A STUDY ON EVALUATION OF HYDROPOWER POTENTIAL OF KAPOLOGWE WATERFALLS IN RUNGWE DISTRICT FOR RURAL ELECTRIFICATION

BY

CHARLES JOACHIM MGINA

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A DISSERTATION SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF ENGINEERING IN RENEWABLE ENERGY OF MBEYA UNIVERSITY OF SCIENCE AND TECHNOLOGY

NOVEMBER 2025

CERTIFICATION

We, the undersigned, certify that we have read and hereby recommend for acceptance by the Mbeya University of Science and Technology, a dissertation entitled, **"A Study on evaluation of hydropower potential of Kapologwe waterfalls in Rungwe District for rural electrification"** in a fulfilment of the requirements for the award of Master of Engineering in Renewable Energy of Mbeya University of Science and Technology.

Name: Prof. Z. Katambara
Date: 22/12/2025
Signature:
Main Supervisor

Name: Dr. Duncan A. Mwakipesile
Date: 23/12/2024
Signature:
Co-Supervisor

Accepted for the Board of……PSRPC……… Name date *Signature*
Dr OJUA F……Signature………

PRINCIPAL/DIRECTOR

DECLARATION AND COPYRIGHT

I, **Charles J. Mgina,** declare that this thesis is my original work and that it has not been presented and will not be presented to any other university for a similar or any other degree award.

Signature

Date 30/11/2025.

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DEDICATION

To my parents, Faith Julius and Joachim Benedict Mgina, as well as my children, Christela and Goodluck Charles Mgina, are honoured by the publication of this work.

ABSTRACT

This study assessed the hydropower potential of the Kapologwe Waterfalls, situated on the Kala River, a tributary of the Kiwira River in Rungwe District, Mbeya Region, using an integrated methodology that combined spatial tools, field measurements, and socioeconomic surveys. Tools such as GPS, current meters, and automatic levels were used to gather geospatial and hydrological data, while structured questionnaires captured energy demand profiles from six villages within the catchment. The analysis incorporated topographic, climatic, land use, soil, and discharge data to characterise the river system. Diversity factor analysis was employed to estimate village-level energy needs, and Karl Pearson's coefficient, along with the Weibull plotting technique, were used to validate hydrological correlations and construct a flow duration curve for the ungauged Kala River. Results indicate a hydropower potential of 7.237 MW, which falls short of the estimated 8.641 MW required to meet the aggregated four-year demand of all six villages. However, the identified potential is sufficient to meet the current demand (7.116 MW) of five villages, making the site viable for phased electrification. The study concludes that the Kapologwe Waterfalls offer a technically feasible solution for decentralised power generation in the Kala catchment. However, to meet long-term and inclusive demand, it is recommended that this resource be supplemented with additional energy sources, such as support from the Rural Energy Agency (REA). Furthermore, future research should focus on optimising turbine design for high-head, low-flow conditions to improve system efficiency, minimize maintenance needs, and extend equipment lifespan.

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ABBREVIATIONS AND ACRONYMS

AHEC	Alternate Hydro Energy Centre
Aug	Area of ungauged catchment (2km),
BOT	Bank of Tanzania
DEM	Digital Elevation Model
DHP	Direct Hydro Power
DoE	Department of Energy
DU	Diversified Unit load
ED	Electricity Demand
ESHA	European Small Hydropower Association
FAO	Food and Agriculture Organization of the United Nations
GDP	Gross Domestic Product
GIS	Geographical Information System
GPS	Global Positioning System
HP	Hydropower
HRET	Hydro Resource Evaluation Tool,
Hspw	Height of the spillway crest
IEA	International Energy Agency
IHA	International Hydropower Association
KWF	Kapologwe Waterfalls
MAF	Mean Annual Flow

MCA	Multi-criteria analysis
MD	Market demand
MGP	Mini-Grids Partnership
MHP	Micro Hydro Project
MHP	Micro Hydropower
MHS	Min Hydropower System
NBCBN	Nile Basin Capacity Building Network Research Cluster.
NC	Number of Consumers
NLCD	National Land Cover Database
PH	Potential Power House
POP	Population
ROR	Runoff River
RS	Regional Secretariat
SHP	Small Hydropower
ST	Sediment Tank
TANESCO	Tanzania Electricity Supply Company
TMA	Tanzania Meteorological Agency
TD	Total load after n years,
URT	The United Republic of Tanzania

LIST OF SYMBOLS

Symbols	**Description**	**Unit**
A	Area of the orifice	m2
A_{gaug}	Catchment areas at gauged sites	m2
A_s	Settling basin area	m2
A_{ung}	Catchment area un-gauged site	m2
B	Canal bed width	M
C	Coefficient of discharge of the orifice.	–
C	Coefficient of discharge	–
d	Internal diameter of the pipe	M
D	Load power	kW
$d_{airvent}$	Internal diameter of air vent	Mm
Du	Diversified unit load	–
E	Energy consumption of an appliance	–
E	Roughness height	Mm
E	Young’s Modulus	N/m2
f	frequency of network	Hz
F	Safety factor	–
$F_{lowgaug}$	Flow at gauged catchment	m3/s
F_{lowung}	Flow at ungauged catchment	m3/s
g	acceleration due to gravity	m2/s
P	Power	W
H	Gross head	M
h	Head of Water.	M
H	Net Head	M
Hc	Water Depth	M
Hd	design flood level	M
H_{flood}	Height of the flood level in the canal	M

HL	The head Loss	M
H_n	Net head	M
hr - hc	Difference - Headrace Canal Water Level	M
hr	Normal flood level	–
h_r	River head	M
h_s	Submergence head	M
h_{sp}	Height of the spillway crest from canal bed	M
h_{surge}	Surge head	M
KVA	Apparent Power	kVA
kW	Real Power	kW
kWh	Electric Energy	kWh
L	Length	M
L_s	Length of settling basin	M
$L_{spillway}$	Length of the spillway	M
Lspw	Length of Spillway	M
Mg	mean annual flow (gauged catchment)	m3/s
Mug	mean annual flow (ungauged catchment)	m3/s
MW	Mega Watt	–
N	Turbine Speed Rpm	
nc	Roughness Coefficient	–
n_c	Roughness coefficient	–
Np	side slope	m/m
N_s	Specific speed of turbine	–
n_z	No. of nozzles in the turbine (s)	–
P	Hydropower potential	kW
P	Power	W
P	Wetted perimeter	m
Pa	atmospheric pressure at water surface	Pa

Pw	pressure of water	Pa
Q	Flow Discharge	m3 /s
Q_{design}	Design flow in headrace canal	m3/s
Q_{mean}	Mean Flow	m3/s
r	Coefficient of correlation	
R	Hydraulic radius	m
Rpm	revolution per minute	
S	Ultimate tensile strength of the material	N/m2
Sc	Silt concentration flow	kg/m3
SL	Canal slope	m
SL	slope	m/m
S_{load}	Sediment storage	kg
T	Top width of canal	m
T	Canal top width	m
T_c	Critical time	sec
$t_{effective}$	Effective thickness of the penstock	mm
TWh	Terawatt hour	
V	velocity of flow	m/s
Vm	mean velocity	m/s
VO	Velocity	m/s
V_o	Velocity through the orifice	m/s
V_{osilt}	Volume of silt stored in the basin	m3
γ	Annual load growth rate	kg/m3
η	efficiency of turbine (dimensionless)	
H	Efficiency of the turbine (dimensionless)	–
ρ	water density	kg/m3
ρS	Density of silt	kg/m3

CHAPTER ONE
INTRODUCTION

1.0 Introduction

Chapter One introduces the foundation of this study by presenting the global and national context of hydropower as a vital renewable energy source. It outlines the persistent challenges of rural electrification in Tanzania and highlights the untapped potential of small-scale hydropower resources such as the Kapologwe Waterfalls. The chapter identifies the research gap stemming from limited site-specific assessments in rural contexts and formulates the problem statement that guides the study. It then establishes the main and specific research objectives, along with the central and subsidiary research questions. Furthermore, the chapter explains the significance of the study to stakeholders, policymakers, and rural communities, demonstrating its contribution to both academic knowledge and practical energy planning. Finally, it clarifies the scope of the study, focusing on the hydrological and socio-economic assessment conducted within the Kapologwe area.

1.1 Background

Hydropower remains a key source of renewable energy, contributing significantly to global electricity production due to its reliability and low greenhouse gas emissions. According to Santl and Steinman (2015), despite being in use for over a century, hydropower continues to be the most widespread source of renewable electricity. In 2018, hydropower generated 4,325 TWh globally, accounting for 68.7% of all renewable energy output and 17.3% of total global electricity production (IEA, 2019). The International Hydropower Association (IHA, 2015) estimates that hydropower supplies approximately 16% of the world’s electricity, a figure that is steadily increasing, particularly in regions with untapped hydropower potential.

In Tanzania, electricity supply remains unreliable, especially in rural areas. There are regions frequently experience unscheduled outages and inconsistent daily supply.

Expanding access to electricity through decentralized solutions—such as mini-grids—can reduce pressure on the main grid and provide reliable energy to remote communities. However, hydropower development in Tanzania is hindered by limited research, high initial investment costs, and a national preference for centralized power systems (Bishoge *et al*., 2018). The long lead times associated with grid expansion, as well as transmission losses from distant generating stations, further delay electricity access for off-grid communities.

Hydropower, as a clean and renewable energy source, is essential not only for economic growth but also for enhancing the quality of life in underserved regions. Access to affordable and reliable electricity is a foundational requirement for Tanzania's socio-economic transformation. Recognizing this, the government has made significant investments in large-scale hydropower infrastructure, notably the ongoing construction of the Julius Nyerere Hydropower Project (2115 MW) and its associated 400 kV substation. While such initiatives are vital for national supply, local-scale renewable energy solutions remain critical for addressing persistent energy shortages in rural areas with high resource potential.

Despite extensive studies on hydropower potential across Tanzania (Mhilu, 2004; URT, 2012; Kiwia, 2013; Kichonge *et al.*, 2015; WB, 2015; Mdee, 2018), there is limited research specifically targeting small-scale hydropower development in local contexts. One such unexplored resource is the Kapologwe Waterfalls**,** located in the Kala River, a tributary of the Kiwira River in the Kisondela Ward of Rungwe District. The energy potential of this site has not been systematically evaluated, despite its proximity to rural communities that face chronic energy insecurity.

The Kapologwe Waterfalls represent a promising opportunity for rural electrification, given their perennial flow and elevation drop. Local hydro resources like these can effectively meet the basic electricity needs of households and small enterprises, yet remain underutilized due to a lack of awareness, technical expertise, and investment. According to Mtalo (2005), decentralized hydro schemes can significantly improve livelihoods in rural Tanzania if properly harnessed. Furthermore, access to electricity supports key economic sectors such as agriculture, tourism, industry, and mining (Bishoge, 2018), and contributes directly to achieving national development goals and international targets such as the Sustainable Development Goals (SDGs).

This study aims to bridge the knowledge gap by assessing the hydropower potential of the Kapologwe Waterfalls. Specifically, it seeks to evaluate the feasibility of harnessing this resource to meet the electricity demands of six villages in Kisondela Ward. The study also contributes to the broader effort to identify viable off-grid renewable energy sources that can reduce energy poverty and enhance resilience in rural communities. According to Kihwele *et al*. (2010), the availability of clean and reliable energy is a fundamental enabler for national economic development. Therefore, unlocking local hydropower resources like those at Kapologwe can contribute meaningfully to Tanzania's energy transition and rural development agenda.

1.2 Problem Statement

In Tanzania, the national grid, powered mainly by hydropower, remains the primary source of electricity. However, this centralised system fails to serve many rural areas, where communities continue to experience limited or no access to reliable electricity. According to URT (2012), the lack of a consistent and dependable energy supply is a significant barrier to both economic and social development in the country. While these rural regions remain underserved, they often possess untapped small-scale hydropower resources that could meet local energy needs for households, social services, and small enterprises. Mtalo (2005) notes that hydropower resources have the potential to enhance rural livelihoods, yet they remain largely unutilised due to limited research, technical capacity, and investment. In addition, existing energy infrastructure frequently falls short of delivering the level of reliability required to support rural development, despite the country's considerable hydropower potential.

Several studies have examined Tanzania's hydropower potential (Mhilu, 2004; URT, 2012; Kiwia, 2013; Kichonge *et al*., 2015; WB, 2015; Mdee, 2015). However, there has been no specific assessment of the Kapologwe Waterfalls on the Kala River in Kisondela Ward, Rungwe District, which may offer a viable site for rural electrification. This lack of site-specific data presents a critical research gap. Addressing this gap is essential for exploring decentralised energy solutions that are responsive to the needs of rural communities. Assessing the hydropower potential of the Kapologwe Waterfalls could provide vital technical and socio-economic insights to support the development of localised energy systems. This, in turn, would contribute to improving energy access, enhancing rural productivity, and reducing dependency on the unstable national grid.

Therefore, this study aims to evaluate the hydropower potential of the Kapologwe Waterfalls to develop a sustainable rural electricity supply in Kisondela Ward, Rungwe District. As Kihwele *et al*. (2010) emphasise, providing affordable and reliable energy is a key driver of national development. Harnessing local natural resources such as Kapologwe's hydropower potential could play a transformative role in closing Tanzania's rural energy gap

1.3 Research Objectives

1.3.1 Main Objective

The primary objective of this study is to evaluate the hydropower potential of the Kapologwe Waterfalls on the Kala River in Kisondela Ward, Rungwe District, to support rural electrification and sustainable energy access for the surrounding communities.

1.3.2 Specific Objectives

i. To analyse the key hydrological parameters (flow and head) of the Kapologwe Waterfalls.
ii. To estimate the current and projected electricity demand of the surrounding villages within the catchment area.
iii. To develop a preliminary design for a mini-hydropower system based on the hydrological and demand characteristics.

1.4 Research Questions

The central research question guiding this study is:

Is the hydropower potential of the Kapologwe Waterfalls sufficient to generate reliable electricity that can support socio-economic development in Kisondela Ward?

To address this overarching question, the following sub-questions are considered:

i. What are the hydrological characteristics and energy potential of the Kapologwe Waterfalls?
ii. What is the current and forecasted power demand of the surrounding villages in the Kisondela Ward?
iii. How can a mini-hydropower plant be technically designed to utilise the Kapologwe site for rural electrification?

1.5 Significance of the Research

This study contributes both practical and academic value in the field of decentralised renewable energy development. Specifically, it provides crucial insights into the hydropower potential of the ungauged Kapologwe/Kala River catchment, offering hydrological and socioeconomic data that can inform site-specific energy planning.

The results of this research are expected to benefit key stakeholders, including the Ministry of Energy, TANESCO, local government authorities, and development partners involved in rural energy access and infrastructure planning. These stakeholders may use the findings to initiate or support the design, funding, and implementation of small-scale hydropower projects in the region.

Moreover, this study helps to address a critical knowledge gap in existing literature by focusing on a specific, understudied site. It adds to the national body of knowledge on mini-hydropower development and supports evidence-based decision-making for energy policy and project planning.

Beyond its technical contributions, the study fosters local capacity development by promoting practical solutions and design methodologies that can be replicated in other similar rural contexts. It also encourages innovation in the deployment of off-grid energy solutions, ultimately contributing to improved livelihoods and accelerated rural development.

1.6 Scope of the Study

All required information for this Dissertation including data collection, and data tests was conducted within Kapologwe waterfalls in Rungwe District and in some cases, the literature review was applied.

CHAPTER TWO

LITERATURE REVIEW

2.0 Introduction

This chapter reviews the current state of knowledge related to the assessment of hydropower potential from waterfalls, with a focus on mini-hydropower systems. The study begins by defining key concepts relevant to the topic. It proceeds to discuss critical components, including hydrological potential, energy demand, the design of mini-hydropower plants, and associated socio-economic and environmental considerations. Furthermore, it explores emerging tools and methodologies applied in feasibility studies and highlights lessons learned from related case studies to contextualise the study of Kapologwe Waterfalls.

2.1 Concepts

2.1.1 Hydropower

Hydropower refers to the generation of electricity using the kinetic and potential energy of flowing or falling water. Although it constitutes a portion of the total global energy mix, it remains the most significant contributor among renewable energy sources (Ellabban, 2014). Its accessibility in mountainous and riverine regions makes it a practical option for electrifying remote and off-grid areas. Additionally, hydropower offers low operational costs, long project lifespans, and the potential for multi-purpose use, including irrigation and flood control.

