EVALUATION OF EMBANKMENT DAM FAILURE AND REMEDIAL MEASURE

(CASE STUDY OF GOMIT DAM IN AMHARA REGION)

BY

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ABSTRACT

The main objective of this research is to evaluate the causes of Gomit embankment dam failure, emphasis on seepage and slope failure using Geo studio softwares and finally to come up with the possible remedial measure. Analysis of seepage was done using seepage analysis methods by analytical using Phreatic line and numerical using Geo studio software of Seep/W) method at normal and current pool level, compared with the current measured seepage at downstream i.e.0.004m^3/s. The result shows that there was excessive seepage passing through the dam body and foundation. Analysis of static slope stability is performed to assess the safe design of upstream and downstream slopes of the dam. Slope/W software is used to calculate the factor of safety under the standard loading conditions for limit equilibrium methods of Ordinary, Bishop, Janbu and Morgenstern–Price method and compared with the observed downstream slope failure at steady state seepage. The result shows the dam was safe under this loading condition. However, from field observation, the downstream slope was failed. The major cause for downstream slope failure is the steady state loading condition in which the shear strength parameters of cohesion and angel of internal friction(**C** and Φ) value at current condition don't attain the real value stated at the design document. The current **C** and Φvalue, the critical slip surface and the minimum factor of safety obtained from the back calculation using slope failure model just at failure. The calculated factor of safety just at failure is 0.996, which is less than the stability safety factor (FOS=1) and resulting slope failure at downstream that was similar to observed failure. Finally, since the dam was endangered to public life and property, the possible remedial measures proposed for seepage control are grouting, impervious blanket, rehabilitating the existing filter and toe-drains, adding both toe drains and drainage zones, and for downstream slope failure by removing the weak zone and failure part and fill with similar material, raise and re-constructing berms, regular maintenance and cover the downstream slope with grass.

Key words - Seep/W, Slope/W, Limit equilibrium method, Factor of safety

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LIST OF ACRONYMS

AAU	Addis Ababa University
C	Cohesion of Soil
CH	High Plasticity Clay
D/S	Downstream
FOS	Factor Of Safety
Ff	Force equilibrium
Fm	Moment equilibrium
GW	Well graded Gravel
Kh	Permeability along horizontal
Kv	Permeability along horizontal
LE/LEM	Limit Equilibrium/Method
M-PM	Morganstern-Price Method
NPL	Normal Pool Level
OM/OMS	Ordinary method/of Slice
Mv	Volume of Compressibility
Phi	Angel of Friction
RDD	Rapid Draw Down
U/S	Upstream

1. Introduction

1.1 Background

The historical records of dam indicate that on the average about 10 significant dam failures have occurred somewhere in the world in each decade and many more damaging near failures have occurred (safety of Dams, Flood and Earthquake criteria, National Academy Press,1985). Embankment dams have been involved in the largest number of dam failures, followed in order by gravity dams, rock fill dams and multiple and single arch dams.

An embankment dam is a massive artificial dam that are more common than any other type of dams because of various reasons like the use of ordinary technology construction method utilizing cheap raw soil materials and subsurface materials, no need of a particular valley shape etc. The geometry of embankment dams depends on burrowed soil materials, subsurface conditions and type of construction. It is typically created by the placement and compaction of a complex semi-plastic mound of various compositions of soil, sand, clay and/or rock. It has a semi-pervious waterproof natural covering for its surface and a dense, impervious core. This makes such a dam impervious to surface or seepage erosion.

The two principal types of embankment dams are earth and rock-fill dams, depending on the predominant fill material used. Earthen dams are composed of suitable soils obtained from borrow areas or required excavation which are then spread and compacted in layers by mechanical means. They require very large quantity of materials. It is necessary to utilize the soils available in large quantities near the site. The construction of earthen dam, upto1930 was based mostly on experience. But now with the advance knowledge of soil mechanics, these dams are designed and constructed scientific basis, with the increased knowledge of the behaviour of soils and the development of earth moving machinery, earth dams can be constructed economically even up to the height of 250m to 300m. Earth dams may be constructed as homogeneous or zoned dams.

Homogenous embankment is a dam which made up of only one type of material that available at the vicinity of dam site. These dams are easy and cheap to construct but cannot be used to make multipurpose large dams. For large multipurpose dams, zoned embankment dam, which made up of different material, is used. Zoned dams are constructed from distinct and separable fill zones and generally preferred since zoning permits the use of several different material types in the embankment that may be available from borrow areas or required excavations.

All earthen dams are subjected to some seepage passing through, under and around them. If uncontrolled, seepage may be detrimental to the stability of the structure as a result of excessive pore water pressure or by internal erosion. Generally, flow through the dam and foundation causes seepage forces, pore water pressures and hydraulic gradients. If these forces are not limited to allowable ranges, they may render some severe problems such as instability of slopes, piping, etc, which may lead to failure of the dam. So seepage should be effectively controlled to preclude structural damage or interference with normal operation. (Safety of Dams: Flood and Eathquake Criteria." National Academy Press, Washington, D.C, 1985).

Earthen dams must function safely under everyday operations as well as under unusual conditions such as excessive leakage, instability and major damage during floods and earthquakes. The structural stability of an earthen dam is to be assessed to ensure that the safety of people and property and if a risk dos exist, corrective actions need to be taken. Conditions for earthen dam to be safe and stable; (1) The dam should not be overtopped by flood water (2) The seepage line should be well within the downstream face of the dam (3) The upstream and downstream face should be stable under the worst conditions (4) There should be no opportunity for free flow of water from upstream to downstream (5) The foundation shear stresses should be within the safe limits (6) The upstream slope should be protected from wave action and borrowing animals (7) The dam and foundation should be safe against piping.

Dam safety does not depend only on proper design and construction but also on monitoring instruments that installed in the dam body and foundation. An effective dam safety monitoring is essential for dams to check the design weather optimized or not, to manage the risk associated with operation and maintenance. Frequent instrument reading and making field observation use to provide data and information which can be used to assess the performance of the dam. Many embankment dams are constructed in Ethiopia most of which are used for irrigation purpose. However, their capacity reduces frequently before their design life time due to a number of reasons. The main causes of capacity reduction are Hydrological, Structural and Hydraulic failure of which hydraulic failures contributes 58% in Amhara region [Tefra B. 2006].

The Gomit dam constructed by Commission for Sustainable Agricultural and Environmental Rehabilitation in Amhara Region (Co-SAERAR) to alleviate the food shortage problems of the drought affected area of the Estie woreda through the use small scale irrigation project, to promote and encourage sustainable agricultural crop production. The dam was built for irrigating 90 ha of command area. However, at current condition the dam was exposed to both seepage and slope failure. In this study, dam failure analysis, seepage and slope stability has done using analytical and numerical methods for the standard or critical loading condition to evaluate the previous and current performance of the dam. Absence of dam monitoring instruments and measured data, absence of relevant data's in the design document are the main challenges of this thesis work.

1.2 Statement of the problem

The main problem of Gomit embankment dam is seepage through the dam body, foundation and right abutment and slope face failure at the downstream. So the study focuses on the causes of embankment dam failure and to put appropriate measure. Additionally, the current performance of the dam (the amount of seepage passing through the dam body and foundation) analyzed by different methods and check with the actual records at downstream.

.1.2 Objective of the Research

1.2.1General Objective

The general objective of the research is to evaluate Gomit embankment dam failure and to come up with possible remedial measure.

1.2.2 Specific Objective

The specific objectives of the research are;-

- To evaluate seepage through the dam body and foundation
- To evaluate the static stability of upstream and downstream of the dam

- To evaluate the current performance of the dam

- To recommend the possible remedial measure for the result of the study

1.3 Scope of the Research

The scope of the research focusing on the analysis of embankment dam failure, emphasis on seepage and structural/slope failure of the dam and to put appropriate remedial measure for the result of the study. The study is based on the available data on the design document that kept on Amhara Region Water Resource Development Office. The research does not include the hydrological failure analysis (overtopping, overturning instability), settlement and deformation (stress-strain) analysis, dynamic/earthquake analysis and failure by foundation sliding.

2. LITRETURE REVIEW

2.1Embankment Dam Failure

Failures of earthen embankment dams can generally be grouped into three classifications: hydraulic, seepage and structural. (Garg, S.K. (2005). “Irrigation Engineering and Hydraulic Structures,” Nineteenth Revised Edition)

2.1.1 Hydraulic Failures

Above 40% of embankment dam failures are hydraulic failures due to the reason from the uncontrolled flow of water over and adjacent to the embankment lead to the erosive action of water on the embankment slopes. Embankment dam are not normally designed to be overtopped and therefore are particularly susceptible to erosion. A well vegetated earth fill dam may withstand limited overtopping if its top is level and water flows over the top and down the face in an evenly distributed sheet without becoming concentrated in any one area.

(i) Overtopping of dams

Over topping failures result from the erosive action of water on the embankment. Erosion is due to un-controlled flow of water over, around, and adjacent to the dam. Earth embankments are not designed to be over-topped and therefore are particularly susceptible to erosion. Once erosion has begun during over-topping, it is almost impossible to stop. A well vegetated earth embankment may withstand limited overtopping if its crest is level and water flows over the crest and down the face as an evenly distributed sheet without becoming concentrated. The owner/operator should closely monitor the reservoir pool level during severe storms. When free board of dam or capacity of spillway is insufficient, the flood water will pass over the dam and wash it downstream and lead to total dam failure.

(ii)Erosion of downstream toe

Heavy cross-current water from spillway buckets, heavy rainfall and tail water erode the toe of the dam at the downstream and lead to dam failure.

(iii) Erosion of the Upstream face

During winds, the waves developed near the top water surface may cut into the soil of upstream dam face which may cause slip of the surface leading to failure.

(iii) Erosion of the Downstream face

Due to rainfall, snow and winds the downstream surface of the dam also erodes. During heavy rains, the flowing rain water over the downstream face can erode the surface, creating gullies, which could lead to failure.

2.1.2 Seepage Failures

More than 30% of embankment dam failures are due to seepage. Seepage always occur in earth dams. It does not harm for stability if it is with in the design limits or controlled in velocity and quantity. Seepage occurs through the body of earthen embankment and its foundation. Uncontrolled seepage can erode fine soil material from the downstream slope or foundation and continue moving towards the upstream slope to form a pipe or cavity to the pond or lake often leading to a complete failure of the embankment. Seepage can also cause slope failures by saturating the slope material, thereby weakening the adhesive properties of the soils and its stability. Burrows or holes created by animals such as the groundhog, woodchuck, or muskrat create voids in the embankment or dike, which weaken the structure and may serve as a pathway for seepage. Tree roots can provide a smooth surface for seepage to travel along. When trees die, their decaying roots may leave passageways for seepage to concentrate in. Pipes through the embankment may also provide smooth surfaces for seepage to concentrate along as well. Studying the causes of destruction in 200 destroyed embankment dam around the world, about 25% of destruction has been due to wash out of the fine granules of the body or the dam foundation (Foster and Fell 1999). The major seepage failures are;-

(i) Piping through the dam body

When seepage starts moving through poor soils in the body of the dam, small channels are formed and which transport fine materials to downstream. As more materials are transported to the downstream, it result in gradual formation of drain (pipe) from upstream to downstream through which water flows and thus lead to dam fails.

(ii) Piping through the foundation

When highly permeable strata of gravel, sand or cavities are present in the foundation of dams, it permits heavy seepage of water through it and causing erosion of soil which will result in the formation of piping. The concentrated seepage at high rate will erode soil which will cause increase flow of water and soil. As a result, the dam will settle or sink and leading to failure.

(iii) Sloughing of downstream side of the dam

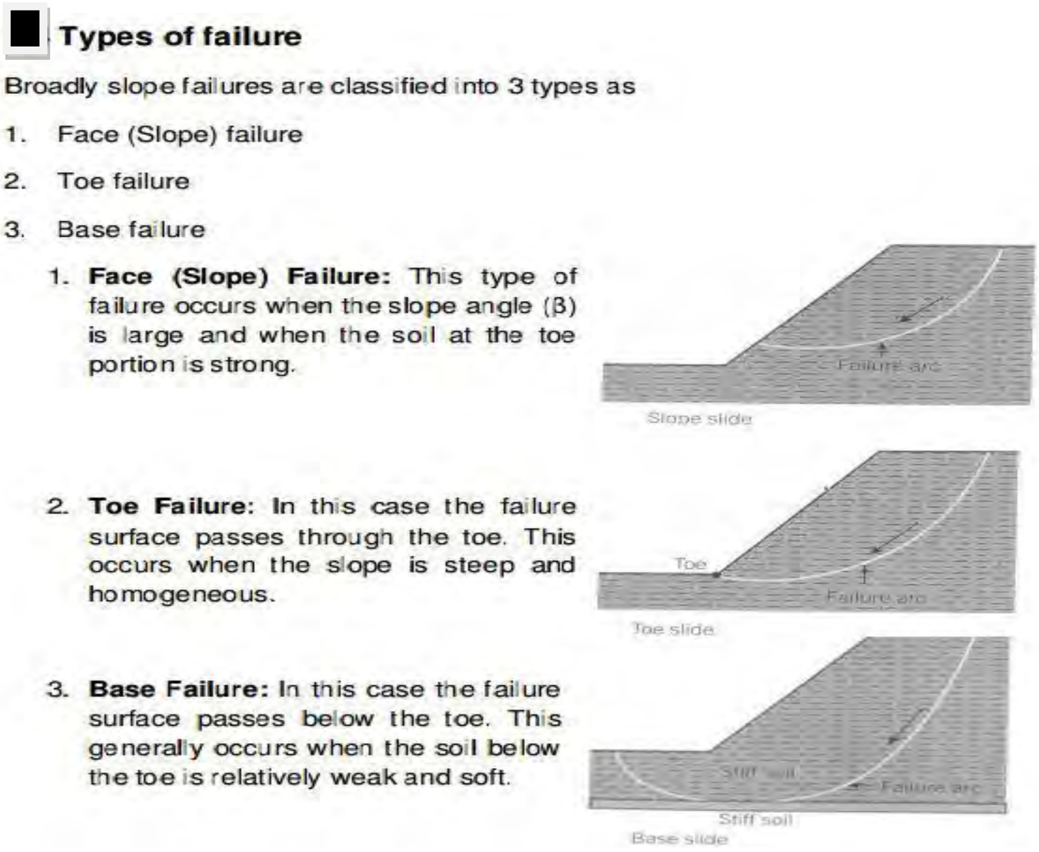
The process of failure due to sloughing starts when the downstream toe of the dam becomes saturated internally and starts eroded, causing small slump or slide of the dam. The small slide leaves a relative steep face, which also becomes saturated doe to seepage and also slumps again and form more unstable surface. The process of saturation and slumping continues, leading to failure of dam.

2.1.3 Structural Failures

About 25% to 30% of failure is attributed to structural failure, which is mainly due to shear failure causing slide along the slopes. Structural failures involve the separation (rupture) of the embankment materials and/or its foundation. This type of failure is more prominent in large embankment dams. Structural failure of an earthen embankment may take on the form of a slide or displacement of material in either the downstream or upstream face. Sloughs, bulges, cracks or other irregularities in the embankment generally are signs of serious instability and may indicate structural failure. The failure may be due to;

(i) Slide in embankment

When the slopes of the embankment are too steep, the dam may slide causing failure. This might happen when there is a sudden or rapid drawdown, which is critical for the upstream side because of the development of extremely high pore pressures, which decreases the shearing strength of the soil. The downstream side can also slide especially when steady state seepage occur. Upstream embankment failure is not as serious as downstream failure.



Types of failure

Broadly slope failures are classified into 3 types as

1. Face (Slope) failure
2. Toe failure
3. Base failure

1. **Face (Slope) Failure:** This type of failure occurs when the slope angle (β) is large and when the soil at the toe portion is strong.
2. **Toe Failure:** In this case the failure surface passes through the toe. This occurs when the slope is steep and homogeneous.
3. **Base Failure:** In this case the failure surface passes below the toe. This generally occurs when the soil below the toe is relatively weak and soft.

Fig-1 Type of slope failure of embankment dam

(ii) Foundation slide

When the foundation of an earth fill dam is composed of fine silt, clay, or similar soft soil,the whole dam fail due to water thrust. If seams of fissured rocks, such as soft clay, or shale exist below the foundation, the side thrust of the water pressure may shear the whole dam and cause its failure. In such failure the top of the dam gets cracked and subsides, the lower slopes moves outward and forms large mud waves near the dam heel.

2.1.4 Others

(i) Faulty construction and poor maintenance

When during construction, if the compaction of the embankment is not properly done(under or over compaction),poor quality of compaction water, wrong placement of materials in different zones, blind drains due to mixing of soils, after construction poor maintenance of the dam like non regular repair of gullies, rain cuts, settlement, pitching will lead to total dam failure.

(ii) Earthquake

It may cause the following types of failure to earth fill dam;

- Cracks may develop in the core wall causing leakage and piping failure
- Slow waves may setup due to shaking of reservoir bottom, dam Settlement which may reduce freeboard and fault movement in the dam site reducing reservoir capacity and causing overtopping.
- Shear slide of the dam and failure of slope pitching.

2.2 Seepage Analysis

2.2.1 Seepage through the Dam and Foundation

Flow of water through soils is called seepage. Seepage takes place when there is difference in water levels on the two sides of the structure such as a dam or a sheet pile. All embankment dams have some seepage as the impounded water seeks paths of least resistance through the dam and its foundation. Seepage will become a problem only if it carries dam materials also along with it. Seepage must be controlled to prevent the erosion of embankment or its foundation. One of the main problems in design of an earth dam is the amount of seeping water leaks through the body and its foundation. Therefore an accurate estimate of the amount of seeping water is very important from the economical and technical points of view. Flow of water in the body and foundation of an earth dam causes seepage forces, pore water pressures and hydraulic gradients. If these forces are not limited to allowable ranges, they may render some severe problems such as instability of slopes, piping, etc., which may lead to failure of the dam.

Seepage can emerge anywhere on the downstream face, beyond the toe, or on the downstream abutments at elevations below normal pool. Seepage may vary in appearance from a "soft" wet area to a flowing "spring." It may show up first as an area where the vegetation is lush and darker green. Cattails, reeds, mosses, and other marsh vegetation often become established in a seepage area. Another indication of seepage is the presence of rust-colored iron bacteria. Due to their nature, the bacteria are found more often where water is discharging from the ground than in surface water. If the seepage forces are large enough, soil will be eroded from the foundation and be deposited in the shape of a cone around the outlet. If these "boils" appear, professional advice should be sought immediately. Seepage flow which is muddy and carrying sediment (soil particles) is evidence of "piping," and is a serious condition that if left untreated can cause failure of the dam. Piping can most often occur along a spillway or other conduit through the embankment, and these areas should be closely inspected.

