NTNU – Trondheim
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Planning and evaluation of landfill at Nideng in Klæbu

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Geotechnics and Geohazards
Submission date: June 2014
Supervisor: Arnfinn Emdal, BAT

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ABSTRACT: *A landfill is to be established at Nideng in Klæbu which is a municipality within the sør-trøndelag region in Norway. The landfill composed of mass deposit of peat which is characterised by fibrous and organic materials. The topography of the project site is primarily ridges with valleys and stream passing through the site. The quaternary geological map shows mountains and weathering material at the project site as well as thick marine deposit. The site has undergone landslide activities in the past and is believed that the ridges left by the slides consist of clay, sand and gravel. Plans for developing the area have been launched, which means that proper stability evaluations have to be carried out.*

The primary objective of the study is stability evaluation of the peat mass deposit.

The slide software based on limit equilibrium methods and plaxis which is based on finite elements method were employed for the slope stability evaluation.The factor of safety based on undrained conditions estimated by the slide software for profiles A, B, C are 0.25, 0.26 and 0.43 respectively. The calculated safety factor for the undrained condition has not satisfied the minimum safety factor required. The presence of the peat layer which is characterised by high in-situ void ratio, high water content and high values of compression index presents a pre-existing sliding surface causing instability of the peat mass embankment. The existence of the peat layer controls the shape of the failure surface. An employment of a berm composed of gravel with a modification of the actual embankment height and placing of layers of sand material of thickness 1m each in the peat embankment help to increase the factor of safety of the embankment for the undrained case significantly. The factor of safety for profile A increases from 0.25 to 1.2, profile B also increase from 0.26 to 1.1 and profile C also increases from 0.43 to 1.1. The factor of safety based on drained conditions estimated by the slide software for profiles A, B, C are 1.44, 1.53 and 1.51 respectively and the plaxis are also 1.33, 1.44 and 1.66 respectively. Using geogrid with tensile stiffness of EA=50 kN/m with lengths varying from10m to 130m and a spacing of 2m in the peat layer at Profile A for the drained case, the factor of safety increases by almost 21% decreasing the horizontal displacements of the embankment and there by increasing the stability of the peat embankment.

Keywords:

1. landfill
2. stability
3. peat
4. factor of safety

Master Thesis, Spring 2014

Student:

Abdulah Hafizulah Amin Mohammed

Title of project work:

Planning and evaluation of landfill at Nideng in Klæbu

Background
A landfill site is a site for the disposal of waste materials by burial and is the oldest form of waste treatment. Historically, landfills have been the most common method of organized waste disposal and remain so in many places around the world. Some landfills are also used for waste management purposes, such as the temporary storage, consolidation and transfer, or processing of waste material (sorting, treatment, or recycling).
A landfill is to be established at Nideng in Klæbu which is a municipality within the sør- trøndelag region in Norway. The topography of the project site is primarily ridges with valleys and stream passing through the site. The quaternary geological map shows mountains and weathering material at the project site as well as thick marine deposit, which may indicate clay that could potentially be sensitive. The site has undergone landslide activities in the past and is believed that the ridges left by the slides consist of clay or sand / gravel. At this stage it is difficult to infer whether it is possible to establish a landfill in this area. Uncertainties are greatest where there are large hoydeforskjeller and greatest chance for quick clay deposits.

Project description
The main task of the project is based on the following components: development of a baseline for landfill planning, subsurface investigations, geotechnical assessment (stability evaluation), hydrogeological characterisations and developing conceptual landfill.

Report

The product of the project should be a scientific report in English stating scope, purpose, methods, results and references. It should conclude with the findings of the study and may propose areas of further research. The report is to be handed in no later than June 10th, 2014, as two bounded originals and one electronic PDF-file.

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Ass. Professor Arnfinn Emdal
NTNU

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I would like to express my gratitude towards my parents and my big sister and her husband for their unflinching and immense support throughout my life. Without their support, it would have been impossible for me to complete my college and graduate education seamlessly.

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My final appreciation goes to the Lord God Almighty for his care and protection throughout this project work.

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CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND OF STUDY

Landfills are the final repositories for unwanted or unusable wastes. Until the middle of this century, nearly all wastes were discarded in open, unengineered dumps. Topographical anomalies that lended themselves naturally to dumping were typically selected for dump sites. The most common waste dumps were natural depressions (creeks, low-lying areas, and flood plains) that were otherwise of little use and mined-out areas, e.g., sand or gravel quarries [1].

The design concept for a landfill depends on the ground conditions, the geology and hydrogeology of the site, the potential environmental impacts and the location of the landfill. The investigations for a landfill should provide sufficient information to enable the formulation of a site specific design. Landfill practice is dynamic in that it will change with both advances in technology and changes in legislation. To incorporate such advances and changes a periodic review of the design should be carried out, as the lifespan of a landfill site from commencement to completion is long compared to other construction projects [19].

A landfill is to be established at Nideng in Klæbu which is a municipality within the sør-trøndelag region in Norway. The landfill is composed of mass deposit of peat which is characterised by fibrous and organic materials. The topography of the project site is primarily ridges with valleys and stream passing through the site. The quaternary geological map shows mountains and weathering material at the project site as well as thick marine deposit. The site has undergone landslide activities in the past and is believed that the ridges left by the slides consist of clay or sand / gravel. Plans for developing the area have been launched, which means that proper stability evaluations have to be carried out.

The main task of the project is based on the following components: development of a baseline for landfill planning, subsurface investigations, geotechnical assessment (stability evaluations), hydrogeological characterisations and developing conceptual landfill.

1.2 SITE LOCATION AND DESCRIPTION

The landfill site is located at Nideng in Klæbu which is a municipality within Sør- Trøndelag region. The site is approximately 11km from Trondheim. The site is approximately located on latitude of 63°19′9.42″N and longitude of 10°25′5.87″E.

The eastern portion of the landfill site indicates a valley with steep slopes and the western part of the landfill is located in an area with several small valleys.

Figure 1.1: Location map of the project site [2].

1.3 VEGETATION AND DRAINAGE

The vegetation is mainly woodland with grass and well drain by streams and rivers.

1.4 SOIL

The soil types at the landfill site are dominated by weathering material (decomposed sedimentary rocks) as well as thick marine deposit which may indicate clay that could potentially be sensitive. It is believed, however, that the ridges left by previous slides consist of clay or sand / gravel.

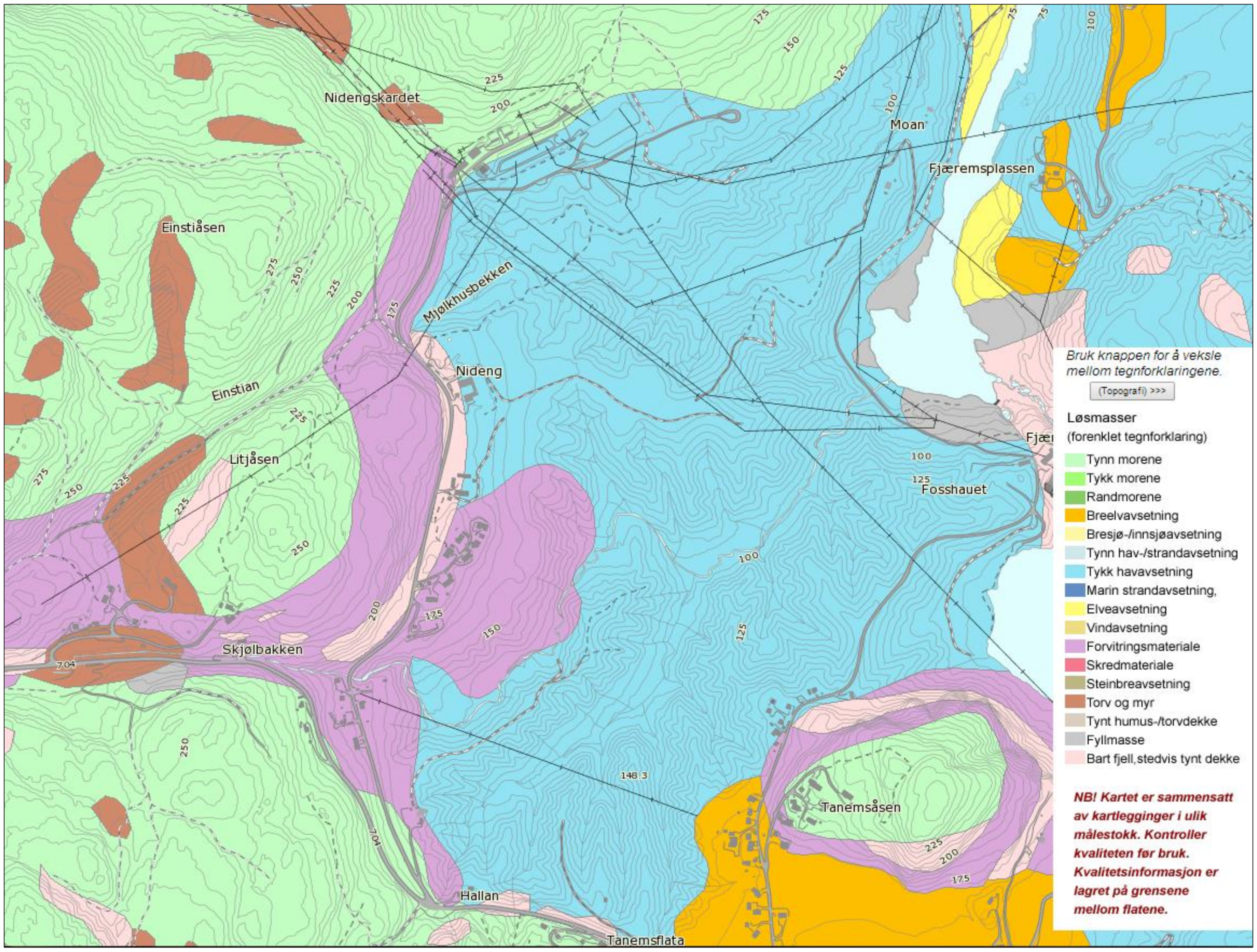


Figure 1.2: Map showing the soil types at the project site [3].

1.5 Geology

Geologically, Norway is a typical hard rock province, forming part of the so-called “Baltic Precambrian Shield”. About 2/3 of the bedrock is Precambrian, and the remaining 1/3 is Paleozoic (mainly Cambro-Silurian, also often referred to as Caledonian), i.e. more than 250 million years old. In the Precambrian areas gneisses and granites are most frequent, but rock types such as gabbro, amphibolite, quartzite and sandstone are also found. The Cambro-Silurian province consists mainly of rock types like mica schist, phyllite, marble and greenstone. Due to several epochs of orogeny, with the Caledonian as the latest, most bedrock in Scandinavia are highly metamorphosed [5].

The Nideng Township where the landfill is to be situated is dominated by sedimentary rocks composed mainly of sandstone, shale, limestone, conglomerate and breccia.

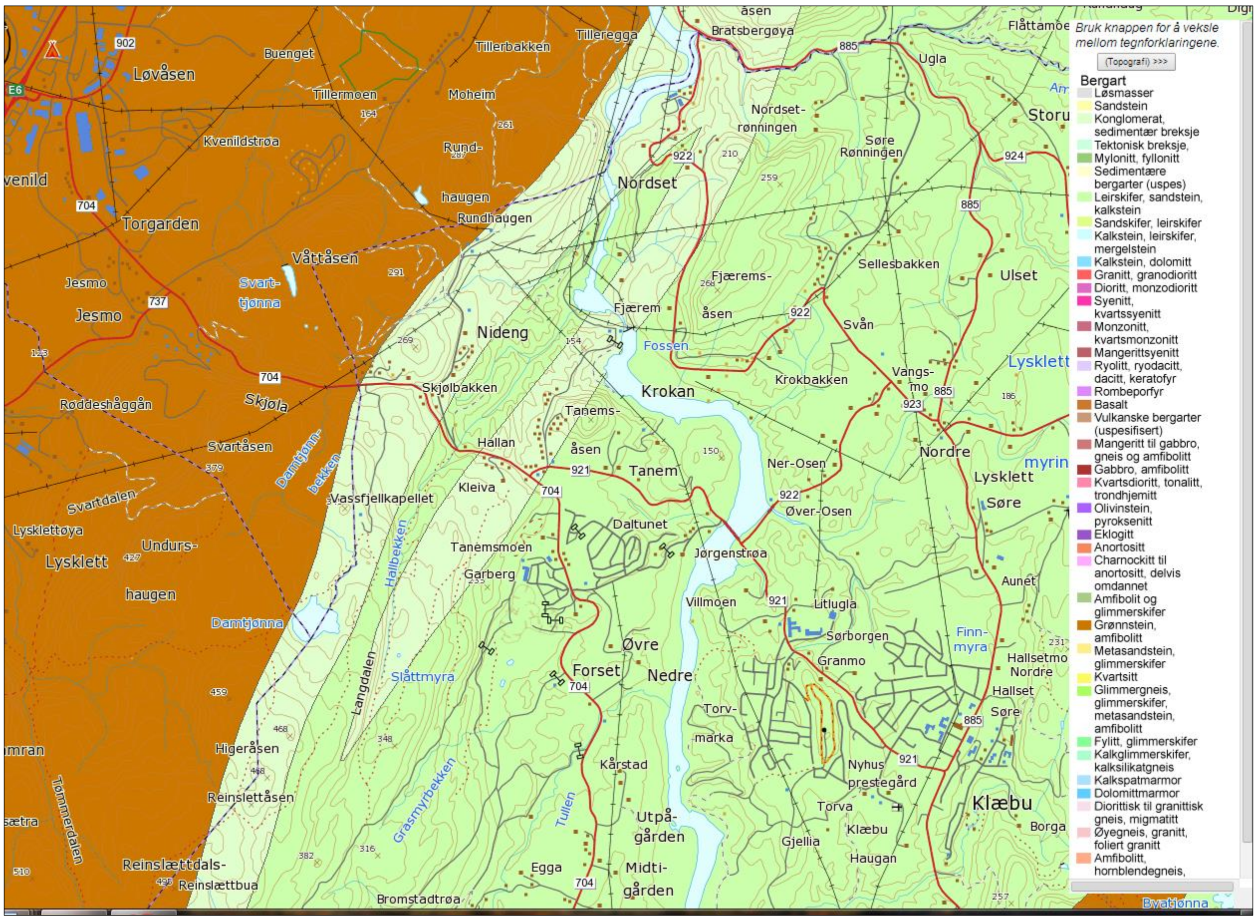


Figure 1.3: Map showing the geology of the project site (Nideng) [4].

CHAPTER TWO

LANDFILL DESIGN

2.1 SOLID WASTE MANAGEMENT

Solid waste management may be defined as the discipline associated with the control of generation, storage, collection, transfer and transport, processing and disposal of solid wastes. Integrated solid waste management includes the selection and application of suitable techniques, technologies and management programs to achieve specific waste management objectives and goals [6]. Current solid waste management technologies can be summarized as:

- Source reduction
- Recycling
- Waste transformation
- Landfilling

2.1.1. Source Reduction

It involves diminishing waste amount, volume and toxicity at the source of waste generation [7]. Source reduction is the most effective way which reduces the quantity of waste, the cost of associated with its handling, and its environmental impacts. Waste reduction may occur through the design, manufacture, and packaging of products with minimum toxic content, minimum volume of material, or a longer life and also at the household, commercial, or industrial facility through selective buying patterns and the reuse of products and material [8].

2.1.2. Recycling

It involves the separation and collection of waste materials; the preparation of these materials for reuse, reprocessing, and remanufacture; and the reuse, reprocessing, and remanufacture of these materials. Recycling is an important factor in helping to reduce the demand on resources and the amount of waste require disposal by landfilling [8]. Reusing waste products can be simply made by the public by returning drink containers to bottling manufacturers and the donation of used clothes, shoes, furniture, and electrical products to charities and retailers. Product recycling primarily involves melting glass and metals, pulping of paper waste so that the end product is useful as a raw material to manufacturers. Benefits of waste recovery include conserving finite resources, lowering the need for mining or harvesting virgin material, reducing inert residues from incinerators, and fewer demands on landfills [7].

2.1.3. Waste transformation

It involves the physical, chemical, or biological alteration of wastes. Typically, the physical, chemical, and biological transformations that can be applied to municipal solid wastes are; to improve the efficiency of solid waste management operations and system, to recover reusable and recyclable materials, and to recover conversion products and energy in the form of heat and combustible biogas. The transformation of waste materials usually results in the reduced use of landfill capacity [8]. Transformation examples include mechanical clipping, shredding, and grinding, thermal combustion, and composting organic food and yard waste [7]. A benefit of thermal incineration is the potential for energy generation while reducing waste volume up to 90% [8].

2.1.4. Landfilling

It is the process by which the solid wastes that cannot be recycled nor further used; the residual matter remaining after the recovery facility and after the recovery of conversion products and energy is placed in a landfill. Although there is a public opposition to landfills, it is necessary and there is no combination of waste management technique that does not require landfilling to make them work. Landfilling includes monitoring of the incoming waste stream, placement and the compaction of waste, and installation of landfill environmental monitoring and control facilities.

2.2 SANITARY LANDFILL

Sanitary landfills are designed to protect humans and the environment from harmful gases and fluids by using methane collection vents and leachate liners and collection pipes. A modern, well-constructed landfill can be characterized as an engineering structure that consists primarily of a liner, leachate collection and removal system, gas collection and control system and final cover. Many landfills are designed for 20 or 30 year life span and still require post closure monitoring up to 30 years to ensure the environmental health.

The landfill is usually double-lined to trap leachate. Synthetic liners include plastic geomembranes, geomats, geogrids, and geotextiles that commonly contain bentonite clays [8]. In a sanitary landfill, waste is contained in a cell which is covered with a layer of soil and compacted at the end of each working day. The dimensions of the cell depend on the volume of waste received and the availability of cover material. The cell thickness may range from 8 to 30 ft (2.4 – 9.1 m) but typically it is 15 ft (4.6m). The usual slope of the working face is 3 horizontal to 1 vertical (3:1) which allows reasonable compaction and easier capping and vegetative growth on the side slopes of the landfill. The width of the working face is usually limited to 2 ft (0.6 m). The first lift of the waste is usually 5 ft (1.5 m) or less with careful removal of the oversize pieces to prevent damage of the underlying leachate collection system. The compaction equipment moves from the bottom to the top of the working face. The thickness of the daily cover is 6-12 in (150- 300 mm).If a lift surface is expected to be exposed over 30 days then an intermediate cover is applied. The intermediate cover is typically 1 ft thick and more resistant to erosion than the daily cover [10]. When the landfill's operational life has ended, a final layer of soil and optimal synthetic liners are added along with a vegetative cover to limit percolation and erosion.

The steps involves in the construction of landfill are:

- Landfill footprint layout
- Sub-base grading
- Cell layout and filling
- Temporary cover selection
- Final cover grading
- Final cover selection

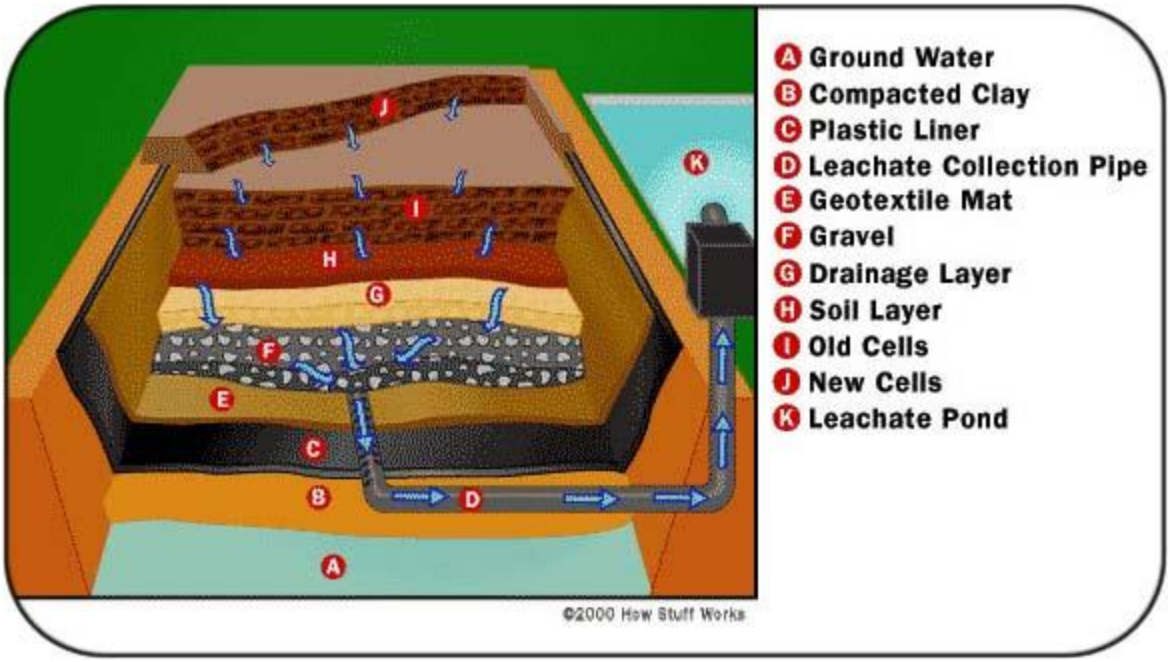


Figure 2.1: Cross-section of modern Sanitary Landfill [11].

2.3 CONSTRUCTION METHODS FOR A SANITARY LANDFILL

The construction method and subsequent operation of a sanitary landfill are mainly determined by the topography of the terrain, although they also depend on the type of soil and the depth of the water table. There are two basic ways of making a sanitary landfill [13].

2.3.1 Trench method

This method is used in flat regions and consists of periodically digging trenches two or three meters deep with a backhoe or a track-type tractor. Some trenches have been dug as deep as 7 m. The solid waste is placed and spread in the trench, later to be compacted and covered with the excavated soil.

Special care should be taken during rainy periods, since water can flood the trenches. To prevent this, drainage ditches should be dug around the perimeter to divert the waters, and internal drainage can also be provided for the trenches. In extreme cases a roof can be erected over them, or the accumulated water can be pumped out. The slopes or walls should be cut corresponding to the settling angle of the excavated soil.

The digging of trenches demands favorable conditions with regard to the depth of the water table as well as to the type of soil. Terrain with a high water table or one close to the surface is not appropriate because of the risk of contamination of the aquifer. Rocky terrain is not suitable either, because it is difficult to dig [13].

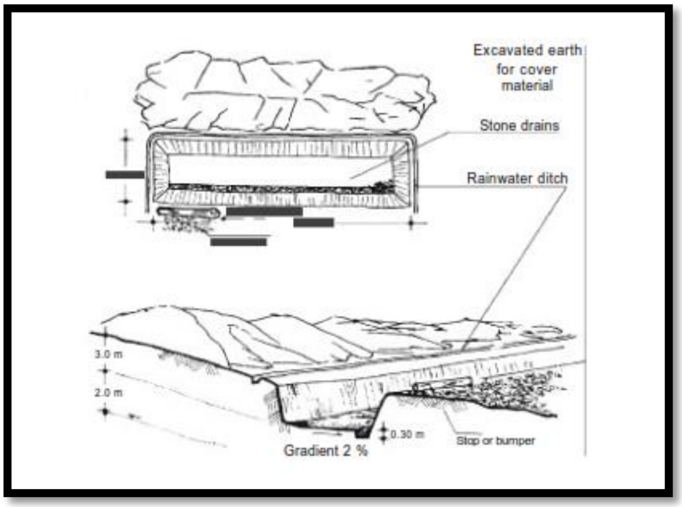


Figure 2.2: Trench method for making a sanitary landfill [13].

2.3.2 Area Method

In relatively flat areas where it may not be feasible to dig pits or trenches to bury the waste, it can be deposited directly on the original ground, which should be raised several meters after the terrain has been made waterproof. In these cases the cover material will have to be brought from other places or, if possible, extracted from the surface layer. The pits are made with a gentle slope to prevent landslides and ensure greater stability as the landfill rises [13].

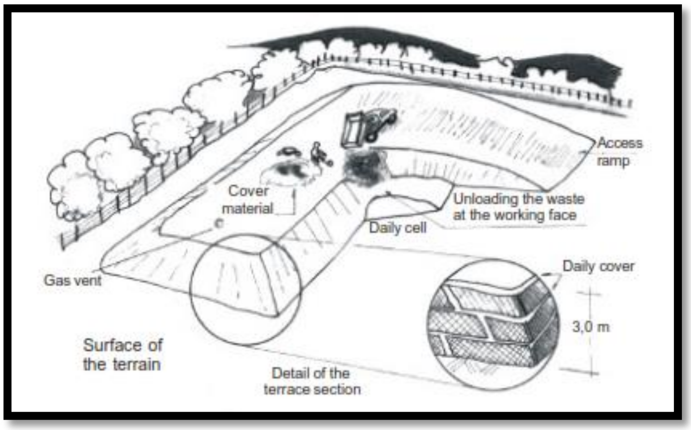


Figure 2.3: Area method for making a sanitary landfill [13].

2.4 LANDFILL SITE SELECTION

The major goal of the landfill site selection process is to ensure that the disposal facility is located at the best location possible with little negative impact to the environment or to the population. For a sanitary landfill siting, a substantial evaluation process is needed to identify the best available disposal location which meets the requirements of government regulations and best minimizes economic, environmental, health, and social costs. Evaluation processes or methodologies are structured to make the best use of available information and to ensure that the results obtained are reproducible so that outcomes can be verified and defended [14].

2.4.1. Criteria for Landfill Siting

There are a number of criteria for landfill site selection. These are environmental criteria, political criteria, financial and economic criteria, hydrologic and hydrogeological criteria, topographical criteria, geological criteria, availability of construction material and other criteria. Each criterion will be discussed briefly in the next sections.

2.4.1.1. Environmental Criteria

2.4.1.1.1. Ecological value of the flora and fauna

The direct and indirect spatial use of a landfill will destroy the actual vegetation and fauna. When making a decision, the ecological value of the actual vegetation and fauna should be evaluated carefully for the candidate area. Ecological value is based on diversity, naturalness and characteristic feature. An example of indirect use is the disturbance of the quietness in the surroundings caused by the activities on the landfill.

2.4.1.1.2. Odour and dust nuisance

A new landfill should not be located within a distance of a housing area because of the dust and odour emissions. Dependent of the local wind direction and speed, the safe distance necessary to locate a landfill site should be determined to prevent sensing dust and odour. The problems of odour and dust can also be minimized by proper soil cover.

2.4.1.1.3. Nuisance by traffic generation

A new landfill will generate more traffic. How much more traffic depends of the distance to the collection area, the kind of transport and the use of transfer stations. Access roads passing through housing areas will cause more nuisance than access roads through the open country side.

So, routing vehicle traffic through industrial, commercial or low density population areas decreases the noise impacts of landfill related vehicles.

2.4.1.1.4. Risks for explosion or fire

Because of the presence of landfill gas, there is a chance for explosion and/or fire. Soil cover also functions to smother fires and to form a barrier preventing the spreading of fires. Proper policing of incoming trucks can further reduce fire risk by minimizing the dumping of flammable loads [15].

2.4.1.1.5. Other nuisance for neighbouring area

Other nuisance includes vermin that is attracted by the organic parts of the waste on the landfill (rats, mice, birds, insects), windblown litter, noise caused by construction, compaction or trucks on the landfill. The daily cover is a solution for nuisance developed by the presence of vermin. Continuous grading of soil cover to fill in low spots is essential to prevent the development of stagnant pools of water in which mosquitoes can breed.

2.4.1.2. Political Criteria

2.4.1.2.1. Acceptance by the local municipalities

The political acceptance of a new landfill location can differ in each region and sometimes the potential sites are located in different regions. The level of political acceptance has influence on the willingness of the local municipalities to make their regional physical plans and to give permission for the construction of a landfill. The unwillingness will cause to a delay of the decision on the landfill location.

2.4.1.2.2. Acceptance by the pressure groups involved

The acceptance by the public of a landfill in their own region or municipality is an important factor in the decision making process. The so-called NIMBY (not in my backyard) syndrome is becoming a common attitude. The influence of the public is significant if there are local groups which are well organized and having good relations with the local authorities and the media (papers, radio and television). The level of the public acceptance can be measured how far the local pressure groups are succeeding to delay the decision making process.

2.4.1.2.3. Property of the landfill area

The ownership of the needed land for the landfill is very important. Public ownership is easier than private ownership because the private ownership will give problems with the cost of the land. Sometimes, expropriation is needed and this procedure will cause delays.

2.4.1.3. Financial and Economical Criteria

2.4.1.3.1. Costs of land

Costs of the land depend on the land prices which can differ for each location. The actual use of the land is important for the price which influences the level of compensation for the owner or actual users. The potential landfill with the lowest costs is more preferable.

2.4.1.3.2. Costs for the access of the landfill

Costs for the access of the landfill depend on the condition and the presence of roads close to the landfill. If reconstruction of actual roads is needed, the costs will increase. Because of that road network is an important factor to locate a landfill site.

2.4.1.3.3. Transport costs

Transport costs are determined by the transport distances from the source of waste generation, the way of transport and the way of collection. The other factors affecting transport costs are the need for waste transfer stations and the possibility to use railways.

2.4.1.3.4. Costs for personnel, maintenance and environmental protection

The costs for personnel will not differ so much between the different potential landfill sites. Maintenance depends on the availability of soil needed for the daily or regular covering and for the stability of the landfill. If the soil is not available in the area, it should be imported which increases the maintenance costs. Extra technical provisions should be placed to prevent the pollution of the soil, groundwater and surface water at the landfill. Monitoring the drainage system and the quality of the leachate and surface water are also important factors in the maintenance costs. The potential landfill with the lowest maintenance costs is more suitable for a landfill.

2.4.1.3.5. Costs for the after-care

The costs for after-care is not only dependent on the kind of final use but also on provisions to monitor the groundwater quality, existence of gas, the winning of gas, the stability of the completed landfill. Needed provisions are depending on the characteristics of the filled waste, the kind of subsoil, the hydrogeological situation, and the kind of final use.

2.4.1.4. Hydrologic/Hydrogeological Criteria

2.4.1.4.1. Surface water

The landfill site should not be placed within surface water or water resources protection areas to protect surface water from contamination by leachate. Safe distances from meandering and non-meandering rivers should be achieved to prevent waste from eroding into rivers and major streams. A landfill should not be located within 100 feet (30.48 m) of any non-meandering stream or river, and at least 300 feet (91.44 m) from any meandering stream or river. Large ponds, lakes, and reservoirs should have a buffer zone of land to prevent blown debris and runoff from harming aquatic habitats. Large bodies of water (greater than 20 acres (80937.45 m^2) of surface area) should be at least 100 feet (30.48m) from any landfill site. If the regional drinking water is supplied by surface water impoundments, it may be necessary to exclude the entire watershed that drains into the reservoir from landfill sites [16].

In case of a high velocity of the surface flow there will be more dilution of an eventually contamination. The potential landfill location with the highest velocity of the overland flow will get the highest ranking score.

The major concern of siting landfills within floodplains is the downstream effect from waste carried away during episodes of higher water levels. Since major rivers have a higher discharge and greater downstream influence, no landfill should be sited within the floodplains of major rivers [16]. The construction of a landfill within the 100-year flood stage of a minor river or stream is not safe.

2.4.1.4.2. Groundwater

To protect subsurface drinking water, landfills should not be situated over high quality groundwater resources. Fresh groundwater (total dissolved solids>1000mg/l) should be avoided or protected with a compound liner system and monitoring wells [16]. Since potential leachate leaks will travel down gradient, landfills should be placed greater than 304.8m (1000 feet) up-gradient from water wells. Aquifer depths less than 15.24m (50 feet) should be considered less suitable than sites with a depth-to-groundwater of 60.96 m (50 to 200 feet) [17]. Table 2.1 and Table 2.2 show landfill suitability based on depth to groundwater and amount of dissolved solids.

Table 2.1: Groundwater depth and landfill suitability [17].

Depth to Groundwater	Suitability
Over 60m (200ft)	High
15 to 60 m	Moderate
Under 15m (50ft)	Low

Table 2.2: Groundwater quality and landfill suitability [16].

Groundwater Quality (TDS in mg/l)	Suitability
Over 10000	High
1000 to 10000	Moderate
Under 1000	Low

A high velocity of the groundwater flow is increasing the spreading of eventually leachate beneath the landfill. The velocity of the groundwater flow is dependent of porosity of the soil and the filtering speed. The potential landfill location with the lowest velocity of the groundwater flow is more suitable for a landfill.