2.1.2 Small-Scale Hydropower

Small-scale or mini-hydropower systems typically range in capacity from 0.5 to 10 MW (Tsoutsos *et al*., 2007). These systems are suitable for both individual and community-based applications. Globally, hydropower—both large and small—accounts for about 19% of total electricity production, making it the most significant renewable source (Paish, 2002). Mini-hydropower schemes are significant in developing countries where grid extension is uneconomical.

2.1.3 Hydropower Potential

The theoretical hydropower potential is defined as the maximum energy that could be harnessed from water flow and head without accounting for technical or environmental limitations (Chandy *et al*., 2012). The hydropower potential of a site primarily depends on two key factors: flow rate and hydraulic head (Paish, 2002; Khan and Zaidi, 2015). Systematic measurement and analysis of these parameters are crucial for determining the site's actual generation capacity. Additionally, potential is often categorised as theoretical, technical, and economic, each accounting for successive levels of practical constraints.

2.1.4 Water Falls

Waterfalls represent natural elevation drops in a river system that convert potential energy into kinetic energy. They are often formed by erosional and tectonic processes (Galton and Mizoguchi, 2009). Some waterfalls serve as geomorphosites, offering both ecological and hydrological significance (Kale, 2015; Goudie, 2020). Waterfalls in mountainous regions can serve as ideal sites for micro and mini-hydropower projects due to their naturally high heads.

2.1.5 Power Transmission System

The power transmission system transfers mechanical energy from the turbine to the generator. Components include shafts, bearings, couplings, and belts. Cost-effective coupling methods such as V-belt or flat-belt systems are commonly used in mini-hydropower setups (Pandey, 2007). This study proposes a direct-drive system where the turbine and generator shafts are directly coupled. Direct-drive systems are preferred for their simplicity and reduced maintenance requirements.

2.2 Evaluation of Hydropower Potential

Assessing hydropower potential involves analysing both natural and technical parameters. Tools such as Geographic Information Systems (GIS) and Remote Sensing (RS) have been extensively used to delineate catchments and analyse hydrological variables (Pandey, 2015; Teshome *et al*., 2020; Yevalla, 2018). DEMs are often used to extract slope, elevation, and drainage patterns, which are then fed into hydrologic models to estimate runoff and design discharge (Raghu, 2015).

Studies have demonstrated that integrating GIS and RS provides robust frameworks for evaluating hydropower potential. Multi-Criteria Analysis (MCA) has also been used to evaluate trade-offs between environmental, economic, and technical factors in hydropower development (Vassoney *et al*., 2017). MCA helps decision-makers prioritise sites based on multiple indicators. Other studies incorporate hydrological simulation tools like HEC-HMS and SWAT to forecast river flow dynamics. The methodology proposed for this study aligns with these best practices by combining hydrological analysis, field data, and GIS-based modelling.

2.3 The Nearby Villages' Need for Electricity

Accurate estimation of energy demand is a critical step in determining the technical and economic feasibility of mini-hydropower systems. In rural electrification projects, demand forecasting must reflect local consumption behaviours and socio-economic trends to ensure that the system is neither overdesigned nor underperforming. One widely used method for estimating energy demand involves a straightforward engineering approach based on appliance usage. The total energy requirement (E) for a given user group can be calculated using Equation 2.1.

$$E = S \times N \times P \times H \tag{2.1}$$

Where:

S = Appliance saturation level

N = Number of households

P = Power rating (kW)

H = Hours of use

This approach enables the aggregation of electricity demand across different types of appliances and customer categories, giving a sectorial overview of expected energy consumption. In addition to the engineering method, econometric modelling is often used for long-term electricity demand forecasting. This method combines economic theory with statistical tools to estimate the relationship between electricity demand (ED) and several influencing variables. A general functional form of the econometric model is:

$$ED = f\left(Y, P_I, P_J, POP, T\right) \qquad 2.2$$

Where:

Y = Household or sectorial income/output

P_i = Price of electricity

P_j = Price of substitute fuels (e.g., charcoal, kerosene)

POP = Population

T = Technology level

The relationship between these variables and electricity demand can be estimated using regression techniques, such as the ordinary least squares (OLS) method or time series analysis, to provide forecasts under various policy or market scenarios (Ibitoye, 2013).

For practical planning purposes, TANESCO recommends a 25-year load forecasting horizon for areas targeted for grid or mini-grid electrification. During the initial years post-commissioning, rapid growth in energy consumption is expected due to increased household connections and demand stimulation. Typically, energy demand is projected to grow at 25% per year for the first four years, eventually stabilising at annual growth rates of:

(i) 4% for residential users
(ii) 3% for commercial users
(iii) 2% for light industrial users
(iv) 2% for public lighting

To analyse consumption patterns, statistical methods such as the Shapiro-Wilk test may be applied to assess the distribution and variability of load data. This helps identify anomalies and validate the assumptions of the forecasting model. Furthermore, spreadsheet-based demand modelling is a common tool for rural electrification planning. Ibitoye (2013) demonstrated how such tools can incorporate historical consumption patterns, appliance inventories, and future expansion plans to build a comprehensive demand profile. These projections are essential for ensuring that the mini-hydropower system can reliably serve its intended users over its design life.

Ultimately, combining engineering-based calculations with econometric and statistical analyses provides a robust framework for assessing energy demand. This ensures that the hydropower system is appropriately scaled to meet current needs while being adaptable to future growth.

2.4 Miniature Hydroelectric Plant Design

Mini-hydropower systems are commonly designed using a run-of-river (RoR) configuration, which diverts a portion of a river's flow through a turbine without the need for large-scale reservoir storage. These systems typically comprise key components such as weirs, intake structures, settling tanks, penstocks, turbines, generators, power transmission systems, and control units. The effectiveness and reliability of a mini-hydropower installation depend heavily on site-specific factors including local topography, hydraulic head, soil conditions, and flow characteristics (Raghu, 2015; Ahlborga and Sjöstedt, 2015).

The design process begins with a detailed hydrological assessment and catchment modelling to estimate the design discharge. This requires the delineation of drainage patterns and runoff estimation, which are facilitated by advances in computer technology. Geographic Information Systems (GIS) and rainfall-runoff hydrologic models are widely used in this context. Digital Elevation Models (DEMs), in particular, are critical for analysing terrain features, identifying flow accumulation points, and estimating runoff depth and hydrographs near gauging stations (Raghu, 2015).

Once the design discharge is established, civil structures such as power canals and penstocks are laid out in alignment with site contours and geological conditions to ensure structural safety and cost-effectiveness. The selection of turbine type—such as Pelton, Francis, or Kaplan—depends primarily on the available head and flow rate. Proper turbine selection is essential for achieving optimal energy conversion efficiency.

The civil works, including intake structures and open channels, must be carefully designed to handle the expected discharge and ensure durability. The alignment of open channels is typically determined by local topography, the availability of right-of-way, and the presence of existing or planned infrastructure. When designing the channel cross-section, key considerations include hydraulic capacity, channel slope, invert elevation, and the type of lining material used. These design parameters are selected to maintain flow stability, prevent erosion, and minimise head losses.

Ultimately, the hydropower capacity of the site is calculated based on the derived flow and head values, and a suitable site is selected for plant construction. Environmental safeguards are also incorporated into the design to minimise ecological disruption and preserve downstream flow regimes. The integration of hydrological modelling, GIS tools, and engineering design ensures a comprehensive and site-adapted approach to developing sustainable mini-hydropower systems.

2.5 A Comparison of Energy Demand and Hydropower Potential

The ultimate viability of a mini-hydropower project depends on the balance between estimated power generation and local energy demand. Methods such as appliance inventory analysis, empirical load models, and diversity factor adjustments are used to estimate household and community-level electricity needs (De Vita *et al*., 2006; Mwangomo, 2010).

In rural contexts, where energy demand is evolving, uncertainty in timing and scale is significant. Reducing this uncertainty through localised assessments is essential for effective planning. Decision-support tools such as RET Screen and HOMER can be used to model energy balance, simulate scenarios, and evaluate financial feasibility. This study will compare the estimated power output of the Kapologwe Waterfalls with the projected demand from nearby villages to determine feasibility, prioritise beneficiaries, and guide system design. A successful match between potential supply and demand not only justifies investment but also supports sustainable rural development and aligns with national energy access goals.

CHAPTER THREE

RESEARCH METHODOLOGY

3.0 Introduction

Chapter Three provides a comprehensive description of the research methodology employed to assess the hydropower potential of the Kala River, with a focus on the Kapologwe Waterfalls. The chapter details the study area, including its topography, hydrology, and local communities, highlighting the river's suitability for mini-hydropower development. It outlines methods for estimating the river's head and flow rate using GPS, field surveys, and hydrological correlation with the nearby gauged Kiwira River, alongside techniques for developing flow duration curves. The chapter further explains procedures for evaluating hydropower potential, selecting appropriate turbines, and analyzing electricity demand for surrounding villages. Finally, it presents the design of essential civil and hydraulic infrastructure, including headworks, forebay, penstock, canals, settling basins, spillways, and the powerhouse, integrating engineering, hydrological, and community-based assessments to provide a robust framework for sustainable energy development.

3.1 Description of the Study Area

The Kala River, a tributary of the Kiwira River, originates in the Rungwe Volcanic Highlands, located in the southern part of Rungwe District within Tanzania's Mbeya Region. The Kiwira River itself begins in the foothills of the Poroto Mountains and flows southward, ultimately discharging into Lake Nyasa near Itungi Port in the Kyela District. This study focuses on the middle reaches of the Kala River, with particular attention to the Kapologwe Waterfalls, a prominent hydrological feature with significant potential for mini-hydropower development (Figure 3.1). Located approximately 25 kilometres southeast of Tukuyu Township, the waterfalls lie at an elevation of around 960 meters above sea level in Kisondela Ward, Rungwe District. Nestled within a dramatic basalt rock gorge, the Kapologwe Waterfalls offer both aesthetic and technical advantages: their natural height difference (head) enhances their suitability for hydropower, while their scenic appeal has made them an emerging eco-tourism destination in Tanzania’s Southern Highlands.

The Rungwe Highlands, ranging from 1,500 to 2,500 meters above sea level, form the upper catchment of the Kala River. This region benefits from high rainfall, dense forest cover, and volcanic geology, which together ensure perennial river flow, ideal conditions for run-of-river hydropower schemes. The Kisondela Ward, which stands to benefit directly from this proposed development, includes several rural villages: Kisondela, Bugoda, Lutete, Kibatata, Ndubi, Mpuga, and Isuba. These communities are primarily reliant on subsistence agriculture and have limited access to grid electricity. Establishing a mini-hydropower system at the Kapologwe site holds significant promise for improving energy access, stimulating economic development, and enhancing social services throughout the ward.

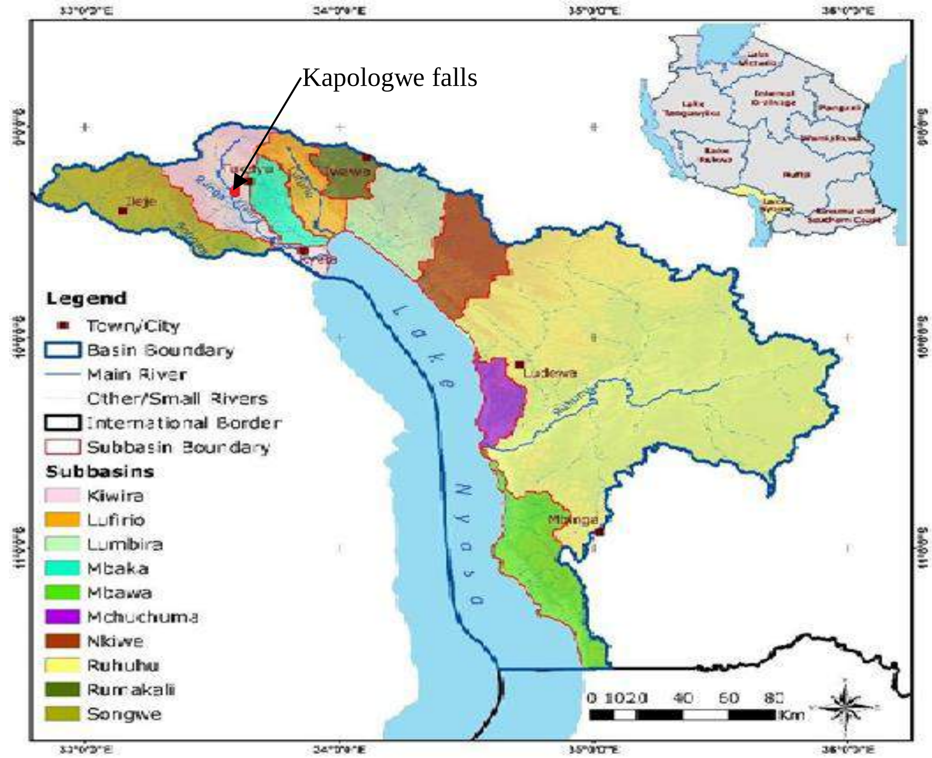

Figure 3. 1: Kapologwe Water Falls in Lake Nyasa Basin. Modified from (URT, 2016)

3.2 Investigating the Hydropower Potential

To calculate the hydropower potential in Watts of Kala waterfalls, the study utilised the head (H) in meters and the flow rate (Q) in cubic meters per second (m^3/s). A GPS, orthographic techniques, field surveying, or any combination of these methods may be used to estimate the gross head (ESHA, 2004). GPS was used in this study to determine the location and elevation of the weir and sediment tank. The location of the weir were chosen based on the river characteristics. A straight river reach that is stable, narrow, and has a gentle slope and adequate ponding requirements. The mean flow rate was calculated using the area ratio. The plan aimed to ensure that the satisfactory requirements for the flow along the Kala River, which is not gauged, are developed. Figure 3.2 provides the summary of the methodology.

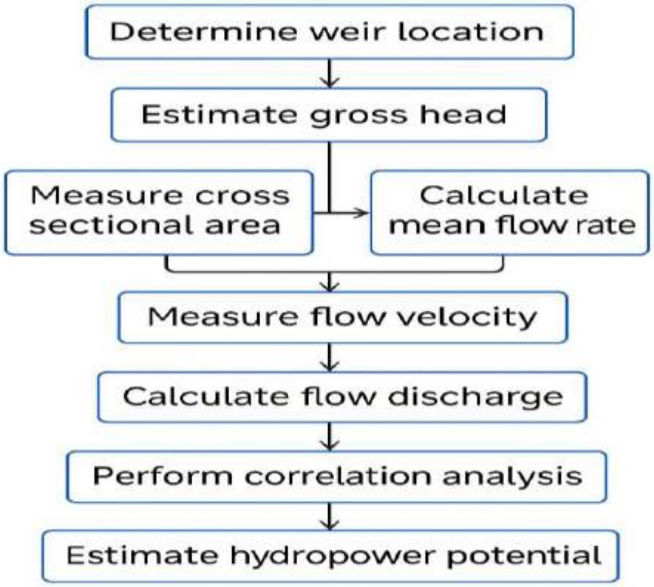

Figure 3. 2: Flow diagram for the methodology

3.2.1 Head (H) Dimensions

When generating hydropower, the head is the distance that a given water source has to fall before the point where power is generated. Ultimately, the force responsible for hydropower is gravity, so a hydroelectric plant with a tall/high head can produce more power than a similar plant with a short/low head. In short, for a given water flow, a larger head will be converted into greater kinetic energy. That energy is then harnessed by a water wheel or water turbine to create usable hydropower. Direct distance measurement and water pressure are the two most common methods for measuring head (Daniel, 2004). The total head (H) of the Kala waterfalls was measured using a GPS mobile set of instruments. The coordinates of the potential sediment tank (ST) and potential power house (PH) wase measured using a GPS map. Equation 3.1 was used to calculate the head as follows:

$$H = PH - ST \tag{3.1}$$

3.2.2 Hydrological analysis

Since the Kala River is not gauged, but the Kiwira River nearby is gauged, the area ratio method was used for hydrological analysis of the Kala River. Since the Kala River cannot be gauged, correlation analysis was required to ascertain whether the hydrological characteristics of the two catchment areas were comparable. This relationship was assessed using Karl Pearson's Coefficient.

