ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
School of Civil and Environmental Engineering

Hydraulic Analysis of Rock Fill Dam under Karst Environment (Weyib River)

A thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science In Civil Engineering (Major in Hydraulic Engineering)

By:
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Addis Ababa, Ethiopia
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Approval by Board of Examiners

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Chairman (department of graduate committee) Signature Date

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Advisor

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Internal Examiner

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External Examiner

GRATITUDE

I first of all would like to send my gratitude's to **My God** for the ways that he showed me in his **silent and firm support and guidance** in my journey of thesis paper finalization. His wisdom and guidance has given me the opportunity to learn the skills necessary to become a successful engineer.

I would also like to send my appreciation to my adviser Dr**. Daneal F/Sillassie for** the time and guidance he has given me throughout my research here at the University of Addis Ababa.

I would also like to show my appreciation to **Professor Asfawosen Asrat, ERA, OWWDSE**, **My family and friends** specially **Endalkachew Girma, Habesh Mohammed, Belay Gedamu, Yidnekachew Abebe, Amha Fekadu, Tomas Guta, Zerihun Estifanos, and Abush Molla** that have all been **valuable assets** to the completion of my research.

Finally, I would like to send great appreciation to **AAiT** for their **prestigious assistance** with **extraordinary testing** and **causive and impressive guiding** played a major role in the successful completion of **post graduate career.**

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DEDICATION

To My Mother W/o Abaynesh Tilahun

And

Her crown child Mihret Fekadu.

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Acronym

USBR: United States Bureau of Reclamation

USSD: United States Society of Dams

BGS: British Geological Society

UNESCO: United Nations Education and Science Organization

GSNA: Geological Society of North America

K-I: Karst Geological Class-One

IACDS: Inter America Committee for Dam Safety

HMS: Hydraulic module software

HEC: Hydraulic Engineering Center

USGS: United States Geological Society

FERC: Federal Energy Regulatory Commission

Abstract

In this paper, the result on research of hydraulic analysis of rock fill dam under Karst foundation is presented. Karst foundation is known for its rock fill dam's water tightness problems resulted from soluble nature of lime stone and progressive leakage. The hydraulic flow feature in this foundation has heterogeneity characterized by temporal and spatial variation. Hydraulic problems related to Karst foundation are possible causes of dams' failures.

It is well known that the key factors in the analysis of rock fill dams are direct methods of foundation investigations. But, under Karst environment, the data obtained through the process has point data nature and the foundation has heterogeneity. Therefore, such methods will result better if supported by alternative ways.

One of the alternative ways of estimating the hydraulic and material properties of the dam and its foundation was estimating the average saturated hydraulic conductivities of the aquifer/ foundation and relating obtained result with Karst hydraulic and material parameteric values of similar Karst features of other places. To this regard the results obtained through published other researches were used for estimation, comparison and verification proposes.

The location of the research area is between two river gauge stations that represent Karst aquifer/foundation. The river is Weyib which is found in south east part of Ethiopia. The river gauge stations were Alemkerem and Sofumar found at close distance and the area in between them was selected by taking the advantages of their spatial and temporal flow difference and its prominent Karst formation.

In estimating of average saturated hydraulic conductivities of the aquifer/ foundation, average base flow estimation was conducted by three methods. The methods were USGS programme software, Excel hydrograph separation and Chpmans' algorithm using excel. Stream flow measured at Sofumar and Alemkerem were used for base flow estimations.

As per the result obtained through different methods, USGS result was selected after comparisons of the methods. During comparision, USGS software programme was selected for it has the smallest values of standard deviation of base flow to the average value.

Average base flow result obtained were ***8.67cumec*** and ***18.49cumec*** for Alemkerem and Sofumar respectively. Average base flow difference obtained was ***9.82cumec*** which was used for estimation of average saturated hydraulic conductivies of the aquifer and foundation. Again, based on the two contributing sides area sizes of the river, base flow results obtained were ***1.89cumec*** and ***7.91cumec*** for the left and right side respectively.

During estimation of average saturated hydraulic conductivities, unconfined well hydraulics formula was used. Based on the result, estimated values were ***6.90E-04 m/sec*** and ***1.5E-04m/sec*** for the left and right side of the river. Once the average values and the average saturation level/river bed were obtained, the respective values of different levels of the aquifer/ foundation were calculated with respect to it. In doing so, empirical formula was used and the results of the two methods were compared with scientific out puts of local and international researches and found sufficient and comparable. The ranges of values obtained by British geological society were ***1.14E-03 m/sec*** and ***1.14E-07m/sec*** and the ranges of values in this research were found almost similar.

During the analysis of the dam, the aquifer was considered as foundation and the average values were required to choose dam type having similar properties with the foundation especially under saturated condition. According to different references, rock fill dam materials under compaction could have hydraulic conductivities values of ***1E-05 m/sec*** to ***1E-07m/sec.***

Hence, in this research, homogeneous rocks fill dam, averagely addressing the hydro-geological variation was hydraulically analyzed. For the sake of convenience, the location in between the two gauge stations with its geo reference data was taken into account and the cross section of dam and the foundation was divided in two different layers with a total of ***250meters*** above and below the river bed level. The base of karstification level was the water table/river bed level and a total of ***500m*** depth was considered. The layering or zoning were for every ***10m*** to ***20m*** depth and karstification level findings with their respective values found similar with other extensively conducted researches results.

The dam under consideration was with and without upstream water proof material. The “without” condition was for estimating leakage magnitude through foundation and dam body where as the “with” condition is required for estimating the water proof material properties parametric values. ***Geo-Studio, 2012 software*** was used in calculating seepage magnitudes.

The results obtained by software were compared with quantitative analysis done by means of excel spreadsheet.

Safe slope values obtained considering equivalent dynamic loading conditions at downstream face and the dam and its foundation as equivalent porous media were ***1.87*** and ***2.55*** for the **"with'** and the **"without"** condition. Correspondingly, seepage values obtained were ***1.04E-04 cum/sec*** and ***2.87E-07 cum/sec*** respectively. ***As per the analysis, rock fill dam was found impossible without abutment and dam water proof treatment works***. Dam, foundation and abutment treatment for water proof works resulted as 10^3 to 10^5 times the foundation and dam materials saturated hydraulic conductivities parameteric values and these values were similar with other researches results.

By using spreed sheet quantitative analysis, obtained values for normal effective stress were **49.99 Kpa and 99.79KPa** for ***10mts and 20mts heights*** respectively. Correspondingly, these values were comparable with Geo-Studio software outputs which were ***50kKPa*** and ***100KPa***.

Observed and measured seepage data were taken from dams under operation found in different parts of the world. These data were systematically referred and arranged to predict the maximum safe dam heights with manageable seepage magnitude. ***No rock fill dam without foundation, abutment and dam water proof treatment work, dam height up to 70mts with all parts water proof treatment and above 70mts higher seepage related risks that require proper management were concluding remarks of this hydraulic analysis of rock fill dam under Karst environment.***

The research had resulted in a better understanding of the threat posed by Karstic physical nature considering dams. This had led to analyze the hydraulic properties, material strength and safe dam height and type under Karst environment. Hence, it can be taken as a base for hydraulic analysis of rock fill dam in other similar areas and for those who have keen interest in further research.

Chapter One -Introduction

1.1 Research Overview

In this paper, the analysis of rock-fill dam on Karst foundation is discussed focusing on the hydraulic stability of the structure under a set of conditions expected normally to occur. Failures associated with hydraulic flow condition as largely reported in many references and hydraulic flow conditions that must be considered under complex hydro-geologic settings are discussed well.

Chapter-One: embankment and rock fill dam types, dam material composition and hydraulic bevaiour of embankment dam especially rock fill materials under saturation were discussed well. The definition of Karst and its environment and hydraulic problems of dams under Karst environment were also explained. Problem statement of the research area was also explained including the hydro-geological features of the research area. In this chapter one of the major issues that were discussed was also how the research problem was addressed. The general and specific objectives, the scope and significance of the research output were put in plain words in this chapter.

Chapter-Two: reviewed literatures were discussed as per the requirement. Karst definition, Karst types hydraulic problems of dams under Karst environment, seepage controlling mechanisms so far developed, some measured and observed magnitudes of seepage and case studies were discussed very well.

Chapter-Three: What data, materials, assumptions, approach were used and followed was discussed in this chapter. How base flow was estimated by different methods, estimation of hydraulic and material parameteric values, Anlysis of seepage and related risks and comparison of result obtained with similar method of estimation and similar researches out puts were discussed in detail in this chapter.

Chapter-Four: The results obtained by this research were discussed using graphs and tables. The significant levels of obtained results were also discussed with comparisons of the result obtained**.**

Chapter-Five: Some concluding remarks and recommendations of the research out puts were discussed in this chapter. The strength, limitation of the research and some research areas that need further studies were also indicated.

1.2 Back Ground

1.2.1 Embankment Dam

The analysis of modern embankment dams is far more complex today than was the case just 100 years ago. Most of the major unknowns and uncertainties involved with embankment dams 100 years ago have been removed by the evolution of engineering experience, research, knowledge, and education **(USBR 2002).**

Embankment dams are the widest spread kind of water-retaining structures. They can be defined as dams constructed of natural material obtained from borrow pits located in the vicinity of the dam site. Material obtained from excavation of foundations and appurtenant structures is also very often used.

The classification of embankment dams is determined in several ways. In accordance with the kind of material of which they are constructed, embankment dams can be divided into: **earth dams,** the larger part of whose body (over 50%) is constructed from fine-grained earth materials of clay, loam, sand, or sandy-gravel materials; **earth–rock** dams, whose basic body volume is constructed from coarse-grained gravel or rock materials, while water impermeability is provided by means of a water- proof element made of fine-grained earth material**; rock fill dams**, whose body is constructed from coarse-grained material, while water impermeability is achieved by means of an element made of artificial material. **(Ljuborim Tanchev 2014).**

According to the position of the water proof element, it is possible to distinguish dams that have a: **facing,** when the upstream slope is lined with a water proof element; **core,** in which case the water proof element is positioned inside the body of the dam. If the core has been constructed from some artificial material, some- times it is called a diaphragm wall.

1.2.2 Rock Fill Dam

Rock fill dams offer the choicest of all alternatives for the construction of high dams under complex geological settings. Rock fill dam is a type of dam widely constructed nowadays. Compacted soft and hard rocks are also used in the construction of dams.

Compacted soft rocks (fragments evolving rocks such as schist, marls and other clayey rocks) are different from rockfill dams (fragments of hard rock) because of the geological nature of the rocks **(Civ. Eng. China 2010).**

Soft Rock fill dam after compaction and hydration, the large fragments of soft rock degrade and result in a material intermediate between soil and rock, relatively impervious, but more compressible and more sensitive to wetting and drying cycles than earth rockfill **(Civ. Eng. China 2010).**

The hydro-mechanical behavior of such material helps to encounter larger strain including high seismic load. Rock fills have higher shear strength and they perform as Mohr/Coulomb material with no cohesion but an angle of internal friction more than 35^0. Effectively compacted rock fills can have density of $2000 kg/m^3$ with angle of friction up to 65^0 **(Roland Pusch 2008).**

1.2.3 Karst Environment

The term Karst represents a distinctive topography that indicates dissolution of major soluble rocks by surface water or ground water. Although; commonly associated with carbonate rocks (limestone and dolomite). Karst features vary in range from small to entire drainage systems which cover hundreds of square miles, and wide Karst plateaus.

The formation of caverns and channels are the direct result of chemical dissolution, which at a certain stage of the process could be supported by an erosive action of water. As turbulence of water is increased, the quantity of solute is also increased. Depending on the overall configuration of Karst environment, it differs as matured and un-matured Karst systems.

Hydraulical flow conditions in Karst can be diffuse, conduit and mixed system. The conductivity of Karst conduits does not influence the drainage of the low permeability matrix. In this case the drainage process is influenced by the size and hydraulic parameters of the low permeability blocks alone. This flow condition has been defined as matrix-restrained (Diffuse) flow regime **(Worthington 2009).**

During the base flow recession of premature Karst systems and the flood recession of mature systems, the recession process is dependent not only on the hydraulic parameters and the size of the low-permeability blocks, but also on conduit conductivity, and the total extent of the flow regime. This flow condition has been defined as conduit flow regime **(Worthington 2009).**

1.2.4 Problems Related To Dams under Karst Environment

Since the time, dam engineering community has learned, as we normally do, from failures or incidents that solution features do create a significant risk to the safety of dams. When a dam is constructed over a Karst foundation, there is likely a network of soil-filled and open solution features. The impoundment of a pool creates a flow gradient due to the differential in the head between the reservoir and the tail water **(USSD 2009).**

The impoundment of pool also significantly increases the pore pressure present in the foundation. Rock surrounding the solution features will provide support for the overburden pressures resulting in a very low confining stress in the soil-filled solution features. The high pore pressure and low confining stress couple to make the soils very weak (a zero effective stress condition is likely) and easily eroded even when subjected to small flow gradients **(USSD 2009).**

This explains how soil-filled solution features can turn into a network of open voids during the life of a dam. Dam safety concerns begin when an open feature is exposed to the foundation or embankment **(USSD 2009).**

1.3 Problem Statement

1.3.1 Problems Related To Karst

Karst is defined as a terrain, generally underlain by limestone or dolomite, in which the topography is chiefly formed by the dissolving of rock, and which is characterized by sinkholes, sinking streams, closed depressions, subterranean drainage and caves. A wide range of closed surface depressions, a well-developed underground drainage system, and strong interaction between circulation of surface water and groundwater typify Karst. Due to very high infiltration rates, especially in bare Karst, overland and surface flow is rare in comparison with non-Karst terrains **(UNESCO 2010).**

Carbonate rocks are more soluble than many other rocks. They are subject to a number of geomorphologic processes. The processes involved in the weathering and erosion of carbonate rocks are many and diverse. The varied and often spectacular surface landforms are merely a guide to the presence of unpredictable conduits, fissures and cavities beneath the ground.

Diversity is considered the main feature of karstic systems, which are known to change over time and in space so that an investigation of each system on its own is required **(UNESCO 2010).**

Interactions between the surface and subsurface in Karst are very strong **(Bonacci 1987).** Groundwater and surface water are hydraulically connected through numerous Karst features that facilitate the exchange of water between the surface and subsurface **(Katz et al 1997).**

An important issue in studying this research is that subsurface water is highly heterogeneous in terms of the location of conduits, the location of vertically moving water, and flow velocities. **(UNESCO 2010).**

Rock fill dams on Karst foundation requires carful analyses of seepage and slope stability of dams. The awareness of changes in the behavior of dam and surrounding environment under different water level is crucial in predicting the possible causes of failures and taking necessary control measures and methods. According to available statistics, piping through dam and foundations are the primary reason for dam failure.

Seepage through the dam and leakage through the foundation are a major problem that occurred in many places after the first filling of reservoir. Hence, the possibility of water flow is uncertain in Karst landform which requires careful analysis before and after construction.

In some cases, under Karst environment, despite an extensive investigation and engineering measures, the results were in adequate relative to the time and cost incurred. This is mainly due to the consequences of karstification and change in the hydraulic behavior of the foundation especially under the dam body.

1.3.2 Karst Features of the Research Area

1.3.2.1 Geological & Geomorphologic Settings

The Sofumar Cave System and the cavernous river system developed on limestone beds. The limestone unit, with a total exposed thickness of less than 100 m in the vicinity of the caves Fig.7 consists of thin, limestone beds intercalated with abundant marl, and sandy limestone beds at the top, and massive, crystalline limestone beds intercalated with thin marl and mudstone beds at the bottom. The limestone beds are usually very fine and lithographic, and rarely contain fossils. At least two types of limestone units have been identified i.e., limestone with rare dolomites and calcite joined by entirely crystallized cement. Sandstone conformably overlies the limestone unit. In the vicinity of the caves, the sand- stone beds are very thin **(Asfawosen Asrat 2015).**

1.3.2.2. Surface Hydrology & Landforms

The River Weyib, which starts from the foot of the southeastern highlands to the west of Sofumar drains to Genale River further to the south. Weyib River upstream, the sink and downstream the resurgence, winds down a narrow and steep, forested gorge, leaving in between the dry valley. The area is generally characterized by motional landforms (cliffs and gorges) rather than depositional landforms. There are numerous dry valleys connecting to the main river gorge. However, all the tributaries are generally dry. Some evidences of depositional (colluvial and alluvial) features are observed in the old dry valley where some colluviums of limestone, sandstone and basalt boulders are deposited by gravity action over old, semi consolidated pebbles and conglomerates presumably deposited by the River Weyib when it was flowing along the now abandoned meandering valley **(Asfawosen Asrat 2015).**

1.4 Research Question

In this research, the major issues to be addressed is the hydraulic analysis of rock fill dams mainly seepage, slope stability and related failure risks. For analysis of such engineering parameters, geological and hydro-geological data are key element. According to researchers, the heterogeneous geological formations and point nature of data need analysis and assessments before and after operations **(Milanovich Peter 2004).**

In this research, the heterogeneity and anisotropy representing spatial and temporal hydraulical and geological variations were well addressed by estimating the average saturated hydraulic and material parametric values of the aquifer as dam, abutment and foundation. Correspondingly, obtained values were assigned for the assumed rock fill dam to make the analysis simple and the results valuable.

1.5 Research Objectives

1.5.1 General Objectives

The general objective of this research was to conduct hydraulic analysis of rock fill dam at selected locations within research area through estimated hydraulic and material properties parametric values. The analysis was mainly for seepage and related risk of dam, abutment and foundation considering hydraulic and material homogeneity between them.

1.5.2 Specific Objectives

The specific objectives of this research were to assess base flow difference, to estimate hydraulic and mechanical properties parametric values, analysis of seepage and related risk under stable slope through arranged water levels and dam type scenarios.

1.6 Scope of the Research

The hydraulic analysis mainly focused on estimation of seepage values under different assumed water levels and section of rock fill dam on stable slope of abutment and dam body. The research location under any circumstances cannot be taken as dam location and the dam as proposed dam and the study did not include any other hydraulic and material engineering problems.

During the analysis, no previously proposed dam was considered for there is no detail feasibility data that can be used for comparison. For the research output evaluation and comparisons, local and international published scientific research works, software outputs excel based quantitative analysis were used.

1.7 Significance of the Research

This research lacks data obtained by direct method of investigations and case study research couldn't be processed. The results obtained can be an indicative for seepage and related risk of dams under similar sites. Data obtained can be used in preliminary works and calibration of other similar works. The research results can also be used as an initial for future research works of others.

Chapter Two - Literature Review

2.1 Karst and Karstification

2.1.1 Karst Types

2.1.1.1 Morphological Classification

Using morphological features as the base, Karst divided in to three types named as holocaust, merokarst and transitional type **(Cvijic 1926).**

Holocaust (Complete Karst): develops in areas comprised entirely of soluble carbonate rocks. It is characterized by the existence of surface and underground Karst phenomena, making feasible further development and the creation of new Karst phenomena. The vast bare and rocky land, without arable land and with or without the presence of vegetation gives very specific appearances to holocaust regions.

- **Merokarst (Incomplete Karst):** have many properties of non Karst area and the carbonate rocks are far less subjected to karstification. Therefore; the Karst phenomena are infrequent and the depth of karstification is limited.

- **Transitional type:** Karst has a degree of karstification as to fall between Holocaust and Merokarst. It is mainly found in limestone's that are isolated by impermeable less soluble sediments.

2.1.1.2 Hydro-Geological Classification

This classification is based up on differences in geological characteristics and above all, upon lithological and structural characteristics. According to this classification, there are two clearly different types of Karst **(GSNA 1988).**

- **Platform Karst:** is characterized by horizontal or gently sloping strata and by platform relief. Carbonate rocks usually contain a higher percentage of Marley material. These sediments are often found tightly imbedded between impermeable rocks or lying over impermeable rocks.
- **Geosynclines Karst:** Develops in distinctly folded and ruptured or faltered carbonate rocks. With favorable climatic conditions, the synclinal fold regions are excellent environments for the maximal karstification process **(Komatrina 1973).**

2.1.1.3 Tectonic Classification

In this classification there are two main types of Karst **(Herak 1976).**

- **Epiorogenic Karst:** develops within the carbonate or other soluble rocks deposited in Sea or under freshwater conditions.
- **Orogenic Karst:** develops in carbonate and other soluble rocks that were subjected to strong tectonic movements, very often with over thrusts as extreme structural forms.

2.2 Karstification Level

Karst is directly related with the carbonate rocks, more specifically limestone and dolomites. Limestone is the most important rock in which karstification takes place, dolomites susceptibility to karstification depends upon the thickness and position within the geological structure **(GSNA 1988).**
Karstification level can be determined by empirical formula that relates the average hydraulic conductivities of the Karst lime stone with the relief or height of the land form. The formula expression stated as follows.

The formula expression stated as,

$$\mathbf{Ki = 23.97e^{-0.012h}}$$

Where, e =2.72

H is height of aquifer (depth of river)

Ki Is height of aquifer (depth of river)

By investigating the formation and distribution of Karst porosity associated with Karst springs, Le Grand, and La Moreaux concluded that karstification decreases with depth and that “progressive concentration of water in the upper zone of saturation produces master conduits there, but causes no appreciable increase in flow and solution at great depths.

The results of permeability investigations from 146 deep observation wells drilled in regions with elevations of 200 m 1000 m above mean sea level have been, generally speaking karstification decreases with depth. The zone of greatest karstification, and at the same time the highest porosity, is at the depth ranging from surface to 10 to 20 m (epikarst zone).

If we assume that karstification decrease with depth according to an exponential law, then KI= $a.e^{-bH}$ where, KI is the index of karstification, H is the depth in meters, e is the natural logarithm, a and b are coefficients. The variation of karstification with depth can be expressed by KI = $23.9697e^{-0.012H}$, it is evident that karstification in the surface zone of 0 to 10 m is about 30 times greater than at a depth of 300 m **(GSNA 1988).**

For depths greater than 300 m, the index of karstification approaches its minimum value, somewhat greater than zero. The zones where the karstification extends to depths to 300 to 500 m and locally even deeper are not very large and do not involve a considerable rock volume. Karstification is associated with tectonic zones and has been expressed in two dimensions. The highest intensity of karstification is in the aquifer section with the largest storage capacity in the zone of water table fluctuations. Karstification decreases below the lowest ground water levels. The base of karstification and the minimum water level coincide **(GSNA 1988).**

Generally, the slope of the base of karstification leads to the zone of the aquifer discharge. The most active Karst channels are directly above the base level. The existence of Karst drains below this level is not excluded, but they are rare and of limited transport capacities. **Vlahovic (1975)** analyzed many exploratory borings from Yugoslavia. He was able to prove that in the zone above the lowest levels of the water table there are 3.3 times more Karst channels and caverns than in the zone below.

2.3 Engineering Classification of Karst Limestone

As per the study indicated in international journal of scientific research volume 4, issue 5, May 2013 and Ford and Williams, 1989 classification, the engineering aspects of Karst environment has geomorphologic characteristics of the five classes **(Abdeltewab 2014).**

- **K I:** Only in deserts and glacial zones, or on impure carbonates, sinkholes rare, rock head almost uniform; minor fissures; low secondary permeability, Caves are rare and small; some isolated relict features.
- **K II:** Minimum in temperate regions, small suffusion or dropout sinkholes; open stream sinks, many small fissures, Fissures are widespread in the few meters nearest surface, Caves are many and small (size less than 3m across).
- **K III:** Common in temperate regions; minimum in the wet tropics, many suffusion and dropout sink-holes; large dissolution sinkholes; small collapse and buried sinkholes extensive fissuring; relief less than 5m , loose blocks in cover soil, extensive secondary opening of most fissures, caves are many (size less than 5m across at multiple levels).
- **K IV:** Localized in temperate regions; normal in tropical regions, many large dissolution sinkholes; numerous subsidence sinkholes; scattered collapse and buried sinkholes, Pinnacled; relief of 5-20m; loose pillars, extensive large dissolution openings, on and away from major fissures, caves are many (size greater than 5m across at multiple levels).
- **K V:** Only in wet tropics, very large sinkholes of all types; remnant arches; soil compaction in buried sinkholes, tall pinnacles; relief of greater than 20 m; loose pillars undercut between deep soil fissures.

2.4 Hydraulic Failure Mechanisms In Rock Fill Dams

2.4.1 Seepage & Slope Stability

Rock fill dams on Karst foundation requires carful analyses of water tightness and slope stability of reservoirs. The awareness of changes in the behavior of dam and surrounding environment under different water level is crucial in predicting the possible causes of failures and taking necessary control measures and methods. According to available statistics, piping through dam and foundations are the primary reason for dam failure.

Seepage through the dam and leakage through the foundation are a major problem that could occur after the first filling of reservoir. Hence, the possibility of water flow is uncertain in Karst landform which requires careful analysis before and after construction. In some cases, under Karst environment, despite an extensive investigation and engineering measures, the results were in adequate relative to the time and cost incurred. This is mainly due to the consequences of karstification and change in the hydraulic behavior of the foundation especially under the dam body **(IACDS 2015).**

2.4.1.1 Internal Erosion

Recently, as a result of numerous failures, the term internal erosion has replaced the term piping as the all-inclusive term per international practice. The term "internal erosion" encompasses a variety of more specific terms that are found in literature. Some of the terms that have been used include concentrated leak erosion, scour, seepage erosion, piping, backward erosion piping, internal instability internal migration, contact erosion, heave, blowout, tunneling, and saturation failure **(IACDS 2015).**

2.4.1.1.1. Concentrated leak erosion

Concentrated leak erosion, also referred to as scour, seepage erosion, or tractive force erosion, can occur when preferential flow paths develop in earth embankments, their foundations, or at contacts between the fill and concrete structures or bedrock. In this mechanism of internal erosion, soil particles are detached form along the preferential flow path and the soil is then eroded by water flowing at relatively high velocity. The eroded particles are then carried through the preferential flow path to the filter face or other downstream exit. Concentrated leak erosion is responsible for the majority of catastrophic dam failures due to internal erosion **(IACDS 2015).**

2.4.1.1.2 Geologic defects in the foundation

Foundation defects have been the root cause of some historic dam failures and incidents. Foundation cleaning, preparation, and treatment are critical aspects of dam construction that often receive inadequate attention or concern. Some of the more common geologic conditions that may contribute to development of preferential seepage paths include openly fractured or jointed foundation soils or rock, open- work gravel or gap-graded soil foundations, Karst or other geologic irregularities that provide concentrated leaks through the foundation **(IACDS 2015).**

2.4.1.1.3 Backward erosion piping

Backward erosion piping is erosion that initiates at an exit point and progresses backward towards the reservoir. Particles of soil are carried away by the seepage until eventually a pipe, is formed from the downstream exit point to the reservoir. Unlike concentrated leak erosion, a discrete flow through the embankment or a geologic defect in the foundation is not a necessary condition to initiate backward erosion. A continuous layer susceptible to piping, beneath a layer or structure capable of forming a roof, is required for backward erosion piping **(IACDS 2015).**

2.4.1.1.4 Vertical Seepage paths/heave and blowout

Evaluation of seepage forces and pore pressures at the downstream toe of an embankment is complicated and requires careful consideration of site conditions. Simplified formulae for estimating exit gradients and safety factors can be easily misused without an understanding of the specific "failure mechanism" being considered. For a condition at the downstream toe to progress and develop into internal erosion mechanism, which results in breach of an embankment, requires additional considerations **(IACDS 2015).**

2.4.1.1.5 High Exit gradients and uplift pressures

If analyses indicate the potential for seepage gradients to approach the critical gradient or for uplift pressures to be near the resisting overburden pressures, it is possible that the embankment and foundation may experience possibly cracking of a low permeability confining layer. Defects such as cracks, roots, penetrations, burrows, thinning, etc., may affect the performance of confining layers. Seepage flow could be concentrated through the crack that develops (Blowout/uplift), where defects exist **(IACDS 2015).**

2.4.2. Seepage Magnitudes of Dams under Operation

Observed and measured magnitudes of seepage through dam bodies and foundations at different parts of the world have been showing today that leakage through dams is inevitable under Karst environment.

Table 1 Measured Seepage loses of dams under Karst environment

Ser. No.	Reservoir Name	Country	Seepage/ Leakage Cum/sec
1	Keban	Turkey	26
2	Camaras	Spain	11.2
3	Marun	Iran	10
4	Great Fall	USA	9.5
5	Canelles	Spain	8
6	Mavrovo	Mecodenia	9.5
7	Busko Blato	BiH	40
8	Hutovo	BiH	10
9	El Kajon Dam	Hunduras	NA
10	Hails Bar	USA	54
11	Montejaque	Spain	4
12	Vertac	Montenegro	25
13	Visegrad	Bosinia	14
14	Iliki	Greese	13
15	Salakovich	BiH	10
16	Lar	Iran	10.8
17	Atatuk	Turkey	11
18	Mosul	Iraq	NA
19	Samanalawewa	Sri Lanka	2
20	Perdikas	Greese	NA
21	May	Turkey	NA

(Adapted from Milanovich Peter2014)

2.5 Base flow Separation Methods

2.5.1 Hydrograph Separation Method

Hydrograph separation method aims to distinguish stream flow derived from surface runoff and that derived from ground water, based solely on time series record of stream flow. The method is popular because it operates only using readily available stream flow data. The base flow separation method relies on the principle that runoff events are of relatively short duration whereas groundwater responds more slowly to rainfall recharge.