2.2.2 Purpose of Seepage Analysis

Dams must be designed and maintained to safely control seepage. Excessive seepage leads to dam safety issues, if not treated carefully. Seepage analyses are carried out for the following reasons;-

- To estimate the phreatic surface within an embankment
- To estimate pore pressures within an embankment or foundation
- To estimate exit gradients and/or uplift pressures at the toe of an embankment
- To estimate the amount of seepage flow that may pass through an embankment or foundation,
- To evaluate the relative effectiveness of various seepage reduction measures,
- To estimate the amount of seepage flows intercepted by drainage features and to size and optimize the configuration of these types of drainage features,
- To evaluate the effectiveness of, or to aid in the design of, dewatering systems. All these are factors taken into consideration while suggesting remedial measures in existing dams.

2.2.3 Seepage Analysis Method

There are various methods available for determining the amount of seepage passing through the dam. Some are the analytical, numerical and graphical methods.

The analytical methods of solving the seepage equations in porous environments are based on solving the governing differential equations using simplifying hypotheses. Those hypotheses are acceptable in certain conditions and therefore, the scope of their applications is limited due to the problems with special geometry and boundary conditions. The analytical calculation of seepage in dams has received many efforts. Analytical method by phreatic line was used for an embankment dam to calculate the amount of seepage passing through the dam body.

Assumption for seepage flow

- Both soil media and water are incompressible (no change in volume, density=constant)
- Flow is laminar and Darcy's Law is valid.
- Seepage water flow under hydraulic gradient only (i.e. difference in piezometric head)
- Steady state seepage (Flow characteristics do not change with the change in time)
- Hydraulic boundary condition at entry and exit are known.

2.2.3.1 Analytical using Phreatic Line

Line of seepage or Phreatic line is defined as the line with in a dam section below which there are positive hydrostatic pressures in the dam. On the line itself the hydrostatic pressure is equal to atmospheric pressure or zero. Above the saturation line there will be a zone of saturation in which the hydrostatic pressure is negative. The saturation line should not strike the downstream face of the dam. Minimum cover over the seepage line should be 2m. The appreciable flow through the dam body below phreatic line reduces the effective weight of the soil. And thus reduces the shear strength of the soil due to pore pressure.

It is therefore absolutely essential to determine the position of the phreatic line, as in position will enable us to determine the following things;-

- It gives us a divide line between the dry and submerged soil. The soil above the seepage line will be taken as dry and the soil below the seepage line shall be taken as submerged for computation of shear strength of soils.
- It represents the top streamline and hence, help us in drawing the flow net.

- Seepage line determination helps to ensure that it does not cut the downstream face of the dam. This is extremely necessary for preventing softening or sloughing of the downstream slope of the dam.

There are different analytical methods for determining the phreatic line depends on the type of filter drain and the angle between the discharge face the phreatic line and the amount of seepage passing through the body of earthen dam can be calculated as follows.

Homogeneous dam with Horizontal filter

The horizontal filter pushes down the seepage line (discharge face horizontal), except the upstream face it has parabolic shape and the amount of seepage passing through the dam body calculated using Kozney's analytical solution.

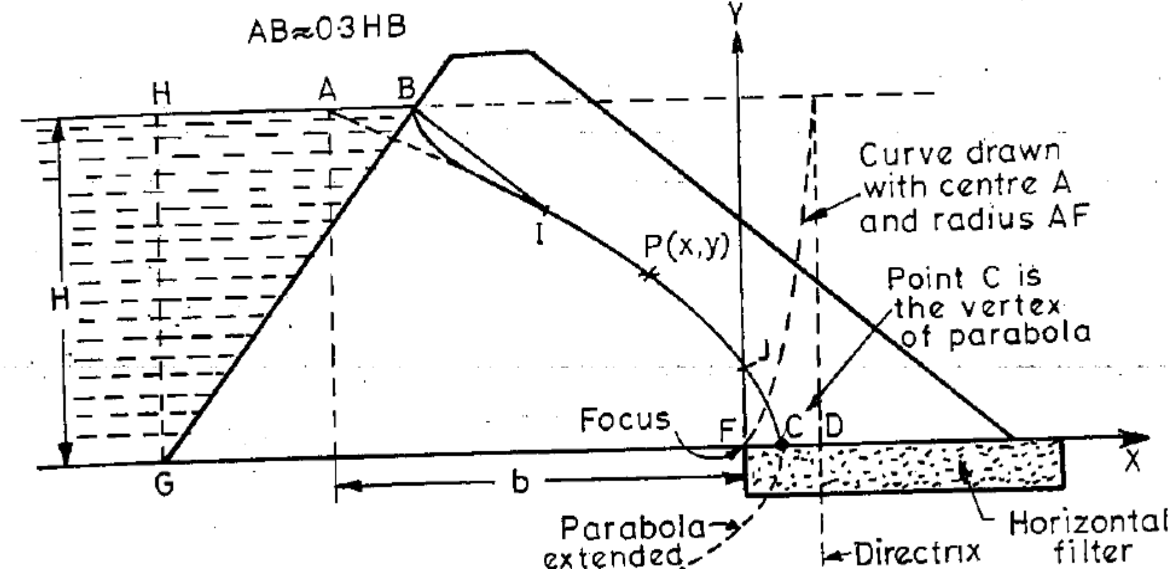


Fig-2 Homogeneous Earthen dam with horizontal filter drain

The Kozney's Analytical formula fora=180^0 (for horizontal filter)

$S = (b^2+h^2)^{0.5} - X$

$Y = (S^2+2SX)^{0.5}$

At F; X = 0 , Y= S

At point C (vertex); Y= 0 , X = -0.5S

The discharge (m3/s) through the body of the dam per unit length

$$Q= K S$$

Where, K is the hydraulic conductivity (m/s)

Homogeneous dam without filter

Phreatic line analyses vary with the type of dam, its drainage system within it and angle on inclination on the downstream face of the dam. Therefore, the analysis for a homogeneous dam without any drainage system and its angle of inclination less than 90^0 is, Casadragde has shown that the Phreatic line coincides with the base parabola, provided the slope of the of the downstream face is flat. Schaffernake and Van Iterson gave an approximate analytical solution for determination of the distance a, the phreatic line cuts the downstream face from the toe, for the slope angle $\alpha < 30^0$

Schaffernak and Iterson's solution for $\alpha < 30^0$

$$Q = K\ a \sin\alpha \tan\alpha$$

$$a = b/\cos\alpha - (b^2/\cos^2\alpha - h^2/\sin^2\alpha)^{0.5}$$

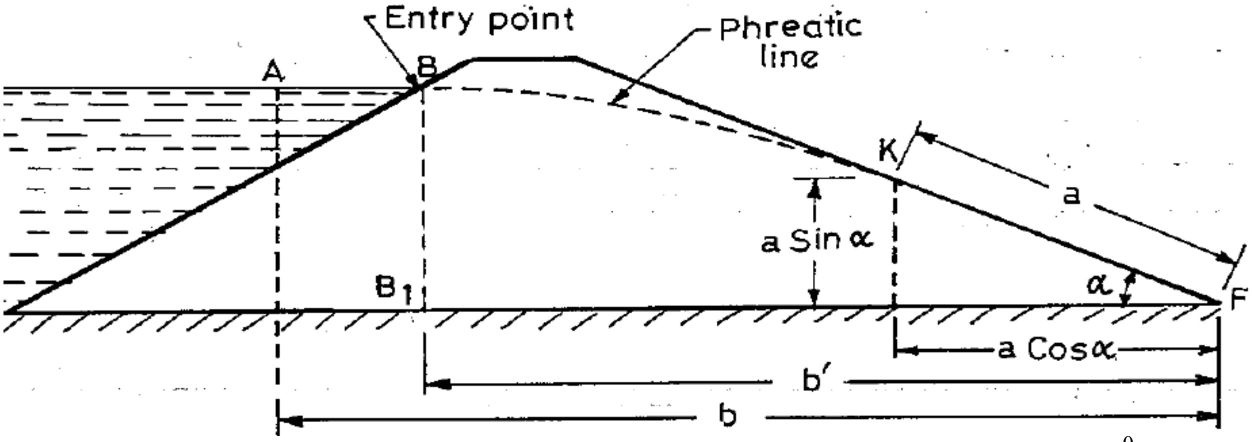


Fig-3 Homogeneous dam without filter and the discharge face(a)< 30^0

L.Cassagrande solution

$Q = Ka\sin^2\alpha$

For α between 30^0 and 60^0,

$a = (b^2+h^2)^{0.5}-(b^2-h^2\cot^2\alpha)^{0.5}$

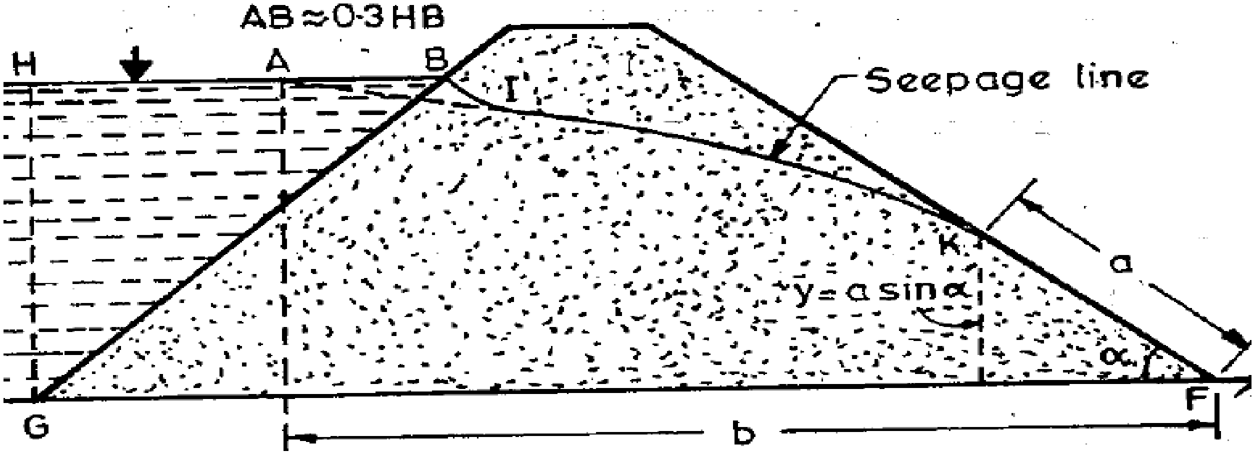


Fig-4 Homogeneous dam without filter and the angel of discharge face a<60^0

Cassagrande has given a general solution to evaluate Δa for various inclination of discharge face. Let a be the angel which the discharge face makes with the horizontal. The various values of $\Delta a/a+\Delta a$ have been given by Cassagrande, as shown in table below

Table1 Cassagrande value for various inclination angle of discharge face with horizontal

a in degree	$\Delta a/(a+\Delta a)$	Remarks
30^0	0.36	Note:- Intermediate values can be interpolated, or read from a graph between a and
60^0	0.32	

90^0	0.26	$\Delta a/(a+\Delta a)$ plotted with the values given here.
120^0	0.18	
135^0	0.14	
150^0	0.1	
180^0	0	

$(a+\Delta a)$ is the distance of the focus from the point where the parabola cuts the downstream face. Δa can be connected by a general equation of

$$\Delta a = (a + \Delta a)\left[\frac{180^\circ - \alpha}{400^\circ}\right]$$

Homogeneous dam with rock toe

In a homogeneous dam a rock pieces or boulders larger than 20cm size are provided to prevent slogging of the toe due to seepage flow and increase the stability of the dam. The height of rock toe is usually kept 30 to 40% of the reservoir head. It keeps the phreatic line well within the section and also facilitates drainage. The∂(angle between the bottom/base of the dam and filter drain) rock toe either 90^0 or greater than 90^0 and the solution using L.Cassagrande described above.

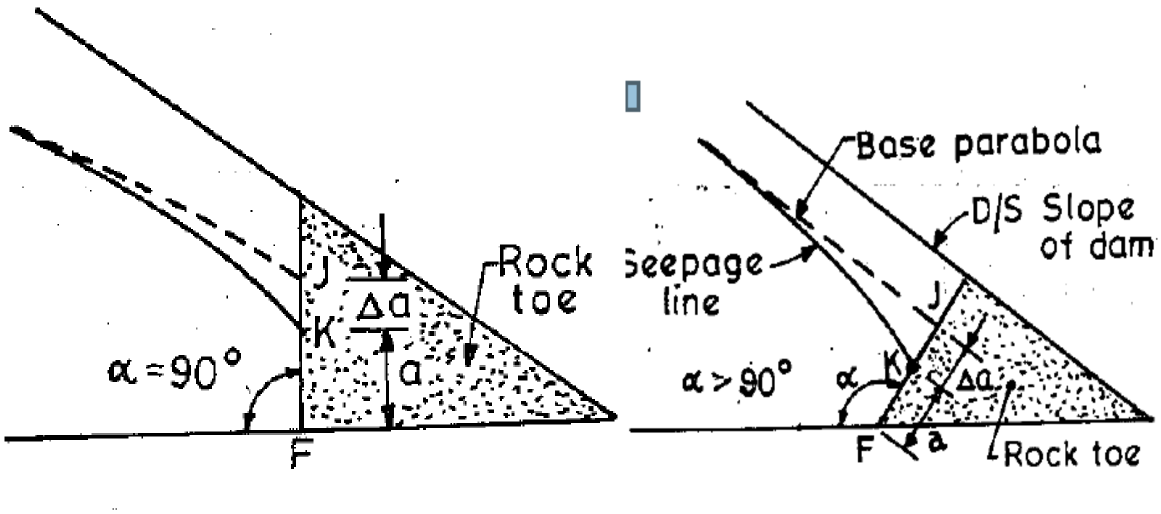


Fig-5 Homogeneous dam with discharge face at $\alpha \geq 90^0$

To summarize the relation between the type of filter drain and the angel between discharge face and horizontal and the analytical solution to correct the phreatic line at exit point.

Table 2 Analytical solution for different type of filter drain

Dam filter type	Discharge face angel, a	Seepage analysis by PL method	Discharge
Dam with Horizontal filter	$a=180^0$	Kozney' analytical method	Q=KS
Dam without filter	$a<30^0$	Schaffernak and Iterson's method	Q=Kasina tana
	$30^0<a<60^0$	Cassagrande's method	$Q=Kasin^2a$
	$a<90^0$	Cassagrande's using table or graph	>>
Dam with rock toe	$a=90^0$	Cassagrande's method	$Q=Kasin^2a$
	$a>90^0$	Cassagrande's method	>>

Zoned dam

If the upstream and downstream shell materials are assumed to be infinitely permeable or a chimney drain is provided, the phreatic line can be computed by the method for a homogenous dam with its focus at F and with no shell material. However, if the permeability of the d/s shell material is not several times the core, the composite section approach is suitable.

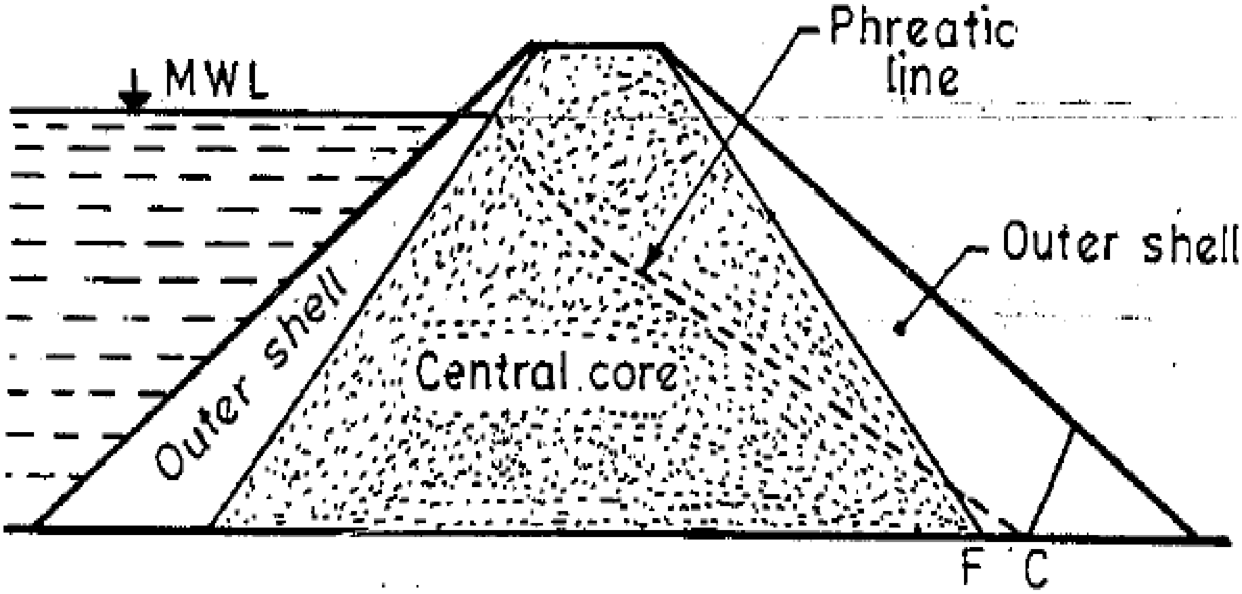


Fig-6 Phreatic line of zoned dam having central impervious core

2.2.3.2 Graphical by Flow net

Flow net is a 2D plan view or x-section diagram of “flow lines” which are lines representing the direction of seepage flow in a porous or permeable medium.

It is drawn by free hand sketching and making suitable adjustment and corrections until to draw the flow line and equipotent line intersect at right angle.

A flow net consists two sets of lines which must be perpendicular to each other;

- Flow lines:-shows direction of seepage water.
- Equipotential lines: - are lines of constant hydraulic head in the porous medium and shows the distribution of potential energy.

According to Casagrande (Casagrande, 1940), the following rules should be obeyed in drawing flow nets:

- Flow lines and equipotential should always be perpendicular to each other, in a homogeneous isotropic system, and form curvilinear "squares".
- Flow lines should always be parallel to an impermeable boundary, and equipotential lines are always perpendicular to it.
- Flow lines should always be perpendicular to a constant head boundary, and equipotential lines are always parallel to it.