A high groundwater level or a nearby high river level will cause more risk to pollute the groundwater or river water. The potential landfill location with the lowest groundwater or river level is more suitable for a landfill. Impermeable layers in the subsoil are minimizing the risk of polluting the groundwater. Especially clay layers have a low permeability. The location with subsoil layers which have a high impermeability is more preferable to locate a landfill.

2.4.1.5. Topographical Criteria

The topography of an area is an important factor on site selection, structural integrity, and the flow of fluids surrounding a landfill site because it has important implications for landfill capacity, drainage, ultimate land use, surface and groundwater pollution control, site access and related operations [15].

2.4.1.6 Geological Criteria

The geology of an area will directly control the soil types created from the parent material, loading bearing capacity of the landfill's foundation soil, and the migration of leachate. Rock and its structure type will determine the nature of soils and the permeability of the bedrock. Geologic structure will influence the movement of leachate and potential rock-slope failure along joints and tilted bedding planes.

Comparing extreme permeability rates, unfractured crystalline rocks will transmit little (if any) fluids whereas poorly cemented sandstones will allow rapid transport of fluids. Due to higher permeability rates, sandstone is less suitable as landfill bedrock than other sedimentary rocks such as limestone and shale. Limestone are more suitable than shale due to susceptibility of the carbonate rocks to dissolution from low pH leachate, and are commonly associated with discontinuities and karst features such as collapses, sinkholes, and caverns. Shale formations are well suited for landfill sites since shales commonly act as a retarding bed slowing or confining the transmission of fluids.

Table 2.3 summarizes some of the various rock types of suitability for landfill siting.

Table 2.3: Landfill suitability of bedrock [10]

Rock Type	Suitability
Unfractured crystalline	Very high
Shale and clay	High
Limestone	Fair to poor
Sandstone	Poor to very poor
Unconsolidated sand/gravel	Unsuitable

The structure and orientation of discontinuity planes will have a direct impact on the movement of leachate and on the structural integrity of the bedrock material. Sites composed of tilted rocks greater than 45 degree dip have the potential for rock-slope failure along discontinuities and should be considered an unstable area. Leachate flow will follow down-dip directions. To limit the spread of leachate, landfills should not be situated on the axis of anticlines and structural domes. In addition to the spreading of landfill leachate, anticlines and domes are often associated with oil and natural gas fields and should be avoided. In contrast, synclines and structural basins are the best sites for leachate to pool into [18]. Regions that are faulted are not suitable for landfill because a fault can act as a conduit of leachate transport and can reduce the structural integrity of bedrock supporting the landfill and its equipment.

The sanitary landfill should be located preferably on a terrain of sandy-silty-clayey soils (loamy coarse sand, predominantly clayey loam); also suitable are silty-clayey soils (heavy predominantly silty, predominantly silty clayey, light clayey silty) and clayey-silty ones (heavy clayey silty and clayey).

2.5 LINING SYSTEMS

2.5.1 Functions of a lining system

The lining system protects the surrounding environment including soil, groundwater and surface water by containing leachate generated within the landfill, controlling ingress of groundwater, and assisting in the control of the migration of landfill gas. The selected liner system must achieve consistent performance and be compatible with the expected leachate for the design life of the facility [19].

2.5.2 Requirements of liner systems

The following sections list options for liner systems for non-hazardous, hazardous and inert landfills. Figure 2.5 illustrates the minimum requirements for each landfill type.

2.5.2.1 Hazardous waste landfill

At minimum a composite liner should be used for hazardous waste landfill facilities. Two options are presented that may be used. The option to be used is dependent on the nature of the waste materials being deposited. Alternative systems may be considered for pre-treated hazardous wastes, example solidification, stabilisation and vitrification of hazardous wastes.

Option 1: Single Composite Liner

The liner system should consist of the following:

- A minimum 0.5m thick leachate collection layer having a minimum hydraulic conductivity of $1x10^{-3}$ m/s;

- The upper component of the composite liner must consist of a flexible membrane liner. At minimum a 2mm HDPE or equivalent flexible membrane liner should be used, as it is sufficiently robust but at the same time not prone to excessive cracking and construction difficulties;
- Base and side wall mineral layer of minimum thickness 5m having a hydraulic conductivity less than or equal to $1x10^{-9}$m/s; and
- A minimum 1.5m of the 5m thick mineral layer should form the lower component of the composite liner and should be constructed in a series of compacted lifts no thicker than 250mm when compacted.

Option 2: Double Composite Liner

This system has two composite liners on top of each other with a leachate detection system between each layer. It should consist of the following:

- A minimum 0.5m leachate collection layer having a minimum hydraulic conductivity of $1x10^{-3}$ m/s;
- Top composite liner consisting of a minimum 2mm HDPE or equivalent flexible membrane liner; and a 1m thick layer of compacted soil with a hydraulic conductivity less than or equal to $1x10^{-9}$ m/s constructed in a series of compacted lifts no thicker than 250mm when compacted or a 0.5m artificial layer of enhanced soil or similar giving equivalent protection to the foregoing also constructed in a series of compacted lifts no thicker than 250mm when compacted;
- A minimum 0.5m thick leachate detection layer having a minimum hydraulic conductivity of $1x10^{-3}$ m/s or a geosynthetic material that provides equivalent performance; and
- Bottom composite liner consisting of a minimum 2mm HDPE or equivalent flexible membrane liner upper component; base and side wall mineral layer of minimum thickness 4m having a hydraulic conductivity less than or equal to $1x10^{-9}$m/s; and a minimum 1m of the 4m thick mineral layer should form the lower component of the composite liner and should be constructed in a series of compacted lifts no thicker than 250mm when compacted [19].

2.5.2.2 Non-hazardous biodegradable waste landfill

For all non-hazardous waste landfills at minimum a composite liner system should be used.

The liner system should at minimum consist of the following components:

- A minimum 0.5m thick leachate collection layer having a minimum hydraulic conductivity of $1x10^{-3}$ m/s;
- The upper component of the composite liner must consist of a flexible membrane liner. At minimum a 2mm HDPE or equivalent flexible membrane liner should be used; and
- The lower component of the composite liner must consist of a 1m layer of compacted soil with a hydraulic conductivity of less than or equal to $1x10^{-9}$m/s constructed in a series of compacted lifts no thicker than 250mm when compacted or a 0.5m artificial layer of enhanced soil or similar giving equivalent protection to the foregoing also constructed in a series of compacted lifts no thicker than 250mm when compacted [19].

2.5.2.3 Inert waste landfill

The liner system for an inert landfill should at minimum meet the following requirements:

- Base and side wall mineral layer of minimum thickness 1m with a hydraulic conductivity less than or equal to $1x10^{-7}$m/s or a 0.5m artificial layer of enhanced soil or similar giving equivalent protection to the foregoing [19].

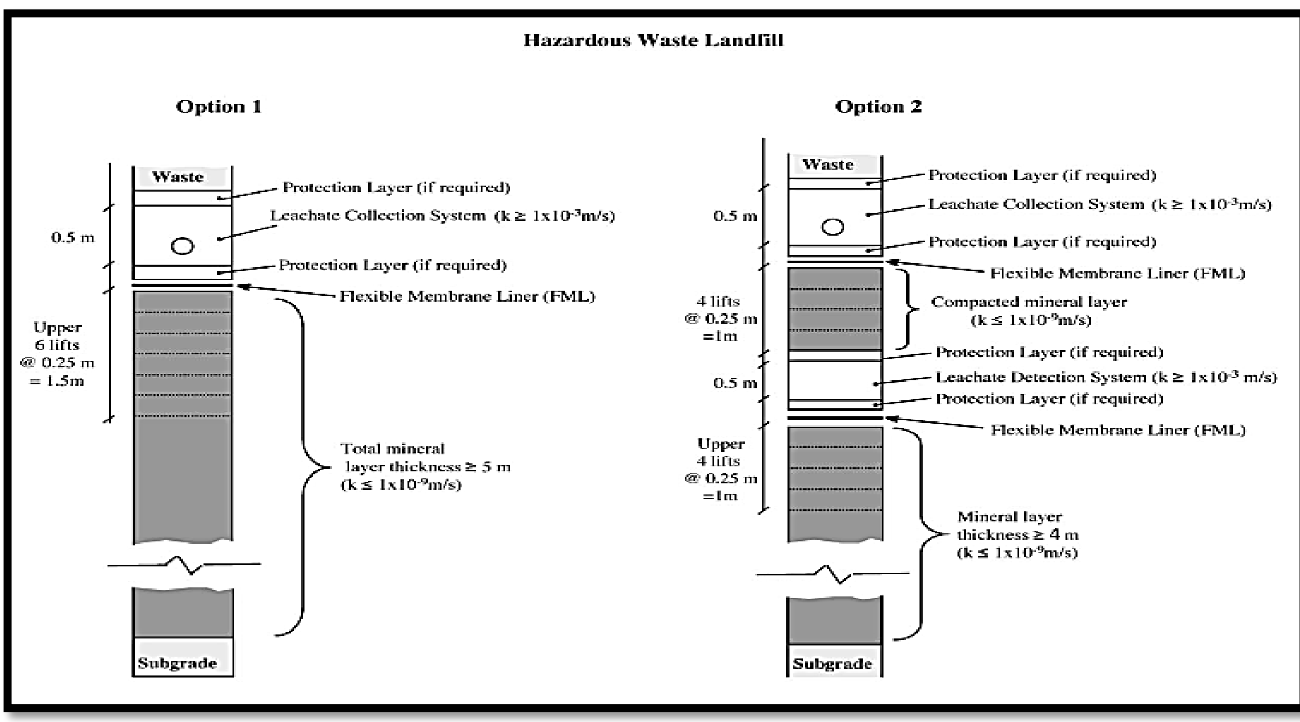


Figure 2.4 (A): Lining system for hazardous waste landfill [19].

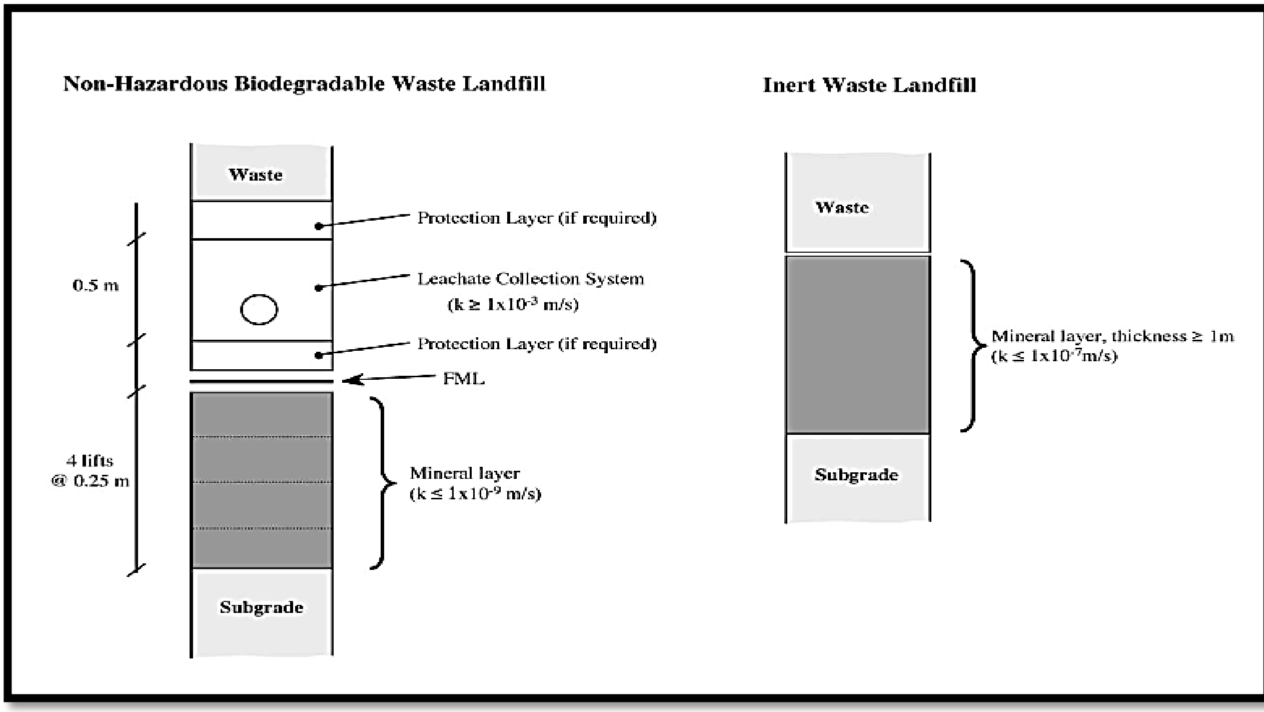


Figure 2.4 (B): Lining system for Non-hazardous waste and inert waste landfill [19].

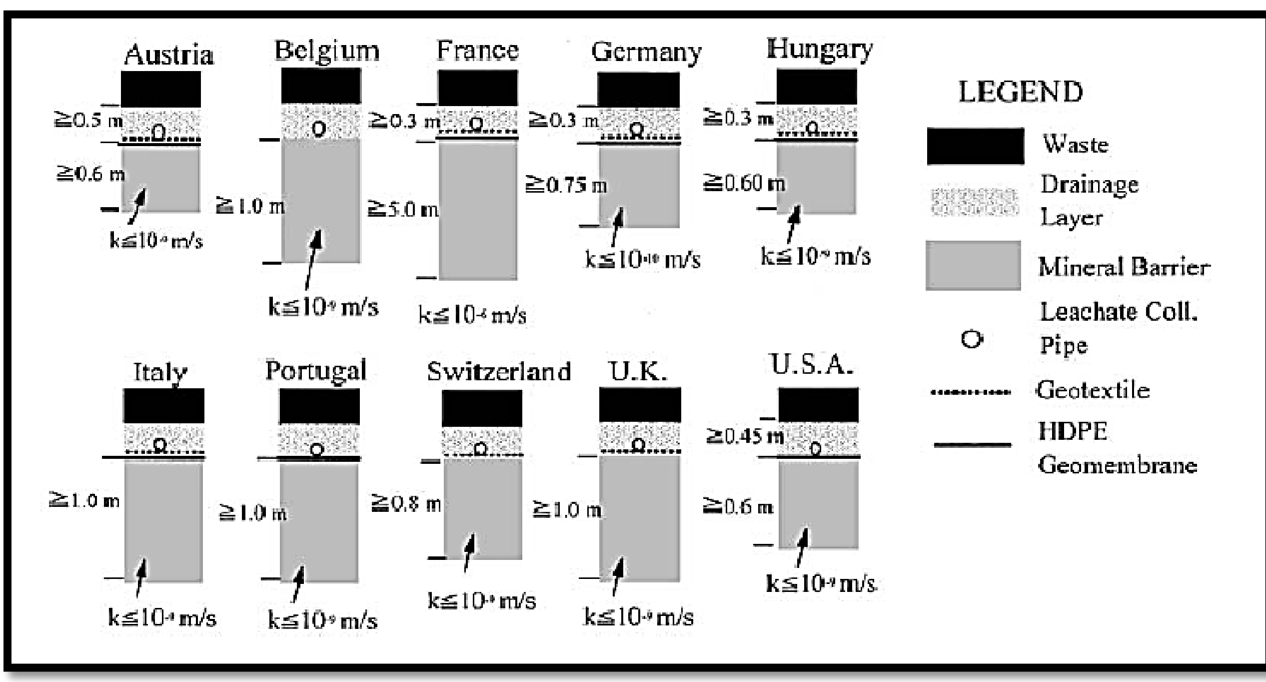


Figure 2.5: European and American bottom liner systems for municipal solid waste [20].

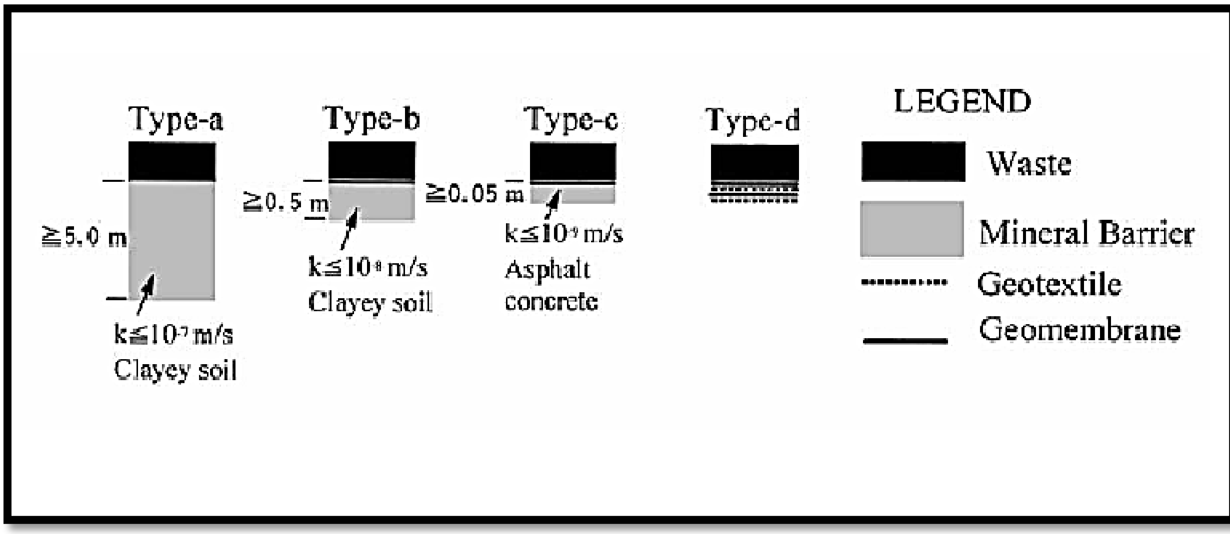


Figure 2.6: Japanese bottom liner systems [20].

2.5.2 Soil permeability

The greater or lesser ease with which water seeps through a soil. The permeability coefficient (k) is an indicator of the greater or lesser difficulty with which a soil resists seepage of water through its pores. In other words, it is the speed with which the water crosses different types of soil. To illustrate these parameters better, we present Figure 2.7, which shows the type of soil and its relation to the permeability coefficient.

K (cm/s)	10^2	10^1	$10^0=1$	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
K (ft/day)	10^5	10,000	1,000	100	10	1	0.1	0.01	0.001	0.0001	10^{-5}	10^{-6}	10^{-7}
Relative Permeability	Pervious				Semi-Pervious			Impervious					
Aquifer	Good					Poor				None			
Unconsolidated Sand & Gravel	Well Sorted Gravel		Well Sorted Sand or Sand & Gravel			Very Fine Sand, Silt, Loess, Loam							
Unconsolidated Clay & Organic					Peat		Layered Clay			Fat / Unweathered Clay			
Consolidated Rocks	Highly Fractured Rocks				Oil Reservoir Rocks			Fresh Sandstone		Fresh Limestone, Dolomite		Fresh Granite	

Figure 2.7: Permeability coefficient k (cm/s) (Logarithmic Scale) [13].

CHAPTER THREE

STABILITY OF LANDFILL

3.1 INTRODUCTION

The stability of landfills is controlled in broad terms by the following factors:

- the properties of the supporting soil;
- the strength characteristics and weight of the fill material;
- inclination of the slope;
- leachate levels and movements within the landfill;
- type of cover; and
- cover resistance to erosion.

In all cases, the presence of water acts as a destablising agent in reducing the strength and increasing the destablising force. Assessment of the stability of solid waste landfills is somewhat less reliable than for soil embankments. The unit weight of refuse and its strength are difficult to determine and could vary over a wide range. Assessment of these variables is largely based on case histories and site-specific investigations [10]. Because of the extreme variability in refuse composition, the usual soil sampling and testing of soils on relatively small samples is not applicable for refuse such as typical municipal waste.

Potential instability could occur in the foundation soil, the refuse, or the cover. In all cases the safety margin is expressed in terms of the factor of safety, F, and is defined as

$$F = \frac{Available\ strength\ along\ the\ potential\ failure\ surface}{Mobilised\ strength\ along\ the\ potential\ surface}$$

If the landfill rules of various countries are analyzed it can be seen that most regulations provide for maximum inclination of the slopes of 1V: 2.5H (World Bank) to 1V:4H (Canada). The Safety Factor of slopes with these inclinations is usually acceptable but depends on the properties of the waste, climatic conditions and altitude [26]. Generically speaking, the legislation states that an appropriate Safety Factor, to ensure the stability of a slope to avoid slide, can vary between 1.3 and 1.5 [27, 28].

The value of factor of safety should be matched to uncertainty of analysis conditions. Recommended minimum values of factor of safety are shown in Table 3.1 by Duncan and Wright [34] and Table 3.2 from the U.S Army corps of Engineers' slope stability manual based on experience.

Table 3.1: Recommended minimum values of factor of safety by Duncan and Wright [34].

Cost and consequences of slope failure	**Uncertainty of analysis conditions**	
	Small	Large
Cost of repair comparable to incremental cost to more conservatively designed slope	1.25	1.5
Cost of repair much greater than incremental cost to construct more conservatively designed slope	1.5	2.0 or greater

3.2 COMPUTATIONAL METHODS

Limit equilibrium (LE) and finite element analysis are very important methods for different applications because of their ability to determine stability of geotechnical structures. The limit equilibrium method is based on the comparison of driving forces and/or moments to the resisting forces and/or moments from material strength acting within the soil mass.

In the limit equilibrium method, the available shear strength along a potential sliding surface is reduced by a factor of safety so that the mass contained within the sliding surface and the free surface is in a state of equilibrium. The limit equilibrium methods do not determine the displacement within the soil and waste mass.

The finite element method gives the stress-strain response of the mass caused by the forces that are imposed on it. This method is more accurate and considers estimation of stresses and deformations. This method has become successful because of the incorporation of representative stress-strain parameters. The stress-strain parameters for waste needed to perform finite element analysis are more difficult to obtain than the strength parameters needed in the limit equilibrium analysis.

3.2.1Limit Equilibrium Methods

Limit equilibrium methods are still currently most used for slopes stability studies. These methods consist in cutting the slope into fine slices so that their base can be comparable with a straight line then to write the equilibrium equations (equilibrium of the forces and/or moments). According to the assumptions made on the efforts between the slices and the equilibrium equations considered, many alternatives were proposed (Table 3.2). They give in most cases rather close results. The differences between the values of the safety factor obtained with the various methods are generally lower than 6% [35]. The traditional methods of slices used are those of Fellenius [21] and Bishop [22]. On figure 3.1 is represented the cutting of a portion of slope potentially in rupture. The equilibrium of slice *i* on the horizontal is written:

$$\mathrm{d}H_i - \sigma_i \tan\tan\alpha_i \mathrm{d}x + \tau_i \mathrm{d}x = 0$$

The forces applied on the i^{th} slice are defined in figure 3.1. H_i and H_{i+1} are horizontal inter-slice forces. V_i and V_{i+1} are vertical inter-slice forces.

Table 3.2: The main limit equilibrium methods [35].

Methods	**Equilibrium conditions satisfied**	**Slip surface**	**Use**
Ordinary Method of Slices (Fellenius, 1927)	Moment equilibrium about center of circle	Circular slip surface	Applicable to non-homogeneous slopes and c-ø soils where slip surface can be approximated by a circle. Very convenient for hand calculations. Inaccurate for effective stress analyses with high pore water pressures.
Bishop's Modified Method (Bishop, 1955)	Vertical equilibrium and overall moment equilibrium	Circular	Applicable to non-homogeneous slopes and c-ø soils where slip surface can be approximated by a circle. More accurate than Ordinary Method of slices, especially for analyses with high pore water pressures. Calculations feasible by hand or spreadsheet.
Janbu's Generalized Procedure of Slices (Janbu,1968)	Force equilibrium (vertical and horizontal)	Any shape	Applicable to non-circular slip surfaces. Also for shallow, long planar failure surfaces that are not parallel to the ground surface.

Morgenstern & Price's Method (Morgenstern & Price's, 1965)	All conditions of equilibrium	Any shape	An accurate procedure applicable to virtually all slope geometries and soil profiles. Rigorous, well established complete equilibrium procedure.
Spencer's Method (Spencer, 1967)	All conditions of equilibrium	Any shape	An accurate procedure applicable to virtually all slope geometries and soil profiles. The simplest complete equilibrium procedure for computing factor of safety.

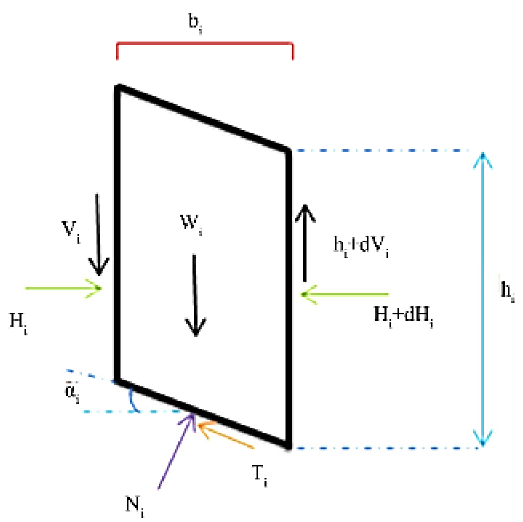


Figure3.1: Circular failure surface and forces acting on a single slice according to Bishop and Fellenius methods [21, 22].

W_i is the weight of i^{th} slice. N_i and T_i are resultant of the normal and tangential forces acting on the i^{th} slice base of length l_i and inclination α_i with respect to the horizontal (figure 3.1).
The equilibrium of slice i on the vertical is written:

$$\mathrm{d}V_i - \gamma_i h_i \mathrm{d}x + \sigma_i \mathrm{d}x + \tau_i \tan\tan\alpha_i \mathrm{d}x = 0$$

where γ_i is the unit weight of slice i.

In the method of Fellenius [21], we make the assumption that dH_i and dV_i are nil, which implies that the normal stresses are estimated by:

$$\sigma_i = \gamma h_i \cos^2 \alpha_i$$

By using the total definition of the safety factor, we obtain the equation:

$$F_{Fel} = \frac{\sum_{i=1}^{n} \left(C' + \left(\gamma h_i \cos^2 \alpha_i - u_i \right) \tan \varphi' \right) \frac{1}{\cos \alpha_i}}{\sum_{i=1}^{n} \gamma h_i \sin \alpha_i}$$

In Bishop's method of [23], we make the assumption that d$V_i = 0$. Thus, by considering the total definition of the safety factor, we obtain: $F_{Bish} = F\ (F_{Bish})$.

$$F_{\text{Bish}} = \frac{\sum_{i=1}^{n} \left(C' + \left(\gamma h_i - \left(\frac{C'}{F_{\text{Bish}}} + \sigma' \frac{\tan \varphi'}{F_{\text{Bish}}} \right) \tan \alpha_i - u_i \right) \tan \varphi' \right) \frac{1}{\cos \alpha_i}}{\sum_{i=1}^{n} \gamma h_i \sin \alpha_i}$$

The general procedure in all these methods can be summarized as follows:

- Assumption of the existence of at least one slip surface;
- Static analysis of normal and tangential stresses on the slip surfaces;
- Calculation of the safety factor *F*, defined like the ratio of the shear strength on effective shear stress along the failure surface considered;
- Determination of the critical failure surface with safety factor *F* minimum, among the whole analyzed surfaces.

3.2.2 Finite element analysis

The finite element method is a numerical procedure in which the mass under consideration is represented by an assemblage of elements interconnected at a finite number of nodal points [24]. The difference between the finite element method and the limit equilibrium approach is that there is no need to estimate the failure geometry in finite element analysis. When you know the stress conditions, the corresponding strains can be determined from the stress-strain behavior of the material under consideration. The finite element analysis gives result which shows a mesh with stress or deformation vector.

3.2.21 Plaxis 2D

The PLAXIS 2D program is a two-dimensional finite element program used to perform deformation and stability analysis for various types of geotechnical applications. Real situations may be modeled either by a plane strain or an axis symmetric model. The program uses a convenient graphical user interface that enables users to quickly generate a geometry model and finite element mesh based on a representative vertical cross-section of the situation at hand. To carry out a finite element analysis using the PLAXIS 2D program, the user has to create a two dimensional geometry model composed of points, lines and other components, in the X-Y plane and specify the material properties and boundary conditions.

In principle, first draw the geometry contour, then add the soil layers then structural objects, then construction layers, then boundary conditions and then loading. It is important to realize that the finite element mesh must be regenerated when the geometry of the existing model is changed. PLAXIS 2D computes the global safety factor by the phi/c reduction method. This method uses the load advancement number of steps. The incremental multiplier is used to specify the increment of the strength reduction of the first calculation step. The strength parameters are successively reduced automatically until all the additional steps have been performed. The strength of interfaces is also reduced in the same way. The last step should result in a fully developed failure mechanism. If a failure mechanism has not fully developed, then the calculation must be repeated with a larger number of additional steps.

3.2.3 Other Software for Stability Analysis

3.2.3.1 GeoSuite

GeoSuite (GS) Stability is a 2D slope stability program for evaluating stability of circular or non-circular failure surfaces in slopes. It is a part of the Novapoint GeoSuite Toolbox, which is a tool for making presentations and calculations in geotechnical design.

Like the rest of the GeoSuite package, GS Stability is based on AutoCAD. This means that users who are familiar with AutoCAD or other CAD software probably will recognize the basic drawing functions. The user can specify terrain geometry, soil layering, groundwater level, load conditions and other factors as needed, and the program can then search for a critical shear surface within specified search criteria.

There are four different methods of calculation to choose from:

- Force equilibrium
- Bishop simplified
- Bishop modified
- BEAST 2003

GS Stability can model various inputs, such as lime cement columns, soil nailing reinforcements and earthquake accelerations, and can also do stepwise construction of embankments or excavations and model variations in groundwater level and soil layering.

3.2.3.2 Slide

SLIDE software, developed by Rocscience Inc Toronto Canada, is also used for slope stability analysis for soil and rock slopes. The software is also 2D-LEbased computer program, which can be applied to evaluate the stability for circular or non-circular failure surfaces (SLIDE 2003). Slide 6.0 is the only slope stability software with built-in finite element groundwater seepage analysis for steady state or transient conditions. Flows, pressures and gradients are calculated based on user defined hydraulic boundary conditions. Seepage analysis is fully integrated with the slope stability analysis or can be used as a standalone module. Slide 6.0 has extensive probabilistic analysis capabilities you may assign statistical distributions to almost any input parameters, including material properties, support properties, loads, and water table location. The probability of failure/reliability index is calculated, and provides an objective measure of the risk of failure associated with a slope design. Sensitivity analysis allows you to determine the effect of individual variables on the safety factor of the slope.

Slide offers no less than 17 different material strength models for rock and soil including Mohr-Coulomb, Anisotropic and Generalized Hoek-Brown. Support types include tieback, end anchored, soil nail, micro pile and geotextile. Back analysis allows you to determine the required support force for a given safety factor. Advanced search algorithms simplify the task of finding the critical slip surface with the lowest safety factor.

3.3 MODES OF SLOPE FAILURE

Principal modes of failure in soil or rock are (i) rotation on a curved slip surface approximated by a circular arc, (ii) translation on a planar surface whose length is large compared to the depth below ground, and (iii) displacement of a wedge shaped mass along one or more planes of weakness. Other modes of failure include toppling of rock slopes, falls, block slides, lateral spreading, earth and mud flow in clayey and silty soils, and debris flows in coarse-grained soils.

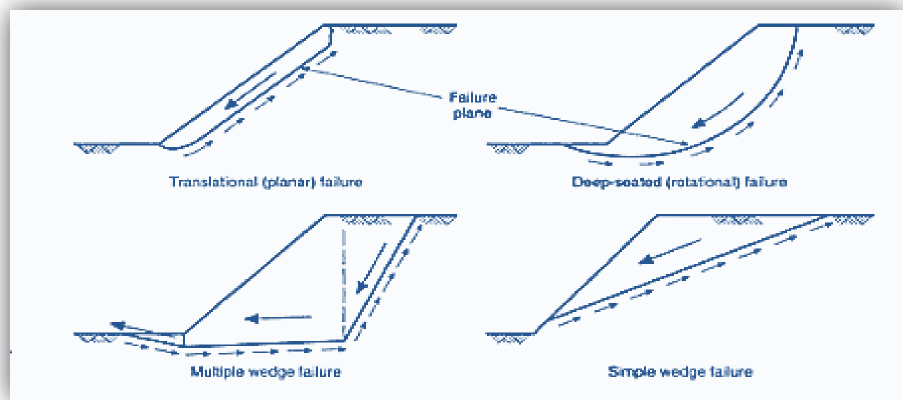


Figure 3.2: Modes of slope failure [34].

