GEOTECHNICAL ASPECTS OF POLDER EMBANKMENT IN SELECTED COASTAL AREA OF BANGLADESH

by

Md. Zakir Hossain

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The thesis titled **"GEOTECHNICAL ASPECTS OF POLDER EMBANKMENT IN SELECTED COASTAL AREA OF BANGLADESH"** Submitted by **Md. Zakir Hossain,** Roll No. **100604212(P)**, Session October' **2006**, has been accepted as satisfactory in partial fulfillment of the requirement for the degree of **M. Engg. (Civil and Geotechnical)** on **September 18, 2013**.

BOARD OF EXAMINERS

____

Dr. Md. Jahangir Alam — Chairman
Associate Professor — (Supervisor)
Department of Civil Engineering
BUET, Dhaka - 1000.

____

Dr. Abu Siddique — Member
Professor
Department of Civil Engineering
BUET, Dhaka - 1000.

____

Dr. Eqramul Hoque — Member
Professor
Department of Civil Engineering,
BUET, Dhaka-1000

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

(Md. Zakir Hossain)

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S = Degree of saturation;

C_c = Compression index;

I_p = Plasticity index;

q_u = Unconfined compressive strength

SPT = Standard penetration test;

w_L = Liquid limit;

w_n = Natural Water Content;

w_p = Plastic limit;

γ_d = Dry unit weight;

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ABSTRACT

Polder embankments are vulnerable to breaching during storm surges. To investigate the causes of failure of embankments, polder no. 14/1 was selected for this study. This polder is in Koyra Upazilla under Khulna district. This polder is near Sundarbans. Two SPT bore holes were done near Patakhali closure, one at the toe of existing embankment and another adjacent to the closure. Due to very soft and non-plastic soil, undisturbed soil samples could not be collected from any borehole. Disturbed samples were collected from two boreholes at 1.5 m interval along with field SPT-N value. Bulk soil samples were collected from adjacent to closure at 1.5 m depth by manually excavating the top soil. Two in-situ field density test using sand cone method were performed; one on the top of the existing embankment and another at the toe of the existing embankment. Necessary index tests were done to classify the soils. Double hydrometer ratio tests were performed to determine the dispersiveness of the soil. Standard Proctor Compaction test were performed on bulk sample.

The sub-surface soil consists mainly of lean clay in the upper layers up to the depth of eleven meters whereas the layers below this consist of non-plastic silt with sand. Since the soil is non-plastic i.e. the soil possesses no cohesion, it is vulnerable to erosion and so if any failure takes place in the embankment due to any cause it propagates quickly and cannot be stopped and becomes a catastrophe within days. Double hydrometer ratio test results on lean clay show that the soil samples collected from the surface and the boreholes are medium dispersive. When water flows through it or in contact with it the soil particles are easily washed away with water. So, it might be the cause of failure of the embankment. In situ density of soil is very low. Usually no compaction is done during construction of polder embankment. Because it is very expensive to carry compaction machine to the site as it is very remote place in Bangladesh. So the embankment remains at high void ratio which is susceptible to piping and erosion.

CHAPTER 1: INTRODUCTION

1.1 GENERAL

A polder is a low lying tract of land enclosed by embankments (barriers) known as dykes, that forms an artificial hydrological entity, meaning it has no connection with outside water other than through manually operated devices. Polders are most commonly found, though not exclusively so, in river deltas, former fen lands, and coastal areas. Dykes are mostly built using locally available materials and each material has its own risk factor: sand is prone to erosion by water while dry peat is lighter than water, making the barrier potentially unstable in very dry seasons.

Earth embankments in Bangladesh are beset with multi-facetted problems. The design and construction methods used to build the embankments, the nature and extent of erosive forces to destabilize them and above all the attitude of the local people for whom they are built altogether determine the magnitude and degree of instability. Most earthen embankments face light to moderate erosion problems arising out of rainfall splash, animal actions and the nature of human uses. Some of the critically positioned submerged types of embankment in *haor* areas of the eastern part and river embankments of the main land are subjected to turbulent water currents and changes in river courses. The problem is acute in offshore islands and coastal belts where the embankments are in addition exposed to erosion by sea waves and tidal fluctuation of water levels.

In the coastal belt and offshore islands severe bank erosion problems occur frequently. New accretion and shifting of the bank line due to erosion happen almost in the same way as those observed in the inland rivers. However, the nature and extent of erosive forces damaging the seashore and successively the dykes differ in certain aspects from those of the inland riverbanks.

The process of erosion gradually destroys the shore lands/riverbanks, foreshore areas/berms and successively the earthen embankments engulfing the plain agricultural lands, habitats and many important installations. In the affected reaches, the embankments built with adequate setback are found to disappear within a year or two of the start of erosion. Generally new embankments are constructed along separate alignments (further back in the countryside), simultaneously adopting sufficient measures to check the bank erosion. Whenever the situation compels to protect the high-value assets and important installations as the losses are really irreparable or would have a great impact on the national economy, sophisticated structural means of proven engineering solutions involving huge costs are often adopted to combat the erosive forces.

1.2 BACKGROUND OF THE STUDY

The southern part of Bangladesh mostly the coastal districts are divided into many parts of land like separated islands by the tributaries of the main rivers of Bangladesh. Existing ground surface is slightly above the mean water level of surrounding rivers. So they are frequently washed away by the daily tidal surges. To protect those lands and their inhabitants from these regular tidal surges and storm surges due to cyclone, embankments were built surrounding the island which is known as Polder.

There are 123 polders in the coastal districts of Bangladesh. Some are large in area whereas some are small. Of them ten polders are in Satkhira, 26 in Khulna, 5 in Bagerhat, 23 in Patuakhali, 19 in Borguna, 19 in Cox's Bazar and rest of the polders are situated in other coastal districts. These polders were constructed during late sixties. Since then they had been working well up to recent years. But they were severely damaged recently by the cyclone Aila in April 2009. After Aila, during tidal flow the saline water enters into the polder and submerged everything. This affects those polder areas through permanent inundation, drainage congestion, storm surge inundation and increased salinity intrusion of low-lying areas. As a result shrimp farm, agricultural land, wetlands, infrastructure and all other income generating works become severely damaged. The green crop land disappears and it becomes a desert of mud (Figure 1.1). So the suffering of the islanders knows no bound. Bangladesh Water Development Board (BWDB) tried their best to rebuild the damaged part of embankment. But it was not always possible to rebuild the dyke and keep the closure in position.

Increasing rates of sea level rise caused by global warming are expected to lead to permanent inundation, drainage congestion, salinity intrusion and frequent storm surge inundation. Sea level rise is a growing threat for the coastal regions of Bangladesh. Bangladesh is one of the mostly densely populated countries of the world where 28% of the total population live in the coastal area. The trend of tropical cyclones hitting the Bangladesh coast is not steady. It has fluctuated in the past century. Presently, there is an increasing trend. Higher population density increases vulnerability to climate change especially water related disaster in Bangladesh [1]. Most of the casualties from cyclones in Bangladesh, as in other parts of the world, are caused by storm surges as for climate change will increase the height of storm surges, leading to greater coastal flooding [2].

Failures of earthen embankment dams or dikes can generally be grouped into three classifications: hydraulic, seepage, and structural. Hydraulic failures from the uncontrolled flow of water over and adjacent to the embankment are due to the erosive action of water on the embankment slopes. Seepage can cause slope failures by saturating the slope material, thereby weakening the cohesive properties of the soil and its stability. Whereas structural failures involve the separation (rupture) of the embankment material and/or its foundation. Structural failures are sometimes initiated by shrimp farmers by making holes (dyke piercing) under embankment to allow water entering into their shrimp farms.

So, it is necessary to identify the causes of embankment failure of polders and find out the remedial measures for repairing and construction of embankment.

1.3 OBJECTIVES OF THE STUDY

The objectives of the study are as follows:

i. To identify the causes of failure of embankments of Polders.
ii. To suggest a method to repair the embankments.

1.4 METHODOLOGY

Polder no. 14/1 which consists of two union councils named South Betkashi and North Betkashi (Figure 1.2) was selected for this study. These two union councils are under the jurisdiction of Koyra Upazilla under Khulna district. This polder is near Sundarban. Patakhali , a village in South Betkashi, was severely damaged along with its protection embankment by the cyclone Aila. Part of the embankment surrounding this village was so badly damaged that saline water, during tidal hour, intruded into the island. The consequence was the disappearance of all type of green due to salinity and it brought limitless sufferings to the islanders. BWDB attempted twice to rebuild the affected part of the dyke but they failed. It could be due to the insufficiency of knowledge on soil property of the area. A reconnaissance was done in the locality of Polder area and embankment to understand the situation. Information was collected from the local people about the lifestyle and economic condition of villagers.

Two SPT bore holes were done near Patakhali closure, one at the toe of existing embankment and another adjacent to the closure. Due to very soft and non-plastic soil, undisturbed soil samples could not be collected from any borehole. Disturbed samples were collected from two boreholes at 1.5 m interval along with field SPT-N value. Bulk soil samples were collected from adjacent to closure at 1.5 m depth by manually excavating the top soil. Two in-situ field density test using sand cone method were performed; one on the top of the existing embankment and another at the toe of the existing embankment. Necessary index tests were done to classify the soils. Double hydrometer ratio tests were performed to determine the dispersiveness of the soil. Standard Proctor Compaction test were performed on bulk sample.

1.5 ORGANIZATION OF THE THESIS

The thesis is arranged into five chapters and one appendix. In Chapter One, background and objectives of the research is described. Chapter Two contains the literature review where history, works and researches on embankment failure are described.

Chapter Three describes the testing program. Chapter Four contains results and discussion. Chapter Five contains the conclusions and recommendations for further research. All graphs of testing results are presented in Appendix A.

(a)

(b)

Figure 1.1: Situation of villages after 2 years of devastation caused by Aila in 2009; (a) country side (b) river side.

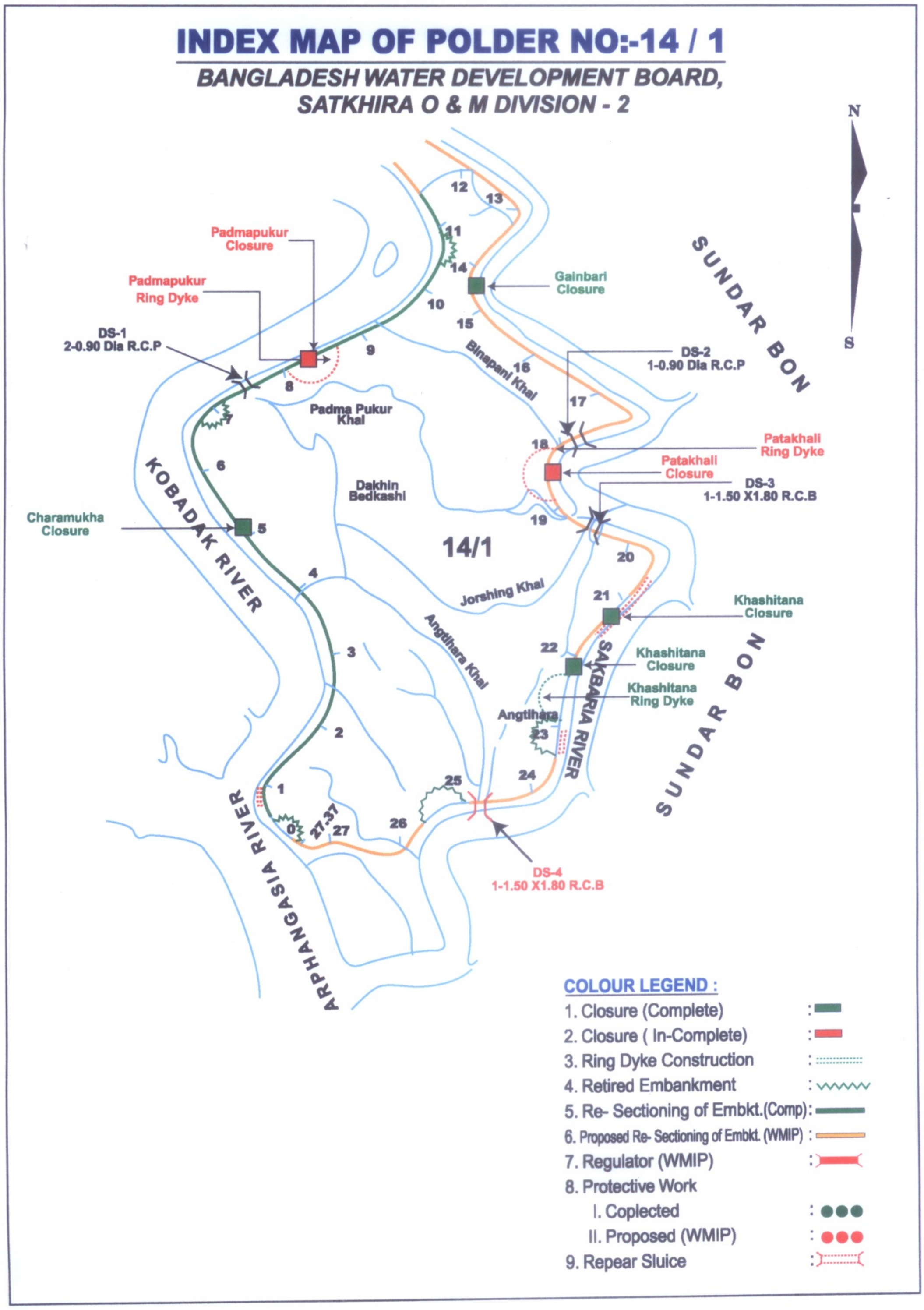

Figure 1.2: Index map of Polder no. 14/1, Patakhali, Dakkin Betkashi, Koira, Khulna

CHAPTER 2: LITERATURE REVIEW

2.1 INTRODUCTION

The literature review is consisting of (a) geotechnical failure mechanisms (b) causes of erosion of polder embankments and their effects (c) measures taken for erosion control and (d) different models for embankment erosion control. Agarwala et al [1] discussed the climate change in Bangladesh and focus on the causes of coastal flooding and Sundarbans. Ali [2] worked on vulnerability of Bangladesh to climate change and finds the sea level rise through tropical cyclones and storm surges. Hossain and Sakai [4] learn the severity of flood embankments in Bangladesh through their works and suggest its remedial approach. Islam et al [5] studied in-situ shear strength of Vetiver grass rooted soil to protect embankment from erosion. Several studies have been done on river embankment and bank failure in various locations of Bangladesh by Hossain et al [5].

Sustainable embankment construction and repair is necessary to protect the inhabitants of polders in Bangladesh. Until now few researches have been done in Bangladesh addressing this problem. Hossain and Sakai [4] identified that as most of the embankments are made of earth or soil only without any surface covering, these are washed away easily with the raindrop and current or flow of water. Therefore they suggested that following remedial measures should be taken for sustainable embankment in Bangladesh: (i) construction of concrete embankment instead of earthen one, (ii) Protection to the surface of earthen embankment to prevent washing out of soil, (iii) Use of locally available recycled and waste materials for earth reinforcement and (iv) application of soil cement composite with mesh as a reinforcement. Islam et al [5] studied the effectiveness of Vetiver grass rooted soil for sustainable embankment. They concluded that Vetiver is effective to protect the coastal embankment against erosion, runoff, wave action, flood and cyclonic tidal surge etc.

2.2 GEOTECHNICAL FAILURE MECHANISMS

Examples of failure mechanisms are illustrated in figure 2.1 [11].

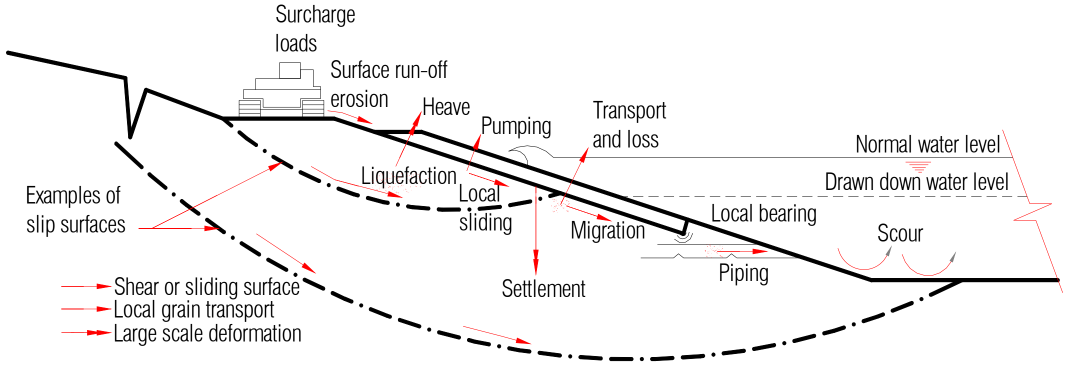

Figure 2.1: Subsoil failure mechanism [11].

