



**ANALYSIS OF SEEPAGE AND ITS IMPACT ON EARTH DAM:
CASE STUDY GERESEGEN EARTH DAM, TIGRAY, ETHIOPIA**

M Sc THESIS

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**ANALYSIS OF SEEPAGE AND ITS IMPACT ON EARTH DAM:
CASE STUDY GERESEGEN EARTH DAM, TIGRAY, ETHIOPIA**

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(Submission Sheet- 1)

We, the undersigned members of the Broad of Examiners of the final open defense by **Berihu Haile Hagos** have read and evaluated his thesis entitled “**Analysis of Seepage and Its Impact on Earth Dam: Case study Gerebsegen Earth Dam, Tigray, Ethiopia**”, and examined candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of **Master of Science in Water Resource Engineering** with **Specialization in Dam Engineering**.

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DECLARATION

I hereby declare that this MSc dissertation is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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LIST OF ABBRIVATIONS

| | |
|-----------------|--|
| ANCOLD | Australia National Commissions On Large Dam |
| ARL | Actual Reservoir Level |
| ATSWWC | Akam Three S Water Works Consultant |
| C | Cohesion, Counts |
| Ch | Chainage |
| D/S | Down Stream |
| DCL | Dam Crest Level |
| E | Exponential |
| FEM | Finite Element Methods |
| Fs | Factor of Safety |
| Fsmin | Factor of Safety minimum |
| GLE | General Limit Equilibrium |
| ICOLD | International Commissions On Large Dam |
| LEM | Limit Equilibrium Methods |
| m.a.s.l | meter above sea level |
| Mm ³ | Million cubic meters |
| M-PM | Morgenstem- price Method |
| MRL | Minimum Reservoir Level |
| NPL | Normal Pool Level |
| OGI | Original Ground Level |
| PDE | Partial Differential Equation |
| q | Seepage rate |
| SBM | Simplified Bishop Method |
| SEEP/W | Finite element software for ground water (SEEPPage for Window) analysis |
| SLOPE/W | Limit equilibrium software for slope stability (SLOPE for window) analysis |
| TWRB | Tigray Water Resource Bureau |
| TWWDSE | Tigray Water Works, Design and Supervision Enterprise |
| u | Pore Pressure |
| U/S | Upstream |
| USACE | United States Army Corps of Engineers |
| USBR | United States Bureau of Reclamation |
| UTM | Universal Transverse Mercator |
| VWC | Volumetric Water Content |
| Ø | Angle of friction |

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ABSTRACT

Gerebsegen dam, zoned earth type of dam with height of 46m was constructed for the purpose of irrigation and Mekelle city water supply. It is located in the Southwestern part of Mekelle city, capital city of Tigray regional state, Northern part of Ethiopia. The dominant geological formations of the site are dolerite shale and marl-limestone intercalations, characterized by cavernous, joints and karastic features which can be responsible for the excessive seepage and presents serious problems in the dam foundation, reservoir and abutments that may be so extensive as to make corrective measures impracticable. The main intention of this thesis work was to evaluate the performance of the dam related to seepage and slope stability, for which Geo-Studio-2007 software (SEEP/W and SLOPE/W) was used throughout the analysis. Prior to the analysis using the model, seepage dominantly flowing through the foundation and abutments was measured using current and float method. The measured value was 0.2053 m³/sec and the annual amount of water loss due to seepage computed was 6,474,340.80 m³/year. By selecting three different sections of the dam during the model analysis, the average seepage flux computed at the normal pool level (NPL) and actual reservoir level (ARL) for every meter length of reservoir were 3.304E-05 m³/sec/m and 2.252E-05 m³/sec/m respectively. But the value computed during designing of the project where only one section of central foundation was considered, was 1.031E-07 m³/sec/m, which ignores the two left and right foundations which are found to be pervious and critical to the water tightness of the dam. The simulation results of SLOPE/W revealed that the downstream under steady state seepage, upstream and downstream under steady state seepage with earthquake, rapid draw down and rapid draw down with earthquake slope factor of safety are 1.805, 1.423, 1.366, 1.883 and 1.203 respectively, indicating the dam is safe against slope failure and the amount of shell materials used for the construction of the dam were excess. But the uncontrolled excessive seepage can have an adverse impact on the constructed earth dam; for which the stability of the dam against the excessive seepage needs to be assessed regularly.

Keywords: *Gerebsegen dam, Earth dam, Seepage analysis, Slope stability, Geo-Studio Software (SEEP/W, and SLOPE/W).*

1 INTRODUCTION

1.1 Background of the Study

The construction of dams ranks with the earliest and most fundamental of civil engineering activities. All great civilizations have been identified with the construction of storage reservoirs appropriate to their needs, in the earliest instances to satisfy irrigation demands arising through the development and expansion of organized agriculture. Dam is a hydraulic structure of fairly impervious built across a river to create a reservoir on its upstream side for impounding water for various purposes. These purposes may be for any of several reasons, such as water supply, irrigation, energy generation, flood control, navigation fishing and recreation. Many dams fulfill a combination of these functions (Novak et al., 2007).

There are two types of modern dam namely embankment dam and concrete dam. Embankment dam which derives its strength from position, internal friction and mutual attraction of particles, is a water impounding structure constructed from fragmental natural materials excavated or obtained close to the dam site. The natural fill materials are placed and compacted without the addition of any binding agent, using high capacity mechanical plant, which rely on their weight to resist the flow of water, just like concrete gravity dams (Jansen, 2006).

Relative to concrete dams, embankment dams offer more flexibility; and hence can deform slightly to conform to deflection of the foundation without failure and it consist of homogeneous earth-fill dam, zoned earth-fill dam, and rock-fill embankment dam. An earth fill dam is constructed primarily of engineering soils compacted uniformly and intensively in relatively thin layers and at controlled moisture contents. Thus, earth fill dams, constituent over 50% selected soil material. On the other hand, rock fill embankment dams are constructed of rock fill materials which contain over 50% of compacted rock fill materials. But concrete dams are classified in two three main categories. These are gravity, arch and buttress dams (Arora, 2002; ICOLD, 2007). Generally several factors are considered in selecting the type of dam such as site conditions, topography, geology and foundation conditions, material availability, environmental and economic conditions. Dam body and its foundation must be sufficiently watertight to control seepage and maintain the desired

reservoir level, the dam materials and foundation must be strong to resist load, it must have sufficient spillway, and outlet works capacity and freeboard to prevent floodwater from overtopping the dam crest (USBR, 1987).

Water molecules flow in the soil porous media due to their potential energy and gradually lose their energy because of friction in this environment. This phenomenon is called water seepage in the soil porous environment which has undesirable effects are such as loss of water stored behind the dam structure, generation of pore pressure in porous media and reduction of effective stress between soil particles and thus reduction of its shear strength, creating uplift pressure, movement of soil particles and occurrence of internal erosion in the environment. So that seepage behavior and control is very important to focus of earth dam, and is one of the major factors of accidents. This seepage can cause weakening in the earth dam structure, followed by a sudden failure due to piping or sloughing, seepage through earth dam is difficult to analyze especially dams with multiple zones (Dahande and Tahri, 2015).

Many embankment dams are constructed in Northern Ethiopia most of which are used for irrigation purposes. However, their capacity reduces frequently before their expected design life due to different reasons. As different researches indicate, among the different types of known causes of dam failure, the main cause of dam capacity reduction in the regions is seepage failure. Dams with excessive seepage through their dam bodies or abutments may face failure or collapsing within short or long time depending on the amount of water seeping and/or location of seepage exit point from the reservoir (Mohammed, 2009).

Gerebsegen dam is one of the dams constructed in the northern part of Ethiopia with seepage dominantly through the foundation and abutments were observed. In the long run, this seepage may have an effect on the stability of the dam itself, which can endanger the existence of the dam. Gerebsegen dam was constructed for the purpose of irrigation and water supply of Mekelle city, the capital city of Tigray region. Any impact or failure to this dam will have a negative impact on the water supply of the city and irrigation lands found in the downstream side of the dam. This may lead to a water use conflict between the farmers and the city. In order to avoid the aforementioned problems and impacts on the dam, the degree or level of

stability problems which can be resulted from the excessive amount of seepage must be analyzed. Analyzing of the impacts can help draw or provide remedial measures if needed.

1.2 Statement of the Problem

Gerebsegen dam is one of the dams constructed in the northern part of Ethiopia for the purpose of irrigation and water supply for the city of Mekelle. As it is observed after completing its construction, seepage from the dam dominantly through foundation and abutments was observed. This seepage should be measured using appropriate methods in order to see whether it is excess enough to have an impact on the dam itself and/or on the amount of water to be used for the city water supply and irrigation purpose. In addition, this excessive seepage may have an effect on the stability of the dam itself, which can endanger its very existence. In order to avoid the aforementioned problems and impacts on the dam, the degree or level of stability problems which can be resulted from the excessive amount of seepage must be analyzed. Analyzing the impacts can help to draw or provide remedial measures if needed.

1.3 Research question

- What is the main casual source of seepage problem of Gerebsegen earthen dam?
- Is the quantity of water seeping through the body of the dam and its foundation within allowable limit?
- How is the effect of seepage on the embankment dam stability?

1.4 Research Objective

1.4.1 Objective

The main objective of the research is to evaluate the performance of an embankment dam related to seepage analysis and slope stability.

1.4.2 Specific objectives

- To assess the seepage problems and measures the actual existing seepage flux
- To determine the design seepage quantity through the body of the dam and its foundation and the pore water pressure distribution using numerical model

- To determine the slope stability of the upstream and downstream of the dam under steady state or normal reservoir operation and sudden drawdown conditions

1.5 Scope of the Research

The scope of this research focus on performance evaluation of Gerebsegen earth-dam after analysis of seepage and slope stability at steady and transient state condition using Geo-Studio 2007 software and identifying the seepage sources based on the water tightness of the geological formation and seepage impacts on the dam and its function and finally propose possible remedial measures.

1.6 Significance of the study

Many seepage problems and failures of embankment dams have occurred because of inadequate seepage control measures, poor/incomplete treatment and preparation of the foundation and abutments. Seepage can lead to piping and sliding, both of which can lead to dam failure. If seepage occurs without dislodge and removing soil/rock particles, no structural damage will result. However, if soil/soluble rock particles are eroded away in seepage, severe problems may develop. The evaluation of safety of earth dams depends on the proper design, construction and monitoring of the actual behavior during the construction and operation of the structure.

The significance of this research study is to address, investigate and evaluate the performance of Gerebsegen dam in relation to seepage analysis and to assess the main problem that can lead to failure of Gerebsegen dam so as to device a mitigation measures. In addition to this, the research output is expected to be used as a guide in planning and designing of new similar projects and make proper maintenance of existing once. It is also helps new research works as spring board for other researchers of similar topic for other dams constructed in other areas.

2 LITERATURE REVIEW

2.1 General

Embankment dams are the oldest and most common wide spread type of dams accounting about 75% the world dams. Thousands of embankment dams exceeding 20meters in height have been constructed throughout the world. Currently, China is the leader in embankment dam construction. It was compared by various forms of concrete dam, but developments since the 1930s in geotechnical science, understanding of behaviors through instrumentation and improvement of earth-moving machinery has made it therefore most type of dam through the world. The significance of the embankment dams emanates from its important to mankind in both technically and economically feasible, due to easily available construction materials, simple design, a variety foundation relative to concrete dam (ICOLD, 2007). In order to achieve the desired target from embankment dams, care must be taken in understanding the causes of dam failures so as to have proper design and construction process. As embankment dams are mainly conditional by the potential risks arising from backwater and water accumulation, the development of seepage through the body, foundation and abutment of a dam can provide basic information on the state of the art of the hydraulic structure and on the possibilities of its safe operation. Moreover, this can help improving methods of seepage and stability analysis which can lead to safe slope construction and proposing proper remedial measures after construction of the target dam.

2.2 Causes of Failure of Earth Embankment Dam

An embankment dam failure is defined as collapse or movement of part of a dam or its foundation, so that the dam cannot retain water. In general, a failure results in the release of large quantities of water, imposing risks on the people or property downstream. The failure mode of an embankment dam is directly connected with the type of cause of failure and the type of the dam. Failures of earth dams are mainly caused by the improper design, lack of investigation, inadequate care in construction and poor maintenance. Various causes of failure can be grouped in to three categories: hydraulic failures, seepage failure and structural failures. Based on the studies on 300 dam failure and it is concluded that 35% of the world's dam failure is caused by direct overflow due to flooding or loss of free board; other 25% of

failure is caused because of foundation problems such as excessive seepage, abnormal increases of pore-pressure and internal erosion and also improper design and construction contribute the remaining 40% (Biswas and Chatterjee, 1971; Ambikaipaha, 2011).

Hydraulic failures occur when the water level surpasses the height of the dam crest, causing it to spill over the dam. Case histories reveal that inadequate capacity or partly/fully blocked spillway, earthquake forces, insufficient estimation of the amount of flooding has often led to the failure of embankment dams (Novak et al., 2007; Tigist, 2008).

2.2.1 Structural failures of the Embankment dams

Structural and slope instability can cause embankment dam failure. Instability of a dam is a serious problem and can cause sliding and movement of the embankment or foundation. Sliding can occur in embankment dams from slopes that are too steep, high pore pressures due to inadequate drainage, and loss of shear strength due to liquefaction of loose granular materials. This can lead to development of sloughs, cracks or other irregularities in the embankment which are generally sign of serious instability and may indicate structural failure (Narita, 2000).

2.2.2 Seepage failures of Embankment dams

Seepage is one of the most important issues considered during design, construction and maintenance of dam. The volume of seepage does not only mean losses of water, which are of a certain hydraulic significance, but seepage also has a certain hydrodynamic effects. This can result in destruction of the stability of dam body or subsurface of the dam and thus endanger the safety of its hydraulic structure and operation as a whole. Most dam designers considered the allowable seepage of embankment dam to be less than $0.03 \text{ m}^3/\text{sec}$ is satisfactory at the normal pool level or full reservoir level. The high rate of leakage (more than $0.06 \text{ m}^3/\text{sec}$) through earth cores is not generally acceptable as it could promote piping effects (Jansen, 2006). Many seepage problems and failures of dams have occurred worldwide because of many reasons (Gebremedhin et al., 2016; Von Thun, 1996). Von Thun (1996) indicated that in the western sector of the USA 60% of earth dams higher than 15 m (in height) have failed due to the process of piping/leakage and their cause of inadequate seepage control measures or

incomplete cleanup and preparation of the cores, foundations, and abutments are among factors which can result in dam failure. In 1954, Middlebrooks specifically identified seepage problems through the bodies and subsoil of dams as the second most frequent cause of failures and breakdowns of dams in the USA, reaching a proportion of 30.5% (Lukac, 2006). His study result was also supported by ICOLD statistics which summarized the causes of failures and breakdowns of dams in Western Europe and the USA for the period of 1900 -1975, and he determined a 38% rate of seepages from piping in failures and breakdowns of dams.

The main causes of seepage problems may lead to the following dam failures. The first one is when seepage starts through poor soils in the body of the dam, small channels are formed which transport material downstream. As more materials are transported downstream, the channels grow bigger and bigger which could lead to wash out of the dam. The second is when highly permeable cavities or fissures or strata of gravel or coarse sand present in the dam foundation and/or abutments, it may lead to heavy seepage. The concentrated seepage at high rate will erode soil which causes increase flow water and soil. As a result the dam will settle or sink leading to failure. The third is the process of failure due to slough starts when the downstream toe of the dam becomes saturated and starts getting eroded, causing, small slump or slide of the dam. The process of saturation and slumping continues, leading to failure of the dam. Some failures studied those at Baldwin Hills, Teton, and Malpasset had all been caused by complex seepage problems involving the dam and foundation contacts (Jansen, 2006).

In the last two decades, 54 micro-dam reservoirs have been constructed in Northern Ethiopia to fight the recurrent drought and improve agricultural productivity through irrigation. About 60% of micro-dam reservoirs are constructed around Mekelle area; excessive leakage through dam body, foundation and abutment is found to be one of the major problems identified (Gebremedhin et al, 2013). They stated that the hydraulic conductivity of the alternating sequences of the limestone-shale-marl intercalation unit ranges from 10^{-4} to 10^2 cm/s and was found to be responsible for the excessive leakage of the Hashenge and Arato microdams in the Mekelle Outlier. This indicates a special attention is needed to the issues of excessive seepage within the dam body and foundation or around it. Gebremedhin et al., 2016 were also studied that micro-dam reservoirs constructed on sedimentary rocks in the region have the geotechnical problems related to the engineering and hydro-geological properties of

limestone-shale-marl intercalation widely outcropped in the Mekelle outlier. One of the failures studied at Hizaetiwedicheber dam (Mekelle area); where the present study area is located in the downstream of Hizaetiwedicheber dam identified that limestone of the area is the main aquifer system with highest hydraulic conductivity value ranging from $1.226E-04$ m/sec (10.6 m/day) to $3.634E-04$ m/sec (31.4 m/day).

Haregewoin (2006) studied the major seepage problems of the micro dams in Mekelle area which indicated about 60% of the studied reservoirs lose water by excessive seepage through the dam foundation and abutment into the bedrock. These were easily observed downstream of the dams where new springs have developed and the base flow of the existing streams increased in the dry season. This excessive seepage was related to the geological formations on which the reservoirs were built. About 50 percent of the dams are located on thin Agulae shales, overlaying a highly jointed Antalo shale and limestone intercalation.

2.3 Impact of Seepage on Embankment Dam

As to the Jansen, (2006) and Dahande and Tahri, (2015) excessive seepage is critical impact and significant destructive nature during and after construction of embankment dams. As far as the impact on dam safety of seepage into and through the foundation, permeable soil conditions can be interpreted to either enhance dam safety or act against it. Embankment could pose danger to human lives and properties due to some extent for their failures due to overtopping, piping and other factors. Some of the more critical consequences include: 1) Breaching of the embankment from slope instability induced by loss of material and/or shear strength. 2) Loss of significant amounts of water stored behind the dam structure (Dahande and Tahri, 2015). 3) Movement of soil particles and occurrence of internal erosion in the environment (Rahmani, 2009). 4) Maintenance problems or loss of useful areas on the downstream slope or areas downstream of the embankment.

According to Fell et al. (2005), in their excellent textbook about geotechnical engineering of dams, four conditions must exist for internal erosion and piping to occur. These are: “1) There must be a seepage flow path and a source of water; 2) There must be erodible material within the flow path and this material must be carried by the seepage flow; 3) There must be an

unprotected exit, from which the eroded material may escape and 4) For a pipe to form, the material being piped, or the material directly above, must be able to form and support 'roof' for the pipe".

2.4 Seepage Control Measures

All embankment dams are subject to some seepage passing through, under, and around them. Seepage control is necessary to prevent excessive uplift pressures, instability of them downstream slope, piping through the embankment and/or foundation, and erosion of material by migration into open joints in the foundation and abutments. If uncontrolled, seepage may be detrimental to the stability of the structure as a result of excessive internal pore water pressures or by piping (Berrones and Acosta, 2011). For existing dams, records or evidence that seepage flows have removed any significant degree of fine grained material must be evaluated. Any such records require further field investigation. Seepage discharge should be effectively controlled to prevent structural damage or interference with normal operations.

As to the Singh and Varsheny, (1995) in the evaluation of seepage reduction or seepage control measures to dam safety the keys to controlling seepage to prevent piping are 1) the protective control measures such as relief wells, weighted graded filters, horizontal drains, or chimney drains, toe drains, filters and transition zones designed to prevent movement of soil particles that they control and safely discharge seepage for all conditions. 2) Contacts of seepage control features with the foundation, abutments, embedded structures, etc., designed to prevent the occurrence of piping and/or hydro fracturing of embankment and foundation materials. 3) Grouting, cut-off trenches, and upstream impervious blankets (Singh and Varsheny, 1995). In many existing dams there are unfiltered seepages outlets which are uninspectable because they are located under the tail water on the downstream side of the dam (USACE, 1993). Generally, for existing embankments, all seepage records compiled during the existence of the structure should be reviewed for significant trends or abnormal changes. The causes of any abnormalities should be determined as accurately as possible.

2.5 Seepage in Embankment Dams and Importance of Seepage Analysis

Dams are valuable but potentially dangerous assets if their problems are not solved on time. A minor problem can turn into a major reconstruction project or even result in a complete dam failure. The rate of seepage at which water moves through the embankment depends on the characteristics of soil in the embankment, how well it is compacted, the foundation and abutment preparation, and the number and size of cracks and voids within the embankment. Many seepage problems and failures of earth dams have occurred because of inadequate seepage control measures. The forces resulting from the seepage of water through an embankment have a considerable effect on dam stability. Seepage control is necessary to prevent excessive uplift pressures, instability of the downstream slope, piping through the embankment and/or foundation, and erosion of material by migration into open joints in the foundation and abutments (Tigist, 2008; Berrones and Acosta, 2011).

If seepage is uncontrolled, it affects the stability of dam. Because of this reason Seepage is very important, as it affects the stability of dam. Because of its importance, the determination of the seepage through an earth dam has received a great deal of attention, to design the seepage mitigation measures and limits the position of phreatic line. If the surface seepage intersects the face of the dam, erosion may result and possible failure of the dam. Thus, it is necessary to measure the level of phreatic line and rock fills are used at the downstream toe or gravel blankets to intersect the line of seepage before it reaches the downstream toe. On the other hand, the amount of seepage and water loss through the dam foundation and body as well as calculation and determination of its value and also methods to prevent and reduce the amount of seepage and water loss are particularly important (Ersayin, 2006).

2.6 Seepage Analysis of Embankment Dam

Seepage is the main aspects and its control enjoys main position in designing, construction and maintenance of any dam. Design of embankment dam which consider the most important features are required foundation treatment plan, abutment stability with water tightness, seepage conditions, stability of slopes adjacent to control structure approach channels and stilling basins, stability of reservoir slopes, and ability of the reservoir to retain the water

stored. These major features should be studied with reference to field conditions and to various alternatives before initiating detailed stability or seepage analyses. This makes design of an embankment dam complex because of the unknown materials property of foundation. Earth dams should be designed to utilize available material to the best advantage and to conform to actual conditions at site. Detail design sometimes will be influenced heavily by the strengths of foundation and construction materials (Novak et al., 2007).

Seepage analysis is necessary to determine the quantity of water passing through the body of embankment dam and its foundation. The distribution of pore water pressure can also be obtained by the seepage analysis. Seepage occurs through all earth dams. If the foundation is pervious, seepage also occurs through it. Seepage is one of the important factors causing failure of embankment dam and hence seepage analysis of embankment dam is of greater importance. Some seepage problems can be evaluated through the use of simplified methods depends on saturated flow theory and highly idealized conditions may be appropriate for preliminary evaluations of seepage issues (Gopal, 2014; Dahande and Tahri, 2015).

Various methods like analytical and numerical were used for calculating flow rates (seepage), hydraulic gradients, water pore pressure, and flow velocities from dam. Among the methods, analytical method previously used by researchers like Schaffernak (1917), Casagrande (1937). In general, exact and analytical solutions are laborious when geometric, hydraulic and boundary conditions become complex. It requires many assumptions and only simple and straight forward seepage problem can be solved. Nowadays, numerical methods are preferred with increasing frequency due to their easy adaptation and automation to widely varying conditions, and in general because of their capability for solving complex problems. Numerical methods have been applied by different authors (Freeze 1971; Lam et al., 1987; Huang and Jia, 2009). Numerical techniques such as finite element method and finite differences are used to determine the seepage through the earth dam. Among these numerical methods, finite element method is the most popular and widely used method (Berrones and Acosta, 2011). Various software used in the seepage analysis of an embankment dams are: SEEP/W, MODFLOW, PLAXIS, Flac3D, ANSYS, PDEase2D, SVFLUX etc (Abhilasha and Balan, 2014).

Fakhari and Ghanbari, (2013) have used finite element to solve seepage problem of almost 600 geometric models of embankment dams with clay core using both numerical and analytical methods, the obtained result exhibits numerical (SEEP/W) of seepage analysis was more accurate about 18%. Since most of the engineering problems have a relatively complex boundary condition, using numerical methods for solving the differential equations are very common. This is because of its important advantage of fast solving any complex seepage problems of earth dam; easy availability of personal computers/laptops, and availability of commercial software at reasonable cost; contribute to making numerical modeling solutions more advantageous over physical modeling in many real life situations.

2.6.1 SEEP/W Software

Numerical modeling can be a useful tool in geotechnical engineering since a wide variety of problems is theoretically possible to model. Embankment dams, that geometrically are quite simple structures, are especially attractive to model numerically in order to get an early notion of potential hazardous processes, not visible on the outside but that can be in progress inside the dam. The main problem with numerical modeling is that those performing such analyses normally need specialist knowledge in a wide range of technical subjects. At the same time, numerical software is within reach for almost anyone with a computer. Not seldom, commercial software are extremely user-friendly which makes it easy to produce results, even wrong ones, often presented in the form of colorful plots (Fell et al., 2005).

SEEP/W (SEEPage for Windows) is a finite element software product that is coming under Geo-studio, used to model the movement of water and pore-water pressure distribution within porous materials like soil and rock. It is formulated on the basis of flow of water through saturated soil and follows Darcy's Law. The numerical model SEEP/W can be employed to carry out simulation of seepage and phreatic surface in homogenous and non homogenous earthen dams (SEEP/W, 2008; Arshad and Babar, 2014). The software is capable enough to resolve the problems ranging from simple saturated steady state issues to saturated/unsaturated time dependent problems (transient state). Good quality graphic output allows a visual display of equipotential lines and flow paths, from which contours can be plotted for different properties like pore pressures, seepage velocities, and gradients. With SEEP/W simulation, the

seepage performance can be obtained and the materials properties of embankment and its foundation such as effective hydraulic conductivity and hydraulic gradient which are useful for indicating the potential for internal erosion to initiate can be revealed (Novak et al., 2007;SEEP/W, 2008).

According to Hasani et al., (2013) a research on seepage analysis in Ilam earth fill dam has been done by SEEP/W software. In order to evaluate the type and size of mesh size on the total flow rate and total head through the dam cross section, four mesh size such as coarse, medium, fine and unstructured mesh is considered. Result showed that average flow rate of leakage under the different mesh size for Ilam dam equal 0.836 liters per second for the entire length of the dam. Noori and Khaleel, (2011), during their analysis work on seepage and stability of Duhok dam have used a finite element method through a computer program, SEEP-2D, to determine the free surface seepage line, the quantity of seepage through the dam, the pore water pressure distribution, the total head measurements and the effect of anisotropy of the core materials.

According to the work of Farzin et al., (2013), it has been compared SEEP/W soft ware simulation results with Schaffernak and Casagrande's analytical solutions, which showed that numerical solution, give more seepage rate than the two others solutions. Fakhari and Ghanbi also conducted on seepage estimation for 120 earth dams by seepage model and found $1.271 \cdot 10^{-2}$ cumecs/m and $8.53 \cdot 10^{-6}$ cumecs/m the recommended as suitable method (Fakhari and Ghanbari, 2013). According the allowable resulting discharge of seepage loss shouldn't exceed 0.56 liters/minute/meter (9.33×10^{-6} m³/sec/m) for high earth dams greater than 40 meter (Look, 2007).

In Ethiopia, many researchers have conducted researches on seepage analysis of embankment dams: like (Abebe, 2014; Tigist, 2008, Bahru, 2014) worked on estimation of seepage rate through the dam body and its foundation using SEEP/W. They found that the seepage through Kesem embankment dam in Afar region, Zana Med Micro dam (Amhara Region) and Welkay Rock fill dam (Tigray Region) and found seepage flux of $1.274 \cdot 10^{-5}$ m³/s/m., $3.18 \cdot 10^{-8}$ m³/s/m and $2.9 \cdot 10^{-5}$ m³/s/m respectively. In view of all above facts, the present research work

is designed to model seepage analysis of an earthen dam by using finite element (SEEP/W) approach.

2.6.2 Seepage Discharge measurement

Seepage measurement is necessary to make regular measurements of total leakage as well as the paths and amounts of individual leakage from the dam body and its foundation or its abutments. It is to emphasize that routine observation of where seepage is emerging can be as useful a guide as the actual measurement. However, as indicated by most of the reporting countries in ICOLD (1989) and ANCOLD (1983), it is usual to measure the quantity of seepage. It is preferable to collect the seepage close to the downstream toe of the impervious zone, and to isolate areas from each other so the readings are not excessively by flow through rock fill zones and runoff from abutments. The authors know of cases where drainage trenches to subdivide the foundation into collection areas under rock fill were filled rapidly with fines from the rock fill, making the system ineffective. The portion of the seepage that appears at the ground surface may be collected by ditches or pipe drains and measured by means of weirs, current meter, float flow or other similar devices (USBR, 1997; Singh and Varsheny, 1995).

Current meter method: The first discharge measurements in Australia were performed in the 1950's using a current meter device now day in the whole world is used standard instrument for discharge measurement. The current meter was mainly performed with the stream for method due to the fact that about 70% of the gauge in Styria, Australia is situated at more or less shallow rivers. It is very important to find the right measuring parameters as varying from river to river and having an important influence to the accuracy of the seepage discharge measurement. The sensor can be used on the rod as well as on cableway devices. Its big advantage is that the flow velocity in the verticals (Geopacks, 2001).

Float method: If a flow meter is not accessible, floats can be used to estimate stream discharge which is simple and inexpensive. The process involving in the float method of measuring velocity is by observing the time for a floating body to cross a known length and noting its position in the channel. The surface float moves with the same velocity as the

surface of the water. A subsurface float consists of a submerged float attached to a surface float by an adjustable line and it measures directly the mean velocity (Joy et al., 2005).

2.7 Slope Stability Analysis of Embankment Dam

Embankment dam stability must be assessed in relation to the changing conditions of loading and seepage regime which develop from construction through first impounding into operational service, including reservoir drawdown. The soil mass in an embankment dam, because of its slope is subjected to the actuating (or driving) forces which tend to cause movement (or sliding) of the soil mass. This movement is resisted by the stabilizing (or resisting) forces which develop at the potential sliding surface because of the shear strength of the soil. The side slopes of earth dam will remain stable if the sum the resisting forces on every possible surface of failure (or surface of slippage) is greater than the sum of the actuating forces. The side slope of an earth dam should be safe against shear failure (Novak et al., 2007).

The major methods for determination of this slope stability are Limit equilibrium method and finite element methods. Limit equilibrium method is the simplest methods of analysis employ a single simple algebraic equation to compute the factor of safety. However, the finite element methods are applied to determination of stresses and deformations in excavated slopes and embankments. It has also used to calculate the factor of safety defined in the same way as that used in limit equilibrium analysis (Duncan et al., 2015).

2.7.1 Limit Equilibrium Analysis

In the conventional limit equilibrium approach, the stability of a slope measured by factor of safety (F_s), which is defined as the ratio between the shear strength of the soil to shear stress required to maintain the equilibrium of the slope (Duncan et al., 2015). A slip surface, which can be planar, circular or non-circular in shape, is required to be assumed prior to the equilibrium analysis. At that point of failure the shear strength is assumed to be fully mobilized along the slip surface and F_s is assumed to be constant for the entire slip surface. The stability analysis eventually involves an iterative process until the critical slip surface is found out where the critical slip surface is defined as the slip surface with lowest F_s . Over the

years, many studies have been conducted to investigate the computational accuracy of different limit equilibrium methods and to develop techniques for searching the critical slip surface (Burman et al., 2015).

