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Embankment Dam Safety Monitoring Through Seepage Analysis

(Case study Gilgel-Gibe I dam)

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Embankment Dam Safety Monitoring Through Seepage Analysis

(Case study: Gilgel Gibe I Hydropower Dam)

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Embankment Dam Safety Monitoring Through Seepage Analysis

(Case study: Gilgel Gibe I Hydropower Dam)

This is to certify that the thesis prepared by Abrehet Mekonnen, entitled: Embankment Dam Safety Monitoring Through Seepage Analysis for case of Gilgel Gibe I Hydropower Dam submitted in partial fulfillment of the degree of Masters of Science (Civil and Environmental Engineering (Major Hydraulic Engineering)) complies with the regulation of the university and meets the accepted standards with respect to originality and quality.

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Addis Ababa, August 2017

Abrehet Mekonnen

LIST OF ABBREVIATIONS AND SYMBOLS

2-D Two dimensional

AFRD Asphalt faced rock fill dam

CFRD Concrete faced rock fill dam

ETCOLD Ethiopian Commission on Large Dams

FEM Finite Element Method

Ha Hectare

ICOLD International Commission on Large Dams

K Hydraulic conductivity

LR Linear regression

MW Mega watt

PHC Potential Hazard Classification

R^2 *Coefficient of fitness*

RMSE Root mean square error

USACE United States Army Corps of Engineers

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ABSTRACT

Gilgel-Gibe I Dam is located in Sekoru woreda Jima zone of Oromia region. It is embankment asphalt faced rock fill dam. Although dams are constructed with great attention to carefully survey, design, and construction such as Gouhou dam in China, Baldwin Hills Dam in Los Angeles, Teton Dam in USBR have been failed and result serious accidents in the world. This study is intended to devise a methodology for embankment dam safety monitoring through seepage analysis of Gilgel-Gibe I Dam. Seepage was numerically performed using 2-D FEM steady state analysis to investigate the seepage in the dam body. Trial seepages were produced for different reservoir water level and trial hydraulic conductivity assigned to asphalt face. The trial seepages generated from seep/w software of Geo-studio 2012 package, and measured (recorded) seepage data are an input to linear regressions. Then, the LR coefficients were calculated by optimizing the seep/w results with the measured seepages using MATHLAB model. The calculated seepage concides with the measured seepage with RMSE, Mean error , mean absolute error being closer to zero while the r^2 *value being aproximately one. By combining these calibrated LR coefficient and seep/w seepage result predicted seepage is developed. From this by adding and subtracting RMSE, upper and lower limit is developed. The upper limit is checked against piping and slope stability. It is the critical value beyond which cause piping, instability and disaster. The curve obtained can be used to monitor the dam safety against seepage for future operation.*

Key words: Dam safety monitoring; Gilgel Gibe I dam, linear regression, SEEP/W

1. INTRODUCTION

1.1 General Back ground

Construction of dams across rivers for different purposes dates back to about 3000 B.C. in Egypt, and then into the Middle East cultures associated with the Euphrates and Tigris Rivers. The dam built at Sadd-el-Kafara, Egypt, around 2600 BC, is generally accepted as the oldest known dam of real significance (P.Novak, 2004). The choice of dam type can be limited by the availability of construction materials, the economic wise, but provided the difference in cost is small, the choice between a concrete and an embankment dam is often influenced by previous and local practice and preference of the designer. The dam engineer is required to synthesize design solutions, which, without compromise on safety, represent the optimal balance between technical, economic and environmental considerations (P.Novak, 2004). The two principal types of embankment dams are earth and rock-fill dams, depending on the predominant fill material used. Trial and error led to an early preference for masonry or quarried rock construction because the successful masonry dams required less material to remain stable and are not as susceptible to failure from uncontrolled seepage (TADS, 2017).

Dam construction in Ethiopia started in the late thirties. The first modern dam is Aba Samuel dam, constructed on the Akaki River, tributary of the Awash River and was commissioned in 1939. Now a day in Ethiopia the demands for irrigation is arising through the development and expansion of organized agriculture, to satisfy this demand many embankment dams are constructed and most of which are used for this purpose. Currently, there are more than 50 small and large dams in operation. Though previously the country was mainly involved in small-scale irrigation projects, that do not need sophisticated studies and techniques; recently there are large-scale irrigation projects such as Tendaho, Ribb, Arjo Dedessa, Kesem (ETCOLD, 2014).

In recent years' clay core rock fill dams (zoned dams) are being built in Ethiopia. The current design practice for most dams (Upper Guder, Tendaho, Middle Awash, Megech, Rib, Lower Awash and Kessem) are clay core rock- fill dam supported on the upstream and downstream sides by compacted rock fill material. On the other hand, asphalt core and asphalt faced rock fill dam supported either both side or downstream sides by compacted rock fill material has been built like Wolkayite and Gilgel Gibe I dam respectively.

The status of a hydraulic structure in the process of construction and in operation also changes, due to outer loads. Moreover, this kind of change is often hidden, slow, and not easy to detect. Accidents or loss can be avoided if we can get relevant information before the accident, analyze the information and make judgments, in order to take timely and effective measures.

This paper is intended to devise a methodology for Gilgel Gibe I Dam safety monitoring through Seepage analysis expectations for a given head in the reservoir by making use of seep/w and statistical approaches. The predicted monitoring seepage is checked against piping and slope stability. Such results will be used for early warning system, which is vital in monitoring the safety of an embankment dam.

1.2 Statement of Problem

Ethiopia is building a number of earth and rock fill dams. Even though dams are valuable resources, they become more expensive to repair if problems are not solved on time. A minor problem can turn into a major reconstruction project or even result in a complete dam failure. Most embankment dams have seepage through foundation within the dam body and abutment because of water moving through the embankment and foundation materials. Since most dams are constructed to serve at least the design period, dam safety monitoring is important. Although dams are constructed with great attention to carefully survey, design, and construction such as Gouhou dam in China, Baldwin Hills Dam in Los Angeles, Teton Dam in USBR have been failed and result serious accidents in the world (USACE, 2006). Therefore, all embankment dams in service should be systematically evaluated for their safe performance under all operational conditions. Seepage monitoring would be vital parameter in identifying the status of the dam safety, as seepage could happen due to different abnormalities of the dam. Besides, from the total number of dams the failed dam percentage for earth and rock-fill is 70% more than gravity dams (Bulletin 99, 1995).

Gibe I hydropower project is a cascade dam so the failure of this dam results in catastrophic damage for the subsequent or downstream dams and users. It is therefore important to monitor the behavior of Gibe I dam during subsequent operation of the reservoir to assess its safety on a continual basis.

Besides, assessing the safety of existing dam condition is important to serve the designed period and in order to decrease potential hazard. Thus, this research tries to define the safe seepage rate through the embankment of Gibe I. Once such safe seepage rate for a given water level in the reservoir is defined any seepage which will occur in the dam body shall comply with this seepage rate. If the observed seepage is beyond what have been proposed (upper limit) it signals that the dam is not safe. Such indicators will warrant further investigation into the causes of such observed seepage variations beyond the expected one. By doing so the safety of the dam situation could be identified and monitored. Thus, this research has tried to set a defined seepage quantity for a given water level based on historical seepage data. The study has selected Gilgel-Gibe I hydropower dam as a case study.

1.3 Objectives

General objective

The overall objective of this study is to devise a methodology for embankment dam safety monitoring through seepage analysis of Gilgel Gibe I dam.

Specific objective

- To define the normal seepage quantity and rate for a given reservoir water level
- To develop dam safety monitoring curve
- To assess the current state of dam safety monitoring of Gilgel Gibe I dam

1.4 Scope of the study

Seepage verses reservoir water level for different hydrogeological parameters were used to predict safe seepage for Gilgel Gibe I dam. The predicted seepage is checked against piping and slope stability. Efforts were made to include foundation seepage analysis but seepage verse reservoir water level data is available on the asphalt facing only, the scope of the research has been limited to determine the safe seepage of the super structure.

1.5 Research question

- Why existing embankment dams require safety monitoring?
- Could a monitoring curve be developed for existing embankment dam?

2. LITERATURE REVIEW

No dam can be considered one hundred percent safe as there will never be a complete understanding of the uncertainties associated with natural and manmade destructive forces, material behavior and construction processes. A dam, which is safe at the time of completion, does not automatically remain safe. A few dam owners may believe that a dam, which was safe at the time of its completion, will always remain safe. Life-span of a dam is as long as it is safe, i.e. as long as proper monitoring can be guaranteed (ETCOLD, 2014).

Dams are constructed for various purposes like flood control, navigation, water supply, recreation, power generation etc. Earth dams have always been associated with seepage as they impound water in it. The water seeks paths of least resistance through the dam and its foundation. Seepage becomes a safety problem when embankment or foundation materials are moved by the water flow, or when excessive water pressure builds up in the dam or its foundation (Ali Beheshti, 2013). In the construction of a rock-fill dam, a wide range of materials can be used from sound, free draining rock to the more friable materials such as sandstone and silt shale materials. The friable materials are better for filling the gaps to provide better compaction but since the shear strength of these materials are not as high as sound rock fill; the stability design of the slope should be studied carefully. Seepage through the foundation and dam body must be controlled and collected to ensure safe operation.

2.1 Gilgel Gibe I dam

Gilgel Gibe I dam is commissioned in January 2004 GC. The Gilge Gibe (little gibe) crossing the region from south west to north east is a tributary of the great gibe (known as the omo river further downstream of the confluence) and is extremely variable in course and gradient. The main objective of this project is to generate 184MW hydroelectric power by three Francis turbines installed in an underground powerhouse. The height of dam from the general foundation level is 40m, which created a reservoir of 63 square kilometers; dam crest length of 1700m, dam crest width of 7m, and the crest elevation is 1675 m (a.m. s. l). And the deepest foundation point is approximately at elevation of 1635 m (a.m. s. l). It is basaltic rock-fill embankment with upstream bituminous facing.

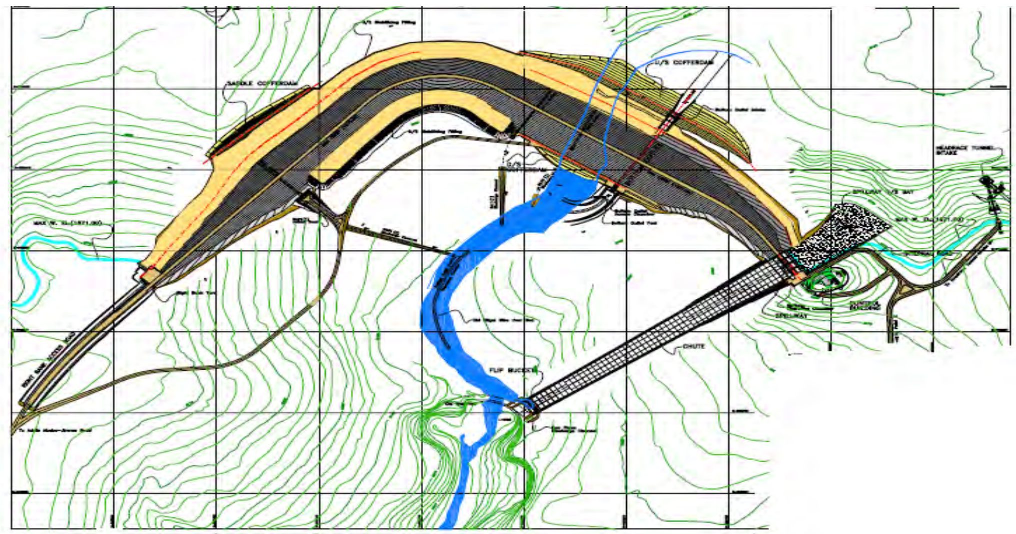

Figure 2. 1: General layout of Gilgel Gibe I dam

Various types of instruments such as electromagnetic settlement gage, inclinometer, displacement sensors, dam foundation pore water pressure, topographical measurement, water level probe etc. were installed throughout the dam at different elevations and section.

2.2 Potential Hazard Classification (PHC)

In order to give more attention how much the dam failure results in damage the potential hazard classification know how is important. The French Committee on Dams and Reservoirs has developed a classification system for dams with two of the main parameters usually used in the determination of a dam break flood, height and storage volume of dam. These two parameters are combined as $H^2.\sqrt{V}$ with H = maximum height of dam walls in meters, measured from riverbed level and V =storage volume of reservoir at full supply level in million cubic meters. This combined parameter is used for the classification into low $(H^2.\sqrt{V} < 20)$, medium $(H^2.\sqrt{V} < 200)$ and high as $(H^2.\sqrt{V} \geq 200)$ potential hazard classes (Small Dam, 2011). According to this Gilgel Gibe I, with height 40m and storage volume 839million m^3 is classified as high PHC**.**

2.3 Couse of failures

Due to the stored water and seepage through dam body and foundation condition, there are problems to the dam, its foundation as well as impounded water. Among this:

(a) Foundation Deficiencies. -These defects are associated with the quality of the foundation or with the foundation treatment. Differential settlements, slides, excessive pressures, weak seams or zones, and inadequate control of seepage are all common potential failure mechanisms within a foundation. Visible cracks in a dam can be indicative of foundation movement. Marginal foundation stability can sometimes be identified by a thorough examination of design and construction records (USBR, 1995).

Regional subsidence caused by the extraction of ground water or hydrocarbons can cause settling of the foundation and cracking of the dam. Settlement of the dam and foundation and the resulting cracking can also occur from the collapse of foundation soils caused by loading subsequent wetting of the foundation materials.

This collapse of foundation soils can occur in fine sands and silts with low densities and low natural moisture contents. The settling and subsequent cracking of embankment materials can be especially disastrous if the embankment contains soils which readily crack when deformed. Seepage through the foundation can cause piping of solid materials or the erosion of soluble materials by solutioning. This removal of foundation material forms voids which can increase until a portion of the remaining unsupported material collapses and failure of a section of the foundation occurs (USBR, 1995)**.**

(b) Inadequate Seepage Control Seepage problems can occur in either concrete or embankment dams as well as through or along the foundations. Uncontrolled seepage through an embankment dam can cause the movement of soil to unprotected exits, creating voids, and leading to “piping” failures. Improper compaction; differential settlements; pervious embankment materials; or the presence of ice lenses, roots, stumps, or debris in an embankment resulting from inadequate construction control can cause excessive seepage through the embankment.

Uncontrolled seepage can result in excessive pore pressures in an embankment or foundation. This can cause a weakening of the soil mass and can result in springs, sand boils, abutment failures, and upstream or downstream slope failures. Excessive pore pressures can be caused during construction by placing embankment material too rapidly or by placing material which is too wet, by percolation of water through areas of pervious material in the embankment or along joints in the foundation which are connected to the reservoir, or by a rapid drawdown of the reservoir. Settlement cracks caused by a compressible material in the embankment or foundation can also provide seepage paths (USBR, 1995).

(c) Reservoir Margin Defects -Events or conditions occurring within a reservoir basin that could lead to or indicate possible catastrophic failure includes landslides, active faulting, seiches (seismic or landslide induced waves), shoreline erosion, and reservoir failure due to piping.

