DELHI TECHNOLOGICAL UNIVERSITY

CERTIFICATE

This is to certify that the project report entitled "**PARAMETRIC STUDY OF STABILITY OF EMBANKMENTS**" is a bona fide record of work carried out by Ashish Kumar Kashyap (2K12/GTE/03) under my guidance and supervision, during the session 2014 in partial fulfillment of the requirement for the degree of Master of Technology (Geotechnical Engineering) from Delhi Technological University, Delhi.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

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JULY-2014

Preface

At first I express my deep praise to God for the project was completion in time. From the very beginning of my undergraduate studies at KIET, Ghaziabad, I like Geotechnical Engineering and very keen in learning latest softwares used in Civil Engineering.

In 2013, I met Dr. Raju Sarkar, Geotechnical specialist in Delhi Technological University and Discussed about master thesis project. In this regard I would like to thanks my guide who gave me the opportunity to work with slope stability analysis with modern geotechnical software. I am grateful to him for enduring advice, keen interest, directing me toward such an interesting problem.

I also, express my profound gratitude, indebtedness and thanks to Head of Department of Civil Engineering Department Prof. A. Trivedi,M. Tech thesis guide Dr. Raju Sarkar Sir, M. Tech project Incharge Prof. A K Sahu, Delhi Technological University for their valuable advice, assistance, encouragement, guidance, interest and cooperation and supervision of all stage in this project work. Their Guidance has benefited me greatly.

This has been a wonderful year for me to have experienced studying in Delhi Technological University. With my friends this has been more colourful who supported me during my study. Living in DTU would never have been as wonderful without all of you.

Lastly, I would like to express my gratitude to my family members and one of my best friends for their continuous support.

DTU, June 2014

Ashish Kumar Kashyap

Abstract

The project work focuses on stability of embankment with the Limit Equilibrium Method computer program Slope/W. The parametric study of stability of embankments have been analysed with this software.

The basic parameters like friction angle, cohesion etc. are needed when we use limit equilibrium method while in finite element method additional parametric information are needed regarding the potential performance of a slope. Depicting the result it is mandatory to use the effective shear strength characterization of the soil while we perform the stability analysis. A division should be made between drained and undrained strength of cohesive soil. Drained condition are the condition where drainage is allowed, while undrained condition are the condition where drainage is constrained .The worst case occurs when the river water level is increased speedily, and then quickly goes down while the water table in the embankment is retained on an tremendously high level so that the low effective stresses might lead to failure.

In Slope/W analysis we consider critical slip surface failure. At different slope factor of safety is calculated and each time one parameter is kept variable and others are kept constant to get the effect of that parameter on factor of safety.

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Chapter -1

INTRODUCTION

Analysing the stability of earthen embankment in soil is an important and challenging aspect of Civil Engineering. Slope instability is a geo-dynamic process that naturally shows the geo-morphology of the earth. The major concern when those unstable slopes have an effect on the safety of people. Concerns with slope stability have some of the most important advances in our understanding of the complex behaviour of soils. Detailed engineering and research studies performed over the past 75 years provided a sound set of soil mechanical principles with which to solve practical problems of slope stability.

Experiences with the behaviour of slopes, and their failure, has led to development of improved understanding of the changes in soil properties that can occur with time, the limitations of laboratory and in situ testing for evaluating soil strengths, development effective types of instrumentation to observe the behaviour of slopes, detailed understanding of the principles of soil mechanics that connect soil behaviour to slope stability, and improved analytical procedures augmented by extensive examination of the mechanics of slope stability analyses, detailed comparisons of different parameters, and use of softwares to perform thorough analyses. Through these advances, the slope stability evaluation has entered a mature phase, where experience and judgment, which continue to be of prime effect, have been combined with improved understanding and advance methodology to improve the level of understanding that is achievable through systematic observation, testing, and analysis.

This project provides the general information required for slope stability analysis, suitable methods of analysis with the use of computers softwares like SLOPE W , and examples of common stability problems in the location of the places like India, Pakistan etc.

Normally, the stability analysis of soil techniques is classified into three categories (Bishop, 1954):

1) Limiting analysis approach;
2) Limiting equilibrium approach;
3) Displacement-based approach.

In the first method of soil stability analysis technique, i.e. the limiting analysis techniques of upper bound solution and lower bound solution are involved. They are based on theory of classical plasticity related to flow rule. Their application is limited to simple geometry ideal material. Instead, the displacement based approach is a more modern development method. This method includes various methods like finite elements method, boundary element method and the discrete element method. In these the discrete element method is very much useful for stability analysis of rock slopes.

This study is focused on the limiting equilibrium analysis and considering Simplified Bishop method. The limit equilibrium methods study the equilibrium of the soil mass going to slide down by the action of gravity. Rotational movement or transitional movement is measured on known (or assumed) slip surface below rock mass or soil mass.

The analysis of stability also requires the estimation of the actuating forces like gravity forces, seepage forces and earthquake forces. In addition, the definition of the shape of the mess of the soil involved in failure is also a prerequisite for the solution. It is relatively simple to define the shape of the failure surface, but several trials are needed to determine the case which leads to the minimum value of the factor of safety.

Generally, a plane strain condition is assumed- a situation which will be true when the length of the slope is large compared to its cross-section. A typical cross-section is investigated, considering unit thickness, ignoring the strains in the perpendicular direction. Such a two-dimensional analysis is known to give a conservative value of factor of safety for an actual three-dimensional problem.

The factor of safety, defined as a ratio of available to that required shear resistance for equilibrium. If FOS is less than 1.0, slope is considered as unstable and if factor of safety is greater than or equal to 1.0, slope is considered as stable. The methods of slices are the most common limit equilibrium techniques. In this method soil mass is dvided into vertical slices. The FOS of specific methods can vary because methods differs in satisfied equilibrium conditions and assumptions involved.

Material presented in this dissertation report is an effort to develop a computer program on Fortran 90 capable of analysing the slope stability of an embankment or a soil mass with the effect of seepage, describe the procedures for applying this computer program to a broad spectrum of practical slope configuration. The program is based on the simplified

Bishop method of slices, which assume a circular slip surface. It is used to search for co-ordinates of the centre and radius of the critical circle which has the least factor of safety and also determined that minimum factor of safety for critical slip circle.

Chapter -2

2.1 Background

The stability analysis of emabnakment of Jamuna river was studied from International Journal of Science and Engineering Investigation. Jamuna, one of the largest river and play an significant role in socio-economic factor. This river has often required earthen emabankment in variours places to prevent river erosion. Earthen embankment tends to get washed away by heavy rainfall . The main cause of failure was embankment cutting by local people, erosion , sliding and seepage.Further during construction poor quality earthwork may be one of the reason of failure.Inadequate compaction and or insufficient laying of topsoil layer , river migration and cutting by public. Among above reasons the improper design methodology is one of the reason of embankment failure.

Embankment failure is one of the common phenomena in a low laying country. Every year earthen embankments are facing serious difficulties like erosion, breaching. The major causes are considered due to the use of geotechnically unstable materials, improper methodology of construction, seepage. In this study the problem is considered from geotechnical point of view where the geotechnical properties of failed Jamuna embankment materials were investigated. The stability analysis technique of embankment has been reviewed through a case study of Jamuna river embankment.

The river erosion creates a problem to almost every aspect including the livelihood, the riverside inhabitants, agriculture and geology of the flood plain alongside the river. This increasing problem of embankment failure has been treated with great importance as well as field investigations were conducted to prevent this problem. Hence, the present work paid attention to investigate the mechanical properties of the failed Jamuna river embankment materials at Siraj. The sustainable method was developed as well to protect the bank of big river embankment like Jamuna.

The site was chosen with a view to finding out the problem regarding embankment failure and to come out with an economic solution. The site was found very fragile considering the weak slope stability. The main reason for this problem was soil erosion was insufficient soil properties. The soil was collected from the weakest point of the riverside. The soil was brought under several laboratory tests. The test results shows that this continuous failure was caused for lack of slope stability due to poor soil properties. From the test result some analysis were run by slope stability software to find out the existing factor of safety. Many of the analysis gave low factor of safety some of which were below allowable limit. Hence the analysis was run again after mixing sand with the main sample and as an accompanying solution.

The program STB2010 uses Bishop's Method. In this method, the safety factor of a slope is determined by comparing the moment of the weight of a soil wedge about the centre of a slip circle, with the resisting moment provided by the shear stresses along the slip surface. It is assumed that on the vertical side planes of the slices only horizontal (normal) stresses are acting and no shear stresses. The first basic equation is Coulomb's Equation for the shear stress along the lower part of a slice,

$$t = [c + (s - p) \times \tan(\Phi)] \div F \qquad (1)$$

Where, t is the shear stress,

c is the cohesion,

s is the stress normal to the sliding plane,

p is the pore water pressure,

Φ is the angle of internal friction and

F is the safety factor.

The second basic equation is the equation of vertical equilibrium of a slice,

$$W \times h = s + t \tan \alpha \qquad (2)$$

Where, W is the average unit weight of the slice,

h is its height and

α is slope of the slip surface at the slice considered.

Equilibrium of moment with respect to the centre of the circle leads to a formula from which the safety factor can be calculated.

2.2 Raising of Mangla Dam, Pakistan

The Mangla Dam is one of the largest embankment dams in the world. The 260 square km reservoir is formed by four major dams, each with a different cross-section. The original designs included provision for raising the dams by up to 12 m to remove the effects of future sedimentation. In 2000, capacity lost due to sedimentation became a significant issue and the Government has decided to exploit the raising provisions of the original design.

The Dam serves the world's largest irrigation network, bringing water to 120 000 square kilometres of land and serving people who live in the vast Indus River basin. It is the world's fifth largest earth-filled dam and has the largest- capacity spillway, capable of discharging 26 500 m3/s, over ten times of the average flow over Niagara Falls.

The scope of the Feasibility Study included the following:

• The foundation conditions were to check their adequacy for raising, including the results of investigations carried out for Kalabagh Dam.

• The data of the foundation and structures to evaluate whether they supported raising in accordance with the previous provisions or to some other optimal level.

• The availability of the material and configuration of raising for the various embankments.

• Analysis, supervision of additional investigations. The Feasibility Study was to look at raising the dams by 6, 9 and 12 m and concluded that raising was still not accurate and that the optimum raising was by 9 m. This raising by 9 m presented the lowest cost/benefit ratio.

• The crest of the embankment dams was raised by 9 m and a 600 mm high parapet along both sides of the crest was constructed. The upstream shoulder of the Sukian Dyke was also widened to accommodate its vertical impervious core. Total embankment length is 13 km and additional fill volume required 30 000 000 m3.

• The placement of 1 500 000 m3 of additional material under water to the upstream toe weight berm on the Intake Embankment

• To reduce seepage impervious blankets on the floor of the reservoir in selected locations through the foundations.

• A new 250 m long x 25 m high RCC weir to control flows to the Emergency Spillway with design capacity of 6 500 m^3/s.

Table 2.1 Specification of Mangla Dam

Main Components	Original Parameter	Raised Parameter
Reservoir		
Maximum conservation level	366.5m	378.7m
Maximum operating level	317.1m	317.1m
Gross storage capacity	7253Mm3	9132 Mm3
Area	258 Km2	324 Km2
Main Dam And Intake Embankment		
Maximum Height	138.5m	147.6m
Crest length	3140m	3400m
Jari Dam		
Maximum Height	83.5m	92.6m
Crest length	4420m	5340m
Kakra Dam		
Maximum Height	38m	47m
Crest length	580m	610m

Main Spillway Maximum Discharge Capacity	28,620 cumecs	-do-
Emergency Spillway Maximum Discharge Capacity Control Weir Crest Elevation Control Weir Length	6520 cumecs	-do- 378m 250m
Power Tunnel and Power Station No. of Tunnel Inner Diameter Length No. of generating Units Installed Capacity	6 7.8 to 9.4m 476m 11 1000 MW	-do- -do- -do- -do- 1,180 MW

Project Benefits:

Additional Average Annual Water Availability 3551Mm3

Additional Average Annual Generation 643GWh

Environmental Impact: Additional Water for Irrigation

Additional Power Generation

Improved Flood Mitigation

Improved Socio-Economic Condition

2.3 Embankment Design

2.3.1Embankment Materials

The structure of the various embankments varied according to foundations and the materials available locally and total 30 Million m3 of material was required. The materials which were used in the embankments were similar with those used for the original construction. However, borrow materials were required to produce filters and drain material. Washing of gravels was also needed to remove fines. In the event materials supply issues were encountered but these were due to land leasing complications. The quantities of seepage through the zones of the embankments were estimated using the finite element software SEEP/W. Filters and drains were provided in the sections of all the embankments to prevent piping and without surcharging the drainage system. Since they met modern standard, the specified gradation envelopes of filter were the same as those used in the construction of the existing Mangla dam. These zones were extended in the raised section. Slope Stability Analysis. The Morgenstern and Price method, programmed in SLOPE/W (GEO-SLOPE 2002), was used for slope stability analyses of the embankments. In the two dimensional analysis, the calculated factors of safety varied from section to section and in some cases fell below acceptable values. To account for these, the whole failure mass was divided into a number of sections, following Sherard's procedure. Three dimensional restraint was involved where the two dimensional factor of safety was less than the target factor of safety. The following cases were analysed for each section of embankment:

• Normal drawdown from 1242 ft. to 1040 ft.: The pore pressures were calculated from the record of piezometers. The target factor of safety for this case was 1.5.

• In steady seepage with full reservoir the downstream slope analysis was critical. The target factor of safety was 1.5.

• Drawdown from 1242 ft. to 1040 ft.: No dissipation of pore pressures was assumed and factor of safety of no less than unity was required.

A high factor of safety from static slope stability analysis provides additional assurance. The design parameters for both foundation and fill were produced from testing carried out at Mangla.

2.3.2 Instrumentation

The dams at Mangla were instrumented during the original construction. About 670 piezometers of various types were installed to analyse the behaviour of earth fill embankments. Settlement gauges, and survey markers were installed to monitor the foundations and fill settlement. The large number of instruments were still in service four decades after installation. However, with the passage of time some of the installations were blocked by foreign material or torpedoes stuck inside the casing during observation. The instrumentation changes for monitoring construction and future performance of the raised dam include the following.

• Installation of standpipe piezometers in the raised sections of the embankments, in the foundations.

• Extension of the standpipes of the old standpipe piezometers.

• Installation of V-notches in seepage measurement chambers, in downstream nullahs and in the drainage gallery of the newly constructed Control Weir.

• Installation of survey markers on the raised crest, slopes.

• Extending casings of the old settlement gauges and slope indicators.

• Construction of galleries for the instrument which were fixed in fill of the raised embankments and construction of new instrument houses at toe of the raised embankments.

• Construction of terminal panels of vibrating wire piezometers installed in the raised portion of the embankments.

Chapter- 3

LITERATURE REVIEW

3.1 Ground Investigation

The necessary borehole information should be obtained before any further examination of an existing slope, or the ground into which a slope is to be built. This information provides details of the soil layer, water content and the existing water level. The presence of any plastic layer along which shear could take place will be noted. Ground investigations also include:

- In-situ and laboratory tests and aerial photographs.
- Study of maps and memoirs to indicate soil conditions.
- Observing the slope for the study in this, field investigations have been done and use of cone penetration test (CPT) for evaluation of geotechnical parameters.

3.2 Geotechnical Parameters

The parameters values needed in the analysis must be determined before geotechnical analysis.

3.2.1 Unit weight

It is defined as the ratio of the total weight of the soil to the total volume of the soil. Unit weight, γ, is determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field. Measuring unit weight of soil directly in the field might be done by sand cone test, rubber balloon.

3.2.2 Cohesion

Cohesion, c, is usually determined from the Direct Shear Test in the laboratory. Unconfined Compressive Strength can be determined using the Triaxial Test or the Unconfined Compressive Strength Test in the laboratory. There are also correlations for Suc with shear strength as estimated from the field using Vane Shear Tests.

3.2.3 Friction Angle

The angle of internal friction, φ, can be determined in the laboratory by the Direct Shear Test or by Triaxial test in the laboratory. For our analysis we will use values taken from research papers and Delhi Technological University soil sample.

3.2.4 Young's Modulus of Soil

It may be estimated from empirical correlations, laboratory test results on undisturbed specimens and results of field tests. Laboratory test that might be used to estimate the soil modulus is the triaxial test.

3.3 Types of soil

We classify soils, or more properly earth materials, for their properties relative to foundation support. These systems are designed to analyse some of the engineering properties and behaviour of a soil based on a few simple laboratory or field tests.

3.4 Basic Requirement for Slope Stability Analysis

The slope stability analyses are performed for drained conditions or undrained conditions, the most basic requirement is that equilibrium must be satisfied. All body forces, and all external loads, including those due to water pressures acting on external boundaries, must be included in the analysis. These analyses provide two useful results:

(1) The total normal stress on the shear surface and

(2) The shear stress required for equilibrium.

The factor of safety for the shear surface is defined as the ratio of the shear strength of the soil divided by the shear stress required for equilibrium. The normal stresses along the slip surface are needed to evaluate the shear strength excluding for soils with $\varphi = 0$, the shear strength depends on the normal stress on the potential plane of failure.

In effective stress analyses, the 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 along the shear surface. These pore pressures can be evaluated with relatively good accuracy for drained conditions, where their values are determined by steady seepage boundary conditions. Pore pressures can be evaluated accurately for undrained 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. Total stress analyses are applicable only to undrained conditions. The basic premise of total stress analysis is this: the pore pressures due to undrained loading are determined by the behaviour of the soil. For a given value of total stress on the potential failure plane, there is a one value of pore pressure and therefore a unique value of effective stress. It is true that shear strength is really controlled by effective stress; it is possible for the undrained condition to relate shear strength to total normal stress, because effective stress and total stress are uniquely related for the undrained condition. Clearly, this line of reasoning does not apply to drained conditions, where pore pressures are controlled by hydraulic boundary conditions.

3.5 Drained and Undrained Strength

A distinction should be made between drained and undrained strength of cohesive materials. As cohesive materials or clays generally have less permeability compared to sand, thus, the movement of water is restricted. So, for clay, it needs years to dissipate the excess pore water pressure before the effective equilibrium is reached. Drained condition refers to the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted.

3.5.1 Analyses of Drained Conditions

Drained conditions are those where changes in load are slow, or where they have been in place long enough, so that all of the soils reach a state of equilibrium and no excess pore pressures are caused by the loads. In drained conditions pore pressures are controlled by hydraulic boundary conditions. The water within the soil may be static, or may be seeping, with no change in the seepage over time and no increase or decrease in the amount of water within the soil. If these conditions prevail in all the soils at a site, or if the conditions at a site can reasonably be approximated by these conditions, a drained analysis is appropriate. A drained analysis is performed using:

- Total unit weights
- Effective stress shear strength parameters
- Pore pressures determined from hydrostatic water levels or steady seepage analyses.

3.5.2 Analyses of Undrained Conditions

Undrained conditions are those where changes in loads occur more than water can flow through the soil. The pore pressures are controlled by the behaviour of the soil in response to changes in external loads. If these conditions occur in the soils at a site, or if the conditions at a site can be approximated by these conditions, an undrained analysis is appropriate. An undrained analysis is performed using:

• Total unit weights

• Total stress shear strength parameters.

3.6 Short-Term Analyses

Short term refers to conditions during construction—the time immediately following the change in load. If constructing a sand embankment on a clay foundation takes two months, the short-term condition for the embankment would be the end of construction, or two months. Within this period of time, it would be an approximation that no drainage would occur in the clay foundation, whereas the sand embankment would be fully drained.

3.7 Long-Term Analyses

After a period of time, the clay foundation would reach a drained condition, the analysis for this condition would be performed as discussed earlier under ‘‘Analyses of Drained Conditions’’, because long term and drained conditions have the same meaning. Both of these terms refer to the condition where drainage equilibrium has been reached and there are no excess pore pressures due to external loads.

3.8 Pore Water Pressures

For effective stress analyses the basis for pore water pressures should be studied. If pore water pressures are based on measurements of groundwater levels in bore holes, the measured data should be described and summarized in appropriate figures. If seepage analyses are performed to calculate the pore water pressures, the method of analysis, including software, which was used, should be described. Soil properties should include the permeability. Appropriate flow nets or contours of pore water pressure, total head, or pressure head should be presented to summarize the results of the analyses.

3.9 Soil Property Evaluation

The soil properties used in a stability evaluation should be described and laboratory test data should be presented. If properties are estimated based on experience, or using correlations with other soil properties or from data from similar sites, this should be explained. Results of laboratory tests should be summarized to include index properties, water content, and unit weights. For compacted soils, suitable compaction moisture–

density data are useful. A summary of shear strength properties is important and should include both the original data and the shear strength envelopes used for analyses (Mohr–Coulomb diagrams, modified Mohr–Coulomb diagrams). The laboratory data that are used in slope stability analyses are the unit weights and shear strength envelopes. If many more extensive laboratory data are available, the information can be presented separately from the stability analyses in other sections. Only the summaries of shear strength and unit weight information need to be presented with the stability evaluation in such cases.

3.10 Circular Slip Surface

In limit equilibrium stability analyses is the requirement to analyse many trials slip surfaces and find the slip surface that gives the lowest factor of safety. Included in this trial approach is the form of the slip surface; that is, whether it is circular, piece-wise linear or some combination of curved and linear segments. Slope/W has a variety of options for specifying trial slip surfaces. The position of the critical slip surface is affected by the soil strength properties. The position of the critical slip surface for a purely frictional soil ($c = 0$) is radically different than for a soil assigned untrained strength ($\varphi = 0$). This complicates the situation, because it means that in order to find the position of the critical slip surface, it is necessary to accurately define the soil properties in terms of effective strength parameters.

3.11 Factor of Safety

In slope stability, and in fact generally in the area of geotechnical engineering, the factor which is very often in doubt is the shear strength of the soil. The loading is known more accurately because usually it merely consists of the self- weight of the slope. The FoS is therefore chosen as a ratio of the available shear strength to that required to keep the slope stable. For highly unlikely loading conditions, accepted factors of safety can be as low as 1.2-1.25, even for dams e.g. situations based on seismic effects, or where there is rapid drawdown of the water level in a reservoir. The allowable limit for factor of safety is 1.5 for undrained analysis and 1.3 for combined or drained analysis.

3.12 Types of Slope Failure

Depending on the geological situations, the soil or rock slopes can fail in different manners. Table 3.1 presents a classification of geologic failure patterns and the related essentials of slope failure and engineering failure patterns (Hunt, 1984). The failure forms of interest for this research are infinite slopes, planar, and circular rock and soil slidesCircular failure surface analysis is used for thick residual or colluvial soil, soft marine clay, shale, and firm cohesive soil (Hunt, 1984).

Table 3.1: Comparison of Elements and Classification of Geological and Engineering Failure Forms (Hunt, 1984)

Geologic Failure Forms	Elements of Slope Failures*						Engineering Failure Forms+		
	Slope Inclination	Slope Height	Material Structure	Material Strength	Seepage Forces	Runoff	Infinite Slope	Planar Failure Surface	Circular Failure Surface
Falls	P	N	P	P	P	P	N	N	N
Planar Slides	P	S	P	P	P	M	A	A	N
Rotational Slides (Rock)	P	P	P	P	P	M	N	N	A
Rotational Slides (Soil)	P	P	P	P	P	M	N	N	A
Spreading/Progressive Failure	S	M	P	P	P	N	N	N	N
Debris Slides	P	M	P	P	P	N	S	S	N
Submarine Slides	S	S	P	P	P	N	N	N	S

NOTE:

* P - Primary Cause, S - Secondary Cause, M - Minor Effect, N - Little/No Effect

+A - Application, S - Some Application, P -Poor Application, N - No Application

3.13 Traditional Slope Stability Analysis Methods

For the purposes of this review, traditional slope stability analysis methods are defined as those which treat slopes as deterministic situations with uniquely defined parameters. The following description are based on the work of Das (1994). Traditional methods use principles of static equilibrium to evaluate the balance of available and required shear resistance. The factor of safety, defined as a ratio of available to that required shear resistance for the equilibrium. If FOS greater than 1.0 is considered as a stable slope; if FOS less than 1.0 then the slope is considered as a unstable slope or slope failure.

For a given slope, a FOS of 1.0 indicates the critical height of a slope.

Figure 3.2 reviews the types of traditional slope analysis methods, these are:

a) Infinite slope without seepage

b) Infinite slope with seepage

c) Finite slope with planar failure surface

d) Circular failure in homogeneous clays: (Φ=0 and Φ>0)

Figure 3.1 determines that slope geometry is defined by two parameters, the height of the slope, H, and the angle of the slope with the horizontal plane, β. These soil parameters are:

c = cohesion

γ = unit weight of soil

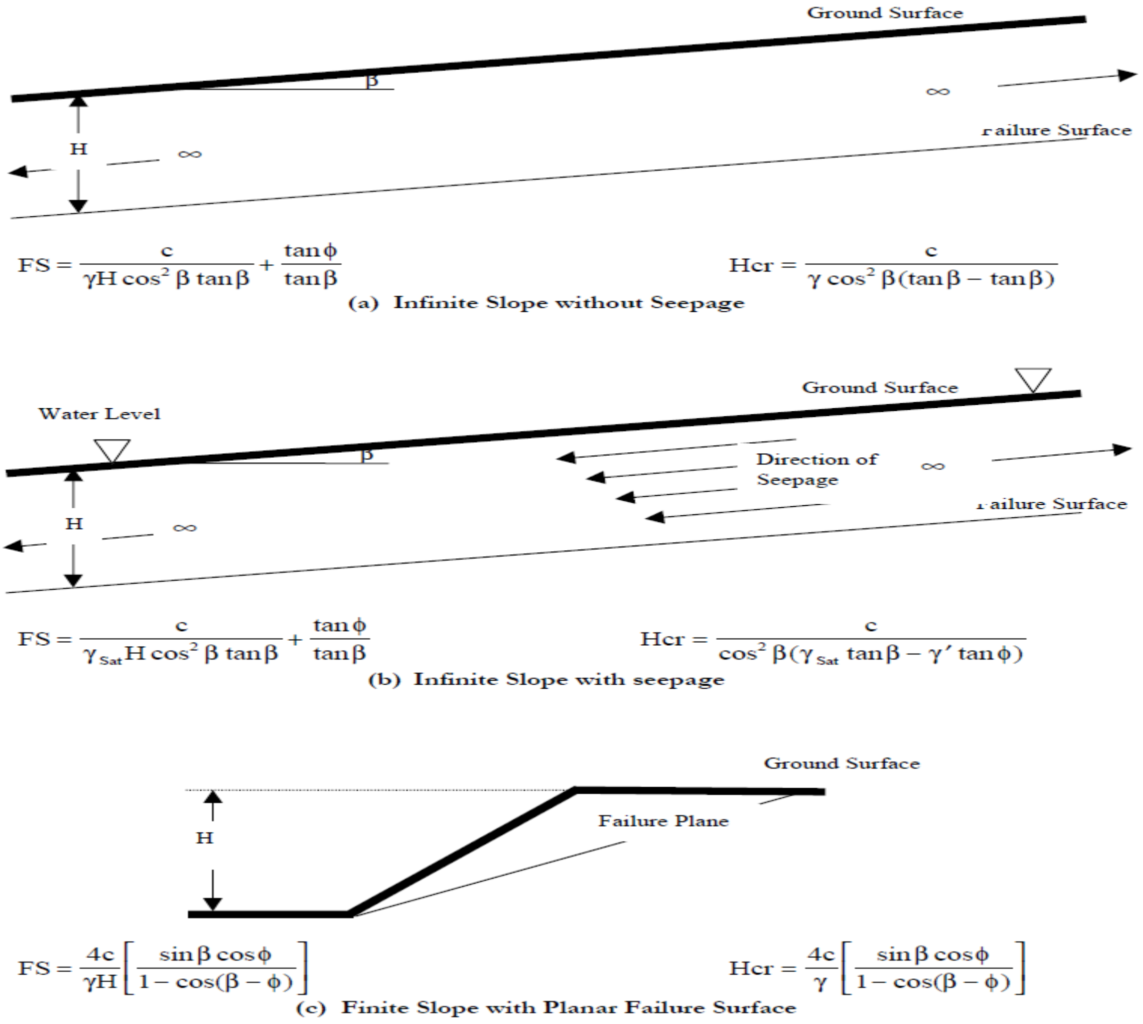

Figure 3.1 Traditional Slope Stability Analysis Methods (Das, 1994)

2. Method of Analysis

The method of analysis used in this computer program is based on a limiting equilibrium condition and the method called is the simplified Bishop method of slices (Bishop, 1954). Figure 3.2 shows a free body (or vertical slice of soil lying above an assumed circular slip surface); using known or assumed forces acting on the slice, the shearing resistance of the soil required for equilibrium is calculated. The ratio of shear strength of the soil to the calculated shearing resistance required indicates the factor of safety for the slope. Forces and dimensions on figure 3.2 are defined as follows:

E_n, E_{n+1}	are the resultants of the total horizontal forces on the slice,
X_n, X_{n+1}	are the vertical shear forces,
W	is the weight of the slice,
N	is the total normal force on the base,
R	is the radius of the slip circle,
S	is the shear force on the base,
U	is the boundary water force,
L	is the arc length of an assumed slip surface for the slice,
b	is the width of the slice,
θ	is the angle between S and the horizontal,
O	is the centre of the circle and point of rotation, and
x	is the horizontal distance from the centre of the slice to 0.

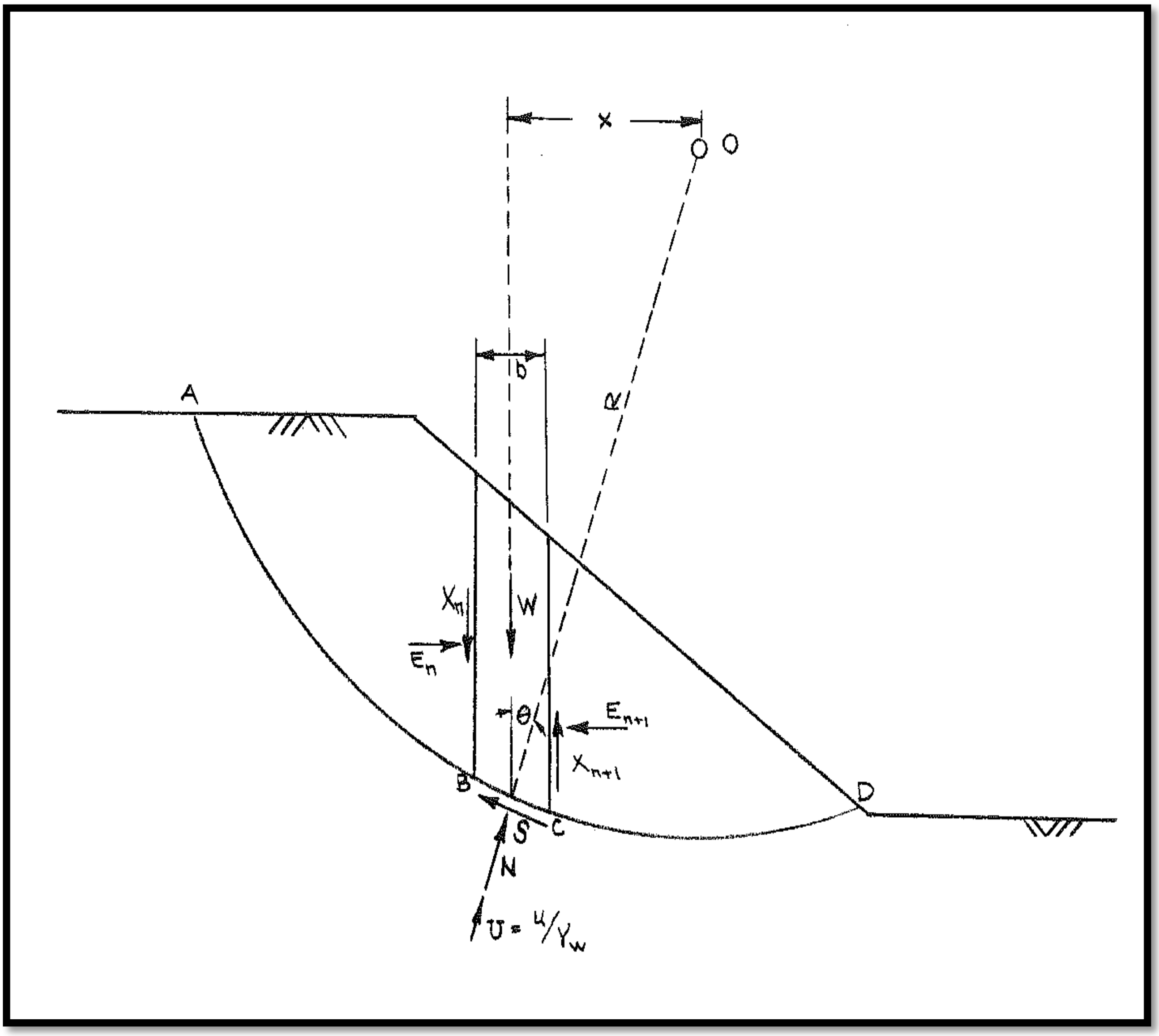

Figure 3.2 Forces on slice in Bishop Method (Bishop, 1954)

Using the definition of factor of safety and Mohr-Coulomb failure criterion, the mobilized shear stress can be written in terms of the shear strength as

$$S = (c' + \sigma' \tan \phi') / F \quad (3.1)$$

Taking moments around 0 of the weight of the soil and the external forces acting on the slice on the circular arc and assuming equilibrium conditions:

$$\Sigma W_x = \Sigma SR = \Sigma sLR \quad (3.2)$$

Noting that σ = N/L – u, it follows from Equations 2.1 and 2.2 that

$$F = R\,\Sigma\,(c'L + (N\text{-}uL) \tan\phi) / \Sigma W_x \quad (3.3)$$

Where u = boundary pore water pressure. Bishop observed that more accurate solutions (especially for deep slip circles where an appreciable change in *0* can occur) were obtained by solving for and resolving the normal forces vertically. Doing so, and letting L = b sec θ, the factor of safety becomes

$$F = \frac{R}{\sum \mathrm{W_x}} \sum [\frac{(c'b+(W+X_n-X_{n-1})tan\emptyset')sec\theta}{1+\frac{tan\theta tan\emptyset'}{F}} \qquad (3.4)$$

Horizontal side forces do not appear in Equation 4 since forces were resolved vertically.

In addition to the circular failure assumption, Bishop concludes that (X_n – X_{n+1}) can be taken to be zero throughout the arc without significant error, typically less than one percent. This conclusion was verified by Whitman and Bailey (1967). They solved many problems using this assumption and a statically accurate method (Morgenstern-Price) and found that the resulting error was seven percent or less. Usually, the error was two percent or less.

Noting that x = R sin θ, Equation 3.4 can be simplified to

$$F = \frac{1}{\sum \mathrm{W\,sin\theta}} \sum [\frac{(c'b+(W-U)tan\emptyset')sec\theta}{1+\frac{tan\theta tan\emptyset'}{F}}] \qquad (3.5)$$

U is seepage force and equal to (U=u*b)

From the above equation factor of safety is obtained. The method of slices has gained in popularity in the methods of analysis, due to its ability to accommodate complex geometrics and variable soil and water pressure conditions (Terzaghi and Peck, 1967). Subsequently, various new methods based on this concept have been developed (Wright, 1969). A comparison of some methods of analysis has been published by Fredlund and Krahn (1977), as summarised in Table 3.2. Their research aimed to compare the FOS obtained by each method.

Table 3.2: Methods of slides comparisons (adapted from Fredlund and Krahn, 1977; Corps of Engineers, 2003)

Method	Factor of Safety (FS)		Interslice Force Assumption (H=horizontal, V=vertical)
	Force Equilibrium	Moment Equilibrium	
(1) Ordinary (Swedish or USBR)	-	Yes	Ignore both H, V
(2) Bishop's Simplified	**-**	**Yes**	**V ignored, H considered**
(3) Janbu's Simplified	Yes	-	V ignored, H considered
(4) Janbu's 'Generalised'	Yes	-	Both H, V considered
(5) Spencer	**Yes**	**Yes**	Both H, V considered
(6) Morgestern-Price	**Yes**	**Yes**	Both H, V considered
(7) Lowe-Karafiath	Yes	-	Both H, V considered
(8) Corps of Engineers	Yes	-	Both H, V considered

Fredlund & Krahn (1977) concluded that FOS from analysis methods (1) to (6) are very similar (difference <0.1%). All methods have the same form of the normal force equation with the exception of the Ordinary method. The differences in the various methods are the assumptions relating to the inter slice forces. For instance, the Ordinary method ignores inter slice forces (V=H=0); Simplified Bishop's method assumes inter slice forces are horizontal (V=0, H>0); Spencer's method assumes all inter slice forces are parallel (V>0, H>0) with an unknown inclination which is computed through iterations; Morgenstern and Price's method relates the shear force (V) to the normal force (H), where $V=\lambda f(x) H$.

The first three methods - Ordinary method, Bishop's simplified and Janbu's simplified, ignore vertical inter-slice forces. Due to the assumption that effective normal and pore pressure forces do not affect the moment equilibrium since they are directed through the centre of the circle, therefore, Ordinary method, Bishop's simplified and Janbu's simplified, should not be used to compute an FOS for noncircular failure surfaces (Abramson et al., 2002).

Abramson et al., (2002) told that vertical inter-slice forces were not considered in Ordinary method, Bishop's simplified and Janbu's simplified. They also assumed that the effective normal and pore pressure didn't affect the moment equilibrium as they were directed through the centre of the circle, therefore Ordinary method, Bishop's simplified and Janbu's simplified should not be used to evaluate the FOS for noncircular failure surfaces. It was also stated that Bishop's method cannot be applied to analyse horizontal force equilibrium and Janbu's simplified can't be applied to analyse moment equilibrium. They also told that Spencer's method or the Morgensters-Price's method can be used to analyse the force and moment equilibrium. In the Janbu's simplified method the final FOS was obtained by multiplying the evaluated FOS with a modification factor, However, FOS value obtained from Bishop's method and Janbu's method normally differ by +15 % to the calculated FOS from Spencer's method or the Morgensters-Price's method (Abramson et al., 2002).

In the Corps of Engineers (2003) method FOS was determined with the use of force equilibrium analysis by considering the inclination of the inter slice force. The difference was that the Corps of Engineers method offered an over determined system, where moment equilibrium was not satisfied for all slices (Abramson et al., 2002).

The latest method for limit equilibrium analysis is that proposed by Fredlund et al. (1981) and Chugh (1986) namely, general limit equilibrium (GLE). The method can determine FOS by satisfying both force and moment equilibrium. It also can be used for analysing circular and noncircular failure surfaces. Furthermore, the GLE has the ability to model a discrete version of the Morgenstern and Price (1965) procedure, and to implement the Spencer's method directly by using a constant inter slice force function (Abramson et al., 2002).

In conclusion, it is very important for a geotechnical engineer to have a comprehensive understanding of the limit equilibrium methods. A large range of method procedures, from simple to complex analysis, requires a geotechnical engineer to have an ability to choose the most suitable method for particular slopes. The use of computer analysis can be the best solution for complex equations in limit equilibrium analysis

Chapter-4

Methodology

Many different solution techniques for slope stability analyses have been developed over the years. Analysis of slope stability is one of the oldest types of numerical analysis in geotechnical engineering. In this project we will use both Limit Equilibrium Method and Finite Element Method for our analysis. The modern geotechnical software programs is utilized, i.e. Slope/W.

4.1 Slope/W

4.1.1 Limit Equilibrium Methods

Modern limit equilibrium software is making it possible to handle ever- increasing complexity within an analysis. It is now possible to deal with complex stratigraphy, highly irregular pore-water pressure conditions, and various linear and nonlinear shear strength models, almost any kind of slip surface shape, distributed or concentrated loads, and structural reinforcement. Limit equilibrium formulations based on the method of slices are also being applied more and more to the stability analysis of structures such as tie-back walls nail or fabric reinforced slopes, and even the sliding stability of structures subjected to high horizontal loading arising, for example, from ice flows.

4.1.2 Defining the Problem

A limit equilibrium analysis was carried out using the Slope/W software for the slope stability of the natural slope. The geometry was created in .dxt format and imported into the Slope/W program. The analysis type is then selected and it is determined that failure will follow a right to left path. The Morgenstern- Price analysis and half-sine function was selected but the software also gives the result of factor of safety for Ordinary, Bishop and Janbu analysis type.

4.1.3 Modeling

The most common way of describing the shear strength of geotechnical materials is by Coulomb's equation which is:

$\tau = c + \sigma n' \tan\varphi$ … … … … … … … (4.1)

where, τ is shear strength (i.e., shear at failure), c is cohesion, $\sigma'n$ is normal stress on shear plane, and φ is angle of internal friction. The equation 4.1 represents a straight line on shear strength versus normal stress plot . The intercept on the shear strength axis is the cohesion c and the slope of the line is the angle of internal friction φ. The strength parameters c and φ can be total strength parameters or effective strength parameters. Slope/W makes no distinction between these two sets of parameters. Which set is appropriate for a particular analysis is project- specific, and is something you as the software user, need to decide. From a slope stability analysis point of view, effective strength parameters give the most realistic solution, particularly with respect to the position of the critical slip surface.

4.1.4 Analysis Type

An analysis of slope stability begins with the hypothesis that the stability of a slope is the result of downward or motivating forces (i.e., gravitational) and resisting (or upward) forces. The resisting forces must be greater than the motivating forces in order for a slope to be stable. The relative stability of a slope (or how stable it is at any given time) is typically conveyed by geotechnical engineers through a factor of safety Fs defined as

$$Fs = \frac{\sum R}{\sum M} \qquad (4.2)$$

The equation states that the factor of safety is the ratio between the forces/moments resisting (R) movement and the forces/moments motivating (M) movement.

4.1.4.1 Ordinary method of slices

This method neglects all interslice forces and fails to satisfy force equilibrium for the slide mass as well as for individual slices. However, this is one of the simplest procedures based on the method of slices (Fellenius, 1936). This method assumes a circular slip surface and it is also known as the Swedish Method of Slices or the Fellenius Method.

4.1.4.2 Simplified Bishop

The simplified Bishop method assumes that the vertical interslice shear force does not exist and the resultant interslice force is therefore horizontal (Bishop, 1955). It satisfies the equilibrium of moment but not the equilibrium of forces.

