c in triangular pattern can be preferable, cost effective.
2) PVDs (width= 100 mm and thickness= 4 mm) @ 1.50 m c/c either in square or triangular pattern can be preferable, cost effective.

It is recommended to use PVD considering cost, time of the project and efficiency of drain having adequate consolidation time with spacing of drain @ 1.25-1.50 m c/c and equivalent diameter of drain of 66 mm in square pattern (as it is easier to install than triangular pattern) and its installation is quick and simple, long term improvement, cost–effective, less environmental problems regarding disposal of spoil materials, elimination of costlier backfill of drains, greater assurance of vertical drainage path and resistance towards lateral displacement under the vertical or horizontal soil movements.

5.3 Conclusions on Parametric Study of Sand Drains and PVDs

1) Consolidation time increases with increasing spacing of drain, for a certain value of co-efficient of consolidation for horizontal flow, degree of consolidation, soft soil layer thickness and for specific equivalent diameter of sand drain.

2) Consolidation time decreases with increasing co-efficient of consolidation for horizontal flow, for a certain value of spacing of drain, degree of consolidation, soft soil layer thickness and for specific equivalent diameter of sand drain.

3) Consolidation time increases with increasing degree of consolidation, for a certain value of co-efficient of consolidation for horizontal flow, spacing of drain, soft soil layer thickness and for specific equivalent diameter of sand drain.

4) Consolidation time increases with increasing soft soil layer thickness, for a certain value of co-efficient of consolidation for horizontal flow, degree of consolidation, spacing of drain and for specific equivalent diameter of sand drain.

5) Consolidation time decreases with increasing diameter of sand drain, for a certain value of co-efficient of consolidation for horizontal flow, degree of consolidation, soft soil layer thickness and for spacing of drain.

6) From the analysis and design of sand drains and prefabricated vertical drains, it is found that comparing with for bilateral flow consolidation time is less than the consolidation for unilateral flow irrespective of square or triangular pattern.

7) It has also been found that comparison to square pattern consolidation time less than for triangular pattern both for sand drains and prefabricated vertical drains irrespective of unilateral or bilateral.

5.4 Recommendations for Future Study

The present study has covered analysis of designs of sand drains and prefabricated vertical drains varying various design parameters. The study was limited to laboratory test and field observation. It is necessary to extend this study considering in following aspects,

1) The present study was carried out considering a certain value of the ratio of permeability of soils and discharge capacity of drain. Further study may be carried out varying the ratio of permeability of soils and discharge capacity of drain to show the varying the type of soil to be improved.

2) Small scale laboratory test can be carried out for Sand Drain and PVD to show the effects of parameters including co-efficient for horizontal flow, diameter of drain, spacing of drain, degree of consolidation etc.

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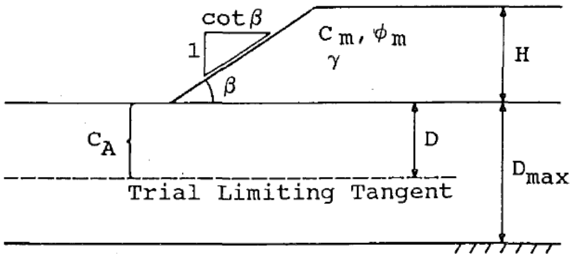
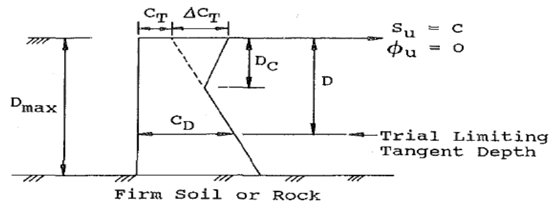
APPENDIX A

Design Calculation for Soil Improvement Works of the Proposed Khan Jahan Ali Airport at Bagerhat showing Stability Analysis and Consolidation Settlement & Time for Consolidation Settlement





C_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$C_A = 0.35\ C_T + 0.65\ C_D$, for $\Delta C_T = 0$.

$C_A = 0.35\ C_T + 0.65\ C_D + 0.35\ (\frac{D_C}{D})^{1.1}\ \Delta C_T$, for $D > D_C$

$C_A = 0.35\ (C_T + \Delta C_T) + 0.65\ C_D$, for $D \leq D_C$

Both C_T and ΔC_T can be positive (as shown), or negative.

Stability analysis of embankments on soft ground by B. K. Low (1989)

$$(F_s)_D = N_1 \frac{C_A}{\gamma H} + N_2 \cdot \left(\frac{C_m}{\gamma H} + \lambda \tan \phi_m\right)$$

where $N_1 = 3.06 \left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 \left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53} \right] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 \left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303 \frac{\cot^2\beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 \left(\frac{D}{H}+\frac{1}{2}\right) - \frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2 - \frac{1}{24}(\cot^2\beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02\cot\beta}{D/H}, \text{ or Fig. 4}$$

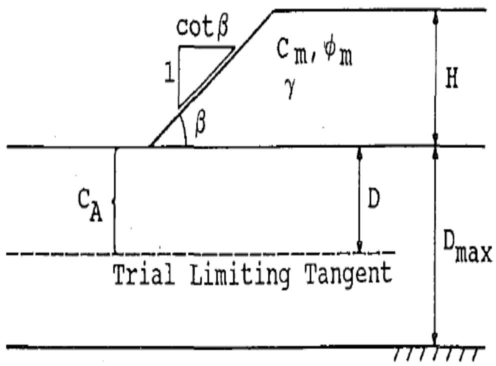
(For $D/H \geq 0.5$)

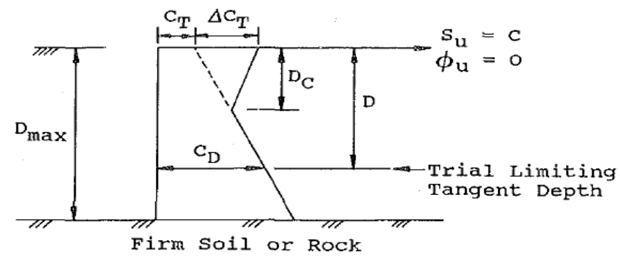
considering a surcharge of 100kPa on the runway pavement; H=6 m; Depth of runway soil layer, D=8 m

H (m)	D (m)	D/H	β (°)	α_1	α_2	λ	N_1	N_2	N_1+N_2	C_m (kPa)	Φ_m (°)	ϒ (kN/m³)	C_T (kPa)	C_D (kPa)	C_A (kPa)	$(F_s)_D$
6	1	0.17	26.565	2.02	0.92	0.43	3.63	3.28	6.91	30	10	18	10	10.00	10.00	1.50
6	2	0.33	26.565	2.09	1.18	0.31	4.26	2.31	6.57	30	10	18	10	10.62	10.40	1.18
6	3	0.50	26.565	2.22	1.51	0.27	4.53	1.79	6.32	30	10	18	10	10.75	10.49	1.02
6	4	0.67	26.565	2.38	1.89	0.25	4.68	1.46	6.14	30	10	18	10	11.50	10.98	0.95
6	5	0.83	26.565	2.57	2.33	0.24	4.78	1.24	6.02	30	10	18	10	12.00	11.30	0.90
6	6	1.00	26.565	2.78	2.84	0.23	4.85	1.08	5.93	30	10	18	10	13.75	12.44	0.90
6	7	1.17	26.565	3.00	3.40	0.22	4.91	0.95	5.86	30	10	18	10	14.40	12.86	0.89
6	8	1.33	26.565	3.22	4.02	0.22	4.95	0.86	5.81	30	10	18	10	15.00	13.25	0.88

Here it is clear that factor of safety is not acceptable from D=2 m depth

Suitable $(F_s)_D \geq 1.2$; is OK ; but in this it is not OK; several stage of loading is required; let's study how much stage is needed





C_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$C_A = 0.35\ C_T + 0.65\ C_D$, for $\Delta C_T = 0$.

$C_A = 0.35\ C_T + 0.65\ C_D + 0.35\ \left(\frac{D_C}{D}\right)^{1.1} \Delta C_T$, for $D > D_C$

$C_A = 0.35\ (C_T + \Delta C_T) + 0.65\ C_D$, for $D \le D_C$

Both C_T and ΔC_T can be positive (as shown), or negative.

Stability analysis of embankments on soft ground by B. K. Low (1989)

$$(F_s)_D = N_1 \frac{C_A}{\gamma H} + N_2 \cdot \left(\frac{C_m}{\gamma H} + \lambda \tan \phi_m\right)$$

where $N_1 = 3.06 \left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 \left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53}\right] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 \left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303 \frac{\cot^2 \beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 \left(\frac{D}{H}+\frac{1}{2}\right) - \frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2 - \frac{1}{24}(\cot^2 \beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02 \cot \beta}{D/H}, \text{ or Fig. 4}$$

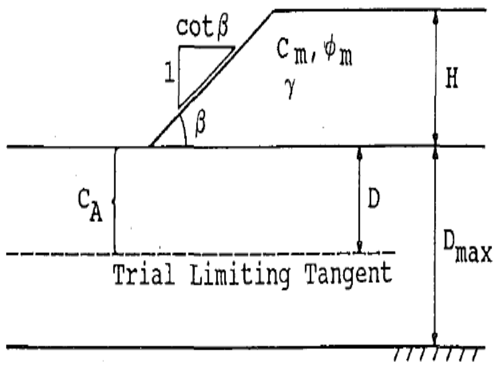
(For $D/H \ge 0.5$)

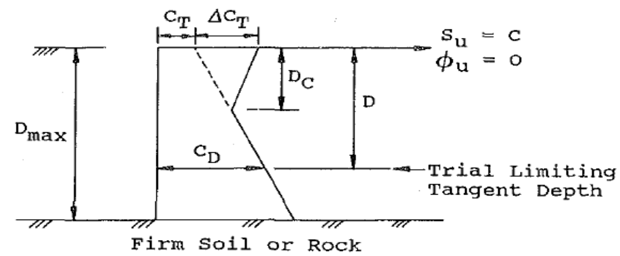
considering a surcharge of 100kPa on the runway pavement; H=3.5 m; Depth of runway soil layer, D=8 m

1st stage of loading

H (m)	D (m)	D/H	β (°)	α_1	α_2	λ	N_1	N_2	N_1+N_2	C_m (kPa)	Φ_m (°)	Ɣ (kN/m³)	C_T (kPa)	C_D (kPa)	C_A (kPa)	$(F_s)_D$
3.5	1	0.29	26.565	2.06	1.10	0.33	4.14	2.52	6.66	30	10	18	10	10.00	10.00	2.00
3.5	2	0.57	26.565	2.28	1.66	0.26	4.60	1.63	6.23	30	10	18	10	10.62	10.40	1.61
3.5	3	0.86	26.565	2.60	2.40	0.24	4.79	1.21	6.00	30	10	18	10	10.75	10.49	1.43
3.5	4	1.14	26.565	2.97	3.31	0.23	4.90	0.97	5.87	30	10	18	10	11.50	10.98	1.35
3.5	5	1.43	26.565	3.35	4.40	0.22	4.98	0.81	5.79	30	10	18	10	12.00	11.30	1.31
3.5	6	1.71	26.565	3.76	5.66	0.21	5.04	0.69	5.73	30	10	18	10	13.75	12.44	1.35
3.5	7	2.00	26.565	4.17	7.09	0.21	5.08	0.61	5.69	30	10	18	10	14.40	12.86	1.35
3.5	8	2.29	26.565	4.59	8.70	0.21	5.12	0.54	5.66	30	10	18	10	15.00	13.25	1.36

Here H=3.5 m for 1st stage loading and $(F_s)_D \ge 1.2$; is OK





c_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$c_A = 0.35\ c_T + 0.65\ c_D$, for $\Delta c_T = 0$.

$c_A = 0.35\ c_T + 0.65\ c_D + 0.35\ (\frac{D_C}{D})^{1.1}\ \Delta c_T$, for $D > D_C$

$c_A = 0.35\ (c_T + \Delta c_T) + 0.65\ c_D$, for $D \leq D_C$

Both c_T and Δc_T can be positive (as shown), or negative.

Stability analysis of embankments on soft ground by B. K. Low (1989)

$$(F_s)_D = N_1 \frac{c_A}{\gamma H} + N_2 \cdot \left(\frac{c_m}{\gamma H} + \lambda \tan \phi_m\right)$$

where $N_1 = 3.06 \left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 \left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53} \right] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 \left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303 \frac{\cot^2 \beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 \left(\frac{D}{H}+\frac{1}{2}\right) - \frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2 - \frac{1}{24}(\cot^2 \beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02 \cot \beta}{D/H}, \text{ or Fig. 4}$$

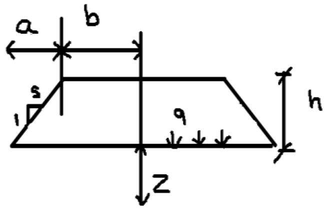
(For $D/H \geq 0.5$)

considering a surcharge of 100kPa on the runway pavement; H=6 m; Depth of runway soil layer, D=8 m

2nd stage of loading

H (m)	D (m)	D/H	β (°)	α_1	α_2	λ	N_1	N_2	N_1+N_2	C_m (kPa)	Φ_m (°)	Υ (kN/m³)	C_T (kPa)	C_D (kPa)	C_A (kPa)	$(F_s)_D$
6	1	0.17	26.565	2.02	0.92	0.43	3.63	3.28	6.91	30	10	18	19.07	19.07	19.07	1.80
6	2	0.33	26.565	2.09	1.18	0.31	4.26	2.31	6.57	30	10	18	19.07	19.69	19.47	1.54
6	3	0.50	26.565	2.22	1.51	0.27	4.53	1.79	6.32	30	10	18	19.07	19.82	19.56	1.40
6	4	0.67	26.565	2.38	1.89	0.25	4.68	1.46	6.14	30	10	18	19.07	20.57	20.05	1.34
6	5	0.83	26.565	2.57	2.33	0.24	4.78	1.24	6.02	30	10	18	19.07	21.07	20.37	1.30
6	6	1.00	26.565	2.78	2.84	0.23	4.85	1.08	5.93	30	10	18	19.07	22.82	21.51	1.31
6	7	1.17	26.565	3.00	3.40	0.22	4.91	0.95	5.86	30	10	18	19.07	23.47	21.93	1.31
6	8	1.33	26.565	3.22	4.02	0.22	4.95	0.86	5.81	30	10	18	19.07	24.07	22.32	1.32

Here H=6 m for 2nd stage loading and $(F_s)_D \geq 1.2$; is OK



Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 500 ft from North End

$I_z = 1/\pi\ [\ \tan^{-1}(b/z) + (1+b/a)\ \{\ \tan^{-1}(a/z+b/z) - \tan^{-1}(b/z)\}]$

$\Delta e = C_c\ /\ (1+e_0)\ \log\ (\Delta p + p_o\ ')/p_o'$

$S_c = \Delta eH.1000$ mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

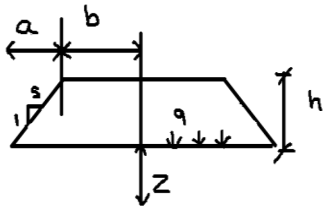
h= 2.5

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p=2I_zq$	ϒ' (kN/m³)	$P_0'=ϒ'z$	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.5000	45.00	8.3	4.15	1	0.2	1	0.1073	107.3473
1.5	3.33	15.24	0.5000	45.00	8.3	12.45	1	0.2	1	0.0664	66.4090
2.5	2.00	9.14	0.4998	44.98	8.3	20.75	1	0.2	1	0.0501	50.0755
3.5	1.43	6.53	0.4994	44.95	8.3	29.05	1	0.2	1	0.0406	40.6085
4.5	1.11	5.08	0.4988	44.90	8.3	37.35	1	0.2	1	0.0343	34.2820
5.5	0.91	4.16	0.4979	44.81	8.3	45.65	1	0.2	1	0.0297	29.7026
6.5	0.77	3.52	0.4966	44.70	8.3	53.95	1	0.2	1	0.0262	26.2091

Total=354.6340



Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 500 ft from North End

I_{z1} = [$\tan^{-1}$ (a/z+b/z)- $\tan^{-1}$(a/z) +z(a+b)/(z^2+$(a+b)^2$]/π

I_{z2} = [$\tan^{-1}$ (a/z+2b/z)- $\tan^{-1}$ (a/z+b/z) + 2((a+b)/a) * { $\tan^{-1}$ 2(a/z+b/z)- $\tan^{-1}$ (a/z+2b/z)-(a+b)/ (z^2+$(a+b)^2$)]/π

Δe= C_c / (1+e_0) log (Δp+p_0 ')/p_0'

S_c = ΔeH.1000 mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

h= 2.5

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	I_Z=I_{z1}+ I_{z2}	Δp=2I_zq	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.0317	1.43	8.3	4.15	1	0.2	1	0.013	12.84
1.5	3.33	15.24	0.0927	0.0000	0.0928	4.17	8.3	12.45	1	0.2	1	0.013	12.56
2.5	2.00	9.14	0.1474	0.0001	0.1476	6.64	8.3	20.75	1	0.2	1	0.012	12.06
3.5	1.43	6.53	0.1940	0.0004	0.1943	8.75	8.3	29.05	1	0.2	1	0.011	11.43
4.5	1.11	5.08	0.2324	0.0007	0.2331	10.49	8.3	37.35	1	0.2	1	0.011	10.75
5.5	0.91	4.16	0.2636	0.0013	0.2649	11.92	8.3	45.65	1	0.2	1	0.010	10.08
6.5	0.77	3.52	0.2888	0.0021	0.2909	13.09	8.3	53.95	1	0.2	1	0.009	9.43

Total=79.15

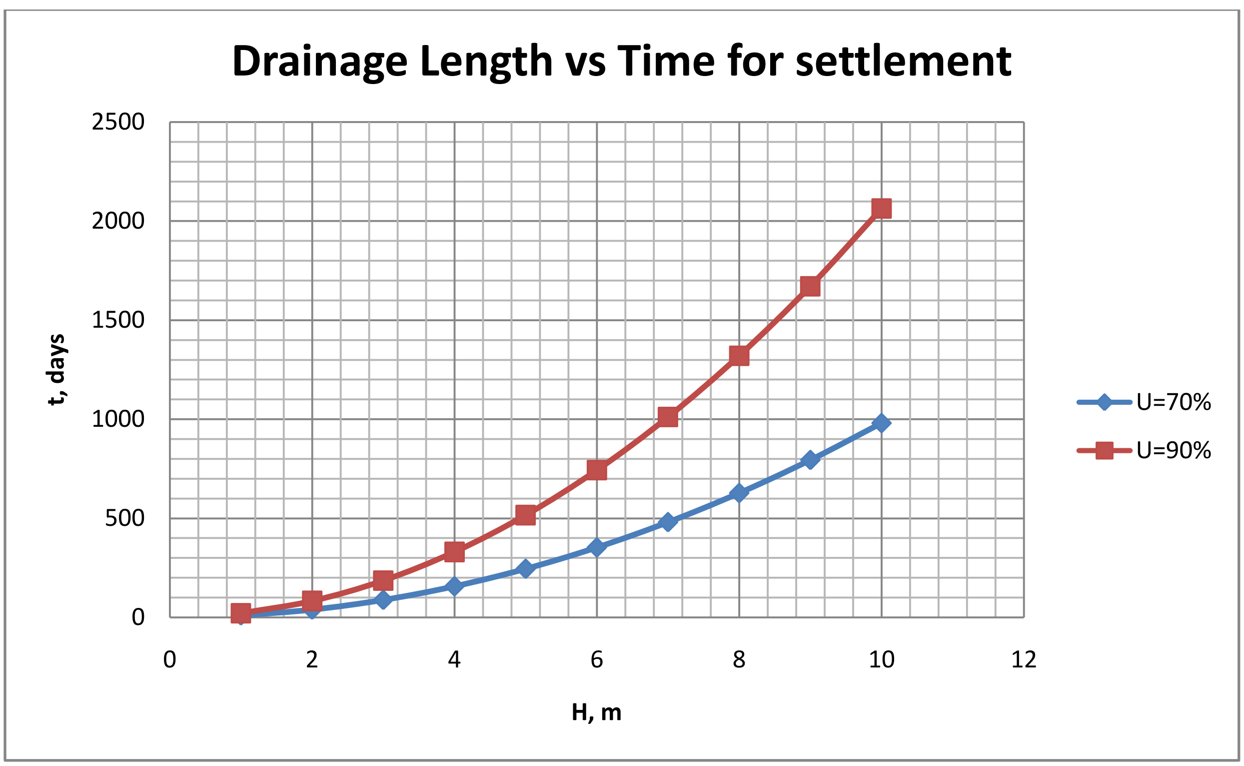
Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

