



DEVELOPING OPTIMAL RESERVOIR OPERATION
(CASE OF KOKA MULTIPURPOSE RESERVOIR, ETHIOPIA)

MSC THESIS

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DECLARATION

I, **Gemechu Shiferaw Belachew**, hereby declare that this is my original work and I have not and will not submit it to any other university for consideration of a similar degree or other equivalent awards. All sources of information used in the creation of this thesis have been properly acknowledged.

Signature_____

Date_____

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Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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ABBREVIATIONS AND ACRONYMS

ALT	Alternatives
AMSL	Above Mean Sea Level
ARBA	Awash River Basin Authorities
Arc-GIS	Aeronautical Reconnaissance Coverage Geographic Information System
ASD	Above Site Datum
CFCAS	Canadian Foundation for Climate Atmospheric Science
CP	Computational Point
DEM	Digital Elevation Model
DMI	Domestic, Municipal, and Industry
EEP	Ethiopian Electric Power
FSL	Full Supply Level
HEC-DSS	Hydrologic Engineering Center- Data Storage System
HEC-HMS	Hydrologic Engineering Center- Hydrologic Modelling System
HEC-ResSim	Hydrologic Engineering Center- Reservoir Simulation Model
MOL	Minimum Operation Level
MW	Mega Watt
MWR	Ministry of Water Resources
MWh	Mega Watt hour
NMA	National Meteorological Agency
NSE	Nash Sutcliffe Efficiency
R ²	Coefficient of Determination
RMSE	Root Mean Square Error
USACE	United States of Army Corps of Engineering
WEAP	Water Evaluation And Planning

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ABSTRACT

Reservoir operation is the most challenging task in the management of water resources systems. Hence, water resources models are needed to minimize these challenges, as much as possible if they are supplemented with sound engineering judgments. In this study, the rainfall-runoff simulation HEC-Hydrologic Model System (HEC-HMS) and reservoir simulation model (HEC-ResSim) were applied to the koka multipurpose reservoir to optimize the power fluctuation observed and water scarcity due to reduced water release from the reservoir. Inflow was generated by the HEC-HMS model by using twenty years of daily meteorological data collected from NMA. The generated inflow was calibrated and validated with 16 years of observed flow data from Melka Kunture, Hombole, and Mojo gauging stations. The performance of the model was evaluated by NSE, R², RMSE, and PBIAS performance indices criteria reviewed in different literature. For example, the NSE value of Melka Kunture, Hombole, Mojo, and Koka Inlet is 0.75, 0.78, 0.63, and 0.8 for calibration, and 0.7, 0.72, 0.53, and 0.75 for validation, respectively. The value of R² for Melka Kunture, Hombole, Mojo and Koka Inlet is also 0.74, 0.77, 0.64 and 0.77 for calibration and 0.74, 0.75, 0.52 and 0.76 for validation respectively. Although the model slightly underestimates the flow, for both calibration and validation, the model shows acceptable performance to generate an inflow of the upper awash watershed. To simulate the reservoir the inflow generated by the HEC-HMS model, reservoir physical, and operational data were collected from governmental organizations and provided to the model. Since the HEC-ResSim model cannot optimize the constraints directly, trial and error have been applied through a prioritization rule between three main demands; i.e., Hydropower, Irrigation, and Domestic, Municipal and Industrial demands. The best alternative was selected based on the power target, release target, and pool elevation target. Accordingly, from the three alternatives applied in the simulation, ALT-1 gives maximum power and maximum release that supports the downstream water needs. When power demand was given the highest rule priority, the reservoir generate an average energy of 504.76MWh per day or 55GWh per year which is greater than the power generated in ALT-2 and ALT-3 by 32% and 64%, respectively. The reservoir reaches its minimum elevation of 103.93m in June (except in the drought year of 2003 and 2016) and its maximum elevation of 110.39m in August. Generally, the reservoir can support the downstream water needs safely if the operation will be conducted by the power demand priority rule.

Key Words: HEC-HMS, HEC-ResSim, Flow Generation, Reservoir Operation, Rule Priority

1. INTRODUCTION

1.1. Background of the Study

Fresh water is the world's most essential but limited resource of the 21st century. These priceless resources need proper management because it is not uniformly dispersed throughout time and place (Adom and Simatele 2021; Al. 2022; Ampitiyawatta 2020; Katusiime and Schütt 2020;). Water resource systems management encompasses planning, developing, distributing, controlling, maintaining, and managing water resources and their related infrastructures to optimize the utilization of water. Managing a water resource system is a significant task, including building a new system, expanding the current system, and operating an existing one (Aghdaie 2019; Jebbo and Awchi 2016). Therefore, management of this resource is a complex and varied topic that requires consideration of a broad range of social, economic, and environmental interests (Seyoum and Theobald 2014). One of the main means of managing and controlling the water resource systems is through the reservoirs. Reservoirs are the most efficient structure in the management and development of water resources constructed for water supply, irrigation, domestic, industrial, hydropower generation, and flood mitigation (Abdulateef et al., 2021; Ahmad et al., 2014).

The Operation and management of a reservoir are one of the most challenging tasks in the management of a water resources system since the problem deals with many complicated variables and uncertainties, such as inflows, return flows, storages, diversions, inter/intra-basin water transfers, irrigation, and industrial and/or municipal water supply demands (Fayaed S.Sabah 2013; Rani and Madalena 2010). One of the objectives of reservoir operation is to ensure adequate water supply for different purposes, but it is not clear how to achieve this goal, especially in multi-purpose reservoirs because depletion of stored water or inflow shortage into the reservoir and hence conflicts may occur between purposes. Therefore, an effective and efficient reservoir operation may be needed by planners and operators with multi-objective management that consider water supply, water allocation, and flooding problems. (Jebbo and Awchi 2016; López 2020).

Nowadays, planners and operators need new technologies that can be used to quickly develop alternative decisions by representative models. The time frame for decision-making may be extremely short, the information available is generally scarce, and the predictability of the meteorological situation is very limited. Therefore, much literature abounds with many techniques which may help dam operators in reservoir management, setting up and using them

allows for exploring different alternative management scenarios for water resources planning and management (López 2020). The two approaches widely adopted for the planning and operation of reservoir systems are simulation and optimization. Application of these models for determining the optimal operating policy for reservoirs is very important in water resources planning and management (Ivanova et al. 2017). The choice of approach in analyzing the problem depends upon the availability of the data, the nature or type of the objective function and its binding constraints, and the system configuration. However, it reveals that no general technique is available to grasp whole water resources optimally (Ampitiyawatta 2020).

Most of the reservoirs in the world still manage predefined operating rules based on different simulation models. Simulation models (Jaiswal 2021) are effective tools for studying the operation of complex physical and hydrological characteristics of a reservoir system including the experience and judgment of operators. They work based on physical relations with a series of operational rules to simulate new conditions and system behavior under a specified policy (Babazadeh et al., 2007). Simulation models can provide a more realistic and detailed representation of reservoir system operations and much lower computational demands than optimization models for all but the most straightforward cases. Simulation models also are more common in practice, and therefore are more likely to be trusted as a standard of comparison (Jay R. Lund, 2014). Therefore this study was conducted using the HEC-ResSim reservoir simulation model.

HEC-ResSim reservoir simulation model is a computer program applicable for hydrologic and hydraulics of reservoir system simulation (Jain 2018). This program simulates reservoir operation, including all characteristics of a reservoir and channel routing downstream. The model also allows users to define alternatives and run simulations simultaneously to compare results (Ziaei and Hosseinipou 2014). Therefore, this research focuses on the operation of the koka multipurpose reservoir through the use of a simple simulation model (HEC-ResSim) to simulate the current power generation and water release for downstream users based on different alternatives.

1.2. Statement of the Problem

Awash River basin is one of the major river basins in Ethiopia and the most utilized River basin so far. Even though this basin is the most utilized river basin in Ethiopia but, still not fully developed and optimally allocated. Currently, the basin faces two extreme challenges; flooding in the rainy season (July to September), and water stress in the dry season (April to June) due to the intensive irrigation development in the basin particularly along the Awash River (ARBA, 2017). The intense rainfall in the uplands of the basin flows to the reservoir and is stored in the early days of the rainy season. This stored water causes flooding to the settlements found across the stretch of river courses when appropriate provisions weren't taken.

The second major challenges in the basin is the increase of water demand which enhance the conflicts among all the users due to less rainfall during dry season. Currently, there are many small to large irrigated farms found in this area which is entirely dependent on water release from the Koka reservoir during dry months. All downstream requirement during dry season was made based on the average reservoir release of 40m³/sec (ARBA, 2017) . Therefore, Hydropower and downstream irrigation needs are in conflicts, because, serving both of these functions requires a tradeoff between purposes that is defined by the target storage level of the reservoir. So there is an expectation that the water released from Koka reservoir may not meet fully meet the demands of the basin in the near future.

Therefore, to solve these problems, Koka reservoir needs proper operation and management as improper operation of reservoirs will lead to technically and economically inefficient to meet the desired objectives. Reservoir simulation models have a significant role to generate maximum power and water releases for downstream water needs. This study attempts to estimate the effective utilization of koka reservoir water uses based on reservoir simulation with the help of the HEC-ResSim model.

1.3. Objectives

1.3.1. General Objective

The main objective of this study is to develop reservoir operation rules for optimal water use of Koka Reservoir.

1.3.2. Specific Objectives

- ❖ To generate reservoir inflow from the upper awash watershed using HEC-HMS Model.
- ❖ To simulate the optimum power and water release that can meet downstream irrigation demand using the HEC-ResSim model.
- ❖ To develop the operational guide curves of the koka reservoir that can meet the downstream water needs.

1.4. Research questions

- ❖ How much inflow yield is estimated from the upper awash watershed?
- ❖ Does the Koka reservoir operate at its optimal condition to generate the estimated power?
- ❖ Does the Koka Reservoir supply enough water for all of the sectors downstream the area regarding current scenarios?

1.5. Significance of the Study

The study contributes to the efficient operation and management of the Koka dam through the important outputs consecutively evaluated under current operation condition. The study also develops a reservoir operation curve that will serve as a decision-making tool for planners and operators under varying hydrological conditions without any conflicts between hydropower and irrigation with efficient utilization of the reservoir storage. Generally, this work indicates the options for the operation of the reservoir that can minimize the possible conflict that arises in hydropower generation and different downstream water users by proper allocation of water between users without significant effect on the water resource of the basin.

1.6. Scope of the study

The scope of this research work is limited to determining the reservoir operation guide curves to optimize power and release from koka reservoir using historical hydrometeorological data., The reservoir simulation model (HEC-ResSim) and rainfall-runoff simulation model (HEC-HMS) were implemented after data were collected and analyzed. The study did not consider the impact of upstream structures, sedimentation and climate change on reservoir operation .

2. LITERATURE REVIEW

2.1. Reservoirs

Reservoirs are an important infrastructure for water resources management, providing social and economic benefits such as hydropower production, water supply, drought and flood control, irrigation, and recreation (Ávila et al. 2022). The primary function of the storage reservoir is to smooth out the natural variability of the hydrological system to allow human activity to be supported by a constant, or a regular, seasonally varying supply. Hence reservoirs serve as temporary storages, capturing high flows whose water can then be available for use during periods of low natural flow (Pereira et al., 2002)

Reservoirs can be constructed for a single purpose or for multi-purpose. single purpose reservoirs are developed to serve only one purpose, which may be flood control or any of the conservation uses such as irrigation, power generation, navigation, industrial use, municipal water supply etc. Multi-purpose reservoirs are constructed to serve more than one purpose which may be a combination of any of the conservation uses with or without flood control. The various purposes for which a reservoir is constructed and its functioning are irrigation, power, generation, municipal and industrial, navigation, recreation, water quality control, etc. (Jain 2018; Sinha 2005)

2.2. Principle of Reservoir Operation

Reservoir operation is a method of allocating stored water in the reservoir among different upstream and downstream users based on actual operational experience (Jain & Singh, 2003). The operation and management of a reservoir system are very challenging since the problem deals with many complicated variables and uncertainties, such as inflows, return flows, storages, diversions, inter/intra-basin water transfers, irrigation, and industrial and/or municipal water supply demands. The decision process that identifies the operations of any system is also, complicated due to conflicts existing among the various purposes and the choice of modeling technique usually depends on the reservoir-specific system characteristics, data availability, the objectives specified, and the constraints imposed for the reservoir (Rani and Madalena, 2010). According to the study of Shemeles (1998) the selection of modeling techniques should depend on the operation procedure that will best achieve the stated objective(s) to solve the optimal water releases at one or more. It was customary for a long time to establish operating rules based on personal judgment alone. As water resource systems

more complex, a concrete decision rule is needed between reservoirs, users, and purposes. The rules were, generally simple:

- ❖ Store all inflow unless needed to meet a target output;
- ❖ When available, release water from storage to fulfill immediate needs;
- ❖ Study all damaging floods on record in the flood control analysis.

Therefore, the complexity of a reservoir operation problem is dependent on the compatibility of the various intended purposes. If they are compatible, less effort is needed for coordination. Efficient operation of multiple reservoirs requires a high degree of intelligence, intuition, and experience to ensure that they are not left partially empty at the end of the rainy season, or be found full at the time of arrival of a series of peak floods, which lead to heavy releases causing floods in the downstream areas (Mamman, 2018). However, there is no universal solution for reservoir operation problems. Therefore, it is important to study the problems and determine optimal reservoir operation guides for each scheme.

2.3. Multi-Purpose Reservoir Operation

Reservoirs can be used for multiple purposes such as irrigation, municipal and industrial water supply, hydropower generation, flood protection, water quality management, recreation, low-flow augmentation, and so on. Therefore, multipurpose reservoir operation needs to address various interactions and trade-offs between the objectives, which are sometimes competitive or conflicting. For example, releases may be required for hydropower generation, at the same time as releases need to be restricted for the prevention of downstream flooding. To solve the conflicts between the objectives, an optimization model can be applied to determine the optimal release decisions of a system. An objective function is required to set up an optimization to achieve the water management goals in which the different sub-objectives are considered. The optimal solution can be calculated by giving a relative penalty to each of these sub-objectives to indicate the relative importance of the sub-objectives. (Lin and Rutten 2016). For example, a reservoir such as a koka multipurpose reservoir that provides irrigation, power generation, and flood control can cause competition and conflicts between objectives of the water uses. The following conflicts may be raised while operating a multipurpose reservoir such as koka (Jain, 2018; Sinha, 2005)

Conflict in Space: These types of conflicts occur when a reservoir is required to satisfy divergent purposes, for example, water conservation and flood control. The conservational demands are best served when the reservoir is as fuller as possible at the end of the filling

period. On the other hand, for flood control purposes, empty storage space in the reservoir needs to be maximized for safely absorbing the flood waters.

Conflict in Time: The temporal conflicts in reservoir operation occur when the use pattern of water varies with the purpose. The conflicts arise because release for one purpose does not agree with that for the other purpose. For example, irrigation demands may show one pattern of variation depending upon the crops, season, and rainfall; while the hydroelectric power demands may have a different variation. In such situations, the aim of deriving an operating policy is to optimally resolve these conflicts.

Conflict in Discharge: The conflicts in daily discharge are experienced for a reservoir, which serves more than one purpose. In the case of a reservoir serving for consumptive use and hydroelectric power generation, the releases for the two purposes may vary considerably in the span of one day. Many times, a small conservation pool is created on the river downstream of the powerhouse, which is used to dampen the fluctuations in releases for meeting varying power demands.

In general, the operation of a multi-purpose reservoir should be governed by how various uses of the reservoir have been combined. In simple terms, one can state that the objective of operating such systems is to manipulate supply to reduce competition among demands and to increase complementarity among them and the intention of the author is also similar to this.

2.4. Reservoir operation policy

Reservoir operating policies are based on dividing the total storage capacity into designated pools or vertical zones. These zones are used to regulate the release of water from storage at any time of the year depending on the reservoir state, likely inflows to the reservoir, water demands, release capacity, and downstream constraints for example, the river channel capacity (Khan and Tingsachali, 2010). The reservoir operation policy also guides the reservoir to ensure improved overall performance in terms of its reliability and vulnerability. This policy is mostly used in planning studies, particularly for firm yield studies but is rarely used in day-to-day operations because of its rigidity. However, Revelle (1999) proposed the use of a rationing rule to minimize the drawback caused by this rule, especially in the event of drought.

Generally, Reservoir operation policies typically divide the total storage capacity into designated pools (Ralph A. Wurbs 2005). A typical reservoir consists of one or more of the vertical zones, or pools, illustrated by

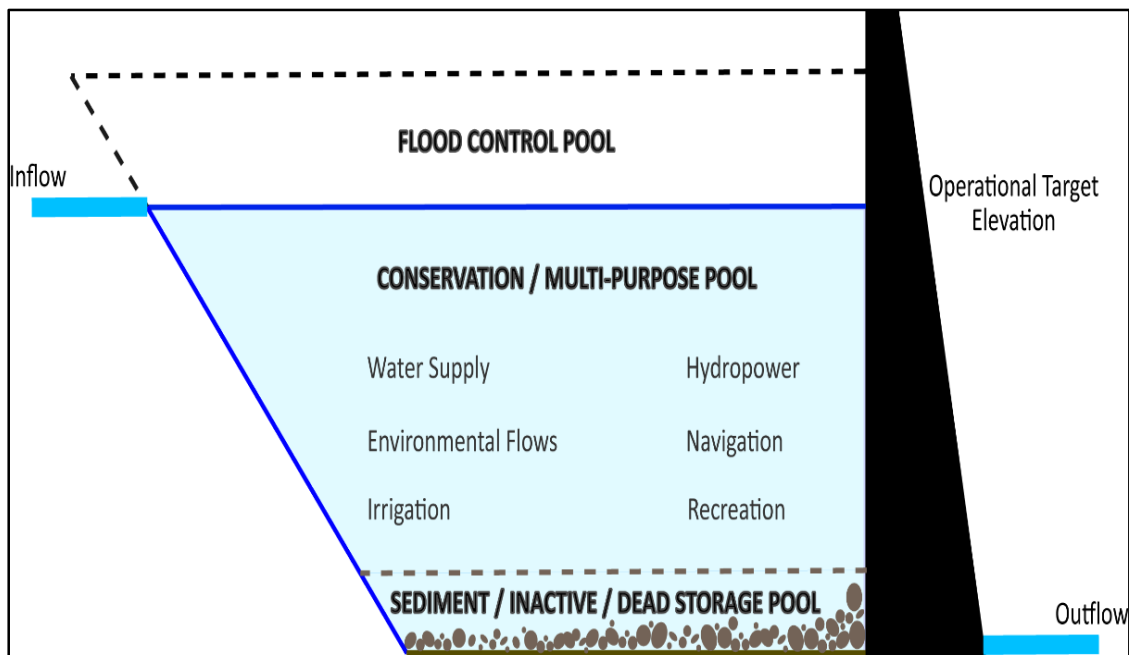


Fig 2. 1 Reservoir Pool Zones

Surcharge Zone: is generally the storage space above the flood control zone between the flood control level and maximum water level. This space is occupied mostly during high floods and the releases from this zone are due to a trade-off between structural safety and downstream flood damages.

Flood Control Zone: This is the storage space marked as temporary storage for absorbing high flows for alleviating downstream flood damages. This space should be emptied as soon as possible to negotiate the next flood event. Gated spillways allow the top of the flood control pool to exceed the spillway crest elevation

Conservation Zone: This storage space is used for the conservation of water for meeting various demands such as municipal and industrial water supply, irrigation, navigation, hydroelectric power, and in-stream flow maintenance. The reservoir water surface is maintained at or as near the designated top of the conservation pool elevation as stream flows and water demands allow. Drawdowns are made as required to meet the various water needs. This zone is generally between flood control level and minimum operation (down sill level).

Buffer Zone: This is the storage space above the dead storage level which is used to satisfy only very essential water needs in case of extreme situations.

Dead Storage Zone: This storage space is also called an inactive zone. This is the lowest zone in which the storage is meant to absorb some of the sediments entering the reservoir. Water releases or withdrawals are normally not made from the inactive pool, except through the

natural processes of evaporation and seepage. It may provide a portion of the sediment reserve, head for hydroelectric power, and water for recreation and fish habitat.

2.5. Reservoir Rule/Guide Curve

A rule curve is a graphical representation specifying ideal storage or space planned to be achieved in a reservoir, under different conditions of probabilities of inflows and/or demands, during various periods in a year. They apply to reservoirs being operated in a steady-state condition. There are several types of rules but each indicates the desired or required reservoir release or storage volumes at any particular time of year. Some rules identify storage volume targets (rule curves) that the operator is to maintain, if possible, and others identify storage zones, each associated with a particular release policy (Loucks and van Beek 2017). The Rule Curves are generally derived by operation studies using historic or generated flows in case long-term historic flows are not available. Many times due to various conditions like low inflows, minimum requirements for meeting demands, etc., it may not be possible to stick to the target level stipulated by Rule Curve. In such a situation as far as possible, the reservoir level should be brought to the stipulated target level at the earliest simple rule curve should base the release of the next time solely on the current storage level and the current time of the year (Sinha 2005)

2.6. Hydrological System modeling

A model is a simplified representation of the real-world system. Hydrological models represent the physical/ chemical/biological characteristics of the catchment and simulate the natural hydrological processes (Moradkhani and Sorooshian 2008). Hydrological models are essentially mathematical models where the physical processes of the hydrologic cycle are described by a set of mathematical equations, logical statements, boundary conditions, and initial conditions, expressing relationships between inputs, variables, and parameters (Jain 1993).

Hydrological models are used to develop stream flow hydrographs required as input for the stream hydraulics models and the reservoir/river system operation models. Generally, hydrological models simulate the hydrologic processes by which precipitation is converted to stream flow. There is no single way to classify the hydrological models. Based on the criteria of interest to describe and discuss strength, capability, and limitations, the hydrological models can be classified in different ways. For example, many scholars (such as Bergström and Graham, 1998; Cunderlik, 2003; Devia et al., 2015; Jain, 2018; Moradkhani, and Sorooshian,

2008) divided the hydrological model based on different criteria/characteristics in one way or another. Accordingly, Based on the description of the physical process, the hydrological models can be classified as empirical (data-driven models), conceptual, and physical, based on the spatial representation hydrological model may be lumped, semi-distributed or distributed and based on the aspect of randomness it can be deterministic, or stochastic.

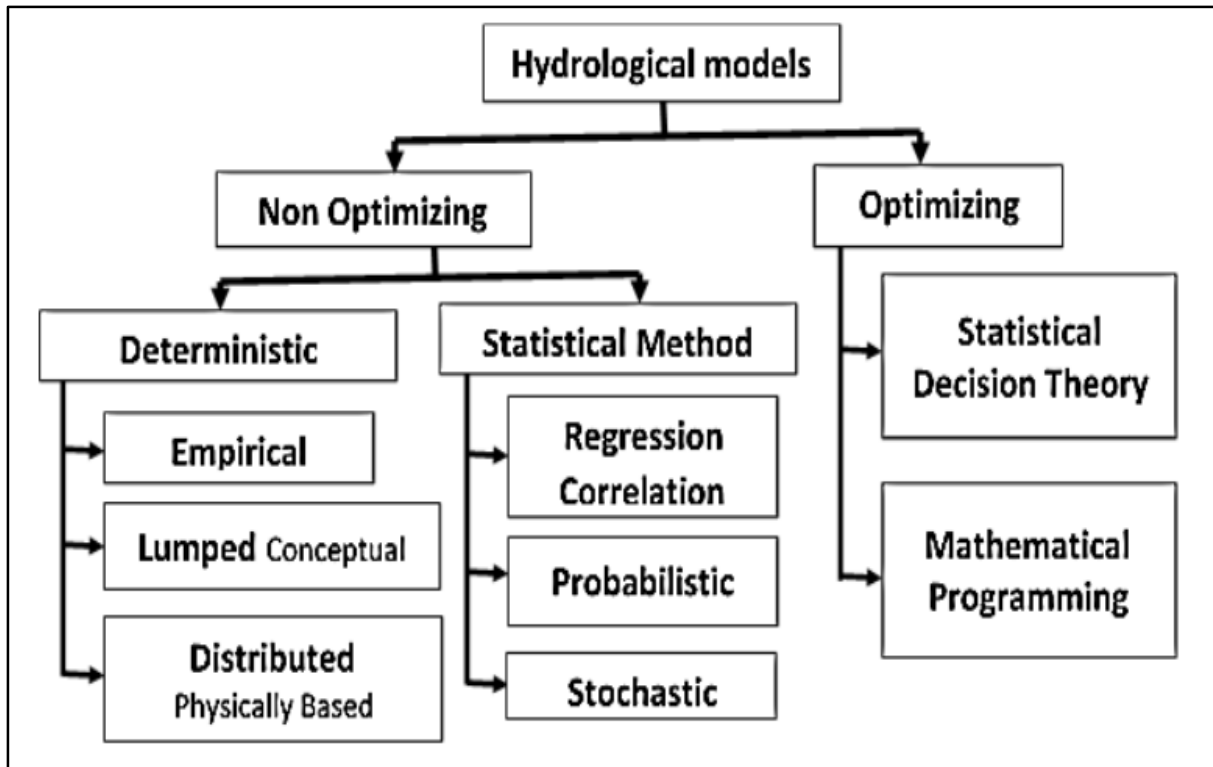


Fig 2. 2 Classification of Hydrological Models (Jain 2018)

In this review, one of the most important classifications based on the process was discussed as follows.

Empirical models (Data Driven model)

These are observation-oriented models which take only the information from the existing data without considering the features and processes of the hydrological systems hence these models are also called data-driven models. This model is also known as a “black box” model because it involves mathematical equations derived from concurrent input and output time series and not from the physical processes of the catchment. Unit hydrograph model, Linear regression and gauge to gauge correlation models, Autoregressive (AR) models (e.g., ARX, ARMAX), and Hydro-informatics based models those based on the concept of working neurons (e.g., Artificial Neural Network (ANN), fuzzy logic, and genetic algorithms) are the common types of the empirical model. (Devia et al., 2015; Jain, 2018)

Lumped methods (Parametric models)

Parameters of lumped hydrologic models do not vary spatially within the basin and thus, basin response is evaluated only at the outlet, without explicitly accounting for the response of individual sub-basins. If the interest is primarily in the discharge prediction only, then these models can provide just as good simulations as complex physically based models(Knights et al, 2017). Many conceptual models have been developed with varying degrees of complexity. E.g., Streamflow Synthesis and Reservoir Regulation (SSARR), Hydrologiska Byråns Vattenbalansavdelning (HBV) and TANK models are among a few. (Devia et al., 2015; Jain, 2018)

Semi-distributed models: Parameters of semi-distributed (simplified distributed) models are partially allowed to vary in space by dividing the basin into many smaller sub-basins. The main advantage of semi-distributed models is that their structure is more physically based than the structure of lumped models and that they are less demanding on input data than fully distributed models. (Cunderlik 2003). A drawback of the semi-distributed model is the manipulation of input data. For example, spatially-distributed rainfall data must be averaged within the sub-area, or rain gauge data at specific locations must be distributed to the area using the Thiessen Polygon method. TOPMODEL and HEC-HMS models are semi-distributed conceptual model which uses land surface slope and soil characteristics to sub-divide the catchment. (Knights et al, 2017)

Distributed models: Parameters of distributed models are fully allowed to vary in space at a resolution usually chosen by the user. Distributed models generally require large amounts of (often unavailable) data for parameterization in each grid cell. However, the governing physical processes are modeled in detail, and if properly applied, they can provide the highest degree of accuracy. MIKE SHE model and SWAT model are the best examples of a physically based model. (Knights et al, 2017)

2.7. River/Reservoir Systems Modeling

Wurbs (1994) reviewed different river/reservoir operation models that have been developed for the operation of the reservoirs and river systems. Various types of input data are required for reservoir system analysis models. Essentially all models require, for each reservoir, a relationship between elevation, storage volume, and water surface area. These models mainly required to allocate of reservoir storage capacity and stream flow between multiple uses and users to;

- ❖ minimizing the risks and consequences of water shortages and flooding;
- ❖ optimizing the beneficial use of water, energy, and land resources; and
- ❖ Minimizing adverse environmental impacts.

To evaluate the system's performance, it is necessary to have a mathematical model, which is a simplified and rational representation of reality. The model conceptualizes the real system and makes the actual situation less complex. Using the models, various alternative systems and policies can be evaluated without interfering with the real system or having a prototype. The mathematical models provide a link between the description of the system and electronic computers, utilizing operational mathematical techniques. The most popularly followed approaches are simulation and optimization (Sinha 2005)

2.7.1. Simulation Techniques

Simulation is a modeling technique used to imitate the behavior of a system on a computer, representing all the characteristics of the system largely by a mathematical or algebraic description. Simulation models provide the response of the system to certain inputs, which include decision rules that allow the decision-makers to test the performance of existing systems or a new system without actually building it. A typical simulation model for a water resources system is simply a model that simulates the interval-by-interval operation of the system with specified inflows at all locations during each interval, specified system characteristics, and specified operating rules (Husain 2012). They also, compute reservoir storage levels, releases, and discharges at pertinent stream locations for specified sequences of hydrologic inputs (stream flow and reservoir evaporation rates), demands for releases or withdrawals for beneficial purposes, and operating rules. Constraints such as storage capacities, outlet and conveyance capacities, and requirements for maintaining minimum stream flows, are also reflected in the models (Wurbs 1991). Generally, simulation models permit a very detailed and realistic representation of the complex physical, economic, and social characteristics of a reservoir system. The concepts inherent in the simulation approach are easier to understand and communicate than other modeling concepts. These are some of the advantages simulation models have over other types of reservoir analysis (Simonovic 1992).

2.7.2. Optimization Techniques

Optimization is the science of choosing the best solution from many possible alternatives, without having to evaluate beyond all possible alternatives. Optimization implies the use of an appropriate optimization model in conjunction with an optimization algorithm. Optimization

methods find out a set of decision variables such that the objective function is optimized (Sinha 2005). They are formulated in terms of determining values for a set of decision variables that will maximize or minimize an objective function subject to constraints. The objective function and constraints are represented by mathematical expressions as a function of the decision variables. For a reservoir operation problem, the decision variables are typically release rates and end-of-period storage volumes. Constraints typically include storage capacities and other physical characteristics of the reservoir stream system, diversion or stream flow requirements for various purposes, and mass balances (Jain 1993). In a broad sense, optimization includes human judgment, the use of simulation and/or optimization models, and the use of other decision-support tools. Optimization models automatically search for an optimal set of decision variable values (Labadie 2004). The most common optimization algorithm adopted for solving water resource problems is linear programming, dynamic programming, nonlinear programming, and goal programming.

2.7.3. Simulation vs Optimization Techniques

There is a distinction between simulation and optimization modeling. The simulation addresses the "what if" question, whereas optimization addresses the "what should be" question. Optimization models need explicit expressions of objectives. Unlike simulation models, optimization models are based on objective functions of unknown decision variables that are to be maximized or minimized. Optimization is often useful, not for finding the very best solution, but for eliminating the worst alternatives from further consideration (Loucks and van Beek 2017). Optimization models aim to identify optimum decisions for system operation that maximizes certain given objectives while satisfying the system constraints. On the other hand, simulation models are used to explore only a finite number of decision alternatives so that the optimum solution may not necessarily be achieved. However, many simulation models now involve a certain degree of optimization and the difference between the optimization and simulation models is becoming less distinct. For a given operating criteria, the performance of a reservoir system may be evaluated by analyzing the computed time sequence of levels, storage, discharges, hydropower, etc. (Husain 2012).

All optimization models also simulate the system. An optimization approach may involve numerous iterative executions of a simulation model, possibly with the iterations being automated to various degrees. Mathematical programming algorithms are embedded within many reservoir/river system simulation models to perform certain computations. Alternative simulation models may be applied in essentially the same manner even though one incorporates

an optimization algorithm to perform certain computations and the other does not (Danielaini 2011). In practice simulation models are an important tool for reservoir systems planning and management studies because they are usually based on mass balance equations and represent the hydrological behavior of reservoir systems using inflows and other operating conditions (Rani and Madalena 2010).

Generally, simulation models permit a very detailed and realistic representation of the complex physical, economic, and social characteristics of a reservoir system. The concepts inherent in the simulation approach are easier to understand and communicate than other modeling concepts. These are some of the advantages simulation models have over other types of reservoir analysis. (Simonovic 1992).

2.8. Hydrological and Reservoir Simulating Models

The structure or behavior of the system under study is typically so complicated that an analytical solution is not attainable. However, more recently, the trend has developed general simulation models that can be applied to any basin or reservoir system (McCartney and Lacombe 2011). Some of the common and most applicable hydrologic/reservoir simulation models are:

2.8.1. VIC model (Variable Infiltration Capacity model)

It is a semi-distributed grid-based hydrology model which uses both energy and water balance equations. The main inputs are precipitation; minimum and maximum daily temperature and wind speed which allows many land cover types within each model grid. VIC simulates saturation excess runoff by considering soil heterogeneity and precipitation. It consists of three layers. The top layer allows quick soil evaporation, the middle layer represents a dynamic response of soil to rainfall events and the lower layer is used to characterize the behavior of soil moisture. (Devi et al. 2015).

2.8.2. TOPMODEL

Topmodel is a rainfall-runoff model that has its origins in the recognition of the dynamic nature of runoff-contributing areas in the 1960s and 1970s. It is a conceptual model designed to capitalize on information that is connected to runoff. It can be applied to a single unit or multiple units of sub-basins with the aid of elevation data (usually in the grid) for the drainage basin area. The model can also be referred to as a variable contributing area conceptual model as its parameters can be measured theoretically. It helps in the prediction of the hydrological

behavior of basins. The major factors considered in this model are the catchment topography and soil transmissivity (Abdulkareem et al. 2018).

