ENHANCEMENT OF THE STABILITY AT THE SITE OF AN ANCIENT LANDSLIDE IN A ROAD CUTTING WITH DRAINAGE AND REINFORCEMENT – CASE HISTORY AT GINIGATHHENA

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Thesis/Dissertation submitted in partial fulfillment of the requirements for the degree of Master of Science / Master of Engineering in Foundation Engineering and Earth Retaining Systems

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DECLARATION

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ABSTRACT

Engineers involved in infrastructure development projects in the hilly terrain of Sri Lanka encounter ancient landslides which could be triggered by rainfall or construction activities. One such landslide was encountered during the widening of a bridge in the main connecting road between central hill country and capital; Avissavella – Hatton – Nuwaraeliya road at bridge no. 48/2 near Ginigathhena. Extensive mitigation measures had to be designed with detailed attention to construction sequence in order to prevent reactivation of the slide.

A valley area had been formed by the previous landslide. Morphology of the area is a sloping land with undulating topography towards upper slope. This has led to the formation of a waterlogged marshy area on a flat land at immediate upper slope and a stream flowing through valley. Water table of the area is quite high. The landslide got activated due to a minor excavation at the toe region for the bridge widening. There had been no rain when the slide was activated. Subsequent rain cause further activation of the landslide. Further widening is necessary according to the new highway design.

Ground water regime management and geometry modification are the two primary approaches used in enhancing the safety margins of the site. Surface and subsurface drainage improvement by various methods such as; cutoff drains, berm drains, trench drains and horizontal drains were introduced for lowering the ground water table. The stability of the steep cuts necessary to accommodate the increased road width was enhanced further by the use of soil nailing. Top down approach was adopted to ensure the safety of the slope during construction. Drainage measures were very effective in economizing the soil nailing design. The analysis and design of stabilizing measures were done using GeoStudio Seep/W and Slope/W software.

Design outcomes were confirmed by monitoring of ground water table and surface movements of the slope.

Key Words – Landslide, Stabilization, Drainage Improvement, Reinforcement, Monitoring

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21st August 2019

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CHAPTER 1 - INTRODUCTION

1.1 Background

Sri Lankan landform experiences significant changes in elevation from lands at sea level in the coastal regions to the central regions with elevations around 2,000 m with many variations in geological formations and rock types. This variation has created many steep slopes all around the central region of the country. Sri Lanka has three main geological regions as Highland Complex, Vijayan Complex and Vanni Complex with two other minor regions as Kadugannawa Complex and Limestone as shown in Figure *1.1* (Cooray, 1984). Hilly area mainly belongs to Highland Complex which is composed of inter banded metamorphic rocks with granulite facies which formed under medium pressure and very high temperature conditions.

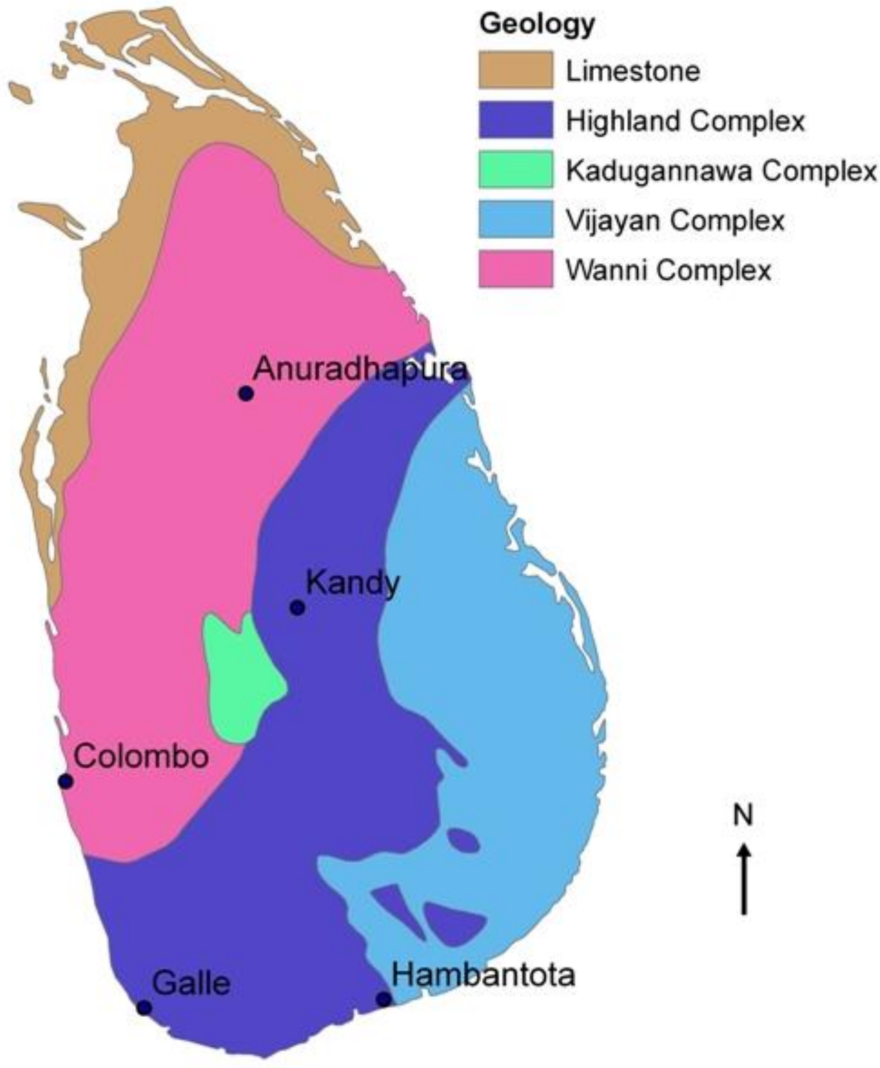


Figure 1.1 : Geology Map of Sri Lanka *(Cooray, 1984)*

Residual soils formed by in-situ weathering of parent metamorphic rocks are the most abundant in the island. Colluvial soils formed as a result of historical landslides are present at the lower levels of the slopes. Alluvial soils and organic soils are present in the flood plains. There is an extensive road network connecting the capital Colombo and all regional cities. The road network that traverses through the highly variable terrain in the island experience significant variations in the elevations. In some instances, the variations could be quite sudden. The road network mostly follows contour lines adopting bridge structures wherever necessary.

To implement the necessary widening and expansion of the road network to meet the demands of the economic development and population growth it would be necessary to excavate into the hillside. This would involve cutting into; fresh rock, rocks of different degrees of weathering, residual soils or colluvial soils. With the high degree of variability associated with these formations due to inherited characteristics of metamorphic parent rock and weathering under conditions of high rainfall and high temperatures, many complicated situations would arise. Addressing such issues is a major challenge encountered by Sri Lankan geotechnical engineers.

The ancient landslide at bridge No. 48/2, near Ginigathhena area at Avissawella – Hatton - Nuwaraeliya Road (A007) got reactivated with an attempt to widen the road in 2014. This is an extremely important road connecting Colombo to the central town of Nuwaraeliya used for both passenger and freight transport and it has to be rectified effectively within a short period of time.

This rectification was done under Climate Resilience Improvement Project (CRIP) executed by Ministry of Irrigation with the financial assistance of International Development Agency (IDA), World Bank and implemented by Department of Irrigation (ID), Mahaweli Authority of Sri Lanka (MASL) and Road Development Authority (RDA). National Building Research Organization (NBRO) was assigned the responsibility of designing the rectification measures and the supervision of the construction.

1.2 Basic description of the landslide

The main landslide area is a pseudo-stable colluvium mass spreading over an area of about 800 m^2 with a total potential area of about 3,000 m^2. Therefore, a large quantity of soil and rock boulders could be released if this mass moves at once by a failure triggered during a rainy period. It is clearly visible that present unstable features may develop further and such situation would lead to deep seated failures possibly demolishing the bridge no. 48/2 on the main road.

There is also the need to widen the bridge and approach to the bridge which requires further cutting into the slope. A small scale cutting back at the toe of the slope in the approach to the bridge triggered a movement of a large mass. Hence the need to plan an appropriate construction sequence with a comprehensive assessment of the subsoil profile and ground water condition was highlighted. A complete design was done and implemented with a comprehensive monitoring system to assess the effectiveness of the implemented rectification measures.

The main affected landslide area just above the road level is a sloping land with undulating topography towards the upper slope consists with thick colluvium layer, deposited by a previous landslide. Immediately at the upper slope of the landslide, there is a flat, water logged marshy area. Two major streams are flowing into this marshy area from the upper slope and finally come together to flow as one stream through the landslide area. Seepages and high yielding springs from the unstable mass, upper slope and even through the fractured bedrock along the road cuttings are prominent.

Beyond the thick Colluvial deposit closer to the toe region, the upper part of the slopes consists of residual and colluvial soils. The residual soil present is mainly clayey soil with silt and sand while colluvium deposits have big boulders with varying sizes embedded in a matrix of clayey silt. The thickness of the overburden varies from place to place.

According to information received from the community living in the area and the field evidence, number of cutting failures have been occurred within the area during rainy seasons. Additionally, there are some evidences of an old landslide just above the current landslide.

1.3 Conceptual design of the rectification and monitoring

The rectification of the impending landside should be designed with the careful consideration of the construction sequence. The widening at the toe needs removal of mass and making the slope at the toe steeper which will clearly reduce the safety margins and trigger the failure. As such, it is necessary to reduce the potential sliding mass by excavation and removal of significant amount of soil from the top and the middle. High stagnant water table conditions witnessed at the top and middle of the landslide should be lowered. The rectification process had to be kept within a limited area without extending further back into the slope. This makes it necessary to maintain steeper slopes that would demand reinforcement through techniques such as soil nailing. To economize the nailing designs, the ground water table should be maintained at a sufficiently low level.

As such, the rectification process would include;

- Surface drainage measures such as cut off drains, trench drains
- Sub surface drainage through sub horizontal gravity drains
- Reinforcement of slopes through soil nailing
- Minimization of infiltration with berm drains cascade drains and surface covering vegetation
- Toe support with gravity type toe retaining structures

The implementation of the rectification measures should follow a top down approach. The safety margins should be evaluated at each stage in the construction sequence.

The effectiveness of all these rectification measures should be evaluated during the construction and also during the period of service. As such, appropriate instrumentation should be installed at relevant locations at the necessary time in the construction process. The design and implementation of the instrumentation is also an essential part of the project. Some design modifications are to be done after the interpretation of the instrumentation data.

1.1 Scope and objectives of the thesis

- Understanding the behavior of ground water table with respect to combination of different rainfall intensities and different drainage improvement measures

- Assessment of slope stability with respect to various drainage improvement techniques for different rainfall patterns

- Assessment of slope stability with respect to slope reinforcing by soil nailing and economizing the nail design by drainage improvement

- Confirmation of design outcomes by field monitoring

1.2 Thesis outline

The thesis is structures in the following manner

- Chapter 2 – Present a review of current knowledge on different forms of rain induced slope failures, appropriate rectification processes and analytical techniques that can be adopted. Applications in number of case studies are cited in this chapter.

- Chapter 3 – After studying the background of landslide, conceptual design was done. Detail investigation including field investigation, laboratory investigation and topographical survey were carried out. Rectification measures were designed using SLOPE/W software based on those results. Field monitoring scheme was adopted to monitor the behavior of the slope after rectification. This Chapter presents these aspects.

- Chapter 4 – Seepage model was developed using SEEP/W software and model verification was done by comparison with field monitoring data. Then slope's response to a hypothetical peak rainfall which is based on observations during the monitoring period was studied by coupling the analyses SEEP/W and SLOPE/W software. These aspects are presented in this Chapter.

- Chapter 5 – Presents the concluding remarks and recommendations for landslide mitigation designs and future studies.

CHAPTER 2 - LITERATURE REVIEW

This Chapter presents a review of current knowledge of different forms of rain induced slope failure, appropriate rectification processes and analytical techniques that can be adopted. Most appropriate monitoring techniques are also discussed. Applications of rectification measures in number of case studies are also cited in this chapter.

2.1 Landslides in Sri Lanka

2.1.1 Introduction

Landslide is defined as "the downward movement of a rock, debris, or earth under gravity" (Cruden, 1991). A landslide could trigger by an event such as extreme rainfall or an earthquake under the background of a certain causative factors related to the geomorphological conditions and sub soil conditions. Terzaghi (1950) has divided causes of landslide in to two categories as external causes which results increasing of shear stress (e.g. geometrical changes, unloading the slope toe, loading the slope crest, shock and vibration, drawdown, changes in water regime) and internal causes which results reduction of shear strength (e.g. progressive failure, weathering, seepages, erosion).

According to Cruden & Varnes (1996) and Wieczorek (1996) the causative factors of landslides can be divided mainly into four categories.

1. Physical processes – Intense rainfall, rapid snow melt, water level change, volcanic eruption, earthquake etc.
2. Man-made processes – excavation of the slope at toe, loading on the slope at crest, improper dumps of soil and waste, vegetation removal, water leakage from services, improper drainage management etc.
3. Ground condition (geological) – weathering process, weak geological features etc.
4. Geomorphological processes–many kinds of erosion of the slope and its toe, tectonic uplift, volcanic uplift, deposition loading of the slope or its crest etc.

Intense rainfall is the major triggering factor causing landslides in Sri Lanka while improper slope excavation, improper soil & waste dumping, unplanned drainage systems, removal of vegetation and poor maintenance of slopes, facilitate the initiation of slope instabilities.

Practically, stability of a slope could be at either of three stages as; stable, marginally stable and actively unstable (Crozier, 1986).When the margin of stability of a slope is sufficiently high to withstand all destabilizing forces, it is considered as a stable slope. When a slope will fail at some instance in response to the destabilizing forces attaining a certain level of activity, it is considered as marginally stable slope. If a slope's destabilizing forces produce continuous or intermittent movement, it is considered as an actively unstable slope. Popescu (2002) has described that factors which makes a slope marginally stable from stable are preparatory factors and factors which makes a slope unstable from marginally stable condition are triggering factors as shown in Figure 2.1. However, in general, only term "triggering factor" is referred as the most critical reason or reasons for activation of a landslide.

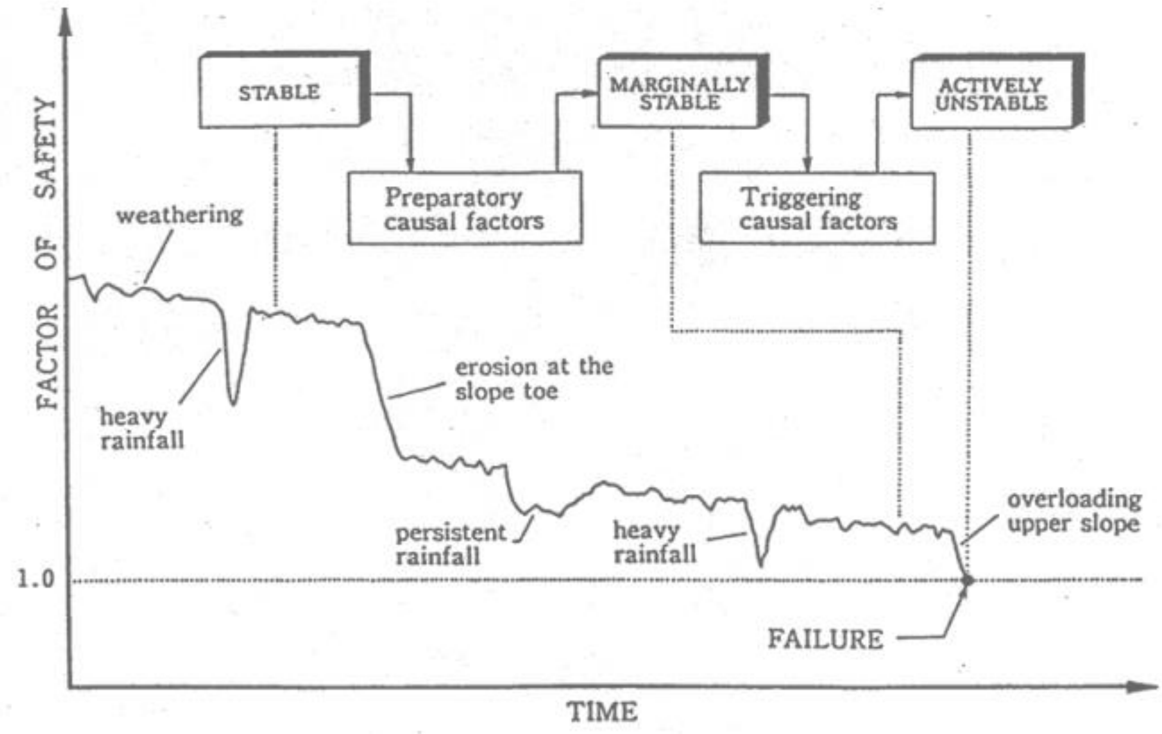


Figure 2.1 : Example of FoS variation with time *(Popescu, 2002)*

2.1.2 Classification of landslides

According to Varnes (1978) the landslide classification has two terms as,

- The first term describes the material type – rock, earth, soil, mud, debris etc.
- The second term describes the type of movement – fall, topple, slide, spread, flow

<u>Slide</u>

Although many types of mass movements are included in the general term "landslide," the more restrictive use of the term refers only to mass movements, where there is a distinct zone of weakness (slip surface, surface of rupture) that separates the slide material from more stable underlying material. Landslides at Watawala railway line and Kahagolla in A7 road are two similar cases that can be placed under this category. There is a clear well defined failure surface that can be stabilized by well designed rectification measures.

The two major types of slides are rotational slides and translational slides. Both types of slides are common to Sri Lanka.

- Rotational Slide (Figure 2.2) – The slip surface is curved upward concavely (spoon-shaped) and the slide moves roughly rotational about an axis that is parallel to the ground surface and transversely across the slide. The rotation surface may be circular or non-circular depending on the ground condition. This type of failures commonly occur in the soil slopes where the bedrock is quite deep.

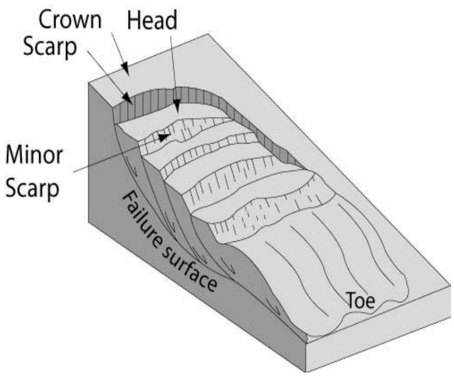


Figure 2.2 : Rotational Slides *(British Geological Survey, 2019)*

- Translational Slide (Figure 2.3) – The landslide mass moves along a roughly planar surface with little rotation or backward tilting. This type of slides commonly occurs along a geological discontinuity such as faults, joints, bedding planes or interface between soil and rock. Rain induced landslides in slopes of long extent could also follow this mode of failure.

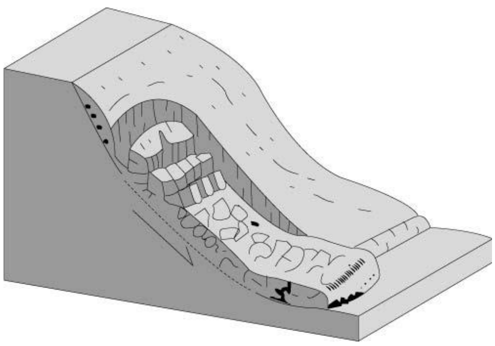


Figure 2.3 : Translational Slides *(British Geological Survey, 2019)*

Fall and Topple

Fall and topple are mainly described in rock and sometimes in soil slopes. It is a phenomenon where a part of rock or soil mass detached from the main parent body along adversely oriented discontinuities and falling or rolling downwards. Rock falls and topples are common natural hazards in Sri Lanka.

Flow

There are several types of flows as debris flow, mud flow etc. In a flow, the disintegrated materials are flowing without a predefined path or boundaries. After a slide, fall or topple, a debris flow can be initiated with the accumulated disturbed materials. This mode of failure is quite common under heavy prolonged rainfall conditions in Sri Lanka where a failure that initiated as a slide converted to a flow or debris flow causing a movement of huge mass of land with all property and humans within it causing massive destruction. Recent landslides at Aranayaka (2016) and Meeriyabedda (2014) comes under this category. Movement of the mass may have extended down to the bedrock and the debris accumulated at the toe should be managed to a stable landform by removal or proper landscaping. Accumulation of debris from rock falls may also end as a debris flow. Structures of different form may be constructed to contain the debris flow.

2.1.3 Landslide situation in Sri Lanka

It was noted that during and after the last quarter of 20th century, landslides have become a major natural disaster in Sri Lanka, especially in the hill country during the monsoons with heavy rainfall (Ratnayake & Herath, 2005). The central highland covers nearly 20% of the total land area of the country where about 30% of the total population lives, and thirteen districts were identified as prone to landslide hazards. Nearly 1,000 human lives were lost while over 300,000 people were made homeless due to cataclysmic landslides events occurred during the last few decades. Loss of property and infrastructure and the damages caused to the national development was enormous.

Figure 2.4 illustrate the distribution of recorded landslides in Sri Lanka occurs up to 2017 (Bandara & Jayasingha, 2018).

Figure 2.5 presents the number of deaths due to landslides event in Sri Lanka and it shows the remarkable increase of the number of deaths in recent decades (Bandara & Jayasingha, 2018).

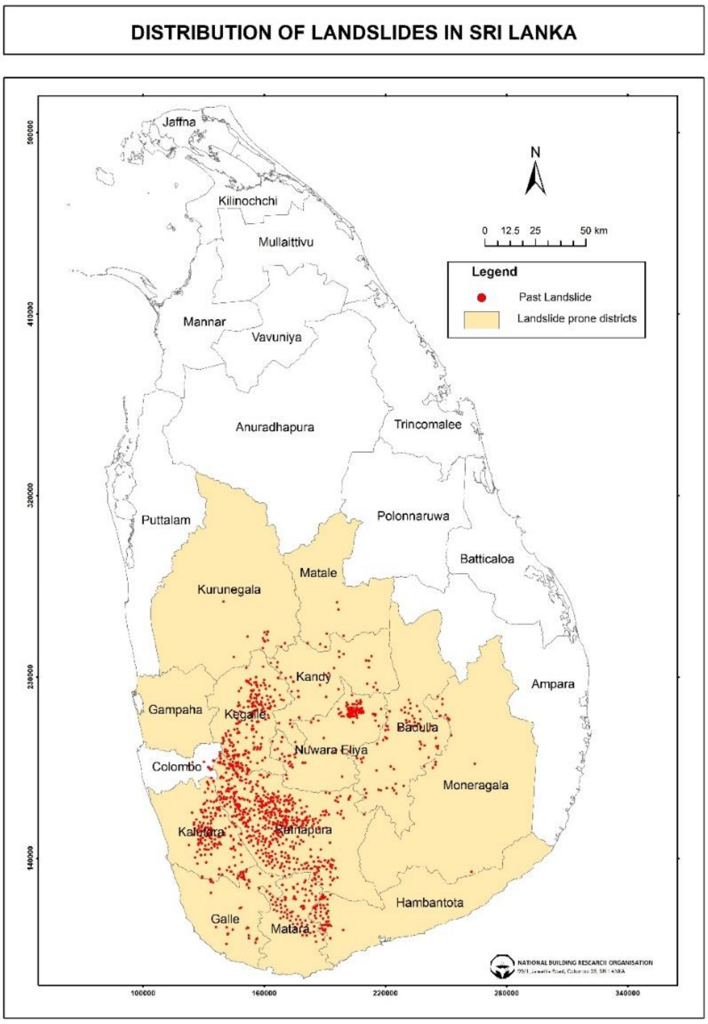


Figure 2.4 : Distribution of Landslides in Sri Lanka *(Bandara & Jayasingha, 2018)*

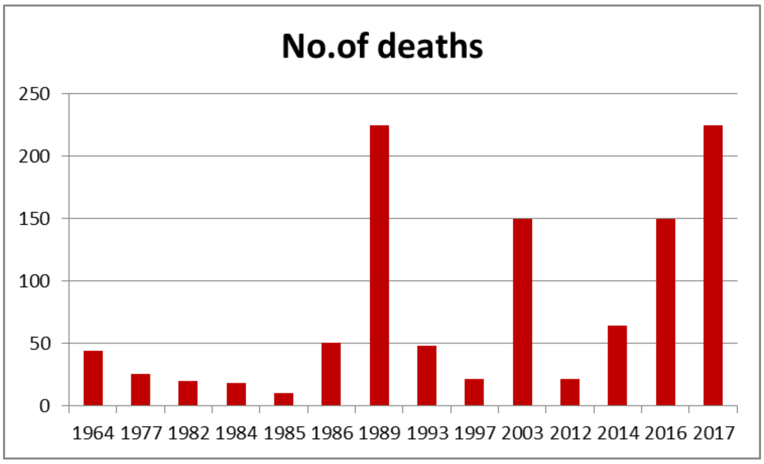


Figure 2.5 : Number of Deaths due to Landslides *(Bandara & Jayasingha, 2018)*

Almost every year in recent past, number of landslides events have been reported during the monsoon periods from all over the landslide prone areas in Sri Lanka. Other than the effect to the human lives, properties, environment and day to day life is severely affected during these periods and it leads to creates many social and economical issues. Also, large number of officials and other personals are involved in disaster management during these periods and huge amount of money is expended by the government to facilitate those activities and aid the people who are affected. Finally, all those factors are directly or indirectly affect the economy of the country adversely.

Other than the rainfall, there are several causes which contributes to the landslides in Sri Lanka. Many landslide events, especially cutting failures, are reported along the road network due to not applying proper mitigation measures during cutting and poor maintenance. Also improper construction of building in hilly terrains, poor drainage management in slopes and removal of vegetation of slopes have led to many failures. Considering the requirements, National Building Research Organisation (2015) has published a manual for hazard resilient housing construction against natural hazards like landslides, floods, cyclones, high winds, earthquakes, tsunamis etc.

There had been several extreme rainfall events reported in recent past years which lead to activation of devastating landslides. Those extreme events may have occurred due to the climatic changes that occur all over the world. Other factors described above had aggravated the situation of landslides.

2.2 Assessment of slope stability

The assessment of the stability of slope in a quantified approach is critically important. The stability of a slope is mostly assessed by the computation of a factor of safety through a mechanistic approach.

2.2.1 Factor of Safety

Definition

Stability of a slope is assessed by the concept of Factor of Safety (FoS) and it may be defined as the ratio between maximum shear strength the soil can sustain to shear stress mobilized for equilibrium.

$$FoS = \frac{Maximum\ shear\ strength\ the\ soil\ can\ sustain}{Shear\ stress\ mobilized\ for\ equilibrium} = \frac{Shear\ strength\ of\ the\ soil}{Shear\ stress\ developed} \tag{2.1}$$

$$FoS = \frac{\tau_f}{\tau_m} \tag{2.2}$$

Shear strength is expressed by Coulomb failure criterion in term of total stress as,

$$\tau_f = c + \sigma\ tan\,\emptyset \tag{2.3}$$

Where c is the cohesion, σ is the total normal stress on the failure surface and $\emptyset$ is the angle of internal friction of the slope material.

However, the most accurate analysis of slope stability for rain induced landslides is the effective stress analysis, since the pore water pressure change is the triggering factor of those failures and effective stress analysis accounts for the change of pore pressures. Further, Bishop & Bjerrum (1960) showed that the safety margin of a cut slope is critical in long term and it is required to carry out an effective stress analysis. Coulomb equation was modified by Terzaghi (1943) combining with the principle of effective stress for the drained condition as,

$$\tau_f = c' + \sigma'\ tan\emptyset' \tag{2.4}$$

where c', $\emptyset'$ are the effective soil parameters and the σ' is the effective normal stress on the failure surface given by,

$$\sigma' = \sigma - u \tag{2.5}$$

where u is the pore water pressure and this is only applicable to the saturated soils and it becomes more complex in unsaturated conditions (Bishop, et al., 1960; Bishop & Bjerrum, 1960).

For unsaturated soils, shear strength is given by,

$$\tau_f = c' + (\sigma - u_a)tan\emptyset' + (u_a - u_w)tan\emptyset^b \tag{2.6}$$

where u_a is the air pressure, $(u_a - u_w)$ is the value of matrix suction and $\emptyset^b$ is the angle of shearing resistance due to suction.

Regardless of whether the total stress or effective stress was used, definition of the factor of safety remains same as the ratio of shear strength of the soil to shear stress developed. In every slope stability analyzing method, value of the safety factor is assumed to be a constant along the slip surface and it is taken as an overall average value. Factor of safety is a quantitative estimate of a stability of a slope (Bishop, 1955). Figure 2.6 illustrated the shear resistance and shear force acting on a moving mass during a landslide movement.

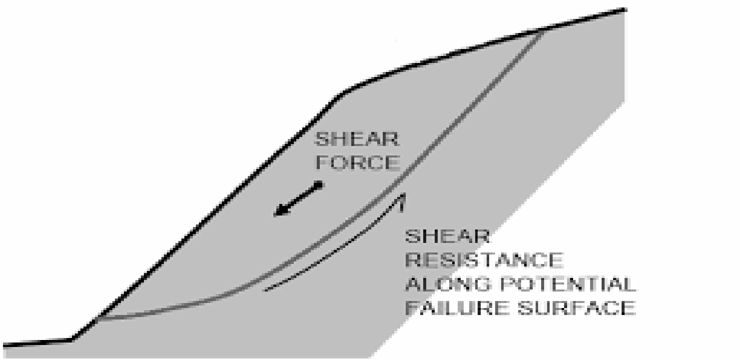


Figure 2.6 : Forces during Landslide Movement *(Rotaru, et al., 2007)*

Design FoS

Three stages of stability condition of a slope described by Crozier (1986) as stable, marginally stable and actively unstable can be defined in term of Factor of Safety categories as follows.

- Unstable ➔FoS< 1
- Critically Stable ➔FoS = 1
- Stable ➔FoS> 1

A major issue which arise in slope stability analysis is selections of design safety factor. Many factors are affecting the selection of design safety factors such as;

- Availability and reliability of subsoil data
- Location
- Influence on socio-economic factors
- Type of structure
- Level of risk that the designer willing to take

Selection of the design safety factor will significantly affect the cost and risk of the design. Different values are suggested for design safety factor in different sources such as text books, code of practices, guidelines, papers, researches etc.

Hong Kong, Geotechnical Manual for Slopes (GEO, Hong Kong, 1984) recommend minimum factor of safety of 1.4 for new slopes for ten year return period against high level risk to life and / or economic. Recommended factor of safety values by GEO, Hong Kong (1984) are summarized in Table 2.1.

Table 2.1: Recommended Factor of Safety *(GEO, Hong Kong, 1984)*

ECONOMIC RISK \ RISK TO LIFE		Recommended Factor of Safety against Loss of Life for a Ten-year Return Period Rainfall		
		Negligible	Low	High
Recommended Factor of Safety against Economic Loss for a Ten-year Return Period Rainfall	Negligible	>1.0	1.2	1.4
	Low	1.2	1.2	1.4
	High	1.4	1.4	1.4

Note : (1) In addition to a factor of safety of 1.4 for a ten-year return period rainfall, a slope in the high risk-to-life category should have a factor of safety of 1.1 for the predicted worst groundwater conditions.
(2) The factors of safety given in this Table are recommended values. Higher or lower factors of safety might be warranted in particular situations in respect of economic loss.

U.S. Department of Transportation, Soils and Foundations Reference Manual (Federal Highway Administration, 2006) suggest factor of safety as low as 1.25 for highway embankment side slopes and it should be increased to minimum of 1.30 to 1.50 for critical slopes whose a failure would cause significant damage. For the slopes made of fined grained soil, minimum factor of safety of 1.50 is recommended. Generally higher factor of safety values are considered for natural slopes due to the uncertainty about homogeneity of the soils than in embankment slopes that are constructed under well controlled condition.

Probabilistic approach of evaluating safety of a slope

The computation of the factor of safety of a natural slope is affected by many uncertified due to heterogeneous nature of the associated parameters Therefore, conventional approaches for stability analysis cannot be used independently for risk associated slope safety designs. Probabilistic slope stability analysis is developed as a solution for this problem. In this method the variability of most significant parameters are accounted by taking them as random variables. The numerical values of the parameters are assigned a mean and a standard deviation based on the experimental data. When the factor of safety is evaluated through the mathematical formulations the probability the factor of safety will be less than unity is computed and termed the probability of failure. Kulathilaka & Mettananda (2002) developed a formulation on this approach and applied to evaluate the stability of some Sri Lankan slopes. This is the microscopic approach in evaluating the probability of failure. Alternatively, the probability of failure is evaluated in a macroscopic formulation in the form of hazard zonation maps (landslide susceptibility maps) by assigning relative values to various factors affecting a landslide. On that basis, NBRO has developed landslide hazard zonation maps for critical districts in Sri Lanka as a 1:10,000 scale.

2.2.2 Deterministic methods for stability analysis

The conventional deterministic methods end up computing a FoS for the slope. Two deterministic approaches for assessing the stability of a slope are;

- Limit equilibrium approach
- Finite element approach

Limit equilibrium method is the most widely used method for slope stability analysis, because of its relative simplicity and early origins. In limit equilibrium method a trial failure surface is selected on which the failure is assumed to be occurring and safety margin along that surface is estimated through a mechanistic analysis. Number of trial failure surfaces are considered and the corresponding trial surface which gives the minimum FoS is considered as the critical failure surface. Designer must ensure that all possibly potential failure modes are accounted by considering a sufficient number of trial failure surfaces based on the geometry of the slope and the sub soil characteristics. There are many methods available for analyzing the stability of a slope along an assumed failure surface. Selection of the most suitable method for the analysis based on the key features of available methods, is a crucial decision to be taken by the designer.

Development of methods of slope stability analysis has commenced as early as mid of 19th century. The Culmann method (Culmann, 1866) is one of the earliest method for stability analysis which is based on the assumption that the failure occurs in a plane through the toe of the slope (Taylor, 1948). However, these kind of plane failure surfaces are observed only in very steep slopes and generally curved failure surfaces are observed in relatively flat surfaces. Therefore, this method of slope stability is used rarely for slope stability analysis now. The commonly used methods for calculation of slope stability are based on the assumption of circular failure surface, because most slope failure surfaces are approximately circular (Yuen, n.d.). Initially, friction circle method was developed considering the stability of whole mass as it is. There are several chart solutions developed for this method such as Taylor’s charts (Taylor, 1948), Cousin’s charts (Cousins, 1978) etc. However, these chart solutions are only applicable to homogeneous soils.

In case where the slope consists of different types of soils or unusual seepage pattern exists, other methods of analysis are required. Over the last six decades, methods of slices are widely used for such instances and many methods of slices have been developed by different researchers. In all methods of slices, failure mass is broken up into a series of slices and the equilibrium of each slice is considered. Accuracy of those methods depends on the assumptions used to eliminate the static indeterminacy of the solution. Several simplified methods like ordinary method of slices ((Fellenius, 1936)& (May & Brahtz, 1936)), Bishop’s simplified method (Bishop, 1955) and Janbu’s simplified method ((Janbu, et al., 1956)& (Janbu, 1957)) etc. are widely used due to the simplicity of analysis and ability to do the computations with the help of a simple calculator. Comparison of Bishop’s simplified method with linear and non-linear finite element analyses shows only very small difference when applied to circular failure surfaces (Wright, et al., 1974).

On the other hand more theoretically sound analysis methods such as; Bishops general method (1955), Janbu’s rigorous method (Janbu, 1954), Spencer’s method (Spencer, 1967), Morgenstern & Price method (Morgenstern & Price, 1965) and Sarma method (Sarma, 1973) consider both moment and force equilibrium and formulated to analyze both circular and non circular failure surface. However, these methods involve very complex calculation and need a computer program to solve. The developments in computer technology has made them applicable for common use.

A summary and comparison of the available methods of slices for slope stability analysis are presented in Table 2.2 (Duncan & Wright, 1980).

Table 2.2 : Comparison of the Slope Stability Analysis Methods *(Duncan & Wright, 1980)*

Method (1)	Characteristics (2)
Slope Stability Charts (Janbu 1968; Duncan et al. 1987)	Accurate enough for many purposes Faster than detailed computer analyses
Ordinary Method of Slices (Fellenius 1927)	Only for circular slip surfaces Satisfies moment equilibrium Does not satisfy horizontal or vertical force equilibrium
Bishop's Modified Method (Bishop 1955)	Only for circular slip surfaces Satisfies moment equilibrium Satisfies vertical force equilibrium Does not satisfy horizontal force equilibrium
Force Equilibrium Methods (e.g. Lowe and Karafiath 1960; U.S. Army Corps of Engineers 1970)	Any shape of slip surfaces Do not satisfy moment equilibrium Satisfies both vertical and horizontal force equislibrium
Janbu's Generalized Procedure of Slices (Janbu 1968)	Any shape of slip surfaces Satisfies all conditions of equilibrium Permits side force locations to be varied More frequent numerical problems than some other methods
Morgenstern and Price's Method (Morgenstern and Price 1965)	Any shape of slip surfaces Satisfies all conditions of equilibrium Permits side force orientations to be varied
Spencer's Method (Spencer 1967)	Any shape of slip surfaces Satisfies all conditions of equilibrium Side force are assumed to be parallel

2.2.3 Finding of critical failure surface

The trial failure surface which gives the minimum FoS value is considered as the critical slip surface and corresponding minimum Factor of Safety value is considered as the safety margin of the slope. Alternative trial failure surfaces are considered in all the aforementioned analysis methods and critical slip surface in which the factor of safety is lowest is found. Followings are several methods that have been developed in order to select different trial failure surfaces for the analysis in a logical manner.

- Circular Failure Surfaces
    - Grid search
    - Line search (Steepest descent, Orthogonal lines, Inclined orthogonal lines)
    - Simplex optimization
- Non-Circular Failure Surfaces
    - Dynamic programming
    - Random line segments
    - Incremental adjustment

2.2.4 Stability analysis with SLOPE/W (GEO-SLOPE International Ltd., 2017)

Computer software are commonly used in geotechnical engineering for slope stability analysis based on limit equilibrium approach, due to the difficulty of manually solving a problem with a large number of trial failure surfaces. The introduction of powerful desktop personal computers in the early 1980s, made it economically viable to develop commercial software. Among the many available software for soil slope stability analysis, GoeStudio SLOPE/W software is the most commonly used software.

Analysis type

Limit equilibrium slope stability analysis in SLOPE/W software can be performed for many analysis methods such as ordinary method of slices, Bishop's simplified method, Janbu's simplified method, Spencer method, Morgenstern-Price method, Lowe-Karafiath method, Sarma method, Janbu's generalized method etc. Selection of most appropriate method for analysis shall be done wisely by the designer considering the type of failure and other factors.

Critical slip surface

SLOPE/W allows for analysis of both circular and non-circular slip surfaces. Finding the critical slip surface is a major concern in slope stability analysis and SLOPE/W provides several methods to perform that task such as entry and exit, grid and radius, block specified and fully specified. Critical slip surface is found by means of trial and error approach in every method other than fully specified method. As in type of analysis, selection of most appropriate method for finding critical slip surface shall be done wisely by the designer.

Material models

Several material models are available with the SLOPE/W software such as Mohr-Coulomb, Spatial Mohr-Coulomb, Undrained, High strength, Bilinear etc. and there is a unique model for bedrock as impenetrable (bedrock) in more recent versions. Selection of the suitable material models shall be done wisely by the designer according to the requirement.

2.3 Seepage analysis

2.3.1 Requirement of seepage analysis

Most slopes in Sri Lanka are made with residual soils weathered from metamorphic parent rocks by various weathering processors and lie at the location of the parent rock. In most situations, ground water level in those slopes are generally low in dry seasons with high matric suctions at upper levels. During the periods of rain, significant infiltration may cause loss of matric suction, rise of the ground water table and development of perched water table. There are many drainage improvement techniques that could reduce the rainwater infiltration and cause rapid dissipation of the excess pore water pressures. All the above phenomena need to be considered in a stability analysis regarding the rain induced slope failures. As such, accurate modeling of the problem is critically important.

2.3.2 Saturated and unsaturated condition of soils

Soils are formed by solid particles with voids in between. Voids may be filled with either air or water. When all the voids in soil are filled with water, it is known as saturated soil. When only a part of voids in soil is filled with water and remaining voids are filled with air, it is known as unsaturated soil. Generally, soil below the ground water level is completely saturated and soil above the ground water level is unsaturated (Figure 2.7). In fine grained soil, saturated zone may extend to some height above the ground water level by capillary rise.