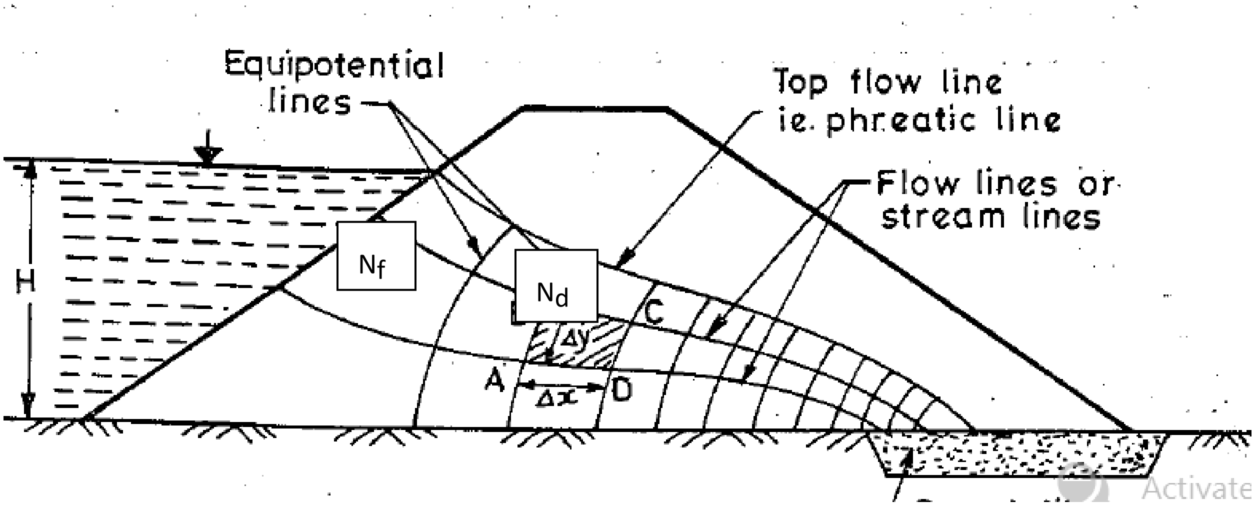


Fig-7 Flow lines and Equipotential lines of homogeneous dam (Flow net)

For Isotropic soil; the hydraulic conductivity is constant in all direction (horizontally as well as vertically, $K_h = K_v = K$).The amount of seepage though the dam body and foundation computed by flow net analysis, using Darcy's law and applying the principle of continuity between each pair flow lines, it is evident the velocity must vary inversely with the spacing and flow through the field or through the channel containing this square. The amount of seepage per unit length

$$q = K\frac{Nf}{Nd}H$$

Where,

q = the discharge per unit width

K = hydraulic conductivity

Nf = number of flow lines

Nd = number of equipotential drops

H = total head (the difference of upstream and downstream water level)

For Non-isotropic soil; the coefficient of permeability is different in the horizontal and vertical direction ($K_h \neq K_v$). The flow net drawn to the same vertical scale but to a transformed horizontal scale. All horizontal dimensions shall be reduced by multiplying them by a factor equal to

$$K = \sqrt{Kh\ Kv}$$

$$q = \sqrt{Kh\ Kv}\,\frac{Nf}{Nd}\,H$$

Where,

K= hydraulic conductivity in the horizontal (K_h) and vertical (K_v) direction.

q = the discharge per unit width

K = hydraulic conductivity

N_f= number of flow lines

N_d= number of equipotential drops

H = total head (the difference of upstream and downstream water level)

2.2.3.3 Numerical using SEEP/W Model

Numerical analysis using computer programs are widely used to model a variety of seepage flow conditions in embankment dams. Numerical analysis of an embankment dam is a process in which the problem is represented as it appears in the actual condition of real world and is interpreted in abstract form.

Geo studio software is one of geotechnical program that used for doing analysis like seepage, static slope, dynamic, stress-strain and also fast water drop in the reservoir. The software include 8-products; SEEP/W for seepage analysis, SLOPE/W for static stability, QUAKE/W for dynamic stability, SIGMA/W for stress-strain deformation, TEMP/W for geo thermal, CTRAN/W, AIR/W for airflow, VADOSE/W for vadose zone and covers.

SEEP/W software calculates the leak/seepage using partial differential equations makes the water flow. The model is constructed to solve 2-Dimensional velocities and gradients of flow. Computations include flow quantities and uplift pressure at user selected locations in the model.

Flow situations with multiple soil layers, flow directions of seepage water can be analysed. Under steady state conditions, the difference between input flux and output flux is zero at all times. Good quality output graphics allows a visual display of equipotential lines and flow paths, and contours can be plotted for different properties like pore pressure, seepage. The procedure for doing analysis

2.3 Stability Analysis

2.3.1 Stability of an Embankment Dam

Slope stability is the resistance of inclined surface to failure by sliding or collapsing. Successful design of slope requires geological information and site characteristics, e.g. properties of soil mass, slope geometry, groundwater conditions, alternation of materials by faulting, joint or discontinuity systems, movements and tension in joints, earthquake activity etc.

The presence of water has a detrimental effect on slope stability. Water pressure acting in the pore spaces, fractures or other discontinuities in the materials that make up the pit slope will reduce the strength of those materials.

2.3.2 Purpose of Stability Analysis

Slope stability analysis is performed to assess the safe design of human-made or natural slopes (e.g. embankments, road cuts, excavations, landfills etc.) and the equilibrium conditions. The main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization.

Satisfactory behavior of an embankment dam under loading conditions not expected exceed during the life of the structure should be considered indicative of satisfactory stability, provided that adverse changes in the physical condition of the embankment do not occur. Significant events that can cause static instability of an existing embankment dam after years of satisfactory service include;-

- An unusually severe drawdown of reservoir. The severity of draw down can be in terms of rapid rate or a lower level than before.
- An usually high and perhaps sustained reservoir water level.
- A prolonged dry period followed by rain. The dry period can cause desiccation cracks to develop in some; cracks with the water and precipitate slides.
- Gradual development of an adverse seepage pattern through the dam and /or its foundation.
- Gradual loss of strength in clay, shale or over consolidated clays due to swelling.

2.3.3 Static Instability Indicator

A need for evaluating the static stability of an existing embankment dam and its foundation is indicated if;-

- There is an appropriate slope stability failure.
- There are longitudinal cracks on the dam crest or slopes.
- There are wet areas on the downstream slope or toe portion.
- There is erosion or sloughing near the downstream toe of the dam resulting in local over steepening of the downstream slope.
- Surface measurement points indicate movements.
- Internal instrumentation indicates movements.

- Internal instrumentation indicates excessive pore pressure in the dam and/or foundation.

- There are bulges in the ground surface beyond the toes of the slopes.

- There is a need for dynamic stability analysis.

- Review of design and construction, records indicate the presence of previously unrecognized but potentially harmful geologic conditions on its foundation may be required to change reservoir operations or for raising the dam.

2.3.4 Stability Analysis Methods

Choice of correct analysis technique depends on both site condition and the potential mode of failure, with careful consideration being given to the varying strengths, weaknesses and limitations inherent in each methodology. Before the computer age, stability analysis was performed graphically or by using a hand held calculator.

Limit equilibrium methods investigate the equilibrium of a soil mass tending to slide down under the influence of gravity. Different movement is considered on an assumed or known potential slip surface below the soil or rock mass. All these methods are based on the comparison of forces, moments, or stresses resisting movement of the mass with those that can cause unstable motion (disturbing forces). The output of the analysis is a factor of safety, defined as the ratio of the shear strength (or, alternatively, an equivalent measure of shear resistance or capacity) to the shear stress (or other equivalent measure) required for equilibrium. If the value of factor of safety is less than 1.0, the slope is unstable.

The limit equilibrium method assumes the following five conditions:

1. “Each point within the soil mass must be in equilibrium.

2. The Mohr-Coulomb failure condition must not be violated at any point.

3. The strains that occur must be related to the stresses through a stress-strain relationship suitable for the soil.

4. The strains that occur at each point must be compatible with the strains at all surrounding points.

5. The stresses within the soil must be in equilibrium with the stresses applied to the soil.”

A wide variety of slope stability software uses the limit equilibrium concept with automatic critical slip surface and FOS determination.

2.3.4.1Ordinary Method

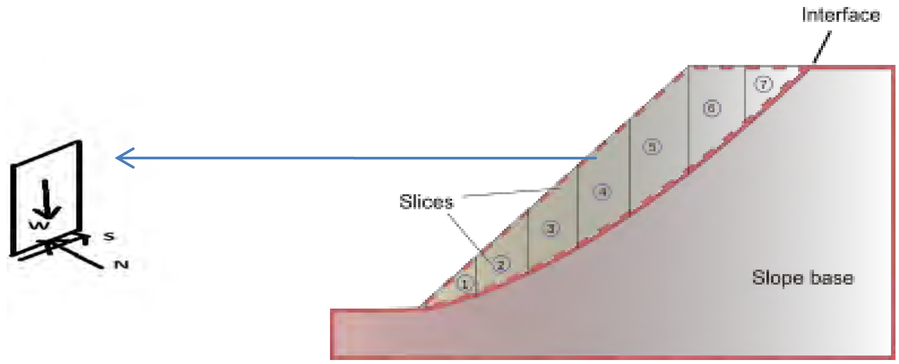


Fig-8 Division of slope masses in the method of slices.

The upstream sliding mass above the failure surface is divided into a number of slices. The forces acting on each slice are obtained by considering the mechanical (force and moment) equilibrium for the slices. Each slice is considered on its own and interactions between slices are neglected because the resultant forces are parallel to the base of each slice. However, Newton's third law is not satisfied by this method because, in general, the resultants on the left and right of a slice do not have the same magnitude and are not collinear.

For the ordinary method of slices, the resultant vertical and horizontal forces are

$$\sum \mathrm{Fv} = o = w - Ncosa - Tsina$$

$$\sum \mathrm{Fh} = 0 = \mathrm{kw} + \mathrm{Nsina} - \mathrm{Tcosa}$$

Where k represents a linear factor that determines the increase in horizontal force with the depth of the slice. Solving for N gives

$$N = W\cos\alpha - kW\sin\alpha\,.$$

In OM, all inter-slice forces are ignored. The slice weight resolved into parallel and perpendicular or normal forces to compute the gravitational driving force and the available shear strength. The factor of safety

$$\text{FOS} = \frac{Shear\ resistance}{Shear\ mobilized} = \frac{\sum[(cB + N\tan\emptyset)]}{\sum W\sin\alpha}$$

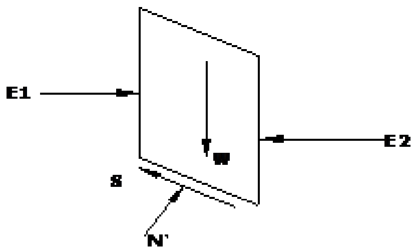
C=cohesion	∅=Friction angle
B=Slice base length	w=Slice weight
N=base normal (wcosα)	α=Slice base inclination

Limitation of OM

- Inaccurate FOS for flat slopes with high pore pressures;
- only for circular slip surfaces;
- Assumes that normal force on the base of each slice is Wcosα ; one equation (moment equilibrium of entire mass), one unknown (factor of safety).Due to poor force polygon(not closed in free body diagram) it gives unrealistic FOS and Consequently should not be used in practice. It used for historic reasons and for teaching purpose.

2.3.4.2Bishop's Method

The Bishop's method is slightly different from the OMS in that normal interaction forces between adjacent slices are assumed to be collinear and the resultant inter-slice shear force is zero. The constraint introduced by the normal forces between slices makes the problem statically indeterminate. As a result, iterative methods have to be used to solve for the factor of safety. The factor of safety for moment equilibrium in Bishop's method can be expressed as



Where,

N’ = normal force

S = shear resistance/shear strength of the soil

E_1,E_2 = inter-slice normal force

$$\text{FOS} = \frac{1}{\sum wsin\alpha}\sum\left[\frac{cB+wtan\emptyset-\frac{cB}{FOS}sin\alpha tan\emptyset}{\text{ma}}\right]$$

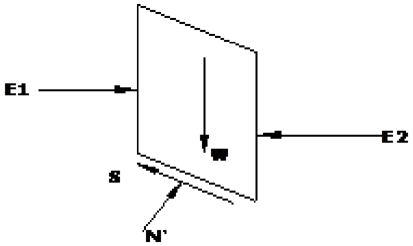
$$m_a = \cos\alpha + \frac{sin\alpha tan\emptyset}{FOS}$$

Initial guess of factor of safety taken from OM then calculate FOS again compute m_a(until the two reached to convergence).

A slices method of slope stability analysis which involves a different procedure and gives different answers compared with the Ordinary Method of Slices has been proposed by Bishop (1955). With this method, the analysis is carried out in terms of stresses instead of forces which were used with the Ordinary Method of Slices. The stresses and forces which act on a typical slice and which are taken into account in the analysis as shown in Fig.

It is accurate method; only for circular slip surfaces; satisfies vertical equilibrium and overall moment equilibrium; assumes side forces on slices are horizontal.

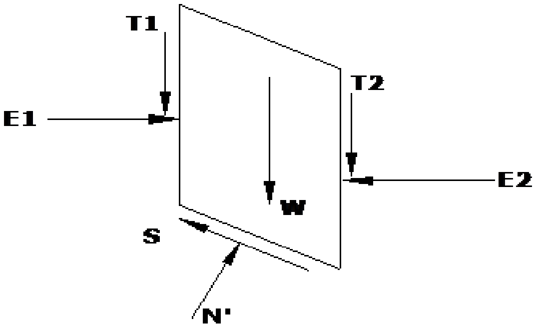
2.3.4.3 Janbu's Simplified Method



It is similar to Bishop's method except it satisfies only overall horizontal force equilibrium, but not overall moment equilibrium. The slice force polygon closure is actually better than Bishop's. The janbu's simplified FOS is actually too low, even though the slices are in force equilibrium. FOS lie on force equilibrium curve, since force equilibrium sensitive to the assumed inter-slice shear, ignoring inter-slice shear, as in the Janbu's method, makes the resulting FOS too low for circular slip surface. Force equilibrium method; applicable to any shape of slip surface; assumes side forces are horizontal (same for all slices); factors of safety are usually considerably lower than calculated using methods that satisfy all conditions of equilibrium.

2.3.4.4 Morgensertn and Price's Method

Morgensertn and Price's method (Morgenstern and Price 1965) Satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes that inclinations of side forces follow prescribed pattern, and allowed for various user specified inter-slice force functions. Side force inclinations(inter-slice forces) are calculated in the process of solution so that all conditions of equilibrium are satisfied; accurate method.



Where,T_1,T_2 = Inter-slice shear force

The inter-slice functions available in the slope/w are

- Constant function
- Trapezoidal function
- Half sine function
- Data-point function
- Clipped sine function

Selecting constant inter-slice function make M-P similar to Spencer methods method. Generally rigorous methods converge worse than the simpler methods (methods that do not include all inter-slice forces and do not satisfy all equations of equilibrium sometimes can error on the unsafe side for convergence problems include too steep sections of slip surface, complex geometry, a significant jump in surcharge etc. If no result is obtained, we recommend slight change of input data, e.g. less steep slip surface input more points into slip surface etc. or using of some of the simpler methods.

2.3.4.5 Spencer's Method

Spencer's Method (Spencer 1967) Satisfies all conditions of equilibrium; applicable to any shape of slip surface; assumes that inclinations of side forces are the same for every slice; side force inclination is calculated in the process of solution so that all conditions of equilibrium are satisfied; accurate method.

Two FOS with respect to force and moment equilibrium(F_f and F_m respectively), constant relationship between inter-slice shear and normal forces, through an iterative until the two FOS be the same or in other word finding shear normal ratio that makes the two FOS equal, mean that both moment and force equilibrium are satisfied. The force polygon of free body diagram is closure and it means very good result.

Spencer's Method of analysis requires a computer program capable of cyclic algorithms, but makes slope stability analysis easier. It is not as accurate as the Modified Bishop's method, but is acceptably accurate in engineering practices

Comparisons

The assumptions made by a number of limit equilibrium methods are listed in the table 3.3 below.

Table-3 Comparison of the different equilibrium methods

Method	Assumption and behavior
Ordinary method of slice	-Normal inter slice forces (horizontal) are neglected. -Inter slice Shear forces (vertical) are neglected. -Not used in practice b/c of unrealistic FOS.
Bishop's simplified	- There are normal inter slice forces(horizontal) -There are no inter-slice shear forces. -Satisfy overall vertical force and moment equilibrium -Not satisfy overall horizontal force equilibrium
Janbu's simplified	-Resultant inter-slice forces are horizontal. -Satisfy all force equilibrium -Not satisfy moment equilibrium
Morgenstern-Price method	-Consider both shear and normal inter-slice force. -Satisfy both moment and force equilibrium -The direction of the resultant inter-slice forces is defined using an arbitrary function. -The fractions of the function value needed for force and moment balance is computed. -Allows for a variety of user selected inter-slice force function (eg. constant, half-sine, trapezoidal) -Selecting constant function make M-P method similar to Spencer method.

Spencer method	- Consider both shear and normal inter-slice force. -Satisfy both moment and force equilibrium -Assume a constant inter-slice force function (ratio of shear to normal is a constant between all slices). -The resultant inter-slice forces have constant slope throughout the sliding mass.

Table-4 Static equilibrium satisfied by the different methods

Method	**Force balance (vertical)**	**Force balance (horizontal)**	**Individual slice moment**	**Overall Moment**
Ordinary MS	YES	NO	NO	YES
Bishop's simplified	YES	NO	NO	YES
Janbu's simplified	YES	YES	YES	NO
Morgenstern-Price	YES	YES	YES	YES
Spencer	YES	YES	YES	YES

.

2.3.4.6 Fellenius-Jumikis Method

Determination of the minimum factor of safety for a slope is very crucial for the design of Earth fill dam, it is important to locate the most critical slip circle with as few trials as possible. In a random trial and error approach, the three geometric parameters, namely, the center of rotation, the radius of slip circle and the distance of intercept in front of the toe are varied and the minimum factor of safety obtained. This requires a very large number of trials, but computers have made the method feasible. However, it is known that there is a certain pattern in slip circle behavior and knowledge of this pattern can be used to advantage and the number of trials reduced.

Fellenius (1936) proposed an empirical procedure to find the center of the most critical circle in a $\varphi u = 0$ soil. The center O for the toe failure case can be located at the intersection of the two lines drawn from the ends A and B of the slope at angles α_1 and α_2 (Figure 3.9). The angles α_1 and vary α_2 Varies with downstream slope. Table 3.5 gives these values.Before running any slope stability analysis computer program (SLOPE/W), a scale diagram of Gomit dam was drawn using AutoCAD2016 and Felleniu's- Jumikis method was used in order to obtain a very approximate indication of the location of the most critical slip circle center in the Gomit zoned dam. In order to find out the worst case, numerous slip circles should be assumed and factor of safety calculated for each circle. In order to reduce the number of trials, Fellenius has suggested a method of drawing a line PQ representing the locus of the critical slip circle.

