Kristina Shrestha

Investigating the Technical Concept of Retrofitting of Non-Hydro Reservoir and Dam

Master's thesis in Hydropower Development
Supervisor: Tor Haakon Bakken
Co-supervisor: Leif Lia
June 2022

Master's thesis

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Abstract

The number of dams build explicitly for the purpose of power production is increasing gradually due to the global increase in energy demand. This increment in energy demand has caused gradual shift from fossil fuels to renewable energy options. This has encouraged developments, innovations, and optimizations in the renewable energy sector. One of the developments in this sector is retrofitting of the preexisting non-power dams. Retrofitting of a non-power dam is done by installing power generating unit while maintaining the integrity and the compatibility with the purpose the dam was originally built for. However, before any intervention to dam there should be proper analysis whether this technology will affect the existing water demand site. Similarly, the economics involving the costs of different retrofitting alternatives should also be investigated prior to its implementation.

This study aims at finding the hydropower potential of the most feasible option of a non-powered dam without the intervention of existing water use. For this, a study location is selected in Kiringya district in Central Kenya on the River Thiba on the tributary of River Tana. The study is based on Upper Tana subbasin on Thiba Dam, whose main purpose is to supply water to Mwea Irrigation Scheme (MIS). For evaluating the water availability, a hydrological model (WEAP) is used which is calibrated from period between 1978 to 1995 with monthly time step where the parameters are calibrated for the streamflow discharge data of an unregulated sub-basin. CROPWAT software is used to find the Irrigation Water Requirement (IWR) in MIS. Using this IWR, the water demand for MIS is fixed which governs the release of water from the dam. Two technical solutions have been opted for retrofitting viz. pipe and weir scheme. Economic analysis for both schemes is done, and the most feasible option is recommended.

The result from the hydrological model shows NSE and PBIAS of 54% and -8.27% in calibration respectively. There is monthly variation in irrigation water demand in MIS with an annual total of 567.5 mm/ha. The potential from pipe scheme is 4.91 MW whereas from diversion weir scheme is 3.7 MW. Both selected schemes are financially sound; however, the pipe scheme is more feasible than weir scheme from all financial indicators. It is evident from this study that retrofitting of dam for power generation is economically beneficial than construction of completely new hydropower in a pristine river. From this study, it is also evident that the scheme which taps the water directly from the dam and utilizes its head has more benefit than other options in retrofitting.

Acknowledgement

This thesis is submitted on behalf of partial fulfillment of the requirement for Master of Science in Hydropower Development at Norwegian University of Science and Technology (NTNU), Trondheim. This study has been done on spring semester between January 2022 to June 2022 under the supervision of Professor. Tor Haakon Bakken and co. supervision of Professor. Leif Lia.

During this study the knowledge gained from two years of Master program has been significantly useful as it covers the topics from hydropower planning, hydrological modelling, economics, water resource modelling. The additional asset while doing this thesis was knowing the use of different software like WEAP, QGIS and CROPWAT which can be directly or indirectly useful for me in further professional career.

There are many people who helped me through my journey while doing this study. First, I would like to thank my supervisor Prof. Tor Haakon Bakken and co. supervisor Prof. Leif Lia for their continuous support and guidance throughout my journey from the very beginning. I am very grateful for their quick response for every problem that I faced during this study. Further, I would like to thank Malthe Winje Infrapower AS, Oslo for providing me the reference and necessary data for doing this thesis research. I would also like to thank Prof. Knut Alfredsen and Prof. Oddbjørn Brunland for their wonderful guidance during this study. Finally, I would like to thank my friend Sanyam Ghimire for his technical advice and my family and friends for their support during this period.

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Abbreviations

CWR	Crop Water Requirements
DC	Deep Conductivity
DEM	Digital Elevation Model
DWC	Deep Water Conductivity
ET	Evapotranspiration
FAO	Food and Agriculture Organization
GIS	Geographic Information System
GWh	Giga Watt Hour
ha	Hectares
HPP	Hydro Power Plant
IWR	Irrigation Water Requirements
KWh	Kilo Watt Hour
LCOE	Levelized Cost of Electricity
masl	Meters above sea level
MCM	Million Cubic Meter
MIS	Mwea Irrigation Scheme
NOK	Norwegian Kroner
NPD	Non-Powered Dam
NSE	Nash- Sutcliffe Efficiency
NVE	Norwegian Water Resource and Energy Directory
PBIAS	Percentage Bias
PET	Potential Evapotranspiration
PFD	Preferred Flow Direction
PH	Powerhouse
QGIS	Quantum GIS
ROR	Run of the River
RRF	Runoff Resistance factor
RSHP	Rukenya Small Hydropower Project
RZC	Root Zone Conductivity
USD	United States Dollar
WEAP	Water Evaluation and Planning
yrs	Years

1 Introduction

1.1 Background

Water is a vital natural resource essential for all form of life living in this planet. People have been managing water since ancient age for different purposes like drinking, household needs, irrigation, recreation, industry, agriculture, electricity. The present water shortage is one of the primary issues of the world which will be more critical in the future (Gupta et al., 1999). Hence, study of the proper and optimal distribution of water and possible hydro electricity generation from the available water resources in different facilities are so much important to encounter current and future crisis related to water and energy.

Renewable source of energy is the form of naturally replenishing energy. Hydropower is one of the renewable sources of energy which is dependent on the water cycle. The proper use of energy is primary in the today's world of energy demand. Experts from all over the world are in option of the utilization of renewable energy source (Shahzad, 2015).

A sustainable hydropower project needs a proper planning and careful design to meet all the challenges (Kaunda et al., 2012). The construction of reservoir is useful in storing the water in dry period and helps in flood retention. Even though reservoir equally important in the areas where there is climate change, it has its own negative side effects for the surrounding ecosystem. The construction of dam that will outrage the river flow and make fluctuations on the natural ecological system. Retrofitting of non-powered dam can be one of the best choices for the increasing renewable energy with only few alternations to the dam site without the intervention in flow.

Hydropower projects can be made multipurpose schemes. These schemes can be irrigation, drinking water , recreation, cultural heritage, pisciculture etc. (IRENA, 2020b). There are many large dams and reservoirs in the world other than those built for hydropower production. Taking reference from ICOLD database published in 2019, which contain almost all the information about large dams and reservoirs, most of the dams in Africa, Asia and Europe are not meant for power production. Only about 20% of the large dam around the world are used for generating electricity. With all these dams having at least some potential of hydropower production will definitely give a huge contribution in renewable source of energy when combined in total. So, it is of paramount importance to see the possibility of retrofitting of dam and hydropower generation to prevent the additional impact on environment, optimal usage of available resources and scientific distribution of water for social welfare. In simple words, adding or expanding hydroelectric facilities to an existing dam those which has not been used for power production is called retrofitting of dams (Kao et al., 2009).

1.2 Objectives

The main objectives of the study are as follows:

1. Identify a non-hydropower dam with a potential for retrofitting, where technical information about the dam and dam site can be found.
2. Assess the overall hydrological potential for retrofitting with use of modelling tool WEAP for selected reservoir /dam.
3. Assess the possible technical solution for the installation / implementation of hydropower technology in the selected/studied sites (dam /reservoir).
4. Estimate the economic costs of the investigated technical solution.

1.3 Study approach

For this study, initially, the location of Non-Powered Dam (NPD) and water demand site was found out using Google maps. Based on literature review, data availability and consultations with professors, Thiba Dam and its periphery was selected as the study area. Information regarding the dam was obtained from Malthe Winje Infrapower AS and relevant literature reviews. From the acquired data, the corresponding coordinates of study area was transferred to QGIS to analyze spatial information and composing graphical maps. WEAP was set up for catchment delineation mode to create the catchment area of basin. The vector layer from QGIS of different projected coordinates were exported to WEAP for full detail and exact location in basin. After the schematic setup in WEAP, hydrological model was simulated. Water demand was evaluated using CROPWAT which was further added in WEAP model to decide the release from dam. Two technical solutions were selected for evaluation of retrofitting. Finally, the possible technical solution was investigated and proceeded with the economic analysis for determining the best possible solution.

1.4 Organization of Study

Chapter 1	includes general introduction and overview of study and outlines its objective
Chapter 2	provides general theory and literature review in relevance with this study.
Chapter 3	describes about study area
Chapter 4	contains details about the data used for this study
Chapter 5	gives information about the materials used and methods used for the analysis in this study
Chapter 6	presents the results from the analysis done
Chapter 7	discusses the results of the study and its limitations
Chapter 8	presents the conclusion of this study

2 Theory and Literature review

This section is for the detail discussion of the concept of generation of hydropower and its connection with retrofitting of non-hydropower reservoir and dams. This section also describes basic principles of chosen software (WEAP, CROPWAT) while conducting this study. It also contains the literature review of cost economic analysis.

2.1 Hydropower

Hydropower is one of the largest renewable sources of energy for which uses water flow to move the turbine. The generated mechanical force can be used as producing electricity power. USBR, (2011) proposed the equation for calculating the potential hydropower generation in megawatt hours [MWh] as in Equation (2-1).

$$P = \frac{\mathrm{Q} * \Delta H * \eta * \mathrm{T}}{11800} \tag{2-1}$$

Where, P is potential hydropower generation, Q is discharge in [ft^3/s], ΔH is effective head [m], η is efficiency, T is total generation period in Hours [H].

Capacity factor is also one of the essential parameters to evaluate the generation of electricity. Power generation increases as the capacity factor increases (Asfaw & Hashim, 2011).

The actual hydropower output is primarily determined by hydrology, different operational constraints, river type, therefore the computed installed capacity tend to overestimate or underestimate when using historic capacity factor. Capacity factor defined as the actual energy generation to the ratio of installed capacity (Hadjerioua et al., 2012). The equation for calculating capacity factor is given in Equation (2-2).

$$\text{Capacity Factor} = \frac{\text{Annual Produced Energy}}{\mathit{Installed\ Capacity}} \tag{2-2}$$

Where, Annual Produced Energy [MWh/year], Installed Capacity [MWh/year]

2.2 Past Studies on Retrofitting of Dams

The already existed dams around the world can be retrofitted with turbines for electrical power production. The power generating units can be incorporated within the dam body if possible, or it could also be built outside in some convenient location. Out of several thousand dams across the globe, many of them are just fulfilling their primary purpose of construction for example irrigation or navigation while some of them pose good power production. So, it is of important to look at the scope and possibility of retrofitting for the mutual benefit in the sector of energy, environment, and economy in overall. There are some challenges related to structural stability issues when dams are modified into another form because these dams in their design period might not have considered future structural changes. A thorough study and behavior of dam in question is must to avoid any catastrophe by dam failure once they come into operation.

Retrofitting the existing dams with additional installation of hydromechanical equipment also has some major issues related to the structural integrity and safety of dam, construction cost and complex engineering tasks. Kao et al., (2009) describes in his paper about a new approach based on innovative hydro turbines is presented in his paper to tackle hydropower development's needs and challenges. This new approach has four design steps 1) an updraft flow arrangement, 2) a vertical flow control valve in place of wicket gats, 3) draft tube being replace with a divergent runner flow chamber and 4) exit flow at surface tail water level. This new way of design is supposed to be a cost-effective option, need less time, environmentally friendly, easy operation and maintenance and low fish mortality.

Zhou et al., (2019) proposed a new energy efficient way for low head dam with introduction of siphon hydro turbine. This does not disturb dam body and no threats are imposed to the structural integrity of dam. Hence, this syphon turbine is good for ultra-low head dam retrofitting. For dams with higher head, the syphon may pose serious risks with the fish passage and give higher fish mortality rate.

Another best method to retrofit the existing dam is with the employment of matrix of low head turbines combined to produce optimum amount of energy for both in the low and high flow situation (Cora et al., 2020). In this design concept, some of the turbines can be shut down or run for certain time to match the flow coming to the turbine inlet which optimize the generation efficiency.

2.2.1 Description of Rukenya Small Hydropower Project (RSHP)

There was already a prefeasibility study done in regard with the construction of a small hydropower in Thiba Dam. The preliminary data required for this study was acquired by Malthe Winje Infrapower AS, Oslo. As per the PDFs and official report published by Malthe Winje Infrapower AS, the Kenyan company M/s Hydel Engineering and Construction Ltd (HECL) has envisaged Rukenya Small Hydropower Project (RSHP) [3.5MW] to enhance national energy security as well as create employment and income generation within the country and utilize the water for power generation from NPD (Thiba dam). In addition, they have proposed a project site location for intake site and powerhouse between Thiba dam and Mwea Irrigation Scheme (MIS).

2.3 Practical considerations for retrofitting

The dam retrofitting for the energy generation should not stress the water availability and distribution to already existed water demand for example irrigation, drinking water supply or navigation. For this to happen, the extra water which spills to the waste can only be utilized for power production which therefore do not interfere with other users and consumers. This also makes sure that the other users do not need to go for compromise for possible water scarcity (Bakken et al., 2016).

The possible environmental impact with the dam retrofitting is unknown and Yuguda et al., (2020) describes possible impacts once the retrofitting starts rebuilding the dam. They proposed three possible options for retrofitting which are modification of the embankment, or one spillway bay or one scour radial gate. In their study, it was noted that 90 % of the environmental impacts occur during the construction phase and steel is dominating of all three options in material input. To minimize these impacts, it was suggested that strategic planning, new material, and manufacturing technique are required to improve the performance.

Hydropower can be regulated within a short period of time which gives flexibility for the power system. Hence, it is necessary to find out the existing grid system and the possible power demand fluctuations to meet the production power (IRENA, 2012). In dam retrofitting, most of the power generation is when there is extra water available which is surplus water from the primary water consumer. If the power production and demand period could not be matched, there may arise a question of dam retrofitting and power generation. Hence, it is always a good practice to be clear in these technical aspects from the planning stage of the project site area.

2.4 Water Evaluation and Planning (WEAP)

WEAP is a software developed by Stockholm Environment Institute (SEI) in 1988 which is used for integrated water resource planning. In this study the version WEAP:2021.0103 is used. A time series of temperature, precipitation, evapotranspiration, and runoff data is used to simulate the water system (Yilmaz, 2015).

According to Yates et al., (2005),WEAP model has two primary functions:

- Simulating natural hydrological processes (e.g., evapotranspiration, runoff, and infiltration) to assess the availability of water in a catchment.
- Simulating the influence of consumption and non-consumption use of water on natural system to estimate the impact of human water use.

The WEAP application is based on the following steps described below:

- It defines the time frame, spatial boundary, and system configuration.
- It establishes the current account which provides the scenario based on water demand and supply to the system.

2.4.1 Catchment Simulation Method

There are five different methods for catchment simulation (Sieber & Purkey, 2015) . The different methods are

1. Rainfall runoff method
2. Simplified coefficient method
3. Plant growth method
4. Soil moisture method
5. MABIA method

Soil Moisture Method

Among the five different method Soil Moisture Method is used for the catchment simulation process. This method is used because of versatile and specific infiltration calculation calculating runoff from time step to time step considering the initial soil moisture method. In addition, this method uses the algorithm for the runoff, evaporation, and evapotranspiration calculation. WEAP is a semi distributed hydrological model where the parameters are distributed for different sub-basins but are averaged in an individual basin. So, it does calculation in each sub-basin with different set of parameters for each sub-basin. For simulation process time series data like precipitation, temperature, humidity data are required. The factors like wind speed, cloudiness, albedo constant, freezing point, melting point can be entered to the model according to the location of catchment otherwise WEAP assign default values.

The climatic data such as temperature, precipitation, windspeed, and humidity are used to calculate the Evapotranspiration (ET) on WEAP. Evapotranspiration is the combination of evaporation and transpiration where water is lost by evaporation from soil surface and transpiration by crop. The main factor affecting ET are radiation, air temperature, humidity, windspeed (Allen et al., 1998).The actual evapotranspiration (ET_c) is obtained by multiplying the crop coefficient (K_c) times the theoretical evapotranspiration (ET_o) (Batchelor, 1984). The actual ET_c is illustrated in the Equation (2-3)

$$ET_c = ET_o * K_c \tag{2-3}$$

In order to calculate ET for a standard crop, the modified version of Penman-Monteith is used (Sieber & Purkey, 2015). Based on the snow cover, the albedo differs from 0.15 to 0.25, whereas the soil heat flux is excluded in the second modification. The crop coefficient can be selected by land use data available. Land cover bands within the different elevation can be automatically generated by catchment delineation mode. According to Allen et al., (1998) the crop coefficient values are based on FAO Irrigation and drainage paper no.56. If crop coefficient values are not determined, then it automatically refers to the default value 1.

Irrigation water demand need to be specified manually. The irrigation water requirement is derived from the river withdrawal catchment area. An annual activity level for the withdrawal must be specified and amount of each withdrawal can be attributed to percentage of monthly variation.

A two-bucket model is the Soil Moisture Method, which separates the soil into two different layers. The first bucket is called root zone and second bucket is called deep zone. Different process of water balance occurs in each layer. The catchment has a subclassification defined by the disaggregation that was considered appropriate to represent processes such as evapotranspiration, runoff, infiltration, and percolation. Each subclassification of the catchment corresponds to the root zone, while deeper zone is assigned to the entire catchment. It can be said that the hydrological model that follows the soil moisture has nine parameters that influence the water balance processes. This method converts the climate in each catchment into flows to the rivers or groundwater nodes simulating processes such as runoff, interflow, percolation, or base flow. The surface runoff is controlled by runoff resistance factor. The direct runoff only occurs when the root zone is fully saturated. The root zone outflows are the interflow and percolation, which depend on the soil water capacity, the soil water conductivity, and the preferred flow direction. The preferred flow direction divides the water flow between interflow and percolation. The deep zone also has a deep-water capacity and deep-water conductivity, which controls the base flow. It is necessary to set the initial condition of some parameters. Z1 and Z2 represent the initial relative water storage of the root zone and the deep zone, expressed as a percentage of total water capacity, respectively. It is the water balance between inflows and outflows, where the difference between the in and out each of the two layers represents the moisture changes in the root zone and deep zone. Schematic representation of two bucket model is illustrated in Figure 2-1.

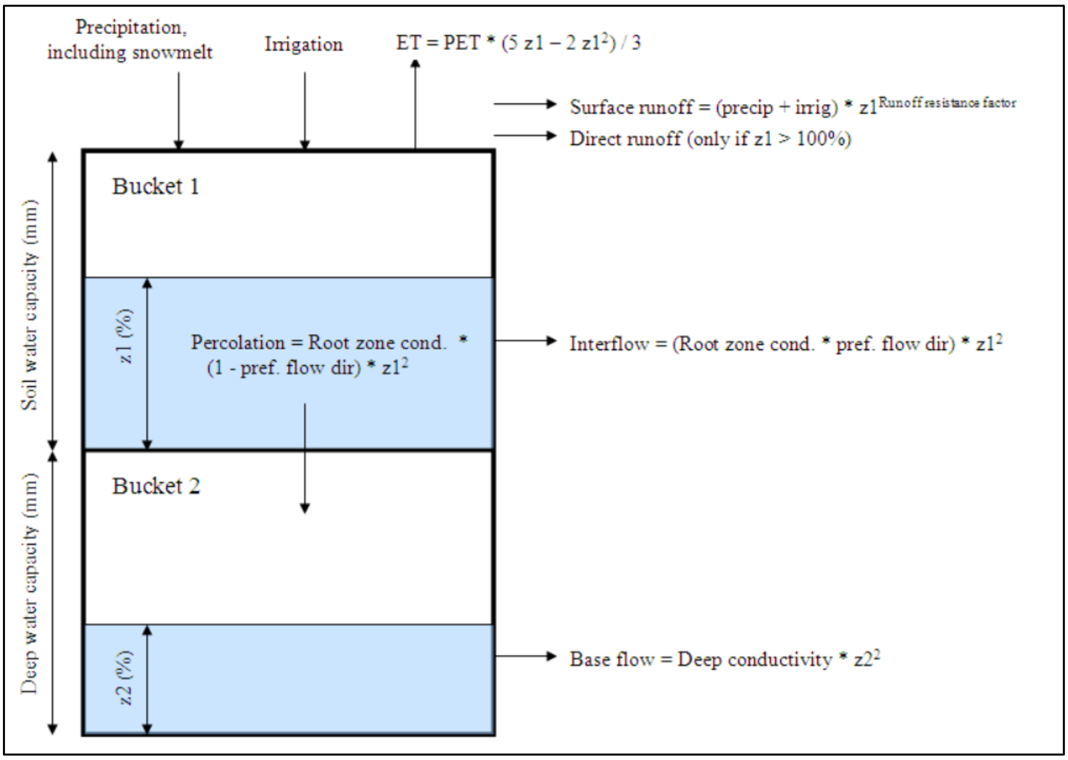

Figure 2-1: Schematic representation of two bucket hydrological model used in WEAP (Sieber & Purkey, 2015).