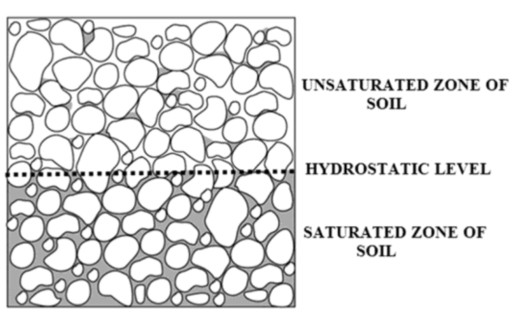


Figure 2.7 : Saturated and Unsaturated Zones of Soil *(Belciu, et al., 2017)*

2.3.3 Hydraulic properties of unsaturated soil

Darcy's law is applied to study the water flow in saturated soils. Water flow through unsaturated soils is also generally governed by Darcy's law as similar to saturated soil (Fredlund & Rahardjo, 1993). In addition, there are two major differences in water flow in an unsaturated soil as compared with the saturated soil;

- There exists a storage term which represents the variation of water content with matric suction
- The water coefficient of permeability depends strongly on matric suction

It should be noted that no volume change in soil is considered during the infiltration process. The storage term in unsaturated flow is not a constant but dependent on the suction (or water content) in an unsaturated soil and can be characterized by the Soil Water Characteristic Curves (SWCC). Therefore, SWCC and water coefficient of permeability are the most important hydraulic properties for unsaturated soils.

Soil water characteristic curves (SWCC)

The most fundamentally important feature of the unsaturated soils is the SWCC, which shows the variation of the matric suction with the volumetric water content. The volumetric water content (θ) is defined as the amount of water contained within the pores of soil.

$$\theta = \frac{V_w}{V_s} \tag{2.7}$$

where, V_w = volume of water content

V_s = volume of soil

Figure 2.8 presents an idealized SWCC with two characteristic points as A* and B*. A* corresponds to air entry value, at which point air starts to enter the soil and, soil is nearly saturated or saturated prior to point A*. B* corresponds to the residual water content, at where, there is only little amount of water by which the effect of pore pressures is negligible and soil is considered as dry beyond that point. Stage between the A* and B* is the most concern in unsaturated soils in which the soil properties are strongly related to its water content or negative pore water pressures.

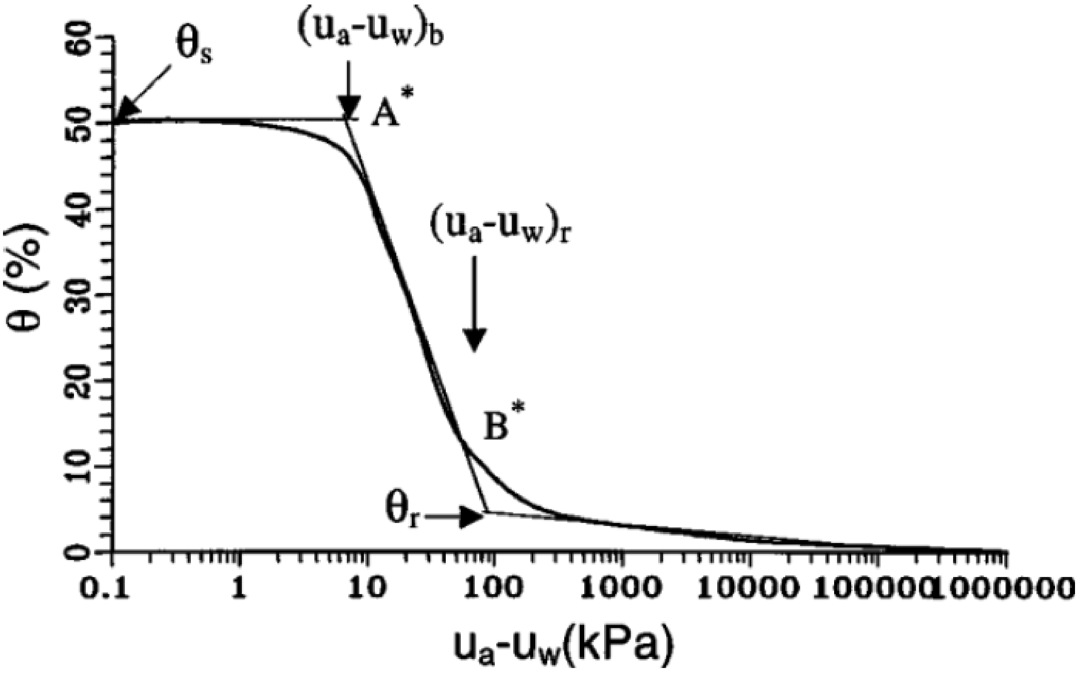


Figure 2.8 : Idealized Soil-Water Characteristic Curve *(Fredlund & Rahardjo, 1993)*

Hydraulic conductivity function

The coefficient of permeability of unsaturated soil depends on the degree of saturation or negative pore water pressures of the soil. The relationship of the degree of saturation to negative pore-water pressure can be represented by a SWCC. Therefore, the water coefficient of permeability for unsaturated soils with respect to negative pore-water pressure bears a relationship to the SWCC, and it can be estimated from the saturated permeability and the SWCC (Fredlund, et al., 1994).

2.3.4 Process of infiltration

The flow of water through both saturated and unsaturated soil that follows Darcy's Law which states that,

$$q = ki \tag{2.8}$$

where q is the specific discharge k is the hydraulic conductivity and i is the gradient of total hydraulic head. Although Darcy’s law is derived for saturated soil, it is also applied to the flow of water through unsaturated soil as well. The only difference is that under conditions of unsaturated flow, the hydraulic conductivity is no longer a constant, but varies with changes in water content and indirectly varies with changes in pore-water pressure.

Darcy's Law is often written as;

$$v = ki \tag{2.9}$$

where, v is the Darcian velocity. The actual average velocity at which water moves through the soil is the linear velocity, which is equal to Darcian velocity divided by the porosity of the soil. In unsaturated soil, it is equal to Darcian velocity divided by the volumetric water content of the soil.

According to the Darcy-Buckingham equation, horizontal and vertical water flux (q_x and q_z) in unsaturated soil are expressed as;

$$q_x = -k(\psi)\left(\frac{\partial\psi}{\partial x}\right) \tag{2.10}$$

$$q_z = -k(\psi)\left(\frac{\partial\psi}{\partial z} + 1\right) \tag{2.11}$$

where $k(\psi)$ is the hydraulic conductivity function of negative pore water pressure ψ (matric suction). The equation for continuity of water is expressed as,

$$\frac{\partial\theta}{\partial t} = -\left(\frac{\partial q_x}{\partial x} + \frac{\partial q_z}{\partial z}\right) \tag{2.12}$$

where t is the time.

Substituting above equations yields the two-dimensional, vertical and horizontal flow equation for soil water (Richard's Equation),

$$\frac{\partial}{\partial z}\left(k(\psi)\left(\frac{\partial\psi}{\partial z}\right)\right) + \frac{\partial}{\partial x}\left(k(\psi)\left(\frac{\partial\psi}{\partial x}\right)\right) + \frac{\partial}{\partial z}(k(\psi)) = c(\psi)\left(\frac{\partial\psi}{\partial t}\right) \tag{2.13}$$

where $c(\psi) = \frac{\partial\theta}{\partial\psi}$is the water capacity function defined as the slope of the Soil Water Characteristic Curve. Solving equations requires the SWCC and hydraulic conductivity function.

2.3.5 Determination of characteristics of unsaturated soils in Sri Lanka

Research done for the purpose of determining hydraulic parameters of unsaturated soils in Sri Lanka is quite limited. Vasanthan (2016) conducted a study with undisturbed samples obtained from the failed slope at Chainage 42+340 to 42+400 of Southern Express way. Results of that study was used by Idirimanna (2016) for the back analysis of the failure occurred at the same location.

In the study, Vasanthan (2016) conducted several experiments on the two undisturbed soil samples collected at the location. Particle size distribution and Atterberg limit tests were done and based on the results, soil was identified as Sandy Silt. Then following parameters were determined by various experiments.

- Permeability function – Using the wetting path and drying path test on a sample with tensiometers in the method of continuous measurement
- Soil Water Characteristic Curve (SWCC) – using Pressure Plate apparatus, Direct shear test with tensiometers
- Shear strength parameters – Direct shear test with the measurement of suction

The results of the permeability function obtained are shown in Figure 2.9.

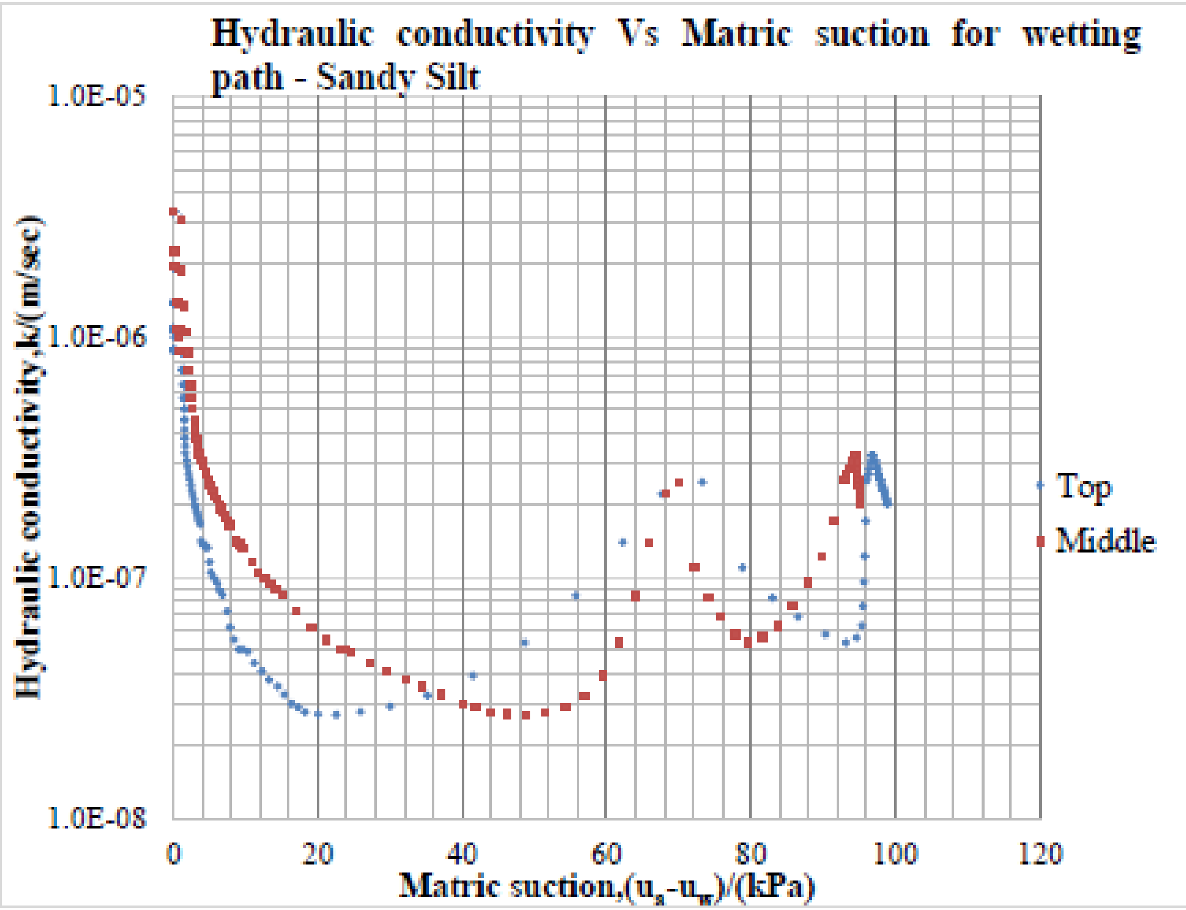


Figure 2.9 : Hydraulic Conductivity vs Matric Suction – Sandy Silt *(Vasanthan, 2016)*

The SWCC obtained for the soil samples using different methods such as wetting path, drying path and pressure plate and, comparison with the results of other researches are presented in Figure 2.10.

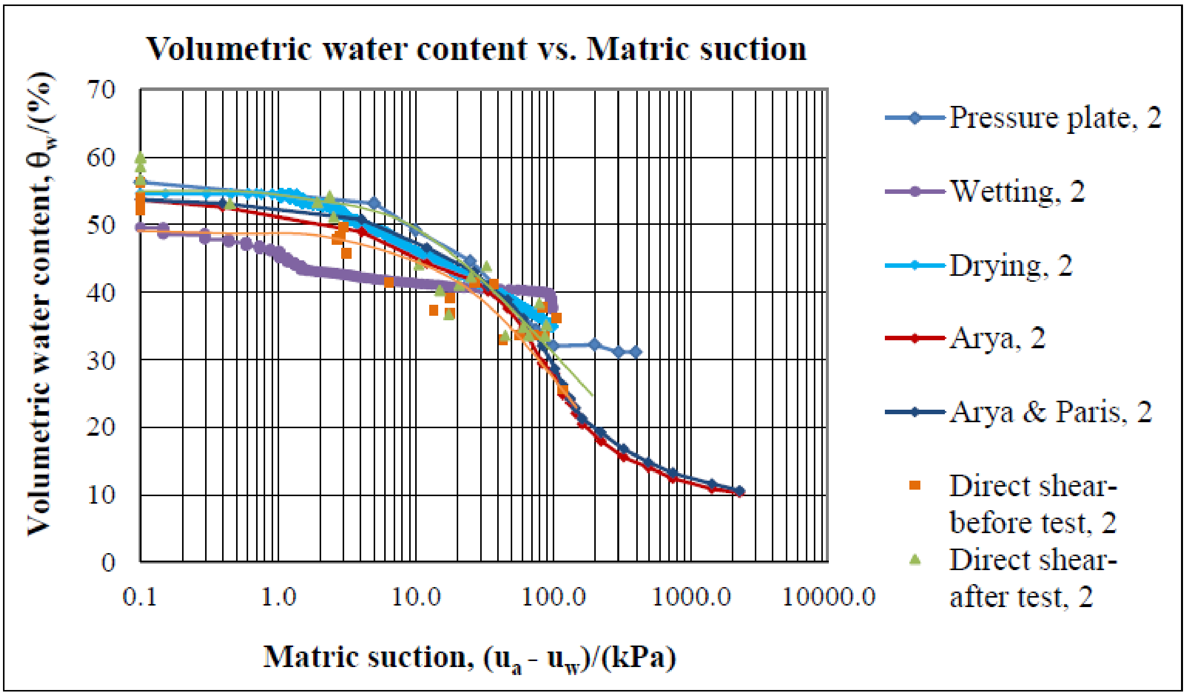


Figure 2.10 : SWCC for Silty Sand form Various Methods *(Vasanthan, 2016)*

2.3.6 Seepage analysis with SEEP/W - (GEOSLOPE International Ltd., 2017)

SEEP/W has the facility to simulate the movement of liquid water or water vapor through saturated and unsaturated porous media through finite element analysis. Basically steady-state and transient analysis can be done using SEEP/W and, resultant pore water pressures and water table can be used as input data for slope stability analysis by SLOPE/W.

Both saturated-only and saturated-unsaturated soil models are used in the SEEP/W analysis and parameters required for saturated-unsaturated soil model is summarized in Table 2.3.

Table 2.3 : Parameters Required for SEEP/W Analysis

Parameter	Symbol	Unit
Hydraulic Conductivity Function	$K(u_w)$	m/s
Soil Structure Compressibility	β	m^2/kN (1/kPa)
Volumetric Water Content Function	$\theta_w(u_w)$	
Anisotropy Ratio	Ky' / Kx'	
Rotation Angle	α	Degrees

Volumetric water content function

GeoStudio provides a number of methods for estimating the volumetric water content functions such as,

- Closed form equation according to the technique developed by Fredlund and Xing (Fredlund, et al., 1994)
- Closed form equation according to the technique developed by Genuchten (Genuchten, 1980)
- Sample volumetric water content functions available
- Modified Kovacs model (Aubertin, et al., 2003)
- Directly entering tabular data for volumetric water content and suction

Hydraulic conductivity function

GeoStudio provides two routines to estimate the hydraulic conductivity function from the saturated hydraulic conductivity and the volumetric water content function.

- Fredlund and Xing equation (Fredlund, et al., 1994)
- Equation proposed by van Genuchten (Genuchten, 1980)

Boundary conditions

The solution of the FEM equations is constrained by boundary conditions specified across the domain. There are mainly five types of boundary conditions available in SEEP/W such as total head, total flux, unit flux, unit gradient and pressure head. The potential seepage face review boundary should be used if a free surface (i.e., pore pressure equal to zero) may develop along the boundary. A seepage face review is also required if the applied water flux boundary condition is in excess of the infiltration capacity of the soil. The review process ensures that the maximum pore-water pressure along the discharge surface or on the infiltration boundary is zero. A potential seepage face review can be completed when using these boundary conditions: total head, total flux, unit flux and pressure head.

2.3.7 Modeling with SEEP/W

Infiltration process into a slope made with unsaturated soils was modeled by Sujeevan and Kulathilaka (2011) considering three types typical cut slopes from Southern Transport Development Project (STDP). The Case 1 was a slope made of a uniform residual soil. In Case

2, a less weathered layer (Weathered Rock-WR) underlies a thick residual soil layer (boundary of residual soil and weathered rock is the line IC in Figure 2.11). In Case 3, the less weathered layer underlies a thin residual soil layer (boundary of residual soil and weathered rock is the line JC in Figure 2.11). The second layer (WR) is with greater shear strength and lower permeability.

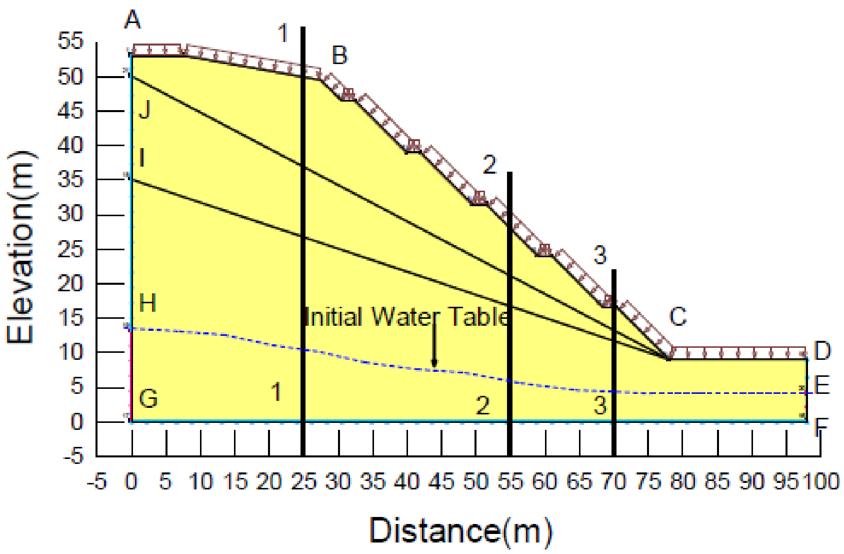


Figure 2.11 : SEEP/W Model with Boundary Condition *(Sujeevan & Kulathilaka, 2011)*

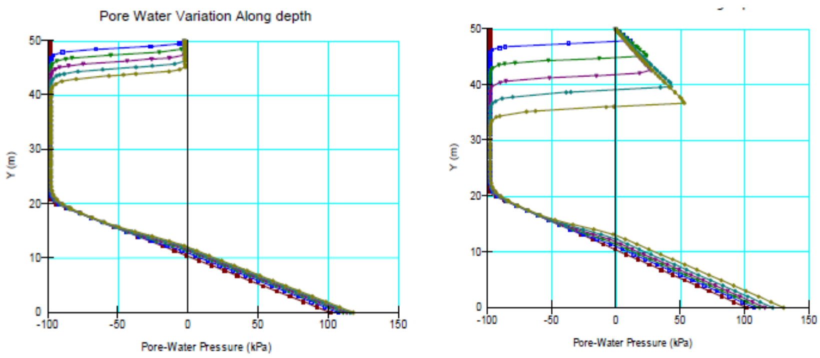
Following boundary conditions were used in the analysis;

- AB, BC, CD= I_r (Rainfall intensity)
- AH, DE, FG=Q=0m^3/s (No flow Boundary)
- EF, GH=h_t (Total head at sides)

Analysis was carried out for rainfall intensities of 5 mm/hr, 20 mm/hr, and 40 mm/hr. For each rainfall intensity pore water pressure variations was obtained for; 1, 2, 3, 4 and 5 days after the infiltration of rain water.

For all cases, the groundwater table is taken to be in the residual soil. Initially, the pore water pressure was considered to be increasing hydrostatically below the ground water table. Above the ground water table, the pore water pressures are taken as negative. The negative value was considered to increase linearly towards the ground surface. Two cases of negative pore water pressures were analyzed; a profile with linear increase and a profile with linear increase with an upper limit for matric suction at 100 kN/m^2. Only the case with an initial upper limit of 100 kN/m^2 on negative pore water pressure is presented here. As there were no experimentally determined SWCC and permeability functions for Sri Lankan soils, some appropriate standard curves available in the SEEP/W software were used in the analysis.

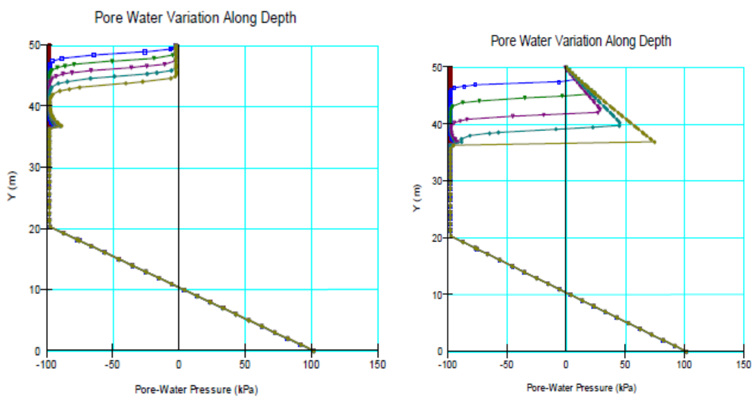
The changes in the pore water pressure regime with the rainfall was plotted for section 1 & section 2 (Figure 2.12). For homogenous condition (Case 1), at 5 mm/hr rainfall, as the rain progressed, matric suction depleted and eventually become zero close to the surface of the slope. At 20 mm/hr rainfall, not only the matric suctions were lost but also positive pore water pressures were developed at the top level – a perched water table condition. The development of the positive pore water pressure and the rise of the ground water table were more significant at lower levels.



(a) 5 mm/hr (b) 20 mm/hr

Figure 2.12 : Results of Case 1 _ Section 1-1 *(Sujeevan & Kulathilaka, 2011)*

When a highly weathered rock layer is underlying the residual soil, the downward movement of water is hampered and water gets accumulated at the boundary. It makes high positive pore water pressure with hydrostatic gradient above that boundary. In the meantime, there is no rise in ground water table due to this barrier effect. Variation of the pore water pressures with time for Case 3 with upper limit for matric suction, for 5 mm/hr and 20 mm/hr rainfalls are shown in Figure 2.13.



(a) 5 mm/hr (b) 20 mm/hr

Figure 2.13:- Results of Case 3 _ Section 1-1 *(Sujeevan & Kulathilaka, 2011)*

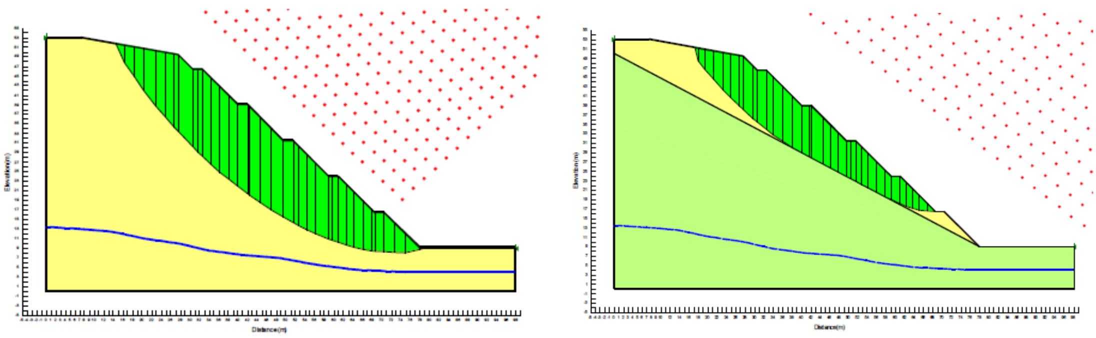
2.3.8 Coupled analysis of SEEP/W and SLOPE/W

Requirement of the seepage analysis is to assess the stability of slopes against different seepage conditions such as rainfall, drainage improvement etc. In order to fulfill this requirement, slope stability analysis should be done based on the results of seepage analysis. GeoStudio software (from version 2007 onwards) has unique feature in this regard to couple the SLOPE/W analysis with SEEP/W analysis and thereby resultant pore pressure conditions by SEEP/W analysis can be used for SLOPE/W analysis. Software also facilitate to perform stability analysis for different time intervals during the seepage process.

Assessment of infiltration effect on slope stability

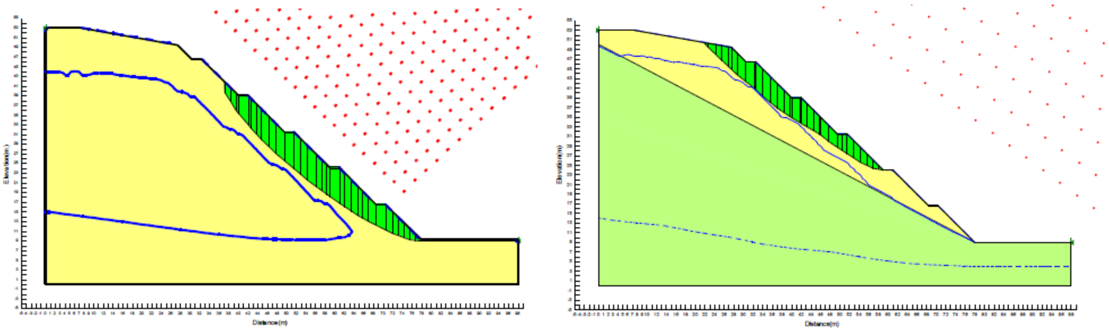
Kulathilaka and Sujeevan (2011) has coupled the SEEP/W analysis with SLOPE/W analysis by incorporating the pore water pressures derived from SEEP/W analysis to SLOPE/W analysis. This analysis was performed to extend the study on seepage further to assess how the stability of the slope is affected by infiltration with different rainfall intensities and different stratum of soil slope.

The shape of the critical failure surface was corresponding to the duration of the rainfall. At the initial stages of the rainfall or during the dry season, the prevailing high matric suctions near the surface induces greater shear strength. As such, critical failure surfaces are quite deep as shown in Figure 2.14 for the uniform slope and two layered slope. In two layered slope, critical slip surface follows the boundary of the two layers. With the rainfall infiltration the loss of matric suction and development of perched water table has developed near the surface. This loss of matric suction would reduce the apparent cohesion closer to the ground surface. As such, the critical failure surfaces corresponding to the latter stages are much shallower as illustrated by Figure 2.15.



(a) Uniform slope (b) Two layered slope

Figure 2.14 : Critical Failure Surface at Dry Condition *(Kulathilaka & Sujeevan, 2011)*



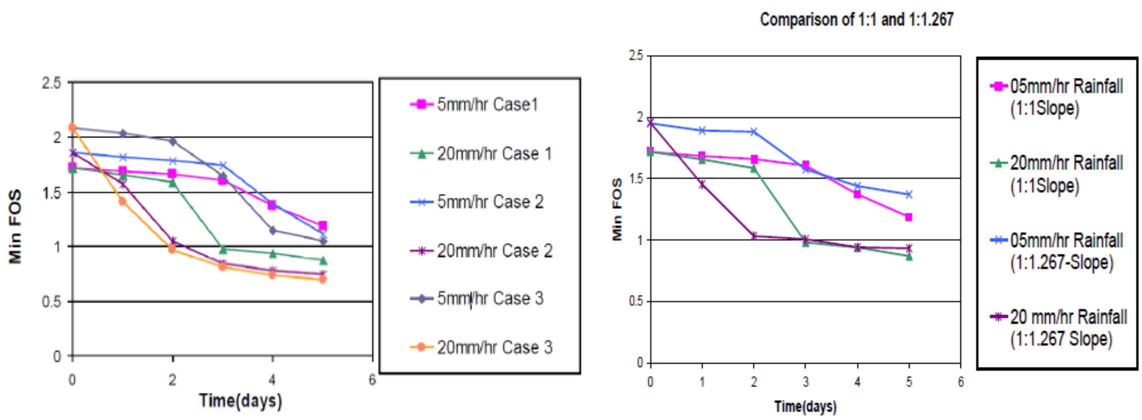
(a) Uniform slope (b) Two layered slope

Figure 2.15 : Critical Failure Surface at Wet Condition *(Kulathilaka & Sujeevan, 2011)*

It was found that the rainfalls of greater intensity are more unfavorable. But rainfall of intensity much greater than the saturated permeability of the soil will contribute to runoff. According to the analysis done by Kulathilaka and Sujeevan (2011), when a layer of much lower permeability (less weathered rock) underlies the residual soil, it will obstructs the infiltration and create a built up of excess pore water pressure at the boundary causing a negative effect on the stability of the slope.

Kulathilaka and Sujeevan (2011) presented the reduction of the safety margins of the slope with the rainfall, for the three idealized geological conditions as, uniform residual soil (case 1), thick layer of residual soil underlain by weathered rock (Case 2) and a thin layer of residual soil underlain by weathered rock (Case 3), (Figure 2.16 (a)).

Further, the study has been extended for a slope with much smaller gradient (1:1.267). Initially, the flatter slope had higher FoS values. However, as the rainfall progressed, FoS of falter slope is relatively less than the steeper slope as shown in Figure 2.16 (b) (i.e. 1^{st}& 2^{nd} day for 20 mm/hr rainfall and 3^{rd} day for 5 mm/hr rainfall).



(a) Different cases (b) Different slope angles – Case 1

Figure 2.16 : Variation of FoS with Duration of Rainfall *(Kulathilaka & Sujeevan, 2011)*

As such, it is clear that if measures in the form of minimizing infiltration are not adopted, making the slope flatter alone would not ensure safety (Kulathilaka & Sujeevan, 2011).

2.4 Design of mitigation measures / slope strengthening measures

If the existing safety margin obtained through the stability analysis of a slope under critical condition is inadequate, proactive techniques should be adopted to mitigate the risk. If a failure has already occurred the slope has to be rectified to enhance the safety margins to an acceptable level to prevent further failures. If the cost of the mitigation is very high, different approach is adopted. First, slope monitoring programme is established in order to find the risk level and critical location. Then, based on the results of the monitoring, control measures are implemented to minimize the damage and prevent loss of human lives.

Design of mitigation measures are done in staged approach. First, inexpensive simpler techniques are considered for enhancing the safety margins of slopes. If safety margins cannot be enhanced up to acceptable level by those measures, more expensive methods are adopted.

2.4.1 Drainage improvement

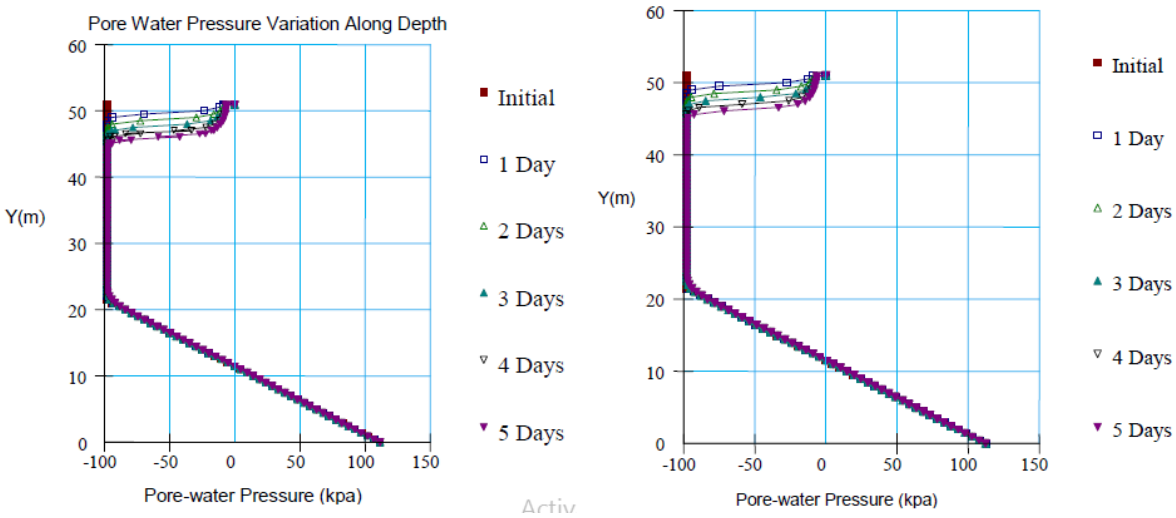
Drainage improvement is the most simple and essential mitigation measure for rain induced landslides and slope failures, which is relatively inexpensive. Therefore, drainage improvement is considered as the primary mitigation measures. Drainage improvement can be considered in two stages as surface drainage improvement and subsurface drainage improvement.

Surface drainage improvement

Surface drainage improvement is considered as the primary and most cost effective mitigation measure for rain induced landslides. Function of the surface drainage improvement is to minimize the infiltration due to rainfall and surface water flow from upper slope. Cutoff drains are provided to divert surface runoff from the unstable mass, landscaping to prevent stagnation of water within the unstable mass, use of berms at regular intervals with berm drains to rapidly divert water away from the unstable slope and cascade drains in steep slopes for rapidly discharge of surface water are some widely used surface drainage improvement measures. In addition to those techniques, vegetation by turfing, hydroseeding or planting is used for minimizing infiltration, surface erosion and gully erosion. When the rainwater infiltration is less, the possibility of development of perched water table is reduced and rise of ground water level is also minimized, thereby enhancing the safety margins of slope.

Kulathilaka and Kumara (2013) showed the effectiveness of the surface drainage improvement in cut slope stability during heavy rains by extending the same analysis done by Kulathilaka and Sujeevan (2011) for surface drainage improved condition. The influence of surface drainage measures were modeled with the software SEEP/W by incorporating a 100 mm thick layer of very low permeability 10^{-20} m/s over the berms representing the berm drains and a thin layer of low permeability over the slope surface to represent the vegetation cover. A parametric study was done by varying the permeability of the thin vegetation layer over the range 10^{-7} m/s to 10^{-9} m/s. According to the results of the analysis, 100 mm thick vegetation cover with sufficiently low permeability of 10^{-7} m/s along with berm drains and cascade drains cause a significant reduction in infiltration. All three cases of subsoil conditions analyzed by Kulathilaka & Sujeevan (2011) was analyzed.

Figure 2.17 shows the pore water pressure distribution of the Case 1 slope with surface drainage improvement for different rainfall intensities. Analysis results shows that, with the presence of surface drains complete depletion of the matric suction and development of positive pore pressures (perched water table) is prevented and rise of ground water table is minimized. Similar effects were observed for the Case 2 and Case 3 also.

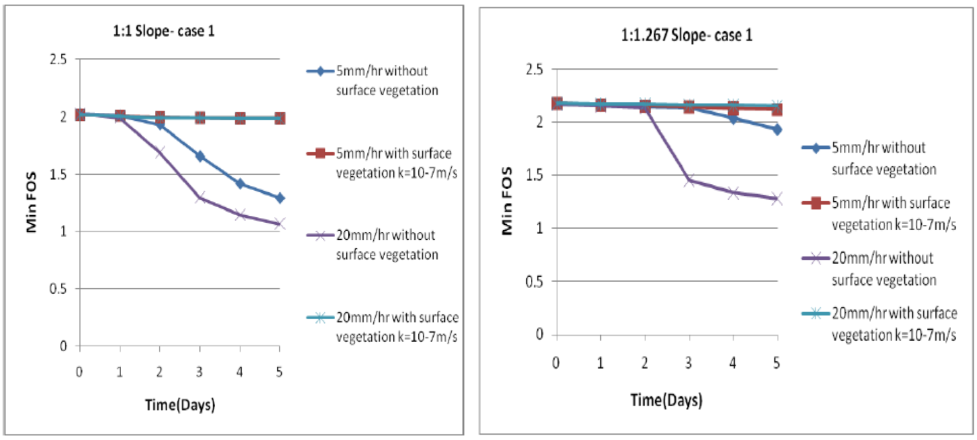


(a) 5 mm/hr (b) 20 mm/hr

Figure 2.17 : Results of Case 1 _ Section 1-1 *(Kulathilaka & Kumara, 2013)*

The effectiveness of the berm drains and vegetation cover in maintenance of a sufficient safety margin in the slope even with a prolonged rainfall is illustrated in Figure 2.18. It could be seen that the vegetation cover which reduced the infiltration and loss of matric suction had been effective in maintaining a significant margin of safety during the prolonged rainfall. When only berms drains are provided without protective vegetation cover, the FoS reduced significantly as the rain persists. When the vegetation cover is present the reduction of the factor of safety with the prolonged rainfall is very minimal. The difference of the factor of safety values corresponding to rainfall intensities of 5 mm/hr and 20 mm/hr is negligible.

Comparison with variation of the FoS of a mild slope with gradient 1:1.267 illustrated that reduction of the slope angle alone will not ensure a sufficient safety margin under prolonged rainfall condition. The provision of adequate surface drainage measures ensure that safety margins will not reduce significantly with the prolonged rainfalls.



(a) 1:1 slope (b) 1:1.267 slope

Figure 2.18 : Variation of FoS with Duration of Rainfall *(Kulathilaka & Kumara, 2013)*

Importance of surface drinage improvement was illustrated by Idirimanna (2016) by back analysis of failure occurred at Welipanna, Southern Expressway.

This is a cut slope of height about 35 m and there were four berms. Berm drains and cascade drains were constructed with turfing as surface protection (Figure 2.19 (a)). Site geology was quite non uniform. There were rock outcrops and five different joint systems. Boudinage structures embedded in the soil mass were identified.

A failure occurred in the evening is illustrated in Figure 2.19 (b). The tension crack that showed up in the morning indicating the imminent failure can be seen in Figure 2.19 (a). The failure was back analyzed in the form of a parametric study considering possible scenarios of defective system of surface drains and the presence of relict joints. Identification of faults in the system of drains prompted this approach.

(a) Slope just before failure (tension crack open) (b) Slope after failure

Figure 2.19 : Failure at Welipenna on Southern Expressway *(Idirimanna, 2016)*

Stability of the slope before failure was analyzed against the actual rainfall encountered at the time of failure, which is a prolong rainfall about 6 days with maximum of 7 mm/hr. Surface drainage system had been constructed for this slope at the road construction stage. However, surface drains had not functioned properly at the time of failure. The analysis was done for four cases such as; with & without surface drains and with & without relict joints systems. Results of analysis are presented in Figure 2.20. It clearly shows that the FoS value is less than unity at the time of failure when there are relict joints and when the surface drainage system is not present or else not functioning well. That implies that if the drainage measurers were well maintained and functioning well, this failure could have been prevented.

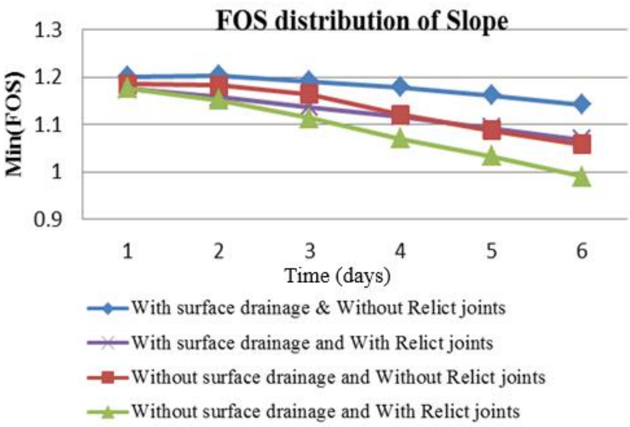


Figure 2.20 : FoS Variation with Time *(Idirimanna, 2016)*

Further, the modeling of infiltration process under different conditions revealed that the rise of ground water table is quite significant at the toe of the slope. A lesson learn was that when natural slopes are excavated into steeper profiles for construction of highways there should be a series of sub horizontal drains at the toe level even if the ground water table is found to be lower than the toe level in general.

Subsurface drainage improvement

Although surface drainage improvement minimizes the infiltrations, ground water table may rise due to remaining infiltration and subsurface water flow. Such rise of ground water table may lead to lowering the safety margin of the slope below acceptable level. Subsurface drainage improvement is required in such situation to lower the ground water table. Trench drains, sub horizontal perforated gravity drains and drainage wells are some types of subsurface drains in use.

Sub horizontal gravity drains (HD) are perforated or slotted pipes installed in to the slope in an appropriate orientation (Figure 2.21). The simplest form is a straight pipe of appropriate length placed at a desired horizontal spacing at different level in the slope (Figure 2.22).