All reservoirs leak to some extent; however, recognizing and evaluating conditions that could lead to increased seepage are critical. Perviousness is a primary concern in any reservoir located in unconsolidated material and many sedimentary rocks. Specific items to be considered as percolation routes are buried channels, fault zones, joints, solution channels, and other forms of primary and secondary permeability. Indicators of excessive permeability are observed leakage, unexpected ground-water fluctuations, water boils, unexplained reservoir losses, and new springs (USBR, 1995). The main source of seepage within asphalt concrete facing is cracks in the asphalt concrete facing in the structure. Formed drains installed in the dam are designed to intercept the seepage and reduce the pressures which could develop along lifts or cracks.

2.4 World experience on Dam Failures

The failure of dam throughout the world had result in catastrophic damage of life, structure and crops. This could result from poor initial design or construction, lack of maintenance and repair, or the gradual weakening of the dam through the normal aging processes. That it is necessary to have a monitoring and surveillance system established, is highlighted by number of dams which experience accidents and failures, often after many years of operation. As an example, dams failed due to seepage problem are listed below (USACE, 2006).

Gouhou Dam

On August 27, 1993, a Concrete Faced Dam failed in China (Gouhou Dam) one day after the reservoir level reached the top of the slab. The failure was due to failure of the gravel shell. Although many CFRD's have been built from freely draining rock fill and clean gravels with no stability problems with slopes ranging from 1.3 to 1.6: 1, this particular dam had a 1.5: 1 downstream slope, but it was constructed with sandy gravels with about 40% of the particles finer than 5 mm. With the leakage through the face and perimeter joint, the dirty shell materials were not pervious enough to conduct the flow at low gradients and a phreatic surface raised high enough in the shell that the normal CFRD slopes could not be maintained, and the dam failed (USACE, 2006).

Baldwin Hills Dam

Baldwin Hills Dam in the Los Angeles area was a 71m high homogeneous earth-fill embankment dam has been constructed on April 18, 1951. The impervious member was a 1.5m thick compacted earth lining which was constructed on an asphaltic membrane. The dam failed by piping on December 14, 1963. Although there was a pea gravel and clay tile drainage system under the bottom of the reservoir, there was not a drain or filter system between the upstream slope of the embankment and the downstream slope of the homogeneous embankment. It is possible that the distortions of the embankment due to the differential settlements in the area due to oil extraction was a factor in cracking the lining which resulted in uncontrolled seepage downstream of the lining on the upstream slope of the embankment because there was no downstream drain or filter zones in the embankment (USACE, 2006).

Walter Bouldin Dam

In February, 1975, the Walter Bouldin Dam in Alabama failed. The 50.1m high embankment dam just to the left of the powerhouse breached. This location is where cretaceous fine sandy silts could have piped undetected into the tailrace channel from seepage lines in the foundation of the left embankment dam, as there was no cutoff to bedrock beneath the left embankment dam immediately adjacent to the left side of the tailrace channel. These seepage lines were not filtered in the design and could have exited into the tailrace channel below water level where the piping would have been uninspected (USACE, 2006).

Teton Dam

On June 5, 1976, the 126 m high Teton Dam of the USBR failed during first filling, which had been initiated in October 1975. The failure of Teton Dam was a clear Case of piping because the silt core in the rock cutoff trench in the right abutment was directly placed against open jointed rhyolite without a filter between the silt and the jointed rock (USACE, 2006).

2.5 Dam monitoring

No dam can be considered one hundred percent safe as there will never be a complete understanding of the uncertainties associated with natural and manmade destructive forces, material behavior and construction processes. This process consists of the collection, recording, analysis and presentation of data from measuring devices installed at or near dams. The instruments should monitor the key performance indicators that provide early warning of the development of the identified potential failure modes.

2.5.1 International dam monitoring

The current condition of the visible features at the dam is determined by an onsite examination or instrument reading. The dam, appurtenant structures, and mechanical equipment should be examined to determine if they are performing as expected. Regions of distress, unexpected movements, unusual seepage or leakage, mechanical and electrical equipment malfunctions, and all other observations related to the safety of the dam should be identified and recorded. The results of the instrumentation observations and analyses may reveal or forecast dangerous conditions.

For existing dam's temperature measurement is used to provide valuable information about seepage flow and ongoing internal erosion in embankment dams. It is common practice in Germany and Sweden. Since, it is very easy to measure and can give useful information about the presence of anomalous flows being transported from different regions of a dam (Johansson, 2012).

In 1965, a concrete dam in Catagunya stressed dam was monitored for deflection. The principal load to concrete dam is water load and temperature. An observed value of temperature and reservoir water level was used as input for multiple linear regressions to predict the deflection. In certain circumstance, it may be possible to establish the regression equation by calculation of deflections for various combinations of temperatures and water load. On continuing plot of deflection against time, values from the regression equation can be compared with observed values, any significant difference can result in detailed investigation (Thomas, 1976).

The common indicators of deterioration of an embankment dam are deformation and seepage flows. These can be identified by visual surveillance or by instrumentation.
Seepage flows

- Sudden emergence of seepage or leakage on the downstream slope, valley sides or valley bottom
- Increase in flow rate or turbidity of existing seepage flows
- Marked change in piezometric level within dam or foundation
- High pore pressure downstream of the water tight element

2.5.2 Dam monitoring in Ethiopia

A guideline called ETCOLD has been established in 2014 due to;

- Ageing of dams
- More development and expansion of urbanization downstream of dams, which were not existing during the construction period,
- More water storage dams (large and complex) are being planned constructed and operated, which need state of the art design, construction and operational safety guideline

- There had not been any dam safety related guidelines, standard or regulation to guide designers, contractors, dam owner, operators and decision makers, and other reasons.

According to the guideline increased seepage or turbidity could indicate piping of embankment dam.

Dam monitoring in Ethiopian situation, there are devices which are installed to measure a particular parameter of interest for monitoring of dam besides the visual inspections. Dam monitoring is one means of determining trends in structural performance.

A Threshold value is used in the analysis or design, or is established from the historic record. An Action Level is the instrument reading that triggers increased surveillance or an emergency action. Threshold and Action limits should be established based on theoretical or analytical studies (e.g. uplift pressure readings above which stability guidelines are no longer met) or on measured behavior (e.g. seepage from an embankment dam) (TADS, 2017).

Some of the items to be monitored include:

- Reservoir water levels, which provide a record of the loadings on the dam;
- Seepage, which may be measured at any point on the dam, abutments, or reservoir rim or even well downstream of the dam, and is probably the best indicator of dam's performance;
- Rainfall (at dam and in catchment), which may relate to the amount of seepage;
- Pore-water pressures and water table levels, which, may be related to seepage, reservoir level and rainfall (TADS, 2017).

2.5.3 Instrumentation and Monitoring of Dams and Reservoirs

For the safety and normal operation of a dam, precise information is required from instrumentation and monitoring of dam's body, the surrounding foundations, the reservoir and the river basin. Instrumentation and monitoring of dam bodies serves two purposes: assessment of dam safety, and improvement of design procedures and practice for future dams.

Specific reasons for instrumentation include:

- Warning of a problem
- Analyzing and defining a problem
- Evaluating remedial action performance
- Proving behavior is as expected

The parameters to be measured, and the appropriate instruments, are as follows;

- Leakage or seepage losses: drainage holes, V-notch weirs.
- Deformation (concrete dam): plumb lines, external targets.
- Deformation (embankment dam): differential settlement gauges, external targets.
- Uplift pressure: Borden tubes, piezometers.
- Pore pressure: piezometers.
- Earthquake motion: strong motion seismographs (Hirose, 2017).

A. Longitudinal inspection galleries

For visual surveillance of the behavior of the dam's interior (seepage, occurrence of cracks), as well as for installation of various measuring instruments, there are constructed special horizontal or inclined galleries, as well as vertical manholes. The galleries have minimum dimensions of 2×1.2 m. They are usually 2–3m wide and over 3m high. Sometimes, the lowermost inspection gallery can also serve for the execution of a grout curtain below the dam. The entrance into the horizontal galleries is either through manholes (inclined or vertical) from the crest, or directly from the banks like in case of Gibe I dam. As well as with an appropriate access, the gallery must also be provided with devices for ventilation and lighting (Tanchev, 2014). Drainage is constructed in the dam's body to collect all the seepage from dam body and foundation, to dispose to downstream through transversal galleries. The entire longitudinal inspection gallery of Gibe I dam has been covered, from the right bank access to the spillway control chamber in the left bank. The gallery has 60 sections from which from section 60 to section 25 the largest portion of the foundation is treated with plastic diaphragm, and the rest from section 25 to section 1 is founded on curtain grout treated foundation.

There are three transversal galleries which originate from the longitudinal gallery and cross the dam from upstream to downstream.

Transversal Gallery 51

This gallery is located on the right abutment in the depressed area of the saddle cofferdam where black-cotton soil has appeared in the foundation. It gathers all the dam drainages between the curve vertex of the dam and the right yard. The V-notch weirs are located at the end of the gallery, in the access chamber, for the measurement of the infiltrations originating from the dam stretch between sections 50 and 63 as well as those comprising among sections 50 and 34-38

Transversal Gallery 11

The function of this gallery is to discharge the leakages drained from the right abutment in the stretch between sections 25 and 34-38

Transversal Gallery 10

The function of this gallery is to discharge the leakages drained from the central area of the dam and, in particular, from those originating between sections 16 – 21 and 21-25. Substantially this is the river-bed area and thus the area in which the infiltrations are intercepted by the grouting curtain, and not the plastic diaphragm.

B. Pore pressure measurement in foundation

The measurement of pore pressure in the embankment and foundation of a dam can give vital quantitative information for use in assessing stability, Potential heave conditions in foundation and for identifying unusual seepage pressure. The correlation between observed trend of seepage and piezometric pressure is useful. When both are decreasing, it is safe situation otherwise monitoring is mandatory (ICOLD, 2000)**.** For this reason, in Gibe I dam the pore pressure transducers were installed on the foundations of the right embankment of the dam at a variable depth of between 1.20 and 2.60m.

The 10 measurement sections contain 4 piezometers each, for a total of 40 installed piezometers were installed in the surface layer of the foundation upper tuff to monitor pore pressure trends during construction and operation.

2.6 Seepage control method

A. Dam body

Embankment zoning normally includes a water barrier zone Core or upstream impervious facing protected by filtered drainage zones and supported by stability zones.

Asphalt facing

Asphaltic concrete facings also found a wide application in the world practice of construction of rock-fill dams, as watertight elements. The main advantages of asphaltic concrete facings are as follows:

- Asphaltic concrete produced for application in hydraulic engineering, is almost completely watertight;
- From a structural point of view, it is very simple because it is constructed as a monolithic unit, without joints;
- Asphaltic concrete linings, i.e. facings, have plastic properties and so are capable of following the deformations of the slope, without the occurrence of significant cracks; besides, there has been proved the ability of asphaltic concrete of self-healing (self-repair) under the effects of water pressure. Mechanization of the entire construction process considerably facilitates the decrease in workforce and material resources, as well as time necessary for construction (Tanchev, 2014)

Filter

The main function of the filter is to prevent wash out of the fines from the core or weak foundation but it also acts as a drain for the seepage water. Filters are used to prevent the intergranular water that percolates through embankment dams and their foundations from moving soil particles, while at the same time allowing the water to escape without building up excessive pore water pressures. The seepage water is often collected into a drainage system on the downstream side. A proper filter design and filter construction are fundamental for the safety of the dam (TADS, 2017). For Gibe, I the filter is designed between embankment and the weathered material of the foundations (Upper tuff) above the paleo channel or even in weathered basalt. The foundation surface has been treated in order to protect the surface layer creating a sort of “interlocking” between the “Upper Tuff” and the Fine Filter

B. Foundation

Foundation and abutment seepage reduction methods are designed to control seepage through the foundation and abutments of a dam. They may be used in conjunction with embankment seepage reduction methods. Foundation and abutment seepage reduction methods include (TADS, 2017):

- Grouting curtain; Grouting of foundation is used to reduce seepage flow through foundation but their effectiveness depends on geologic conditions. It is usually performed from the crest of the dam especially constructed galleries within the embankment close to the contact with foundation, or from adits driven the abutments. The depth of grouting may range from 30 to 40 percent of the head of the water on good foundation and to 70 percent of head on poor foundations (Fell, 2005) for Gibe I dam a plastic diaphragm wall 25m long was placed in most erosive zone. Their connection to bedrock was sealed cement grout. The total length of plastic diaphragm covers 1120 meter from the total dam length
- Cutoffs
- Upstream impervious blankets
- Downstream seepage berms

2.7 Slope stability

Stability problems in embankment dams are almost always preceded or accompanied by seepage problems. It is therefore essential to understand the seepage occurring through the dam and its foundation prior to doing stability analysis. Pore-water pressure and seepage measurements are the best indicators of dam safety condition (USBR, 1995).

2.7 Estimation of seepage

A. Numerical model method; for seepage analysis discretize the continuous flow medium into manageable units and then reunite the discrete parts through the use of continuity equations and describe the internal mechanism of flow by the Darcy equation. Computerized numerical methods are recommended because of;

- Complex systems can be modeled easily
- No transformation of dimensions
- Parameter variation is much more expedient
- Results are printed digital form for each node

The computer program is 2D SEEP/W finite element program

Much computer software has come in general use, and any hard computations and simulation can be carried out through them by giving them appropriate inputs and data. SEEP/W is a finite element which is capable enough to solve steady stead vertical section modes of the embankment and or foundation. The numerical model SEEP/W can be employed to carry out simulation of seepage and Phreatic surface. The SEEP/W program is capable enough to simulate quite effectively seepage rates and phreatic surfaces in homogenous and non-homogenous dams, groundwater flow, seepage and excessive pore water pressure problems within the porous media such as soil and rock (Krah, 2012).

Steady state seepage

Seep/W governed by Darcy's law, laminar flow is assumed and satisfy Laplace equation. The steady state analysis of Seep/W assumes that the water inflow is equal to the water outflow. During the analysis of rapid drawdown stability, transient seepage, in which the inflow is not equal to outflow, is used. Darcy's law is the basic premise upon which almost all seepage analysis is based. In its simplest equation

$$q = kiA \qquad equation\ 2.1$$

Where q is the flow rate, k is the hydraulic conductivity, i is hydraulic gradient and A is the cross-sectional area, can be used when a gross estimate of flow is needed, or when a back calculation of average hydraulic conductivity is needed. The general governing differential equation for two-dimensional seepage under steady-state conditions can be expressed as (Krah, 2012).

$$(\partial^2 H / \partial x^2) + (\partial^2 H / \partial y^2) = 0 \qquad \text{equation } 2.2$$

where:

H= the total head

Hydraulic conductivity

In unsaturated soil, the difference between the air and the water pressure, the matric suction varies the volume of water stored in the soil. To account for this the sample function in the software are used by inserting the saturated water content for the materials. Saturated water content is almost equal to the porosity of the soil. There are theoretical models that relate the soil water retention and hydraulic conductivity characteristic curves. The van Genuchten uses the Mualem theory and provide for a smoother representation of the inflection point in the characteristic curve near saturation. Hydraulic conductivity of materials is also defined with respect to the matric suction. Having the boundary condition of the static water pressure up to the normal water level in the upstream side, zero pressure water is expected at the downstream of the dam (Krah, 2012).