The parameters for the two-bucket model by (Sieber & Purkey, 2015) are as follows:

- Kc - The crop coefficient, relative to the reference crop, it depends on land class type
- Runoff Resistance Factor (RRF)- It controls the response of surface runoff. It depends on the land slope.
- Preferred Flow Direction (PFD) -It is used to separate the flow out of top bucket between interflow and flow to the lower bucket. Values differ depending on the type of land class.
- Root Zone Conductivity (RZC) – In the top bucket when the relative storage Z1= 1.0, the conductivity rate will be partitioned between the interflow and flow to the lower bucket depending on the preferred flow direction. Rates can vary depending on the type of land classes.

- Deep Conductivity (DC) – At full saturation (Z2=0), conductivity at the deep layer (bottom layer) controls the transmission of baseflow. This does not vary according to the land class. It has a single value for whole catchment.
- Deep Water Capacity (DWC): DWC is water holding capacity of lower bucket. This does not vary according to the land class. It has a single value for whole catchment.
- Soil Water capacity (SWC): SWC is the water holding capacity of the upper bucket. It varies according to the land class.
- Initial Z1- Z1 is the root zone water storage capacity as a percentage of total storage capacity.
- Initial Z2- Z2 represents the percentage of total effective storage indicated in the lower soil bucket (deep water capacity).

2.4.2 Reservoir Zones and Operation

Reservoir is classified into four zones viz. inactive/dead storage zone, buffer zone, conservation zone and flood control zone. Active zone of the reservoir is used for withdrawing water to specific purposes. Buffer and conservation zones are active zones. Water in dead zone is not available for use and kept for sediment deposition purposes. The flood control zone is always left empty in order retain water during high flood events. So, the water level will never reach beyond conservation zone under normal operating circumstances.

WEAP allows the water to be fully utilized to meet the water demand downstream and demand of energy to hydropower until the level is in the conservation zone. Proceeding the water from the conservation zone to buffer zone the water availability will only subject to a constraint expressed in percentage (Hashimoto et al., 1982). The inactive pool does not have water available for allocation, but evaporation may bring the water from the reservoir into the pool under extreme conditions.

To define the zone properly the volume must be allocate corresponding their height. Buffer coefficient slows down the volume of water release from the buffer zone. The range of buffer coefficient is between (0-1). The coefficient near 1.0 will release more rapidly while coefficient near 0 will preserve the water maintaining the water level at buffer zone leaving the unmet demand at demand site. To summarize, buffer coefficient determines the amount of cutback based on volume at which the top of buffer represents the release volume to be cutback. If reservoir zones are not specified, then the system will assume the whole reservoir as conservation zone. In Figure 2-2), schematic representation of reservoir model (left) represents the reservoir model when zones are defined, (right) represents the reservoir model when zones are not defined. Q represents the outflow from the dam.

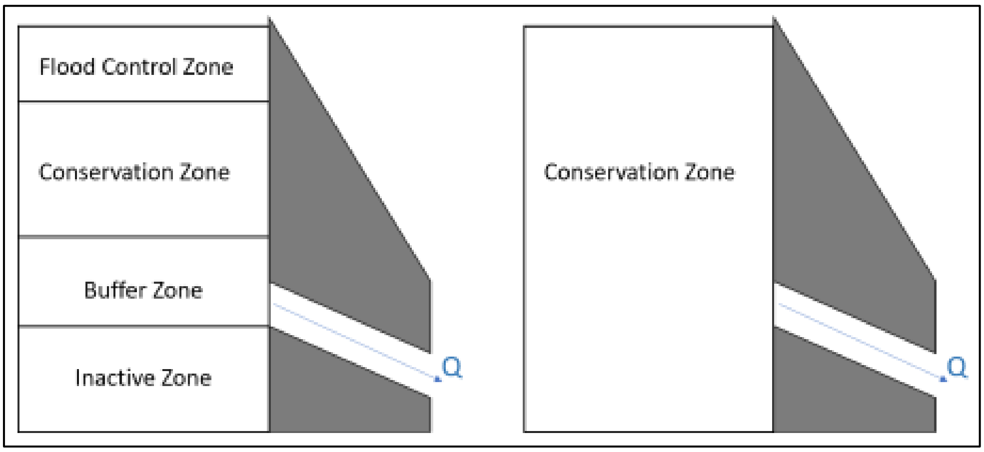

Figure 2-2: Schematic representation of reservoir zone used in WEAP (Sieber & Purkey, 2015). Left side of figure represents reservoir models when zones are defined, and right side of the figure represents when the reservoir zones are not defined.

2.4.3 Hydropower Generation

WEAP calculates hydropower generation using the available water in the reservoir or in a river for a run of river power plant. Individual withdrawals, hydropower needs, reservoir filling, and environmental demands are all prioritized in terms of demand for water from the reservoir. If water volumes are not sufficient, the lowest priority demands will not be met.

WEAP determines the output of energy based on the amount of water available for hydropower generation. In this case, the output is dependent on the amount of water which passes through the turbine during the timestep and the HGF, or hydro generation factor. There are five factors involved in this process: available head, plant efficiency, density of water, and gravitational force. Water levels at the beginning of the time step are subtracted from the water level of the tailwater to estimate the available head. General efficiency factor (η) is liable for the head losses and energy losses. A plant factor can be adjusted for seasonal hydropower production control in WEAP. The resulting equation of Hydropower Generation factor (HGF) given in Equation (2-4).

$$\mathrm{HGF} = \frac{\rho . H . PF . \eta . \mathrm{g}}{1\ 000\ 000\ 000} \tag{2-4}$$

The energy generated is computed as in (2-5)

$$E = HGF.V \tag{2-5}$$

Where, E is energy generated in [GJ], HGF [GJ/m^3], ρ is the density of water [1000kg/m^3], H is the head [m], PF is the plant factor, g is acceleration due to gravity [9.81m^3/s], V is the water volume through the turbine during the timestep [m^3/timestep]

2.5 CROPWAT

CROPWAT is a computer program developed by Land and Water Division of Food and Agriculture Organization (FAO) in Rome to calculate crop water requirements and irrigation water requirements. It follows the guidelines of Irrigation and Drainage Series, paper No. 56 "Crop Evapotranspiration - Guidelines for computing crop water requirements (Allen et al., 1998). For the calculation of CROPWAT it requires the data on soil, climate, and crop requirement. To estimate reference evapotranspiration, crop water requirement (CWR) and to support Irrigation water requirement (IR_n) in the model the calculation is based on reference evapotranspiration (ET_o) which is as per Penman-Monteith and other crops parameters. The equation for Penman- Monteith method from FAO Irrigation and Drainage Paper 56 as in Equation (2-6)

$$Et_o = \frac{0.408\,\Delta(R_n - G) + \gamma \frac{900}{T + 273} u_2 \ (e_s - e_a)}{\Delta + \gamma\,(1 + 0.34\,u_2)} \tag{2-6}$$

Where, ET_0 is reference evapotranspiration [mm/day], Rn is net radiation at the crop surface [MJ m^{-2} day^{-1}], G is soil heat flux density [MJ m^{-2} day^{-1}], T is mean daily air temperature at 2 m height [^{0}C], U_2 is wind speed at 2 m height [m /s], e_s is saturation vapor pressure [KP_a], e_a is actual vapor pressure [KP_a], e_s-e_a is saturation vapor pressure deficit [KPa], Δ is slope vapor pressure curve [KPa ^{0}C $^{-1}$], γ is psychrometric constant [KPa ^{0}C $^{-1}$]

2.5.1 Irrigation water requirement

Irrigation Water Requirements is the total quantity of water needed during the cropping period for cultivation of crop (Ali, 2010). It is important to distinguish between crop water requirements and irrigation requirements. Water used by crop for the cell construction and transpiration is called crop water requirement whereas, the irrigation water is the water that needs to be provided through the irrigation system to meet the full requirement for crop.Net

Irrigation Requirement (IR_n) does not consider losses associated with applying the water. The net irrigation requirement from field balance (Allen et al., 1998) as in Equation (2-7)

$$IR_n = (K_c ET_o - P_{eff}) \quad (2\text{-}7)$$

Where, IR_n is net irrigation requirement [mm], K_c is crop coefficient, ET_o is theoretical crop evapotranspiration [mm], P_{eff} is effective rainfall [mm], T is the total growing period of crop. Effective rainfall is calculated by choosing USDA Soil Conservation Method as in Equation (2-8) when total rainfall < 250mm and equation (2-9) when total rainfall >250 mm.

$$P_{eff} = \mathrm{TR} * \frac{(125 - 0.2 * TR)}{125} \quad (2\text{-}8)$$

$$P_{eff} = 125 + 0.1 * \mathrm{TR} \quad (2\text{-}9)$$

Where, P_{eff} is the effective rainfall [mm], TR is the total rainfall [mm],

2.6 Cost Estimation

Cost estimation for the hydropower plant is necessary for proper budget allocation of project. For hydropower plant the cost breakdown structure is divided into three parts as illustrated in Figure 2-3) proposed by O’Connor et al., (2015).

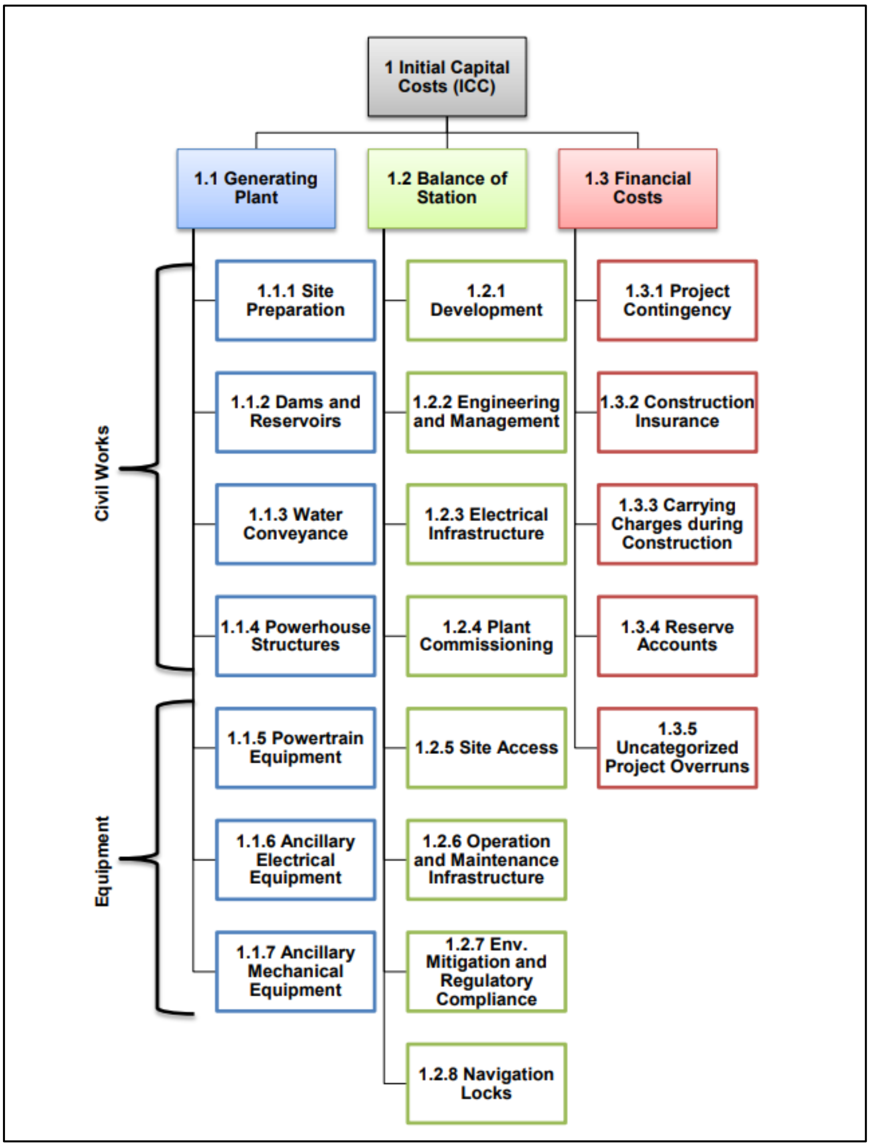

Figure 2-3: Hydropower cost breakdown structure (O'Connor et al., 2015).

The cost calculation for NPD may be little different than the newly constructed dams. For an NPD the construction of new powerhouse and other technical installation to the existing dam can only be considered for estimation. The costs associated with civil works and equipment account for approximately 81% of the total cost of the NPD development project, with equipment being the major cost component (O'Connor et al., 2015).

The average cost statistics for 36 engineering and 2 construction stage NPD projects illustrated by (O'Connor et al., 2015) is presented in Figure 2-4.

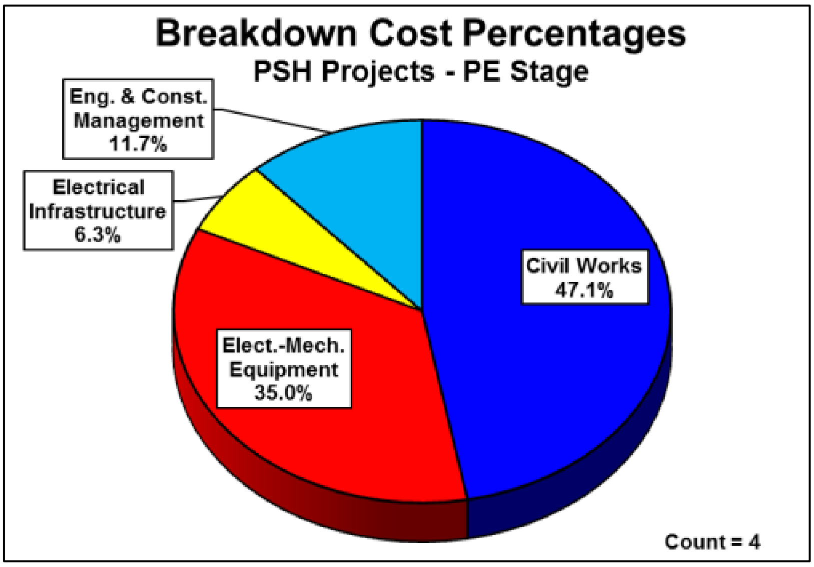

Figure 2-4: Overview of cost distribution for 83 NPD projects (O'Connor et al., 2015).

2.7 Hydropower Revenue and Economical Analysis

The major revenue source in most of the project is energy generation (Somani et al., 2021).

The main evaluation criteria for economic analysis are:

- Internal Rate of Return
- Net Present Value
- Benefit/Cost ratio (B/C ratio)

Net Present Value (NPV)

Considering the value of time, Net Present Value is a common measure to evaluate the economic viability of project. It measures the value of the project based on its costs and benefits over entire lifespan in relation to a particular year. NPV is evaluated by taking the difference between the present value of cash outflows (i.e., cost of construction) from the present value of cash inflows for the project over its lifetime. NPV is calculated by using Equation (2-10):

$$\text{NPV}= \sum_{t=0}^{n} \frac{(Cost-Revenue)_t}{(1+i)^t} \tag{2-10}$$

Where, i is the discount rate, t is the total number of years for project.

Discounting is the process of revaluing future costs and benefits to make them comparable to present values. It is necessary to bring the project's future streams of costs and benefits to a common denominator.

NPV must be positive for a project to be economically viable. This is the way to compare the liability of project. The positive value of NPV represent the profitability of the project. When there is more than one project to be evaluated and compared with same NPV, it is preferred to choose the project with lowest total cost.

- Internal Rate of Return (IRR)

An internal rate of return is an appraisal method that computes the discount rate for discounted cash flow investments, from which the net present value is zero.

- Payback period

Payback period is the time it takes the cash inflows from a capital investments project equal to the cash outflows, usually expressed in years. Payback period cannot be the basis to evaluate the project. It ignores some aspects of overall project.

- B/C ratio

B/c ratio involves the calculation of a ratio of benefit to cost. The B/C ratio will give the result in determining the acceptability of the project i.e., B/C >1 the project is acceptable.

- Levelized Cost of Energy (LCOE)

The LCOE is a process to compare the cheapest energy supply source or to estimate the competitiveness of an energy supply projects in the market. Calculating and comparing the LCOE can measure the value across the longer term showing projected life cycle costs.

The cost is calculated by discounting the total cost of the project through its lifetime divided by the discounted energy price. LCOE is evaluated based on the Equation (2-11).

$$LCOE = \frac{\sum_{t=1}^{n} \frac{I_t + M_t + F_t}{(1+r)^t}}{\sum_{t=1}^{n} \frac{E_t}{(1+r)^t}} \tag{2-11}$$

Where, r is the discount rate [%], n is the number of lifetimes considered, I_t is the investment cost, M_t is the operational and maintenance cost, F_t is the fuel cost, E_t is the electricity generation [KWh], t is the timestep.

3 Study Area

This section highlights the detail description of study area and main project features for which this thesis study is carried out.

3.1 Description of Study Area

Thiba River and its catchment is selected as the study area for this thesis work. The catchment area is located at Upper Tana basin in the Kirinyga district in Central Kenya. Thiba river is tributary of Tana River which is one of the longest rivers in Kenya. The study area lies in UTM Zone 37N within latitudes between 0° 11' S and 0 ° 48' S and longitudes between 37° 12' E and 37º 42' E. The total catchment area of Upper Tana basin is 2038 km^2 which is also represented as study area in Figure 3-1. In Figure 3-1 the overall study area including MIS is shown with the total catchment area of 2038 km^2. Similarly, the catchment area at the gauging station (4DA10) is 318 km^2.

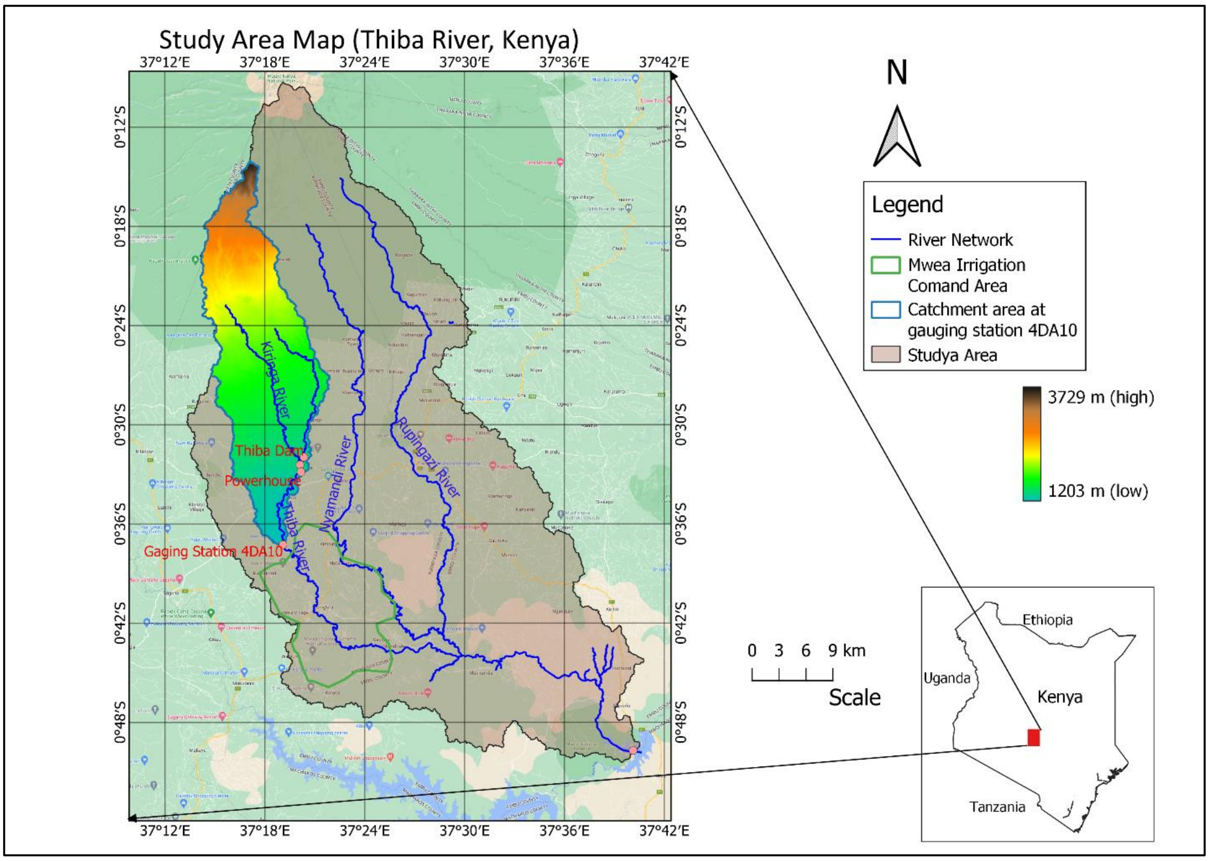

Figure 3-1: Location map of the study area. The study area is the catchment that encompasses rivers and the Mwea Irrigation Scheme(MIS) whereas the focus area is the catchment at gauging station 4DA10 where hydrological model is set up and its elevation above mean sea level is represented by the color ramp. Catchment of study area, gauging station(4DA10) and

Thiba Dam is 2038 km^2, 318 km^2 and 128 km^2 respectively. The MIS lies downstream of the 4DA10 gaging station.