2.8.3. SWAT model (Soil and Water Assessment Tool)

SWAT is a semi-distributed, continuous watershed modeling system, which simulates different hydrologic responses using process-based equations. The model computes the water balance from a range of hydrologic processes such as evapotranspiration, snow accumulation, snowmelt, infiltration, and generation of surface and subsurface flow components. The model is physically based, computationally efficient, and capable of continuous simulation over long periods. Major model components include weather, hydrology, soil temperature and properties, plant growth, nutrients, pesticides, bacteria and pathogens, and land management (Jain 1993; P. W. Gassman et al. 2007).

2.8.4. HEC-HMS Model

The HEC-HMS software simulates many hydrologic processes such as infiltration, evapotranspiration, snowmelt, soil moisture accounting, etc. for continuous simulation and procedures such as unit hydrograph and hydrologic routing. Advanced capabilities are also provided for gridded runoff simulation using the linear quasi-distributed runoff transform (Mod Clark). Supplemental analysis tools are provided for model optimization, forecasting stream flow, depth area reduction, assessing model uncertainty, erosion, sediment transport, and water quality (Jain 2018).

2.8.5. River Ware Model

River Ware can model the physical (hydrologic) processes associated with reservoirs, the river reaches, gains and losses, distribution canals, agricultural consumptive uses, shallow groundwater interaction, and conjunctive use. It can model some water quality constituents such as total dissolved solids, temperature, and dissolved oxygen. Man-made infrastructure such as spillways and outlet works, hydropower plants at dams, inline power and pumping plants, diversion structures, wells, and distribution canals, and associated energy generated or consumed are also represented (Zagona 2016)

2.8.6. River Simulation Model (RIBASIM)

RIBASIM is a generic model package to simulate the hydrological behavior of river basins for varying current and future hydrological, climate, and anthropogenic conditions. The model is a comprehensive and flexible tool to link the hydrologic water inputs at various locations in a basin to the diverse water-using activities taking place at the same analytical scale. By

processing this combination of inputs, RIBASIM can evaluate a large set of system modeling options and outputs, for example, measures related to infrastructure and operational management, changes in water distribution and consumption patterns across sectors, and changes in hydro-meteorological conditions. The model provides an efficient handling and structured analysis of the large amounts of data commonly associated with (complex) water resources systems. (van der and Boccalon, 2013)

2.8.7. Water Evaluation and Planning Model (WEAP)

The Water Evaluation and Planning (WEAP) System was developed and distributed by the Stockholm Environmental Institute Boston Center in Boston, Massachusetts. WEAP is a reservoir/river/use system water balance accounting model that allocates water from surface and groundwater sources to different types of demands. The modeling system is designed as a tool for maintaining water balance databases, generating water management scenarios, and performing policy analyses. The model also, places the demand side of the equation water use patterns, equipment efficiencies, re-use, prices, hydropower energy demand, and allocation on an equal footing with the supply side stream flow, groundwater, reservoirs, and water transfers. (Sieber, 2005; Ralph A Wurbs, 2005b).

2.8.8. MODSIM Model

MODSIM is a generic river basin management decision support system originally conceived in 1978 at Colorado State University, making it the longest continuously maintained river basin management software package currently available. MODSIM is designed as a generalized river basin management decision support system (DSS) designed as a computer-aided tool for developing improved basin-wide and regional strategies for short-term water management, long-term operational planning, drought contingency planning, water rights analysis, and resolving conflicts between urban, agricultural, and environmental concerns. The model also provides for integrated evaluation of hydrologic, economic, environmental, and institutional/legal impacts as related to alternative development and management scenarios, including the conjunctive use of surface water and groundwater resources (Labadie, 2010).

2.8.9. HEC-ResSim Model

HEC-ResSim is one such reservoir simulation model. It has been developed by the Hydrologic Engineering Center of the US Army Corps of Engineers to aid engineers and planners in predicting the behavior of reservoir systems in water management studies, and to help reservoir operators plan releases in real-time during day-to-day and emergency operations. The program

represents the physical behavior of reservoir systems with a combination of hydraulic computations for flows through control structures, and hydrologic routing to represent the lag and attenuation of flows through segments of streams. It represents operating goals and constraints with an original system of rule-based logic that has been specifically developed to represent the decision-making process of reservoir operation (Klipsch and Evans 2010)

2.9. Model Selection Criteria

Cunderlik (2003) lists the following criteria which can be used for choosing the right hydrologic model. These criteria are always project dependent since every project has its specific requirements and needs. Each model may be selected based on the capability of the model to answer the following question.

- ❖ Does the model predict the variables required by the project such as peak flow, event volume, hydrograph, long-term sequence flows, etc.? (i.e., the capability to predict the required variables)
- ❖ Is the model capable of simulating regulated reservoir operation, snow accumulation and melt, single event or continuous processes (i.e., its estimation for desired output)
- ❖ Can all the inputs required be provided within the time and cost constraints of the project? (i.e, input data availability)
- ❖ Does the investment appear to be worthwhile for the objective of the project? (i.e., whether the model commercial or free version)

2.10. Description of Selected Models

Each model type serves a certain purpose, and a particular model type may not categorically be considered more appropriate than others in all situations. The choice of a suitable model structure relies heavily on the function that the model needs to serve. Hydrological model selection depends on the objective of the modeler and the availability of hydro-meteorological as well as other necessary data. In this study, the selection of a model mainly depends on the criteria set by (Cunderlik 2003). Furthermore, it may depend on the availability of data, the purpose of the study, the required output accuracy, the size of the catchment area required to be studied, the simplicity of the model, the versatility of the model, the compatibility of the model with another model, powerfulness of the model, and the previous application of the model in the study area were also considered. Therefore, considering all the criteria mentioned above and the researcher's interaction with the model, the hydrologic simulation model (HEC-

HMS) and reservoir simulation model (HEC-ResSim) models were selected for this study particular study. The detailed description of these models will be discussed as follows.

2.10.1. HEC-HMS Hydrological Model

HEC-HMS (Hydrologic Engineering Center-Hydrological Modeling System) is the United State Corps of Engineers' hydrologic system computer program developed by the Hydrologic Engineering Center (HEC). The HEC-HMS model is a generalized modeling system capable of representing many different watersheds. The model is developed to simulate the precipitation-run-off processes of dendritic watershed systems using observed or/and predicted precipitation to generate stream flow hydrographs at pertinent locations. The model is also, applied across a wide range of geographic areas to simulate large river basin water supply and flood hydrology, as well as supporting small urban or natural watershed run-off modeling. The model is very adaptable because it includes a variety of model choices for each segment of the hydrologic cycle. It has been used in many studies for achieving goals in flood damage reduction, reservoir and system operation, floodplain regulation, environmental restoration, and water supply planning, among others (Scharffenberg et al., 2010).

HMS Model Components

HEC-HMS model components are used to simulate the hydrologic response in a watershed. In HEC-HMS, a project consists of a basin model, a meteorological model, and control specifications. HMS model components consist of the following four models. (Fleming and Brauer, 2015)

i. Basin Model Components

The main purpose of the basin model is to convert atmospheric conditions into stream flow at a specific location in the watershed. The hydrologic elements of a basin model such as sub-basin, reach, junction, sink, etc., are used to break the watershed into manageable entities. HEC-HMS consists of four mathematical models, for the calculation of Precipitation-Runoff-Routing hydrological processes simulation.

HMS contains the following four main mathematical models.

- ❖ Models that compute runoff volume
- ❖ Models of direct runoff,
- ❖ Models that estimate base flow (overland flow and interflow)
- ❖ A model that calculates channel flow

Models that compute runoff volume (direct runoff models): this model can account for overland flow, storage, and energy losses as water runs off a watershed and into the stream channel. The systems type transformation included in HMS consists of methods such as Kinematic wave, Mod Clark, SCS unit hydrograph, Snyder unit hydrograph, User-specified s-graph, and User-specified unit hydrograph

Models of direct runoff (Loss Models): The loss models in HEC-HMS normally calculate the runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. Deficit and constant (also gridded), Exponential, Green and Ampt (also gridded), Initial and constant, SCS curve number (also gridded), Smith Parlange and Soil moisture accounting (also gridded) are the most known base flow calculation methods available in HEC-HMS model.

Models that estimate Base flow: Base flow is the sustained or "fair-weather" runoff of prior precipitation that was stored temporarily in the watershed, plus the delayed subsurface runoff from the current storm. Some conceptual models of watershed processes account explicitly for this storage and the subsurface movement. However, this accounting is not necessary to provide the information for many water resources studies.

HEC-HMS includes three alternative models of base flow:

- ❖ Constant, monthly-varying value.
- ❖ Exponential recession model.
- ❖ Linear-reservoir volume accounting model.

A model that calculates channel flow (Routing Model): The reach element is used to convey stream flow downstream in the basin model. Inflow into the reach element can come from one or many upstream hydrologic elements. Outflow from the reach is calculated by accounting for translation and attenuation of the inflow hydrograph. Channel routing in HEC-HMS is estimated by one of the following methods. The channel routing methods available in HMS include Lag; Modified Pulls, Muskingum, Kinematic wave, and Muskingum Cunge.

ii. Meteorologic Model Component

The meteorologic model is responsible for preparing the boundary conditions that act on the watershed during a simulation. The meteorologic model is prepared to use with one or more basin models. If the basin model contains sub-basin elements, then the meteorologic model must specify how precipitation will be generated for each sub-basin. Evapotranspiration should be included in the meteorologic model when the basin model is configured for a continuous

simulation. Methods available in the HEC-HMS model for estimation of basin average precipitation or grid cell precipitation are Frequency Storm, Gage Weights, Gridded Precipitation, Inverse Distance, SCS Storm, Specified Hyetograph, etc.

iii. Control Specifications Component

Control specifications are one of the main components of the hydrology project. Their major purpose is to control when simulations start and stop, and what time interval is to be used in the HEC-HMS hydrology simulation. The control specifications also include the time interval that will be used to perform computations during a simulation. The time interval defined in the control specification will also be used in displaying time-series results from the simulation.

iv. Input Data Components

Time-series data (Precipitation gages, Discharge gages, Stage gages, Temperature gages, Solar radiation gages), paired data (Storage-discharge functions, Elevation-storage functions, Elevation-area functions, Elevation-discharge functions, Inflow-diversion functions), and gridded data (Precipitation grid sets, Temperature grid sets, Solar radiation grid sets, Crop coefficient grid sets, Storage capacity grids) are often required as a parameter or boundary conditions in the basin. Input data can be entered manually or referenced to an existing record in an HEC-DSS file.

Model Parameters Calibration and Validation

Calibration

Unfortunately, it is not, in general, possible to estimate the parameters of models by either measurement or prior estimation. Studies that have attempted to do so have generally found that even using intensive series of measurements of parameter values, the results have not been entirely satisfactory. Prior estimation of feasible ranges of parameters also often results in ranges of predictions that are wide and may still not encompass the measured responses all of the time (Jain 1993). Therefore calibration process finds the optimal parameter values that minimize the objective function. Further, the calibration estimates some model parameters that cannot estimate by observation or measurement, or have no direct physical meaning.

Calibration uses observed hydrometeorological data in a systematic search for parameters that yield the best fit of the computed results to the observed runoff. Calibration can be either manual or automated (optimization). Manual calibration relies on the user's knowledge of the basin's physical properties and expertise in hydrologic modeling (CFCAS, 2004). In the

automated calibration model parameters are iteratively adjusted until the value of the selected objective function is minimized. The systematic search process is called optimization. Optimization begins from initial parameter estimates and adjusts them so that the simulated results match the observed stream flow as closely as possible.

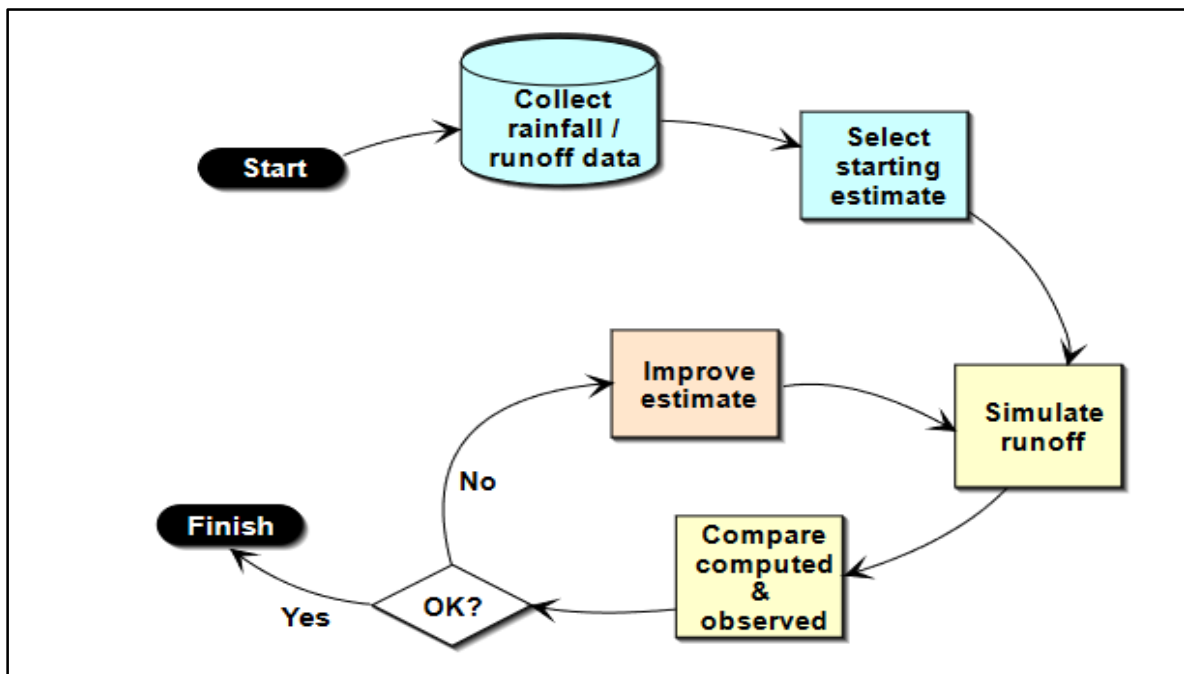


Fig 2. 3 Schematic of Calibration Procedure (USACE 2000)

In HEC-HMS two different search algorithms are provided that move from the initial estimates to the final best estimates: Nelder and Mead search algorithm and the Univariate Gradient search algorithm. A variety of objective functions are provided to measure the goodness of fit between the simulated and observed stream flow in different ways. These objective functions are: peak weighted root mean square error (RMSE), percent error peak, sum absolute errors, and sum squared residuals. The goal of all four calibration schemes is to find reasonable parameters that yield the minimum value of the objective function (USACE, 2000; Yener et al., 2012).

A sum of absolute errors. This objective function compares each ordinate of the computed hydrograph with the observed, weighting each equally. Thus, this function implicitly is a measure of fit of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs. If the value of this function equals zero, the fit is perfect: all computed hydrograph ordinates equal exactly the observed values.

A sum of squared residuals. This is a commonly-used objective function for model calibration. This function is a measure of the comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs.

Percent error in peak. This measures only the goodness-of-fit of the computed-hydrograph peak to the observed peak. This objective function is a logical choice if the information needed for designing or planning is limited to peak flow or peak stages.

Peak-weighted root means square error. This function is an implicit measure of comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs.

Validation Model validation is the process of testing the model's ability to simulate observed data other than those used for the calibration, with acceptable accuracy. During this process, calibrated model parameters are not subject to change, their values are kept constant. The quantitative measure of the match is again the degree of variation between computed and observed hydrographs. One-third of historical data was used for model validation.

2.10.2. HEC-ResSim Reservoir Simulation Model

The Reservoir System Simulation (HEC-ResSim) software was developed by the U.S. Army Corps of Engineers, as a successor to the "HEC-5, Simulation of Flood Control and Conservation Systems" program. HEC-ResSim is a decision support tool that meets the needs of modelers performing reservoir project studies as well as meeting the needs of reservoir regulators during real-time events. HEC-ResSim is unique among reservoir simulation models because it attempts to reproduce the decision-making process that human reservoir operators must use to set releases. Additionally, ResSim is comprised of a graphical user interface (GUI), a computational program to simulate reservoir operation, data storage and management capabilities, and graphics and reporting facilities. ResSim has three sets of functions called modules that provide access to specific types of data within a watershed. These modules are watershed setup, reservoir network, and simulation. Each module has a unique purpose and associated set of functions accessible through menus, toolbars, and schematic elements (Klipsch and Hurst, 2021)

Watershed Setup Module

The objective of the Watershed Setup Module is to provide a common framework for the creation of features of a watershed for modeling with one or more HEC modeling applications. Activities associated exclusively with this module include creating new background maps,

establishing and editing stream alignment, defining watershed configuration, assembling configuration to describe the watershed’s physical features, and viewing lists of schematic elements that we have created.

Reservoir Network Module

The purpose of the reservoir network module is to isolate the development of the reservoir model and alternatives from the analysis results. This module provides the user to create a reservoir network and schematic, describe the physical and operational features of the created network, assemble the alternatives that they want to analyze, and view reports that list your network elements by type

Simulation Module

This module is used to isolate the analysis of results from the model development process. Once the reservoir model (network) was created and at least one alternative was defined, the module was used to create and configure the simulation which compute results, viewed outputs, and made changes to the model as appropriate to achieve the desired operation.

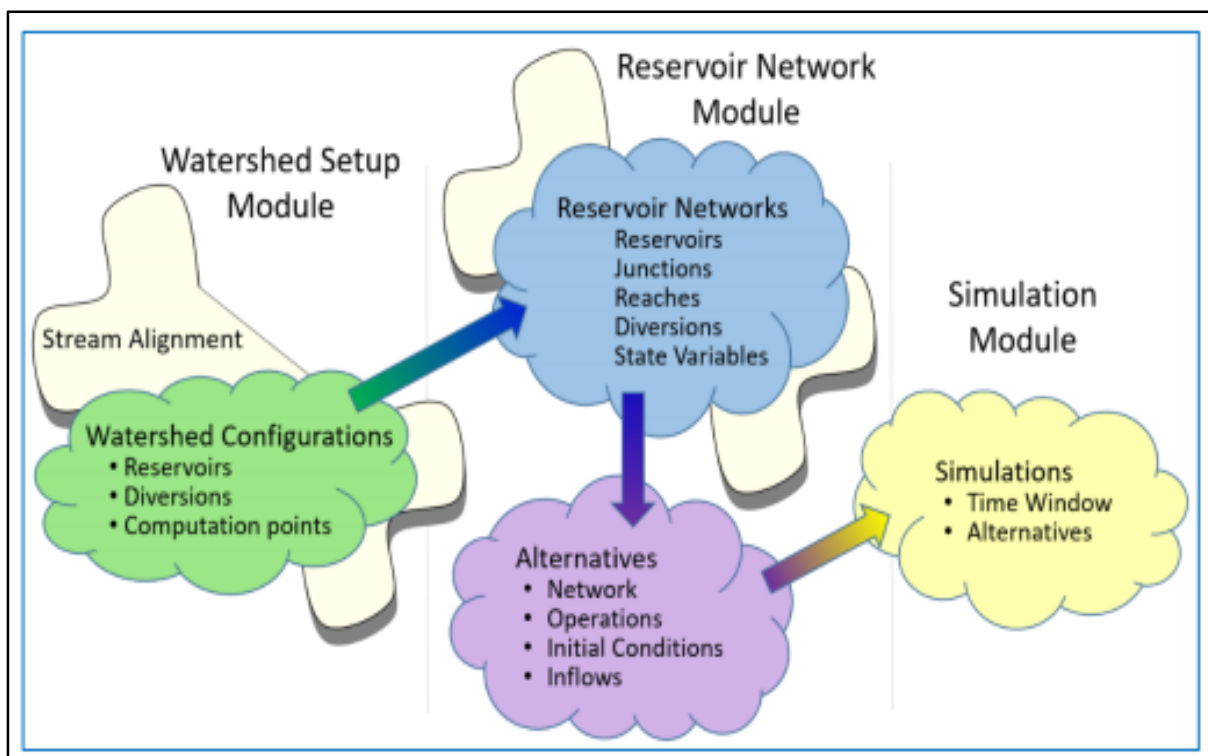


Fig 2. 4 Illustration of Basic Concepts between Modules in HEC-ResSim

2.11. Previous Studies in the Study Area

Many consulting companies, organizations, and scholars have conducted studies concerning water resources planning and management in the awash basin. Their study focuses on specific challenges and the use of particular models. This study has reviewed a few of these studies as follows:

HALCROW (1989) conducted an extensive study on the surface water resources of the Awash River Basin while carrying out the Awash Master Plan study for the development of surface water resources on different topics like climate and hydrology, dams and hydropower, river basin modeling e.t.c. and put forward some important proposals. According to Halcrow's (1989) study the major source of irrigation water for various demands in the basin was the release from the Koka reservoir. The live storage required at Koka can sustain 68800ha of irrigation land 850 Mm³ and 660 Mm³ giving priority either for power generation or irrigation, respectively by considering the annual sedimentation inflow rate until 2008. Still today, most studies conducted on the Awash River basin were based on modeling results of Halcrow (1989) considering the irrigation development scenarios of sustainability and expansion of irrigation area for 40 years by raising the koka dam by three meters plus construction of Kesem and Tendaho dam. Many organizations including EEP use this Halcrow proposal for various studies. However, for reasons such as the quality and quantity of metrology data available at the time and the complex problems that naturally arise, it is very difficult to use this proposal as it is.

Booker Tate & MCE (2003) (cited in (Merga, 2020; Woldemariam, 2009) conducted a study on the Wonji/Shoa sugar factory expansion studies. Their study mainly focuses on the estimation of inflow into koka reservoir using Halcrow's (1989) regression equation, the evaporation of koka reservoir from meteorological data collected from Wonji, Adama, and Batu stations using the penman method, and finally, discusses the impact of irrigation expansion koka reservoir in detail by using Hydro 10 model with two reservoir operation.

Awash Basin Authority (ARBA 2017a), has been formed with many mandates and responsibilities concerning the basin's water resources development and management. In line with its mandate, the Basin Authority initiates 8 years River Basin Strategies Plan to develop and manage the water resources of the basin in a coordinated manner

The main goal of this organization is to allocate water for domestic (18.6 million), livestock (34.4 million), and irrigation (340,000ha) depending on the basin in a sustainable, equitable,

and rational manner at full coverage. According to the study of this authority, the annual water demand of the Basin was estimated as 4.114BCM and this will be forecasted to be 6.56BCM of water in the future. Therefore, to overcome the water scarcity in the basin, the authority allocates water needs through three water development and management strategies.

- ❖ 3.271 BCM with full storage of the capacity of the existing dams
- ❖ 1.160 BCM with the construction of new dams in the basin
- ❖ 1.289 BCM saved with water use efficiency improvement in an irrigated agricultural system
- ❖ 1BCM from groundwater development

Perhaps, there had no well-organized water allocation system in the basin yet except, for the constant water release from Koka reservoir (about 40m³/sec) parallel to produce regulated power throughout the year.

Schemeles (1998) establish operation rule curve for Koka reservoir for the wet season by using Linear programming in MS-FORTRAN language to develop the optimization model and SCS unit hydrograph and curve number concept to model rainfall-runoff. The rainfall-runoff model and the operational rule can be used to facilitate decision-making based on real-time hydrometeorological conditions.

According to his research, the following results have been found:

- ❖ A rainfall-runoff model is developed using the already available and/or easily obtainable hydrometeorological data and has been verified with actually observed data and its performance is satisfactory.
- ❖ The Linear optimization model has been developed which performed well under the given set of objective functions and constraints which output of the model is used to establish the rule curve. This rule curve can effectively be used in conjunction with the rainfall-runoff model for the real-time operation of Koka reservoir.
- ❖ Both the rainfall-runoff and optimization models are quite flexible to be applied with changing conditions like land use, reservoir capacity, demand, etc, whenever such changes do occur.

Therefore, in relative terms, the structure of the models is simple with only a few parameters to determine and their performance is, however, satisfactory

3. MATERIAL AND METHODS

3.1. Study Area Description

The Upper Awash River Basin has located in the highlands of the Ethiopian plateau at an elevation of between 3000 to 1600 m ASL starting from its headwaters at Ginchi 75 km west of Addis Ababa to the Koka Lake (Duguma et al., 2021). The study area covers about 10% of the Awash River basin area which is 110,000 km². The sub-basin lies between the latitude of 8°16" to 9°18" and longitudes of 37°57" to 39°17". There are two functional dams found in Upper Awash Basin: Aba Samuel (1.5 GWh/year) commissioned in 1939, and Awash I (Koka) (110 GWh/year) commissioned in 1960 (Nanesa, 2021). The Koka dam is located at the outlet of the sub-basin at the village of Sire Robi village at about 100 km from the capital city of Ethiopia, Addis Ababa between 8.4684° latitude and 39.1588° longitude. The Dam is one of the oldest dams constructed in late 1960 by the Italian government as a redemption and gesture of apology for invading Ethiopia in the 1935/36 Ethiopian Calendar. At the time it was the largest hydroelectric power plant in Ethiopia which has contributed immensely to the development of the country, still in service. The dam was primarily constructed for hydropower generation, but was later upgraded as a multi-purpose dam for flood regulation during the wet season and ensuring downstream irrigation development downstream during dry season. The dam also supports Awash II and III (165GWh/year each) power stations in addition to the above purposes. (See Fig 3.1)

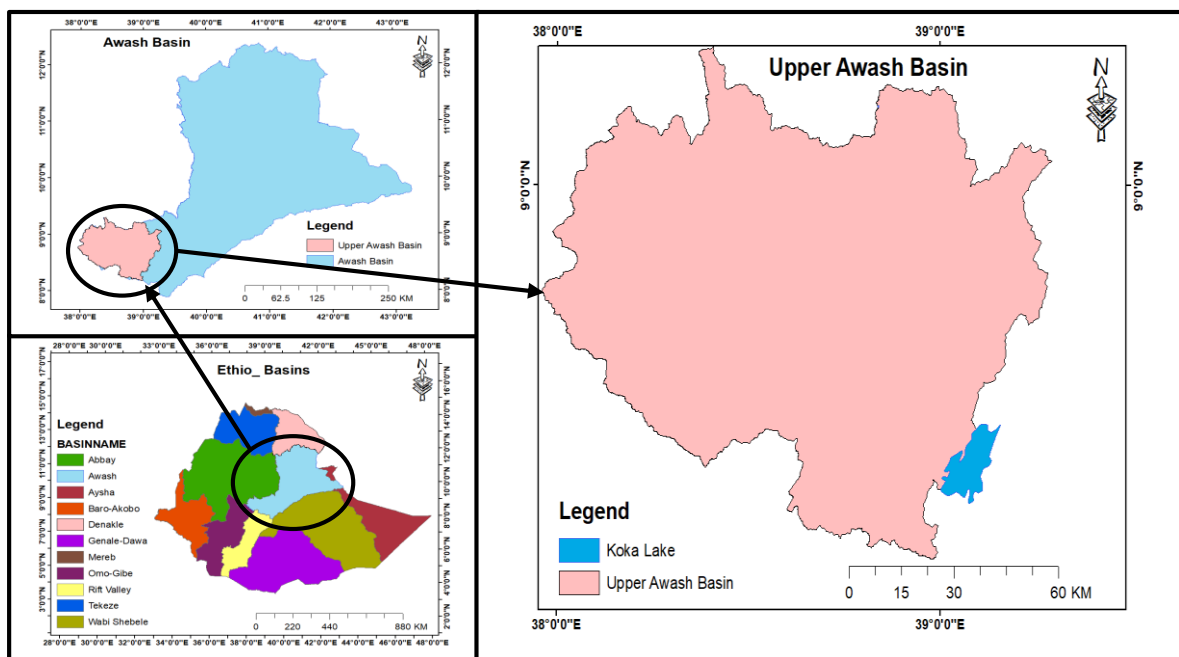


Fig 3. 1 Location Map of Upper Awash Watershed

3.1.1. Topography

The altitude of the Upper Awash River basin is above 3000m a.s.l and rises on the high plateau to the west of Addis Ababa. As one can move from north to south, large elevation drops of about 1400m in 100km distance from north to south are observed. The topography in the area is characterized by moderately dissected to undulating local reliefs. Various interrelated factors like location, altitude, climate, topography, agricultural development, inhabitants, administrative boundaries, etc. contribute to the classification of the basin into zones. Accordingly, HALCROW (1989) divided the whole basin into four distinct zones. These are the upper basin (from headwater to koka dam), upper valley (from koka Reservoir to Awash station), middle valley (from awash station to Mille River), and lower plains (from Mille River to Lake Abe). (See Fig 3.2)

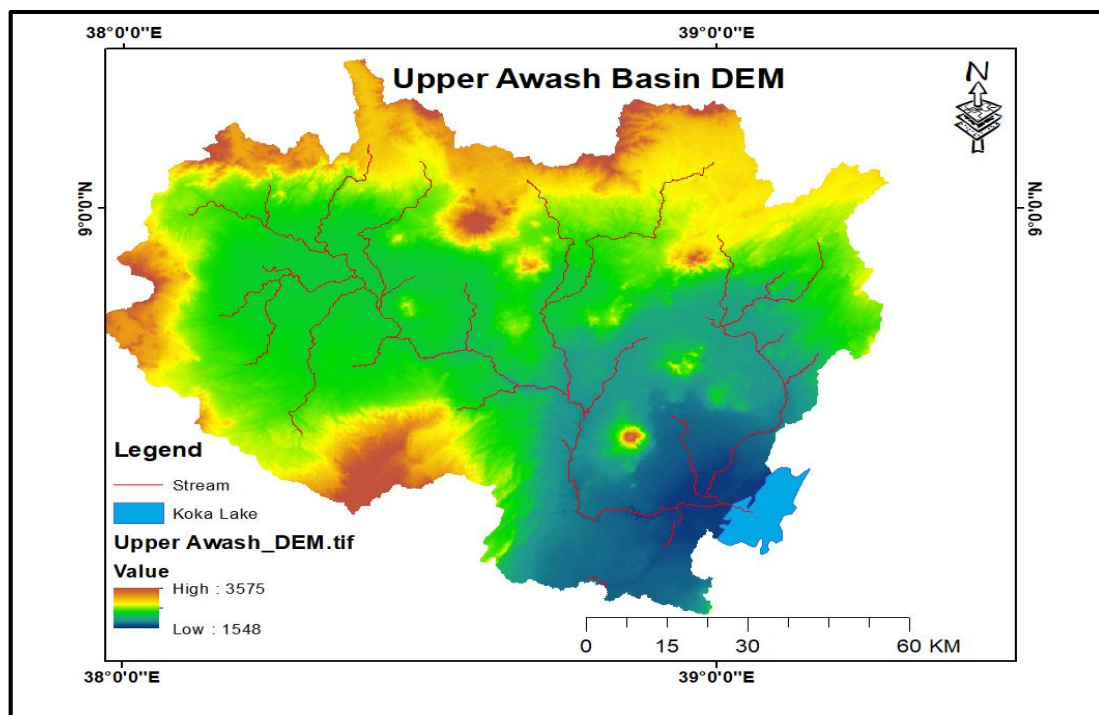


Fig 3. 2 Digital Elevation Model (DEM) of Upper Awash Watershed

3.1.2. Climate

The climate of the Awash Basin is influenced by the Inter-Tropical Convergence Zone (ICTZ), a zone of low pressure that characterizes the convergence of dry tropical easterly and moist southeasterly winds. This convergence is responsible for seasonal rainfall distribution within the basin. As a result of the annual migration of the ITC between May and November, the major rainfall in Ethiopia occurs. Traditionally, the climate of the Awash basin has four seasons: Arfaasaa refers to spring (March, April, and May), Ganna is summer (June, July, and August), Birraa is autumn

(September, October, and November), and Bona is winter (December, January, and February). Arfaasaa and Ganna are wet seasons, while Birraa and Bona are dry seasons, relatively speaking. The heavy rainy season extends to September of the Birraa season (Duguma et al., 2021). The upper awash region gets its main rainy season in July and August when ITCZ is positioned in Northern Ethiopia.

Rainfall

The rainfall of the upper Awash River basin varies with altitude. According to the report of Henock et al. (2014), the annual maximum daily rainfall of the Upper Awash basin varies from 37 mm to 62mm. The major peak rainfall occurred from July to August whereas the minor peak is observed from October to December months. Generally, plateaus between 3000m and 2,500m receive 1,400 - 1,800 mm/year, and regions with altitudes ranging from 1600 to 2500m receive 1 000-1400mm/year. The rainfall distribution of this region is a bimodal type with a mean annual value of greater than 800mm. Fig 3.3 shows the long-term mean rainfall of some selected stations in the upper awash river basin.

