PERFORMANCE EVALUATION OF MICRO HYDROPOWER SCHMES ON UNGAUGED STREAM THE CASE OF BASHIRO-GUTE – (ERERTI RIVER)

MASTERS OF SCIENCE THESIS

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A THESIS SUBMITTED TO
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HAWASSA UNIVERSITY

ADVISORS’ APPROVAL SHEET

This is to certify that the thesis entitled as **‘‘PERFORMANCE Evaluation of Micro Hydropowe Schmes on Ungauged Stream the Case of Bashiro-Gute – (Ererti River)’’** is submitted in partial fulfillment of the requirements for the degree of Masters with specialization in **Hydraulic Engineering,** the post graduate program of the **Faculty of Bio-Systems and Water Resource Engineering** and has been carried out by **Kefyalew Abrham Argo, ID.GHYdr0008/13** under my supervision. Therefore, I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the school.

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Name of Major Advisor	Signature	Date
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We, the undersigned, members of the Board of Examiners of the final open defense by **Kefyalew Abrham Argo** have read and evaluated his thesis entitled **‘‘PERFORMANCE Evaluation of Micro Hydropower Schemes on Ungauged Stream the Case of Bashiro-Gute – (Ererti River)’’** and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of master’s in "Hydraulic Engineering".

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DECLARATION

I declare that this thesis, which I submit to the Faculty of Biosystem and Water Resource Engineering of Hawassa University in partial fulfillment of the requirement of degree master of Science in Hydraulic Engineering, is my own personal effort. Furthermore, I took reasonable caution to ensure that the work is original and does not break copyright law and has not been taken from other sources except where such work has been cited and acknowledged within the text.

Signature----------------------

Date----------------------------

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LIST OF ACRONYMS

CSA – Central Statistical Agency of Ethiopia

DEM-Digital Elevation Model

EEPCo-Ethiopia Electric Power Corporation

GIZ-Germany Agency for International Cooperation

GPS – Global Positioning System

HPP=Hydropower Plant

MHP=Micro hydropower plant

RoR HPPs = Run of River hydropower plants

SHP= International Center on Small Hydro Power

ABSTRACT

This study was aimed to Performance Evaluation of Micro Hydropower Schemes on Ungauged Stream the Case of Bashiro-Gute – (Ererti River). The methodology that was applied in this analysis was quantitative research approach and using GIS software. Specifically, to assess the current status of the scheme, the factors behind the failure and estimating the potential and designing the proposed structures. The stakeholder interview, field observation, document review and secondary data (30-year simulated stream flow) were collected and the analysis was conducted. The physical observation has shown that the existing condition of the components especially the diversion weir and power canal of the Micro hydropower plants was broken and collapsed and do not provide the functions. The causes of failure of the plant were lack of technical feasibility study (structural instability), public participation and consultation shortage on the plant development, lack sense of ownership among the community and stakeholders of the project to protect and manage the plant. The plant development has not followed the scientific design method. The potential assessment of the Ererti River has indicated that 74.9kw, 86.4kw, and 153.6kw actual power could be generated from 0.4 m^3/s, 0.5 m^3/s, and 0.8 m^3/s discharge design at 100%, 95% and 50% exceedance time respectively. For average potential hydropower which was 0.8 m^3/s, the proposed crest dimension was top width 0.8m, bottom width 2.3m, and height 2.65m. The power canal that proposed was width= 0.8m, depth=0.4m with the bed slop of 1:2, wetted perimeter =0.2m and bed slop 1 to 0.01m with 1m head loss. The 300mm diameter of penstock is calculated with the head loss of 0.02m as suitable to transfer the water to the turbine. The net head available to produce the power from 50% exceedance time discharge was 27. 98m.The measured weir crest dimension was top width 0.9m, bottom width 2.0m, and height 2.01m.and the power canal bottom width width= 1m, depth=1.5m with the bed slop 1 to 0.01m and the 300mm diameter of penstock is measured.

Keyword: - *Hydropower, Discharge, Diversion weir, Power Canal, Actual power, Flow Duration Curve*

CHAPTER ONE

1. INTRODUCTION

1.1. Background of Study

The demand for energy is growing throughout the world as energy is the backbone of the growth of a nation. A combination of increasing population growth rate, desire for an improved standard of living has been increased. Currently, public policies have an increased interest in green energy development to satisfy the need of their nation. Reliable access to electricity is basic preconditioning to improve people's lives in rural and urban areas, for enhanced health care, education, and for growth within local economies (Dametew, Design and Analysis of Small Hydro Power for Rural Electrification, 2016.). Worldwide at present time 1lmost 1.5 billion people do not have an access to electric power and 80% of them are in rural area more over it is also estimated to increase in the coming decades (Sørnes, 2010) According to Kyu Kyu Thin, WinWin Zin, Zin Mar Lar Tin San, Akiyuki Kawasaki, Abdul Moiz, and Seemanta Sharma Bhagabati, 2020, commonly there are different renewable energy options including wind, solar, and hydropower. Among these the hydropower is the most apparent renewable energy option. However even if there are abundant resources particularly the developing countries such as sub-Saharan countries live in paradoxically with insufficient energy supply (Hailu, 2022). Despite the fact that vast energy potential existed in the region, more than 620 million people lack access to electricity. Compared to other renewable sources implementation of hydropower has a significant impact on the economic development. The argument that is raising frequently is unlike fossil fuel plants, hydro power doesn't directly contribute to CO2 emissions or other environmental harm.