3.5 SOIL REINFORCEMENT

Soil can resist pressure and shear forces very well, but it is not able to tolerate tensile forces. Reinforced soil is composite material that contains components that can easily stand tensile forces. Nowadays reinforcing materials is widely used to overcome technical problems. Reinforced soil is used in stabilizing embankment (slope), fill dams, retaining walls, foundation and in-situ slope for increasing the shear resistance of soil layer in different earth structures.

Geosynthetics recognized as synthetic materials are used in soil [37]. The specific families of Geosynthetics are the following: Geotextiles, Geogrids, Geomembranes and Geocomposites. When synthetic fibers are made into a flexible, porous fabric by standard weaving machinery or are matted together in woven and nonwoven manner, the product known as "Geotextile". Geogrids are plastics formed into a very open netlike configuration. Geotextiles and Geogrids are used usually as reinforcing material for soil improvement [37]. These reinforcing materials are not susceptible to corrosion, have relatively low stiffness and flexible enough to tolerate large deformation.

3.5.1 Reinforced Steep Slopes

In reinforced steepened slopes, the reinforcement works with the compacted soil to create a stable mass that has enhanced geotechnical properties. Thus, slopes with surface inclinations greater than the natural angle of repose of the soil, can be constructed. The costs associated with the design and construction of reinforced steep slopes is far less than the costs associated with comparable alternates (i.e., cast-in-place concrete walls, soldier piles and lagging, soil nailing, etc.). Figure 3.3 shows steep slope reinforcement.

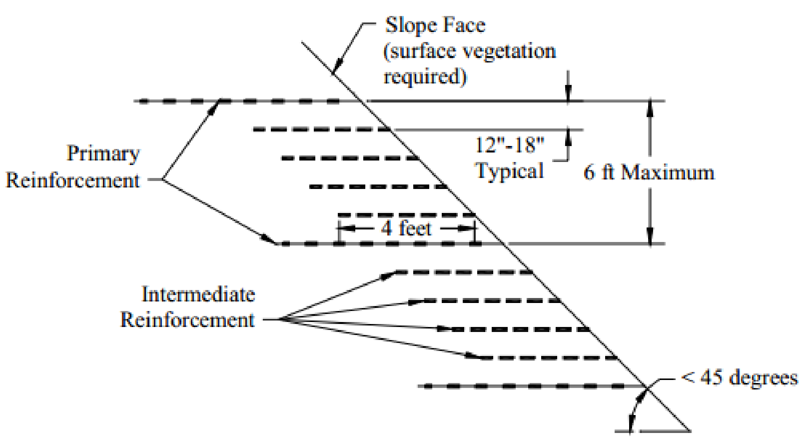


Figure 3.3: Cross section of reinforced slope with slope angle < 45 [38].

3.5.1.1 Design Methods

Reinforced slopes are currently analysed using modified versions of the classic limit equilibrium slope stability methods. A circular or wedge-type potential failure surface is assumed, and the relationship between driving and resisting forces or moments determines the slope’s factor of safety. Reinforcement layers intersecting the potential failure surface are assumed to increase the resisting moment or force. The design process must address all possible failure modes that a reinforced (or unreinforced) slope will potentially experience (figure 3.4). The design process must address:

- Internal stability for the condition where the failure plane crosses the reinforcement,
- External stability for the condition where the failure plane is located outside and below the reinforced soil mass; and,
- Compound stability for the condition where the failure plane passes behind and through the reinforced soil mass.

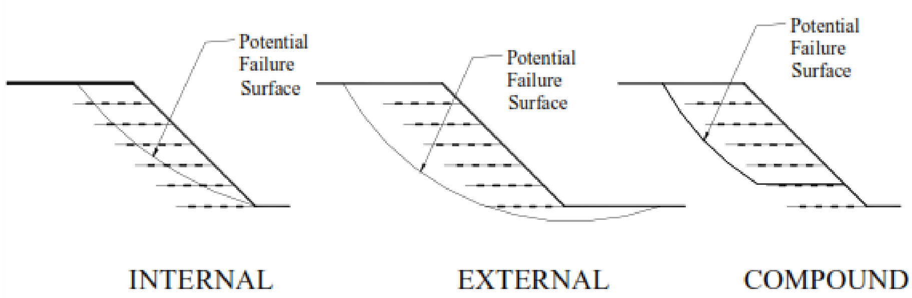


Figure 3.4: Failure modes of reinforced slopes [38].

3.5.1.1.1 Detailed Analysis Method

One approach to the design of reinforced soil slopes is to determine the required strength of reinforcement by means of detailed limiter equilibrium analysis methods such as the Bishop modified method of analysis [38]. The Bishop modified method of analysis can be extended to include the effect of tensile reinforcement. When a failure surface intersects a reinforcement layer, an additional resisting moment is added to the overall moment of equilibrium. A model for a rotational slip surface is presented in figure 3.5. In a more conservative approach, the deformability of the reinforcements is not taken in to account; therefore the tensile force is assumed to be horizontal as shown in figure 3.5.

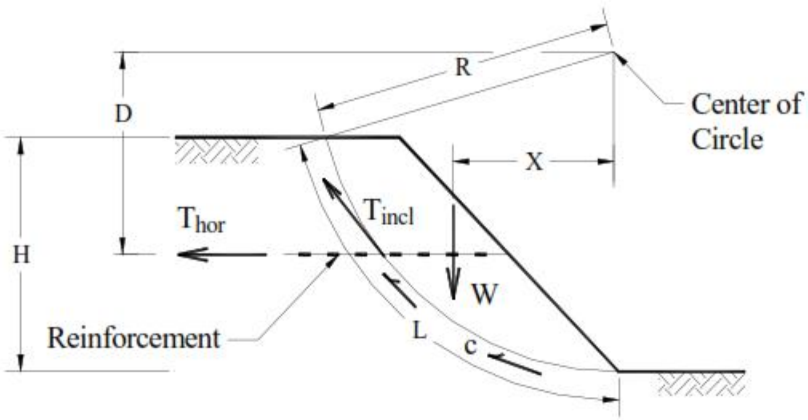


Figure 3.5: Model for detailed analysis [38].

The procedure requires that the most critical surface through the toe be located for the unreinforced case. Since it is assumed that the foundation soils are competent and capable of sustaining the load of the slope construction, only failure surfaces through the toe of the slope need be examined at this point [38]. The factor of safety for the unreinforced section is calculated as follows: $\mathrm{FS_u} = \frac{\mathrm{c \times L \times R}}{\mathrm{W \times x}} = \frac{\mathrm{M_R}}{\mathrm{M_D}}$

where M_R = Resisting Moment

M_D = Driving Moment

c = cohesion (kN/m^3)

L, R, W, x = as defined in Figure 3.5

The contribution of the reinforcement can be added directly to the resisting moment and the factor of safety (FSr) for the reinforced section is calculated as follows:

$$FS_r = \frac{M_R + M_G}{M_D} = \frac{M_R + (T_{hor} \times D)}{M_D}$$

where M_G = the resisting moment due to reinforcement

T_{hor}, D = as defined in figure 3.5

Note that the orientation of the reinforcement tensile force influences the calculation of the resisting moment due to the reinforcement and, thus, the factor of safety. As mentioned previously, the conservative approach is to consider the reinforcement tensile force (T_{hor}) to act horizontally. The maximum value that the resisting moment due to reinforcement can have is:

$$M_G = T_{incl} \times R$$

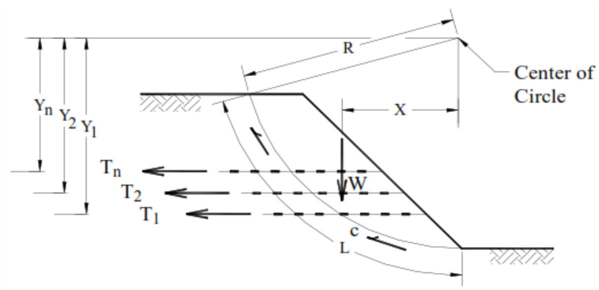


Figure 3.6: Model of multi-layered reinforcement slope [38].

Calculation of the resisting moment due to reinforcement for a multi-layered reinforced slope (figure 3.6), is given below:

$$M_G = \sum_{1}^{n} T_i \times Y_i$$

Finally, the embedment length of the individual reinforcement beyond the critical failure surface must be sufficient to provide adequate pullout resistance. The most frequently used equation to determine the required embedment length (L_e) of a reinforcement element is as follows:

$$L_e = \frac{R_{po} \times FS}{2 \times C_i \times \sigma_n \times tan\emptyset_i}$$

where R_{po} = pullout resistance

C_i = coefficient of interaction for pullout

σ_n = normal stress acting over geogrid anchorage length

ϕ_n = peak angle of friction for the reinforced soil

FS = factor of safety for pullout failure

Depending on the design specifications, minimum embedment lengths of one (1) foot to one (1) meter beyond the failure surface have been required.

3.5.1.1.2 Simple Wedge Methods (Schmertmann et al, 1987)

Two-part wedge, or bilinear, limit equilibrium models provide a method for quickly checking the computer-generated results. Design charts were developed based upon simplified analysis methods of two-part and one-part wedge-type failure surfaces and are limited by the following assumptions [38]:

- Extensible reinforcement elements are used,
- Slopes are constructed with uniform, cohesionless soil; ϕ', c' = 0, analysis appropriate,
- No pore pressures within the slope,
- No seismic loading,
- Competent, level of foundations,
- Flat slope face and horizontal slope crest,
- Uniform surcharge load at top of slope, and
- Horizontal reinforcement layers with coefficient of interaction (C_i) equal to 0.9.

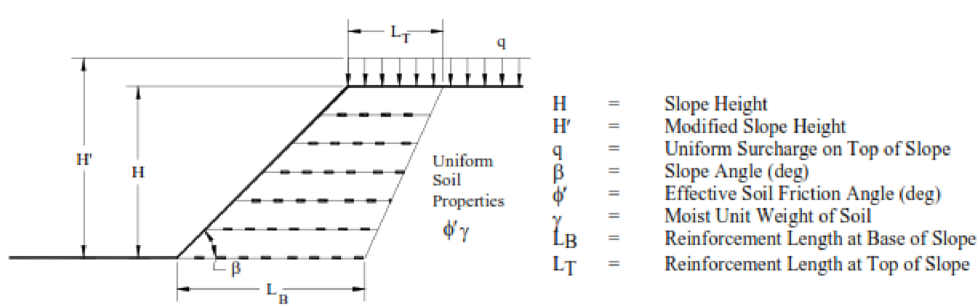


Figure 3.7: Slope geometry and definitions [38].

By definition, solutions for limit equilibrium models for a factor of safety (FS) equal to unity. The target, or desired, overall FS is taken in to account by factoring or reducing the soil shear strengths and is calculated as follows:

$$\phi'_f = tan^{-1}\left(\frac{tan\phi'}{FS}\right)$$

where ϕ' = soil friction angle

ϕ'_f = factored soil friction angle

The next step is to calculate the modified slope height (H') to take in to account any uniform surcharge loading at the top of the slope. The modified slope height is calculated as follows:

$$H' = H + \frac{q}{\gamma}$$

where H, q, and γ are defined in figure 3.7.

From the chart on figure 3.8, determine the force coefficient K and calculate the maximum tensile force requirement (T_{max}) from the following:

$$T_{max} = 0.5 \times K \times \gamma \times (H')^2$$

From the chart on figure 3.9, determine the required reinforcement length at the top (L_T) and at the bottom (L_B) of the reinforced section.

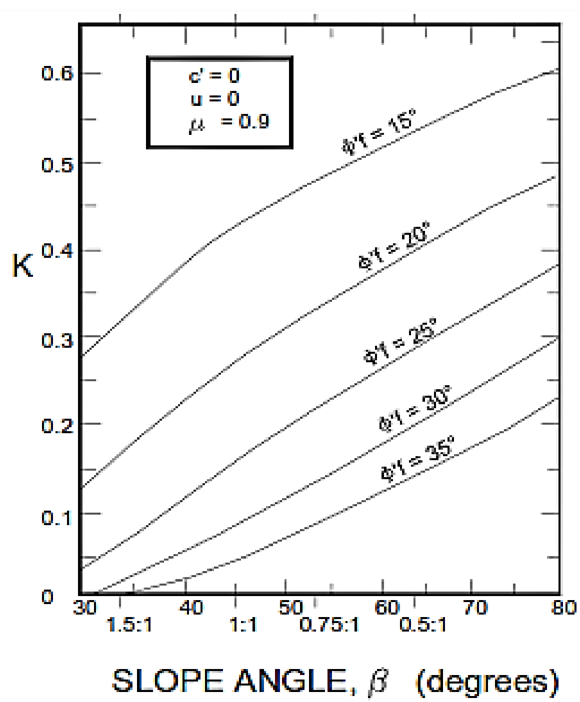


Figure 3.8: Reinforcement Coefficient K [38].

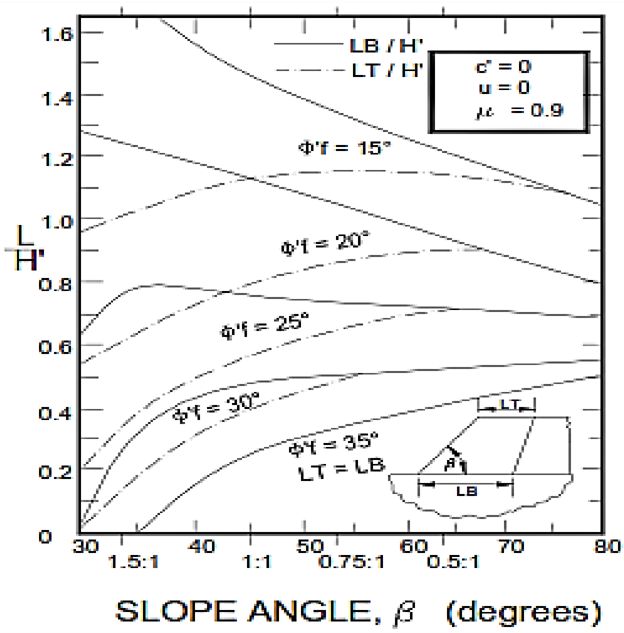


Figure 3.9: Reinforcement length ratios, L_T and L_B [38].

The next step in the procedure is to select the appropriate primary geogrid and calculate the number of layers required. The term primary geogrid layer refers to the geogrid required to satisfy internal, external, and global stability requirements. A t this point in the analysis, the designer must choose a geogrid so that the resulting spacing calculations yield acceptable values. For example, the spacing of primary geogrid layers at the bottom of a slope should not be less than 8 inches to 12 inches. This corresponds to typical earthwork fill thickness. Conversely, the primary geogrid spacing should be no greater than 4 feet. If calculations yield geogrid spacing less than the practical limit, then a stronger primary geogrid should be chosen. Alternatively, if the calculations yield geogrid spacing greater than 4 feet, a lighter geogrid can be selected. To determine the appropriate geogrid, calculate the long-term design strength (LTDS) of the material as follows:

$$LTDS = \frac{T_{ult}}{RF_{CR} \times RF_{ID} \times RF_D}$$

where: T_{ult} = ultimate tensile strength of the reinforcement as per ASTM D6637,
RF_{CR} = reduction factor due to creep,
RF_{ID} = reduction factor due to installation damage, and
RF_D = reduction factor due to durability.

The minimum number of geogrid layers for the reinforced section, is then calculated as follows assuming 100% coverage of the geogrid for a given vertical elevation:

$$N = \frac{T_{\max}}{LTDS} \quad \text{and} \quad S_v = \frac{H}{N}$$

where N = number of geogrid layers (rounded up to the next integer)
T_{max} = the total geogrid force (for a given section)

Note that T_{max} for a low section of slope is equal to the total geogrid force requirement for the entire height of the slope. For higher slope sections, T_{max} can be distributed over several zones. For example, for a three-zone section, one can distribute T_{max} as follows:

$$T_{Bottom} = \frac{1}{2} T_{\max}$$
$$T_{Middle} = \frac{1}{3} T_{\max}$$
$$T_{Top} = \frac{1}{6} T_{\max}$$

In other words, the section is divided in to three zones where there will be three different spacing and geogrid requirements. This results in an efficient and cost-effective design. Pullout embedment lengths have been taken in to considerations in the total length, L_T and L_B, in the chart in figure 3.9.

3.6 RETAINING STRUCTURES

Retaining structures are frequently used to support stable or unstable earth masses. The different types of retaining structures, as shown in figure3.10, are:

- Gravity walls (eg., masonry, concrete, cantilever, or crib walls)

- Tieback or soil nailed-walls
- Soldier pile and wooden lagging or sheet pile walls
- Mechanically stabilized embankments including geosynthetic and geogrid reinforced walls.

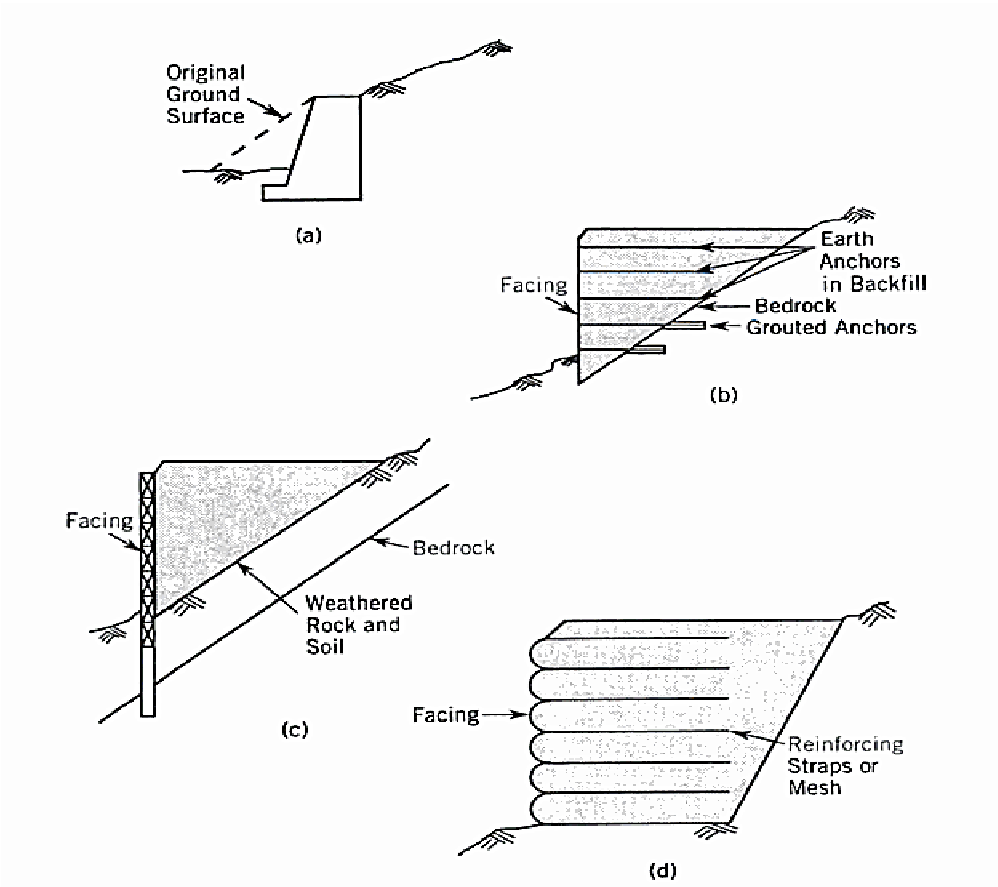


Figure 3.10: Types of retaining structures. (a) Gravity retaining wall. (b) Tieback retaining wall. (c) Sheet pile cantilever wall or soldier pile. (d) Mechanically stablised embankment [25].

The design of retaining structures requires three primary considerations:

- External stability of the soil behind and below the structure
- Internal stability of the retained backfill
- Structural strength of retained wall members.

CHAPTER FOUR

METHODOLOGY AND MATERIALS

4.1 BACKGROUND

The methodology requires gathering relevant data from literature survey of various studies in to in-depth study of engineering properties of soils relating to the site (Nideng). Soil properties are determined by field examination of the soils and by laboratory index testing of some benchmark soils. Established standard procedures are followed. During the survey, total soundings (rotary pressure sounding) are made and examined to identify and classify the soils and to delineate them on the soil maps. Samples are taken from some typical profiles and tested in the laboratory to determine grain-size distribution, plasticity, grains density, consolidation characteristics and shear strength parameters. The construction and stability analysis of the landfill dependent directly in most cases and indirectly always on these engineering properties.

The site for the landfill is characterised lithologically by sand, gravel, clay and silt. The landfill composed of mass deposit of peat which is characterised by fibrous and organic materials. It is assumed that the first layer of the soil is sand with an average thickness of 5m and is underlain by clay material.

Many different solution techniques for slope stability analyses have been developed over the years. Analyse of slope stability is one of the oldest type of numerical analysis in geotechnical engineering. The case study slopes were evaluated by Limit equilibrium methods, using the computer software SLIDE and the FE method using the software PLAXIS. The stability analysis was based on both short term (undrained or total stress analysis) and long term (Drained or effective stress analysis) stability analysis.

4.2 DATA COLLECTION AND UNCERTAINTIES

Accurate data collection is essential to maintaining the integrity of research. Both the selection of appropriate data collection instruments (existing, modified, or newly developed) and clearly delineated instructions for their correct use reduce the likelihood of errors occurring. Consequences from improperly collected data include inability to answer research questions accurately, inability to repeat and validate the study and distorted findings resulting in wasted resources. There could be an error propagation or distortion from the collection of data from the available literatures for this study. The difference between the intended use of this data and the actual use may lead to semantic error. The primary rationale for preserving data integrity is to support the detection of errors in the data collection process, whether they are made intentionally (deliberate falsifications) or not (systematic or random errors). There are two approaches that can preserve data integrity and ensure the scientific validity of study results. Each approach is implemented at different points in the research timeline:

i. Quality assurance - activities that take place *before* data collection begins
ii. Quality control - activities that take place *during* and *after* data collection

4.3 MATERIALS

Naturally occurred soil consists of three phases; solids, liquids and gas. A mixture of these phases has been shown in figure 4.1 by a schematic phase diagram, which shows a soil skeleton. Basic soil properties are important parameters not only to classify and identify the soils, but also to understand the soil behaviour.

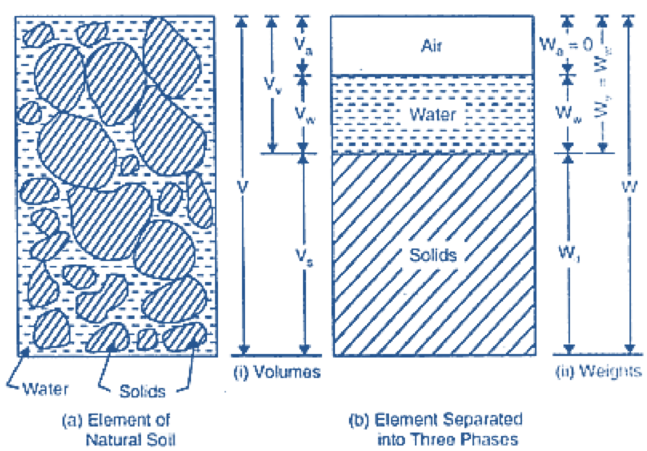


Figure 4.1: Soil as three phase system.

V = total volume, V_a = Volume of air, V_w = Volume of water, V_s = Volume of solids, V_v = Volume of void, W_s = Weight of solid, W_w = Weight of solid.

4.3.1 Physical and Engineering Properties of Peat Material

Peat is an accumulation of disintegrated plant remnants which have been preserved under condition of high water content and incomplete aeration [40]. According to Radforth [41] peat can be divided into three main types for engineering purposes. First, amorphous – granular peats which have high colloidal minerals and seem to be like clay in grain structure where the inter-spaces water is kept locked in an adsorbed condition around particles. The two peat types are fine-fibrous and coarse-fibrous peats which hold the inter spaces water in the peat mass as a free water and these types are described as woodier peat. There have been several studies on the physical and engineering properties of peat in several countries in the world and have been presented in Table 4.1, Table 4.2 and the input parameters for the stability analysis were inferred from these properties.

Table 4.1: Geotechnical paramters of peat in Surfers Paradise, Queensland-Australia [42].

Parameter	Values	Parameter	Value
Liquid Limit	259-305 %	Coeff. of Secondary Consolidation (Cα)	0.03-0.04
Plastic Limit	125 – 207 %	Over Consolidation Ratio (OCR)	1.1 – 2.8 %
Plasticity Index	88 –134	Apparent Cohesion	43-166 kPa
Water Content	168 – 247 %	Apparent Friction Angle	0–14 Degree
Unit weight	1.14 t/m^3	Shear Strength	30–40 kPa
Coeff. of Consolidation (Cv)	1.12 – 40.5 m^2/year	Ash Content	20%
		Specific Gravity	1.572
Coeff. of Volume Compressibil-ity (Mv)	7.00E-05– 1.30E-04 m^2/kN	Permeability (k)	1.40E-05– 2.40E-07 m/sec
Compression Index (Cc)	1.83-2.74	Organic Content (estimated)	63-68 %

Table 4.2: Laboratory shear box test results of peat in different locations in Malaysia [40].

Location	MC	OC	LL	Von Post Scale	C (kPa)	φ
	211	85	294	H1	9-11	9-20
Banting, Selangor	195	79	219	H2	6-11	9-16
	832	84	361	H5	8-10	7-10
	219	94	316	H6	11-12	9-12
	225	85	166	H8	8-12	6-11
Kg. Jawa, Selangor	215	78	180	H3	10-12	6-14
	209	89	325	H6	12-14	7-25
	786	85	368	H8	7-11	8-13
Kg. Jawa, Selangor	680	85	298	H3	11-12	10-15
	747	93	352	H5	10-12	5-10
	720	83	282	H7	7-9	9-12
Dengkil, N. Sembilan	246	98	305	H2	13-17	3-12
	301	98	335	H5	11	13-15
	786	83	377	H8	8-9	12-20

In which MC= Moisture Content, OC = Organic Content, LL = Liquid Limit, C = Cohesion and ϕ = Friction angle

Farrell & Hebib [43] also working at Trinity College Dublin reported results of a comprehensive laboratory investigation into the shear strength of an Irish peat recovered from Raheenmore bog. The peat was about 98% organic and had moisture content ranging between 1200% and 1400%.The peat was found to have undrained shear strength of about 5 kPa. The main findings of this study can be summarized as follows:

- The apparent effective angle of shearing resistance, as measured in undrained triaxial compression tests, was about $\varphi' = 55°$.
- Failure as defined by peak deviator stress was not reached in drained triaxial compression tests.
- An effective angle of shearing resistance $\varphi' = 38°$ was measured in both the direct shear box and the ring shear test, whereas the direct simple shear (DSS) yielded lower value of $\varphi' = 31°$. The concurrence of the results of the ring shear and direct shear box test would suggest that φ' measured is representative of the matrix, whereas φ' measured in triaxial compression is more representative of the matrix and the reinforcing effect of the fibres.

Hanrahan and Walsh [47] and Hanrahan et al. [48] report on a comprehensive triaxial shear test program, involving 130 tests, on remoulded (macerated) peat samples and it was concluded that the qualitative behaviour of peat in its remoulded and undisturbed states is similar and that the strength of peat was frictional (c′ ranges from 5.5 kPa to 6.1 kPa and φ ′ – from 36.6° to 43.5°).An example from this work is shown in figure 4.2 (Sodha [49]). It can be seen that the behaviour of the material under shear in isotropically consolidated triaxial tests is not unlike that of a lightly overconsolidated mineral soil. However, unlike mineral soils, there is a different failure envelopes corresponding to different initial water contents, with φ ′ increasing with decreasing water content.

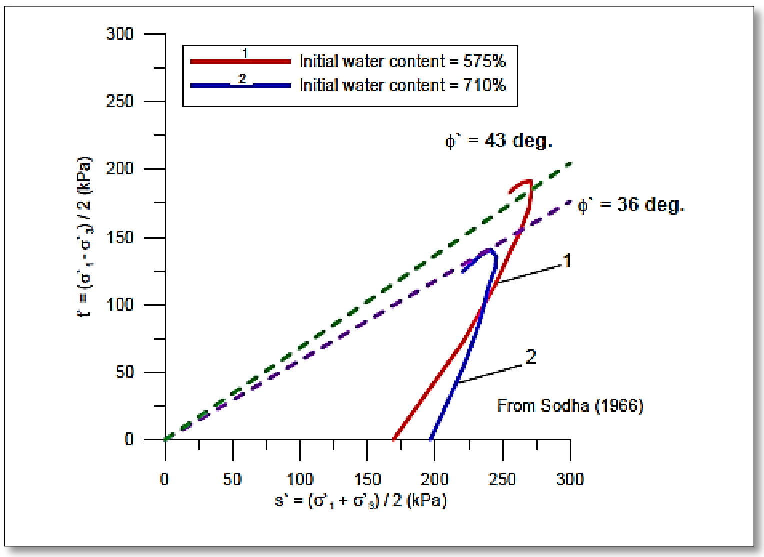


Figure 4.2: Behaviour of macerated peat of different water contents Sodha [49].

A review of the peat properties, as well as the previous research performed shows that peat soils in different regions of the world exhibit markedly different physical and engineering properties. High compressibility, low shear strength, and high moisture content are the main characteristics which define the peat ubiquitously.

Undrained shear strengths (S_u) of peat have been reported to vary from 20 kPa in fibrous peat to below 4 kPa in more humified peats [45]. S_u been shown to vary with several factors, such as degree of humification and water content (see Fig. 4.1, from Helenlund [46]).

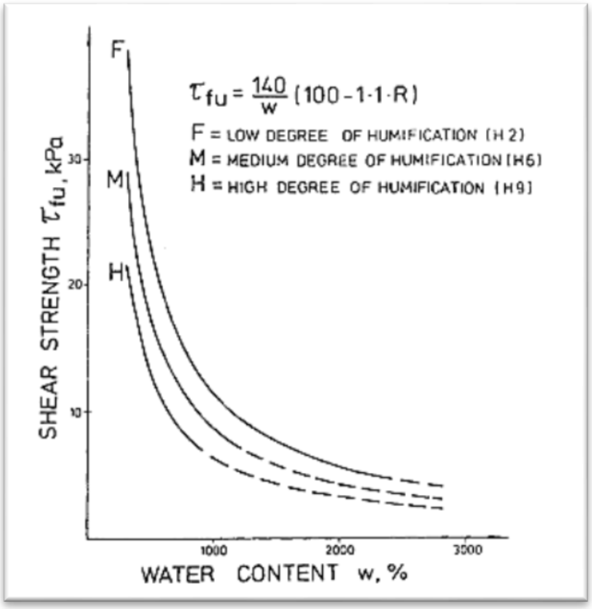


Figure 4.1: Relationship between shear strength, moisture content and degree of humification [46].

Lunne et al. [50] provide a useful review of the use of the CPTU in peat and organic soils. They summarise case histories of work in peat from Holland, Germany and Canada. In conclusion, they suggest that peat is characterised by a high friction ratio (R_f), greater than perhaps 5%, and that negative pore pressures can be developed in fibrous zones.

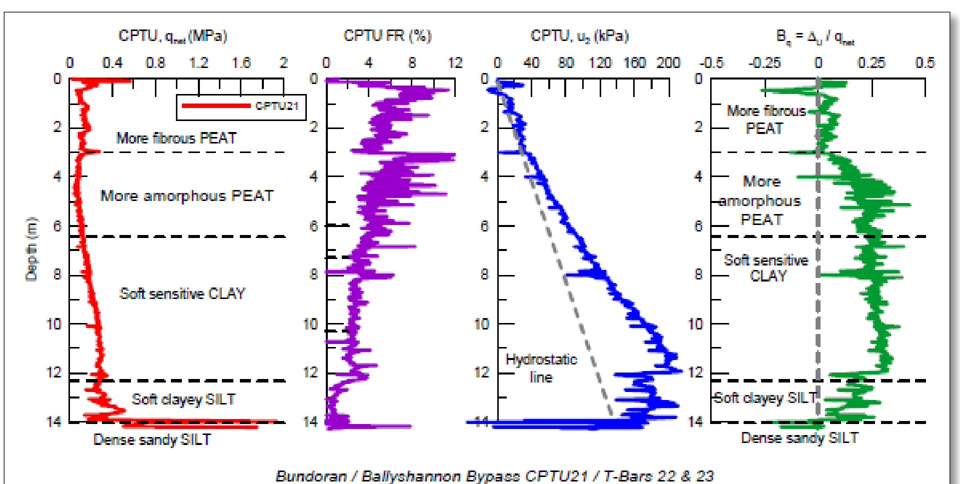


Figure 4.3: Use of CPTU as a profiling tool [45].

