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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

A Thesis in Analysis and Remedial Measures for Slope Instability Related Problems of Selected Sections of LOT II: Belta (KM60+000) – Otolo (KM 87+000) Asphalt Road Project, Southern Part of Ethiopia.

BY

GETINET TILAHUN GEDA

ADVISOR: PROFESSOR ALEMAYEHU TEFERRA

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of science in Civil Engineering

The undersigned have examined the thesis entitled '**ANALYSIS AND REMEDIAL MEASURES FOR SLOPE INSTABILITY RELATED PROBLEMS OF SELECTED SECTIONS OF LOT II: BELTA (KM60+000) - OTOLO (KM87+000) ASPHALT RAOD PROJECT, SOUTHERN PART OF ETHIOPIA**' presented by **GETINET TILAHUN GEDA**, a candidate for the degree of **Master of Science in Civil Engineering** and hereby certify that it is worthy of acceptance.

Prof. Alemayehu Teferra	______________________	_____________
Advisor	**Signature**	**Date**
Dr. Ing. Henock Fikre	______________________	_____________
External Examiner	**Signature**	**Date**
Dr. Ing. Samuel Tadesse	______________________	_____________
Internal Examiner	**Signature**	**Date**
Dr.-Ing Esayas G/Youhannes	______________________	_____________
Chairman	**Signature**	**Date**

DECLARATION

I certify that research work titled "**ANALYSIS AND REMEDIAL MEASURES FOR SLOPE INSTABILITY RELATED PROBLEMS OF SELECTED SECTIONS OF LOT II: BELTA (KM60+000) - OTOLO (KM87+000) ASPHALT RAOD PROJECT, SOUTHERN PART OF ETHIOPIA**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Signature	____________________
Name	Getinet Tilahun Geda
Place	AAIT, School of Civil and Environmental Engineering
Date of Submission	FEBRUARY 2016

Dr.-Ing Esayas G/Youhannes	______________________	_____________
Chairman	**Signature**	**Date**

Dr.-Ing Esayas G/Youhannes	______________________	_____________
Chairman	**Signature**	**Date**

Dr.-Ing Esayas G/Youhannes	______________________	_____________
Chairman	**Signature**	**Date**

Dr.-Ing Esayas G/Youhannes	______________________	_____________
Chairman	**Signature**	**Date**

ABSTRACT

This thesis presents the investigation, analysis and design of proposed remedial measures for three cut slope related instability (land slide) locations stationed at Sta.66+995 to Sta.67+055(RHS), Sta.68+020(ahead) to Sta. 68+120(back) (RHS) and Sta. 72+740 to Sta. 72+790(LHS) of LOT II: Belta – Otolo Asphalt Road Project located in Southern part of Ethiopia.

In this thesis work, first slide characterization of the cut slope instability areas which includes the land slide location, dimensional variables and engineering classification the land slide soil through detailed site investigation and testing was carried out. Analysis of the land slide after the characterization of the cut slide was carried out by the method of back analysis and 2D limit equilibrium stability analysis. Then, residual shear strength variables were back calculated on the onset of failure or for FS=1 for each cut slope instability location. Finally, using the results of the investigation and analysis, combination of control and restraint remedial measures were proposed and designed at the respective land slide locations so that the construction of road at these locations were finalized.

Generally, it was observed that the provision of steep cut slope angles, the presence of relict residual bedding profiles and migration of moisture from prolonged rainfall duration were the main reasons for the instability of the cut slopes under this study.

Key words: Cut slope instability, back analysis, design of remedial structures

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LIST OF ABBREVATIONS

ASTM	American Society for Testing and Materials
CD	Consolidated Drained Test
CDSC	Construction Design Share Company
CU	Consolidated Undrained Test
DDGC	Diriba Defersha General Contractor
EBCS	Ethiopian Building Code Standard
ERA	Ethiopian Roads Authority
FEM	Finite Element Method
IAEG	Engineering Geologist Commission of Landslides
MSE	Mechanically Stabilized Earth Walls
SLOPE/W	One component of a complete suite of geotechnical products called Geostudio that has been designed and developed to be a general software tool for the stability analysis.
UU	Unconsolidated Undrained test

LIST OF SOME SYMBOLS

C'	cohesion in terms of effective stress;
C	cohesion in terms of total stress;
ϕ'	angle of internal friction of soil in terms of effective stress;
ϕ	angle of internal friction of soil in terms of total stress;
ϕ_r	residual internal friction angle;
ϕ_d	developed/calculated internal friction angle;
C_r	residual cohesion;
C_d	developed/calculated cohesion;
CI	Cohesive Index
F_m	factor of safety in terms of moment equilibrium
F_f	factor safety in terms of force equilibrium
F_d	the sum of driving forces for factor of safety formulations
F_r	the sum of resisting forces for factor of safety formulations
M_d	the sum of driving moments for factor of safety formulations
M_r	the sum of resisting moments for factor of safety formulations
γ	unit weight of the backfill materia
H	the height of the retaining wall
K_a	the active earth pressure coefficient

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1. Introduction

1.1. Background

An earth slope is an unsupported inclined surface of a soil mass mainly purpose made for railway formations highway cuts and fills, earth dams, etc or natural slopes. On engineered slopes the cost of earth work would be reduced if the slopes are made steep. However, very steep slopes may not be stable. A common strategy adopted is to make a compromise between economy and safety, where slopes provided are neither too steep nor too flat. Alternatively speaking, steepest slopes which are stable and safe should be provided.

Many road projects which are under construction traversing on escarpment, mountainous and rolling terrain many time pose slope instability problems due to earth cuttings which are required by the road design standard. This problem is mainly attributed due to seasonal change of moisture on the slopes during wet (rainy) and dry season as the presence of water causes both increased stresses(through seepage forces) and loss of shear strength within the slope or failure surface.

Under mountainous terrains conditions where massive earthwork activities with quantities of excavations in the order of 3-4 million cubic meters are designed to meet road standards, cut slope instability problems are the most common and troublesome of major geotechnical difficulties encountered in such road construction projects. And most of the time, there is a general lack of focus for detailed investigation and design for mitigation of slope instability related problems during design of the same road projects. The problem of cut slope instability will not become apparent until excavation works are commenced or finalized to the required lines and grades, however at later stages the effort to correct these problems results in project cost overruns and delay in completion of the project.

1.2. Objective

This research aims at proposing optimal remedial solutions after detailed analysis of selected failed slopes for sections of LOT II: Belta (KM60+000) – Otolo (KM 87+000) Asphalt Road Project, located in South Nations and Nationalities People Regional state of Gamo Gofa Zone , South part of Ethiopia. To reiterate once more**,** the objectives of this research were aimed at investigation and analysis of cut slope instability related problems at three selected locations of the road project and finally to propose and design optimal remedial solutions for these failed cut slopes.

1.3. Scope, method and limitation of the study

Here, three failed cut slopes(landslides) locations were selected at locations Sta.66+995 to Sta. 67+055(RHS), Sta.68+020(ahead) to Sta. 68+120(back)(RHS) and Sta. 72+740 to Sta. 72+790(LHS) of the road stretch.

The scope of this study incorporates detailed investigation, analysis and provision of optimal remedial measures at the selected cut slide locations aimed at restoring the slide sections of the road for traffic.

In order to achieve the objective and the scope of the study, the following methodology were introduced:

- the investigation of selected failed cut slopes were done by field observation, surveying and laboratory tests.
- analysis of failed slopes were carried out by the method of back analysis using limit equilibrium two dimensional slope stability formulations.
- the results of these investigations and analyses were used to propose and design optimal remedial measures for respective cut slide locations.

The use of the results of the analysis and designed and proposed remedial solutions of this study are limited to the respective slide locations while the method of investigation, method of analysis and method of design of remedial measures can be employed at other cut slide locations of the road project.

1.4. Organization of the thesis

The thesis is organized in six main chapters along with appendixes included at the end of the thesis. The introduction chapter highlights the background problem, the objectives and scope of the study. Chapter two presents the literature review of the general slope stability concepts and remedial measures while chapter three is devoted to brief description of the road project with due emphasis on project location, climatic conditions, local geology, topography and terrain classification of the study area. Furthermore, investigation, testing, characterization and analysis of the respective cut slope landslides locations are presented under chapter four. The respective remedial solutions for the slide locations are presented under chapter five while the conclusions and recommendations of this thesis report are discussed under chapter six.

2. Literature Review

2.1. General Slope Instability(Stability) Concepts

2.1.1 Introduction

The increased demand for engineered cut and fill slopes on construction projects have disrupted the delicate balance of natural soil slopes. The need to understand analytical methods, investigative tools, and stabilization methods to solve slope stability problems has only increased through these times due to the increasing demand of such engineered slopes. Slope stabilization or remediation methods involve special construction techniques that must be understood and modeled in realistic ways.

An understanding of geology, hydrology, and soil properties is central to applying slope stability principles properly. Furthermore, analyses must be based upon a model that accurately represents site subsurface condition, ground behavior, and applied loads while judgments regarding acceptable risks or safety factors must be made to assess the results of analyses.

In most applications, the primary purpose of cut slope instability analyses are:

(a) to analyze cut slope slides and understand failure mechanisms and influence of environmental factors.

(b) to enable the redesign of failed cut slopes and the planning and design of remedial measures, where necessary,

2.1.2 Cut Slope Failures or Land Slides

2.1.2.1 Description of Land Slides

When a slope fails, it is often called a landslide or a slope failure. Several classification methods and systems have been proposed for landslides. The one adopted in this thesis report and most consistently around the world is the one proposed by the International Association of Engineering Geologists (IAEG) Commission of Landslides. [6]

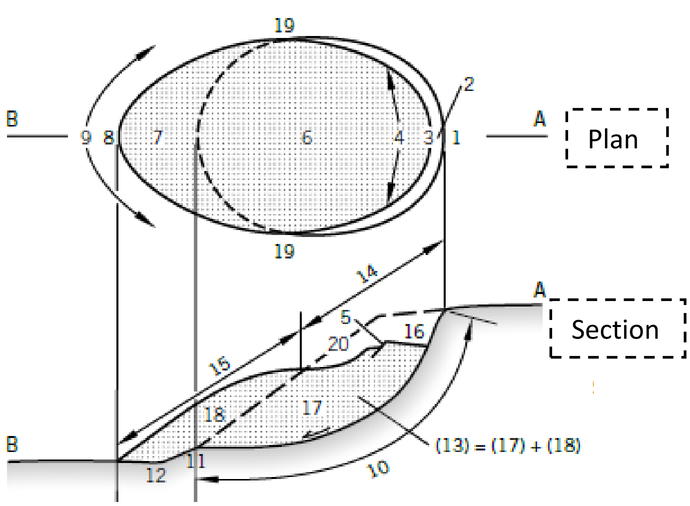


Fig 2.1: land slide features.

PLAN: The dashed line is the trace of the rapture surface on the original ground surface.

SECTION: Cross hatching indicates undisturbed ground and the broken line is the original ground surface. And stippling shows the extent of the displaced material [6].

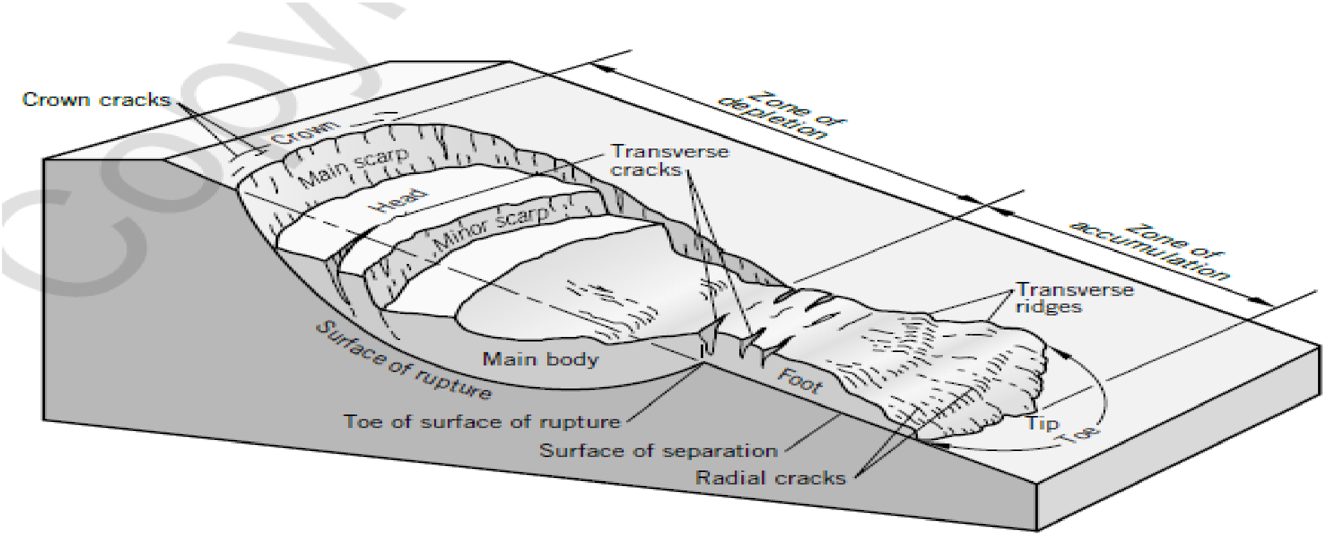


Figure 2.2: Terminology for describing landslide features [6]

Accordingly, the observable typical features of a land slide are shown schematically in the Figure 2.1 and 2.2 above while the description of landslides parts is tabulated hereunder Table 2.1.

Table 2.1: Description of landslides parts [6]

No.	Name	Definition
1	Crown	The practically undisplaced material above the main scarp.
2	Main Scarp	A steep surface on the undisturbed ground at the upper edge of the land slide. Caused by the movement of displaced material (13, stippled area) away from undisturbed ground; it is visible part of surface of rapture, (10)
3	Top	The highest point of contact between the displaced material (13) and main scarp (2).
4	Head	The upper parts of the land slide between the displaced material and main scarp (2).
5	Minor Scarp	A steep surface on the displaced material produced by differential movements within displaced material.
6	Main Body	The part of the displaced material that overlies the surface of rupture between main scarp (2) and toe of surface of rapture (11).
7	Foot	The portion of the land slide that has moved beyond the toe of the surface of rapture (11) and overlies original ground surface (20).
8	Tip	The point on the toe (9) farthest from the top (3) of land slide.
9	Toe	Lower, usually curved margin of the displaced material of the landslide, most distant from the main scarp (2).
10	Surface of Rupture	The surface that forms the lower boundary the displaced material (13), below original ground surface (20) also termed slip surface or shear surface.
11	Toe of Surface Rupture	The intersection (usually buried) between the lower part of the surface of rapture (10) and the original ground surface (20).

No.	Name	Definition
12	Surface of Separation	Part of the original ground surface now overlain by the foot (7) of the land slide.
13	Displaced Material	Material displaced from its original position by landslide movement comprises both depleted mass (17) and accumulation (18).
14	Zone of Depletion	The area within which the displaced material lies below the original ground surface (20).
15	Zone of Accumulation	The area of land slide within which the displaced material (13) lies above the original ground surface (20).
16	Depletion	The volume bounded by the main scarp (2), the depleted mass (17), and the original ground surface (20).
17	Depleted Mass	The volume of the displaced material (13) that overlies the rupture surface (10) but underlies the original ground surface (20).
18	Accumulation	The volume of the displaced material(13) that lies above the original ground surface(20), if left and right used, they refer to flanks as viewed from crown; otherwise use compass direction.
19	Flank	The undisplaced material adjacent to the sides of the rapture surface.
20	Original Ground Surface	The surface of the slope that existed before the slide took place.

Similarly, standardized typical dimensional variables for the land slide (cut slope instability) as per the International association of Engineering Geologists (IAEG) commission of landslides as per Fig.2.3 below; [1]

Table 2.2: Description of landslide dimensional variables [1]

1. Width of Displaced Mass, W_d **: the maximum breadth of the displaced mass perpendicular to the length $L_{d)}$(4)**
2. Width of Rupture Surface, W_r**: The maximum width between the flanks of the landslide, perpendicular to the length, L_r(5).**
3. Total Length, L**: The minimum distance from the tip of the land slide to its crown.**
4. Length of Displaced Mass, L_d**: The minimum distance from tip to the top.**
5. Length of Rupture Surface, L_r: **The minimum distance from the toe of the surface of rupture to the crown.**
6. Depth of the displaced mass, D_d: **the maximum depth of the displaced mass, measured perpendicular to the plain containing W_d (1) and L_d (4).**
7. Depth of the Rupture Surface, D_r: **The maximum depth of the rupture surface below the original ground surface measured perpendicular to the plane containing W_r (2) and L_r (5).**

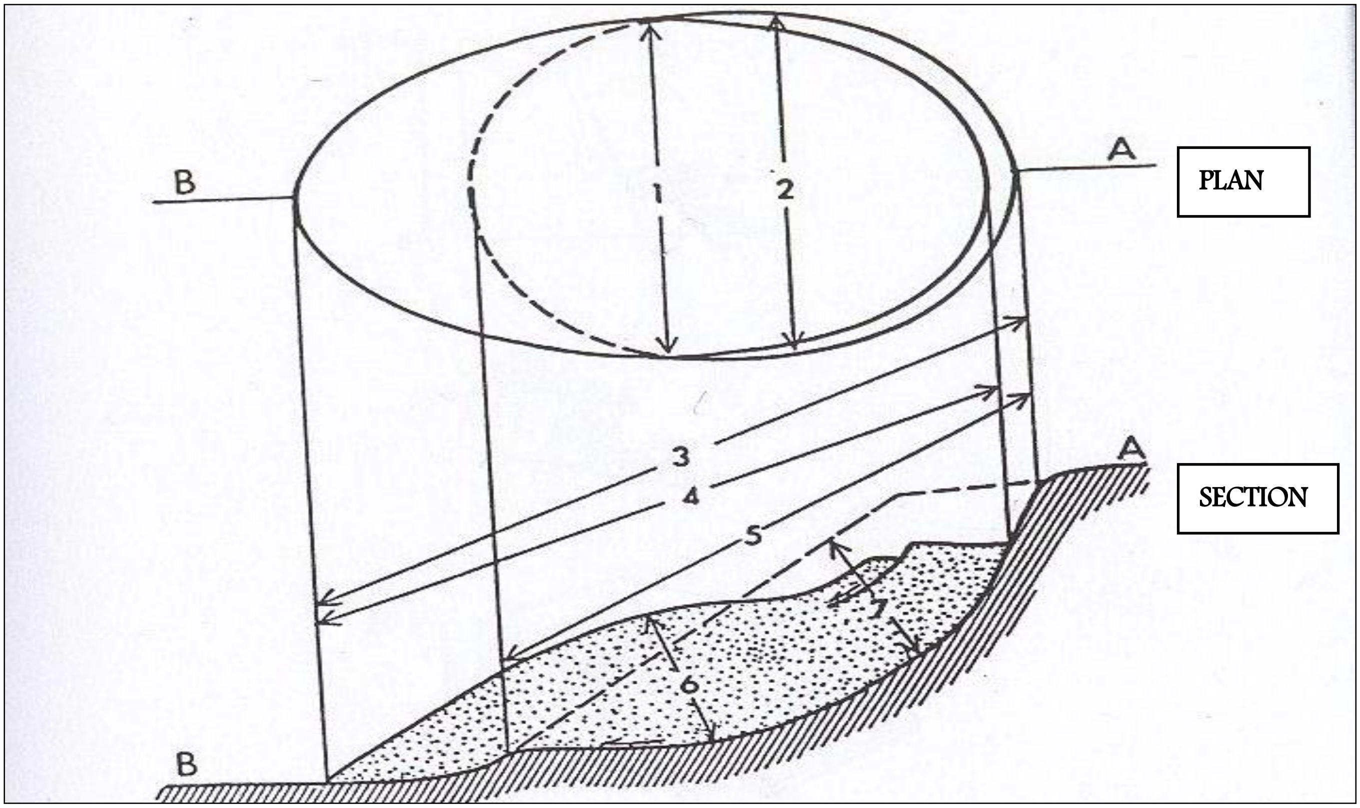


Figure 2.3: Land slide dimensional variables

SECTION: Cross hatching indicates undisturbed ground and the broken line is the original ground surface. PLAN: The dashed line is the trace of the rupture surface on the original ground surface. [1]

2.1.2.2 Land Slide Dimension

There is no reference to classify landslides by size, but to provide some reference Table 2.3 is used in describing landslide size in this thesis. [6]

Table 2.3. Grouping landslides by area in plan

Description	Area, Sq.m.
Very Small	<200
Small	200-2,000
Medium	2,000-20,000
Large	20,000-200,000
Very large	200,000-2,000,000
Huge	>2,000,000

Guide Line: Here, length is measured horizontally not along the slope. If the size is near the border between categories, both sizes are mentioned(e.g. land slide of around 2,000 sq.m. is described as small - to- medium landslide). For slopes Steeper than 45^0 to the horizontal, it is recommended that vertical height replace horizontal length in the area calculation. For Flow slides, it is recommended that the area be based on the eroded bowl at the initiation site, ignoring any further erosion further down slope in the valley below.

2.1.2.3 Landslide Classification

The kinematics of landslides, i.e. how movement is characterized throughout the displaced mass, constitutes another way of classifying landslides. Slope movements or land slides are subdivided into six categories:

(i) Falling

(ii) Toppling

(iii) Sliding

(iv) Spreading

(v) Flowing

(vi) Composites – combination of types

Here, each type of landslide has a number of common modes. falling and toppling are features frequently associated with rock slopes, Fig. 2.4 (A) and(B) below, whereas the latter three are related to soil slopes.

A slide is down slope movement of a soil mass occuring predominantly on surface of rupture or relatively thin zones of intense shear strain. Movement is usually progressive from an area of local failure. The first overt signs of ground movement are usually cracks in the original ground surface along which the main scarp of the slide will form. The slide may be translational or rotational, or a combination of both, which is called a compound slide. Translational slides often involve movement along marked discontinuities or planes of weakness, including previously existing failure planes. In clay soils, translational slides takes place along saturated sand or silt seams, particularly where these zones of weakness dip roughly parallel to the existing slope. Rotational slips have a failure surface that is concave upwards and often occur within an intact soil mass. Classic, purely rotational, slope failures (slumps) most commonly occur in relatively homogeneous material, such as those found in constructed fills and embankments, refer Fig. 2.4 (C),(D) and (E) below.

Spread is defined as an extension of a soil mass combined with a general subsidence of the fractured mass into softer underlying material. The rupture surface is not a surface of intense shear. spreads may result from liquefaction of granular deposits or failure of weak cohesive soils in a slope, reference is made to Fig.2.4 (F) below. They commonly occur on shallow slopes.

A flow, reference is made to Fig. 2.4 (G) and (H) below, is a spatially continuous movement in which surfaces of shear are short – lived., closely spaced, and usually not preserved. The distribution of velocities in the displacing mass resembles that of a viscous liquid. The lower boundary of the displaced mass may be a surface along which appreciable differential movement has taken place or it may be a thick zone of distributed shear. Slides may turn gradually into flows due to changes in water content, mobility, and evolution of movement.[1,6]

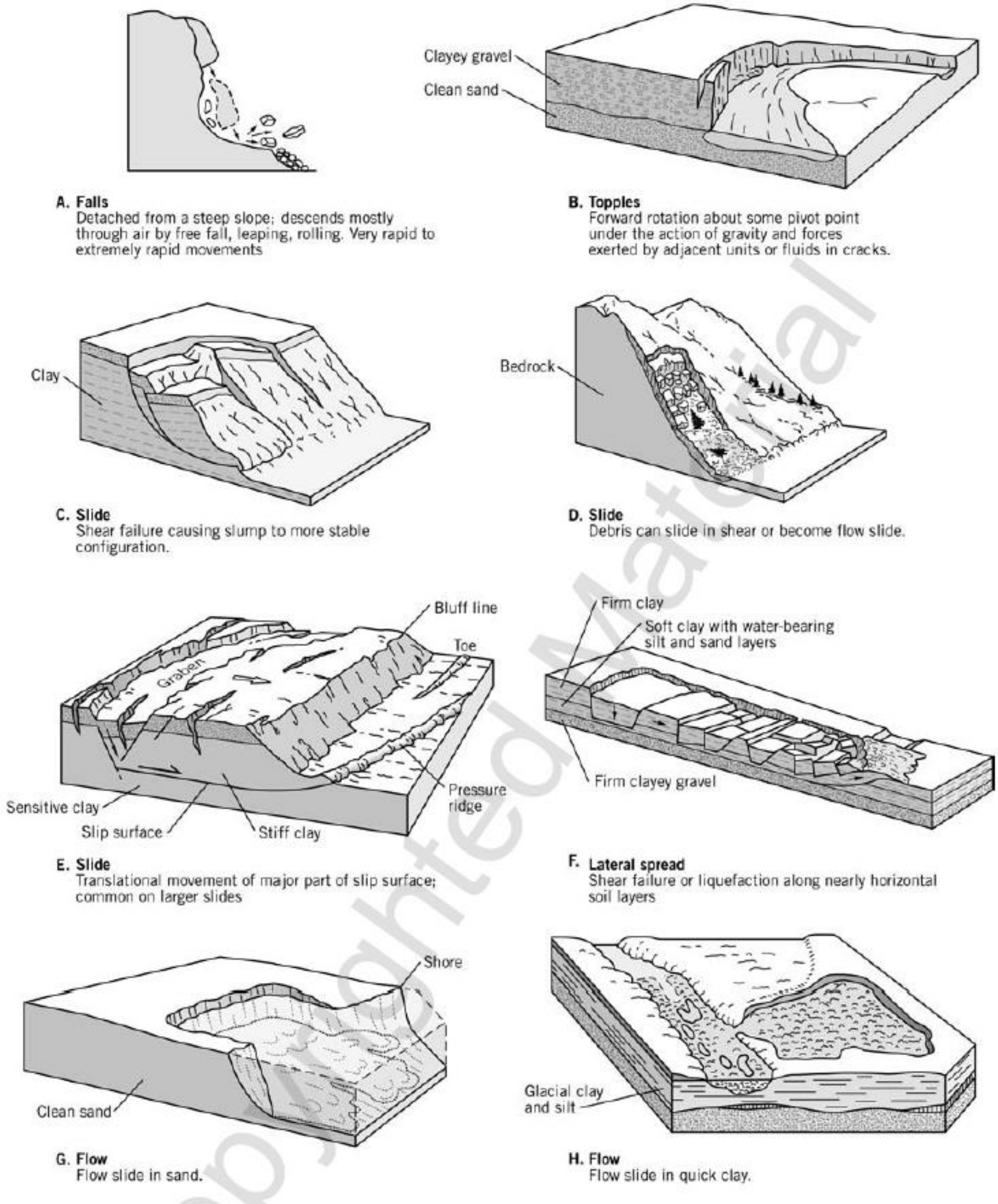


Figure 2.4: Examples of landslide occurrences [6]

2.1.2.4 Factors Contributing to Slope Failures

Slope failures are often caused by processes that increase shear stresses or decrease shear strengths of the soil mass. Processes that most commonly cause an increase in the shear stresses acting on slopes are:

- Removal of Support: erosion, natural soil movements(e.g. falls, slides, settlements), human activity(cuts and excavations, removal of retaining walls, drawdown of bodies of water)
- Overloading: by natural causes(weight of precipitation, accumulation of materials because of past landslides), by human activity(construction of fill, buildings and other overloads at the crest)
- Transitory effects (e.g. Earthquakes)
- Removal of underlying materials that provide support(by rivers, by weathering, by underground erosion due to seepage(piping), by human activity(excavation or mining), by loss of strength of underlying material)
- Increase in lateral pressure(by water in cracks and fissures, by freezing of the water in the cracks, by expansion of clays)

Factors that cause reduced shear strength in slopes are:

- Factors inherent in the nature of the materials(Composition, Structure, Secondary or inherited structure, stratifications)
- Changes caused by weathering and physiochemical activity(wetting and drying processes, hydration, removal of cementing agents)
- Effect of pore pressure
- Changes in structure (Stress release, structure degradation)

Residual soil and weathered bedrock can be weakened by preexisting discontinuities such as faults, bedding surfaces, foliations, cleavages, sheared zones, relict joints, and soil dykes. Relict Joints and structures in residual soils often lose strength when saturated. Slicken sided seams or weak dikes may also preexist in residual soil and weathered rock slopes. Faults, bedding surfaces, cleavages, and foliations have more influence on rock slope stability than soil slope stability.

Slope cuts release residual horizontal stresses and cause expansion of the slope. It is essential not to undermine the toe of the cut slope but to maintain the slope as "loaded" as possible. The main causes of cut slope instability are overcutting the toe or over steepening the slope angle. [1]

2.2. Cut Slope Slide(Instability) Investigations

2.2.1 Field Investigation

The scope of a site investigation depends on the size and complexity of the landslide under study. For a small landslide. The first consideration is to understand the cause of the landslide, then obtain sufficient data to properly model it, and finally determine the options that best suit the situation revealed by the data collection. These options are usually focused on different methods of remediating the landslide. That is, achieving stabilization with a sufficient allowance for uncertainties (usually measured by factor of safety) to be reasonably sure that it will remain permanently stable. For a medium to large landslide, site investigations can be subdivided into two phases: preliminary and general.

Preliminary: This phase includes the site visit, information collection, and planning of a study program.

The following information should be obtained, if it is available: previous site investigation reports; past construction activities on the property; topographic plans of the site; aerial photos; oblique photos of the slide from the air; relevant photos of site development or landslide event; regional geology maps; correspondence pertaining to the landslide site, including prior landslide and maintenance records; and newspaper accounts of the landslide event.

The Site visit is the most important part of the preliminary work because it is the earliest visit to the landslide site and provides the opportunity to obtain photographs and descriptions of the conditions before they are changed. Even if the landslide occurred months or years earlier, it is still the first opportunity to assess the situation and should be used to collect as much information as possible.

Landslides occur in a wide variety of circumstances, and it is difficult to provide advice that is to the point to all landslides. However, the more important requirements of a site reconnaissance are:

- Make a plan of the landslide
- Make a section through the approximate center of the landslide
- Take many photographs from different viewpoints at the site
- Quantify observations as much as possible

General: This is usually the key study, and involves the collection of data to understand and model the landslide. The most important requirement is to put a line of borings through the center of the landslide to determine the geological conditions, measure the depth of slippage, measure groundwater levels near the slip surface, and collect samples for laboratory testing of classification and strength. Other borings can expand this information to other parts of the landslides and occasionally outside the landslide area. [6]

2.2.2 Landslide shear surfaces

Land slide geometry: the shape of the landslide shear surfaces have to be measured for modeling in stability analysis. The more common shapes of the shear surfaces are: Single Plane(or slightly curved) on steep cuts; circular arc in uniform strata such as fills, soft clays, and smaller landslides; double and triple wedges on translational landslides, usually in medium to large size categories. [6]

Depth of Failure Surface: the depth of the failure surface below the crust of the slope is usually equal to the distance from the crest of the slope back to the furthest shear crack. For failures beyond the toe of the slope, the depth of the failure plane at the toe is usually about one-third the distance from the toe to the edge of the mud wave. If the mud wave exists on a continuing slope, the outlet of the failure surface is usually near the top the visible mud wave, reference can be made to Figure 2.5, below.

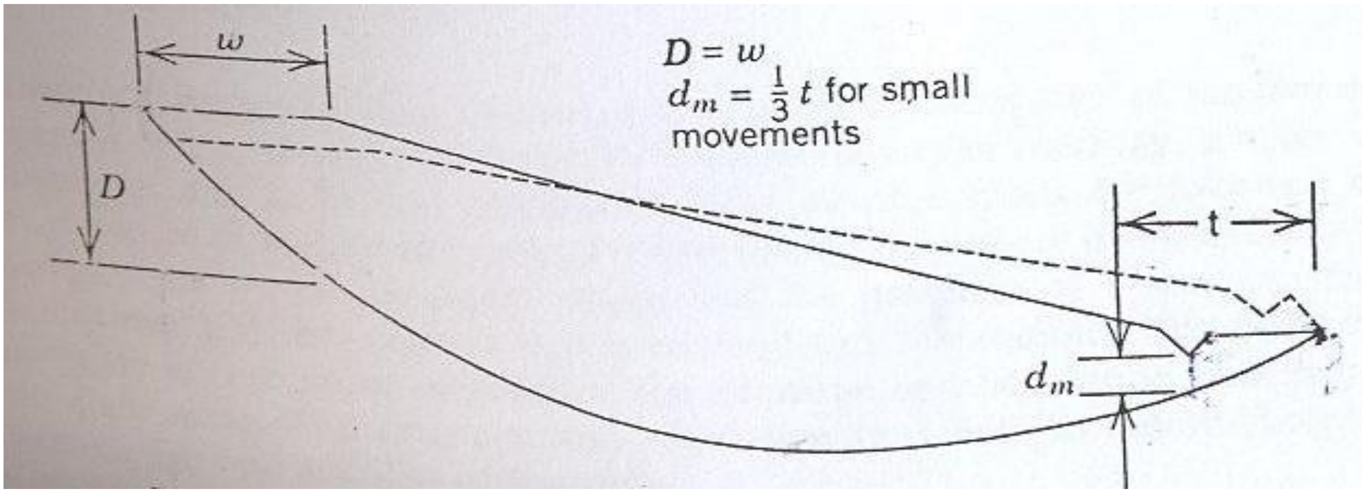


Figure 2.5: Typical depth to failure surface [1]

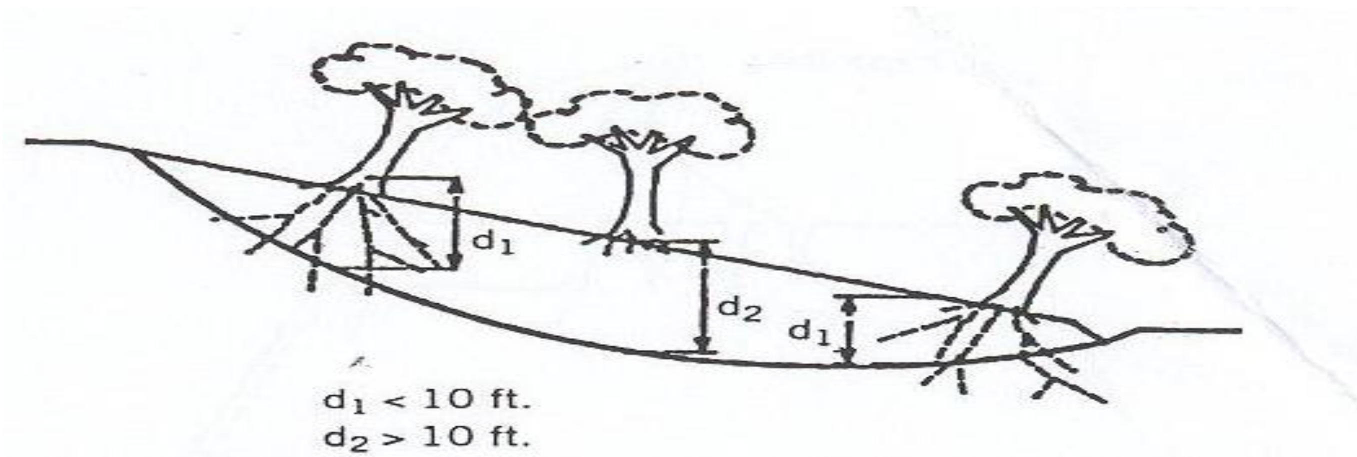


Figure 2.6: Depth of failure surface estimated from trees with deep roots [1]

The depth of the failure plane may be less than 10 feet if the deep routed trees in the sliding mass tend to tilt down slope, as depicted in Figure 2.6 above.

Breaks in buried utilities, such as culverts and sewer pipes, can give a direct visual identification of where the failure surface exists, and some times of how much movement has occurred across the failure surface. [1]

2.3. Analysis of land slide or slope instability

For slope instability analyses to be useful, they must represent the correct problem and correctly formulated. This requires a better understanding of the principles of soil mechanics, knowledge of geology and site conditions, and knowledge of properties of the soil at the site conditions.

2.3.1 Factors to consider in land slide or slope instability analysis

The selection of the proper method for soil slope instability analysis is based on the geometric and time factors. Here, the geometric factors refers to the type of the instability (cut slope, earth dam, embankments, etc), the location of critical failure surfaces, dimensional parameters of the slide and the engineered slope and stratigraphy (isotropic; homogeneous, non –homogeneous; anisotropic). The time factor relates to the short or long term conditions of loading which control whether undrained or drained conditions prevail and hence, whether total or effective stress analysis is applicable in the analysis model.

2.3.1.1 Drained and Undrained Conditions

The crux of the issue is whether or not changes in load cause change in pore pressure. The difference in these conditions is time. Undrained condition signifies a condition where changes in loads occur more rapidly than water can flow in or out of the soil. The pore pressures increase or decrease in response to the changes in loads. While drained signifies a condition where changes in load are slow enough, or remain in place long enough, so that water is able to flow in or out of the soil, permitting the soil to reach a state of equilibrium with regard to water flow. The pore pressures in the drained conditions are controlled by the hydraulic boundary conditions, and are unaffected by the changes in loads.

2.3.1.2 Effective and Total Stress Analyses

In effective stress analyses, the pore pressures along the shear surfaces are subtracted from the total stresses to determine effective normal stresses, which are used to evaluate shear strengths. Therefore, to perform effective stress analyses, it is necessary to know (or to estimate) the pore pressures at every point along the shear surface. These pore pressures can be evaluated with relatively good accuracy for drained conditions, where their values are determined by the response of the soil to external loads.

In total stress analyses, pore pressures are not subtracted from the total stresses, because shear strengths are related to total stresses. Therefore, it is not necessary to evaluate and subtract pore pressures to perform total stress analyses. The total stress analyses are applicable only to undrained conditions. The basic premise of total stress analysis is this: The pore pressure due to undrained loading are determined by the behavior of the soil. For a given value of total stress on the potential failure plane, there is a unique value of pore pressure and therefore a unique value of effective stress. Thus, although it is true that shear strength is really controlled by effective stress, it is possible for the undrained condition to relate for the undrained condition. Clearly, this line of reasoning does not apply to drained condition, where pore pressures are controlled by hydraulic boundary conditions rather than the response of the soil to external loads. [9]

2.3.1.3 Undrained and Drained Shear Strengths

Measured soil strength parameters are not unique. Soil type, strain history, and loading conditions (time) are the major factors to consider in the selection of strength parameters. Here, the strain history controls whether peak, ultimate, or residual strength will prevail, while the time element refers to short term or undrained conditions and long term or drained conditions.

Undrained Strength is the shear strength of the soil when loaded to failure under undrained conditions. In the field, closely approximating undrained conditions result when loads are applied to a mass of a soil faster than the soil can drain. In the laboratory, undrained conditions are achieved by loading test specimens so rapidly that they cannot drain, or by sealing them in impermeable membranes. Here, in order to avoid the high strain rates that are not representative of the field conditions, one needs to control drainage through the use of impermeable membrane rather than very high rates of loading.

Drained Strength is the shear strength of the soil when it is loaded slowly enough so that no excess pore pressures are induced by applied loads. In the field, drained conditions result when loads are applied slowly to a mass of soil, or where they persist for a long enough time so that the soil can drain. In the laboratory, drained conditions are achieved by loading test specimens slowly so that excess pore pressures do not develop as the soil is loaded. [9]

2.3.1.4 Analyses of Drained and Undrained Conditions

End – of – Construction Stability:

Slope stability during and at the end of construction is analyzed using either drained or undrained strengths, depending on the permeability of the soil. Many fine- grained soils are sufficiently impermeable that little drainage occurs during construction. This is particularly true for clays. For this fine grained soils, undrained shear strengths are used, and the shear strength is characterized using total stresses. For soils that drain freely, drained strengths are used; shear strengths are expressed in terms of effective stresses, and pore pressures are defined based on either water table information or appropriate seepage analysis. Undrained strengths for some soils and drained strengths for others can be used in the same analysis.

Long – Term Stability:

Over time after construction the soil in the slopes may either swell (with increase in water content) or consolidate (with decrease in water content). Long – term stability analyses are performed to reflect the conditions after these changes have occurred. Shear strengths are expressed in terms of effective stresses and the pore water pressures are estimated from the most adverse groundwater conditions anticipated during the life of the slope. [9]

2.3.2 Analysis of Slope Instability: Method of Back Analysis

When a slope fails by sliding it can provide a useful source of information on the conditions in the slope at the time of failure as well as an opportunity to validate stability analysis methods. Because the slope has failed, the factor of safety is considered to be unity (1) at the time of failure. Using this knowledge and an appropriate method of analysis it is possible to develop a model of the slope at the time it failed. The model consists of the unit weights and shear strength properties of the soil, ground water, and pore water conditions and the method of analysis, including failure mechanisms. Such model can help in understanding the failure better and be used as a basis for analysis of remedial measures.

The process of determining the conditions and establishing a suitable model of the slope from a failure is termed back-analysis or back calculation.

Back Analysis is probably the most valuable tool available for landslide studies. It provides confidence in ensuring the reliability of remedial work and allows the practitioner to use less conservative factor of safety for landslides than for slope stability calculations where no failure has occurred. Results of the back analysis calculations should provide an unambiguous measure of the shear strength at failure.

General procedure:-

Back Analysis involves an analysis of the stability at the onset of slope instability, knowing that the static factor of safety, FS, then is 1.00. This removes one of the unknowns in the stability calculation. If the soil density and the shape of the shear failure surfaces can be measured with good accuracy, it leaves ground water levels and shear strength on slip surfaces as the principal uncertainties or unknowns. A back analysis can be used to determine the average shear strength parameters acting on the entire slip surface at the time of the land slide activity. Here, parametric analysis can be made by varying the strengths and ground water levels to check that the calculated factor of safety is sufficient to account for uncertainties.

Back-analysis is a useful procedure for developing an analytical model of a failed or failing slope. This model usually consists of five components, where the first four components of the model can be evaluated with reasonable accuracy based on field and laboratory investigations and the fifth component of the model shall be established with back analysis model. Therefore, the components of the model are:

a. Landslide geometry including the ground surface, slip surface, and material boundary locations.
b. Pore water pressures on the sliding surface at time of failure. These are necessary for effective stress analysis.
c. External loads acting on the slope at the time of failure.
d. Unit weights of the materials involved in the land slide.
e. Strength of materials along the failure surface.

Because each pair of shear strength parameters (c–ϕ or c'– ϕ') corresponds to a unique slip surface, the location of the slip surface, along with the knowledge that the slope has failed (i.e. FS=1), can be used to back calculate values for two shear strength parameters (c–ϕ or c'– ϕ').

In the case where there are two or more materials along the sliding surface, back-analysis, such as the one mentioned above, doesn't result in a unique residual friction angle and cohesion value for the materials. In such circumstances, the back – analysis procedure would include seven steps:

i. Laboratory test results and/or correlations with index properties are used to establish trial values of shear strength along the failure surface. Often to simplify the analysis, c' is assumed equal to zero and only the residual friction angle is used.
ii. A stability analysis is performed using slope geometry, groundwater levels, and external loading conditions at the time of failure. The analysis yields a factor of safety, FOS that corresponds to the trial strengths from step (i).
iii. The trial strengths from step(i) are adjusted using the safety factor computed in step (ii), according to the following formulas:

$$\emptyset'_{d}(\text{adjusted}) = \tan^{-1}\left\{\frac{\tan\{\emptyset_d(trial)\}}{FOS}\right\} \quad (1\text{-}1)$$

$$c_d'(adjusted) = \frac{c_d(trial)}{FOS} \quad (1\text{-}2)$$

Here, the adjusted or developed cohesion and angle of internal friction represent back calculated values required to produce a factor of safety of 1. Note that, in general, $\emptyset_d{}'$ is fixed at a certain value and $c_d{}'$ is varied based upon local experience to expedite the time involved in the back-analysis.

iv. One then calculates the depth of the critical slip surface for each pair of values of the strength parameters

v. The back calculated cohesion and friction angle from step (iii) are plotted against the depth of the slip surface, calculated in step (iv).

vi. Finally, the cohesion and friction angle corresponding to the observed slip surface depth (actual) are determined from the plotted results.

vii. The results of Step (iii) can be verified by reanalyzing the slide using the newly calculated strengths. The final back-calculated strengths that produce a safety factor equal to unity are appropriate for existing sliding surface, where the shear strength has been reduced to residual.

Back analysis of Landslides uses the known factor of safety of 1.0 to calculate the shear strength properties along the slip surface. Accordingly, one can determine either (i) the shear strength at the onset of sliding or (ii) the shear strength when the sliding has just stopped moving. For land slide remediation analysis, choice (ii) may be more relevant and would be generally lower than choice (i) in first time slides.

Although it is reassuring to obtain good agreement between laboratory data and the back analyzed strength, it is the back analysis shear strength data that should be used in remedial studies. The reason for preferring the back-analyzed soil strength is that laboratory tests measure only an infinitesimally small part of the shearing surface, whereas back analysis reflects the strength of the entire landslide mass. [6]

2.3.3 Factor of Safety

The factor of safety for slope stability analysis is usually defined as the ratio of the ultimate shear strength divided by the mobilized shear stress at incipient failure. There are several ways in formulating the factor of safety, FS. The most common formulation for FS assumes the factor of safety to be constant along the slip surface, and it is defined with respect to the force or moment equilibrium:

1. **Moment Equilibrium:** generally used for the analysis of rotational landslides. Considering a slip surface, the factor of safety F_m, defined with respect to moment is given by:

$$F_m = \frac{M_r}{M_d} \quad (1\text{-}3)$$

Where: M_r is the sum of the resisting moments

M_d is the sum of the driving moments

For a circular failure surface, the center of the circle is usually taken as the moment point for convenience. For a non – circular failure surface, an arbitrary point for the moment consideration may be taken in the analysis. It should be noted that for methods which do not satisfy horizontal force equilibrium (e.g. Bishop's method), the factor of safety will depend on the choice of the moment point as ' true' moment equilibrium requires force equilibrium. Actually, the use of the moment equilibrium equation without enforcing the force equilibrium cannot guarantee 'true' moment equilibrium.

2. Force Equilibrium: generally applied to translational or rotational failures composed of planar or polygonal slip surfaces. The factor of safety F_f defined with respect to force is given by:

$$F_f = \frac{F_r}{F_d} \qquad (1\text{-}4)$$

Where: F_r is the sum of the resisting forces

F_d is the sum of the driving forces

For 'Simplified methods' which cannot fulfill both force and moment equilibrium simultaneously, these two definitions will be slightly different in the values and meaning, but a single factor of safety is specified for designs. A slope may actually possess several factors of safety according to different methods of analysis. [5]

2.3.4 Slope Stability Analysis

Limit Equilibrium Method (LEM)

A slope stability problem is a statically indeterminate problem, and there are different methods of analysis available to the engineers. Slope stability analysis can be carried out by the limit equilibrium method (LEM). The static limit equilibrium method have two different approaches (a) single free body procedures and (b) method of slices. In the single free body diagram is assumed for the entire mass of the soil is considered to be in equilibrium and a single free body diagram is assumed for the entire mass. Here, infinite slope method, Swedish slip circle method and logarithmic spiral method are some of the examples of single free body methods. But, this method imposes challenges in calculations when used in case of a non-circular or a wedged slip surface. As most of the landslides observed in the real world fail with a non-circular rapture surface, this method was not popularly used. In view of the foregoing, the method of slices was adopted by many engineers to overcome this disadvantage. The method of slices is appropriate to solve both circular and non-circular rapture surfaces. Most of limit equilibrium methods are based on the techniques of slices which can be vertical, horizontal or inclined.

The common features of the methods of slices have been summarized hereunder: [5]

(a) The sliding body over the failure surface is divided into a finite number of slices. The slices are usually cut vertically, but horizontal as well as inclined cuts have also been used by various researchers. In general, the differences between different methods of cutting are not major, and the vertical cut is preferred by most engineers at present.

(b) The strength of the slip surface is modilized to the same degree to bring the sliding body into a limit state. That means there is only a single factor of safety which is applied throughout the whole failure mass.

(c) Assumptions regarding inter-slice fores are employed to render the problem determinate.

(d) The factor of safety is computed from force and/or moment equilibrium equations.

In the method of slices, a failure surface is assumed and the soil mass above the failure surface is divided into slices. A set of stresses is estimated along the potential failure surface so that conditions of global and local static equilibrium are satisfied for the soil mass overlying the slip surface. This state of mobilized stress, which is not necessarily the true state along this surface, provides an approximate solution that can be used to evaluate the factor of safety. A critical slip surface is then searched, for which the factor of safety is minimized. For the computation of the factor of safety, which is defined in terms of strength, the available strength along the slip surface is computed based on Mohr– Coulomb failure criterion.

Limit equilibrium methods have provided a very useful technique for analysis of slopes and other geotechnical engineering problems. However, the procedure does have several inherent weaknesses, which include:

- Incipient failure is assumed at an overall FOS equal to one, which is highly influenced by many variables associated with geological details, material parameters, pore water pressures, and so on.
- The assumtion of constant FOS along the entire slip surface is an oversimplification, especially if different soil materials exist along the failure surface.
- The stress – strain relationship of the soil is neglected, that is, stress –deformation increments and/or decrements within a slope are not simulated by consideration of static equilibrium by itself.

Finite Element Method (FEM)

The limit equilibrium method allows engineers to evaluate the stability of the slopes quickly. However, these procedures are the same whether the analysis considers (1) slope of newly constructed embankment, (2) slope of a recent excavation, or (3) an existing natural slope. The stresses within this slopes are strongly influenced by K_o, the ratio of lateral to vertical normal effective sttresses, but conventional limit equilibrium procedures ignore this important feature. In reality, the stress distributions within these three slopes would be diffferent and hence significantly influence their stability.

The finite element method(FEM) bypasses many of the deficiencies that are inherent with the limit equilibrium methods. For typical cases, the FEM can incorporate incremental construction for embankments and excavations in an attempt to simulate the stress history of the soil within the slope.

The finite element method essentially divides the soil continuum into discrete units, that is, finite elements. These elements are interconnected at their nodes and at predefined bundaries of continuum. The displacement method of formulation of the finite element method is typically used for geotechnical applications and presents results in the form of displacements, stresses, and strains at the nodal points. There are many two and three dimensional computer programs available for finite element anlaysis of slopes and embankments.

Here, in selecting a suiatble program, one must consider:

- The implementation of constituitive model(s) i.e. the types of soil stress-strain relationships that can be used are linear elastic, elasto-plastic, hyperbolic, modified Cam Clay, elasto-visco-plastic and multi-linear elastic models.
- The availability of different types finite elements (e.g. triangular, quadrilateral, or isoparametric).
- The laboratory and field test data required for defining the soil property and hence the soil model.

Inview of the uncertaining and level of familiarity with the FEM, in creating a constituitive model for the analysis, the complex method is not used in this thesis. To this end, the SLOPE/W computer program which is based on limit equilibrium method of analysis was used for the computation of factor of safety (Morgenstern and Price method which satisfies both equilibrium equations and applicable on different slip surfaes) and stability analysis of remediated slopes.[1]

2.4. Remediation of land Slide or Slope Instability

Slope stabilization methods generally reduce driving forces, increase resisting forces, or both. Driving forces can be reduced by excavation of material from the appropriate part of the unstable ground and drainage of water to reduce the hydrostatic pressures acting on the unstable zone. Resisting forces can be increased by

- Drainage that increases the shear strength of the ground
- Elimination of weak strata or other potential failure zones
- Building of retaining structures or other supports
- Provision of in situ reinforcement of the ground
- Chemical treatment (hardening of soils) to increase shear strength of the ground

As an alternative to slope stabilization, the unstable slope can be avoided by adjusting the location of construction or selecting a different site all together. Before the best method can be selected, the actual or potential causes of slope instability must be determined. Quite often there are multiple contributing factors that cause or could cause a landslide or slope instability recurrent.

For remediation of land slide or slope instability one should answer the following questions:

- What is the cause of the cut slope instability?
- What is the amount of remediation needed to maintain stability for reasonably foreseeable future condition?

The methods for correcting and prevention of slope instability related problems are quite many, however a chosen few methods which are related to cut slope instability problems are briefed below. Generally, these methods are control and restraint works. The first three remedial methods are categorized under control works while the fourth method is restraint work.

1. Avoidance: relocation of the highway or road stretch affected by the slide from the limits of the slide area by provision of realignments. This method is cost effective if the relocation is for short stretches of the road section.
2. Movement of Earth: reduction in shear stresses on the slip surface can be achieved by removal of slide material. This can be achieved by the removal of slide material from the head (upperpart) and by flattening of the slide mass.
3. Drainage: provisions results in reduction in shear stresses and increase in shear strength of the soil. Here, drainage can be achieved by the provision of surface and sub-surface drainage provisions. Surface drainage is the provision of surface ditches (furrow ditches) above the crown of the slide mass and the treatment of slide slopes by the construction of rock covering or permeable apron on the surface of sliding mass to control the flow. Furthermore, horizontal and vertical drainages are used to intercept and divert underground water in large masses of soil with underground flow and deep seated sliding masses, underground water in strata or lenses, respectively.
4. Retaining Structures: these structures are basically constructed at base and toe of the sliding surface. Accordingly, support at the base can be achieved by the provision surcharge at toe by construction of rock or earth fill while common or crib retaining wall structures are constructed at the base/toe of the slide mass to restrict movement of the slide mass. Furthermore, piles (concrete or steel) are fixed in the slip surface in order to increase the strength of the slip surface. Alternatively speaking, the strength of the failure surface is increased by the amount of the stress required to make the piles fail. [1]

The most common use of retaining walls for slope stabilization is when a cut or fill is required and there is not sufficient space or right- of -way available for just the slope itself. The wall should be deep enough so that the critical slip surface passes around it with an adequate FOS.