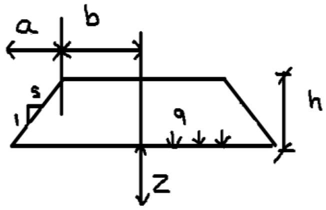
Chainage: 500 ft from North End

Calculation for Consolidation Settlement Time

$t = T_v/H^2/C_v$; H= drainage length

C_V (m^2/year)	U(%)	T_V	H(m)	t (year)	t (days)
15	70	0.403	1	0.02686	10
15	70	0.403	2	0.10743	39
15	70	0.403	3	0.24171	88
15	70	0.403	4	0.42970	157
15	70	0.403	5	0.67141	245
15	70	0.403	6	0.96683	353
15	70	0.403	7	1.31596	480
15	70	0.403	8	1.71881	627
15	70	0.403	9	2.17537	794
15	70	0.403	10	2.68564	980
15	90	0.848	1	0.05653	21
15	90	0.848	2	0.22613	83
15	90	0.848	3	0.50880	186
15	90	0.848	4	0.90453	330
15	90	0.848	5	1.41333	516
15	90	0.848	6	2.03520	743
15	90	0.848	7	2.77013	1011
15	90	0.848	8	3.61813	1321
15	90	0.848	9	4.57920	1671
15	90	0.848	10	5.65333	2063





Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 1500 ft from North End

$I_z = 1/\pi\ [\ \tan^{-1}(b/z) + (1+b/a)\ \{ \tan^{-1}(a/z+b/z) - \tan^{-1}(b/z)\}]$

$\Delta e = C_c\ /\ (1+e_0)\ \log\ (\Delta p+p_o')/p_o'$

$S_c = \Delta eH.1000$ mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

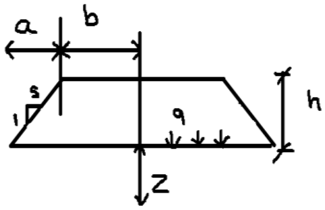
h= 2.5

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.0000	45.7200	0.5000	44.9997	8	4	1	0.2	1	0.1088	108.8134
1.5	3.3333	15.2400	0.5000	44.9959	8	12	1	0.2	1	0.0677	67.6662
2.5	2.0000	9.1440	0.4998	44.9815	8	20	1	0.2	1	0.0512	51.1759
3.5	1.4286	6.5314	0.4994	44.9499	8	28	1	0.2	1	0.0416	41.5867
4.5	1.1111	5.0800	0.4988	44.8952	8	36	1	0.2	1	0.0352	35.1620
5.5	0.9091	4.1564	0.4979	44.8122	8	44	1	0.2	1	0.0305	30.5020
6.5	0.7692	3.5169	0.4966	44.6968	8	52	1	0.2	1	0.0269	26.9409
7.5	0.6667	3.0480	0.4950	44.5457	8	60	1	0.2	1	0.0241	24.1155

Total=385.9626



Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 1500 ft from North End

I_{z1} = [$\tan^{-1}$ (a/z+b/z)- $\tan^{-1}$(a/z) +z(a+b)/(z^2+$(a+b)^2$]/π

I_{z2} = [$\tan^{-1}$ (a/z+2b/z)- $\tan^{-1}$ (a/z+b/z) + 2((a+b)/a) * { $\tan^{-1}$ 2(a/z+b/z)- $\tan^{-1}$ (a/z+2b/z)-(a+b)/ (z^2+$(a+b)^2$)]/π

$\Delta e = C_c / (1+e_0) \log (\Delta p+p_0')/p_0'$

$S_c = \Delta e H.1000$ mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

h= 2.5

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	$I_Z=I_{z1}+I_{z2}$	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.0317	1.43	8	4	1	0.2	1	0.01	13.26
1.5	3.33	15.24	0.0927	0.0000	0.0928	4.17	8	12	1	0.2	1	0.01	12.97
2.5	2.00	9.14	0.1474	0.0001	0.1476	6.64	8	20	1	0.2	1	0.01	12.45
3.5	1.43	6.53	0.1940	0.0004	0.1943	8.75	8	28	1	0.2	1	0.01	11.80
4.5	1.11	5.08	0.2324	0.0007	0.2331	10.49	8	36	1	0.2	1	0.01	11.11
5.5	0.91	4.16	0.2636	0.0013	0.2649	11.92	8	44	1	0.2	1	0.01	10.41
6.5	0.77	3.52	0.2888	0.0021	0.2909	13.09	8	52	1	0.2	1	0.01	9.75
7.5	0.67	3.05	0.3090	0.0032	0.3122	14.05	8	60	1	0.2	1	0.01	9.14

Total=90.88

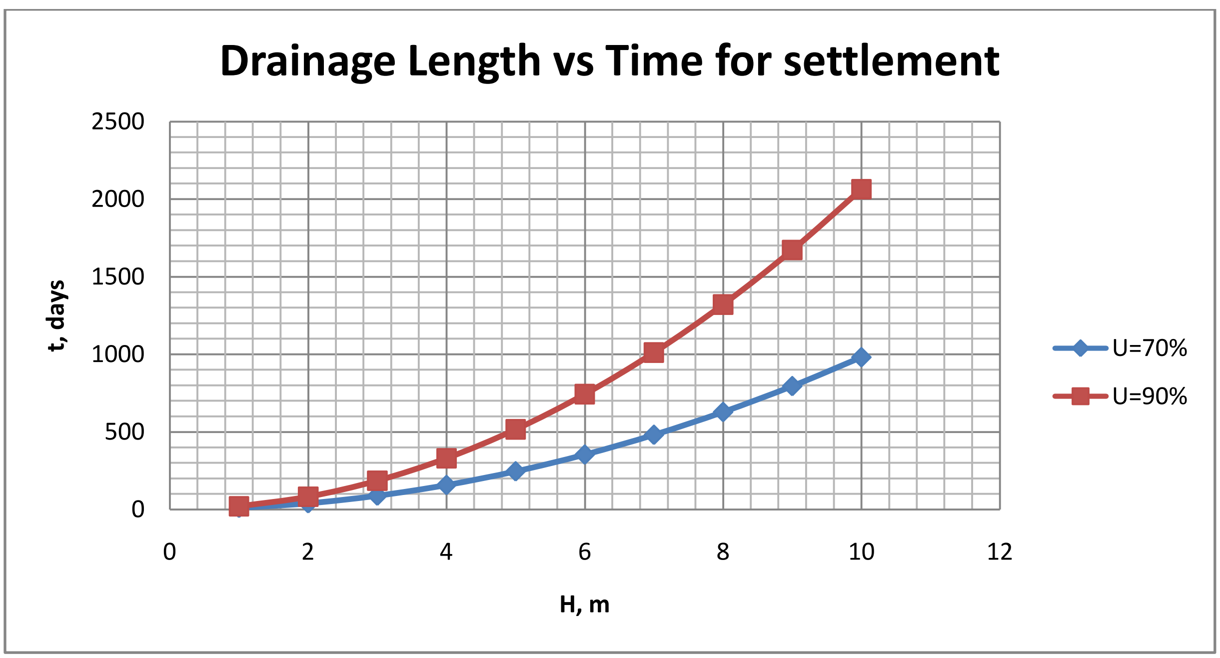
Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

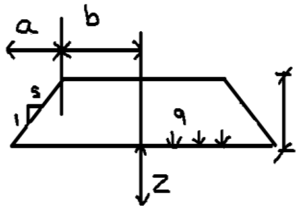
Chainage: 1500 ft from North End

Calculation for Consolidation Settlement Time

$t = T_v / H^2 / C_v$; H= drainage length

C_v (m²/year)	U(%)	T_v	H(m)	t (year)	t (days)
15	70	0.403	1	0.02686	10
15	70	0.403	2	0.10743	39
15	70	0.403	3	0.24171	88
15	70	0.403	4	0.42970	157
15	70	0.403	5	0.67141	245
15	70	0.403	6	0.96683	353
15	70	0.403	7	1.31596	480
15	70	0.403	8	1.71881	627
15	70	0.403	9	2.17537	794
15	70	0.403	10	2.68564	980
15	90	0.848	1	0.05653	21
15	90	0.848	2	0.22613	83
15	90	0.848	3	0.50880	186
15	90	0.848	4	0.90453	330
15	90	0.848	5	1.41333	516
15	90	0.848	6	2.03520	743
15	90	0.848	7	2.77013	1011
15	90	0.848	8	3.61813	1321
15	90	0.848	9	4.57920	1671
15	90	0.848	10	5.65333	2063





Location: Proposed Taxiway of Khan Jahan Ali Airport at Bagerhat

Chainage: 125 ft from West Edge of Runway

$I_z = 1/\pi$ [$\tan^{-1}$ (b/z) + (1+b/a) { $\tan^{-1}$ (a/z+b/z)- $\tan^{-1}$(b/z)}]

$\Delta e = C_c$ / (1+e_0) log ($\Delta p + p_o$ ')/p_o'

S_c = ΔeH.1000 mm

a= 5   q= ϒh =45

b= 22.86   ϒ= 18

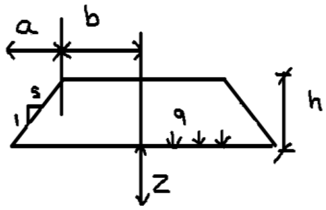
h= 2.5

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p = 2I_zq$	ϒ' (kN/m³)	$P_0' = \Upsilon' z$	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.0000	45.7200	0.5000	44.9997	7.34	3.67	1.28	0.38	1	0.1871	187.0988
1.5	3.3333	15.2400	0.5000	44.9959	7.34	11.01	1.28	0.38	1	0.1177	117.7411

Total=304.8399



Location: Proposed Taxiway of Khan Jahan Ali Airport at Bagerhat

Chainage: 125 ft from West Edge of Runway

I_{z1} = [$\tan^{-1}$ (a/z+b/z)- $\tan^{-1}$(a/z) +z(a+b)/(z^2+$(a+b)^2$]/π

I_{z2} = [$\tan^{-1}$ (a/z+2b/z)- $\tan^{-1}$ (a/z+b/z) + 2((a+b)/a) * { $\tan^{-1}$ 2(a/z+b/z)- $\tan^{-1}$ (a/z+2b/z)-(a+b)/ (z^2+$(a+b)^2$)]/π

$\Delta e = C_c$ / (1+e_0) log ($\Delta p+p_0$ ')/p_0'

S_c = ΔeH.1000 mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

h= 2.5

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	$I_Z=I_{z1}+I_{z2}$	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.032	1.43	7.34	3.67	1.28	0.38	1	0.0238	23.7838
1.5	3.33	15.24	0.0927	0.0000	0.093	4.17	7.34	11.01	1.28	0.38	1	0.0233	23.2693

Total=47.0531

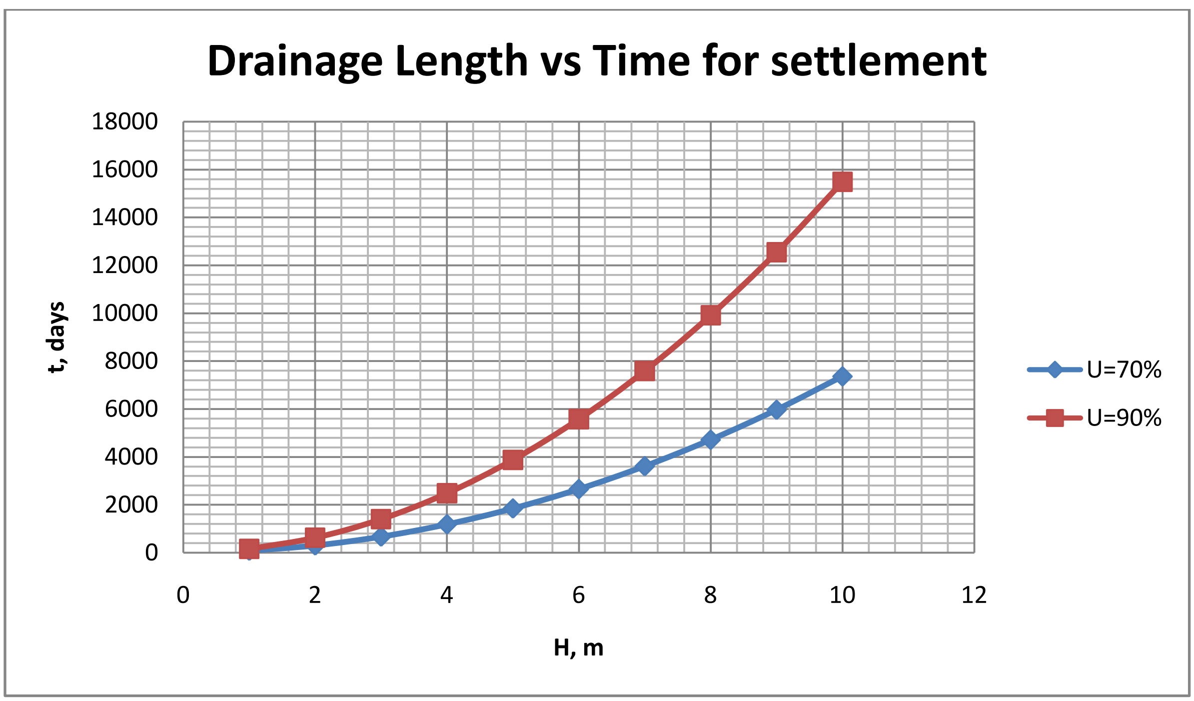
Location: Proposed Taxiway of Khan Jahan Ali Airport at Bagerhat

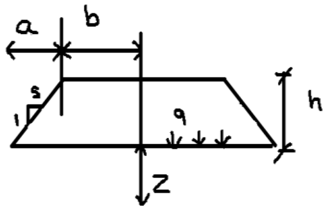
Chainage: 125 ft from West Edge of Runway

Calculation for Consolidation Settlement Time

$t=T_v/H^2/C_v$; H= drainage length

C_V (m^2/year)	U(%)	T_V	H(m)	t (year)	t (days)
2	70	0.403	1	0.20142	74
2	70	0.403	2	0.80569	294
2	70	0.403	3	1.81281	662
2	70	0.403	4	3.22277	1176
2	70	0.403	5	5.03557	1838
2	70	0.403	6	7.25123	2647
2	70	0.403	7	9.86972	3602
2	70	0.403	8	12.89107	4705
2	70	0.403	9	16.31526	5955
2	70	0.403	10	20.14229	7352
2	90	0.848	1	0.42400	155
2	90	0.848	2	1.69600	619
2	90	0.848	3	3.81600	1393
2	90	0.848	4	6.78400	2476
2	90	0.848	5	10.60000	3869
2	90	0.848	6	15.26400	5571
2	90	0.848	7	20.77600	7583
2	90	0.848	8	27.13600	9905
2	90	0.848	9	34.34400	12536
2	90	0.848	10	42.40000	15476





Location: Proposed Apron Area of Khan Jahan Ali Airport at Bagerhat

Chainage: 475 ft from the West edge of Runway

$I_z = 1/\pi\ [\ \tan^{-1}(b/z) + (1+b/a)\ \{\ \tan^{-1}(a/z+b/z) - \tan^{-1}(b/z)\}]$

$\Delta e = C_c\ /\ (1+e_0)\ \log\ (\Delta p + p_0')/p_0'$

$S_c = \Delta e H.1000$ mm

a= 5　　q= ϒh =45

b= 22.86　　ϒ= 18

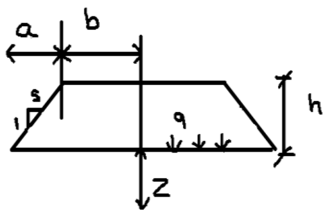
h= 2.5

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.0000	45.7200	0.5000	44.9997	8.53	4.2650	0.92	0.2	1	0.1107	110.6893
1.5	3.3333	15.2400	0.5000	44.9959	8.53	12.7950	0.92	0.2	1	0.0682	68.2103
2.5	2.0000	9.1440	0.4998	44.9815	8.53	21.3250	0.92	0.2	1	0.0513	51.3195
3.5	1.4286	6.5314	0.4994	44.9499	8.53	29.8550	0.92	0.2	1	0.0416	41.5534
4.5	1.1111	5.0800	0.4988	44.8952	8.53	38.3850	0.92	0.2	1	0.0350	35.0396

Total= 306.8121



Location: Proposed Apron Area of Khan Jahan Ali Airport at Bagerhat

Chainage: 475 ft from the West edge of Runway

$I_{z1} = [\tan^{-1}(a/z+b/z) - \tan^{-1}(a/z) + z(a+b)/(z^2+(a+b)^2\,]/\pi$

$I_{z2} = [\tan^{-1}(a/z+2b/z) - \tan^{-1}(a/z+b/z) + 2((a+b)/a) * \{\tan^{-1} 2(a/z+b/z) - \tan^{-1}(a/z+2b/z) - (a+b)/(z^2+(a+b)^2)\}]/\pi$

$\Delta e = C_c / (1+e_0) \log (\Delta p + p_0')/p_0'$

$S_c = \Delta e H.1000$ mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

h= 2.5

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	$I_Z=I_{z1}+I_{z2}$	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.0317	1.43	8.53	4.27	0.92	0.2	1	0.0131	13.0619
1.5	3.33	15.24	0.0927	0.0000	0.0928	4.17	8.53	12.80	0.92	0.2	1	0.0128	12.7741
2.5	2.00	9.14	0.1474	0.0001	0.1476	6.64	8.53	21.33	0.92	0.2	1	0.0123	12.2635
3.5	1.43	6.53	0.1940	0.0004	0.1943	8.75	8.53	29.86	0.92	0.2	1	0.0116	11.6222
4.5	1.11	5.08	0.2324	0.0007	0.2331	10.49	8.53	38.39	0.92	0.2	1	0.0109	10.9307