Several limit equilibrium methods (LEM) have been developed for slope stability analyses. Fellinius, (1936) introduce the first method with the assumption of forces acting on sides of any slice are neglected, referred to as the Ordinary or the Swedish method, for a circular slip surface. Bishop (1955) developed an equation for factor of safety calculation. The method included the inter-slice normal forces but ignored the inter slice shear forces Bishop's simplified method satisfies overall moment equilibrium but not the horizontal overall force equilibrium. The Bishop's simplified method does not satisfy the complete equilibrium, although it gives relatively accurate results for factor of safety. Janbu (1956) developed a simplified method only satisfies the overall horizontal equilibrium and this method does not satisfy the overall moment equilibrium. It included interslice normal forces, but ignored the interslice shear forces. The force polygon closure is better than the Bishop's Simplified method but the factor of safety is too low. Spencer's (1967) method (SM considers both shear and normal interslice forces. Spencer satisfies both satisfies both moment and force equilibrium. He adopted a constant interslice force function between the interslice shear and normal forces. Morgenstern and Price (1965) developed a method similar to the Spencer method, but they allowed for various user-specified interslice force functions (constant, half-sine, data-point specified...). It considers both shear and normal interslice forces, satisfies both moment and force equilibrium, and allows for a variety of user-selected interslice force function (Pasetto, 2014).

2.7.2 SLOPE/W Software Model

In the past various computer-based geotechnical soft-wares have been developed and performed for the slope stability analysis; and those Geo-Slope Software mainly utilize two methodologies, the limit equilibrium models and the finite element analysis based on appropriate soil models which produce great interest both of researchers and of professionals. Among those software, a computer software SLOPE/W is one of the best and has been developed by GEO-SLOPE International Canada, by Fredlund at the University of

Saskatchewan (Krahn, 2004). SLOPE/W is one component in a complete suite of geotechnical products, one of whose powerful features of the integrated approach is that it opens the door to types of analyses of a much wider and more complex spectrum of problems, including the use of finite element computed pore-water pressures and stresses in a stability analysis (SLOPE/W, 2008). In addition, in SLOPE/W five kinds of the limit equilibrium methods are available; these are Ordinary Method, Bishop's Method, Janbu Method, Morgenstern-Price Model, and Spencer Method. The safety factors in the stability analysis by the Limit Equilibrium method as advised by Jasim et al., (2017) have been obtained based on the Morgenstern Method (Morgenstern, 1965). Because of considering the synchronous effect of all the forces, it usually concludes a lower safety factor and has a less error percentage than other methods (Mahdi, 2016).

According to Hasani et al., (2013) a case study on Ilam dam by numerical modeling using Finite Element method and Geoslope software, the stability of embankment dams was analyzed to eliminate defects in the Limit Equilibrium method. During this study, by the use of Geo-slope software, the safety factors against slip and rupture on the slopes of an embankment dam with fixed specifications and geometry were investigated in two ways of Entry and Grid and Radius and exit.

In Ethiopia, many researchers have conducted, like (Abebe, 2014; Bahru, 2014) interested in their research into the subject of the influence of several parameters on the factor safety for slope stability. In view of all above facts, the present study of the factor of safety is calculated using Limit equilibrium methods and Finite element method at different loading condition of an embankment dam.

2.7.3 Loading Condition for stability analysis

The stability of the upstream and downstream slopes of the dam embankment is analyzed for the most critical or severe loading conditions that may occur during the life of the dam. The standards of the slopes must be stable during all conditions of construction phase, full reservoir condition, including rapid drawdown, earthquake, and seepage flow through the embankment, foundation and abutment must be controlled. Different agencies suggest

minimum values within the range of 1.1 to 1.5 depending upon the risks involved. At each of these stages, the mechanics of failure is different and slightly different analytic techniques are used (Novak et al., 2007). The minimum (critical) safety factor for slope stability is different based on the following loading conditions:

End construction stability: - Slope stability during and at the end of construction is analyzed using either drained or untrained strengths, depending on the permeability of the soil. For many embankment slopes the most critical condition is the end of construction. In some cases, however, there may be intermediate conditions during construction that are more critical. In some fill placement operations, including some waste fills, the fill may be placed with slope geometry such that the stability conditions during construction are more adverse than at the end of construction (Duncan et al., 2015).

Steady state seepage condition: - Over time after construction the soil in slopes may either swell (with increase in water content) or consolidate (with decrease in water content). Long-term stability analyses are performed to reflect the conditions after these changes have occurred. Shear strengths are expressed in terms of effective stresses, and the pore water pressures are estimated from the most adverse groundwater and seepage conditions anticipated during the life of the slope. The downstream slope of an embankment dam is more critical for this loading condition (USACE, 2003).

Sudden (Rapid) drawdown condition: Rapid or sudden drawdown is caused by a lowering of the water level adjacent to a slope, at a rate so fast that the soil does not have sufficient time to drain significantly. The potential for upstream failure due to rapid drawdown occurs in embankments constructed of cohesive soils in which excess pore water pressures dissipates at a slow rate than the drop in the reservoir level. These excess pore pressures can lead to reduced shear strength of the embankment soils and slope failure (Duncan et al., 2015). During rapid drawdown, the stabilizing effect of the water on the upstream face is lost, but the pore-water pressures within the embankment may remain high. The case, in which the water has to be quickly lowered in presence of impermeable material, but low permeability materials take a long time to drain in the dam slope, so that the phreatic line falls very slowly, is common problems (Tran, 2004;in Passtto, 2014).

Earthquake (Pseudo static) Slope stability analysis: - Dynamic loads generated by seismic disturbances must be considered in the design of all major dams situated in recognized seismic 'high-risk' regions. Conventional pseudo-static, limiting equilibrium, earthquake stability analysis, a horizontal earthquake force is applied to the sliding body in addition to the static forces. The additional horizontal force is proportional to the total mass of the sliding body, and the factor of proportionality is denoted "earthquake coefficient". This type of analysis is applicable only for dams constructed of materials that do not experience a significant reduction in strength during cyclic loading. For many years, seismic coefficients were estimated based on empirical guidelines and codes. Typical values for seismic coefficients used ranged from about 0.05 to about 0.25(Novak et al., 2007)

3 MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 Location

Gerebsegen Embankment dam is located in Tigray region, southern zone, boundary of Enderta and Hintalowajerat wereda and Maygent Tabia. It is located on river Gerebsegen (sub tributary of the Tekeze River) near to Mekelle city (Figure 3.1). It is geographically located at Longitude 542094 UTM East and Latitude 1482355 UTM North. The dam is found at about 800 km from Addis Ababa and 17 km south west of Mekelle city. It is accessible by all weather roads within the area.

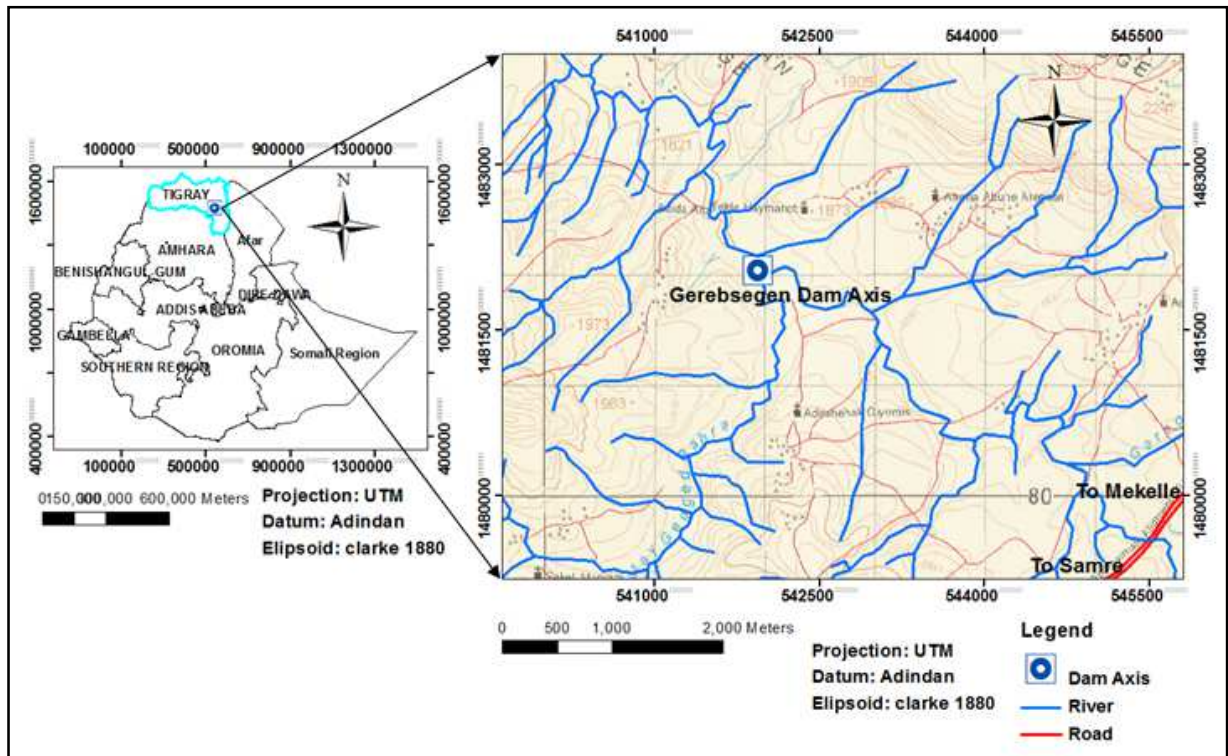


Figure 3.1 : Location map of the study area

3.1.2 Description of the Existing Dam

According to the reports written by TWRB (2014), Gerebsegen embankment dam was built at Gerebsegen River to create a reservoir with an approximate total gross storage capacity of

24.67 Mm³ including a dead storage volume of 6.68 Mm³. The dam with the purpose of water storage for irrigation of about 700 hectares and water supply with the demand of 240 liter/sec was constructed by Tigray water works Construction Enterprise. The dam which was constructed between 2014 and 2016 is zoned type of earth fill dam and has 46 m dam height. The main components of the project include the main dam, two intake outlets for water supply, one intake outlet for irrigation purpose, blanketed upstream reservoir area and side channel spillway. The dam has crest length of 815 m with embankment slopes of 2.5:1 on downstream and 3:1 on upstream side (Figure 3.2). To control seepage losses through the dam body, a clay core having width of 3 m at top with side slope of 0.23:1 was used. The upstream and downstream shells were compacted shale with low permeability. Designed thickness of horizontal blanket drainage is 0.30 m thick of sand at bottom, 0.40 m thick of gravel material at the center, and 0.30 m thick of sand material at the top and total thickness of 1.00 m horizontal drainage system. At the downstream toe of the dam a rock toe combined with a drain was constructed. But the vertical chimney drain has not been designed on both the upstream and downstream side of the core. Due to topographic feature of the site, a natural steep ground favorable for side channel spill way with maximum flood out flow of 290.04 m³/sec is available on the left side of the dam which is under construction (TWRB, 2014).

Table 3.1 : Technical characteristics of the dam (TWRB, 2014).

| | |
|------------------------|--|
| Type of Embankment dam | Zoned Earth Fill dam |
| Embankment Volume | 3.30 M m ³ |
| Purpose | Water supply (7.46 M m ³), Irrigation (9.72 M m ³) |
| Crest Length | 815 m |
| Crest width | 7 m |
| Head water | 41 m |
| Maximum Bottom width | 272 m |
| Normal Pool Level(NPL) | 1859 m.a.s.l |
| Berm width | 3 m (U/S and D/S) |
| Catchment area | 290 km ² |

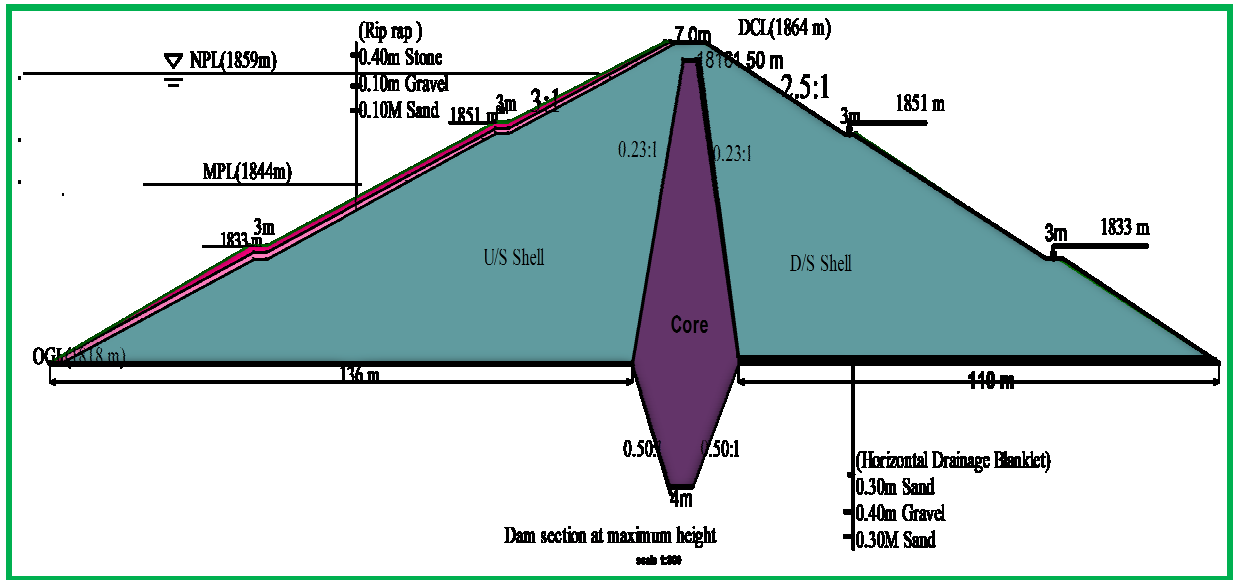


Figure 3.2: Cross section of the Gerebsegen Zoned earth dam (TWRB, 2014)

3.1.3 Topography and Geomorphology

The drainage density and pattern are mainly controlled by the lithology, topography and geological structures of the area. The drainage pattern of the basin can generally be described as dendrite type and flows from East to West direction. Like some of the rivers in the region, it is perennial.

The total catchment area of the dam is 290 km² and which is drained to the Tekeze drainage system. The study area has a wide variety of topographic features ranging from moderate to high hills, faulted blocks, dissected gorges high dolerite hills and alluvial deposits soils in the bed, preferable suitable for the cost earth dam. The dam catchment and command area is characterized by a plain land of command area and high mountainous terrain in the dam catchment. The average elevation of the catchment area has failed about 1920 m.a.s.l and the highland is about 2204.90 m .a.s.l. The command area of this irrigation scheme is found at both sides of the river. This command area is mainly cultivated land and includes uncultivated bush lands (TWRB, 2014).

3.1.4 Climate

Climatic condition of the area is generally classified as semi-arid (Waynadega) zone, with hot winter and relatively short cold summer. The rain season is from June to the end of September and the rest of the year is dry season. The rainfall data for Gerebsegen Dam scheme is taken from Mekelle meteorological station. Hence, the rainfall data of Mekelle is expected to be representative to the study area without significant variation (TWRB, 2014). According to the data taken from Mekelle Meteorological Agency, the mean annual rainfall of the watershed is 555.77 mm.

The monthly mean maximum temperature is 23.5⁰C and the monthly mean minimum temperature 10.4⁰C. The temperature data for Gerebsegen irrigation scheme is taken from Mekelle meteorological station. Actually, there is an elevation difference of more than 250 m between Mekelle and the project study area. Hence, the project area is a bit hotter than Mekelle. Nevertheless, the temperature data of Mekelle is used to represent the study area without any modification (TWRB, 2014).

3.1.5 Geological Foundation of the Dam

Generally important considerations of the dam have to be studied: competent foundation, geological formations and discontinuities, liquefaction prone soils during an earthquake, stability problem, permeability characteristics and differential settlement are the major factors that should be considered thorough the preliminary and detailed investigation. If sedimentary rocks, particularly shale, poorly cemented sandstones and cavernous limestone, they are undesirable, because which are inter bedded and its orientation, leaching of soluble matter, porosity and permeability through the foundation and abutment of the dam. Granites and basalts the type of igneous rock will be much competent for the site of the dam. Yet another point of care should be taken that, the granites and basalts will occur with the sedimentary formations like shale, sandstones and limestone (George, 2008; Turkmen et al., 2001; Asmelash and Meisina, 2015).

According the studies made by ATSWWC (2014), the major geological formations of the project area are mainly Antalo formation, Agulae shale, dolerite and the major soil types are

mainly: alluvial and colluvial deposits and residual soils. Based on the previous study and recent field visit, the geological formation of the target area is very disturbed, and also shows an inter-layering of marl-shale-limestone intercalation with dolerite intrusions (ATSWWC, 2014).

Based on the referred engineering geological mapping and geological structure interpretation, the detailed description on the geological setting of the dam foundation and its vicinity is as follows (Figure 3.3 to Figure 3.5)(ATSWWC, 2014).

- The central parts of the dam foundation site and reservoir area are covered by alluvial deposits. Their thickness varies from 4 m to 10 m in the foundation. It is composed of coarse (dominated by rounded sand, gravel, pebble, and boulders of different origin) and fine materials (silt and clay dominated) of river deposit. The shale-marl-limestone intercalation is observed underlying the alluvial deposit. This unit is observed at river cut and open trench during construction excavated. It is characterized by high degree of weathering and fracturing.
- The two abutments of the dam site have similar lithological and structural setup except small differences in thickness from top to bottom; dolerite sill, shale-marl-limestone intercalation unit and alluvial deposits. While the talus deposits are limited to the top part of the dolerite sills with a variable thickness. The shale is variegated, fissile with variable thickness. The dolerite is mainly outcropped in the right and left abutments of the dam which extends towards the rim of the reservoir area. The mode of occurrence is mainly sill. Generally the dolerite shows high degree of weathering.

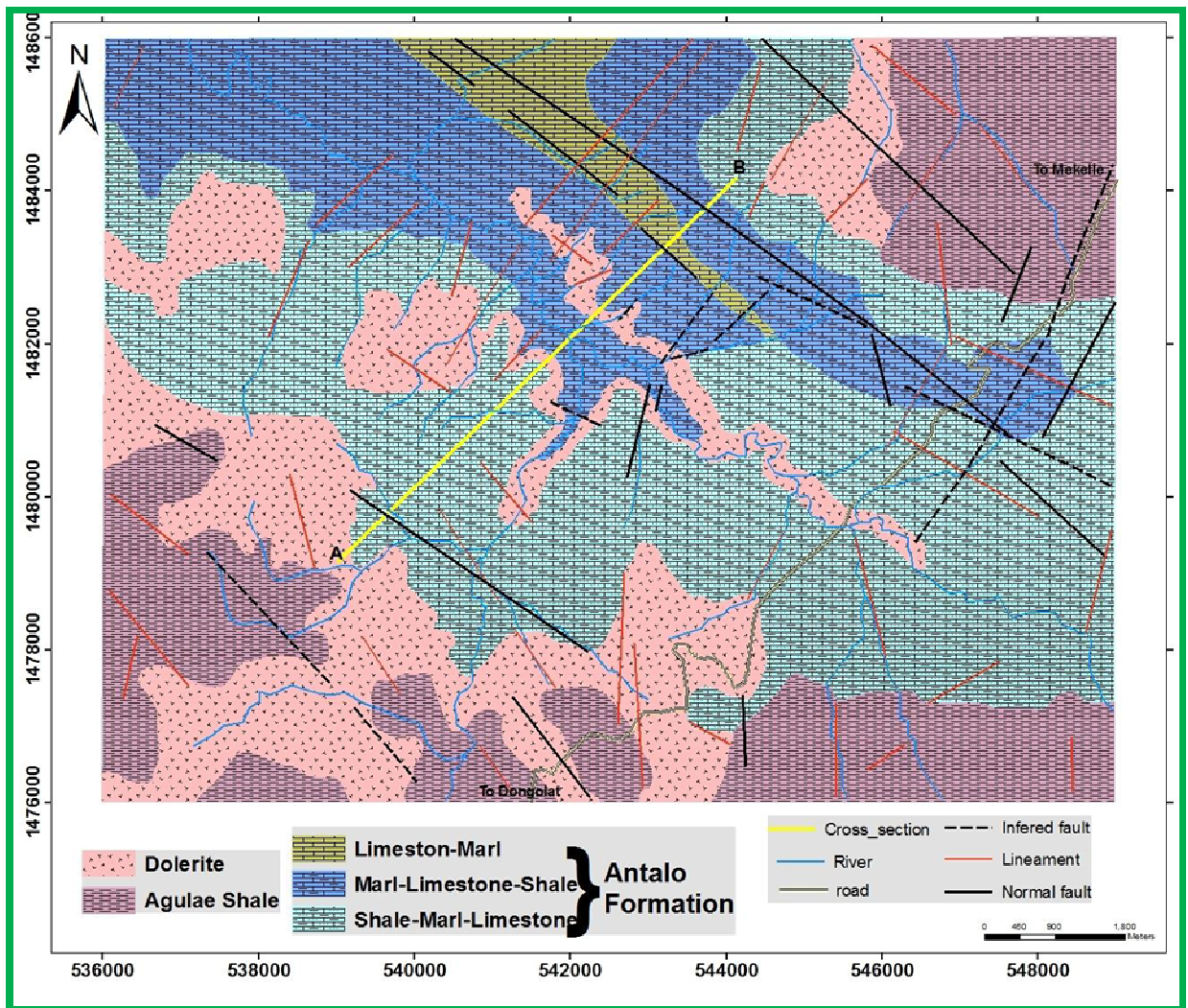


Figure 3.3 : Geological map of the study area (ATSWWC, 2014)

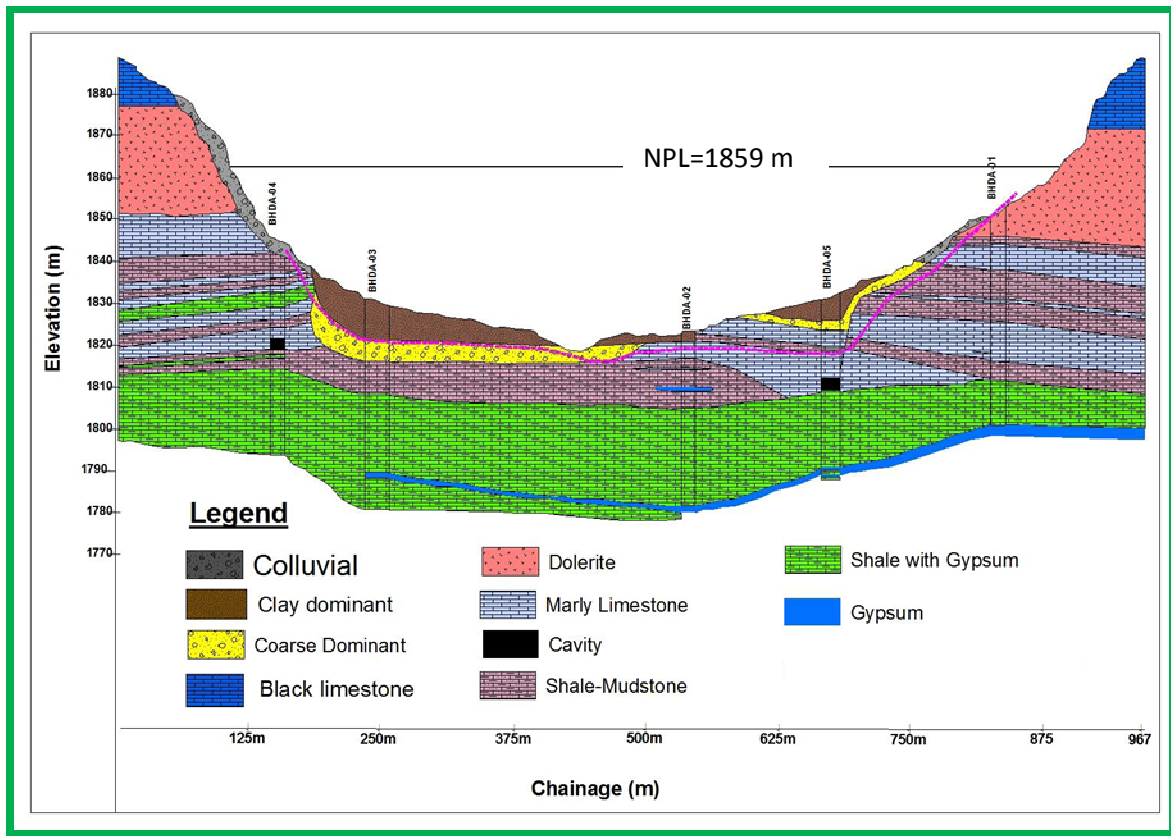


Figure 3.4: Cross sectional profile of the dam axis (ATSWWC, 2014)

Based on the geological map, it is recognized that the vertical succession of the rock layers on the dam axis and reservoir area from top-down is (ATSWWC, 2014):

- Dolerites exposed on the abutment rocks are highly weathered, and interpreted to be highly pervious.
- Shale-marl-limestone exposed on the abutments, reservoir rims and river sections downstream slightly too highly weathered, fractured and interpreted to be highly pervious.
- Quaternary deposits cover the valleys, depressions and flat lands along the reservoir floor. Their thickness is generally variable, but thicker near the river course and thinner toward the steep sides of the flanks and reservoir periphery.

Solution cavities and low angle horizontal bedding planes are very common in the sedimentary rock units of the area.

Generally, the rock units found at the dam site, reservoir area and its surrounding remains highly affected by major and minor fractures. The general orientation of the discontinuity structures on the right and left abutments is not suitable from water tightness perspectives (ATSWWC, 2014).

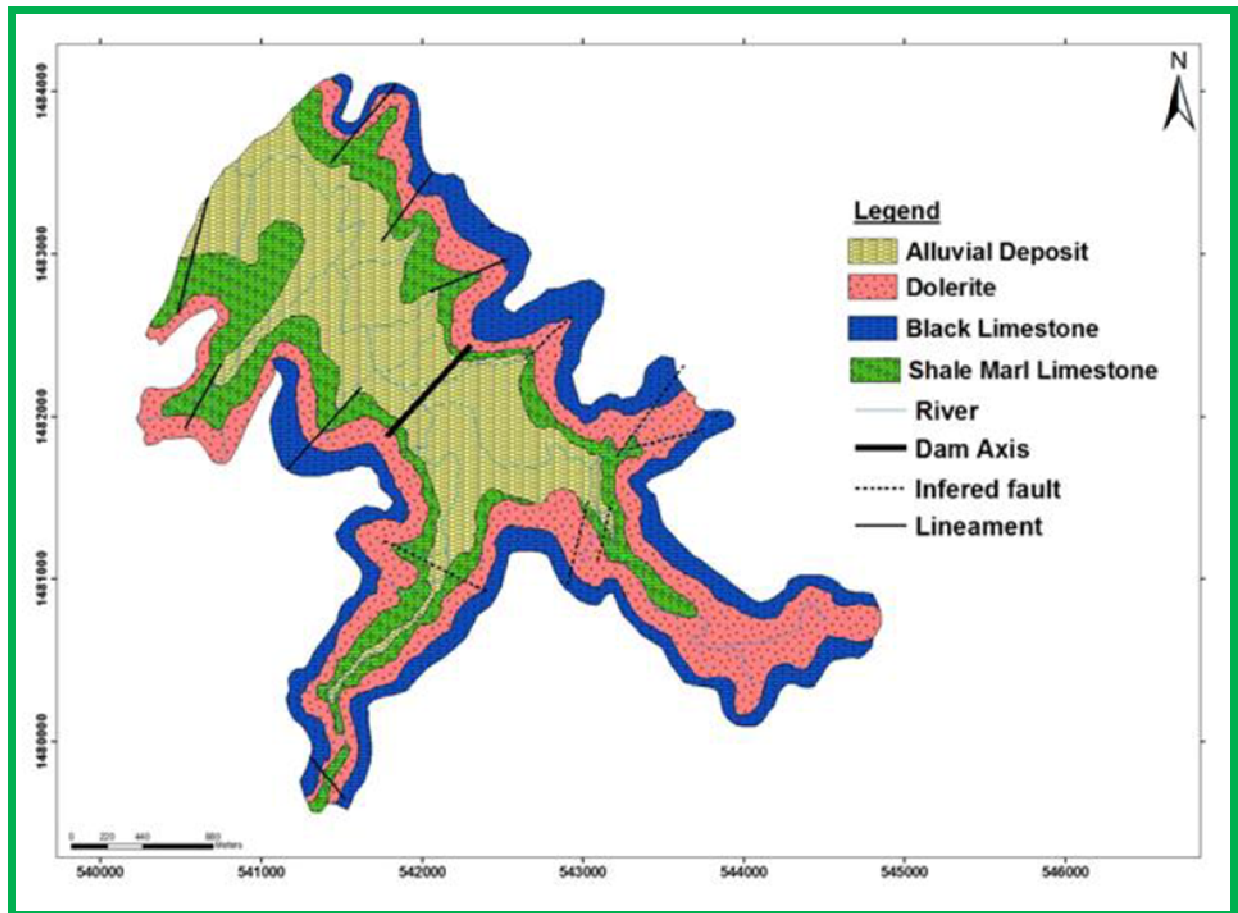


Figure 3.5 : Geological map of the Dam Site (ATSWWC, 2014).

3.1.6 Subsurface Conditions

The proper design of dams requires adequate information on the surface and subsurface engineering geological and/or geotechnical behavior of the earth materials (soils and rocks). According to the investigation and geotechnical characterization of the soils and rocks of the Gerebsegen dam project made by ATSWWC, (2014), surface field observation, test pits, trenches, geophysical (resistivity survey), core drilling, field in situ test and laboratory works

have been conducted. Using all the surface, subsurface and laboratory tests of the geologic materials (soils and rocks) found within the dam foundation and reservoir areas were analyzed, interpreted, classified. Accordingly 10 boreholes were drilled of which five of them are in the dam axis, one at the downstream, one at spillway route along the left abutment and three of them are in the reservoir with depth of drilling 30 to 50 meter (ATSWWC, 2014).

Generally, the main geological formations in the area, the shale-limestone-marl unit at this part of the trench are highly variable in terms of fracturing and voids formation. On the other hand the shale marl is characterized by ductile localized minor folds. The heterogeneity and disturbance of this unit may result due to effect of dolerite intrusion and major and minor fractures. All the results indicate that this rock mass is responsible for the excessive leakage observed from the dam.

3.1.7 Construction Material

The embankment zoning should provide an adequate impervious zone, filter and drainage zones between the core and the shells, and seepage control zones. An impervious clay water-retaining element or core of very low permeability of soil core fills. Zoning of an embankment that places the more pervious material (shell) on each side of the core zone is preferable. This placement improves the stability of the embankment during rapid drawdown conditions and keeps the downstream slope drained for greater effective weight (Singh and Varsheny, 1995).

Based on previous studies conducted by the ATSWWC (2014) the construction materials of the Gerebsegen dam was constructed for various construction materials such as shell, clay core, filter materials, and riprap materials in soils in a variety of quantities and locations within the fill. It was used which assumed the detail field and laboratory works have been done in the pre-construction investigation work. The shell materials were used the weathered shale. Dispersion test was carried out on four samples of the shell sources using the double hydrometer and it was confirmed that all the soil types of the shell sources of the study area are non-dispersive nature as compared with the standard values (ATSWWC, 2014).