Hydrograph base flow analysis uses available stream flow data from a gauging station and provides information on groundwater discharge as a function of time over the period of record. The results relate to the entire catchment upstream of the gauging station and so the method provides a spatially integrated assessment and does not provide any information on where within the catchment the groundwater discharge may be occurring.

2.5.1.1 USGS hydrograph separation Software

This software is prepared as an extension programme for the analysis of base flow as a function of measured or observed stream flows. The method used stream flow as an input and the days of records with formatted form.

2.5.1.2 Excel based hydrograph separation

Professional excel based base flow separation method is available for the hydrograph separation method. Unlike USGS, this method considers the effect of rain fall on base flow magnitude.

2.5.1.3 Hydrograph separation Algorithms

Excel based base flow separation method is possible to use for the hydrograph separation method. This method deploys the use of excel using the formula developed by the scientists like Chapman.

2.5.1.4 Hec- HMS Hydrograph Separation

2.5.1.4.1 Hec-GeoHMS Process

Hec-GeoHMS is powerful software in the process of setting loss, transform and routing method that will be used in Hec-HMS and generate the HMS schematic, such as river reach, sink and sub basin. Catchment can be divided on the basis of stream flow gage station location. To make single sub basin the catchments within the basin can be merged. By merging and split of Basin River reach can be divided and multi routing can be avoided. River length, basin area, longest flow path, centroid of basin are to be considered well to be used in HMS.

2.5.1.4.2 Hec-HMS Base flow modeling

Hec- HMS modeling can be done by three methods

- Constant, monthly- varying values
- Exponential recession model
- Linear reservoir accounting model

2.5.1.4.2.1 Constant, monthly-varying base flow

The simplest base flow method by using Hec-HMS. It uses base flow as a constant flow that is able to vary monthly. In this system, user specified flow can be added to the direct runoff for each simulation step.

2.5.1.4.1.2 Exponential recession model

Hec-HMS comprises a exponential recession model to represent watershed base flow. (Chow, Maidment, and Mays 1988). It defines the relationship of Qn, the base flow at any time t, to an initial value as:

Qt = QoK'

Where, Qo = Initial base flow at time zero; K = an exponential decay constant

Table 2 Recession constant values

Flow component	Recession constant, daily
Groundwater	0.95
Interflow	0.8-0.9
Surface runoff	0.3-0.8

(Adapted from Hec-HMS modeling manual)

Figure 1 Base flow in multi routing

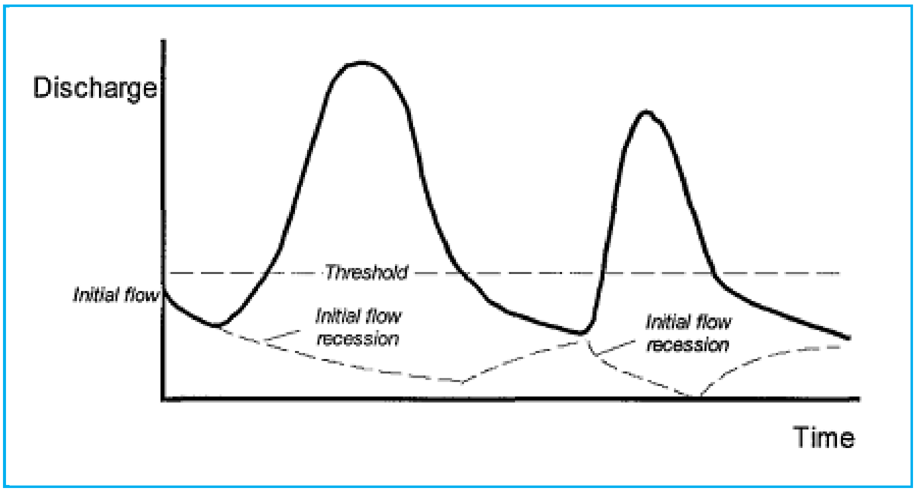
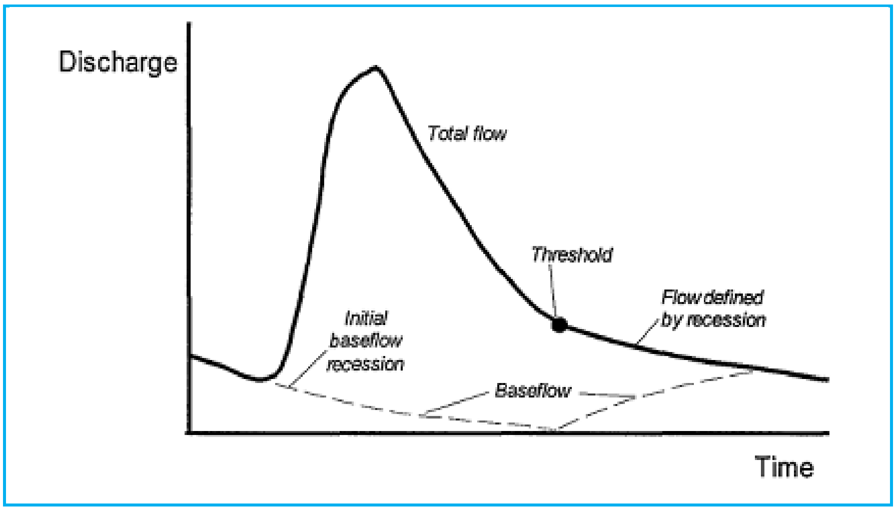


Figure 2 Base flow in single routing



(Adapted from Hec-HMS modeling manual)

2.6 Hydraulic Gradient Analysis

The direction of flow between the groundwater and a river can be determined by comparing the hydraulic heads within the ground water with the water level in the river. If the river level is higher than the groundwater there will be a potential for the river to leak water into the groundwater. Conversely, if the river level is lower than the groundwater level adjacent to the river then there is a potential for groundwater to flow into the river. It is possible to estimate the magnitude of the water exchange using Darcy's Law, which calculates flow as the product of the hydraulic gradient and transmissivity.

$$q = T\frac{\partial h}{\partial x}$$

Where: q is the flow rate (per unit length of river)

T is the transmissivity

h is the hydraulic head

X is distance

As discussed above, groundwater inflow and outflow rates show relatively high variations over small spatial scales. Therefore, piezometers need to be positioned to so that measured hydraulic heads reflect those in the regional aquifer, and not local-scale processes. If the bore is placed very close to the river, then the head gradient can be difficult to measure and the estimated flow rate may not be representative of the larger region. However, as the distance between the bore and the river increases, other problems will be created.

2.6 Flow Difference Method

This method is perhaps the most straightforward of all of the methods for characterizing surface water groundwater interactions. At the simplest level, the flow rate in the stream at a time when surface runoff is likely to be negligible is assumed to be equal to the groundwater discharge rate in the catchment upstream of the gauging station. Usually measurements of stream flow rate can be carried out at a number of locations along the river on the same day and the groundwater inflow (or outflow) is estimated from the differences between adjacent flow gauging.

The method assumes that the changes in river flow are due to groundwater inflows and outflows and so will only be accurate if all other inflows and outflows are negligible or have been quantified. The method is usually applied at sufficient time after rainfall so that it can be assumed that downstream increases in river flow are due to groundwater inflow. However, groundwater inflow will be underestimated if river losses occur over that same reach.

This method provides information on groundwater inflows only on the day (s) on which sampling takes place. The method is most useful for identifying locations of groundwater inflow and for quantifying the spatial variations in inflow rates. Conceptually, if more than one gauging station is located along a reach of river (and there is no tributary inflow between the stations), then differences between these station records can yield some information on temporal variations in groundwater inflow. The existence of more than one gauging station along a river reach, however, would be rare.

2.7 Geo-Studio Software

One of the powerful software in analysis of Steady state seepage analysis is Geo-Slope, 2012 using the state of the art Finite Element Method based computer program – Seep/W. Slope and other analysis like slope can be performed under steady state by Static and dynamic loading conditions. During the analysis, Slope/W, Quake/W can be used under different loading conditions.

2.8 Rock Fill Dams

Rock fill dams offer the choicest of all alternatives for the construction of high dams under complex geological settings. The behavior of such material helps to encounter larger strain including high seismic load. Rock fills have higher shear strength. They perform as Mohr/Coulomb material with no cohesion but an angle of internal friction more than 35^0. Effectively compacted rock fills can have density of 2000kg/m^3 with angle of friction up to 65^0. Compaction of such fills reach very high stability to form vertical stable slope **(Roland Pusch 2008).**

The hydraulic conductivity of this porous media has been subjected to numerous studies and modeling attempt assuming non turbulent flow. Rock fill Materials can have a hydraulic conductivity with the range of 1E-5 m/sec to 1E-7m/sec with and without addition of fines respectively. A rock-fill dam is one composed largely of fragmented rock with an impervious core. The core is separated from the rock shells by a series of transition zones built of properly graded material. A membrane of concrete, asphalt, or steel plate on the upstream face should be considered in instead of an impervious earth core when sufficient impervious material is not available **(Roland Pusch 2008).**

The rock-fill zones are compacted in layers 30 to 60cm thick by heavy rubber-tired or steel-wheel vibratory rollers. It is often desirable to determine the best methods of construction and compaction on the basis of test quarry and test fill results. Free-draining, well-compacted rock fill can be placed with steep slopes if the dam is on a rock foundation. If it is necessary to place rock-fill on an earth or weathered rock foundation, the slopes must, of course, be much flatter, and transition zones are required between the foundation and the rock fill **(Roland Pusch 2008).**

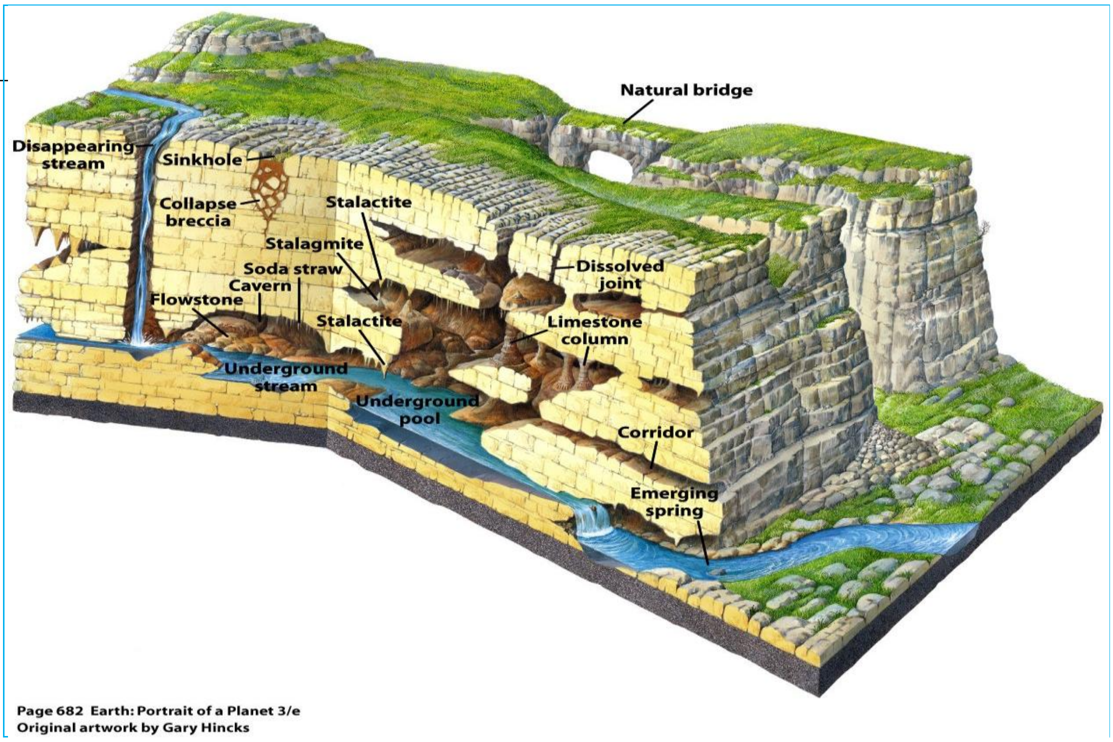
2.8.1 Rock Fill Dams under Karst Environment

Construction of dams and reservoirs in Karst is historically known as a very risky task. In spite of very detailed geophysical investigations and repeated sealing treatments, the possibility for dam failure cannot be eliminated. In the Karst environment, with its highly random distribution of dissolution features, some uncertainties always remain.

The final determination of the adequacy of sealing measures comes after the first reservoir impoundment or even later. In many worldwide examples, water tightness treatment during dam construction was only partially successful, with some remedial work after impoundment being quite common. However, in some cases, the problem is simply too complicated and cannot be overcome **(Peter Milanovich 2014).**

Special approaches have to be undertaken in order to prevent seepage from reservoirs. The key elements are a good geological map and proper geophysical investigations. These investigations are key prerequisites of dam construction in Karst and cutting costs through restricting them usually results in increasing the chance of project failure. To deal with Karst successfully, innovation, engineering practice, execution feasibility, and commercial understanding have to be undertaken. Grouting alone is definitely not adequate in the case of large Karst conduit **(Peter Milanovich 2014).**

Figure 3 Karst Environment Schematic map. (Adapted from Marshak, 2008).

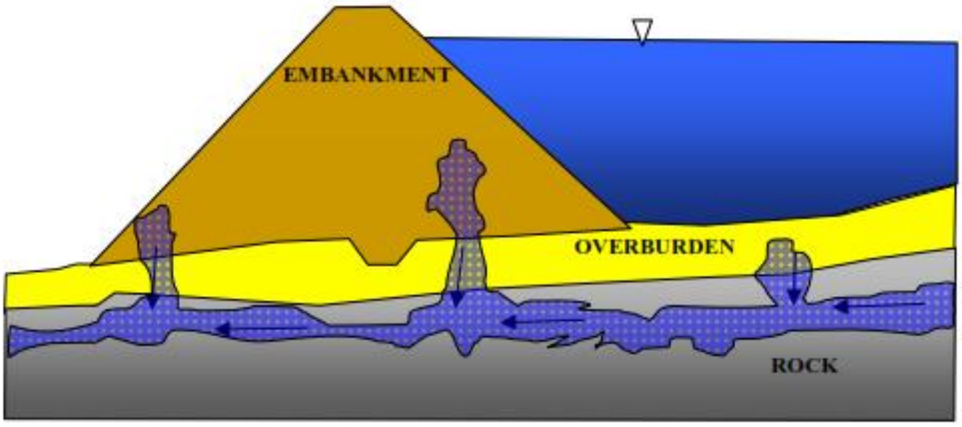


2.8.1.1 Seepage and Related Risks Case Studies

2.8.1.1.1 Solution features

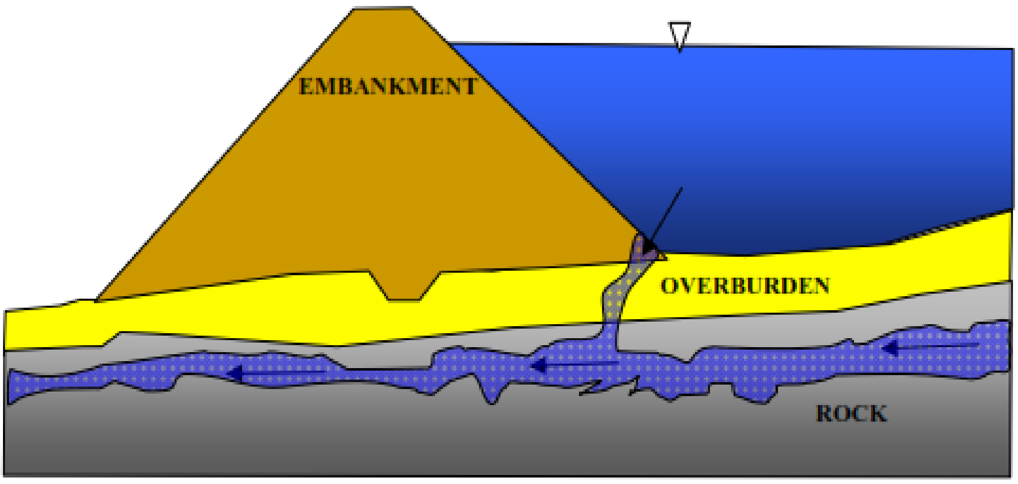
Scenario 1: If the solution feature is a cave with a rock roof and only a limited area that is exposed to the foundation of the dam (Figure 4 & 5), then the flow through the pipe will be limited to the cave opening size and hydraulic capacity. This case may lead to severe leakage or draining of the reservoir but may not always lead to breach. Once soil is eroded from a solution feature, a void is now present with a soil roof. The stability of the roof is dependent on the soil type and the seepage condition present on the roof face. The confining stress on the face of the roof will be very low. Very small seepage gradients across the roof face can now cause erosion and progression of a pipe even in soils that are normally very resistant to scour such as stiff, high plasticity clays

Figure 4 Erosion into Solution Feature (Adapted from USSD, 2009)



If the solution feature does not clog and continues to transport the eroded soil, the pipe can progress to the ground surface. If the pipe enters the reservoir then the flow through the pipe will rapidly erode causing gross enlargement of the pipe and possibly initiate breach of the dam. The likelihood of breach depends primarily on the extent and location of the exposure of the soils to the void. Other factors include dam features, material types, reservoir volume, and the geometry and capacity of the solution features **(USSD 2009).**

Figure 5 Erosion into cave opening (Adapted from USSD 2009)

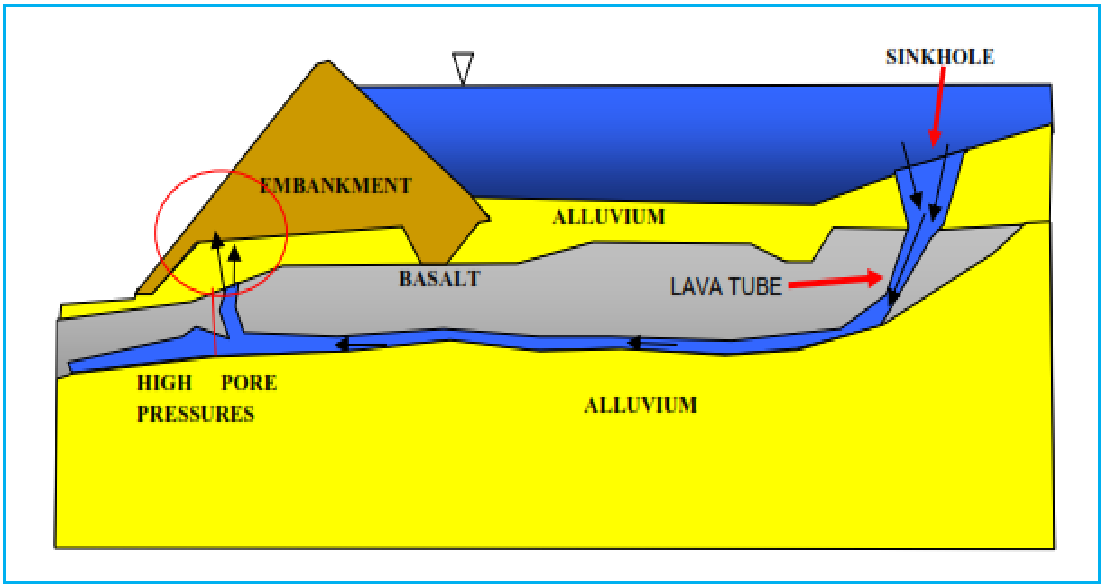


Scenario 2: if in the first scenario, the flow through the solution feature also has some exposure to the soil downstream of the core, high pore pressures can develop at the embankment toe. Full upstream pool heads can be transmitted to the foundation directly below the downstream embankment slope creating a low effective stress condition resulting in slope instability and blowout

A case history example of this scenario is the failure of the Swift #2 canal dam located on the Lewis River in south-central Washington **(Esterle 2006).** At this site, the feature in the foundation rock was actually a lava tube rather than a Karst feature. The same condition could easily develop in a Karst environment. The dam was an 83-foot tall earth embankment. The dam was completed in 1959 and failed on April 21, 2002 **(USSD 2009).**

Over time, the alluvial materials that were deposited over the basalt were eroded into the tube. The void space expanded upstream and developed a sinkhole through the alluvium into the reservoir (Figure 5). The pore pressure quickly increased in the tube and an uplift failure occurred at the toe of the embankment which led to breach (Regan, et al., 2008). Figure 5 show the breach initiation just after heave of the toe. Figure 4 shows the upstream sinkhole revealed after complete loss of the reservoir **(USSD 2009).**

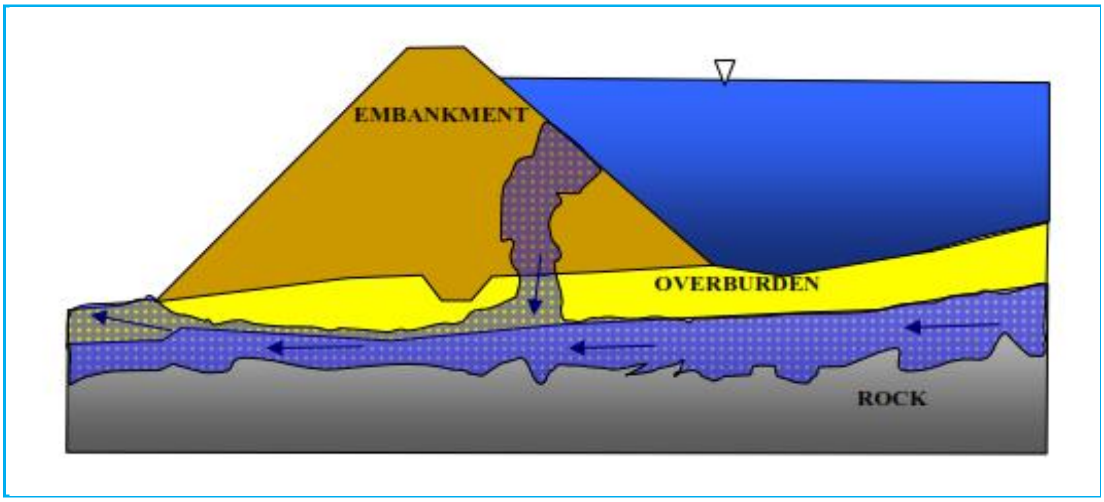
Figure 6 Failure Mechanism (USSD 2009)



Scenario 3: The last condition is where there is continuous exposure to the soil along the so solution feature **(Figure 8).** In this case, if a pipe progresses to the reservoir, gross enlargement will occur along the entire soil/solution feature contact and will likely lead to

catastrophic failure if there is sufficient storage capacity in the reservoir.

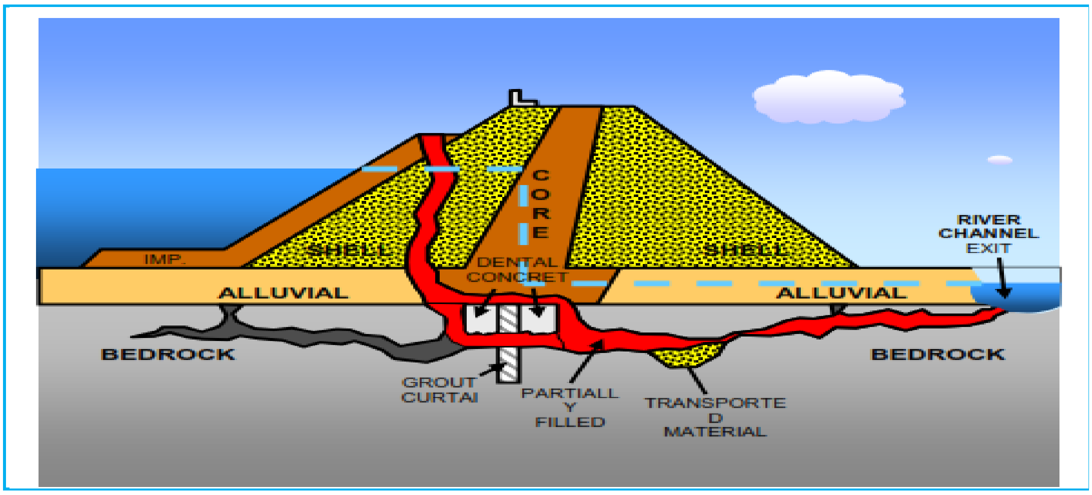
Figure 7 Solution Feature with Continuous Soil Exposure (USSD 2009)



A case history of this scenario is the failure of the East Fork Dam in south-central Kentucky. The dam was 45-feet tall and was completed on November 7, 1978. Heavy rains quickly filled the reservoir in early December of that year. Within two days of filling, the dam failed. The failure was attributed to piping into the Karst foundation. Apparently, an open void network existed below the foundation soils prior to filling the reservoir.

A case history of Clear water dam located in south Missouri, it is 154 foot tall dam (Figure 8) that was completed in 1948. During construction, cut of trench was excavated to the foundation rock. Some large solution features were revealed and treated with concrete grouting. The reservoir experienced the pool of record in May 2002. The following period of low pool, a sinkhole opened up on the upstream embankment face above the pool elevation. It is believed that the sinkhole pipe developed below the treatment through a Karst feature that was originally soil filled but over time was cleared out by erosion.

Figure 8 Seepage representation of Clear water dam (USSD 2009)



2.8.2 .2 Hydraulic Design of Dams under Karst Environment

According to researchers, dams under Karst foundation are to be designed as “design for geology “and “design for hydrogeology”. As the design of any dam, design criterias, guidelines, standards, and manuals must be followed.

Dam safety concern under Karst environment is started in1940’ G.C as flood control measures and the first ground breaking foundation treatment were started for six selected dams in America.

2.8.2.2.1 Hydraulic Design of dam section

Considering the hydraulic condition of the dams, the dam body should be designed for seepage and the phreatic line not to cross the downstream face unless it is made of homogeneous material. The magnitude of seepage shall not create any material erosion and safe out let should be established either as rock toe at the end in case of homogeneous types or filter material passage under the dam at safe location far from the downstream end.

2.8.2.1.2 Hydraulic Design for Settlement and deformation

The hydraulic behavior of dam material shall not create any change under saturation due to static and dynamic loads or combined forms. Settlement due to static and dynamic loads should not be beyond the allowable limits and the deformation that can be created due to the response of grain material under dynamic loads to the water proofing materials or materials with the abutment should be checked for its limits.

2.8.2.1.3 Hydraulic Design for upstream and downstream slope

The upstream and downstream slope of the dam should be designed for stability of sudden drawdown and steady state condition respectively. The slope must be checked for saturated condition since the behavior of materials is expected to change. The recommended value of safety factors for different loading conditions as per the standard is as shown in the table below.

Table 3 Recommended value of safety factors for different loading conditions

Cases	Loading Condition	Critical Slope	FOS
I	End of construction	Upstream	1.3
II	Sudden drawdown	Upstream	1.3
III	Steady state seepage	Upstream	1.5
		Downstream	1.5
IV	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

2.8.2.1.4 Hydraulic Design for dam foundation

The foundation must be designed for seepage control through water proof treatment and should be safe for material erosion and change of material properties. Leakages from the dam to it or through the foundation are the main cause of failure as stated in different manuals and guide lines. The type and the magnitude of foundation treatment mainly depend on dam material, foundation material behaviors under saturated conditions created by the hydraulic gradient of water head in the reservoir or geological defect. There are many methods of foundation treatment but their appropriateness is determined as per the specific site geological and hydrogeological conditions.

2.8.2.1.4 Hydraulic Design for Seepage Control

Karst dissolving behavior can cause significant problems with the structural integrity of various impoundments, specifically dams. Dams located in Karst terrain often times deal with significant groundwater seepage issues. Given sufficient time, groundwater will eventually erode enough of the underlying soil structure to create conditions where dam failures can occur. To prevent dam failures from occurring in karst terrain, grout curtains are typically installed. As previously mentioned, grout curtains are installed to prevent excessive seepage under dams and to minimize groundwater infiltration.

Four types of seepage cutoffs have been successfully utilized in limestone formations: open-cut excavated cutoffs, diaphragm wall cutoffs, secant pile cutoffs, and grouted cutoffs. After falling somewhat out of favor due to lack of success on some projects, recent advances in materials, procedures, and techniques have resulted in practitioners regaining confidence in grouting.

2.8.2.4.4.1 Excavated Open Trench

This method consists of excavating and removing the rock and joint materials to the required depth and length. The excavation is then backfilled with the appropriate concrete mix to form a thick and continuous wall. It results in a positive cutoff since the termination points can be visually inspected before backfilling operations begin. This method is applicable when the voids are discovered during the initial construction, when the overburden or fill above the rock surface is shallow and the required depth is easily obtained by conventional drilling, blasting and excavation equipment. This is a traditional method used for many dams worldwide

2.8.2.4.4.2Diaphragm Wall

This method consists of excavating a narrow trench that is temporarily stabilized by a slurry fluid in a series of continuous panels. After the excavation for a primary panel is completed, the excavation is then backfilled with concrete. Reinforcing is added if required. After completion of several primary panels, secondary panels are then excavated between the existing primary panels and backfilled with concrete to form a continuous wall. It is important that the completed diaphragm wall have adequate structural strength and integrity as well as to provide a positive cutoff for differential hydraulic head.

2.8.2.4.4.3 Secant Pile Wall

This method consists of drilling large primary and secondary piles, in sequence, and then backfilling them with concrete. After two primary piles have been drilled and the concrete has cured, a secondary pile is drilled between them and backfilled with concrete to form a continuous concrete cutoff wall. This is an effective method where the depth of overburden is shallow and the depth of penetration into the rock is significant.