Total=60.6524

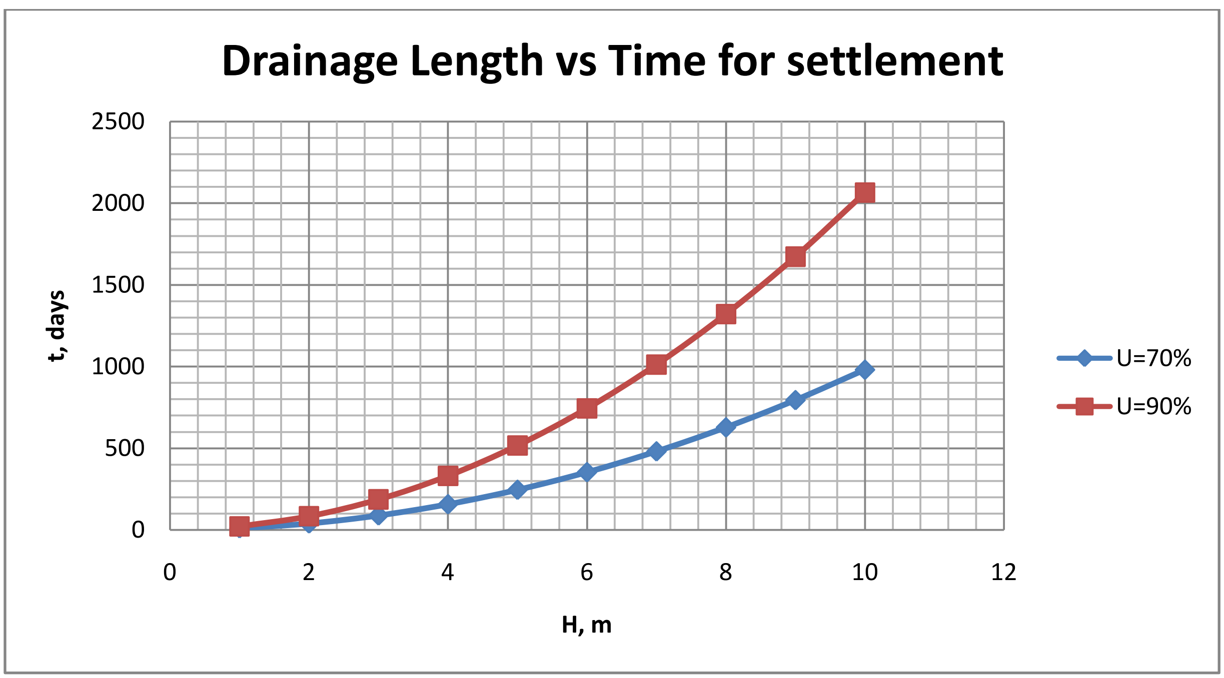
Location: Proposed Apron Area of Khan Jahan Ali Airport at Bagerhat

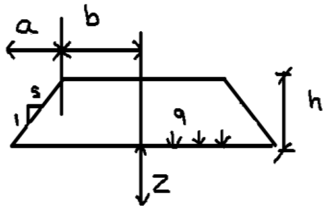
Chainage: 475 ft from the West edge of Runway

Calculation for Consolidation Settlement Time

$t = T_v / H^2 / C_v$; H= drainage length

C_V (m^2/year)	U(%)	T_V	H(m)	t (year)	t (days)
15	70	0.403	1	0.02686	10
15	70	0.403	2	0.10743	39
15	70	0.403	3	0.24171	88
15	70	0.403	4	0.42970	157
15	70	0.403	5	0.67141	245
15	70	0.403	6	0.96683	353
15	70	0.403	7	1.31596	480
15	70	0.403	8	1.71881	627
15	70	0.403	9	2.17537	794
15	70	0.403	10	2.68564	980
15	90	0.848	1	0.05653	21
15	90	0.848	2	0.22613	83
15	90	0.848	3	0.50880	186
15	90	0.848	4	0.90453	330
15	90	0.848	5	1.41333	516
15	90	0.848	6	2.03520	743
15	90	0.848	7	2.77013	1011
15	90	0.848	8	3.61813	1321
15	90	0.848	9	4.57920	1671
15	90	0.848	10	5.65333	2063





Location: Proposed Terminal Building Area of Khan Jahan Ali Airport at Bagerhat

Chainage: 700ft from the West Edge of Runway

$I_z = 1/\pi$ [$\tan^{-1}$ (b/z) + (1+b/a) { $\tan^{-1}$ (a/z+b/z)- $\tan^{-1}$(b/z)}]

$\Delta e = C_c$ / $(1+e_0)$ log $(\Delta p+p_o$ ')/p_o'

$S_c = \Delta eH.1000$ mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

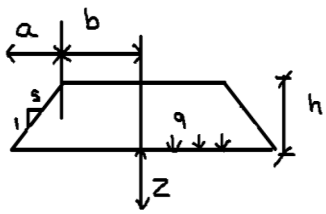
h= 2.5

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.5000	45.00	8.7	4.35	0.81	0.12	1	0.0699	69.9312
1.5	3.33	15.24	0.5000	45.00	8.7	13.05	0.81	0.12	1	0.0430	42.9720
2.5	2.00	9.14	0.4998	44.98	8.7	21.75	0.81	0.12	1	0.0323	32.2788
3.5	1.43	6.53	0.4994	44.95	8.7	30.45	0.81	0.12	1	0.0261	26.1072
4.5	1.11	5.08	0.4988	44.90	8.7	39.15	0.81	0.12	1	0.0220	21.9965
5.5	0.91	4.16	0.4979	44.81	8.7	47.85	0.81	0.12	1	0.0190	19.0290
6.5	0.77	3.52	0.4966	44.70	8.7	56.55	0.81	0.12	1	0.0168	16.7701

Total=229.0847



Location: Proposed Terminal Building Area of Khan Jahan Ali Airport at Bagerhat

Chainage: 700ft from the West Edge of Runway

I_{z1} = [tan^{-1} (a/z+b/z)- tan^{-1}(a/z) +z(a+b)/(z^2+(a+b)^2]/π

I_{z2} = [tan^{-1} (a/z+2b/z)- tan^{-1} (a/z+b/z) + 2((a+b)/a) * { tan^{-1} 2(a/z+b/z)- tan^{-1} (a/z+2b/z)-(a+b)/ (z^2+(a+b)^2)]/π

Δe= C_c / (1+e_0) log (Δp+p_0 ')/p_0'

S_c = ΔeH.1000 mm

a= 5 q= ϒh =45

b= 22.86 ϒ= 18

h= 2.5

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	$I_Z=I_{z1}+I_{z2}$	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.0317	1.43	8.7	4.35	0.81	0.12	1	0.0082	8.1720
1.5	3.33	15.24	0.0927	0.0000	0.0928	4.17	8.7	13.05	0.81	0.12	1	0.0080	7.9915
2.5	2.00	9.14	0.1474	0.0001	0.1476	6.64	8.7	21.75	0.81	0.12	1	0.0077	7.6714
3.5	1.43	6.53	0.1940	0.0004	0.1943	8.75	8.7	30.45	0.81	0.12	1	0.0073	7.2694
4.5	1.11	5.08	0.2324	0.0007	0.2331	10.49	8.7	39.15	0.81	0.12	1	0.0068	6.8360
5.5	0.91	4.16	0.2636	0.0013	0.2649	11.92	8.7	47.85	0.81	0.12	1	0.0064	6.4050
6.5	0.77	3.52	0.2888	0.0021	0.2909	13.09	8.7	56.55	0.81	0.12	1	0.0060	5.9954

Total=50.3406

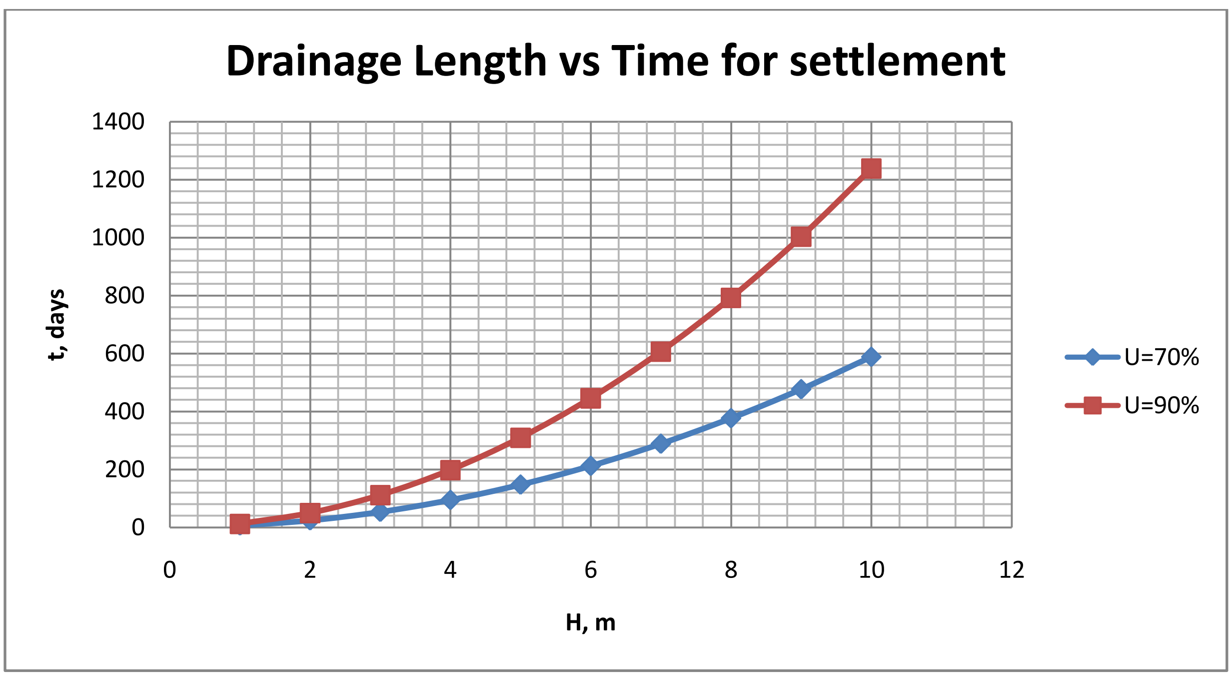
Location: Proposed Terminal Building Area of Khan Jahan Ali Airport at Bagerhat

Chainage: 700ft from the West Edge of Runway

Calculation for Consolidation Settlement Time

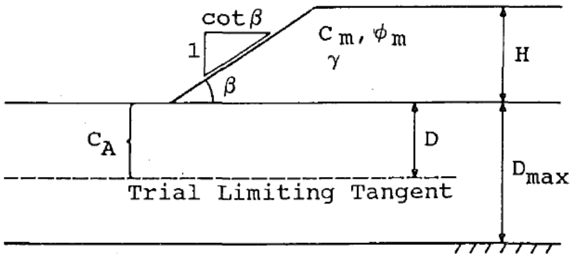
$t = T_v / H^2 / C_v$; H= drainage length

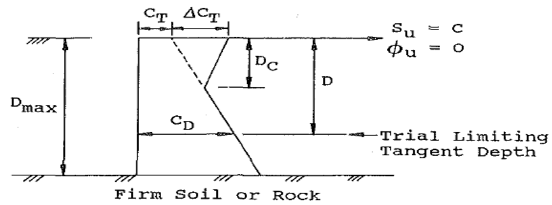
C_V (m^2/year)	U(%)	T_V	H(m)	t (year)	t (days)
25	70	0.403	1	0.01611	6
25	70	0.403	2	0.06446	24
25	70	0.403	3	0.14502	53
25	70	0.403	4	0.25782	94
25	70	0.403	5	0.40285	147
25	70	0.403	6	0.58010	212
25	70	0.403	7	0.78958	288
25	70	0.403	8	1.03129	376
25	70	0.403	9	1.30522	476
25	70	0.403	10	1.61138	588
25	90	0.848	1	0.03392	12
25	90	0.848	2	0.13568	50
25	90	0.848	3	0.30528	111
25	90	0.848	4	0.54272	198
25	90	0.848	5	0.84800	310
25	90	0.848	6	1.22112	446
25	90	0.848	7	1.66208	607
25	90	0.848	8	2.17088	792
25	90	0.848	9	2.74752	1003
25	90	0.848	10	3.39200	1238



APPENDIX B

Details of Calculation for Soil Improvement Works for the Runway Pavement of the Proposed Khan Jahan Ali Airport at Bagerhat





C_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$C_A = 0.35\ C_T + 0.65\ C_D$, for $\Delta C_T = 0$.

$C_A = 0.35\ C_T + 0.65\ C_D + 0.35\ (\frac{D_C}{D})^{1.1}\ \Delta C_T$, for $D > D_C$

$C_A = 0.35\ (C_T + \Delta C_T) + 0.65\ C_D$, for $D \le D_C$

Both C_T and ΔC_T can be positive (as shown), or negative.

Stability analysis of embankments on soft ground by B. K. Low (1989)

$$(F_s)_D = N_1 \frac{C_A}{\gamma H} + N_2 \cdot (\frac{C_m}{\gamma H} + \lambda \tan \phi_m)$$

where $N_1 = 3.06 (\frac{D}{H})^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 [(\frac{D}{H}+1)^{0.53} - (\frac{D}{H})^{0.53}] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 (\frac{D}{H}+\frac{1}{2}) + 0.1303 \frac{\cot^2 \beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 (\frac{D}{H}+\frac{1}{2}) - \frac{1}{2} (\frac{D}{H}+\frac{1}{2})^2 - \frac{1}{24} (\cot^2 \beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02 \cot \beta}{D/H}, \text{ or Fig. 4}$$

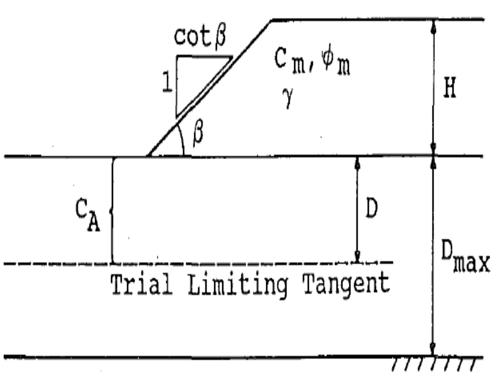
(For $D/H \ge 0.5$)

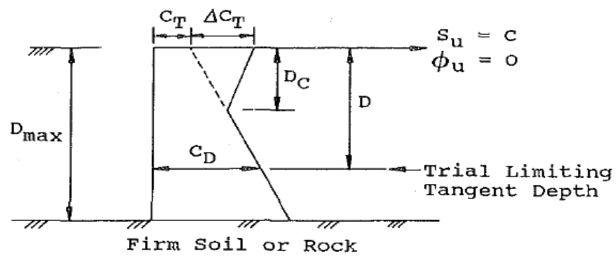
considering a surcharge of 100kPa on the runway pavement; H=6 m; Depth of runway soil layer, D=8 m

H (m)	D (m)	D/H	β (°)	α_1	α_2	λ	N_1	N_2	N_1+N_2	C_m (kPa)	Φ_m (°)	Ɣ (kN/m³)	C_T (kPa)	C_D (kPa)	C_A (kPa)	$(F_s)_D$
6	1	0.17	26.565	2.02	0.92	0.43	3.63	3.28	6.91	30	10	18	10	10.00	10.00	1.50
6	2	0.33	26.565	2.09	1.18	0.31	4.26	2.31	6.57	30	10	18	10	10.62	10.40	1.18
6	3	0.50	26.565	2.22	1.51	0.27	4.53	1.79	6.32	30	10	18	10	10.75	10.49	1.02
6	4	0.67	26.565	2.38	1.89	0.25	4.68	1.46	6.14	30	10	18	10	11.50	10.98	0.95
6	5	0.83	26.565	2.57	2.33	0.24	4.78	1.24	6.02	30	10	18	10	12.00	11.30	0.90
6	6	1.00	26.565	2.78	2.84	0.23	4.85	1.08	5.93	30	10	18	10	13.75	12.44	0.90
6	7	1.17	26.565	3.00	3.40	0.22	4.91	0.95	5.86	30	10	18	10	14.40	12.86	0.89
6	8	1.33	26.565	3.22	4.02	0.22	4.95	0.86	5.81	30	10	18	10	15.00	13.25	0.88

Here it is clear that factor of safety is not acceptable from D=2 m depth

Suitable $(F_s)_D \ge 1.2$; is OK ; but in this it is not OK; several stage of loading is required; let's study how much stage is needed





C_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$C_A = 0.35\ C_T + 0.65\ C_D$, for $\Delta C_T = 0$.

$C_A = 0.35\ C_T + 0.65\ C_D + 0.35\ (\frac{D_C}{D})^{1.1}\ \Delta C_T$, for $D > D_C$

$C_A = 0.35\ (C_T + \Delta C_T) + 0.65\ C_D$, for $D \le D_C$

Both C_T and ΔC_T can be positive (as shown), or negative.

Stability analysis of embankments on soft ground by B. K. Low (1989)

$$(F_s)_D = N_1 \frac{C_A}{\gamma H} + N_2 \cdot \left(\frac{C_m}{\gamma H} + \lambda \tan \phi_m\right)$$

where $N_1 = 3.06 \left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 \left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53} \right] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 \left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303 \frac{\cot^2 \beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 \left(\frac{D}{H}+\frac{1}{2}\right) - \frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2 - \frac{1}{24}(\cot^2 \beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02 \cot \beta}{D/H}, \text{ or Fig. 4}$$

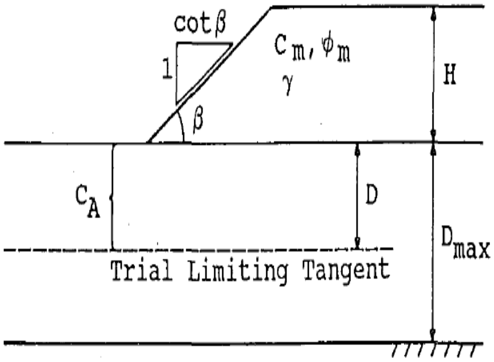
(For $D/H \ge 0.5$)

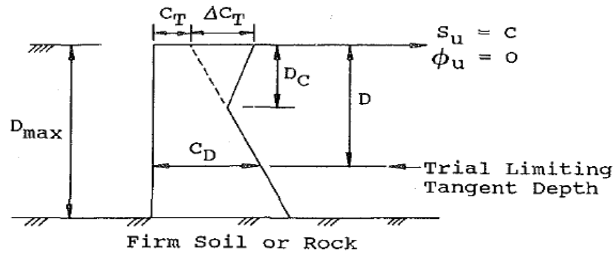
considering a surcharge of 100kPa on the runway pavement; H=3.5 m; Depth of runway soil layer, D=8 m

1st stage of loading

H (m)	D (m)	D/H	β (°)	α_1	α_2	λ	N_1	N_2	N_1+N_2	C_m (kPa)	Φ_m (°)	Υ (kN/m³)	C_T (kPa)	C_D (kPa)	C_A (kPa)	$(F_s)_D$
3.5	1	0.29	26.565	2.06	1.10	0.33	4.14	2.52	6.66	30	10	18	10	10.00	10.00	2.00
3.5	2	0.57	26.565	2.28	1.66	0.26	4.60	1.63	6.23	30	10	18	10	10.62	10.40	1.61
3.5	3	0.86	26.565	2.60	2.40	0.24	4.79	1.21	6.00	30	10	18	10	10.75	10.49	1.43
3.5	4	1.14	26.565	2.97	3.31	0.23	4.90	0.97	5.87	30	10	18	10	11.50	10.98	1.35
3.5	5	1.43	26.565	3.35	4.40	0.22	4.98	0.81	5.79	30	10	18	10	12.00	11.30	1.31
3.5	6	1.71	26.565	3.76	5.66	0.21	5.04	0.69	5.73	30	10	18	10	13.75	12.44	1.35
3.5	7	2.00	26.565	4.17	7.09	0.21	5.08	0.61	5.69	30	10	18	10	14.40	12.86	1.35
3.5	8	2.29	26.565	4.59	8.70	0.21	5.12	0.54	5.66	30	10	18	10	15.00	13.25	1.36

Here H=3.5 m for 1st stage loading and $(F_s)_D \ge 1.2$; is OK





C_A = Equivalent shear strength (over a circular arc), corresponding to a trial limiting tangent of depth D.

