EFFECT OF RECTIFICATION MEASURES IN KAHAGOLLA LANDSIDE –COMPARISON OF MONITORING AND ANALYTICAL DATA

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ABSTRACT

This research paper focuses on creep movements of rainfall-induced landslides with their groundwater level fluctuation, to understand the pore water pressure development in saturated/ unsaturated soil layers in relation to the mechanism of failure. A case study was selected at Kahagolla Sri Lanka, which is a massive creep landslide initiated around 1957 and triggering by prolonged rainfall events. The stabilization of the Kahagolla landslide was carried out under the "Landslide Disaster Protection Project" implemented by the Government of Sri Lanka with the support of Japan.

Detailed geotechnical investigation along with real-time monitoring data showed mainly four slip surfaces along the landslide axis. The main reason for movement is discovered as the rising of groundwater table and subsequent loss in the slip surface strength.

Two-dimensional analyses were carried out with several back analyses and adjusted parameters according to real-time monitoring data. Limit equilibrium analysis coupled with a seepage model was performed to confirm the actual conditions of the landslide occurrence. Thereafter, effects of rectifications were also modeled to access the stability status of the rectified landslide. The performance of the rectification measures was further examined with critical design rainfalls and a threshold for the rectified landslide. The results show an acceptable stabilization of terrain after the construction of counter measures. It can be concluded that the final combination of rectifications has been succeeded in the stabilization of this landslide and the above-mentioned approach is appropriate for use in the simulation of deep-seated landslides.

Keywords: Kahagolla Landslide, Deep seated failure, Drainage wells, Back analysis, Rectification measures

DEDICATION

This thesis is dedicated to my loving parents Mr. P.A. Thilakarathna and Mrs.B.V.R. Chandrakanthi
For their endless love, support and encouragement

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LIST OF ABBREVIATIONS

Abbreviation Description

CH –Chainage

NBRO - National Building Research Organization

SM - Silty Sand

SWCC - Soil Water Characteristic Curve

LDPP – Landslide Disaster Protection Programme

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1. CHAPTER 1: INTRODUCTION

1.1 Background

Landslide-related hazards have become a major issue causing a significant drawback in economy and loss of lives during the past few decades in Sri Lanka. Since 1980, the number of landslide events has been surprisingly increased due to human involvement with uncontrolled land developments and the number of victims has been raised by thousands. The damage assessment of the recent landslide events during the past five years has indicated more than 500 deaths and 30,000 houses damaged in 10 districts around the country. Population growth and scarcity of lands are the main reasons behind the unavoidable settlements in these fragile lands which increased vulnerability towards the community. Therefore, today several government bodies focus on disaster prevention programs through structural and non-structural mitigation works. Hazard mapping, early warning, and landslide forecasting through continuous monitoring have been lately updated with modern technology, but in advance, the Sri Lankan government has moved towards applying engineering solutions for landslide mitigation as a long-term strategy.

Most of the landslides in mountainous regions are triggered by prolonged or heavy rainfall, which infiltrates into the soil and alters the pore water pressure within the slope. Changes in land use pattern has also resulted in increasing the infiltration into the lands. The influence of these factors over a long period has caused to create massive landslides around Sri Lanka, which are very challenging to mitigate.

Kahagolla landslide is such a well-known massive landslide in Sri Lankan landslide history. It is located at the 10km post of A016 road in Badulla district. Records show that it was initiated around 1957 and movements have shown periodically at a slow rate. It sporadically damages Haputale-Bandarawela road and pushes it around at the location. It has extended over 4.5 hectares in Kahagolla tea estate with a length and a width of 300m and 150m respectively. Continuing creep movement of this soil mass is still threatening to many lives and properties located just below the landslide area.

Aiming at reducing the risks associated with landslide disasters in the national road network in highland areas, the "Landslide Disaster Protection Programme (LDPP)" was initiated in 2014 by the Ministry of highways under the funding of the Japan International Cooperation Agency (JICA). Accordingly, Kahagolla Landslide was also selected for mitigation with the latest technologies that are used worldwide. The following countermeasures were adopted after a detailed investigation.

1. Earth removal works
2. Counterweight embankment work
3. Subsurface drainage and deep drainage well development
4. Surface drainage development
5. Ground anchor works
6. Road widening with light weight embankment

1.2 Problem Identification

The landslide mass mostly consists of colluvium soils in the form of silty sand and sandy silt mixture up to a considerable depth. The low permeable characteristic of soil layers and their layering nature maintain the groundwater level at considerably high elevation throughout the year. During rainy seasons, the groundwater level rises almost to the surface level, triggering the movements of unstable soil masses. Therefore, surface and subsurface drainage systems would play a significant role in the mitigation.

Although an initial design was implemented after many types of research and detailed investigations of subsurface soil layers, later it was discovered that a rock layer underneath, creates an obstacle for constructing subsurface drainage systems as planned earlier. Therefore, the subsurface drainage system was located away from the main axis of the landslide.

As such, one of the main purposes of this research is to model the process closely using actual site-specific geological and hydrological data obtained through recent investigations and monitoring. This data helps to assess the stability of mitigated landslide during a heavy prolonged rainy period.

1.3 Objectives

Objectives of this study are,

1. To study the physical mechanism of landslide induced by infiltration of rainfall and changes to the pore pressure regime will be studied through Geoslope SEEP/W software. The result will be incorporated into the stability analysis by SLOPE/W
2. To study the effectiveness of countermeasures applied for the landslide rectification
3. To study the response of the rectified slope to critical design rainfalls and finding the threshold limit for the landslide

1.4 Methodology Applied

1. The infiltration of rainfall into the slope was modelled for actual prolonged rainfall event which occurred after a long dry season. Accordingly, the initial ground water table corresponding to a dry season was obtained through the monitoring data. In the absence of actual experiment results for Soil-Water Characteristic curve and permeability function, these parameters were changed within an acceptable range, until achieving the observed ground water fluctuation, through a process of back analysis.

   Using this process, Soil-Water Characteristic curves and the permeability functions of different soil layers were established.

2. A back analysis of stability was carried out using the critical water level after the rainfall, and varying the shear strength parameters within an acceptable range to obtain the observed failure surface.
3. The effect of proposed rectification measures on the lowering of ground water table and hence enhancement of FOS was analyzed. In this study a special technique of permeable zone was used to model the inter-connected drainage wells. Consequently, the improvement of safety margin with applied countermeasures was obtained.

4. A critical design rainfall was applied on the rectified slope to understand its response. This is to determine whether the rectified landslide could withstand a heavy rainfall without reaching critical state of stability.

1.5 Thesis Outline

The outline of the thesis is as follow.

Chapter 2 of this thesis reviews the available literature related to landslide triggering factors for deep-seated failures considering specific features that are relevant to the terrain of the case study. Rectification measures, assessment of stability with appropriate methods, and similar case studies are also reviewed for a suitable approach to the analysis.

Chapter 3 presents the history of occurrence of Kahagolla Landslide and presents the design of countermeasures. Some important observations made during the monitoring stage are also described here.

Chapter 4 describes the approach used for the initial design of rectifications and detail of countermeasures. Some design changes carried out during the construction works are also reported in this chapter.

Chapter 5 presents the model of seepage analysis and incorporated stability analysis for the worst-case condition of the landslide. The procedure followed for back analysis and verification process of two models are discussed in this chapter.

Chapter 6 discusses the model with rectification measures and strategies followed to model them according to site conditions. Some back analysis and verifications that were carried out for this model and factor of safety values obtained for each rectification measures are also described in this chapter.

Chapter 7 presents the response of the rectified landslide for a critical design rainfall event and assess that efficiency of the countermeasures.

Chapter 8 summarizes the findings and makes suggestions for the research.

2. CHAPTER 2: LITERATURE REVIEW

According to USGS (United State Geological Society), Landslide is defined as any type of downward movement of the ground including soil, debris, rocks under major influence of gravity. This term can be used on following modes of failure; falls, topples, slides, spreads, flows. In this chapter, Literature review provides an insight into deep-seated landslides starting from their triggering factors to effects of rectification measures.

2.1 Introduction to Rainfall Induced, Deep-seated Landslides

Deep-seated landslides are often large-scale, slow-moving mass on the rock surface which is having a depth of greater than 5m with a limited runout distance. Usually they occur as translational slides, rotational slides, or large block slides. Slides can be formed as a result of changing hydrological or geological process in the area such as increase of ground water level or earth shattering. After formation, it can persist from few years to centuries. (Lin & Chen, 2020).

Even though every rainfall directly does not trigger deep-seated landslides, many case studies evidence that they normally activate during heavy rainfall situations. In the study of deep-seated landslides, it is also worth exploring regional rainfall pattern, mechanism of rainfall infiltration as the rain process and the rate of infiltration into the ground. The infiltration process of rain water in to the soil is mostly influenced by both gravity and capillary forces which acts toward vertical direction to cause percolation downward. Capillary forces can also act in lateral direction and divert water from the large pores to capillary pore spaces which are in much smaller in dimensions. As this process continues, soil layers become saturated with percolated water at greater depths and groundwater encounters with increased resistance to flow due to reduced extent or dimension of flow channels, increased length of channels, or an impermeable barrier such as rock or clay (Gray & Norum, 1967). Therefore, as rainfall continues over a long period of time, the groundwater table rises inside the soil mass, until it triggers a failure.

2.2 Rainfall Pattern of the Terrain of Case Study

Landslides are more frequent in tropical countries, where prolonged or intense rainfall situation occur in an environment with steep slopes, sparse vegetation and incoherent fine grained soils including colluvium and residual soils. Consistent with the worldwide rainfall trigged landslides catalogue published by NASA, most landslides occur during the northern hemisphere summer coinciding with the tropical cyclone and East Asian monsoon seasons. In Sri Lanka, landslides are more often during Southwest monsoon rainfall, which cumulative amount varies from 100 mm to 3000 mm during May-September. Recent researches carried out in Sri Lanka indicated that, the central highland districts are more susceptible for landslide hazards with more than 15-20 days' antecedent rainfall (lower intensities over long time period), while the south western are susceptible with less than 3-5 days' antecedent rainfall (high intensities over short time period). This study also proved that not only the daily rainfall affects for the initiation of landslide but also the antecedent rainfalls influence landslides (Kumara et al., 2018).

The case study area locates in the Badulla district, which is a very famous deep-seated landslide-prone area of the central highland of the country. In 2014, more than 200 people got killed by a major landslide that occurred in this terrain. Research carried out for landslides especially in this terrain, reveals that the rainfall threshold predicted for landslides in Badulla district is considerably greater than the thresholds predicted for the whole country (Nawagamuwa & Perera, 2017). This implies that a Landslide in this terrain would need a high amount of rainfall to get activated.

Another research carried out for rainfall in Badulla district revealed that short-duration rainfall events and high rainfall intensities are required to trigger debris or mud flow, while long-duration rainfall events with low rainfall intensity can trigger both landslide and debris flow. The analysis also discovered that most mass movements (83%) occurred within 5-6 hour of peak rainfall (Perera ,Jayawardana & Jayasinghe, 2017). But the important finding of these rainfall analysis is terrain rainfall pattern has showed increasing trends within last three decades. The annual rainfall trend of the Badulla district as calculated from 1999 to 2018 is 15.8 mm/year which is a prominent rise from the global precipitation rate. The maximum annual rainfall was observed in 2001 as 2525.1mm, while the minimum observed in 2010 as 1034.7 mm. Northeast-Monsoon Season (December-February) shows highest seasonal mean value of 583.2 mm with an increasing trend (Ruwangika et al., 2019). Therefore, the risk of failure would increase at future under the extreme weather conditions. In design perspective, designers have to be more cautious with antecedent rainfalls having ascending pattern when the selecting of counter measures.

2.3 Mechanism of the Failure and Its Triggering Factors

Numerous research studies were carried out to identify the possible failure mechanism of Kahagolla landslide since 1989, and it was estimated that the main trigger of the occurrence of the landslide is the supply of underground water. The pore pressure which acts on the moving mass is increased by a continuous supply of groundwater thereby creating an uplift pressure at the slip surface. A GPR survey was conducted in 2016 and detected groundwater table at a shallow depth from the ground surface (Abeysinghe et al., 2017).

However, Researchers were not able to find a possible way of recharging the groundwater table of the area. In 1989, it was suspected that there might be a contribution of groundwater supply from an impounding reservoir, located at the opposite side of the mountain. This reservoir locates at 1500 MSL elevation and the dip direction of underlying rock masses also points towards the landslide. But the investigations carried out so far are inadequate to determine whether a marble layer has formed a subterranean connection between this reservoir and the Kahagolla earth slip. (Bandara et al., 2002). Figure 2.1 illustrates the cross section of the area indicating possibility of water seepages from the reservoir to landslide.

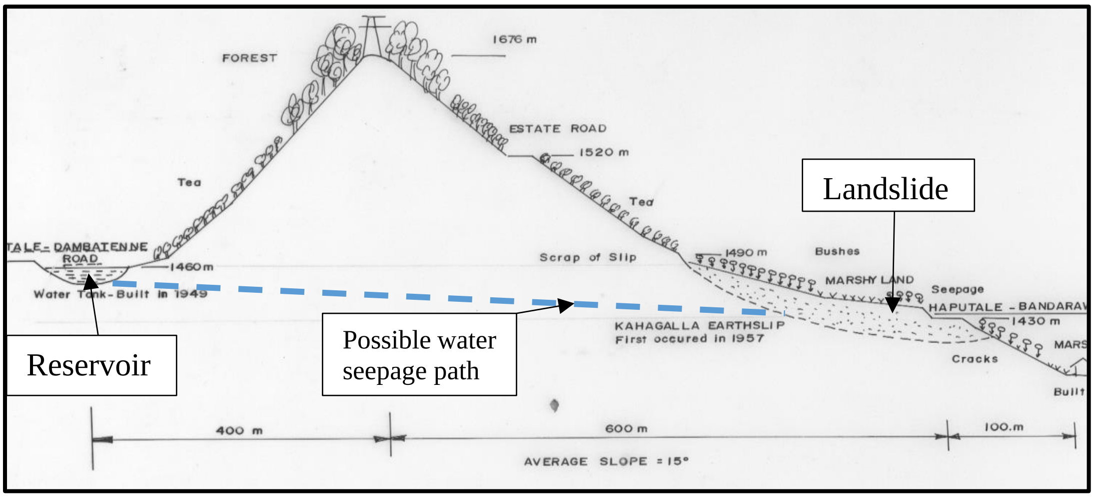


Figure 2.1: Sketch of the cross section and the area affected by the Kahagolla landslide Source: (Engineering Geological report of Kahagolla Landslide and Mitigation of Its Impacts,2002)

2.4 Ground Water Recharging by Rainwater Infiltration

Various studies have been carried out to forecast the groundwater level fluctuations in rainfall-induced landslides worldwide. Some of them attempted to characterize the groundwater system through numerical models. These analyses reveal that most of the deep-seated landslides are triggered by positive pore water pressure developed due to the rising of the groundwater table (Van,Buma & Van, 1999). When the groundwater level reaches 4m below the ground surface, most landslides get reactivated (Caris & Van, 1991). It was found that a soil profile with a shallow water table quickly responds to rainfall showing a close relationship between precipitation and groundwater recharging (Wu, Zhang & Yang, 1996). In 2009, it was discovered that cumulative precipitation is equal or greater than potential recharge and amount of evaporation together, thereby making the infiltration one of the dominant factors contributing to the groundwater level fluctuation (Schwartz & Schreiber, 2009).

In 2019, a study was carried out for landslides located at Nawalapitiya. This landslide is located in the highland complex, which have similar geological conditions as Kahagolla, and both are deep-seated failures initiated by the rising of the groundwater table. The rainfall infiltration in to the landslide was modelled as a multi-tank system as illustrates in figure 2.2.

It was found that the occurrence of heavy rainfall is associated with significant variation in the groundwater level. This model was successful in predicting fluctuation of groundwater level with a reasonable accuracy (Gunathilake et al., 2019). Therefore, it can be concluded that the terrain groundwater level is be mainly influenced by the rainwater infiltration.

Unfortunately, multi tank system model cannot be used for predicting groundwater level after the construction of rectifications, especially for the control works of groundwater drainage. Therefore, the next intention was finding case studies relevant to quantifying the effectiveness of ground water control works.

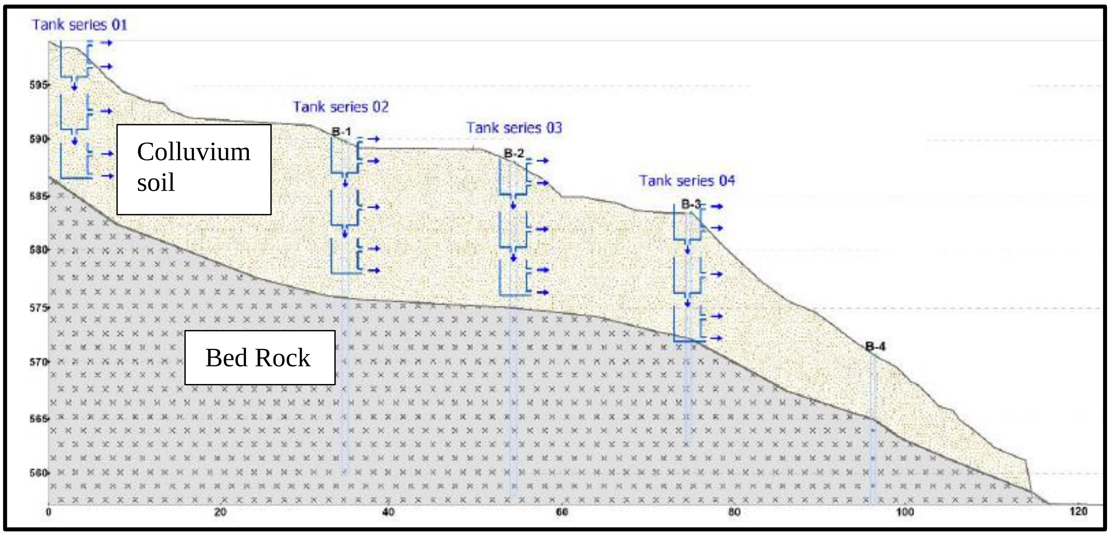


Figure 2.2: A Multi tank system modelled for Nawalapitiya Landslide

Source: (Gunathilake, Bandara & Weerasinghe, 2019)

2.5 Landslide Rectification Measures

Prior to the detail study of the ground water control works, it is worth exploring all types of countermeasures briefly with their efficiency as mitigation measures.

According to Japanese design manuals, landslide rectification works can be divided into two main groups; control works and Resistant works. Landslide control works mainly involve modification of existing features such as topography, groundwater, and other conditions that indirectly control the landslides' movements. Resistant works such as Piles and shafts, ground anchors, soil nails directly involve in preventing mass movement as resistant forces.

2.5.1 Landslide Control Works

These methods are very effective and inexpensive for most cases and commonly used in mitigation works. Surface drainage measures are constructed to control landslide movements by minimizing rainfall infiltration into the slope. Surface drainage measures are designed to fulfil following functions; drainage collection and channel works to remove water collected from the landslide at a fast rate. These are often combined with subsurface drains.

Horizontal drainage measure involves drilling a series of horizontal borings directed towards the slip surface, to reduce the groundwater level effectively. A small drilling angle is maintained to continue the flow under gravity. This measure is very popular and commonly used due to its safeness, inexpensive, and effective usage but difficult at the maintenance.

Drainage well, although not commonly used is an efficient countermeasure for deep-seated failures. Wells are constructed with a set of radially placed horizontal drains inside of the well. This measure often reduces the length of horizontal drains normally required as a rectification measure alone and significantly improves the effectiveness. Drainage wells can be constructed to work under both gravity and pumping. Drainage tunnels are also a desirable option, but extremely expensive method compared to other controlling measures.

Earth removal work in the upper landslide body entered the design and practice of landslide rectifications lately, which can increase the factor of safety margin by reducing the driving force of unstable mass. Recently, earth removal works are carried out together with the counterweight embankment method, which can be used the same removal mass at the toe area as a compacted fill (Japaneese landslide society, 2002).

2.5.2 Landslide Resistant Works

Typical landslide Resistant measures are steel piles, concrete shafts, ground anchors, and soil nails, which are efficient in improving stability, but expensive options. Concrete piles are used in large projects, where the control works are not satisfactorily achieved a safety margin. Ground anchors are frequently used in the rectifications of landslides, which use the tensile forces as a resistant force.

2.6 Effects of Drainage Control and Other Measures in Stabilizing Landslides

Watawala is the most well-known rectified landslide, Sri Lanka in 1993, which is mainly based on drainage control works to stabilize the massive landslide mass. Movements had been periodically shown in this landslide starting from 1980, disrupting the Badulla-Colombo railway line. The landslide is well known for the applications of directional drilling techniques first time for landslides stabilization as well.

This landslide locates in Nuwara-Eliya District, which has a highly fractured and faulted metamorphic rock as a basement at the location. This basement consists of metamorphic rocks such as quartzite, marble, charnockitic gneiss. However, limestone has also been found at two boreholes locations at 66 m depth. The geomorphology of this area shows a V-shaped valley and the whole landslide mass consists of thick sandy silt and clayey silt colluvium layer. The thickness of the colluvium layer gradually increases from top to bottom. Figure 2.3 illustrates the geological profile of the landslide.

The ground shows the signs of an artesian aquifer at the top of the colluvium layer and the water table is highly sensitive to rainfall events. Test results reported high permeability properties in soil samples indicating ground water control works are more effective in stabilizing the landslide.

According to the literature, six wells were installed in the landslide body, with 200 mm diameter and an approximate depth of 60m. The unstable ground was supported with 150 mm steel casings and electric submersible pumps were installed in each well, so the whole system is supposed to work under pumping.

The computerized directional drilling technique, which is a successfully adopted from oil well technology, was used to install the horizontal drains up to 600m distance. Eleven horizontal drains were constructed with a diameter of 130mm at the bottom of the landslide through the colluvium layer. The slip surface scrap, near the railway track was stabilized using the soil nail technique. In additionally surface drainage, surface regarding and re-vegetation measures also implemented to prevent rain water infiltration. The expected drawdown of the water table was 5m from the highest GWL (Chandler,Broise & Partners, 2000). Figure 2.4 and 2.5 illustrate the rectification works carried out for Watawala earth slide.

After the construction of rectification measures, monitoring equipment was installed at the site to measure post-construction performance. Accordingly, manual piezometers were installed in 16 different locations, and 60 survey points were established to detect the landslide movements. A relationship between rainfall and total discharge was discovered for rainfall that occurred in 1995, showing a prominent success of the groundwater control work. The following conclusions were made after observing the monitoring data.

The system quickly responses to the heavy rainfall nearly after 24 hours occurring of maximum rainfall, showing that subsurface drains are well-arranged within the landslide. It also indicates that the colluvium layer is more permeable compared to the value obtained from the laboratory permeability test. The discharge from the subsurface drains is also sensitive to isolated storms but shows a noticeable response to heavy rainfall occurred in successive days.

The total discharge from subsurface drains is owned by 63% from the precipitation amount on the landslide area, which is quite high, thus flow accumulation from the surrounding strata is suspected. This verifies by the artesian aquifer located at the upper slope.

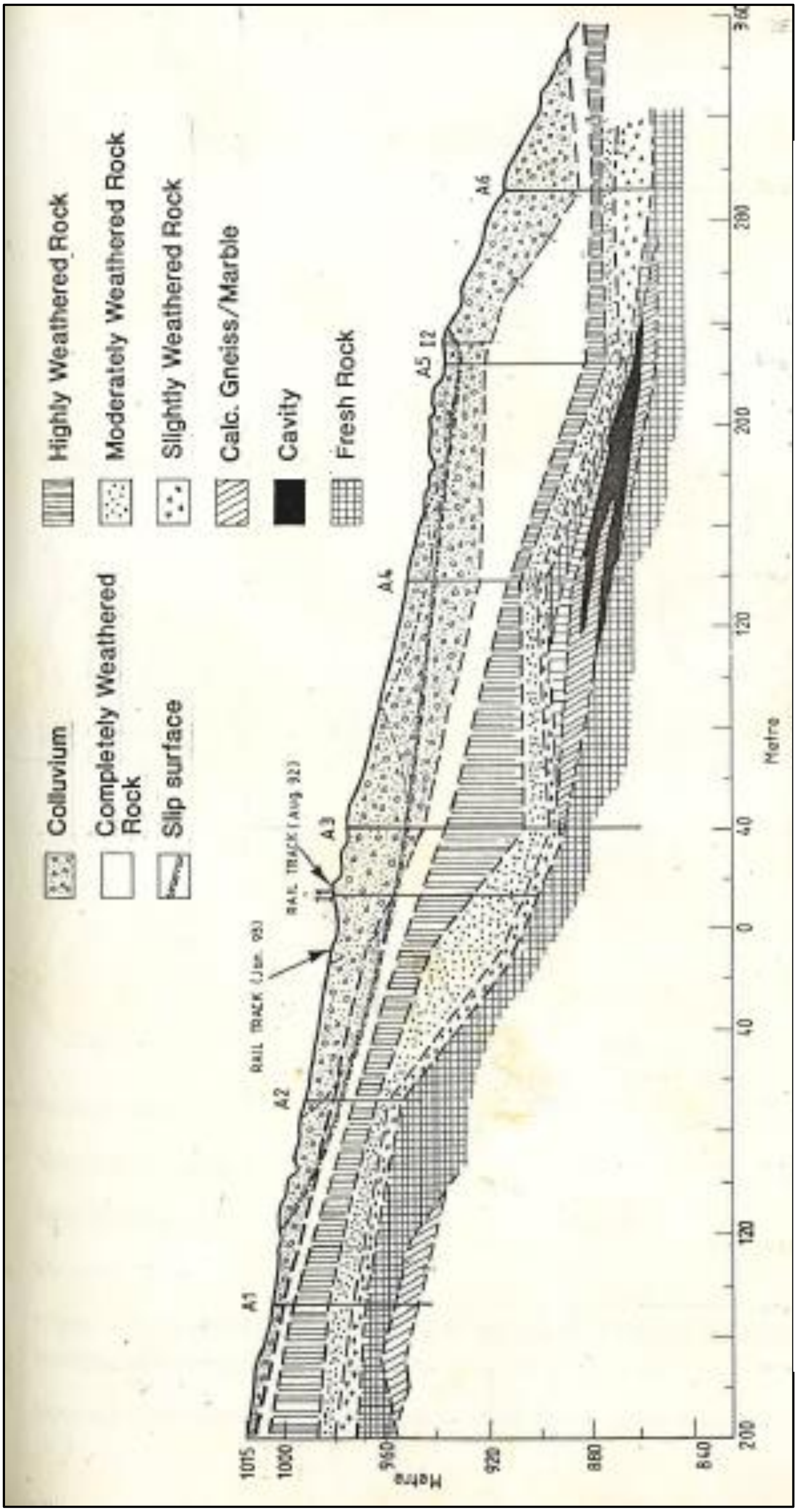


Figure 2.3: Longitudinal cross section of Watawala landslide with subsurface soil/rock profile and borehole locations

Source: (Chandler,Broise & Partners , 2000)

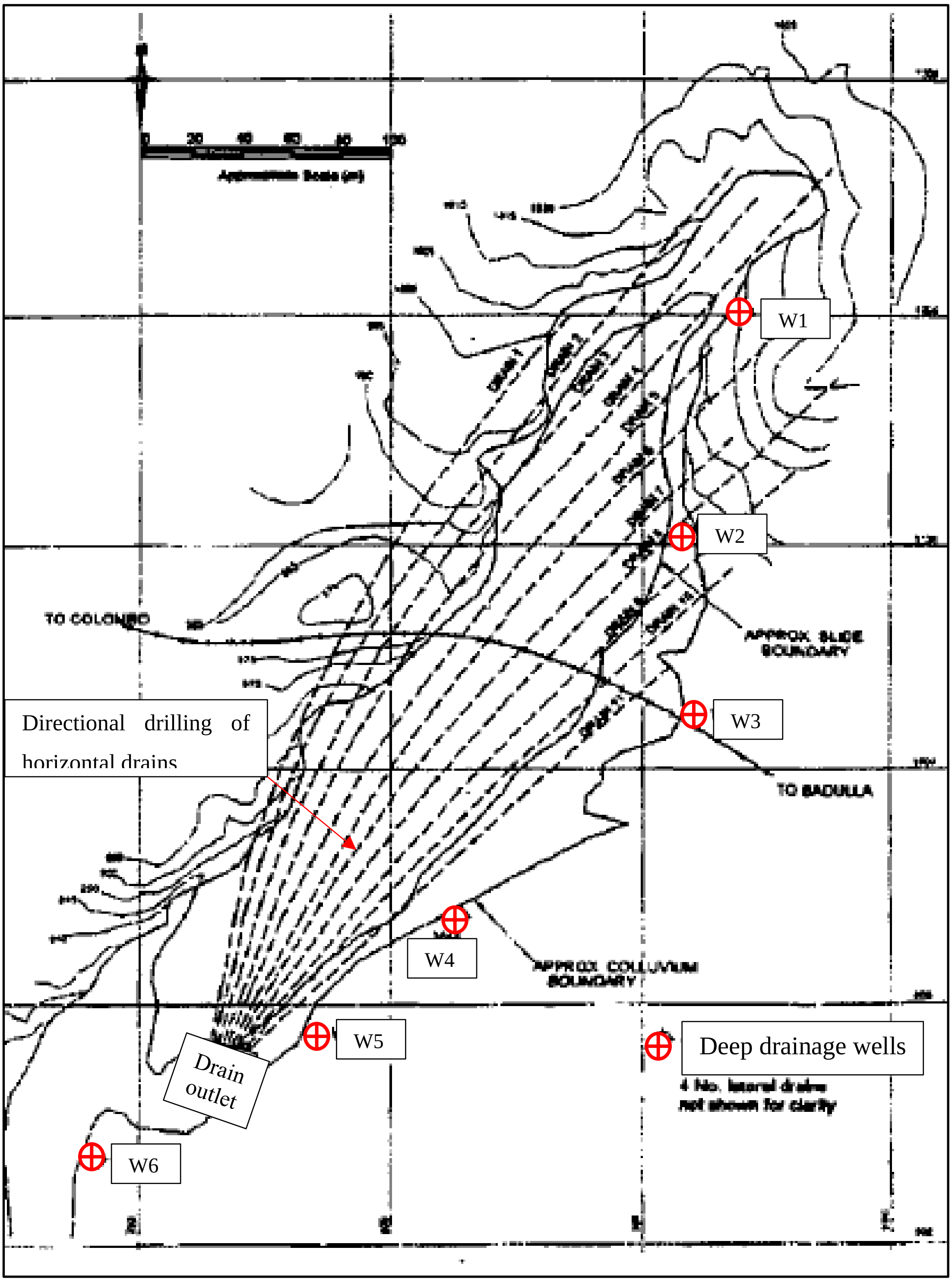


Figure 2.4: Plan View of Rectification Measures in Watawala Landslide

Source: (The Watawala Earthslide;Investigations and Diagnostics, 1993)

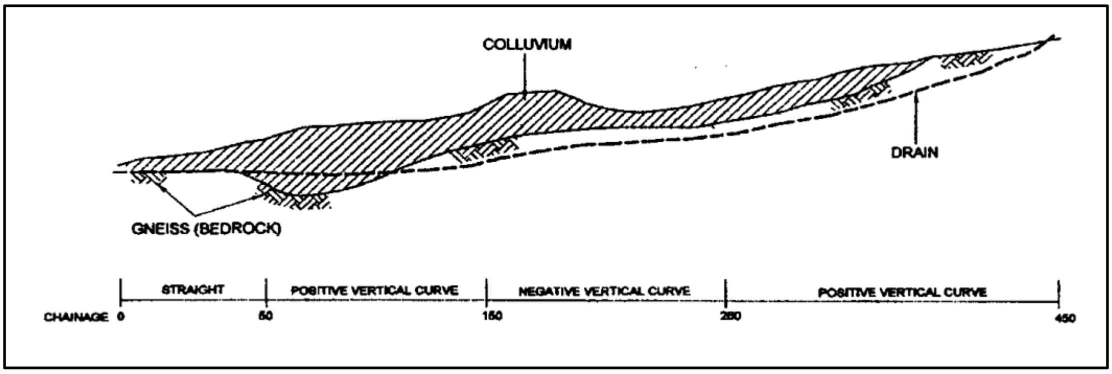


Figure 2.5 :Typical underdrain profile

Source: (Chandler,Broise & Partners, 2000)

2D and 3D slope analysis had been performed for the post-construction stage as well for the reduced groundwater table (-5m). The shear strength parameters have been used for the colluvium as c’=5 kN/m^2 and ϕ’=18^0 which are very lower values than the results obtained through the laboratory tests.