Dispersive nature of the clay core was also checked and which is a non-dispersive and not susceptible to piping erosion. The filter (sand) was used in different part of the dam such as in

the horizontal filter and upstream work etc. But not available in that area because which have very poor quality and not fit filter requirement. Thus, sand was brought from the Agulae and Giba rivers for the construction industry around Mekelle, and the coarse filter material was used within Gerebsegen river bed and the fresh black crystalline limestone and dolerite were used for the riprap of the dam (ATSWWC, 2014).

3.2 Methodology

In order to answer the questions pointed out in the research question and meet objectives of the research work, first collection of all available data and information including as-built drawings, geotechnical engineering material properties, hydraulic parameters and geological cross-sections of the Gerebsegen dam area was done. The actual value of the seepage through the dam body, its foundation and around it has been determined during the field work by applying: direct field measurement using current method and float method and also the results obtained from the two methods were compared with SEEP/W model results. The actual reservoir level was also measured by leveling instrument to obtain the drawdown of the reservoir. Having geotechnical engineering material properties of the dam material, seepage (SEEP/W) and slope (SLOPE/W) analysis of the dam was done. Finally, based on the results obtained from the measurements and analysis of the seepage and slope stability, evaluation the dam, conclusions and recommendation on the possible remedial measures were drawn (Figure 3.6).

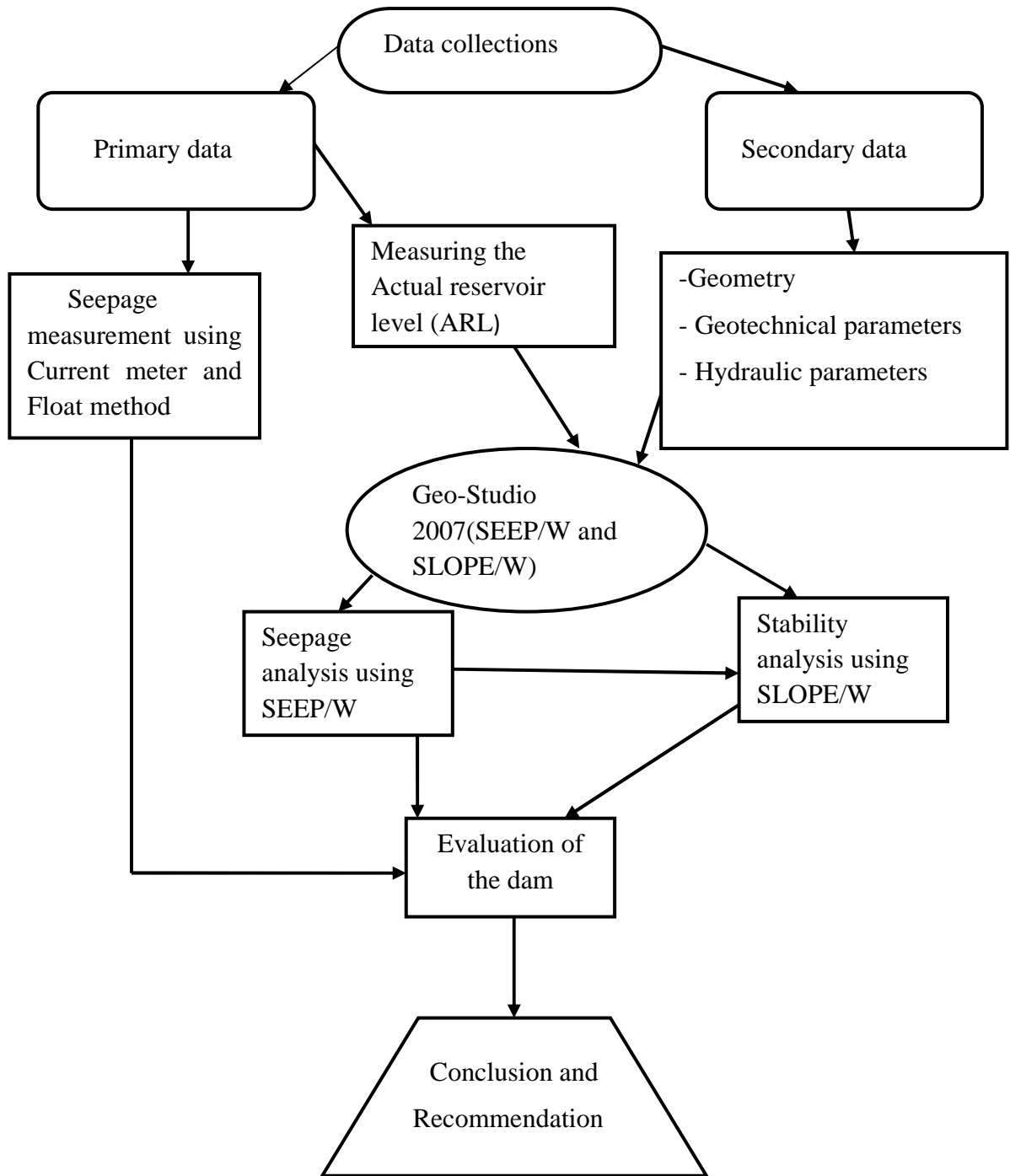


Figure 3.6 : The flow chart of the research methodology

3.2.1 Methods used

1. Soft ware: Geo-Studio 2007

Geo-studio, which includes SLOPE/W for slope stability, SEEP/W for groundwater seepage, SIGMA/W for stress-deformation, QUAKE/W for dynamic earthquake, TEMP/W for geothermal, CTRAN/W for contaminant transport, AIR/W for air flow, VADOSE/W for vadose zone and covers is software products based on finite element code that can be used to evaluate the performance of dams and levees with varying levels of complexity (SEEP/W, 2008). Therefore, the finite element method (SEEP/W) used in this research work is the best tool for analyzing seepage flow in an earth-fill dam. In addition, SLOPE/W software used under different conditions to evaluate slope stability of dams was also applied in this research work. Morgenstern-Price method which, applies minimum safety factor considered as a safety factor of slope stability, was used during the SLOPE/W model analysis.

I. SEEP/W: The SEEP/W program is capable enough to simulate quite effectively seepage rates and phreatic surfaces in homogenous and non homogenous earthen dams. It is 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. The software is capable enough to resolve the problems ranging from simple saturated steady state issues to saturated/unsaturated time dependent problems (SEEP/W, 2008).

II. SLOPE/W: SLOPE/W is a software product that uses theories and principles of the limit equilibrium methods (LEM) to compute the factor of safety of earth slopes, that developed by GEO-SLOPE (Geo-Studio). International Canada is used for slope stability analysis. The comprehensive formulation of SLOPE/W makes it possible to easily analyze both simple and complex slope stability problems using a variety of methods to calculate the factor of safety (SLOPE/W, 2008).

Many different method of slope stability analysis have been developed and adopted since many years. The procedure of computation in most methods are very similar, all the methods are targeted to compute the factor of safety. Some of the common methods are Ordinary or Fellenius, Bishop, Janbu, Spencer and Morgenstern-Price Method (Novak et al., 2007). The

stability analysis using SLOPE/W is included following components: Drawing geometry description of the stratigraphy and shapes of the potential slip surface, soil strength parameters properties used to describe the soil (material) and assigning for the corresponding soil layer, defining the water table, selection of analysis method (Spencer, Morgenstern-Price method), problem solving and display the results (Krahn, 2004).

2. Current flow meter (Physical): The measurement of the stream flow by directly measuring the current to determine water velocity. Each point velocity measurement is then assigned to a meaningful part of the entire cross section passing flow. The velocity-area principle is used to compute discharge from current-meter data. Total discharge is determined by summation of partial discharges (USBR, 1997; Geopacks, 2001).

3. Float Method (Physical): Fairly simple for measuring flow rate through an open channel is the float method. Although, not as accurate as a measuring device such as current meter. This method to comparing with current meter and which involves measuring the surface velocity of the water with a floating object, and then multiplying this velocity by width and average depth of the channel. The needed equipments are under this such as stopwatch, measuring tape, visible buoyant, marker, data collection sheet and pens (Joy et al., 2005, USBR, 1997).

4. Leveling instrument: It is the simple and most accurate instrument which used for measuring vertical height or elevation (Landgate, 2013). For this research leveling instrument has been used for measuring the actual reservoir level with difference draw down water level due to reservoir losses such as seepage at different time fluctuations by considering the reference of bench mark of the dam.

3.2.2 Data collections

Before conducting of any research, it is imperative to make a tough search for the data needed. Therefore, the primary assignment of the study is getting relevant information and data of the study area. The data are collected during the field study as well as other relevant data has been taken from the previous assessments done by Tigray Regional Water Resource Development Bureau, Tigray Water Works Design and Supervision Enterprise, Akam Three S Water Works

Consultants and the designer's technical report and personal interview with personnel who were in chief of construction supervision.

In order to collect the relevant primary data, practical field visit has been carried out for the following purposes:

- Visual inspection of the dam surveillance, locations and the quantity of seepage flow rate has been measured by measuring the velocity of the water which was used the float and current meter method for comparing alternatives for more accurate cross checking.
- Conducting survey on the crest of the dam structure was done in order to check the reservoir water level.
- Interviewing of the operator, residential engineer, supervisors, engineering geologists of the dam and beneficiaries of the command area.
- Collecting of pictures that show the reservoir water level, the existing seepage areas and measuring methods, the dam body with digital camera.

During the study work some of the secondary data collected includes:

- Dam cross sectional design and Geological setting of the study area.
- Engineering geological material properties of the study area, which are important inputs for Geo-studio model.
- Construction material property (hydraulic conductivity, unit weight, angle of internal friction, cohesion of soil) from laboratory testing for core material, shell material, foundation material, and filter.

3.2.3 Data analysis

The quantity of seepage, the seepage path and the velocity of seepage water are of great concern in analyzing the structural behavior of a dam which is occurred in large quantity of seepage through the abutments and the foundation.

After collecting all the relevant data, analysis have been done using different methods of analyzing seepage and compare with the actual existing seepage problem and slope stability

with minimum standard factor of safety. The two methods used were SEEP/W and SLOPE/W of Geo-Studio model.

3.2.4 General Seepage observation condition of the Dam

Water seepage is common problem in most of dam sites, in which the impounded water finds the escape routes with least resistant through the dam body and foundation. The observational method of seepage analysis and slope stability analysis involves visually inspecting the area of dam body embankment, the surrounding conditions and all potentially related factors (Jansen, 2006; Hasan et al., 2014).

As it was observed and collected the information on the reservoir filling and draw down process during the recent field visit and construction period, more than three meter of drawdown was observed due to the seeping of water from the dam in only six months. Since during initial impoundment of the reservoir of the seepage conditions was not performed as regularly as required. This was due to the lack of safety management procedures in the dam. So, during this studying regular recording of the seepage rates and as I had measured the actual reservoir level from January 2017 at 1851 m.a.s.l till 06 August, 2017 up to the 1847.75 m.a.s.l (Appendix table1).

The measured reservoir water level was taken for seepage analysis at 1850 m.a.s.l but that of seepage exit point water level is at 1828 m.a.s.l the downstream of the dam(. This indicates that the reservoir level is higher than the static water level on the seepage exit points, which can give a clue on the interconnection condition between the reservoir and the seepage exit points through the left and right abutments as shown in Figure 3.7.

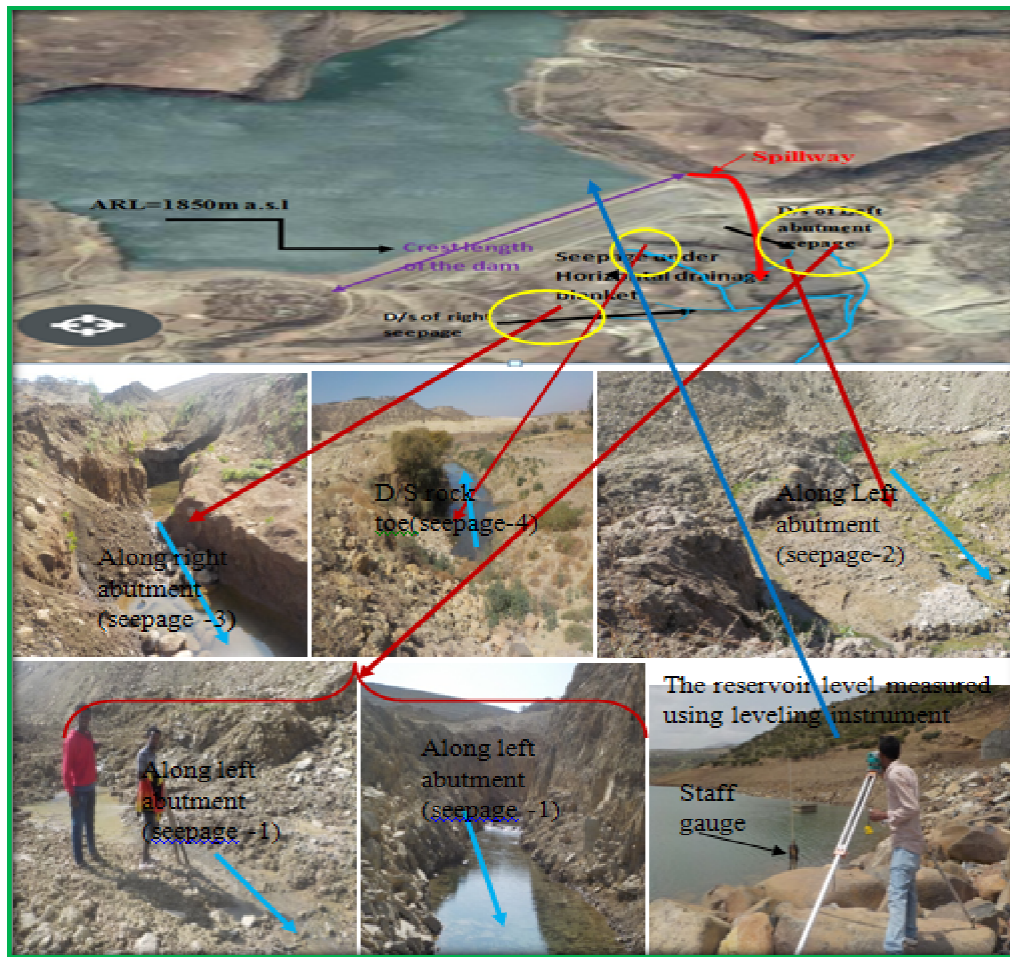


Figure 3.7: The reservoir conditions and the existing seepage from the downstream abutments and foundation of the dam

3.2.5 Field Seepage Measurements

The most common and direct method of estimating discharge is the velocity-area method. This technique requires measurement of flow velocity, channel width and the depth of water flow at cross flow vertical sections. During the studies for time the existing seepages were measured and repeated. The discharge of seepage loss (Q) refers to the horizontal water flow occurring at the surface in flow. The discharges of seepage Gerebsegen dam was collected in the drain channel flow which is defined in well shaped streams to be suitable measure. There are different types of flow measurement methods for seepage flow discharge, such as float method, V-notch method, bucket and stopwatch, current meter etc (USBR, 1997). Therefore,

from these above measurements, it was used the float method and current meter measurements during the field seepage flow.

$$Q = A \times V \dots\dots\dots \text{(Equation 3.1)}$$

Where, Q is discharge seepage flow (m³/sec), V is flow velocity (m/sec), and A is cross-sectional area (m²), this cross-sectional area the flow is defined as the width (m) and depth of the defined channel by which divided into rectangular section.

1. Float flow method: This method, which is inexpensive and simple, measures surface velocity from mean velocity, is obtained using a correction factor. The basic idea is to measure the time that it takes the object to float a specified distance downstream. With a floating object the surface velocity of the river is measured. Multiply the velocity by the cross-sectional area and by a correction factor (0.85). Since the top of the channel flow faster than the bottom due to friction against the flow bed, the friction correction factor events out the flow. For rough or rocky bottoms, multiply the velocity by 0.85. The mean river velocity can be approximated by the following formula (Joy et al., 2005):

$$V_{Mean} = 0.85 V_{surface} \dots\dots\dots \text{(Equation 3.2)}$$

Where, V_{mean} is mean velocity and V_{surface}.

During field work, measuring of seepage flows occurred due to impoundment of water on both abutments and foundation of the dam was done (Figure 3.9). Seepage water collected in a drain trench of the dam is taken through open channels for discharge into the downstream of the dam. The following steps were used the conditions for float during observation:

- 1) Located the spot in the open channel of the seepage that will act as the cross sectional of the channel.
- 2) Using the meter stick, the depth of the channel was measured at selected intervals along with the width of the channel.
- 3) Once these data were gathered, multiply each depth by the interval it was taken in and add all the amounts together. This calculation is the area of the cross section of the selected channel.

- 4) The length of the channel flow and typical width of the river was decided to send the floating material down.
- 5) Using the stop watch, the time it takes the float to travel down the length of the selected channel flow from step 4 was recorded.
- 6) Step five was repeated 5 times to determine the average time taken for the float to travel the flow. The float was thrown into the water at different distances from the shoreline in order to gain a more accurate average.
- 7) The flow of channel length found in step 4 was divided by the average time in step 6 to determine the average velocity of the flow.
- 8) The velocity found in step 7 was multiplied by the friction correction factor (0.85).

2. Current meter method: The current flow meter is an electronic device that counters from the impeller stick or anemometer proportional to velocity. The total number of counters per unit of time (normal one minute) can be converted in to a velocity value by referring to calibration charts or using formulae is shown in the Figure 3.8. So, the usual most important equipments are needed: Geopacks stream flow meter, stopwatch, measuring tape, ranging pole, data collection sheet and pens (Geopacks, 2001).

The cross-sectional area of the channel is perpendicular to the predominant flow direction. Measurements of channel widths and depths are then recorded using data sheet provided. This depth can be measured using the rule or staff. The cross sectional area of the flow is illustrated in the following equation (USBR, 1997):

$$A = W \times d \dots\dots\dots \text{(Equation 3.3)}$$

Where, A is cross sectional area of the flow (m²), W is width of the defined channel (m) and d is wetted depth of the define channel (m).

Determine the flow speed using the graph provided (Figure 3.8). Alternatively, the flow speed (V), in m/s is given by the following empirical formula in which C is the counts per minute recorded by the basic flow meter to water velocity (V) in m/s. This formula was used from the operational manual of current meter during my study filed work (Geopacks, 2001):

$$V = (0.000854 C) + 0.05 \dots\dots\dots \text{(Equation 3.4)}$$

Where, V= water velocity in m/s, and C=the number of counts per minutes.

During the field study work, the seepage water collected in a drain trench of the dam is taken through open channels for discharge into the downstream of the dam (Figure 3.9). In order to determine the depth and width of the selected channel, planting a wooden stake onto one bank and attaching a tag line running the line across the channel perpendicular to flow and also Applying the second wooden stake to the opposing bank and attach tag line again is needed. This line must be kept in place for the duration of the measuring, as it serves as a marker for subsection intervals necessary for readings of the water place a staff gauge, which was followed by the next step of connecting the counter meter with the current meter. Once meter is assembled and water depth has been established, a flow measuring method must be chosen, from which revolutions is counted using a digital meter reader each being at 60 seconds.

Generally current meter is more accurate than float method. Because the velocity is not the same all places in the stream which can be affect by the slope (friction against the flow bed) and external wind disturbed. In the current meter for the discharge flow measurement of the cross sectional was divided into rectangular subsection to get more accurate (Geopacks, 2001).

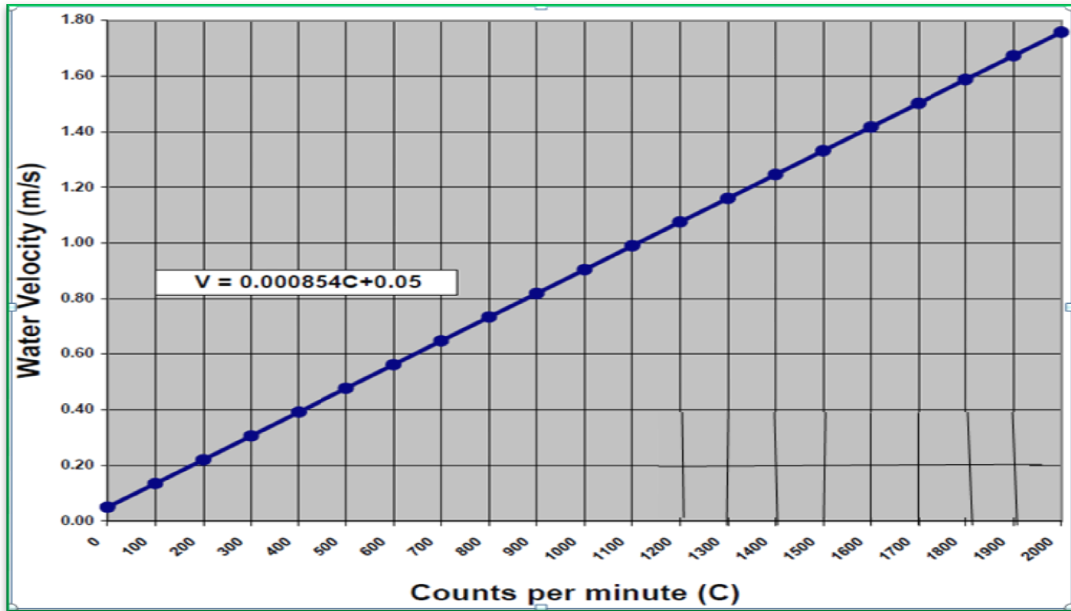


Figure 3.8: Water velocity Calibration chart (Geopacks, 2001)

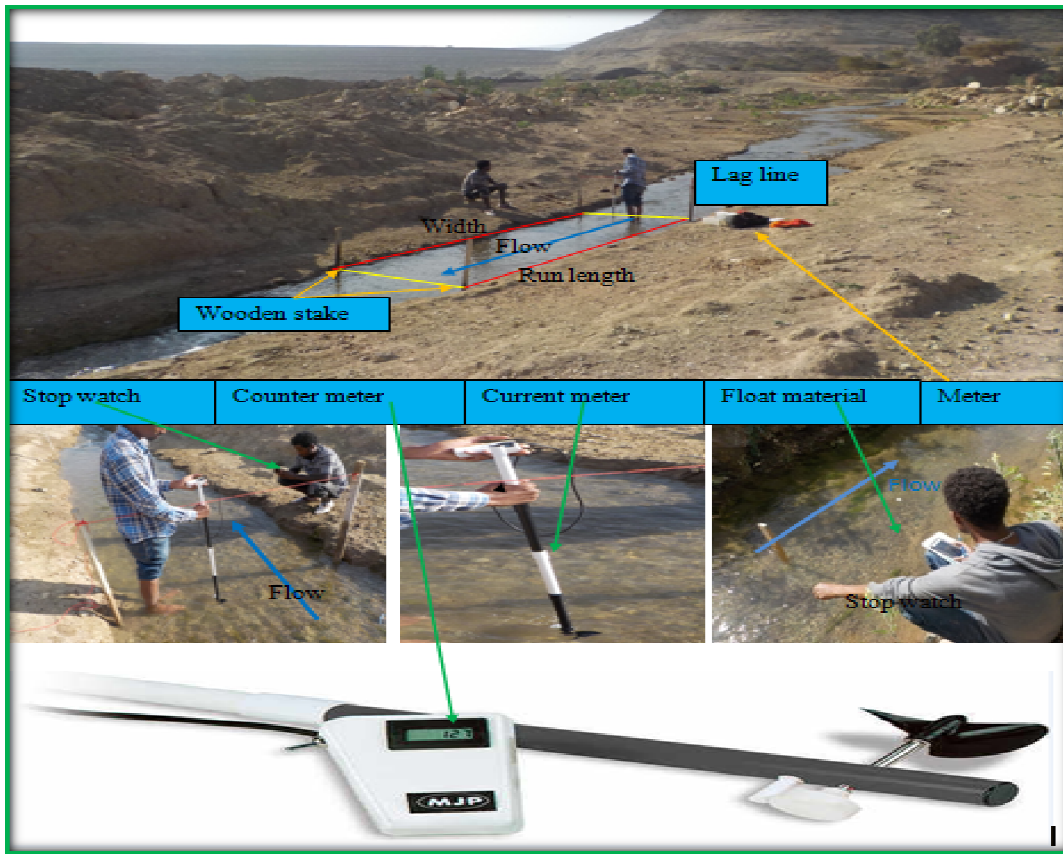


Figure 3.9: Measuring seepage flow using current meter and float method at the site.

3.2.6 Seepage analysis with SEEP/W Software

There are two fundamental types of seepage analysis: steady state and transient analysis. A steady-state seepage analysis is an analysis type where water pressures and water flow rates are at equilibrium with the defined boundary conditions and are not changing with time in the embankment and foundation strata. In contrast, in a transient analysis, seepage analysis the amount of flow into the system may differ from the flow out of the system because the system stores or releases water and the pore-water pressures are changing with time in the embankment and foundation strata. In general, a transient analysis can provide more accurate results when soil conditions are modeled; however, they are significantly more complicated than steady-state analyses. Both the initial conditions as well as future boundary conditions

must be provided (Broaddus, 2015). In this research both steady state and transient state analysis was carried out.

The SEEP/W software was used to conduct the finite element modeling for seepage analysis. The following partial differential equations (PDE) for two-dimensional domain can be expressed as assuming that flow follows Darcy's law (SEEP/W, 2008). The differential equation can be written as follows for the two dimensional for steady state:

$$\frac{\partial}{\partial x} (k_x \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y} (k_y \frac{\partial H}{\partial y}) + Q' = 0 \dots\dots\dots \text{(Equation 3.5)}$$

The governing PDE for seepage through a heterogeneous, anisotropic, saturated, unsaturated soil can be derived by satisfying conservation of mass for a representative elemental volume. If the assumption is made that the total stress remains constant during a transient process (rapid draw down or rapid fill of reservoir), the differential equation can be written as follows for the two dimensional transient case:

The governing PDE can be written as follows for the two dimensional transient case:

$$\frac{\partial}{\partial x} (k_x \frac{\partial H}{\partial x}) + \frac{\partial}{\partial y} (k_y \frac{\partial H}{\partial y}) + Q' = \frac{\partial \theta}{\partial t} \dots\dots\dots \text{(Equation 3.6)}$$

Where, k_x and k_y are the coefficients of hydraulic conductivity in the x and y direction, respectively (meter per second), where the ratio k_x and k_y is assumed to be 1 in this research, H is the total head being the summation of pressure head (h) and elevation head (z) in m, and Q' is the boundary flux (m^3/s), θ is the volumetric water content and t is the time. For steady state seepage, the flux entering and leaving an elemental volume is the same at all times. But under transient states that the difference between the flow (flux) entering and leaving an elemental volume at a point in time is equal to the change in storage of the soil systems. More fundamentally, it states that the sum of the rates of change of flows in the x-and y-directions plus the external applied flux is equal to the rate of change of the volumetric water content with respect to time (Thieu et al., 2001; SEEP/W, 2008).

1. Finite Element Method (FEM) meshes formation: Finite element numerical methods are based on the concept of subdividing a continuum into small pieces, describing the behaviour or actions of the individual pieces and then reconnecting all the pieces to represent the behaviour of the continuum as a whole. The three main important parts of finite element analysis are: discretization (dividing the domain into small areas called elements), specifying and assigning material properties and specifying boundary conditions. Basic requirements in a finite element analysis are geometry, finite element mesh, material properties, and boundary conditions. The finite element seepage equation can be expressed in a general form of using (SEEP/W, 2008; Melih, 2014):

$$[K]\{H\} = \{Q\} \dots\dots\dots \text{(Equation 3.7)}$$

Where, [K] is the element properties matrix, {H} is the vector of nodal heads, {Q} is the applied flux vector.

2. Setting of Boundary Conditions: In SEEP/W the boundary condition is necessary to always considered of both the pressure head and elevation components. In steady state analysis, all the boundary conditions are fixed. The boundary condition of the Gerebsegen dam is carried out for the seepage analysis through the dam at two scenarios, by means of the normal pool level of the reservoir (1859 m) and the actual reservoir water level (1850 m). In transient analysis, the condition on the boundary condition may change with time. On the upstream side, where the reservoir contacts the embankment dam and the original ground surface, the total head is equal to the full supply level. The upstream boundary condition is therefore a constant total head and the downstream boundary condition (zero total head) was defined on the downstream side of the exit.

Before calculating the seepage, first by selecting permeability sections of the dam foundation, in three sections (the number of these sections based on geological properties in the area obviously by increasing the number of sections more accurate seepage analysis can be done) based on the thickness of the layers of soil and rock, and the thickness of each layer. The cross sections of the dam chainage (i.e. Ch: 0+065 m, 0+480 m, and 0+638 m) were selected in such a manner that include the highest section of the dam body and more critical in sectional of

geological conditions for model using SEEP/W soft ware and its permeability coefficients values of various materials were taken from the design report of Gerebsegen Dam Project (2014).

Table 3.2: Permeability coefficient characteristics of the materials and foundation of the Gerebesegen earth dam (ATSWWC, TWRB, 2014):

| Type of Material | Standard of Permeability coefficient (K) in m/sec (USBR, 1987 and George ,2008) | Permeability coefficient, K (m/sec) |
|--------------------------------|---|-------------------------------------|
| Clay core | $<10^{-9}$ | 6.86×10^{-10} |
| Shell | $10^{-5} - 10^{-7}$ | 3.46×10^{-8} |
| Foundation (Marly Limestone) | $10^{-4} - 10^{-5}$ | 1.7×10^{-5} |
| Foundation (shale-mudstone) | 10^{-6} | 2.6×10^{-6} |
| Foundation (Shale with Gypsum) | 10^{-7} | 1.17×10^{-7} |
| Fine filter sand (F1) | $10^{-2} - 10^{-3}$ | 1×10^{-3} |
| Coarse filter gravel (F2) | $1 - 10^{-3}$ | 1.07×10^{-3} |

USBR (1987) describes soils with permeability less than 1×10^{-8} m/s as impervious, those with permeability between 1×10^{-8} m/s and 1×10^{-6} m/s as semi-pervious, and soils with permeability greater than 1×10^{-6} m/s is pervious. Accordingly, the permeability value of the shell material falls in the semi- permeable range.

3. Volumetric water content (VWC): It is important to understand the relationship between pore-water pressure and water content in seepage analyses. One of the required input parameters for a transient analysis is the volumetric water content function. In a saturated soil, all the voids are filled with water and the volumetric water content of the soil is equal to the porosity of the soil. The SEEP/W has different methods available to develop a 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 (sample function). Based these methods, the sample function is used water content functions for different the embankment materials are adopted from SEEP/W modeling engineering manual. Therefore, the volume water content functions (Table 3.3 and appendix figure I) of material properties have been directly taken from SEEP/W sample functions for the Gerebsegen dam.

Table 3.3: The volume water content functions of the embankment dam material (SEEP/W, 2008).

| Type of Material | Volumetric water content (VWC) | Remark |
|-----------------------------------|--------------------------------|-----------------|
| Clay core | 0.47 | Sample function |
| Shell | 0.45 | Sample function |
| Foundation -1 (MarlyLimestone) | 0.38 | Sample function |
| Foundation -2 (Shale with Gypsum) | 0.43 | Sample function |
| Fine filter -1 (Sand) | 0.35 | Sample function |
| Coarse filter-2 (Gravel) | 0.30 | Sample function |

The reason why volumetric water content of the clay core become higher than the other material is because of the pores between the individual clay particles in the clay is small. It is often difficult to identify a specific air-entry value for clay, as consolidation and changes to the structure of the clay can release water from the system over a significant range before air actually enters the pores. Since clay tends to be compressible, the slope of the function in the positive pore-water pressure region tends to be steeper and becomes a significant parameter to consider in saturated seepage analyses (SEEP/W, 2008).