An example from some recent work in Ireland during the ground investigation for the Bundoran/Ballyshannon bypass in County Donegal is shown in figure 4.3. This site is underlain by approximately 6.5 m of peat (raised bog) over soft sensitive clay. As suggested by Lunne et al. [50], R_f values are high in the peat being in the range from 4% to 12%. The more fibrous upper peat is clearly distinguished from the deeper more amorphous peat by the higher net cone resistance and the pore pressure being either hydrostatic or slightly negative. In contrast, positive excess pore pressures are generated in the more amorphous lower peat. It can also be seen that the pore pressure parameter B_q is particularly useful in delineating the two separate peat zones. In the fibrous zone B_q it is close to zero, whereas in the more amorphous zone B_q is about 0.25.

It would seem there is much promise in the use of the CPTU as a profiling tool in peat soils. However further work is necessary to relate measured (or derived) CPTU parameters to actual properties of the peat.

4.3.2 Physical and Engineering Properties of Sand, clay and silt

The physical and engineering properties of sand, clay, silt and gravel are presented in Table 4.3 from the Håndbok 016 – Geoteknikk i vegbygging. The input paramters for stability analysis was based on these properties in the Table 4.3.

Table 4.3: Physical and Engineering properties of sand, clay, silt and gravel.

Soil Type	γ_{sat} (kN/m^3)	E (MPa)	ν	a (kPa)	φ(^{o})	k (m/year)	c_u (kpa)	Reference
Clay	19	1-3	0.33	0-20	26	10^{-5} - 10^{-2}	15-50	[38]
Sand	17-18	15-50	0.3	0-10	33-36	10^{2} - 10^{4}		[38]
silt	18-19	20	0.33	0-10	31-33	10^{-3} - 10^{2}	25-70	[38]
Gravel	18-19	70-170	0.33	0-10	36-38	10^{3} - 10^{5}		[38]

In which: γ_{sat}: Saturated unit weight; E: Soil Young's modulus; ν: Poisson's ratio; a: Soil attraction; φ: Angle of internal friction; and k: permeability coefficient.

CHAPTER FIVE

NUMERICAL MODELLING

5.1 DESCRIPTION OF THE PROJECT SITE

The landfill to be established at Nideng in Klæbu covers an area of about 130.4 decars and topography of the project site is primarily ridges with valleys and stream passing through the site. The quaternary geological map shows mountains and weathering material at the project site as well as thick marine deposit. The site has undergone landslide activities in the past and is believed that the ridges left by the slides consist of clay or sand / gravel. The eastern portion of the landfill site indicates a valley with steep slope and the western part of the landfill is located in an area with several small valleys.

Plans for developing the area have been launched, which means that proper stability evaluations have to be carried out. Three selected profiles for stability evaluations at the site are shown on Figure 5.1, whereas a cross-section through the slope in all the profiles is given in Figure 5.2.

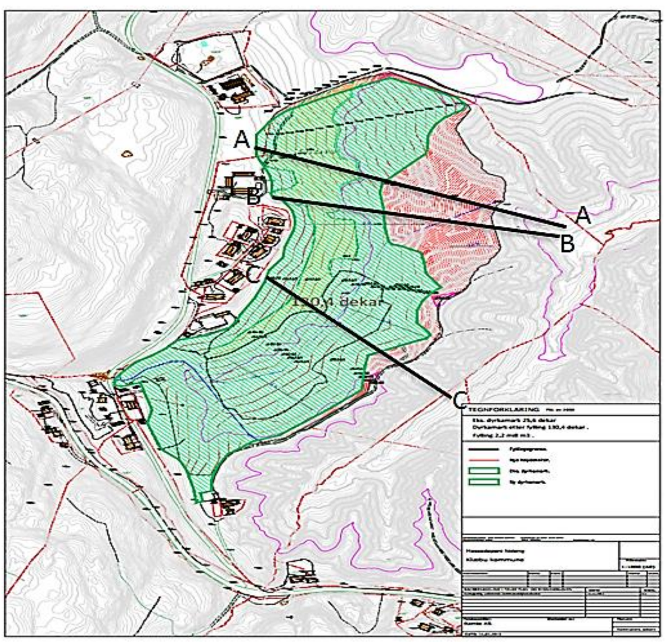


Figure 5.1: Plan view showing the selected profiles.

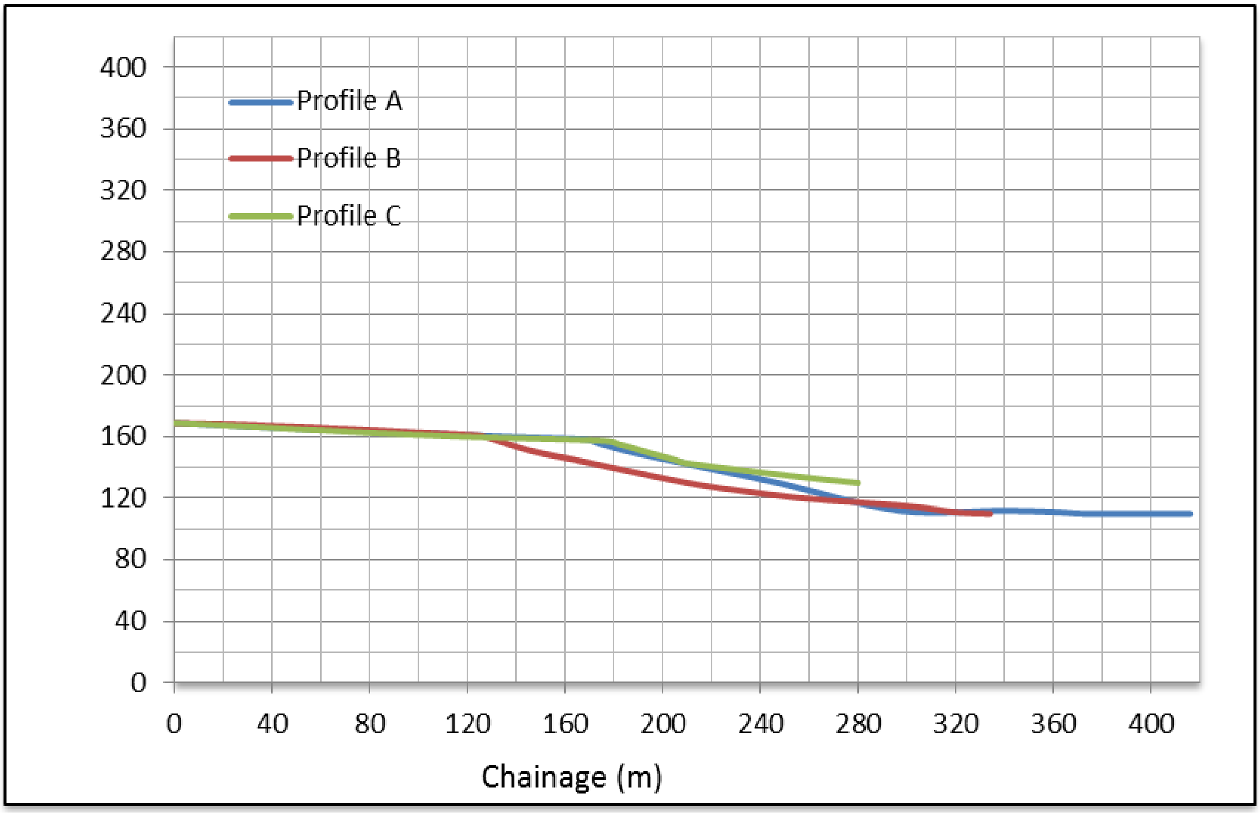


Figure 5.2: Cross-section through the slopes in Profile A, B and C.

5.2 MATERIAL PROPERTIES AND SLOPE STABILITY EVALUATIONS

The slope analysis is correlated from soil investigation reports, along with the data presented in Håndbok 016 – Geoteknikk i vegbygging. These properties are presented in Table 5.1.The material properties of each of the layers of the landfill were assigned to the model. The embankment of the slope is composed of peat with organic material. The subsoil consists of 5m of sand and underneath the sand layer is clay/silt. The case study slopes were evaluated by limit equilibrium methods, using the computer software SLIDE and the finite element method using the software PLAXIS. The stability analysis was based on both short term (undrained or total stress analysis) and long term (Drained or effective stress analysis) stability analysis. The Mohr-Coulomb model was selected for each material because the Mohr-Coulomb criterion better represents the soil behavior in addition to the physical meaning of its parameters (Das, 1983; Bishop, 1966).

Table 5.1: Soil Properties.

Soil Type	γ_{sat} (kN/m^3)	E (MPa)	ν	a (kPa)	φ(°)	k (m/year)	c_u (kpa)	Reference
Clay	19	1-3	0.33	0-20	26	10^{-5} - 10^{-2}	15-50	[38]
Sand	17-18	15-50	0.3	0-10	33-36	10^{2} - 10^{4}		[38]
silt	18-19	20	0.33	0-10	31-33	10^{-3} – 10^{-2}	25-70	[38]
peat	8 - 12	0.3	0.33	0	23	10^{-2} -10^{-1}	15	[42]
Gravel	18-19	70-170	0.33	0-10	36-38	10^{3} - 10^{5}		[38]

In which: $\gamma_{sat:}$ Saturated unit weight; E: Soil Young's modulus; ν: Poisson's ratio; a: Soil attraction; φ: Angle of internal friction; and k: permeability coefficient.

5.3 MODELLING WITH SLIDE

Initial step for analyzing the model, is to create the geometry of the model. The geometry characteristics such as embankment height, slope and crest width. The other geometry which should be defined is under laying soil profile such as thickness of the soft layer. The second step is to provide the material properties of the embankment and the under laying soil. For present investigation the geometrical model of the various profiles are shown in figure 5.3.

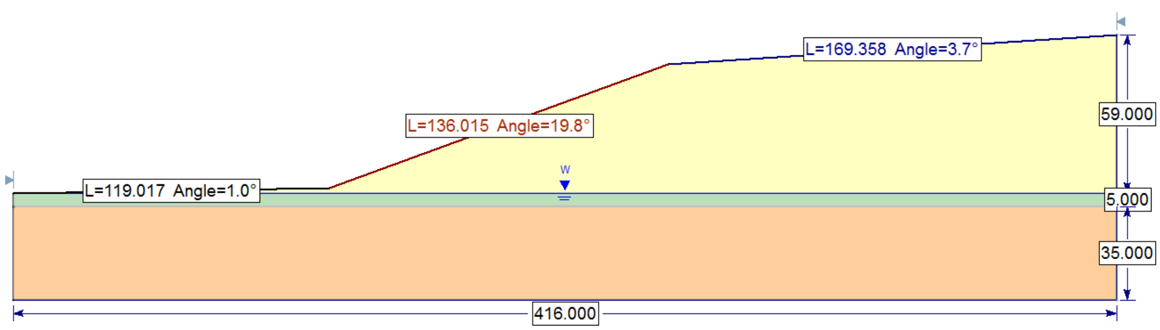


Figure 5.3a: Geometrical Model of profile A.

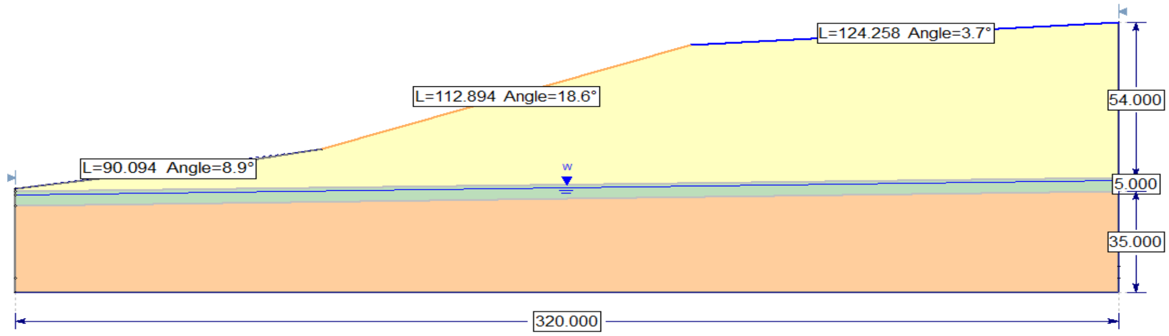


Figure 5.3b: Geometrical Model of profile B.

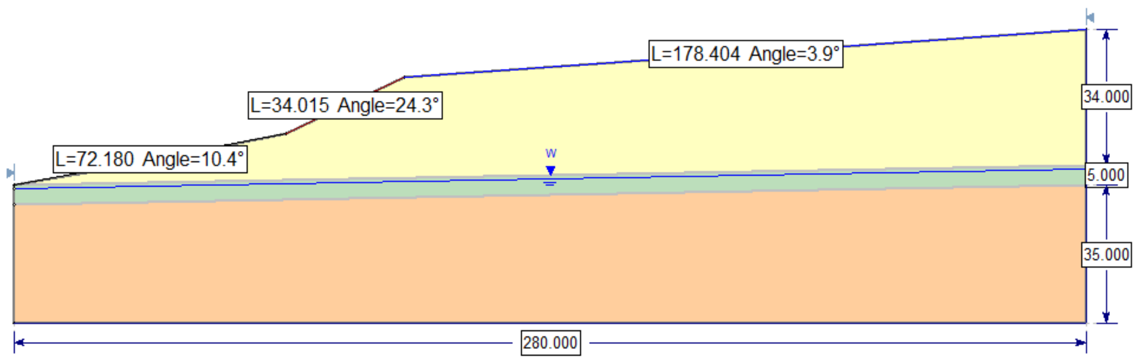


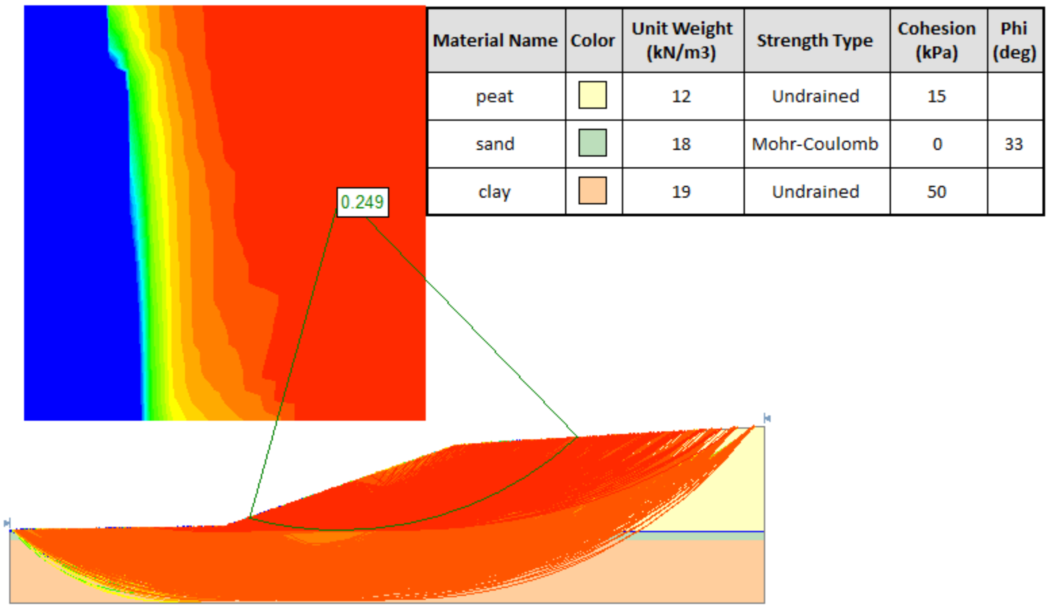
Figure 5.3c: Geometrical Model of profile C.

The material parameters used for this analysis is based on conservative approach and the input materials are shown in table 5.2. The factor of safety was computed based on Mohr Coulomb failure criterion. Both the undrained and drained stability analysis by the Slide software was based on Janbu Simplified method. Ground water table was taken to be 1m below the natural ground surface.

Table 5.2: Input Parameters.

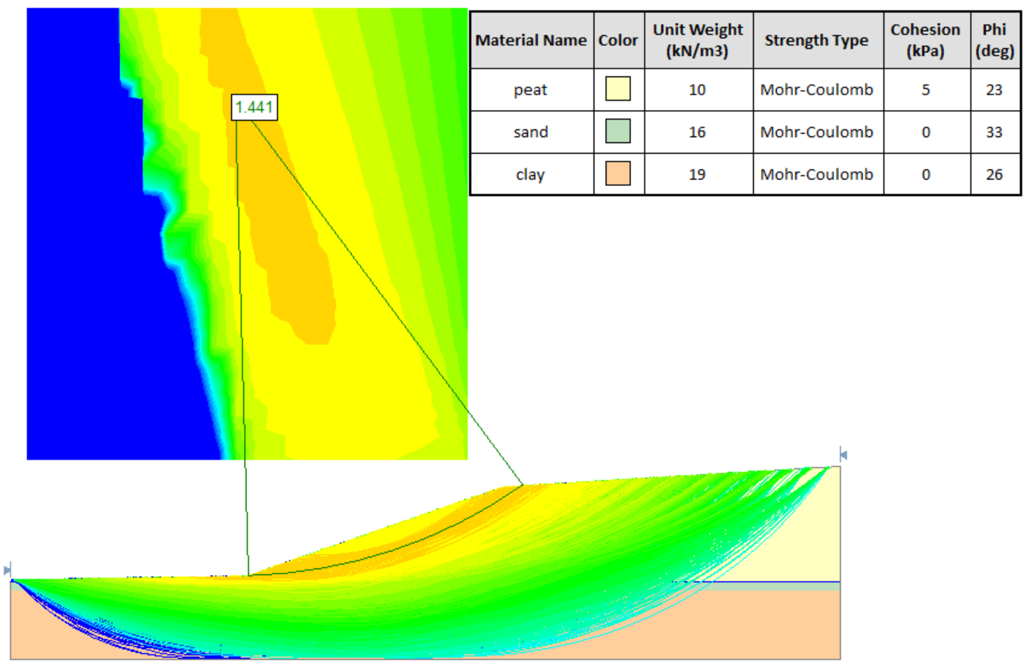
Soil Type	γ_{sat} (kN/m^3)	C (kPa)	φ(°)	C_u (kpa)
Clay	19	0	26	50
Sand	18	0	33	
silt	18	0	31	50
peat	12	5	23	15
Gravel	18	1	36	

The computed results from the slide software indicating the location of the failure zones and the factor of safety for both undrained and drained conditions for the various profiles are shown in figure 5.4, 5.5 and 5.6. Analysis of the slope showed that the calculated safety factor for the undrained case ranges from 0.25 to 0.48 and for the drained case range from 1.44 to 1.53. An appropriate safety factor to ensure the stability of a slope to avoid slide, can vary between 1.3 and 1.5 [27, 28]. The calculated safety factor has not satisfied the minimum safety factor required. The presence of the peat layer presents a pre-existing sliding surface that reduces the safety factor into such low values. The existence of the peat layer controls the shape of the failure surface. The failure surface passes through the peat layer because it is the weakest layer in the whole slope's soil profile. The slope shear strength parameters also have influence on the slope safety factor. The soil cohesion parameter for the various material layer was set to zero whcich could also be factor for the reduce factor of safety. There is no doubt that increasing the value of the cohesion force and the friction angle will increase the slope safety factor. Soil reinforcement like geotextiles and geogrid can also be used to improve the slope stability. Table 5.3 shows the summary of the factor safety for the various profiles.



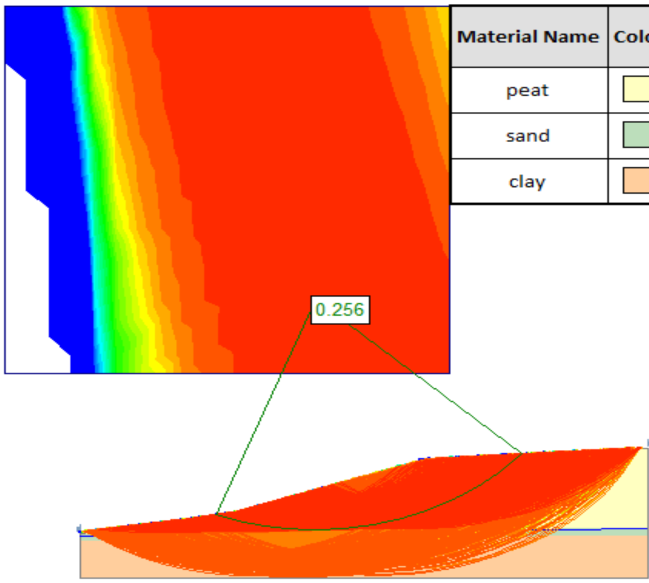
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		12	Undrained	15	
sand		18	Mohr-Coulomb	0	33
clay		19	Undrained	50	

Figure 5.4a: Stability assesment based on undrained conditions for profile A.



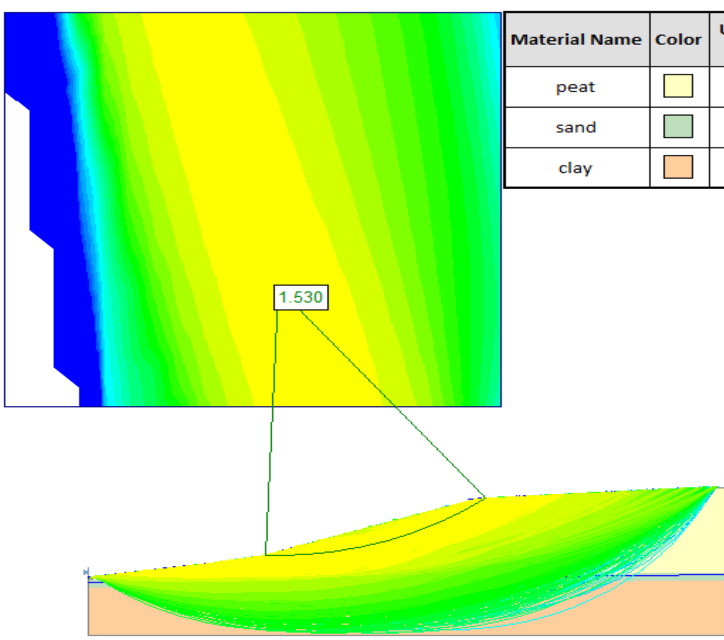
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

Figure 5.4b: Stability assesment based drained conditions for profile A.



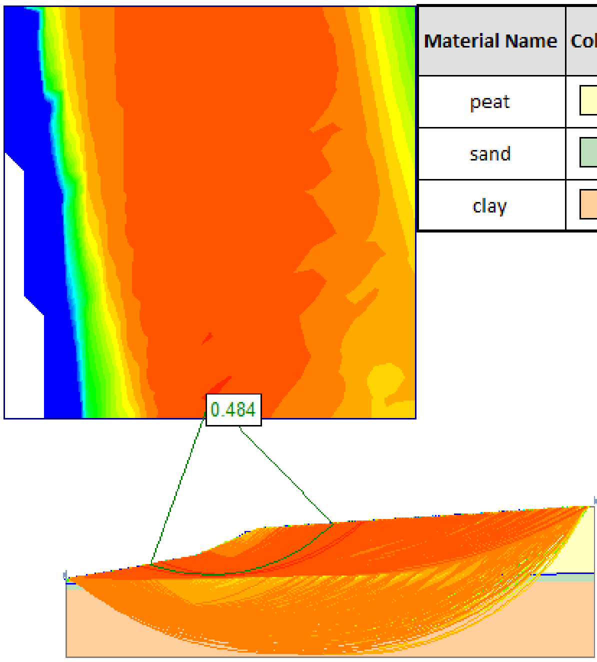
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		12	Undrained	15	
sand		18	Mohr-Coulomb	0	33
clay		19	Undrained	50	

Figure 5.5a: Stability assesment based on undrained conditions for profile B.



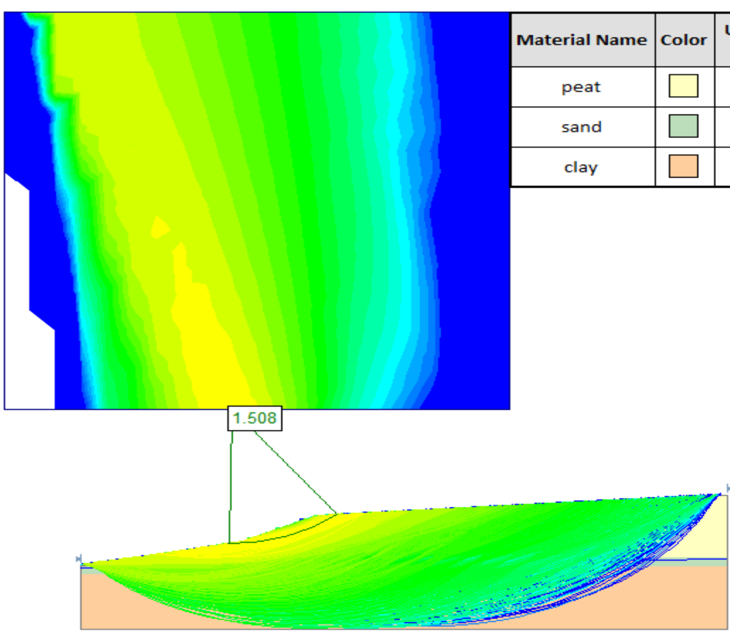
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		12	Mohr-Coulomb	5	23
sand		18	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

Figure 5.5b: Stability assesment based on drained conditions for profile B.



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		12	Undrained	15	
sand		18	Mohr-Coulomb	0	33
clay		19	Undrained	50	

Figure 5.6a: Stability assesment based on undrained conditions for profile C.



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		12	Mohr-Coulomb	5	23
sand		18	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

Figure 5.6b: Stability assesment based on drained conditions for profile C.

Table 5.3: Summary of factor of safety.

Profile	FS (Undrained)	FS (Drained)
A	0.25	1.44
B	0.26	1.53
C	0.43	1.51

5.3.1 Effects of cohesion

There is no doubt that increasing the slope shear strength parameters of the peat material will consequently increase the slope safety factor. Figure 5.7 shows the relation between the soil cohesion (c) and the safety factor of the slope (F.S.) for different soil cohesion of 0 kpa, 5 kpa, 10 kpa and 15 kpa at effective angle of internal friction of 23^0 of the peat material for profile A. Increasing the soil cohesion resulted in a noticeable increase in the safety factor for the slope. The figure 5.8 shows the stability assessment based on the influence of soil cohesion.

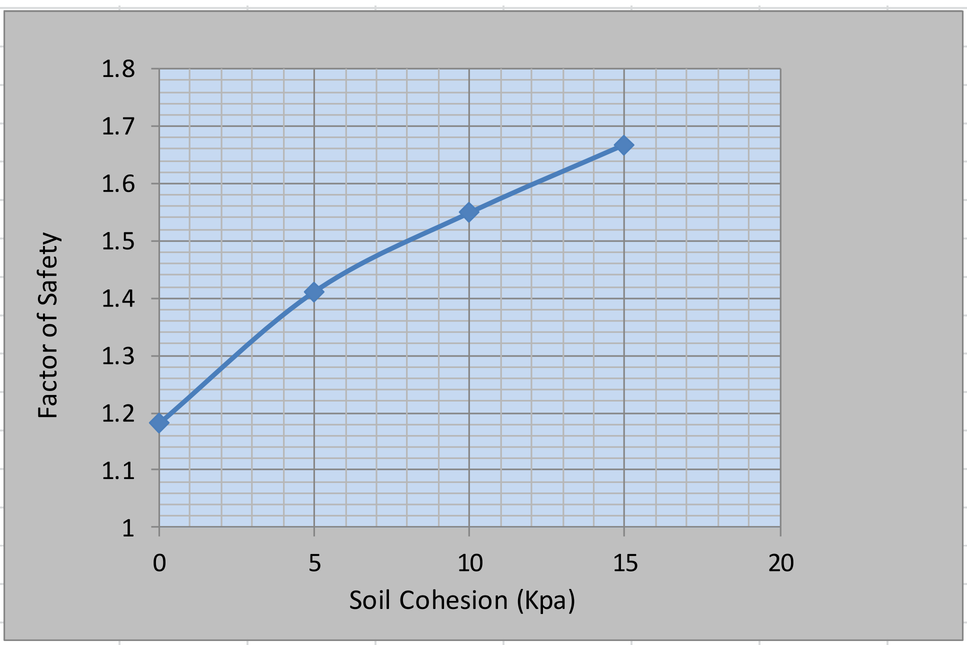


Figure 5.7: Effect of Soil Cohesion on the F.S. (c = 0 kpa, 5 kpa, 10 kpa, 15 kpa, $\varphi = 23^0$).

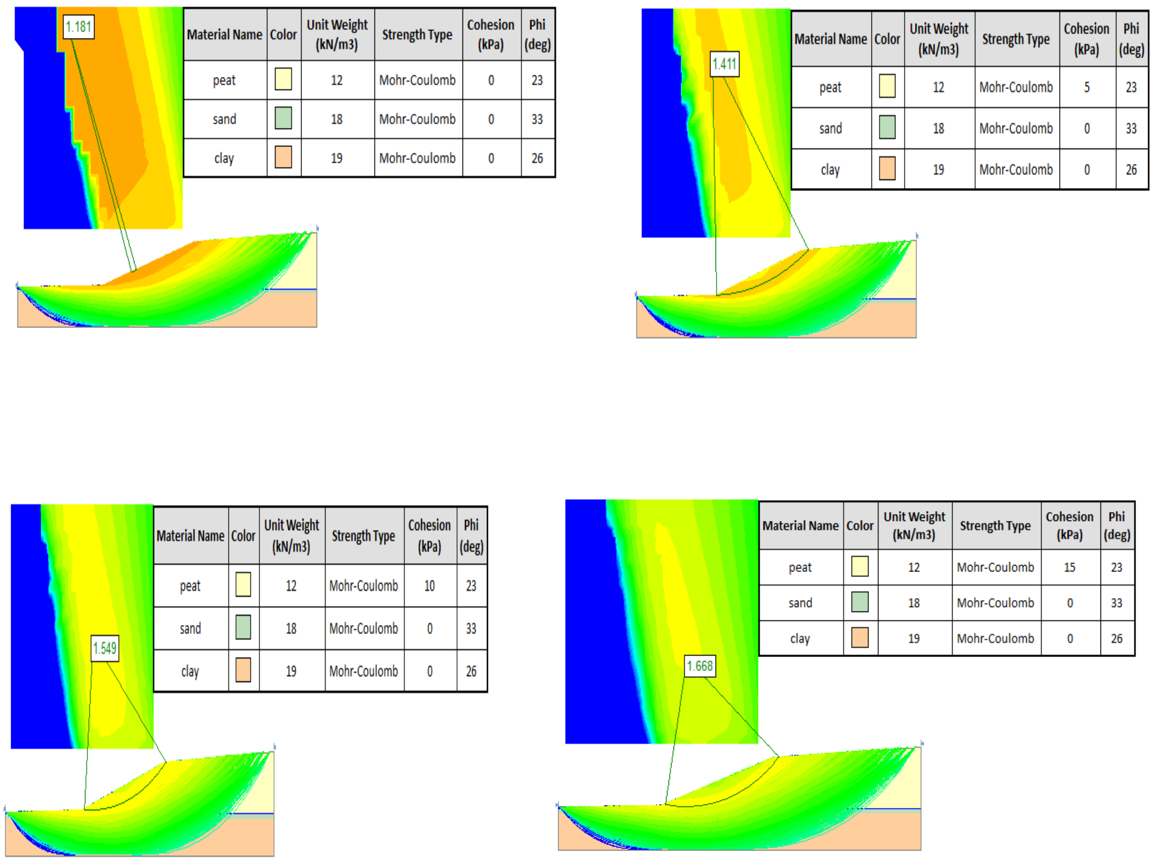


Figure 5.8: Stability assessment of profile A based on influence of soil cohesion.

5.3.2 Effect of friction angle

The frictional angle (ϕ) of the peat material was varied from 20^0 to 50^0 and their effect on the factor of safety is shown in figure 5.9 for profile A. From the figure below it can be seen that with an increase of the friction angle of the peat material the factor of safety increases significantly. The figure 5.10 shows the stability assessment based on the influence of friction angle.