However surprisingly, Ethiopia is a heavily dependent on conventional fuel, ranking third in Africa after Chad and Eritrea. Nevertheless, conventional fuels account for more than 99.9% of the country's rural energy use. Around 95% of residential energy is used in the home overall needs are met from biomass thereby contributing to deforestation, loss of soil nutrients and organic matter. Even though, the country has a substantial potential for renewable energy resources, the existed hydropower schemes are a very limited stage. According to UNIDO, ICSHP, (2016), globally about 16.6% of the electricity is produced from hydropower while in Ethiopia it remains under 3%. The East Africa countries such as Ethiopia, Tanzania and Zambia have the largest SHP (small hydropower) potential as reported to date.

According to Alam Hossain Mondal, Abiti Getaneh Gebremeskel, Kiflom Gebrehiwot and Claudia Ringler, (2018) Ethiopia has high potential of renewable energy for example hydropower potential is 45GW, wind energy is 10 GW, geothermal potential is 5 GW, and the potential for solar irradiation ranges from 4.5 kWh/m^2/day to 7.5 kWh/m^2 /day. Even though there is abundance of potential, only less than < 1% each is exploited ((Kumsa, 2020) and (Dagmawi Mulugeta Degefu, 2015). In urban area 87% of the population have accessed while in Rural only 5% accessed with the connection of grid line. Majority of the population in the country which is almost 83% resides in rural area (Alam Hossain Mondal, Abiti Getaneh Gebremeskel, Kiflom Gebrehiwot and Claudia Ringler, 2018). To solve this problem the government of Ethiopia has plan for universal electricity access, efficiency improvement, developing decentralized off-grid power generation, and exporting electricity to neighboring countries (EEPC, 2014a, Hailu, 2022). Based on these plans now a day a number of mega and small-scale hydropower projects are under construction since the topography is suited to the use of mega and micro-hydropower plant project development.

Alam Hossain Mondal, Abiti Getaneh Gebremeskel, Kiflom Gebrehiwot and Claudia Ringler, (2018) claimed that under the universal Electrification scenario the power generation capacity grows faster, from 4.23 GW in 2014 to 69.3 GW in 2050 to meet expected electricity demand. However so far, out of the total potential for micro hydropower plants in the country, only less than 1 % is developed (Degefu, 2015, Kumsa, 2020). Various micro hydropower plants (MHP) schemes were installed in the period between 1950 and 1970 however almost all of them are not operational now a day (Tibebu Tsegaye Zigale, 2019).

Bashiro Gute, Gobacho-I and Gobacho-II schemes which are located at Bona-Zuria woreda are among the micro hydropower plants that aimed to generate electric power. The implementation of Bashiro gute hydropower scheme was started in 2008 and completed in 2011. It is a diversion type of hydropower system. The length of the power canal starting from the headwork to the forebay is about 100 m (masonry lined canal) and 30m penstock with internal diameter of 320mm. The main source for this scheme is Ererte River which is the tributary of Genale river basin. Its design discharge was 300 liter per second and head was 15.5m and supposed to generate 33KWH power for local domestic demands. The Gobacho-I scheme was also started in 2008 and completed in 2011. The main source for this scheme is Gange River which is the tributaries of Genale river basin. Its design discharge design was 138 liter per second and head was 8.13m, and which supposed to generate 11 kilowatt powers. The length of the power canal starting from the headwork to the forebay is about 16 m (masonry lined canal) and penstock is 10m with internal diameter of 320mm. Gobacho-II hydropower scheme was started in 2009 and completed in 2012. The length of the power canal starting from the headwork to the fore bay is about 10 m (masonry lined canal) and penstock is 62m with internal diameter of 320mm. The main source for this scheme is Gange River which is the tributary of Genale river basin. Its design discharge was 138 liter per second and head was 25m. The scheme supposed to generate 33KWH power. However, the schemes have not been giving the intended benefits due to the failure of the head work and conveyance structure. Due to the limitation of time, data access and financial resource only Bashiro Gute hydropower scheme was aimed to evaluate. The main aim of these researches is to investigate current situation of the existing scheme and identifying the factors responsible for the failure of the scheme and assessing the potential of the existing micro-hydropower scheme and comparing with past design to provide suitable remedial strategies.

1.2. Problem Statement

The Bashiro-gute hydropower schemes were developed by GIZ and were expected to solve the power demand gap of Bashiro gute kebele. However, the schemes are failed to provide the required power for the community. According to the kebele administration information the scheme was failed after two year it was installed. Typically, the diversion weir and power canal collapsed and which could not pass the inputs which is discharge to forebay and penstock. However, no one can find any researches that were conducted on the examination of cause of failure previously.

1.3. Objectives

1.3.1. General Objective

The general objective of this study was to evaluate the performance of Hydropower components of the existing scheme of Bashiro-Gute (Ererti River).