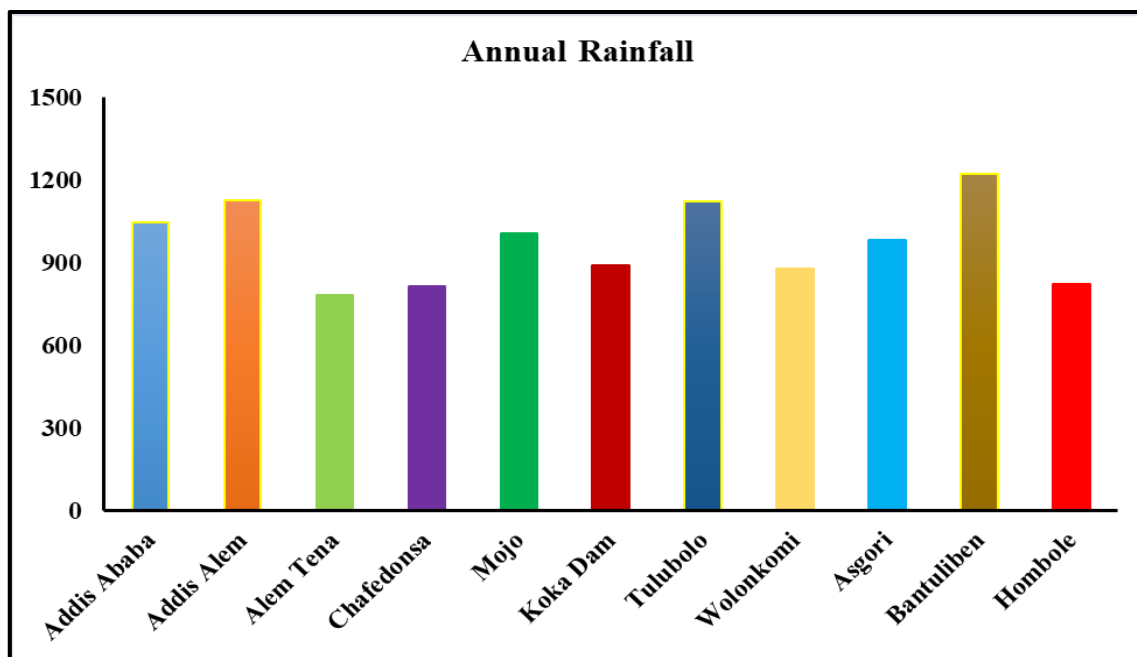


Fig 3. 3 Annual Rainfall of Selected Meteorological Station

Temperature

The temperature of the Awash River basin varies considerably in altitude. The temperature of the Awash River Basin ranges from moist in its upper basin to arid in its lower basin. According to the study of HALCROW (1989) the mean annual temperature ranges from 16.7°C at Addis Ababa to 29°C at Dubti. Based on the eleven selected stations for analysis (See Fig 3.4) the

annual mean, maximum and minimum temperature found in the upper awash was estimated as 16.56°C to 21.39°C, 22.8°C to 27.8°C and 10°C to 14.9°C respectively. See

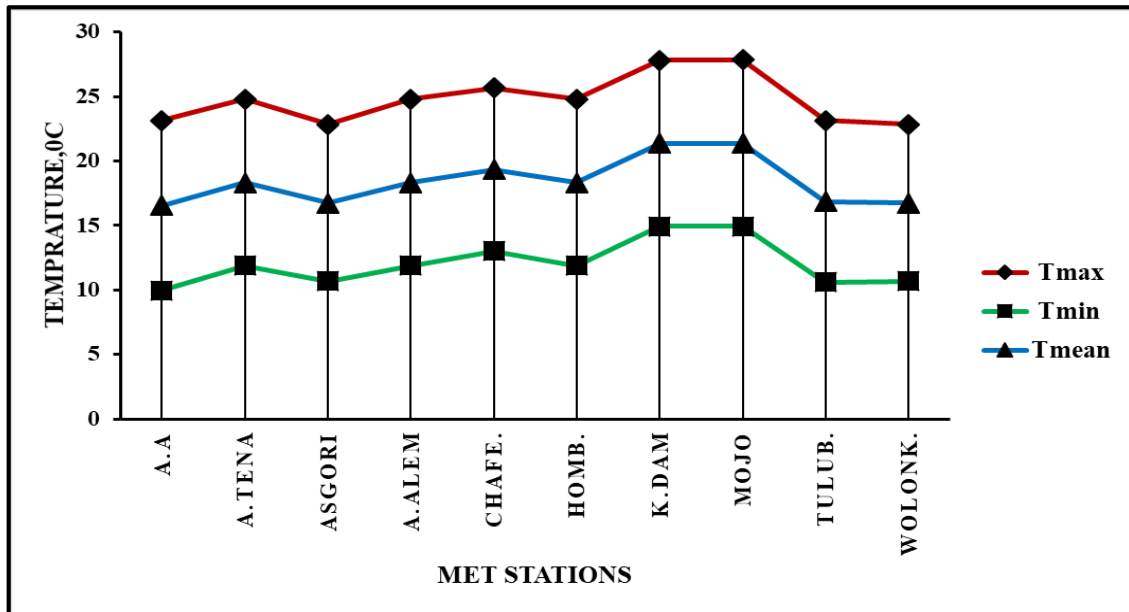


Fig 3. 4 Long-Term Annual Temperatures of Selected Stations

3.1.3. Land Use/Land Cover

The land use condition in the Upper Awash catchments includes mainly cultivated agricultural land, grassland, and forestland, rural and urban settlements. It is estimated that 67% is intensively cultivated, 25.5% is moderately cultivated, 4.5% is bushland or shrubland or wooded grassland, and 3% is an urban area and alpine vegetation (Gobena, 2010).

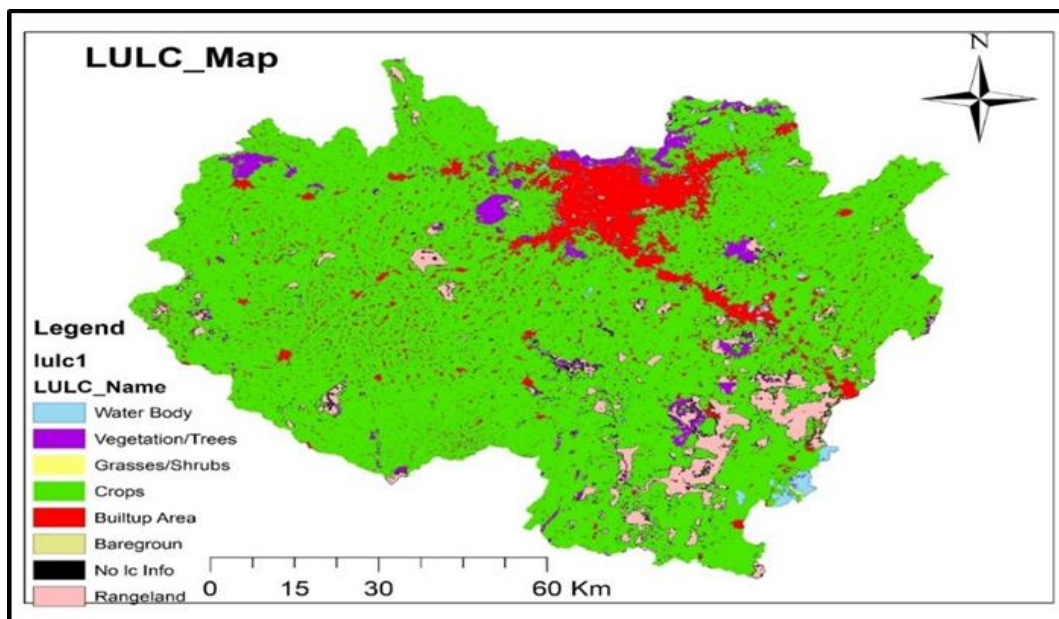


Fig 3. 5 Land Use/Cover of Upper Awash Watershed

3.1.4. Soil

The soil type in the upper awash sub-basin is diverse. The major soils classified in the upper awash basin are Vertisols, Luvisols, Cambisols, Nitisols, Andosols, Fluvisols, Regosols, and Leptosols, the list of the major soil groups and their area coverage, their geographic distribution is shown in a map. Orthic Luvisols are the most extensive soils of the sub-basin; they cover about 40 % of the total soil type (Tafa, 2019).

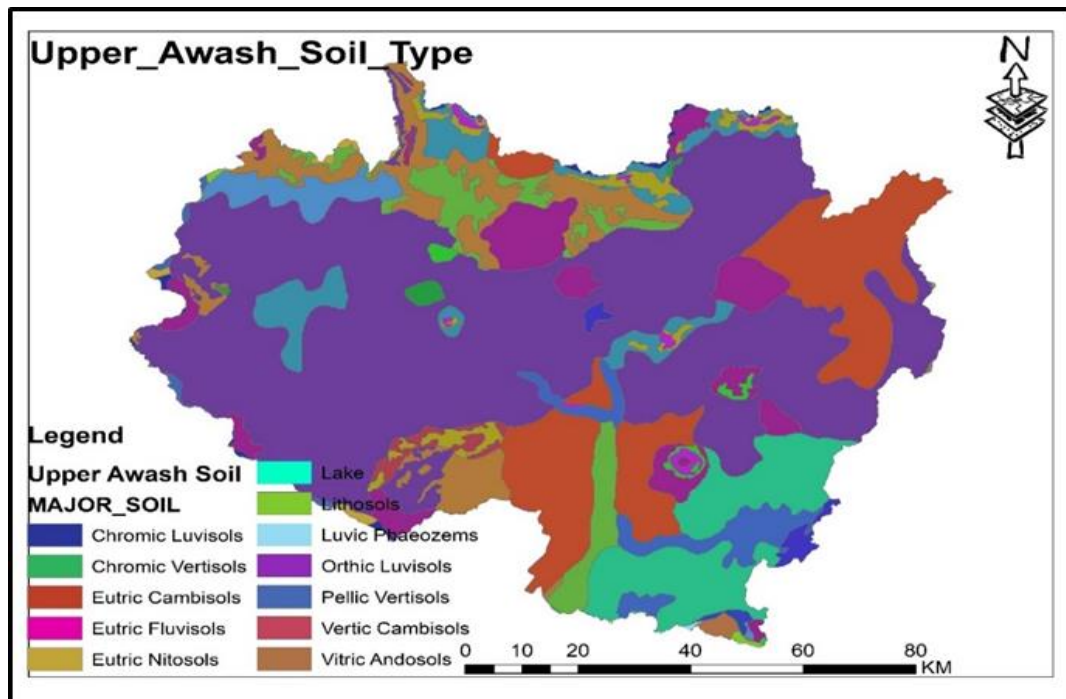


Fig 3. 6 Soil Type of Upper Awash Watershed

3.1.5. Features of Koka Hydropower Plant

Koka dam is a concrete gravity dam located about 100km south of the capital Addis Ababa. It was commissioned in 1960 after three years of construction. It is the only multipurpose reservoir found in the whole of the awash basin being utilized for both Hydropower generation and Irrigation purposes. Its crest elevation is at 112.8m a.s.d with a maximum height of 23.8m and a crest length of 426m. Koka reservoir was created as a result of the construction of the Koka dam in 1960. The reservoir covers about 177 km² area with mean depth, maximum length, and maximum width of 9m, 20 km, and 15 km respectively. The minimum and maximum water levels in the reservoir are 100.3 m and 110.3m above the site datum, respectively. At the initial period of construction, the reservoir storage capacity was estimated as 1850MCM with 1650MCM active storage and 180MCM for dead storage. According to the bathymetric survey conducted by MWR in 1999, the capacity of the Koka reservoir has been

reduced from 1660 Mm³ in 1959 to 1186 Mm³ (MWR 1999). The reservoir has lost a considerable amount of about 40% of its storage due to a siltation issue and clogs the bottom outlet completely to the extent that it is no more usable. The major constructions in the dam are the ogee spillway, a bottom outlet, and an intake tunnel are the major construction components of the koka dam. Four radial gate spillway was found at the center of the dam with a crest and top of gate level at 104.6m and 110.3m, respectively. The spillway has a capacity of 1028m³/s at an elevation of 110.3m and 1322m³/s at 111.8m. Koka reservoir has an orifice-type intake at the left side of the dam. The water from the dam passes through a pressure tunnel of length 71.5m, and a 145.37m concrete pipe of a diameter of 5.5m with 70cm earth cover. A surge tank of 20m in height and 18m in diameter is found to regulate the water hammer effect. Koka power plant has three penstocks of 3.5m diameter and variable lengths of 50.7m, 53m, and 59m. Three vertically oriented Francis turbine is located in the power with three units of generators. At Koka powerhouse, there are three vertical Francis turbines with installed power, firm power, regulated discharge, and net head of 43.2MW, 34.5MW, 126.9m³/s, and 42m respectively(NCORE Workshop 2017; Shemeles 1998; Tumebo 2008)

Table 3. 1 Koka Hydropower System and its Salient Features

Salient Feature	Level and other Remarks
Dam	
Type of Dam	Concrete Gravity Dam
Crest Length	426 m
Height	23.8 m
Crest Width	10 m
Crest Level	1593.2(112.8m) ASL(ASD)
Minimum Foundation Level	1551.4 m
Reservoir	
Catchment Area	177 km ²
Storage Volume	1186 m ³
Maximum length	20 km
Maximum Width	15 km

Mean Depth	9 m
Maximum Flood Control Level	1592.2(111.8m) ASL(ASD)
Maximum Operating Level	1590.7(110.3m) ASL(ASD)
Minimum Operating Level	1580.7(100.3m) ASL(ASD)

Spillway

Type of Spillway	Ogee Spillway
Spillway Top of the Gate level	1590.7(110.3m) ASL(ASD)
Spillway Invert Level	1584.7(104.6m) ASL(ASD)
Type of Spillway Gate	Radial Gate
Radial Gate (No, wxh)	4,12x6

Power House

Maximum Capacity	4X250=1000 m ³ /s
Installed Capacity	43.2MW
Firm Capacity	34.5MW
Firm Power production	80 GWh
Installed Power Production	110 GWh
Regulated Flow Through Penstock	3x42.3 = 126.9 m ³ /s

Power Intake

Height	5.15 m
Length	6.1 m
Intake Invert Level	1570.9(90.5m) ASL(ASD)
Maximum Capacity@110.3m	260 m ³ /s

Others

Bottom Outlet Invert Level	1569.5(89.1m) ASL(ASD)
General Silt Level of the Dam	1575.4(95.0m) ASL(ASD)

Minimum Foundation Level of the Dam	1551.4(71.0m) ASL(ASD)
Tail water Level	1548(68.3m) ASL(ASD)

3.2. Data Collection

Data is an essential parameter for all hydrological and simulation models in water resources studies. As the goal of this study is to optimize the operation of the Koka reservoir for different purposes, data that support this goal should be collected. Accordingly, spatial and time series data were collected from different agencies and organizations in Ethiopia.

Table 3. 2 Data Type and Respective Organization/Agencies

Data category	Data length	Sources
Hydrological and Spatial data	Daily	Ministry of Water Resources
Meteorological data	Daily	National Meteorological Agency
Reservoir Physical and Operational data	Monthly	Ethiopia Electric Power (EEP)

3.3. Methodology

The methodology is the overall steps of the research which describes the activities carried out through the working period. To conduct research with a clear methodology having good-quality data is a must. Data might be primary or secondary. For this particular research secondary data were used. These data were collected from respective organizations and their quality and quantity were analysed by using different empirical formulas because the result mainly depends on the quantity and quality of data used.

The analyzed meteorological data such as rainfall, temperature, and spatial data were inputted into HEC-HMS hydrological model to generate the inflow from catchments to the reservoir for different time horizons, and the performance of the model was checked. The generated flow obtained from HEC-HMS, net reservoir evaporation loss, reservoir physical data, and reservoir operational data were the main inputs for the HEC-ResSim model. The optimized power and the release from the reservoir considering the objective function and different constraints was the expected output of the model. After all, the data are configured, a simulation was performed

and results were discussed. Based on the simulation results and objectives of the study, different conclusions and recommendations were drawn.

The following Fig.3.7, clearly shows the overall approach and steps followed in the design and analysis of this study

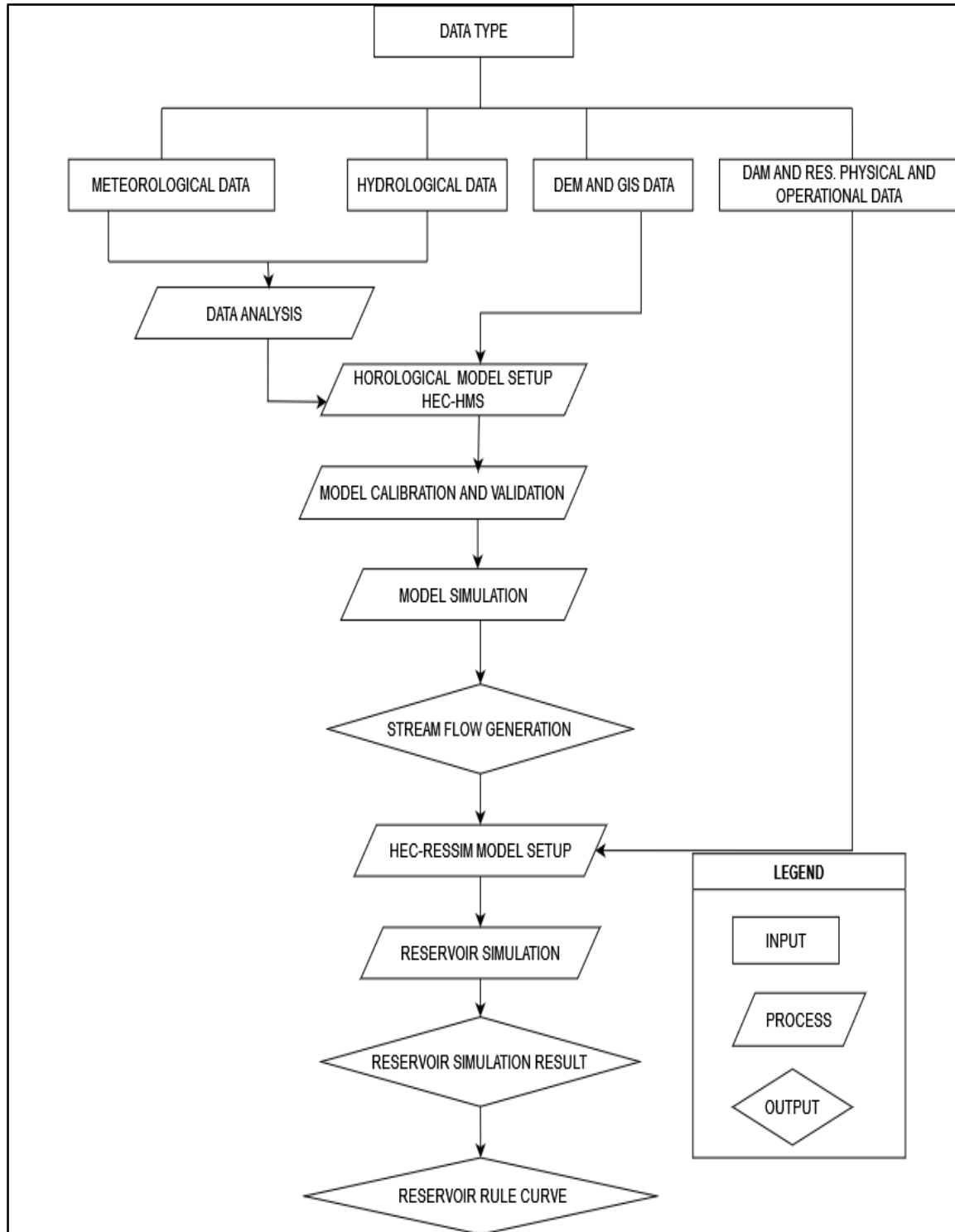


Fig 3. 7 General framework of the study

3.4. Material Used

For the appropriate implementation of this study ArcGIS, Hydrologic Modeling System (HEC-HMS), Reservoir System Simulation model (HEC-ResSim), Data Storage System (HEC-DSS) tool, and Excel Spreadsheet were used for data analysis and reservoir simulation. To meet the objective of the study, secondary data from different sources were used.

3.5. Hydro-Meteorological Data Analysis

General

The effectiveness of hydrological analysis and hydrological modeling is greatly influenced by hydro-meteorological data quality. The prerequisite that hydrologists must examine and fulfill is preserving the quality of hydro-meteorological data. Thus, these data should be stationary, consistent, and homogeneous. If it does not fulfill one of the above criteria, it will result in a big problem that contradicts the actual situation. To determine whether the data meet these criteria, the researcher needs a simple but efficient screening procedure. Therefore, data consistency, homogeneity, and infilling were checked by different empirical methods.

3.5.1. Meteorological Data Analysis

Metrological data are among the most important time series data needed in the design and analysis of water resources projects. According to the report of ARBA (2017a), 56 meteorological stations were installed in the Awash River Basin, in which 34 stations were found in the upper awash basin. For this research, data from 23 meteorological stations found in and nearby of the sub-basin were collected from National Meteorological Agency. Of these stations, only 2 stations are first-class stations (Addis Ababa @ Bole and Bishoftu) and the rest 21 stations are third-class. The location of the station (i.e., within or nearby to the sub-basin), availability, and quality of data were considered as selection criteria. Accordingly, eleven (11) representative meteorological stations were considered for this study. The meteorological data collected from these stations were precipitation, temperatures (minimum and maximum), relative humidity, sunshine hour, and wind speed.

Table 3. 3 Summary of selected Meteorological stations

S.N	Station Name	Latitude	Longitude	latitude	Years of data	Annual RF
1	A.A @Bole	8.98	38.8	2354	1985-2020	1049
2	Addis Alem	11.024	37.06	2400	1987-2020	1126

3	Alem Tena	8.29	38.91	1656	1987-2020	782
4	Asgori	8.8	38.34	2072	1985-2020	985
5	Chafedonsa	8.97	39.13	2402	1985-2020	816
6	Wolonkomi	9.004	38.257	2165	1985-2020	880
7	Tulubolo	8.66	38.21	2190	1987-2020	1123
8	Mojo	8.61	39.11	1771	1985-2020	1007
9	Hombole	8.36	38.78	1743	1985-2020	823
10	Koka Dam	8.471	39.157	1625	1985-2020	891
11	Bantu Liben	8.6185	38.357	2167	1985-2020	1207

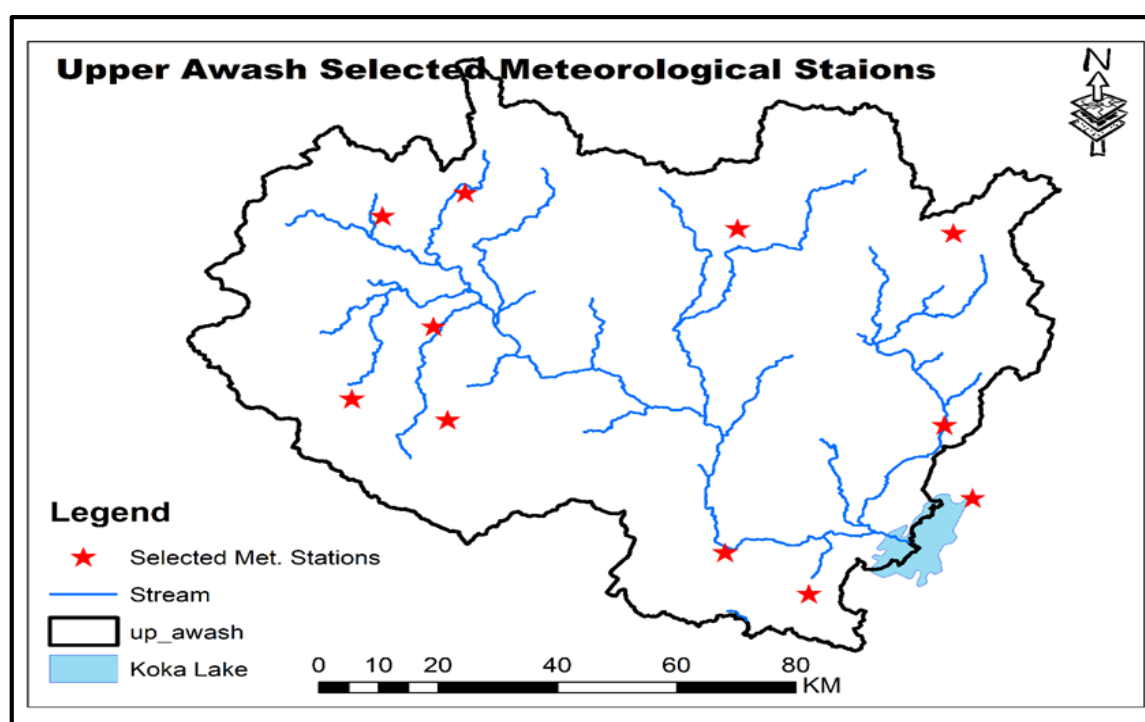


Fig 3. 8 Selected Metrological Stations in the Study area

Homogeneity Test

The quality and reliability of the data obtained from the meteorological stations depend on many factors. Meteorological gauging stations are influenced by the location of the station, the tool and method used, and the observation quality and the time series might gain an inhomogeneous structure. For this reason, the reliability and quality of the data to be used in

the modeling of hydrology and water resources processes should be tested statistically (Allami et al., 2014; Firat et al., 2012). The homogeneity of eleven selected stations in the study area was checked by non-dimensional values of monthly average rainfall records using the following non-dimensional equation.

$$P_i = \left(\frac{\bar{P}_i}{P_j} \right) * 100\% \dots \dots \dots \text{Eq 3.1}$$

\bar{P}_i = Over years averaged monthly precipitation of station i.

P_j =Over year's average yearly precipitation of all station

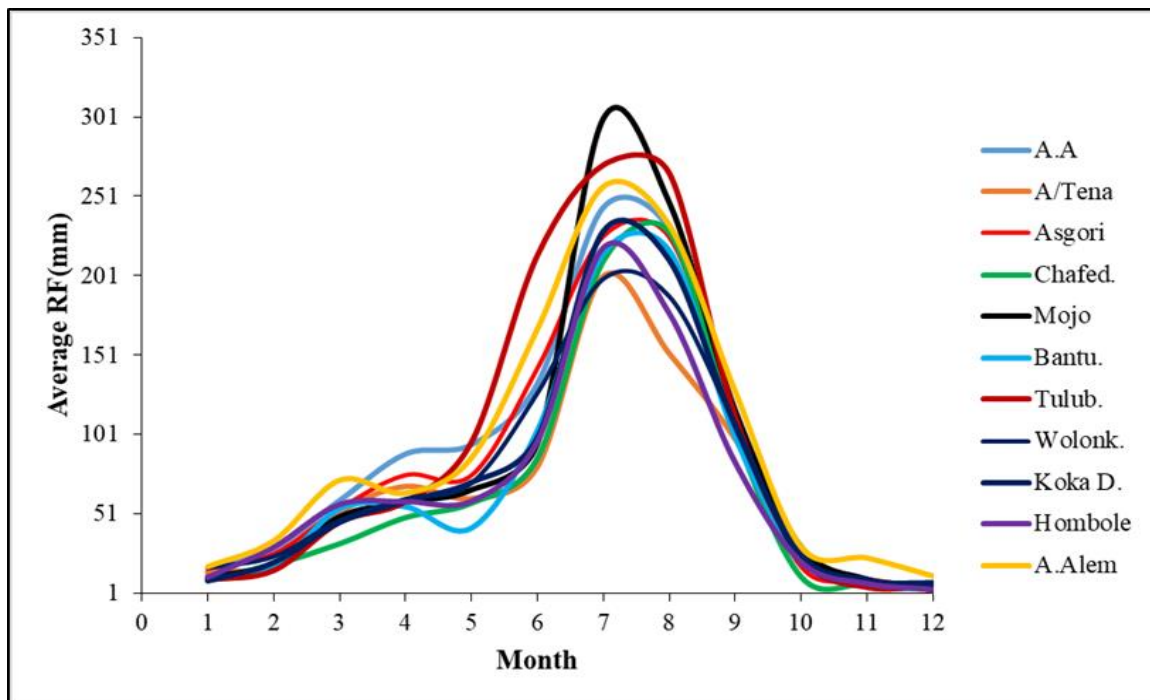


Fig 3. 9 Non-dimensional rainfall for selected meteorological stations

Test for Consistency of Record

If the conditions relevant to the recording of a rain gauge station have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. Shifting of the rain gauge station to a new location, the neighborhoods of the station undergoing a marked change, change of ecosystem due to calamities, and occurrence of observational error from a certain date are some of the most common causes of the inconsistency of records. The inconsistency of the record is checked by the double-mass curve technique. This technique is based on the principle that when each recorded data comes from the parent population, they are consistent. If a double-mass curve has a constant slope, the record is consistent unless the data is not consistent and need to be adjusted (corrected) by the following equation.

$$P_{cx} = P_x \frac{M_c}{M_a} \dots \dots \dots \text{Eq 3.2}$$

P_{cx} = corrected precipitation at any time t_1 at station x

P_x = originally recorded precipitation at any time t_1 at station x

M_c = corrected slope of double mass curve

M_a = original slope of double mass curve

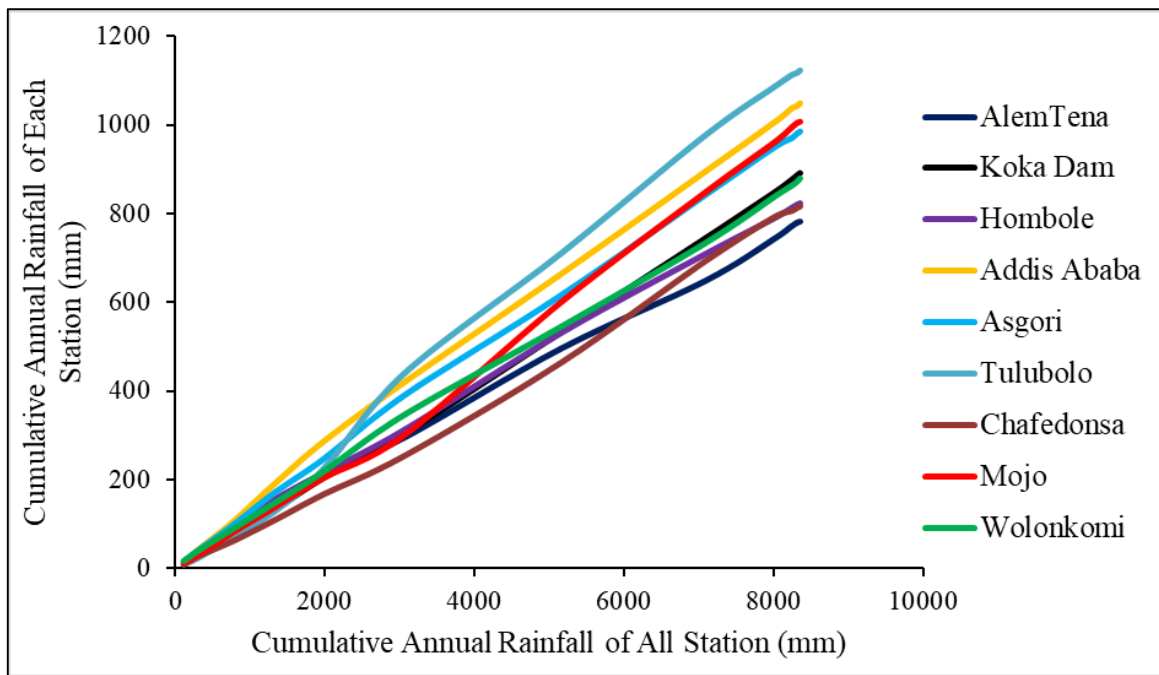


Fig 3. 10 Double Mass Curve of Selected Station

Filling Missing Rainfall Data

The accurate planning and management of water resources depend on the presence of consistent and exact precipitation data in meteorology stations. The outcome of data analysis depends on the quality and completeness of the data. Lack of good quality rainfall data will have a bad implication on the process of analyses and subsequently, leads to biased results of the analysis (Egigu 2020). Data may be missed due to many reasons such as failure or absence of the observer to make the necessary visit to the gauge, destruction of recording gauges, and instrument failure because of mechanical or electrical malfunctioning. Any such causes of instrument failure reduce the length and information content of the precipitation record. Most of the stations in upper awash have missed data records and it is necessary to estimate the missing records to keep the continuous time series of the data. The missing data can be

estimated by using the data of the neighboring stations. Several methods have been proposed in different literature for the estimation of missed data. The most common and well-known methods are the simple Arithmetic Mean Method, Normal-Ratio Method, and Inverse Distance Weighted interpolation method.

Arithmetic Mean Method

The arithmetic means method considers the mean annual rainfall of a station with missing data with that of an adjoining station thus too shallow. This assumes equal weights from all nearby rain gauge stations and uses the arithmetic mean of the precipitation data. This method is preferable when the following three conditions are fulfilled.

- ❖ the missing stations say x is within 10% of the normal annual rainfall of the surrounding stations
- ❖ Data of at least three surrounding stations, called index stations are available within the basin and
- ❖ The index stations are evenly spaced around the missing station and should be as close as possible

The method yields a good estimate in a flat area if the gauges are uniformly distributed and the individual gauges catch does not vary widely from the mean. The arithmetic mean is estimated by:

$$P_x = \frac{1}{n} \sum_{i=1}^N P_i \dots \dots \dots \text{Eq 3.3}$$

Where, P_x = the station with the missing record (mm)

P_i = index station (mm)

N = number of index stations

Normal-Ratio Method

The normal ratio method is preferable when the normal annual precipitation of the index station differs by more than 10% of the missing station. This method is applied when there are large differences in elevation (for example, regions where orographic effects are present) or where average annual rainfall is low. The rainfall of the surrounding index stations is weighted by the ratio of normal annual rainfall by using the following relation.

$$P_x = \frac{N_x}{n} \left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_3} \dots + \frac{P_n}{N_n} \right] \dots \dots \dots \text{Eq 3.4}$$

P1, P2, P3, Pn = Rainfall data of index stations (mm/d)

N1, N2, N3, Nn = Normal Annual rainfall of index station (mm)

Px, Nx=the corresponding values of the missing station x

n = number of stations surrounding the station x

Inverse Distance Method

In this method, the weight for each station is assumed to be inversely proportional to the squared distance of the target station from the neighboring station with data. This method involves computing weights of the surrounding rain gauges based on their distances from the gauge with missing data. As this method is strictly based on distance, it is not satisfactory for hilly regions. These distances are computed by establishing the asset of axes running through its gauge. The missing rainfall data of station x is computed by the following equation.

$$P_x = \frac{\sum_{i=1}^N W_i P_i}{\sum_{i=1}^N W_i} \dots \dots \dots \text{Eq 3.5}$$

$$\text{where } W_i = \frac{1}{D_i^2} \text{ and } D_i^2 = (\Delta X^2 + \Delta Y^2)$$

Pi= the rainfall of the surrounding stations

Wi = the weight for the rainfall depth Pi at gage i

($\Delta x^2 = \Delta y^2$) = the difference between the surrounding station from the station with missing data (0, 0)

N = number of the station which cannot be more than 5

The best method to estimate missing rainfall data can be different for each climatic zone depending on the rainfall pattern and spatial distribution. It's important to remember that no method is better or worse than any other. As a result, it is not wise to rely just on one method to complete the missing data. The choice of a certain technique to achieve better outcomes is significantly influenced by the distribution of the data. Hence, a detailed analysis of the data distribution is required before applying any of the strategies mentioned (Egigu 2020). For this study, missing data were filled by the inverse distance method because it has been advocated to be the most accurate method and has been applied by several researchers (Armanuos et al. 2020; Das 2019; Egigu 2020; Lam et al. 2015; Moeletsi et al. 2016; Silva et al. 2007) worldwide and in Ethiopia.