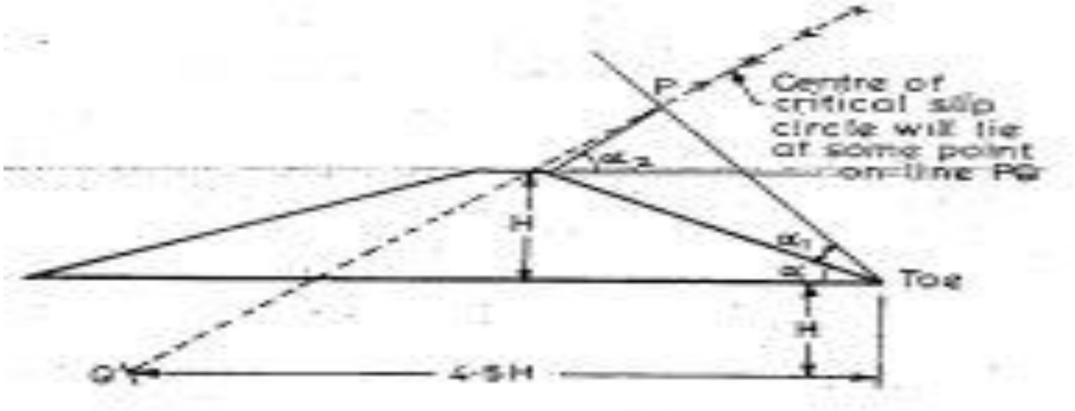


Fig-9 Locus of critical circle for downstream slope

Table-5 Fellenius’s criteria for locating the most critical slip surface

Downstream slope angel(o)	Slope ratio(H:V)	Directional angles	
		∂_1(bottom angel)	∂_2(top angel)
60	0.58:1	29	40
45	1:1	27.5	37
33.8	2:1	25	35
26.6	3:1	25	35
18.4	4:1	25	35
11.3	5:1	25	35

The above guidelines can only serve as pointers and in any stability analysis a sufficiently large number of trials have to be made to locate the correct critical circle. It is found that the factor of safety is more sensitive to lateral shifting of the centre than to vertical movements. Moreover, the guidelines according to Fellenius - Jumikis method, can only serve as a general and approximate way for the determination of the critical slip circle center.

2.3.5 Shear Strength of Soil

Introduction

Shear strength of soil is the resistance of soil mass against shear forces. It is essential for meaningful analysis of slope stability. There is a strong correlation between incidence of embankment slope failures and the use of fine grained/highly plastic soil in embankments. Excess pore pressures often develop during rapid construction of fine-grained fill zones, resulting in reduced shear strength and potentially unstable conditions during or shortly following construction. Early studies indicated that the correlation between fineness of soil and the susceptibility to sliding was strong enough to outweigh the influence of all other factors, including steepness of slopes, construction methods, and reservoir activity. For reasons of safety and economy, a zoned embankment consisting of a central or sloping impervious core flanked by zones of higher-strength pervious materials, should always be constructed in areas where there is a variety of soils available.

Determination of shear strengths

Shear strengths of fill materials should be based on tests performed on laboratory compacted specimens. Its values used in the stability analyses for each condition of drainage are determined from laboratory tests on specimens of the material which are compacted to the density and water content that simulates the conditions anticipated in the dam. Tests corresponding to the three conditions of drainage are:

- Q or Unconsolidated-Undrained (UU) test in which no initial consolidation is allowed under the confining pressure and the water content is kept constant during shear.

- R or *R'* Consolidated-Undrained (CU) tests in which consolidation is allowed under initial stress conditions but in which the water content is kept constant during application of shearing stresses. The *R'* test is identical to the R test except that pore water pressure measurements are made on it.

- S or Consolidated-Drained (CD) test in which consolidation is permitted under the initial stress conditions and also for each increment of loading during shear.

For undrained conditions, the water is not able to flow in or out of the soil rapidly enough in response to the loading conditions, resulting in an increase or decrease of the pore water pressures. Depending on the loading conditions and the permeability of the fill material within the embankment, an engineer could be considering drained or undrained conditions, or both (in the case of a free-draining shell material and impervious core material), in the analysis of the stability of an embankment dam.

For drained conditions, water is able to flow in or out of the soil rapidly in response to the loading conditions that the soil reaches a state of equilibrium and there is no increase of pore water pressures within the soil. Long-term conditions are analyzed using drained strengths expressed in terms of effective stress parameters (c' and ϕ’).

Total and Effective Stresses

“Total stress (*σ*) is the sum of all forces, including those transmitted through inter-particle contacts and those transmitted through water pressures, divided by the total area. Total area includes both the area of voids and the area of solid.” The total stress analysis is used in the design of embankments for loading conditions during construction, rapid drawdown, and earthquake. In total stress analysis, the shear strength of soil includes the effect of pore pressure. It should be used only in cases where the soils behave drained and piezometer data are available. The cases that can be analyzed by the effective stress method are partial pool and steady seepage.

“Effective stress (σ’) includes only the forces that are transmitted through particle contacts. In effective stress analysis, the shear strength of soil is evaluated on the effective normal stress basis and explicit account is taken of the pore pressure in the stability calculation. It is equal to the total stress minus the water pressure (*u*).”

The shear strength of a soil is a function of the cohesion of the soil (*c*), the internal angle of friction of the soil (*Φ*), and the normal stress (*σ*). The shear stress at failure (S) is expressed by the Mohr-Coulomb failure law as:

Su = $c + \sigma \tan \Phi$ for a total stress analysis

Sd = $c' + \sigma' \tan \Phi'$ for a effective stress analysis

Where c and c' are the cohesion intercepts and Φ and Φ' are the friction angles for the total and effective stress shear strength envelopes, respectively. The constant c and values depend upon the type of soil and moisture content in it. Sands and gravels are cohesion less soils and approximately equal to zero; while Φ is approximately zero for saturated cohesive impervious clays. The figure below shows the shear strength envelopes that are developed from Mohr circles for total and effective stresses.

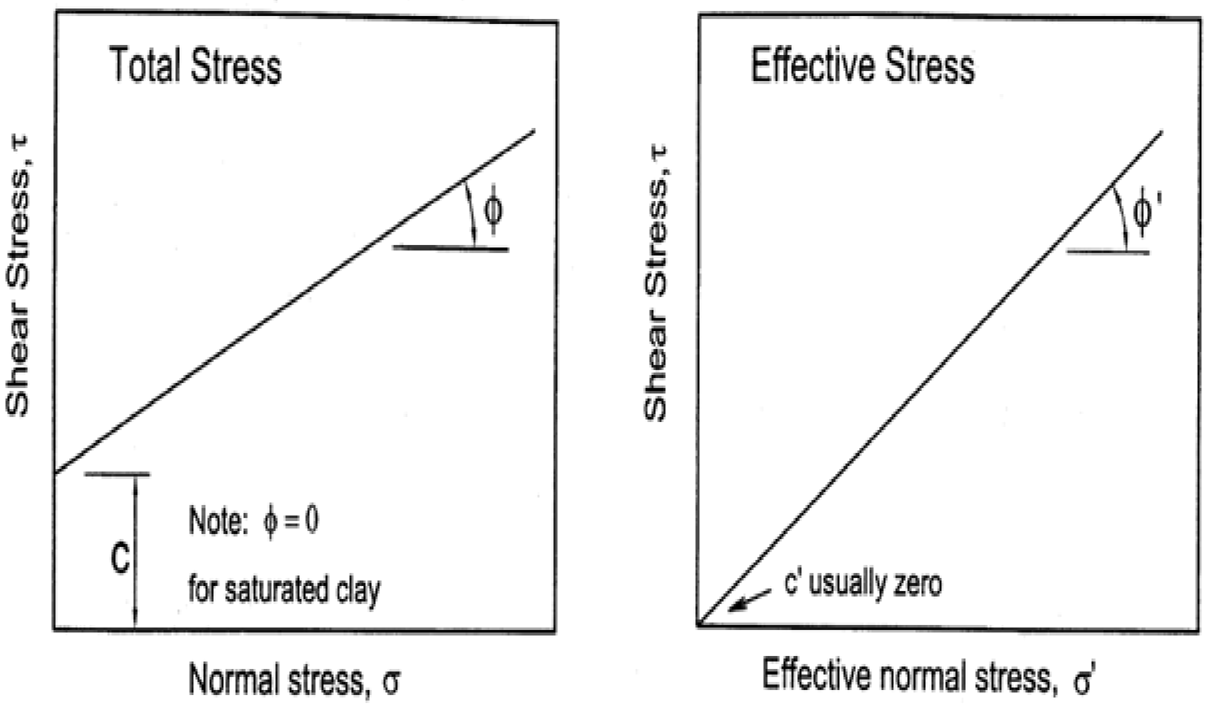


Fig-10 Linear (Mohr-Coulomb) Strength envelope

2.3.6 Standard Loading Condition

An embankment and its foundation are subject to shear stresses imposed by the weight of the embankment and by pool fluctuations, seepage, or earthquake forces. Loading conditions vary from the commencement of construction of the embankment until the time when the embankment has been completed and has a full reservoir pool behind it. The standard loading conditions encompasses the following conditions at various stages from end of construction through the operational stage of the completed embankment.

2.3.6.1 End of Construction Loading Condition

At the end of construction, an embankment dam is still undergoing internal consolidation under its own weight. For homogeneous dams or for zones in dams constructed from impervious materials, pore water pressures will be built up during construction due to the inability of the impervious soil mass to drain rapidly during consolidation. The type of stress analysis that applies to this loading condition is the total stress analysis. Because of the difficulty in estimating pore water pressures within the embankment during this stage of loading, an effective stress analysis is not generally used. The analysis may, however, be conducted using pore pressure responses in previously constructed dams that used materials, construction methods, and construction schedules similar to those for the proposed dam. For pervious zones in the embankment where drainage can occur rapidly, S strengths should be used in the analysis. The end of construction analysis using shear strengths obtained from the U-U test as representative of the strength available in the impervious zones of an embankment represents a lower limit of stability since consolidation is progressing during the course of construction. If there are any serious questions about stability during construction, the only positive method to determine the stability is to install piezometers and evaluate the stability during construction.

2.3.6.2 Sudden or Rapid Drawdown

In the sudden drawdown loading condition, the structure has been subjected to a prolonged high pool during which time a steady seepage condition has been established through the embankment. The soil in the embankment below the phreatic surface is in a completely saturated state and is fully consolidated under the weight of the overlying material. If subsequently the reservoir pool is drawn down faster than pore water can escape, so excess pore water pressures develop. Consequently, the reduced factor of safety following a reservoir drawdown is due primarily to the existence of high residual pore water pressures (drawdown pore water pressures) acting inside the upstream slope. The unit weights of the soils to be used in analyzing the "before drawdown" condition will be the moist weights above the line of saturation and submerged weights below. In analyzing the "after drawdown" condition, moist unit weights will be used for the zone above the original phreatic surface, saturated unit weights will be used within the drawdown zone, and submerged weights will be used below the level of drawdown.

2.3.6.3 Steady Seepage Loading Condition

Steady state seepage develops after a reservoir pool has been maintained at a particular elevation for a sufficient length of time to establish a steady line of saturation through the embankment. The seepage forces which develop in the steady state condition act in a downstream direction, and therefore the condition of steady seepage through an embankment may be critical for downstream slope stability. The upstream slope need not be analyzed for this condition if the upstream slope was designed for rapid drawdown. The seepage forces can be conservatively estimated by assuming a horizontal phreatic line from the elevation of the pool to the downstream limit of the impervious zone. However, high abutment groundwater tables may cause the phreatic surface to be higher in the vicinity of the abutments. It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage (or steady seepage) conditions with the reservoir at its normal operating pool elevation (usually the spillway crest elevation) since this is the loading condition the embankment will experience most. This condition is also called steady-state seepage under active conservation pool (USBR). Unit weights to be used in the analysis will be the moist unit weight above the line of saturation and submerged weights below this line.

Factor of safety

Factor of safety is the ratio of shear resistance to shear failure. The factor of safety includes a margin of safety to guard against ultimate failure, to avoid unacceptable deformation, and to cover uncertainties associated with the measurement of soil properties or the analysis used. The computed value of Factor of safety greater than the minimum safety acceptance mean the slope of the dam stable against mobilized. In selecting a minimum acceptable factor of safety an evaluation should be made on both the degree of conservative with which assumptions were made in choosing soil strength parameter, pore water pressure and method of analysis. The table below shows the minimum required safety factor for the standard loading conditions.

Table-6 The minimum acceptable FOS of earthen dam after Jansen R. et al. 1988

Standard loading condition	Minimum Safety factor	
	Downstream slope	Upstream slope
1. under construction; end of construction With earthquake loading in addition (pseudo static)	1.25 1.0 (1.1)	1.25 1.0 (1.1)
2. Steady seepage long-term operational at partial pull, Upstream slope With earthquake loading in addition (pseudo static)	1.5 1.25	1.5 1.25
3. Rapid drawdown With earthquake loading in addition (pseudo static)	- 1.25 (1.2)	- 1.25 (1.2)
4. Seismic loading with 1,2, or 3 above	1.1	1.1

Table-7 The minimum acceptable safety factor provided by U.S. Army of Corps

Slope of the dam	End of construction before impoundment	Operation condition
Upstream slope	1.3	1.5
Downstream slope	1.3	1.5

2.4 Seepage Controlling Measure

Measures to control seepage through the dam body and foundation must consider the type of dam, foundation, and abutments as a unit. They may form a complicated structure through which seepage occurs at the embankment and its foundation. This can make precise detection and remedial control difficult. Measure action may range from continued or additional monitoring to rebuilding or abandonment of the dam. Most of the remedies can be adapted to embankment dams are listed below.

• Monitoring

In all cases, monitoring seepage is essential, and in some instances may be the only action necessary. Monitoring seepage and seepage control measures can lead to a rational conclusion with a minimum of expenditure. The most common and easiest monitoring is simply to rely on visual observations and inspections at various intervals and reservoir elevations. Periodic photographing and videotaping of potential distressed areas can provide valuable documentation. If the latter monitoring methods are used, it is vital that the camera always be in the same position to permit easy interpretation. If not already accomplished during construction, a common recommendation may be to install instrumentation such as piezometers, observation wells, and seepage collection systems to determine more definite patterns of seepage behavior. Review the data on a regular basis to detect any major seepage changes and long-range trends. If monitoring indicates that a potentially dangerous seepage problem may exist, consider permanent structural or regulatory measures (i.e. permanent reservoir level restriction). If monitoring is selected as a remedial measure, it may also be desirable to consider automated instruments with predetermined criteria or values to flag undesirable behavior.

• Lowering the Reservoir

The most direct method to reduce or stop seepage is to lower the reservoir and restrict the reservoir level. Lowering and restriction of the reservoir level may not, however, be an acceptable permanent solution. Flood inflows may cause the reservoir to rise above restricted levels, and the benefits of the project will probably be greatly diminished or lost altogether. If this alternative is selected, care should be taken to lower the reservoir at such a rate so as to prevent possible flooding downstream and also to reduce the risk of an upstream slope failure from rapid drawdown.

• Grouting

Grouting is often attempted as a means of controlling seepage through soluble rocks. However, this method frequently is not successful or is only temporarily successful because the solution passages usually are partially filled with residual clay or other material that erodes when subjected to changed seepage forces. If one passage is plugged with grout, the seepage often finds another passage around the plug. Therefore, more aggressive methods may be required to ensure permanent and reliable seepage control in soluble rock. These seepage control methods require careful analysis on a case-by-case basis. In current practice, more use is being made of positive remedies such as concrete cutoff walls at the dam centerline. Pressure grouting using a mixture of cement and water or other materials is probably the most frequently used method to remedy serious foundation or abutment seepage problems in rock. However, it should not be used indiscriminately. Before any grouting effort, conduct an investigation to determine the seepage conditions and locations.

• Cutoff Walls

Cutoff walls are uniquely suited to remedy seepage through an existing embankment. The cutoff can be extended to remedy foundation and abutment seepage problems as well. However, cutoff walls are extremely costly and thus are usually considered only as a last resort. Use caution when designing a cutoff wall for an embankment. The wall should be located as far upstream as possible because uplift pressures will increase upstream of the wall. If a slurry trench is chosen for a particular cutoff design, consider these major factors:

- The effect on the stability of the embankment due to excavation of a long slurry-filled trench, and the introduction of a plane of weakness through the dam (in the case of soil-bentonite backfill). A cement-bentonite or concrete backfill may be placed in panels in separately excavated elements rather than in an open trench. Relatively weak soil bentonite backfill may be unacceptable.
- The possibility of cracking of the wall if the completed wall is not sufficiently plastic.
- The ability to tie the slurry trench to other existing or proposed seepage control measures. If a competent upstream blanket exists, the trench may be tied to the blanket or it may be placed through the dam and tied to the impervious core.

• Upstream Impervious Blankets

An impervious blanket immediately upstream of a dam can be used to seal the reservoir bottom and sides and thereby reduce seepage quantities and pressures beneath a dam. If the dam contains a permeable shell, the impervious blanket must be extended up the embankment slope to effectively control the seepage problem.

If it is determined that sealing of the reservoir bottom and sides immediately upstream of the embankment will be useful in reducing undesirable seepage quantities and pressures beneath the embankment, an upstream impervious blanket may be employed. If successful and economically feasible, this is one of the most efficient measures since the source of water is controlled upstream of the embankment and its foundation. Fine grained materials placed on the upstream embankment slope may be removed during drawdown because of low saturated strength and high saturated weight. If seepage can also go through the upstream portion of the embankment and then into the foundation an upstream blanket will be less effective and another remedy may be necessary..

• Downstream Berm

A downstream berm can be used as a remedial treatment against seepage forces and uplift pressures on the downstream face of the dam. A berm may prevent blowout by increasing the overlying weight sufficiently to resist the uplift pressures. If the berm is of low permeability, the seepage will be forced to exit further downstream. The design of a pervious berm should ensure there is no upward migration of fine particles from the foundation. This implies filter compatibility between the berm and the foundation. An added advantage of a downstream berm is to increase slope stability because of the additional resistance to sliding provided by the berm.

In some cases, it may be necessary to add on to the downstream side of the dam using impervious elements, filter and drainage chimneys and blankets, and an outer stability shell.

• Drainage

Drainage can also be used as a treatment to control seepage. Generally, the work can be performed from the downstream side of the dam. Seepage emerging on or at the toe of the downstream slope will normally be controlled by one of the methods previously mentioned; Expedient installation of filter materials and a toe drain can help prevent piping of embankment and foundation materials and may increase embankment stability, but will not normally reduce seepage quantities. Horizontal drains of slotted pipe normally do not have a filter envelope and would generally be used for seepage or as an expedient measure until a more permanent solution could be installed [www. U.S Army Crops of Engineers, 1993].

3. MATERIALS AND METHODOLOGY

The materials used for embankment dam failure analysis focusing on seepage and stability analysis are Geo studio 2007 software products of Seep/W and Slope/W,Ms excel, AutoCAD2012, Digital camera, Global positioning Satellite (GPS), Flow current meter and Tape meter. In addition to that, the analysis of the problem is supported with data from literature review to model out the problem using Slope/w.

3.1 Description of the Study Area

3.1.1 Location and Topography

Gomit dam is found in Amhara region, South Gondar Administration Zone, Estie woreda about 9km far away from the capital of the woreda in South direction. The geographical location of this site lies on the coordinates of $11^0$33'43" North latitude and $38^0$46'20" East longitude with an average altitude of 2375 above mean sea level. Geographically the woreda is found in the North western highlands of Ethiopia and it is bounded by Blue Nile (Abay) River in the South part and the area is free from any tectonic and seismic activity or risk. The dam is situated between valley gorges having wider reservoir area and agricultural land. The total catchment area is 23.43km^2.the dam was constructed on the river of Gomit to irrigate 90ha of agricultural land on the left side of the river by impounding the flood for dry season irrigation.