3.2.3 Area Ratio Method

The area ratio approach assumes the presence of a gauged catchment in the vicinity of the ungauged catchment and having similar characteristics. According to Emerson *et al*., (2005), the topography, land use, geomorphology, and lithology of the two catchments must be hydrologically identical. ArcGIS software was used to determine the ungauged stream's catchment area. To define the necessary location, a Digital Elevation Model (DEM) was used. Equation 3.2 was used to calculate the mean annual flow (MAF) for the ungauged catchment area in the manner described below.

$$M_{ug} = \frac{A_{ug} * M_g}{A_g} \tag{3.2}$$

Where A_{ug} is the unmeasured catchment's area (in km^2), M_{ug} is the unmeasured catchment's mean annual flow (in m^3/s), and M_g is the unmeasured catchment's mean annual flow (in m^3/s).

3.2.4 Field Survey Method

To gauge the flow rate, a three-point method similar to that described in (DOE, 2009) was used. The velocities at 20% (V20%), 60% (V60%), and 80% (V80%) from the surface of

Water River was measured using an electromagnetic current meter. Then, using Equation 3.3, determine the river's mean flow rate (Vm). Measurements of the flow will be taken throughout the dry season, the rainy season, and immediately after the rainy season.

$$Vm = 0.25 \times (V0.2 + 2V0.6\, V0.8) \tag{3.3}$$

3.2.5 The Kala River's Cross-Sectional Area

The location where the axis of the river bed is straight and the cross section of the river is nearly uniform is where the stream flow will be measured. A measuring tape was used to determine the river's width. The average depth (m) was calculated by summing the depth measurements. The cross-sectional area was calculated by multiplying the outcomes by the width of the river, as shown in Equation 3.4.

$$s = b\left(\frac{h+\cdots+h}{n}\right) \tag{3.4}$$

Where b is the width, h is the depth, and S is the estimated area.

Equation 3.5 was used to determine the flow of the Kala River during the dry season, the rainy season, and immediately following the rain. ESHA (2004) used a schematic diagram (Figure 3.3) to illustrate the process.

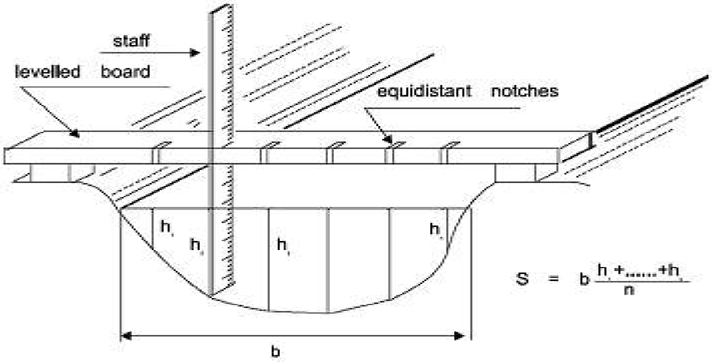

Figure 3. 3: Measuring the cross-sectional area (ESHA, 2004)

3.2.6 Stream Flow Discharge

The continuity equation, as shown in Equation 3.5, was used to calculate the flow rate from the cross-sectional area and velocity.

$$Q = Vm \times A \tag{3.5}$$

Whereby; flow discharge = flow rate = Q m^3/sec

Flow of velocity = Vm/sec

Vm stands for mean speed, and A for estimated area.

3.2.7 Analyses of Hydrological Correlation

The correlation analysis was used to determine whether two catchments have the same hydrological characteristics. The Nyasa River Basin Water Office and Tanzania Meteorological Agency (TMA) Mbeya Office provided the average monthly flows (m^3/s) for Kiwira River and rainfall (mm) for Kisondela Ward, respectively. Equation 3.6 and Karl Pearson's coefficient of correlation (r) enable the computation and determination of whether gauged and ungauged points correlate.

These are the values for Karl Pearson's coefficient of correlation:

$$r = \frac{\sum xy}{(\sum x^2)(\sum y^2)} \qquad (3.6)$$

Where y denotes Kala stream's monthly rainfall data and x denote Kiwira's monthly average flow rates. R values range from +1 to -1, with +1 denoting perfect direct correlation, -1 denoting perfect inverse correlation, and 0 denoting no correlation at all.

3.2.8 Estimating a Flow-Duration Curve for an Ungauged Catchment

By using a curve derived for a gauged site along the same stream or in a neighbouring catchment, a flow duration curve for an ungauged catchment was estimated. The ratio of the catchment of the Kala River to the flow of the nearby gauged Kiwira catchment was multiplied to determine the flow of the Kiwira River. The Kiwira River's flow enabled the plotting of a flow duration curve, which will show the percentage of time against discharge for an ungauged site.

The stream flow data was arranged in descending order of stream discharges to create the flow duration curve. The daily flow data that are available for a decade were used to create a range of values known as class intervals. Plotting relationship calculations was used to determine the plotting position, and the results was plotted on logarithmic paper. Equation 3.7 gives the percentage likelihood that any flow magnitude Q will be matched or exceeded.

$$Pp = \frac{m}{N+1} \times 100\% \qquad (3.7)$$

Where m = Ranked position on the listing, N = Number of events for the period of record, and PP = Probability that a given flow will be equalled or exceeded (% of time).

3.3 Hydropower Potential

The effective head, flow rate, and turbine efficiency all influence the amount of power produced by hydraulic turbines. To calculate the power potential at Kala River, Equation 3.8 was used.

$$P = Q \times g \times H \times \eta \qquad (3.8)$$

Where P is the Kala River's potential, ρ is water density, g is the acceleration of gravity, H is the river's head, and η is the efficiency of a particular turbine.

3.4 Selection of the Turbine

Turbines come in a variety of varieties. The site data, particularly the head and flow discharge, determine the type of turbine that should be used. Efficiency and cost are additional technical criteria. A chart for choosing a turbine based on head and flow discharge is shown in Figure 3.4.

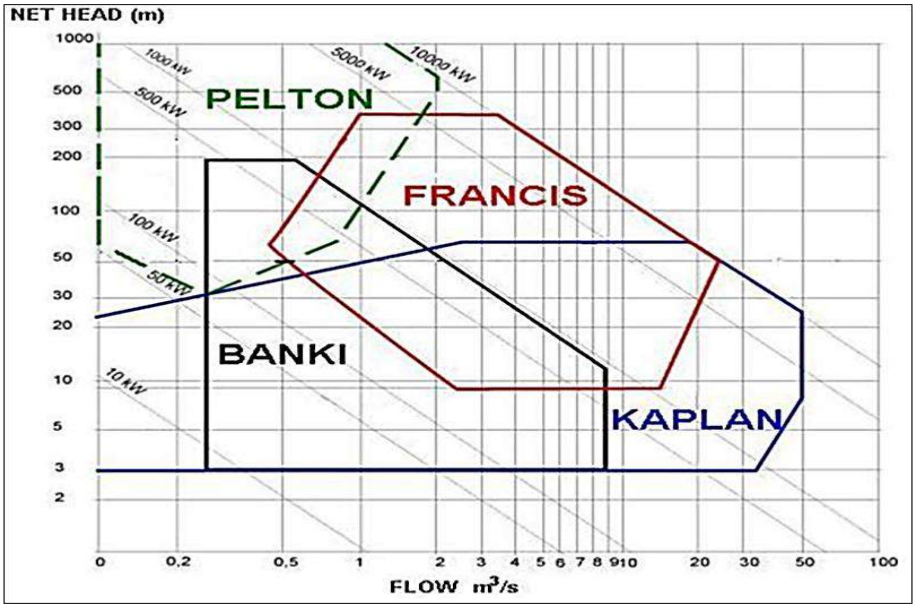

Figure 3. 4: Turbine Selection Chart (Inforse, 2014)

3.5 Hydro-Power Demand of Neighbouring Villages

The following techniques were used to estimate demand for rural electricity, according to Mehra (2001): trend analysis, the end-use approach, the econometric approach, and load and energy forecasting.

3.5.1 Demand Analysis

3.5.1.1 Trend Analysis

Trend analysis extrapolates current rates of electricity demand from historical data. To forecast future changes in electricity demand, trend analysis focuses on past variations or movements in electricity demand.

3.5.1.2 End-Use Approach Method

The various uses of electricity in the residential, commercial, agricultural, and industrial sectors of the economy are the focus of the end-use models for electricity demand. The relationship given below identifies the sector's end-use methodology.

The current study employed the Load and Energy Forecast method to analyse the power needs of six villages in the Kala River sub-catchment: Kisondela, Bugoda, Lutete, Kibatata, Ndubi, and Isuba. To determine the electrical load demand of the six villages in Kala catchment's Kisondela ward, a survey was conducted. Structured interviews and focus groups were used to collect the data. The study's primary focus areas were residential customers, light commercial customers, light industries, and other social services. The load demand for each village was calculated using the diversified unit load and group diversity factors for classified loads in Appendix 1. Equation 3.9 was used to determine the individual diversified maximum power in each village, and the total demand for all twelve villages was calculated by:

$$MD = NC \times DU \times Dg \quad (kW) \qquad \textit{(3.9)}$$

The number of consumers (NC), the diversified unit load (DU), the market demand (MD), and the group diversity factor (Dg) are all included.

Equation 3.10 was used to estimate the demand forecast for these twelve villages over the next four and twenty-five years.

$$TD = D(kW) \times (1 + \text{⍰}) \times n \qquad (3.10)$$

Where γ is the annual load growth rate (which is 0.25 for the first four years), D is the load in kW, TD is the total load after n years, and n is the number of years.

3.6 Design of the Initial Mini-Hydropower Plant

Civil structures, mechanical components, and electrical/electronic components are typically the main components of mini hydro power plants. Current field conditions will determine the subcomponents of civil structures. Designs for civil structures relevant to the Kala River catchment are presented in this chapter. These include the forebay tank, penstock, settling basin, headrace canal, and spillway. Additionally, the right turbine and generator have been chosen. The typical main parts are depicted in Figure 3.5, along with a powerhouse where the selected turbine and generator will be installed.

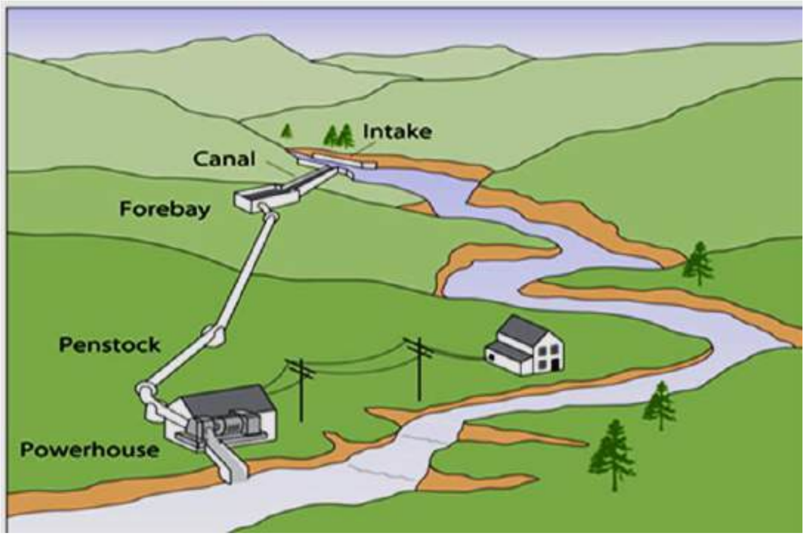

Figure 3. 5: The general layout of the SHP and its major parts

3.6.1 The Head Works

This is the diversion point of the water conveyance system. Water can travel to this point in run-of-river systems either by gravity or by pumping. The goal is to direct the necessary amount of water from the source to the hydroelectric plant's waterways. The intake must divert the required volume of water into a power canal or a penstock with the least amount of head loss and environmental impact possible (Korkomaz, 2007).

The intake (Weir, Orifice, and Approaching Canal) and the settling basin are significant parts of the headworks. A weir or an orifice can be used in the design of a check structure in the headwork. The study recommends explicitly using a submerged orifice based on the physical characteristics of the Kala River.

3.6.1.1 Design of Side Intake Orifice

The conveyance system that transports designed discharge from the intake to the headrace (forebay, penstock, and powerhouse) and prevents flooding is made possible by the side intake. Figure 3.6 shows a schematic representation of a longitudinal section of an intake.

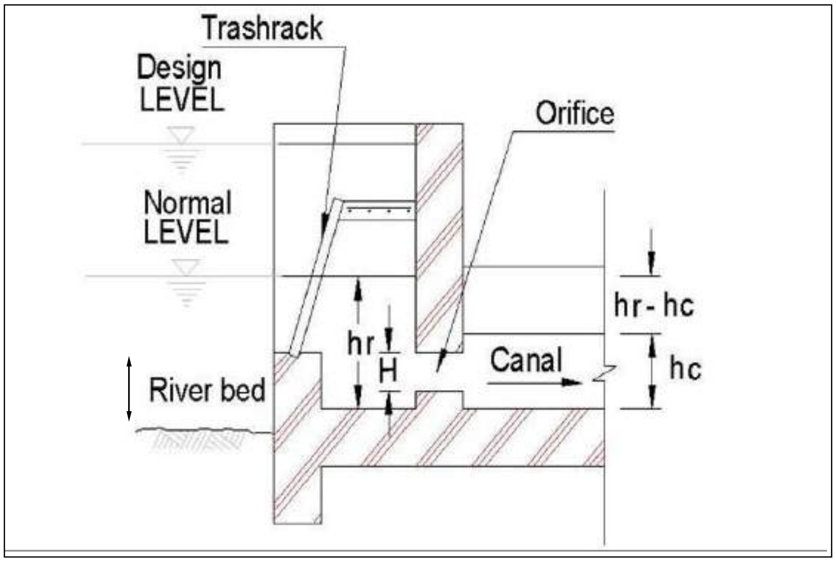

Figure 3. 6: The layout of the power house's intake to conveyance system

During the low-flow season, it should be sized so that it is submerged at the time of the design flow. This help to control excess flows during floods. Using Equation 3.11, the discharge through the orifice is calculated.

$$Q = AVO = A \times C \sqrt{(2 \times g \times (hr - hc)} \qquad (3.11)$$

Where Q is the orifice's discharge in cubic meters per second, VO its velocity in meters per second, hr - hc its difference in water level between the river and the headrace canal, and C its coefficient of discharge.

According to AHEC (2011), for a sharp-edged and roughly finished, fully submerged concrete or masonry orifice structure, the coefficient of discharge (C) value varies depending on the materials used to develop the structure of the intake. The value of C can range from 0.6 to 0.8 for a precisely finished and smooth opening. In addition, since a higher water level in the river results in a greater head at the orifice, the value of hr - hc depends on the discharge in the river. During typical flow, the orifice's velocity ranges between 1.0m/s and 1.5m/s. To prevent drawing bed load into the intake, the velocity should be less than 1.0 m/s if the orifice does not have a trash rack.

3.6.1.2 Headrace Canal Design

The headrace canal transports the water to the fore-bay after it enters through the intake. A canal's cross-sectional profile, slope, and roughness all affect the flow it can carry (ESHA, 2004). Canals can occasionally be replaced with pipes. The proposed materials for the canal's construction will be determined by the topographic survey's findings regarding the site's geographic situation and other obvious factors, such as the availability of labour and materials (Kunwor, 2012). According to Kunwor (2012), the most crucial factor to consider when building a headrace canal is to maintain a slight elevation in the canal's slope. This is because a higher slope can result in a higher water velocity, which can then cause erosion in the surface of the headrace canal. The headrace for a small-scale hydropower plant typically utilises an exposed structure, such as an open channel or a covered channel, due to the generally low volume of water conveyance (DOE, 2009).

The Criteria for the Headrace Canal's design is based on the following factors, which influence the canal's dimensions and cross-section: capacities, velocity, side slope, head loss and seepage, stability, and economics. The topography and soil type results of the site survey, particularly for open channels, will determine the canal's design parameters.

The headrace canal will be designed according to the following process:

i. Select the appropriate canal type based on the stability and site conditions.
ii. Using Table 3.1, select a suitable velocity (V) for the chosen canal type, and use Table 3.2 to determine the roughness coefficient (nc). If chosen velocities are very close to maximum velocity, unacceptable head loss may result.