Gravity retaining walls, mechanically stabilized earth (MSE) walls, which are not pre-stressed, must move before they can develop resistance to stabilize a land slide. Such walls can be designed using the following three main steps:

a. Using conventional limit equilibrium slope stability analyses, determine the force required at the location of the wall to stabilize the slope (i.e. to raise the factor of safety of the slope to the desired value). These analyses can be performed using any method in which an external force of specified location, direction, and magnitude can be included. The analyses are performed using repeated trials. The magnitude of the force is varied until the desired factor of safety is achieved. Each of the analyses with a new trial force should search for the location of the critical slip surface. This critical slip surface is not the same as the critical surface with no stabilizing force but is often close to that surface. The magnitude of the stabilizing force can be determined with acceptable accuracy using a force equilibrium analysis, with the direction of the slide forces between slices assumed to be the average of the slope inclination and the failure surface inclination, or using a method that satisfies all conditions of equilibrium. The position of the stabilizing force can reasonably be assumed to be about 0.4H above the bottom of the wall to the surface of the slope.[1]

b. Using conventional retaining wall design procedures, determine the external dimensions of the retaining wall, MSE wall, or soil nailed wall required for global wall stability, with the force determined in step(a) applied to the wall. The considerations for external stability of the wall include sliding, overturning, bearing capacity, position of the resultant force on the base, and deep -Seated sliding (failure through the foundation beneath the wall).[3] Accordingly, the different wall stability criteria (Static Case) are:[1]

- Factor of safety Sliding ≥1.5
- Factor of Safety Overturning ≥ 2.0
- Factor of Safety Global ≥ 1.2 - 1.5
- Factor of Safety Bearing capacity ≥ 3.0

Furthermore, for remedial structure constructed in seismically active locations one needs to consider the dynamic lateral earth pressure exerted on the wall due to earthquake loadings. It is believed that the increased lateral earth pressure during earthquakes induces sliding and/or tilting to the wall structure. Because of the complex soil-structure interaction (mode of wall movement) during earthquakes, the lateral earth pressure theory based on the fully plastic solution (also known as pseudostatic method) which is widely used by most of the design engineers, is adopted in the design of the retaining walls. For Gravity and Cantilever walls, the procedure most widely used in practice is the Mononobe - Okabe method. This method is based on the theory of fully plastic solution or Pseudostatic method where the stability of Coulomb type wedge in a manner similar to that of static analysis is used while an equivalent seismic load assumed to act at the center of gravity of the wedge with horizontal and vertical magnitudes equal to the mass of the wedge times the horizontal and vertical accelerations respectively.[10]

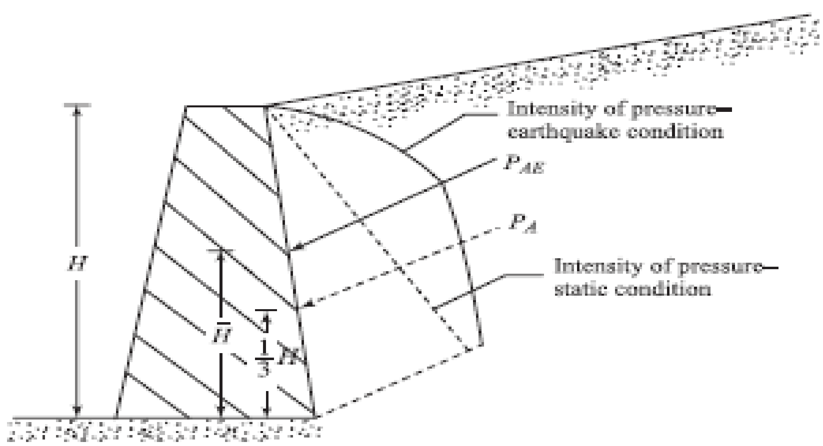


Figure 2.7: Point of application of the resultant active force

The line of action of the resultant force from static and dynamic lateral active pressures (Reference is made to Figure 2.7) is determined for [11]:

- Rotation about the bottom of the wall: the procedures shall be applied as shown in Figure 2.8 below.

$$\bar{H} = \frac{(P_A)\left(\frac{1}{3}H\right) + (\Delta P_{AE})(0.6H)}{P_{AE}}$$

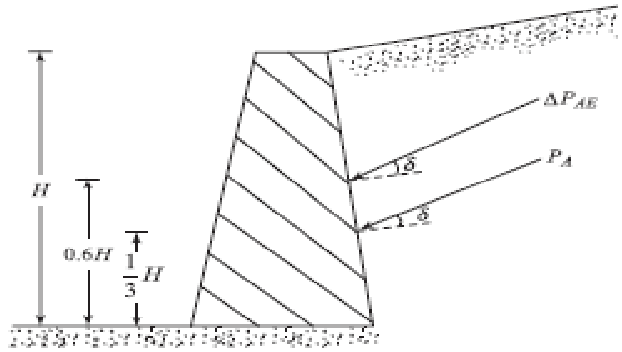


Figure 2.8: Line of action for rotation of the wall about the toe.

- Translation of the wall: the method proposed as per Figure 2.9 shall be applied.

$$\bar{H} = \frac{(P_A)(0.42H) + (\Delta P_{AE})(0.48H)}{P_{AE}}$$

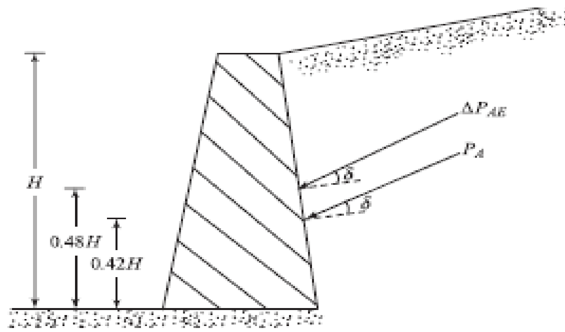


Figure 2.9: Line of action for translation of the wall.

The factor of safety for Pseudostatic method of analysis of mechanically stabilized earth structures is taken to be 75% of the static stability criteria stated above. [6]

The forces that should be considered in the design of retaining walls include:[2]

- Active and passive earth pressures: these pressures should be calculated using the appropriate theories. In most cases, however, because of its simplicity, one uses the Coulomb's Earth Pressure Formulae. The distribution of the contact pressure under the base of the retaining wall is assumed to be planar. Hence the usual flexural formula is used.
- Dead weight including the weight of the wall and portion of the soil mass that is considered to act on the retaining structure
- Surcharge including live loads, if any
- Contact pressure under the base of the structure

c. Using conventional design procedures, evaluate the requirements for internal strength. For gravity walls, these include the shear and moment capacity of the footing and stem. For mechanically stabilized earth walls, these include the length of reinforcement, and spacing of reinforcement. For soil nailed walls, these include nail capacity, nail length, and nail spacing.

3. Brief Description of the Road Project

3.1 Background Information of the Road Project

The Federal Democratic Republic of Ethiopia (FDRE), represented by **Ethiopian Roads Authority**, has allocated sufficient budget for the Detailed Engineering Design, Tender Document Preparation of **Arbaminch – Kemba - Sawla Road Project (~138KM)**. The consultancy service for the Detailed Engineering Design, Tender Document Preparation of the project has been undertaken by **MH Engineering PLC** as per the agreement signed with the **Ethiopian Roads Authority** on the 10th of October 2007.

In line with this, **Diriba Defersha General Contractor** has signed a contract with Ethiopian Roads Authority for the construction of **Belta- Otolo Road Project(~30.3KM)**, which is a part of Arbaminch – Sawla Road Project: Lot 2: **Belta(KM60+000) - Otolo(KM87+000)**. The road project begins at Belta town which is the end control point of Lot 1: Arbaminch –Belta Road Project.

The scope of the contract is the construction of 30.3 km Road to DS5 Double Surfaced dressing standard with an initial project amount of ETB: 623,018,262.70. The construction period for this contract was 1095 calendar days (Three years), running between September 23, 2011 and September 22, 2014.

In the course of preparations for construction, the contractor, Diriba Defersha GC, had identified sections with design gaps which are preliminarily found difficult and require design revision. Accordingly, from the point of view of multi-dimensional aspects of highway engineering design and construction, the original design was reviewed and clear and major design gaps have been identified including:

- Very deep box cut cross sections
- Steep and long grades
- Steep grades at Switch back curves
- Triple handling of material due to switch back curves at box cuts
- Ground profile grades well above ruling gradients for route location
- Gaps in material slope identification

Finally, Contractor's, design review team has concluded that the design gaps and the limitations negatively affect the economy, safety, serviceability, construction, operation, maintenance and aesthetics of the project road and need to be improved and hence their negative impacts should be minimized and/or avoided. Accordingly, improvement opportunities which had mainly included two major realignments identified as Realignment 1 and Realignment 2 which generally follow the natural contours in harmony with the surrounding natural topography and mostly resulted in hill side cut cross sections that flash with the ground at the valley side and that minimize box cut cross sections, and steep grades were proposed, investigated and designed.

The investigations and design activities included route location, topography surveying, geotechnical investigations, geometric design, hydrology-hydraulics analysis and design and structural design of these realignments. The results of the investigations and the design had revealed that the proposed realignments have minimized or avoided the identified major design gaps and limitations enhancing the project road serviceability, safety, stability, and reducing the social and environmental impacts.

Accordingly, the Employer, ERA, with due consultation with the Supervision Consultant, Pure Consulting Engineers, had approved the design review prepared by the Contractor which also resulted in initial construction cost savings amounting 46.76mln ETB.[7]

3.2 The Road Project Location

The Arbaminch-Sawla project road is located in the southern part of the country in Southern Nations and Nationalities People Regional State with its start at a village called Shele Mazoria, 20Km from Arbaminch on the Arbaminch – Jinka road and connects Arbaminch to Sawla via Kemba town. The project road, Belta –Otolo, forms the middle lot of the Arbaminch – Sawla road project stretching between Belta and Otolo towns from KM 60+000 to KM 87+000 (~30.3kms). The Control points of the project are Belta Town, Kemba Town and Otolo Village as depicted in Figure 3.1 below.

3.3 Accessibility to the Project

The Arbaminch Sawla project road starts from a junction on Arbaminch (Gamo Zone) - Jinka road which is 470Km from the capital along Addis Ababa – Buta Jira – Hossaina- Sodo – Arbaminch – Jinka road and traverses in the westerly direction to the end of the project, Sawla Town (Gofa Zone), passing through a number of towns and rural villages with a total estimated length of 138.30Km. Out of the total length, the first 60km section of the project has been allocated under Lot 1 and the second lot starts from Belta town which is 30.3 km section of the road project.

The project road will shorten the additional 270Km trip from Arbaminch to Sawla, through Arbaminch – Sodo – Sawla, by nearly half length and will also connect the towns and many villages of the two ethnic groups, Gamo and Gofa in the Gamo and Gofa Zones. Reference is made to Figure 3.1 and 3.2 below.

3.4 Land Use and Cover

The land use pattern along the road alignment can be defined as mostly cultivated land with the small portion covered by bushes, shrubs, eucalyptus trees, bamboo trees and Enset. Bushes cover limited part of the project area, which is used for grazing. Mostly people are dependent on the farming and cattle breeding which is main livelihood for them.

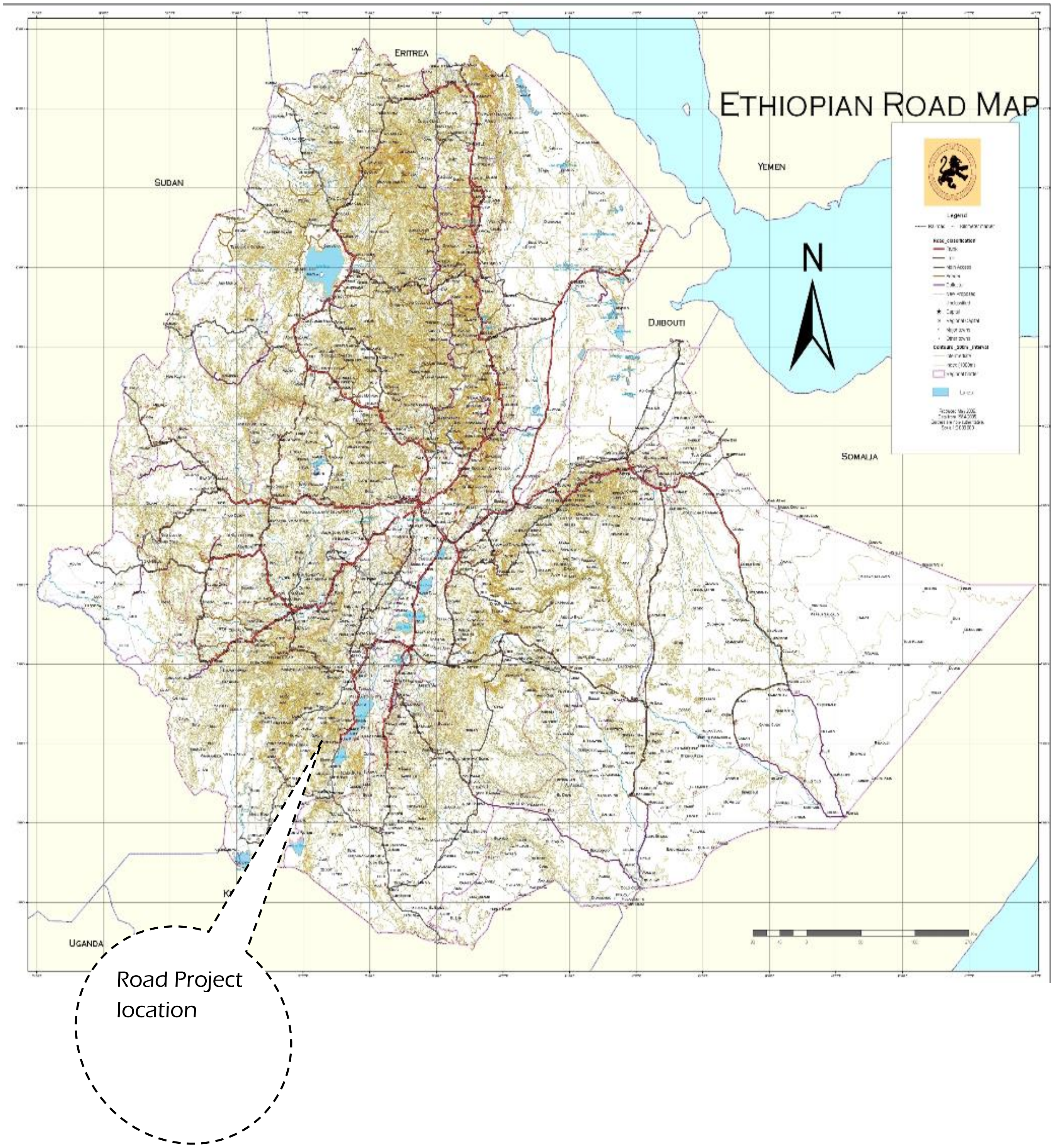


Figure 3.1: The road project (study area) general location

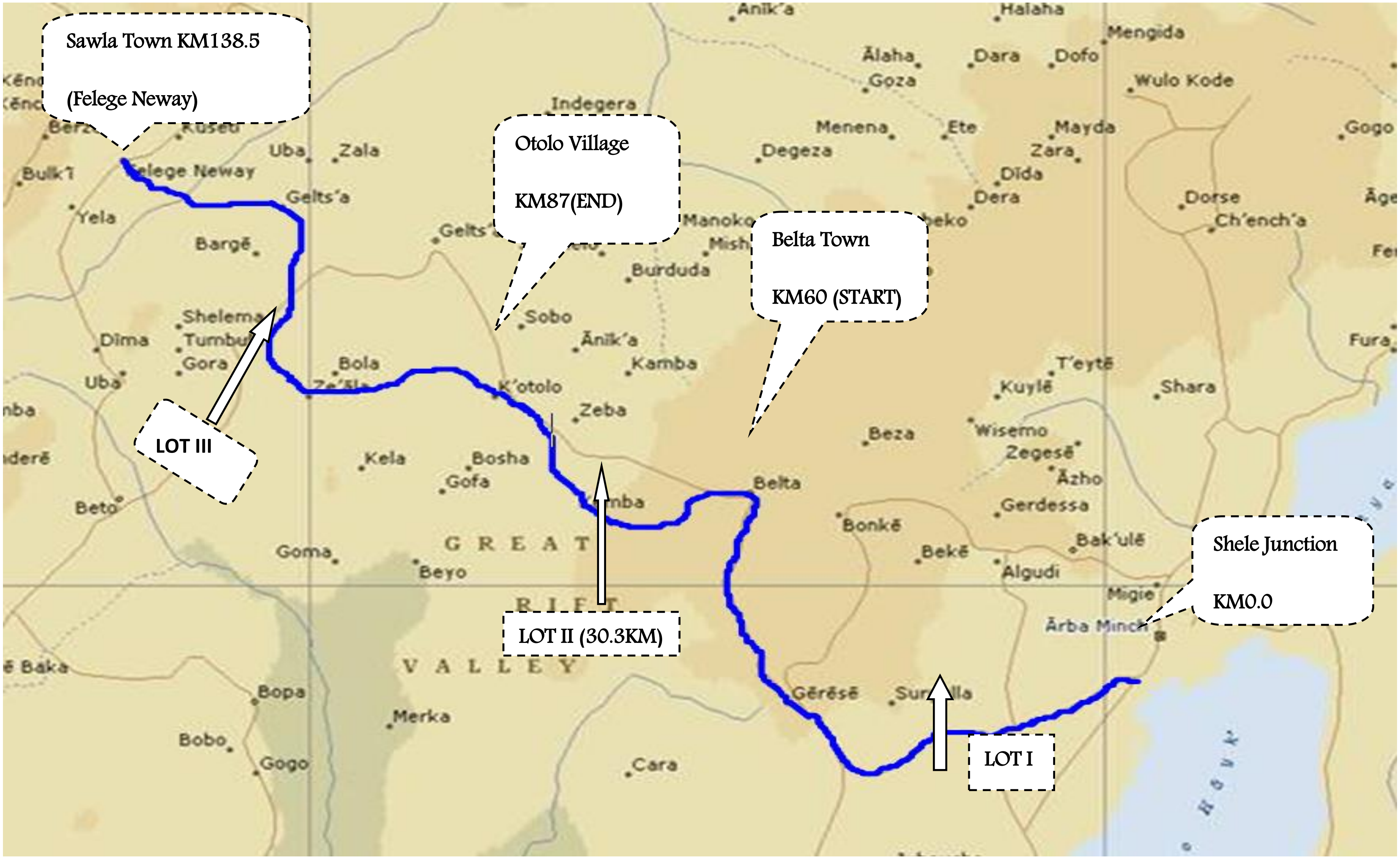


Figure 3.2: The project road location (study area) and the road alignment with control points

3.5 Topography and Climate

Topography and Terrain of the Project Area

The terrain of the road alignment can be generally described as escarpment/mountainous over the large portion of the road with some sections characterized by rolling and flat physiographic features. From the terrain classification road, the stretches from Sta.67+000 to Sta.67+060 and from Sta.68+020AHD to 69+140BCK are classed as escarpment(transverse terrain slope in excess of 50%) while the stretch Sta.72+740 to 72+790LHS is classed as mountainous(transverse terrain slope from 25% to 50%) topography as depicted in Figures 3.3 and 3.4 below.

Figure 3.3: Vertical profile of designed road alignment

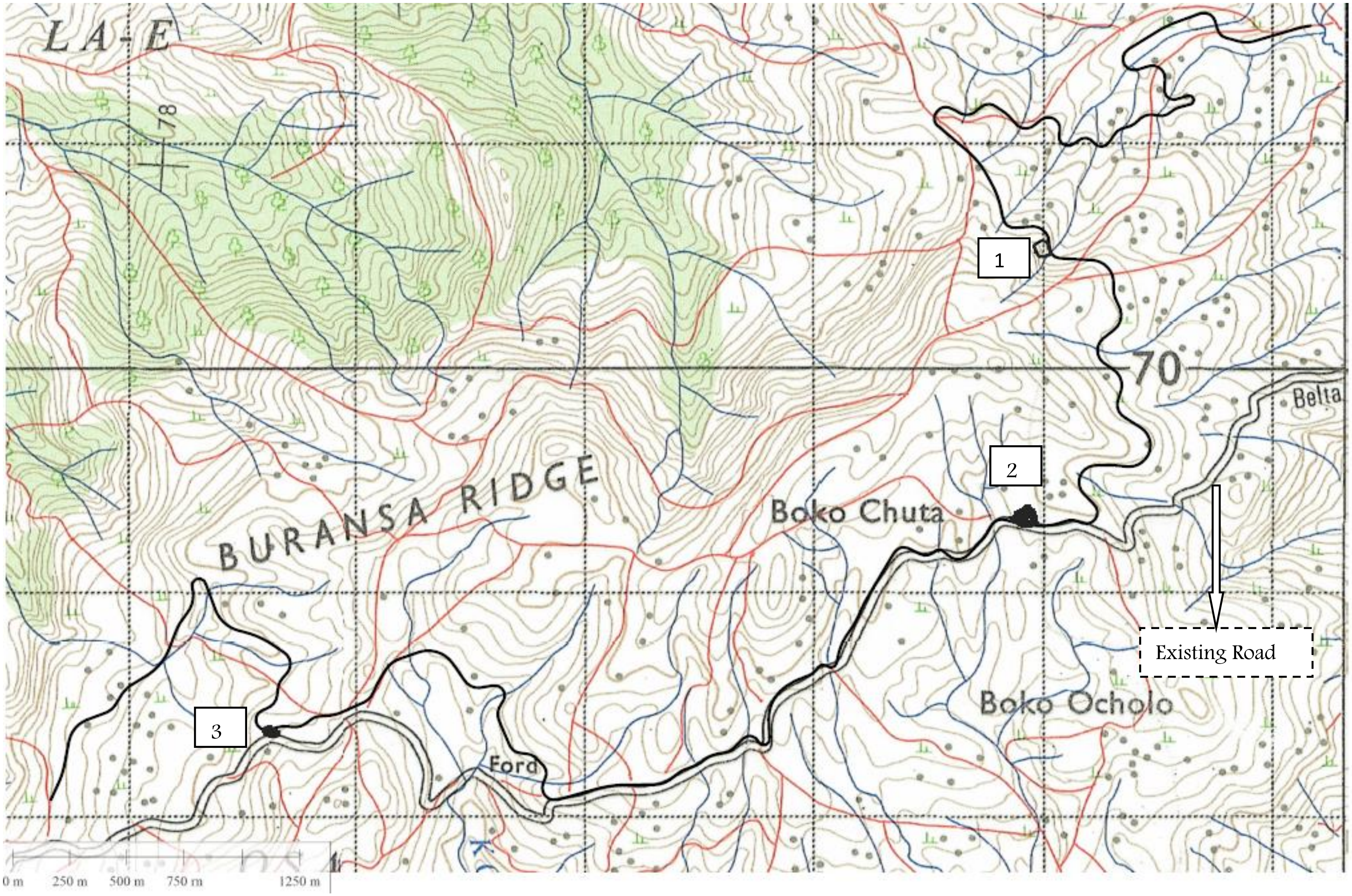


Figure 3.4: Topographic map and slide Location 1, 2 & 3 with the designed road alignment

Climate of the Project Area

Rainfall: According to ERA (2002) drainage design manual the project road is located in rainfall regime B2. The rainfall in the project area is bimodal - the first rainy season is from March-May and the second is from September-November. Rainfall increases as one goes from the beginning of the project to the end of the project i.e. about 900 mm/year to 1400 mm/year in Sawla.

Temperature: Under normal conditions, air temperature decreases with increasing altitude. The Mean maximum and minimum temperature at the road project is 34 degree centigrade in february and 12 degree centigrade in december respectively. [7]

3.6 Geology of the Project Area

Regional Geology

The Jima volcanic rocks cover most part of South west highlands of Ethiopia. Thick basaltic base rock and thin Salic rocks form the dominant terrain throughout the Gamo-Gofa zone and also thin layer 1-5m of residual reddish brown and light yellowish silty clay soils on top in hilly and mountainous terrains as shown under Figure 3.6 below. The residual soils get thicker in rolling terrains. The volcanic rocks are dark in color, fine grained, aphanitic to porphyritic texture, massive to widely jointed, and moderately fractured. They cover the hilly and mountainous sections of the project area. The Alluvial and Lacustrine deposits from the flat terrains of the route with reddish silty clay soil formations and brownish clay soil in very flat terrains which have poor sub grade and drainage characteristics. They cover the flat sections with reddish silty clay soil formations and brownish sand and silts around the rivers.

Pertaining to the regional geological formation the sub grade soils along the project route are generally similar. The pre-dominant sub-grade soil is reddish to reddish grey clay soil. They are residual soils formed from decomposition of parent rock from weathering.

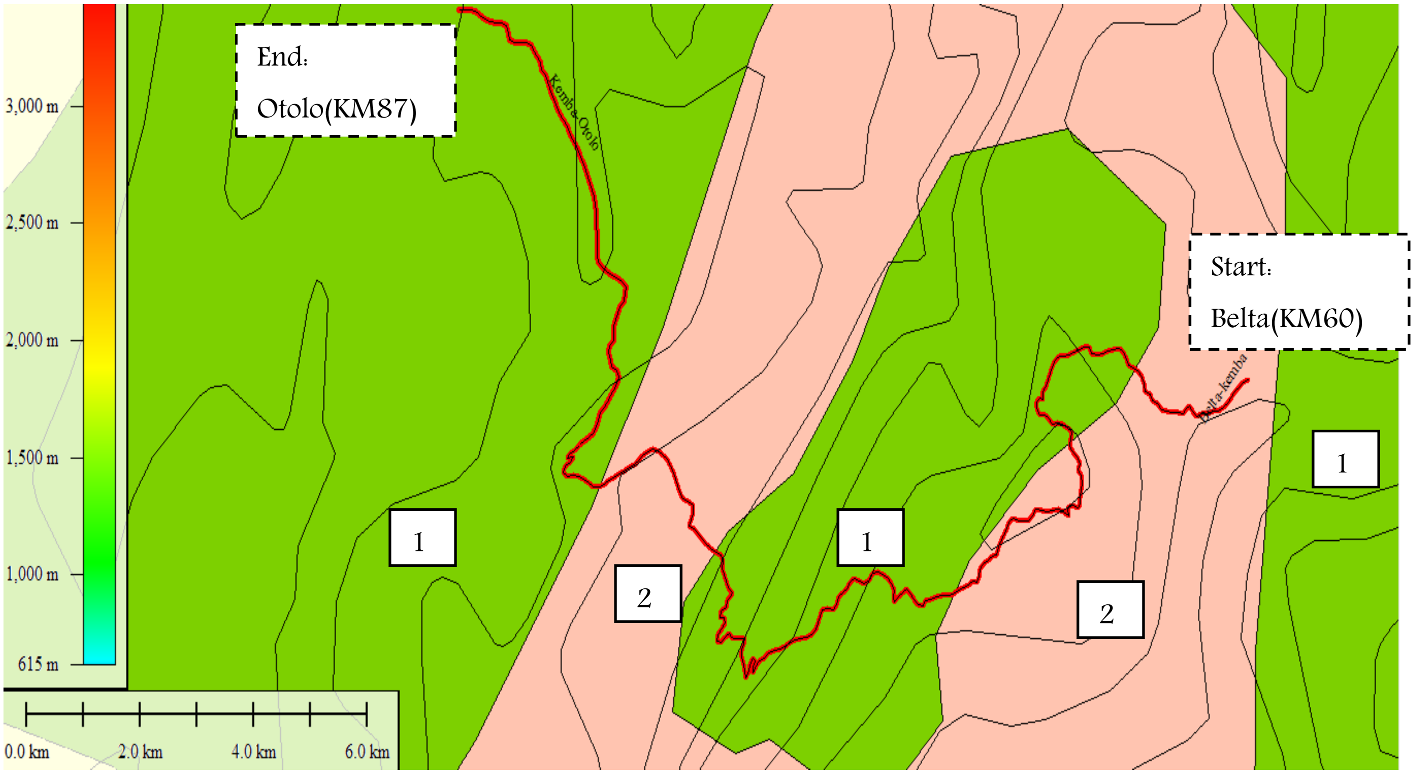


Figure 3.5: Regional geological setting of the designed road corridor

1 **Jima volcanic:** Basalt with Pyroclastic ash, tuff and rhyolite.

2 **High grade metamorphic rocks:** Biotite quartzo-feldspathic gneiss.

Local Geology

The two major dominant litho logical groups observed in the project road are: first, the high grade metamorphic rock composed of biotite bearing quartzo-feldspathic gneiss which is mainly observed at the start of the project and all the way up to 62+000km. As far as it is the oldest formation of the region it occupies low topography and is characterized by light grey color, well foliated and banded with alternating colors of light and dark grey minerals, completely weathered and highly decomposed, easily dozeable. Similar rock exposure is also exposed in the valleys between Kemba town and Korza River.

The second dominant lithologic unit exposed overlying the basement rocks is volcanic rocks of Jima group which are mainly forming the high topography, sharp ridge, mountainous and plateaus, but locally exposed as slope deposits composed of angular rock fragments of rhyolite, and lithic trachyte, tuffs and associated ash mostly exposed at the foot of slopes, especially as an isolated boulders and blocks at the foot of Dolja mountain slopes.

The geomorphology of the road project area as a whole seems to be a reflection of the underlying geology with steep-sided mountainous and escarpment formed of rhyolites and trachytes and associated saddles of strong rock units and long, gentle slopes and topographic breaks found on soft rocks, with basaltic plateau.[7]

Figure 3.6: Well foliated and banded biotite bearing stratum of quartzo-feldspathic gneiss exposed at Km60+200

Figure 3.7: Isolated rock boulder at the foot of mountain chain of Dolja, resulted from rock fall

4. Selected Unstable(Slide)Cut Slope Locations of the Road Project

4.1 Sta.66+995 – Sta.67+055(RHS)

4.1.1 General Features of the Cut Slope Failure (Slide)

This land slide (excavation initiated slope failure) had occurred after the excavation within the section was finalized to the sub grade level. The common excavation to spoil activity in this slide section was performed in box cut location while the LHS cut slope was removed in order to facilitate side wasting of the excavated material on the existing natural slope.

Rotational type of slide through the base of toe of the excavated slope with downward (from top of the slope) and outward (from the slip surface) movement of the soil mass peculiar to rotational type of slide with composite slide surface has been observed. The material on the exposed part of the excavated RHS slope looks visually homogeneous with different beddings of soil layer dipping in to the toe of the slope vertically and dipping from the RHS to the LHS slide boundary longitudinally while the material on top part of the slide plane (boundary) is distinctly granular. It is observed that the slide had originally started at the outer boundary of the slide plane after the removal of toe support by excavation. Here reference is made from Figure 4.2 to 4.4.

From the surveying of the cut slope failure, the plan area, the surface area and the perimeter covered by the sliding mass are 4381.79m^2, 3765.20m^2 and 232.27m respectively. According to the classification of landslides, this slide is categorized as medium landslide (Cut Slope Failure) [2]. During the field investigation i.e. before and after failure of the cut slope, no seepage or ground water flow is observed on the slope as referred from Figure 4.1 below.

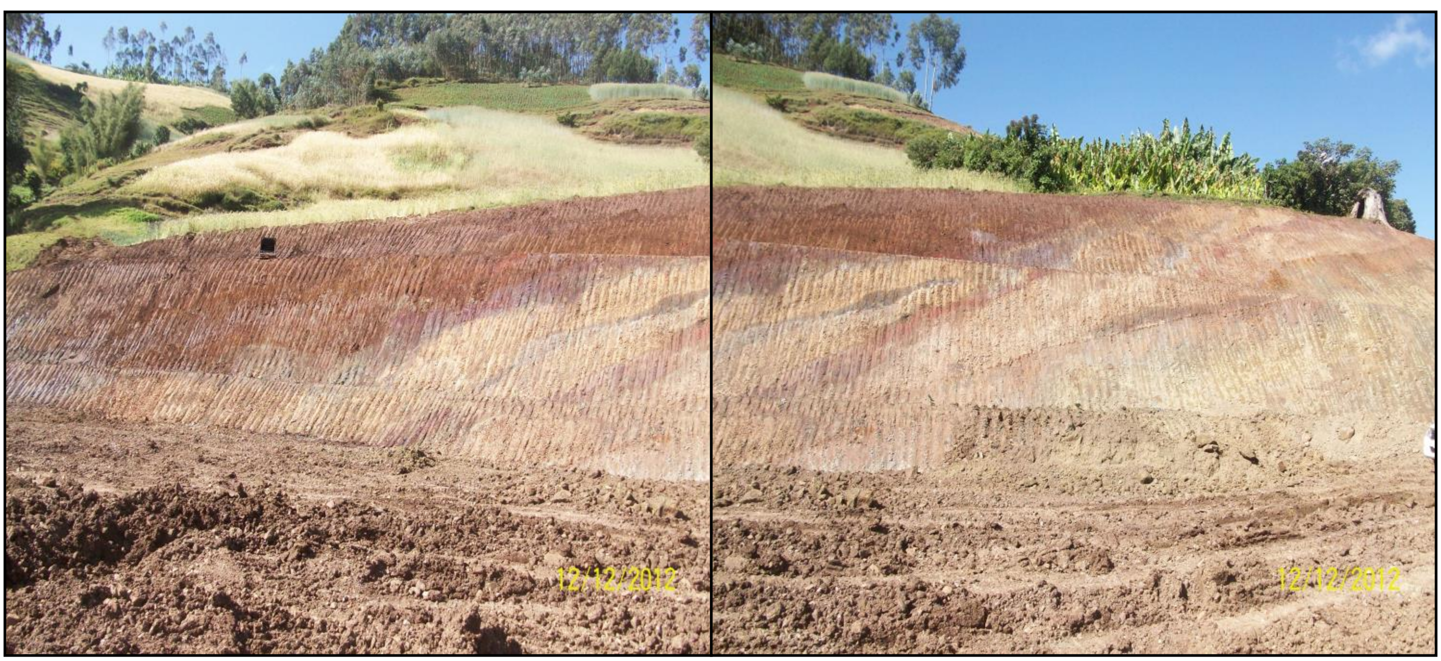


Figure 4.1: LHS and RHS limits of the excavated slope before failure of cut slope

Figure 4.2: Front view of the cut slide manifestation

Figure 4.3: Top view of the cut slide boundary: main scarp

Figure 4.4: View of the failed cut slope RHS and LHS boundary

The geometrical parameters of the designed slope are shown in Figure 4.5:

- Slope height RHS=13.64m (Two benches at 6m height with 1:1 slope + Stepped with 2 berms provided for 3m length and 10% dipping angle in to the slope + 1mini bench at the toe of the slope)
- The depth of Excavation at center of the road is 10.39m while LHS and RHS Excavation limits (offsets) from the centerline of the road are 13.79m and 26.32m respectively.
- The designed volume of common excavation to spoil within the limits of this slide is 20,593cu.m of earth.

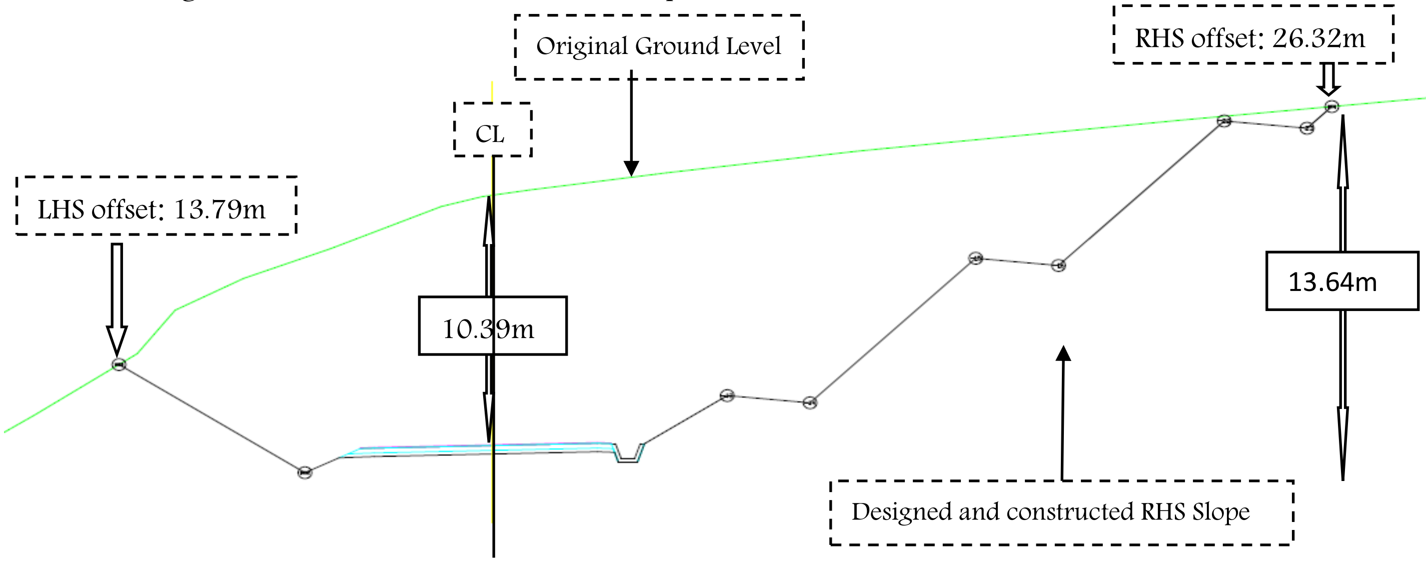


Figure 4.5: Designed typical section at Sta.67+030

Similarly, the geometry of exposed slide plane and slide material are described here under Figure 4.6:

- The location of the Crest and Toe of the failed soil mass are 63.28m and 1.95m, respectively from the Centerline of the road. The height of the main scarp varies over the slide boundary from 5 to 7.5m at the center, from 2 to 4.5m at the LHS and from1.5 to 2.5m at the RHS as depicted in Figures 4.3 and 4.4 above.
- The Crest height when measured from the design sub grade level is 19.14m with dipping ground profile in to the LHS as shown in Figure 4.6.

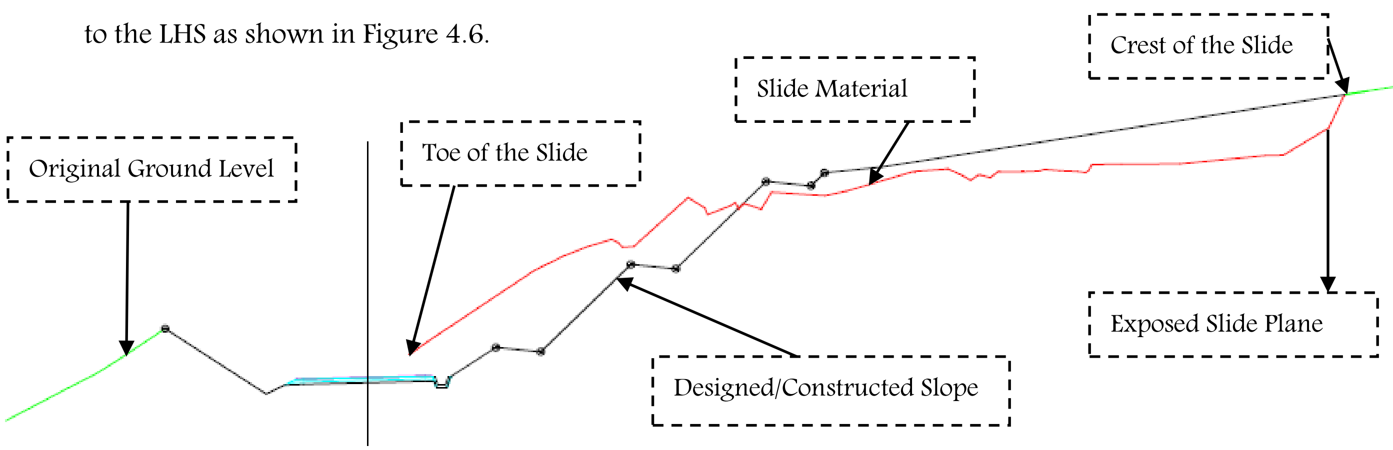


Figure 4.6: Typical slide section at Sta.67+030

The dimensional properties of the slide from the typical slide cross section of Figure 4.6 and plan drawing of Figure 4.7 are tabulated hereunder:

Table 4.1: Dimensional variables of the slide from Sta.65+995 to Sta.67+055(RHS)

Dimensional Variable	**Measured Value(m)**	**Remark**
Length of Displaced mass, L_d: **the minimum distance from tip to the top.**	Tip=1.95m and Top=60.76m from Centerline of the Road. Hence, **L_d= 58.81**	The horizontal distance from the X-sectional drawing is taken, Figure 4.6.
Total Length, L: **The minimum distance from the tip of the land slide to its crown.**	Tip=1.95m and crown=63.29m from Center line of the Road. Hence, **L=61.34**	Crown: Practically un displaced material adjacent to the highest parts of main scrap.
Length of Rupture Surface, L_r: **The minimum distance from the toe of the surface of rupture to the crown.**	Toe=4.9m and Crown=63.29m from Center line of the Road. **L=58.39**	The toe of the slide plane is used for measurement.
Width of Displaced Mass, W_d : **the maximum breadth of the displaced mass perpendicular to the length L_d**	**W_d=66.76**	The width perpendicular to the cross-section at 67+030 is used.
Width of Rupture Surface, W_r: **The maximum width between the flanks of the landslide, perpendicular to the length, L_r.**	**W_r=66.76**	Reference is made to the plan drawing, Figure 4.7.

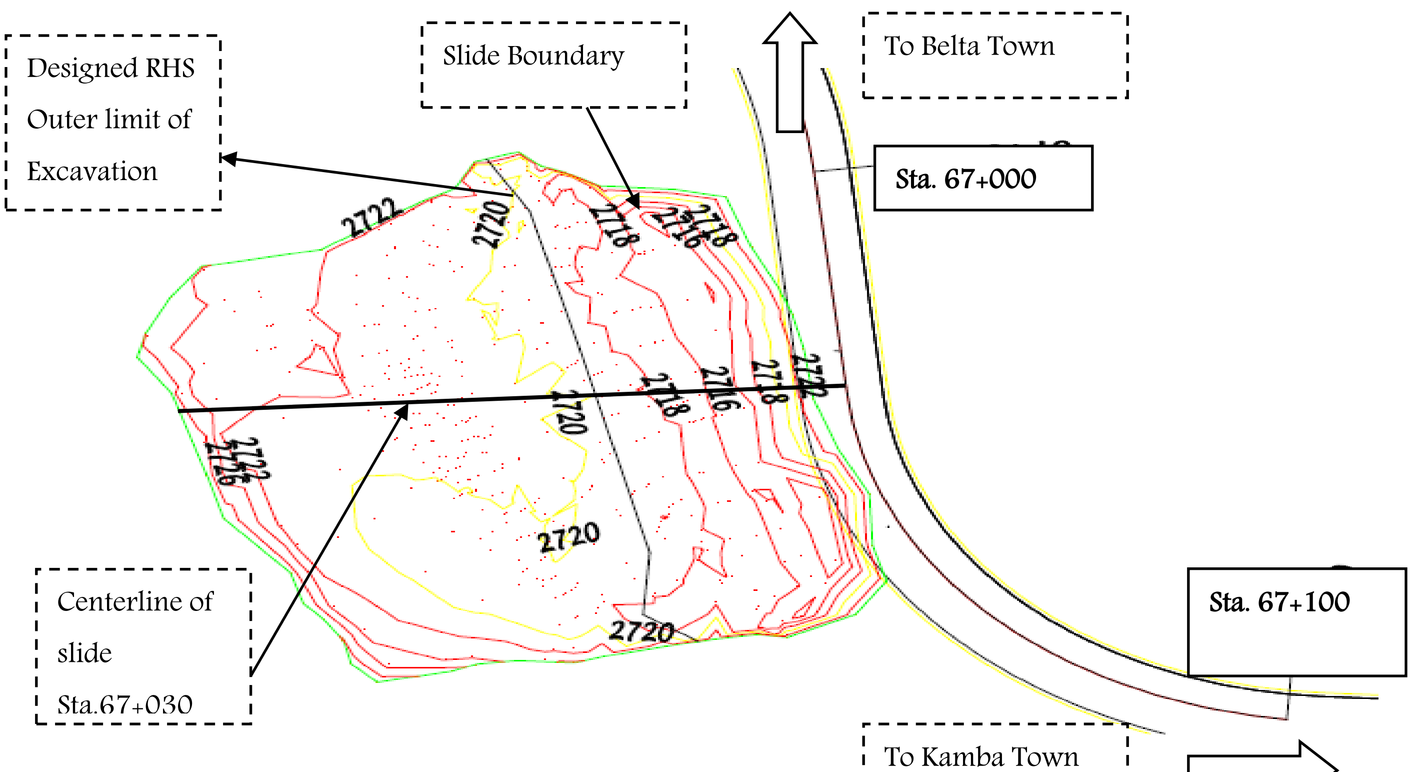


Figure 4.7: Plan drawing of the slide mass with contour

4.1.2 Sampling, Testing and Cut Slope Failure(Slide) Characterization

Undisturbed and disturbed soil samples from the slide plane and slide material respectively were sampled and tested for classification, strength determination and hence for the characterization of the slide mass. From the tests, one can determine the strength parameters and index properties for the slide plane and index properties for slide mass, which eventually is useful to characterize the slide mass and sliding plane. All the index and strength test reports which were tested in Diriba Defersha GC project site (contractor's) laboratory, Construction Design Share Company's and Gondwana Engineering's material testing departments are annexed under Annex A.

Table 4.2: Description of sample location and test results

Sample location	Tests and results	Slide material and slide plane characterization
Station : 67+030RHS Offset from centerline: 60.8m, Depth from the OGl:6.5m Sample type: disturbed material sampled from: exposed slide plane	Atterberg Limits: • Natural moisture content, W_n:26.9% • Specific Gravity=2.23 • Unit weight(bulk),γ_b:16KN/m^3 • Unit weight(dry),γ:13.05KN/m^3 Sieve + hydrometer Analysis: Using ASTM classification, • Pass no. 200 sieve: 40% • clay fraction: 17% • silt fraction: 23% • sand fraction: 28% and, • gravel fraction: 32%.	➢ According to Sieve + Hydrometer analysis: % retained on sieve No.200: >50% hence, soil is course grained. According to ASTM D 422; Sand=28% and Gravel =32% Hence , **Sandy gravel with acceptable amount of fines i.e. silt and clay**
Station : 67+030RHS Offset from Centerline: 10.56m Depth from the OGl:11.88m(2.10m from the sub grade) Sample Type: Disturbed Material Sampled from: Slide material highly plastic weak layer.	Atterberg Limits: • Liquid Limit, LL=60% • Plastic Limit, PL=37% • Plasticity Index, PI=23% • Cohesive Index, $CI = \frac{PI}{PL}$ =0.62 Sieve + Hydrometer Analysis: • Pass No.200= 65% • Gravel=0%, • Sand =35%, • Silt =65% and • Clay=0%	➢ According to the classification of fine grained soils, ASTM, where 50% or more by weight passing sieve No.200: A-Line =0.73(LL-20) Below A line LL >50 % Retained on sieve No.200:>30% Sand=35% and Gravel =0% Sandy Silt soil ➢ According to classification of silts and clays based on Cohesive Index, CI:[2] CI =(0.6-0.8) : Very Silty Clay ➢ According to Sieve + hydrometer analysis: **Sandy silt soil with few**

		traces of clay
Station : 67+030RHS Offset from Centerline: 10.56m Depth from the OGl:12.38m(2.30m from the sub grade level) Sample Type: Disturbed Material Sampled from: Slide plane material	Atterberg Limits: • Liquid Limit, LL=56% • Plastic Limit, PL=36% • Plasticity Index, PI=20% • Cohesive Index, $CI = \frac{PI}{PL}$ = 0.56 Sieve+ Hydrometer Analysis: • Pass No.200 =60% • Gravel =33%, • Sand =17%, • Silt =49% and • Clay=11%	➢ According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200: A-Line =0.73(LL-20) Below A line LL >50 Gravely silt soil ➢ According to classification of Silts and clays based on Cohesive Index, CI:[2] CI =(0.4-0.6) : Clayey silt soil ➢ According to Sieve + hydrometer analysis: **Gravely silt soil with traces of sand and clay**

Owing to the insitu formation of the soil, i.e. residual soil formation, all index tests are carried out as per the procedures set out for testing of residual soils [8].

From the index test results (sieve + hydrometer) it was observed that:

- ➢ The material on the slide plane ranges from gravel with considerable amount of fines(top exposed part) and gravely silt soil(at the toe /interface of the slide plane) while the displaced or failed cut slope material ranges from clayey to sandy silt soil. From the foregoing it can be stated that, the material on the slide plane and the failed or displaced material are distinct.
- ➢ The overall slope material is heterogeneous, and the slide plane is approximated by composite failure surface and;
- ➢ Relatively the same material extends below the toe of the slope as the exposed slide plane material as depicted in the slope profile and index tests conducted;

Figure 4.8: Granular exposed part of the slide plane material

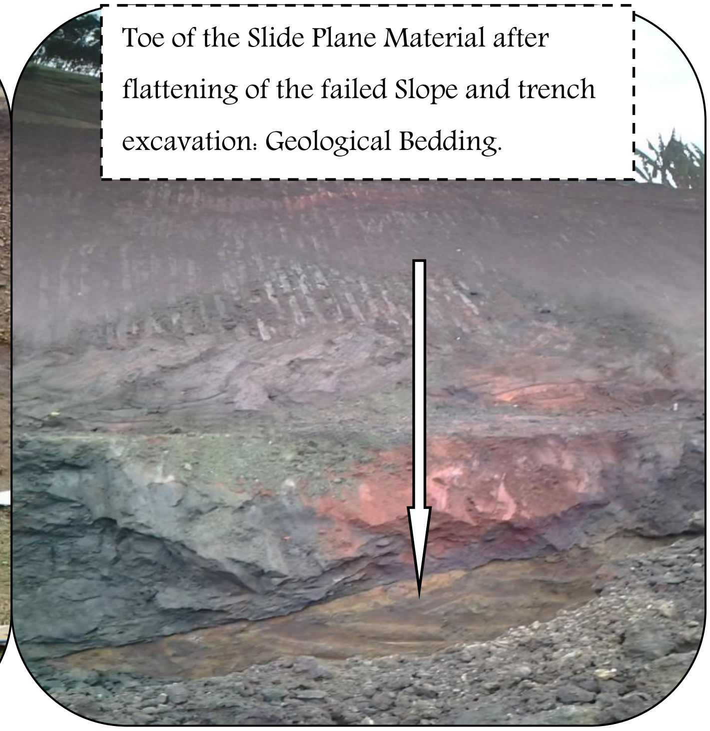


Figure 4.9: Toe of the slide plane material

4.1.3 Analysis of Slide or Cut Slope Instability

The cut slope was failed during the construction period due to undermining of the toe of the existing slope, and the existence of distinct bedding and layering(which are taken as weakened planes) of the in situ soil coupled with moisture migration from surface run off during the rainy seasons. Furthermore, the reason for failure of the cut slope was also that steeper slope angle of 45^0 or 1:1 slope ratio, when compared with the height of the bench cut, was constructed. The design of the cut slope should consider the insitu material types, the soil moisture and the uncertainty over long term (end of construction) ground conditions rather than just taking the preliminary design slope angles which were given in the contractual design standards which had substantially proved working under homogeneous cohesion less (sandy soil) soil profiles for the first 6-8km of the project road. [7]

Since the slope material is heterogeneous, the mobilized shear strength between the two materials along the slip surface should be simulated in the lab to determine the shear strength parameters of the slope during failure. However, due to the inherent difficulties in the classical testing of such heterogeneous materials (between slip surface and slide material) by simulating the actual site conditions in the laboratory, the use of back analysis is of prime importance under such conditions.

Accordingly, analytic model for back analysis using the data obtained from the site is created in order to determine the residual shear strength parameters on the slip surface which will be used for the design of remedial structure and reinstatement of the slide mass.

In order to approximately identify the slip surface for use in back analysis of the cut slope, the following points and line segment of the slide plane were used as depicted under Figure 4.10 below and Figures 4.8 and 4.9 above:

- Toe location and the depth of the Slip Surface (bedding) i.e. 1.4m and 2.3m below the designed sub grade level
- The outer Crown of the slide surface i.e. at 63.28m offset from the centerline.
- The top exposed part of the slide plane, head scarp i.e. from the Crown to the Top of the displaced soil mass i.e. 4.86m

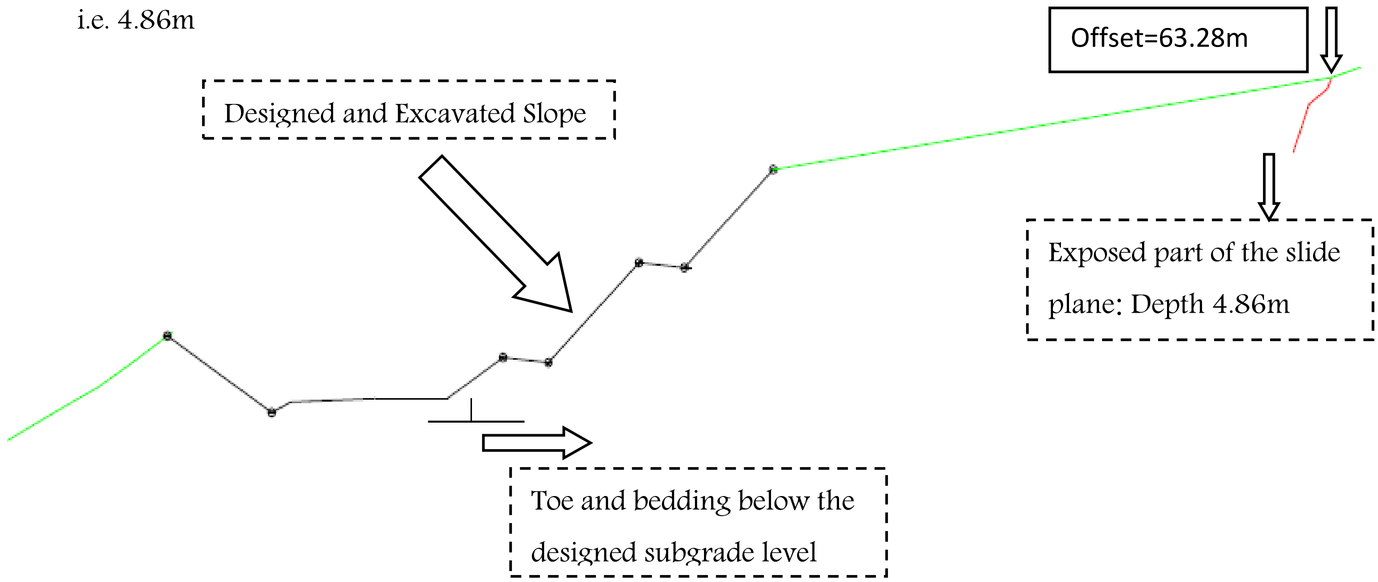


Figure 4.10: Approximated toe and crown of the Slide Surface Location

Next, Back Analysis method coupled with the actual location of the slip surface revealed from site investigations shall be used to back calculate the strength parameters on the slip surface just at the onset of failure of the cut slope.

Accordingly, the steps and the assumptions in back analysis for this failed cut slope shall involve the following:

i. Assume a combination of strength parameters (c and ϕ)(Table 4.3) to result in a range of dimensionless parameter related to the depth of the slip surface, $\lambda_{c\phi}$, as given in equation (1-5) below;

$$\lambda_{C\emptyset} = \frac{\gamma H \tan \emptyset}{C}, \qquad (1\text{-}5)$$

Where:

γ: is unit weight of the soil (kN/m^3), take=13.87kN/m^3

c and $\emptyset$: are Strength parameters (kPa and degrees, respectively)

H: The Cut Slope Height (m), take 13.64m

Here, the strength variables are selected so as to produce a range of dimensionless parameters ($\lambda_{c\phi}$) by taking the range of expected strength values in the failed cut slope, but the values did not necessarily produce a factor of safety of 1.

Table 4.3: Assumed strength variables and dimensionless parameter

Assumed Strength Variables	c(kpa)	40	30	40	25	15	5	10	5
	∅(Deg.)	5	10	20	15	10	5	15	10
Dimensionless Parameter,	$\lambda_{c\phi}$	27.00	10.00	7.00	6.00	5.00	4.00	3.00	2.00

ii. Create the slope geometry in Geo studio 2004 SLOPE/W Software as shown on Figure 4.11 using data from the designed slope and slide plane as depicted in figures 4.5, 4.6 and 4.10 above.