Bank failure stability can be considered in terms of

(a) Overall stability – the stability of the bank as a whole

(b) Local stability – the stability of elements of the revetment or bank structure in resisting the influence of local loads, hydraulic or otherwise : local stability problems may ultimately lead to failure of the bank as a whole.

2.2.1 Overall Stability Failure Mechanisms

Slip circles and deep sliding – this is a conventional soil mechanics stability problem: the influence of rapid drawn-down should be considered. Pre-existing slip planes within the soil, or lenses and bands of weaker material can have a significant effect on bank stability. Site investigations should be planned to enable such situations to be identified. Figure 2.2 classifies various failure mechanisms.

2.2.2 Local Stability Failure Mechanisms

Failure of embankment and slope may be initiated by various local stability problems. These are described below:

Local sliding: local shear failure parallel to the slope either within the soil mass or at the soil-revetment interface.

Local bearing failure: local shear failure in the soil in planes not parallel to the slope.

Scour: removal of soil particles from the soil-water interface by current or wave induced shear forces within the water way; possibly in combination with hydraulic gradient forces or by rainfall runoff above the water line.

Piping: a form of concentrated seepage through a section of soil which is either more permeable than its surroundings or is subject to a particularly high hydraulic gradient. Concentration of flow may lead to transport of soil particles and loss by regressive erosion. Once transport is established, the hydraulic gradient will increase and transport more particles resulting in an erosion pipe.

Migration: transport of material within the soil mass beneath the revetment structure. Migration may be parallel to the bank causing slumping and S-profile or through the revetment in which case material is lost from the bank. A special case is suffusion, which is the transport of the finest particles from between a soil skeleton formed by coarser material.

Liquefaction: complete loss of grain to grain contact by an increase in pore water pressure or by shock loading of a loosely compacted granular soil. Consequent loss of effective stress effects in a zero shear strength and the soil behaving as a liquid.

Pumping or sand boiling: extrusion of particles under shock loading or impact in connection with liquefaction at an interface.

Settlement: deformation due to reduction in volume of soil. Settlement may be caused by

- Consolidation – squeezing out of pore water from a cohesive soil
- Compression – more dense packing of particles due to extra loading
- Migration – loss of material from within the bank
- Shrinkage – drying out of saturated cohesive soil
- Loss of apparent cohesion – some sandy soils exhibit apparent cohesion when dry. When saturated this cohesion is lost resulting in settlement.

Heaving: deformation due to increase in volume of soil. Heave may be caused by:

- Frost heave – expansion of ice crystals within the soil mass on freezing
- Swelling – certain soils take up moisture and swell after removal of confining pressure.

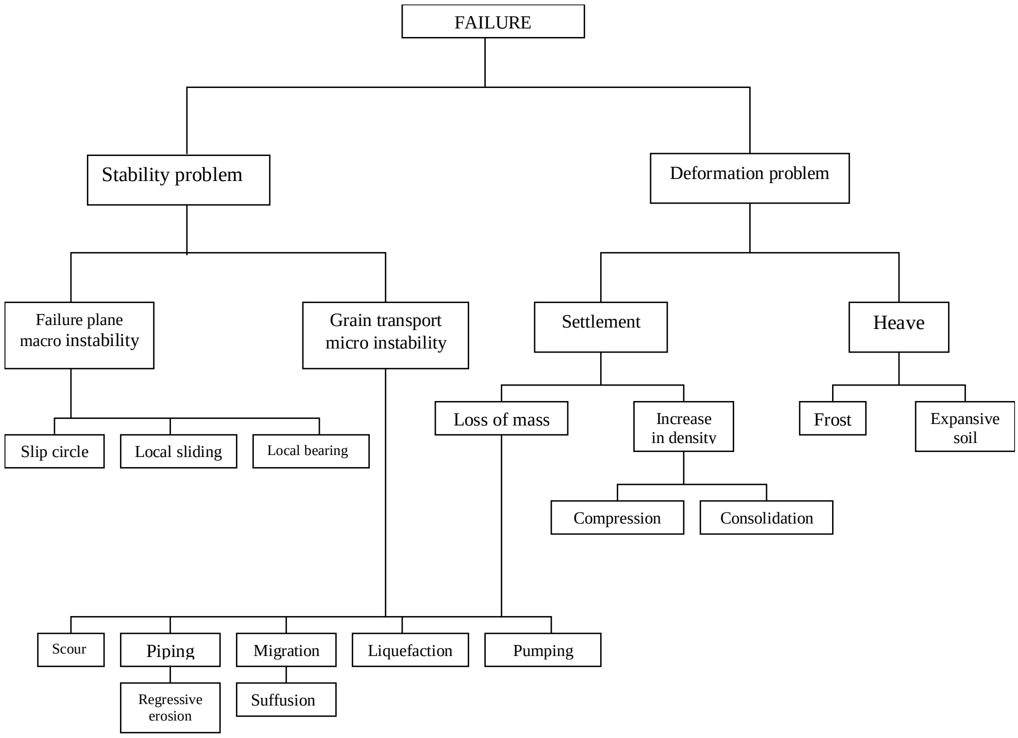

Figure 2.2: Classification of various failure mechanisms [11].

2.3 IDENTIFICATION OF SOILS SUSCEPTIBLE TO DOWNSLOPE MIGRATION

The problem of down-slope migration is most apparent in silts, sandy silts and fine sands. The minor cohesive properties of these types of soils tend to encourage high mobility of small single grains at relatively low hydraulic gradients [11].

The distinction between soils susceptible to down-slope migration and those not susceptible is related to grain size, permeability and degree of cohesion. The following procedure gives a general method of identifying soils susceptible to down-slope migration. There may however be exceptions to the conditions given here and the designer must use his own judgment in such cases.

Soils susceptible to down-slope migration will satisfy the condition that a proportion of particles must be smaller than 0.06 mm. Additionally the soil will satisfy at least one of the following criteria:

1. The coefficient of uniformity,

$$C_u = \frac{d_{60}}{d_{10}} < 15$$

The limit of 15 is a conservative estimate based on practical experience. For soils with a coefficient of uniformity greater than 15 a secondary filter will normally become established.

2. 50 % or more of the particles will lie in the range 0.02 mm < d < 0.10 mm.

3. The plasticity index

$$I_p < 15$$

If I_p is unknown at the preliminary design stage then the following criterion may be substituted,

$$\frac{proportion\ of\ clay\,(d < 0.002mm)}{proportion\ of\ silt\,(0.002 < d < 0.06\,mm)} < 0.5$$

2.4 METHODS OF PREVENTING DOWNSLOPE MIGRATION

There are three ways of preventing down-slope migration.

(a) Attenuation of the Hydraulic Gradient

This can be achieved by incorporating a granular sub-layer between the geo-textile and the cover layer. In practice the granular material should be fine enough to provide an adequate damping effect yet coarse enough to be retained by the cover layer. The sub-layer should have a thickness of between 100 mm and 500 mm for the conditions given at the beginning of this section 300 mm has been found to be adequate.

(b) Stabilization of the Soil Surface

Experience has shown that a thick layer of coarse fibers attached to the back of the geo-textile filter integrates with the soil surface, thus reducing down-slope migration of soil particles within this layer.

The physical requirements of this layer are dependent upon the range of the soil grading are given in Table 2.1.

Table 2.1: Requirements of the coarse layer.

Characteristic of Coarse Layer	Range A Grading Curve	Range B Grading Curve
0_{90}	$0.3 < 0_{90} < 1.5$mm	$0.5 < 0_{90} < 2.0$ mm
Thickness	$5 < t_{gg} < 15$ mm	$5 < t_{gg} < 20$ mm

As well as these requirements experience has shown that the filter fabric should have a minimum thickness of about 5 mm.

(c) Application of a Constraining Load

This can be achieved by using a heavy weight cover layer which imparts a significant load into the bank slope thereby resisting uplift pressures caused by excess pore water pressures. However at present, research is still being carried out in this area and as a practical means of preventing downslope migration either alternative (a) or (b) is suggested.

Figure 2.3 illustrates alternatives (a) and (b) together with the case where the soil is not susceptible to downslope migration.

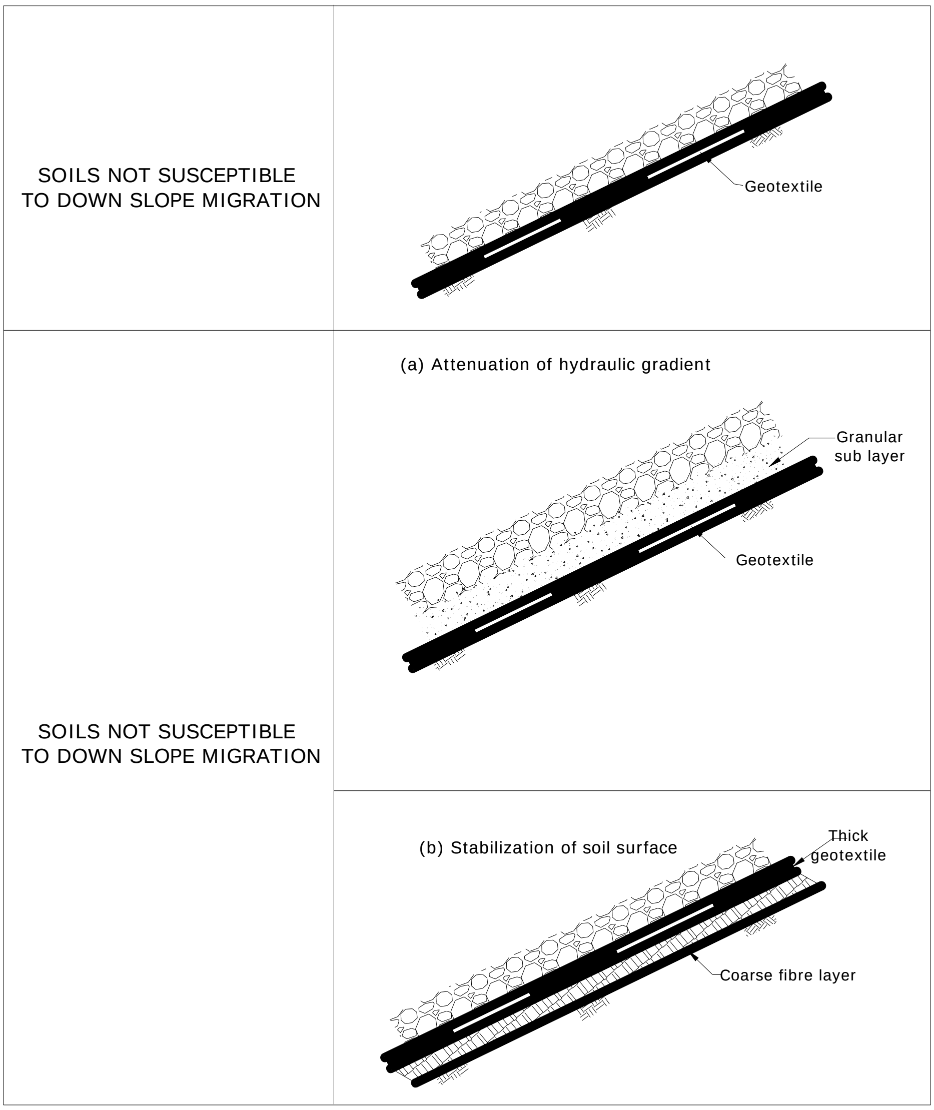

Figure 2.3: Recommended filter system [11].

2.5 GEOTECHNICAL INVESTIGATION OF MEGHNA-DHONAGODA IRRIGATION PROJECT (MDIP) EMBANKMENT

(a) Introduction

Meghna Dhonagoda Irrigation Project (MDIP) in Matlab Upazilla of Chandpur district was completed in 1987. This project is encircled by a 60.7 km of flood embankment around the side of the river Meghna and Dhonagoda. After completion of the embankment, severe seepage and boiling were observed along the countryside of the embankment during floods. The embankment was breached during 1987 and

1988 floods causing severe damages. During the floods of 2004, a breach at Nandalalpur occurred but no damage could be caused as the BWDB authorities repaired the breach quickly with the help of local people. Furthermore, sliding of the slope of embankments occurred on the countryside at about 60 places, seepage problems developed at about 25 km length and boiling occurred at about 120 locations. The damages were severe and it remained as a threat to future performance of the embankment during floods.

(b) Observed Distresses in the Embankment

Four major types of failure/damage in the embankment due to 2004 flood were observed:

- Blowout of the upper soil on the countryside close to the embankment due to uplift pressure transferred through the sand layer.

- Initiation of boiling through a sandy channel with progressive removal of sand creating a tunnel that finally enlarged and sometimes creating breach in the embankment.

- Erosion of soil from toe of the embankment due to seepage creating instability at the toe and resulting in slides on the country side, and

- Rain cuts and washout or removal of sandy soil within the embankment damaging the embankment sections.

(c) Subsoil Exploration

A detailed soil investigation was carried out on the sub-soil at the embankment site. The soil investigation was conducted with the following principal objectives:

- Identification and classification of the foundation soil.

- Determination of stratigraphic sequences of sub-soil strata by drilling boreholes to adequate depths.

- Evaluation of consistency and relative density of the sub-soil by carrying out specific in-situ test, and
- Assessment of strength, compressibility, expansibility and permeability properties of the sub-soil from laboratory testing.

The field investigation at the embankment site consisted of drilling of boreholes, recording density/stiffness characteristics of soil layers by carrying out Standard Penetration Tests (SPT), collection of sufficient numbers of disturbed and undisturbed tube samples. Also portable seismograph was used to support the result obtained from SPT-N values. A total of fifty six boreholes at eleven sites were drilled vertically at this site using wash boring technique.

The depth of boreholes below the ground surface varied from 32 ft to 52 ft. In general five boreholes (BH-1, BH-2, BH-3, BH-4 and BH-5) were drilled at each site across an embankment section. The chainage-wise borehole sites and locations are presented in Figure 2.4.

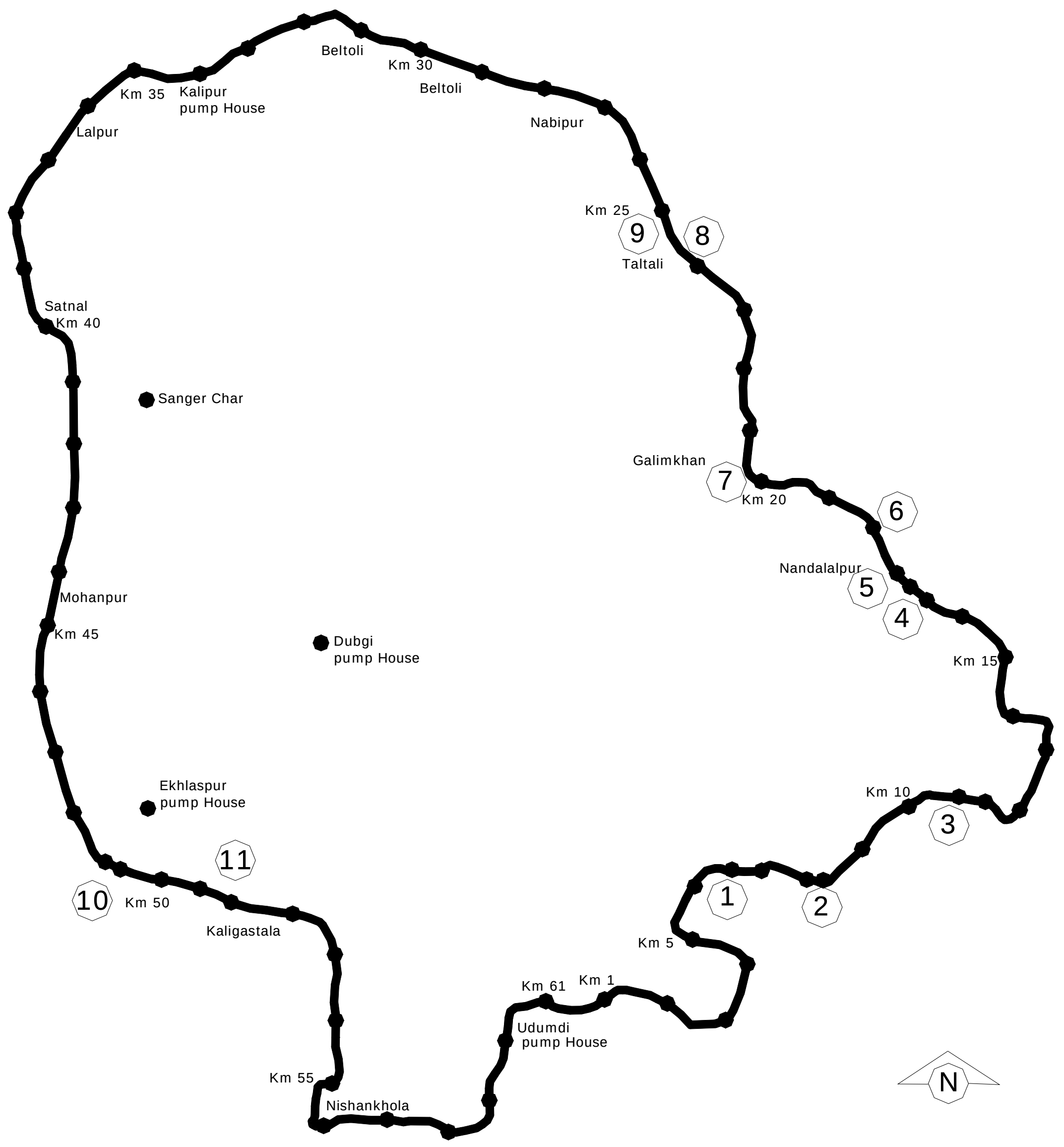

Figure 2.4: Location of boreholes marked by circles and distress areas are marked by numbers in the MDIP.