3.2 Hypsographic Curve

A hypsometric curve shows the slope of the basin by plotting elevation versus the cumulative area of the basin. From analysis of Digital Elevation Model, it shows that the topography of upper Tana river basin extracted from the gauging station (4DA10) varies between 1300 masl to 3200 masl. Figure 3-2 represents the hypsometric curve of Upper Tana basin above gauging station point (4DA10).

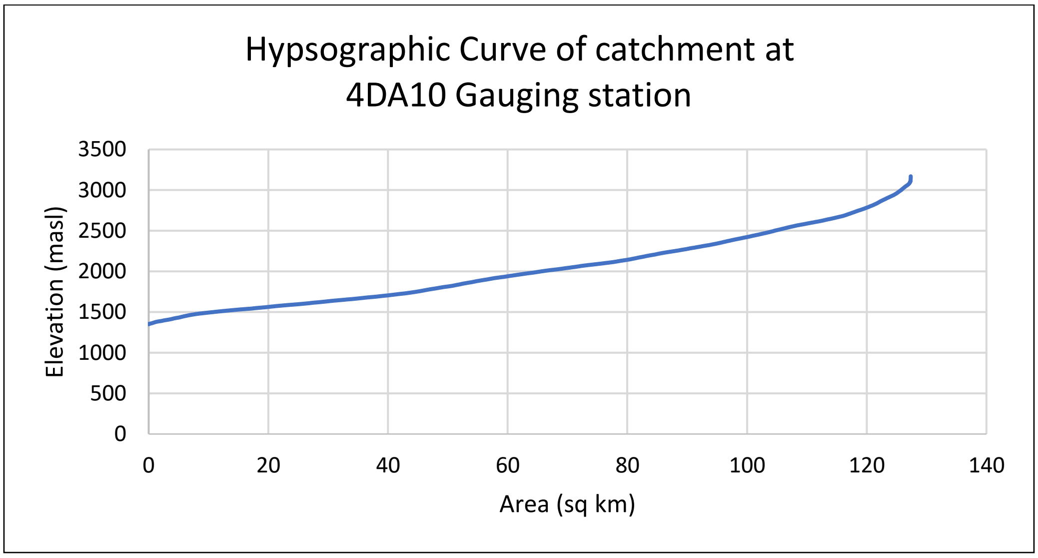

Figure 3-2: Hypsometric curve of catchment area at gauging station (4DA10).

3.3 Land use of study area

The land use data obtained from European Space Agency's Climate Change Initiative Land Cover database (ESA-CCI-LC) shows that majority of the land is covered by agriculture which is 71.59% of the total land use. Similarly, forest covers the second highest range of land use which is 27.40% of total land cover area. The land use map is shown in Figure 3-3 and the area covered by each land use is tabulated in Table 3-1.

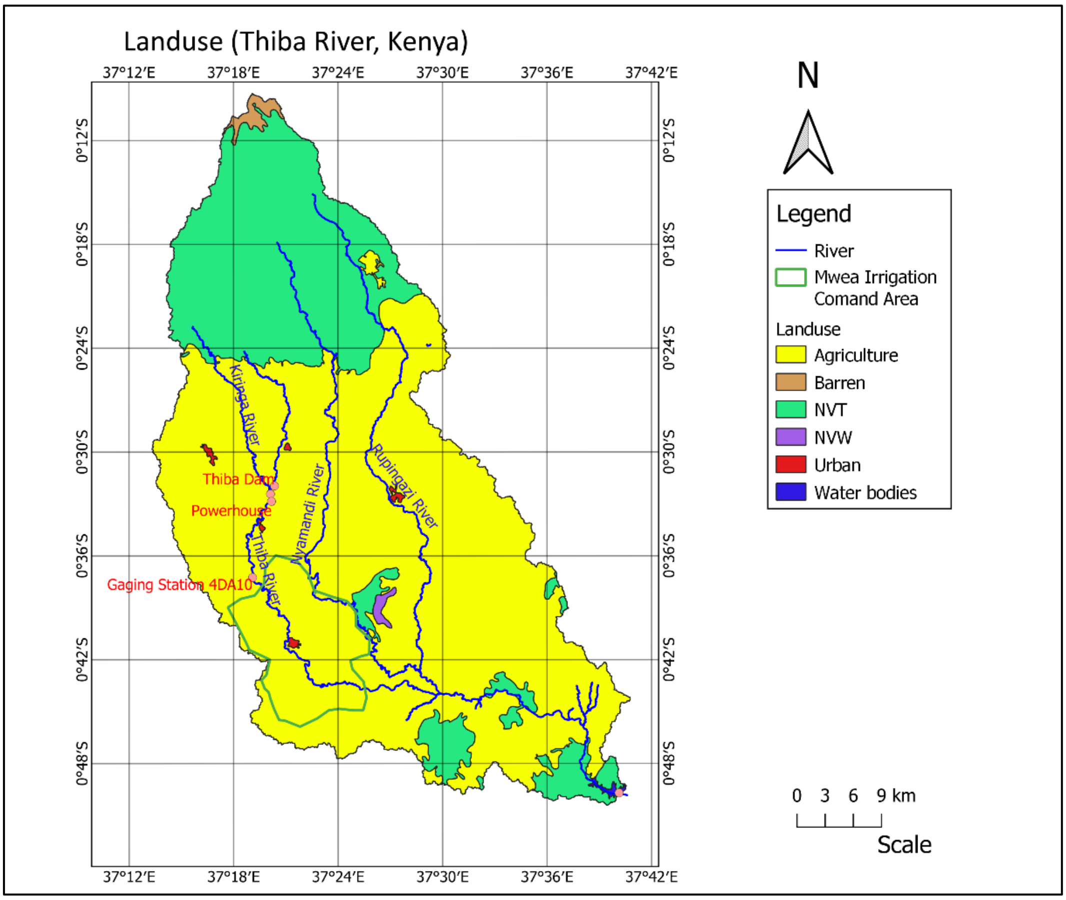

Figure 3-3: Land use of Upper Tana basin. NVT and NVW denote Natural Vegetation Terrestrial and Natural Vegetation Aquatic respectively.

Table 3-1: Land use map of Upper Tana basin.

Land Cover	**Code**	**Area (km^2)**	**% of Area Cover**
Agriculture	AG	1457.59	71.53
Natural vegetation terrestrial	NVT	558.26	27.40
Barren	BA	10.61	0.52
Urban	UR	4.46	0.22
Natural vegetation aquatic	NVW	4.32	0.21
Water bodies	WB	2.55	0.13

3.4 Dam and reservoir

According to the PDFs and official report published by Malthe Winje Infrapower, The salient features of the Thiba Dam are tabulated in Table 3-2.

Table 3-2: Sailent features of Thiba dam.

Main Data	**Unit**	**Value**
Catchment Area	km^2	128
Crest Level	masl	1385
Full Supply Level	masl	1380
Minimum Operating Level	masl	1369.7
Coffer Dam Crest level	masl	1369
Gauging Station Name	4DA10	

3.5 Climate at Study Area

The climatic conditions around the study area are tropical with two rainy seasons characterized by a short rainy season from April to May and a long rainy season from October to November (Akoko et al., 2020). About 930 mm of rain falls on average every year, while the temperature ranges between 14°C and 31°C. Relative humidity ranges between 55% and 70%.

4 Data Acquisition

This section gives description about the data, its collection and sources. The data set required for the study were collected from various sources. The climatic data required were obtained from Terrestrial Hydrology Research group (Princeton University) and AQUASTAT Climate Information Tool (https://aquastat.fao.org/climate-information-tool/). The flow data at gauging station (4DA10) for calibration of model was obtained from Malthe Winje Infrapower AS.

The major data set that is required are listed below:

4.1.1 Climate data

Precipitation, Temperature, Wind

Precipitation, temperature, wind, land cover data set were obtained from Terrestrial Hydrology Research Group (Princeton University). These meteorological datasets were dated from 1978 to 1995. The data are based on Princeton Satellite data with raster grid 720 rows * 1440 columns in coordinates 90N 180W – 90S 180E with a slope of 0.25° or about 28 km (Jayantari et al., 2019).

Humidity

The humidity data is obtained from AQUASTAT Climatic Information Tool (https://aquastat.fao.org/climate-information-tool/). These data are gridded dataset with horizontal resolution of 0.1° x 0.1° with temporal coverage from 1979 to present. These datasets are monthly averages with a relative humidity at a height of 2m above the surface. In this study to obtain the average humidity from the whole subbasin, random 2 coordinates were selected within the sub basin and average value from the 2 coordinates were used for the further calculations.

4.1.2 Digital Elevation Model

The high resolution available HydroSHEDS DEM with spatial resolution of data 3 arc seconds (90 meters) is used in this study. It is believed that the 90m data is capable of producing relatively accurate catchment area and stream network.

4.1.3 Land Cover

Using digital elevation data in each catchment, WEAP can calculate the land area in each elevation band. The built-in database for WEAP is from the European Space Agency’s Climate Change Initiative Land Cover database (ESA-CCI-LC) (https://www.esa-landcover-cci.org/). It includes 22 different types of land cover which can be rename or aggregated according to our needs.

An overview of spatial and temporal information of collected data is tabulated in the Table 4-1.

Table 4-1: Description of Data Collection.

Type of Data	**Source**	**Frequency**	**Time**
Discharge data	Malthe Winje Infrapower AS	Monthly Discharge	1978-1995
Precipitation	Terrestrial Hydrology Research Group (Princeton University)	Monthly Precipitation	1978-1995
Temperature	Terrestrial Hydrology Research Group (Princeton University)	Monthly Temperature	1978-1995
Wind Speed	Terrestrial Hydrology Research Group (Princeton University)	Monthly Wind Speed	1978-1995
Humidity	AQUASTAT Climatic Information Tool	Monthly percentage of Relative Humidity	January to December
DEM	Shuttle Radar Topography Mission (SRTM) 90m resolution		
Land Cover	European Space Agency’s Climate Change Initiative Land Cover database	Varies according to each elevation band in catchment area	

5 Methodology

In this section, the methodology utilized for this study as well as the main assumptions applied to retrofitting is presented. Firstly, the setting up of the WEAP model is explained. After that, the process of calculation of water demand is explained followed by proposal of the technical solution (alternatives) of retrofitting. Finally, the economic analysis is performed for each of the proposed solution.

The general methodology used during this study process is illustrated in Figure 5-1.

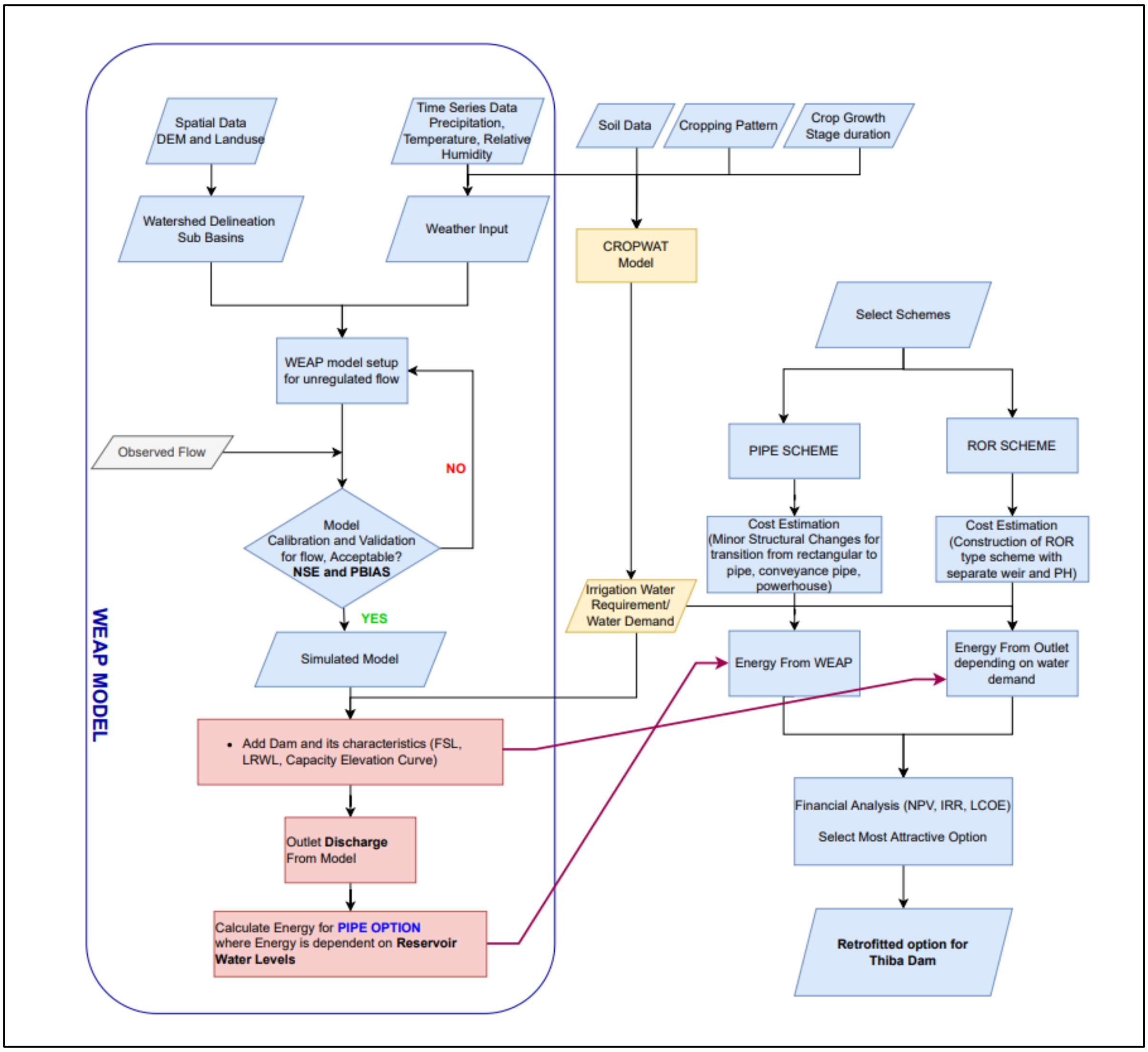

Figure 5-1: Schematic representation of methodology used during study process.

5.1 Method for estimation of retrofitting potential

5.1.1 Main assumption

The foremost assumption is that this retrofitting process and the energy generated will have minimum obstructive effect on existing water demand site. In this study, the current water use is for agricultural project (Mewa Irrigation Site). In addition to the first assumption there must be minimum environmental flow requirement must be meet for the surrounding environment to maintain a healthy ecosystem and there will be construction of pipeline to the proposed powerplant to increase the hydraulic head.

The water balance for the catchment is performed to evaluate the potential from the main assumption.

5.1.2 Choice of the case study

The study was chosen based on the following criteria. The first criteria there must be a potential of retrofit to the existing NPD whose main purpose is to regulate the water for other water demand rather than energy production. Second, the data for streamflow gauging station and technological description of reservoir must be available.

In Kenya, to expand the irrigable area as well as increase cropping intensity in Mwea Irrigation Scheme, a dam is constructed across the Thiba River. As explained in Section 2.2.1, to create energy security and employment within the country RSHP [3.5MW] is proposed near the village Njega on Thiba River in Kirinyaga County. They proposed a diversion weir and powerhouse that would be suitable for HPP, however, in this study there are two other technical solutions proposed and compared the best ones based on the economic analysis.

5.1.3 Tools

1. WEAP is considered suitable software for this study. The reasons for choosing WEAP software for this study is illustrated as below:

- It introduces the significant technological advance such as modern graphic user interface for solving robust algorithm water allocation problem and integrated set of hydrological components (Yates et al., 2005).
- It is possible to enter the information the information of reservoir and hydropower generating units, water demand capacity, water withdrawal points, environmental flow manually.

- It is possible to create more than one scenario and compare the results in many possible ways.

2. QGIS is used for analyzing geo referenced data, analyze spatial information, exporting graphical maps in form of raster, vector layers.
3. CROPWAT (FAO Irrigation Drainage NO. 56) is used to evaluate the water requirements for irrigation based on climate, soil, crop data of the area.

5.2 WEAP hydrological year setup

In the model setup, the hydrological year starts in January and ends in December. Therefore, the study period will run from January 1, 1978, to December 31, 1995. The current year of scenario is setup in 1978 and the reference year of scenario is setup between 1979- 1995. Here, the current year refers to the warmup period of the model and the reference year refers to the years where the model performance metrices (Section 5.6) are evaluated. Figure A-1 shows the schematic view of WEAP software with model setup.

5.3 Catchment delineation and reservoir

Upper Tana subbasin is created automatically using Catchment Delineation mode. The Upper Tana subbasin is again divided into four parts namely Thiba subbasin, Nyamandi subabsin, Rupingazi subbasin and South subbasin which is named after the rivers generated by automatic catchment delineation mode. Rivers created by automatic delineation mode are named for their historic names of river. The coordinates for the study like reservoir point, intake location for irrigation site, diversion weir, the command area for Mwea Irrigation Site, Gauging Station [4DA10] were first created in QGIS and respective vector layer were extracted from the QGIS to WEAP. Figure 5-2 represents Upper Tana subbasin created by catchment delineation mode in WEAP and further divided into four subbasin. The red triangle represents the NPD Thiba dam for this study purpose.

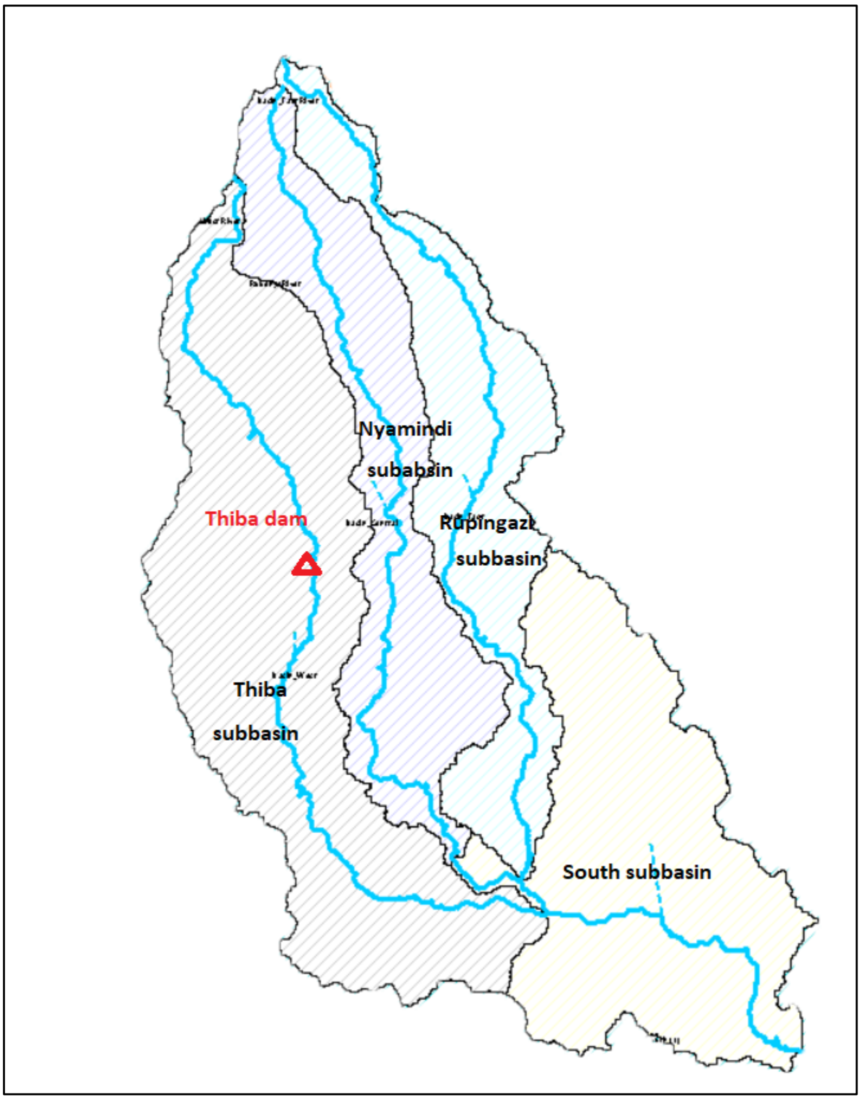

Figure 5-2: Upper Tana subbasin created by catchment delineation mode in WEAP and further divided into four subbasin. The red triangle represents the NPD Thiba dam.

5.4 WEAP Setup Details

Two scenarios were introduced in WEAP setup:

1. Scenario 1: This scenario represents without introduction of reservoir and hydropower plant.
2. Scenario 2: This scenario represents the introduction of reservoir and hydropower plant.

After catchment setup from catchment delineation mode and setting up the respective coordinates using vector layer from QGIS, the river nodes were and different links to the river were setup.

Reservoir nodes is set in Thiba river. It releases the water directly to demand sites. The salient feature of dam is taken from PDFs and official report but storing capacity of dam is evaluated based on raster calculation and DEM analysis of volume- area- elevation. From the obtained detail set of value from volume- area- elevation curve from Figure 5-3 is inserted manually for the reservoir elevation corresponding to its volume in WEAP.