1.3.2. Specific Objectives

- To assess the existing situation of the Bahiro-Gute hydropower components on the Ererti River.
- To investigate factors of the failure of the hydropower scheme.

- To investigate and design the potential of the Hydropower

1.4. Research Questions

- What is the status of the schemes at current time?
- What are the causes for the mal-functional of the schemes?
- What is the potential of the river to generate power for the community?

1.5. Significance of the Study

This study has a significant benefit for the community since it has provide scientifically studied information on the situation and cause of failure of the project. This enables the community to manage and own the project properly. It also gives the information of failure area where the correction has to be taken for the local government and community. The national and regional government may take the result as a baseline and input for rural electrification strategy design since the detail information that was studied has shown the energy resource potential in order to address the issue of energy access in society.

CHAPTER TWO

2. LITERATURE REVIEW

The majority of developing nations are experiencing what many refer to as an "energy crisis," which is characterized by the depletion of domestic energy sources and a reliance on imported fuel (IMF, 2022). The energy crisis is making the food crisis worse by speeding up deforestation and destroying farmland. Furthermore, the ability of the affected countries to purchase food whenever necessary is being weakened by their reliance on imported petroleum. To feed their nation and harness the development sustainable energy utilization is much important. According to UNIDO and ICSHP (2016) explanation energy is one of the most significant economic, environmental and development issues facing the world today. Globally about 1.2 billion people which is 17% of the world's population have no access to electricity. Currently, the need for clean and sustainable sources of energy is growing more in the uncertainty of climate. Moreover, the geopolitical and economic uncertainty over traditional fossil-fuel markets highlights the importance of energy diversification and independence.

The report that was published by world Hydropower Atlas 2000, has indicated that technically feasible hydro potential at world scale is estimated at 14,370 TWh/year, which covers 100% of today's global electricity demand (Dametew, 2016). The proportion of electric generation source that was indicated in the figure 2.1 below has shown that the hydropower share is the second following coal at global scale.

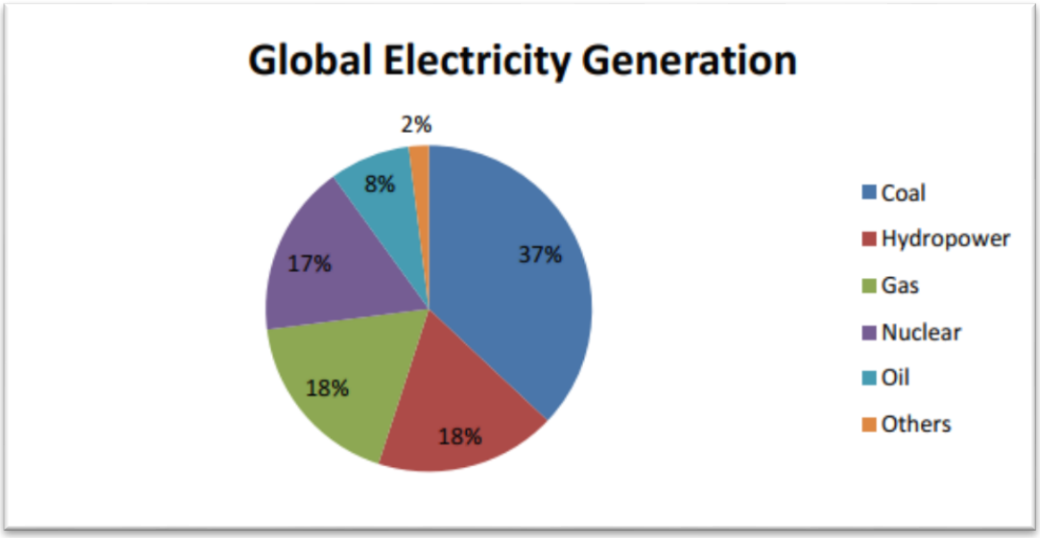


Fig.2.1. Global Electricity Generation in World

2.1. History of hydropower

The concept of hydropower has started 2000 years ago according to the eaplanation of U.S. Department of energy in 2008 (AJALA, 2014). During this period, it was found that the force of running water could be used to simplify human tasks by turning shafts to mills for grinding grain and cutting wood, operating hammers in Western European ironworks, and industrial power. Among the sustainable energy source hydropower is the most preferable source where water resource sufficiency is existed. It is the oldest and widely utilized form of renewable energy at global scale with over 1.2 TW of installed capacity spanning six continents. Installing the scheme requires both water flow and a drop in height called a head to produce useful power (Dametew, 2016, UNIDO, ICSHP, 2016).

The power produced by utilizing the potential energy held in moving water is known as hydropower. The generation of energy could be explained by the law of conservation of energy. According to the AJALA, (2014), during the hydropower generation four energies takes place which are potential, kinetic, mechanical and electrical energies. The potential energy of flowing water is converted to kinetic energy in the penstock whereas kinetic energy of the flowing water turns the blades of the turbine and converted to mechanical energy. Finally, the turbine shaft rotates the generator and produce electrical energy. The power generated is calculated using the equation 2.1. below (ESHA, 1998).

$$P == n\rho gQH \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\ldots eq2.1$$

Where, P is the power in watts, n is the general efficiency of the plant ρ is the density of water in kg/m^3 g is the gravitational acceleration in m/s^2 Q is the discharge, passing through the turbine in m^3/s H is the gross head of the water in m

There are many different project scales and types for hydropower which are micro, Small-scale Hydropower, and mega-hydropower projects.