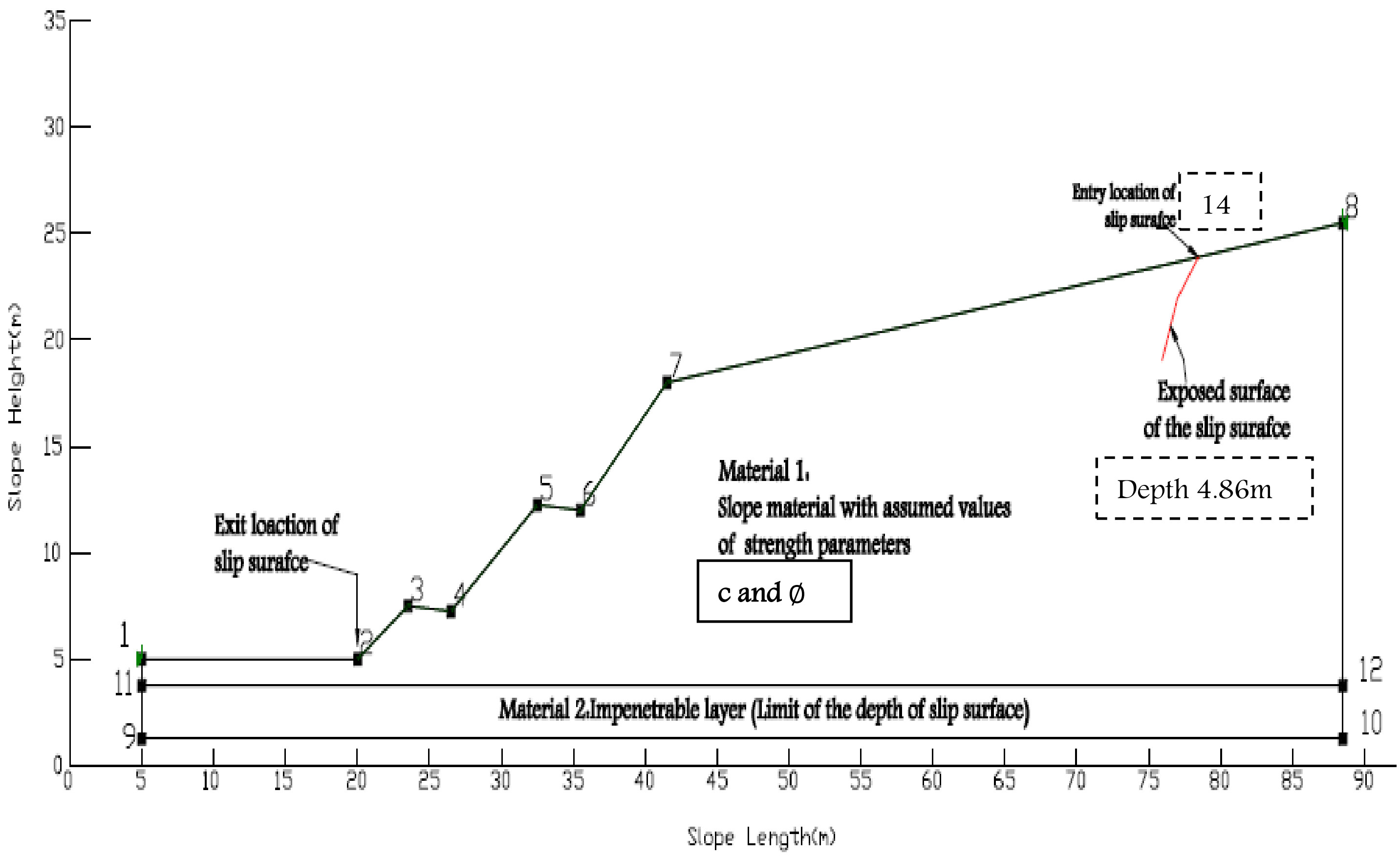


Figure 4.11: Slope Geometry in Geo Slope/W 2004

iii. The slope is characterized in Geo slope Slope/W as soil material underlain by bed rock or impenetrable layer to simulate the interface between the distinct slope materials and the observed depth of the slip surface. Further, in the analysis of the slope, the slip surface is construed to pass through the toe of the slope and crest of the slide plane by Entry and Exit Option, reference is made from Figure 4.11.

iv. In the analysis of the slope for factor of safety determination Morgenstern-Price Method is used, since the slide surface is presumed to be composite one.

v. Using the combination of strength parameters (c and $\emptyset$) assumed in (i) calculate the corresponding factor of safety and the depth of the slip surface referenced from reference point 14 as shown on figures from C.1 to C.9 which are annexed in Appendix C. In this analysis output, the reference point 14 is the location of the scarp while the reference point 13 is the actual location of exposed part of the rapture surface determined from field observation. In this regard, the depth of the slip surface is measured directly from reference point 14 and the location of the slip surface above or below reference point 13 by exporting the results of the analysis to AUTOCAD. Reference was made to figures from 4.1 to 4.8 under appendix C.

vi. Calculate the developed shear strength parameters for Factor of Safety corresponding to FS=1, from (v) above, using equations 1-6 and 1-7, where the results of the calculations are tabulated under Table 4.4 below;

$$\emptyset_d(\text{adjusted}) = \tan^{-1}\left\{\frac{\tan\{\emptyset_d(trial)\}}{FOS}\right\} \quad (1\text{-}6)$$

$$c_d(adjusted) = \frac{C_d(trial)}{FOS} \quad (1\text{-}7)$$

Table 4.4: Summary of analysis results for the back calculated shear strength parameters

S/N	Dimensionless parameters $\lambda_{c\emptyset} = \frac{\gamma H \tan\emptyset}{c}$	Assumed strength variables c	Assumed strength variables ϕ	Factor of Safety(M-P) Critical Surface Entry and Exit	Calculated (developed) strength variables for FS=1, Entry and Exit c_d	ϕ_d	Depth of the critical surface at Pt 14(m) Critical Surface Entry and Exit
1	27	40	5	1.748	22.88	2.87	6.2
2	10	30	10	1.762	17.03	5.72	3.51
3	7	40	20	2.801	14.28	7.41	3.52
4	6	25	15	1.893	13.21	8.06	3.51
5	5	15	10	1.19	12.61	8.43	3.52
6	4	5	5	0.498	10.04	9.97	3.52
7	3	10	15	0.688	14.53	21.29	3.52
8	2	5	10	1.893	2.64	5.32	3.52
			Slope Height=13.64m		Bulk Unit Weight=13.87kN/m^3		

vii. Next, prepare a plot of the developed strength parameters (c_d and $\emptyset_d$) corresponding to a Factor of Safety, FS=1 and the depth of the slip surface measured in (v) above from analysis results.

viii. Finally, calculate from the plot prepared in (vii) above the corresponding back analysis strength values (c and $\emptyset$) corresponding to the actual depth of slip surface i.e. 4.86m, reference is made to figures from 4.1 to 4.8 as indicated in appendix C.

ix. Accordingly, from the use of back analysis method, the results of the developed internal friction angle and cohesion are 4.25^0 and 20kPa respectively as indicated in Figures 4.12 and 4.13 below.

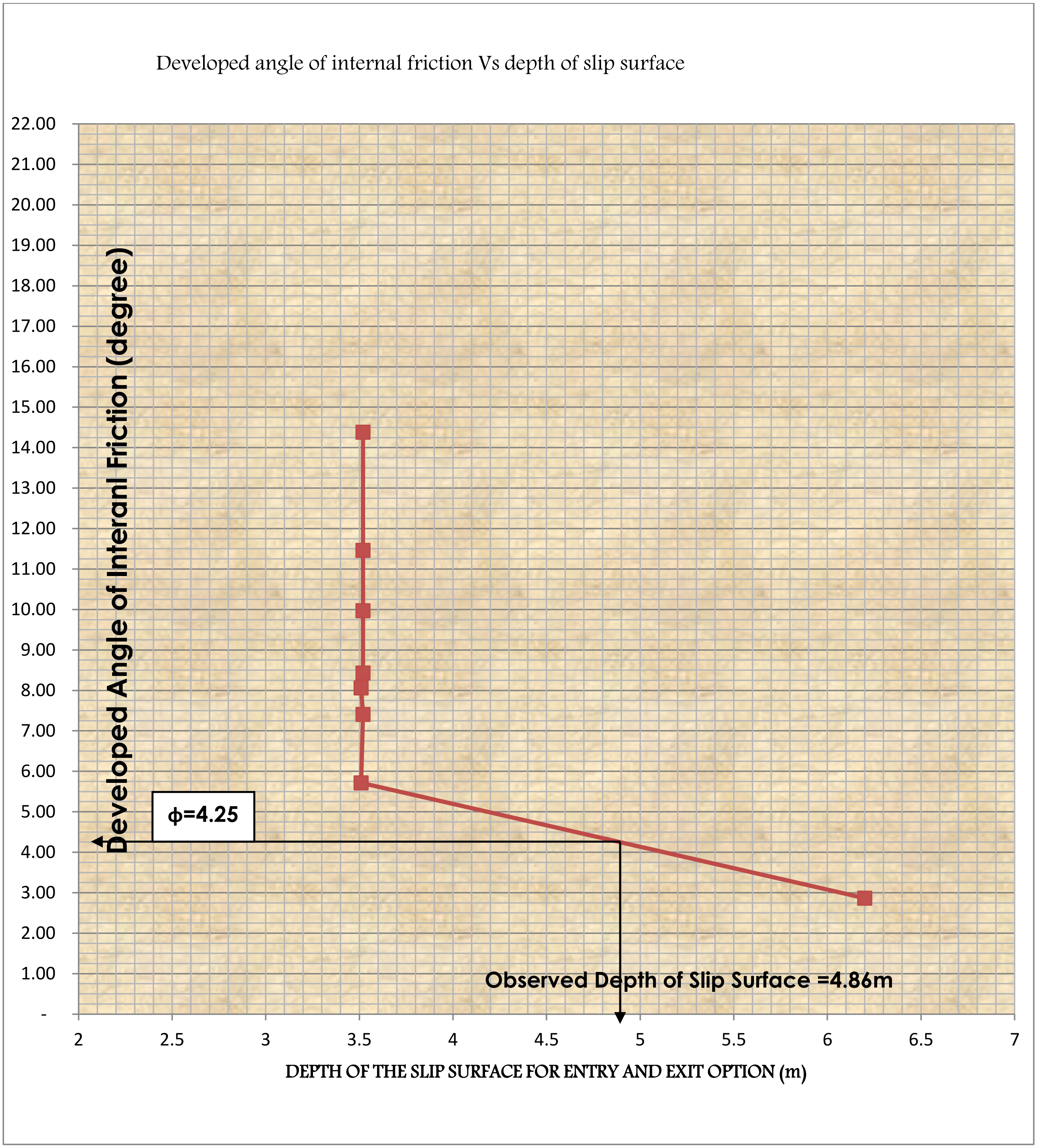


Figure 4.12: Back calculated friction angle Vs depth of critical slip surface

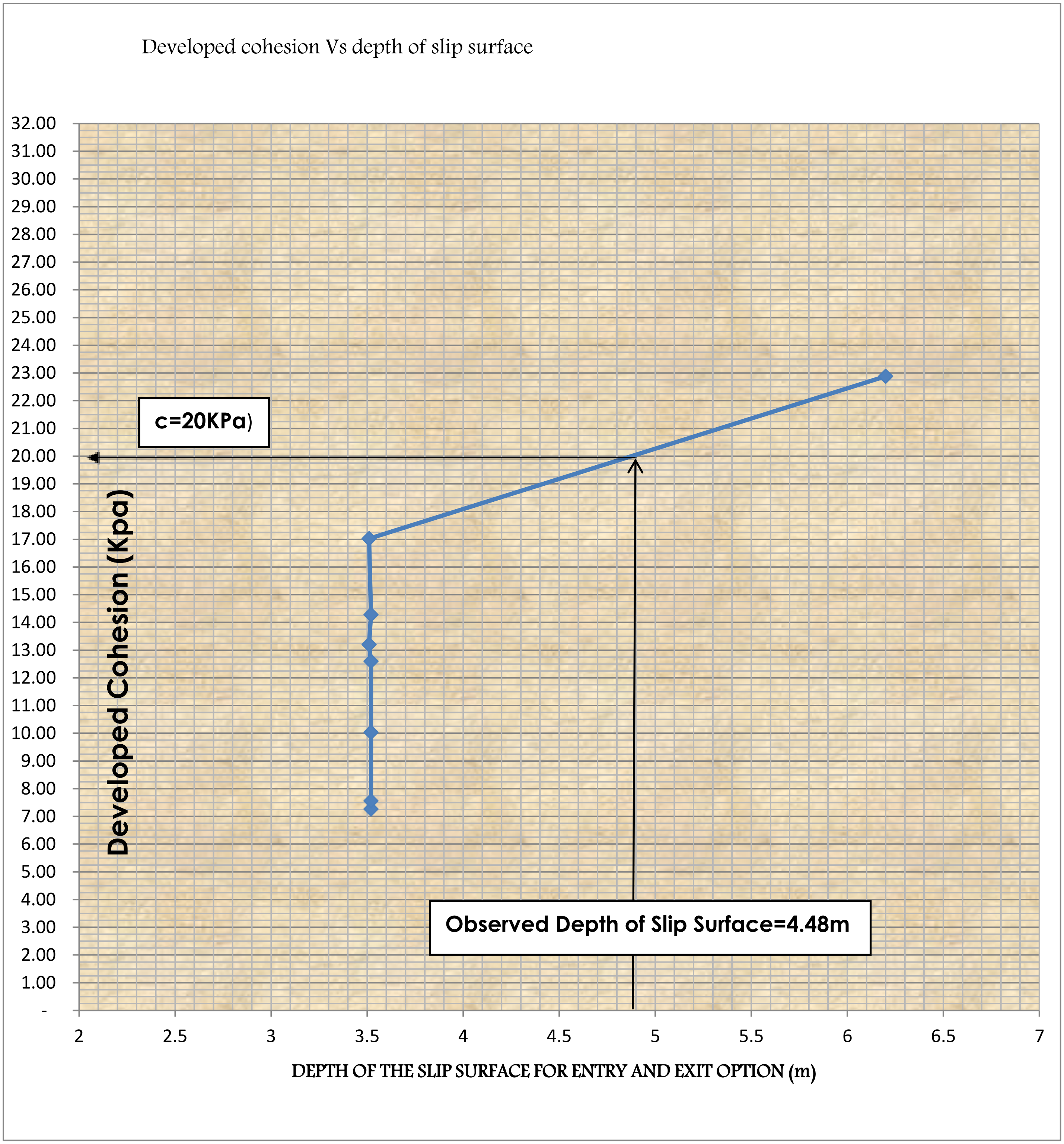


Figure 4.13: Back calculated cohesion Vs depth of critical slip surface

4.2 Sta. 68+020(Ahead) – 68+120(Ahead)RHS

4.2.1 General Features of the Cut Slope failure(Slide)

The land slide (cut triggered slope failure) occurred after the excavation within the section was finalized to the depth of sub grade level in box cut section except on some width of LHS side of the road cross section which was left for traffic accommodation. A rotational type of slide through the base toe of the excavated slope with a downward (from top of the slope) and outward (from the slip surface) movement of the soil mass peculiar to rotational type of slide has been observed. Initially, during the progress of the common excavation works, wedge type of slope failure day lighting on the cut slope was observed on the slope as shown on Figure 4.14 and progressed to a distance about 80.4m from the center of the road to a stable position as per the typical slide cross section made at Sta.68+080 RHS, Figure 4.16.

The total plan area, surface area and perimeter covered by the sliding mass are 7488.46m^2, 9300.84m^2 and 337.01m respectively. According to the classification of landslides this slide is categorized as medium landslide (Slope Failure). [2] During field investigation, no seepage or ground water flow was observed on the slope or failed mass.

Figure 4.14: Initial Wedge Slide day lighting on the slope during progress of excavation work

Figure 4.15: Common Excavation work on progress

The designed slope of the road shown as in Figure 4.16 below has the following geometrical or dimensional properties:

- Slope Height RHS=12.25m (Two benches at 6m height with 1:1 slope + Stepped with 2 berms provided for 3m length and 10% dipping angle in to the slope + 1mini bench)
- The depth of Excavation at center of the road is 9.36m while LHS and RHS Excavation limits (offsets) from the centerline of the road are 18.76m and 26.47m respectively.
- The designed volume of common excavation within this slide section is 29,363Cu.m of earth.

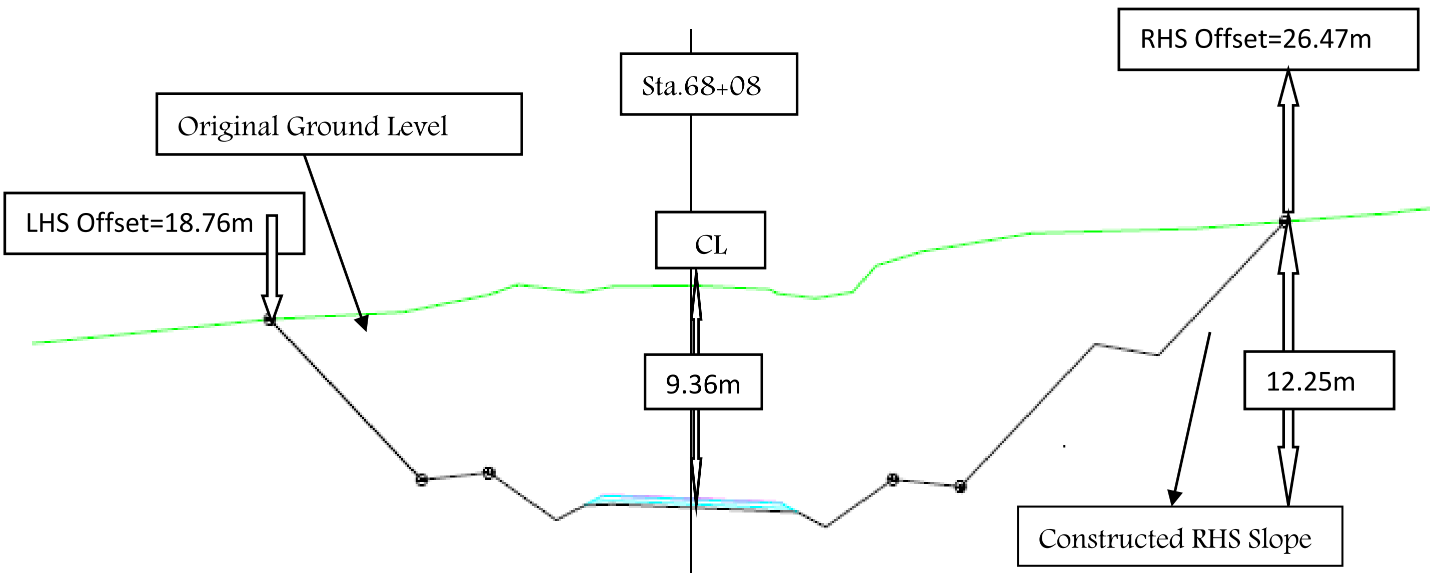


Figure 4.16: Designed typical road Section at Sta.68+080

Slide Plane and slide material:

The location of the Crest and Toe of the failed soil mass are 80.37m and (-) 10.07m, respectively from the Centerline of the road. Crest Height when measured from the design sub grade Level is 18.29m with dipping ground profile in to the LHS as shown in Figure 4.26 below. The slide is characterized as retro progressive slide as there was continued subsidence on the top part of the slide.

The front part of the slide mass with a width of 10-12m has translated in the form of a block from the excavated slope for a distance of 5-7m forward which decreases in width as one moves to the LHS and RHS. The top part of the slide mass has moved vertically (subsided) for a depth of 2-5m over two thirds of the slide cross section.

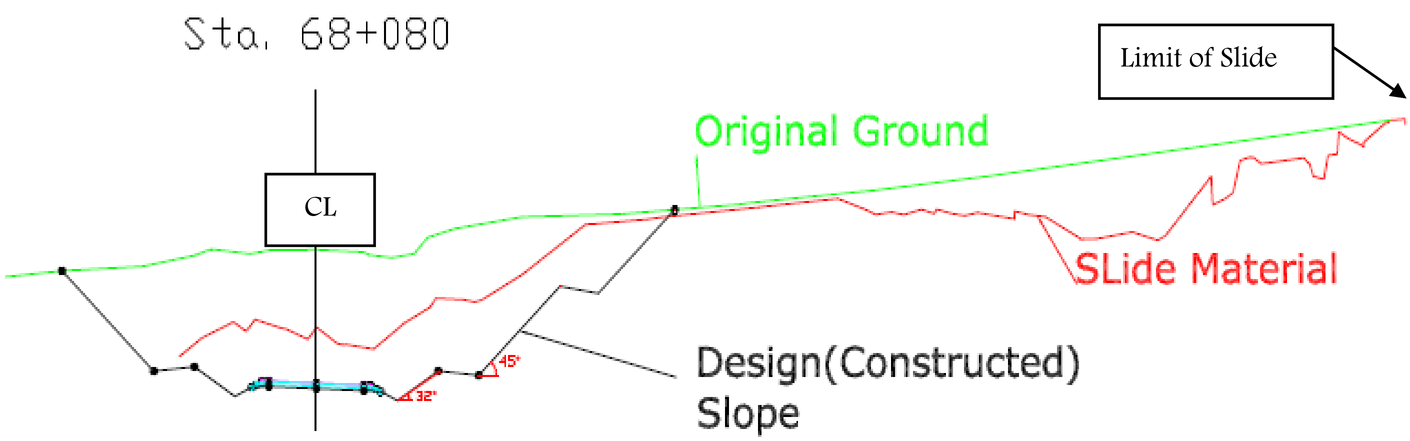


Figure 4.17: Typical slide section at sta.68+080 showing the slide extent

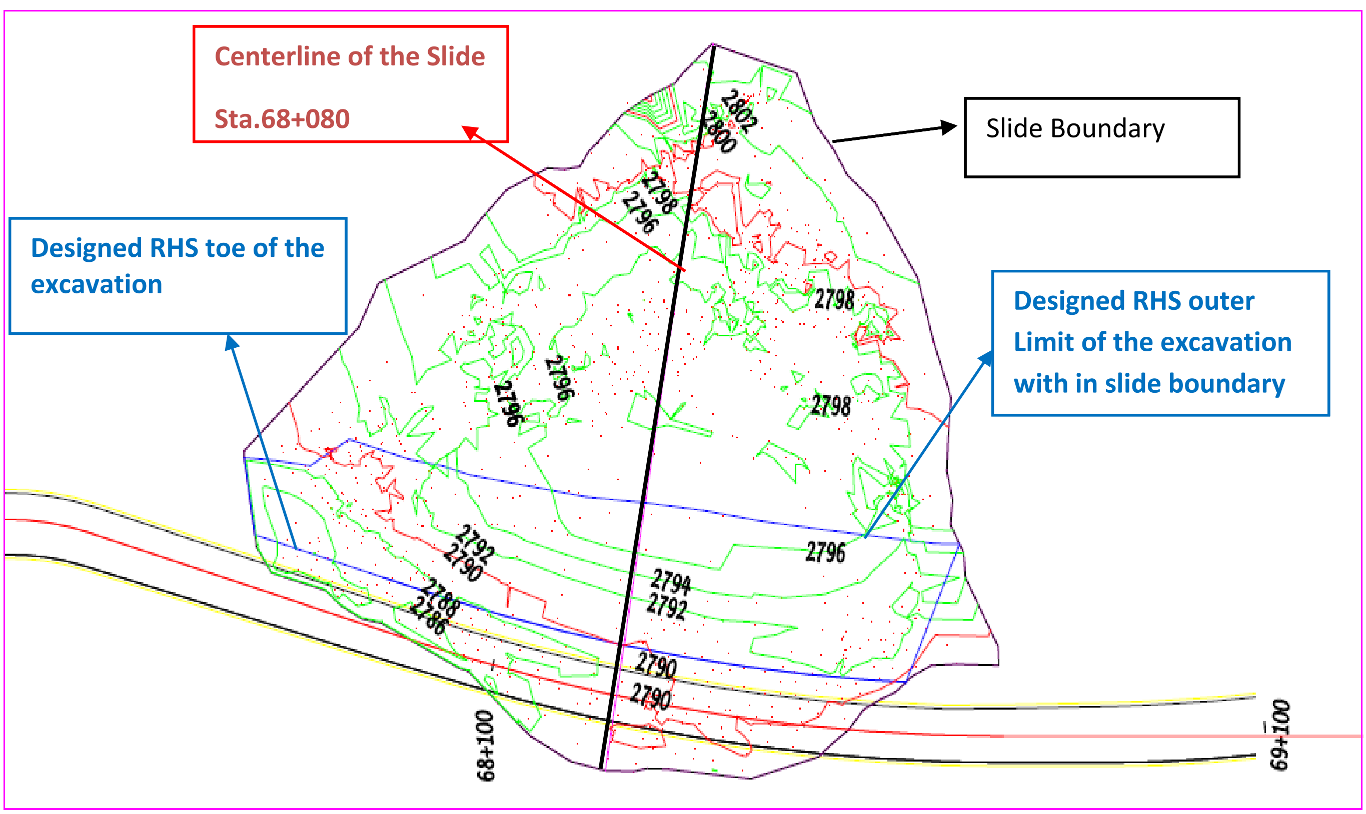


Figure 4.18: Plan drawing of the Slide mass with contour lines

The dimensional Variables of the slide calculated from the typical slide cross section(Figure4.17) and plan drawing(Figure 4.18) are tabulated hereunder Table 4.5.

Table 4.5: Dimensional variables of the slide 68+020RHS (ahead) - 68+140(RHS)

Dimensional Variable	Measured Value(m)	Remark
Length of displaced mass, L_d: the minimum distance from tip to the top.	Tip=-10.07m and Top=80.00m from Center line of the Road. Hence, **L_d= 90.07m**	The horizontal distance from the x- sectional drawing is taken, Figure 4.17.
Total Length, L: The minimum distance from the tip of the land slide to its crown.	Tip=-10.07m and Crown=80.37m from Center line of the Road. Hence, **L=90.44m**	Crown: Practically un displaced material adjacent to the highest parts of main scrap.
Length of rupture surface, L_r: The minimum distance from the toe of the surface of rupture to the crown.	Toe=6.81m and Crown=80.38m from Center line of the Road. **L=73.57m**	The toe of the slide plane is used for reference.
Width of displaced mass, W_d : the maximum breadth of the displaced mass perpendicular to the length L_d	**W_d=118.87m**	Reference is made to the plan drawing of the slide, Figure 4.18.
Width of rupture surface, W_r: The maximum width between the flanks of the landslide, perpendicular to the length, L_r.	**W_d=118.87m**	Reference is made to the plan drawing of the slide, Figure 4.18.

Figure 4.19: Slide at Start of Sta.68+020(Ahead)

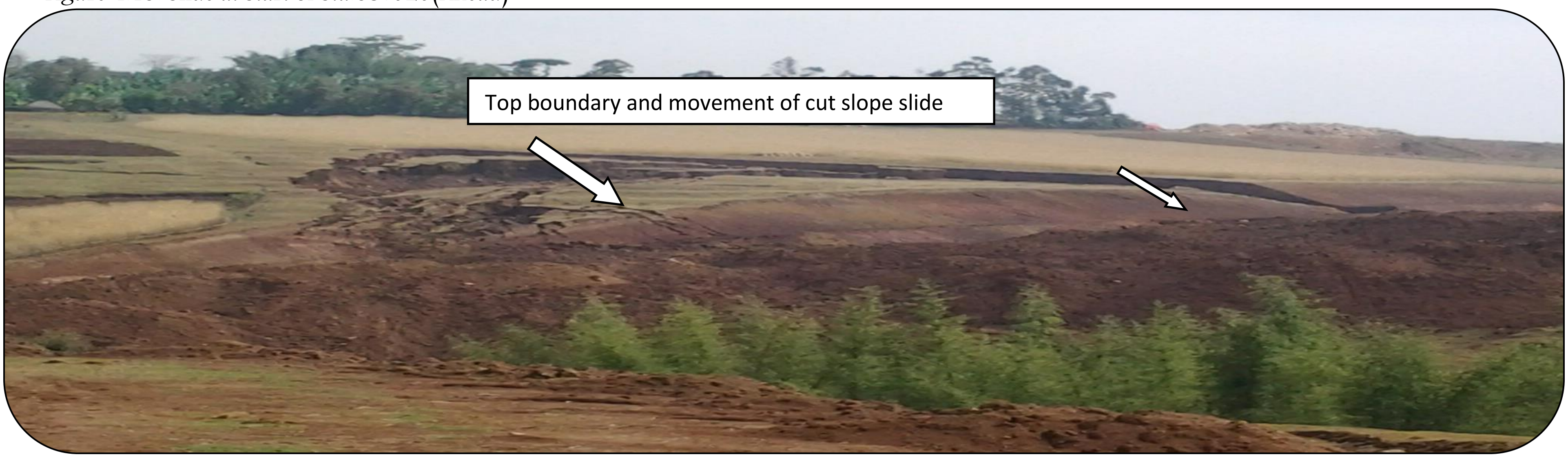


Figure 4.20: Slide manifestation from 68+020(Ahead) to 68+140(Back) RHS

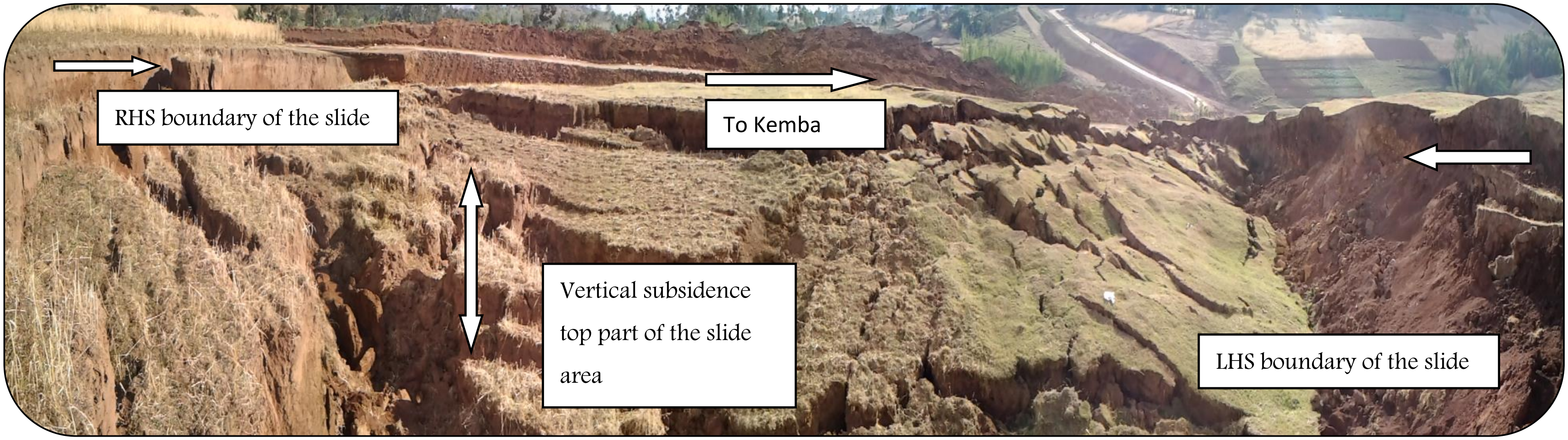


Figure 4.21: Top view of the slide

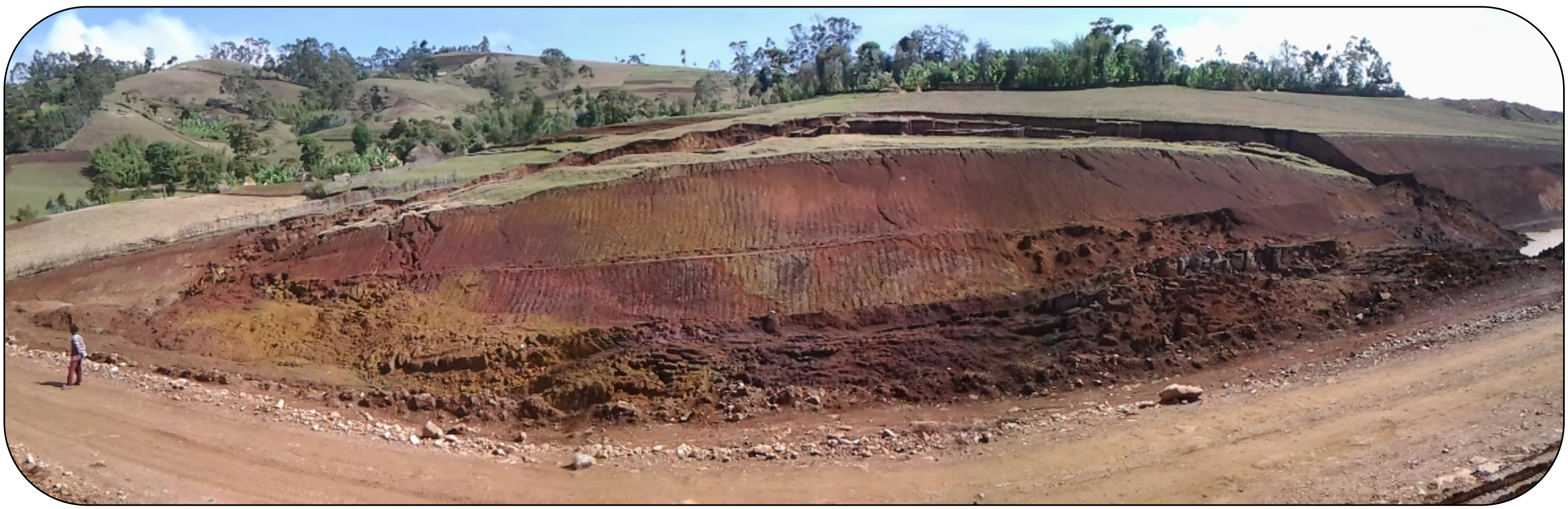


Figure 4.22: Front view of the slide and block movement of the slide mass

4.2.2 Sampling, Testing and cut slope failure(Slide) characterization:

Undisturbed and disturbed soil samples from the slide plane and slide material respectively are sampled and tested for classification, strength determination and hence for the characterization of the slide mass. From the tests, one can determine the strength parameters and index properties for the slide plane and index properties for slide mass, which eventually is useful to characterize the slide mass and sliding plane. All the index and strength test reports which are tested in Diriba Defersha GC project site (contractor's) laboratory, Construction Design Share Company's and Gondwana Engineering's material testing departments are annexed under Annex A.

Table 4.6: Description of sample location and test results

Sample location	Tests and results	Slide material and slide plane characterization
68+080RHS: Offset=80m , Depth=1.75m (on the slide plane)	Atterberg Limits: • Liquid Limit,LL=58% • Plastic Limit, PL=38% • Plasticity Index, PI=20% • Cohesive Index, $CI = \frac{PI}{PL}$ =0.6 Unit Weight(bulk), γ_b:13.87KN/M^3 Hydrometer Analysis: • pass no. 200 sieve=86% • clay fraction=20%, • silt fraction=66%, • sand fraction=14% and, • gravel fraction=0%.	➢ According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200 A-Line =0.73(LL-20) Below LL >50 % Retained on sieve No.200: 15-30% sand=14% and gravel =0% Hence, silt with sand. ➢ According to the classification of Silts and clays based on Cohesive Index, CI: CI =(0.4-0.6) : clayey silt ➢ According to hydrometer analysis: **sandy silt soil with traces of clay**
68+080RHS: Offset= 0m, Depth=0.3m (accumulated material at the center of the road)	Atterberg Limits: • Liquid Limit,LL=65% • Plastic Limit, PL=40% • Plasticity Index, PI=25% • Cohesive Index, $CI = \frac{PI}{PL}$: 0.6 Unit Weight(bulk), γ_b:13.87KN/M^3 Hydrometer Analysis: • pass no. 200 sieve=65% • clay fraction=15%, • silt fraction=50%, • sand fraction=34% and, • gravel fraction= 1%.	➢ According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200 A-Line =0.73(LL-20), Below LL >50 % Retained on sieve No.200: >=30% sand=34% and gravel =1% Hence , sandy silt ➢ According to the classification of Silts and clays based on Cohesive Index, CI:[2] CI =(0.4-0.6) : clayey silt ➢ According to Hydrometer analysis: **sandy silt soil with clay fraction**

Sample Location	Tests and Results	Slide material and Slide plane Characterization
68+120 RHS, Offset=3.5m, Depth from the OGL=10.9m	Atterberg Limits: • Liquid Limit,LL=64% • Plastic Limit, PL=42% • Plasticity Index, PI=22% • Cohesive Index, $CI = \frac{PI}{PL}$: 0.5 Unit weight(bulk), γ_b:13.87KN/M^3	➢ According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200: A-Line =0.73(LL-20) Below LL >50 Clayey Silt soil ➢ According to the classification of silts and clays based on Cohesive Index, CI:[2] CI =(0.4-0.6) : **clayey silt**

Owing to the insitu formation of the soil all index tests are carried out as per the procedures set out for testing of residual soils [8].

According to the index test results based on hydrometer + sieve Analysis the followings were concluded:

- The material on the slide plane and displaced or failed material have the same material properties, i.e. sandy silty soil.
- The overall slope material is homogeneous, and the slide plane is approximated to be composite and;
- The same material extends below the toe of the slope as the slide plane material.

4.2.3 Analysis of Slide or cut slope instability

The failure of the cut slope was first noted during the progress of excavation within the slide area on the second bench cut slope with a wedge failure daylighting on the slope. The excavation of the road was finalized within the slide section to the sub grade level and the cut slope failure had propagated upslope (retrogression) to a stable position.

The instability of the cut slope primarily was the result of the steepness of the constructed cut slope angle i.e. 45^0 or a slope ratio of 1:1(horizontal to vertical) which was simply adopted from design provisions. [7]. The cut slope angles provided in the design provisions are recommended for preliminary design purposes, are indicative and require site specific assessment and design especially for depth of cuts in excess of 5m deep. To this end, mobilized shear strength between the slide material and slide plane was less than the amount required for stability of the excavated slope and the cut slope has failed.

Although it is impossible to identify all points of the slip surface on the entire length, three points can be identified on the slip surface:

- Toe of the slip surface i.e. the toe of cut slope

- The outer crest of the slide surface measured at the offset of 80.37m from the centerline of the road.
- Depth of failure surface: The depth of the failure surface below the crest of the slope is usually equal to the distance from the crest of the slope back to the furthest shear crack as depicted under figure 4.31 below and Figure 2.5 above.[1]

Back Analysis method coupled with the actual location of the slip surface revealed from site investigations shall be used to back calculate the strength parameters on the slip surface just at the onset of failure of the cut slope.

Accordingly, the steps and the assumptions in back analysis for this failed cut slope shall involve the following:

i. Assume a combination of strength parameters (c and ϕ) to result in a range of dimensionless parameter related to the depth of the slip surface, $\lambda_{c\phi}$, as given in equation (1-5) above and tabulated in Table 4.6 below.

Table 4.7: Assumed Strength Variables and Dimensionless Parameter

Assumed Strength Variables	**c(kPa)**	40	20	10	5	5	40
	ϕ(Deg.)	5	15	25	30	10	20
Dimensionless Parameter ($\lambda_{C\emptyset}$)		**25**	**5**	**2**	**1**	**2**	**7**

ii. Create the slope geometry in Geo studio 2004 SLOPE/W Software as shown on Figure 4.23 below using data from the designed slope and slide plane as depicted in figures 4.16 and 4.17 above ;

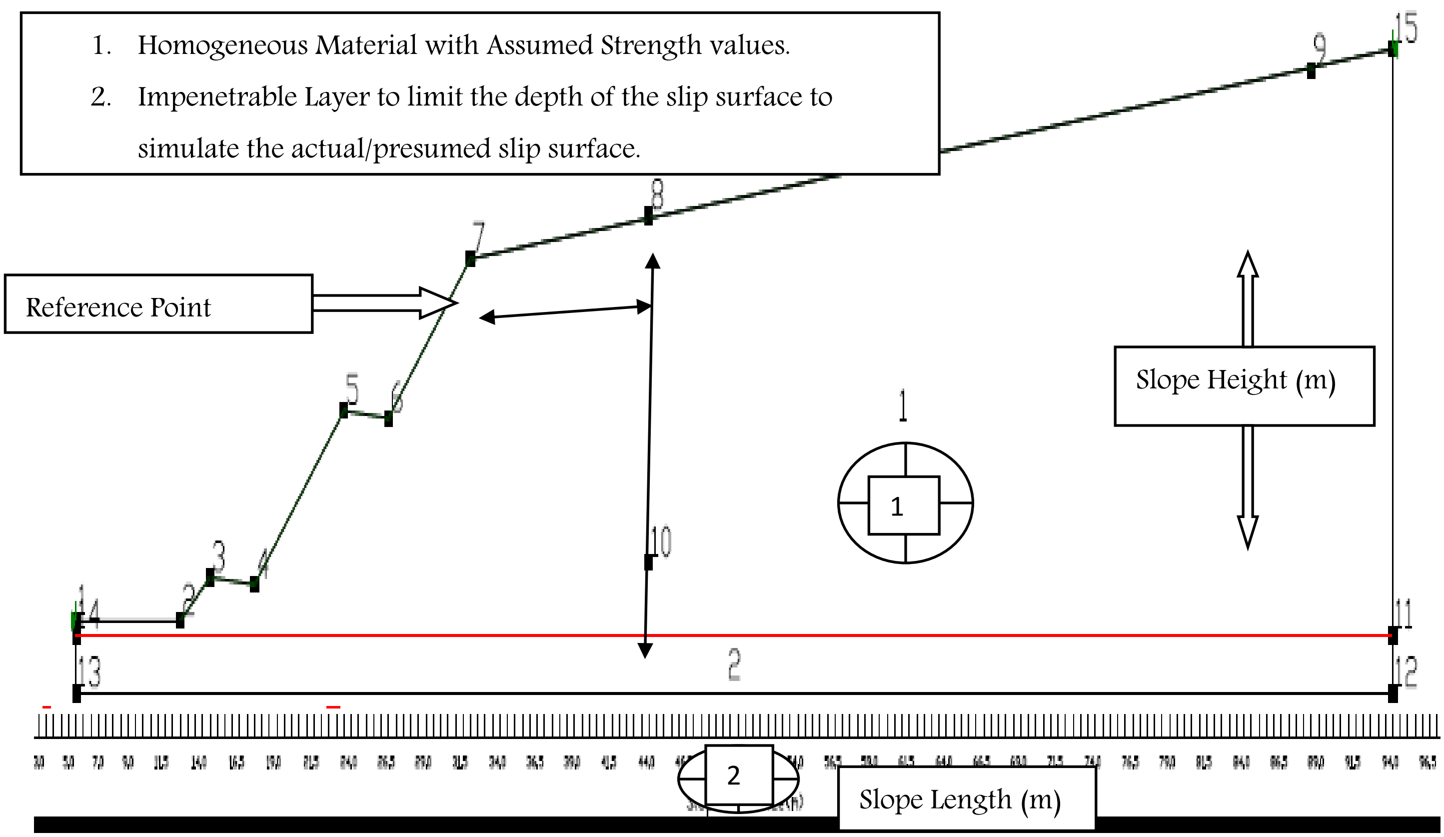


Figure 4.23: Slope Geometry in Geo Slope/W 2004

iii. The slope is characterized in Geo slope Slope/W as soil material underlain by bed rock or impenetrable layer to simulate the interface between the distinct slope materials and the depth or shape of the slip surface. Further, in the analysis of the slope the slip surface is construed to pass through the toe of the slope and crest of the slide plane by Entry and Exit Option, where reference is made to Figure 4.23 above;

iv. Using the combination of strength parameters (C and Ø) assumed in (i) calculate the corresponding factor of safety and the depth of the slip surface referenced from reference point 7 as shown on figures from C.9 to C.14 which are annexed in Appendix C. In this analysis output, the reference point 7 is the location of the scarp while the reference point 10 is the actual location of exposed part of the rapture surface determined from field observation. In this regard, the depth of the slip surface is measured directly from reference point 7 and the location of the slip surface below reference point 7 by exporting the results of the analysis to AUTOCAD.

v. Calculate the developed shear strength parameters corresponding to FS=1, from (iv) above, using equations 1-6 and 1-7 as tabulated hereunder Table 4.8;

Table 4.8: Summary of analysis results for the assumed strength values

S/N	Dimensionless Parameters	Assumed Strength Variables		Factor of Safety(M-P)	Developed Strength Viables for FS=1, Entry and Exit		Depth of the Critical Surface at head of the Slope(Pt 7)(m)
	$\lambda_{c\emptyset} = \frac{\gamma H \tan \emptyset}{c}$	c	ϕ	Critical Surface Entry and Exit	c_d	ϕ_d	Critical Surface Entry and Exit
1	25	40	5	2.115	18.91	2.37	13
2	5	20	15	3.284	6.09	4.65	8.79
3	2	10	25	2.604	3.84	10.04	8.79
4	1	5	30	2.723	1.84	11.78	8.79
5	2	5	10	1.068	4.68	9.37	8.79
6	7	40	20	3.616	11.06	5.71	13
			Slope Height=12.5m		Bulk Unit Weight=13.87kN/m^3		

vi. Next, prepare a plot of the calculated shear strength parameters (c and ϕ) corresponding to a Factor of Safety, FS=1 and the depth of the slip surface measured in (v) as depicted under Figures 4.23 and 4.24;

vii. Finally, calculate from the plot prepared in (vii) above the corresponding back analysis strength values (c and ϕ) corresponding to the actual or assumed depth of slip surface i.e. 12.78m directly measured between reference point 7 and assumed point 10 as indicated on figures C.9 to C14.

viii. Finally, from the use of back analysis, the results of the developed internal friction angle and cohesion are 4.8^0 and 10.8KPa respectively.

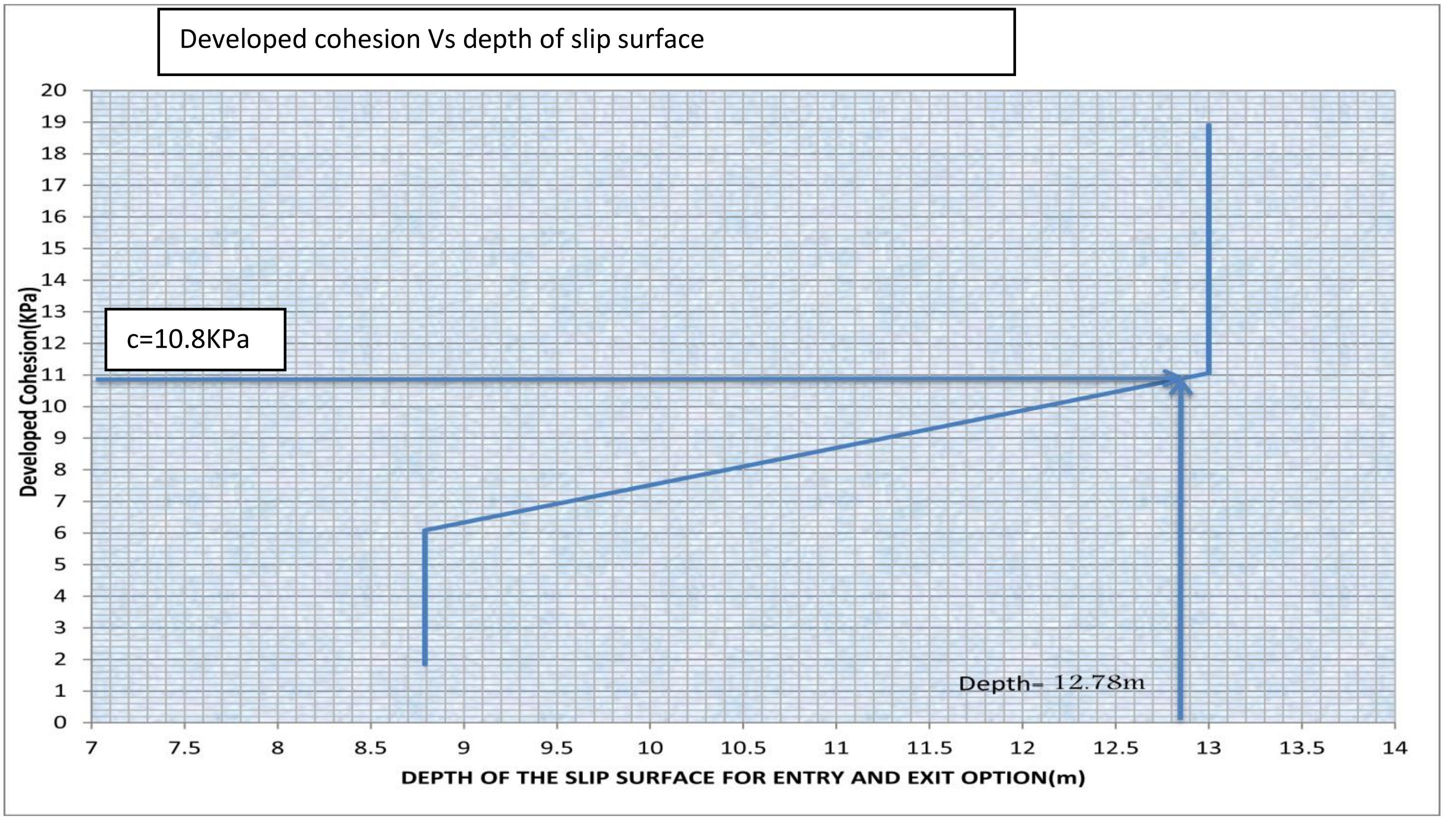


Figure 4.24: Calculated (developed) cohesion for a FS of 1 and depth of slip surface

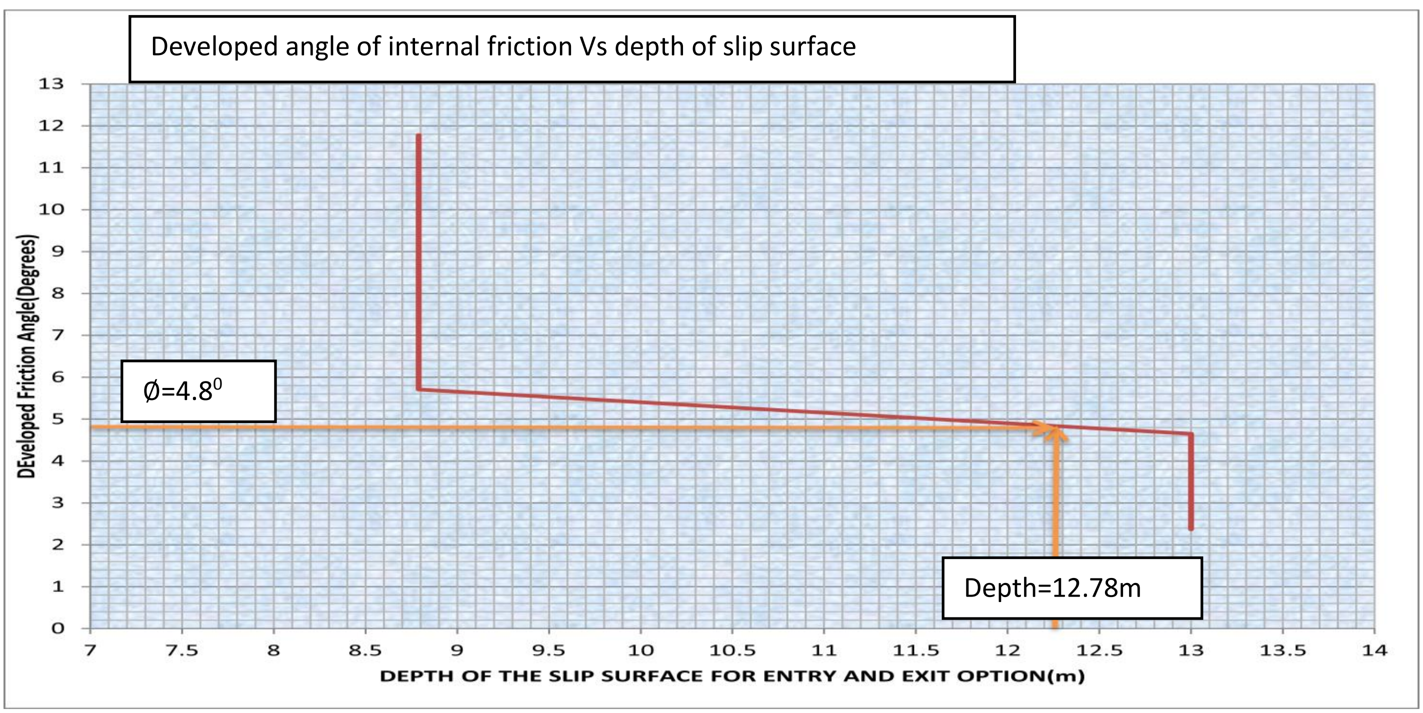


Figure 4.25: Calculated (developed) angle of internal friction for a FS of 1 and depth of slip surface

4.3 Sta.72+740 to Sta. 72+790(LHS)

4.3.1 General Features of the Cut Slope failure(Slide)

This Land slide (cut triggered slope failure) has occurred after the excavation within the section was finalized to the depth of sub grade level in box cut section. A rotational type of slide through the base toe of the excavated slope with a downward (from top of the slope) and outward (from the slip surface) movement of the soil mass peculiar to rotational type of slide was observed.

The total plan area, surface area and perimeter covered by the sliding mass are 2264.65m^2, 2730.60m^2 and 186.16m respectively. According to the classification of landslides, this slide is categorized as medium landslide (Slope Failure). [2] During, field investigation no seepage or ground water flow was observed on the slope or failed mass.

The geometrical parameters of the designed slope as per Figure 4.26:

- Slope height RHS=17.96m (Three benches at 6m height with 1:1 slope + stepped with 3 berms provided for 3m length and 10% dipping angle in to the slope + 1mini bench at the toe of the slope)
- The depth of excavation at center of the road is 12.02m while LHS and RHS excavation limits (offsets) from the centerline of the road are 33.94m and 20.49m respectively.
- The designed volume of common excavation to spoil work within the limits of this slide was 27, 894cu.m of earth.

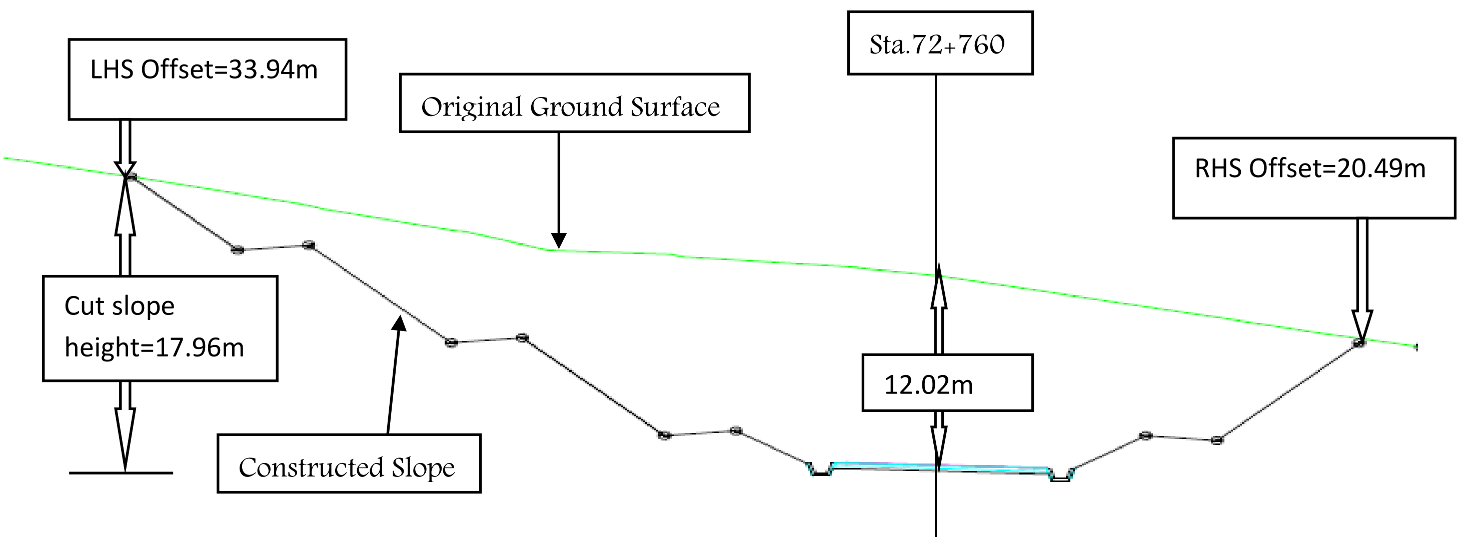


Figure 4.26: Designed typical section at Sta.72+760

Slide plane and slide material:

- The locations of the crest and toe of the failed soil mass are (-) 34.85m and 10.95m, respectively from the Centerline of the road.