(e) Findings of BRTC, BUET about the problems in MDIP

Based on subsoil investigation data and their analyses, they concluded that the embankment and the foundation soil of MDIP embankment mainly consists of alluvial silty sand. This type of soil is prone to distress especially when used in earthen embankment. During the recent floods, especially in 2004, the embankment was under tremendous threat to breach. The post flood observation of the sites revealed that the vulnerability of the embankment was primarily because of water seepage through the embankment, especially through the foundation soil. This allowed the finer particles to migrate through the interconnecting voids thus forming an effective water channel within the soil mass and resulted in boiling at several locations.

From field SPT values and empirical correlations, the relative density of the embankment and foundation soil was estimated to be in the range of 20% to 55%. From direct shear test, apparent cohesion was found to be small, a maximum value of 16 kN/m^2 was obtained. The values of internal angle of friction of the soils were found to be in the range of 27^o and 36^o (void ratio 0.81 to 1.13 i.e. porosity 0.45 to 0.53). constant head permeability test was carried out using a special constant head permeameter. In these tests the boiling phenomenon was also simulated for the field conditions. The laboratory determined permeability was of the order of $3x10^{-3}$ m/s which agreed reasonably well with those estimated using well known Allen Hazen's empirical formula.

2.6 CAUSES OF EROSION OF POLDER EMBANKMENT

Earth embankments in Bangladesh are beset with multi-facetted problems. The design and construction methods used to build embankments, the nature and extent of erosive forces to destabilize them and above all the attitude of the local people for whom they are built altogether determine the magnitude and degree of instability.

Most earth embankments face light to moderate erosion problems arising out of rainfall splash, animal action and nature of human uses. Some of the critically positioned submerged types of embankment in haor areas of the eastern part and river embankments of the main land are subjected to turbulent water currents and changes in river courses. The problem is acute in offshore islands and coastal belts where the embankments are in addition exposed to erosion by sea waves and tidal fluctuation of water levels.

In the coastal belts and offshore islands, severe bank erosion problems occur frequently. New accretion and shifting of the bank line due to erosion happen all most in the same way as those observed in the inland rivers. However the nature and extent of erosive forces damaging the seashore and successively the dykes differ in certain aspects from those of the inland riverbanks.

The process of erosion gradually destroys the shore lands, river banks, foreshore areas and successively the earthen embankments engulfing the plain agricultural lands, habitant and many important installations. In the affected reaches, the embankments built with adequate setback are found to disappear within a year or two of the start of erosion. Generally new embankments are constructed along separate alignments (further back in the country side), simultaneously adopting sufficient measure to check the bank erosion. Whenever the situation compels to protect the high-value assets and important installations as the losses are really irreparable or would have a great impact on the national economy, sophisticated structural means of proven structural solutions involving huge costs are adopted to combat the erosive forces. Thus erosion creates a double disaster – erosion of resources from where it is mostly wanted and simultaneous accumulation where it is unwanted.

The most common causes are natural forces and human interference.

2.6.1 Natural Forces

The natural forces cause erosion of the embankments in the following ways:

Rainfall impact: Mean annual rainfall varies from about 1500 mm in the southwest (Khulna district) to over 3570 mm in the southeast (Cox's Bazar). The heaviest rainfall occurs in July and ranges from 350 mm to over 875 mm accordingly. The slope erosion caused by rain runoff is enormous and its speed/force grows exponentially towards the toe. Toe erosion is the combined effect of runoff and wave action. The main features of rainfall impact are:

- The embankment crest is mainly affected with the formation of gullies and initiation of piping action leading to collapses in combination with either.

- Surface runoff caused by rainfall results in sheet erosion and the formation of gullies and drills on the poorly protected embankment shoulders, slopes and toes.

- Flooding (monsoon/periodic floods, and those created by storms/cyclones).

- The high head of water on the riverside induces piping across the embankment which may lead to breaching and collapse of the polder system.

- Monsoon flooding often gives rise to serious erosion of embankments by undermining due to current, vortex and wave forces.

Wave action: Tidal waves cause damage to the embankments located too near to the sea. A severe hydraulic load is steadily exerted on the toes and slopes and causes erosion. Cyclonic storms in the coastal zone (occurring repeatedly) act upon the water surface, causing it to advance towards the shore with enormous hydraulic loads. The wave thus formed eventually hit the embankment toe and slopes. The high hydraulic loads exerted on the embankment cause erosion and if there is overtopping, the physical structure of the embankment is destroyed.

Turbulent water currents:

- The high velocity flow of water associated with vortex motion in rivers and estuaries often causes erosion of the banks by undermining, and the eventual collapse of the embankment threatens unless protective measures are taken.

- At the mouth of a branch river or canal, especially in the surrounding of sluice gates, the turbulent water current erodes the bank and subsequently the embankments.

- The presence of continuous borrow-pits on a river or sea side induces undercutting of the embankment toes and slopes due to complete inundation of the riverbank or seashore during the monsoon. The borrow-pit and adjoining lowlands inundated induce a parallel water current to flow along very near the embankment toes and slopes, thereby eroding the surfaces rapidly.

Wind action: The slow and steady action of wind in the relatively sparse fields and coastlines blows away the top soil of the embankments where it is sandy or a mix of silt and sand. The embankment crest and bare surfaces are gradually eroded to leave patch holes and undulated surfaces for further decaying by rainfall splash, runoff or wave action.

2.6.2 Human Interference

The human interference responsible for major embankment erosion is quite diverse in nature and often varies accordingly to the lifestyle and manner of using the embankments of the inhabitants of different areas. The most commonly observed erosion problems out of the varied human uses are noted below:

Travel paths for men and cattle: The people living around use the embankments as the main travel paths. The crests thus serve as a rural communication road between villages. Different type of vehicles moves regularly on these earthen embankments in the dry season. Movement of bullock carts in the rainy season inscribe deep path marks along the track which induce further decaying of the embankment crest by trapping of the rainwater inside. The people and their cattle, while moving along the damaged crest, often tend to take a better alternative route along the shoulders, slopes and even toes. Gradually the shoulders and slopes are also affected.

Homesteads and Agricultural Practices: Those who have lost their land and properties due to river erosion and acquisition land for development activities and practically have no shelter find no other way than to become squatters to live on the embankments. The presence of homesteads on the embankment gives rise to various other associated uses, including raising agricultural produce for subsistence and intensive cattle grazing. By loosening the soil and denuding it, repeated soil tillage results in severe erosion.

Cattle Grazing: Cattle, mainly belonging to people living on the embankment, cause erosion by uncontrolled browsing of natural grasses. When the embankment is overgrazed plant species and the vegetative cover, especially the grasses, show retarded growth, weaken and cannot continue to ensure adequate protection of the embankment. The squatters, most of them are poor and landless, prefer to keep goats as pet animals with their limited scope at the embankment homesteads. The grazing of goats is particularly harmful to the vegetative cover.

Public cuts: Public cuts and tubes linking a river or seaside with the countryside of its embankment are frequently observed. These cuts weaken the embankments, exposing them to slow but continual erosive forces. During flood or cyclone storm, breaching or major erosion occurs at those points. The people mainly cut the embankment to fulfill their purposes:

- To get rid of poor and inadequate drainage condition of the existing structures, they arrange quick removal of excess floodwater from the polder area to the river or the sea. This is failure of the planning and design end not to take care of this problem appropriately.

- They create temporary irrigation inlets for applying sweet river water to the copping fields when there are prolonged droughts in the polder area. This problem arises due to a need of adequate people participation during planning and design of the embankment.

- For short-term economic purposes yielding individual-level benefits, sometimes people allow river or seawater to penetrate inside the polder for shrimp cultivation or any other fishing requirement or salt planning.

Unplanned afforestation of embankment slopes: Afforestation without appropriate planning and management techniques destroys the undergrowth grass cover and becomes ineffective for erosion protection. In such cases, afforestation results in the weakening of the embankment without any substantial contribution to its ability.

2.7 CAUSES OF FAILURE OF EARTH FILL EMBANKMENTS

Failures of earthen embankment dams or dikes can generally be grouped into three classifications: hydraulic, seepage and structural.

2.7.1 Hydraulic Failures

Hydraulic failures from the uncontrolled flow of water over and adjacent to the embankment are due to the erosive action of water on the embankment slopes. Earth embankments or dikes are not normally designed to be overtopped and therefore are particularly susceptible to erosion. A well vegetated earth embankment or dike may withstand limited overtopping if its top is level and water flows over the top and down the face in an evenly distributed sheet without becoming concentrated in any one area.

Hydraulic failures may be related directly or indirectly to the following:

1) Overtopping - Wave-overtopping takes place when waves meet a submerged reef or structure, but also when waves meet an emerged reef or structure lower than the approximate wave height. During over-topping, two processes important to the coastal processes take place: wave transmission and the passing of water over the structure.

2) Wave Erosion - Notching of upstream face by wave action reduces the embankment cross section thickness and weakens embankment material.

3) Top Erosion - Erosion of downstream toe of the earth slope caused by misdirected spillway outlet discharge.

4) Gullying - Rainfall erosion of embankment slopes. Also caused by traffic from people and vehicles.

2.7.2 Seepage Failures

Most embankments exhibit some seepage. However, this seepage must be controlled in velocity and quantity. Seepage occurs through the earthen embankment or dike and/or through its foundation. Seepage, if uncontrolled, can erode fine soil material from the downstream slope or foundation and continue moving towards the upstream slope to form a pipe or cavity to the pond or lake often leading to a complete failure of the embankment. This action is known as “piping.” Seepage failures account for approximately 40 percent of all embankments or dike failures.

Seepage can also cause slope failures by saturating the slope material, thereby weakening the adhesive properties of the soil and its stability. Burrows or holes created by animals such as the groundhog, woodchuck, or muskrat create voids in the embankment or dike, which weaken the structure and may serve as a pathway for seepage.

Tree roots can provide a smooth surface for seepage to travel along. When trees die, their decaying roots may leave passageway for seepage to concentrate in. Pipes through the embankment may also provide smooth surface for seepage concentrate along as well.

2.7.3 Structural Failures

Structural failures involve the separation (rupture) of the embankment material and/or its foundation. This type of failure is more prominent in large embankment dams. However, it is not exclusive to large dams and similar occurrences may be seen on earthen embankments or dikes in New Hampshire. Structural failure of an earthen embankment may take on the form of a slide or displacement of material in either the downstream or upstream face. Sloughs, bulges, cracks or other irregularities in the embankment or dike generally are signs of serious instability and may indicate structural failure.

2.7.4 Other

Tree growth on an earthen embankment or dike can be a contributing factor in the failure of an earthen structure and part of any one of the three previously described type of failures. Tree growth directly on the crest or top of the structure could lead to a hydraulic failure should the tree be blown over. This may displace embankment material within the root ball creating a low area susceptible to flows from the impoundment. Tree root systems may also create seepage paths through an earthen embankment or dike and structural failure of an upstream or downstream slope could occur with the displacement of a large tree implanted within the earth slope.

CHAPTER 3 : TEST PROGRAM

3.1 GENERAL

Damaged embankment of polder no. 14/1 at Patakhali, Dakkhin Betkashi, Koyra under the district of Khulna was selected as the study area for this project. Reconstruction of two closures at Patakhali and Padmapukur was going on in full swing at that time. Sufficient amount of bulk samples were collected from the study area. Determination of SPT and sample collection was done at two locations: one at the damaged part of the old embankment and the other is at the proposed construction area near Patakhali closure. Laboratory tests were performed at the Geotechnical Laboratory of Civil Engineering Department of Bangladesh University of Engineering and Technology (BUET) to find index properties, double hydrometer ratio, Standard Proctor Density and optimum moisture content.

3.2 POLDER EMBANKMENT SECTION

Existing polder embankment section was measured during site visit. A typical section of old embankments is shown in Figure 3.1. A typical section of new embankment was collected from BWDB engineers and shown in Figure 3.2.

3.3 FIELD TESTS

Two points at the site of proposed closure were selected for collecting soil sample and field tests. The bore hole locations and bulk sample location are shown in Figure 3.3. It is a very remote area near Sundarbans and very difficult to reach. So, all the field works were completed in one tour. Field density, SPT values and collection of samples were the main tasks which were performed at the site.

SPT values of the sub-soil strata were collected from both old (damaged) and new (proposed) embankment area. A 21.5m deep boring was dug at the old embankment (shown in Figure 3.4) and SPT for every 1.5 m interval was recorded. Samples, at the same interval, for laboratory tests were collected carefully and preserved. Undisturbed samples at every 3m depth were tried to collect by inserting Shelby tube but it was not possible to collect any undisturbed Shelby tube sample. Because the soil throughout the depth was soft grey lean clay, non-plastic silt, silt with sand and sandy silt. On the other hand, a 15.5 m boring was done at the new embankment area (shown in Figure 3.5). SPT values at 1.5 m intervals were recorded and disturbed samples for lab test were collected. Like the first boring it was not possible to collect any undisturbed sample from this borehole. Here the soil was lean clay, silt with sand and sandy silt.

Approximately 15 kg bulk sample (see Figure 3.3) was collected from adjacent to closure area at 1.5 m depth by manually excavating top soil. Two in-situ field density test using sand cone method were performed; one on the top of the existing embankment and another at the toe of the existing embankment (see Figure 3.3).

3.4 LABORATORY TESTS

A detailed laboratory investigation was carried out on soil samples collected from the boreholes drilled at the site. All samples were taken to Geotechnical Laboratory of BUET to perform Atterberg Limits, Specific Gravity Test, Sieve Analysis, Hydrometer Analysis, Standard Proctor Compaction Test, and Double Hydrometer Ratio Test.

The Double Hydrometer Ratio Test also known as Soil Conservation Service Laboratory Dispersion Test was performed to identify the dispersiveness of soil. The particle size distribution of the soil is first determined using the standard hydrometer test where the soil is dispersed in distilled water with chemical dispersant. A parallel hydrometer test is then made on a same soil specimen but without a chemical dispersant. The percent dispersion is the ratio of the dry mass of particles smaller than 0.005mm diameter of the second test to the first expressed as a percentage.

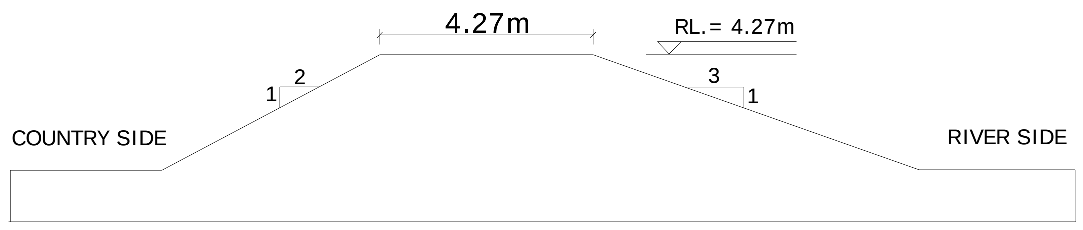

Figure 3.1: Typical section of old embankments.

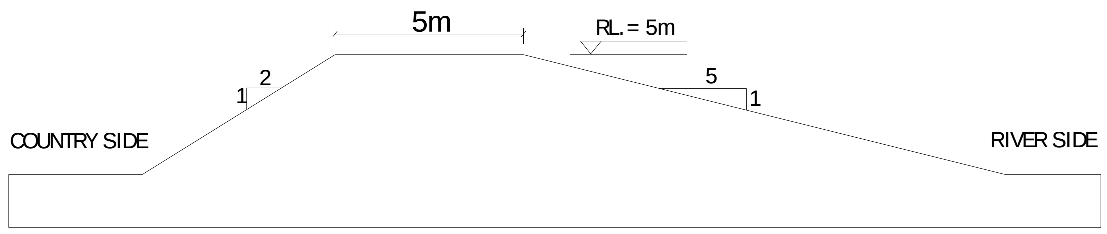

Figure 3.2: Typical section of new embankments.