2.8.2.4.4.4 Grouting

Historically, grouting operations in limestone have not been very successful because of our lack of understanding of the characteristics of limestone foundations and with the conventional equipment and procedures, it was difficult to control the grout mixes and injection pressures needed to ensure that the grout filled all of the voids. Monitoring of grout takes and the results were accomplished manually after a zone had been grouted. After a hole was completed an assessment was made.

This process made it difficult to measure the degree of success of grouting an area or the project as a whole. While grouting was successful in stopping muddy water from flowing from the toe of the Wolf Creek Dam between 1968 and 1970, additional studies concluded that because of the high head on the foundation, grouting could not be considered as permanent long term treatment and a diaphragm cutoff wall was then installed.

2.9 Types of Dams Affected by Seepage/Internal Erosion

(Adapted from FERC guide manual)

Seepage may develop and stay within the same soil horizon, or it may pass into other horizons, to a free face downstream of the structure, depending upon the permeability of the soils and other subsurface features. Movement of soil by subsurface flow is typically initiated by a new or elevated groundwater gradient and/or flow velocity. Therefore, any change in seepage conditions can result in the transportation of soil particles if the conditions are conducive to the movement of particles of soil. More details about the conditions susceptible to internal erosion and the likelihood of internal erosion developing are discussed below.

According to FERC guide manual, types of structures affected by internal erosion generally include:

- Earth and rock fill dams on an alluvial soil foundation
- Earth and rock fill dams on a solution-prone, karstic, or highly weathered rock foundation
  Earth and rock fill dams on steep rock abutments
- Earth and rock fill dams on rock foundations with naturally occurring voids or discontinuities
- Earth and rock fill dams with manmade conduits penetrating the embankment and/or through bedrock
- Earth dams without a system of filters and drains to collect, control, and release seepage
- Concrete dams on an unprepared rock foundation with erodible seams, joints, fractures, layer, or other flaws within the rock
- Concrete dams abutting earth fill embankments
- Concrete dams on soil foundations
- Concrete lined spillways with soil foundation
- Concrete guide walls penetrating embankments or separating aquifers and a groundwater gradient
- Pump stations and forebay
- Earthen levees

- Levee T-walls
- Lock walls on rock foundations with naturally occurring conduits or discontinuities
- Lock walls

The key thing to keep in mind is that every foundation is unique to the geologic conditions of the structure foundation

The following sections provide general discussion about the more common structures that can be affected by internal erosion and piping. More details about these and other types of structures can be located in some of the references noted at the end of this chapter, as well as general design manuals for embankment dams. Prior to the evaluation of a specific project, more research should be performed to get a full understanding of the project design features.

2.9.1 Earth and Rock Fill Dams

Modern earth and rock fill dams are designed to filter, collect, and convey seepage away from the downstream toe in a controlled and collected manner. This is typically done by placing one or more zones of a graded filter in the embankment that prevents the migration of soils from one zone of the embankment into another zone, or around a pipe that is intended to collect and covey seepage away from the dam.

If the filter system is constructed incorrectly, becomes damaged by settlement or earthquake-induced deformations, erosion of one zone into adjacent downstream zones can lead to a potentially catastrophic consequence. Evidence of this erosion may be difficult to detect until piping of soil is observed, subsidence is noted, a sinkhole appears above the area of erosion, or the embankment dam fails catastrophically.

Embankment materials are often placed directly upon an alluvial soil foundation soils, although this is not as common in current practice in areas with liquefaction concerns. Since dams are typically constructed across existing stream channels, the foundation soils can range from clean sands and gravels to well-graded soils with clays.

Modern design standards require a filter analysis between the foundation and the soils placed in direct contact with the foundation to ensure filter compatibility between the soil types. The analysis must evaluate the anticipated direction of flow whether it would be from the embankment into the foundation, or from the foundation into the embankment. This will dictate design criteria.

In older dams, small containment ponds or dams, or in long canals and levees, it may be typical that no filter is placed on the foundation below the embankment or in the embankment as a result of the size of the structure and the costs associated with constructing the filter system. Depending upon the development of the internal phreatic surface, the lack of a foundation filter may not be an issue.

Typically, the gradient is downward from the reservoir through the embankment and into the foundation. However, it is not uncommon for upward seepage to develop from the foundation into the embankment or just downstream of the toe of the embankment. This can result in the development of a sand boil and a significant loss of effective stresses with an increased potential for piping to occur.

Another area of concern is where earth and rock fill dams are placed against a steep abutment, narrow trench, in and over a solution feature, or over a conduit with soil backfill or a portion of the conduit that extends above the foundation. In these instances, differential settlement can result in the development of transverse cracks or arching of the soil which results in a significant reduction in vertical and lateral stress.

When the lateral stress drops below the seepage pressure, hydraulic fracturing can occur and a transverse crack can form in the cohesive embankment material or clay core. If this crack conveys unfiltered water or causes damage to the filter, internal erosion may commence. Wherever a discontinuity is encountered, a potential flaw can exist in the embankment. This flaw could allow hydraulic fracturing to occur. Embankment loads will result in arching effect from the low modulus material to the high modulus material. No matter how well compacted, the modulus of soil fill will always be much lower than rock or concrete, resulting in a reduction in vertical and lateral stress in the soil embankment. This could allow for internal erosion and piping commencing through this area.

2.9.2 Penetrations through Dams

Embankment dams have historically been constructed with outlet conduits passing through the embankment. Seepage collar construction was one common method used to lengthen the seepage path along the conduits and reduce the potential for internal erosion and piping. The lesson learned from problems with the historical use of seepage collars is that because it is difficult to adequately compact the soil adjacent to the collars, internal erosion and piping is actually more likely.

As compression of this poorly compacted soil occurs in contrast to the rigidity of the conduit and collar, the stress field in the soil arches onto the conduit and the collar, which reduces the minor principal stress and facilitates the development of hydro fracturing at the interface increasing the potential for internal erosion. With the well compacted soil above the conduit, and the conduit adjacent to the seepage path, an erosion feature can easily form along the length of the conduit resulting in the failure of the embankment.

Contrary to the design intent, the use of seepage collars actually reduces the safety of a dam from seepage induced internal erosion. Current, modern design standards for conduits through embankments typically require the conduit to be encased in concrete with battered sides to allow for good compaction. In addition, filter diaphragms are also used to prevent internal erosion pathways along the conduit and dam or foundation fill contacts.

This same concern exists with any penetrating structure through an embankment dam. Any vertical wall, retaining wall counter forts, or other artificial penetration through an embankment must be closely evaluated and discussed when discussing the potential for internal erosion and piping.

2. 9.3 Concrete Dams

While concrete dams are not generally subject to internal erosion, the foundation may be susceptible if the dam is constructed on soil. Foundation internal erosion through the foundation may result in a significant change in uplift pressures that may affect the static stability of the structure. The concrete dam can easily form a roof over the erosion feature that may not collapse, but allows the erosion to continue unbated.

Seepage gradients within the soil in contact with the base of the concrete dam may also increase. The erosion may not be evident until there is discolored water noted at the toe of the dam or there is a significant change in uplift pressure, which would be difficult to comprehensively monitor. The change in uplift pressure may be detected in serendipitously located piezometers, or is more often indicated by a significant change in measured seepage at the toe of the dam.

When a concrete dam abuts an earth fill embankment, differential settlement and arching of the soil stress field may occur. The interface between the concrete and the earth embankment is similar to the interface between an earth embankment and a steep rock abutment or along a penetration through an embankment. This allows for a higher potential for internal erosion and piping to occur along this interface.

2.9.4 Spillways

Spillways founded on soil foundations can be particularly susceptible to internal erosion and piping. If not constructed with care on soil foundations, spillway chutes can easily crack, settle or otherwise experience movement that could create some type of an offset. Many spillways are also designed with weep or drain holes to prevent or mitigate excess pore pressure accumulation behind the slab or walls. If these are not properly designed to filter soils behind the spillway or become plugged and are not maintained, pore pressures can build up beneath the spillway leading to a greater risk of preferential flow path development. Erosion of soil and/or rock at the toe of the spillway may also be caused by subsurface discharge flows. If this continues undetected (toe of spillway is always submerged), this could result in an increased seepage gradient and greater potential for piping.

There are two primary concerns for spillways on soil:, slab-jacking and cavitation pressures. With gated structures, the spillway inlet is typically below water during many times of the year; this includes intake walls that typically become the spillway chute walls. As discussed above in penetrations through embankments, this situation raises concerns that should be evaluated as well. This provides the opportunity for seepage to develop and collect below the slab. Any open crack or drain hole provides a possible unfiltered exit.

A second and perhaps more significant concern with spillways is erosion or scour potential. Although not actually related to internal erosion, the magnitude and direction (upstream higher than downstream side of crack, or vice versa) of the offset, creates a potential for slab jacking or cavitation of the concrete slab. If a portion of the slab is lost while the spillway is operating, scour of the soil foundation of the spillway is inevitable and could result in an uncontrolled release of the reservoir. This makes it crucial to closely monitor the behavior of the spillway slab and evaluate the structure under extreme conditions.

2.9.5 Ancillary Structures

Ancillary structures associated with dams may include intake structures, retaining structures, power houses, guide walls, etc. These structures may be susceptible to internal erosion when a seepage gradient is present along the side or below the structure. These solid structures can easily form a roof for the development of pipe along the structure. If there is no filter to control the erosion of fine grained materials at material interfaces, the embankment could fail along one of these structures.

2.9.6 Locks and Dams

Locks and dams are subject to erosion hazards similar to these described above for ancillary structures. Unlike dams, the largest gradient occurs when the head differential between the upper and lower pool is the greatest and the embankment acts like a typical embankment dam. During high discharge or when the gates are being operated, the gradient across the structure may be low. With the fill and discharge of locks during operation, there are other considerations to take into account when discussion internal erosion. Any repeated charging and discharge of soil raises a concern of excessive movement of material with each cycle.

The operation of a lock is essentially the same as performing cycles of rapid fill and drawdown of a reservoir. Embankments are typically designed for such an operation by providing riprap and riprap bedding, and in some instances, an upstream filter when it is known that the reservoir will go through multiple fill and drain cycles. As the reservoir is drained, bank storage in the soil will drain and seepage forces can increase significantly, resulting in a higher likelihood of soil transportation. After many cycles of this type of reservoir operation, it is possible to erode a sufficient amount of soil from the embankment that could either result in an upstream slope failure, or create a seepage path that could quickly develop a pipe through the embankment and cause the embankment to fail.

2.9.7 CFRD Rock fill Dams

The concrete face rock fill dam, CFRD, had its origin in the mining region of the Sierra Nevada in California in the1850s. Experience up to 1960 using dumped rock fill, demonstrated the CFRD to be a safe and economical type of dam, but subject to concrete face damage and leakage caused by the high compressibility of the segregated dumped rock fill. As a result, the CFRD became unpopular. (**Z. Erkay Kemaloğlu, et.,al 2016)**

With the advent of vibratory-roller-compacted rock fill in the 1950s, the development of the CFRD resumed. Although design is largely based on precedent, there has been continuous progress in design aspects and in construction methods (ICOLD, Nov.2014). In recent years, concrete face rock fill dams have been a major dam type as a result of their good performance and low cost compared to earth fill dams. However design of CFRDs is still based on experiences and engineering judgment (**Z. Erkay Kemaloğlu, et.,al 2016)**

To define and check the performance of the dam throughout of life time and estimate the performance of the dam against an extreme event, such as an earthquake or a flood, several analyses like static and dynamic stability, settlement and displacement, and seepage are suggested. **(Z. Erkay Kemaloğlu, et.,al 2016)**

Leakage is a key parameter that relates to the overall performance of the CFRD. Large leakage rates are an indication that damage has occurred to the perimeter joint and/or that the concrete face has cracked to some extent. Seepage through the foundation may also be a contributing factor to large leakage rates **(Z. Erkay Kemaloğlu, et.,al 2016)**

The purpose of the seepage analysis is to define the pore-water pressures in the dam foundation system needed for the slope stability and stress/displacement calculations. The seepage analyses were performed by means of the SEEP/W module of the GeoStudio Finite Element Method geotechnical software package.

The seepage analyses have been carried out for the cases of different water levels. Permeability values for foundation rock and dam body rock fill are necessary. For stability purposes and to decrease the seepage through dam's foundation and body, a vertical cut-off wall can be built. **(Z. Erkay Kemaloğlu, et.,al 2016)**

Chapter Three - Materials & Methodology

3.1 Description of the Research Area

The research area is located in Bale zone of Oromiya regional state. Weyib River has a catchment area of 4480.36SqKm and 3670.16 Sq.Km at Sofumar and Alemkerem river gauge stations respectively. Topography of the area is characterized by steep to flat slope with maximum and minimum elevation of 2200m a.s.l and 1200m a.s.l at Alemkerem and Sofumar respectively.

The climate of the area is characterized by midland with bimodal rainfall pattern. The average temperature ranges from 28^{o}C to 40^{o}C and the annual rainfall also ranges in between 650mm to 1250mm.. The observed flow measured in the range of 0.79m^{3}/sec and 127.56 m^{3}/sec. The research area is located in between the two gauge stations that can be representative for karstic featured catchments with similar physiographic and climatologically conditions.

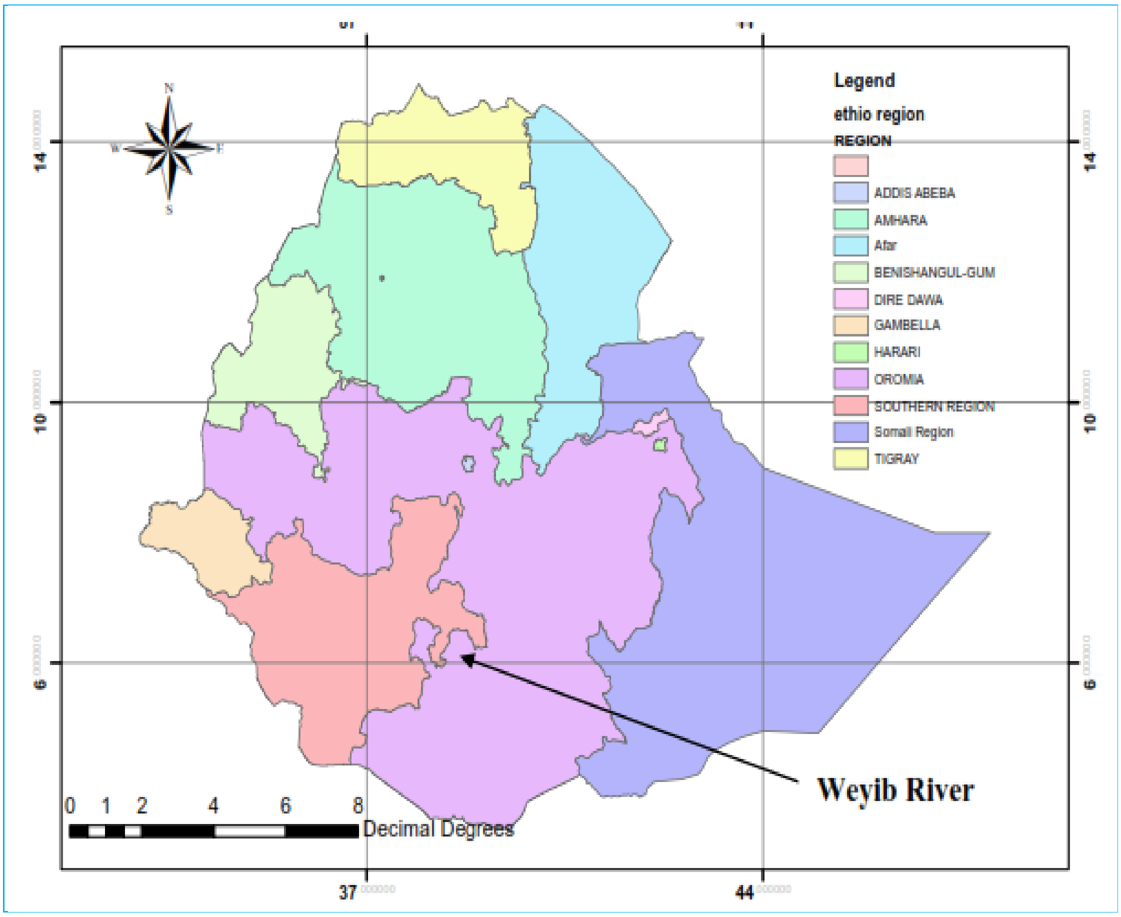
3.2 Material & Data

Data, documents and materials collection for the research was carried out from different offices and respective organizations. Some of them are as presented in the table below.

Table 4 Material and used data

Item No	Data and Material	Number of gauge stations	Data Type	Observation Period	Sourse
1	Stream flow	2	Daily	1992-2004	MOI&E
2	Rainfall	5	Average Monthly	1992-2004	MOI&E
3	Master Plan Study				MOI&E
4	Research Documents				Published
5	Global mapper software				Freely available
6	Hec-GeoHMS 2012				Freely available
7	Geo-Studio, 2012				One month trial
8	XL Stat,2017				One month trial
9	USGS Programme Separation Progarmme				Freely available
10	Excel Hydrograph Separation Progarmme				Freely available

Figure 9 Location Map of the River



3.2.2 Data Analysis

3.2.2.1 Observed Data

Data obtained were hydrological and monthly rainfall of 13 (thirteen) years and above observed and measured records. Data of two different gauging stations with daily stream flow and monthly mean rain fall of five rainfall stations were obtained.

Table 5 Stations and record periods

Item No	Data	Number of gauge stations	Data Type	Observation Period
1	Stream flow	2	Daily	1992-2004
2	Climate			
2.1	Rainfall	5	Average Monthly	1992-2004
2.2	Tempreture	5	Average Monthly	1992-2004

3.2.2.2 Base stations

Ginir rain fall station and Sofumar river gauge station which are found within the catchment were considered as rainfall and stream flow base stations for the estimation of missed data for they have full time observed and measured records and enough data for the required analysis.

Figure 10 Ginir station monthly average rainfall data

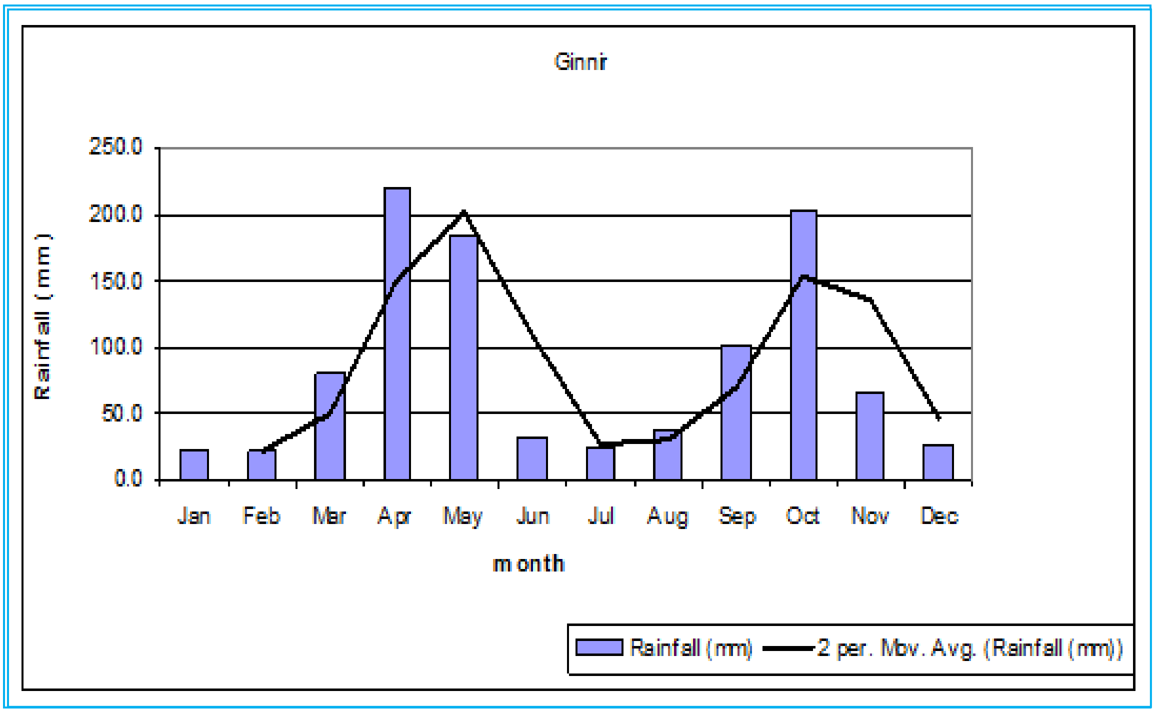


Figure 11 Sofumar gauge station average daily stream flow

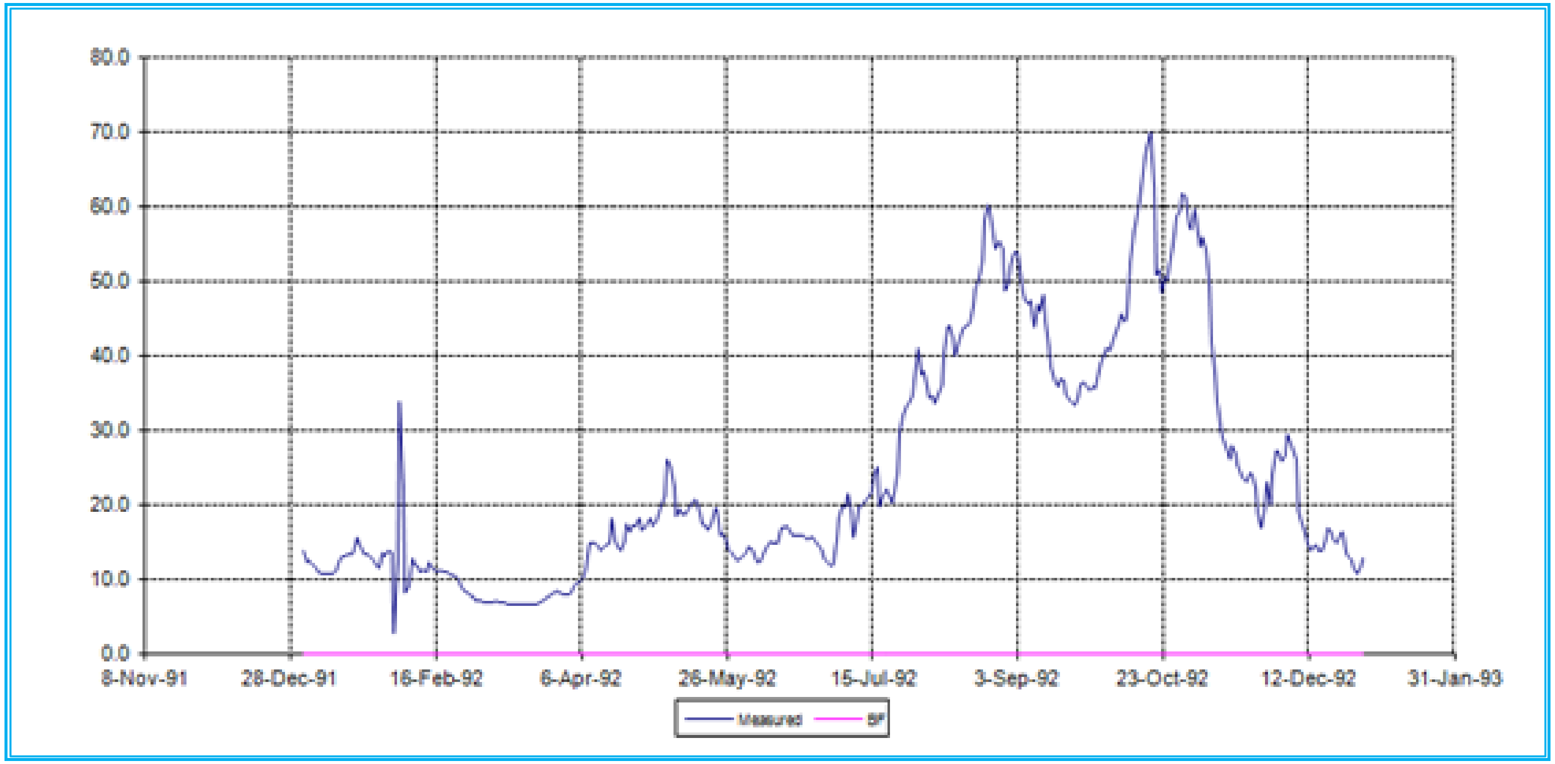


Figure 12 Rainfall gauge stations location within the catchment

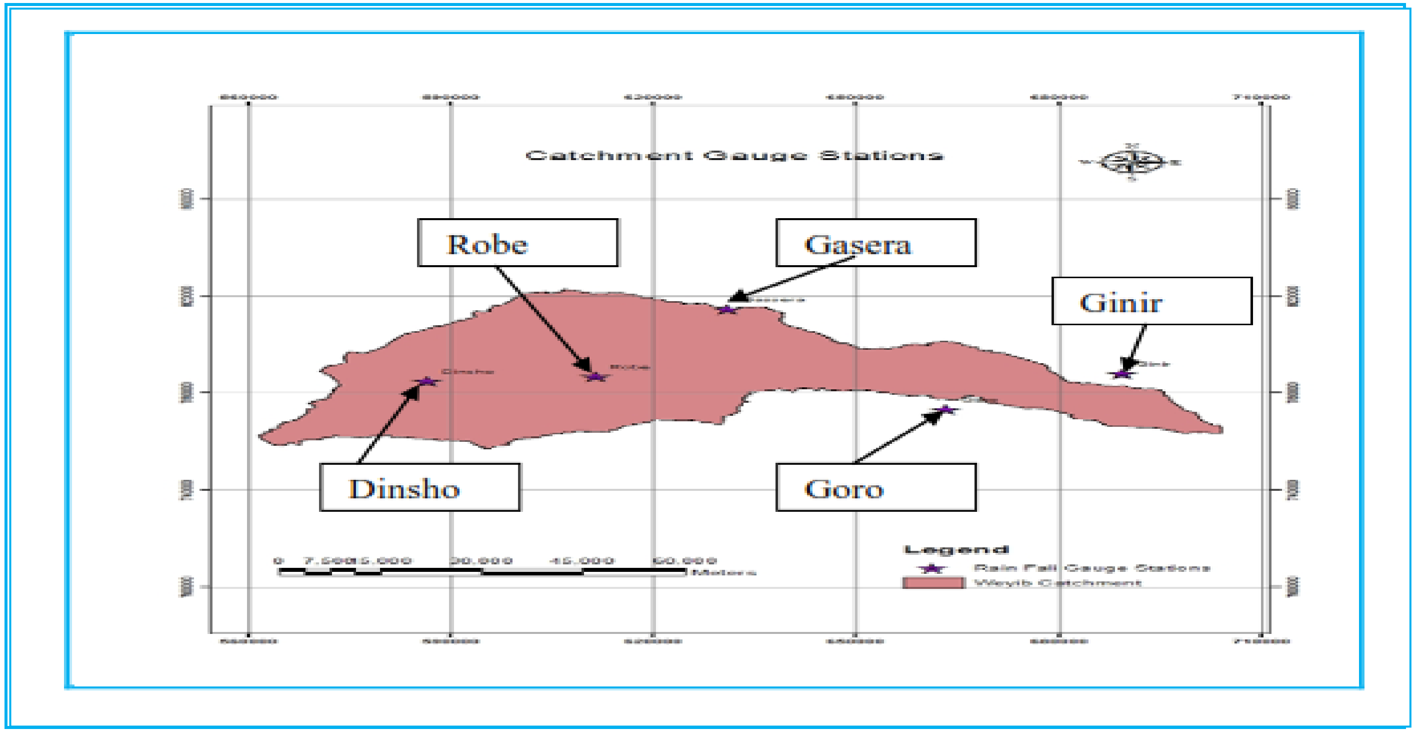
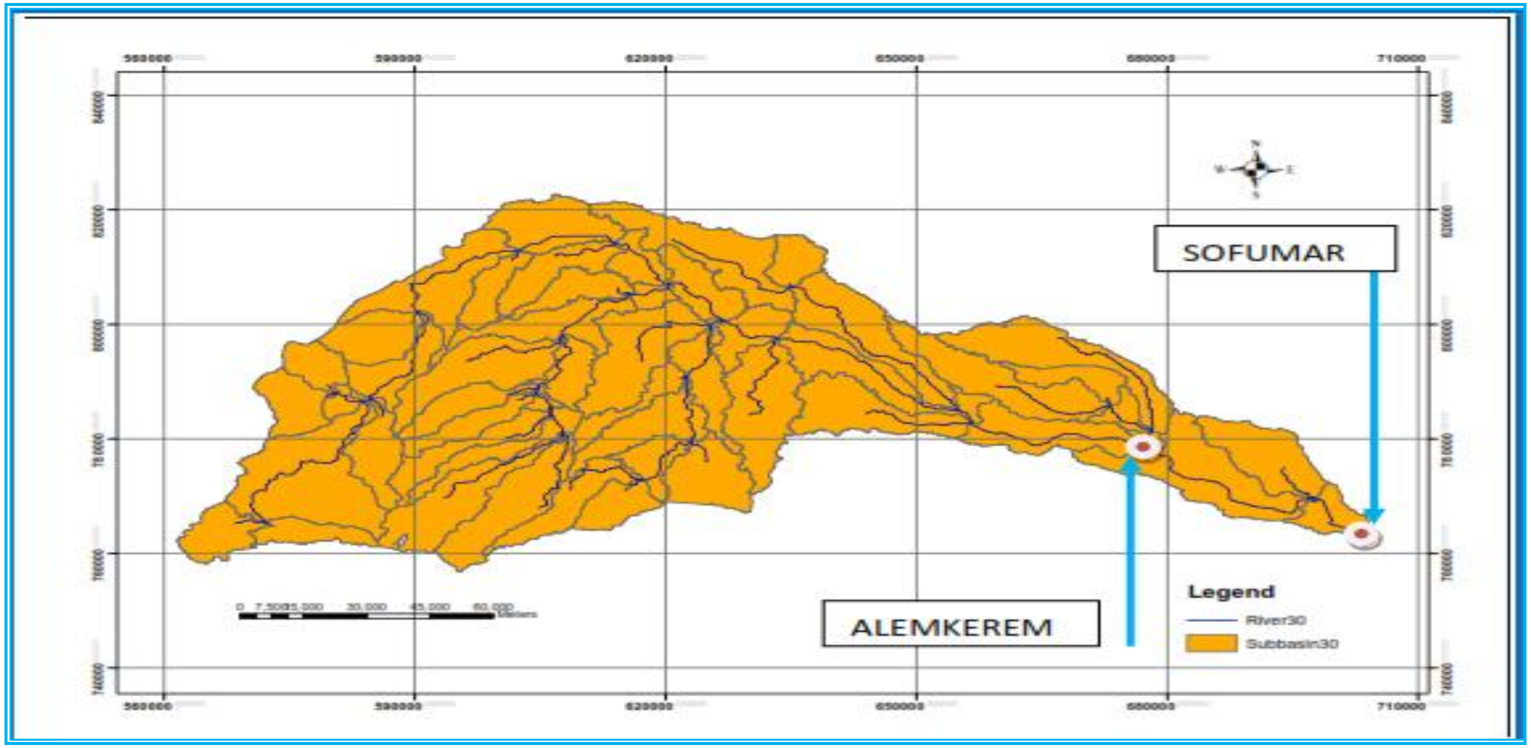


Figure 13 Stream flow gauge stations location within the catchment



3.2.2.3 Missed Data Estimation

Some of the methods used in estimating missed data were normal ratio and Thiesson polygon methods. Estimation was for the other rainfall stations using base station data. After data estimation for each rainfall station was conducted, catchment representative rain fall data was estimated by Thiesson polygon.