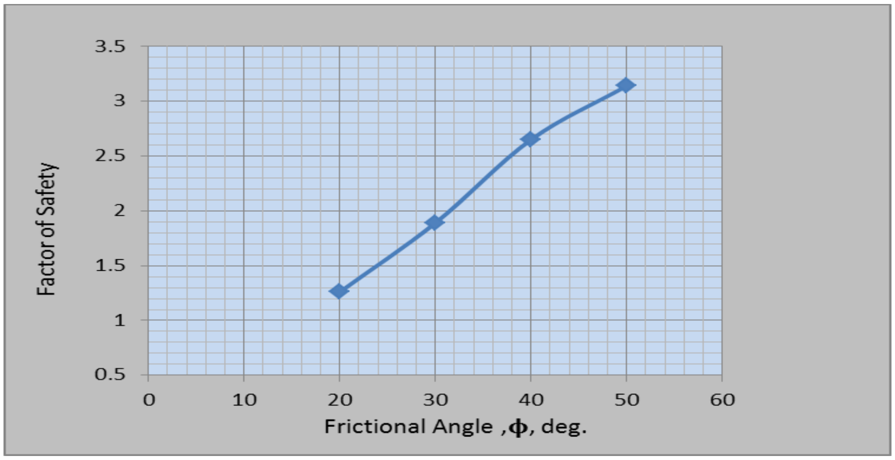


Figure 5.9: Effect of frictional angle on the Factor safety.

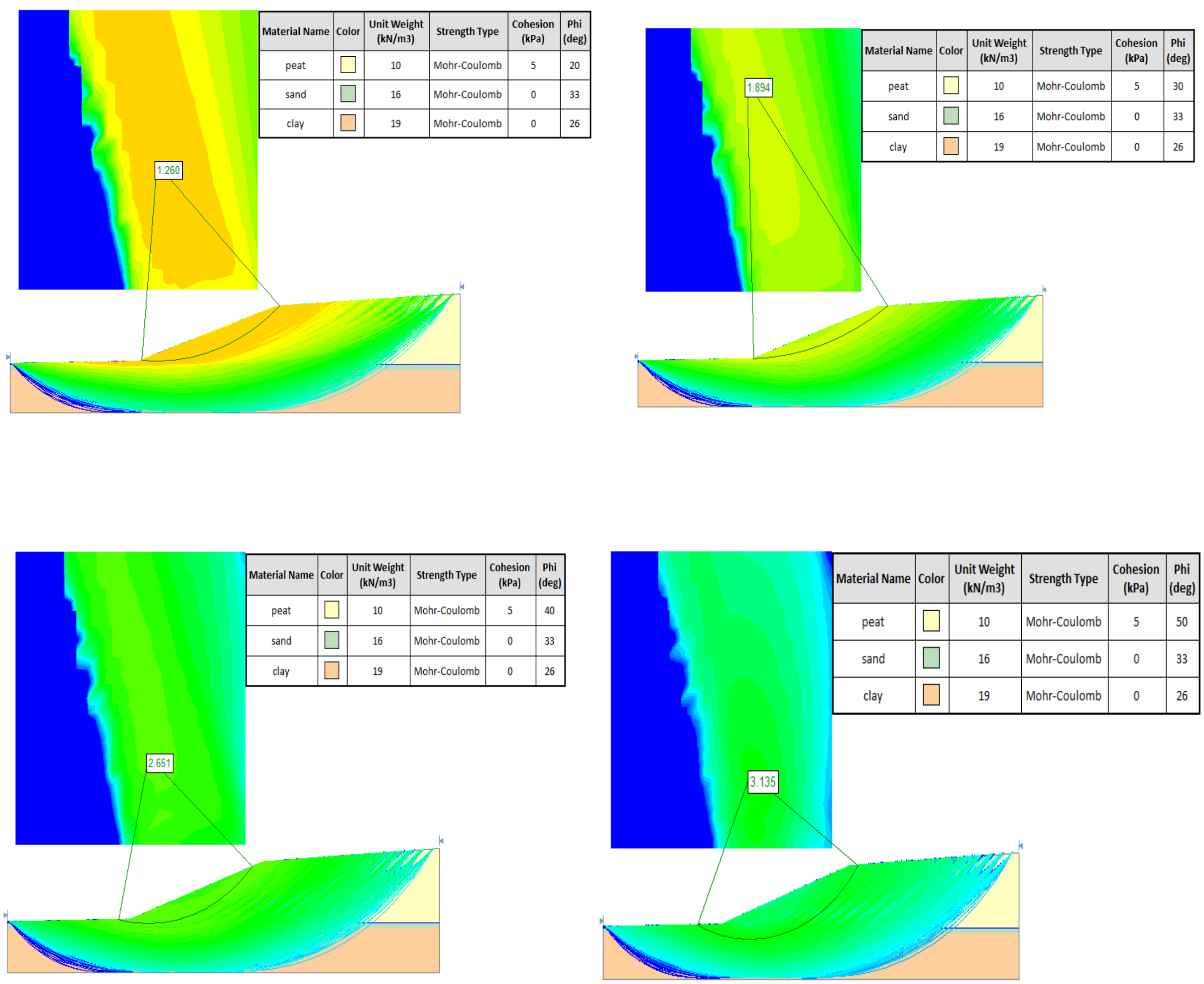


Figure 5.10: Stability assessment of profile A based on influence of friction angle.

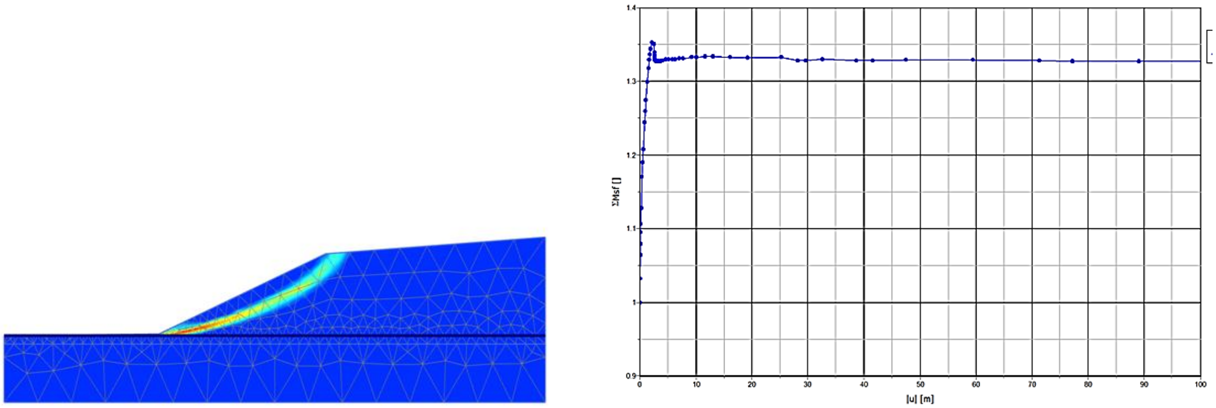
5.4 MODELLING WITH PLAXIS

The slopes of the various profiles were modelled in the input module of PLAXIS, based on 15 noded elements in a plane strain model. A sufficient, well-refined mesh was generated to obtain the least possible factor of safety. Similarly, the soil properties in each layer were defined using a Mohr-Coulomb soil model. The input parameters used include Young's modulus and Poisson's ratio which are the elastic properties of the layers and the effective cohesion, the effective friction angle and the dilatancy angle which are the strength parameters for the layers. The dilatancy angle was assumed to be zero for all the layers since it is a reasonable value. Moreover, the initial stresses due to soil and ground water conditions were computed in the calculation module of PLAXIS. The input parameters are shown in Table 5.4.

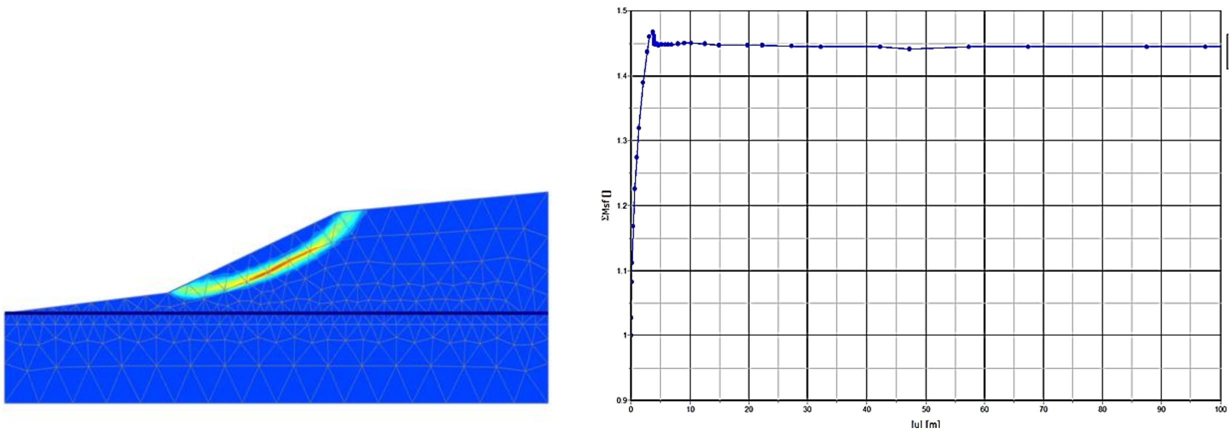
Table 5.4: Input parameters.

Soil Type	γ_{sat} (kN/m^3)	C (kPa)	φ(^{o})	ν	E (MPa)	k (m/year)	c_u (kpa)
Clay	19	0	26	0.33	1.5	10^{-3}	50
Sand	18	0	33	0.3	30	10	
peat	12	5	23	0.33	0.3	10^{-2}	15
Gravel	18	1	36	0.3	30	10	

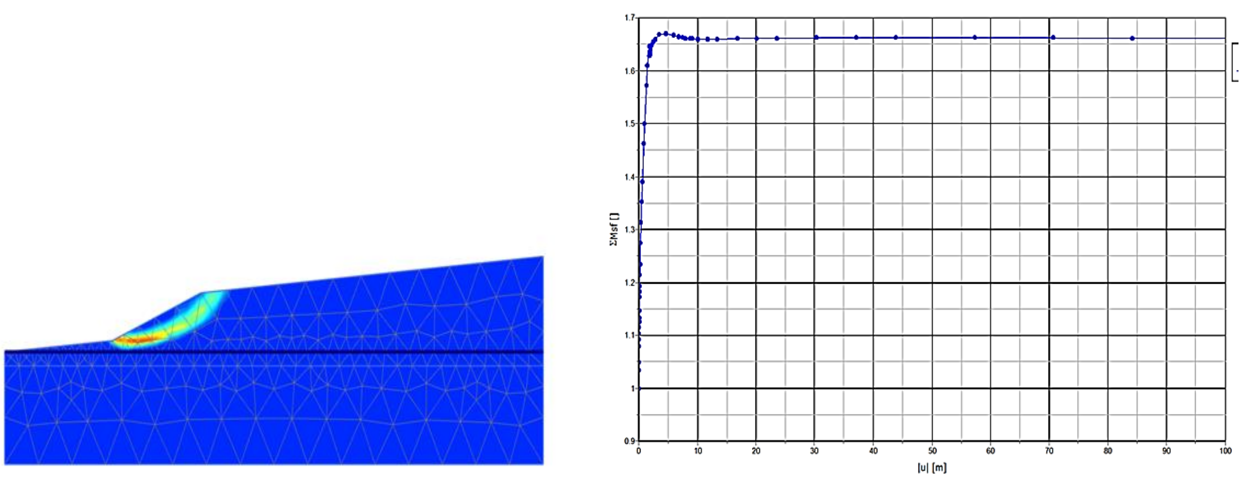
PLAXIS 2D computes the global safety factor by the phi/c reduction method. This method uses the load advancement number of steps. The number of steps used for the landfill models ranged from 300 - 850. The incremental multiplier was set to the default value which is 0.1. The strength parameters are successively reduced automatically until all the additional steps have been performed. The strength of interfaces is also reduced in the same way. The last step should result in a fully developed failure mechanism. A failure mechanism was fully developed when the line displaying the factor of safety was constant at a certain value. The computed results from the plaxis indicating the location of the failure zones and the development of the factor of safety against toe displacement for drained conditions for the various profiles are shown in figure 5.11. Analysis of the slope showed that the calculated safety factor for the drained case range from 1.33 to 1.66. Table 5.5 shows the summary of the factor safety for the various profiles.



(a) Profile A



(b) Profile B



(c) Profile C

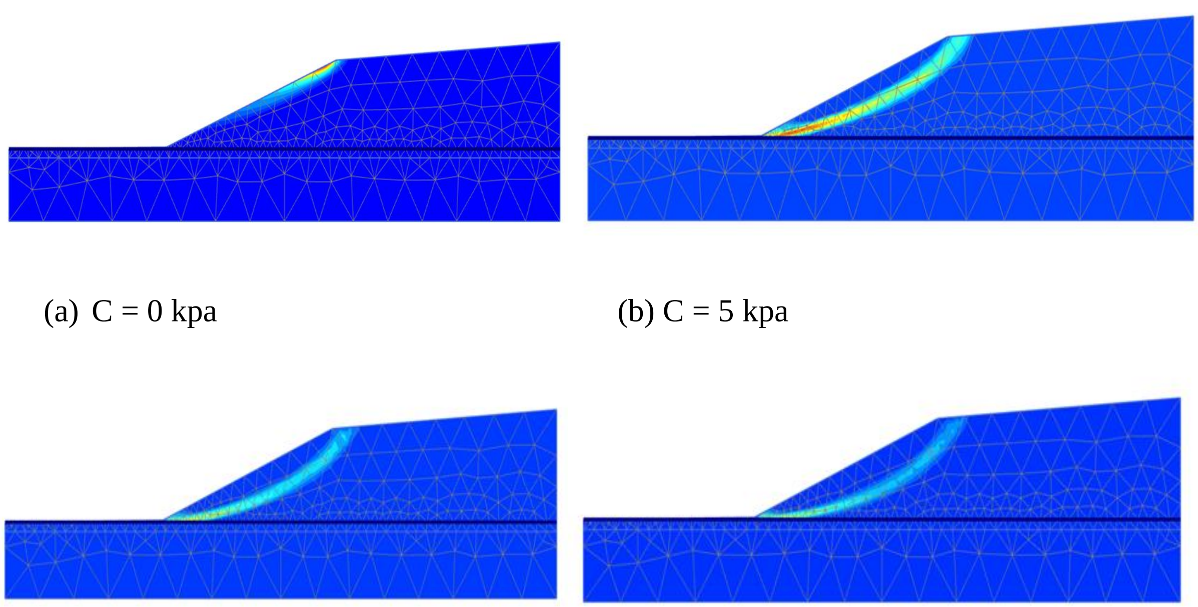
Figure 5.11: Maximum shear strain development of the embankment with factor of safety curve.

Table 5.5: Summary of factor of safety.

Profile	FS (Drained)
A	1.33
B	1.44
C	1.66

5.4.1 Effects of cohesion

The cohesion of the peat material were varied from 0 to 15 kpa at an effective angle of internal friction of 23^0 for profile A. Increasing the soil cohesion resulted in a noticeable increase in the safety factor for the slope. The figure 5.12 shows the development of the shear strain and the graph of factor of safety against cohesion are shown in figure 5.13.



(a) C = 0 kpa (b) C = 5 kpa

(c) C = 10 kpa (d) C = 15kpa

Figure 5.12: Maximum shear strain development of the embankment with variation of the cohesion of the peat from 0 to 15 kpa.

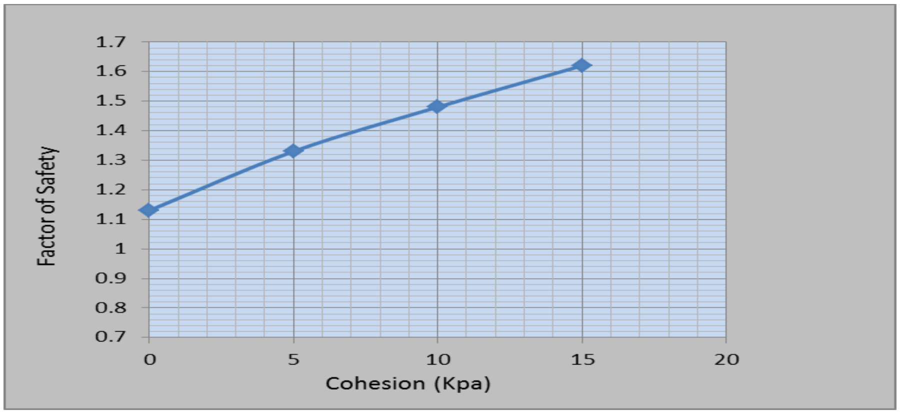
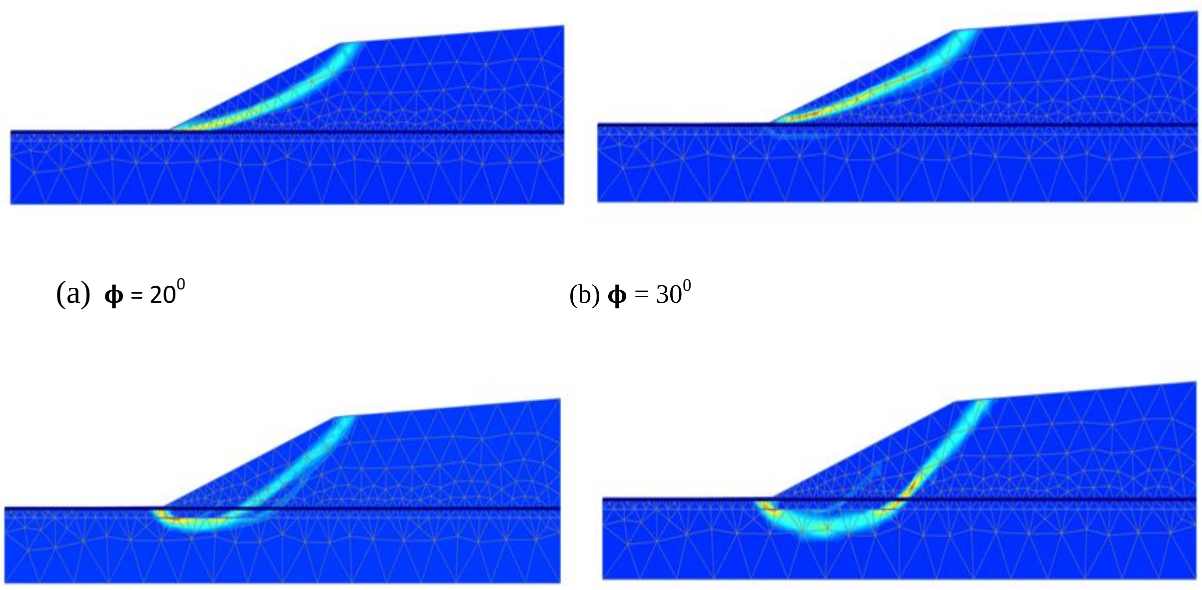


Figure 5.13: Effect of Soil Cohesion on the F.S. (c = 0 kpa, 5 kpa, 10 kpa, 15 kpa, $\varphi = 23^0$).

5.4.2 Effect of friction angle

The frictional angle (ϕ) of the peat material was varied from 20^0 to 50^0 at cohesion of 5 kpa for the same profile A. The figure 5.14 shows the development of the shear strain and the graph of factor of safety against the friction angle are shown in figure 5.15. From figure 5.15 below it can be seen that with an increase of the friction angle of the peat material the factor of safety increases significantly.



(a) $\phi = 20^0$ (b) $\phi = 30^0$

(c) $\phi = 40^0$ (d) $\phi = 50^0$

Figure 5.15: Maximum shear strain development of the embankment with variation of the friction angle of the peat from 20^0 to 50^0.

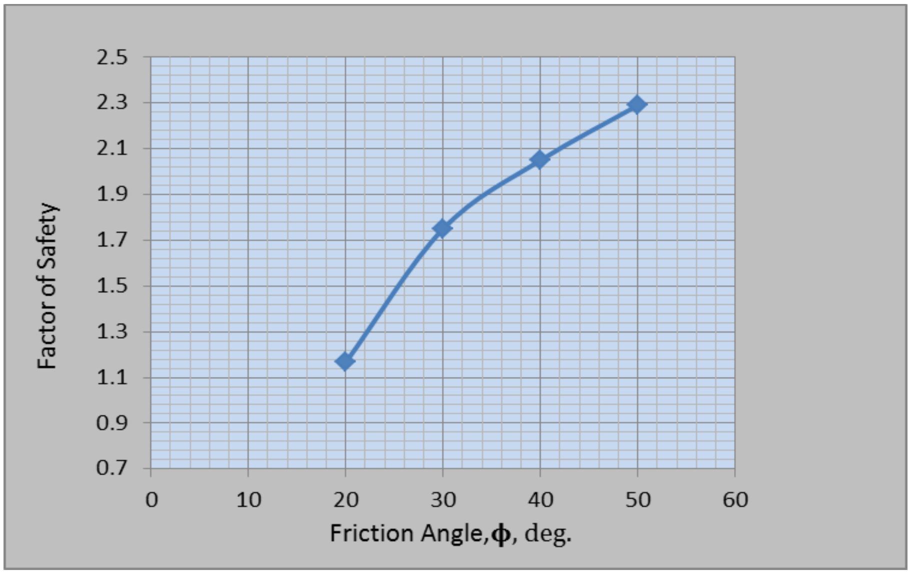


Figure 5.15: Effect of frictional angle on the Factor safety.

5.5 COMPARISON OF RESULTS BETWEEN THE SLIDE AND PLAXIS 2D

The geometry of the landfill and the material input parameters used in both the PLAXIS model and the SLIDE model were the same. Both programs computed different factors of safety as shown in Table 5.6. From Table 5.6, it can be seen that there are variations of the computed factor of safety by both programmes.

Table 5.6: Comparison of factor of safety computed by slide and plaxis.

Profile	FS with plaxis (Drained)	FS with slide (Drained)
A	1.33	1.44
B	1.44	1.53
C	1.66	1.51

5.6 EFFECT OF BERM ON THE PEAT MASS EMBANKMENT

Slope instability is often due to gravitational and seepage forces during construction of excavation. Often berms are used to improve the stability of slopes and embankments as the berm provides a greater resisting moment along the developing slip mechanism. On the whole, the increase in stability produced by the berm does not increase without limit as the position of the failure mechanism may change as the berm width or height is increased. The berm employed for the stability enhancement of the embankment consists of gravels and the input parameters used is shown in Table 5.1 and the geometric model of the peat embankment with the berm which is in triangular form is also shown in figure 5.16.

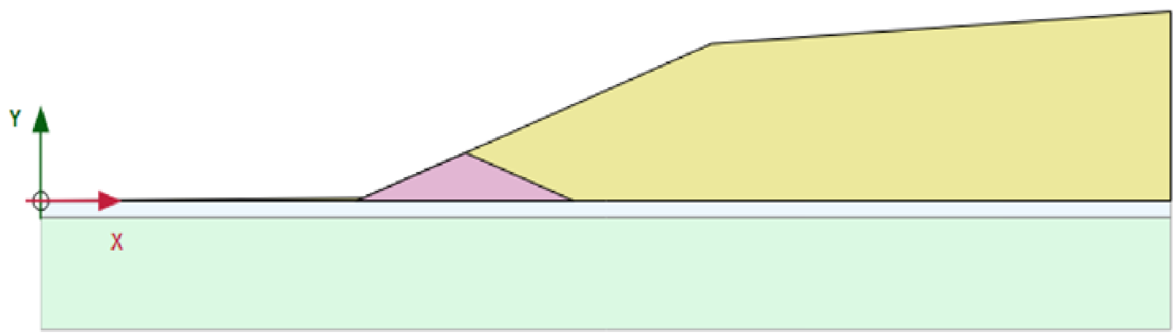


Figure 5.16: Geometric model of the peat embankment with the Berm.

From the modelling, the stability of the embankment increases steadily with increase in the height of the back berm which indicates how effectively to enhance stability of embankment with back berm. The relationship between the safety factor of the peat embankment and the height of the berm is showed in table 5.7 and Figure 5.17.

Table 5.7: Summary of factor of safety of the peat embankment with the height of the berm.

Height of berm, H_b	Factor of Safety
0m	1.33
5m	1.37
10	1.39
15m	1.42

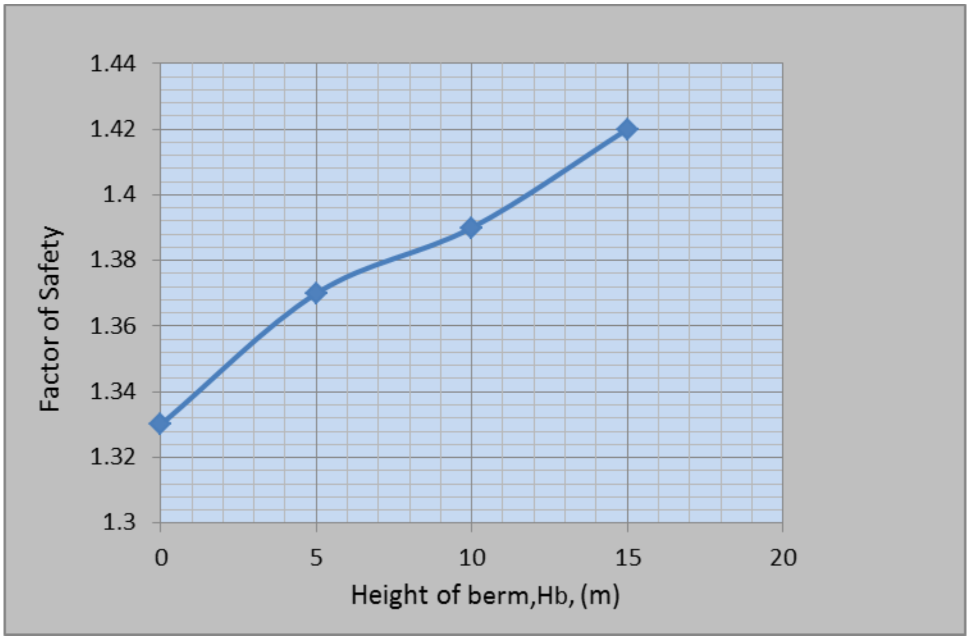
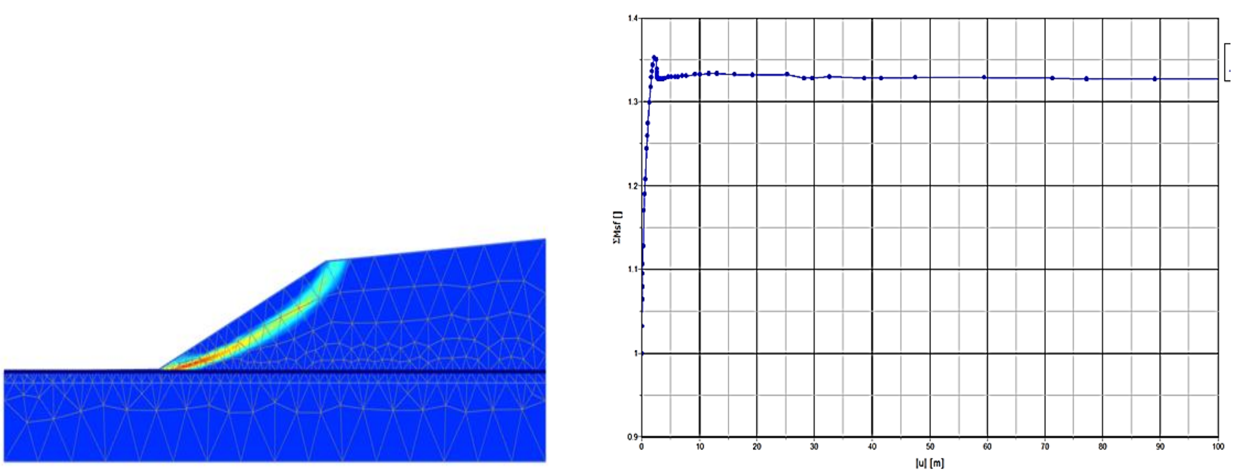
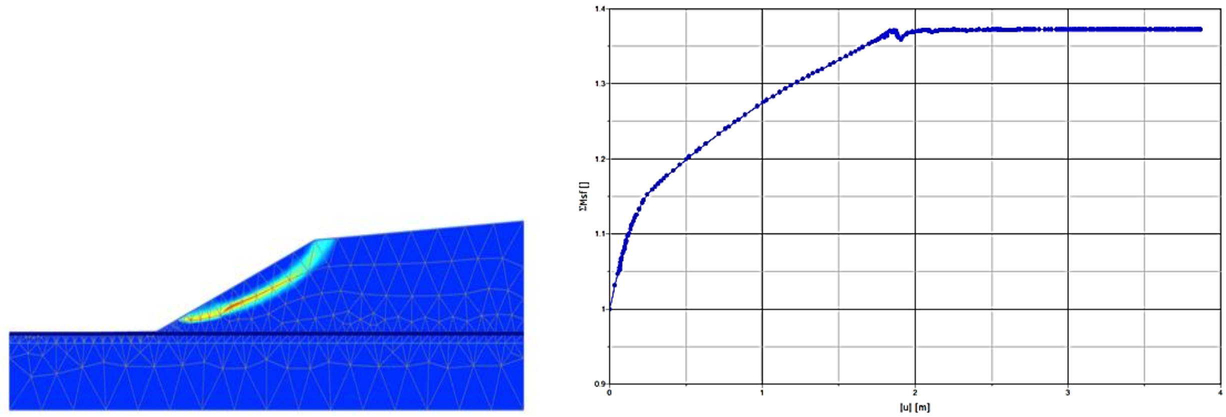


Figure 5.17: Relationship between the factor of safety of the peat embankment and the height of the berm.

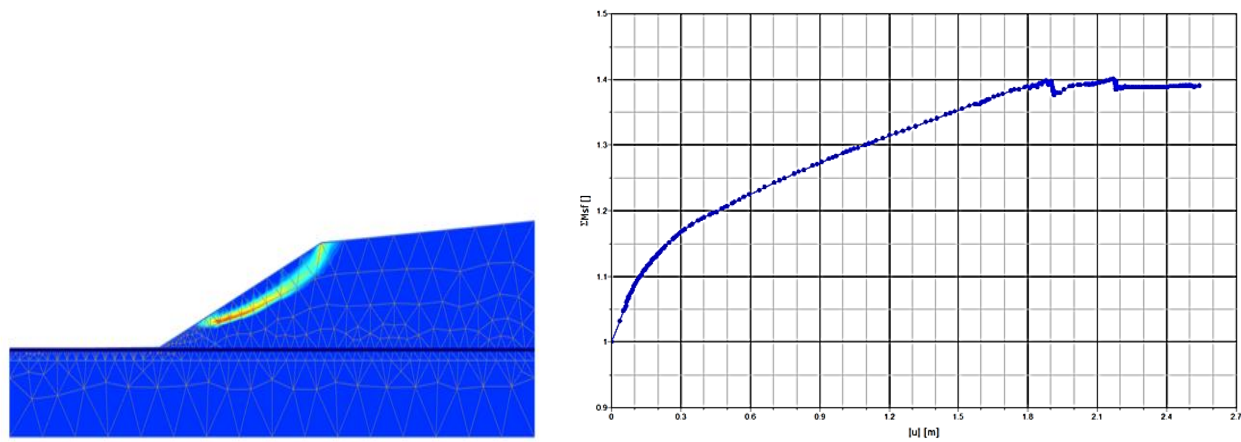
The maximal shear strains development of the embankment with the factor of safety curve are showed in Figure 5.18. It can be found that the volume of soil to be slipped included the toe of embankment when there is no berm but with an employment of the berm, the toe of the embankment has been "protected" by back berm effectively and the slipping surface changed and cross the top of back berm.



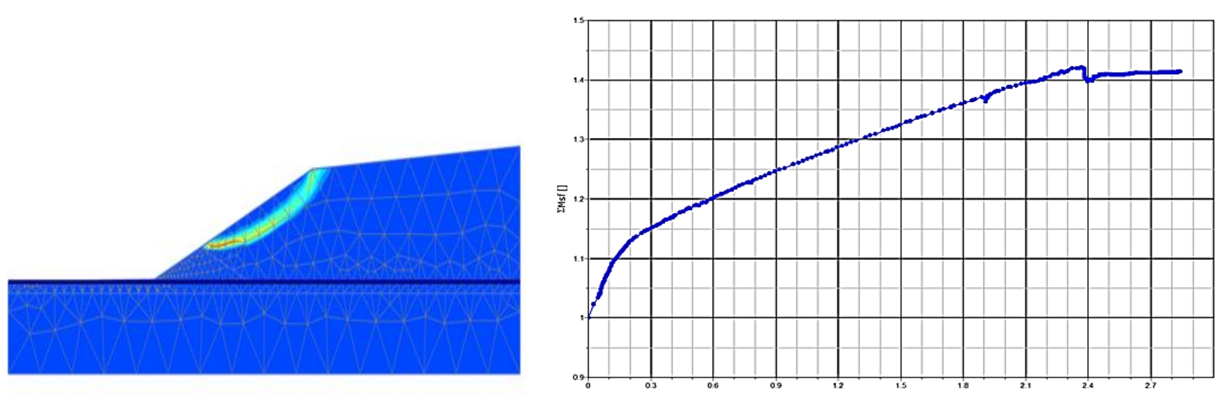
(a) $H_b = 0$



(b) H_b = 5m



(c) H_b = 10m



(d) H_b = 15m

Figure 5.18: Maximum shear strain development of the embankment with factor of safety curve.