This method uses the horizontal forces equilibrium equation to obtain the factor of safety. It does not include interslice forces in the analysis but account for its effect using a correction factor. The correction factor is related to cohesion, angle of internal friction and the shape of the failure surface (Janbu et al., 1956).

4.1.4.3 Spencer Method

This is a very accurate method which satisfies both equilibrium of forces and moments and it works for any shape of slip surface. The basic assumption used in this method is that the inclinations of the side forces are the same for all the slices.

4.1.4.4 Morgenstern and Price

Morgenstern and Price proposed a method that is similar to Spencer's method, except that the inclination of the interslice resultant force is assumed to vary according to a "portion" of an arbitrary function. This method allows one to specify different types of interslice force function (Morgenstern & Price, 1965).

4.1.4.5 General Limit Equilibrium

This method can be used to satisfy either force or moment equilibrium, or if required, just the force equilibrium conditions. It encompasses most of the assumptions used by various methods and may be used to analyse circular and noncircular failure surfaces (Ferdlund, Krahn, & Pufahl, 1981).

4.1.5 Slip Surface for Circular Failure Model

After the material input and pore pressure was assigned, a slip surface was defined. The analyses were performed for two failure models namely the circular failure model and block failure model. There were several methods for defining the slip surface for the circular failure but the entry and exit method was selected. One of the problems with the other methods is how to visualize the extents or the range of the trial slip surface. This difficulty is solved by the entry and exit method because it specifies the location where the trial slip surfaces should enter the ground surface and where they should exit.

4.1.6 Verification and Computation

When the slip surface has been specified, then Slope/W runs several checks to verify the input data using the verify/optimize data command in the Tools menu. When the verification is completed and there are no errors, then Slope/W computes the factor of safety using the method of slice selected. The minimum factor of safety is obtained for that particular analysis.

Chapter-5

5.1 Raised Embankment Design

The raised embankment design is the most common construction technique used in tailings storage facilities. The three principal designs are downstream, upstream and centreline structures, which designate the direction in which the embankment crest moves in relation to the starter dyke at the base of the embankment wall (Vick 1990). Modified centreline is another method rarely used which a combination is between upstream and centreline construction.

As the name suggests the embankment is raised at certain time intervals to increase the available volume for the storage of tailings and water, thus they have a lower initial capital cost than retention dams because fill material and placement costs are phased over the life of the impoundment. The choices available for construction material are increased as smaller quantities are needed at any one time. For example, retention dams generally use natural soil whereas raised embankments can use natural soil, tailings, and waste rock in any combination (Vick 1990). The most common materials used for embankment raises are waste mine rock, natural borrow soils, underground roadway development material, cyclone tailings (coarse fraction) and hydraulically deposited tailings. With the developments of high capacity earthmoving equipment in the 1940's raises can be compacted in a similar manner to that of water retention dam construction techniques (Martin, Davies et al. 2002). Today, raised embankment construction is almost always mechanised to gain the level of compaction required to increase the safety and lower the risk of instability of the storage facility. As the name suggests the embankment is raised at certain time intervals to increase the available volume for the storage of tailings and water, thus they have a lower initial capital cost than retention dams because fill material and placement costs are phased over the life of the impoundment. The choices available for construction material are increased as smaller quantities are needed at any one time. For example, retention dams generally use natural soil whereas raised embankments can use natural soil, tailings, and waste rock in any combination (Vick 1990). The most common materials used for embankment raises are waste mine rock, natural borrow soils, underground roadway development material, cyclone tailings (coarse fraction) and hydraulically deposited tailings. With the developments of high capacity earthmoving equipment in the 1940's raises can be compacted in a similar manner to that of water retention dam construction techniques (Martin, Davies et al. 2002). Today, raised

embankment construction is almost always mechanised to gain the level of compaction required to increase the safety and lower the risk of instability of the storage facility.

5.1.1 Upstream Method

The upstream method is the lowest initial cost and most popular design for a raised tailings embankment in low risk seismic areas. One of the reasons for this is mainly due to the minimal amount of fill material required for initial construction and subsequent raising which normally consists entirely of the coarse fraction of the tailings.

The construction of an upstream designed embankment starts with a pervious (free draining) starter dyke foundation. The tailings are usually discharged from the top of the dam crest creating a beach that becomes the foundation for future embankment raises (Vick 1990). Figure 2 shows a simplistic diagram of the stages of construction of an upstream raised embankment. Where the tailings properties are suitable, natural segregation of coarse material settles closest to the spigot and the fines furthest away (not always the case with thickened tailings discharge). Cyclones can be used to accelerate this particle segregation for certain tailings characteristics to send the slime proportion to the centre of the impoundment and the sand fraction to the beach behind the crest. The conventional method of upstream raises relies on no compaction of the spigotted beach that forms the embankment shell (Martin 1999). Today compaction by earthmoving equipment is common to increase the degree of safety of raised embankments. Generally the settled coarse fraction from the spigots/discharge point is used as the raise material for the embankments. For multiple spigot discharge a series of shallow pits are dug in front of the spigots (once the tailings have dried and consolidated) and tailings placed on the embankment crest, then they are compressed, the tailings lines lifted and reassembled then normal operation commences.

It is not surprising that the upstream method is the most common design to fail causing huge environmental consequences all over the world (ICOLD and UNEP 2001). Davies et al. (2000) note that there are reported to be just over 3500 tailings dams worldwide of which 50% are of the upstream design. It was also noted that the key failure mode of upstream embankments is a static/transient load induced liquefaction flowslide event. This is not surprising considering the low relative density of the tailings and the potential for water mismanagement to generate high saturation of the embankment and subsequently creating liquefaction induced flows of the tailings.

This coarse beach material is essential for upstream designed impoundments to aid drainage and prevent saturation of the embankments. This allows for a stronger and more permeable crest to develop which reduces the height of the phreatic surface as the embankment progressively rises. The best way to reduce the phreatic surface is to have a wide Beach above Water (BAW) between the dam crest and the supernatant pond (free water) within the impoundment (Shaheen, Martin et al. 2003). The closer the supernatant pond is to the dam crest, the higher the phreatic surface of the embankment and thus the greater the risk of failure. The filter under-drain system of the embankments is a key component in reducing the phreatic surface of an upstream designed embankment (ICOLD and UNEP 2001).

Upstream embankments are suited to areas where the climate is arid, minimal amounts of water require storage in the impoundment and rapid water accumulation is improbable (e.g. upstream water inundation and flooding). This helps to promote wide beaches and prevent frequent water level deviations that can dramatically alter pond geometry, freeboard and the phreatic surface within the impoundment area. Upstream embankments are not suited to areas of seismic activity as the risk of liquefaction increases as a result of the potential for dynamic loading by earthquakes. In some countries, for example Chile, upstream construction is not permitted for this reason.

Rates of rise of upstream embankments have to be controlled to prevent increased pore pressures that can reduce the shear strength of the fill material (Jakubick, McKenna et al. 2003). Excessive rates of rise have been a trigger for static liquefaction that has been the underlying cause for many upstream tailings impoundment failures (Davies, McRoberts et al. 2002).

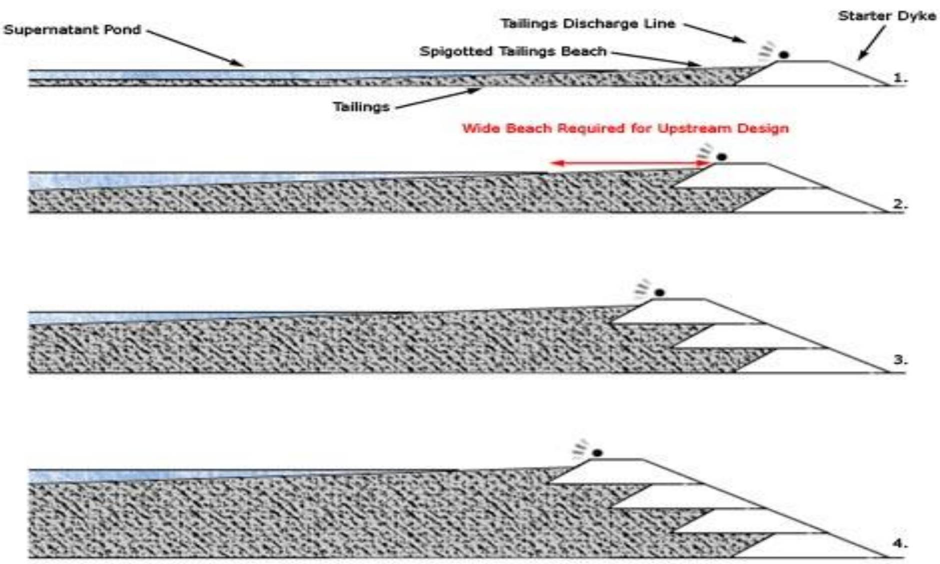

Figure 5.1 Upstream method of construction (Davies, McRoberts et al. 2002)

5.1.2 Downstream Method

The downstream design was developed to reduce the risks associated with the upstream design, particularly when subjected to dynamic loading as a result of earthquake shaking (ICOLD and UNEP 2001). The installation of impervious cores and drainage zones can also allow the impoundment to hold a substantial volume of water directly against the upstream face of the embankment without jeopardising stability.

The downstream embankment design starts with an impervious starter dyke unlike the upstream design that has a pervious starter dyke. The tailings are at first deposited behind the dyke and as the embankment is raised the new wall is constructed and supported on top of the downstream slope of the previous section (figure 3). This shifts the centreline of the top of the dam downstream as the embankment stages are progressively raised (Vick 1990). An advantage to the downstream design is that the raised sections can be designed to be of variable porosity to tackle any problems with the phreatic surface of the embankment. This can be particularly useful where a processing plant has made changes to increase efficiency and as a result alter the tailings characteristics. This may result in pumping more water to the tailings facility or alter the drainage characteristics of the newly deposited tailings.

The downstream design is very versatile for a range of site specific design parameters and behaves similarly to water retention dams. Their main advantage is that the downstream design can have unrestricted heights due to each raise being structurally independent of the tailings. The main disadvantage is the cost of raising the embankment as large volumes of fill are required which increases exponentially as embankment height increases, and subsequently a large area around the dam itself is required as the toe of the dam moves out as more raises are added. This can cause problems where limited space has been taken into consideration prior to building, or if property line and utilities are in close proximity. Although a downstream embankment can theoretically have no height limit, the dam's ultimate height is determined by the restriction of the advancing toe (Vick 1990).

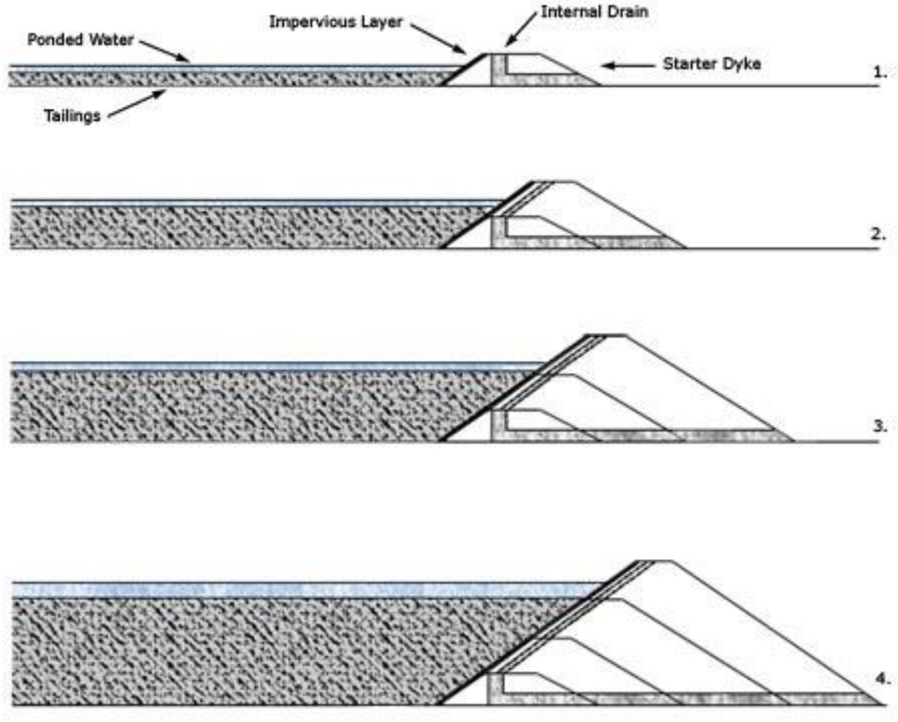

Figure 5.2 Downstream method of construction (Vick 1990)

5.1.3 Centreline Method

The centreline method is really a compromise between both the upstream and downstream designs (Benckert and Eurenius 2001). It is more stable than the upstream method but does not require as much construction material as the downstream design. Like the upstream method the tailings are generally discharged by spigots from the embankment crest to form a beach behind the dam wall. When subsequent raising is required, material is placed on both the tailings and the existing embankment. The embankment crest is being raised vertically and does not move in relation to the upstream and downstream directions of subsequent raises, hence the term, centreline design.

The design incorporates the internal drainage zones that are similarly found in the downstream method. Therefore, the free water can encroach closer to the dam crest than the upstream method without the worry of increasing the phreatic surface and causing a potential failure risk. However, a centreline dam cannot be used as a large water retention facility solely due to the subsequent raises being partially built on consolidated tailings. A suitable decant system needs to be installed to prevent the free water submerging the beach around the dam crest.

In many cases the centreline design is a good compromise between the seismic risk and the costs associated with construction (EC 2004).

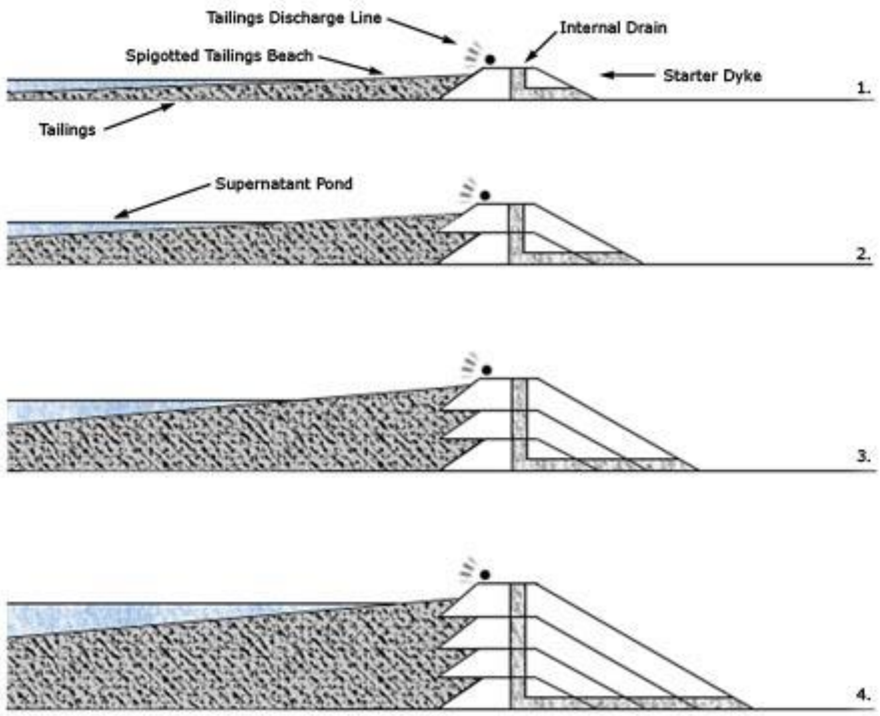

Fig 5.3 Centreline method of construction (EC 2004)

5.1.4 Modified Centreline Method

Modified centreline is a compromise between the upstream and centreline methods to reduce the volume of construction material placed in the downstream shell of the embankment. The angle of the upstream crest advance over the tailings is calculated during the design phase following stability and seepage analyses. Rockfill is usually utilised in this technique to gain a higher angle, rather than the coarse tailings fraction to reduce risk of instability.

In countries where upstream construction is not permitted (i.e. due to seismic risk), the modified centreline method may also not be permitted due to the concept of partially placing construction material on the existing tailings beach.

Chapter-6

Results and Discussion

The stability of natural slopes ware analysed for different parametric conditions by using Limit Equilibrium Methods (LEM) slope stability software Slope/W. Results from slope stability analysis are presented in Appendix A and Appendix B. Appendix A presents different figures drawn in Slope W software for different parametric conditions are as follows:

1. Eight different slopes are taken and factor of safety for different slope condition are calculated using Bishop Method, Price method and Janbu method.
2. The slopes taken are as 1:1.5, 1:2, 1:2.5, 1:3, 1:3.5, 1:4, 1:4.5, and 1:5.
3. Factor of safety have been calculated for stepwise raised embankment in three stages.
4. Factor of safety have been calculated for three different water level conditions.
5. For different values of cohesion and friction angle taken from different research paper have been enlisted.
6. The method of upstream has been employed in raised embankment.

Appendix B shows the safety factors calculated by slope/W utilizing the Morgenstern-Price methods, Ordinary method, modified Bishop Method and Janbu method .A comparison should be made for factor of safety for different parametric condition. As cohesive materials or clays generally possess less permeability compared to sand, thus, the movement of water is restricted whenever there is change in volume. So, for clay, it takes years to dissipate the excess pore water pressure before the effective equilibrium is reached. Shortly, drained condition refers to the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted. Besides, the drained and undrained condition of cohesive soils, it should be noted that there is a decline in strength of cohesive soils from its peak strength to its residual strength due to restructuring.

In the following diagrams drawn with the help of Slope W software, parametric study of stability of slopes have been analysed. The embankment cross section is drawn by upstream method.

6.1 Analysis of stability of embankments

6.1.1 Analysis for slope 1:1.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:1.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

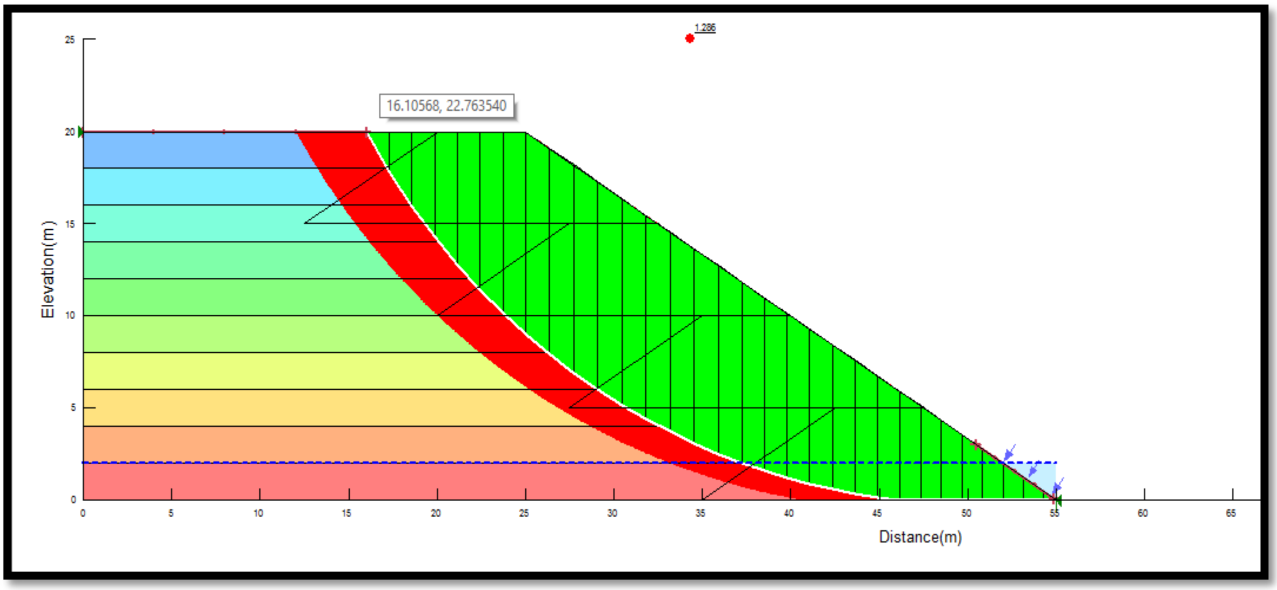

Figure 6.1 Embankment cross section for 1:1.5 slope

The critical slice information are as follows:

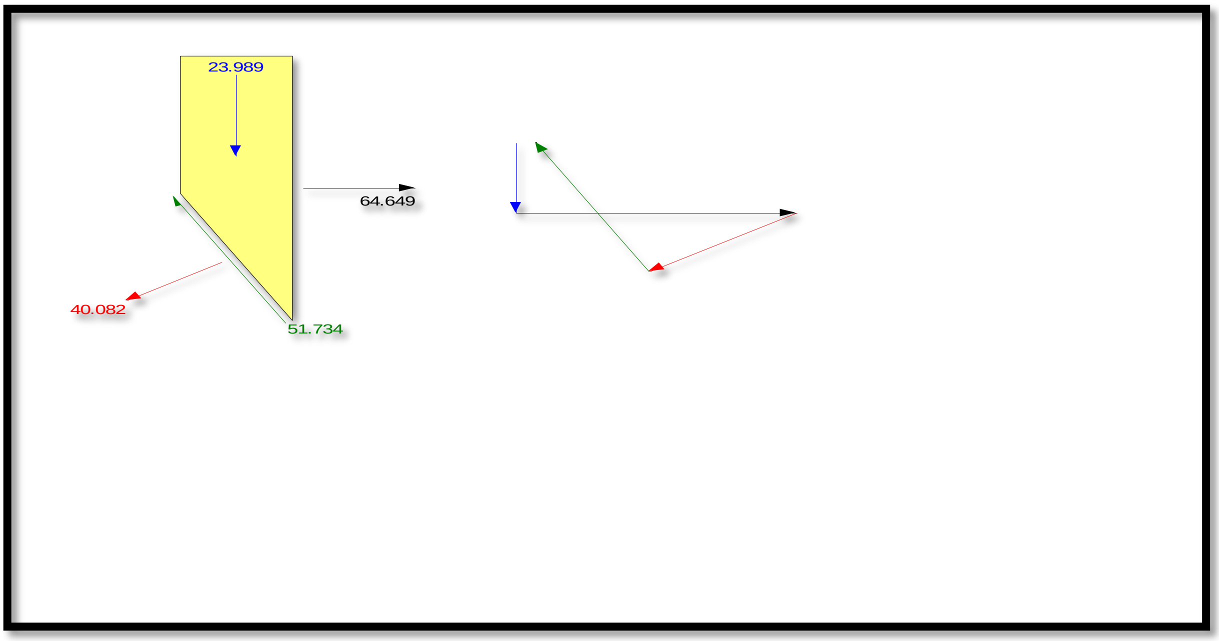

Figure 6.2 Critical slice for 1:1.5 slope

Slice 1 - Bishop Method	
Factor of Safety	1.2857
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	78.01 kN
Pore Water Pressure	-165.92 kPa
Pore Water Force	-417.53 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	1.2857 m
Mid-Height	1.0816 m
Base Length	2.5165 m

Base Angle	-59.275 °
Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	23.989 kN
Base Normal Force	-40.082 kN
Base Normal Stress	-15.928 kPa
Base Shear Res. Force	66.517 kN
Base Shear Res. Stress	26.433 kPa
Base Shear Mob. Force	51.734 kN
Base Shear Mob. Stress	20.558 kPa
Right Side Normal Force	-64.649 kN

The variation of frictional stress with slices are as follows:

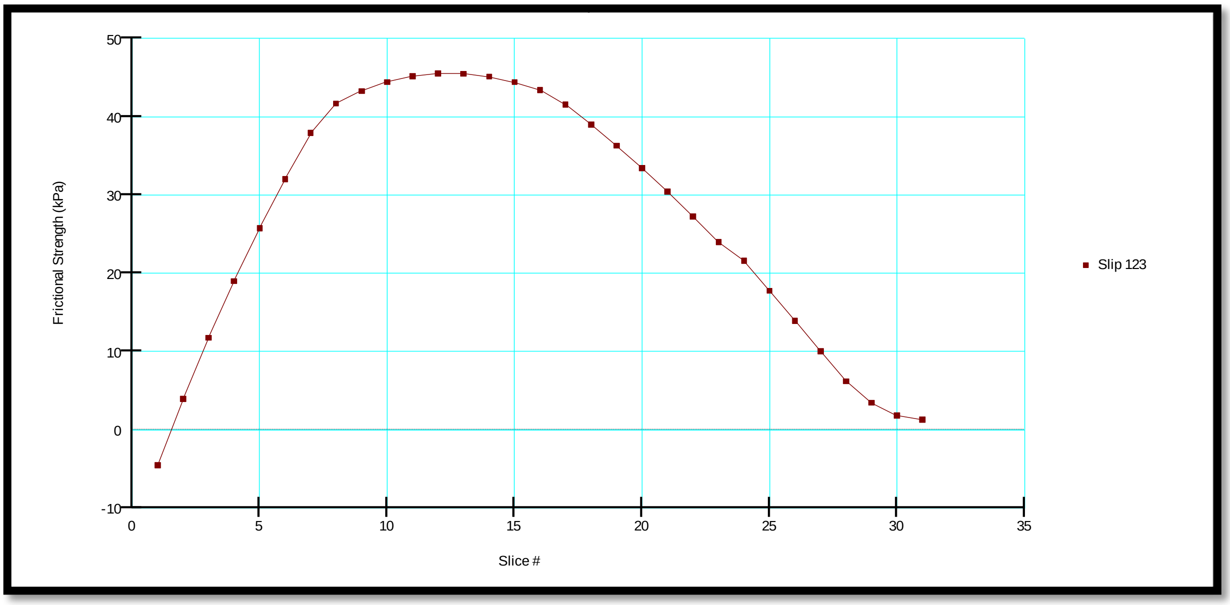

Figure 6.3 Variation of frictional stress with slices

6.1.2 Analysis for Slope 1:2

The embankment has the following parameters:

1. Height of the embankment: 20 m

2. Slope given on both side of embankment: 1:2
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

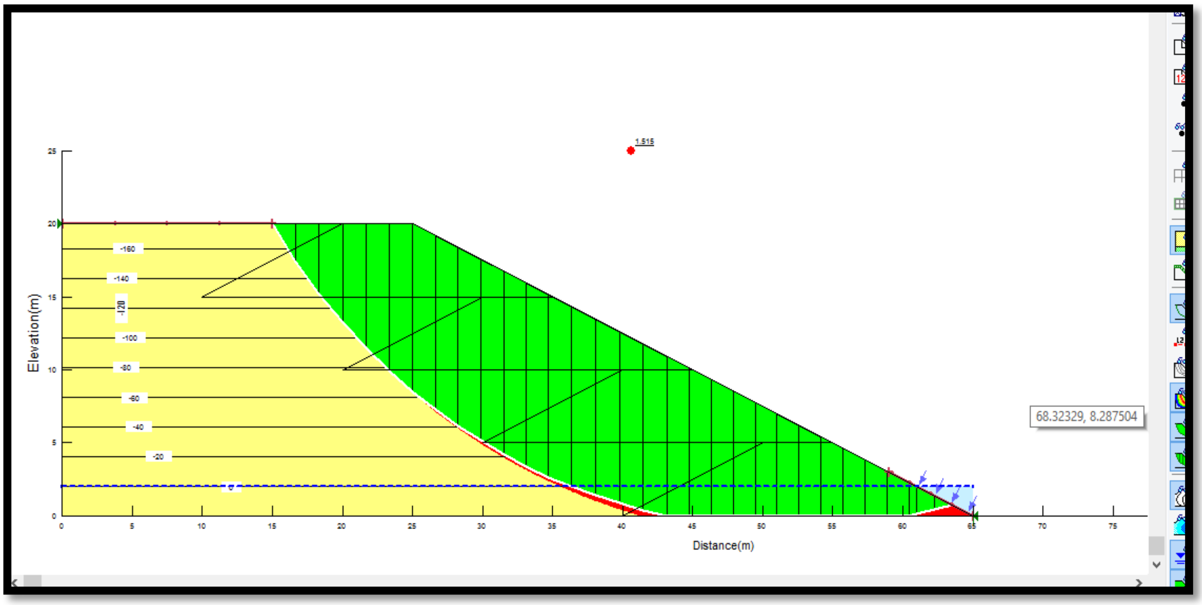

Figure 6.4 Embankment cross section for 1:2 slope

The critical slice information are as follows:

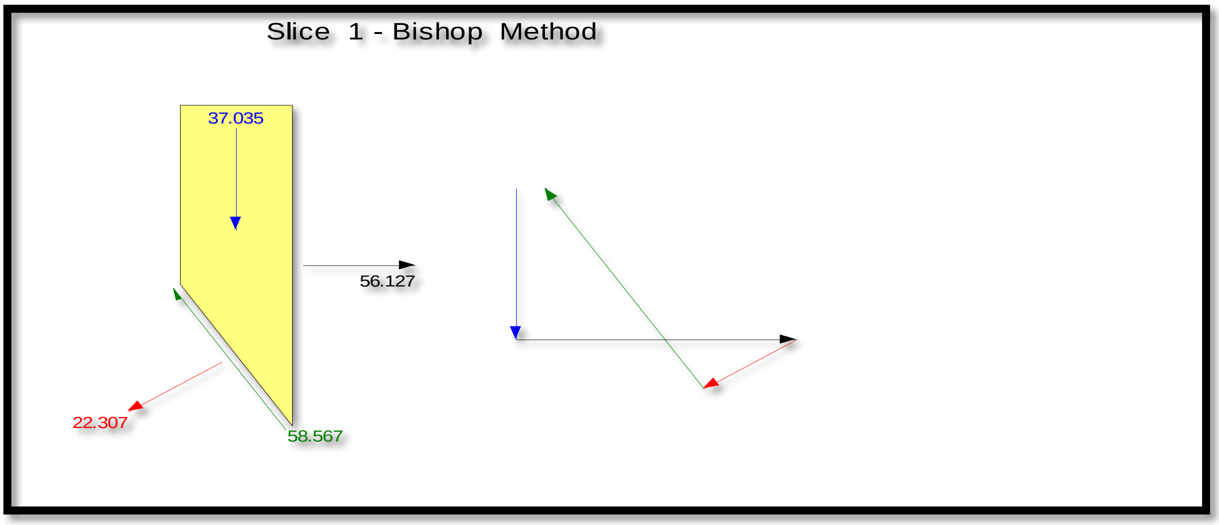

Figure 6.5 Critical slice for 1:2 slope

Slice 1 - Bishop Method	
Factor of Safety	1.5149
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	95.121 kN
Pore Water Pressure	-163.89 kPa
Pore Water Force	-502.89 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	1.6667 m
Mid-Height	1.2882 m
Base Length	3.0684 m
Base Angle	-57.101 °
Anisotropic Strength Mod.	1

Applied Lambda	994
Weight (incl. Vert. Seismic)	37.035 kN
Base Normal Force	-22.307 kN
Base Normal Stress	-7.2699 kPa
Base Shear Res. Force	88.725 kN
Base Shear Res. Stress	28.915 kPa
Base Shear Mob. Force	58.567 kN
Base Shear Mob. Stress	19.087 kPa
Right Side Normal Force	-56.127 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN
Point Load	0 kN
Reinforcement Load Used	0 kN
Reinf. Shear Load Used	0 kN
Surcharge Load	0 kN

The variation of frictional stress with slices are as follows:

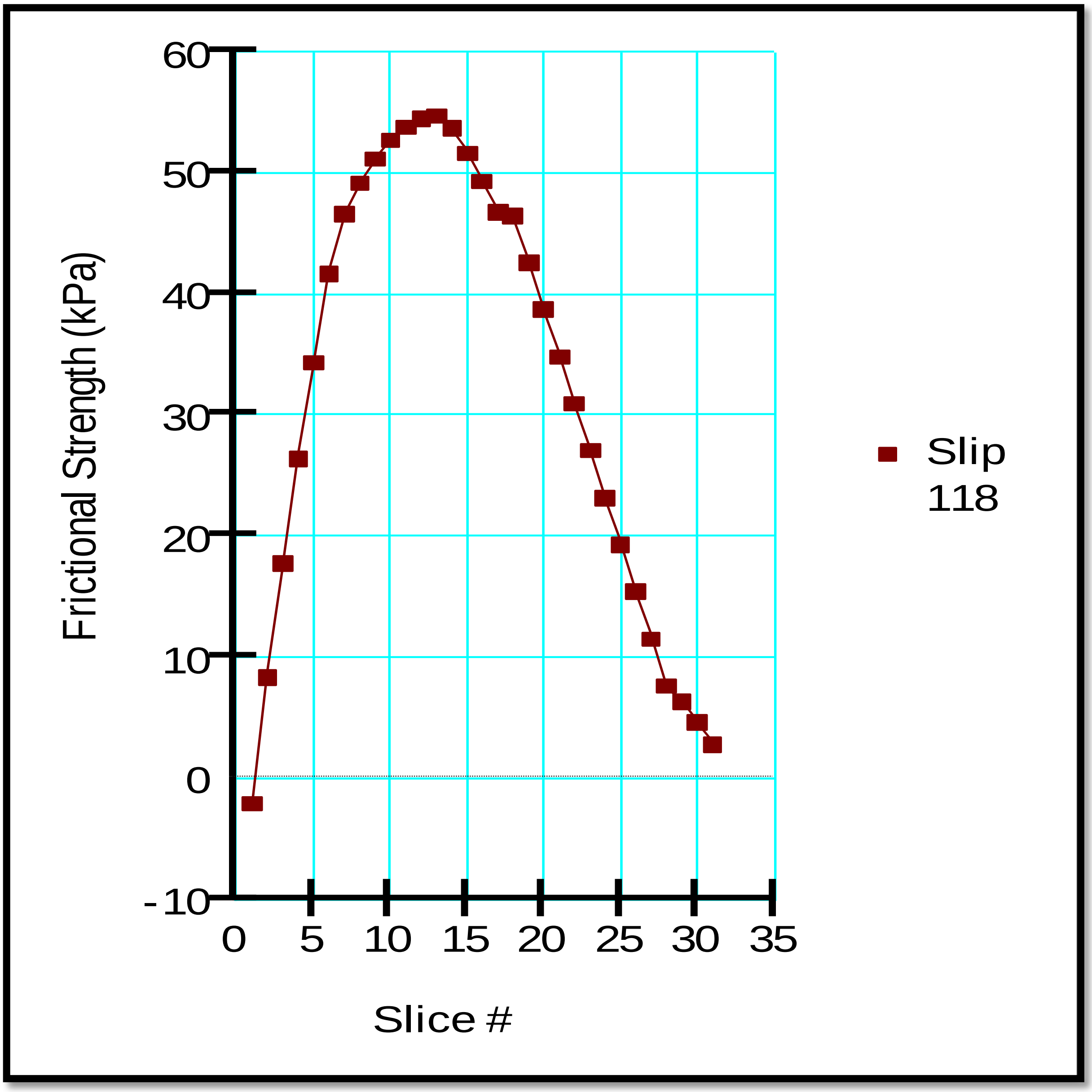

Figure 6.6 Variation of frictional stress with slices

6.1.3 Analysis for Slope 1:2.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:2.5

3. Cohesion : 31kPa

4. Friction Angle: 16^0

5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

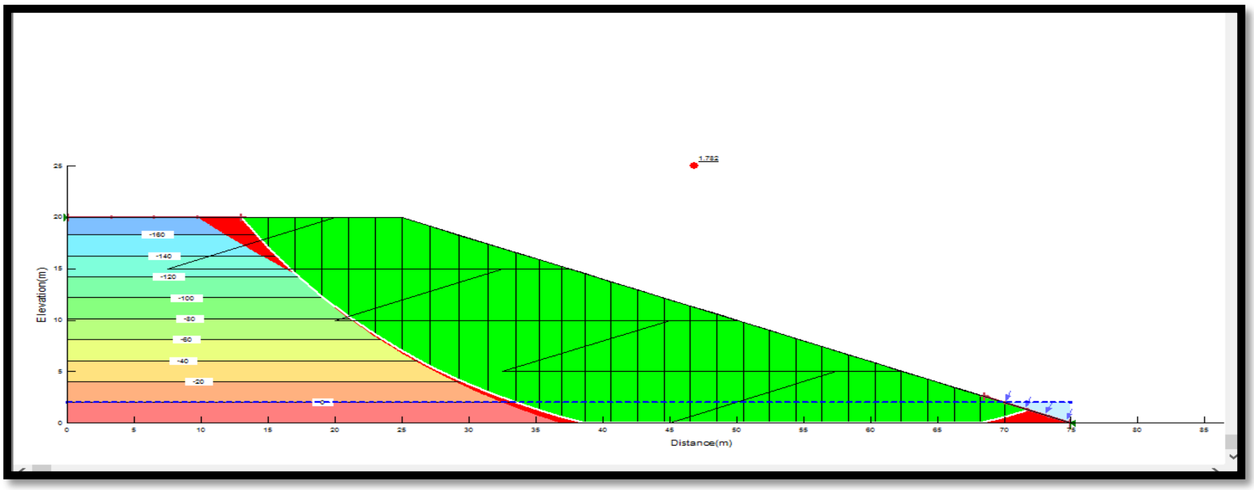

Figure 6.6 Embankment cross section for 1:2.5 slope

The critical slice information are as follows:

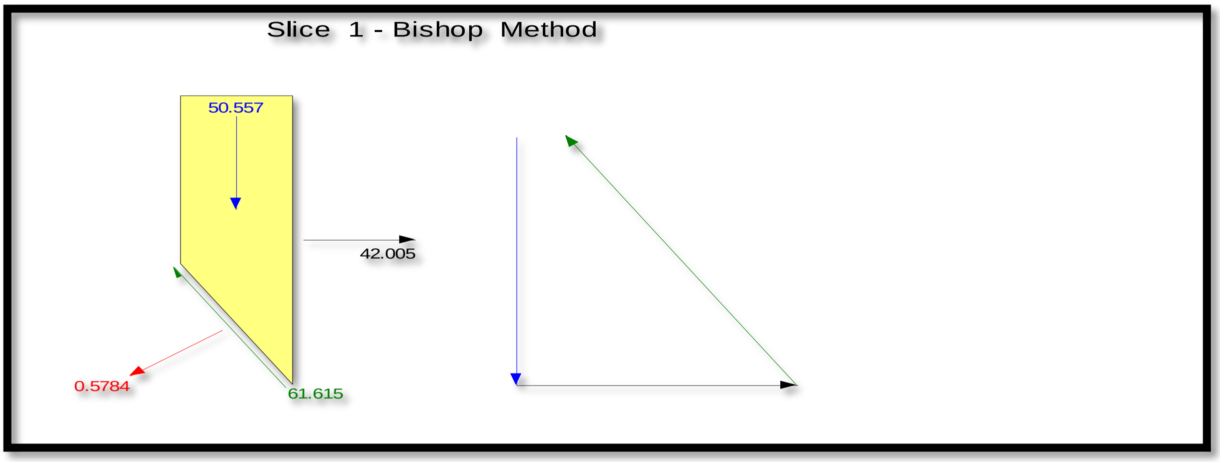

Figure 6.7 Critical slice for 1:2.5 slope

Slice 1 - Bishop Method

Factor of Safety	1.7825
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	110 kN
Pore Water Pressure	-162.15 kPa
Pore Water Force	0 kPa
Pore Air Pressure	
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	2 m
Mid-Height	1.4654 m
Base Length	3.5482 m
Base Angle	-55.691 °
Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	50.557 kN
Base Normal Force	-0.5784 kN
Base Normal Stress	-0.16301 kPa
Base Shear Res. Force	109.83 kN
Base Shear Res. Stress	30.953 kPa
Base Shear Mob. Force	61.615 kN
Base Shear Mob. Stress	17.365 kPa
Right Side Normal Force	-42.005 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN
Point Load	0 kN

Reinforcement Load Used	0 kN
Reinf. Shear Load Used	0 kN
Surcharge Load	0 kN

The variation of frictional stress with slices are as follows:

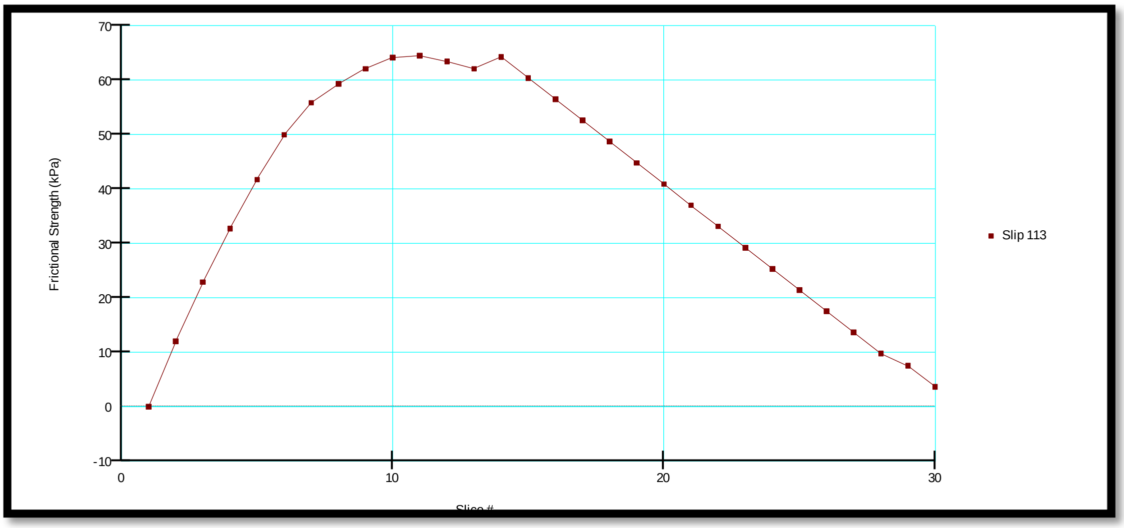

Figure 6.8 Variation of frictional stress with slices

6.1.4 Analysis for Slope 1:3

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:3
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