3.2.2.3.1. Normal ratio method

The most common method used to estimate missed rainfall data is Normal ratio method and the method is based on past observation of the nearest gauge station. This method was appropriate for a catchment with different climatological and physiographic features.

The formula used in the analysis can be expressed as follows:

$$Px = \frac{1}{m} * \sum \frac{Nx}{Ni} * P_I$$

Where, Px= Estimate of monthly rainfall for the un-gauged station

P_i = Monthly rainfall values of observed gauge station

Nx= Annual average value of observation gauge

N_i = Average annual rainfall values of missed data gauge

P_x= Monthly rainfall values of missed data gauge station

m = Number of gauge stations

3.2.2.3.2 Thiessen Polygon Method

Thiessen polygon method was carried out using GIS and the stations coordinate as an input for the preparation of the polygon. Estimation of representative raifall values of the basin was obtained as an arithmetic mean of multiplication of respective polygon area of each station with rainfall of each station

Formula expression:

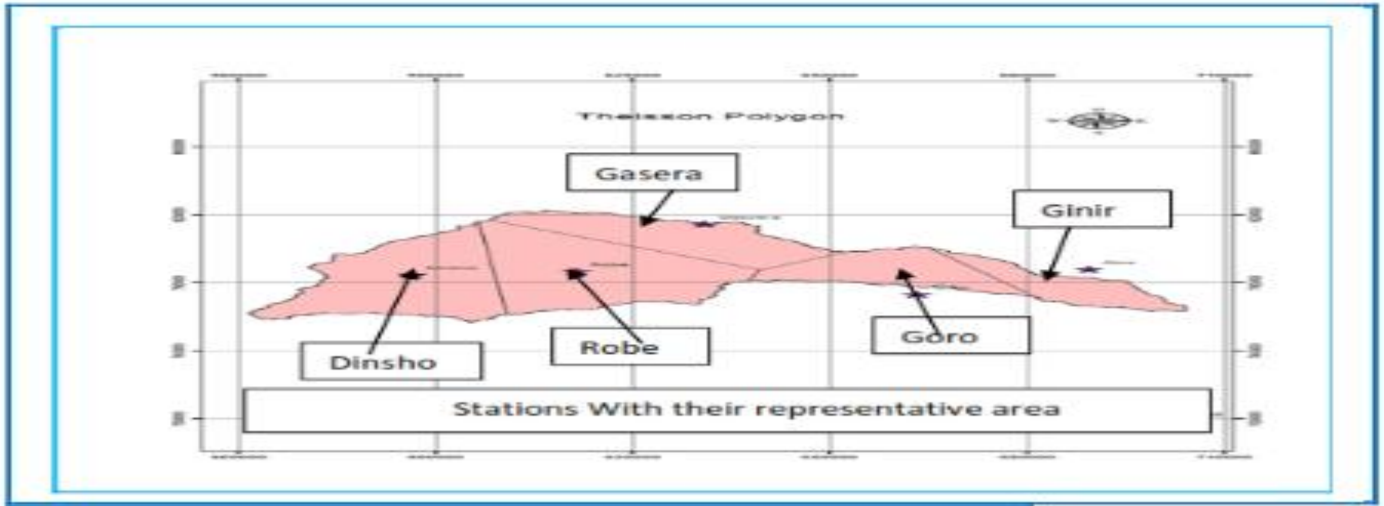
$$\mathbf{Px} = \frac{\sum \mathbf{Ai*Pi}}{\sum \mathbf{Ai}}$$

Where, Px= estimate of monthly rainfall for the basin

Pi = Monthly rainfall values

Ai = Area of respective gauge stations

Figure 14 Thiesson Polygon and gauge stations with respective areas



Ser No	Name	Area (Sqm)	Area (SKm)
1	Goro	600691460.90	600.69
2	Dinsho	1277524624.00	1277.52
3	Robe	1375713145.00	1375.71
4	Gasera	799710777.90	799.71
5	Ginir	426721026.30	426.72
Total Area		4480.361034	

3.2.2.3.3 Area ratio method

Catchment area to stream flow ratio was used in estimating missed data for Alemkerem station. Emperical formula was used for estimation which was developed by Dr. Admasu Gebeyehu.

Formula expression:

$$(Ab|Am)^{0.7} = \frac{Qm}{Qi}$$

Where, Qm = Missed stream flow

Qb = Base stream flow

Am = Missed stream flow area

Ab = Base stream flow area

The results of estimation were presented as follows.

Figure 15 Goro station rainfall data

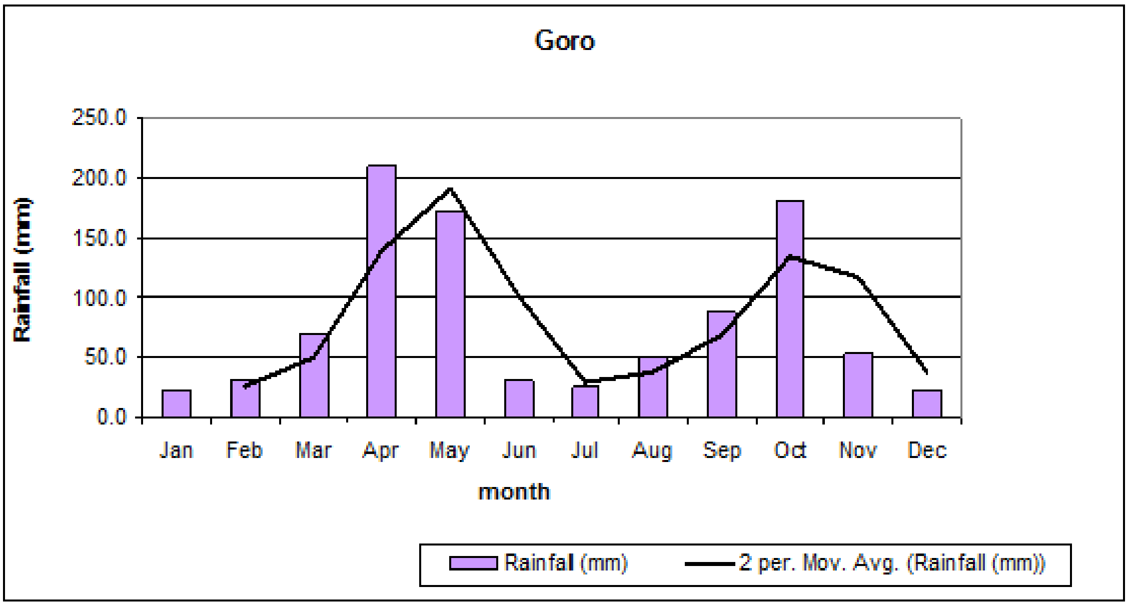


Figure 16 Gasera station rainfall data

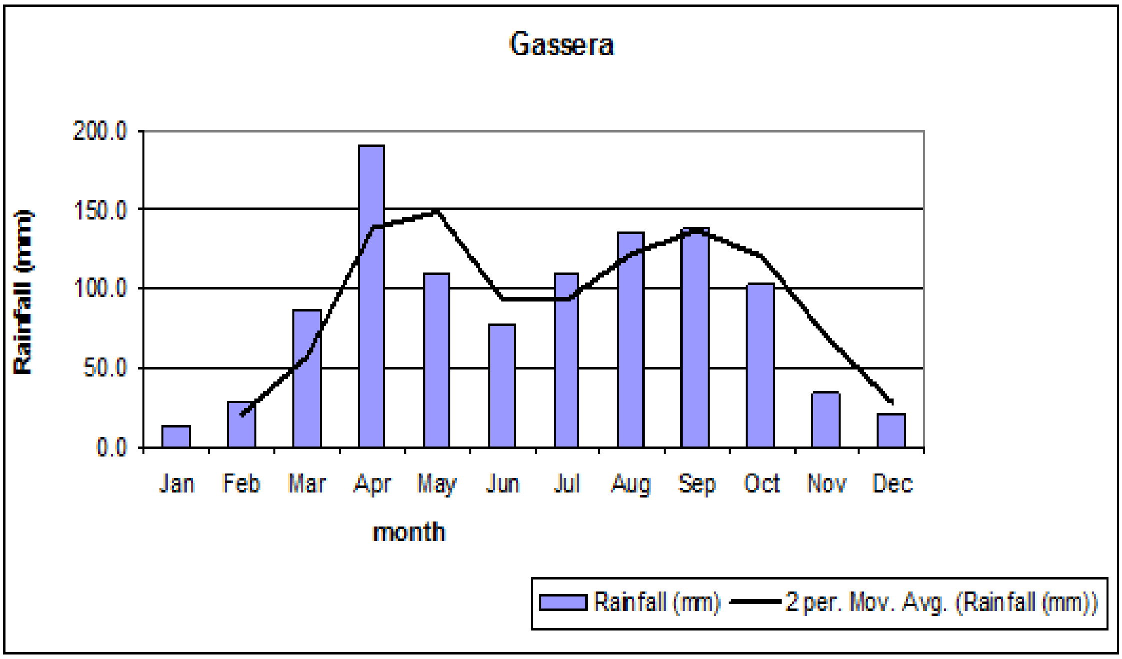


Figure 17 Robe station rainfall data

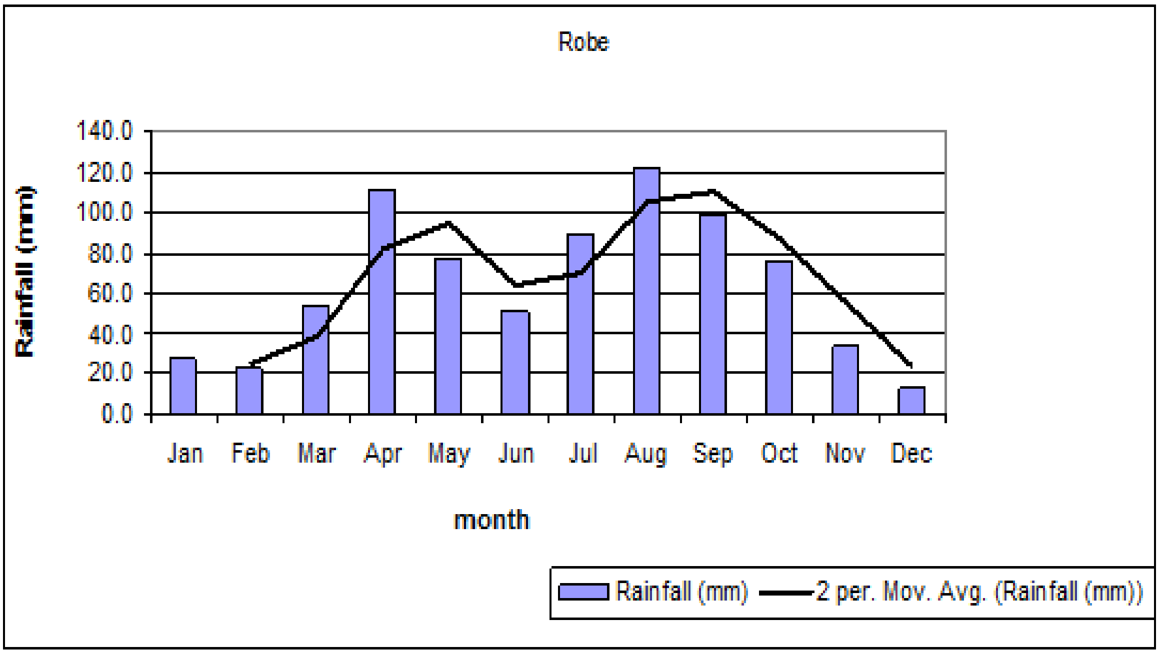
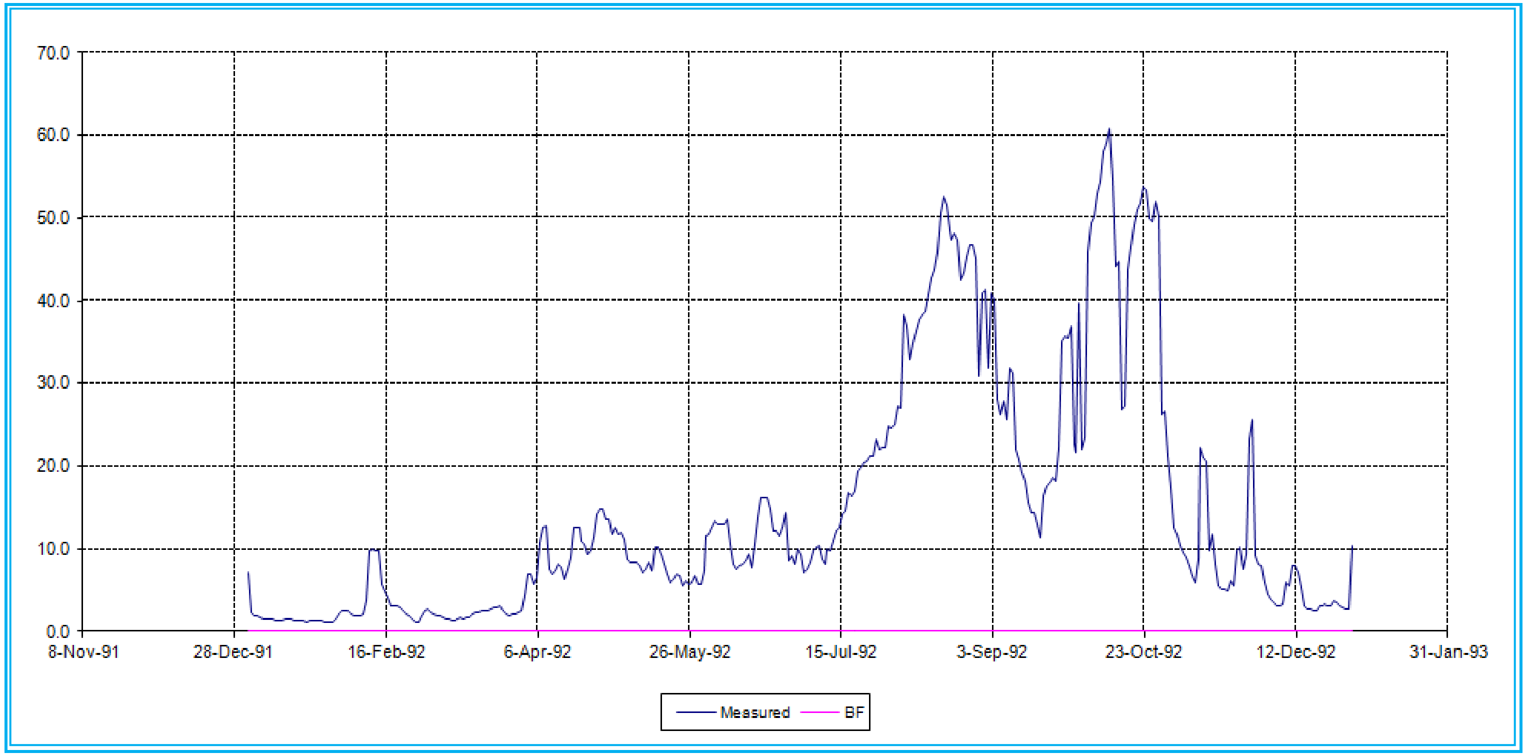


Figure 18 Alemkerem gauge station average daily stream flow



3.2.2.2.4 Estimated catchment representative rain fall

Estimate obtained by each method is compared and the suitability is determined based on how close are the values in a given time series**.** In the process of estimation, the centroid of the catchment is found nearer to Gasera station and it is used for comparisons with basin representative data. Whereas, Ginir station is used in estimating missed data of other stations since it has full observed data values for the selected periods and located near to gauge stations.

In estimating the basin values, the following steps are followed.

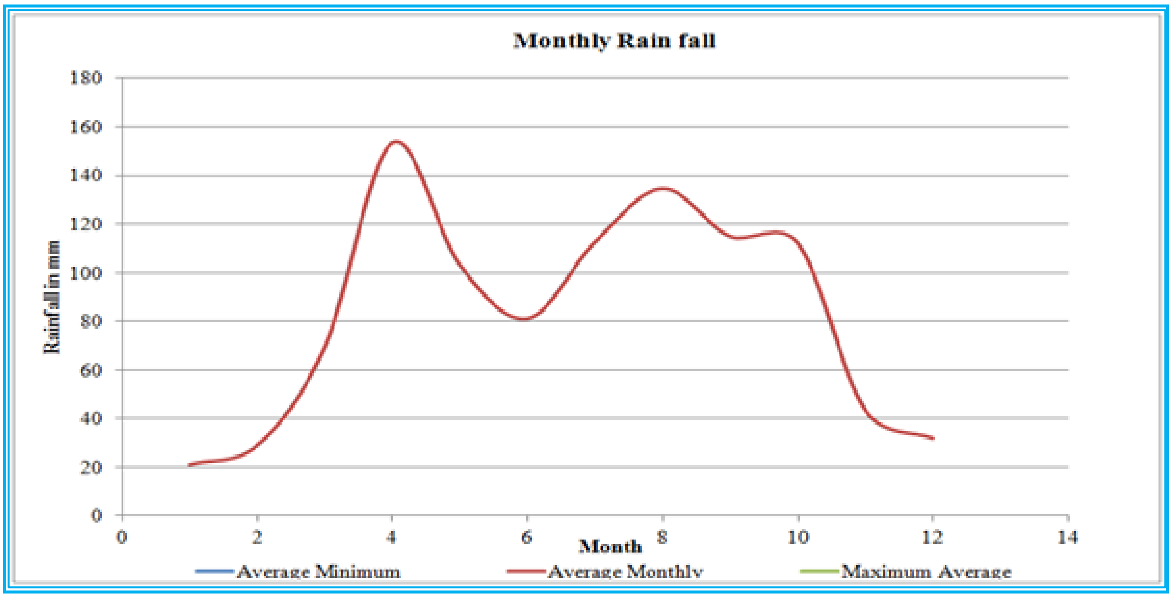
First: Missed values of each station are estimated by normal ratio method

Second: After missed values estimation, a basin representative value of the catchment is computed using Thiessen polygon method.

Third: Data estimation is checked by comparing computed basin representative data values with the nearest station to the basin centroid.

Fourth: The record durations of stream flows coincided with the rain fall to make the analysis easy.

Figure 19 Basin Representative rain fall data



3.2.2.4 Data Quality Analysis

Missed data estimation of each station was carried out but the estimated data quality check was done only for basin representative rain fall and stream flow data. Basin representative data value was obtained by Thiessen polygon method and it was checked with Gasera rain fall measuring station for it is located near to basin centroid as shown in the figures below. Stream flow data were checked also for time series and regression analysis

Figure 20 Linear regression analysis using XLSTAT, 2017 for rainfall data

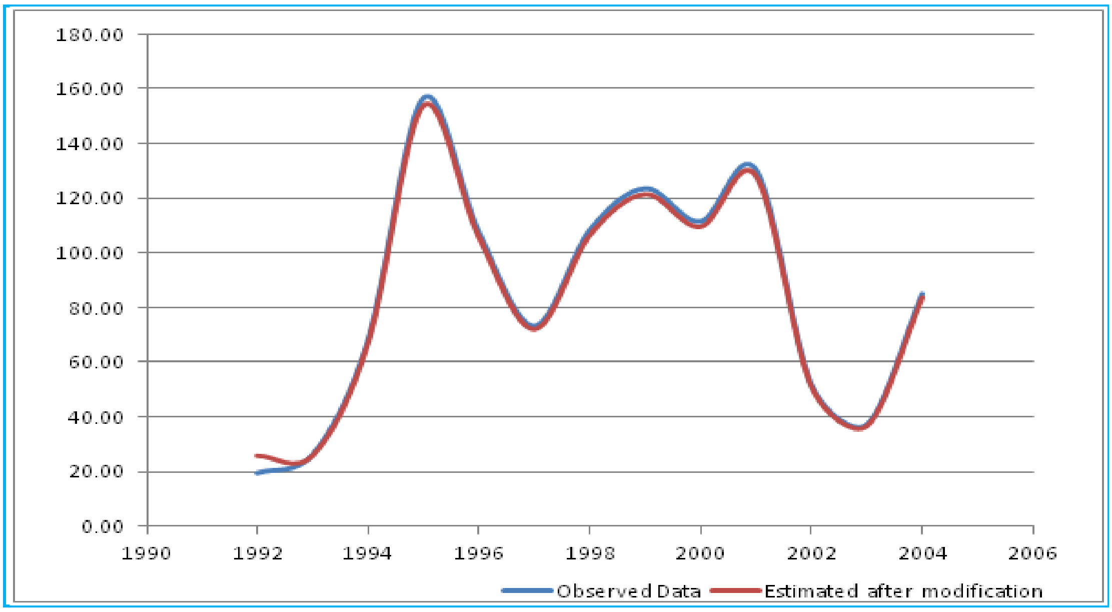


Figure 21 Mass curve of observed and estimated rain fall data

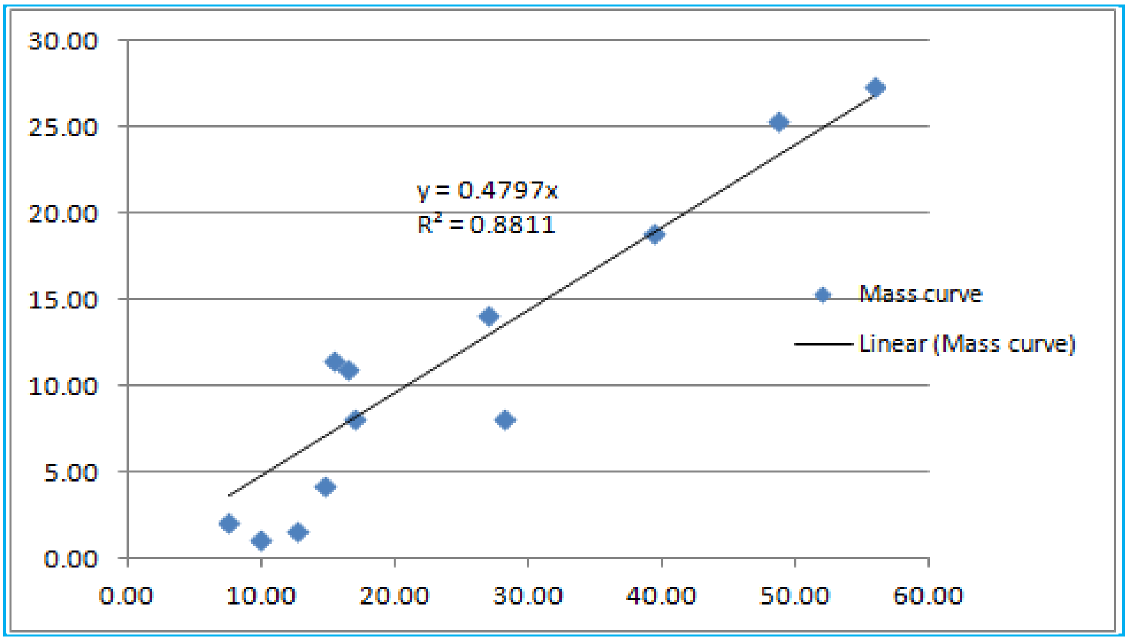
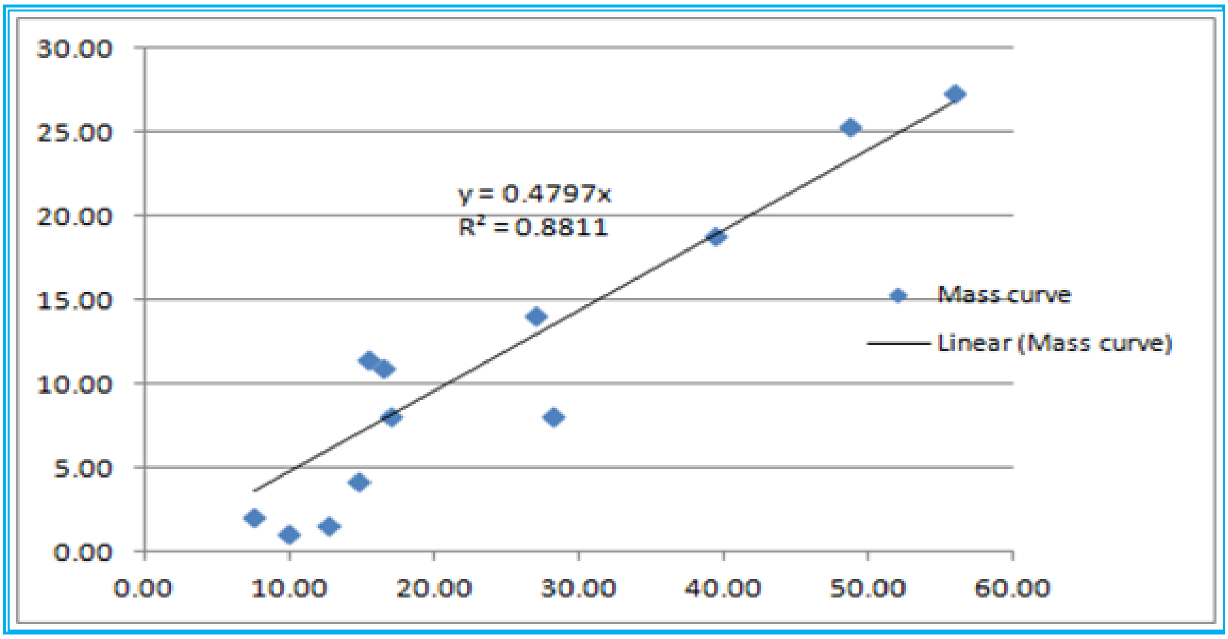


Figure 22 Time series representation of stream flow of Sofumar and Alemkerem

Figure 23 Regression analysis of stream flow of Sofumar and Alemkerem



From the above time series and regression analysis, consistency of observed and estimated data found satisfactory. This comparison was done again by using XLSTAT, 2017 software trial version.

3.3 Methodology

3.3.1 Estimating Base Flow Difference

3.3.1.1 Stream Flow Separation Process

Observed and measured stream flow data obtained from two stream gauges were used in the analysis. During the analysis, USGS programme software, excel based programme, and excel based algorithm using Chapman’s formula.

3.3.1.1.1 USGS Base flow separation programme software

Thirteen years daily flow data were averaged as yearly average daily data and prepared for the software as an input with required formatted files. The out puts of base flow and stream flows were as shown in the figure below.

3.3.1.1.2 Excel based hydrograph separation

Unless USGS programme, this one was prepared including rainfall data as monthly average input. The base flow response to the rainfall was well addressed in this method of analysis. That is why the basin rainfall estimation is required since stream flow is the accumulation of runoff over the entire catchment and a slow gradual release of ground water to the streams. The out puts of base flow and stream flows are as shown in the figure below

3.3.1.1.3 Excel based flow separation using Chapman's Formula

Unless USGS programme and excel hydrograph flow separation this one was prepared without including rainfall data as monthly average input. The base flow response to the rainfall was well addressed in this method of analysis. The out puts of base flow and stream flows are as shown in the figure below.