2.1.1. Small scale Hydropower

Hydropower on a small-scale is one of the most cost-effective energy technologies to be considered for rural electrification in less developed countries (Boniface, 2013). This ideas was also supported by Dametew, (2016) that by saying it is one of the most environmentally caring energy technologies. Even though there is internationally accepted definition of small hydropower, in China, it can refer to capacities up to 25 MW, in India it limited to 15 MW while Sweden accepts below 1.5 MW as small hydropower. In Ethiopia the definition was range up to 500kw power.

2.2. Small Hydropower potential In Ethiopia

The report on micro hydropower has indicated that Ethiopia is the second highest potential country next to Kenya for small hydropower generation (Liu, 2013). According to the report the country has 1,500 MW potential for small hydropower. However, at the region the project suffered from lack of hydrological data, inadequate awareness, lack of master plan, difficult site access due to lacking road infrastructure. The other barriers that hiders the functionality of a project is the lack of Investment and participation of private companies. In addition to these limited human resource capacity, especially technical know-how, needs to be improved in view of the poor maintenance and management of small hydropower plants. Similarly, Ethiopia has been facing such challenges in installation of small hydropower plant. Generally, the countries potential of hydropower has tabulated below in the table 2.1. Below

Table 2.1: Hydropower Potential of Ethiopia

Name of River	Number of potential sites			Total
Basin	Small Scale 40 MW	Medium 40-60	Large above 60MW	
Abbay	74	11	44	129
Rift Valley Lakes	7		1	8
Awash	33	2		35
Omo-Gibe	4		16	20
Genale Dawa	18	4	9	31
Wabi-shebele	9	4	3	16
Baro Akobo	17	3	21	41
Tekeze Angereb	11	1	8	20
Total	173	25	100	300

[Source: Damte, 2016]

This report indicates the potential on the large river but the country has a number of river stream that is suitable for hydropower development. However, according to the German Agency for Technical Cooperation Report in 2010 depicted that small and micro-hydropower are not yet developed on a larger scale in Ethiopia. The study area three small hydropower plant which are Bashiro-erert Gobecho and Hagara Sodicha was inaugurated in February 2012 according to Liu, (2013) in Sidama Zone at that time but Now Sidama Regional state. According to the report the plants were developed in partnership with Sidama Mines, Water and Energy Agency, the Sidama Development Association and local communities, and with the support of GIZ.

2.2.1. Classifications of Hydropower Installation

Hydropower installation is classified in different types based on various criteria such as operational feature, basis of operation, purpose of development, based on uses hydraulic feature plant capacity operational head. The design of structure, hydrologic and hydraulic design, the service provision is depended on the types of hydropower (Gaudard L. and Romerio F., 2014).

2.2.2. Run-of-river

A tiny hydro power station without a reservoir or storage space is known as a run-off-river hydro power plant. This type of hydropower energy production depends on the timing of the flow. according to Gaudard L. and Romerio F., (2014) run of river generates the base-load power with seasonal fluctuations. The most crucial factors in the design of a run of river type hydropower plant are flow volume and head. It primarily uses the river's available flow to generate energy for electricity. Thus, generation is on rainfall and runoff and is subject to significant daily, monthly, and seasonal variations. The Run of River hydropower plants (RoR HPPs) will have more irregular generation profiles if even short-term storage is excluded, especially if they are located in tiny rivers or streams with highly variable flows. According to the fig 2.2. Below a portion of the river water may be diverted to a pipeline or channel (penstock) in for this type of hydropower plant. RoR projects can take the shape of cascades down a river valley, frequently with a reservoir-type HPP in the valley's upper reaches that enables both to take advantage of the combined capacity of the multiple power plants. It is inexpensive to install and have low environmental impact as long as the entire ecosystem is adequately protected (Kiraga, 2021, Kumar, A. et al., 2011).