Volumetric Water Content

As water flows through soil, certain amounts of water are stored or retained within the soil structure, being these functions of the pore-water pressure and the characteristics of the soil structure. Since it can sometimes be difficult or time consuming to obtain a volumetric water content function, it may be of benefit to be able to develop an estimation of the volumetric water content function using either a closed-form solution that requires user-specified curve-fitting parameters, or to use a predictive method that uses a measured grain-size distribution curve. SEEP/W has four methods available to develop a volumetric water content function. One is to estimate a data point function using a predictive method based on grain size, one is to base your function of a sample set of functions built into the software, and two are closed form equations based on known curve fit parameters.

B. Graphical flow net construction Flow nets are one of the most useful and accepted methods for solving the Laplace equation. If boundary conditions and geometry of a flow region are known and can be displayed two dimensionally, a flow net can provide a strong visual sense of what is happening (pressures and flow quantities) in the flow region. A flow net is two sets of orthogonal (intersecting at right angles) curves. One set of curves represents flow paths (flowlines) through the porous media, while curves at right angles to the flow paths show the location of points within the porous media that have the same piezometric head (equipotential lines) (TADS, 2017).

2.8 Dam safety analysis/assessment

The purpose of all dam safety analysis/assessments for the whole life cycle of a dam (planning, design, operation, etc.) is to determine the capability of the dam system to retain the stored volume under all conditions and to pass flows around and through the dam in a safe, controlled manner. Dam safety analysis should consider the full range of applicable conditions in order to determine how the structures are expected to perform and what amount of deviation from the normal condition is tolerable. Design, construction, and operation should be integrated in the analysis to ensure that the design intent has been incorporated into the dam (TADS, 2017).

The surveillance and monitoring program should provide regular monitoring of dam performance, as follows:

- Compare actual and design performance to identify deviations;
- Detect changes in performance or the development of hazardous conditions;
- Confirm that reservoir operations are in compliance with dam safety requirements; and
- Confirm that adequate maintenance is being carried out.

3. METHODOLOGY

3.1 General Description of the case study dam

Gilgel-Gibe I Dam is located in Sekoru woreda Jima zone of Oromia region. The ground distance is 260 km SW of Addis Ababa and 70km NE of Jima (Fig 3.1). Geographically the area is located between 7°20'N, 8°14'22''N latitude and 6°31'05'', 37°28'36''E longitude. It is flat plateau about 1650 m above mean sea level (a.m. s. l) consisting of a series of gently sloping low hills and broad plains surrounded by hills or mountains.

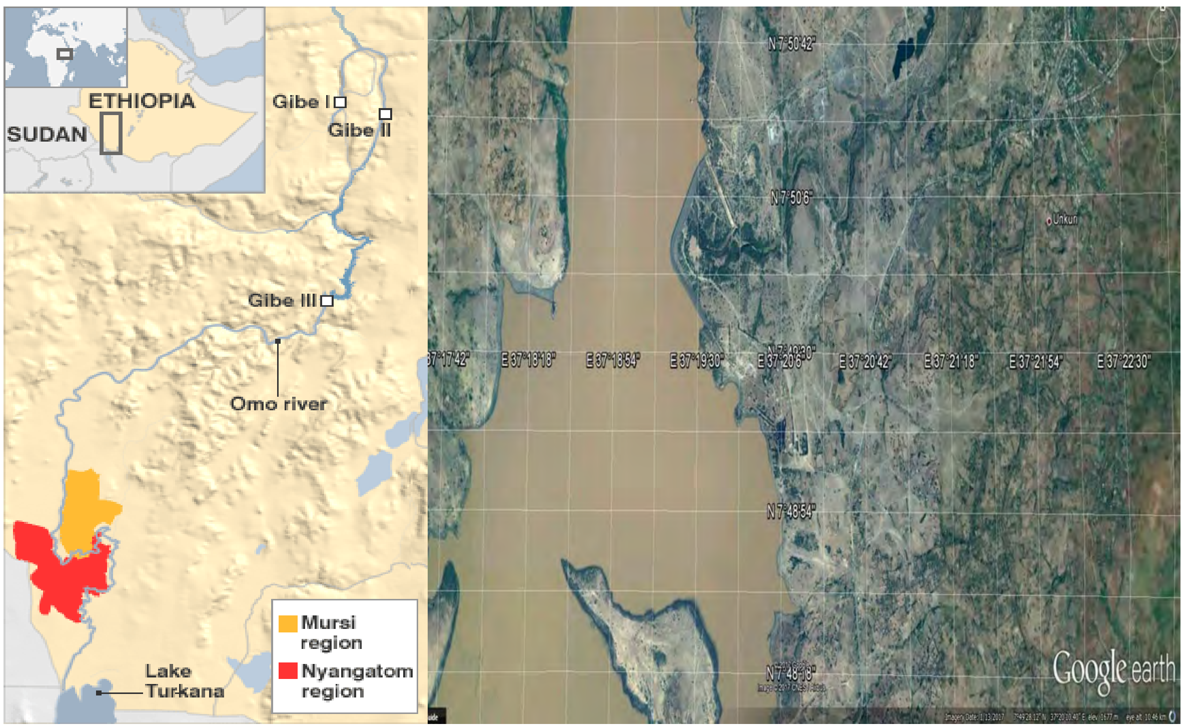

Figure 3.1: location of Gilgel gibe I dam

3.2 Conceptual modeling

In the process of analyzing the dam safety based on seepage, the first step was collecting the measured seepage data verses elevation corresponding to date with supposition of that the dam is safe. It is because the seepage losses from the bituminous lining are minimal than the calculated value in the design document (i.e.: 1.74×10^{-5} m^3/s) and therefore acceptable. Besides there is no visually observable defects like excessive seepage or turbidity at the dam

The dam is examined for steady-state seepage conditions, since it is stored water the dam body is saturated so there is no loss or addition of water inside the dam body. The first step is calibrating the model (seep/w) by taking a single reservoir water level for four trial hydraulic conductivity values of the asphalt face (upstream impervious face) to generate trial seepage. Therefore, the model is calibrated for under and over estimation of seepage, the process is repeated to generate different trial seepages. The number of modeling depends on the number of measured data and trial hydraulic conductivity. Then, each trial generated from seep/w, and measured (recorded) seepage data are an input to multiple linear regressions. Finally, the LR coefficients were calculated by optimizing using MATHLAB model until minimum error found. Then by adding and subtracting RMSE from the predicted seepage upper and lower boundary is developed. With this seepage whether the dam is safe against piping and slope stability was checked. So, the dam safety was defined and monitoring curve was developed from the predicted seepage corresponding to reservoir water level. (See the flow chart figure below)

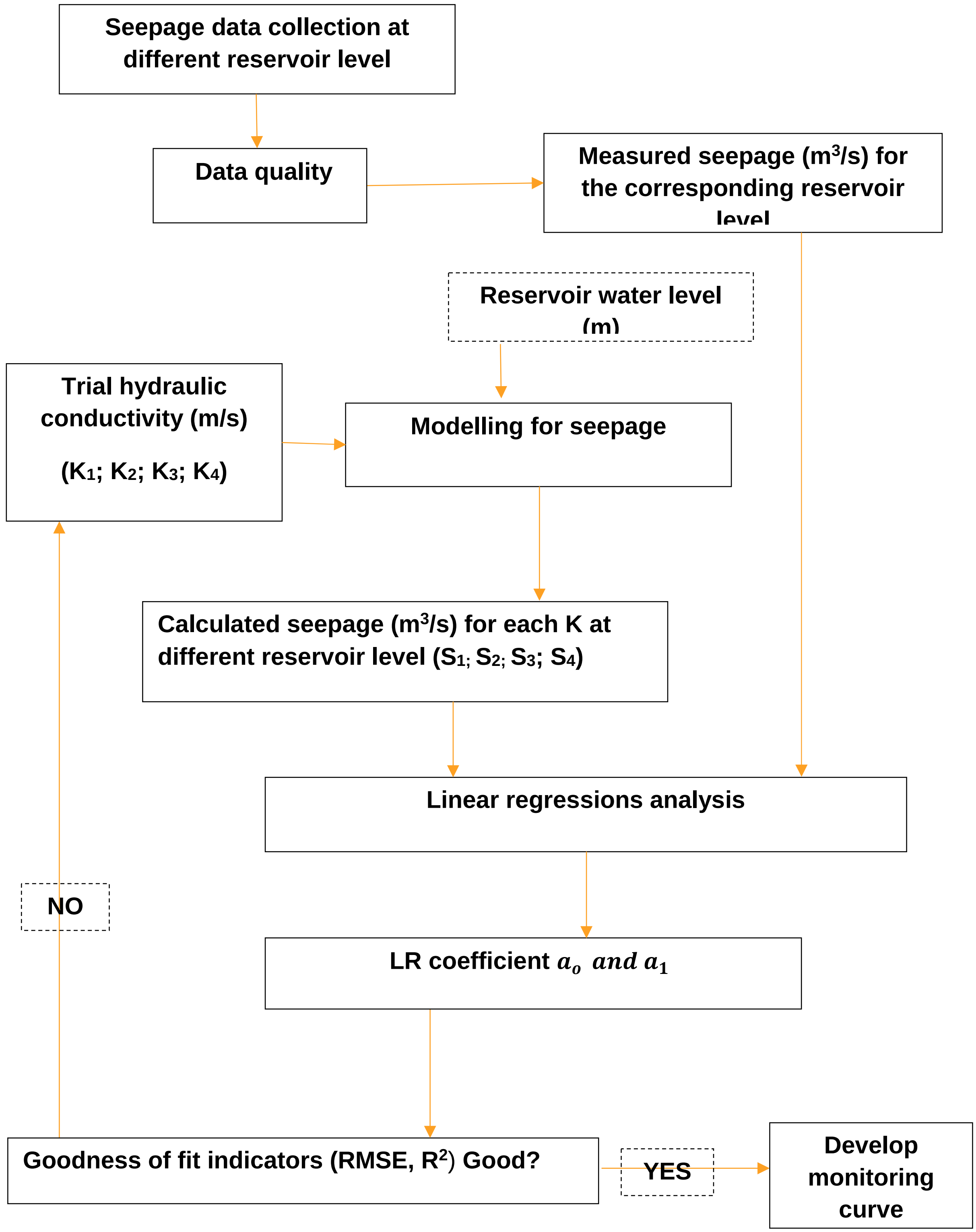

Figure 3. 2: modeling flow chart

3.3 Data collection

The following data were used as an input for steady state analysis of seep/w model

- Measured seepage
- Reservoir water level
- Hydrogeological parameters of material (conductivity)
- Construction drawing
- Material property of the dam and foundation

3.3.1 Primary data

The primary data are collected by the researcher for the first time and it is unique. It was collected from Gilgel Gibe hydropower project site office to cross check the secondary data. Valuable information needed to assess the dam and foundation seepage was gathered through

- Interviewing the resident engineer

Figure 3. 3: Primary data collection

- Physical observation of the dam site, gallery, V-notch and drain hole location
- Photos that show reservoir water level, dam and appurtenant condition have been collected with digital camera.

3.3.2 Secondary data

This are the data that have been collected and recorded by the project site and Ethiopian electric power head office who supervise the Gilgel-Gibe I hydroelectric project. The raw data on seepage water, reservoir water level, gallery and drain hole location and design drawings are obtained. All necessary information is gathered for using numerical modeling of 2D finite element method analysis of the cross sections of dams is listed as the following items:

- ✓ Data on measured seepage discharges and the corresponding reservoir level;
- ✓ As built drawings;
- ✓ Design analyses and reports, and construction plans.

3.4 Data quality

Data quality is the condition of a set of values that fit for the intended use in operation, decision making, and planning. Problems with data quality do not only arise from incorrect data, but inconsistent data is a problem as well. The basic criterion in selection of a case study is the reservoir has to pound maximum reservoir level the dam is in operation. Therefore, the trend of measured seepage data verses reservoir water level and reservoir water level verses date was observed using graph for different drain hole of the gallery block. From the measured seepage data, there were frequently and slightly dripping drain holes which are difficult to be measured. This is because of low permeability of asphalt face. There is fluctuation in reservoir level because of seasonal and operational variation. That is during summer (rainy season) there is rise in water level but during dry season there is decline in water level due to water released for hydropower generation and evaporation. As reservoir water level increase the measured seepage shall increase any trend that does not respect such reality was ignored. Area without continuous reading or no reading of seepage corresponding to reservoir level and date were not considered for analysis.

3.5 Measured Seepage

For reducing the seepage, uplift and the mechanical and physical-chemical effect of the seepage water, gallery is constructed in the dam's body. The gallery has 60 sections. Based on the measured seepage collected from the office the largest leakages are located among sections 21 and 17 (between gallery Ga10 and the bottom outlet), as foreseeable, being this the riverbed area and thus the area with major hydraulic head. Gallery ten is located near to bottom outlet toward the left bank covers from chainage 524.005m to 624.182m which is approximately one hundred-meter (100.177m) length. It is located in the lowest elevation at 1634m above mean sea level relative to other galleries. It has five blocks (10.1, 10.2, 10.3, 10.4, and 10.5) and each block have two drain holes spaced at ten meter mutually. Therefore, as typical section for this analysis section 21 chainage 550m part for Gallery ten (Ga10) is selected. And this typical section is selected because of;

- Sufficient recorded seepage is collected at this area
- This is the area of riverbed and
- The major hydraulic head is obtained here

From this section, Block number 10.4 was the target area. See the figure below for the location of the galleries and sections.