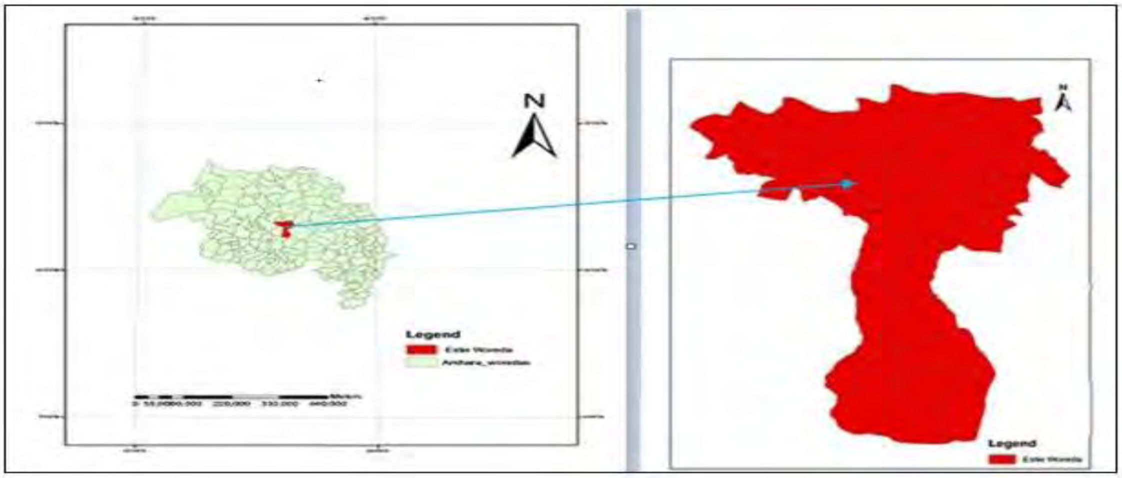


Fig-11 Topographic map of Amhara Region, Estie Woreda

3.1.2 Climate

The project area is classified as Dega agro-climatic zone with an average altitude of 2343 m above mean sea level with two rainfall seasons of “kiremt” and non-promising “belg”. Though it has two rainy seasons, the rainfall nature is uncertainly distributed and erratic as it is intercepted by Gunna mountain.

Climatic data taken from two metrological stations, the temperature and rainfall data from Mekaneselam station (the capital city Estie Woreda) and a very small scaled isohytal map of the area from Debretabour station. The rainfall pattern of the two stations is almost similar and hence the Debretabour city with altitude of 2690m a.s.l meteorological data is taken for the analysis. The mean annual rainfall varies from 1200 to 1600mm, the mean monthly maximum and minimum temperatures are 23.98^0c and 8.85^0c respectively. The relative humidity varies from 57% in April to 89% in July.

3.1.3 Geology

Most part of Estie Woreda and vicinity of the area are covered by volcanic rocks of aphanitic and trachy basalt, tuff, and consolidated red ash rock formations.

As the other parts of the Woreda, the areas around Gomit micro earth dam project and its catchment are dominantly covered by volcanic rocks of aphanitic and highly vesiculated trachy basalt, tuff and consolidated red ash. Especially all parts of the dam axis and most parts of the reservoir area (both right and left parts) are covered by highly vesiculated and weathered tranchy basalt rock and this rock is continuous both sides (upstream and downstream) the project site and around the spill way site. While the ridge lands which are found on the left reservoir and upstream ends are covered by highly weathered basalt rock and there is local dyke, which surrounds the pick land.

3.2 Logical Framework and Methods

In order to achieve the objective of the research, a logical frame work/flow chart prepared for indicating the steps/ procedure and the methods used in the analysis of seepage and slope stability as shown below.

The analysis of seepage was done using analytical and numerical methods. Seepage through the dam body and foundation evaluated for both homogeneous and zoned dam by phreatic line and Geo studio software of Seep/W. The stability of upstream and downstream slope of the dam was performed using Geo studio software Slope/W. For the current performance of the dam, seepage and slope stability analysis was done using the above methods at current water level.

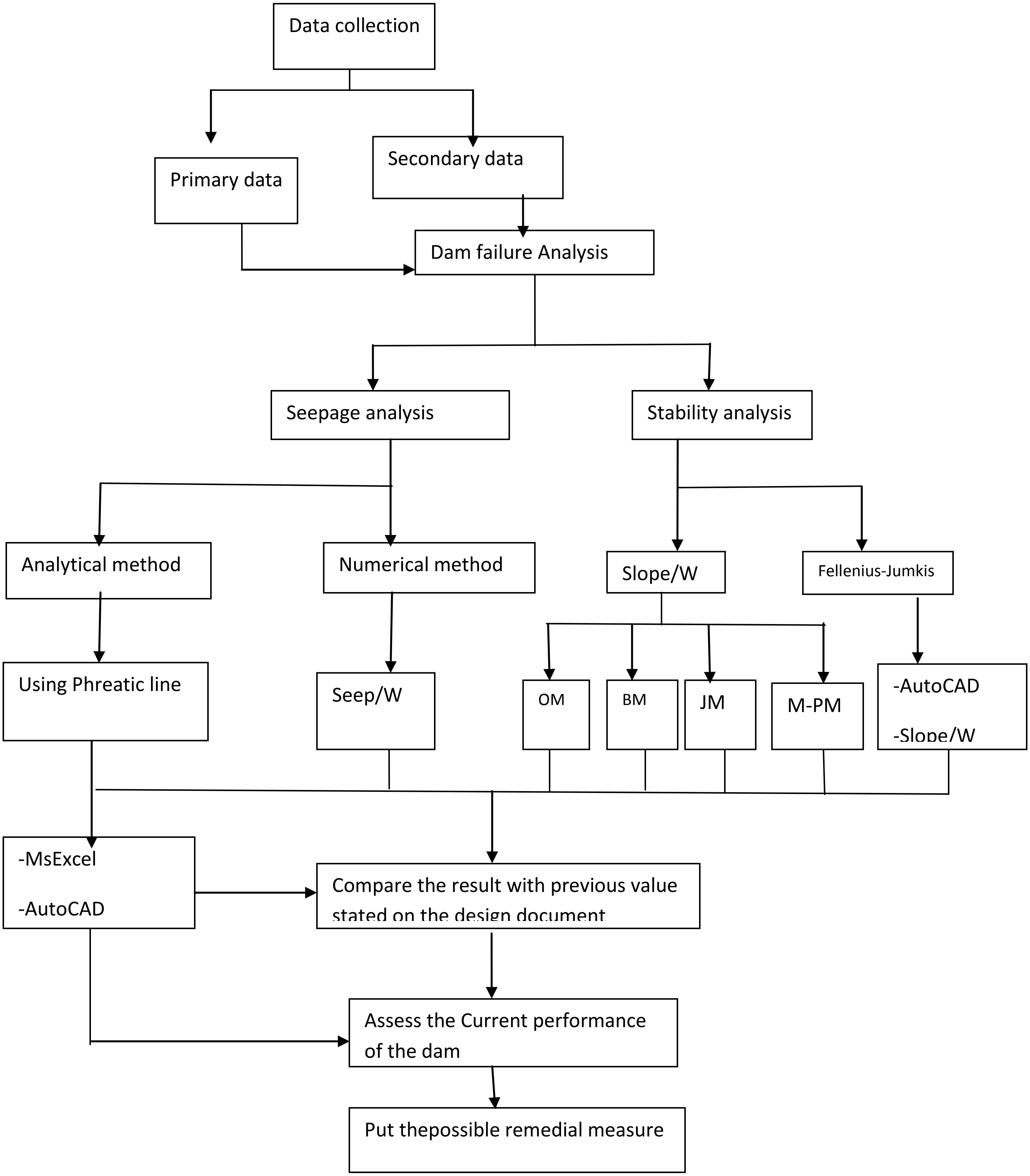


Fig-12 logical frame work

3.3 Data Collection Method

3.3.1 Primary Data Collection

- Dam site visiting and visual inspection about the current condition of the dam.
- Interviewing the beneficiaries and operator about the past condition of the dam.
- Identify the main problem of the dam supporting with pictures using digital camera.

3.3.2 Secondary Data

- The available data was collected from the past reports and files kept by the Amhara Region Water Resource Development Bureau and other responsible organization (relevant NGOs) for further interpretation and analysis.
- Manuals, guidelines and standards for the design and analysis of embankment dams
- The secondary data collected from the office are;-
  - Topographic map of the dam site
  - Design documents that contain the main hydrological, structural & soil data.

In order to achieve the objective of this study, the main data taken from the design document are the dam profile and property of construction materials and foundation.

- Dam profile; - the top and bottom width, the height of dam, the u/s and d/s slope, the normal, maximum and minimum water level of the dam, the length horizontal filter, depth of cut-off.
- Property of material; - the laboratory test result of the core, shell material and foundation (dry, saturated and submersed unit weight, cohesion of soil, angle of internal friction, permeability/ hydraulic conductivity of the soil).

Tests were made on different construction and foundation materials. The test results are analyzed and the stability analysis was performed using these foundation and construction material physical and shear parameters. From the design document the test results summarized as follows.

Table-8 Property of dam material and foundation

Materials	Class	g dry (g/cc)	Permeability coefficient (cm/sec)	C Kpa	Ø $(^{o})$	ρ bulk (g/cc)	ρ sub (g/cc)	ρ sat (g/cc)
Shell	GW	1.43	$1.53x10^{-6}$	3.8	39^{o}	1.86	0.87	1.87
Core	CH	1.34	$2.3x10^{-8}$	14	26^{o}	1.76	0.82	1.82
Foundation	CH	-	$3.42x10^{-4}$	9	24^{o}	1.53	0.66	1.66

3.3.2.1 General Description of the dam and Finding out the problems

Gomit dam is a zoned type earth fill dam composed of a central impervious core flanked by zones of materials considerably more pervious called shells. The dam of 20m height and 324m crest length with upstream and downstream side slope of 2:1 and 2.5:1 respectively. A riprap material of poorly graded was provided to protect the erosive action of waves on the upstream and protection grass on the downstream. The volume of water stored at normal pool level (elevation of NPL=2667m) is 10.61Mm^3. The total embankment volume is 93236.76 m^3 and the total reservoir/catchment area was 73.964 ha. Spillway was provided on the right side of the. It is an ogee type spillway designed to convey safely the maximum designed flood of 87.84m3/sec with over the crest length of 25m. Outlet work provided which made up of steel pipe with 0.6m diameter. Trash racks, collard stilling basin and valves are also provided.

The main problems were identified through the dam site visiting and by interviewing the operators and beneficiaries and shown in photo. The main problems are;-

- Seepage through the dam body, foundation, right abutment and left side of spillway foot. This result in high reduction of water storage on the upstream and endanger for the coming dry month.

- Face (slope) failure on the downstream slope of the dam. This was happened before 3years.

So the analysis will be done to identify the cause, to determine whether or not an observed or perceived problem in serious and represents an unacceptable risk and, to develop effective remedial/action at reasonable cost.

21/7/08

Photo-1 Seepage water at downstream around the outlet

Photo-2 Seepage water from the foot of spillway side wall entering to outlet

Photo-3 Slope face failure at the downstream

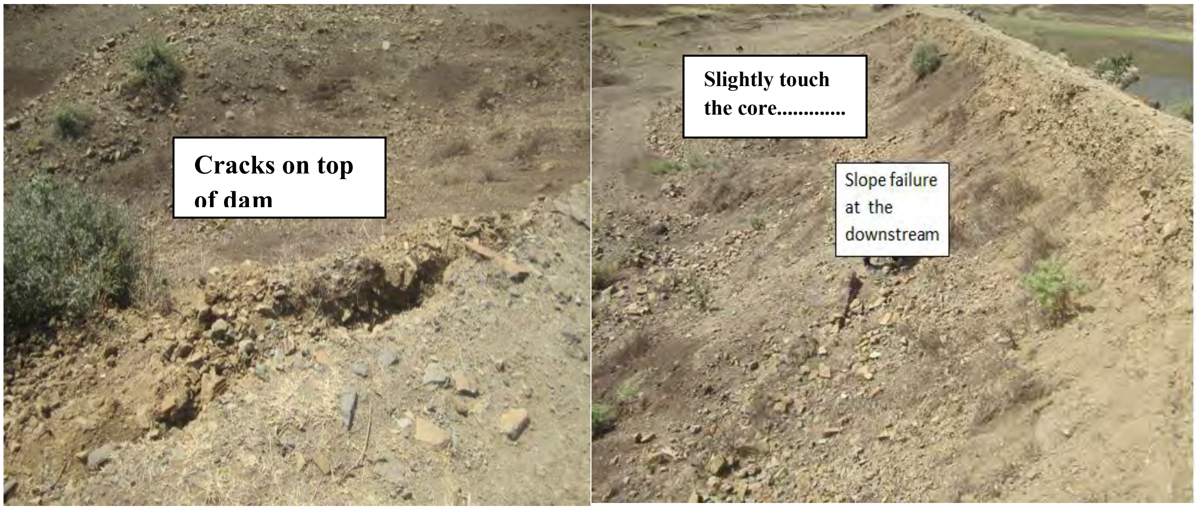


Photo-4 Slope failure and top width crack at current condition

3.4 Data analysis and Procedures

Analysis of the Gomit embankment dam failure was done using analytical by Phreatic line and numerically by Geo studio software(SEEP/W and SLOPE/W) for both seepage and stability analysis .The choice to use the different analysis method should be based on complexity of the conditions to be analyzed and the objective of the analysis. Computer programs are rapid methods and provide a means for detailed analysis of seepage and stability.

The study is based on visual observation and available data present at the design document focusing on the evaluating the previous design results, current condition or the performance of the dam and monitoring instruments.

3.4.1 Analysis of Seepage

3.4.1.1 Analysis by phreatic line

Seepage line is used to estimate the amount of seepage passing through the body of the dam with a horizontal filter. The first step is determination of phreatic or seepage line of the dam. In the design document the amount of seepage estimated for the Gomit dam with horizontal filter by considering the dam as a homogenous. But actually the dam was a zoned dam as mentioned earlier. Now the analysis is carried for both homogenous and zoned dam to check and compare the amount of seepage passing through the dam body. The available data obtained from design document for estimating seepage are geometry of the dam and permeability coefficient of core and shell material

Homogenous Dam

Since the Gomit dam has a horizontal filter drain, the analysis was done homogenous dam with horizontal filter. The geometry of the dam points plotted on Autocad-2016. The phreatic line determined using parabolic equation. The horizontal filter drain makes the phreatic line with in the body of the dam as shown on the fig below.

Zoned dam

In the analysis of embankment dam with central core material/zoned dam, if the upstream and downstream shell materials are infinitely permeable or chimney drain is provided, the phreatic line computed by the method for homogenous dam with its focus at F and with no shell material. However, if the permeability of shell material is not several times of core material, the composite section approach is used for the analysis. So, first the impact of permeability of shell material with the core material should be checked and if the ratio of permeability of shell material with core material is greater than 20, the effect of shell material on the core will be neglected.

Kshell=1.53x10^{-6}cm/sec

Kcore=2.3x10^{-8}cm/sec

Ratio(Ks/Kc) = 66.5 >>20

From the above result, the effect of shell material on the core is neglected and analysis made by considering the core material as a homogenous dam without a horizontal filter material and chimney drain. The core material is highly plastic clay with permeability of 2.3*10^{-8}cm/sec.

3.4.1.2 Numerical by seep/w software

SEEP/W is a useful tool that uses numerical modeling to solve complex groundwater seepage problems. This part of the software is to investigate the leak and flow of water in the soil. The procedure that use for analyzing the problem are listed below.

(1) Analysis Types

There are two fundamental seepage analysis type, the steady state and transient type. The analysis type of this research is steady state, which is the total seepage passing through the dam body and foundation do not change with the change in time. This type of analysis does not consider how long it takes to achieve a steady condition. The model will reach a solved set of pressure and flow condition for the given set of unique boundary conditions applied to it and that is the extent of the analysis. In the study the analysis type carried by steady state for homogenous and zoned dam with foundation.

(2) Importing the geometry of the dam and draw region

The geometry of the dam is the profile of the dam that is, the impervious core and shell material, the dam height, the upstream and downstream slope of the dam, the top and bottom width, the maximum, normal and minimum water level, and horizontal filter length and an inclined chimney drain.

Three methods used for importing the geometry of the dam. For simple dam profile direct sketch of the dam , for complex dam profile copy the coordinate points of the dam from excel sheet and paste on key point or by importing the dam profile after saved in AutoCAD dxf file. The second method will be selected for importing the geometry.

- Regions (closed polygon) are created by connecting points and are used to define areas of different material properties and conditions. It is important to note that regions cannot overlap one another and must maintain continuity (no gaps).

(3) Assign/Input materials

The next step in developing a numerical model of SEEP/W is to assign the type and property of materials in to the regions using criterion. Step-by-step instructions to assign materials are provided below. Utilize the KEYIN function and click on MATERIALS. Add a new material, provide it a name (core, shell and foundation) and assign it a color.

Use the material model drop down menu to select one of the options:

A. Saturated Only – Use if a steady state analysis is conducted on a domain that will remain saturated for the entire duration of the simulation. This used for foundation.
B. Saturated/Unsaturated – Use if unsaturated zones are expected to occur. Use for core and shell material.

(4) Setting Boundary Condition

When specifying seepage boundary condition, there are two fundamental options- specifying flux (Q) or head (H).Specifying flux is the Neumann boundary and specifying constant head for stored water at lakes, river, or reservoir is the Dirichilete boundary condition. In this research, the normal and current water level (head) set as a boundary and a special condition that makes the pressure zero is the actual boundary condition used at the downstream toe side. In the draw boundary condition menu of SEEP/W, if the option pressure head, P is selected and action 0 is set for the downstream face slope, SEEP/W find the z coordinate of each node and sets the total head equal to the z coordinate. To assign boundary conditions, setting up the boundary conditions in the model is an essential component as the solution is dependent upon the type of boundary conditions defined in the model. The fundamental types of boundary conditions are described below.

- Potential Seepage Face – A boundary condition where both the head and total flux are unknown along a downstream slope. This allows the solver to locate the position where a seepage face may develop.
- Head – A boundary condition used where there is free water present within the domain such as a reservoir behind a dam (on upstream slope), or a river on the flood side of a dike.
- Zero Pressure – A boundary condition that used to model a drains and areas where pore-water pressure dissipates near instantly such as at horizontal filter drain.

(5) Locate the flux section

SEEP/W has the ability to compute the instantaneous seepage volume rate that flows across a user defined section for a steady state or transient analysis. This is a very useful tool for isolating flow volumes to specific regions of interest and it can save you manually adding up individual nodal flows in the case of a drain or seepage face that is comprised of many nodes. The objective is to compute the total flow across a vertical section of the element. Flux sections can be used in many ways, because they can be drawn any place across which you want to know the flux.

(6) Verify and optimize the data

Each analysis and input data are verify automatically before solving, and any errors are reported on the verification tools. Verify tool checks for errors in the overlapping geometry lines, input data and region that do not have any data points.