However, the authors have mentioned that the values were derived after further investigation of failure mechanism at Watawala landslide. A high factor of safety (more than 1.5) was obtained indicating a stabilized situation in the landslide area.

Unfortunately, monitoring data of the post-construction stage is not available to quantify the drawdown amount of water table due to drainage works. And also, there was no record of any verification activity to find whether the predicted drawdown of water table had been achieved or not.

It is a great loss that the data of monitoring for such mitigation works and data about the fluctuation of ground water level after rectifying the landslide are not available. Records and analysis with continuous monitoring is essential in these kinds of mitigations specially which can be vulnerable again with a system failure. Even though many drainages well construction projects of rectifying large-scale landslides of Sri Lanka have been carried out, the efficiency in drawdown of water table has not been studied thouhgrouhly.

Unfortunately, a similar method has been followed at the design stage of Kahagolla landslide mitigation, without a proper analysis of achieving forecasted the average drawdown of water table at a prolonged rainy period. Instead of that, designers presented some empirical values according to their experience gained at the monitoring stages of previous cases as illustrates in table 2.1 and figure 2.6.

Table 2.1: Expected Drawdown of Water Table for Control Works

Source: (Oriantal Consultants, 2016)

Type of ground water control work	Maximum drawdown of water table
Horizontal boring	3m
Drainage well	5m
Drainage tunnel	8m

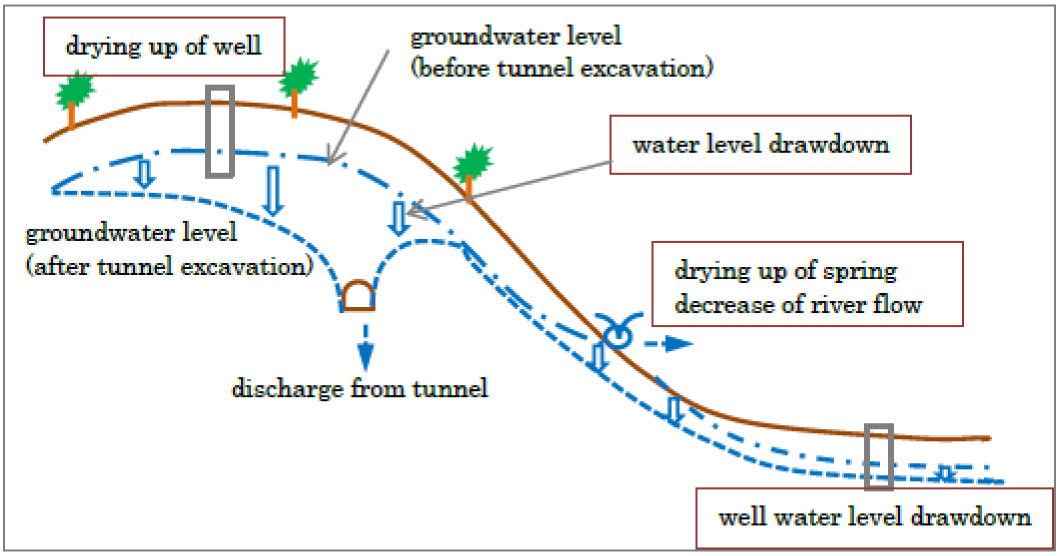


Figure 2.6: Typical Drawdown pattern of Water Table after Construction of Rectification Works

Source: (Guideline for Environmental Impact Study -Ground Water, 2018)

It should be noted that these values can be varied with Locations of counter measures, morphology of the area, geology, and supply of groundwater etc. The level of drawdown at a short period was expected as 2m which is half of the value for quantity of ground water level reduction for both horizontal boring and drainage wells (Oriental consultants, 2016).

2.7 Landslides Modelling with Seepage Models

A successful attempt of modelling seepage through drainage wells were found at a case study of Li-Shan landslide in Taiwan. Li Shan landslide is a deep-seated landslide that has a varying sliding depth of 30m to 70m. The annual rainfall recorded is approximately 2242 mm and maximum monthly rainfall recorded at the site is 514 mm. The landside is located near the Li shan fault where the tectonic activities still present in the area. Geology of this area has been categorized to Miocene Lu-shan formation which is highly fragmented metamorphic rock basement. The site includes high graded "slate" rock type while the mudstones present in the colluvium layer. Both colluvium and the highly weathered rock layers are subjected to slow movements along the valley.

In 1990, sudden and massive collapse had been reported in this landslide due to a torrential rainfall occurred over 5 successive days, damaging to a highway in Taiwan. In 1995, construction of remedial measures was started after a proper detail investigation and the design stage. Accordingly, counter measures had been selected including drainage galleries, drainage wells, subsurface drains, submerged dams and check dams. Figure 2.7 illustrates the plan view of rectification measures constructed at Li-Shan landslide.

Fifteen drainage wells of diameter 3.5m were driven into 15 to 40m depth. Series of horizontal drains in length of 40 m-70 m had been installed at 3 different elevations in each drainage well. Addition to that two drainage tunnels with sub vertical horizontal drains were constructed in the stable fresh rock at 80m depth, which is not influenced by the landslide movement.

In 2008, after the completion of the project, researchers had performed both 2D and 3D analysis together with monitoring data and found a quick computation method to evaluate the effectiveness and efficiency of a subsurface drainage system. Analysis had been carried out using Geoslope software to model transient seepage at two typhoon events and succeeded in modelling as actual site conditions. The model was verified with the monitoring data collected at the site before and after the construction of remedial measures. The factor of safety was also found coupling with the Geoslope SLOPE/W analysis and discovered that the applied drainage control system is very efficient in stabilizing large scale of landslides. This analysis also revealed the ability of modelling the effect of countermeasures specially the ground water control systems with a seepage model (Lin et al., 2001)

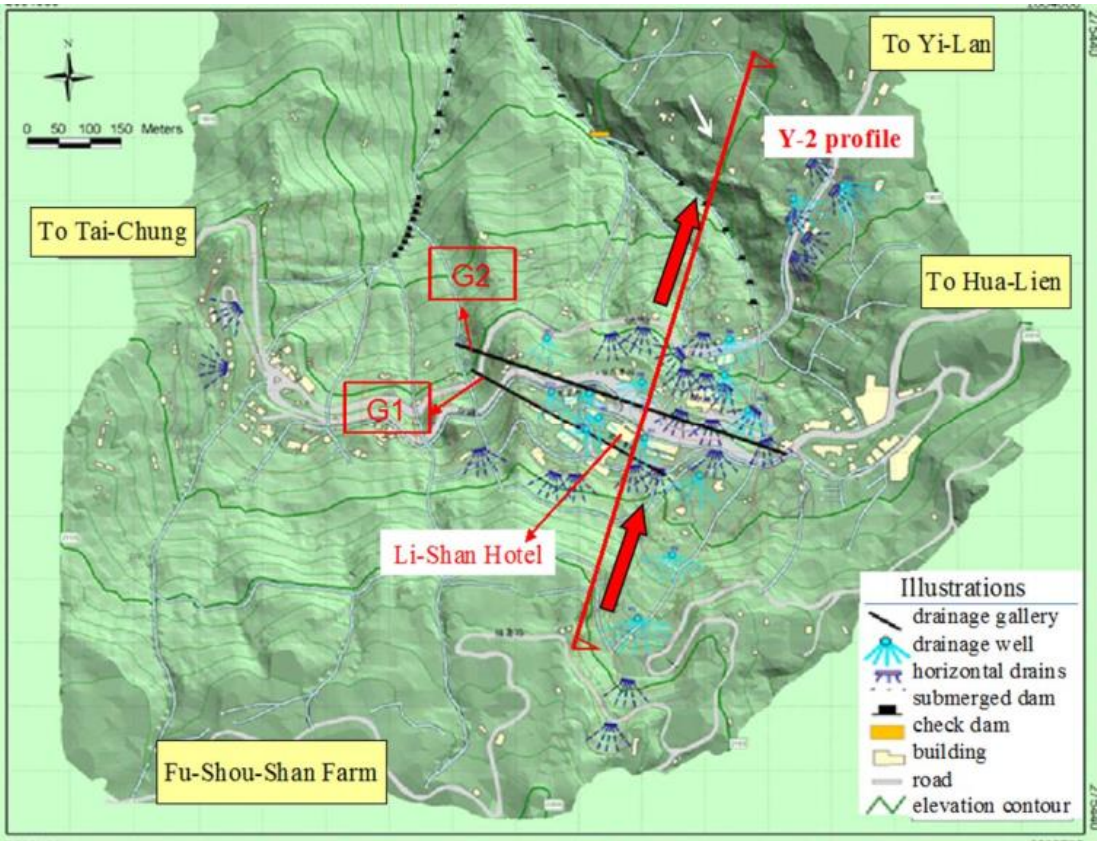


Figure 2.7: Plan view of Subsurface drainage system at Li-Shan Landslide at Taiwan Source: (Lin et al., 2001)

2.8 Material and hydraulic properties applying to the seepage models

Lately, seepage analysis which has an ability of modelling transient stages with varying boundary conditions and time is widely used in large scale slope stability projects. It also has the capability of modeling pore pressure and subsequent matric suction in unsaturated soil regions. However, the data required for these types of analysis are very complex in nature, when the attempts are made with laboratory experiments for hydraulic properties of soils. As an example, in order to model unsaturated soil regions in seepage analysis, two functions are required; hydraulic conductivity and Soil-Water characteristic curve. The methods used to determine the hydraulic conductivity function have been thoroughly studied by many researchers in both empirical and experimental ways. Some of them were succeeded in deriving two functions close to actual curves and gave reliable results as compared to laboratory tests. Hydraulic conductivity function derived from volumetric water content function with regard to negative pore water pressure distribution (or matric suction) is such an appropriate method widely used in stability analysis software. Because the hydraulic conductivity is a function of water content and indirectly is a function of pore water pressure, both curves share similar fundamental shapes and features.

In the case study of Li Shan Landslide all the SWCC curves of each soil layers have been derived using the method proposed by Fredlund and Xing (1994). This method is based on volumetric water content and saturated water content etc. The hydraulic conductivity function had been derived using the SWCC and the saturated permeability obtained through the field tests. Material properties such as friction angle and the cohesion had been determined through the direct shear tests (Lin et al., 2001).

2.9 Boundary Conditions applying to the seepage models

Applying of boundary conditions plays a fundamental role in geotechnical modelling process since it defines the problem and lead the model to achieve realistic solutions for the analysis. The case study carried out for Li-Shan landslide in Taiwan, describes the important of defining the model with correct type of boundary conditions which represent the actual site circumstances. In this case study, the horizontal drains had been modelled as line type of potential free seepage face (total flux = 0 with potential seepage) to represent actual site conditions. It further describes that; zero pressure condition (h_p =0) gives unrealistic results when the horizontal drain locates below the ground water table and the pressure inside the horizontal drain eventually rises as it fills with water. Therefore, zero pressure condition may not exist inside the horizontal drains. Instead of that, “potential free seepage face with zero total flux” boundary condition would give varying pressure and flow conditions inside the horizontal drain. (Lin et al., 2001).

However, this boundary condition removes the water from the system at the applied location. Therefore, applying of boundary conditions available in some software are a bit suspicious when it compares with actual site conditions, which would rather persuade the user to model with a different technique or strategy. Above figure 2.8 represents the boundary conditions applied for a model of rectified landslide in Taiwan.

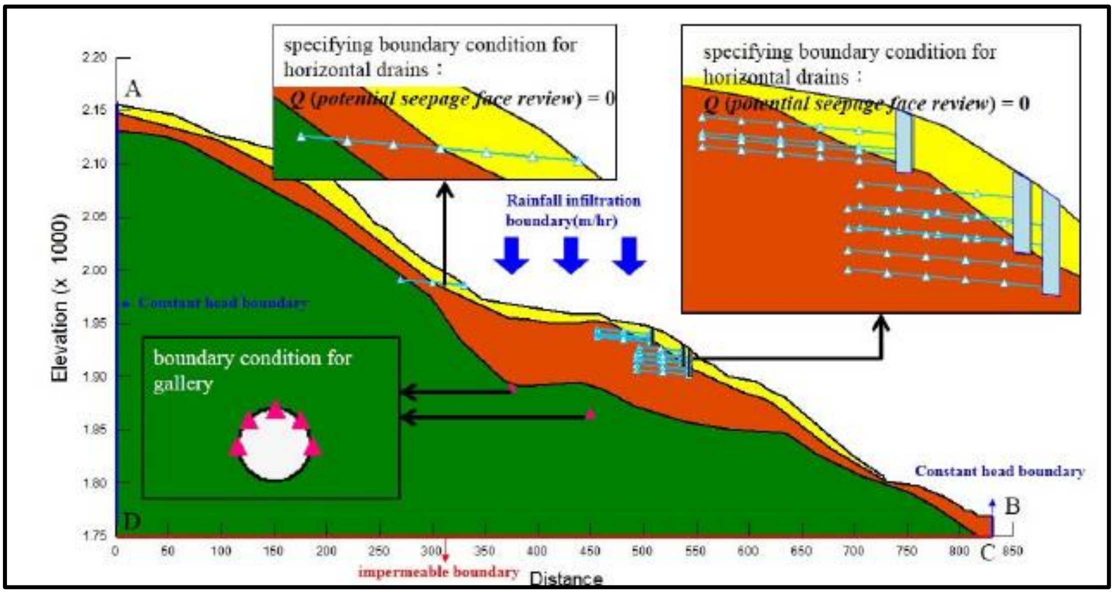


Figure 2.8 : Boundary Conditions Applied to Model to Represent the Rectification Measures

Source: (Lin et al., 2001)

3. CHAPTER 3: HISTORY AND DETAILS OF INVESTIGATIONS

3.1. History of the Landslide and Introduction to the Landslide Mitigation Project

Kahagolla landslide locates in the Badulla district, which is very famous for deep-seated landslide-prone areas in the country. It situates along the Beragala – Hali-Ela Road in between the culverts no. 11/3 and 11/6, at an elevation of 1430 m MSL. The first activation of this landslide was reported in 1957, and the affected area was an estate managed tea plantation. After the movements observed in the ground, the land was abandoned for decades. And also, it periodically damaged the road pushing it around this location and threated many houses located downslope.

Thereafter, the affected slope creeped at different locations in wet seasons. It was observed that these creep movements appearing towards north- west direction, because of debris deposited from a past landslide was accumulated in this area. The tea estate was destabilized and destroyed with appearing of tension cracks, scars, lumps indicating clear signs of many slip surfaces around the area as shown in figure 3.1.

Figure 3.1: Unstable Features of Old Landslide

Furthermore, tension cracks appeared in the road section, restricting the main transportation system among the cities including Colombo, Ratnapura and Badulla. The trace of road A016 was changed shifting it toward the mountain side by RDA after few decades. Figures no. 3.2 shows the changes of road alignment due to landslide threat.

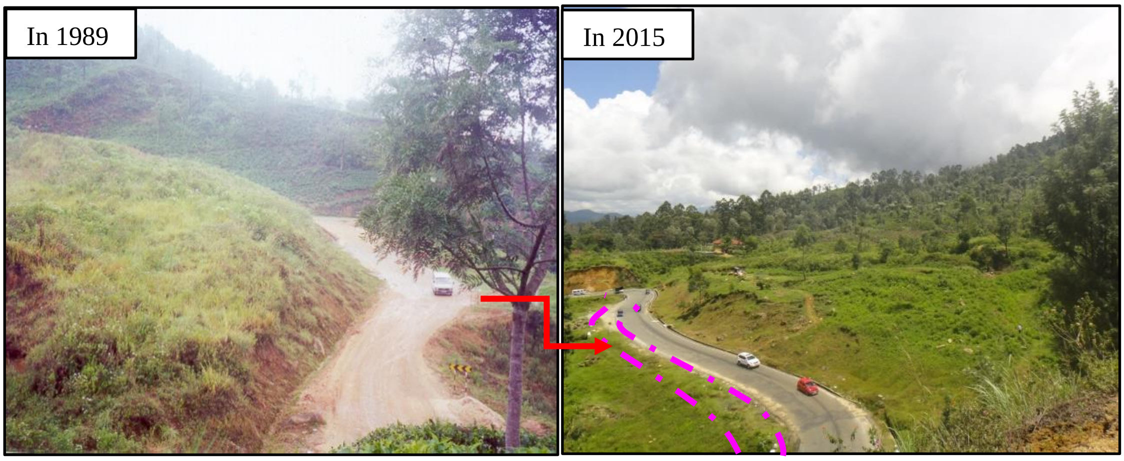


Figure 3.2: Shifted Road trace towards the mountain after few decades

The Landslide was recognized as highly active and suspected for possible destruction due to sudden deformation of the ground. Therefore, a decision was taken for emergency evacuation of residents from the toe area by the government authorities. But the decision was abandoned after some residents refused to evacuate and resettled in the same land. Therefore, the evacuation neither solved the problems about the vulnerability towards the community or difficulties of transportation along the road.

In 2014, the Kahagolla site was selected to be rectified as one of the most significant sites under "Landslide Disaster Protection Project of the National Road Network" program (LDPP), which was implemented by the Road Development Authority (RDA) with the financial and technical corporation of Japan International Cooperation Agency (JICA). The project was successfully completed in 2019, using Japanese technologies for rectification measures. Later, some modifications were added to the original design due to unforeseen difficulties during the constructions of countermeasures.

3.2. General Details and Morphology of the Landslide

General details of the landslide are as follows.

- Coordinates -6°47'32.4"N, 80°58'25.1"E
- Average Elevation -1430 m MSL
- GS Division -Panketiya

- DS Division -Haputhale
- Area -4.5 Hectares
- Length -500 m
- Width -150 m
- Depth -29 m

Most active landslides can be found in specific locations, where some sort of landform patterns exist on the ground. Topographical survey and Arial photograph interpretation were conducted in order to identify distinct features of landslide during detailed investigations. Results of interpretation clearly showed a landslide located in a rolling slope of colluvium, formed by a huge collapse of the mountain slope. Several landslide blocks were identified in the upper slope and locations of the tension cracks were mapped on the topography map as shown in figure 3.3.

3.3. Geology of the Area

Most of the rocks in Sri Lanka are composed of highly crystalline, non-fossiliferous metamorphic rocks that were created in the Precambrian age. This Precambrian basement is subdivided into three major litho-tectonic units based on their rock types, metamorphic grade, and isotopic characteristics.

1. The Highland Complex (HC)
2. The Wanni Complex (WC)
3. The Vijayan Complex (VC)

Highland Complex is the largest rock unit in the Precambrian basement and featured by many geological structures due to various deformational events that occurred at the formation. Rocks in this basement are metamorphosed under high pressure and high-temperature conditions (7-10 kBar, 710-900 C^0). This complex is usually composed of two main types of rocks, namely metasedimentary and meta-igneous (Coorey, 1984). Gneisses, sillimanite-graphite gneisses, quartzite, marbles, and some Charnockite are common types of rocks that are available in the granulite facies rocks of the Highland Series.

This landslide locates in the middle part of the Highland Complex, which is bounded by the Vijayan complex in east and Wanni complex in West. The rarity of rock outcrops and inconsistency in strikes and dips had made it difficult to recognize the geological structures by visual observation. The average strike direction and dip angle of bedrock were observed as N60W and 7W respectively.

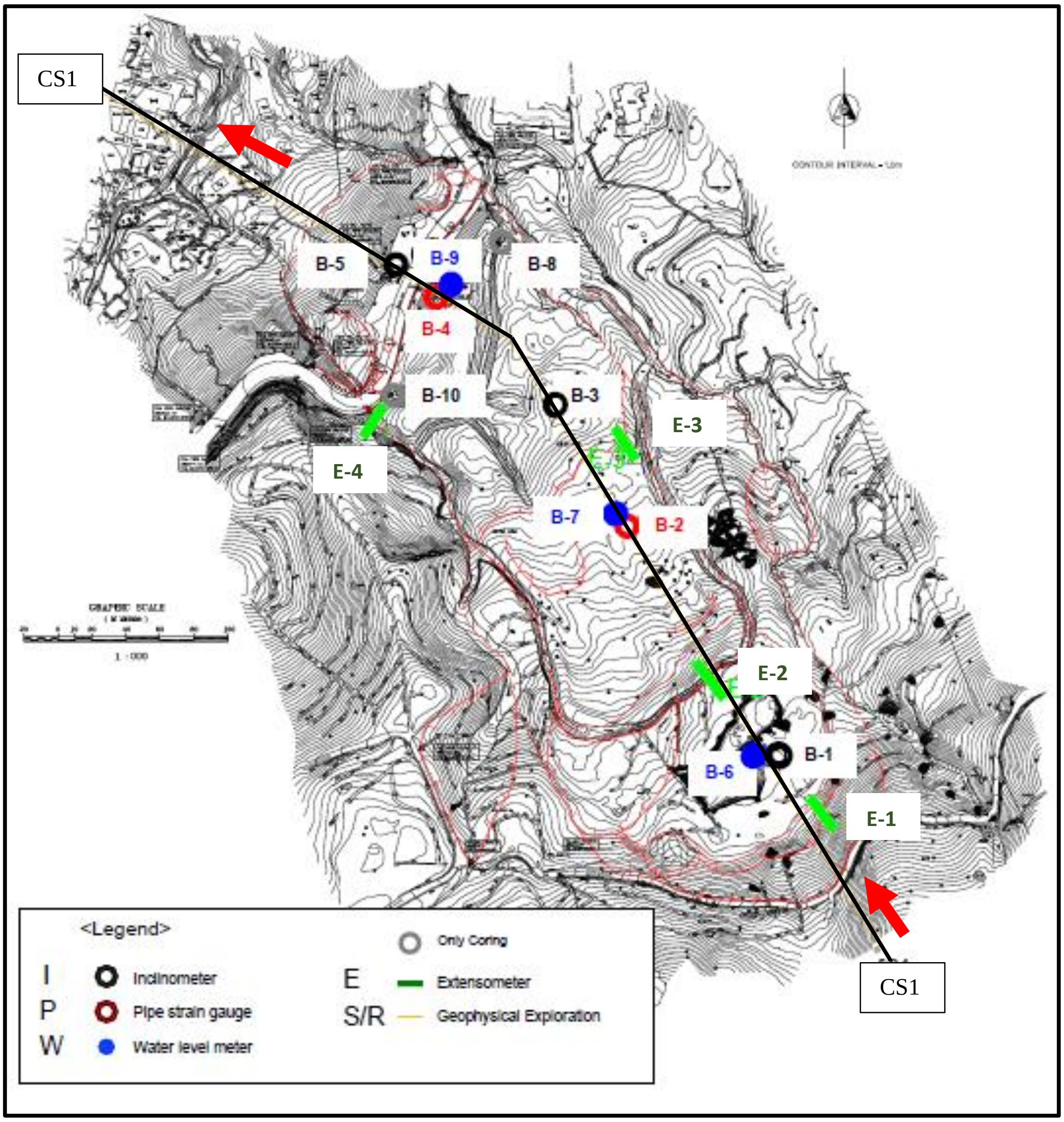


Figure 3.3: Landslide features and locations of monitoring instruments

Source: (Oriantal Consultants, 2015)

According to the lithological data observed from the borehole logs, the rock level encountered at intermediate depth from 12.55 m to 29.4 m within each borehole locations. Table 3.1 illustrates rock types underlying by the landslide that were identified through samples collected from core drilling.

Table 3.1: Lithology at the Kahagolla Site

Rock Type	Mineralogical Assemblage
Charnockitic gneiss	Hypersthene, Quartz, Feldspar, Biotite
Biotite gneiss	Quartz, Feldspar, Biotite
Hornblende biotite gneiss	Garnet, Hornblende, Quartz, Feldspar, Biotite
Calc gneiss	Biotite, Dolomite, Calcite, Pyroxene
Khondalite	Garnet, Sillmanite, Quartz, Feldspar

Figure 3.4 indicates the subsurface profile with major rock types that were found during the borehole investigations.

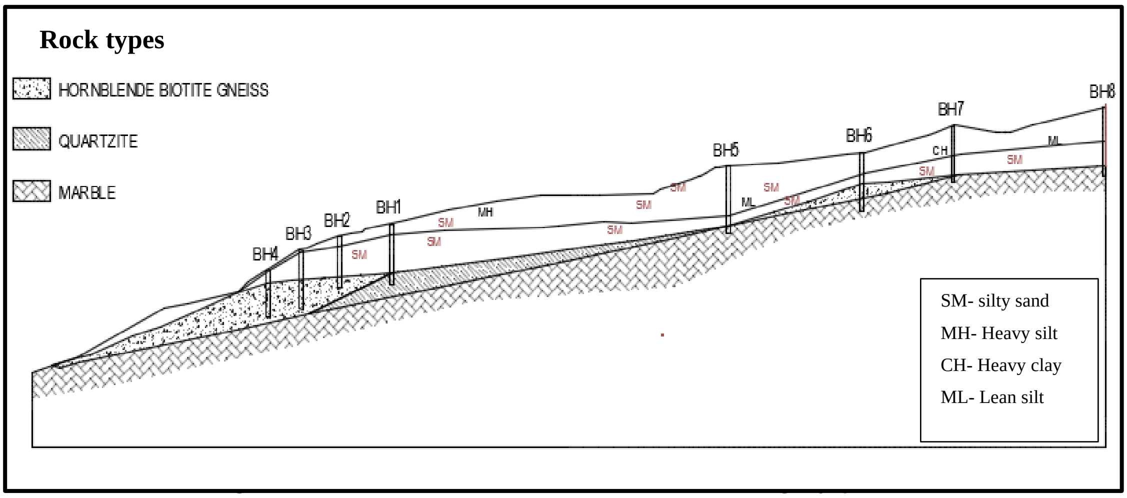


Figure 3.4: Subsurface profile along the main axis interpreted from Borehole Data Source: (Engineering Geological report of Kahagolla Landslide and Mitigation of Its Impacts, 2002)

Borehole investigations further revealed that bedrock is highly jointed with two sets of joints. Marble rocks found within the boreholes have a varying foliation dip direction from $10^0 – 30^0$, while others appear un-foliated and massive. Geo-technical investigations conducted in the area have found such inconsistencies in foliation even within the same cores that were drilled. These variations in the dip angle signify local variations in the geological structure with minor folding. The figure 3.5 illustrates such minor folding found within borehole no.5.

Figure 3.5: Folds in the core samples collected from bore hole no.5 (Source: Oriantal Consultants, 2015)

3.4. Drilling Investigations

Standard penetration test (SPT) was conducted at every 1.0 m depth interval for seven boreholes and samples were collected using Raymond sampler and Double tube core barrel. In addition to the SPT values, data such as ground conditions, weathering conditions, groundwater level, the situation of spring water, the color of the water were also recorded. The depth of drilling varies from 38m to 50m in different boreholes.

Borehole investigations revealed considerable information about subsurface layers. The uppermost layers of soil overburden consist of both colluvium and residual soils. These subsoil layers consist of clay, silt, sand, and gravel in different combinations (ex: clayey silt, silty sand, silty clay, clayey sand, silty gravel, sandy silt, and gravelly silty sand). However, the majority of the soil mass was identified as silty sand (SM as indicated in figure 3.4). Additionally, boulders can be found in different depths in different sizes. The thickness of overburden varies between 21m to 29m just above the road level whereas it decreases gradually towards the downhill. Bedrock exposes at the toe area of the slope. The slip surface could not be predicted at this stage. However, the thickness of the colluvium layer is considerable and varying from 20m to 29m. This implies the landside is a deep-seated failure. The figure 3.6 illustrates the borehole profiles obtained through the drilling investigations.

3.5. Geophysical Exploration

Seismic exploration and high-density electrical exploration were conducted to understand the properties of subsoil profile close to the surface. Seismic exploration was carried out for three traverse lines, which represent the upper part, middle, and the lower part of the slope separately. The velocity profile was prepared accordingly, which shows the subsoil profile according to layer density.

The two-dimensional electrical resistivity survey helped to understand about the weathering state of each layer, depth to the bedrock, aquifers and their continuity, which was very useful in the design of sub-surface drainage. The sub soil profile obtained from the resistivity survey along the main axis of the landslide is shown in figure 3.7.

3.6. Laboratory Testing

Twelve different locations at five boreholes were selected to obtain the samples during the drilling process and the following laboratory tests were carried out in accordance with the ASTM standards. Accordingly, following tests were conducted whenever necessary.