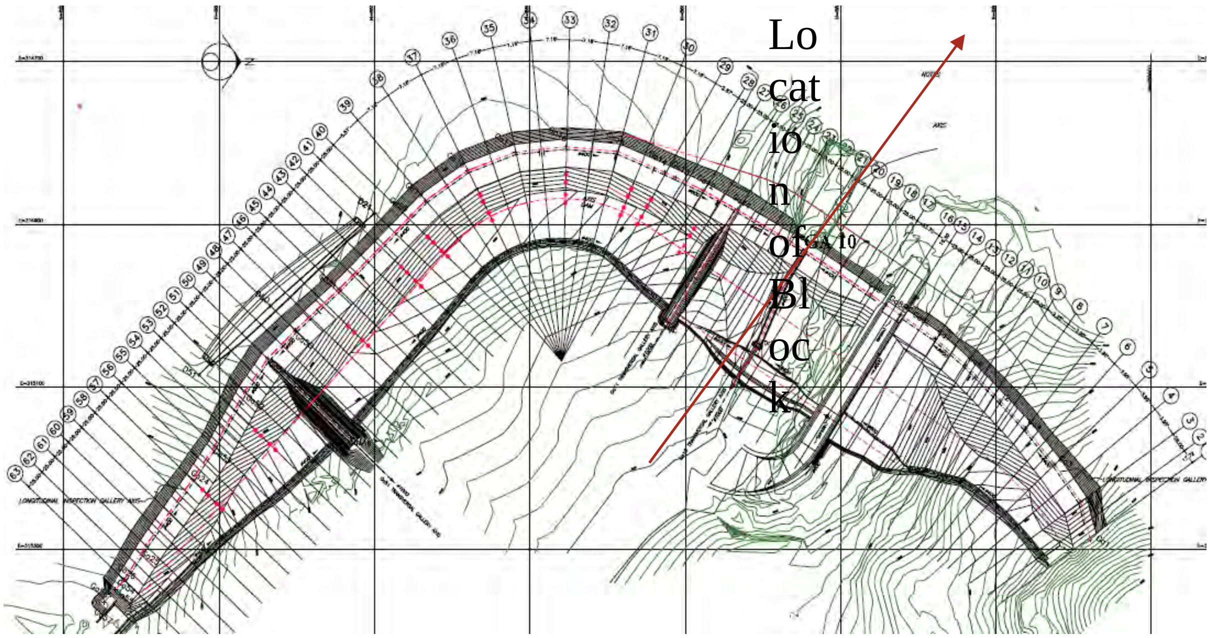

Figure 3.4: Location of galleries

Since the asphalt face is highly impermeable the collected seepage data trend was increased slowly corresponding with reservoir water level increment. The seepage collected is through the asphalt facing only. A representative monitored seepage data was taken for a repeated seepage reading at single reservoir level on different dates. Table 3.1 shows the measured seepage as well as the reservoir water level.

Table 3.1: measured seepage @ block 10.4

RWL(M)	1666	1666.5	1669	1669.5	1670	1670.5	1671
Measured seepage(m^3/s)	$8x10^{-6}$	$8.5x10^{-6}$	$1.1x10^{-5}$	$1.15x10^{-5}$	$1.2x10^{-5}$	$1.25x10^{-5}$	$1.3x10^{-5}$

3.6 Modeling for seepage and slope stability through the embankment dam

In this research, a powerful program of Seep/w and Slope/w software program were used, which are among from GeoStudio packages.

There are a number of software products in the world. Well known software products dealing with seepage problems are the software packages SEEP/W (GEO-SLOPE, Canada), DIANA (TNO Company, The Netherlands), SV Flux (Soil Vision Systems, Canada), (Tanchev, 2014). Seep/w software is selected among others since it works based on discretization of the continue flow medium into manageable units and then reuniting or meshing the discrete parts through the use of continuity equations. Its comprehensive formulation enables to consider analyses ranging from simple, saturated steady-state problems to sophisticated, saturated / unsaturated and time-dependent problems. It has the option of computing seepage forces and has the capability of modeling axisymmetric radial flow problems (Krah, 2012).

3.6.1 Seep/w

Seep/w is a numerical model that can mathematically simulate the real physical process of water flowing through a particulate medium. It is based on finite element computer enabled software which is capable enough to solve groundwater flow, seepage and excessive pore water pressure problems within the porous media such as soil and rock. Since SEEP/W gives more precise result when the elements are fined, mesh size was approximately 4m wide and quad-triangular type of element for the analysis. The material properties for each zone with proper geometry are made as input to the software respectively and verification for each region has been made accordingly.

3.6.2. Slope/w

SLOPE/W is the leading slope stability software product for computing the factor of safety of earth and rock slopes. SLOPE/W can effectively analyze both simple and complex problems for a variety of slip surface shapes, pore-water pressure conditions, soil properties, analysis methods and loading conditions (SLOPE/W, 2012).

3.7. Seepage Analysis using seep/w model

Seep/w is governed by Darcy's law and satisfies Laplace equation. In this research, the inputs for seep/w model are reservoir water level and hydraulic conductivity. Changes in reservoir water level and hydraulic conductivity have an effect on magnitude of seepage and pore water pressure. To model all materials in the dam body of this typical section is assigned with respective hydraulic, volumetric and other parameters of the soil property. The cross section is discretized into fine elements and connected at nodes. The software calculates the seepage and pore water at nodes.

$$q = kiA \qquad equation\ 3.1$$

Where q is the flow rate, k is the hydraulic conductivity, i is hydraulic gradient and A is the cross-sectional area, can be used when a gross estimate of flow is needed, or when a back calculation of average hydraulic conductivity is needed. Laplace expressed as.

$$(\partial^2 H / \partial x^2) + (\partial^2 H / \partial y^2) = 0 \qquad equation\ 3.2$$

where:

H= the total head

Seepage analysis was performed with one distinct situation of dam operation that is steady state flow and with crack. Different reservoir water levels were considered. The dam body and its foundation have ten types of materials and it is defined as "saturated/unsaturated" material model. Regarding the hydraulic parameters, it should be inserted by using functions of volumetric water content variation and permeability variation.

Asphaltic concrete is a thermoplastic material; that is to say, it changes its properties with changes in temperature, so that at high temperatures it becomes soft, while at low temperatures it becomes rigid. And when it is exposed to light and ultraviolet rays, has the property of ageing, i.e. it loses certain properties in the course of time (Tanchev, 2014). Due to this, crack may occur at most frequently fluctuating reservoir water level. For this paper at reservoir water level 1666m crack is assumed in order to have conservative seepage. It is assigned as boundary condition to the asphalt face in order to analyze the predicted seepage against piping and pore water pressure.

1. Volumetric water content

For the presented study, the function of volumetric water content was determined as "Data point function". This function is estimated based on a "sample function" which is in this case, different sample materials. Posterior, it is required to insert the initial condition of water volume value in "Saturated WC" was set based on porosity of the material. The coefficient of compressibility is the load of water on soil when saturated. It is 10^{-3} 1/k pa to 10^{-4} 1/k pa for clay soil and for other type of soil ranges up to 10^{-6} 1/k pa (Dr.A.R.Arora, 2004). The residual soil moisture content which the soil retains under high pressure is quite small amount of soil moisture content between 5 to 12% of volumetric moisture content (Jose_Navar2, 2016). It is different for all material models since it depends based on volumetric water content or porosity.

2. Hydraulic conductivity

Permeability function is also a "Data point function", estimated using Van Geunchten method from moisture function. Since, Van Geunchten method uses the Mualem theory. It provides for a smoother representation of the inflection point in the characteristic curve near saturation.

The bedding, filter layers, gravels and rock shells make no contribution to dissipating the hydraulic head on the downstream side of the dam body. But only the asphalt face is subjected to different hydraulic parameters because only the asphalt face contributes to the dissipation of hydraulic head. Four types of hydrogeological parameter (k_1, k_2, k_3 and k_4) were applied to the asphalt facing. This is because that the trial calculated seepages using this hydrogeological parameter have to be coincided with measured seepage.

3. Boundary conditions

Boundaries define the entry and exit or limits and conditions of flow in the cross-section being analyzed. In a steady state analysis, there are two choices of boundary conditions: a constant pressure (or head) and a constant flux rate. The input data for boundary conditions, called (H), has in "constant" option1666m, 1666.5m, 1669m, 1669.5m, 1670, 1670.5m and 1671m for "Action" applied to the upstream face and a magnitude of 0 m pressure head and 0 m^3/s total fluxes is applied to ground surface and downstream seepage face respectively.

3.8 Slope stability analysis

The stability of an embankment depends on the pore developed either in the dam body or foundation, characteristics of the foundation and fill materials, on the geometry of the embankment section, and additional factors such as presence of water, loading conditions etc. A crack boundary with constant head at 1666m is applied to asphalt face. This section is to analyze the instability occurred on the slope due to crack. To do this the SLOPE/W component of the GEOSTUDIO software has been used.

The SLOPE/W component of the software uses the limit equilibrium analysis technique in the numeric. Limit equilibrium technique of numerical analysis tries to satisfy the equilibrium of statics (SLOPE/W, 2012).

The analysis used in this slope stability determination is the Spencer type. The Spencer type slope stability analysis tries to satisfy both the horizontal and moment equilibrium of statistics. The pore water pressure found in the Seep/W component is a parent for this analysis, to know the saturated part of the dam and foundation. The constitutive model for the materials is Mohr-Coulomb.

The slope stability was observed for the predicted upper limit seepage, in which the downstream face of the dam is at steady state condition.

3.9 Linear regressions

Different techniques are used to investigate the association of variables. If all the variables (dependent and independent) are in linear form, the regression is referred to as the linear regressions. In this paper, there are measured seepage data obtained from the site as dependent and calculated seepages obtained from analysis result of SEEP/W as independent variables written in linear form. So, linear regressions are used since it is the simple and easiest method (Gujarati:, 2004).

The general expression of linear regressions can be written as follows:

$$\boldsymbol{y = a_o + a_1 x_1} \qquad \qquad \boldsymbol{equation\ 3.3}$$

$$where \qquad y = measured\ seepage\ data$$

$$x_1\ is\ calculated\ seepage\ from$$

$$seep\ w\ for\ different\ water\ levels$$

$$a_o, a_1\ is\ LR\ coefficient$$

The relationship was studied obtaining measured values and the SEEP/W results from the analysis of LR by using MATLAB model. The solved results of the equations using MATLAB yields LR coefficients. The goodness of fit a statistical analysis of a regression line root mean square error, Mean error and mean absolute error and r^2 were used to determine how much the calculated seepage deviates from the measured seepage.

$$r^2 = \frac{\left(\sum y_i \hat{y}_i\right)^2}{\left(\sum y_i^2\right)\left(\sum \hat{y}_i^2\right)} \qquad \textit{equation 3.4}$$

Where; $\hat{y}_i = \boldsymbol{predicted\ seepage}$

$\mathbf{y_i} = \boldsymbol{meaured\ seepage}$

$r^2 = \ cofficient\ of\ determination$

r^2 ranges from 0 to 100%. If r^2 is less than 74% or the error is maximum seep/w modeling will be repeated by adjusting the hydraulic conductivity. But if r^2 is greater than 74% or the error is minimum the adjusted LR coefficients will be used to calculate the predicted seepage (Gujarati:, 2004).

Finally, by observing the nature of the correlation between predicted seepage and the measured seepage result; against piping and slope stability, safe monitoring curve was developed for Gibe I dam.

4. RESULT AND DISCUSSION

Seepage analysis was successfully carried out with the use of two dimensional models. The dam is in operation for almost fourteen years. The monitored available seepage recorded data and other parameters are used for running the analysis. Four hydraulic conductivity values were selected and assigned to the asphalt face or impervious part because the analysis is concerned to the asphalt face.

4.1 Results of Seepage Analysis

The Finite Element Models used in the analyses and the computed discharges are shown on respective table and Figures. Owing to the great difference between permeability of materials in the dam's body and its facing, the seepage line through the facing suddenly goes down, while through the dam's body it is not much raised above the foundation, with an exit at the downstream end. The seepage from seep/w considers only the losses from the bituminous lining because the measured seepage collects from the drain hole that is along the asphalt face. The seepage flow was calculated based on the seepage flux along the asphalt face and multiplied by ten meter which is the distance between drain holes inside the gallery of the dam.

The figure that depicts calculated seepage with respect to permeability is depicted below (see figure 4.1) and the rest of figures are on appendix A.

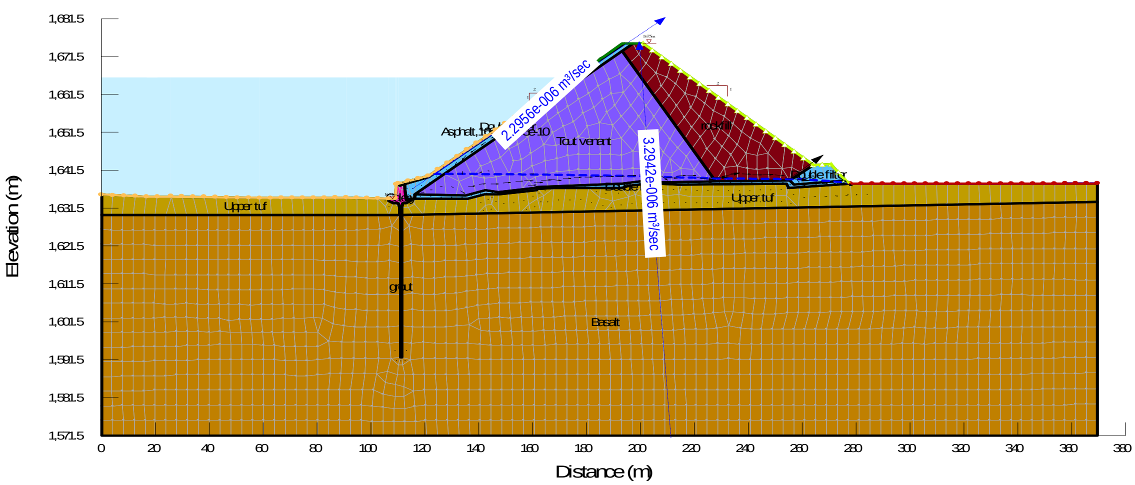

Figure 4. 1: Reservior water level at 1666m k=8.5x10^{-10}m/s

The flow vectors did not cross the grouting since it is highly impermeable. They are concentrated toward the coffer dam or filter it is important that the water is properly drained away and the quantity of drained water is tolerable and small to drain properly outside of the dam body. The same was conducted for different water level and the corresponding seepage of the asphalt face is as depicted in table 4.1.

Table 4. 1: Seepage result @ block 10.4

RWL(M)	Calculated seepage in m^3/s			
	$K=4.5x10^{-11}$	$K=2.5x10^{-10}$	$K=8.5x10^{-10}$	$K=1x10^{-9}$
1666	$1.46x10^{-6}$	$7.61x10^{-6}$	$2.3x10^{-5}$	$2.7x10^{-5}$
1666.5	$1.53x10^{-6}$	$7.96x10^{-6}$	$2.39x10^{-5}$	$2.79x10^{-5}$
1669	$1.82x10^{-6}$	$9.53x10^{-6}$	$2.84x10^{-5}$	$3.33x10^{-5}$
1669.5	$1.88x10^{-6}$	$9.55x10^{-6}$	$2.94x10^{-5}$	$3.45x10^{-5}$
1670	$1.95x10^{-6}$	$9.84x10^{-6}$	$3.09x10^{-5}$	$3.55x10^{-5}$
1670.5	$2.01x10^{-6}$	$9.97x10^{-6}$	$3.22x10^{-5}$	$3.70x10^{-5}$
1671	$2.10x10^{-6}$	$1.02x10^{-5}$	$3.47x10^{-5}$	$3.84x10^{-5}$

Different trial hydraulic conductivity ($K=4.5x10^{-11,}$ $K=2.5x10^{-10,}$ $K=8.5x10^{-10,}$ $K=1x10^{-9}$) were used to calculate the predicted seepage. Finally, conductivity $K=8.5x10^{-10}$ is selected; since it represents the dam's permeability or the calculated seepage is closer to the measured seepage with r^2 of 0.89.