- The crest height when measured from the design sub grade Level is 18.16m with dipping ground profile in to the right hand side as shown in Figure 4.27.
- The head scarp height (exposed part of the slide plane) varies from 5.5m to 8.0m over the slide boundary when moved to the center of the slide mass. Due to the vertical inclination of the head scrap (>64degrees from the horizontal) the slide mass material has moved far from the toe of the slide and occupied the full width of the finished road. The slide has started at the top of the slide crest and advanced down the slope.

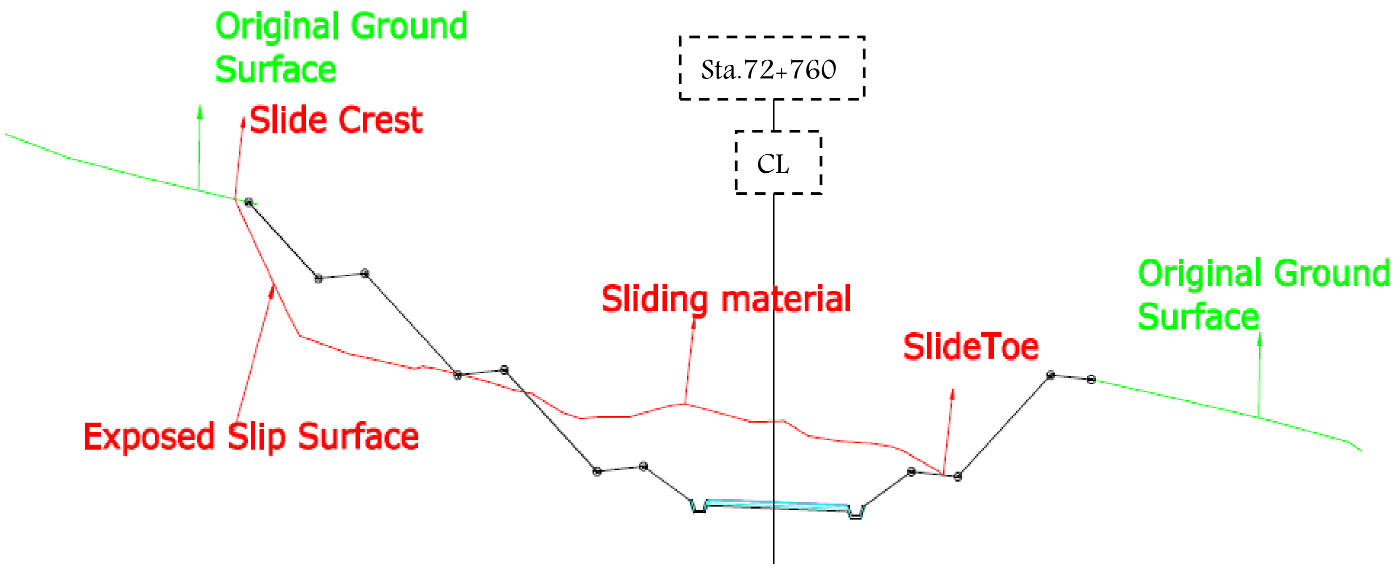


Figure 4.27: Typical slide cross section at Sta.72+760

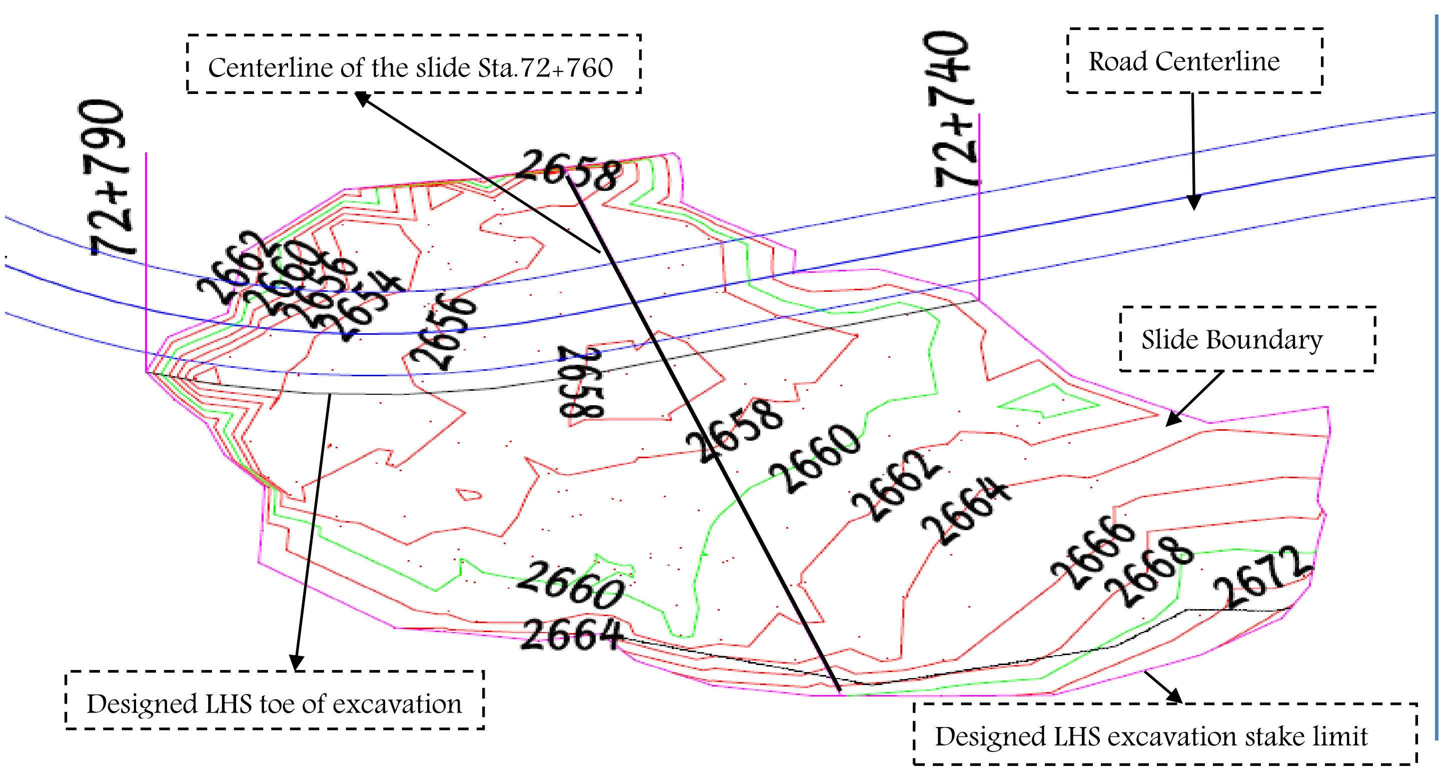


Figure 4.28: Plan drawing of the slide mass with contours

The dimensional properties of the slide from the typical slide cross section and plan drawings as per Figures 4.27 and 4.28 respectively, are tabulated hereunder:

Table 4.9: Dimensional variables of the failed cut slope from Sta.72+740 to 72+790(LHS)

Dimensional Variable	Measured Value(m)	Remark
Length of displaced mass, L_d: the minimum distance from tip to the top.	Tip=10.95m and Top= (-) 30.66m from Center line of the Road. Hence, **L_d= 41.61m**	The horizontal distance from the x-sectional drawing is taken, Figure 4.27.
Total length, L: The minimum distance from the tip of the land slide to its crown.	Tip=10.95m and Crown = (-) 34.85m from Center line of the Road. Hence, **L=45.80m**	Crown: practically un displaced material adjacent to the highest parts of main scrap.
Length of rupture surface, L_r: The minimum distance from the toe of the surface of rupture to the crown.	Toe=10.95m and Crown=(-) 34.85m from Center line of the Road. **L=45.80m**	The toe of the slide plane is used for reference.
Width of displaced mass, W_d : the maximum breadth of the displaced mass perpendicular to the length L_d	**W_d=47.28m**	Reference is made to the plan drawing of the slide, Figure 4.28.
Width of rupture surface, W_r: The maximum width between the flanks of the landslide, perpendicular to the length, L_r.	**W_d=47.28m**	Reference is made to the plan drawing of the slide, Figure 4.28.

Figure 4.29: LHS view of cut slope failure (slide)

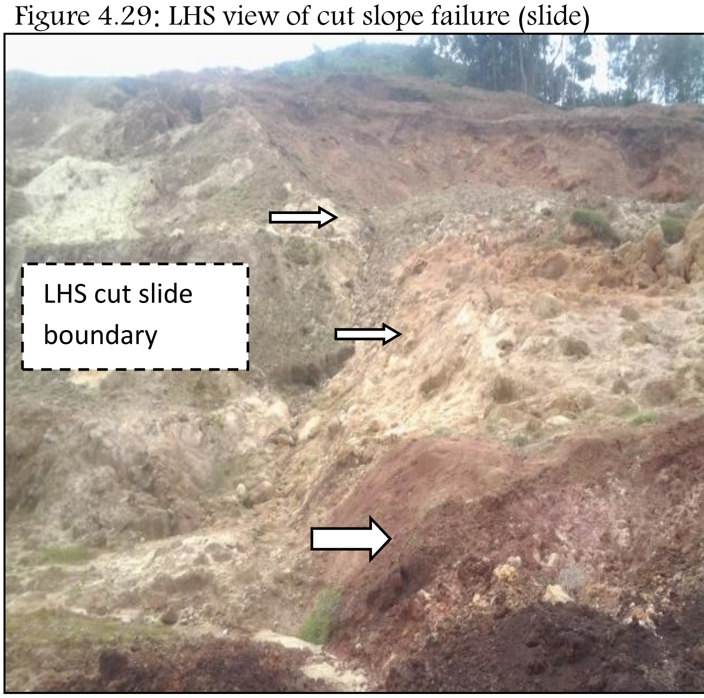
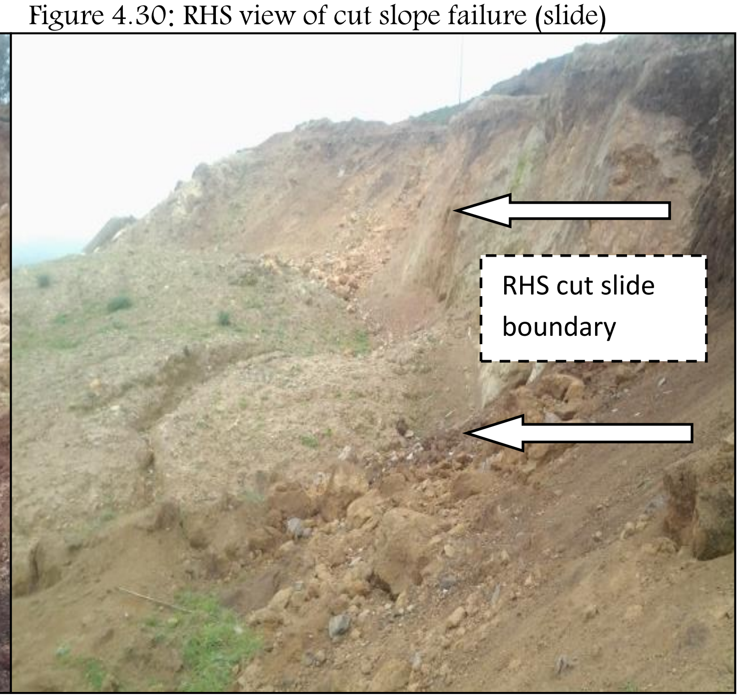


Figure 4.30: RHS view of cut slope failure (slide)



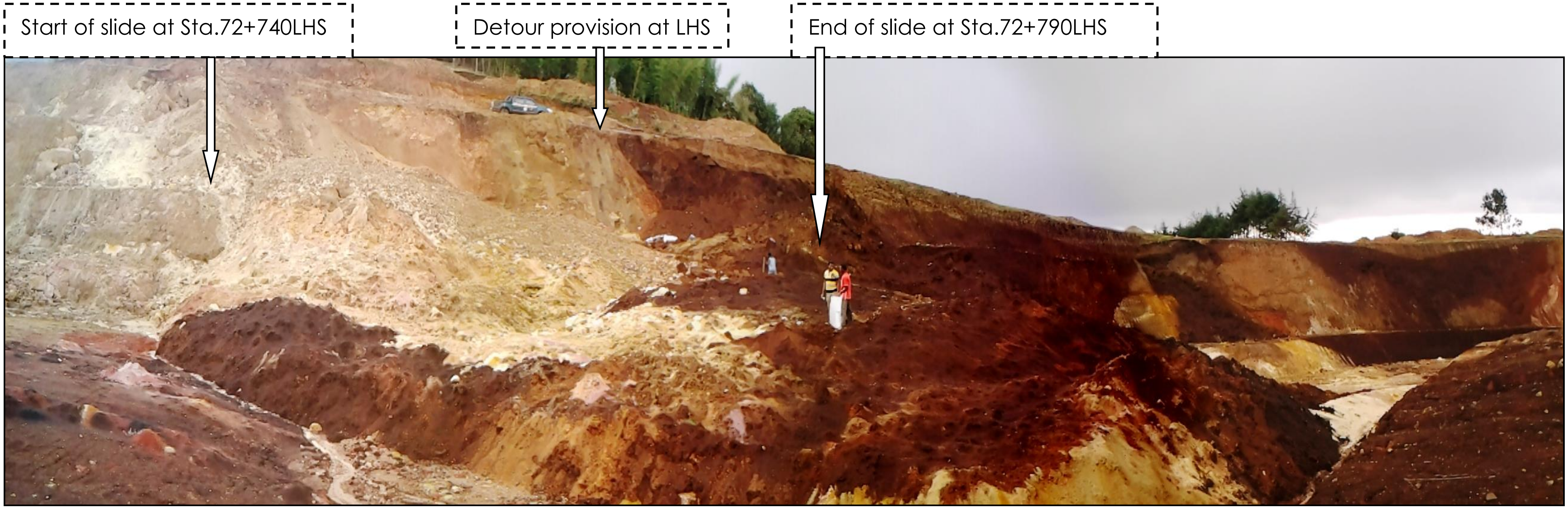


Figure 4.31: Front side of the slide from sta.72+740 to sta.72+790LHS

Figure 4.32: Displaced material at foot of the cut slope slide from Sta. 72+740 to Sta.72+790 LHS: front view

4.3.2 Sampling, Testing and cut slope failure(Slide) characterization:

Undisturbed and disturbed Soil samples from the slide plane and slide material respectively are sampled and tested for classification, strength determination and hence for the characterization of the slide mass. From the tests, one can determine the strength parameters and index properties for the slide plane and index properties for slide mass, which eventually is useful to characterize the slide mass and sliding plane. All the index and strength test reports which are tested in Diriba Defersha GC project site (contractor's) laboratory, Construction Design Share Company's and Gondwana Engineering's material testing departments are annexed under Annex A.

Table 4.10: Description of sample location and test results

Sample Location	Tests and Results	Slide material and slide plane Characterization
72+760LHS : Offset=29.46m , Depth=6.50m (on the slide plane)	Atterberg Limits: • Liquid Limit,LL:62% • Plastic Limit, PL:38% • Plasticity Index, PI:24% • Cohesive Index, $CI = \frac{PI}{PL}$: 0.6 Hydrometer Analysis: • Pass no. 200 sieve:69% • Clay Fraction:12%, • Silt Fraction:57%, • Sand Fraction:30% and, • Gravel Fraction: 1%.	➢ According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200 A-Line =0.73(LL-20) Below LL >50 % Retained on sieve No.200: >30% Sand=30% and Gravel =1% Hence, Silt with Sand. ➢ According to hydrometer analysis: **Sandy Silt soil with clay fractions** According to classification of Silts and clays based on Cohesive Index, CI:[2] CI =(0.4-0.6) : Clayey Silt
72+760HS: Offset=0m, Depth=0.3m (accumulated material at the center of the road)	Atterberg Limits: • Liquid Limit,LL:64% • Plastic Limit, PL:37% • Plasticity Index, PI:27% • Cohesive Index, $CI = \frac{PI}{PL}$: 0.7 Hydrometer Analysis: • Pass no. 200 sieve:47% • Clay Fraction:4%, • Silt Fraction:30%, • Sand Fraction:53% and, • Gravel Fraction: 0%.	➢ According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200 A-Line =0.73(LL-20) Below LL >50 % Retained on sieve No.200:>30% Sand=53% and Gravel =0% Hence , Silt with Sand ➢ According to Hydrometer analysis: **Silty Sand soil with few clay fraction** ➢ According to classification of Silts and clays based on Cohesive Index, CI:[2] CI =(0.6-0.8) : Very Silty Clay

Sample Location	Tests and Results	Slide material and Slide plane Characterization
72+790 LHS, Offset=3.0m, Depth from the OGL=12.15m	Atterberg Limits: Liquid Limit,LL:53% Plastic Limit, PL:35% Plasticity Index, PI:18% Cohesive Index, $CI = \frac{PI}{PL}$: 0.5	According to the classification of Fine grained soils, ASTM, where 50% or more by weight passing sieve No.200: A-Line =0.73(LL-20) Below A line LL >50 Clayey Silt soil According to classification of Silts and clays based on Cohesive Index, CI:[2] CI =(0.6-0.8) : Clayey Silt

Owing to the insitu formation of the soil, i.e. residual, all index tests are carried out as per the procedures set out for testing of residual soils [8].

From the index test results tabulated in Table 4.10 above (sieve +hydrometer) it is observed that:

- The slide plane and displaced or failed material have distinct material properties, i.e. Sandy Silt and Silty Sand soil respectively;
- The overall slope material is heterogeneous, and the slide plane is approximated to be composite failure surface and;
- The same material extends below the toe of the slope as the slide plane material;

4.3.3 Analysis of Slide or Cut Slope Instability

The cause of this slide was the result of:

- the removal of the toe support
- the mobilized shear strength between the slide material and slide plane is less than the amount required for stability of the excavated slope when coupled with increase of moisture from rainfall during the wet seasons.
- the slide mass material and the material on the slide plane are different and hence distinct geological bedding was present. This relict bedding could also be a source of weak soil interface leading to the failure of the cut slope.
- The provided slope angle 45^0 or ratio 1:1(H: V) for construction of the slope was steep and resulted in a factor of safety of less than 1.

In order to approximately identify the slip surface, the following points and line segment of the slide plane are used as depicted in Figure 4.33 below:

- Toe of the slip surface i.e. the sub grade level
- The outer crown of the slide surface at the offset of 34.85m from the centerline of the designed road

- The top exposed part of the slide plane i.e. from the crown to the top of the displaced soil mass i.e. the depth from Pt.No. 12 to Pt.No. 13 as referred from figure 4.34 below.

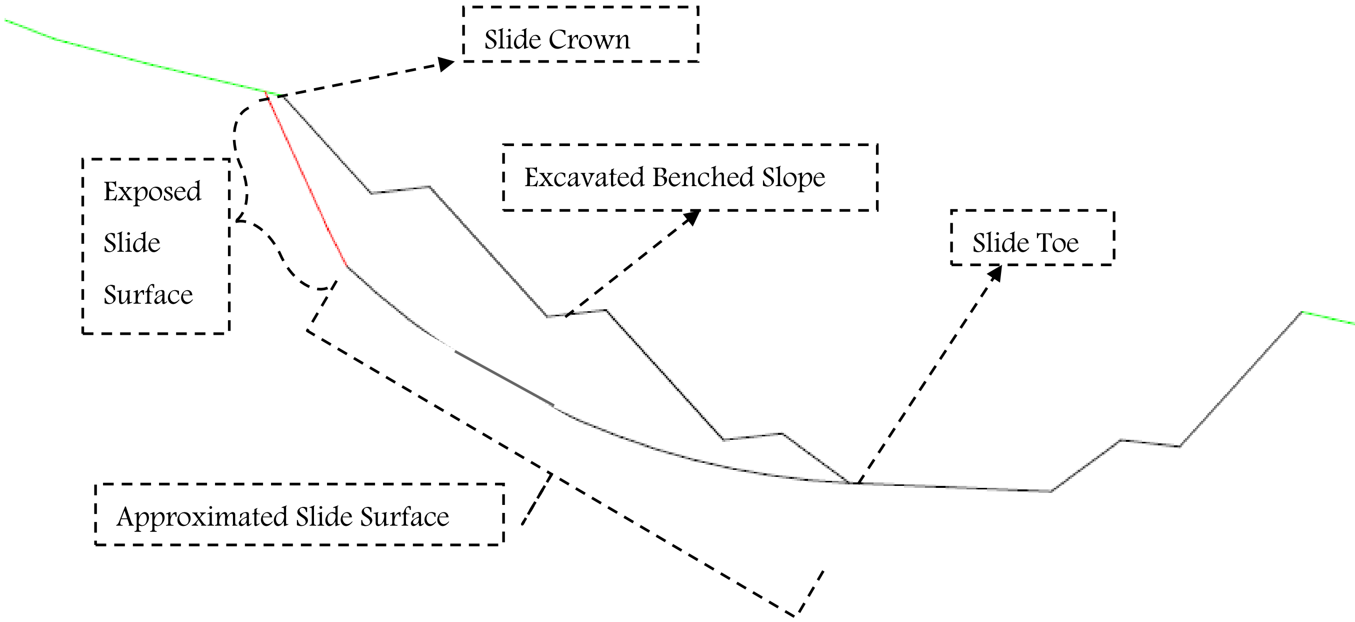


Figure 4.33: Approximated slip surface location

Next, back analysis method coupled with the actual location of the slip surface revealed from site investigations shall be used to back calculate the strength parameters on the slip surface just at the onset of failure of the cut slope.

The steps and the assumptions in back analysis for this failed cut slope would share the same steps indicated in the aforementioned slide locations and hence a summary of the results and calculations are displayed hereunder.

i. Assume a combination of strength parameters (C and ∅) to result in a range of dimensionless parameter related to the depth of the slip surface, $\lambda_{c\phi}$, as given in equation 1-5 and tabulated under Table 4.11 below

Table 4.11: Assumed shear strength values and dimensionless parameters

Assumed Strength Variables	c	40	40	20	5	10	10	5	5
	ϕ	5	20	15	10	25	30	30	40
Dimensionless parameters,	$\lambda_{c\phi}$	1.00	3.00	4.00	9.00	12.00	15.00	29.00	42.00

ii. Create the slope geometry in Geo studio 2004 SLOPE/W Software as shown on Figure 4.34;

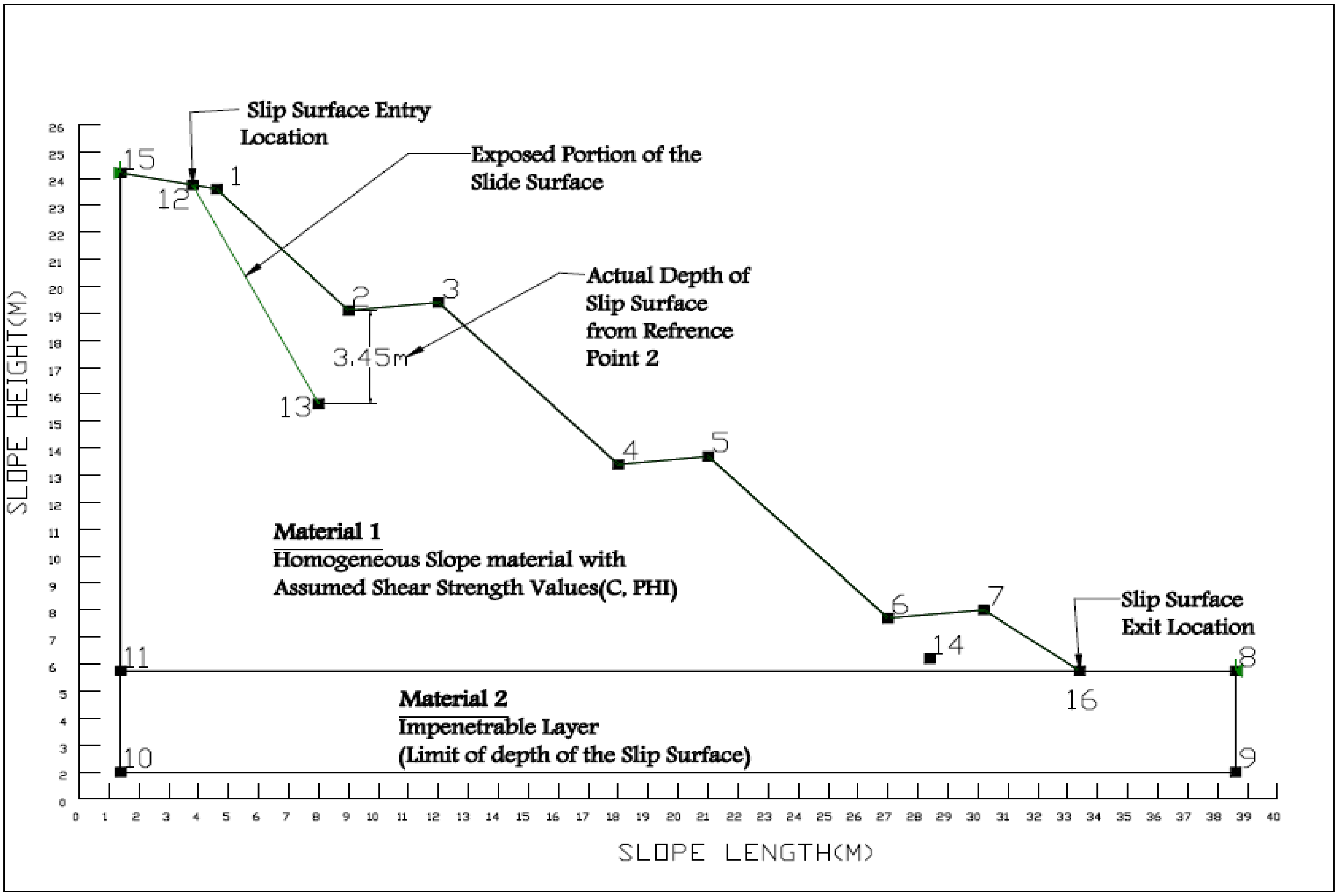


Figure 4.34: Slope geometry for analysis in GEO SLOPE/W 2004

iii. Using the combination of strength parameters (c and ∅) assume in (i) calculate the corresponding factor of safety and the depth of the slip surface referenced from point 2 as shown on figures from C.15 to C.23 which are annexed in Appendix C. In this analysis output, the reference point 2 is the first bench of the slope while the reference point 13 is the actual location of exposed part of the rapture surface determined from field observation. In this regard, the depth of the slip surface is measured directly between reference point 2 and the location of the slip surface below reference point 2 by exporting the results of the analysis to AUTOCAD.

iv. Next, prepare a plot of the calculated strength parameters (c and ∅) which are tabulated under Table 4.12 below corresponding to a Factor of Safety, FS=1 and the depth of the slip surface measured in (iii).

v. Finally, calculate from the plot prepared in (iv) above the corresponding back analysis strength values (C and ∅) corresponding to the actual depth of slip surface i.e. 3.45m measured directly between reference from point 2 and observed point 13 as indicated under figures from C.15 to C.23 under appendix C.

vi. Accordingly, from the use of back analysis the results of the developed internal friction angle and Cohesion are 15.75^0 and 9.25kPa respectively.

Table 4.12: Summary of analysis results for the back calculated shear strength parameters

S/N	Dimensionless Parameters $\lambda_{c\phi} = \frac{\gamma H \tan\phi}{c}$	Assumed Strength Variables c(kPa)	Assumed Strength Variables ϕ(deg)	Factor of Safety(M-P) Critical surface entry and exit	Developed Strength Viables for FS=1, Entry and Exit Option c_d	Developed Strength Viables for FS=1, Entry and Exit Option ϕ_d	Depth of the Critical Surface at head of the Slope(Pt 2)(m) Critical surface entryt and exit
1	1.00	40	5	1.826	21.91	2.74	5.76
2	3.00	40	20	2.428	16.47	8.53	5.76
3	4.00	20	15	1.404	14.25	10.81	5.76
4	9.00	5	10	0.595	8.40	16.52	3.10
5	12.00	10	25	1.404	7.12	18.38	3.10
6	15.00	10	30	1.614	6.20	19.69	3.10
7	29.00	5	30	1.353	3.70	23.12	3.09
8	42.00	5	40	1.848	2.71	24.43	3.09
Slope Height=18.08m				Bulk Unit Weight =13.78kN/m3			

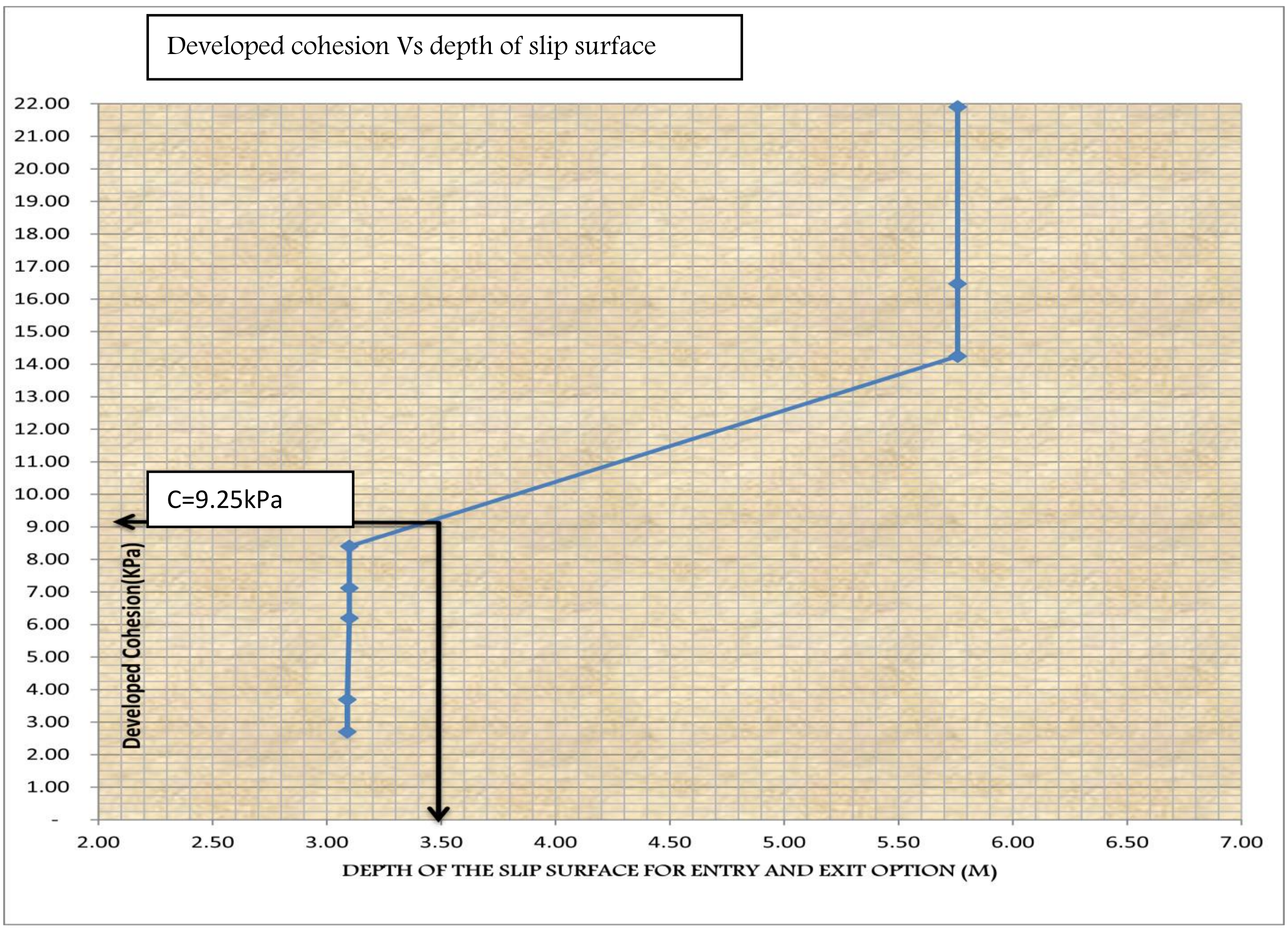


Figure 4.35: Back analysis values of developed angle of internal friction for a Factor of Safety of 1.0

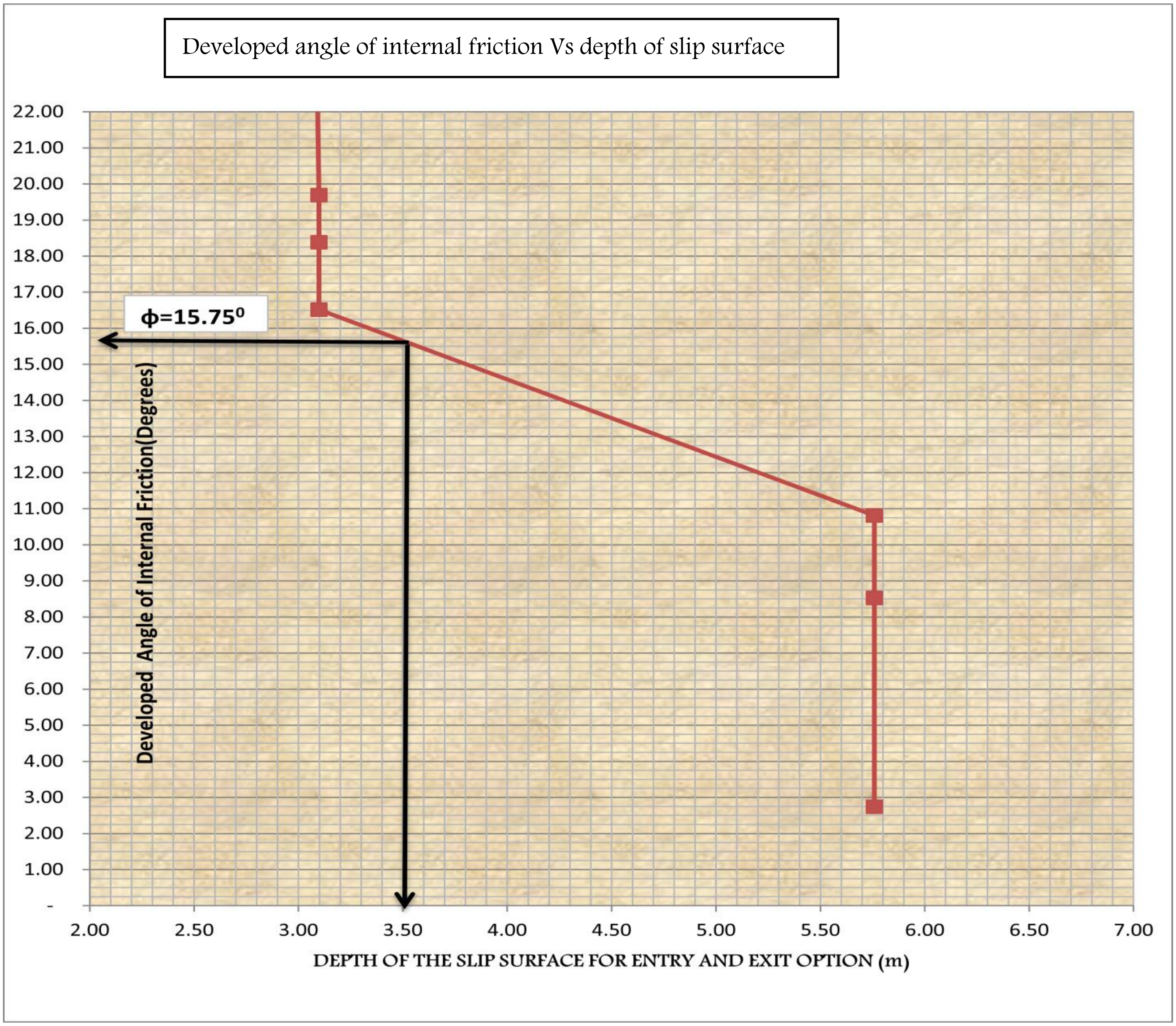


Figure 4.36: Back analysis values of developed angle of internal friction for a Factor of Safety of 1.0

5. Remedial Measures (Solutions) for Respective Land Slide (Cut Slope Instability) Locations

Once a landslide is identified on a natural, cut or fill slope, the remedial measures must provide a reduction in the driving forces, an increase in the resisting forces, or combination of both. A list of the most common methods that can be used to stabilize or remedy the effects of landslides can be stated in different standards and literatures. While one remedial measure may be sufficient to minimize the effect of a cut triggered landslide, most remedial works usually involve a combination of two or more methods. Generally, the selection of appropriate remedial measures depends on engineering feasibility, economic viability and environmental acceptability. [1]

To reiterate once more, there are two major countermeasures for landslide preventions.

i. Control works: are basically the methods to avoid or reduce the triggering factors i.e. up rise of groundwater level, artificial change on natural/cut slope angle and height.
ii. Restraint measures: stabilize the landslide by construction of structures to counteract the driving force of the landslide.

In this thesis study, the list of remedial measures proposed and designed for respective slide locations are outlined below. However, the specific proposed remedial measures at each cut slide locations depends on the investigation and analysis of the respective slides.

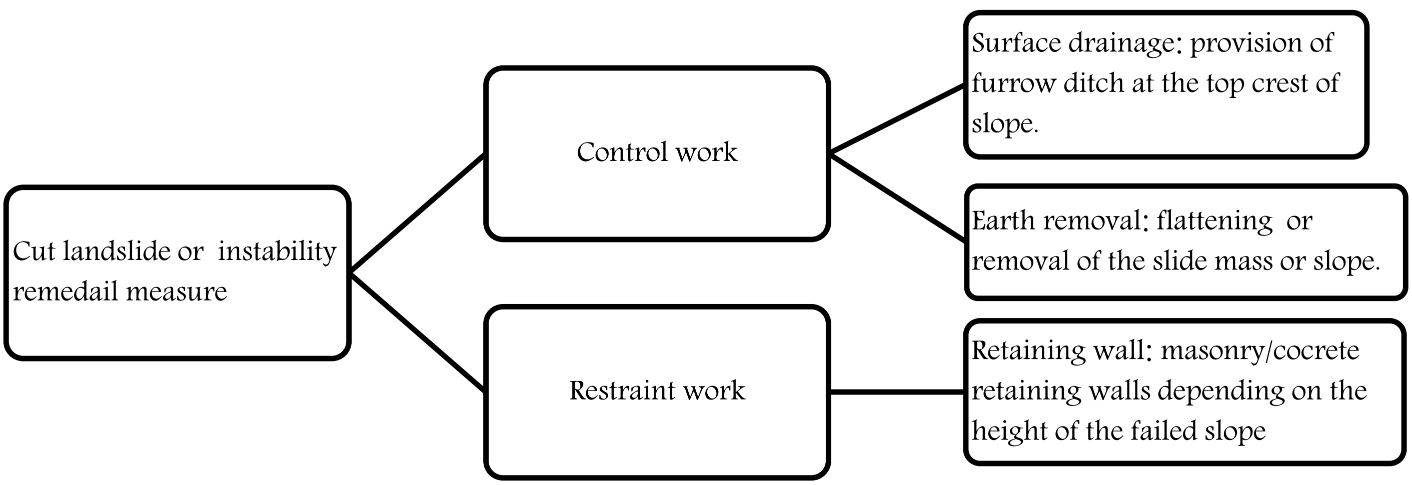


Table 4.13: Cut slope instability or landslide countermeasures

5.1 Cut Slope Instability(Land Slide) Location 1: Sta.66+995 to Sta.67+055 (RHS)

In Summary, at this slide location it was observed that:

- the depth of the slip surface at the toe is measured to be 1.4m below the designed sub grade level while a highly to moderately weathered rock material is found at a depth of 2.3m from the designed sub grade level.
- there were no ground water or seepage forces on the slope or on slide mass.
- from back analysis of the failed cut slope a factor of safety close to 1 (i.e. 0.962) was obtained using shear strength parameters of ϕ=4.25^0 and C=20kPa.

From the investigation of the cut slide or instability the reasons for failure of the cut slope was as a result of:

- ❖ the construction of steep cut slope angle of 45^0 or 1:1 slope ratio (H: V) for a total cut height of 13.64m resulting in a factor of safety of less than 1.
- ❖ the presence of extended period of wet (rain fall) season per year which makes cut slopes wet.
- ❖ the relict bedding planes which could be considered as weak planes in residual soil formations.

From results of the investigation the proposed remedial solutions for the remediation of the failed cut slope are the combinations of:

1. Control works:
   - ➢ Slope flattening and earth removal: since the slide material has occupied the road section during failure earth removal is necessary to open the road. Furthermore, slope flattening is performed starting from the top of the slide mass in order to reduce the surcharge load on the slip surface.
   - ➢ Surface drainage: provide furrow ditch at the top of the crown to collect surface drainage out of the slide area.
2. Restraint works: selected to be gravity retaining wall, stone mortared wall, due to the following reasons:

- The failure is due to the undermining of the toe of the slope coupled with increase of shear stresses due to rainfall and a rigid structure can accommodate such failures or flexible structure like gabions are not desired as any eminent (additional) movements of such wall will disturb other adjacent structures, like road side ditches, and road furniture.
- The free height of the retaining wall to be provided will be short as much failed soil mass shall be removed by flattening of the slope or reducing the overall slope angle.
- Construction material, mainly masonry rock, can be produced on site at reasonable distances and good workmanship can be achieved.

Accordingly, the remediation structure shall be constructed in base up method where the bottom of the slide shall be excavated after flattening of the slide material during dry weather conditions. Minimum of 1m clear excavation width from the heel of the wall with 1:1 slope shall be provided during excavation for safety precautions and placing of granular(sub base) or backfill(Rock fill) material.

Furthermore, excavation and construction shall be done for a length of 4-5m at a time (providing construction joint) in order to:

- minimize the disturbance due to the removal of toe support of the slide mass during excavation and construction of the wall;
- provide construction joints and;
- design the wall which negotiates with the horizontal curvature of the road.

Finally, provide weep holes diameter 110mm at 2.0m center to center horizontally and vertically to collect water from the backfill material which eventually discharge it to longitudinal drainage ditch.

The detailed design computations for the design of the remedial structure is annexed under Appendix B.

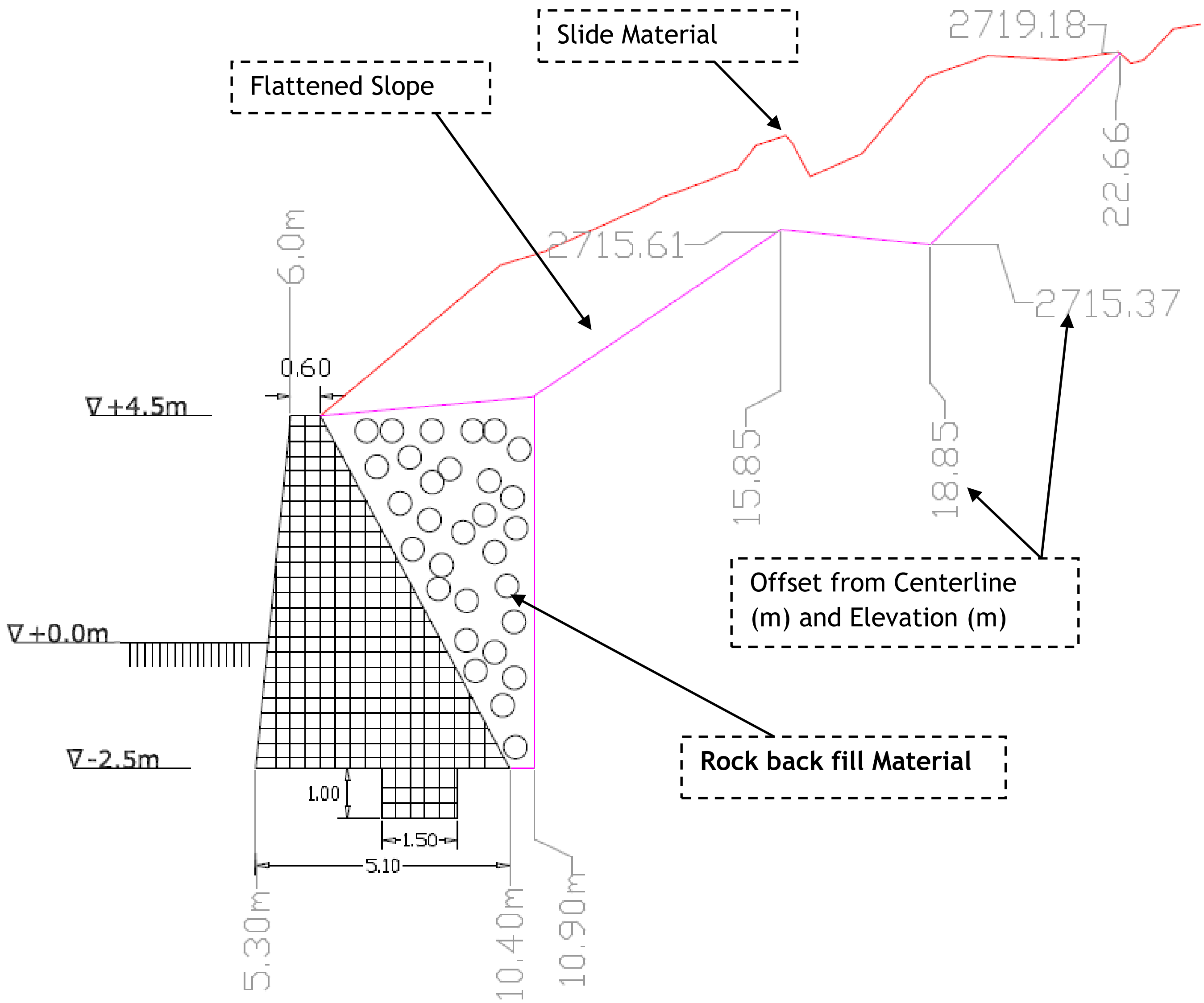


Figure 5.1: Typical Drawing of Masonry Retaining Wall with Shear Key at Sta.67+030RHS

5.2 Cut Slope Instability(Land Slide) Location 2: Sta.68+020(Ahead) to Sta.68+140(Ahead) (RHS)

Generally, at this slide location it was observed that:

- there were no ground water or seepage forces on the slope or on slide mass.
- from back analysis of the cut slope a factor of safety close to 1 was obtained using combination of shear strength parameters and observed depth of slip surface. The corresponding back calculated shear strength parameters (angle of internal friction and cohesion) were 4.8^0 and 10.8kPa, respectively.

The proposed remedial measures for the remediation of the cut slope failure are slope flattening and restraint wall. Accordingly, the restraint work selected to be reinforced concrete cantilever retaining wall due to the following reasons:

- The failure is due to the provision of steep slope angles, undermining of the toe of the slope coupled with increase of shear stresses due to rainfall. A rigid structure can be used to reinstate such failures while flexible structure like gabions are not desired as any (additional) movements of such wall will disturb other adjacent structures, like road side ditches, and road furniture.
- In order to keep the slide material in place after the removal of the slide material from the road cross section free height of the retaining structure of 7m should be provided. In view of the above, a cantilever retaining wall should be designed to accommodate the failed mass.

Accordingly, the remediation structure shall be constructed in base up method where the bottom of the slide shall be excavated after flattening of the slide material during dry weather conditions. Minimum of 1m clear excavation width from the heel of the wall with 1:1 slope shall be provided during excavation for safety precautions and placing of backfill (Rock fill) material.

Furthermore, excavation and construction shall be done for a length of 7-10m at a time (providing construction joint) in order to:

- ❖ minimize the disturbance due to the removal of toe support of the slide mass during excavation and construction of the wall,
- ❖ provide construction joints and design the wall which negotiates with the horizontal curvature of the road.

The proposed dimensions and the provisions of reinforcements are provided in Figure 5.2 and 5.3 below and one needs to provide weep holes of diameter 50mm at 1.5m C/C horizontal and vertical spacing so that percolated water shall be collected by longitudinal drainage ditch.

The detailed design computations for the design of the remedial structure is annexed under Appendix B.

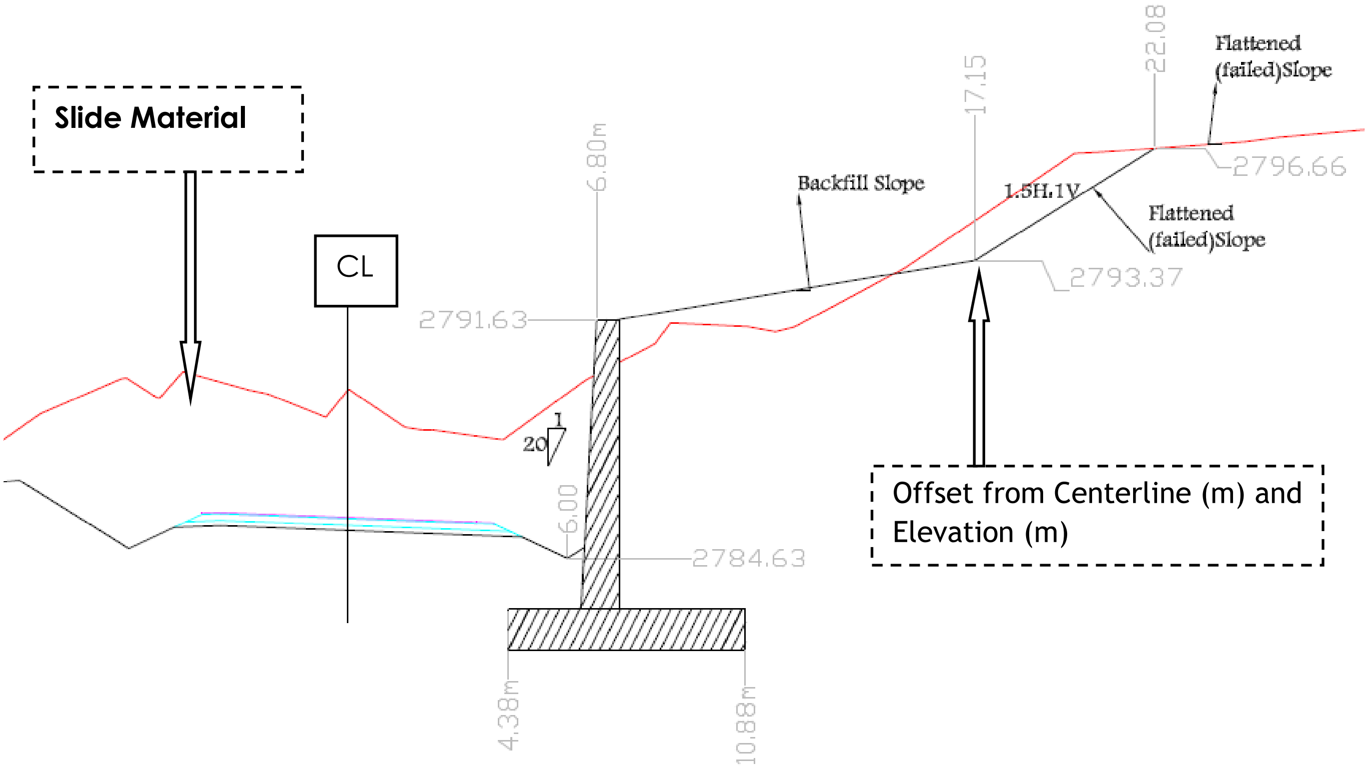


Figure 5.2: Typical Drawing of cantilever retaining wall at Sta.68+080RHS

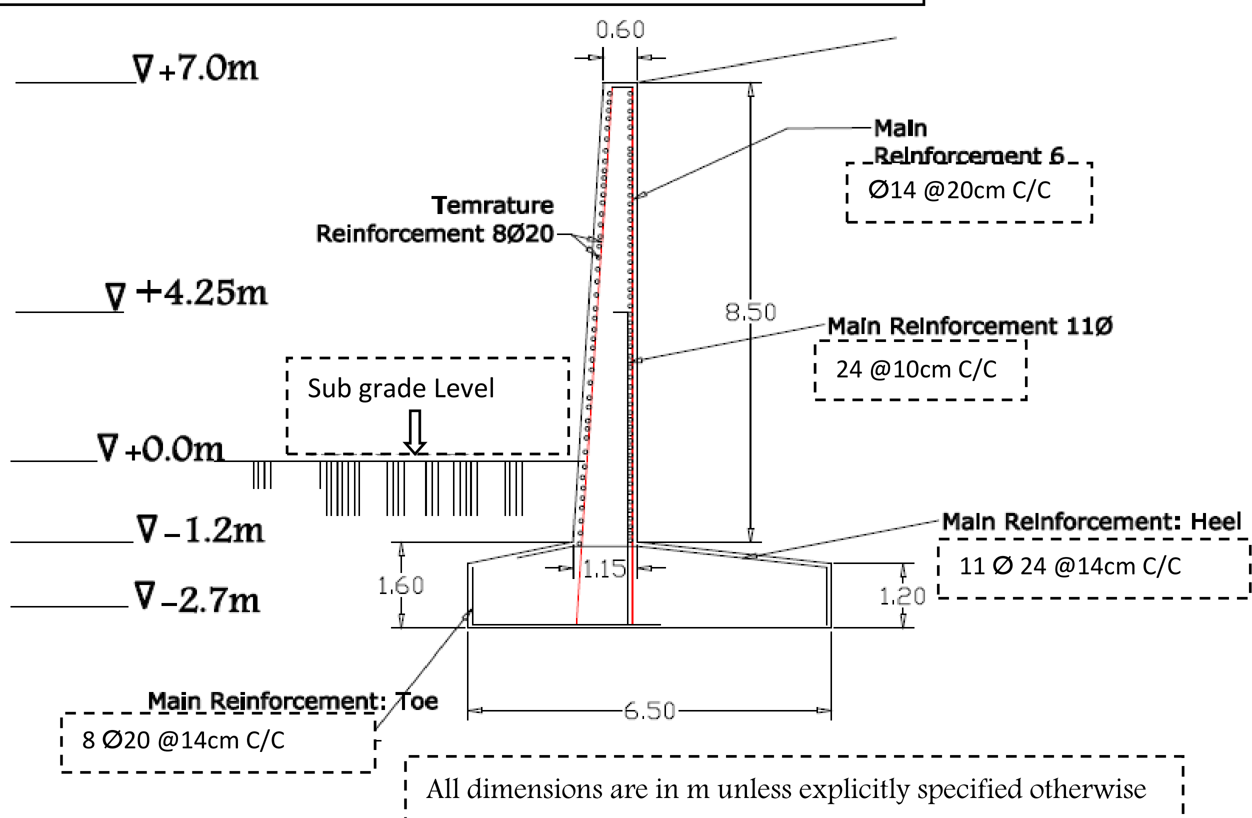


Figure 5.3: Typical structural drawing of cantilever retaining wall at Sta.68+080RHS

5.3 Cut Slope Instability(Land Slide) Location 3: Sta.72+740 to Sta.72+790 (LHS)

Generally, at this slide location it was observed that:

- there were no ground water or seepage forces on the slope or on slide mass.
- a factor of safety close to 1 was obtained from back analysis of the cut slope using combination of shear strength parameters and observed depth of slip surface. The corresponding back calculated shear strength parameters (angle of internal friction and cohesion) were 15.75^0 and 9.25kPa, respectively.