1. Tri axial test (CU)
2. Specific gravity test
3. Water content test
4. Particle size distribution analysis
5. Atterberg limits
6. Wet density test
7. Direct shear test

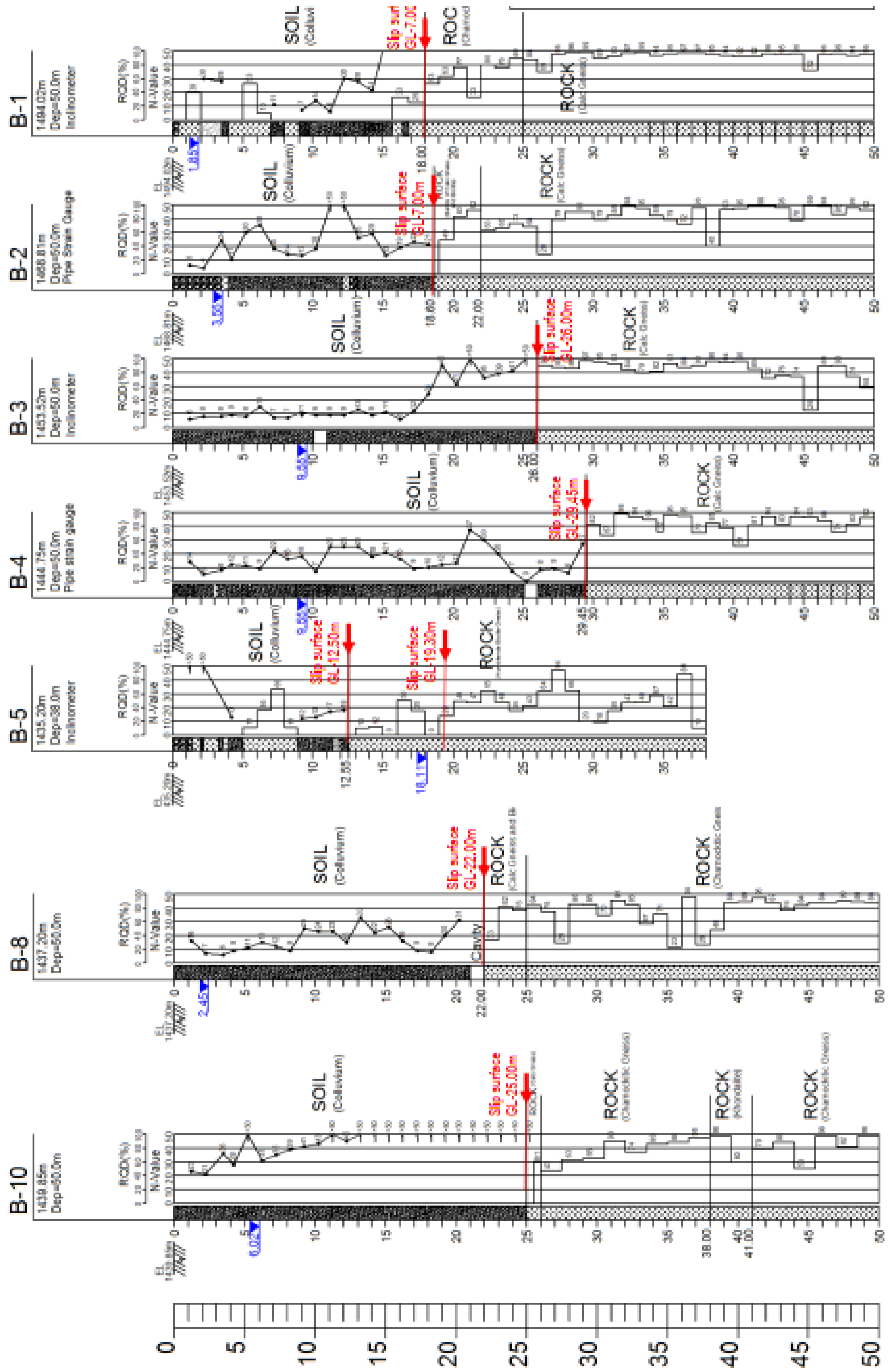


Figure 3.6: the borehole profiles obtained through the drilling investigations at Kahagolla Landslide

Source: (Oriantal Consultants, 2015)

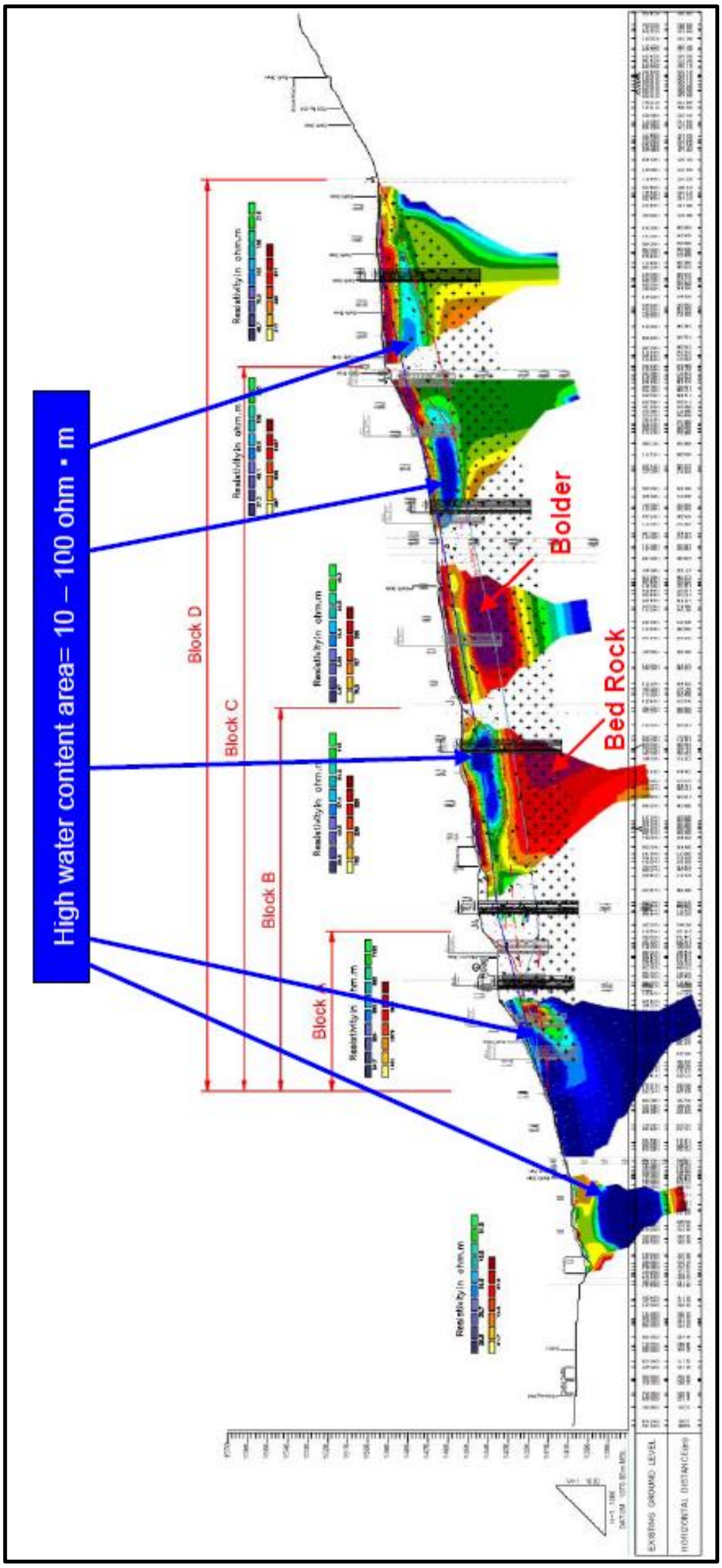


Figure 3.7: Resistivity Cross-Section at section 1-1 A016-010, Kahagolla

Source: (Oriantal Consultants, 2015)

According to the ASTM soil classification system, ten samples out of twelve were found as silty sand, consist of nearly 60% of sand, 23% of silt, and 13% of clay. Eight samples showed plasticity behavior with plasticity index in the range 8-34 with a significant amount of clay. The natural water content was recorded between 18 and 62, representing both unsaturated and saturated states.

Contrast to the above results, the undisturbed block sample obtained from the 1m below from the ground level showed high plasticity silt with a considerable amount of clay percentage. This sample was especially collected for testing of shear strength parameters. The unconsolidated Tri-axial test and direct shear test were proceeded accordingly. Both results showed same friction angle but the different cohesion values (Refer Appendix 2 for the summary of laboratory test results).

3.7. Landslide Monitoring

Landslide monitoring is an essential component of the design of countermeasures, which provides an understanding of the underlying mechanism of a landslide, collecting a wide range of data for design purposes. Main observations of the landslide monitoring were carried out to reveal the relationship with rainfall patterns and slip occurrence. The depth of slip occurrence, slide direction and its velocity, groundwater level fluctuation and its relation to slip occurring were used to model the actual site conditions at the analysis. Installed monitoring instrument at Kahagolla Landslide are listed in table 3.2 and locations of the instrument are given in the figure 3.3.

Kahagolla Landslide Monitoring process were divided in to four categories and real time monitoring data was obtained over two years to gather sufficient amount of data.

- Monitoring rainfall data → Rain gauge
- Monitoring surface deformation → Extensometer
- Monitoring ground water level → Ground water level meter
- Monitoring subsurface (slip surface) movement → Pipe strain gauge and Inclinometer

Table 3.2: Installed Monitoring Equipment at Kahagolla Landslide (in figure 3.3)

Type of Monitoring Instrument	Quantity	Location
Extensometer	4	E1, E2, E3, E4
Pipe strain gauge	2	B2, B4
Inclinometer	3	B1, B3, B5
Ground water level meter	3	B6, B7, B9

3.8. Results of Investigations and Monitoring Data

According to the monthly rainfall data collected from 2008 to 2014, the annual average rainfall is about 2397 mm/year, which is recorded from the nearest rain gauge at Pitaratmalai of Department of Meteorology. An unusual heavy rainfall of 600 mm/month was observed in October-December 2015 and associated creep movements were observed.

It was identified that the main triggering factor of landslide movement is rainwater infiltration and groundwater level rising during a prolonged rainfall event. The monitoring data indicates that the groundwater level is sensitive to both isolated storms and prolonged rainfall events but the movements in extensometers are only visible for more than 15 days antecedent rainfalls. The inclinometers did not indicate any sign of movements. But the pipe strain gauges fixed at the B4 location showed clear signs of movement at two depths for a prolonged rainfall event while the other one fixed at B2 location clearly indicated movements at several depths up to 24m.This reveals that the total colluvium layer up to 29m depth is subjected to move during a prolonged rainfall event. The increased positive pore pressure pushes the soft colluvium stratum while decreasing the shear strength at the rock-soil contact. The morphology of the catchment area also causes to gather a huge amount of water supply inside the landslide, therefore waterlogged areas appear frequently. The displacements of the landslide recorded in the monitoring instruments with corresponding daily rainfall and groundwater table are shown in figure 3.8 and 3.9. The monitoring data obtained from the pipe strain gauges and inclinometers revealed the depth to the slip surface at each borehole location as illustrated in table 3.3.

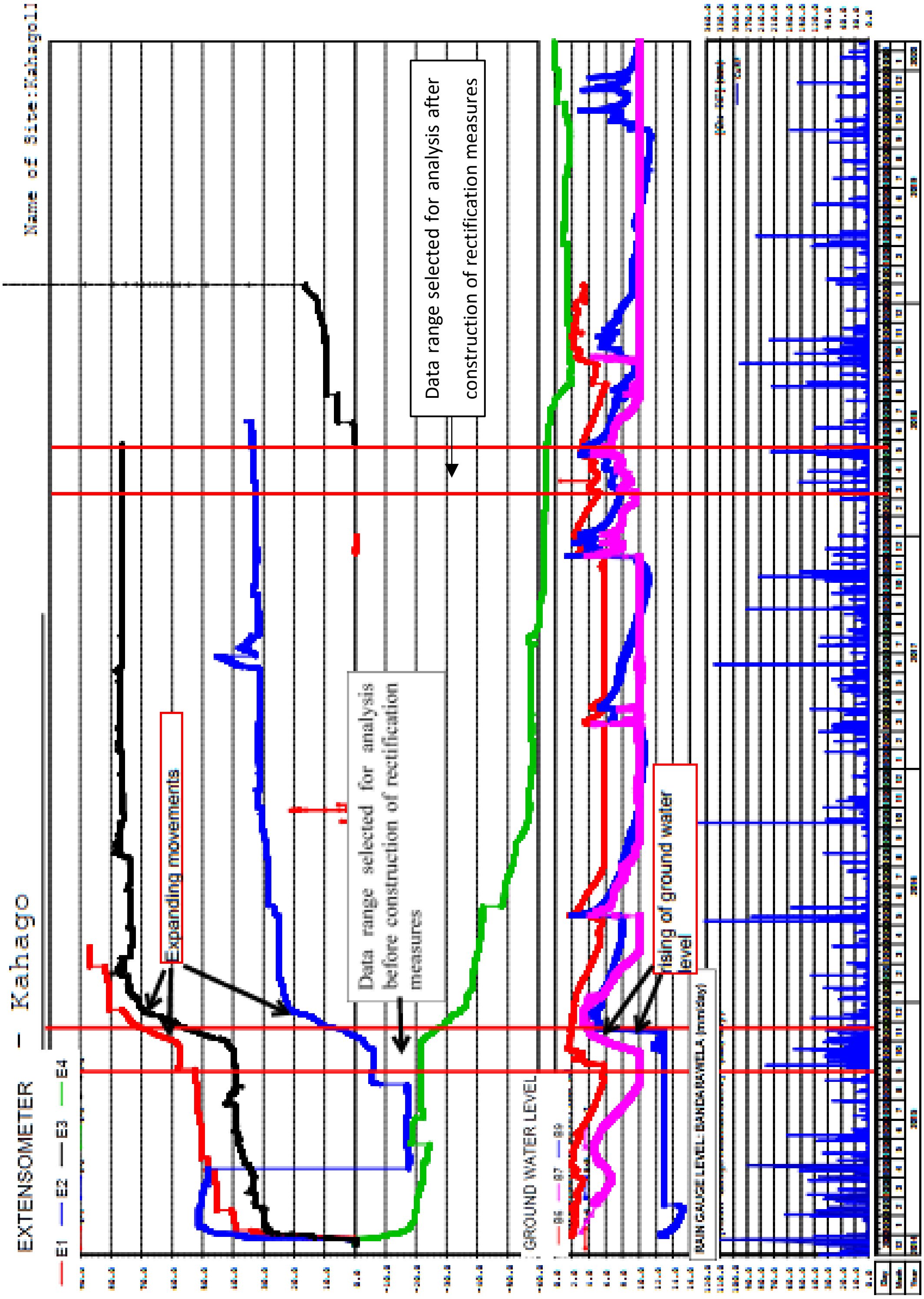


Figure 3.8 : Displacements recorded in Extensometers (Planview in figure 3.3)

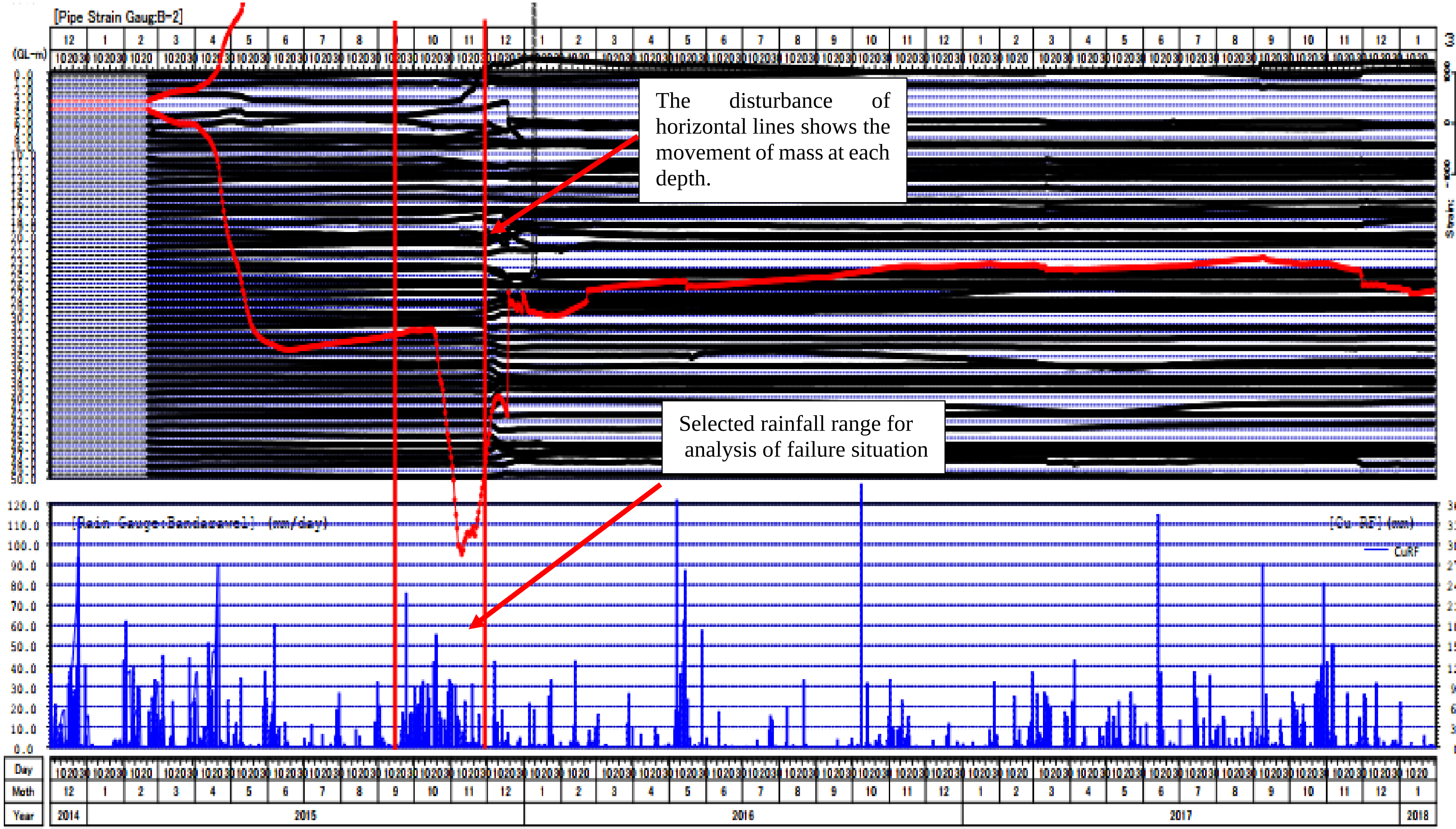


Figure 3.9 :Displacement recorded in Pipe strain gauge at B2 (Planveiwin figure 3.3)

Table 3.3: Estimation of Slip surface by monitoring instruments

Bor. No	Depth (m)	Accumulation depth of displacement (GL-.m)		Geology (GL.-m)		SPT	Ground Water level(m)		Estimation of Slip Surface (GL-.m)
		Pipe Strain gauge	Inclinometer	S: Soil R: Rock		N-Value	HWL (m)	Date	
B-1	50.0	-	None	S	0.00-18.00	6-50	-	-	18.00m
				R	18.00-50.00	-			
B-2	50.0	3.50 4.50	-	S	0.00-18.60	4-50	-	-	18.60m
				R	18.60-50.00	-			
B-3	50.0		None	S	0.00-26.00	6-50	-	-	26.00m
				R	26.00-50.00	-			
B-4	50.0	11.50 13.501 7.50 22.50 24.50	-	S	0.00-29.45	0-37	-	-	29.45m
				R	29.45-50.00	-			
B-5	38.0	-	None	S	0.00-12.55	12-50	-	-	12.55m 19.30m
				R	12.55-38.00	-			
B-6	20.0	-	-	S	0.00-20.0	-	1.45	31-Dec	
B-7	20.0	-	-	S	0.00-20.0	-	3.07	5-Jan	
B-8	50.0	-	-	S	0.00-22.00	6-33	-	-	22.00m
				R	22.00-50.00	-			
B-9	20.0	-	-	S	0.00-20.00	-	12.56	1-Jan	
B-10	50.0	-	-	S	0.00-25.00	21-50	-	-	25.00m
				R	25.00-50.00	-			

Some remarkable changes were confirmed at all the extensometers fixed at the site during the investigation period, with clear signs of expanding movements as the water table rises. Therefore, the water levels in three gauges were recorded as the extensometers start to move and used to find the critical water level as illustrated in table 3.4.

Table 3.4: Critical water level obtained through monitoring data

Bore hole No:	Observed period	Critical water level (0-GL, m)	Relevant Extensometer for each slip surface
B 6	From: 20 September 2015 To :25 November 2015	-2.00	E 1 (for J1)
B 7	From: 20 September 2015 To :25 November 2015	-4.70	E 2 (for J2)
B 9	From: 20 September 2015 To :25 November 2015	-14.76	E 3 (for J3)

Based on the field reconnaissance survey results, the landslide area was divided into four blocks as illustrated in table 3.4 and figure 3.10.

Table 3.5: Landslide Block Division based on field reconnaissance survey

Block no	State of Activity
J 4	Extremely high activity (Mainly rainy season) Size: Length:85m, Width:120m, Depth:13m Locates at the toe of landslide across the A016 road
J 3	High activity (Mainly rainy season) Size: Length:190m, Width:150m, Depth:20m Abnormal surface with scarps and a small water pool is observed
J 2	High activity (Mainly rainy season) Size: Length:370m, Width:180m, Depth:30m Scarp is clear. Abnormal surface is observed
J 1	High activity (Mainly rainy season) Size: Length:500m, Width:180m, Depth:30m Not remarkable movement is observed. High possibility of influence reaching to this block

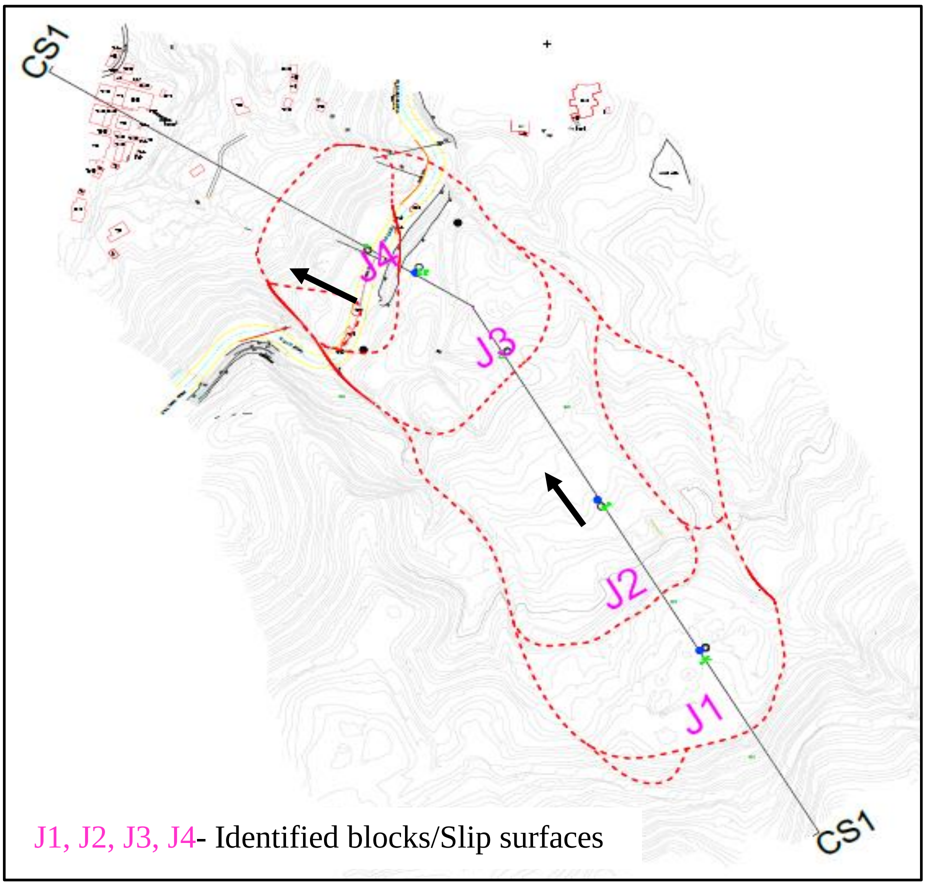


Figure 3.10: Identified main blocks and the longitudinal axis of the landslide

Based on the information obtained from field investigations, variations of subsoil strata are summarized in table 3.3 and shown in figure 3.7.

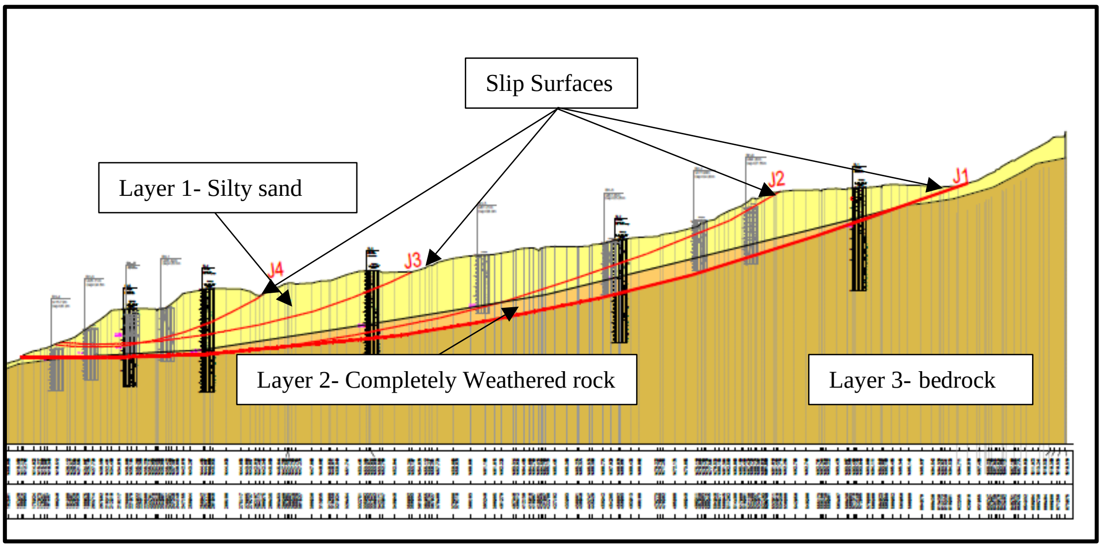


Figure 3.11: Subsoil profile with identified slip surfaces along CS1

Table 3.6: Details of Sub Soil Profile

Layer No	Soil Type	Description	Range Of field SPT "N"	Depth Range (m)				
				BH-01	BH-02	BH-03	BH-04	BH-05
Layer 1	Silty gravel/ Gravelly silt/ Gravelly sand/ Sandy silt/ Silty sand	Loose to very dense soil layer with rock boulders	4-36	0.00-9.75	0.00-13.20	0.00 - 15.00	0.00-17.00	0.00-3.30
Layer 2	Silty sand/ Sandy silt Weathered rock	Loose to very dense silty sand and sandy silt soil layer with rock boulders	6-50	9.75-18.00	13.20-18.60	15.00-26.22	17.00-29.45	3.30-12.55
Layer 3	Rock	Moderately weathered to fresh rock layer	-	>18.00	>18.60	>26.00	>29.45	>12.55

3.9. Conclusions of the Detail Investigation and Monitoring

Through the various type of investigations and long-term monitoring process, sufficient amount of data was gathered to perform the analysis. The key variables which govern the design such as, locations of aquifers, geological formation and its minor structures, locations of slip surfaces and their status, were successfully investigated.

Additionally, it was able to find a relationship in between ground water level fluctuation and the movement of landslide which is the most important discovery of the detailed investigations. Data collected through pipe strain gauges, inclinometers and boreholes were mainly used to estimate the depth of each slip surface while data of water level gauges and extensometers were used to identify the critical-water level and high-water levels that described in detail in next chapter.

However, the rock layers in shallow depth found at B5 borehole location disturbed the earlier plan of placing drainage wells, making it difficult to drill through it. Therefore, the proposed locations of drainage wells were changed in order to construct possible connections among them.

4. CHAPTER 4: DESIGN AND CONSTRUCTION OF COUNTERMEASURES

4.1 Introduction to design Procedure adopted by JICA

The adopted design procedure was mostly based on the data collected during the monitoring period and the detail investigations. The prominent feature of this design procedure is successfully adopting the relationship in between rising of groundwater level and the movements of the landslide mass, which was obtained after the processing of monitoring data with a great effort. Limit equilibrium method, the conventional way of calculating factor of safety had been used to reveal the stability for each pre-defined slip surface as mentioned in chapter 3. Additionally, some assumptions were also made by the Japanese designers, where the observations during the detail investigations are not sufficient to perform the analysis.

4.2 The Relationship in Between the Groundwater Level Fluctuation and The Movement of Landslide

As prolonged rainfalls continue over time, the groundwater level rises until it triggers the movements of the landslide mass. According to the definition of factor of safety for stability of slopes, the value reduces to unity (FOS =1) as the mass starting to move. The groundwater level at the activation of landslide is called “Critical water level” (CWL) while the ultimate level that can rise at the site is called “High water level” (HWL). The figure 4.1 explains the relationship among each phenomenon and the factor of safety concept. Monitoring data that were collected of CWL and HWL for the analysis are illustrated in table 4.1.

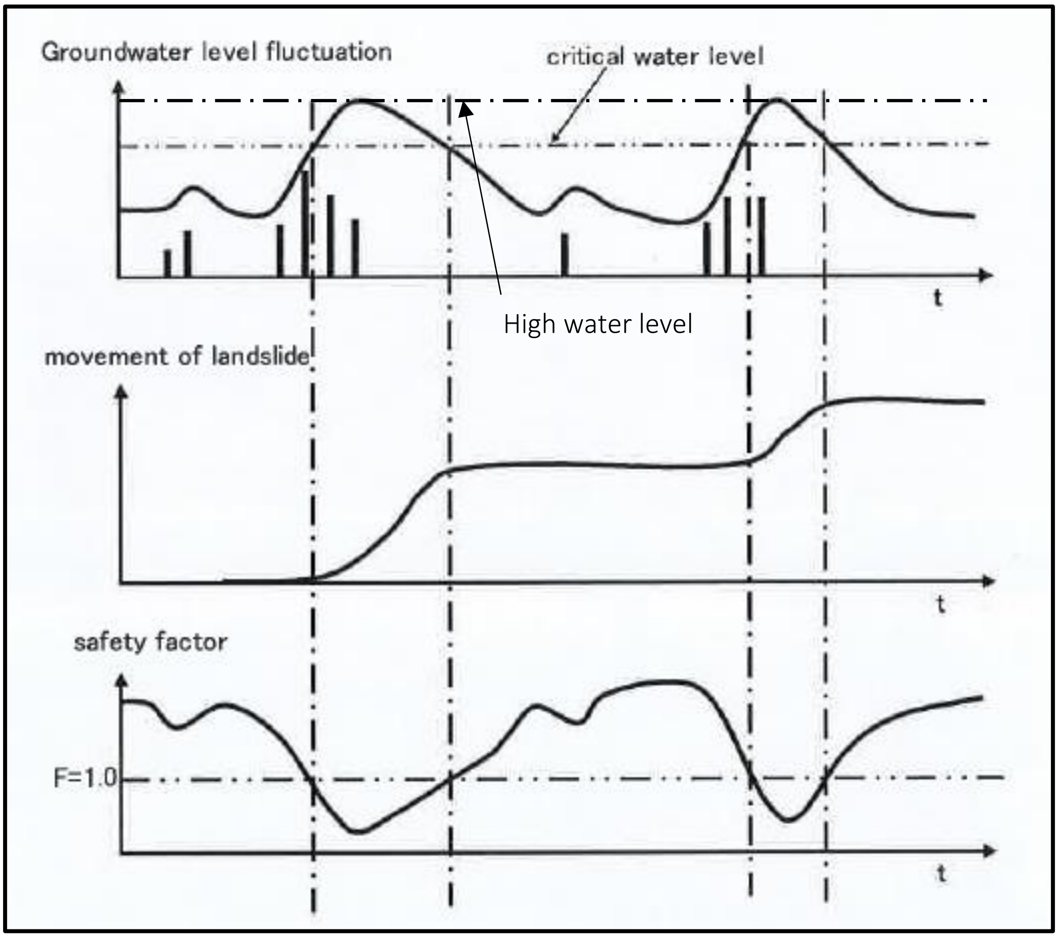


Figure 4.1: Changes of the ground water level affect the movements of mass and change the factor of safety (Source: Oriantal Consultants, 2016)

Table 4.1: Specific Groundwater levels used for Analysis (Source: Oriantal Consultants, 2016)

Borehole no.	CWL	Observed period	HWL	Observed date
B6	-2.00 m	From 29/12/2014 To 30/04/2015	-1.47 m	31/12/2014
B7	-4.70 m		-3.07 m	05/01/2015
B9	-14.76 m		-12.56 m	01/01/2015

4.3 Selection of Counter Measures

LDPP project adopted Japanese technologies and their design standards for the design of countermeasures. Based on the results of the detailed investigations, suitable combinations of countermeasures were considered and examined for the most appropriate combination of countermeasures in terms of both economic efficiency and design requirements. Table 4.2 describes the selected combinations of countermeasures with their cost of constructions. The design overall factor of safety value is 1.15 (Oriental Consultants, 2016).