3.3.1.1.4 Comparison of flow separation methods

As per the Anlysis conducted for the three alternative methods of hydrograph separation, USGS is selected for it has minimum standard deviation for annual base flow values.

Table 6 Comparison of base flow separation methods (Simulated and Analytic)

Ser No.	Hydrograph Separation Method	Maximum Stream Flow Cumec	Average Stream Flow Cumec	Average Baseflow Flow Cumec	Standard Deviation value of Base flow	Remark
I	Sofumar					
1	USGS	69.98	24.52	18.49	10.50	Selected
2	Chapman Algorithm	69.98	24.52	20.05	16.0	
3	VBI Excel	69.98	24.52	18.64	10.47	
II	Alemkerem					
1	USGS	60.86	14.79	8.67	7.00	Selected
2	Chapman Algorithm	60.86	14.79	11.7	12.13	
3	VBI Excel	60.86	14.79	8.67	7.04	
III	Flow Difference For Comparison and Computation	9.12	9.82	8.35	9.97	

Figure 24 Stream flow and base flow at Sofumar (USGS)

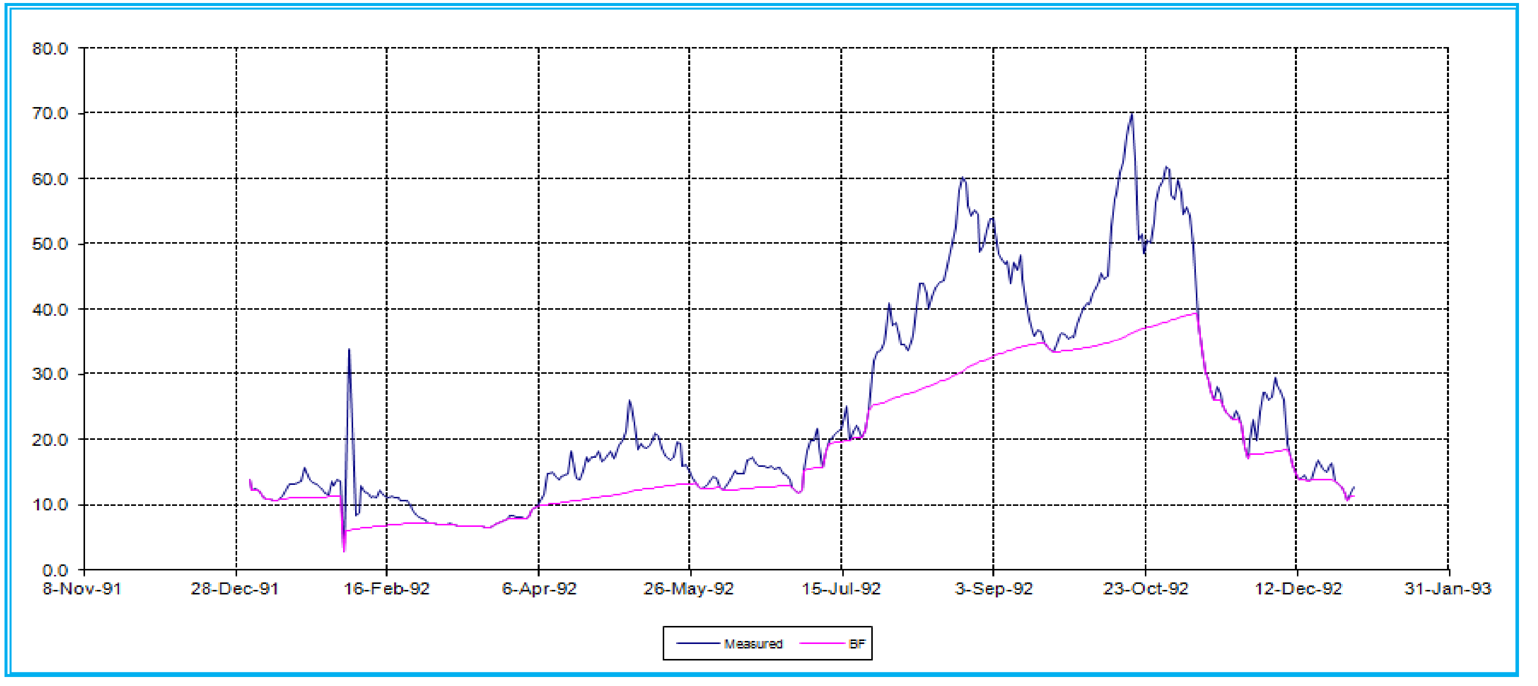


Figure 25 Stream flow and base flow separation at Alemkerem (USGS)

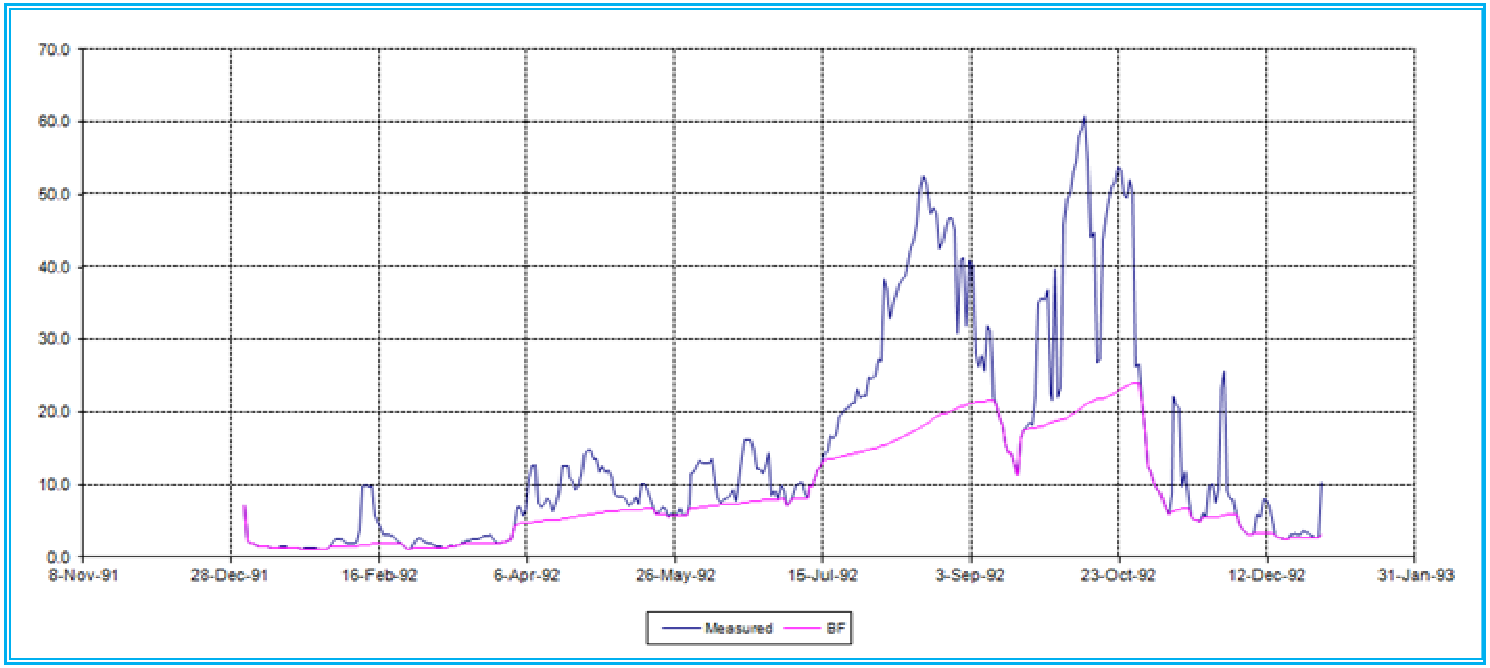


Figure 26 Stream flow and base flow at Sofumar (Excel based hydrograph Separation)

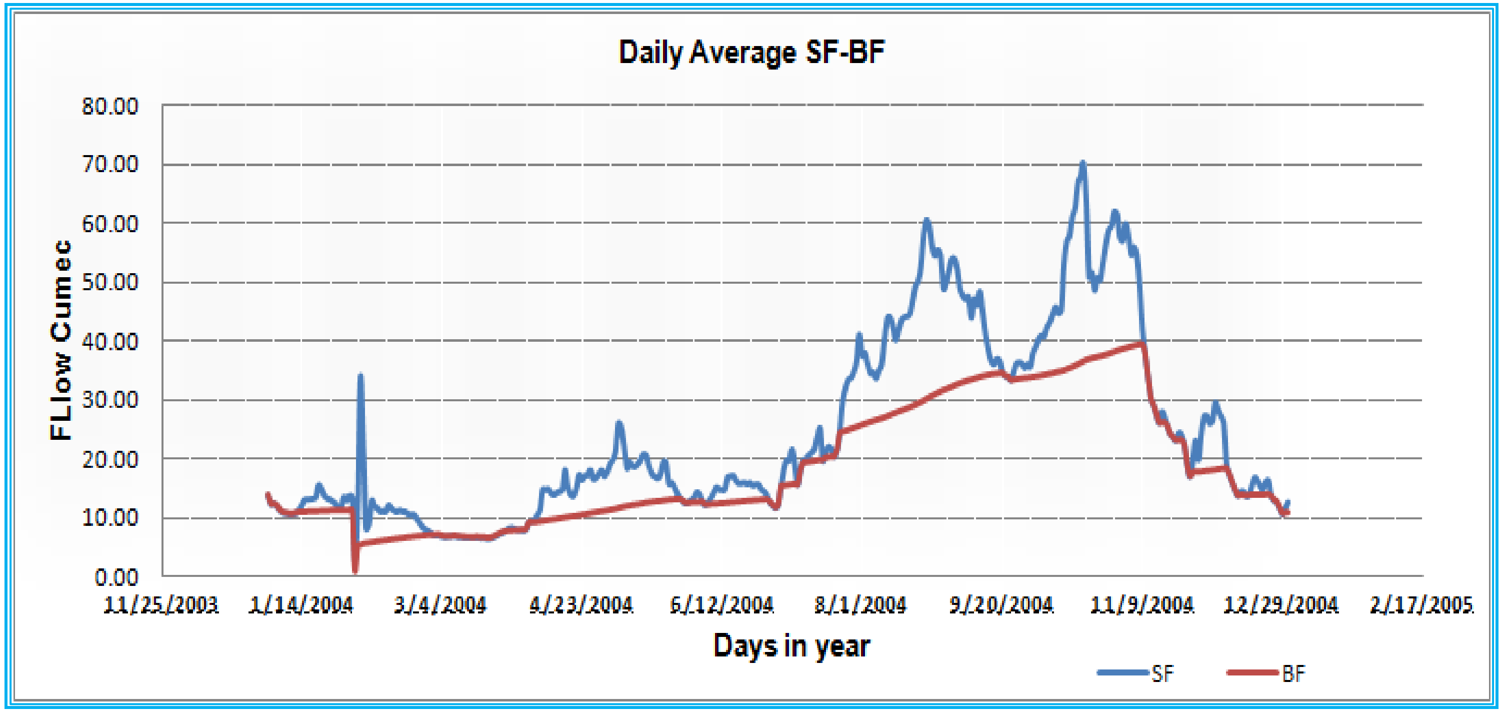


Figure 27 Stream flow data and base flow separation at Alemkerem (Excel based hydrograph Separation)

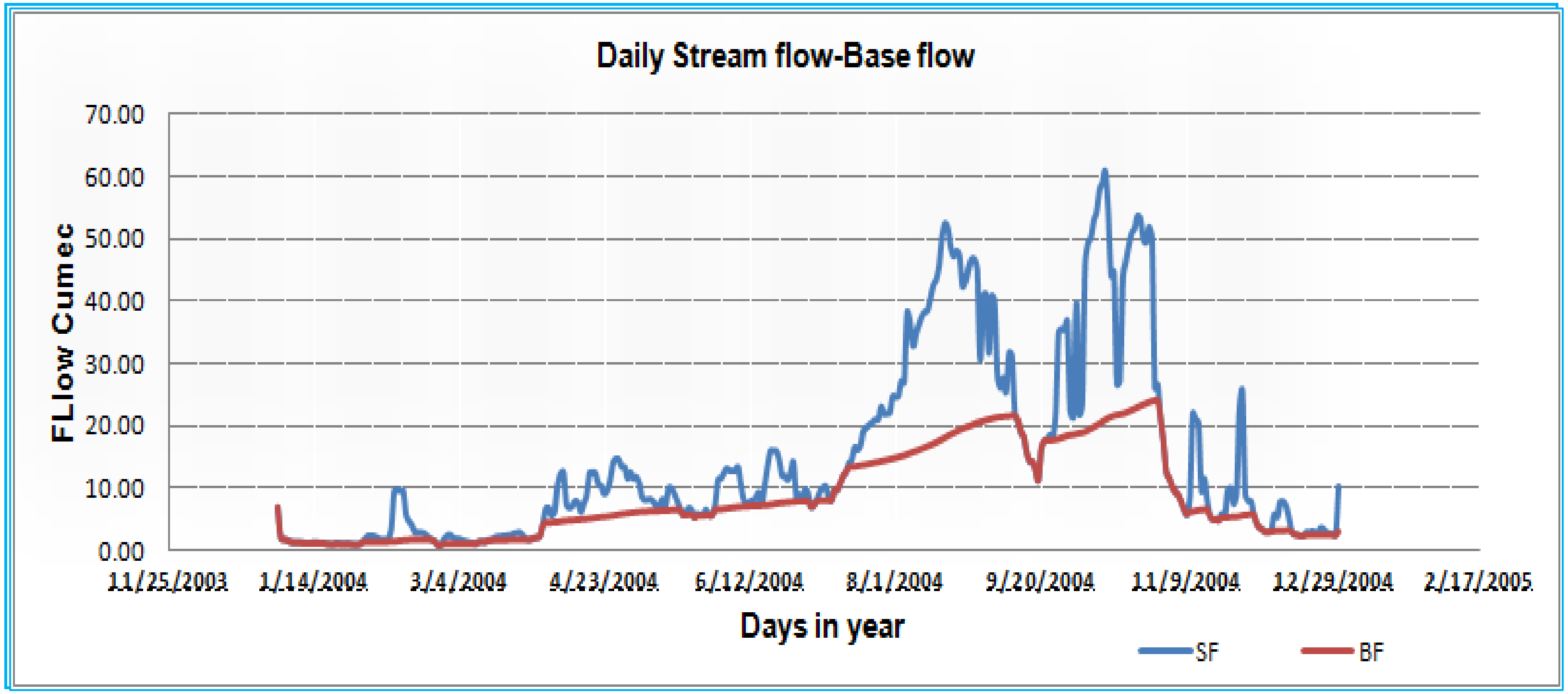


Figure 28 **Stream flow data and base flow separation at Alemkerem (Using Chapman's Formula)**

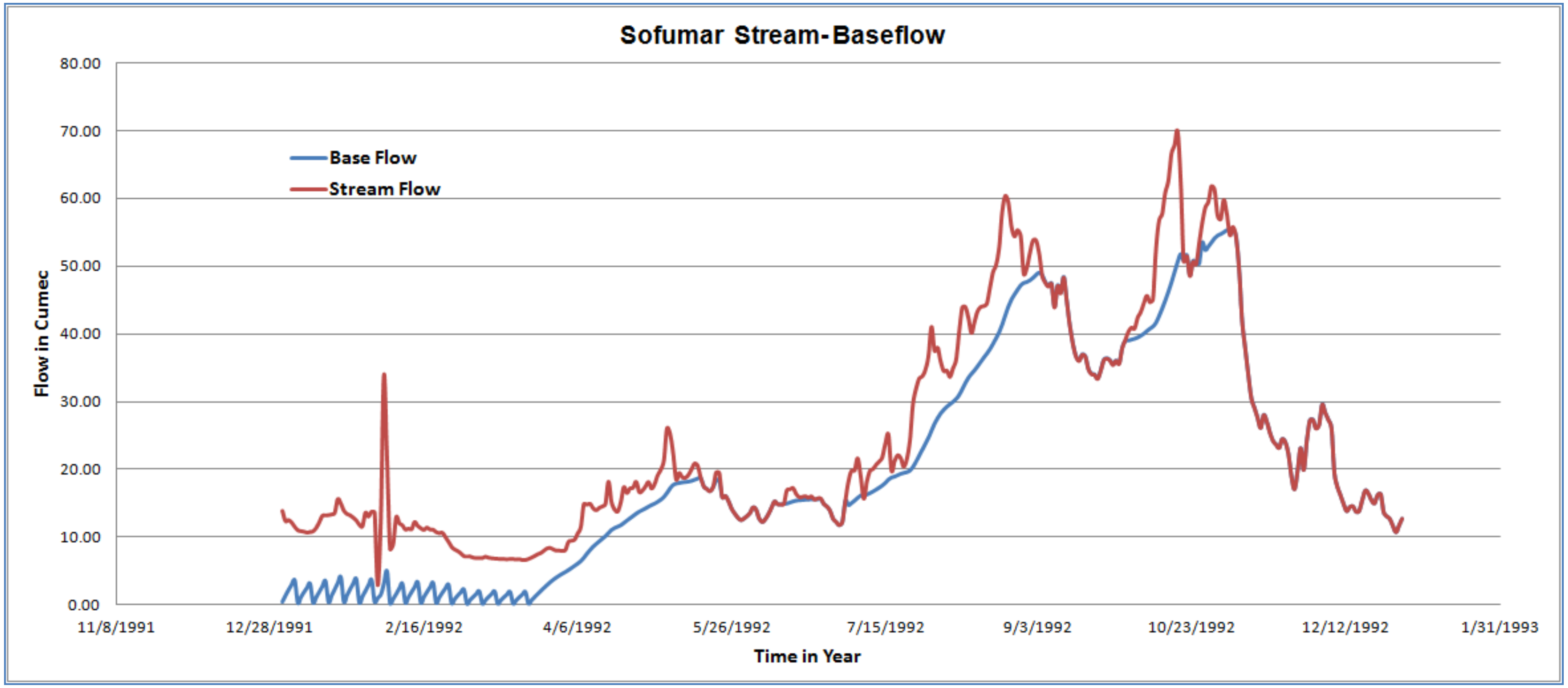
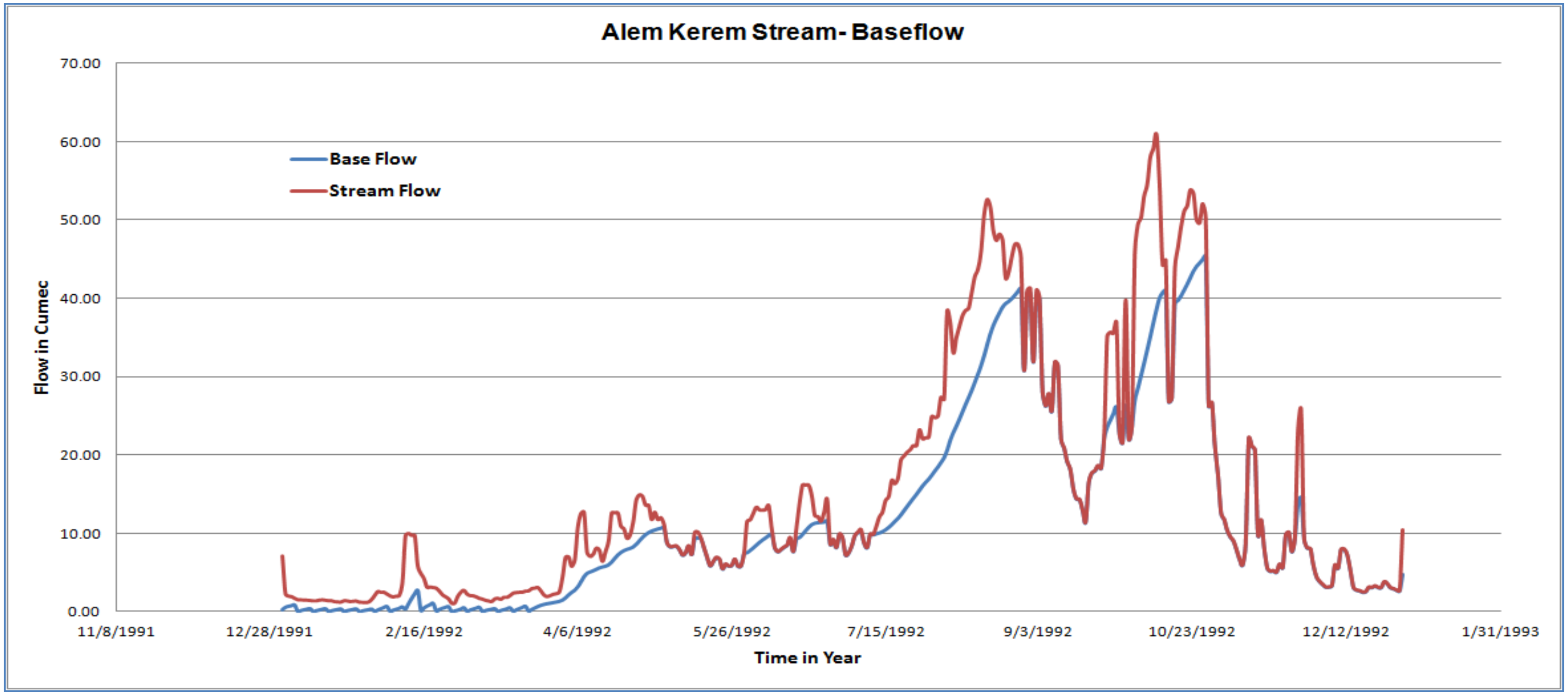


Figure 29 Stream flow data and base flow separation at Alemkerem (Using Chapman's Formula)



3.3.2 Estimation of Average Saturated Hydraulic Conductivities

In estimating the average saturated conductivity, base flow difference of the two stations, average catchment width, river bed elevations, respective catchment area of stations were used as input data. Besides, base flow considered for the two sides of the river were depending on size of their area contribution.

Unconfined well hydraulics formula was used in estimating the required parameters and the formula used is as presented below:

$$Q = \frac{\frac{3.14K(H2^2 - H1^2)}{LnR1}}{R2}$$

Where, K is hydraulic conductivity

H_2 Maximum depth of Aquifer

H_1 well depth

R_1 river width to the center of the well (half length of the river bottom width)

R_2 Maximum width of aquifer to well center (to the river center)

Table 7 Left and right side average saturated hydraulic analysis input data

Item No	Gauge Station	River Length	Difference		Elevation(m)		Average base flow
		m	m	SqKm	Right Side	Left Side	Cumec
1	Sofumar	242580	58260		1540.00	1800.00	18.49
2	Alemkerem	184320		810.20	1490.00	1490.00	8.67

Table 8 Left and right side average saturated hydraulic conductivities.

Description	Radius of river	Radius of aquifer	(H1-H2)^2	Ln(R/r)	Saturated hydaraulic conductivity (K)	Claculated base flow
	m	m			m/sec	Cumec
Right Side	15	1127.62	17500	4.32	0.00015	1.96
Left Side	15	2798.21	189100		6.91788E-05	7.86

Figure 30 Representative model for estimating average saturated hydraulic conductivity analysis

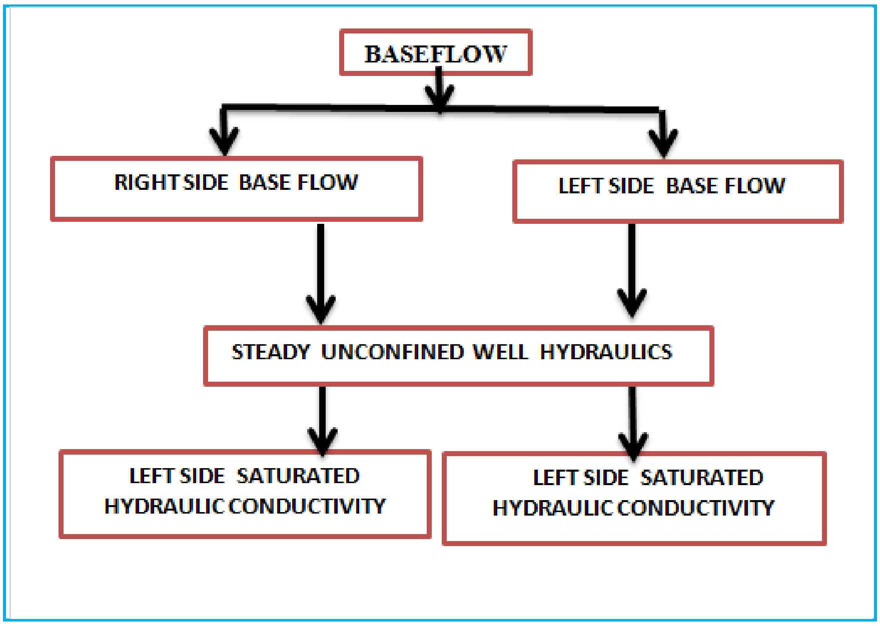
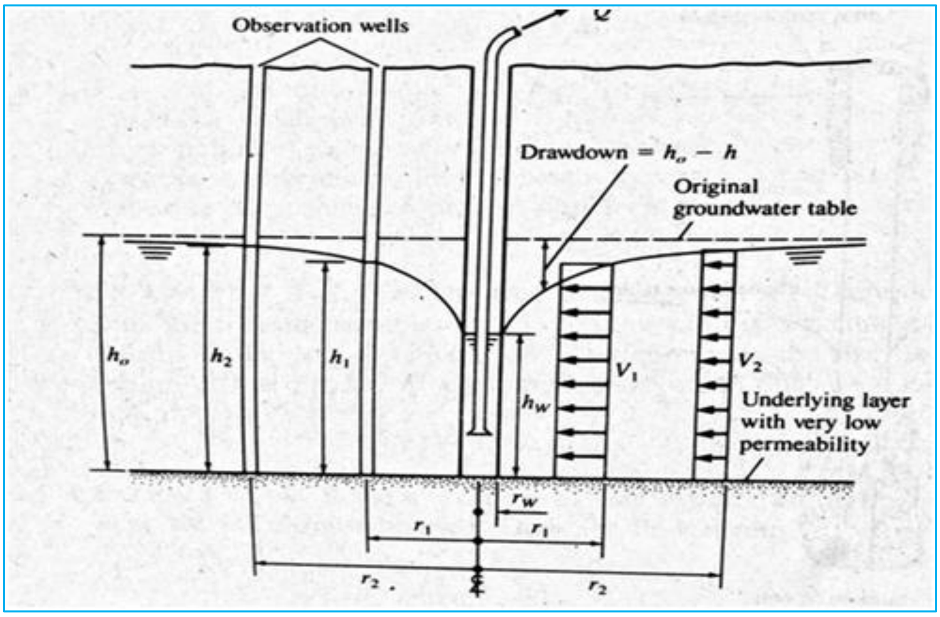


Figure 31efinition Sketch of Model for well hydraulics



(Adapted from Advanced hydraulic engineering Book)

3.3.3 Estimating Karstification Level of the Research Area

The scale of karstification was estimated by using a known empirical formula developed by Le Grand and Las Moreaux. Estimation of average saturated hydraulic conductivity at every 20m depth of the river was carried out by taking the average value at the river bed considering it as the water table level/karstification base level.

The formula expression stated as,

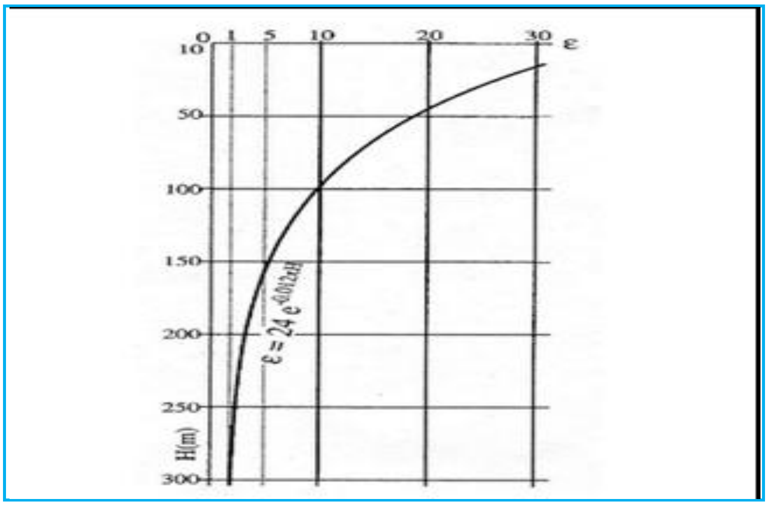
$$Ki = 23.97e^{-0.012h}$$

Where, e =2.72

H is height of aquifer (depth of river)

Ki is height of aquifer (depth of river)

Figure 32 **Graph showing Depth and Karstification Relation (Le Grande & Las Moreaux)**



3.3.4 Estimating Hydraulic and Geologic Settings

Estimation of hydraulic and geologic setting of the research area was done by using the equation mentioned above, the average saturated hydraulic conductivities and comparing the result with the geological data of extensive research carried out by British geologic society under Karst environment **(Fig. 35).**

3.3.4.1 Approach for estimating karstification level and geologic settings

In this research, in order to obtain the required results, the following considerations and assumptions were taken in to account.

- The karstification level of the research area was estimated using the average saturated hydraulic conductivity at river bed level/base of karstification level
- Karstification levels of the river at required location were estimated using empirical formula developed by researchers and different zoning of vertical section of the river were categorized relative to the river bed.