3.5.2. Hydrological Data Analysis

Stream flow records are among the most valuable of all hydrologic data because they represent an integration of all hydrologic factors. The hydrological data were collected from the Ethiopian Ministry of Water Resources. The Ministry operates 21 hydrometric stations in the upper awash basin out of 67 stations installed in the whole basin (Mengistu, 2008). For this study, three gauging sites that are found at key positions and nearby the outlet are chosen for analysis of river flow data. The selected stations are Awash at Melka Kunture, Awash at Hombole, and Mojo at Mojo IV. Flow data of these hydrometric stations were obtained from the ministry on daily basis in excel format.

Table 3. 4 Major Hydrological Station in Upper Awash Basin

Station	River	Station Name	Lat.	Long.	D/Area	Data Length	Annual Flow
031012	Awash	@M.Kunture	8.42	38.36	4456	1995-2016	985
031004	Awash	@Hombole	8.23	38.47	7656	1997-2016	1292
031014	Mojo	@Mojo	8.36	39.05	1264	1996-2016	150

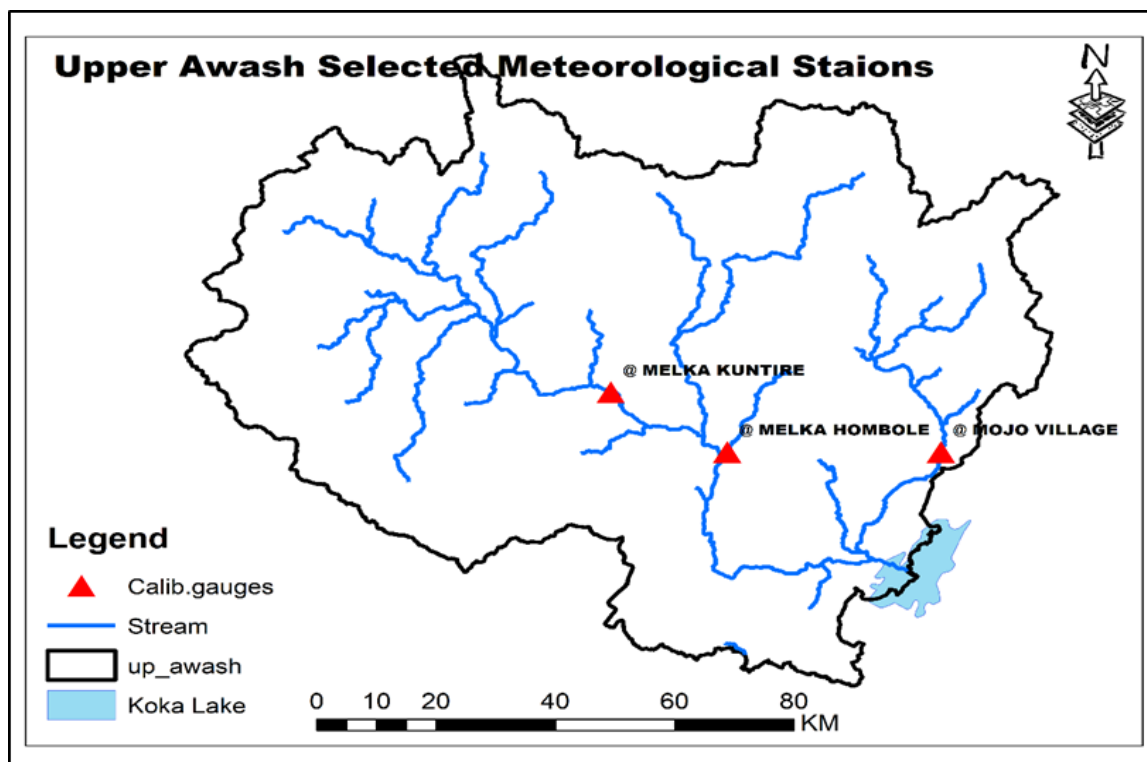


Fig 3. 11 Selected Hydrological Gauging Stations of the study area

Rough Screening

This will allow visual detection of whether the observational data have been consistently or accidentally credited to a wrong day, or whether they contain misplaced decimal points. Visual observation of daily flow records implied no minor errors such as exaggerated numbers, misplaced decimal points, very high flows during dry months, and/or very low flows during rainy months. Thus, the stream flow records look good enough for all stations.

Filing of Missed Data

There are some periods with missed and short gauge records in all river flow data. Hence, to make use of partially recorded data, missing values need to be filled in sequence. The missing values were filled with the simple arithmetic mean method because all stations fulfill the criteria of the arithmetic mean method and yields good results than inverse distance method.

3.6. Reservoir Evaporation

Evaporation from open water sources is rarely measured directly, even in small water bodies. Evaporation can be determined by pans or by an empirical formula such as water balance, energy balance, mass transfer, or a combination of energy balance and aerodynamic techniques. The selection of measuring techniques usually depends on the depth of the water body and the availability of weather data or micrometeorological equipment (Jensen, 2010). According to FAO Irrigation and Drainage paper 56 by (Anderson and French 2019), the conversion of reference evaporation to evaporation of open water, with a depth higher than 5m, clear of turbidity, in a temperate climate, would be varied between 0.65 and 1.25. The lower values “correspond to the period when the water body is gaining thermal energy”, and the higher to the period “during the fall and the winter when heat is released from the water body”.

For this study, koka reservoir evaporation was estimated by ETo- Calculator program by using Adama meteorological station’s data because this station is the first class station found nearby to the reservoir. The program requires data like long-term mean daily maximum temperature (°C), daily minimum temperature (°C), [RH] relative humidity (%), wind speed (km/day), sunshine hours per day, and solar radiation (MJ/m²/day) by using penman-Monteith method. The aridity correction factor for Ethiopia was estimated to be 1.2 (ARBA 2017b; Sileshi Gobena 2010). Therefore, to get the evaporation of koka reservoir the evaporation estimated by the ETo-Calculator program was multiplied by an aridity factor of 1.2.

Table 3. 5 Koka Reservoir Evaporation

Month	Evaporation (At Adama Met. Station)	Evaporation (Transferred to Reservoir)
Jan	147.95	177.55
Feb	146.36	175.64
Mar	169.37	203.25
Apr	157.64	189.16
May	155.56	186.68
Jun	146.45	175.75
Jul	136.40	163.68
Aug	134.15	160.97
Sep	130.36	156.44
Oct	152.46	182.96
Nov	138.82	166.58
Dec	139.78	167.74

3.7. Flow Transfer to Dam Site

Koka reservoir receives inflow from two rivers: the Awash River (Main River) and the Mojo River (tributary river). However, the total inflow into the koka reservoir is not directly known from gauging stations. Therefore, it will be necessary to transfer the inflow to the reservoir inlet from most nearby gauging, i.e. Hombole (42km) and mojo (35km) upstream of the reservoir inlet.

Table 3. 6 Selected Gauging Station with Drainage Area

Station Name	Drainage area (km ²)	Percent of Coverage	River
Melka Kunture	4515	36	Awash
Melka Hombole	6640	53	Awash
Mojo	1440	11	Mojo
Koka Inlet	12595	100	Outlet

Halcrow (1989) developed the regression equation by considering the ungauged area below gauging stations to calculate the total inflow at the koka inlet.

$$Q@koka\ inlet = 1.065 * Q@Hombole + 1.18 * Q@Mojo \dots \dots \dots Eq. 3.6$$

Hence, for this study, the total inflow at the koka reservoir inlet was calculated using Eq.6

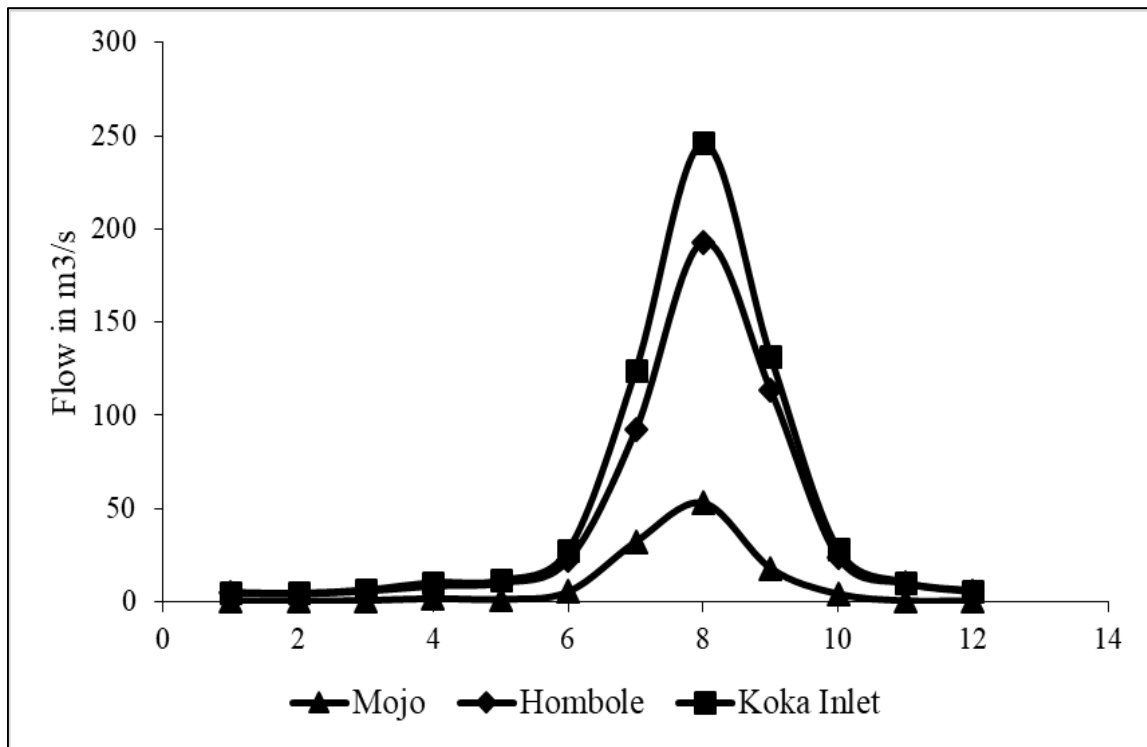


Fig 3. 12 Transferred flow to koka reservoir inlet

3.8. HEC-HMS Model Setup

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the precipitation runoff processes of dendritic watershed systems. The software's design allows applicability in a wide range of geographic areas for solving diverse problems including large river basin water supply and flood hydrology, and small urban or natural watershed runoff. A model of the watershed is constructed by separating the hydrologic cycle into manageable pieces and constructing boundaries around the watershed of interest. The software is one of the only hydrologic simulation products to place such an equal value on both natural and urbanized watersheds, and can handle watersheds with a mixture of development conditions. HEC-HMS also is one of the only hydrologic simulation tools to include both event and continuous simulation capabilities. This flexibility allows a watershed model developed for one purpose, to be repurposed with a minimal amount of effort (USACE 2013).

3.8.1. HEC-HMS Model Components

HEC-HMS model components are used to simulate the hydrologic response in a watershed. HEC-HMS Model setup consists of four main model components: basin model, meteorological model, control specifications, and input data (time series, paired data, and gridded data).

Basin Models: The basin model represents the spatial configuration of the watershed. This project was established in the latest version of HEC-HMS 4.10 through the GIS menu which is an extension in HEC-HMS that supports the identification of the river network and division of the catchment (basin area) into many sub-areas. The domain of the HEC-HMS model was generated from a digital elevation model (DEM) of 20m resolution which was taken from the Ministry of Water Resources, GIS, and Remote Sensing Department. Before HEC-HMS version 4.4, the Geospatial Hydrologic Modeling Extension (HEC-GeoHMS), was needed to create basin and meteorological models using GIS datasets. HEC-GeoHMS is a plugin to ESRI's ArcGIS software. After the release of HEC-HMS version 4.4, basic watershed delineation tools are now provided directly within HEC-HMS. Nowadays, integrated GIS tools allow the user to create a basin model from a digital elevation model. In this research, the model divided the upper awash sub-basin into eleven sub-basins, seven reaches, and eight junctions as appropriate for the study. The model delineated area of the sub-basin as 11595 km² which is nearly equal to the area reported in different kinds of literature. (See Fig 3.13)

The catchment characteristics consist of runoff volume models, direct runoff models (overland flow and interflow), base flow models, and channel flow models. The basin models used for this study are summarized as follows:

Deficit and Constant Loss method:-The Deficit and Constant Loss Method uses a single soil layer to account for continuous changes in moisture content. Infiltration only occurs when the soil layer is saturated. This method is generally used in combination with a meteorological model that computes evapotranspiration. Maximum storage specifies the amount of water the soil can hold, specified as a depth. The constant rate defines the infiltration rate when the soil layer is saturated. The percentage of the sub-basins which is directly connected with the impervious area can be specified (USACE, 2022).

Clark Unit Hydrograph Transform: - Clark unit hydrograph is a synthetic unit hydrograph method. This means that the user is not required to develop a unit hydrograph through the analysis of past observed hydrographs. Time of concentration (T_c) and storage coefficient (R) are the two important parameters in the Clark unit hydrograph transforming excess rainfall into

runoff. There are various methods available to estimate Tc and R and some of the quickest ways are to use empirically derived equations and/or regional regression equations.

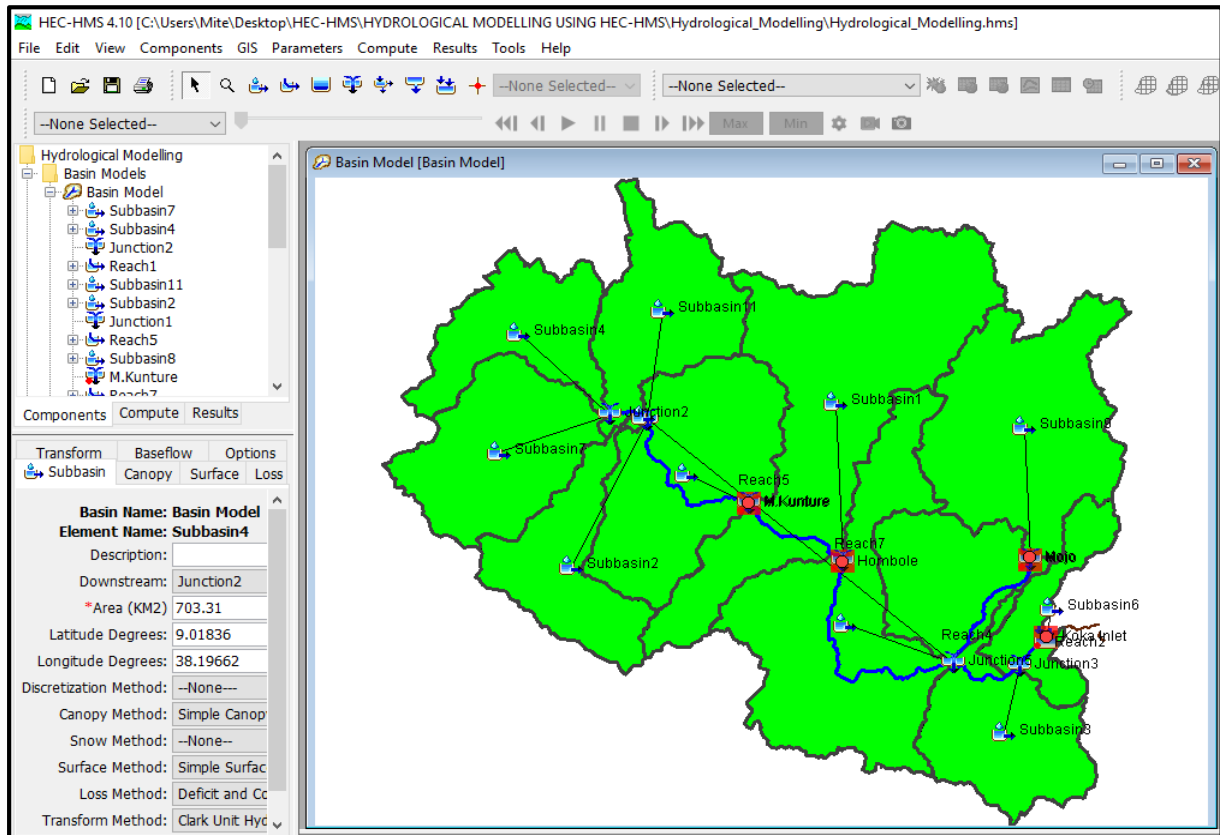


Fig 3. 13 Schematic of Basin Model

Regression equations are developed for regions with specific types of terrain, land use, basin/river slopes, and watershed sizes (Press 1996; USACE 2022). For this study Time of concentration (Tc) and storage coefficient (R) were estimated from equation 3.7 and equation 3.8 respectively.

$$T_c = 2.2 * \left[\frac{L * L_c}{\sqrt{\text{Slope}_{10-85}}} \right] \dots \dots \dots \text{Eq 3.7}$$

Where Tc = Time of concentration in hrs.

L = longest flow path in km

Lc = centroidal flow path in km

Slope₁₀₋₈₅ = average slope of the flow path represented by 10 to 85 percent of the longest flow path (m/km).

Once the time of concentration is estimated, the storage coefficient will be 65% of the time of concentration.

$$\frac{R}{R + T_c} = 0.65 \dots \dots \dots \text{Eq 3.8}$$

Linear Reservoir Base flow: The linear reservoir base flow method, as its name implies, uses a linear reservoir to model the recession of base flow after a storm event. It is the only base flow method that conserves mass within the sub-basin. This method can be used with one, two, or three reservoirs. Partition fractions are used to split the inflow to each of the reservoirs. The inflow is multiplied by the partition fraction to determine the amount of inflow going to each reservoir. The sum of the partition fractions must be less than or equal to one. If the sum of the fractions is less than one, the remaining percolated water is considered aquifer recharge. If the sum of the fractions is exactly equal to one, then all percolation will become a base flow and there will be no aquifer recharge (USACE, 2022).

Muskingum Routing: - Muskingum routing method uses a simple conservation of mass approach to route flow through the stream reach. However, it does not assume that the water surface is level. By assuming a linear, but non-level, water surface it is possible to account for increased storage during the rising tide of a flood wave and decreased storage during the falling side. By adding a travel time for the reach and a weighting between the influence of inflow and outflow, it is possible to approximate attenuation. This method requires primarily three parameters: Muskingum K, Muskingum X, and a number of sub-reaches. The Muskingum K parameter is equivalent to the travel time through the reach. This parameter can be estimated from knowledge of the channel cross-section properties and flow properties. For this research, the Muskingum K was estimated using kirpich’s formula for time of concentration due to the lack of river cross-sectional and flow data channels. The time of concentration represents the time required for a drop of water to travel from the most hydrologically remote point in the sub-catchment to the outlet (Asefa, 2011)

$$K = 0.0078 * L^{0.77} * S^{-0.385} \dots \dots \dots \text{Eq. 3.9}$$

Where,

K= travel time for a drop of water to travel from the remotest point outlet (minute)

L = Length of channel/ditch from headwater to outlet in m and

S = average watershed slope, m/m

The Muskingum X parameter is a dimensionless coefficient that lacks a strong physical meaning. It is the weighting between inflow and outflow influence and ranges from 0.0 to 0.5. In practical application, a value of 0.0 results in maximum attenuation, and 0.5 results in no attenuation. The maximum amount of attenuation will occur when the channel routing computation is done in one step. For most applications, an initial estimation of 0.25 is taken and further refined through model calibration.

Number of sub-reaches: The number of sub-reaches parameter affects attenuation. One sub-reach results in the maximum amount of attenuation and increasing the number of sub-reaches approaches zero attenuation. An initial estimate of this parameter can be obtained by dividing the Muskingum K parameter by the simulation time step Δt . For natural channels that vary in cross-section dimension, slope, and storage, the number of sub-reaches can be treated as a calibration parameter (Press, 1996).

$$\text{No of sub reaches} = \frac{K}{\Delta t} \dots \dots \dots \text{Eq. 3.10}$$

Where is K=travel time of the flood wave

Δt =computation interval

In this study, simulation was applied based on 24hr time step and only one sub reaches was considered since the ratio of Muskingum K and computation interval is less than one.

Precipitation method: The most important process for hydrologic modeling is precipitation. Precipitation determines how water enters the watershed being modeled. HEC-HMS provides a variety of options to define each meteorological element. These methods are used to distribute observed rainfall over the basin by methods such as user hyetograph, user gauge weighting, inverse distance gauge weighting, and gridded precipitation. These methods can provide additional capability to describe rainfall in space and time. For this thesis, a user-specified precipitation method was used. This method was selected because of its simplicity it can provide a specific precipitation time series for each subbasin element in the watershed model and it can handle all of the atmospheric conditions over the watershed. Precipitation is a major and a must element in the meteorological model if the watershed has a sub-basin, but the other meteorological model elements are optional. Elements within the basin model will use the potential evapotranspiration and then compute actual evapotranspiration based on available water in the soil and possibly other factors

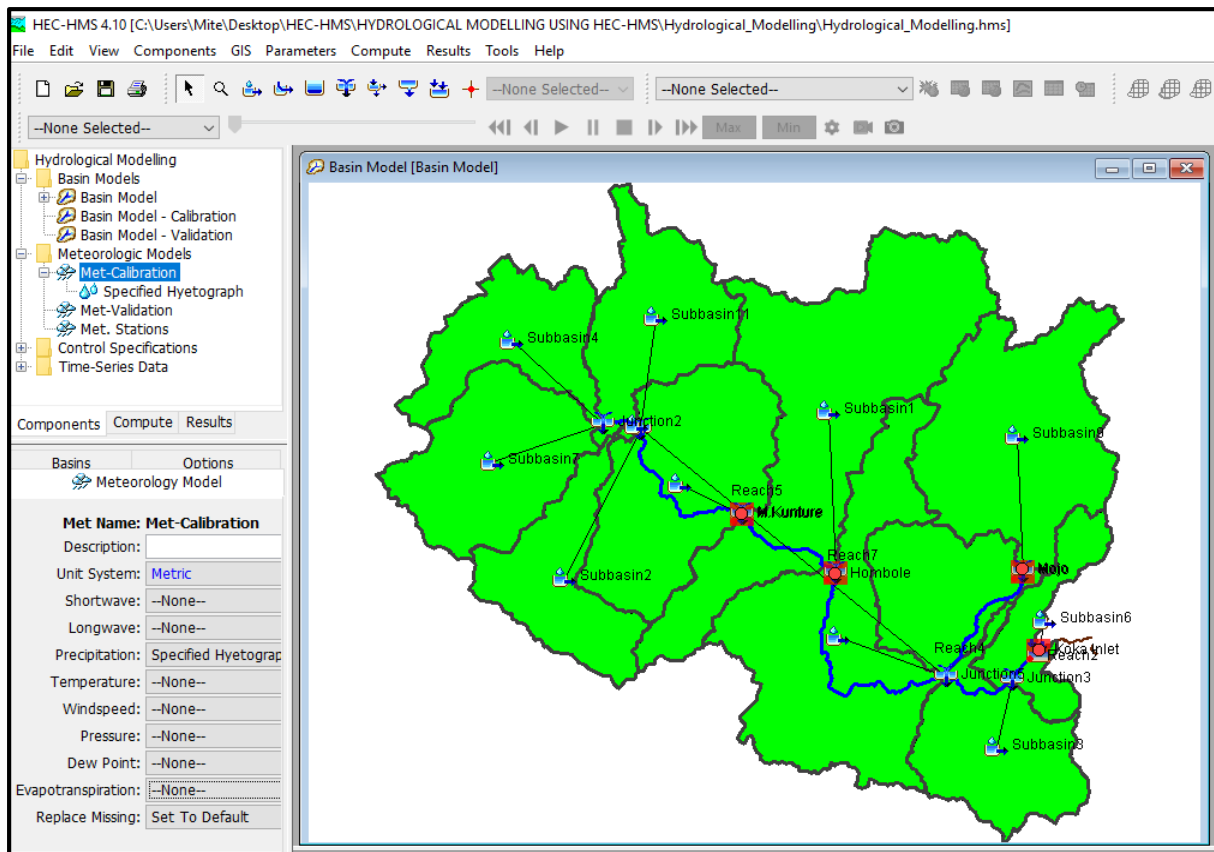


Fig 3. 14 Schematic of Meteorological Stations

Control Specification Model: The control specification model specifies the beginning and ending date and time of a simulation, and the time interval for calculations. Most model elements compute at the time interval specified in the control specifications. However, some elements use adaptive time stepping and may run as short as one-second intervals. These special elements only record results at the specified time interval. For this research, the model runs daily from the period of 1997-2020.

Time series Data Model: Real-time series data for all the meteorological elements defined in the meteorological model are fed in this part. Apart from the above-mentioned meteorological element, discharge data can also be supplied for calibration and simulation of the developed model. Eleven meteorological stations and four hydrological stations data were added by automatic data entry using HECDSS, the Hydrologic Engineering Center Data Storage System.

3.8.2. HEC-HMS Model Performance

The model performance is typically evaluated from the comparison of simulated and observed discharge data in terms of mean, standard deviation, maximum daily discharge, and total discharge using commonly used indices (Jain 1993). To evaluate the model performance, three

statistical indices are commonly used to evaluate the model performance: Coefficient of determination (R^2), Nash and Sutcliffe efficiency (NSE), and index of agreement/ Percent Bias (PBIAS). These are computed as follows.

Nash Sutcliffe Efficiency, NSE

The Nash and Sutcliffe coefficient (NSE) is a measure of efficiency that relates the goodness-of-fit of the model to the variance of measured data. NSE indicates how well the plot of observed versus simulated data fits the 1:1 line (Nash and Sutcliffe 1970).

$$NSE = 1 - \frac{\sum_{i=1}^n (Q_o - Q_s)^2}{\sum_{i=1}^n (Q_o - \overline{Q_o})^2} \dots \dots \dots \text{Eq. 3.11}$$

Q_o = Observed flow

Q_s = Simulated flow

$\overline{Q_o}$ = Average observed flow

NSE ranges between $-\infty$ and 1 (1 inclusive). $NSE = 1$ is considered as optimal value (the simulated values perfectly match with observed values). Values between 0 and 1 are generally viewed as acceptable levels of performance, whereas values less than 0 indicate that the mean observed value is a better predictor than the simulated value, which indicates unacceptable performance.

Coefficient of Determination (R^2)

The coefficient of determination (R^2) measures the degree of collinearity between simulated and measured data. The coefficient of determination R^2 is the squared value of the coefficient of correlation. The correlation coefficient is an index of the degree of the linear relationship between observed and simulated data and ranges between -1 and 1. It is estimated as

$$R^2 = \frac{\sum_{i=1}^n ((Q_o - Q_s)(Q_o - \overline{Q_o}))^2}{\sum_{i=1}^n (Q_o - \overline{Q_o})^2 \sum_{i=1}^n (Q_s - \overline{Q_s})^2} \dots \dots \dots \text{Eq. 3.12}$$

Q_o = Observed flow

Q_s = Simulated flow

$\overline{Q_o}$ = Average observed flow

$\overline{Q_s}$ = Average simulated flow

The coefficient of determination (R^2) is the number between 0 and 1, with higher values of R^2 indicating less error variance, and typically values greater than 0.5 are considered acceptable (Arnold et al. 2012; Liew, Arnold, and Garbrecht 2003).

Percent Bias, PBIAS: is a measure of the average tendency of the simulated data to be larger or smaller than their observed counterparts

$$D = 100\% * \left[\frac{(\sum_{i=1}^n(Qo)) - \sum_{i=1}^n(Qs)}{\sum_{i=1}^n(Qo)} \right] \dots \dots \dots \text{Eq. 3.13}$$

Qo = Observed flow

Qs = Simulated flow

The percent difference (D) can vary between ∞ but $-\infty$ it performs best when a value of 0 (zero) is generated. The optimal value of PBIAS is 0.0, with low-magnitude values indicating accurate model simulation. Positive values indicate model underestimation bias, and negative values indicate model overestimation bias (Gupta and Das, 1999)

3.9. HEC- ResSim Model Setup

HEC-ResSim represents a significant advancement in the decision support tools available to water managers. The model uses an original rule-based approach to mimic the actual decision-making process that reservoir operators must use to meet operating requirements for flood control, power generation, water supply, and environmental quality. The program has a graphical user interface (GUI) and utilizes the HEC-Data Storage System (HEC-DSS) for storage and retrieval of input and output time-series data and they are the watershed setup/or watershed configuration, the reservoir network and the simulation scenario module. The program consists of three modules. These are Watershed Setup, Reservoir Network, and simulation. Each module has a unique purpose and an associated set of functions accessible through menus, toolbars, and schematic elements.

3.9.1. Koka Watershed Setup

Koka Reservoirs watershed setup was created by specifying the directory location, naming the watershed, specifying the units of measure as SI units, and the Time Zone as GMT+3, Addis Ababa. Once it has been created the new watershed, ResSim generates a new directory hierarchy in the “base” working directory. The background image that describes the Geo-referenced area of the koka watershed and the physical arrangement of the stream network was imported from ArcGIS file (See figure 3.15). After importing the background map and stream

alignment, the project elements such as reservoir and computational points will be drawn, using water shed setup module tools. Finally, the watershed configuration will be created for the network setup.



Fig 3. 15 Koka Watershed Setup

3.9.2. Koka Reservoir Network setup

A reservoir network is a collection of watershed elements connected by routing reaches (See Fig 3.16). In this module, river schematization, and description of the physical and operational elements of koka reservoirs were built and the required alternatives were developed. Using configurations that were created in the koka watershed setup module as a template, based on a reservoir network module, routing reaches were added, and possibly other network elements to complete the connectivity of the reservoirs network schematic. Koka Reservoir (KR) and Koka Dam (KD), have been created in koka network module with their respective element trees.

The modeling elements that make up a reservoir network include reservoirs, reaches, junctions, diversions, reservoir systems, and state variables. Each of these elements consists of one or more sub-elements. The following sections will describe some of these elements created in the koka reservoir setup



Fig 3. 16 Koka Reservoir Network Setup

Junctions (CP1 to CP5): Junctions are key locations to identify and manage flow across the network. Since HEC-ResSim does not calculate runoff, all local flows must be introduced at junctions as external flows. The flow out of a junction is simply the sum of the flows into the junction. The computation points added in the koka Watershed setup automatically become junctions in the reservoir network module, hence the main task in the reservoir network is adding local flow to junctions and routing reaches. In the koka watershed network module, junctions such as CP3 (Hombole), CP4 (Mojo), and CP5 (Mojo) are local inflow junctions to the reservoir whereas CP1 and CP2 are inflow to outflow from the reservoir respectively.

Routing: Routing is a process used to estimate the temporal and spatial variations of a flood hydrograph as it moves through a river reach. The effects of storage and flow resistance within a river reach are reflected by changes in the hydrograph shape and timing as the flood wave moves from upstream to downstream. Routing in HEC-ResSim is handled by a few hydrologic routing methods, which are null (no lag or attenuation), Variable Lag & K, Coefficient, Modified Puls, and Muskingum. In this study, the Muskingum method was selected, due to the method only requires the assignment of only two parameters: wedge-storage coefficient (X) and flood wave travel time (K). The parameter X (ranges between 0 and 0.5) was estimated as

0.15 through trial and error. But the Muskingum K parameters were directly adopted from the hydrologic routing model developed in HEC-HMS as the reason that the reach used in the two models was the same.

Table 3. 7 Values of K and X for Melka Kunture, Hombole, and Mojo rivers

Computation point		Reach Parameters			
From	To	Length (km)	Slope (%)	X	K
Melka Kunture	Hombole	29.89	0.642	0.15	2.21
Hombole	Koka Inlet	61.51	0.317	0.15	4.97
Mojo	Koka Inlet	34.69	0.45	0.15	2.82

Reservoirs (Koka Reservoir-KR): The reservoir is the most complex element in HEC-ResSim. The physical data of a reservoir are represented by a pool and one or more dams. The pool contains the reservoir's elevation-storage-area relationship and can optionally include evaporation and seepage losses.

Dam (Koka Dam-KD): The dam represents both controlled and uncontrolled outlets. An outlet without a control structure (gate) is called uncontrolled and one with a gate or valve is referred to as a controlled outlet. An overflow spillway is an example of an uncontrolled outlet. The advanced outlet types are power plants and pump both of which are controlled outlets with additional features to represent their special purposes. The power plant adds the ability to compute energy production to the standard controlled outlet. The Koka dam releases water through three types of outlets: spillway outlet, power plant outlet, and bottom outlet.

Alternatives

In HEC-ResSim, an Alternative is a construct that represents the combination of a reservoir network, the selection of an active operation set for each reservoir in the network, and the specification of the starting (or look back) conditions and inflow time-series data for the network. A Simulation is a time window over which to compute and analyze one or more alternatives.