Table 4.2: Comparison chart for selection of Counter Measures

Combination No.	Combination 1	Combination 2	Combination 3
Type of Counter measures	Counterweight embankment Horizontal boring works Drainage wells Pile works Light weight embankment	Counterweight embankment Horizontal boring works Earth removal work Drainage wells Ground anchor works Light weight embankment	Horizontal boring works Drainage wells Pile works
Overall safety factor	1.154	1.150	1.150
Direct cost of Construction	1,248,690,000 Yen	1,104,200,000 Yen	1,456,140,000 Yen
Remarks	Not recommended due to high cost	Adopted due to lowest cost	Not recommended due to high cost

4.4 Selection of Shear Strength Parameters

There were some practical problems with soil parameters obtained from laboratory testing, which did not represent the properties developed at the slip surfaces.

As an example, the analysis for critical situation had been carried out for the shear strength parameters obtained from the block sampling which are $\phi=32^0$ and $C=10$ kN/m^2 (in Appendix 2). The factor of safety values calculated for all four slip surfaces showed values more than 1, indicating results obtained from the analysis are not consistent with the data obtained from landslide deformation monitoring. Figure 4.2 illustrates the obtained unrealistic factor of safety values for corresponding high shear strength values and for all slip surfaces.

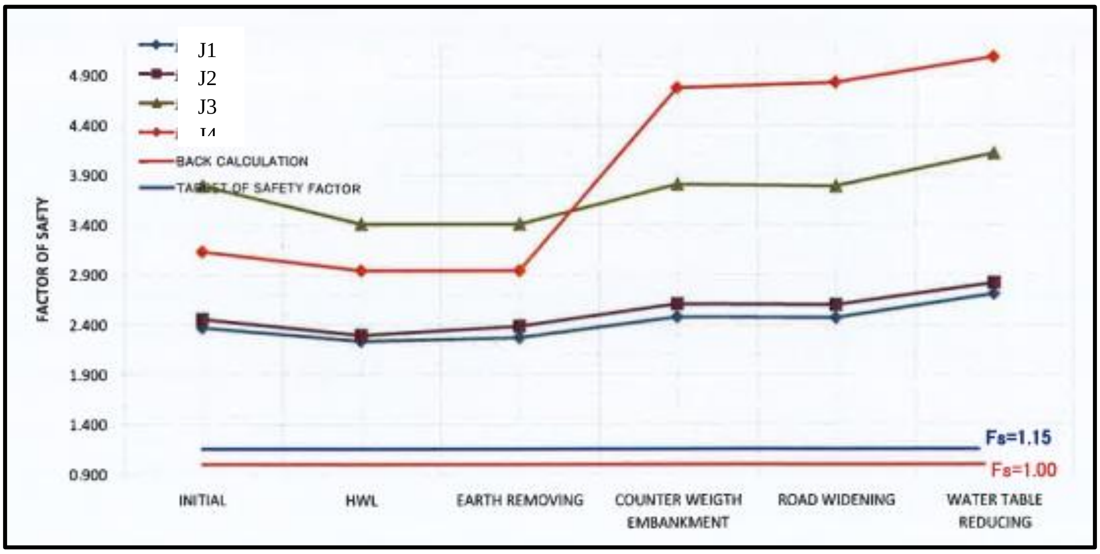
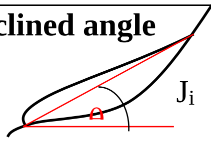


Figure 4.2: Factor of safety values obtained for each stage, each slip surface for block sampling (Source: Oriental Consultants, 2016)

Thus, Design Engineers had to use another method which was developed based on their experience gained in mitigation projects, in order to find corresponding shear strength values. Table 4.3 illustrates some of the parameters that are used to find the friction parameters in a particular landslide regardless the triggering factors.

Table 4.3: Relationship with inclined slip surface and angle of friction

Source: (Oriantal Consultants, 2016)

Inclined angle 	**ϕ' Internal friction angle**	**Coefficient of correlation**
0 < θ < 10	9	0.944
10 < θ < 15	14.8	0.951
15 < θ < 20	20.7	0.935
20 < θ < 25	23.6	0.941
25 < θ < 30	27.9	0.93
30 < θ	30	0.944

The friction angle was found according to the inclination of slip surfaces using above table and it was found as 10^0. Next the cohesion value was found modelling the factor of safety as 1.0 for the occurrence of the landslide movement. The groundwater level at this moment was kept in CWL (in table 4.1). Figure 4.3 illustrates the calculations of back analysis for the slip surface J1. The obtained shear strength parameters are illustrated in table 4.4.

Then the highest water level recorded at the site was modeled to represent the worst-case scenario and the lowest factor of safety was obtained for each slip surface. The countermeasures were applied hereafter to increase the factor of safety. In this design, the factor of safety was set to 1.15 considering the influence factors. (FOS-Prefectural Government office in Japan). Then factor of safety values was obtained for sequence of counter measures for each slip surface as mentioned in table 4.5.

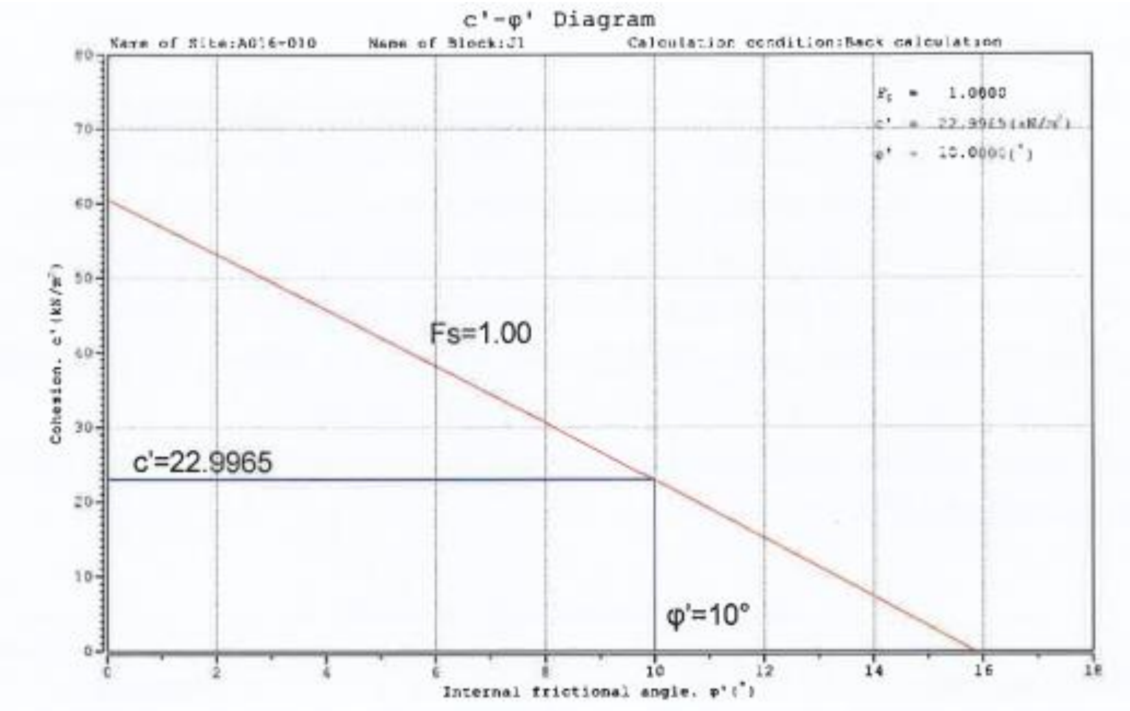


Figure 4.3: Back calculation for stability analysis (slip surface J1)
Source: (Oriantal Consultants, 2016)

Table 4.4: Shear strength parameters used for each slip surface (Results of Back analysis)

Block (Slip surface)	Initial safety factor (CWL)	Wet unit weight(kN/ m^3)	Friction angle	Cohesion (kN/m)	Remark
J1	1.00	18.0	10.0	23.00	The friction angle is estimated by inclination of the slip surface
J2	1.00	18.0	10.0	20.62	
J3	1.00	18.0	10.0	0.37	
J4	1.00	18.00	10.0	5.68	

Table 4.5: Obtained factor of safety values

Activity order	Counter measures	Factor of safety			
		J1 block	J2 block	J3 block	J4 block
1	Critical water level	1.0	1.0	1.0	1.0
2	High water level	0.957	0.954	0.892	0.945
3	2+ Earth removal work	1.014	0.986	0.892	0.945
4	3+ Counterweight embankment works	1.11	1.091	1.109	1.93
5	4+ Road widening	1.12	1.089	1.104	1.972
6	4+ Drainage well+ horizontal drains (3m drawdown of WT)	1.21	1.152	1.192	2.041

Overall design safety is achieved

Optimum combination of countermeasures combination 2 in table 4.2 was selected to rectify the landslide and located as in figure 4.4.

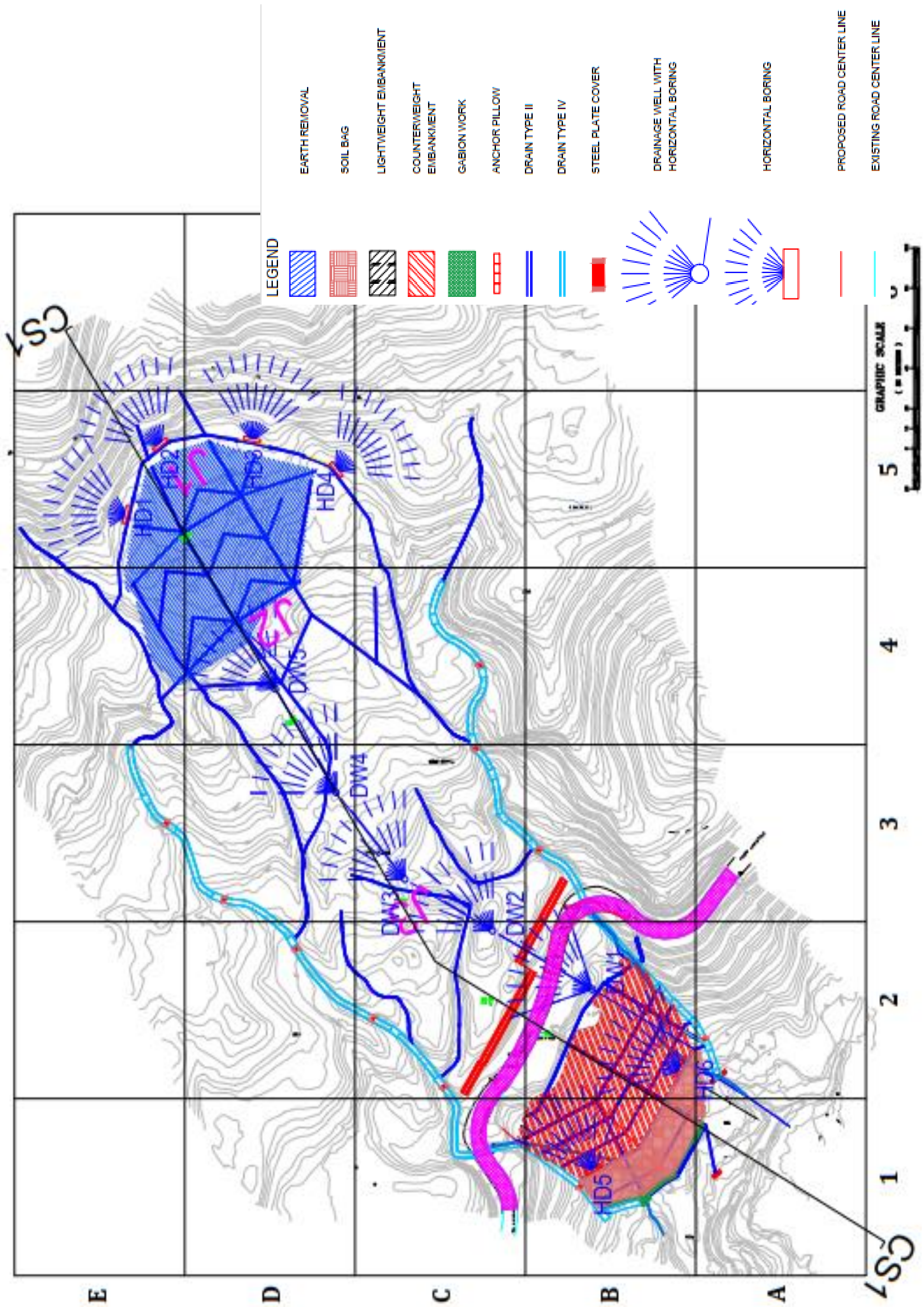


Figure 4.4: Optimum combination of countermeasures applied to the landslide

4.5 Construction of Countermeasures

4.5.1 Surface Drainages

Structures of drains had been carefully designed following the shapes of land deformation. Accordingly, valley paths inside the landslides were chosen to collect water easily to drain sections, minimizing the excavations. Wide and shallow precast sections were used around the boundary of the landslide with sub-surface drains underneath them. This control work aims to prevent water from entering into the landslide and collecting a considerable percentage of water before infiltrating into the soil.

Figure 4.5 illustrates typical drain sections constructed on the ground and valleys. Figure 4.6 shows a surface drain constructed at the toe of the landslide area.

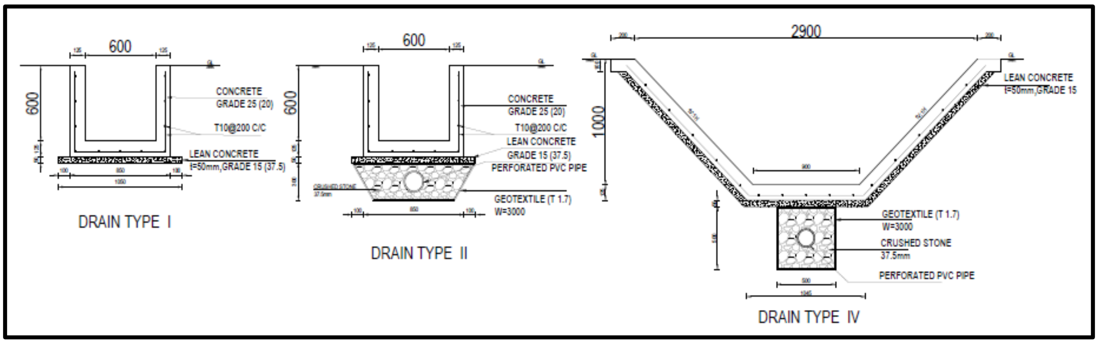


Figure 4.5: Types of drain sections constructed along the landslide

Figure 4.6: Outlet of Surface drainage system (A2 in figure 4.3)

4.5.2 Earth Removal Works

Earth removal work was carried out at the upper region of the landslide to reduce the driving force, thereby improving the stability of the landslide. 1:8 mild slope was maintained at cut slope. Figure 4.7 and 4.8 illustrate the earth removal area with a sudden change of slope.

Figure 4.7: Earth removal and slope protection at the Landslide upper area (D4, D5, E4, E5 in figure 4.3)

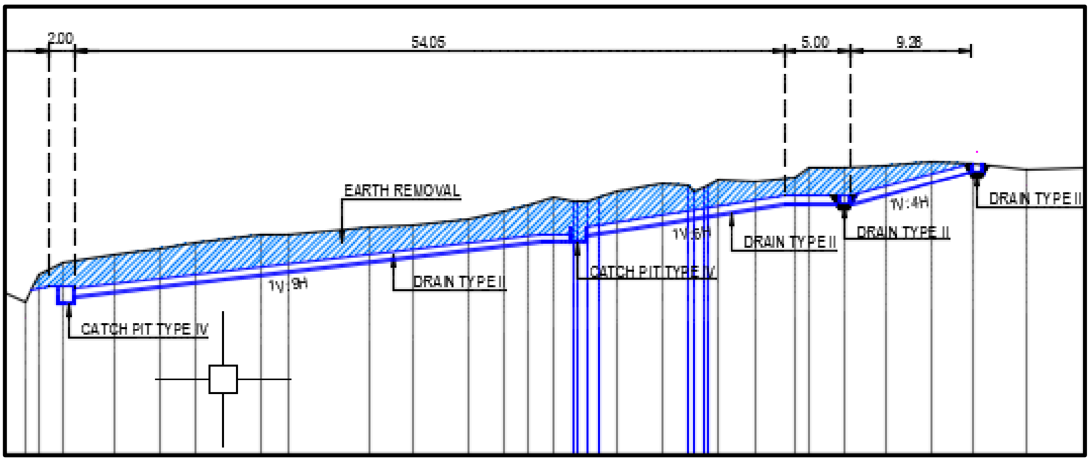


Figure 4.8:Typical cross section of Earth removal

4.5.3 Counterweight Embankment Works

An embankment at the toe area of the slope was constructed to increase the resistant force by filling the earth. 1:2 filling slope was maintained with a 3m width berm at every 10m distance. Drainage inside the filling was improved with gravel drains. The counterweight embankment after the construction is shown in figure 4.9 and 4.10.

Figure 4.9: Counterweight embankment work at the landslide toe area (B1, B2 in figure 4.3)

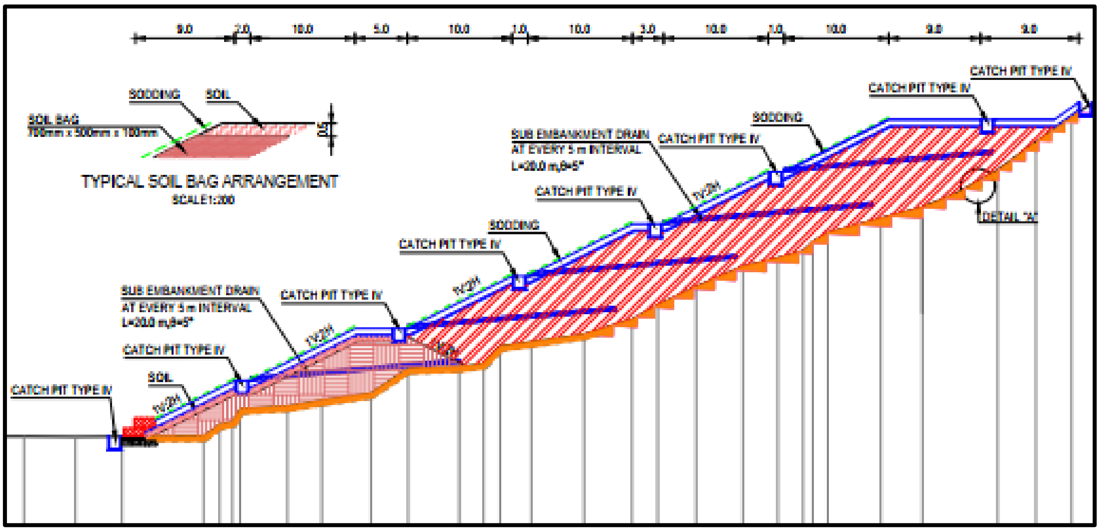


Figure 4.10: Typical cross section of Counterweight Embankment works

4.5.4 Ground Water Drainage Work: Horizontal Drains

Fan type Horizontal drains were selected to lower the groundwater table effectively in narrow and long landslide. Each fan has 10 pipes in 50m length. Four fans were constructed at the crown area of the landslide while the other five constructed inside the landslide. These drains were installed at 12m deep inside the drainage wells. Every fan was constructed close to the slip surface. The expected drawdown of the water table is 3m from the highest water level that occurred at the site. Figure 4.11 and 4.12 illustrates the plan view of constructed ground water drainage work and the horizontal drains (HD 1 to HD 6) located at the toe area.

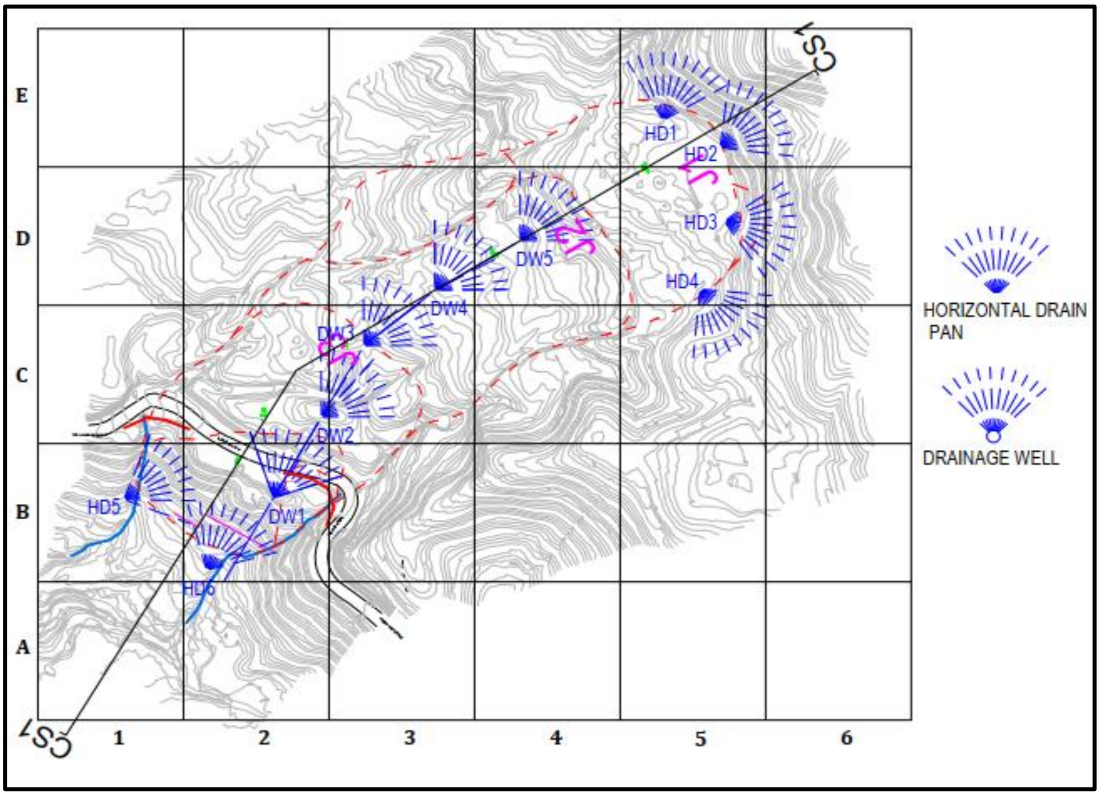


Figure 4.11: Plan view of the ground water control works

Figure 4.12: Horizontal drain outlets at toe area (B1, B2 in figure 4.9)

4.5.5 Ground Water Drainage Work: Drainage Wells

Five drainage wells (DW1 to DW 5) were constructed inside the landslide body to collect the groundwater from horizontal drains and themselves. The diameter of a well was 3.5m and made with steel lining plates. All wells were connected with discharge pipes at the bottom of the well. The whole system is supposed to work under gravity and pressure. The lowest drainage well collects all the water coming from the other wells and discharges it at the toe of the landslide. These wells were designed above the slip surface. The inside the drainage well after the construction and the typical cross section of the well are shown in figure 4.13 and 4.14.

Figure 4.13 : Inside a drainage well (B2, C2, C3, D3, D4 in figure 4.9)

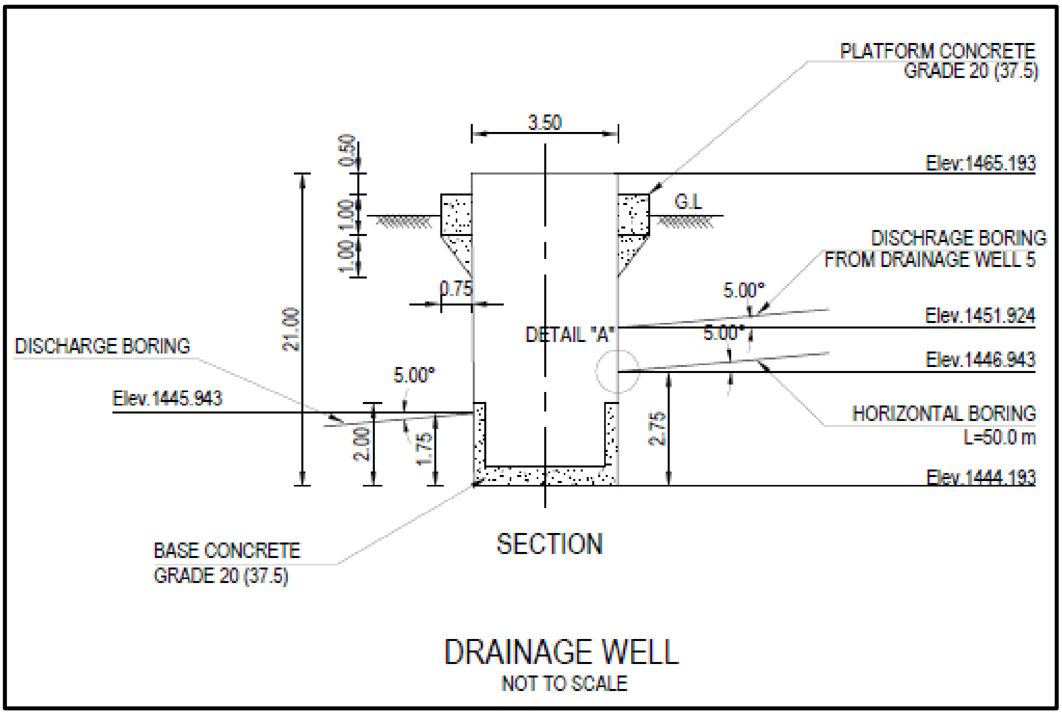


Figure 4.14 : Typical cross section of the drainage well

4.5.6 Ground Anchor Works

Here, addition to the above landslide controlling measures, ground anchor works were also adopted to the design as a resistant measure. Accordingly, 134 ground anchors were constructed just above the road level with two rows. Each nail was anchored to the bedrock with 10m length while total length was 50m in each. The arrangement of ground anchor after the construction is shown in figure 4.15. The figure 4.16 shows the nail arrangement in the plan view and cross section.

Figure 4.15 : Ground anchor beside the road (C2, B2, B3 in figure 4.8)

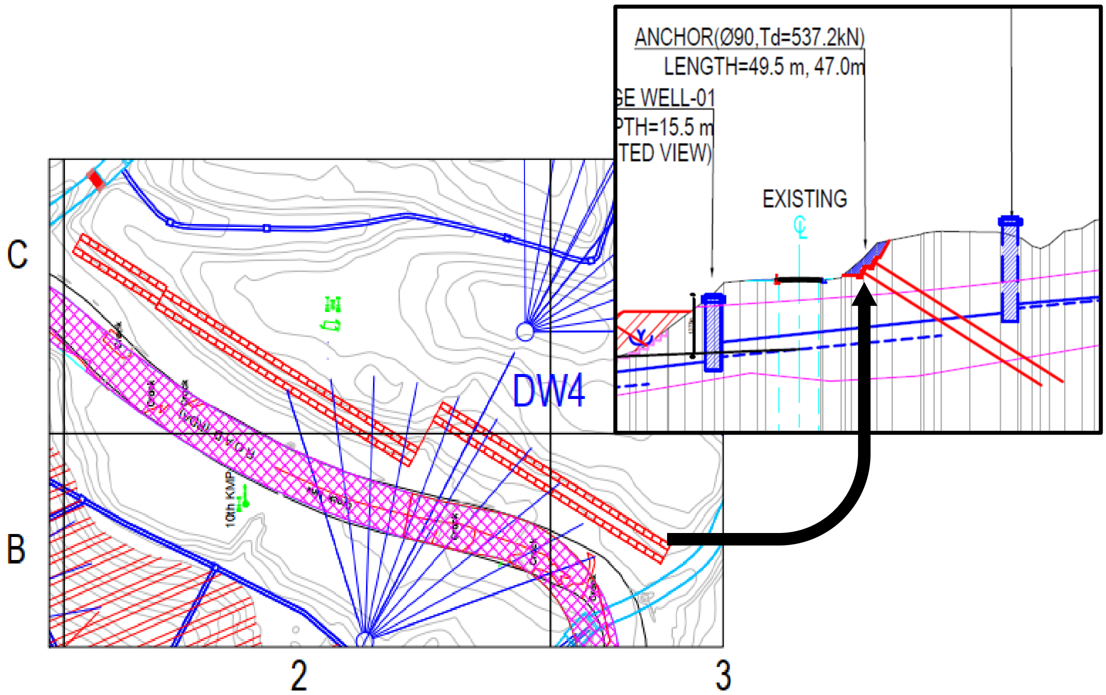


Figure 4.16: Plan view and cross section of ground anchor arrangement

Two lines of soil anchors were constructed at the site with spacing of 2m and 67 numbers for each line. The design conditions of anchoring system are as in table 4.6.