- During setting of the profile of the abutment (aquifer), the range of values for Karst limestone were compared with the data from scientific researches obtained by published British geologic society and Middle East geological researches.
- Depending on the hydraulic features of each profile, the corresponding geologic features of the abutment were estimated as highly, moderately and slightly weathered or mixed geologic nature of lime stone.
- The values obtained compared with the values of research results conducted in other similar research areas, with local researchers' geologic data and master plan as shown in Figures below.
- Engineering parametric values like porosity, angle of friction, unconfined and confined strength of the abutment and dam body were estimated by matching the geologic features parameteric values of the area with the range of values available from similar research.

Table 9 **Estimated Hydraulic and geologic parameters**

Ser No	Elevation a.sl m	Karsttification Level	Depth of aquifer m	e (Value)	Factor of karstification	e ^(-0.012H)	Average Hydraulic m/sec	Hydraulic Conductivity m/sec	Karsttification Condition
1	1800	23.97	0	2.72E+00	23.97	1.00	0.00006900	0.0019858	Medium to High
3	1780	18.86	20	2.72E+00	23.97	0.79	0.00006900	0.0015621	
4	1760	14.83	40	2.72E+00	23.97	0.62	0.00006900	0.0012288	
5	1740	11.67	60	2.72E+00	23.97	0.49	0.00006900	0.0009666	
6	1720	9.18	80	2.72E+00	23.97	0.38	0.00006900	0.0007604	
7	1700	7.22	100	2.72E+00	23.97	0.30	0.00006900	0.0005982	
8	1680	5.68	120	2.72E+00	23.97	0.24	0.00006900	0.0004706	
9	1660	4.47	140	2.72E+00	23.97	0.19	0.00006900	0.0003702	
10	1640	3.51	160	2.72E+00	23.97	0.15	0.00006900	0.0002912	
11	1620	2.76	180	2.72E+00	23.97	0.12	0.00006900	0.0002291	
12	1600	2.18	200	2.72E+00	23.97	0.09	0.00006900	0.0001802	
13	1580	1.71	220	2.72E+00	23.97	0.07	0.00006900	0.0001417	
14	1560	1.35	240	2.72E+00	23.97	0.06	0.00006900	0.0001115	Low to Medium
15	1540	1.06	260	2.72E+00	23.97	0.04	0.00006900	0.0000877	
16	1520	0.83	280	2.72E+00	23.97	0.03	0.00006900	0.0000690	
17	1500	0.66	300	2.72E+00	23.97	0.03	0.00006900	0.0000543	
20	1480	0.41	340	2.72E+00	23.97	0.02	0.00006900	0.0000336	
21	1460	0.32	360	2.72E+00	23.97	0.01	0.00006900	0.0000264	
22	1440	0.25	380	2.72E+00	23.97	0.01	0.00006900	0.0000208	
23	1420	0.20	400	2.72E+00	23.97	0.01	0.00006900	0.0000164	
24	1400	0.16	420	2.72E+00	23.97	0.01	0.00006900	0.0000129	
25	1380	0.12	440	2.72E+00	23.97	0.01	0.00006900	0.0000101	Low to Lower
26	1360	0.10	460	2.72E+00	23.97	0.00	0.00006900	0.0000080	
27	1340	0.08	480	2.72E+00	23.97	0.00	0.00006900	0.0000063	
28	1320	0.06	500	2.72E+00	23.97	0.00	0.00006900	0.0000049	

Table 10 Hydraulic Conductivity Values of Karst Formations (British Geologic Survey /Report /CR/06/160N)

Typical ranges in hydraulic conductivity of common rock types (after Lewis, 1989)

Lithology	Hydraulic conductivity (m/day)
Clay*	5×10^{-7} to 10^{-3}
Loess	10^{-2} to 1
Silt	10^{-3} to 10^{-1}
Sand	10^{-1} to 5×10^{2}
Gravel	5×10^{1} to 5×10^{4}
Sand and gravel	5 to 10^{2}
Till	10^{-7} to 5×10^{-1}
Halite	5×10^{-9} to 5×10^{-3}
Limestone, dolomite	5×10^{-6} to 10^{0}
Karstic limestone	10^{-1} to 10^{3}
Chalk	Up to 5
Sandstone	5×10^{-3} to 2×10^{1}
Shale	5×10^{-8} to 10^{-4}
Lignite	10^{-1} to 10
Friable tuff	2×10^{-2} to 2
Welded tuff, ignimbrite	5×10^{-8} to 2×10^{-1}
Dense basalt	10^{-6} to 10^{-3}
Fractured basalt	10^{-4} to 1
Vesicular lava	10^{-4} to 10^{-3}
Lava	Less than 5×10^{-9} to 10^{3}
Slate	5×10^{-9} to 5×10^{-6}
Schist	10^{-7} to 10^{-4}
Dense crystalline rock	5×10^{-8} to 10^{-5}
Fractured crystalline rock	10^{-3} to 10

Karstified lime stone hydraulic conductivity range of values

3.3.5 Comparison of Hydraulic and Geologic Settings of Research Area

The comparison is done by referring the geologic setting obtained from published scientific papers conducted at local and international level. **The range of values obtained for karstified lime stone were 10^{-2}m/day to 10^{1} m/day or (1.15E-07 – 1.15E-02) m/sec.**

Figure 33 **Research area geological maps (Surface Geology)**

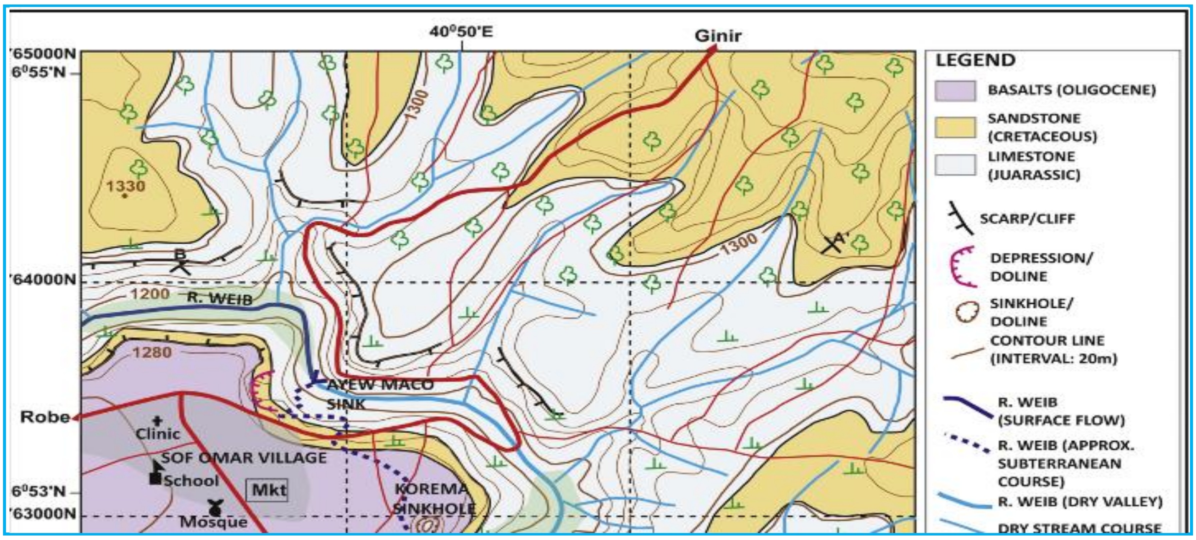
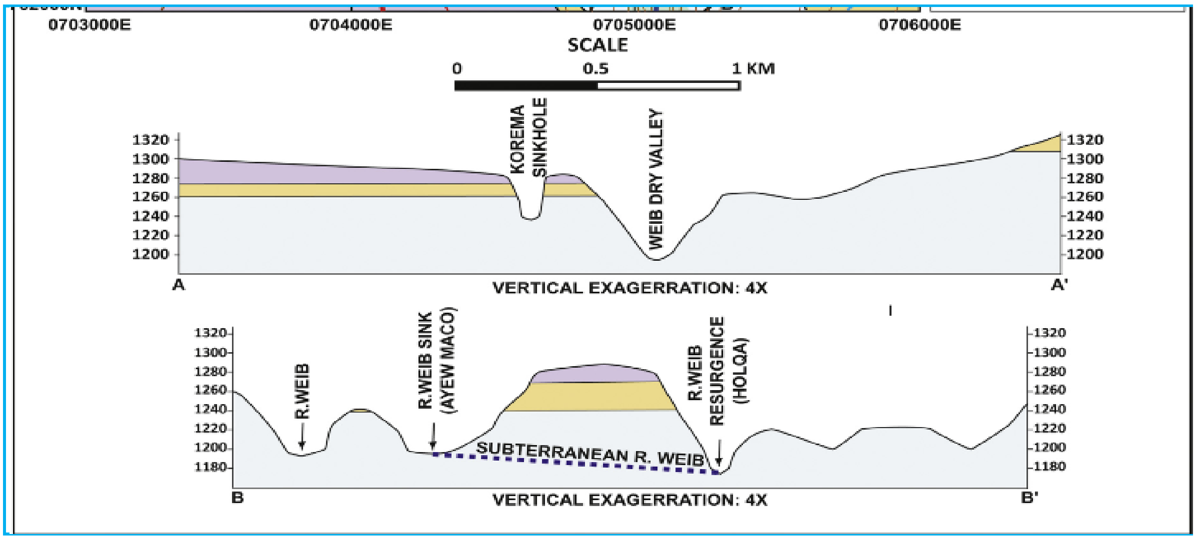


Figure 34 **Research area geological maps (Cross-Section)**



Source: (Asfawosen Asrat, 2015)

Figure 35 **Geological Map of Research Area (Plan view)**

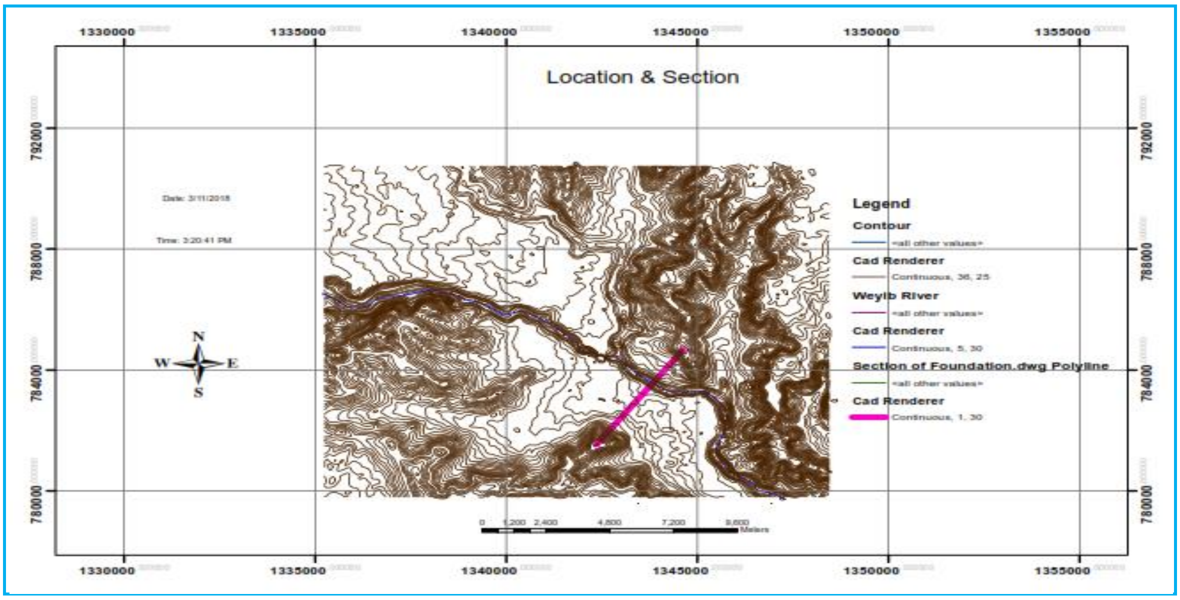
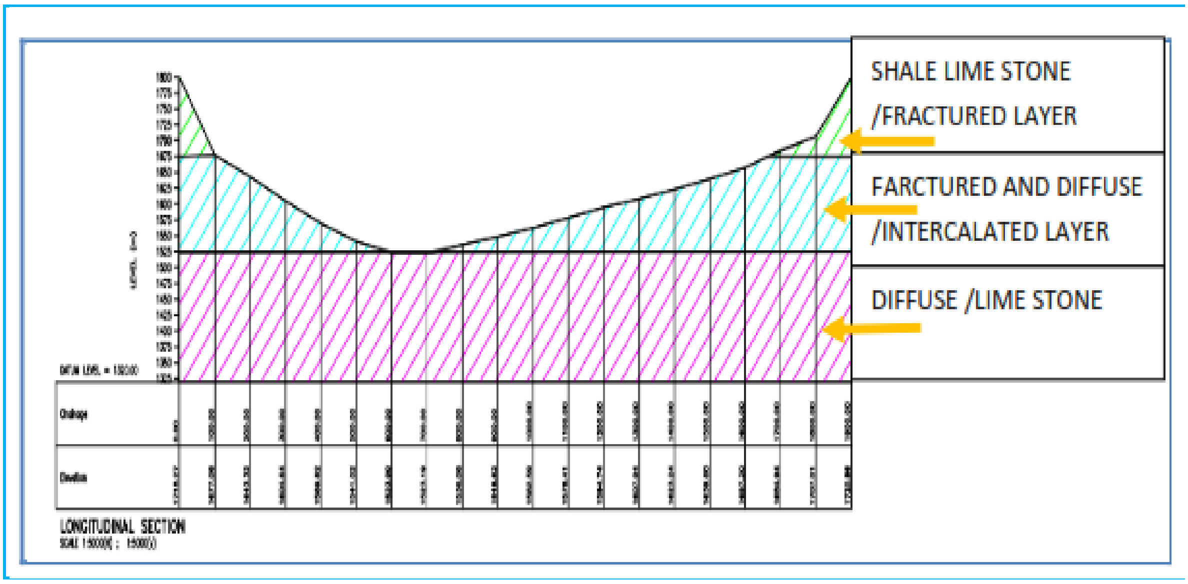


Figure 36 **Geological Cross-section of Research Area (Research out Put**)



As per the study by international journal of scientific research volume 4, issue 5, May 2013 and Ford and Williams, 1989 classification, the engineering aspects of Karst environment has geomorphologic characteristics of the fourth classes(Class-IV).

As per again the study by the Department of Geology and Geography, Georgia Southern University, Statesboro, GA 30460-8149 and Department of Earth and Atmospheric Science Purdue University, West Lafayette, IN 47907-13,

As per also the study conducted by local researcher of the area and based on the above assumptions, estimations and comparisons, the values obtained for further analysis were reasonably represent the specific area geological features for research propose.

Table 11 Summary table for estimated hydraulic and material Properties range of values

Karsttification Condition	Remark		Strength Property Friction Angle Ø	Cohesion	Dry Density KG/Cum	Porosity %	Poisson"s Ratio	UCS Mpa	Young"s Modulus Mpa
Medium to High	Effect of Karst	Shale Lime Stone	20-30	50-60 KN	above 1800	5 to 25	0.1 to 0.36	20 to 55	15 to 55

Table 12 Summary table for estimated hydraulic and material range of values for assumed dam analysis

Material	Hydraulic conductivies m/sec	Strength Property Friction Angle Ø	Cohesion	Dry Density KG/Cum	Porosity %	Poisson"s Ratio	UCS Mpa	Young"s Modulus Mpa	Assumed hieght for analysis m	Sourses
Embankement	6.91E-05	35	50	1800	25	0.18	20	40	10	Reference and research out puts under similar area
Foundation	6.91E-07	30	60	2000	15	0.18				
Concrete	6.91E-09	9	500	2300						

3.3.6 Rock Fill Dam Hydraulic Analysis

3.3.6.1 Selected Type and Section of Dam

Approach

Since the main aim of this research is to analyze seepage magnitudes and related hydraulic risks, the slope of the dam should be stable and the material strength of the dam should not be changed for the assumed dam height under any load. Hence, slope and material strength checking by software and analytic method using spread sheet were conducted to obtain reliable seepage magnitude useful for further analysis.

Dam Type

Rock fill dam with and without upstream cover which is homogeneous with the abutment (environment) was considered for the analysis since rock fill dam was considered as equivalent porous media and its hydraulic behavior resembles with averagely saturated and averagely karstified Karst hydrogeological feature.

Assumed Dam Height and Freeboard

Dam height for the analysis was assumed based on the hydraulic conductivities change obtained for each 20mts of the aquifer or abutment. Since the change in hydraulic properties estimated were similar with the changes obtained by different researches, dam height for analysis was assumed as 10mts for the sake of comparison and estimating seepage related risks of dam under Karst environment. Net freeboard requirements can be determined using the procedures described in Saville, McClendon, and Cochran (1962). In this research free board was considered allowing for settlement only.

Top Width, Alignment and Slope

Top width of an earth or rock-fill dam within conventional limits has little effect on stability and is governed by whatever functional purpose the top of the dam must serve. In this research, the minimum top width required for the dam irrespective of height is considered. Axes of embankments must be straight or most economical alignment fitting the topography and foundation conditions.

Sharp changes in alignment should be avoided which could cause concentration of seepage and possibly cracking and internal erosion. In this research, alignment perpendicular to the valley formations is considered. It may be more economical to flatten embankment slopes to attain the desired stability. In this research, the abutment properties and slope were taken similar with the assumed dam.

3.3.6.2 Analysis of Seepage Using Geo-Studio Software, 2012 &Analytic Method

The seepage flow through and under a dam produces both surface and subsurface flow to downstream of the dam. The portion of the total seepage that emerges from the ground, or is discharged from drains in the dam, its foundation, or abutments can be measured directly or an estimate of the quantity of subsurface flow from flow net seepage analyses.

3.3.6.2.1 Seepage Analysis using Geo-Studio, 2012

One of the basic requirements for analysis of embankment dams is to ensure safety against internal erosion, piping and development of excessive pore pressures. Steady state seepage analysis is carried out to estimate the amount of seepage through the embankments and foundation materials. The analysis was conducted using the state of the art Finite Element Method based computer program – Seep/W using Geo-slope studio, 2012. Permeability of materials for different zones of foundations was estimated depending on the depth of foundation.

Approach

In order to obtain reliable seepage magnitudes, stable slope analysis were carried out as per the above estimated materials parametric values. As it was advised by many researchers, rock fill dam with upstream cover should be checked for Settlement, downstream slope and response of grain materials to deformation of rigid parts of the dam.

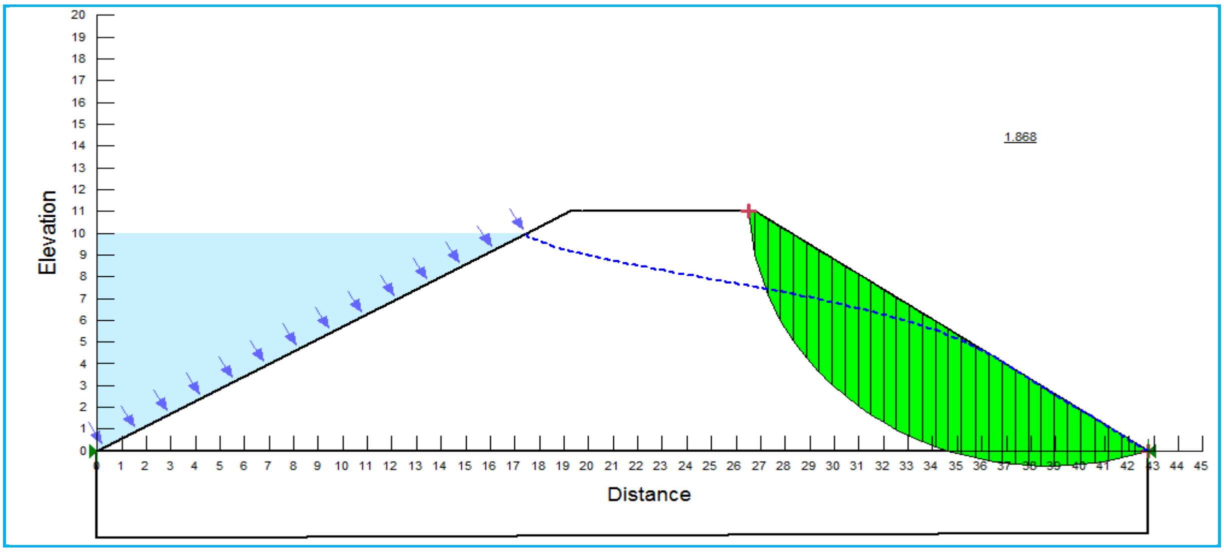
Table 13 Summary table for safety factor under various loading conditions

Cases	Loading Condition	Critical Slope	FOS
I	End of construction	Upstream	1.3
II	Sudden drawdown	Upstream	1.3
III	Steady state seepage	Upstream	1.5
		Downstream	1.5
IV	Steady state seepage with earthquake	Upstream	1.1
		Downstream	1.1

In this research, stability of assigned slope under equivalent dynamic loading was checked and the result obtained was presented in figures and tables below.

Slope stability safety factor obtained for dam without upstream cover was **1.87** which was safe and more than the recommended value of **1.3** under equivalent dynamic loading conditions. The slope magnitude, which was **1.75**, was assigned considering the foundation and embankment materials.

Figure 37 **Assigned Slope and obtained safety factor without upstream cover**



Slope stability safety factor obtained for dam without upstream cover was **2.55**which was safe and more than the recommended value of **1.3** under equivalent dynamic loading conditions. The slope magnitude, which was **1.75**, was assigned considering the foundation and embankment materials.

Figure 38 **Assigned Slope and obtained safety factor with upstream cover**

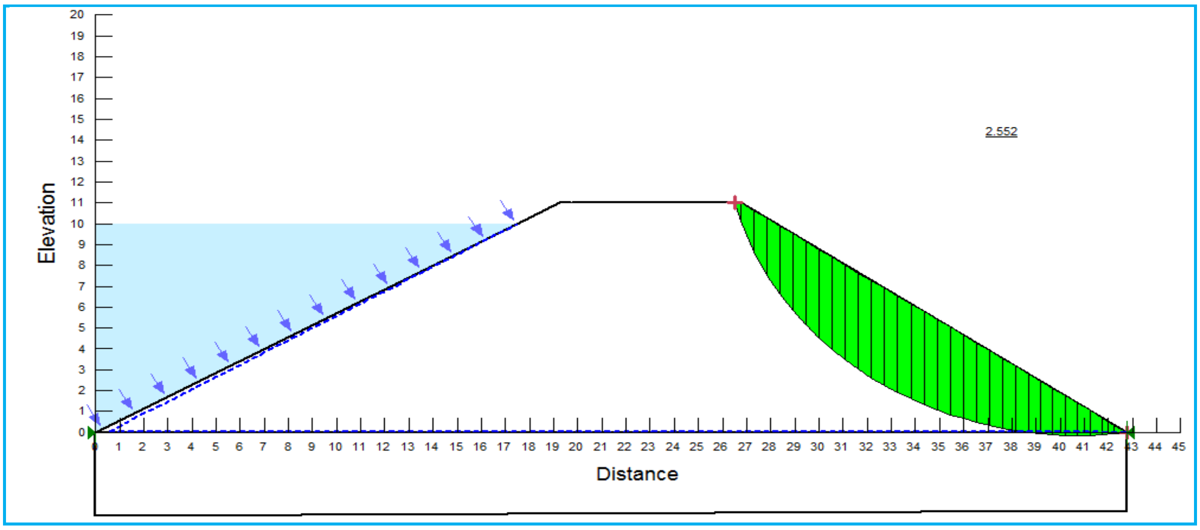


Figure 39 Seepage value obtained for without upstream cover (1.55E-04) m/sec

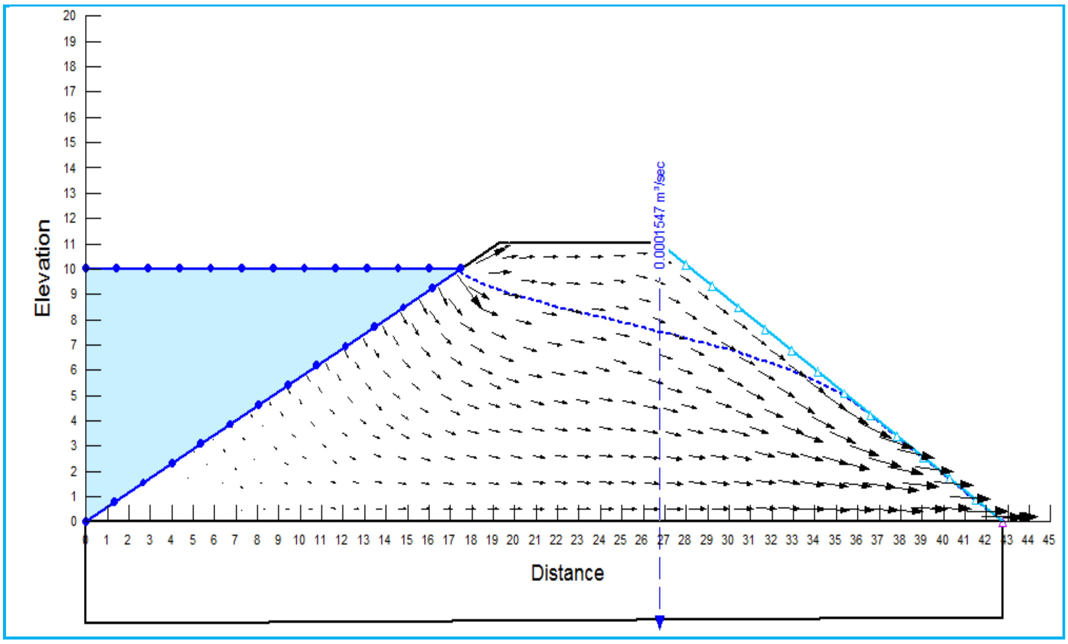


Figure 40 Seepage value obtained for with upstream cover (2.80E-07) m/sec

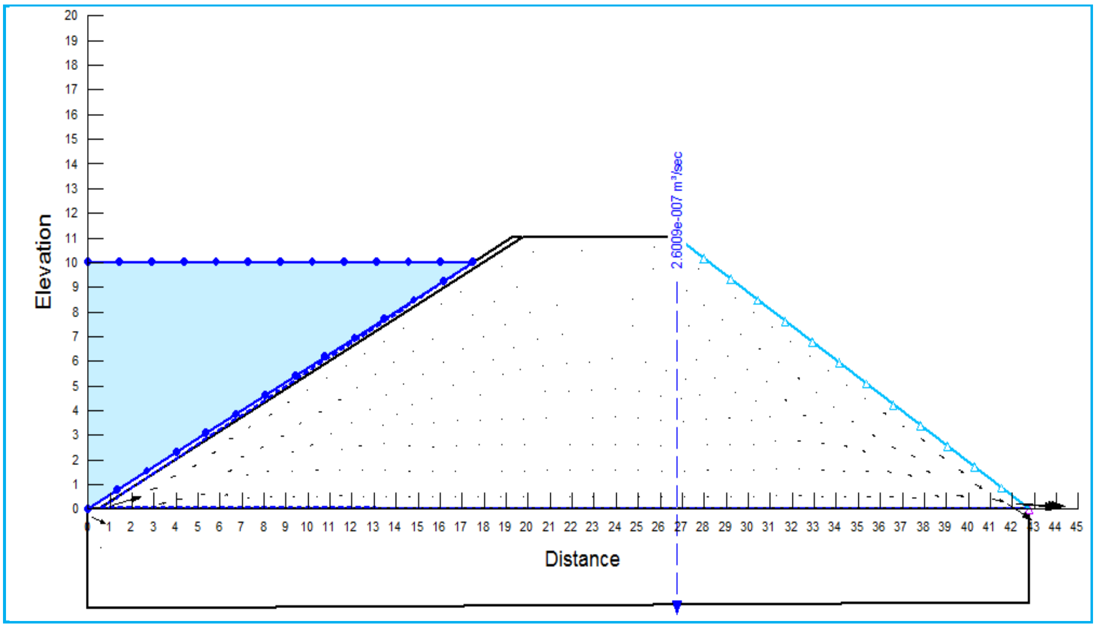


Figure 41 Pore water pressure for without upstream cover

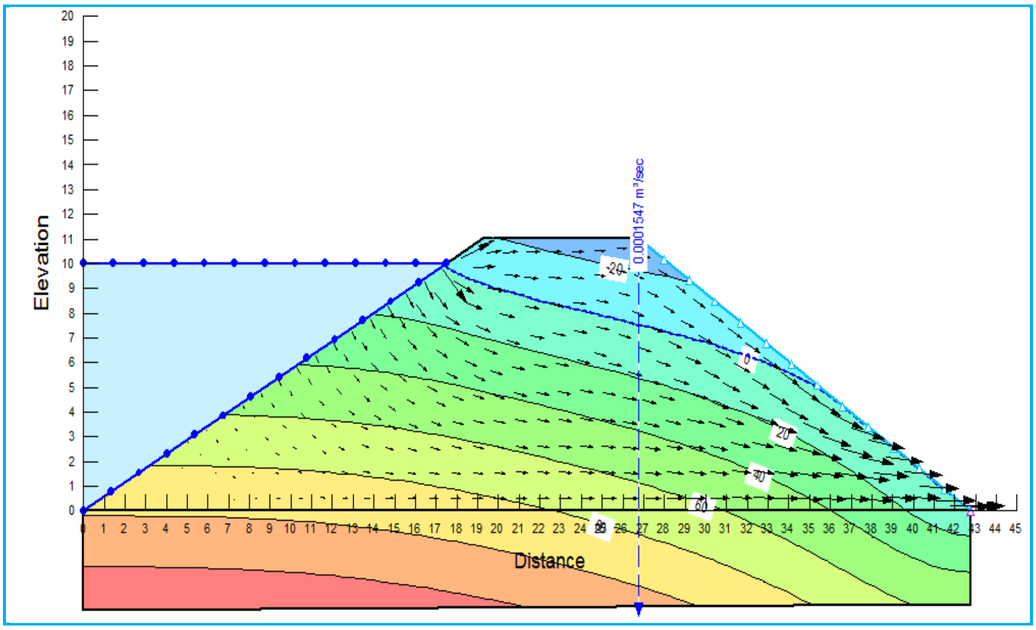
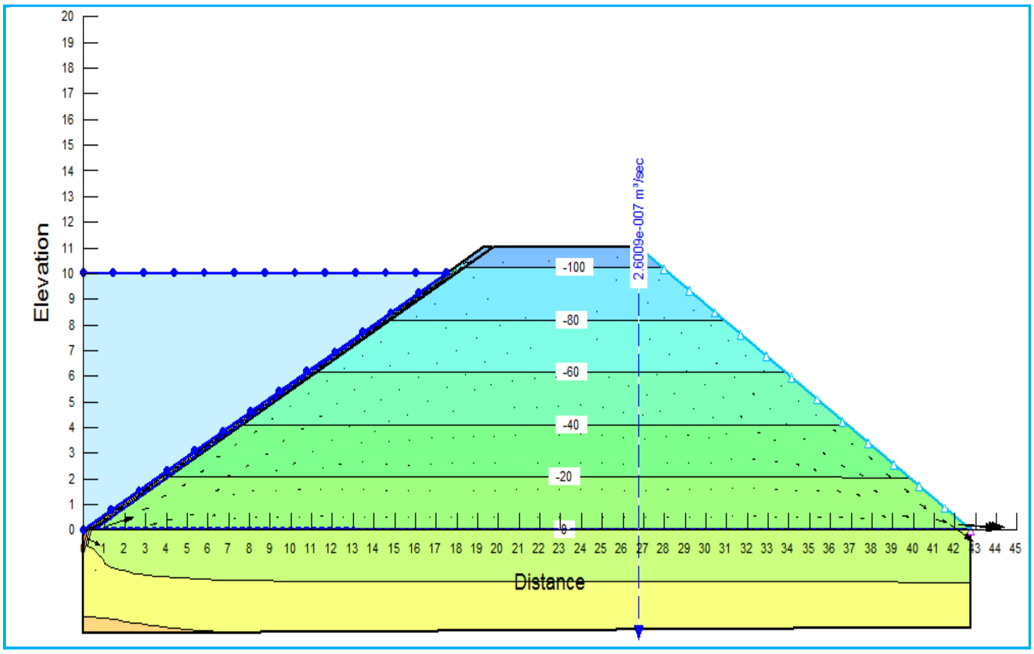