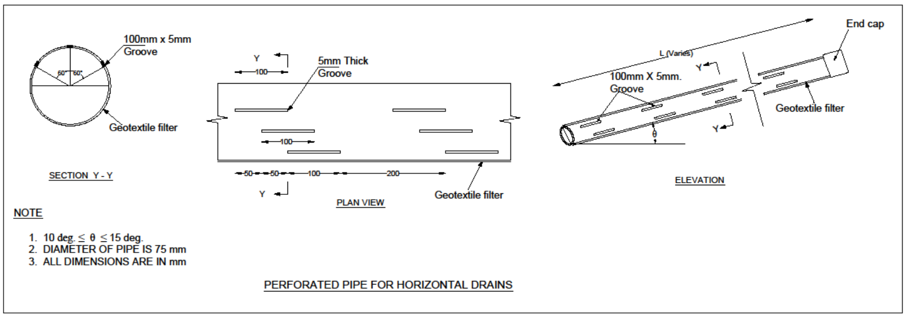


Figure 2.21 : Perforated Pipe for HD *(National Building Research Organisation, 2016)*

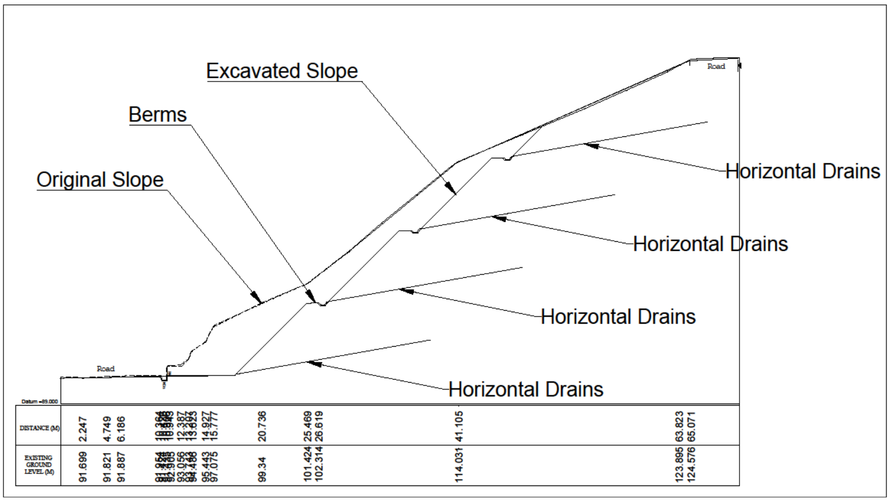


Figure 2.22 : Arrangement of HD *(National Building Research Organisation, 2016)*

Those may be installed at several levels in a slope and are particularly useful in cut slopes of limited extent. Other forms of subsurface drains such as, directional drains following the failure surface (Watawala) or series of radial drains at different levels (Badulusirigama) may be required potential landslides of longer extent.

2.4.2 Use of earth retaining structures

If the required safety margins cannot be achieved through surface and subsurface drainage improvement, earth retaining structures of different forms are used to enhance the safety margins.

Retaining structures that can be used to enhance the stability of slopes may be of externally stabilized type that take the integrated force exerted by the soil mass or internally stabilized type that reinforces the soil and enhance the shear strength.

Gravity retaining walls made of random rubble masonry, mass concrete, gabion or crib systems may be placed at the toe of a slope to enhance the safety. They take up the force exerted by the soil mass behind it and resist it by the self weight, without getting overturning or sliding forward. The wall should be of sufficient weight and foundation should be placed on a competent soil layer to ensure that there is no bearing capacity failure. The possibility of deep seated failure should also be checked. Such gravity retaining structures constructed at the toe will take up a significant space. The cost of the toe gravity retaining wall can be minimized if the wall is done with a back batter.

Embedded type structures made of techniques of driving prefabricated sheet of steel or concrete (sheet piles) or forming in-situ concrete piles extended beyond the potential failure zone is another form externally stabilized structures. The necessary depths of embedment should be evaluated by a detail design.

Alternatively, the potential failure mass should be reinforced by intrusion of elements that can take up tensile stress at regular intervals vertically and horizontally. Those intrusions should extend beyond the potential failure surface. With initiation of movements tensile force will mobilized in the intrusions and shear resistance in the potential failure plane will be increased. These intrusions should be of sufficient tensile strength and should embedded to sufficient distance to prevent getting pulled out.

Soil nailing is a passive system where the intrusions are not pretensioned and forces are mobilized with movement in the soil mass. In anchors the elements are pretensioned to a sufficient tensile force. Both systems do not take additional space and can be installed at different height of the slope.

Soil nailing

As discussed previously, soil nailing is a technique of reinforcing in-situ soil by using passive intrusions from the facing and proceeding from top down. Followings are some advantages of soil nailing techniques with compared to externally stabilized systems.

- Low Cost
- Light Construction Equipment
- Adaptability to Different Soil Conditions
- Flexibility
- Reinforcement Redundancy

The mechanism of stabilization in soil nailing is illustrated in Figure 2.23.

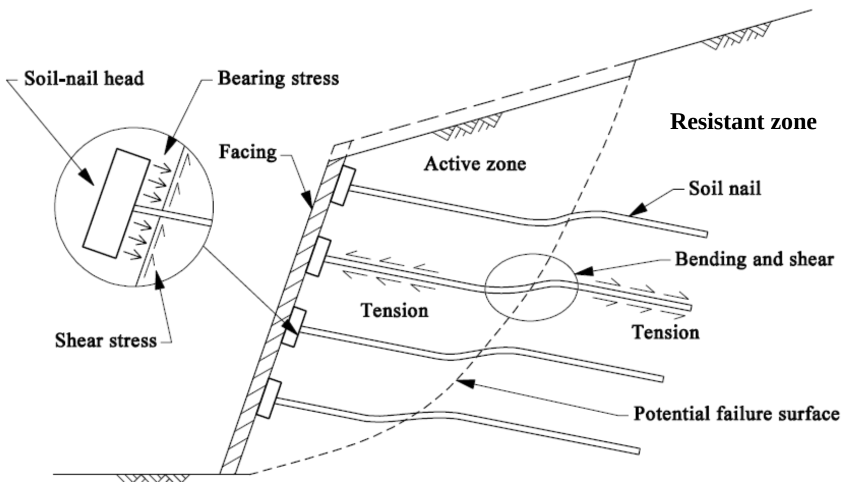


Figure 2.23 : Soil nailing system *(GEO, Hong Kong, 2008)*

Soil Nailing systems are generally assed using two zone model, namely the active zone and the resistant zone, which are separated by a potential failure surface as illustrate in Figure 2.23. The active zone is the region in front of the potential failure surface, where it has a tendency to detach from the soil-nailed system. The resistant zone is the region behind the potential failure surface, where it remains more or less intact. The soil nails act to tie the active zone to the resistant zone. (GEO, Hong Kong, 2008).

Soil nails are functioning when the slope experiences some movement. Tensile stress developed in the soil nails which transfers to the resistance zone reducing the shear stress required to be mobilized for stability and increases the normal stress on the failure surface (Equation 2.14). Both these phenomena increase the safety margin of a slope. For this to be effective, nails should extend to a sufficient distance beyond the failure surface (into the resistant zone) and should be capable of mobilizing a sufficiently large tensile force. The critical factors in a soil nailing design are the pullout resistance the nail develops at the interaction of soil/grout interface in the resistant zone and the tensile strength of the reinforcement bar.

Two analytical models were developed by Kulathilaka & Mettananda (2002) for analysis of stability of soil-nailed system. Analytical model based on the Bishop's Simplefied Method (Bishop, 1955) can be used for circular failure surfaces and it is described below.

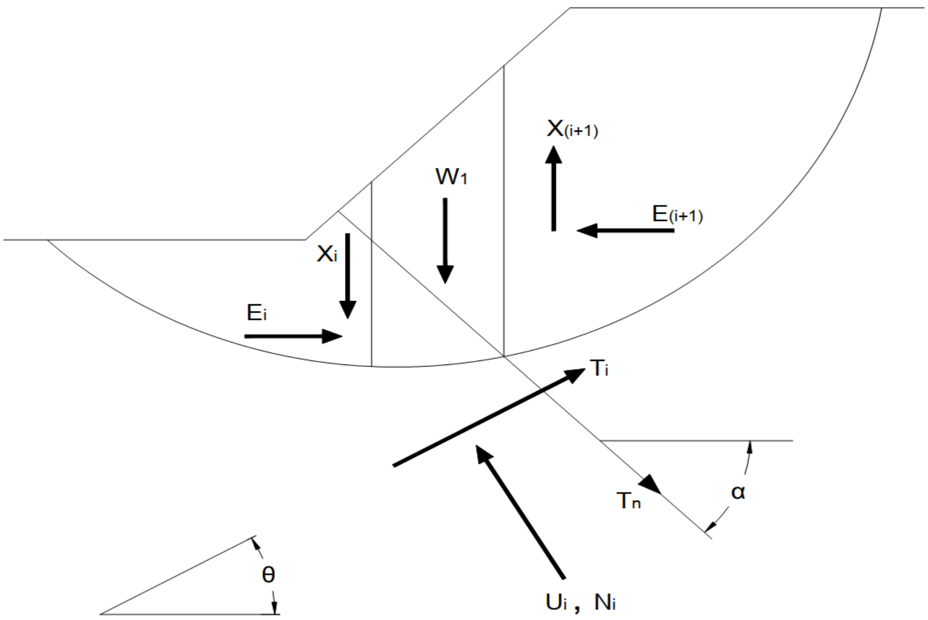


Figure 2.24 : Forces on Potential Failure Mass *(Kulathilaka & Mettananda, 2002)*

Figure 2.24 illustrate the forces acting on a particular slice *i*.

Where;

- $T_{N,i}$ is the total nail force acting on the slice *i*, which is acting at an angle of α_i to the horizontal, and also passing through the center of the failure surface segment of the slice *I*,
- E_i, E_{i+1} are the interslice normal effective stresses
- T_i, N_i' are resultants of the shear and normal effective stresses acting along the segment of the failure arc
- U_i is the pore water pressure forces acting along the segment of the failure arc
- W_i is the weight of the slice

Resultant FoS is given by;

$$FoS = \frac{\sum\left\{\left[c'\Delta x_i + (w_i + Q_i - U_i \Delta x_i + T_N \sin\alpha)\tan\varphi'\right]\left[{}^{1}/_{M_i(\theta)}\right]\right\}}{\sum\left[(w_i + Q_i)\sin\theta_i - T_{N,i}\cos(\theta_i + \alpha_i)\right]} \tag{2.14}$$

Where:

$$M_i(\theta) = \left(\cos\theta_i + \frac{\tan\varphi' \sin\theta_i}{FoS}\right) \tag{2.15}$$

Since the FoS presented at the both sides of the equation, iteration process is used for solving the equation. FoS calculation equation (Equation 2.8) shows that nail force $T_{N,i}$ has increased the shear resistance force and reduced the shear stress force. Thereby FoS has been increased by application of soil nails.

Design of soil nailing systems is done considering the global stability. Generally, GeoStudio SLOPE/W software is used for design of soil nail systems with an appropriate method for stability analysis such as Bishop method or Spencer method.

In soil nailing technique reinforcement bars (galvanized steel bars of 25 mm - 32 mm diameter) are installed in to drill holes of 100 mm - 125 mm diameter and grouted. The nails are installed with a downward inclination of 15^0-20^0 and typically with 1.5 m - 2.5 m vertical and horizontal spacing. Length of the nails and spacing are based on the design requirement. Typical nail lengths are in the range of 6 m – 16 m.

With particularly high slopes and with deep failure surfaces, the required length of reinforcement would be high and soil nailing technique may not be effective. If the stability analysis indicates that reinforcement of excessive lengths are required, it is customary to use anchors which are pre-stressed. Cable anchors installed to the designed length are post tensioned to a designed stress level.

The nails should be connected at the slope surface to have and integrated action and an appropriate surfacing should be provided. Different options available for surfacing are;

- Connecting the nail heads by beams and shotcreting the surface fully
- Connecting the nail heads by beams and strengthen the slope in between the beams with the help of a steel mesh and introduce vegetation
- Using a concrete block as the nail heads and connecting them with high tensile steel mesh and introduce vegetation by hydroseeding with the help of a coir mesh

In addition to the global stability considered by the conventional slope stability analysis. It is necessary to ensure the stability of the facing as well. With the techniques of introducing vegetation in between the nail heads, the stabilized slope will blend well with the natural environment as in contrast to a shotcreted facing.

2.5 Combine effect of drainage improvement and soil nailing

Surface water runoff and existing groundwater conditions should be properly controlled to ensure satisfactory performance of a soil-nailed system, both during construction and throughout its design life. Concentrated surface water flows may result in erosion, washout failures, or shallow landslides. Build-up of high groundwater pressures behind the system may result in reduction of its overall stability. With high ground water table the required nail lengths to satisfy the stability would be very long. High groundwater levels may also adversely affect the grout quality as well as accelerate the corrosion rate of steel reinforcement. Suitable surface drainage provisions and subsurface drainage provisions should be provided to soil-nailed systems based on the actual site conditions (GEO, Hong Kong, 2008). Further, combine effect of soil nailing and drainage improvement is important in economizing the soil nail design.

2.6 Case histories

There are many landslide mitigation projects implemented all over the landslide prone areas in the country with the funding of several agencies such as World Bank, Japanese International Cooperation Agency (JICA) and Government of Sri Lanka etc. Several landslide mitigation projects carried out adopting the aforementioned concepts are briefly discussed in the proceeding Sections.

2.6.1 Rectification of Watawala landslide

Rectification of Watawala landslide is an example for landslide mitigation with surface and subsurface drainage improvement.

Watawala landslide mitigation is the first large scale landslide mitigation work carried out in Sri Lanka. It was a landslide that initiated in 1950's and had been moving periodically in the periods of heavy rain. A major failure occurred in June 1992 damaging nearly 100 m length of the Colombo - Badulla railway track between Galboda and Watawala (Figure 2.25). The landslide was found to cover an area of approximately 22,920 m^2 and about 322,700 m^3 slide volume.

Figure 2.25 : Watawala Landslide after Failure *(Rajaratnam & Bhandari, 1994)*

Failure surface was identified by detailed geotechnical investigation (Figure 2.26) and, design shows that the slope is quite stable when the ground water table is low. Therefore, well designed surface and subsurface drainage improvement measures were adopted in order to maintain the ground water level below the critical level. Subsurface drainage improvement was considered as the major component and long sub horizontal gravity drains were installed along the slip surface to facilitate rapid dissipation of pore pressures.

With the deep failure surface of very long extent, perforated pipes in one direction would not be feasible. Therefore, directional controlled boring with aid of computer controlled drilling machine was used for installation of sub horizontal gravity drains. Perforated HDPE (High Density Polyethylene)) pipes that could withstand a large strain up to about 800% was used as drainage pipes.

Additionally, several automatically operated pumping wells were installed to pump out the ground water if the ground water level rise above the critical level due to malfunction of subsurface drains. Mitigation measures are shown in schematically Figure 2.27.

With aforementioned drainage improvement measures Watawala landslide was stabilized and there was no any reactivation since the mitigation.

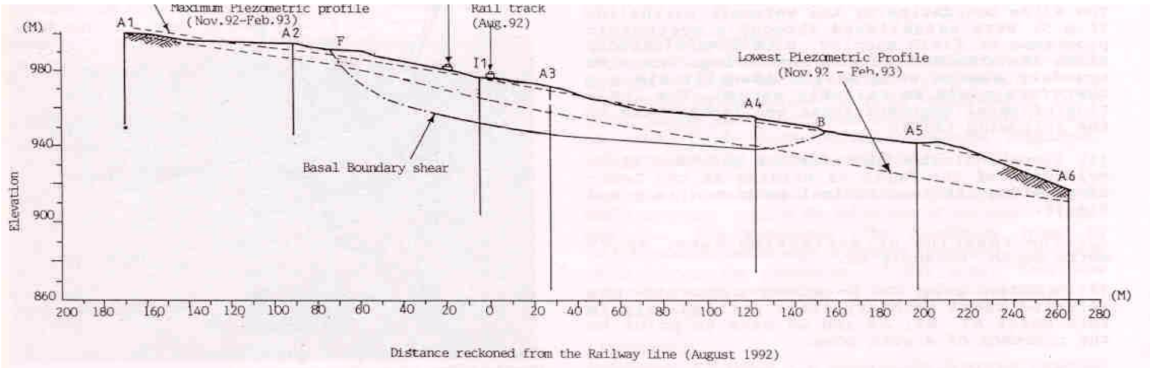


Figure 2.26 : Slip Surface of Watawala Landslide *(Rajaratnam & Bhandari, 1994)*

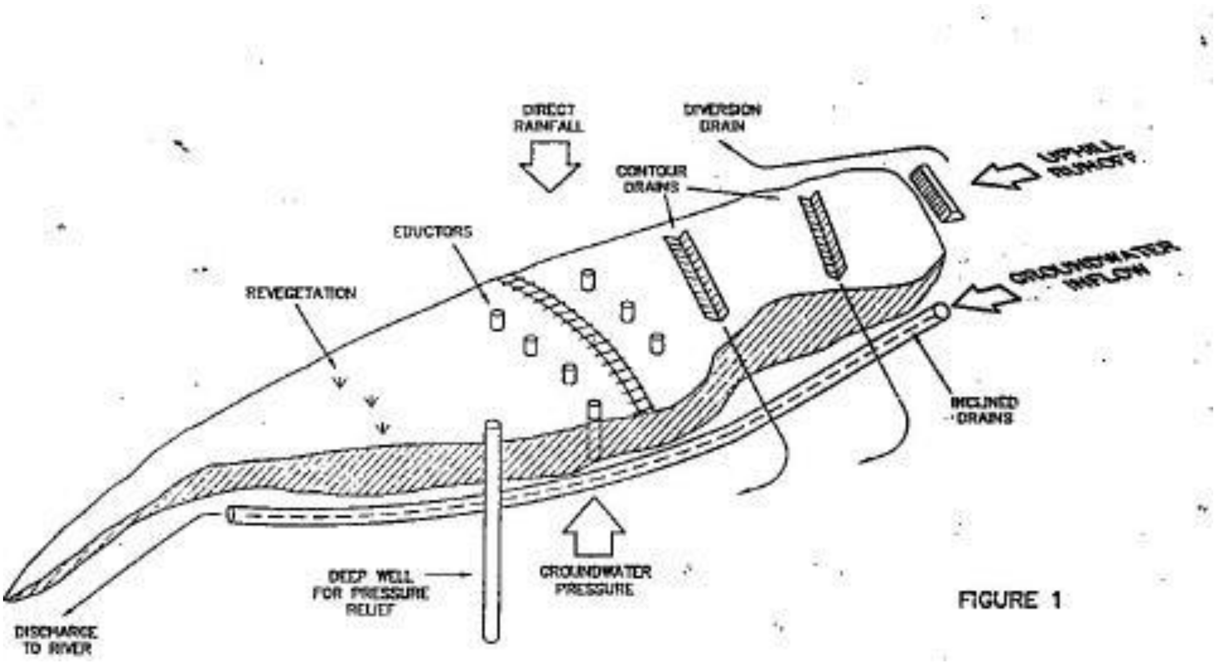


Figure 2.27 : Mitigation Measures of Watawala Landslide *(Rajaratnam & Bhandari, 1994)*

2.6.2 Rectification of Badulusirigama landslide

Badulusirigama is village located close to the Uva-Wellassa University and a landslide was occurred in 2011 damaging the whole village by debris flow. The landslide was identified as a shallow and long extend slide with three moving segments by a detailed geotechnical investigation. Landslide had been occurred on the thick colluvium soil layer Cross section through the landslide is presented in Figure 2.28.

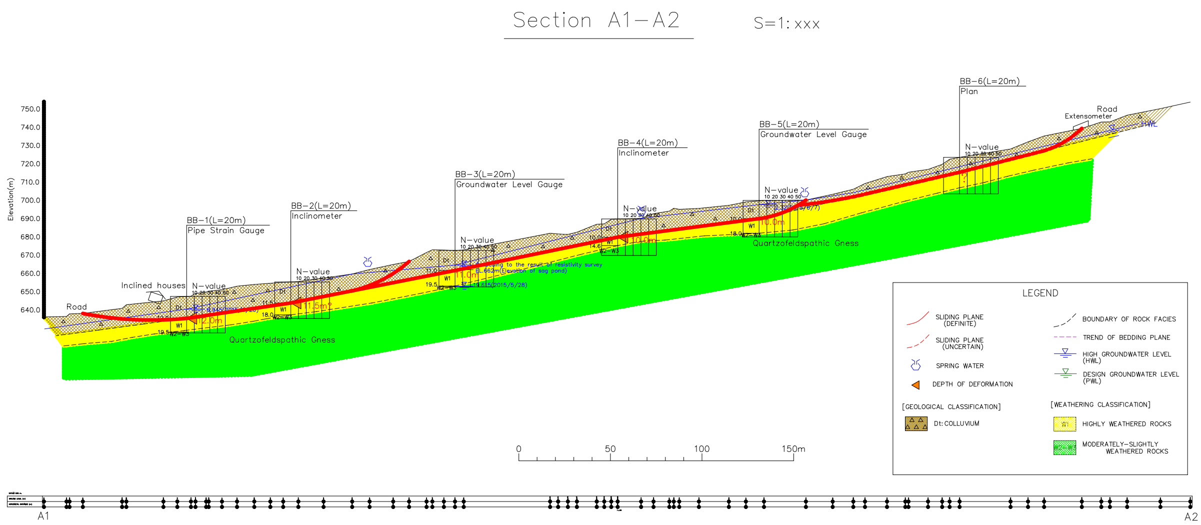


Figure 2.28 : Cross Section through the Landslide *(Amada, 2016)*

Subsurface and surface drainage improvement was applied as mitigation measures in order to maintain the stability of the slope by keeping ground water table at lower levels. Considering the geometry of the slope and landslide area, sub horizontal gravity drains were installed in an radial orientation at different levels covering the entire landslide mass (Figure 2.29).

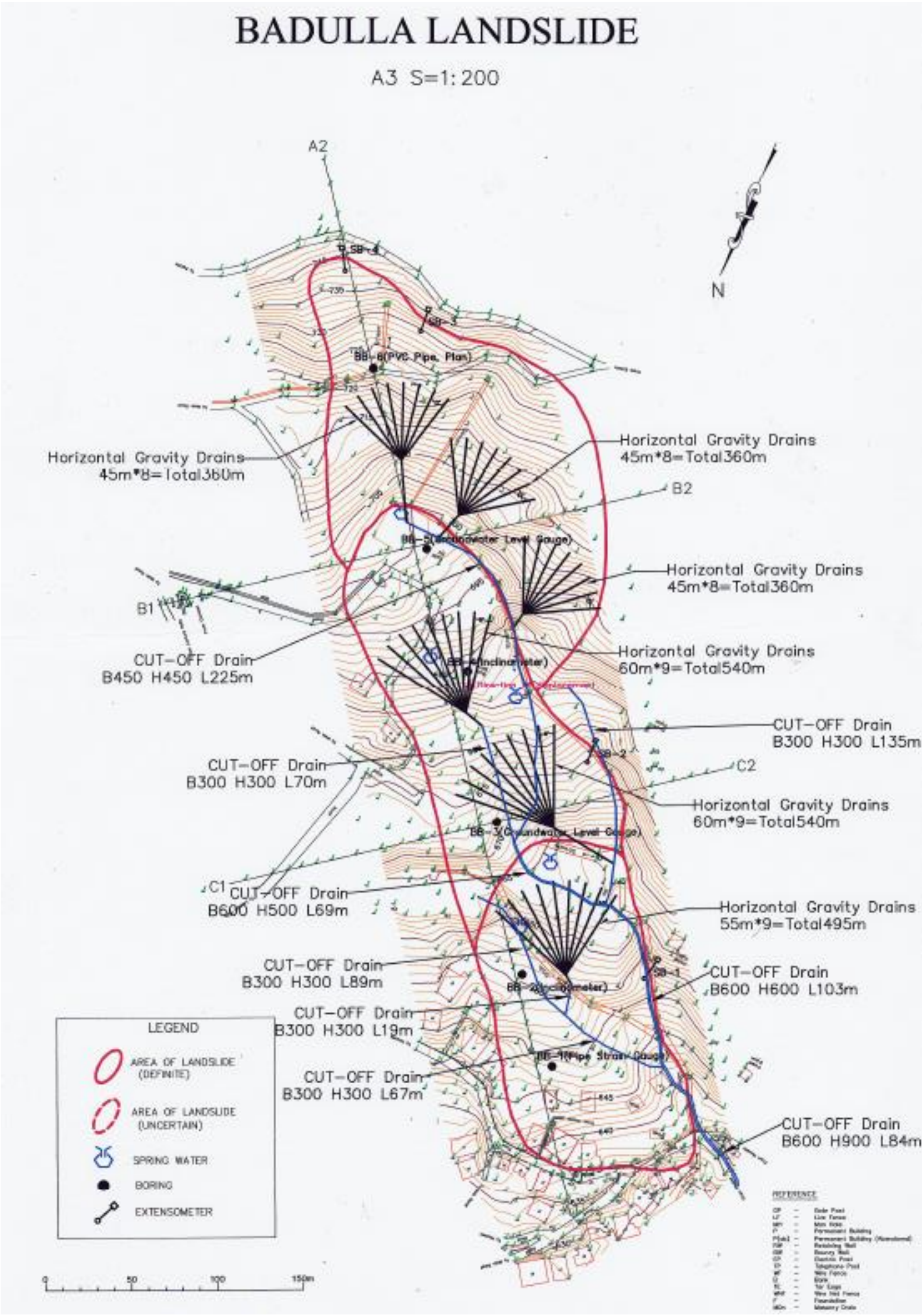


Figure 2.29 : Plan View of Mitigation Measures of Badulusirigama *(Amada, 2016)*

2.6.3 Rectification of failure at Welipenna on Southern Expressway

Rectification of failure at Welipenna is a good example to emphasize the importance of drainage improvement on slope stability and mitigation with combination of mitigation measures.

The failure occurred in a cut slope at location 42+340 to 42+400 in the Southern Expressway after few days of rain few years after commissioning the road. The failure surface was quite shallow. The debris of the failure were quite dry and the ground water table was quite low. Close examination indicated that one cascade drain had cracked sometime prior to failure and water had been leaking into the slope. Joint systems were identified in the rocks in the area and adversely oriented relict joints filled with water were identified during the rectification process. Back analysis of the reveals that the failure could have been prevented if drainage system was functioned well (Idirimanna, 2016).

Analysis of the stability of the slope sections that prevailed after failure indicated that the slope profile is not safe unless high matric suction values exist. As such, rectification measures were adopted involving; strengthening the existing scar of failure (with some minor cutting back) by soil nailing after removal of the debris, reconstruction of surface drainage and use of a series of long sub horizontal drains. After mitigation of the upper area of the slope, toe protection is provided by a gabion retaining structure. Soil nailing design involved nails of 16 m length at the highest location. Later it was decided to use cable anchors in place of those long nails. The other nails were of length 12 m. Number of long horizontal drains were done at identified locations of water filled relict joints to facilitate rapid release of water.

Staged construction with proper sequence was adopted to maintain the stability of the slope during construction. Soil nailing and anchoring was done simultaneously with the excavation of the slope. Excavation for toe retaining structure was done after stabilizing the upper slope. The installation of sub horizontal gravity drains were done last after completing all the grouts work in the nailing to prevent any grout intrusion through the interconnected relit joints. Figure 2.30 shows the mitigation measures applied for slope strengthening.

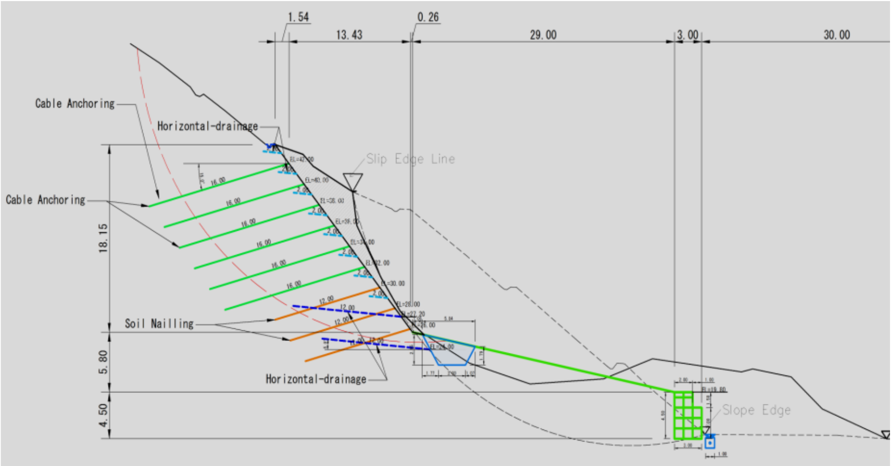


Figure 2.30 : Mitigation Measures of Welipenna Landslide *(Idirimanna, 2016)*

Figure 2.31 shows the conditions just after failure (a) and after the rectification (b).

(a) After Failure (b) After Rectification

Figure 2.31 : Failure at Welipenna on Southern Expressway *(Idirimanna, 2016)*

2.6.4 Rectification of failures at Kandy - Mahiyangana road

Several slope failures have occurred on the Kandy – Mahiyangana road after widening. Among those failures eighteen locations were identified as critical locations and mitigation project was carried out for slope strengthening.

Cutting failures had been occurred in most of the locations where steep high cuts were done without any drainage measures. Failures observed in some locations further propagated during the periods of rainy weather.

As failures of all locations were caused by rainfall, surface drainage improvement was considered as the primary mitigation measure. Cutoff drains were constructed above the crests of the unstable areas in order to divert water away and discharge in to the lower areas by means of cascade drains. Surface drainage improvement within the unstable masses was done to rapidly dissipate the water away. In some locations required safety margins were achieved only by surface drainage improvement. Sub horizontal gravity drains were used in some locations. In addition, soil nailing was also used in some locations.

In some locations (Location 08 and Location 18) Gabion walls were used as toe retaining structures along with drainage measures. In Location 08 failures has propagated and propagated further back (Figure 2.32 (a)), and original design had to be revised to adopt the new scope. Gabion wall was constructed at the toe to support loose materials in the landslide body. Compaction of top soil layers and landscaping was done to reduce infiltration and prevent water stagnation. Berm drains connected to cascade drains were used to rapid discharge of rain water away from the landslide body. Turfing was done on the slope to prevent erosion. Sub horizontal gravity drains were installed at berm levels and toe level to lower the ground water table. Scar of 1 m – 2 m height at the crest was cut back into 55^0 angle. With all these mitigation measures required safety margin was achieved. Slope after mitigation is presented in Figure 2.32 (b).

(a) After Failure (b) After Rectification

Figure 2.32 : Kandy – Mahiyangana Road Location 08

Some locations had steep high cuts and required safety margins could not be achieved only by surface drainage improvement. Therefore, soil nailing had to be installed in order to enhance the safety margins of the slopes up to required level. Soil nails of 25 mm diameter with lengths varying from 6 m – 12 m were installed and grouted in to 100 mm diameter bore holes. Surface protections was done by shotcreting or vegetation on the slope based on the site conditions. Shotcreting was used in locations where heavy erosion is possible and vegetation is not applicable due to soil condition. Figure 2.33 shows a typical location with soil nailing, drains and surface protection with both shotcreting (for closely spaced fractured rock mass) and vegetation (residual soils present at the surface) as mitigation measures. Figure 2.34 shows a typical location with soil nailing, drains and surface protection with individual concrete pillow, high tensile wire mesh and vegetation as mitigation measures

Figure 2.33 : Soil Nailing and Facing with Shotcreting and Vegetation

Figure 2.34 : Mitigation Measures with Soil Nailing and Facing with Individual Pillows Connected by High Tensile Wire Mesh, and Vegetation

2.6.5 Rectification of valley failure on Sothern Expressway Extension

There was a valley (formed probably by an earlier landslide) on Sothern Expressway Extension Galle – Matara section from chainage 41+630 to 41+635 and all the water in the natural slope above were flowing towards this valley. The underlying rocks are mostly gneisses rocks and weathered with lateritic caps present at the crest of the original ridge. A weak clayey soil formed by the weathering of feldspar rich migmatitic gneissic rock is seen in the lower levels of the cut. The failure (Figure 2.35 (a)) had been triggered by excessive infiltration into this soil formation during the prolonged period of rainfall. Berm drains or any other surface drainage measures had not been constructed by the time of failure.

The stability was restored by soil nailing after the removal of the debris. Shotcreting has to be used for surface erosion protection. The excessive amount of runoff moving towards the valley has to be systematically directed downward by the construction of a basin drain and connecting the same to a cascade drain. Slope after rectification is shown in Figure 2.35 (b).

This can be considered as an illustration of combined use of drainage and reinforcement to enhance the stability of a slope to desired level.

(a) After Failure (b) After Rectification

Figure 2.35 : Failure at 41 km on Southern Expressway Extension

2.7 Concluding comments

Rain induced slope failures in residual and colluvial soil formations are quite complex. Inherited variability of the soil formation with material of different levels of weathering and hence of different shear strength and permeability makes the response to a rainfall very complicated to analyze. The presence of relict joints adds to the complexity.

The rainwater infiltration into such slopes and its effect on the pore pressure regime can be understood to a reasonable level by careful idealization of the slope and assignment of appropriate hydraulic characteristics.

The changes to the pore pressure regime obtained by such analyses can be incorporated into the stability analyzes. Stability analyzes are generally done with limit equilibrium approach and it is necessary to have an advanced method such as Spencer method which can handle both circular and non-circular failure surfaces and consider both moment and force equilibrium in the computation process.

Computer software are used for seepage and stability analysis. In this regards commercial software GeoStudio SEEP/W and SLOPE/W are used. The modeling of SEEP/W and SLOPE/W can be combined together.

If the slope is found to be unstable by the analysis under critical rainfall, the stability can be enhanced with mitigation measures such as,

- Surface drainage
- Subsurface drainage
- Gravity retaining structures at the toe
- Embedded retaining structures inserted deeper beyond the potential failure surface
- Slope reinforcing by soil nails / anchors

The design of rectification measures should be done with a staged approach. Some slopes can be stabilized with surface drainage measures only. Some may need subsurface drains in addition to surface drains. The subsurface drainage measures can vary from simple form of perforated drain pipe in one direction to drainage pipes in radial directions at different levels, directional drains at deeper levels or drainage wells.

When the safety levels achieve by drainage measures are not adequate, structural measures in the form of gravity structures at the toe and reinforcement of soil will have to be used additionally. In the work reported in this research, these multiple methods of rectifications were used.

CHAPTER 3 - BACKGROUND OF THE FAILURE, DETAIL INVESTIGATION, DESIGN OF MITIGATION MEASURES AND MONITORING

This chapter describes the background of the landslide, initial investigation and detail geotechnical investigation carried out to obtain the data for the rectification of the slope. The proposed mitigation design and the field monitoring scheme is also presented.

3.1 Reactivation of the Ginigathhena landslide

3.1.1 Background

The ancient landslide at bridge no. 48/2, near Ginigathhena area at Avissawella-Hatton-Nuwaraeliya road was reactivated by some minor excavation at toe for widening of the bridge.

The landslide is located at the upper slope of the bridge no 48/2 (Longitude 80^0 27.717'E, Latitude 06^0 59.480'N). The area belongs to Pitawala GN divisions of Ambagamuwa divisional secretariat in Nuwaraeliya district. The average elevation of this location is about 407 m from the MSL. The area can be easily accessible from Avissawella-Hatton-Nuwaraeliya road (A007) via Ginigathhena (Figure 3.1).

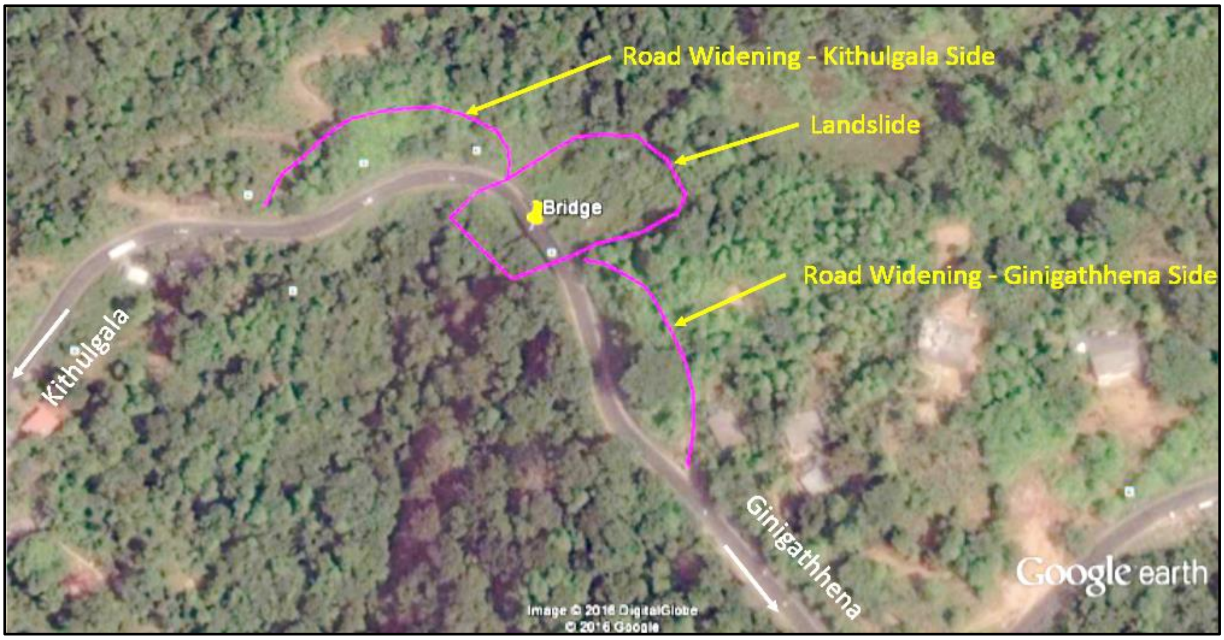


Figure 3.1 : Location Map

A mass movement had occurred on 22nd June 2014 followed by the excavation works at the toe of the soil slope (Figure 3.2). Excavation had been done for the proposed widening of the bridge. It was reported that there was no rain at the time of failure, but it has been further activated with continuous heavy rain received few days later into the area. According to information received from the community living in the area and the field evidence, numbers of other cutting failures had occurred within the area during rainy seasons. Geomorphological features at the site indicates that there had been a landslide earlier (Figure 3.3). Several tension cracks have been developed in the middle and head part of the landslide and some boulders were slightly moved towards the road side as shown in Figure 3.4.

(a) Failure

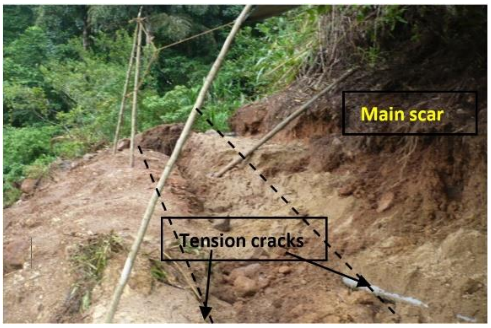
(b) Toe of the failure

(c) Crest of the failure

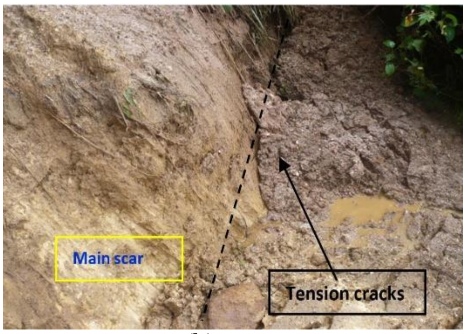
(d) Stream beside landslide

Figure 3.2 : Landslide on June 2014 *(National Building Research Organisation, 2014)*

Figure 3.3 : Ginigathhena Landslide Body



(a) Tension crack



(b) Main scar

(c) Loose mass

(d) Open tension crack

(e) Unstable boulders

Figure 3.4 : Landslide Body *(National Building Research Organisation, 2014)*

The subsequent investigations revealed that the main landslide area is a pseudo-stable colluvium mass spreading over an area of about 800 m^2 with total potential area of about 3,000 m^2. Therefore, a large quantity of soil and rock boulders can be released if this mass is released at once by a failure during rainy period. It is clearly visible that present unstable features may develop further and such situation would lead to a deep seated failure demolishing the bridge no. 48/2 in the main road thereby severely disrupting main traffic connection of Avissawella-Hatton-Nuwaraeliya Road (A007). This road is the main access and means of transportation of economical products such as vegetables, tea to the nearby cities and even to Colombo via Avissawella where a lot of communities around Ginigathhena area depend on. Therefore, mitigation of this landslide is critically important to secure the economy of the region and of the country as well.

Also, the widening of the bridge is necessary to cater increasing traffic flow. This required a cut back in to the slope at the landslide and at two adjacent ends. This cutting back has to be designed to be stable against all aspects.

Considering all the above facts, overall impact and the consequences, requirement of rectification is justifiable and the landslide without mitigation measures can be a serious issue to all the above elements at risk.

3.1.2 Geological and hydrological features

While the major landslide area consists of thick colluvial deposit, the uppermost slopes consist of residual and colluvium soil. The residual soil mainly consists of clayey soil with silt and sand while colluvial deposits are mainly consist of the same soil with big boulders with varying sizes. The thickness of the overburden varies from place to place as found by the results of the geotechnical investigation.