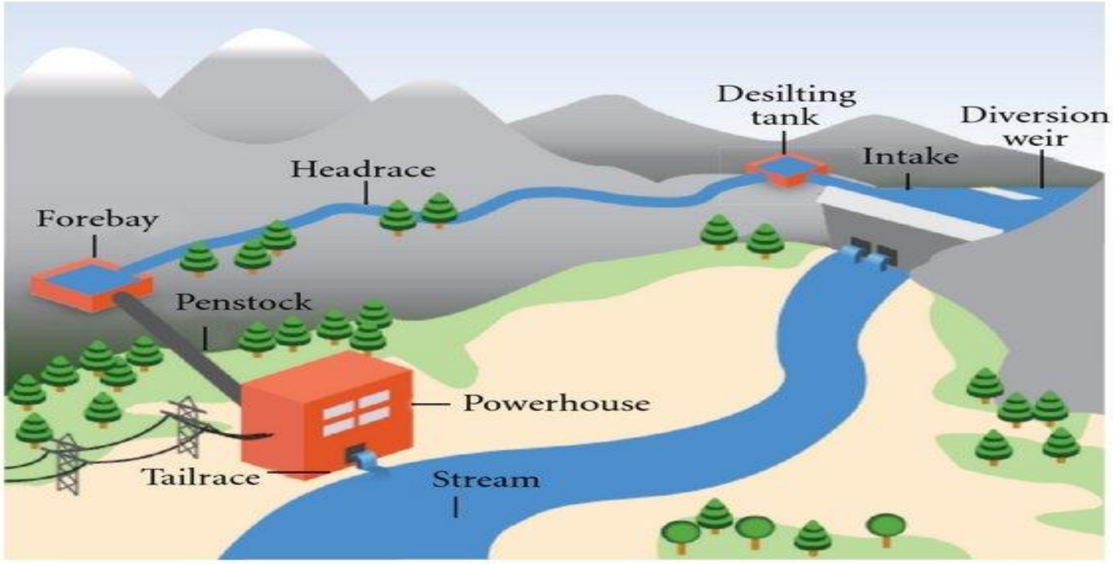


Fig: 2.2. Schematic diagram of run - of – river hydropower system *(Source: Kumar, et al., 2011)*

Storage Hydropower

Storage hydropower refers to hydropower projects that include a reservoir because they store water for later use. This type of hydropower installation is differed from the run of river types by the storage timing. According to Gaudard L. and Romerio F., (2014) argument it is suitable for mountainous region or in area where large reservoir exists. For this type of hydropower, the head of water determines the energy production even with minimal storage volume. The topography of the area influences the kind and the reservoirs layout. The generating station is frequently connected to the lake acting as a reservoir in these types of locations by tunnels rising beneath the lake (lake tapping) as it is shown in the figure 2.3 below. But as Gaudard L. and Romerio F., (2014) championed this type of power production may aggravate a significant environmental impact.

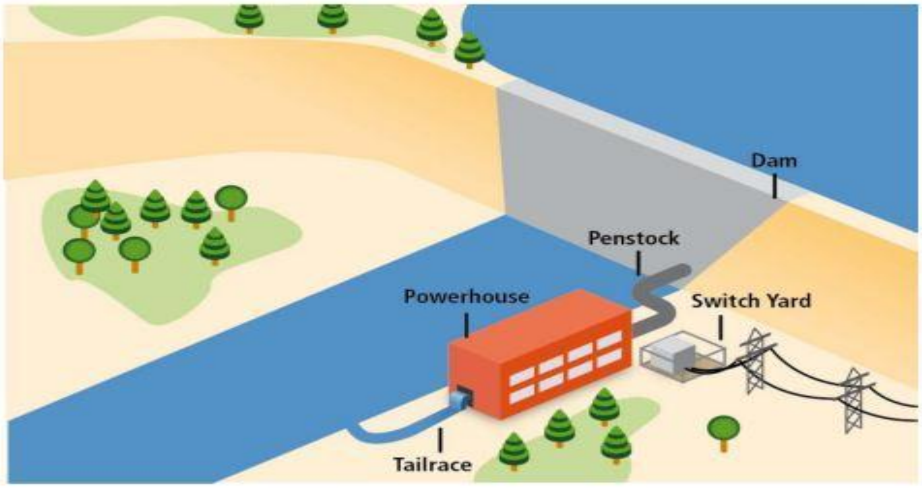


Fig. 2.3 Typical hydropower plant with reservoir (Source: Kumar, et al., 2011)

Pumped storage

Pumped storage facilities are actually storage facilities rather than energy sources. In such a system, water is often pumped from a lower reservoir into an upper reservoir during off-peak times, and flow is then reversed to produce power during the daily peak load period or at other times of necessity (Fig. 2.4). According to Gaudard L. and Romerio F., (2014) The effectiveness pumped installation type hydropower depends thus on the ratio between peak and off-peak price.

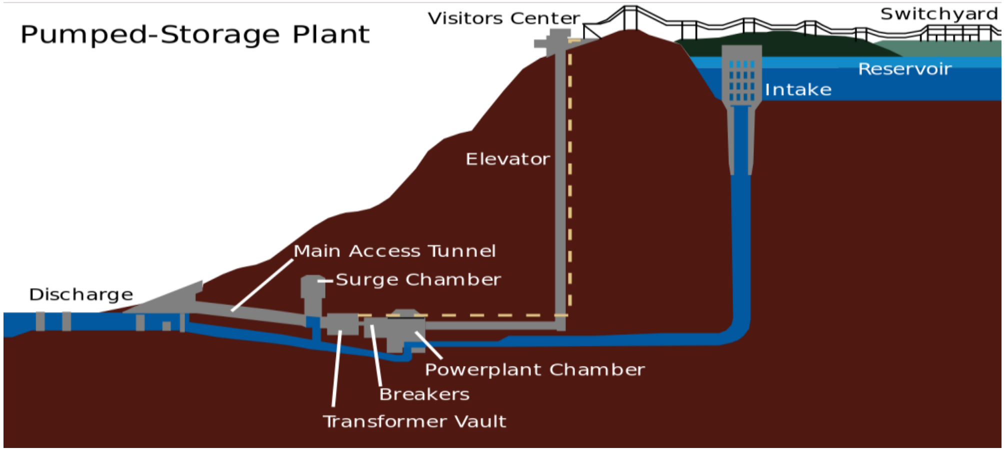


Fig. 2.4 Typical pumped storage project (Source: Kumar, et al., 2011)