Calculation for volume - area- elevation curve

The data for volume- area- elevation curve plays an important role in estimating the surface area and storage volume at any depth of the reservoir (Sayl et al., 2017).

A flow direction raster was generated using QGIS hydrological tools, then a flow accumulation raster was obtained subsequently, a stream raster was generated by selecting (1000cells) as the threshold point for stream generation. Then the dam location was selected as the pour point for watershed generation.

Using the calculator calculating elevation for which the volume is required by subtracting the required elevation with DEM followed by reclassifying of raster by positive (1) and inundation area. The inundation area raster is further averaged to find the average depth of inundation. Obtained average inundation depth were multiplied with inundation to find the capacity (volume) stored by dam in the specific elevation. This process was repeated from riverbed level to highest flood level. The curve showing reservoir volume and reservoir area at their respective elevation is illustrated in Figure 5-3.

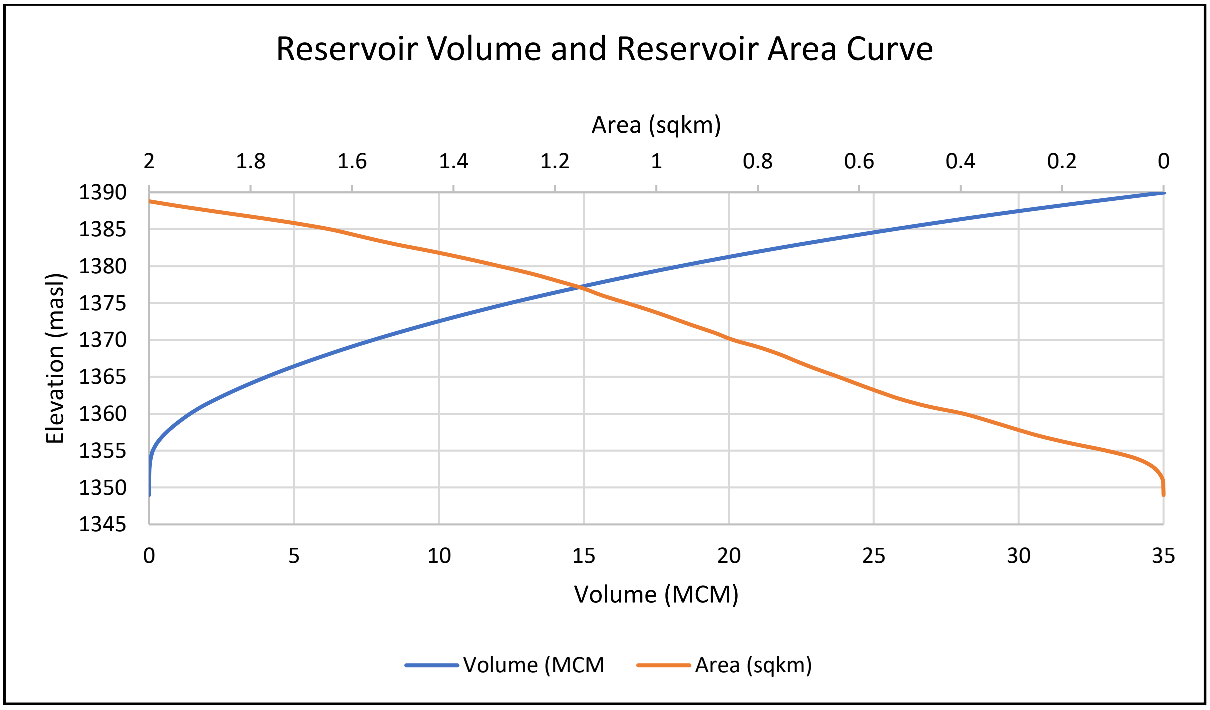

Figure 5-3: Curve showing volume- area- elevation of reservoir. 1380 masl is the highest-level water regulated and 1365 masl is the bed level of the dam.

The regulation simulation is not used in WEAP means; the whole volume of reservoir is considered as conservation zone. Consequently, whole volume of water is available for release to the demand site as illustrated in Figure 2-2. Run of river hydropower nodes is presented for proposed diversion weir scheme almost 1km downstream of Thiba dam. Streamflow gauge is placed at the downstream of Thiba River named as 4DA10. Monthly streamflow data from 1978 to 1995 is added using Read from File function in WEAP.

WEAP only introduce the runoff/ infiltration nodes but does not calculate the runoff distribution within the catchment. Infiltration nodes are automatically linked to downstream catchments, which enables the flow of water to the catchment downstream. Inflow to the upstream side of reservoir therefore should be accounted by adding the extra infiltration link. Therefore, the share of runoff is divided into two parts into upstream and downstream of reservoir.

Transmission links delivers the water from river to the demand sites and return flow directs the water that is not consumed to the river. In this study the water withdrawal is downstream the dam, therefore the water withdrawal and return flow links are connected in between the dam and main water withdrawal site.

The environmental flow requirement in the catchment area is also taken from the PDF and official report from Malthe Winje Infrapower AS. It is documented that there must be minimum environmental release of 0.82 m^3/s throughout the year. The requirement is included in WEAP as flow requirement node to the downstream of dam.

The main water demand to this study area is Mwea Irrigation Scheme (MIS). Due to the unavailability of dataset for the water demand CROPWAT is used as a relevant software to calculate the water demand in monthly timestep series. The value for the monthly variation is set up in WEAP for demand area site.

CROPWAT

An FAO recommended software is used find the actual water demand for the irrigable area of Mwea Irrigation Scheme (MIS). This water demand governs the release of water from the Thiba dam which is used by the retrofitted hydropower scheme to generate electricity. Here, the water demand is the Irrigation Water Requirement (IWR) of MIS. The interactive and user-friendly interface of the CROPWAT software helps to find the crop water and subsequently IWR in a location with specified meteorological data, soil characteristics and crop properties. CROPWAT integrates the computational procedures presented in FAO56 for crop evapotranspiration which has been widely used in research and has more than 11,500 citations in various articles (Pereira et al., 2015). All the relevant data require for finding the IWR is given in Table 5-1. And the workflow of CROPWAT with required data is given in Figure 5-4.

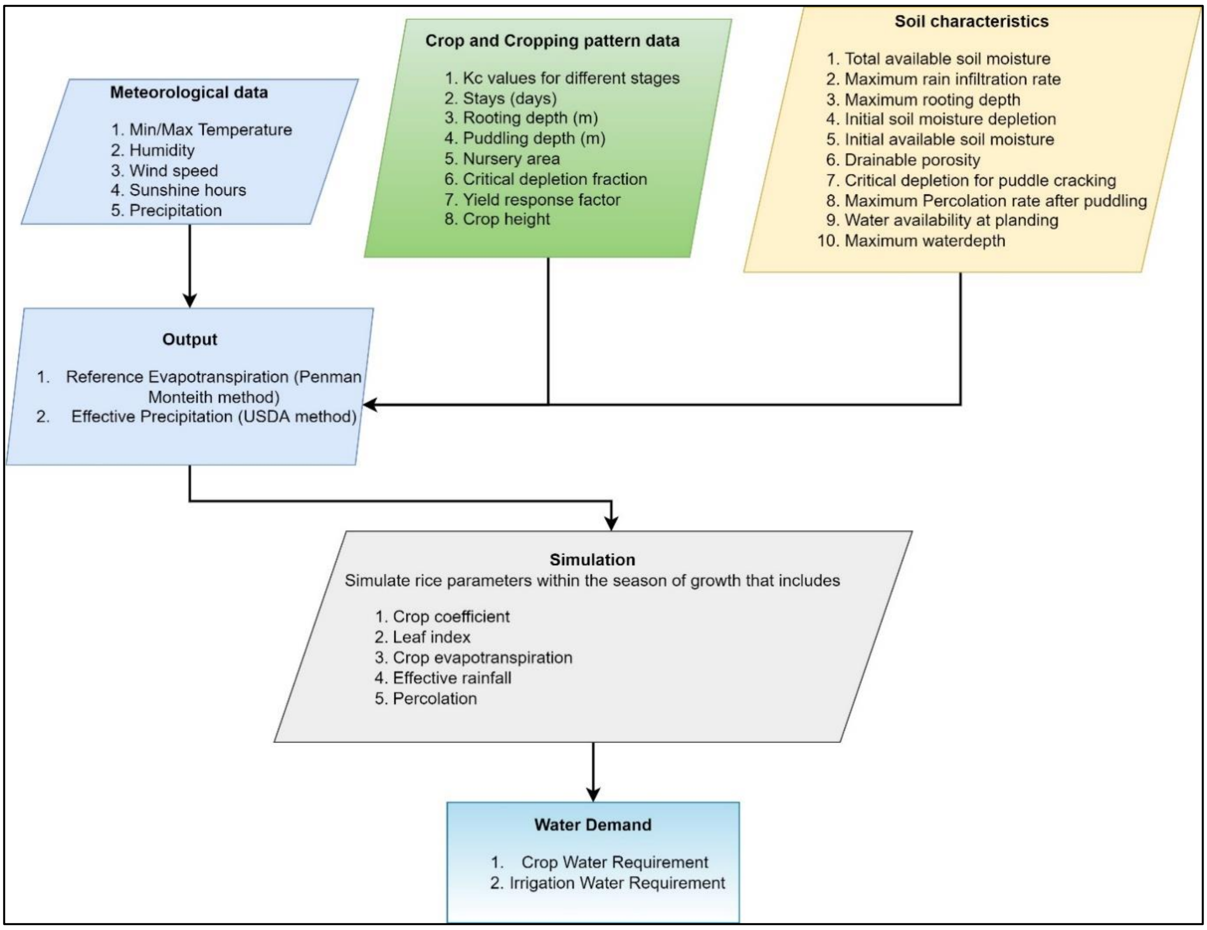

Figure 5-4: Schematic workflow of CROPWAT model.

Table 5-1: Data acquisition for CROPWAT model.

Data	**Sources**
Meteorological Data	Princeton Dataset
Crop Data	(Allen et al., 1998)
Crop Calendar	(Akoko et al., 2020)
Soil Data	(Ochieng, 1981), (NRCCA, 2010), (Brouwer et al., 1985)

The meteorological data are minimum and maximum temperature, air humidity, wind speed, sunshine hours, or interchangeably solar radiation. From these data, the Penman-Monteith method is used to calculate the reference crop evapotranspiration in (mm/month) as shown in Figure A-2.

The average monthly precipitation for each month is entered to find the effective rainfall using USDA soil conservation service method. After this the crop properties in terms of planting date, harvest, K_c values, stage (days), rooting depth (m), pudding depth (m), nursery area (%), critical depletion, yield response factor and crop height (m) are entered as shown in Figure A-3. Most of the required crop data are taken (Allen et al., 1998). Further, sowing and harvesting period is selected as proposed by Samejima et al., (2020), in MIS the sowing pattern is between July and August and harvesting period between December and January.

Finally, characteristics of soil in the catchment is entered. Most of the soil is lateritic clay loams in the Mwea Irrigation Scheme (Ochieng, 1981). So, it is assumed that the clay loam is the dominant soil type in the Mwea irrigation scheme, and the values are entered accordingly in Figure A-4 is self-explanatory which illustrate the soil data required for estimating Irrigation Water Requirement.

5.4.1 Calibration

The unregulated subbasin from period 1978 to 1995 is chosen as calibration catchment. The most important part of the calibration considered is the minimized difference between observed and simulated streamflow. A monthly timestep is followed to minimize the difference between observed and simulated data. The catchment delineation mode on WEAP is used to separate the sub basin within the catchment. Specifically, streamflow gauge (4DA10) situated along the Thiba River is calibrated for this subbasin. Therefore, using climatic dataset from Terrestrial Hydrology Research Group (Princeton University) and land cover data set from the European Space Agency’s Climate Change Initiative Land Cover database (ESA-CCI-LC) simulation of model is performed at each elevation band at spatial resolution of 3s. Using digital elevation data WEAP automatically creates elevation band branches within the catchment.

For calibration the following parameters were chosen:

- Deep water capacity (DWC)
- Runoff resistance factor (RRF)
- Soil water capacity (SWC)
- Deep conductivity (DC)
- Root zone conductivity (RZC)
- Preferred Flow Direction (PFD)
- Z1 and Z2
- Crop coefficient (K_c)

Key assumptions for the above used parameters is created because it is convenient way to perform the calibration as it will automatically assign the variables to parameters for each elevation band.

5.5 Sensitivity Analysis

The purpose of the sensitivity analysis is to identify the key parameters that affect model performance and it plays an important role in model parameterization, calibration, optimization and uncertainty quantification (Song et al., 2015). In calibration process, sensitivity analysis is used to determine the best parameter values for the best model performance (Eryani et al., 2022).The sensitivity analysis for calibrated parameter was carried out by varying each parameter individually keeping the other parameter constant (Silva et al., 2017). The default parameters are used to vary by ± 50% and resulting changes in volume are registered in the total water balance. According to Nearing et al., (1990) the sensitivity parameters S is given in Equation (5-1):

$$\mathrm{S} = \frac{O_2 - O_1 / \bar{O}}{I_2 - I_1 / \bar{I}} \tag{5-1}$$

Where, I_1 and I_2 are the least and greatest values for input parameters and O_1 and O_2 are the output obtained from two input parameters. $\bar{I}$ are the averages from input parameters I_1 and I_2 and $\bar{O}$ is the averages from O_1 and O_2.

5.6 Model Evaluation Technique

The performance and behavior of hydraulic models are evaluated by comparing observations and simulations variables. Hydrologist uses efficiency criteria to compare observed values to model simulations (Waseem et al., 2008). Hydrological model's accuracy depends mainly on the characteristics of discharge hydrographs, including their volume, shape, peak flows, base flows and timings (Ghimire, 2021), Following parameters are chosen to determine the model's performance.

1. Percentage Bias (PBIAS)

PBIAS measures the average of simulated flow volumes to differ from their observed volumes by larger or smaller amounts; the optimum value for PBIAS is 0; positive values signify underestimation, while negative value indicate overestimation (Gupta et al., 1999). It is calculated by Equation (5-2).

$$\text{PBIAS} = \frac{\sum_{t=1}^{n}(Q_{i,obs} - Q_{i,sim})}{\sum_{i=1}^{n} Q_{i,obs}} * 100\% \tag{5-2}$$

Where, $Q_{i,obs}$ is the observed flow [m^3/s] in time step [i] , $Q_{i,sim}$ is the simulated flow in [m^3/s] in timestep [i].

2. Nash Sutcliffe Efficiency (NSE) coefficient

NSE is a measure of the relationship between residual variance and variance of flow where the optimum value is 1, and values should be greater than 0 to indicate the model is least minimally accepted (Gupta et al., 1999). It is calculated by Equation (5-3)

$$\text{NSE} = 1 - \frac{\sum_{t=1}^{n}(Q_{i,obs} - Q_{i,sim})^2}{\sum_{t=1}^{n}(Q_{i,obs} - \bar{Q})^2} \tag{5-3}$$

Where, $Q_{i,obs}$ is the observed flow [m^3/s] in time step [i] , $Q_{i,sim}$ is the simulated flow in [m^3/s] , $\bar{Q}$ is the averaged of observed flow in in [m^3/s] in timestep [i].

3. Coefficient of determination (R^2 value)

$$R^2 = \left[\frac{n(\sum Q_{obs}Q_{sim}) - (\sum Q_{sim})(\sum Q_{obs})}{\sqrt{\left[n\sum {Q_{obs}}^2 - (\sum Q_{obs})^2\right]\left[n\sum {Q_{sim}}^2 - (\sum Q_{sim})^2\right]}}\right]^2 \tag{5-4}$$

Where, n is the number of data, Q_{obs} is the observed and Q_{sim} is the observed discharges. The value close to 1 represents better fit between the observed and simulated discharges and 0 represents bad resemblance of simulated with observed discharges.

5.6.1 Model Performance Rating

Using performance rating for the recommended statistics and project specific information general guidelines for evaluating the model have been developed for each monthly timestep by (Mancosu et al., 2015). The model evaluation criteria by Moriasi et al., (2015) on monthly temporal scale is presented in Table 5-2: .

Table 5-2: Model Evaluation Criteria

Performance Rating	**PBIAS**	**NSE**	**R^2**
Very good	PBIAS < ±5	0.8 > NSE	R^2 >85
Good	±5 ≤ PBIAS < ±10	0.7 < NSE ≤ 0.8	0.75 < NSE ≤ 0.85

Satisfactory	$\pm 10 \leq$ PBIAS $< \pm 15$	$0.5 <$ NSE ≤ 0.7	$0.6 <$ NSE ≤ 0.75
Unsatisfactory	PBIAS $> \pm 15$	NSE ≤ 0.5	$R^2 \leq 0.6$

5.7 Technical Solutions

There are two schemes considered as a technical solution for the implementation of hydropower technology for NPD. The two schemes were selected to assess the differences between their production capacities and economic feasibility. The schematic plan view of the two schemes is shown in Figure 5-5. Schemes on the right side of the river represent diversion weir schemes, and schemes on the left side of the river represent pipe schemes. The two schemes share the same powerhouse location downstream of the dam. The methods for water conveyance for two schemes is however different and will be described in following sections.

Figure 5-5: Proposed layout of two schemes for retrofitting of Thiba Dam. Scheme towards the right side of the river represents the layout of diversion weir scheme and left side of the river represents the layout of pipe scheme.

5.7.1 Detail descriptions of technical solutions

1. Scheme 1: Pipe scheme

This pipe scheme is newly proposed solution in this study, it is assumed that that pipe is inserted from the bottom outlet of the reservoir and connected to the powerhouse. In this scheme the hydraulic head is based on the reservoir elevation to the tailwater elevation for each time step. The maximum gross head equals to the difference in full supply level or maximum regulated water level in reservoir and the tailwater level at 1302 masl. The full supply water level of the reservoir is at 1380m. The utilization of the head due to impounding of the reservoir is the main advantage of this scheme. The design discharge is taken as the guaranteed flow of 7.56m^3/s to avoid the uncertainties in the discharges. The installed capacity is 4.91 MW and the annual generation from this scheme is 43.01 GWh. In this scheme the total length of waterway (pipe) will be 2040 m. Figure 5-6 shows the longitudinal profile illustrating the reservoir water level, proposed waterway, and powerhouse for pipe scheme.

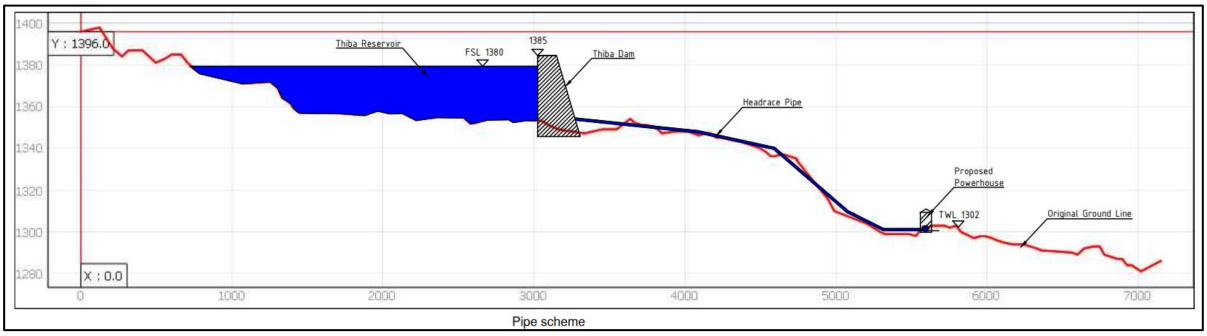

Figure 5-6: Longitudinal profile of pipe scheme.

2. Scheme 2: Diversion Weir scheme

Another scheme is called diversion weir scheme or simply weir scheme. The motivation to propose this scheme is taken from the official report by Malthe Winje Infrapower. A diversion weir is proposed just downstream of the confluence of Thiba and Kiringya river. This scheme cannot take advantage of the head of the dam but is benefited from the increased discharge from the Kiringya river. The water from the bottom outlet of reservoir is diverted by diversion weir at the elevation of 1352.5m and connected to a powerhouse at elevation 1302m. In this scheme hydraulic head is independent to the reservoir water level. The gross head equals to diversion weir level minus tail water level. The design discharge is considered as 9.05m^3/s. The installed capacity is 3.7 MW and the annual generation from this scheme is 33.04 GWh. In this scheme the total length of waterway (pipe) will be 845 m. Figure 5-7 shows the longitudinal profile illustrating the diversion weir level, proposed waterway, and powerhouse for diversion weir.