Figure 42 Pore water pressure for with upstream cover



3.3.6.2.2 Seepage analysis by excel spreed sheet

The quantitative analysis of all stress conditions were also done by spread sheet for each zoning/layers of foundation considering 10m and 20m height as shown in the table below. The general formula used in quantitative analysis was as expressed below.\

$$\boldsymbol{\delta e = \delta t - P = (\gamma sb + \gamma wdn) - (\gamma w\varphi)}$$

Where,

δe = Effective stress

δt = Total stress

γsb = Specific weight for drained material

d = depth of water

n = Total porosity of solid

φ = pressure above reference plane

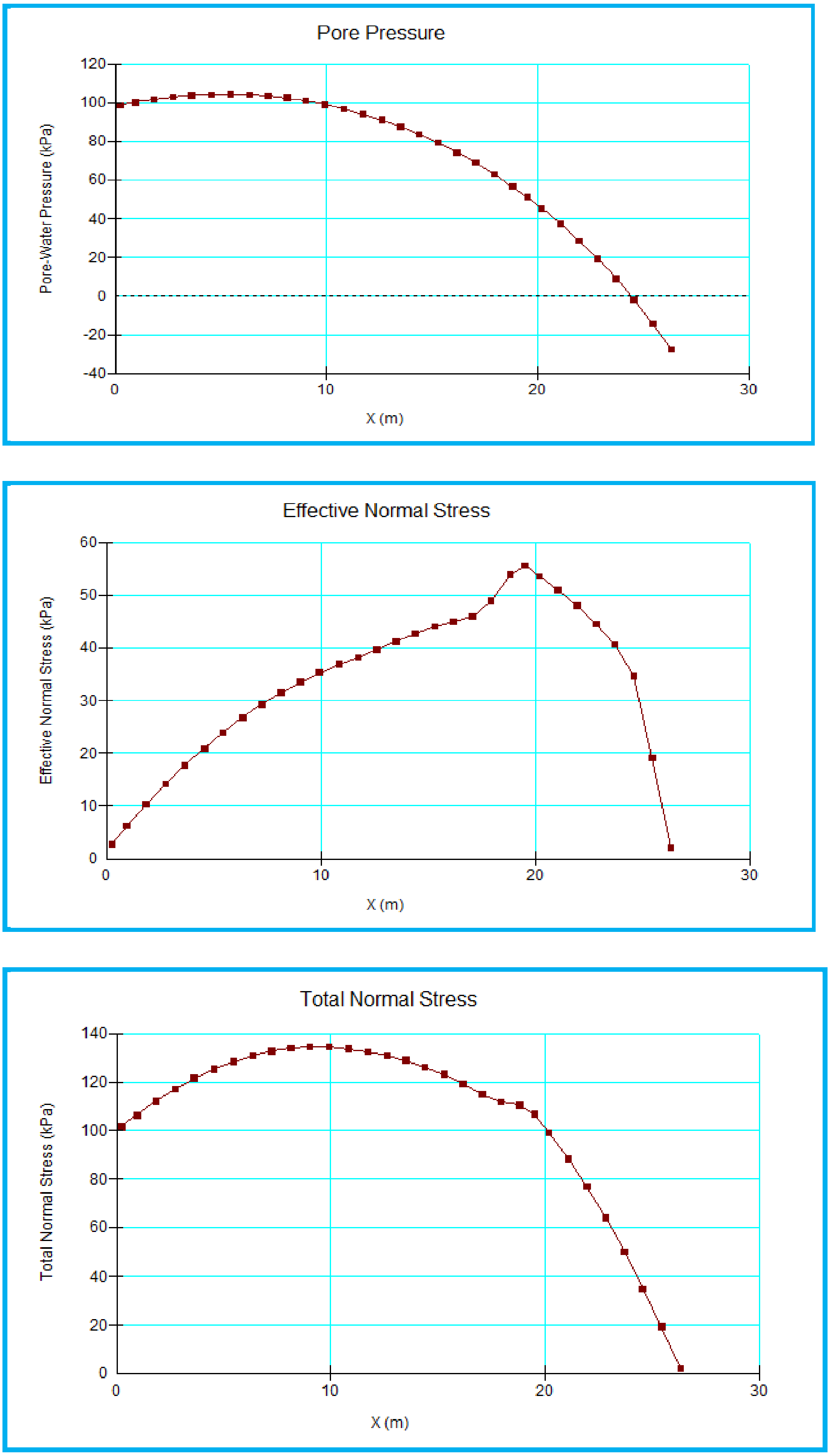
Table 14 Summary of Seepage value based on Relative hydraulic conductivity

Hieght Of Dam	Partial Height	Stable Side Slope of Abutment	Width of Dam	Hydraulic Gradient	Effective Hydraulic Permeability	Relative Hydraulic Permeability	Area of Cross Section	Average Estimated Seapage
m	m	m/m	m	m/Sec	m/Sec	m/sec	sqm	Cum/sec
480.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0005560
460.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0004374
440.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0003441
420.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0002707
400.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0002129
380.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0001675
360.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0001318
340.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0001036
320.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000815
300.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000641
280.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000505
260.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000397
240.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000312
220.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000246
200.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000193
180.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000152
160.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000094
140.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000074
120.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000058
100.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000046
80.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.0000036
60.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.00
40.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.00
20.00	20.00	1V:1.75H	100.00	0.28	0.00	0.00	2000.00	0.00
0.00	0.00	1V:1.75H	30.00	0.28	0.00	0.00	0.00	0.00
							Average	0.000104

Table 15 Quantitative analysis of stress using spread sheet

Ser No	Elevation a.sl	Partiial Head	Total head	Dry Density	Aceelartion of gravity	Density of water	Porosity	Total stress	Pore Pressure	Effective Stress	Normal Total Stress	Normal Effective Stress
	m	m	m	KN/Cum	Kg/sqm/sec	Kg/cum	%	Kpa	Kpa	Kpa	Kpa	Kpa
1.00	1800.00	20.00	480.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
3.00	1780.00	20.00	460.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
4.00	1760.00	20.00	440.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
5.00	1740.00	20.00	420.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
6.00	1720.00	20.00	400.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
7.00	1700.00	20.00	380.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
8.00	1680.00	20.00	360.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
9.00	1660.00	20.00	340.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
10.00	1640.00	20.00	320.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
11.00	1620.00	20.00	300.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
12.00	1600.00	20.00	280.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
13.00	1580.00	20.00	260.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
14.00	1560.00	20.00	240.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
15.00	1540.00	20.00	220.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
16.00	1520.00	20.00	200.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
17.00	1500.00	10.00	180.00	1800.00	9.81	1000.00	15	191.30	98.10	93.20	110.95	54.05
19.00	1490.00	10.00	170.00	1800.00	9.81	1000.00	15	191.30	98.10	93.20	110.95	54.05
20.00	1480.00	20.00	160.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
21.00	1460.00	20.00	140.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
22.00	1440.00	20.00	120.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
23.00	1420.00	20.00	100.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
24.00	1400.00	20.00	80.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
25.00	1380.00	20.00	60.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
26.00	1360.00	20.00	40.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
27.00	1340.00	20.00	20.00	1800.00	9.81	1000.00	15	382.59	196.20	186.39	221.90	108.11
28.00	1320.00	0.00	0.00	1800.00	9.81	1000.00	15	0.00	0.00	0.00	0.00	0.00
20m Head							15	353.16	181.11	172.05	204.83	99.79
10m Head							15	176.58	90.55	86.03	102.42	49.90

Figure 43 Analysis of stress by Geo-Slope, 2012



3.3.7 Rock-Fill Dam Seepage Related Risk

Risk analysis is carried out considering flow through the dam body and foundation with and without upstream cover material. During the analysis; average river width, safe side slope of the embankment, assumed dam height, and total water contact width are well established. The two hydraulic flow conditions used in the analysis are expressed well below.

3.3.7.1 Diffuse Method of Analysis

Fracture size of 0.5cm and less than is considered as matrix/ or diffuse flow as it is stated in many research papers. Diffuse method of analysis is mainly for comparison proposes with Ge0-slope 2012 software output. Diffuse method is applicable under the maximum seepage value of 0.005 cumec which has a width less than 0.5cm fracture or equivalent pipe diameter. The formula used is expressed as follows as it is given by Darcy.

$$\boldsymbol{Q} = \mathbf{KIA}$$

Where,

Q= Discharge in Cumec

A= Area in Sqm

I= Hydraulic gradient

K= Hydraulic Permeability

3.3.7.2 Conduit Method of Analysis

Conduit method of analysis is applicable for seepage values greater than 0.005 cumec which has a greater diameter than 0.005m fracture width or equivalent pipe diameter. The formula used for conduit analysis is expressed as follows:-

$$\boldsymbol{Q} = \left[\boldsymbol{2dgA^2}\right]^{\mathbf{0.5}} * [\boldsymbol{I}]^{\mathbf{0.5}}*$$

Where,

Q= Discharge in cum/sec

D= Conduit size, m

A= Area in Sqm

f= Friction factor = 0.1-0.01

I= Hydraulic gradient

g= Gravitational acceleration

Table 16 Initial discharge and fracture size for conduit flow

Hydraulic Gradient	Average Friction Factor	Diameter m	Cross Sectional area	Aceeleration of Gravity	Discharge
m/Sec		D1	Sqm	sqm/sec	cum/sec
0.25	0.055	0.05	0.0019625	9.81	0.004
0.25	0.055	0.1	0.00785	9.81	0.023
0.25	0.055	0.5	0.19625	9.81	1.310
0.25	0.055	0.75	0.4415625	9.81	3.611

The initial seepage magnitude and minimum dam height for initiation of conduit / pipe flow /were as indicated in the tables above which are 0.005cumec and 10m height respectively.

3.3.7.3 Manageable Seepage with Maintenance

Safe and economical maintenance during operation is the base for estimating the maximum allowable seepage magnitudes. Average values of percentages have been taken based on the ratio of what has been measured and what have been controlled on dams in the world. As per the value obtained from world dams under Karst environment, the average value and percentage estimated through systematic analysis is summarized in the tables below.

Table 17 Managed Seepage Value of dams under Karst environment

Ser. No.	Reservoir Name	Country	Seepage Cum/sec	Managed Seepage Cum/sec
1	Keban	Turkey	26	10
2	Camaras	Spain	11.2	2.6
3	Marun	Iran	10	0
4	Great Fall	USA	9.5	0.2
5	Canelles	Spain	8	0
6	Mavrovo	Mecodenia	9.5	0
7	Busko Blato	BiH	40	5
8.00	Hutovo	BiH	10.00	1.00

Source: (Milanovich 2004)

Table 18 Managed Seepage Percentage Value of dams under Karst environment

Ser. No.	Reservoir Name	Country	Seepage Cum/sec	Managed Seepage Cum/sec	Alloable With Maintenance %	Weighted Value %	Weighted Value cumec
1	Keban	Turkey	26	10	38.46	16.7	2.6
2	Camaras	Spain	11.2	2.6	23.21		
3	Marun	Iran	10	0	0.00		
4	Great Fall	USA	9.5	0.2	2.11		
5	Canelles	Spain	8	0	0.00		
6	Mavrovo	Mecodenia	9.5	0	0.00		
7	Busko Blato	BiH	40	5	12.50		
8.00	Hutovo	BiH	10.00	1.00	10.00		

Hence, weighted average seepage value is 2.6 cumec and 17% from average total seepage value of 15.5cumec. In this analysis, the minimum weighted value (2.6 cumec) is taken as the limit for seepage allowable value with 17% for limiting height of dam with corresponding discharge of 0.442 cumec.

Table 19 Analysis of hydraulic risk of dam without upstream cover

Ser No.	Average Unit Seepage	Hiegt of Dam	Stable Slope	Average River Bottom Width	River Top Width	Area of Sepage Contact	Total Seepage	Remark
		m	H:V	m	m	Sqm	cumec	
1	1.55E-04	0	1.75	30	30	0	0.000	Possibility of conduit formation
2	1.55E-04	5	1.75	30	47.5	237.5	0.037	
3	1.55E-04	10	1.75	30	65	650	0.101	
4	1.55E-04	15	1.75	30	82.5	1237.5	0.192	
5	1.55E-04	20	1.75	30	100	2000	0.310	
6	1.55E-04	25	1.75	30	117.5	2937.5	0.455	
7	1.55E-04	30	1.75	30	135	4050	0.628	
8	1.55E-04	35	1.75	31	153.5	5372.5	0.833	
9	1.55E-04	40	1.75	32	172	6880	1.066	
10	1.55E-04	45	1.75	33	190.5	8572.5	1.329	
11	1.55E-04	50	1.75	30	205	10250	1.589	
12	1.55E-04	55	1.75	30	222.5	12237.5	1.897	
13	1.55E-04	60	1.75	30	240	14400	2.232	
14	1.55E-04	65	1.75	30	257.5	16737.5	2.594	
15	1.55E-04	70	1.75	30	275	19250	2.984	
16	1.55E-04	75	1.75	30	292.5	21937.5	3.400	
17	1.55E-04	80	1.75	30	310	24800	3.844	
18	1.55E-04	85	1.75	30	327.5	27837.5	4.315	
19	1.55E-04	90	1.75	30	345	31050	4.813	
20	1.55E-04	95	1.75	30	362.5	34437.5	5.338	
21	1.55E-04	100	1.75	30	380	38000	5.890	
22	1.55E-04	105	1.75	30	397.5	41737.5	6.469	
23	1.55E-04	110	1.75	30	415	45650	7.076	
24	1.55E-04	115	1.75	30	432.5	49737.5	7.709	
25	1.55E-04	120	1.75	30	450	54000	8.370	
26	1.55E-04	125	1.75	30	467.5	58437.5	9.058	
27	1.55E-04	130	1.75	30	485	63050	9.773	
28	1.55E-04	135	1.75	30	502.5	67837.5	10.515	
29	1.55E-04	140	1.75	30	520	72800	11.284	

Table 20 Analysis of hydraulic risk of dam without upstream cover

Ser No.	Average Unit Seepage	Hiegt of Dam m	Stable Slope H:V	Average River Bottom Width m	River Top Width m	Area of Sepage Contact Sqm	Total Seepage cumec	Remark
1	2.60E-07	0	1.75	30	30	0	0.000	No possibility of conduit formation
2	2.60E-07	5	1.75	30	47.5	237.5	0.000	
3	2.60E-07	10	1.75	30	65	650	0.000	
4	2.60E-07	15	1.75	30	82.5	1237.5	0.000	
5	2.60E-07	20	1.75	30	100	2000	0.001	
6	2.60E-07	25	1.75	30	117.5	2937.5	0.001	
7	2.60E-07	30	1.75	30	135	4050	0.001	
8	2.60E-07	35	1.75	31	153.5	5372.5	0.001	
9	2.60E-07	40	1.75	32	172	6880	0.002	
10	2.60E-07	45	1.75	33	190.5	8572.5	0.002	
11	2.60E-07	50	1.75	30	205	10250	0.003	
12	2.60E-07	55	1.75	30	222.5	12237.5	0.003	
13	2.60E-07	60	1.75	30	240	14400	0.004	
14	2.60E-07	65	1.75	30	257.5	16737.5	0.004	
15	2.60E-07	70	1.75	30	275	19250	0.005	
16	2.60E-07	75	1.75	30	292.5	21937.5	0.006	Possility of Conduit formation
17	2.60E-07	80	1.75	30	310	24800	0.006	
18	2.60E-07	85	1.75	30	327.5	27837.5	0.007	
19	2.60E-07	90	1.75	30	345	31050	0.008	
20	2.60E-07	95	1.75	30	362.5	34437.5	0.009	
21	2.60E-07	100	1.75	30	380	38000	0.010	
22	2.60E-07	105	1.75	30	397.5	41737.5	0.011	
23	2.60E-07	110	1.75	30	415	45650	0.012	
24	2.60E-07	115	1.75	30	432.5	49737.5	0.013	
25	2.60E-07	120	1.75	30	450	54000	0.014	
26	2.60E-07	125	1.75	30	467.5	58437.5	0.015	
27	2.60E-07	130	1.75	30	485	63050	0.016	
28	2.60E-07	135	1.75	30	502.5	67837.5	0.018	
29	2.60E-07	140	1.75	30	520	72800	0.019	

Chapter Four - Research Out Puts

As per the result of this research, previously conducted in the area and other similar Karst environment of the world, hydraulic analysis of dam is possible at different stage like study, design, construction and operation due to heterogeneous nature of geo-physical formations and point nature of parametric observation data.

4.1 Base Flow Difference between Gauge Stations

Base flow estimation of the research area using USGS software programme, excel based hydrograph separation and Excel based hydrograph by Chapman's algorithm were conducted. The average base flow values obtained. The comparison was done between the methods to check the reliability of the results obtained through standard deviation least values. As per the comparison, USGS software Programme result was selected. The flow difference of the two gauge stations was found ***9.82cum/sec.*** Again, based on the two contributing sides area sizes of the river, base flow results obtained were ***1.89cumec*** and ***7.91cumec*** for the left and right side respectively.

Table 21 Comparison table of Hydrograph separation methods

Ser No.	Hydrograph Separation Method	Maximum Stream Flow Cumec	Average Stream Flow Cumec	Average Baseflow Flow Cumec	Standard Deviation value of Base flow	Remark
I	Sofumar					
1	USGS	69.98	24.52	18.49	10.5	Selected
2	Chapman Algorithm	69.98	24.52	23.94	15.0	
3	VBI Excel	69.98	24.52	18.64	10.5	
II	Alemkerem					
1	USGS	60.86	14.79	8.67	7.00	Selected
2	Chapman Algorithm	60.86	14.79	14.61	12.99	
3	VBI Excel	60.86	14.79	8.67	7.04	
III	Flow Difference For Comparison and Computation	9.12	9.73	9.82		

4.2 Average Saturated Hydraulic Conductivity Values

After estimation of the flow contribution of both sides of the river, unconfined well hydraulic formula was used in estimating the values of left and right side of the river. The results obtained were ***1.50E-04*** and ***6.90E-05m/sec*** respectively.

Table 22 Saturated hydraulic analysis using well hydraulics model

Description	Radius of river	Radius of aquifer	(H1-H2)^2	Ln(R/r)	Saturated hydaraulic conductivity (K)	Claculated base flow
	m	m			m/sec	Cumec
Right Side	15	1127.62	17500	4.32	0.00015	1.96
Left Side	15	2798.21	189100		6.90379E-05	7.84

4.3 Estimation of Hydro-Geologic Settings

By using empirical formula, different hydraulic conductivity values of the left and right sides of the river at different layers of aquifer/foundation was carried out. Based on the result obtained, it was found possible to categorize different horizons of 250m above and 250 below the river bed, considering the discharge level/ water level/river bed as base of karstification or location of average saturated hydraulic conductivity.

The results obtained were again compared with relevant values of international and local researches and the range values were found useful to set the level, hydraulic and geologic conditions. Based on the hydraulic and geologic classification, some supportive material strength properties were possible to estimate and the results were also compared with similar scientific research papers for their reliability. Hydro-geological cross section of the research area and estimated material properties were found as summarized in the figure and table below.

Figure 44 Estimated Hydro-Geologic Settings of the research area (Cross-Section)

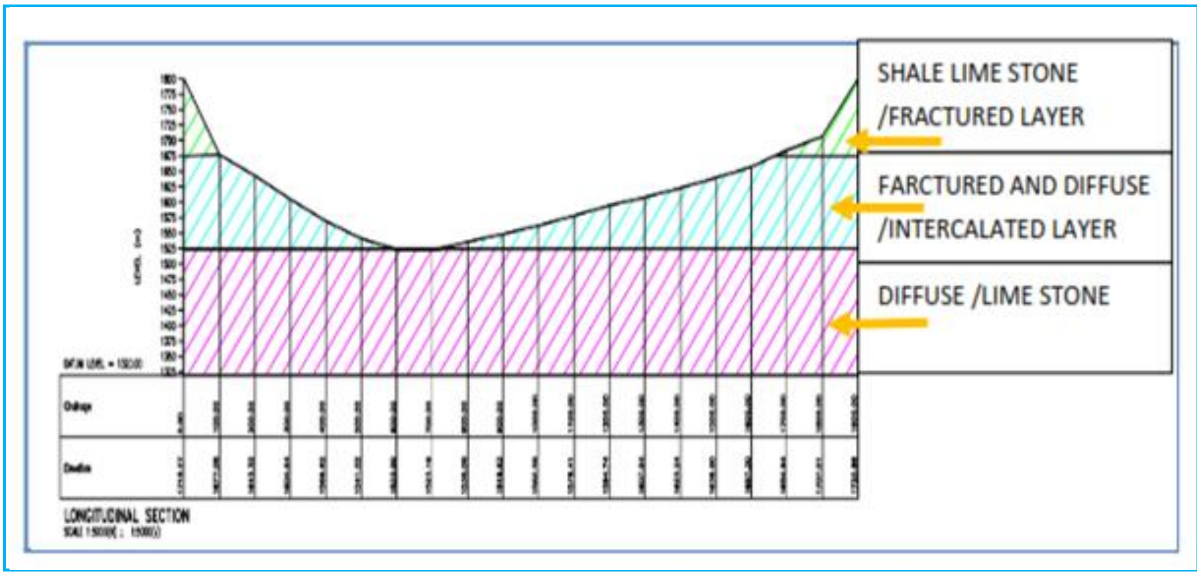


Table 23 Estimated Material Strength Parameteric Values

Material	Hydraulic conductivities	Strength Property		Dry Density	Porosity		UCS	Young"s Modulus	Assumed hieght for analysis	Sourses
	m/sec	Friction Angle Ø	Cohesion	KG/Cum	%	Poisson"s Ratio	Mpa	Mpa	m	
Embankement	6.91E-05	35	50	1800	25	0.18	20	40	10	Reference and research out puts under similar area
Foundation	6.91E-07	30	60	2000	15	0.18				
Concrete	6.91E-09	9	500	2300						

4.4 Slope, Seepage & Related Risk Analysis

Slope analysis was done and the assumed slope of the dam and its environment was found safe as per the standard. The allocated slope was ***1V:1.75 H*** values and the stability were checked for all loading conditions, like steady state for static, dynamic and combined loads. The safe values obtained for equivalent dynamic loading at downstream slope were ***1.87*** and ***2.55*** for the with and the without upstream cover conditions.

Seepage values obtained in the analysis using geo-slope 2012 software and analytic method using spreadsheet for both diffuse and conduit hydraulic conditions were found comparable and the average values obtained were ***1.04E-04 cum/sec*** and ***2.87E-07 cum/sec*** for ***the with out*** and ***with upstream cover conditions.***

Stress condition of the dam and foundation under saturation were checked using ***excels based analysis and Geo-Studio 2012 software*** and the result were found comparable for the assumed dam height. The values obtained for normal effective stress were **49.99 Kpa and 99.79KPa** for ***10mts and 20mts heights*** respectively. These values were comparable with software outputs which were ***50kKPa*** and ***100KPa*** respectively.

Seismic loading has brought some normal stress values change which can show material beviour alteration but not found significant in this research for one thing the area is not susceptible for seismic loads and the other thing it was considered for equivalent dynamic loading condition considering ratio of stress to strain as a function of permanent water level. ***This can be controlled also by changing the density of material to the required levels.***

4.4.1 Arranged Dam Scenarios & Seepage Related Risk Analysis

Rock fill dam with and without upstream concrete face were carefully analyzed using obtained data under saturated and unsaturated conditions by the software. Diffuse and conduit hydraulical flow conditions analysis were carried out assuming different water levels or dam heights which corresponds to the hydro-geologic settings.

Manageable seepage magnitudes and their weighted percentage were obtained from observed and measured values of dams of the world under similar environment. The data obtained systematically used to predict safe height of dam under safe water proof work and sufficient management during the operation life of the dam.

The safe maximum manageable seepage magnitude taken was ***0.442 cumec*** considering highly fractured layers obtained through estimating the karstification level. ***The safe heights of the dam, without conduit creating conditions were found 0m and 70m for the with out and with water proof covering respectively. The dam above 70m was conduit creating conditions for all scenarios.***

Chapter Five - Conclusions & Recommendations

5.1 Conclusions

Hydraulic analysis of rock fill dam under Karst environment was carried out and the following concluding remarks were obtained.

- All dams in the world constructed under Karst environment have leakage and related problems that are manageable to some extent in some countries and no controlling is achieved so far in others.
- As per the international engineering classification of Karst environment, the research area corresponds to Class K-IV having all features of the class.
- Karst environment needs carful study for safe utilization of the recourses found within it.
- The analysis conducted by software's and other methods using formulas are found comparable for base flow, seepage and stress values.
- Based on the result obtained, the average values of base flows at two gauge station were different.
- The base flow at each gauge station was found useful in estimating the base flow difference in between them, that is, the research area location
- As per the contributing area, the magnitudes of flow for the left and right side of the river found different and the average saturated hydraulic conductivities of the two sides were also different in similar manner.
- As per the result obtained in the course of estimating the karstification level of the area, different layers hydraulic conductivity, geological setting and hydraulical engineering parametric values estimations were found possible.
- The different layers of geological and hydraulical settings were found helpful in arranging different scenarios for the analysis of slope, water tightness and related risk subjected to respective water level.
- Homogeneous Rock fill dam is found representative for heterogeneous nature of Karst environment and the average saturated hydraulic and engineering parameters in addressing and averaging spatial and temporal difference.
- Seepage, slope and water tightness analysis with and without concrete facing have shown different
- Hydraulic and material strength parametric values obtained through analysis were within the range of corresponding values of homogeneous rock fill.

- Seepage, slope, stress and related others analysis by means of software and spread sheet are found comparable and reasonable.
- Seepage values obtained through observation and measuring of dams constructed under Karst environments of the world were found useful for systematic prediction of the maximum possible manageable magnitudes.

5.2 Recommendations

The concluding remarks summarized above were the base to reach at some recommendations as stated below:

5.2.1 General Recommendations

- Through Safe, economical and environmentally friendly technological interventions resource abstraction can be achieved with some manageable risks.
- Local and international research conducted so far on dams under Karst environment indicated some useful parametric values that made the research to be conducted with systematical approach. Similarly this research results can be an indicative and useful for some related works which are relevant to this research.