Table 4.6: Design conditions of anchors

Detail of Anchor	Ground anchor line 1	Ground anchor line 2
Length	49.5 m	47.5 m
Direction	150^0	150^0
Pullout resistance	600 kPa	600 kPa
Resistance reduction factor	2.5	2.5
Bond length	8 m	8 m
Bond diameter	0.09m	0.09m
Anchor Spacing	2 m	2 m
Tensile capacity	550 kN	550 kN

4.6 Conclusion About the Design and Construction Sequence

Applying above counter measures, overall factor of safety of 1.15 was achieved for the analysis. A typical cross section drawn across the entire landslide was used, which is not straight. This longitudinal cross section with counter measures is illustrated in figure 4.17. The drawdown of the water table had been roughly estimated in this stage based on the experience gained in previous designs and constructions. However, it is essential to develop a seepage model with actual pore water pressure conditions which is not performed by the designer.

A top to bottom construction procedure was followed for the most of the rectification measures. The construction works commenced with the excavation of drainage wells. Some difficulties found here when the ground consists of massive boulders and couldn't keep the excavation straight. As a solution, steel plates were fixed around the well just after the excavation, which made the ground more stable without collapsing. The drilling works for horizontal drains were proceeded next, inside of the drainage wells. The ground modification works, such as earth removal works and counterweight embankment works were commenced simultaneously. The excavated soil from the upper slope was packed in soil bags and used to build a counterweight embankment at the downslope. The surface drainage works were completed on the modified ground.

Type IV drain were constructed along the valley to carry rainwater from the upper slope. Finally, ground anchor works were completed.

An obstacle was found in between drainage well no.1 and 2 (DW 1 and 2). The rock layers underneath this area made it difficult to drill for the discharge boring in between two drainage wells. Finally, a decision was taken to shift the locations of two drainage wells from the middle to the west direction to avoid the rock layer. The connections were made among the drainage wells later through the discharge pipes by directional drilling. However, a considerable change was discovered from the early assumption of the expected drawdown of the water table in this area. The rock layer underneath and shifting the drainage well slightly made the groundwater accumulation in the middle part of the landside toe area. This phenomenon was discovered during the post-monitoring period which was not anticipated during the design stage. This research aims to discover effects after the construction of rectification measures including such situations.

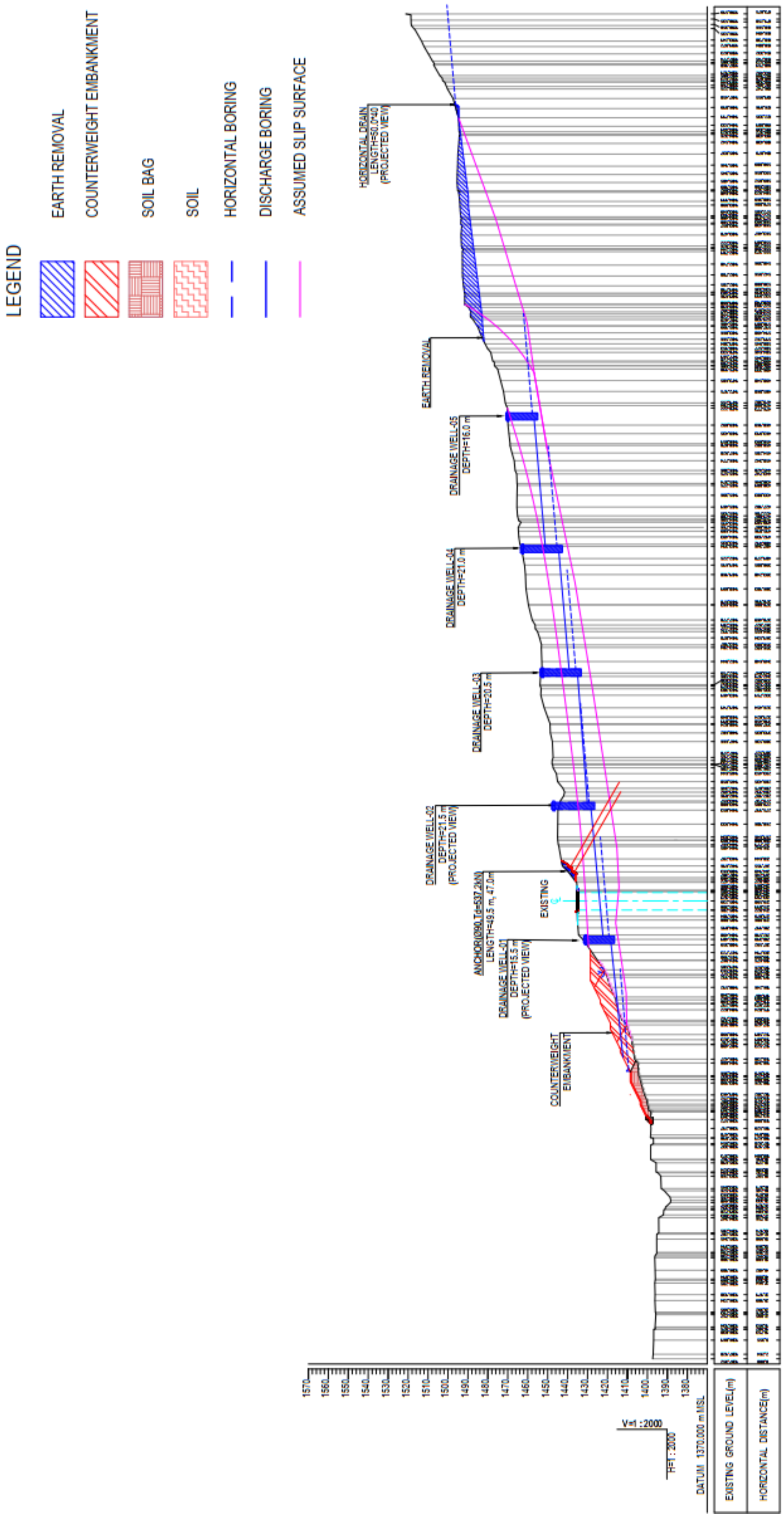


Figure 4.17: Typical Longitudinal cross section along the landslide with applied countermeasures

5. CHAPTER 5: BACK ANALYSIS OF THE LANDSLIDE

5.1 Introduction to Back Analysis

The back analysis is a popular method in slope stability analysis when there are some uncertainties about parameters at the moment of failure. This method is widely used for deep-seated landslides, where the groundwater table plays a significant role in triggering the landslide as well as the cross-section is available before the failure occurs.

In this research, the failure of the Kahagolla landslide was back analyzed by simulating events that occurred with the storms, which trigged its movement. A two-dimensional slope model coupled with a seepage model was developed to grasp the actual site conditions such as pore pressure generated across the profile. A typical cross-section drawn across the axis of the landslide was selected since the landslide shows creep movements, which has not changed the profile significantly from the original. The main aim of performing these analyses is to obtain appropriate values for some unknown parameters to represent the actual site conditions and then forecast the performance of rectification measures.

A 2D infiltration model was created in GeoStudio SEEP/W 2012 software using available data to analyze the infiltration behavior of both saturated and unsaturated soil. The modelling of seepage across the profile requires hydraulic properties of each soil type such as the soil-water characteristic curve, volumetric water content, residual water content, saturated permeability, etc. Further, it requires boundary conditions corresponding to the site, mesh properties, initial conditions of the water table, and some material properties. The software can analyze transient stages under user defined varying boundary conditions with time. The generated results can be incorporated easily with slope stability analysis under SLOPE/W software.

5.2 Procedure of Back Analysis of Failure Situation

Absence of field data relevant to some essential parameters made it necessary to perform several back analyses for both models. First, the seepage model was developed by available information on geology, hydrology, and topography through the results of desk studies on preliminary and detailed investigations. The two-dimensional subsurface profile was prepared based on the data collected from borehole logs, resistivity data, and laboratory tests. Boreholes drilled along the longitudinal axis revealed the layered structure of the soil profile, which can be divided by their density, soil type, and shear strength parameters. Accordingly, soil profile was divided into three layers based on their properties as described in detail in chapter 3.

After preparing the subsurface soil profile, the next step was applying the boundary conditions to represent the actual site conditions. Accordingly, majority of boundary conditions were applied related to the hydrostatic pressure and flows considering their varying nature with time. Special attention was given for aquifers shown in resistivity profiles. Moreover, the monitoring data related to ground water level was used at this stage in order to define the initial ground water table.

To build the seepage model in SEEP/W software, some of the hydraulic properties were selected from literature review. Accordingly, some layers were analyzed with data available on recognized soil types while others with assumed values. The parameters of assumed values are saturated permeability, saturated volumetric water content etc. The sample curves for soil water characteristic curve and volumetric water content which are available in this software corresponding to each soil type were also used. The final values were selected from performing analyses for assumed values within an acceptable range mainly based on their soil type. The groundwater table observed in landslide monitoring data were compared with the observed results and parameters (saturated permeability, saturated volumetric water content curves) were adjusted until the results of the numerical model agree with the observed ground water levels.

Then the corresponding slope stability states were calculated for two stages named “critical water level” and “high water level”. The critical water level is chosen with the activation of landslide which is described in detail in the chapter 4. The high-water level was obtained from the monitoring data where the highest water level recorded at the site. Some slip surfaces identified through the monitoring data of strain gauges and extensometers were used in the back analysis procedure. Only the shear strength properties of colluvium layer were varied due to all slip surfaces lying within this layer. In contrast to the method used by the Japanese designer, only one set of appropriate shear strength parameters were obtained for all four slip surfaces.

The flow chart in figure 5.1 illustrates the procedure of back analysis.

5.3 Initial Boundary Conditions

It is necessary to apply some boundary conditions to the model for calculating the pore water pressure in soil profile with varying circumstances. The most common boundary conditions are unit flux, total flux, pressure head, and zero pressure condition that can be modeled as a line, node, or area type. These boundary conditions help to develop the model close to the site conditions.

In this research, several types of boundary conditions were applied as in figure 5.2. The rainfall was modeled over the upper boundary of the profile AB, BC, and CD, as a unit flux with a function that varies with time. Zero total flux was applied to the side of the slope above the water table at AH and DE for non-lateral flowing of water infiltrated. The bottom boundary of profile GF also was kept at zero total flux situation to water to simulate no flow of groundwater table to further down. The initial total head of 20m and 205m was applied to the sides of the profile GH and FE, below the water table to maintain minimum depth to the groundwater table. These heads were selected using water level gauges fixed at the site before the starting of particular rainstorm. The 10m mesh was created in soil profile considering the wide area selected for the model. The coordinates of each boundary type are given in table 5.1.

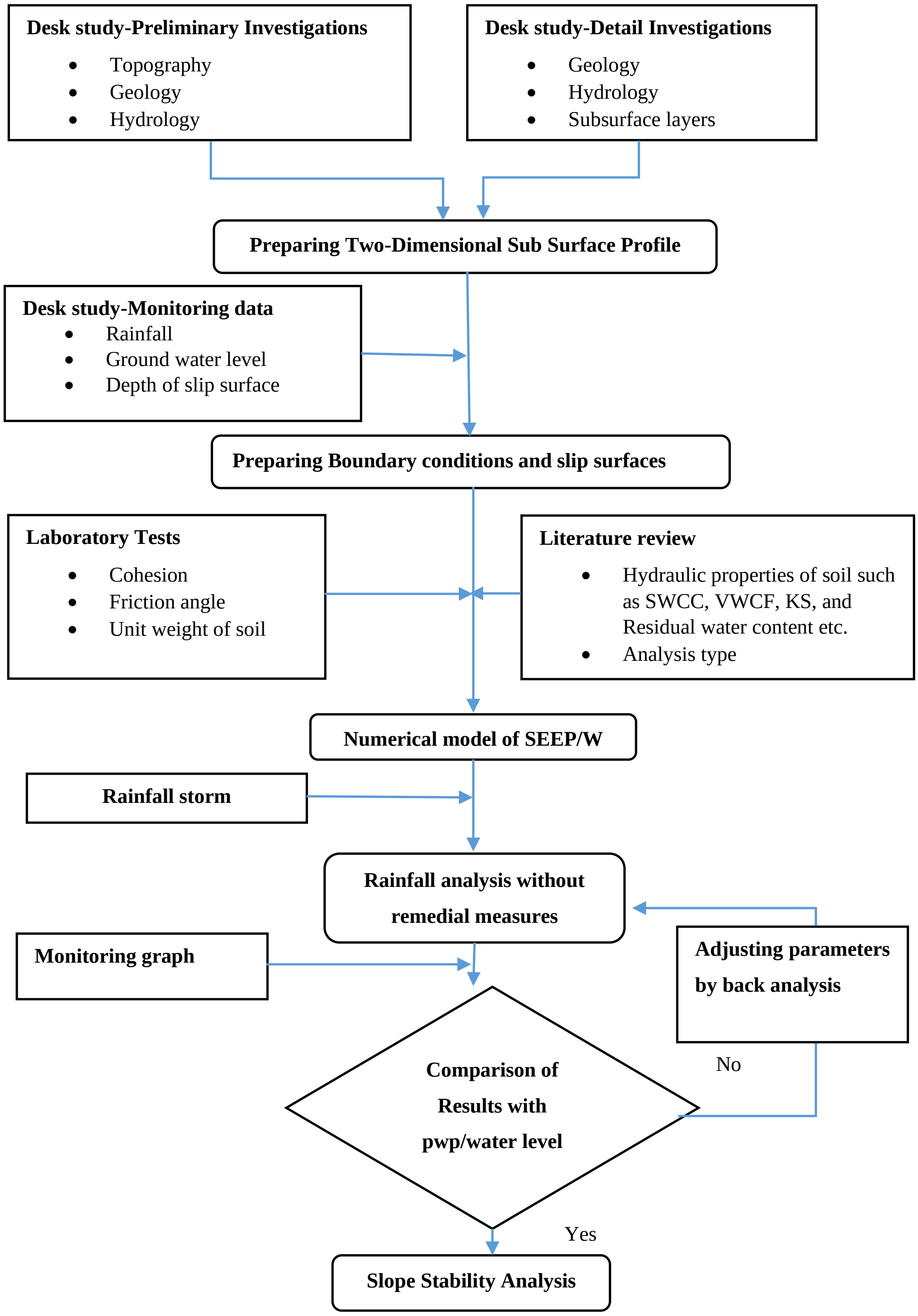


Figure 5.1: Procedure for back analysis of the failure situation

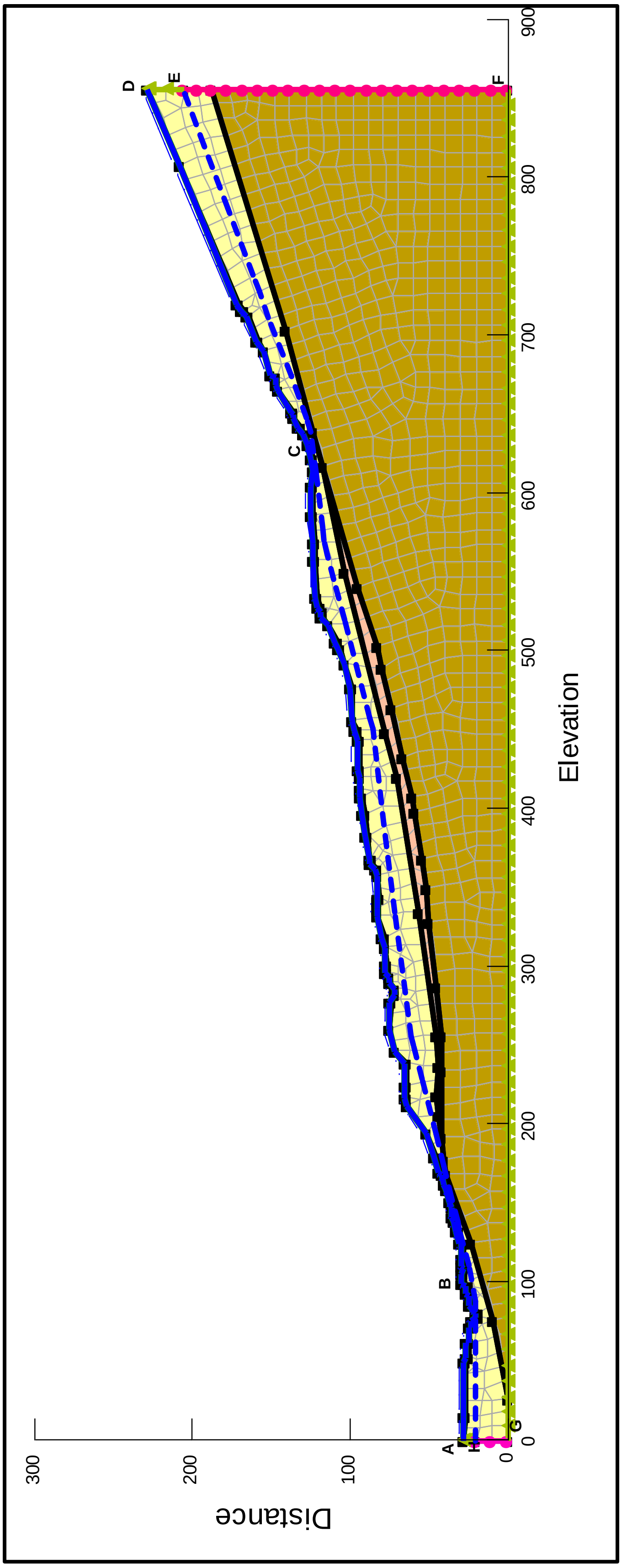


Figure 5.2: Boundary conditions applied to the profile along CS1

Table 5.1: Detail of Boundary Conditions

Name of the Boundary condition	Type of boundary condition	Stating point		End point	
		X coordinate	Y coordinate	X coordinate	Y coordinate
GH-Left Head	Head =20 m	0.000	0.000	0.000	20.000
EF-Right Head	Head =205 m	856.000	205.00	856.000	0.00
AH-Zero seepage	Total flux=0 m^3	0.000	27.500	0.000	20.000
DE-Zero seepage	Total flux=0 m^3	856.00	227.500	856.000	205.000
GF- Zero seepage	Total flux=0 m^3	0.000	0.000	856.000	0.000
ABCD-Rainfall	Unit flux= func.	0.000	27.500	856.00	227.500

A rainfall hyetograph occurred on 2015-09-20 to 2015-11-25 was selected to create the rainfall boundary condition (figure 5.3). This hyetograph consists of 67 daily rainfalls until the peak ground water level occurred at the site. The rainfall data was gathered from the station fixed at Bandarawela central college, which locates 5 km away from the landslide. The rainfall event had been recorded before the installation of remedial measures.

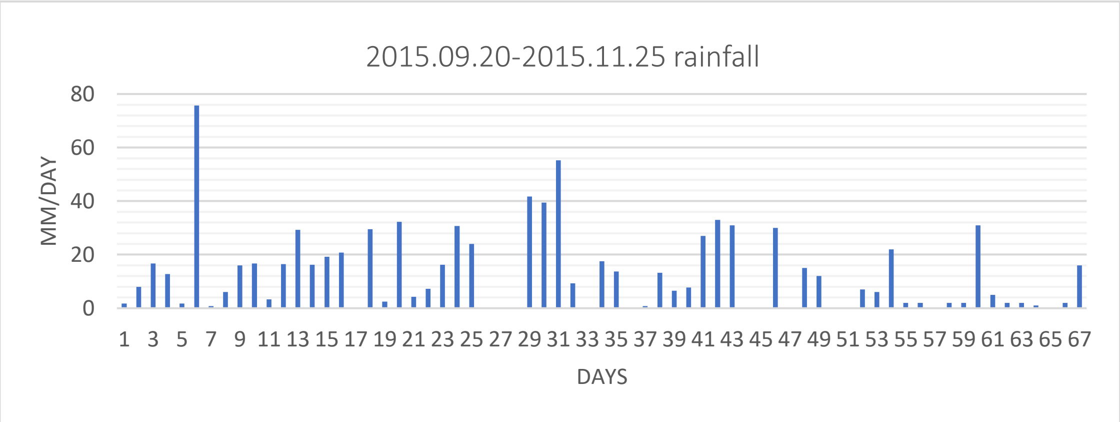


Figure 5.3: Rainfall selected for analysis before construction of rectification measures (from figure 3.9)

This rainfall event was carefully chosen at the starting point of a rainy season of 67 days, followed by an approximately dry season of 3 months. It was observed through the monitoring graphs that the groundwater table was at the lowest level and in steady-state before the rainfall event.

5.4 Hydraulic Parameters of Soil Layers

The saturated hydraulic properties of the soils are essential when the effects of matric suction in seepage analysis is needed to be considered. In this research, the effect of vegetation cover is also included by modeling it as a 200 mm thick layer. The hydraulic conductivity function and soil-water characteristic curve for silty sand was obtained by the literature review. However, due to the lack of experimental data relevant to the hydraulic properties of other soil layers, assumed values were used to estimate the parameters. The final values were chosen after adjusting values and comparing the numerical results with monitoring data. The results of hydraulic properties relevant to each soil layer are presented in table 5.2. The layers are presented in figure 5.4.

Table 5.2: Hydraulic properties of layers

Layer name	Saturated permeability	Sat. volumetric water content
Vegetation cover	6.5×10^{-8} m/s	0.5
Colluvium- silty sand	1×10^{-6} m/s	0.52
Completely weathered rock	1×10^{-8} m/s	0.4
Slightly weathered rock	1×10^{-9} m/s	0.23

The final volumetric water content curves selected for each layer are presented in figure 5.5 while, final soil water characteristic curves of each layer are presented in figure 5.6.

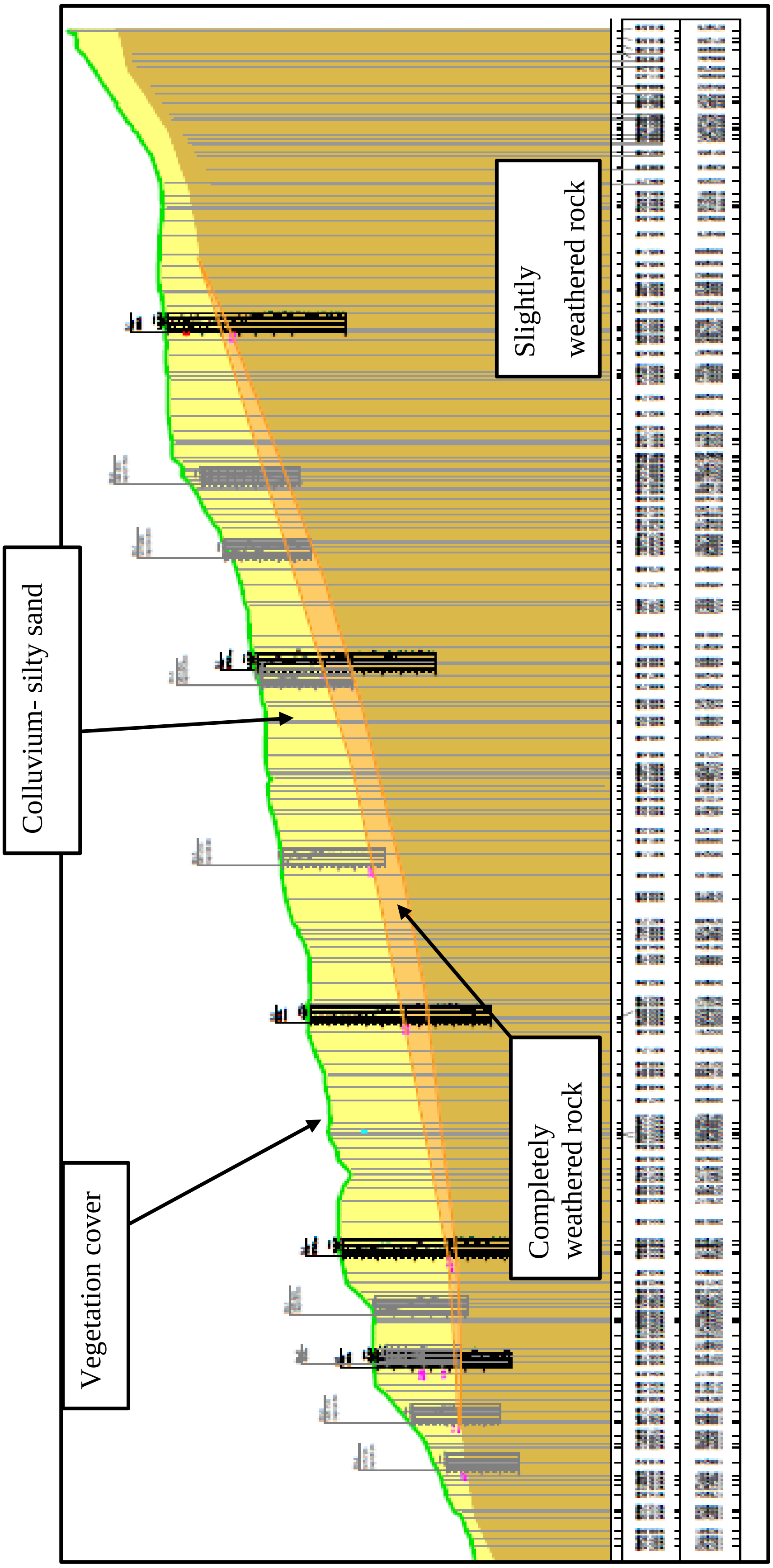
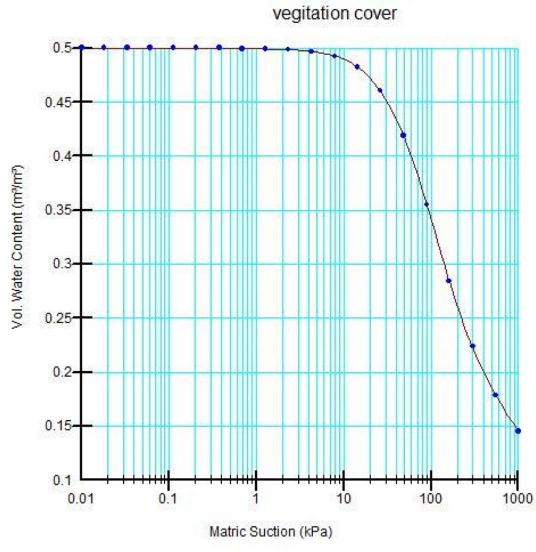
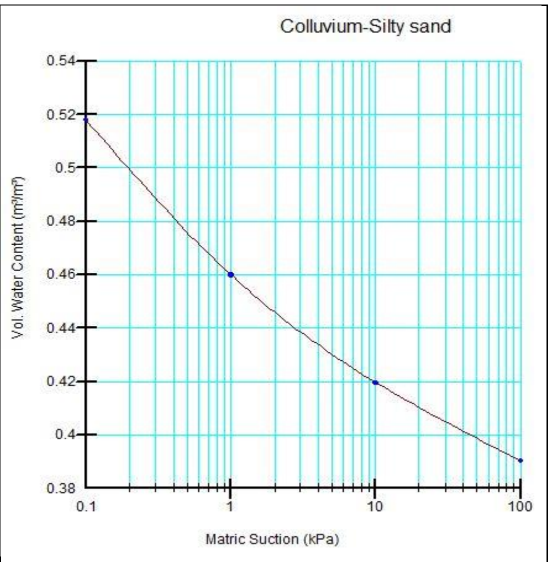


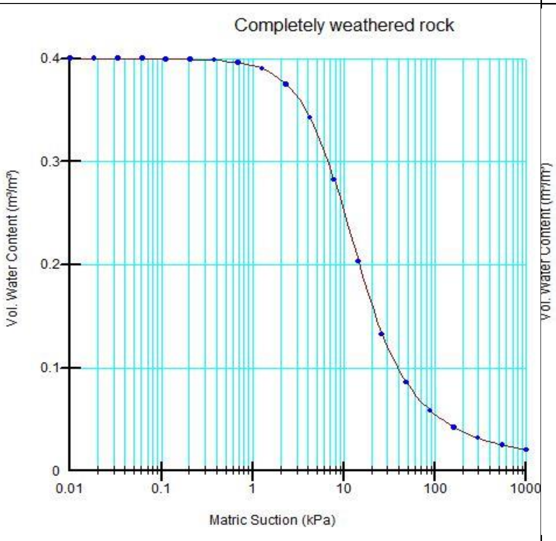
Figure 5.4: Selected layers for soil profile along CS1



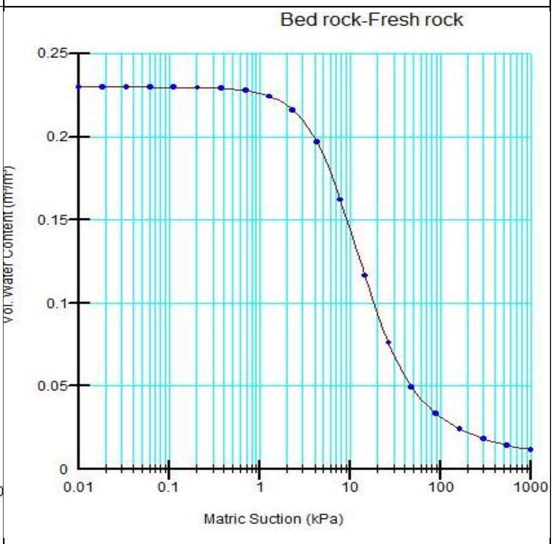
Vegetation cover (sample curve of clay)



silty sand-colluvium (sample curve of silty sand)

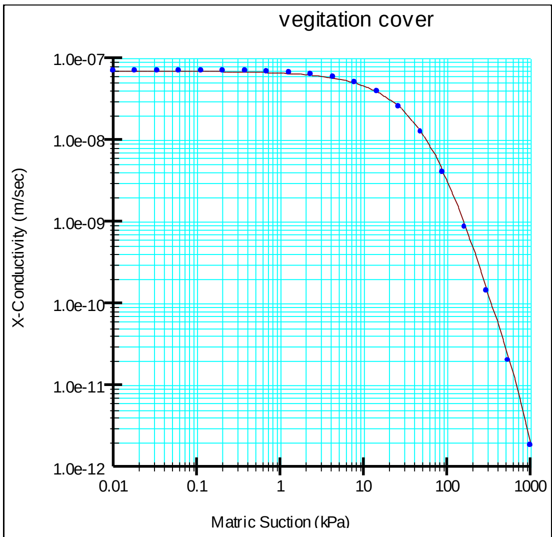


Completely weathered rock (sample curve of silty sand)

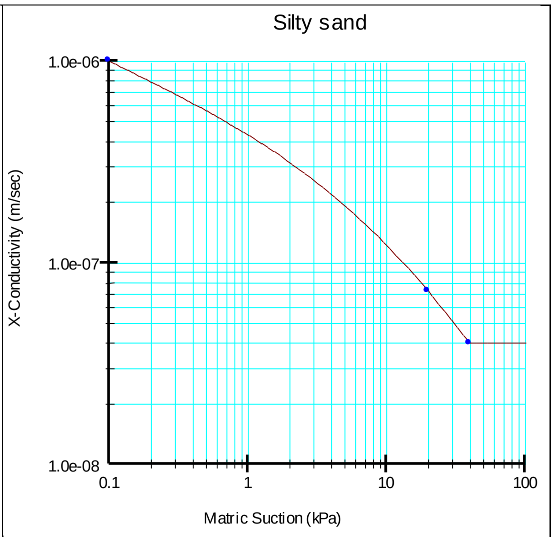


Slightly weathered rock (sample curve of silty sand)

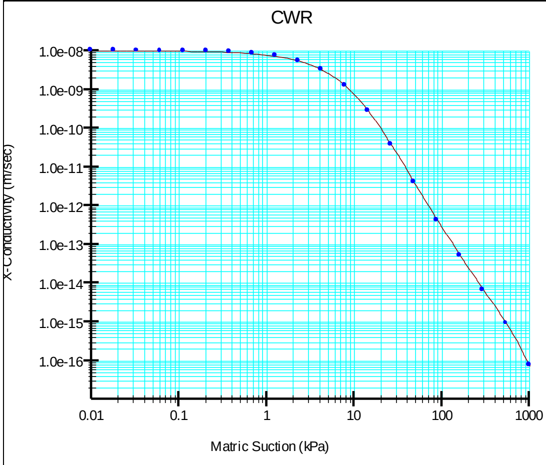
Figure 5.5: Soil water characteristics curves for Soil layers



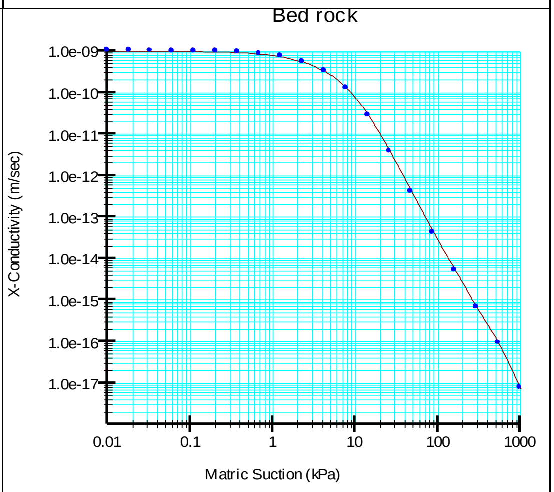
Vegetation cover (sample curve of clay)



silty sand-colluvium (sample curve of silty sand)



Completely weathered rock (sample curve of silty sand)



Bedrock (sample curve of silty sand)

Figure 5.6: Variation permeability with matric suction (Source: sample curves of SEEP/W software)

5.5 Shear Strength Parameters of Soil Layers

The shear strength parameters obtained through the laboratory tests lead to high factor of safety values for every slip surface, which are more than the 1.2, indicating those movements are unlikely to occur (figure 4.2). Therefore, a critical water level was chosen from the monitoring graph to determine the shear strength parameters from the back analysis procedure. The shear strength parameters were adjusted until FOS approaches 1.0 for the most critical slip surface (J4) as shown in table 5.3. Since the movements of extensometers were recorded 30.5 days after the selected rainstorm, the water level after 30 days was used to analyze the parameters. Figure 5.7 indicates the subsurface profile used for the analysis.