$C_A = 0.35\ C_T + 0.65\ C_D$, for $\Delta C_T = 0$.

$C_A = 0.35\ C_T + 0.65\ C_D + 0.35\ (\frac{D_C}{D})^{1.1}\ \Delta C_T$, for $D > D_C$

$C_A = 0.35\ (C_T + \Delta C_T) + 0.65\ C_D$, for $D \leq D_C$

Both C_T and ΔC_T can be positive (as shown), or negative.

Stability analysis of embankments on soft ground by B. K. Low (1989)

$$(F_s)_D = N_1 \frac{C_A}{\gamma H} + N_2 \cdot \left(\frac{C_m}{\gamma H} + \lambda \tan \phi_m\right)$$

where $N_1 = 3.06 \left(\frac{D}{H}\right)^{0.53} \alpha_1^{1.47} / \alpha_2$

$$N_2 = 1.53 \left[\left(\frac{D}{H}+1\right)^{0.53} - \left(\frac{D}{H}\right)^{0.53} \right] \alpha_1^{1.47} / \alpha_2$$

$$\alpha_1 = 1.564 \left(\frac{D}{H}+\frac{1}{2}\right) + 0.1303 \frac{\cot^2 \beta + 1}{\frac{D}{H}+0.5}$$

$$\alpha_2 = \alpha_1 \left(\frac{D}{H}+\frac{1}{2}\right) - \frac{1}{2}\left(\frac{D}{H}+\frac{1}{2}\right)^2 - \frac{1}{24}(\cot^2 \beta + 1)$$

$$\lambda \simeq 0.19 + \frac{0.02 \cot \beta}{D/H}, \text{ or Fig. 4}$$

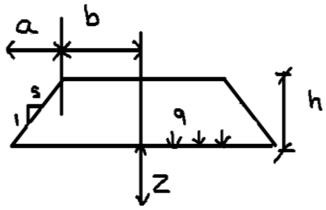
(For $D/H \geq 0.5$)

considering a surcharge of 100kPa on the runway pavement; H=6 m; Depth of runway soil layer, D=8 m

2nd stage of loading for 90% consolidation

H (m)	D (m)	D/H	β (°)	α_1	α_2	λ	N_1	N_2	N_1+N_2	C_m (kPa)	Φ_m (°)	Υ (kN/m³)	C_T (kPa)	C_D (kPa)	C_A (kPa)	$(F_s)_D$
6	1	0.17	26.565	2.02	0.92	0.43	3.63	3.28	6.91	30	10	18	19.07	19.07	19.07	1.80
6	2	0.33	26.565	2.09	1.18	0.31	4.26	2.31	6.57	30	10	18	19.07	19.69	19.47	1.54
6	3	0.50	26.565	2.22	1.51	0.27	4.53	1.79	6.32	30	10	18	19.07	19.82	19.56	1.40
6	4	0.67	26.565	2.38	1.89	0.25	4.68	1.46	6.14	30	10	18	19.07	20.57	20.05	1.34
6	5	0.83	26.565	2.57	2.33	0.24	4.78	1.24	6.02	30	10	18	19.07	21.07	20.37	1.30
6	6	1.00	26.565	2.78	2.84	0.23	4.85	1.08	5.93	30	10	18	19.07	22.82	21.51	1.31
6	7	1.17	26.565	3.00	3.40	0.22	4.91	0.95	5.86	30	10	18	19.07	23.47	21.93	1.31
6	8	1.33	26.565	3.22	4.02	0.22	4.95	0.86	5.81	30	10	18	19.07	24.07	22.32	1.32

Here H=6 m for 2nd stage loading and $(F_s)_D \geq 1.2$; is OK



<u>Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat</u>

<u>Chainage: 1500 ft from North End</u>

$I_z = 1/\pi\ [\ \tan^{-1}(b/z) + (1+b/a)\ \{\ \tan^{-1}(a/z+b/z) - \tan^{-1}(b/z)\}]$

$\Delta e = C_c\ /\ (1+e_0)\ \log\ (\Delta p + p_o')/p_o'$

$S_c = \Delta e H.1000$ mm

a= 5 q= γh =63

b= 22.86 γ= 18

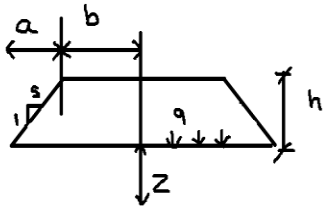
h= 3.5

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p=2I_z q$	γ' (kN/m³)	$P_0'=\gamma' z$	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.0000	45.7200	0.5000	62.9996	8	4	1	0.2	1	0.1224	122.4013
1.5	3.3333	15.2400	0.5000	62.9942	8	12	1	0.2	1	0.0796	79.5847
2.5	2.0000	9.1440	0.4998	62.9740	8	20	1	0.2	1	0.0618	61.7912
3.5	1.4286	6.5314	0.4994	62.9298	8	28	1	0.2	1	0.0512	51.1548
4.5	1.1111	5.0800	0.4988	62.8532	8	36	1	0.2	1	0.0439	43.8688
5.5	0.9091	4.1564	0.4979	62.7371	8	44	1	0.2	1	0.0385	38.4863
6.5	0.7692	3.5169	0.4966	62.5755	8	52	1	0.2	1	0.0343	34.3088
7.5	0.6667	3.0480	0.4950	62.3640	8	60	1	0.2	1	0.0310	30.9502

Total=462.5462



Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 1500 ft from North End

I_{z1} = [$\tan^{-1}$ (a/z+b/z)- $\tan^{-1}$(a/z) +z(a+b)/(z^2+$(a+b)^2$]/π

I_{z2} = [$\tan^{-1}$ (a/z+2b/z)- $\tan^{-1}$ (a/z+b/z) + 2((a+b)/a) * { $\tan^{-1}$ 2(a/z+b/z)- $\tan^{-1}$ (a/z+2b/z)-(a+b)/ (z^2+$(a+b)^2$)]/π

$\Delta e = C_c / (1+e_0) \log (\Delta p+p_0')/p_0'$

$S_c = \Delta eH.1000$ mm

a= 5 q= ϒh =63

b= 22.86 ϒ= 18

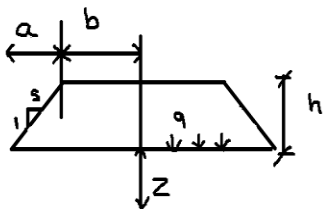
h=3.5

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	$I_Z=I_{z1}+I_{z2}$	$\Delta p=2I_zq$	ϒ' (kN/m³)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.0317	2.00	8	4	1	0.2	1	0.02	17.60
1.5	3.33	15.24	0.0927	0.0000	0.0928	5.84	8	12	1	0.2	1	0.02	17.23
2.5	2.00	9.14	0.1474	0.0001	0.1476	9.30	8	20	1	0.2	1	0.02	16.58
3.5	1.43	6.53	0.1940	0.0004	0.1943	12.24	8	28	1	0.2	1	0.02	15.75
4.5	1.11	5.08	0.2324	0.0007	0.2331	14.69	8	36	1	0.2	1	0.01	14.86
5.5	0.91	4.16	0.2636	0.0013	0.2649	16.69	8	44	1	0.2	1	0.01	13.97
6.5	0.77	3.52	0.2888	0.0021	0.2909	18.33	8	52	1	0.2	1	0.01	13.11
7.5	0.67	3.05	0.3090	0.0032	0.3122	19.67	8	60	1	0.2	1	0.01	12.32

Total=121.42



Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 1500 ft from North End

$I_z = 1/\pi\ [\ \tan^{-1}(b/z) + (1+b/a)\ \{\ \tan^{-1}(a/z+b/z) - \tan^{-1}(b/z)\}]$

$\Delta e = C_c\ /\ (1+e_0)\ \log\ (\Delta p + p_o')/p_o'$

$S_c = \Delta e H.1000$ mm

a= 5 q= ϒh =108

b= 22.86 ϒ= 18

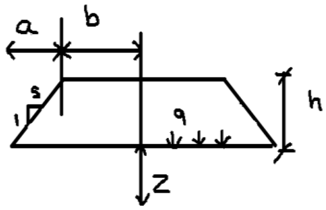
h= 6

s= 2

Calculations for Consolidation Settlement at centre of Embankment

Z (m)	a/z	b/z	I_z	$\Delta p = 2I_zq$	ϒ' (kN/m^3)	P_0'=ϒ'z	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.0000	45.7200	0.5000	107.9994	8	4	1	0.2	1	0.1447	144.7156
1.5	3.3333	15.2400	0.5000	107.9901	8	12	1	0.2	1	0.1000	99.9964
2.5	2.0000	9.1440	0.4998	107.9555	8	20	1	0.2	1	0.0806	80.6029
3.5	1.4286	6.5314	0.4994	107.8797	8	28	1	0.2	1	0.0686	68.5997
4.5	1.1111	5.0800	0.4988	107.7484	8	36	1	0.2	1	0.0601	60.1301
5.5	0.9091	4.1564	0.4979	107.5493	8	44	1	0.2	1	0.0537	53.7101
6.5	0.7692	3.5169	0.4966	107.2722	8	52	1	0.2	1	0.0486	48.6137
7.5	0.6667	3.0480	0.4950	106.9097	8	60	1	0.2	1	0.0444	44.4330

Total=600.8014



Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 1500 ft from North End

$I_{z1} = [\tan^{-1}(a/z+b/z) - \tan^{-1}(a/z) + z(a+b)/(z^2+(a+b)^2)]/\pi$

$I_{z2} = [\tan^{-1}(a/z+2b/z) - \tan^{-1}(a/z+b/z) + 2((a+b)/a) * \{ \tan^{-1} 2(a/z+b/z) - \tan^{-1}(a/z+2b/z) - (a+b)/(z^2+(a+b)^2)\}]/\pi$

$\Delta e = C_c / (1+e_0) \log (\Delta p + p_0')/p_0'$

$S_c = \Delta e H . 1000$ mm

a= 5 q= ϒh =108

b= 22.86 ϒ= 18

h= 6

s= 2

Calculation for Consolidation Settlement at Embankment Toe

Z (m)	a/z	b/z	I_{z1}	I_{z2}	$I_Z=I_{z1}+I_{z2}$	$\Delta p=2I_zq$	ϒ' (kN/m³)	$P_0'=ϒ'z$	e_0	C_c	H (m)	Δe	S_c (mm)
0.5	10.00	45.72	0.0317	0.0000	0.0317	3.43	8	4	1	0.2	1	0.03	26.87
1.5	3.33	15.24	0.0927	0.0000	0.0928	10.02	8	12	1	0.2	1	0.03	26.36
2.5	2.00	9.14	0.1474	0.0001	0.1476	15.94	8	20	1	0.2	1	0.03	25.45
3.5	1.43	6.53	0.1940	0.0004	0.1943	20.99	8	28	1	0.2	1	0.02	24.29
4.5	1.11	5.08	0.2324	0.0007	0.2331	25.18	8	36	1	0.2	1	0.02	23.03
5.5	0.91	4.16	0.2636	0.0013	0.2649	28.61	8	44	1	0.2	1	0.02	21.75
6.5	0.77	3.52	0.2888	0.0021	0.2909	31.42	8	52	1	0.2	1	0.02	20.53
7.5	0.67	3.05	0.3090	0.0032	0.3122	33.72	8	60	1	0.2	1	0.02	19.37

Total=187.66

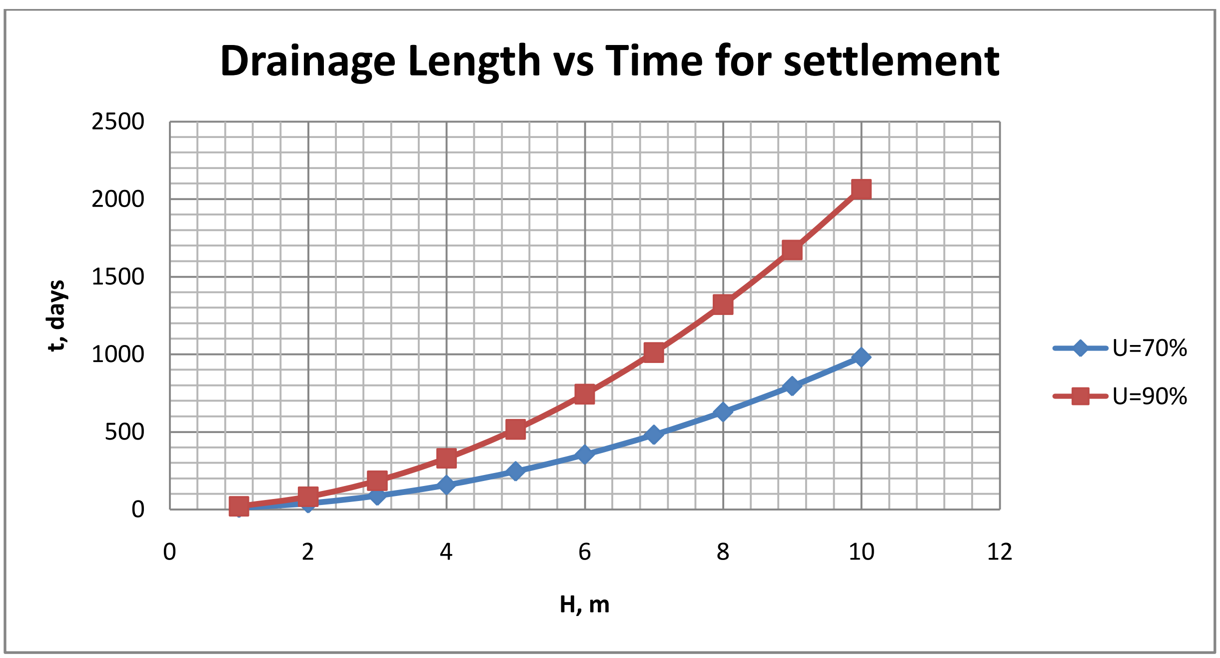
Location: Proposed Runway of Khan Jahan Ali Airport at Bagerhat

Chainage: 1500 ft from North End

Calculation for Consolidation Settlement Time

$t=T_v/H^2/C_v$; H= drainage length

C_V (m^2/year)	U(%)	T_V	H(m)	t (year)	t (days)
15	70	0.403	1	0.02686	10
15	70	0.403	2	0.10743	39
15	70	0.403	3	0.24171	88
15	70	0.403	4	0.42970	157
15	70	0.403	5	0.67141	245
15	70	0.403	6	0.96683	353
15	70	0.403	7	1.31596	480
15	70	0.403	8	1.71881	627
15	70	0.403	9	2.17537	794
15	70	0.403	10	2.68564	980
15	90	0.848	1	0.05653	21
15	90	0.848	2	0.22613	83
15	90	0.848	3	0.50880	186
15	90	0.848	4	0.90453	330
15	90	0.848	5	1.41333	516
15	90	0.848	6	2.03520	743
15	90	0.848	7	2.77013	1011
15	90	0.848	8	3.61813	1321
15	90	0.848	9	4.57920	1671
15	90	0.848	10	5.65333	2063



$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

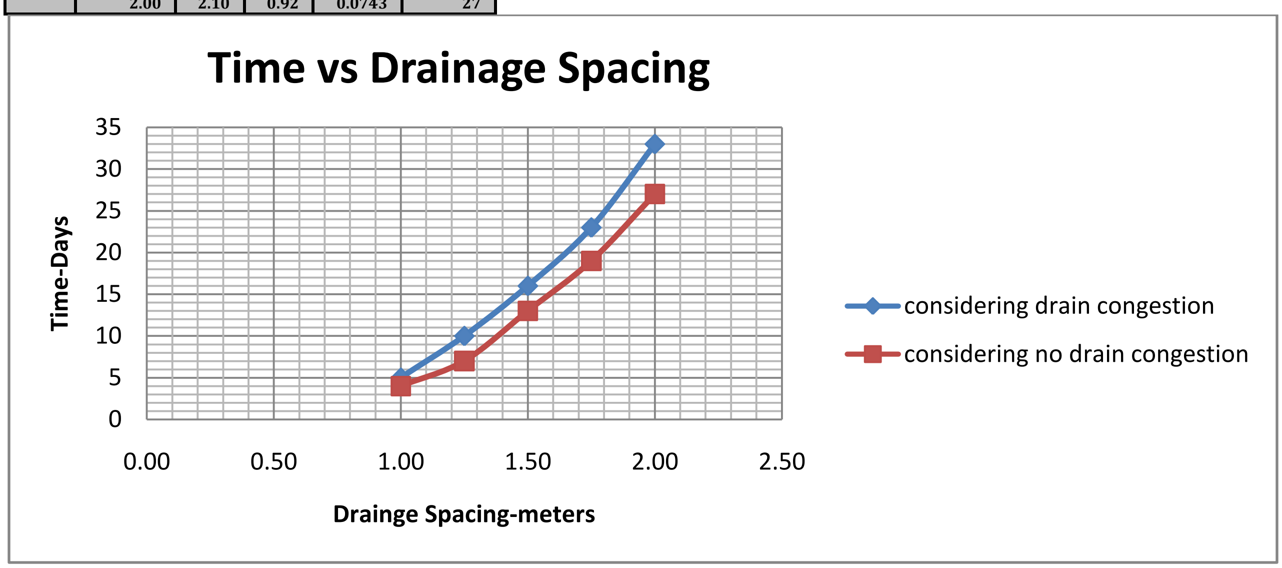
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	4.00	1.60	0.92	0.0143	5
1.25	1.31	6.56	4.00	1.60	0.92	0.0263	10
1.5	1.58	7.88	4.00	1.60	0.92	0.0427	16
1.75	1.84	9.19	4.00	1.60	0.92	0.0636	23
2	2.10	10.50	4.00	1.60	0.92	0.0893	33

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.92	0.0105	4
1.25	1.25	1.31	0.92	0.0205	7
1.5	1.50	1.58	0.92	0.0343	13
1.75	1.75	1.84	0.92	0.0522	19
2	2.00	2.10	0.92	0.0743	27