Considering the undrained case, the factor of safety for profiles A, B, C are 0.25, 0.26 and 0.43 respectively. The calculated safety factor for the undrained condition has not satisfied the minimum safety factor required. The presence of the peat layer presents a pre-existing sliding surface that reduces the safety factor into such low values. The existence of the peat layer controls the shape of the failure surface. The failure surface passes through the peat layer because it is the weakest layer in the whole slope's soil profile. An employment of a berm composed of gravel with modification of the embankment height and placing of separate layers of sand material of thickness 1m each in the peat embankment help to increase the factor of safety of the embankment significantly. For profile A, the height of the embankment was reduced from 59m to 30m with a berm of height 11m composed of gravel with a cohesion of 1kpa and a friction angle of 38^0 and two separate layers of sand materials with a cohesion of 3.5kpa and a friction angle of 36^0 of thickness 1m each placed in the peat embankment as shown in figure 5.19a, the factor of safety increases from 0.25 to 1.2 as shown in figure 5.19b.

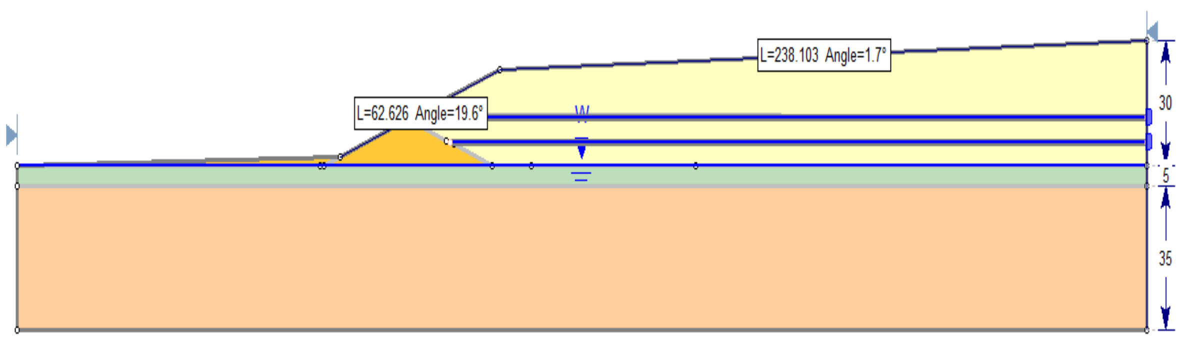
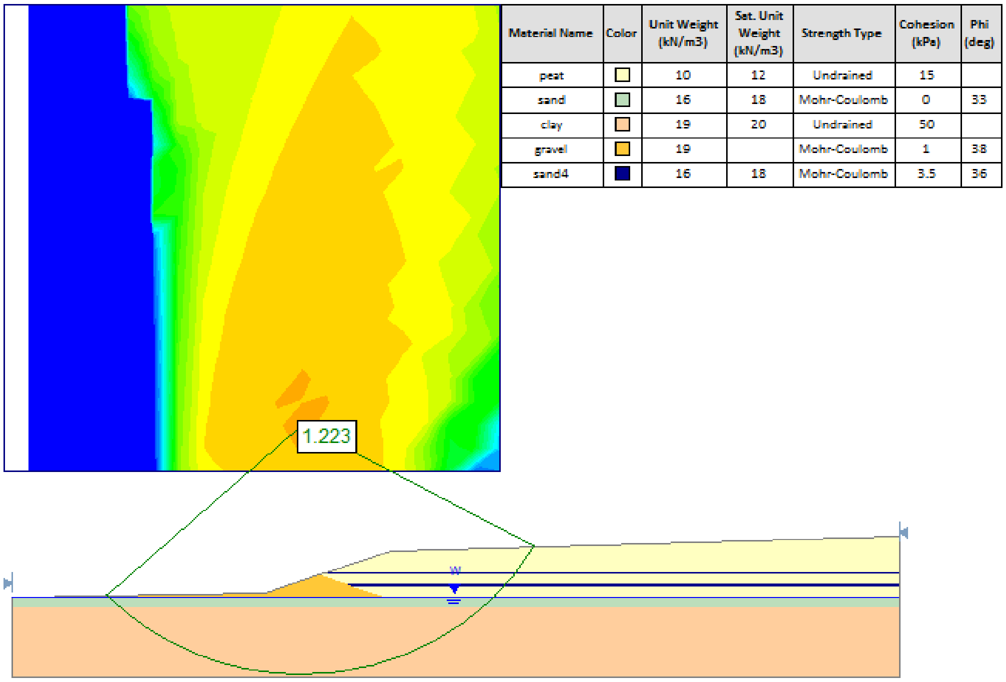


Figure 5.19a: Stablised model for the undrained case for profile A.



Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	12	Undrained	15	
sand		16	18	Mohr-Coulomb	0	33
clay		19	20	Undrained	50	
gravel		19		Mohr-Coulomb	1	38
sand4		16	18	Mohr-Coulomb	3.5	36

Figure 5.19b: Stablity analysis for the undrained case for profile A.

For profile B, the height of the embankment was reduced from 55m to 27m with a berm of height 16m composed of gravel with a cohesion of 1kpa and a friction angle of 38^0 and three separate layers of sand materials with a cohesion of 3.5kpa and a friction angle of 36^0 of thickness 1m each placed in the peat embankment as shown in figure 5.20a, the factor of safety increases from 0.26 to 1.1 as shown in figure 5.20b.

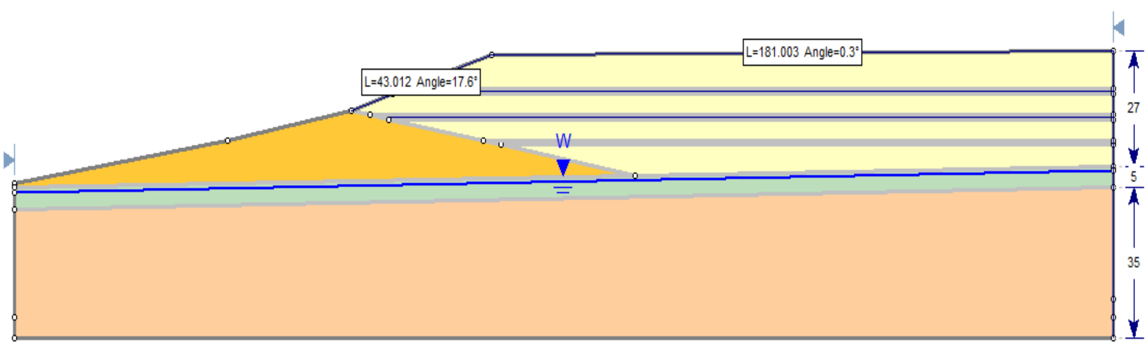
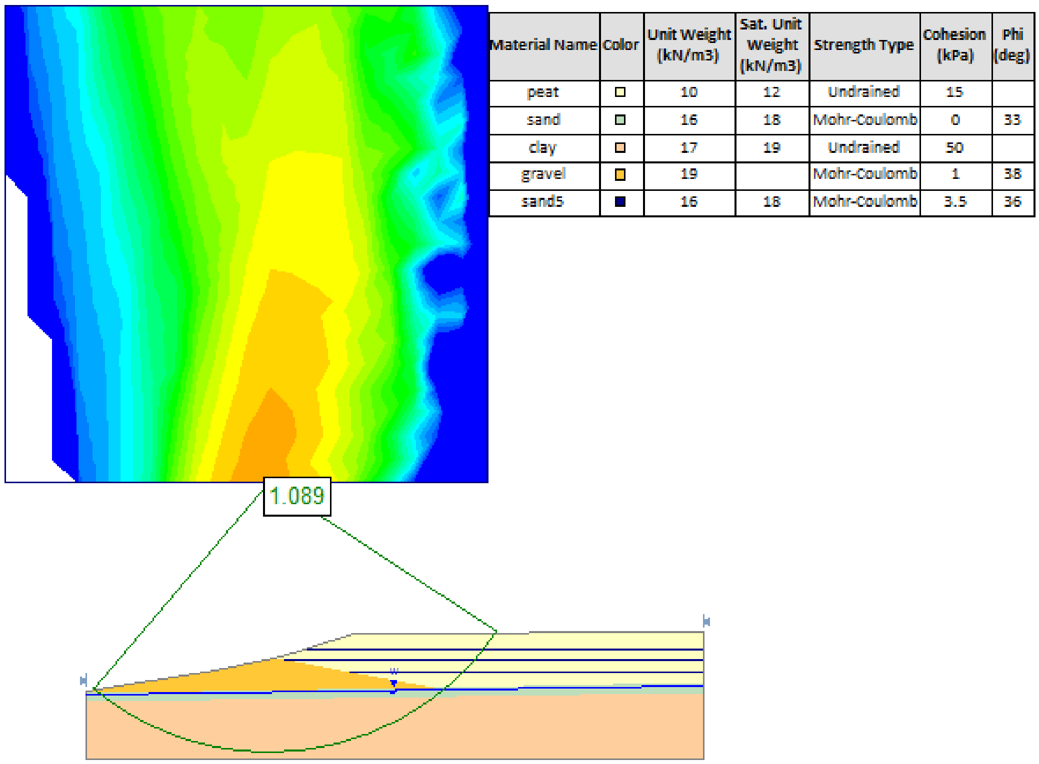


Figure 5.20a: Stablised model for the undrained case for profile B.



Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	12	Undrained	15	
sand		16	18	Mohr-Coulomb	0	33
clay		17	19	Undrained	50	
gravel		19		Mohr-Coulomb	1	38
sand5		16	18	Mohr-Coulomb	3.5	36

Figure 5.20b: Stablity analysis for the undrained case for profile B.

For profile C, the height of the embankment was unaltered with a berm of height 14m composed of gravel with a cohesion of 1kpa and a friction angle of 38^0 and three separate layers of sand materials with a cohesion of 3.5kpa and a friction angle of 36^0 of thickness 1m each placed in the peat embankment as shown in figure 5.21a, the factor of safety increases from 0.43 to 1.1 as shown in figure 5.21b.

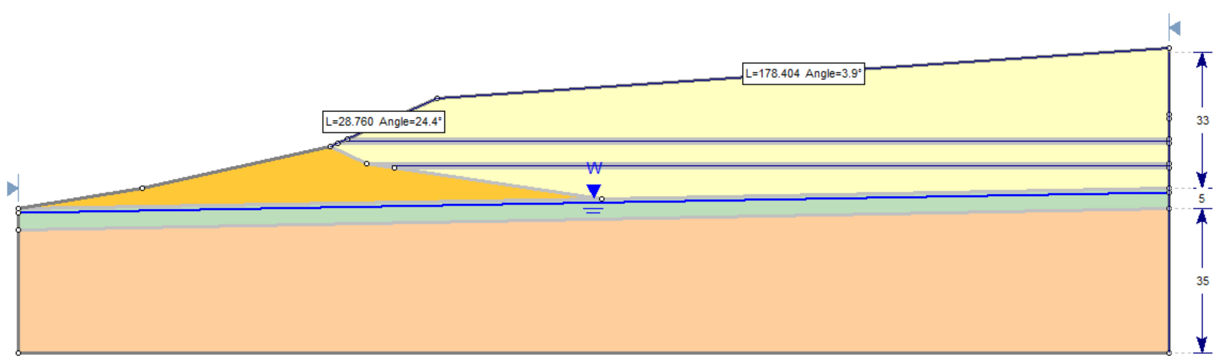
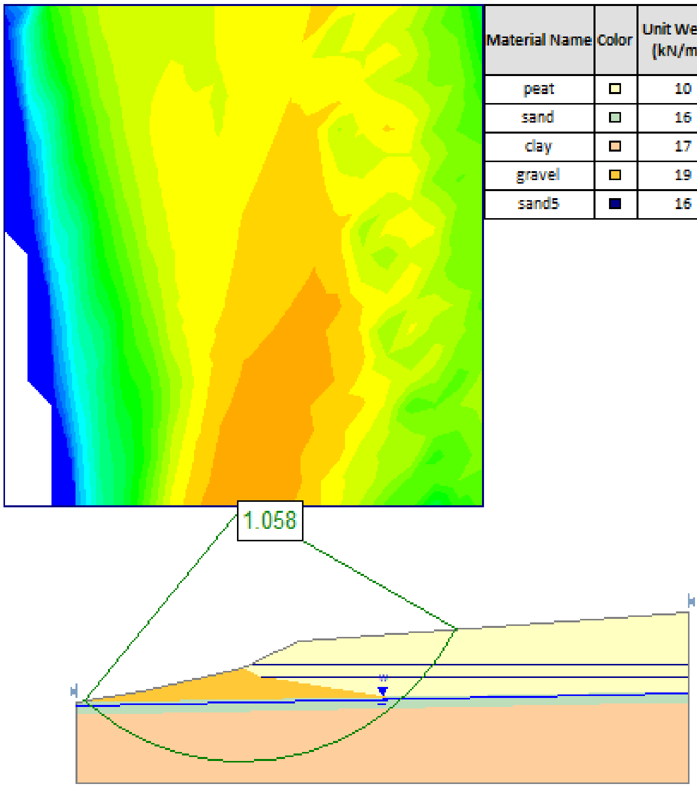


Figure 5.21a: Stablised model for the undrained case for profile C.



Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	12	Undrained	15	
sand		16	18	Mohr-Coulomb	0	33
clay		17	19	Undrained	50	
gravel		19		Mohr-Coulomb	1	38
sand5		16	18	Mohr-Coulomb	3.5	36

Figure 5.21b: Stablity analysis for the undrained case for profile C.

5.7 MODELLING WITH GEOGRID

Geogrid is a synthetic planar structure formed by a regular network of tensile strength elements with apertures of sufficiently large size to allow for interlocking with the surrounding soil so as to perform the primary function of reinforcement. The most critical slope is profile A where the factor of safety is below the minimum requirement of stability. Geogrid employment in this critical slope will help to enhance the stability of the slope. The reinforcement geometrical and mechanical characteristics obviously influence slope performance, both with regard to stability conditions and to global displacements. In the numerical analyses the reinforcements are modelled as flexible elastic elements that can sustain only tensile forces (no compression). The only property to define is the axial stiffness EA, that is the ratio of the axial force per unit width and the axial strain. Different values of the axial stiffness can be so determined by the knowledge of the normalized isochronous curve of the reinforcement, having defined the ultimate tensile strength. As far as soil-geogrid interaction is concerned, around the reinforcements set of interfaces (suited to model bond mechanisms) and refinement have been applied. Geogrid with tensile stiffness of EA=50 kN/m is selected for this study. A soil interface with strength reduction factor of R_{inter} = 0.7 were employed around the geogrid to model the soil-structure interaction. The length of the geogrid varies from 10m to 130m with a spacing of 2m. Figure 5.22shows the generated mesh by the model whiles Figure 5.23 shows the results from the geogrid modelling and Table 5.8 also gives a summary of the factor safety both for with and without geogrid.

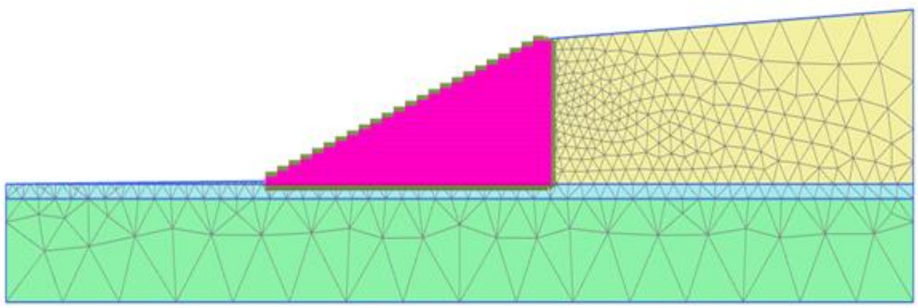


Figure 5.22: Generated mesh of the model.

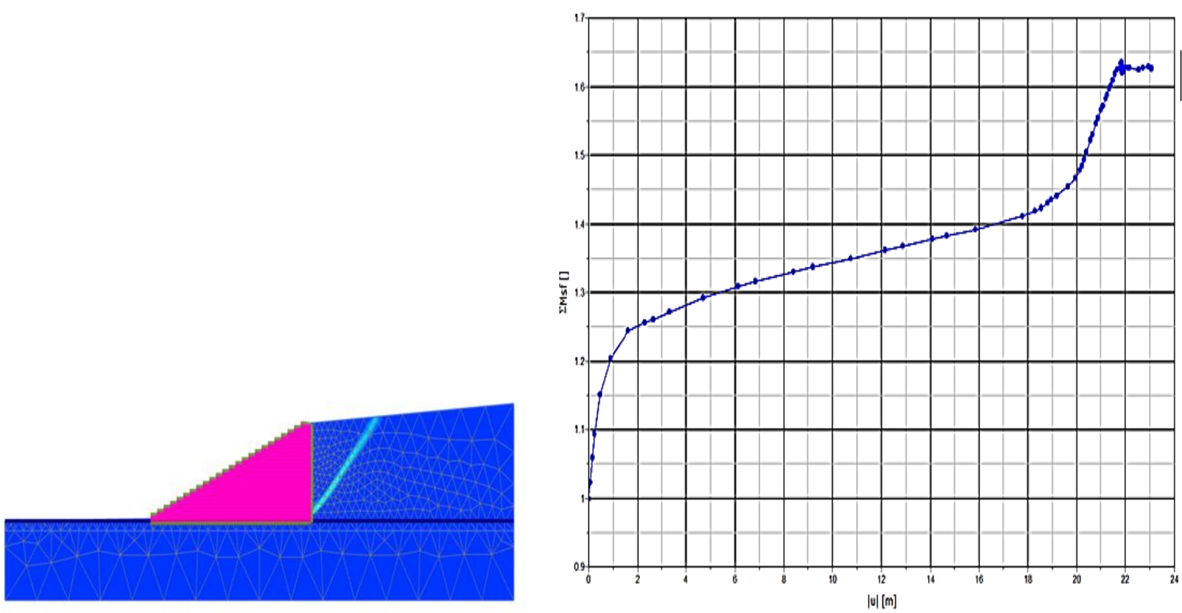


Figure 5.23: Modelling of the embankment with geogrid and the factor of safety curve.

Table 5.8: Comparisons of factor of safety results.

Profile	FS (Without geogrid)	FS (With geogrid)
A	1.33	1.63

Using the geogrid between the top of the major slope of the embankment and the embankment base decrease vertical and horizontal displacement. As decreasing horizontal displacements of embankment has important role in stability of embankment. For the stability analysis of the profile A, the factor of safety increases by almost 25% with the installations of the geogrid.

The slide analysis also indicated an increment of the factor of safety with the employment of geogrid with same specifications as in the plaxis analysis. The factor of safety without geogrid was 1.44 and the factor of safety with geogrid is also given as 1.74 as show in figure 5.24.

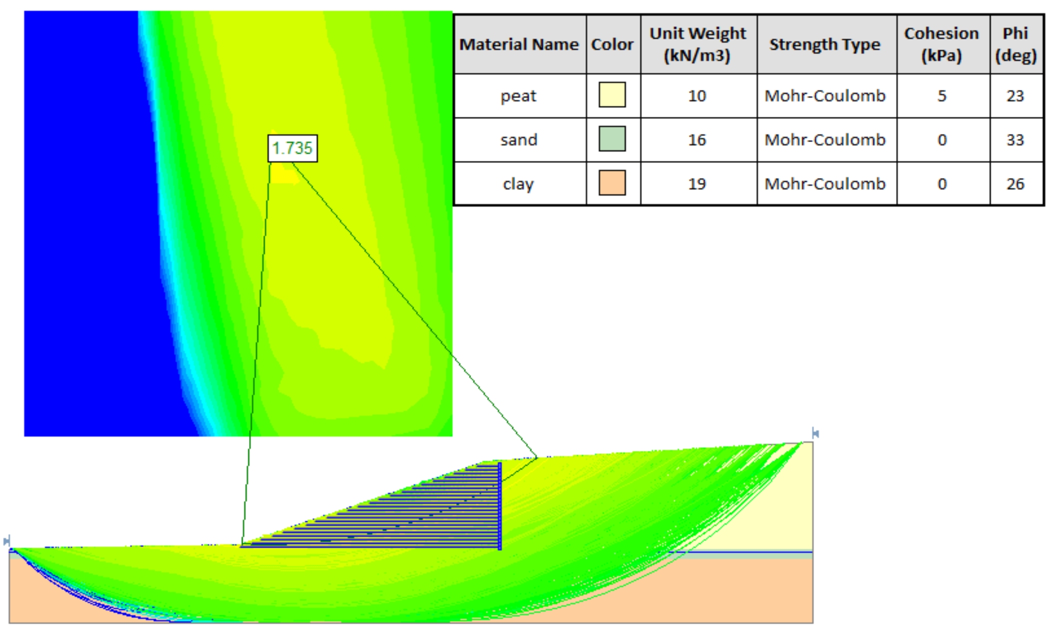
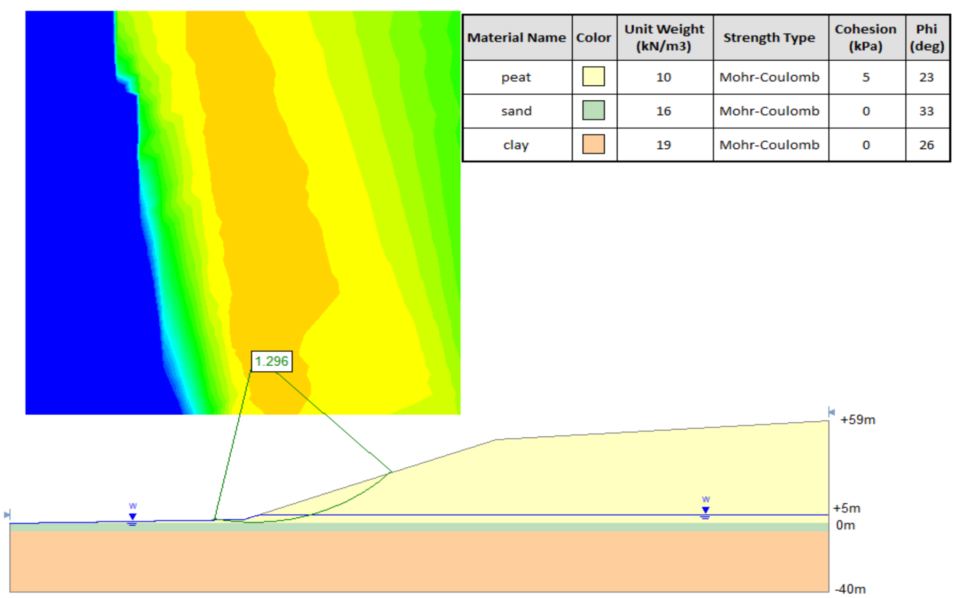


Figure 5.24: Slide analysis of the peat embankment with geogrid.

5.8 EFFECT OF GROUNDWATER ON THE PEAT EMBANKMENT

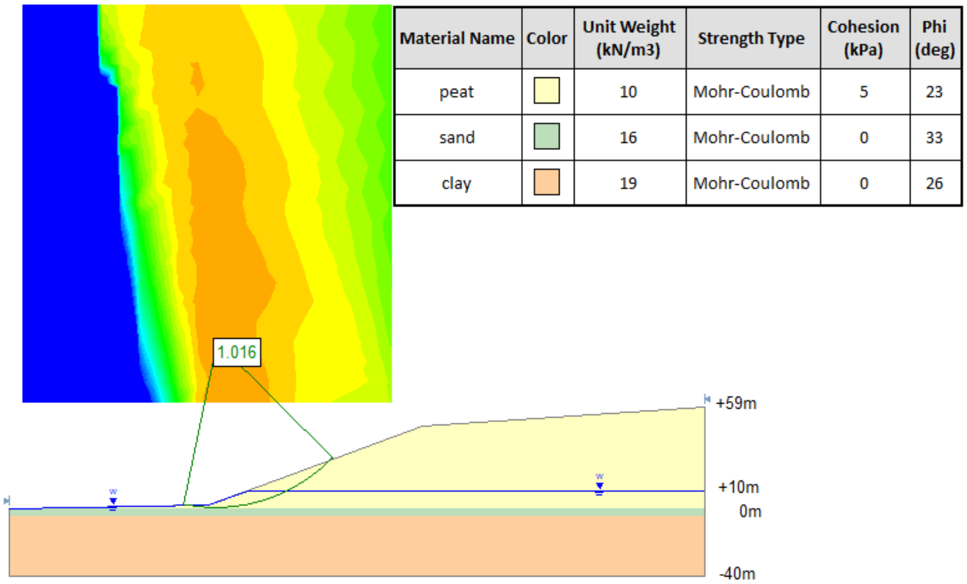
Groundwater exists nearly everywhere beneath the surface of the earth. It is water that fills the pore spaces between grains in rock or soil or fills fractures in the rock. The water table is the surface that separates the saturated zone below, wherein all pore space is filled with water from the unsaturated zone above. Changes in the level of the water table occur due changes in rainfall. The water table tends to rise during wet seasons when more water infiltrates into the peat and falls during dry seasons when less water infiltrates. Such changes in the level of the water table can have effects on the stability of the peat embankment.

Considering profile A, raising the groundwater level from -1m to +5m (within the peat embankment) the factor of safety margin reduces as a result of increase in the pore water pressure and above +10m the factor of safety is less than one as a result of saturation of the peat material with the angle of repose reduced to very small value. Figure 5.25 shows the stability assessments of the peat embankment by slide based on groundwater level change and also figure 5.26 also shows a graph of effects of variations of the groundwater level on the factor of safety.



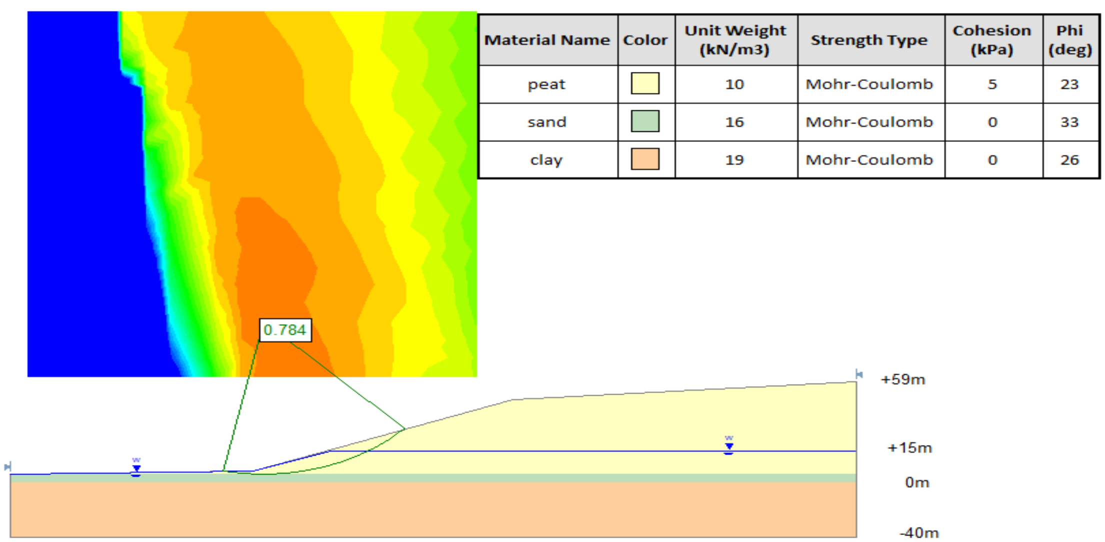
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

a) GWL = +5m



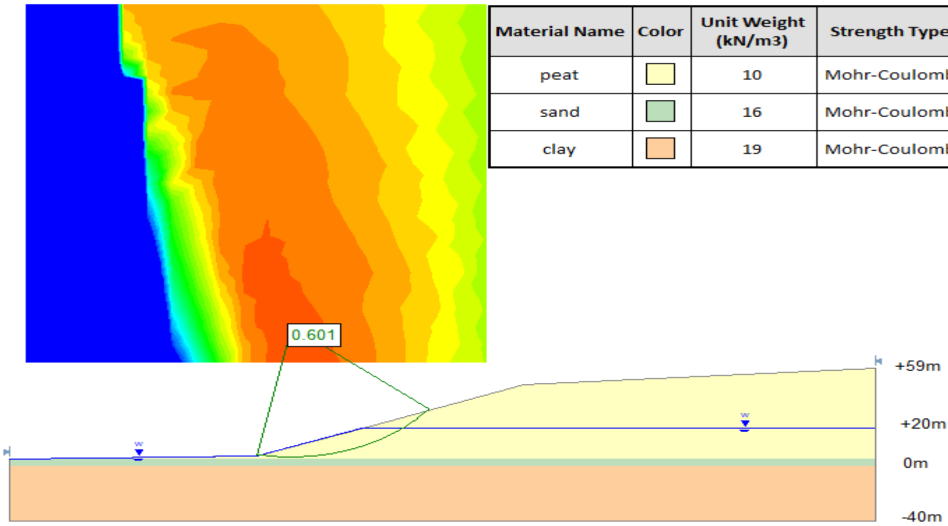
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

b) GWL = +10m



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

c) GWL = +15m



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

d) GWL = +20m

Figure 5.25: Stability assessments of the peat embankment based on the influence of groundwater level.

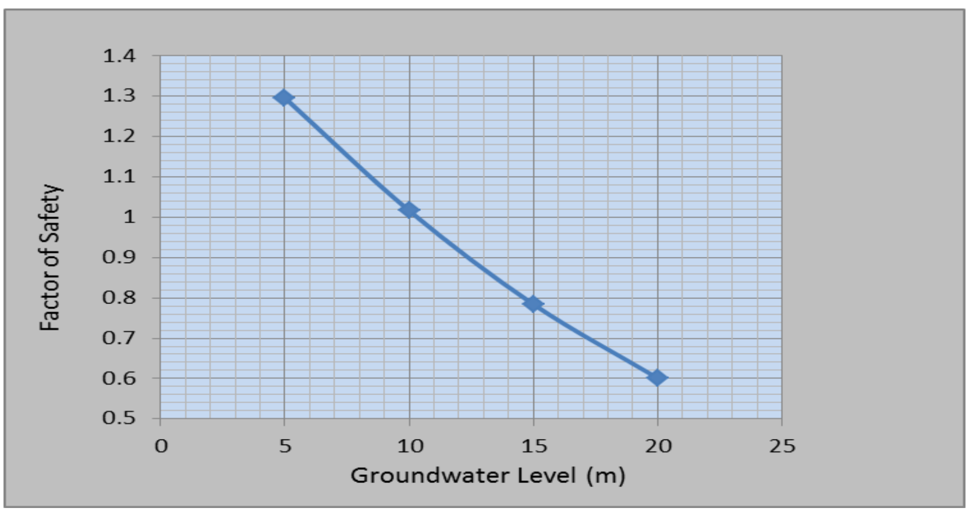
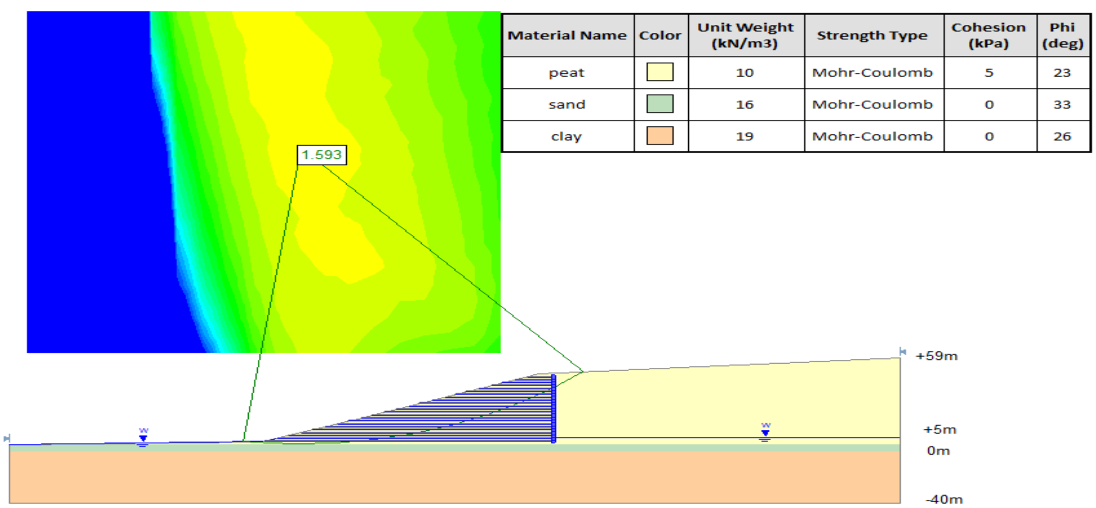
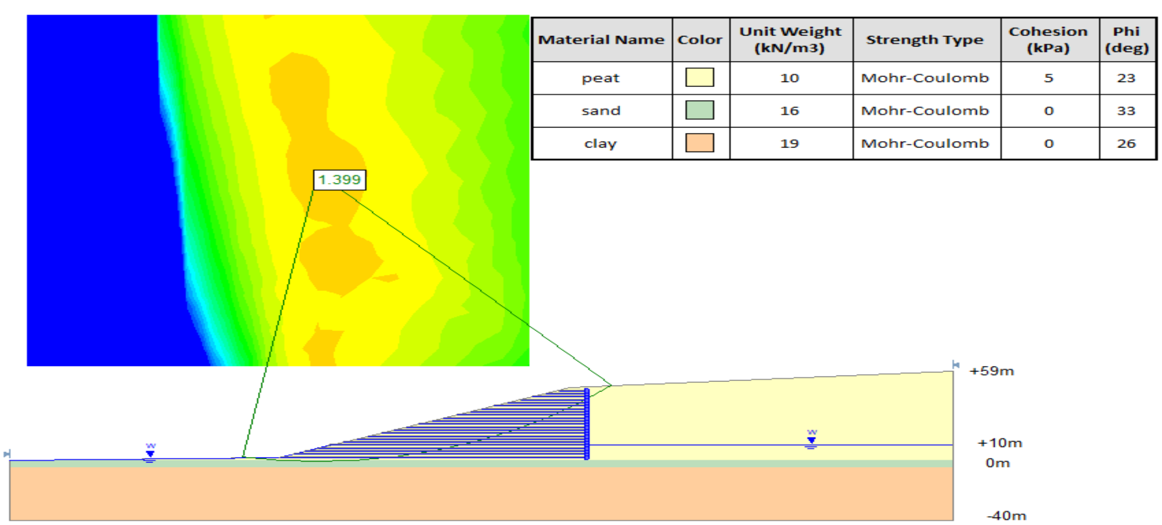


Figure 2.26: Effects of variations of the groundwater level of the peat embankment on the factor of safety.