The landslide area is generally underlain by highly crystalline metamorphic rock of Highland complex (HC). The Highland complex rocks (ages 2.0-3.4 Ga) metamorphosed to granulite grade is the centrally located, NE-SW trending belt comprising mainly Charnockitic Gneisses and Granulites, Metasediments, Basic Granulites, Gneisses and Migmatites. The metasediments include Quartzites, Marbles, Pelitic Gneisses and Garnet-Sillimanite-Schist (Khondalites). The rock found at the adjacent hill can be identified as highly fractured Charnockitic gneiss with thick quartzite layers as shown in Figure 3.5.

Figure 3.5 : Exposed bedrock *(National Building Research Organisation, 2014)*

The area under investigation is a sloping land with undulating topography. Immediately above the activated landslide, there is a flat, water logged marshy area which can be identified as a major cause for the high ground water table prevailing in the area (Figure 3.6). Two major streams are flowing into this marshy area from the upper slope and finally come together to flow as one stream through the landslide area (Figure 3.2 & Figure 3.7). Seepages and high yielding springs are observed in the unstable mass, upper slope and even through the fractured bedrock along the road cuttings.

(a) Marshy area (b) Water logging

Figure 3.6 : Water Log at Marshy Area *(National Building Research Organisation, 2014)*

(a) Stream in marshy area

(b) Stream flowing down

Figure 3.7 : Streams in Marshy Land *(National Building Research Organisation, 2014)*

3.1.3 Mechanism of failure

As discussed in the previous section, the main affected area, just above the road level is a thick colluvial deposit, resulting from a previous landslide. This mass has been in a critically stable condition for a long time and movement has been initiated due to the removal of toe support by the excavation activities. By the field observation and available information, it is reasonable to assume that this is a type of rotational debris slump with a debris flow at the lower part. Sudden subsidence has been developed in the crown area due to the movement of the soil mass. Tension cracks with about 1 m width provide evidence for this subsidence. According to the morphology of the area, the landslide consists of different portions, which are moving with different rates. Lower part of the landslide is the most active area mainly because of loosely compacted, highly permeable colluvium overburden with high water content acting as an effluent area of the catchment. Several tension cracks observed at the surface of the unstable mass provide evidence for few slip surfaces above the main slip surface.

3.2 Conceptual design of rectification measures

After the background study of the landslide, conceptual design was done in order plan the investigation and obtain required information for the detail design.

Several key factors that need to be taken in to account in design of mitigation measures were identified as;

- Stabilization of the slope with a sufficient safety margin to prevent future failures

- Allocation of space necessary for proposed bridge and road widening
- Lowering the high ground water table prevailing in the area
- Minimize influence to the natural environment and hydrology of the area
- Minimize damage to the privately owned lands in the area
- Ensuring the stability of the slope during construction and operation even against an extreme rainfall event

The widening of the bridge and road requires an excavation at toe region of the slope. Removal of mass at toe will make the slope steeper at the toe region which will clearly reduce the safety margins. As such, it is necessary to reduce the potential sliding mass by excavation and removal of significant amount of soil from the top and the middle. Debris and very loose coluvial soil on the landslide body need to be removed. In order to enhance the slope stability and for better surface drainage management several berms to be made on the slope during the excavation.

Considering the effect on the natural environment and damage to private lands, cutting back of the slope and removal of mass has to be kept within a limited area. This makes it necessary to maintain steeper slopes and that will demand reinforcement. Soil nailing will be considered as the reinforcing technique. In addition, toe retaining structure might also be required.

Prevailing condition of water regime and necessary improvements may be listed as;

- High stagnant water table conditions were witnessed at the top and middle of the landslide and surface water flow from the upper slope was observed during the rain periods. Surface drainage improvement with cutoff drains, berm drains and cascade drains would be required to minimize the infiltration and facilitate surface runoff preventing stagnation of water.
- To economize the nailing designs, the ground water table should be maintained at a sufficiently low level. Subsurface drainage improvement with horizontal drains would be required for lowering the ground water table.
- Trench drains may be required to dry out the marshy land at the crest area.
- Surface cover with vegetation is required to minimize the infiltration, prevent erosion and to blend the rectification measures with the natural environment.

In order to ensure the stability of the slope during constructions, top down approach must be adopted during construction. Slope excavation needs to commence from top of the slope only after construction of drains above the crest area. Excavation downwards should be done only after the installation of soil nails and surface & subsurface drainage measures at the upper levels. The rectification of the impending landside should be designed with the carefully consideration of this construction sequence and the safety margins should be evaluated at each stage in the construction sequence.

In summary, the rectification process would include;

- Surface drainage measures such as cut off drains, trench drains
- Sub surface drainage through sub horizontal gravity drains
- Reinforcement of slopes through soil nailing
- Minimization of infiltration with berm drains cascade drains and surface covering vegetation
- Toe support with gravity type toe retaining structures

Detailed investigation of the landslide area is required to proceed with the detailed design of rectification measures.

3.3 Detail investigation for the design of rectification measures

Detailed investigation was carried out to obtain required information for the detailed design (National Building Research Organisation, 2014) and it consists of two parts as;

- Topographical survey
- Geotechnical investigation

3.3.1 Topographical survey

Topographical survey was carried out by a licensed surveyor in order to get the morphology of the area and it had two segments as contour survey and cross sectional survey. Contour survey of 1 m interval was done and several cross sections were taken covering the unstable area. Figure 3.8 shows the developed contour survey map of the area after topographical survey.

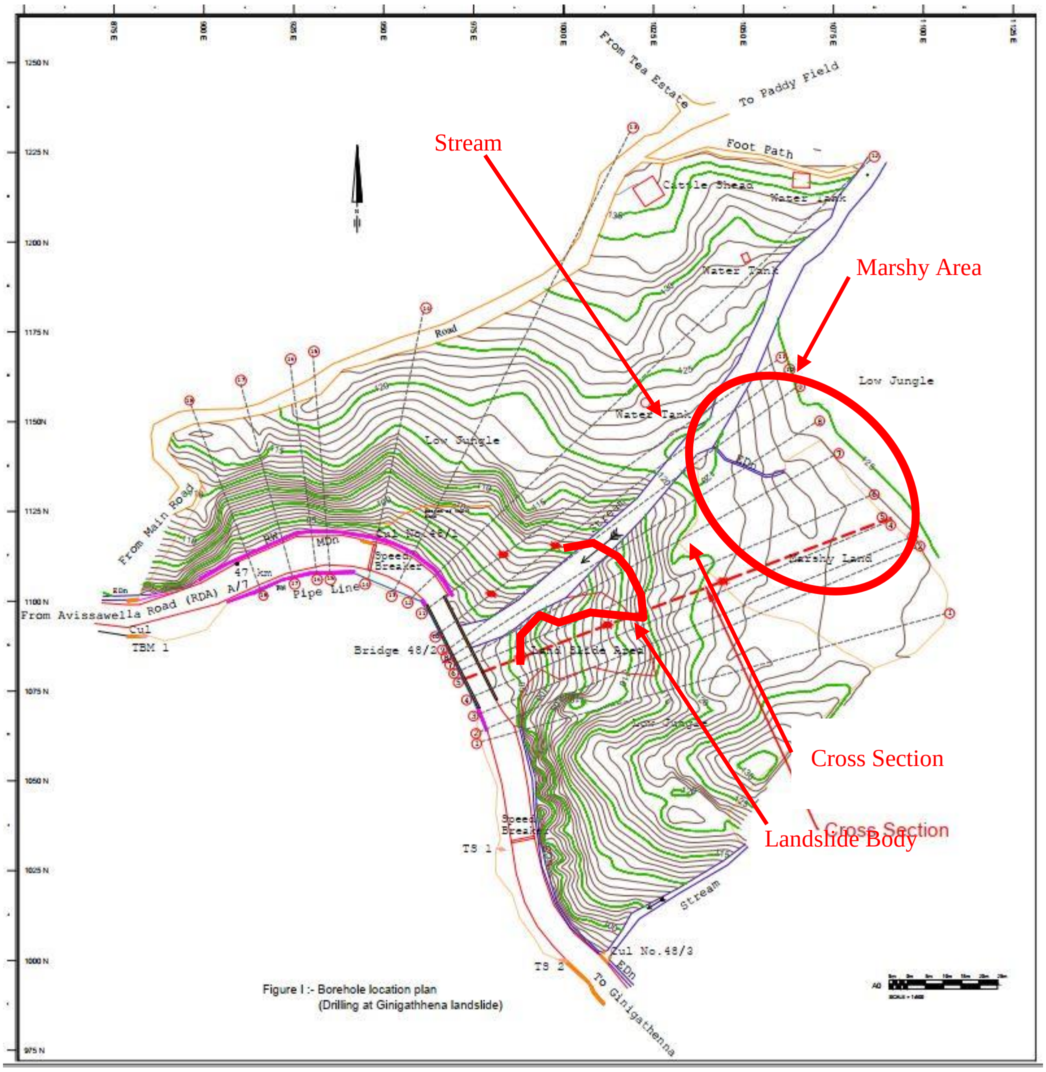


Figure 3.8 : Survey Map *(National Building Research Organisation, 2016)*

3.3.2 Field investigation

Geotechnical investigation in landslide mitigation projects are important to identify the subsoil profile, determination of soil parameter and identification of any features of slip surface development etc. Field investigation with boreholes was carried out as the first part of the geotechnical investigation to get the sub soil profile along with ground water condition, undisturbed soil samples were obtained for laboratory testing to get the parameters necessary for the design.

Total of six boreholes were advanced covering the two unstable slope areas which are located each side of the stream flowing in the middle. Three boreholes are located on the studied area under this Thesis, which is located at the left side (Ginigathhena side) of the stream. Boreholes are located along the landslide axis that was identified during the initial investigation. Location of the boreholes are tabulated in the Table 3.1 and depicted in the Figure 3.9.

Table 3.1 : Borehole Locations

Borehole Number	Location	
BH 1	Left side of the stream (studied area)	Toe of the slide
BH 2		Middle of the slide
BH 3		Top of the slide
BH 4	Right side of the stream	Toe of the slide
BH 5		Middle of the slide
BH 6		Top of the slide

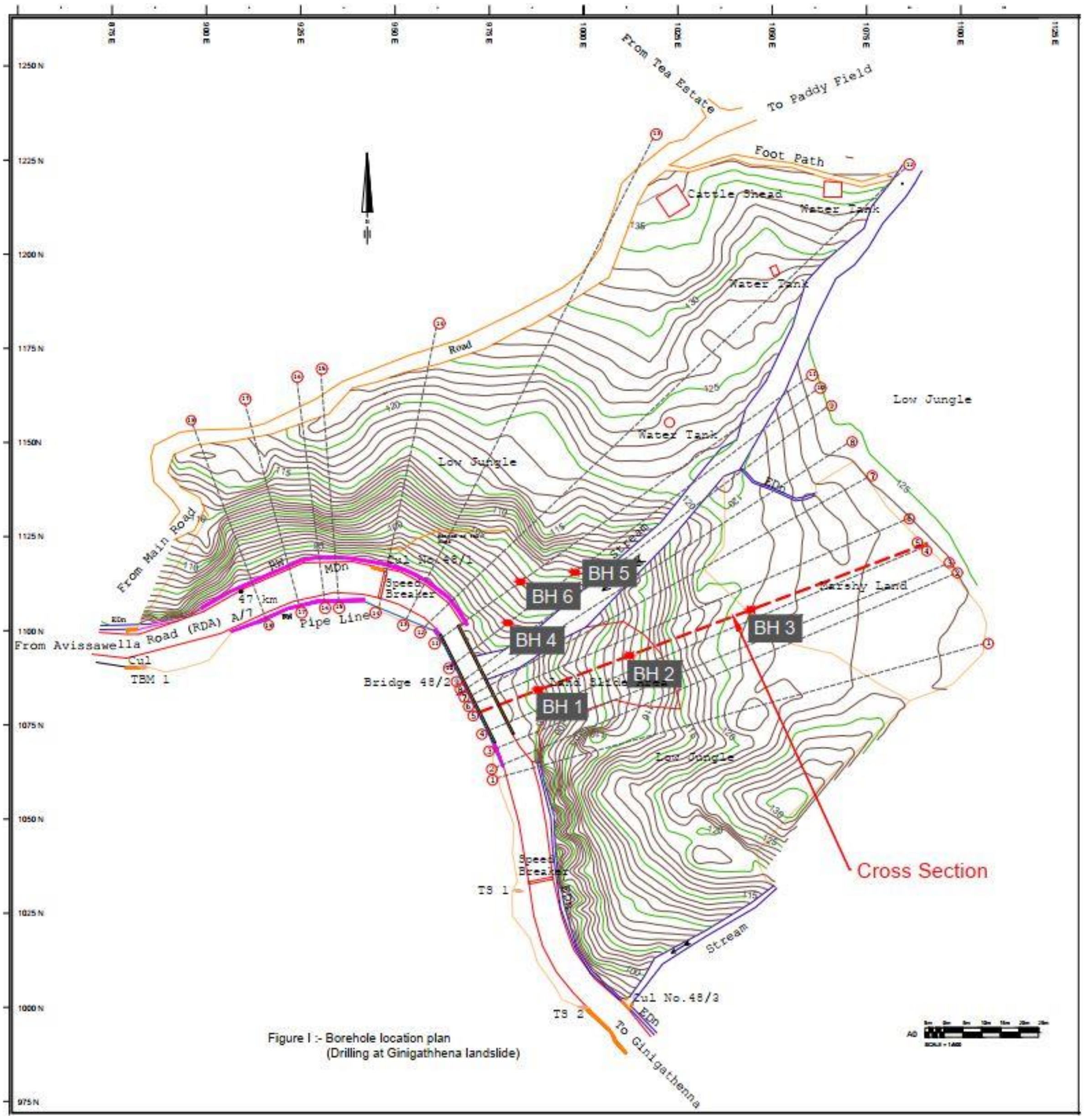


Figure 3.9 : Boreholes Locations Map and Cross Section

Boreholes are advanced with wash boring technique and Standard Penetration Tests (SPTs) were conducted 1 m intervals. Triple tube core barrel is used for drilling and taking samples of rock.

Since a borehole termination criterion is not defined in the scope of the investigation, boreholes were terminated after drilling a considerable depth through rock. However, BH 1 is terminated above the rock level after drilling through completely weathered rock (CWR) under condition of hammer re-bounced for more than 3 m. This depth is well below the area affected by failure. Termination depths and termination soil condition is summarized in Table 3.2 and detailed boreholes logs are presented in Annex A.

Table 3.2 : Boreholes Termination Details

Borehole Number	Termination Depth	Termination Soil Condition
BH 1	9.90 m	Hammer re-bounce > 3 m in CWR
BH 2	14.90 m	Quartzite Rock CR = 58% , RQD = nil
BH 3	18.85 m	Charnockitic Gneissic Rock CR = 100% , RQO = 72 %

3.3.3 Laboratory testing

Laboratory testing was done as the second part of the geotechnical investigation to get sub soil properties. Basic index tests were conducted to determine the particle size distribution and plasticity characteristics. The test results are summarized in Table 3.3.

Table 3.3 : Summary of Index Test Results

Borehole No.	Depth of Sample Collection m	Soil Classification (BS 5930:2015)	Natural Moisture Content %	Specific Gravity	Grain Size Distribution				Atterberg Limits		
					Gravel %	Sand %	Silt %	Clay %	LL %	PL %	PI %
BH 1	3.00-3.45	GM			70	17	13				
	4.00-4.45	SM			22	62	16				
	6.00-6.45	GM			57	32	11				
BH 2	7.00-7.45	SM			29	42	16	13	37	27	10
BH 3	1.00-1.45	MS			6	52	27	15	42	32	10
	3.00-3.45	SM			35	44	21				

3.3.4 Development of subsoil profile

Different subsoil layers were identified based on the data acquired through the borehole investigations. The summary is presented in Table 3.4. It is graphically presented in Figure 3.10 using the cross section through the boreholes as shown in Figure 3.9.

Table 3.4 : Summary of Ground Condition

Borehole	Depth (m)	Soil Type	Ground Water Level (m)	Field SPT Value
BH 1	0.0 – 3.0	Very loose Silty Sand	7.1	0 – 5
	3.0 – 6.6	Medium dense Silty Sand / Silty Gravel		12 – 23
	6.6 – 9.9	Very dense Silty Sand (CWR)		HB
	>9.9	Bedrock		
BH 2	0.0 – 2.7	Very loose Silty Sand	1.2	3 – 6
	2.7 – 8.3	Medium dense Silty Sand / Silty Gravel		13 – 31
	8.3 – 9.9	Very dense Silty Sand (CWR)		HB
	>9.9	Bedrock		
BH 3	0.0 – 3.0	Very loose Silty Sand	1.2	1 – 3
	3.0 – 7.9	Medium dense Silty Sand / Silty Gravel		13 – 44
	7.9 – 17.2	Very dense Silty Sand (CWR)		19 - HB
	>17.2	Bedrock		

..

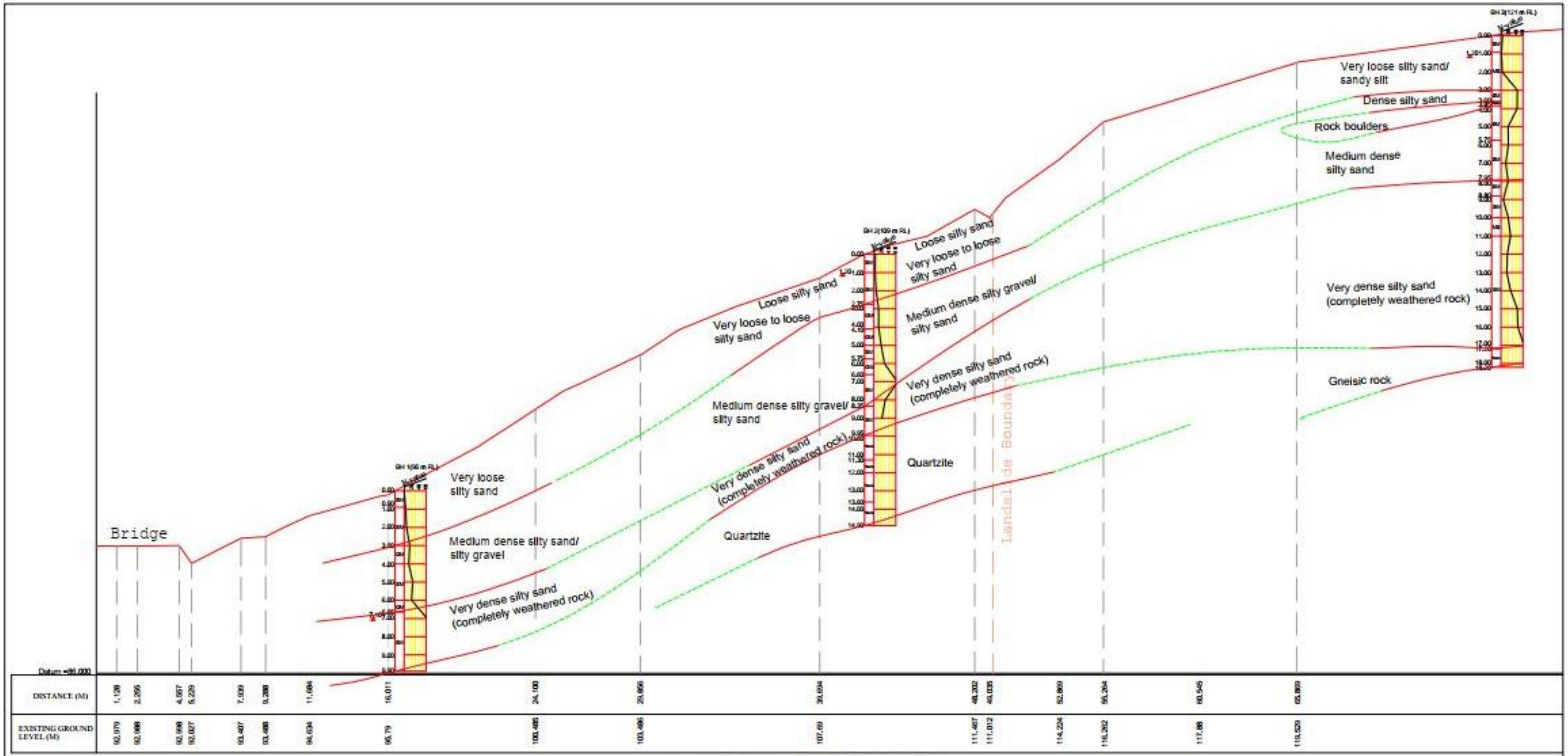


Figure II(c) :- Cross sections at BH1, BH2 and BH3
(Drilling at Ginigathhena landslide)

Figure 3.10: Subsoil Profile *(National Building Research Organisation, 2014)*

3.4 Design of rectification measures

Detailed design of the landslide mitigation measures was carried out using identified subsoil profile. Total mitigation area consists of three parts as landslide area and two adjacent areas at each side of the landslide body as illustrated in Figure 3.1. Only the mitigation measures of area marked as “Landslide area” are considered in this study. Mitigation for the landslide area is proposed on main four components as;

- Drainage improvement
- Geometry modification
- Reinforcing the slope
- Toe retaining structure

3.4.1 Computer model for stability analysis

Analyzes were to be done for the evaluation of effectiveness of proposed stabilization measures. An appropriate number of trial failure surfaces were to be analyzed under limit equilibrium formulation. Hence GeoStudio (2016) software SLOPE/W and SEEP/W were used in this study.

SLOPE/W has capacity to perform stability analysis based on method of slices using many different methods. There are several key input data for stability analysis with soil nailing such as,

- Slope geometry with subsoil profile
- Analysis type
- Slip surface options
- Soil model and parameters
- Reinforcement parameters

<u>Slope geometry with subsoil profile</u>

Geometry of the slope obtained from the topographical survey and subsoil profile developed by the detail geotechnical investigation as described in previous section was used for the analysis.

Analysis type

More precisely, method of analysis should have the capacity to analyze both circular and non-circular slip surfaces. To be accurate it has to satisfy both force and moment equilibrium. Consequently, Spencer's method (Spencer, 1967) was selected as for the stability analysis.

Slip surface

Grid and radius method available with the SLOPE/W software was used for selecting trial failure surfaces and finding the critical failure surface and corresponding minimum Factor of Safety value for circular failure surfaces.

Soil model and parameters

Mohr-Coulomb soil model is used for the analysis. Soil parameters are decided based on the geotechnical investigation, field observation and experienced gain through similar site condition. Table 3.5 summarizes the properties used for each type of material in the analysis.

Table 3.5 : Sol Parameters Used for the Analysis

Soil Type	**Notation**	**ϒ (kN/m^3)**	**C$^/$(kPa)**	**$\Phi^/$(degree)**
Top Soil Layer		17	7	26
Residual Soil Layer 3		19	12	31
Slightly Weathered Rock		22	25	50

Ground water table

There are several options to assign ground water table to the analysis. Most common options are Piezometric Line option and By Parent Analysis option. Water table is given manually in Piezometric Line option. In the analyzes where the infiltration is modelled, the output of the SEEP/W analyze was incorporated in to SLOPE/W software.

Reinforcement parameters

Tor steel bars of 32 mm diameter inserted and grouted into 125 mm boreholes, were used as soil nails. Angle of the soil nails are kept as 15^0 downwards from the horizontal. Pullout resistance of the nails at different levels of embedment were evaluated by the following formula.

$$\tau_f = C_w + \sigma_{av} \tan \delta \tag{3.1}$$

where τ_f is the pullout resistance in kPa, C_w is the adhesion between grout-soil interface of the resistance zone, δ is the angle of interface friction between grouted body & soil and σ_{av} is the average of horizontal and vertical soil stress measured at the mid length of the resistant part of the soil nail given by;

$$\sigma_{av} = (\sigma'_v + \sigma'_h)/2 \tag{3.2}$$

Where;

$$\sigma'_h = K_a \sigma'_v \tag{3.3}$$

Where;

K_a is the coefficient of lateral earth pressure

Pullout resistance of nails are varied nail to nail with the overburden height and soil type of the embedment. Safety factors of 1.5 and 2.0 was assigned against pullout resistant and tensile strength of the reinforcement bar. Reinforcement spacing was kept as 2.0 m in horizontal direction and 2.5 m in vertical direction.

<u>Developed computer model</u>

The SLOPE/W model developed from investigation data is presented in Figure 3.11. Ground water table is given as the water level from borehole investigation.

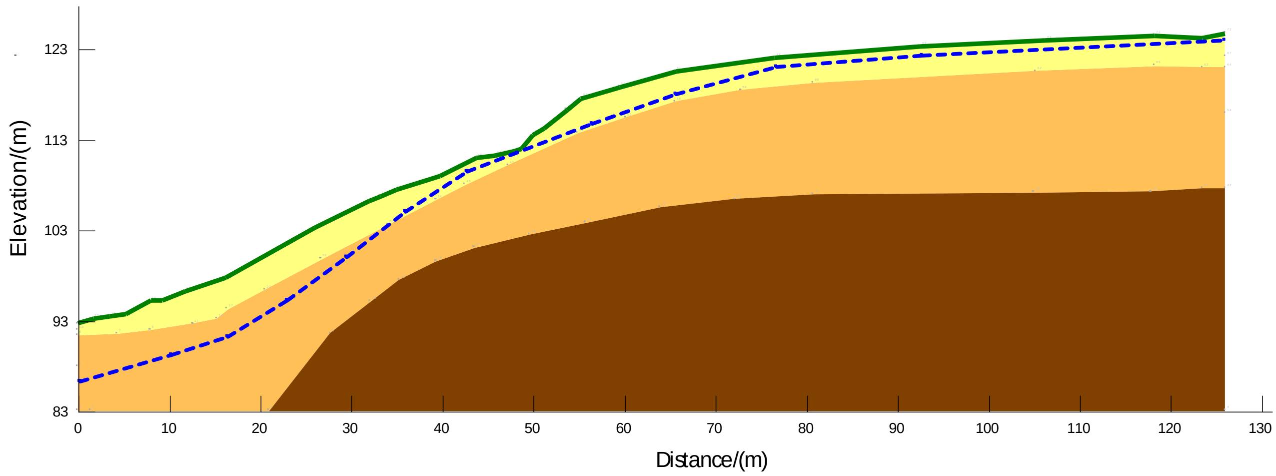


Figure 3.11 : SLOPE/W Model Developed

Design Factor of Safety

Considering the critical condition of the Ginigathhena site and devastating impact if the failures occurs following safety margins were adopted in the design,

- Short term stability during construction – 1.1
- Long term stability in service condition – 1.3

3.4.2 Back analysis of failure

After developing the SLOPE/W model, back analysis was done to model the failure occurred with the toe excavation and thereby verification of the model. Subsoil profile obtained by geotechnical investigation and soil parameters as described above was used. Since there was no rain at the time of failure (National Building Research Organisation, 2014), infiltration analysis was not incorporated. Therefore, the ground water level was taken as the water level observed during the borehole investigation.

First, stability condition of the natural slope before excavation was analyzed and the resultant critical slip surface is shown in Figure 3.12. FoS of the original condition is 1.126, which is a quite low value and slope can be considered as at critical stability state. Excavation at the toe of this critically stable slope has initiated a slope failure. Results of the stability analysis of the excavated slope presented in Figure 3.13 reveals that the critical slip surface is a shallow. FoS of the critical slip surface is 0.896 and therefore, failure would have occurred along that slip surface. Stability of the slope after the occurrence of the first failure was analyzed by removing the region of the failed mass. The results of the analysis showed the critical shallow failure surface with a FoS of 0.963 as depicted in Figure 3.14. Next analysis was done removing the region of second failure and it results a shallow and lengthy transitional type critical slip surface as shown in Figure 3.15. The FoS of this slip surface is 1.062, which is at the critical stable state and small change in ground water level or slope geometry could triggers the next failure. The summary of back analysis is presented in Table 3.6.

This series of analysis has revealed the progression of a slope failure, towards the upper slope. The succession of these mechanisms may not have been visible since it has occurred very quickly one after another. However, according the eye witnesses, a small failure has occurred initially and it has progressed further up the slope within a short period of time. The failure had been propagated further in to the slope during heavy rainy periods. SLOPE/W model has predicted the failure to a reasonable level.

Since the failed soil mass had not been removed from the toe but had accumulated there with the support from the bridge piers, it has given an additional toe resistance to the upper slope and prevented further propagation of slope failure. The tension cracks appeared on the upper slope may have caused due to small movement of the slope with lower safety margins.

Table 3.6 : FoS of back analysis

Condition	FoS
Natural Slope	1.126
After Excavation - Step 1	0.896
After Excavation - Step 2	0.963
After Excavation - Step 3	1.062

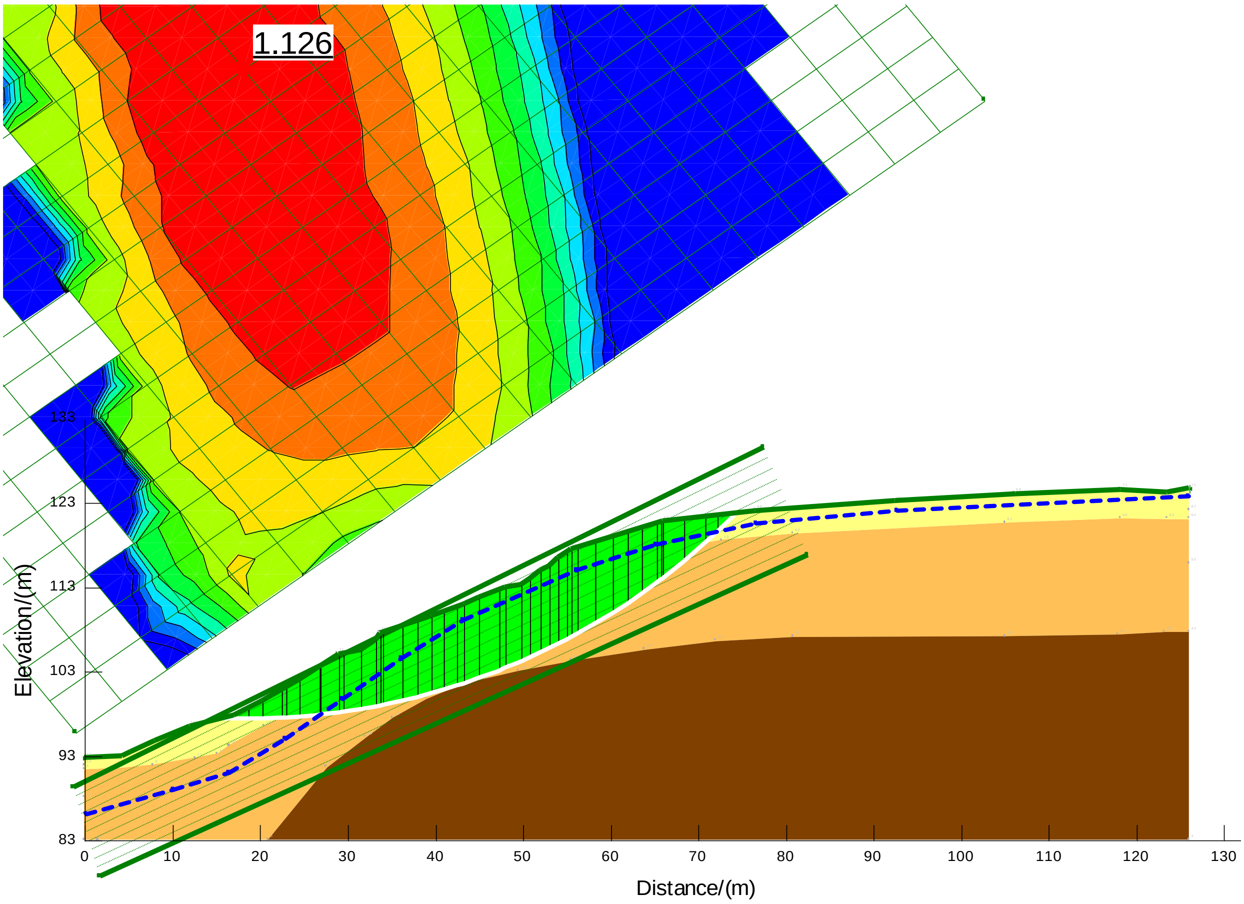


Figure 3.12 : Stability of the Natural Slope Prior to and Excavation

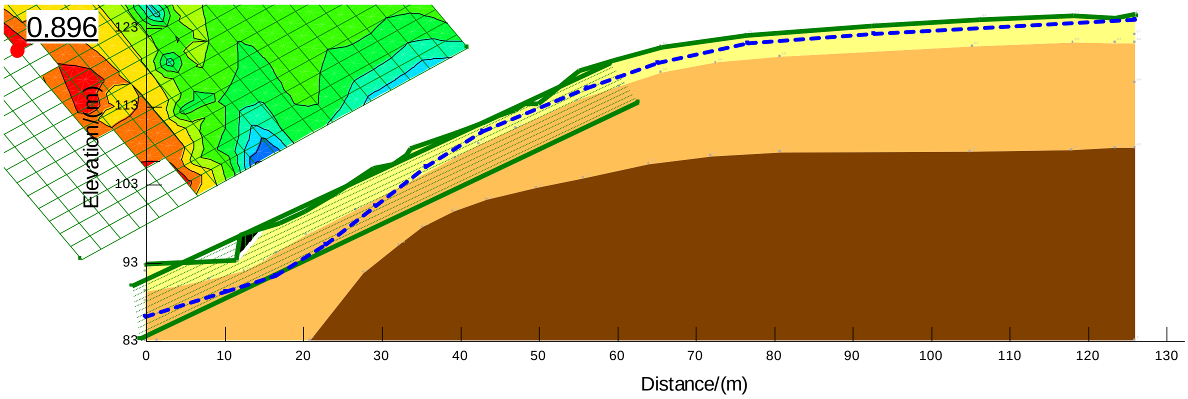


Figure 3.13 : Stability of the Slope after Excavation

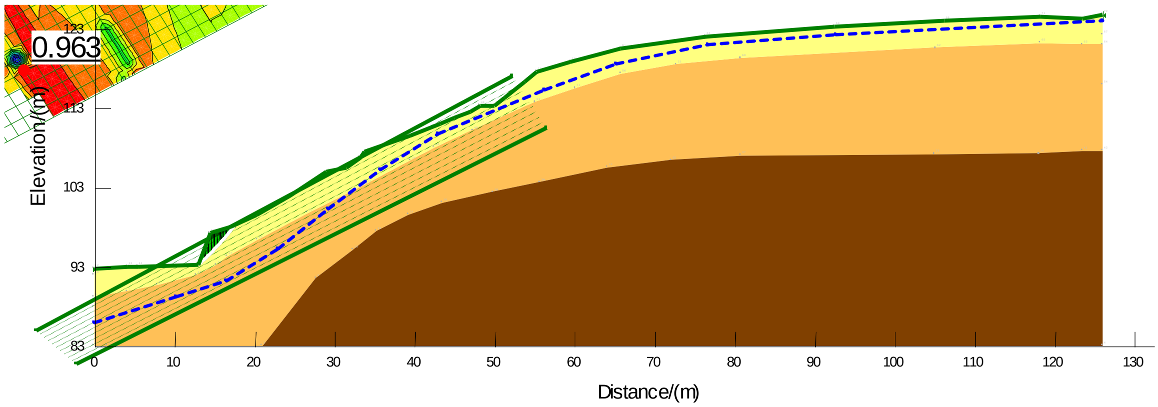


Figure 3.14 : Stability of the Slope after First Failure

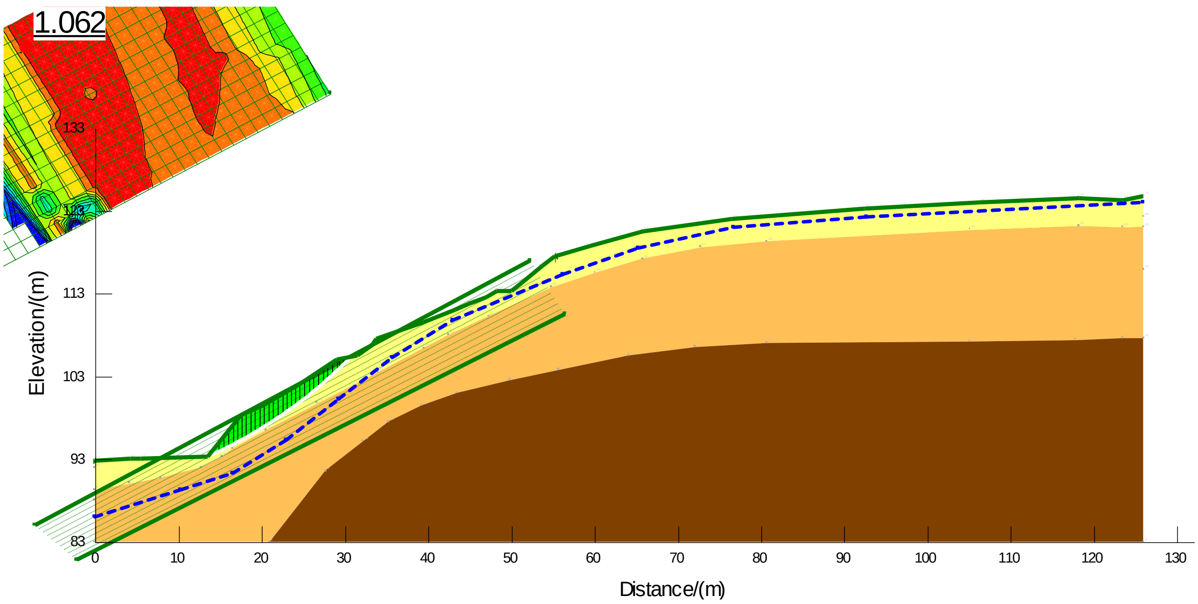


Figure 3.15 : Stability of the Slope after Second Failure

3.4.3 Drainage Improvement

After development of SLOPE/W model, detailed design of mitigation measures was carried out. Since the high ground water table was identified by field observations and borehole investigations, surface and subsurface drainage improvement is considered to be essential.

First consideration was to drain out the water stagnation in the marshy land at the upper slope, which is a major reason for high ground water table prevailing in the area. It is also necessary to prevent the surface water flow from marshy area in upper slope. This flow is occurring through two small streams.

Two surface drains (Figure 3.16 - DS (M) A) were constructed at the marshy area for this purpose and water from those drains were diverted to the main stream.

Since the ground water level of the marshy land at top of the slope was almost at the surface level and water logging is occurring during rainy periods, trench drains were constructed (Figure 3.16 – DS (S) A) below the DS (M) A drains to lower the ground water level.

Cut off drain was constructed just above the soil nailing area (Figure 3.16– DS (C) A) to divert the ground water flow away from the soil nailing area.

Slope was excavated with two berms and berms were sealed with berm drains (Figure 3.16 – DS (B) A & Figure 3.22 – Berm Drains). The excavated slope was covered with vegetation to prevent soil erosion and minimize the rainwater infiltration.

Typical arrangement of trench drain with surface drain is shown in Figure 3.17. All surface drains were constructed as cast in-situ reinforced concrete drains.

In addition to the trench drains, sub horizontal gravity drains of 20 m length were installed into the slope as subsurface drainage improvement (Figure 3.22). Those sub horizontal gravity drains were installed at every berm level in 5 m spacing along the slope. Additional sub horizontal gravity drains were installed where ever necessary during construction specially in areas of seepages and high ground water level.