3.9.3. Koka Simulation Setup

The purpose of the Simulation module is to isolate the output analysis from the model development process. The simulation window comprises starting time of the simulation, the

look back (the time required for equilibrium or ‘warm-up period’ of the model before the starting simulation time), and the end time of the simulation. Once the reservoir model is complete and the alternatives have been defined, the Simulation module is used to configure the simulation. The computations are performed and results are viewed within the simulation module. Inside the simulation folder, there will be a copy of the watershed which includes only those files needed by the selected alternatives in the DSS file. After a successful simulation, the results can be viewed, analyzed and revisions made and additional simulations performed. HEC-ResSim performs simulations starting from 15 minutes up to the daily time step. HEC-ResSim cannot yet perform simulations on a monthly time step but can interpolate monthly data to daily data (Klipsch and Hurst, 2021).

3.9.4. Koka Reservoir Physical Data

The definition of physical parts is one of the most important parts in the HEC model. Even small changes affect significantly the system behavior and the impacts deteriorate or meliorate the result in the simulation part. Input that should be considered for the physical part consists of the reservoir pool characteristics which are defined by the storage-elevation-area curve, Evaporation loss, Spillway capacity curve, hydropower plant data (outlet, installed capacity, hydraulic loss, station use, and overload factor) and tailwater elevation

Koka Reservoir Storage-Elevation-Area Curve

The elevation storage area curve is the main characteristic of the reservoir pool defining the surface area and the volume of storage at the respective elevation. The reservoir surface area is mainly utilized to compute the reservoir evaporation loss, and the storage is used to estimate the stage or elevation at any time based on the storage equation. Due to the lack of a current bathymetric survey, the storage elevation area of 1999 conducted by the MWR was used for this study.

Table 3. 8 Koka Reservoir Storage- Elevation-Area Capacity

Elevation (m)	Volume (m ³)	Area (ha)
97.60	20000	1
98.00	92000	20.8
99.20	1040000	80.2
100.00	4204000	1585

101.20	23794000	5680
102.00	57282000	7880
103.20	142476000	10520
104.00	218888000	11840
105.20	355744000	13160
106.00	459924000	13760
107.20	600528000	14240
108.00	727204000	14840
109.20	970966000	16160
110.30	1186173000	17700

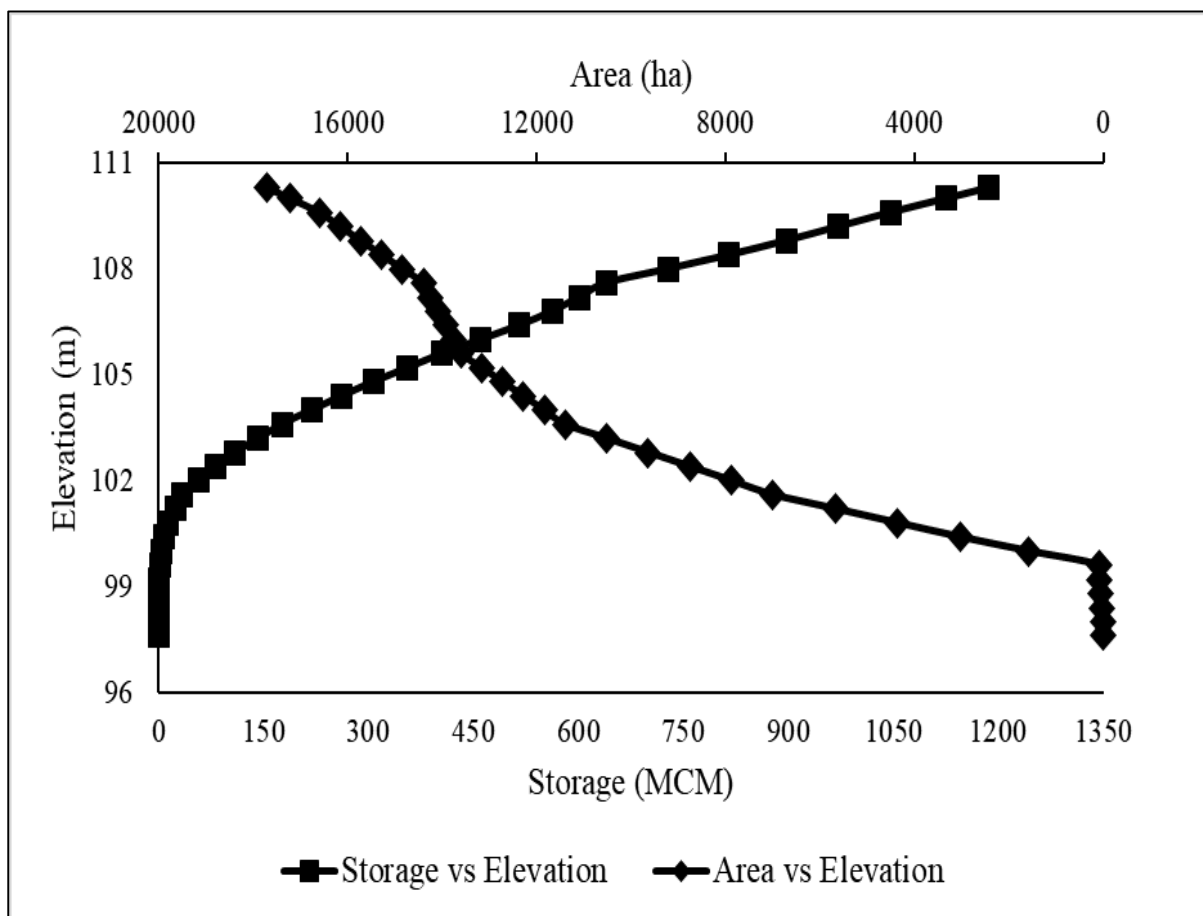


Fig 3. 17 Storage-Elevation-Area Capacity Curve of Koka Reservoir

Net Reservoir Evaporation

During rainy months, net evaporation (evaporation minus precipitation) on the pool will not significantly affect modeling operations. However, it has significant impact on reservoir operation if this loss is not considered during dry months. In this study, the evaporation data series has been determined using ETo-Calculator Software on a long-term monthly mean basis from Adama meteorological station because this station is the most neighbor station to the study area.

Table 3. 9 Koka Reservoir Net Evaporation

	Rainfall	Evaporation	Net Evaporation
Month	mm/month	mm/month	mm/month
Jan	8.93	177.55	168.61
Feb	22.55	175.64	153.09
Mar	45.78	203.25	157.47
Apr	60.96	189.16	128.21
May	70.90	186.68	115.78
Jun	102.27	175.75	73.48
Jul	228.96	163.68	-65.28
Aug	210.83	160.97	-49.86
Sep	110.06	156.44	46.37
Oct	25.48	182.96	157.47
Nov	9.99	166.58	156.59
Dec	5.76	167.74	161.97

Koka Dam: The Koka hydropower dam performs two major functions. It creates the head and storage necessary to move the turbines, and the storage used to maintain the daily or seasonal flow release for downstream irrigation demand. It has a crest elevation of 112.8m at maximum flood level and a crest length of 426m. The koka dam has gated outlets at different levels to manage the quantity of water released from the corresponding zones of the reservoir. These

outlets are low-level outlets (referred to as spillways in this case), hydropower outlets (power plant in this paper), and bottom outlets (compensation outlets in this report).

Koka Dam Spillway: Spillways typically represent structures at the top of the dam that allows water to go over the dam top in a controlled or uncontrolled manner. If no gates are selected, then flow over the spillway is unrestricted. When gates are included, the flow over the spillway will be controlled by the gates. Koka dam consists of an ogee-type spillway controlled by four radial gates with a width of 10m each controlling an ogee overflow section, located in the central part of the dam. It releases a maximum capacity of 1000 m³/s at 110.3m (maximum operating level) to the river axis. There are three different methods for computing outflow through a spillway: Broad-Crested, Ogee, and User Specified. Elevation versus maximum capacity relation for the main spillway of Koka dam at different elevations was developed from a basic orifice equation (USBR 2014)

$$Qg = CA\sqrt{2gH} \quad (\text{Orifice Equation}) \dots \dots \dots \text{Eq. 3.14}$$

Where

Q = the total discharge (m³/s)

C = Coefficient of discharge, which is 0.6 for vertical wall or wheel-mounted gate, 0.65 for radial gate, and 0.9 for bell mouth entrance conditions,(USBR 2013, 2014)

A = the area of the orifice opening (i.e., the product of the opening width (L) and the minimum dimension (d) between the top of the flow surface and the bottom of the opening) (m²).

H = the total hydraulic head above the orifice opening centerline elevation (m)

g = the acceleration due to gravity (m/s²)

Table 3. 10 Koka Reservoir Spillway Release Capacity

Elevation (m)	Spillway Capacity (m ³ /s)
104.60	0.00
105.30	93.44
106.30	186.80
106.80	280.40
107.30	373.80

107.80	467.24
108.30	560.68
108.80	654.12
109.30	747.56
109.80	841.00
110.10	934.44
110.30	1028.00
111.80	1322.00

Koka Dam Power Intake: Intake structures direct water from the reservoir into the penstock or power conduit. Gates or valves are used to cut off the flow of water to permit emergency unit shutdown or turbine and penstock maintenance. The intake of the Koka hydropower plant is an orifice type intake which consists of structures such as a trash rack, service vertical lift gate (now fully open), and one radial gate of 4 m x 4 m which diverts the water to a rectangular tunnel of 5.5 m diameter. The invert of the radial gate is at 90.5 m. The radial gate operates on the range from the full reservoir level (FRL) of 110.30m above the site datum to the minimum operating level (MOL) of 100.3 m (ARBA 2017b). The average release through penstock of the koka hydropower plant of the year 2015-2021 shown in the table below was taken from the awash Basin Authority Office in excel format.

Table 3. 11 The Average Release Through Units

Month	Average Flow through Penstocks (m ³ /s)
Jan	36.90
Feb	40.32
Mar	37.70
Apr	33.74
May	34.87
Jun	32.43

Jul	50.02
Aug	108.32
Sep	157.63
Oct	46.47
Nov	36.04
Dec	38.23

Koka Dam Bottom Outlet: Koka dam bottom outlet is located below the low-level (spillway) outlet, with intake on the right vertical wall of the spillway water passage. It is located at 89.1m and has three gates. Emergency outlet with bulkhead of 2m by 4.58m at the top (now fully opened), roller gate overhauled at the middle with a size of 4m by 4m (closed), and radial gate at the lower part with a size of 2m by 2m (operational). Its design capacity was 260m³/s when the water level reaches 110.3m. According to the report of (NCORE Workshop 2017), the bottom outlet of the koka dam is now completely clogged due to a siltation issue to the extent that it is no more usable and dredging has been stopped years ago. Therefore bottom outlet wasn't considered in this study.

3.9.5. Koka Reservoir Operational Data

An operation set consists of three basic features: Zones, Rules, and the identification of the Guide curve.

Zones

Zones are operational subdivisions of the reservoir pool. The division of zone depends on the purpose of the reservoir, but ResSim establishes a default set of three zones: Flood control, conservation, and inactive as default. Each zone has its own operation rule for flood control, conservation, environment, and any additional activity either downstream or within the dam body. The release is not allowed from the inactive zone, and any rules cannot be added to this zone. The conservation zone is the preferred zone for normal reservoir operation. In the conservation zone, all releases are based on the objective of meeting downstream flow requirements and the reservoir tries to stay within this zone. In the case of the koka reservoir, the conservation zone extends from the top of the inactive zone (100.3m) to the bottom of the flood control zone (110.3m). The flood control zone is the zone set aside for flood control

storage. It extends from the top of the conservation zone to the maximum operation level (111.8m for this analysis). So, for the existing condition, the flood control zone extends from 110.3m to 111.8m.

Rules

Reservoir operational management requires a set of operational rules like release regulations, schedules, policies, or plans that best meet a set of objectives. Rules describe a minimum or maximum constraint on the reservoir releases based on several variables such as reservoir water level, date, inflow, outflow, and many others depending on the information found at the outlet. In ResSim, rules are applied to selected zones of the reservoir to describe the different factors influencing the release decision when the reservoir elevation is within each zone. Each zone can contain a different set of rules depending on the flow limits and requirements of that zone within the regulation plan. In the case of this study, two operational rules were implemented on the reservoir pool and hydropower plant to define three alternatives based on the availability and feasibility of the data in this study. These rules were: **the release function rule** and the **power schedule rule**. The release Function rule as a function of date was applied for all downstream demands because this rule is more flexible and can be assigned to any release element. Power guide curve operation was selected for hydropower generation because it can be assigned to a reservoir element (pool or outlet).

Guide Curve

The main goal of reservoir operating rules is to guide release decisions for the reservoir operators based on the existing condition. HEC-ResSim requires every reservoir to have a target elevation. The reservoir's target elevation is called its Guide Curve and is presented as a function of time. The rule curves define the individual reservoir storage targets at different times of the year and a release schedule typically indicates the total release to be made from the reservoir system as a function of water available in the system and time of the year. Guide curve operation oversees releases to maintain that storage level. For this study, the guide curve is set in the conservation zone. The storage above this guide curve is the flood control pool and the one below it is the conservation pool. The criteria for determining the release from the reservoir is then based on where the current reservoir pool elevation is concerning the guide curve. When the reservoir pool elevation is below the guide curve in the conservation zone/pool, the reservoir reduces the release of water as possible to refill the pool; if the reservoir pool elevation is above the guide curve in the flood control pool, then the reservoir releases

more water as possible than is entering the pool to draw down the pool (Klipsch and Hurst 2021)

3.9.6. Koka Hydropower Plant Parameters

The koka hydropower plant parameters inputted into HEC-ResSim models were: Installed power of 43.2MW, firm power is 34.5MW, station use is null, the total head loss is assumed as 2m, the net head is 42m, overall efficiency is adopted from (Müller et al. 2016) as 90% and the average tailwater elevation is at 66.3m ASD. The monthly strategic power and energy data planned for the year 2021 was taken in excel format from the Power operation and generation office and directly used in this study.

Table 3. 12 Table Strategic Power and Energy Planned in 2021

Months	Average Load (MW)	Monthly Energy	Monthly Energy
		First plan (MWh)	Second Plan (MWh)
Jan	14	9500	10080
Feb	14	9500	10080
Mar	11	9300	7920
Apr	11	8800	7920
May	10	9060	7200
Jun	9	7200	6480
Jul	14	31104	31104
Aug	14	31104	31104
Sep	14	31104	31104
Oct	14	12400	10080
Nov	14	10080	10080
Dec	13	10600	9360

3.9.7. Downstream Water Needs

Even though Koka reservoir was constructed in 1959 for hydropower generation, later it is forced to support irrigation water demand and domestic, municipal, and industrial (DMI) water demand particularly, during the dry season. In the latest study by ARBA (2017), Awash River Basin has an irrigation potential of about 200,000ha, of which about 69,000ha (NCORE Workshop 2017) were found in the upper valley (from koka dam to Amibara). Wonji Sugar Factory, Metehara Sugar Factory, Africa Juice Factory, and Upper Awash Agro-Industry are a few of the large irrigation schemes found in the upper valley of the Awash basin and their irrigation water demand relies on the release of the koka reservoir. The domestic, municipal, and industrial (DMI) water requirement is usually a steady demand, whereas irrigation has a strongly varying demand according to the cropping pattern and climate. Most of the water consumption for domestic, industrial, and municipal (for Adama Woreda, Dodota Woreda, Boset Woreda, and Fentale Woreda) found in the upper valley of Awash Basin also depends on release from the koka reservoir. In this analysis, Water demands for irrigation, DMI, and environmental flow during the dry season were adopted from the report of the Awash River Basin Water Resource Administration Directorate (ARBA 2020). The details of each demand are summarized in Appendix E

Table 3. 13 Water Allocation from Koka Reservoir (ARBA 2020)

Month	Irrigation Demand (m ³ /s)	DMI Demand (m ³ /s)	Environmental Flow (m ³ /s)
Jan	29.77	7.31	3.44
Feb	26.15	7.02	3.02
Mar	27.28	7.02	3.18
Apr	22.86	6.64	2.78
May	28.71	7.14	3.56
Jun	28.79	6.91	3.73
Jul	10.97	7.11	3.79
Aug	10.47	7.11	3.64
Sep	25.23	7.11	3.82

Oct	27.62	7.11	3.76
Nov	33.14	7.49	3.85
Dec	29.97	7.34	3.49

3.9.8. Koka Reservoir Operation Alternatives

In HEC–ResSim, alternatives were developed from the reservoir network (previously created from a watershed configuration) and operation rules. ResSim has several operation rules that are used as a method of resolving conflicts with one or another release from the reservoir. The availability of the rule type varies, depending on the release element. In this study, demand and release elements were prioritized to look for optimal release and power generation. According to the prioritization rule, the release regulation was distributed throughout the conservational zone and minimum operation level by sequential release allocation approach. The power plant outlet element gets the release first until it reaches release capacity. After the capacity through the powerhouse is reached, the bottom outlet and spillway-controlled outlet start to release water.

Water release from the koka reservoir downstream is differentiated between constant demand and time variable demand. The domestic, municipal, industrial (DMI), and environmental water requirement is usually a steady demand, whereas irrigation has a varying demand according to the cropping pattern and climate. For this study, three alternatives were selected based on the prioritization rule between the water demands of hydropower, irrigation, and DMI, so that maximum energy and downstream water release was expected. In the simulation, monthly Energy, monthly Irrigation, monthly DMI, and monthly environmental flow were applied in the conservation zone.

Table 3. 14 Demand Prioritization in Conservation Zone

Zone	Alternative 1 (ALT-1)	Alternative 2 (ALT-2)	Alternative 3 (ALT-3)
Conser vation zone	Hydropower	Irrigation	DMI
	Irrigation	Hydropower	Hydropower
	DMI	DMI	Irrigation
	Environmental	Environmental	Environmental

3.9.9. Reservoir Optimization

The Reservoir Simulation model determines the reservoir operation release rule over a given period with known stream flows at the input point to the reservoir and other control points throughout the system for both periods. For this particular study, the HEC-ResSim model was used to operate the koka reservoir as best to meet the power generation and flow requirements of different water uses. This study uses simulation techniques by incorporating optimization techniques. However, the optimization technique is an indirect approach to determining the optimal solution by successive trial and error procedures. Through each iteration step, the simulation method enables the behavior of water level, the power generation output, and downstream water requirement. The system reservoir simulation considers a comparison among alternatives to obtain the maximum value of an objective function. The search for the best alternative in the process depends on the analyst's skill at effectively manipulating the design factors and operating policy. The HEC-ResSim model can compare alternatives to obtain the maximum value of the objective function in the simulation reservoir system. For this study, demand priority was applied to the koka reservoir release for power, irrigation, and downstream release. Therefore, the alternative that fulfills the following condition will be selected as the best alternative.

- ❖ maximum system power generation,
- ❖ Maximum water release for downstream
- ❖ Minimum spillage loss.

Here in this thesis demands were prioritized as alternatives to come up with optimal power and downstream release. Monthly Irrigation demand, DMI demands, and environmental flows were taken from the previous study report of (ARBA 2020) as stated in section 3.9.8.

3.9.10. Storing Time-Series Data in HEC-DSS Tool

HEC-DSSVue is a tool for transferring time series data from HEC-DSS database storage to a working space or it allows to access data stored in the HEC-DSS database. The tool is comprised of two visual basic executables that utilize an object library and object classes within the database structure (DSS catalogs) and contains all relevant records and descriptors to automatically transfer the time series data during the simulation process. The time series data of the reservoirs have been stored in the HEC-DSSVue file. The rainfall and stream flow time series was stored in the HEC-DSS tool and provided to the models. The DSS pathname consists of six parts in the following format.

A/B/C/D/E/F

Where:

A - Group name for the data such as a watershed name, study name, or any identifier which allows the records to be recognized as belonging to a group.

B – This part identifies location such as site name or organization ID

C – The parameter of the data such as flow, precipitation, storage, or evaporation.

D – The start date of the time series.

E – The time interval for regular data or the block length for irregular interval data.

F – An optional descriptor that can be used for additional information about the data.

E.g. /Upper Awash Basin/Hombole/Flow/1Day/MWR/

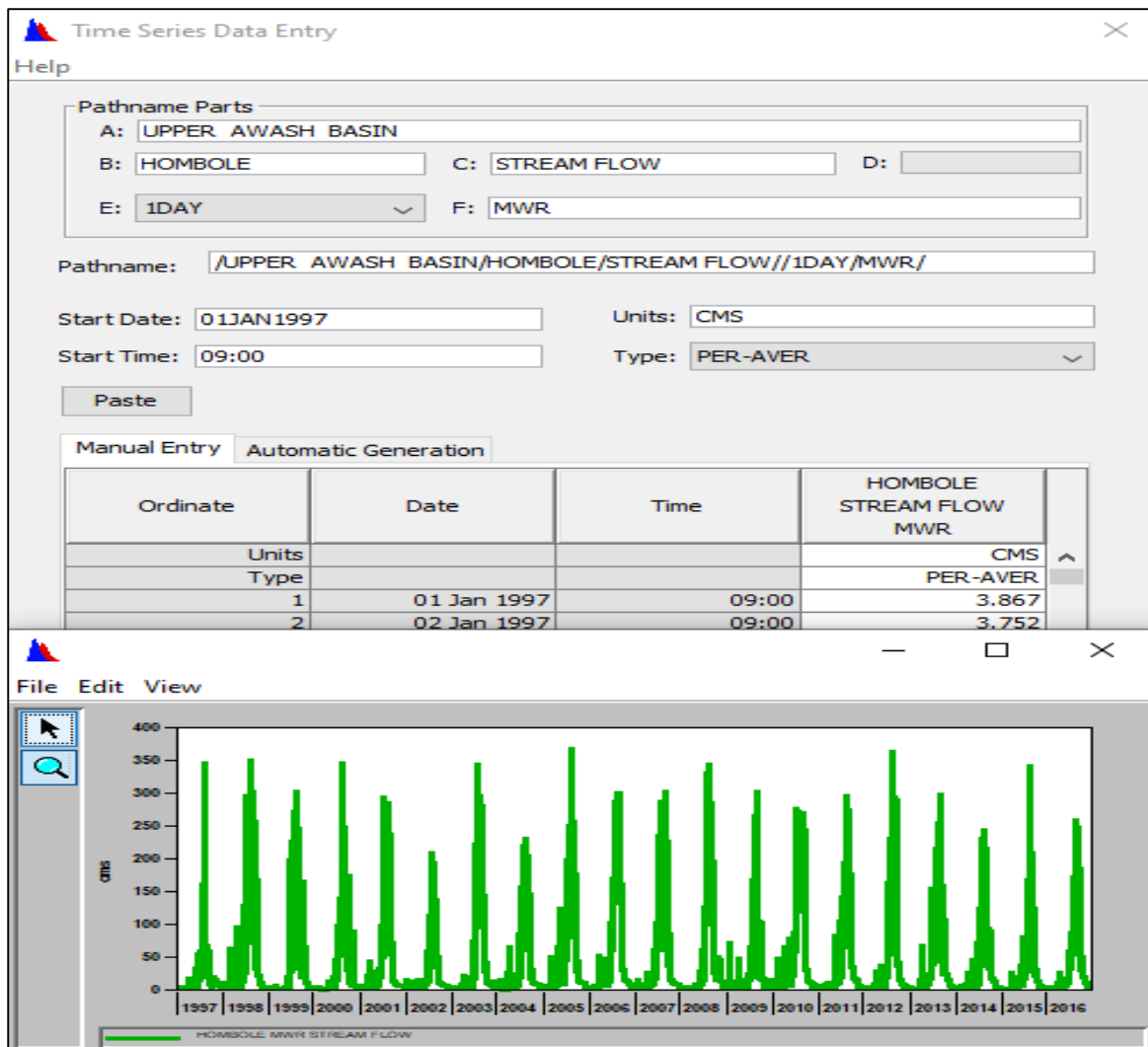


Fig 3. 18 Sample Stored Data in DSS

3.9.11. Reservoir Simulation Procedures

Koka Hydropower reservoir modeling for optimal operation consists of the following steps:

- i. Preparation of the time series data obtained from the Ministry of Water Resources and storing the data set in the HEC Data Storage System (HEC-DSS).
- ii. In the launched watershed module, import the georeferenced river map processed in ArcGIS 10.4 and draw alignment, computational points, and nodes.
- iii. From the reservoir network module, draw reaches (including the reach parameter) and assign the junction properties.
- iv. Add the Storage-elevation-area curve and evaporation loss to the reservoir pool, controlled (spillway and power plant) and uncontrolled (if any) gates to the dam tree, and finally zones and rules to the operational tab.
- v. Assigning the required information or data for each gate, zone, and rules prepared through manual or spreadsheet.
- vi. Set alternatives and access the input time-series data via HEC-DSS tools to the assigned junctions in step iii.
- vii. Run the model from the simulation setup for each alternative set.
- viii. Analyze the computation result and select the best alternative based on the criteria mentioned in section 3.9.6
- ix. Finally, select the best alternative that fulfills the condition stated in step viii based on the priority of demands and analyze the discussion of results.

3.9.12. Reservoir Operation Assumptions and Initial Conditions

The following assumptions and initial conditions were used in this study:

- i. Reservoir storage was considered constant (i.e. no reduction due to sediment)
- ii. Seepage through the reservoir and the body of the dam was neglected.
- iii. Seepage and evaporation through any reach are negligible and assumed to be zero
- iv. Only free water surface evaporation loss was accounted for.
- v. In this case, the reservoir's initial condition or look back assumed was:
 - ❖ The reservoir conservation level is at 110.3m.
 - ❖ Spillway can release a maximum of 1000m³/s when all four gates opened.
 - ❖ The maximum discharge for power generation is 126.9m³/s

4. RESULT AND DISCUSSION

General

HEC-HMS Hydrological model and HEC-ResSim Reservoir simulation model were importantly used to attain the goal of this study. The inflow to the reservoir is the main input data for reservoir simulation and hence, water availability in the upper Awash River basin was simulated with the help of the HEC-HMS model then reservoir operation was followed by using the HEC-ResSim model. Hence, these models can support the decision-making process which will enable the government and policymakers to formulate and implement water resource management options and appropriate response strategies.

4.1. HEC-HMS Simulation Results

In this study, HEC-HMS Hydrologic Model was used for the upper awash sub-basin for Stream flow simulation. The basin model was created in the HEC-HMS model through the GIS menu and the parameters of the loss method, transform method, base flow method, and routing methods are estimated for calibration by the optimization trial manager which is later adjusted by manual calibration to get the acceptable value of NSE and R^2 .

4.1.1. HEC-HMS Model Calibration and Validation

Before calibration and validation proceeded, the model was initialized with the default parameter of the model using rainfall data for 23 years (1997-2020). After the model was initialized calibration and validation were followed at four gauge stations found in the site based on their contribution to the flow to the reservoirs. These stations are Awash@Melka_Kunture, Awash@Melka_Hombole, Mojo@MojoIV, and koka inlet at the Upper Awash Basin outlet. For all gauging stations, calibration was conducted by two third of daily observed data (1997-2010) and validation was made by one-third of daily observed data (2011-2016). The model simulates a peak flow of $502.2\text{m}^3/\text{s}$ on 17 Aug 1999 and a volume of 49.7BCM at koka inlet.

Calibration and validation results of the koka inlet were shown as a sample whereas the graph of the other three stations was summarized in Appendix E.

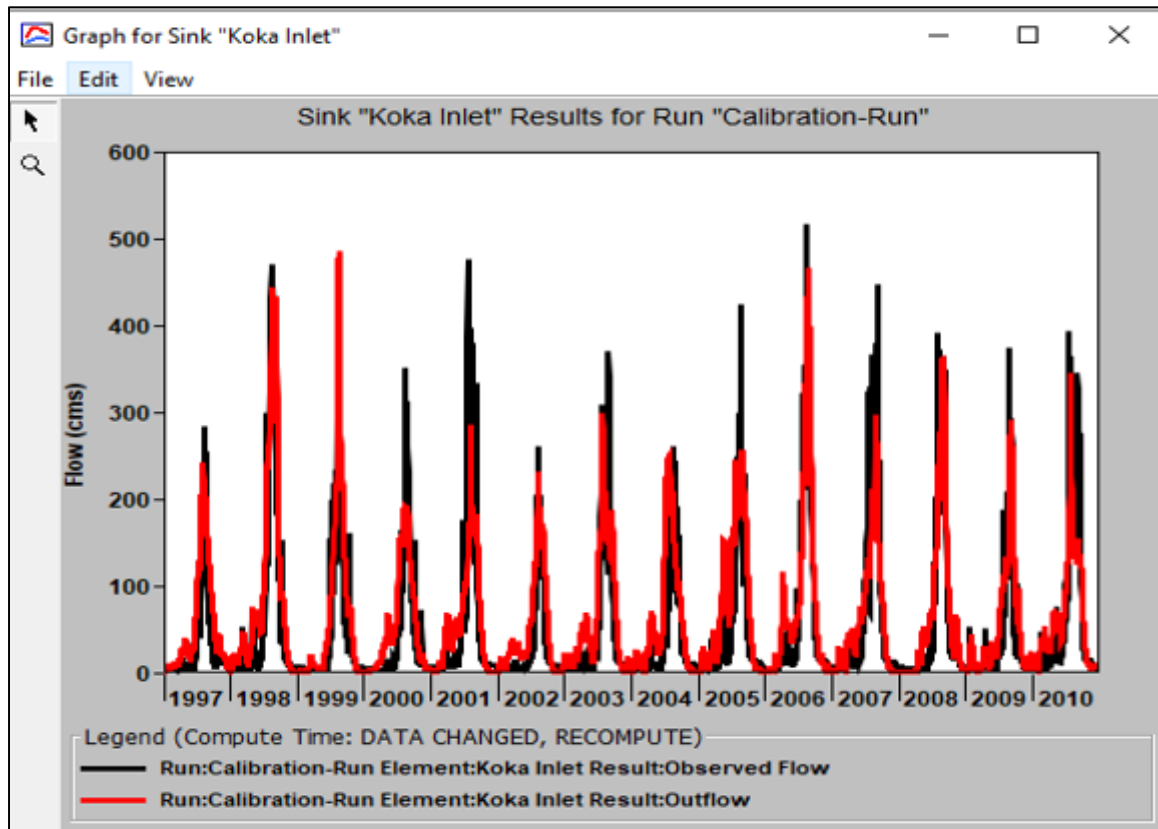


Fig 4. 1 HEC-HMS Model Calibration at Koka Reservoir Inlet

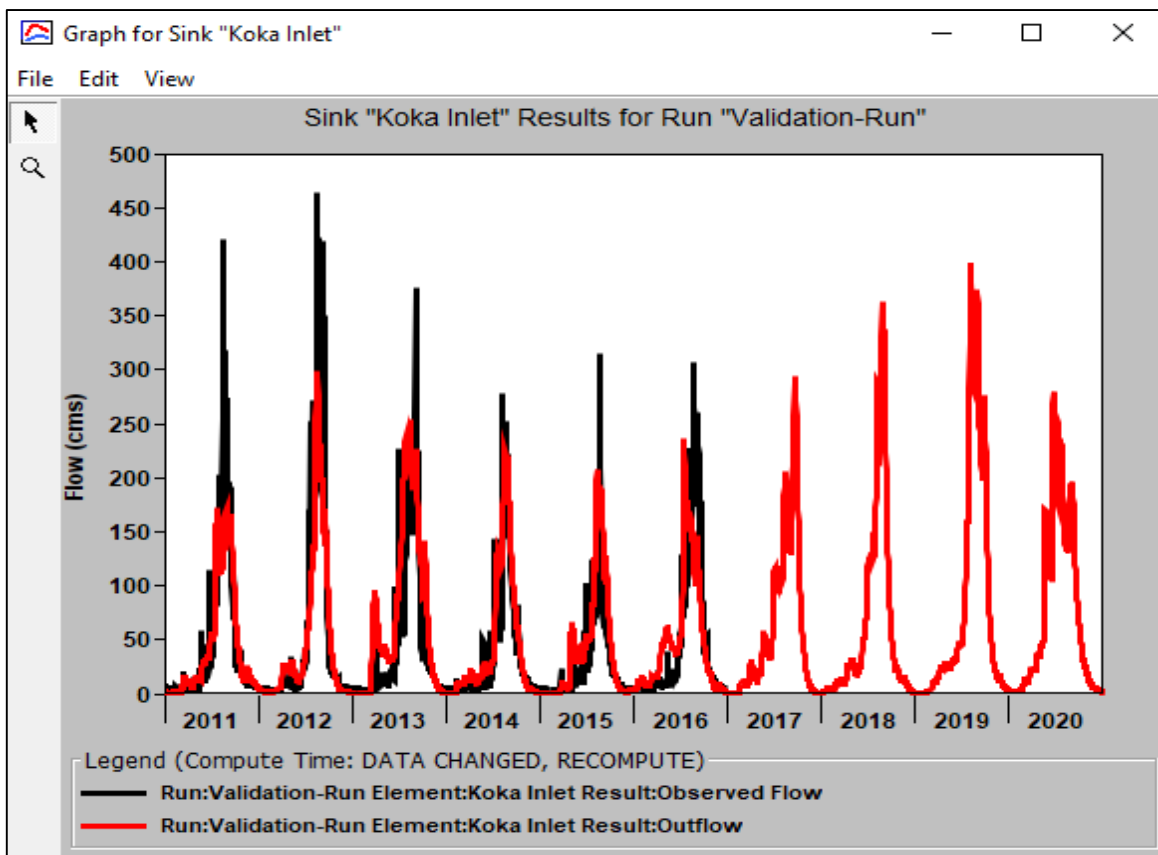


Fig 4. 2 HEC-HMS Model Validation at Koka Reservoir Inlet

As one can observe from Figures 4.1 and 4.2 above, the trend and shape of the simulated and observed hydrographs are nearly similar for both calibration and validation runs. However, the model slightly overestimates and mostly underestimates the peak flow in all gauges. For instance, at the koka inlet, the model overestimates only in the year 1999 during the calibration run, and slightly underestimate in the other periods of the simulation run during both the calibration and validation run. In general, the HEC-HMS model performs well for all sub-basins unless the mojo subbasin. For the other three upstream gauging stations i.e., Melka Kunture, Hombole, and Mojo the Hydrograph results of the Calibration and Validation Run was shown in Appendix E.