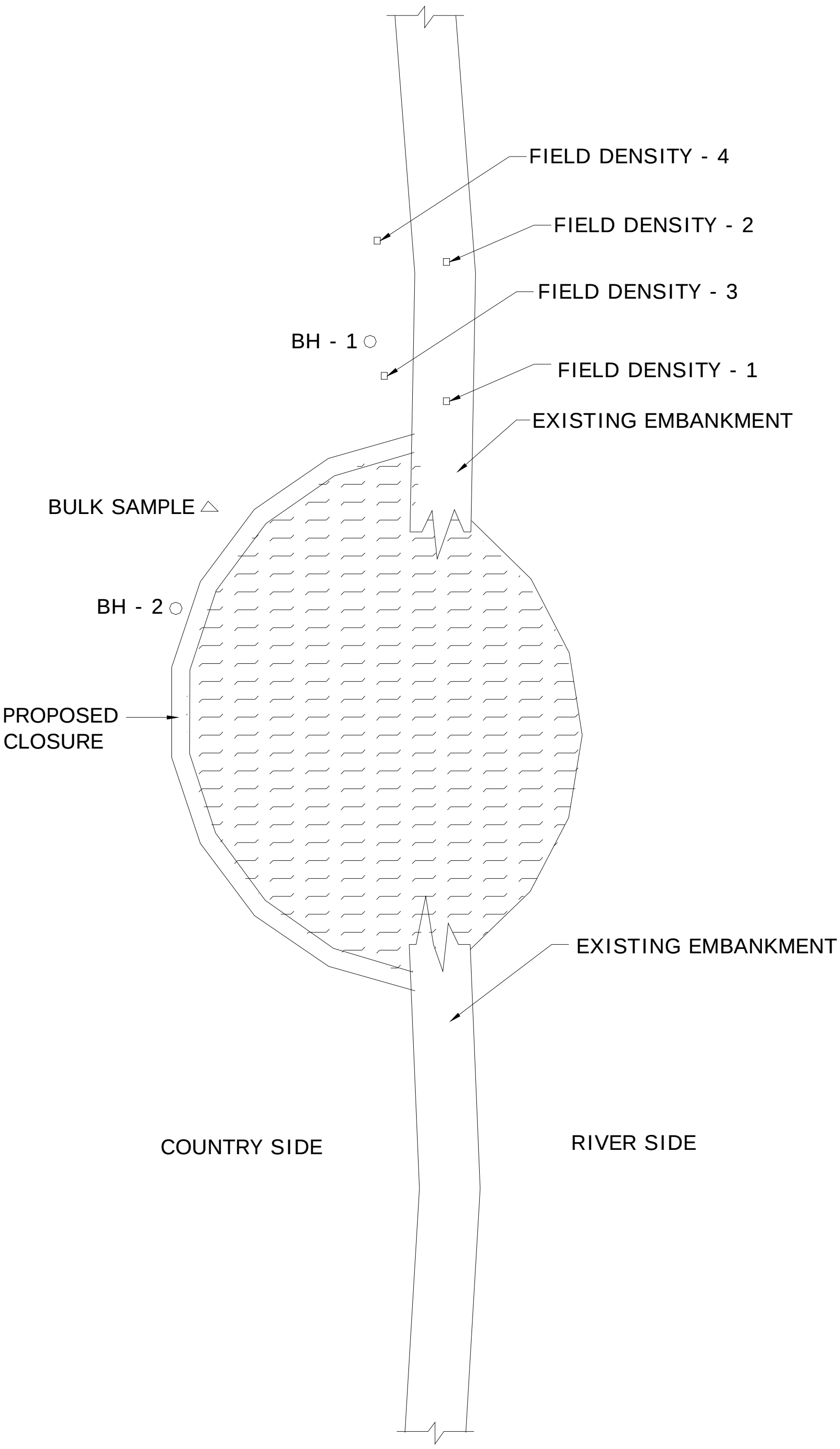

Figure 3.3: Locations of SPT boreholes, field density tests and bulk soil sample.

Figure 3.4: First borehole, BH-1 (21.5 m deep), near old embankment at Patakhali closure, Koyra, Khulna.

Figure 3.5: Second borehole, BH-1 (15.5 m deep), near new embankment at Patakhali closure, Koyra, Khulna.

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 GENERAL

Results of field and laboratory tests have been presented in this chapter. It is found that the sub-surface soil consists mainly of fat and lean clay in the upper layers up to the depth of eleven meters whereas the layers below this consist of non-plastic silt, silt with sand and sandy silt. These subsoil strata together with double hydrometer ratio helped to find the cause of failure of polder embankments.

4.2 ATTERBERG LIMITS

Atterberg Limits of disturbed samples collected from different depths of the boreholes were performed to determine Liquid Limit, Plastic Limit and Plasticity Index [8]. A summary of the Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) is shown in Table 4.1, 4.2 and 4.3. It may be noted that LL and PL for soil samples vary from 40 to 57 and from 14 to 28 respectively. Plasticity Index varies from 17 to 37.

Table 4.1, 4.2 and 4.3 shows that Atterberg limits were not applicable for some samples collected from different depths of the boreholes. The samples which were brought under this test showed low values of liquid limit and plastic limit and therefore low value of plasticity index. It means soil has low cohesion and decaying characteristic against flowing water.

Table 4.4 and 4.5 shows the Liquidity Index, Flow Index and Toughness Index of the soil samples collected from the site vary from 0.118 to 0.545, 3.5 to 63 and 0.3 to 9.4 respectively. Most of the samples show high value of flow index and very low value of toughness strength which indicates soil has low shear strength. So, shear failure may be one of the causes of the failure of embankment.

4.3 GRAIN SIZE DISTRIBUTION

Wet Sieving method of sieve analysis was performed on 23 soil samples collected from two boreholes as per [9] in order to determine the grain size distribution of the collected soil samples at different depths. Percentage of sand fraction is shown in Table 4.1, 4.2 and 4.3. It may be noted that percent sand ranges from 0 to 49.6. Below 7.5 m depth of BH-2 which is adjacent to closure, have more percentages of sand than that of BH-1. This might be the one reason why it was very difficult to construct the closure in several attempts.

4.4 USCS CLASSIFICATION OF SOIL SAMPLES

For classification of the soil samples Unified Soil Classification System [8] was used. Tables 4.1, 4.2 and 4.3 show the USCS classification of soils. At old embankment area sub soil layers consist of lean clay and non-plastic silt whereas lean clay, silt with sand and sandy silt dominates in the layers of soil at the new embankment area. Bulk sample collected from the surface is classified as lean clay.

According to above classification it is seen that soil forming the sub-soil strata consists of lean clay, non-plastic silt and silt with sand which are very weak against flowing water. This is why it is very difficult to repair any broken embankment against tidal flow and rapid draw down during ebb-tide which easily washes away soil particles.

4.5 IDENTIFICATION OF SOILS SUSCEPTIBLE TO DOWNSLOPE MIGRATION

The problem of downslope migration is most apparent in silts, sandy silts and fine sands. The minor cohesive properties of these types of soils tend to encourage high mobility of small single grains at relatively low hydraulic gradients [11].

The distinction between soils susceptible to downslope migration and those not susceptible is related to grain size, permeability and degree of cohesion.

In most of samples shown in the figures (Figure 4.3, Figure 4.4 and Figure 4.5) it is seen that 85 % of the soil particles have diameter smaller than 0.06 mm. That means it satisfies the susceptibility of downslope migration as per PIANC [11].

Additionally, the coefficient of uniformity of the soil samples calculated from the above mentioned graphs are 7.2, 7.8 and 6.8 respectively, all of which are significantly below the upper limit 15. This criteria of downslope migration susceptibility is also satisfied by the embankment and subsoil. An average of 40% of the particles lies in the range 0.02 mm < d < 0.10 mm, which does not satisfy the criteria of downslope migration susceptibility. Plasticity Index of the collected samples from embankment and subsoil under embankment was found to be greater than 15 which satisfy the criteria of downslope migration susceptibility of soil. Considering all these criteria and soil properties, it can be concluded that embankment soil and subsoil are susceptible to downslope migration.

4.6 FIELD SPT VALUE

SPT bore log of BH-1 and BH-2 are shown in Figure 4.1 and Figure 4.2. Bore log found from old embankment area reveals that the upper crust of the soil is very soft to medium stiff up to the depth of 14 m through which SPT value varies from 1 to 7. But the value increases rapidly within the depth of 15.5 meter to 18.5 meter from 15 to 52. It then remains unchanged at a value of around 50 up to the depth of 21.5 m.

The second bore log shows that at the proposed new embankment area soil is soft up to the depth of 6 m through which SPT value varies from 1 to 5. But within the next three meters i.e. at 9 meter depth it reaches its peak at a value of 42. Then it shows a trend of fluctuation. At 11 m and 12.5 m it climbs down to 16 and 10 respectively whereas by the next 1.5 m it again jumped to 42.

At the old embankment the soil profile represents that soil is soft up to a significant depth of the sub soil, so it could be a cause why the embankment washed out.

4.7 FIELD DENSITY

Sand Cone Method was used to find the field density of existing embankment and in situ soil at the toe of embankment. The locations of field density tests are shown in Figure 3.3. The first two tests were on the crest of the embankment and another two tests were 0.5 m below the land surface at the toe. Field density test results are shown in Table 4.4. It was found that field dry density at the crest and toe of embankment was 16.4 kN/m^3 (Percent compaction = 98%) and 12.2 kN/m^3 (Percent compaction = 73) respectively. Natural Moisture Content at the crest and toe of embankment was 12.2% and 29.6% respectively.

Standard Proctor Compaction test has been performed on bulk samples collected from 1.5 depth of the new construction site. To get representative value, two tests were conducted on the same sample. It is found that the maximum dry density 16.6 kN/m^3 attains in the soil sample at an optimum moisture content of 17%. The results of the tests are shown in the Figure 4.9. Standard Proctor Dry density was also determined for determination of field density which is shown in Table 4.4.

It is seen that field dry density of the soil on the crest is higher than that of the toe. This is because of the natural compaction by movement of men and cattle. Another cause may be that the soil on the crest dries in the sun and shrink in the dry season and gets its maximum field density. On the other hand soil at the toe represent the in-situ density of recently deposited alluvial soil which more susceptible to erosion.

4.8 HYDROMETER ANALYSIS

Hydrometer analysis was done using bulk soil samples and other samples collected from BH-1 and BH-2. Test results are shown in the Figure 4.3, Figure 4.4 and Figure 4.5. For the entire graphs uniformity coefficient, C_u of the soil is about 7.5 which satisfies the criteria of soil susceptible to downslope migration.

4.9 DOUBLE HYDROMETER RATIO TEST

To know the dispersiveness of soil, double hydrometer ratio was found out using the data from hydrometer analysis. The particle size distribution is first determined using the standard hydrometer test in which the soil specimen is dispersed in distilled water with a chemical dispersant. A parallel hydrometer test is then performed on the same soil specimen, but without a chemical dispersant. The percent dispersion is the ratio of the dry mass of particles smaller than 0.005 mm diameter of the test without dispersing reagent to the test with dispersing reagent expressed as a percentage [10]. According to this method the double hydrometer ratio of bulk sample B-1, disturbed sample D-2 (BH-1) and D-3 (BH-2) are 43%, 35% and 37% respectively. The criteria for evaluating degree of dispersion using results from the double hydrometer test are shown in Table 4.5 [6]**.** The results obtained from the tests fall in the medium dispersive range of the table. The double hydrometer ratios obtained from hydrometer analysis are presented in the Figure 4.6, Figure 4.7 and Figure 4.8.

The double hydrometer ratio obtained from the test results shows that soil collected from the site falls into medium dispersive group which means that soil would be easily washed away with flow of water. So this may be one of the causes of failure of the embankment and difficulties aroused during construction of closure.

4.10 POLDER EMBANKMENT CONSTRUCTION PRACTICES

During field test visit to the polder it was observed that some parts of an embankment was being constructed using locally available silty fine sand which is covered by silty clay. A typical section of this embankment is shown in Figure 4.10. Since the cover soil is medium dispersive, the embankment might be easily washed out when overtopping of storm surges will occur.

4.11 CAUSES OF FAILURE OF EMBANKMENT

The sub-surface soil consists mainly of fat and lean clay in the upper layers up to the depth of eleven meters whereas the layers below this consist of non-plastic silt with sand. Since the soil is non-plastic i.e. the soil possesses no cohesion, it is vulnerable to erosion and so if any failure takes place in the embankment due to any cause it propagates quickly and cannot be stopped and becomes a catastrophe within days.

Double hydrometer ratio test results on lean clay show that the soil samples collected from the surface and the boreholes are medium dispersive. When water flows through it or in contact with it the soil particles are easily washed away with water. So, it might be the cause of failure of the embankment.

In situ density of soil is very low. Usually no compaction is done during construction of polder embankment. Because it is very expensive to carry compaction machine to the site as it is very remote place in Bangladesh. So the embankment remains at high void ratio which is susceptible to piping and erosion.

Embankments are widely used as the travel paths for men and cattle in the rural area. Poor people use embankments for constructing their homesteads and agricultural purposes. They often cut it to get rid of poor and inadequate drainage or to take water inside for farming. Due to this public cuts embankments get decayed and become weak. It loses the strength to withstand against any strong tidal surge or any natural disaster like cyclone and finally collapse.

4.12 POSSIBLE SOLUTIONS TO THE PROBLEM

Many works have been done by different researchers on increasing the stability of in-situ soil mass. After studying related literature and observing the test results of this study following solutions may be adopted to solve the problem.

i. Bhuvaneshwari and Soundara [6] has worked on dispersiveness of clayey soil. After an intensive research work they have suggested that smaller percentage of lime was not sufficient to reduce dispersiveness, the addition of 5% lime decreased the double hydrometer ratio to 9.5 and the combination of 2% lime and 9% fly ash, 2% Lime + 11% fly ash, 2% lime +15% fly ash, the ratio was nearly 1and 0 indicating non dispersiveness. So, during construction of embankment, the embankment may be covered by an improved soil mixed with lime and fly ash.

ii. Embankments are widely used as the travel paths for men and cattle in the rural area. Poor people use embankments for constructing their homesteads and agricultural purposes. They often cut it to get rid of poor and inadequate drainage or to take water inside for farming. Due to this public cuts embankments get decayed and become weak. It loses the strength to withstand against any strong tidal surge or any natural disaster like cyclone and finally collapse. Increasing awareness among the people who live near the embankment about the adverse effects of those practices may reduce the possibility of failure.

iii. The shoulders and side slope of embankment can be vegetated with grass and shrubs so that the soils can be protected against being washed out or erosion. In this case salinity could be a problem that can have adverse effect on plantation. Some selected type of grass which can grow in saline water can be planted to overcome the problem. It will be easier and economic since these types of plants are available in the nearby Sundarbans.

iv. Concrete block can be used as revetment to protect the embankment from high wave of tide. But the materials needed for preparing blocks are unavailable in the area and very costly. So, it may be economic.

v. Crest of the embankment can be covered with brick soling or lime mixed soil or cement mixed soil. In this case, bricks are available in the area but cement and lime are to be brought from outside which will be expensive. So brick soling may be a solution to protect the crest from erosion.

vi. Islam et al [4] has studied the effectiveness of Vetiver grass against the protection of embankment. He discussed that the special attributes of Vetiver grass is its longer life, high resistance to extreme climatic variation, finely structured root system, cost effectiveness and environment friendliness. It is found that in the climatic and soil condition of the coast of Bangladesh, Vetiver grass can grow and survive. He also studied that strong and finely structured root system of vetiver increases the shear strength of the soil. It means that vetiver will be effective to protect the coastal embankments against cyclonic tidal surge. so in the side slope, Vetiver grass may be planted to protect the embankment.

vii. According to PIANC, there are three ways of preventing serious downslope migration: (a) *attenuation of the hydraulic gradient* – this can be achieved by incorporating a granular sublayer between the geotextile and the coverlayer. In practice the granular material should be fine enough to provide an adequate damping effect yet coarse enough to be retained by the cover layer. The sublayer should have a thickness of between 100 mm and 500 mm. (b) *stabilization of the soil surface* – experience has shown that a thick layer of coarse fibres attached to the back of the geotextile filter integrates with the soil surface, thus reducing downslope migration of soil particles within this layer. (c) *application of a constraining load* – this can be achieved by using a heavy weight coverlayer which imparts a significant load into the bank slope thereby resisting uplift pressures caused by excess pore water pressures.

Table 4.1: USCS Classification of soil samples collected from BH-1 at Old Embankment of Patakhali closure, Koyra, Khulna.