(7) Analysis Result

The final step is solving the problem and analyzing the results and make conclusions. Seepage analyses are often conducted for three major applications: calculating flow rates, gathering hydraulic gradient data for determining factors of safety against piping and to be used as a parent analysis for a slope stability analysis.

3.4.2 Data Analysis for Slope Stability

The Geo Studio software of Slope/W was used for the analysis of static slope stability of Gomit earth-fill dam at standard loading condition. The procedure of slope stability will be listed below.

(1) Defining the geometry

The geometry defined in terms of regions or n-side closed polygon for doing the analysis. A region can have only one type of material (soil type). The same type can be assigned to many different regions, but each region can have only one soil type. Similar methods used for importing the geometry of the dam.

(2) Defining material property

Mohr-Coulomb strength parameters C (cohesion) and Ø (phi) are available to describe the property of material in terms of soil strength. The parameters may be total or effective depending on the pore-water pressure conditions specified. Un-drained strengths are specified by making Ø zero. The Property of material used for doing the analysis described in table 3.9.

(3) Defining the trial slip surface

Finding the critical slip surface involves a trial procedure. A possible slip surface is created and the associated factor of safety is computed. This is repeated for many possible slip surfaces and at the end; the trial slip surface with the lowest FOS is deemed the governing or critical slip surface. The trial slip surface defined in terms of

- Slip surface shape- circular and non- circular. Circular slip surface selected for the study. It is similar with the observed slope failure.
- Slip surface location- upstream and downstream.
- Method- for the study, two methods used for determining the position of slip surface i.e, entry-exit and grid and radius for Fellenius method.

(4) Loading condition

The Gomit dam and its foundation was analyzed and designed against failure by slope instability. Consideration of loading conditions which may result to instability must be made for all likely combinations of reservoir, seepage conditions, both after and during construction. The standard loading conditions that are examined in the static stability analysis are end of construction, steady state seepage and rapid draw down.

- End of construction

The condition of end of construction may be critical for both upstream and downstream slope stability. There is no water in the reservoir and embankment dam. The bulk unit weight and the total or un-drained shear strength parameters of soils are used in this loading case.

- The steady state seepage condition

The condition of steady state seepage through an embankment may be critical for the downstream slope stability. The bulk unit weight and effective or drained shear strength parameters of soils are used in loading case.

- Rapid draw down

The condition of through an embankment may be critical for the downstream slope stability. The bulk unit weight and the effective or drained shear strength parameter of soils are used in this loading case.

(5) Analysis method

The limit equilibrium method methods consist of cutting the slope into a number of slices and applying appropriate equilibrium equation (force and /or moment equilibrium). In LE, the methods used for the stability analysis of both upstream and downstream slopes are the ordinary, Bishop’s simplified, Janbu’s and Morgan stern - price method.

(6) Analysis result

The Slope/W software computes the shear stress of slip surface corresponding to shear strength to determine the factor of safety. The analysis result shows the critical slip surface and the minimum factor of safety.

4. RESULT AND DISCUSSION

4.1 Result of seepage Analysis

The amount of seepage passing through the dam body using phreatic line calculated for both homogeneous and zoned dam as shown in Fig13 and 14.

For zoned dam, the presence of a chimney drain and horizontal filter drain within the seepage model, the phreatic surface changed as compared to the analysis was done for homogeneous. The phreatic surface moved horizontally from the normal pool elevation and was intercepted by chimney filter drain where the zero pressure boundaries conveyed the majority of seepage water through the drain. The analysis also shows that the shape and position of phreatic line depends solely on the actual boundary conditions, geometry and material properties of soils. The phreatic line and the total estimated seepage through the dam body was done on Ms Excell worksheet as shown on the appendix. The phreatic line for both homogeneous and zoned dam at normal pool level was drawn and as shown in Fig 17and Fig18 respectively.

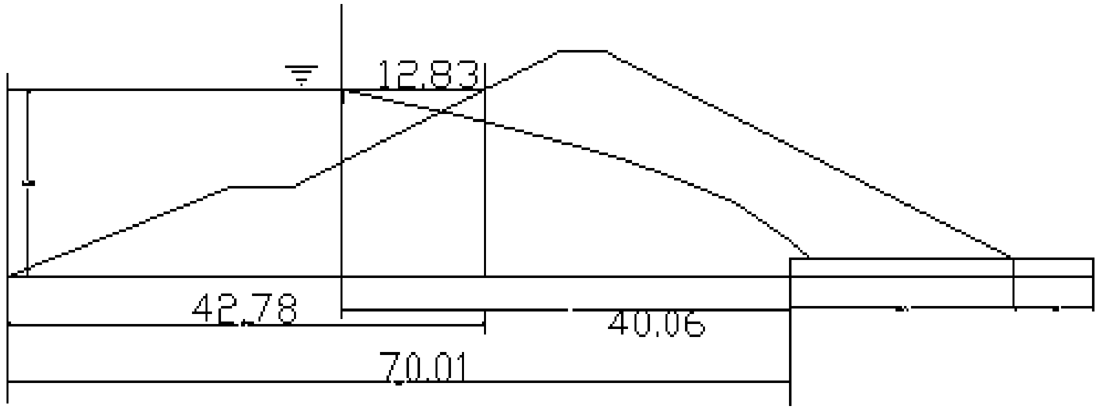


Fig-13 Phreatic line at NPL for homogeneous dam

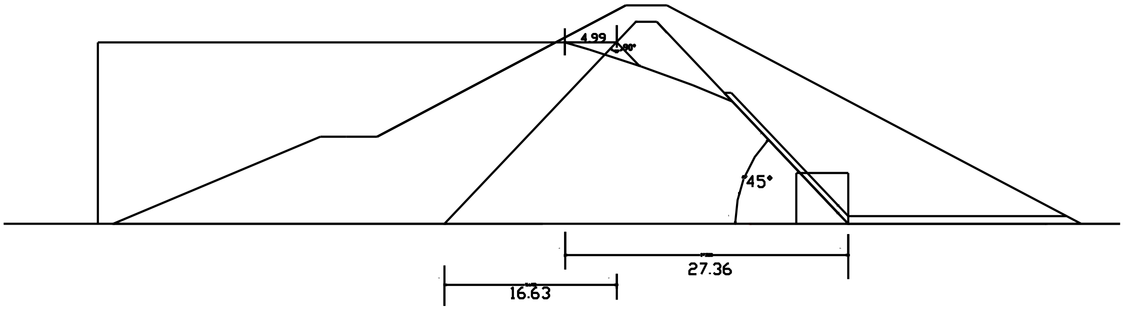


Fig-14 Phreatic line at NPL for Zoned dam

For numerical method, Seep/w software model was calculating the seepage water at normal pool level for the three cases; homogeneous, zoned and zoned dam with foundation. At the center of the dam, the flux section is drawn as a blue dashed line with arrows at the top (endpoint) of each section across the seepage flow and the value labeled on it as shown in Fig15, 16 and 17.

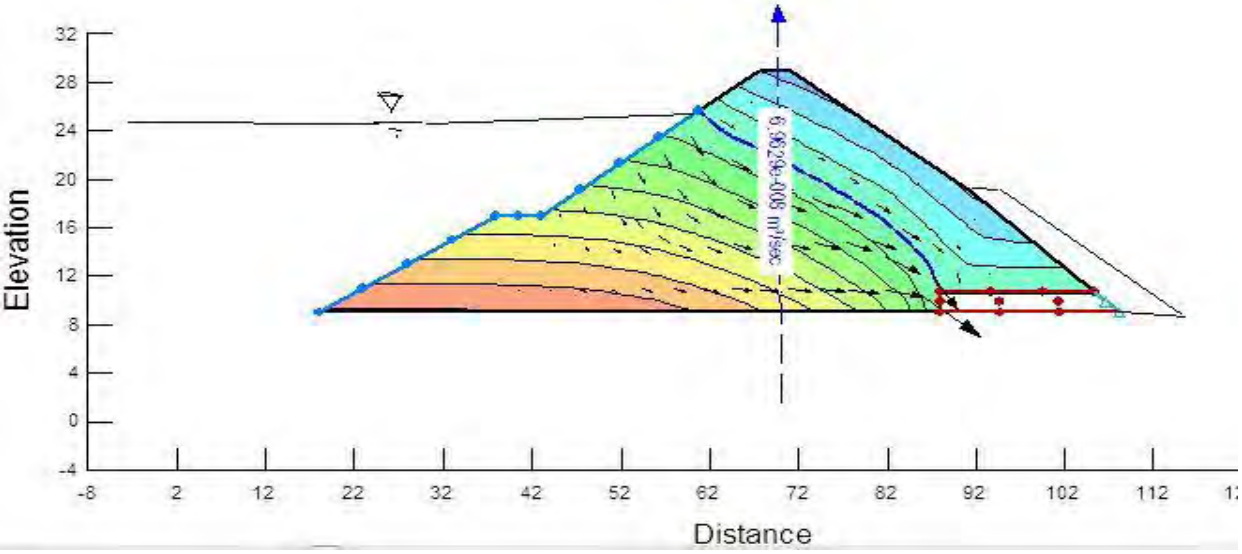


Fig-15 Seepage through the homogeneous dam

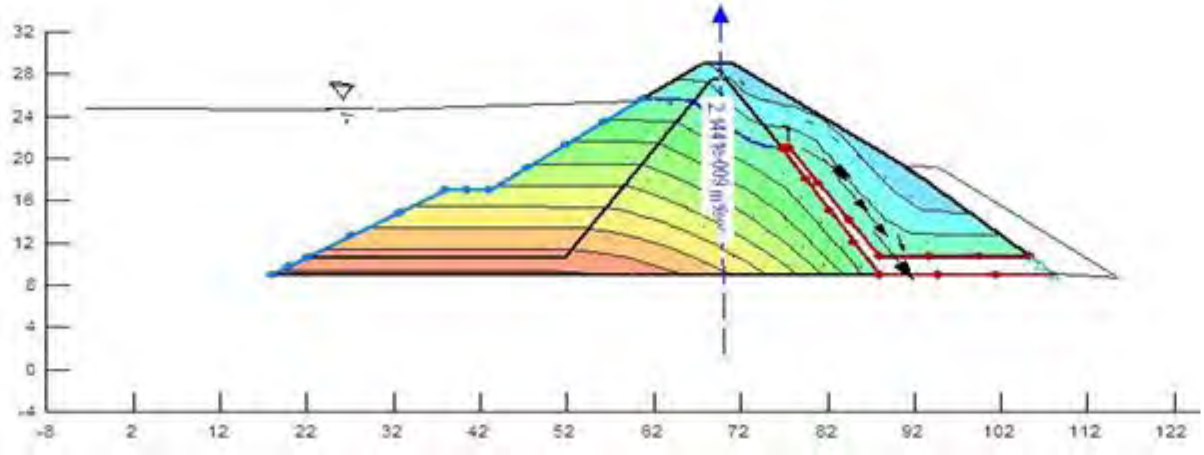


Fig-16 Seepage through zoned dam

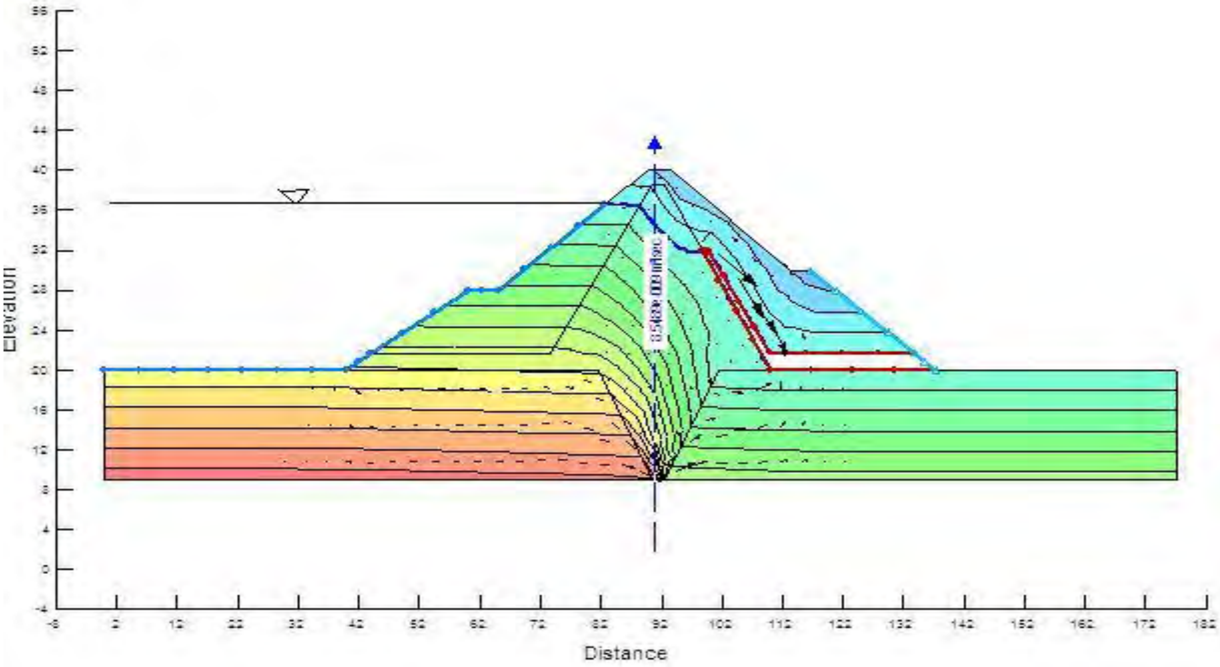


Fig-17 Seepage through zoned dam with foundation at NPL

To compare the computed result of seepage analysis with the previous study, the result for both analytical and numerical methods was shown in Table 9.

Table-9 Result of seepage at normal pool level

Analysis type	Methods	Homogeneous dam with horizontal filter (m^3/s)	Zoned dam with chimney & horizontal filter (m^3/s)	Zoned dam including foundation with chimney and filter (m^3/s)
Seepage	Analytical by PL	$1.65*10^{-4}$	$2.73*10^{-5}$	-
	SEEP/W	$2.2*10^{-4}$	$2.0*10^{-6}$	$3.5*10^{-6}$
Seepage at the design document	Analytical by PL	$3.18*10^{-5}$	-	-

From Table 9, the study result show that the maximum seepage through the dam body calculated using Seep/W software for homogeneous dam with horizontal filter is $2.2*10^{-4}m^3/s$. However, since Gomit dam was a zoned dam, only the maximum value of seepage between zoned dam with chimney drain and horizontal filter($2*10^{-6}m^3/s$) and zoned dam with chimney drain and horizontal filter including foundation($3.5*10^{-6}m^3/s$) should be compared with the value at the design document($3.18*10^{-5}m^3/s$). This result shows that the value of seepage estimated in the design document is higher than the calculated value seepage. This means the design document estimate safe amount of seepage passing through the dam body and foundation. So, in the evaluation of the previous study of seepage analysis, the results indicate that there is no problem in quantifying the amount of seepage through the dam body foundation.

During the current condition of evaluation, the result of direct measured value of seepage and Seep/W software at current elevation 2359.76 m or at a water depth of 9.4 m shown in Table 10.

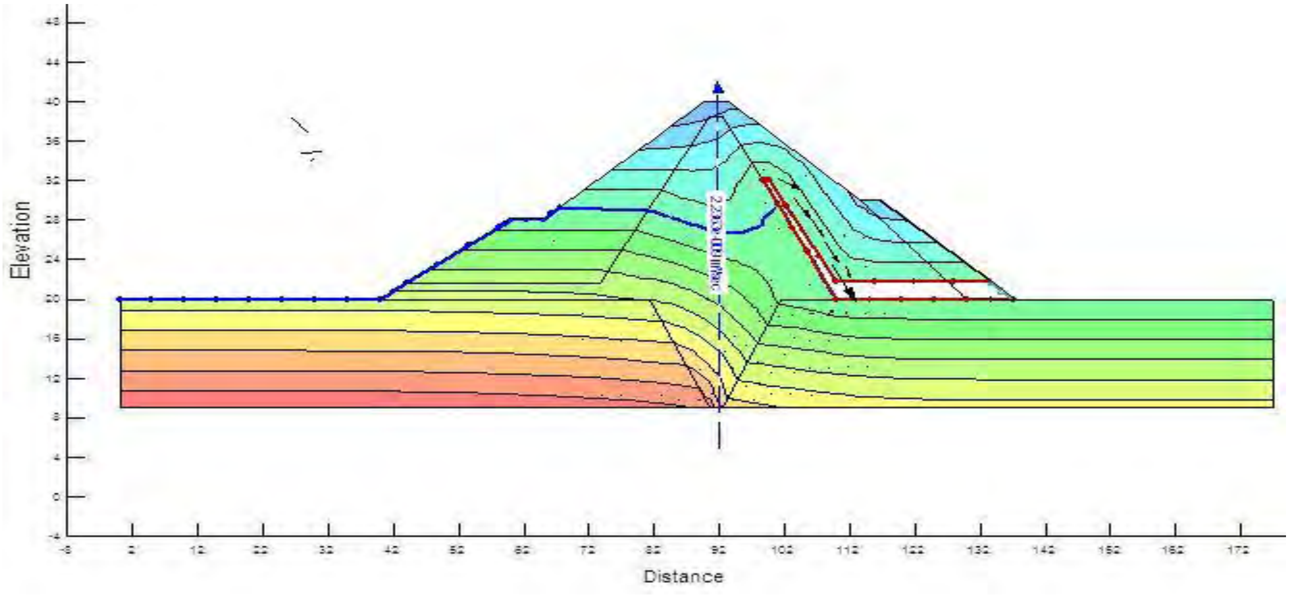


Fig-18 Seepage through zoned dam with foundation at current water level

Table-10; Result of seepage at current water level

Analysis type	Methods	Zoned dam including foundation(m^3/s)
Seepage	Seep/w	$7.26*10^{-7}$
Seepage	Measured seepage from foot of spillway side wall	0.0002
	Measured seepage at downstream	0.0038
Total seepage	Measured at downstream	0.0040

From Table10, the measured seepage is higher than the estimated value using software at both current water and normal pool level. This shows that there is an excessive seepage through the dam body and foundation which may lead to seepage failure as mentioned earlier. The causes for excessive seepage flow through the dam body and foundation will be described below.

4.1.1Causes of Excessive Seepage

Performing seepage analysis requires significant judgment and relies on many assumptions. To alleviate some of uncertainty, it is important to obtain the as Built drawing done by contractor, subsurface and the dam body materials data from several sources like geological and soil laboratory results in order to get detail idea on the dam and foundation materials.

The seepage through the dam body, foundation, around conduits and in the joints is exposed to an adjacent layer, there will be a potential for erosion and removal of the soils which will allow further progression of the piping. Due to the absence of installed and/or measured data from dam monitoring instruments and absence of all available data in the design document, it is difficult to judge the analysis without a complete understanding and detail investigation of the problem. One or more problems which are stated below can lead to excessive seepage passing through the dam body and foundation.