Sand drain design (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

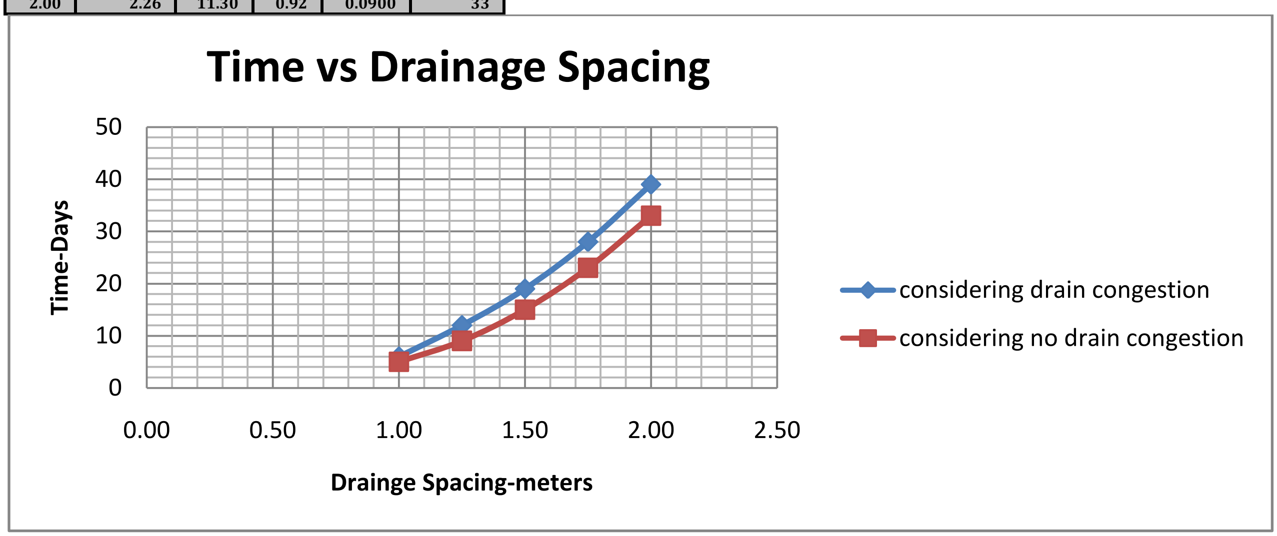
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	4.00	1.60	0.92	0.0175	6
1.25	1.41	7.06	4.00	1.60	0.92	0.0321	12
1.5	1.70	8.48	4.00	1.60	0.92	0.0517	19
1.75	1.98	9.89	4.00	1.60	0.92	0.0767	28
2	2.26	11.30	4.00	1.60	0.92	0.1073	39

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	5.65	0.92	0.0132	5
1.25	1.41	7.06	0.92	0.0253	9
1.50	1.70	8.48	0.92	0.0419	15
1.75	1.98	9.89	0.92	0.0634	23
2.00	2.26	11.30	0.92	0.0900	33



Sand drain design (square)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

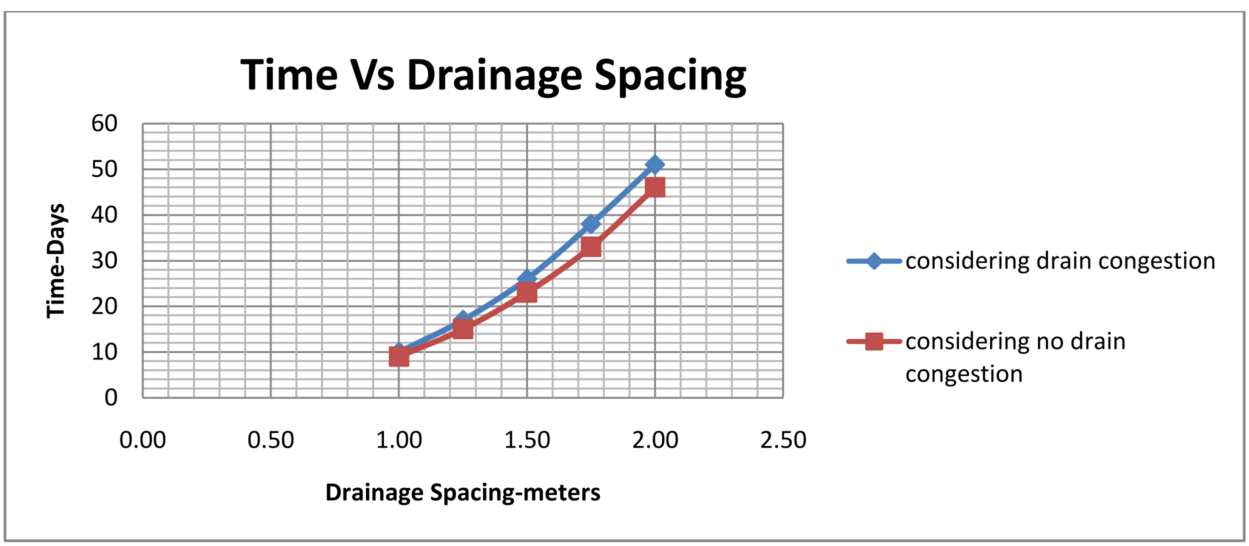
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	4.00	1.60	0.92	0.0271	10
1.25	1.31	19.82	4.00	1.60	0.92	0.0464	17
1.50	1.58	23.79	4.00	1.60	0.92	0.0716	26
1.75	1.84	27.75	4.00	1.60	0.92	0.1029	38
2.00	2.10	31.72	4.00	1.60	0.92	0.1406	51

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.92	0.0234	9
1.25	1.31	19.82	0.92	0.0406	15
1.50	1.58	23.79	0.92	0.0632	23
1.75	1.84	27.75	0.92	0.0914	33
2.00	2.10	31.72	0.92	0.1256	46



PVD design (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

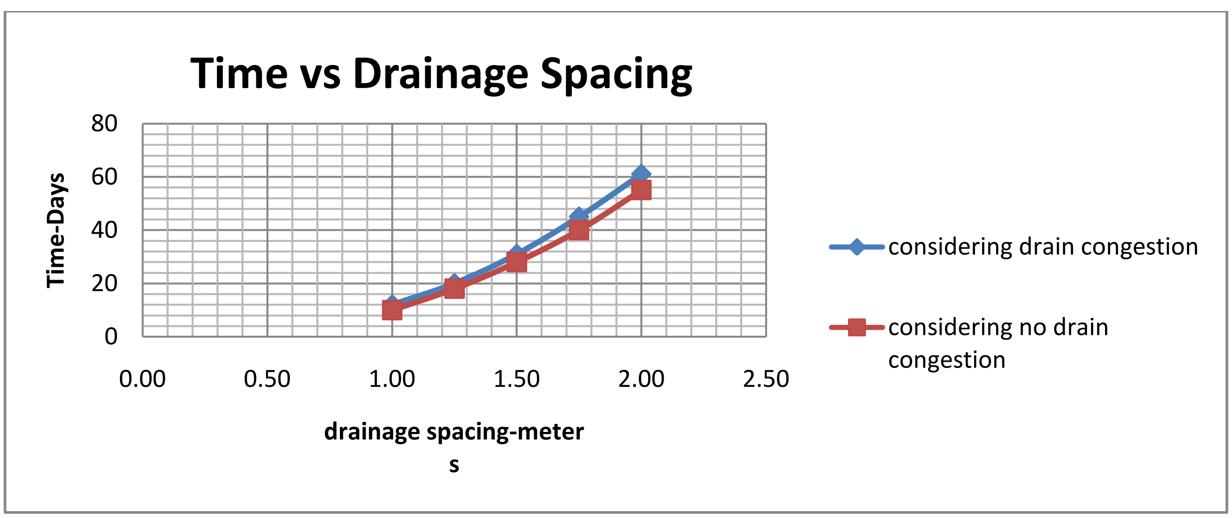
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	4.00	1.60	0.92	0.0324	12
1.25	1.41	21.33	4.00	1.60	0.92	0.0553	20
1.50	1.70	25.60	4.00	1.60	0.92	0.0851	31
1.75	1.98	29.87	4.00	1.60	0.92	0.1222	45
2.00	2.26	34.13	4.00	1.60	0.92	0.1667	61

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.92	0.0280	10
1.25	1.41	21.33	0.92	0.0485	18
1.50	1.70	25.60	0.92	0.0754	28
1.75	1.98	29.87	0.92	0.1089	40
2.00	2.26	34.13	0.92	0.1494	55



PVD design (square)

APPENDIX C

Details of Calculation of Design of Sand Drains and Prefabricated Vertical Drains for Parametric Study for the Runway Pavement of the Proposed Khan Jahan Ali Airport at Bagerhat

t=(D^2/8C_h)*[ln (D/d)-0.75]*ln(1/1-U) ; Considering no drain congestion

t=(D^2/8C_h)*[ln (D/d)-0.75+π*z*(2I-z)*(k_c/q_w)]*ln(1/1-U) ; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

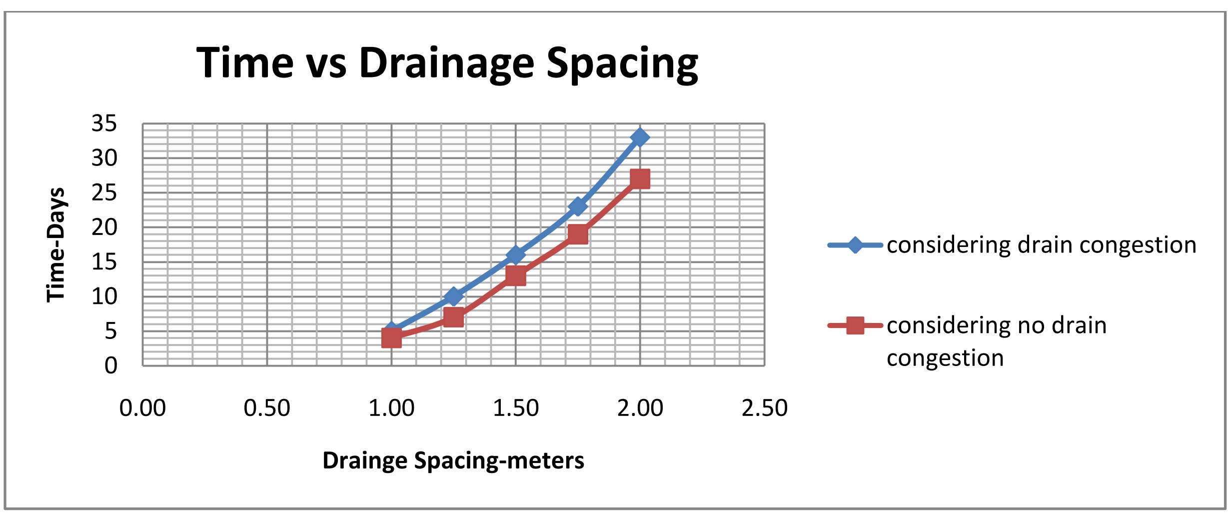
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	4.00	1.60	0.92	0.0143	5
1.25	1.31	6.56	4.00	1.60	0.92	0.0263	10
1.5	1.58	7.88	4.00	1.60	0.92	0.0427	16
1.75	1.84	9.19	4.00	1.60	0.92	0.0636	23
2	2.10	10.50	4.00	1.60	0.92	0.0893	33

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.92	0.0105	4
1.25	1.25	1.31	0.92	0.0205	7
1.5	1.50	1.58	0.92	0.0343	13
1.75	1.75	1.84	0.92	0.0522	19
2	2.00	2.10	0.92	0.0743	27



Sand drain design (triangular)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 8 m

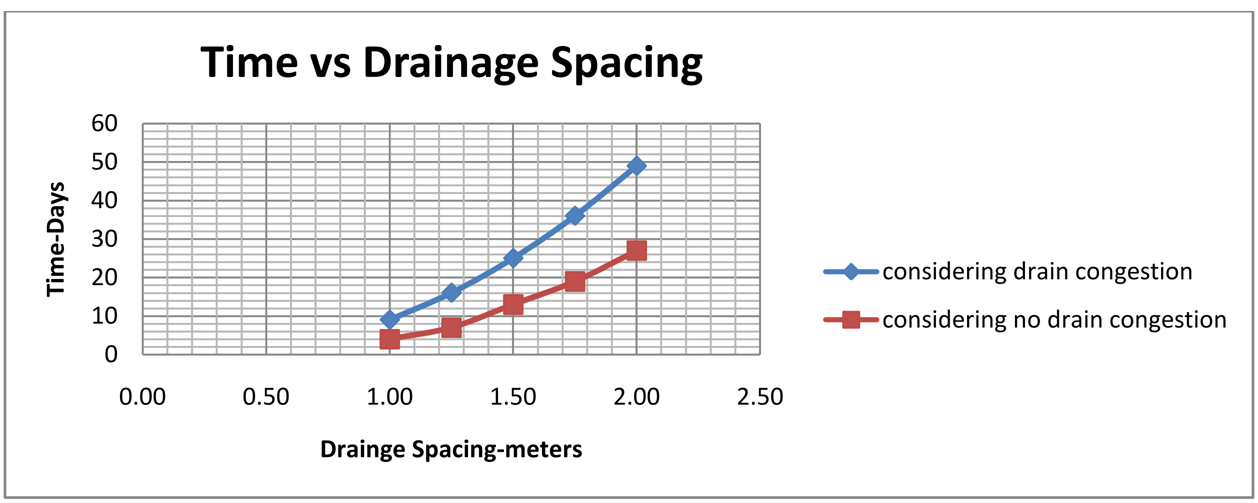
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	8.00	3.20	0.92	0.0255	9
1.25	1.31	6.56	8.00	3.20	0.92	0.0438	16
1.5	1.58	7.88	8.00	3.20	0.92	0.0679	25
1.75	1.84	9.19	8.00	3.20	0.92	0.0979	36
2	2.10	10.50	8.00	3.20	0.92	0.1340	49

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.92	0.0105	4
1.25	1.25	1.31	0.92	0.0205	7
1.5	1.50	1.58	0.92	0.0343	13
1.75	1.75	1.84	0.92	0.0522	19
2	2.00	2.10	0.92	0.0743	27



Sand drain design (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

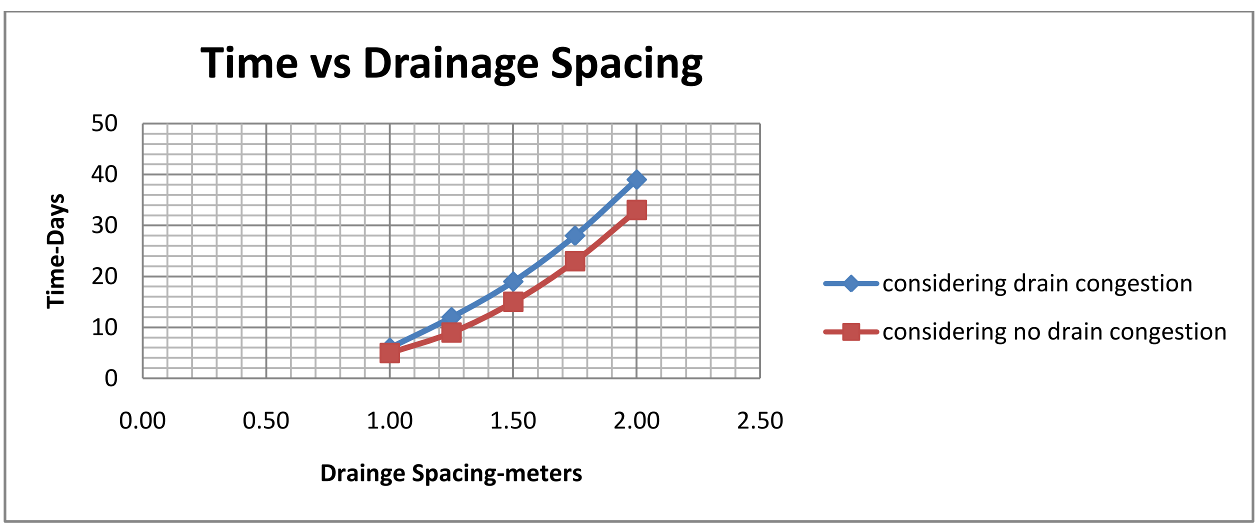
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	4.00	1.60	0.92	0.0175	6
1.25	1.41	7.06	4.00	1.60	0.92	0.0321	12
1.5	1.70	8.48	4.00	1.60	0.92	0.0517	19
1.75	1.98	9.89	4.00	1.60	0.92	0.0767	28
2	2.26	11.30	4.00	1.60	0.92	0.1073	39

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	5.65	0.92	0.0132	5
1.25	1.41	7.06	0.92	0.0253	9
1.50	1.70	8.48	0.92	0.0419	15
1.75	1.98	9.89	0.92	0.0634	23
2.00	2.26	11.30	0.92	0.0900	33



Sand drain design (square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 8 m

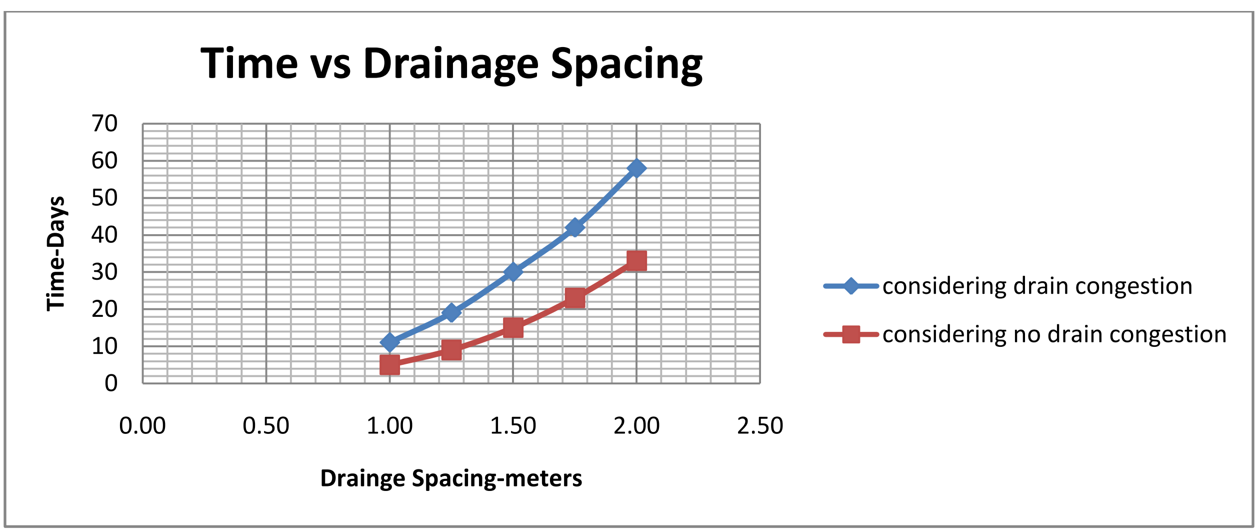
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	8.00	3.20	0.92	0.0305	11
1.25	1.41	7.06	8.00	3.20	0.92	0.0523	19
1.5	1.70	8.48	8.00	3.20	0.92	0.0808	30
1.75	1.98	9.89	8.00	3.20	0.92	0.1164	42
2	2.26	11.30	8.00	3.20	0.92	0.1592	58