The summary of setup procedures of seepage analysis with SEEP/W model:

- Three cross- sections of the dam were selected/drawn on SEEP/W

- The individual materials were assigned to their regions.
- The assigned dam cross-sections were discretized to small elements to apply FEM.
- The material and material models was assigned to the individual regions.
- Type of analysis has been fixed both steady state and transient case.
- Upstream and downstream boundary condition was determined based on the boundary conditions.
- Finally the problem was solved and results were analyzed and discussed

So that, the discharge of the seepage was calculated by analyzing the resultant discharge of the dam cross-section using SEEP/W software, which is calculated from the total discharge of seepage along 815 m length of the dam.

Pore-water pressure distribution

Finite element seepage analysis can be computing the pore-water pressure distribution for heterogeneous earth dam under both steady state and transient boundary conditions. If seepage analyses are performed to compute the pore water pressures, the method of analysis, including SEEP/W software, which was used. Appropriate contours of pore water pressure, total head, or pressure head are presented (Krahn et al., 1996). Initial analysis of the calculation is steady state; the rapid filling and rapid draw down of the reservoir are transient state analysis.

Evaluation of Horizontal and Vertical (or Inclined) drainage blankets

Vertical (or inclined) and Horizontal drainages are used to control seepage through the embankment by preventing materials eroded through a crack in the core from washing in the downstream shell by seepage water under reservoir head (Singh and Varsheny, 1995; Bahru, 2014). For evaluation these drains sizes Darcy's law was used accordingly.

- The thickness of horizontal blanket drains (t) required to discharge the seepage flow from the following equation based on Darcy's law.

$$q = \frac{kt^2}{2l} \dots\dots\dots \text{(Equation 3.8)}$$

Where: q=Seepage flux rate per unit width of the filter drain

k=permeability of the filter material of the drain

t=thickness of the filter drain

l=width of the downstream shell at the drain level

- The thickness of inclined drain, t, is given also by Darcy's law.

$$q = kiA \dots\dots\dots \text{(Equation 3.9)}$$

Where: q= rate of discharge seepage flux per unit width of the filter drain

k=permeability of the filter material of the drain

A=cross-sectional area of flow i.e. $A=lxt= t$

i=hydraulic gradient

To determine thickness of vertical (or inclined) and horizontal drainages blanket which applicable the Darcy's law is an equation that relates only application to flow through saturated materials and deals mostly analysis under laminar flow (or steady state) condition.

3.2.7 Slope Stability analysis with SLOPE/W Software

The stability of an embankment depends on the characteristics of the materials and its foundation and fill materials, on the geometry of embankment section, and the additional factors such as presence of water, loading conditions etc. SLOPE/W is the most common and popular software application used for the analysis of slope stability. The Limit Equilibrium Method formulation is based on two factors of safety equations and allows for a range of interslice shear normal force assumptions. One equation gives the factor of safety with respect to moment equilibrium (F_m), while the other equation gives the factor of safety with respect to horizontal force equilibrium (F_f). The LEM method satisfies both moment and force equilibrium by finding the cross-over point of the F_m and F_f (SLOPE/W, 2008; Pasetto, 2014).

SLOPE/W Software was used to conduct the limit equilibrium analysis for stability. Factor of safety values for the circular slip surface determined using entry and exit method to specify the location and shape of the trial slip surfaces. Modern limit equilibrium software is making it possible to handle ever increasing complexity within an analysis. It is now possible to deal with complex stereography, highly irregular pore-water pressure conditions, and various linear and nonlinear shear strength models, almost any kind of slip surface shape, and distributed or concentrated loads.

An analysis of slope stability begins with the hypothesis that the stability of a slope is the result of downward or motivating forces (i.e. gravitational) and resisting (or upward) forces. The resisting forces must be greater than the motivating forces in order for a slope to be stable. The relative stability of a slope (or how stable it is at any given time) is typically conveyed by geotechnical engineers through a factor of safety (Fs) defined as follows (Krahn, 2004):

$$F_s = \frac{R}{M} \dots\dots\dots \text{(Equation 3.10)}$$

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

The limit equilibrium methods use the Mohr-Coulomb expression to govern the shear strength along the sliding surface of embankment structures. The shear stress at which a soil fails in shear is expressed as the shear strength of the soil and the shear strength is usually expressed by the Mohr-Coulomb linear relationships defined as (Duncan et al., 2014):

Shear strength (available):

$$\tau_f = C + (\sigma - u) \tan \phi \dots\dots\dots \text{(Equation 3.11)}$$

Shear stress (mobilized):

$$\tau = \frac{C + (\sigma - u) \tan \phi}{F_s} \dots\dots\dots \text{(Equation 3.12)}$$

Where F_s , σ , u , C , and ϕ = factor of safety, normal stress, pore pressure, cohesion and friction angle respectively. The available shear strength depends on the type of soil and the effective normal stress, whereas the mobilized shear stress depends on the external forces acting on the soil mass. The procedures for static slope stability are well established, and have been concisely documented by Duncan et al., (2014). Duncan recommends that evaluation of a slope focus on defining geometry, shear strengths, unit weights, and pore-water pressures.

The following data of dam material properties parameters were used as input data (Table 3.4) when the stability analysis has been performed. All the data have been taken from laboratory soil test resulted of geotechnical engineering reports of SLOPE/W for stability of Gerebsegen multipurpose dam project and cross section (Figure 3.2).

Table 3.4: Materials properties of Gerebsegen Earth dam (ATSWWC, TWRB, 2014).

| Type of material | Unit weight, γ (KN/m ³) | Internal Angle, ϕ (°) | Cohesion, C (Kpa) | Remark |
|------------------------------|--|----------------------------|-------------------|--------|
| Clay core | 13.13 | 14.03 | 44.25 | |
| Shell | 15.23 | 29 | 25.5 | |
| Foundation (Marly Limestone) | 15.52 | 29 | 2.7 | |
| Fine filter (Sand) | 18 | 33 | 0 | |
| Coarse filter(Gravel) | 18 | 34 | 0 | |

The summary of setup procedures of slope analysis with SLOPE/W model

- The maximum cross section of the dam was drawn on SLOPE/W
- The individual material was assigned to their region
- The strength parameters and the material model were assigned to each region
- Morgenstern-Price method of analysis was used to more accurate
- The volumetric water contents were taken from SEEP/W document
- Finally the problems were solved and the results were analyzed and discussed

Methods of slope stability and loading conditions

The stability of the Gerebsegen Dam has been analyzed the limit equilibrium method (LEM) according to Morgenstern-price presented using SLOPE/W method. The usual failure of an earth embankment consists of the sliding of a large mass of soil along a curved surface. The loading conditions considered in the present analysis are minimum water level 1844 m, maximum water level 1859 m. The stability analyses had been conducted in order to determine the factor of safety for various slip surfaces.

Table 3.5: Various load cases and minimum required factor of safety slope stability for Earth and Rock-fill Dams at Full Supply Condition (Novak et al., 2007):

| Design of Loading condition | Critical slope | Factor of safety (FS min) |
|--|----------------|---------------------------|
| Steady state seepage | Downstream | 1.50 |
| | Upstream | 1.10 |
| Steady state seepage with seismic load | Downstream | 1.10 |
| | Upstream | 1.20 |
| Sudden draw down with seismic load | Upstream | 1.10 |

Stability analysis of downstream slopes under steady state seepage condition:

When the reservoir has been full long enough for seepage water to percolate all the way through the embankment, the pressure in the pore water in the downstream portion reaches its highest values (Duncan et al., 2014). Under this condition the downstream slope may have its lowest factor of safety against sliding. The phreatic surface computed with the help of SEEP/W was used to set up the pore water pressure line in the stability analysis.

Stability analysis of upstream slope under sudden drawdown condition

Reservoirs may be drawn down in a matter of days or weeks. Stability computations were performed for the upstream slope for conditions occurring when the water level adjacent to the slope is lowered rapidly. For analysis purposes, it was assumed that drawdown is very fast, and no drainage occurs in materials with low permeability. The critical situation occurs when a reservoir has been at high level for an appreciable period of time so that the embankment material has come to equilibrium (consolidated) under the high reservoir condition. Generally, the upstream face of the impervious zone of the dam is critical for stability (Novak et al., 2007; Pasetto, 2014).

Dynamic (Pseudo static) slope stability analysis

Earthquakes expose slopes to dynamic loads that can reduce the soil shear strength and cause instability. One of the earliest procedures for analysis of seismic stability is the pseudostatic procedure in which the earthquake loading is represented by a static force; equal to the soil weight multiplied by a seismic coefficient the pseudostatic force is used in a conventional limit equilibrium slope stability analysis. Most commercial slope stability programs can accommodate the use of a seismic coefficient (Duncan et al., 2014). This analysis is similar to the above cases with an additional earthquake loading. The intensity of a shock is expressed by acceleration coefficients horizontal (α_h) and vertical (α_v) each representing the ratio of peak ground acceleration to gravitational acceleration (g). The horizontal ground acceleration (α_h) and vertical ground acceleration (α_v) considering the vertical acceleration to be 50% of the horizontal acceleration (Novak et al., 2007). According to the Ethiopian Building codes standards, the ground acceleration value for western margin of the Afar depression (where Gerebsegen dam is located) considered be a peak ground acceleration (PGA) value of 0.10g. When an earthquake condition is considered, the maximum credible earthquake with seismic factor of 0.10g is taken (ATSWWC, 2014).

4 RESULTS AND DISCUSSIONS

4.1 Seepage analysis (using current meter and float methods)

Previous studies indicate seepage of water through, around or under a dam is expected in all embankment dams and even in concrete dams, which can be considered as one of the best indicators of a dam's performance. From this, seepage from any dam must be controlled and checked with respect to the allowable amount indicated during its design and construction period. According to Jansen (2006), the allowable seepage from embankment dams should not be in excess of $0.03 \text{ m}^3/\text{s}$ (30 l/s).

As it was observed during the field study to the dam site and its surroundings, no indication of seepage through the dam body was observed and therefore no wet area on the downstream of the dam body despite its horizontal drainage system collects part of the water losses and creating stream along the river. The main seepage exit point is along the left abutment/spillway route/ at about 300 m length from the centerline of the dam axis and has significant seepage losses. In addition, seepage is also observed along the right abutment, near to the location of water supply treatment plant and due to this reason, the areas along both abutments become wetted as shown in appendix figure D and appendix figure E.

In order to obtain the approximate values of the seepage or water loss from the dam, both float flow and current meter of flow measurement methods were applied. The amount of seepage measured using the two methods along both abutments and foundation outside the dam body were found to be almost similar result as shown (Figure 4.1 and Table 4.1). The average total seepage losses measured using current meter and float flow method were about $0.2053 \text{ m}^3/\text{sec}$ (205.3 l/s) and $0.202 \text{ m}^3/\text{sec}$ (202 l/s) respectively. But, along the left abutment and rock toe (seepage-1 and seepage-4) the amount of seepage measured using float method was slightly different from the results obtained using current method (Figure 4.1), thus the results obtained using the current meter were considered and taken as to estimate the amount seepage from the dam. The seepage analysis shows that, the amounts of seepage measured at seepage-1, seepage-2, and seepage -4 of the exit area were slightly decreased in the last six months. But seepage-3 was different from others exiting seepage which has measured almost constant in the last six months. The results of seepage measurement at each seepage area, starting from

the month January 28 to August 08, 2017 at different reservoir levels of 1851.05 m to 1847.75 m a.s.l are shown in Appendix table 1. So that, the water seepage discharge shows a tight correlation with the reduced water level progress. The result indicates that, the volume of water measured from the surface seepage and obtained from the drawdown of the reservoir level were almost similar.

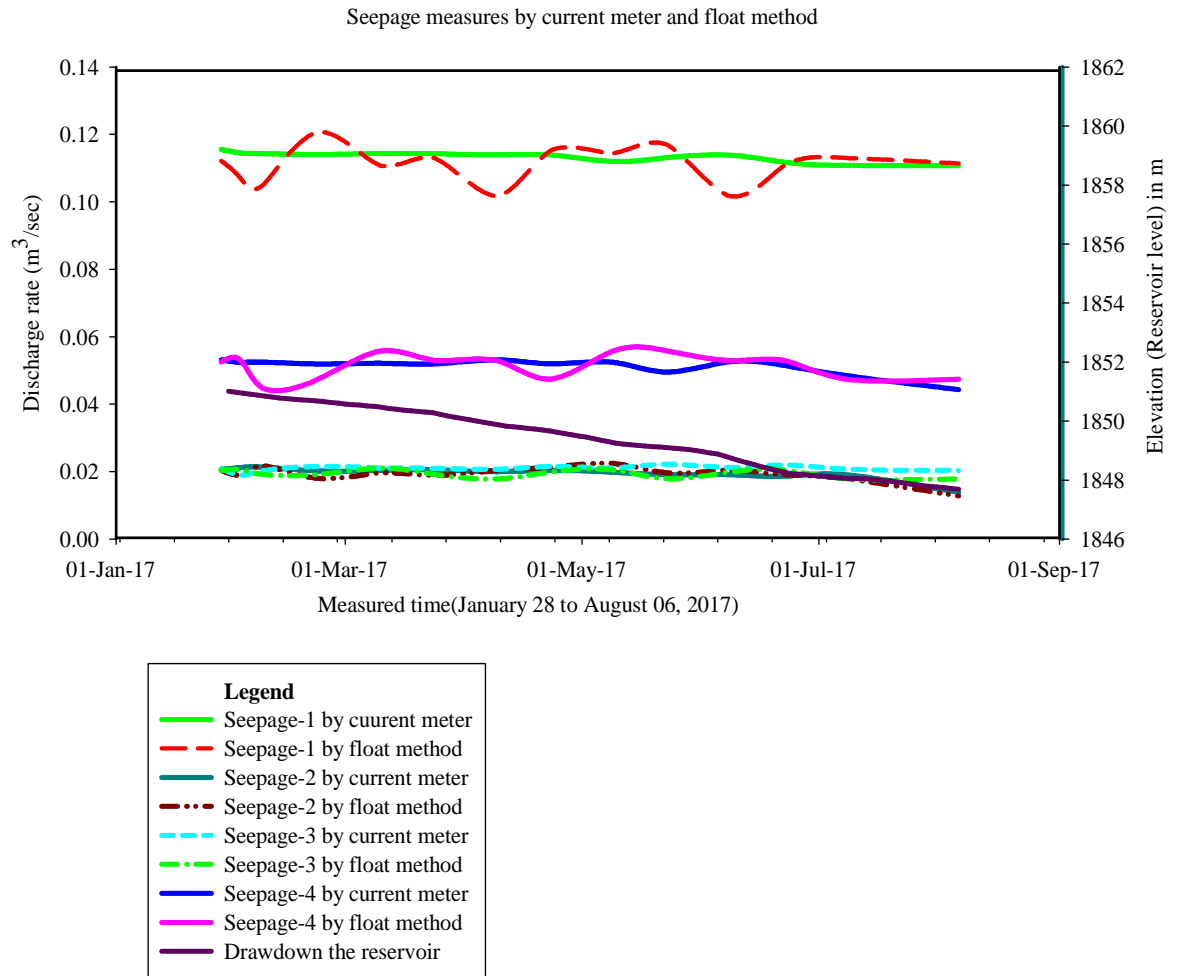


Figure 4.1: Actual seepage rates losses measured and reservoir head draw down

The seepage amount $0.021 \text{ m}^3/\text{sec}$ (10% of the total seepage) was measured at the right abutment of the dam. Around $0.133 \text{ m}^3/\text{sec}$ (65% of the total seepage) was recorded in the left downstream side of the dam (along the spill way route) and $0.051 \text{ m}^3/\text{sec}$ (about 25% of the total seepage) discharged along the downstream rock toe of the dam. Therefore, the total

combination amount of seepage through the dam body, foundation as well as abutments measured becomes 0.2053 m³/sec (Table 4.1) and the annual amount of water loss due to seepage from 1851.05 m.a.s.l. to 1847.75 m.a.s.l. of reservoir water level computed was 6,474,340.80 m³/year. Therefore, excess water loss through seepage may endangers for the existence of the dam.

Table 4.1: Discharge measured using float and current method at downstream area of the dam.

| No | Location of excess seepage | Seepage discharge flow (Q) in m ³ /sec | | Annually Seepage losses(m ³ /year) (qx3600x24x365) | Status of the total seepage |
|---|----------------------------------|---|----------------------|--|-----------------------------|
| | | Float method | Current meter method | | |
| 1 | Along Left abutment (Seepage-1) | 0.111 | 0.1134 | 3,573,825.07 | 55% |
| 2 | Along Left abutment (Seepage-2) | 0.020 | 0.020 | 620,649.20 | 10% |
| 3 | Along Right abutment (Seepage-3) | 0.019 | 0.021 | 661,103.54 | 10% |
| 4 | Downstream rock toe (Seepage-4) | 0.0514 | 0.0513 | 1,617,283.67 | 25% |
| Total seepage losses from the dam reservoir | | 0.202 | 0.2053 | 6,474,340.80 | 100% |

Compared to Jansen (2006) the standard design and allowable seepage expected (0.03 m³/sec) from the target dam, the total amount of seepage measured during this study is more than the allowable amount of seepage. This seepage resulting from target dam is the indicator of excessive loss of water.

The total amount of seepage measured on the downstream area of the surface is probably due to discontinuities and untreated of highly fracture limestone-shale-marl intercalation of the geological foundation of the dam constructed. In case of rocks, the hydraulic conductivity could be highly affected by discontinuities of the formation of the rock. These geological and structural conditions make this unit permeable and responsible for excessive seepage losses.

4.2 Seepage Analysis (using SEEP/W Model)

Steady state seepage analysis uses the principle “consider the condition without respect to time and the distribution of total head and pressure head”. Seepage analyses through the dam and the foundation have been conducted using finite element method based computer program-SEEP/W from Geo-slope international, 2007. The discharge quantified using SEEP/W is used to evaluate thickness of filter, and is help full while working with water balances, in addition, to compute phreatic surface, to set up the pore water pressure line and is used in computing slope stability analysis of the dam.

As the results obtained using the SEEP/W are presented in (Figure 4.2 to 4.4 and Table 4.2) the expected quantity of the seepage (through the dam body and its foundation) for every meter length of the dam and for the three selected sections were computed. At the three selected section chainage (Ch.) of 0+065 m, 0+480 m and 0+638 m of the actual water level (ARL) of the reservoir, the quantified seepage fluxes obtained are 2.040×10^{-5} m³/sec/m, 2.993×10^{-6} m³/sec/m, and 4.417×10^{-5} m³/sec/m respectively. The average seepage flux computed at the normal pool level for every meter length of reservoir was 3.304×10^{-5} m³/sec/m (Appendix Figure A). But the value from the design document computed using SEEP/W-2004 and estimated at one maximum selected section and one foundation of hydraulic parameter was 1.031×10^{-7} m³/sec/m (Appendix Figure J), which is lower than the value computed during this study. At the actual water level of the reservoir, the average seepage flux computed was 2.252×10^{-5} m³/sec/m. The aforementioned results indicate, the computed seepage flux at the two selected critical sections (left and right abutment) are slightly higher than the maximum middle selected section of the dam. This could be due to the presence of highly pervious shale/marl-limestone intercalation is shown in Figure 4.5.

Table 4.2: The computed seepage flux using SEEP/W software at selected sections

| Seepage flux through, q in ($m^3/sec/m$) | Selected section at different chainage (Ch.) of the dam for ARL (1850 m.a.s.l) | | | 815 m of total length (L) of the dam | |
|--|--|--------------------|--------------------|--------------------------------------|---|
| | at the Ch: 0+065 m | at the Ch: 0+480 m | at the Ch: 0+638 m | Average q flux ($m^3/sec/m$) | Discharge seepage flux= $(q*L)$ m^3/sec |
| The dam body and foundation | 2.040E-05 | 2.993E-06 | 4.417E-05 | 2.252E-05 | 0.01835 |
| The foundation | 2.0398E-05 | 2.974E-06 | 4.416E-06 | 2.166E-05 | 0.01765 |

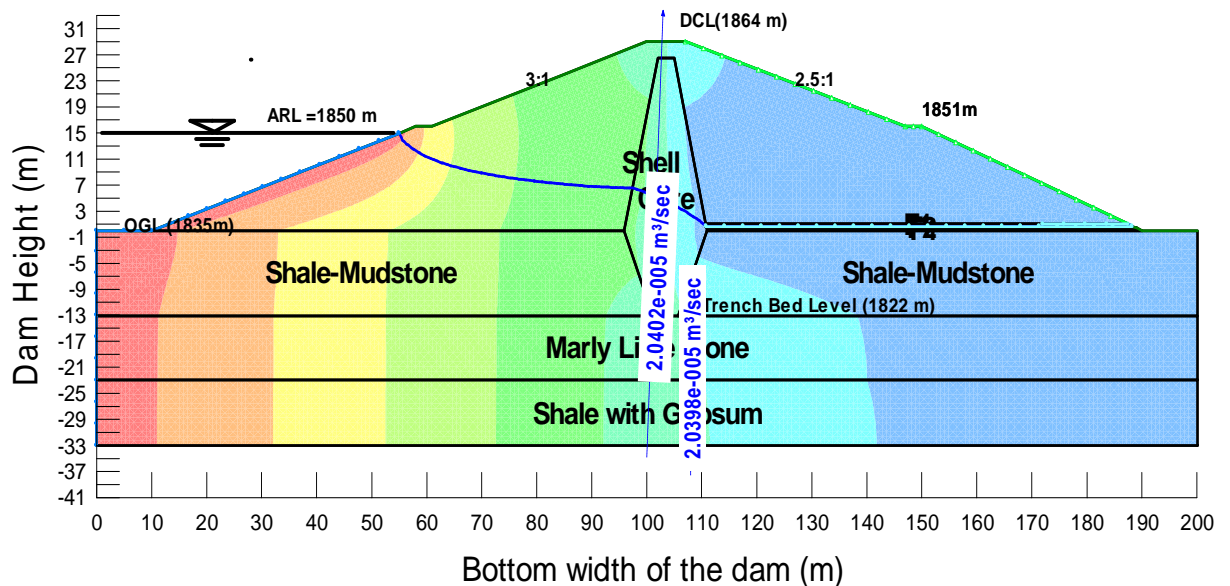


Figure 4.2: Seepage analysis result for steady state at chainage Ch: 0+065m

As the results shows that, the computed seepage flux (Figure 4.2 to Figure 4.4 and Table 4.2) at the two selected critical sections (left and right abutment) are higher than the maximum middle selected section of the dam. The obtained seepage flux through the foundation is too much higher than the dam body, because of the depth of excavation is not reached bed rock

(shale with gypsum). The shale-marl-limestone unit at this part of the trench is highly variable of in terms fracturing and voids formation.

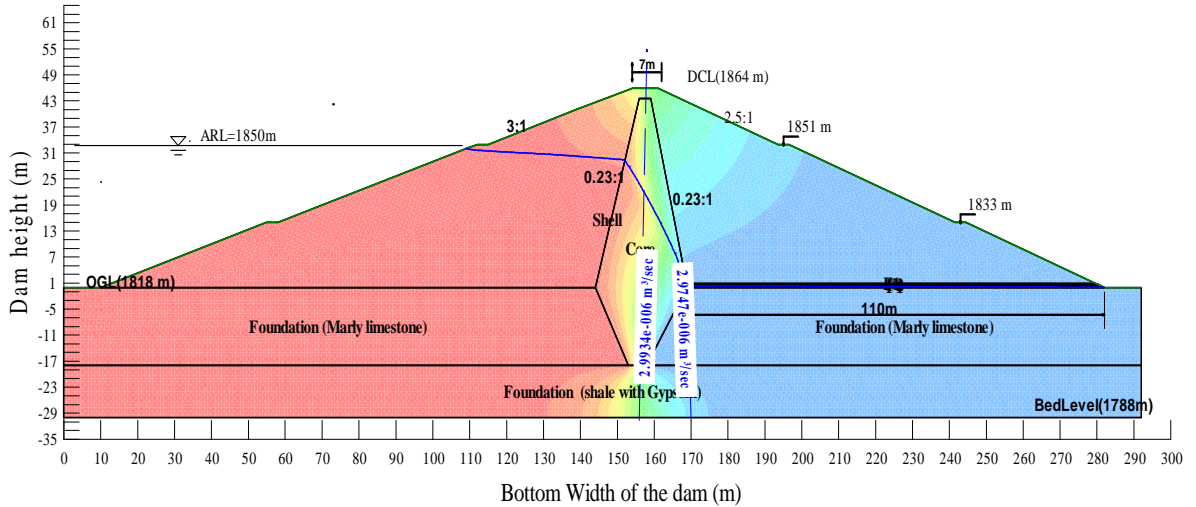


Figure 4.3: Seepage analysis result for steady state condition at chainage Ch: 0+480m

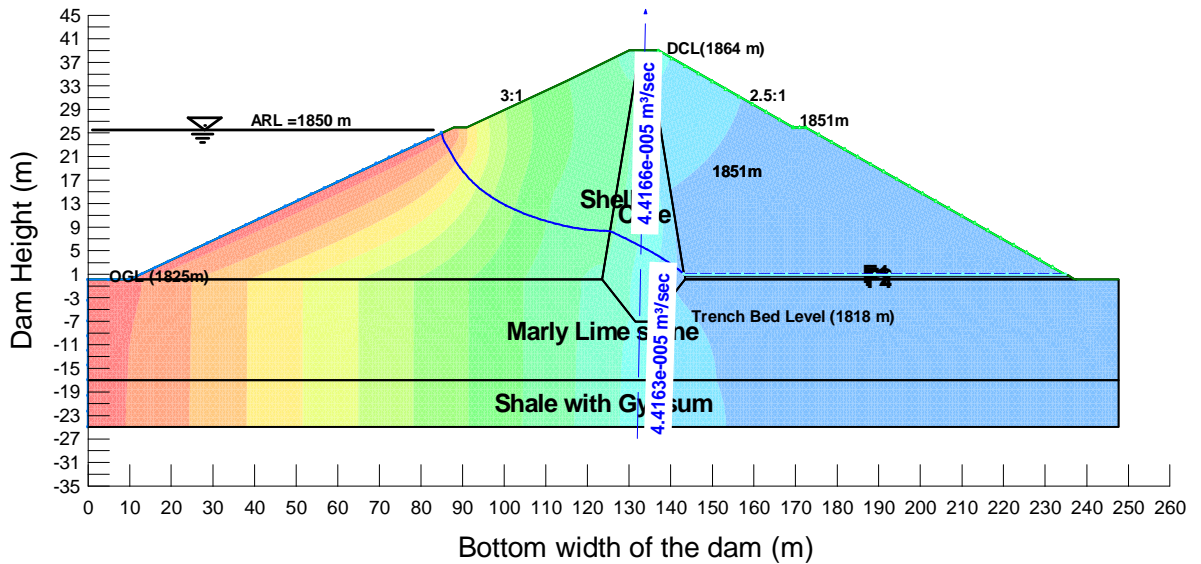


Figure 4.4: Seepage analysis result for steady state condition at chainage Ch: 0+638m

The actual average seepage through dam body and its foundation of the dam with 815 m of total length, quantified are shown in (Table 4.2 and 4.4) in the steady seepage case analysis

using SEEP/W model is $2.252\text{E-}05 \text{ m}^3/\text{sec}/\text{m}$ ($0.01835 \text{ m}^3/\text{sec}$), and the annual amount of water loss due to seepage at 1850 m a.s.l. computed was $578,685.60 \text{ m}^3/\text{year}$.

Look (2007) indicates the allowable seepage flux from earth dams with height of 40 m and above should not be in excess of $9.33 \times 10^{-6} \text{ m}^3/\text{sec}/\text{m}$ ($7.60 \times 10^{-3} \text{ m}^3/\text{s}$). Compared to the standard allowable seepage expected from the earth dam, the amount of seepage computed using SEEP/W is more than the expected amount of seepage. This is due to the geological and geotechnical characteristics of the area where the dam was constructed.

4.2.1 Seepage analysis with revised hydraulic conductivity of Marly Lime stone

The type and condition of geological formation of target area play a major role in the stability and water tightness of a dam to be constructed. Different geological formations have different properties of stability and water tightness. As a result of this, detailed investigation on the water tightness and stability of geological formations is important before constructing of any dam. According to ATSWWC (2014) and observed at open trench during construction excavation, the dominant geological formations of the site are dolerite shale and marl-limestone intercalations. The central parts of the dam foundation and reservoir area are covered by alluvial deposits. The shale-marl-limestone intercalation is observed underlying the alluvial deposit. The left and right abutments of the dam site have similar lithological and structural setup except small differences in thickness from top to bottom. Lime stones and related carbonate rocks vary widely in their suitability as foundations for dams and reservoirs. Thin-bedded, deformed, and cavernous lime stones present serious problems that may be so extensive as to make corrective measures impracticable. These sedimentary rocks of the area are characterized by cavernous, joints and karastic features which can be responsible for the seepage and presents serious problems in the dam foundation, reservoir and abutments. The shale is variegated, fissile with variable thickness. It is characterized by high degree of weathering and fracturing and interpreted to be highly pervious (Figure 4.5 and Appendix Figure F).

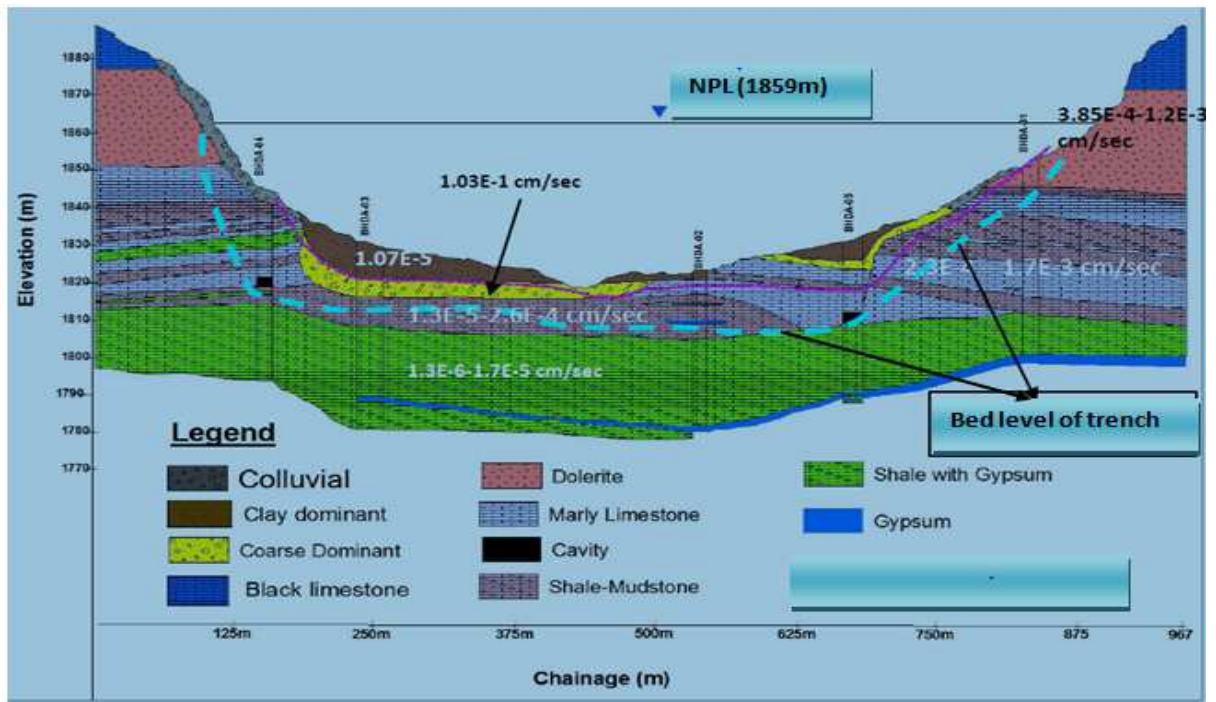


Figure 4.5: Cross section showing various lithologies of the foundation and abutment (ATWWC, 2014).