- Poor/under compaction

  Construction related defects like compaction are very difficult to detect. Compaction is a process to increase the density of soil mass by mechanical means, which usually involves rolling, vibrating, tamping or combination of these process, such process is to rearrange the soil particles with expulsion of air in the voids of soil mass without major changes in the pore water except for granular soil, in which water is possible be pressed out.

  Lower compactness of soil skeleton implies lower soil strength, differential settlement, lower compressibility higher permeability and higher susceptibility of other engineering property to water content alteration. So if compaction not properly done during early stage of construction it leads to excessive seepage through the dam body and foundation and finally total dam failure.

- Improper design and wrong placement of filters

  Gradation curve and placement of the filter materials and drainage design are the critical part of embankment dam design, but these are also not stated in the design document. The design document stated that the designer to use the available filter materials obtained from the dam site if the coefficient permeability ratio of shell material to core is greater than 25. Poor design of filter create preferential flow path of crack through the dam body, foundation and at contact between the embankment and spillway and increase the seepage flow to downstream of the dam.

- Problem on laboratory test and result

  The laboratory test and result of construction materials have an effect on the formation of excessive seepage through the dam body and foundation. If proper care was not given in sample taking and result or generally some quick laboratory test give uncorrect results.

- Insufficient length and thickness of impervious blanket

  Excessive seepage through the dam body and foundation will appear in zoned dam due to insufficient length and thickness of horizontal blanket on the upstream that does not connected to central core of the dam. The Gomit dam has an impervious blanket that connected to the central core of the dam to seal the upstream stored water passing into the foundation. However, the presence of cracks or other preferential flow paths are more likely to occur at the upper part of horizontal blanket and resulting excess seepage through the dam foundation.

- High intra and inter layer permeability variation

  For non-isotropic soils, the permeability is different in the horizontal and vertical direction, so if a high permeability in the horizontal (intra) than vertical (inter) or Kh>Kv, excessive seepage water can pass from upstream to downstream.

4.2 Result of Stability Analysis

Determination of the potential failure surface (slip surface) and the corresponding forces tending to cause slip and to restore or stabilize the sliding mass, and the computation of available factor of safety are in the stability analysis. The determination of the critical failure surface (a failure surface for which factor of safety is minimum) is central to a slope stability analysis. The Stability problem of Gomit embankment dams was preceded or accompanied by seepage problems. It is essential to doing the stability after the understanding of seepage occurring through the dam and its foundation. For static stability analysis, Slope/W software used to evaluate the type of potential failure surface which is similar to that of observed failure (i.e. circular) and the location of potential failure surface that is both upstream and downstream slope surface analyzed using limit equilibrium of different methods for the standard loading conditions.

End of construction

At the end of construction, significant pore pressure development is expected in the embankment body and foundation either from rainfall or compaction water. So the upstream and downstream slope stability of Gomit earth fill dam was assessed at this loading condition using the slope/w and the result shown in Fig 19 and 20 respectively.

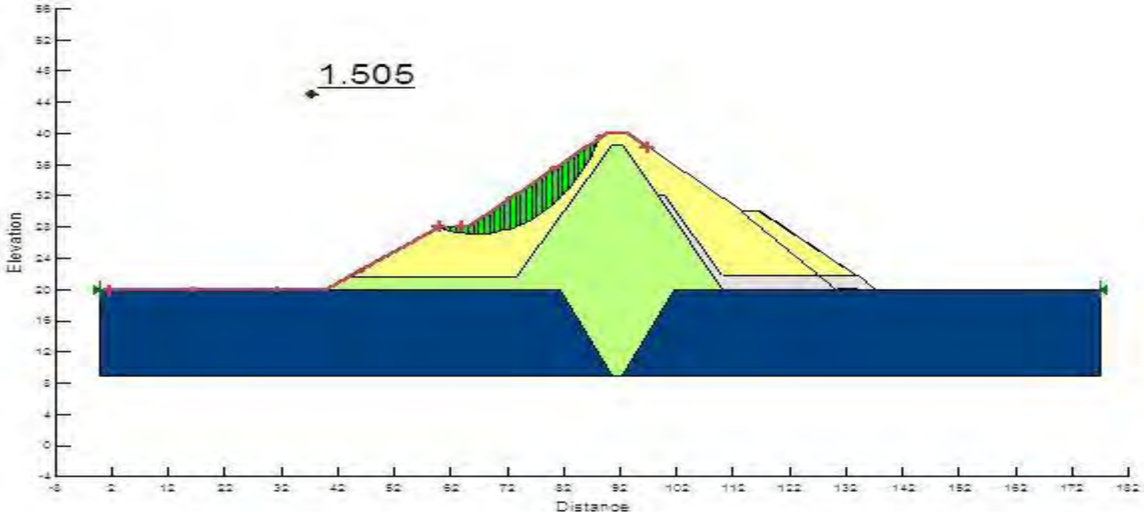


Fig 19 The minimum FOS for Upstream at the end of construction

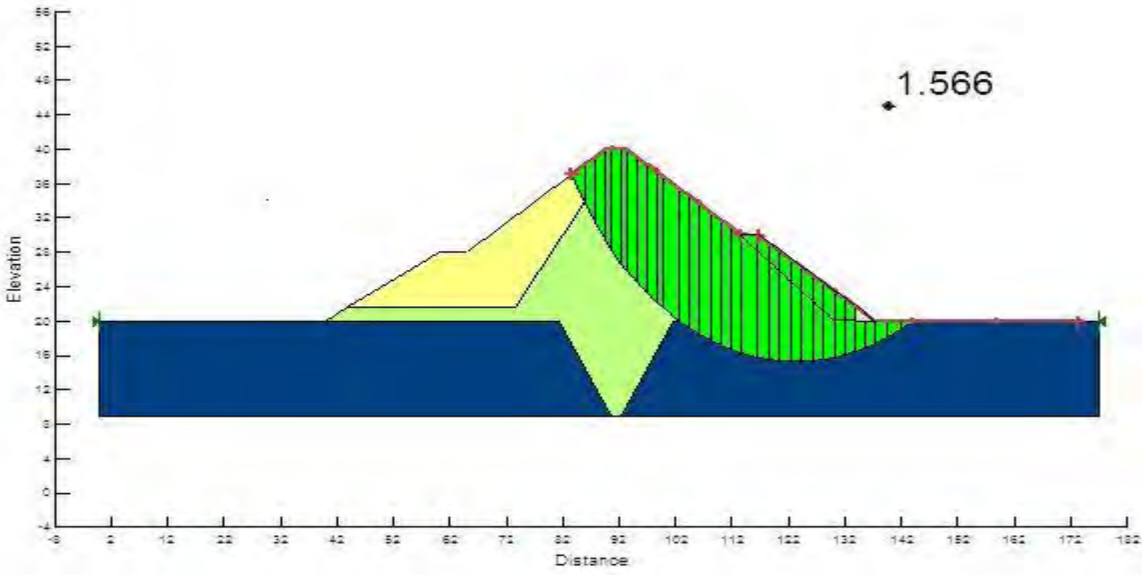


Fig 20 The minimum FOS for Downstream at the end of construction

Steady state seepage

For the long-term operation, the phreatic surface within the embankment has been established and critical for downstream slope. So the downstream slope stability of the dam was assessed and the result shown in Fig below

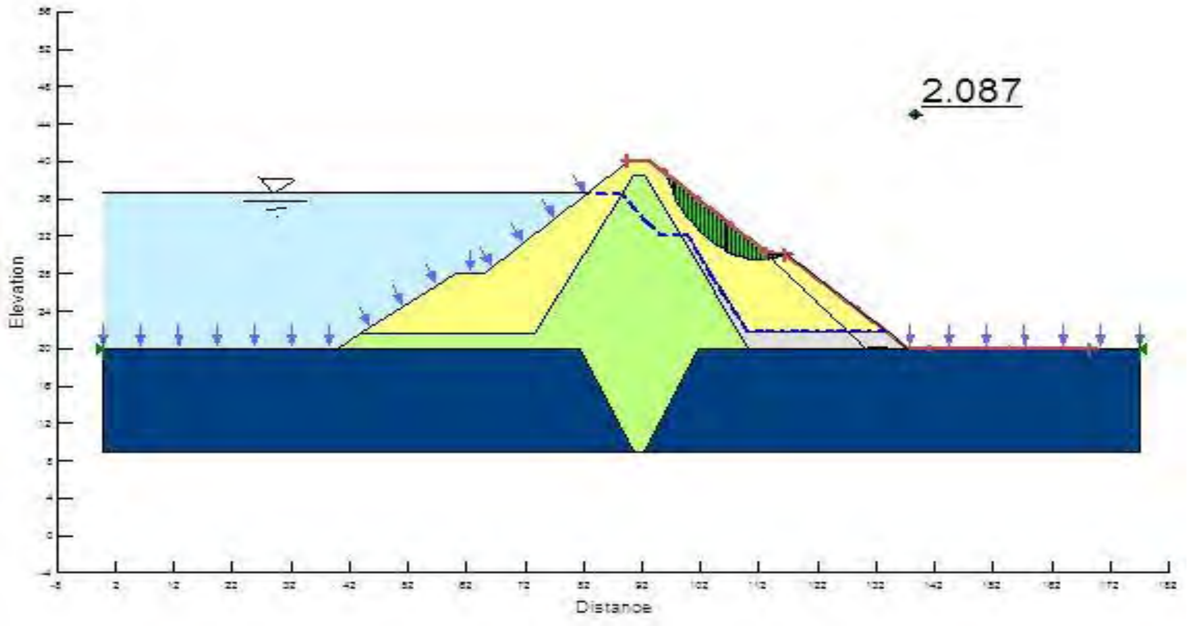


Fig 21 The minimum FOS for Downstream at the Steady state of Seepage

Rapid draw down

When water is suddenly withdrawn or in other words if the level of water in the reservoir reduces suddenly, the soil on the upstream face of the dam body may be highly saturated and has pore water pressure that tries to destabilize the dam and if this force is high enough, it can fail the dam. So the upstream slope stability of the dam was assessed and the result shown in Fig 22.

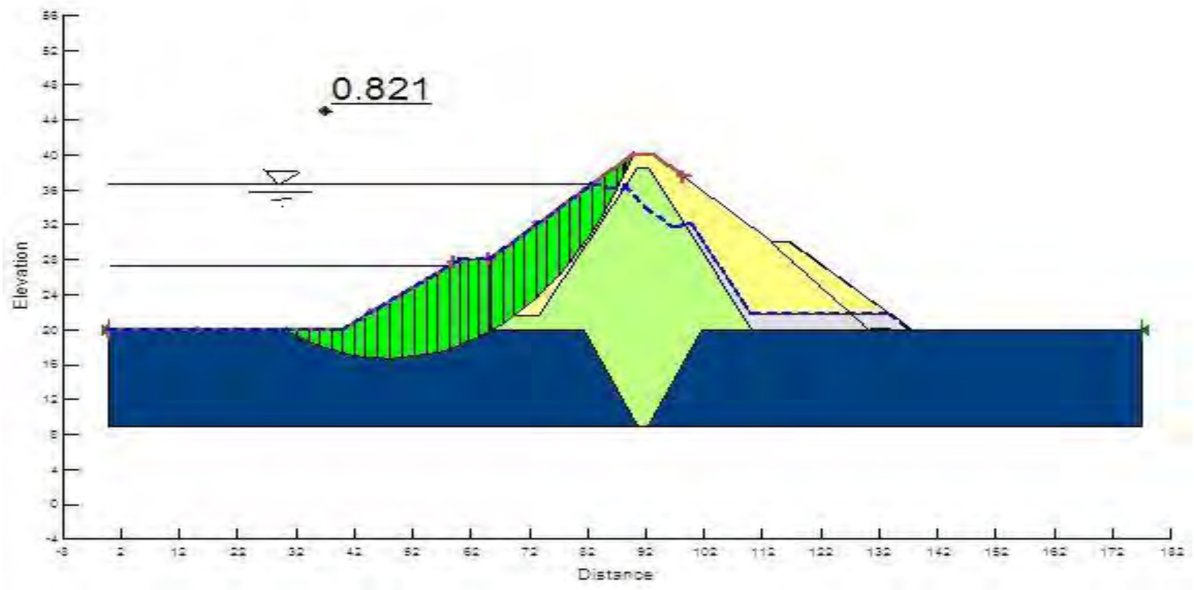


Fig 22 The minimum FOS for Upstream at Rapid draw down

Fellenius-Jumkis Method

For Fellenius-Jumkis method, the locus of the critical slip circle including the intersecting line of the two angles (line PQ is the possible center of the critical slip surface) drawn by AutoCAD and shown in fig below and analysis was done on this critical line.

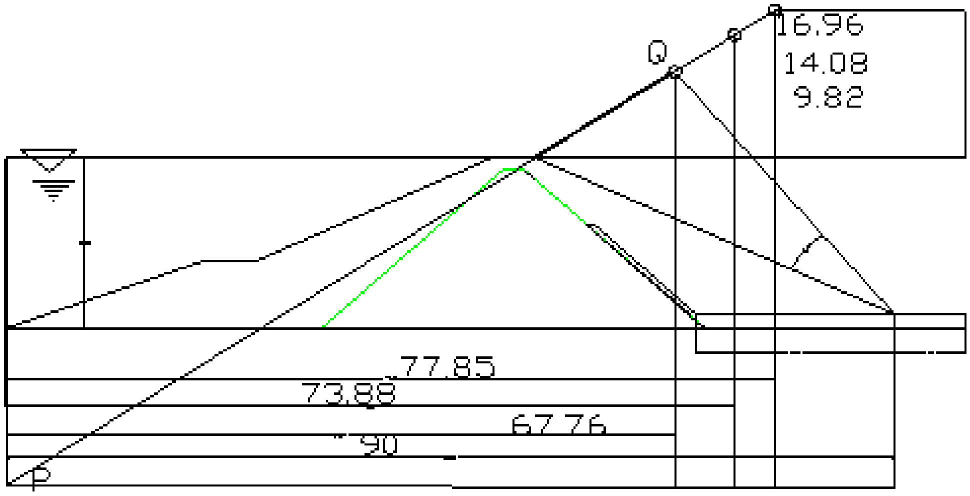


Fig 23 The center of critical slip surface using Fellenius-Jumicks Method

The possibility of the most critical surface for downstream slope lies on the line of PQ. To know the cause of the downstream slope failure, the Fellenius-Jumicis critical failure determination was done by taking the points on the line of PQ and the result of minimum FOS is 2.02 as shown in Fig 24.

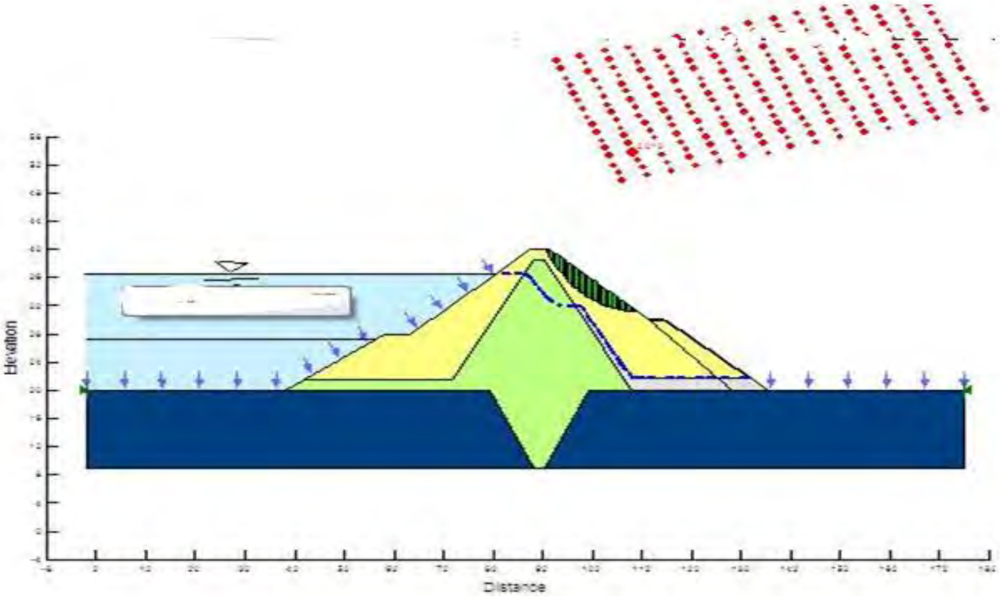


Fig 24 The minimum FOS for critical slip surface using Fellenius-Jumicks Method

Table 11 FOS result for critical surface

Analysis type	Methods	Minimum factor of safety
		Steady-state seepage (D/S)
Static stability analysis for Fellenius-Jumikis critical failure points and the minimum FOS for point	OM	2.022
	BM	2.013
	JM	2.119
	M-PM	2.043

Stability analysis was done by Slope/W software using grid and radius slip surface locating method and the minimum factor of safety using different stability methods as shown in the above table. The result shows that the dam was stable, because the value of FOS is greater than the minimum acceptable value provided by U.S Army.

For evaluating the previous result, the static slope stability analysis by Geo studio product of SLOPE/W for the different limit equilibrium method (Ordinary, Bishop's, Janbu's and Morgenstern-Price method) using entry and exit trial slip surface at standard loading condition summarized and compared as shown in Table12.

Table 12 Evaluating the previous study of static slope stability at NPL

Analysis type	Methods	The minimum factor of safety			
		End of construction		Rapid/Sudden draw down	Steady-state seepage
		U/S	D/S	U/S	D/S
Static stability	OM	1.505	1.566	0.844	2.08
	Bishop's	1.638	1.720	1.015	2.24
	Janbu's	1.508	1.561	0.926	2.07
	M-P	1.638	1.726	1.014	2.23
Stability in the design document	Fellenius method	-	-	1.31	1.544

For stability analysis of embankment dams, the recommended factors of safety will vary with loading conditions. The factor of safety for a short term (end of construction) and a long term operation (steady state and rapid draw down) should be greater than the minimum acceptable value stated by U.S Army corps (shown on Table 7).

From Table12, the minimum factor of safety calculated for end of construction (FOS= 1.505) compared with the minimum acceptable safety factor provided by U.S Army Corps (FOS=1.3). The result shows that the dam was stable at the end of construction The current study result for long term shows that the dam was safe for steady state seepage (FOS=2.07) but upstream slope will be slide for rapid draw down (FOS=0844). In the design document, the calculated factor of safety for a long term operation stated was also near to the minimum acceptable safety factor i.e 1.54 for steady state seepage and 1.31 for rapid draw down, which is critical/endangered value. From the field observation and literature review, for a long term operation of steady state seepage, the downstream slope failure model will be done below by correcting the material data mainly the cohesion and angel of friction value of core and shell material. The downstream slope failure occur at asteady state seepage condition. The observed slope failure drawn using AutoCAD as shown in fig below.