4.3 Result of slope stability analysis

The results of slope stability analysis by limit equilibrium method are the factor of safety, normal and shear stresses along the shear surface, and the normal and shear stresses along the inter slice boundaries. Rock fill shell is free draining the seepage penetrates through asphalt facing toward foundation. The rock fill will not develop pore water but the upper foundation is weak and weathered so it is exposed to develop pore water pressure. This pore water pressure has significant effect on slope stability of the dam even though the dam is rock fill and free draining.

The head of phreatic line has increased up due to additional pore pressure developed on the foundation. The factor of safety for a static condition at steady state seepage where the shear parameters are effective, the minimum required factor of safety is 1.5 (USBR, 1995). However, the slope/w analysis result is below the minimum safety factor set as shown below and the rest of figures are on appendix A. It indicates that unsafe or unstable.

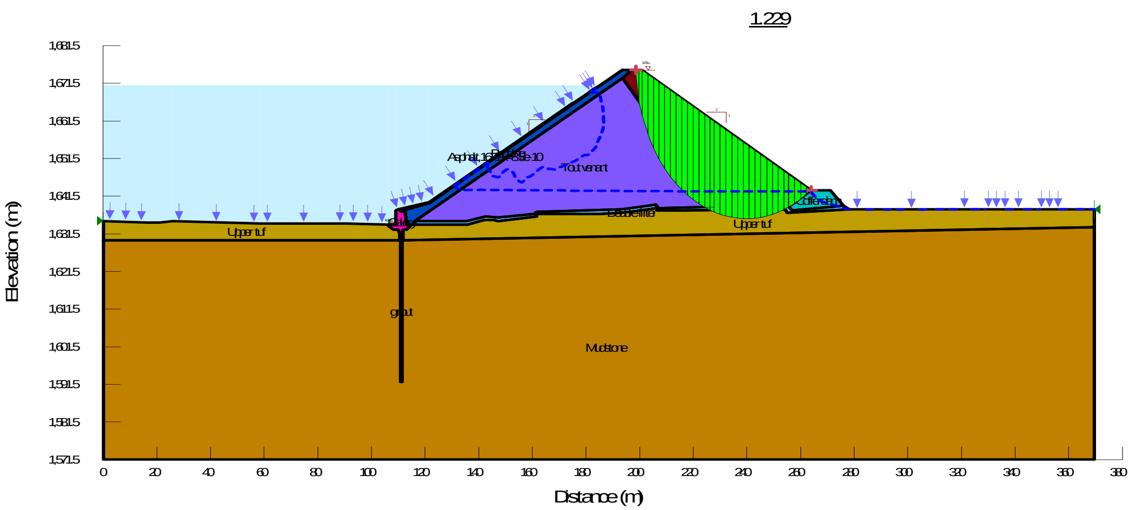

Figure 4. 2: reservoir water level at 1671m k=8.5x10^{-10}m/s

4.2 Linear regression analysis

Seep/w analysis has been done succesfully. For given reservior level and hydrogeological parametre the seep/w has generated seepage,pore water pressure and and other results at the selected section. The calculated seepage and measured seepage are input for linear regrasion analysis. The predicted monitoring seepage was estimated by the LR equation.

$$\boldsymbol{y = a_o + a_1 x_1} \qquad \textbf{equation 4.1}$$

$$\textbf{where} \quad \boldsymbol{y}, \textbf{is Predicted seepage}$$

$$x_1, \textit{is calculated seepage for trial } K$$

$$a_0, a_1 \textit{ are adjusted LR coefficients}$$

There are about eight (8) LR equations. Holding this equation to solve using MATLAB model to obtain the value of regrassion coefficients which are 0 and 0.4335 for a_o, a_1 respectively. The goodness fit for the linear regression analysis was checked with RMSE, mean error, mean absolute error and r^2.

Table 4. 2: Goodness fit

RMSE m^3/s	Mean error m^3/s	Mean absolute error m^3/s	R^2
$3.97x10^{-6}$	$-3.97x10^{-6}$	$3.97x10^{-6}$	0.89

The goodness of indicator obtained can be seen from table 4.2. The goodness of fit indicators identify wheather the measured seepage coincides with the predicted seepage or not. From Table 4.2 it is evident that the model result concides with the measured result with RMSE of $3.97x10^{-6} m^3/s$, Mean error of $-3.97x10^{-6}$ m^3/s mean absolute error of $3.97x10^{-6}$ m^3/s and r^2 value of 0.89. The RMSE, Mean error and Mean absolute error being closer to zero indicate the acceptable results. While the r^2 value being greater than 0.74 clearly show the result is again acceptable.

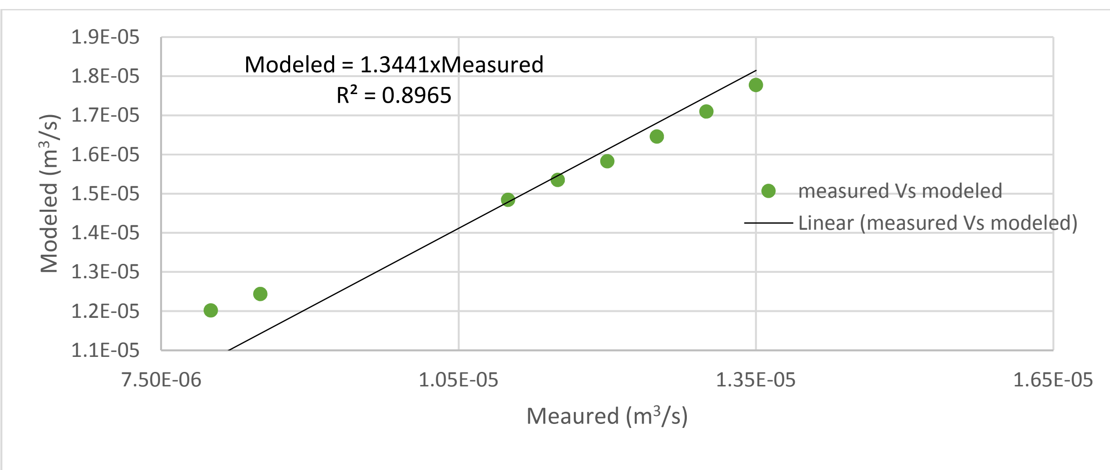

Figure 4. 3: Coefficient of determination

4.3 Monitoring Curve

Once the seep/w model parameters are fixed as shown in the seep/w model section and the regression coefficients in section 4.2, it is now possible to create a monitoring curve which defines the safe seepage rate in the Gilgel Gibe I embankment dam asphalt concrete face. The coeffients of LR has been multplied with the seep/w generated seepage to calculate a predicted seepage. Upper and lower limit curve is developed by adding or subtructing RMSE to/from the predicted seepage.

Table 4. 3: Predicted seepage

RWL(M)	Predicted seepage (m^3/s)	Lower limit(m^3/s)	Upper limit(m^3/s)
1666	$1.20x10^{-5}$	$8.04X10^{-6}$	$1.60X10^{-5}$
1666.5	$1.24x10^{-5}$	$8.46X10^{-6}$	$1.64X10^{-5}$
1669	$1.48x10^{-5}$	$1.09X10^{-5}$	$1.88X10^{-5}$
1669.5	$1.53x10^{-5}$	$1.14X10^{-5}$	$1.93X10^{-5}$
1670	$1.58x10^{-5}$	$1.18X10^{-5}$	$1.98X10^{-5}$
1670.5	$1.65x10^{-5}$	$1.25X10^{-5}$	$2.04X10^{-5}$
1671	$1.71x10^{-5}$	$1.31X10^{-5}$	$2.11X10^{-5}$

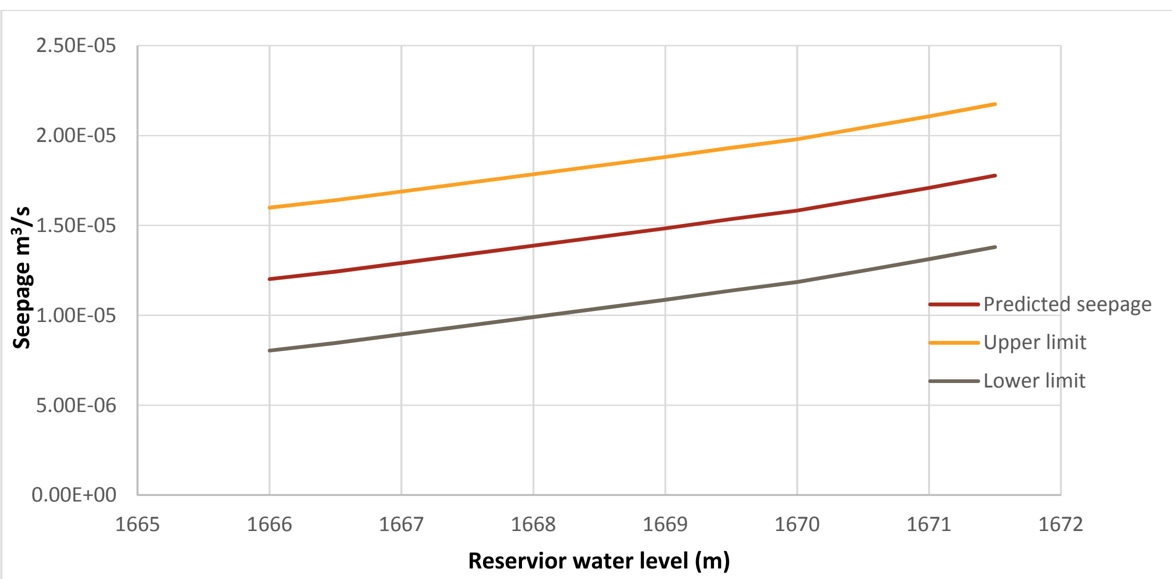

Figure 4. 4: Monitoring curve

For the upper limit curve or monitoring curve, seep/w and slope/w analysis is done in order to check against piping and slope stability due to pore water pressure developed. From the most fluctuating reservior water level range (1666m-1671m) at 1666m crack is assumed on the facing and at this a head boundary is set. When asphalt is exposed to weather variation, cracking; the most common failure type might happen.

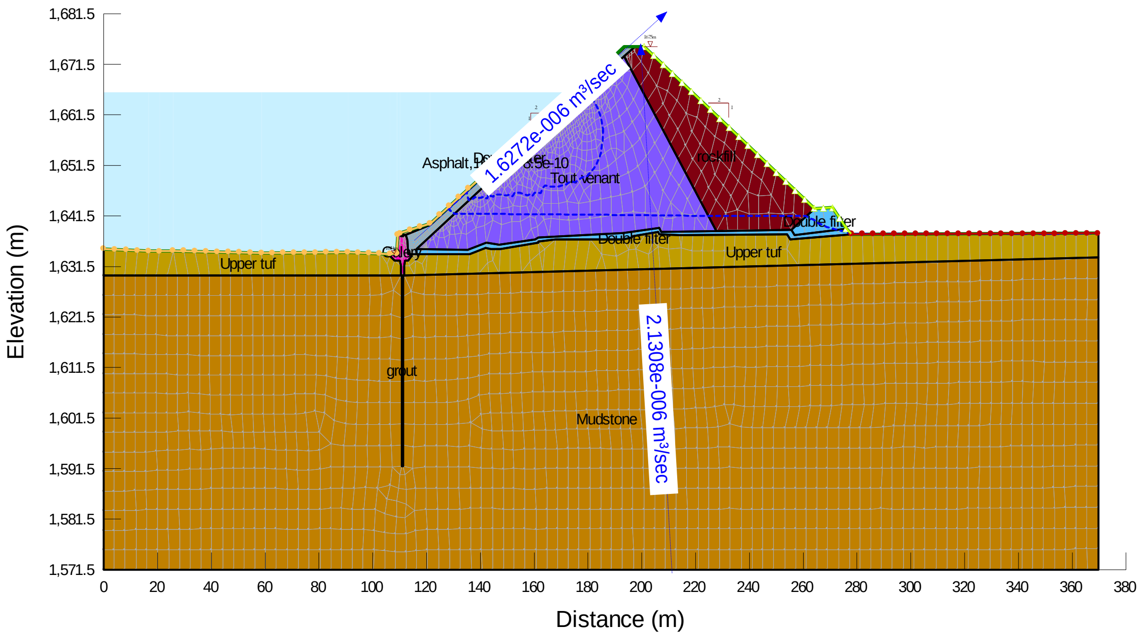

Figure 4. 5 reservoir water level at 1666m k=8.5x10^{-10}m/s

Concentrated seepage flow is located at the crack toward the bedding. The seepage pressure is produced by the friction between the percolating water and the walls of the voids and can be described as a “drag”. That is piping is occurred when seepage flow from impervious to pervious region with high pressure (Dr.A.R.Arora, 2004). The water flows from upstream face to the bedding material makes friction between the water and the bedding material voids tends to lift the sand grains toward the rock fill.

Table 4. 4 : Safety factor against piping

RWL(m)	Head		ΔH	Length		ΔL	i_{cal}	Fs
1666	1664.98	1644.61	20.37	173.45	135.79	37.66	0.541	2.403
1666.5	1665.07	1644.75	20.32	173.57	136.07	37.50	0.542	2.399
1669	1666.06	1645.34	20.72	175.45	137.25	38.20	0.542	2.397
1669.5	1666.12	1645.50	20.62	175.45	137.5	37.95	0.543	2.393
1670	1666.95	1645.69	21.26	177.00	137.9	39.10	0.544	2.391
1670.5	1667.38	1645.56	22.24	178.09	137.98	40.11	0.554	2.345
1671	1669.15	1648.55	21.6	182.55	143.78	38.77	0.557	2.333

$I_{critical} = (\gamma_{sub} / \gamma_w)$

$i_{cr} = \gamma_{sat} - \gamma_w / \gamma_w$ *equation 4.2*

$i_{cal} = \Delta H / L$ *equation 4.3*

γ_{sub} is submerged unit weight of bedding material

γ_{sat} Saturated unit weight of bedding material=23kN/m^3

γ_w unitweight of water

Factor of safety = i_{cr} / i_{cal} *equation 4.4*

ΔH is the head lost between the last two equipotential lines, and L is the length of the flow element. A factor of safety of 3 to 4 is considered adequate for the safe performance against piping (A.R.Arora, 2004). When, seepage is observed above the upper limit curve the factor of safety will be below the recommended i.e piping is started as depicted in table 4.4.

A paper on construction quality monitoring of embankment dam for the case of Kesem dam has been done in November 2014 by Abebe Arega. This research has used measured pore water pressure and seep/w model to predict a pore pressure in the dam body for under construction dam. The coefficients were obtained from the correlation of measured and modeled pore water pressure using MLR model. The coefficients obtained have resulted in an r^2 value of nearly unity. This indicates the result of the pore water pressure from the model is almost the same as the result obtained from the recorded data. Hence, he concluded that the technique of construction quality monitoring used in Kesem dam has promising application if used.