Table 5.3: Shear strength parameters of soils obtained from back analysis

Layer name	Color code	Unit weight	Cohesion	Friction angle
Colluvium- silty sand		15 KN/m^3	5.25	17
Completely weathered rock		18 KN/m^3	8	20
Bed rock		20 KN/m^3	20	35

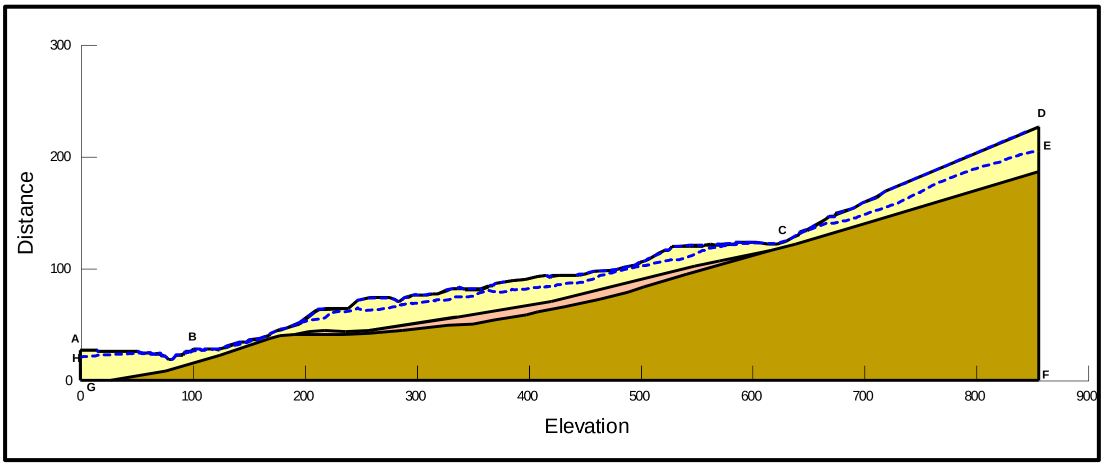


Figure 5.7: Critical Water level occurred after 30.5 days of prolonged rainfall event

Then the factor of safety values for the condition of high-water level were calculated for each slip surface to ensure the all slip have factor of safety lower than the selected design value of 1.2.

5.6 Verification of Seepage Model before installation of rectifications

For the verification of the seepage analysis, the ground water level fluctuation of the landslide was used as an indicator. The initial analysis was carried out for the critical situation without remedial measures until obtaining the groundwater level. Therefore, some adjustments were carried out on hydraulic properties in each soil layer to obtain a peak groundwater level similar to the monitoring data.

Monitoring graphs of the water table at three locations (BH 6, 7, 9) were used in this context. It is assumed that the pore pressure distribution below the groundwater table is positive and above is negative in all unsaturated soils. The negative pore water pressure was limited to -5 kN/m^2.

With the results of the analyses, some adjustments were carried out for hydraulic conductivity and the saturated water content of each soil layer. As shown in figure 5.8, the groundwater table was obtained after 67 days of prolonged rainfall was compared with the highest recorded groundwater table from monitoring at the site. The maximum difference of 5m height was obtained only at the borehole no.9 location, computed value is lower than the actual water table. Other observation points indicated slight variations from the monitored water table. The possible explanations for the deviation occurred at the borehole location 9 are as follows.

The borehole 9 locates near the borehole 5 where the rock layer is found at shallow depth. Therefore, the earlier plan of drainage well locations had to be shifted from the landslide axis. Water easily accumulates in this location as the flow is restricted by the shallow bedrock.

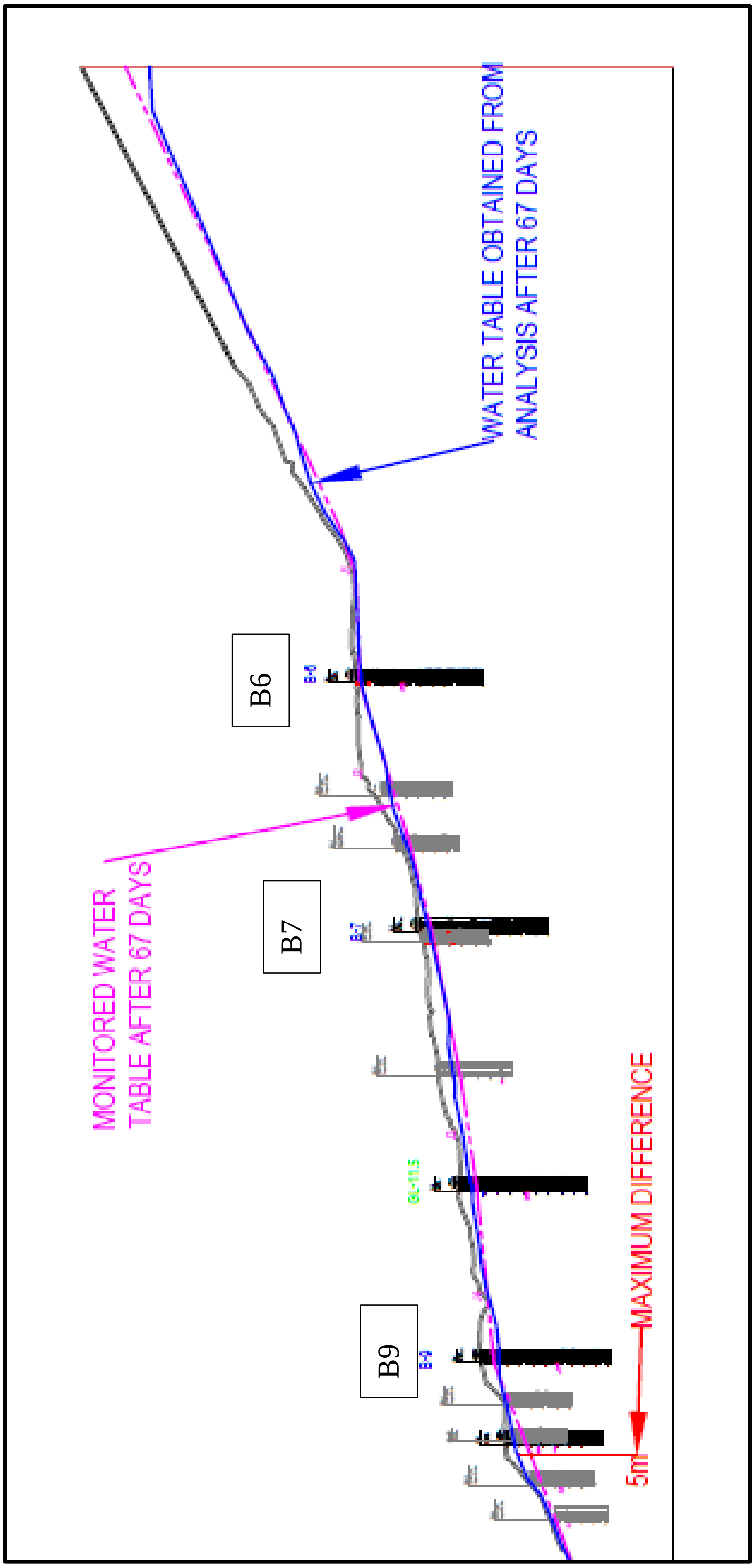


Figure 5.8: Comparison of ground water table between simulation and observed data

The rise of the groundwater table was also compared at each three borehole locations with respect to monitoring data for the whole time period of rainfall as given in figure 5.9, 5.10 and 5.11.

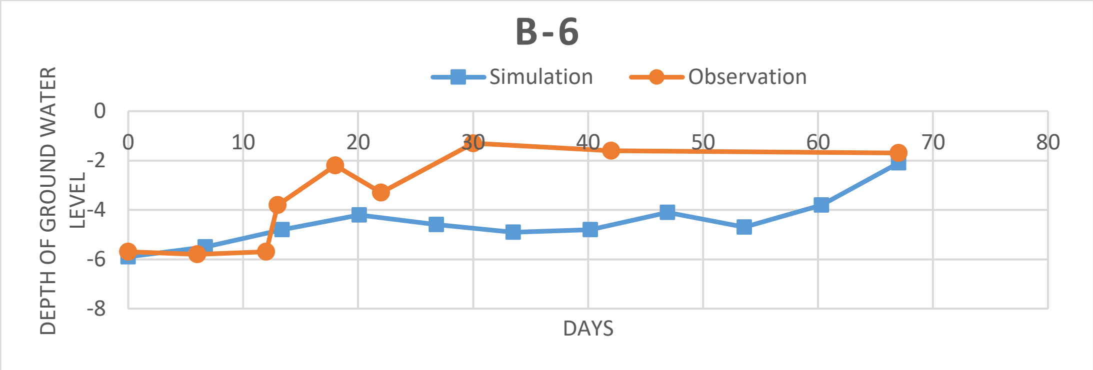


Figure 5.10: Ground water table comparison at Borehole no.6

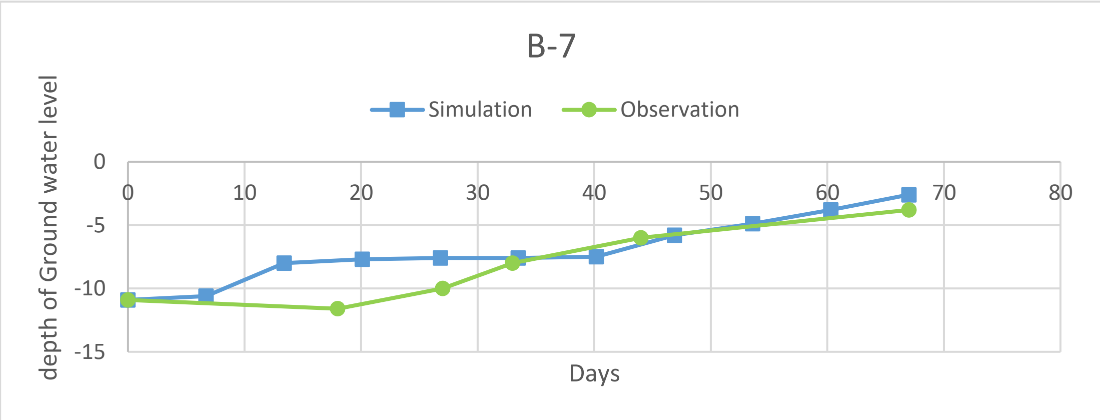


Figure 5.9: Ground water table comparison at Borehole no.7

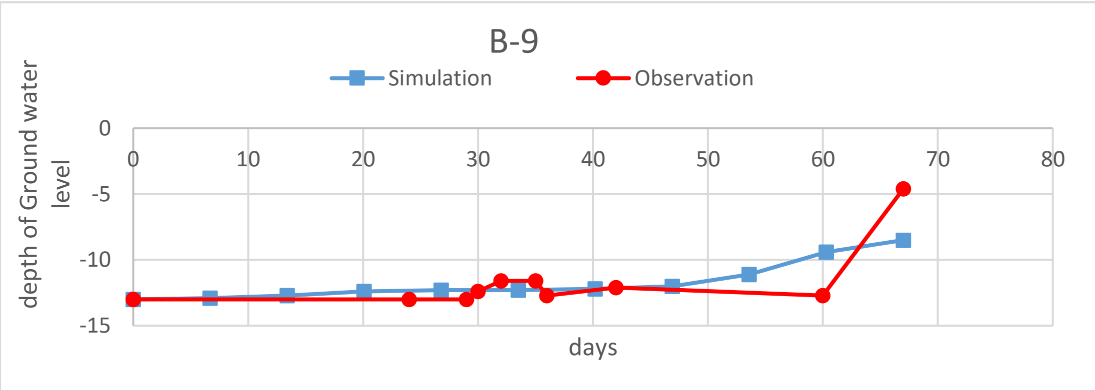


Figure 5.11 : Ground water table comparison at Borehole no.9

Above graphs show slight variations at the middle part of each graph. Especially at borehole 6 and 9 graphs show slight variations from the monitoring data at each location. This can happen due to spatial variability that exists in hydraulic properties in the ground even though earlier assumptions were made as one value for each layer. Especially the aquifers that exist on the ground provide clear evidence on the difference of characteristics such as hydraulic conductivity within the same layer.

Since the final water levels obtained through the simulations are fairly close to the monitored water levels and all curves follow the shape of the monitored water level fluctuations, it can be concluded that this model can be used to predict the pore water pressure generated in actual soil profile.

Conclusively, the proposed model can be used to predict the ground water behavior for existing situation and the forecasting the landslide behavior under different rainfall.

5.7 Results of Back Analysis before construction of counter measures

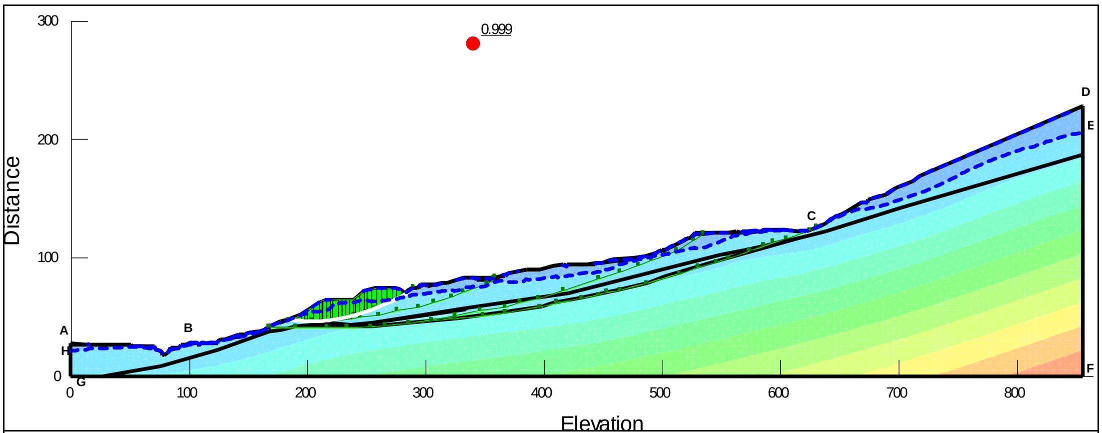
After the verification of the seepage model, the generated pore water pressure was incorporated with slope model. Then back analysis for the critical water level were proceeded and results were obtained for all four slip surfaces as follows. The groundwater table generated by the model after 30 days were used as the critical water level. Obtained factor of safety values for the landslide through the back analysis are presented in table 5.4.

Table 5.4: Factor of safety values for critical water level

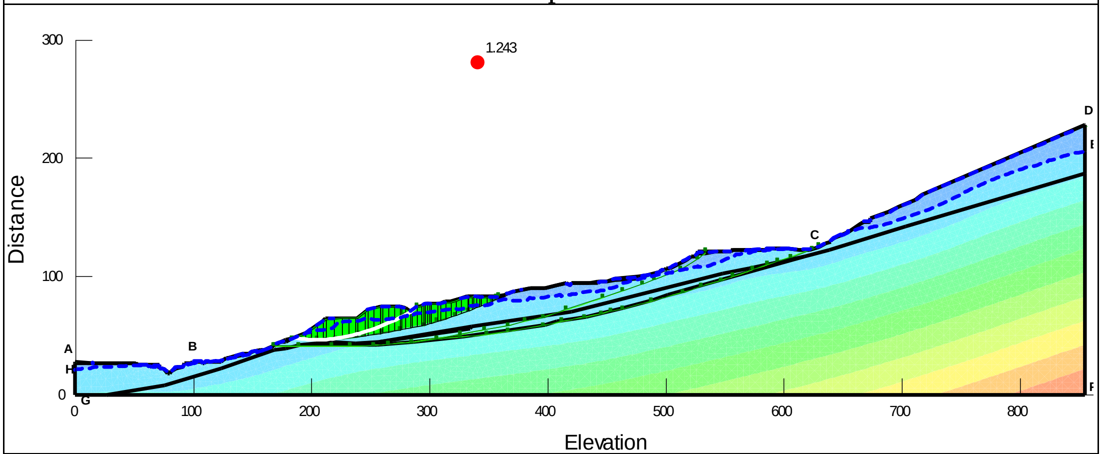
Slip surface	Factor of safety for critical situation
J 4	0.999 (fig. 5.12.a)
J 3	1.243 (fig. 5.12.b)
J 1	1.249 (fig. 5.12.c)
J 2	1.299 (fig. 5.12.d)

FOS $\simeq$ 1 achieved for CWL

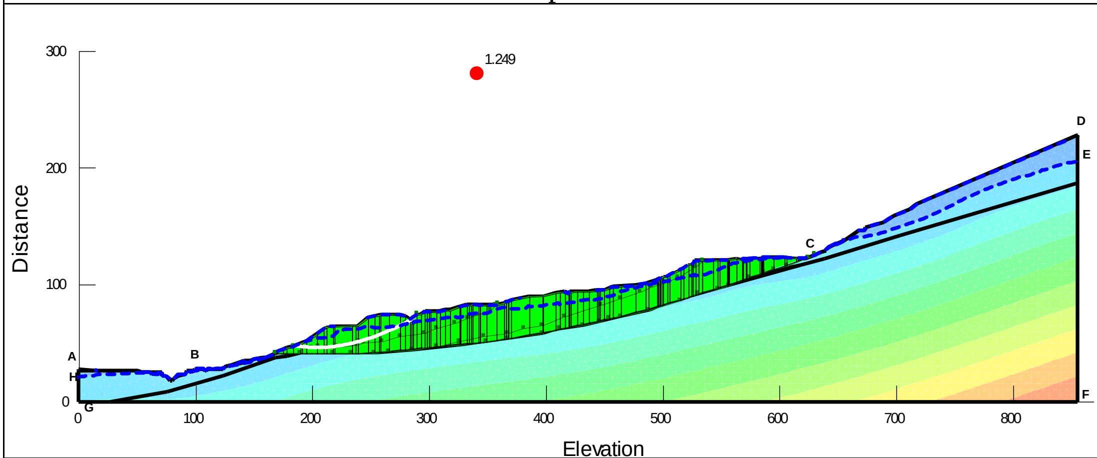
It shall be noted that there is a slight variation of two water levels obtained through the analysis and monitoring data, which is negligible (figure 5.9, 5.10, 5.11). The results are presented in figure 5.12.



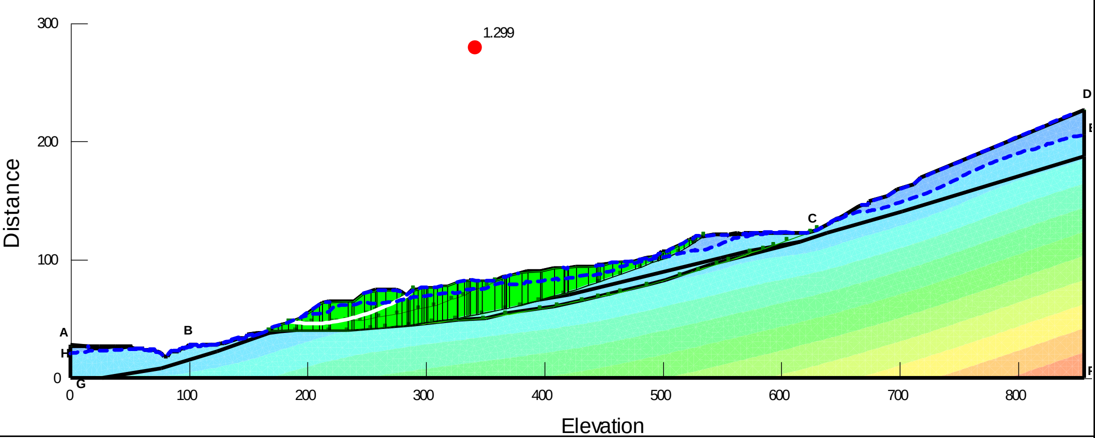
a. Critical water level and slip surface 4



b. Critical water level and slip surface 3



c. Critical water level and slip surface 1



d. Critical water level and slip surface 2

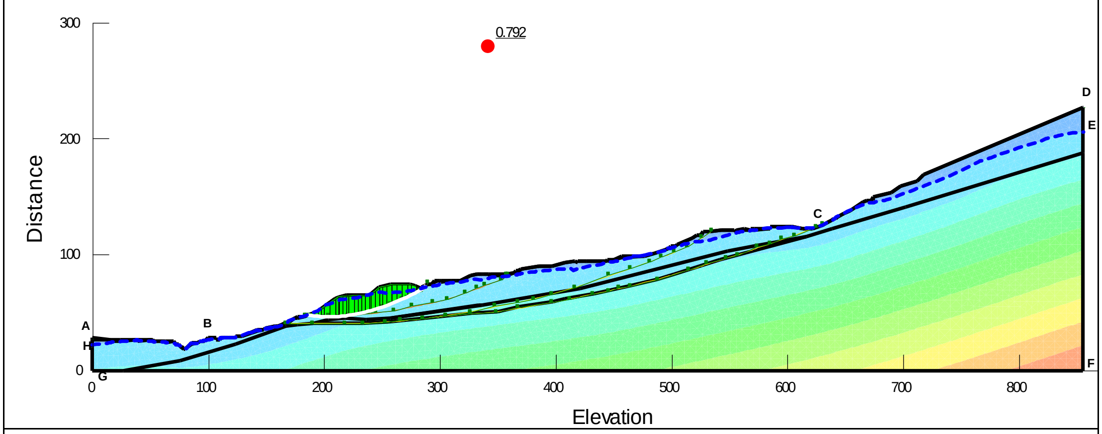
Figure 5.12 : Factor of safety values for each slip surface at the critical water level (obtained through Geoslope SLOPE/W 2012 software)

After the establishment of shear strength parameters from critical water level, the model was used to calculate the factor of safety values for high water level occurrence. The ground water level obtained after 67 days was used keeping other parameters unchanged. Obtained factor of safety values for each slip surface are presented in table 5.5.

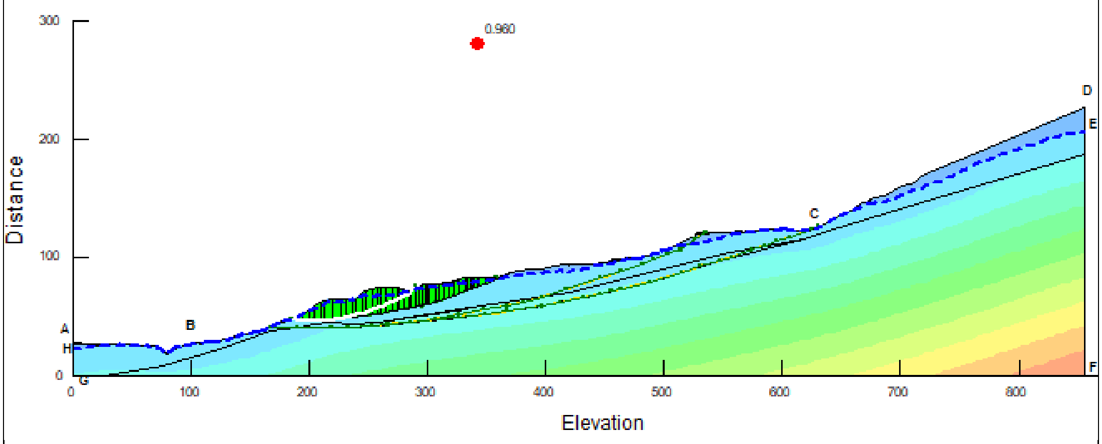
Table 5.5: Factor of safety values for high water level

Slip surface	Factor of safety for high water level occurrence
J 4	0.792 (fig. 5.13.a)
J 3	0.960 (fig. 5.13.b)
J 1	1.063 (fig. 5.13.c)
J 2	1.064 (fig. 5.13.d)

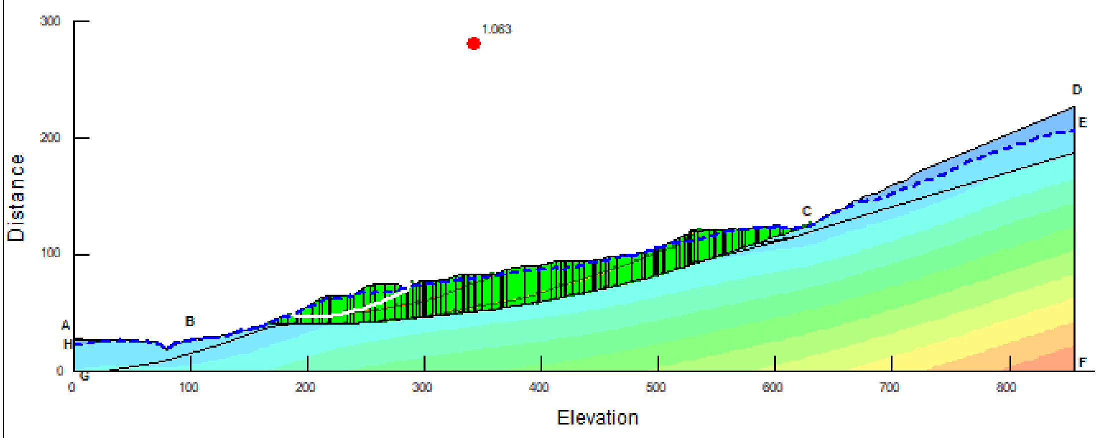
The results are presented in figure 5.13.



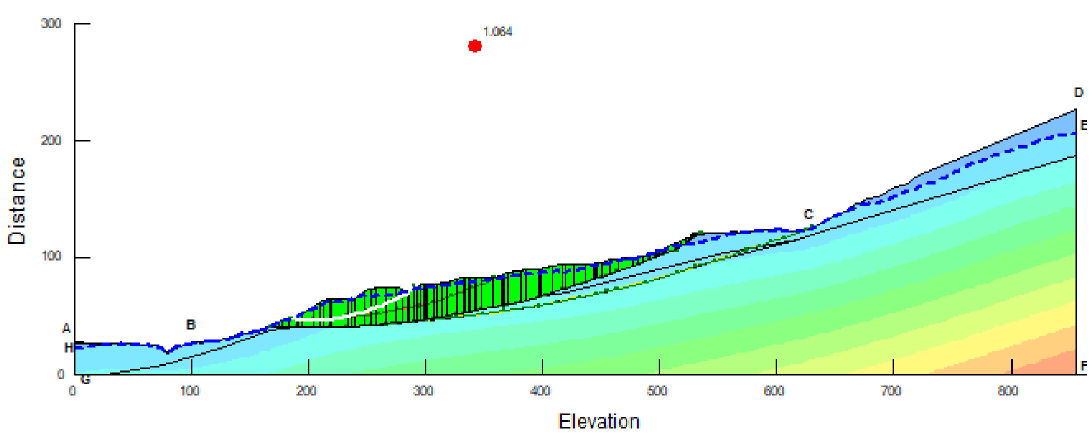
a. High water level and slip surface 4 (J4)



b. High water level and slip surface 3 (J3)



c. High water level and slip surface 1 (J1)



d. High water level and slip surface 2 (J2)

Figure 5.13 : Factor of safety values for each slip surface at the high-water level (obtained through Geoslope SLOP/W 2012 software)

The above stability results clearly illustrates that the landslide responds to the level of the groundwater table with high sensitivity. The factor of safety is drastically reduced even for a slight variation of the water table, making a threat of failure.

Therefore, lowering the groundwater table contributes to decreasing the rate of lowering the factor of safety, thereby keeping the factor of safety at an acceptable level. Moreover, it can be concluded that at the high-water level occurrence, the above landslide is highly active and vulnerable. Factor of safety of some of the slip surfaces are only and slightly greater than unity, making landslide is marginally stable. The records of monitoring instruments verify these results by showing clear signs of movements at all the slip surfaces. Moreover, all four blocks are below the design safety factor and need to rectify by applying appropriate countermeasures.

6. CHAPTER 6: ANALYSIS OF RECTIFICATION MEASURES

After the calibration of the parameters of each soil layer, the model was used to analyze the effects of rectification measures. This chapter describes the procedure of analyzing rectification measures with some assumptions for a real rainfall situation and its results.