Table3.1: Recommended side slopes and canals velocities (Pandey, 2007)

Canal material	**Side slope**	**Maximum recommended velocity for canals (v)**	
Sandy Loam	1.5 to 2.0	0.4	0.7
Loam	1.0 to 1.5	0.5	0.8
Clay Loam	1.25	0.6	0.9
Clay	1.0	0.8	1.0
Stone masonry with mud	0.5 to 1.0	1.0	1.0
Stone masonry with cement	0 to 1.5	1.5	1.5
Concrete	0 to 1.5	2	3.0

Table 3.2: Roughness coefficients for different natural canals (NLCD, 2011)

Land Cover Definition	**Range of Manning's Roughness Coefficient n**
Open Water	0.020 – 0.05
Developed, Open Space	0.030 – 0.05
Developed high intensity	0.12 – 0.2
Baren Land (Rock/Sand/Clay)	0.023 – 0.03
Shrub/Scrub	0.07 – 0.16
Woody Wetlands	0.045 – 0.15
Emerged Herbaceous Wetlands	0.05 – 0.085

Using Equation 3.12, determine the cross-sectional area (A).

$$A = {}^{Q}\!/_{V} \qquad (3.12)$$

Where Q refers to the design flow, choose the side slope (Np) based on Table 3.1. Remember that Np equals h/v, as shown in 8. Np is the ratio of the side wall's horizontal length divided by its vertical height. Using Equations 3.13 through 3.29, one can determine the canal height (Hc), canal bed width (B), and canal top width (T);

$$X = 2\surd\,((1 + N\_p2)) - 2Np \qquad (3.13)$$

$$Hc = \frac{A}{(X + NP)} \qquad (3.14)$$

$$B = X \times HC \qquad (3.15)$$

$$T = B + (2 \times HC \times NP) \qquad (3.16)$$

X is the factor used to optimize the canal shape.

For a rectangular canal N = 0 and X = 2

$$H_C = \sqrt{\frac{A}{2}} \quad (3,17)$$

$$T = B = 2Hc \quad (3.18)$$

The velocity must be under 80% of the critical velocity

The velocity in a long canal must be lower than 80% of the critical velocity to achieve stable and uniform flow.

$$V_C = \sqrt{\frac{A * g}{T}} \quad (3.19)$$

For a rectangular canal

$$V_C = \sqrt{(HC \times g)} \quad (3.20)$$

$$P = B + 2H_C\sqrt{(1 + N2P)} \quad (3.21)$$

For a rectangular canal

$$P = B + 2HC \quad (3.22)$$

Calculate the hydraulic radius (R) as follows:

$$R = A/P \quad (3.24)$$

The slope (SL) can be found from Manning’s equation.

$$S_L = \left[\frac{n_c \times V}{R^{0.667}}\right]^2 \quad (3.25)$$

Now all dimensions required for the construction of the canal are known.

- Calculate the head loss in the canal by:

$$Head\ loss = L \times SL \quad (3.26)$$

L stands for the canal section's length. For Q 500 l/s and for 500 l/s to Q > 1000 l/s, a freeboard of roughly 300 mm and 400 mm, respectively, will be offered. Calculate the size of the largest particle that will be transported through the canal: This is necessary to account for design uncertainties.

$$d = 11\,RS1 \qquad (3.27)$$

Where d denotes the largest particle that could exist, R denotes the hydraulic radius in meters, and S represents the ultimate tensile strength of the material in N/m2.
A possible flood flow in the canal must be checked to ensure that it can be accommodated without exceeding 50% of the freeboard.

Avoid using canals with a width of under 300 mm, as they are more susceptible to blockage. Smaller sizes are also challenging to build, as stone masonry canals are complex to construct.

3.6.1.3 Settling Basin

The primary goal of the Settling Basin is to reduce sediment, which has a detrimental effect on the Min Hydropower System's components. The design's fundamental principles include allowing sediment to settle before it enters the canal and slowing the water speed to prevent it from being carried away. The heads of the basins are typically constructed. To flush the accumulated undesirable sediments, gate valves will be used. Particles larger than 0.2 to 0.3 mm can be settled in the basin (Harvey, 1983). The standard de-sanding basin is shown in Figure 3.7 below.

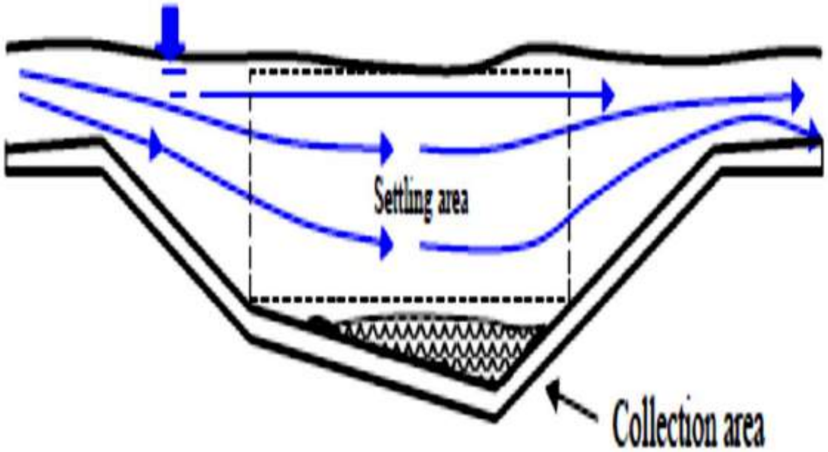

Figure 3. 7: Settling Basin

The procedure is as shown in Equations 3.28 – 3.32

The depth of the basin is given by

$$D = 1.3\sqrt{Q} \tag{3.28}$$

Specific volume of desalting tank

$$Vs = 50.7\sqrt{Q} \tag{3.29}$$

Tank Volume

$$(VT) = VS \times Q \tag{3.30}$$

Length of the desalting is given by;

$$L = \sqrt{\frac{V_T}{4 \times D}} \tag{3.31}$$

The following relationship can calculate suitable basin width (W);

$$W = \frac{L}{4} \tag{3.32}$$

3.6.1.4 Spillway

To manage excess water resulting from flooding and to minimise its adverse impacts on other components of the Mini Hydropower System (MHS), as in Figure 3.8 adequately designed spillways are essential. These structures are specifically engineered to handle overflow conditions by safely redirecting surplus water back to the source. Spillways are commonly integrated into both the de-sanding basin and the forebay, serving as critical safety features that prevent overloading of the intake and downstream systems (Pandey, 2006).

The standard weir equation (Equation 3.33) will be used to calculate the length of the spillway (Lspw);

$$\text{Lspillway} = \frac{(\text{Qflood} - \text{Qdesign})}{\text{Cw (hflood)}^{L}} \tag{3.33}$$

Where hflood denotes the height of the flood level in the canal (m), hsp denotes the height of the spillway crest from the canal bed (m), and Qflood denotes the flood flow via intake (m3/s). Therefore, Hspw = hhn, HOT = Hmf- hhn, and Lspw = Lw. The height of the spillway crest (Hspw) will be aligned to the normal flow surface level or water depth.

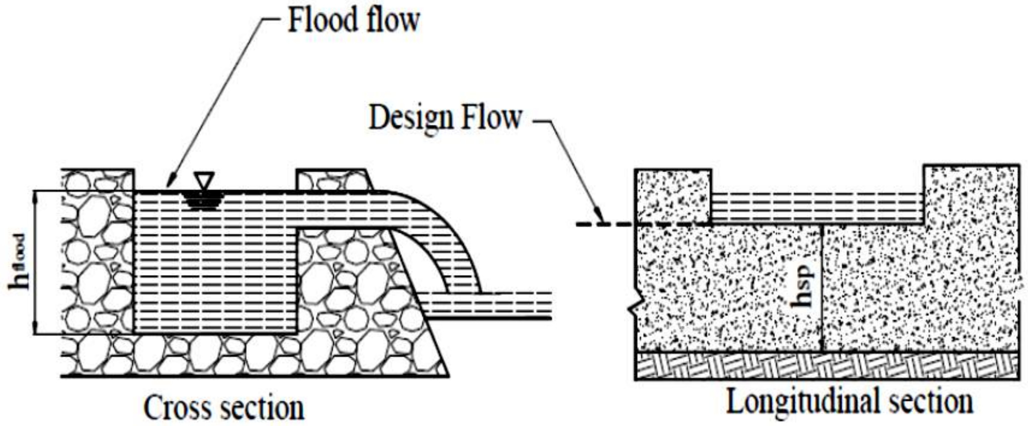

Figure 3. 8: Typical MHS spillway (Pandey, 2006)

3.6.2 Forebay Tank

The forebay tank, as depicted in Figure 3.9, is a pool located at the end of the headrace canal where the penstock pipe draws water. Munyaneza *et al*. (2015) assert that the primary function of the forebay is to minimise air entry into the penstock pipe, which in turn could prevent cavitations (explosion of trapped air bubbles under high pressure) of both the penstock pipes and the turbine. Additionally, as the water speed slows down in the fore bay, it may lead to particle sedimentation, necessitating the previously mentioned spillway construction. Similar to this, Munyaneza *et al*. (2015) suggested installing trash racks to filter out fine sediments before water from the forebay enters the penstock pipes. The fore bay tank is structurally similar to the settling basin, with the exception that the trash rack and entrance into the penstock pipe replace the outlet transition. Equation 3.34 can be used to determine the minimum submergence head for the penstock pipe.

$$h\_s \geq \frac{1.5V_2}{2g} \tag{3.34}$$

Where V_2 represents the penstock's speed

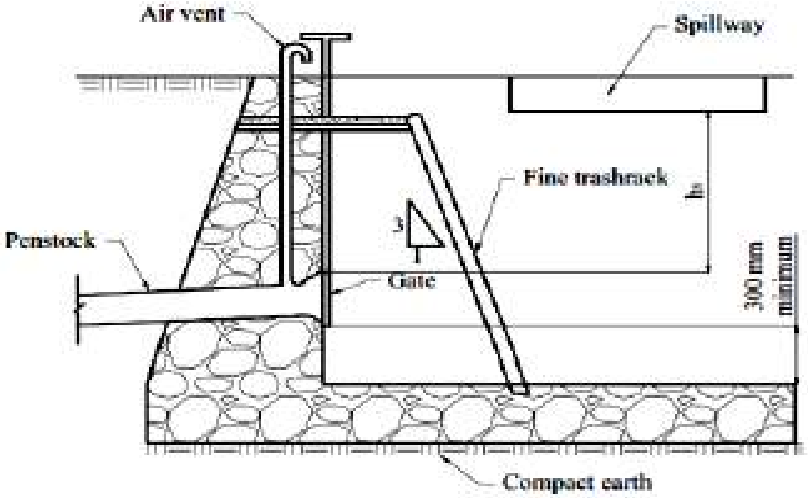

Figure 3. 9: The forebay (Pandey, 2007)

3.6.3 Penstock Pipes

It is one of the most crucial parts of the hydropower system because it is here that the water's potential energy is changed into kinetic energy. Penstock pipes, or close conduit pipes, are essentially pipes that help move water from the forebay tank to the turbine. Steel, high-density polythene, and polyvinyl chloride are frequently used as penstock materials. The penstock typically has a water velocity of 3 m/s and is situated at a slope of more than 45 degrees (Sarasa, 2015). Sliding-type expansion joints are positioned between two consecutive pipe lengths to mitigate the risk of contraction and expansion of penstock pipes resulting from changes in seasonal temperatures. To prevent the penstock from moving in undesirable directions, an anchor block, which is essentially a mass of concrete fixed into the ground, is used. Depending on the environment, the material of the penstock, the ambient temperature, and other factors, penstocks may be installed above or below the ground (ESHA, 2004).

The length of a penstock (L) will be calculated from a topographic map during its design. To minimise head loss, it is good practice, according to ESHA (2004), to consider 5% of the gross head. The typical components of a penstock assembly are shown in Figure 3.10.

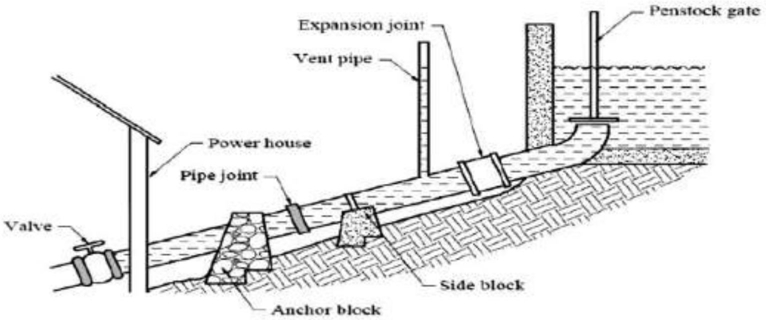

Figure 3. 10: Penstock Assembly Components (Pandey, 2007)

Mild steel, Upvc, concrete, and ductile iron are the most frequently used materials for penstock pipes (Pandey, 2006). Other materials can also be used. As shown in Table 3.3, these have various characteristics.

Table 3.3: Materials for penstock pipe (Pandey, 2006)

S/N	Material	Friction loss	Weight	Corrosion	Cost	Jointing	Pressure
1	Mild steel	♦♦♦	♦♦♦	♦♦♦	♦♦♦♦	♦♦♦♦	♦♦♦♦♦
2	Upvc	♦♦♦♦♦	♦♦♦♦♦	♦♦♦♦	♦♦♦♦	♦♦♦♦	♦♦♦♦
3	Concrete	♦	♦	♦♦♦♦♦	♦♦♦	♦♦♦	♦
4	Ductile iron	♦♦♦♦	♦	♦♦♦♦	♦♦	♦♦♦♦♦	♦♦♦♦

The material type is more advantageous based on the more "stars" it has.

3.6.4 Tailrace

The headrace canal, which was previously covered in this section, is very similar to the tailrace. The only distinction between it and the headrace canal is that it is located at the conclusion of the civil components and is used to return water to its source after it has been used in the micro hydro plant. As a result, the components and specifications for the tailrace will be identical to those for the headrace canal.

3.6.5 Powerhouse Parts

In a powerhouse, water's mechanical energy is transformed into electrical energy. Typically, a powerhouse is composed of electromechanical components, including drive systems, generators, and turbines. The protection of the electro-mechanical equipment is the powerhouse's primary purpose. The majority of small-scale hydropower plants fall into the category of "the above-ground type," "the semi-underground type," and "the underground type" of powerhouses (Pandey, 2004).

Hydroelectricity is produced using either synchronous or induction generators, which are the two main types of generators used. The primary categories of generators used frequently in large-scale power generation are synchronous generators. Induction generators are commonly used when the output power levels are low (generally less than 10 MW). Due to their ability to run at variable speeds with a constant frequency, low cost, and reduced maintenance requirements compared to synchronous generators, induction generators are also the preferred type of generator in MHPs (Micro Hydro Projects) (Kunwor, 2012). Both of these generators have the option to operate either standalone or in conjunction with the grid (Upadhayay, 2009). The kind of generator chosen will depend on the Kala River's potential for hydropower for the study's purposes.

Using the formula below, the frequency of the power network and the number of poles will determine the rated rotational speed for the synchronous generator (Equation 3.35)

$$P = \frac{120f}{N} \tag{3.35}$$

P is for pole count, N is for maximum rotational speed (rpm), and f is for network frequency (Hz).

The original turbine speed and the rated generator speed are chosen, along with either direct coupling or indirect coupling using a power transmission facility (gear or belt), in order to match the proper speed ratio between the turbine and generator. It is also important to consider the combined cost of the turbine, transmitter, and generator. Typically, mini-hydropower plants use 4 to 8 poles to reduce expenses (Pandey, 2004). Based on the number of poles.

Table 3.4: Displays the generator synchronisation speed

Number of poles	**Frequency**		**Number of poles**	**Frequency**	
	50Hz	**60Hz**		**50Hz**	**60Hz**
2	3000	3600	16	375	450
4	1500	1800	18	333	400
6	1000	1200	20	300	360
8	750	900	22	272	327
10	600	720	24	250	300
12	500	600	26	231	377
14	428	540	28	214	257

Note: High-speed generators are smaller and less expensive than low-speed generators, as noted.

3.6.6 Systems of Drives

The primary function of the drive systems is to transfer energy from the turbine to the generators at a constant voltage and frequency while moving in the required direction and at the required speed. Similar to other drive systems, the drive systems in a hydropower system also include the generator shaft, turbine shaft, bearings, couplings, gearboxes, and belts and pulleys. The various drive system types that are frequently used in the MHS (Min Hydropower System) include gearbox drive systems, timing belt and sprocket pulley systems, "V" or wedge belts and pulleys, and direct drive (Kunwor, 2012). A direct-drive system is one in which the turbine shaft and generator shaft are directly joined. The drive systems most frequently used in hydropower systems are "V" or wedge belts and pulleys. The timing belt and sprocket pulley, on the other hand, are commonly used in small systems (less than 3 kW) where efficiency is crucial. When drive belts are ineffective, gearboxes are often the preferred solution for large machines. Gearboxes aren't used as frequently in hydropower systems because of the high costs of maintenance and alignment. Direct, 1:1 coupling of the generator and turbine is the most effective and dependable drive system.