The model performance was also checked using the quantitative values of model performance such as Nash-Sutcliffe Model Efficiency Coefficient (NSE), Coefficient of determination (R^2), and Percent difference as shown in Table 4.1 below.

Table 4. 1 HEC-HMS Model Performance Indices

Parameter	Daily Calibration				Daily Validation			
	M.Kunture Gauge	Hombole Gauge	Mojo Gauge	Koka Inlet	M.Kunture Gauge	Hombole Gauge	Mojo Gauge	Koka Inlet
NSE	0.7	0.78	0.64	0.81	0.73	0.76	0.53	0.77
R2	0.74	0.77	0.64	0.77	0.74	0.75	0.52	0.76
PB	0.36	2.69	5.41	8.76	5.79	3.77	-2.58	2.78
RMSE	0.5	0.5	0.6	0.4	0.5	0.5	0.8	0.5

As seen from table 4.1 above, the result of calibration and validation shows an acceptable range of performance indices based on the criteria pointed out by different researchers under section 3.7.1. The model results are very good for Koka Inlet and Hombole stations, good for the Melka Kunture station, and satisfactory for the Mojo station for calibration and validation. In general, overall model performance indicates that a strong correlation exists between observed and simulated inflows and, hence, the model can predict the water potential of the upper awash basin.

Therefore, from now onwards the simulated flow gained from HEC-HMS is used as an observed flow for HEC-ResSim in reservoir simulation.

4.2. HEC-RESSIM Simulation Result

The HEC-ResSim model was configured to simulate the reservoir behavior, using simulated hydrologic data of 20 years from 2000 to 2020. Accordingly, the simulation result (inflow-outflow, reservoir storage, reservoir release, and generated power) were presented in the following section in tabular form and graphically with a detailed discussion.

4.2.1. Reservoir Inflow and outflow

Reservoir inflow time series data are the main input data in HEC-ResSim model and required at the most upstream reservoir elements. These inflow time series were simulated in HEC-HMS model at: Melka Kunture, Hombole and Mojo gauging stations. These inflow sources are located at considerable distances from reservoir inlet, so HEC-ResSim routes the flow to compute the total inflows to the reservoir. HEC-ResSim model calculates the inflow and outflow at reservoir inlet and outlet point. Accordingly the reservoir inflow and outflow report is shown in table 4.2 and the minimum and maximum inflow to the koka reservoir varies between 2.18m³/s and 507.99 m³/s whereas the maximum and minimum outflow varies between 6.64 m³/s to 666.9 m³/s.

Table 4. 2 Summary of Simulated Inflow-Outlet Flow of All Alternatives

Location/Parameter	Average	Maximum	Minimum
CP1 (Inflow to Reservoir)			
Regulated Flow (cms)	70.87	507.99	2.18
CP2 (Outflow from Reservoir)			
Regulated Flow (cms)	91.2	666.9	6.64
Unregulated Flow (cms)	70.87	507.99	2.18
CP3 (Flow at Hombole Gauging Station)			
Regulated Flow (cms)	68.10	488.98	0.41
CP3 to CP1(Flow from Hombole to Koka Reservoir)			
Regulated Flow (cms)	68.10	496.2	1.66
CP4 (Flow at Mojo Gauging Station)			
Regulated Flow (cms)	2.76	20.90	0.00

CP4 to CP1 (Flow from Mojo to Koka Reservoir)			
Regulated Flow (cms)	2.77	22.35	0.00
CP5 (Flow at Melka Kunture Gauging Station)			
Regulated Flow (cms)	28.38	207.70	0.27
CP5 to CP3 (Flow from Melka Kunture to Hombole)			
Regulated Flow (cms)	28.38	215.5	0.12
Koka Reservoir			
Regulated Flow (cms)	91.2	666.9	6.64

4.2.2. Reservoir storage of All Alternatives

Table 4. 3 Summary of Reservoir Storage for All Alternatives

Alternatives	Minimum Pool Storage (MCM)	Average Pool Storage (MCM)	Maximum Pool Storage (MCM)
ALT-1	126	708	1198
ALT-2	566	938	1191
ALT-3	1018	1133	1196

According to (ARBA 2017b), currently, the sediment accumulation has been approaching 102m ASD due to the dredging intake being clogged. Hence, it is difficult to operate the power intake at the critical condition when reservoir storage falls below 200MCM. From Fig 4.3 the reservoir storage falls below 200MCM in the dry months of the years 2003 and 2016. Therefore, the reservoir wasn't generating power during these years for certain months. But, when Irrigation or DMI has given priority the reservoir store much water in each simulation year. In reality, the operators do not recommend the reservoir store water all year because it will cause flooding in the next rainy year. Fig 4.4 and Fig 4.5, show that the system doesn't release sufficient water through the turbine to generate power when Irrigation or DMI is given priority. Therefore, less power will be expected in these alternatives compared to ALT-1 even

though the reservoir stores much water in the conservation zone. Therefore, since the reservoir did not release enough water and these alternatives are not recommended.

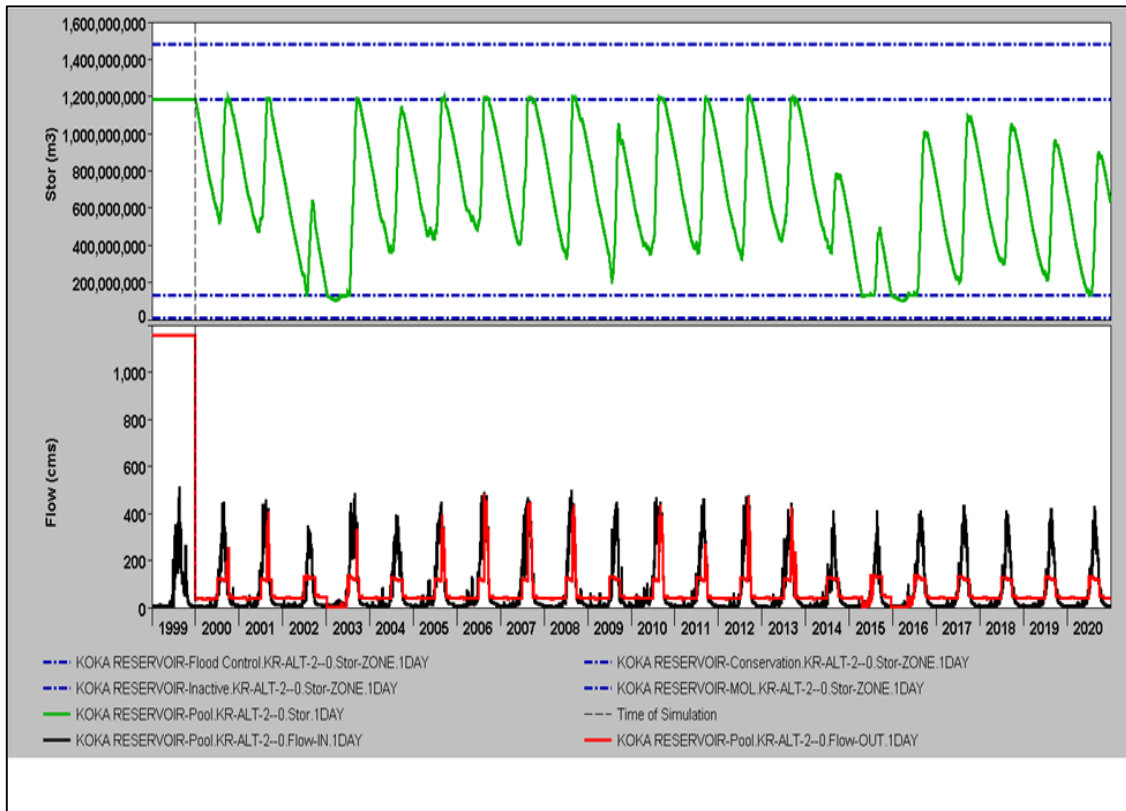


Fig 4. 3 Koka reservoir pool level and inflow-outflow rate for Alternative 1

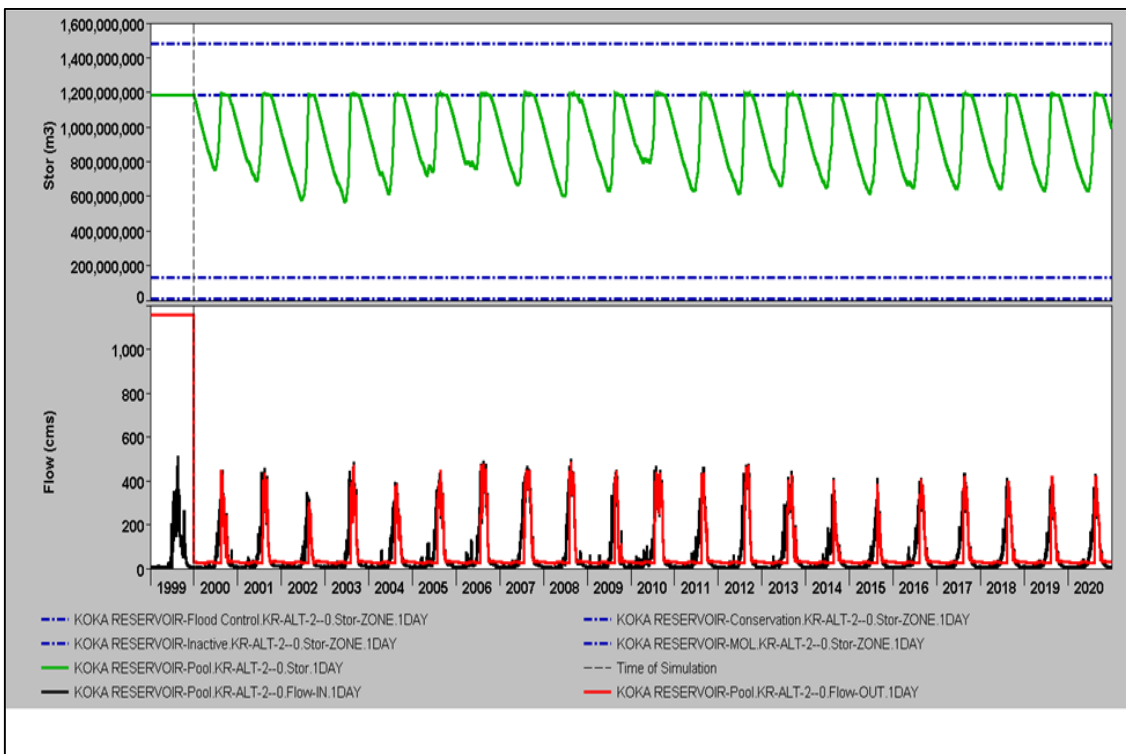


Fig 4. 4 Koka reservoir pool level and inflow-outflow rate for Alternative 2

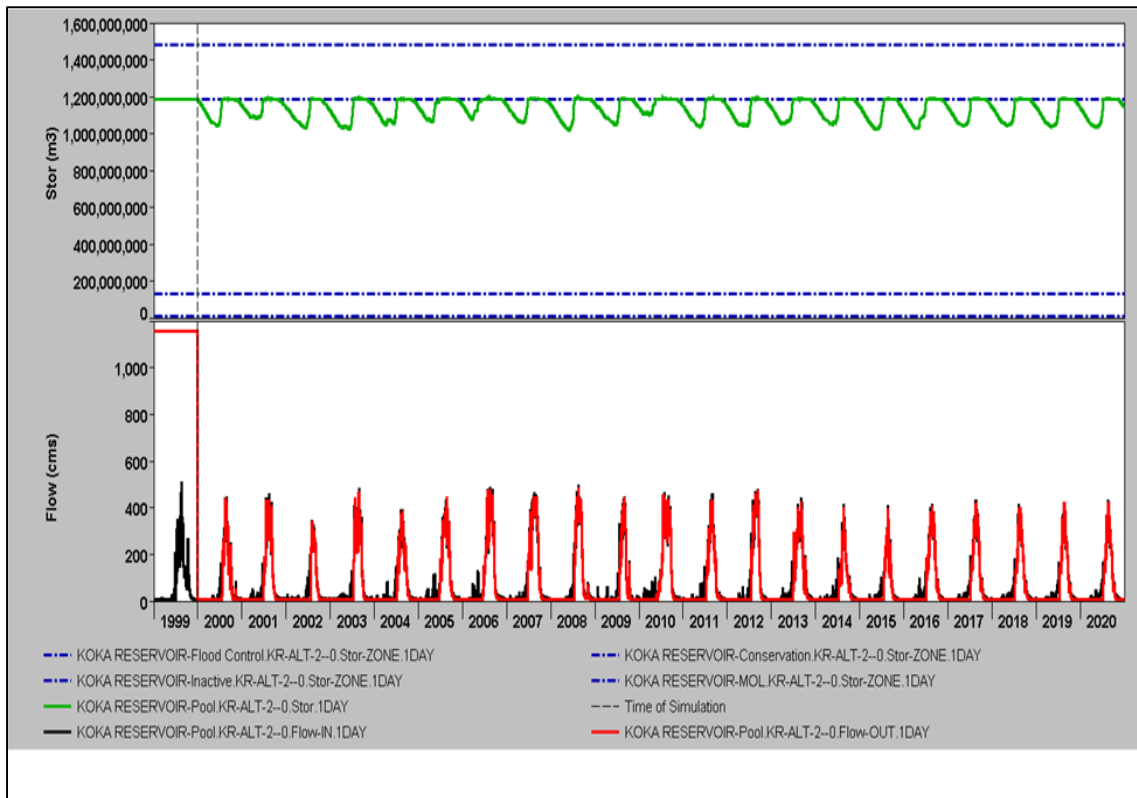


Fig 4. 5 Koka reservoir pool level and inflow-outflow rate for Alternative 3

4.2.3. Reservoir Release of All Alternatives

The koka reservoir releases water through a power plant outlet (primary), low-level spillways, and evaporation from the reservoir. As seen from table 4.5 the power plant releases an average of 62.72m³/s, 58.87m³/s, and 55.52m³/s for ALT-1, ALT-2, and ALT-3 respectively. The release observed in ALT-1 is more reasonable than other alternatives because the minimum release through power plant units shows an increment of 14.2% over the average water required for irrigation suggested by (ARBA 2020). If the release from the koka spillway is greater than 300m³/s, it causes serious damage to downstream settlements unless early warning was taken (Shemeles 1998). Therefore spillway release in all alternatives (Table 4.4) is less than the suggested value, thus not causing serious damage to downstream settlements. The reservoir also, losses an average of 5.7BCM per year of water by evaporation.

In general, ALT-1 seems the best alternative, as the water released through the power unit can generate maximum power as well as support downstream irrigation without any deficiency. The water released through the spillway also cannot cause any catastrophic damage to downstream settlements.

Table 4. 4 Summary of Water Release through Different Outlets of All Alternatives

Alternatives	Flow Through Power Plant (cms)			Flow Through Spillway (cms)		
	Minimum	Average	Maximum	Minimum	Average	Maximum
ALT-1	28.64	62.72	326.15	0	2.23	199.98
ALT-2	22.86	58.87	326.15	0	1.32	116.76
ALT-3	6.64	55.52	326.15	0	1.42	116.76

4.2.4. Power and Energy Generation of All Alternatives

Table 4. 5 Summary of Simulated Power and Energy of All Alternatives

Alternatives	Parameters	Average	Maximum	Minimum
ALT-1	Generation Efficiency	0.9	0.9	0.9
	Power Head (m)	39.58	42.07	34.46
	Hydraulic Head (m)	2	2	2
	Energy Generated per Time Step (MWh)	504.77	1036.8	240.0
	Power Generated (MW)	21.03	43.2	10.0
	Plant Factor	0.49	1.00	0.3
	Flow for Power Generation	59.76	136.53	32.87
ALT-2	Generation Efficiency	0.9	0.9	0.9
	Power Head (m)	40.92	42.02	38.96
	Hydraulic Head (m)	2	2	2
	Energy Generated per Time Step (MWh)	379.64	1036.8	192.54
	Power Generated (MW)	15.82	43.2	8.02
	Plant Factor	0.37	1.00	0.19
	Flow for Power Generation	43.4	116.77	22.86
ALT-3	Generation Efficiency	0.9	0.9	0.9
	Power Head (m)	41.82	42.02	41.36

Hydraulic Head (m)	2	2	2
Energy Generated per Time Step (MWh)	307.21	1036.8	58.18
Power Generated (MW)	12.8	43.2	2.42
Plant Factor	0.3	1.00	0,06
Flow for Power Generation	34.58	116.74	6.64

4.2.5. Reservoir Simulation

As one can easily understand from the summary Table 4.5 above, the alternatives applied to different reservoir water requirements, the model gives different energy and power for each alternative. The maximum energy generated from ALT-1, ALT-2, and ALT-3 is 504.77GWh, 379.64GWh, and 307.21GWh per day, respectively. According to the optimization rule described in section 3.9.6, the alternative with higher energy was selected as the best alternative. Therefore, the energy generated in ALT-1 (504.77MWh) is greater than the other two alternatives. From now onwards, all results obtained under ALT-1 will be discussed as an optimal alternative for the Koka reservoir.

4.2.6. Power and Energy Generated

Table 4. 6 Summary of Simulated Power and Energy

Power Parameters	Minimum	Average	Maximum
Power Generated (MW)	10.0	21.03	43.2
Energy Generated per Time Step (MWh)	240.0	504.77	1036.8
Flow for Power Generation (cms)	32.87	59.76	136.53

Figure 4.6 shows that the simulated power follows an irregular pattern through its simulation period. The plant generates maximum power of 43.2MW during rainy months (from the beginning of July to the end of September) and a minimum power of 10MW for the rest months excluding the years 2003 and 2016. The plant generates null power for about five months (from January to June) in the years 2003 and 2016, due to the reservoir pool level falling below the minimum operating (103m) level in the years 2003 and 2016. In general, power generated

under this alternative was validated with the power planned in the year 2021 by EEP and shows good agreements with the correlation of 0.993 as shown in Fig 4.7.

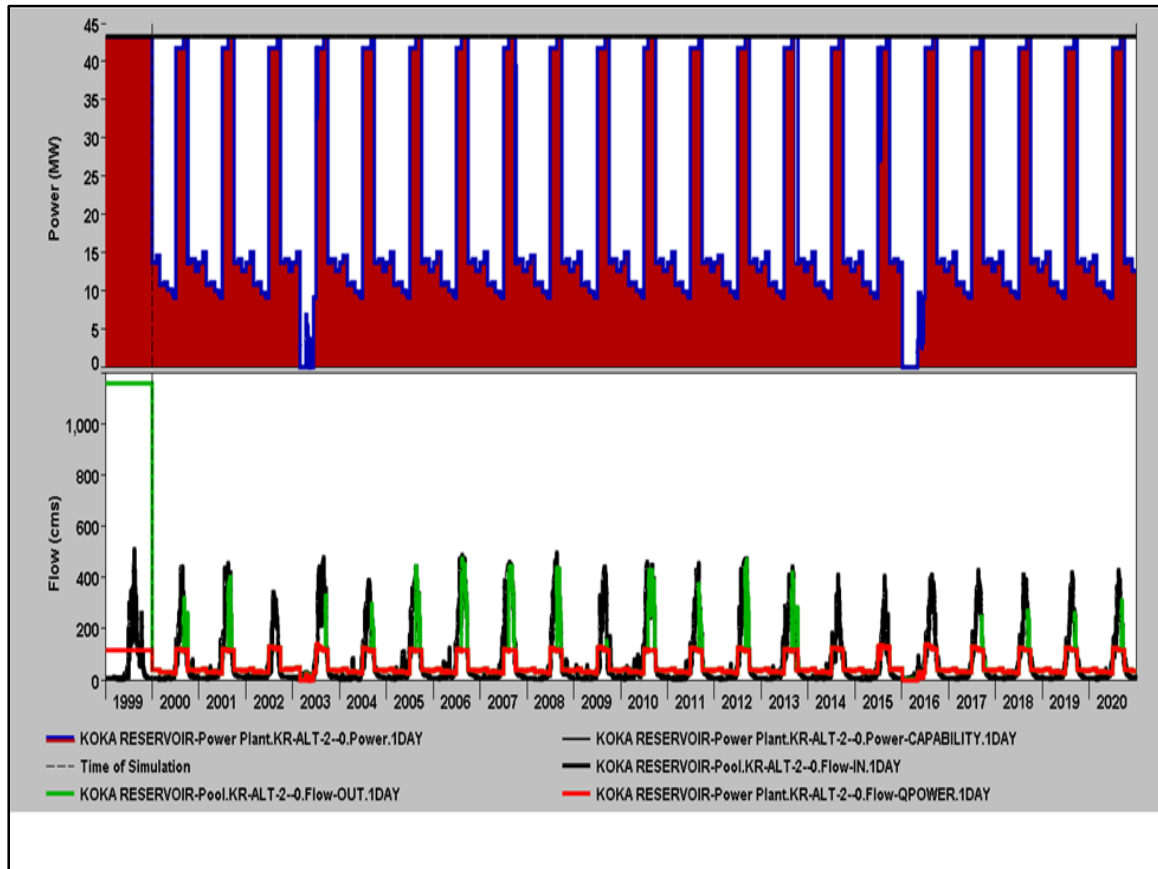


Fig 4. 6 Simulated Koka Power Plant, and Power flow, Plots.

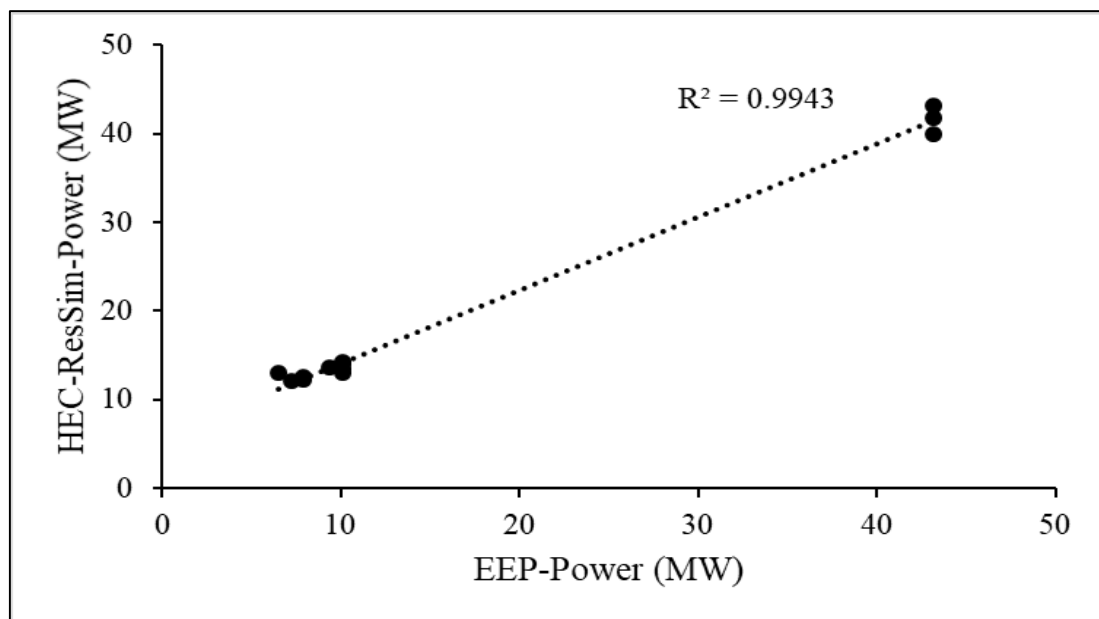


Fig 4. 7 Correlation between ResSim and EEP Power

4.2.7. Firm and Secondary Power

The amount of power that can be generated with certainty without interruption at a site, is the firm power whereas power generated more than 50% of the time is the secondary power. This varies in amount from month to month and year to year, depending on the hydrological conditions (Kumar 2011).

The power duration curve was analyzed in an Excel spreadsheet using the power obtained from ALT-1. The power and energy that could be available all the time are 10MW per day and 240MWh per day, respectively. The secondary power is 14MW per day which is equivalent to the minimum design power of EEP.

Table 4. 7 Power Duration Curve

Month	Power in MW			Descending		Probability (%)
	Minimum	Average	Maximum	Order	Rank	
Jan	12.77	12.06	12.77	43.20	1	8.30
Feb	13.65	12.67	14.14	41.81	2	16.70
Mar	12.50	11.31	12.50	41.11	3	25.00
Apr	12.22	11.09	12.22	17.03	4	33.33
May	12.18	11.14	12.18	14.25	5	41.67
Jun	10.00	10.00	10.00	14.00	6	50.00
Jul	41.81	41.11	41.81	13.65	7	58.33
Aug	41.81	42.02	43.20	12.77	8	66.67
Sep	43.20	43.20	43.20	11.31	9	75.00
Oct	16.67	17.03	43.20	11.14	10	83.33
Nov	14.00	14.00	14.00	11.09	11	91.67
Dec	14.25	14.01	14.25	10.00	12	100.00

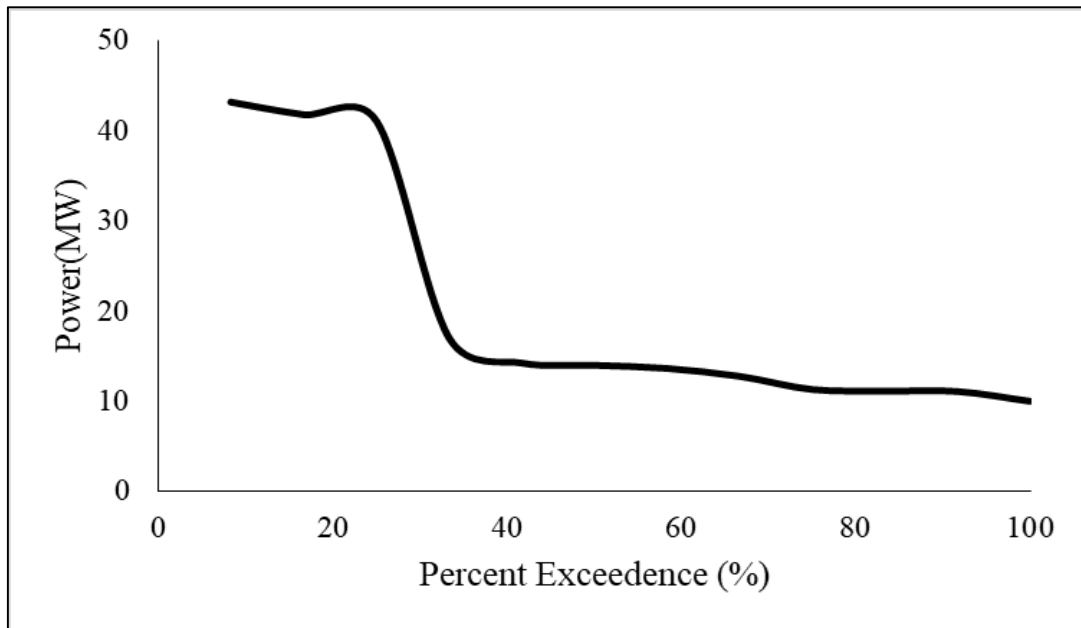


Fig 4. 8 Koka Power Plant Power guide curve

4.2.8. Koka Reservoir Release

Table 4. 8 Water Release through Different Outlets

Outlets	Minimum	Average	Maximum
Flow Through Power Plant (cms)	28.64	62.72	326.15
Flow Through Spillway (cms)	0	2.23	199.98

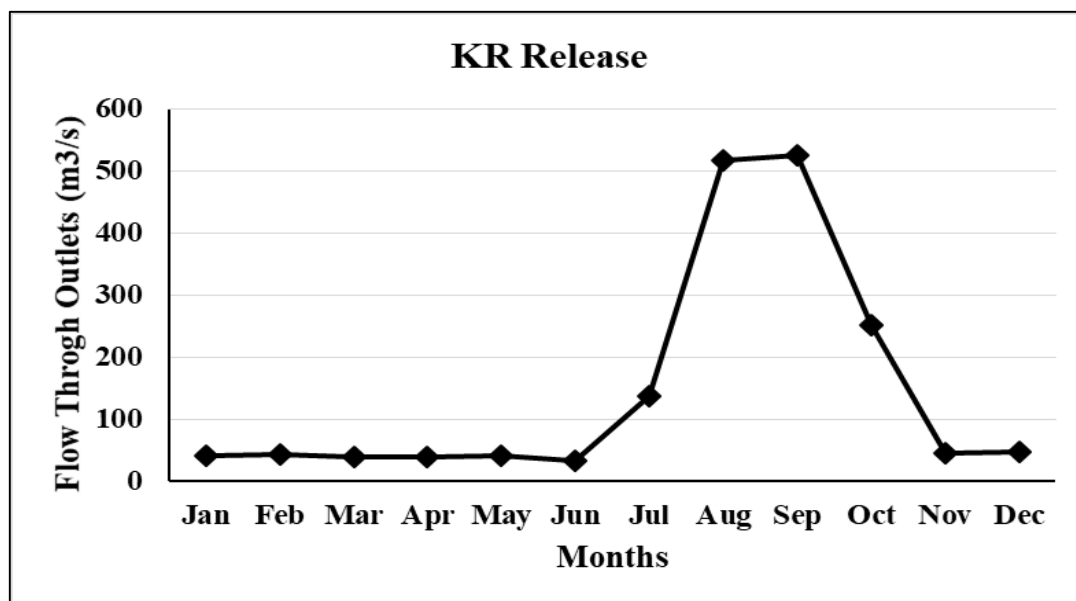


Fig 4. 9 Maximum Release Through Outlets

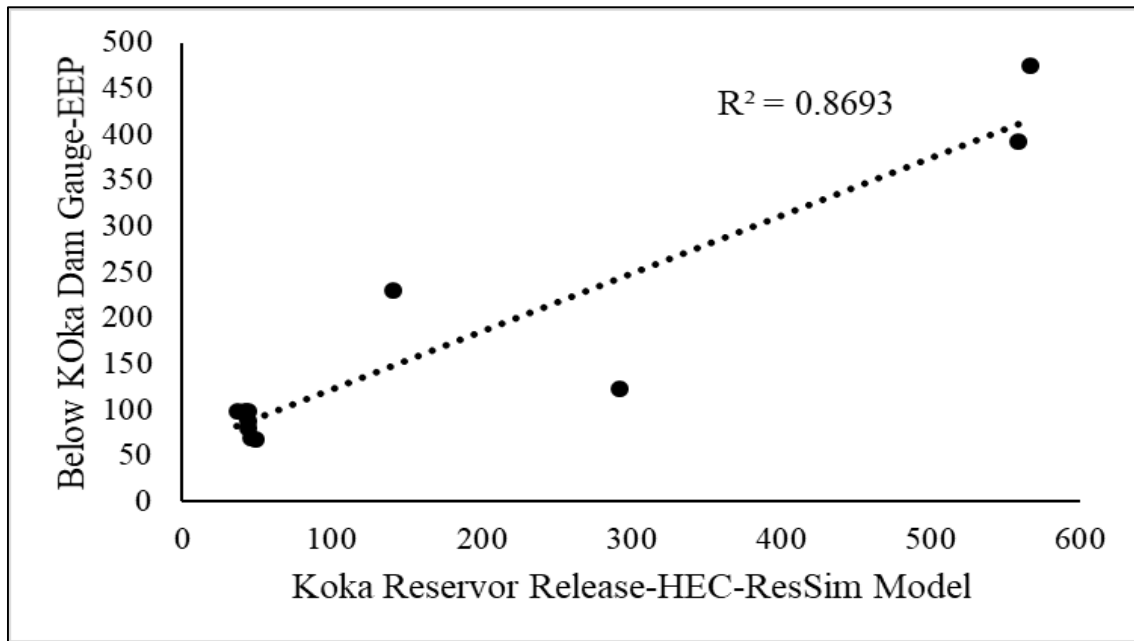


Fig 4. 10 Correlation b/n ResSim Release and Flow at Below Koka Gauge Station

Fig 4.10 above shows that the simulated water release by the HEC-ResSim model shows a good agreement with flow measured at the gauging station found immediately below the dam, and therefore the model performs well.

4.2.9. Koka Reservoir Guide Curve

A guide curve is a guideline that proposes the status of the reservoir as a function of time for one year. As discussed in section 3.9.6 the release decision logic in ResSim starts and ends with the guide curve. According to this logic, the reservoir releases more water than is entering the pool when the reservoir’s pool elevation is above the guide curve (above 110.3m in this case), and the reservoir releases less water than is entering the pool when below the guide curve (103.0m in this case).

As anyone can observe from Fig 4.11, the reservoir elevation trajectory line oscillates in the conservation zone. But, the reservoir pool elevation crosses the minimum operation level twice (2003 and 2016) in the simulation period. Apart from this, the pool elevation rises to the reservoir FSL (i.e., 110.3m) during the wet months (July to September) and drops down towards MOL (i.e., 103.0m) during the dry season. In general, the reservoir attain its maximum pool level of 110.39m in August and minimum of 103.93m in June.