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	5.65	0.92	0.0132	5
1.25	1.41	7.06	0.92	0.0253	9
1.50	1.70	8.48	0.92	0.0419	15
1.75	1.98	9.89	0.92	0.0634	23
2.00	2.26	11.30	0.92	0.0900	33



Sand drain design (square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 4 m

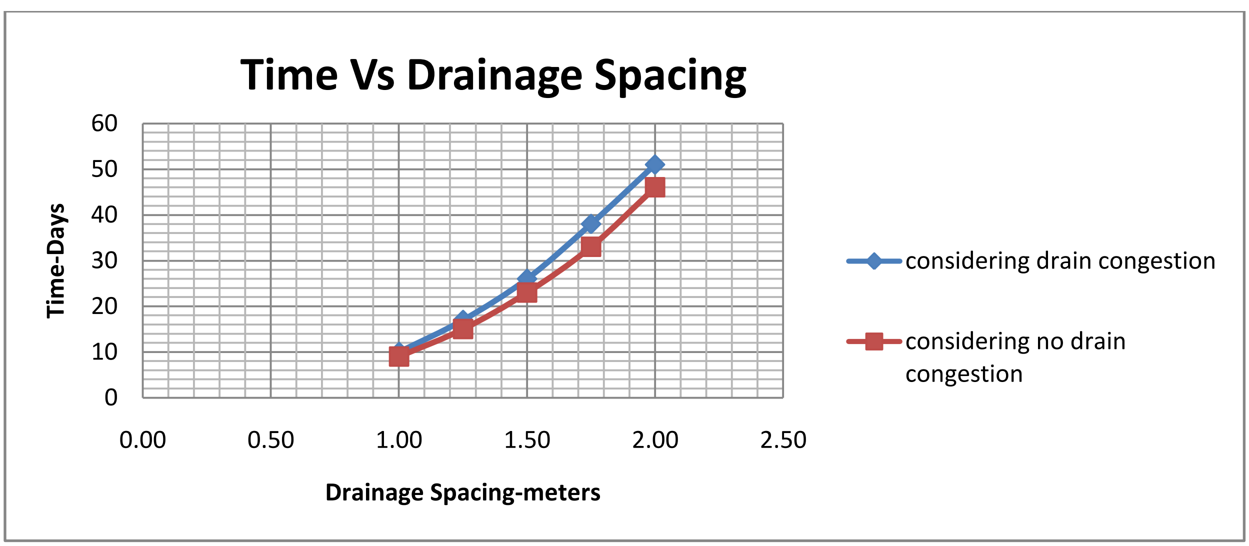
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	4.00	1.60	0.92	0.0271	10
1.25	1.31	19.82	4.00	1.60	0.92	0.0464	17
1.50	1.58	23.79	4.00	1.60	0.92	0.0716	26
1.75	1.84	27.75	4.00	1.60	0.92	0.1029	38
2.00	2.10	31.72	4.00	1.60	0.92	0.1406	51

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.92	0.0234	9
1.25	1.31	19.82	0.92	0.0406	15
1.50	1.58	23.79	0.92	0.0632	23
1.75	1.84	27.75	0.92	0.0914	33
2.00	2.10	31.72	0.92	0.1256	46



PVD design (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 8 m

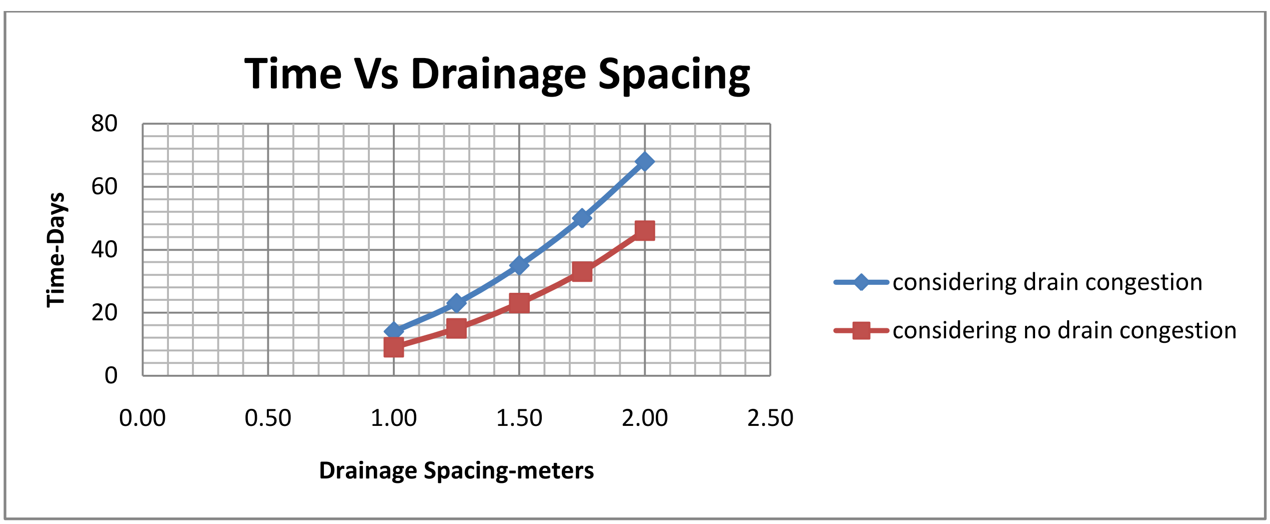
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	8.00	3.20	0.92	0.0383	14
1.25	1.31	19.82	8.00	3.20	0.92	0.0639	23
1.50	1.58	23.79	8.00	3.20	0.92	0.0967	35
1.75	1.84	27.75	8.00	3.20	0.92	0.1372	50
2.00	2.10	31.72	8.00	3.20	0.92	0.1853	68

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.92	0.0234	9
1.25	1.31	19.82	0.92	0.0406	15
1.50	1.58	23.79	0.92	0.0632	23
1.75	1.84	27.75	0.92	0.0914	33
2.00	2.10	31.72	0.92	0.1256	46



PVD design (triangular)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

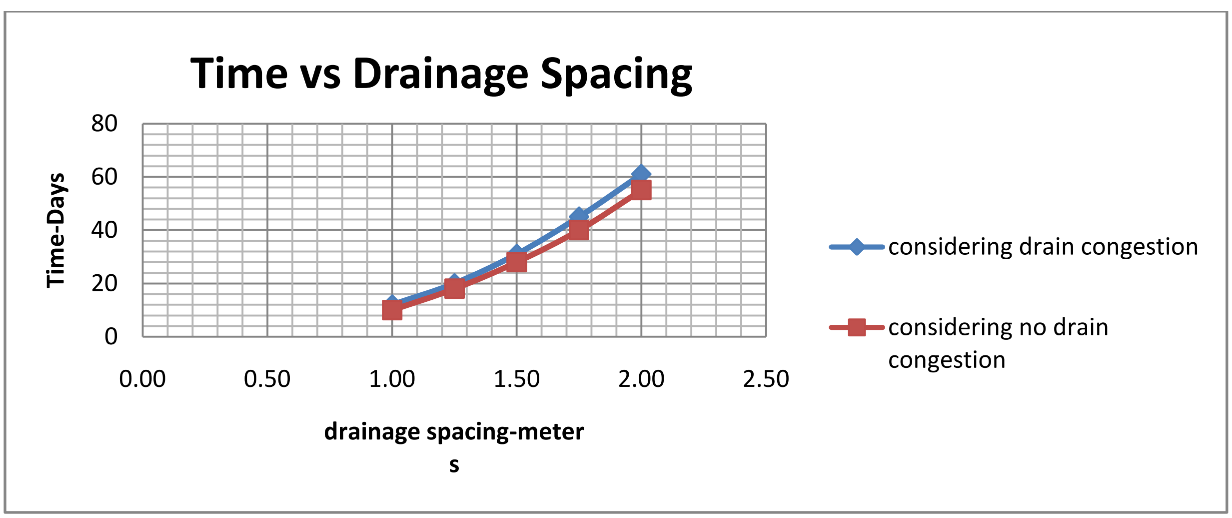
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	4.00	1.60	0.92	0.0324	12
1.25	1.41	21.33	4.00	1.60	0.92	0.0553	20
1.50	1.70	25.60	4.00	1.60	0.92	0.0851	31
1.75	1.98	29.87	4.00	1.60	0.92	0.1222	45
2.00	2.26	34.13	4.00	1.60	0.92	0.1667	61

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.92	0.0280	10
1.25	1.41	21.33	0.92	0.0485	18
1.50	1.70	25.60	0.92	0.0754	28
1.75	1.98	29.87	0.92	0.1089	40
2.00	2.26	34.13	0.92	0.1494	55



PVD design (square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 8 m

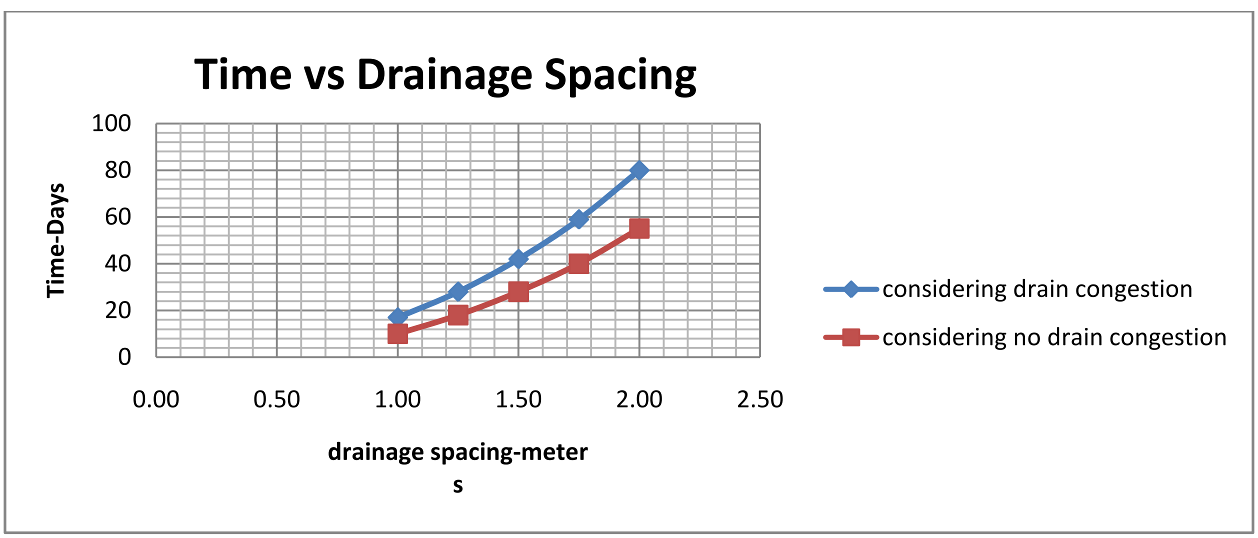
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	8.00	3.20	0.92	0.0453	17
1.25	1.41	21.33	8.00	3.20	0.92	0.0755	28
1.50	1.70	25.60	8.00	3.20	0.92	0.1143	42
1.75	1.98	29.87	8.00	3.20	0.92	0.1619	59
2.00	2.26	34.13	8.00	3.20	0.92	0.2186	80

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.92	0.0280	10
1.25	1.41	21.33	0.92	0.0485	18
1.50	1.70	25.60	0.92	0.0754	28
1.75	1.98	29.87	0.92	0.1089	40
2.00	2.26	34.13	0.92	0.1494	55



PVD design (square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

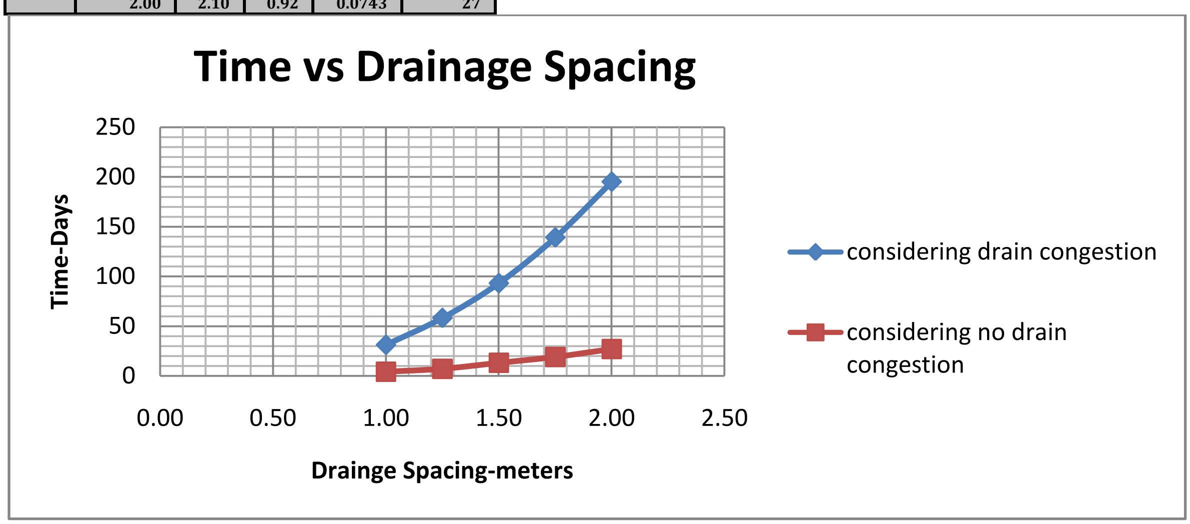
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	4.00	1.60	0.92	0.0856	31
1.25	1.31	6.56	4.00	1.60	0.92	0.1581	58
1.5	1.58	7.88	4.00	1.60	0.92	0.2562	93
1.75	1.84	9.19	4.00	1.60	0.92	0.3815	139
2	2.10	10.50	4.00	1.60	0.92	0.5355	195

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.92	0.0105	4
1.25	1.25	1.31	0.92	0.0205	7
1.5	1.50	1.58	0.92	0.0343	13
1.75	1.75	1.84	0.92	0.0522	19
2	2.00	2.10	0.92	0.0743	27



Sand Drain (Triangular)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

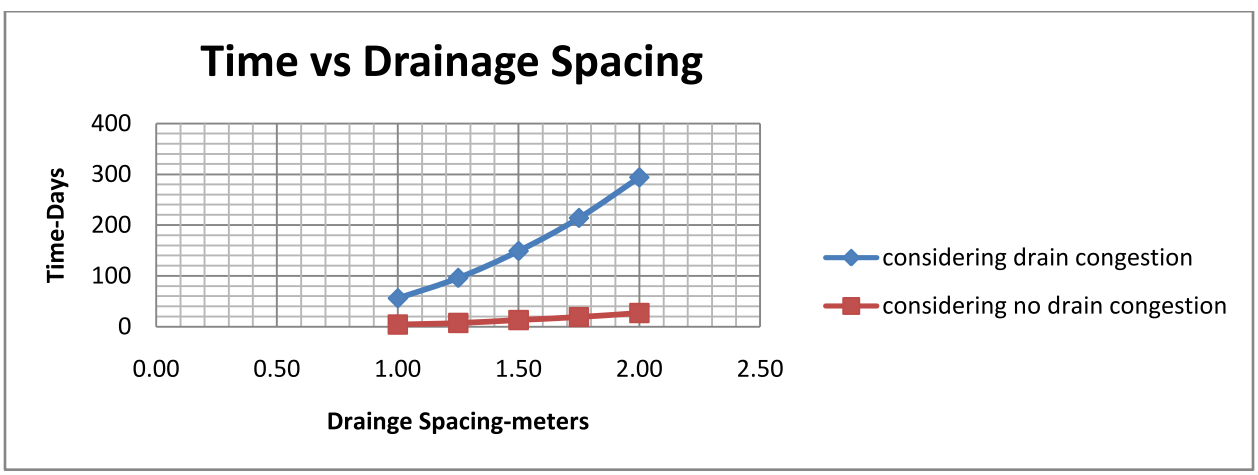
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	8.00	3.20	0.92	0.1528	56
1.25	1.31	6.56	8.00	3.20	0.92	0.2630	96
1.5	1.58	7.88	8.00	3.20	0.92	0.4073	149
1.75	1.84	9.19	8.00	3.20	0.92	0.5873	214
2	2.10	10.50	8.00	3.20	0.92	0.8042	294

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.92	0.0105	4
1.25	1.25	1.31	0.92	0.0205	7
1.5	1.50	1.58	0.92	0.0343	13
1.75	1.75	1.84	0.92	0.0522	19
2	2.00	2.10	0.92	0.0743	27



Sand Drain (Triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

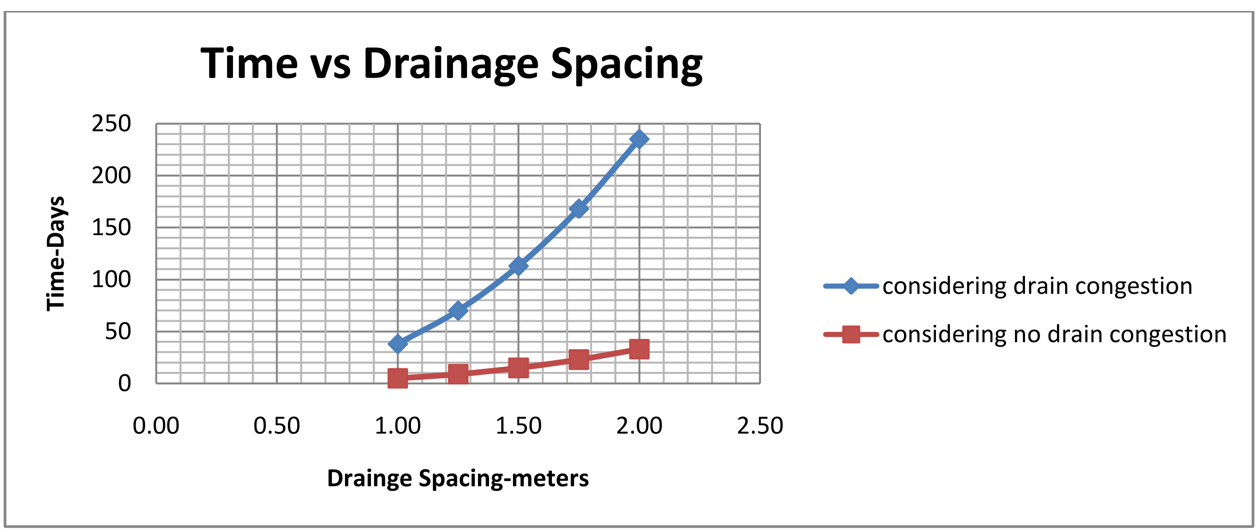
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	4.00	1.60	0.92	0.1051	38
1.25	1.41	7.06	4.00	1.60	0.92	0.1923	70
1.5	1.70	8.48	4.00	1.60	0.92	0.3100	113
1.75	1.98	9.89	4.00	1.60	0.92	0.4600	168
2	2.26	11.30	4.00	1.60	0.92	0.6439	235

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1	1.13	5.65	0.92	0.0132	5
1.25	1.41	7.06	0.92	0.0253	9
1.5	1.70	8.48	0.92	0.0419	15
1.75	1.98	9.89	0.92	0.0634	23
2	2.26	11.30	0.92	0.0900	33