Sample Depth (m)	Sample ID	Sand Fraction (%)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Color	USCS Classification
1.5	D-1	6.0	50	19	31	Dark Grey	Fat Clay
3.0	D-2	4.8	51	18	33	Dark Grey	Fat Clay
4.5	D-3	7.6	51	25	26	Dark Grey	Fat Clay
6.0	D-4	6.6	57	20	37	Dark Grey	Fat Clay
7.5	D-5	5.7	44	14	30	Dark Grey	Lean Clay
9.0	D-6	2.8	N/A	N/A	N/A	Grey	Non Plastic Silt
10.5	D-7	2.6	43	23	20	Grey	Lean Clay
12.0	D-8	3.3	N/A	N/A	N/A	Grey	Non Plastic Silt
13.5	D-9	4.7	N/A	N/A	N/A	Grey	Non Plastic Silt
15.0	D-10	24.9	N/A	N/A	N/A	Grey	Non Plastic Silt with sand
16.5	D-11	6.2	N/A	N/A	N/A	Grey	Non Plastic Silt
18.0	D-12	4.8	N/A	N/A	N/A	Grey	Non Plastic Silt
20.0	D-13	5.7	N/A	N/A	N/A	Grey	Non Plastic Silt
21.5	D-14	5.1	N/A	N/A	N/A	Grey	Non Plastic Silt

Table 4.2: USCS Classification of soil samples collected from BH-2 at New Embankment of Patakhali closure, Koyra, Khulna

Sample Depth (m)	Sample ID	Sand Fraction (%)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Color	USCS Classification
1.5	D-1	0	N/A	N/A		Grey	Non Plastic Silt
3.0	D-2	0	45	19	26	Dark Grey	Lean Clay
4.5	D-3	5.7	45	28	17	Dark Grey	Lean Clay
6.0	D-4	3.8	40	18	22	Dark Grey	Lean Clay
7.5	D-5	22.8	N/A	N/A	N/A	Grey	Silt with Sand
9.0	D-6	26	N/A	N/A	N/A	Grey	Silt with Sand
10.5	D-7	35.2	N/A	N/A	N/A	Grey	Sandy Silt
12	D-8	36.9	N/A	N/A	N/A	Grey	Sandy Silt
13.5	D-9	46.6	N/A	N/A	N/A	Grey	Sandy Silt
15	D-10	49.6	N/A	N/A	N/A	Grey	Sandy Silt

Table 4.3: USCS Classification of bulk sample collected from 1.5 m depth

Sample Depth (m)	Sample ID	Sand Fraction (%)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Color	USCS Classification
1.5	BL-1	0	45	24	21	Dark Grey	Lean Clay
1.5	BL-2	0	43	23	20	Dark Grey	Lean Clay

Table 4.4: Index properties of clayey soil collected from BH-1

Sample Depth (m)	Sample ID	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Liquidity Index I_L	Flow Index I_F	Toughness Index I_T
1.5	D-1	50	19	31	0.354	21	1.5
3.0	D-2	51	18	33	0.364	3.5	9.4
4.5	D-3	51	25	26	0.192	48	0.5
6.0	D-4	57	20	37	0.270	21	1.8
7.5	D-5	44	14	30	0.533	7.5	4
10.5	D-7	43	23	20	0.350	10.5	1.9

Table 4.5: Index properties of clayey soil collected from BH-2

Sample Depth (m)	Sample ID	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Liquidity Index I_L	Flow Index I_F	Toughness Index I_T
3.0	D-2	45	19	26	0.423	22	1.2
4.5	D-3	45	28	17	0.118	63	0.3
6.0	D-4	40	18	22	0.545	20.5	1.1

Table 4.6: Field density results

Sl. No.	Location	Depth (m)	Field density (kN/m^3)	Natural Moisture Content (%)	Standard Proctor Density (kN/m^3)	Percent Compaction
Field density - 1	Crest of embankment	0.2 m	16.4	12.2	16.6	98
Field density - 2	Crest of embankment	0.2 m	16.1	10.5	16.6	97
Field density - 3	Toe of embankment	0.5 m	12.2	29.6	16.7	73
Field density - 4	Toe of embankment	0.5 m	12.6	31.0	16.7	75

Table 4.7: Degree of dispersion from double hydrometer ratio test [6]

Percent dispersion	**Degree of dispersion**
<30	Non-dispersive
30 to 50	Intermediate Dispersive
>50	Highly Dispersive

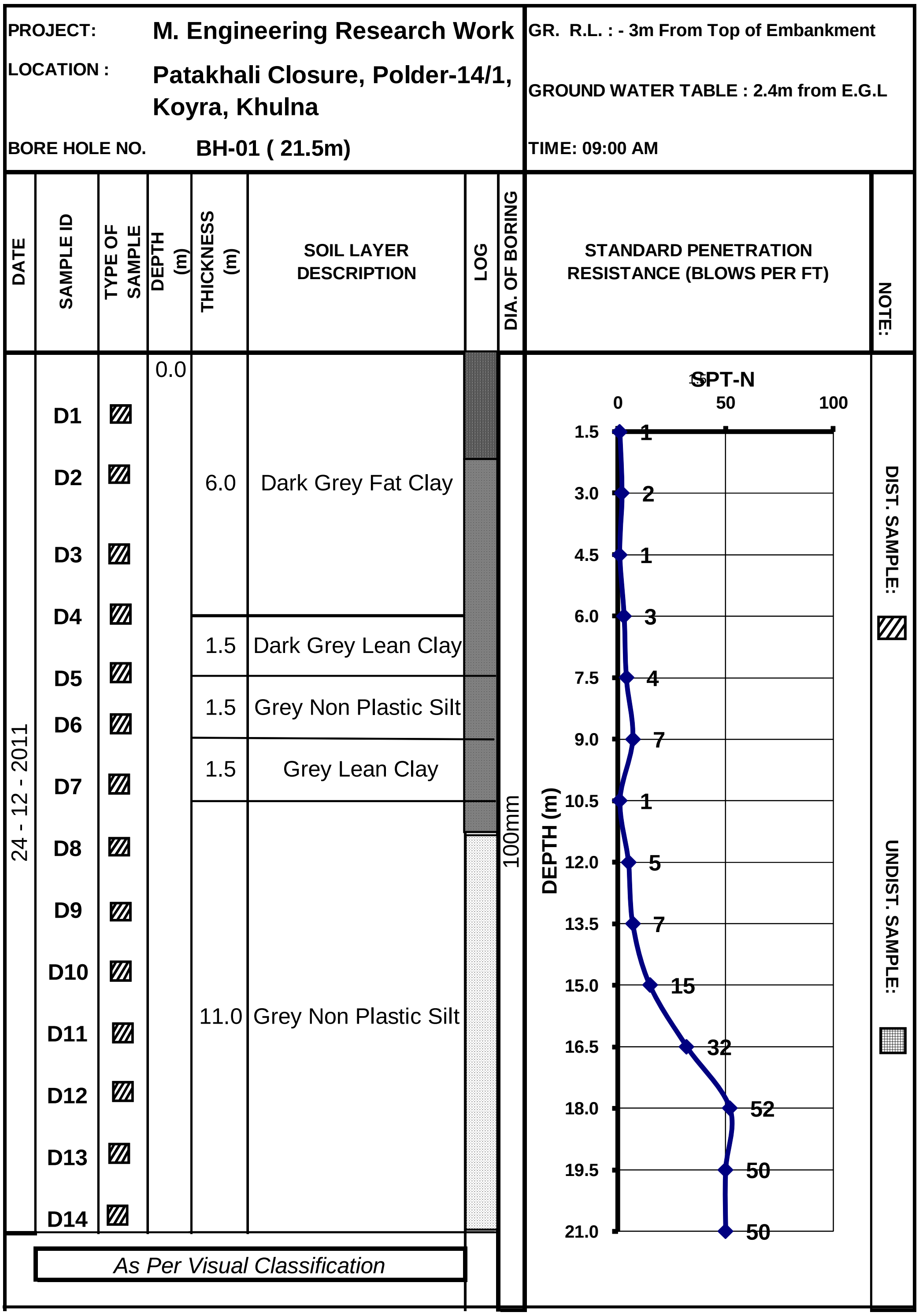

Figure 4.1: Bore log of BH-1 near old embankment at Patakhali closure, Koyra, Khulna

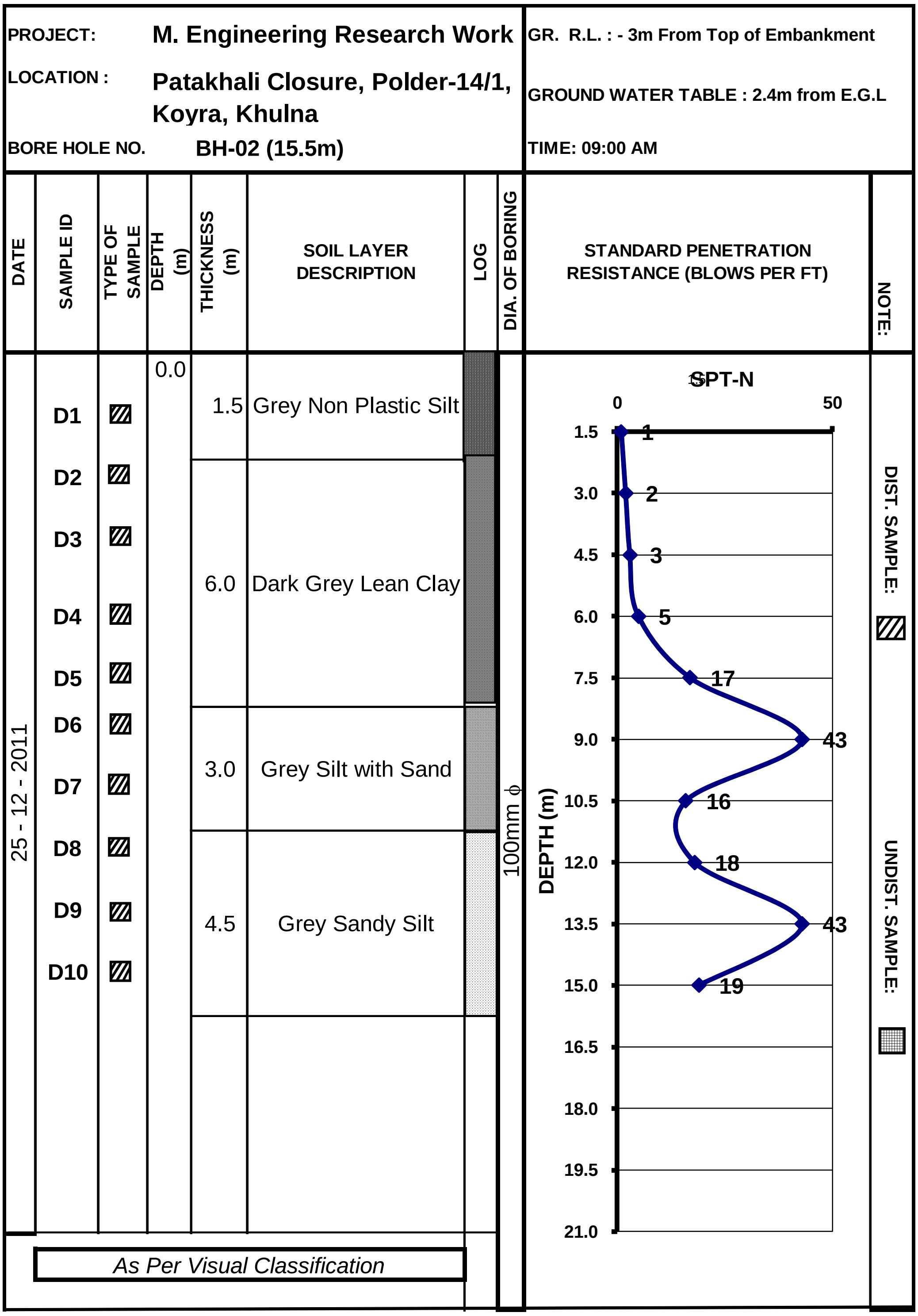

Figure 4.2: Bore log of BH-2 near new embankment at Patakhali closure, Koyra, Khulna

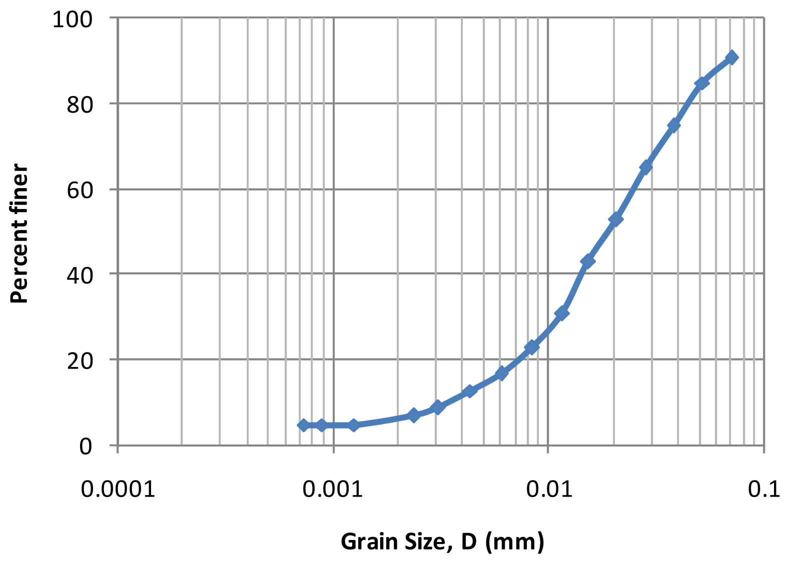

Figure 4.3: Grain size distribution curve of bulk soil sample **B-1** (Lean Clay) collected from Patakhali closure, Koyra, Khulna.

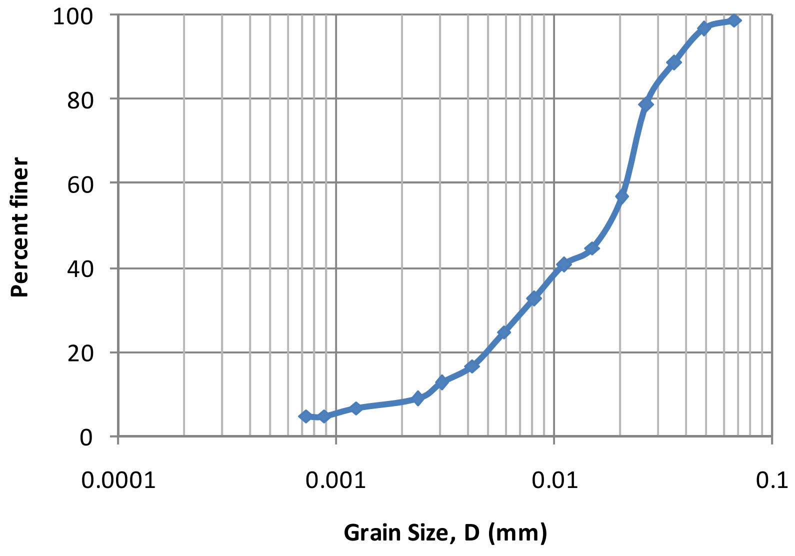

Figure 4.4: Grain size distribution curve of disturbed soil sample **D- 2** collected from BH-1, Patakhali closure, Koyra, Khulna.

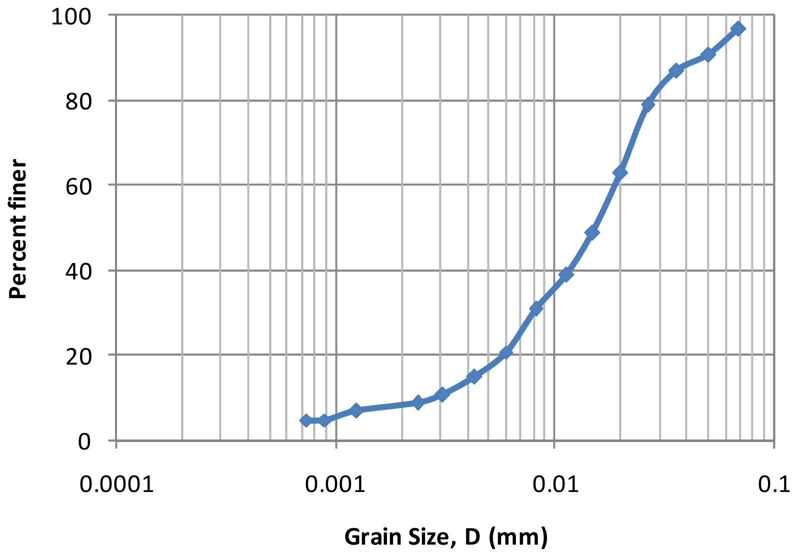

Figure 4.5: Grain size distribution curve of disturbed soil sample D-3 collected from BH-2, Patakhali closure, Koyra, Khulna.