Gebremedhin et al., (2016), in their study for Hizaetiwedicheber (Mekelle area); where the present study area is located, identified that limestone of the area is the main aquifer system with highest hydraulic conductivity value ranging from $1.226E-04$ m/sec (10.6 m/day) to $3.634E-04$ m/sec (31.4 m/day) which can be related to the target dam. The foundation and reservoir rim geological formation (limestone-shale marl intercalation) and setup of the Gerebsegen dam is similar to Hizaetiwedicheber dam site. Considering the tests done on both dams, the average hydraulic conductivity value of $2.43E-04$ m/sec was used during the SEEP/W model simulation.

Table 4.3: The computed seepage flux (K of Marly limestone =2.43E-04, Gebremedhin et al, 2016) using SEEP/W model at selected sections.

| Seepage flux through, q in m ³ /sec/m) | Selected section at different chainage(Ch) of the dam for ARL(1850 m) | | | 815 m of total length (L) of the dam. | |
|--|---|--------------------|--------------------|--|--|
| | at the Ch: 0+0 65 m | at the Ch: 0+480 m | at the Ch: 0+638 m | Average q flux (m ³ /sec/m) | Discharge seepage flux(Q) =(q*L) m ³ /sec |
| The dam body and foundation | 1.260E-04 | 3.188E-06 | 6.217E-04 | 2.503E-04 | 0.2039 |
| The foundation | 1.259E-04 | 3.17E-06 | 6.21E-04 | 2.500E-04 | 0.2038 |

Taking this value of average hydraulic conductivity during the SEEP/W model analysis, the amount of seepage computed was 2.503E-04m³/sec/m (0.2039 m³/sec) (Appendix Figure B Fig.1 to Fig.3), and the annual amount of seepage computed was 6,430,190.04 m³/year (Table 4.4), which is more than the amount of seepage computed using the design hydraulic conductivity value. Therefore, excess water loss through seepage is expected which can endanger its existence of the dam. All the geological formation indicates that this rock mass is the cause for excessive seepage loss.

Table 4.4: The results of measured and simulated annual seepage losses of the dam

| Type of seepage Analysis | Recommended allowable seepage (Jansen, 2006;Look , 2007) | Seepage losses rate, q (m ³ /sec) | Annual seepage losses(m ³ /year) |
|---|--|--|---|
| | | | (q*3600*24*365) |
| Current meter | 0.03 m ³ /sec | 0.2053 | 6,474,340.80 |
| SEEP/W Model | 9.33x10 ⁻⁶ m ³ /sec/m (7.60x10 ⁻³ m ³ /s) | 0.01835 | 578,685.60 |
| SEEP/W (used; k=2.43E-04 for marly limestone) | 9.33x10 ⁻⁶ m ³ /sec/m (7.60x10 ⁻³ m ³ /s) | 0.2039 | 6,430,190.04 |

The aforementioned results indicate, the amount of seepage flow from the dam coming through the central foundation and abutments was measured using current meter and float method is higher than the amount of seepage computed using the model. But, the amount of seepage computed using the model by taking the average hydraulic parameters measured by Gebremedhin et al., (2016), during their study for Hizaetiwedicheber dam near to the study area, which is relatively near to the actual amount of excess seepage measured during this study. Compared to the standard allowable seepage expected from the earth dam, the amount of seepage both computed and measured are more than the expected.

Previous studies indicate that water flow through the soil media due to their potential energy and gradually lose their energy because of friction in this environment. This phenomenon is called water seepage in the soil porous environment which has negative effects are such as loss of water stored behind the dam structure, generation of pore pressure in porous media and reduction of effective stress between soil particles. Thus, reduction of its shear strength, creating uplift pressure, movement of soil particles and internal erosion could be created.

4.2.2 Analysis of Pore-water pressure distribution

Due to lateral flow, pore-water pressures may increase in areas of initially low excess pore water pressures (beneath the toe of the embankment). The value of pore water pressure distributions from SEEP/W modeling are shown in Figure 4.6 and table 4.5 under steady state seepage. Therefore, the pressure distribution was also resulted as expected with minimum at the top and maximum at the bottom along with the depth of water increment. In addition, the computed phreatic surface as shown by the blue line is used to set up the pore water pressure line and is used in computing slope stability analysis of the dam. It has been seen that the blue phreatic line is well aligned with the filter material of the dam body (not intersected with downstream slope).

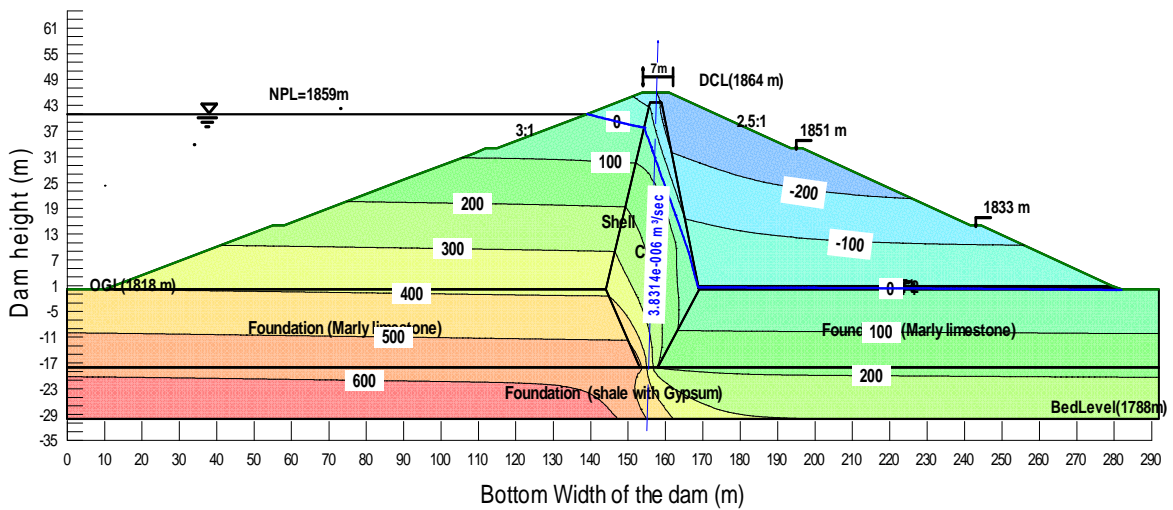


Figure 4.6 : Pore water pressure distribution for steady state at chainage Ch: 0+480 m

For transient state (the rapid fill) condition of seepage analysis six time steps (1, 5, 10, 60, 100,297,1314,5400 days) of reservoir filling were used during the SEEP/W software model simulation to assess the reaction of the constructed dam to the amount of water stored and assess the pore water pressure along the dam body (Figure 4.8 and Table 4.5). Prior to running of transient condition, a steady state analysis was established at the time where the reservoir is empty. In other words, a total head of 0 m is assigned to the upstream face of the embankment dam as an initial condition. Then the total head is increased from 0 m to 41 m in instantaneous time, exponentially. Water level in the reservoir is considered to rapidly increase during the rising stage of single reservoir fill. The simulation duration is selected as 4800 days, which is sufficient time for the flow to reach a condition where almost steady-state conditions are observed for the flow. It clears from this figure 4.7 that after 4800 days, the seepage rate does not change with respect to time.

Table 4.5: The hydraulic parameters of steady state and transient state result of seepage analysis of the dam.

| Hydraulic parameters | Method of seepage analysis using SEEP/W Model | | | | | | | | |
|---------------------------------------|---|--------|---------|---------|----------|----------|-----------|-----------|------------------------|
| | Transient (reservoir filling with time, days) | | | | | | | | steady state condition |
| | 1 day | 5 days | 10 days | 60 days | 100 days | 297 days | 1314 days | 4800 days | |
| Seepage flux (m ³ /day/m) | 0.188 | 0.246 | 0.269 | 0.296 | 0.303 | 0.316 | 0.326 | 0.329 | 0.331 |
| Maximum pore-water pressure (Kpa) | 665.8 | 679 | 683.8 | 689.7 | 691.20 | 693.30 | 694.60 | 694.7 | 695.5 |

The output and evaluation result of the analysis indicate, both the seepage quantity and pore-water pressure were increased gradually to the final steady state flux at rapid filling of reservoir time stages. In addition, the gap between the position of phreatic line of the dam and that of the result obtained from the steady state condition is relatively large, which in turn needs long time to attain the normal phreatic line and therefore saturating of the shell materials of the dam. This can be due to using of low permeability shell construction material, which can play a significant role in the stability of the dam.

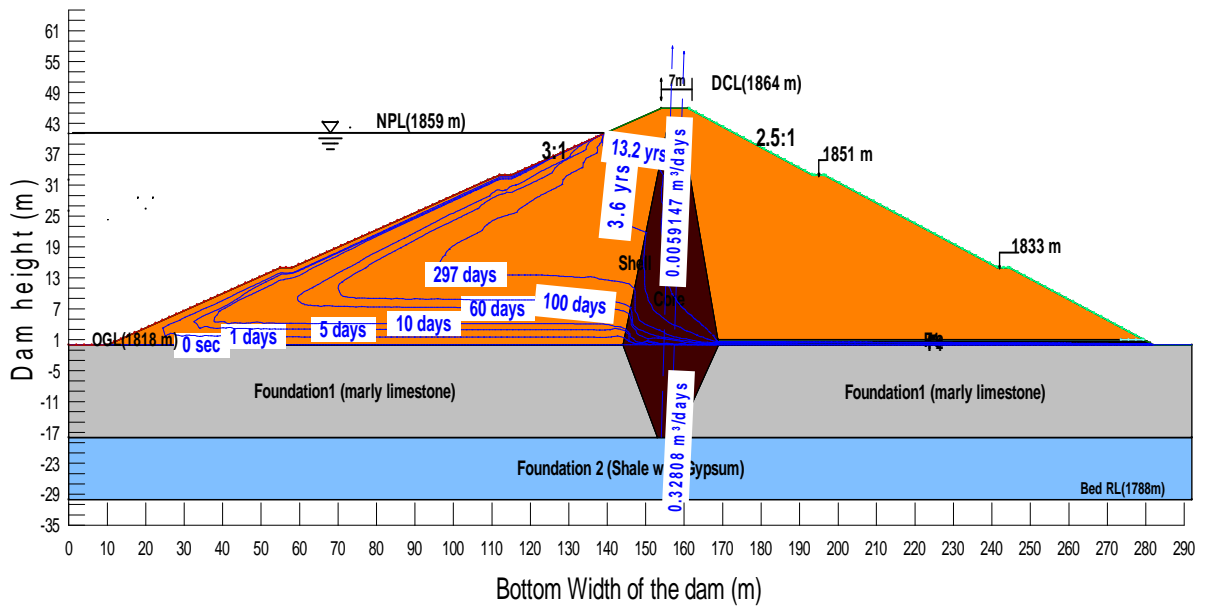


Figure 4.7: Seepage analysis result for transient state condition (rapid filling of reservoir)

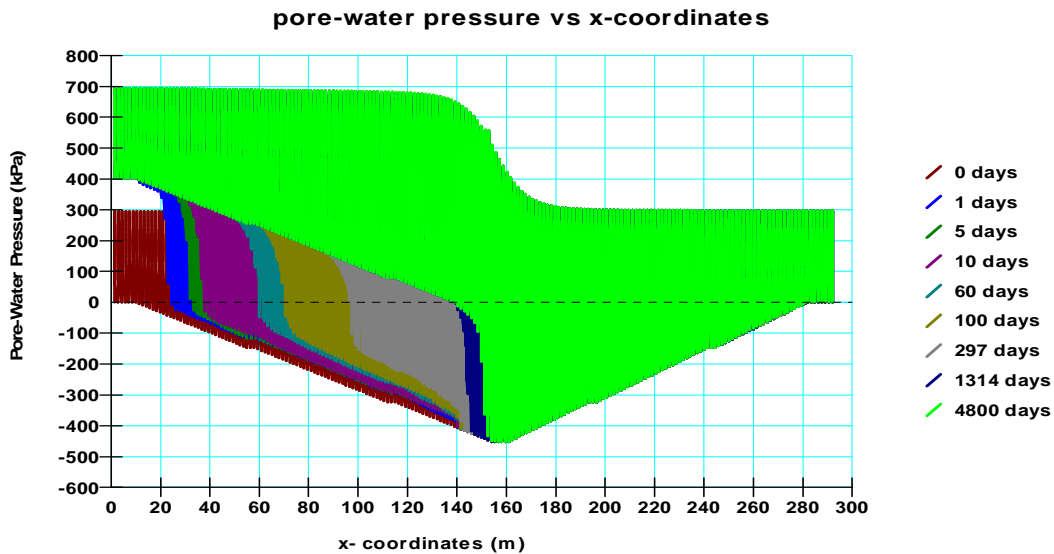


Figure 4.8 : Pore-water pressure versus x-coordinate at transient state at chainage Ch: 0+480m

4.2.3 Evaluation of Horizontal and Vertical (or Inclined) drainage blankets

In practice, the minimum thickness of a filter layer is that which can be placed without danger of gaps or discontinuity. In order to evaluate the inclined and horizontal filter thickness, seepage analyses through dam body and its foundation was conducted using SEEP/W

software. The computed seepage fluxes (Appendix Figure A2) were used to calculate the filter thickness of the dam.

Table 4.6: Evaluation of Horizontal and Vertical (or Inclined) drainage blankets

| different parameters of the dam | | | | | | Thickness of filter blanket (t) in meter | |
|---|-------------------------|--|-------------------------------------|--|---------------------------------|--|---------------------------------------|
| Seepage flux q at NPL through (m ³ /sec/m) | | Angle seepage face(α) $=\tan^{-1}$ (1/0.23) | Hydraulic gradient (i)=sin α | permeability of filter material (k) in m/sec | width of d/s shell (l) in meter | Inclined (t)=q/k.i | Horizontal (t)=(2ql/k) ^{1/2} |
| Dam body | dam body and foundation | | | | | | |
| 8.99E-07 | 3.304E-05 | 77 ⁰ | 0.97 | 0.001 | 110 | 0.001 | 2.70 |

According to Singh and Varsheny (1995), the minimum thickness of an inclined filter layer is determined from consideration of filter thickness required for drainage, allowance for intermixing with adjoining zones depending on compaction equipment, minimum width required for compaction and earthquake effects. Generally, a minimum thickness of 2.0 m inclined filter is recommended. Where filter materials are scarce, a horizontal filter width of 1.0 to 1.5 m can be also used.

For the inclined filter, the seepage flux (q) at normal pool level through dam body at angle of discharge face with horizontal (α) i.e. \tan^{-1} (1/0.23) =77⁰ and slope core zone of 0.23:1 computed at steady state was 8.99E-07 m³/sec/m (Appendix Figure A2). The hydraulic conductivity value taken was 1.0E-04 m/sec. Darcy's law was used to calculate thickness of the inclined filter (t) (equation 3.9) and hence, t=0.001 meter and considering times by a factor of safety of 2 (i.e. t=2x0.001 m=0.002 m) (Table 4.6). This shows the calculated thickness of inclined filter (0.002 m) using Darcy's law is much less than the minimum thickness (2.0m) indicated by Singh and Varsheny (1995). As it is known that, the inclined filter is used to protect the core from any possibility of piping or internal erosion, cracking and earthquake etc.

Without taking these into consideration, the dam was constructed without any inclined filter, which can endanger the stability and also the existence of the dam.

Thickness of the horizontal filter (t), calculated using Darcy's law was 2.70 m (Table 4.6) (equation 3.8). As it can be seen from the result, the calculated value is greater than both the designed (1.0 m) and the minimum thickness (1-1.5 m) indicated by Bharat and Singh (1995). This shows, large amount of seepage loss was simulated than the amount of seepage flux expected from the design of the dam. In earth fill dam, the inclined filter is connected to a horizontal drainage blanket at the dam base to lead the water drain safely without disturbing the downstream slope of the dam. In other words, the horizontal drain has to be of sufficient cross-section to convey the maximum quantity of seepage estimated to come through the dam section with a conservative factor of safety. In this dam, the horizontal filter used during construction was less than the required amount of filter thickness, which can cause accumulation of uplift water pressure and therefore, it can have stability problem on the dam.

4.3 Slope Stability analysis (using SLOPE/W model)

There are many methods which can be used in the analysis of slope stability of embankment dams, but in this study, SLOPE/W software using the Morgenstern-price method was used to evaluate the stability of dam with functional of water storage and consideration of downstream slopes under steady state seepage condition and upstream slope under sudden drawdown condition, slope stability analysis under steady state with seismic load condition and slope stability under sudden drawdown with seismic load condition. In addition, the location of phreatic surface is crucial for slope stability analysis.

4.3.1 Downstream slope stability analysis at steady state seepage condition

The stability analysis under steady state condition has been checked by considering normal pool level at 1859.0 m a.s.l for the downstream of the dam. The phreatic surface computed with the help of SEEP/W was used to set up the pore water pressure line in the stability analysis. The green colour shown in Figure 4.9 indicates the critical slip surface and factor of safety of the dam. It is known that at steady state condition, the downstream slip surface is more critical than upstream one. The downstream computed factor of safety of the dam was

1.805 (Figure 4.9) and (Table 4.8). The output result as well as the design of the dam of factor of safety is higher than the minimum standard required factor of safety (1.5) indicated by Novak et al., (2007) which is 1.742, so that can be concluded as the downstream slope of the dam as stable.

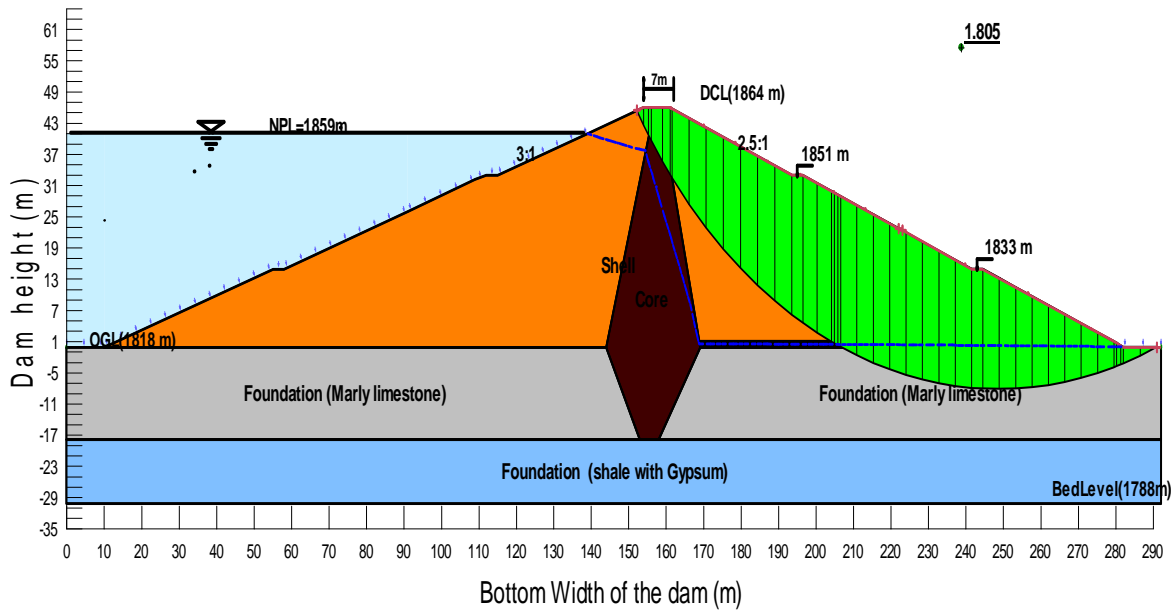


Figure 4.9: Result of stability of downstream slope for steady state seepage condition

4.3.2 Pseudo static slope stability analysis at steady state seepage

The pseudostatic force is used in a conventional limit equilibrium slope stability analysis. The seismic stability of a slope is strongly influenced by its static stability. The pseudostatic force is treated as a static force and acts in only one direction, whereas the earthquake accelerations act for only a short time and change direction, tending at certain instances in time to stabilize rather than destabilize the soil (Novak et al., 2007). The stability analysis for upstream and downstream under steady state with earthquake loading condition has been checked by considering NPL.

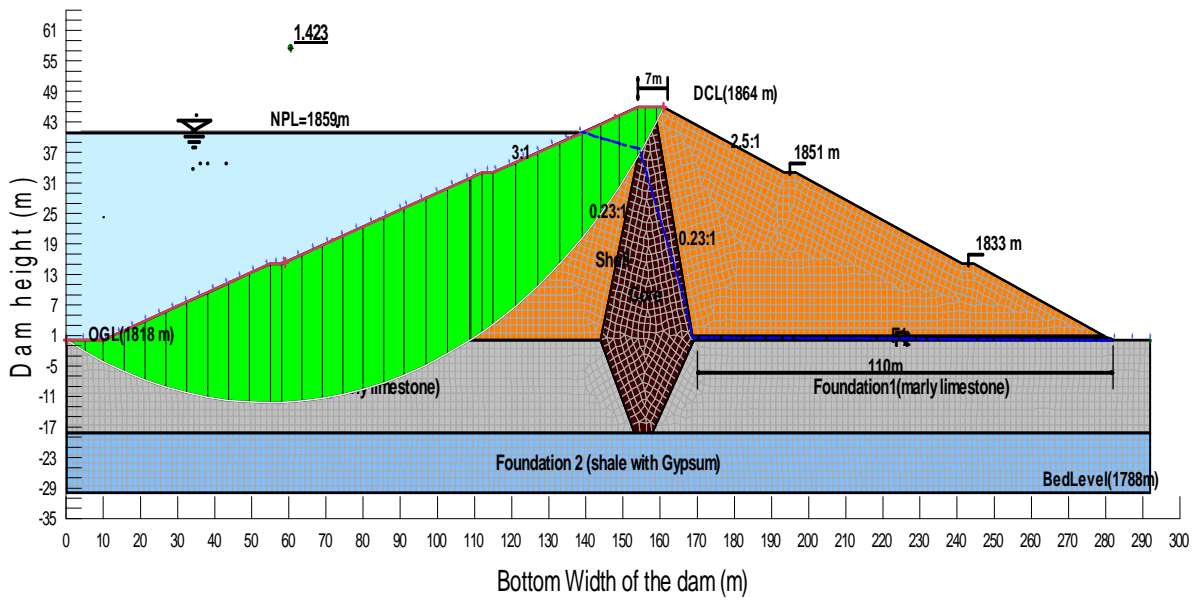


Figure 4.10: Result of stability of upstream slope for steady state seepage with seismic load

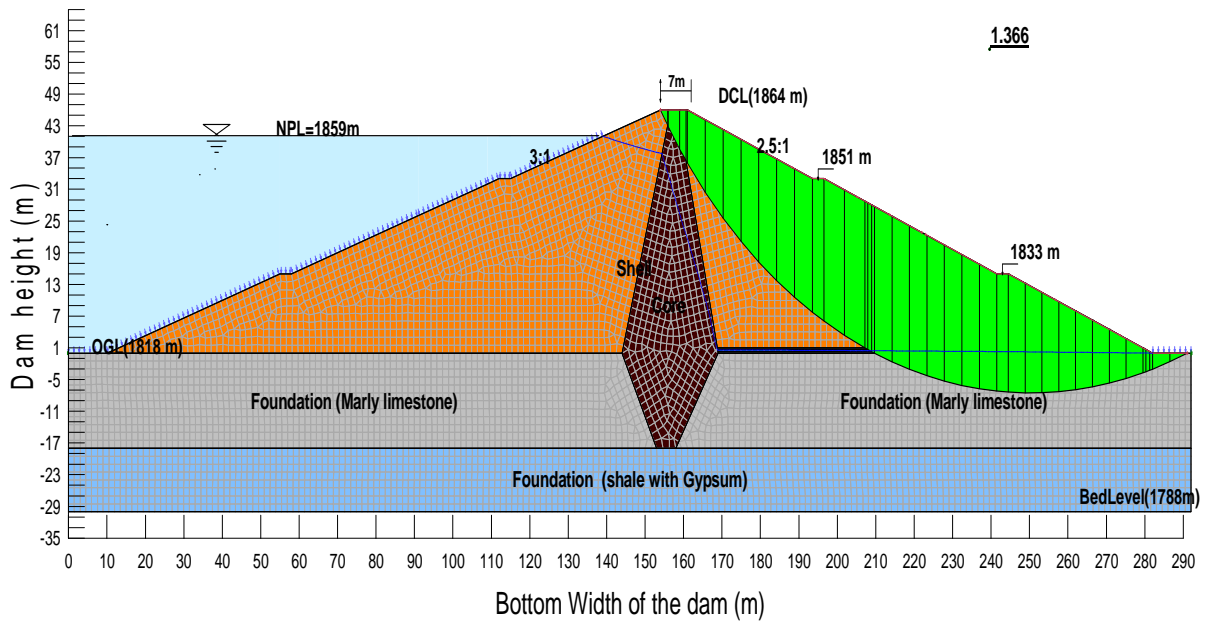


Figure 4.11: Result of stability of downstream slope for steady state with seismic load

As shown in the Figure 4.10 and Figure 4.11 the stability condition of the safety factor for upstream and downstream slope the computed factor of safety in LEM method are 1.423 and 1.366 respectively. The results of the pseudostatic steady state analysis are also shown in

Table 4.8. This result indicates a safe (i.e. achieved minimum factor of safety greater than 1.10) against slope failure for the Gerebsegen dam in the event a full reservoir.

4.3.3 Upstream slope stability analysis under sudden drawdown condition

In actually, determination of the rate of drawdown that would prevent upstream slope instability is dependent on permeability and geometry of the embankment zoning. SLOPE/W software model was used for examining the slope stability of the dam in the rapid drawdown (transient state) by limit equilibrium condition using Morgenstern-Price method of slices at critical loading conditions. The analysis of safety factor for each step was determined by using the SEEP/W analysis results as input data. For the Gerbesegen dam, the total boundary conditions pass instantaneously from 41 m (NPL) to 26 m (outlet level) with a drawdown of 15 m (Figure 4.12).

For transient state condition of the slope stability analysis of the dam, the analysis results of SEEP/W of eight time steps (1, 5, 10, 20, 30, 60 and 360 days) starting from the instantaneous time of sudden drawdown were used during the SLOPE/W software model simulation to calculate factor of safety of the constructed dam at each time step (Figure 4.12, Figure 4.13, and Table 4.7). Embankment dams may become saturated by seepage flow during long term reservoir stage. If the reservoir is drawdown quickly, can happen that the pore-water pressure cannot spillage, so high level of the pore-water pressure and consequently reduced stability will result. The case, in which the water has to be quickly lowered in presence of impermeable material in the dam slope, so that the phreatic line falls very slowly, is common problems (Tran, 2004; Pasetto, 2014). In case of rapid draw down, it is assumed that the pore water pressure within the embankment continuous to reflect the original water level. The lag of the phreatic line depends on factors such as: permeability of soil, drawdown rate and slope gradient. Highly permeable materials drain quickly during rapid drawdown, but low permeability materials take a long time to drain. The rapid drawdown condition is critical for the upstream slope of the Gerebsegen earth dam.

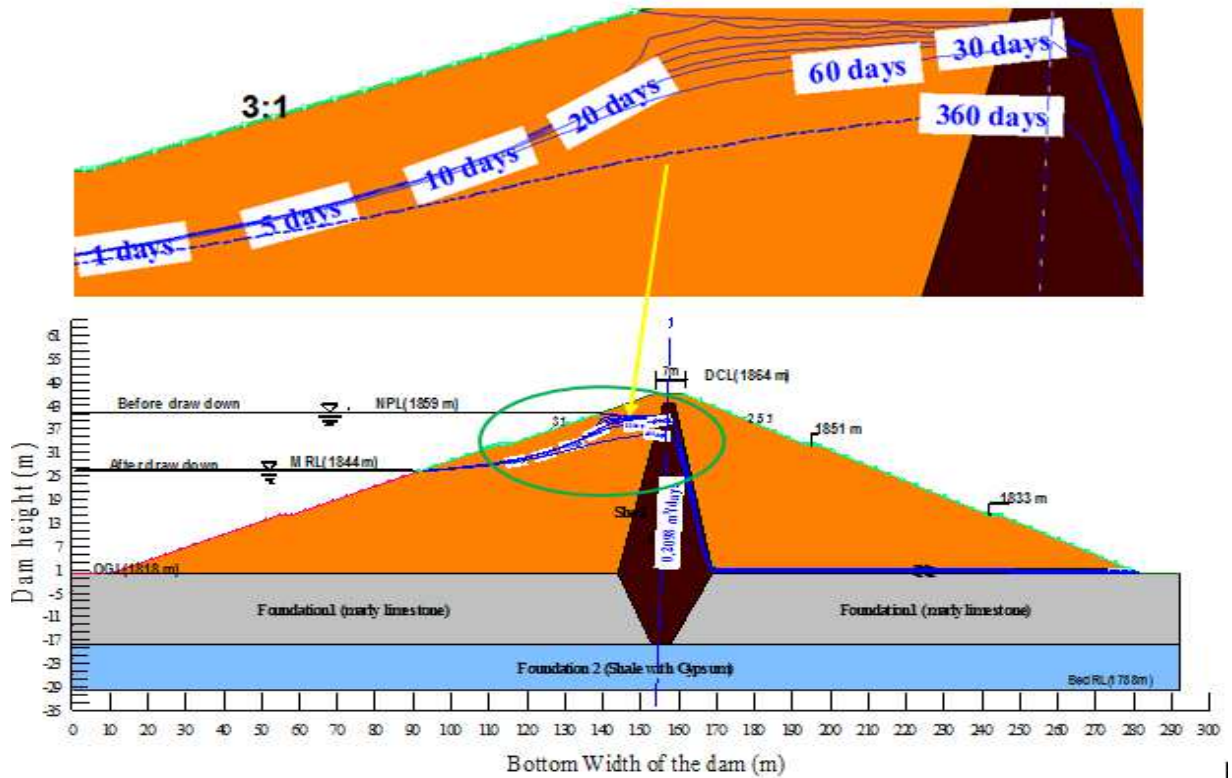


Figure 4.12: Seepage flow the days (1, 5, 10, 20, 30, 60, and 360) after rapid draw down

From the analysis results, it can be noted that the phreatic line starting from the day one to the final time stage (360 days) falls slowly, which shows the shell materials used for the construction of the dam is less permeable and as a result of which upstream slope pore water pressure of the shell material will not be dissipated quickly. In other words, due to the presence of low permeability shell material, no significant change in the saturation level inside the slope takes place when the reservoir level goes down. But in the case of factor of safety of the upstream slope stability, the result obtained throughout the whole time stage used during the analysis are in the range of safe value. The minimum factor of safety obtained was 1.833 as well as the specified in the design of the dam (1.76) exceeding the desired safety factor specified by the Novak et al., (2007). From this, it can be concluded that, the amount of materials used for the construction of the dam are excess.