From the investigation of the cut slide it was observed that:

- the slide mass has moved and occupied the full width of the road
- the exposed portion of the slide plane at upper portion and LHS and RHS of the toe of the slide planes remain undisturbed and stable.

Therefore, the removal of the slide material from the road cross section will not affect the overall stability of the failed slope as the failed slope has defined a stable slope.

Accordingly, the remedial solution for this slide section shall consist of the following:

- The failure mass should be removed from the road way and also some removal and flattening of the slide material existing out of the roadway width shall also be made to a depth below the height of the failed mini bench.

- Here, the aim of remedial structure is to provide a berm support at the foot of the slide mass as the slide plane is stable and no cracks and retro progressive slides were noted after the slide and it was also witnessed that detour road runs on the top of the slide slope. Hence, provide masonry retaining wall of unsupported height equivalent to the height of the failed mini bench i.e. 2.5m as depicted below (Figure 5.4 and 5.5).

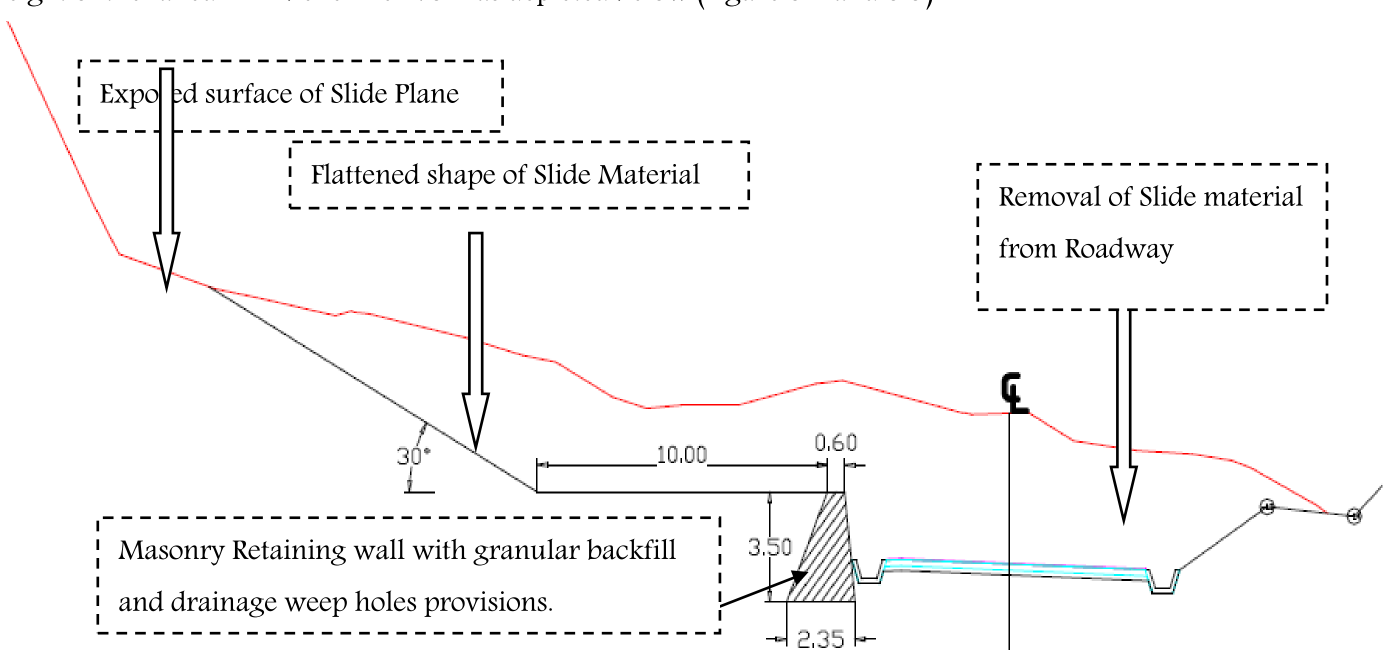
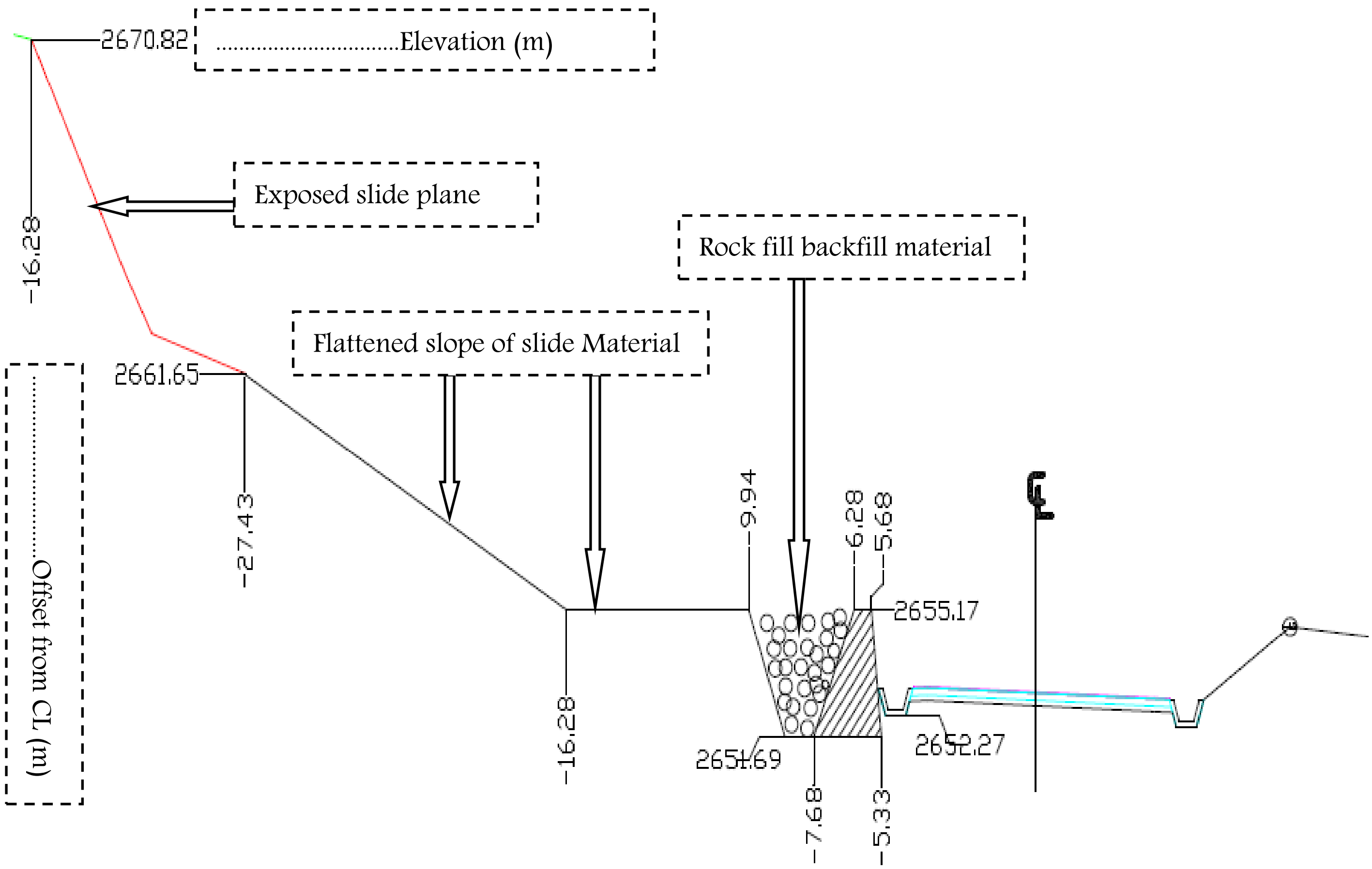


Figure 5.4: Provision of retaining wall Sta.72+760LHS



Note: All indicated dimensions are in meters unless explecitly stated otherwise.

Figure 5.5: Typical cross section of masonry retaining wall and flattened slope at Sta.72+760LHS

Here, provide weep holes diameter 110mm at 2.0m center to center horizontally and vertically to collect water from the backfill material which eventually discharges to longitudinal lined drainage ditch.

Chapter 6: Conclusions and Recommendations

6.1 Conclusion

The conclusions of this thesis report are:

1. The remedial solutions for respective cut slide locations depend on the type of instability and magnitude of remediation required. In this regard, the combinations of control and restraint remedial measures which were designed and proposed at each slide locations in order to restore the road to traffic were:
   - ❖ Control works of the nature removal of failed slope material from the road way, flattening of the failed cut slope and provision of furrow ditch at the outer boundary of the slide mass to convey the surface water flow were proposed at the three cut slide locations.
   - ❖ Restraint works include the provision of 4.5m free height + 2.5m keyed foundation masonry retaining wall and 9.7m cantilever reinforced concrete retaining wall at the toe of the flattened cut slide slopes of slide locations 1 and 2 respectively were provided. Furthermore, 3.5m height masonry retaining wall just to replace the failed berm slope at the toe of the third failed cut slope was proposed.
2. The provision of steep cut slope angles, the presence of relict residual bedding profiles and migration of moisture from prolonged duration and intense rainfall were the main reasons for the instability of the cut slopes under this investigation.
3. The back analysis shear strength parameters at each cut slide locations were obtained with the use of limit equilibrium 2D analysis. The results of back analysis shear strength values (cohesion and angle of internal friction) at cut slide locations 1, 2 and 3 are (15^0, 9.25kPa), (4.8^0, 10.8kPa) and (4.25^0, 20kPa) respectively.

6.2 Recommendations

The case studies made in this thesis project have focused on medium scale cut slope instability (landslides) in road project, however the method of investigations, cut slope instability characterizations, method of analysis and design of remedial structures and method of remediation of the instability can be taken as a template for similar cut slope instability related problems in road or other projects.

To this end, for further improvement in the investigation, analysis and remediation of cut slope instability (landslides) related problems the followings are recommended:

I. The shear strength parameters at each slide locations were obtained from the use of back analysis. However, in order to check the degree of accuracy of the shear strength values obtained from back analysis it is important to determine the residual shear strength parameters through laboratory testing by simulating the prevailing site conditions.

II. Two dimensional limit equilibrium analysis was used to calculate the factor of safety of the failed slopes at respective slide locations. Accordingly, the shear strength values obtained under this study were conservative. However, for design of remedial measures, especially restraints works, the shear strength values obtained from the use of 3D finite element analysis methods which consider the three dimensional end or side effects should be used.

REFERENCES

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Appendix –A

LABORATORY TEST RESULTS

Slide Location 1: Sta.66+995 to Sta.67+055(RHS)

Company Name	Form Nº
CONSTRUCTION DESIGN SHARE SCo.	OF/CDSCo./123

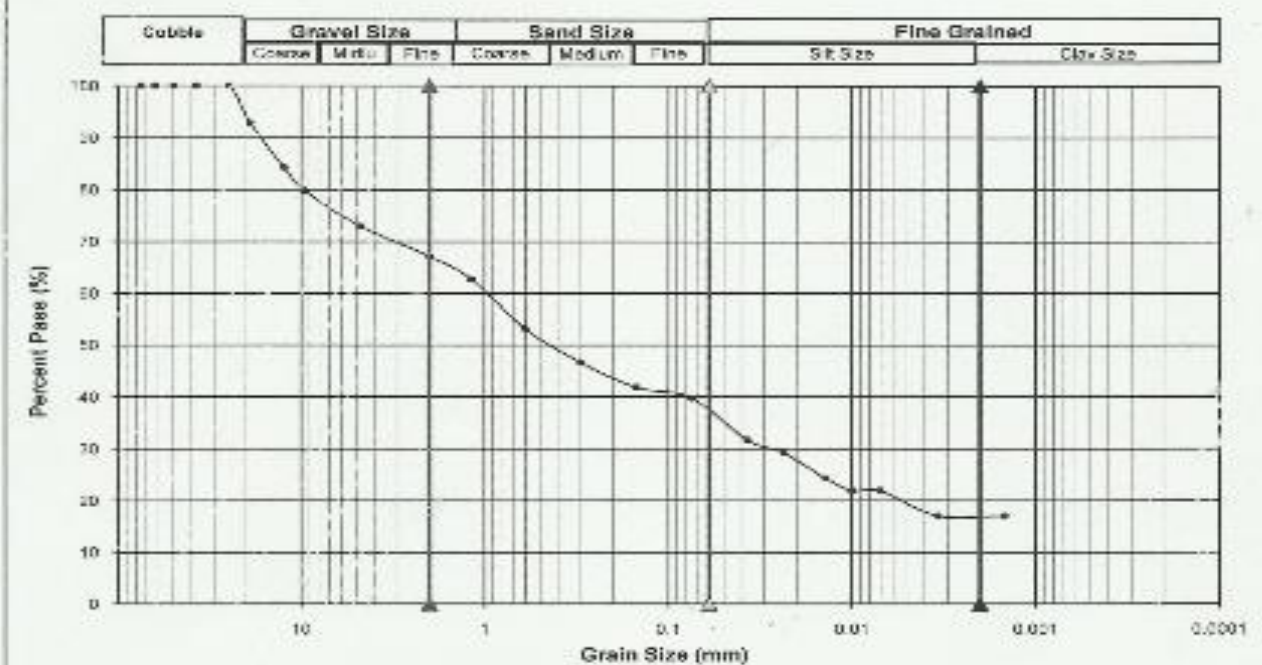
Title	Issue NO	Page Nº
Grain Size Distribution	1	Page 1 of 1

W.O/Proj. No	019841
Date	25/02/2014
Sample ID:	Gs-03/14/1969

Project :- Thesis / Belta-Otolo Road Project Lot-II /
Client :- Getinet Tilahun
Location:- Belta-Otolo
Description:- Brown Silty Gravel With Some Sand and Clay
Test method :- ASTM D 422

Station :- 67+030 RHS
Depth (m) :- 6.50
Sliding plane at outer surface

Specific gravity 2.23



Tested by: Kokobe Lemma
Date : 04/03/2014
Checked by: Abate Legesse
Date: 05/03/2014

Approved by :- Anley Abebe
Date :- 06/03/2014

Construction Design S.Co.
Central Laboratory

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

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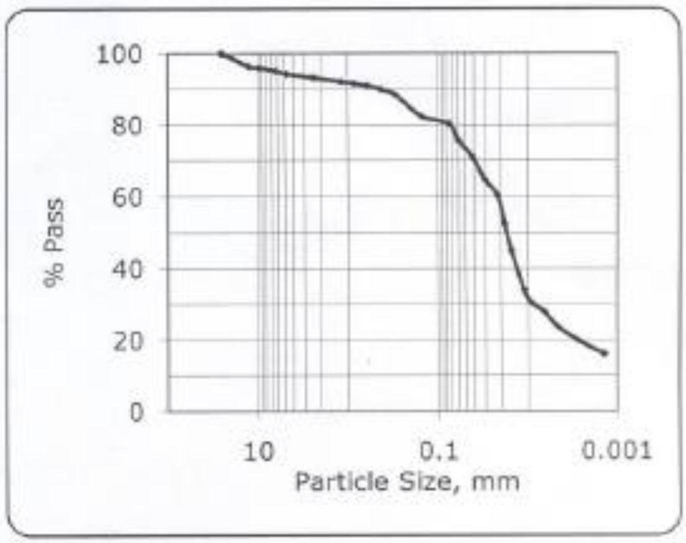
		Document No.: LAB-F-014	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 88

Project:- Belte - Otolo (Lot II)
Client:- Getinet Tilahun
Station:- 67 + 030, RHS
Sample of:- Slide slope Material
Description of Sample:- Pinkish brown Silty clay GRAVEL
Depth Sampled:- Top slid material 8.50m from OGL
Lab. Number:- BO. Hydrom. Slop. 4/14
Date Tested:- 11/08/14
Date Issued:- 19/08/14

ASTM SIEVE (MM)	CUMULATIVE % RETAINED	% PASS
25	0.0	100
19	1.3	98.7
12.5	3.6	96.4
9.5	4.0	96.0
6.3	4.9	95.1
4.75	5.7	94.3
2.36	6.8	93.2
1.18	7.9	92.1
0.850	8.4	91.6
0.600	9.1	90.9
0.425	10.1	89.9
0.300	11.6	88.4
0.150	17.6	82.4
0.075	19.7	80.3
0.05855	24.6	75.4
0.04185	29.0	71.0
0.03006	35.4	64.6
0.02180	39.7	60.3
0.01823	47.6	52.4
0.01514	55.2	44.8
0.01105	66.1	33.9
0.00922	69.5	30.5
0.00664	72.2	27.8
0.00478	76.2	23.8
0.00279	80.1	19.9
0.00140	84.0	16.0



Remark:-

Reported by:- Lab. Engineer

Approved by:- Mat. Engineer

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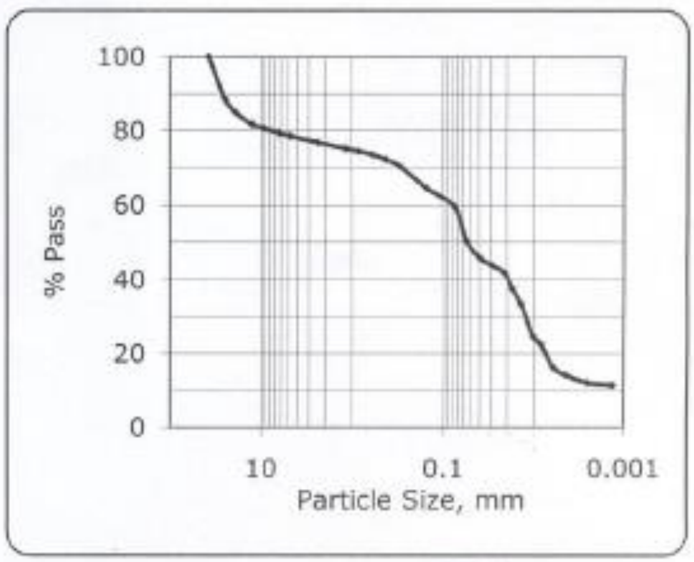
	Document No.: LAB-F-014	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 88

Project:-	Belte - Otolo (Lot II)
Client:-	Getinet Tilahun
Station:-	67 + 030, RHS
Sample of:-	Slide slope Material
Description of Sample:-	Brownis Silty clay with few GRAVEL
Depth Sampled:-	2.10m from Subgrade
Lab. Number:-	BO. Hydrom. Slop. 1/14
Date Tested:-	11/08/14
Date Issued:-	19/08/14

ASTM SIEVE (MM)	CUMULATIVE % RETAINED	% PASS
37.5	0.0	100
25	11.3	88.7
19	15.1	84.9
12.5	18.3	81.7
9.5	19.3	80.7
6.3	20.6	79.4
4.75	21.4	78.6
2.36	23.1	76.9
1.18	24.8	75.2
0.850	25.5	74.5
0.600	26.4	73.6
0.425	27.7	72.3
0.300	29.6	70.4
0.150	35.6	64.4
0.075	40.4	59,6
0.05590	49.7	50.3
0.04062	54.0	46.0
0.02918	56.1	43.9
0.02116	58.2	41.8
0.01753	62.5	37.5
0.01390	66.8	33.3
0.01031	75.3	24.7
0.00848	77.4	22.6
0.00619	83.8	16.2
0.00443	85.9	14.1
0.00259	88.1	11.9
0.00135	88.7	11.3



Remark:-

Reported by:- Lab. Engineer

Approved by:- Mat. Engineer

ARBAMINCH KEMBA SAWLA ROAD PROJECT
LOT II BELTA OTOLO ROAD PROJECT

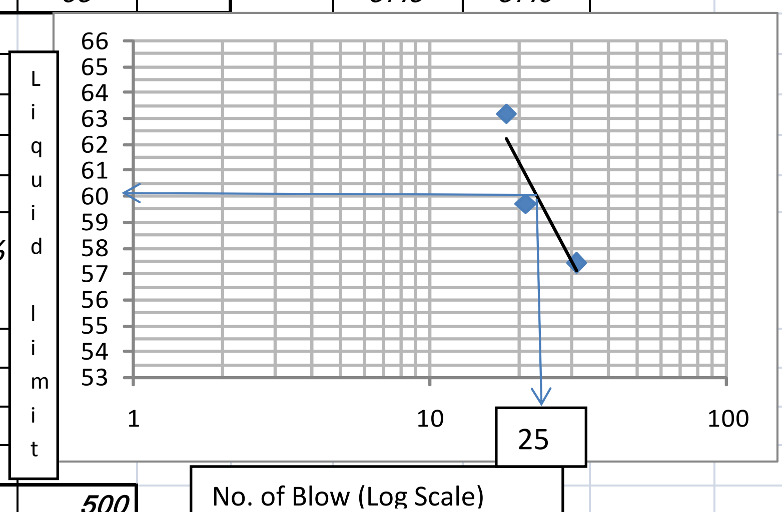
CONTRACTOR	Diriba Defersha General Contractor
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES
CLASSIFICATION METHOD : AASHTO M - 145

		Wet Method			
STATION:	67+030	DATE SAMPLED :	27/03/14	DATE TESTED :	1/4/2014
LOCATION :	Off Set 10.56 RHS	DEPTH:	10.56m	TEST REF. No. :	
SOIL DESCRIPTION :		2.10mt from sub grade material (Clay Material)			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		31	21	18			
Container No.		A-6	A -7	A - 8		A-9	A-10
Wet Soil+Cont	g	31.18	32.28	29.56		23.05	26.11
Dry Soil+Cont	g	27.08	28.04	25.85		21.66	24.62
Mass Container	g	19.94	20.94	19.98		17.93	20.66
Mass Moisture	g	4.1	4.24	3.71		1.39	1.49
Mass Dry Soil	g	7.14	7.1	5.87		3.73	3.96
Moist Content	%	57	60	63		37.3	37.6

LIQUID LIMIT :	60.0
PLASTIC LIMIT :	37.0
PLASTICITY INDEX :	23.0



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	3	3.0	1	99
0.425	102	105.0	21	79
0.075	88	193.0	39	61

Weight of tota sample befor washing	500
Weight of tota sample After washing	194
Soil Clasification	A-7-5(14)

REMARKS : ____________________

M. Technician	Material Engineer
	Tessema B.

ARBAMINCH KEMBA SAWLA ROAD PROJECT
LOT II BELTA OTOLO ROAD PROJECT

CONTRACTOR	Diriba Defersha General Contractor
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

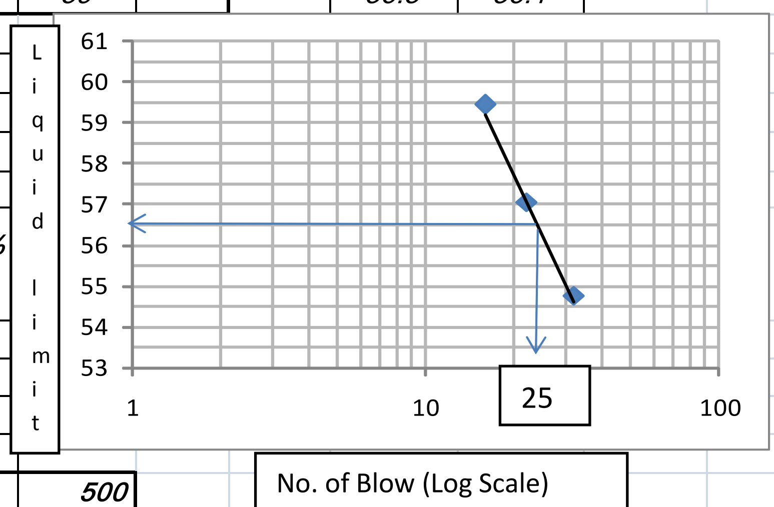
CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES
CLASSIFICATION METHOD : AASHTO M - 145

Wet Method

STATION:	67+030	DATE SAMPLED :	27/03/14	DATE TESTED :	1/4/2014
LOCATION :	Off Set 10.56m RHS	DEPTH:	12.38mt	TEST REF. No. :	
SOIL DESCRIPTION :		OFF SET 2.30mt FROM SUB-GRADE (weathered Material)			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		32	22	16			
Container No.		B-6	B -7	B - 8		B-9	B-10
Wet Soil+Cont	g	32.46	31.64	33.16		28.65	26.44
Dry Soil+Cont	g	28.21	27.84	28.91		27.06	24.63
Mass Container	g	20.45	21.18	21.76		22.70	19.62
Mass Moisture	g	4.25	3.8	4.25		1.59	1.81
Mass Dry Soil	g	7.76	6.66	7.15		4.36	5.01
Moist Content	%	55	57	59		36.5	36.1

LIQUID LIMIT :	56.0
PLASTIC LIMIT :	36.0
PLASTICITY INDEX :	20.0



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	20	20.0	4	96
0.425	82	102.0	20	80
0.075	145	247.0	49	51

Weight of tota sample befor washing	500
Weight of tota sample After washing	248
Soil Clasification	A-7-5(8)

REMARKS :

M. Technician	Material Engineer
	Tessema B.

Slide Location 2: Sta.68+020(AHD) to Sta.68+140(AHD) (RHS)

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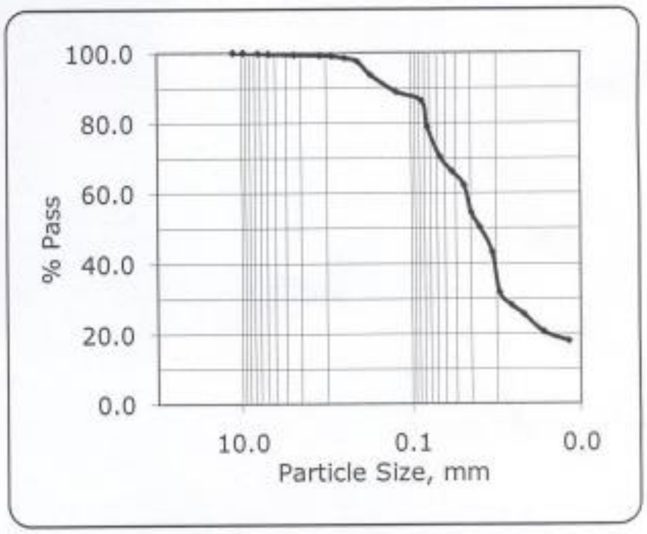
		Document No.: LAB-F-014	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 88

Project:- Belte - Otolo (Lot II)
Client:- Getinet Tilahun
Station:- 68 + 080 , RHS
Sample of:- Slide slope Material
Description of Sample:- Reddish brown Silty Clay mixed with highly weathered to decomposed GRAVEL
Depth Sampled:- 1.75 m from OGL
Lab. Number:- BO. Hydrom. Slop. 6/14
Date Tested:- 11/08/14
Date Issued:- 19/08/14

ASTM SIEVE (MM)	CUMULATIVE % RETAINED	% PASS
12.5	0.0	100.0
9.5	0.0	100.0
6.3	0.1	99.9
4.75	0.3	99.7
2.36	0.5	99.5
1.18	0.7	99.3
0.850	0.9	99.1
0.600	1.4	98.6
0.425	2.4	97.6
0.300	6.2	93.8
0.150	11.0	89.0
0.075	13.6	86.4
0.06402	21.1	78.9
0.04589	29.6	70.4
0.03316	33.8	66.2
0.02394	37.7	62.3
0.01978	45.5	54.5
0.01562	49.6	50.4
0.01117	56.8	43.2
0.00922	68.2	31.8
0.00664	71.8	28.2
0.00473	74.5	25.5
0.00276	79.4	20.6
0.00139	82.0	18.0



Remark:-

Reported by:- Lab. Engineer

Approved by:- Mat. Engineer

Company Name		Form N°
	CONSTRUCTION DESIGN SHARE SCo.	OF/CDSCo./123
Title: Grain Size Distribution	Issue NO: 1	Page N°: Page 1 of 1

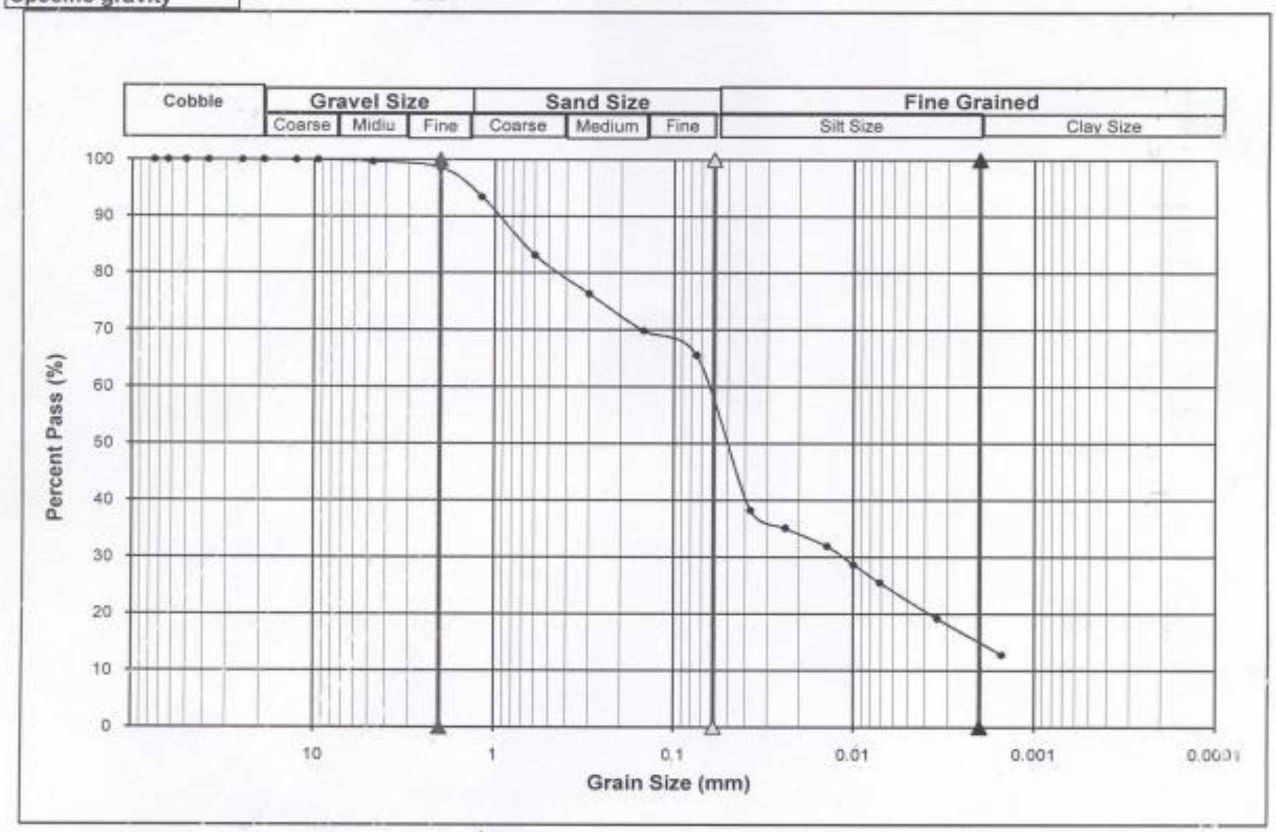
W.O/Proj. No	019641
Date	25/02/2014

Sample ID:	Gs-03/14/1956

Project :- Thesis / Belta-Otolo Road Project Lot-II /
Client :- Getinet Tilahun
Location:- Belta-Otolo
Description:- Grey Sand Silt With Some Clay
Test method :- ASTM D 422

Station :- 68+080
Sliding material at center line

Specific gravity 2.62



Tested by: Kokobe Lemma
Date : 04/03/2014
Checked by: Abate Legesse
Date: 05/03/2014

Approved by :- Anley Abebe
Date :- 06/03/2014

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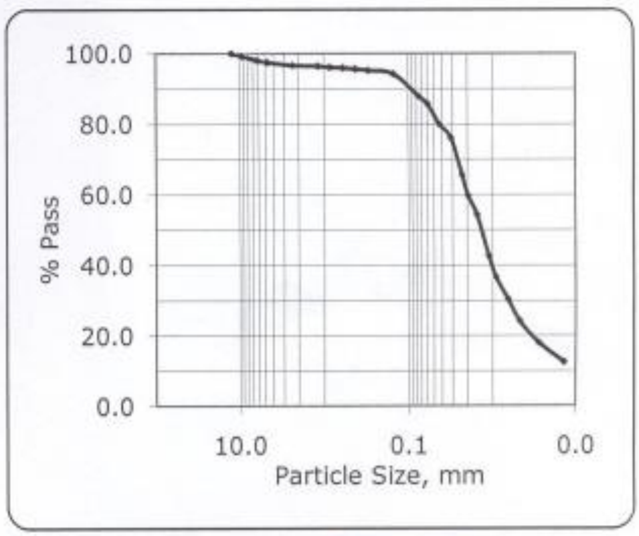
		Document No.: LAB-F-014	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 88

Project:- Belte - Otolo (Lot II)
Client:- Getinet Tilahun
Station:- 68 + 080 , RHS
Sample of:- Slide slope Material
Description of Sample:- Brownish Silty CLAY
Depth Sampled:- 1st trench
Lab. Number:- BO. Hydrom. Slop. 5/14
Date Tested:- 11/08/14
Date Issued:- 19/08/14

ASTM SIEVE (MM)	CUMULATIVE % RETAINED	% PASS
12.5	0.0	100.0
9.5	0.8	99.2
6.3	1.9	98.1
4.75	2.5	97.5
2.36	3.3	96.7
1.18	3.6	96.4
0.850	3.9	96.1
0.600	4.1	95.9
0.425	4.4	95.6
0.300	4.8	95.2
0.150	5.8	94.2
0.075	12.1	87.9
0.05919	14.3	85.7
0.04317	20.0	80.0
0.03128	23.9	76.1
0.02315	34.4	65.6
0.01954	40.3	59.7
0.01532	45.8	54.2
0.01105	57.4	42.6
0.00912	63.4	36.6
0.00652	69.6	30.4
0.00473	75.8	24.2
0.00276	82.0	18.0
0.00139	87.5	12.5



Remark:-

Reported by:- Lab. Engineer

Approved by:- Mat. Engineer

ARBAMINCH KEMBA SAWLA ROAD PROJECT
LOT II BELTA OTOLO ROAD PROJECT

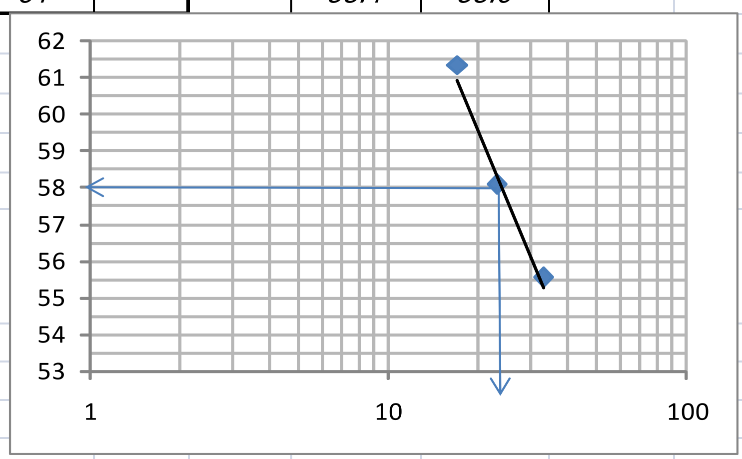
CONTRACTOR	Diriba Defersha General Construction Company
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES
CLASSIFICATION METHOD : AASHTO M - 145

		Wet Method			
STATION:	68+080	DATE SAMPLED :	20/02/14	DATE TESTED :	26/02/16
		DEPTH:		TEST REF. No. :	
Location		Sliding Material on the outer Stake 1.75 mts			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		33	23	17			
Container No.		B-1	B-2	B-3		B-4	B-5
Wet Soil+Cont	g	38.64	39.73	37.83		24.13	23.64
Dry Soil+Cont	g	32.93	33.52	32.18		22.95	22.32
Mass Container	g	22.65	22.83	22.97		19.88	18.85
Mass Moisture	g	5.71	6.21	5.65		1.18	1.32
Mass Dry Soil	g	10.28	10.69	9.21		3.07	3.47
Moist Content	%	56	58	61		38.4	38.0

LIQUID LIMIT :	58.0
PLASTIC LIMIT :	38.0
PLASTICITY INDEX :	20.0



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	12	12.0	2	98
0.425	85	97.0	19	81
0.075	135	232.0	46	54

Weight of tota sample befor washing	500
Weight of tota sample After washing	233
Soil Clasification	A-7-5(9)

REMARKS : ______________________

M. Technician	Material Engineer
	Tessema B.

ARBAMINCH KEMBA SAWLA ROAD PROJECT

LOT II BELTA OTOLO ROAD PROJECT

CONTRACTOR	Diriba Defersha General Construction Company
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES

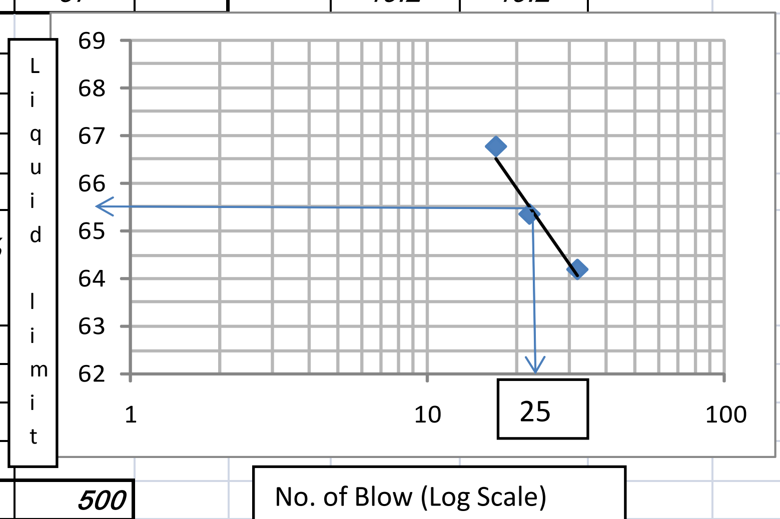
CLASSIFICATION METHOD : AASHTO M - 145

		Wet Method			
STATION:	68+080	DATE SAMPLED :	20/02/14	DATE TESTED :	25/02/16
		DEPTH:	10.9m	TEST REF. No. :	
Location		Sliding Material On the Center Line, Depth =0.3m			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		32	22	17			
Container No.		K-1	K-2	K-3		K-4	K-5
Wet Soil+Cont	g	32.63	35.57	34.84		24.51	29.06
Dry Soil+Cont	g	27.22	30.5	30.00		22.91	27.27
Mass Container	g	18.79	22.74	22.75		18.93	22.82
Mass Moisture	g	5.41	5.07	4.84		1.60	1.79
Mass Dry Soil	g	8.43	7.76	7.25		3.98	4.45
Moist Content	%	64	65	67		40.2	40.2

LIQUID LIMIT :	65.0
PLASTIC LIMIT :	40.0
PLASTICITY INDEX :	25.0

Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	37	37.0	7	93
0.425	132	169.0	34	66
0.075	89	258.0	52	48



Weight of tota sample befor washing	500
Weight of tota sample After washing	265
Soil Clasification	A-7-5(9)

REMARKS : ______________________

M. Technisian	Material Engineer
	Tessema B.

ARBAMINCH KEMBA SAWLA ROAD PROJECT
LOT II BELTA OTOLO ROAD PROJECT

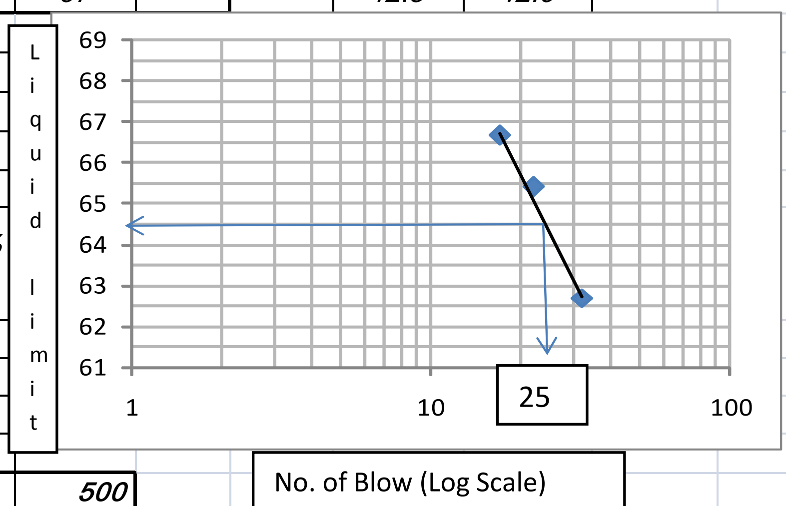
CONTRACTOR	Diriba Defersha General Construction Company
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES
CLASSIFICATION METHOD : AASHTO M - 145

		Wet Method			
STATION:	68+120	DATE SAMPLED :	28/03/14	DATE TESTED :	1/4/2014
		DEPTH:	1.7 mt from SG	TEST REF. No. :	
LOCATION :		3.5 mt offset from Center Line			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		32	22	17			
Container No.		A-6	A-7	A-8		A-9	A-10
Wet Soil+Cont	g	35.97	35.72	32.93		24.44	28.33
Dry Soil+Cont	g	30.86	30.52	27.69		22.76	26.71
Mass Container	g	22.71	22.57	19.83		18.81	22.85
Mass Moisture	g	5.11	5.2	5.24		1.68	1.62
Mass Dry Soil	g	8.15	7.95	7.86		3.95	3.86
Moist Content	%	63	65	67		42.5	42.0

LIQUID LIMIT :	64.5
PLASTIC LIMIT :	42.0
PLASTICITY INDEX :	22.5



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	64	64.0	13	87
0.425	118	182.0	36	64
0.075	52	234.0	47	53

Weight of tota sample befor washing	500
Weight of tota sample After washing	234
Soil Clasification	A-7-5(10)

REMARKS : ____________________

M. Technician	Material Engineer
	Tessema B.

Slide Location 3: Sta.72+740 to Sta.72+790(LHS)

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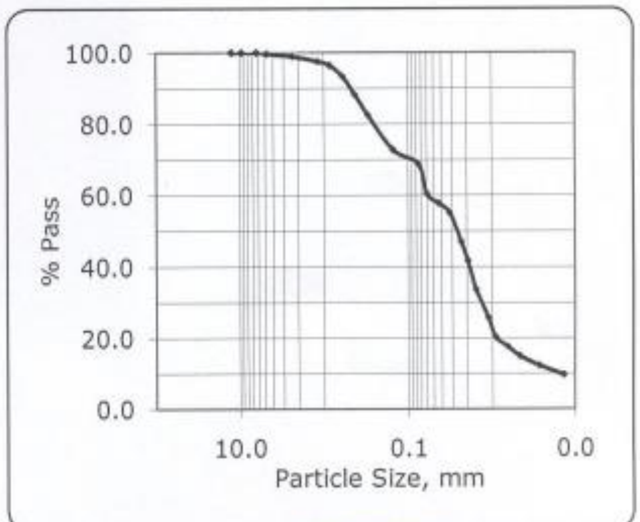
		Document No.: LAB-F-014	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 88

Project:- Belte - Otolo (Lot II)
Client:- Getinet Tilahun
Station:- 72 + 760
Sample of:- Slide slope Material
Description of Sample:- Light to yellowish brown Silty Clay highly weathered decomposed GRAVEL
Depth Sampled:- 6.50 m.
Lab. Number:- BO. Hydrom. Slop. 8/14
Date Tested:- 11/08/14
Date Issued:- 19/08/14

ASTM SIEVE (MM)	CUMULATIVE % RETAINED	% PASS
12.5	0.0	100.0
9.5	0.0	100.0
6.3	0.0	100.0
4.75	0.3	99.7
2.36	1.0	99.0
1.18	2.4	97.6
0.850	3.5	96.5
0.600	6.6	93.4
0.425	11.7	88.3
0.300	17.4	82.6
0.150	27.2	72.8
0.075	31.2	68.8
0.05985	39.6	60.4
0.04294	42.2	57.8
0.03218	44.9	55.1
0.02352	52.9	47.1
0.01958	58.3	41.7
0.01551	66.3	33.7
0.01105	74.3	25.7
0.00918	79.7	20.3
0.00656	82.4	17.7
0.00469	85.0	15.0
0.00272	87.7	12.3
0.00138	90.4	9.6



Remark:-

Reported by:- Lab. Engineer

Approved by:- Mat. Engineer

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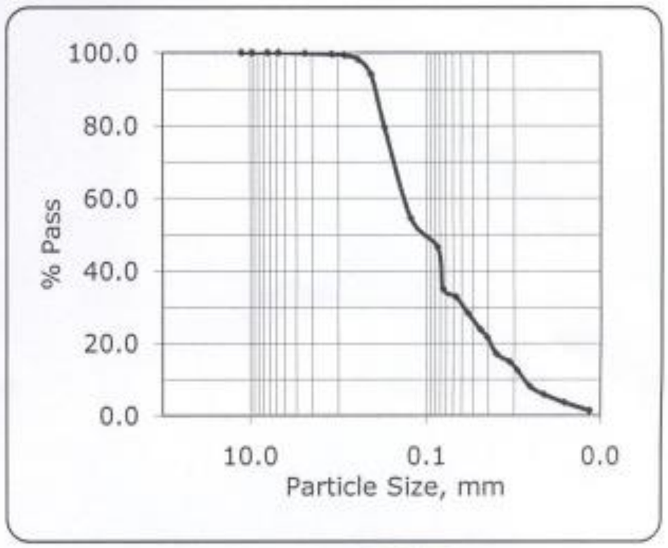
		Document No.: LAB-F-014	Revision No. 0	Effective Date: 10/01/2013

TEST METHOD : AASHTO T - 88

Project:- Belte - Otolo (Lot II)
Client:- Getinet Tilahun
Station:- 72 + 760 center lane
Sample of:- Slide slope Material
Description of Sample:- Brownish Silty CLAY
Depth Sampled:- 0.0 m.
Lab. Number:- BO. Hydrom. Slop. 7/14
Date Tested:- 11/08/14
Date Issued:- 19/08/14

ASTM SIEVE (MM)	CUMULATIVE % RETAINED	% PASS
12.5	0.0	100.0
9.5	0.0	100.0
6.3	0.0	100.0
4.75	0.0	100.0
2.36	0.1	99.9
1.18	0.4	99.6
0.850	0.6	99.4
0.600	1.8	98.2
0.425	5.9	94.1
0.300	20.5	79.5
0.150	45.5	54.5
0.075	53.4	46.6
0.06518	64.8	35.2
0.04647	67.1	32.9
0.03391	71.6	28.4
0.02452	76.1	23.9
0.02038	78.4	21.6
0.01601	82.9	17.1
0.01136	85.1	14.9
0.00928	87.4	12.6
0.00660	91.9	8.1
0.00456	94.1	5.9
0.00265	96.4	3.6
0.00135	98.7	1.4



Remark:-

Reported by:- Lab. Engineer

Approved by:- Mat. Engineer

ARBAMINCH KEMBA SAWLA ROAD PROJECT
LOT II BELTA OTOLO ROAD PROJECT

CONTRACTOR	Diriba Defersha General Construction Company
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

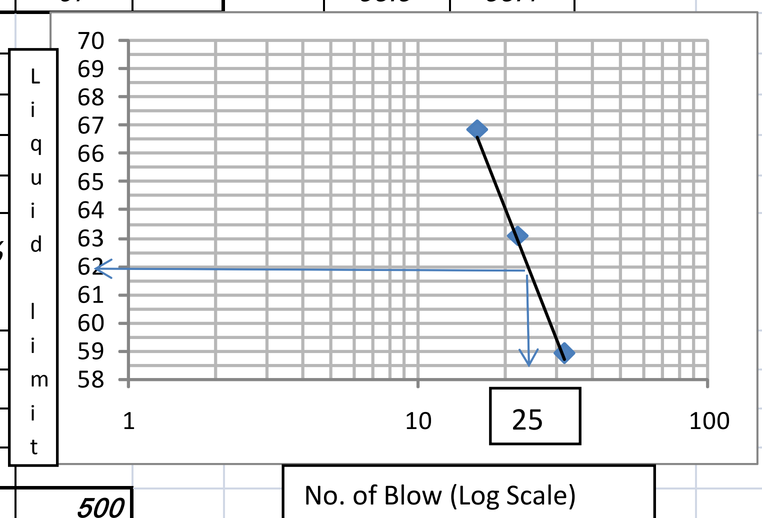
CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES
CLASSIFICATION METHOD : AASHTO M - 145

Dry Method

STATION:	72+760 LHS	DATE SAMPLED :	28/03/14	DATE TESTED :	1/4/2014
LOCATION :	Slide Slope	DEPTH:	6.5mt	TEST REF. No. :	
SOIL DESCRIPTION :		SLIDING SLOPE FROM THE TOP SECTION			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		32	22	16			
Container No.		C-1	C -2	C - 3		C-4	C-5
Wet Soil+Cont	g	35.49	31.33	30.32		28.24	25.29
Dry Soil+Cont	g	30.76	26.92	25.88		26.66	23.74
Mass Container	g	22.73	19.93	19.24		22.53	19.70
Mass Moisture	g	4.73	4.41	4.44		1.58	1.55
Mass Dry Soil	g	8.03	6.99	6.64		4.13	4.04
Moist Content	%	59	63	67		38.3	38.4

LIQUID LIMIT :	62.0
PLASTIC LIMIT :	38.0
PLASTICITY INDEX :	24.0



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	9	9.0	2	98
0.425	117	126.0	25	75
0.075	159	285.0	57	43

Weight of tota sample befor washing	500
Weight of tota sample After washing	285
Soil Clasification	A-7-5(6)

REMARKS : ______________________

M. Technician	Material Engineer
	Tessema B.

ARBAMINCH KEMBA SAWLA ROAD PROJECT
LOT II BELTA OTOLO ROAD PROJECT

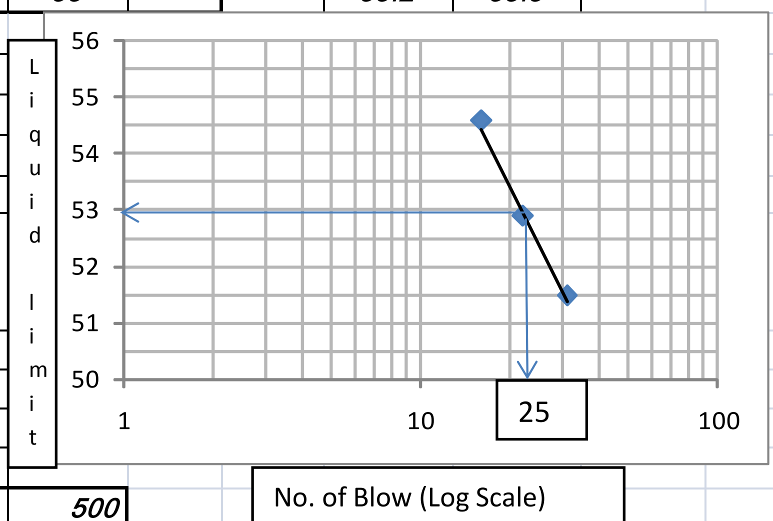
CONTRACTOR	Diriba Defersha General Contractor
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES
CLASSIFICATION METHOD : AASHTO M - 145

		Wet Method			
STATION:	72+790 LHS	DATE SAMPLED :	28/03/14	DATE TESTED :	1/4/2014
LOCATION :	Off Set 3.5 mt C.L	DEPTH:	1.7 mt	TEST REF. No. :	
SOIL DESCRIPTION :					

LIQUID LIMIT						PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		31	22	16			
Container No.		A-1	A-2	A-3		A-4	A-5
Wet Soil+Cont	g	31.37	32.6	31.23		23.26	27.57
Dry Soil+Cont	g	27.39	27.86	27.3		22.12	26.30
Mass Container	g	19.66	18.9	20.1		18.88	22.72
Mass Moisture	g	3.98	4.74	3.93		1.14	1.27
Mass Dry Soil	g	7.73	8.96	7.2		3.24	3.58
Moist Content	%	51	53	55		35.2	35.5

LIQUID LIMIT :	53.0
PLASTIC LIMIT :	35.0
PLASTICITY INDEX :	18.0



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	55	55.0	11	89
0.425	113	168.0	34	66
0.075	78	246.0	49	51

Weight of tota sample befor washing	500
Weight of tota sample After washing	249
Soil Clasification	A-7-5(7)

REMARKS : ______________________

M. Technician	Material Engineer
	Tessema B.

ARBAMINCH KEMBA SAWLA ROAD PROJECT

LOT II BELTA OTOLO ROAD PROJECT

CONTRACTOR	Diriba Defersha General Contractor
CONSULTANT	Pure Consulting Engineer Pvt Ltd Company

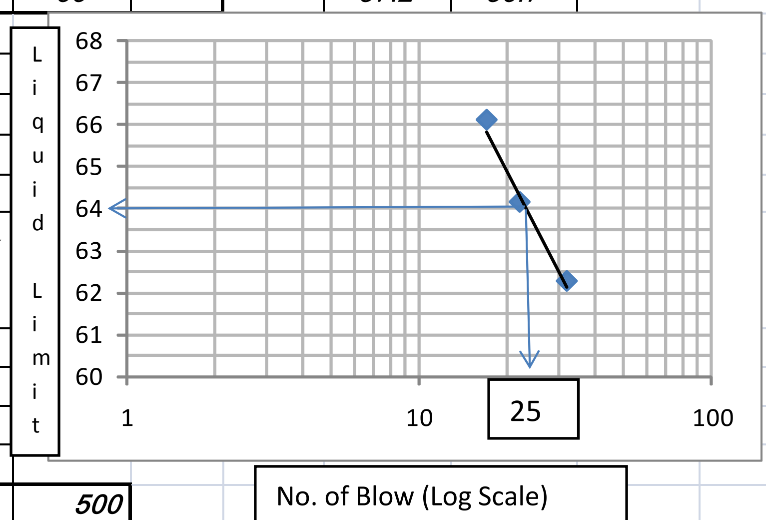
CLASSIFICATION OF SOIL AND SOIL-AGGREGATE MIXTURES

CLASSIFICATION METHOD : AASHTO M - 145

		Wet Method			
STATION:	72+760	DATE SAMPLED :	20/02/14	DATE TESTED :	25/02/16
		DEPTH:	0.3m	TEST REF. No. :	
Location		Sliding Material On the Center Line			

		LIQUID LIMIT				PLASTIC LIMIT	
Test No.		1	2	3	4	1	2
Number of blows		32	22	17			
Container No.		B-1	B-2	B-3		B-4	B-5
Wet Soil+Cont	g	38.44	36.29	37.13		24.46	24.42
Dry Soil+Cont	g	32.38	31.03	31.49		23.21	22.92
Mass Container	g	22.65	22.83	22.96		19.85	18.83
Mass Moisture	g	6.06	5.26	5.64		1.25	1.50
Mass Dry Soil	g	9.73	8.2	8.53		3.36	4.09
Moist Content	%	62	64	66		37.2	36.7

LIQUID LIMIT :	64.0
PLASTIC LIMIT :	37.0
PLASTICITY INDEX :	27.0



Sieve Sizes	Wt. Retained	Cum wt retad	Cum % Retaind	Cum. % Pass
4.75				100
2.00	2	2.0	0	100
0.425	171	173.0	35	65
0.075	82	255.0	51	49

Weight of tota sample befor washing	500
Weight of tota sample After washing	331
Soil Clasification	A-7-5(10)

REMARKS :

M. Technician	Material Engineer
	Tessema B.

Company Name: CONSTRUCTION DESIGN SHARE CO.	Form No: OF/CDSCo./117	
Title: LABORATORY TEST RESULT	Issue NO: 1	Page N°: Page 1 of 1

W.O.No :- 020549
Date :- 09/05/2014

Project :- Thesis / Belta - Otolo Road Project Lot - II /
Client :- Getinet Tilahun
Location :- Belta - Otolo
Object :- Undisturbed Soil Samples
Test Type :- UU Triaxial Shear Test & Various

N°	Station	Depth (m)	Moisture Content (%)	Dry Density (Kg/m^3)	Bulk Density (Kg/m^3)	Cell Pressure (KN/m2)	Triaxial Shear Strength C (KN/m^2)
1	68+030 RHS Offset 3m	9.00 From OGL	28.21	1217	1535	150	201
2	72+940 Offset 3m	12.15 From OGL	26.16	1196	1533	200	283

Note

Four graphs for grain size distribution test result are drawn and attached here with.