6.1 Modelling of Rectification Measures

The rectification measures implemented at the site have a three-dimensional complex nature that cannot be easily simplified to a two-dimensional model. As an example, the site consists of drainage wells along the main axis with a pan structure of horizontal drains fixed at the bottom. Nevertheless, each drainage well is connected with a discharge boring to the other well located below, therefore it was a very challenging task to model the subsurface drainage system.

1. The whole subsurface drainage system is inter-connected with the soil profile therefore flow cannot release freely from the system applying some boundary conditions.
2. Since the whole subsurface drainage system is inter-connected, all pipes, horizontal drains and drainage wells are subjected to varying pressures according to the external environment. Furthermore, the pressure inside a pipe would be governed by the other component of the system

The following assumptions were made for the simplification of the structure to model in a two-dimensional profile.

1. a permeable layer is modeled in the profile to represent the effects of pan structure of horizontal drains on the width of landslide.
2. The effect of the water infiltration into drainage well directly from soil is negligible compared to the landslide width, but specially modeled to represent

the critical situation that can occur at the site (The infiltration of water from drainage well to soil also can take place in the down wells).

First, the subsurface drainage system was modeled along the cross-section, and some of the soil regions were redrawn keeping material properties unchanged. Then the 10m mesh was generated for the profile to carry out the seepage analysis through finite element technique. It should be noted that most of the rectification works as well as monitoring instruments have been closely located to the main axis of the landslide. Therefore, these monitoring instruments such as groundwater level gauges are directly affected by a rectification measure constructed nearby.

Accordingly, the whole subsurface drainage system including drainage wells, lateral drains, and discharge pipes was modeled as regions instead of applying boundary conditions to represent the actual site conditions. This new strategy was used to model the unique nature of rectifications of the case study. Suitable material properties such as hydraulic conductivities were assigned by proceeding back analysis which is described later in this chapter.

The computed ground water level was compared with monitoring data available after the construction of rectification measures. Adjustment of hydraulic parameters were carried out in an iterative procedure. The final combination of parameters was selected, to closely represent the observed ground water table. The seepage model with rectification measures was verified in this way and corresponding pore water pressure regime was obtained.

Then, other rectification measures such as ground anchors, earth removal works, counterweight embankment works were modeled in the cross section and corresponding increase of factor of safety values were calculated for each slip surface. Filling material properties were found from literature and anchor properties were collected from design data. The procedure for the analysis of rectification measures is given in figure 6.1.

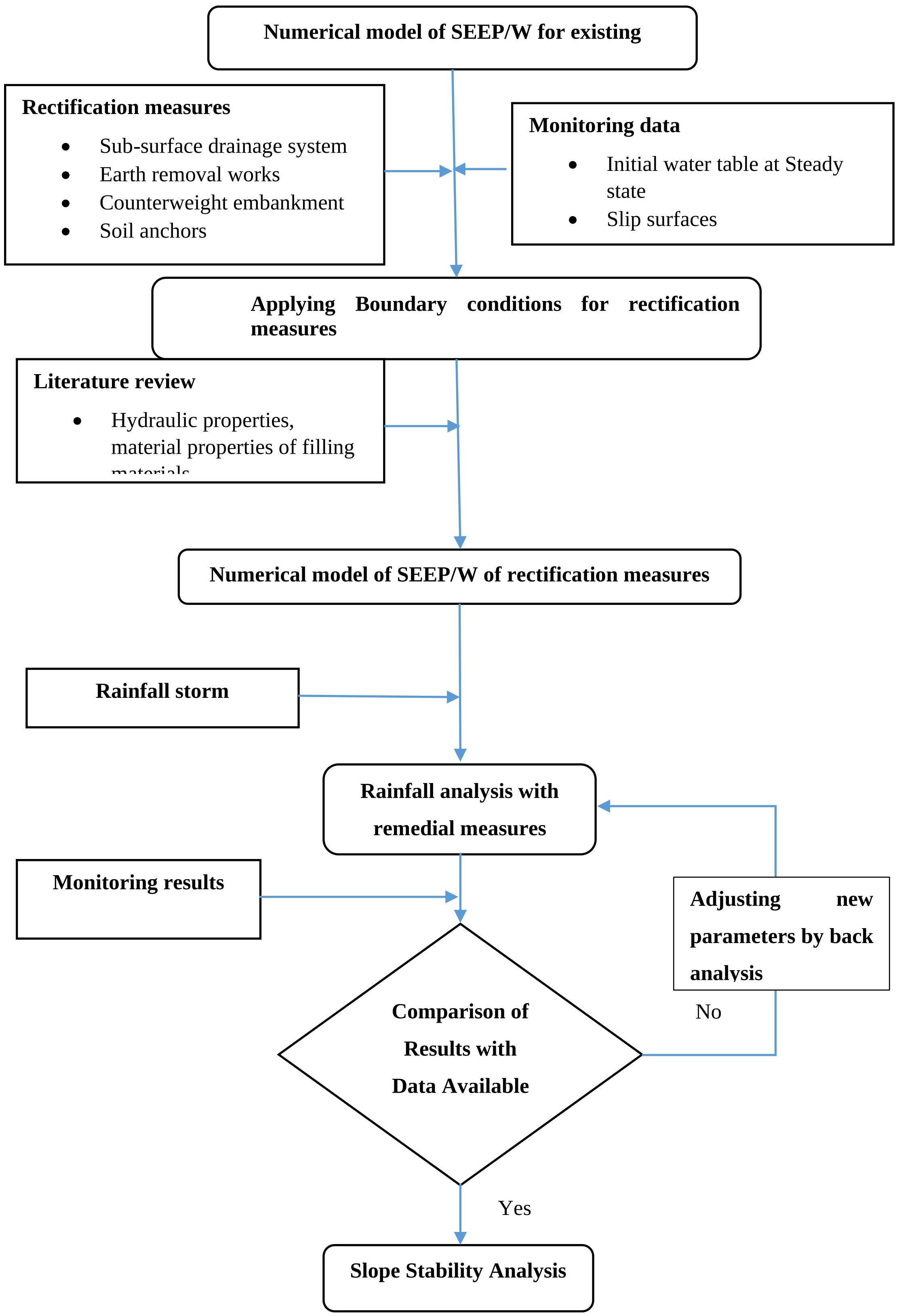


Figure 6.1: Analysing Procedure of the effect rectification measures for the stabilization of Kahagolla Landslide

6.2 Boundary Conditions for the Model with Rectification Measures

Modeling of rectification measures by applying boundary conditions was a difficult task due to the complex nature of the drainage system. The whole subsurface drainage system is connected and acts as a system of pipes that can carry water top to the bottom, rather than directly removing water from underground. This causes an accumulation of underground water at the toe area of the landslide and a sudden rise of the water table in this region. Moreover, many conditions such as pressures and flows vary with time and rainfall, making the boundary conditions indeterminate. As an example, zero pressure condition never exists in a lateral drain when the drainage well is filled with water above its opening. Therefore, the boundary conditions applied to the model give unrealistic results when compared with the monitoring results after the construction works. Finally, a solution was found by modeling the subsurface drainage system as regions with more permeable materials.

The monitoring graphs of water levels were again used for the back analysis procedure. One of the advantages of this model is, it allows to change of water pressure inside the pipes and drains while any fluctuations take place in drainage wells. And also, the flows through these pipes can vary according to the water level fluctuation and finally reach a steady condition. A possibility was discovered that the water can flow in the transverse direction at the toe of the landslide. That means water can infiltrate again into the soil through both lateral drains and drainage wells. Therefore, modelling as more permeable regions gives a realistic approach to the problem.

Discharge Pipes were modeled with zero seepage boundary conditions to separate flow through the pipe from the surrounding soils. The figure 6.2 illustrates the model with new region and boundary conditions that were used to analyze the rectification measures.

During the stages of constructions, some of the monitoring instruments were damaged. Therefore, Data from the post monitoring stage could not be gathered in some locations. Instead of that, some check boreholes were established at selected locations and data were gathered manually.

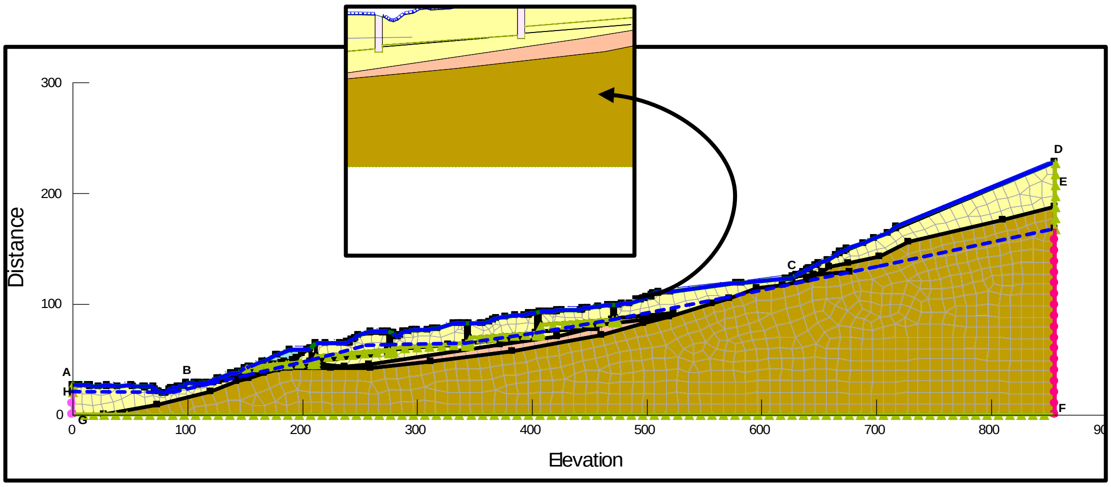


Figure 6.2: Boundary conditions applied to the profile with rectification measures

Hyetograph corresponding to a rainfall occurred on 2018.05.01 to 2018.06.30 was selected for the analysis of rectification measures. It should be noted that the constructions of the subsurface drainage system were not fully completed at that moment. Therefore, records of water level gauge located at Borehole 9 were selected, assuming others have no difference from the steady-state. Figure 6.3 represents the rainfall hyetograph that was used for the analysis.

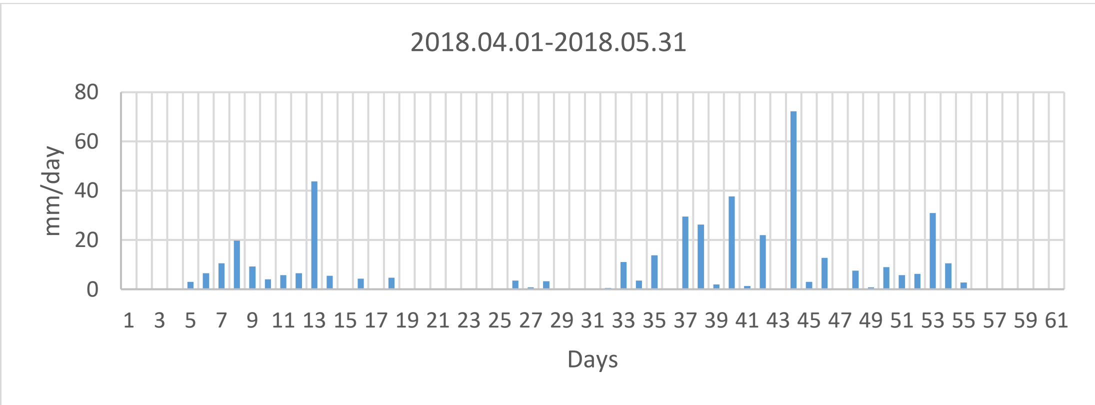


Figure 6.3: rainfall hyetograph used after the construction of rectification measures

Table 6.1: Detail of Boundary Conditions

Name of the Boundary condition	Type of boundary condition	Stating point		End point	
		X coordinate	Y coordinate	X coordinate	Y coordinate
GH-Left Head	Head =20 m	0.000	0.000	0.000	20.000
EF-Right Head	Head =205 m	856.000	205.00	856.000	0.00
AH-Zero seepage	Total flux=0 m^3	0.000	27.500	0.000	20.000
DE-Zero seepage	Total flux=0 m^3	856.00	227.500	856.000	205.000
GF- Zero seepage	Total flux=0 m^3	0.000	0.000	856.000	0.000
ABCD-Rainfall 1(with potential seepage)	Unit flux= func.	0.000	27.500	856.00	227.500
Discharge Boring 1-Zero seepage (final end to drainage well 1)	Total flux=0 m^3	151.1	41.6	208.2	47.5
	Total flux=0 m^3	151.5	41.8	208.2	47.7
Discharge Boring 2- Zero seepage (drainage well 1 to 2)	Total flux=0 m^3	211.6	51.2	275.3	57.1
	Total flux=0 m^3	211.6	51.4	275.3	57.3
Discharge Boring 3 -Zero seepage (drainage well 2 to 3)	Total flux=0 m^3	278.7	59	342.1	63.7
	Total flux=0 m^3	278.7	59.2	342.1	63.9
Discharge Boring 4- Zero seepage (drainage well 3 to 4)	Total flux=0 m^3	345.5	68	404	73
	Total flux=0 m^3	345.5	68.2	404	73.2
Discharge Boring 5-Zero seepage (drainage well 4 to 5)	Total flux=0 m^3	407.4	79.7	470.4	85.1
	Total flux=0 m^3	407.4	79.9	470.4	85.3

6.3 Hydraulic and Material Parameters of Soil

Table 6.2 presents the final values of hydraulic parameters used to analyze drainage system those were adjusted from back analysis.

Table 6.2: Hydraulic properties used for subsurface drainage system

Layer name	Color code	Saturated permeability
Lateral drains		$1x10^{-5}$ m/s
Discharge pipes and well		$1x10^{-4}$ m/s

New material was introduced for the modeling of counterweight embankment, the soil bags filled with the soil removed from the upper area. Since the soil bags are included in the design, the shear strength parameters also depend on the properties of soil bags. The parameters found in the literature review and the design report are given in table 6.3. Sub soil profile used for the analysis is indicated in figure 6.4.

Table 6.3: Shear strength parameters of soils and other layers

Layer name	Color code	Unit weight	Cohesion	Friction angle
Soil bags		16 KN/m^3	14	25
Colluvium- silty sand		15 KN/m^3	5.25	17
Completely weathered rock		18 KN/m^3	8	20
Bed rock		20 KN/m^3	20	35

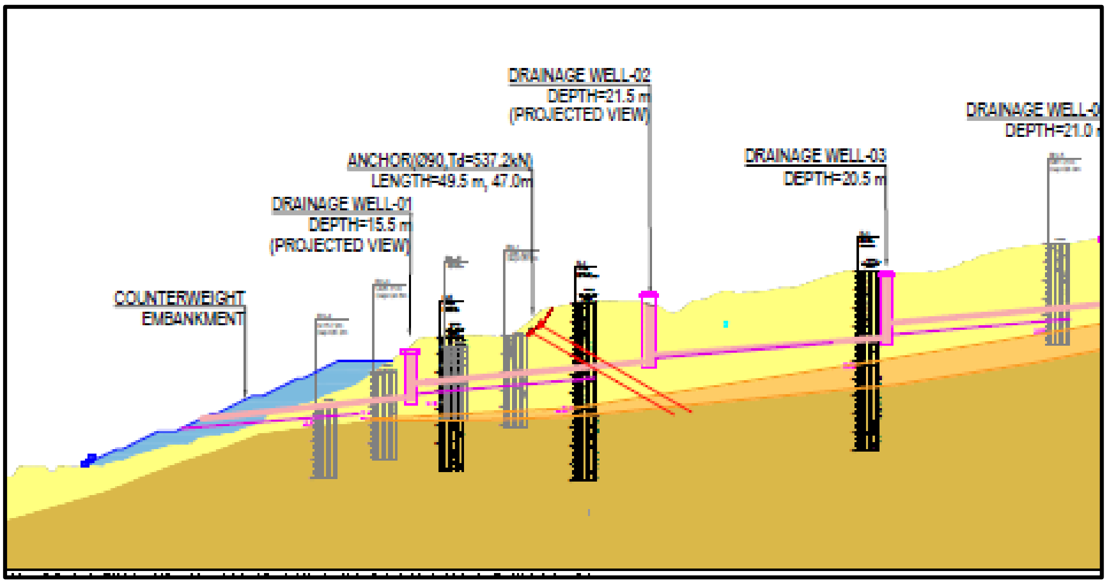


Figure 6.4: Part of the Soil profile along CS 01 used for analysis of rectification measures (Layers were according to table 6.2 and 6.3.)

The shear strength parameters of materials used for the modeling of rectification measures were assigned as same as the silty sand layer. Therefore, the effect of the drawdown of the water table will be only accounted for here in the construction of subsurface drainage works.

6.4 Numerical Simulation for Subsurface Drainage System

After the analysis of the existing situation and verifying the model, another model was created with rectification measures. Then the next rainstorm was used to analyze the groundwater level behavior and verifications were carried out with monitoring data. The comparison results of two water table at the borehole no. 7, and other two check boreholes are given in figure 6.5, 6.7 and 6.8.

The comparisons of two water tables in above three figures show slight variations during 61 days. That can happen due to spatial variability that exists in hydraulic properties in the ground even though we assume one value for each layer. Generally, the water table has been drawn down according to the observation made at the site.

The different gaps could be observed between two groundwater levels after the completion of rectification measures. The maximum gap of water tables was observed as approximately 2m from the highest record of each the borehole. This implies that even though the ground observations are highly sensitive to the rainfall records, simulations are less varying with the infiltrations.

However, the rising of the water table near the discharge end of the subsurface drainage system could be observed in the simulation. This feature is also visible in monitoring graphs due to groundwater accumulation at the toe area.

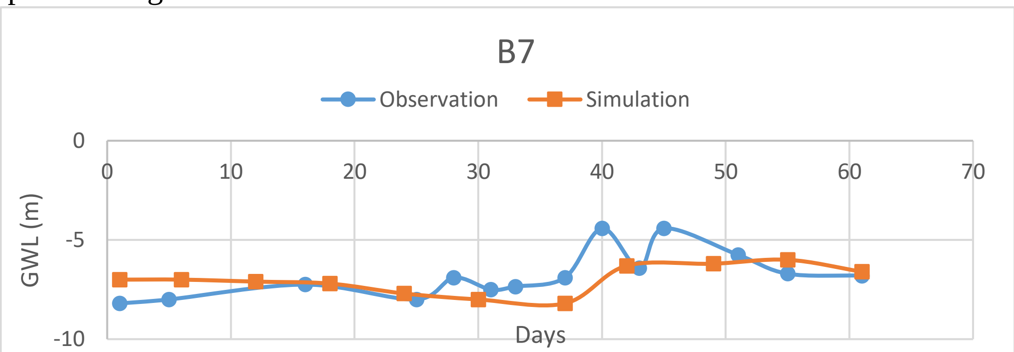


Figure 6.5 : Ground water table comparison at Borehole no.7 (Model 2)

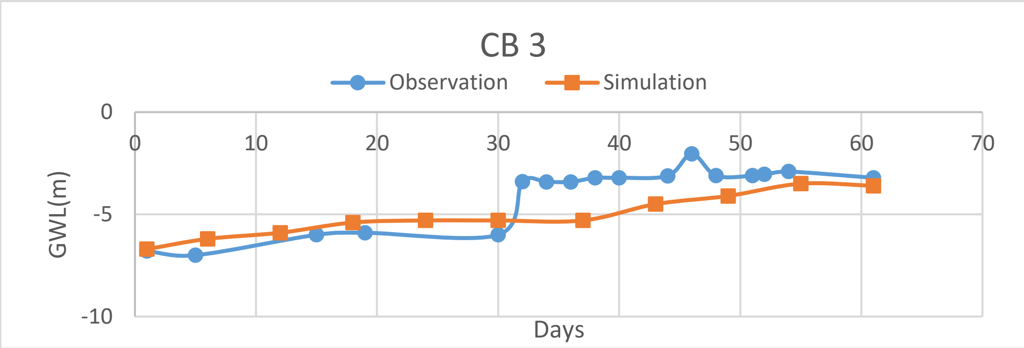


Figure 6.7 : Ground water table comparison at Check boring no. 3 near the drainage well 4 (Model 2)

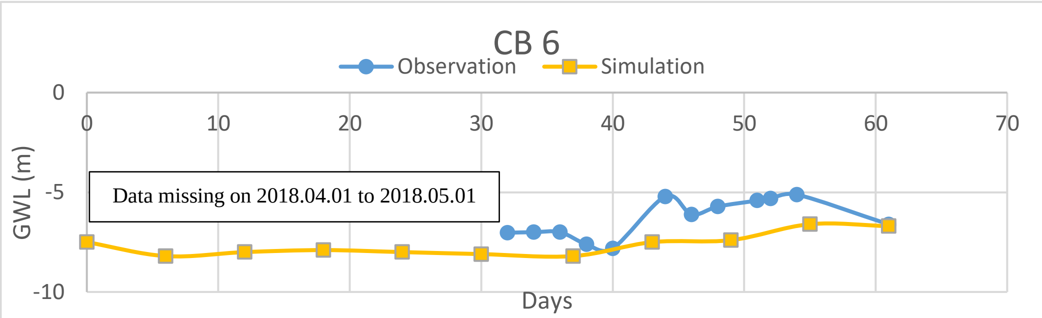


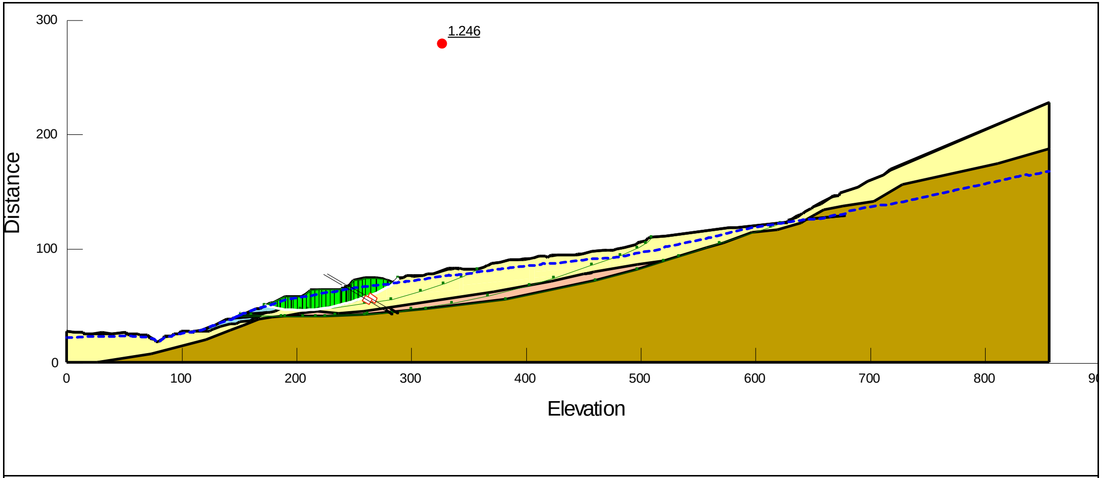
Figure 6.6: Ground water table comparison at Check boring no. 6 near the drainage well 2 (Model 2)

6.5 Results of Slope Stability Analysis for Rectification Measures

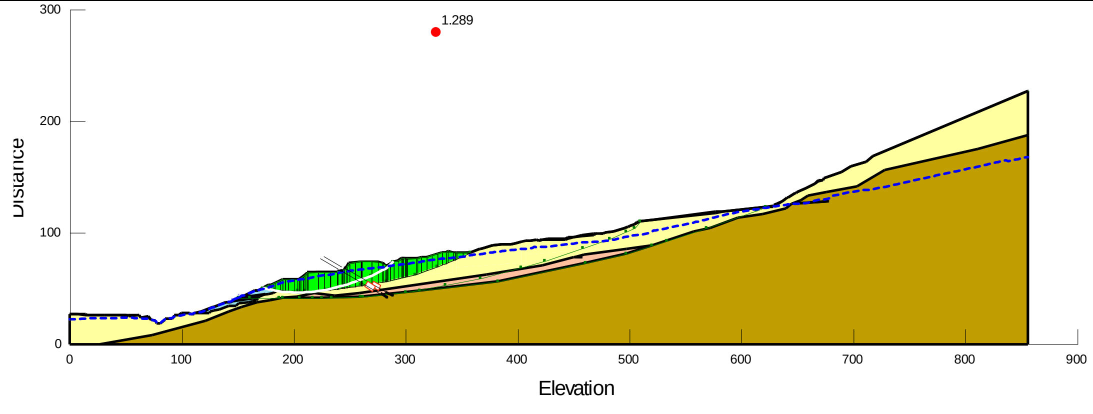
The pore pressure generated by the seepage model at the high-water level was used for stability analysis. The other countermeasures such as anchoring system, earth removal, and counterweight embankment were also modeled and the results were obtained. The factor of safety values obtained through the analysis is given in the table in 6.4 and figure 6.9.

Table 6.4:Factor of safety values for high water level after the construction of counter measures

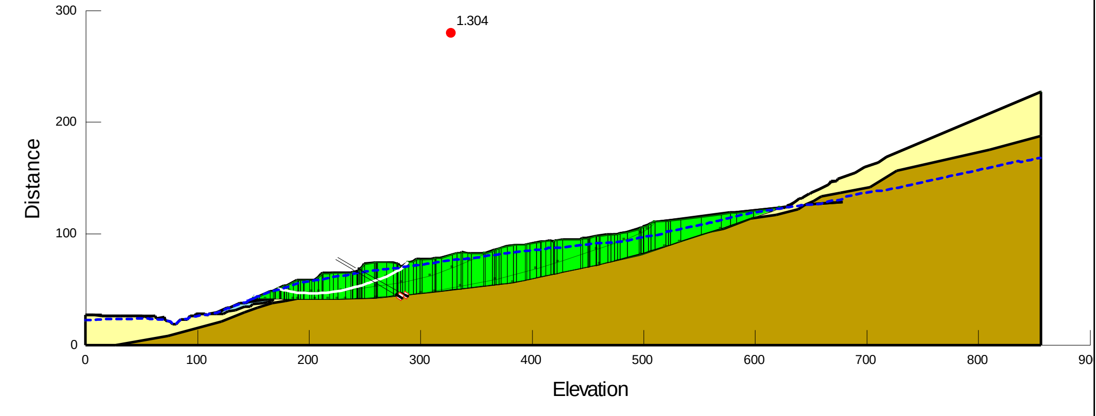
Slip surface	Factor of safety for high water level
J 4	1.246 (fig. 6.8.a)
J 3	1.289 (fig. 6.8.b)
J 1	1.304 (fig. 6.8.c)
J 2	1.511 (fig. 6.8.d)



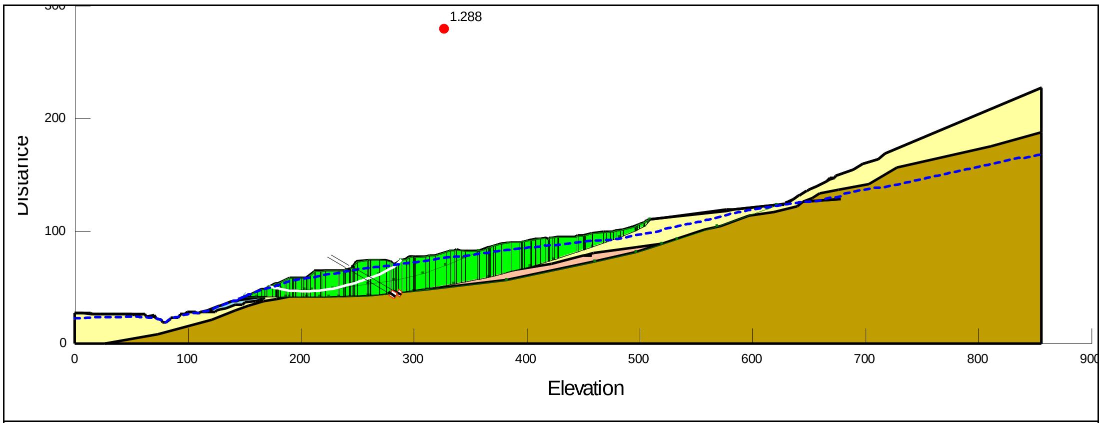
a. High water level in slip surface 4 after applying of countermeasures (47 days of rainfall)



b. High water level in slip surface 3 after applying of countermeasures (47 days of rainfall)



c. High water level in slip surface 1 after applying of countermeasures (47 days of rainfall)



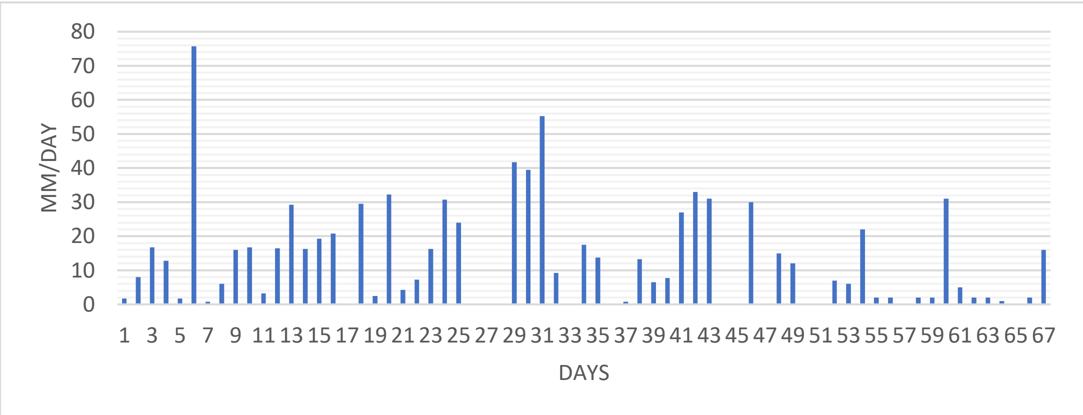
d.High water level in slip surface 2 after applying of countermeasures (47 days of rainfall)

Figure 6.8: Factor of safety values after construction of countermeasures

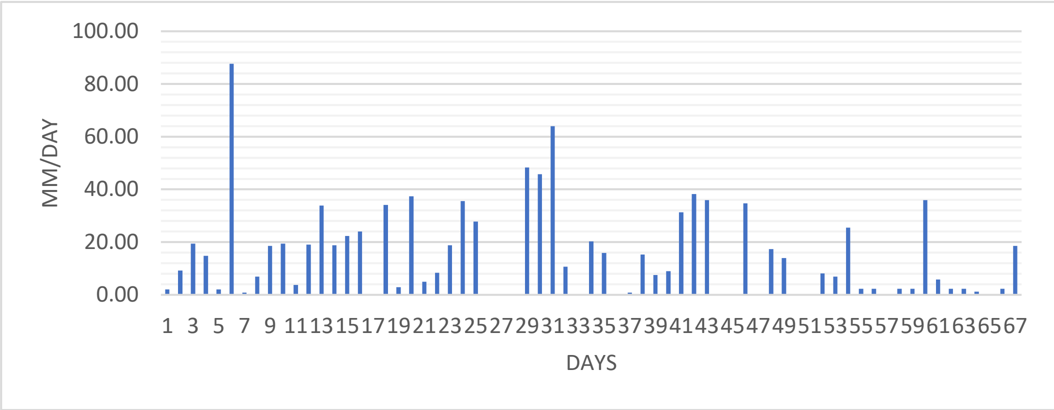
The results show that the factor of safety values remain above the design factor of safety (FOS des =1.2) which pairs with the site conditions for particular rainfall. However, a high rainfall event shall be also modeled to forecast the behavior of rectified landslide at a very critical rainfall situation. This can be used to assess the effectiveness of the rectified measures under future expected high prolonged rainfall situations.