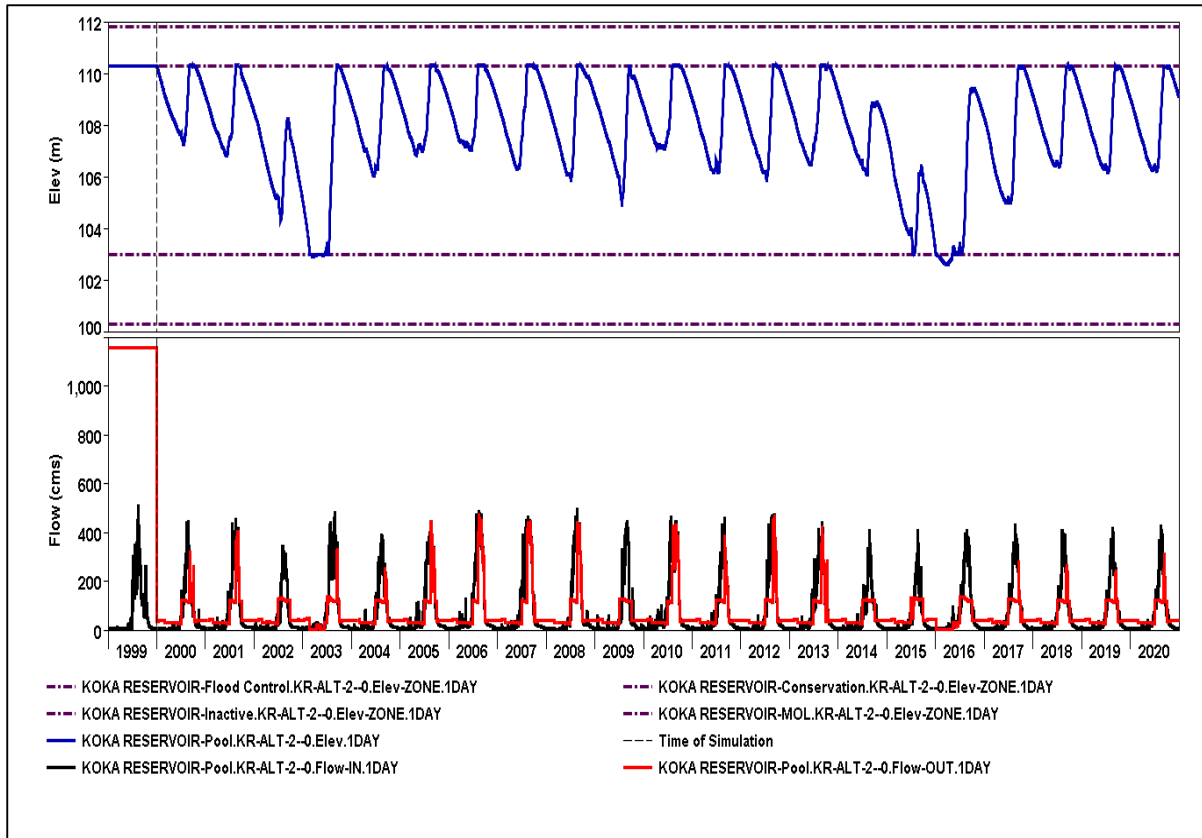


Fig 4. 11 Maximum, average, and minimum reservoir level

It can be seen from fig. 4.12 above, just during the wet season, the guide curve enlarges and increases the electricity generation capacity to avoid spilling, since the reservoirs tend to release more water than entering into it. After ceasing the wet season, the guide curve decreases and reduces water usage for electricity generation to avoid water level falling excessively which enables reservoirs to capture more water than outflow from it. This shows that there was a big possibility to generate reasonable power and can meet the downstream demands without water scarcity during the dry season.

The minimum pool elevation for HEC- ResSim simulation and EEP operation is 103.93m and 103.7m, respectively in June. The reservoir also, gains its mean maximum elevation in September for HEC-ResSim and also for EEP operation. In general, the reservoir is safe for power generation and also, supports downstream users without any water scarcity when power was given priority.

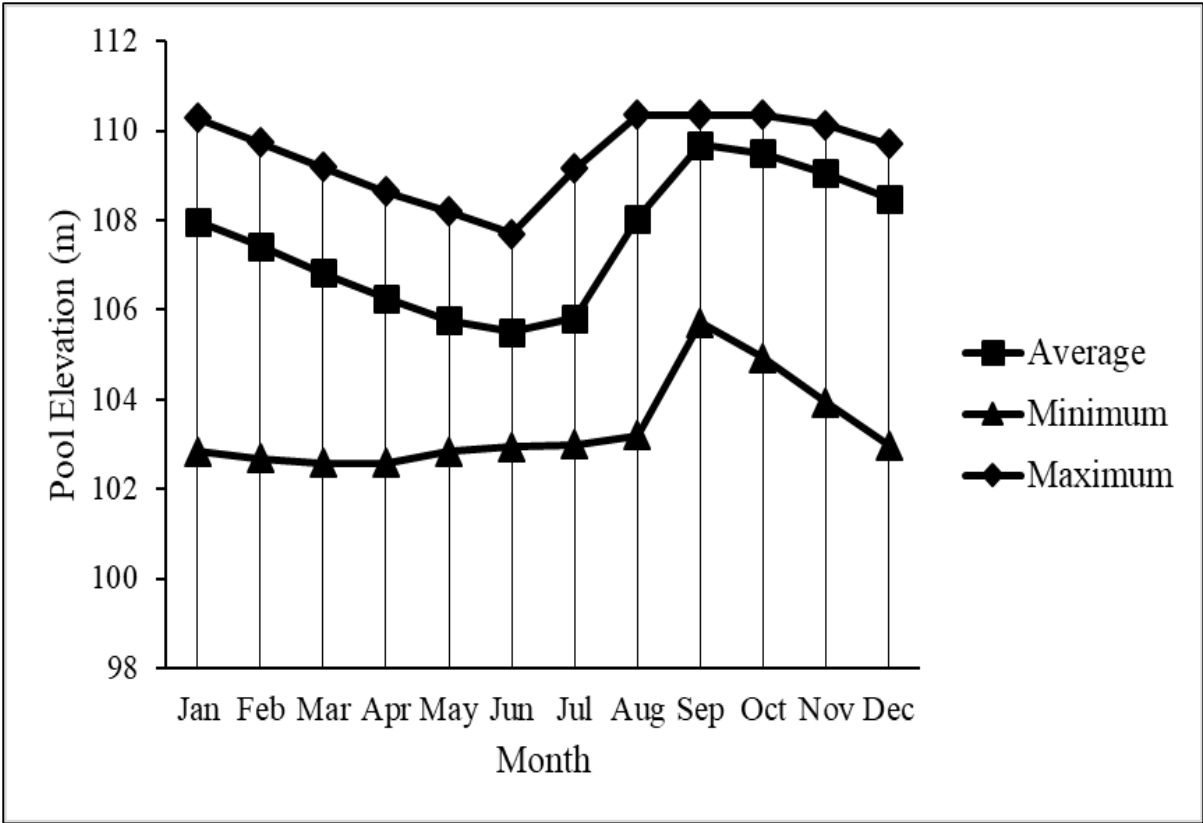


Fig 4. 12 Guide curve for koka reservoir

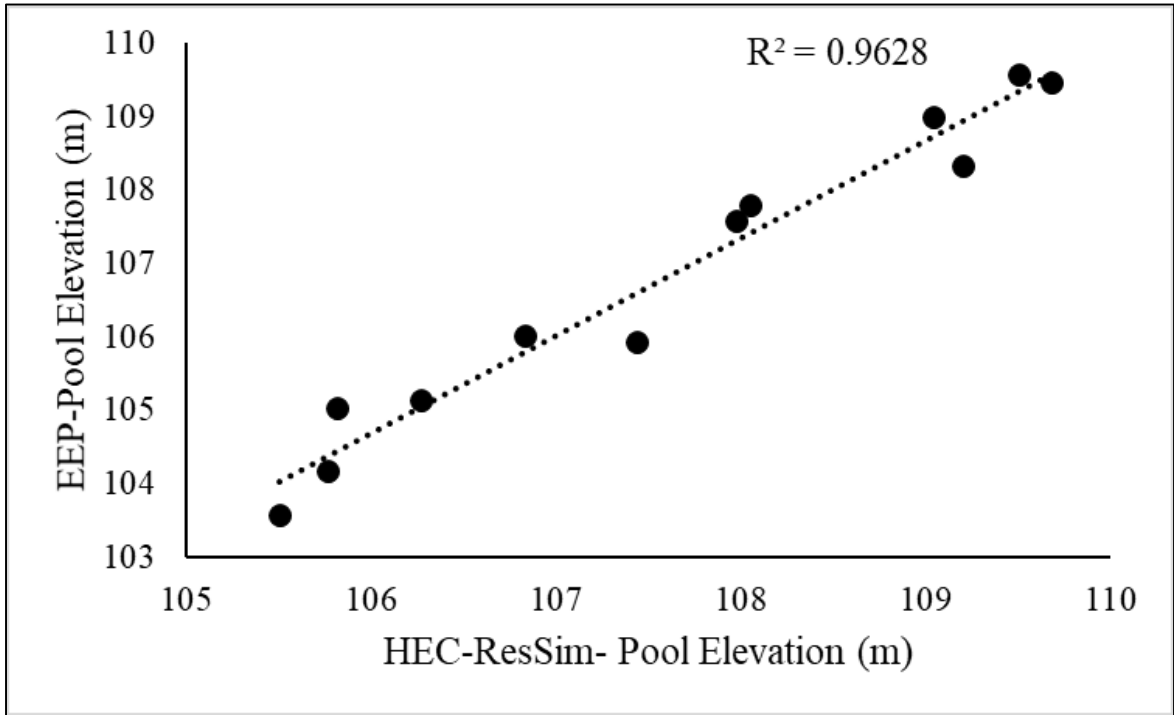


Fig 4. 13 Comparison of HEC-ResSim and EEP study Pool Elevation

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

Freshwater is one of the most important resources our world has. Therefore, it is very important to preserve and manage these valuable assets properly. Water resource problems are quite complex that they are not amenable to a purely mathematical formulation. At best, the involved variables in every model can only be as complex as the modeler can visualize them. In reservoir operation, for instance, release decisions are made in face of uncertainty which includes hydrological uncertainty and meteorological uncertainty. Water resource models are used to minimize these uncertainties, as much as possible because these models can play a major role if they are supplemented with sound engineering judgment in the analysis of problems and interpretation of results. In this study, the inflow to reservoirs was generated by the HEC-HMS model whereas Reservoir operation was done by the HEC-ResSim model to facilitate decision-making based on the available hydro-meteorological data to meet the objective of this study

Koka Reservoir inflow was generated by the HEC-HMS model by using twenty years of daily meteorological data collected from NMA. The generated inflow was calibrated and validated with 16 years of observed flow data of three gauging stations located in the upper awash watershed, namely Melka Kunture, Hombole, and Mojo stations. The performance of the model was evaluated by observing the values of NS, R^2 , RMSE, and PBIAS performance indices reviewed in different literature. The performance of the model at the selected gauge was summarized in the table below.

Parameter	Daily Calibration				Daily Validation			
	M.Kunture Gauge	Hombole Gauge	Mojo Gauge	Koka Inlet	M.Kunture Gauge	Hombole Gauge	Mojo Gauge	Koka Inlet
NSE	0.74	0.78	0.64	0.81	0.73	0.76	0.53	0.77
R2	0.74	0.77	0.64	0.77	0.74	0.75	0.51	0.76
PB	0.36	2.69	5.41	8.76	5.79	3.77	-2.58	2.78
RMSE	0.5	0.5	0.6	0.4	0.5	0.5	0.8	0.5

In general, the model performance indices show the acceptable correlation between observed and simulated inflows and, hence, the HEC-HMS model can predict the water potential of the upper awash basin.

The main objective of this study is to maximize power generation and downstream water release from the koka multipurpose reservoir based on the limited storage and available inflow from the upper awash watershed. All releases from the reservoir mainly depend on the objective of meeting all downstream requirements, particularly irrigation requirements. To achieve this goal HEC-ResSim 3.3 reservoir simulation model was applied. The Koka Reservoir physical and operation data were collected from EEP and AWBA governmental organizations. Specifically, the monthly irrigation demand, DMI demands, and environmental flows were directly adopted from the study report of Awash River Basin Authority conducted by the Water Resources Administration Directorate department in 2020. This study uses three alternatives in the HEC-ResSim model to get the optimal reservoir operation. However; the simulation models do not optimize directly as an optimization model, rather they find the optimal solutions by indirect approach by trial and error. The HEC-ResSim model can compare alternatives to obtain the maximum value of the objective function in the simulation reservoir system. The best alternative will be selected if the alternative should fulfill the minimum nodal requirements (power or irrigation) and simultaneously minimized storage and surpluses over the simulation period. Here in this thesis water release prioritization between three demands: Hydropower, Irrigation, and DMI demands were applied to select the rule with maximum power and downstream release at the same time minimum spillage.

When the highest rule priority is given to Irrigation or DMI demands, the reservoir does not release enough water through the power plant, even though the release from the alternatives can be met their demands. But, when the highest rule priority was given to power demand, the reservoir releases reasonable water through the power plant outlet to generate power, and average water was allocated to downstream demands. Hence operating the reservoir by the highest rule priority (ALT-1) of power demand will give the best result by compromising all demands and constraints of the reservoir. ALT-1 generates a firm power of 10MW and an average energy of 504.64MWh per day which shows an increment of 32% over-irrigation rule priority and 64% over DMI rule priority. The reservoir releases a maximum of 326.15m³/s and 199.98m³/s water through a power plant and spillway outlets respectively. The power plant outlet releases an average minimum discharge of 28.64m³/s which is greater than the average irrigation water demand (25.08m³/s) suggested by ARBA (2020). The reservoirs achieve a

minimum elevation of 103.93m in June (except in the years 2003 and 2016) and a maximum elevation of 110.39m in August. In general, eventhough the effect of climate change and sedimentation were not considered in this study, the reservoir is safe for power generation and also, supports downstream users if power was given the highest rule priority.

5.2. Recommendation

The author of this research recommended the following points to be included in future reservoir studies for better koka reservoir operation.

- ❖ Previously, four bathymetric surveys, i.e., during 1973, 1981, 1988, and 1999 have been carried out on the Koka reservoir. The last bathymetric survey has been carried out 24 years ago by MWR in 1999. At the time survey was conducted, the reservoir lost 40% of its storage capacity and it is very difficult to estimate how much the reservoir lose its storage capacity, now. Therefore, since no bathymetric survey has ever been made in recent times it is highly recommended to use the recent data if available or take any other methods to know the current reservoir storage capacity.
- ❖ The total inflow into the koka reservoir is not directly known from the gauging station, due to the lack of gauge stations at the inlet of the reservoir. Thus, the researcher highly recommends the ministry to install a new gauge station at the reservoir inlet and repair the existing gauges.
- ❖ Nowadays climate change and its impact is becoming a hot issue in different natural and manmade systems in different ways. Therefore, it is recommended to consider the effect of climate change, so as gate reliable information on the operation of the koka reservoir.
- ❖ Reservoir Sedimentation is one of the major problems in the planning of storage dams due to soil erosion and poor land management which causes a significant reduction of reservoir capacity. In this research, the effect of sedimentation was not studied. Thus, the researcher suggests, knowing the effect of sedimentation on reservoir operation.
- ❖ The HEC-ResSim optimal result is based on a successive trial and error procedure that is not fully guaranteed for the optimal value. Hence it is recommendable to recheck using optimization models.

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APPENDICES

Appendix-A: Meteorological Data

ADDIS ABABA													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	29.10	0.00	22.10	66.80	44.80	128.00	257.00	160.70	94.70	58.60	15.30	0.00	877.10
1998	66.60	40.00	43.80	99.80	197.70	111.60	270.70	236.80	173.40	139.40	0.00	0.00	1379.80
1999	4.40	0.00	35.00	17.80	30.50	104.60	294.00	270.50	62.80	127.10	0.00	0.00	946.70
2000	0.00	0.00	17.60	87.80	95.20	102.10	192.90	221.90	157.50	19.60	7.50	0.00	902.10
2001	0.00	10.30	174.30	14.80	116.70	166.00	289.40	207.30	113.30	10.60	0.00	0.00	1102.70
2002	30.60	25.90	79.40	36.60	49.60	115.50	213.90	233.60	72.60	0.50	0.00	32.80	891.00
2003	4.80	34.10	48.90	111.50	18.00	111.00	204.30	238.40	130.20	4.60	0.00	33.30	939.10
2004	26.10	11.70	32.40	7.00	114.50	236.90	230.10	122.10	50.00	0.60	0.00	0.34	831.74
2005	55.40	14.10	41.80	116.20	164.60	159.10	174.30	248.00	77.60	25.80	7.20	0.00	1084.10
2006	2.00	36.60	107.80	93.90	37.80	115.10	313.20	331.10	132.50	35.90	0.00	0.00	1205.90
2007	9.90	21.30	61.10	86.80	134.00	157.60	191.30	305.40	130.90	37.20	0.10	0.00	1135.60
2008	0.00	0.00	0.00	34.00	75.30	73.10	295.10	259.10	192.70	22.20	53.10	0.00	1004.60
2009	40.90	0.00	12.40	46.10	52.00	77.50	238.20	269.50	86.10	42.40	2.00	79.90	947.00
2010	0.40	115.20	75.60	159.50	94.70	107.20	320.20	138.80	105.00	0.00	13.80	15.70	1146.10
2011	3.40	13.60	27.90	85.20	148.00	182.00	296.50	141.30	0.00	11.90	0.00	2.26	912.06
2012	0.00	0.00	34.50	75.10	58.50	72.80	228.80	281.60	176.90	1.20	0.00	2.21	931.61
2013	0.00	0.00	63.50	114.40	78.50	101.40	157.60	270.20	126.70	45.30	3.20	0.00	960.80
2014	0.00	41.70	29.70	33.70	62.10	41.80	179.70	253.60	95.10	34.80	0.00	0.00	772.20
2015	0.00	0.00	21.30	169.10	135.60	153.50	203.00	95.10	0.00	1.30	1.80	0.02	780.72
2016	133.90	33.50	178.90	142.10	179.70	197.90	23.60	17.30	0.00	2.86	0.78	0.06	910.60
2017	0.00	13.60	56.00	26.60	103.50	44.10	217.00	241.70	260.50	5.30	0.00	0.00	968.30
2018	0.00	30.80	21.70	145.70	87.10	198.10	259.00	352.80	53.50	51.50	5.20	0.00	1205.40
2019	0.00	0.00	51.90	109.00	72.40	125.10	417.00	352.30	230.70	0.00	34.72	10.06	1403.18
2020	3.20	97.70	104.60	186.40	354.20	363.20	193.70	27.40	0.00	54.39	5.27	3.53	1393.59
MEAN	17.11	22.50	55.93	86.08	104.38	135.22	235.85	219.85	105.11	30.54	6.25	7.51	1026.33

ADDIS ALEM													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	22.65	22.53	79.48	53.03	108.85	201.90	245.80	172.10	95.10	68.80	90.20	12.00	1172.43
1998	61.10	86.80	107.40	67.30	101.10	208.10	289.50	251.00	154.00	53.90	21.90	0.00	1402.10
1999	29.50	3.30	28.60	13.50	74.60	197.50	255.00	262.20	51.80	116.60	0.00	0.00	1032.60
2000	0.00	0.00	12.50	103.10	135.40	168.10	266.60	238.20	186.80	33.60	17.40	29.20	1190.90
2001	0.00	0.00	169.40	28.20	124.30	233.90	309.70	265.00	43.30	21.40	0.00	0.00	1195.20
2002	51.60	84.10	77.80	26.40	35.00	174.50	333.10	264.60	38.80	0.00	0.00	62.40	1148.30
2003	0.00	65.80	109.60	84.10	0.00	173.10	226.00	232.10	165.30	11.00	6.60	4.00	1077.60
2004	17.50	27.60	70.50	140.60	62.90	143.10	190.40	212.50	172.50	33.60	1.30	2.00	1074.50
2005	14.20	15.20	148.20	45.10	96.90	90.40	262.35	224.00	99.00	21.90	12.70	0.00	1029.95
2006	0.00	83.50	101.00	61.10	83.20	148.90	334.30	265.40	135.50	22.50	0.00	9.80	1245.20
2007	12.10	59.30	120.10	94.90	123.40	170.10	237.30	204.30	75.00	23.10	0.00	0.00	1119.60
2008	1.90	12.50	14.60	36.90	125.60	145.10	291.50	268.10	85.60	46.90	32.60	19.80	1081.10
2009	43.20	2.80	48.60	11.30	51.50	89.30	297.60	318.30	62.10	27.85	6.60	39.60	998.75
2010	0.40	87.20	99.00	112.80	73.70	156.10	340.10	250.40	233.40	8.80	2.00	24.80	1388.70
2011	23.90	12.30	69.10	19.40	95.10	74.20	62.90	222.10	312.40	0.00	307.00	42.20	1240.60
2012	0.00	0.00	0.00	102.90	17.70	61.70	318.80	271.90	136.20	0.00	0.00	11.20	920.40
2013	3.40	0.00	46.30	78.00	58.20	115.40	317.40	246.10	151.30	29.50	19.00	1.50	1066.10
2014	14.10	53.90	31.70	49.70	121.20	87.50	238.10	126.10	119.90	48.80	1.10	0.00	892.10
2015	0.00	5.00	20.10	5.60	99.00	147.70	176.00	167.50	137.90	4.40	1.70	13.90	778.80
2016	61.00	26.00	26.60	140.80	173.90	132.30	129.50	260.60	103.40	32.90	0.00	0.00	1087.00
2017	23.00	28.73	42.18	68.90	113.80	369.10	254.20	278.70	359.80	17.40	20.70	0.00	1576.50
2018	31.00	41.50	22.00	44.80	55.70	236.90	262.70	260.70	127.20	19.80	40.70	0.00	1143.00
2019	0.00	42.40	100.00	84.40	97.20	221.30	229.50	56.20	5.60	51.40	60.70	0.00	948.70
2020	23.00	28.73	42.18	68.90	107.92	221.46	210.38	204.74	146.78	25.18	24.76	0.00	1104.02
MEAN	18.06	32.88	66.12	64.24	89.01	165.32	253.28	230.12	133.28	29.97	27.79	11.35	1121.42

ALEM TENA M													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	10.40	0.00	32.30	85.60	9.30	158.40	194.60	151.20	70.70	48.00	6.00	0.00	766.50
1998	7.10	21.80	36.30	102.60	33.10	49.80	216.40	190.70	52.00	93.40	0.00	0.00	803.20
1999	2.40	0.00	19.70	6.80	19.20	80.80	227.50	165.60	71.80	119.90	0.00	0.00	713.70
2000	0.00	0.00	0.00	127.10	89.00	51.70	207.80	144.00	155.80	52.70	54.20	0.00	882.30
2001	0.00	24.60	68.20	15.70	113.80	113.90	212.20	155.30	72.80	0.00	0.00	0.00	776.50
2002	1.50	35.40	24.30	82.90	15.50	45.00	181.10	150.70	33.10	0.50	0.00	19.00	589.00
2003	19.00	16.70	116.30	54.00	29.40	84.60	392.50	105.40	96.50	0.00	0.00	21.70	936.10
2004	89.40	0.00	58.40	174.30	0.00	78.80	122.90	145.00	110.60	40.50	0.00	2.50	822.40
2005	32.70	1.20	100.90	130.70	80.40	55.10	103.60	237.00	136.00	12.50	5.50	0.00	895.60
2006	1.20	64.90	147.90	46.40	48.90	55.30	166.50	124.50	69.40	27.70	0.00	15.40	768.10
2007	10.10	3.71	13.30	47.80	101.50	61.70	156.60	172.20	125.50	36.80	5.20	0.00	734.41
2008	0.00	0.00	0.00	36.00	90.10	162.80	244.80	226.80	121.00	31.50	44.60	0.00	957.60
2009	120.90	0.00	19.60	28.80	2.55	61.30	147.60	65.00	57.50	80.40	0.00	5.55	589.21
2010	0.00	55.40	134.90	58.70	95.80	81.60	185.50	144.70	64.00	0.00	16.20	1.50	838.30
2011	0.00	0.80	45.70	0.00	24.00	11.00	157.10	235.20	216.00	0.00	0.08	0.00	689.88
2012	0.00	0.00	0.00	50.70	32.20	6.91	6.49	192.00	247.70	0.00	0.00	0.50	536.50
2013	4.20	0.00	129.80	4.00	77.20	97.10	285.20	152.30	157.70	36.00	1.50	0.00	945.00
2014	0.00	19.00	46.30	4.00	158.10	15.50	258.90	228.30	112.00	90.30	1.50	0.00	933.90
2015	0.00	0.00	15.30	0.00	161.70	103.00	146.20	178.30	51.40	1.50	3.70	0.70	661.80
2016	9.80	0.00	9.40	162.60	179.80	137.20	360.60	104.80	88.50	30.70	15.40	0.00	1098.80
2017	0.00	37.40	67.30	17.80	178.00	39.90	240.00	86.20	163.30	0.00	0.00	0.00	829.90
2018	8.09	2.55	24.21	179.60	30.10	90.00	161.80	173.20	22.20	0.00	16.70	1.03	709.48
2019	0.00	0.00	46.46	87.04	33.64	183.81	135.38	224.68	168.40	33.27	45.14	0.00	957.82
2020	3.80	7.76	50.13	177.13	97.12	156.80	384.00	232.20	73.80	5.20	7.40	7.90	1203.24
MEAN	13.36	12.13	50.28	70.01	70.85	82.58	203.97	166.05	105.74	30.87	9.30	3.16	818.30

ASGORI													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	69.20	70.40	31.20	80.80	79.70	166.50	315.40	312.60	51.60	20.40	0.00	0.00	1197.80
1998	3.50	0.00	69.20	8.90	66.00	137.20	254.80	199.10	130.60	59.10	0.00	0.00	928.40
1999	0.00	0.00	0.00	63.20	111.60	238.10	167.90	143.60	1.40	8.30	0.00	0.00	734.10
2000	1.40	12.50	103.40	22.80	127.90	183.90	253.80	128.30	55.80	5.80	0.00	0.00	895.60
2001	27.70	41.40	48.10	41.00	52.20	97.30	258.40	252.70	93.40	0.00	0.00	20.70	932.90
2002	27.70	41.40	48.10	41.00	52.20	97.30	258.40	252.70	93.40	0.00	0.00	20.70	932.90
2003	45.10	24.50	65.20	119.80	43.90	169.60	264.80	272.00	107.90	2.00	1.30	10.50	1126.60
2004	30.80	4.50	44.60	172.00	24.60	151.80	183.60	173.80	74.30	57.10	0.00	0.00	917.10
2005	28.90	2.40	77.20	57.90	140.80	149.00	164.20	146.80	123.20	10.50	0.00	0.00	900.90
2006	0.20	11.50	18.49	11.93	19.65	46.06	60.18	32.02	79.10	2.70	0.00	1.20	283.03
2007	8.70	2.44	83.00	143.50	64.60	198.00	212.20	179.70	141.70	11.00	0.00	0.00	1044.84
2008	0.00	0.00	0.90	60.90	103.20	140.30	283.20	190.20	74.00	22.60	65.00	0.00	940.30
2009	31.10	1.50	1.00	26.20	64.70	95.40	205.90	266.20	67.80	20.58	59.10	27.40	866.88
2010	0.03	89.20	83.70	102.30	52.20	164.60	256.30	144.50	98.60	10.50	43.00	8.10	1053.03
2011	14.80	5.20	37.50	15.10	61.80	171.00	155.60	256.10	189.90	0.00	3.10	0.00	910.10
2012	0.00	0.00	1.20	60.70	41.50	134.70	211.30	230.30	83.60	2.70	0.00	7.60	773.60
2013	2.70	0.00	391.60	103.50	77.40	111.10	228.70	165.20	129.00	74.70	8.00	0.00	1291.90
2014	0.00	54.20	38.00	57.40	118.40	100.70	213.50	205.60	106.70	8.70	2.90	0.00	906.10
2015	0.00	0.00	11.70	0.80	77.30	152.20	196.30	226.70	110.90	0.40	0.00	10.90	787.20
2016	33.70	7.70	14.39	134.10	64.60	71.00	276.00	180.70	70.60	0.70	3.90	1.30	858.69
2017	0.00	117.90	23.70	21.90	132.70	97.30	147.90	202.30	163.80	2.10	1.20	0.00	910.80
2018	1.52	11.80	28.70	82.80	137.50	180.30	216.90	217.60	23.40	31.59	5.44	0.00	937.55
2019	0.00	7.58	57.57	130.28	35.24	184.55	231.57	320.56	386.26	26.91	30.86	7.07	1418.45
2020	6.26	37.13	29.86	60.25	84.50	132.44	209.48	207.98	105.94	7.09	7.44	3.49	891.83
MEAN	13.89	22.64	54.51	67.46	76.42	140.43	217.76	204.47	106.79	16.06	9.63	4.96	935.03

BANTU LIBEN													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	0.00	0.00	0.00	109.20	17.40	187.90	278.40	232.60	87.30	53.70	31.80	0.00	998.30
1998	60.80	9.00	68.60	227.30	127.00	150.90	430.60	433.40	87.80	0.00	0.00	0.00	1595.40
1999	0.00	0.00	49.90	0.00	20.90	146.30	225.90	285.45	74.80	39.10	0.00	0.00	842.35
2000	21.85	9.75	69.55	89.85	57.02	181.83	314.17	281.11	79.32	15.47	5.30	6.75	1131.96
2001	0.00	6.20	148.20	47.70	76.10	159.80	307.10	259.80	2.80	0.00	0.00	0.00	1007.70
2002	33.30	26.50	76.50	60.10	62.10	207.80	270.10	216.00	12.60	0.00	0.00	19.20	984.20
2003	37.00	16.80	74.10	94.80	38.60	238.30	372.90	259.40	210.60	0.00	0.00	21.30	1363.80
2004	74.60	0.00	39.50	160.10	43.20	333.20	302.10	280.10	77.40	18.90	0.00	0.00	1329.10
2005	29.20	7.20	100.50	78.60	323.40	268.50	434.40	434.40	166.90	0.00	0.00	0.00	1843.10
2006	0.00	0.00	214.30	133.10	116.10	273.10	325.30	360.40	141.80	5.10	0.00	0.00	1569.20
2007	8.90	0.00	69.70	38.90	139.70	180.20	332.90	292.70	97.00	0.00	0.00	0.00	1160.00
2008	11.93	6.63	0.00	30.50	69.85	144.70	366.00	260.60	92.15	5.33	74.60	22.20	1084.48
2009	13.31	8.28	17.43	20.40	59.60	130.85	227.93	243.05	93.21	5.38	0.00	44.40	863.83
2010	0.00	0.00	0.00	0.00	0.00	0.00	181.20	196.80	0.00	0.00	0.00	28.35	406.35
2011	6.40	0.00	0.00	16.90	45.20	189.20	0.00	212.10	129.80	16.20	0.00	12.30	628.10
2012	32.40	26.50	63.40	104.90	62.10	265.20	244.00	44.40	229.60	0.00	130.20	5.10	1207.80
2013	0.00	0.00	80.50	60.50	75.50	190.30	299.80	295.60	211.70	18.80	0.00	0.00	1232.70
2014	0.00	17.00	54.60	6.10	27.00	226.30	323.70	390.80	227.10	9.88	37.85	1.28	1321.60
2015	0.00	0.00	0.00	31.90	56.90	115.70	297.50	390.00	120.50	0.00	0.00	0.00	1012.50
2016	21.00	0.00	0.00	201.50	158.40	292.50	533.80	394.30	143.10	20.70	21.20	0.00	1786.50
2017	0.00	24.10	37.70	39.50	158.10	183.30	232.60	174.70	166.50	12.34	14.76	0.32	1043.93
2018	37.00	24.10	7.00	84.80	53.70	102.40	243.50	284.50	154.80	16.52	36.00	0.40	1044.72
2019	11.60	13.04	19.86	72.76	90.82	184.04	326.22	326.86	162.40	11.89	21.96	0.40	1241.85
2020	13.92	12.25	12.91	86.09	103.58	175.59	326.72	314.07	149.46	12.29	18.79	0.22	1225.90
MEAN	17.22	8.64	50.18	74.81	82.59	188.66	299.87	285.96	121.61	10.90	16.35	6.76	1163.56

CHAFEDONSA													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	5.97	0.00	5.70	61.60	12.10	84.00	144.60	152.60	66.50	66.60	18.30	0.00	617.97
1998	18.70	46.40	48.70	39.90	27.30	87.00	217.50	262.60	96.50	36.60	0.00	0.00	881.20
1999	0.00	0.00	22.80	0.00	14.40	37.45	232.90	264.00	48.90	26.90	0.00	0.20	647.55
2000	0.00	0.00	23.80	40.00	18.90	96.20	182.90	226.80	29.70	8.45	19.10	0.00	645.85
2001	0.00	23.80	109.70	17.40	69.10	89.00	202.60	111.20	46.60	0.00	0.00	6.30	675.70
2002	0.00	0.00	41.70	20.00	17.00	59.60	156.00	265.60	53.40	0.00	0.00	12.00	625.30
2003	12.10	43.40	54.30	78.00	0.00	73.70	228.30	353.00	113.80	0.00	0.00	15.50	972.10
2004	14.20	1.30	36.00	72.20	4.30	122.30	223.10	184.90	108.80	0.00	5.60	0.80	773.50
2005	62.80	24.00	47.60	142.70	113.20	181.00	186.90	186.90	83.60	0.00	0.00	0.00	1028.70
2006	2.50	10.40	28.50	148.40	27.50	113.10	346.50	166.80	148.30	22.50	0.00	18.70	1033.20
2007	29.20	15.20	55.60	78.10	96.80	179.80	217.20	326.00	104.50	0.00	2.10	0.00	1104.50
2008	0.00	0.00	0.00	41.70	86.70	72.30	314.80	265.00	83.60	10.80	29.60	0.00	904.50
2009	45.70	0.00	23.00	21.80	63.70	63.40	278.00	333.30	103.20	0.00	13.50	24.80	970.40
2010	0.00	46.40	26.30	100.00	89.10	144.90	247.50	207.10	161.80	0.00	0.00	7.00	1030.10
2011	0.00	0.00	46.80	48.70	29.90	65.90	220.80	275.00	108.30	0.00	0.00	0.00	795.40
2012	0.00	0.00	10.40	87.10	61.70	134.50	239.20	312.70	103.10	0.00	0.00	0.40	949.10
2013	1.10	0.00	78.50	53.10	351.10	89.40	257.60	262.20	180.00	43.90	3.30	0.00	1320.20
2014	0.00	25.60	19.40	5.20	36.00	79.00	203.90	264.90	83.00	38.80	1.50	0.00	757.30
2015	0.00	0.00	15.30	0.00	59.70	90.30	137.50	222.40	69.10	1.10	6.10	0.00	601.50
2016	19.00	1.00	28.50	151.20	79.00	108.50	229.90	52.58	80.60	7.71	52.20	0.00	810.19
2017	0.00	0.00	28.60	11.10	164.90	53.50	249.80	254.30	187.50	0.10	0.00	0.00	949.80
2018	31.00	10.00	9.05	15.57	26.60	111.80	277.70	277.20	71.00	7.80	12.00	0.00	849.72
2019	0.00	15.52	2.04	17.38	37.81	80.40	358.20	233.00	202.90	23.90	39.40	1.43	1012.00
2020	1.30	0.09	8.41	21.57	171.80	97.30	272.50	373.00	135.00	22.20	9.29	6.88	1119.34
MEAN	10.15	10.96	32.11	53.03	69.11	96.43	234.41	243.04	102.90	13.22	8.83	3.92	878.13