2.3. Components of Hydropower plant

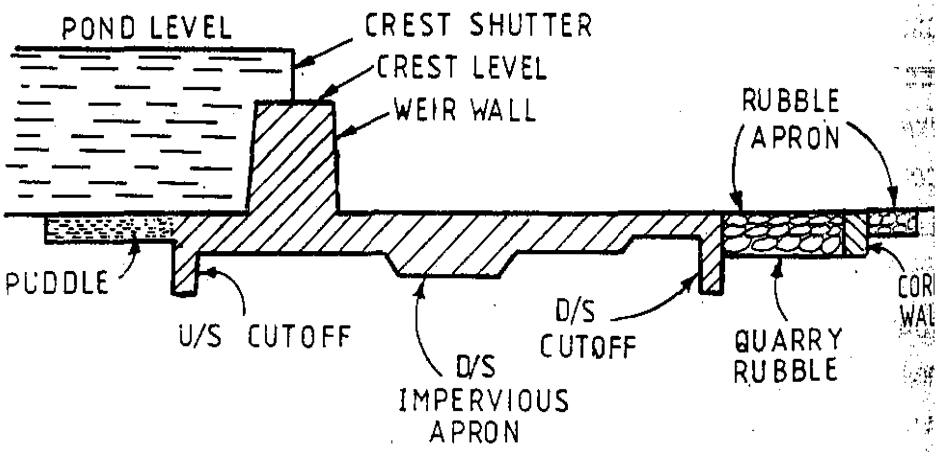
A complete hydropower system consists of the following major components, which were discussed in this section. The main component includes Gates which are barriers to control and regulate the flow. The gates are used for power intakes, bottom outlets, or river diversion works. The other is penstock which is a conduit or pipe that transport water from the intake section to the powerhouse. To withstand the pressure which called “water hammer” Penstocks are made of steel. However, when the powerhouse is located far from the dam and reservoir water has to be transferred by open channels, tunnels with pressure, or pressure shafts, which depending upon the rock type can be unlined, lined with concrete or steel lined. The other components are surge tank which located between the power house and water intake structure to absorb any water surge. Turbine is the electromechanical part of the hydropower component that converts the kinetic/potential energy of water into mechanical energy. The other electromechanical part is generator which converts the mechanical energy in to electrical energy. Finally, the electromagnetic powers transferred to the transformer are which changed the electromagnetic currents in to high voltage and connected it to power line (Corà, 2020; Damte, 2016). This study was conducted on the Run of River type of hydropower plant. AJALA, (2014) explained the main components of such type of scheme by dividing in to two headings such as civil works and electromechanical works. According to the explanatio the civil works of a run-of river SHP scheme include water conveyance structures, head pond or forebay, penstock, powerhouse and tailrace structures while electromechanical component comprises turbine, generator, governor, and regulator (Yassen, 2014).

2.3.1. Civil Work Components

The civil work component of the hydropower includes the diversion weir, intake, sand trap, spillway, headrace canal; settling basin, fore-bay tank, penstock pipes, and tailrace are among the civil components included in this section (Garg, 1976). The civil components include diversion headwork such as weir or barrage, under sluice, divide wall, fish ladder, canal head work, silt excluder, guide bank and marginal bunds.

A. Diversion weir

Based on shape and construction material diversion weirs are commonly divided in to three categories which are masonry weirs with vertical drop, rock-fill weirs with sloping aprons; and concrete weirs with sloping glacis. Among these weirs in Ethiopia masonry type of weir is used for hydropower and irrigation development. As it was explained by Garg, (1976), Masonry weir consists of a horizontal floor and masonry crest with vertical or nearly vertical downstream face as it was seen in the figure 2.5 below. This type of weir is considered as old head-works and is particularly suitable for hard clay and consolidated gravel foundations.



[Source Garg, 1976]

Figure 2.5 Masonry weir

The technical efficiency of hydropower schemes depends on the suitability and economic dimension of the weir structures. The components of vertical drop weir as it was shown in the figure 2.4 above include crest, upstream and downstream cutoff, impervious apron, and rubble apron. The effectiveness of the weir structure determined with these component dimensions. According to the hydraulic

B. Under sluice

A conveyance system called a headrace canal is used to move designed discharge from one location to another. In general, pipe systems are employed for specific challenging terrain while canal systems are used in all micro hydropower schemes.

C. Settling basins

The civil component known as the Settling Basin is responsible for settling sediments down in the basin for regular flushing. The exact size of the specified proportion of sediment needs to be captured, settled, stored, and flushed since silt is harmful to civil and mechanical structures and elements. Only by reducing the turbulence of the water carrying the silt can this be accomplished. Constructing settling basins along the conveyance system will reduce turbulence. The transit velocity and turbulence are greatly reduced by the geometrical and greater flow areas of settling basins, allowing the necessary sediments to settle. The resulting settled sediment must be properly re-flushed into the natural waterways.