3.6.7 Selection of Turbine

Since the net head of the mini hydropower system at Kala Stream is 598.1m and the design discharge is 1.54 m^3/s, the appropriate turbine for this scheme is the Pelton with an efficiency of 80% to 90% and a rated power capacity of 72378.07kW, according to the turbine chart in Figure 3.4.

Calculation of the Pelton turbine's specific speed, n in revolutions per minute (rpm) for Pelton 1 Jet (ESHA, 2004), is as shown in Equation 3.35:

$$\mathrm{N_S} = \frac{85.49}{(\mathrm{H_n})^{0.243}} \qquad (3.35)$$

The specific speed (Ns), which increases with more jets, grows as the square root of the number of jets. According to ESHA (2004), the Pelton turbine's two (2) jets have a specific speed of 35.32 9. Once the precise speed is known, it is simple to estimate the turbine's rotational speed.

$$N = \frac{N_S \times H_n^{\frac{5}{4}}}{\sqrt{P}} \tag{3.36}$$

Where Ns = Specific Turbine Speed, N = Turbine Rotational Speed, Q = Flow in Cubic Meters per second, and H_n = Net Head in Meters. As the amount of power available is 72,378.07kW, by applying Equation (3.36), the type of generator to be used for generating 9383.946 kW of power can be chosen based on the turbine speed (N) in revolutions per minute. Equation 3.37 yields the number of poles (P).

$$P = 120f/N \tag{3.37}$$

3.7 Summary of Methodology

The methodology used to evaluate the hydropower potential of the Kala River waterfalls combines hydrological analysis, geospatial techniques, engineering design, and community-based assessments of energy demand. It followed a structured multi-step approach:

3.7.1 Site Selection and Head Estimation

The location of the weir and sediment tank was identified using GPS data, focusing on a stable, straight river reach with gentle slope and suitable ponding. Elevation measurements were taken to estimate the gross and net head, which are critical for assessing energy potential.

3.7.2 Flow Rate Estimation

Since the Kala River is ungauged, the study relied on data from the nearby gauged Kiwira River. The area ratio method was applied to estimate flow rates by comparing catchment areas with similar hydrological characteristics. Field surveys were also conducted using a current meter to measure water velocities at different depths, and the cross-sectional area of the river was calculated to determine the actual flow.

3.7.3 Hydrological Correlation and Flow Duration Curve

Statistical correlation analysis, specifically Karl Pearson's coefficient, was used to verify the hydrological similarity between the Kala and Kiwira Rivers. Using Kiwira’s long-term flow data, a flow duration curve was developed to understand flow variability and ensure system reliability under different hydrological conditions.

3.7.4 Hydropower Potential Assessment

The hydropower potential was calculated by integrating the net head, flow rate, and an assumed turbine efficiency. This helped estimate the total theoretical power output that could be generated from the Kala River site.

3.7.5 Turbine Selection

Based on the site's high head and moderate flow, a Pelton turbine was identified as the most appropriate choice. Turbine selection was guided by standard turbine charts, accounting for efficiency, head, and discharge characteristics.

3.7.6 Electricity Demand Analysis

Electricity demand for six villages within Kisondela Ward was assessed using a load and energy forecasting method. Data was gathered through household surveys, interviews, and focus group discussions to estimate current and future energy needs across residential, commercial, and institutional sectors.

3.7.7 Design of Civil and Hydraulic Infrastructure

Technical designs were prepared for critical components of the proposed mini-hydropower plant, including the head works, forebay tank, headrace canal, penstock, settling basin, and spillway. These designs considered site-specific topographic and hydrological data to ensure optimal performance, structural integrity, and minimal environmental impact.

This methodology provides a robust basis for evaluating the Kala River's hydropower viability, with the results presented in the following section to inform decision-making on energy infrastructure development.

CHAPTER FOUR
RESULTS AND DISCUSSION

4.0 Introduction

This section comprises three sections, the first section presenting information on the investigation of the hydrological potential of KWF, the second section analysis the power demand of surrounding villages and the last section provides information on the preliminary design of the anticipated mini-hydropower plant for KWF. The results of Karl Pearson's coefficient of correlation analysis and distinct mathematical analysis applied to different factors as outlined in the other sections presented in the Appendix.

4.1 Investigation of the Hydrological Potential

4.1.1 Hydrological Analysis

The longitudinal profile of the river segments and the mean river parameters including the cross-sectional area, were determined using rise and fall methods and land traversing techniques in the hydrological analysis. Tables 3.3 and 3.4 display the flow rate, and Figure 3.10 shows the longitudinal profile of the Kala River from which the weir elevation and power house elevation were determined.

4.1.2 Hydraulic Head of Kala River

The graphical presentation of the relative position of the land features on the earth's surface or under the water surface, shown in Figure 3.10, involves the rise and fall of the elevations. The results, when viewed from 2300 meters above the shoulder of Kapologwe waterfalls, demonstrate a fall towards the waterfalls' shoulder. The highest elevation of 988.85 m and the lowest elevation (at the waterfalls' shoulder) of 309.27 m, as shown in Figure 4.1, demonstrate a hydraulic head of 679.58 m, which can provide energy to run an installed turbine. These findings imply that variations in head, which characterise the potential energy of flow, are what cause water to move. In this case, changes in gravitational potential energy resulting from elevation changes are what cause the flow. In other words, the water moves from an elevation of 629.58 m to a low potential energy.

But this only represents the gross head, from which the net head is calculated as the gross head less 5% (of the gross head), or 598.1 m. Berrada (2019), ESHA (2004), and Pandey *et al*. (2015) employed land surveying methods to determine site characteristics, provide additional support for this strategy. The data was used to calculate the ideal net mechanical power, the net head of the plant, and the pressure losses. These latter ones were used to dimension the plant's conduit, turbine, and generator in accordance with market standards and supply availability.

4.1.3 Discharge

4.1.3.1 Dry Season Flows

The discharge or flow rate was calculated using Equation 5 above**.** The mean measurements for the dry season (October and November) are shown in Table 4.1. Since the cross-section of the flow was small, the velocities were taken at 0.6% of the subsection depth (across the flow). This is illustrated as $0.6D_1$ - $0.6D_3$ for the three segments, along the cross-section as shown in Table 4.1.

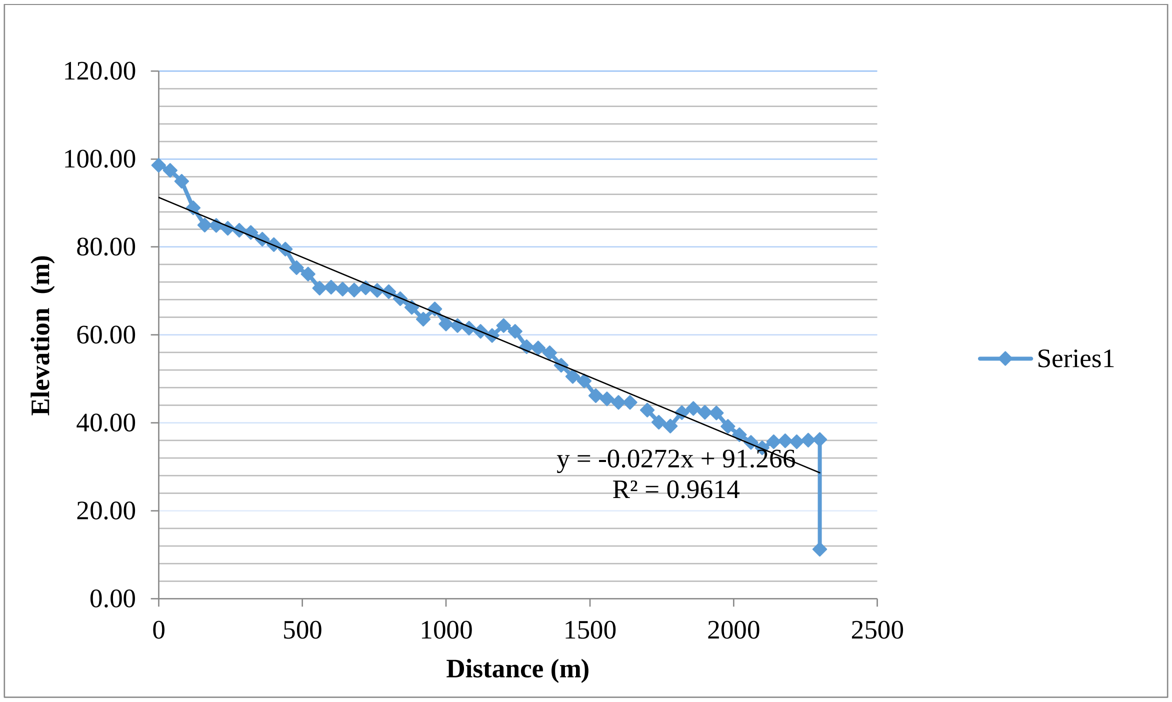

Figure 4. 1: Longitudinal Profile of Kala River

Furthermore, the results in Table 4.1 show that the average velocity is 0.45 m/s, the average depth is 0.88 m, and the average width is 5.14 m. Therefore, the calculated cross-sectional area of the flow, as obtained from Equation 3.4, is 4.53 m^2, and the calculated flow rate from Equation 3.5 is 2.02 m^3/s. The results are shown in Table 4.1.

These results show that the highest flow rate is observed in sections 5 to 6. The flow in section one is 1.77 m^3/s, and that in section 10 is 1.88 m^3/s, which denotes an increase of 6.2%. However, within the river terrain, in sections 5 and 6, the flow rate increased further to 2.77 m^3/s and 2.79 m^3/s, respectively, as these sections were the deepest. This implies an increase from 56.5 to 57.6%. There is a high likelihood that the cause of this increase is due to the increased gradient and, consequently, velocity in this area. Figure 4.1 shows that the elevation in this area is 596.45 m, and at 1360 m, the elevation is 490.89 m, resulting in a distance of 160 m, which yields a gradient of 8%. Moreover, the area comprises bedrock.

4.1.3.2 Wet Season Flows

By using Manning's equation to estimate mean flow velocity and roughness in natural channels, based on the characteristics of the river, it was found that natural channels with Shrub/Scrub (Table 3.4) have a selected ρ value of 0.07-0.16, so the chosen value is 0.15. The lowest and highest cross-sectional areas are 8.23 and 10.35 m^2, while the derived minimum and maximum velocity of flow are 2.70 m/s and 3.27 m/s, respectively, as detailed in Table 4.2. Similarly, the mean discharge for both the dry and wet seasons is 15.24 m^3/s, as determined using Equation 3.1. These results suggest that it is possible to estimate wet season flows by examining the physical features of the river. Furthermore, the range of wet and dry season flows is 26.43, which is 13 times more than that of the dry season.

Table 4.2: Wet season velocity, cross-sectional area and discharge

SECTION	Mean Depth(m)	Mean Width(m)	Area (m^2)	P	R=A/P	$R^{2/3}$	$S^{1/2}$	N	V (m/s)	Flow (m^3/s)
1	0.89	9.25	8.23	11.03	0.75	0.9	0.5	0.15	3.00	24.7
2	0.85	9.7	8.25	11.4	0.72	0.81	0.5	0.15	2.70	22.3
3	1.02	9.6	9.79	11.64	0.84	0.89	0.5	0.15	2.97	29.0
4	0.95	9.45	8.98	11.35	0.79	0.86	0.5	0.15	2.87	25.7
5	0.98	9.95	9.75	11.91	0.82	0.98	0.5	0.15	3.27	31.9
6	0.99	8.9	8.81	10.88	0.81	0.98	0.5	0.15	3.27	28.8
7	1.05	9.15	9.61	11.25	0.85	0.95	0.5	0.15	3.17	30.4
8	0.89	9.95	8.86	11.73	0.75	0.93	0.5	0.15	3.10	27.5
9	1.04	9.85	10.24	11.93	0.86	0.91	0.5	0.15	3.03	31.1
10	1.04	9.95	10.35	12.03	0.86	0.96	0.5	0.15	3.20	33.1
$Mean_{Wet}$	**0.97**	**9.89**	**9.29**	**11.51**	**0.81**	**0.92**	**0.5**	**16.95**	**3.06**	**28.45**

The overall mean dry and wet season average discharge = ½ x (2.02 + 28.45) =**15.24** m^3/s

4.1.4 Correlation Analysis

Since the Kala River is ungauged, there was a need to prove if the river has the same hydrology as a gauged river in a similar environment.

Table 4.3: Average monthly flows of Kiwira River for 10 years (m3/sec)

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
2012	0.40	5.17	35.48	49.57	57.92	44.33	23.16	13.80	9.20	6.87	5.28	4.264
2013	2.94	9.15	38.49	36.46	26.42	39.62	17.85	11.86	9.05	7.45	5.21	4.909
2014	5.66	8.49	14.30	13.79	40.93	42.02	18.01	10.78	8.58	7.03	6.32	5.443
2015	5.60	12.00	25.59	26.57	31.17	25.79	11.67	8.56	7.20	6.26	5.05	3.557
2016	1.70	1.82	5.50	16.99	20.69	27.91	17.99	12.17	9.21	7.77	6.14	3.741
2017	1.97	38.52	55.98	58.22	54.58	38.85	24.67	16.87	11.72	9.69	7.92	6.312
2018	4.86	15.80	30.68	58.07	56.74	54.97	34.05	20.91	13.68	9.73	5.73	3.741
2019	3.82	15.24	24.92	28.48	44.79	47.21	21.81	12.44	9.31	7.06	5.30	3.825
2020	6.56	8.00	30.34	30.92	43.41	33.01	16.08	11.19	8.92	7.21	5.39	3.288
2021	2.25	3.83	9.91	14.21	19.38	28.75	15.33	10.92	8.60	7.08	5.43	4.802
Mean	**3.58**	**11.80**	**27.12**	**33.33**	**39.60**	**38.25**	**20.06**	**12.95**	**9.55**	**7.61**	**5.78**	**4.388**

Source: REN21, (2014)

Results in Table 4.3 show the average monthly flows for the gauged Kiwira River, which is in the same environment. Appendix 3 presents the correlation analysis results for the Kala and Kiwira rivers. As it is known, Kala is a tributary of the Kiwira River, while the Kiwira River and the Songwe River both pour their water into Lake Nyasa.

The results show that the peak monthly flow of 39.60 m^3/s is experienced in the month of March, and the lowest mean flow of 3.58 m^3/s is observed in the month of November for Kiwira River. Table 4.3, Figure 4.2, and Table 4.4 show the average monthly flows of the Kiwira River and the rainfall (mm) for the Kala catchment.

The data in Table 4.3 on monthly flows for the past decade show that the stream is perennial, thus water flows throughout the year. This explains that there are high chances that streams flowing in the exact geographical location and catchment have similar characteristics. In October and November, the river flows are at their lowest volumes, with a mean of less than 4.4 m^3/s.

For July to November, the standard deviation results (Appendix 5) are relatively close to the corresponding means of 9.55, 7.61, 5.78, 4.388, and 3.58. This indicates that the variance of the results is generally slight.

In addition, Figure 4.2 demonstrates that there is little to no rainfall between June and October. The peak period for rainfall is from February to April. The stream flow is also at its lowest and highest points during the same months. These findings demonstrate the relationship between rainfall and stream flow, showing that, between January and October, increased rainfall results in an increase in flow volume, and vice versa. The rate at which the rainfall increases, however, exceeds the extent of the increased water flow in the stream, and this trend is most pronounced between the months of January and November. This may be because the rainy season is at its most active during this time. Due to the fact that this time frame follows the dry season and that the Poroto Mountains are the source of tributaries that feed the Kala and Kiwira rivers, adequate time is needed, among other things, for infiltration, deep percolation, saturation, and the formation of runoff from the catchment areas into tributaries of the Kala and Songwe rivers.