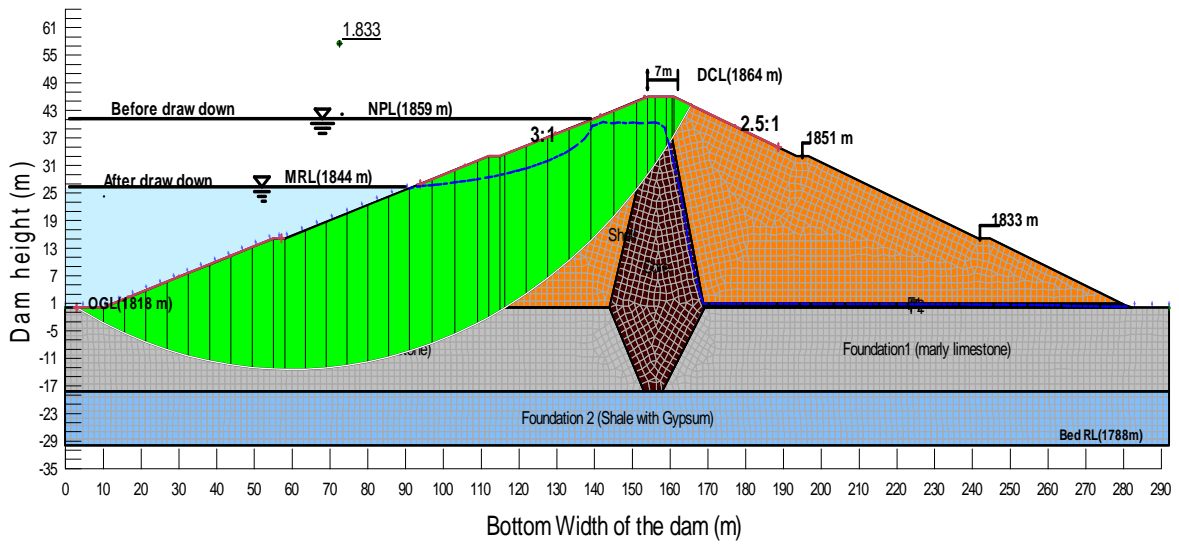


Figure 4.13: U/S slope of the minimum stability factor using SLOPE/W result with 1day sudden drop in water level of reservoir.

Table 4.7: The results safety factor of the upstream slope stability analysis under sudden draw downs condition:

| Time loading (day) | Water height in the reservoir | Recommended Factor of Safety(F _{smin}) | Computed Factors of safety | Slope stability |
|-----------------------|----------------------------------|---|----------------------------------|--------------------|
| | | Novak et al. | Morgenstern- Price Method | |
| 1 | 26 | 1.2 | 1.833 | Upstream slope |
| 5 | 26 | 1.2 | 1.836 | |
| 10 | 26 | 1.2 | 1.839 | |
| 20 | 26 | 1.2 | 1.842 | |
| 30 | 26 | 1.2 | 1.844 | |
| 60 | 26 | 1.2 | 1.847 | |
| 360 | 26 | 1.2 | 1.865 | |

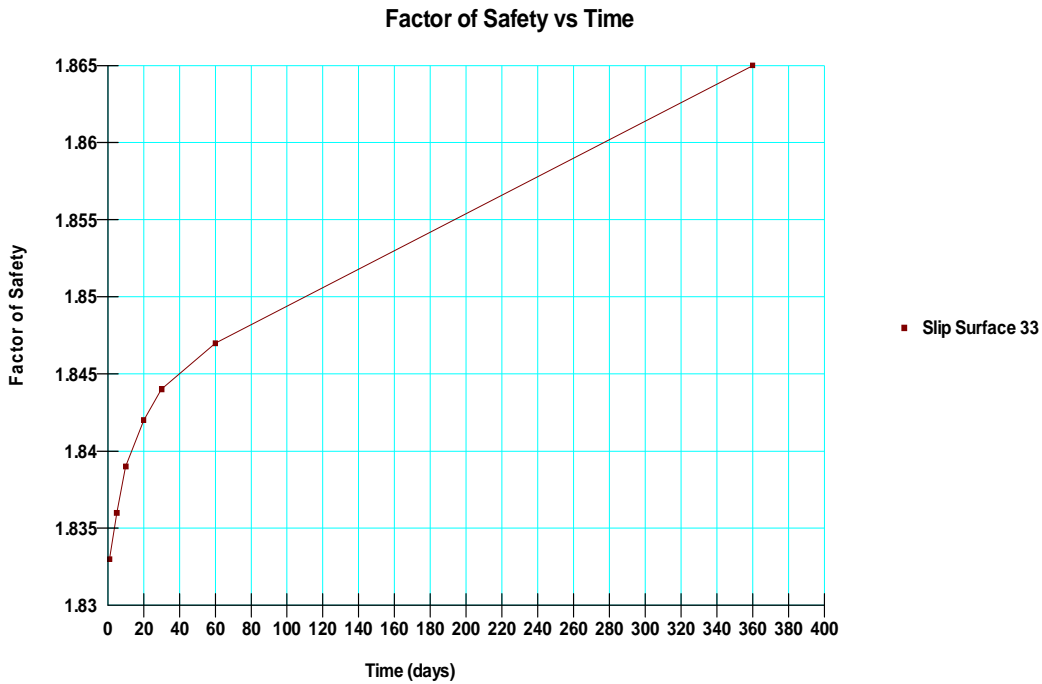


Figure 4.14: Trend of safety factor during rapid drawdown (Upstream slope)

The trend of the safety factor (Figure 4.14) shows that, the increase in safety factor throughout the analysis time stage is almost uniform.

4.3.1 Pseudo static slope stability analysis under sudden drawdown

The drawdown is known as one of the most dangerous conditions for the upstream side slope. When the countervailing upstream water pressure has disappeared, it causes a danger to the upstream slope. The upstream shell cannot stay stable under the hydrodynamic pressure due to rapid drawdown. Soils inside the dam body remain saturated and seepage commences from it towards the upstream slope. Seepage and hydrodynamic pressures create downward forces acting on the upstream slope. Those are adverse to the stability and create a critical condition to the upstream slope.

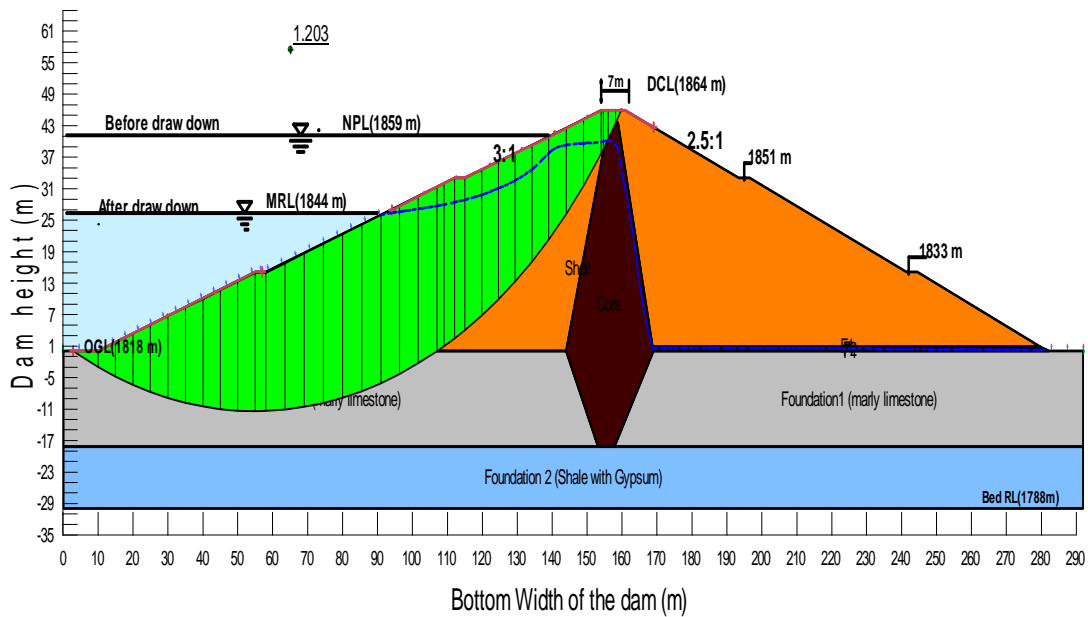


Figure 4.15: U/S slope of the minimum stability factor using SLOPE/W result after 1day sudden drop with seismic load in water level of reservoir.

In the above Figure 4.15 shows that the value and location of most critical slip surface for rapid drawdown condition with seismic load effect. It shows that the value of factor of safety of upstream decrease when rapid drawdown condition and seismic load effect. The factor of safety obtained was 1.203 more than the desired safety factor specified by the Novak et al., (2007).

Table 4.8: Summary of the computed results and recommended of the factors of safety for different loading conditions of Gerebsegen earth dam:

| Analysis Loading condition | Slope | Recommended Factor of Safety(F_{smin}) by Novak et al. | Designed F_s using Morgenstern-Price method | Computed F_s using Morgenstern-Price method | Status |
|-----------------------------------|-------|--|---|---|--------|
| Steady state seepage | D/S | 1.50 | 1.742 | 1.805 | Ok |
| Steady state seepage with seismic | U/S | 1.10 | 1.388 | 1.423 | Ok |
| | D/S | 1.10 | 1.263 | 1.366 | Ok |
| Sudden draw down | U/S | 1.20 | 1.76 | 1.833 | Ok |
| Sudden draw down with seismic | U/S | 1.10 | 1.246 | 1.203 | Ok |

The summary result safety of factors of the performed analysis, it can be concluded that the Gerebsegen dam satisfies the entire requirement by the Novak et al., (2007) recommendations as shown in the Table 4.8. The stability analysis results of computed factors of safety, which have almost similar as compared the specified in the designed factors of safety as stable against slope failure under the four different loading conditions.

5 Conclusions and Recommendations

5.1 Conclusions

- The excessive seepage flow from Gerebsegen dam can have a future impact on the constructed dam, for which the amount of water flow coming through the central foundation and abutments was measured using current meter and float method. The total combination amount of seepage measured was $0.2053 \text{ m}^3/\text{sec}$ and the annual amount of water loses due to seepage from 1851.05 m to 1847.75 m a.s.l computed was $6,474,340.80 \text{ m}^3/\text{year}$, which matches the amount of water expected to be supplied for Mekelle water supply. As the purpose of the constructed dam is for both Mekelle city water supply and irrigation; a water use conflict between the irrigation user farmers and the city could be an issue for the coming years.
- A computer model based on finite element method using SEEP/W software has been used to analyze the seepage rate through dam body and its foundation at steady state condition. By selecting three different sections of the dam, the average seepage flux computed at the normal pool level (1859 m a.s.l) and the actual reservoir level (1850 m a.s.l) for every meter length of reservoir were $3.304 \times 10^{-5} \text{ m}^3/\text{sec}/\text{m}$ and $2.252 \times 10^{-5} \text{ m}^3/\text{sec}/\text{m}$ respectively. But the value computed during designing of the project where only one section of central foundation was considered, and so only one hydraulic parameter (conductivity) taken was $1.031 \times 10^{-7} \text{ m}^3/\text{sec}/\text{m}$, which ignores the two left and right foundations which are found to be pervious and critical to the water tightness of the dam. As a result of this, the amount of seepage computed using the model, taking the hydraulic parameters measured during the designing time of the project become less than the actual amount of seepage measured during this study.
- According to the previous studies done on the area, the geological coverage of the reservoir area and abutments is dominated by weathered dolerite overlying highly to completely weathered marl-limestone, interpreted as pervious and easily erodible. In addition, the geology of the area was affected by jointing; weathering process, fault and the present of thin bedded plane can act as pass way for water/seepage from the

dam reservoir to the downstream of the dam. The amount of seepage computed using the model by taking the average hydraulic parameters measured by Gebremedhin et al., (2016), during their study for Hizaetiwedicheber dam near to the study area become $2.503E-04 \text{ m}^3/\text{sec}/\text{m}$ ($0.2039 \text{ m}^3/\text{sec}$), which is relatively near to the actual amount of excess seepage measured during this study. This excess seepage, which was not taken in to account during the design process may have an adverse impact on the constructed dam like piping of the dam foundation effect. Form this, it can be concluded that designing or constructing of a dam without detail investigation can limit its potential to fulfill the demanded target.

- For the transient analysis, the output and evaluation result of the analysis indicate, both the seepage quantity and pore- water pressure were increased gradually to the final steady state flux at rapid filling of reservoir time stages. In addition, the gap between the position of phreatic line of the dam and that of the result obtained from the steady state condition is relatively large, which in turn needs long time to attain the normal phreatic line and therefore saturating of the shell materials of the dam. This can be due to using of low permeability shell construction material, which can play a significant role in the stability of the dam.
- In earth dam, the inclined filter is connected to a horizontal drainage blanket at the dam base to lead the water drain safely without disturbing the downstream slope of the dam. But the dam was constructed without placing of inclined filter and therefore will lose the service of the filter mentioned above.
- The factor of safety computed from the downstream slope stability analysis at steady state seepage and steady state seepage with seismic load condition as well as upstream slope stability at rapid drawdown condition and rapid drawdown with seismic load condition using SLOPE/W model, and also the factor of safety indicated in the design of the dam is higher than the minimum standard required factor of safety indicated by Novak et al., (2007), which can be concluded as the downstream and upstream slope of the dam as stable against slope failure. But the uncontrolled excessive seepage can have an adverse impact on the constructed earth dam; for which the stability of the dam against the excessive seepage.

5.2 Recommendations

- As it was observed during this study, the hydraulic parameters of the geological materials of the central and both abutments of the dam were underestimated, therefore detailed investigation should be done in order to assess the possibility of minimizing the excessive seepage emanating from the dam.
- As it was known that uncontrolled seepage can have an adverse impact on any constructed earth dam, the stability of the dam against the excessive seepage should be assessed regularly. This can be used as an input for further controlling of its sustainability of the dam.
- The most appropriate actions are installation of drainage measures and barriers which would minimize the risk of piping, cracking, or increase pore-water pressure. The practical option for the Gerebsegen dam should be extend an upstream impervious blanket to cover over the permeable geological formation on both abutments. In addition, the most practical options would be to construct upstream impervious blanket, to improve the capacity of the downstream relief wells and pervious blanket should be installed with ditch drains at the particular location, this should be properly maintained such that the seepage water could at least be used-provided of course that the ongoing seepage does not compromise the safety of the dam.
- As it was observed 65% of the amount of seepage measured during this study, which can be used for irrigation with very little effort comes from the left abutment, along the alignment of the irrigation canal. Therefore using this water until the remedial measures of seepage minimizing mechanism are implemented can play a measure role in minimizing or avoiding water use conflict between the irrigation user farmers and Mekelle city water supply.
- Any possibility of using the excessive seepage coming from the reservoir, which matches the amount of water expected to be supplied for Mekelle water supply by constructing any structure downstream of the dam should be assessed. This option can

be applied if the stability of the dam against the excessive seepage from the reservoir is confirmed.

- In order to have detailed information on how the amount of water for irrigation and Mekelle water supply should be allocated, detailed water balance of the area should be done by considering the actual amount of seepage from the reservoir.

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Appendices

Appendix Figure A: SEEP/W Software results for steady state at Normal pool level

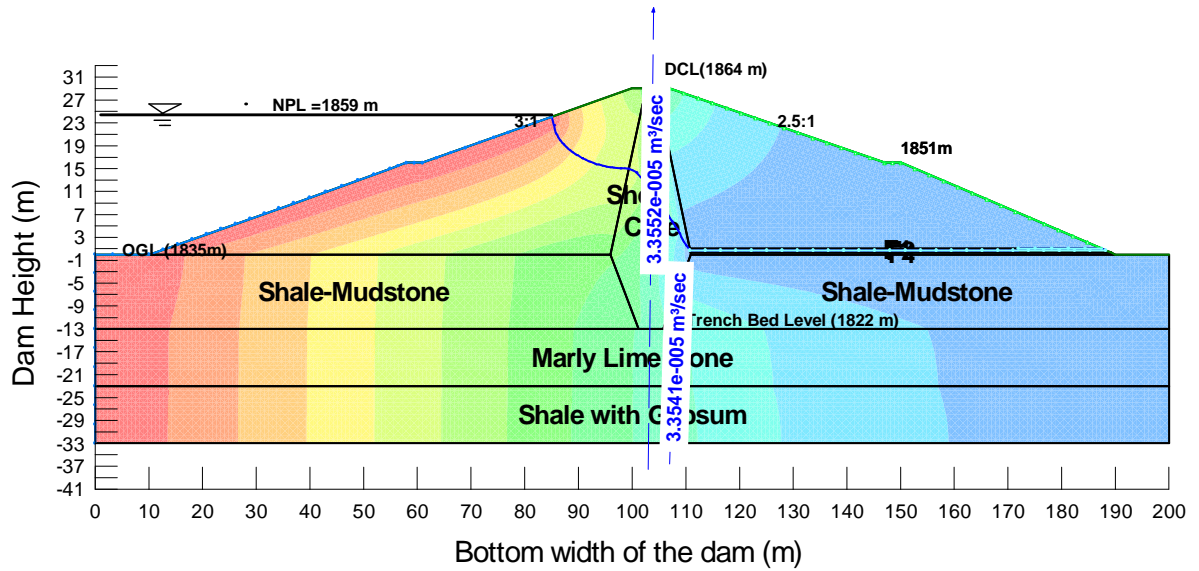


Figure A 1: Seepage through the dam body and foundation at the chainage Ch: 0+065m

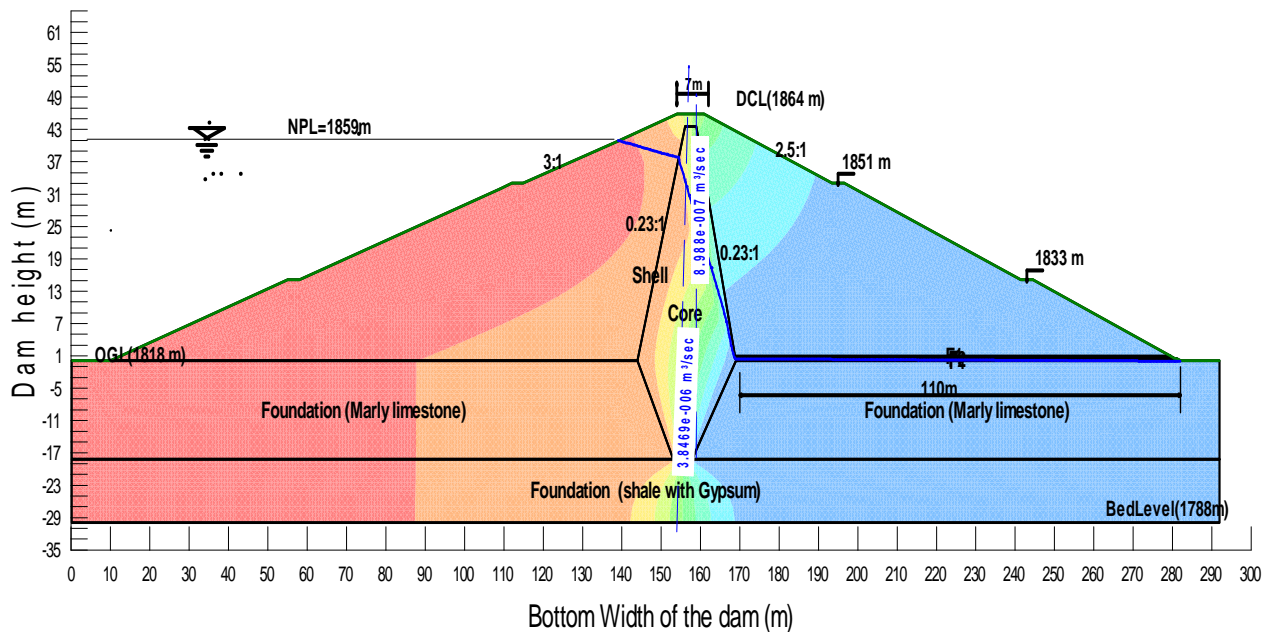


Figure A 2: Seepage through the dam body and foundation at the chainage Ch: 0+480m

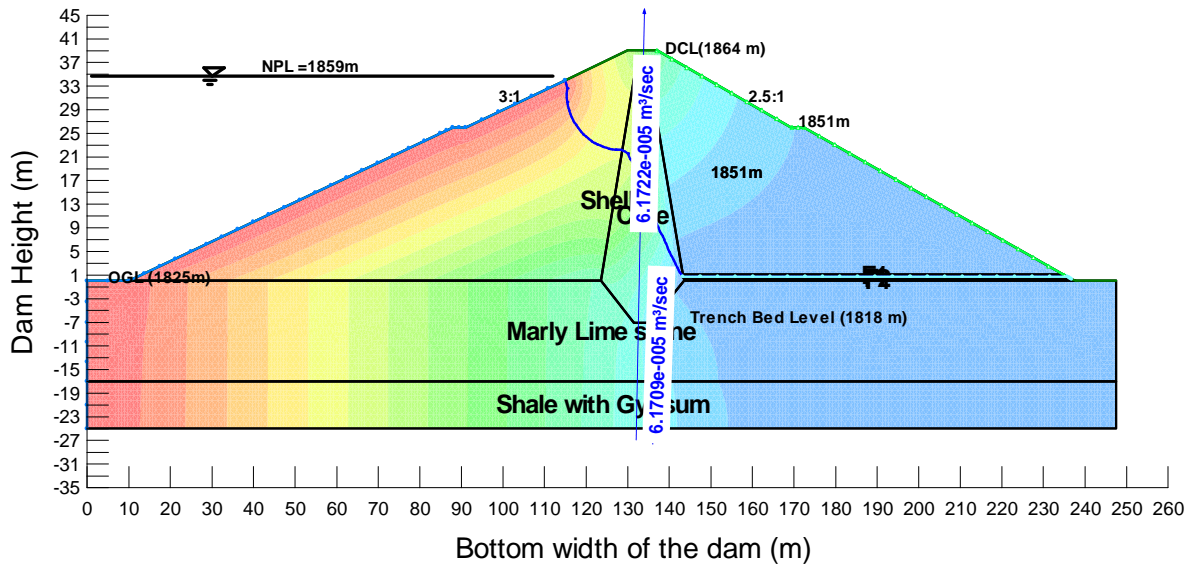


Figure A 3: Seepage through the dam body and foundation at the chainage Ch: 0+638m

Appendix Figure B- SEEP/W results at steady state when revised $k=2.43E-04m/sec$ for marly limestone foundation

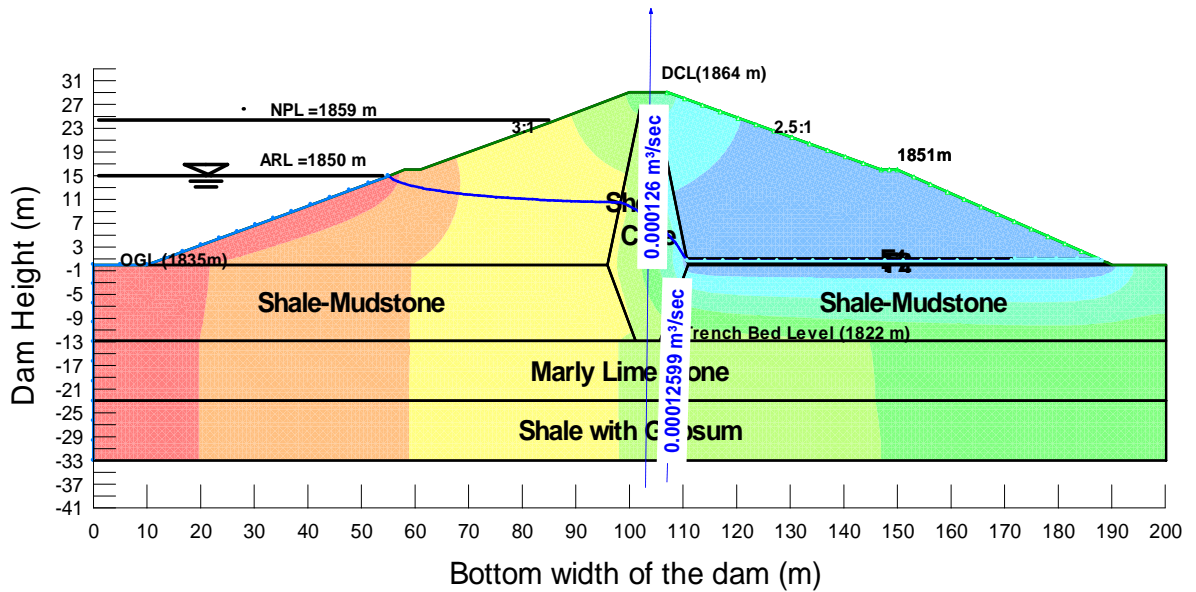


Figure B. 1: Seepage through the dam body and foundation at the chainage Ch: 0+065m

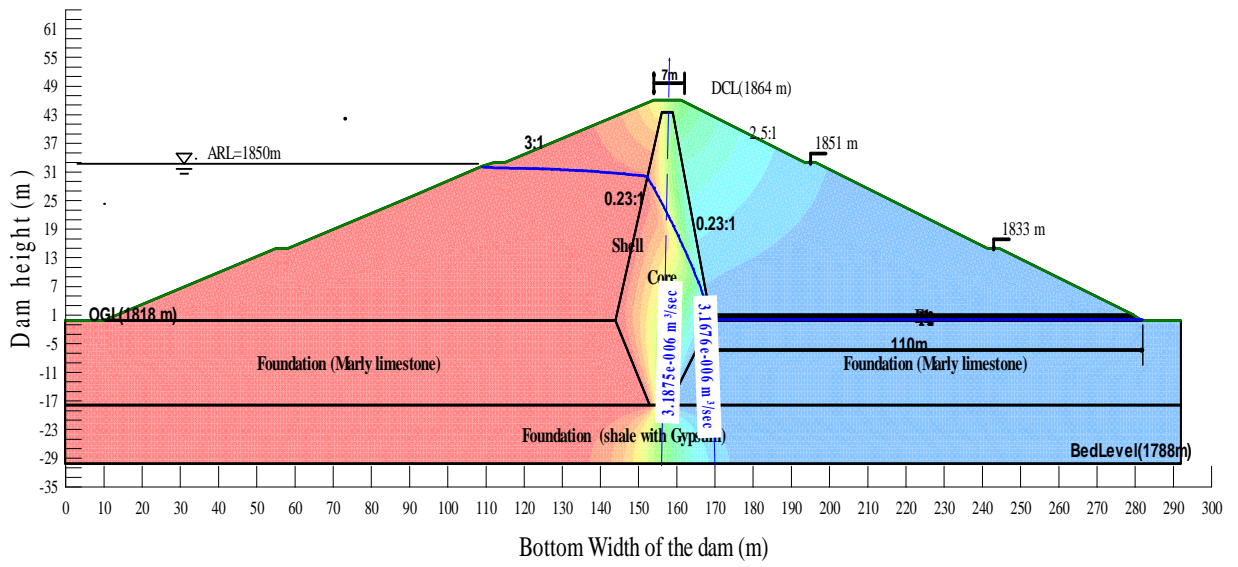


Figure B. 2: Seepage through the dam body and foundation at the chainage Ch: 0+480m

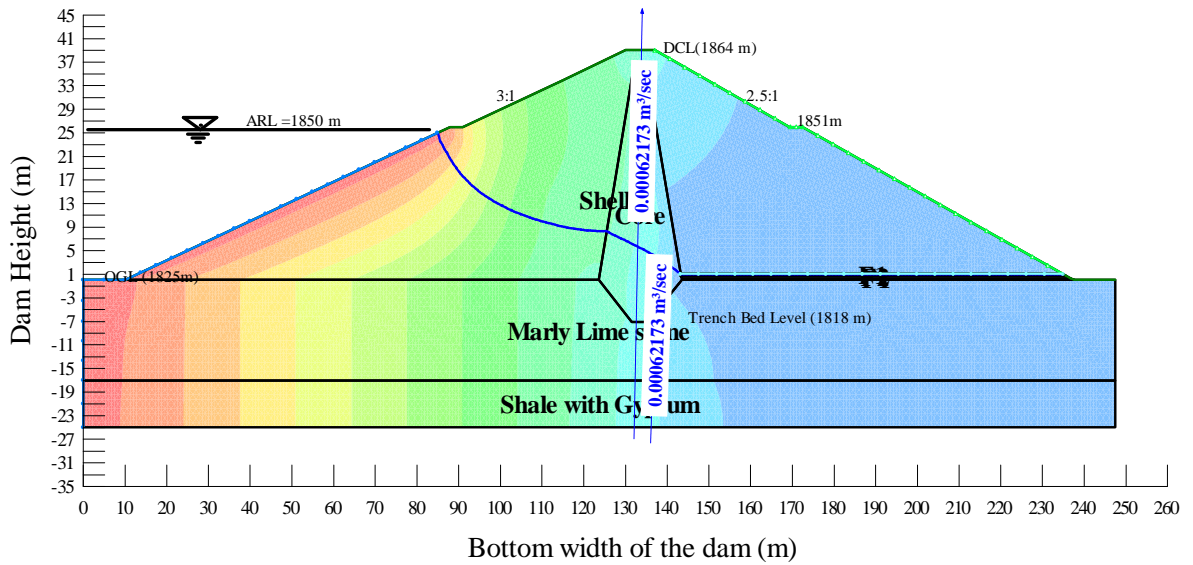


Figure B. 3: Seepage through the dam body and foundation at the chainage Ch: 0+638m

Appendix Figure C: Slope stability results using SLOPE/W under rapid draw down.

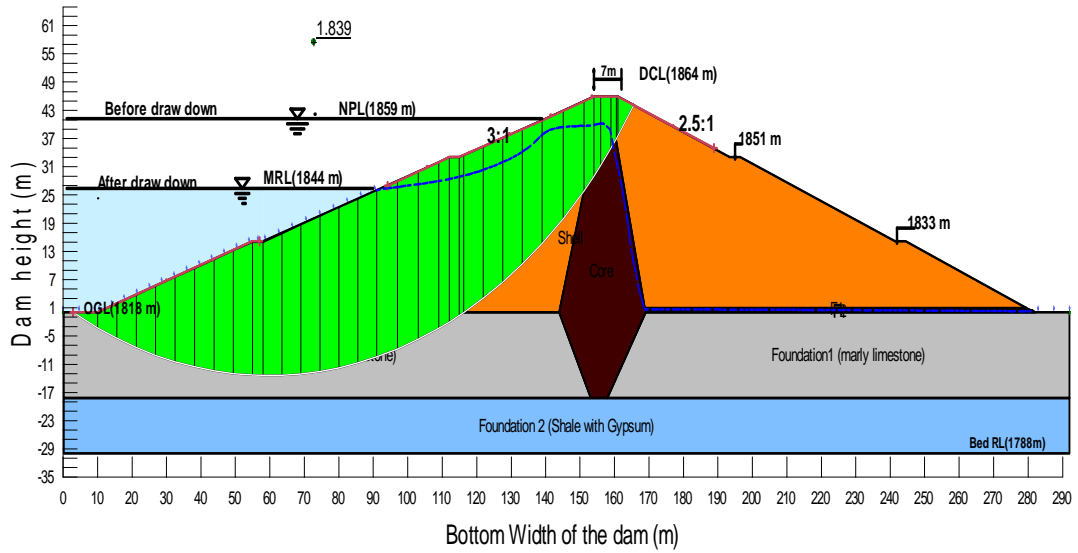


Figure C. 1: U/S slope of the factor safety using SLOPE/W result within 5day sudden drop in water level of reservoir.