We reported one point triaxial shear strength test for each samples .

Tested by :- Yeshi Tadesse
Date :- 25/06/2014
Checked by :- Abate Legesse
Date :- 26/06/2014

Approved by :- BLAT
Date :- 31/07/2014

Construction Design S.co.
Central Laboratory

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

Appendix –B

DESIGN OF REMEDIAL STRUCTURES

Appendix –C

PICTURES OF FACTOR OF SAFETY ANALYSIS

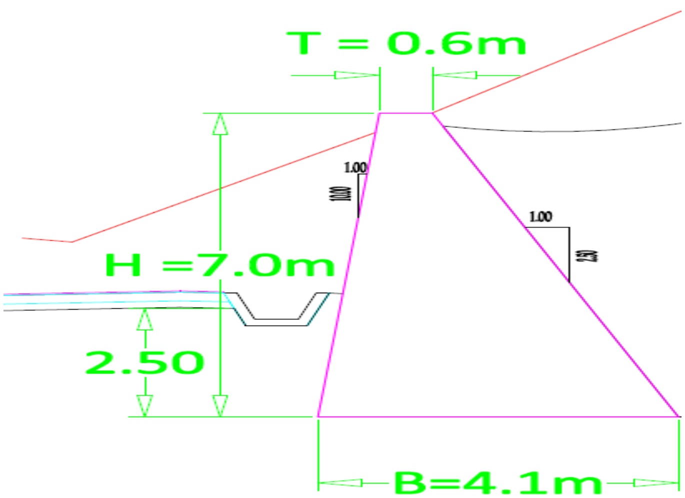
Design of Remedial Structure:

1. Sta. 66+995 – Sta.67+055(RHS)

The common procedures for the design of retaining wall shall be as follows:

- **The geometry of the failed soil mass and tentative dimensions of the retaining wall as shown below in Figure B.1 are:**

Total Height H=7.0m, Free Height H_{fr}=4.5m, Embedded Height H_{emb}=2.50m, Top width T= 0.6m, Battering Slope (Back fill Side, V:H) 2.5:1 and (Front Side, V:H) 10:1 and Bottom width B=4.10m.



The structural excavation for the retaining wall should be made as per shown in the Figure B.1 for safety related issues. However, for the analysis of the retaining wall one assumes the backfill material i.e. directly above the heel of the wall, to be the effective back of the wall and shall be backfilled at 8-15^0 slope from the horizontal to create a smooth transition with the flattened slope.

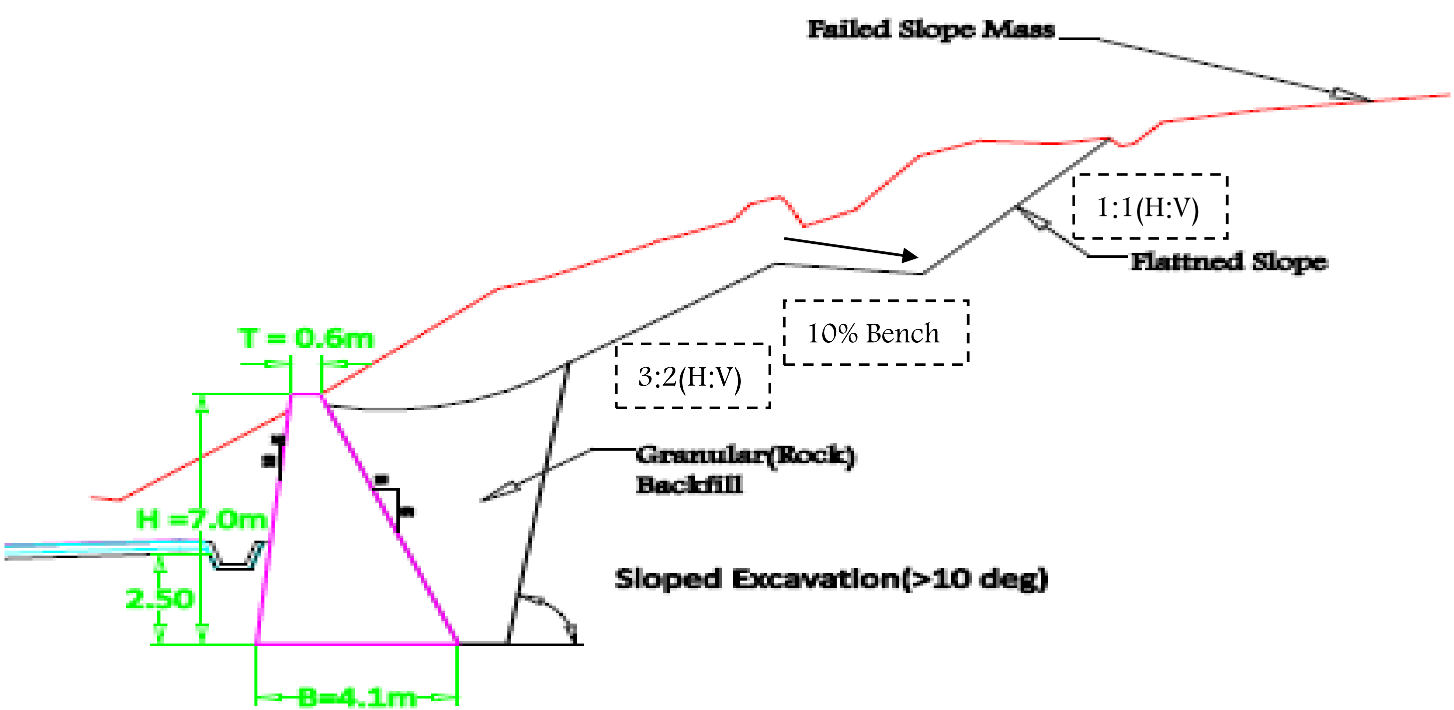


Figure B.1: Tentative dimensions of retaining wall and failed slope to be flattened

- **The computation of the Vertical Dead Loads:**

For convenience, the acting loads on the retaining wall are segmented and tabulated hereunder.

Segmental Loads	Unit Weight (kN/m^3)	Vertical Loads(Weights) (kN/m)
W1: granular backfill	20	0.3125*20=6.25
W2: Class B Masonry	23	2.45*23=56.35
W3: granular backfill	20	9.597*20=191.94
W4: Class B Masonry	23	9.8*23=225.4
W5: Class B Masonry	23	4.2*23=96.6

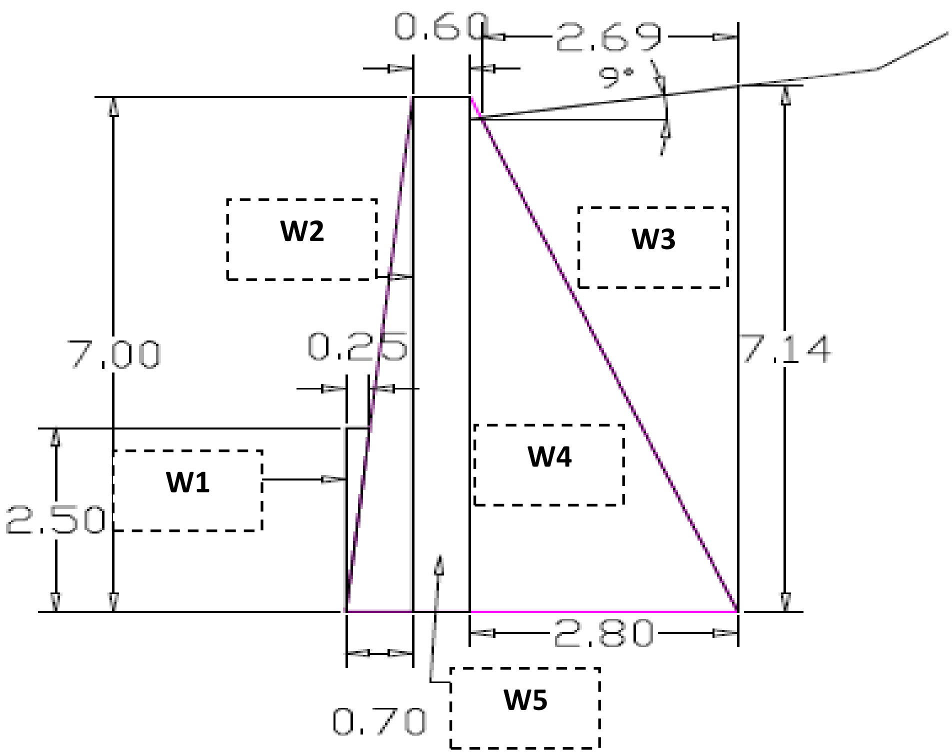


Figure B.2: Tentative dimensions of retaining wall and vertical load components

- **Calculation of lateral Pressures:**

The Coulomb's lateral earth pressure theory shall be used for the computation of the active earth pressure. For use of the forgoing, the following presumed/assumed values are made:

- Backfill slope shall be, $\beta = 9^0$ from the horizontal and $\propto = 68^0$;
- For ease of construction and timely retaining of the soil mass(for development of lateral pressure at rest) rock fill backfill(nearly cohesion less) shall be (hand)placed and presumed value of drained friction angle of 35^0 shall be used;
- Wall friction angle, $\delta' = 24^0$ is used;

The active Pressure, Pa, can be determined using coulomb's formula:

$$P_a = \frac{1}{2}\gamma H^2 K_a$$

Where:

γ : unit weight of the backfill material

H : the height of the retaining wall

K_a : the active earth pressure coefficient given by :

$$K_a = \frac{\sin 2(\propto + \emptyset')}{\sin 2\alpha \sin(\alpha - \delta)\left[1 + \left(\frac{\sin(\emptyset' + \delta)\sin(\emptyset' - \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}\right)^{\frac{1}{2}}\right]^2}$$

where:

$\propto$ −batttering of the retainig wall from the horizontal, heel side

$\emptyset'$ − Effective Friction angle of the backfill material

δ − Friction angle between the masonry retaining wall and backfill material

β − backfill slope with the horizontal

Inserting the above values in the above equation, the coefficient of active lateral pressure,

K_a equals **0.52**

Since the total active lateral force is inclined at an angle of , $\delta = 24$ degree to the normal drawn to the back of the wall or [δ + (90-∝)] degrees from the horizontal, the horizontal component of the active coefficient,

K_{ah} equals: Cos (46)*0.52= **0.36**

And similarly the vertical component of the active earth pressure coefficient:

K_{av}: equals **0.37**

Therefore, the distribution of the horizontal and vertical forces from lateral earth pressure are:

$$P_a = \frac{1}{2}\gamma H^2 K_a = 0.5 * 20 * 7^2 * 0.52 = \mathbf{254.8kN/m}$$

P_{ah} = **176.4kN/m** and P_{av} = 181.3kN/**m**

The Summation of Vertical and Horizontal Forces:

$$\sum V = W_1 + W_2 + W_3 + W_4 + W_5 + P_{av}$$
$$= 6.25 + 56.35 + 191.94 + 225.4 + 96.6 + 181.3 = \mathbf{757.84kN/m}$$

$$\sum H = P_{ah} = \mathbf{176.4kN/m}$$

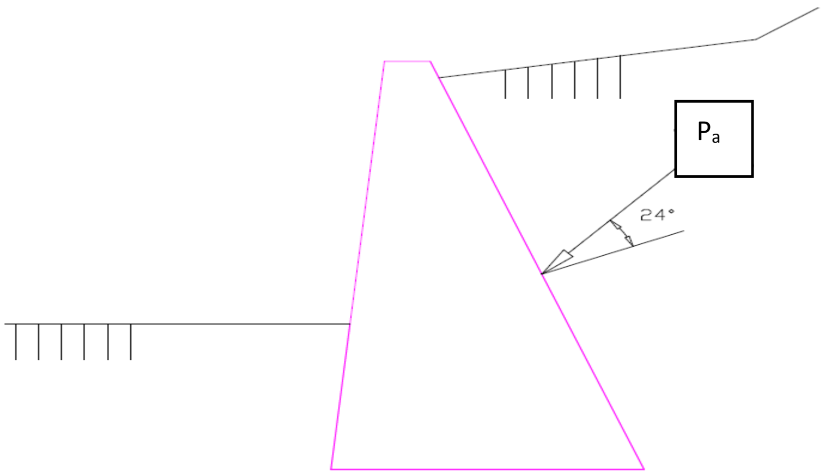


Figure B.3: Active lateral earth pressure

Static Stability Criteria's of the Retaining Wall

i. Stability against Overturning:

Taking moment about the Toe of the retaining wall and considering clockwise direction as positive:

$\sum M_{toe}$= W_1*(0.25/3)+W_2*(0.7/3)+W_3*((2.69/3)+1.3)+W_4*((2.8/3)+1.3)+W_5*((0.6/2)+0.7)
+P_{av}*(4.1-((H/3)/tan68))-P_{ah}*(H/3)

$\sum M_{toe}$= 0.521+13.148+421.628+503.393+96.6+572.413-411.60=1,196.10kN-m/m

Next, the location of the resultant vertical and horizontal forces are;

$\sum \mathrm{V}\overline{\mathrm{X}} = \sum$ Moments of Vertical Forces: stabilizing moments

$$\overline{\mathrm{X}} = \frac{1607.71}{757.84} = 2.12\mathrm{m}$$

$\sum \mathrm{H}\overline{\mathrm{Y}} = \sum$ Moments of Horizontal Forces: overturning moments

$$\overline{\mathrm{Y}} = \frac{411.60}{176.4} = 2.33\mathrm{m}$$

The location of the Resultant Force from the toe of the retaining wall;

$\omega = \tan^{-1}\frac{\sum V}{\sum H} = \tan^{-1}\frac{757.84}{176.4} = 76.89^0$

X=$\overline{\mathrm{X}}$ - $\overline{\mathrm{Y}}$ tan13.21=2.12-2.33*tan13.21=1.57m

Next, checking the location of the resultant force of the wall and the backfill whether it lies at the middle third of the wall base to maintain a positive at the uphill base of the wall i.e. heel;

e=B/2-X=4.1*0.5-1.57=0.48m while B/6=4.1/6=0.68m, here as B/6 >e , the resultant lies within the middle third or Kern, as indicated in Figure B.4 below.

Finally, the Factor of Safety against sliding is given as:

$\mathrm{F_o} = \frac{\sum \text{Stabilizing Moments}}{\sum \text{Overturning Moments}}$

$$F_o = \frac{1607.71}{411.60} = 3.91 > 2.0 \text{......................OK!}$$

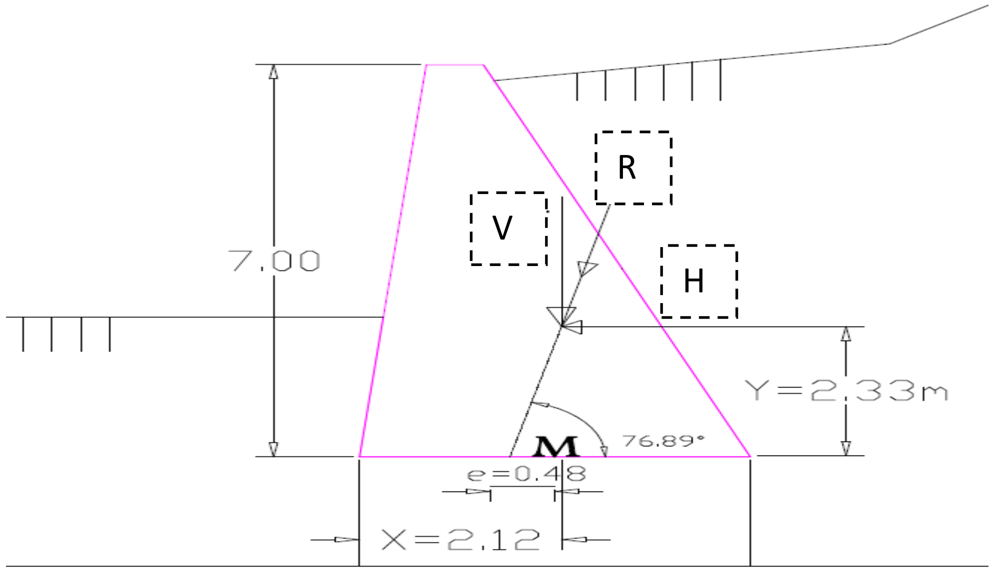


Figure B.4: Resultant force location

ii. **Stability against Sliding along the base:**

The Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Horizontal Resisting Forces}}{\sum \text{Horizontal Sliding Forces}}$$

Here, horizontal resistance along the base of a retaining wall is provided by friction between the base and the underlying foundation soil. The friction between the masonry base and soil is likely to be less than the shear strength of cohesion less soil in the foundation. Generally, these values suggest the following values for base/soil friction angle δ': [3]

- $\delta' = 30^0$ for coarse -grained soil containing no silt and clay
- $\delta' = 24^0$ for coarse -grained soil containing silt
- $\delta' = 20^0$ for thin layer of granular fill overlying cohesive soils

Therefore, taking $\delta' = 24^0$ and neglecting the horizontal passive resistance in front of the wall, we have;

$$F_o = \frac{\sum V \tan \delta'_g}{\sum H} = \frac{757.84 * \tan 24}{176.4}$$

$F_o = 1.91 > 1.5$ .... OK!

iii. **Bearing Capacity Failure about the Toe of the Wall:**

The computation of contact pressure under the base of the wall shall be made using the flexure formula. To determine the maximum and minimum contact pressures it is necessary to calculate the moment of the forces acting on the wall about the center of the base.

$M_{toe} = \sum V\bar{X} - \sum H\,\bar{Y}$

$M_M = \sum V\left(\bar{X} - \frac{B}{2}\right) - \sum H\,\bar{Y}$

$M_M = M_{toe} - \sum V\frac{B}{2}$

= 1,196.10-(757.84*4.1*0.5)=-357.47kN-m/m

To this end, the contact pressure under the base of the wall (B x 1) from flexure formula is given as:

$$\sigma_{toe/heel} = \frac{\sum V}{B} \pm \frac{6M_M}{B^2}$$

$\sigma_{toe/heel} = \frac{757.84}{4.1} \pm \frac{-6*357.47}{4.1^2} = 184.84 \pm 127.59$

$\sigma_{toe} = 312.43kN/m^2$, $\sigma_{heel} = 57.25kN/m^2$ and $\sigma_{av} = 184.84kN/m^2$

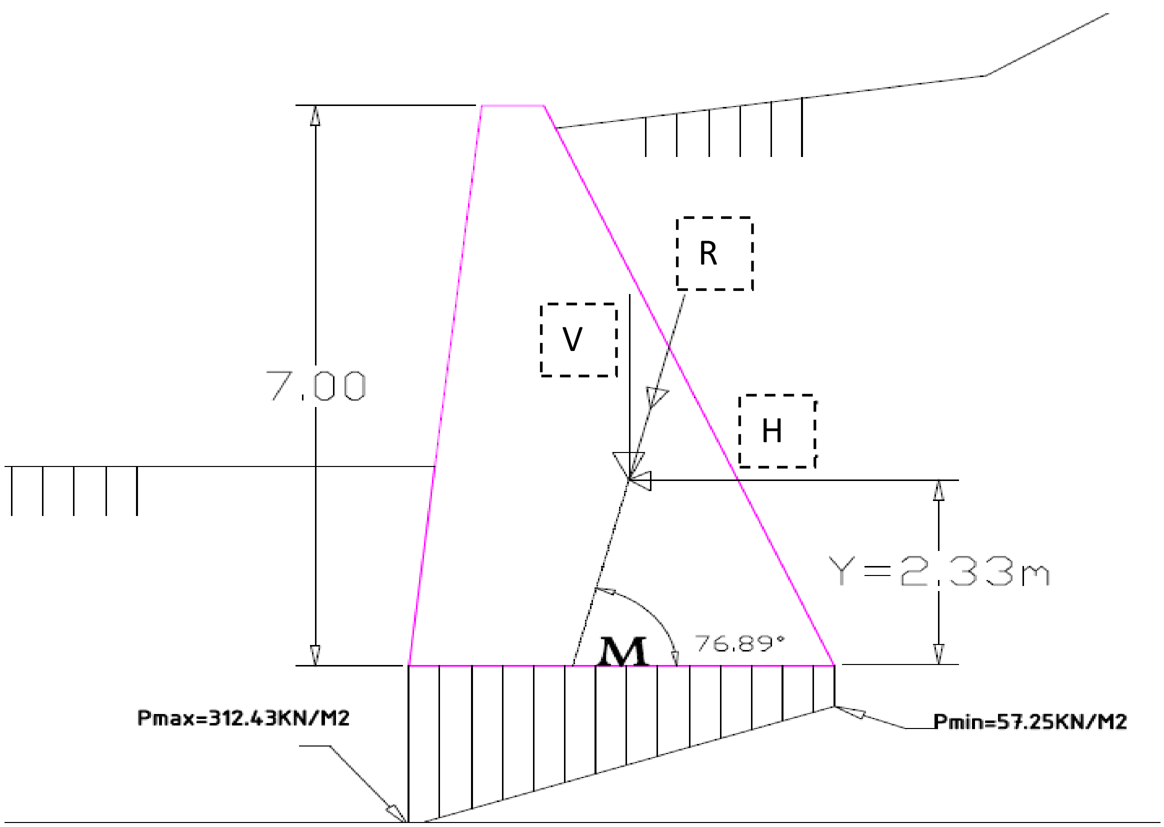


Figure B.5: Contact pressure distribution under the base of the wall

The Bearing Capacity of the foundation material shall be taken from presumptive bearing capacity values as elaborate soil investigation was not done on the weathered rock foundation material. The presumed design bearing resistance for vertical load which shall be reduced at least by 25% shall be used to account for the horizontal load component.[12]

Factor of safety against bearing capacity failure at the toe shall be given as:

$$F_B = \frac{\text{Pressumed Design Bearing Resistance}}{\text{Toe contact Pressure}} = \frac{850}{312.43} = 2.72 > 2.5\ OK$$

iv. **Deep Seated foundation failure Stability : Static and Dynamic conditions**

Overall stability of the wall system within the context of slope stability must also be assessed to ensure that no failure occurs either in the backfill or the native soil. As such, a separate analysis for slope stability must be performed on the zone in the vicinity of the wall using conventional limit equilibrium slope stability methods.

The slope stability model that is used for checking the proposed remedial structure and flattened slope is referred from Fig.B.8 and this slope stability analysis is carried out with the use of Geo Slope V.7 (Slope/W) software.

The Steps involved in the analysis of the designed remedial structure are bulleted hereunder;

- **Analysis Method/Model:** Since the solution is sensitive to the location of the axis of rotation (i.e. fully specified slip surfaces from the use of back analysis and field observations) methods that satisfy both force and moment equilibrium shall be used with shear strength values obtained from back analysis.
- **Use of a High - soil strength model to represent non - water materials or Surcharge loading:** The failure of the retaining wall is usually a result of the undercutting of the retaining wall, not shearing of the wall itself. For this reasons, the strength of the retaining wall itself becomes trivial, but the weight of the wall acting as a stabilizing force is critical. Accordingly, two methods are used to model the retaining wall with nearly close Factor of Safety results :

  Method 1: the gravity retaining wall and the backfill material shall be modeled as a High-strength soil model(C=5000Kpa and $\phi=45^0$) with an appropriate unit weight ($23kN/m^3$) that ensures that the weight of the wall and the granular backfill is included in the analysis.

  Method 2: the retaining wall and the rock backfill material shall be modeled as a surcharge loading at the toe of the flattened slope by introducing the weight of the wall as area loading i.e. $23kN/m^2$.
- **Slip Surfaces:** fully specified surface, referenced from back analysis and field observation, obliged to pass within the native soil being retained and just below the backfill material and retaining wall is analyzed.

Accordingly, a factor of safety of 1.365 is calculated is taking the foregoing inputs in to Geo slope software V.7. a value which is in the range required for global stability 1.2-1.5 as displayed under FigB.5(a)

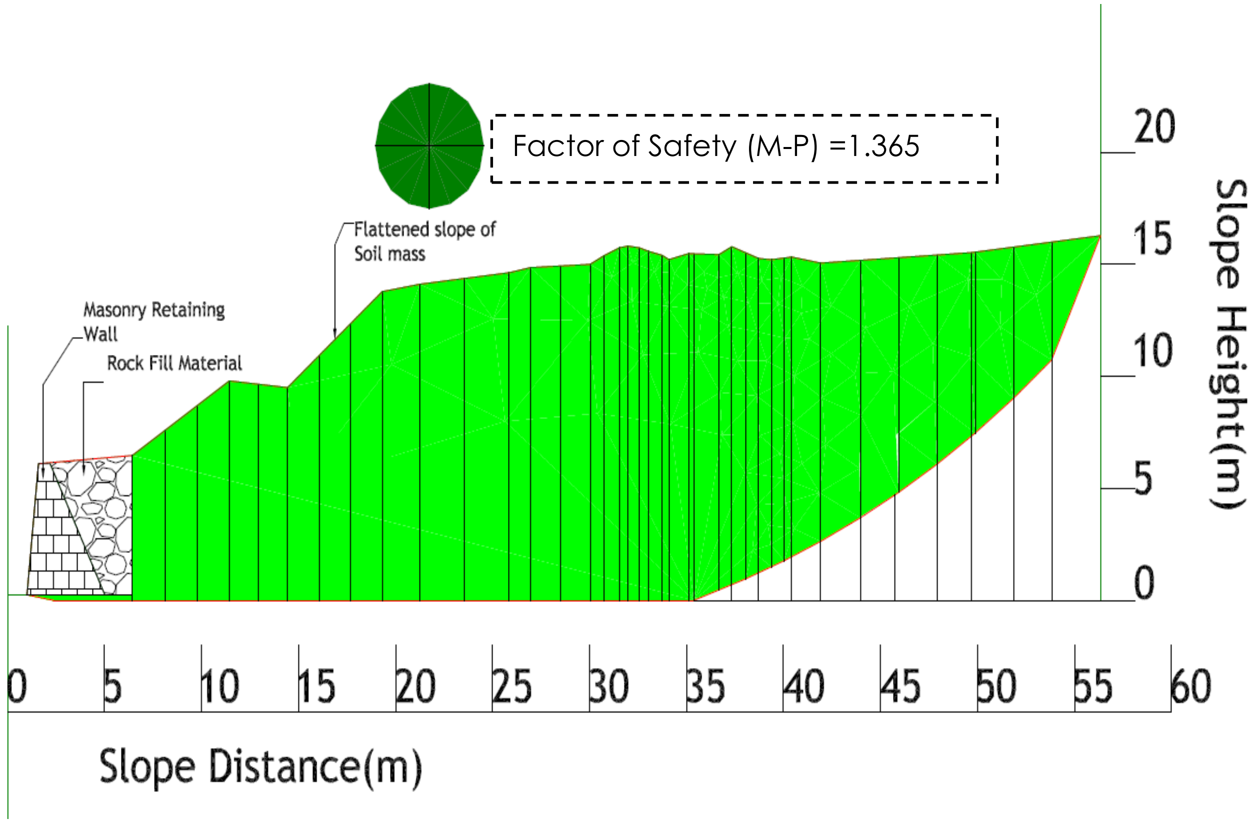


Figure B.5 (a): Remedial structure and flattened slope of global slope stability analysis

Similarly the global stability of the slope for seismic loading conditions is performed with the use of pseudostatic approach where the critical seismic coefficient corresponding to the factor of safety of 1 is calculated from the variation of horizontal seismic coefficients for the slope geometry in consideration while the stability of the slope to survive the design horizontal acceleration of 0.1 is compared with the critical horizontal seismic coefficient. [1]

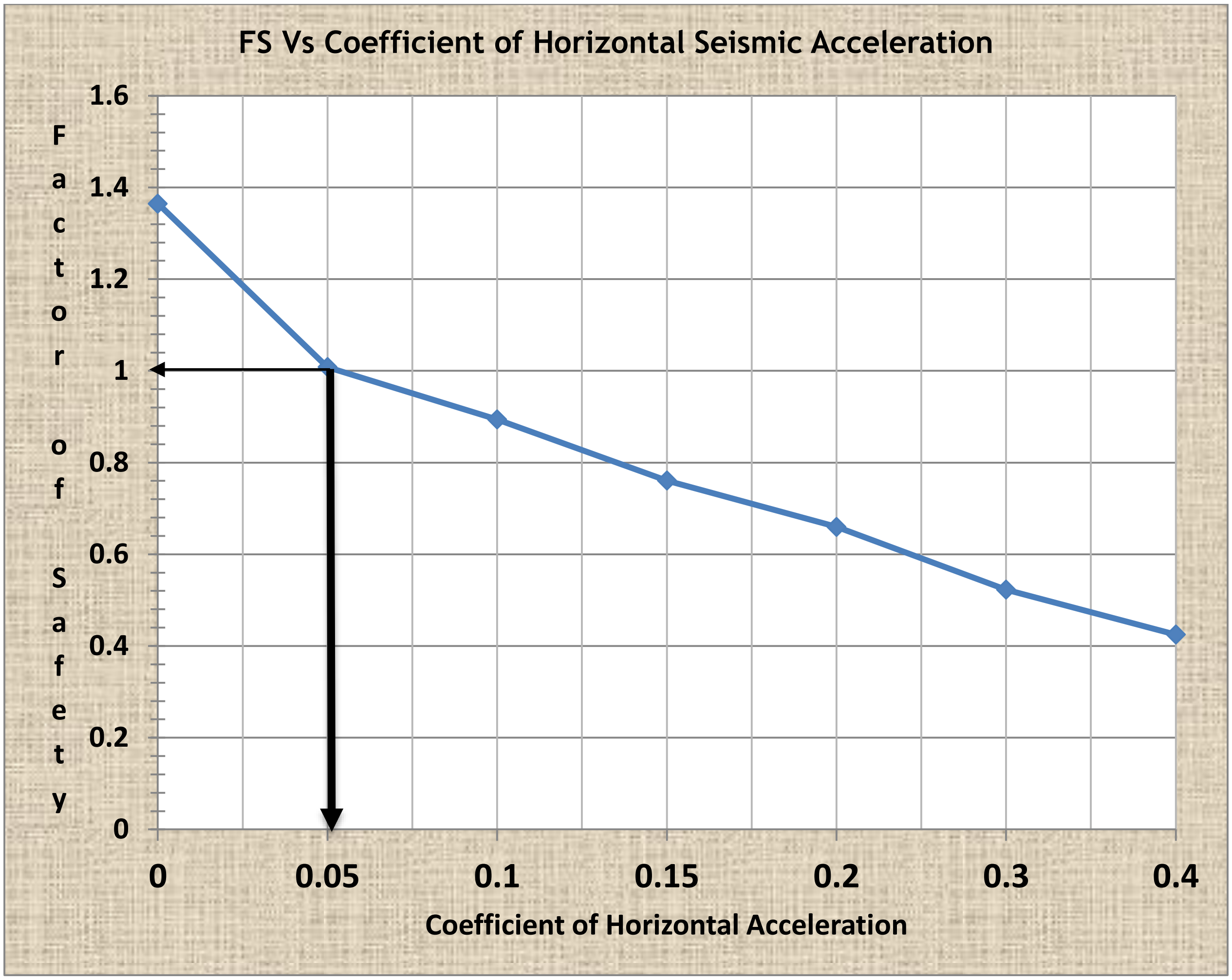


Figure B.5 (b): Variation of FOS with horizontal seismic coefficient

From figure B.5 (b) the critical horizontal seismic coefficient corresponding to a factor of safety of 1 is calculated to be K_y=0.05 which is less than the design horizontal seismic coefficient but equal to $0.5a_h$. Hence the slope is expected to survive the design earthquake with slightly minor damages.

Dynamic and Static Stability Criteria's of the Retaining Wall

The study area resides between Arbaminch and Sawula (Felege Neway) towns where these towns are located within Zone 4 and Zone 3 respectively of the seismic hazard zones. In view of the foregoing, the design bedrock acceleration of gravity of (Horizontal) are 0.1 and 0.07 for Arbaminch and Sawula Towns respectively. To this end, the horizontal and vertical component of earthquake acceleration coefficient of the study area is selected to be 0.1 and 0 respectively. [11]

Therefore, using the procedures of Mononobe Okabe lateral earth pressure equations for seismic forces one can calculate the active seismic force acting on the retaining wall accordingly while reference is made on Figure B.6 below; [4]

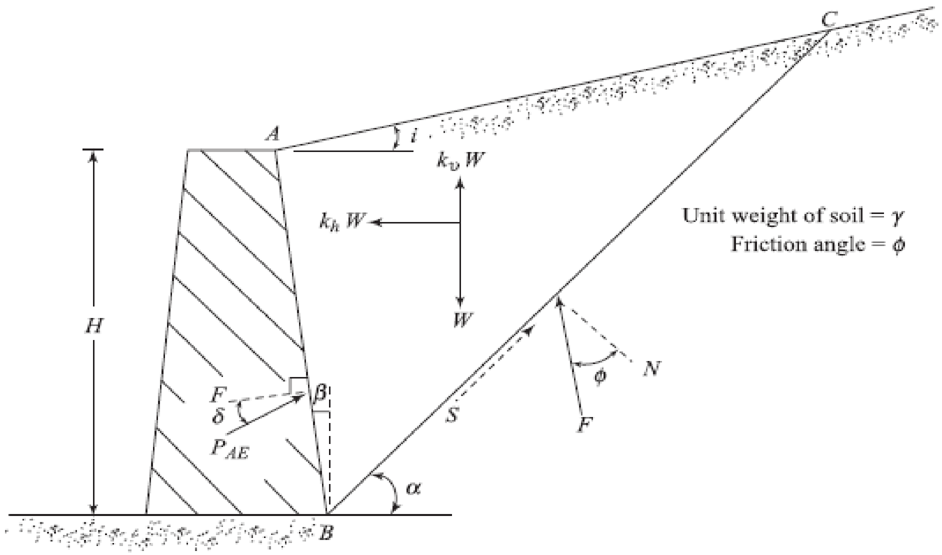


Figure B.6: Derivations of Mononobe Okabe seismic induced lateral active earth pressure equations

$$P_{AE} = \frac{1}{2}\gamma H^2(1-k_v)K_{AE}$$

Or after simplification and to keep similarity with lateral earth pressure equation (Static case)

$$P_{AE} = P_A(i', \beta')(1-k_v)\ (\overset{*}{p})$$

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^2}$$

$$\theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$$

Where:

$$i = 9^0, \beta = 22^0, \delta' = 24^0, \emptyset' = 35 \text{ and } \gamma = 20\ KN/M^3$$

Calculate the seismic inertia angle, $\theta = \tan^{-1}\frac{K_h}{1-K_h} = \tan^{-1}\left(\frac{0.1}{1-0}\right) = 5.71^0$

Calculate, $i' = i + \theta = 9 + 5.71 = 14.71^0$

Calculate, $\beta' = \beta + \theta = 22 + 5.71 = 27.71^0$

Calculate, the coefficient of active lateral earth pressure, $K_A(i', \beta')$, using following equation and inserting the values:

$$K_A(i', \beta') = \frac{\cos^2(\phi-\beta')}{\cos^2\beta'\cos(\delta+\beta')\left[1+\left\{\frac{\sin(\delta+\phi)\sin(\phi-i')}{\cos(\delta+\beta')\cos(\beta'-i')}\right\}^{1/2}\right]^2}$$

$K_A(i', \beta')$=0.699

Calculate, $P_a = \frac{1}{2}\gamma H^2 K_a$ =342.51KN/M and

Calculate, P*, using the following equation

$$\overset{*}{p} = \left(\frac{\cos^2 \beta'}{\cos\theta \cos^2 \beta} \right)$$

p*=0.921 and hence, P_{AE} equals 315.45kN/m

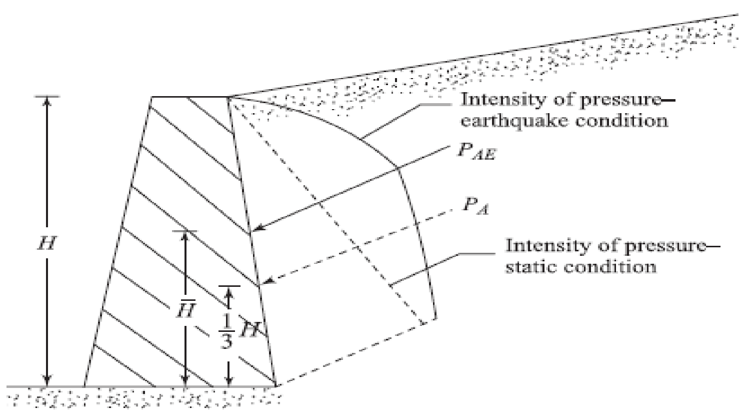
While, the vertical and horizontal components of seismic lateral active pressures similar to the static active lateral pressure case, P_{aeh} = **219.13kN/m** and P_{aev} = 226.96kN/**m** and further,

$$\sum V = W_1 + W_2 + W_3 + W_4 + W_5 + P_{av} + P_{aev}$$
$$= 6.25 + 56.35 + 191.94 + 225.4 + 96.6 + 181.3 + 226.96 = \mathbf{984.80kN/m}$$
$$\sum H = P_{ah} + P_{aeh} = 176.4 + 219.13 = \mathbf{395.53kN/m}$$

Next, Stability criteria for the retaining wall from static and dynamic effects of loadings shall be computed and factor of safety corresponding to overturning, sliding, bearing capacity and deep seated foundation failure shall be calculated and checked against the allowable factor of safety which is 0.75 times the allowable factor of safety for static loading conditions.[6]

i. **Stability against Overturning:**

In the computation of the overturning moments of the retaining wall about the toe, one needs to locate the location of the resultant active lateral seismic force using the procedure given for rotation about the bottom of the wall, as shown in figure B.7 below:[9]



$$\bar{H} = \frac{(P_A)\left(\frac{1}{3}H\right) + (\Delta P_{AE})(0.6H)}{P_{AE}}$$

Figure B.7: Location of Resultant Active Lateral Seismic Force

Here ΔP_{AE} and P_A act at 0.6H and H/3 respectively, from the bottom of the wall. $\Delta P_{AE} = P_{AE} - P_A$

Hence, inserting the values, the location of the resultant active lateral seismic force, $\bar{H}$ =2.63m.

$\sum M_{toe}$= W_1*(0.25/3)+W_2*(0.7/3)+W_3*((2.69/3)+1.3)+W_4*((2.8/3)+1.3)+W_5*((0.6/2)+0.7)

+P_{av}*(4.1-((H/3)/tan68))+P_{aev}(4.1-($\bar{H}$/tan68))-P_{aeh}* $\bar{H}$-P_{ah}*(H/3)

$\sum M_{toe}$= 0.521+13.148+421.628+503.393+96.6+572.413+689.37-576.31-411.60

=1,309.16kN-m/m

Next, the location of the resultant vertical and horizontal forces are;

$\sum V\overline{X} = \sum$ Moments of Vertical Forces: stabilizing moments

$$\overline{X} = \frac{2{,}297.08}{984.80} = 2.33\text{m}$$

$\sum H\overline{Y} = \sum$ Moments of Horizontal Forces: overturning moments

$$\overline{Y} = \frac{987.91}{395.53} = 2.50\text{m}$$

The location of the Resultant Force from the toe of the retaining wall;

$\omega = \tan^{-1}\frac{\sum V}{\sum H} = \tan^{-1}\frac{984.80}{395.53} = 68.12^0$

X=$\overline{X}$ - $\overline{Y}$ tan21.88=2.33-2.50*tan21.88=1.33m

Next, checking the location of the resultant force of the wall and the backfill whether it lies at the middle third of the wall base to maintain a positive contact pressure at the uphill base of the wall i.e. heel;

e=B/2-X=4.1*0.5-1.33=0.72m while B/6=4.1/6=0.68m, here as B/6 < e , the resultant lies outside the middle third or Kern. Hence, one can enlarge the base of the retaining wall by providing 0.3m depth lean concrete footing of width 5.1m. So that B/6=5.1/6=0.85m which is greater than e=0.72m and hence the resultant lies within the middle third of the base.

Finally, the Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Stabilizing Moments}}{\sum \text{Overturning Moments}}$$

$F_o = \frac{2{,}297.08}{987.91} = 2.33 > 1.5$(i.e. 2.0x 0.75).......................OK!

ii. Stability against Sliding along the base:

The Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Horizontal Resisting Forces}}{\sum \text{Horizontal Sliding Forces}}$$

Here, horizontal resistance along the base of a retaining wall is provided by friction between the base and the underlying foundation soil. Therefore, taking $\delta' = 24^0$ and neglecting the horizontal passive resistance in front of the wall, we have;

$$F_o = \frac{\sum V \tan \delta'_g}{\sum H} = \frac{984.80 * \tan 24}{395.53}$$

$F_o = 1.11 < 1.125$ (i.e 0.75x1.5)

Since the factor of safety is less than 1.125 one needs to key the base to the foundation material (weathered rock).

iii. Bearing Capacity Failure about the Toe of the Wall:

The computation of contact pressure under the base of the wall shall be made using the flexure formula. To determine the maximum and minimum contact pressures it is necessary to calculate the moment of the forces acting on the wall about the center of the base.

$$M_{toe} = \sum V\bar{X} - \sum H\,\bar{Y}$$

$$M_M = \sum V\left(\bar{X} - \frac{B}{2}\right) - \sum H\,\bar{Y}$$

$$M_M = M_{toe} - \sum V\frac{B}{2}$$

= 1,309.16-(984.80*4.1*0.5)=-709.68kN-m/m

To this end, the contact pressure under the base of the wall (B x 1) from flexure formula is given as:

$$\sigma_{toe/heel} = \frac{\sum V}{B} \pm \frac{6M_M}{B^2}$$

$$\sigma_{toe/heel} = \frac{984.80}{5.1} \pm \frac{-6*709.68}{5.1^2} = 193.10 \pm 163.71$$

$\sigma_{toe} = 356.81kN/m^2$, $\sigma_{heel} = 29.39kN/m^2$ and $\sigma_{av} = 193.10kN/m^2$

The Bearing Capacity of the foundation material shall be taken from presumptive bearing capacity values as elaborate soil investigation was not done on the weathered rock foundation material. The presumed design bearing resistance for vertical load which shall be reduced at least by 25% shall be used to account for the horizontal load component.[11]

Factor of safety against bearing capacity failure at the toe shall be given as:

$$\mathrm{F_B} = \frac{\text{Pressumed Design Bearing Resistance}}{\text{Toe contact Pressure}} = \frac{850}{356.81} = 2.38 > 1.875 \dots\dots\dots OK!$$

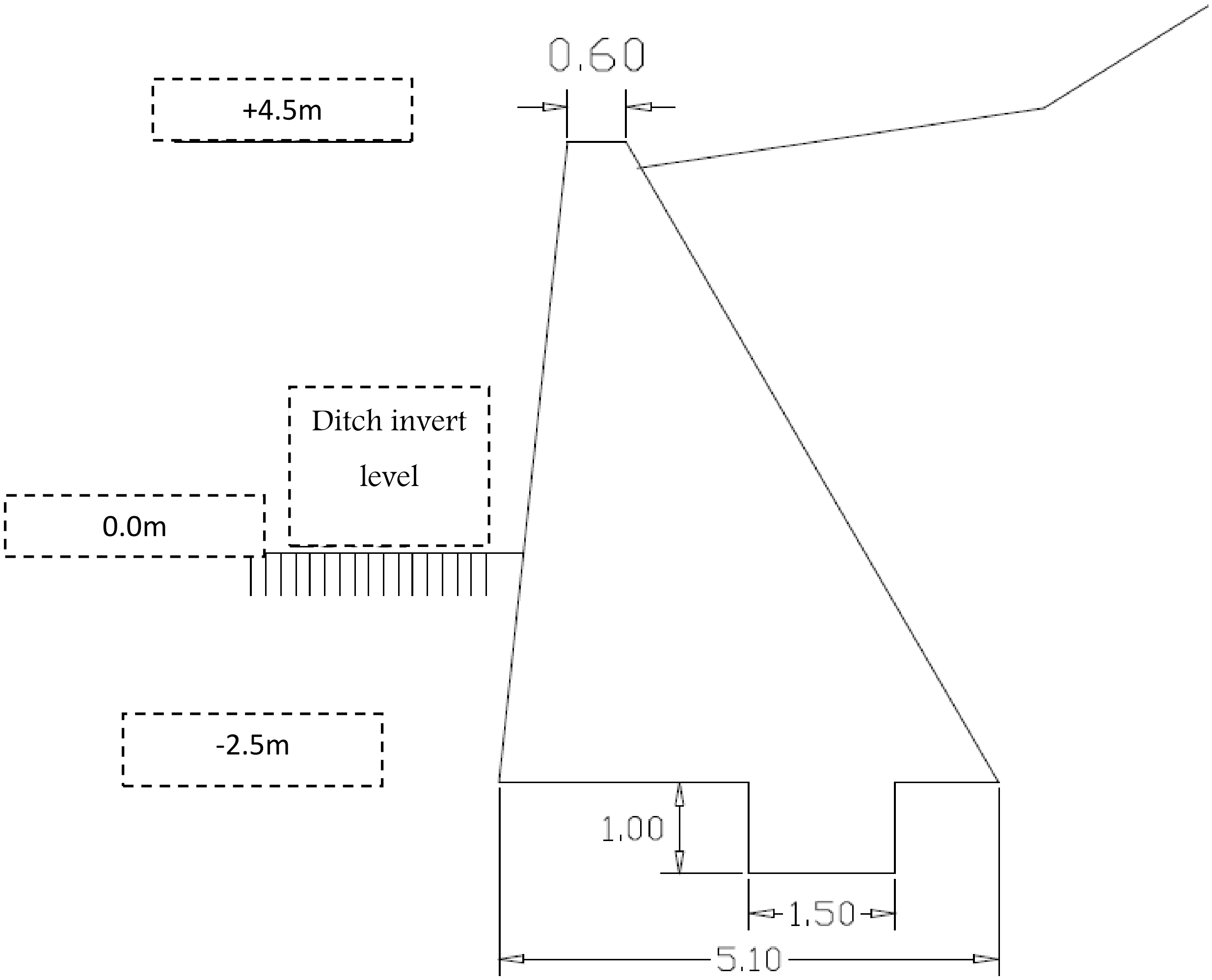


Figure B.8: Designed retaining wall cross-section with shear key

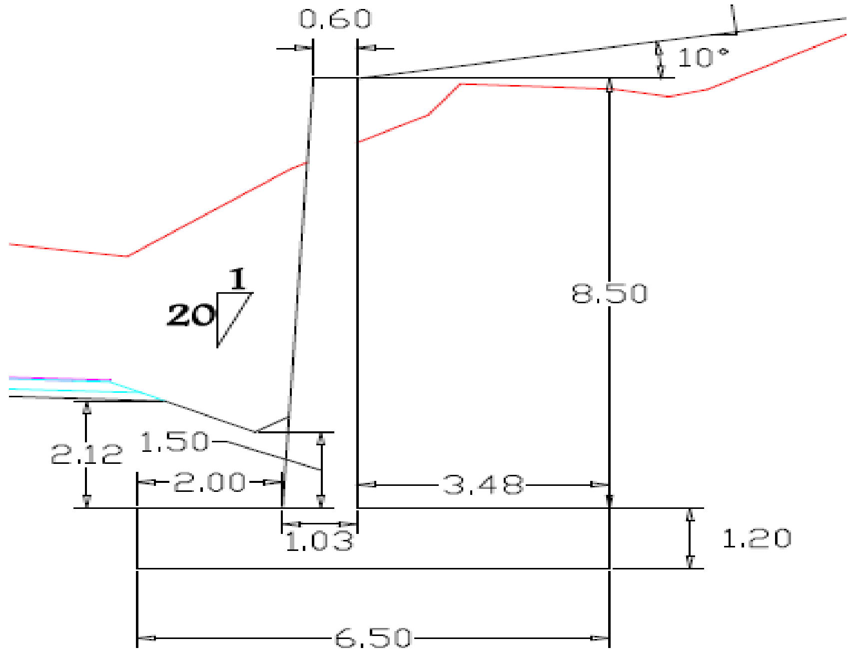
Provide weep holes diameter 150mm at 2.0m center to center horizontally and vertically to collect water from the backfill material which eventually discharge it to longitudinal drainage ditch. Furthermore, provide a flattened slope as per figure B.1 above and furrow ditch at the top of the reinstated slope to facilitate surface runoff coming from the hill side. Furthermore, as the height of the slide slope decreases when one goes to the LHS and RHS of the cross section under investigation accordingly, the height of the wall shall be reduced, during the construction of the remedial structure.

2. Sta. 68+020(ahead) to Sta.68+140(ahead)(RHS):

The common procedures for the design of retaining wall shall be as follows:

- **The geometry of the failed soil mass and tentative dimensions of the retaining wall as shown below in Figure B.9 are:**

  Total Height H=9.7m, Free Height H_{fr}=7.0m, Embedded Height H_{emb}=1.50m (from the ditch invert), Top width T= 0.6m, Battering Slope ((Front Side, V:H) 20:1 and Bottom width B=6.5m.



The structural excavation for the retaining wall should be made at least 1m wide for safety related issues. However, for the analysis of the retaining wall one assumes the backfill material i.e. directly above the heel of the wall, to be the effective back of the wall and shall be backfilled at 10degrees slope from the horizontal to create a smooth transition with the flattened slope.

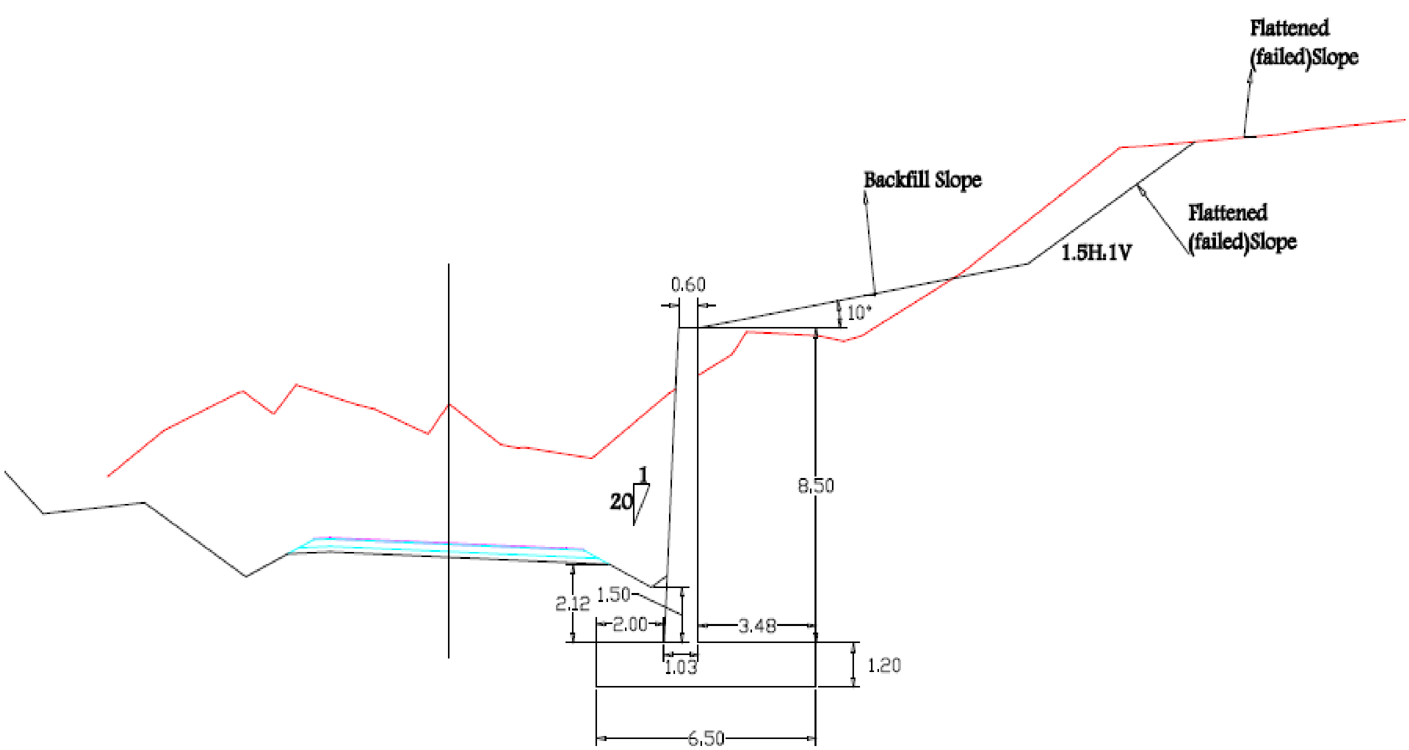
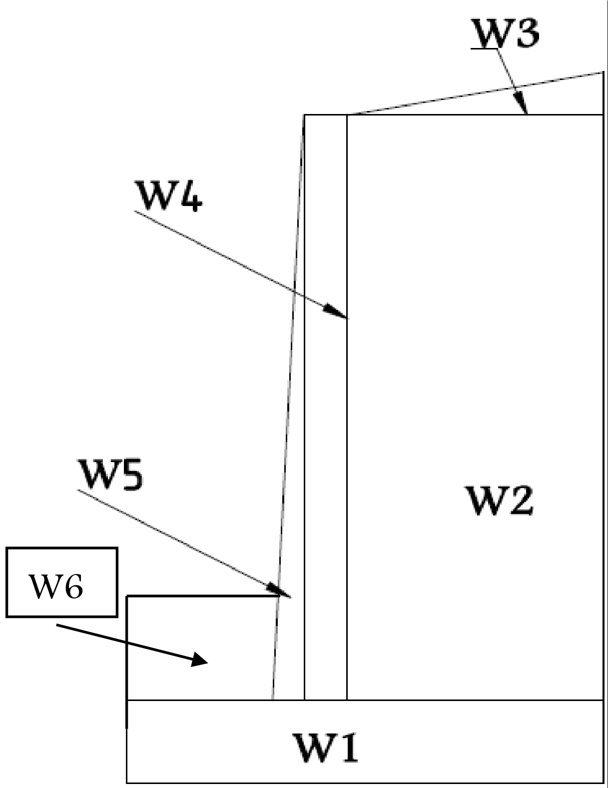


Figure B.9: Tentative dimensions of retaining wall and Flattened Slide Material

- **The computation of the Vertical Dead Loads:**

  For convenience, the acting loads on the retaining wall are segmented and tabulated hereunder.