5.2.2 Specific Recommendations

- Dams under considerations and similar areas, before and after intervention the right and left side of the river abutments are better to be checked differently.
- Rock fill dam is impossible to be constructed without abutment, foundation and embankment water proof treatments.
- Homogeneous Rock fill dam intervention needs dam and foundation water proof works. As per this research results and similarly conducted, abutment, foundation and embankment water proof works should be in the order of 10^2 and 10^5 time the hydraulic parametric values of the foundation and dam material respectively.

- Homogeneous Rock fill dam intervention in similar areas, dam height above 70m even after all dam and foundation treatment works has a possibility of creating piping through the dam and its foundation that can bring serious risk without care of manageable seepage loses.

- As data of engineering geologic are mandatory, hydrogeological in situ data are very essential for water proofing works of dam under Karst environment.

- Since the aim of this research is seepage and related risks of rock fill dam under Karst environment, hydraulic risks like fracturing, deformations of rigid dam parts, response of dam material to deformation and settlements under combined loads are still areas of further researches.

Chapter Six – References

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Chapter Seven - Appendixes

Table 24 Data quality check for outlier using XLSTAT; 2017

XLSTAT 2017.02.44369 - Grubbs test for outliers - Start time: 6/3/2017 at 3:10:32 PM / End time: 6/3/2017 at 3:11:11 P

Data: Workbook = BookG.xlsx / Sheet = Dinsho / Range = Dinsho!$B$1:$N$14 / 13 rows and 13 columns

Alternative hypothesis: Two-sided

Significance level (%): 5

Iterations: Maximum: 1

Number of simulations: 1000000

Run agε

Summary statistics:

Variable	Observations	Obs. with missing data	Obs. without missing data	Minimum	Maximum	Mean	Std. deviation	Remark
Jan	13	1	12	0.00	137.00	31.18	46.39	No Outlier
Feb	13	1	12	0.00	180.00	33.12	52.68	No Outlier
Mar	13	0	13	7.00	218.60	86.48	62.85	No Outlier
Apr	13	0	13	103.40	383.70	192.89	79.76	No Outlier
May	13	2	11	37.80	404.10	141.23	116.83	No Outlier
Jun	13	1	12	44.90	288.80	100.29	70.03	No Outlier
Jul	13	0	13	88.50	329.00	192.35	76.14	No Outlier
Aug	13	3	10	88.70	290.50	186.33	73.29	No Outlier
Sep	13	1	12	58.80	329.40	133.03	72.54	No Outlier
Oct	13	1	12	21.80	414.00	167.00	126.95	No Outlier
Nov	13	0	13	6.30	226.10	70.87	75.46	No Outlier
Dec	13	0	13	0.00	164.00	47.43	43.72	No Outlier
Annual	13	0	13	875.50	2328.70	1391.02	433.65	No Outlier

XLSTAT 2017.02.44369 - Grubbs test for outliers - Start time: 6/3/2017 at 3:15:06 PM / End time: 6/3/2017 at 3:15:35 PM								
Data: Workbook = Book111.xlsx / Sheet = Goro / Range = Goro!$A$1:$M$10 / 9 rows and 13 columns								
Alternative hypothesis: Two-sided								
Significance level (%): 5								
Iterations: Maximum: 1								
Number of simulations: 1000000								
again:								
Variable	Observations	Obs. with missing data	Obs. without missing data	Minimum	Maximum	Mean	Std. deviation	Remark
Jan	12	1	11	0.00	153.00	26.08	52.53	No Outlier
Feb	12	1	11	0.00	155.00	29.28	55.32	No Outlier
Mar	12	1	11	2.00	118.70	51.78	45.62	No Outlier
Apr	12	0	12	98.00	394.80	213.03	99.68	No Outlier
May	12	0	12	57.30	279.70	170.71	73.29	No Outlier
Jun	12	1	11	8.50	87.10	34.44	27.84	No Outlier
Jul	12	1	11	0.00	52.00	26.95	19.28	No Outlier
Aug	12	0	12	0.00	112.40	56.47	39.85	No Outlier
Sep	12	0	12	0.00	145.10	80.19	43.23	No Outlier
Oct	12	2	10	105.00	409.00	212.14	104.75	No Outlier
Nov	12	1	11	0.00	183.70	54.39	59.90	No Outlier
Dec	12	2	10	0.00	89.00	23.03	32.56	No Outlier

XLSTAT 2017.02.44369 - Grubbs test for outliers - Start time: 6/3/2017 at 3:16:25 PM / End time: 6/3/2017								
Data: Workbook = BookGO.xlsx / Sheet = Gasera / Range = Gasera!$A$1:$M$14 / 13 rows and 13 columns								
Alternative hypothesis: Two-sided								
Significance level (%): 5								
Iterations: Maximum: 1								
Number of simulations: 1000000								
Run again:								
Summary statistics:								
Variable	Observations	Obs. with missing data	Obs. without missing data	Minimum	Maximum	Mean	Std. deviation	Remark
Jan	13	0	13	0.00	102.00	19.78	29.23	No Outlier
Feb	13	0	13	0.00	178.00	26.91	49.11	No Outlier
Mar	13	1	12	0.00	125.30	75.51	41.38	No Outlier
Apr	13	0	13	59.70	294.10	164.78	59.91	No Outlier
May	13	1	12	8.30	181.20	101.06	55.08	No Outlier
Jun	13	0	13	36.30	176.50	87.31	40.39	No Outlier
Jul	13	1	12	20.10	381.00	113.72	98.34	No Outlier
Aug	13	0	13	31.70	267.60	133.55	55.41	No Outlier
Sep	13	0	13	56.20	254.20	122.58	48.83	No Outlier
Oct	13	0	13	19.40	240.00	108.48	65.90	No Outlier
Nov	13	0	13	0.00	203.00	38.28	53.27	No Outlier
Dec	13	2	11	0.00	83.60	23.79	27.34	No Outlier

XLSTAT 2017.02.44369 - Grubbs test for outliers - Start time: 6/3/2017 at 3:18:19 PM / End time: 6/3/2017 at 3:18:55 PM

Data: Workbook = BookGAS.xlsx / Sheet = Robe / Range = Robe!$A$1:$M$14 / 13 rows and 13 columns

Alternative hypothesis: Two-sided

Significance level (%): 5

Iterations: Maximum: 1

Number of simulations: 1000000

Run again:

Summary statistics:

Variable	Observations	Obs. with missing data	Obs. without missing data	Minimum	Maximum	Mean	Std. deviation	Remark
Jan	13	0	13	0.00	77.60	26.70	23.90	No Outlier
Feb	13	2	11	0.00	68.70	16.14	21.60	No Outlier
Mar	13	0	13	0.00	135.30	60.33	42.71	No Outlier
Apr	13	2	11	46.40	174.60	112.07	42.04	No Outlier
May	13	2	11	22.70	132.60	71.21	29.39	No Outlier
Jun	13	1	12	23.90	110.00	62.15	25.43	No Outlier
Jul	13	0	13	44.10	167.00	97.57	36.79	No Outlier
Aug	13	2	11	69.90	249.30	140.07	55.07	No Outlier
Sep	13	2	11	83.00	138.60	111.31	17.23	No Outlier
Oct	13	0	13	34.20	195.00	91.22	43.83	No Outlier
Nov	13	0	13	3.50	109.90	38.02	32.37	No Outlier
Dec	13	0	13	0.20	83.90	23.04	27.19	No Outlier

XLSTAT 2017.02.44369 - Grubbs test for outliers - Start time: 6/3/2017 at 3:57:50 PM / End time: 6/3/2017 at

Data: Workbook = Book111.xlsx / Sheet = Ginir / Range = Ginir!$A$1:$M$14 / 13 rows and 13 columns

Alternative hypothesis: Two-sided

Significance level (%): 5

Iterations: Maximum: 1

Number of simulations: 1000000

Run again:

Summary statistics:

Variable	Observations	Obs. with missing data	Obs. without missing data	Minimum	Maximum	Mean	Std. deviation	Remark
Jan	13	0	13	0.00	200.50	39.98	59.342	No Outlier
Feb	13	0	13	0.00	118.40	17.42	40.278	No Outlier
Mar	13	0	13	0.00	193.30	54.69	61.242	No Outlier
Apr	13	0	13	66.10	288.10	185.18	62.711	No Outlier
May	13	0	13	31.00	270.50	102.75	63.425	No Outlier
Jun	13	0	13	2.70	80.40	34.92	20.141	No Outlier
Jul	13	0	13	0.00	45.80	17.82	14.787	No Outlier
Aug	13	0	13	0.00	68.60	34.46	23.972	No Outlier
Sep	13	0	13	30.70	135.20	84.55	35.717	No Outlier
Oct	13	0	13	84.50	301.60	191.78	66.165	No Outlier
Nov	13	0	13	3.20	241.30	59.82	64.819	No Outlier
Dec	13	0	13	0.00	93.10	35.25	32.050	No Outlier

XLSTAT 2017.02.44369 - Grubbs test for outliers - Start time: 6/3/2017 at 4:13:11 PM / End time: 6/3/2017 at 4:13:47 F								
Data: Workbook = Hydrology Weyib (Alemkerem) (Autosaved)f (Autosaved).xls / Sheet = AK / Range = AK!$A$2:$M$15 / 13 rows and 13 columns								
Alternative hypothesis: Two-sided								
Significance level (%): 5								
Iterations: Maximum: 1								
Number of simulations: 1000000								
Summary statistics:								
Variable	Observations	Obs. with missing data	Obs. without missing data	Minimum	Maximum	Mean	Std. deviation	Remark
Jan	13	0	13	0.80	6.19	1.62	1.46	No Outlier
Feb	13	1	12	0.27	2.61	1.08	0.68	No Outlier
Mar	13	0	13	0.66	8.51	2.10	2.12	No Outlier
Apr	13	3	10	1.10	19.94	6.55	5.51	No Outlier
May	13	0	13	1.68	18.53	8.08	4.69	No Outlier
Jun	13	1	12	1.50	25.69	6.99	7.72	No Outlier
Jul	13	1	12	2.18	44.75	13.85	11.51	No Outlier
Aug	13	6	7	6.95	39.63	26.80	12.65	No Outlier
Sep	13	1	12	6.85	32.83	19.28	6.91	No Outlier
Oct	13	4	9	5.60	108.91	27.43	31.37	No Outlier
Nov	13	2	11	1.40	19.56	7.08	5.59	No Outlier
Dec	13	0	13	1.30	12.72	4.23	3.75	No Outlier

Table 25 Dinsho estimated data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	0.00	70.70	66.20	188.10	324.30	288.80	135.00	218.90	329.40	414.00	217.90	74.60	193.99
1993	137.00	180.00	7.00	281.60	147.60	184.40	290.00	74.83	147.00	290.00	23.10	5.90	147.37
1994	0.00	7.20	51.20	383.70	404.10	103.20	329.00	290.50	91.10	21.80	145.30	57.80	157.08
1995	0.00	69.90	218.60	254.00	57.05	65.90	188.00	163.70	88.60	163.00	14.60	50.20	111.13
1996	33.40	14.10	83.80	222.70	115.70	96.70	225.00	135.00	146.00	47.10	52.10	2.50	97.84
1997	27.80	0.00	101.00	190.90	63.70	85.10	125.00	85.44	85.44	317.00	226.10	77.10	115.38
1998	115.00	26.00	92.20	122.90	85.60	61.00	143.00	285.80	194.20	266.00	53.60	0.00	120.44
1999	24.60	5.80	162.70	104.70	93.50	84.10	187.00	218.00	136.50	60.50	33.40	36.10	95.58
2000	0.00	0.00	45.10	174.40	73.20	47.10	184.00	228.00	94.60	169.00	18.50	48.70	90.22
2001	0.00	0.00	101.20	103.40	147.00	55.03	314.00	88.70	58.80	105.00	34.00	9.80	84.74
2002	0.00	2.80	156.60	117.30	67.63	44.90	88.50	89.10	71.50	99.40	6.30	164.00	75.67
2003	8.40	0.00	14.90	167.30	37.80	86.90	150.00	145.60	132.10	44.70	36.60	50.80	72.93
2004	0.00	0.00	4.60	193.23	147.00	70.06	20.17	164.70	160.77	22.73	28.23	115.87	77.28

Table 26 Goro estimated data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	91.26	91.26	0.00	168.80	144.30	91.26	46.60	95.60	139.40	161.00	79.40	89.00	99.82
1993	33.00	155.00	3.60	303.60	197.80	21.60	0.00	0.00	0.00	74.83	0.00	0.00	65.79
1994	0.00	0.00	31.10	115.10	279.70	30.80	28.20	55.90	74.90	203.00	66.10	17.10	75.16
1995	0.00	16.00	118.70	289.10	113.50	13.80	42.00	32.90	90.10	159.00	2.40	14.50	74.33
1996	10.20	0.00	117.40	394.80	219.30	87.10	11.90	77.10	67.30	105.00	13.40	82.61	98.84
1997	12.40	0.00	53.20	210.30	57.30	59.90	6.80	112.40	72.80	294.00	183.70	40.60	91.95
1998	153.00	63.20	2.00	98.00	215.10	9.40	79.26	5.50	60.90	154.00	37.50	0.00	73.15
1999	0.00	0.00	58.90	125.40	220.60	8.50	52.00	88.40	145.10	409.00	52.60	0.00	96.71
2000	0.00	0.00	71.45	71.45	71.45	71.45	0.00	71.45	71.45	71.45	71.45	71.45	53.59
2001	0.00	0.00	54.93	54.93	54.93	54.93	0.00	54.93	54.93	54.93	54.93	54.93	41.20
2002	0.00	0.00	67.63	67.63	67.63	67.63	67.63	0.00	67.63	67.63	67.63	67.63	50.73
2003	0.00	0.00	59.35	59.35	59.35	59.35	59.35	59.35	59.35	59.35	59.35	59.35	49.46
2004	0.00	0.00	70.06	70.06	70.06	70.06	0.00	70.06	70.06	70.06	70.06	70.06	52.54

Table 27 Robe estimated data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	49.20	91.26	27.10	98.80	104.00	62.20	52.20	201.30	103.00	101.00	58.70	45.00	82.81
1993	43.40	68.70	0.00	118.40	87.20	59.00	58.00	168.10	131.80	102.00	4.90	0.20	70.14
1994	0.00	0.00	28.60	75.63	73.90	55.10	100.00	168.00	111.10	130.00	76.60	4.50	68.62
1995	0.00	17.10	38.20	155.00	51.60	23.90	167.00	112.90	107.30	70.80	4.70	14.30	63.57
1996	30.20	32.80	106.10	174.60	82.61	110.00	109.00	82.61	82.61	34.20	62.50	9.70	76.41
1997	6.30	0.00	88.80	135.40	49.40	59.20	81.90	75.80	118.40	127.00	109.90	43.90	74.67
1998	77.60	79.26	38.30	95.00	79.26	50.10	99.00	92.00	138.60	195.00	51.70	1.90	83.14
1999	10.30	34.50	135.30	60.50	66.90	40.70	147.00	140.10	95.20	106.00	15.00	1.40	71.08
2000	15.10	0.00	13.90	71.90	132.60	71.45	44.10	249.30	83.00	97.40	28.40	9.50	68.05
2001	22.20	10.90	118.10	63.60	72.50	77.40	119.00	111.20	123.80	66.80	26.70	7.20	68.28
2002	29.60	2.30	89.80	46.40	54.30	35.70	67.60	69.90	120.20	45.70	3.50	83.90	54.08
2003	7.70	0.00	55.60	163.00	68.20	103.50	98.60	152.20	92.00	49.20	37.20	64.20	74.28
2004	55.50	11.20	44.50	110.70	22.70	69.00	125.00	70.06	70.06	60.70	14.50	13.80	55.64

Table 28 Gasera estimated data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	2.50	50.00	0.00	166.50	113.50	150.30	381.00	267.60	254.20	36.00	25.50	47.40	124.54
1993	43.80	178.00	0.00	201.10	159.30	98.20	26.90	136.90	76.40	151.00	0.00	74.83	95.54
1994	0.00	0.00	54.00	294.10	68.70	81.10	195.00	110.50	85.20	69.10	71.70	75.63	92.09
1995	0.00	32.60	122.60	219.10	59.10	36.30	148.00	141.40	126.30	92.60	5.00	0.00	81.92
1996	21.60	13.80	125.30	191.50	181.20	99.90	39.90	132.20	118.50	19.40	29.40	3.80	81.38
1997	5.40	0.00	85.90	179.70	56.20	62.90	74.70	118.80	56.20	188.00	203.00	13.10	86.99
1998	102.00	43.60	31.10	85.40	87.60	46.00	118.00	154.60	114.20	175.00	14.40	0.00	80.99
1999	0.00	0.00	92.20	59.70	147.10	94.30	118.00	96.70	117.20	240.00	18.30	0.00	81.96
2000	1.70	0.00	26.70	180.10	71.45	54.30	99.40	177.10	119.60	117.00	42.50	8.10	74.83
2001	0.00	31.80	91.70	116.40	179.70	104.20	63.80	160.70	129.50	81.60	25.70	26.30	84.28
2002	24.30	0.04	116.20	145.10	65.20	60.10	79.80	31.70	113.30	92.40	0.00	83.60	67.65
2003	14.50	0.00	105.40	136.00	86.80	176.50	59.35	131.30	107.00	30.20	42.80	53.30	78.60
2004	41.40	0.00	55.00	167.40	8.30	70.90	20.10	76.70	175.90	118.00	19.40	26.10	64.93

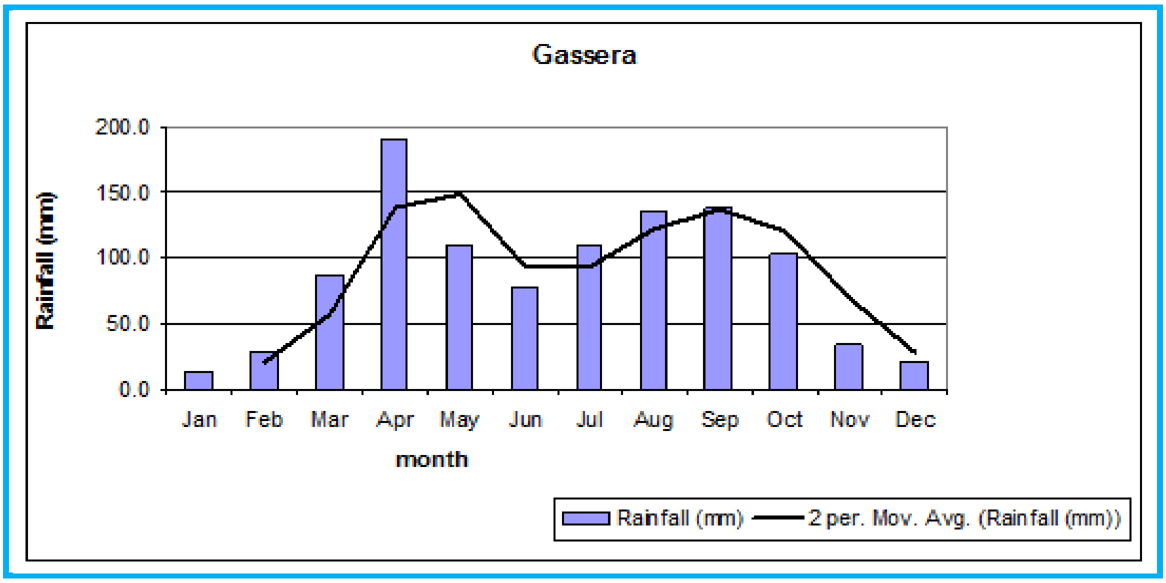

Table 29 Ginir station rainfall data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	84.80	3.00	0.00	206.10	157.80	24.50	29.60	67.20	125.20	241.70	86.70	68.50	91.26
1993	73.80	118.40	0.00	143.60	31.20	24.80	16.60	48.20	135.20	291.40	3.20	11.60	74.83
1994	0.00	0.00	41.00	252.10	103.30	35.60	28.30	25.80	63.30	197.20	125.00	36.00	75.63
1995	0.00	8.60	193.30	161.80	82.50	25.70	12.10	2.70	30.70	142.60	16.00	8.60	57.05
1996	29.20	0.00	87.80	288.10	270.50	80.40	23.40	46.50	53.10	84.50	22.90	4.90	82.61
1997	0.00	0.00	19.90	158.20	31.00	61.20	12.30	22.90	105.70	301.60	241.30	71.20	85.44
1998	200.50	96.50	18.60	188.80	137.50	36.00	39.00	9.00	53.10	145.10	25.70	1.30	79.26
1999	0.00	0.00	125.80	151.50	73.00	18.30	45.80	26.70	95.40	163.00	26.50	0.00	60.50
2000	0.00	0.00	22.00	279.30	119.20	29.70	0.00	55.00	74.90	213.00	40.80	23.50	71.45
2001	0.00	0.00	60.90	66.10	101.30	36.60	1.10	57.30	114.50	183.00	17.00	22.50	55.03
2002	44.50	0.00	127.50	133.10	43.50	24.60	6.30	0.00	122.00	243.00	26.00	41.10	67.63
2003	0.00	0.00	10.30	159.80	105.50	53.90	17.10	68.60	32.40	103.00	68.50	93.10	59.35
2004	85.80	0.00	3.90	218.90	79.50	2.70	0.00	18.10	93.70	184.00	78.10	76.00	70.06

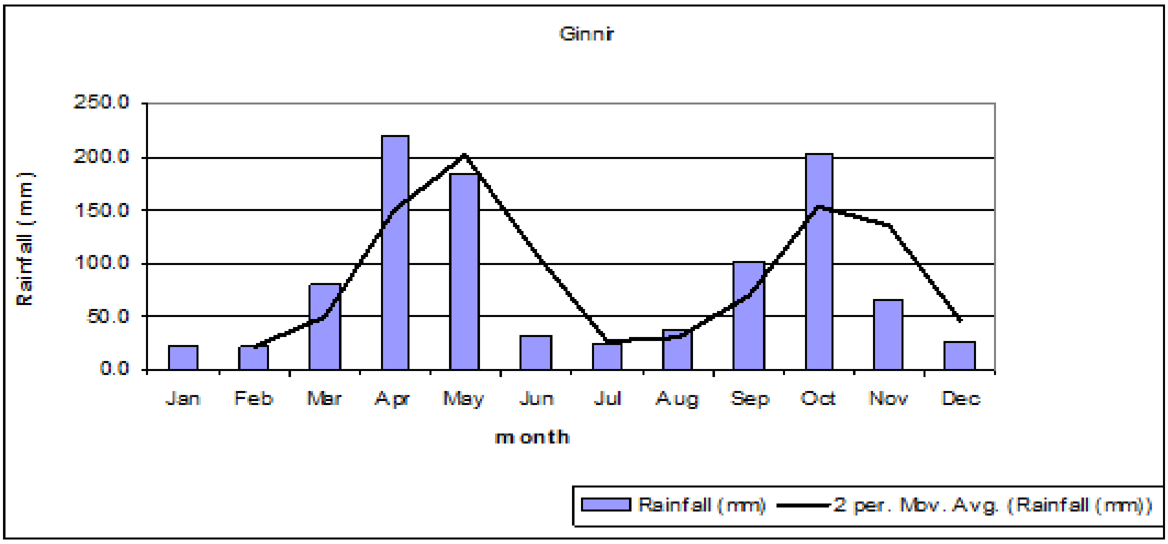

Table 30 Robe station rainfall data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	49.20		27.10	98.80	104.00	62.20	52.20	201.30	103.00	101.00	58.70	45.00	82.05
1993	43.40	68.70	0.00	118.40	87.20	59.00	58.00	168.10	131.80	102.00	4.90	0.20	70.14
1994	0.00	0.00	28.60		73.90	55.10	100.00	168.00	111.10	130.00	76.60	4.50	67.98
1995	0.00	17.10	38.20	155.00	51.60	23.90	167.00	112.90	107.30	70.80	4.70	14.30	63.57
1996	30.20	32.80	106.10	174.60		110.00	109.00			34.20	62.50	9.70	74.34
1997	6.30	0.00	88.80	135.40	49.40	59.20	81.90	75.80	118.40	127.00	109.90	43.90	74.67
1998	77.60		38.30	95.00		50.10	99.00	92.00	138.60	195.00	51.70	1.90	83.92
1999	10.30	34.50	135.30		66.90	40.70	147.00	140.10	95.20	106.00	15.00	1.40	72.04
2000	15.10	0.00	13.90	71.90	132.60		44.10	249.30	83.00	97.40	28.40	9.50	67.75
2001	22.20	10.90	118.10	63.60	72.50	77.40	119.00	111.20	123.80	66.80	26.70	7.20	68.28
2002	29.60	2.30	89.80	46.40	54.30	35.70	67.60	69.90	120.20	45.70	3.50	83.90	54.08
2003	7.70	0.00	55.60	163.00	68.20	103.50	98.60	152.20	92.00	49.20	37.20	64.20	74.28
2004	55.50	11.20	44.50	110.70	22.70	69.00	125.00			60.70	14.50	13.80	52.76

Table 31 Gasera station rainfall data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	2.50	50.00		166.50	113.50	150.30	381.00	267.60	254.20	36.00	25.50	47.40	135.86
1993	43.80	178.00	0.00	201.10	159.30	98.20	26.90	136.90	76.40	151.00	0.00		97.42
1994	0.00	0.00	54.00	294.10	68.70	81.10	195.00	110.50	85.20	69.10	71.70		93.58
1995	0.00	32.60	122.60	219.10	59.10	36.30	148.00	141.40	126.30	92.60	5.00	0.00	81.92
1996	21.60	13.80	125.30	191.50	181.20	99.90	39.90	132.20	118.50	19.40	29.40	3.80	81.38
1997	5.40	0.00	85.90	179.70	56.20	62.90	74.70	118.80	56.20	188.00	203.00	13.10	86.99
1998	102.00	43.60	31.10	85.40	87.60	46.00	118.00	154.60	114.20	175.00	14.40	0.00	80.99
1999	0.00	0.00	92.20	59.70	147.10	94.30	118.00	96.70	117.20	240.00	18.30	0.00	81.96
2000	1.70	0.00	26.70	180.10		54.30	99.40	177.10	119.60	117.00	42.50	8.10	75.14
2001	0.00	31.80	91.70	116.40	179.70	104.20	63.80	160.70	129.50	81.60	25.70	26.30	84.28
2002	24.30	0.00	116.20	145.10	65.20	60.10	79.80	31.70	113.30	92.40	0.00	83.60	67.64
2003	14.50	0.00	105.40	136.00	86.80	176.50		131.30	107.00	30.20	42.80	53.30	80.35
2004	41.40	0.00	55.00	167.40	8.30	70.90	20.10	76.70	175.90	118.00	19.40	26.10	64.93

Table 32 Dinsho station rainfall data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992	0.00	70.70	66.20	188.10	324.30	288.80	135.00	218.90	329.40	414.00	217.90	74.60	193.99
1993	137.00	180.00	7.00	281.60	147.60	184.40	290.00		147.00	290.00	23.10	5.90	153.96
1994	0.00	7.20	51.20	383.70	404.10	103.20	329.00	290.50	91.10	21.80	145.30	57.80	157.08
1995	0.00	69.90	218.60	254.00		65.90	188.00	163.70	88.60	163.00	14.60	50.20	116.05
1996	33.40	14.10	83.80	222.70	115.70	96.70	225.00	135.00	146.00	47.10	52.10	2.50	97.84
1997	27.80	0.00	101.00	190.90	63.70	85.10	125.00			317.00	226.10	77.10	121.37
1998	115.00	26.00	92.20	122.90	85.60	61.00	143.00	285.80	194.20	266.00	53.60	0.00	120.44
1999	24.60	5.80	162.70	104.70	93.50	84.10	187.00	218.00	136.50		33.40	36.10	98.76
2000		0.00	45.10	174.40	73.20	47.10	184.00	228.00	94.60	169.00	18.50	48.70	98.42
2001	0.00		101.20	103.40	147.00		314.00	88.70	58.80	105.00	34.00	9.80	96.19
2002	0.00	2.80	156.60	117.30		44.90	88.50	89.10	71.50	99.40	6.30	164.00	76.40
2003	8.40	0.00	14.90	167.30	37.80	86.90	150.00	145.60	132.10	44.70	36.60	50.80	72.93
2004	0.00		4.60	193.23	147.00		20.17	164.70	160.77	22.73	28.23	115.87	85.73

Table 33 Goro station rainfall data

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1992				168.8	144.3		46.6	95.6	139.4	161	79.4	89	115.51
1993	33.00	155.00	3.6	303.6	197.8	21.6	0.00	0.00	0.00		0.00	0.00	64.96
1994	0.00	0.00	31.1	115.1	279.7	30.8	28.2	55.9	74.9	203	66.1	17.1	75.16
1995	0.00	16.00	118.7	289.1	113.5	13.8	42	32.9	90.1	159	2.4	14.5	74.33
1996	10.20	0.00	117.4	394.8	219.3	87.1	11.9	77.1	67.3	105	13.4		100.32
1997	12.40	0.00	53.2	210.3	57.3	59.9	6.8	112.4	72.8	294	183.7	40.6	91.95
1998	153.00	63.20	2.00	98	215.1	9.4		5.5	60.9	154	37.5	0.00	72.60
1999	0.00	0.00	58.9	125.4	220.6	8.5	52	88.4	145.1	409	52.6	0.00	96.71
2000	0.00	0.00											0.00
2001	0.00	0.00											0.00
2002	0.00	0.00											0.00
2003	0.00	0.00											0.00
2004	0.00	0.00											0.00