HOMBOLE													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	9.50	37.80	31.80	27.40	23.50	82.10	184.10	127.60	47.90	22.00	3.00	0.00	596.70
1998	17.60	26.30	52.00	43.80	38.00	66.40	224.60	284.30	57.00	62.10	0.00	0.00	872.10
1999	3.40	0.00	8.60	4.20	0.80	93.90	200.90	188.80	42.20	116.50	0.00	0.00	659.30
2000	7.00	14.20	58.80	37.30	54.20	106.30	194.30	158.40	115.80	16.90	0.00	0.00	763.20
2001	0.00	16.40	116.50	63.80	123.80	158.70	187.70	102.80	26.60	0.00	0.00	0.00	796.30
2002	7.40	15.10	77.70	32.30	38.20	48.40	220.50	47.60	16.70	0.00	0.00	15.60	519.50
2003	15.20	14.80	58.30	150.90	13.70	107.50	326.10	160.90	28.90	0.00	0.00	18.30	894.60
2004	15.70	0.00	28.90	144.90	0.00	101.20	119.90	147.60	60.20	12.00	0.00	0.00	630.40
2005	32.60	3.80	24.00	65.20	81.00	188.10	196.40	45.30	48.90	6.90	5.00	4.60	701.80
2006	36.70	45.40	112.30	58.90	87.40	201.20	120.00	89.50	0.00	0.00	19.90	0.00	771.30
2007	40.20	3.80	43.40	26.70	113.50	180.40	135.00	159.20	106.60	15.50	0.00	0.00	824.30
2008	0.00	0.00	0.00	0.00	56.80	69.40	232.40	202.60	91.20	8.70	133.20	0.00	794.30
2009	98.90	0.00	8.20	18.30	34.80	103.80	181.10	173.60	110.00	93.30	0.00	8.60	830.60
2010	0.00	73.40	96.40	132.40	74.70	95.70	165.40	156.20	137.90	26.80	37.00	2.10	998.00
2011	3.60	0.00	46.70	45.00	41.20	68.80	189.90	174.40	103.70	0.00	14.10	0.00	687.40
2012	0.00	0.00	70.10	61.10	29.70	55.80	333.40	150.20	142.80	4.90	0.30	0.00	848.30
2013	0.00	0.00	168.30	17.30	55.90	115.70	201.60	154.40	127.20	27.50	0.00	0.00	867.90
2014	0.00	1.20	30.40	22.10	53.10	6.00	185.60	136.20	122.90	8.30	0.00	0.00	565.80
2015	0.00	0.00	39.30	23.50	139.60	170.10	149.80	219.20	107.10	9.80	1.00	0.00	859.40
2016	8.90	0.00	5.40	151.90	114.00	75.80	330.90	70.70	66.30	0.00	1.30	0.00	825.20
2017	0.00	32.10	146.80	20.40	124.70	40.10	213.60	168.90	155.10	0.00	0.00	0.00	901.70
2018	0.00	14.60	55.00	190.30	93.60	139.00	100.40	149.40	23.50	3.50	4.10	0.00	773.40
2019	2.40	11.70	61.90	96.90	118.40	37.30	199.20	152.40	219.90	3.40	1.60	0.00	905.10
2020	2.80	14.60	95.90	124.00	108.30	91.20	388.80	218.80	114.40	3.40	1.60	0.00	1163.80
MEAN	12.58	13.55	59.86	64.94	67.45	100.12	207.57	151.63	86.37	18.40	9.25	2.05	793.77

MOJO													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	30.40	0.00	37.60	16.80	16.10	130.60	216.10	158.60	77.80	55.30	21.00	0.00	760.30
1998	32.30	7.60	50.70	46.00	46.00	112.10	161.30	274.80	149.00	154.10	0.00	0.00	1033.90
1999	3.23	0.00	18.40	0.00	4.20	90.90	541.50	362.00	67.40	87.40	0.00	0.00	1175.03
2000	0.00	0.00	12.00	10.10	33.20	128.00	285.90	230.80	114.10	12.60	25.40	0.00	852.10
2001	0.00	28.50	79.40	22.50	111.00	131.50	184.70	175.60	54.90	0.00	0.00	0.00	788.10
2002	0.00	0.00	8.90	13.70	6.50	62.40	193.50	169.20	95.20	0.00	0.00	14.90	564.30
2003	40.60	55.90	74.90	84.30	23.20	129.00	393.20	148.60	96.10	0.00	0.00	14.30	1060.10
2004	15.50	0.00	99.30	114.60	9.80	21.79	393.80	201.20	116.90	99.10	32.40	0.00	1104.40
2005	12.47	19.10	160.50	149.60	195.40	200.50	320.60	308.70	143.50	14.60	10.90	0.00	1535.87
2006	1.60	38.50	90.50	61.60	19.20	109.00	344.40	362.70	131.80	24.90	2.60	12.30	1199.10
2007	54.10	23.70	83.80	49.60	111.00	129.70	222.80	227.90	91.40	7.60	0.00	0.00	1001.60
2008	1.50	0.00	0.00	61.90	76.70	137.00	431.00	404.40	188.70	89.40	50.80	0.00	1441.40
2009	39.20	6.00	26.30	62.90	12.70	95.30	213.80	330.30	66.20	214.50	6.50	34.60	1108.30
2010	0.00	85.20	7.70	151.90	119.50	73.40	254.20	170.40	246.70	0.00	1.40	0.00	1110.40
2011	0.00	0.00	47.40	54.90	122.00	54.60	193.00	175.60	170.40	0.00	0.00	0.00	817.90
2012	0.00	0.00	37.80	43.00	10.98	13.34	524.40	419.70	147.00	0.00	0.00	0.00	1196.22
2013	0.00	0.00	85.80	32.90	121.30	98.50	420.00	232.30	69.20	36.40	0.00	0.00	1096.40
2014	0.00	16.80	51.90	2.90	7.60	27.40	260.70	257.20	154.30	96.30	0.00	0.00	875.10
2015	0.00	0.00	0.00	0.00	186.30	117.60	156.90	275.20	93.90	8.50	21.00	0.00	859.40
2016	12.10	32.70	18.20	258.70	83.60	117.70	531.60	112.90	76.80	0.00	31.50	0.00	1275.80
2017	0.00	32.90	57.40	17.80	155.20	44.10	319.10	211.60	198.30	0.00	0.00	0.00	1036.40
2018	31.00	6.50	38.28	70.26	100.81	133.40	351.40	297.80	43.20	0.00	17.50	0.00	1090.15
2019	0.00	11.11	42.10	60.06	98.47	134.30	463.80	261.00	238.60	2.13	17.50	17.10	1346.17
2020	10.78	20.80	38.99	44.40	50.60	113.30	172.40	471.50	106.40	0.53	16.63	4.28	1050.60
MEAN	11.87	16.05	48.66	59.60	71.72	100.23	314.59	260.00	122.41	37.64	10.63	4.06	1057.46

TULUBOLO													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	0.00	0.00	36.30	60.70	42.10	203.00	343.20	68.72	78.80	63.30	37.10	0.00	933.22
1998	0.00	18.30	0.00	90.40	155.30	317.00	343.60	340.90	113.80	68.90	0.00	15.70	1463.90
1999	0.02	0.00	56.00	0.00	79.10	270.10	356.70	449.00	140.30	5.50	0.00	0.00	1356.72
2000	0.51	0.00	1.80	160.80	131.90	222.00	320.70	226.70	114.30	6.30	26.40	4.60	1216.01
2001	0.00	3.20	98.10	36.40	68.60	186.50	224.00	160.10	35.60	6.90	0.00	0.00	819.40
2002	46.30	9.10	41.70	60.60	43.30	224.90	236.80	241.00	77.20	0.00	0.00	40.60	1021.50
2003	40.00	11.70	49.10	94.30	23.80	127.10	315.10	199.10	91.50	0.00	0.00	7.70	959.40
2004	52.50	0.00	10.80	92.90	44.70	270.90	330.60	185.00	163.80	0.00	2.20	5.20	1158.60
2005	26.00	0.00	53.90	180.90	165.00	208.40	177.30	203.20	83.80	18.30	5.50	0.00	1122.30
2006	0.00	13.30	106.40	63.20	91.40	197.90	260.80	138.20	162.00	5.50	0.00	0.30	1039.00
2007	6.80	12.90	40.60	49.40	121.50	257.60	206.20	40.05	147.20	13.10	0.00	0.00	895.35
2008	0.00	0.00	7.40	48.40	124.40	234.10	281.00	306.00	90.80	47.00	0.00	0.00	1139.10
2009	41.40	0.00	16.20	9.40	67.10	59.20	264.10	331.00	75.60	89.10	0.00	25.60	978.70
2010	0.00	103.90	81.60	82.20	186.30	281.90	349.20	157.40	108.10	0.00	10.20	34.00	1394.80
2011	3.40	17.80	53.10	14.50	75.50	188.80	111.10	237.10	132.40	0.00	0.00	0.00	833.70
2012	0.00	0.00	23.00	90.60	77.70	107.20	218.20	253.00	122.40	7.50	0.00	0.00	899.60
2013	0.00	0.00	34.10	71.70	128.90	252.00	411.90	454.00	217.10	111.90	0.00	2.30	1683.90
2014	0.00	32.10	53.60	22.00	77.50	95.20	310.90	333.90	79.80	14.20	0.00	0.00	1019.20
2015	0.00	1.60	45.80	0.00	128.10	113.50	207.20	209.80	93.20	0.20	6.40	21.30	827.10
2016	21.80	10.70	62.40	100.00	158.30	98.80	241.00	149.30	99.20	14.90	15.80	0.00	972.20
2017	0.00	61.50	51.30	43.40	166.20	188.20	145.60	247.90	137.40	3.80	0.50	0.00	1045.80
2018	1.66	22.50	9.79	10.65	13.80	293.30	232.80	295.70	37.90	137.40	23.50	0.00	1079.00
2019	0.00	0.40	37.86	48.34	105.47	189.70	257.60	277.66	112.43	41.41	6.60	5.10	1082.56
2020	3.91	0.20	118.00	66.40	110.00	197.40	258.50	274.60	96.70	46.26	7.54	4.10	1183.61
MEAN	10.18	13.30	45.37	62.38	99.42	199.36	266.84	240.81	108.81	29.23	5.91	6.94	1088.53

WOLONKOMI													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	38.10	0.00	18.00	78.70	36.10	138.10	276.50	208.20	121.20	45.00	47.00	0.00	1006.90
1998	50.10	39.80	36.80	51.40	87.20	221.70	241.20	220.30	138.10	70.00	2.40	0.00	1159.00
1999	4.80	0.00	8.20	26.80	62.10	127.80	197.10	175.60	83.60	107.30	0.00	0.00	793.30
2000	0.29	2.50	20.47	4.90	25.52	36.74	185.70	202.10	145.90	55.00	26.50	9.40	715.00
2001	0.70	8.90	115.10	26.80	110.30	161.50	267.60	211.20	57.70	15.20	0.00	0.00	975.00
2002	38.00	16.80	43.80	32.20	34.50	195.50	224.20	175.90	45.20	2.00	0.00	97.70	905.80
2003	19.80	0.00	103.50	130.30	14.90	177.60	315.80	185.10	61.80	0.00	2.30	2.40	1013.50
2004	49.40	2.40	22.80	173.10	22.40	30.24	249.30	198.70	164.40	10.00	0.00	0.00	922.74
2005	62.80	0.00	81.50	126.80	110.60	151.60	264.30	118.20	77.30	3.40	2.50	0.00	999.00
2006	0.00	25.40	87.20	70.00	67.60	34.32	92.13	67.00	40.92	17.20	0.03	0.25	502.05
2007	30.20	28.70	29.20	28.35	13.12	167.40	219.10	190.30	85.30	20.70	0.01	0.00	812.38
2008	0.00	0.00	0.00	21.60	113.50	28.08	133.40	38.20	14.99	4.54	12.93	0.01	367.25
2009	20.10	0.39	29.20	25.30	16.15	23.71	51.18	66.22	64.10	39.10	10.40	14.90	360.75
2010	19.60	29.70	84.60	72.00	84.70	143.00	254.10	195.00	102.00	0.00	31.70	22.80	1039.20
2011	2.95	1.08	7.55	3.10	15.36	181.50	194.00	202.00	87.00	0.00	27.70	0.00	722.24
2012	0.00	0.00	0.00	63.50	24.80	26.98	42.50	46.14	16.91	0.60	0.07	1.52	223.02
2013	4.70	0.00	26.00	69.20	75.00	170.10	195.60	123.30	141.40	28.20	25.90	0.00	859.40
2014	0.00	47.10	28.70	58.90	88.50	67.70	155.60	239.30	110.70	46.90	12.20	0.00	855.60
2015	0.00	0.00	2.96	0.21	19.28	113.90	168.80	118.70	108.10	0.20	4.00	27.80	563.94
2016	29.50	5.40	20.90	164.50	91.40	116.60	179.30	185.90	94.80	13.60	3.50	0.00	905.40
2017	0.00	64.80	64.80	38.70	153.60	65.40	177.00	160.70	168.60	12.00	7.00	0.00	912.60
2018	4.40	60.80	26.40	28.50	37.00	191.90	127.70	198.60	99.10	16.00	22.50	0.00	812.90
2019	0.00	0.00	22.16	53.33	63.12	238.50	185.00	150.50	179.60	18.70	9.84	5.61	926.36
2020	4.30	0.00	27.30	70.30	193.20	207.20	233.20	142.90	119.48	12.13	9.37	6.64	1026.02
MEAN	15.82	13.91	37.80	59.10	65.00	125.71	192.93	159.17	97.01	22.41	10.74	7.88	807.47

Appendix-B: Hydrological Data

	AVERAGE FLOW OF HOMBOLE GAUGE STATION												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	4.04	3.22	2.97	5.50	4.11	15.69	57.71	119.28	32.38	10.03	10.68	6.10	271.70
1998	5.01	3.69	11.16	8.21	11.31	24.62	137.37	366.56	161.03	41.71	9.09	5.46	785.23
1999	4.98	4.10	4.58	3.44	3.56	20.80	110.42	269.34	62.90	47.58	8.64	5.12	545.44
2000	4.15	3.75	1.94	3.15	5.67	12.57	63.10	178.61	90.99	30.78	12.48	6.04	413.21
2001	5.73	5.24	11.07	7.52	11.73	43.33	140.46	189.32	78.70	11.72	8.21	6.12	519.15
2002	5.81	4.21	4.88	5.69	5.16	12.01	59.54	143.31	49.31	9.59	7.58	3.82	310.90
2003	3.61	3.20	4.83	10.32	6.66	22.77	119.50	184.30	103.20	15.67	8.86	6.00	488.91
2004	3.68	2.85	3.89	14.74	5.29	20.49	88.59	166.73	87.11	16.74	9.33	4.49	423.93
2005	5.13	3.79	10.43	7.71	30.29	26.01	124.39	192.58	94.34	18.21	11.14	5.63	529.67
2006	4.31	4.00	9.73	17.12	15.50	26.20	155.89	269.65	143.56	18.66	11.81	6.01	682.45
2007	5.00	5.47	5.15	8.22	11.83	39.81	127.92	253.47	158.64	25.10	11.11	7.49	659.19
2008	4.66	4.16	3.31	3.72	5.64	16.80	118.76	216.79	159.12	18.41	22.48	6.08	579.91
2009	9.51	4.25	3.43	9.52	5.31	7.42	45.25	185.45	111.66	25.88	6.16	6.77	420.62
2010	4.54	12.56	10.97	21.57	19.45	35.68	162.37	178.77	153.67	18.51	9.99	5.36	633.45
2011	4.83	3.96	5.43	3.78	11.23	27.06	68.99	202.57	126.23	21.68	9.78	5.78	491.30
2012	4.27	3.74	3.15	10.67	8.25	12.19	93.95	262.43	239.54	19.20	7.73	5.21	670.34
2013	4.21	3.39	6.42	13.63	11.81	31.92	93.58	169.78	137.59	40.52	12.16	4.98	530.00
2014	3.86	4.45	4.74	6.52	10.31	14.19	55.98	147.20	84.69	32.04	10.02	4.68	378.67
2015	3.94	3.25	4.83	3.71	9.65	23.81	45.77	118.51	63.50	16.53	5.74	3.97	303.20
2016	4.07	3.58	4.79	8.63	10.00	20.53	72.32	174.48	131.33	27.07	8.91	4.71	470.43
MEAN	4.77	4.34	5.89	8.67	10.14	22.69	97.09	199.46	113.47	23.28	10.10	5.49	505.38

	AVERAGE FLOW OF MELKA KUNTURE GAUGE STATION												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	1.81	1.20	0.82	2.27	1.60	6.56	45.86	87.55	20.85	5.06	4.04	1.71	179.33
1998	2.37	1.37	4.35	2.05	4.70	16.16	78.84	191.86	107.89	24.84	3.69	2.18	440.30
1999	1.62	0.94	1.38	0.66	1.61	8.78	76.42	108.00	47.45	33.51	3.62	1.77	285.74
2000	1.66	1.09	0.86	1.82	2.97	6.35	42.07	115.78	66.48	21.46	3.78	2.50	266.82
2001	1.77	1.54	2.84	2.64	4.92	22.86	74.99	133.47	63.15	6.14	3.03	2.30	319.65
2002	3.43	1.99	2.44	4.81	2.08	11.95	57.95	128.12	42.74	3.76	1.82	2.05	263.13
2003	1.89	1.06	2.82	4.41	3.48	14.17	122.38	147.39	105.21	8.87	2.85	2.26	416.79
2004	2.14	1.44	1.38	8.79	2.95	20.02	78.94	138.47	65.51	11.24	3.23	2.10	336.21
2005	1.92	0.98	3.69	3.64	15.95	15.88	101.85	163.29	81.46	11.18	3.39	2.70	405.92
2006	1.71	1.29	5.53	9.60	18.28	19.56	135.76	263.77	120.95	7.16	3.13	2.10	588.84
2007	1.46	1.97	1.54	2.69	4.09	39.50	108.27	205.38	156.10	15.32	2.91	1.99	541.23
2008	1.38	0.98	0.67	1.11	1.97	9.18	124.80	218.01	112.06	5.96	8.14	2.67	486.95
2009	2.15	1.66	0.97	1.77	1.96	3.37	45.35	190.94	98.52	10.40	2.53	2.58	362.19
2010	1.54	3.72	5.52	10.44	8.26	27.51	132.87	158.89	121.88	9.55	3.17	2.41	485.77
2011	2.09	1.11	2.50	1.18	3.44	20.79	65.01	181.69	99.29	8.94	3.26	1.86	391.15
2012	1.12	0.71	0.72	3.04	4.40	9.96	76.39	178.14	122.48	9.44	2.41	1.75	410.55
2013	2.04	0.76	1.43	3.86	6.57	25.04	107.78	181.98	108.97	36.66	3.89	2.31	481.29
2014	1.56	1.43	2.17	2.39	6.33	7.71	41.47	101.25	59.65	13.67	2.76	1.83	242.21
2015	1.24	0.73	1.02	0.82	3.04	7.92	42.11	209.01	51.90	4.00	1.50	0.97	324.24
2016	0.96	0.69	0.84	5.46	22.96	17.20	66.94	167.60	85.75	15.94	2.64	1.72	388.68
MEAN	1.79	1.33	2.18	3.67	6.08	15.52	81.30	163.53	86.91	13.16	3.29	2.09	380.85

	AVERAGE FLOW OF MOJO GAUGE STATION												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	12.54	15.92	19.61	18.50	16.07	533.74	507.97	854.60	193.56	37.86	15.74	14.83	186.74
1998	12.54	15.92	19.61	18.50	16.07	533.74	531.92	874.99	254.04	37.86	15.74	14.83	195.48
1999	15.02	11.61	16.71	11.88	13.41	441.10	473.74	983.92	278.69	137.87	10.71	9.84	200.38
2000	11.07	9.09	8.65	8.36	13.73	265.57	379.47	790.54	268.36	19.58	12.25	11.19	149.82
2001	11.23	9.24	44.73	12.43	41.63	216.00	562.47	1057.40	245.62	15.49	12.87	13.30	186.87
2002	13.69	10.65	14.93	12.15	13.90	53.73	379.49	440.66	67.32	13.33	11.09	14.29	87.10
2003	17.28	12.78	22.32	34.22	18.54	81.50	834.98	1552.32	272.74	16.62	11.80	13.61	240.73
2004	12.50	6.61	19.29	54.65	9.07	38.46	219.55	837.24	93.63	17.37	11.76	12.29	111.03
2005	14.25	5.27	46.19	79.84	166.00	79.12	424.48	649.78	234.70	17.36	16.15	14.12	145.60
2006	12.96	16.71	24.22	72.25	47.21	70.97	626.83	590.19	247.35	19.07	14.77	14.37	146.41
2007	11.12	12.96	13.38	31.50	95.41	219.68	416.22	1106.22	558.10	33.65	18.43	19.98	211.39
2008	18.99	6.40	2.93	14.49	21.29	61.51	365.74	888.20	214.67	22.68	29.52	15.93	138.53
2009	16.06	5.95	5.23	25.84	13.77	12.93	200.40	555.08	241.25	44.36	11.46	14.25	95.55
2010	10.09	8.52	19.85	66.34	95.97	92.64	450.44	451.93	391.06	19.91	13.03	10.94	135.89
2011	6.94	0.22	30.83	40.82	24.68	99.83	435.79	447.05	366.87	102.02	27.37	15.82	133.19
2012	14.06	7.26	6.32	20.55	15.58	54.36	259.00	826.46	164.49	17.13	13.69	11.97	117.57
2013	9.59	5.53	19.07	17.86	20.04	116.34	816.81	293.41	195.00	33.77	31.35	24.35	131.93
2014	19.91	25.65	16.62	12.35	46.85	40.11	941.27	597.09	289.70	69.53	31.11	32.29	176.87
2015	32.80	14.71	10.41	8.42	106.75	132.17	162.90	432.74	72.42	22.04	20.48	18.57	86.20
2016	26.36	20.18	13.51	10.38	76.80	86.14	552.08	514.92	181.06	45.79	25.79	25.43	131.54
MEAN	14.95	11.06	18.72	28.57	43.64	161.48	477.08	737.24	241.53	37.16	17.75	16.11	150.44

	AVERAGE INFLOW AT KOKA INLET												
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	4.71	4.01	3.80	6.48	4.90	34.83	78.14	155.09	41.05	11.92	11.91	6.98	363.84
1998	5.74	4.51	12.53	9.37	12.57	44.33	163.77	419.12	180.12	45.67	10.22	6.30	914.26
1999	5.80	4.78	5.43	4.07	4.23	37.12	133.16	319.15	76.44	55.20	9.57	5.77	660.71
2000	4.78	4.31	2.35	3.64	6.49	22.40	79.66	216.17	106.01	33.43	13.70	6.80	499.74
2001	6.48	5.91	13.26	8.43	13.86	53.48	168.06	236.35	92.15	12.99	9.18	6.95	627.10
2002	6.63	4.88	5.68	6.48	5.95	14.62	75.87	167.09	54.80	10.65	8.45	4.53	365.63
2003	4.41	3.87	5.88	12.15	7.71	27.01	154.68	247.26	119.16	17.23	9.83	6.83	616.03
2004	4.33	3.27	4.78	17.55	5.93	23.12	101.56	205.06	95.95	18.39	10.34	5.18	495.47
2005	5.93	4.23	12.63	10.92	37.71	30.38	146.42	226.44	108.44	19.97	12.42	6.46	621.94
2006	5.02	4.87	11.16	20.68	18.06	30.31	186.60	306.56	161.28	20.49	13.08	6.88	785.00
2007	5.69	6.30	5.93	9.82	15.73	49.85	149.90	306.27	187.89	27.83	12.45	8.64	786.29
2008	5.58	4.65	3.62	4.46	6.70	19.98	138.49	260.05	176.75	20.35	24.94	6.99	672.56
2009	10.65	4.74	3.83	11.02	6.10	8.34	54.78	215.74	127.10	29.02	6.95	7.68	485.95
2010	5.16	13.68	12.34	25.22	23.87	41.14	187.72	205.24	176.93	20.37	11.08	6.07	728.82
2011	5.37	4.22	6.79	5.42	12.77	32.21	87.79	230.42	146.88	26.43	11.34	6.67	576.31
2012	5.01	4.24	3.57	12.06	9.29	14.83	108.56	306.63	260.70	21.01	8.70	5.94	760.54
2013	4.80	3.81	7.47	15.12	13.24	37.94	126.48	190.45	153.15	44.26	14.01	6.10	616.85
2014	4.76	5.67	5.60	7.36	12.52	16.48	90.53	176.37	100.02	36.40	11.73	6.04	473.49
2015	5.27	4.00	5.48	4.24	13.78	29.84	54.10	140.42	70.09	18.33	6.80	4.84	357.18
2016	5.20	4.53	5.54	9.55	13.18	24.79	95.15	202.73	146.01	30.33	10.37	5.85	553.22
MEAN	5.57	5.02	6.88	10.20	12.23	29.65	119.07	236.63	129.05	26.01	11.35	6.38	598.04

AVERAGE OUTFLOW AT D/S OF KOKA DAM													
YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SUM
1997	36.35	45.98	53.07	62.48	69.31	63.55	48.13	53.37	40.09	38.99	36.44	30.06	577.83
1998	31.93	32.36	30.17	32.95	32.36	36.09	36.24	198.02	129.81	65.56	45.13	47.39	718.00
1999	48.21	51.50	55.79	55.66	57.74	70.59	74.91	188.34	75.96	54.74	52.85	54.52	840.83
2000	48.21	47.96	49.78	42.73	77.20	67.22	38.93	75.44	70.13	68.61	46.66	47.98	680.87
2001	49.67	48.70	46.46	45.23	49.16	45.23	46.15	51.51	66.53	56.77	55.87	57.25	618.52
2002	54.68	54.48	54.54	52.98	54.65	54.34	52.21	54.28	55.52	55.97	44.44	34.07	622.16
2003	35.30	35.33	37.27	33.50	29.39	27.97	38.21	46.05	88.57	45.81	44.13	45.31	506.83
2004	43.87	37.29	36.90	43.55	44.86	44.94	48.64	49.97	35.70	34.93	29.92	34.00	484.58
2005	37.85	40.99	39.79	40.52	31.69	33.35	34.09	41.15	50.52	32.17	27.04	29.42	438.59
2006	36.03	39.52	40.31	41.84	41.05	34.56	30.34	104.12	188.03	45.66	29.77	32.76	664.01
2007	25.17	24.33	25.33	30.32	22.34	24.86	25.90	105.91	237.86	22.83	21.89	22.42	589.14
2008	20.56	27.42	31.68	33.63	25.14	25.14	24.81	39.94	175.33	44.14	21.38	21.33	490.50
2009	21.21	21.12	21.14	21.42	21.42	20.72	20.83	21.90	21.75	22.63	21.60	21.59	257.35
2010	23.21	22.75	22.24	22.70	23.11	25.54	36.40	67.24	48.69	36.45	31.96	26.45	386.73
2011	25.21	24.38	23.34	23.98	24.80	30.36	51.96	112.57	75.62	50.26	42.33	31.30	516.12
2012	22.09	21.58	21.88	22.47	22.44	22.29	55.84	168.86	21.79	22.45	22.00	21.93	445.63
2013	24.29	22.08	22.40	22.48	28.80	40.11	63.79	75.02	89.96	36.00	42.43	25.38	492.74
2014	23.70	22.69	22.47	22.91	24.79	29.58	52.00	105.92	59.01	36.29	34.68	26.26	460.30
2015	23.82	22.68	22.52	22.96	25.21	30.59	55.90	115.59	61.60	36.25	35.36	26.22	478.69
2016	23.48	21.49	22.32	22.70	25.31	30.64	56.88	116.35	58.09	32.75	33.62	24.95	468.57
MEAN	32.74	33.23	33.97	34.85	36.54	37.88	44.61	89.58	82.53	41.96	35.98	33.03	536.90

Appendix-D: Downstream Water Requirement

Months	Domestic	Municipal	Industry	DMI	Env'tal
Jan	1.50	1.66	4.15	7.31	3.44
Feb	1.50	1.37	4.15	7.02	3.02
Mar	1.50	1.37	4.15	7.02	3.18
Apr	1.50	0.99	4.15	6.64	2.78
May	1.50	1.49	4.15	7.14	3.56
Jun	1.50	1.26	4.15	6.91	3.73
Jul	1.50	1.46	4.15	7.11	3.79
Aug	1.50	1.46	4.15	7.11	3.64
Sep	1.50	1.46	4.15	7.11	3.82
Oct	1.50	1.46	4.15	7.11	3.76
Nov	1.50	1.84	4.15	7.49	3.85
Dec	1.50	1.69	4.15	7.34	3.49

Name of user	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun
wonji main	4.9	4.5	4.71	4.84	4.56	4.31	3.83	3.72
Dodota pump	1.68	1.56	1.62	1.67	1.57	1.49	1.32	1.28
waketiyo pump	0.44	0.4	0.42	0.43	0.41	0.39	0.34	0.33
welenchti intake	3.15	2.9	3.07	3.11	2.95	2.6	3.01	2.15
wonji kuriftu cooperatives	0.12	0.11	0.12	0.12	0.11	0.1	0.12	0.08
Boku Kurabo Cooperatives	0.1	0.09	0.09	0.09	0.08	0.06	0.08	0.03
Adulak Boku Cooperatives	0.1	0.1	0.1	0.1	0.08	0.06	0.09	0.04
Wake-Mia Cooperatives	0.15	0.14	0.15	0.14	0.12	0.09	0.13	0.05
Wake-Tiyo Cooperatives	0.07	0.07	0.07	0.07	0.06	0.04	0.06	0.02
Africa juice @ degaga	0.1	0.1	0.1	0.1	0.09	0.07	0.09	0.04
Tibila farmers @Tibila diversion	2.06	1.96	2.04	1.96	1.63	1.22	1.71	0.72
Africa juice @memberehiwot	0.03	0.03	0.03	0.03	0.03	0.02	0.03	0.01
upper awash Ag.ind @abadiska	0.36	0.32	0.3	0.22	0.26	0.2	0.32	0.39
upper awash Ag.ind @Nura Hera	1.32	1.17	1.09	0.79	0.94	0.82	1.67	2.02
Farmers @Nura Hera	0.44	0.39	0.36	0.25	0.31	0.22	0.35	0.42
Farmers @Merti woreda	0.39	0.34	0.32	0.22	0.27	0.19	0.3	0.37
Farmers @Jeju woreda	0.45	0.4	0.37	0.26	0.31	0.22	0.36	0.43
Farmers @mertijeju main canal	1.56	1.39	1.29	0.9	1.08	0.77	1.24	1.49
upper awash Ag.ind @mertijeju	0.24	0.22	0.2	0.14	0.17	0.14	0.27	0.33
Awash winery @mertijeju	0.22	0.19	0.18	0.13	0.16	0.12	0.18	0.21
Fentale project @kawa diversion	3.62	3.21	2.97	2.07	2.5	1.78	2.86	3.44
Bole gobgota main canal	0.69	0.61	0.57	0.39	0.47	0.34	0.54	0.65
farmers @ abadir	0.33	0.29	0.27	0.19	0.23	0.16	0.26	0.31
metehara SF @ abadir	2.95	2.61	2.58	2.23	2.56	2.21	2.73	3.03
metehara SF @ merti canal	5.44	4.82	4.76	4.11	4.72	4.08	5.03	5.59
farmers @ merti canal	0.38	0.34	0.31	0.22	0.26	0.19	0.3	0.36
Middle Awash @melka sedi weir	5.31	4.94	4.63	4.08	4.51	4.98	6.85	8.49
Total irrigation demand from koka	38.46	34.91	34.39	30.23	31.8	27.84	35.56	37.28
Total irrigation demand from koka	36.6	33.2	32.72	28.86	30.44	26.87	34.07	36
Sub total upper awash	33.14	29.97	29.77	26.15	27.28	22.86	28.71	28.79
Remarks	Additional 2 m3/s for industry use in all months for Wonji Sugar Factory							
	Additional 0.25 m3/s for industry use in all months for Merti Agro Industry							
	Additional 1.5 m3/s for industry use in all months for Meetehara Sugar Factory							

Appendix-E: Calibrated and Validated Graphs by HEC-HMS Model