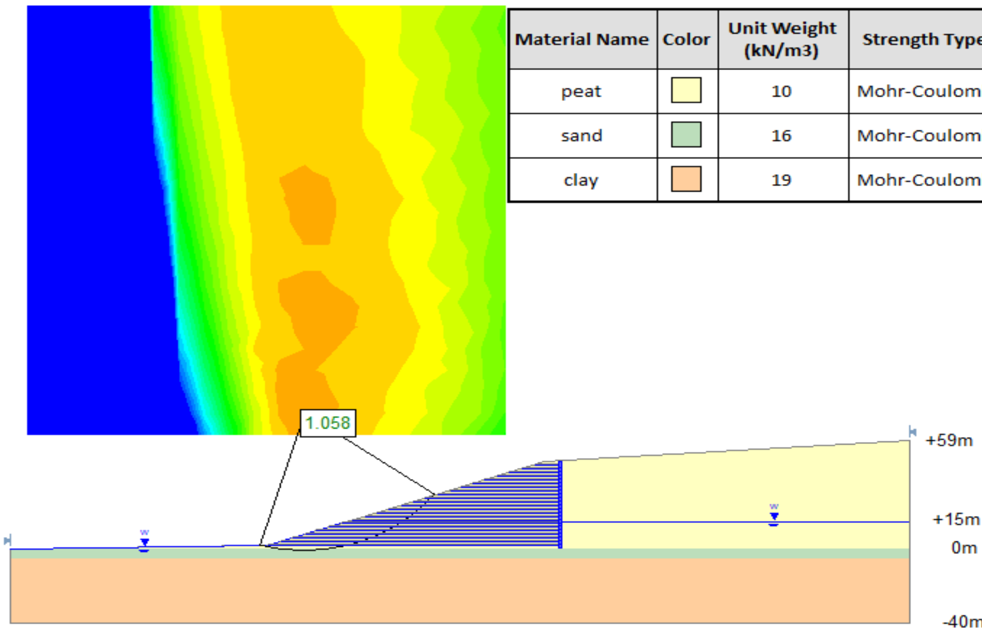
With an employment of geogrid reinforcement with tensile stiffness of EA=50 kN/m, strip coverage of 100%, pullout strength adhesion of 5kpa, pullout strength friction angle of 30^0 and length of the geogrid with variation from 10m to 130m with a spacing of 2m, it was realized that the factor of safety is less than one when the ground water level was above +15m and the factor of safety below +15m was much higher than the peat embankment without geogrid. The geogrid were embedded in thin layers of sand which improve the performance of the peat embankment and also provide drainage paths in the lateral extent preventing pore water pressure generations. Figure 5.27 shows stability assessments of the peat embankment with geogrid based on the influence of changes in the groundwater level. Table 5.9 gives a summary of the factor of safety of the peat embankment with and without geogrid based on the variations of the water table and figure 5.28 also shows a graph indicating the factor of safety with and without geogrid based on the influence of the groundwater level.



a) GWL = +5m

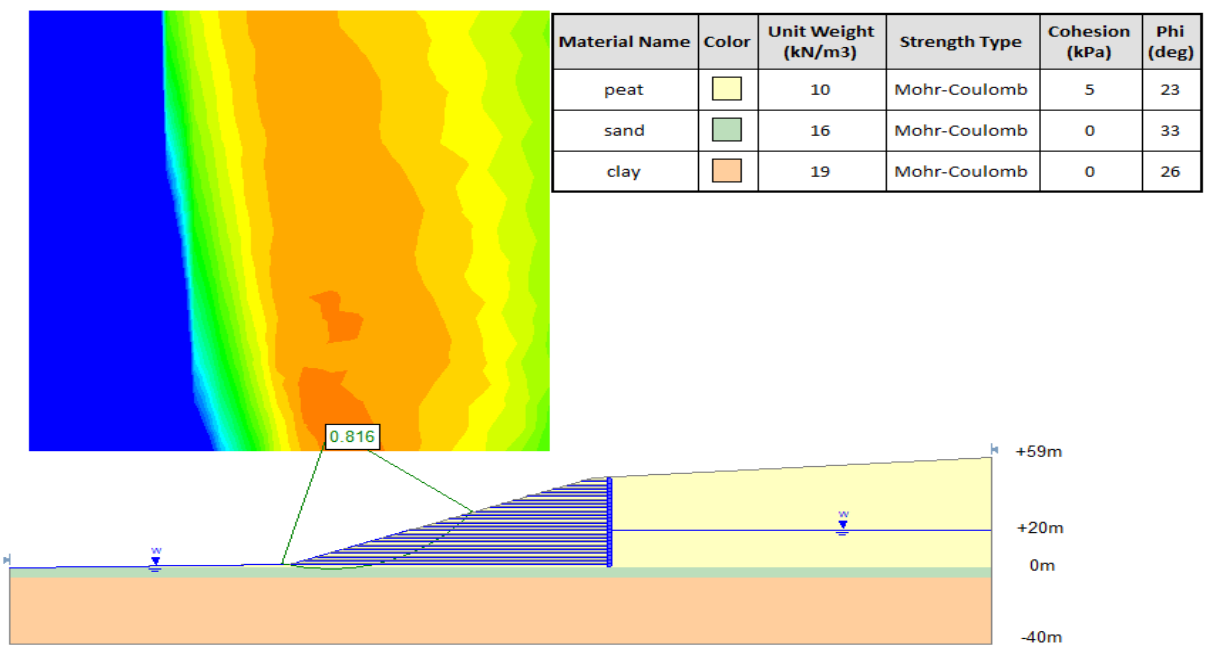


b) GWL = +10m



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

c) GWL = +15m



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

d) GWL = +20m

Figure 5.27: Stability assessments of the peat embankment with geogrid based on the influence of changes in the groundwater level.

Table 5.9: Summary of the factor of safety of the peat embankment with and without geogrid based on the variations of the groundwater level.

Ground water level (GWL)	FS (Without geogrid)	FS (With geogrid)
+5m	1.29	1.60
+10m	1.02	1.40
+15	0.78	1.06
+20m	0.60	0.82

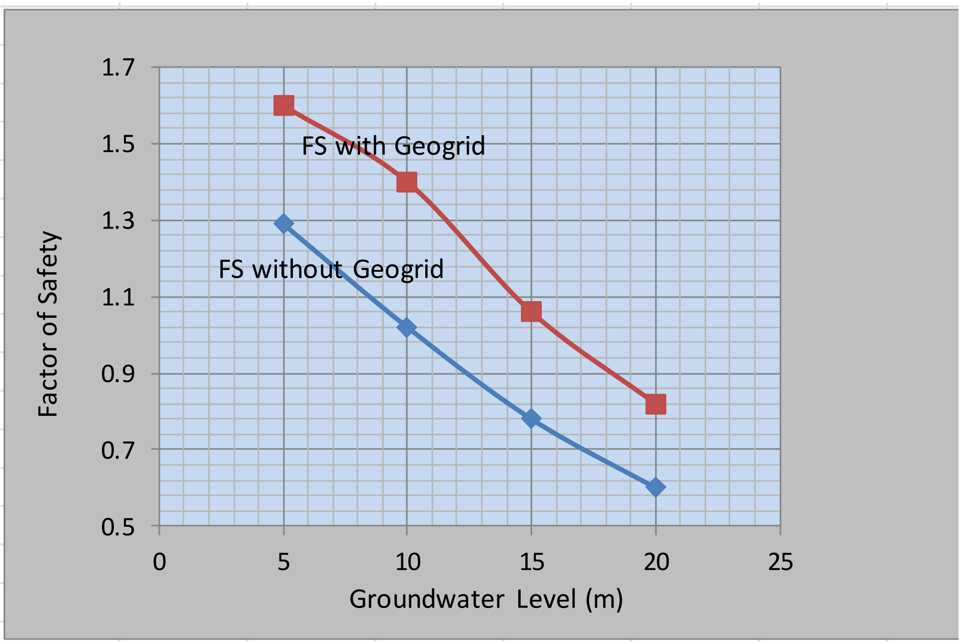


Figure 5.28: Influence on the factor of safety with and without geogrid based on the changes in the groundwater level.

CHAPTER SIX

CONCLUSION AND RECOMMENDATION

6.1 CONCLUSION

The slide software based on limit equilibrium methods and plaxis which is based on finite elements method were employed for the slope stability evaluation. The factor of safety based on undrained conditions estimated by the slide software for profiles A, B, C are 0.25, 0.26 and 0.43 respectively. The calculated safety factor for the undrained condition has not satisfied the minimum safety factor required. The presence of the peat layer which is characterised by high in-situ void ratio, high water content and high values of compression index presents a pre-existing sliding surface causing instability of the peat mass embankment. The existence of the peat layer controls the shape of the failure surface. An employment of a berm composed of gravel with a modification of the embankment height and placing of separate layers of sand material of thickness 1m each in the peat embankment help to increase the factor of safety of the embankment for the undrained case significantly. The factor of safety for profile A increases from 0.25 to 1.2, profile B also increase from 0.26 to 1.1 and profile C also increases from 0.43 to 1.1.

The factor of safety based on drained conditions estimated by the slide software for profiles A, B, C are 1.44, 1.53 and 1.51 respectively and the plaxis are also 1.33, 1.44 and 1.66 respectively. There exist a linear relationship between the factor of safety and the cohesion of the peat material which was exhibited by the drained stability analysis by both slide and plaxis software and is given by the equation FS = 0.03c + 1.21 for slide and FS = 0.03c + 1.15 for plaxis where c is the cohesion of the peat material for a frictional angle of 23^0. Therefore with an increment in the cohesion of the peat material, the factor of safety also increases. There also exist a linear relationship between the factor of safety and the frictional angle of the peat material and is given by the equation FS = 0.06$\boldsymbol{\phi}$ – 0.01 for slide and FS = 0.04$\boldsymbol{\phi}$ + 0.48 for plaxis where $\boldsymbol{\phi}$ is the frictional angle of the peat material for a cohesion of 5kpa. Therefore an increment in the frictional angle of the peat material, the factor of safety also increases.

Berm made of gravel was employed for the drained case for profile A which have the least factor of safety to provide a greater resisting moment along the developing slip mechanism of the peat embankment. With no berm in the peat embankment the factor of safety was 1.33, for a berm of height 5m, 10m, 15m the factor of safety were 1.37, 1.39 and 1.42 respectively. The stability of the embankment increases steadily with increase in the height of the back berm which indicates how effectively to enhance stability of embankment with back berm. It can be found that the volume of soil to be slipped included the toe of embankment when there is no berm but with an employment of the berm, the toe of the embankment has been "protected" by back berm effectively and the slipping surface changed and cross the top of back berm.

Results reveal that use of geogrid between the top of the major slope of the embankment and the embankment base decrease vertical and horizontal displacement in the drained case for profile A. As decreasing horizontal displacements of embankment has important role in stability of embankment. For the stability analysis of the profile A, using geogrid with tensile stiffness of EA=50 kN/m with lengths varying from10m to 130m and a spacing of 2m, the factor of safety by the plaxis increases from 1.33 to 1.63 and for the slide analysis also increases from 1.44 to 1.74. . The optimum place for geogrid layer is between bed and embankment. If the geogrid layer covers the bottom of embankment completely it will cause the least displacements.

As the groundwater level tends to rise during the wet seasons in the peat mass the factor of safety decreases as a result of increase in the pore water pressure which reduces the effective stress holding the peat particles together and the angle of repose reduced to very small value. From the studies it was realized that employment of geogrid embedded in thin layers of sand enhances the performance of the peat embankment and also provide drainage paths in the lateral extent preventing pore water pressure generations.

6.2 RECOMMENDATION

- Employment of full scale field and laboratory investigations and the results used for the stability evaluation of the peat materials.
- For peat materials containing organic layers, chemical soil treatment such as lime columns is considered an effective way to increase the slope safety factor and to increase the shear strength parameters in the failed region, but further parametric study, laboratory, and field tests should be performed in order to have a better view for stabilizing the soft soil with lime columns and to determine the different columns characteristics. Feasibility study should also be performed in such case.
- Careful and proper monitoring on the performance of the embankment during and after construction through instrumentation scheme.
- Detailed hydrological and hydrogeological assessment of the project site.
- Full time proper supervision of the construction works by qualified personnel/engineer.

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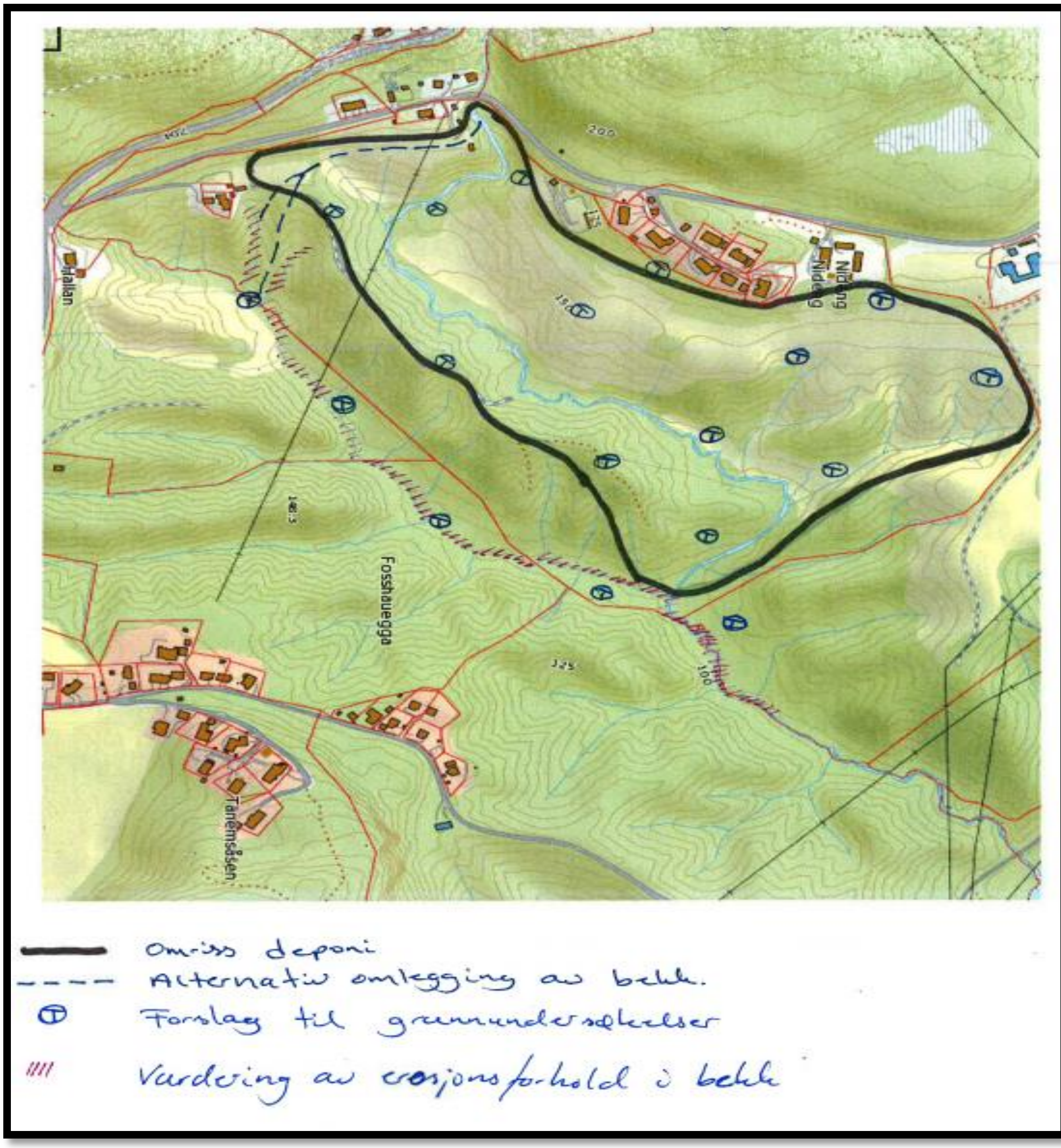
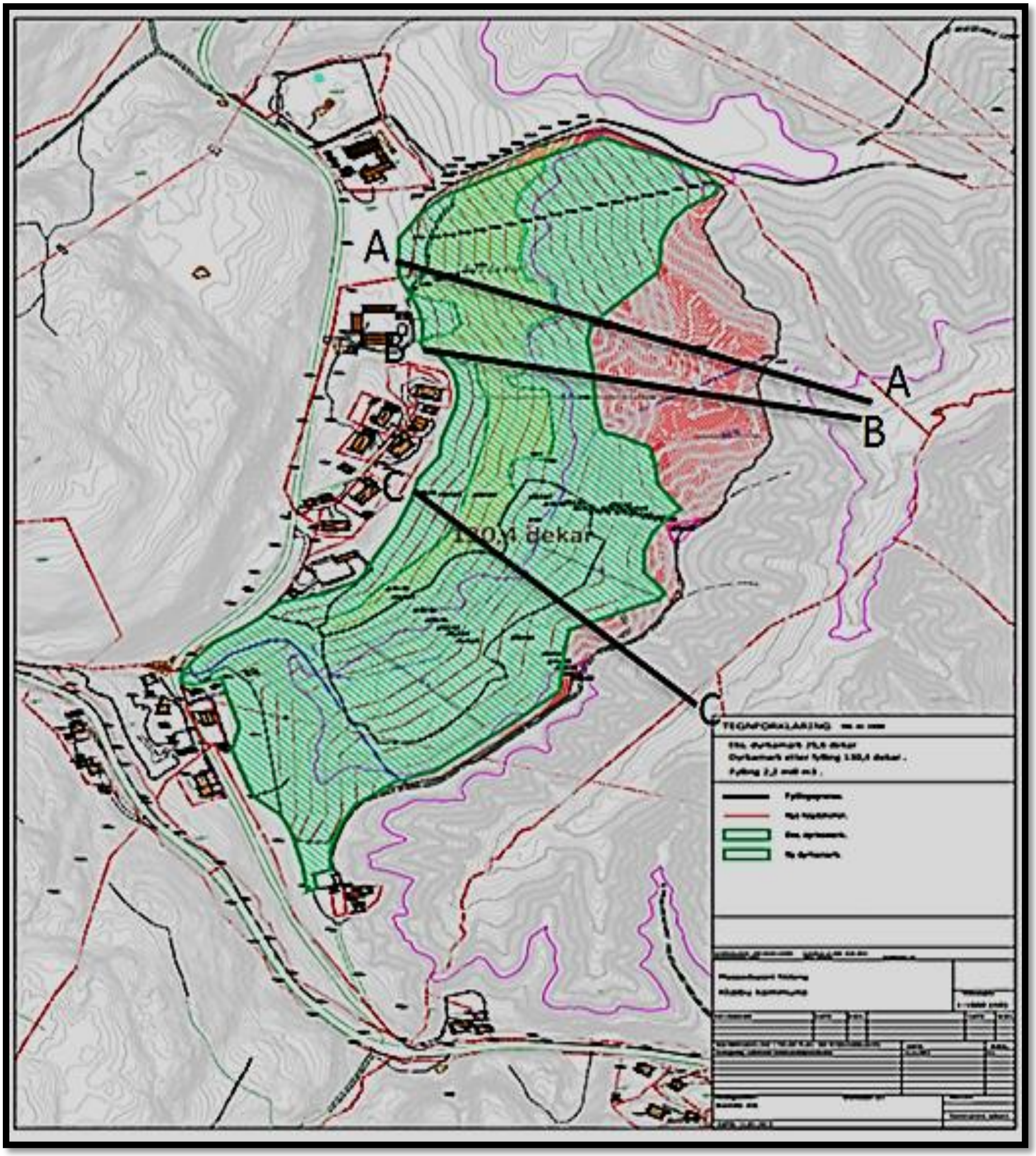
APPENDICES

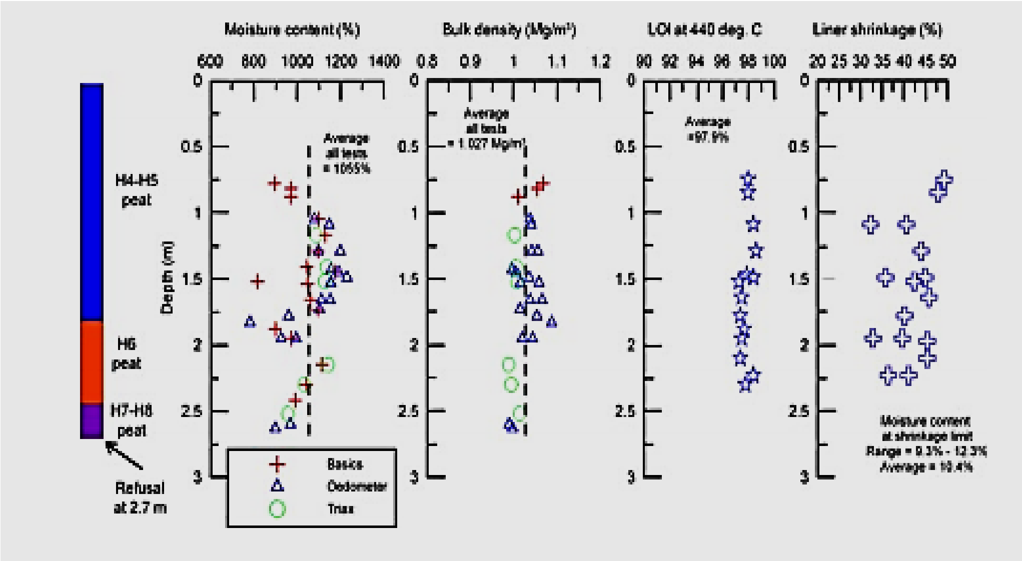
APPENDIX A1 – PROJECT SITE LAYOUT WITH PLANNED INVESTIGATIONS AND DESIGNS.



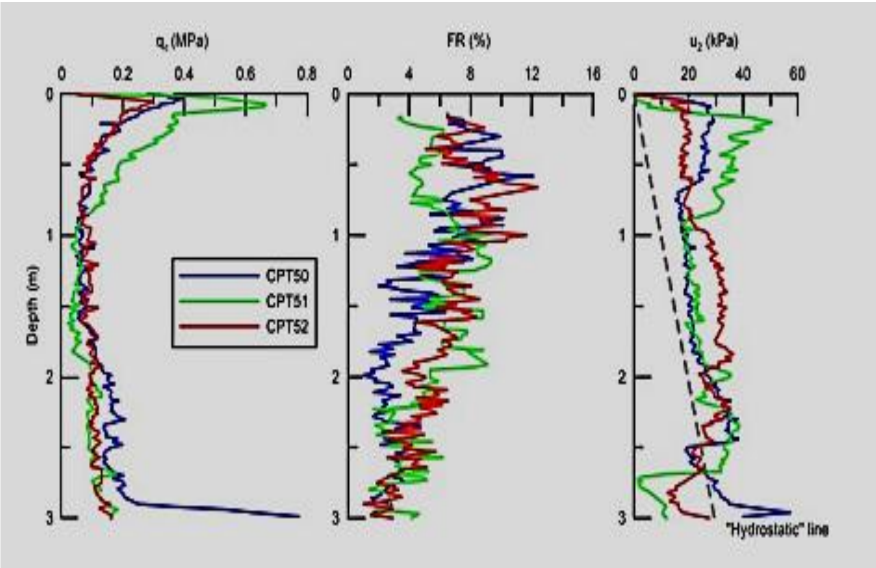
APPENDIX A2 –TERRAIN MODEL WITH THE VARIOUS PROFILES.



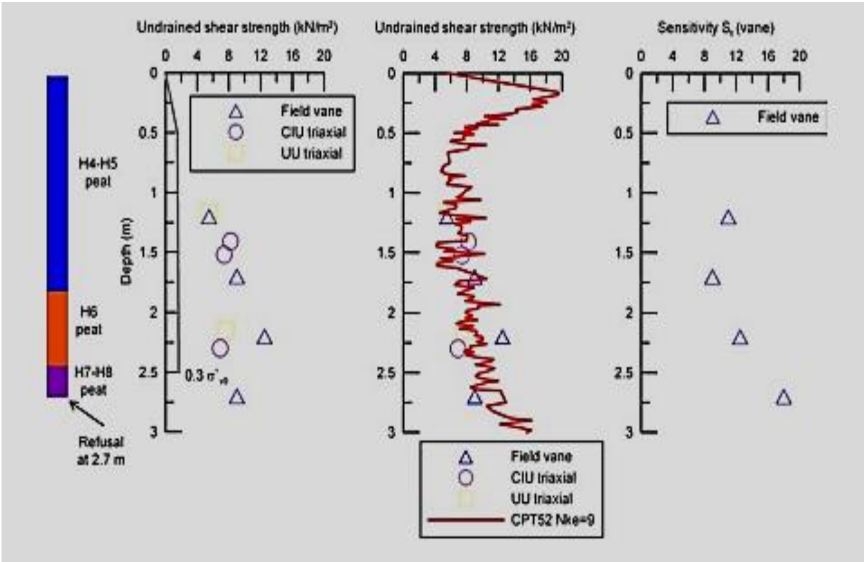
APPENDIX B1 – TYPICAL INDEX PROPERTIES AND CLASSIFICATION OF PEAT.

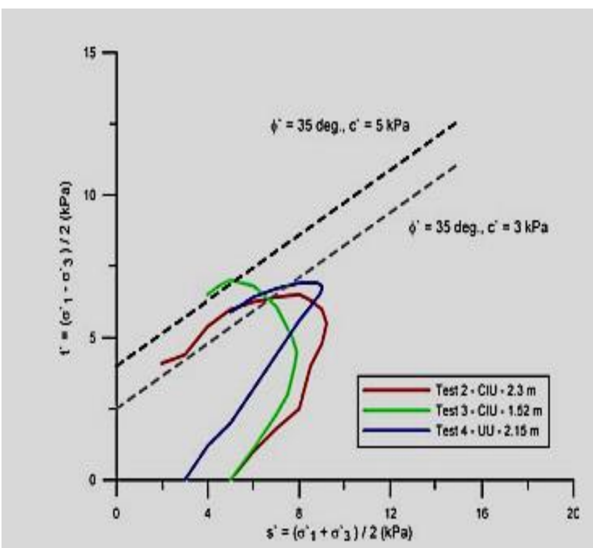


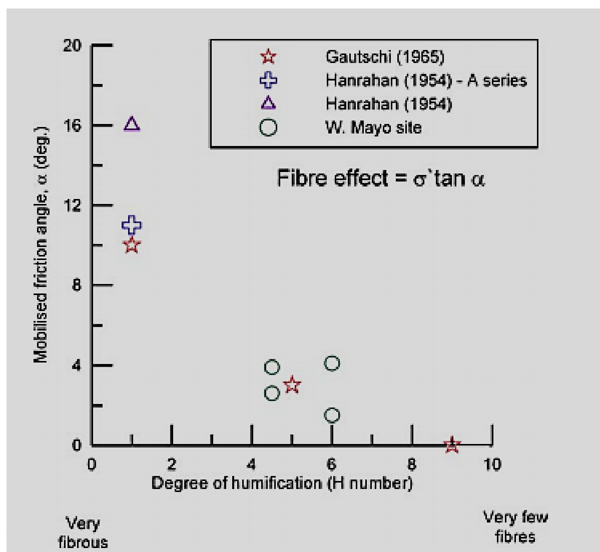
APPENDIX B2 – TYPICAL CPTU TEST RESULTS IN PEAT.



APPENDIX B3 – TYPICAL UNDRAINED SHEAR STRENGTH OF PEAT.