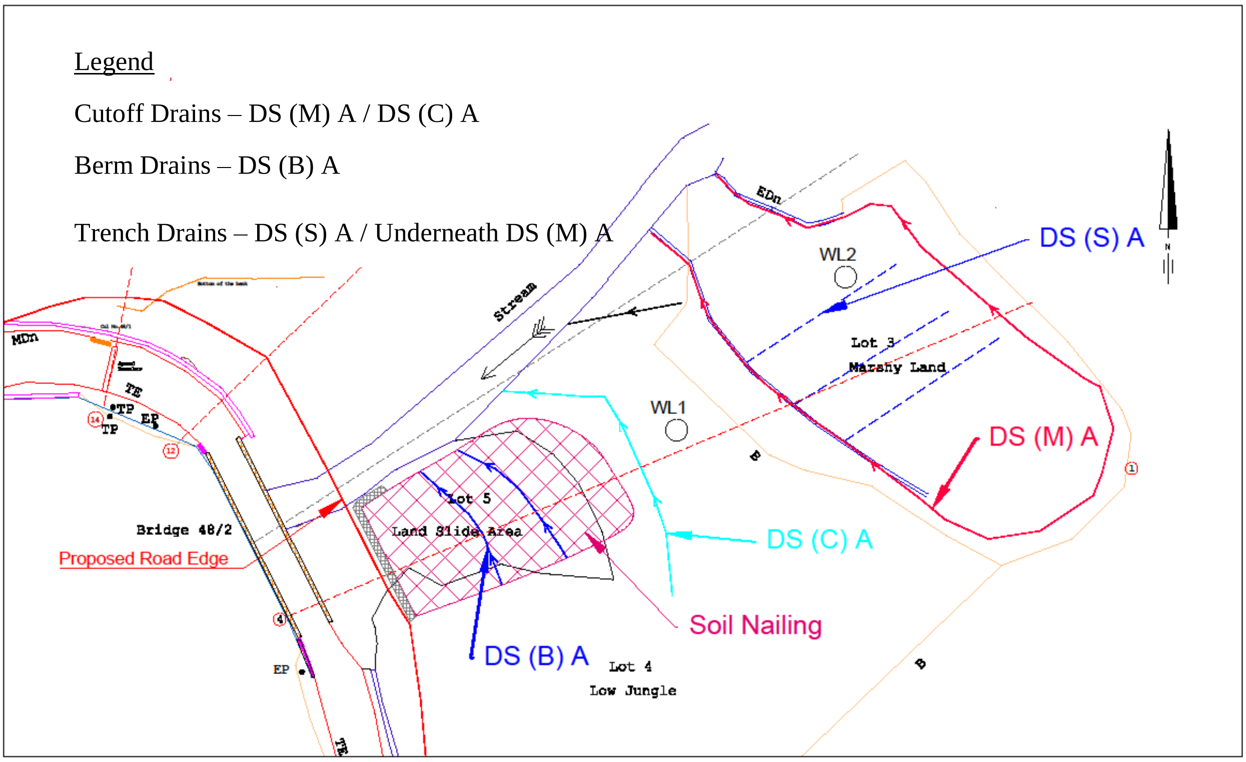


Figure 3.16 : Plan view of proposed mitigation measures

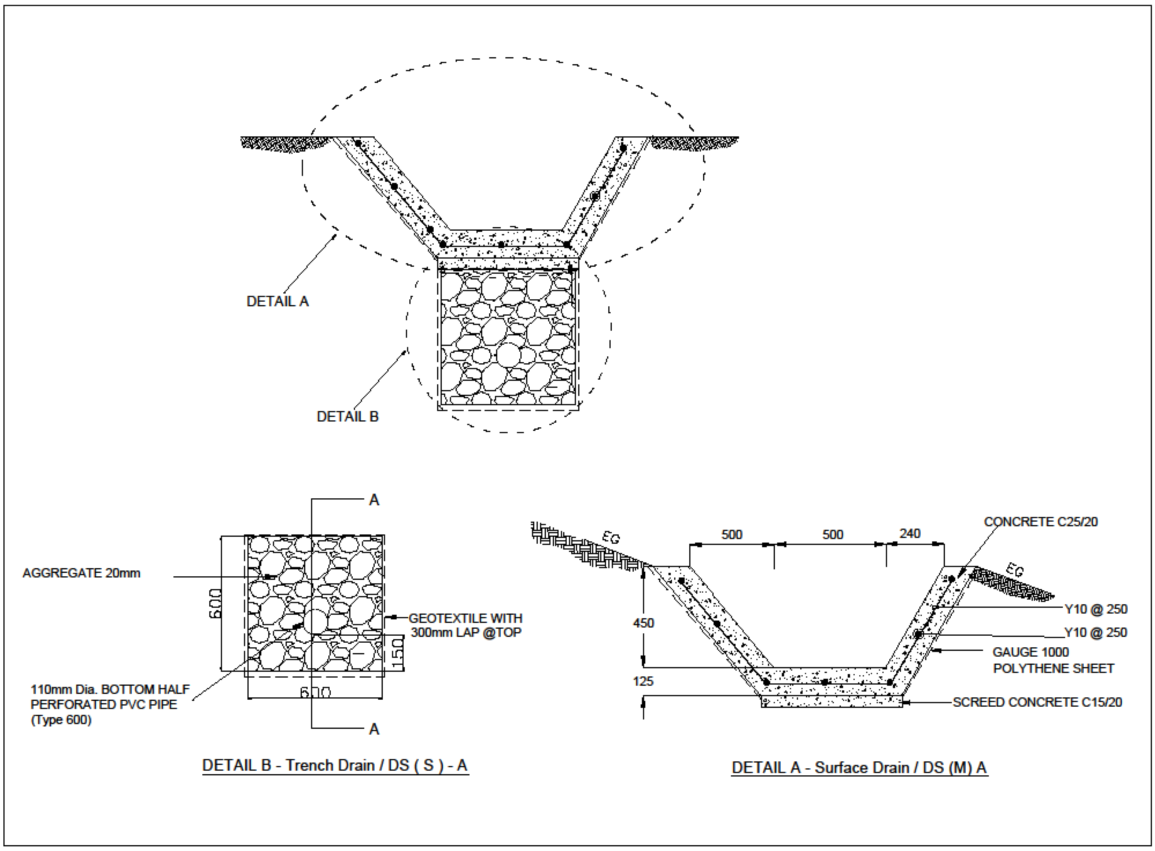


Figure 3.17 : Detail of surface and trench drain

3.4.4 Modification of slope geometry

With the proposed widening of the road and the bridge, the new road edge is located about 10 m away from the existing edge. Therefore, 10 m wide excavation in to the slope at the toe was required. Removal of material at toe will lead to the instability of the upper area. Soil mass at the top and middle area need to be reduced in order to maintain the slope stability.

In order to accommodate the aforementioned requirement and in the purpose of removing the loose debris of the landslide mass, reshaping of existing slope was taken place. Two berms with three slopes were created in the 26 m high slope. Angle of the top slope was 50^0 and the angle of the middle and bottom slopes was 30^0. In order to maintain the stability of the slope and for the convenience of the construction, top down approach should be adopted in the excavation of the slope.

However, the modified slope geometry was not in a stable condition on its own. Stability analysis results shows that the FoS is only 0.928 after geometry modification with high ground water table (Figure 3.18), and slope was not stable under this condition. Application of drainage improvement as mentioned above would lower ground water table and slope stability analysis with low ground water table shows that FoS had improved to 1.103 (Figure 3.19), but it was not acceptable in the long run.

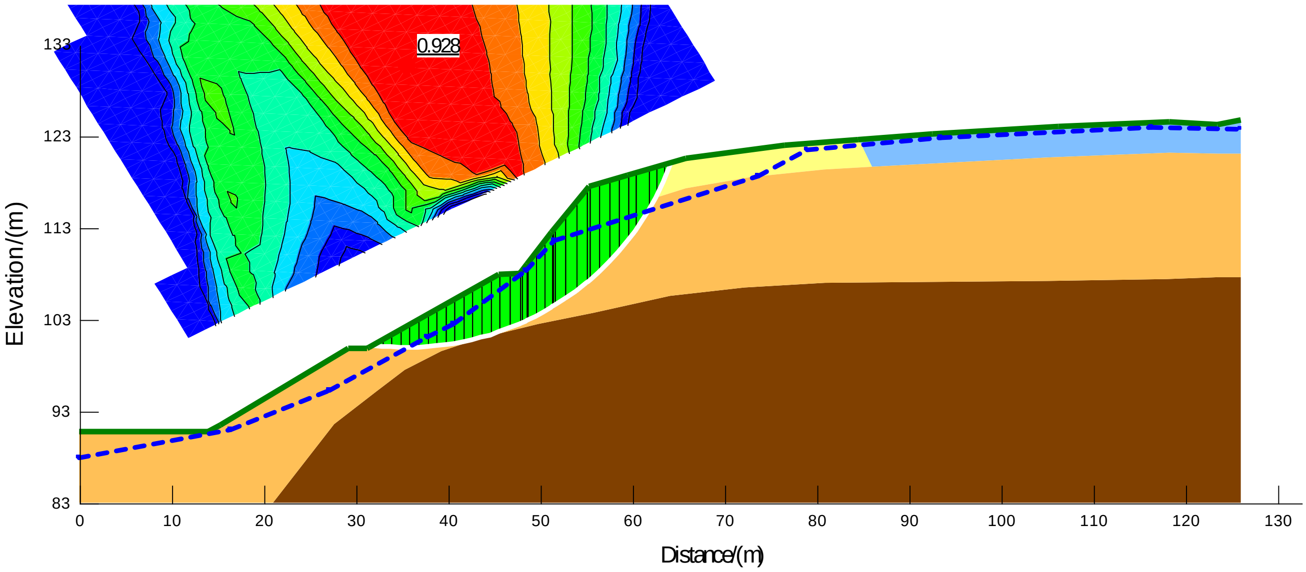


Figure 3.18 : Slope Stability Analysis – Geometry Modification with High GWT

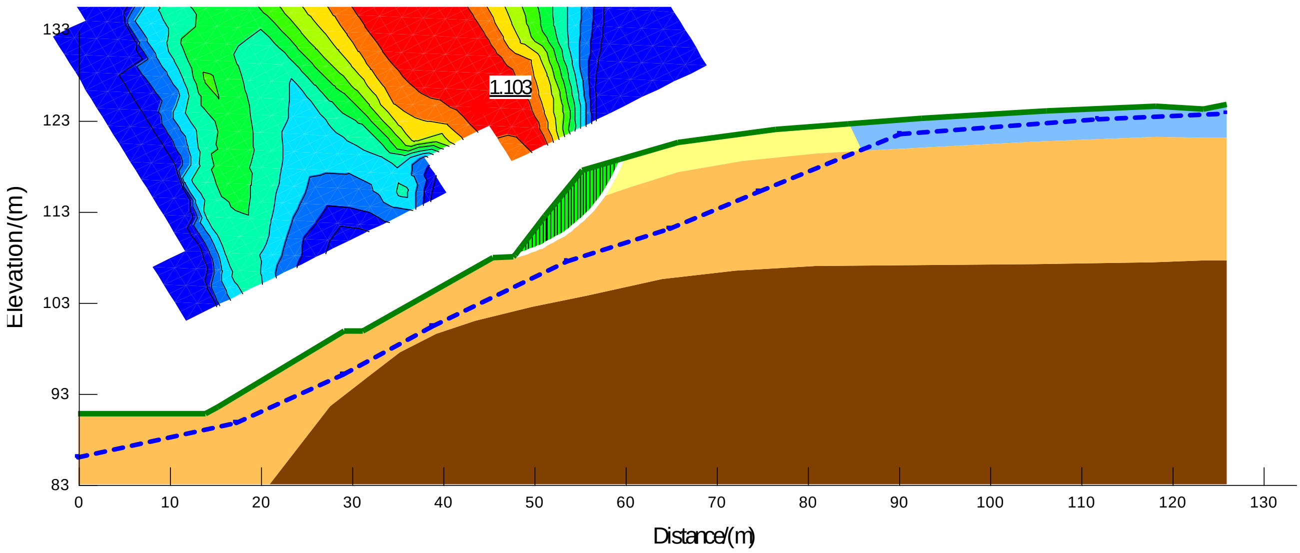


Figure 3.19 : Slope Stability Analysis – Geometry Modification with Low GWT

Stability of the modified slope could be enhanced by further removal of soil mass at top and middle by reducing the slope angle. Cutting back in to the slope was required for reducing the slope angle and it was not feasible considering the limitation of land acquisitions and environmental effects. Also, the stream which flows at a side of the landslide would be exposed due to further cutting back of the slope. Therefore, the modified geometry of the slope is required to be maintained and slope strengthening measures are required.

3.4.5 Reinforcing the slope

Considering the slope geometry and subsoil condition, soil nailing is proposed as the best means of strengthening the slope. Four numbers of 16 m long soil nails are installed on the top slope while one number of 16 m soil nail & two numbers of 12 m soil nails, and four numbers of 8 m soil nails are installed on the middle and bottom slope respectively. All the reinforcement bars are 32 mm in diameter and installed into the 125 mm diameter borehole and grouted. Nail spacing was kept as 2.0 m in horizontal direction and 2.5 m in vertical direction.

Pullout resistances were calculated for each nail level considering the soil overburden on the resistance zone of the soil nails. Applied pullout resistances for soil nails in the analysis are summarized in Table 3.7.

Table 3.7 : Nail Pullout Resistance

Stage	Nail No. from Top	Nail Length (m)	Embedment (m)	σ_v (kPa)	σ_h (kPa)	Pullout Resistance (kPa)
Stage 1	1	16	6	111.0	55.5	50
	2	16	8	148.0	74.0	70
	3	16	10	185.0	92.5	90
	4	16	12	222.0	111.0	100
Stage 2	5	16	11	203.5	101.75	100
	6	12	8	148.0	74.0	70
	7	12	8	148.0	74.0	70
Stage 3	8	8	4	74.0	37.0	40
	9	8	4	74.0	37.0	40
	10	8	4	74.0	37.0	40
	11	8	4	74.0	37.0	40

Stability analysis results shows that the FoS of reinforced slope with high ground water level is 1.132 and it is not sufficient for long term condition (Figure 3.20). When the ground water table was further lowered, FoS of reinforced slope had increased to 1.470 satisfying the long term stability condition (Figure 3.21). Further lowering the ground water table was proposed to be achieved by surface and subsurface drainage improvement and such drainage improvement measures are essential to maintain the stability of the slope. A slope cross section with all the proposed mitigation measures are presented in Figure 3.22.

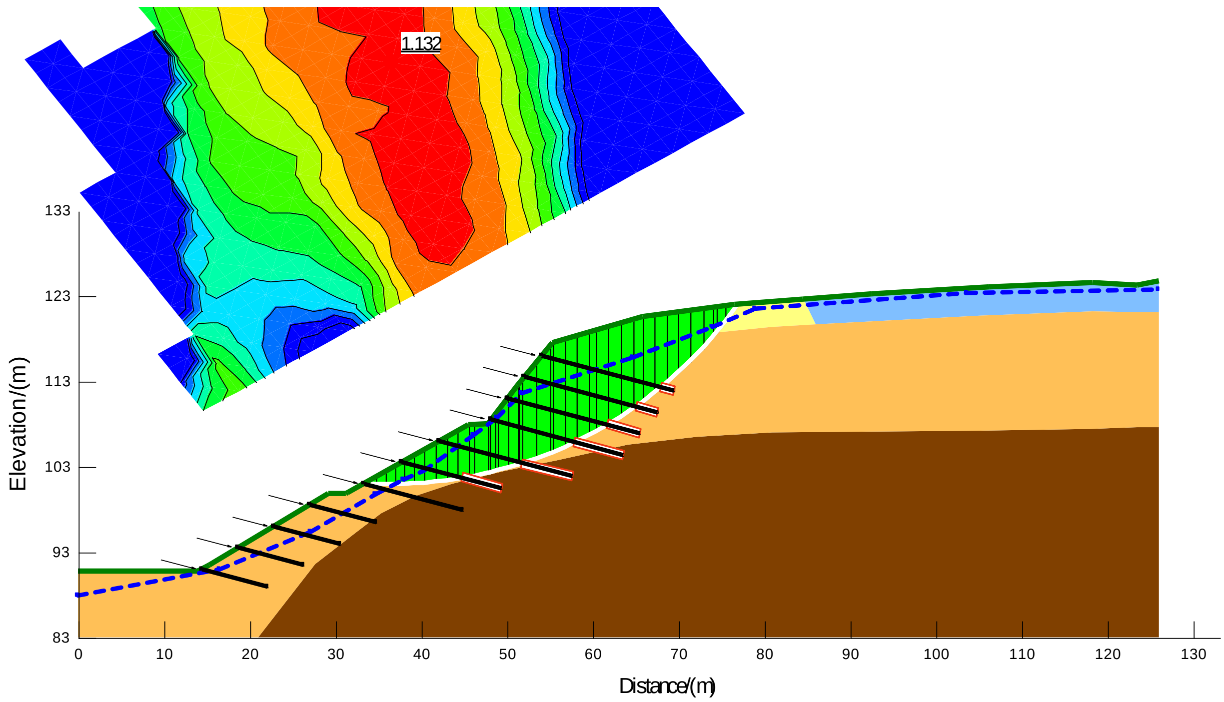


Figure 3.20 : Slope Stability Analysis of Nailing System with High GWT

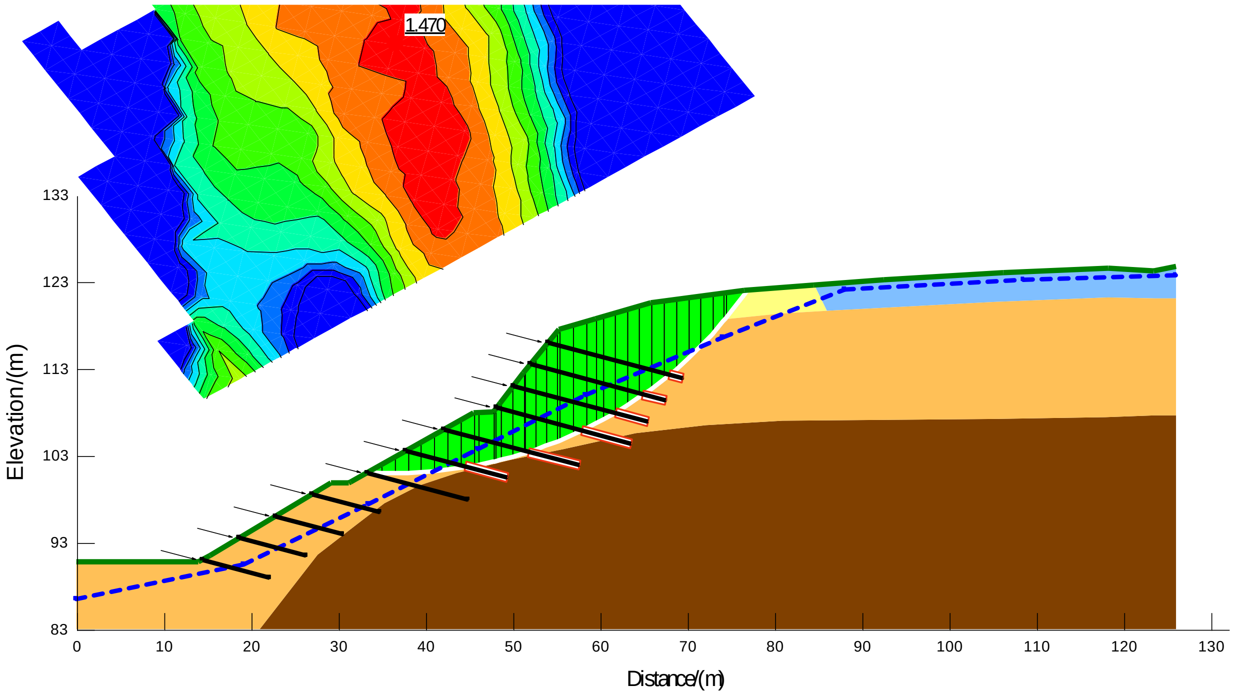


Figure 3.21 : Slope Stability Analysis of Nailing System with Low GWT

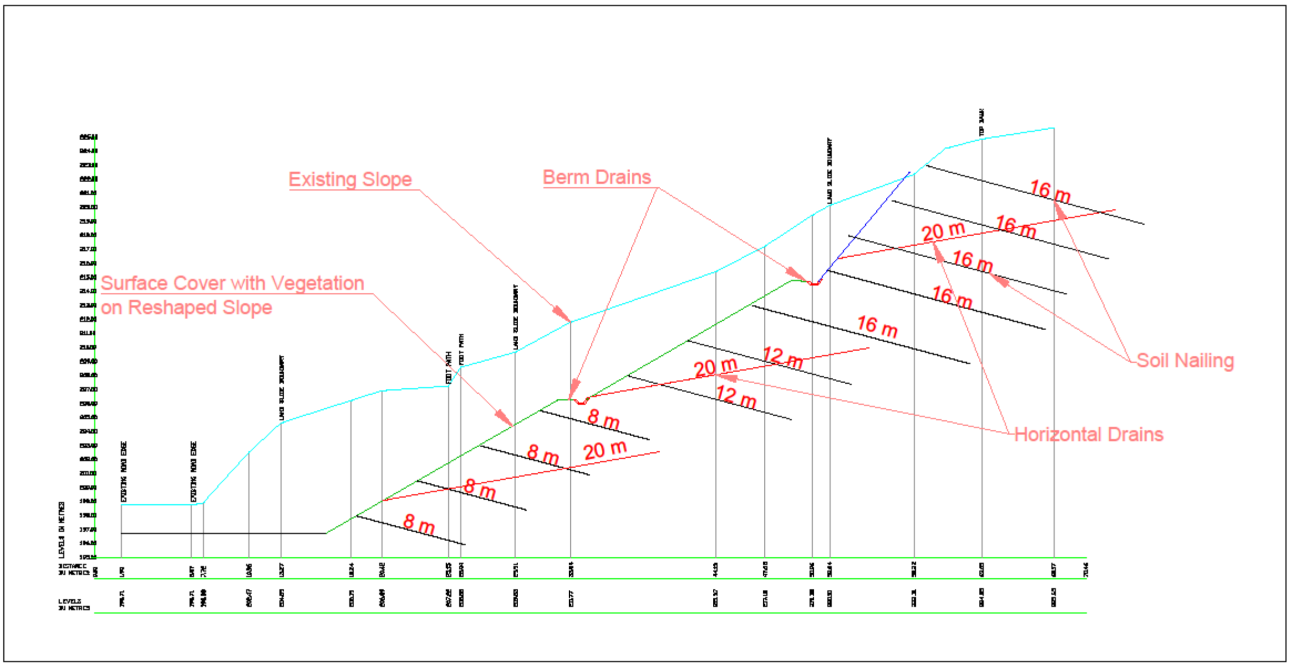


Figure 3.22 : Cross section of proposed mitigation measures

3.5 Field monitoring

With the proposed rectification measures a field monitoring scheme is proposed. It was done with the following objectives.

- Evaluation of the effectiveness of the applied mitigation measures
- Design and applying of further mitigation measures required if slope strengthening was not achieved by the already applied mitigation measures
- Observations from field monitoring to be used in future designs of mitigation measures

With those objectives, a monitoring programme was applied at site for the monitoring of,

- Rainfall
- Ground water level and,
- Slope movement

Rainfall monitoring is done by a manual rain gauge installed at the site premises and rainfall in recorded two times a day as at 8.00 am and 4.00 pm. Accuracy of the measurement is 5 mm. Rainfall measurement has been done throughout the construction period and is continuing.

Two water level monitoring wells were installed at the upper slope area. Locations of monitoring wells and boreholes done for the geotechnical investigations are shown in Figure 3.16 (WL 1 & WL 2). Installation of monitoring wells done after excavation of the slope up to top berm level and installation of sub horizontal gravity drains at top berm level. Water level of the monitoring wells are measures daily basis using a Dip Meter.

Two fixed points were established on the slope to measure the movement of the slope with respect to given bench marks.

CHAPTER 4 - STUDY OF THE BEHAVIOR OF RECTIFIED SLOPE UNDER CRITICAL RAINFALL

This chapter presents the results of the study carried out on the behavior of the rectified slope under critical rainfall.

4.1 Computer software for seepage analysis

In order to analyze the effect of the infiltration of rainfall and the drainage improvement on the ground water table, a transient seepage analysis by finite element method is required. The effect of the rainfall infiltration and drainage improvement should be analyzed with respect to the slope stability. In order to achieve this objective, the results of the seepage analysis and slope stability analysis are required to be coupled together. Selection of computer software to perform the analyses was done considering above requirements.

With availability of the licensed version, GeoStudio (2016) software suite was selected as the best software to fulfill all the above requirements. SEEP/W & SLOPE/W software in GeoStudio (2016) were selected for seepage analysis and slope stability analysis respectively. The results of the seepage analysis could be transferred to the stability analysis software.

4.1.1 Developing the SEEP/W model

SEEP/W software is based on a finite element formulation and has a capacity to simulate the movement of liquid water or water vapor through saturated and unsaturated porous media. There are several key input data for the analysis such as;

- Slope geometry and subsoil profile
- Analysis type
- Material model and parameters
- Boundary condition

After developing the SEEP/W model, it has to be verified by simulating an actual seepage event.

<u>Slope geometry and subsoil profile</u>

Geometry of the slope found by the topographical survey and subsoil profile developed by the detail geotechnical investigation as described in Chapter 3 was used for the analysis. Slope profile developed is presented in Figure 4.3.

Analysis type

Two fundamental types of finite element seepage analysis are transient & steady-state and those are available in SEEP/W software. The definition of steady-state means that the ground water regime is not changing with time and transient means it is changing always. In a steady-state analysis all the boundary conditions are fixed. But in a transient analysis the boundary conditions can also be a function of time or response to flow amounts. Transient analysis was selected for the study on the purpose of simulating the variation of ground water regime with time and, the results of each time step is used in the stability analysis to study the variation of safety margins of the slope with time. Total duration and time steps of the analysis were selected based on the requirement.

Material model and parameters

There are three types of material models available with the SEEP/W software, namely; saturated / unsaturated, saturated only and interface. Since the ground water table is below the surface level, both saturated & unsaturated soil conditions are present in the ground. Also, fluctuation of ground water table & rainwater infiltration is expected during the construction and service. Therefore, saturated / unsaturated soil model was selected for the analysis with an exception for modeling of berm drains.

The key input parameters of saturated / unsaturated soils are;

- Volumetric Water Content function (SWCC)
- Hydraulic Conductivity function

Volumetric water content function (SWCC)

Since the research done for developing Soil Water Characteristic Curve in Sri Lankan context are limited, results of the study done by Vasanthan (2016) for residual soil was used as the Soil Water Characteristic Curve, although the soil condition of the two cases are somewhat different. Comparison of the soil index properties, used by Vasanthan (2016) for the study and Ginigathhena site are done in Table 4.1. The comparison indicated that the use of results obtained by Vasanthan (2016) is reasonably satisfactory for the Ginigahthena site.

Saturated volumetric water content was considered as 0.518. The Soil Water Characteristic Curve used for the residual and colluvium soils in the analysis is illustrated in Figure 4.1.

Table 4.1 : Comparison of Ginigathhena Soil Index Properties with Vasanthan (2016)

Source / Location	Classification	Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
Vasanthan (2016)	MS	54	43	11	2	43	35	20
Ginigathhena (BH2)	SM	37	27	10	29	42	16	13
Ginigathhena (BH3)	MS	42	32	10	6	52	27	15

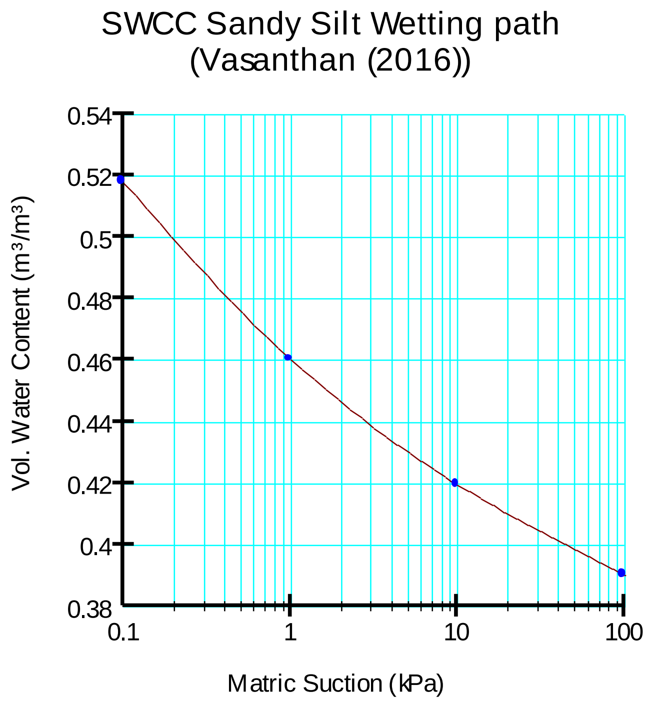


Figure 4.1 : SWCC used for the Analysis *(Vasanthan, 2016)*

Hydraulic conductivity function

Hydraulic conductivity function of unsaturated soils can be obtained through Soil Water Characteristic Curve and saturated permeability. Soil Water Characteristic Curve is already defined and saturated permeability of residual soils is determined by trial and error process, which will be discussed in next section. Hydraulic conductivity function used for the analysis is presented in Figure 4.2.

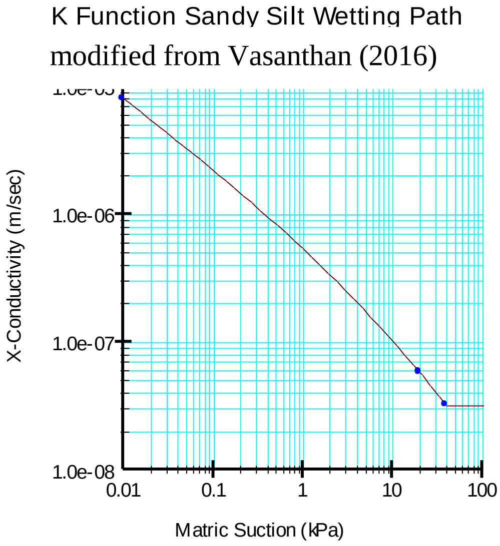


Figure 4.2 : Hydraulic Conductivity Function used for the Analysis *(Vasanthan, 2016)*

Modeling of vegetation cover and berm drains

The combine effect of surface drainage improvement, which includes cutoff drains and slope covering with turfing, is modeled by a thin layer of low saturated permeability in the order of 10^{-7} m/s as proposed by Kulathilaka & Kumara (2013). Later this range is used by other researchers; Dharmasena & Kulathilaka (2015), Idirimanna (2016), Jayakody, et al. (2018) and confirm the suitability for the analysis. Therefore, the effect of surface drainage improvement is modeled by a 100 mm thick layer of saturated permeability of 5×10^{-7} m/s.

Separate layer is used to model the berm drains at berm levels with a saturated permeability of 1×10^{-20} m/s as suggested by Dharmasena & Kulathilaka (2015). Thickness of this layer is kept as 100 mm.

<u>Boundary condition</u>

Suitable boundary conditions were applied for the model boundaries to represent the actual seepage condition of the slope as illustrated in Figure 4.3.

- Boundary A-B-C-D-E ➔ Rainfall intensities were applied on the ground surface as Unit Flux boundary by a step data point function according to the relevant rainfall intensities. Potential Seepage Face Review function was enabled in the ground surface since it is a free surface and rainfall flux may be greater than the infiltration capacity of the soil. Water ponding and developing pore pressures on the ground surface is prevented and pressure head at ground surface is always made equal to zero.

- Boundary E-F, A-I ➔ No flow boundary condition was applied on the two sides of the slope model above the ground water level, as a zero Total Flux boundary, to restrict the lateral flow above the ground water table.
- Boundary F-G, I-H ➔ Initial total heads were applied at the sides of the slope model below the ground water level, as Heads boundary with respective values, to maintain the minimum depth to ground water table.
- Boundary G-H ➔ No flow boundary condition was applied on the bottom of the slope model, as a zero Total Flux boundary, to restrict the flow of ground water to further down.
- Boundary J-K ➔ Zero pressure boundary condition was applied to lines with no thickness as plain strain idealization to simulate perforated pipes of sub horizontal gravity drains which were installed at some horizontal spacing. No dimensions were given for simulating the horizontal drains and only length was indicated.

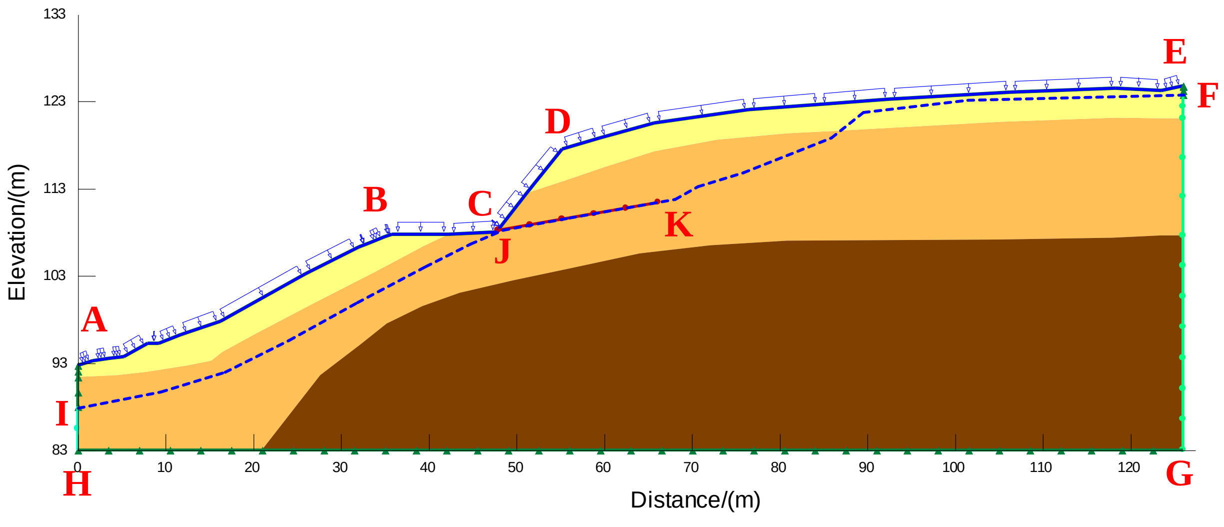


Figure 4.3 : Slope Geometry with Sub Soil Profile and Boundary Conditions

4.1.2 Model verification

Verification of the developed SEEP/W model was done using rainfall and ground water level monitoring data. The peak daily rainfall during the of rainfall monitoring period is recorded as 300 mm/day on 20th May 2018 and significant rise of ground water table was observed with that high rainfall. Therefore, rainfall intensities of 20 days, starting from 18th May 2018 was modeled as shown in Figure 4.4 and variation of ground water level was compared with the actual water level measurement. Although the rainfall is measured twice a day, daily rainfall is modeled as a step function of Unit Flux boundary condition as illustrated in Figure 4.5.

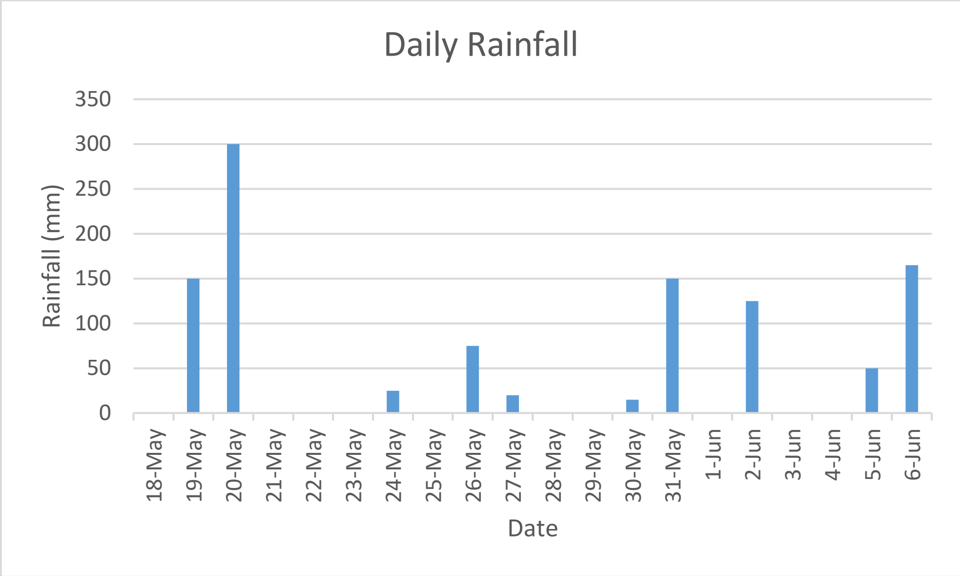


Figure 4.4 : Daily Rainfall Record form 18th May 2018 to 06th June 2018

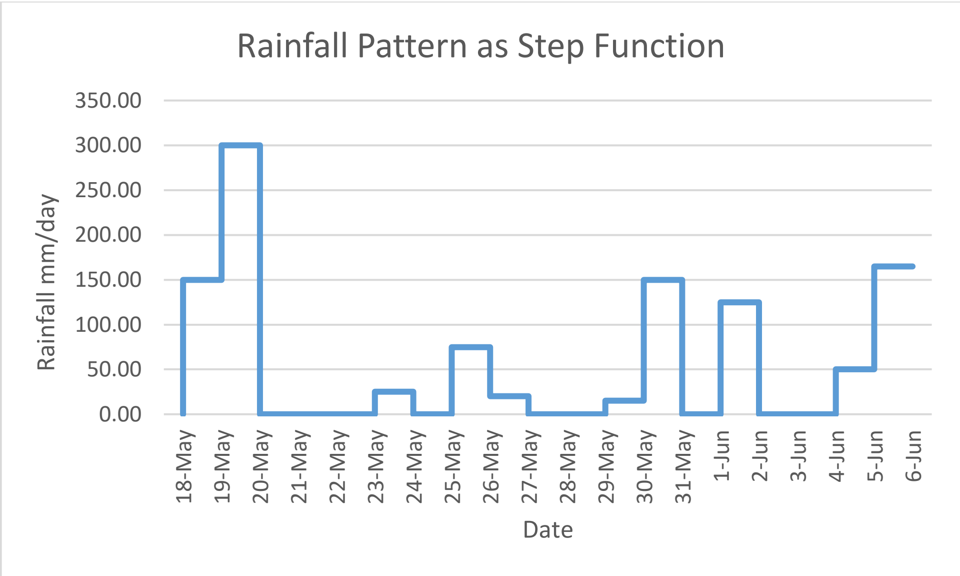


Figure 4.5 : Rainfall Pattern used for Model Verification

Surface drainage improvement of upper slope area had been already completed and sub horizontal gravity drains at the top berm level were already installed by 18th May 2018. The ground water levels observed during the geotechnical investigation phase was modified based on the data of monitoring wells to be used as initial water level for the seepage analysis.

Water level meter, WL2 is installed in the marshy land and water level is almost at the ground level. Variation of rainfall intensities has minimum effect on the ground water level of the marshy area due to low permeability and high water content. Therefore, only the water level of WL1 was used for the comparison of actual water level with results of the analysis.

Figure 4.6 presents the daily rainfall intensities and compares the observed water level in WL1 and the water level estimated from the analysis at the location WL1 for different permeability values, from 18th May 2018 to 6th June 2018.

Analysis was conducted for different saturated permeability values of residual soil, in the range of 8 x 10^{-5} m/s to 1 x 10^{-6} m/s and the saturated permeability value, which gives the most comparatively accurate results that represent the actual ground water condition, is taken as 8 x 10^{-6} m/s. Therefore, saturated permeability of residual soil is taken as 8 x 10^{-6} m/s for further analyses.

Figure 4.6 reveals the rise of ground water table with high intensity rainfall and lowering of water level in dry days. Also, development of perched water table and functionality of sub horizontal gravity drains are highlighted.

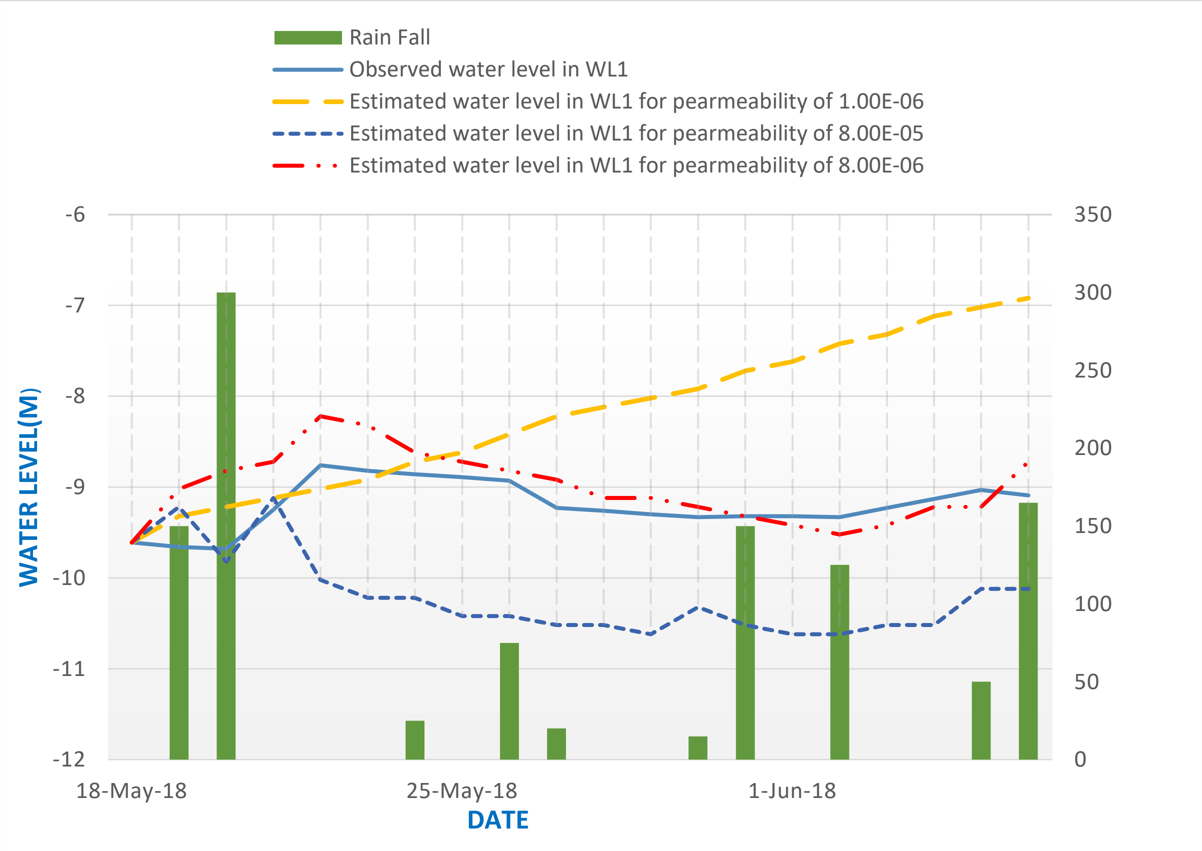


Figure 4.6 : SEEP/W Model Verification

4.2 Analysis the response of the slope to critical rainfall

Response of the slope to a critical hypothetical rainfall event after implementation of the proposed slope stabilization measures was analyzed using the geotechnical models developed by SEEP/W and SLOPE/W software. Analyses was carried out for the critical rainfall pattern given here under different conditions of drainage improvement and reinforcement

4.2.1 Rainfall

Effect of rainfall on the ground water regime was analyzed by SEEP/W software. Among infinite possibilities of rainfall patterns, a hypothetical critical rainfall pattern was considered in the analysis.