D. Spill way

A spillway is a design that securely directs surplus water to a river. Flood-related surplus water needs to be removed using spillways in order to reduce the Construction of settling basins and fore-bays, from which the surplus water is securely transferred to the water source, is a common practice when building large spillways.

E. Fore-bay tank

The civil part known as the fore-bay tank is what allows consistent flow into the turbine through the penstock. Before the water enters the penstock, For-bay serves as the final settling basin, allowing the final particles to settle. Depending on its size, the fore bay may also serve as a water storage reservoir. It will be feasible to seal off the penstock's entrance using a sluice. To stop big particles from entering the penstock, a trash rack needs to be placed in front of it.

F. Penstock Pipe

This is the conduit used to deliver pressurized water from the fore-bay tank to the turbine. In addition to transporting water to the turbine, the enclosure also generates head pressure as the vertical drop heightens. The turbine is at the bottom of the pipe, where the penstock concentrates all water power.

2.3.2. Powerhouse Components

The powerhouse is made up of electromechanical components such driving systems, generators, and turbines. A turbine is a device that transforms water pressure from falling water into mechanical shaft power that can be used to power an electric generator. The site criteria determine which turbine is best suited for a certain hydro site. A power speed characteristic is present in all turbines. This means that at a specific speed, head, and flow configuration, they will function most effectively. As a result, the choice is also influenced by the needed speed of the generator connected to the turbine. The turbine's ability to produce electricity at part flow conditions is another crucial consideration. The head under which a turbine operates is the primary factor in determining its design speed.

2.4. Hydropower scheme Performances

The performance of hydropower scheme depends on the stability of the structure, management capacity and its cost recovery ability. It is inevitable that the project may not operate as the design due to failure of hydraulic structures with regard to technical performance (Tilaye, 2009). According to (Murry Rust, D., Hammond, & Snellen, 1992) performance simply defined as “the level of achievement of desired objectives”. The hydropower scheme performance is checked by evaluating the water intake, transportation and power development system and management practices. The hydropower planning, design and construction and generating power are mainly dealing with creation of physical infrastructure to intake and transport water to the next structure. These physical facilities such as weir, power canal and forebay need to be properly operated to ensure the storing, transport and delivery of water at the right time in adequate quantity. Proper maintenance operation is need to ensure the capabilities of these structures to deliver the intended power.

2.4.1. Diversion weir performance

The instability of diversion weir is caused not only because of the unbalanced moment, but due to the scouring effect of the foundation. Typical the analysis of stability is important where the structure foundation is pervious foundations. For such case the design of the components of the diversion structure has to be checked based on the surface and sub-surface flow scenario. All the forces that act on the structure are then calculated to ensure its stability against overturning and sliding (José Ignacio Sarasúa, Paz Elías, GuillermoMartínez-Lucas, Juan Ignacio Pérez-Díaz, 2014, Temesgen, 2017). The structure stability would be calculated based on the theories that were discussed here below. Therefore, the indicators of stability of the diversion structure selected for this research were stability against, overturning and sliding. During the process the safety against sliding, safety against overturning and stability against uplift force are going to be checked.

2.4.2. Managerial Problems

The hydropower scheme failure also may be cause by the managerial Problems such due to in sufficient knowledge of the importance of local partnership during hydropower Policy formulation (Maginn, 2007). Knowing the community culture is an important on the process of decision making. Typical in the rural area the community groups that are directly impacted by the hydropower project are women. Their participation on the decision making in full scale determines the success and failure of the scheme. Practically however the women participation was limited according to the argument of (Gupte, 2003). The other issues regarding managerial problem are lack of appropriate ownership and share of responsibility. Different studies for example Barnett (2000) has shown that about 74% of failures were due to managerial and social problems in Pakistan. Similarly, the study result in Zimbabwe and Mozambique found that the development of effective renewable energy needs financial success. According to the study privately owned schemes has financial success than domestic use community-based micro-hydro projects. The hydropower in Ethiopia typically the small hydropower plant failed in different parts of the country. In the study area Bashiro Gute (ererti) failed to give the intended services. Therefore, this research was aimed to assess the causes of failures by checking the participation of community, type of ownership and existence of guidelines and other required documents.