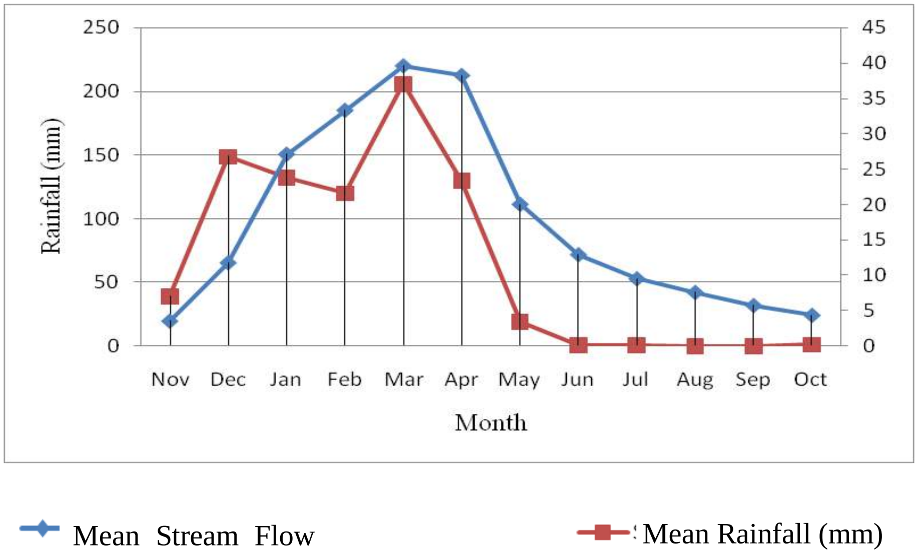

Figure 4. 2: Comparison of monthly Rainfall (mm) against stream flow (m3/s) for the Kiwira River

In order to determine whether gauged and ungauged points are correlated, data were computed using Karl Pearson's coefficient of correlation (r), as shown in Table 4.4 and the corresponding mathematical equation. The correlation coefficient is 0.78 based on the findings. This explains why the gauged Kiwira and Kala Rivers have a direct correlation.

The findings of the correlation analysis of the Kala River and Songwe River are presented in Table 4.4. According to Kentel and Ergen (2015), streams with comparable conditions have r values between +1 and -1, where +1 denotes a perfect direct correlation. Due to their similar hydrology, the findings in Table 4.4 demonstrate the correlation between the Kala River and the Kiwira River. Gang *et al*., (2015), Archfield and Vogel (2010), and Kentel and Ergen (2015) provide evidence in support of findings regarding the correlation of two streams in comparable ecological zones. Due to the fact that the Kiwira River and Kala River flow through the same area, there is a good chance that their environmental influences on behaviour will be similar.

Table 4.4: Calculation of Karl Pearson’s coefficient of correlation (r)

X	Y	X^2	Y^2	XY
27.1	132.3	735.5	17503.3	3588.0
33.3	120.0	1110.9	14400.0	3999.6
39.6	206.0	1568.2	42436.0	8157.6
38.3	130.0	1463.1	16900.0	4972.5
20.1	19.4	402.4	376.4	389.2
13.0	0.8	167.7	0.6	10.4
9.6	0.8	91.2	0.6	7.6
7.6	0.3	57.9	0.1	2.3
5.8	0.0	33.4	0.0	0.0
4.4	1.3	19.3	1.7	5.7
3.6	39.4	12.8	1552.4	141.1
11.8	149.0	139.2	57360.3	1758.2
Sum		**5801.6**	**150531.3**	**23032.1**

$$r = \frac{23032.1}{\sqrt{(5801.86)(150531.32)}} = 0.78$$

4.1.5 Curve of Flow Duration

The flow duration curve depicted in Figure 4.3 and the information shown in Appendix 2 were created using the average monthly flow data. According to the findings (Figure 4.2), the flow rate decreases as the percentage time increases. Similarly, as the percentage of days the stream flows decreases, the flow rate increases. The daily mean flow duration values presented for the month of November have a minimum flow of 1.54 m^3/s and a 92.31%-time interval, which has been exceeded, according to the statistical analysis of the stream flow duration. These results indicate a high flow that has been exceeded on 92.3% of the flow record's days, or the probability of an exceedance, which denotes a low flow condition in the stream. This indicates that 1.54 m^3/s or more is present in 92.31% of all daily mean flows in the record.

Based on these findings, 7.69% of the total daily mean flow in the records may correspond to the probability of obtaining stream flows less than 1.54 m^3/s. The discharge used to determine the hydropower potential was 1.54 m^3/s.

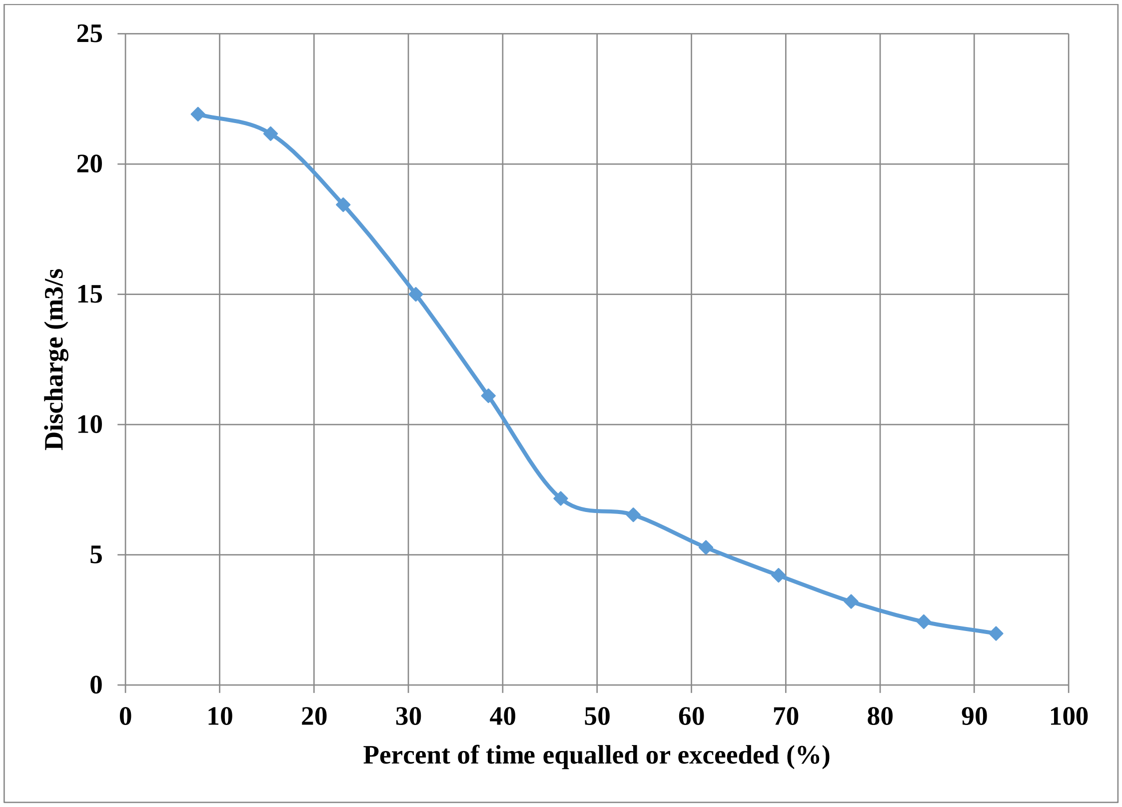

Figure 4. 3: Flow duration curve for the Kala River

These findings also demonstrate that the likelihood of the stream in the Kala River continuing to flow is high, but that the probability decreases with the proportion of time that its standards have been met or exceeded. Therefore, there is a good chance that the anticipated flow for Kala River hydropower production will decline over time. The findings indicate that the design flow should be limited to less than 100% of the time if the Kala River hydropower system is to be independent of any other energy or utility backup; otherwise an additional source of energy will be needed to supplement this source.

4.1.6 Selecting a Turbine

Using standard charts, the type of turbine, its efficiency, and its power capacity were determined, given the head and the design flow rate of the hydropower system. The Pelton type of turbine, with an efficiency of 80–90% and a rated power capacity of 7237.8 kW, is the appropriate turbine for this scheme because the net head of the mini hydropower system at Kala stream is 598.1 m and the design discharge is 2.02 m^3/s. These findings are related to the normative charts for small hydro turbine selection put forth by DHP (2017) and HRET (2017). Sangal *et al*. (2013) evaluated the effectiveness of using turbine selection charts to choose the type of turbine and its corresponding power output. The study supported the effectiveness of standard charts in determining the most suitable types of turbines. The Pelton turbine is suitable for hydropower generation at the Kapologwe waterfalls, according to these results.

4.1.7 Potential for Hydropower

The potential weir, sediment tank, and power house are shown in Table 4.5, along with their locations and elevations. The elevation of the power house is 309.27 m, while the diversion point in the Kala River is 946.27 m. Due to the mathematical difference between the elevation of the diversion site (upstream) and the power house, these data show that the gross head of the Kala River hydropower site is 637 m.

Table 4.5: Latitude, Longitude and Elevation for the location of the hydropower system

Location	Latitude South	Longitude East	Elevation (m)
Potential Weir	93 885 94	33 610 010	946.27
Sediment Tank	93 864 77	336 09 860	935.42
Potential Power house	93 919 55	336 128 16	309.27
Foot of Dam	93 932 52	336 13 207	87.59
Dam Shoulder	93 930 98	336 13 250	327.59

Pandey (2015) also recommends a 5% head loss as the net head loss. The net head for the system is 598.1 m when the 5% head loss is taken into account. Therefore, based on the mathematical calculation, the available power potential is 7237.9 kW.

4.2 Power Demand of Neighbouring Villages

4.2.1 Potential Consumers and Power for Various Loads

The Ward has a population of 12,200 people, with six village offices and four primary schools having the most power users. Additionally, the ward lacks garages. Despite the large number of consumers, there is a chance that consumer groups will use more energy. The total power load demand for the six villages, as analysed in the results, is 1728.36 kW, with households having the highest load at 1586 kW. Similar to this, commercial establishments experience the lowest load, with a total load of 36.57 kW, as shown in Table 4.6.

Table 4.6: Average power for various load classifications in Kisondela ward

Classification of Load	Category	Kw Per User	Consumers	Load (kW)
Household		0.13	12,200	**1586**
Education	P/School	1.58	4	6.32
	S/School	24	2	48
	Subtotal	**5.85**	**12,206**	**54.32**
Commercial	Tailoring	1.45	1	1.45
	Carpentry	2	1	2
	Grain Mills	15	2	30
	Garage	2.25	0	0
	S/Restaurant	0.12	2	0.24
	Bar	1.14	1	1.14
	Shops	0.16	2	0.32
	S/Workshop	1.42	1	1.42
	Subtotal	**1.83**	**10**	**36.57**
Public Service	Mosque	0.17	1	0.17
	Churches	0.43	1	0.43
	W/Pump	33.33	1	33.33
	Go down	0.5	1	0.5
	Office	2.65	6	15.9
	Dispensary	0.38	3	1.14
	Subtotal	**1.75**	**13**	**51.47**
	TOTAL			**1728.36**

URT (2015)

4.2.2 Potential Consumers and Power Demand for Various Loads

Table 4.6 indicates that after four years, the combined load for the six villages of Kisondela, Bugoda, Lutete, Kibatata, Ndubi, and Isuba will be 8641.80 kW. These findings help to explain why each of the six villages has an average power demand of 1440.3 kW. The power demand projections for these villages will be compared with the power potential of the Kala River to determine the viability of this potential. The Kala River has the potential to provide hydroelectric power to the communities living in the villages within its catchment.

Menezes (2013), employed a similar technique to determine power demand and compared the results with other methods, including the sampling of monitored data and a "bottom-up" approach to estimate likely power, supports the methodology used in assessing this power demand and supply. These findings rely on an accurate assessment of the catchment's hydropower demand.

4.2.3 Hydropower Potential

Hydropower potential means: an amount of water (flow) which flows down a certain height. To utilise such, the produced electricity is to be transported by power line to potential users. Hydropower offers a significant potential for renewable energy production.

In other way, the hydropower potential is calculated by multiplying the gravitational constant by the flow rate, hydraulic head, water density, and desired turbine efficiency.

4.2.4 Selecting a Turbine

The type of turbine as depicted in Figure 3 is determined by the site data, including the head and flow discharge, through the use of established guides (DHP, 2017; HRET, 2017). Instead of using measurements obtained over a short period of field measurement, estimated flow data from ten years was used, which is more reliable.

Pelton turbines were chosen as the most appropriate turbine type for the identified field features, with a flow rate of 1.542 m^3/s and a net head of 598.1 m, as shown in Figure 1. According to the standard selection chart, Pelton turbines have a flow rate of 0 to 2 cumecs and a head range of 30 to 1000 m.

4.2.5 Engine Effectiveness

Adejumob and Shobayo (2015)'s Appendix 1 show the trend of turbine efficiency for various flow rates, from which the turbine efficiency was obtained. Based on this, the Pelton turbine is capable of 95% efficiency.

Peak efficiency was found to range between 80 and 90% in several studies (Gupta *et al*., 2014; Cobb and Sharp, 2013, and Adejumob and Shobayo, 2015). Because of this, the current research has chosen to use an efficiency of 80% (0.8).

The hydropower potential (P) was found to be 7237.9 kW by using Equation 3.8. Based on the results above, thus 7.23MW Comparing Hydropower Potential and Power Demand

The development of a decentralised power system in the area was facilitated by matching the hydropower potential with energy demand. Figure 4.4 compares the trend in power demand with the Kala River's power potential. According to the findings, the cumulative power demand for the six villages in the sub catchment is rising at an increasing rate, as shown in Table 4.6, increasing from 8641.80 kW in the first four years to 6608.9 kW and above in the succeeding years. The power demand, however, exceeds the Kala River's (7,237.9 kW) capacity. These findings imply that the Kala River's capacity to provide electricity to the six nearby villages for a duration of 1-4 years is insufficient. There is a good chance that, even with a suitable installed turbine, the energy needs of the six villages will not be met by the Kala River's potential for three to four decades.

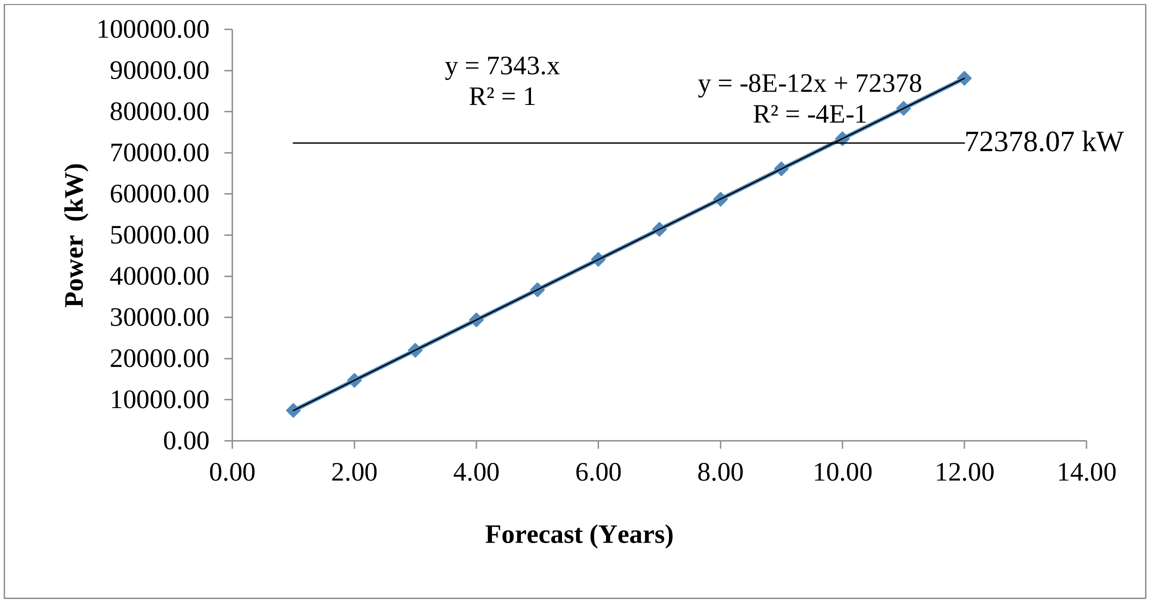

Figure 4. 4: A forecast of power demand versus power potential for Kala River

In addition to this, analysis was conducted to determine which size of community would be best served by the 7237.9 kW of available power potential. Results in Appendix 6, Appendix 7, and Tables 4.7 and 4.8 indicate that the estimated power potential demand for five of the six villages in the Kala catchment—Kisondela, Bugoda, Lutete, Ndubi, and Isuba is 7116.85 kW. The power potential of Kala waterfalls is 121.05 times greater than this power demand. These findings imply that the Kala River and Kapologwe Waterfalls' power pot entail is sufficient to provide electricity to the five villages in the catchment.