4.4 The current state of Dam safety monitoring of Gilgel Gibe I dam

The flow coming out from the dam drain galleries is measured and monitored by means of the drain holes and V notches purposely expected. There is no specific threshold value of total or single seepage flow for the criteria for routine or alert procedure application. With full reservoir water level estimated seepage, talking for each reservoir level safety is difficult; since the dam impound throughout the year is not full.

For stable reservoir water level the seepages through dam foundation and dam body for the application of the routine procedure, stable conditions or vary according to rain/dry season trends. But alert procedure will be activated as soon as increasing with time (for foundation drains only if independently from season trend) or for local drains when turbidity or excessive seepage flow.

For raising reservoir water level to have routine procedure the leakages through the drainage holes set up in the inspection gallery of a dam body and at the ground surface downstream if, season dependent increased. But, when there is time dependent increment and for local drains when turbidity of existing seepage flows alert procedure is taken.

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Data of good quality were obtained and used for the analysis of GeoStudio 2012 (seep/w). Different reservoir level and trial hydraulic conductivity was an input to seep/w software in order to calculate trial seepage. Finally, a conductivity of 8.5×10^{-10} m/s ($K = 8.5 \times 10^{-10}$ m/s) that represent the dam's permeability with r^2 of 0.89 was selected.

Since full reservoir water level could not pound throughout the year, for most fluctuating reservoir water level safe seepage is predicted. LR was used to define the coefficients by using measured and calculated seepage as input then predicted seepage is obtained. The predicted seepage and the measured seepage were checked for deviation and the goodness fit found to be acceptable. From the predicted seepage, a monitoring curve is developed by adding and subtracting RMSE. The upper limit is checked against piping and slope stability and it is critical point. Since the seepage above the critical value or upper limit has a significance effect on piping and slope stability, any seepage above it implies safety problem. Accordingly, the methodology adopted in deciding the safe and critical seepage through the asphalt concrete face is good enough. The curve obtained can be used to monitor whether the dam is safe against seepage or not for future operation.

Safety monitoring is the "eyes and ears" (LI, 2015) and it is also an important fundamental task for dam safety monitoring. Internal erosion from dam body or foundation, which is one of the major reasons for embankment dam failure, causes an increased seepage flow due to loss of fines. Methods that are able to register small changes in the seepage flow rate relative to the monitoring curve (upper limit) developed along the entire dam are therefore important. Because they can detect internal erosion or any defects at an early stage before it starts to affect the safety of the dam.

5.2 Recommendation

Existing dam safety monitoring through seepage analysis was effective in this paper. The result tells not only the present condition of the dam but also used to predict the condition for future. Any seepage above the predicted or upper limit curve represents that the asphalt facing had exposed to any defect. So, urgent scrutiny and monitoring of the dam problem is important.

Gilgel Gibe I monitoring method have no threshold limit corresponding to each reservoir water level for routine or alert procedure application of seepage monitoring. Rather sudden emergence of seepage or turbidity of leakage observed at the drainage holes set up in the inspection gallery of a dam body and at the ground surface downstream may indicate local seepage failure in the foundation or the dam are indicators to know the safety of the dam condition. However, in this paper a threshold value has been set for each reservoir water level to have safe operation of the dam. Since it is asphalt concrete face, unless there is problem the seepage will not increase from the predicted quantity.

- So, any increment from the predicted seepage range even it is not excessive or turbid, immediate assessment of the cause is important.
- In Germany and Sweden embankment dam safety is monitored through temperature measurement. Since, it is very easy to measure and can give useful information about the presence of anomalous flows. Therefore, it is recommended to adopt this method if subsequent modeling is done.
- The methodology adopted here could also be used for monitoring safety of an embankment dam. However, the linear regression analysis can be replaced by any other statistical analysis which could be used to fit modeled values to a measured one.

6. REFERENCES

1. (ETCOLD), E. C. O. L. D., 2014. *ETHIOPIAN DAM SAFETY GUIDLINE.* s.l.:s.n.
2. (TADS), T. A. F. D. S., 2017. *http://www.deq.state.ms.us.* [Online]
   Available at: Training Aids For Dam Safety
3. Ali Beheshti, A. K. a. H. A., 2013. Seepage Analysis of Rock-Fill Dam Subjected to Water Level Fluctuation A case study on Gotvand-Olya Dam. *Iranica Journal of Energy & Environment 4,* pp. 155-160.
4. Bulletin 99, I., 1995. Dam failure statistical analysis. In: Paris: s.n.
5. Dr.A.R.Arora, ,2004. *Soil mechanics and foundation engineering.* Delhi: Lomus offset press.
6. Fell, R., ,2005. *Geotecnical Engineering of Dams.* Britain: A.A Balkema publishers leiden.
7. Foster M., F. V. N. J. S. G. D. R., April 28-May 2, 2008. *SEEPAGE AND PIPING TOOLBOX CONTINUATION, PROGRESSION,INTERVENTION AND BREACH.* Portland, Oregon, s.n.
8. Gujarati:, 2004. *Basic econometrics.* s.l.:© The McGraw−Hill.
9. Hirose, T., ,2017. *WATER STORAGE, TRANSPORT, AND DISTRIBUTION – Instrumentation and Monitoring of Dams and Reservoirs Mizuno and Toshio Hirose.* [Online]
   Available at: http://www.eolss.net/Sample-Chapters/C07/E2-12-02-06.pdf
   [Accessed 04 01 ,2017].
10. ICOLD, I. C. O. L. D., ,2000. *REHABLITATION OF DAMS AND APURTENANT WORKS, bulletin119.* Paris: boulevard haussmann.
11. Johansson, S., ,2012. *Experiences from internal erosion detection and seepage monitoring based on temperature measurements on Swedish embankment dams.* Paris, s.n.
12. Jose_Navar2, 2016. *https://www.researchgate.net/profile/Jose_Navar2.* [Online]
    Available at: https://www.researchgate.net
    [Accessed 29 April 2017].
13. Krah, J., ,2012. *Seepage Modeling with SEEP/W.* Calgary, Alberta, Canada: GEO-SLOPE International Ltd.
14. LI, J., 2015. *UPDATING AND MODERNIZING THE DAM SAFETY MONITORING FACILITIES OF THE CHARSEN RESERVOIR.* China, s.n.
15. P.Novak, A. C. a. R., 2004. *Hydraulic Structures Third Edition.* London and New York: s.n.

16. SLOPE/W, 2012. *Stability modeling with SLOPE/W 2012 Version.* s.l.:Geo-Slope international ltd,.

17. Small Dam, D., 2011. *SMALL DAMS Design, Surveillance and Rehabilitation.* s.l.:s.n.

18. Tanchev, L., ,2014. *Dams and Appurtenant Hydraulic Structures.* London, UK: CRC Press/Balkema.

19. Thomas, H. H., ,1976. *The engineering of large dams.* london newyork, sydney toronto: jhon willey and sons.

20. USACE, 2006. *US Army Corps of Engineering General Design and Construction Considerations for Earth and Rock-Fill Dams.* s.l.:s.n.

21. USBR, 1995. *Safety Evaluations of Existing Dam (SEED).* Denver: United States Government Printing Oflice.

22. Xiao-dong Chen, H.-x. G. &. E.-x. S., 2008. Analysis method for slope stability under rainfall action. *Landslides and Engineered Slopes – Chen et al. (eds)*, pp. 1507-1515.