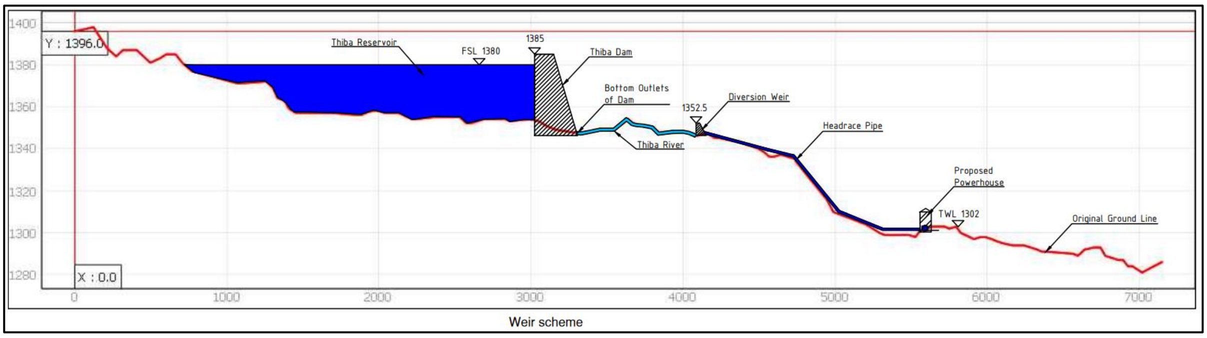

Figure 5-7: Longitudinal profile for diversion weir scheme.

For both the schemes the following assumptions are implemented:

- There are possibilities of construction of new pipelines aiming to increase the hydraulic head for the study. Environmental impact is excluded from this study from the construction of new waterway.
- When considering head losses and turbine efficiency ,85% is considered a normal efficiency for NPD (Hadjerioua et al., 2012).
- Plant factor is considered 100%.

5.8 Economic Analysis

Economic Analysis is a good way to compare best alternatives between different schemes. The cost for the selected schemes for civil works, mechanical equipment, electromechanical equipment's were computed for small- scale hydropower plant (with a generating capacity of up to 10 MW) (SWECO Norge AS, 2012) guidelines given by cost basis curve [Norwegian Water Resources and Energy Directory (NVE), 2012]. The following elements are presented in the cost estimation tabulated in Table 5-3.

Table 5-3: Parameters included in cost estimation for economic analysis with their respective sources.

Parameters	**Source**	**Curve**	**Assumption**
Waterway	NVE < 10MW	Fig 2.5.1	Steel Pipe
Surface Powerhouse	NVE < 10MW	Fig 2.4.1	Head between 10m to 50m
Intake	NVE < 10MW	Fig 2.3.1	Cost Intake > 1MW
Turbines	NVE < 10MW	Fig 3.2.2	500 KW -10000KW
Generator	NVE < 10MW	Fig 4.2.1b	1300KW – 10000KW
Transformer	NVE < 10MW	Fig 4.3.1b	1.4MW – 10MW

Transportation and access road cost are excluded from this study as there is already existing dam which is supposed to be access by roadways. The extra cost while cost calculation like unforeseen cost, administration and planning cost, compensation cost & finance cost are also eliminated from this study.

As electromechanical equipment is sold internationally, prices are assumed to be the same in Kenya and Norway. Tax and subsidies are excluded from the study. The cost price of dam is excluded from this study. As, it is a retrofitting project the other miscellaneous prices for dam retrofitting like maintenance cost is not included in this study. Pipe diameter is optimized to find the most economical diameter for the steel pipe. It is done by calculating the installation cost and present worth of loss of revenue due to head loss throughout the lifetime of project. From this the most economic diameter is considered for the design. The detail optimization is in illustrated in (Figure A-5 , Figure A-6). In both the scheme steel pipe is used for durability and less maintenance throughout the lifetime of period. Francis turbine is selected as per available head and flow discharge as per (IRENA, 2012) for both the schemes.

The two schemes are taken for considerations for retrofitting of the dam. Both schemes are laid out in such a way that the layouts are almost optimized when they are positioned. Some of the alternatives that were included during brainstorming were not taken into further consideration and were rejected from screening and consultations. Proper site selection and hydro scheme design are key challenges. Therefore, proper dimensioning and optimization of the elements of civil structures during engineering design and implementation stages are key factors to reduce construction cost for project (Jager et al., 2011).The most favorable options are named henceforth pipe scheme and diversion weir scheme as shown in Figure 5-6 and Figure 5-7 respectively. To find whether the proposed schemes are economically feasible or not, it is important to find the cost of the retrofitting for both scheme and the benefits that each scheme would generate if it was operated.

As the pipe scheme doesn’t have its separate headworks and rely completely on the bottom outlet of the dam, it requires a transition from the rectangular outlet to the circular pipe to a powerhouse where the electricity is generated can be shown in Figure 5-5. Without the need of construction of the headworks explicitly for the power generation, it is believed to decrease the cost of construction significantly. This scheme generates power which is controlled by the discharge as well as the head in the reservoir. So, it can also take an advantage from the dam height which in turn increases the benefits from the sale of electricity.

The diversion weir scheme doesn't take advantage from the head from the dam but from the additional discharge of the Kiringya river. Both options seem to have their own advantage. So, financial analysis is required to find whether these retrofitted options are feasible or not.

There were some hurdles in conducting financial analysis of the retrofitted scheme. The proposed schemes are to be constructed in Kenya but the rate or the cost curves were not available for Kenya in particular. NVE provides cost basis for hydropower projects (SWECO Norge AS, 2012) less than 10 MW only for Norway, which couldn't be used directly. So, some comparisons had to be made using specific investment cost from Norway and Kenya. Belbo,(2016) suggests that the specific construction cost of hydropower (<10MW) in western and eastern Norway is around 3.79 NOK/KWh and 4.7 NOK/KWh respectively. This rate of 2016 with the inflation of 10% (SWECO Norge AS, 2012) in construction materials in Norway will make the specific construction cost of average of 7.52 NOK/KWh (0.752 USD/KWh) after 6 years in 2022.

In the same way, the exact specific construction cost values in Kenya could not be obtained in a single literature. So, the annual generation and total investment cost of some Kenyan hydropower projects under construction in 2022 had to be taken as a reference. This was obtained from Tembo power (Tembo Power, 2022), as this organization is involved in developing multiple hydropower projects of small scale. The initial cost and annual generation of hydropower developed by Tembo power is shown in Table 5-4.

Table 5-4: Cost and generation of some under construction small and medium scale hydropower projects in Kenya in 2022 (Tembo power).

Hydropower	**Capacity (MW)**	**Location**	**Annual Output (GWh)**	**Investment Cost (Million USD)**	**Specific investment cost (USD/MWh)**
Kaptis	14.7	Yala River, Western Kenya	86.4	51.5	0.596
Buchangu	4.7	Western Kenya	21.66	18	0.831
Kenya 3,4 and 5	47	Western Kenya	224	156	0.696

From Table 5-4, it is evident that the specific construction cost for medium hydropower of around 5MW in Kenya is approximately 0.7 to 0.9 USD/KWh in 2022. This reflects that the specific construction cost of a hydropower project in Kenya and Norway are comparable. So, it is assumed that there is no conversion needed for the cost of hydropower project in Kenya calculated from (NVE) cost curves.

According to MOE Kenya, (2012) the average feed in tariffs for small renewable projects connected to the grid is about 0.1USD/KWh for small scale hydropower project. Other financial considerations made to perform financial analysis of the projects are given in Table 5-5.

Table 5-5: Financial considerations/ assumptions made to evaluate schemes.

Parameters	**Assumption**	**Source**
Total study period	30 years	(IRENA, 2020b)
Project Finance structure	70:30 (debt to equity ratio 70%)	
Energy rates	0.1 USD/KWh	(MOE Kenya, 2012)
Operation and Maintenance cost	2% of the project cost	(IRENA, 2020b)
Loan Term	12 years (payment on quarterly basis)	
Currency conversion rate from NOK to USD ($)	1 USD ($) = 10 NOK	2022 Currency Conversion
Discount Rate	10%	

6 Results

This section is presented by results of the calibration model from WEAP. Then, the results from water demand from CROPWAT. It precedes with the result of power production from two schemes and the economic analysis are provided.

6.1 Model Calibration and evaluation

Calibration:

The main objective of calibration is to find the set of parameters that minimizes the difference in observed outflow with the simulated outflow by changing the input parameters. This process is known as calibration. Calibration of Thiba subbasin was performed using 18-year monthly data from 1978 to 1995 AD. Out of this, first year was regarded as warmup period or reference year.

Firstly, the sensitivity analysis was performed to find the sensitive parameters of the model. This helps us to identify the parameters that largely influence the model performance. After the sensitive parameters have been found out, the model was run by changing one or multiple sensitive parameters. After each run, the model performance metrices mentioned in Section 5.6.1 were checked along with the flow hydrograph Figure 6-2 and its shape, volume, peak, base, and medium flows were also checked. After the sensitive parameters were calibrated, less-sensitive parameters were changed, and fine tuning of the model was done.

The modelled outflow was compared against the observed discharge and the performance of simulation was evaluated using Percentage Bias (PBIAS), coefficient of determination (R^2), Nash-Sutcliffe efficiency (NSE).

Table 6-1 shows the statistical indicators of model performance in calibration from WEAP model. It is observed from Table 6-1 that the NSE, PBIAS and R^2 values in calibration were found to be 0.54, -8.27% and 0.54 respectively. Taking reference from Moriasi et al., (2015), the model is deemed to be “satisfactory” in terms of NSE, “good” in terms of PBIAS and “not satisfactory” in terms of R^2. The model seems to perform satisfactorily in terms of NSE and PBIAS but doesn’t do good when R^2 is considered. However, the flow duration curve and monthly averaged observed and simulated runoff show good resemblance in observed and simulated values.

From the sensitivity analysis, model sensitivity is highest for run off resistance factor and lowest for preferred flow direction. The sensitivity index of parameters on the water balance is illustrated in Figure 6-1.

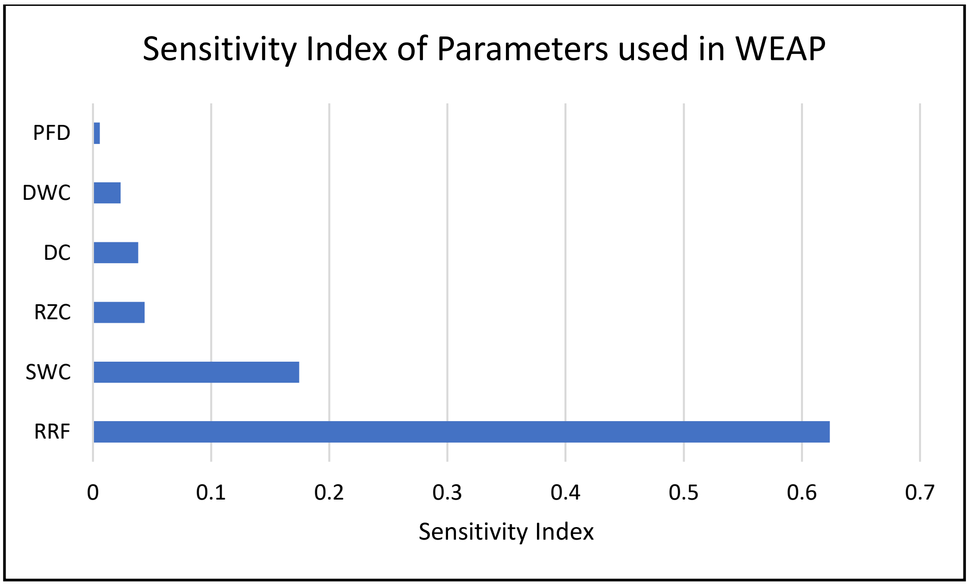

Figure 6-1: Sensitivity Index of Parameters

Table 6-1: Statistical indicators of model performance in calibration.

Station	Period	NSE	R^2	PBIAS
4DA10	1978-1995	0.54	0.54	-8.27%

Nine parameters were used to calibrate the WEAP model. The parameters used in WEAP model for calibration is presented in Table 6-2.

Table 6-2: Calibrated Parameters for WEAP.

Parameters	RRF	SWC	DWC	PFD	DC	RZC	Z1 & Z2	K_c
Unit	[-]	[mm]	[mm]	[-]	[mm/month]	[mm/month]	%	[-]
Value	3.4	923	2054	0.5	457	670	5	1.2

The Figure 6-2 represents the calibration of total monthly flow from 1978 to 1995.

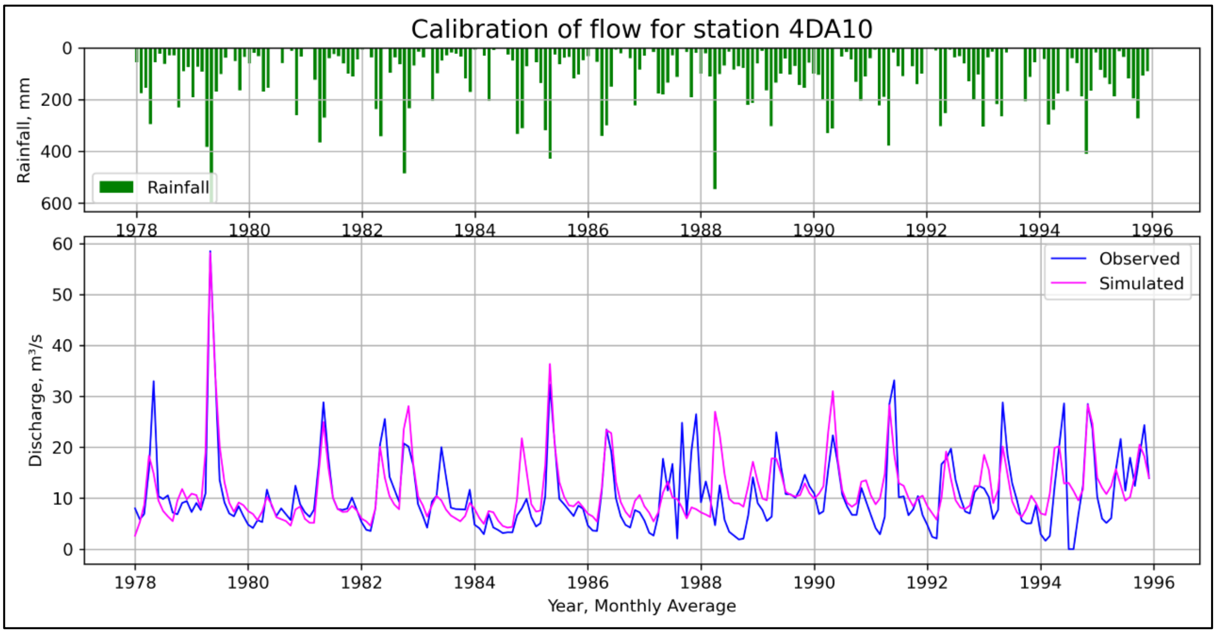

Figure 6-2: Calibration of average monthly flow from year 1978 to 1995 AD. The blue line plot shows the observed hydrograph, magenta line plot shows the simulated hydrograph, and the inverted green bar chart shows the monthly hyetograph.

Duration curve of the observed and simulated flow helps to identify the ability of the model to simulate the high, medium, or low flows. The flow duration curve showing the observed and simulated streamflow is illustrated in Figure 6-3.

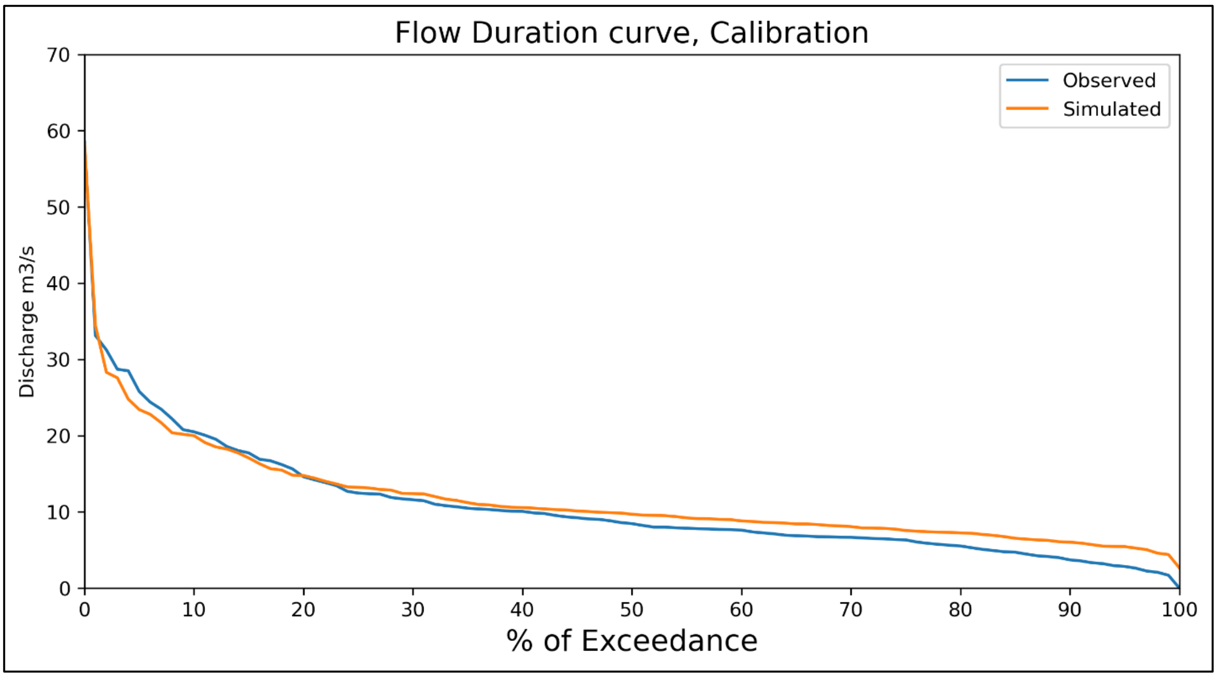

Figure 6-3: Flow duration curve for observed and simulated streamflow.

Likewise, the total monthly averaged observed and simulated flow values are shown in Figure 6-4.

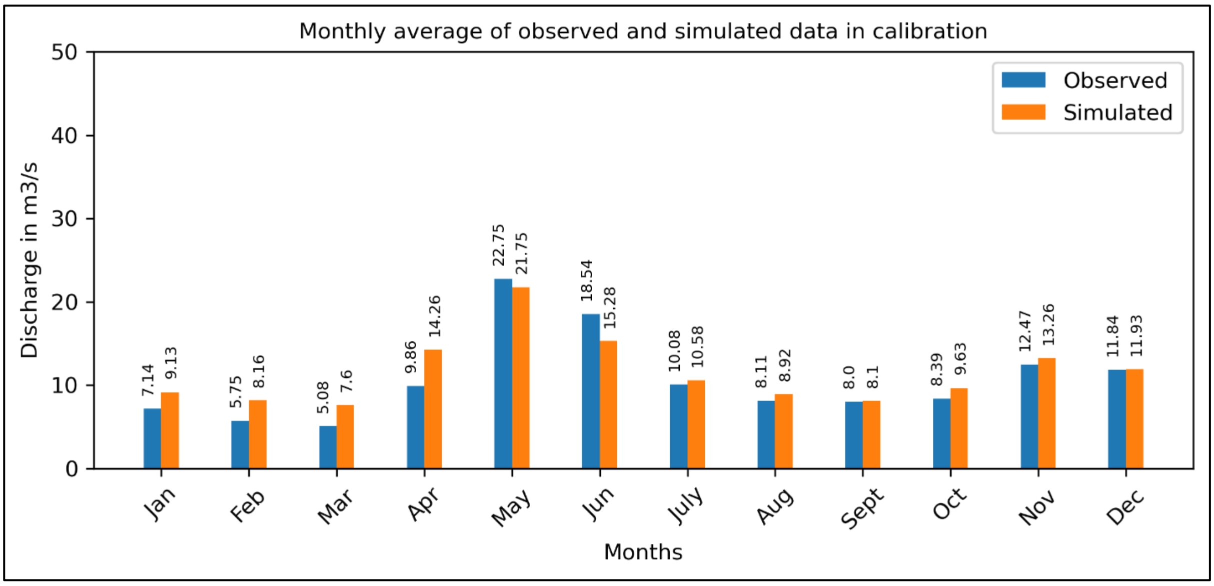

Figure 6-4: Monthly Average of Observed and Simulated Calibration.

It can be seen from the Figure 6-5 that the model has overestimated the volume in total study period of 18 years by 8.27 %. The total cumulative volume of calibration from year 1978 to 1995 is illustrated in Figure 6-5.