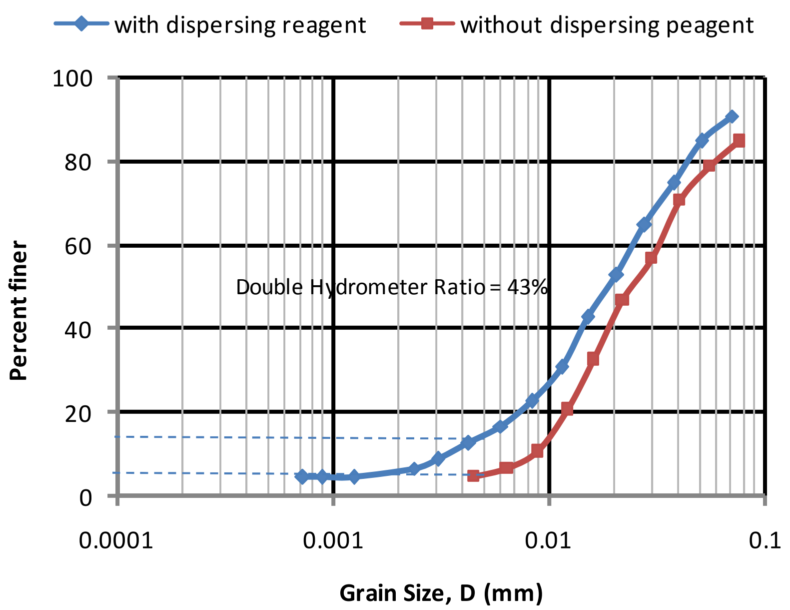

Figure 4.6: Double hydrometer ratio test result of bulk soil sample **B-1 (Lean Clay)** collected from Patakhali closure, Koyra, Khulna.

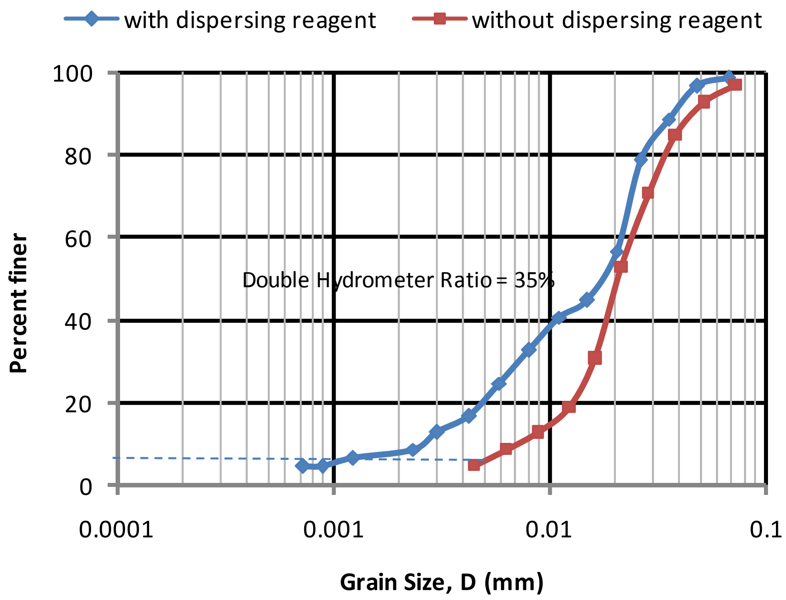

Figure 4.7: Double hydrometer ratio test result of disturbed soil sample **D-2 (Lean Clay)** collected from BH-1, Patakhali closure, Koyra, Khulna.

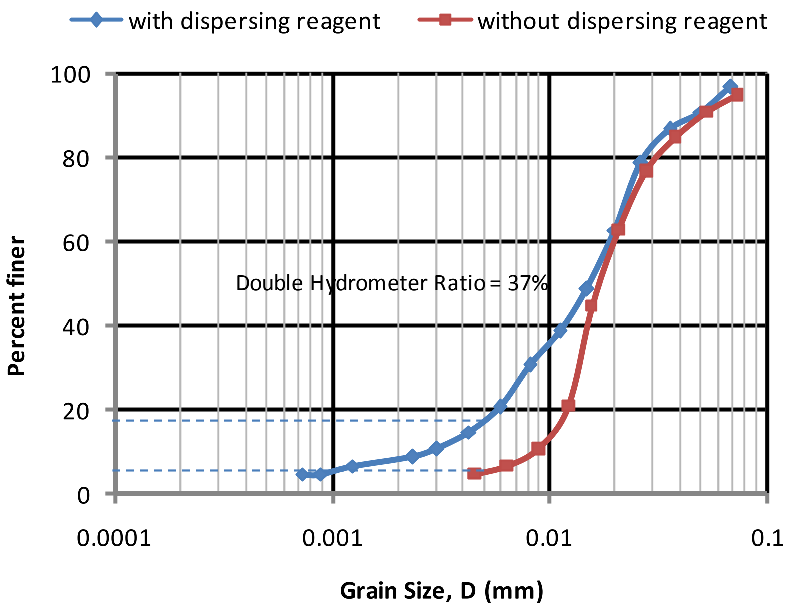

Figure 4.8: Double hydrometer ratio test result of disturbed soil sample **D-3 (Lean Clay)** collected from BH-2, Patakhali closure, Koyra, Khulna.

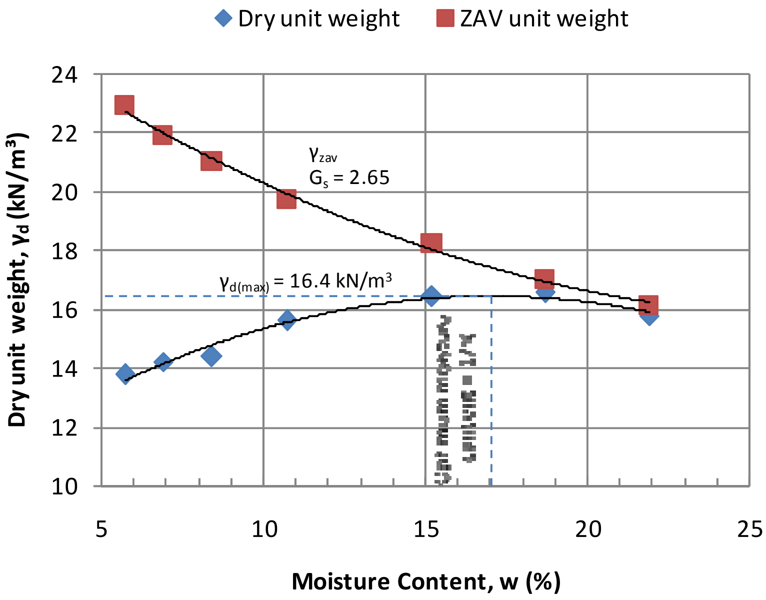

Figure 4.9: Standard Proctor Compaction Test results of bulk soil sample B-1 (Lean Clay) collected from Patakhali closure, Koyra, Khulna.

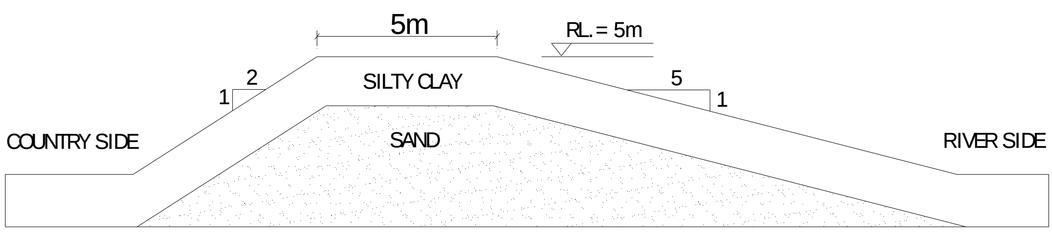

Figure 4.10: Typical section of embankment constructed in Padmapukur, Koyra, Khulna

CHAPTER 5: CONCLUSIONS

5.1 GENERAL

In this study, USCS soil classification, dispersivity, unconfined compression strength maximum dry density of soil were determined from the samples collected from surface and boreholes through several tests performed in the field and in the laboratory. Then these are studied and analyzed and compared with specifications. Conclusions were made according to the results and comparison. The study was undertaken to show the causes of failure of the embankment by studying and analyzing the properties of the soil sample collected from the site.

5.2 CONCLUSIONS

Field and laboratory tests were performed on the samples collected from the breached embankment site of the selected polder. From the test results following conclusions may be drawn:

i. The sub-surface soil consists mainly of lean clay in the upper layers up to the depth of eleven meters whereas the layers below this consist of non-plastic silt with sand. Since the soil is non-plastic i.e. the soil possesses no cohesion, it is vulnerable to erosion and so if any failure takes place in the embankment due to any cause it propagates quickly and cannot be stopped and becomes a catastrophe within days.

ii. Double hydrometer ratio test results on lean clay show that the soil samples collected from the surface and the boreholes are medium dispersive. When water flows through it or in contact with it the soil particles are easily washed away with water. So, it might be the cause of failure of the embankment.

iii. In situ density of soil is very low. Usually no compaction is done during construction of polder embankment. Because it is very expensive to carry compaction machine to the site as it is very remote place in Bangladesh. So the embankment remains at high void ratio which is susceptible to piping and erosion.

5.3 RECOMMENDATIONS FOR FUTURE STUDY

From the lessons of the present study, the recommendations for future study may be summarized as follows:

i. In this study, only two boreholes were dug to collect soil samples and their test results are analyzed. More vulnerable sites of the embankment of the coastal polder should be included and extensive study could be taken place to find out the property and performance of the soil.

ii. More attention should be paid during sampling and transportation of the samples to avoid irregularities and incoherence of test results.

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Appendix A

WASH SIEVE GRAIN SIZE ANALYSIS TEST RESULTS

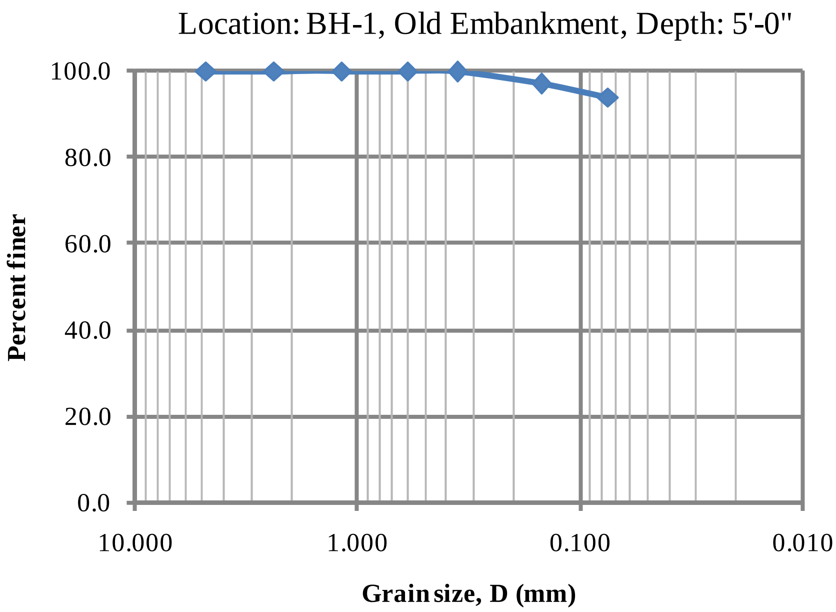


Figure A.1: Typical wash sieve grain size analysis curve of sample (D-1) collected from BH-1 near Old embankment Patakhali closure, Koyra, Khulna.

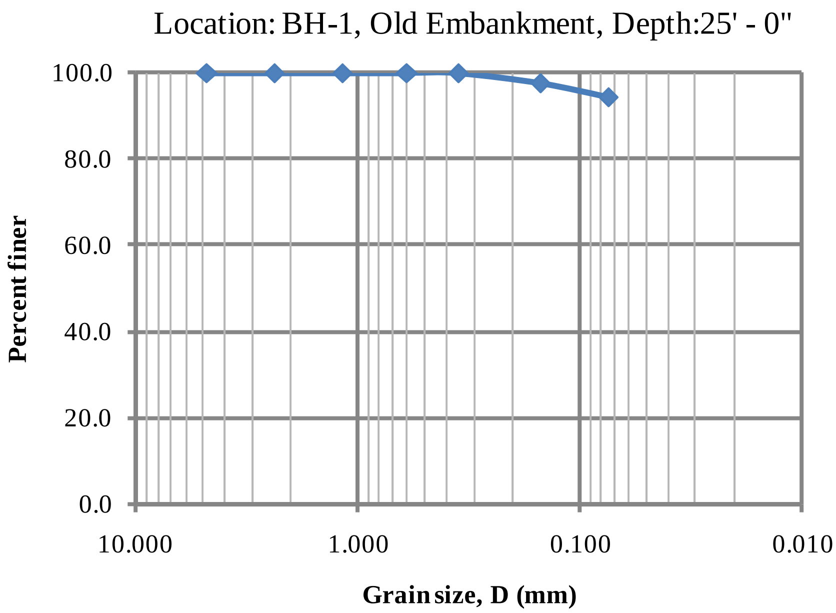


Figure A.2: Typical wash sieve grain size analysis curve of sample (D-5) collected from BH-1 near Old embankment Patakhali closure, Koyra, Khulna.

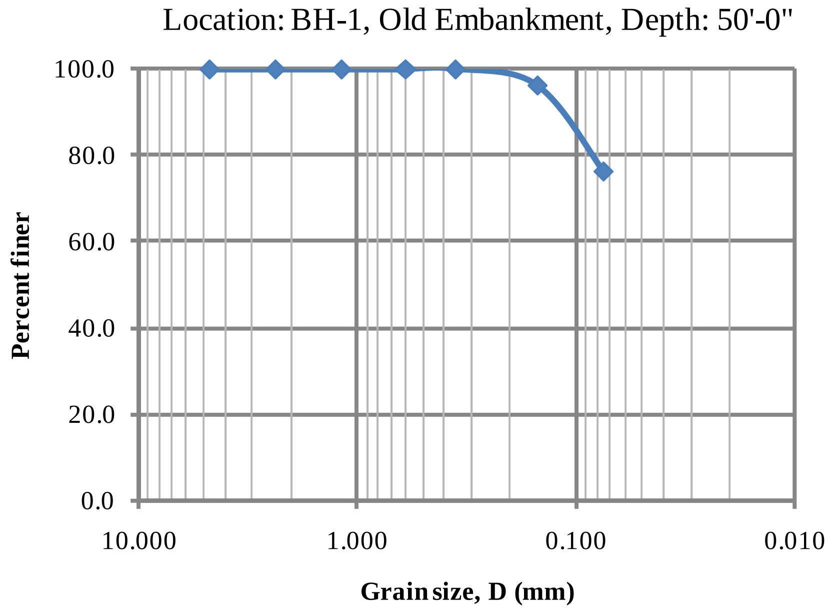


Figure A.3: Typical wash sieve grain size analysis curve of sample (D-10) collected from BH-1 near Old embankment Patakhali closure, Koyra, Khulna.

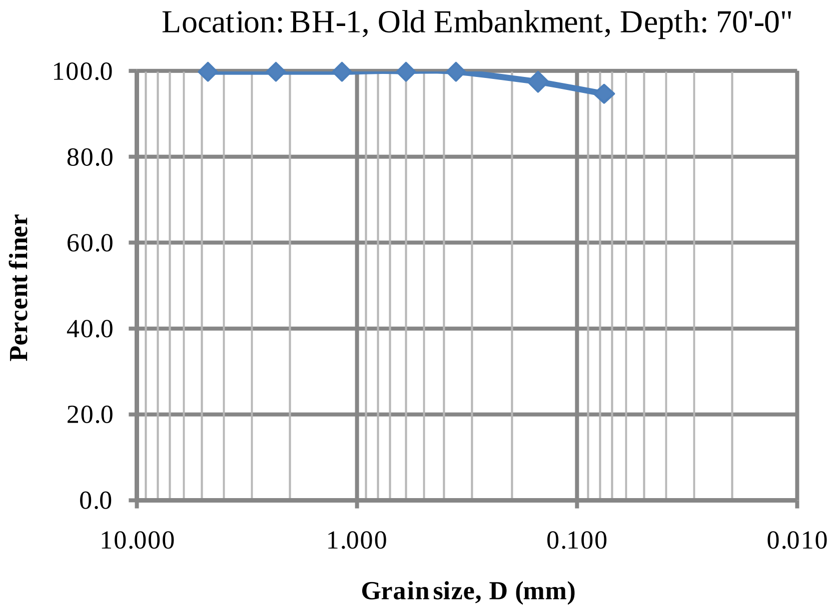


Figure A.4: Typical wash sieve grain size analysis curve of sample (D-14) collected from BH-1 near Old embankment Patakhali closure, Koyra, Khulna.