Segmental Loads	Unit Weight (kN/m^3)	Vertical Loads(Weights) (kN/m)
W1: Concrete Wall	25	7.8*25=195
W2: Rock/granular fill	20	29.58*20=591.60
W3: Rock/granular fill	20	1.08*20=21.60
W4: Concrete Wall	25	5.1*25=127.50
W5: Concrete Wall	25	1.81*25=45.16
W6: Rock/granular fill	20	3*20=60

Figure B.10: Vertical load components on the Retaining Wall

- Calculation of lateral Pressures:

  The Coulomb's lateral earth pressure theory shall be used for the computation of the active earth pressure. For use of the forgoing, the following presumed/assumed values are made:

  - Backfill slope shall be, $\beta = 10^0$ from the horizontal and $\propto = 90^0$;
  - For ease of construction and timely retaining of the soil mass(for development of lateral pressure at rest) rock fill backfill(nearly cohesionless) shall be (hand)placed and presumed value of drained friction angle of 35^0 shall be used;
  - Wall friction angle, $\delta' = 24^0$ is assumed;

  The active Pressure, P_a , can be determined using coulomb's formula:

$$P_a = \frac{1}{2}\gamma H^2 K_a$$

Where:

γ : unit weight of the backfill material

H : the total height of the retaining wall

K_a : the active earth pressure coefficient given by :

$$K_a = \frac{\sin 2(\propto + \emptyset')}{\sin 2\alpha \sin(\alpha - \delta)\left[1 + \left(\frac{\sin(\emptyset' + \delta)\sin(\emptyset' - \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}\right)^{\frac{1}{2}}\right]^2}$$

where:

$\propto$ −batttering of the retainig wall from the horizontal, heel side

$\emptyset'$ − Effective Friction angle of the backfill material

δ − Friction angle between the masonry retaining wall and backfill material

β − backfill slope with the horizontal

Inserting the above values in the above equation, the coefficient of active lateral pressure,

K_a equals 0.275

Since the total active lateral force is inclined at an angle of , $\delta = 24$ degree to the normal drawn to the back of the wall or [δ + (90-$\propto$)] degrees from the horizontal , the horizontal component of the active coefficient,

K_{ah} equals : Cos(24)*0.275= 0.251

And similarly the vertical component of the active earth pressure coefficient:

K_{av}: equals 0.111

Therefore, the distribution of the horizontal and vertical forces from lateral earth pressure are:

$$P_a = \frac{1}{2}\gamma H^2 K_a = 0.5 * 20 * 9.7^2 * 0.275 = \mathbf{258.75kN/m}$$

P_{ah} = **236.38KN/M** and P_{av} = 105.24kN/**m**

The Summation of Vertical and Horizontal Forces:

$$\sum V = W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + P_{av}$$
$$= 195 + 591.6 + 21.6 + 127.5 + 45.16 + 60 + 105.24 = \mathbf{1146.1kN/m}$$

$$\sum H = P_{ah} = \mathbf{236.38kN/m}$$

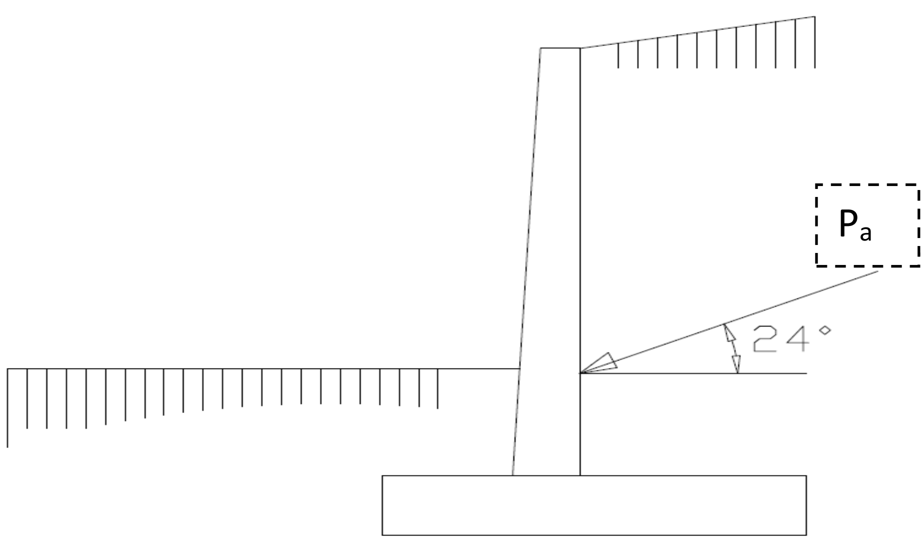


Figure B.11: Active lateral earth pressure

Static Stability Criteria's of the Retaining Wall

i. **Stability against Overturning:**

Taking moment about the Toe of the retaining wall and considering clockwise direction as positive:

$\sum M_{toe}$= W_1*(6.5/3)+W_2*(3.48/2+3.03)+W_3*((2*3.48/3)+3.03)+W_4*((0.6/2)+2.43)+W_5*((2*0.43/3)+2) W6*(2/2) +P_{av}*3.03-P_{ah}*(9.7/3)

$\sum M_{toe}$= 633.75+2821.93+115.56+348.08+150.53+60+318.88-764.30=3,624.43KN-M/M

Next, the location of the resultant vertical and horizontal forces are;

$\sum V\bar{X} = \sum$ Moments of Vertical Forces: Stabilizing moments

$$\bar{X} = \frac{4388.73}{1146.1} = 3.83\text{m}$$

$\sum H\bar{Y} = \sum$ Moments of Horizontal Forces: Overturning moments

$$\bar{Y} = \frac{764.30}{236.38} = 3.23\text{m}$$

The location of the Resultant Force from the toe of the retaining wall;

$\omega = \tan^{-1}\frac{\sum V}{\sum H} = \tan^{-1}\frac{1146.1}{236.38} = 78.35^0$

X=$\bar{X}$ - $\bar{Y}$ tan11.65=3.83-3.23*tan11.65=3.16m

Next, checking the location of the resultant force of the wall and the backfill whether it lies at the middle third of the wall base to maintain a positive at the uphill base of the wall i.e. heel;

e=6.5/2-X=6.5*0.5-3.16=0.09m, while B/6=6.5/6=1.08m, here as B/6 >>e , the resultant lies within the middle third or Kern , as shown in figure B.12 below

Finally, the Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Stabilizing Moments}}{\sum \text{Overturning Moments}}$$

$F_o = \frac{4388.73}{764.3} = 5.74 > 2.0$........................OK!

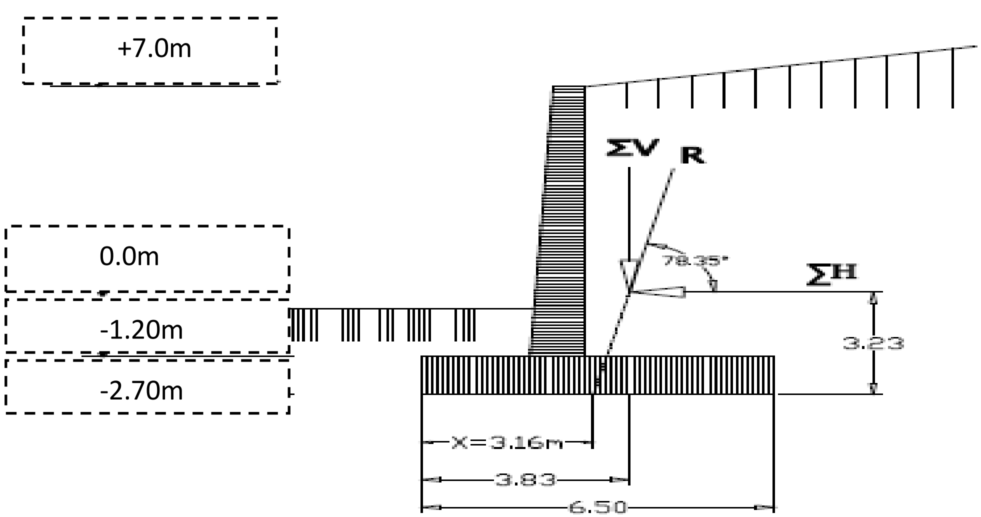


Figure B.12: Resultant force location

ii. **Stability against Sliding along the base:**

The Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Horizontal Resisting Forces}}{\sum \text{Horizontal Sliding Forces}}$$

Here, horizontal resistance along the base of a retaining wall is provided by friction between the base and the underlying foundation soil. The friction between the concrete base and soil is likely to be less than the shear strength of cohesion less soil in the foundation. Generally, using following values for base/soil friction angle δ':[6]

- $\delta' = 30^0$ for coarse -grained soil containing no silt and clay
- $\delta' = 24^0$ for coarse -grained soil containing silt
- $\delta' = 20^0$ for thin layer of granular fill overlying cohesive soils

Therefore, taking $\delta' = 24^0$ and neglecting the horizontal passive resistance in front of the wall, we have;

$$F_o = \frac{\sum V \tan \delta'_g}{\sum H} = \frac{1146.1 * \tan 24}{236.38}$$

$F_o = 2.16 > 1.5$ ………………… OK!

iii. **Bearing Capacity Failure about the Toe of the Wall:**

The computation of contact pressure under the base of the wall shall be made using the flexure formula. To determine the maximum and minimum contact pressures it is necessary to calculate the moment of the forces acting on the wall about the center of the base.

$$M_{toe} = \sum V\bar{X} - \sum H\,\bar{Y}$$

$$M_M = \sum V\left(\bar{X} - \frac{B}{2}\right) - \sum H\,\bar{Y}$$

$$M_M = M_{toe} - \sum V\frac{B}{2}$$

= 3,624.43-(1146.1*6.5*0.5)=-100.40kN-m/m

To this end, the contact pressure under the base of the wall (B x 1) from flexure formula is given as:

$$\sigma_{toe/heel} = \frac{\sum V}{B} \pm \frac{6M_M}{B^2}$$

$$\sigma_{toe/heel} = \frac{1146.1}{6.5} \pm \frac{-6*100.40}{6.5^2} = 176.32 \pm 14.26$$

$\sigma_{toe} = 190.58kN/m^2$, $\sigma_{heel} = 162.06kN/m^2$ and $\sigma_{av} = 176.32kN/m^2$

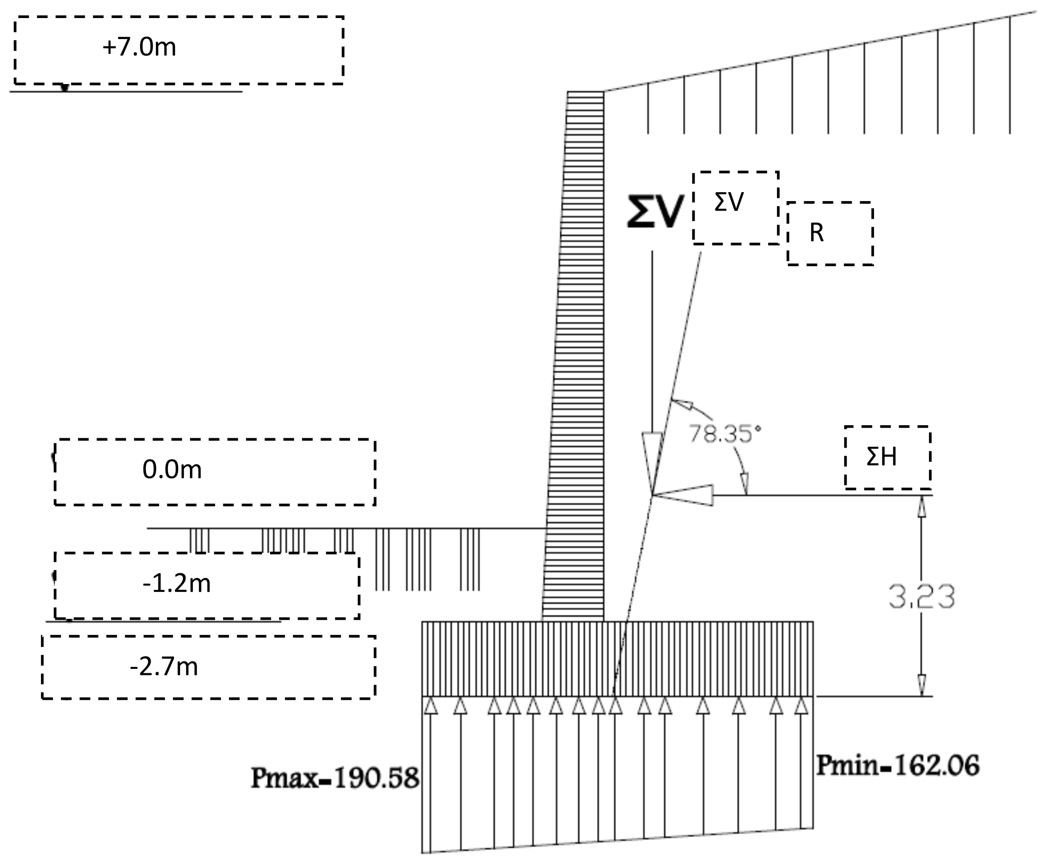


Figure B.13: Contact pressure distribution under the base of the wall

The Bearing Capacity of the foundation material shall be calculated from undrained loading conditions laboratory results to simulate initial loading conditions. Accordingly, ϕ_u=0 and C_u=201kN/m^2 , γ_t= 15.06kN/m^3, B=6.5m. e=0.09m and B'=6.5-0.09*2=6.32m, D_f=2.70m.

According to DIN 4017: the bearing capacity for initial loading conditions shall be given by:

$$q_{ult} = C_u N_c i_c S_c' + \gamma_1 D_f N_q i_q s_q'$$

Shape factors, S_c' *and* S_q' =1(for continuous footing) and Bearing capacity coefficients, N_c=5.0 and N_q=1.0

Inclination Factors, i_q=1, while

$i_c = 0.5 + 0.5\sqrt{1 - \frac{H_f}{A'C_u}}$, inserting the values stated below, i_c =0.896

H_f(Critical Horizontal load)=Fs*H=2*236.38=472.76KN/M and A'=6.32m^2.

$q_{ult} = 201 * 5 * 0.896 * 1 + 15.06 * 2.7 * 1 * 1 * 1 = 941.14$ kN/m^2

Factor of safety against bearing capacity failure (short term stability) at the toe shall be given as:

$$F_B = \frac{qult}{\text{Toe contact Pressure}} = \frac{941.14}{190.58} = 4.94 > 2.5\ OK$$

iv. **Deep Seated foundation failure Stability**

Overall stability of the wall system within the context of slope stability must also be assessed to ensure that no failure occurs either in the backfill or the native soil. As such, a separate analysis for slope stability must be performed on the zone in the vicinity of the wall using conventional limit equilibrium slope stability methods. The failure of the retaining wall is usually a result of the undercutting of the retaining wall, not shearing of the wall itself. For this reasons, the strength of the retaining wall itself becomes trivial, but the weight of the wall acting as a stabilizing force is critical.

Accordingly, the model of slide mass and designed remedial structure is prepared using site observations and back analysis results and the same is analyzed in GEOSLOPE V.7 software [10] by simulating the conditions aforementioned above. Reference is made to Fig B.13(a)

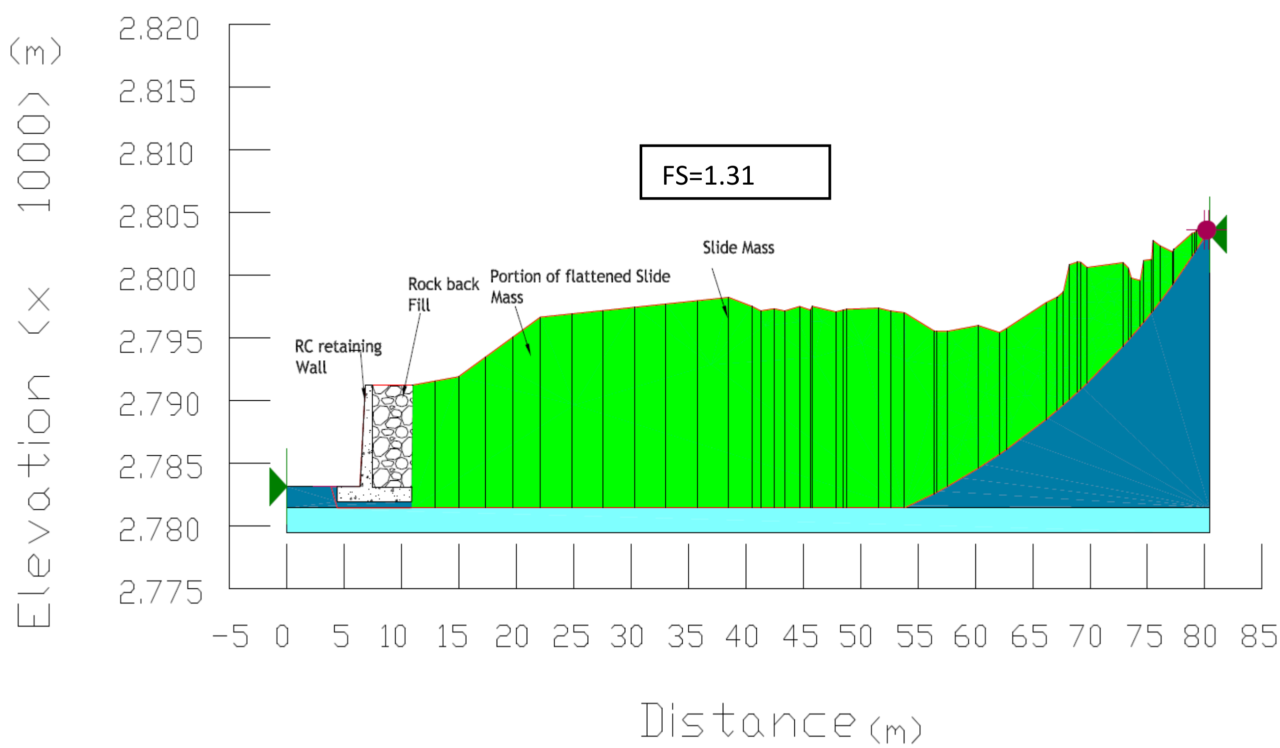


Figure B.13 (a): Designed remedial structure and slide mass FOS: static stability

Dynamic Stability and Static Criteria's of the Retaining Wall

The study area resides between Arbaminch and Sawula (Felege Neway) towns where these towns are located within Zone 4 and Zone 3 respectively of the seismic hazard zones.[8] In view of the foregoing, design bedrock acceleration of gravity of (Horizontal) 0.1 and 0.07 for Arbaminch and Sawula Towns respectively. To this end, the horizontal and vertical component of earthquake acceleration coefficient of the study area is selected to be 0.1 and 0 respectively.[9]

Therefore, using the procedures of Mononobe Okabe lateral earth pressure equations for seismic forces one can calculate the active seismic force acting on the retaining wall accordingly while reference is made on Figure B.9 above;

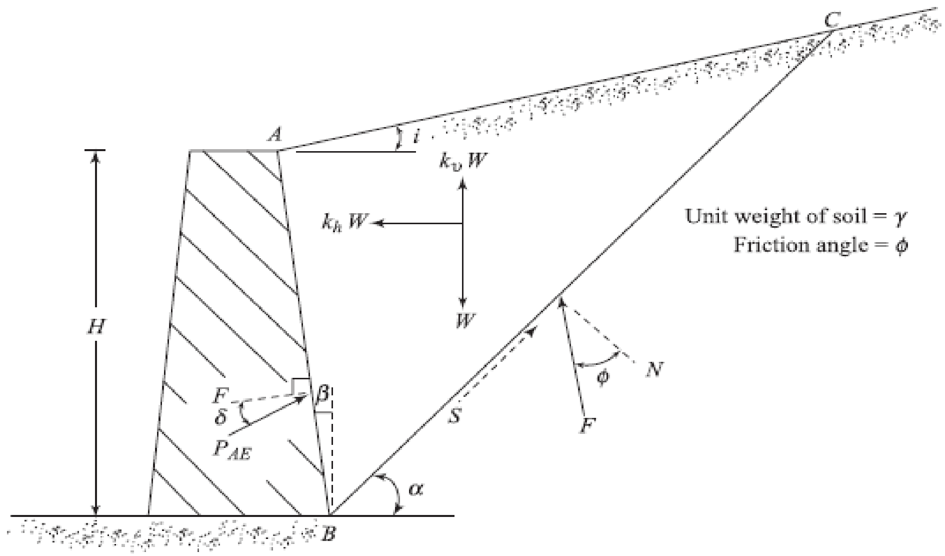


Figure B.14: Derivations of Mononobe Okabe seismic induced lateral active earth pressure equations

$$P_{AE} = \frac{1}{2}\gamma H^2(1-k_v)K_{AE}$$

Or after simplification and to keep similarity with lateral pressure equation (Static)

$$P_{AE} = P_A(i', \beta')(1-k_v)\,(\overset{*}{p})$$

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]^2}$$

$$\theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$$

Where:

$$i = 10^0, \beta = 0^0, \delta' = 24^0, \emptyset' = 35 \text{ and } \gamma = 20\,KN/M^3$$

Calculate the seismic inertia angle, $\theta = \tan^{-1}\frac{K_h}{1-K_h} = \tan^{-1}\left(\frac{0.1}{1-0}\right) = 5.71^0$

Calculate, $i' = i + \theta = 10 + 5.71 = 15.71^0$

Calculate, $\beta' = \beta + \theta = 0 + 5.71 = 5.71^0$

Calculate, the coefficient of active lateral earth pressure, K_A(i', β'), using following equation and inserting the values:

$$K_A(i',\beta') = \frac{\cos^2(\phi-\beta')}{\cos^2\beta'\cos(\delta+\beta')\left[1+\left\{\frac{\sin(\delta+\phi)\sin(\phi-i')}{\cos(\delta+\beta')\cos(\beta'-i')}\right\}^{1/2}\right]^2}$$

$K_A(i', \beta')$=0.356

Calculate, $P_a = \frac{1}{2}\gamma H^2 K_A$ =0.5*20*9.7²*0.356=334.96kN/m and

Calculate, P*, using the following equation

$$\overset{*}{p} = \left(\frac{\cos^2\beta'}{\cos\theta\cos^2\beta}\right)$$

p*=0.995 and hence, P_{AE} equals 333.29kN/m

While, the vertical and horizontal components of seismic lateral active pressures similar to the static active lateral pressure case, P_{aeh} = **304.47kN/m** and P_{aev} = 135.56kN/**m** and further,

$$\sum V = W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + P_{av} + P_{aev}$$

$$= 195 + 591.6 + 21.6 + 127.5 + 45.16 + 60 + 105.24 + 135.56 = \mathbf{1281.66kN/m}$$

$$\sum H = P_{ah} + P_{aeh} = 304.47 + 236.38 = \mathbf{540.85kN/m}$$

Next, Stability criteria for the retaining wall from static and dynamic effects of loadings shall be computed and factor of safety corresponding to overturning, sliding, bearing capacity and deep seated foundation failure shall be calculated and checked against the allowable factor of safety which is 0.75times the allowable factor of safety for static loading conditions.[6]

i. **Stability against Overturning:**

In the computation of the overturning moments of the retaining wall about the toe, one needs to locate the location of the resultant active lateral seismic force using the procedure given by Seed and Whitman (1969) for rotation about the bottom of the wall, reference is made to Exhibit 5.7 above.

$$\overline{H} = \frac{(P_A)\left(\frac{1}{3}H\right) + (\Delta P_{AE})(0.6H)}{P_{AE}} \qquad \Delta P_{AE} = P_{AE} - P_A$$

Here ΔP_{AE} and P_A act at 0.6H and H/3 respectively, from the bottom of the wall.

Hence, inserting the values, the location of the resultant active lateral seismic force, $\overline{H}$ =3.822m.

$\sum M_{toe}$= W_1*(6.5/3)+W_2*(3.48/2+3.03)+W_3*((2*3.48/3)+3.03)+W_4*((0.6/2)+2.43)+W_5*((2*0.43/3)+2)

W6*(2/2) +P_{av}*3.03+P_{aev}*3.03−P_{ah}*(9.7/3)−P_{aeh}* $\overline{H}$

$\sum M_{toe}$= 633.75+2821.93+115.56+348.08+150.53+60+318.88+410.75−764.30−1163.68

=2931.50kN-m/m

Next, the location of the resultant vertical and horizontal forces are;

$\sum V\overline{X} = \sum$ Moments of Vertical Forces: Stabilizing moments

$$\overline{X} = \frac{4859.48}{1281.66} = 3.79m$$

$\sum H\overline{Y} = \sum$ Moments of Horizontal Forces: Overturning moments

$$\overline{Y} = \frac{1927.98}{540.85} = 3.57m$$

The location of the Resultant Force from the toe of the retaining wall;

$\omega = \tan^{-1}\frac{\sum V}{\sum H} = \tan^{-1}\frac{1281.66}{540.85} = 67.12^0$

X=$\overline{X}$ - $\overline{Y}$ tan21.88=3.79-3.57*tan22.88=2.28m

Next, checking the location of the resultant force of the wall and the backfill whether it lies at the middle third of the wall base to maintain a positive contact pressure at the uphill base of the wall i.e. heel;

e=B/2-X=6.5*0.5-2.28=0.97m while B/6=6.5/6=1.08m, here as B/6 > e , the resultant lies within the middle third or Kern.

Finally, the Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Stabilizing Moments}}{\sum \text{Overturning Moments}}$$

$F_o = \frac{4859.48}{1927.98} = 2.52 > 1.5$(i.e. 2.0x 0.75)......................OK!

ii. Stability against Sliding along the base:

The Factor of Safety against sliding is given as:

$$F_o = \frac{\sum \text{Horizontal Resisting Forces}}{\sum \text{Horizontal Sliding Forces}}$$

Here, horizontal resistance along the base of a retaining wall is provided by friction between the base and the underlying foundation soil. Therefore, taking $\delta' = 24^0$ and neglecting the horizontal passive resistance in front of the wall, we have;

$$F_o = \frac{\sum V \tan \delta'_g}{\sum H} = \frac{1281.66 * \tan 24}{540.85}$$

$F_o = 1.06 < 1.125$ (i.e 0.75x1.5)

Since the factor of safety is less than 1.125 one needs to key the base to the foundation soil.

iii. Bearing Capacity Failure about the Toe of the Wall:

The computation of contact pressure under the base of the wall shall be made using the flexure formula. To determine the maximum and minimum contact pressures it is necessary to calculate the moment of the forces acting on the wall about the center of the base.

$M_{toe} = \sum V\bar{X} - \sum H\,\bar{Y}$

$M_M = \sum V\left(\bar{X} - \frac{B}{2}\right) - \sum H\,\bar{Y}$

$M_M = M_{toe} - \sum V\frac{B}{2}$

= 2931.5-(1281.66*6.5*0.5)=-1233.90kN-m/m

To this end, the contact pressure under the base of the wall (B x 1) from flexure formula is given as:

$$\sigma_{toe/heel} = \frac{\sum V}{B} \pm \frac{6M_M}{B^2}$$

$\sigma_{toe/heel} = \frac{1281.66}{6.5} \pm \frac{-6*1233.90}{6.5^2} = 197.18\ \pm 175.23$

$\sigma_{toe} = 372.41kN/m^2$, $\sigma_{heel} = 21.95kN/m^2$ and $\sigma_{av} = 197.18kN/m^2$

The ultimate Bearing Capacity of the foundation material under undrained loading conditions shall be given as:

$$q_{ult} = 5.14C_u\left[1 + 0.9146\left(\frac{B}{L}\right)\right]\left[1 + 0.4\left(\frac{D_f}{B}\right)\right] + q :\ for\ \frac{D_f}{B} \le 1$$

In view of the foregoing equation, the undrained cohesion of the material under dynamic loading conditions can be approximated by the $\left[\frac{C_{u(dyn)}}{C_{u(Sta)}}\right]$=1.5 as per the tests carried on Buckshot Clay by Carroll (1963) [7]

Hence, C_u=1.5*201=301.5KN/M^2, γ_t= 15.06KN/M^3, B=6.5m. e=0.97m and B'=6.5-0.97*2=4.56m, D_f=2.70m.

$$q_{ult} = 5.14 * 301.5[1 + 0.9146\ (4.56)]\left[1 + 0.4\left(\frac{2.7}{4.56}\right)\right] + 15.06 * 2.7 = 9951.35\,KN/M^2$$

Factor of safety against bearing capacity failure at the toe shall be given as:

$$\mathrm{F_B} = \frac{\text{Pressumed Design Bearing Resistance}}{\text{Toe contact Pressure}} = \frac{9951.35}{372.41} = 26.72 >> 1.875 \ldots\ldots\ldots..OK!$$

Structural Design of the Cantilever Retaining Wall

The tentative dimensions of the retaining wall satisfy both static and dynamic stability criteria and hence structural design of the retaining wall shall be carried out according to the Ultimate Limit State Design Method.

Grade of Concrete is C-30 and reinforcing steel is f_{yk}=300mPa body forces both from dynamic and static stability criteria shall be used.

A. **Design of the Stem**

In the design of the stem the body forces acting of the wall are the lateral earth pressure acting from seismic and static forces.

Moment at A:

M_A=0.5*4.25*48.84*(4.25/3)=147.04kNm/m

Design Moment at A, M=1.3*147.04=191.15kNm/m

Reinforcement bar at Level A:

$A_s = \rho bd$

where b=1.0m and d=0.7625, rebar cover is 5cm and $\rho = geometrical\ ratio\ of\ reinforcement$

$$\rho = \frac{1-\left(1-\frac{2M}{f_{cd}bd^2}\right)^{1/2}}{\frac{f_{yd}}{f_{cd}}}$$

$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{300}{1.15} = 260.87$MPa and $f_{cd} = \frac{0.85*f_{ck}}{\gamma_c} = \frac{0.85*(\frac{30}{1.25})}{1.5} = 13.6$MPa

$$\rho = \frac{1-\left(1-\frac{2M}{f_{cd}bd^2}\right)^{1/2}}{\frac{f_{yd}}{f_{cd}}} = \frac{1-\left(1-\frac{2*191.15}{13.6*1*1000*0.7625^2}\right)^{1/2}}{\frac{260.87}{13.6}} = 0.0013$$

$A_s = \rho bd = 0.0013*100*76.25 = 9.15\,cm^2/m$ =6 ϕ14mm @20cm(C/C)/m

Moment at B:

M_B=0.5*8.5*97.69*(8. 5/3)=1176.35kNm/m

Design Moment at A, M=1.3*1176.35=1529.26kNm/m

Reinforcement bar at Level B:

$A_s = \rho bd$

where b=1.0m and d=0.98, rebar cover is 5cm and $\rho = geometrical\ ratio\ of\ reinforcement$

$$\rho = \frac{1-\left(1-\frac{2M}{f_{cd}bd^2}\right)^{1/2}}{\frac{f_{yd}}{f_{cd}}}$$

$$\rho = \frac{1-\left(1-\frac{2*1176.35}{13.6*1*1000*0.98^2}\right)^{1/2}}{\frac{260.87}{13.6}} = 0.0049$$

$A_s = \rho bd = 0.0049*100*98 = 48.02\,cm^2/m$ =11 ϕ 24mm @10cm (C/C)/m

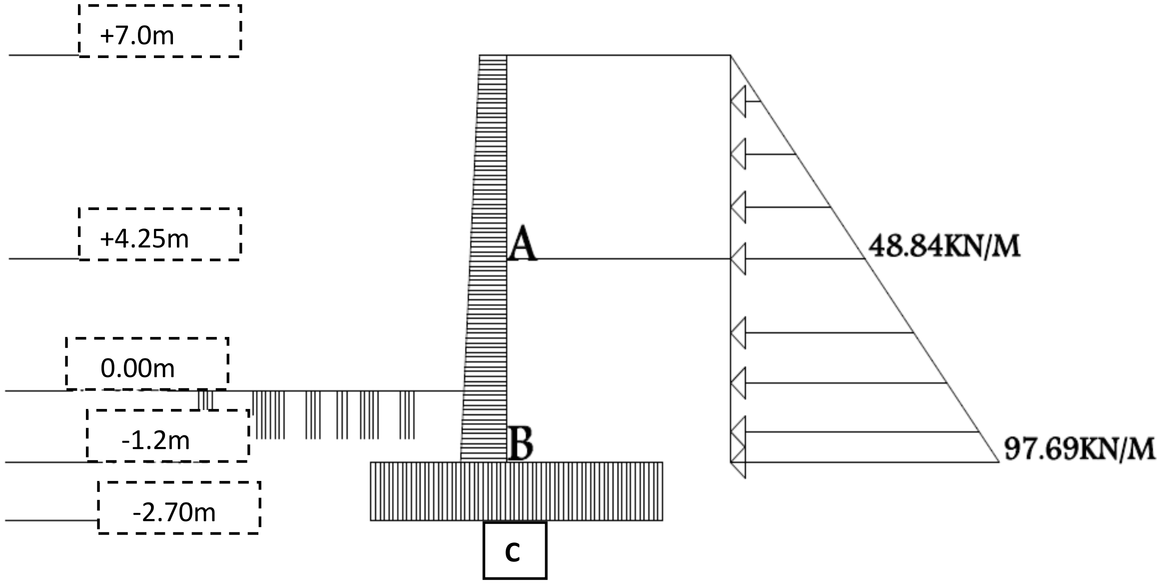


Figure B.15: Design of the stem

B. Design of the Heel

For the design of the heel, the weight of the soil resting on the heel including the vertical component of the lateral forces from static and dynamic effects, reaction from the base and weight of the heel are considered. It is to be noted here that the vertical component of the lateral earth pressure will not create a moment for the design of the heel through point C.

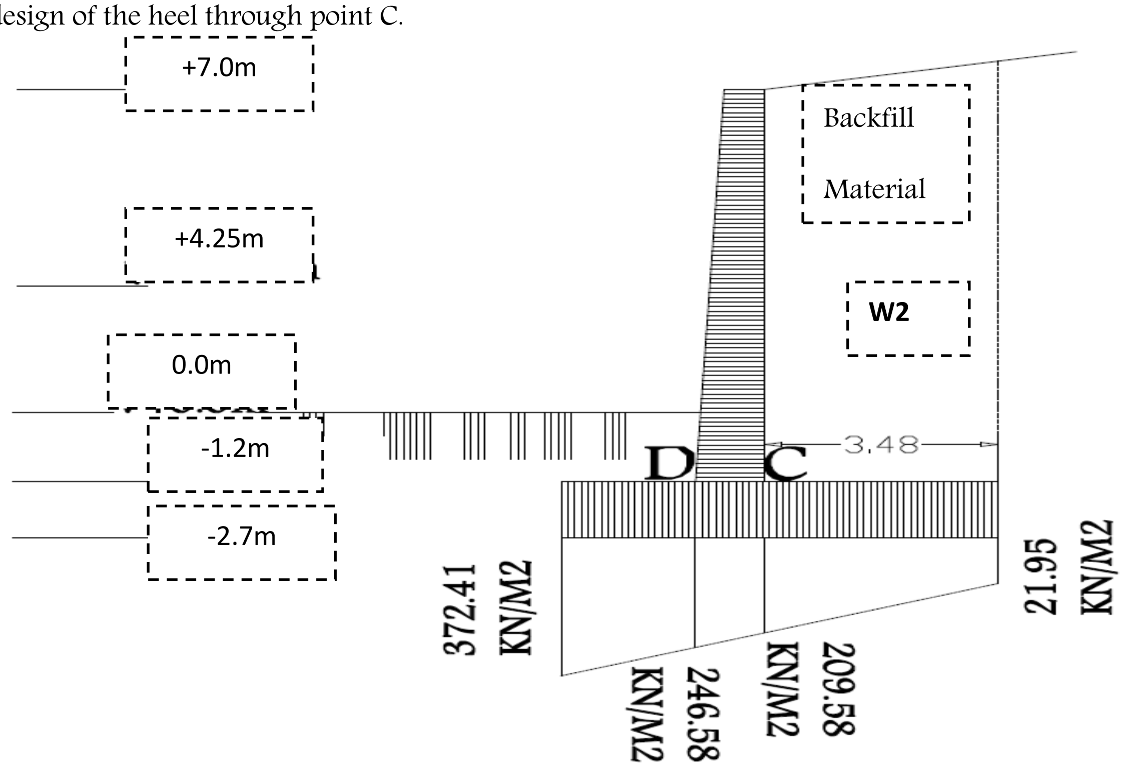


Figure B.16: Design of the toe and the heel

Moment At C:

M_{C}= –[W_2*(3.48/2)+W_3*(2*3.48/3)+W_{01}*(3.48/2)]•**1.6**+[3.48*21.95*(3.18/2)+0.5*187.63*3.48*(3.48/3)]***1.3**

=–591.60*(3.48/2)+21.6*(2*3.48/3)+104.4*(3.48/2)]•**1.6**+**1.3***[3.48*21.95*(3.18/2)+0.5*187.63*3.48*(3.48/3)]

M_{C}=–1261.15*1.6+500.17*1.3=–1367.62kNm/m

Reinforcement bar at Level C: shall be placed at the top, with 5cm concrete cover.

$A_s = \rho bd$, where b=1.0m and d=1.15m and the geometrical ration of reinforcement, ρ, shall be calculated from:

$$\rho = \frac{1-\left(1-\frac{2M}{f_{cd}bd^2}\right)^{1/2}}{\frac{f_{yd}}{f_{cd}}} = \frac{1-\left(1-\frac{2*1367.62}{13.6*1000*1.0*1.15^2}\right)^{1/2}}{\frac{260.87}{13.6}} = 0.00413$$

$A_s = \rho bd = 0.00413 * 100 * 115 = 47.47\, cm^2/m$ =11 ϕ 24mm @10cm (C/C)/m

C. **Design of the Toe:**

M_D=[–25*1.2*2*(2/2)]*1.6+[246.58*2*(2/2)+125.83*2*0.5*2*(2/3)]*1.3=763.21

$A_s = \rho bd$, where b=1.0m and d=1.15m

$$\rho = \frac{1-\left(1-\frac{2M}{f_{cd}bd^2}\right)^{1/2}}{\frac{f_{yd}}{f_{cd}}} = \frac{1-\left(1-\frac{2*763.21}{13.6*1000*1.0*1.15^2}\right)^{1/2}}{\frac{260.87}{13.6}} = 0.00226$$

$A_s = \rho bd = 0.00226 * 100 * 115 = 25.99\, cm^2/m$ =8 ϕ 20mm @14cm (C/C)/m

(place the rebar at the top).

D. **Checking for Shear Stress:**

Stem:

The horizontal forces acting the stem: $\sum H = 0.5 * 97.69 * 8.5 = 415.18kN/m$

Design Concrete Resistance: 1.3*$\sum H = 1.3 * 415.18 = 539.74kN/m$

Concrete Resistance (punching shear resistance); $V_{Rdl} = 0.25 * f_{ctd} * k_1 * k_2 * u * d$ and ; $k_1 = 1 + 50\rho$ and $k_2 = 1.6 - d \geq 1.0$, (d in meters)

Using ρ =0.0049, $k_1 = 1 + 50 * 0.0049 = 1.245$ and taking d=1.03m, $k_2 = 1.6 - 1.3 = 0.3$, since $k_2 \geq 1$, one takes $k_2 = 1.0$ and design tensile strength of concrete, $f_{ctd} = \frac{0.35*\sqrt{f_{ck}}}{\gamma_c} = \frac{0.35*\sqrt{30/1.25}}{1.5} =$ 1.52

$V_{Rdl} = 0.25 * f_{ctd} * k_1 * k_2\, u * d = 0.25 * 1.52 * 1.245 * 1 * 1.03$ =0.487MN/M=487KN/M

since $\sum H > V_{Rdl}$, increase the width of the d to 1.15m so that, $d = \frac{\sum H}{V_{Rdl}} = \frac{539.74}{487} = 1.15$

$V_{Rdl} = \frac{1.15}{1.03} * 487 = 543.74$kN/m ………….OK!

Toe:

Using ρ =0.00226, $k_1 = 1 + 50 * 0.00226 = 1.113$ and taking d=1.2m, $k_2 = 1.6 - 1.2 = 0.4$, since $k_2 \geq 1$, one takes $k_2 = 1.0$ and design tensile strength of concrete, $f_{ctd} = 1.52$

$V_{Rdl} = 0.25 * f_{ctd} * k_1 * k_2\, u * d = 0.25 * 1.52 * 1.113 * 1 * 1.2$ =0.508mN/m=508kN/m

The vertical forces acting against the contact pressure should be reduced from $\sum V$

Hence:

V_{act}= Contact Pressure-weight of footing-Overburden Soil

V_{act}=(246.58*2+0.5*125.83*2)-25*1.2*2-1.5*15.06*2=513.81kN/m

$\sum V = 1.3 * V_{act}$ =667.95kN/m, here since the imposed force is still greater than the shear stress of the concrete, V_{Rdl} one should increase d to:

=1.2*(667.9/508)=1.6m.

The design dimensions and the provisions of reinforcements are provided in Figure B.16 below and one needs to provide weep holes of diameter 50mm sloped at 10% and placed at 1.5m C/C horizontal and vertical spacing and hence flow shall be collected by longitudinal drainage ditch.

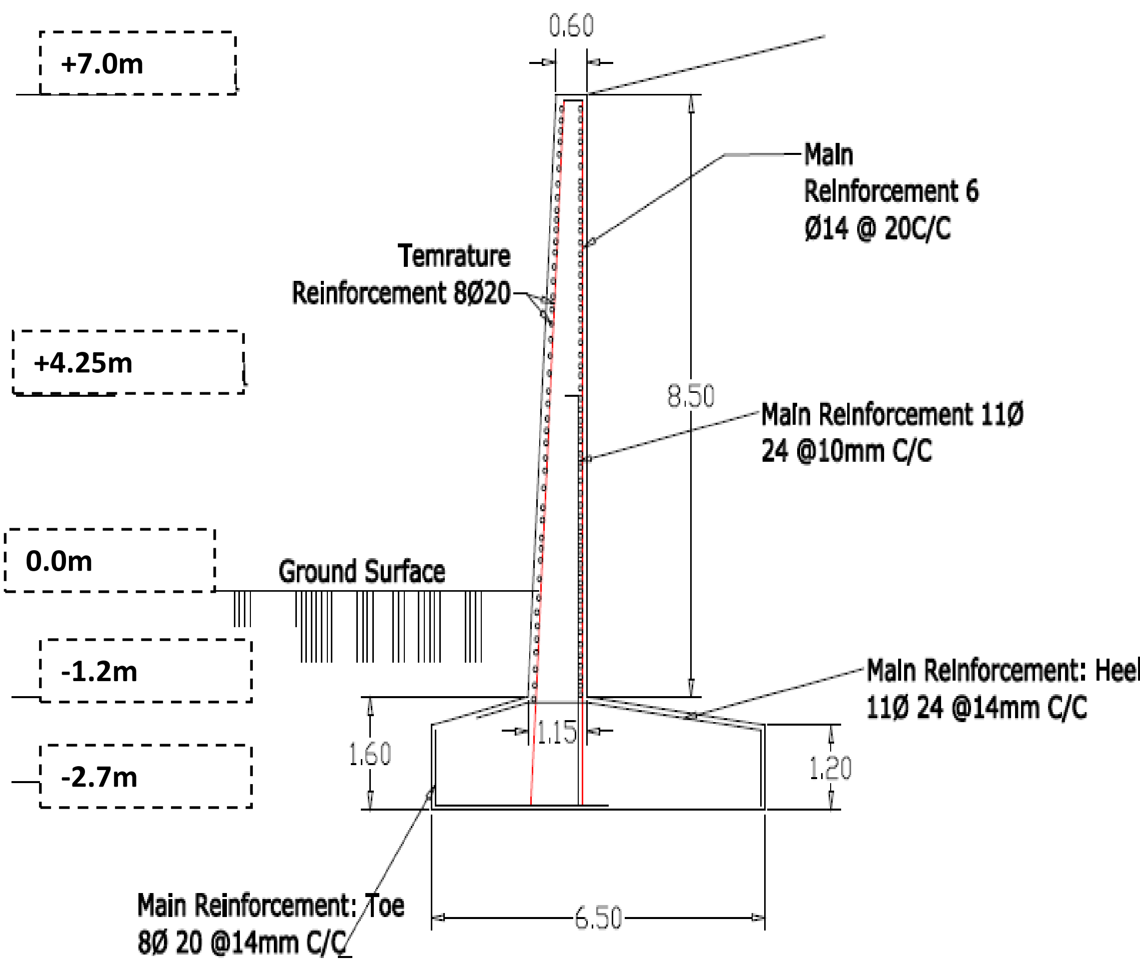


Figure B.17: Structural design of the wall and reinforcement bar provisions

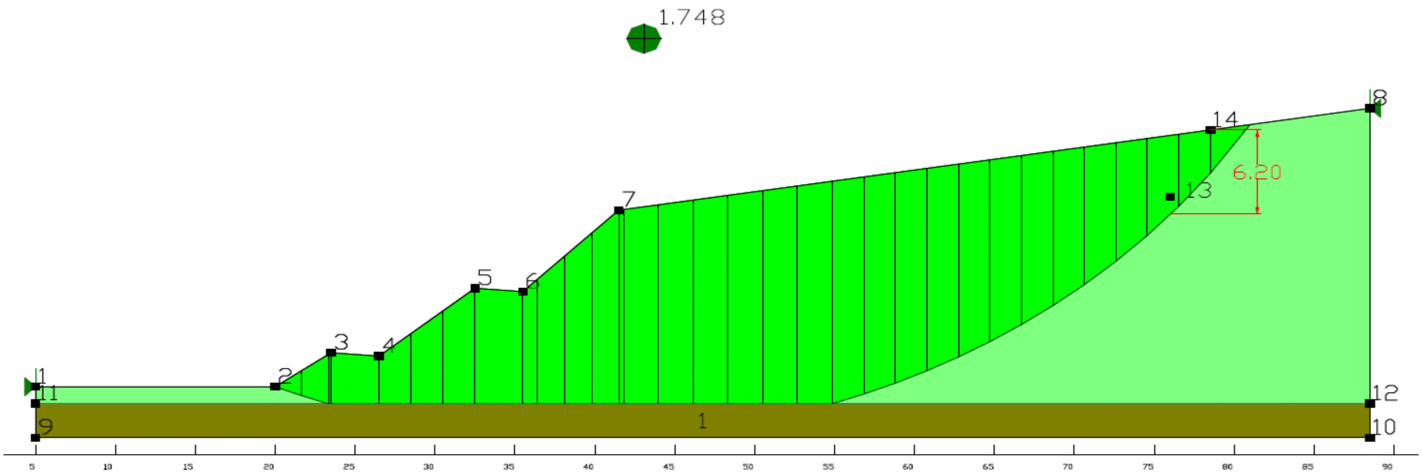


Figure C.1: Slope analysis for c=40kPa and ϕ=5^0, depth of slip surface=6.20m and Factor of Safety= 1.748

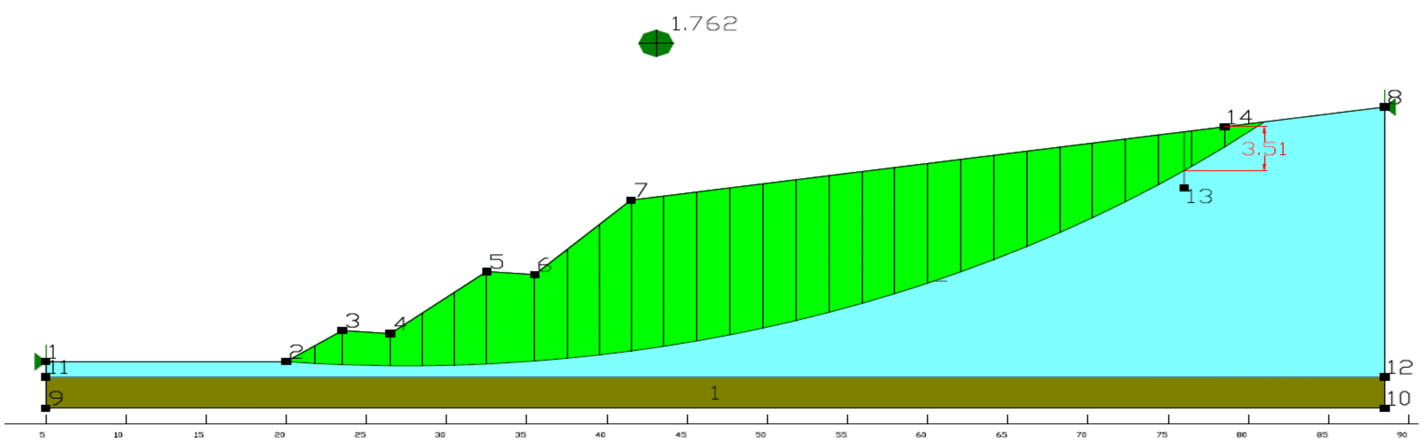


Figure C.2: slope analysis for c=30kPa and ϕ=10^0, depth of slip surface=3.51m and Factor of Safety= 1.762

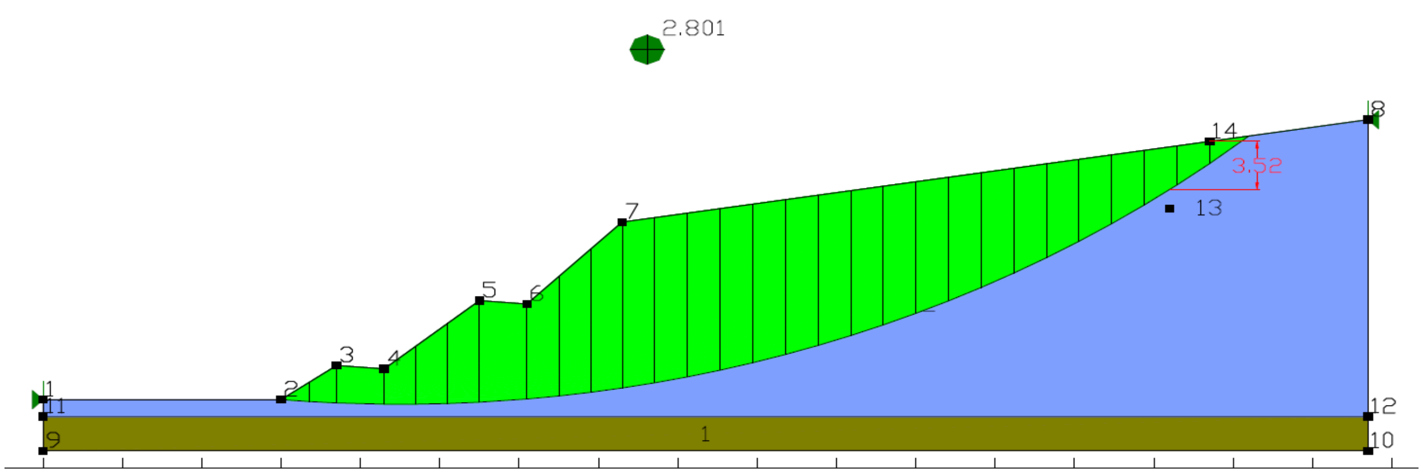


Figure C.3: slope analysis for c=40kPa and ϕ =20^0, depth of slip surface=3.52m and Factor of Safety= 2.801

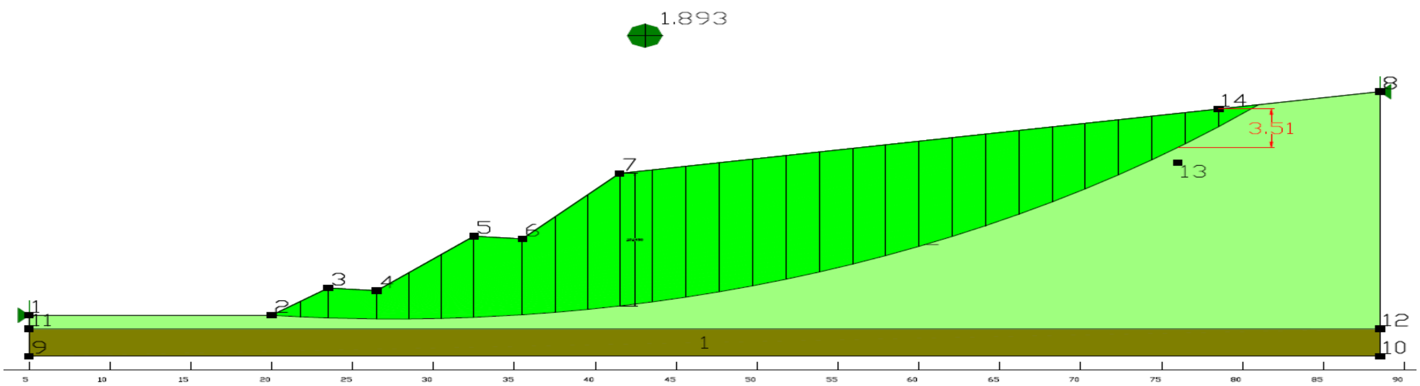


Figure C.4: Slope analysis for c=25kPa and ϕ =15^0, depth of slip surface=3.51m and Factor of Safety= 1.893

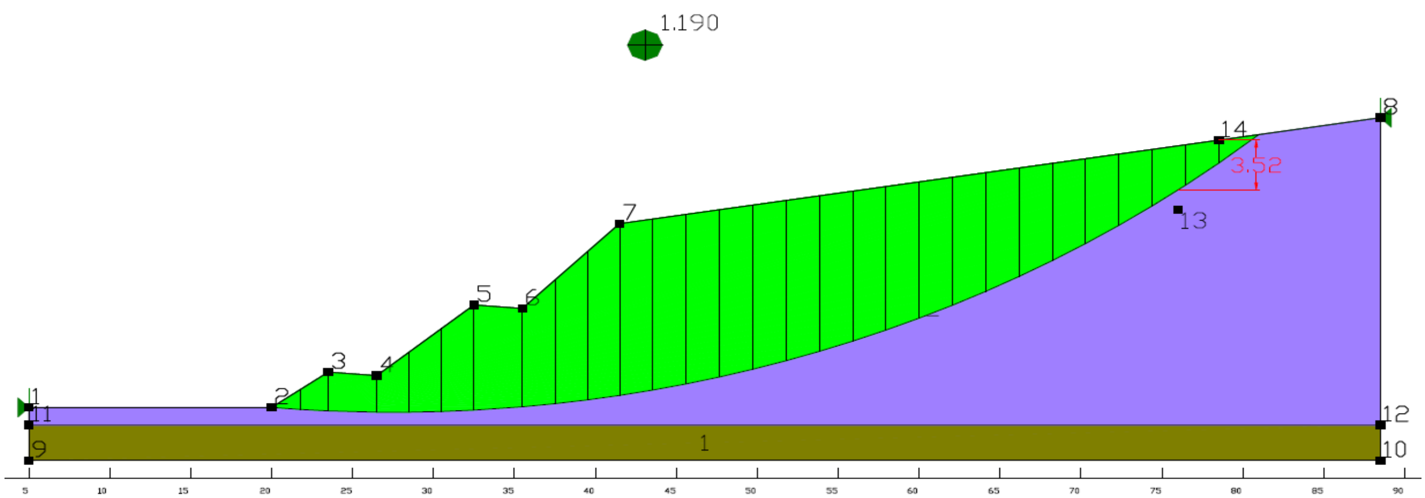


Figure C.5: Slope analysis for c=15kPa and ϕ =10^0, depth of slip surface=3.52m and Factor of Safety= 1.190