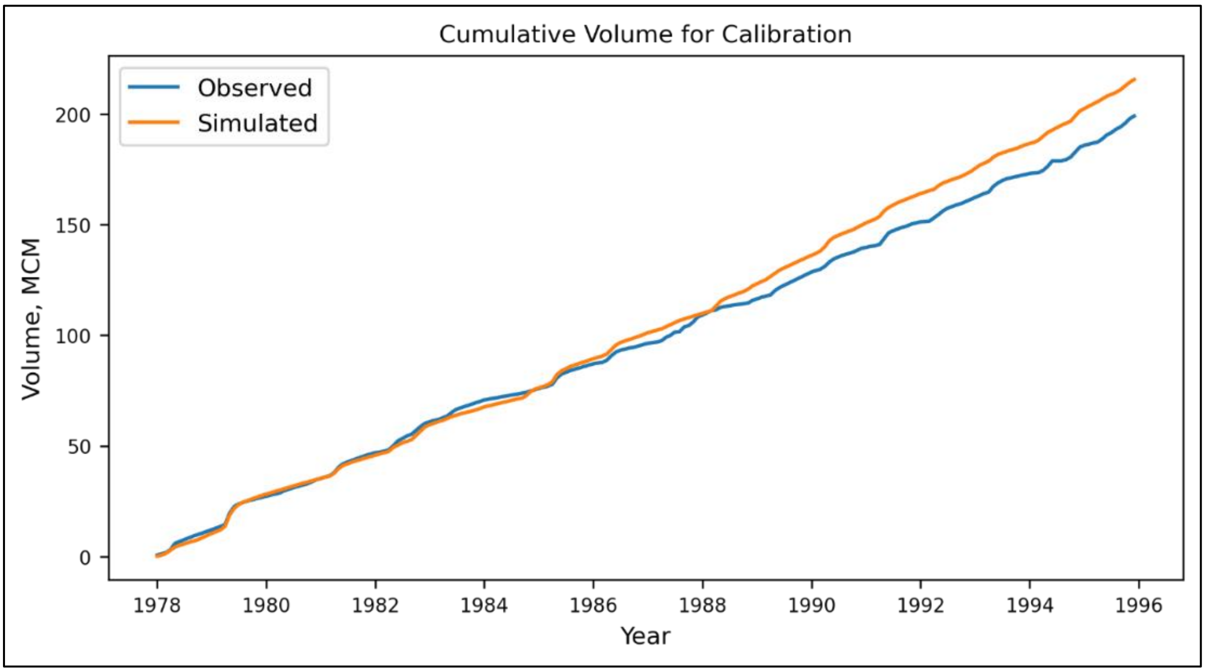

Figure 6-5: Yearly Cumulative Volume for Calibration from year 1978 to 1995 AD.

Figure 6-6 shows the scatter plot of the observed and simulated discharges. This plot helps to know the linearity between the observed and simulated discharges.

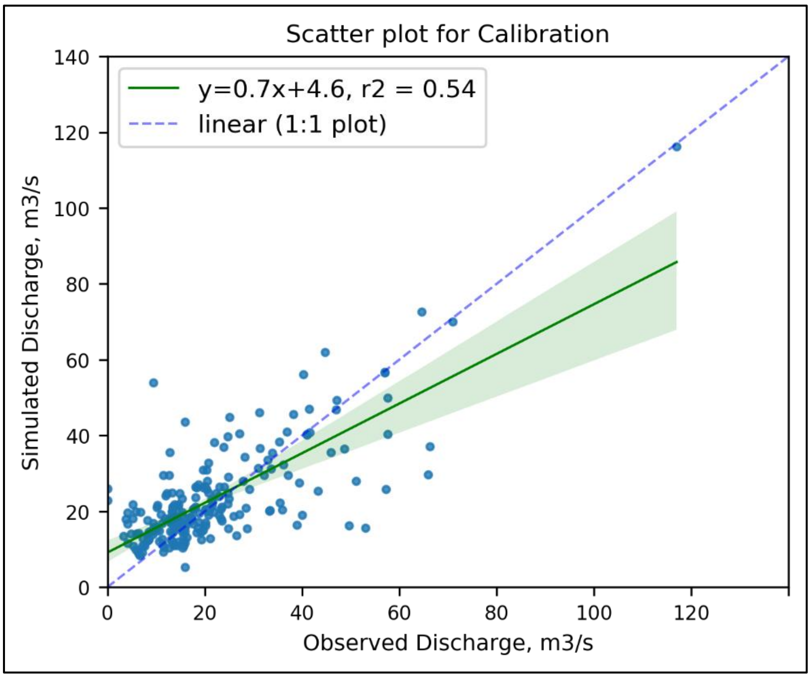

Figure 6-6: Scatter plot for Calibration. Dotted blue is 1:1 line and green line represents linear relationship between observed and simulated discharges.

6.2 Water Demand

Figure 6-7 represents the monthly average rainfall, potential evapotranspiration, and temperature trend in Thiba subbasin from year 1978 to 1995. The irrigation water requirement for each month and per decade is tabulated in Table A-1.

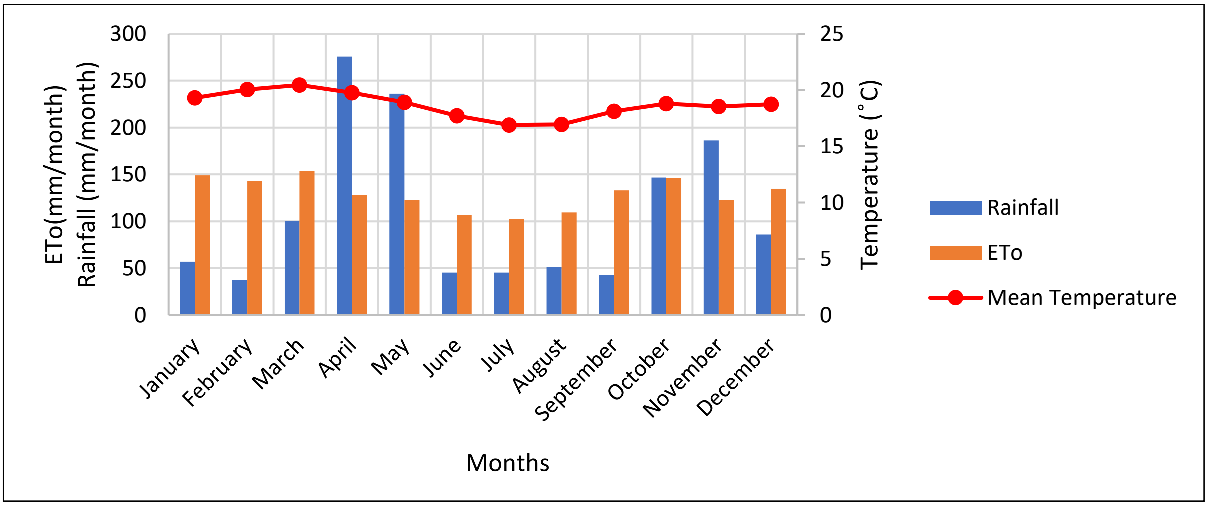

Figure 6-7: Combined Potential Evapotranspiration (Et_o), rainfall and temperature trends in Thiba subbasin from year 1978 to 1995. The bar charts refer to the Et_o and rainfall to be referred to left axis and the temperature represented by line plot to be referred.

Comparison of Available flow and Water Demand shows that the river flow is highest in the month of May with average of 22.17 m^3/s. The higher water deficit was observed in month of

July whereas other months don't face any deficit due to surplus water availability in the river. This result is also in accordance with the values obtained from (Akoko et al., 2020). However, there two months viz. August and September in addition to July where water deficit occurs in (Akoko et al., 2020). This is because the literature has assumed different cropping pattern in the system as compared to this study. The available water flow at Thiba dam site and irrigation water requirement is shown in Figure 6-8.

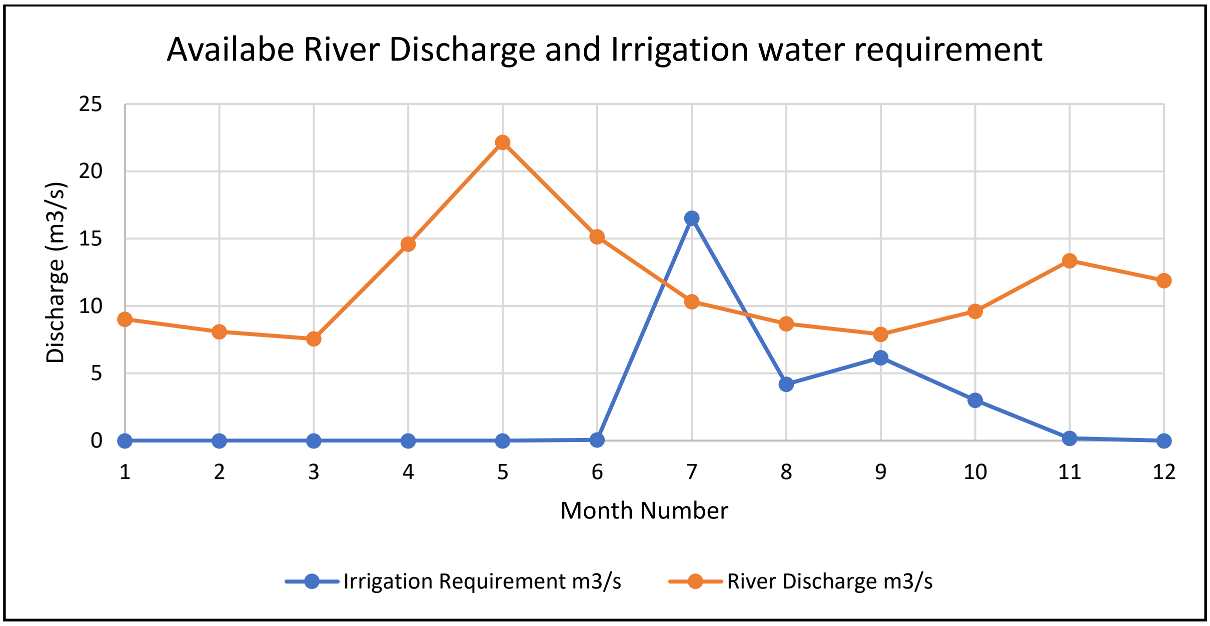

Figure 6-8 : Available Water Discharge and Irrigation Water Requirement.

6.3 Hydropower Generation

The monthly flows at two stations were extracted from simulated WEAP model by assigning nodes at intake of pipe and weir schemes. Flow duration curve for the pipe and weir scheme is presented in Figure 6-9. When comparing the flow at intake of two schemes, discharge of diversion weir scheme is higher than the pipe scheme, because of the confluence of River Kiringya with the River Thiba at the location of diversion weir. The design discharges for pipe and weir schemes have been fixed at 100% of guaranteed flow of 7.56 m^3/s and 9.05 m^3/s The annual average hydropower production is 43.01GWh for pipe scheme and 33.04GWh for diversion weir scheme. The average annual electricity consumption per household is 2501 kWh and the average monthly energy consumption per household is 208 kWh in Kenya (Magambo, 2010). Considering this value and hydropower production from estimated from pipe scheme can serve up to 16,500 and diversion weir scheme can serve up to 12,700 household in Kenya. Salient features of two selected schemes are tabulated in Table 6-3.

Table 6-3: Salient features of selected schemes.

Type of Project	**Pipe Scheme**	**Diversion Weir Scheme**
District	Kiringya , Central Kenya	Kiringya , Central Kenya
River	Thiba River	Thiba River + Kiringya River
Catchment Area at Intake	128 km^2	197 km^2
Design Discharge	7.56 m^3/s	9.05 m^3/s
Full Supply Level	1380 masl	1352 masl
Tail Water Level	1302 masl	1302 masl
Gross Head	78 m	50 m
Rated Net Head	77.88 m	49.97 m
Installed Capacity	4.91 MW	3.7 MW
Annual Energy	43.01 GWh	33.04 GWh

Annual total hydropower generation is presented in Figure 6-10 and monthly average hydropower generation is presented in Figure 6-11. Both of the figures are presented from scenario 2 after the considering reservoir and run of river power plant for diversion weir.

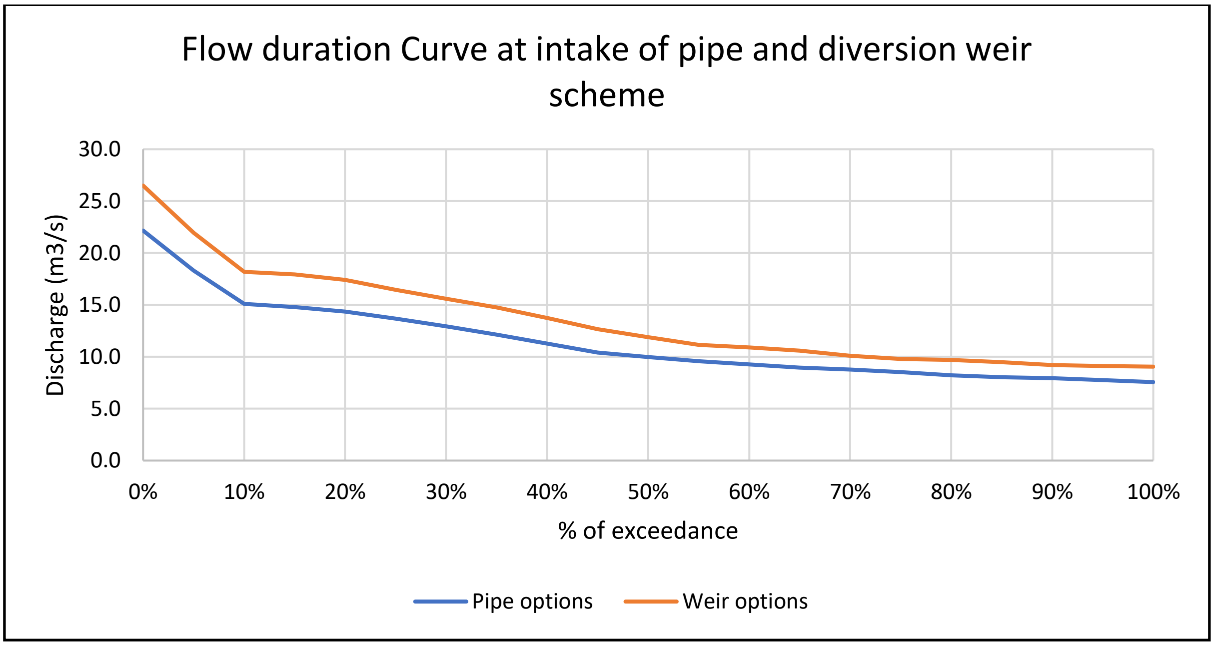

Figure 6-9: Flow duration curve at intake of pipe and diversion weir scheme. The flow duration curve at diversion weir scheme is higher than pipe scheme because of confluence of Kiringya river with Thiba river.

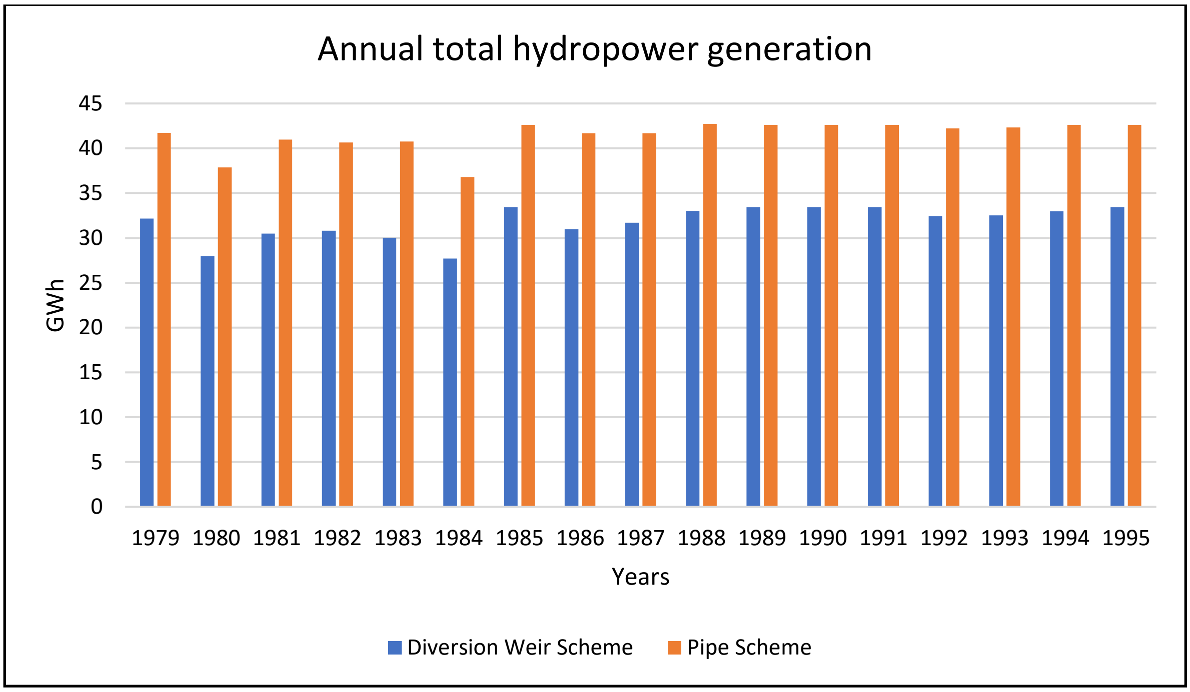

Figure 6-10: Annual total hydropower generation from Scenario 2 when reservoir and diversion weir are introduced to see the possible power production each year of calibration after the fulfilling of water demand to MIS.

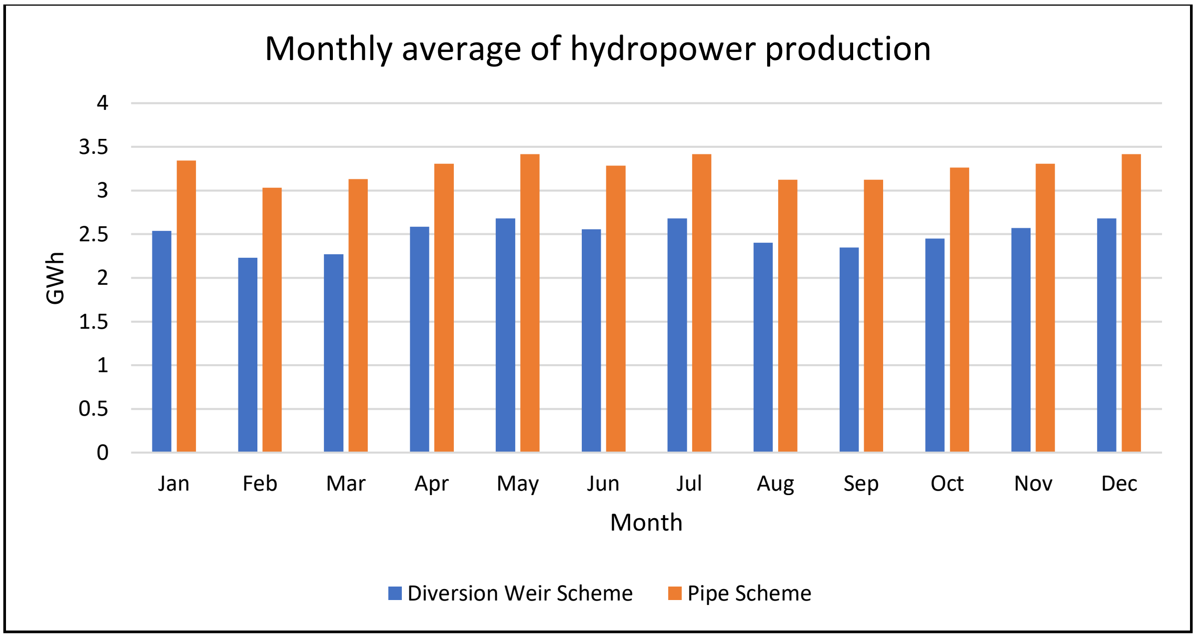

Figure 6-11: Monthly average hydropower production from pipe and diversion weir scheme from Scenario 2 when reservoir and diversion weir are introduced to see the possible power production each month.

6.4 Economic Analysis

From the financial assumptions and considerations mentioned in Section 5.8 the financial indicators for both pipe and weir scheme are evaluated. From the analysis, it is seen that both schemes are financially attractive. However, pipe scheme is better in all the financial indicators. The total cost of construction for pipe and weir scheme are 21.03 Millon USD and 16.97 Million USD respectively. The investment cost breakdown for different components of the retrofitted scheme is given in Table 6-4. The annual revenue generation from pipe and weir scheme is 4.3 million USD and 2.9 million USD for pipe and weir scheme respectively. With almost similar scale of investment, pipe scheme seems attractive with IRR of 17.31% as compared to its counterpart, weir scheme which has IRR of 14.25%. The cash flow diagram is presented in Figure A-7 and Figure A-8 for pipe scheme and diversion weir scheme respectively. All the financial indicators that are considered in financial analysis are shown in Table 6-5.

Table 6-4: Investment cost for different components for pipe and weir scheme.

S.No	**Cost Element**	**Pipe Scheme Cost (Million USD)**	**Weir Scheme Cost (Million USD)**
a.	**Civil Works**		
1	Waterway/Steel Pipe	8.64	4.27
2	Powerhouse		
	Surface Powerhouse	2.26	1.93
3	Intake	0.46	0.48
4	Diversion Weir and Headworks	0.00	1.31
b.	**Mechanical Equipment**		
1	Turbines	6.90	6.63
c.	**Electro Technical Equipment**		
1	Generator	1.13	1.04
2	Transformer	0.28	0.20
3	Switching Station	0.95	0.74
d.	Operation & Maintenance Cost	0.41	0.33
	Total Cost	**21.03**	**16.97**

Table 6-5: Results from economic analysis for pipe and weir scheme.