7. APPENDIX A

7.1 Appendices to chapter four

7.1.1 SEEP/W result

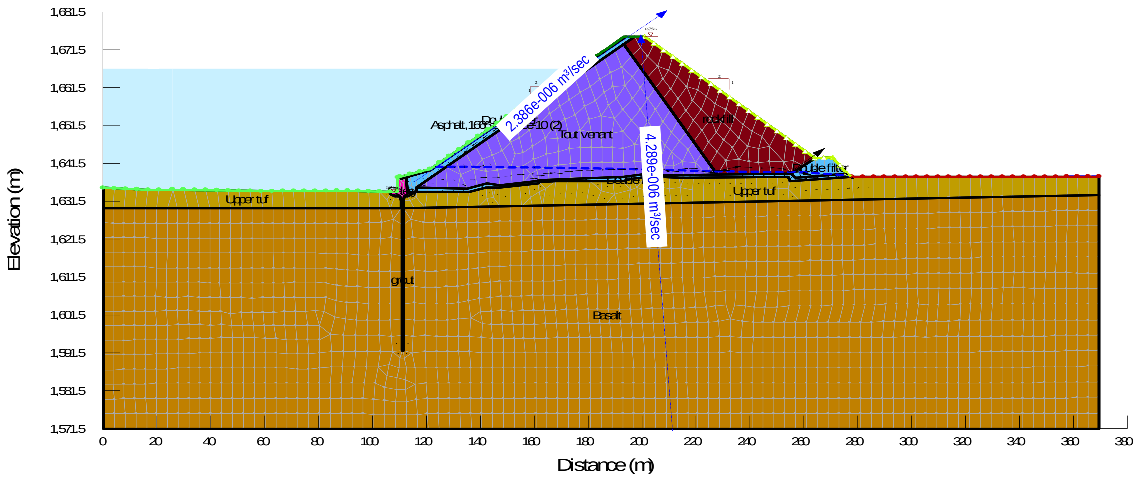

Figure A . 1: Reservoir water level at 1666.5m k=8.5x10^{-10}m/s

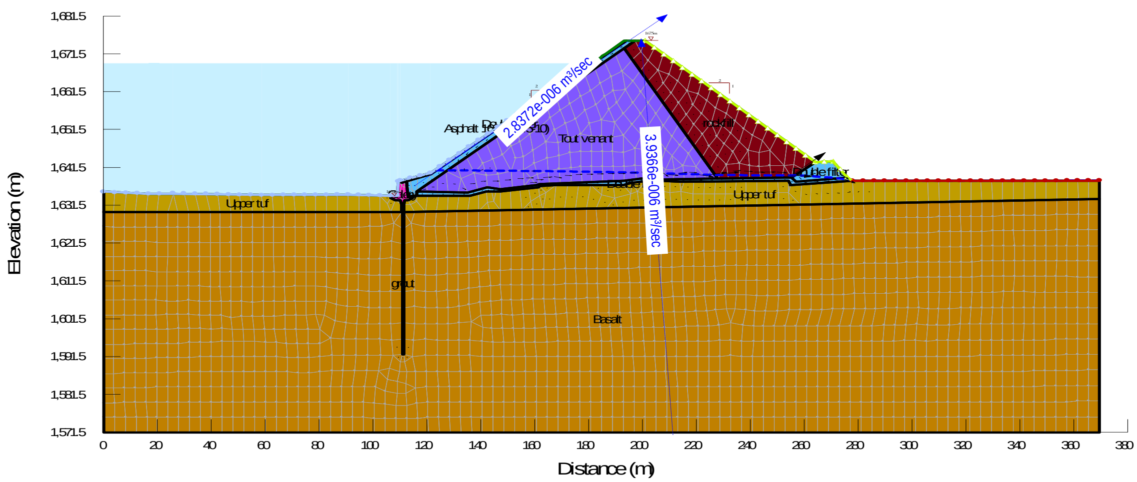

Figure A . 2: Reservoir water level at 1669m k=8.5x10^{-10}m/s

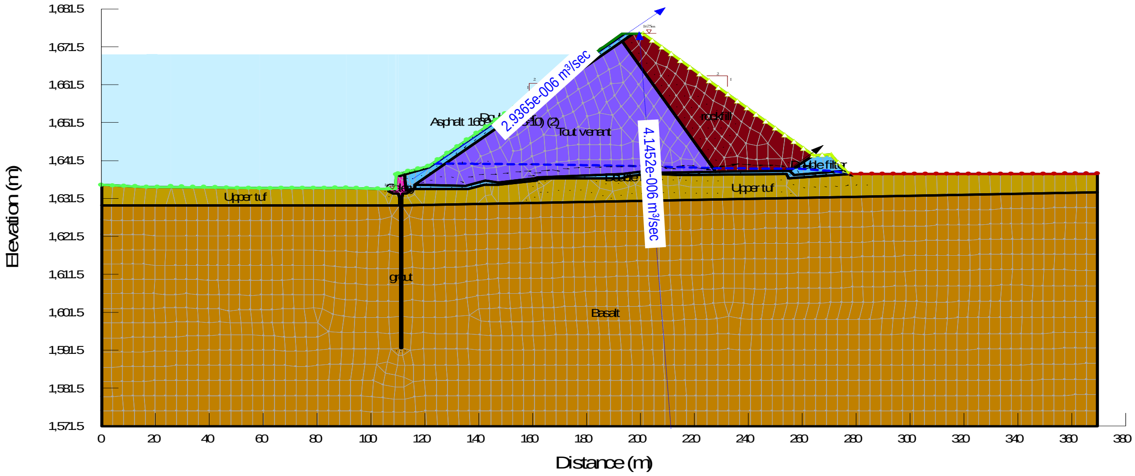

Figure A . 3: reservoir water level at 1669.5M k=8.5x10^{-10}m/s

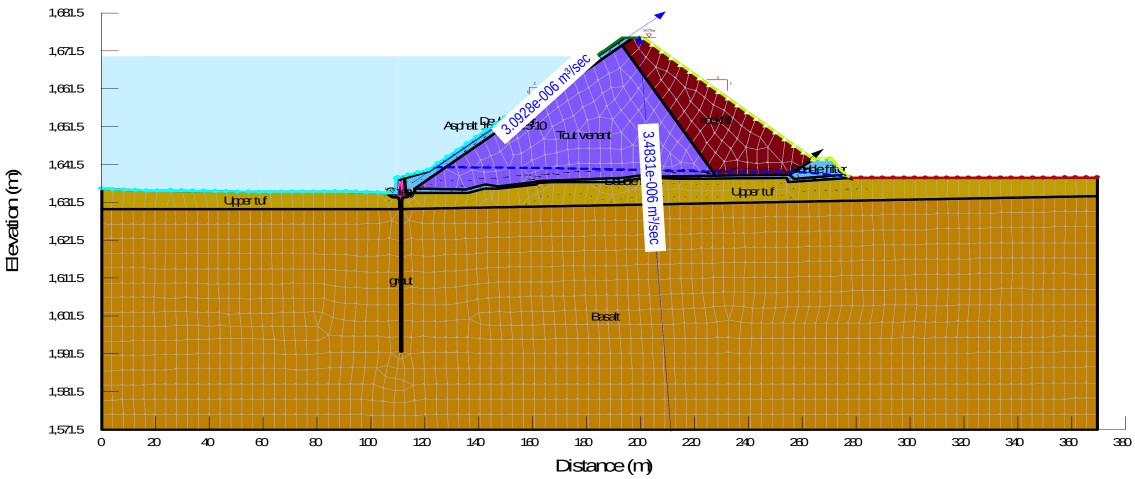

Figure A . 4: Reservoir water level at 1670m k=8.5x10^{-10}m/s

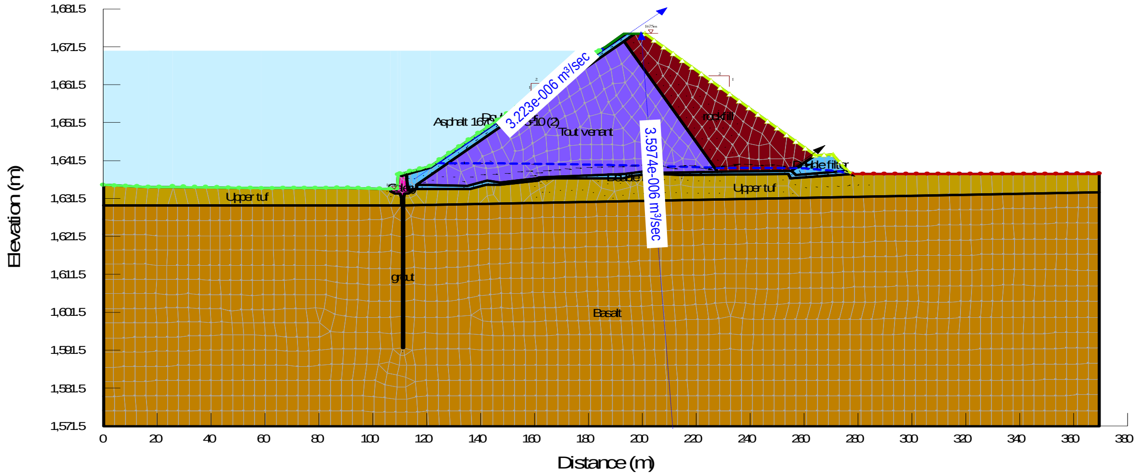

Figure A . 5: Reservoir water level at 1670.5m k=8.5x10^{-10}m/s

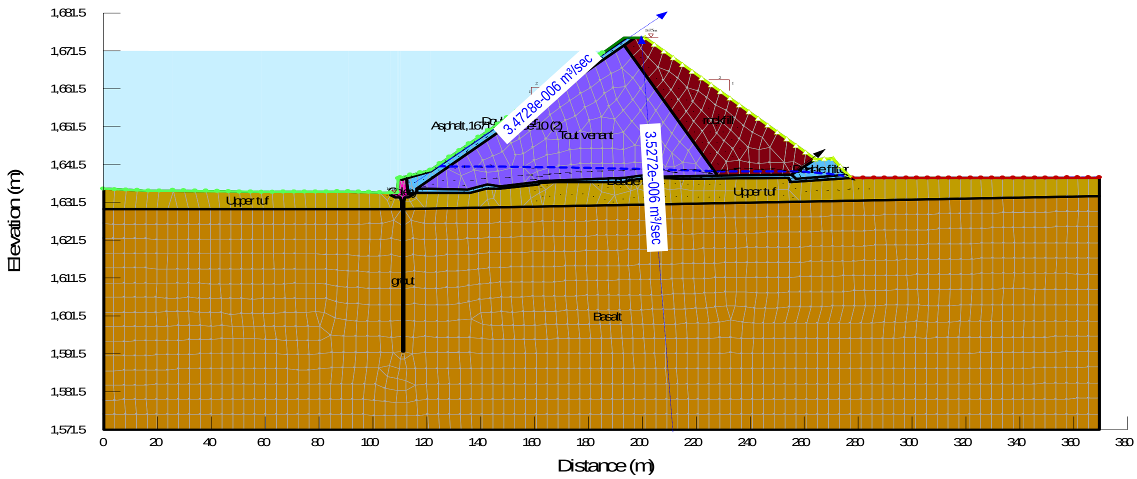

Figure A . 6: Reservoir water level at 1671m k=8.5x10^{-10}m/s

7.1.2 SLOPE/W result

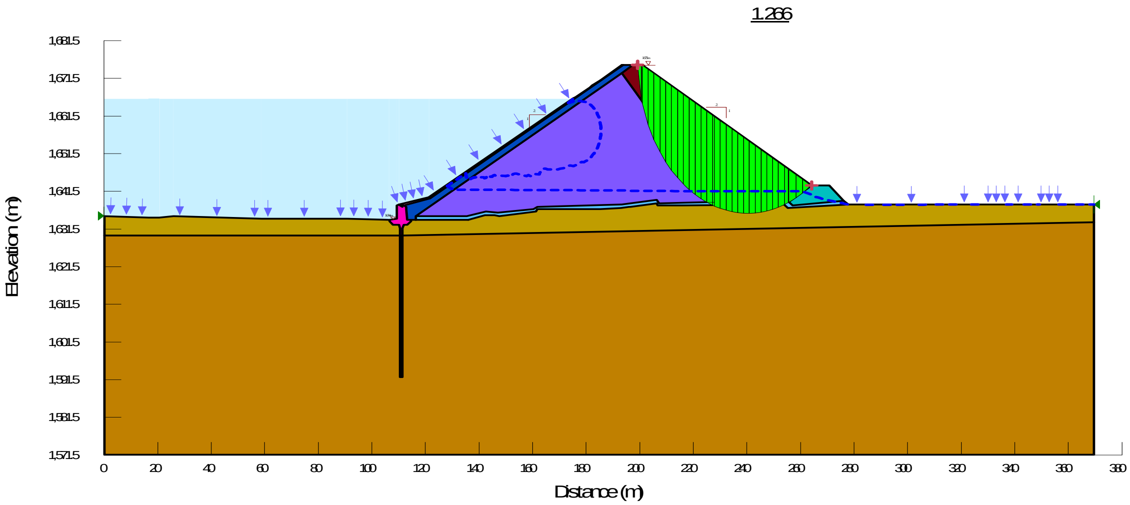

Figure A . 7: reservoir water level at 1666 M k=8.5x10^{-10}m/s

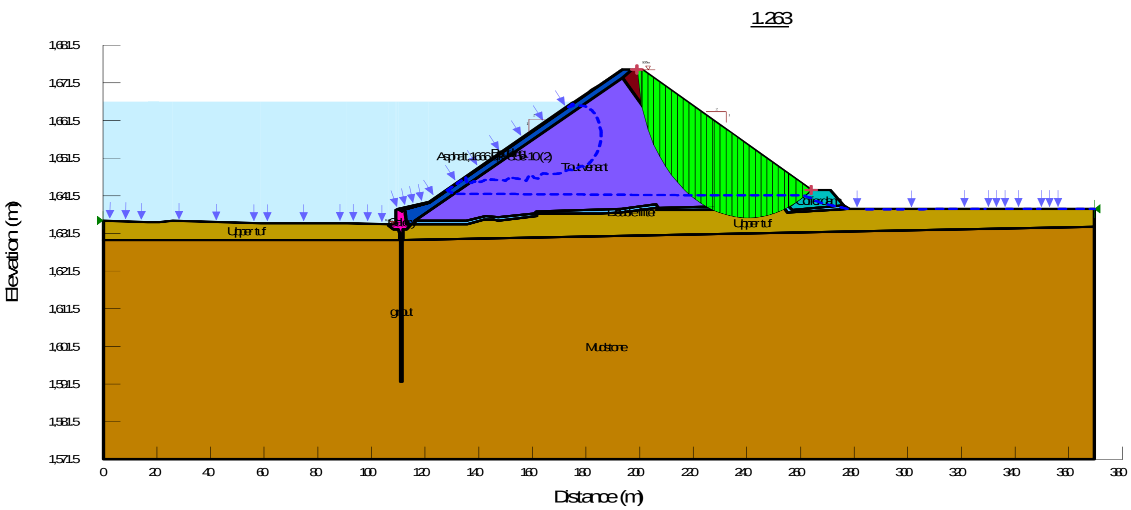

Figure A . 8: reservoir water level at 1666.5M k=8.5x10^{-10}m/s

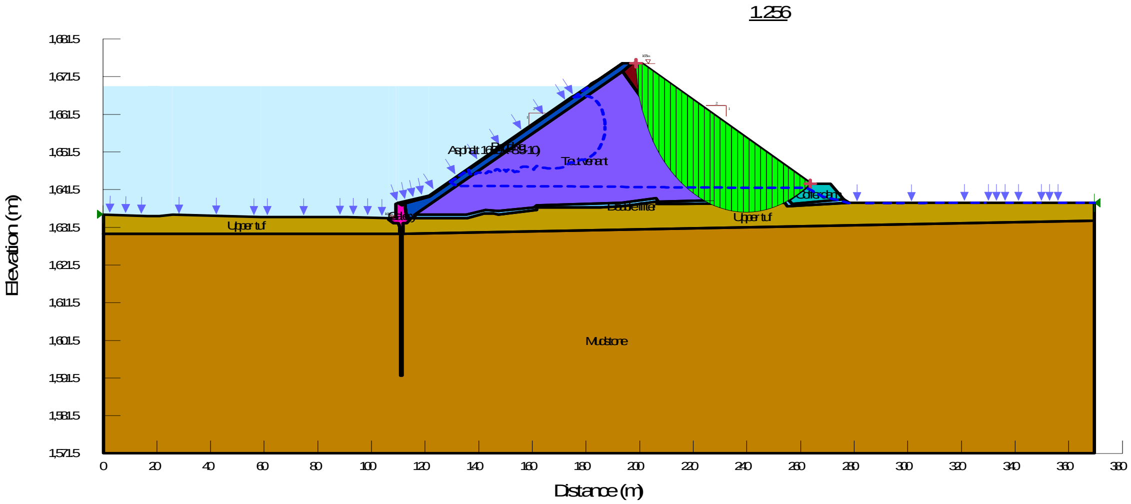

Figure A . 9: reservoir water level at 1669m k=8.5x10^{-10}m/s