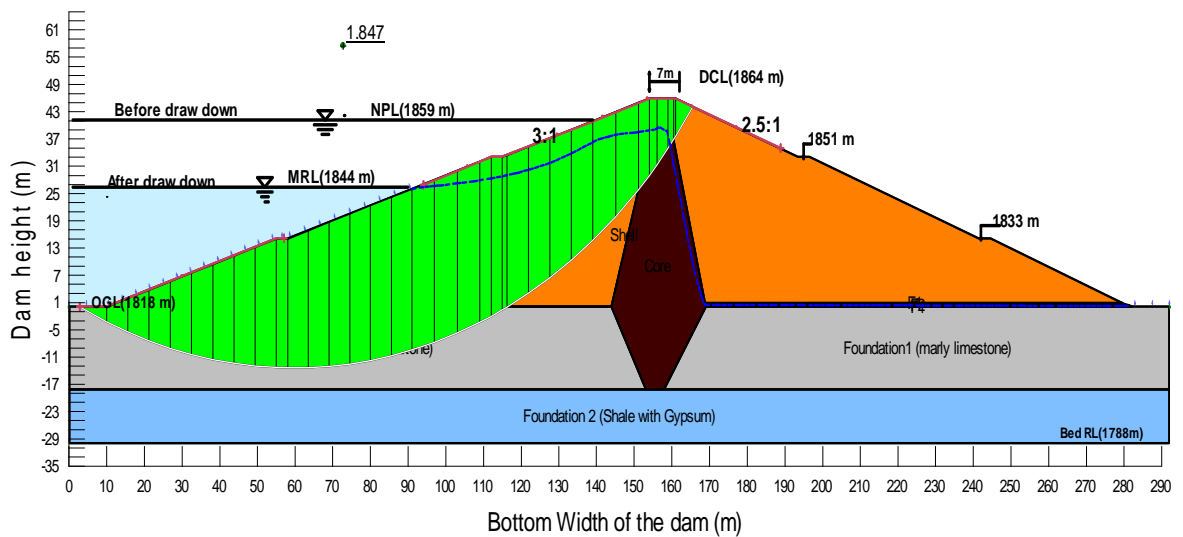


Figure C. 2: U/S slope of the factor safety using SLOPE/W result within 60day sudden drop in water level of reservoir.

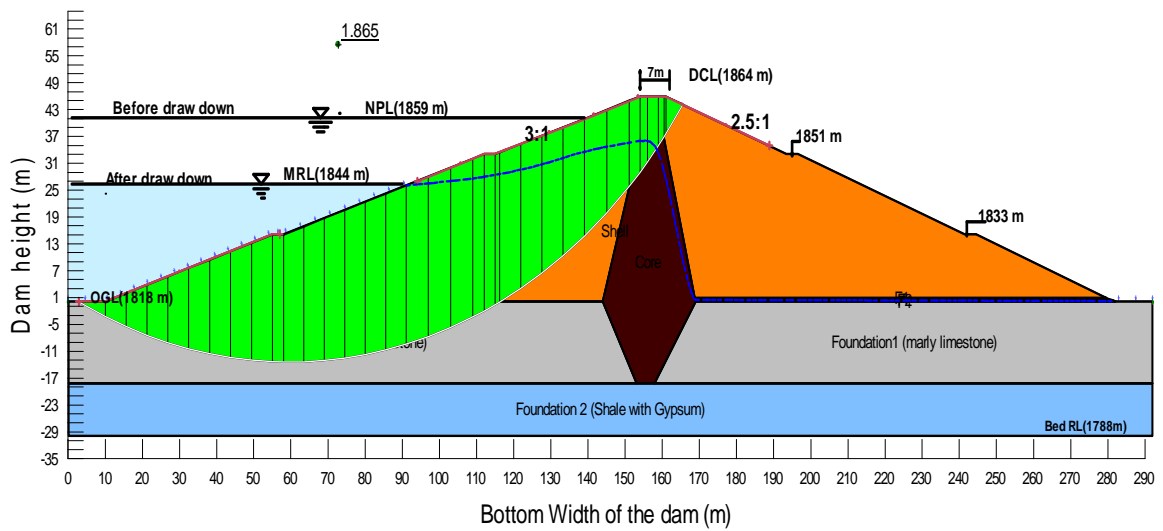


Figure C. 3: U/S slope of the factor safety using SLOPE/W result within 360day sudden drop in water level of reservoir.



Appendix Figure D: The existing seepage at different location area on the downstream of the dam.



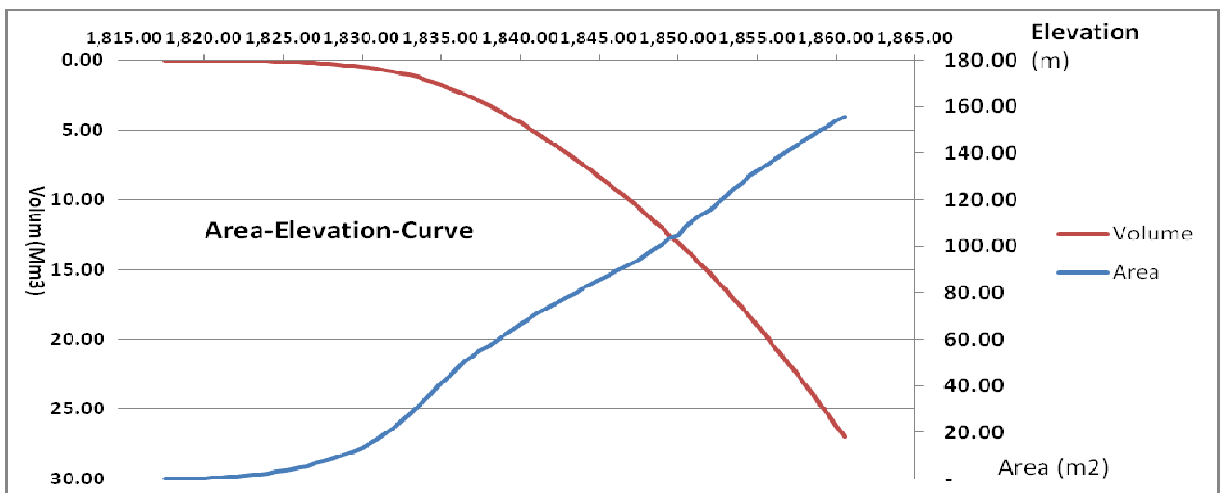
Appendix Figure E: location of seepage on the downstream area of the dam.



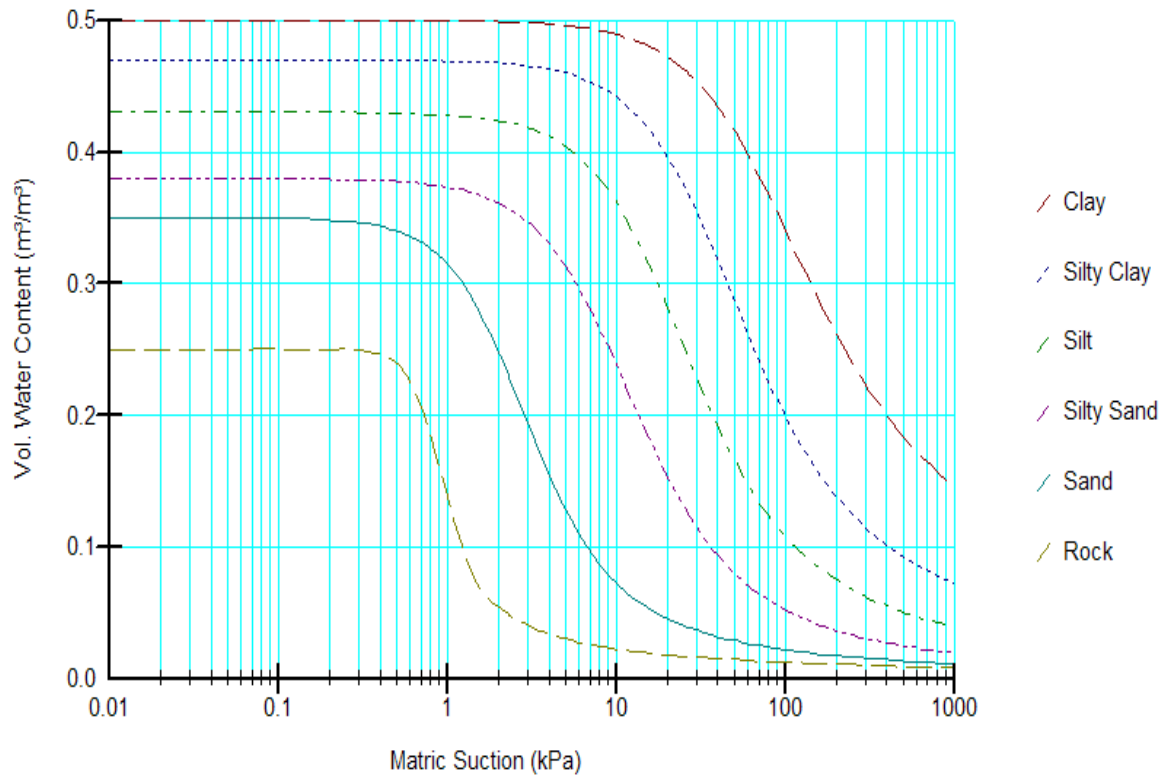
Appendix Figure F: during the trench excavation of the geological foundation (shale-limestone intercalation in the Gerebsegen dam).



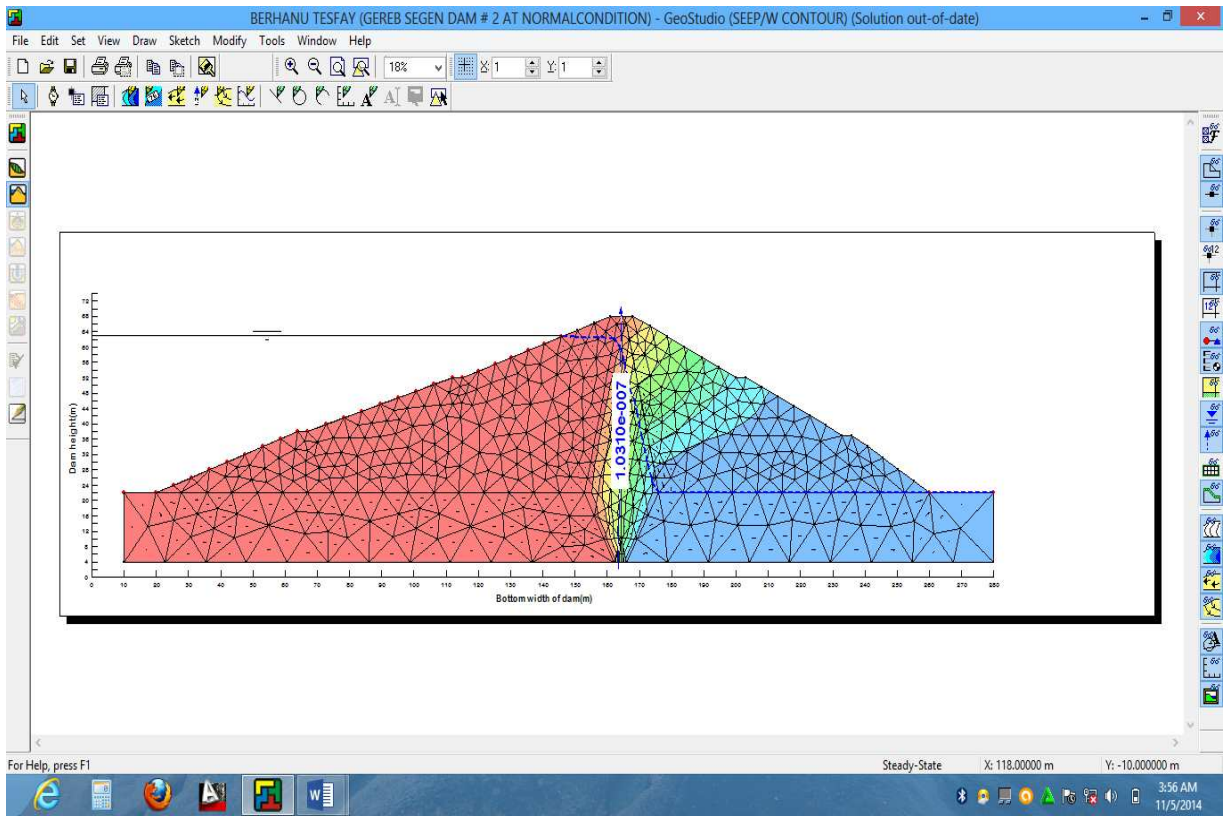
Appendix Figure G: Photographs showing the weathering open jointed and voids on the left reservoir rim area.



Appendix Figure H: Area-Elevation-Curve of Gerebsegen dam (TWRB, 2014)



Appendix Figure I: Volumetric water content sample function from (SEEP/W, 2008).



Appendix Figure J: Seepage flux through the dam body and foundation at the chainage Ch: 0+480m (TWRB, 2014)

Appendix table 1 : Summery seepage discharge measurements using Current meter and Float method during at the field time

| Date observation | Actual Reservoir Level(m) | Location seepage | | | | | | | | Design of reservoir | |
|--|---------------------------|-----------------------------------|-------------|----------------------------------|-------------|------------------------------------|-------------|-----------------------------------|-------------|---------------------|-----------------------------|
| | | seepage one (m ³ /sec) | | seepage two(m ³ /sec) | | seepage three(m ³ /sec) | | seepage four(m ³ /sec) | | Elevation (m.a.s.l) | Capacity (Mm ³) |
| | | Curent meter method | Float methd | Curent meter method | Float methd | Curent meter method | Float methd | Curent meter method | Float methd | | |
| 28/01/2017 | 1851.05 | 0.116 | 0.112 | 0.021 | 0.020 | 0.021 | 0.020 | 0.05319 | 0.0525 | 1851.00 | 14.16 |
| 02/02/2017 | 1850.94 | 0.115 | 0.107 | 0.021 | 0.019 | 0.019 | 0.020 | 0.05243 | 0.053 | | |
| 08/02/2017 | 1850.82 | 0.114 | 0.105 | 0.021 | 0.022 | 0.020 | 0.019 | 0.05247 | 0.0446 | | |
| 22/02/2017 | 1850.67 | 0.114 | 0.121 | 0.020 | 0.018 | 0.022 | 0.019 | 0.05191 | 0.0474 | 1850.50 | 13.61 |
| 10/03/2017 | 1850.47 | 0.114 | 0.111 | 0.020 | 0.020 | 0.021 | 0.021 | 0.05214 | 0.0558 | | |
| 24/03/2017 | 1850.25 | 0.114 | 0.113 | 0.021 | 0.019 | 0.021 | 0.019 | 0.05194 | 0.053 | 1850.00 | 13.07 |
| 09/04/2017 | 1849.86 | 0.114 | 0.102 | 0.020 | 0.020 | 0.021 | 0.018 | 0.05319 | 0.053 | | |
| 23/04/2017 | 1849.65 | 0.114 | 0.115 | 0.020 | 0.021 | 0.022 | 0.020 | 0.05194 | 0.0474 | 1849.50 | 12.55 |
| 09/05/2017 | 1849.27 | 0.112 | 0.114 | 0.020 | 0.022 | 0.021 | 0.020 | 0.05243 | 0.0558 | | |
| 23/05/2017 | 1849.11 | 0.113 | 0.117 | 0.019 | 0.020 | 0.022 | 0.018 | 0.04953 | 0.0558 | 1849.00 | 12.04 |
| 08/06/2017 | 1848.80 | 0.114 | 0.102 | 0.019 | 0.020 | 0.021 | 0.020 | 0.05263 | 0.053 | | |
| 22/06/2017 | 1848.25 | 0.112 | 0.111 | 0.019 | 0.019 | 0.022 | 0.020 | 0.05152 | 0.053 | 1848.50 | 11.54 |
| 08/07/2017 | 1848.05 | 0.111 | 0.113 | 0.019 | 0.018 | 0.021 | 0.018 | 0.04851 | 0.0474 | 1848.00 | 11.05 |
| 06/08/2017 | 1847.75 | 0.111 | 0.111 | 0.014 | 0.013 | 0.020 | 0.0179 | 0.04434 | 0.0474 | 1847.50 | 10.57 |
| Q _{average seepage loses} | | 0.1134 | 0.1110 | 0.020 | 0.019 | 0.021 | 0.0194 | 0.0513 | 0.0514 | 3.5 | 3.59 |
| Tatal seepage Losses | | | | | | | | 0.205 | | | |
| Tatal seepage Losses from 28/01/2017 to 06/08/2017 (1851.05m-1847.75 m a.s.l) | | | | | | | | 3,369,192.35 | | 3,590,000.00 | |
| The variation may be due evaporation and other consuptions | | | | | | | | | | 220,807.65 | |

Appendix table 2: Field data seepage discharge measurements using float method along D/S left abutment (seepage -1 and 2)

Table:Field Data Sheet Showing Discharge Measurements using the float Method

| Location seepage -along Left abutment(seepage- 1) | | 1000 | Coefficient | Month, from January -July,2009E.C | | | | | | | | | | | | | | |
|---|------------------------|-------------------|-------------|-----------------------------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|-----------------------|-------|
| No | Description | unit | 28/01/2017 | 02/02/2017 | 08/02/2017 | 22/03/2017 | 10/03/2017 | 24/03/2017 | 09/04/2017 | 23/04/2017 | 09/05/2017 | 23/05/2017 | 08/06/2017 | 22/06/2017 | 08/07/2017 | 06/08/2017 | Average Discharge (Q) | |
| 1 | Average Width(w) | m | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | | |
| 2 | Average Depth(h) | m | 0.15 | 0.16 | 0.145 | 0.15 | 0.155 | 0.15 | 0.145 | 0.145 | 0.148 | 0.155 | 0.15 | 0.145 | 0.15 | 0.145 | | |
| 3 | Area (A)=wxh | m ² | 0.225 | 0.24 | 0.2175 | 0.225 | 0.2325 | 0.225 | 0.2175 | 0.2175 | 0.222 | 0.2325 | 0.225 | 0.2175 | 0.225 | 0.2175 | | |
| 4 | Run length(L) | m | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | 11 | | |
| 5 | Average travel Time(t) | Sec | 18 | 20 | 19 | 20 | 18 | 19 | 18 | 20 | 18 | 19 | 18 | 20 | 19 | 18 | | |
| 6 | Velocity(V)=0.85(L/t) | m/s | 0.519 | 0.468 | 0.492 | 0.468 | 0.519 | 0.492 | 0.519 | 0.468 | 0.519 | 0.492 | 0.519 | 0.468 | 0.492 | 0.519 | | |
| 7 | Discharge (Q)=A.V | m ³ /s | 0.117 | 0.112 | 0.107 | 0.105 | 0.121 | 0.111 | 0.113 | 0.102 | 0.115 | 0.114 | 0.117 | 0.102 | 0.111 | 0.113 | | 0.111 |

| Location seepage -along Left abutment(seepage- 2) | | 1000 | Coefficient | Month, from January -July,2009E.C | | | | | | | | | | | | | | |
|---|------------------------|-------------------|-------------|-----------------------------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|-----------------------|-------|
| Sr.No | Description | unit | 28/01/2017 | 02/02/2017 | 08/02/2017 | 22/03/2017 | 10/03/2017 | 24/03/2017 | 09/04/2017 | 23/04/2017 | 09/05/2017 | 23/05/2017 | 08/06/2017 | 22/06/2017 | 08/07/2017 | 06/08/2017 | Average Discharge (Q) | |
| 1 | Average Width(w) | m | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.5 | | |
| 2 | Average Depth(h) | m | 0.08 | 0.075 | 0.085 | 0.075 | 0.082 | 0.074 | 0.08 | 0.078 | 0.082 | 0.077 | 0.08 | 0.075 | 0.07 | 0.075 | | |
| 3 | Area (A)=wxh | m ² | 0.06 | 0.0563 | 0.0638 | 0.0563 | 0.0615 | 0.0555 | 0.06 | 0.0585 | 0.0615 | 0.0578 | 0.06 | 0.0563 | 0.0525 | 0.0375 | | |
| 4 | Run length(L) | m | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | | |
| 5 | Average travel Time(t) | Second | 15 | 15 | 15 | 16 | 16 | 15 | 15 | 14 | 14 | 15 | 15 | 15 | 15 | 15 | | |
| 6 | Velocity(V)=0.85(L/t) | m/s | 0.340 | 0.340 | 0.340 | 0.319 | 0.319 | 0.340 | 0.340 | 0.364 | 0.364 | 0.340 | 0.340 | 0.340 | 0.340 | 0.340 | | |
| 7 | Discharge (Q)=A.V | m ³ /s | 0.0204 | 0.0191 | 0.0217 | 0.0179 | 0.0196 | 0.0189 | 0.0204 | 0.0213 | 0.0224 | 0.0196 | 0.0204 | 0.0191 | 0.0179 | 0.0128 | | 0.019 |

Appendix table 3: Field data seepage discharge measurements using float method along D/S right abutment and horizontal filter (seepage -3 and 4).

Location seepage -along D/S rock toe (seepage- 4)

| | | 1000 | Coefficient | | 0.85 | | Month, from January -july,2009E.C | | | | | | | | | | | | | |
|-------|------------------------|-------------------|-------------|------------|------------|------------|-----------------------------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|-----------------------|--|--|--|
| Sr.No | Description | unit | 28/01/2017 | 02/02/2017 | 08/02/2017 | 22/02/2017 | 10/03/2017 | 24/03/2017 | 09/04/2017 | 23/04/2017 | 09/05/2017 | 23/05/2017 | 08/06/2017 | 22/06/2017 | 08/07/2017 | 06/08/2017 | Average Discharge (Q) | | | |
| 1 | Average Width(w) | m | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | | | | |
| 2 | Average Depth(h) | m | 0.1 | 0.095 | 0.085 | 0.085 | 0.1 | 0.095 | 0.095 | 0.085 | 0.1 | 0.1 | 0.095 | 0.095 | 0.085 | 0.085 | | | | |
| 3 | Area (A)=wxh | m ² | 0.15 | 0.1425 | 0.1275 | 0.1275 | 0.15 | 0.1425 | 0.1425 | 0.1275 | 0.15 | 0.15 | 0.1425 | 0.1425 | 0.1275 | 0.1275 | | | | |
| 4 | Run length(L) | m | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | 7 | | | | |
| 5 | Average travel Time(t) | Second | 17 | 16 | 17 | 16 | 16 | 16 | 16 | 16 | 16 | 16 | 16 | 16 | 16 | 16 | | | | |
| 6 | Velocity(V)=0.85(L/t) | m/s | 0.350 | 0.372 | 0.350 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | 0.372 | | | | |
| 7 | Discharge (Q)=A.V | m ³ /s | 0.053 | 0.053 | 0.045 | 0.047 | 0.056 | 0.053 | 0.053 | 0.047 | 0.056 | 0.056 | 0.053 | 0.053 | 0.047 | 0.047 | 0.0514 | | | |

Location seepage -along right abutment(Seepage- 3)

| | | 1000 | Coefficient | | 0.85 | | Month, from January -july,2009E.C | | | | | | | | | | | | | |
|-------|------------------------|-------------------|-------------|------------|------------|------------|-----------------------------------|------------|------------|------------|------------|------------|------------|------------|------------|------------|-----------------------|--|--|--|
| Sr.No | Description | unit | 28/01/2017 | 02/02/2017 | 08/02/2017 | 22/02/2017 | 10/03/2017 | 24/03/2017 | 09/04/2017 | 23/04/2017 | 09/05/2017 | 23/05/2017 | 08/06/2017 | 22/06/2017 | 08/07/2017 | 06/08/2017 | Average Discharge (Q) | | | |
| 1 | Average Width(w) | m | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | | | | |
| 2 | Average Depth(h) | m | 0.08 | 0.075 | 0.075 | 0.07 | 0.077 | 0.075 | 0.07 | 0.078 | 0.075 | 0.07 | 0.078 | 0.075 | 0.07 | 0.07 | | | | |
| 3 | Area (A)=wxh | m ² | 0.06 | 0.0563 | 0.0563 | 0.0525 | 0.0578 | 0.0563 | 0.0525 | 0.0585 | 0.0563 | 0.0525 | 0.0585 | 0.0563 | 0.0525 | 0.0525 | | | | |
| 4 | Run length(L) | m | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | 6 | | | | |
| 5 | Average travel Time(t) | Second | 15 | 14 | 15 | 14 | 14 | 15 | 15 | 15 | 14 | 15 | 15 | 14 | 15 | 15 | | | | |
| 6 | Velocity(V)=0.85(L/t) | m/s | 0.340 | 0.364 | 0.340 | 0.364 | 0.364 | 0.340 | 0.340 | 0.340 | 0.364 | 0.340 | 0.340 | 0.364 | 0.340 | 0.340 | | | | |
| 7 | Discharge (Q)=A.V | m ³ /s | 0.020 | 0.020 | 0.019 | 0.019 | 0.021 | 0.019 | 0.018 | 0.020 | 0.020 | 0.018 | 0.020 | 0.020 | 0.018 | 0.018 | 0.019 | | | |

Appendix table 4: Field data seepage discharge measurements using current meter method along D/S left abutment seepage-1

Table:Field data sheet showing discharge measurements using the current meter method

| Location | Left abutment @chainage 300m from dam axis (Seepage-1) | | | | | 0.000854 |
|--------------|--|--------------|--------------------------------|-------------------|--|---------------------------------------|
| Velocity (V) | 0.000854C+0.05 | | | | | 0.05 |
| Date | Width (W) of interval(m) | Depth(d)in m | Area(A)=W x d(m ²) | Counts (C)/minute | Velocity @0.6d (V)=0.000854C +0.05 (m/s) | Discharge (Q) =A*V(m ³ /s) |
| 28/01/2017 | 0.25 | 0.158 | 0.0395 | 565 | 0.5325 | 0.0210 |
| | 0.25 | 0.160 | 0.04 | 580 | 0.5453 | 0.0218 |
| | 0.25 | 0.165 | 0.04125 | 600 | 0.5624 | 0.0232 |
| | 0.25 | 0.16 | 0.04 | 582 | 0.5470 | 0.0219 |
| | 0.25 | 0.15 | 0.0375 | 413 | 0.4027 | 0.0151 |
| | 0.25 | 0.135 | 0.03375 | 376 | 0.3711 | 0.0125 |
| Total width | 1.50 | | | | Q _{Total} | 0.1156 |
| 02/02/2017 | 0.25 | 0.16 | 0.04 | 570 | 0.5368 | 0.0215 |
| | 0.25 | 0.17 | 0.0425 | 600 | 0.5624 | 0.0239 |
| | 0.25 | 0.165 | 0.04125 | 550 | 0.5197 | 0.0214 |
| | 0.25 | 0.16 | 0.04 | 525 | 0.4984 | 0.0199 |
| | 0.25 | 0.15 | 0.0375 | 414 | 0.4036 | 0.0151 |
| | 0.25 | 0.135 | 0.03375 | 380 | 0.3745 | 0.0126 |
| Total width | 1.50 | | | | Q _{Total} | 0.1145 |
| 08/02/2017 | 0.25 | 0.15 | 0.0375 | 565 | 0.5325 | 0.0200 |
| | 0.25 | 0.155 | 0.03875 | 580 | 0.5453 | 0.0211 |
| | 0.25 | 0.165 | 0.04125 | 589 | 0.5530 | 0.0228 |
| | 0.25 | 0.16 | 0.04 | 580 | 0.5453 | 0.0218 |
| | 0.25 | 0.145 | 0.03625 | 460 | 0.4428 | 0.0161 |
| | 0.25 | 0.135 | 0.03375 | 376 | 0.3711 | 0.0125 |
| Total width | 1.50 | | | | Q _{Total} | 0.1143 |
| 22/02/2017 | 0.25 | 0.15 | 0.0375 | 500 | 0.4770 | 0.0179 |
| | 0.25 | 0.165 | 0.04125 | 566 | 0.5334 | 0.0220 |
| | 0.25 | 0.165 | 0.04125 | 567 | 0.5342 | 0.0220 |
| | 0.25 | 0.16 | 0.04 | 578 | 0.5436 | 0.0217 |
| | 0.25 | 0.154 | 0.0385 | 500 | 0.4770 | 0.0184 |
| | 0.25 | 0.13 | 0.0325 | 376 | 0.3711 | 0.0121 |
| Total width | 1.50 | | | | Q _{Total} | 0.1141 |
| 10/03/2017 | 0.25 | 0.162 | 0.0405 | 565 | 0.5325 | 0.0216 |
| | 0.25 | 0.154 | 0.0385 | 580 | 0.5453 | 0.0210 |
| | 0.25 | 0.158 | 0.0395 | 600 | 0.5624 | 0.0222 |
| | 0.25 | 0.16 | 0.04 | 582 | 0.5470 | 0.0219 |
| | 0.25 | 0.148 | 0.037 | 413 | 0.4027 | 0.0149 |
| | 0.25 | 0.138 | 0.0345 | 376 | 0.3711 | 0.0128 |
| Total width | 1.50 | | | | Q _{Total} | 0.1144 |
| 24/03/2017 | 0.25 | 0.16 | 0.04 | 584 | 0.5487 | 0.0219 |
| | 0.25 | 0.16 | 0.04 | 598 | 0.5607 | 0.0224 |
| | 0.25 | 0.16 | 0.04 | 565 | 0.5325 | 0.0213 |
| | 0.25 | 0.16 | 0.04 | 526 | 0.4992 | 0.0200 |
| | 0.25 | 0.154 | 0.0385 | 418 | 0.4070 | 0.0157 |
| | 0.25 | 0.138 | 0.0345 | 382 | 0.3762 | 0.0130 |
| Total width | 1.50 | | | | Q _{Total} | 0.1143 |
| 09/04/2017 | 0.25 | 0.158 | 0.0395 | 565 | 0.5325 | 0.0210 |
| | 0.25 | 0.157 | 0.03925 | 580 | 0.5453 | 0.0214 |
| | 0.25 | 0.16 | 0.04 | 600 | 0.5624 | 0.0225 |
| | 0.25 | 0.16 | 0.04 | 582 | 0.5470 | 0.0219 |
| | 0.25 | 0.145 | 0.03625 | 413 | 0.4027 | 0.0146 |
| | 0.25 | 0.135 | 0.03375 | 376 | 0.3711 | 0.0125 |
| Total width | 1.50 | | | | Q _{Total} | 0.1139 |
| 23/04/2017 | 0.25 | 0.157 | 0.03925 | 584 | 0.5487 | 0.0215 |
| | 0.25 | 0.16 | 0.04 | 598 | 0.5607 | 0.0224 |
| | 0.25 | 0.165 | 0.04125 | 565 | 0.5325 | 0.0220 |
| | 0.25 | 0.16 | 0.04 | 526 | 0.4992 | 0.0200 |
| | 0.25 | 0.15 | 0.0375 | 418 | 0.4070 | 0.0153 |
| | 0.25 | 0.135 | 0.03375 | 382 | 0.3762 | 0.0127 |
| Total width | 1.50 | | | | Q _{Total} | 0.1139 |

| | | | | | | |
|--|------|-------|---------|-----|--------------------|--------|
| | | | | | | |
| 09/05/2017 | 0.25 | 0.15 | 0.0375 | 570 | 0.5368 | 0.0201 |
| | 0.25 | 0.16 | 0.04 | 590 | 0.5539 | 0.0222 |
| | 0.25 | 0.165 | 0.04125 | 600 | 0.5624 | 0.0232 |
| | 0.25 | 0.145 | 0.03625 | 525 | 0.4984 | 0.0181 |
| | 0.25 | 0.15 | 0.0375 | 450 | 0.4343 | 0.0163 |
| | 0.25 | 0.13 | 0.0325 | 380 | 0.3745 | 0.0122 |
| Total width | 1.50 | | | | Q _{Total} | 0.1120 |
| 23/05/2017 | 0.25 | 0.16 | 0.04 | 570 | 0.5368 | 0.0215 |
| | 0.25 | 0.16 | 0.04 | 600 | 0.5624 | 0.0225 |
| | 0.25 | 0.16 | 0.04 | 550 | 0.5197 | 0.0208 |
| | 0.25 | 0.16 | 0.04 | 525 | 0.4984 | 0.0199 |
| | 0.25 | 0.154 | 0.0385 | 414 | 0.4036 | 0.0155 |
| | 0.25 | 0.138 | 0.0345 | 380 | 0.3745 | 0.0129 |
| Total width | 1.50 | | | | Q _{Total} | 0.1131 |
| 08/06/2017 | 0.25 | 0.157 | 0.03925 | 584 | 0.5487 | 0.0215 |
| | 0.25 | 0.16 | 0.04 | 598 | 0.5607 | 0.0224 |
| | 0.25 | 0.165 | 0.04125 | 565 | 0.5325 | 0.0220 |
| | 0.25 | 0.16 | 0.04 | 526 | 0.4992 | 0.0200 |
| | 0.25 | 0.15 | 0.0375 | 418 | 0.4070 | 0.0153 |
| | 0.25 | 0.135 | 0.03375 | 382 | 0.3762 | 0.0127 |
| Total width | 1.50 | | | | Q _{Total} | 0.1139 |
| 22/06/2017 | 0.25 | 0.15 | 0.0375 | 498 | 0.4753 | 0.0178 |
| | 0.25 | 0.165 | 0.04125 | 566 | 0.5334 | 0.0220 |
| | 0.25 | 0.165 | 0.04125 | 567 | 0.5342 | 0.0220 |
| | 0.25 | 0.16 | 0.04 | 560 | 0.5282 | 0.0211 |
| | 0.25 | 0.154 | 0.0385 | 450 | 0.4343 | 0.0167 |
| | 0.25 | 0.13 | 0.0325 | 376 | 0.3711 | 0.0121 |
| Total width | 1.50 | | | | Q _{Total} | 0.1118 |
| 08/07/2017 | 0.25 | 0.14 | 0.035 | 560 | 0.5282 | 0.0185 |
| | 0.25 | 0.155 | 0.03875 | 570 | 0.5368 | 0.0208 |
| | 0.25 | 0.155 | 0.03875 | 610 | 0.5709 | 0.0221 |
| | 0.25 | 0.160 | 0.04 | 572 | 0.5385 | 0.0215 |
| | 0.25 | 0.158 | 0.0395 | 415 | 0.4044 | 0.0160 |
| | 0.25 | 0.13 | 0.0325 | 370 | 0.3660 | 0.0119 |
| Total width | 1.50 | | | | Q _{Total} | 0.1108 |
| 06/08/2017 | 0.25 | 0.146 | 0.0365 | 560 | 0.5282 | 0.0193 |
| | 0.25 | 0.15 | 0.0375 | 570 | 0.5368 | 0.0201 |
| | 0.25 | 0.155 | 0.03875 | 610 | 0.5709 | 0.0221 |
| | 0.25 | 0.156 | 0.039 | 572 | 0.5385 | 0.0210 |
| | 0.25 | 0.155 | 0.03875 | 415 | 0.4044 | 0.0157 |
| | 0.25 | 0.137 | 0.03425 | 370 | 0.3660 | 0.0125 |
| Total width | 1.50 | | | | Q _{Total} | 0.1107 |
| Average discharge of the seepage(Q) in m ³ /sec | | | | | | 0.1134 |