Sand Drain (Square)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 5 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 8 m

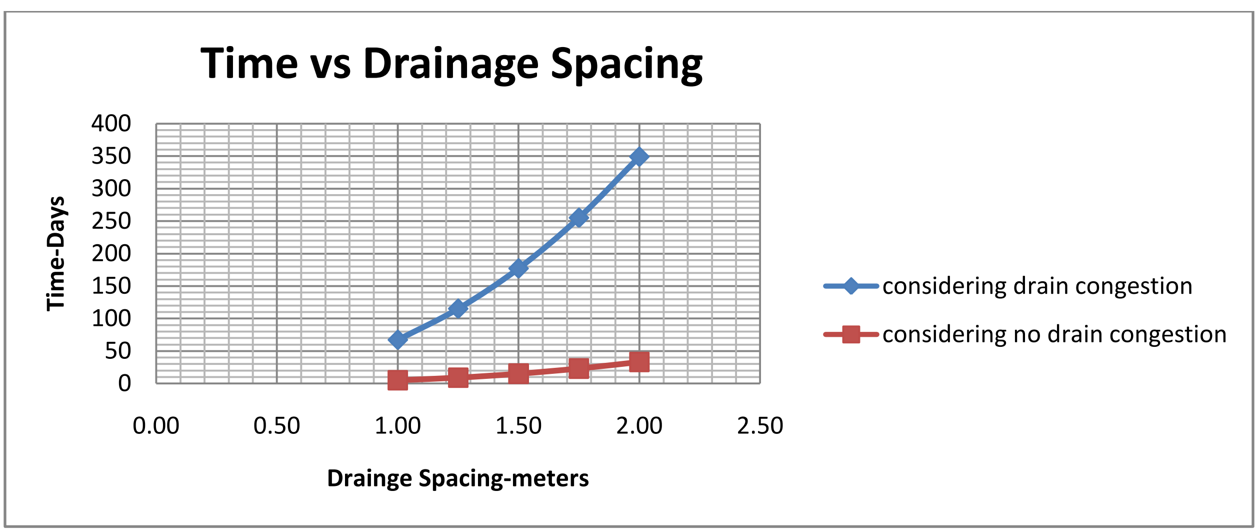
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	8.00	3.20	0.92	0.1829	67
1.25	1.41	7.06	8.00	3.20	0.92	0.3139	115
1.5	1.70	8.48	8.00	3.20	0.92	0.4851	177
1.75	1.98	9.89	8.00	3.20	0.92	0.6983	255
2	2.26	11.30	8.00	3.20	0.92	0.9551	349

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	5.65	0.92	0.0132	5
1.25	1.41	7.06	0.92	0.0253	9
1.50	1.70	8.48	0.92	0.0419	15
1.75	1.98	9.89	0.92	0.0634	23
2.00	2.26	11.30	0.92	0.0900	33



Sand Drain (Square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

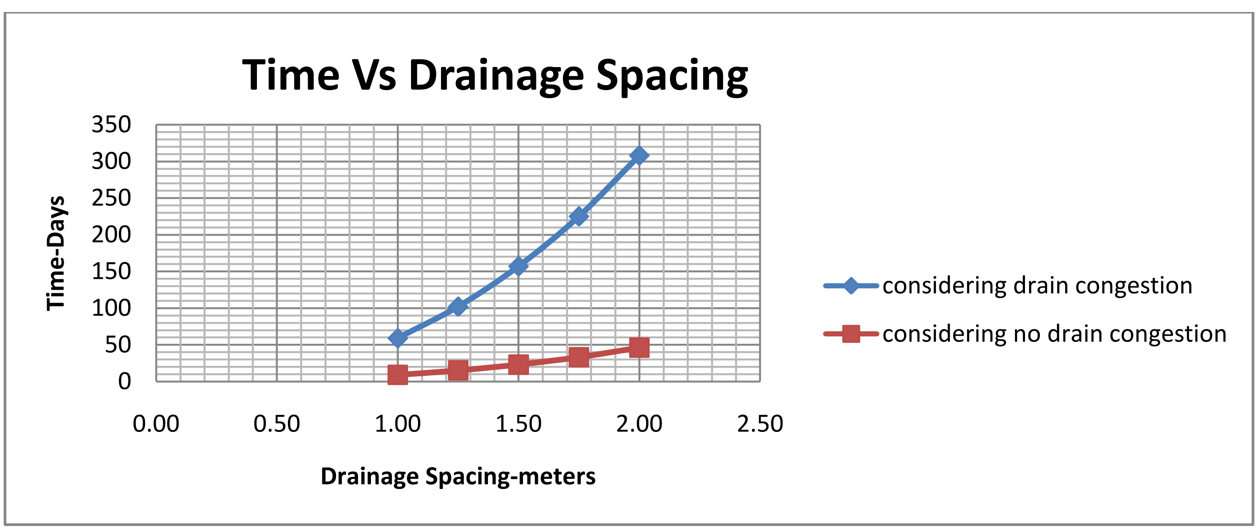
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	4.00	1.60	0.92	0.1626	59
1.25	1.31	19.82	4.00	1.60	0.92	0.2783	102
1.50	1.58	23.79	4.00	1.60	0.92	0.4293	157
1.75	1.84	27.75	4.00	1.60	0.92	0.6172	225
2.00	2.10	31.72	4.00	1.60	0.92	0.8433	308

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.92	0.0234	9
1.25	1.31	19.82	0.92	0.0406	15
1.50	1.58	23.79	0.92	0.0632	23
1.75	1.84	27.75	0.92	0.0914	33
2.00	2.10	31.72	0.92	0.1256	46



PVD (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 8 m

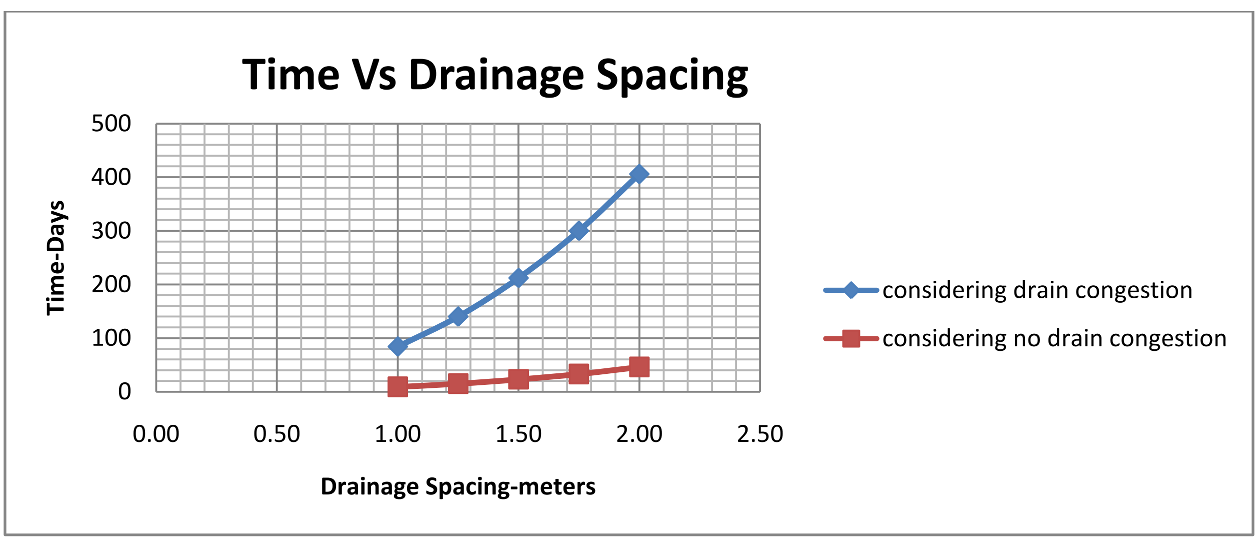
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	8.00	3.20	0.92	0.2298	84
1.25	1.31	19.82	8.00	3.20	0.92	0.3833	140
1.50	1.58	23.79	8.00	3.20	0.92	0.5805	212
1.75	1.84	27.75	8.00	3.20	0.92	0.8230	300
2.00	2.10	31.72	8.00	3.20	0.92	1.1121	406

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.92	0.0234	9
1.25	1.31	19.82	0.92	0.0406	15
1.50	1.58	23.79	0.92	0.0632	23
1.75	1.84	27.75	0.92	0.0914	33
2.00	2.10	31.72	0.92	0.1256	46



PVD (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

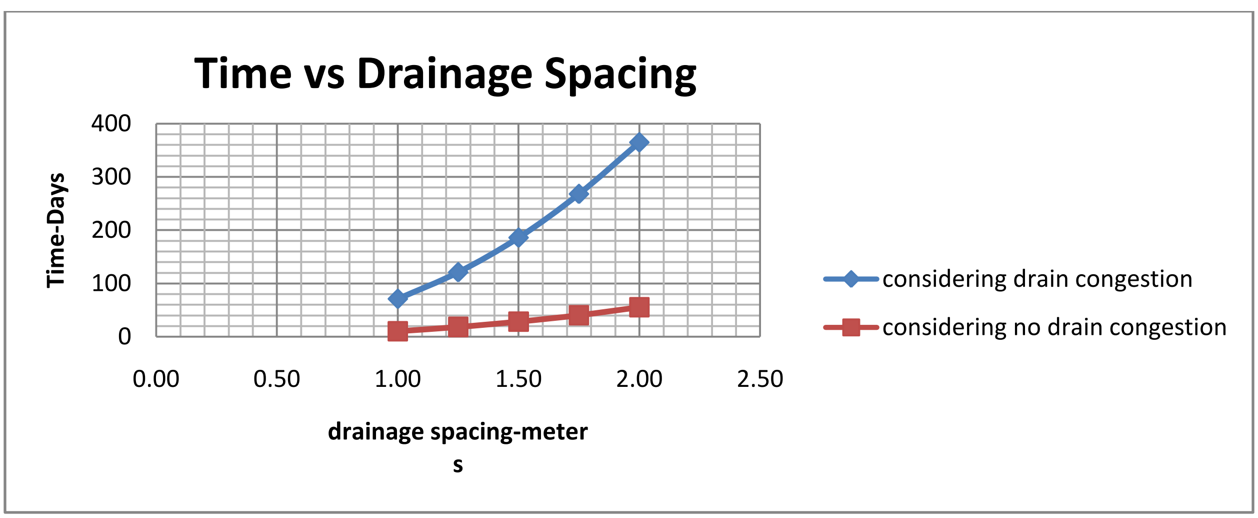
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	4.00	1.60	0.92	0.1942	71
1.25	1.41	21.33	4.00	1.60	0.92	0.3316	121
1.50	1.70	25.60	4.00	1.60	0.92	0.5106	186
1.75	1.98	29.87	4.00	1.60	0.92	0.7330	268
2.00	2.26	34.13	4.00	1.60	0.92	1.0004	365

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.92	0.0280	10
1.25	1.41	21.33	0.92	0.0485	18
1.50	1.70	25.60	0.92	0.0754	28
1.75	1.98	29.87	0.92	0.1089	40
2.00	2.26	34.13	0.92	0.1494	55



PVD (square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 5 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

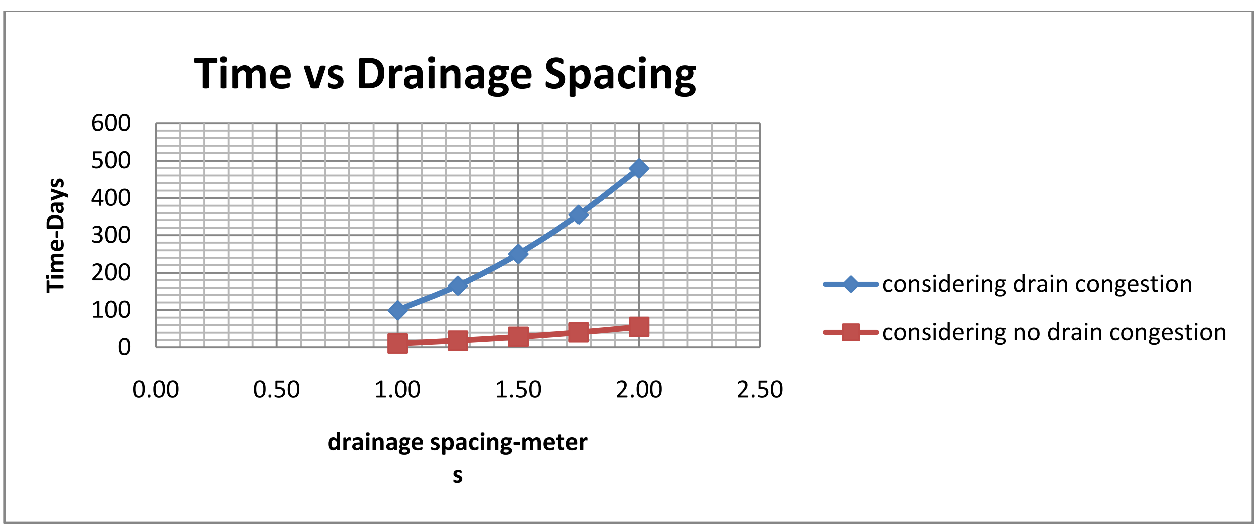
U= 0.92

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	8.00	3.20	0.92	0.2720	99
1.25	1.41	21.33	8.00	3.20	0.92	0.4532	165
1.50	1.70	25.60	8.00	3.20	0.92	0.6856	250
1.75	1.98	29.87	8.00	3.20	0.92	0.9713	355
2.00	2.26	34.13	8.00	3.20	0.92	1.3117	479

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.92	0.0280	10
1.25	1.41	21.33	0.92	0.0485	18
1.50	1.70	25.60	0.92	0.0754	28
1.75	1.98	29.87	0.92	0.1089	40
2.00	2.26	34.13	0.92	0.1494	55



PVD (square)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

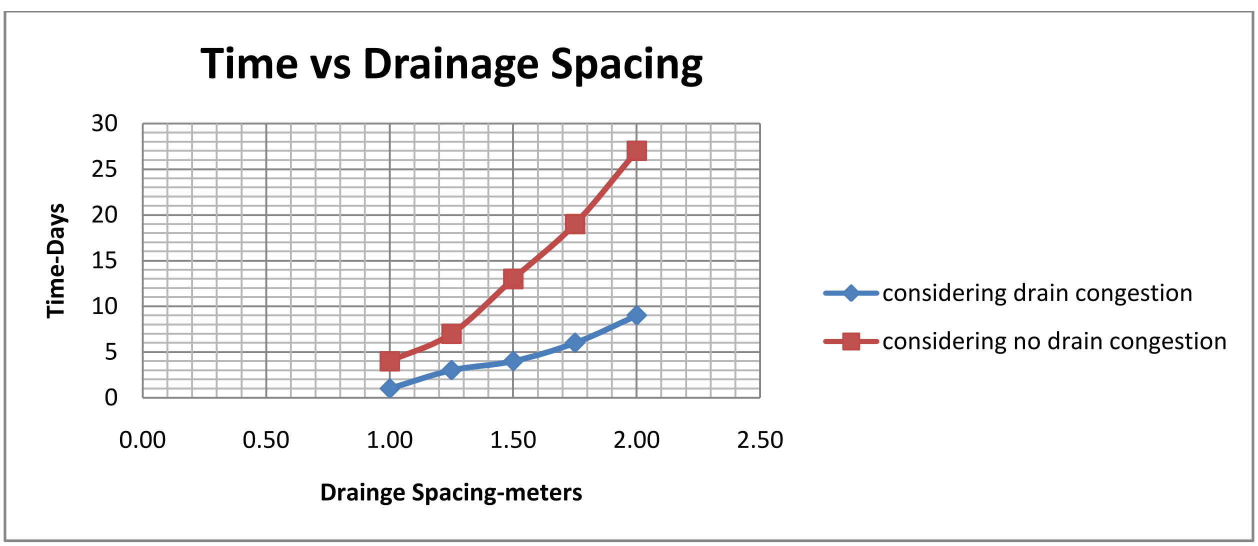
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	4.00	1.60	0.5	0.0039	1
1.25	1.31	6.56	4.00	1.60	0.5	0.0072	3
1.5	1.58	7.88	4.00	1.60	0.5	0.0117	4
1.75	1.84	9.19	4.00	1.60	0.5	0.0175	6
2	2.10	10.50	4.00	1.60	0.5	0.0245	9

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.5	0.0105	4
1.25	1.25	1.31	0.5	0.0205	7
1.5	1.50	1.58	0.5	0.0343	13
1.75	1.75	1.84	0.5	0.0522	19
2	2.00	2.10	0.5	0.0743	27



Sand Drain (Triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 8 m

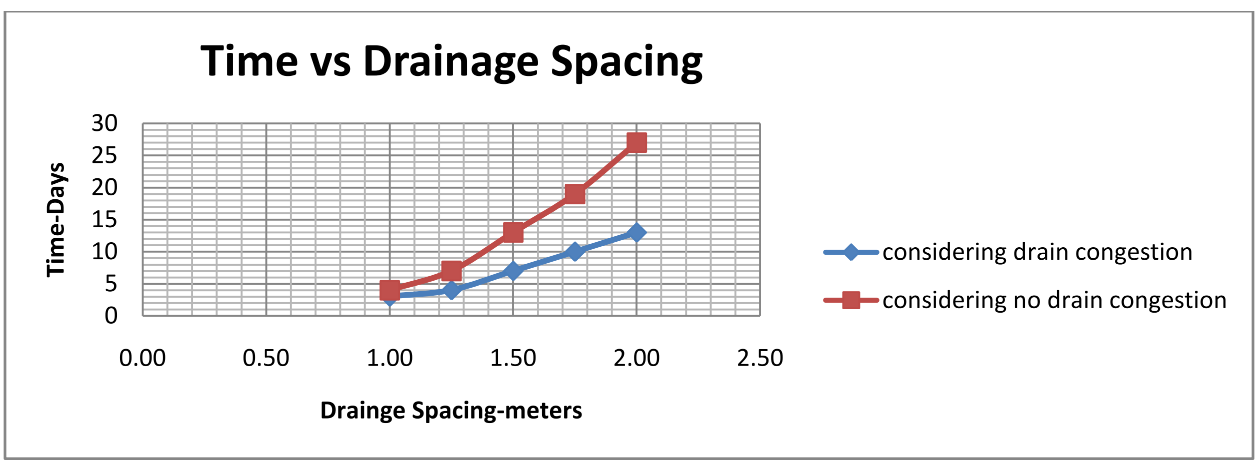
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.05	5.25	8.00	3.20	0.5	0.0070	3
1.25	1.31	6.56	8.00	3.20	0.5	0.0120	4
1.5	1.58	7.88	8.00	3.20	0.5	0.0186	7
1.75	1.84	9.19	8.00	3.20	0.5	0.0269	10
2	2.10	10.50	8.00	3.20	0.5	0.0368	13

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1	1.00	1.05	0.5	0.0105	4
1.25	1.25	1.31	0.5	0.0205	7
1.5	1.50	1.58	0.5	0.0343	13
1.75	1.75	1.84	0.5	0.0522	19
2	2.00	2.10	0.5	0.0743	27