Parameters	**Unit**	**Pipe Scheme**	**Weir Scheme**
IRR of the Project	%	17.31	14.25
NPV(Nominal)	USD	89,082,036	56,988,967
NPV(Discounted)	USD	13,573,819	6,263,653
Normal Payback Period Years	yrs	5.73	6.89
Discounted Payback Period Years	yrs	6.37	11.69
B/C ratio	[-]	1.65	1.37
LCOE	USD/KWh	0.06	0.07
Specific Cost	USD/KWh	0.48	0.57

7 Discussion

7.1 Calibration of WEAP model

Additional analysis can be performed once the hydrological model is simulated properly. So, it is important to discuss and consider the areas of strengths and limitations of the simulated model which helps to interpret results of further analysis by using the same model.

Analyzing the duration curve of observed and simulated flow (Figure 6-3), it is evident that the model fairly underestimates the higher flows having percentage of exceedance of around 5 to 15% and overestimates in low flows particularly from January to April (Figure 6-4).

When we observe the observed and simulated hydrograph (Figure 6-2), we find that there is highest flow of around 57 m^3/s in 1979 throughout the study period. This flow is caused by series of relatively considerable rainfall in prior month and accounting around 600 mm in month of May in particular. However, this phenomenon is not observed in 1988 when the similar magnitude of rainfall in March,1988 is recorded. It is to be noted that March comes after months having series of low rainfall period when the soil is dry and poses high infiltration capacity. This causes the water to infiltrate into soil instead of appearing in river even there is event of heavy rainfall. So, it is obvious that there should be low flow for the similar magnitude of rainfall for months followed by dry periods. However, the observed discharge is relatively lower and couldn't show up the precipitation signal in year 1988 which is evident from Figure 6-2. Additionally, the discharge in the river has reached near zero in year 1994 which is impractical for a perennial river. Nonetheless, the model could capture the precipitation signal of March 1988 which simulated around 27 m^3/s of flow in river. This signifies that the model is working satisfactorily in response to the input precipitation.

In total, the uncertainty involved in observation, low spatial and temporal resolution of meteorological data, lack of heuristic method of parameterization etc. could also have resulted in underperformance of simulated model exhibited by statistical metrices, especially R^2.

Referring to Figure 6-1 in sensitivity analysis of input parameters, run off resistance factor (RRF) is the most sensitive and preferred flow direction is the least sensitive parameters. The increasing RRF allows less water to become streamflow as it percolates into the soil immediately because of resistance to runoff. While it is in the soil, more evaporation is possible enabling higher water loss from system and loss of water that contribute to streamflow. The preferred flow direction is least sensitive among the analyzed parameters. This indicates the direction of flow from root zone to the lower bucket is not so sensitive as RRF is governing the total process.

7.2 Limitations and Uncertainties in WEAP model

There were several limitations while performing the simulation in WEAP model. The most important part of any model is good input data which helps to obtain good outputs. The data from the ground based meteorological stations were not available during the course of this study. So, a gridded data available on the internet had to be used which is generally of low resolution. The gridded meteorological data were obtained from Princeton dataset having spatial resolution of 28 km x 28 km. It is to be noted that the catchment area where the hydrological model was setup was mere 318 km^2 (Figure 3-1) covered only by two grided raster dataset at most. Although the hydrological model is semi-distributed, the input data is lumped within the basin. The grided satellite dataset should be corrected for its biasness which is only possible when ground based data is available (Dangol et al., 2022). This causes uncertainty in meteorological data which is an important input for any hydrological model. The low spatial resolution of climate data from Princeton dataset is therefore not considered the most suitable dataset as it covers limited resolution for this study. While calibrating the WEAP model, some of the parameters went beyond their physical possible ranges due to various uncertainty while maintaining the model performance metrices mentioned in Section 5.6. Therefore, some of the assigned parameters in WEAP model lies at the boundary of their physical ranges at cost of lower NSE and R^2.

Even though WEAP allows for a daily resolution, the calculation is done taking the monthly dataset. The monthly timestep is followed although there was availability of daily dataset to fit the precipitation dataset. Due to the limitations of climatic dataset from real field observations, the climatic data was solely based on Terrestrial Hydrology Research group (Princeton University). Although, it gives the daily set of data for precipitation, it averages the precipitation data for a whole month resulting the same value of precipitation for each day in one month. It is considered that precipitation varies for the different day which can directly affect the calibration of model when calibrating from the same precipitation data throughout the month.

After reviewing the report from Malthe Winje, due to insufficient information about the dam characteristics some assumptions have been made for the buffer coefficient that needed to be introduced on the reservoir data in WEAP model. As illustrated in Figure 2-2, the whole reservoir is considered as conservation zone. The flood control zone, conservation zone, buffer zone and inactive zone (dead zone) are not included in the model. The effect of this will cause the reservoir completely empty to fulfill the water demand. Decreased availability of water would result from implementing dead volumes, whereas implementing water volumes would result in less water spilling from spillways. For each of the different cases, it is necessary to examine how merging different reservoir zones into one impact the hydropower potential. Assuming the dam to work only in conservation zone without applying constraints in water release will not assure reliability of fulfilling the water demand by the dam (Hashimoto et al., 1982).

Internal water transfers are excluded from this study due to insufficient water demand data. If there is a big water demand site for e.g.: group of households upstream of where the pipe scheme and diversion weir scheme is suggested then, it might influence on the power production. To avoid limitations after the study and to know the proper facts of catchment area beforehand such information about internal transfers needed to be available before the investigation of the study. There might be monthly variation of environmental discharge downstream side of the dam to maintain the ecology of the river system, but fixed environmental discharge is also only provided in the official PDFs. It would be better to input the monthly variation of environmental discharge in the WEAP model as it also allows to input the monthly variation in model.

7.3 Method of interpreting hydropower generation

Hydrological models using WEAP were chosen for this study due to their ability to represent a wide range of water uses. There are several types of water demands that can be addressed by WEAP, including hydropower generation, environmental flow requirements, reservoir refilling, and other water demands accounted for as individual withdrawal nodes. This study is especially useful for assigning priorities to water uses which can offer prioritize lower hydropower generation in comparison to other important water demand. Care must be taken not to compromise the water release to the demand that the dam had intended to be built for (Bakken et al., 2016). This study has also considered the same water demand for irrigation to MIS while producing the hydropower. However, this might not be the optimized management policy for a water reservoir where a conflicting demand is introduced. This needs more complex calculations and WEAP alone is not capable of optimizing all those operating policy.

There are certain weaknesses in WEAP to account for hydropower calculations. There is no allowance for the use of multiple turbine units in one hydropower plant due to the hydropower calculations not accounting for the varying turbine efficiency with changing discharges.

Furthermore, when using monthly timesteps means the head and the available flow at the beginning of the month are calculated as constants. They are not considered for possible time-specific time variation changes in water elevation.

The study done by Fjøsne, (2020) the hydropower calculation was based on a monthly timestep, but the gross head was fixed and not affected by the upstream water level. However, in this study, the power production by the pipe scheme depends upon the upstream head in the reservoir at each monthly time step which is more realistic approach than that adopted by Fjøsne, (2020).

7.4 CROPWAT

It is important to find the actual Irrigation Water Requirement (IWR) to fix the water demand for the reservoir since water demand by the irrigable land governs the release of water from the reservoir which then could be used by the hydropower. According to Panigrahi et al., (1992) irrigation water requirement plays an important role as they help to predict the amount of water needed for different crops in the area. Therefore, it is important to know the irrigation water requirement in this study. The water demand calculated by using FAO recommended software, CROPWAT in terms of irrigation water requirement is found to be 567.5 mm. Considering 13,745 hectares of irrigable land in Mwea Irrigation Scheme (MIS), the total average annual irrigation requirement is around 5675 m^3/ha. The considered irrigable land area is referred from Akoko et al., (2020). The calculated values are comparable to the values adopted in Droggers et al., (2011). According to Droggers et al.,(2011) the total irrigated land area for Thiba subabsin is 10,643 ha and the water requirement were set as 500 mm per year equivalent to 5000m^3/ha/year which is close to our calculations from CROPWAT.

However, it is worth mentioning that the climatic data from the low-resolution meteorological dataset and generalization of one soil type over the irrigable land area (i.e., Clay loam) and its characteristics might have increased the uncertainty in the calculations. Similarly, only one type of crop (monsoon rice) has been assumed while calculating the IWR based on Gikonyo et al., (2018) which might not be practical where other crops are also grown.

Comparing the available river discharge and IWR Figure 6-8, there is deficit of water in month of July when the IWR is at maximum. Here, the reservoir will supply the deficit water from its storage and there won't be any decrease in hydropower production.

The IWR seems reasonable and is used further in finding the regulated release from the dam which is considered as discharge for power production.

7.5 Economic Analysis

For over the last decade the increase in the costs of non-renewable energy sources and a increasing environmental impact have followed to the research institutions to investigate more about the renewable energy (Butera & Balestra, 2015). 56% of the hydropower projects already has lower LCOE than the cheapest new fossil fuel- fires cost option in 2020 (IRENA, 2020). LCOE of this retrofitting scheme can be compared to generic hydropower constructed in pristine rivers. Based on the LCOE presented by IRENA, (2020a), the LCOE of new small scale hydropower generation is around 0.1 USD/KWh. The values compared to the LCOE calculated in this study for retrofitted scheme; pipe scheme (0.06 USD/KWh) and diversion weir scheme (0.07 USD/KWh) which are relatively cheaper than values for new hydropower projects. This also demonstrates the relevance of retrofitting of dam.

Out of the two investigated schemes, the pipe scheme is more attractive because of two reasons. One, it can take an advantage of increased head of dam in producing the electricity, two, it doesn't require a separate headworks to divert water from the river. Therefore, the increased discharge by trading off head and need of construction of headworks explicitly for diversion of flow for weir scheme couldn't outweigh the benefits from pipe scheme.

Referring to the official report presented by Malthe Winje, the power generation is 3.5MW with IRR of 11.95%, 15 years of payback period, LCOE of 0.07 USD/KWh, LCOE in present study the pipe scheme generates 4.91MW of installed capacity with IRR of 17.31% , and payback period of 7 years and LCOE of 0.06 USD/KWh. It can be seen from the result that the financial parameters are good in terms of proposed pipe scheme then the scheme presented in the official report of Malthe Winje.

There as some assumptions made to convert the NVE cost curves to specific project location in Kenya. These assumptions are crude because no relevant literatures could be found out to calculate the actual cost of construction and rate of construction materials of hydropower projects in Kenya. With the help of Belbo, (2016); Tembo Power, (2022), the cost of construction in Kenya and Norway seems comparable based on specific construction cost. However, if actual costs were considered, the results would be different. Nevertheless, the retrofitted schemes seem promising financially.

In this study, some of the issues are overlooked due to unavailability of data. The actual geology of the site has not been considered. This will affect the length and alignment of waterway (headrace pipe) and the location of powerhouse. Similarly, knowledge of sediment is essential for planning of hydropower and other water resource projects (Ghimire et al., 2021) which hasn't been incorporated in the study of Thiba Dam.

Electricity prices will fluctuate in the future due to many factors. As a result of this, they are difficult to predict and, in some cases, seen as being the greatest risk in power projects (Noothout et al., 2016). There were no official forecasts found so a fixed firm rate from (MOE Kenya, 2012) is chosen equal to the average price during the operation period which might be uncertain for the cost benefit analysis and there can be certain fluctuations of marginal benefit in the near future. Additionally, new retrofitting technology have not been introduced during the financial analysis of this project such as modification on the existing dams. This cost can increase in the construction cost of project.

This study shows that the retrofitting of existing dam is feasible in financial perspective. The study of retrofitting for NPDs in Spain Fjøsne, (2020) has also concluded that retrofitting option is feasible for most of the dams investigated. However, a general recommendation could not be made for all the NPD around the world and needs to be checked from different approaches and assumptions. Screening and scoping of these NPD for retrofitting can be done using hydrological models and different retrofitting options can be envisaged to find the most optimum one.

7.6 Topics for future studies in retrofitting

The present study of retrofitting of Thiba dam shows a huge possibility of retrofitting demonstrated by economic viability of both considered technical solutions. Using a basin wise scale model, a catalog should be developed to identify possibilities of retrofitting on a boarder perspective. The model should be based on proper input data which would include ground based meteorological stations for better calibration and validation. The well simulated model will then help deducing correct water availability data in a catchment. This model could then be integrated with proper engineering and technical judgement involving economics of retrofitting. This helps in prioritizing the more feasible projects that can be retrofitted with minimum cost and maximum benefits.

There is no standard procedure in retrofitting of the NPD. The guidelines can be developed based on technical and economic criteria. The criteria should also include additional intangible and indirect benefits of NPD like safeguards of environment, CO_2 certificates, access to electricity in rural community and its impacts in national economy, improvement in local livelihood etc. The enhancement in standardization in retrofitting helps the developers to focus on the major issues in retrofitting of the dam and prioritize the problems to be address. Hansen et al., (2021) mentioned the common information risk related to dam safety, human safety, electricity transmission, economic and environmental analysis before the development must be studied carefully by the stakeholders involved in NPD development. So, standardization helps in identifying the risks which assist in tackling those risks beforehand.

Prioritizing water demand allocation can be one of the main challenges for NPD. Further question will be if water demand is maintained or fluctuated then main challenge will be maintaining the economic incentive to projects with higher marginal revenues. Therefore, water resources must be weighed and considered thoroughly in terms of the diverse demands of various sectors beforehand and sound economic concept needed to be developed for maintaining the accountability of the project. So, future studies in retrofitting should move in the direction that addresses aforementioned issues.

8 Conclusion

According to this study the total annual average production from pipe scheme is found to be 43.01 GWh with a corresponding installed capacity of 4.91MW. Similarly, the total annual average production from diversion weir scheme is found to be 33.04 GWh with a corresponding capacity of 3.7 MW. The IRR, LCOE and specific cost of the pipe scheme is 17.31%, 0.067 USD/ KWh and 0.489 USD/ KWh respectively and IRR, LCOE, specific cost of diversion weir scheme is 14.25%, 0.07USD/KWh and 0.5 USD/KWh. Although, both retrofitting schemes are financially viable, pipe scheme is more attractive in economic and production point of view.

The major firmness of this study is considering the existing water use on the basin where the evaluation of retrofitting has been done. Verifiable economic activity of the project can attract investors and intensify the potential of retrofitting project. This study can also provide motivation to the developers of NPD equip their dam with a power generating facility which would give them additional benefits with low-cost investments. Although having some uncertainties in the hydrological model, it can be helpful in assessing additional issues like climate change, change in hydropower production due to increase in water demand, prioritizing one demand in cost of other etc.

Hence, from the result of this study, the retrofitting of NPD could be a better option than developing a new hydropower project in new location. Similarly, retrofitting provides new revenues for utilization of available resources (dam) in an optimum way.

The following topics are recommended for further studies for retrofitting of dam.

- It is common from past assessments that only development potential of retrofitting is focus exclusively but technology development, utilities, and large-scale water systems, or grid management are also the possible areas to be looked at.
- Further studies could focus on possible legislative or institutional hurdles in retrofitting of the dam and its possible solutions.
- Assigning monetary value to indirect and intangible benefits of retrofitting in economic point of view.

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A APPENDIX

A.1 Description of Master's Thesis

NTNU
Norwegian University of
Science and Technology

Faculty of Engineering

Department of Civil and
Environmental Engineering

M.Sc. Thesis in

Water Resources Modelling and Engineering

Candidate: Kristina Shrestha

Title: Investigating the technical concept of retrofitting of non-hydro reservoir and dam.

1. **BACKGROUND**

A large number of the world's large dams and reservoirs are built for other types of use than hydropower production. According to the statistics derived from the International Commission of Large Dams (ICOLD), the purpose of single purpose dams in Asia and Africa is dominated by irrigation, and only 14% and 7%, respectively, are used for hydropower. Retrofitting describes in this context the addition or expansion of an existing dam not used for hydropower with hydroelectric power generation capabilities. The introduction of hydropower technology in these dams will neither introduce any additional environmental impacts.

According to previous studies at NTNU it is a technically possible and economically feasible to re-build (retrofit) some of these non-hydropower dams for the purpose of producing hydropower electricity, without affecting the existing purpose of the dams. As retrofitting of non-hydropower dams and reservoirs is not a well-proven and detailed investigated technical concept, in-depth assessment of technical solutions is needed, as well as the associated costs related to different solutions. This thesis will contribute to a better understanding on how retrofitting technically can be implemented in the real world, reducing technical and economic risks.

2. MAIN QUESTIONS FOR THE THESIS

Key questions to be addressed in the thesis are.

1. Identify a non-hydropower dam with a potential for retrofitting, where technical information about the dam and dam site can be found.
2. Assess the overall hydrological potential for retrofitting with use of modelling tool WEAP for selected reservoir /dam.
3. Assess the possible technical solution for the installation / implementation of hydropower technology in the selected/studied sites (dam /reservoir).
4. Estimate the economic costs of the investigated technical solution.

3. SUPERVISION, DATA, AND INFORMATION INPUT

Professor Tor Haakon Bakken will be the main supervisor of the thesis work, with Professor Leif Lia as co-supervisor on aspects related to the investigation of technical solutions for retrofitting. Discussion with and input from colleagues and other research or engineering staff at NTNU, power companies or consultants are recommended, if considered relevant. Significant inputs from others shall, however, be referenced in a convenient manner.

The research and engineering work carried out by the candidate in connection with this thesis shall remain within an educational context. The candidate and the supervisors are therefore free to introduce assumptions and limitations, which may be considered unrealistic or inappropriate in contract research or a professional engineering context.

4. REPORT FORMAT AND REFERENCE STATEMENT

The report shall be typed by a standard word processor and figures, tables, photos etc. shall be of good report quality, following the NTNU style. The report shall include a summary, a table of content, lists of figures and tables, a list of literature and other relevant references. All figures, maps and other included graphical elements shall have a legend, have axis clearly labelled and generally be of good quality.

The report shall have a professional structure and aimed at professional senior engineers and decision makers as the main target group, alternatively written as a scientific article. The decision regarding report or scientific article shall be agreed upon with the supervisor. The thesis shall include a signed statement where the candidate states that the presented work is his/her own and that significant outside input is identified.

This text shall be included in the report submitted. Data that is collected during the work with the thesis, as well as results and models setups, shall be documented and submitted in electronic format together with the thesis.

The thesis shall be submitted no later than 28^{th} of June 2022.