Appendix table 5: Field data seepage discharge measurements using current meter method along D/S left abutment seepage-2

Table:Field data sheet showing discharge measurements using the current meter method

| Location | Left abutment @chainage 300m from dam axis (Seepage-2) | | | | | 0.000854 |
|--|--|--------------|-------------------------------|-------------------|--|---------------------------------------|
| Velocity (V) | 0.000854C+0.05 | | | | | 0.05 |
| Date | Width (W) of interval(m) | Depth(d)in m | Area(A)=Wx d(m ²) | Counts (C)/minute | Velocity @0.6d (V)=0.000854C +0.05 (m/s) | Discharge (Q) =A*V(m ³ /s) |
| 28/01/2017 | 0.25 | 0.085 | 0.02125 | 330 | 0.3318 | 0.0071 |
| | 0.25 | 0.09 | 0.0225 | 350 | 0.3489 | 0.0079 |
| | 0.25 | 0.075 | 0.01875 | 300 | 0.3062 | 0.0057 |
| Total width | 0.75 | | | | Q _{Total} | 0.0206 |
| 02/02/2017 | 0.25 | 0.084 | 0.021 | 340 | 0.3404 | 0.0071 |
| | 0.25 | 0.09 | 0.0225 | 370 | 0.3660 | 0.0082 |
| | 0.25 | 0.07 | 0.0175 | 330 | 0.3318 | 0.0058 |
| Total width | 0.75 | | | | Q _{Total} | 0.0212 |
| 08/02/2017 | 0.25 | 0.085 | 0.02125 | 335 | 0.3361 | 0.0071 |
| | 0.25 | 0.09 | 0.0225 | 365 | 0.3617 | 0.0081 |
| | 0.25 | 0.075 | 0.01875 | 320 | 0.3233 | 0.0061 |
| Total width | 0.75 | | | | Q _{Total} | 0.0213 |
| 22/02/2017 | 0.25 | 0.075 | 0.01875 | 330 | 0.3318 | 0.0062 |
| | 0.25 | 0.085 | 0.02125 | 375 | 0.3703 | 0.0079 |
| | 0.25 | 0.075 | 0.01875 | 325 | 0.3276 | 0.0061 |
| Total width | 0.75 | | | | Q _{Total} | 0.0202 |
| 10/03/2017 | 0.25 | 0.08 | 0.02 | 325 | 0.3276 | 0.0066 |
| | 0.25 | 0.085 | 0.02125 | 360 | 0.3574 | 0.0076 |
| | 0.25 | 0.075 | 0.01875 | 330 | 0.3318 | 0.0062 |
| Total width | 0.75 | | | | Q _{Total} | 0.0204 |
| 24/03/2017 | 0.25 | 0.085 | 0.02125 | 300 | 0.3062 | 0.0065 |
| | 0.25 | 0.09 | 0.0225 | 360 | 0.3574 | 0.0080 |
| | 0.25 | 0.075 | 0.01875 | 315 | 0.3190 | 0.0060 |
| Total width | 0.75 | | | | Q _{Total} | 0.0205 |
| 09/04/2017 | 0.25 | 0.08 | 0.02 | 315 | 0.3190 | 0.0064 |
| | 0.25 | 0.085 | 0.02125 | 365 | 0.3617 | 0.0077 |
| | 0.25 | 0.075 | 0.01875 | 310 | 0.3147 | 0.0059 |
| Total width | 0.75 | | | | Q _{Total} | 0.0200 |
| 23/04/2017 | 0.25 | 0.085 | 0.02125 | 300 | 0.3062 | 0.0065 |
| | 0.25 | 0.085 | 0.02125 | 375 | 0.3703 | 0.0079 |
| | 0.25 | 0.075 | 0.01875 | 320 | 0.3233 | 0.0061 |
| Total width | 0.75 | | | | Q _{Total} | 0.0204 |
| 09/05/2017 | 0.25 | 0.08 | 0.02 | 315 | 0.3190 | 0.0064 |
| | 0.25 | 0.085 | 0.02125 | 375 | 0.3703 | 0.0079 |
| | 0.25 | 0.07 | 0.0175 | 315 | 0.3190 | 0.0056 |
| Total width | 0.75 | | | | Q _{Total} | 0.0198 |
| 23/05/2017 | 0.25 | 0.08 | 0.02 | 300 | 0.3062 | 0.0061 |
| | 0.25 | 0.08 | 0.02 | 365 | 0.3617 | 0.0072 |
| | 0.25 | 0.075 | 0.01875 | 300 | 0.3062 | 0.0057 |
| Total width | 0.75 | | | | Q _{Total} | 0.0191 |
| 08/06/2017 | 0.25 | 0.084 | 0.021 | 310 | 0.3147 | 0.0066 |
| | 0.25 | 0.09 | 0.0225 | 375 | 0.3703 | 0.0083 |
| | 0.25 | 0.07 | 0.0175 | 315 | 0.3190 | 0.0056 |
| Total width | 0.75 | | | | Q _{Total} | 0.0205 |
| 22/06/2017 | 0.25 | 0.07 | 0.0175 | 310 | 0.3147 | 0.0055 |
| | 0.25 | 0.09 | 0.0225 | 365 | 0.3617 | 0.0081 |
| | 0.25 | 0.065 | 0.01625 | 300 | 0.3062 | 0.0050 |
| Total width | 0.75 | 0.085 | | | Q _{Total} | 0.0186 |
| 08/07/2017 | 0.25 | 0.085 | 0.02125 | 300 | 0.3062 | 0.0065 |
| | 0.25 | 0.075 | 0.01875 | 360 | 0.3574 | 0.0067 |
| | 0.25 | 0.075 | 0.01875 | 310 | 0.3147 | 0.0059 |
| Total width | 0.75 | 0.075 | | | Q _{Total} | 0.0191 |
| 06/08/2017 | 0.25 | 0.09 | 0.0225 | 310 | 0.3147 | 0.0071 |
| | 0.25 | 0.075 | 0.01875 | 365 | 0.3617 | 0.0068 |
| Total width | 0.50 | | | | Q _{Total} | 0.0139 |
| Average discharge of the seepage(Q) in m ³ /sec | | | | | | 0.020 |

Appendix table 6: Field data seepage discharge measurements using current meter method along D/S right abutment seepage-3

Table:Field data sheet showing discharge measurements using the current meter method

| Location | Right abutment @chainage 350m from dam axis (Seepage -3) | | | | | 0.000854 |
|--|--|--------------|-------------------------------|-------------------|--|---------------------------------------|
| Velocity (V) | 0.000854C+0.05 | | | | | 0.05 |
| Date | Width (W) of interval(m) | Depth(d)in m | Area(A)=Wx d(m ²) | Counts (C)/minute | Velocity @0.6d (V)=0.000854C +0.05 (m/s) | Discharge (Q) =A*V(m ³ /s) |
| 28/01/2017 | 0.25 | 0.085 | 0.02125 | 310 | 0.3147 | 0.0067 |
| | 0.25 | 0.09 | 0.0225 | 360 | 0.3574 | 0.0080 |
| | 0.25 | 0.075 | 0.01875 | 330 | 0.3318 | 0.0062 |
| Total width | 0.75 | | | | Q _{Total} | 0.0210 |
| 02/02/2017 | 0.25 | 0.075 | 0.01875 | 335 | 0.3361 | 0.0063 |
| | 0.25 | 0.085 | 0.02125 | 365 | 0.3617 | 0.0077 |
| | 0.25 | 0.065 | 0.01625 | 300 | 0.3062 | 0.0050 |
| Total width | 0.75 | | | | Q _{Total} | 0.0190 |
| 08/02/2017 | 0.25 | 0.08 | 0.02 | 320 | 0.3233 | 0.0065 |
| | 0.25 | 0.085 | 0.02125 | 375 | 0.3703 | 0.0079 |
| | 0.25 | 0.07 | 0.0175 | 340 | 0.3404 | 0.0060 |
| Total width | 0.75 | | | | Q _{Total} | 0.0203 |
| 22/02/2017 | 0.25 | 0.075 | 0.01875 | 335 | 0.3361 | 0.0063 |
| | 0.25 | 0.09 | 0.0225 | 365 | 0.3617 | 0.0081 |
| | 0.25 | 0.075 | 0.01875 | 385 | 0.3788 | 0.0071 |
| Total width | 0.75 | | | | Q _{Total} | 0.0215 |
| 10/03/2017 | 0.25 | 0.08 | 0.02 | 330 | 0.3318 | 0.0066 |
| | 0.25 | 0.085 | 0.02125 | 375 | 0.3703 | 0.0079 |
| | 0.25 | 0.07 | 0.0175 | 380 | 0.3745 | 0.0066 |
| Total width | 0.75 | | | | Q _{Total} | 0.0211 |
| 24/03/2017 | 0.25 | 0.08 | 0.02 | 335 | 0.3361 | 0.0067 |
| | 0.25 | 0.08 | 0.02 | 365 | 0.3617 | 0.0072 |
| | 0.25 | 0.075 | 0.01875 | 380 | 0.3745 | 0.0070 |
| Total width | 0.75 | | | | Q _{Total} | 0.0210 |
| 09/04/2017 | 0.25 | 0.08 | 0.02 | 335 | 0.3361 | 0.0067 |
| | 0.25 | 0.085 | 0.02125 | 365 | 0.3617 | 0.0077 |
| | 0.25 | 0.07 | 0.0175 | 360 | 0.3574 | 0.0063 |
| Total width | 0.75 | | | | Q _{Total} | 0.0207 |
| 23/04/2017 | 0.25 | 0.085 | 0.02125 | 325 | 0.3276 | 0.0070 |
| | 0.25 | 0.085 | 0.02125 | 370 | 0.3660 | 0.0078 |
| | 0.25 | 0.075 | 0.01875 | 365 | 0.3617 | 0.0068 |
| Total width | 0.75 | | | | Q _{Total} | 0.0215 |
| 09/05/2017 | 0.25 | 0.08 | 0.02 | 325 | 0.3276 | 0.0066 |
| | 0.25 | 0.085 | 0.02125 | 370 | 0.3660 | 0.0078 |
| | 0.25 | 0.075 | 0.01875 | 365 | 0.3617 | 0.0068 |
| Total width | 0.75 | | | | Q _{Total} | 0.0211 |
| 23/05/2017 | 0.25 | 0.085 | 0.02125 | 330 | 0.3318 | 0.0071 |
| | 0.25 | 0.09 | 0.0225 | 380 | 0.3745 | 0.0084 |
| | 0.25 | 0.075 | 0.01875 | 360 | 0.3574 | 0.0067 |
| Total width | 0.75 | | | | Q _{Total} | 0.0222 |
| 08/06/2017 | 0.25 | 0.084 | 0.021 | 335 | 0.3361 | 0.0071 |
| | 0.25 | 0.09 | 0.0225 | 360 | 0.3574 | 0.0080 |
| | 0.25 | 0.07 | 0.0175 | 350 | 0.3489 | 0.0061 |
| Total width | 0.75 | | | | Q _{Total} | 0.0212 |
| 22/06/2017 | 0.25 | 0.085 | 0.02125 | 320 | 0.3233 | 0.0069 |
| | 0.25 | 0.09 | 0.0225 | 375 | 0.3703 | 0.0083 |
| | 0.25 | 0.075 | 0.01875 | 365 | 0.3617 | 0.0068 |
| Total width | 0.75 | | | | Q _{Total} | 0.0220 |
| 08/07/2017 | 0.25 | 0.075 | 0.01875 | 325 | 0.3276 | 0.0061 |
| | 0.25 | 0.085 | 0.02125 | 375 | 0.3703 | 0.0079 |
| | 0.25 | 0.075 | 0.01875 | 360 | 0.3574 | 0.0067 |
| Total width | 0.75 | | | | Q _{Total} | 0.0207 |
| 06/08/2017 | 0.25 | 0.08 | 0.02 | 300 | 0.3062 | 0.0061 |
| | 0.25 | 0.085 | 0.02125 | 355 | 0.3532 | 0.0075 |
| | 0.25 | 0.075 | 0.01875 | 360 | 0.3574 | 0.0067 |
| Total width | 0.75 | | | | Q _{Total} | 0.0203 |
| Average discharge of the seepage(Q) in m ³ /sec | | | | | | 0.021 |

Appendix table 7: Field data seepage discharge measurements using current meter method along D/S horizontal filter seepage-4

Table:Field data sheet showing discharge measurements using the current meter method

| D/s river seepage along Rock toe @chainage 30m from D/S end(Seepage -4) | | | | | | |
|---|--------------------------|---------------|-------------------------------|-------------------|--|---------------------------------------|
| Location | Width (W) of interval(m) | Depth(d)in m | Area(A)=Wx d(m ²) | Counts (C)/minute | Velocity @0.6d (V)=0.000854C +0.05 (m/s) | Discharge (Q) =A*V(m ³ /s) |
| Date | Width (W) of | Depth(d) in m | Area(A)=Wxd(| Counts | Velocity @0.6d | Discharge (Q) |
| 28/01/2017 | 0.25 | 0.090 | 0.0225 | 335 | 0.3361 | 0.0076 |
| | 0.50 | 0.107 | 0.0535 | 375 | 0.3703 | 0.0198 |
| | 0.25 | 0.107 | 0.02675 | 360 | 0.3574 | 0.0096 |
| | 0.25 | 0.100 | 0.025 | 346 | 0.3455 | 0.0086 |
| | 0.25 | 0.090 | 0.0225 | 338 | 0.3387 | 0.0076 |
| Total width | 1.50 | | | | Q _{total} | 0.0532 |
| 02/02/2017 | 0.25 | 0.092 | 0.023 | 330 | 0.3318 | 0.0076 |
| | 0.50 | 0.107 | 0.0535 | 360 | 0.3574 | 0.0191 |
| | 0.25 | 0.107 | 0.02675 | 366 | 0.3626 | 0.0097 |
| | 0.25 | 0.100 | 0.025 | 338 | 0.3387 | 0.0085 |
| | 0.25 | 0.091 | 0.022625 | 330 | 0.3318 | 0.0075 |
| Total width | 1.50 | | | | Q _{total} | 0.0524 |
| 08/02/2017 | 0.25 | 0.095 | 0.02375 | 330 | 0.3318 | 0.0079 |
| | 0.50 | 0.108 | 0.054 | 345 | 0.3446 | 0.0186 |
| | 0.25 | 0.108 | 0.027 | 352 | 0.3506 | 0.0095 |
| | 0.25 | 0.105 | 0.02625 | 345 | 0.3446 | 0.0090 |
| | 0.25 | 0.090 | 0.0225 | 330 | 0.3318 | 0.0075 |
| Total width | 1.50 | | | | Q _{total} | 0.0525 |
| 22/02/2017 | 0.25 | 0.100 | 0.025 | 330 | 0.3318 | 0.0083 |
| | 0.50 | 0.107 | 0.0535 | 350 | 0.3489 | 0.0187 |
| | 0.25 | 0.107 | 0.02675 | 345 | 0.3446 | 0.0092 |
| | 0.25 | 0.095 | 0.02375 | 344 | 0.3438 | 0.0082 |
| | 0.25 | 0.090 | 0.0225 | 335 | 0.3361 | 0.0076 |
| Total width | 1.50 | | | | Q _{total} | 0.0519 |
| 10/03/2017 | 0.25 | 0.100 | 0.025 | 330 | 0.3318 | 0.0083 |
| | 0.50 | 0.100 | 0.05 | 368 | 0.3643 | 0.0182 |
| | 0.25 | 0.110 | 0.0275 | 345 | 0.3446 | 0.0095 |
| | 0.25 | 0.100 | 0.025 | 344 | 0.3438 | 0.0086 |
| | 0.25 | 0.090 | 0.0225 | 335 | 0.3361 | 0.0076 |
| Total width | 1.50 | | | | Q _{total} | 0.0521 |
| 24/03/2017 | 0.25 | 0.100 | 0.025 | 330 | 0.3318 | 0.0083 |
| | 0.50 | 0.110 | 0.055 | 325 | 0.3276 | 0.0180 |
| | 0.25 | 0.110 | 0.0275 | 345 | 0.3446 | 0.0095 |
| | 0.25 | 0.100 | 0.025 | 344 | 0.3438 | 0.0086 |
| | 0.25 | 0.090 | 0.0225 | 335 | 0.3361 | 0.0076 |
| Total width | 1.50 | | | | Q _{total} | 0.0519 |
| 09/04/2017 | 0.25 | 0.090 | 0.0225 | 335 | 0.3361 | 0.0076 |
| | 0.50 | 0.107 | 0.0535 | 375 | 0.3703 | 0.0198 |
| | 0.25 | 0.107 | 0.02675 | 360 | 0.3574 | 0.0096 |
| | 0.25 | 0.100 | 0.025 | 346 | 0.3455 | 0.0086 |
| | 0.25 | 0.090 | 0.0225 | 338 | 0.3387 | 0.0076 |
| Total width | 1.50 | | | | Q _{total} | 0.0532 |
| 23/04/2017 | 0.25 | 0.100 | 0.025 | 330 | 0.3318 | 0.0083 |
| | 0.50 | 0.110 | 0.055 | 325 | 0.3276 | 0.0180 |
| | 0.25 | 0.110 | 0.0275 | 345 | 0.3446 | 0.0095 |
| | 0.25 | 0.100 | 0.025 | 344 | 0.3438 | 0.0086 |
| | 0.25 | 0.090 | 0.0225 | 335 | 0.3361 | 0.0076 |
| Total width | 1.50 | | | | Q _{total} | 0.0519 |

| | | | | | | |
|--|------|-------|----------|-----|--------------------|--------|
| 09/05/2017 | 0.25 | 0.092 | 0.023 | 330 | 0.3318 | 0.0076 |
| | 0.50 | 0.107 | 0.0535 | 360 | 0.3574 | 0.0191 |
| | 0.25 | 0.107 | 0.02675 | 366 | 0.3626 | 0.0097 |
| | 0.25 | 0.100 | 0.025 | 338 | 0.3387 | 0.0085 |
| | 0.25 | 0.091 | 0.022625 | 330 | 0.3318 | 0.0075 |
| Total width | 1.50 | | | | Q _{total} | 0.0524 |
| 23/05/2017 | 0.25 | 0.090 | 0.0225 | 320 | 0.3233 | 0.0073 |
| | 0.50 | 0.100 | 0.05 | 360 | 0.3574 | 0.0179 |
| | 0.25 | 0.100 | 0.025 | 350 | 0.3489 | 0.0087 |
| | 0.25 | 0.100 | 0.025 | 330 | 0.3318 | 0.0083 |
| | 0.25 | 0.090 | 0.0225 | 325 | 0.3276 | 0.0074 |
| Total width | 1.50 | | | | Q _{total} | 0.0495 |
| 08/06/2017 | 0.25 | 0.100 | 0.025 | 330 | 0.3318 | 0.0083 |
| | 0.50 | 0.107 | 0.0535 | 360 | 0.3574 | 0.0191 |
| | 0.25 | 0.107 | 0.02675 | 366 | 0.3626 | 0.0097 |
| | 0.25 | 0.095 | 0.02375 | 338 | 0.3387 | 0.0080 |
| | 0.25 | 0.090 | 0.0225 | 330 | 0.3318 | 0.0075 |
| Total width | 1.50 | | | | Q _{total} | 0.0526 |
| 22/06/2017 | 0.25 | 0.092 | 0.023 | 330 | 0.3318 | 0.0076 |
| | 0.50 | 0.100 | 0.05 | 368 | 0.3643 | 0.0182 |
| | 0.25 | 0.107 | 0.02675 | 366 | 0.3626 | 0.0097 |
| | 0.25 | 0.100 | 0.025 | 338 | 0.3387 | 0.0085 |
| | 0.25 | 0.091 | 0.022625 | 330 | 0.3318 | 0.0075 |
| Total width | 1.50 | | | | Q _{total} | 0.0515 |
| 08/07/2017 | 0.25 | 0.085 | 0.02125 | 325 | 0.3276 | 0.0070 |
| | 0.50 | 0.100 | 0.05 | 365 | 0.3617 | 0.0181 |
| | 0.25 | 0.100 | 0.025 | 356 | 0.3540 | 0.0089 |
| | 0.25 | 0.090 | 0.0225 | 337 | 0.3378 | 0.0076 |
| | 0.25 | 0.085 | 0.02125 | 328 | 0.3301 | 0.0070 |
| Total width | 1.50 | | | | Q _{total} | 0.0485 |
| 06/08/2017 | 0.25 | 0.085 | 0.02125 | 300 | 0.3062 | 0.0065 |
| | 0.50 | 0.100 | 0.05 | 320 | 0.3233 | 0.0162 |
| | 0.25 | 0.100 | 0.025 | 320 | 0.3233 | 0.0081 |
| | 0.25 | 0.090 | 0.0225 | 310 | 0.3147 | 0.0071 |
| | 0.25 | 0.085 | 0.02125 | 300 | 0.3062 | 0.0065 |
| Total width | 1.50 | | | | Q _{total} | 0.0443 |
| Average discharge of the seepage(Q) in m ³ /sec | | | | | | 0.0513 |

Appendix table 8: The trench excavated reduced level (RL) and fill volume of the dam (TWWDS, 2014)

Project Name: Gereb Segen Dam
Trench Excavated and Fill volume

| Chainage(m) | Original ground level (OGL) in m | Trench Reduced Level(TRD) in m | Cross-Sectional area(m ²) | Volume(m ³) | Chainage(m) | Original ground level (OGL) in m | Trench Reduced Level(TRD) in m | Cross-Sectional area(m ²) | Volume(m ³) |
|-------------|----------------------------------|--------------------------------|---------------------------------------|-------------------------|-------------|----------------------------------|--------------------------------|---------------------------------------|-------------------------|
| 0 | 1864.00 | 1855.80 | 66.47 | 0 | 410 | 1821.33 | 1810.10 | 171.09 | 1569.05 |
| 10 | 1860.68 | 1852.84 | 62.10 | 642.87 | 420 | 1822.03 | 1810.15 | 188.52 | 1798.01 |
| 20 | 1858.00 | 1838.65 | 264.61 | 1633.58 | 430 | 1822.52 | 1810.97 | 179.52 | 1840.18 |
| 30 | 1855.10 | 1836.43 | 249.03 | 2568.22 | 440 | 1822.75 | 1810.98 | 185.61 | 1825.67 |
| 40 | 1854.05 | 1832.70 | 313.39 | 2812.10 | 450 | 1822.65 | 1811.35 | 173.00 | 1793.05 |
| 50 | 1846.54 | 1832.77 | 149.98 | 2316.81 | 460 | 1823.65 | 1811.65 | 192.00 | 1824.98 |
| 60 | 1843.50 | 1826.78 | 206.66 | 1783.17 | 470 | 1824.67 | 1812.42 | 199.03 | 1955.17 |
| 70 | 1842.56 | 1819.48 | 358.72 | 2826.88 | 480 | 1824.88 | 1812.85 | 192.87 | 1959.51 |
| 80 | 1841.20 | 1815.18 | 442.72 | 4007.19 | 490 | 1823.82 | 1812.97 | 161.15 | 1770.09 |
| 90 | 1836.03 | 1815.26 | 298.55 | 3706.37 | 500 | 1824.38 | 1813.39 | 164.79 | 1629.70 |
| 100 | 1834.75 | 1814.80 | 278.83 | 2886.94 | 510 | 1824.59 | 1813.40 | 170.08 | 1674.37 |
| 110 | 1833.33 | 1814.64 | 423.87 | 3513.52 | 520 | 1824.61 | 1813.46 | 169.03 | 1695.55 |
| 120 | 1831.92 | 1814.25 | 383.14 | 4035.07 | 530 | 1825.06 | 1813.87 | 170.03 | 1695.28 |
| 130 | 1830.90 | 1813.73 | 363.30 | 3732.21 | 540 | 1825.82 | 1814.09 | 184.38 | 1772.02 |
| 140 | 1829.90 | 1813.36 | 339.81 | 3515.51 | 550 | 1825.82 | 1814.25 | 180.01 | 1821.92 |
| 150 | 1828.75 | 1812.80 | 318.02 | 3289.14 | 560 | 1826.42 | 1814.66 | 185.20 | 1826.05 |
| 160 | 1828.25 | 1812.65 | 306.08 | 3120.50 | 570 | 1826.76 | 1814.56 | 123.17 | 1541.86 |
| 170 | 1827.70 | 1812.37 | 296.50 | 3012.90 | 580 | 1828.22 | 1814.12 | 155.91 | 1395.43 |
| 180 | 1827.33 | 1811.95 | 297.89 | 2971.96 | 590 | 1828.15 | 1814.26 | 152.06 | 1539.88 |
| 190 | 1826.85 | 1811.89 | 283.54 | 2907.15 | 600 | 1828.25 | 1814.54 | 148.77 | 1504.15 |
| 200 | 1826.71 | 1811.92 | 278.14 | 2808.40 | 610 | 1828.55 | 1814.54 | 154.13 | 1514.47 |
| 210 | 1826.85 | 1812.01 | 279.72 | 2789.30 | 620 | 1829.41 | 1815.04 | 160.73 | 1574.27 |
| 220 | 1826.71 | 1812.21 | 268.38 | 2740.51 | 630 | 1831.20 | 1817.08 | 156.17 | 1584.48 |
| 230 | 1826.46 | 1812.21 | 260.00 | 2641.90 | 640 | 1832.29 | 1819.41 | 134.45 | 1453.09 |
| 240 | 1826.46 | 1812.20 | 260.32 | 2601.60 | 650 | 1833.05 | 1820.44 | 129.93 | 1321.90 |
| 250 | 1826.41 | 1812.02 | 264.60 | 2624.61 | 660 | 1835.62 | 1820.88 | 167.58 | 1487.52 |
| 260 | 1826.45 | 1811.71 | 276.29 | 2704.47 | 670 | 1837.01 | 1821.75 | 177.38 | 1724.76 |
| 270 | 1826.46 | 1811.50 | 283.51 | 2799.00 | 680 | 1839.89 | 1825.60 | 159.35 | 1683.66 |
| 280 | 1826.45 | 1811.70 | 276.63 | 2800.68 | 690 | 1842.80 | 1827.97 | 169.25 | 1643.00 |
| 290 | 1826.39 | 1811.61 | 277.40 | 2770.15 | 700 | 1845.05 | 1830.70 | 160.36 | 1648.04 |
| 300 | 1826.36 | 1811.66 | 274.79 | 2760.95 | 710 | 1848.57 | 1832.51 | 193.14 | 1767.51 |
| 310 | 1826.42 | 1811.59 | 279.28 | 2770.36 | 720 | 1849.90 | 1832.64 | 217.89 | 2055.15 |
| 320 | 1826.60 | 1810.75 | 188.93 | 2341.07 | 730 | 1849.71 | 1834.65 | 173.58 | 1957.36 |
| 330 | 1826.59 | 1810.45 | 194.75 | 1918.41 | 740 | 1849.54 | 1848.65 | 3.95 | 887.65 |
| 340 | 1827.13 | 1810.32 | 208.42 | 2015.87 | 750 | 1850.14 | 1850.65 | -1.88 | 10.32 |
| 350 | 1827.21 | 1810.20 | 212.79 | 2106.09 | 760 | 1851.57 | 1850.85 | 3.13 | 6.24 |
| 360 | 1819.53 | 1810.21 | 80.71 | 1467.53 | 770 | 1854.05 | 1851.63 | 12.60 | 78.63 |
| 370 | 1819.52 | 1810.25 | 80.07 | 803.92 | 780 | 1855.10 | 1854.45 | 2.80 | 76.99 |
| 380 | 1819.56 | 1810.68 | 114.44 | 972.56 | 790 | 1854.79 | 1854.79 | 0.00 | 14.01 |
| 390 | 1820.01 | 1810.96 | 118.08 | 1162.60 | 800 | 1856.68 | 1856.68 | 0.00 | 0.00 |
| 400 | 1820.72 | 1810.61 | 142.72 | 1304.03 | 815 | 1863.80 | 1863.80 | 0.00 | 0.00 |