7. CHAPTER 7: RESPONSE OF THE RECTIFIED SLOPE TO SEVERAL DESIGN RAINFALLS

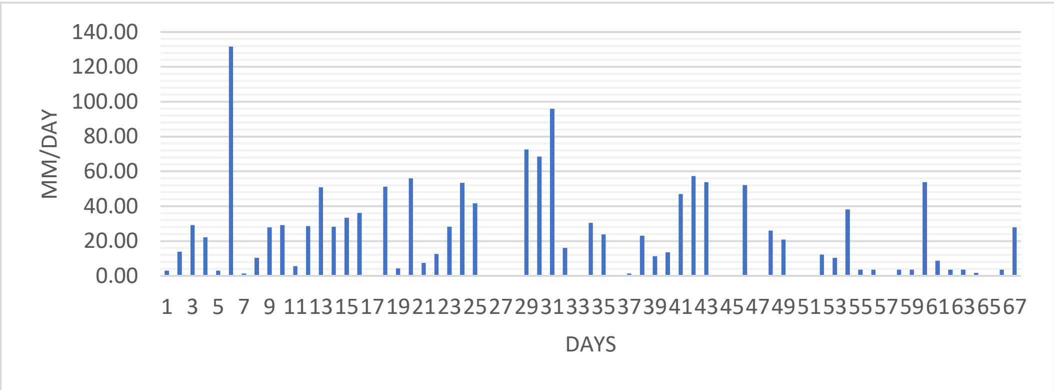
In order to assess the effectiveness of rectification measures, the influence under a high rainfall shall be modeled. Accordingly, several high rainfalls that can be expected at the second inter monsoon (October-November) including maximum rainfall occurred at the area (The same rainfall that was used to analyze the landslide movement in figure 5.3) were considered. The cumulative rainfall values were subjected to weighted distribution over 67 days using the rainfall in figure 5.3 and several design rainfalls were developed. Figure 7.1 illustrates the resulted design rainfalls selected to analyze the model. Then the generated pore water pressure regimes were used to calculate the factor of safety of each slip surface by incorporating with slope model. These values were compared with the design factor of safety as well as critical factor of safety (1.00) to find out whether it achieves a reasonable safety margin and the critical rainfall threshold.



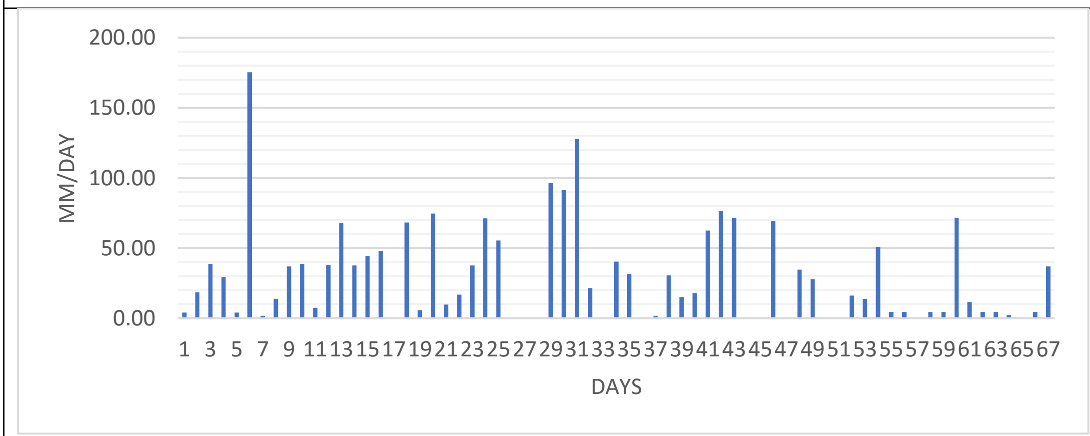
Design rainfall 1: Cumulative rainfall 900 mm



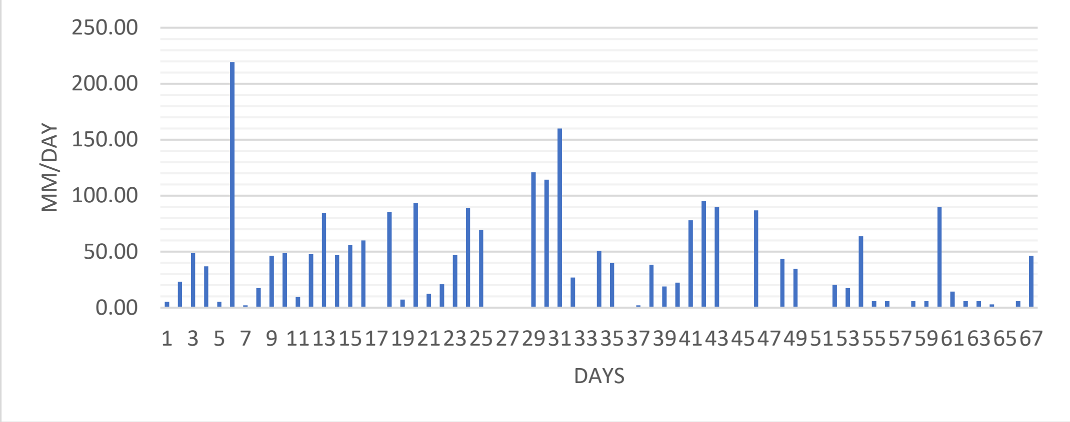
Design rainfall 2: Cumulative rainfall 1000 mm



Design rainfall 3: Cumulative rainfall 2000 mm



Design rainfall 4: Cumulative rainfall 3000 mm



Design rainfall 5: Cumulative rainfall 3500 mm

Figure 7.1: Several rainfalls selected for analysis after the construction of rectification measures

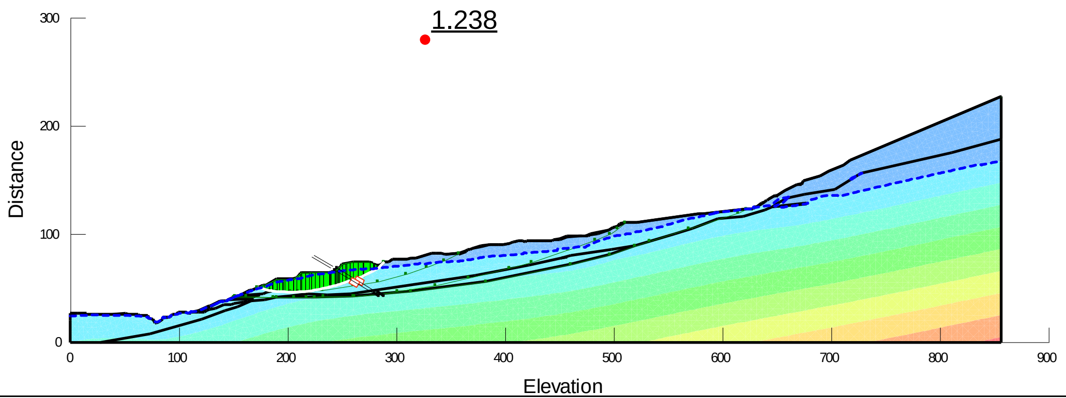
The corresponding safety factors achieved for each design rainfall are illustrated in table 7.1.

Table 7.1: Factor of safety values for high water level at several design rainfalls

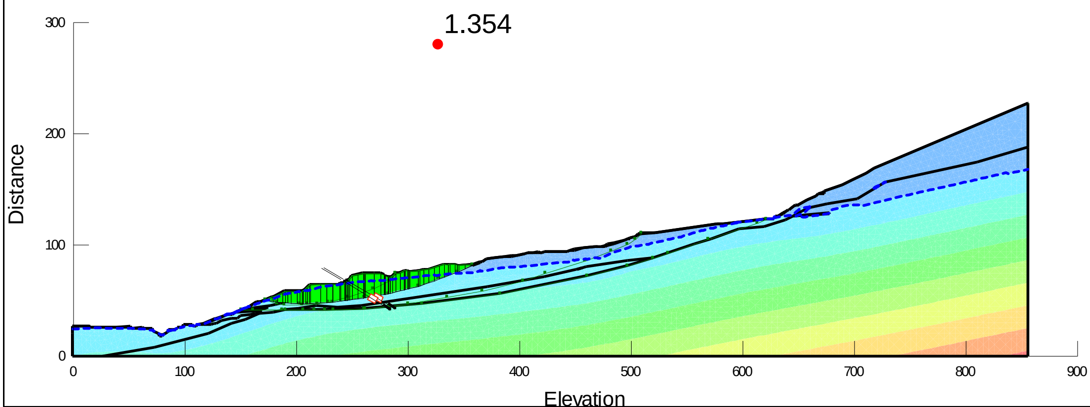
Cumulative design rainfall (mm) over 67 days	Factor of safety of slip surfaces (with mitigation measures)			
	J1 block	J2 block	J3 block	J4 block
863.75 (from 2015-09-20 to 2015-11-25)	1.394 (fig.7.2a)	1.419 (fig.7.2b)	1.354 (fig.7.2c)	1.238 (fig.7.2d)
950 mm (Design rainfall 1)	1.407	1.381	1.326	1.208
1000 mm (Design rainfall 2)	1.375	1.399	1.309	1.191
2000 mm (Design rainfall 3)	1.251	1.225	1.116	1.031
3000 mm (Design rainfall 4)	1.131	1.146	1.043	1.014
3500 mm (Design rainfall 5)	1.117	1.108	1.101	0.992

FOS >1.2   1.2> FOS >1.0   FOS <=1.0

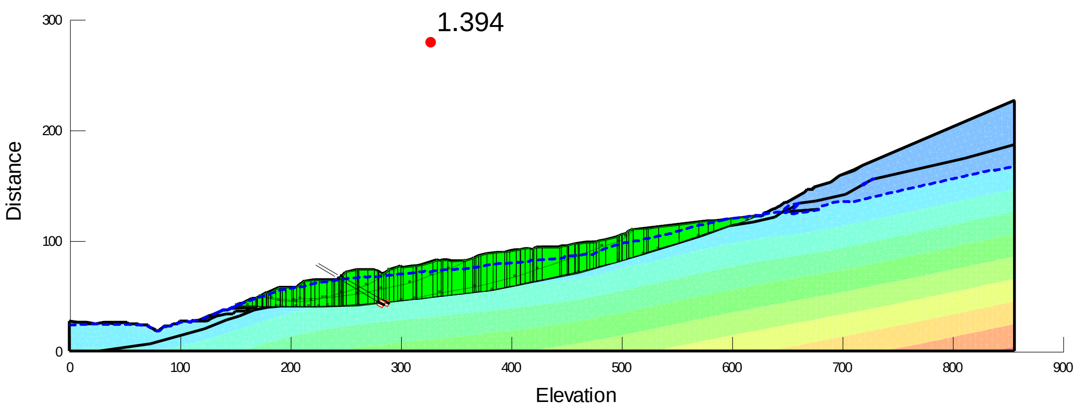
The corresponding safety factors achieved for each slip surface under cumulative rainfall of 863.75 mm are illustrated in figure 7.2.



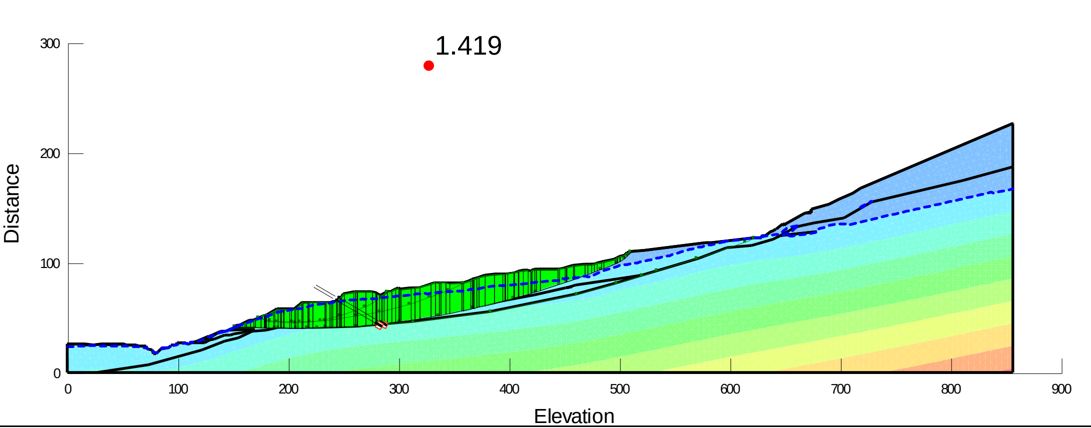
a. High water level in slip surface 4 after applying of countermeasures (67 days of rainfall 863.75 mm)



b. High water level in slip surface 3 after applying of countermeasures (67 days of rainfall 863.75 mm)



c. High water level in slip surface 1 after applying of countermeasures (67 days of rainfall 863.75 mm)



d. High water level in slip surface 2 after applying of countermeasures (67 days of rainfall 863.75 mm)

Figure 7.2: Factor of safety values after construction of countermeasures for highest rainfall record

As results shown in table 7.1, it can be concluded that the above rectification measures would be successful in stabilizing the landslide body, even under the worst-case scenario that has taken place in the past, thereby reducing the risk of failure drastically. It also reveals that the desirable factor of safety (FOS=1.2) lies in between 950-1000 mm cumulative rainfall events. However, activation of landslide movements will be shown in between 3000-3500 mm cumulative rainfall over 67 days which is not very likely to occur. Therefore, it can be concluded that the rectification measures are very effective that could occur the landslide movement up to 3000 mm cumulative rainfall over 67 days.

8. CHAPTER 8: CONCLUSION

8.1 Case Study with Back Analysis

Back analysis is widely used in slope stability studies when the absence of data or test data gives unrealistic results. It is generally accepted that back analysis gives more reliable results compared to the analysis with the laboratory test data, because it represents the average of properties which may vary over a large area. However, always there should be a way to verify the obtained results for this type of analysis and results should be interpreted carefully.

The case study was analyzed with several back analyses in the absence of detail investigations for some parameters. The results were adjusted with monitored data, which produces an effective way to improve the accuracy. Not only shear strength parameters, some hydraulic properties of soils were also able to estimate in this way. The results obtained from both seepage and slope stability models show the applicability of this method in finding a wide range of information, which represent the whole landslide mass. Moreover, this method is a prominent solution in finding thresholds of rectified landslides which are continuously subjected to various rainfall patterns during monsoon periods.

From the back-analysis method, shear strength parameters of slip surfaces were found as 17 and 5.25 kN/m^2 for the friction angle and cohesion at the time of failure. These values are reasonable according to site conditions assuming slurry conditions that may develop at the slip surface.

The hydraulic conductivities of vegetation cover, silty sand layer, completely weathered rock and bed rock were found as 6.5×10^{-8} m/s, 1×10^{-6} m/s, 1×10^{-8} m/s, 1×10^{-9} m/s etc. All values are in acceptable range according to their layer properties and the analysis gives very close results to the monitoring data.

The subsurface drainage system was also modeled as layers in this case study, due to lack of proper boundary condition which represents the actual site phenomenon.

Accordingly, back analysis method was again applied to find the hydraulic properties of materials that were used to model the pipes and drainage wells. The results were obtained as $1x10^{-5}$ m/s and $1x10^{-4}$ m/s for lateral drains alone and both discharge pipes and drainage wells respectively. Even though this system was modeled as layers with some material properties, the main objective behind this technique is finding the speed through each component that conveys the water. The analysis of the case study reveals that even though the sub surface drainage system is a hollow structure, the flow through the system depends on the hydraulic properties of surrounding soil layers, varying pressure head conditions on drainage wells and corresponding pressure heads on discharge borings etc. The obtained values for permeability of each soil layers are much smaller compared to the ability of convey water through the pipes. But it is obvious that the small change of permeability shows considerable draw down of water table and considerable rise of factor of safety even the permeable zone is negligible compared to the landslide area. Moreover, this is the most suitable method in finding flow through the interconnected sub surface drainage system, which has a complex nature with varying conditions of pressure heads, flows and infiltration rates through the soil layers.

8.2 Effects on Rectifications in Stabilization of the Landslide

One of the main objectives of this research is studying the effectiveness of countermeasures applied for the landslide rectification. The continuous monitoring process that carrying out at the site reveals the clear signs of stabilization of the landslide after constructions of rectification measures. Compared to actual conditions, the monitoring data also verifies the situation with no movements visible at the ground for rainfall events occurred after the completion of the project. Observations made in drainage wells during the rainfall events verify the fluctuation of water levels in the area, is within a safe margin. Therefore, it can be concluded that the landslide is in a stability state and its features are visible in the ground.

Stability analysis carried out for the high-water level occurrence before construction of counter measures showed factor of safety values lower than 1, implying the landslide is unstable during the particular rainfall event. Two slip surfaces out of four showed more critical and others are in slightly greater than unity. The most critical slip surfaces were found out at the site with showing clear signs of landslide scar marks and tension cracks that verify the results of stability calculations.

The stability analysis with rectification measures shows factor of safety values more than 1.2 for a high rainfall recorded in the area, implying that the combination of rectification measures has achieved to the target level of stability. The observations that were made at the site also verify the results of stability. It is essential to find the response of the rectified landslide to even worse case that may take in place, in order to forecast about the stability of the rectified landslide. Therefore, the model was again analyzed for several assumed design rainfalls over 67 days which is very similar to the worst case recorded at the site. The results were also used to find the thresholds of the rectified landslide as well. It turned out that the desirable safety margin lies in between 950mm-1000mm and the landslide threshold value lies in between 3000 to 3500 mm cumulative rainfall. However, these results can be varied with different intensity of rainfall and the number of days. Furthermore, following reasons cause for producing different results of design reports and the above analysis are different due to following reasons.

1. Even though both analyses were done according to limit equilibrium method, the equations used for the calculations are different (Janbu method and Morgenstern- price method).
2. The pore water pressure built up in the profile and the effect of matric suction were taken into account in this case study, while the designer used ground water table only for his calculation.

Finally, it can be concluded that the rectified landslide has achieved to a stability level even for a higher rainfall event, showing the success of construction works that carried out.

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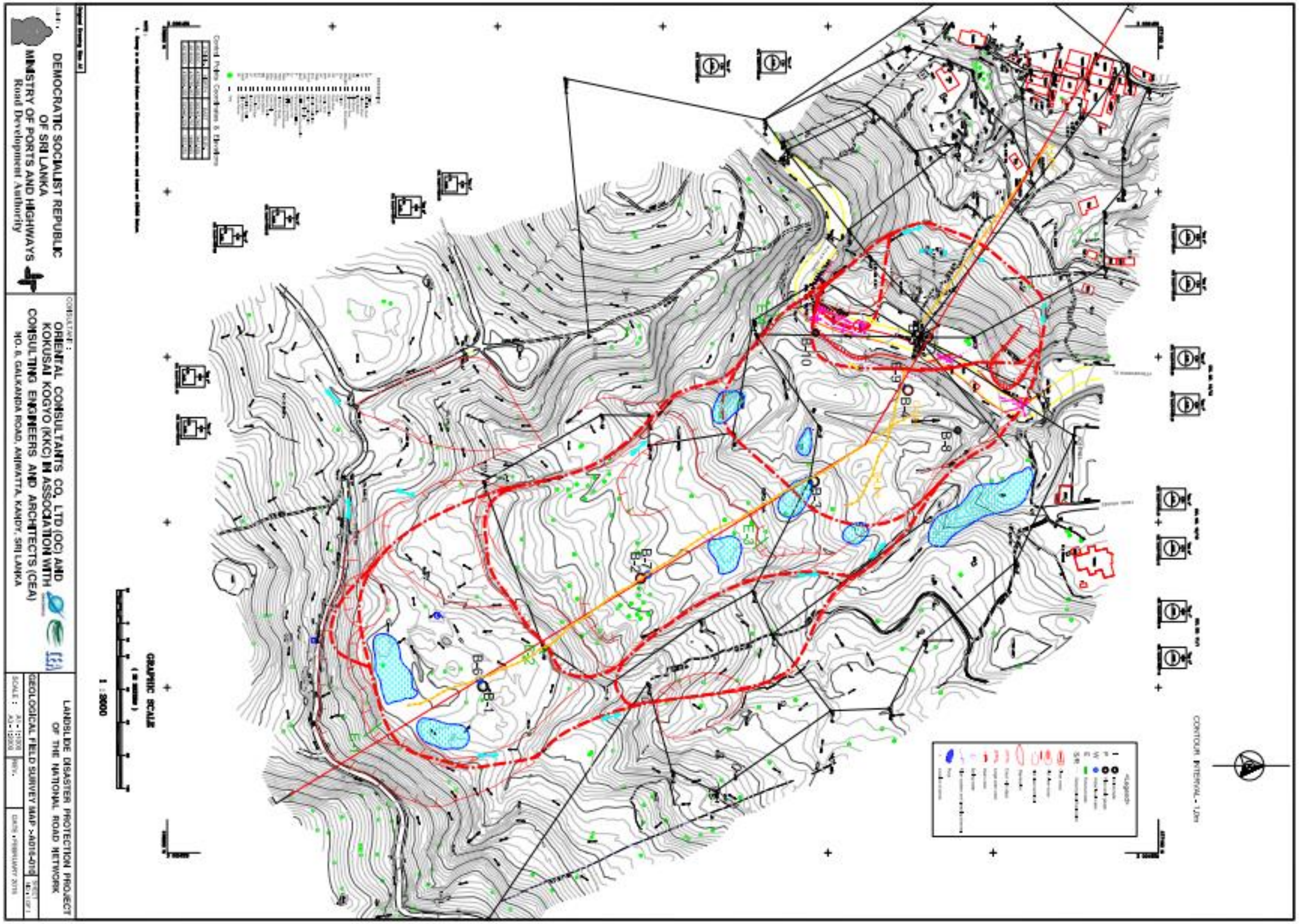
10.APPENDICES

Appendix 1

Site Planview and Investigation Points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OF SRI LANKA
MINISTRY OF PORTS AND HIGHWAYS
Road Development Authority
CONSULTANT :
ORIENTAL CONSULTANTS CO., LTD (OC) AND
KOKUSAI KOGYO (KKC) IN ASSOCIATION WITH
CONSULTING ENGINEERS AND ARCHITECTS (CEA)
NO. 6, GALKANDA ROAD, ANIWATTA, KANDY, SRI LANKA
LANDSLIDE DISASTER PROTECTION PROJECT
OF THE NATIONAL ROAD NETWORK
GEOLOGICAL FIELD SURVEY MAP
GRAPHIC SCALE
CONTOUR INTERVAL - 1.0m
B-1
B-6
B-8
B-10

Appendix 2

Laboratory Test Results

Table 10.1: Summary of the laboratory test results

Drilling No.	Depth (m)	Soil classification	Natural Moisture content (%)	Specific Gravity	Grain Size Distribution				Atterberg Limits		
					Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI
B-1	9.00-9.45	GM	34	2.65	36	20	14	30	56	40	16
	14.00-14.45	SM	32	2.79		62	28	10	Non plastic		
B-2	9.00-9.45	SM	55	2.51		52	44	4	62	44	18
	15.00-15.45	SM	24	2.46	1	75	16	8	Non plastic		
	18.00-18.45	SM	24	2.60	30	52	15	3	Non plastic		
B-3	10.00-10.45	MH	52	2.26		41	29	30	72	38	34
	16.00-16.45	SM	49	2.52		63	29	8	39	31	8
	20.00-20.45	SM	23	2.60	1	69	23	7	32	23	9
B-4	17.00-17.45	SM	53	2.55		52	27	21	74	49	25
	24.00-24.45	SM	62	2.75	1	65	15	20	94	48	46
	28.00-28.45	SM	39	2.36		87	8	4	41	29	12
B-5	10.00-10.45	SM	18	2.62		70	25	5	Non plastic		

Table 10.2 : Results of Laboratory tests (block sampling)

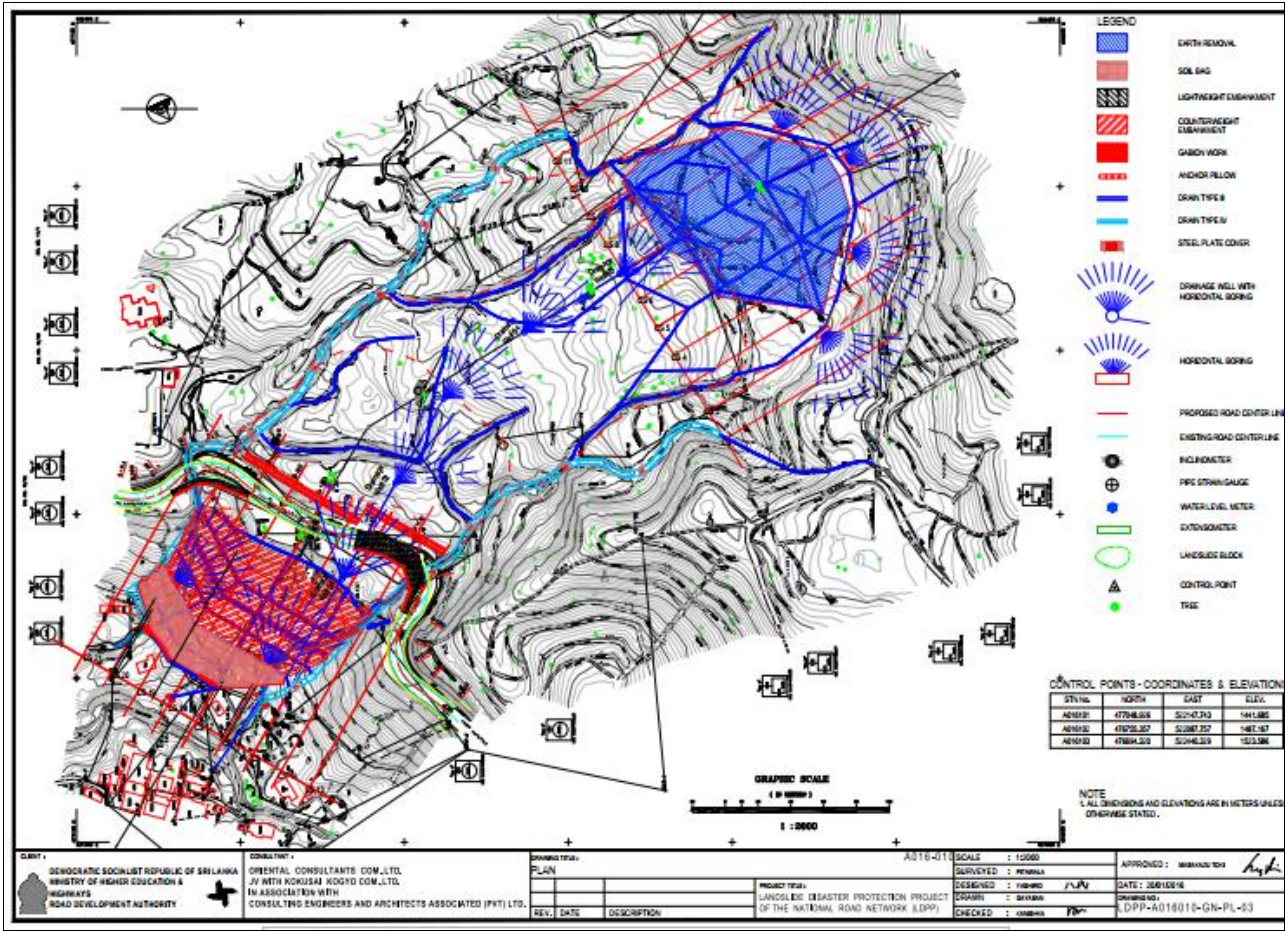
Sample No.	Depth (m)	Soil classification	Natural Moisture content (%)	Specific Gravity	Grain Size Distribution				Atterberg Limits		
					Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	PI
Sample B	GL-1.0m	MH	32	2.61	0	26	36	38	60	45	16

Table 10.3: Results of shear tests (Block Sampling)

Sample No.	Depth (m)	Wet Density (g/cm3)	Triaxial test/ Consolidated undrained		Direct shear	
			C' (kpa)	Φ' (Deg.)	C (kpa)	Φ (Deg.)
Sample B	GL-1.0m	1.51	20	32	10	32

Appendix 3

Plainview and Longitudinal View with Rectification Measures


LEGEND
EARTH REMOVAL
SOIL BAG
LIGHTWEIGHT EMBANKMENT
COUNTERWEIGHT EMBANKMENT
GABION WORK
ANCHOR PILLOW
DRAIN TYPE II
DRAIN TYPE IV
STEEL PLATE COVER
DRAINAGE WELL WITH HORIZONTAL BORING
HORIZONTAL BORING
PROPOSED ROAD CENTER LINE
EXISTING ROAD CENTER LINE
INCLINOMETER
PIPE STRAIN GAUGE
WATER LEVEL METER
EXTENSOMETER
LANDSLIDE BLOCK
CONTROL POINT
TREE
CONTROL POINTS - COORDINATES & ELEVATIONS
STN No.
NORTH
EAST
ELEV.
GRAPHIC SCALE
1 : 2000
NOTE
1. ALL DIMENSIONS AND ELEVATIONS ARE IN METERS UNLESS OTHERWISE STATED.
CLIENT :
DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA
MINISTRY OF HIGHER EDUCATION & HIGHWAYS
ROAD DEVELOPMENT AUTHORITY
CONSULTANT :
ORIENTAL CONSULTANTS CO.,LTD.
JV WITH KOKUSAI KOGYO CO.,LTD.
IN ASSOCIATION WITH
CONSULTING ENGINEERS AND ARCHITECTS ASSOCIATED (PVT) LTD.
DRAWING TITLE :
PLAN
REV.
DATE
DESCRIPTION
PROJECT TITLE :
LANDSLIDE DISASTER PROTECTION PROJECT OF THE NATIONAL ROAD NETWORK (LDPP)
SCALE : 1:2000
SURVEYED
DESIGNED
DRAWN
CHECKED
APPROVED :
DRAWING NO :
LDPP-A016010-GN-PL-03