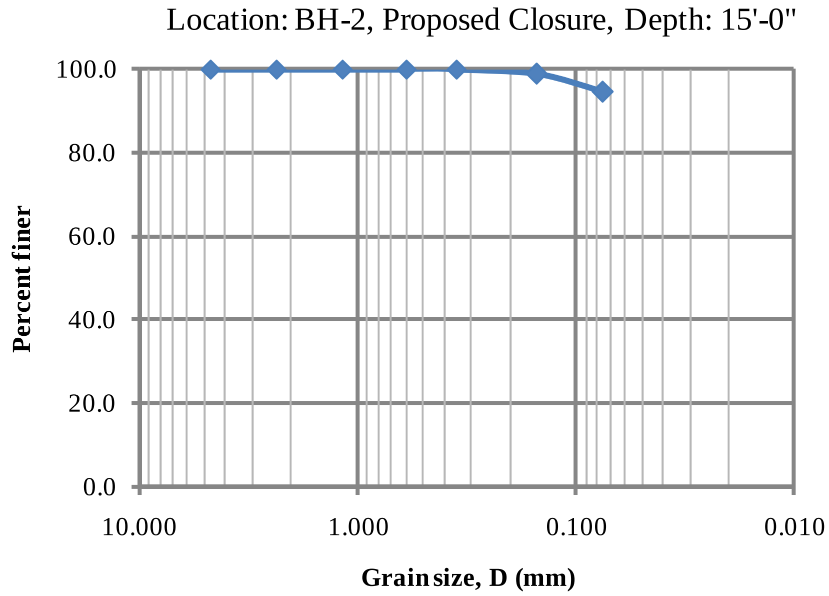


Figure A.5: Typical wash sieve grain size analysis curve of sample (D-3) collected from BH-2 near New embankment Patakhali closure, Koyra, Khulna.

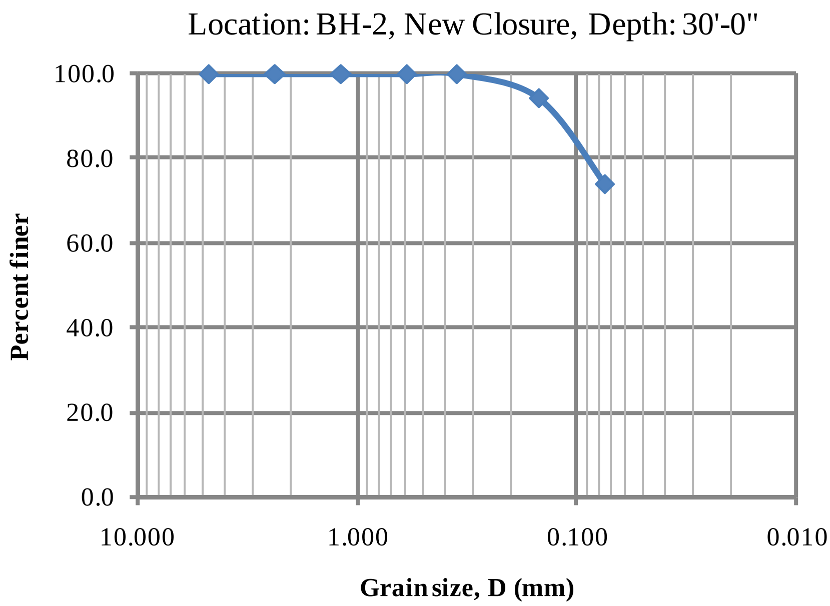


Figure A.6: Typical wash sieve grain size analysis curve of sample (D-6) collected from BH-2 near New embankment Patakhali closure, Koyra, Khulna.

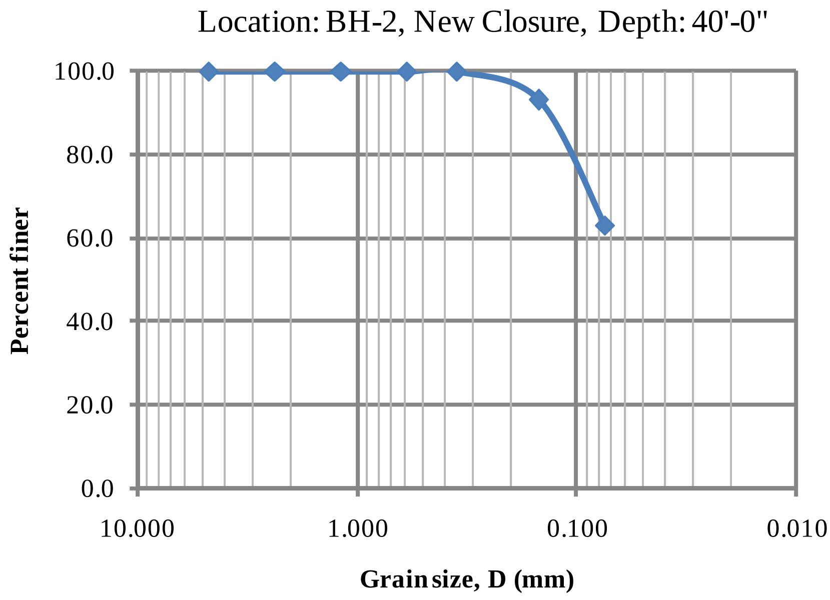
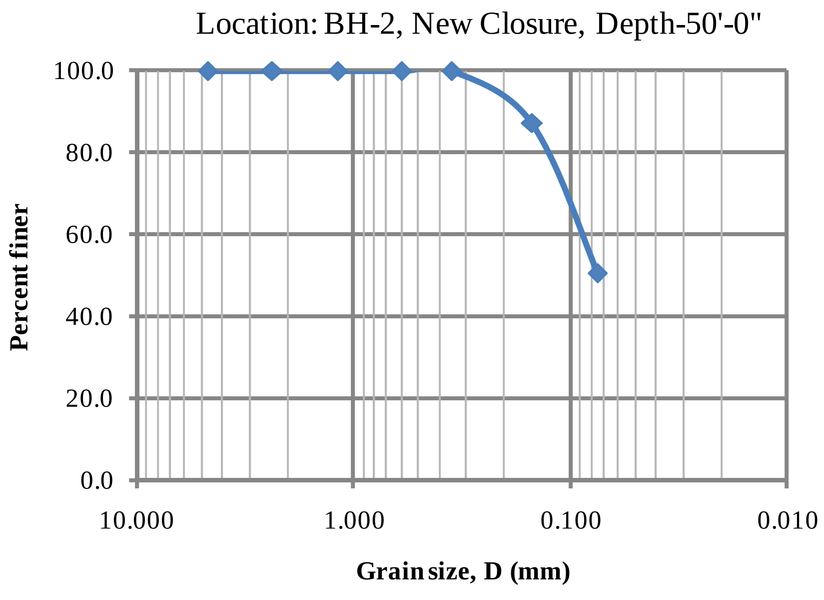


Figure A.7: Typical wash sieve grain size analysis curve of sample (D-8) collected from BH-2 near New embankment Patakhali closure, Koyra, Khulna.



A.8: Typical wash sieve grain size analysis curve of sample (D-10) collected from BH-2 near New embankment Patakhali closure, Koyra, Khulna.

HYDROMETER ANALYSIS TEST RESULTS

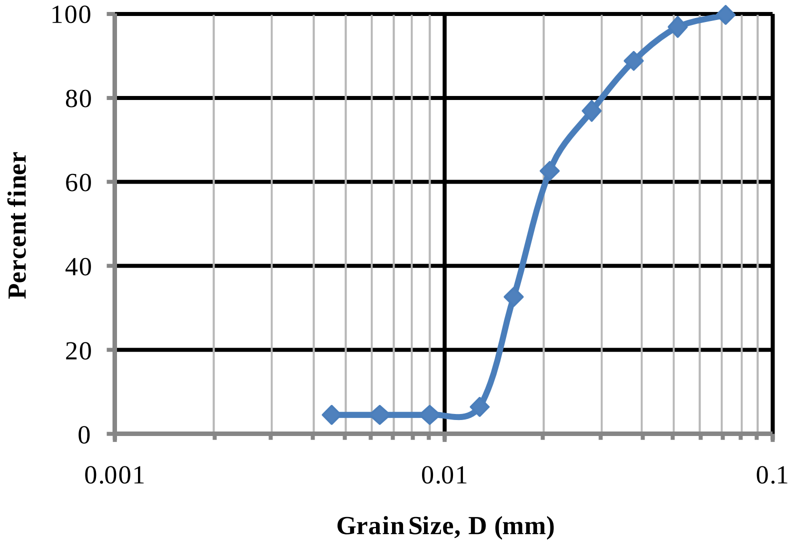


Figure A.9: Hydrometer analysis of Bulk Soil Sample B-1 with saline water.

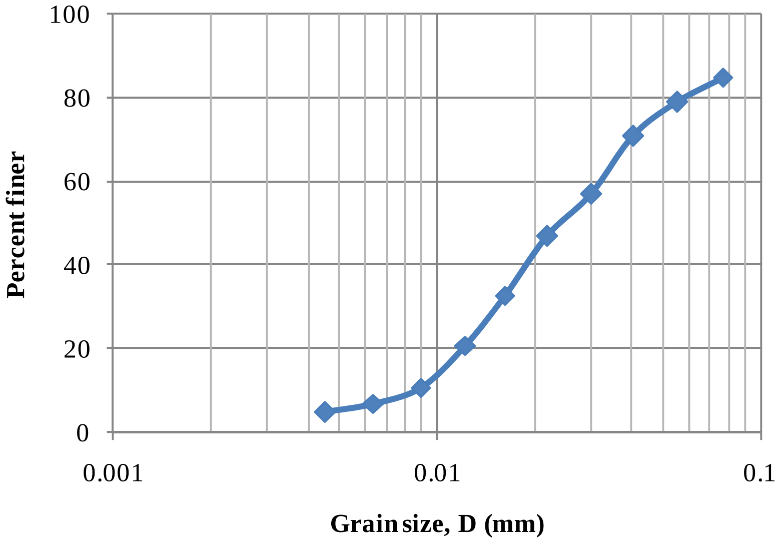


Figure A.10: Hydrometer analysis of Bulk Soil Sample B-1 without dispersing reagent.

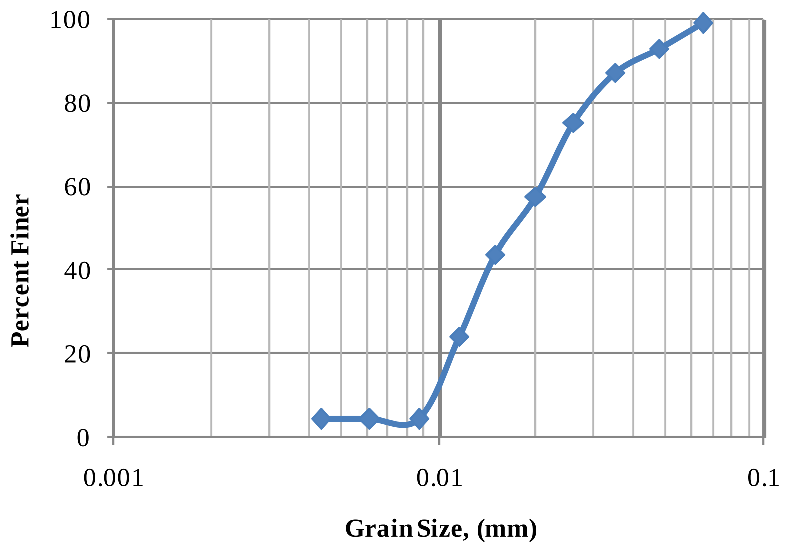


Figure A.11: Hydrometer analysis of the soil sample D-2 collected from BH-1, Old Embankment with saline water collected from the site

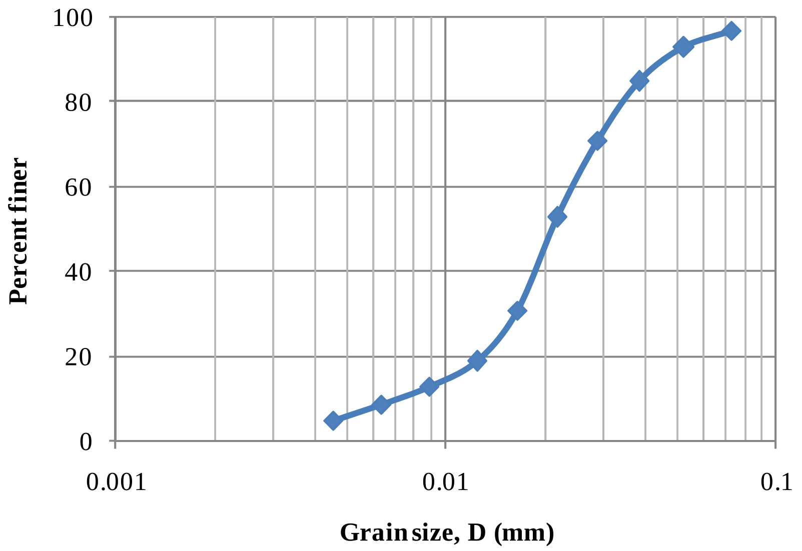


Figure A.12: Hydrometer analysis of the soil sample UD-2 collected from BH-1, Old Embankment without dispersing reagen

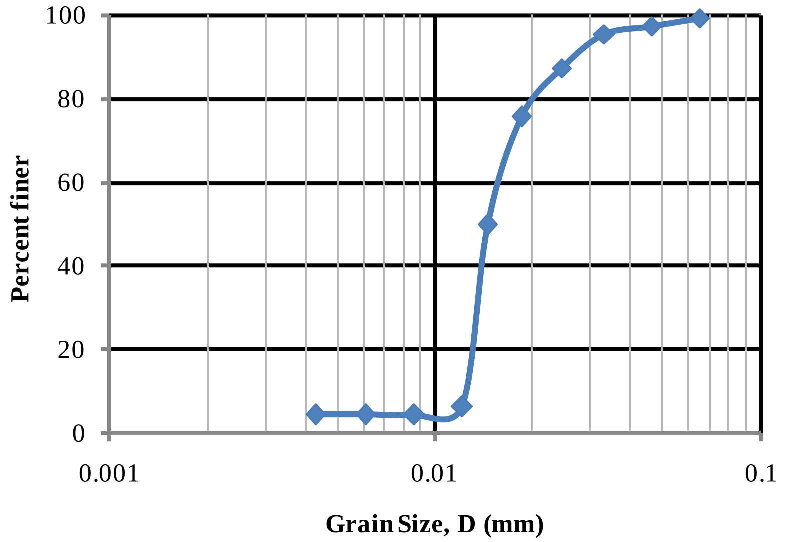


Figure A.13: Hydrometer analysis of the soil sample D-3 collected from BH-2, Proposed New Closure with saline water collected from the site

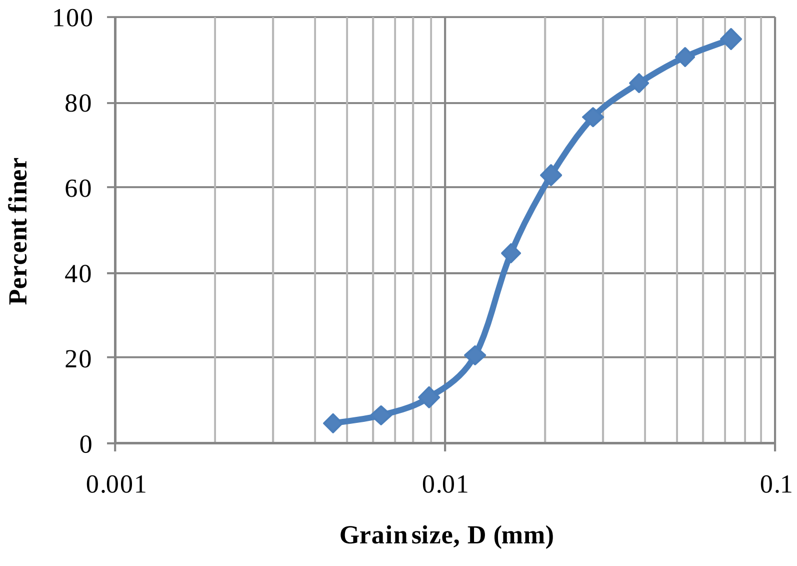


Figure A.14: Hydrometer analysis of the soil sample D-3 collected from BH-2, Proposed New Closure without dispersing reagent

ATTERBERG LIMITS TEST RESULTS

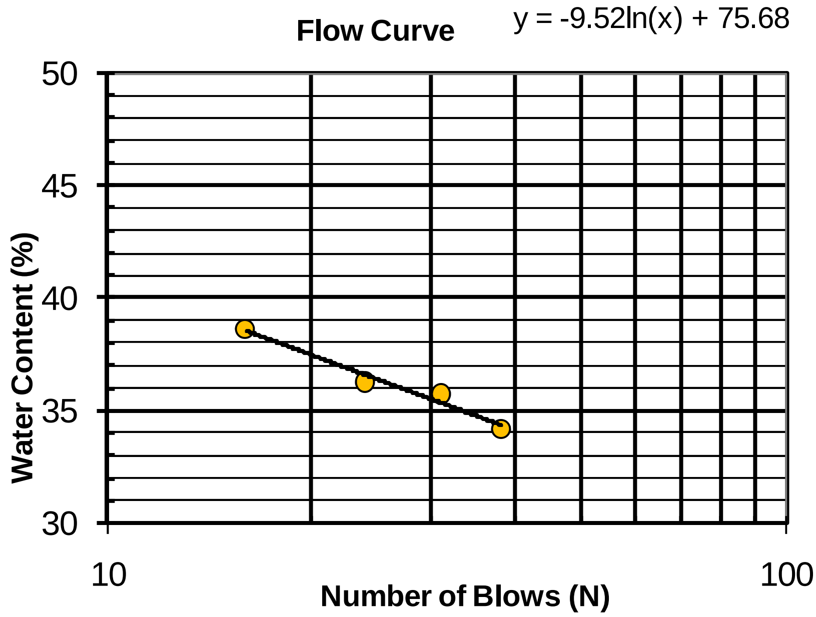


Figure A.15: Flow curve for liquid limit determination of bulk sample B-1, manually collected from 1.5m below top-surface adjacent to the new embankment.