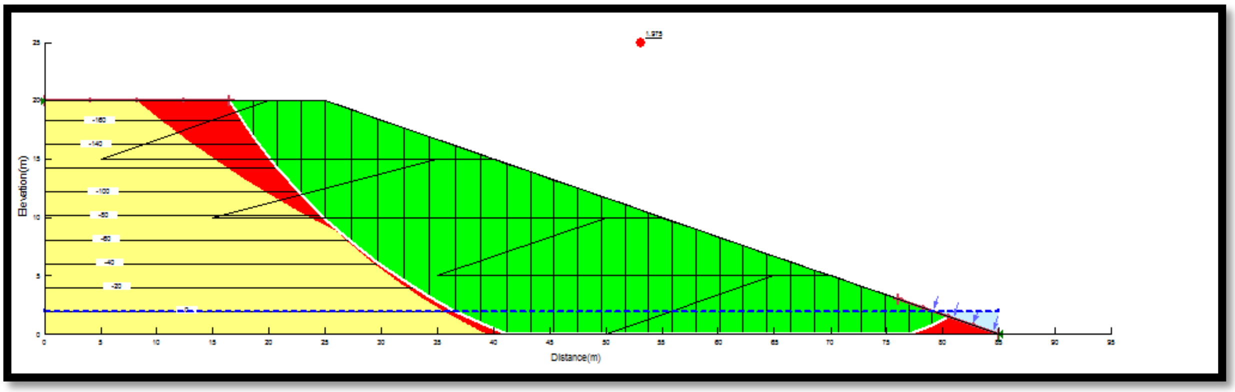

Figure 6.9 Embankment cross section for 1:3 slope

The critical slice information are as follows:

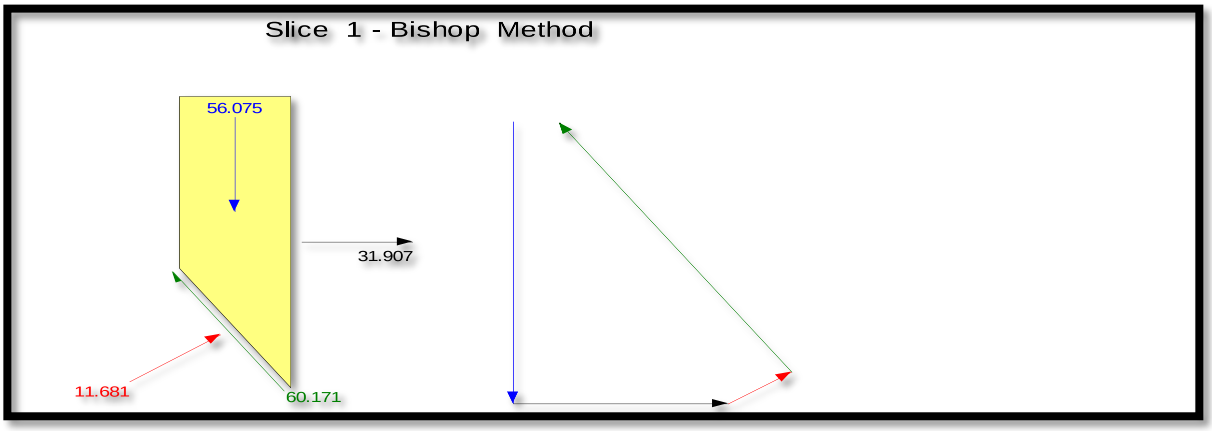

Figure 6.10 Critical slice for 1:3 slope

Slice 1 - Bishop Method	
Factor of Safety	1.9748
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	115.48 kN
Pore Water Pressure	-161.52 kPa
Pore Water Force	-601.69 kN
Pore Air Pressure	0 kPa

Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	2.125 m
Mid-Height	1.5298 m
Base Length	3.7251 m
Base Angle	-55.218 °
Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	56.075 kN
Base Normal Force	11.681 kN
Base Normal Stress	3.1357 kPa
Base Shear Res. Force	118.83 kN
Base Shear Res. Stress	31.899 kPa
Base Shear Mob. Force	60.171 kN
Base Shear Mob. Stress	16.153 kPa
Right Side Normal Force	-31.907 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN
Point Load	0 kN
Reinforcement Load Used	0 kN
Reinf. Shear Load Used	0 kN
Surcharge Load	0 kN

The variation of frictional stress with slices are as follows:

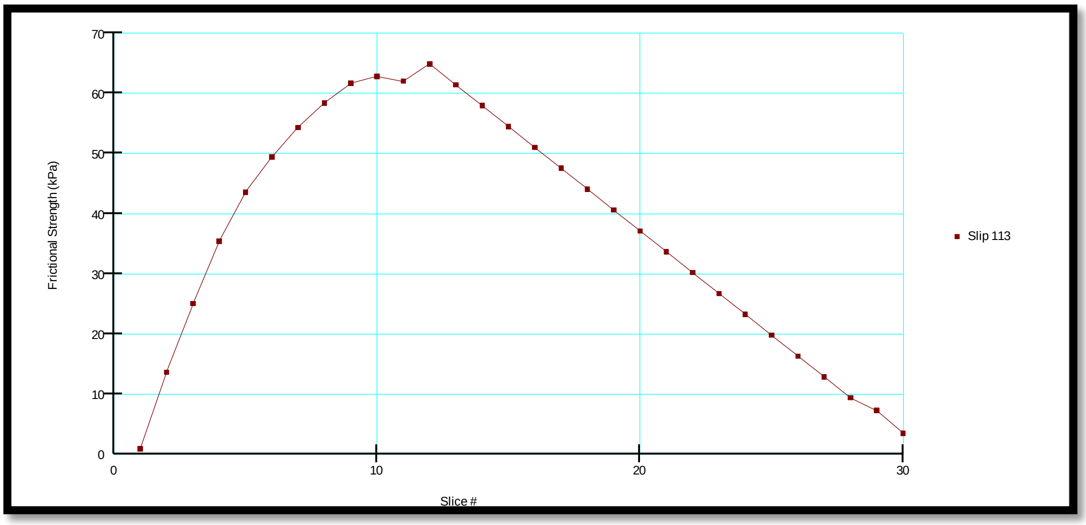

Figure 6.11 Variation of frictional stress with slices

6.1.5 Analysis for Slope 1:3.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:3.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

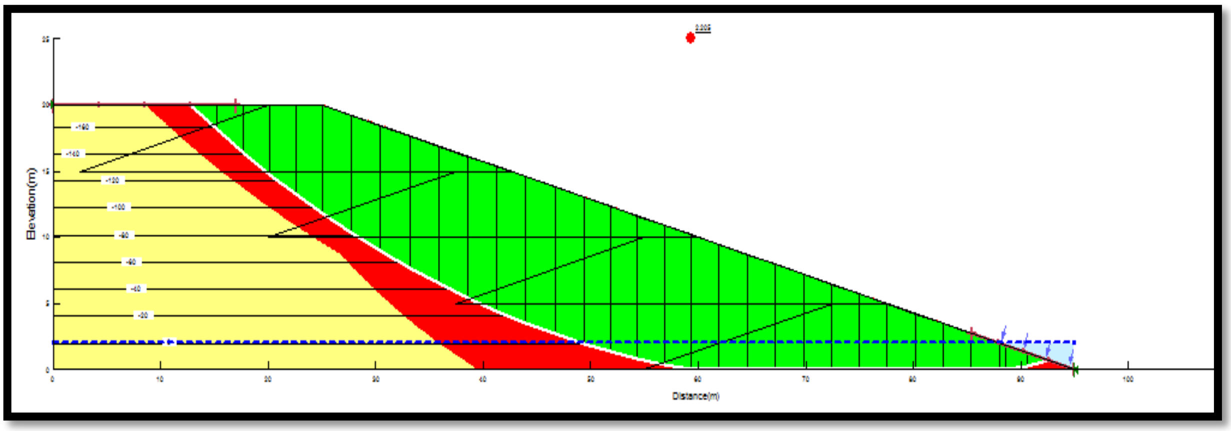

Figure 6.12 Embankment cross section for 1:3.5 slope

The critical slice information are as follows:

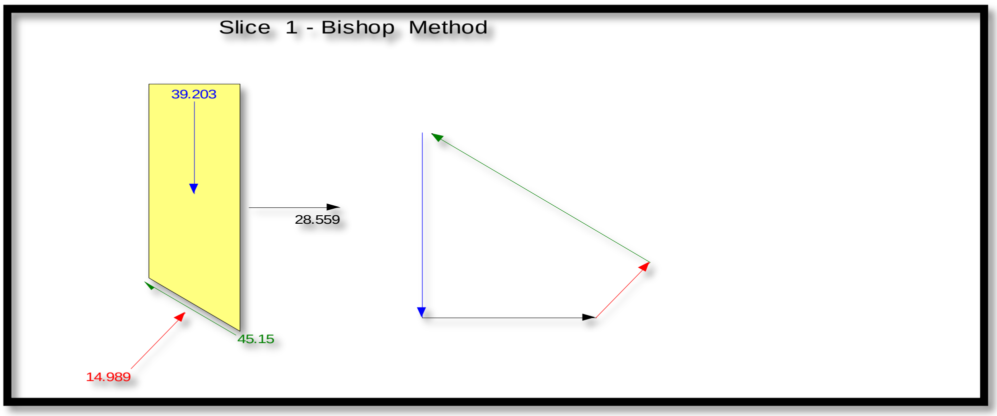

Figure 6.13 Critical slice for 1:1.5 slope

Slice 1 - Bishop Method	
Factor of Safety	2.2052
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	95.268 kN
Pore Water Pressure	-167.43 kPa
Pore Water Force	-514.53 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	2.45 m
Mid-Height	0.9276 m
Base Length	3.0732 m
Base Angle	-37.134 °
Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	39.203 kN

Base Normal Force	14.989 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Base Normal Stress	4.8775 kPa
Base Shear Res. Force	99.566 kN
Base Shear Res. Stress	32.399 kPa
Base Shear Mob. Force	45.15 kN
Base Shear Mob. Stress	14.692 kPa
Right Side Normal Force	-28.559 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN
Point Load	0 kN
Reinforcement Load Used	0 kN
Reinf. Shear Load Used	0 kN
Surcharge Load	0 kN
Polygon Closure	1.5802 kN
Top Left Coordinate	12.75, 20 m
Top Right Coordinate	15.2, 20 m
Base Normal Stress	4.8775 kPa
Base Shear Res. Force	99.566 kN
Base Shear Res. Stress	32.399 kPa
Base Shear Mob. Force	45.15 kN
Base Shear Mob. Stress	14.692 kPa
Bottom Left Coordinate	12.75, 20 m
Bottom Right Coordinate	15.2, 18.1448 m

The variation of frictional stress with slices are as follows:

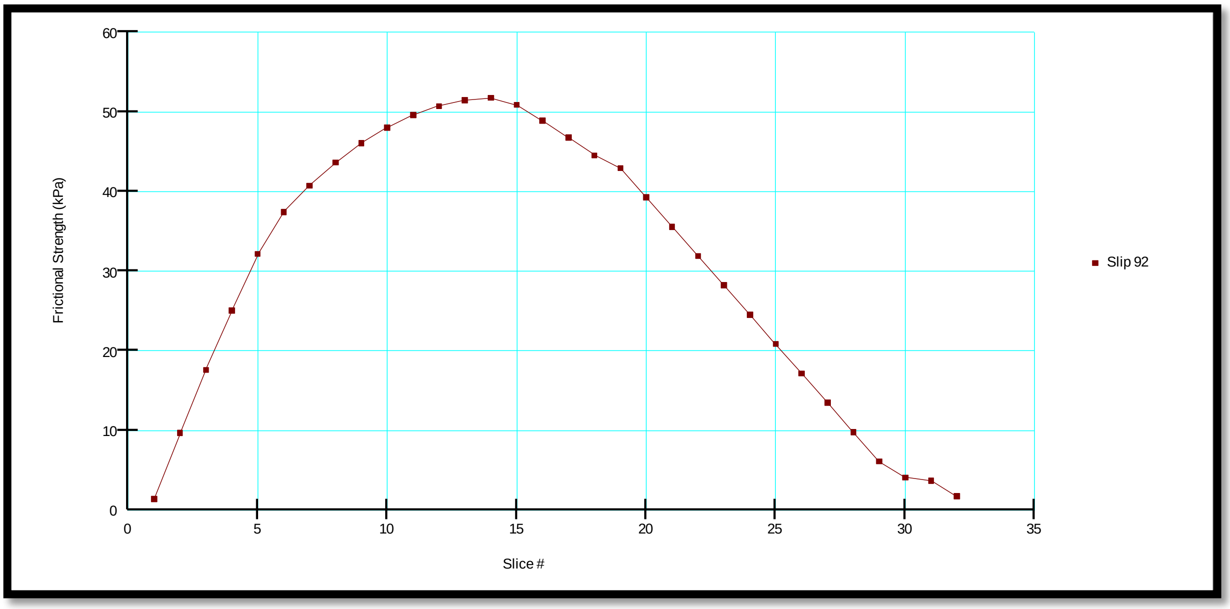

Figure 6.14 Variation of frictional stress with slices

6.1.6 Analysis for Slope 1:4

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:4
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

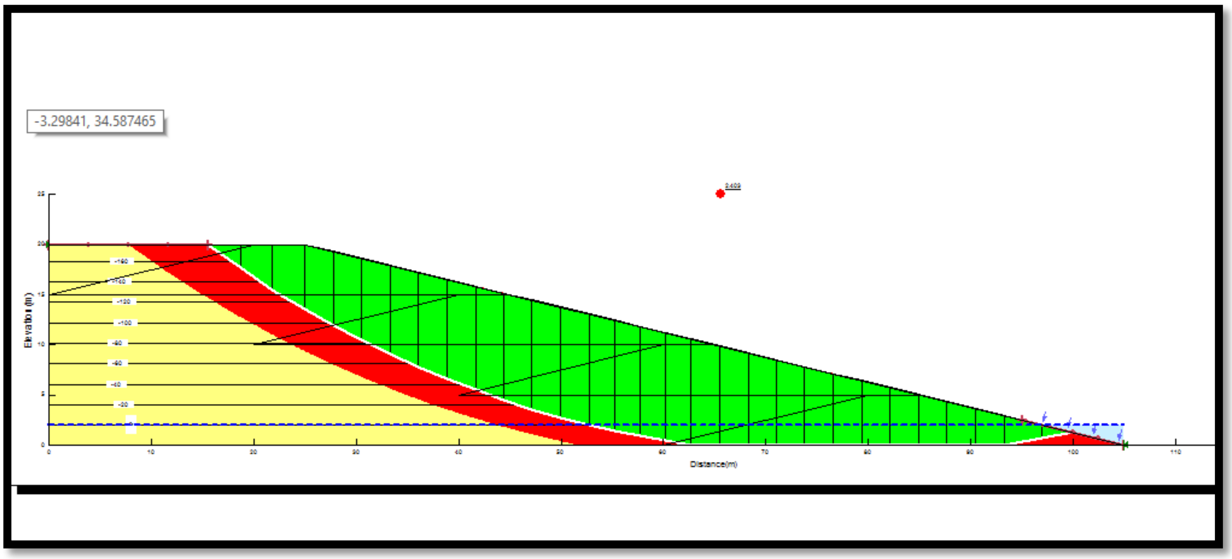

Figure 6.15 Embankment cross section for 1:4 slope

The critical slice information are as follows:

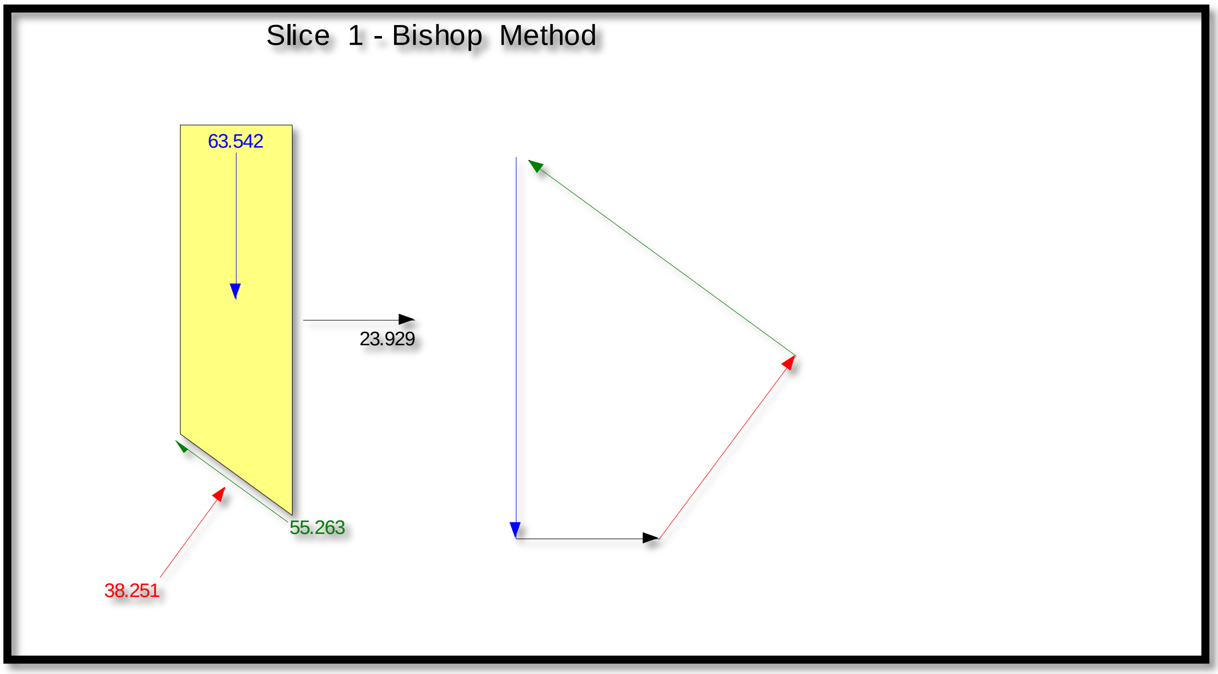

Figure 6.16 Critical slice for 1:4 slope

Slice 1 - Bishop Method	
Factor of Safety	2.4027
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	121.81 kN
Pore Water Pressure	-165.12 kPa
Pore Water Force	-648.82 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	3.1667 m
Mid-Height	1.1632 m
Base Length	3.9294 m
Base Angle	-36.304 °

Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	63.542 kN
Base Normal Force	38.251 kN
Base Normal Stress	9.7344 kPa
Base Shear Res. Force	132.78 kN
Base Shear Res. Stress	33.791 kPa
Base Shear Mob. Force	55.263 kN
Base Shear Mob. Stress	14.064 kPa
Right Side Normal Force	-23.929 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN
Point Load	0 kN
Reinforcement Load Used	0 kN
Reinf. Shear Load Used	0 kN
Surcharge Load	0 kN
Polygon Closure	2.224 kN
Top Left Coordinate	15.5, 20 m
Top Right Coordinate	18.666667, 20 m
Bottom Left Coordinate	15.5, 20 m
Bottom Right Coordinate	18.666667, 17.673514 m
Factor of Safety	2.4027
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	121.81 kN
Pore Water Force	-648.82 kN
Pore Air Pressure	0 kPa

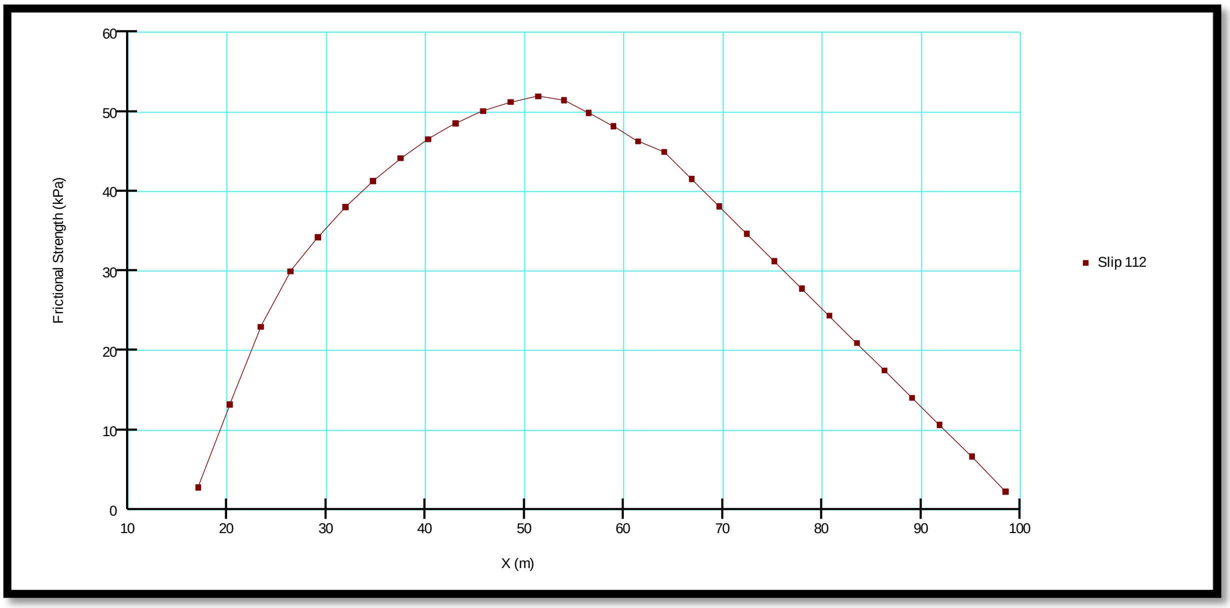

Figure 6.17 Variation of frictional stress with slices

6.1.7 Analysis for Slope 1:4.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:4.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m^3

The cross section is made on Slope W software and the analysis results are as follows:

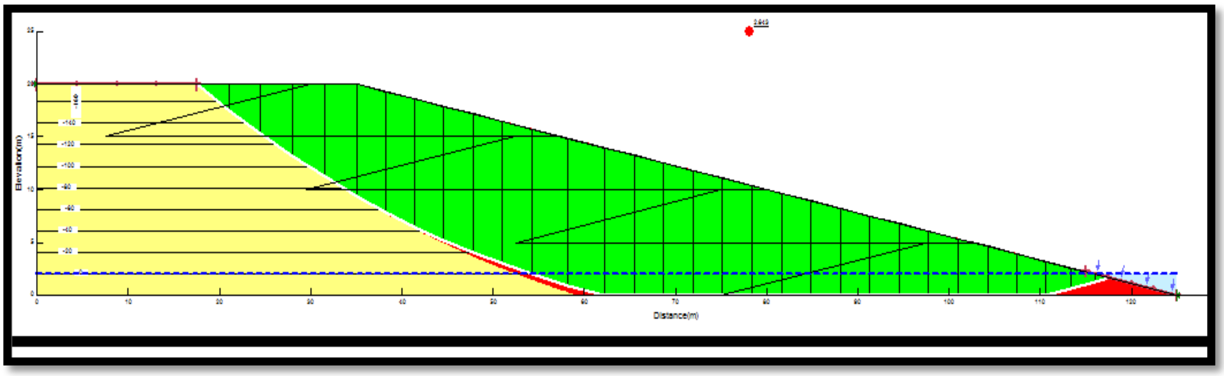

Figure 6.18 Embankment cross section for 1:4.5 slope

The critical slice information are as follows:

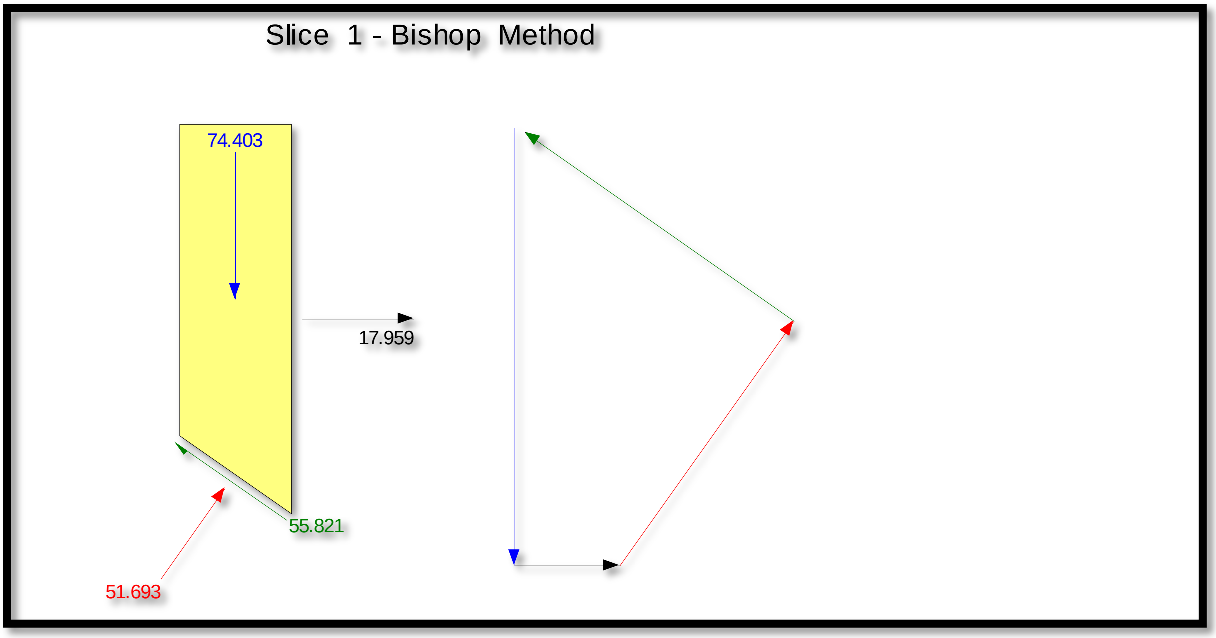

Figure 6.19 Critical slice for 1:4.5 slope

Slice 1 - Bishop Method

Factor of Safety	2.6429
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	132.7 kN
Pore Water Pressure	-164.44 kPa
Pore Water Force	-703.93 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	3.5 m
Mid-Height	1.2323 m
Base Length	4.2807 m
Base Angle	-35.153 °
Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	74.403 kN
Base Normal Force	51.693 kN
Base Normal Stress	12.076 kPa
Base Shear Res. Force	147.53 kN
Base Shear Res. Stress	34.463 kPa
Base Shear Mob. Force	55.821 kN
Base Shear Mob. Stress	13.04 kPa
Right Side Normal Force	-17.959 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN

Point Load	0 kN
Reinforcement Load Used	0 kN
Reinf. Shear Load Used	0 kN
Surcharge Load	0 kN
Polygon Closure	1.8601 kN
Top Left Coordinate	17.5, 20 m
Top Right Coordinate	21, 20 m
Bottom Left Coordinate	17.5, 20 m
Bottom Right Coordinate	21, 17.535312 m
Factor of Safety	2.6429
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	132.7 kN
Pore Water Pressure	-164.44 kPa
Pore Water Force	-703.93 kN

The variation of frictional stress with slices are as follows:

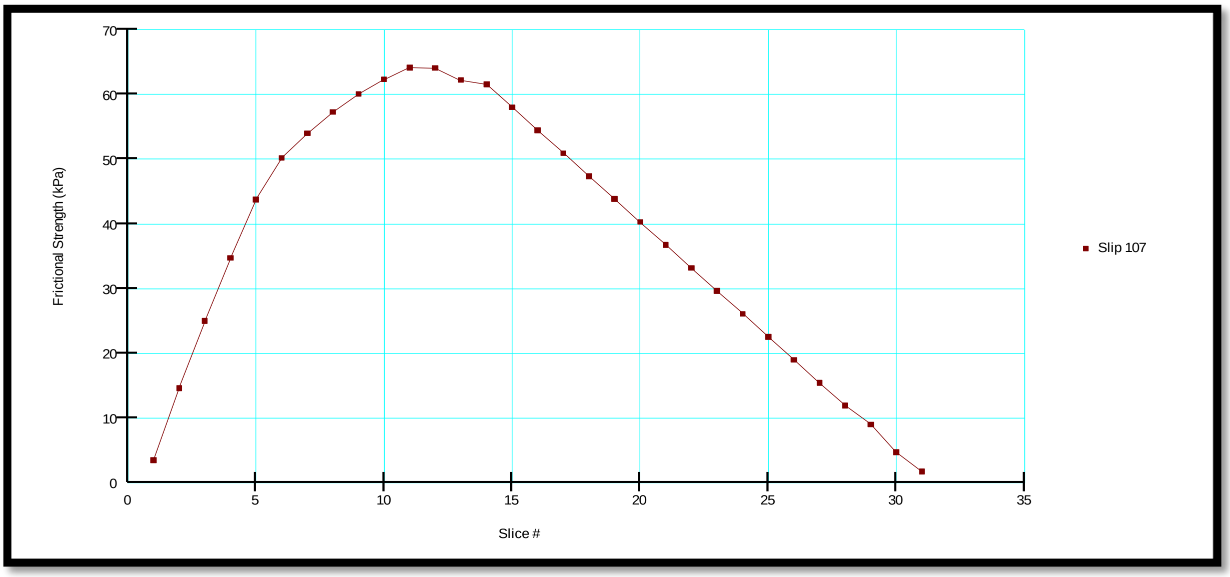

Figure 6.20 Variation of frictional stress with slices

6.1.8 Analysis for Slope 1:5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

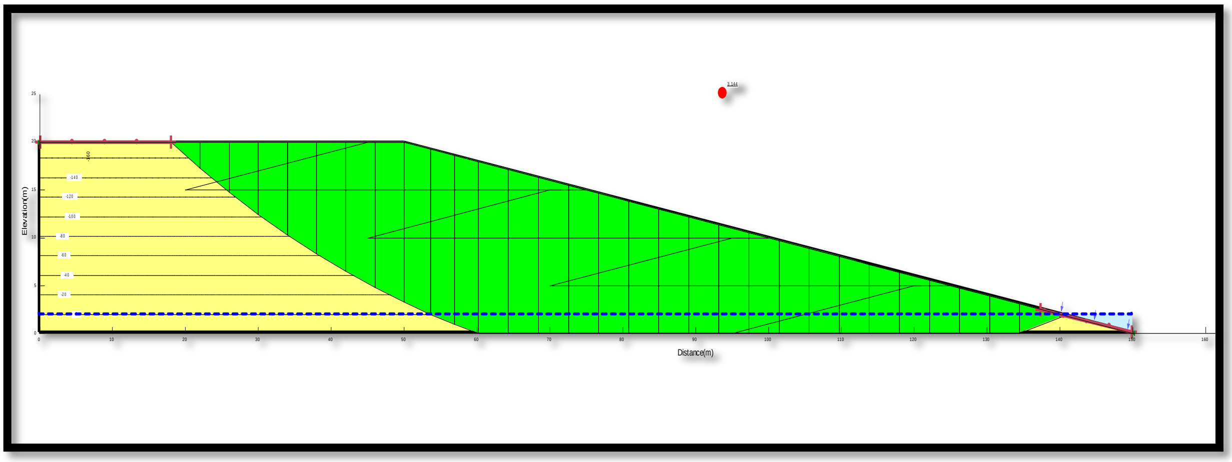

Figure 6.21 Embankment cross section for 1:5 slope

The critical slice information are as follows:

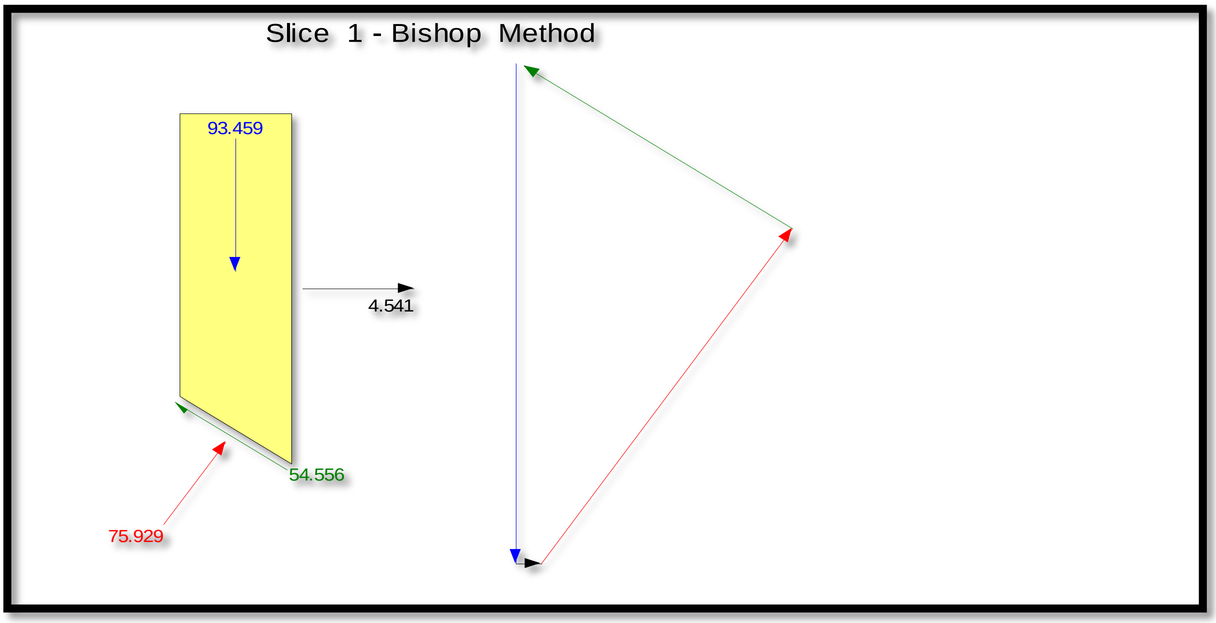

Figure 6.22 Critical slice for 1:5 slope

Slice 1 - Bishop Method

Factor of Safety	3.1442
Phi Angle	16 °
C (Strength)	31 kPa
C (Force)	149.76 kN
Pore Water Pressure	-163.24 kPa
Pore Water Force	-788.62 kN
Pore Air Pressure	0 kPa
Pore Air Force	0 kN
Phi B Angle	0 °
Slice Width	4 m
Mid-Height	1.3545 m
Base Length	4.831 m
Base Angle	-34.107 °

Anisotropic Strength Mod.	1
Applied Lambda	994
Weight (incl. Vert. Seismic)	93.459 kN
Base Normal Force	75.929 kN
Base Normal Stress	15.717 kPa
Base Shear Res. Force	171.53 kN
Base Shear Res. Stress	35.507 kPa
Base Shear Mob. Force	54.556 kN
Base Shear Mob. Stress	11.293 kPa
Right Side Normal Force	-4.541 kN
Right Side Shear Force	0 kN
Horizontal Seismic Force	0 kN
Point Load	0 kN

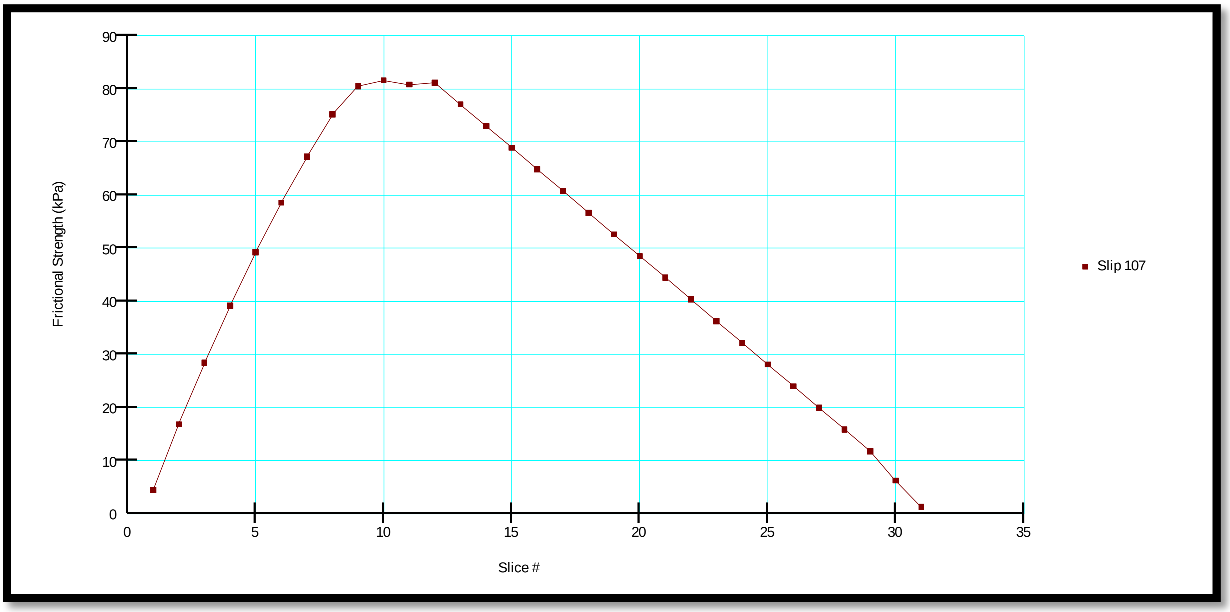

Figure 6.23 Critical slice for 1:5 slope

From above variation of slopes different factor of safety has been obtained. The variation of factor of safety with slopes are as follows

Table 6.1 Variation of FOS with slope of embankment

Slope of embankment	FOS
1:1.5	1.286
1:2	1.515
1:2.5	1.783
1:3	1.975
1:3.5	2.205
1:4	2.403
1:4.5	2.643
1:5	3.144

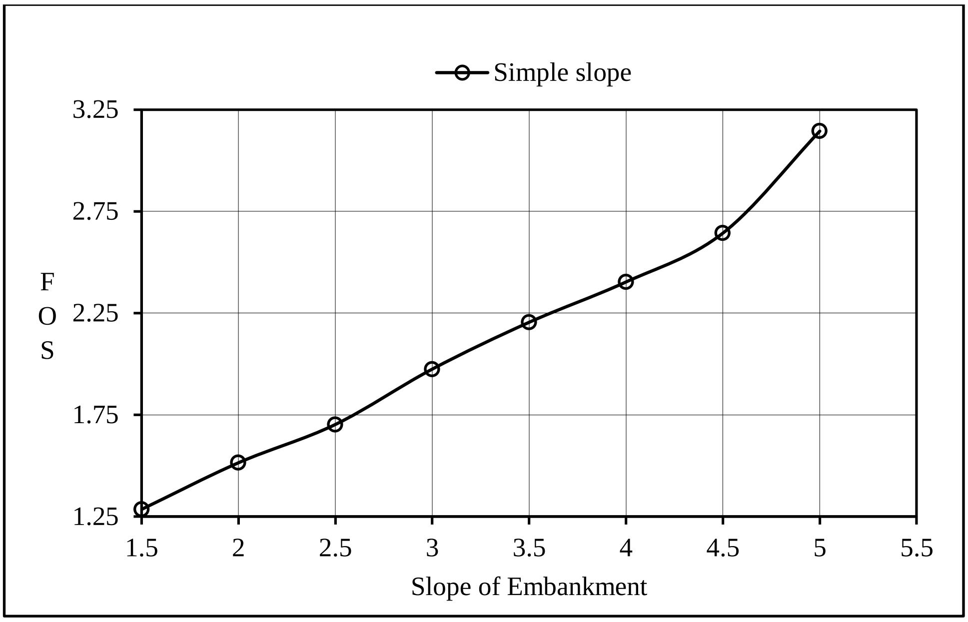

Figure 6.24 Variation of FOS with slope of embankment

6.2 Analysis of Step Slopes

6.2.1 Analysis for Slope 1:1.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:1.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

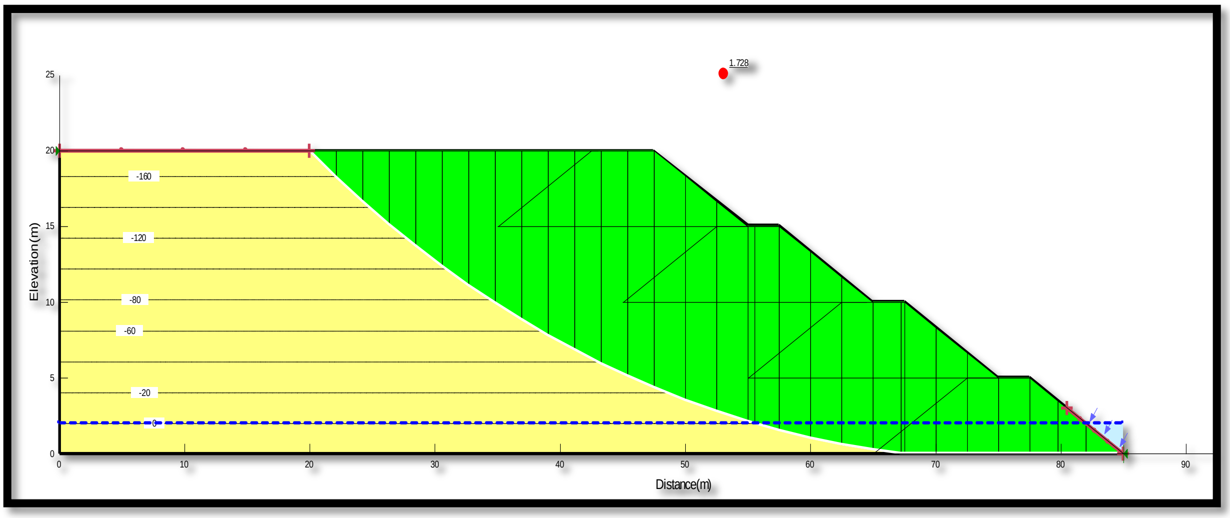

Figure 6.25 Critical slice for 1:1.5 step slope

The critical slice information are as follows:

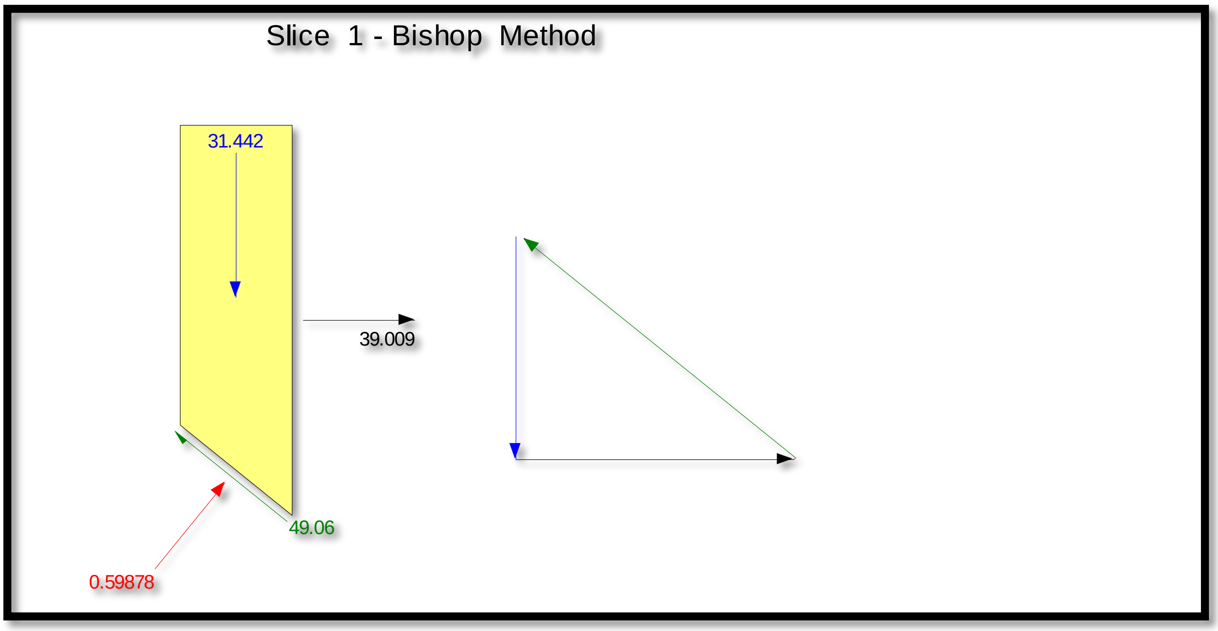

Figure 6.26 Critical slice for 1:1.5 step slope

The variation of frictional stress with slices are as follows:

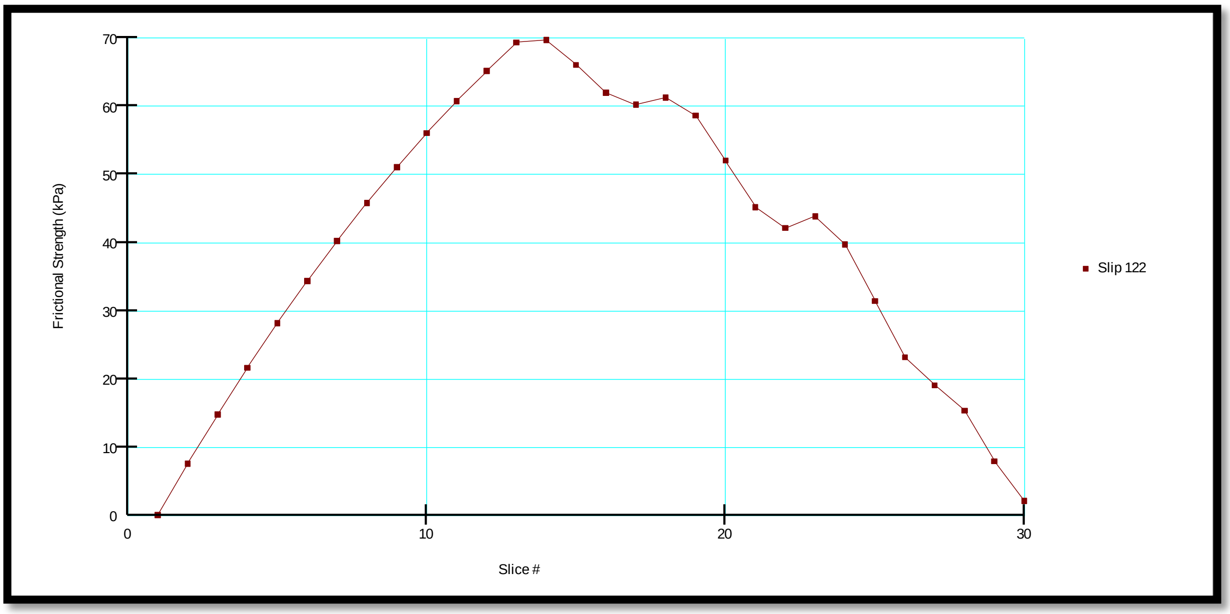

Figure 6.27 Variation of frictional stress with slices

6.2.2 Analysis for Slope 1:2

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:2
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

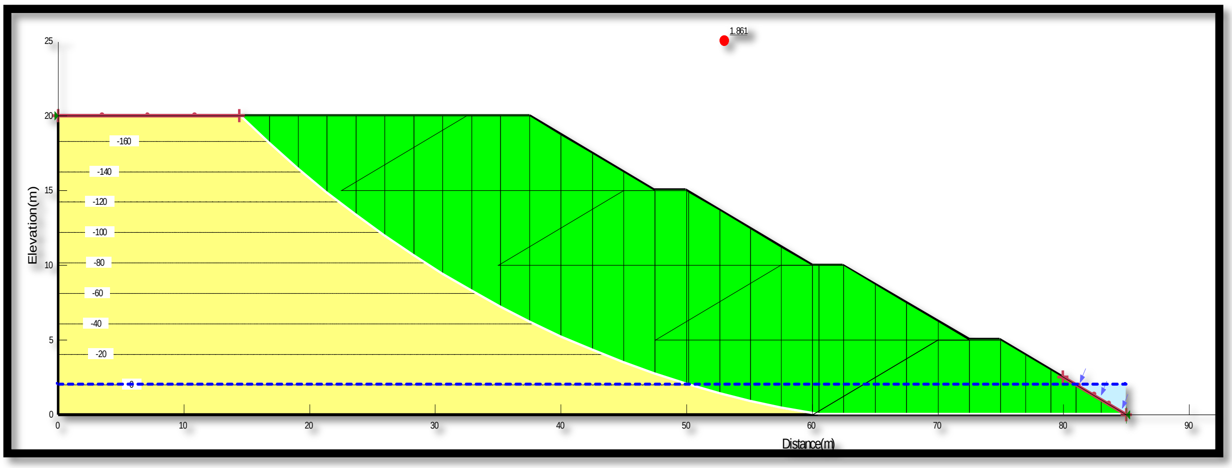

Figure 6.28 Embankment cross section for 1:2 step slope

The critical slice information are as follows:

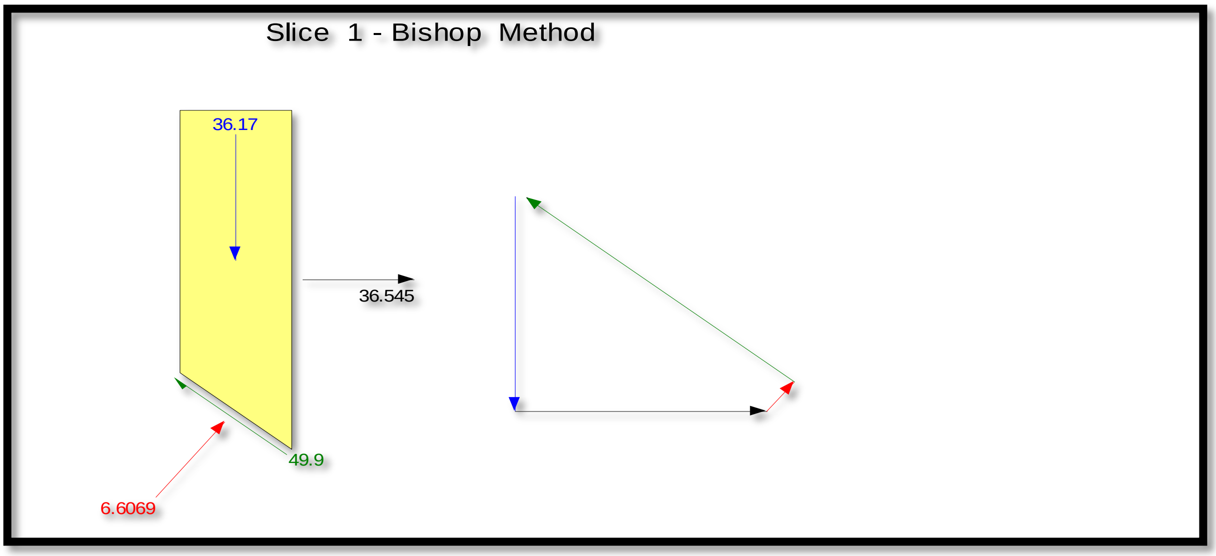

Figure 6.29 Critical slice for 1:2 slope

The variation of frictional stress with slices are as follows:

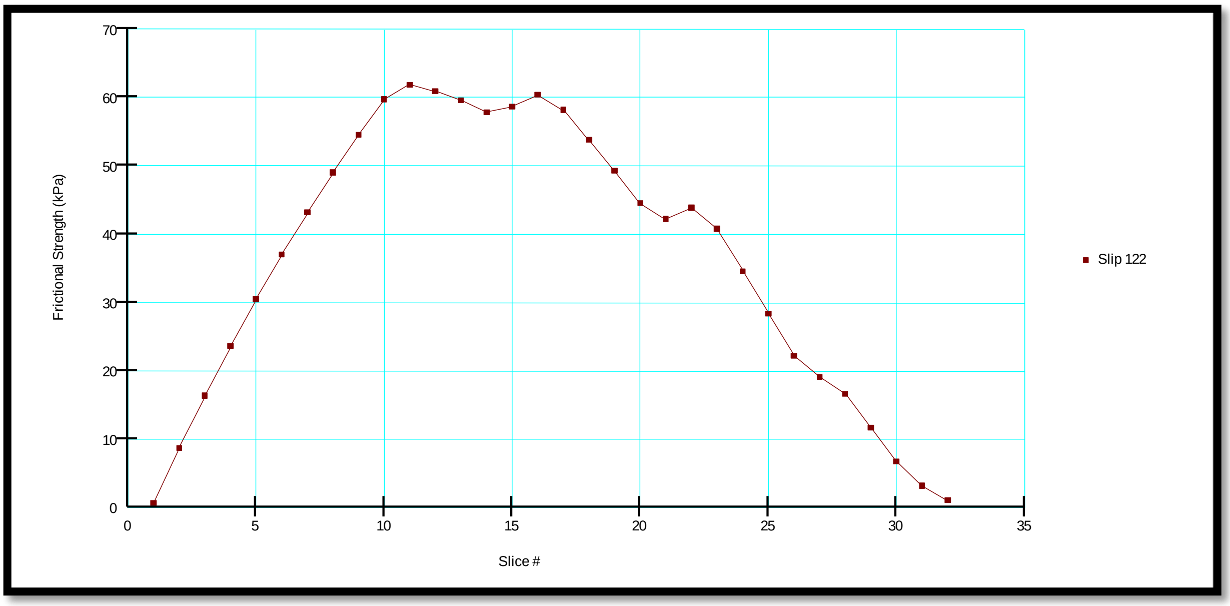

Figure 6.30 Variation of frictional stress with slices

6.2.3 Analysis for Slope 1:2.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:2.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

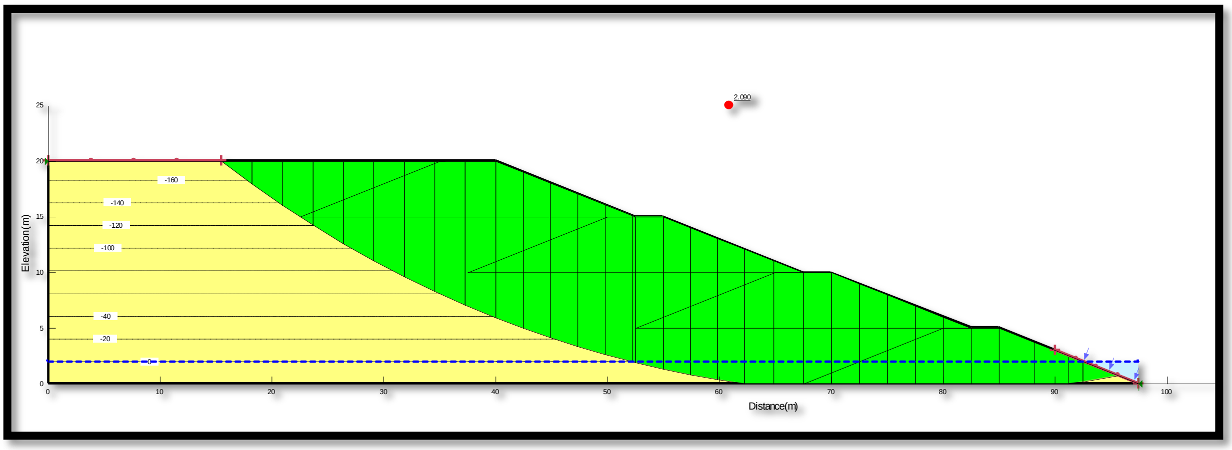

Figure 6.31 Embankment cross section for 1:2.5 step slope

The critical slice information are as follows:

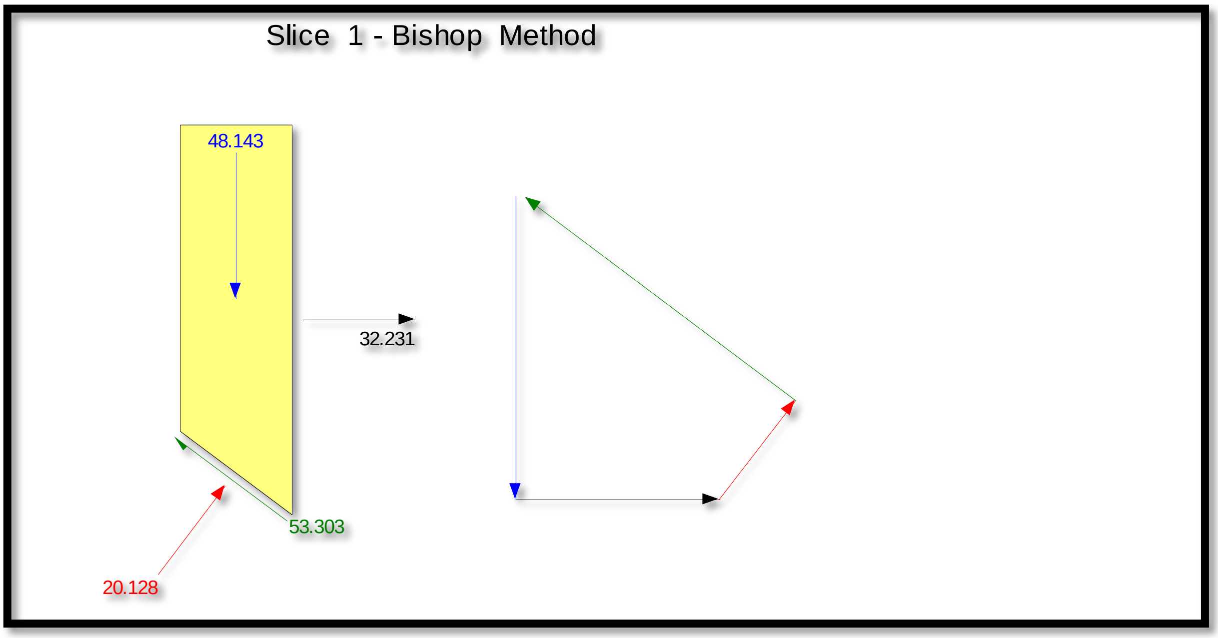

Figure 6.32 Critical slice for 1:2.5 step slope

The variation of frictional stree with slices are as follows:

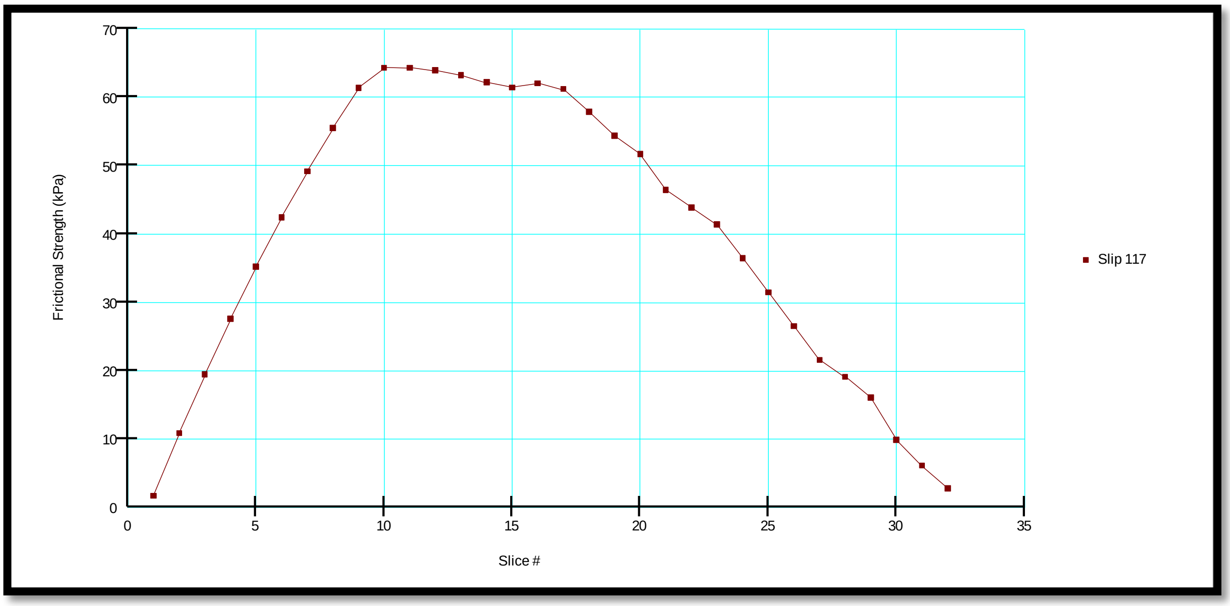

Figure 6.33 Variation of frictional stress with slices

6.2.4 Analysis for Slope 1:3

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:3
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

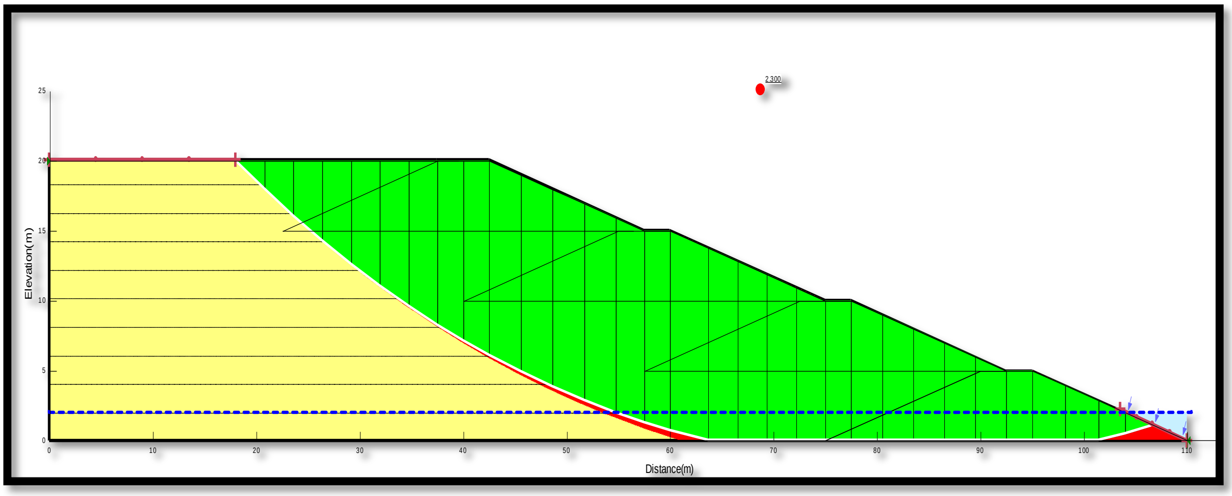

Figure 6.34 Embankment cross section for 1:3 step slope

The critical slice information are as follows:

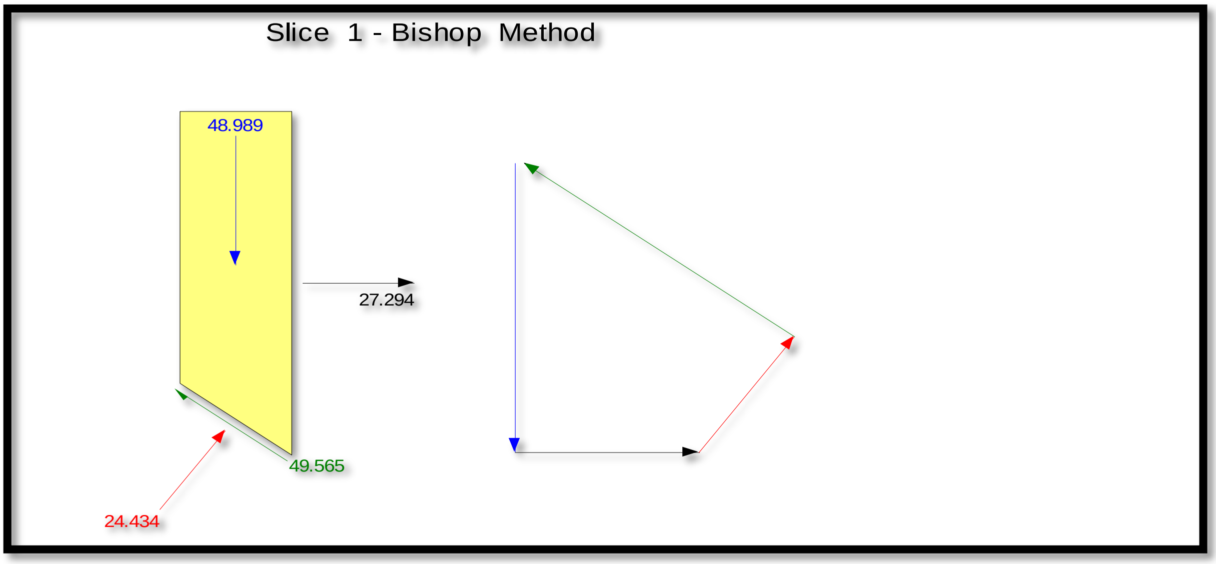

Figure 6.35 Critical slice for 1:3 step slope

The variation of frictional stress with slices are as follows:

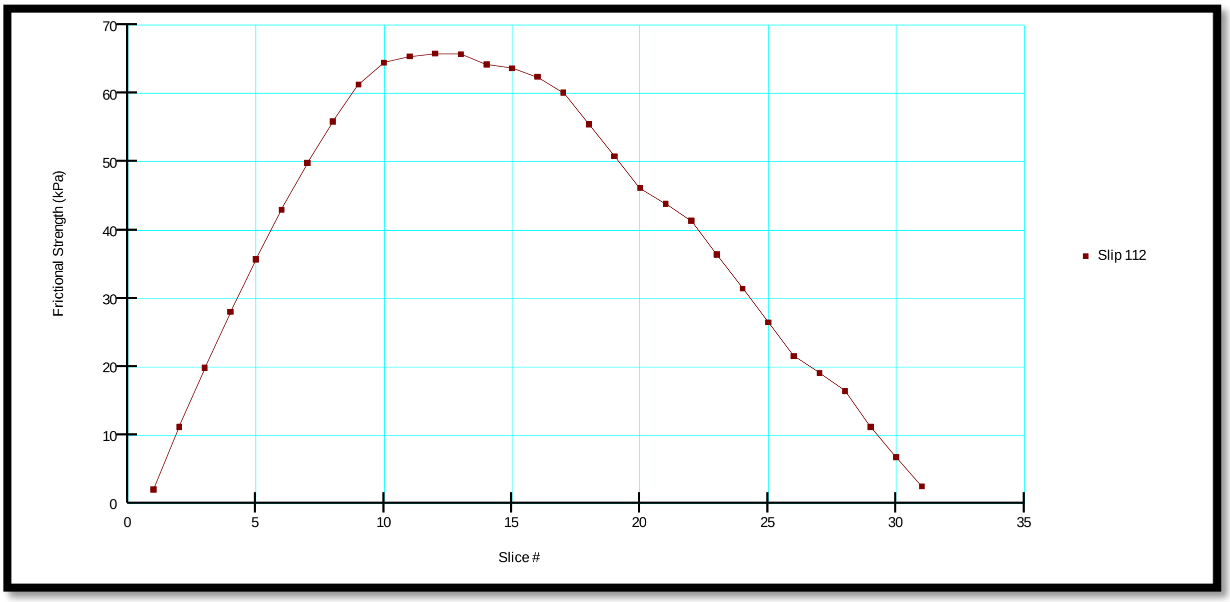

Figure 6.36 Variation of frictional stress with slices

6.2.5 Analysis for Slope 1:3.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:3.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

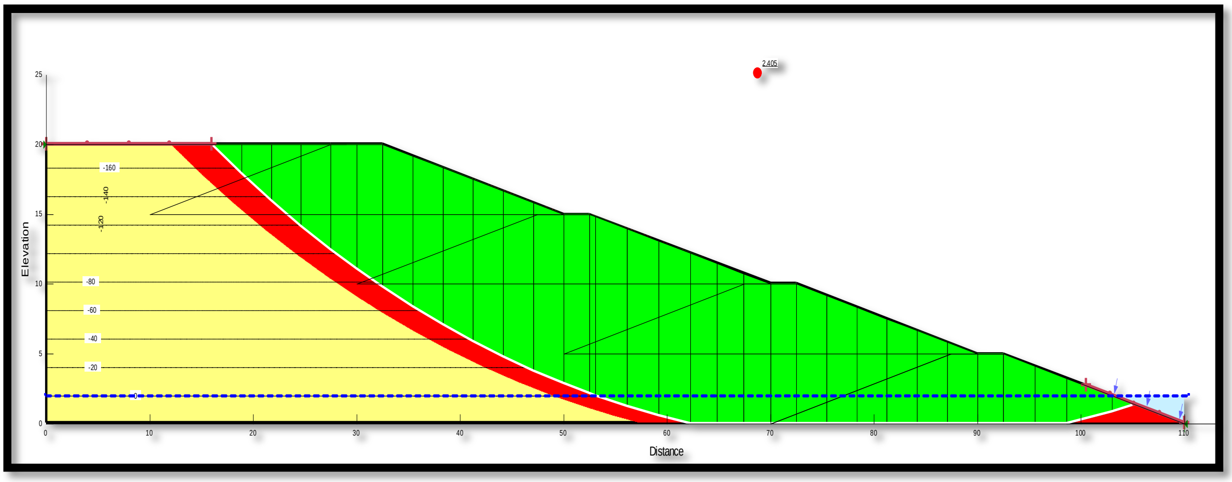

Figure 6.37 Embankment cross section for 1:3.5 step slope

The critical slice information are as follows:

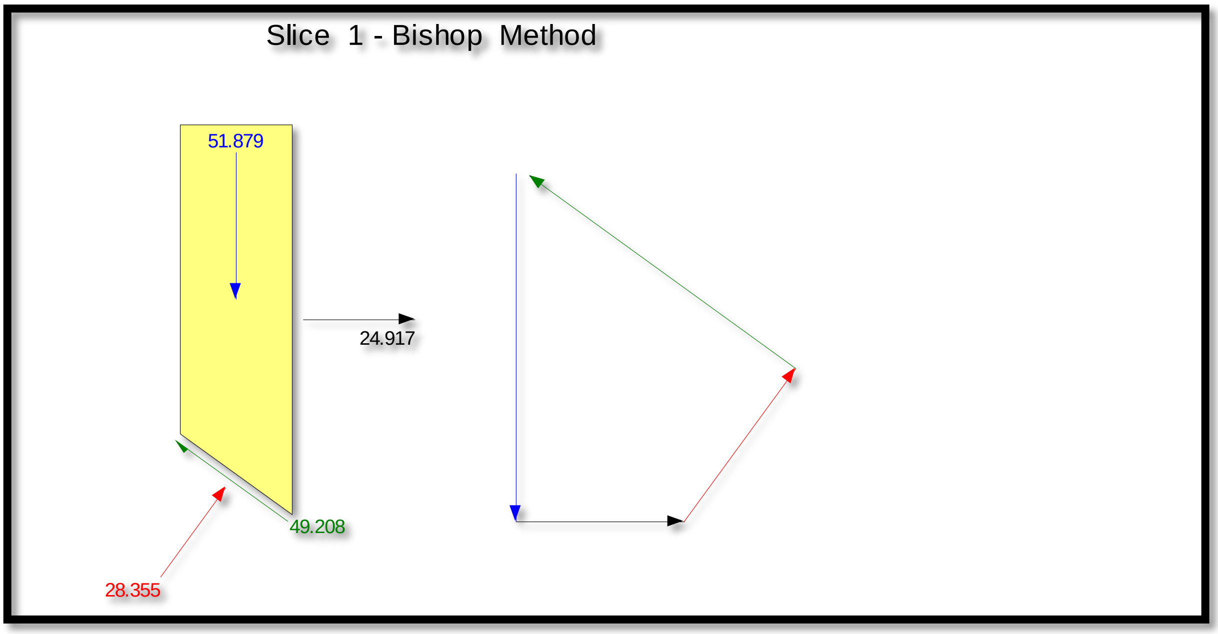

Figure 6.38 Critical slice for 1:3.5 step slope

The variation of frictional stress with slices are as follows:

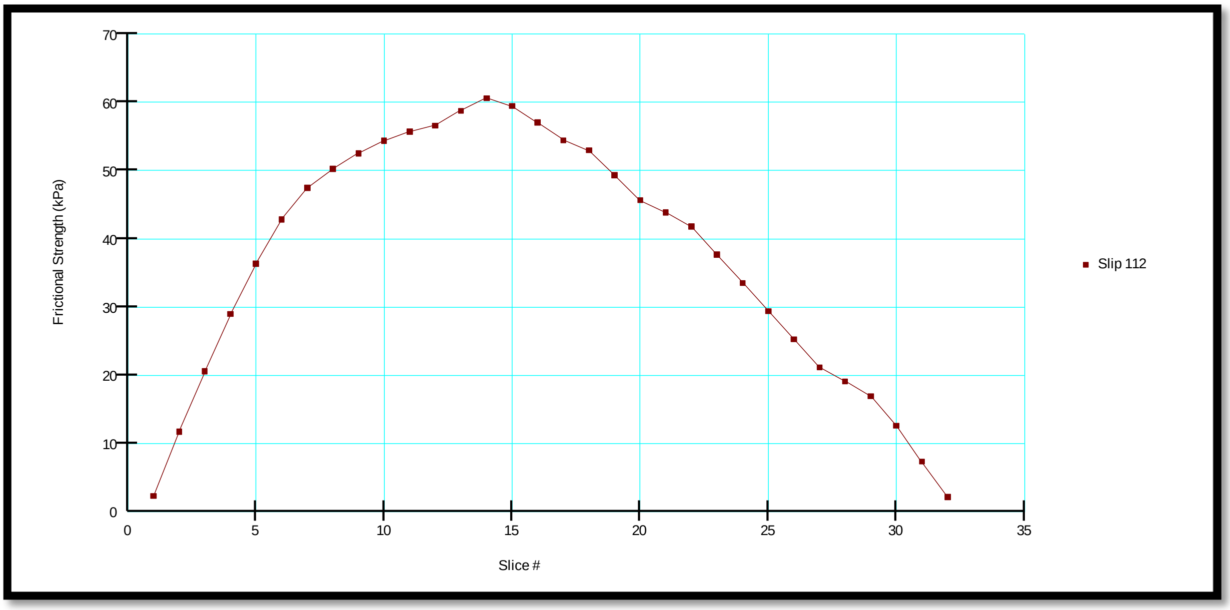

Figure 6.39 Variation of frictional stress with slices

6.2.6 Analysis for Slope 1:4

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:4
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

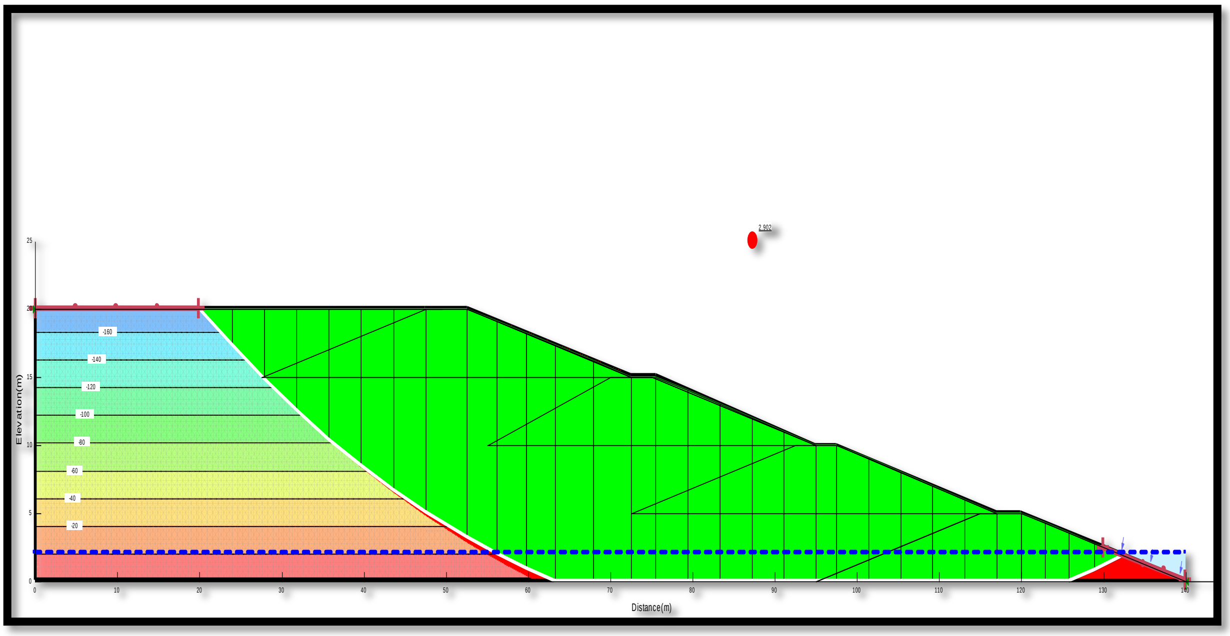

Figure 6.40 Embankment cross section for 1:4 step slope

The critical slice information are as follows:

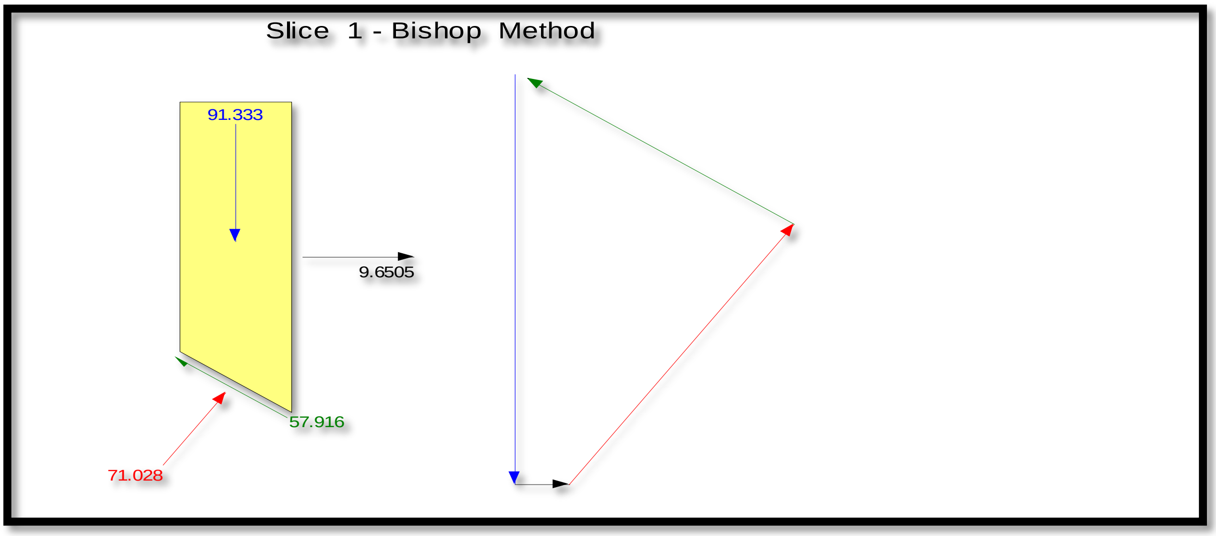

Figure 6.41 Critical slice for 1:4 step slope

The variation of frictional stress with slices are as follows:

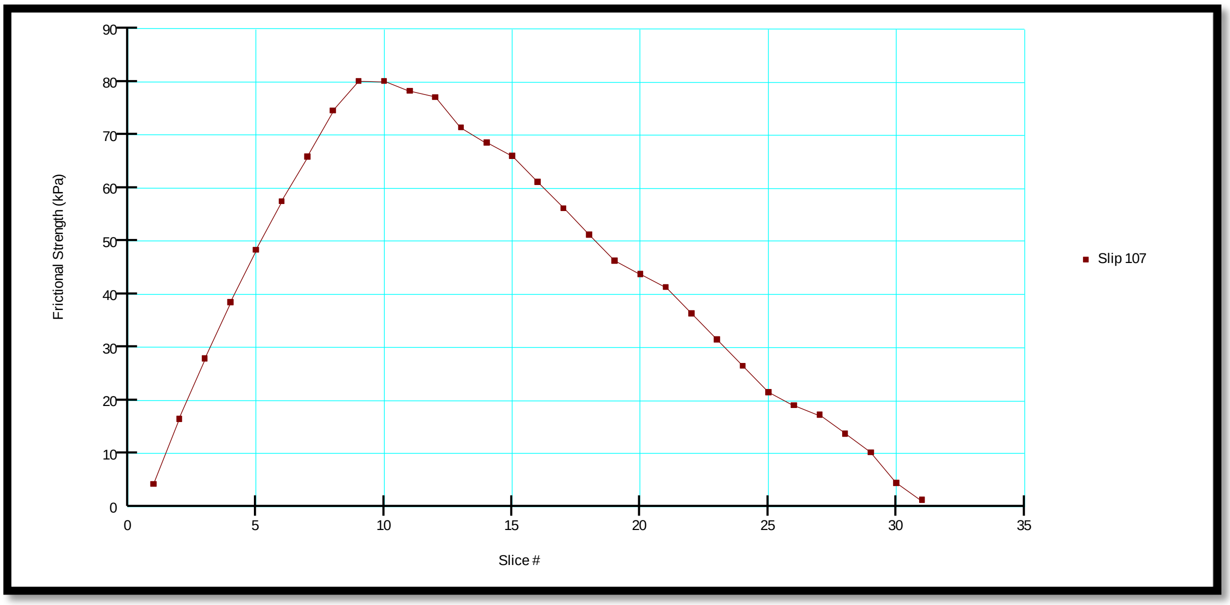

Figure 6.42 Variation of frictional stress with slices

6.2.7 Analysis for Slope 1:4.5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:4.5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

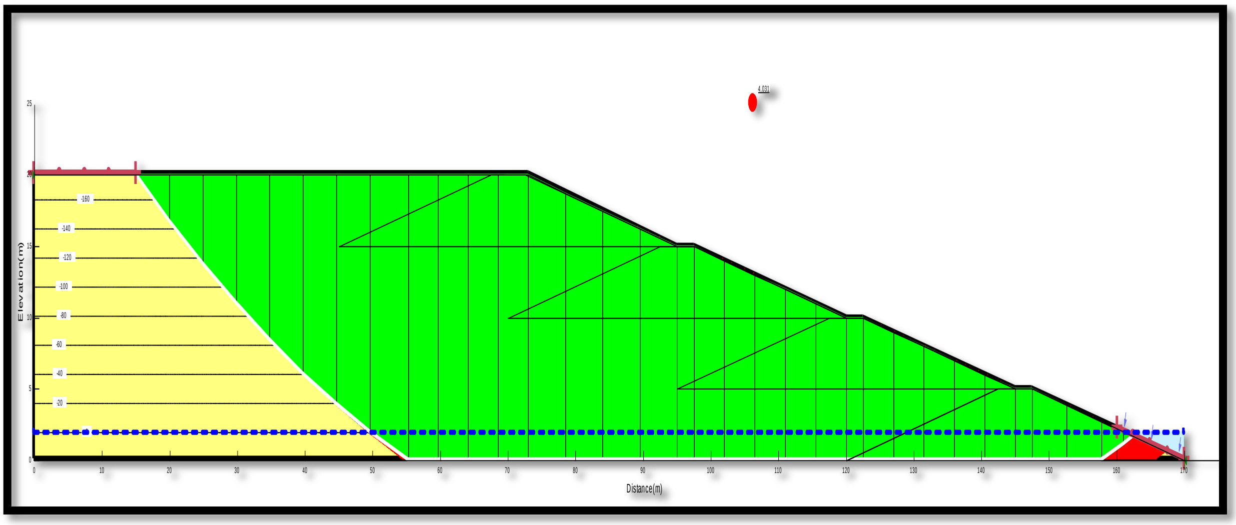

Figure 6.43 Embankment cross section for 1:4.5 step slope

The critical slice information are as follows:

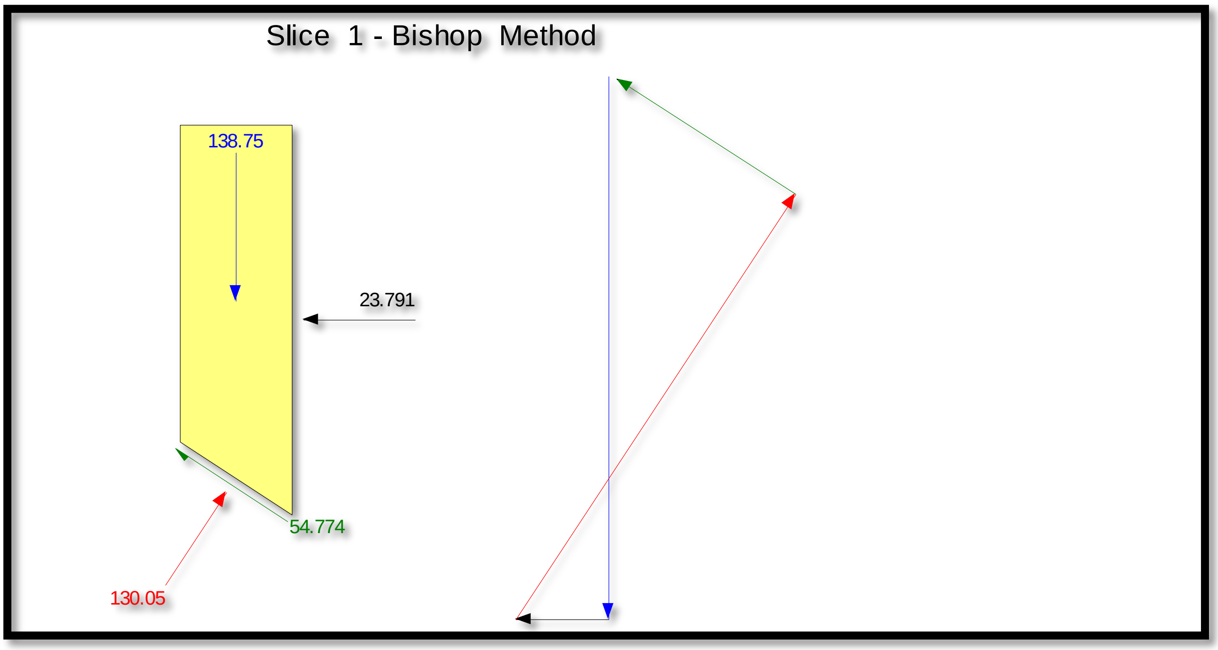

Figure 6.44 Critical slice for 1:4.5 step slope

The variation of frictional stress with slices are as follows:

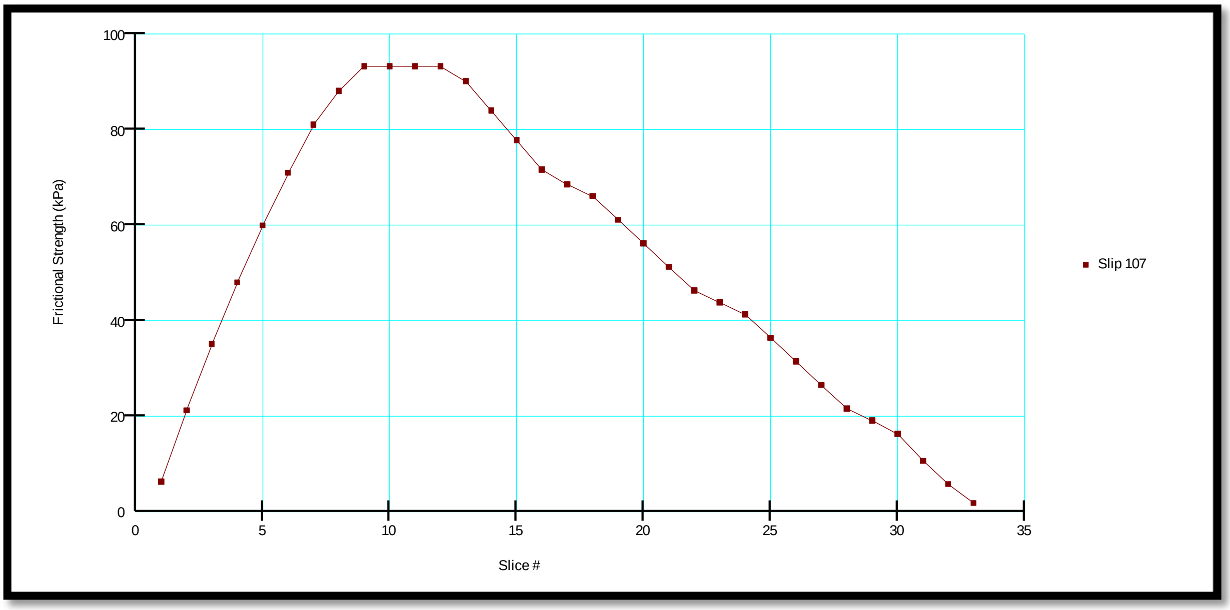

Figure 6.45 Variation of frictional stress with slices

6.2.8 Analysis for Slope 1:5

The embankment has the following parameters:

1. Height of the embankment: 20 m
2. Slope given on both side of embankment: 1:5
3. Cohesion : 31kPa
4. Friction Angle: 16^0
5. Unit Weight : 17.25 kN/m³

The cross section is made on Slope W software and the analysis results are as follows:

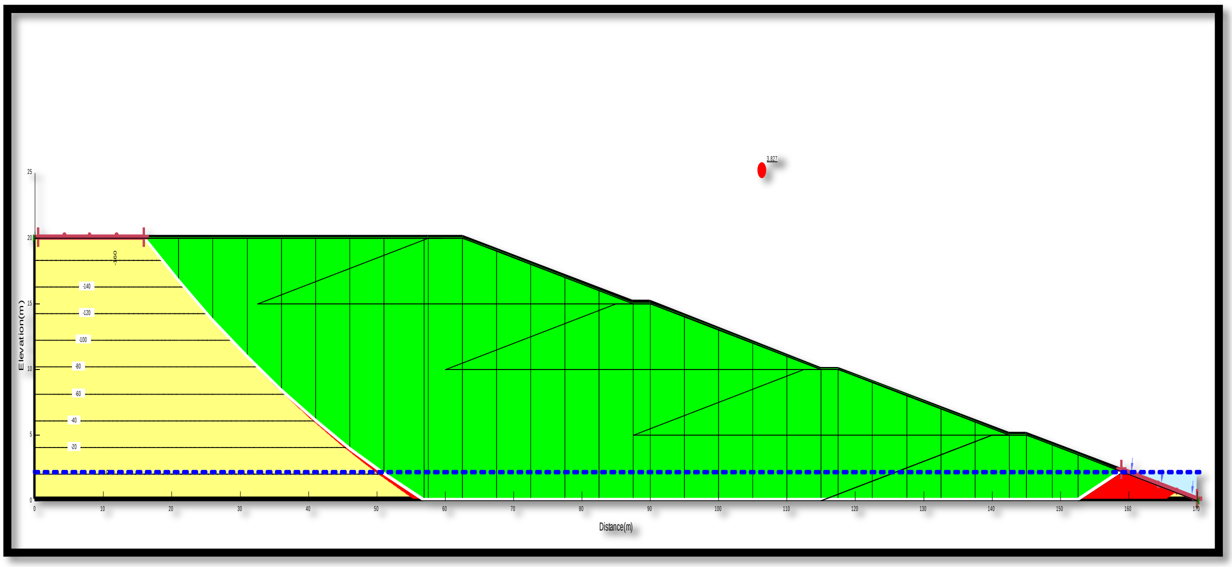

Figure 6.46 Embankment cross section for 1:5 step slope

The critical slice information are as follows:

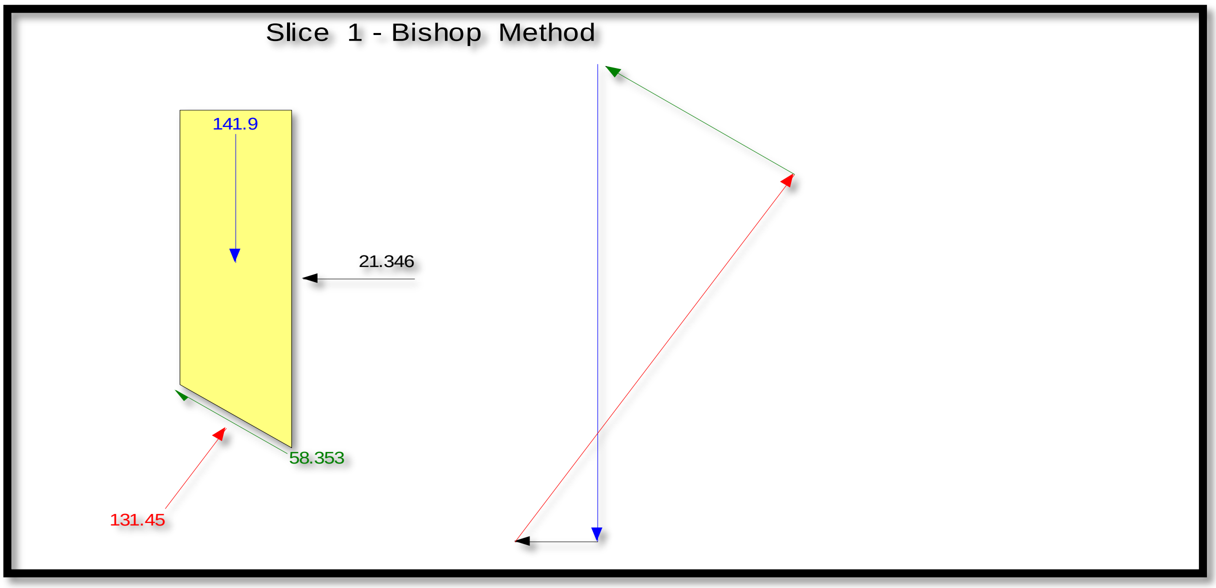

Figure 6.47 Critical slice for 1:5 step slope

The variation of frictional stress with slices are as follows:

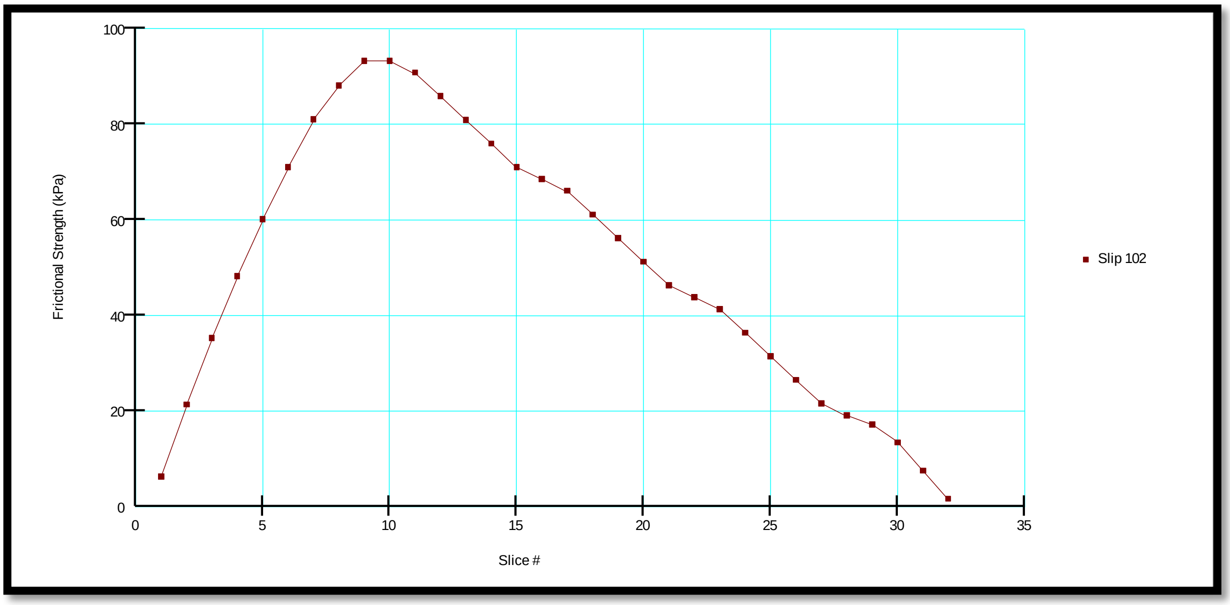

Figure 6.48 Variation of frictional stress with slices

From above data there is variation of FOS with change in step slopes. The graph showing the variation are as follows:

Table 6.2 Variation of FOS with slope of embankment

Slope of embankment	FOS
1:1.5	1.728
1:2	1.862
1:2.5	2.090
1:3	2.300
1:3.5	2.405
1:4	2.908

1:4.5	3.031
1:5	3.862

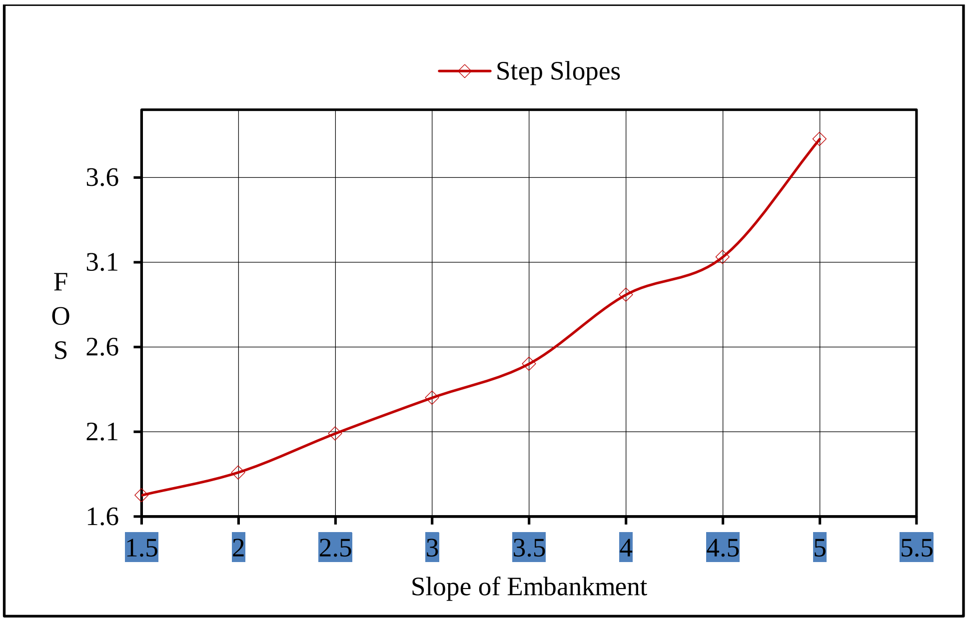

Figure 6.49 Variation of FOS with slope of embankment

Comparison of simple slope with step slope shown that there is increase in FOS in step slope as compared to simple slopes thus less susceptible to failure.

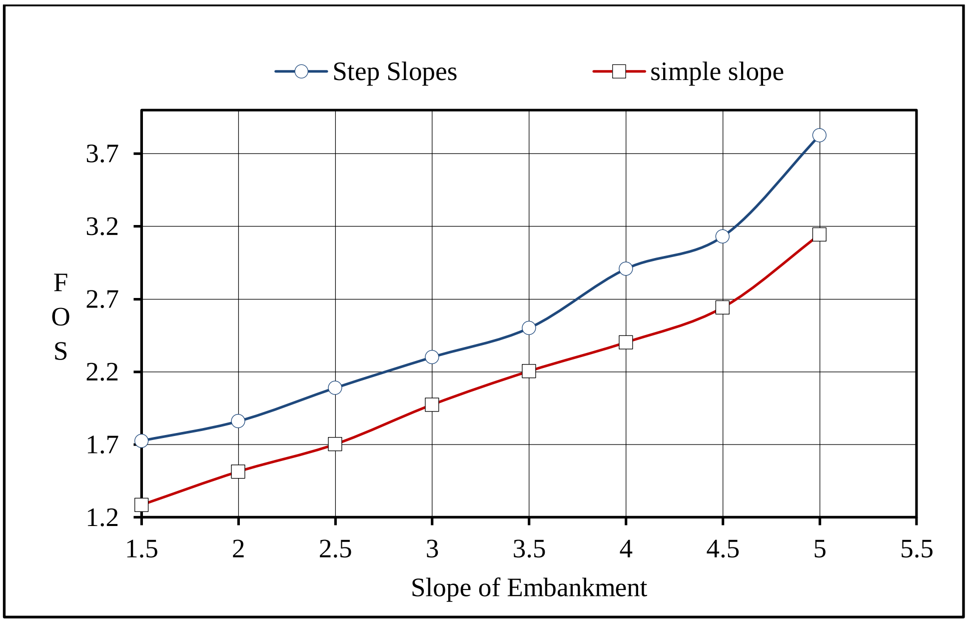

Figure 6.50 Comparison of FOS between simple and step slope

6.3Analysis of FOS At Different Water Level

6.3.1 Analysis for 1:1.5 slope

The water level is kept at 5,10, 15 m and FOS has been calculated and compared.

- At 5 m water level:

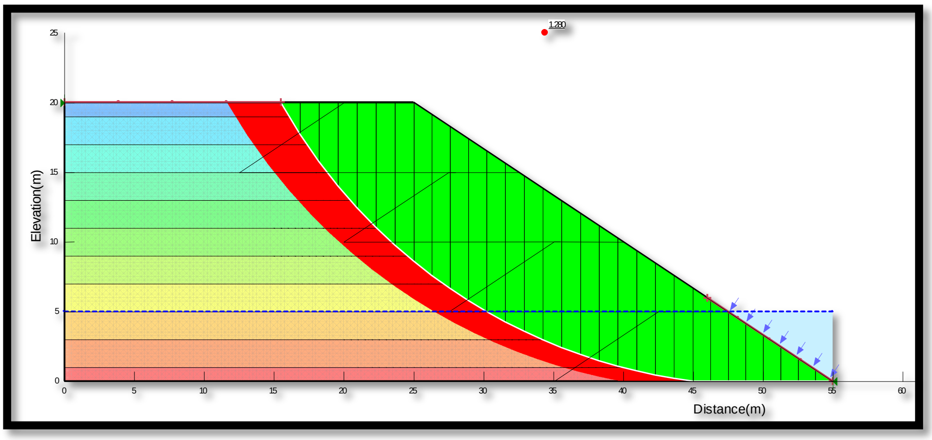

Figure 6.51 Embankment cross section for 1:1.5 slope at 5m water level

Factor of Safety 1.2802

- At 10 m water level:

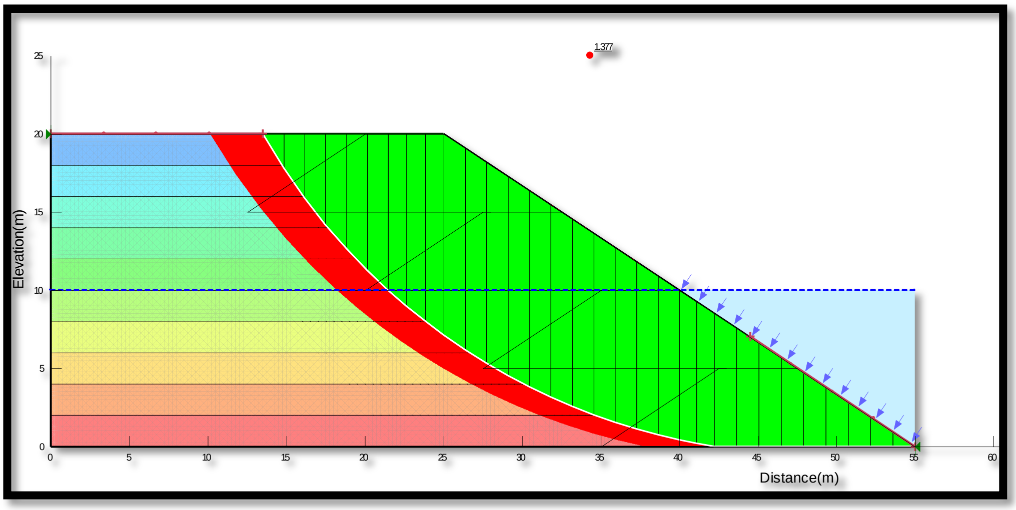

Figure 6.52 Embankment cross section for 1:1.5 slope at 10 m water level

Factor of Safety 1.3775

- At 15 m water level

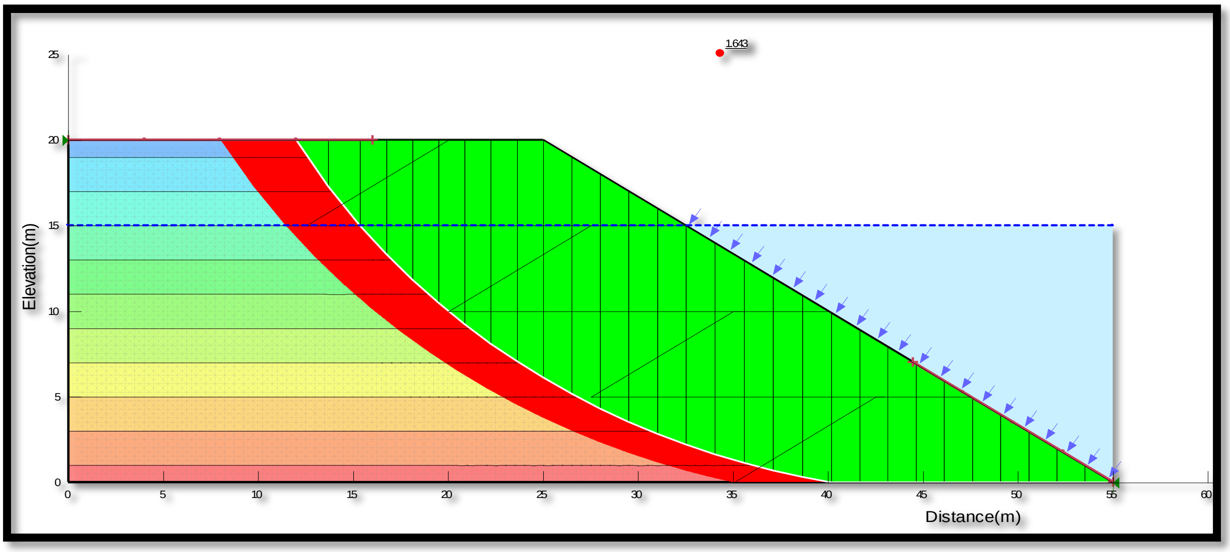

Figure 6.53 Embankment cross section for 1:1.5 slope at 15 m water level

Factor of Safety 1.6431

The comparison at different water level for 1:1.5 slope are as follows:

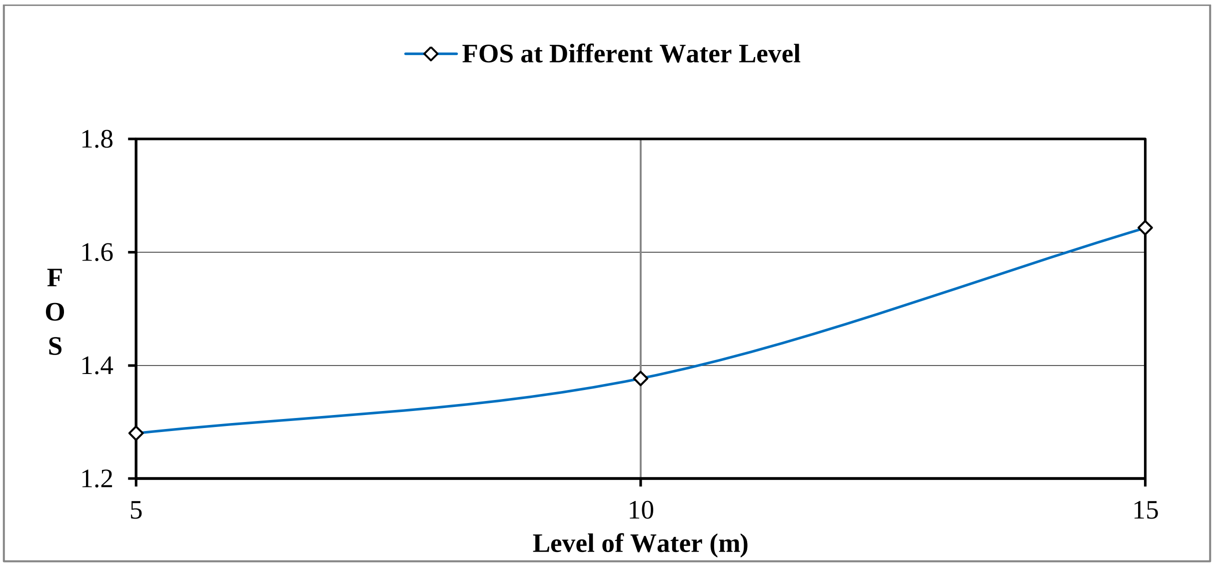

Figure 6.54 Variation of FOS at different water level

6.3.2Analysis for 1:2 slope:

- At 5 m water level

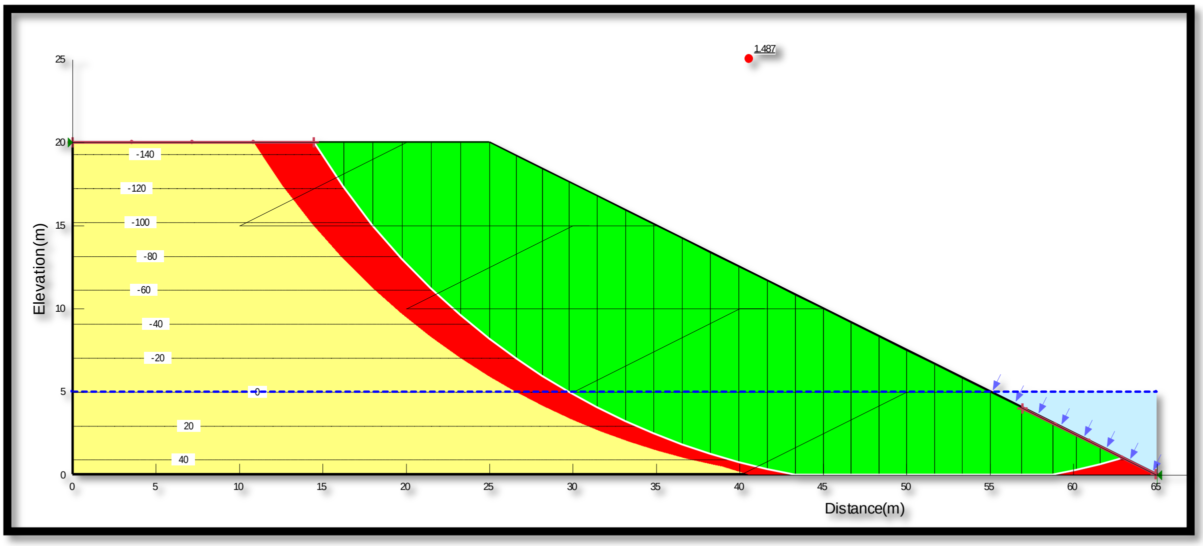

Figure 6.54 Embankment cross section for 1:2 slope at 5 m water level

Factor of Safety 1.4872

- **At 10 m water level**

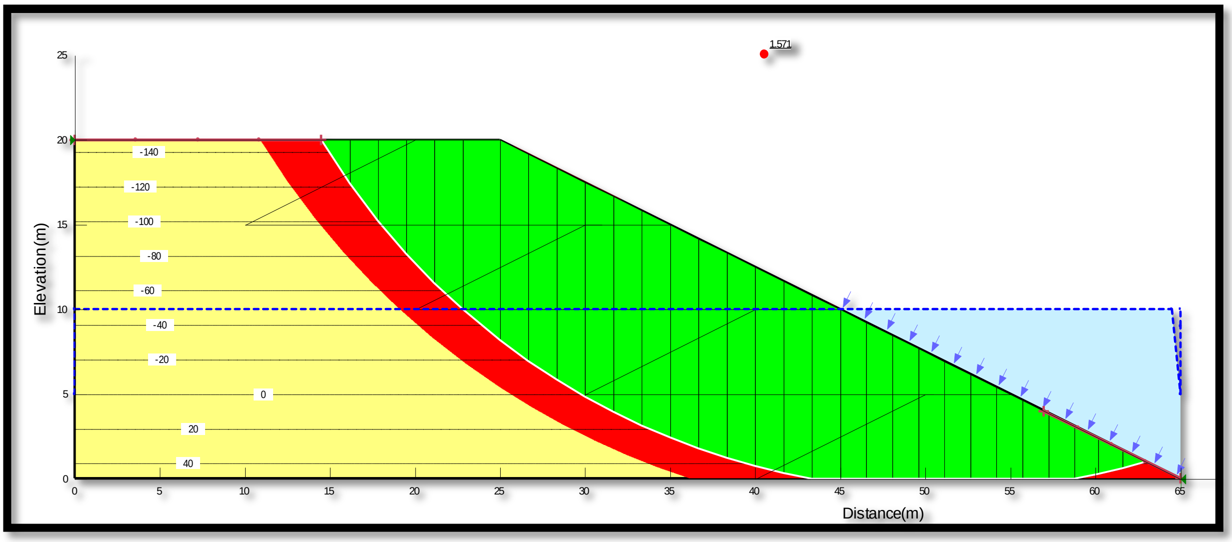

Figure 6.55 Embankment cross section for 1:2 slope at 10 m water level

Factor of Safety 1.5715

- **At 15 m water level**

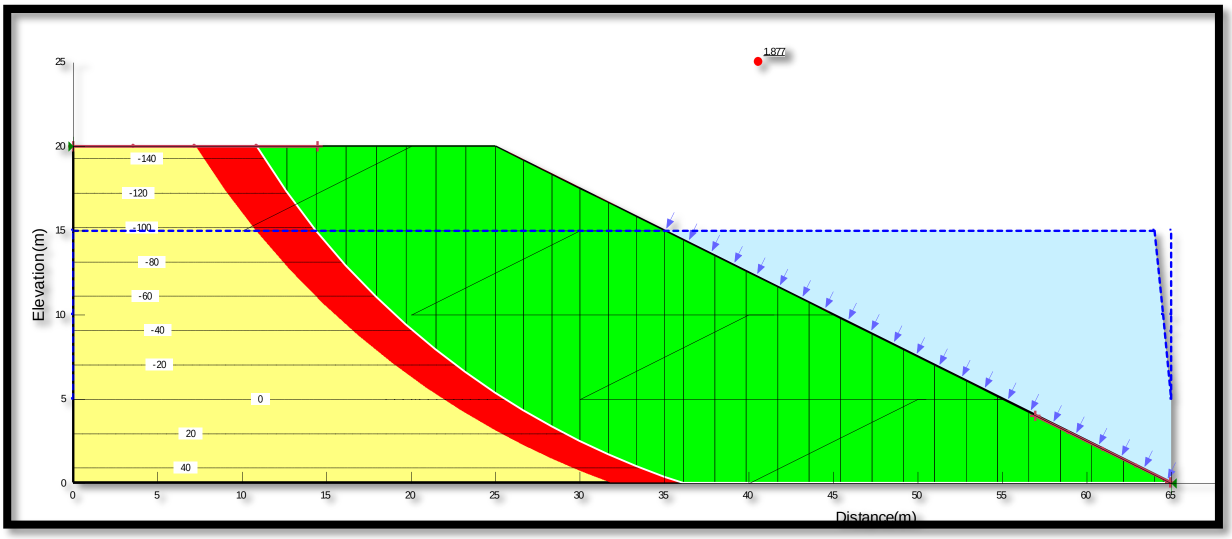

Figure 6.56 Embankment cross section for 1:2 slope at 15 m water level

Factor of Safety 1.877

The comparison curve at different water level for 1:2 slope are as follows:

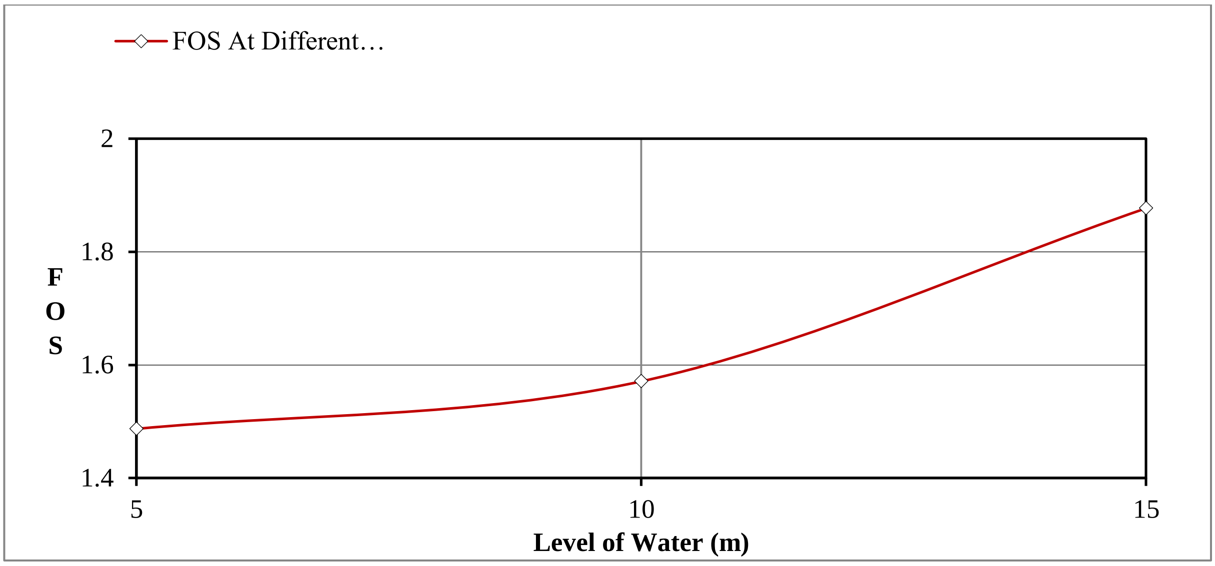

Figure 6.57 Variation of FOS at different water level

6.3.3 Analysis for 1:2.5 slope

- **At 5 m water level:**

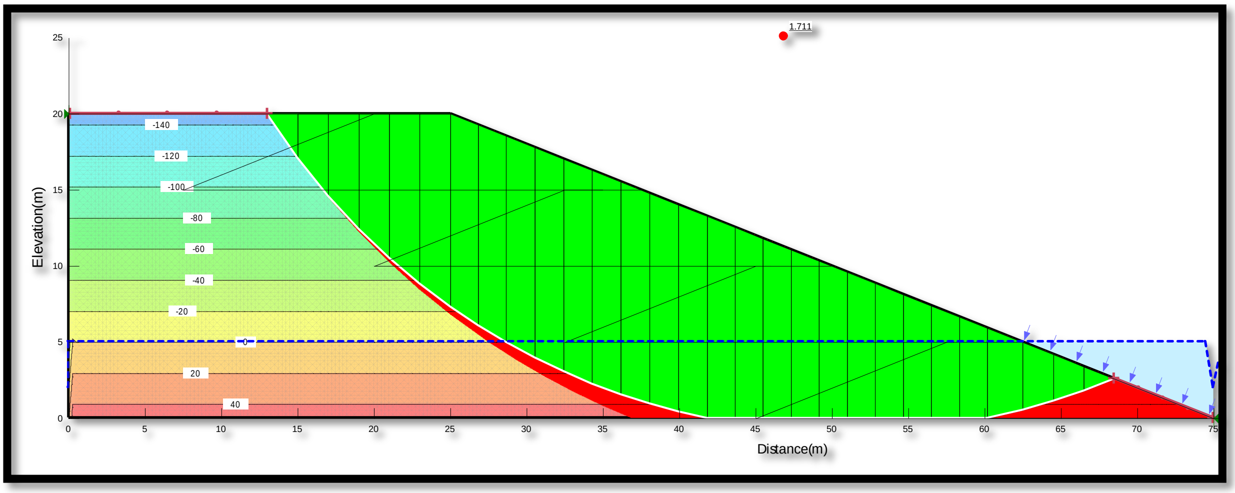

Figure 6.58 Embankment cross section for 1:2.5 slope at 5 m water level

Factor of Safety 1.7109

- **At 10 m water level :**

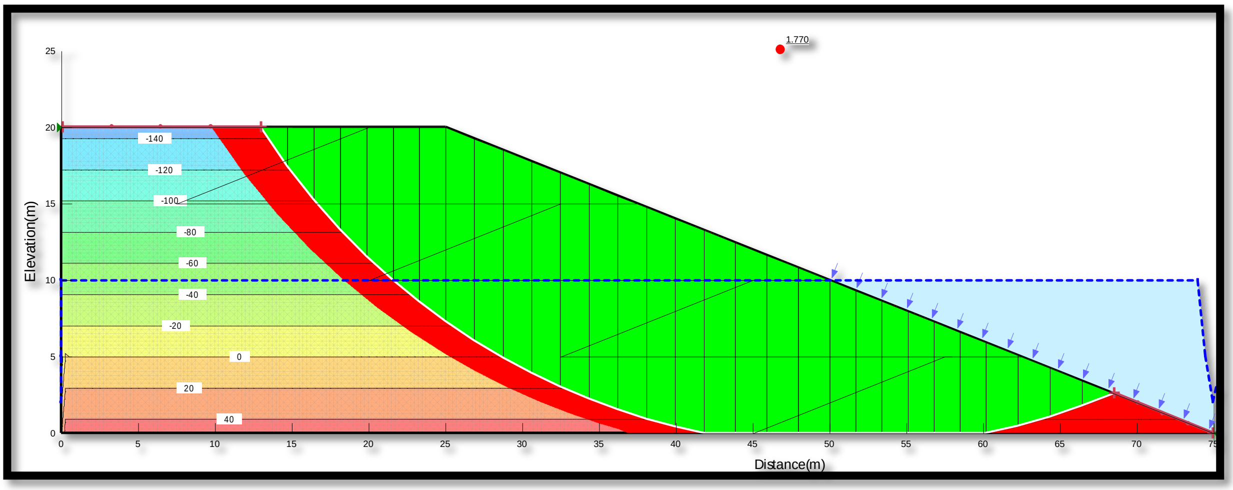

Figure 6.59 Embankment cross section for 1:2.5 slope at 10 m water level

Factor of Safety 1.7701

- **At 15 m water level:**

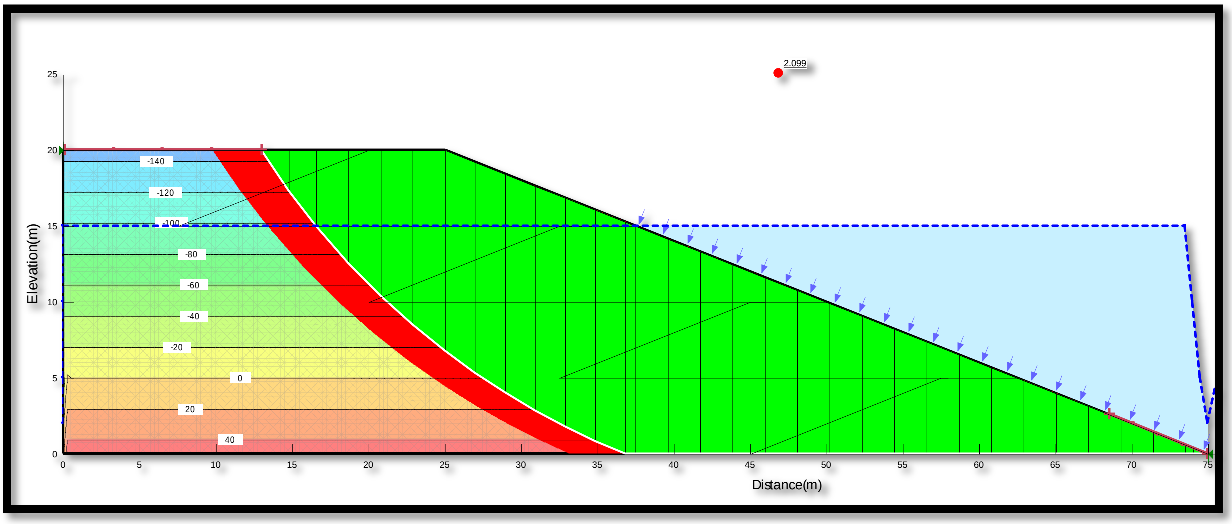

Figure 6.60 Embankment cross section for 1:2.5 slope at 15 m water level

Factor of Safety 2.0987

The comparison curve at different water level for 1:2.5 slope are as follows:

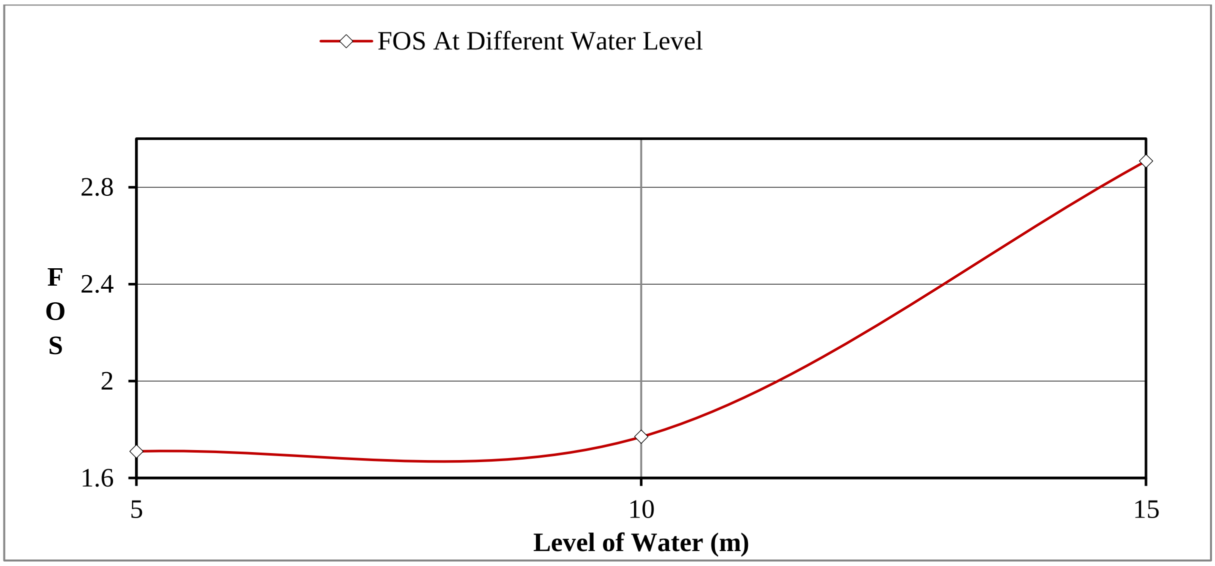

Figure 6.61 Variation of FOS at different water level

6.3.4Analysis for 1:3 slope

- **At 5 m water level:**

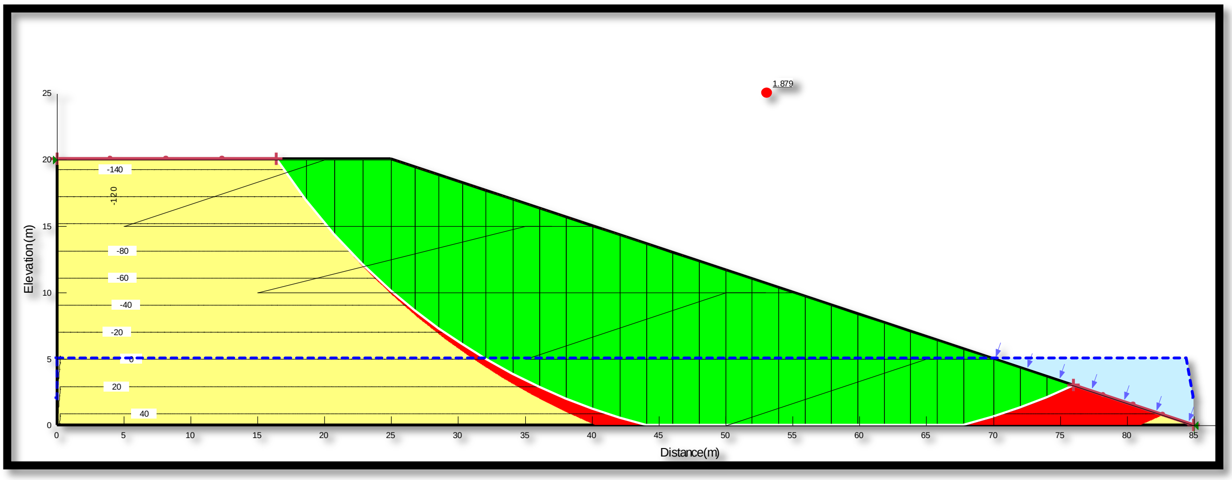

Figure 6.62 Embankment cross section for 1:3 slope at 5 m water level

Factor of Safety 1.8785

- **At 10 m water level:**

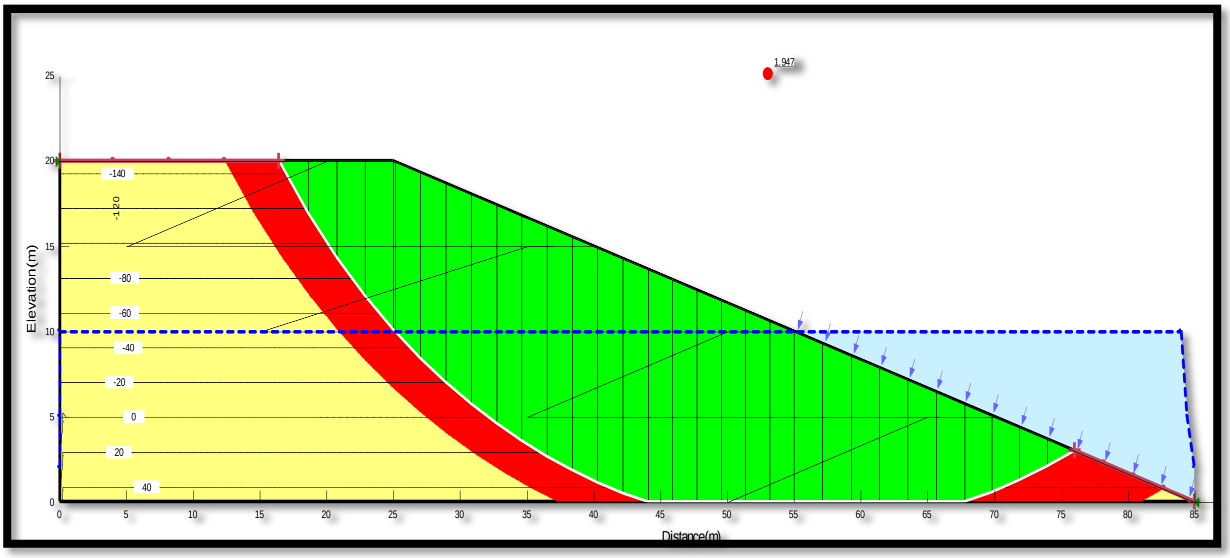

Figure 6.63 Embankment cross section for 1:3 slope at 10 m water level

Factor of Safety 1.9474

- **At 15 m water level:**

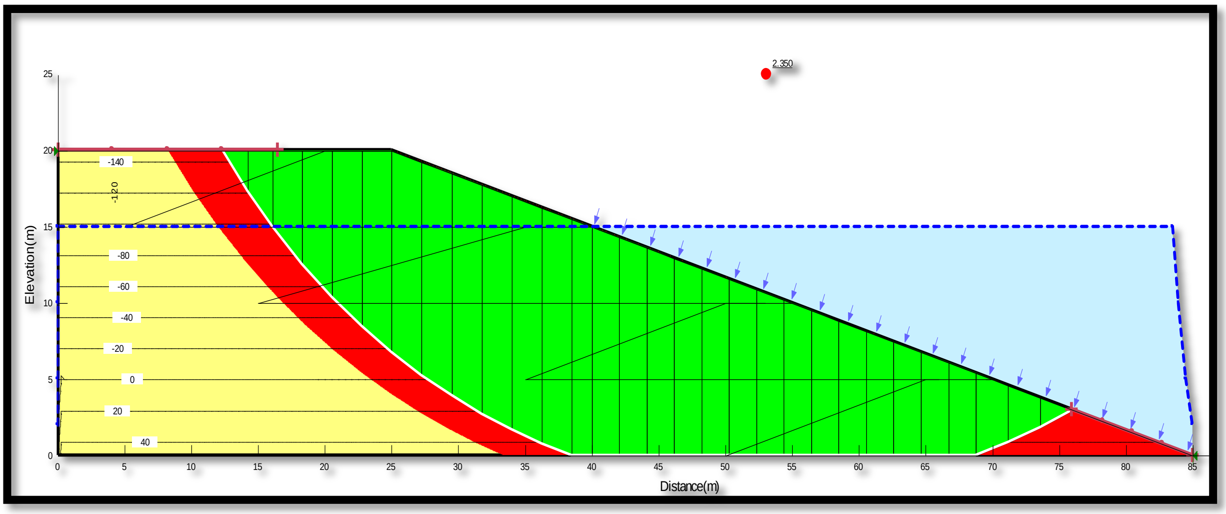

Figure 6.64 Embankment cross section for 1:3 slope at 15 m water leve

Factor of Safety 2.3497

The comparison curve at different water level for 1:3 slope are as follows:

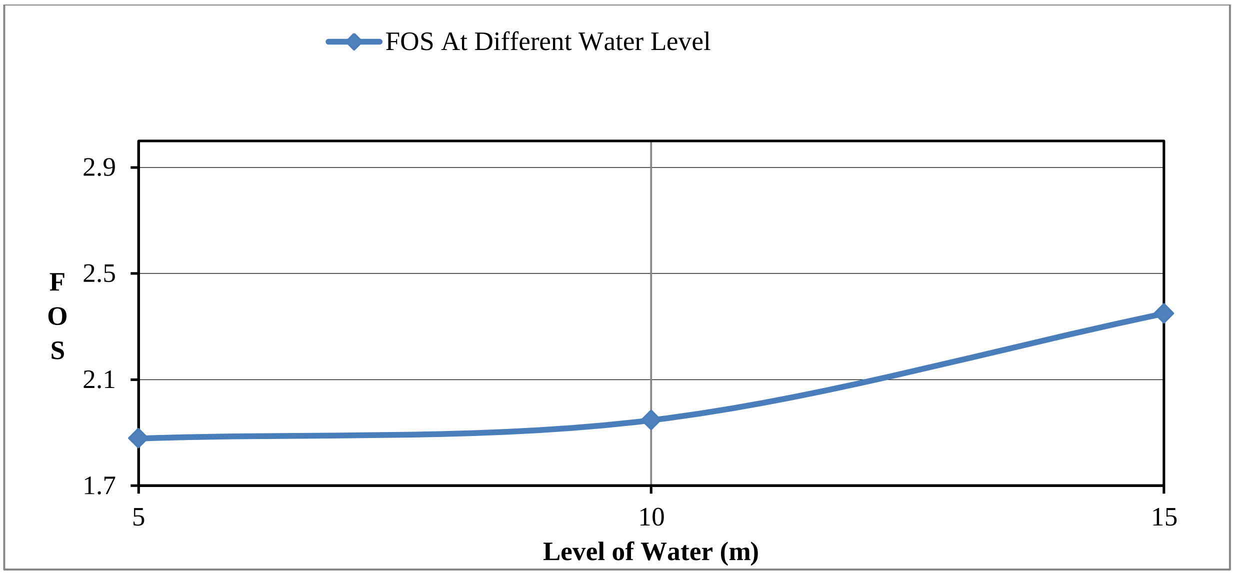

Figure 6.65 Variation of FOS at different water level

6.3.5Analysis for 1:3.5 slope

- At 5 m water level

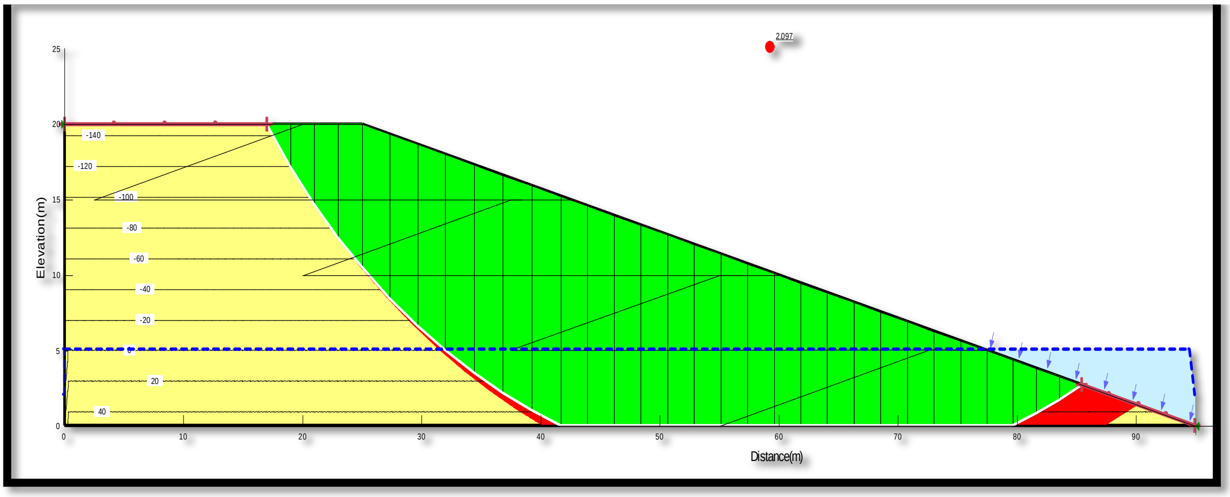

Figure 6.65 Embankment cross section for 1:3.5 slope at 5 m water level

Factor of Safety 2.0965

- **At 10 m water level**

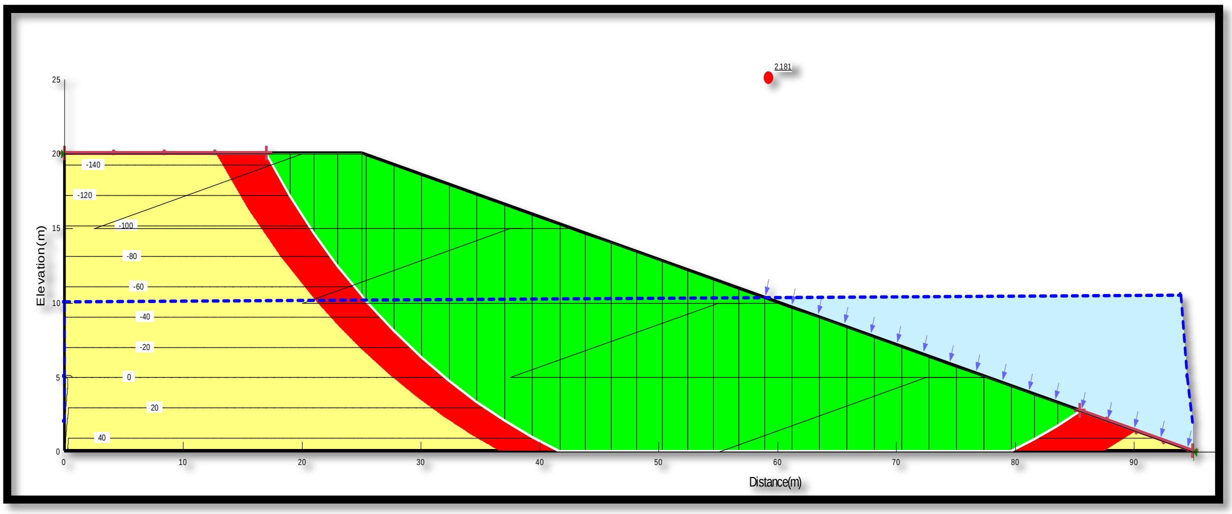

Figure 6.66 Embankment cross section for 1:3.5 slope at 10 m water level

Factor of Safety 2.1812

- **At 15 m water level:**

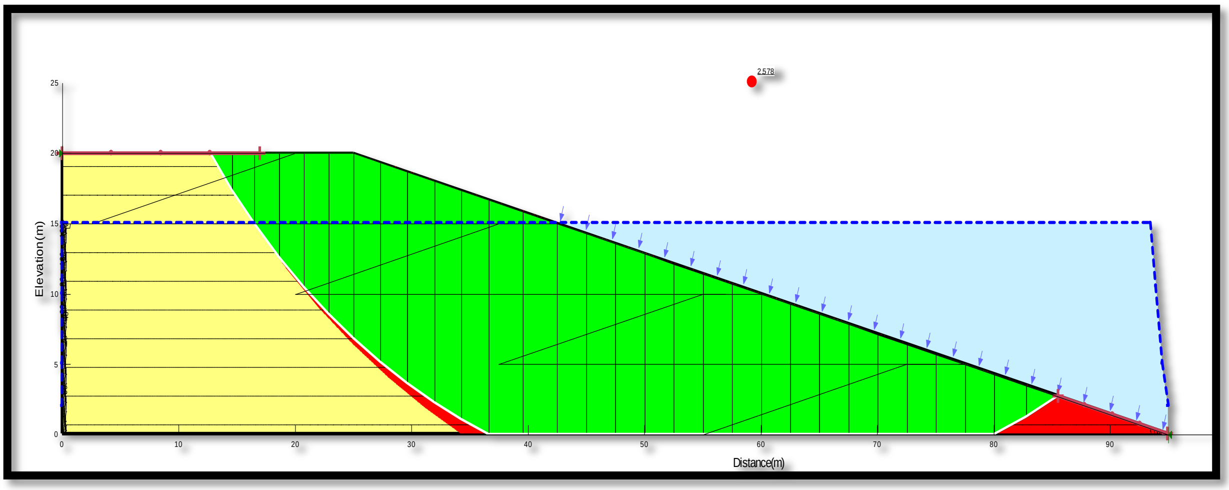

Figure 6.67 Embankment cross section for 1:3.5 slope at 15 m water level

Factor of Safety 2.5779

The comparison curve at different water level for 1:3.5 slope are as follows:

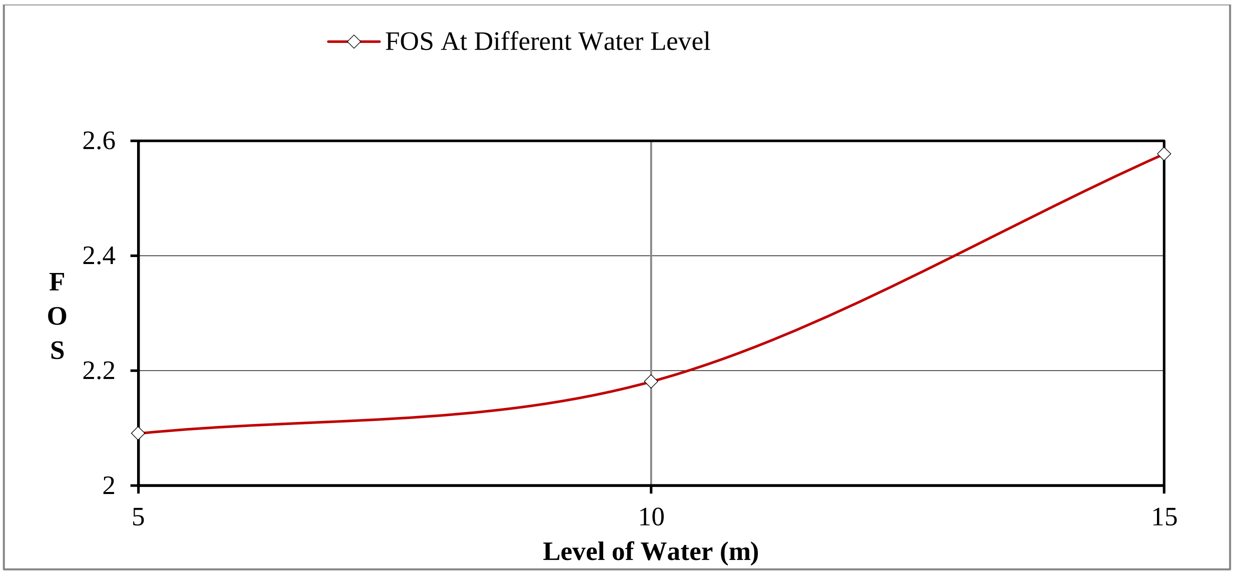

Figure 6.68 Variation of FOS at different water level

6.3.6Analysis for 1:4 slope

- **At 5 m water level:**

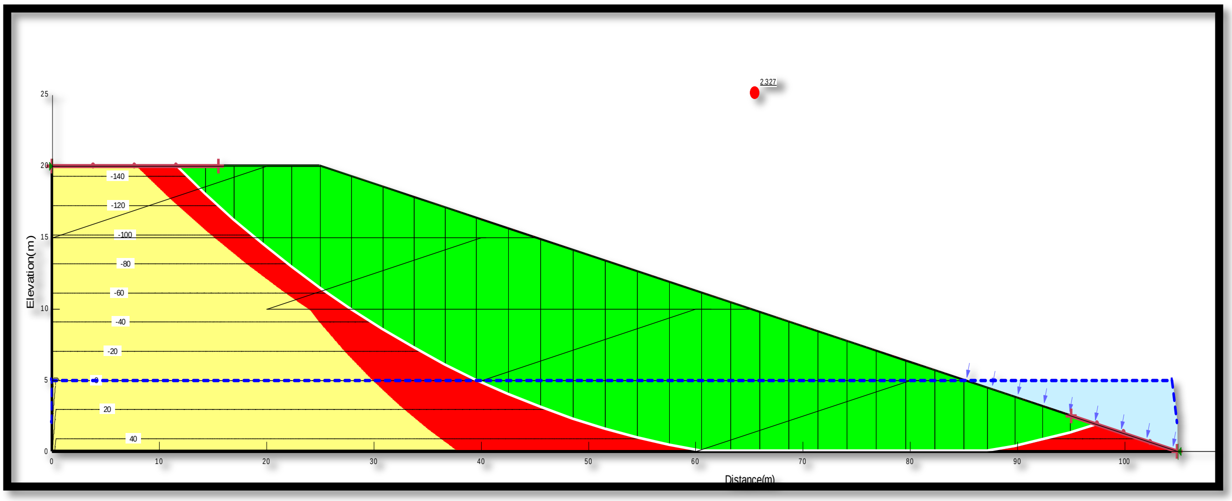

Figure 6.69 Embankment cross section for 1:4 slope at 5 m water level

Factor of Safety 2.3274

- **At 10 m water level:**

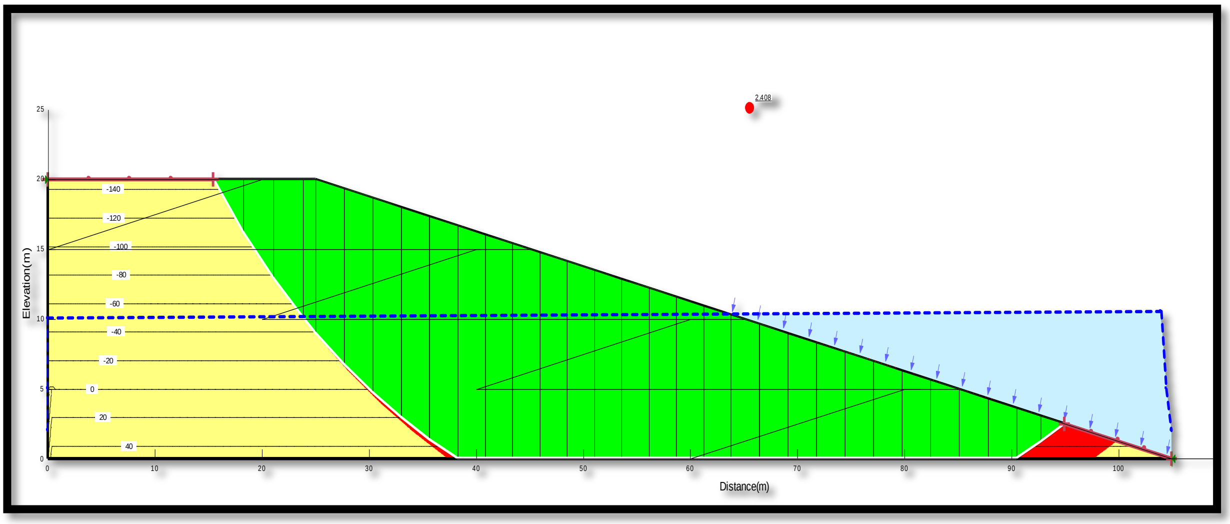

Figure 6.70 Embankment cross section for 1:4 slope at 10 m water level

Factor of Safety 2.4076

- **At 15 m water level:**

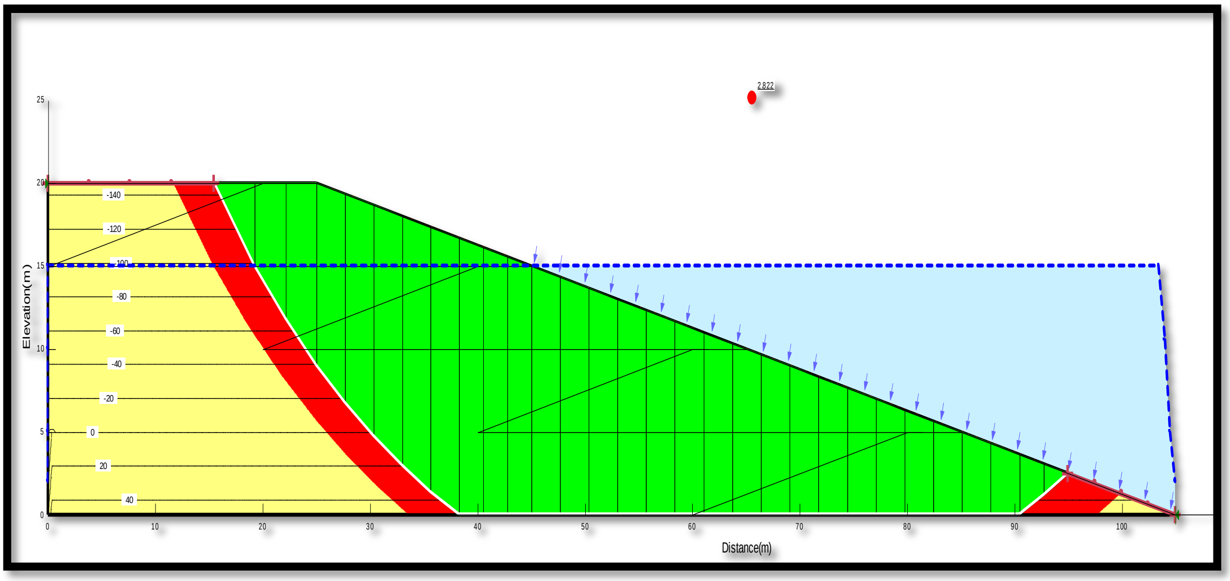

Figure 6.71 Embankment cross section for 1:4 slope at 15 m water level

Factor of Safety 2.8222

The comparison curve for different water level at 1:4 slope are as follows:

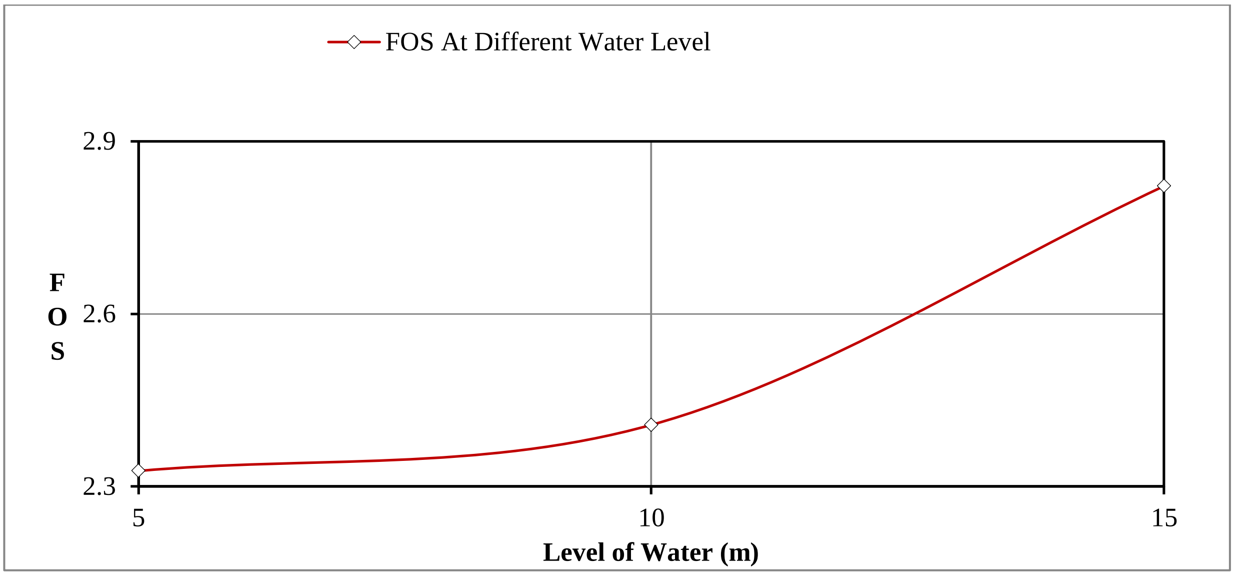

Figure 6.72 Variation of FOS at different water level

6.3.7 Analysis for 1:4.5 slope

- **At 5 m water level:**

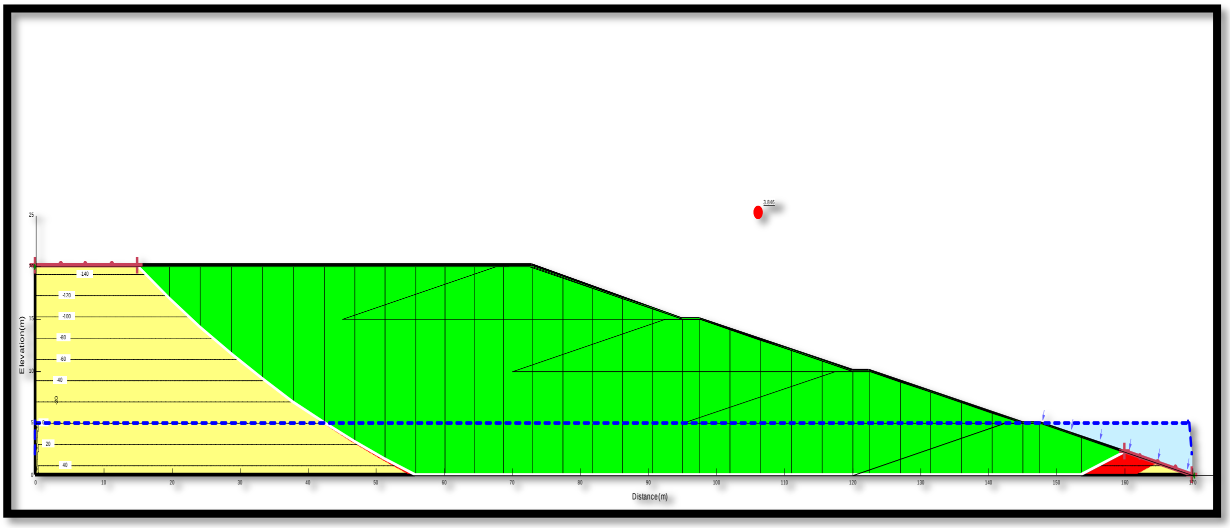

Figure 6.73 Embankment cross section for 1:4.5 slope at 5 m water level

Factor of Safety 3.8461

- **At 10 m water level:**

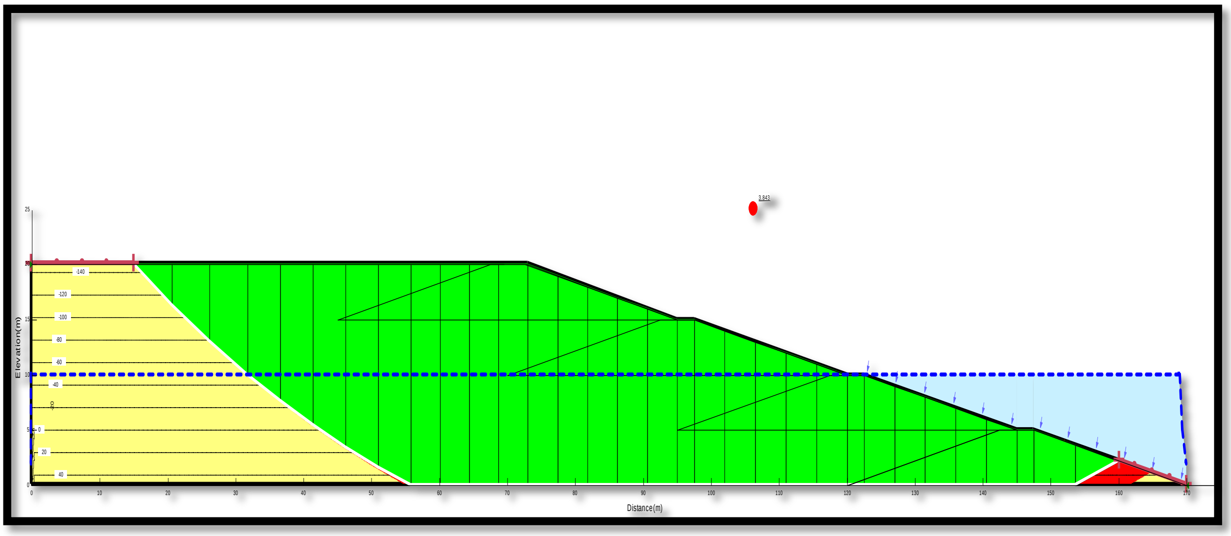

Figure 6.74 Embankment cross section for 1:4.5 slope at 10 m water level

Factor of Safety 3.8429

- **At 15 m water level**

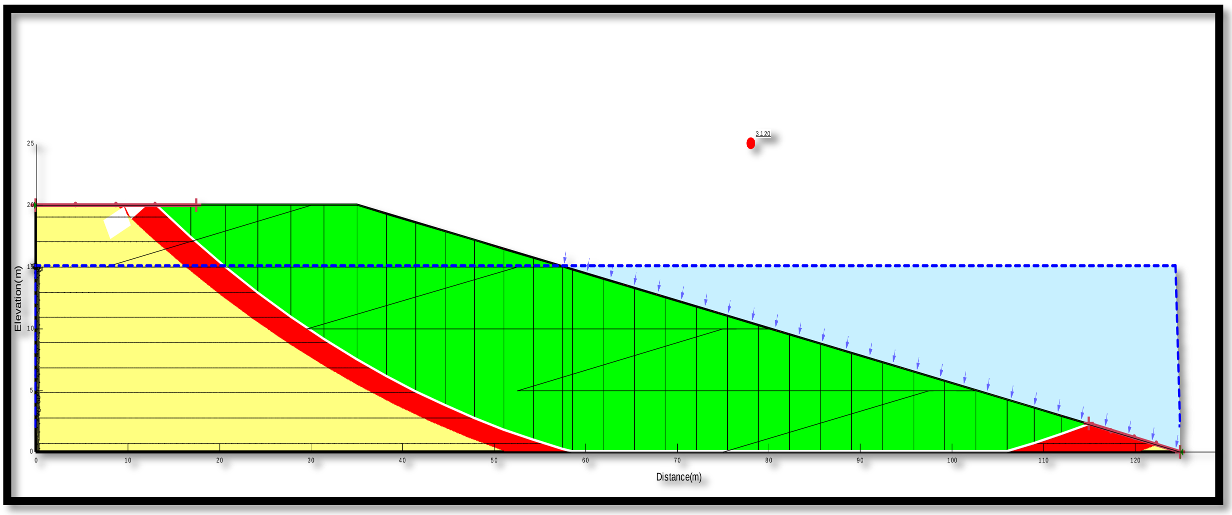

Figure 6.75 Embankment cross section for 1:4.5 slope at 15 m water level

Factor of Safety 3.1199

The comparison curve at different water level for 1:4.5 slope are as follows:

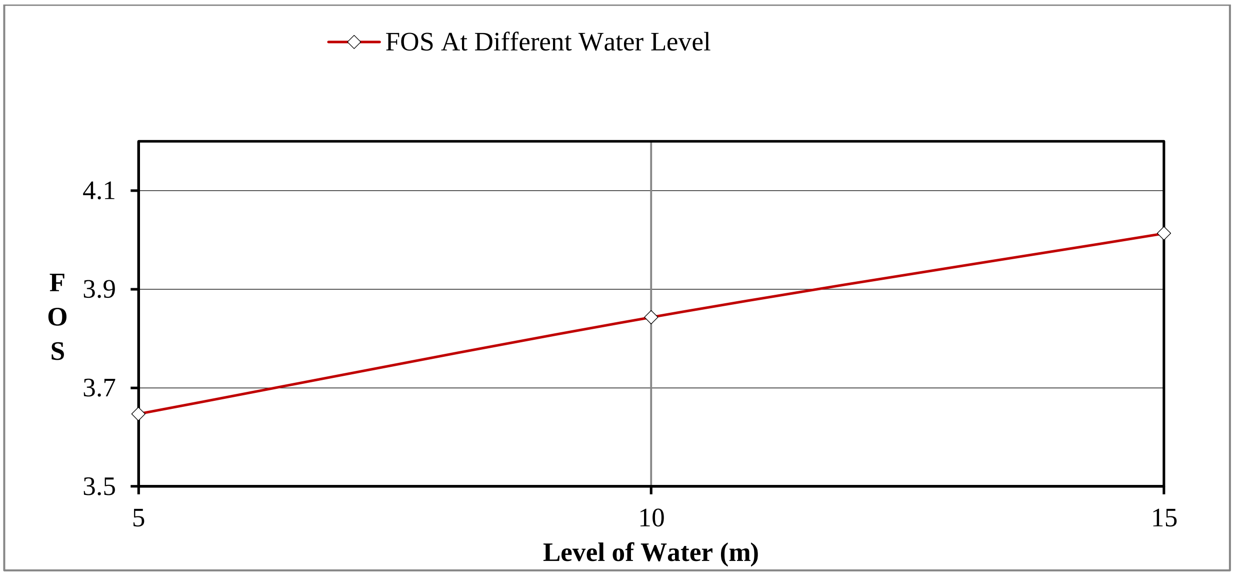

Figure 6.76 Variation of FOS at different water level

6.3.8 Analysis for 1:5 slope

- **At 5 m water level:**

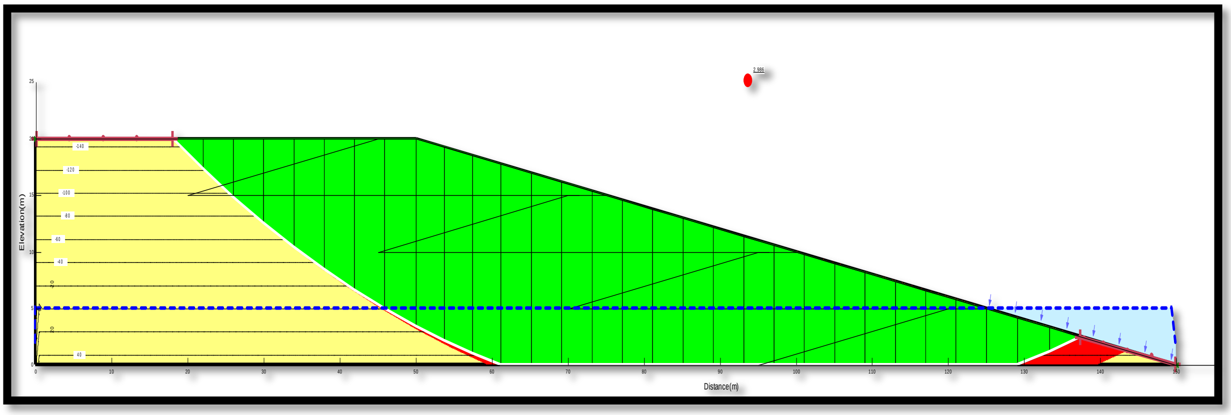

Figure 6.77 Embankment cross section for 1:5 slope at 5 m water level

Factor of Safety 2.9865

- **At 10 m water level:**

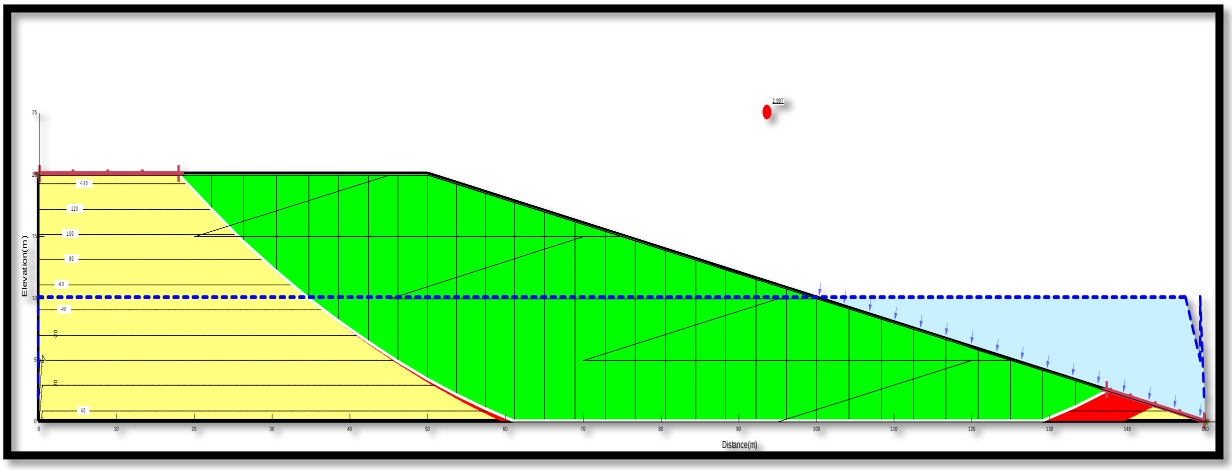

Figure 6.78 Embankment cross section for 1:5 slope at 10 m water level

Factor of Safety 2.9973

- **At 15 m water level:**

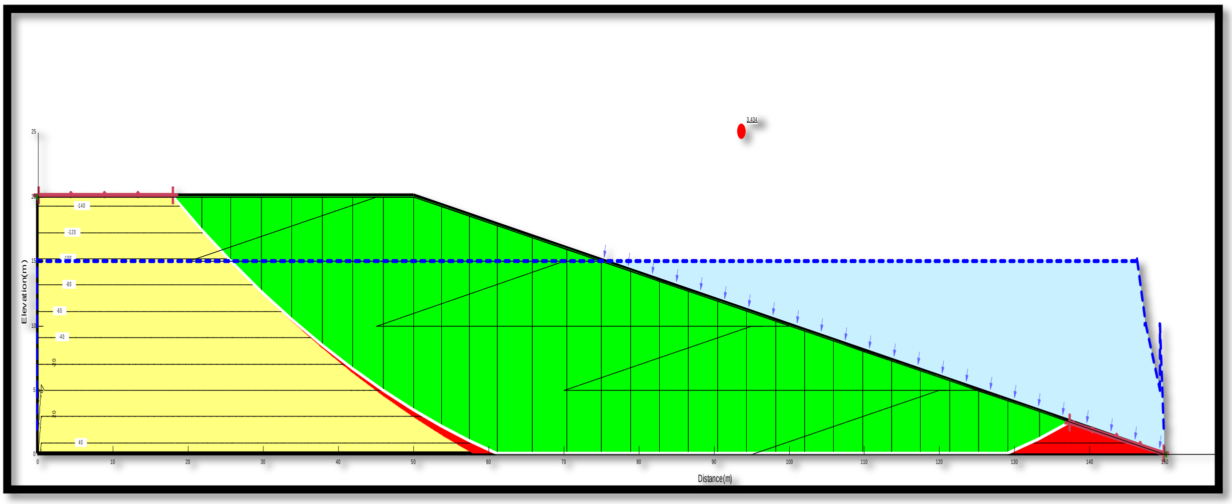

Figure 6.79 Embankment cross section for 1:5 slope at 15 m water leve

Factor of Safety	3.4343

The comparison curve at different water level for 1:5 slope are as follows:

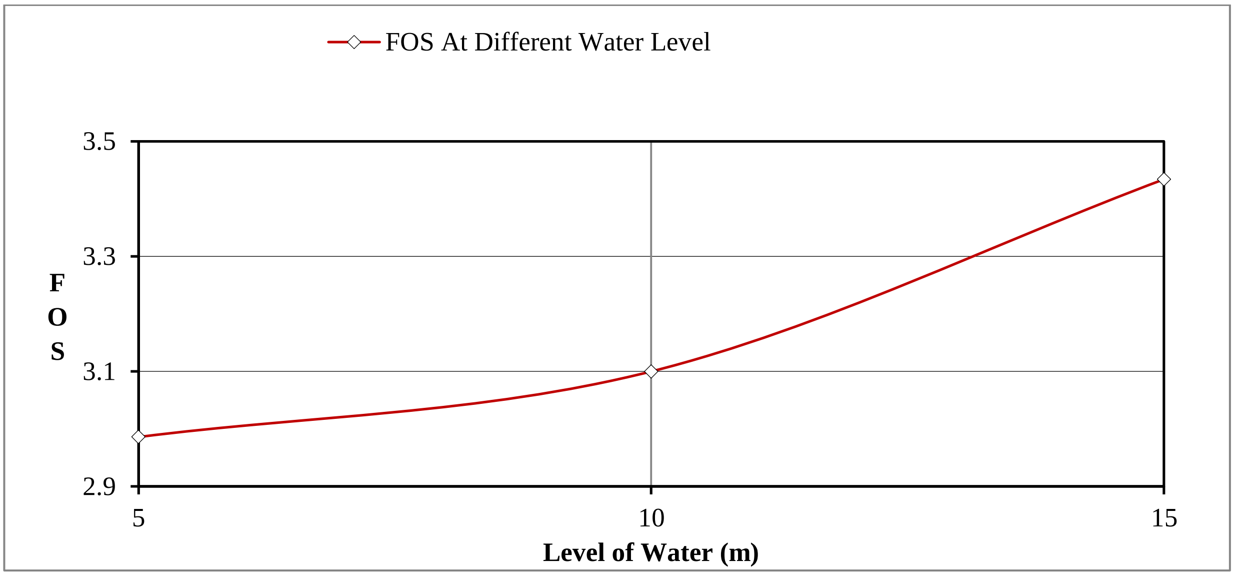

Figure 6.80 Variation of FOS at different water level

6.4 Analysis At Different water level For Step Slopes

6.4.1 Analysis For 1:1.5 Step Slope

- **At 5 m water level:**

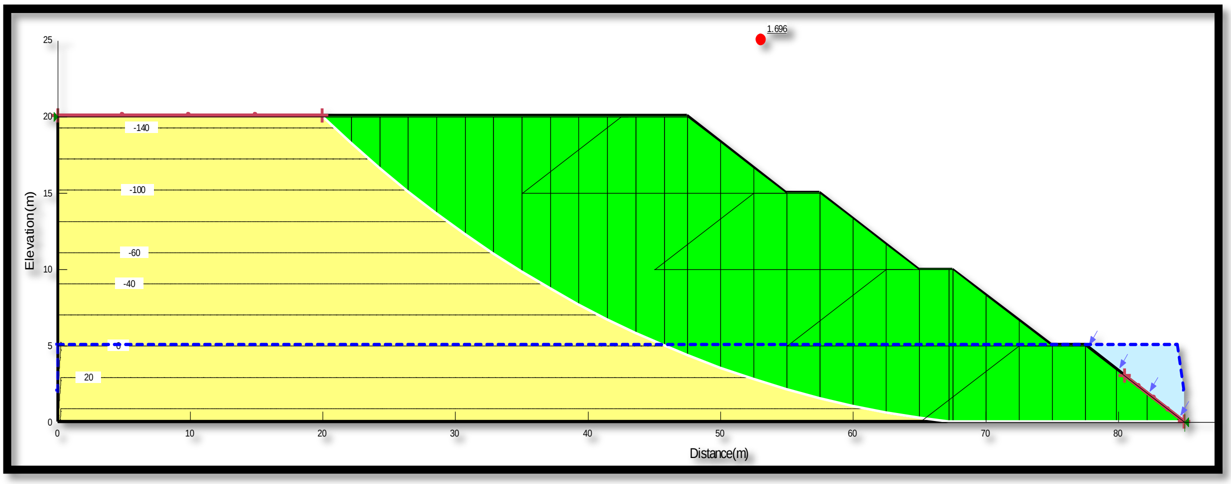

Figure 6.81 Embankment cross section for 1:1.5 slope at 5 m water level

Factor of Safety 1.6959

- **At 10 m water level**

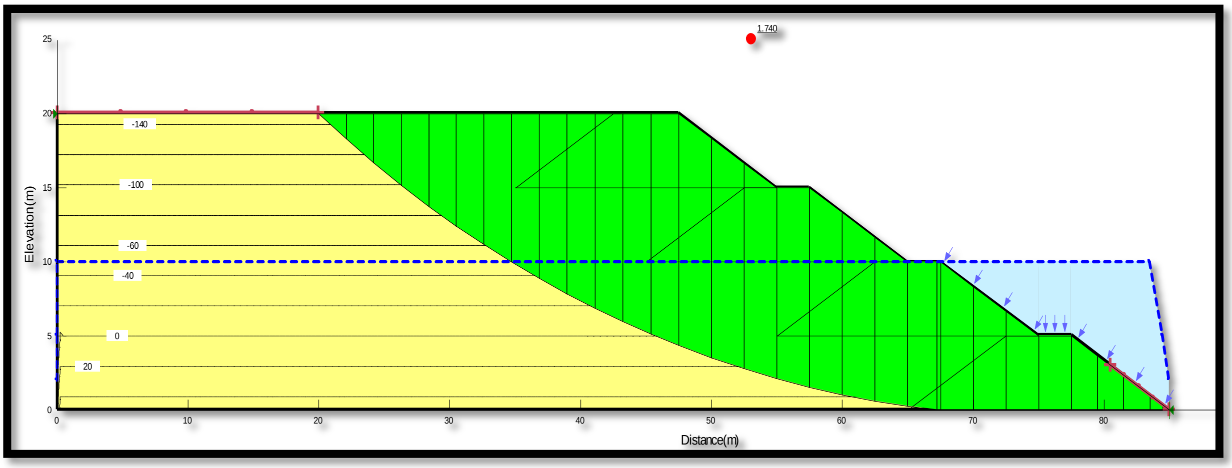

Figure 6.82 Embankment cross section for 1:1.5 slope at 10 m water level

Factor of Safety 1.7395

- **At 15 m water level:**

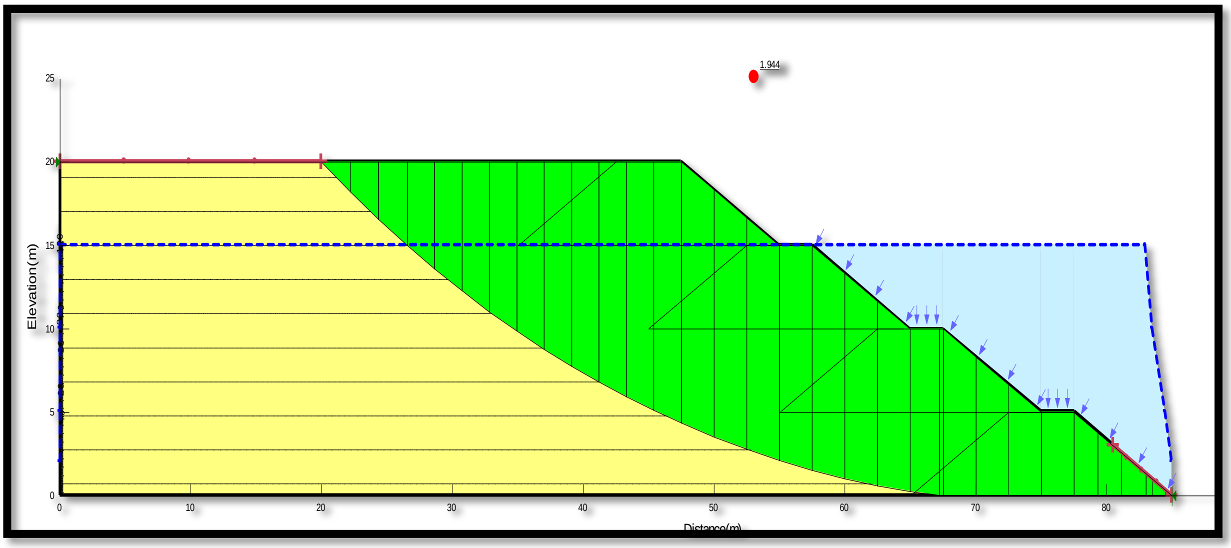

Figure 6.83 Embankment cross section for 1:1.5 slope at 15 m water level

Factor of Safety 1.9445

The comparison curve at different water level for 1:1.5 slope are as follows:

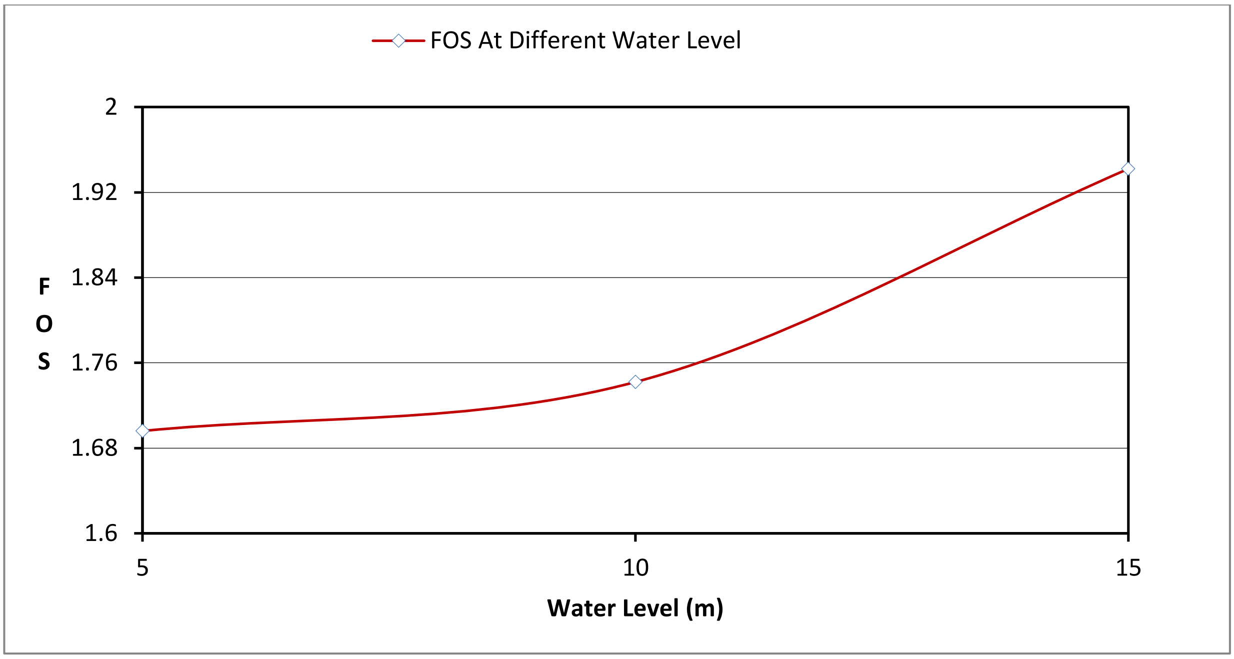

Figure 6.84 Variation of FOS with water level

6.4.2 Analysis for 1:2 Step Slope

- **At 5 m water level:**

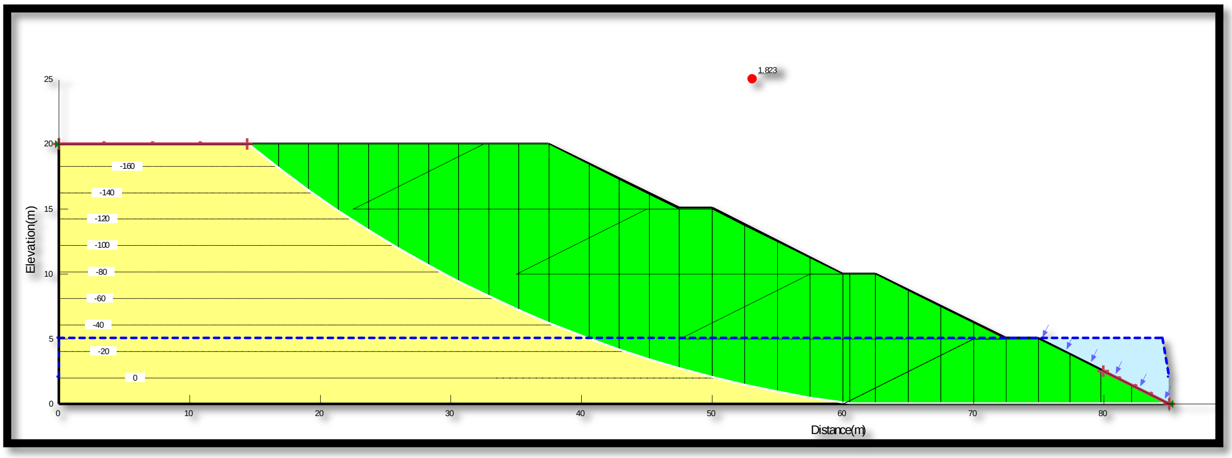

Figure 6.85 Embankment cross section for 1:2 slope at 5 m water level

Factor of Safety 1.8228

- **At 10 m water level:**

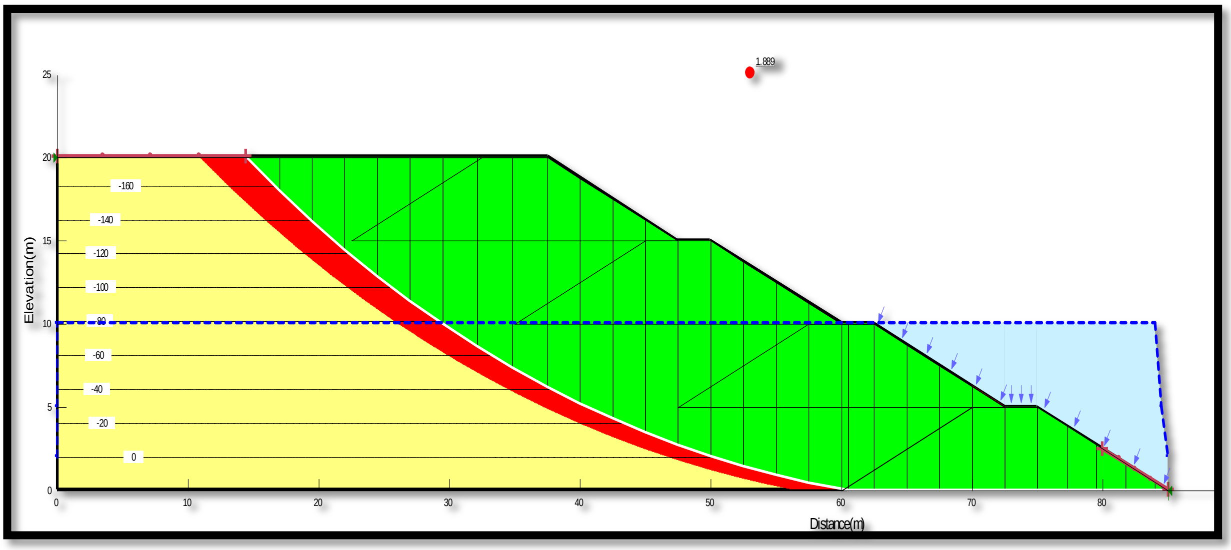

Figure 6.86 Embankment cross section for 1:2 slope at 10 m water level

Factor of Safety 1.8887

- **At 15 m water level:**

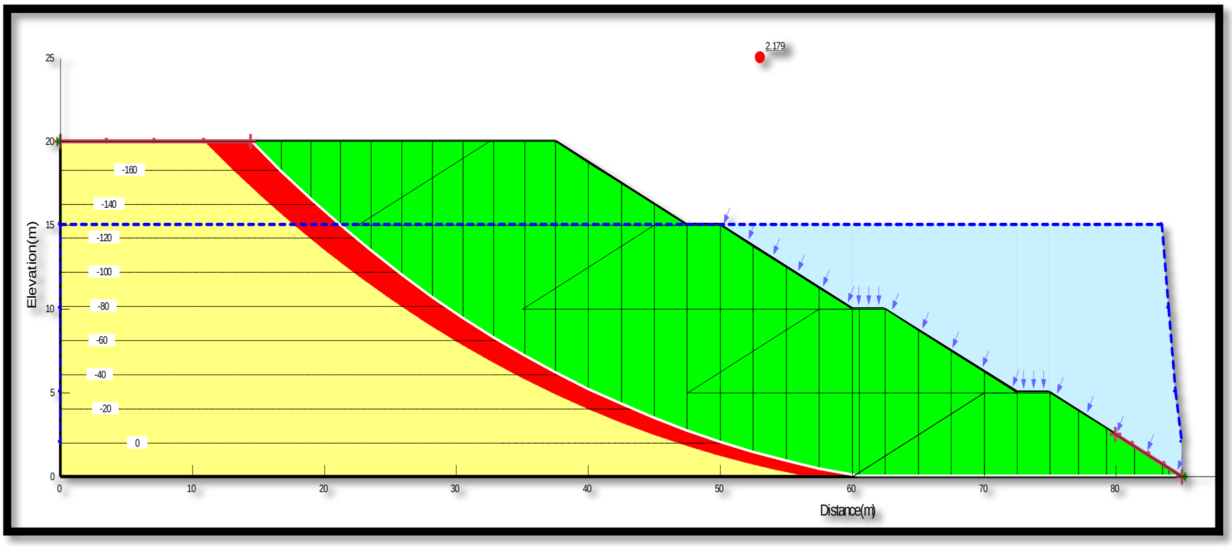

Figure 6.87 Embankment cross section for 1:2 slope at 15 m water level

Factor of Safety 2.1789

The comparison curve at different water level for 1:2 step slope are as follows:

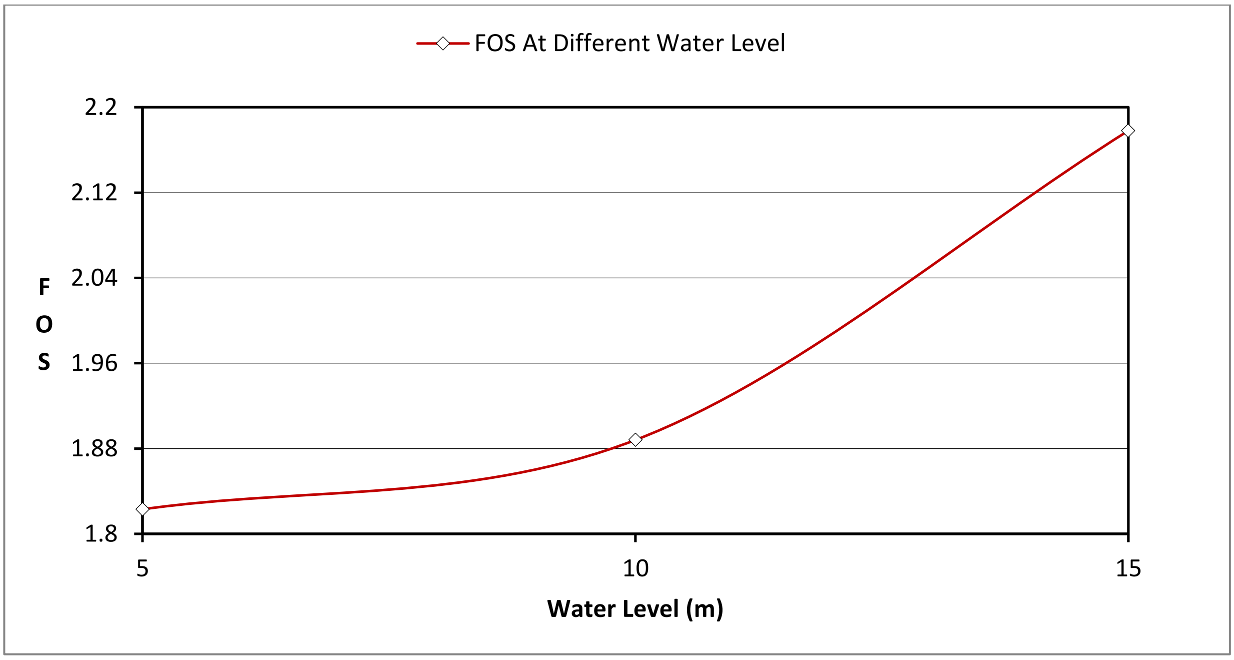

Figure 6.88 Variation of FOS at different water level

6.4.3 Analysis for 1:2.5 Step Slope

- **At 5 m water level:**

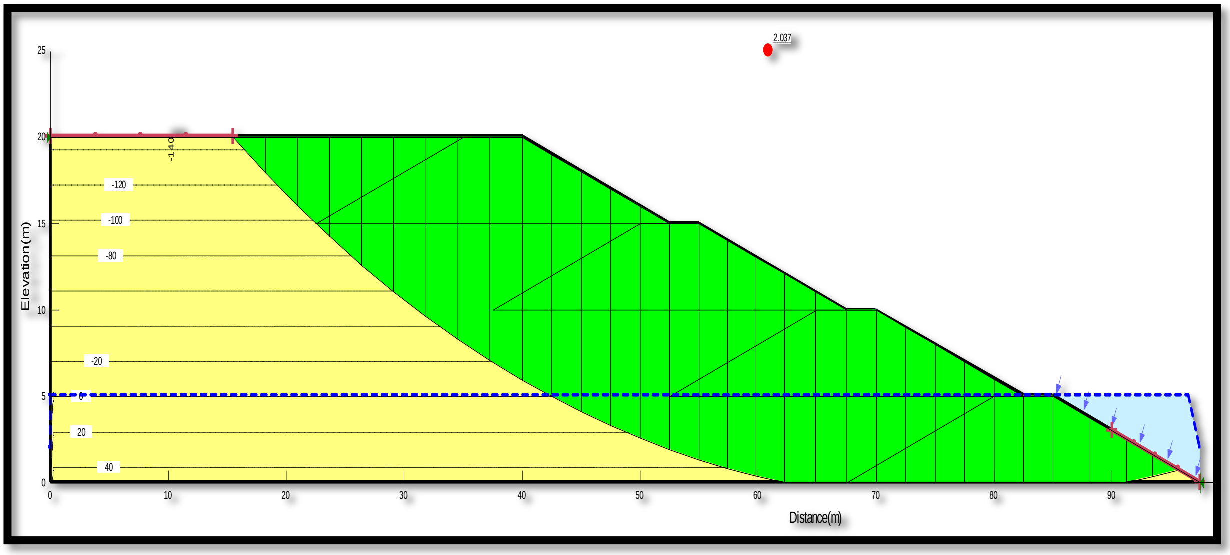

Figure 6.89 Embankment cross section for 1:2.5 slope at 5 m water level

Factor of Safety 2.0365

- **At 10 m water level:**

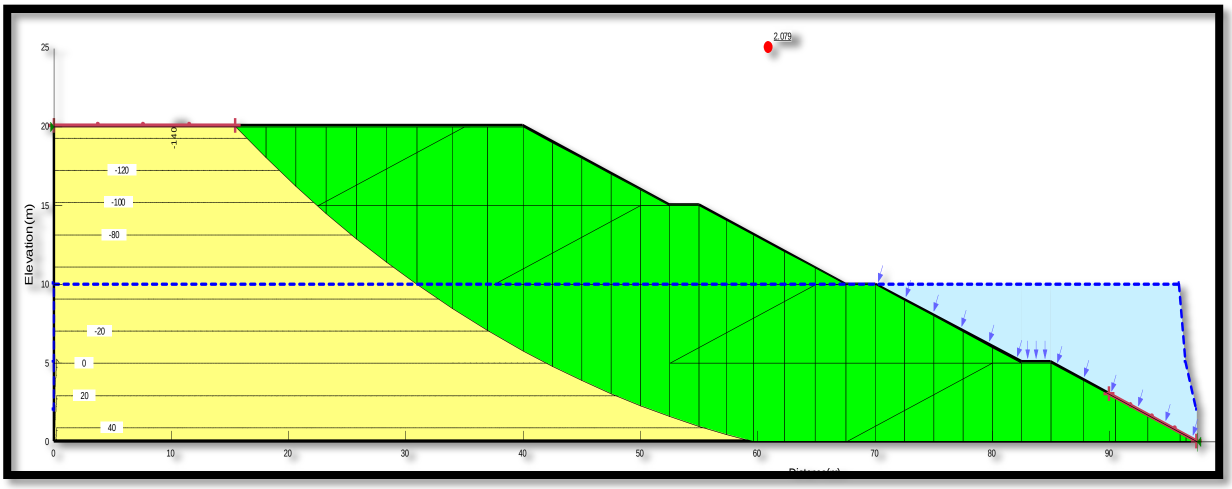

Figure 6.90 Embankment cross section for 1:2.5 slope at 10 m water level

Factor of Safety 2.079

- **At 15 m water level:**

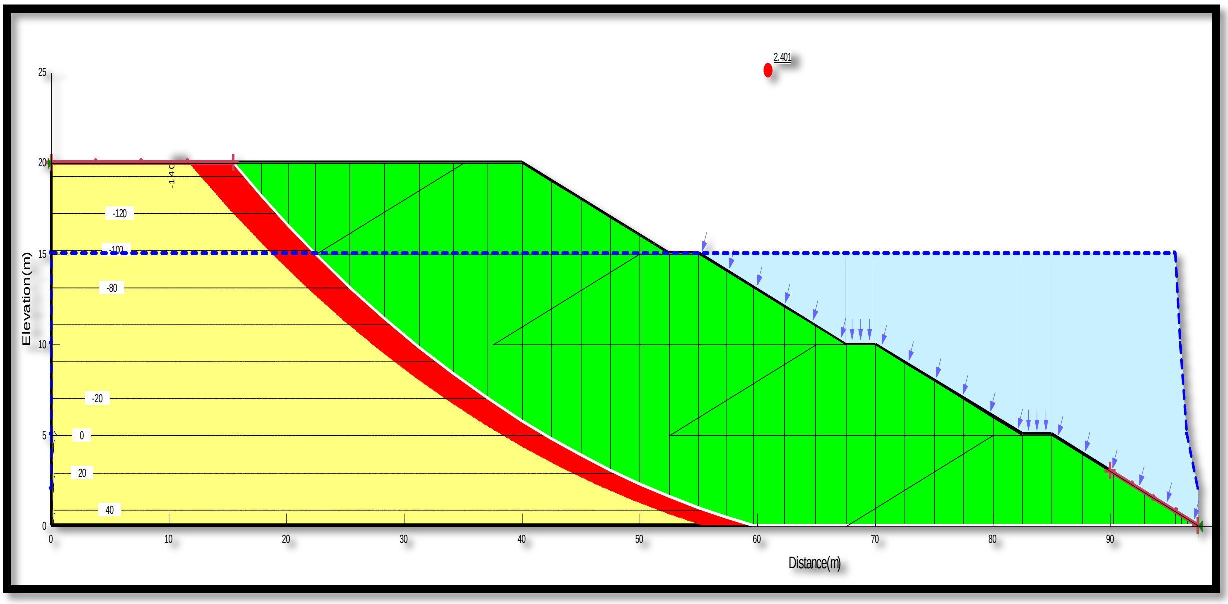

Figure 6.91 Embankment cross section for 1:2.5 slope at 15 m water level

Factor of Safety 2.4011

The comparison curve at different water level for 1:2.5 step slope are as follows:

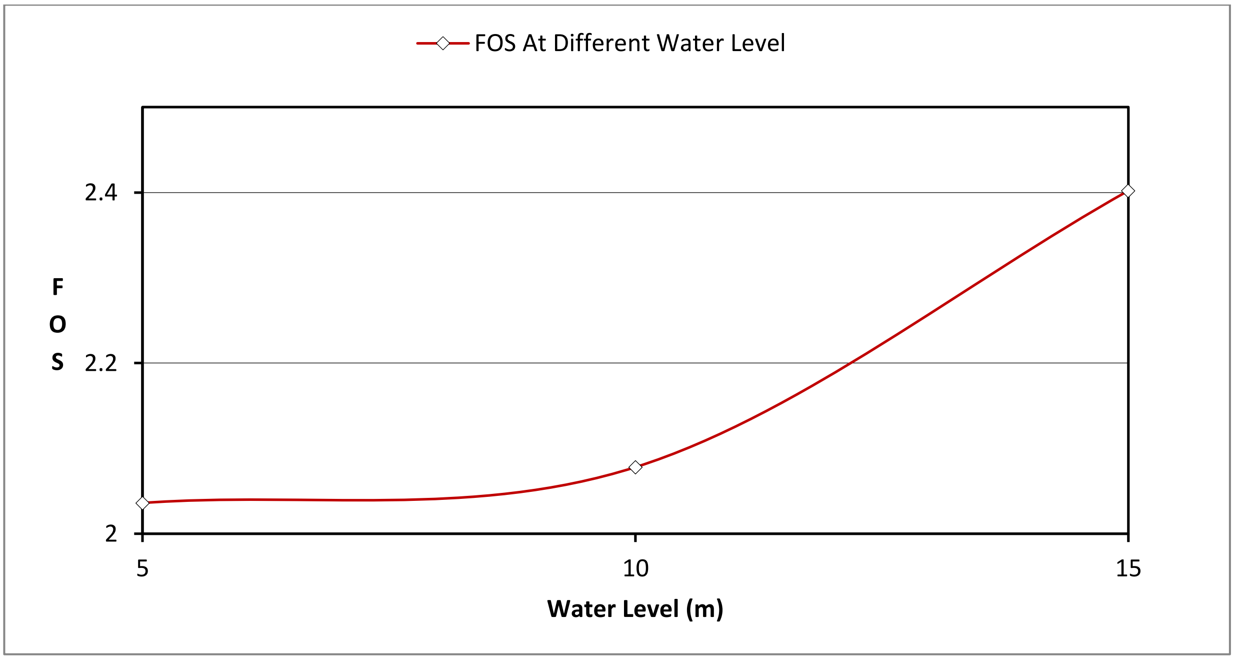

Figure 6.92 Variation of FOS at different water level

6.4.4 Analysis for 1:3 Step Slope

- **At 5 m water level:**

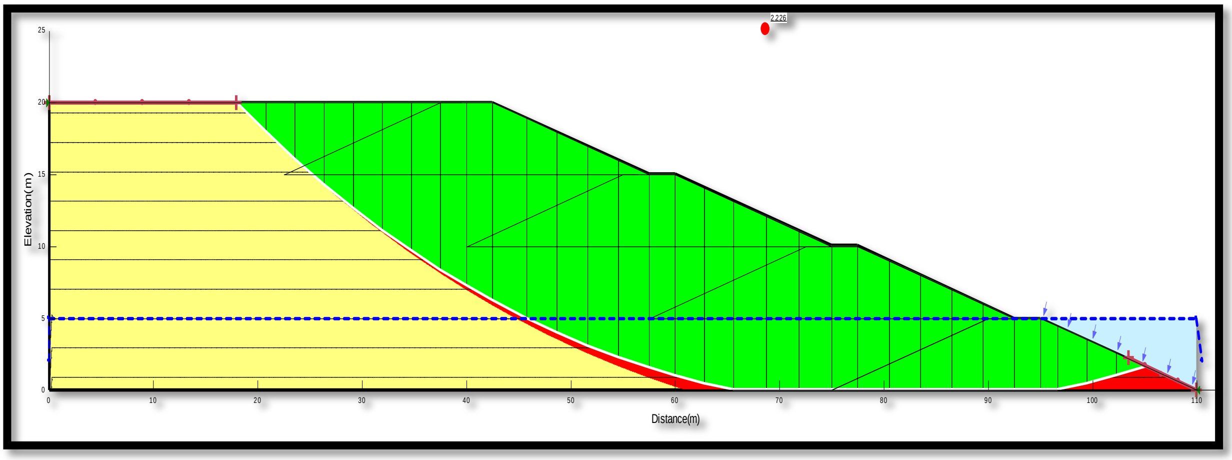

Figure 6.93 Embankment cross section for 1:3 slope at 5 m water level

Factor of Safety 2.2265

- **At 10 m water level:**

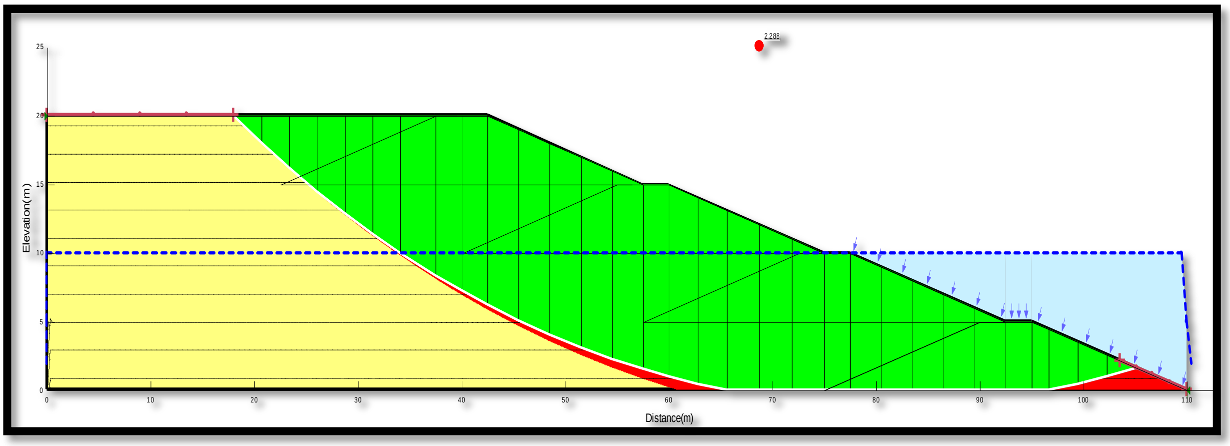

Figure 6.94 Embankment cross section for 1:3 slope at 10 m water level

Factor of Safety 2.288

- **At 15 m water level:**

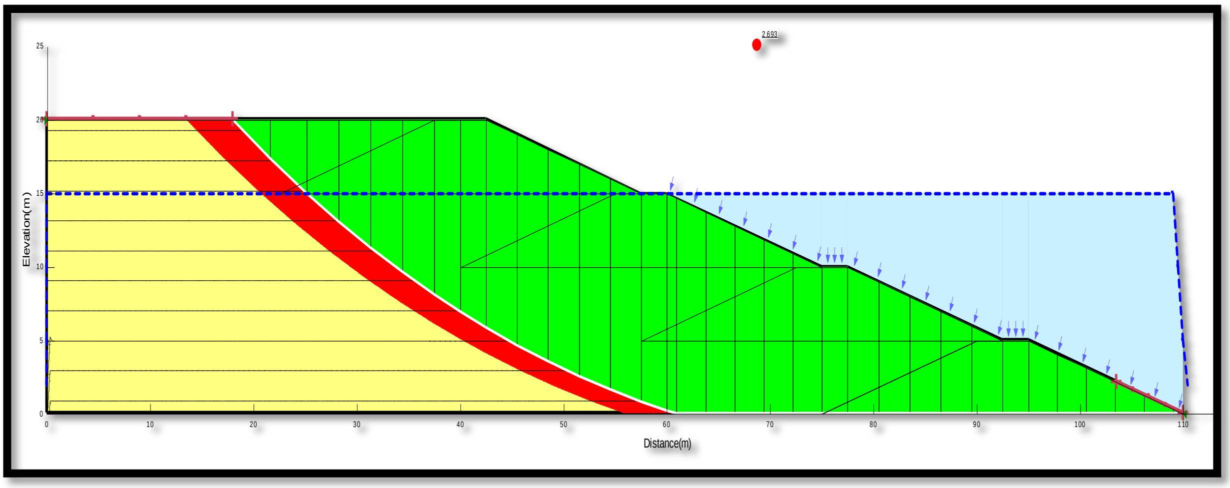

Figure 6.95 Embankment cross section for 1:3 slope at 15 m water level

Factor of Safety 2.6933

The comparison curve at different water level for 1:3 step slope are as follows:

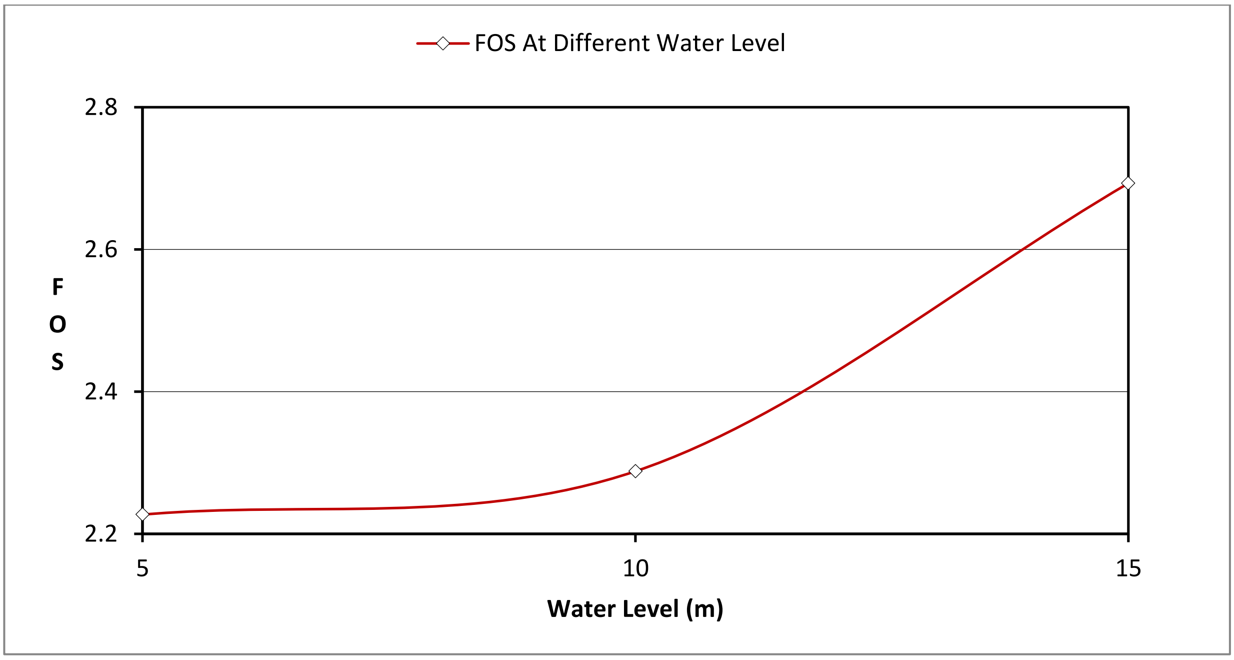

Figure 6.96 Variation of FOS at different water level

6.4.5 Analysis for 1:3.5 Step Slope

- **At 5 m water level:**

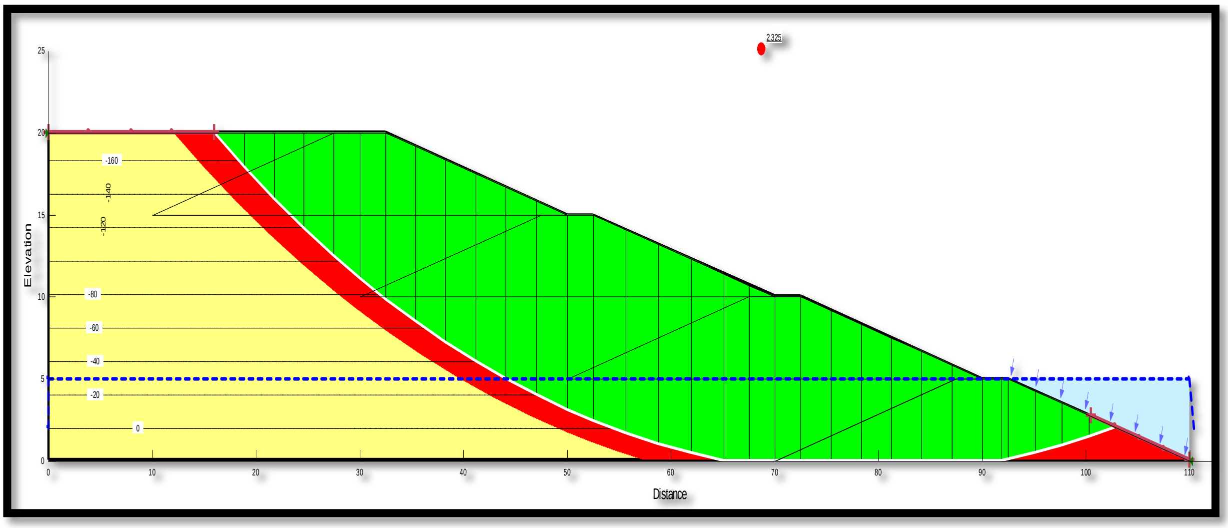

Figure 6.97 Embankment cross section for 1:3.5 slope at 5 m water level

Factor of Safety 2.3253

- **At 10 m water level:**

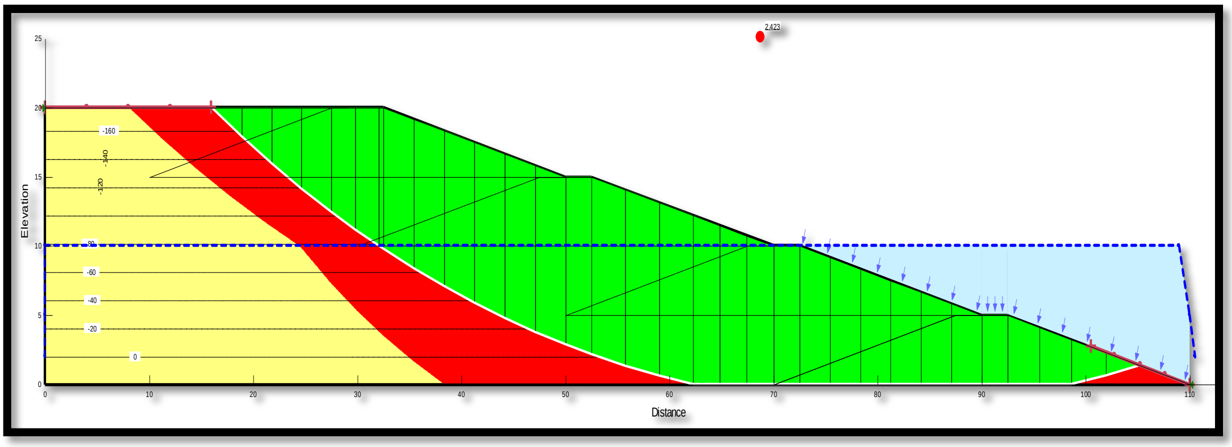

Figure 6.98 Embankment cross section for 1:3.5 slope at 10 m water level

Factor of Safety 2.422

At 15 m water level:

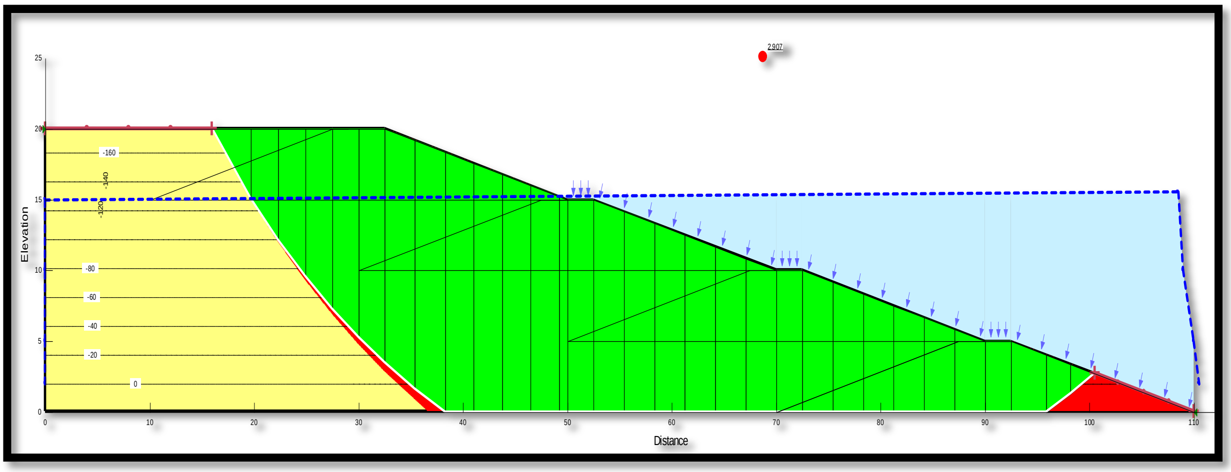

Figure 6.99 Embankment cross section for 1:3.5 slope at 15 m water level

Factor of Safety 2.9073

The comparison curve at different water level for 1:3.5 slope are as follows:

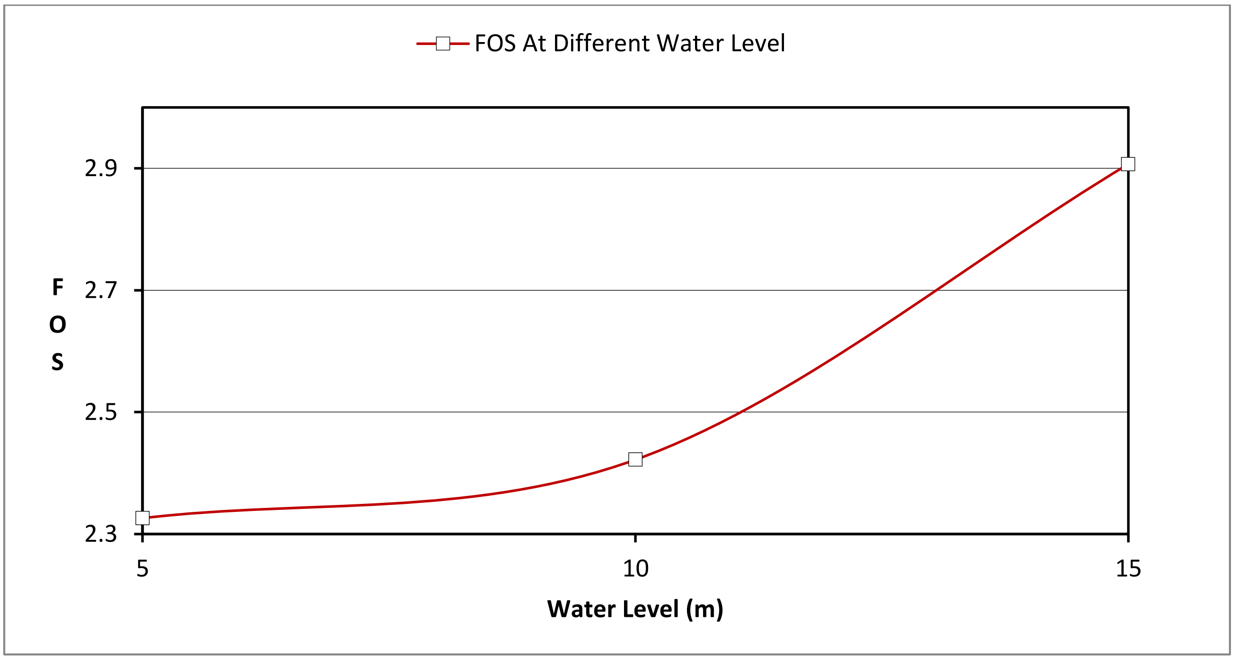

Figure 6.100 Variation of FOS with water level

6.4.6 Analysis for 1:4 Step Slope

- **At 5 m water level:**

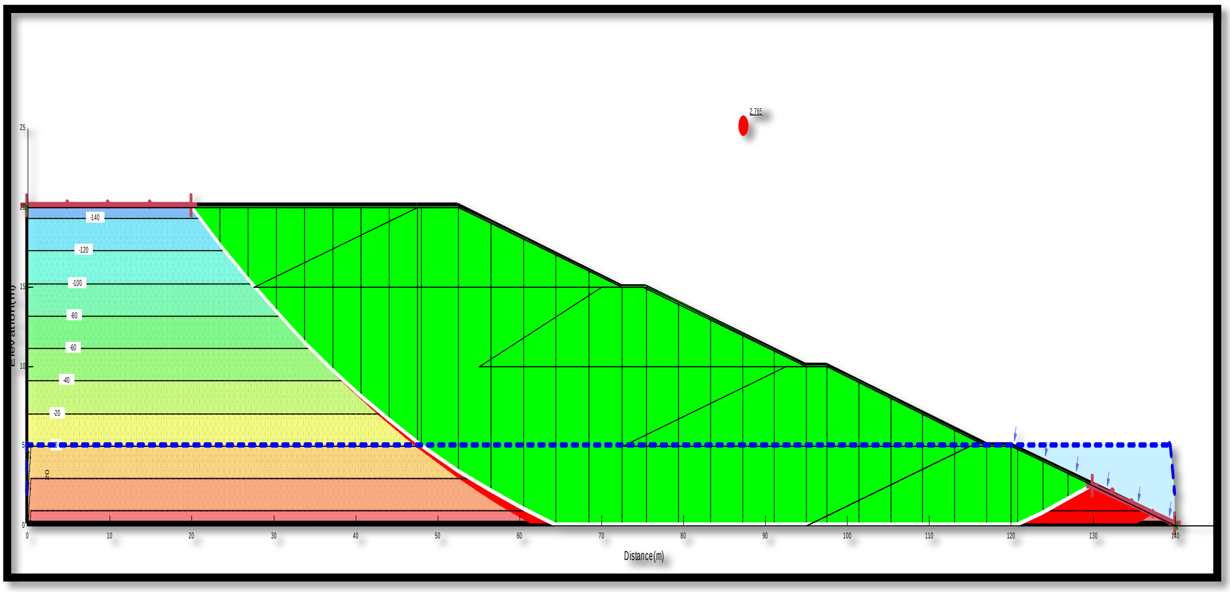

Figure 6.101 Embankment cross section for 1:4 slope at 5 m water level

Factor of Safety 2.765

- **At 10 m water level:**

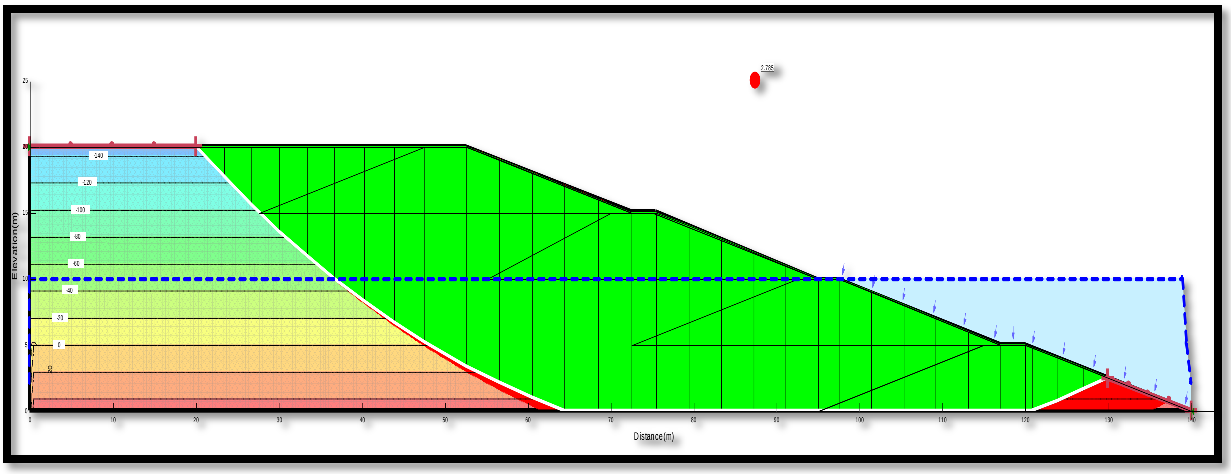

Figure 6.102 Embankment cross section for 1:4 slope at 10 m water level

- **At 15 m water level:**

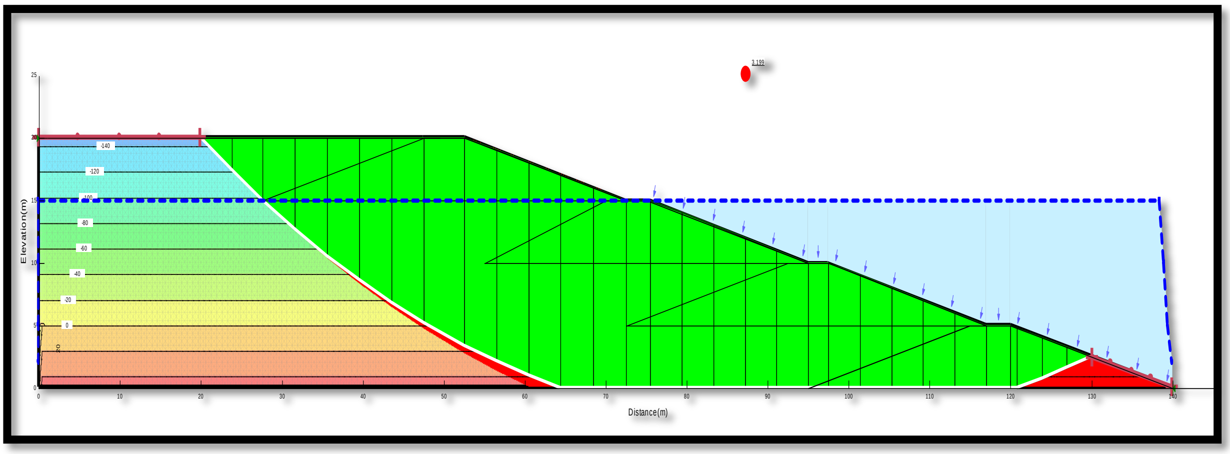

Figure 6.102 Embankment cross section for 1:4 slope at 10 m water level

Factor of Safety 3.1985

The comparison curve at different water level for 1:4 step slope are as follows:

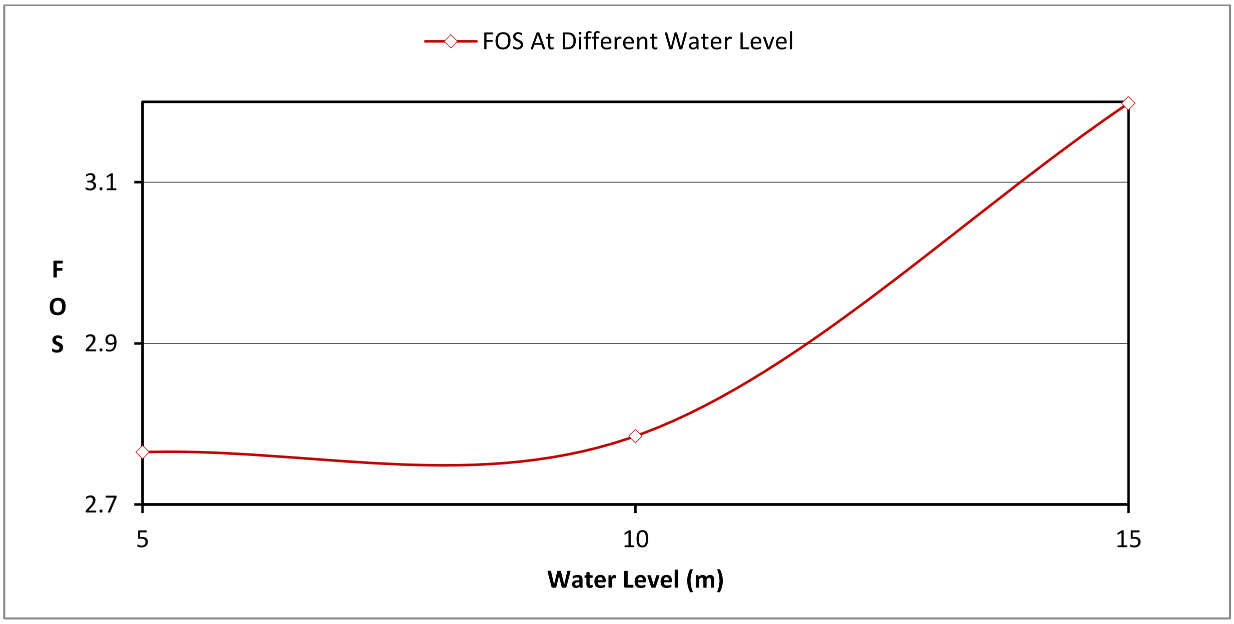

Figure 6.103 Variation of FOS at different water level

6.4.7 Analysis for 1:4.5 Step Slope

- **At 5 m water level:**

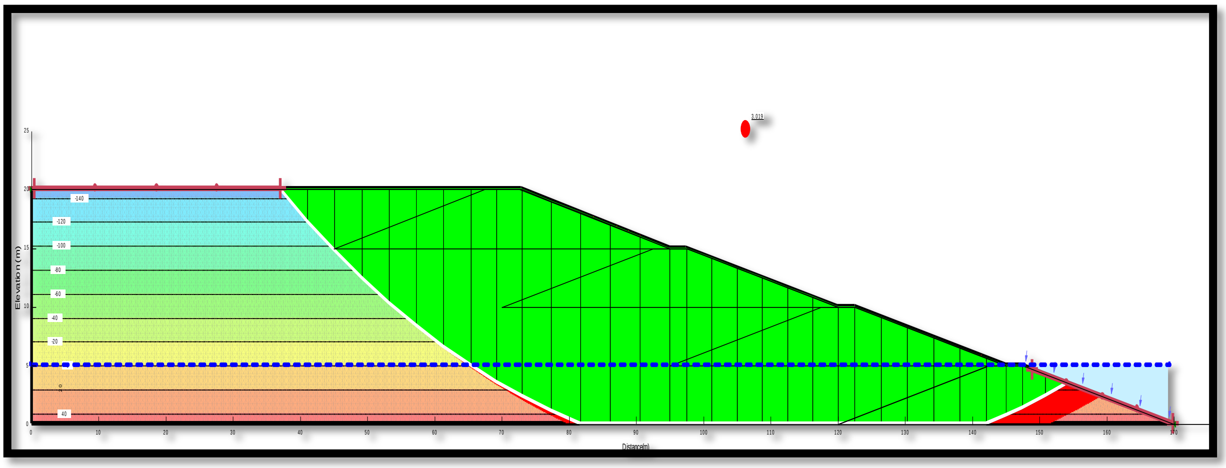

Figure 6.104 Embankment cross section for 1:4.5 slope at 5 m water level

Factor of Safety 3.0186

- **At 10 m water level :**

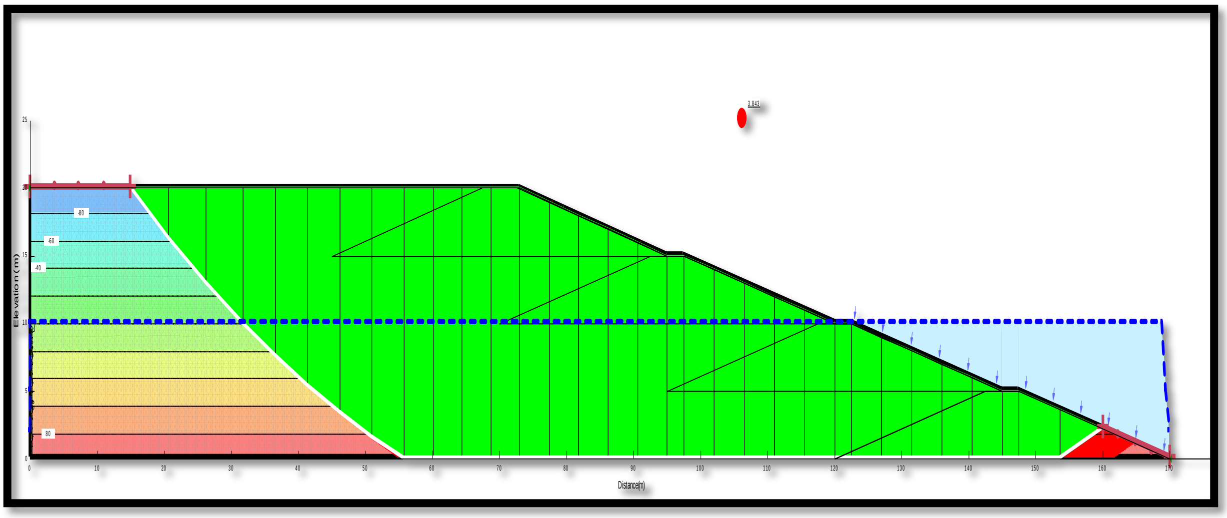

Figure 6.105 Embankment cross section for 1:4.5 slope at 10 m water level

Factor of Safety 3.8429

- **At 15 m water level:**

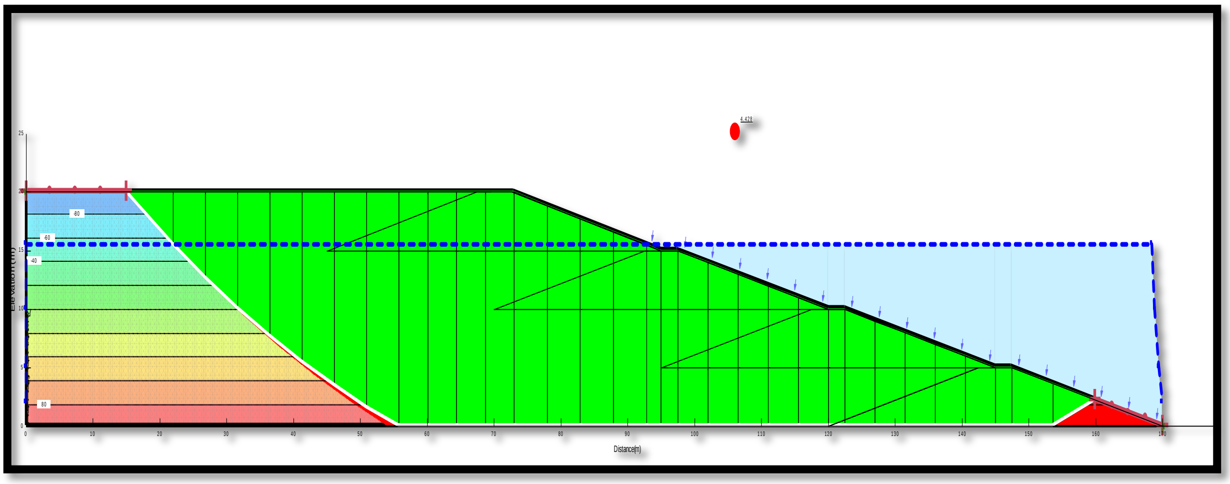

Figure 6.106 Embankment cross section for 1:4.5 slope at 15 m water level

Factor of Safety 4.4278

The comparison curve at different water level for 1:4.5 slope are as follows:

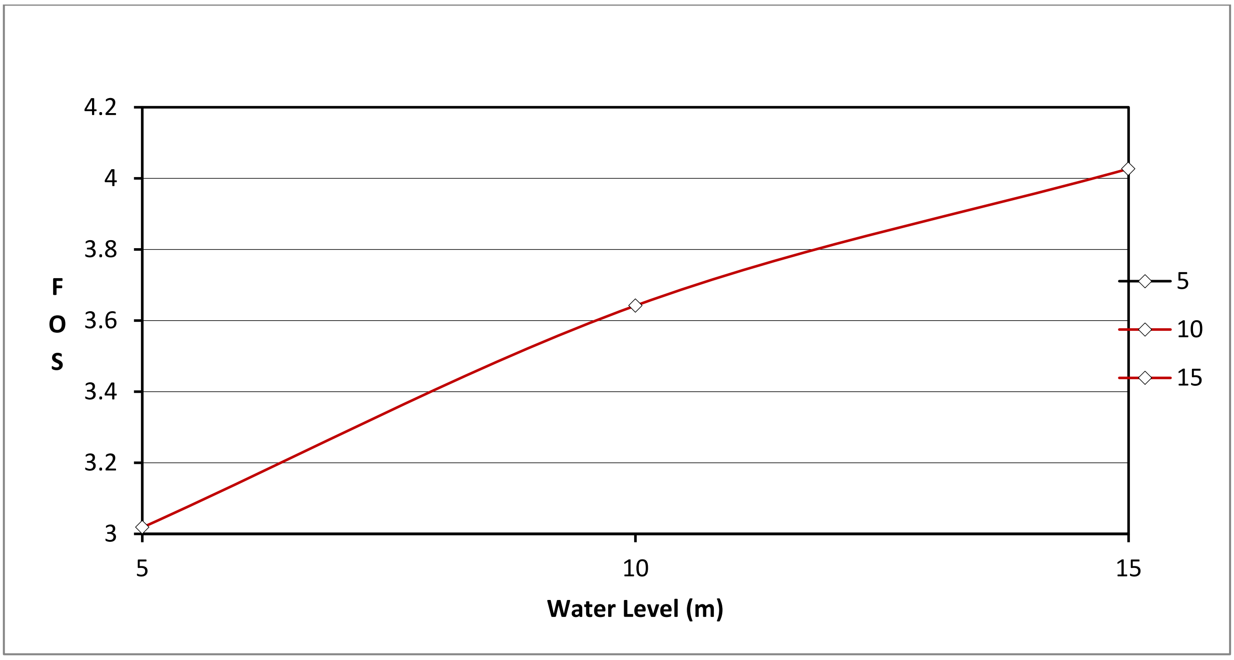

Figure 6.107 Variation of FOS at different water level

6.4.8 Analysis for 1:5 Step Slope

- **At 5 m water level:**

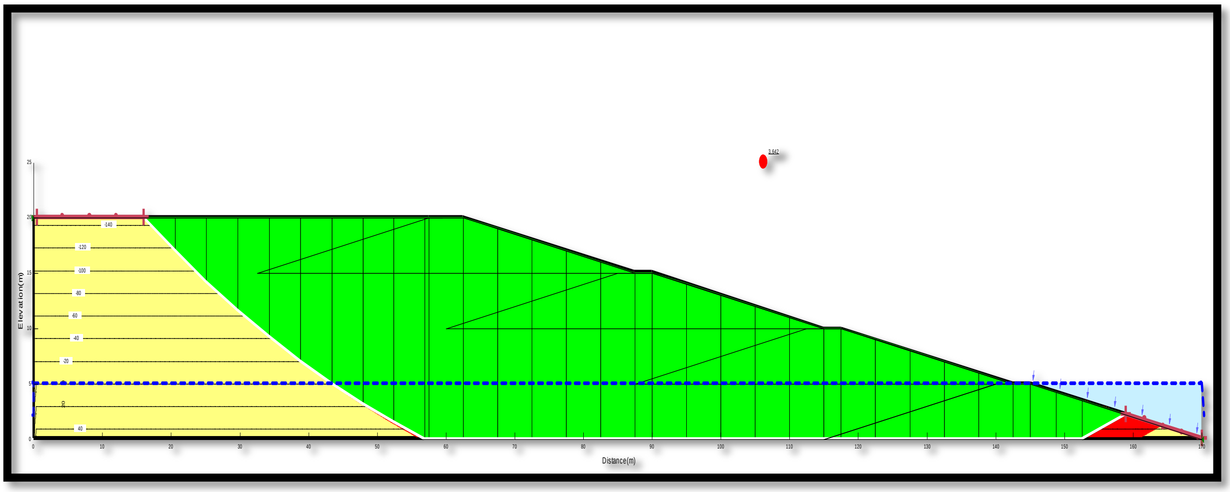

Figure 6.107 Embankment cross section for 1:5 slope at 5 m water level

Factor of Safety 3.6416

- **At 10 m water level:**

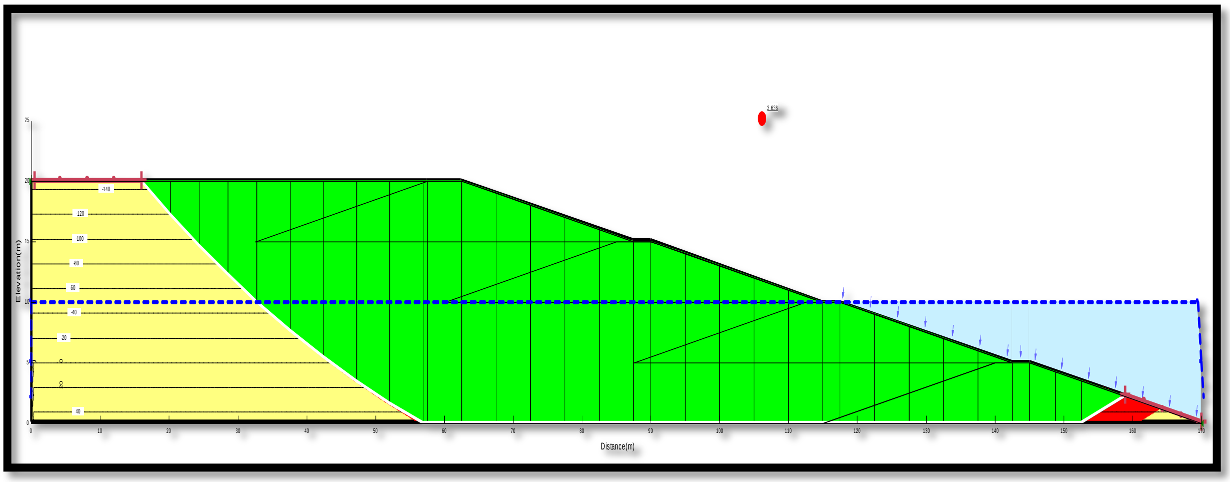

Figure 6.108 Embankment cross section for 1:5 slope at 10 m water level

Factor of Safety 3.6358

- **At 15 m water level:**

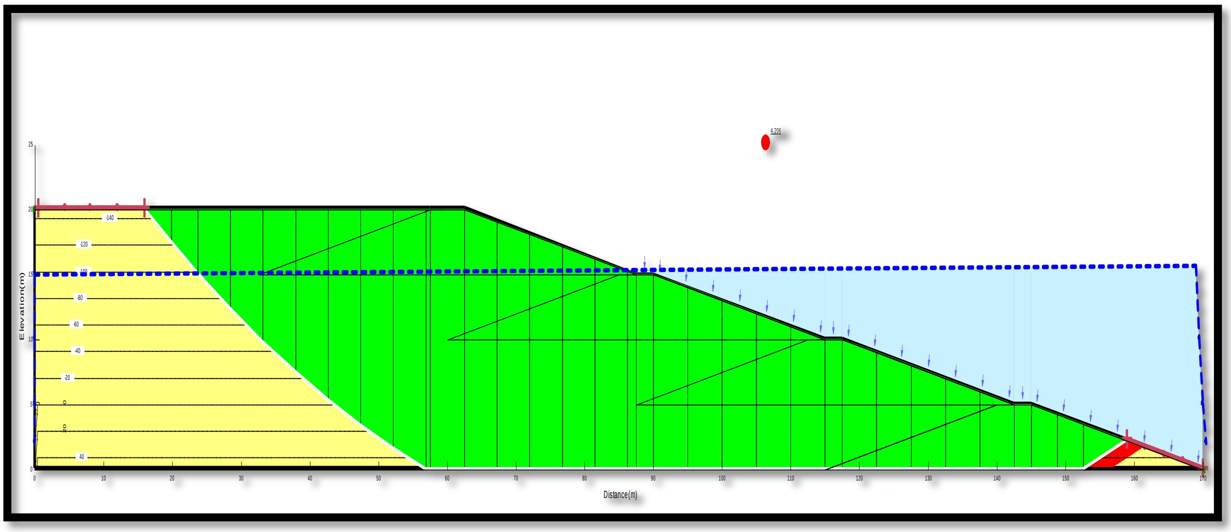

Figure 6.109 Embankment cross section for 1:5 slope at 15 m water level

Factor of Safety 4.2062

The comparison curve at different water level for 1:5 slope are as follows:

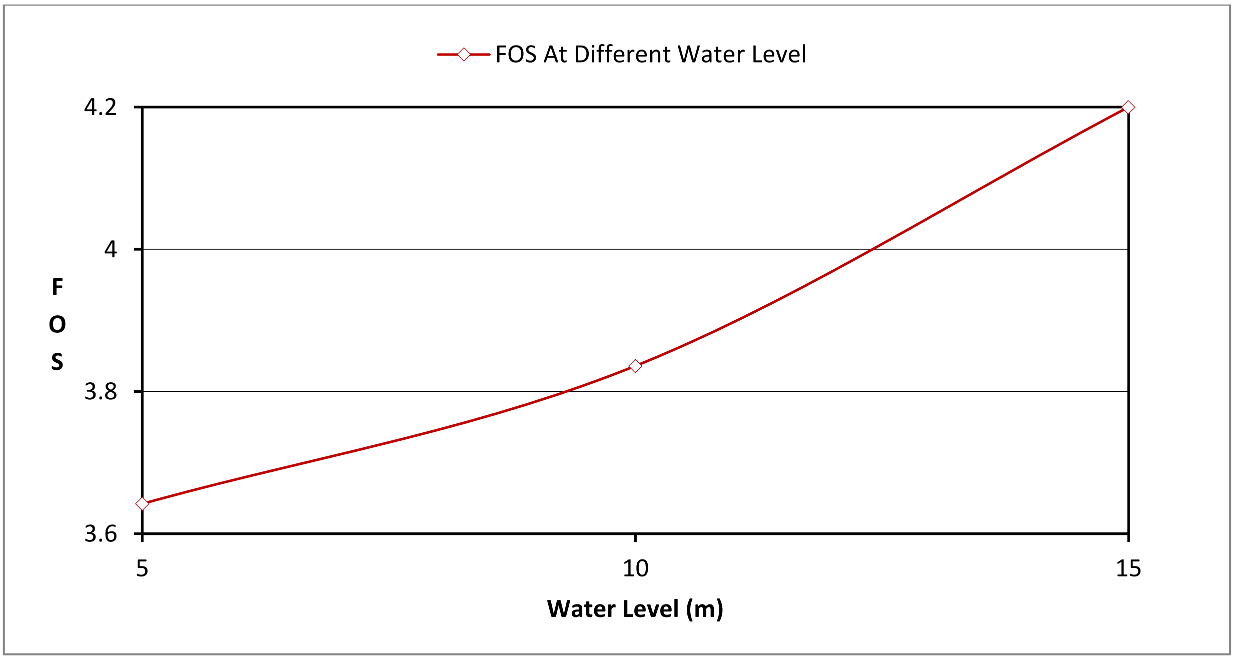

Figure 6.110 Variation of FOS at different water level

6.5 Comparison of FOS between simple and step slope

- **At 5 m water level**:

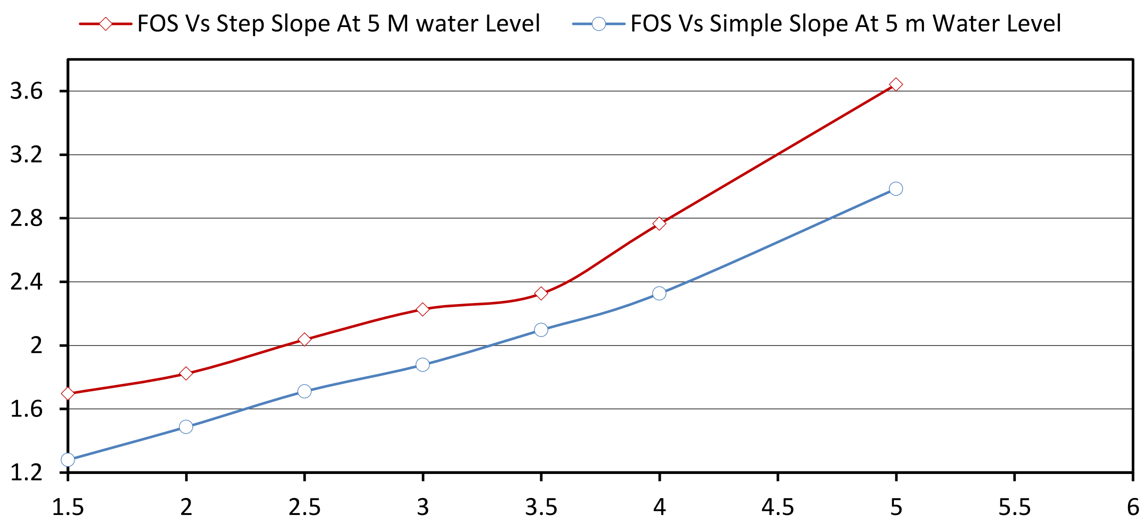

Figure 6.111 Comparison of FOS between simple and step slope at 5 m water level

- **At 10 m water level:**

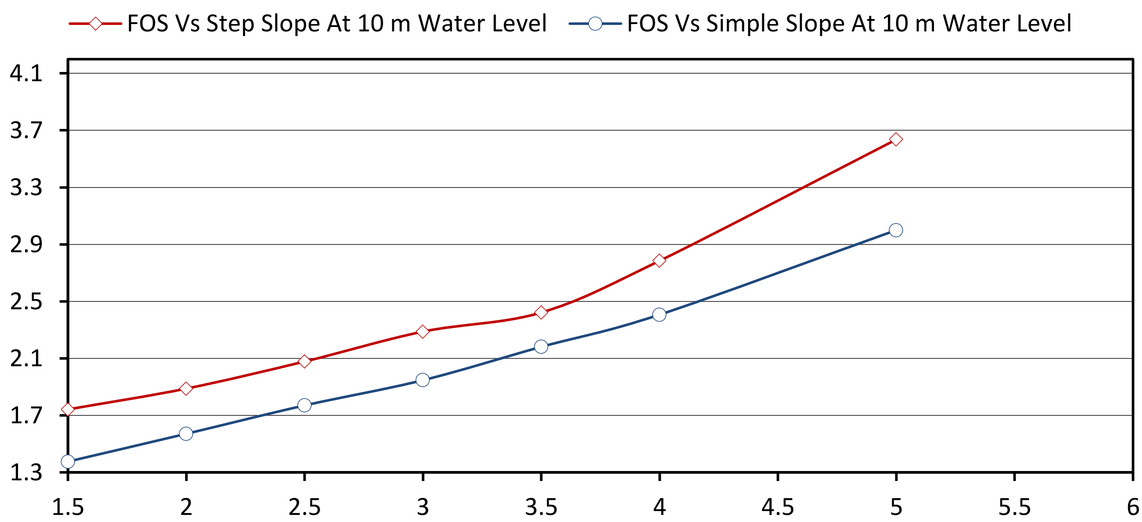

Figure 6.112 Comparison of FOS between simple and step slope at 10 m water level

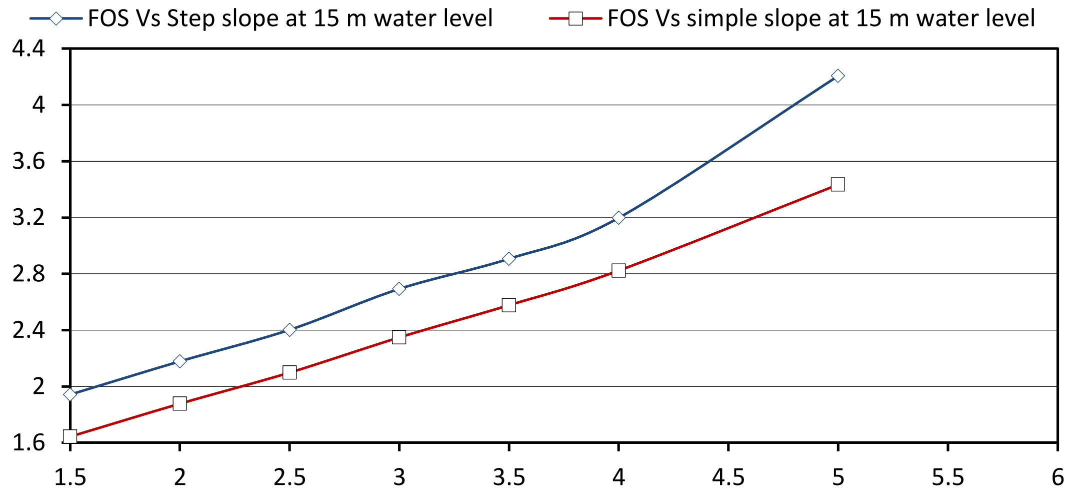

Figure 6.112 Comparison of FOS between simple and step slope at 15 m water level

6.6 Conclusion

- As the slope goes milder the factor of safety will be increased. The steeper the slope lesser will be factor of safety. In steep slopes the weight of the slope is more susceptible to fall than milder slopes.
- On increasing the water level the factor of safety is increased as pores in the soil is filled with water so pore water pressure is increased which holds the soil particles thus increasing the factor of safety.
- Building the dam in different number of stages enhances the factor of safety than compared to the single stage raised embankment. Thus building the dam step wise is advantageous.
- For different values of cohesion and friction angle the dams for different slopes are analysed for different factor of safety. It is concluded that on increasing the values of above parameters the factor of safety gets increased.
- Step slopes are more stable than simple slopes
- From above graphs it is seen that FOS is increased in step slopes as compared to simple slope at every slope value
- The FOS is increased as water level is increased.
- At each water level the FOS is increased in step slopes as compared to simple slope.

- The step slopes are less susceptible to failure.
- The upstream method of construction is advantageous as compared to the downstream and centreline method.

6.8 Future scope

- The upstream method of construction is preferable in future construction.
- As compared to simple slope step slope will be preferred in future construction.
- Sometime there is need to increase the height of the dam thus parametric study will be helpful to estimate the stability of the dam. Example, there is need to increase the height of a dam in southern part of India.
- The dams built in different stages are more stable than a single stage dam which will be preferred in future.

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APPENDIX A

Slope Stability

File Information

Last Edited By: ASHISH KASHYAP
Revision Number: 41
File Version: 8.2
Tool Version: 8.12.3.7901
Date: 21-07-2014
Time: 10:17:00
File Name: slope1.5.gsz
Directory: C:\Users\ASHISH\Desktop\
Last Solved Date: 21-07-2014
Last Solved Time: 10:17:08

Project Settings

Length(L) Units: meters
Time(t) Units: Seconds
Force(F) Units: kN
Pressure(p) Units: kPa
Strength Units: kPa
Unit Weight of Water: 9.807 kN/m³
View: 2D
Element Thickness: 1

Analysis Settings

Slope Stability

Kind: SLOPE/W
Method: Bishop
Settings
- PWP Conditions Source: Piezometric Line
- Apply Phreatic Correction: No
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 m
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

New Material

Model: Mohr-Coulomb
Unit Weight: 17.25 kN/m³
Cohesion': 31 kPa
Phi': 16 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (0, 20) m
Left-Zone Right Coordinate: (16, 20) m
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (50.5, 3) m
Right-Zone Right Coordinate: (54.89972, 0.066854) m
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 20) m
Right Coordinate: (55, 0) m

Piezometric Lines

Piezometric Line 1

Coordinates

	X (m)	Y (m)
Coordinate 1	0	2
Coordinate 2	55	2

Points

	X (m)	Y (m)
Point 1	0	0
Point 2	0	20
Point 3	25	20
Point 4	55	0

Regions

	Material	Points	Area (m^2)
Region 1	New Material	1,2,3,4	800

Current Slip Surface

Slip Surface: 123
F of S: 1.286
Volume: 286.51389 m^3
Weight: 4,942.3646 kN
Resisting Moment: 1,04,992.97 kN-m
Activating Moment: 81,659.008 kN-m
F of S Rank: 1
Exit: (54.89972, 0.066853538) m
Entry: (16, 20) m
Radius: 39.095381 m
Center: (50.232916, 38.882698) m

Slip Slices

	X (m)	Y (m)	PWP (kPa)	Base Normal Stress (kPa)	Frictional Strength (kPa)	Cohesive Strength (kPa)
Slice 1	16.642857	18.918389	-165.91864	-15.927784	-4.5672186	31
Slice 2	17.928571	16.892404	-146.0498	13.702371	3.9290917	31
Slice 3	19.214286	15.109255	-128.56246	40.985611	11.752435	31
Slice 4	20.5	13.516945	-112.94668	66.255981	18.998597	31
Slice 5	21.785714	12.08108	-98.865156	89.76692	25.74025	31
Slice 6	23.071429	10.777482	-86.080763	111.71496	32.033749	31
Slice 7	24.357143	9.5883728	-74.419172	132.2556	37.923682	31
Slice 8	25.681499	8.470371	-63.454929	145.41035	41.695747	31
Slice 9	27.044497	7.4179658	-53.13399	150.9542	43.28542	31
Slice 10	28.407495	6.457014	-43.709936	154.94688	44.430303	31
Slice 11	29.770493	5.5795704	-35.104846	157.4996	45.162284	31
Slice 12	31.133491	4.7791688	-27.255309	158.70305	45.507368	31
Slice 13	32.49649	4.0504838	-20.109095	158.63172	45.486915	31
Slice 14	33.859488	3.3890873	-13.622779	157.34704	45.118539	31
Slice 15	35.222486	2.7912719	-7.7600033	154.89968	44.416769	31
Slice	36.585484	2.2539182	-	151.33129	43.393548	31

16			2.4901753			
Slice 17	37.902234	1.788831	2.0709344	147.00925	41.560394	31
Slice 18	39.172735	1.3902645	5.9796758	141.97372	38.995663	31
Slice 19	40.443236	1.0385378	9.4290595	135.95318	36.280209	31
Slice 20	41.713738	0.7323537	12.431807	128.97231	33.417451	31
Slice 21	42.984239	0.47062171	14.998613	121.05093	30.410013	31
Slice 22	44.25474	0.25243725	17.138348	112.20453	27.259789	31
Slice 23	45.525242	0.077064928	18.858224	102.44453	23.967988	31
Slice 24	46.744443	0	19.614	94.938902	21.599068	31
Slice 25	47.912345	0	19.614	81.508035	17.747829	31
Slice 26	49.080246	0	19.614	68.077168	13.89659	31
Slice 27	50.248148	0	19.614	54.646301	10.045351	31
Slice 28	51.416049	0	19.614	41.215434	6.1941114	31
Slice 29	52.576335	0	19.614	31.640226	3.4484649	31
Slice 30	53.729005	0	19.614	25.920679	1.808411	31
Slice 31	54.60253	0.033426769	19.286184	23.835283	1.3044331	31