Trondheim 15^{th} of January 2022

Tor Haakon Bakken

____

Tor Haakon Bakken, Professor

A.2 Schematic view from WEAP

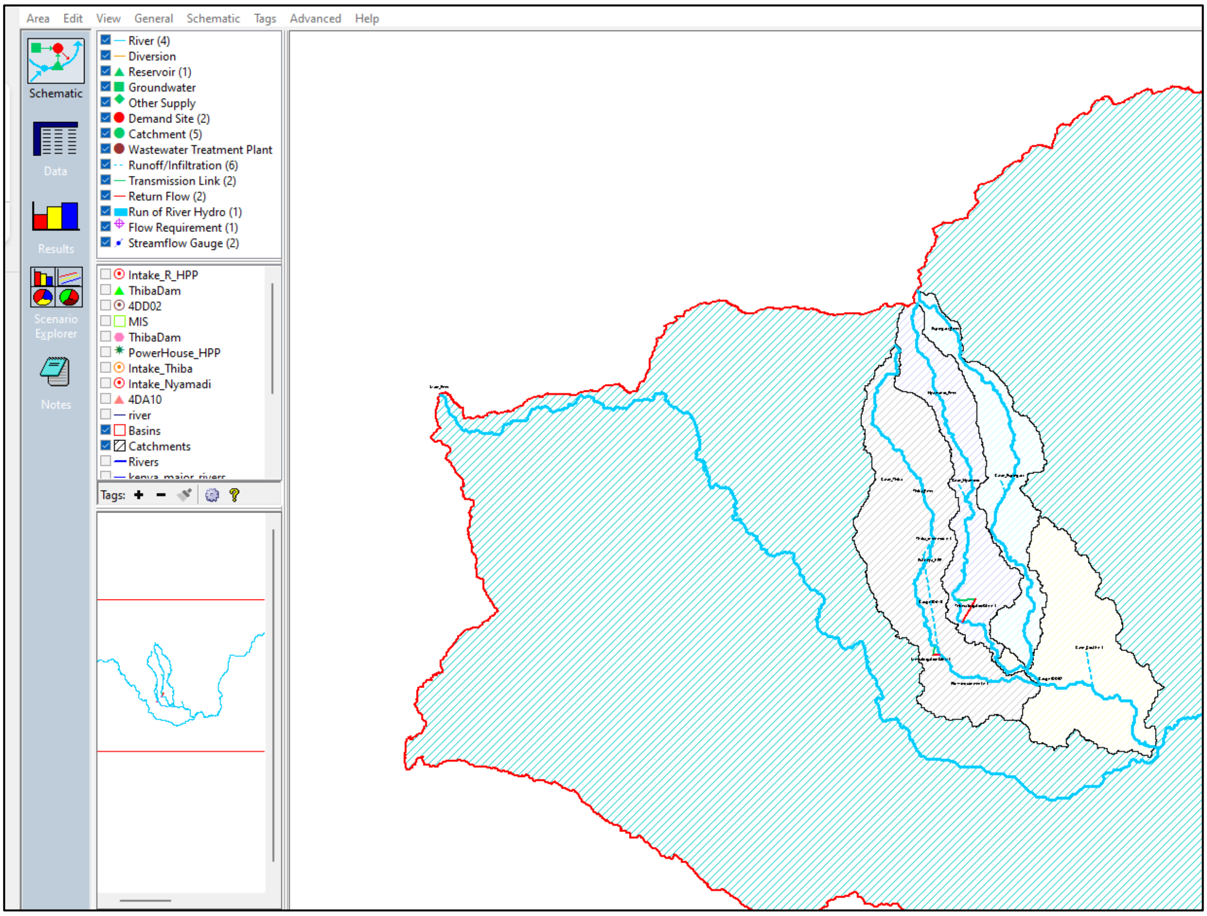
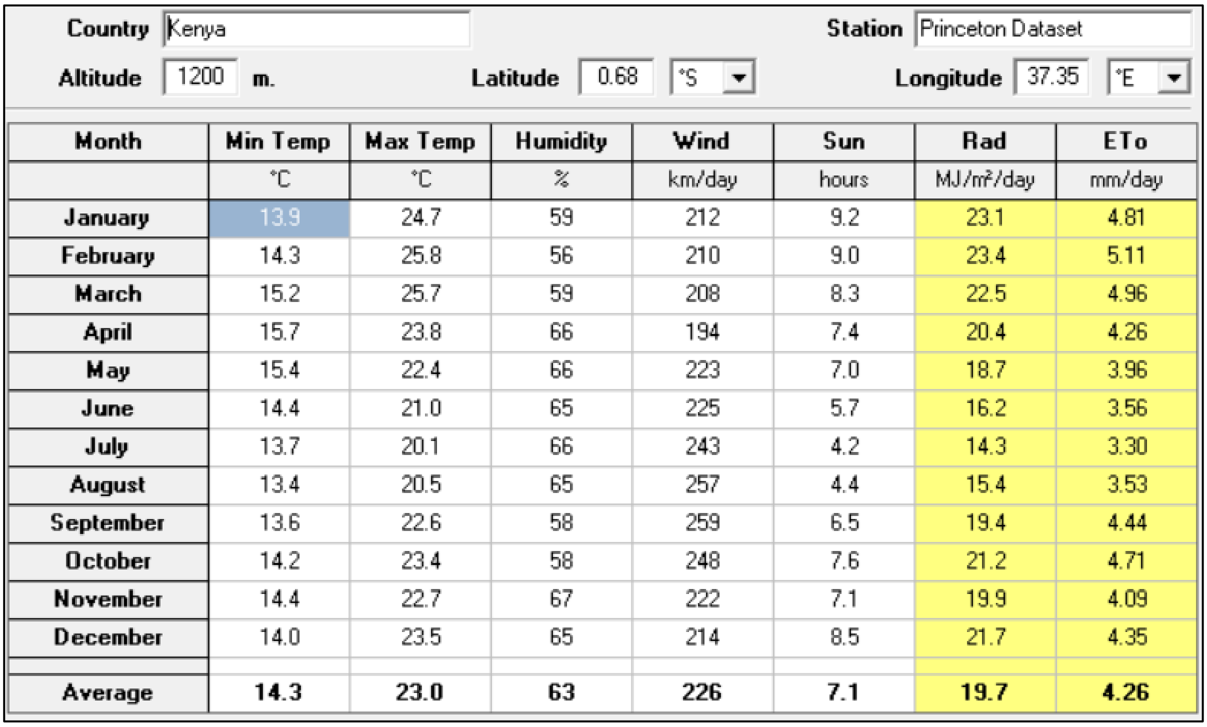


Figure A-1: WEAP setup showing the schematic view of study area.

A.3 CROPWAT



Country: Kenya — Station: Princeton Dataset

Altitude: 1200 m. — Latitude: 0.68 °S — Longitude: 37.35 °E

Month	Min Temp	Max Temp	Humidity	Wind	Sun	Rad	ETo
	°C	°C	%	km/day	hours	MJ/m²/day	mm/day
January	13.9	24.7	59	212	9.2	23.1	4.81
February	14.3	25.8	56	210	9.0	23.4	5.11
March	15.2	25.7	59	208	8.3	22.5	4.96
April	15.7	23.8	66	194	7.4	20.4	4.26
May	15.4	22.4	66	223	7.0	18.7	3.96
June	14.4	21.0	65	225	5.7	16.2	3.56
July	13.7	20.1	66	243	4.2	14.3	3.30
August	13.4	20.5	65	257	4.4	15.4	3.53
September	13.6	22.6	58	259	6.5	19.4	4.44
October	14.2	23.4	58	248	7.6	21.2	4.71
November	14.4	22.7	67	222	7.1	19.9	4.09
December	14.0	23.5	65	214	8.5	21.7	4.35
Average	**14.3**	**23.0**	**63**	**226**	**7.1**	**19.7**	**4.26**

Figure A-2: Average monthly meteorological data input to find monthly ET_o.The meteorological data is derived from Princeton dataset.

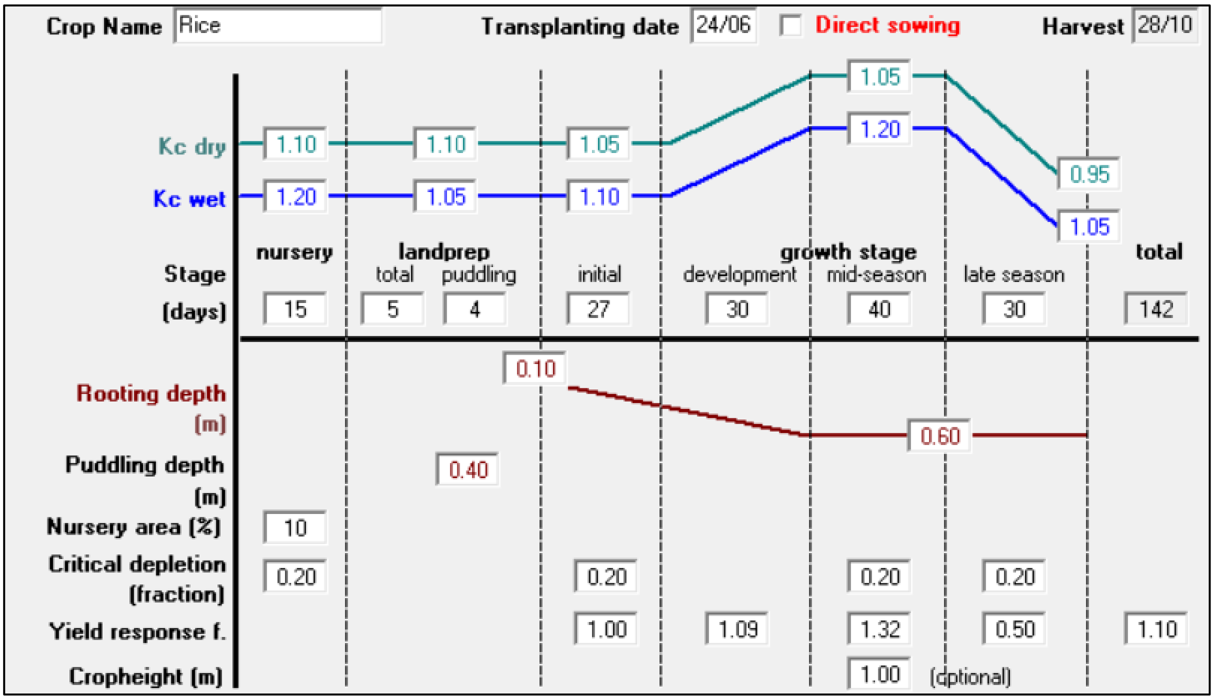


Figure A-3: Crop properties of rice.

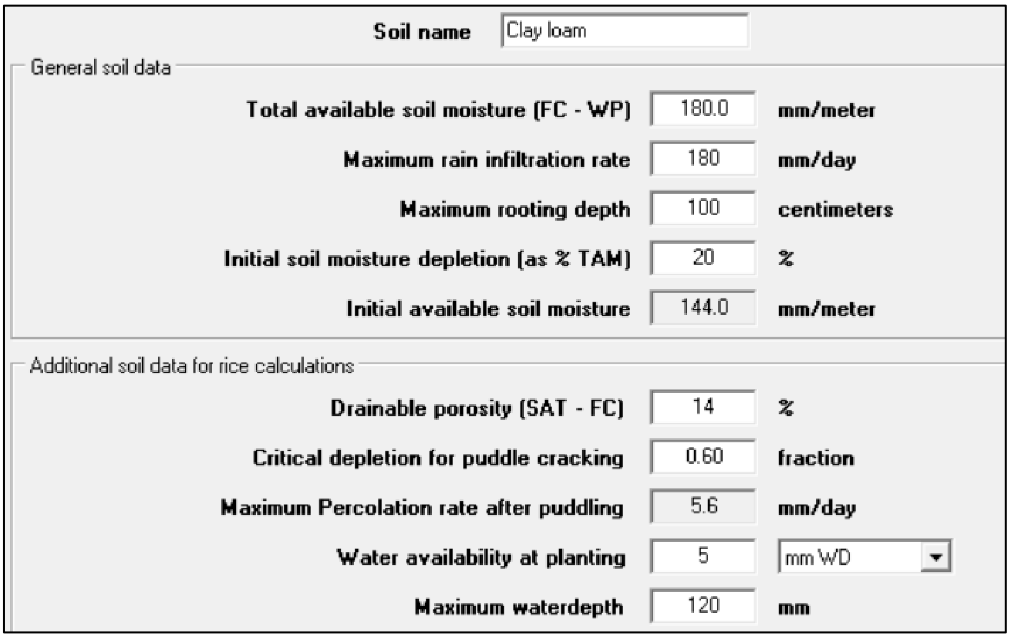


Figure A-4: Soil characteristics input in CROPWAT.

Table A-1: Crop and Irrigation water requirement in each month per decade (10 days)

Month	**Decade**	**Stage**	**Kc**	**Etc**	**Etc**	**Eff rain**	**Irr. Req.**	**Total m3**	**Total month (m3)**
			coeff	mm/day	mm/dec	mm/dec	mm/dec	m3	m^3
Jun	3	Nurs	1.2	0.42	0.4	1.1	0.4	54984	54984
Jul	1	Nurs/LPr	1.19	0.73	7.3	13.9	74.9	10295754	
Jul	2	Init	1.09	3.59	35.9	13.8	210.4	28921584	
Jul	3	Init	1.1	3.72	40.9	14.4	26.5	3642690	42860028
Aug	1	Init	1.1	3.8	38	15.4	22.6	3106596	
Aug	2	Deve	1.12	3.95	39.5	16.1	23.4	3216564	
Aug	3	Deve	1.15	4.41	48.5	15.2	33.3	4577418	10900578
Sep	1	Mid	1.18	4.89	48.9	11.5	37.4	5141004	
Sep	2	Mid	1.19	5.3	53	9.5	43.5	5979510	
Sep	3	Mid	1.19	5.41	54.1	18.8	35.3	4852338	15972852
Oct	1	Mid	1.19	5.52	55.2	31	24.1	3312786	
Oct	2	Late	1.19	5.62	56.2	39.9	16.3	2240598	
Oct	3	Late	1.16	5.21	57.3	41.1	16.1	2213106	7766490
Nov	1	Late	1.1	4.72	47.2	43.9	3.3	453618	
Nov	2	Late	1.05	4.3	34.4	37.7	0	0	453618

A.4 Optimization of Waterway

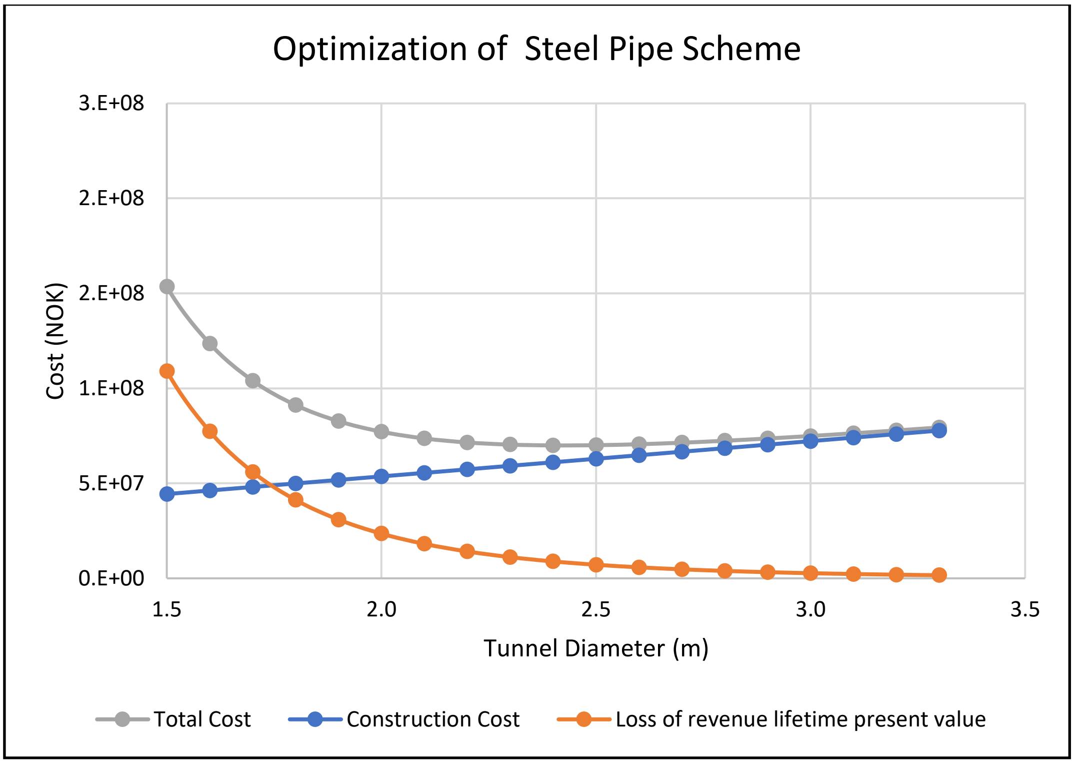


Figure A-5: Graphical optimization of headrace pipe of pipe scheme.

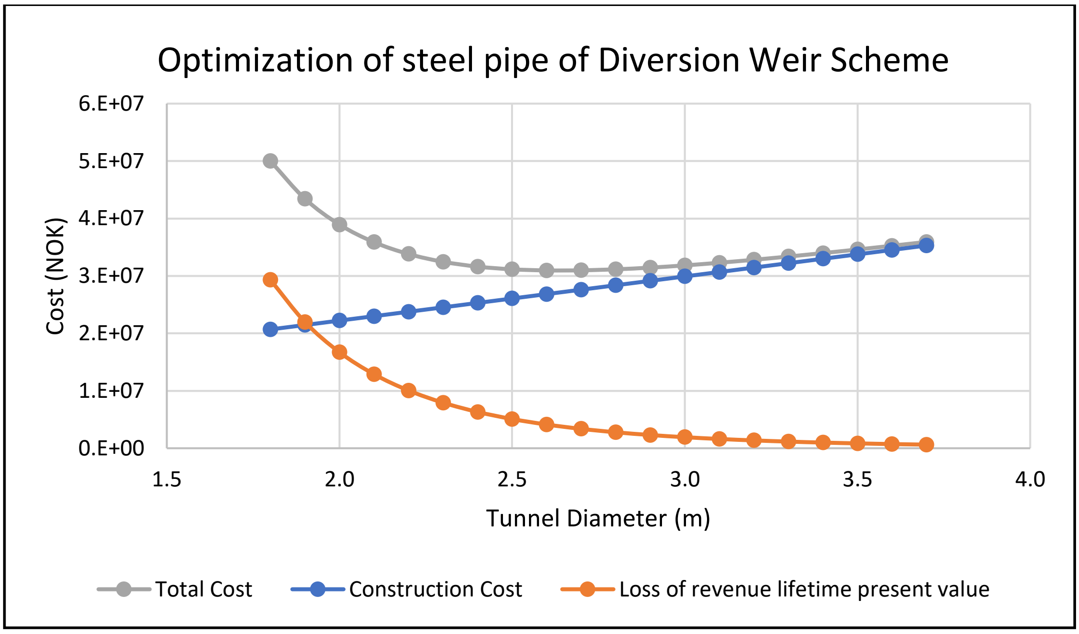


Figure A-6: Graphical optimization of headrace pipe of weir scheme.

A.5 Cashflow diagram

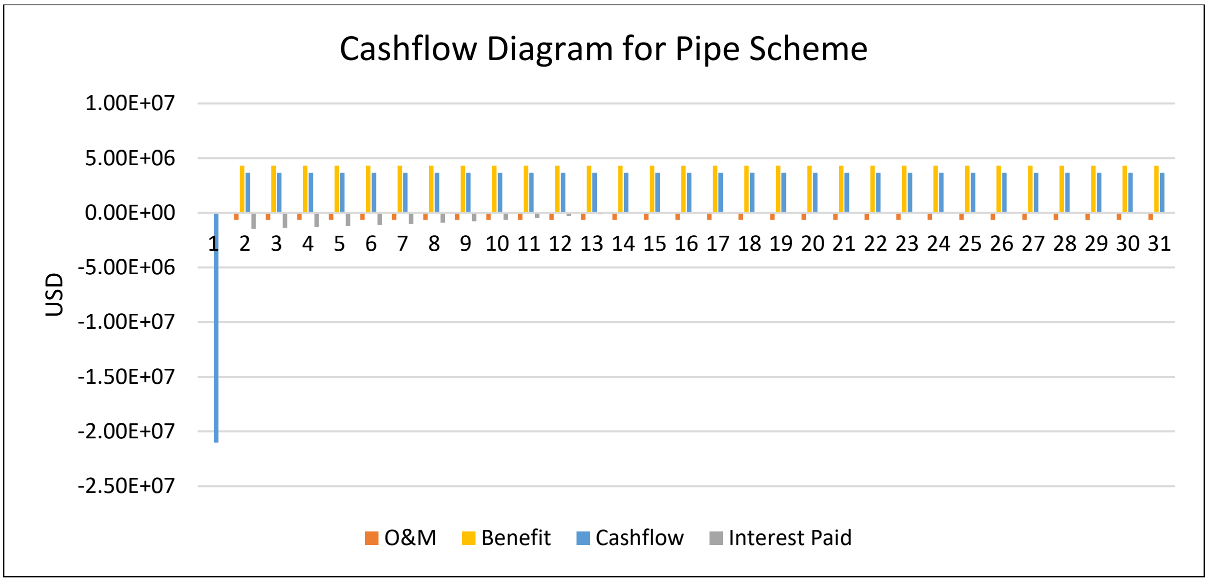


Figure A-7: Cashflow diagram for pipe scheme.

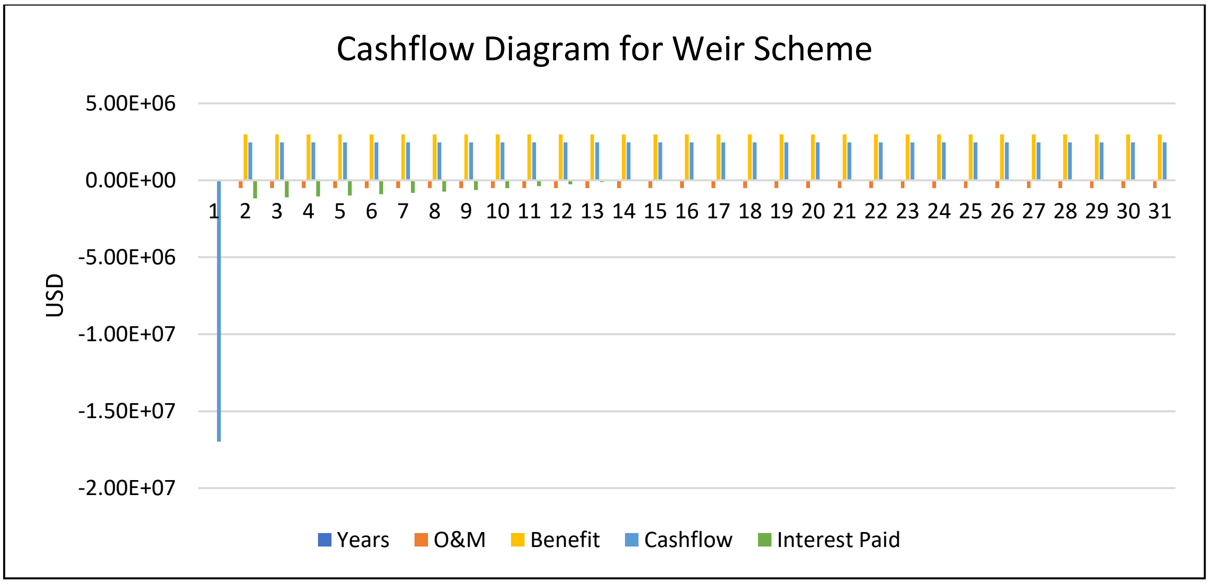


Figure A-8: Cash flow diagram for diversion weir scheme.

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