The peak rainfall of 300 mm/day observed during the monitoring period was considered as the critical rainfall for the analysis. However, 300 mm/day rainfall was applied for two days as the critical rainfall and rainfall of 50 mm/day was applied as the residual rainfall for seven days afterwards. Further, the ground water table at monitoring stage was considerably lower than at the investigation stage. This aspect will be discussed in detail in Section 4.2.2. In order to account for this aspect, five days of dry weather period was considered initially. Total of fourteen days had considered for the analysis.

Step function as shown in Figure 4.7 was applied as the unit flux boundary condition to simulate the critical rainfall pattern.

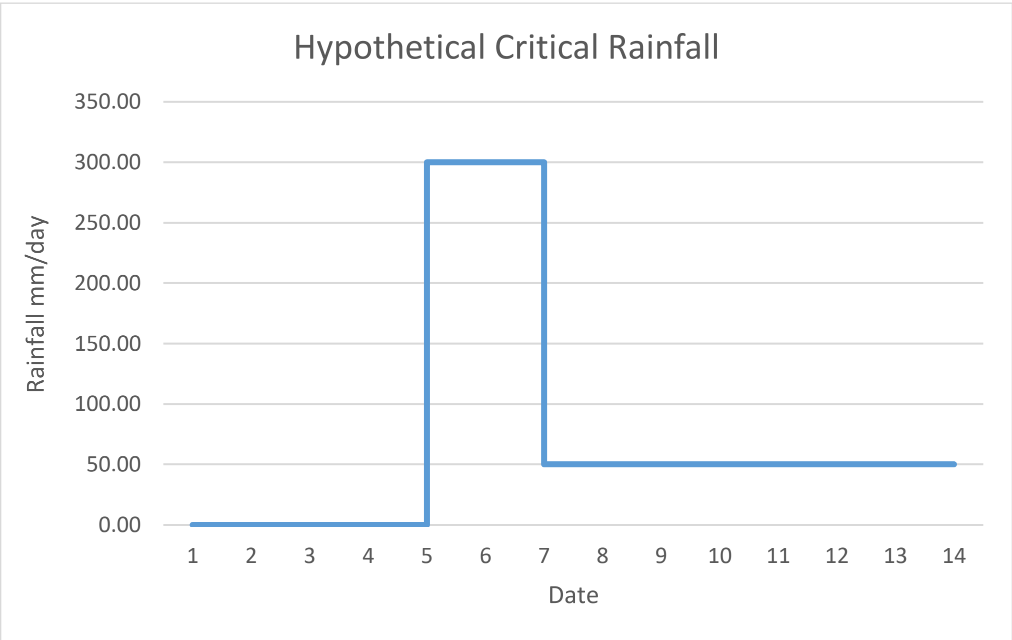


Figure 4.7 : Critical Rainfall Pattern for the Analyses

4.2.2 Drainage improvement and initial ground water level

Effect of drainage improvement on the ground water regime was modeled by SEEP/W software. Drainage improvement consist with main two categories namely; surface drainage improvement and subsurface drainage improvement. Following alternative conditions were considered in the analyses in order to get the effectiveness of each and their combination.

- No drainage improvement
- Surface drainage improvement
- Both surface and subsurface drainage improvement

Water levels measured by the monitoring wells were lower than the water table observed during the geotechnical investigation stage. It is reasonable to assume that the ground water table was lowered due to installation of sub horizontal gravity drains at the top berm level considering the location of the drains and monitoring was done after installation of those drains. Comparison of ground water level from the geotechnical investigation and monitoring is presented in Figure 4.8.

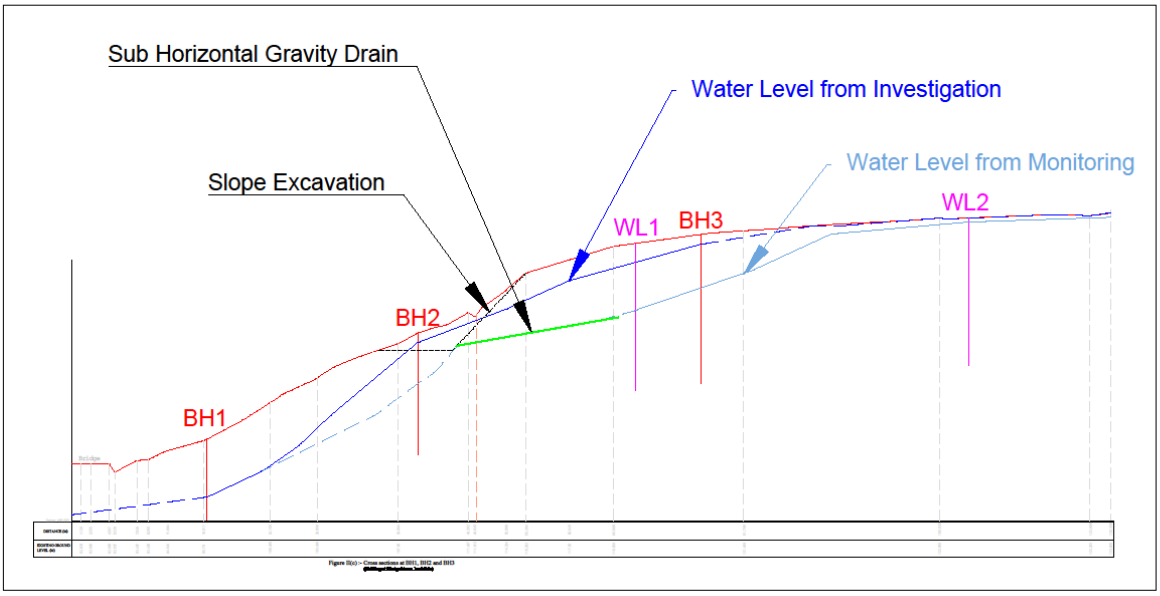


Figure 4.8 : Water Levels Comparison between Investigation and Field Monitoring

The seepage analysis started with the water level which was observed at the investigation stage. A five day long period of dry weather was considered prior to the rainfall even to bring it down to what was observed in the monitoring wells after installation of drainage measures, (Figure 4.7). This simulation lowered the ground water level to that observed by monitoring wells.

As described in Section 3.4.3 trench drains were constructed to lower the ground and surface water in the marshy land. The ground water level in the marshy area was almost at ground level prior to the construction of trench drains. The ground water level measurements done after the construction of the trench drains, in WL2 was varying from 0.5 m – 0.9 m below surface level. Therefore, the effect of trench drain was modelled under two different conditions. When the presence of trench drains were accounted the ground water table was taken to be 0.5 m below the surface (Figure 4.9). The condition without trench drains was accounted by keeping the initial ground water table at the surface (Figure 4.10). Effect of trench drains were incorporated to the analyses with surface drainage condition.

The pore water pressure distribution with depth prior to the rainfall was assumed to be hydrostatic below the ground water table and negative above it in the unsaturated soil mass.

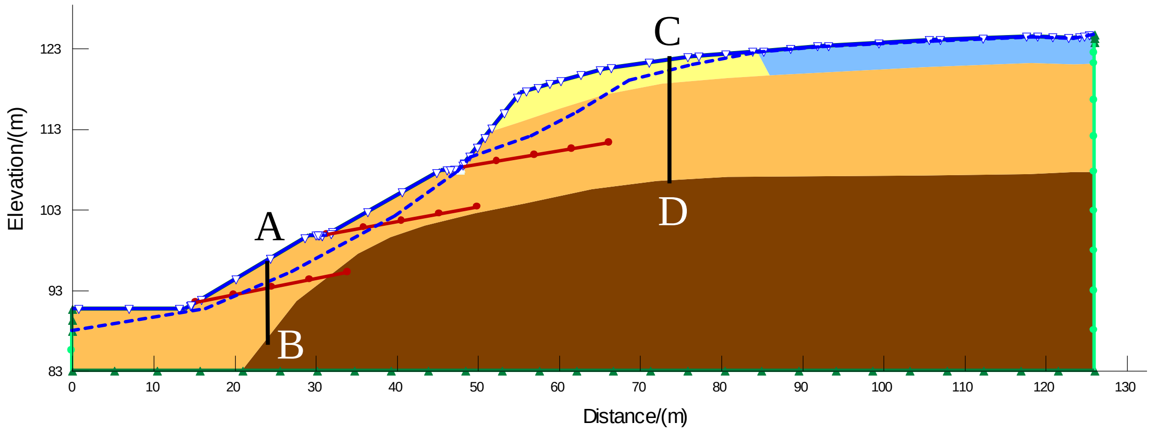


Figure 4.9 : Initial Ground Water Level for the Analysis without Trench Drains

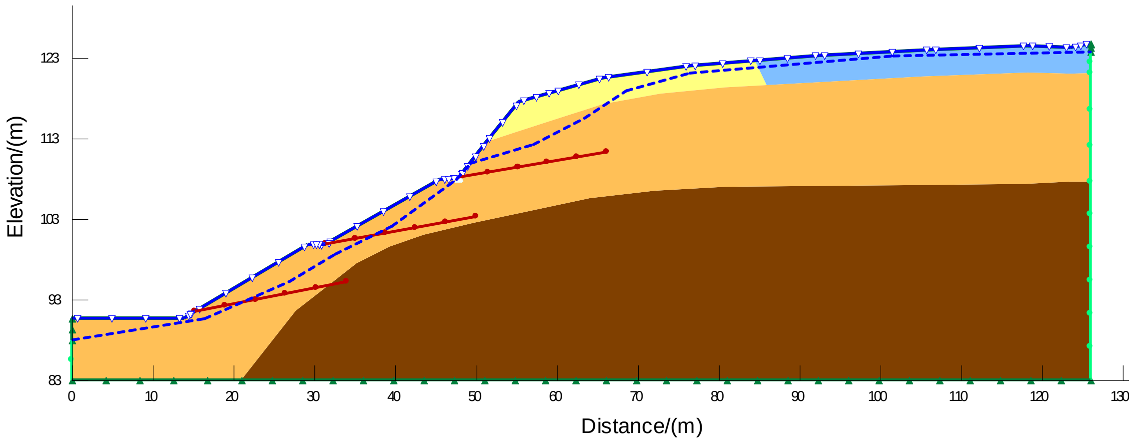


Figure 4.10 : Initial Ground Water Level for the Analysis with Trench Drains

4.2.3 Reshaping and reinforcement of the slope

As discussed in the Section 4.4, slope was excavated in to three segments with two berms and soil nails were installed to improve the stability of the slope up to required safety margins. Importance of the soil nailing in slope stability and significance of adopting top down approach in soil nailing construction was studied by staged construction of reshaping and installing reinforcement. Three stages of construction with excavation of each segment was simulated in the analyses.

<u>Construction Stage 1</u>

Excavation was done up to top berm level and analyses were done for with & without soil nails conditions. Four numbers of 16 m long soil nails were installed on the upper segment as shown in Figure 4.11.

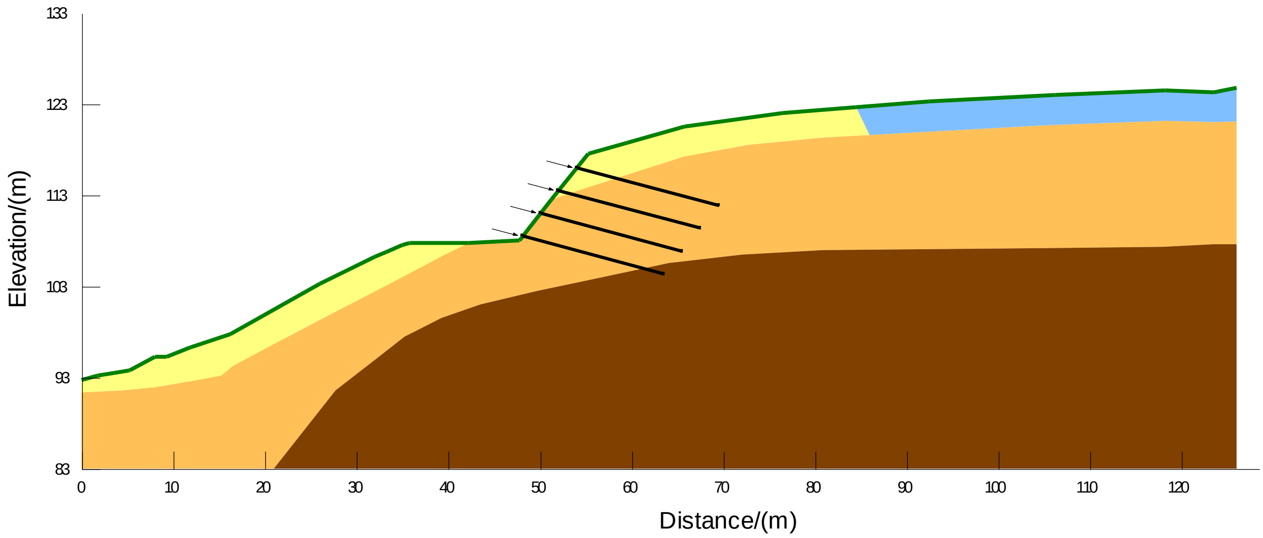


Figure 4.11 : Construction Stage 1

<u>Construction Stage 2</u>

Excavation was done up to bottom berm level and analyses were done for with & without soil nails conditions. One number of 16 m long soil nail and two numbers of 12 m long soil nails were installed as shown in Figure 4.12.

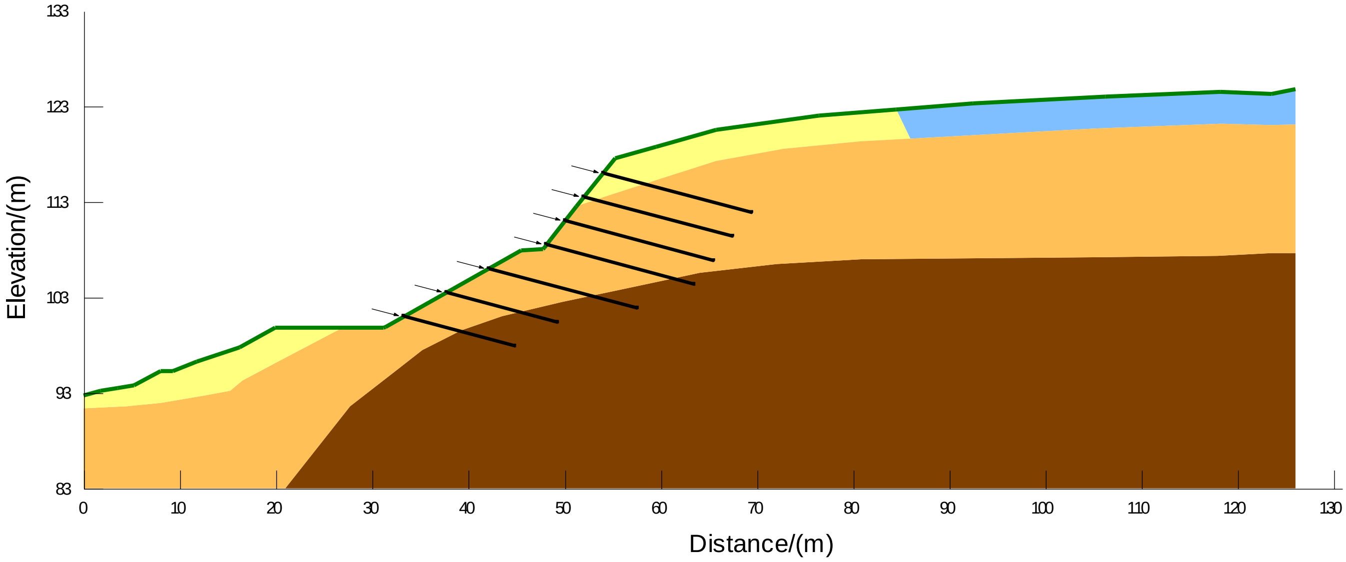


Figure 4.12 : Construction Stage 2

Construction Stage 3

Excavation was done up to top berm level and analyses were done for with & without soil nails conditions. Four numbers of 8 m long soil nails were installed as shown in Figure 4.13.

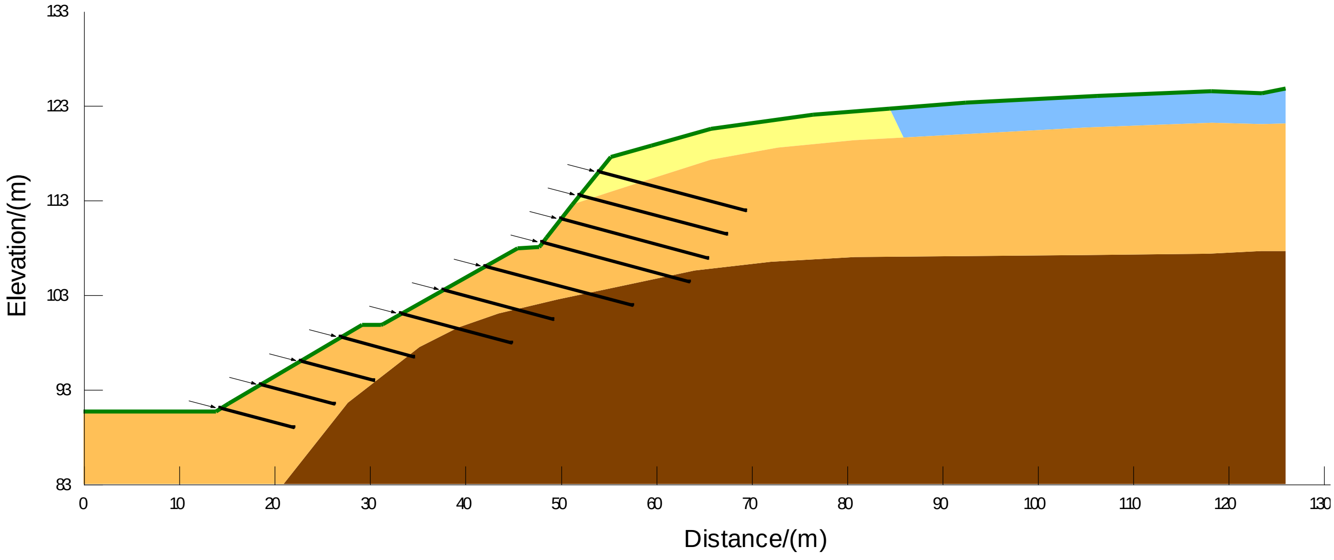


Figure 4.13 : Construction Stage 3

4.3 Summary of types of analyses done

Different types of analyses done is summarized in Table 4.2

Table 4.2 : Summary of the Types of Analysis Carried Out

Construction Stage	Without Drainage Improvement		With Surface Drainage Improvement		With Surface and Subsurface Drainage Improvement	
	Without Nailing	With Nailing	Without Nailing	With Nailing	Without Nailing	With Nailing
Construction Stage 1	✓	✓	✓	✓	✓	✓
Construction Stage 2	✓	✓	✓	✓	✓	✓
Construction Stage 3	✓	✓	✓	✓	✓	✓

4.3.1 Effect of drainage improvement on ground water regime

Effect of the drainage improvement on the ground water regime due to the presented critical rainfall pattern was analyzed by GeoStudio (2016) SEEP/W software using transient type analysis, applying different drainage measures as indicated. The analysis was done at the completion of excavation up to Construction Stage 3. The pore water pressure contours and ground water level corresponding to different analysis after peak rainfall are presented in Figure 4.14 to Figure 4.16.

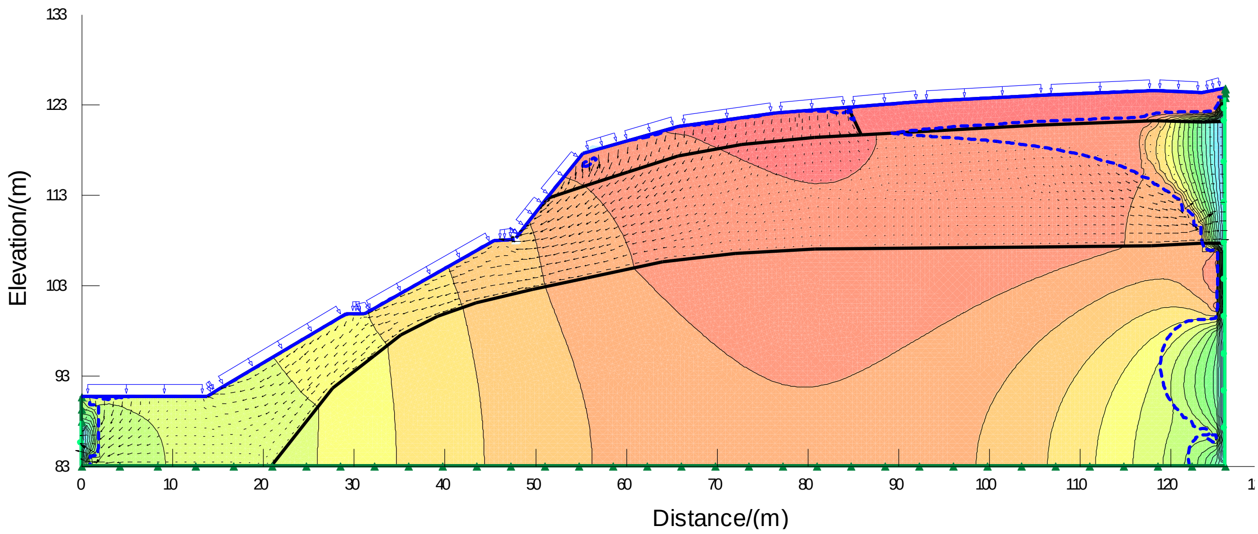


Figure 4.14 : Ground Water Regime with No Drains

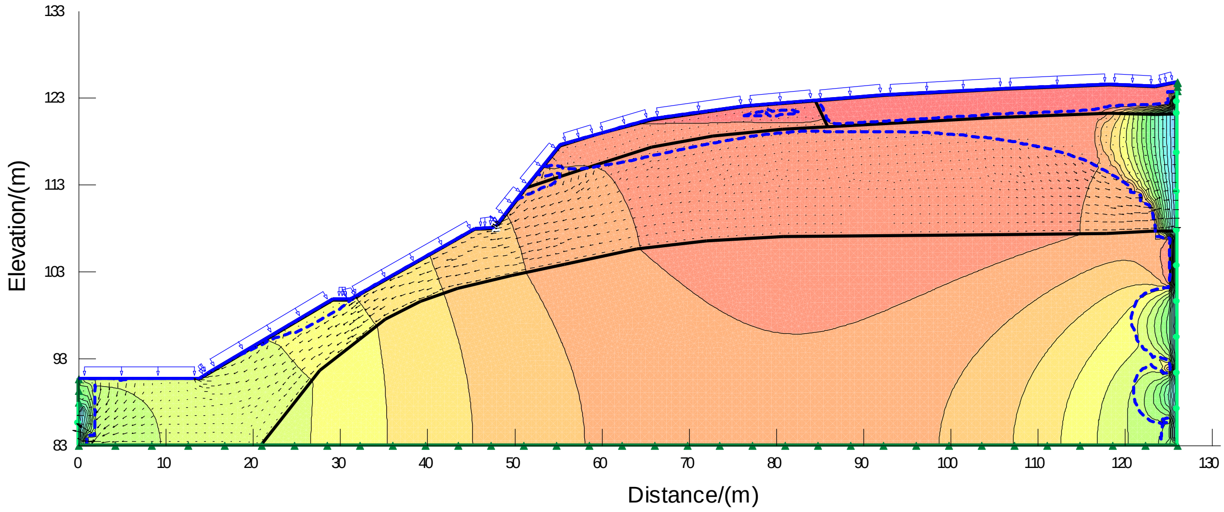


Figure 4.15 : Ground Water Regime with Surface Drains

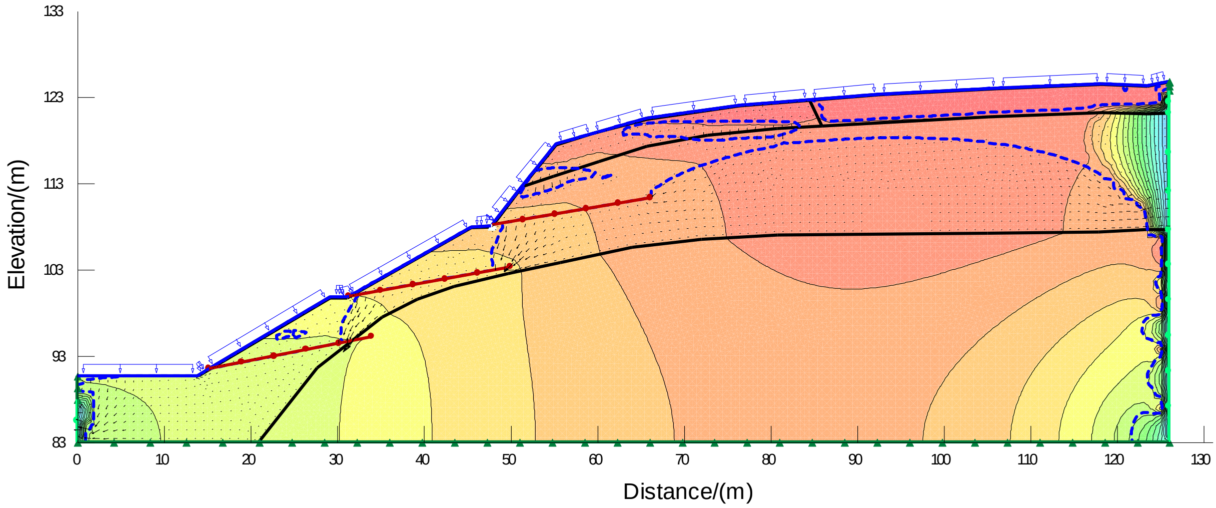


Figure 4.16 : Ground Water Regime with Surface and Subsurface Drains

If no drainage measures are applied, ground water level rises to surface level with the two days of peak rainfall. With drainage improvement, ground water level lowers from the ground level and different types of drainage improvement has different effects. When surface drainage implemented, ground water level reduces relatively parallel to the ground surface. When both surface and sub horizontal gravity drains are used, ground water level further lowered. Ground water level directly above the sub horizontal gravity drain reduces to the drain level and it takes parabolic shape beyond the edge of the drain pipe.

Effect on the ground water regime is further discussed in Figure 4.17 to Figure 4.22, in term of pore pressure distribution along Section A-B & C-D as shown in Figure 4.9. In those illustrations, graphs of different graphs represent the pore pressure distribution at different time steps as described below.

Initial condition

At end of 4th day (end of dry period)

At end of 7th day (end of peak rainfall)

At end of 14th day (end of residual rainfall)

<u>No drains condition</u>

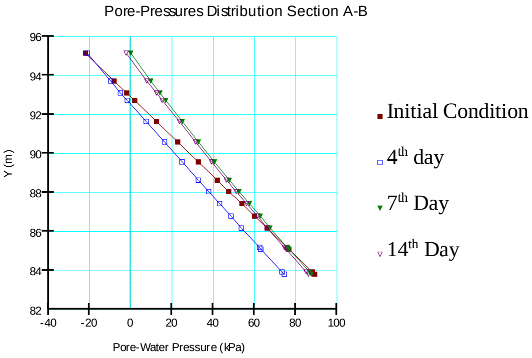


Figure 4.17 : Pore-Pressure Distribution at Section A-B with No Drains

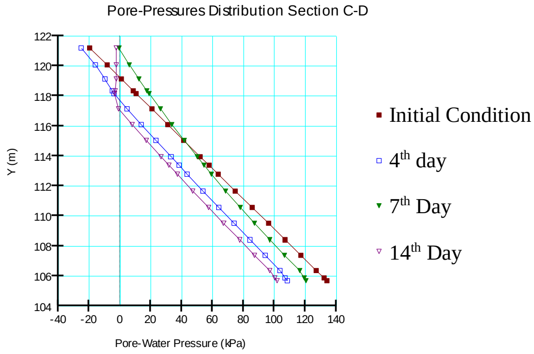


Figure 4.18 : Pore-Pressure Distribution at Section C-D with No Drains

<u>Application of surface drains condition only</u>

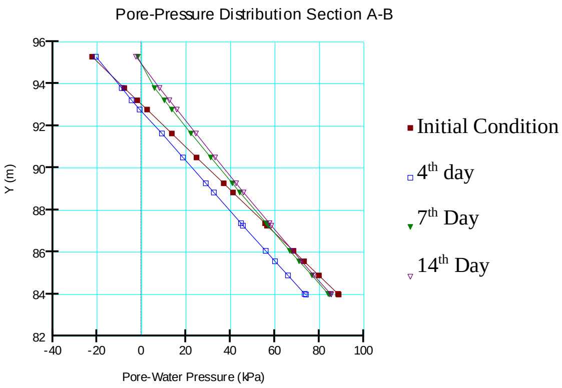


Figure 4.19 : Pore-Pressure Distribution at Section A-B with Surface Drains

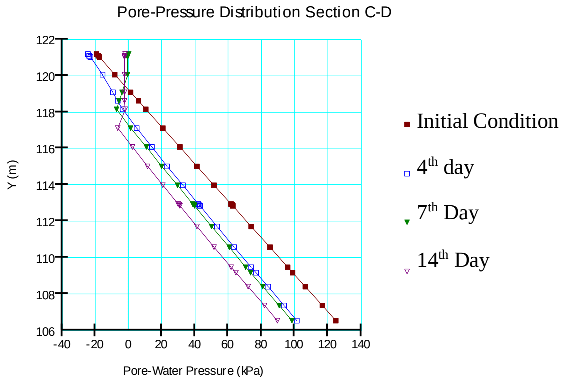


Figure 4.20 : Pore-Pressure Distribution at Section C-D with Surface Drains

The response of the slope to the critical rainfall when only surface drains are present is presented in Figure 4.19 and Figure 4.20. As illustrated in Figure 4.17 and Figure 4.19, behavior of pore pressure distribution of the Section A-B which is at the toe region of the slope is almost same for the both “no drainage improvement” and “only surface drainage improvement” condition. Under both conditions, ground water table has got lowered during the five day long dry period and rises very close to the surface level with the peak rainfall. Thereafter, pore pressure distribution does not vary considerably during residual rainfall period.

As illustrated in Figure 4.18 and Figure 4.20, lowering of ground water table during dry period is almost similar at the Section C-D which is at the crest region of the slope, for the both “no drainage improvement” and “only surface drainage improvement” condition. With the critical rainfall under “no drainage improvement” conditions, ground water level rises almost up to the surface level, just after the peak rainfall. When the surface drainage improvement is done, ground water level does not rise and only the matric suction is lost up to the ground water level due to the peak rainfall. Positive pore pressures near the surface denotes slight development of perched water table. Thereafter, ground water level lowers during the residual rainfall period under both conditions and small matric suction is developed just above the water table when surface drainage improvement is in place.

This results imply that, provision of surface drainage improvement only does not influence the ground water regime significantly although there is some reduction of amount of infiltration during the rains. The effect on the ground water regime at the crest of the slope is more than that at the toe region.

<u>Use of both surface drains and subsurface drains conditions</u>

The response of the slope to the critical rainfall when both surface and subsurface drains are present is present in Figure 4.21 and Figure 4.22. When Figure 4.21 is compared with Figure 4.17 and Figure 4.19, it could be seen that when both surface and subsurface drains are applied, pore pressures are further lowered during the dry period at Section A-B, which is at the toe of the slope. However, with the peak rainfall, rise of ground water table is prevented and only the matric suction above the ground water level is lost and there is not much variation in the pore pressure distribution during the residual rainfall period. This behavior is similar to the case of no drains and surface drains only conditions.

When Figure 4.22 is compared with Figure 4.18 and Figure 4.20, it could be seen that when both surface and subsurface drains are applied, pore pressures has reduced significantly and the ground water table has much lowered during the dry period with compared to the cases of no drains are surface drains only conditions, at Section C-D, which is at the crest region of the slope. With the peak rainfall, ground water level has not changed much and only the matric suction is lost above the ground water level. Positive pore pressures near the surface denotes slight development of perched water table. Afterwards, ground water level has lowered during the residual rainfall and matric suction is developed above the ground water level.

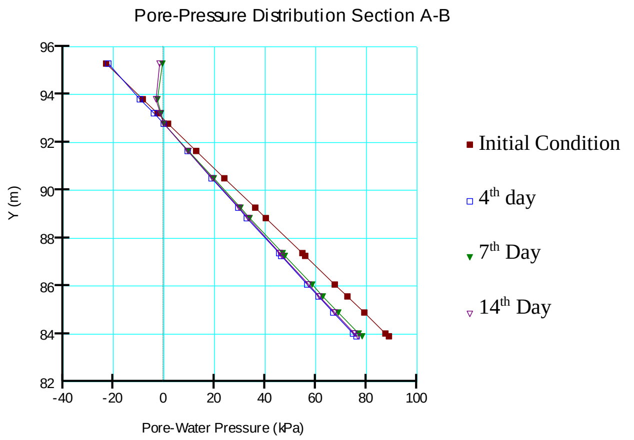


Figure 4.21 : Pore-Pressure Distribution at Section A-B with Surface & Subsurface Drains

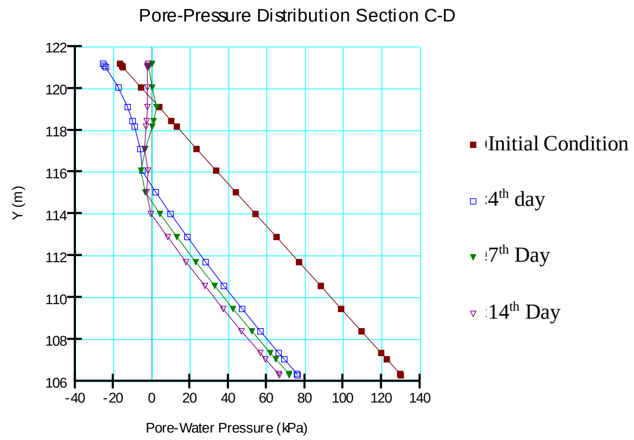


Figure 4.22 : Pore-Pressure Distribution at Section C-D with Surface & Subsurface Drains

This results clearly indicate that the application of both surface and subsurface drainage improvement is more effective in controlling the buildup of high pore water pressures during the periods of heavy rains.

4.4 Effect of drainage and reinforcement on slope stability

Effect of the drainage improvement and reinforcement on the slope stability was assessed by SLOPE/W software using Spencer's method of stability analysis.

Influence of surface and subsurface drains working in combination to reduce the adverse effects of rainfall on the pore pressure regime of the slope was illustrated in preceding section. Results of those analyses done with SEEP/W software are incorporated into the stability analyses done with SLOPE/W software to assess how those drainage measures would influence the safety margins of the slope during rainfall.

Analyses were done here to evaluate how the safety margins varied during the different construction stages, Construction Stage 1, Construction Stage 2 & Construction Stage 3, as outlined before. When the safety margin of the slope is not adequate at a particular stage with only one type of rectification measures (drainage or reinforcement) the combination of measures were considered.

4.4.1 Construction Stage 1

The excavation to Construction Stage 1 with different drainage measures and with & without reinforcement conditions were modeled. The critical rainfall was applied on the slope and the effect was analyzed with SEEP/W software. Since the construction of Stage 1 would take at least three weeks, critical rainfall may occur during construction. The stability was evaluated incorporating the pore water pressures distribution obtained from the seepage analysis into the SLOPE/W software.

Critical failure surfaces and FoS obtained after peak rainfall at Construction Stage 1 is presented from Figure 4.23 to Figure 4.26. The FoS obtained without drainage improvement or reinforcement is 0.783. Therefore, drainage measures were applied and FoSs of 0.874 and 1.046 were obtained for surface drainage only and both surface & subsurface drainage conditions respectively. This FoSs are not adequate. As such, reinforcing with nailing is proposed and FoS has increased to 1.734. The failure surfaces and corresponding to the reinforcement is presented in Figure 4.26. This implies that to complete the Construction Stage 1 with a sufficient safety margin it is necessary to implement both drainage measures and reinforcement.

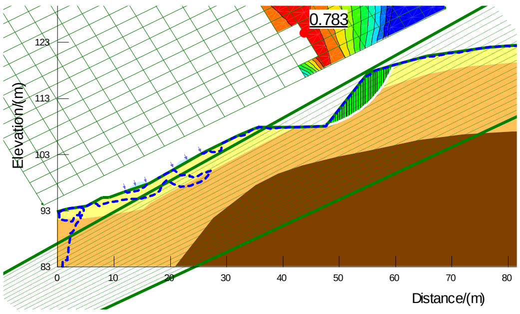


Figure 4.23 : Analysis without Drains and without Nails – Stage 1

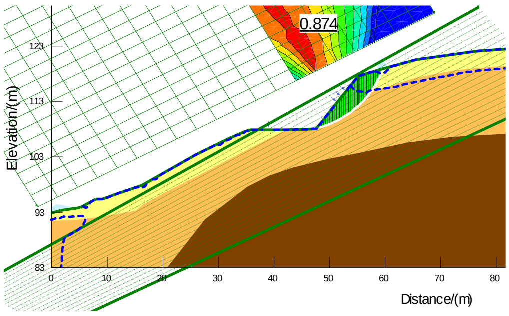


Figure 4.24 : Analysis with only Surface Drains and without Nails – Stage 1

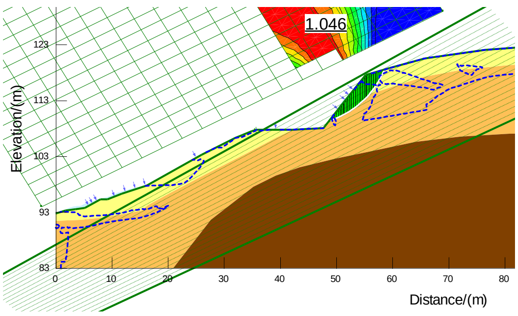


Figure 4.25 : Analysis with Surface & Subsurface Drains and without Nails – Stage 1

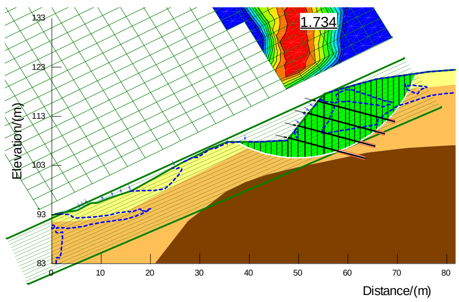


Figure 4.26 : Analysis with Surface & Subsurface Drains and with Nails – Stage 1

The variation of FoS during the construction up to Construction Stage 1 under different rectification measures when the slope was under the influence of critical rainfall is presented in Figure 4.27.