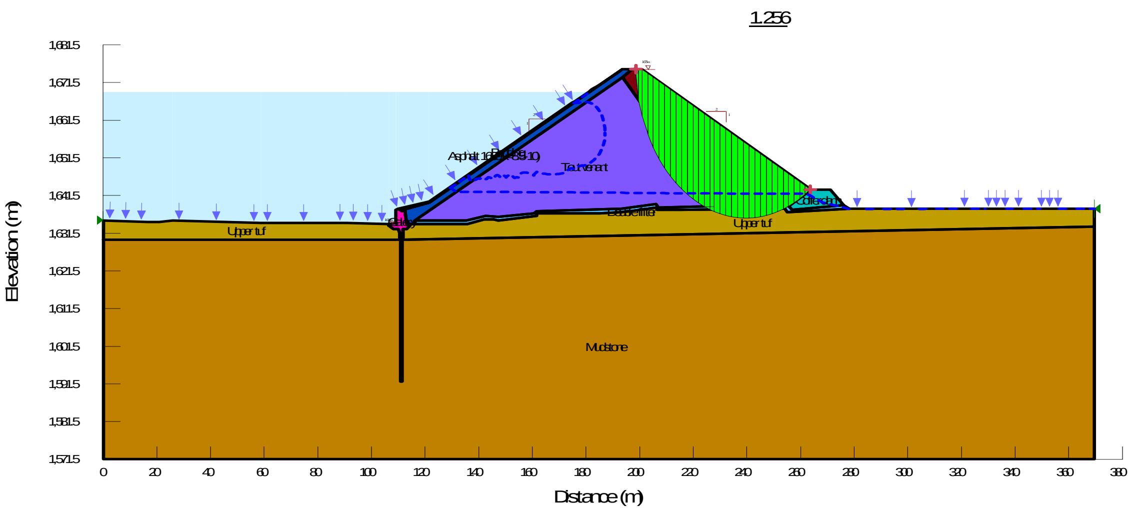

Figure A . 10: reservoir water level at 1669.5m k=8.5x10^{-10}m/s

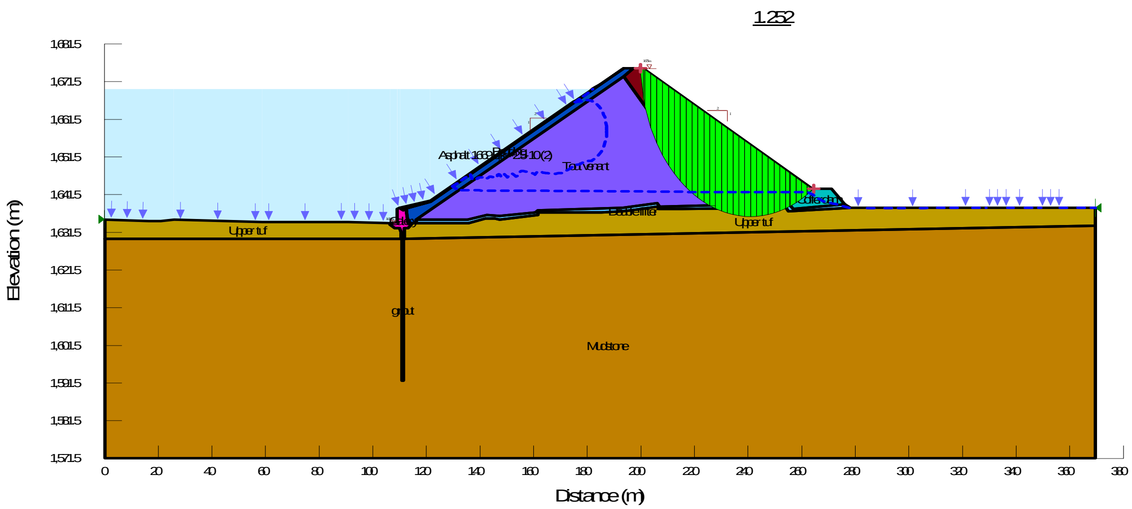

Figure A . 11: reservoir water level at 1670m k=8.5x10^{-10}m/s

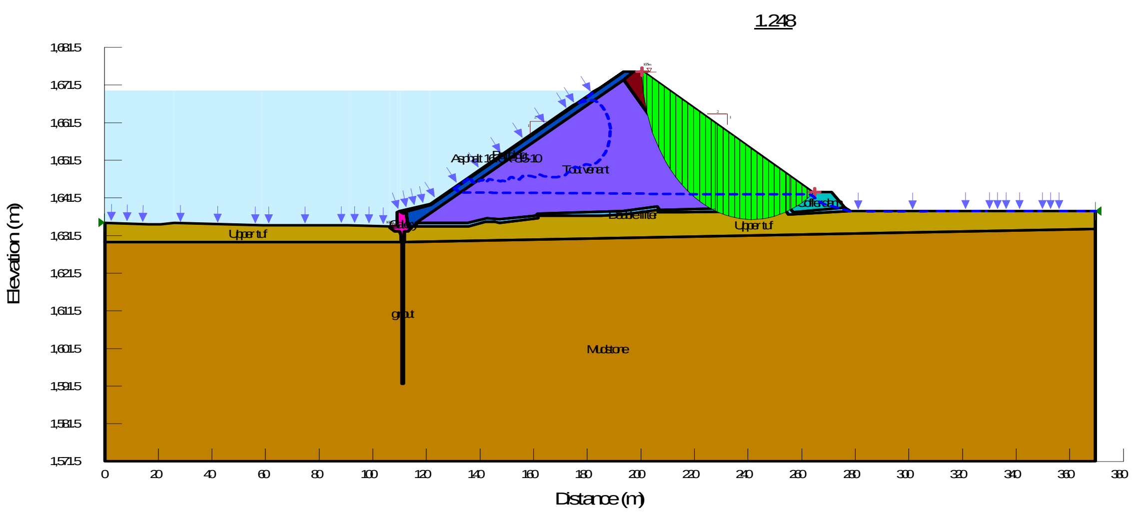

Figure A . 12: reservoir water level at 1670.5m k=8.5x10^{-10}m/s

7.1.3 SEEP/W result against piping

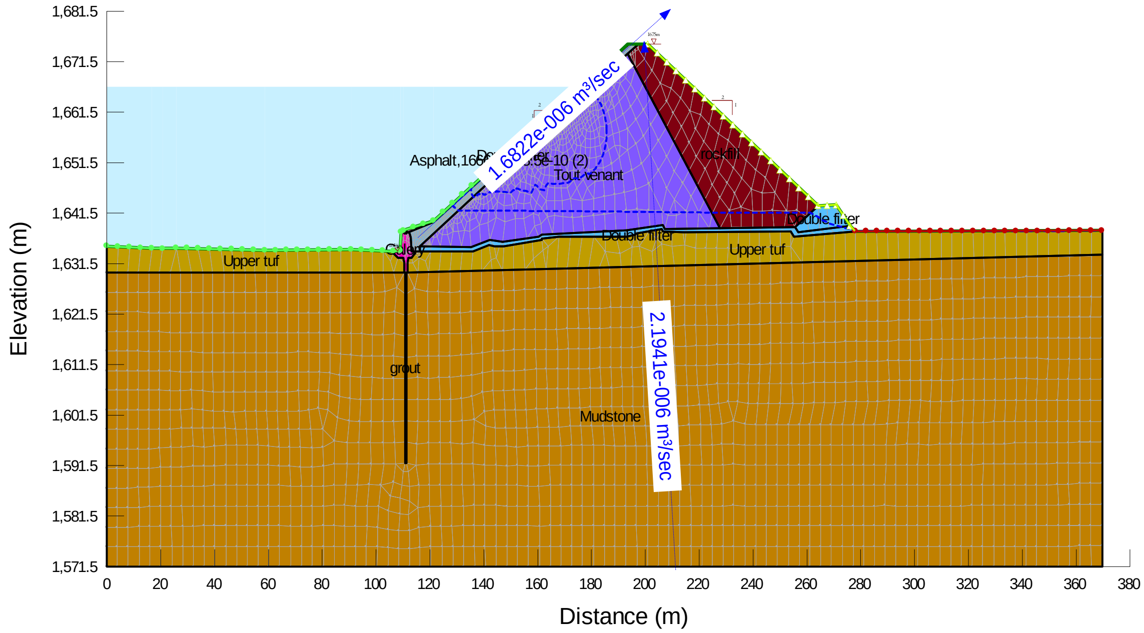

Figure A . 13: reservoir water level at 1666.5M k=8.5x10^{-10}m/s

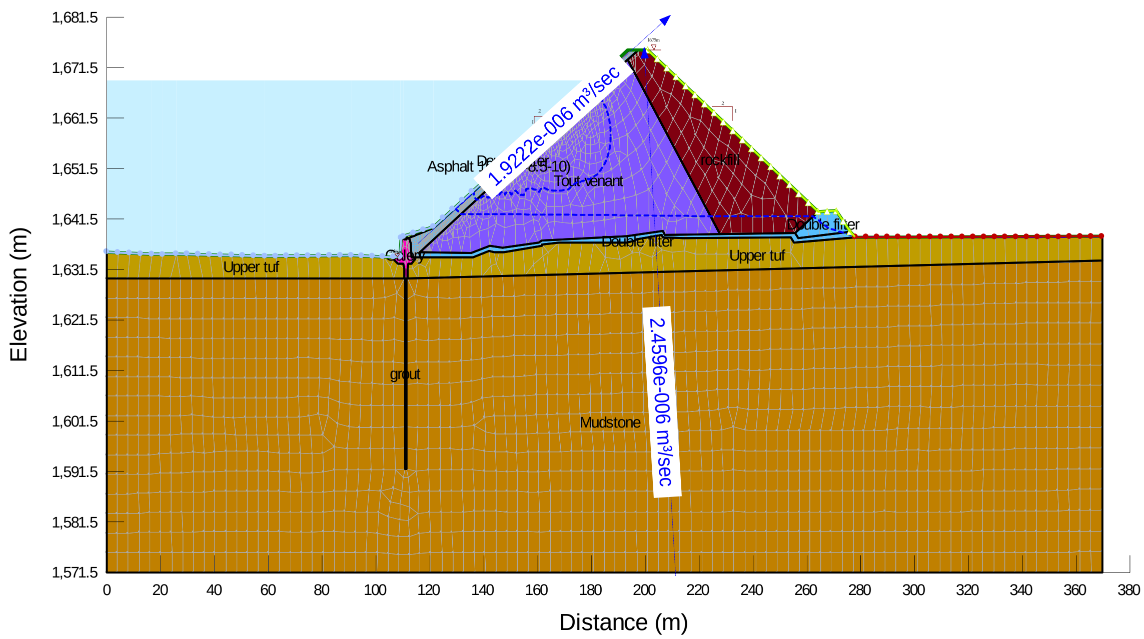

Figure A . 14: reservoir water level at 1669m k=8.5x10^{-10}m/s

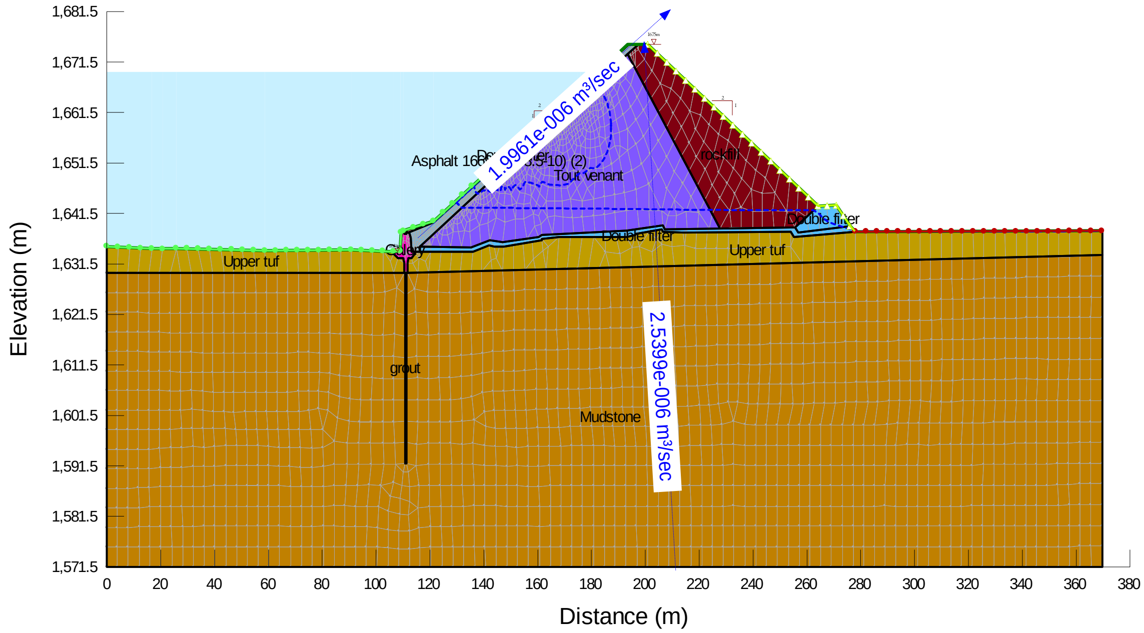

Figure A . 15: reservoir water level at 1669.5m k=8.5x10^{-10}m/s

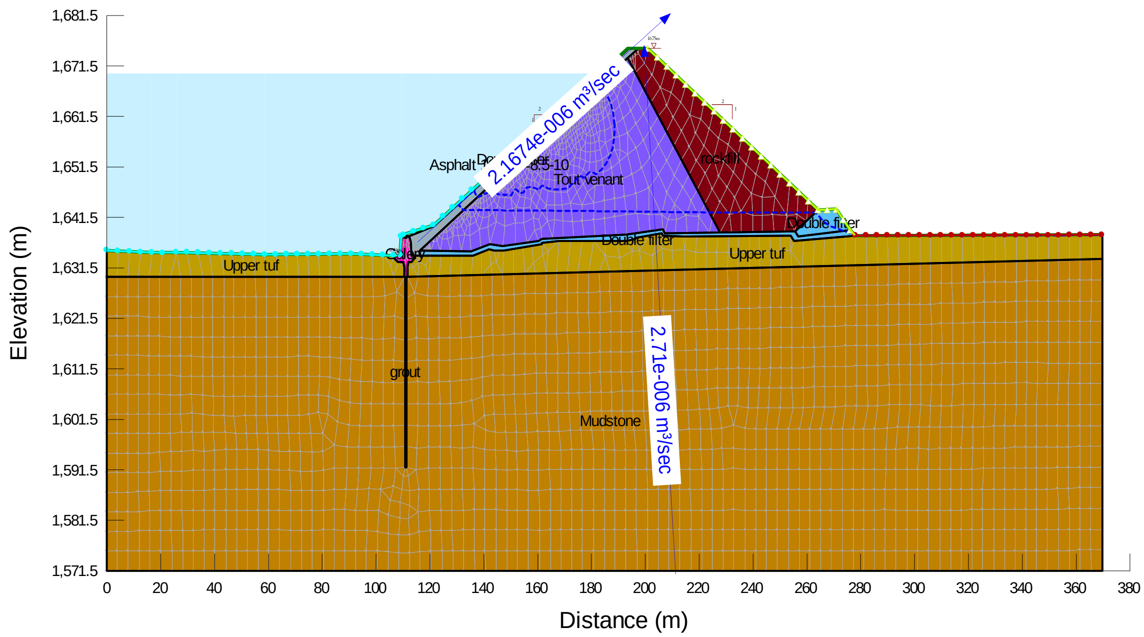

Figure A . 16: reservoir water level at 1670m k=8.5x10^{-10}m/s

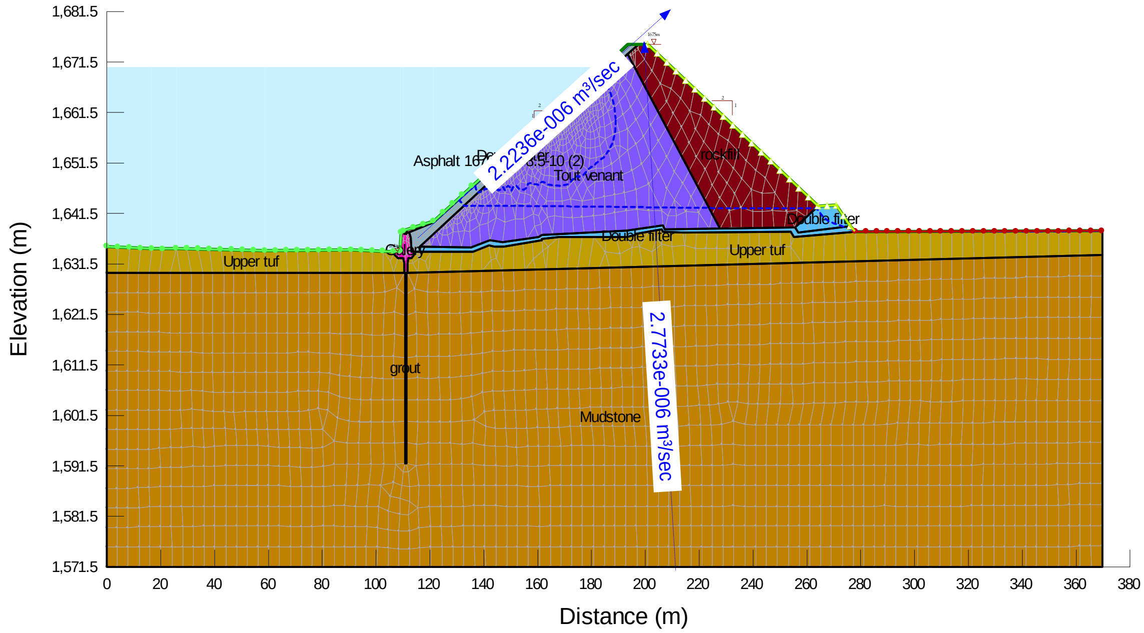

Figure A . 17: reservoir water level at 1670.5m k=8.5x10^{-10}m/s

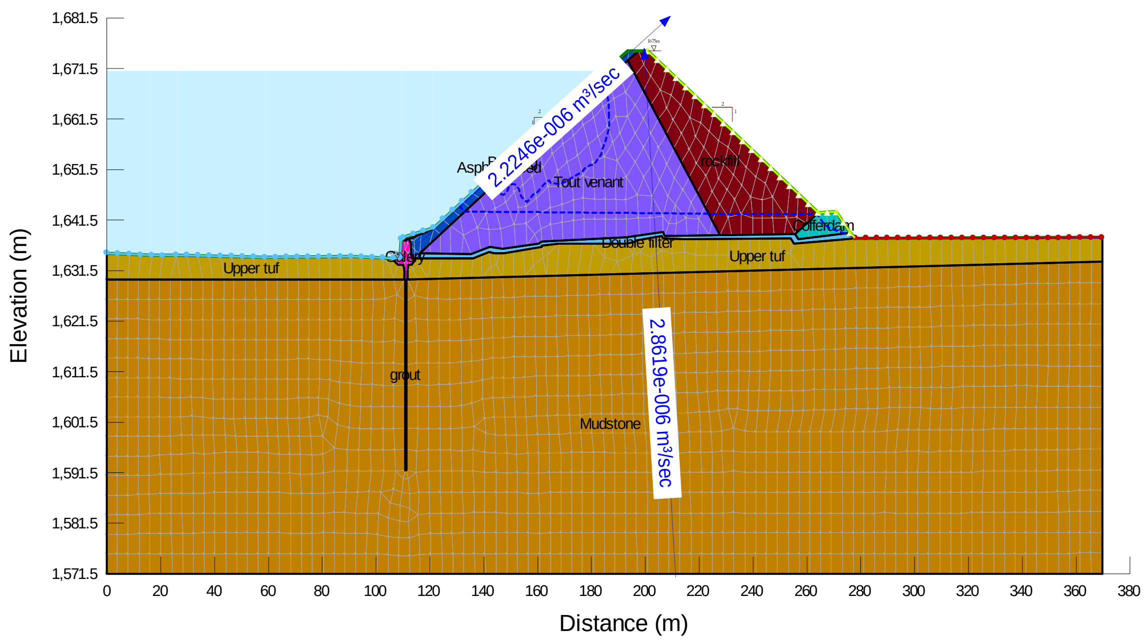

Figure A . 18: reservoir water level at 1671m k=8.5x10^{-10}m/s