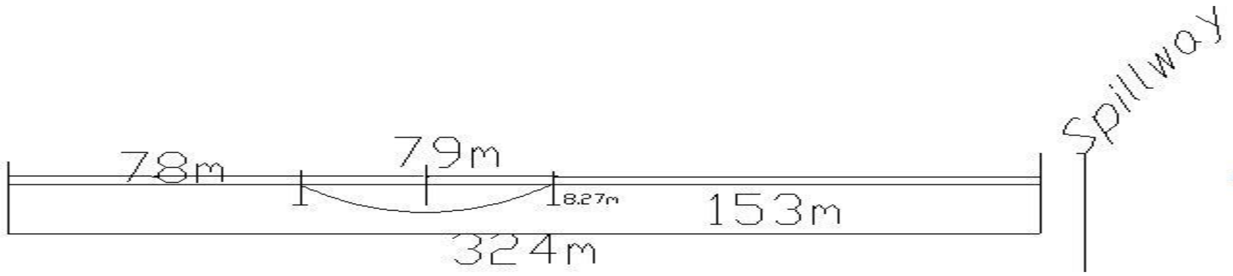


Fig 25 The front view of downstream slope failure part

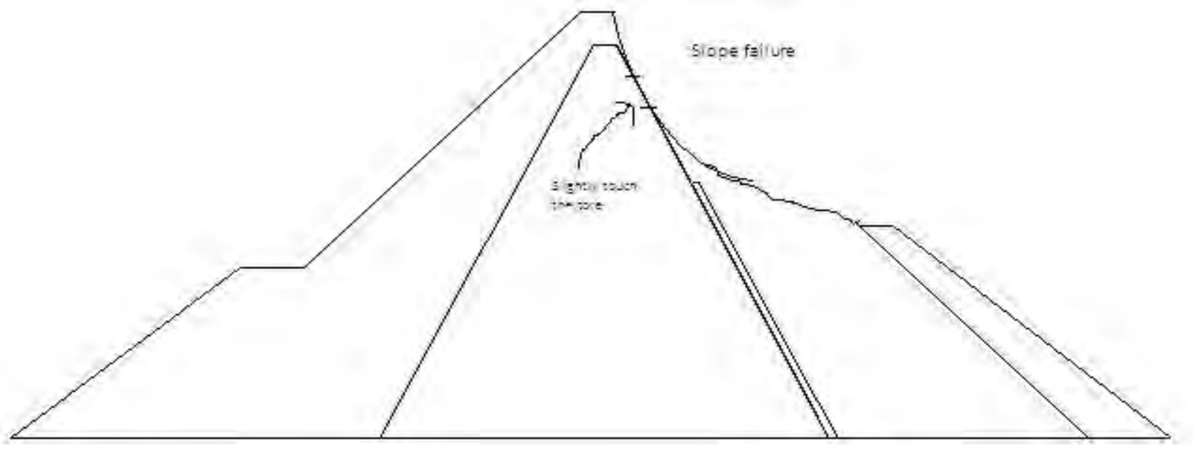


Fig 26 Downstream slope at current condition

4.2.1Causes of Slope failure

Slope failure occurs when the downward movements of material due to gravity and shear stresses exceeds the shear strength. Therefore, factors that tend to increase the shear stresses or decrease the shear strength increase the chances of failure of a slope. Different processes can lead to reduction in the shear strengths of rock mass. Increased pore pressure, cracking, swelling, decomposition of clayey rock fills, creep under sustained loads, leaching, strain softening, weathering and cyclic loading are common factors that decrease the shear strength of rock mass. In contract to this the shear stress may increase due to additional loads at the top of the slope and increase in water pressure in cracks at the top of the slope, increase in soil weight due to increased water content, excavation at the bottom of the slope and seismic effects. In addition to these reasons factor contributing in failure of slope are properties of rock mass, (slope geometry), state of stress, temperature and erosion. There are different factors affecting the slope stability of embankment dams such as

- Geological Discontinuities - Fault, Joint, bedding plane,
- Effect of Water - Ground water, drainage pattern, rainfall, permeability, aquifer
- Construction material Strength - Shear strength, compressive strength, tensile strength
- Geotechnical parameters - Gran size, moisture content, atterberg limit, etc.
- Method of construction - Shovel, dumper, BWE or combination
- Dynamic forces - Blasting, Seismic activity
- Geometry of slope - Height and angle of slope, bench height and angle.

However, the major causes for slope failure of Gomit embankment dam are listed below;-

Laboratory Test Result

In the design document Unconsolidated-Undrained (UU) box shear test was used for all loading conditions. But this test used only for end of construction loading condition. However, the Consolidated-Undrained (CU) shear test used for Rapid draw down and Steady state seepage loading condition. For these two loading conditions, the Consolidated-Drained (CD) shear test used for materials having permeability coefficient greater than10^{-4} cm/sec. Since the permeability coefficient of the foundation ($3.42x\ 10^{-4}$ cm/sec) was greater than10^{-4} cm/sec, CD test should be required but not done.

Sediment load

This is usually a long-term problem if the catchment areas of the dam not properly conserved. The entry and deposition of sediment will decrease the total water storage capacity and the stability of the dam. Sediment load increase the driving force and if the driving force is greater than the resisting force, the soil mass will slide along the slip surface and a slope stability failure will occur. Since the catchment area was not conserved, this has an effect on the stability of the dam due to the entry of sediment increases the driving force.

Settlement

Settlement can arise in zoned dam when due to poor construction techniques. The core has been compacted comparatively more than the other part of the embankment (shell material). The upstream and downstream sides or shoulders of the embankment settle more than the core as they are less well compacted and as a result cracks appear along the crest edges and slides as the settlement takes place.

Loading Condition

If neither erosive action nor natural hazards occurred on the dam, the downstream slope failed at steady state seepage condition. Using the material properties in the design document, the calculated factor of safety for this type of loading conditions is 2.07, which is safe but in reality not safe. From the field observation, for a long term operation of steady state seepage, the downstream slope failure model will be done below by correcting the material data mainly the cohesion and angel of internal friction values of core and shell materials. During construction or after construction in service time the cohesion and angel of internal friction (c and Phi value) not attained the real value stated in the design document. So the C and phi value calculated using back calculation or iteration determined just at failure condition by making factor of safety less than 1.

Table 15 Result of Back calculation value just at failure as compared with design document value

Material	Type	At design document		Back calculation just at failure	
		(Cohesion, C KPa)	Angle friction Ø ^{o}	(Cohesion,C KPa)	Angle of friction.Ø(^{o})
Shell	WG	3.8	39	0	25
Core	CH	14	20	14	19
Foundation	CH	16.6	23	16.6	23

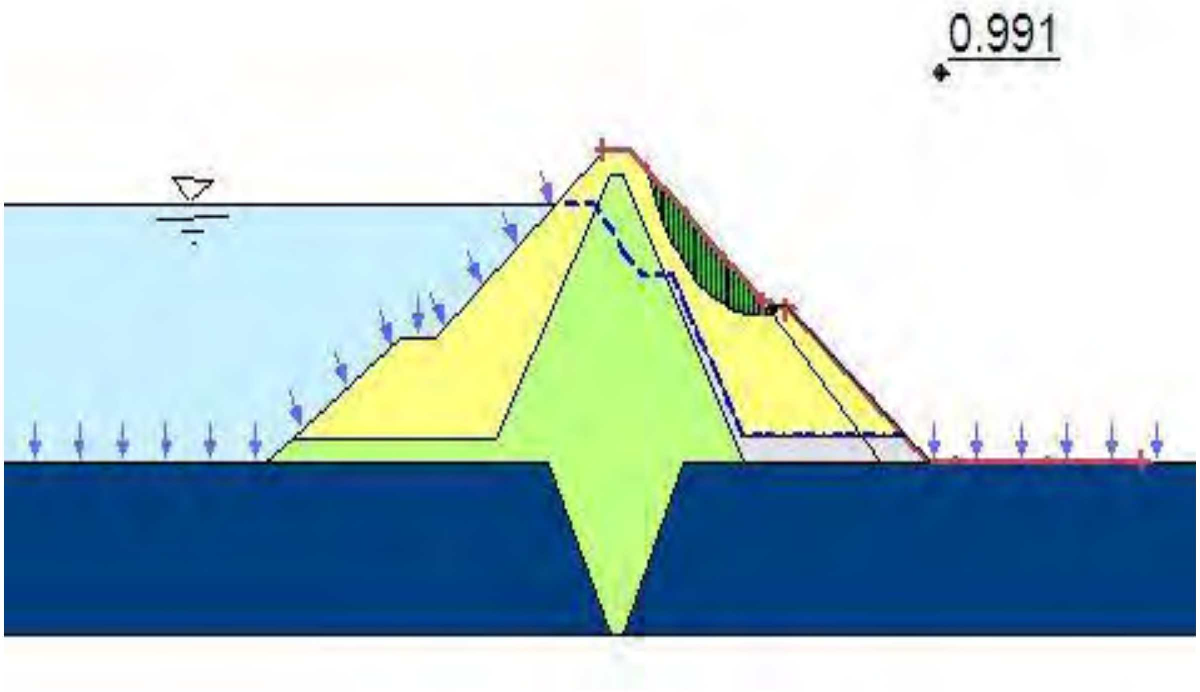


Fig 31 Gomit dam at current condition using Slope/W model

The above fig31 shows that the downstream slope failure for the loading condition steady state seepage using input back calculation (by making Factor of Safety ≤ 1) and these data are different from the data stated in the design document. The result shows the slope failed because of the calculated factor of safety (FOS=0.991) less than the minimum safety factor (FOS >1). For shell material, the angel of friction stated in the design document is higher than the value taken from back calculation which gives similar with the observed failure at the downstream of the dam.

5. CONCLUSION

The main objective was to evaluate the Gomit embankment dam failure, emphasis on seepage and slope failure and finally to come up with the possible remedial measure. The methods used for seepage analysis are analytical by phreatic line and numerical by Geo studio software product of SEEP/W model. The total seepage calculated using analytical by phreatic line for zoned dam and using SEEP/W for zoned dam with chimney drain and horizontal filter including foundation compared with the value stated at the design document. The result shows that the design document has no problem on quantifying the expected quantity of seepage. But at current water elevation, the measured seepage is higher than the value of seepage calculated at the current and normal pool level. This shows that there was excessive seepage passing through the dam body and foundation. The causes for excessive seepage are problem on laboratory test and result, poor filter design, faulty construction and poor maintenance, under compaction, wrong placement filter materials in different zones, if not properly done. The existence of one or more problems, lead to a great loss of reservoir water moving through the dam body and foundation into the downstream, which result in dam failure by seepage.

Slope/W software was used to calculate the factor of safety under the standard loading conditions for limit equilibrium methods of Ordinary, Bishop, Janbu, and Morgenstern-Price method and compared with the observed downstream slope failure at steady state seepage. The result shows the dam was safe under this loading condition. However, from the field observation, the downstream slope was failed and the model was done by correcting the material input data mainly the cohesion and angel of friction value of core and shell material. The factor of safety result just at failure (at current condition) is 0.996 using back calculation, which is less than the stability safety factor (1) and resulting slope failure at downstream that was similar to observed failure. The major cause for downstream slope failure is the steady state loading condition in which the shear strength parameters of cohesion and angel of internal friction(**C** and Φ) value at current condition don’t attain the real value stated at the design document.

6. RECOMMENDATION

The seepage and stability analysis was done using the available data from the design document. The result obtained from the different methods of analysis shows that there is no problem on the estimating seepage. However, the field observation shows slope failure at the downstream and the actual measured value was higher than the estimated seepage. The following major recommendation arises on the basis of the findings:-

- The need to reduce the risk of embankment failure has led to costly remedial measure that are planned and executed without a complete understanding and detail investigation of the problem. So detail analysis and investigation require on the property of dam and foundation materials, degree of compaction, design and placement of filter materials by multi-disciplinary teams before doing the remedial measure.
- Installing dam monitoring instruments, frequent instrument reading and making observation use to provide data and information which can be used to assess the performance of the dam under normal and extreme loading condition and to manage the risk associated with operation and maintenance.

- In addition to the above, the Gomit dam was highly exposed to sedimentation problem due to the absence of soil conservation structures on the catchment areas. So different soil conservation structures like planting trees (covering the area with forest), bunds, and terraces are suitable for controlling sediment entry to the dam.

- Removal of the failure and weak part of the dam and raising the dam height and normal pool level after assessing and adjusting the foundation. This is because of high sediment accumulation on upstream lowered the height and reservoir storage capacity. If the incoming of probable maximum flood will increase the water level and lead to total dam failure by overtopping.

6.1 Remedial Measures for seepage

(1) Grouting at the contact between the embankment and foot of spillway side wall or abutment wall. The contact zone can be treated by cement grout with suitable admixture.

(2) Sufficient length and thickness of impervious blanket on the upstream that connected to central core of the dam can be used to seal the reservoir bottom and sides and thereby reduce seepage quantities and pressures beneath a dam.

(3) Cracks or other preferential flow paths are more likely occur at the upper part of the embankment, at the embankment/abutment contact, at the embankment/foundation contact, around and above a conduit, at the contact between the embankment and spillway are remedied with the effective design of filter. Since proper filter application will stop particle erosion through a crack, a process known as self-healing, to prevent failure of the dam by piping.

(4) Rehabilitating existing toe drains or adding both toe drains and drainage zones.

(5) Downstream pervious berms control seepage by increase the seepage path, ensure no upward migration of fine particles and prevent scouring of d/s. This implies filter compatibility between the berm and the foundation

6.2 Remedial measure for slope failure

(1) Removing the weak zone around the failure part and proper filling and compacting with same materials /well graded gravel or shell material at the top and downstream slope failure part of the embankment dam.

(2) Berms will improve the stability of an embankment dam, a one possible means to prevent such stability problems of this dam is to raise and re-construct a stabilizing berm of coarse material along the d/s toe of the dam. An added advantage of a downstream berm is to increase slope stability because of the additional resistance to sliding provided by the berm. The berm along the dam toe is re-constructed according to the following principles:

(3) The downstream slope should be protected against the erosive action of rain and its runoff. So, cover the exposed area with grass and regular maintenance need on the downstream slope of the dam.

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APPENDIX

Data from design document

(1) Geometry of the dam

- Crest length=324m
- Top width=4m
- Bottom width=91m
- Dam height=20m
- Maximum water level=2368.4m
- Normal water level=2367m
- Crest level of dam=2370.36m
- Bed level=2350.36m
- U/S shell slope=2:1
- D/S slope above the berm=2:1
- D/S slope below the berm=2.5:1
- U/S and D/S core slope=1:1

(2) Property of dam material

(*) Core material

- Type= caly soil
- Permeability coefficient=$2.3\text{x}10^{-8}$cm/s
- Specific gravity=2.68

- Natural moisture content=39%
- Optimum moisture content=31%
- Maximum dry density=1.34g/cc
- Porosity(η)=0.5
- Cohesion of soil=1400kg/m^2
- Angle of internal friction(φ)==24°
- Bulk/dry unit weight(γ_{bulk})=820kg/m^3
- Saturated Unit Weight(γ_{sat})=1820kg/m^3
- Submerged Unit weight(γ_{sub})=1760kg/m^3

(*) Shell material

- Type=Sandy gravel
- Permeability coefficient=1.53x10^{-6}cm/s
- Natural moisture content=
- Optimum moisture content=30%
- Maximum dry density=1.43g/cc
- Porosity(η)=0.5
- Cohesion of soil=380kg/m^2
- Angle of internal friction(φ)=39°
- Bulk/dry unit weight ($\gamma_{\mathbf{bulk}}$)=870kg/m^3

- Saturated Unit Weight(γ_{sat})=1870kg/m^3
- Submerged Unit weight(γ_{sub})=1860kg/m^3

(*) Foundation material

- Depth of foundation= 9m
- Type =Sandy gravel,and clay(cutoff)

(3) Geometry of Spillway

- Type=Ogee shape
- Crest length=25m
- Crest level=2667m
- Design discharge(Q_{design})= 87.84 m^3/s
- Design head(H_d)= 1.40m
- Spillway height= 0.7m

Shell material coordinate

X	Y

0	0
20	8
25.5	8
49.5	20
53.5	20
90.1	1.7

Core material with chimney drain

32	0
50.5	18.5
52.2	18.5
59	12
70.01	1.7
71	0
90.09	0
97.25	1.7

Phreatic line calculation on Ms.Excell

Homogenous Dam

At max water level

X	37.77
Y	18.28
S	4.191069

(1) S=(x^2+y^2)^0.5-x

(2) Y=(S^2+2*S*x)^0.5

x	0	2.5	5	7.5	10	12.5	15	17.5	20
y	4.19	6.206481	7.712052	8.968338	10.06908	11.06082	11.97068	12.8161	13.60911

Q= K*S

Kshell	1.53E-06
S	4.191069
Q	6.41E-06
L	324
Q_{totl}	0.002078

At Norrmal pool Leve (d=16.64m)

X	40.06m
Y	16.64
S	3.318

(1)$S=(x^2+Y^2)^{0.5}- X$

(2)$Y=(s^2+2SX)^{0.5}$

At F; X=0 , Y = S = 3.318m

At V; Y=0 , X = -0.5 S = -1.659

Tabel;- phreatic line points

X	0	2.5	5	7.5	10	12.5	15	17.5	20	22.5
Y	3.318	5.25	6.648	7.796	8.797	9.694	10.515	11.277	11.989	12.663

X	25	27.5	30	32.5	35	37.5	40
Y	13.302	13.911	14.496	15.057	15.598	16.131	16.628

The amount of seepage

$q=K_{shell} \text{ x } S$

$=1.5\text{x}10^{-7} \text{ x } 3.318$

$=5.08\text{x}10^{-7} m^2/s$

The total crest length of the dam is 324m,so the total amount of seepage

$Q=q \text{ x } s$

$=6.4\text{x}10^{-6}\text{x } 324$

$=0.000165m^3/s$

Zoned dam

Zoned dam(NWL=16.64m)

L-Cassagrande solution for $a>30^0$

$a=45^0$

a	Da/a+Da
30	0.36
45	0.34
60	0.32

b 27.36 $b^2=$ 748.5696

h 16.64 $h^2=$ 276.8896…… (b^2*h^2)^0.5 =32.02279

$a = (b^2+h^2)^{0.5}- (b^2-h^2\cos^2 a)^{0.5}$

$(b^2+h^2)^{0.5}$ 32.02 ………………(1)

pie 3.14

$\cos^2 a$ 0.50

$(b^2-h^2\cos^2 a)^{0.5}$ 24.70 ……………….(2)

a 7.32

Da/a+Da 0.34

a+Da

7.32*0.34 = 2.4889

Da= 0.34(7.32+Da)

Da= 2.489+0.34Da

0.66Da = 2.489

Da = 3.771

to check Da/a+Da 0.340 ……………………..correct!!

Seepage

	Q=Kasin^{2}a
Kcore	0.000000023
a	7.32
Pie	3.14
a	0.79
sin^{2}a	0.50
q	8.42369E-08
L	324
Qtotal	2.72927E-05

…………………the maximum seepage inside core material.

Tabel;- phreatic line points

X	0	2.5	5	7.5	10	12.5	15	17.5	20	22.5
Y	4.663	6.712	8.269	9.575	10.724	11.761	12.713	13.599	14.431	15.217

X	25	27.36
Y	15.965	16.64