Sand Drain (triangular)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 4 m

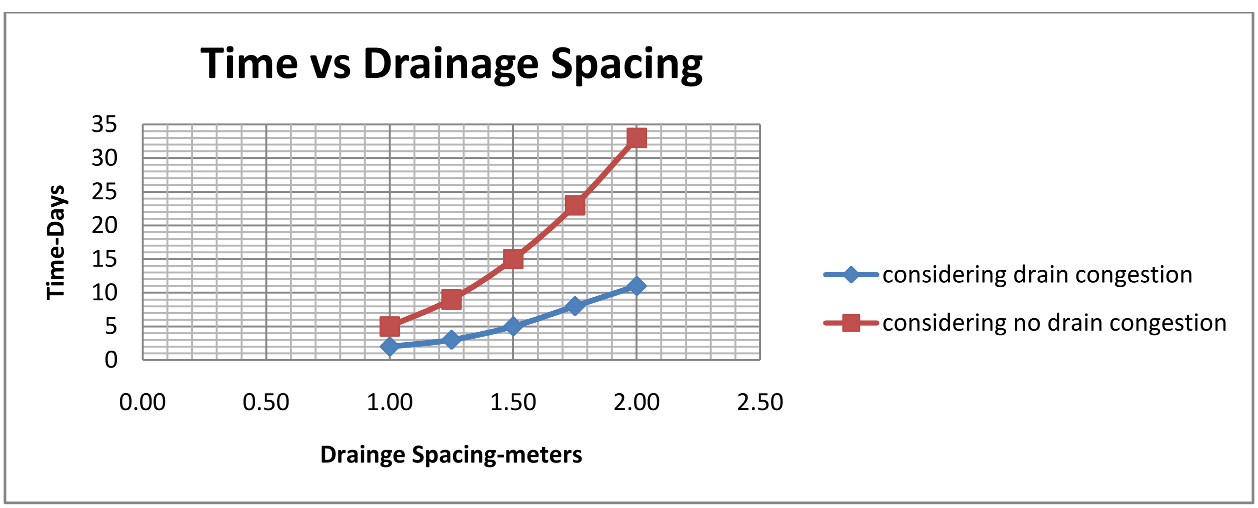
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	4.00	1.60	0.5	0.0048	2
1.25	1.41	7.06	4.00	1.60	0.5	0.0088	3
1.5	1.70	8.48	4.00	1.60	0.5	0.0142	5
1.75	1.98	9.89	4.00	1.60	0.5	0.0210	8
2	2.26	11.30	4.00	1.60	0.5	0.0295	11

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	5.65	0.5	0.0132	5
1.25	1.41	7.06	0.5	0.0253	9
1.50	1.70	8.48	0.5	0.0419	15
1.75	1.98	9.89	0.5	0.0634	23
2.00	2.26	11.30	0.5	0.0900	33



Sand Drain (Square)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^{3}-10^{2}
Medium coarse sand	10^{-3} – 10^{-4}	10^{2}-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 200 mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 8 m

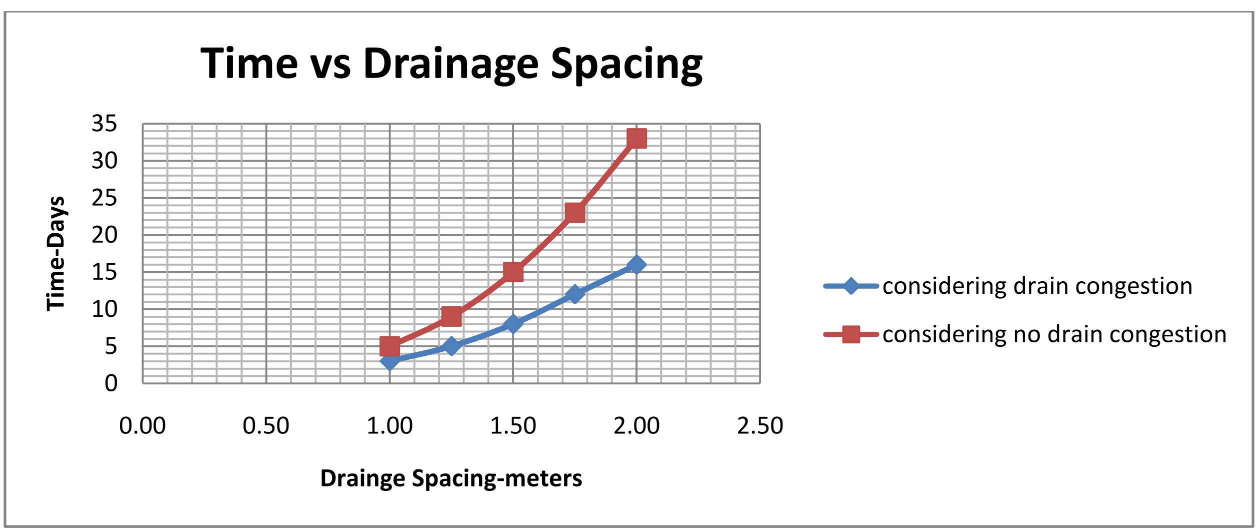
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1	1.13	5.65	8.00	3.20	0.5	0.0084	3
1.25	1.41	7.06	8.00	3.20	0.5	0.0144	5
1.5	1.70	8.48	8.00	3.20	0.5	0.0222	8
1.75	1.98	9.89	8.00	3.20	0.5	0.0319	12
2	2.26	11.30	8.00	3.20	0.5	0.0437	16

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	5.65	0.5	0.0132	5
1.25	1.41	7.06	0.5	0.0253	9
1.50	1.70	8.48	0.5	0.0419	15
1.75	1.98	9.89	0.5	0.0634	23
2.00	2.26	11.30	0.5	0.0900	33



Sand Drain (square)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w= 0.01 m^{-2}

I= 4 m

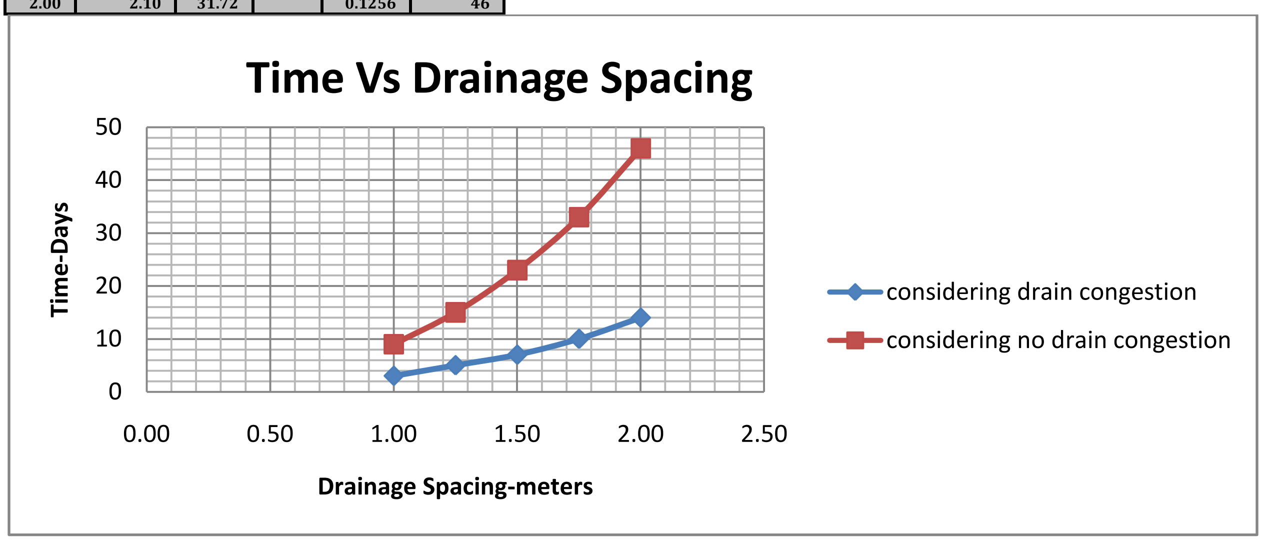
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	4.00	1.60	0.5	0.0074	3
1.25	1.31	19.82	4.00	1.60	0.5	0.0127	5
1.50	1.58	23.79	4.00	1.60	0.5	0.0196	7
1.75	1.84	27.75	4.00	1.60	0.5	0.0282	10
2.00	2.10	31.72	4.00	1.60	0.5	0.0386	14

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.5	0.0234	9
1.25	1.31	19.82	0.5	0.0406	15
1.50	1.58	23.79	0.5	0.0632	23
1.75	1.84	27.75	0.5	0.0914	33
2.00	2.10	31.72	0.5	0.1256	46



PVD (triangular)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 8 m

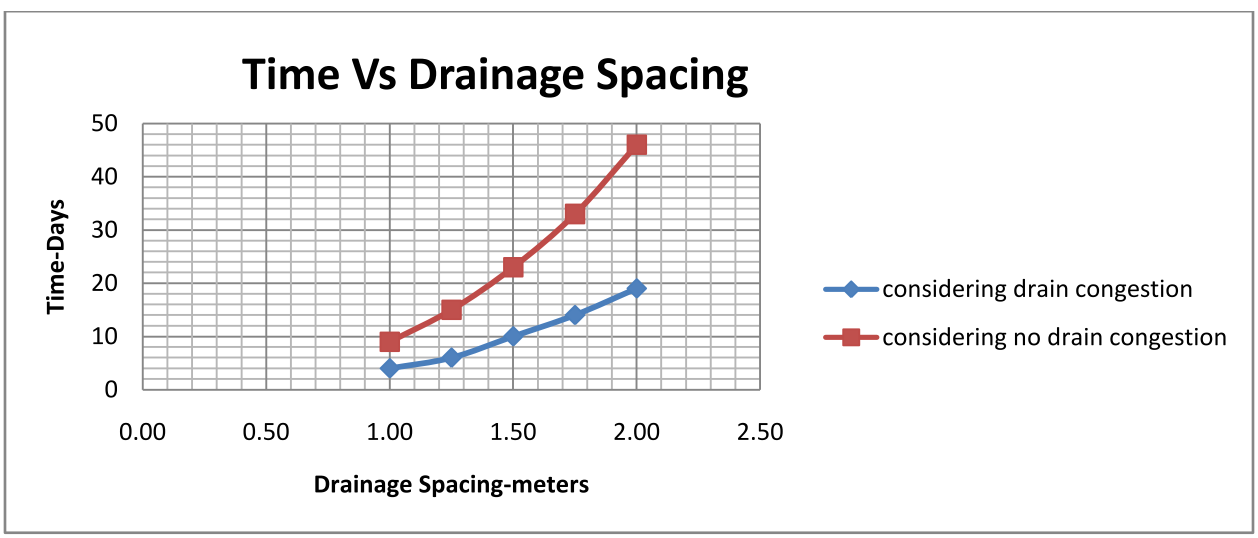
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.05S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.05	15.86	8.00	3.20	0.5	0.0105	4
1.25	1.31	19.82	8.00	3.20	0.5	0.0175	6
1.50	1.58	23.79	8.00	3.20	0.5	0.0266	10
1.75	1.84	27.75	8.00	3.20	0.5	0.0376	14
2.00	2.10	31.72	8.00	3.20	0.5	0.0509	19

Drainage time considering no drain congestion

S(m)	D=1.05S	D/d	U	t(yr)	t(days)
1.00	1.05	15.86	0.5	0.0234	9
1.25	1.31	19.82	0.5	0.0406	15
1.50	1.58	23.79	0.5	0.0632	23
1.75	1.84	27.75	0.5	0.0914	33
2.00	2.10	31.72	0.5	0.1256	46



PVD (triangular)

$t=(D^2/8C_h)*[\ln(D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln(D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 4 m

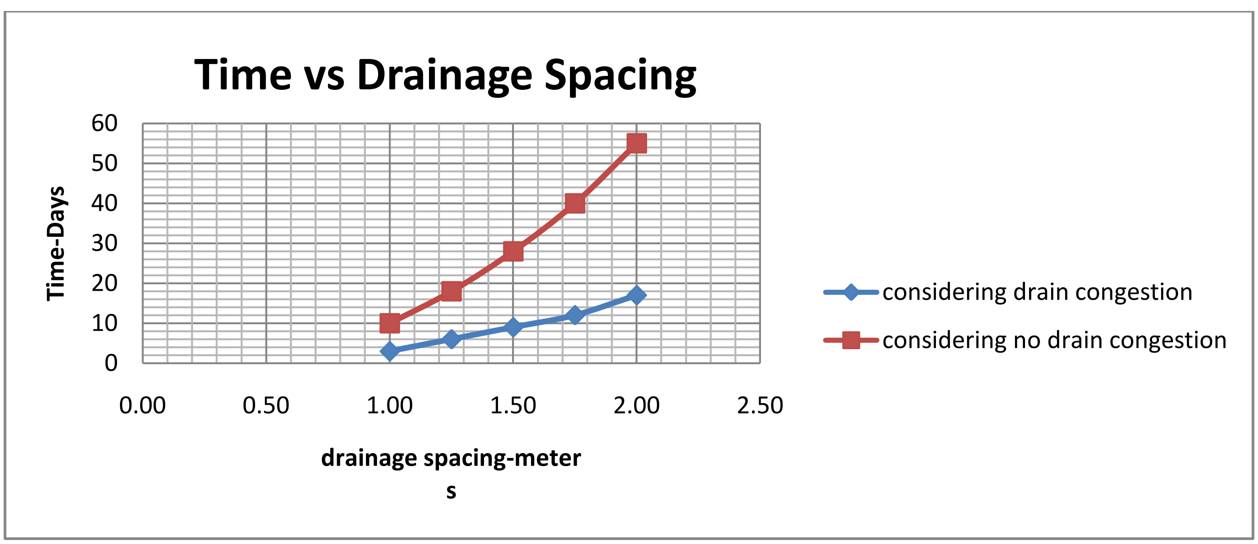
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	4.00	1.60	0.5	0.0089	3
1.25	1.41	21.33	4.00	1.60	0.5	0.0152	6
1.50	1.70	25.60	4.00	1.60	0.5	0.0234	9
1.75	1.98	29.87	4.00	1.60	0.5	0.0335	12
2.00	2.26	34.13	4.00	1.60	0.5	0.0458	17

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.5	0.0280	10
1.25	1.41	21.33	0.5	0.0485	18
1.50	1.70	25.60	0.5	0.0754	28
1.75	1.98	29.87	0.5	0.1089	40
2.00	2.26	34.13	0.5	0.1494	55



PVD (square)

$t=(D^2/8C_h)*[\ln (D/d)-0.75]*\ln(1/1-U)$; Considering no drain congestion

$t=(D^2/8C_h)*[\ln (D/d)-0.75+\pi*z*(2I-z)*(k_c/q_w)]*\ln(1/1-U)$; Considering drain congestion

where, t= consolidation period; D= diameter of drained soil cylinder (m) ; D= 1.05S (triangular pattern) , D=1.13S (square pattern) ; S= spacing of the drain; C_h= horizontal consolidation co-efficient (m^2/year) ; U= average horizontal consolidation degree; d= equivalent diameter of drain=2(a+b)/π ; a= drain width, b= drain thickness; z= distance to the flow point (m); I= drain length at unilateral flow (m) (half length for bilateral flow); k_c=permeability of the soil (m/s); q_w=discharge capacity of the drain

Soil	k_c (m/s)	k_c/q_w (m^{-2})
Coarse sand	10^{-2} - 10^{-3}	10^3-10^2
Medium coarse sand	10^{-3} – 10^{-4}	10^2-10
Fine sand	10^{-4} – 10^{-5}	10^{-1}
Silty sand	10^{-5} – 10^{-6}	1-10^{-1}
Sandy silt	10^{-6} – 10^{-7}	10^{-1}-10^{-4}
Peat	10^{-7} – 10^{-9}	10^{-2}-10^{-4}
Clay	10^{-9} – 10^{-11}	10^{-4}-10^{-6}

d= 66 mm ; a= 100 mm ; b= 4mm

C_h= 30 m^2/year

k_c/q_w = 0.01 m^{-2}

I= 8 m

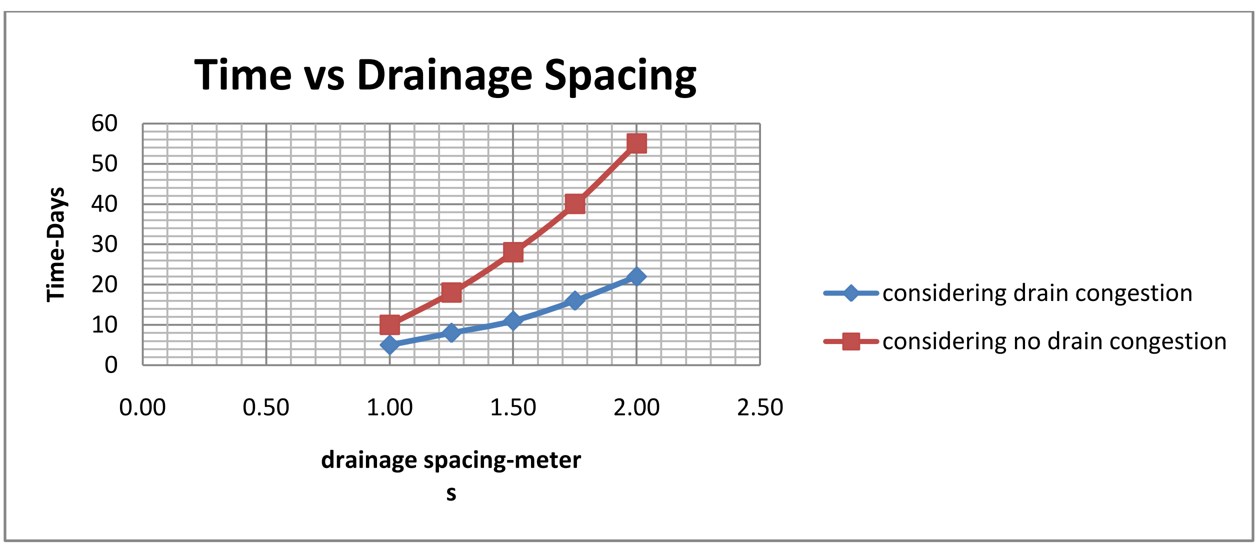
U= 0.5

Drainage time considering drain congestion

S (m)	D=1.13S	D/d	I (m)	z (m)	U	t(yr)	t(days)
1.00	1.13	17.07	8.00	3.20	0.5	0.0124	5
1.25	1.41	21.33	8.00	3.20	0.5	0.0207	8
1.50	1.70	25.60	8.00	3.20	0.5	0.0314	11
1.75	1.98	29.87	8.00	3.20	0.5	0.0444	16
2.00	2.26	34.13	8.00	3.20	0.5	0.0600	22

Drainage time considering no drain congestion

S(m)	D=1.13S	D/d	U	t(yr)	t(days)
1.00	1.13	17.07	0.5	0.0280	10
1.25	1.41	21.33	0.5	0.0485	18
1.50	1.70	25.60	0.5	0.0754	28
1.75	1.98	29.87	0.5	0.1089	40
2.00	2.26	34.13	0.5	0.1494	55



PVD (square)