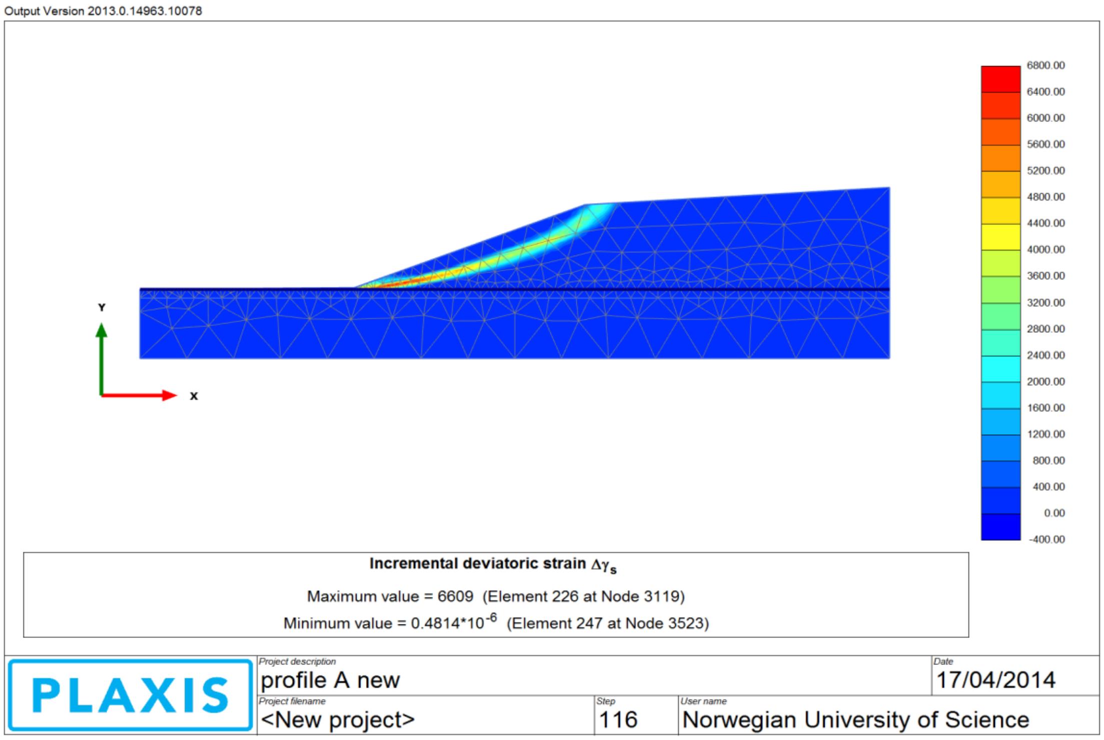


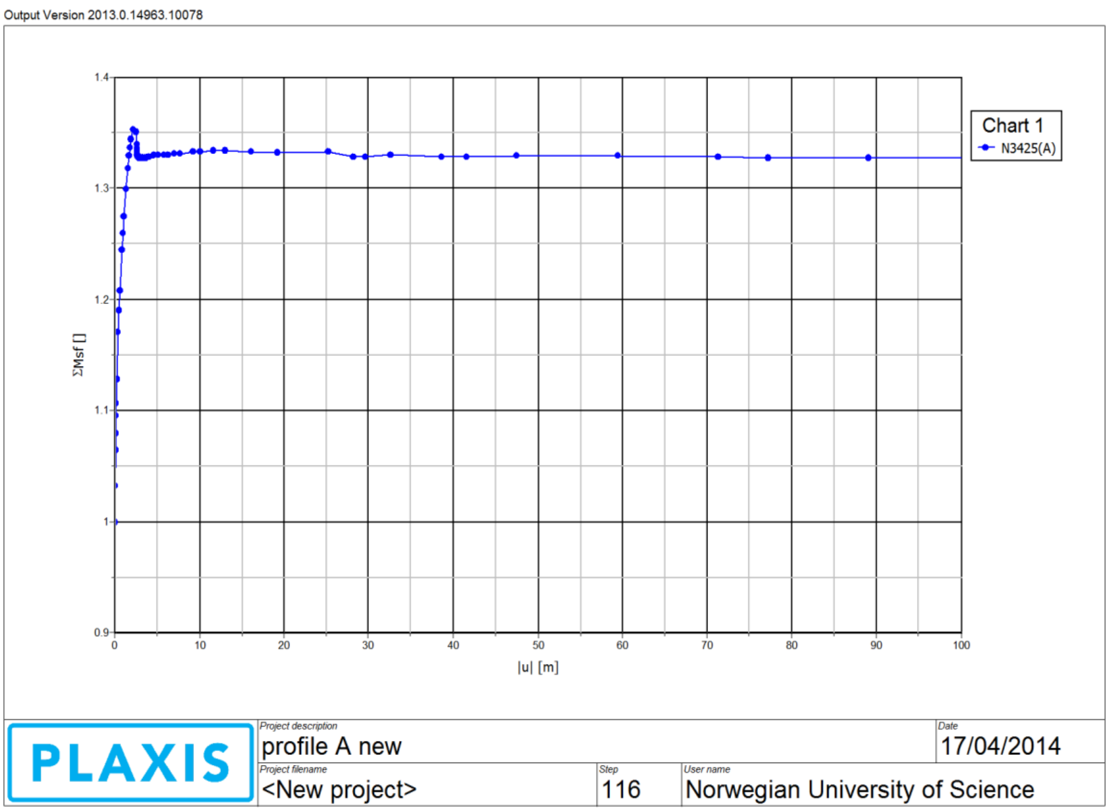


APPENDIX B4 – TYPICAL ENGINEERING PROPERTIES OF SOIL.

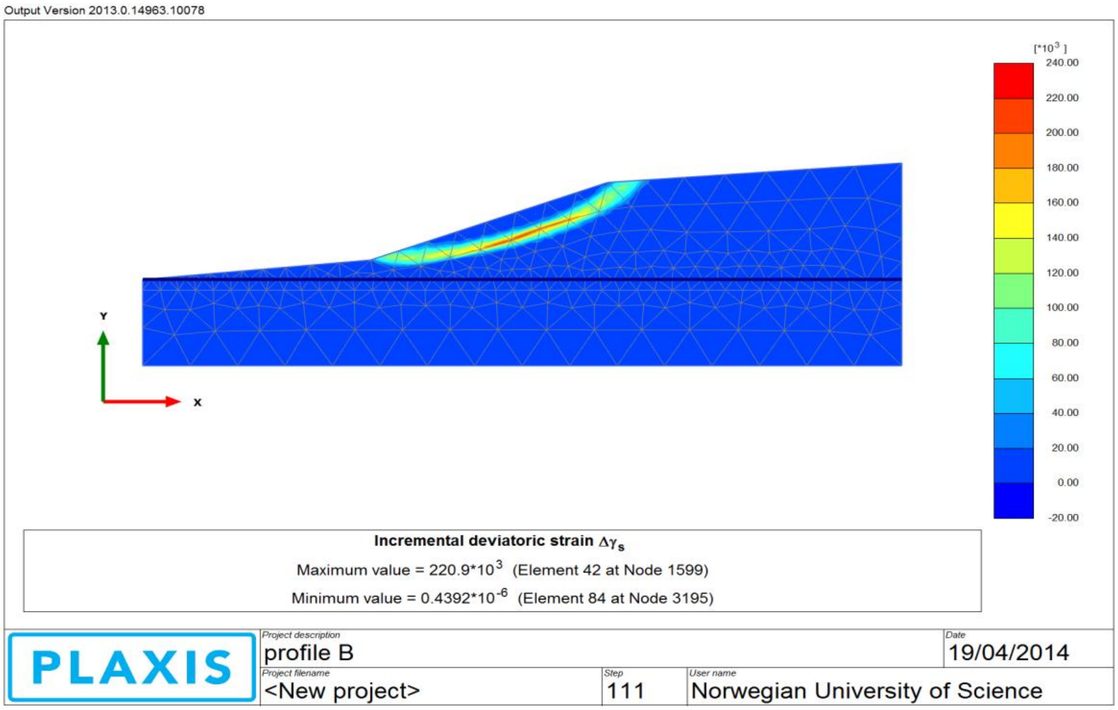
Soil Type	γ_{sat} (kN/m^3)	E (MPa)	ν	a (kPa)	φ(°)	k (m/year)	c_u (kpa)	Reference
Clay	19	1-3	0.33	0-20	26	10^{-5} - 10^{-2}	15-50	[38]
Sand	17-18	15-50	0.3	0-10	33-36	10^{2} - 10^{4}		[38]
silt	18-19	20	0.33	0-10	31-33	10^{-3} – 10^{-2}	25-70	[38]
peat	8 - 12	0.3	0.33	0	23	10^{-2} -10^{-1}	15	[42]
Gravel	18-19	70-170	0.33	0-10	36-38	10^{3} - 10^{5}		[38]

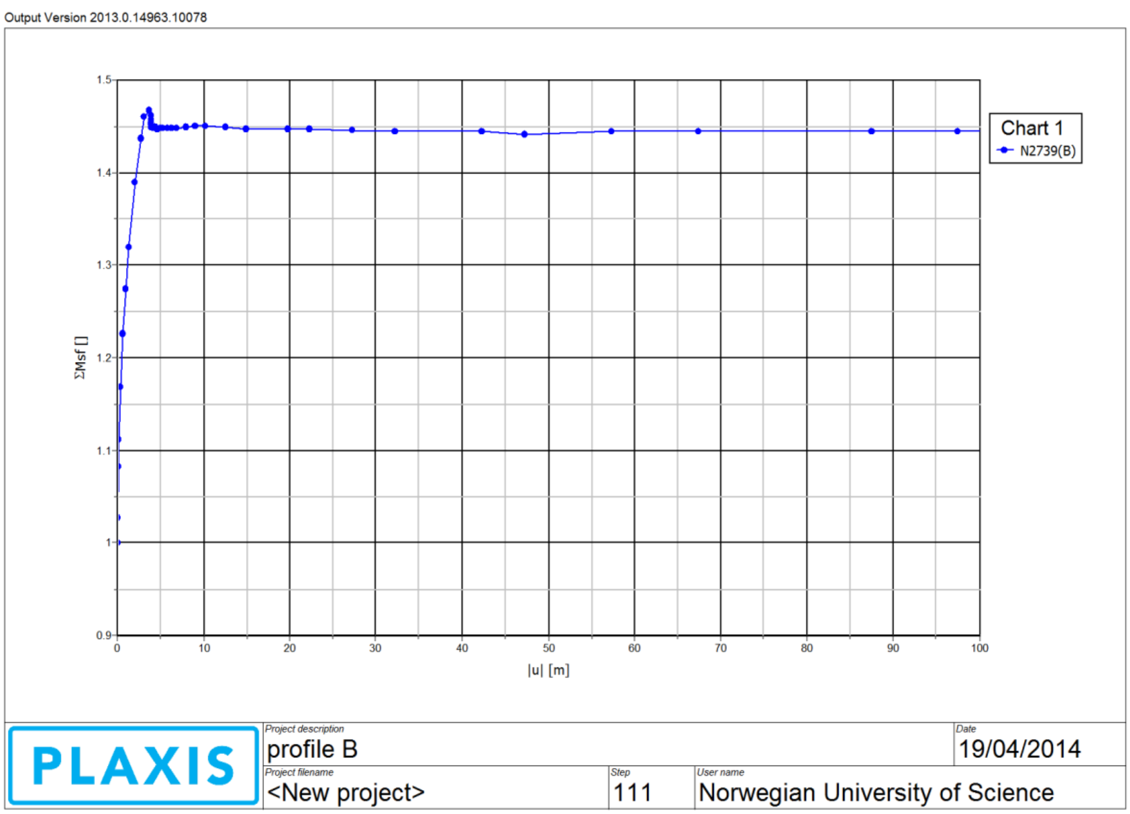
APPENDIX C1 – PLAXIS RESULTS FOR PROFILE A (DRAINED CONDITIONS)



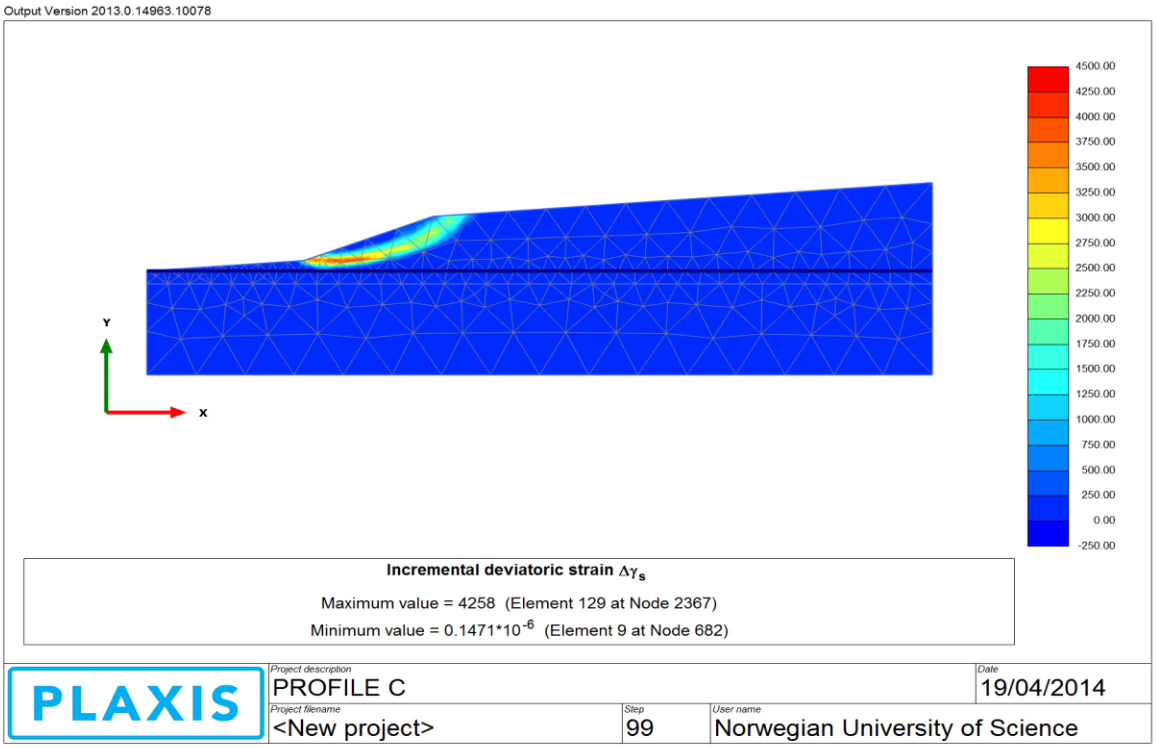


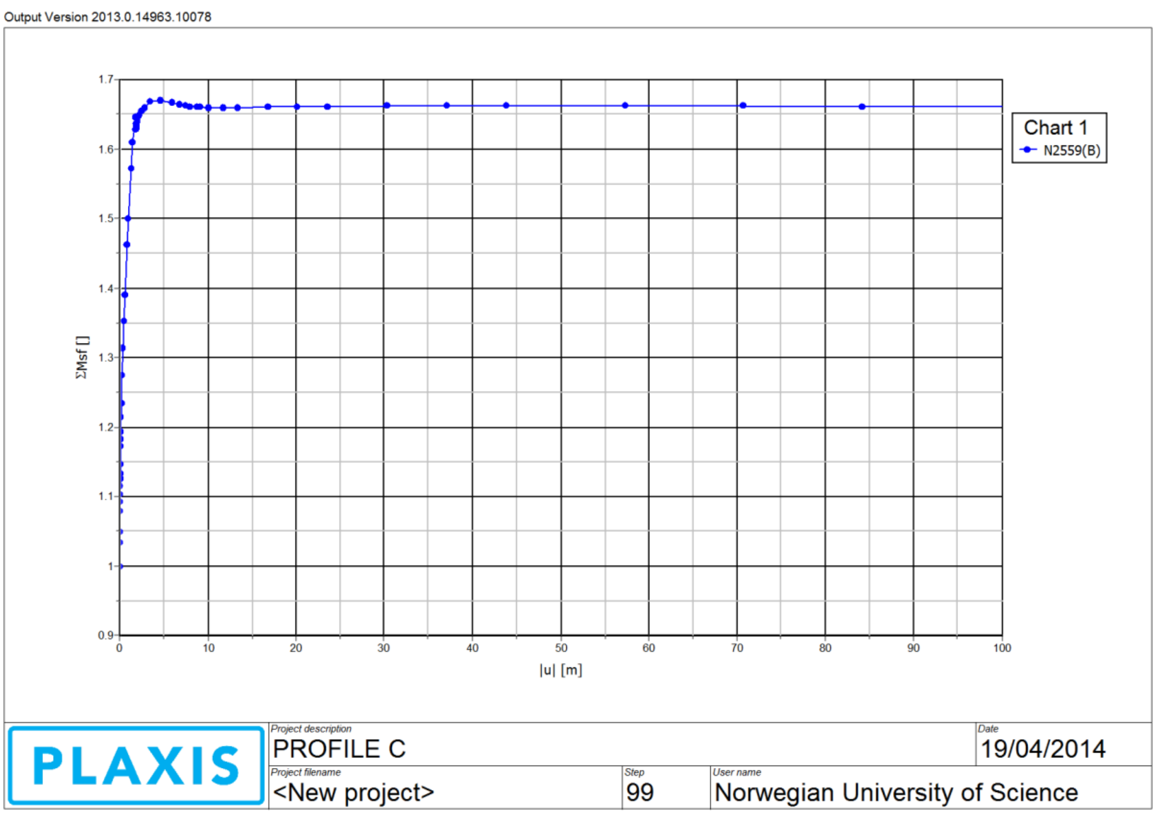
APPENDIX C2 – PLAXIS RESULTS FOR PROFILE B (DRAINED CONDITIONS)



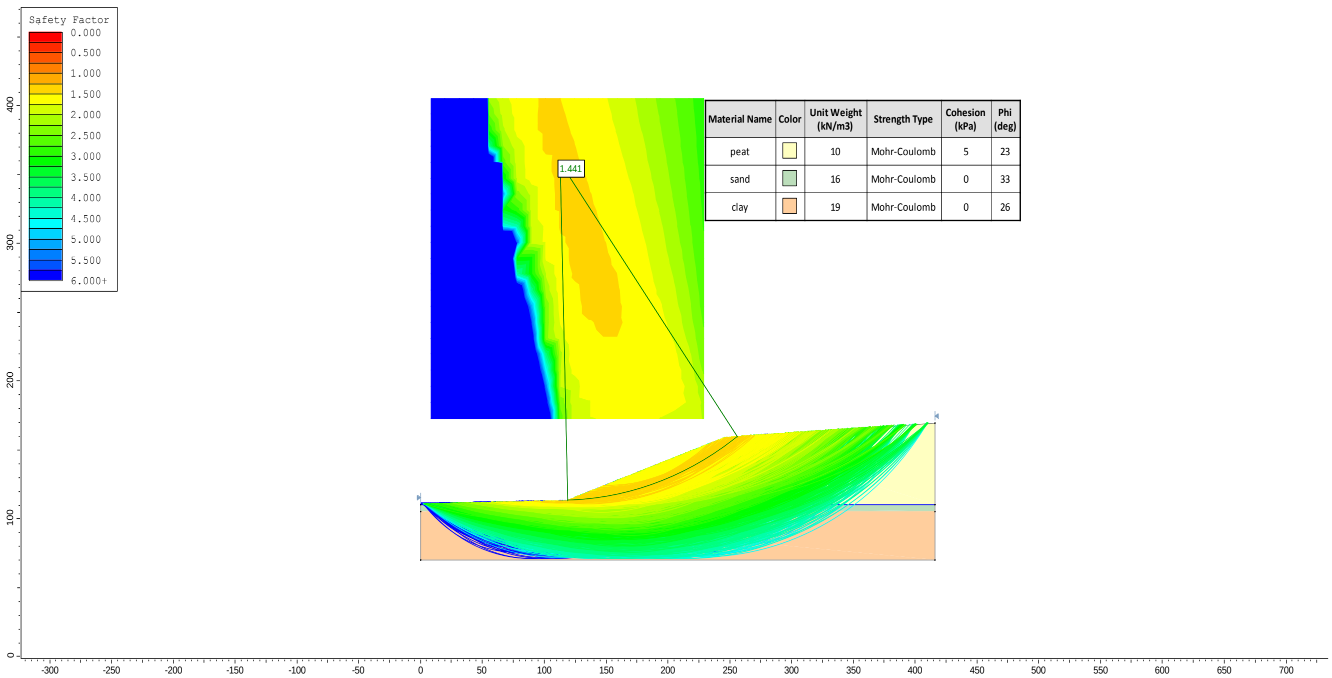


APPENDIX C3 – PLAXIS RESULTS FOR PROFILE C (DRAINED CONDITIONS)



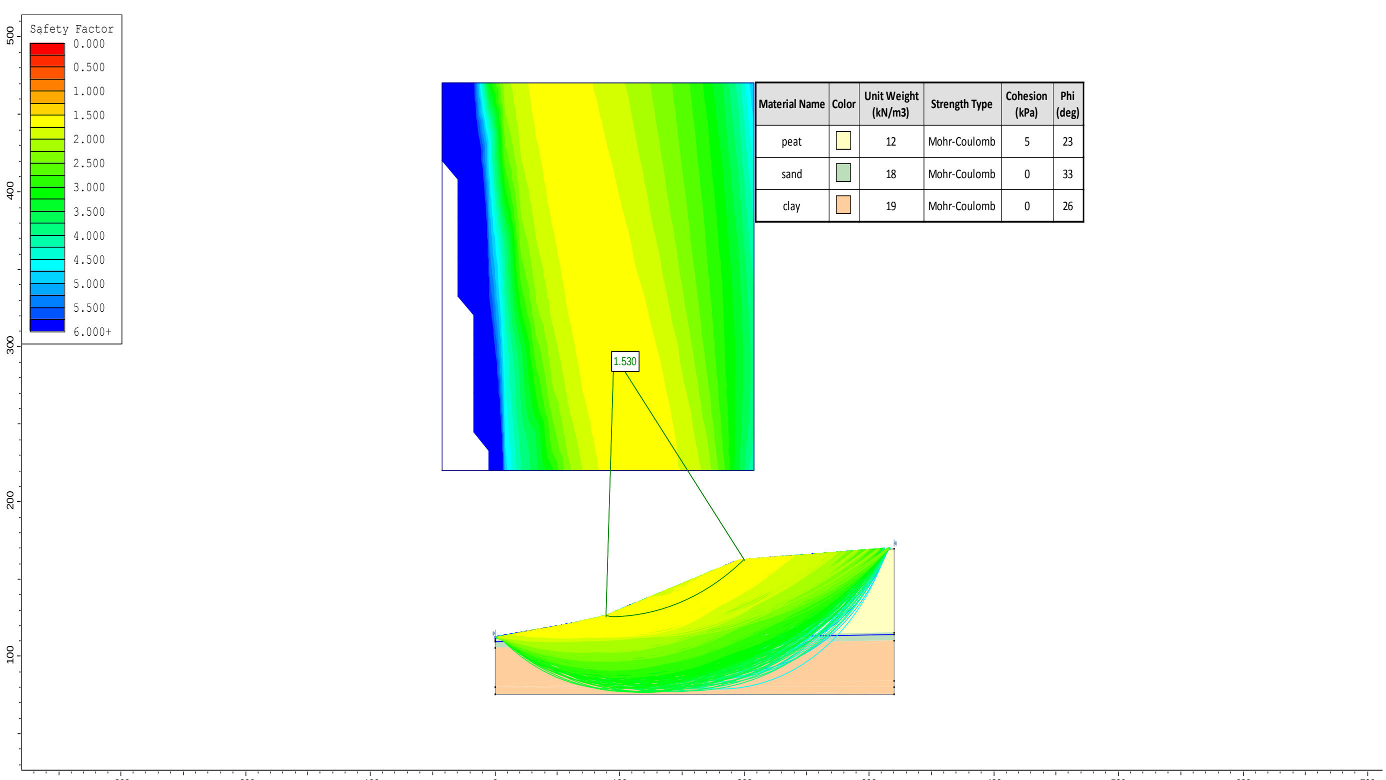


APPENDIX D1 – SLIDE RESULTS FOR PROFILE A (DRAINED CONDITIONS).



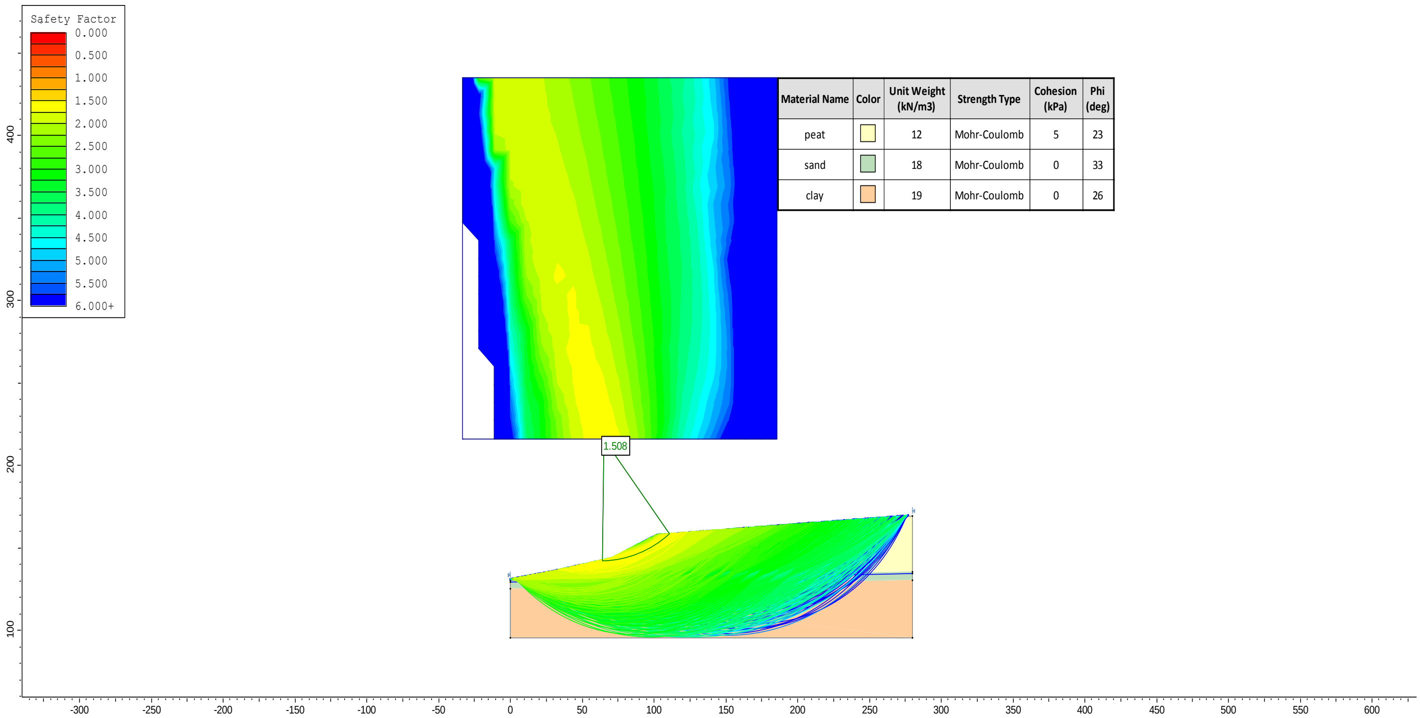
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		10	Mohr-Coulomb	5	23
sand		16	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

APPENDIX D2 – SLIDE RESULTS FOR PROFILE B (DRAINED CONDITIONS).

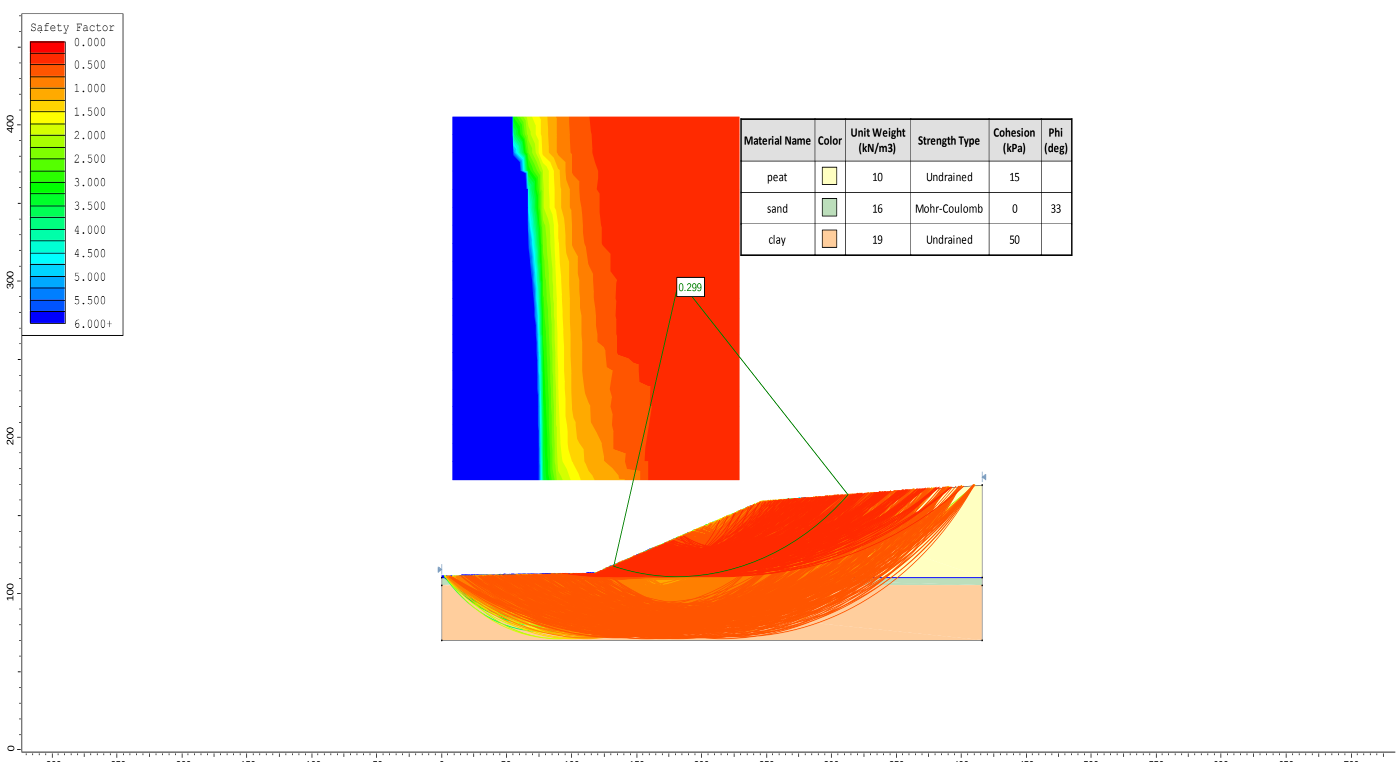


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
peat		12	Mohr-Coulomb	5	23
sand		18	Mohr-Coulomb	0	33
clay		19	Mohr-Coulomb	0	26

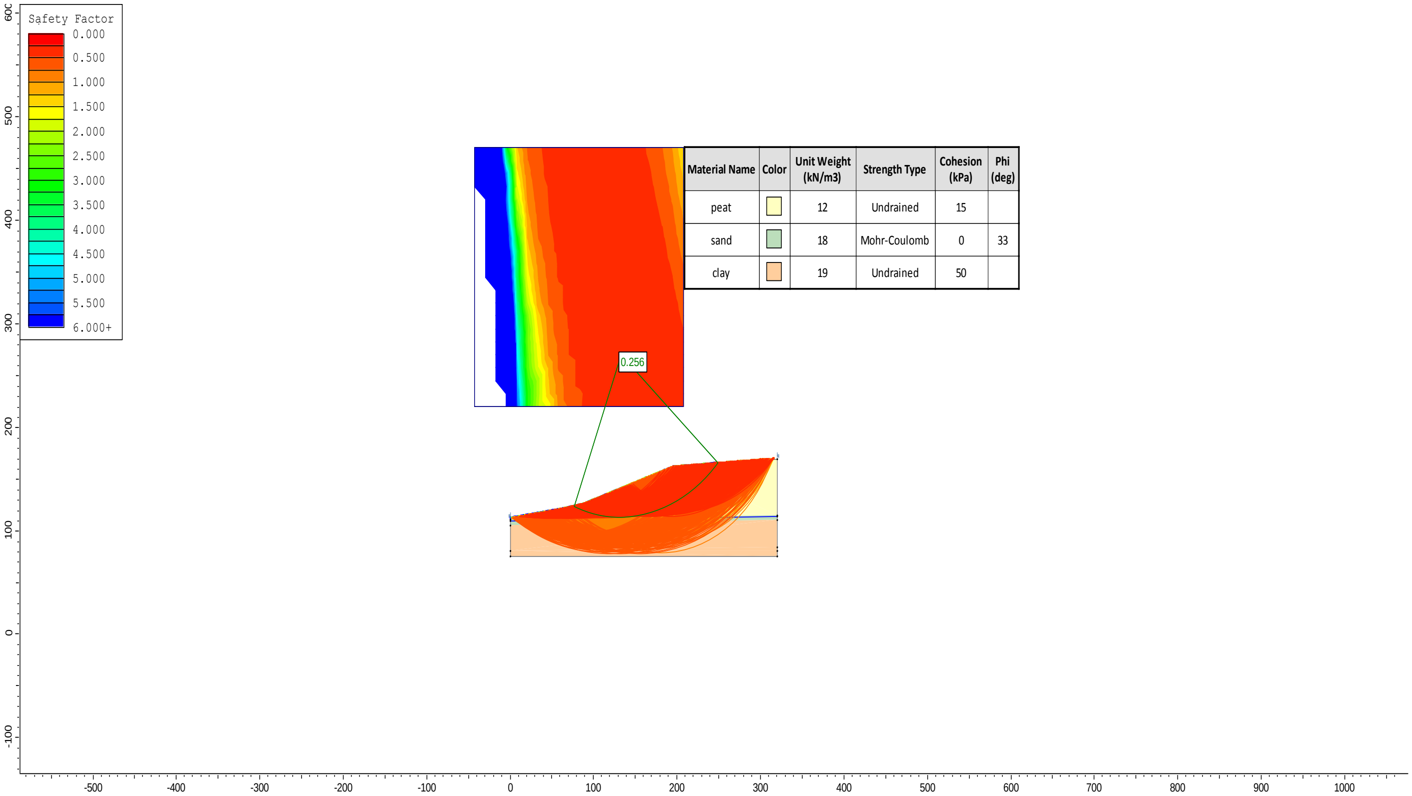
APPENDIX D3 – SLIDE RESULTS FOR PROFILE C (DRAINED CONDITIONS).



APPENDIX D4 – SLIDE RESULTS FOR PROFILE A (UNDRAINED CONDITIONS).



APPENDIX D5 – SLIDE RESULTS FOR PROFILE B (UNDRAINED CONDITIONS).



APPENDIX D6 – SLIDE RESULTS FOR PROFILE C (UNDRAINED CONDITIONS).