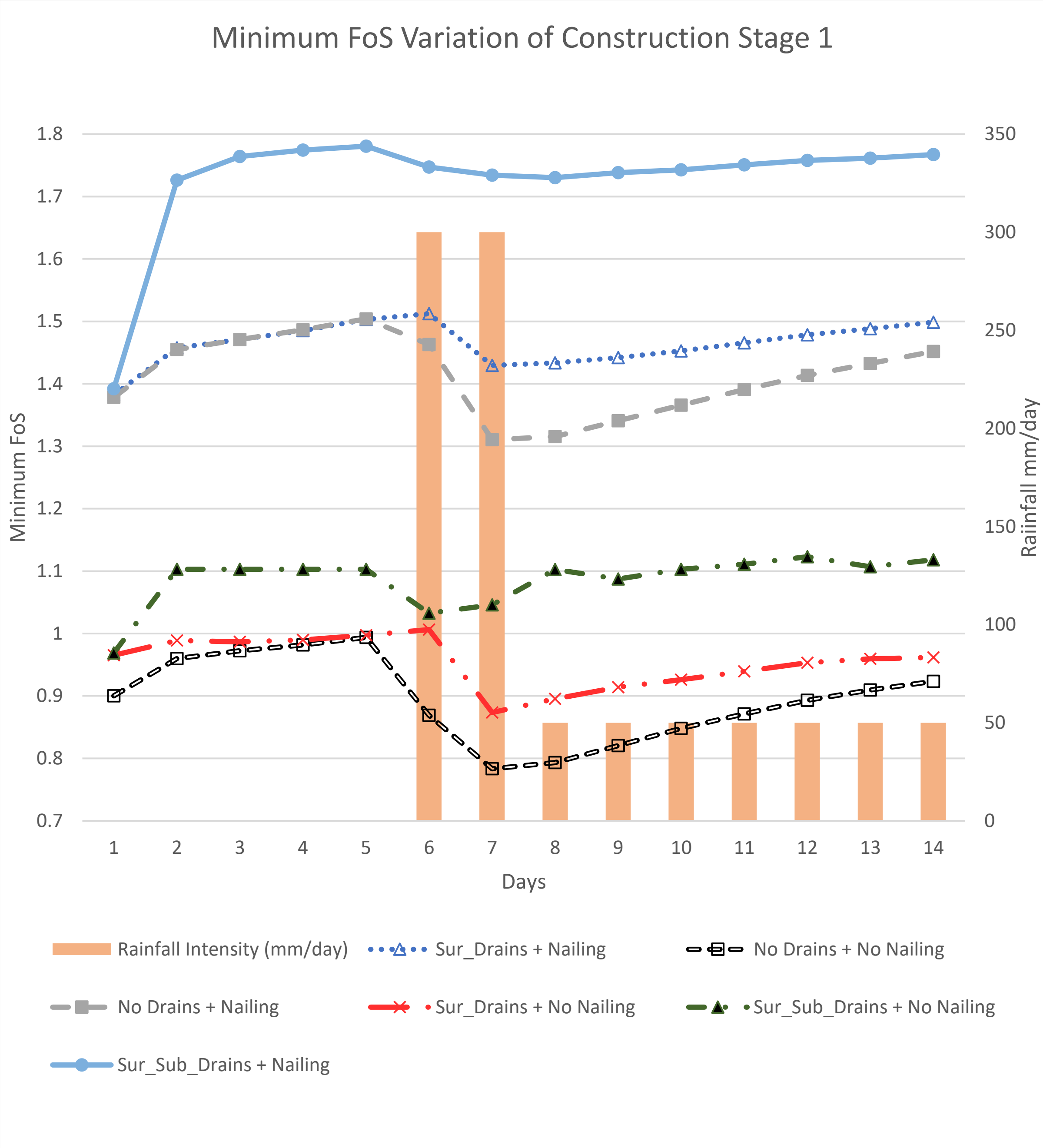


Figure 4.27 : Minimum FoS Variation at Construction Stage 1

During the first four days, which is the dry period, FoS has increased gradually in both "no drains" and "only surface drains" condition. Variation is almost similar for both cases. Surface drains are effective in directing the rainfall away from the slope minimizing the infiltration. As such, under the condition of "no rainfall" they will not have any influence. When the sub surface drainage improvement is applied FoS has rapidly increased and maintained at a higher value. This is due to the effect of subsurface drains to drain out the water already in the slope.

When the peak rainfall is applied, FoS is significantly reduced in both “no drains” and “only surface drains” condition. However, when surface drains are there, reduction of FoS is lesser and delayed compared no drains condition. Reduction of FoS is significantly lesser when subsurface drains are also in place.

FoS again increases with the residual rainfall for “no drains” and “only surface drains” condition. When subsurface drains are in place, only small increase in the FoS is observed during residual rainfall is observed as there is not much reduction of FoS during peak rainfall.

FoS with “no drains” and “only surface drains” condition is less than unity when no nails are applied, which implies the slope is unstable.

These results imply that the surface drainage improvement slightly increases the FoS of the slope and with the combination of surface and subsurface drainage there is a significant improvement of the FoS. Further, application of reinforcement has significantly improved the FoS of all drainage conditions, and it had been possible to maintain the long term stability requirement (FoS > 1.3) for all drainage conditions with the presence of soil nails. When nails are there under “no drain” or “only surface drains” conditions, the FoS values are of the order 1.4 – 1.5. When subsurface drains are introduced this increased to value over 1.7. When nails are not applied, even the short term stability requirement (FoS > 1.1) cannot be achieved.

4.4.2 Construction Stage 2

The excavation to Construction Stage 2 with different drainage measures and with & without reinforcement conditions were modeled. The critical rainfall was applied on the slope and the effect was analyzed with SEEP/W software. Since the construction of Stage 2 would take at least three weeks, critical rainfall may occur during construction. The stability was evaluated incorporating the pore water pressures distribution obtained from the seepage analysis into the SLOPE/W software.

Critical failure surfaces and FoS obtained after peak rainfall at Construction Stage 2 is presented from Figure 4.28 to Figure 4.31. The FoS obtained without drainage improvement or reinforcement is 0.791. Therefore, drainage measures were applied and FoSs of 0.850 and 1.074 were obtained for surface drainage only and both surface & subsurface drainage conditions respectively. This FoSs are not adequate. As such, reinforcing with nailing is proposed and FoS has increased to 1.535. The failure surfaces and corresponding reinforcements are presented in Figure 4.31. This implies that to complete the Construction Stage 2 with a sufficient safety margin it is necessary to implement both drainage measures and reinforcement.

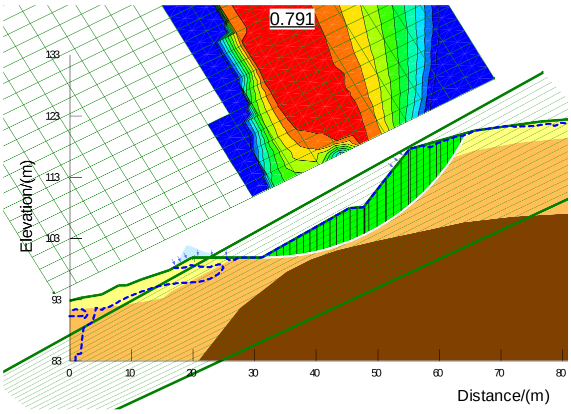


Figure 4.28 : Analysis without Drains and without Nails – Stage 2

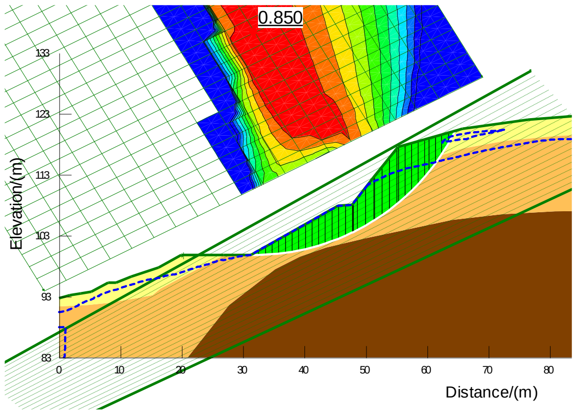


Figure 4.29 : Analysis with only Surface Drains and without Nails – Stage 2

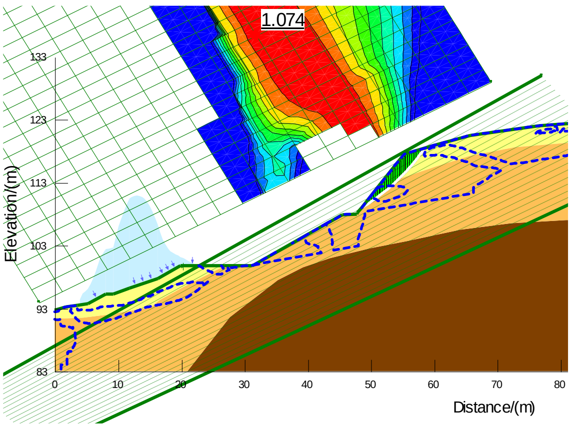


Figure 4.30 : Analysis with Surface & Subsurface Drains and without Nails – Stage 2

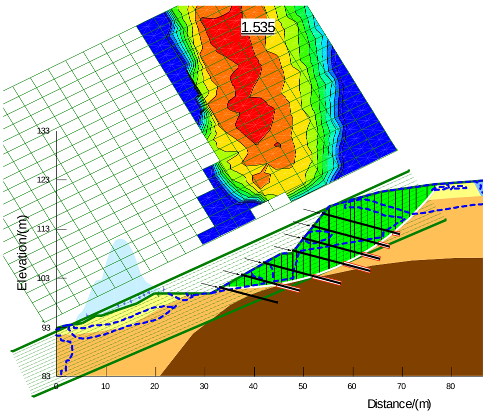


Figure 4.31 : Analysis with Surface & Subsurface Drains and with Nails – Stage 2

The variation of FoS during the construction up to Construction Stage 2 under different rectification measures when the slope was under the influence of critical rainfall is presented in Figure 4.32.

Variation pattern of FoS in Construction Stage 2 is more or less similar that for Construction Stage 1 with several significant features.

FoS slightly reduces after an initial increment, during the dry period in “no drains” and “only surface drains” condition without nails. Increase of the pore pressures close to toe region due to the downward seepage within the soil even during the dry period, may be the reason. However, when subsurface drains are present, FoS increased rapidly and maintained at the same value or slightly increase during the dry period for both with and without nail conditions.

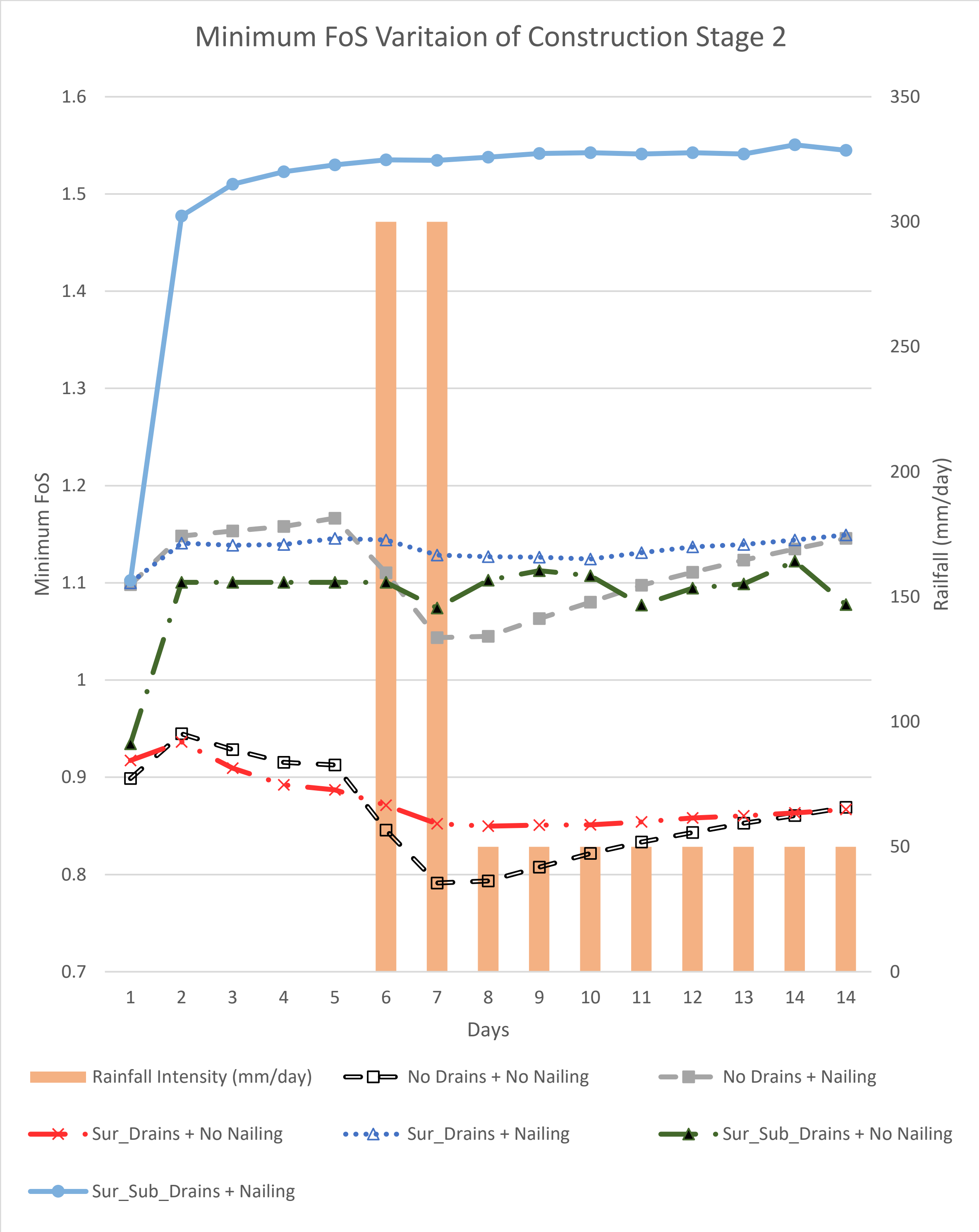


Figure 4.32 : Minimum FoS Variation at Construction Stage 2

As in Construction Stage 1, minimum FoS drastically reduce with the peak rainfall in “no drains” and “only surface drain” condition and reduction of FoS is minimum when subsurface drains are in place.

Afterwards during residual rainfall, minimum FoS increases as in Construction Stage 1, for “no drains” and “only surface drains” condition. Variation of minimum FoS is minimum with the subsurface drainage condition.

The most significant feature is that the minimum FoS is in the same range for both “nailing without subsurface drainage improvement” and “no nailing with subsurface drainage improvement” condition. However, FoS values of this range (1.1) are not acceptable. That highlights the importance of the subsurface drainage improvement when the soil nailing technique is applied. Two techniques are complimentary.

It should be noted that, without nailing FoS is less than unity at the initial stage and it increases above the unity only when subsurface drains are applied. These results imply that the surface drainage improvement slightly increases the FoS of the slope and combination of surface and subsurface drainage improvement has a significant effect on the minimum FoS. Further, application of reinforcement has significantly improved the FoS of all drainage conditions and it was able to maintain the long term stability requirement (FoS > 1.3) only with surface and subsurface drainage improvement together with the presence of soil nails. Also, the short term stability requirement (FoS > 1.1) can only be achieved by combination of soil nailing with drainage improvement.

4.4.3 Construction Stage 3

The excavation to Construction Stage 3 with different drainage measures and with & without reinforcement conditions were modeled. The critical rainfall was applied on the slope and the effect was analyzed with SEEP/W software. Since the construction of Stage 3 would take at least three weeks, critical rainfall may occur during construction. The stability was evaluated incorporating the pore water pressures distribution obtained from the seepage analysis into the SLOPE/W software.

Critical failure surfaces and FoS obtained after peak rainfall at Construction Stage 3 is presented from Figure 4.33 to Figure 4.36. The FoS obtained without drainage improvement or reinforcement is 0.794. Therefore, drainage measures were applied and FoSs of 0.847 and 1.066 were obtained for surface drainage only and both surface & subsurface drainage conditions respectively. This FoSs are not adequate. As such, reinforcing with nailing is proposed and FoS has increased to 1.539. The failure surfaces and corresponding to the reinforcement is presented in Figure 4.36. This implies that to complete the Construction Stage 3 with a sufficient safety margin it is necessary to implement both drainage measures and reinforcement.

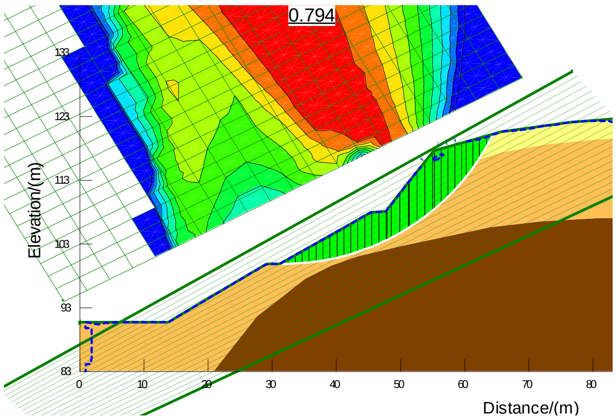


Figure 4.33 : Analysis without Drains and without Nails – Stage 3

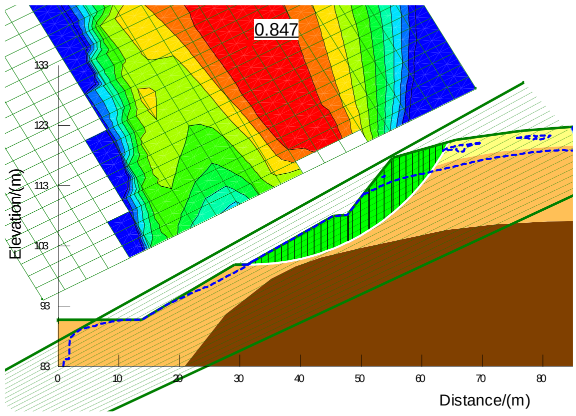


Figure 4.34 : Analysis with only Surface Drains and without Nails – Stage 3

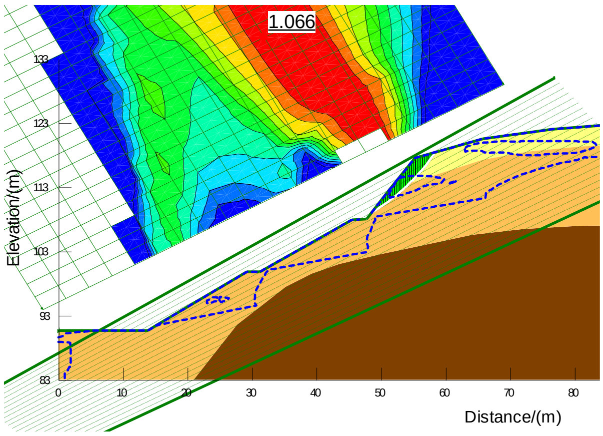


Figure 4.35 : Analysis with Surface & Subsurface Drains and without Nails – Stage 3

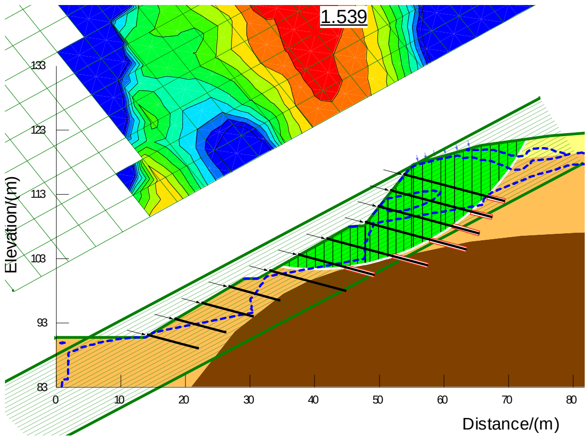


Figure 4.36 : Analysis with Surface & Subsurface Drains and with Nails – Stage 3

The variation of FoS during the construction up to Stage 3 under different rectification measures when the slope was under the influence of critical rainfall is presented in Figure 4.37.

Behavior of the variation of the minimum FoS in Construction Stage 3 is similar to the Construction Stage 2, because the critical slip surfaces in Construction Stage 2 and 3 are similar and excavation and nailing of the bottom segment does not affect the critical slip surface.

Results imply that the surface drainage improvement slightly increases the FoS of the slope and combination of surface and subsurface drainage improvement has a significant effect on the minimum FoS. Further, application of reinforcement has significantly improved the FoS of all drainage conditions. However, it was able to maintain the long term stability requirement (FoS > 1.3) only when there is both surface and subsurface drainage improvement together. It should be also noted that even the short term stability requirement (FoS > 1.1) can only be achieved by combination of soil nailing with drainage improvement. With “only drainage” or “only reinforcement” conditions FoS values are around 1.1.

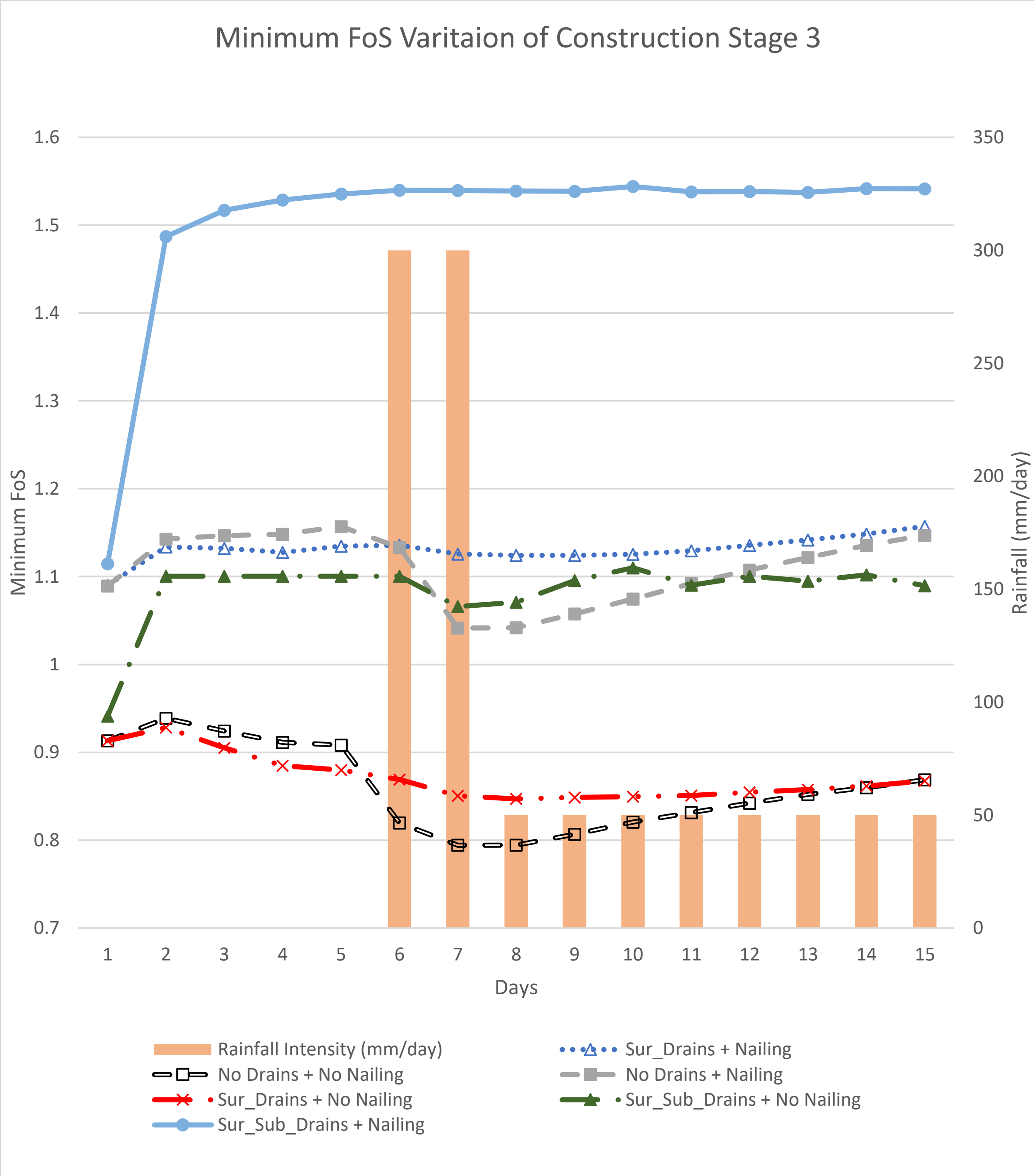


Figure 4.37 : Minimum FoS Variation at Construction Stage 3

4.4.4 Summary of the stability analyses

Behavior of critical slip surface

Above results shows that the critical slip surface has moved deeper in to the slope due to the application of reinforcement by nails resulting the improvement of factor of safety values. Subsurface drainage improvement has restricted the failure to top segment of the slope for Construction Stage 2 and Construction Stage 3, thereby increased the FoS.

Critical slip surface obtained for Construction Stage 2 and Construction Stage 3 are almost similar, since developing the slip surface further downwards in Construction Stage 3 is prevented by the underlain rock layer (Figure 3.10).

Summary of FoS

Summary of the critical FoS after peak rainfall is presented in Table 4.3.

Table 4.3 : Summary of FoS

Slope Condition		Construction Stage 1	Construction Stage 2	Construction Stage 3
Drainage	**Reinforcement**			
No Drains	No Nailing	0.783	0.791	0.794
No Drains	Nailing	1.310	1.044	1.042
Surface Drains Only	No Nailing	0.874	0.850	0.847
Surface Drains Only	Nailing	1.430	1.124	1.124
Surface and Subsurface Drains	No Nailing	1.046	1.074	1.066
Surface and Subsurface Drains	Nailing	1.734	1.535	1.539

Summary of the stability analysis results reveals that the required safety margin after completion of Construction Stage 3 can only be achieved by applying both surface & subsurface drainage along with reinforcement by soil nailing.

4.5 Economizing the nailing design by drainage improvement

The implemented design involves both surface and subsurface drainage together with reinforcement. The effect of surface and subsurface drainage in economizing the nailing design was evaluated by comparing the excavation to Construction Stage 3 under different drainage conditions of;

- without any drainage measures,
- with only surface drainage measures and
- with both surface and subsurface drainage measures (actually implemented desing).

Considering the long term stability, the FoS value to be maintained under any condition was 1.3. All stability analyses were done at the end of peak rainfall (day 7).

4.5.1 Without any drainage measures

Stability of the slope for "no drainage" condition after the peak rainfall was assessed (Figure 4.38) under the designed and implemented nailing pattern of;

- 5 nos. of 16 m long nails in 2.0 m horizontal and 2.5 m vertical spacing
- 2 nos. of 12 m long nails in 2.0 m horizontal and 2.5 m vertical spacing
- 4 nos. of 8 m long nails in 2.0 m horizontal and 2.5 m vertical spacing

Total nail length is 1,624 m.

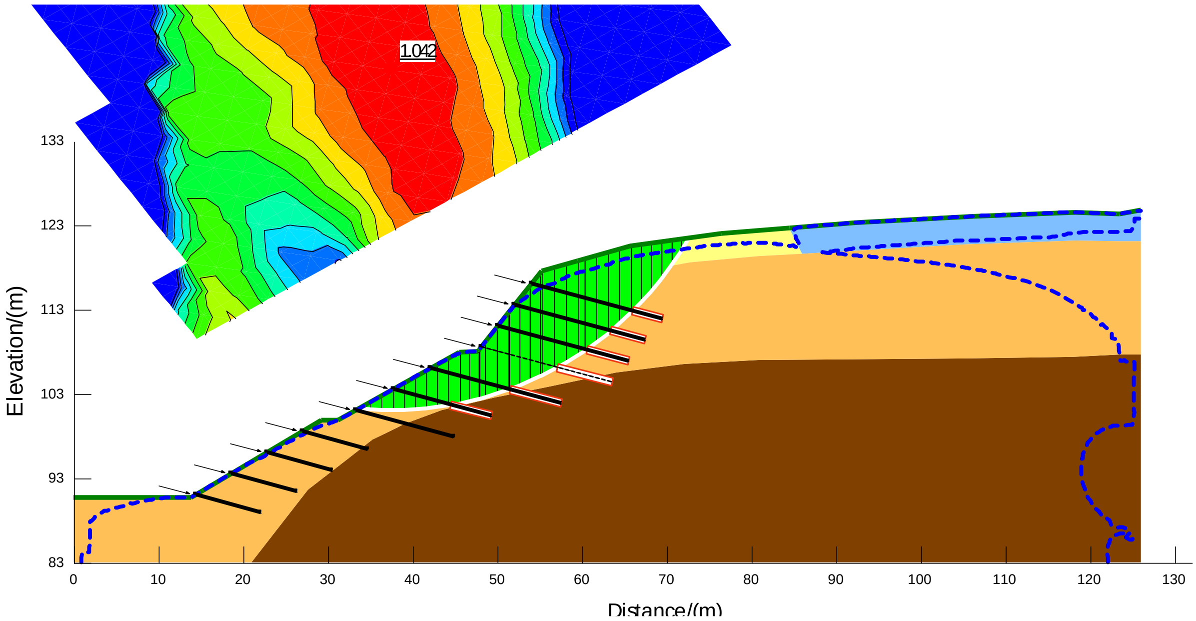


Figure 4.38 : Implemented Soil Nailing Design with No Drainage Improvement

The results of the analysis present in Figure 4.38 and Table 4.3 indicate that when there are no drainage measures a sufficient safety margins cannot be obtained (FoS < 1.3) with the applied drainage arrangement.

Then the nailing pattern was modified to achieve the required FoS of 1.3 with the critical rainfall (Figure 4.39). Since the increasing the nailing length more than 16 m was not practical and effective, nail grid was densified while increasing nail length of only 12 m nails. After several trials, following nailing pattern was obtained;

- 12 nos. of 16 m long nails in 1.5 m horizontal and 1.5 m vertical spacing
- 4 nos. of 8 m long nails in 2.0 m horizontal and 2.5 m vertical spacing

Bottom nailing arrangement was not necessary to change since the critical slip surface not going through that nails area.

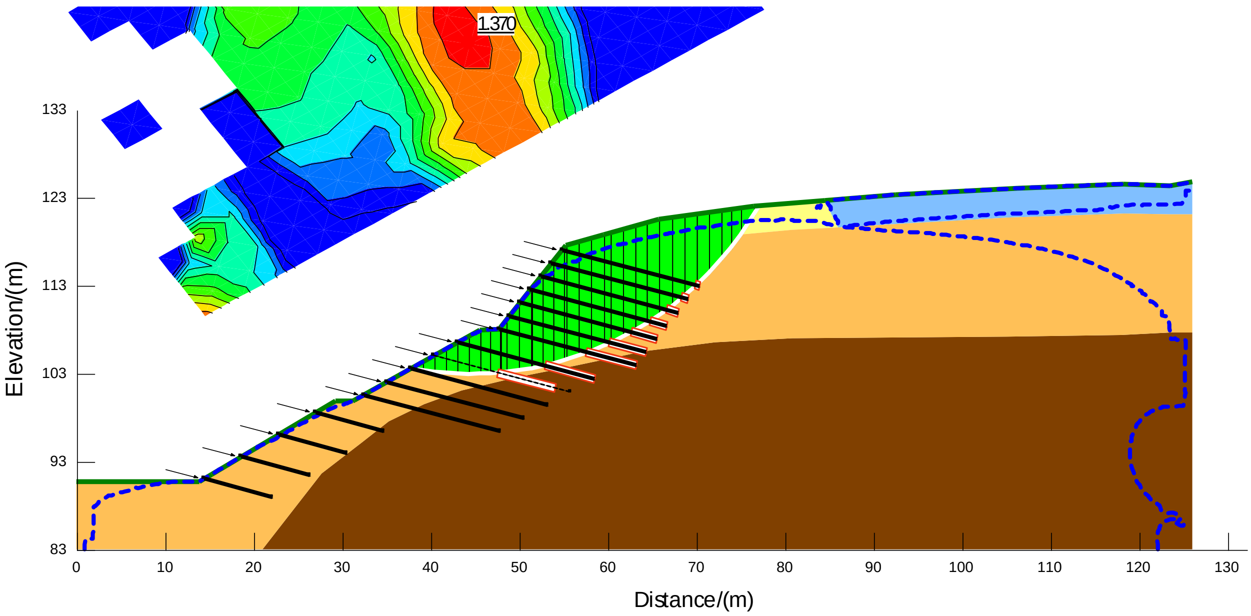


Figure 4.39 : Required Soil Nailing to Achieve Sufficient FoS with No Drainage

As such, if no drainage improvement is done total number of nails and nail lengths were increased very significantly. Thus the, total nailing length was increased to 3,112 m. Further, length of nail connecting beams are required to be increased due to densifying the grid and increasing the number of nails.

4.5.2 With surface drainage only

Stability of the slope for “only surface drainage” condition after the peak rainfall was assessed (Figure 4.40) under the designed and implemented nailing pattern.

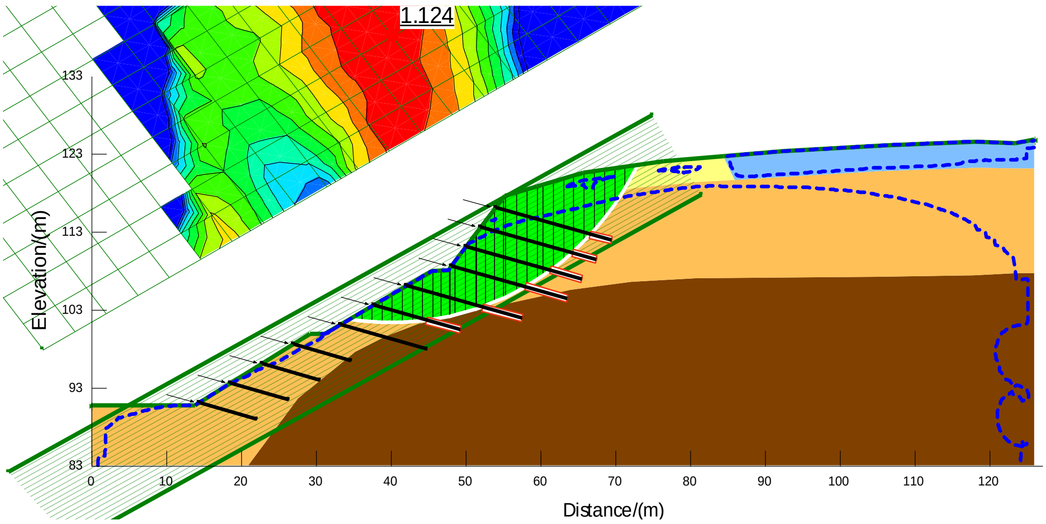


Figure 4.40 : Implemented Soil Nailing Design with Surface Drainage Improvement

The results of the analysis present in Figure 4.40 and Table 4.3 indicate that when there are no drainage measures a sufficient safety margins cannot be obtained (FoS < 1.3).

Then the nailing pattern was modified to achieve the required FoS of 1.3 with the critical rainfall (Figure 4.41). Since the increasing the nailing length more than 16 m was not practical and effective, nail grid was densified while increasing nail length of only 12 m nails. After several trials, following nailing pattern was obtained;

- 9 nos. of 16 m long nails in 1.5 m horizontal and 2.0 m vertical spacing
- 4 nos. of 8 m long nails in 2.0 m horizontal and 2.5 m vertical spacing

Bottom nailing arrangement was not necessary to change since the critical slip surface not going through that nails area.

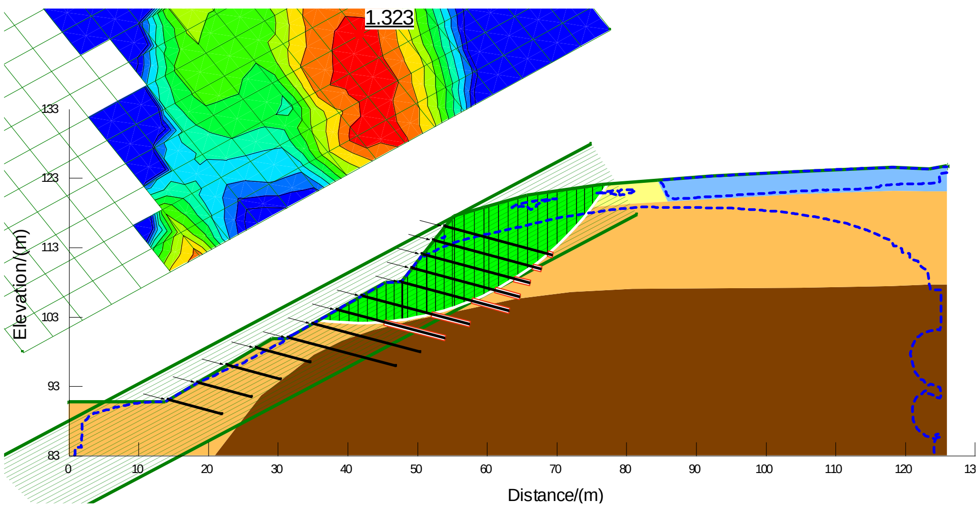


Figure 4.41 : Required Soil Nailing to Achieve Sufficient FoS with Surface Drainage

As such, when "no drainage improvement" or "only partial drainage improvement" is done total number of nails were increased with densifying of the nailing. Total nailing length was increased to 2,624 m. Further, length of nail connecting beams are required to be increased due to densifying the grid and increasing the number of nails.

4.5.3 Cost comparison

Construction cost for each drainage measures were calculated based on the average rates in the industry. Table 4.4 presents the comparison of construction cost under different drainage conditions. It is clear that cost of nailing has significantly reduced by application of drainage measure. Drainage measures comes with a cost. However, cost of drainage measures are less than the cost of nailing. Therefore, total construction cost has reduced by application of drainage measures.

Construction cost without any drainage measures (with only nailing) in Rs. 49.0 Mn. Cost saving by application of surface drainage is Rs. 1.3 Mn and it is 2.65% saving from the no drainage condition. Cost saving by application of both surface and subsurface drainage is Rs. 7.1 Mn and it is 14.49% saving from the no drainage condition.

Table 4.4 : Construction Cost Comparison for Different Drainage Measures

Drainage Condition	Cost of Nailing and Associated Works (Rs. Mn)	Cost of Drainage Improvement (Rs. Mn)	Cost of Excavation and Other Works (Rs. Mn)	Total Cost (Rs. Mn)	Cost Saving
No Drains	41.6	0	7.4	**49.0**	0.00%
Only Surface Drains	34.9	5.4	7.4	**47.7**	2.65%
Both Surface and Subsurface Drains	25.4	9.1	7.4	**41.9**	14.49%

Above results indicate that soil nailing design can be economized by application of drainage measures. Construction cost has significantly reduced by the application of subsurface drains to the slope with surface drains.

Therefore, it is important to have both surface and subsurface drainage improvement for economizing the nailing design.

4.3.2 Importance of the staged construction

Figure 4.42 illustrate the variation of minimum of FoS values at each Construction Stage correspond to different drainage and reinforcement conditions under peak rainfall.

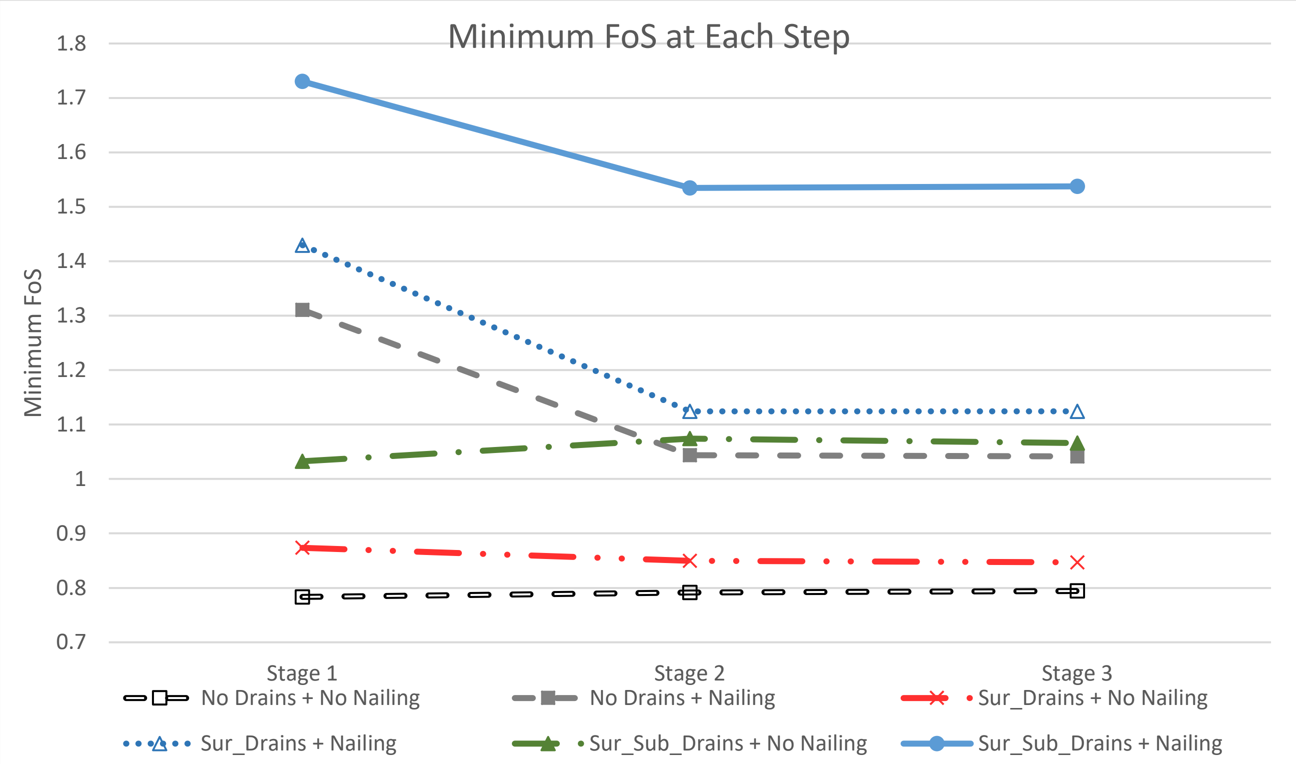


Figure 4.42 : Minimum values of minimum FoS at each Construction Stage

Minimum of FoS values is always less than 1.1 when the soil nails are not applied. Also it is less than unity for the “no drains” and “only surface drains”. Since at least a FoS value of 1.1 is required in short term stability, stability of the Construction Stage 1 cannot be maintained without nailing. Therefore, soil nails should be applied with the excavation adopting top down approach. If the slope is soil nailed in Construction Stage 1, minimum FoS is always above 1.3.