Table 4.7: Forecast for the five villages' average power demand for 1- 4 yrs

Years	**Classified Load**	**n**	**Γ**	**D (Load-kW)**	**(1+γ)**	**Total Load (kW)**
1 – 4	Residential customers	4	0.25	1586	1.25	7,930.00
	Education	4	0.25	54.32	1.25	271.60
	Commercial	4	0.25	36.57	1.25	182.85
	Public Services	4	0.25	51.47	1.25	257.35
Total load after 4 years						**8641.80**

Table 4.8: Forecast for the five villages' average power demand for 36 yrs

Years	**Classified Load**	**n**	**Γ**	**D (Load-kW)**	**(1+γ)**	**Total Load (kW)**
For γ = 0.04, 0.03 and 0.02						
36	Residential customers	36	0.04	54.32	1.04	2033.74
	Education	36	0.03	1640.32	1.03	60,823.07
	Commercial	36	0.02	36.57	1.02	1342.85
	Public Services	36	0.02	51.47	1.02	1,889.98
Total load after 36 Years						**66089.64**

4.3 Design of Orifice for Side Intake

Hydropower Plant's Proposed Initial Design for the Kapologwe Waterfalls, civil structures, mechanical components, and electrical components are typically the main components of any mini hydro power plant. The prevailing field conditions dictated the design of the subcomponents of civil structures. The designs for the side intake, headrace canal and spillway, settling basin, fore bay tank, and penstock civil structures are presented in this section. Additionally, a suitable turbine and generator have been selected. Figure 4 illustrates the main components.

4.3.1 Design of Side Intake Orifice

The study provided a suitable plan for the intake orifice, and sketch drawings, to demonstrate how the side intake orifice will appear once it is constructed. The design flood level, the orifice's area (width and height), and the discharge moving through the orifice are crucial factors to consider. Equation 4.1 was used to determine the flood level. Where hf is the above normal flood level and hr is the normal flood level, both of which are 0.6 meters. Therefore, 1.0 m is used to calculate the design flood level (Hd) for the side intake orifice.

$$H_d = h_r + h_f \tag{4.1}$$

The velocity through the orifice ranges from 1.0 to 1.5 m/s, giving a mean velocity of 1.25 m/s, according to AHEC (2011). The recommended velocity (Vo) for the current study was set at 1.2 m/s. Using this foundation, it is now possible to determine the area of the orifice by using Equation 3.12 as 1.649 m^2. For a rectangular orifice, the area is equal to the sum of the orifice's height and width. If the orifice's height (ho) is set to 0.2 meters, its width (w) will be 8.245 meters. Let the orifice's bottom be 0.2 meters above the riverbed's level. Thus, the bed load will be reduced. Set the headrace canal water level to hc = 0.4 m in relation to the datum as depicted in Fig. 4 (To confirm the submerged state). Since the masonry orifice

A schematic representation of the intake at the point of divergence, where the orifice is situated, as shown in Figure 5. In this case, using the orifice as a fixed throttle to produce head loss is essential. In addition to limiting the flow discharge to the design, the orifice's head loss also affects the flow meter’s ability to calculate flow rates in terms of volume or mass.

The following are the flood conditions and the flow discharge through the orifice which was found to be 4.0740 m^3s^{-1} by using Equation 3.33 of flood therefore equals 4.0740m^3/s. Since the orifice can only take 1.979 m^3/s, this value is crucial when designing the spillway.

It is necessary to build the intake structure upstream of the waterfalls' shoulder at Lat 93 885 94 S and Long 033 610 010 E. The side intake's orifice is constructed to accommodate a flow of 1,979 m^3/s. The orifice has a width of 8245 mm and a height of 4000 mm. It is advised that a trash rack be added to the catchment area due to the state of the area to stop debris from flowing into the power canal.

4.3.2 Head Race Canal Design

The type of canal selected was stone masonry with cement mortar, and the survey data's calculation of the design discharge yields is 1.979 m^3/s. The decision to use this material was made because it is recommended in situations where porous soil is present, as is the case with the Kala dam site. Using other materials, such as earthen or stone, could have resulted in water seeping through the canal surface, potentially causing landslides in the vicinity. Tables 3.1 and 3.2 above provide the side slope and roughness coefficients for various types of canals. In this case, the side slope for stone masonry with a cement mortar is Np = 0.5, (1h/2v), where V is equal to 0.9 m/s. The chosen roughness coefficient, n, is 0.2 because the head race design will be developed with high intensity (Table 3.2). The cross-sectional area of the canal and spillway evaluated using Equation 3.12 was 2.198 m^2 for the discharge of 1.979 m^3.

The value of X is 1.236; thus, using Equation 3.13, the initial water depth (H) of the canal was calculated based on this factor, yielding a depth of 1.094 m. Also, Equation 3.14 was used to calculate the wetted bed width in the canal (B) was found to be 1.352m

Equation 3.15 was then used to determine the top width of the canal (T) given the values for the slope (Np), water depth (Hc), and canal bed width (B), was found to be 2.446 m.

Consequently, the Top width is 2.446 meters from the design water level.

The water velocity in the headrace canal must be 80% less than the critical velocity in order to guarantee a stable and uniform flow. 0.8Vc was checked using the Equation 3.16 and found to be 2.969

As a result, the water's speed in the headrace canal, 2.199 m/s, is less than 80% of the critical speed, 2.375 m/s. Therefore, it can be said that the headrace canal's design is suitable.

Since 2.375 is greater than 2.199, the water's speed in the headrace canal is less than 80% of the critical speed (2.375 m/s), which is 1.8 times the speed of the water (2.199 m/s). Therefore, it can be said that the headrace canal's design is suitable.

After determining the wetted perimeter of the headrace canal, as calculated by equation 3.21, the perimeter was found to be 4.658 m which the design is deemed acceptable.

Wetted perimeter is equal to 4.658 m. Equation 3.24 was used to determine the hydraulic radius "R" based on the wetted perimeter, which was found to be 0.472 m.

Consequently, R = 0.472 m is the hydraulic radius.

Lastly, Equation 3.25 output for the canal's bed slope is 0.000637

For normal masonry using cement mortar, the roughness coefficient is nc, which equals 0.017 (Pandey, 2007). The specified slope (SL) of the required canal bed is 0.000637. Finally, the dimensions of the canal are known. This freeboard allowance of 300 mm is recommended.

4.3.3 Layout of the Spillway

The foundations of the basin and penstock must be kept away from water that exits the flush gate. Otherwise, the installations' supporting soil will be washed away. The spillway drain should have walls and be paved.

Water may flow at a rate of up to 4.0740 m^3/s during peak seasons, which is higher than the design flow of 1.979 m^3/s (Qdesign). The time of the flood (Qflood). When the orifice for side intake was calculated (using Equation 3.33), the Qflood was established. By design, the hover top is assumed to be 100 mm, the weir coefficient (Cw) is 1.6, the flood velocity (Qflood) is 4.0740 m/s, and the design velocity (Qdesign) is zero m^3/s.

Equation 3.33, the conventional weir equation, was used to determine the length of the spillway (Lspw); and found to be 3.268 m. as in Appendix 4. As a result, the normal flow surface level or water depth will be aligned with the height of the spillway crest (Hspw);

Case II

In this case, Cw = 1.6, Qflood = 4.0740 m^3/s, Qdesign = 1.979 m^3/s, and Hovertop = (hflood - hsp) = 50 mm.

The spillway's length in this instance is 4.0740 - 1.979 divided by 1.6 (0.5)1.5 to equal 1.68 meters.

Therefore, a spillway with a length of 3.268 meters is needed to satisfy both of these requirements. It is now necessary to check the head loss in the power canal. According to the preliminary design survey, the headrace canal's length (L) is 109 m, and the slope (SL) was calculated, and it has a value of 0.0019846.

The head loss was found to be 0.21632 m using Equation 3.26. Therefore, the result for the power canal's head loss is 0.13189 m.

The size of the largest particle (D) that can move through the canal at a speed of 0.9 m/s must now be determined. Equation 3.27 was used because it was required, it is undesirable for particles larger than a certain size to pass through the canal.

A particle traveling at 0.9 m/s will have a diameter of 0.0149 m when it enters the canal. The headrace canal would act as a holding area for particles larger than 14.9 mm. The gravel trap must be made to remove all particles larger than 14.9 mm in order to prevent deposition upstream of the settling basin.

The materials for the power canal were selected based on the site's geology, topography, and availability of those materials. For the headrace canal, stone masonry with cement mortar was chosen because mud mortar type would have had to lead seepage of water from the canal, which would have eventually caused landslides, which is not what was intended. The headrace canal must be approximately 109 meters long, with the length chosen to maintain the gross head of 604.2 meters. The headrace canal has a trapezoidal shape, with a depth of 1094 millimetres and a freeboard of 300 millimetres, a bed width of 1352 millimetres, and a spillway length of 3268 millimetres, as shown in Figure 4.3.

Settling Basin (4.5.3)

Using Equation 3.28, the depth of the basin was found to be 1.829 m

Equation 3.29 was used to calculate the precise volume of the de-silting tank and found to be 140.59 m^3.

Equation 3.31 was used to calculate the length of the de-silting basin, which was found to be 4.383 m. Similarly, Equation 3.32 was used to determine another parameter, yielding a value of 1.0959 m.

According to site space and conditions, a design discharge of 1.979 m^3/s was created, and the settling basin was created. The settling basin's suggested dimensions were 1959 mm in width, 43836 mm in length, 900 mm in total depth, and 260 mm in storage depth.

4.3.4 Penstock Pipe Design

Surface roughness, design pressure, jointing technique, weight and ease of installation, site accessibility, terrain, soil type, design life and maintenance, weather conditions, availability, comparative cost, and likelihood of structural damage are all factors to consider.

The material for the penstock pipe was selected as the first step in the design process. Mild steel is considered the best material for this application because it can withstand the pressure and velocity found inside the pipe, as well as its diameter. The mild steel chosen for this power plant's material types has a design diameter of 630 mm, a length of 627 m, and an average pipe velocity of 3 m/s. The estimated 31.8 m head loss is the total. The acceptable head loss for a mini-hydropower plant is between 5% and 10%.

As a result, the penstock's standard diameter is 0.529 m. The pressure informed the choice of penstock material.

4.3.5 Calculating the Pipe's Thickness

This penstock is made of mild steel, which has an ultimate tensile strength of 350×10^6 N/m^2.

The pipe thicknesses are calculated using Equation 36 as follows:

For a pipe with an internal diameter and an internal pressure P, the penstock’s pipe thickness t was found to be 12 mm.

P_w=P_a+ρgh

Where Pw is water pressure, Pa is atmospheric pressure at the water's surface, ρ is water density, g is the acceleration of gravity, and h is the height of the water column. Hence water pressure was found to be 6349460 N/m^2

4.5.5 Fore Bay Tank Design

The fore-bay tank's submerged head (hs), as determined by Equation 3.34, was found to be 0.688 m. Therefore, the fore-bay tank's submergence head should be greater than 0.69 m. In this instance, the storage depth beneath the pipe is 300 mm; however, in some cases, it may be equal to the pipe diameter. In such cases, the larger of the two is advised. The air vent (d) needs to be planned because a gate that closes quickly can result in a vacuum inside the pipe, potentially causing it to collapse. The air vent (d), consequently, an air vent has a diameter of 10 mm.

4.3.6 Turbine and Generator Design

Calculation of the Pelton turbine's specific speed, (n (rpm)), using Equation 3.35 was found to be 18.03385

The specific speed (Ns), which increases as the square root of the number of jets, increases as the number of jets increases. Two (2) Pelton turbine jets have a specific speed of 35.32 (ESHA, 2004). The rotational speed of the turbine can be easily estimated once the specific speed is known. Equation 3.36, gives the turbine's rotational speed of 1243.64 rpm which describes the power demand of 7237.81kW.

It is possible to choose the type of generator to use for the power generation of 7237.81 kW based on turbine speed. In Equation 3.37, the number of poles (P) was found to be 5 poles'

According to the power network's frequency (50 Hz), the rated rotational speed is specified, and the generator has 5 poles. The harmonised speed for a frequency of 50Hz and 6 poles is given as 1000rpm based on the standard Generator synchronisation speed (Pandey, 2004), as shown in Table 3.2.

The Pelton type turbine, with an efficiency of 80 to 90% and a rated power capacity of 7,237.07kW, is the best choice for this scheme based on the turbine chart in Figure 3, because the net head of the small hydropower system at Kala Stream is 598.1m and the design discharge is 2.02 m^3/s. These findings are related to the standard charts for choosing small hydroelectric generators that DHP (2017) and HRET (2017) have proposed.

The required generator has 6 poles, a speed of 1000 rpm, and a power rating of 7237.9kW. The Kala River has a net head of 598.1 meters and a design flow of 2.0 m^3/s. The Pelton turbine is the appropriate type of turbine for this project, as indicated by the turbine chart in Figure 3.4. The chosen generator features a direct drive, 6 poles, and a 1000 rpm speed.

4.3.7 An Overview of the Design

The results are summarised in Table 4.9, along with the critical dimensions of the civil components (designed structures) for the Kala River Hydropower Plant in the Kala Catchment of the Kiwira Basin in the Nyasa River Basin.

Table 4.9: Summary of the critical dimensions of the civil components

Components	**Critical dimensions**
Dimensions of Orifice for side Intake	Design flood level 1 m Area of Orifice 1.649 m2 Delivery discharge 1.979 m3/s Flood discharge 4.0740 m3/s
Dimensions of Headrace Canal	Cross sectional area = 2.198 m2 Optimum Canal Height H c = 1.094 m Canal bed width B = 1.352 m Canal top width T= 2.446 m Canal Velocity Vc = 2.969 m/s Wetted perimeter P= 4.658 m Head loss = 0.21632 m Hydraulic radius = 0.472 Canal bed slope = 0.019846 Size of largest particle 0.0149 = 14.9 mm
Dimensions of Setting Basin	Length = 4.3836 m Expected silt load = 3326.4 kg Volume of silt load = 2.55 m3 Average collection depth = 0.26m Width w = 1.959 m
Spillway	Length 3.268 m
Dimension of penstock assembly	Material = mild stel pipe 280 mm Length = 109 m Pipe diameter = 530 mm Total head loss = 31.8 m
Fore bay	Submergence head = 0 .9 m Diameter of air vent = 63.31mm

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.0 Introduction

This chapter presents the conclusions and recommendations of the study, which assessed the hydropower potential of the Kapologwe Waterfalls in Rungwe District, Mbeya Region, to support rural electrification. The analysis revealed that the site has a power potential of 7,237.8 kW, sufficient to meet the energy needs of five out of six surrounding villages. The study employed a transfer-function approach and hydrological data from the nearby Kiwira River to estimate flows in the ungauged Kala catchment, while a Pelton turbine was identified as the most suitable technology given the site’s high head and moderate flow rate. The chapter concludes that the methodology used is reliable and transferable for similar ungauged sites. It further recommends future research to optimise turbine design and operational performance, ensuring long-term efficiency, mechanical reliability, and sustainability of the hydropower system.

5.1 Conclusion

This study evaluated the hydropower potential of the Kapologwe Waterfalls in Rungwe District, Mbeya Region, with the goal of supporting rural electrification in the surrounding villages. Based on site-specific measurements and hydrological estimations, the waterfalls were found to have a power potential of 7,237.8 kW, driven by a net head of 598.1 meters and a flow rate of 1.54 m^3/s. While this output is slightly below the combined projected energy demand of 8,641.8 kW for all six villages over the next four years, it is sufficient to meet the needs of five villages—Kisondela, Bugoda, Lutete, Ndubi, and Isuba—which collectively require 7,116.85 kW. This makes the site a viable candidate for phased rural electrification. To address the challenge of an ungauged river, the study employed a transfer-function approach using hydrological data from the nearby Kiwira River. The area ratio method, supported by correlation analysis and a generated flow duration curve, allowed for reliable flow estimation under varying hydrological conditions. A Pelton turbine was selected as the most technically appropriate option due to the high head and moderate flow rate at the site. The results demonstrate that this methodology provides a systematic, efficient, and transferable approach for assessing hydropower potential in similar ungauged catchments and supports the design of site-specific mini-hydropower systems.