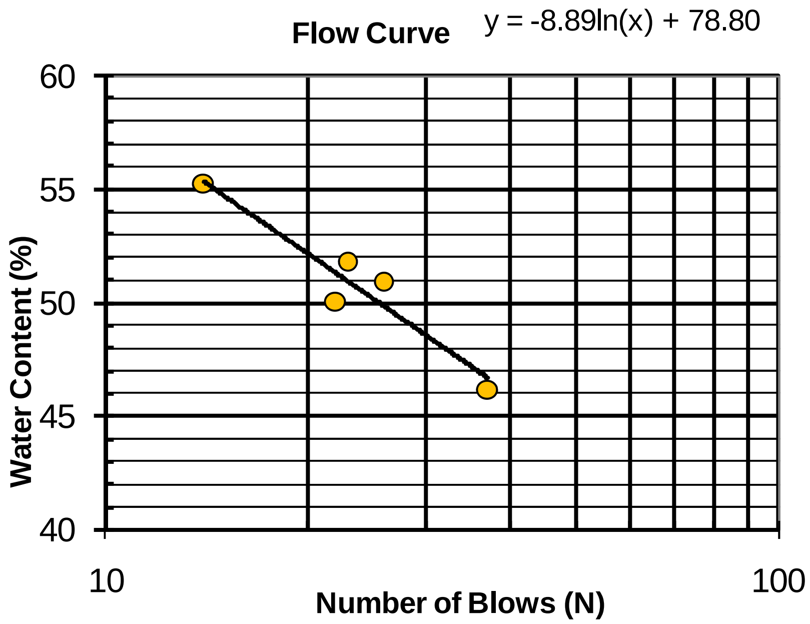


Figure A.16: Flow curve for liquid limit determination of disturbed sample D-1, collected from BH-1 at a depth of 1.5m adjacent to old embankment.

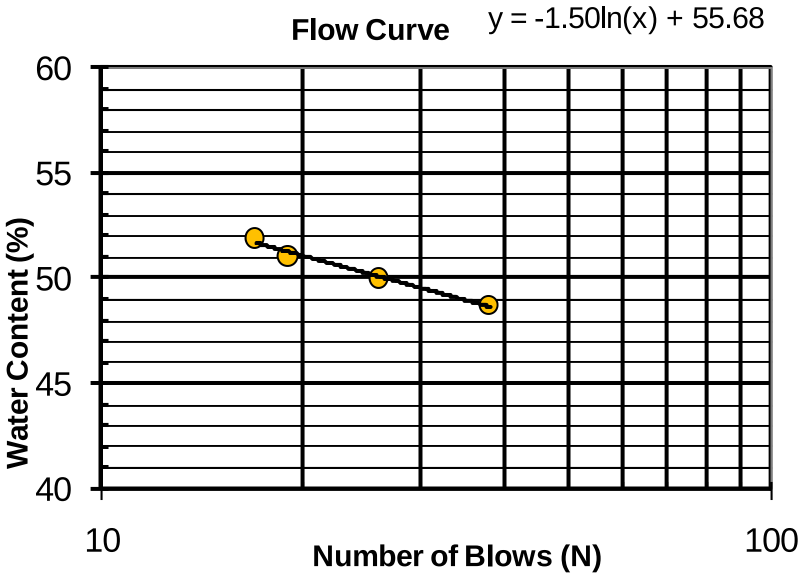


Figure A.17: Flow curve for liquid limit determination of disturbed sample D-2, collected from BH-1 at a depth of 3.0m adjacent to old embankment.

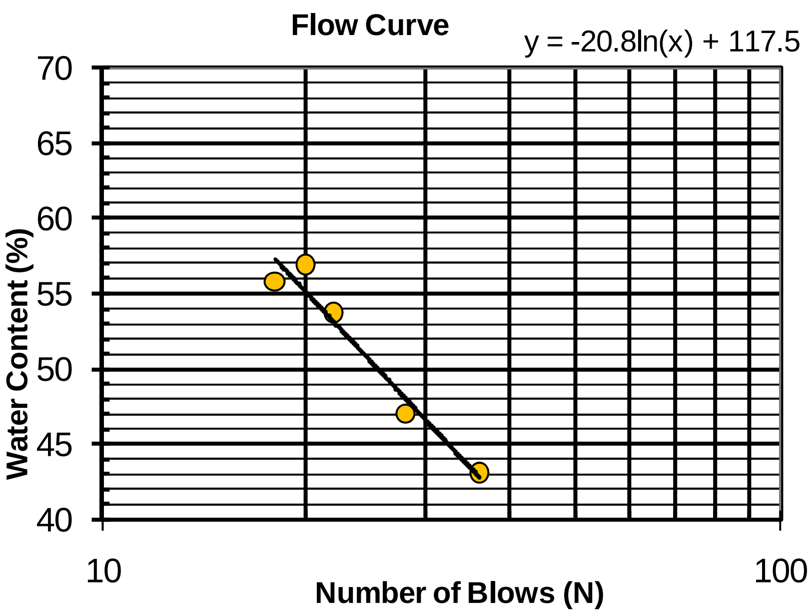


Figure A.18: Flow curve for liquid limit determination of disturbed sample D-3, collected from BH-1 at a depth of 4.5m adjacent to old embankment.

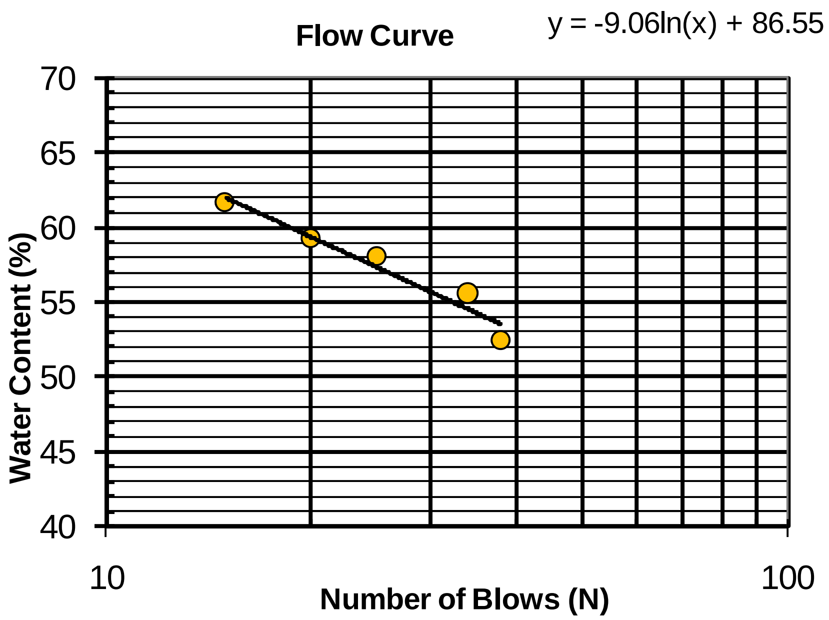


Figure A.19: Flow curve for liquid limit determination of disturbed sample D-4, collected from BH-1 at a depth of 6.0m adjacent to old embankment.

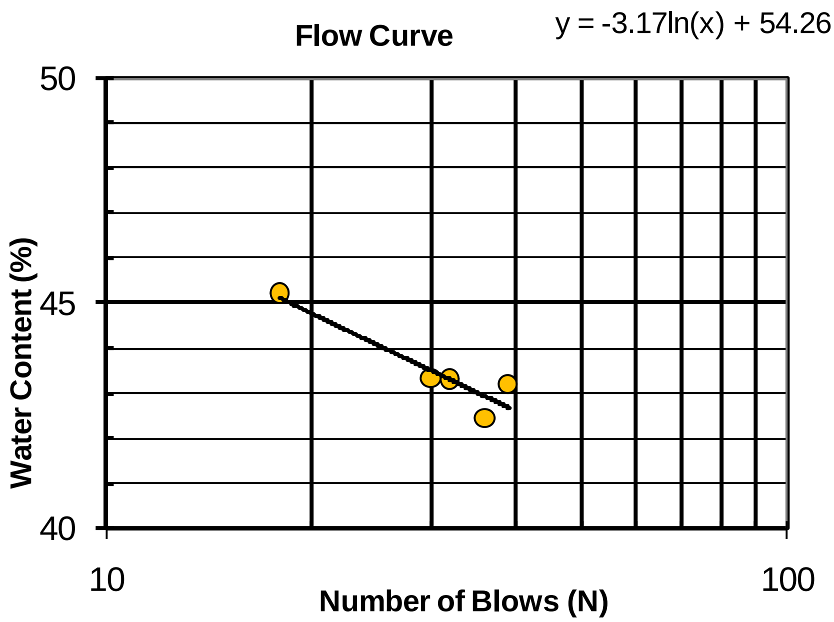


Figure A.20: Flow curve for liquid limit determination of disturbed sample D-5, collected from BH-1 at a depth of 7.5m adjacent to old embankment.

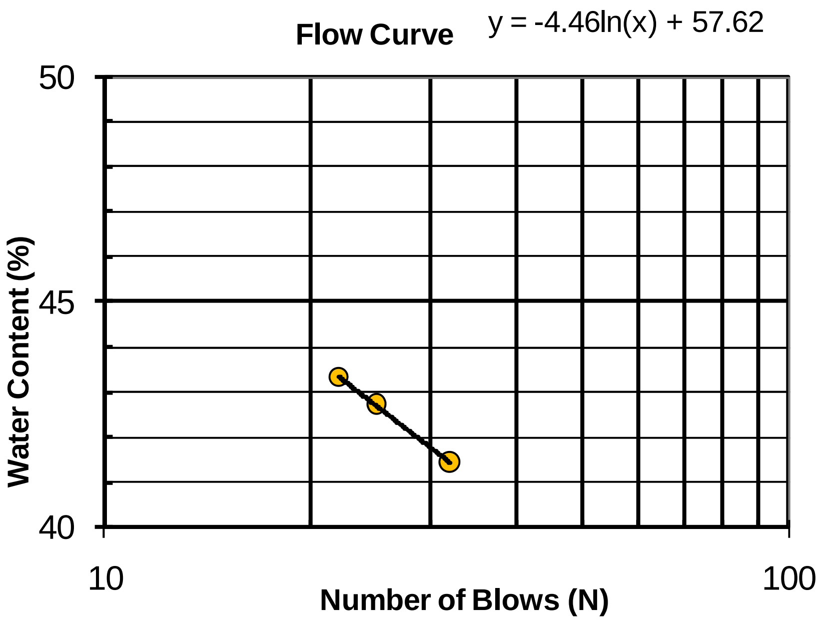


Figure A.21: Flow curve for liquid limit determination of disturbed sample D-7, collected from BH-1 at a depth of 10.5m adjacent to old embankment.

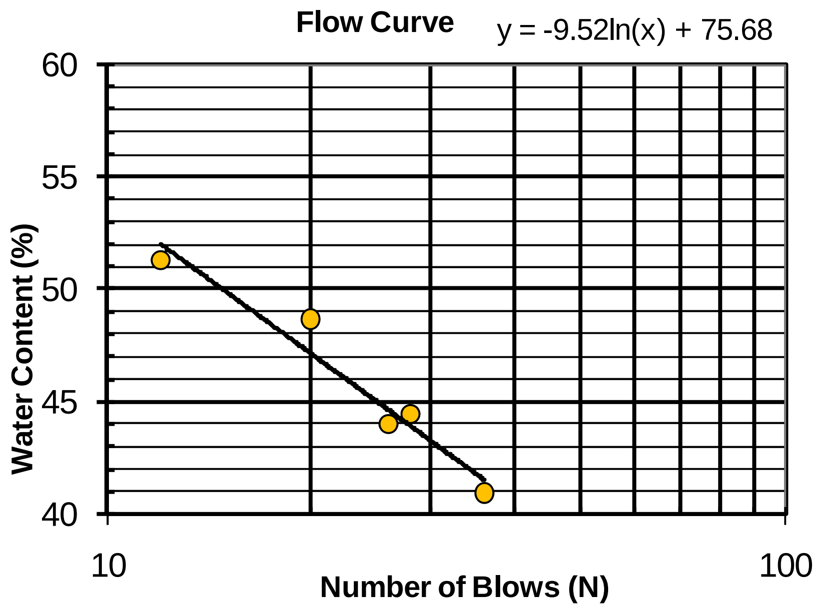


Figure A.22: Flow curve for liquid limit determination of disturbed sample D-2, collected from BH-2 at a depth of 3.0m adjacent to new embankment.

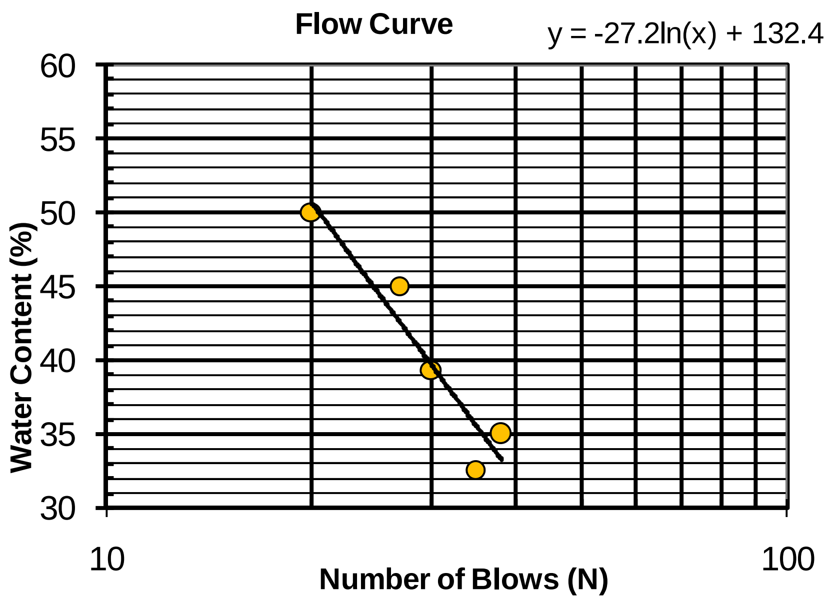


Figure A.23: Flow curve for liquid limit determination of disturbed sample D-3, collected from BH-2 at a depth of 4.5m adjacent to new embankment.

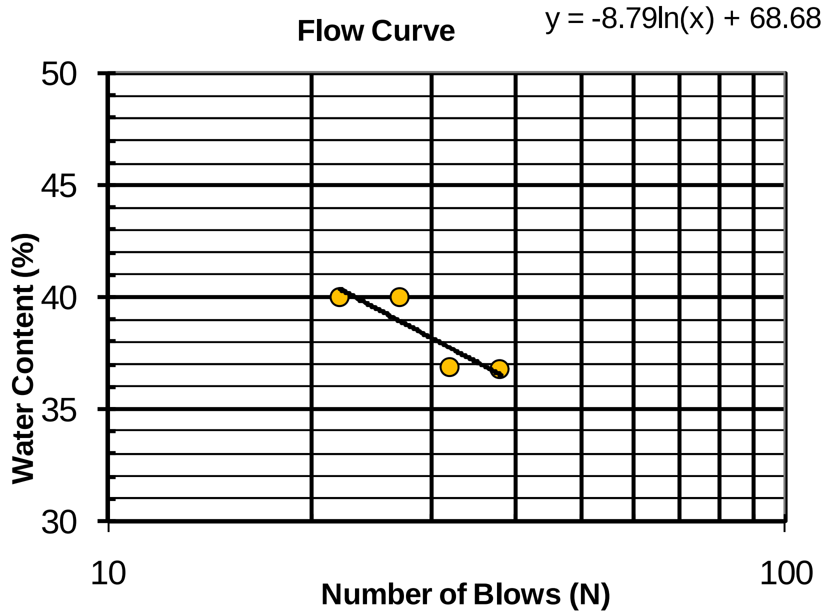


Figure A.24: Flow curve for liquid limit determination of disturbed sample D-4, collected from BH-2 at a depth of 6.0m adjacent to new embankment.