At the Construction Stage 2 minimum value of FoS is lowered below 1.1 when nails are applied without drainage measures. Therefore, top down approach along with drainage measures must be adopted in order to maintain the stability of the slope at Construction Stage 2.

At the Construction Stage 3 minimum value of FoS is below 1.1 implying unstable slope when nails are not applied and nails are applied without drainage measures. Therefore, top down approach must be adopted to maintain the stability of the slope at Construction Stage 3.

Therefore, the short term stability requirement can be achieved only through applying soil nails together with drainage improvement adopting the staged construction.

CHAPTER 5 - CONCLUSIONS AND RECOMMENDATIONS

Notable during past few decades, rain induced landslides have become a major natural disaster in Sri Lanka, in the hill country during the monsoons with heavy rainfall. Considering the devastating nature and effect on the social, economic and environmental aspects by those landslides, many landslide mitigation and slope stabilization projects are being implemented all over the landslide prone areas in the country.

In the rain induced landslides, drainage improvement is considered as a mandatory and vital component in mitigation measures. Surface and subsurface drainage improvement are applied under drainage improvement. When the required safety margins cannot be achieved only through drainage improvement, earth retaining structures are used to enhance the safety margins of the slopes to a desired level. Soil nailing technique is used for the stabilization of the steep high cut slopes adopting the top down approach.

In this study, effectiveness of different types of drainage improvement methods on ground water regime management and resultant effect on the slope stability and, effectiveness of soil nailing on the slope stability are assessed considering the Ginigathhena landslide mitigation project. Also the importance of top down approach was also studied. Further ground water level and rainfall monitoring data was used to confirm the results obtained by the analyzes.

GeoStudio (2016) SEEP/W software, which is based on a finite element formulation, was used for analyzing the effect on the ground water regime by different types of drainage improvement under different rainfall intensities. GeoStudion (2016) SLOPE/W software, which is based on limit equilibrium method, was used for the analysis of slope stability incorporating the results of SEEP/W. Many alternative solutions with different drainage improvement techniques and rainfall intensities were considered in the analysis. Considering the importance of construction sequence, the staged application of different drainage measures were modelled in the study.

Based on the results obtained by seepage and slope stability analyses following conclusions can be made.

- The failure that occurred after excavation for the new bridge abutment was triggered due to removal of toe support with low ground water table. Small failure occurred at the steep cut slope made with the excavation and it was gradually propagated to the upper slope.

- Reasonably accurate seepage model can be developed by SEEP/W software to simulate the drainage measures and rainfall infiltration. SWCC and hydraulic conductivity functions developed for Sri Lankan residual soils by (Vasanthan, 2016) is sufficiently accurate for use in the seepage model.
- Surface drainage improvement alone does not affect the existing ground water regime significantly although it reduces the amount of infiltration during the rains.
- Subsurface drainage improvement significantly affect the existing ground water regime directly and effect of infiltration during rains can be eliminated.
- Applying both surface and subsurface drainage improvement is more effective since both existing ground water regime and rainwater infiltration are directly affected.
- Trench drains were used for lowering the water stagnation at the marshy area at crest during the rainy periods.
- It is necessary to study how the safety margins of the slope will vary during the proposed construction stages. Different rectification measures of drainage and reinforcement should be applied on the correct sequence to ensure that a sufficient safety margin will be maintained at any time during an expected critical rainfall.
- The study indicated how the safety margins of the slope would vary with the rainfall patterns. If the drainage measures are implemented the Factor of Safety should not decrease significantly during a period of high intensity rainfall. The increase of safety margin during the period of dry weather was also clearly observed.
- Use of nailing will apply large tensile forces across potential shallow failure surface and the failure surface will be pushed further deep into the slope. If the soil layers at deeper levels are more competent, this will cause a further increase of safety margins.
- The soil nailing design can be significantly economized by the use of efficient drainage measures. The analyzes indicated that if the water table is not lowered, much longer nails would be required at closer spacing. Further, there would be many practical difficulties in drilling the holes for installation of nails below the ground water table.
- A properly designed combination of drainage and reinforcement will make the project more economized.
- It is necessary to have appropriate monitoring of ground water table during the construction to ensure that drainage measures applied are preforming as expected.
- The subsurface drains installed at a given horizontal spacing is idealized in the plain strain formulation as boundary condition of zero pore water pressure to the length of

the sub horizontal gravity drains. Here it is assumed that the sub horizontal gravity drain spacing is closer enough. If the drainage effect can be simulated three dimensionally, the process can be modeled more accurately and the effect of horizontal spacing of sub horizontal gravity drains can also be studied.

- All rectifications implemented in a slope should be closely monitored periodically to ensure that they perform according to the designed functions. Monitoring of pore water pressure variations and slope movement at identified critical locations is also very important. Slope movement may be captured by either the instrumentation installed at the site or by remote sensing techniques.

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Annex A - Investigation

BH - 01

BOREHOLE LOG

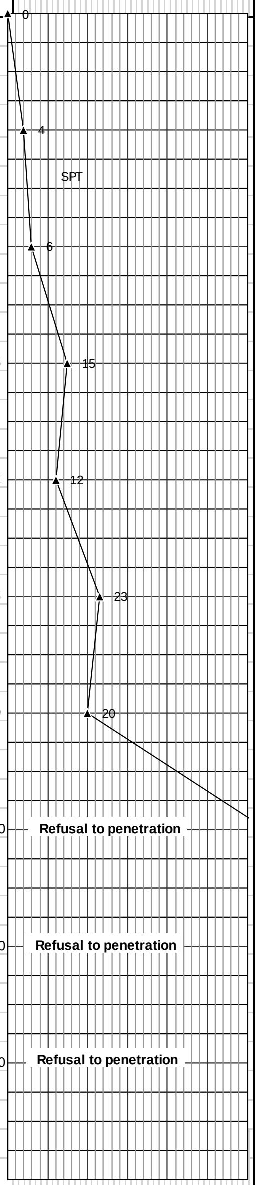
NATIONAL BUILDING RESEARCH ORGANISATION
GEOTECHNICAL ENGINEERING DIVISION
99/1, Jawatta Road, Colombo 05.

SHEET NO. 1 of 1

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 01
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	9.90
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	96.0	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	29/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	29/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SOIL DESCRIPTION	STRATA	GWL	DEPTH TESTED [m]	SPT 1	SPT 2	SPT 3	FOR 30cm
0.00	96.00		GROUND LEVEL							
			Very loose, yellowish brown, silty SAND, angular to subangular, fine to coarse grained with mica and plant roots (top soil), moist	SM		0.00	1	0	0	0
0.90 / 1.00	95.10	0.90				1.00	1	2	2	4
2.00	94.00		Very loose to loose, yellowish brown, silty SAND, fine to medium grained with mica, moist	SM		2.00	3	3	3	6
3.00	93.00	2.10				3.00	9	9	6	15
			Medium dense,orangish brown, silty GRAVEL, angular to subangular, medium grained with sand, moist	GM						
4.00	92.00	1.00				4.00	5	6	6	12
5.00			Medium dense, greyish brown to yellowish brown silty SAND, subangular, fine to coarse grained with gravel moist.(silt content increase with depth)	SM		5.00	16	11	12	23
6.00	90.00	2.00				6.00	6	8	12	20
6.65	89.35	0.65	Medium dense, yellowish brown mottled with grey silty GRAVEL, fine grained with intermediate plastic silt, moist	GM	7.10m GWL, 29/03/2015					
7.00			Very dense, yellowish brown mottled with grey, silty SAND, fine to medium grained with gravel and completly weathered rock fragments, moist (completly weathered rock)	SM		7.00	>50	-	-	>50
8.00						8.00	33	40	10/5	>50
9.00						9.00	44	36	12/5	>50
9.90 / 10.00	86.10	3.25	Borehole terminated at 9.90m depth							

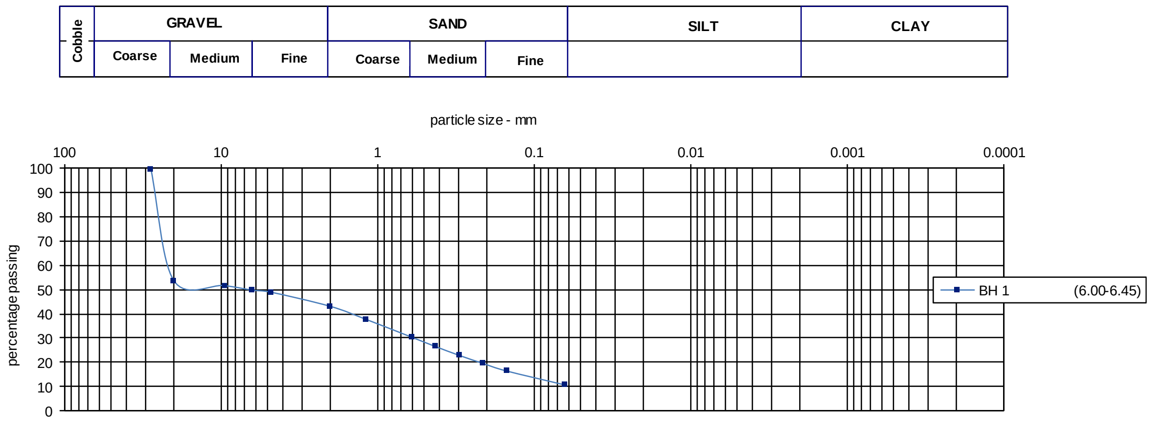
MOISTURE CONTENT - % (10 20 30 40 50 60)
UNDRAINED SHEAR STRENGTH - kN/m²
SPT RESISTANCE - Blows/30 cm (10 20 30 40 50 60)



Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By: GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By: CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date: 16/3/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By: CKJ

NATIONAL BUILDING RESEARCH ORGANISATION	Job Ref.	30/24771
GEOTECHNICAL ENGINEERING DIVISION	Client	GINIGATHHEN A
PARTICLE SIZE DISTRIBUTION CURVE - DRILLING AT GINIGATHHENA LANDSLIDE		

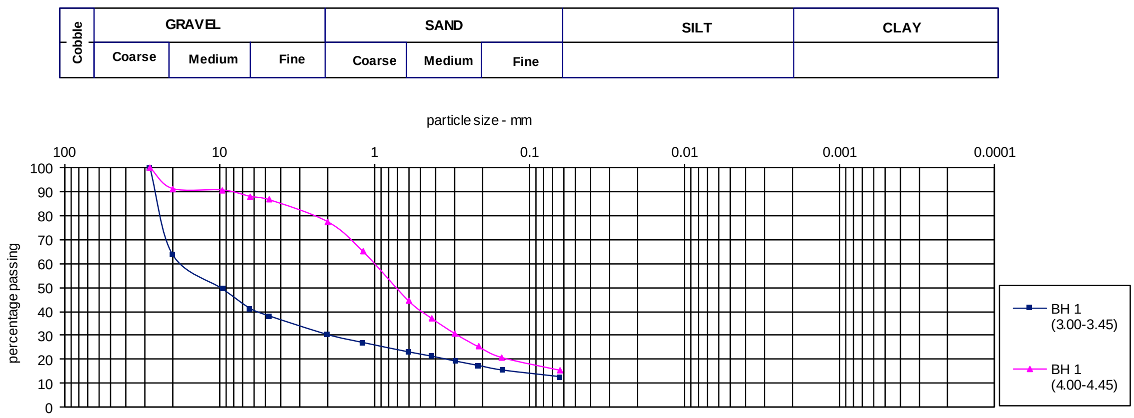
Test Method	BS 1377



Location	Sample No	Depth (m)	Classification	Liquid Limit	Plastic Limit	Plasticity Index	Cobble (%)	Gravel (%)	Sand (%)	Silt & Clay (%)	Clay (%)	Cu	Cc	Remarks
Ginigathhena	BH 1	6.00-6.45	GM					57	32	11				

NATIONAL BUILDING RESEARCH ORGANISATION	Job Ref.	30/24771
GEOTECHNICAL ENGINEERING DIVISION	Client	LANDSLIDE RESEARCH AND RISK MANAHEMEN T DIVISION
PARTICLE SIZE DISTRIBUTION CURVE - DRILLING AT GINIGATHENA LANDSLIDE		

Test Method	BS 1377

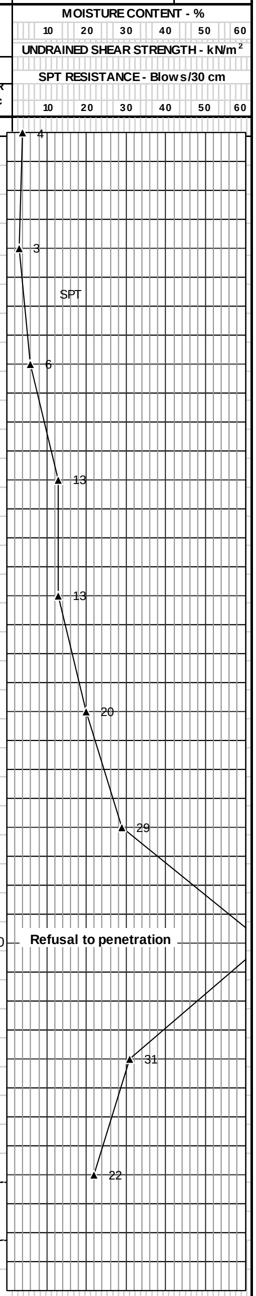


Location	Sample No	Depth (m)	Classification	Liquid Limit	Plastic Limit	Plasticity Index	Cobble (%)	Gravel (%)	Sand (%)	Silt & Clay (%)	Clay (%)	Cu	Cc	Remarks
Ginigathhena	BH 1	3.00-3.45	GM					70	17	13				
Ginigathhena	BH 1	4.00-4.45	SM					22	62	16				

BH – 02

BOREHOLE LOG	NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION 99/1, Jawatta Road, Colombo 05.			SHEET NO. 1 of 2
PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 02
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	14.90
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	109.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	13/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	16/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SOIL DESCRIPTION	STRATA	GWL	DEPTH TESTED [m]	1	2	3	FOR 30cm
0.00	#####		GROUND LEVEL			0.00	2	1	3	4
			Loose, dark brown, silty SAND, fine to medium grained with plant roots (top soil)	SM						
0.90 / 1.00	108.10	0.90				1.00	3	1	2	3
			Very loose to loose, brown, silty SAND, fine to medium grained with gravel moist	SM	1.20m GWL, 16/03/2015					
2.00						2.00	3	4	1	6
2.75	#####	1.85								
3.00			Medium dense, blackish brown, silty GRAVEL, fine to medium grained with completly weathered rock fragments, moist	GM		3.00	13	5	8	13
4.00		1.35				4.00	3	2	1	13
4.10	#####									
			Medium dense, yellowish brown, silty GRAVEL, angular to subangular, medium grained with sand and clay, moist	GM						
5.00	#####	0.90				5.00	7	8	9	20
			Medium dense, brown, silty SAND, fine to medium grained with gravel and rock fragments, moist	SM						
5.75	#####	0.75								
6.00			Medium dense, light yellow, GRAVEL, well graded, angular, coarse grained with sand moist	GW		6.00	16	12	17	29
6.60	#####	0.85								
7.00			very dense,yellowish brown, silty SAND with pockets of clay and mica, moist	SM		7.00	8	11	10/10	>50
8.00						8.00	6	15	16	31
8.35	#####	1.75								
9.00			Dense to medium dense, reddish brown to light brown, silty SAND, angular to subangular, medium to coarse grained with completly weathered rock fragments, moist	SM		9.00	8	10	12	22
						Core Depth m	Core Recovery %	RQD %		Return of Water %
9.95		1.60				9.95 / 11.30	31	Nil		-
10.00	99.05									



Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By: GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By: CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date: 16/3/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By: CKJ

BOREHOLE LOG	NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION 99/1, Jawatta Road, Colombo 05.	SHEET NO. 2 of 2

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 02
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	14.90
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	109.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	13/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	16/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SAMPLE TYPE	SAMPLE NO.	SOIL DESCRIPTION	STRATA	LEGEND	GWL	Y - [g/cm³] OTHER TESTS	Core Depth m	Core Recovery %	RQD %	Return of Water %
10.00					Moderately strong, thinly foliated, dull white, medium to coarse grained QUARTZITE with biotite gneiss inter layers, meta-sedimentary, moderately weathered, highly fractured rock	Rock							
11.00													
11.30	97.70	1.35								11.30 12.00	37	Nil	-
					Moderately strong, thinly foliated, dull white, medium to coarse grained QUARTZITE meta-sedimentary, moderately weathered, highly fractured rock	Rock							
12.00	97.00	0.70								12.00 13.60	45	Nil	-
					Moderately strong, thinly foliated, dull white, medium to coarse grained QUARTZITE meta-sedimentary, moderately weathered, highly fractured rock	Rock							
13.00													
13.60	95.40	1.60								13.60 14.90	58	Nil	-
14.00					Moderately strong, thinly foliated, dull white, medium to coarse grained QUARTZITE with biotite gneiss layers, meta-sedimentary,moderly weathered, highly fractured.	Rock							
14.90 15.00	94.10	1.30											
					Borehole terminated at 14.90m depth.								
16.00													
17.00													
18.00													
19.00													
20.00													

STANDARD PENETRATION TEST DATA — NUMBER OF BLOWS PER 15cm: 1, 2, 3; FOR 30cm

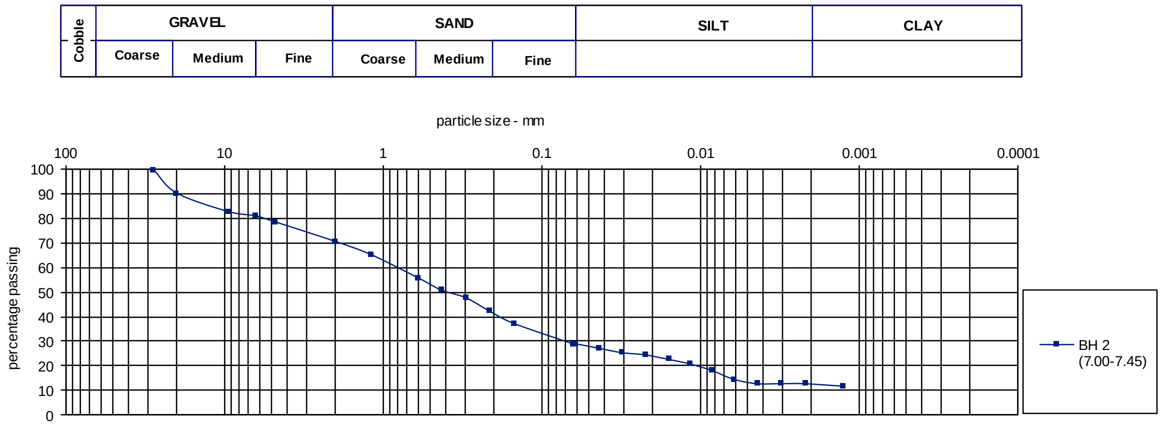
MOISTURE CONTENT - %: 10 20 30 40 50 60
UNDRAINED SHEAR STRENGTH - kN/m²
SPT RESISTANCE - Blows/30 cm: 10 20 30 40 50 60

SPT

Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By	GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By	CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date	16/3/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By	CKJ

NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION	Job Ref.	30/24771
PARTICLE SIZE DISTRIBUTION CURVE - DRILLING AT GINIGATHENA LANDSLIDE	Client	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION

Test Method	BS 1377



Location	Sample No	Depth (m)	Classification	Liquid Limit	Plastic Limit	Plasticity Index	Cobble (%)	Gravel (%)	Sand (%)	Silt & Clay (%)	Clay (%)	Cu	Cc	Remarks
Ginigathhena	BH 2	7.00-7.45	SM	37	27	10		29	42	29	13			

BH – 03

BOREHOLE LOG

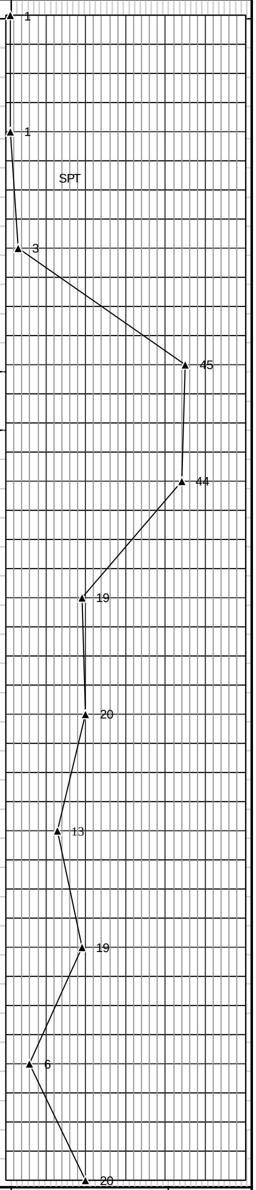
NATIONAL BUILDING RESEARCH ORGANISATION
GEOTECHNICAL ENGINEERING DIVISION
99/1, Jawatta Road, Colombo 05.

SHEET NO. 1 of 2

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 3
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	18.85
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	121.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	18/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	20/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SOIL DESCRIPTION	STRATA	GWL	DEPTH TESTED [m]	Blows 1	Blows 2	Blows 3	FOR 30cm
0.00	121.00		GROUND LEVEL			0.00	0	0	1	1
			Very loose, dark brown, silty SAND, fine to medium grained with plant roots and gravel, moist (top soil)	SM						
0.90 / 1.00	120.10	0.90				1.00	1	0	1	1
			Very loose, reddish brown, sandy SILT, low plasticity with fine to medium grained sand, moist	MS	1.20m GWL, 20/03/2015					
2.00						2.00	2	1	2	3
3.00	118.00	2.10				3.00	7	27	18	45
		0.60	Dense, yellowish brown, silty SAND, fine to medium grained with completely weathered rock fragments, moist	SM		Core Depth m	Core Recovery %	RQD %		Return of Water %
3.60	117.40					3.60 - 3.85	92	Nil		-
3.85 / 4.00	117.15	0.25	Moderately weak, thinly foliated whitish grey, fine to coarse grained, QUARTZOFELDSPATHIC GNEISS, meta-Sedimentary, highly fractured	Rock		4.00	7	21	23	44
			Rock Boulders							
			Medium dense, yellowish brown, silty SAND, fine to medium grained with completely weathered rock fragments, moist	SM						
5.00	116.00	1.15				5.00	7	9	10	19
6.00			Medium dense, dark brown to reddish brown, silty SAND, angular to subangular, fine to coarse grained with gravel, moist	SM		6.00	12	12	8	20
7.00						7.00	10	7	6	13
8.00	113.00	3.00				8.00	9	10	9	19
			Medium dense, yellowish brown mottled with grey, silty SAND, fine to coarse grained with completly weathered rock fractured (completly weathered rock) moist	SM						
9.00	112.00	1.00				9.00	5	3	3	6
			Loose, Yellowish brown, mottled with grey silty SAND, with completly weathered rock fragments, moist.(completly weathered r ock)	SM						
10.00										

MOISTURE CONTENT - % / UNDRAINED SHEAR STRENGTH - kN/m² / SPT RESISTANCE - Blows/30 cm



Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By: GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By: CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date: 13/5/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By: CKJ

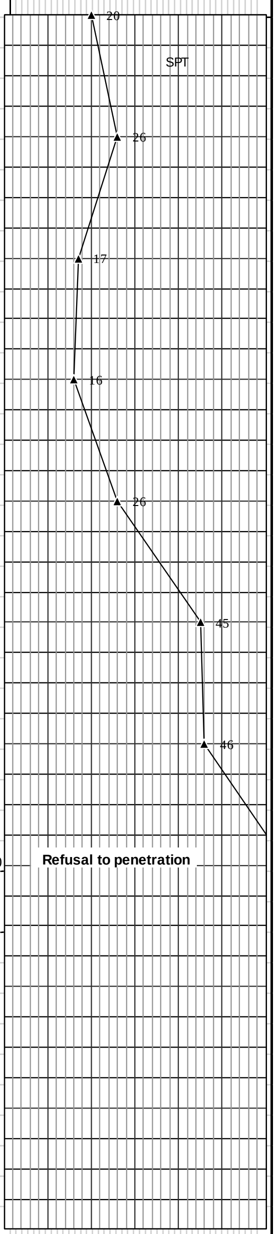
BOREHOLE LOG	NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION 99/1, Jawatta Road, Colombo 05.	SHEET NO. 2 of 2

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 3
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	18.85
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	121.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	18/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	20/03/2015

SOIL PROFILE

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SAMPLE TYPE	SAMPLE NO.	SOIL DESCRIPTION	STRATA	LEGEND	GWL	Y - [g/cm³] OTHER TESTS	DEPTH TESTED [m]	STANDARD PENETRATION TEST DATA – NUMBER OF BLOWS PER 15cm: 1	2	3	FOR 30cm
10.00	111.00	1.00								10.00	6	10	10	20
					Medium dense, yellowish brown mottled with grey sandy SILT, low plasticity with completly weathered rock fragments, moist (completly weathered rock)	MS								
11.00	110.00	1.00								11.00	16	13	13	26
12.00										12.00	10	7	8	17
13.00										13.00	7	6	10	16
					Highly to completely weathered rock (washing sample)									
14.00										14.00	9	12	14	26
15.00										15.00	19	22	23	45
16.00										16.00	19	22	24	46
17.00										17.00	30	15/5	-	>50
17.20	#####									Core Depth m	Core Recovery %	RQD %		Return of Water %
					Strong, thinly foliated, black, medium to coarse grained, BIOTITE GNEISS, meta-igneous, slightly weathered, moderately fractured rock	Rock				17.20 18.10	75	31		-
18.00														
18.10	#####	0.90								18.10 18.85	100	72		-
					Strong, thinly foliated, whitish grey, fine to coarse grained, GARNET BEARING CHARNOCKITIC GNEISS, fresh, moderately fractured rock	Rock								
18.85	102.15	0.75												
19.00														
					Borehole terminated at 18.85m depth									
20.00														

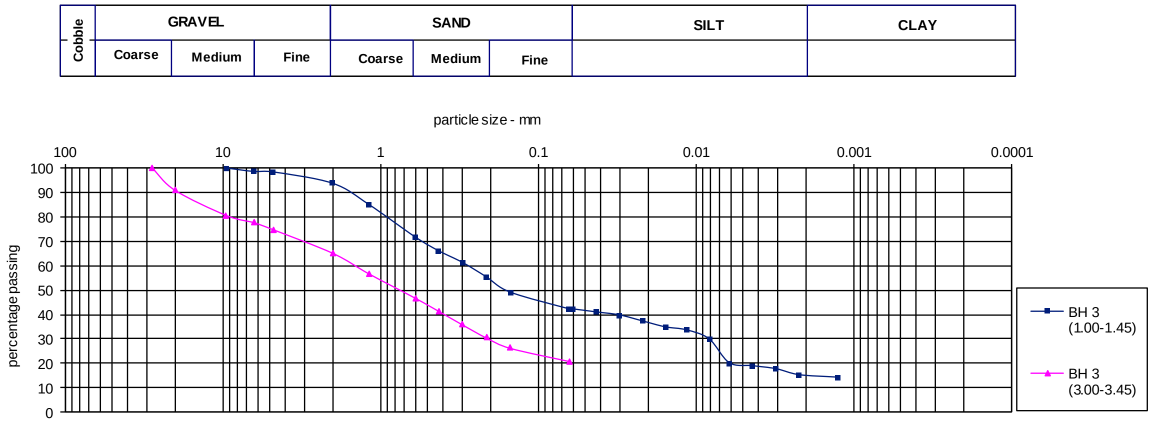
MOISTURE CONTENT - %: 10 20 30 40 50 60
UNDRAINED SHEAR STRENGTH - kN/m²
SPT RESISTANCE - Blows/30 cm: 10 20 30 40 50 60



Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample		Drilled By	GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample		Logged By	CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample		Date	13/5/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample		Checked By	CKJ

NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION PARTICLE SIZE DISTRIBUTION CURVE - DRILLING AT GINIGATHENA LANDSLIDE	Job Ref. Client	30/24771 LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION

Test Method	BS 1377



Location	Sample No	Depth (m)	Classification	Liquid Limit	Plastic Limit	Plasticity Index	Cobble (%)	Gravel (%)	Sand (%)	Silt & Clay (%)	Clay (%)	Cu	Cc	Remarks
Ginigathhena	BH 3	1.00-1.45	MS	42	32	10		6	52	42	15			
Ginigathhena	BH 3	3.00-3.45	SM					35	44	21				

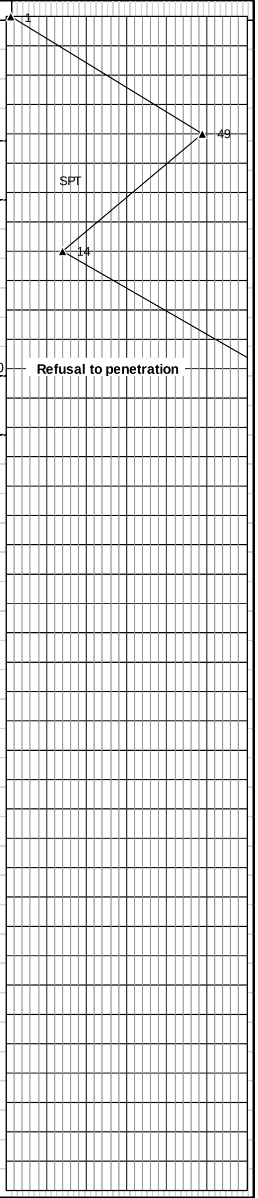
BH – 04

BOREHOLE LOG	NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION 99/1, Jawatta Road, Colombo 05.	SHEET NO. 1 of 1

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 4
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	7.65
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	98.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	21/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	23/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SAMPLE TYPE	SAMPLE NO.	SOIL DESCRIPTION	STRATA	LEGEND	GWL	Y - [g/cm³] OTHER TESTS	DEPTH TESTED [m]	SPT blows per 15cm: 1	2	3	FOR 30cm
0.00	98.00				GROUND LEVEL									
										0.00	1	0	1	1
					Very loose, yellowish brown, silty SAND, angular, medium to coarse grained with gravel, moist	SM								
1.00	97.00	1.00								1.00	12	25	24	49
					Dense, yellowish brown, silty SAND, angular, medium to coarse grained with mica, moist	SM		1.30m GWL, 23/03/2015						
1.50	96.50	0.50								Core Depth m	Core Recovery %	RQD %	Return of Water %	
1.90					Very strong, thinly foliated dark grey, fine to coarse graind, GARNET BEARING CHARNOCKITIC GNEISS, meta-igneous, fresh, slightly fractured	Rock				1.50 2.00	82	66	-	
2.00	96.10	0.40			Rock Boulders					2.00	1	5	9	14
					Medium dense to very dense, orangish brown, silty SAND, angular, fine to coarse grained with completely weathered rock fragments	SM								
3.00		1.15								3.00	17/5	-	-	>50
3.05	94.95									Core Depth m	Core Recovery %	RQD %	Return of Water %	
					Strong, thinly foliated, dull white, medium to coarse grained, QUARTZITE, meta-sedimentary moderately weathered, moderately fractured rock	Rock				3.05 4.90	13	Nil	-	
4.00														
4.90														
5.00	93.10	1.85								4.90 7.65	14	Nil	-	
6.00					Strong, thinly foliated, dull white, medium to coarse grained, QUARTZITE, meta-sedimentary moderately weathered, moderately fractured rock	Rock								
7.00														
7.65	90.35	2.75												
8.00					Borehole terminated at 7.65m depth									
9.00														
10.00														

MOISTURE CONTENT - % / UNDRAINED SHEAR STRENGTH - kN/m² / SPT RESISTANCE - Blows/30 cm



Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By: GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By: CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date: 13/5/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By: CKJ

BH – 05

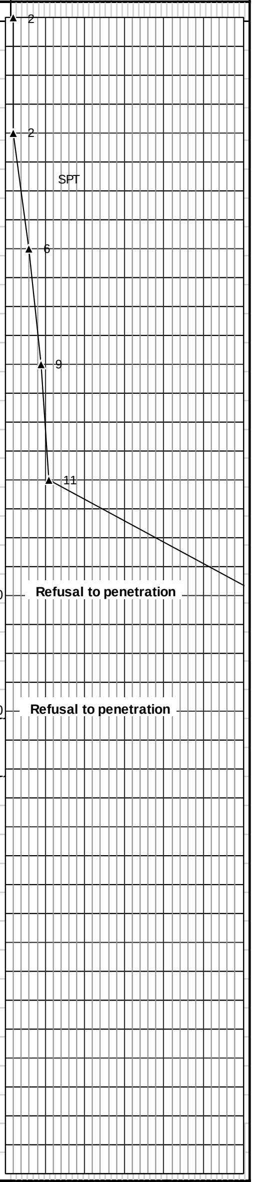
BOREHOLE LOG	NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION 99/1, Jawatta Road, Colombo 05.	SHEET NO. 1 of 2

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 5
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	11.00
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	106.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	24/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	25/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SOIL DESCRIPTION	STRATA	GWL	DEPTH TESTED [m]	1	2	3	FOR 30cm
0.00	#####		GROUND LEVEL							
			Very loose, dark brown, sandy SILT, low plasticity with plant roots and partially de composed woody material, moist	MS	1.60m GWL, 25/03/2015	0.00	1	1	1	2
0.90 / 1.00	105.10	0.90				1.00	1	1	1	2
			Very loose, orangish brown, silty SAND, fine grained with mica, very moist	SM						
2.00	#####	1.10				2.00	2	3	3	6
3.00			Loose to medium dense, yellowish brown mottled with grey, silty SAND, fine to coarse grained with completely weathered rock fragments, moist (completely weathered rock)	MS		3.00	9	5	4	9
4.00						4.00	4	5	6	11
5.00	101.00	3.00				5.00	28	HB	-	>50
6.00			Very dense, dark brown, silty SAND, fine to medium grained with mica and completely weathered rock fragments, moist (completely weathered rock)	SM		6.00	50	-	-	>50

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SOIL DESCRIPTION	STRATA	Core Depth m	Core Recovery %	RQD %	Return of Water %
6.70	99.30	1.70			6.70 8.20	20	Nil	-
7.00			Moderately strong, thinly foliated, dull white, medium to coarse grained, QUARTZITE, meta-sedimentary moderately weathered highly fractured rock	Rock				
8.00								
8.20	97.80	8.20			8.20 9.70	13	Nil	-
9.00			Moderately strong, thinly foliated, dull white, medium to coarse grained, QUARTZITE, moderately weathered, highly fractured rock	Rock				
9.70	96.30	9.70			9.70 11.00	38	Nil	-
				Rock				

MOISTURE CONTENT - % / UNDRAINED SHEAR STRENGTH - kN/m² / SPT RESISTANCE - Blows/30 cm



Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By: GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By: CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date: 13/5/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By: CKJ

BOREHOLE LOG

NATIONAL BUILDING RESEARCH ORGANISATION
GEOTECHNICAL ENGINEERING DIVISION
99/1, Jawatta Road, Colombo 05.

SHEET NO. 2 of 2

PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 5
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	11.00
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	106.00	CHAINAGE / OFFSET	-
CORE SIZE [mm] 54	CASING SIZE NX	CO-ORDINATES	N	DATE COMMENCED	24/03/2015
VANE SIZE [mm*mm] -	UDS SAMPLER SIZE [mm] -		E	DATE COMPLETED	25/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SAMPLE TYPE	SAMPLE NO.	SOIL DESCRIPTION	STRATA	LEGEND	GWL	Y - [g/cm³] OTHER TESTS	DEPTH TESTED [m]	SPT 1	SPT 2	SPT 3	FOR 30cm
10.00					Moderately strong, thinly foliated, dull white, medium to coarse grained, QUARTZITE, meta-sedimentary moderately weathered, highly fractured rock	Rock								
11.00	95.00	1.30												
					Borehole terminated at 11.00m depth.									
12.00														
13.00														
14.00														
15.00														
16.00														
17.00														
18.00														
19.00														
20.00														

STANDARD PENETRATION TEST DATA — NUMBER OF BLOWS PER 15cm

MOISTURE CONTENT - % : 10 20 30 40 50 60
UNDRAINED SHEAR STRENGTH - kN/m²
SPT RESISTANCE - Blows/30 cm : 10 20 30 40 50 60

Natural moisture content, Atterberg Limits (LL, PL)	γ –Wet unit weight	W - Wash sample	Drilled By	GES
SPT 'N', blows/ft	G -Grainsize Analysis	SPT - SPT Sample	Logged By	CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample	Date	13/5/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample	Checked By	CKJ

BH - 06

BOREHOLE LOG	NATIONAL BUILDING RESEARCH ORGANISATION GEOTECHNICAL ENGINEERING DIVISION 99/1, Jawatta Road, Colombo 05.	SHEET NO. 1 of 2

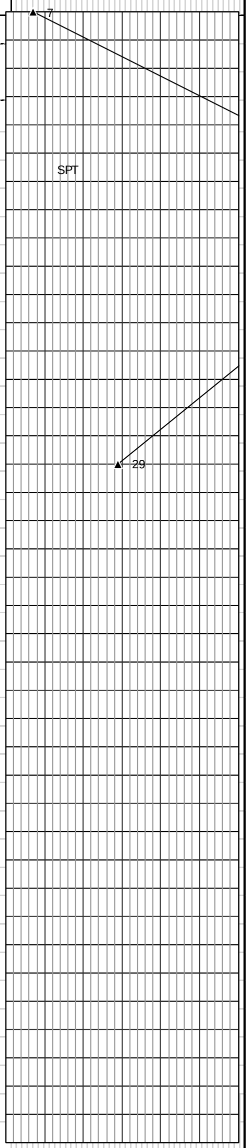
PROJECT	DRILLING AT GINIGATHENA LANDSLIDE	CLIENT	LANDSLIDE RESEARCH AND RISK MANAHEMENT DIVISION	BOREHOLE NO	BH 06
LOCATION	GINIGATHHENA	CONTRACT NO	30/24771	DEPTH OF HOLE (m)	4.50
DRILING METHOD	CORE DRILLING	ELEVATION (m RL)	103.00	CHAINAGE / OFFSET	-
CORE SIZE [mm]	54 / CASING SIZE: NX	CO-ORDINATES	N	DATE COMMENCED	26/03/2015
VANE SIZE [mm*mm]	- / UDS SAMPLER SIZE [mm]: -		E	DATE COMPLETED	28/03/2015

DEPTH [m]	ELEVATION [m RL]	LAYER THICKNESS(m)	SAMPLE TYPE	SAMPLE NO.	SOIL DESCRIPTION	STRATA	LEGEND	GWL	γ - [g/cm³] OTHER TESTS	DEPTH TESTED [m]	SPT 1	SPT 2	SPT 3	FOR 30cm
0.00	#####				GROUND LEVEL									
0.50	#####	0.50			Loose, dark brown, silty SAND, fine to medium grained with plant notes moist	SM				0.00	1	3	4	7
										Core Depth m	Core Recovery %	RQD %	Return of Water %	
1.00					Moderately strong, thinly foliated, dark grey, fine to medium grained, GARNET BEARING CHARNOCKITIC GNEISS, meta-igneous, moderately weathered, moderately fractured rock	Rock				0.50 1.50	22	Nil	-	
1.50	101.50	1.50			(Boulder)					1.50 3.50	10	Nil	-	
2.00					Moderately strong, thinly foliated, dark grey, fine to medium grained, GARNET BEARING CHARNOCKITIC GNEISS, meta-igneous, moderately weathered, moderately fractured rock	Rock								
3.00								3.92m GWL, 28/03/2015						
3.50	99.50	2.00			Boulder									
4.00					Medium dense, dark brown mottled with grey, silty SAND, angular, medium to coarse grained with gravel and completely weathered rock fragments, moist	SM				4.00	9	10	19	29
4.50	98.50	1.00												
5.00					Borehole terminated at 4.50m depth.									
6.00														
7.00														
8.00														
9.00														
10.00														

MOISTURE CONTENT - % (10 20 30 40 50 60)

UNDRAINED SHEAR STRENGTH - kN/m²

SPT RESISTANCE - Blows/30 cm (10 20 30 40 50 60)



Natural moisture content, Atterberg Limits (LL, PL)	γ – Wet unit weight	W - Wash sample		Drilled By	GES
SPT 'N', blows/ft	G - Grainsize Analysis	SPT - SPT Sample		Logged By	CKJ
Vane shear strength, peak	U - Unconfined compression	Undisturbed sample		Date	16/3/2015
Vane shear strength, residual	CU - Consolidated undrained triaxial	Disturbed Sample		Checked By	JU