5.2 Recommendations

Although the hydropower potential of the Kapologwe Waterfalls is sufficient to meet the energy demands of five villages in the Kala catchment, further research is recommended to enhance the technical and operational performance of the system. Future studies should focus on the structural interaction of turbine components, particularly the runner design, to improve mechanical reliability, extend operational lifespan, and minimise maintenance requirements. This is especially important for high-head, low-flow conditions, such as those at Kapologwe, where turbine runners tend to be relatively short and may experience increased stress concentrations. Additionally, targeted optimisation of turbine parameters—including runner geometry, nozzle configuration, and rotational speed—should be pursued to maximise efficiency under site-specific conditions. Addressing these technical aspects would overcome current design limitations and ensure that the selected turbine operates more effectively and sustainably over the long term.

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APPENDICES

Appendix 1: Structured Questionnaire

Name of respondent……………………………………………………………………

Interviewer's name …………………………………………………………………

Date……………………………………………………………………………………….

A. Family profile

1. Number of family members (only living together in the same house)

Female adults at 20 years or over …………………………....

Children less than 20 years…………………………………….

Total…………………………………………………………….

Number of children going to school

University student..

High school student………………………………………....

Secondary school student……………………………………...

Primary school student………………………………………....

Total……………………………………………………….....

2. How many of your family members are earning income in the village? ....

3. How many of your family members are living in other town to work? .... persons

B. Housing

4. How many rooms does your house have? ....rooms

5. What is floor area of your house? ....m

6. What type of roof is used for the house?

Type of roof	**Tick**
Tiled	
GI sheet roof	
Thatched roof	

C. Economic Aspects

House hold income

7. How much is your family earning from agriculture?

Type of crops	**Average amount of production per cropping(kg)**	**Time of cropping per year**	**Average farm gate price (Tsh)**

8. How much did your household spend on the energy-related item for the last month?

No	**Item of expenditure**	**Amount**
Gas		
Solar power		
Car battery charge		
Kerosene		
Diesel oil		
Charcoal		
Fuel wood		
Dry wood		
Dry batteries		
Other		
Candles		
Total		

9. If your village is to be electrified and your house is to be connected with electricity distribution systems, all of your existing costs for lighting and heating as mentioned above may be saved. In this case, how much monthly charge are you willing to pay for new electricity services?

Range(Tsh/month)	**3000**	**5000**	**10000**	**More than 15,000 (Specify9**
Tick				

D. Energy related Property

10. Do you have the following for lighting and/heating

Kind of equipment	**Generator**	**Kerosene**	**Gas fired cooking**	**Car battery**	**Other (specify)**

11. What kind of electrical appliances does your household currently use?

( ) Bulb/fluorescent light ……………………………….units

( ) Radio & cassette recorder set……………………… units

( ) Other, specify……………………………………..units

E. Needs for Electricity

Priority needs 12. Could you give your priority order on the following needs?

Needs	**Priority**
Education	
Health	
Water supply	
Water supply	
Sanitation (toilet, solid, waste, drainage etc)	
Electrification	
Irrigation	
Road improvement	
Others (specify)	

Effort to have access to electricity: Please Tick (✓) the correct answer

13. Has your household ever attempted to have access to electricity (Tick)?

Yes ☐ No ☐

If **Yes** go to question 14

14. What type of electricity generation did your household plan to have access to?

( ) Diesel generator set

( ) Solar home system

( ) Wind power

( ) Micro-hydropower

( ) Biomass

( ) Other specify…………………………………………………………….

Purpose of using electricity

15. If you can have access to electricity, what kind of electrical appliances and how many appliances do you want to use?

( ) Bulb/fluorescent light……………………………………units

( ) TV – set ……………………………………….............. units

( ) Radio & cassette recorder set…………………………........ units

( ) Refrigerator……………………………………………........ units

( ) Air conditioner units

( ) Other, specify……………………………………............. units

16. What kind facility/equipment do you want to use electricity for productive activities?

( ) Saw mill machine

( ) Rice milling machine

( ) Rice dryer

( ) Irrigation pump

( ) Other, specify ……………………………………………..................

17. What public facilities do you think should have access to electricity?

( ) School

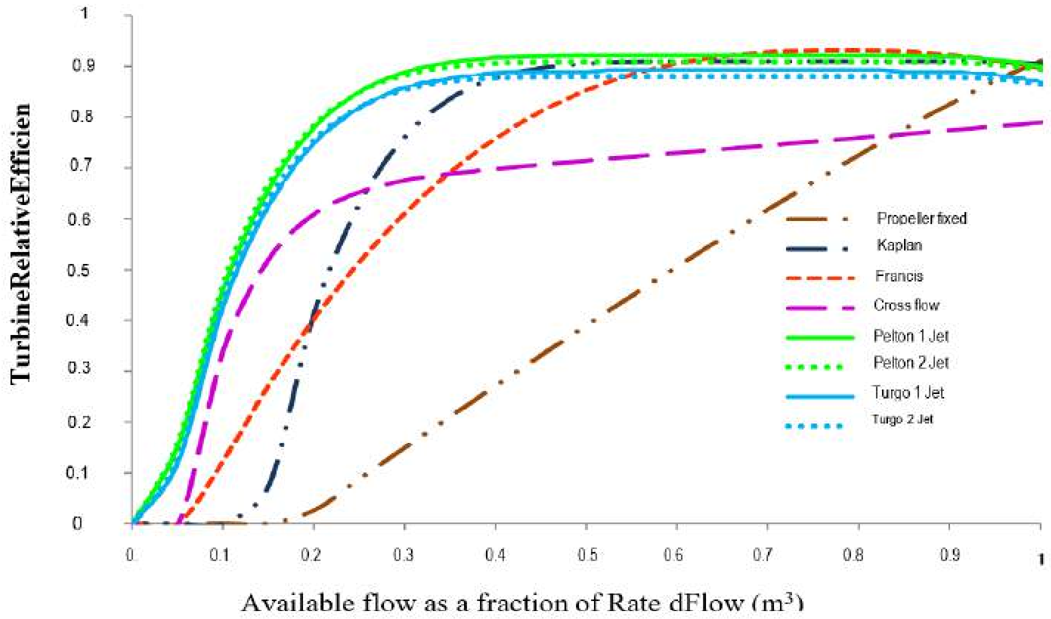
( ) Mosque/church

( ) Clinic/health center

( ) Water pump for drinking water

( ) Others, specify…………………………………………………………..

Appendix 2**:** Turbine efficiency curves (Adejumob and Shobayo, 2015)



Appendix 3: Flow duration of Kala and Songwe River

Collected Data		**Analyzed values**			
Month	**Q (m^3/s)**	**Month**	**Q (m^3/s) descending**	**Rank**	**Pp (%)**
January	15.1	March	21.91	1	7.69
February	18.44	April	21.16	2	15.39
March	21.91	February	18.44	3	23.08
April	21.16	January	15.1	4	30.77
May	11.1	May	11.1	5	38.46
June	7.16	June	7.16	6	46.15
July	5.28	December	6.53	7	53.85
August	4.21	July	5.28	8	61.54
September	3.21	August	4.21	9	69.23
October	2.43	September	3.21	10	76.92
November	1.542	October	2.43	11	84.62
December	6.53	November	1.542	12	92.31

Appendix 4: Average flows and rainfall data (Decade)

Month	Months Average Flows (m^3/s)	Average Rainfall (mm)
Jan	27.1182	132.3
Feb	33.3273	120
Mar	39.6018	206
Apr	38.2452	130
May	20.0638	19.4
Jun	12.9493	0.8
Jul	9.5467	0.8
Aug	7.6142	0.3
Sep	5.7773	0
Oct	4.3882	1.3
Nov	3.577	39.4
Dec	11.8028	149

Appendix 5: Standard deviation of the means of flow

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
2012	0.4	5.17	35.48	49.57	57.92	44.33	23.16	13.8	9.2	6.87	5.28	4.264
2013	2.94	9.15	38.49	36.46	26.42	39.62	17.85	11.86	9.05	7.45	5.21	4.909
2014	5.66	8.49	14.3	13.79	40.93	42.02	18.01	10.78	8.58	7.03	6.32	5.443
2015	5.6	12	25.59	26.57	31.17	25.79	11.67	8.56	7.2	6.26	5.05	3.557
2016	1.7	1.82	5.5	16.99	20.69	27.91	17.99	12.17	9.21	7.77	6.14	3.741
2017	1.97	38.52	55.98	58.22	54.58	38.85	24.67	16.87	11.72	9.69	7.92	6.312
2018	4.86	15.8	30.68	58.07	56.74	54.97	34.05	20.91	13.68	9.73	5.73	3.741
2019	3.82	15.24	24.92	28.48	44.79	47.21	21.81	12.44	9.31	7.06	5.3	3.825
2020	6.56	8	30.34	30.92	43.41	33.01	16.08	11.19	8.92	7.21	5.39	3.288
2021	2.25	3.83	9.91	14.21	19.38	28.75	15.33	10.92	8.6	7.08	5.43	4.802
stdv	**2.04**	**10.44**	**14.87**	**16.98**	**14.56**	**9.39**	**6.25**	**3.54**	**1.83**	**1.17**	**0.86**	**0.96**

Appendix 6: Power Demand for Five Villages

Classification of Load	Category	Kw Per User	Consumers	Load (kW)
Household		0.13	10,100	1313
Subtotal				**1,313**
	P/School	1.58	3	4.74
Education	S/School	23	1	23.00
	Subtotal	**5.85**	**12,206**	**27.74**
	Tailoring	1.45	1	1.45
	Carpentry	2	0	0
	Grain Mills	15	2	30
	Garage	2.25	0	0
	S/Restaurant	0.12	2	0.24
	Bar	1.14	1	1.14
	Shops	0.16	2	0.32
Commercial	S/Workshop	1.42	1	1.42
	Subtotal	**1.83**	**10**	**34.57**
	Mosque	0.17	1	0.17
	Churches	0.43	1	0.43
	W/Pump	33.33	1	33.33
Public Service	Godown	0.5	1	0.5
	Office	2.65	5	13.25
	Dispensary	0.38	1	0.38
	Subtotal	**1.75**	**13**	**48.06**
	TOTAL			**1,423**

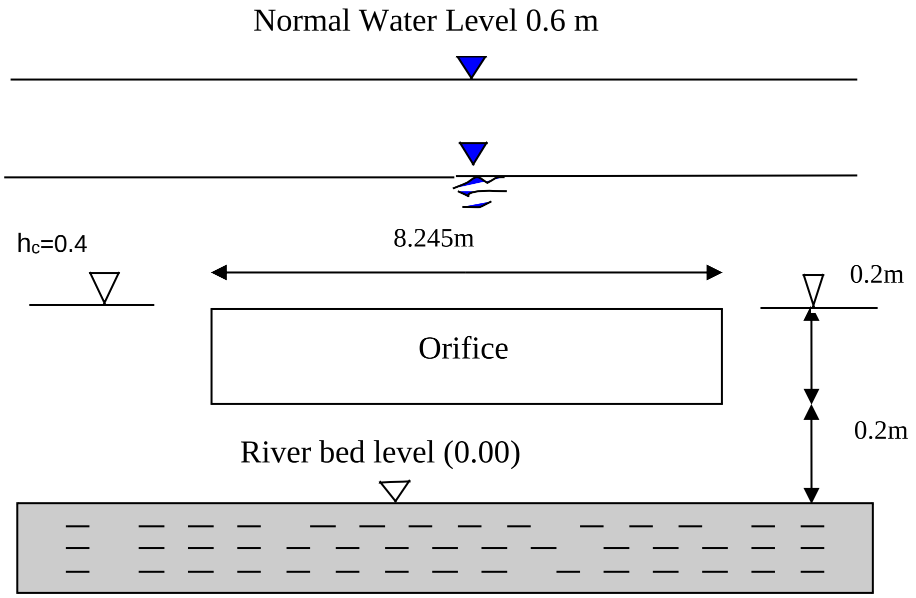
Appendix 7: Average Power Forecast for Five Villages

Based on equation 12 in Chapter Three

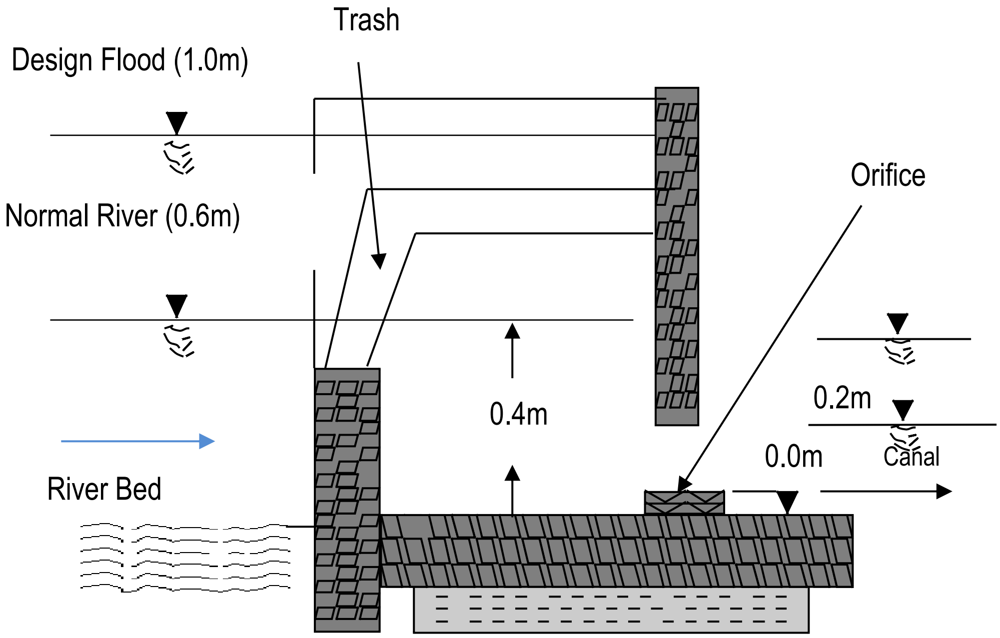
$TD = D\,(kW) * (1 + \gamma) * n$

Years	Classified Load	n	γ	D (Load-kW)	(1+γ)	Total Load (kW)
	Residential customers	4	0.25	1,313	1.25	6565
	Education	4	0.25	27.74	1.25	138.7
	Commercial	4	0.25	34.57	1.25	172.85
1 – 4	Public Services	4	0.25	48.06	1.25	240.3
TOTAL (Kw)						**7116.85**

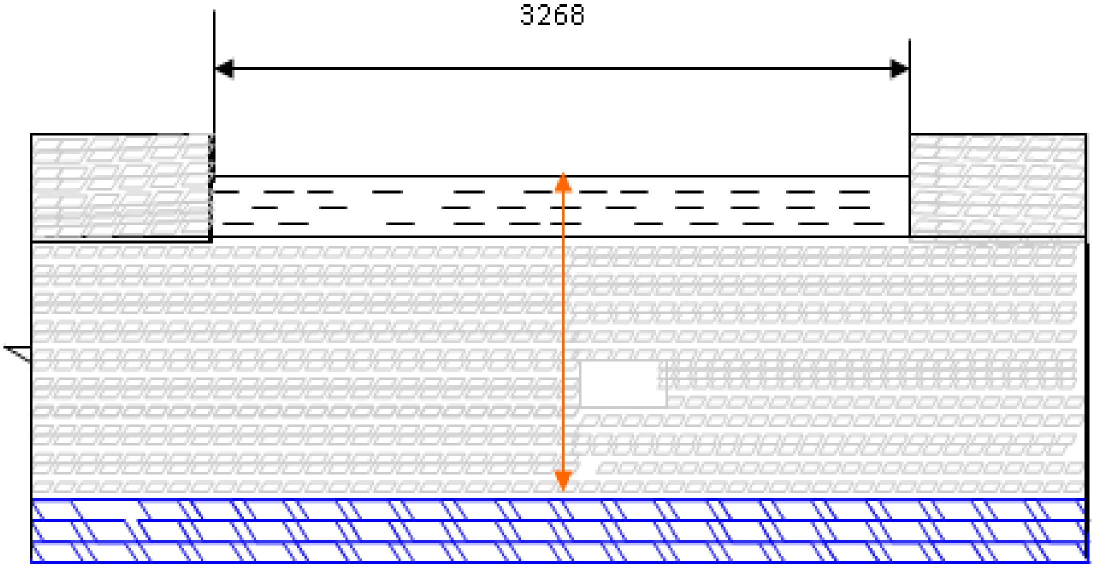
Appendix 8: Dimensions of Orifice and Levels (Dimension in meters)



Appendix 9: Structure of Intake to Power Canal (dimension in m)



Appendix 10: Section of Spillway in mm



Appendix 11: Schematic diagram of headrace canal design