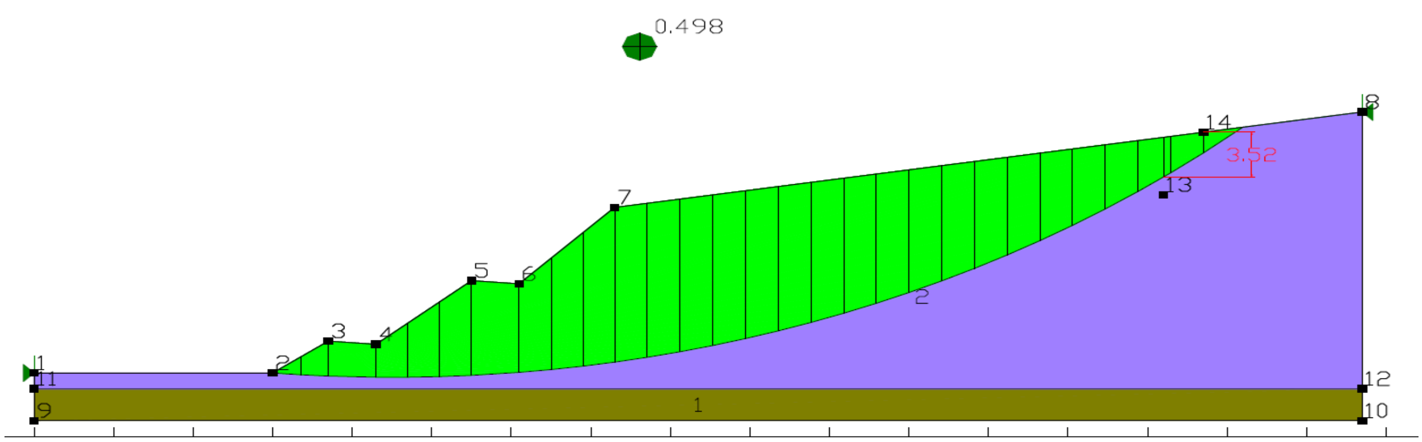


Figure C.6: Slope analysis for c=5kPa and ϕ =5^0, depth of slip surface=3.52m and Factor of Safety= 0.498

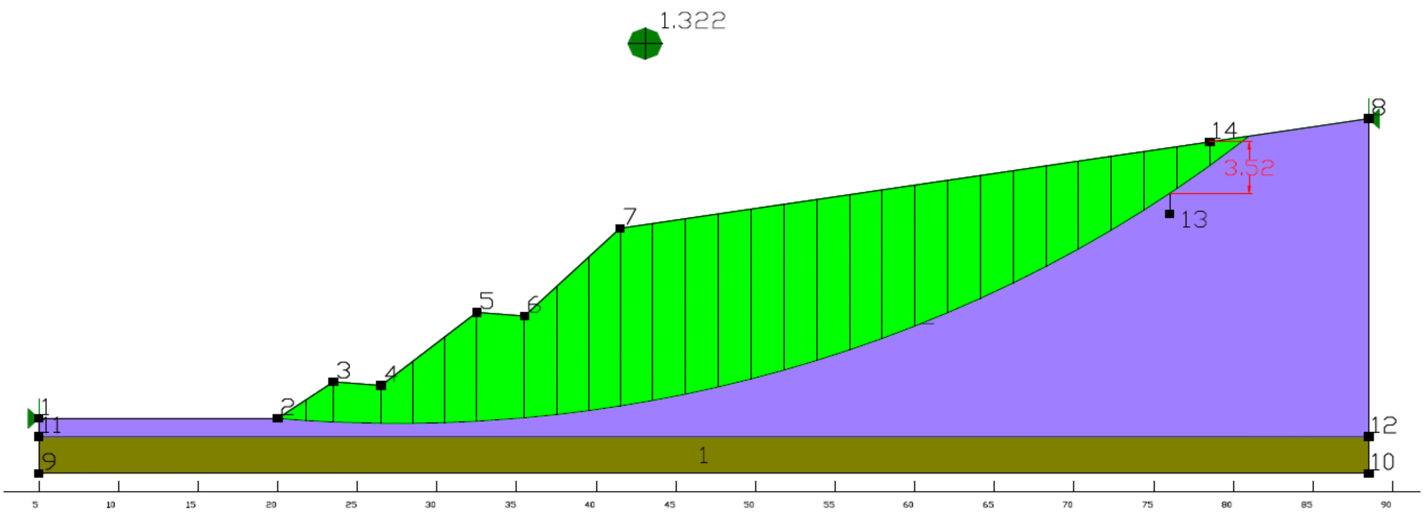


Figure C.7: Slope analysis for c=10kPa and ϕ =15^0, depth of slip surface=3.52m and Factor of Safety= 1.322

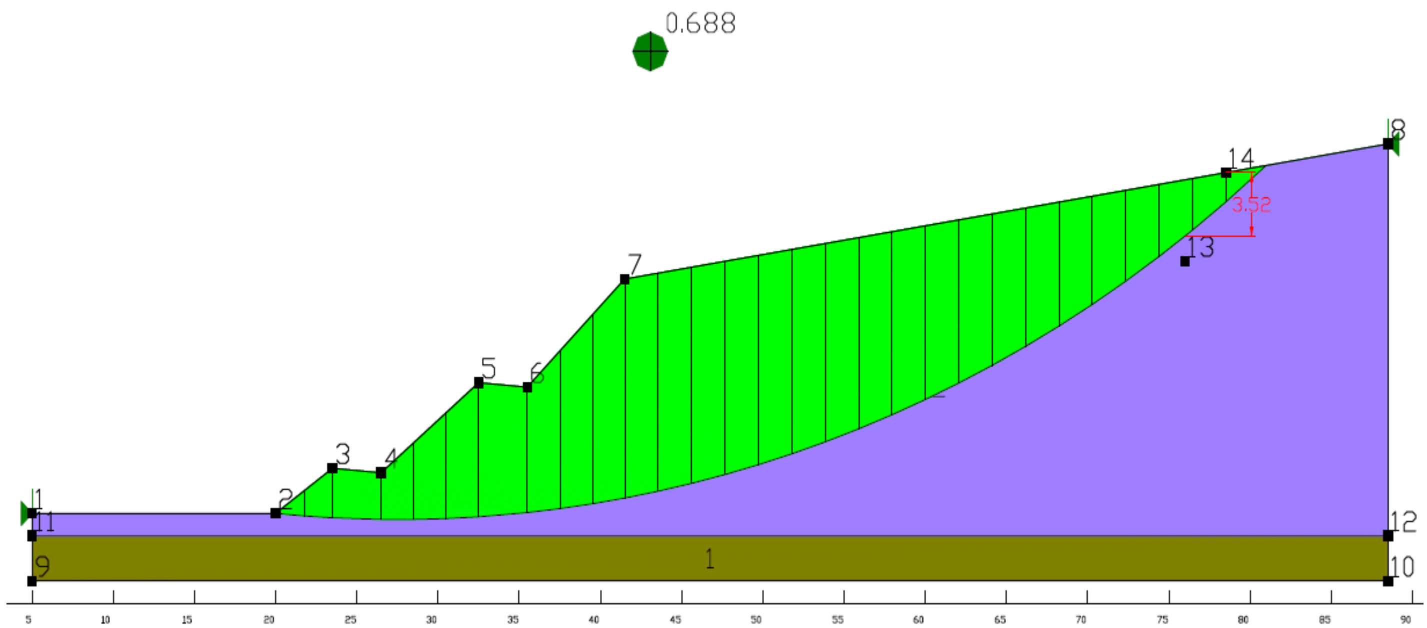


Figure C.8: Slope analysis for c=5kPa and ϕ =10^0, depth of slip surface=3.52m and Factor of Safety= 0.688

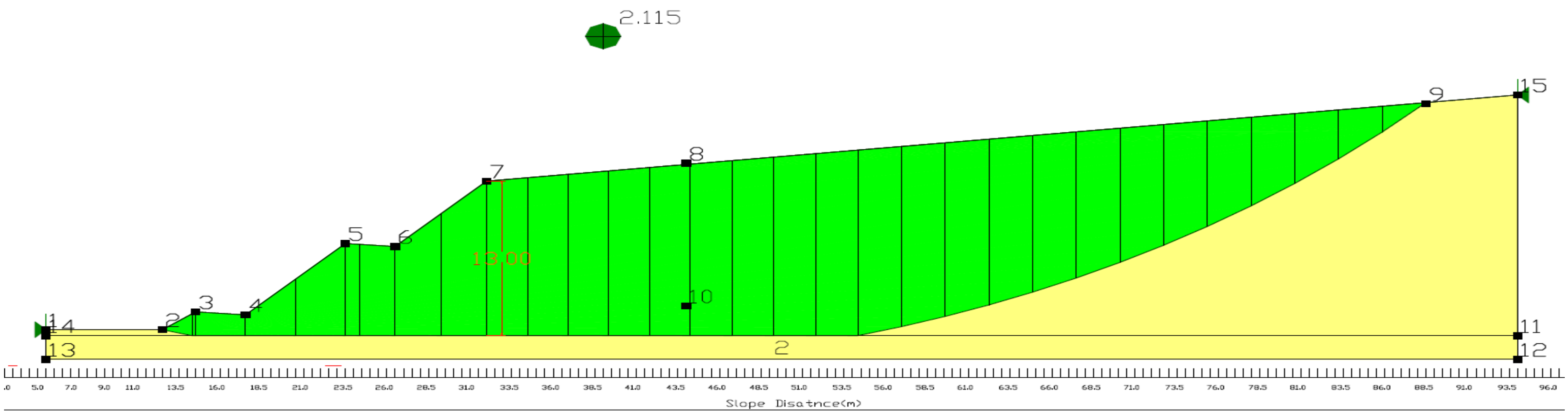


Figure C-9: Slope analysis for c=40kPa and $\phi=5^0$, depth of slip surface=13m and Factor of Safety= 2.115

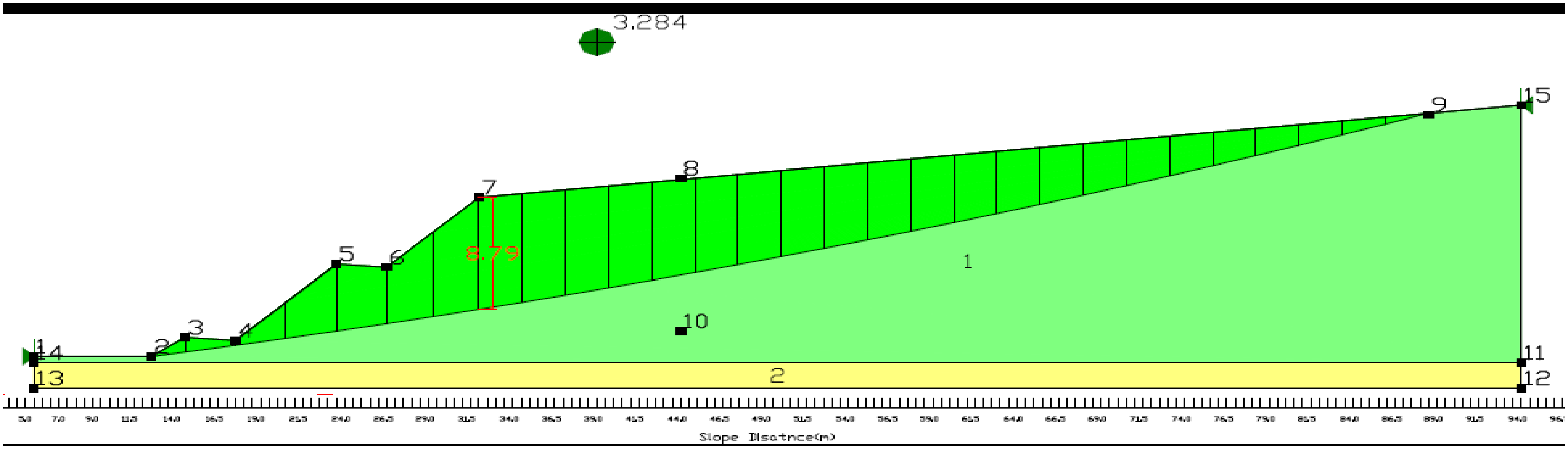


Figure C-10: Slope analysis for c=20kPa and $\phi=15^0$, depth of slip surface=8.79m and Factor of Safety= 3.284

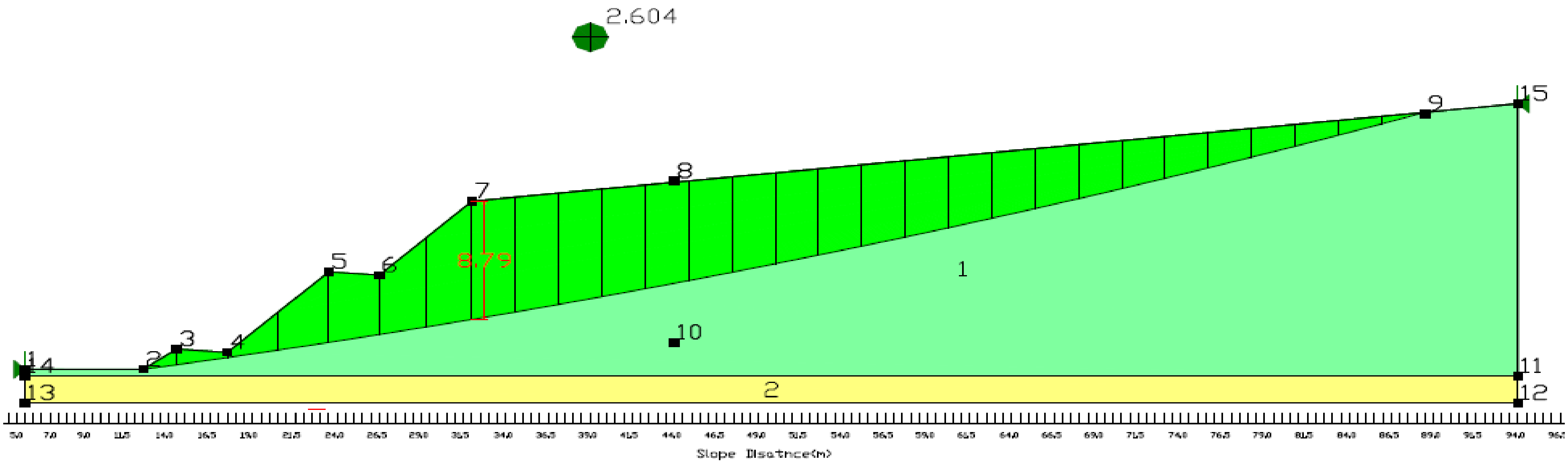


Figure C-11: Slope analysis for c=10kPa and $\phi=25^0$, depth of slip surface=8.79m and Factor of Safety= 2.604

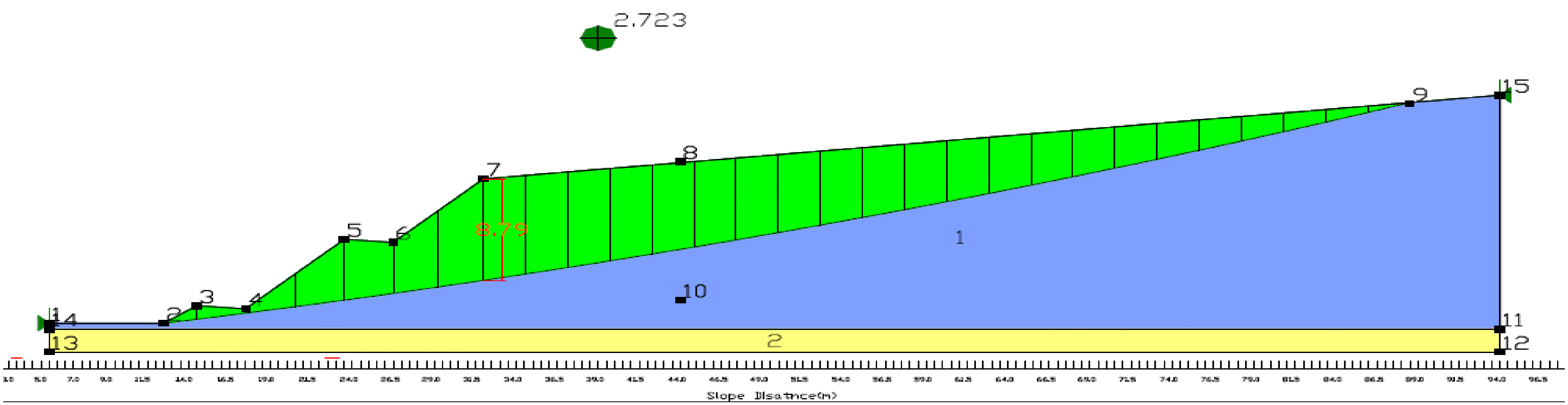


Figure C-12: Slope analysis for c=5kPa and $\phi=30^0$, depth of slip surface=8.79m and Factor of Safety= 2.723

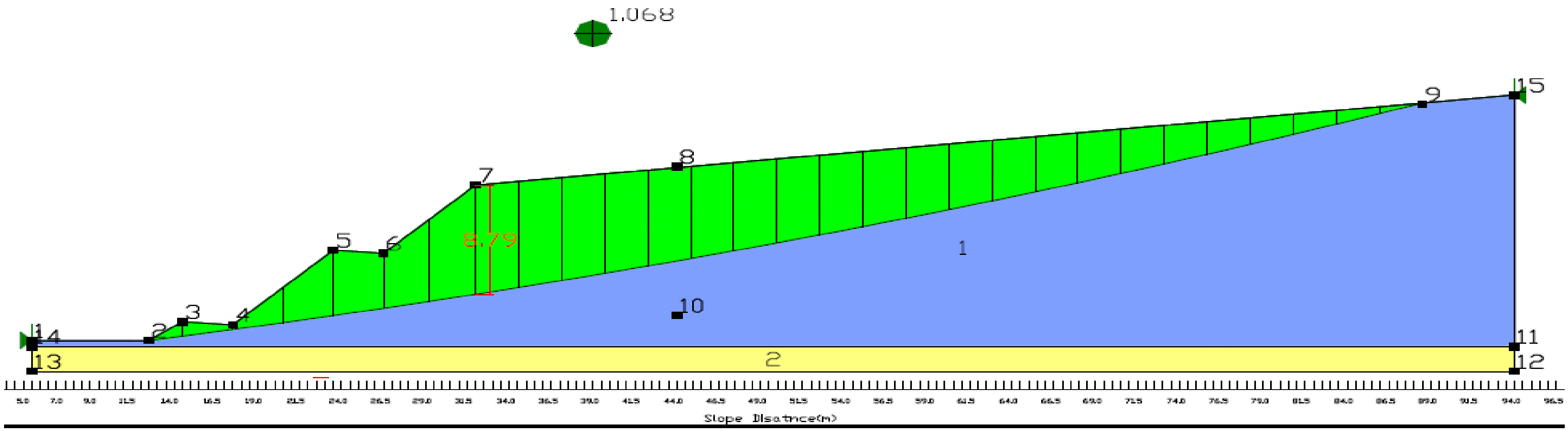


Figure C-13: Slope analysis for c=5kPa and ϕ=10^0, depth of slip surface=8.79m and Factor of Safety= 1.068

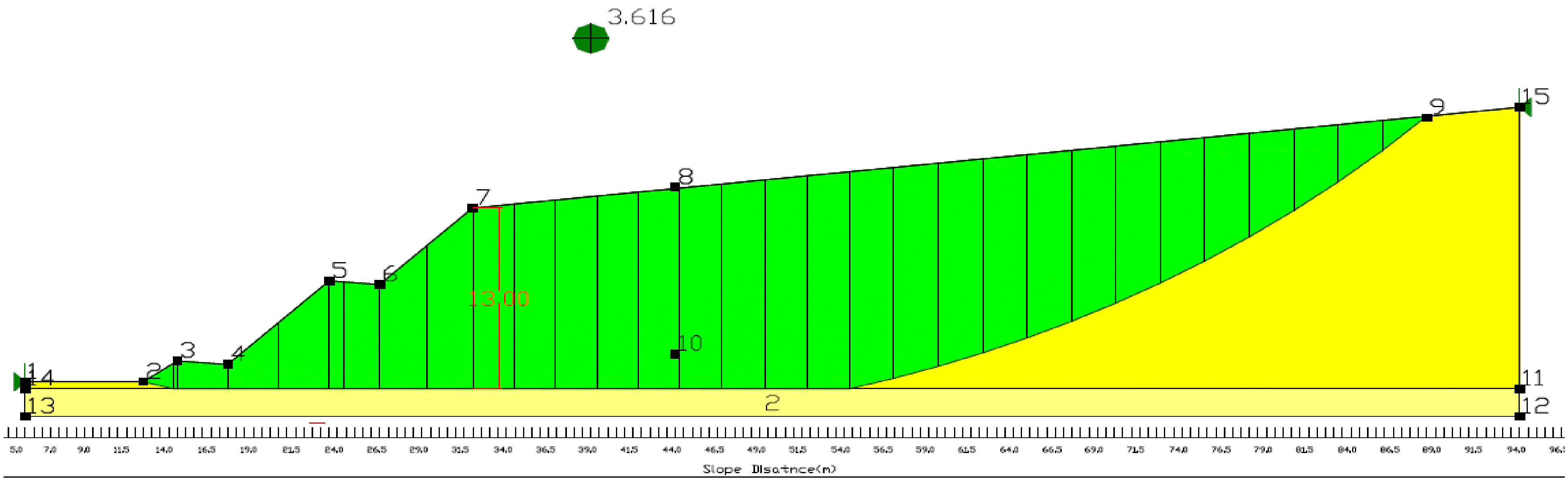


Figure C-14: Slope analysis for c=40kPa and ϕ=20^0, depth of slip surface=13.00m and Factor of Safety= 3.616

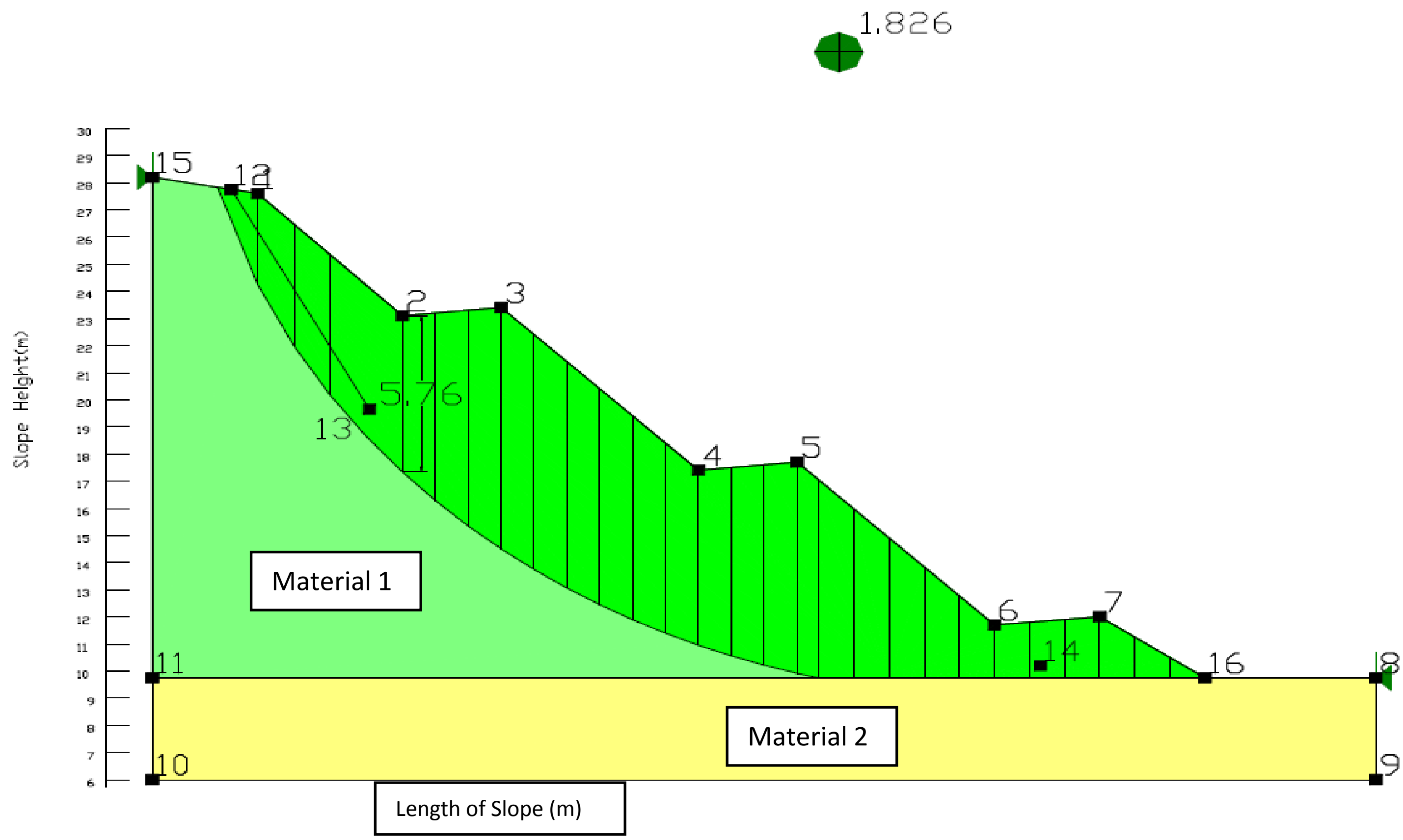


Figure C.15: Slope analysis for c=40kPa and $\phi=5^0$, depth of slip surface=5.76m and Factor of Safety= 1.826

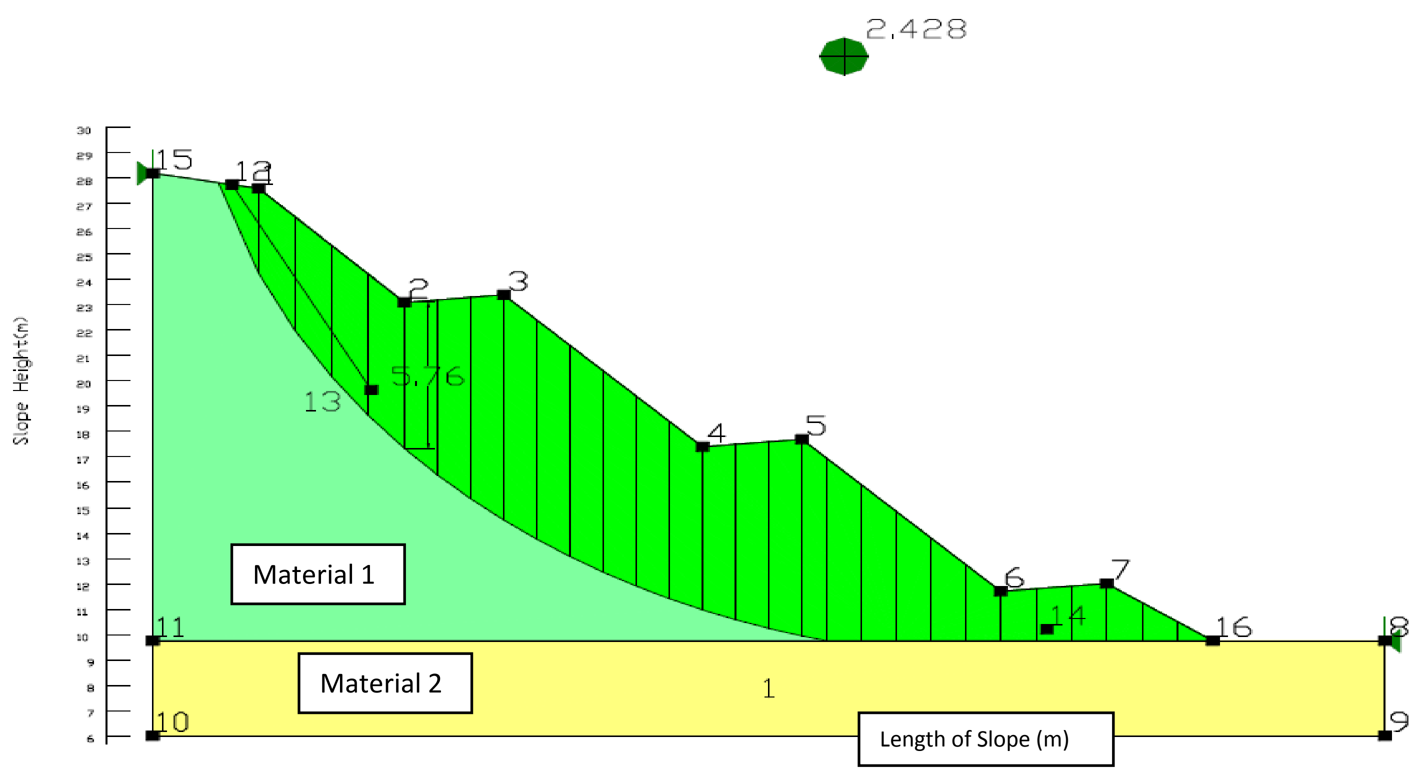


Figure C-16: Slope analysis for c=40kPa and $\phi=20^0$, depth of slip surface=5.76m and Factor of Safety= 2.428

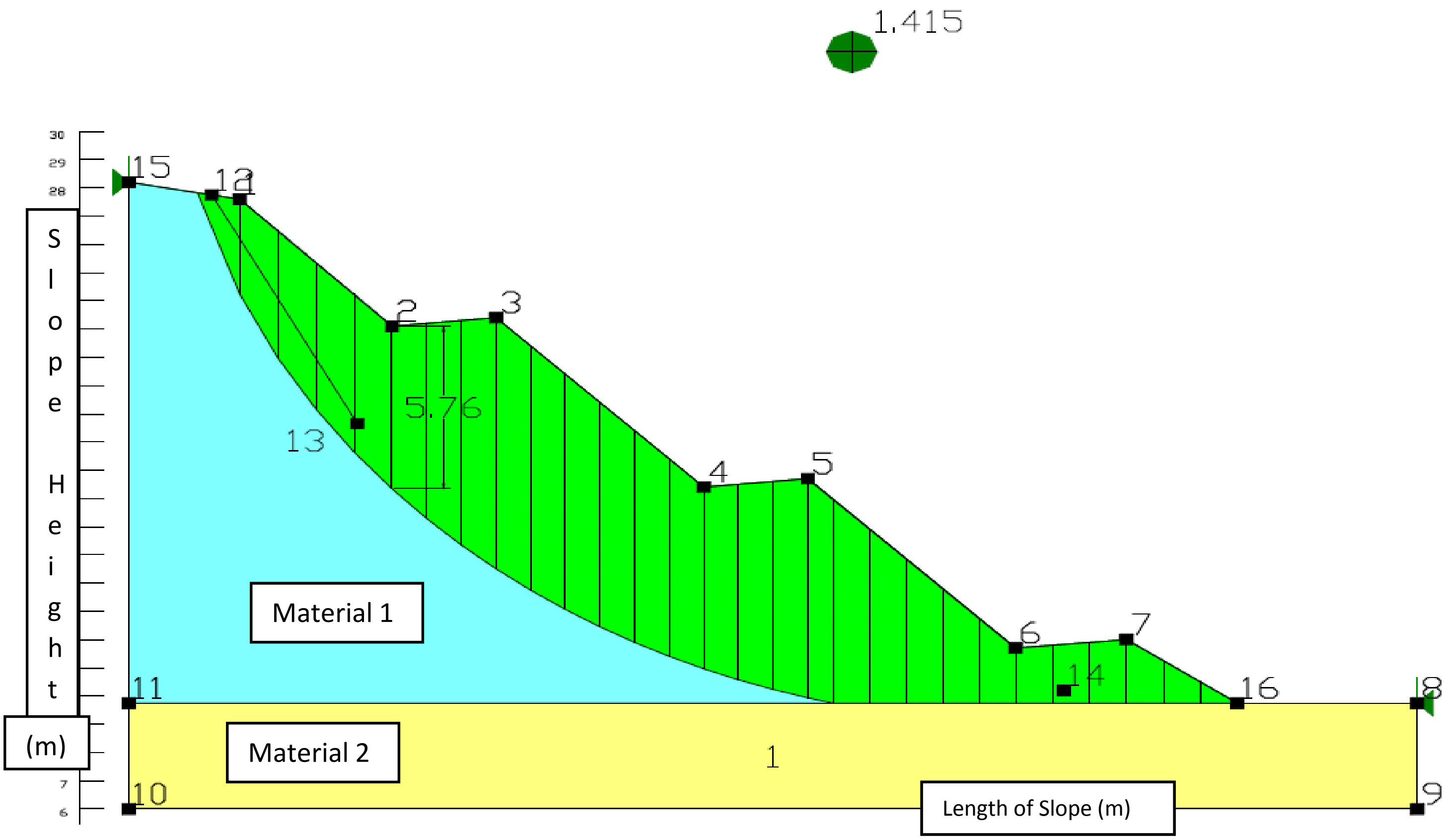


Figure C-17: Slope analysis for c=20kPa and $\phi=15^0$, depth of slip surface=5.76m and Factor of Safety= 1.415

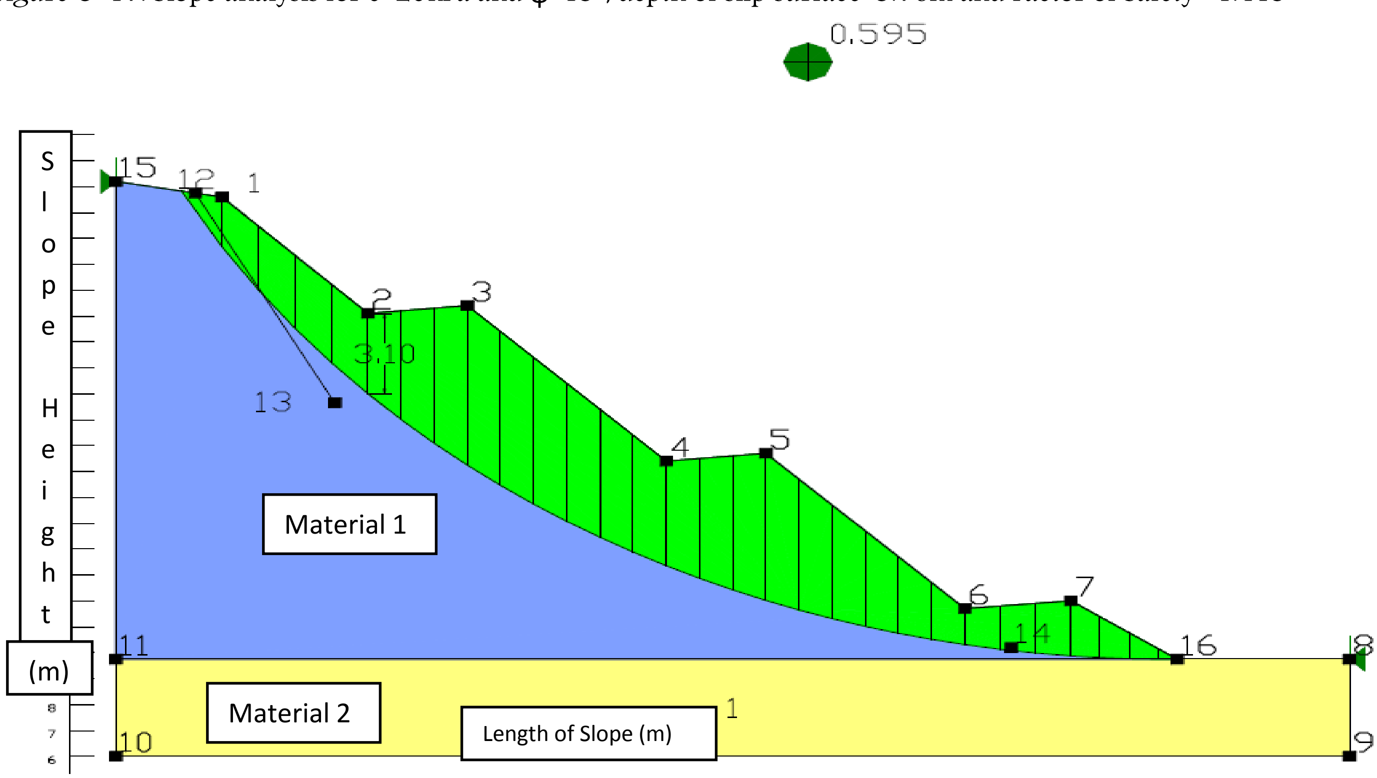


Figure C-18: Slope analysis for c=5kPa and $\phi=15^0$, depth of slip surface=3.10m and Factor of Safety= 0.595

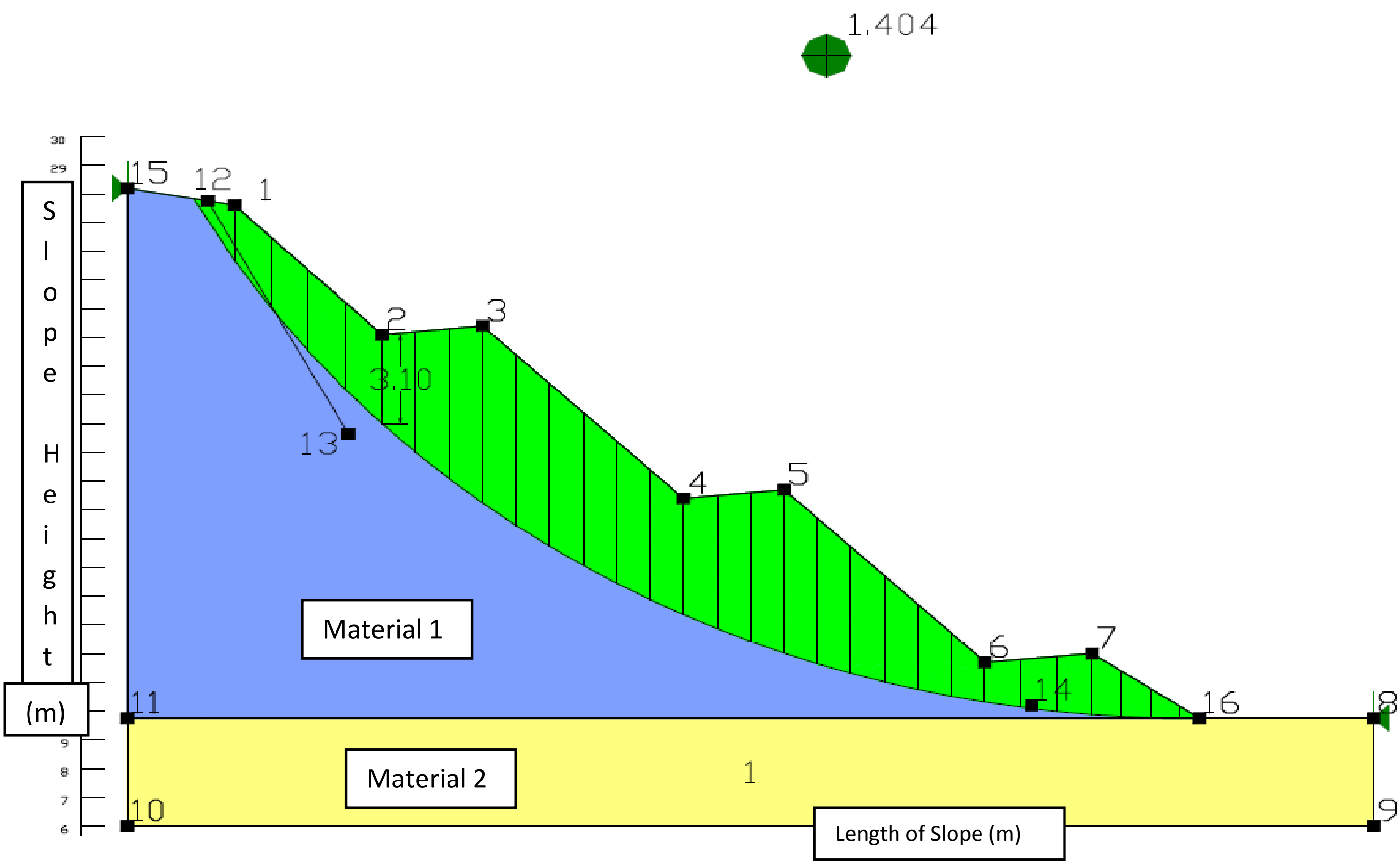


Figure C-19: Slope analysis for c=10kPa and ϕ=25⁰, depth of slip surface=3.10m and Factor of Safety= 1.404

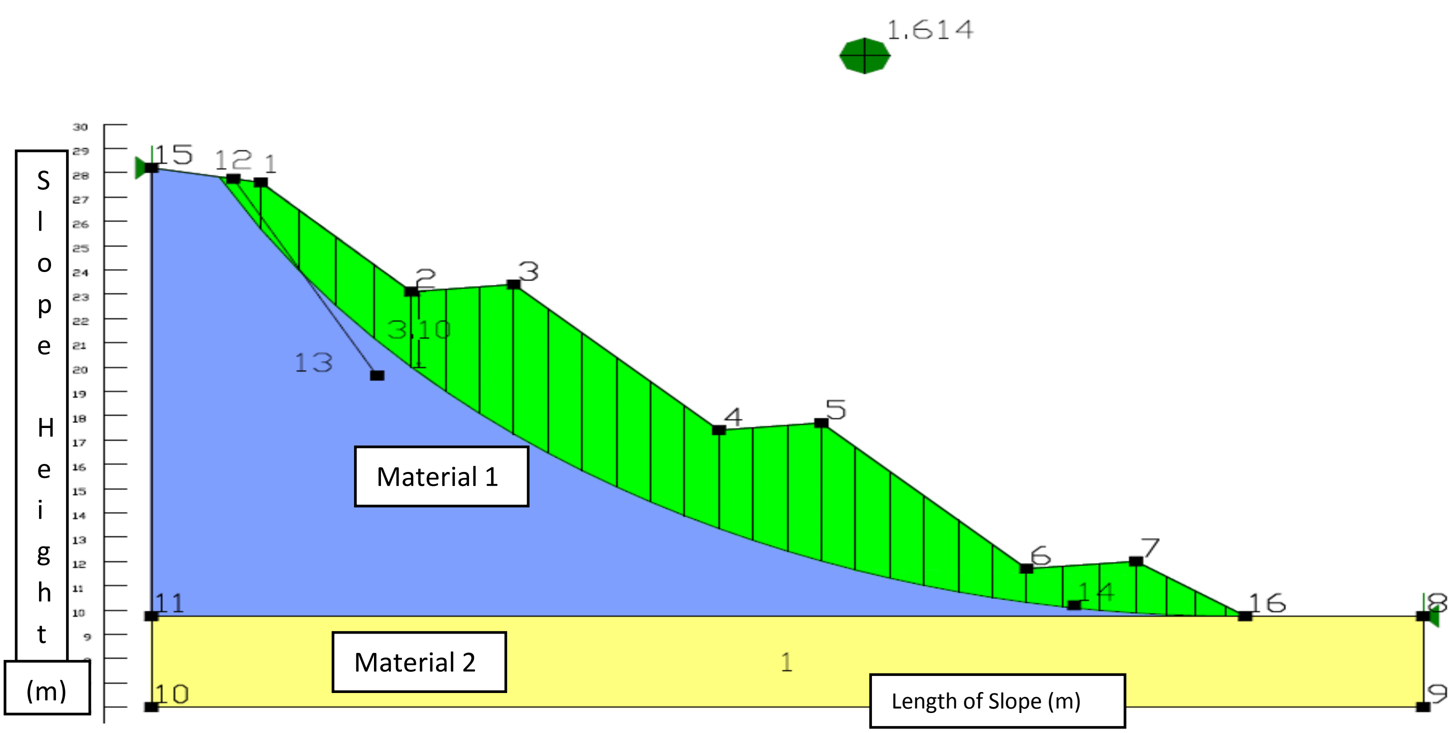


Figure C-20: Slope analysis for c=10kPa and ϕ=30⁰, depth of slip surface=3.10m and Factor of Safety= 1.614

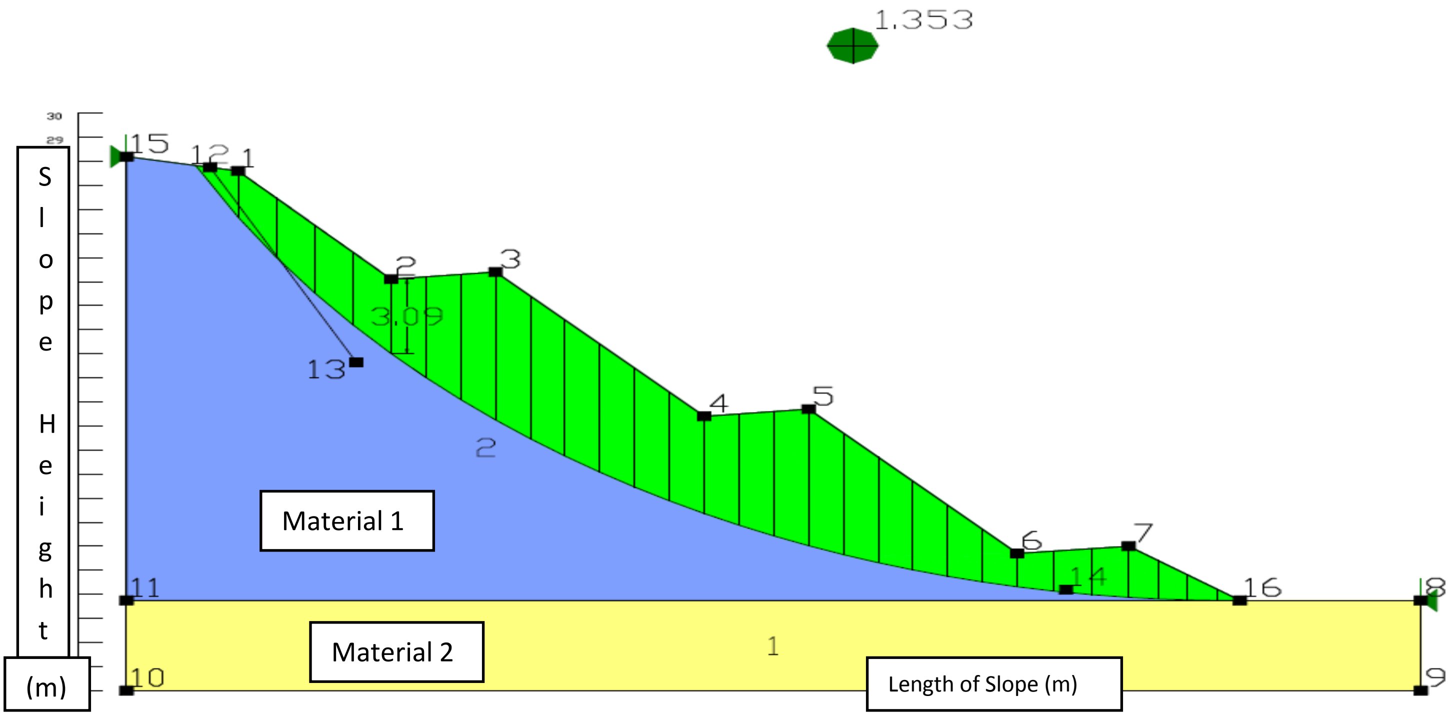


Figure C-21: Slope analysis for C=5kPa and ϕ =30^0, depth of slip surface=3.09m and Factor of Safety= 1.353

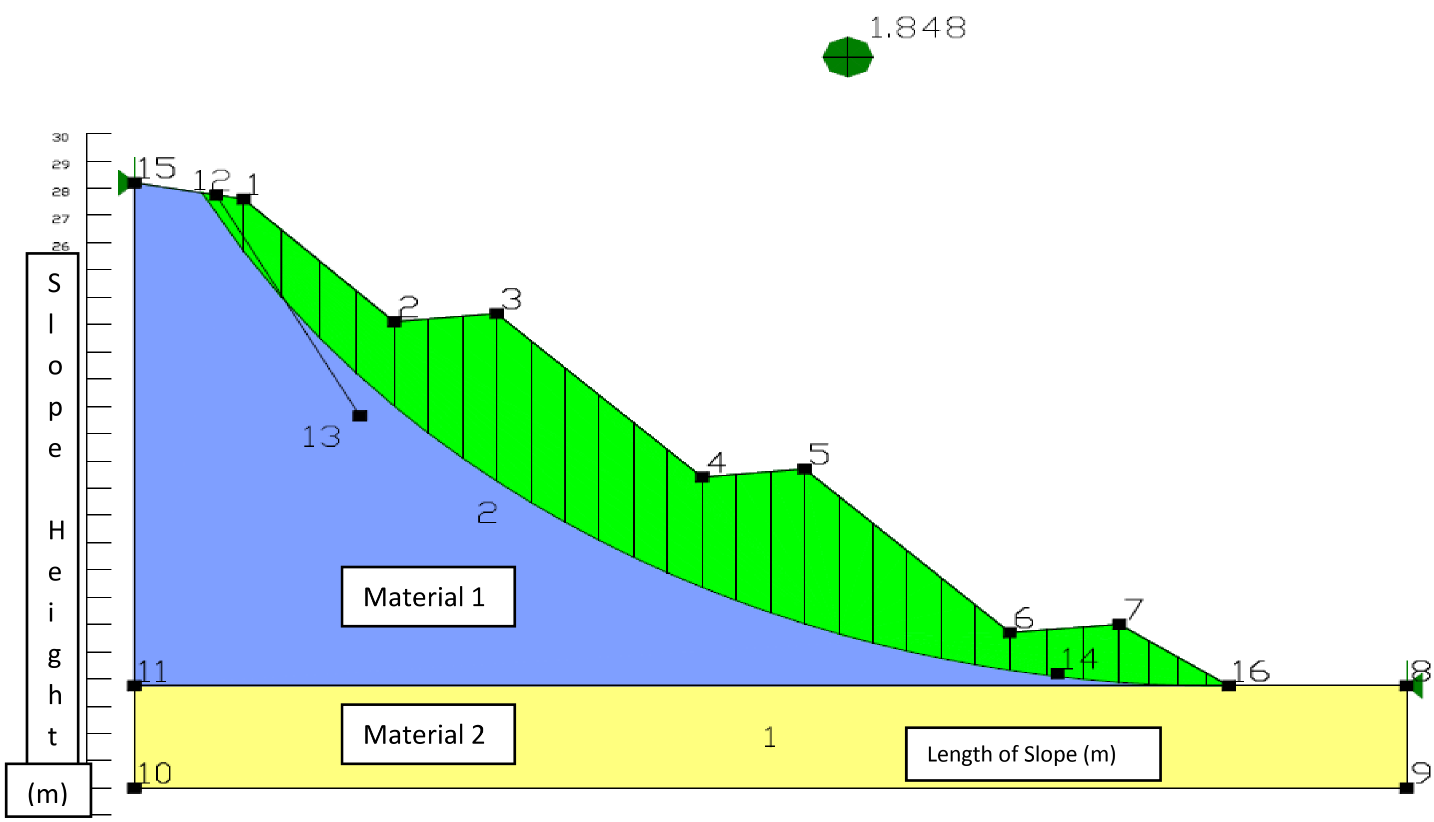


Figure C-22: Slope analysis for C=5kPa and ϕ=40^0, depth of slip surface=3.09m and Factor of Safety= 1.353

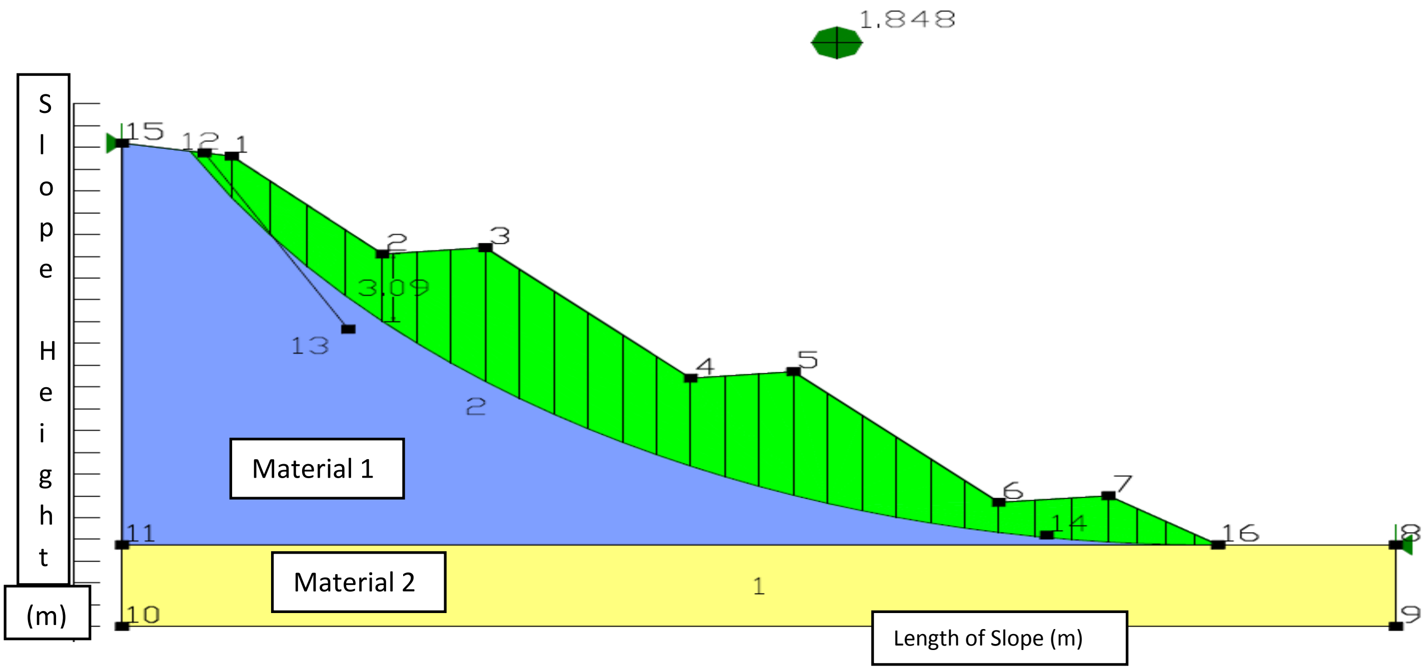


Figure C-23: Slope analysis for C=10kPa and ϕ =40^{0}, depth of slip surface=3.09m and Factor of Safety= 1.848