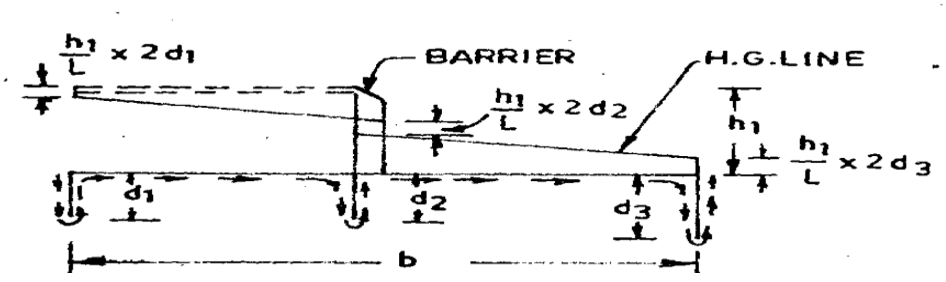
2.5. Weir Design and Seepage Theories

Hydraulic structures such as weir, barrage, dam etc. may found on impervious or pervious foundation. The structure which founded on pervious surface is subjected to seepage of water beneath the structure whereas to all forces to which it will subjected when the foundation is impervious (Garg, 1976). The Causes of failure of weir or barrage on pervious foundation is affected due to subsurface flow and by surface flow. If the failure is caused by piping Garge depicted the seepage retain sufficient residual forces at emerging downstream side. This practice may lead to opining porosity and hollow structure which ended up with failure. Failure by direct uplift that induced by the seepage exerted uplift pressure on the structure which lastly led to failure of structure by rapture. Such failure of structure by subsurface water flow framed by the theory of Bligh's creep.

2.5.1. Bligh's Seepage Theory

According to the Bligh's theory the seepage water flow follows the base of foundation outline of the constructed structures. That means the water creeps along the contour of the structure and the flow length determined as the length of creep. The theory assumes that the head loss is proportional to the creep length which is called the hydraulic gradient (H/L) as shown in the figure 2.5 below

$$L = b + 2d_1 + 2d_2 + 2d_3$$



[Source: Garge, 1976]

Fig 2.6 Bligh's theory

According to the figure 2.6 above the head loss (h) along the creep is calculated using the equation:

$h = \frac{H_L}{L}$…………………………………………………………………...…………………eq2.2.

$$L = 2d1 + b + 2d2 \ + 2d3 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots . \dots\dots . eq2.3$$

Where b= distance between points, d = creep depth

Therefore, the design criterion based on Bligh theory for safety of hydraulic structure on pervious foundation should assume the hydraulic gradient should be within the limit to avoid piping and the floor should be thick enough to avoid rapture. This would be held by determining sufficient creep length between the upper stream and downstream side of the structure using the formula:

$$L = C * H \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots . \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots eq2.4$$

Where, L= creep length, C= is a function of the floor material and H = the difference of water level between upstream and downstream.

Table 2.2 Values of Bligh’s safe hydraulic gradient for different Soil

	Soil type	**Creep coefficient**	**Safe hydraulic gradient**
1	Light sand and mud	18	18-Jan
2	fine sand Coarse	15	15-Jan
3	sand	12	12-Jan
4	Boulder / gravel mixed with sand	9-May	1/5-1/9

The structural stability against different forces of diversion weir structure would be checked based on this theory. Therefore, this study is going to examine the design document and check the safety factor. Since several scholars have thoroughly examined the design of diversion weirs on permeable foundations and come to the conclusion of its construction is directly influenced by the probability of percolation in the permeable soil upon which the apron was constructed (José Ignacio Sarasúa, Paz Elías, Guillermo Martínez-Lucas, Juan Ignacio Pérez-Díaz,José Román Wilhelmi, and José Ángel Sánchez, 2014). Until recently, the creep theory of Bligh (1912) was used to build component-based weirs on sand or alluvial soil. Although this theory makes assumptions about creep length, actual field observations have shown it to be flawed. Based on his research with numerous dams, Lane (1935) suggested a system in which the creep is weighted to account for variation in creep in both vertical and horizontal directions. The method for determining uplift pressure is questioned because it is an empirical method and not based on any mathematical methodology, yet it is an advance over the Bligh's creep hypothesis. Lane therefore proposed a factor of 1/3 for horizontal creep and a value of 1 for vertical creep (Robel, 2009). A composite barrage or weir section is divided into a number of simple standard forms of known analytical solutions using the "method of independent variables," which was developed by Khosla in 1954. These are a straight horizontal floor of negligible thickness with sheet piles at either end, a straight horizontal floor depressed below the bed but without a vertical cut off, and a straight horizontal floor of negligible thickness with sheet pile at some intermediate position. A surface flow problem in a diversion structure, analysis of a hydraulic jump is required.

2.6. The Research Gap

The small hydropower development is challenged by different barriers in Ethiopia for example a large potential area which are remote to the grid and private sector company involvement in the development and ownership is limited. The other gap that is seen at different project site is lack of Environmental Impact Assessment since it is not enforced by the regulator for micro-hydropower projects. The limited participation of upstream and downstream communities during approval of the project determines the success and failure of the projects. The local skills to manufacture operate and maintain the plants are not still well-developed Liu, (2013). Therefore, this study was aimed to assess and evaluate such gaps on the project of Bashiro-Gute. The seasonality of rainfall and reduced availability of water limited the micro and mini hydropower plant development.

CHAPTER THREE

3. MATERIALS AND METHODS

3.1. Description of the Study Area

3.1.1. Location of the study area

The Schemes of Bashiro gute micro hydropower schemes are located at Bona-Zuria woreda in the south-east parts of Sidama regional state, extending from $6^0$31’50” N and 38^o 42'54" E. it is found in Genalle Dawa basin at Basihro Gute catchment. The total area of the catchment is 173.2 km^2.

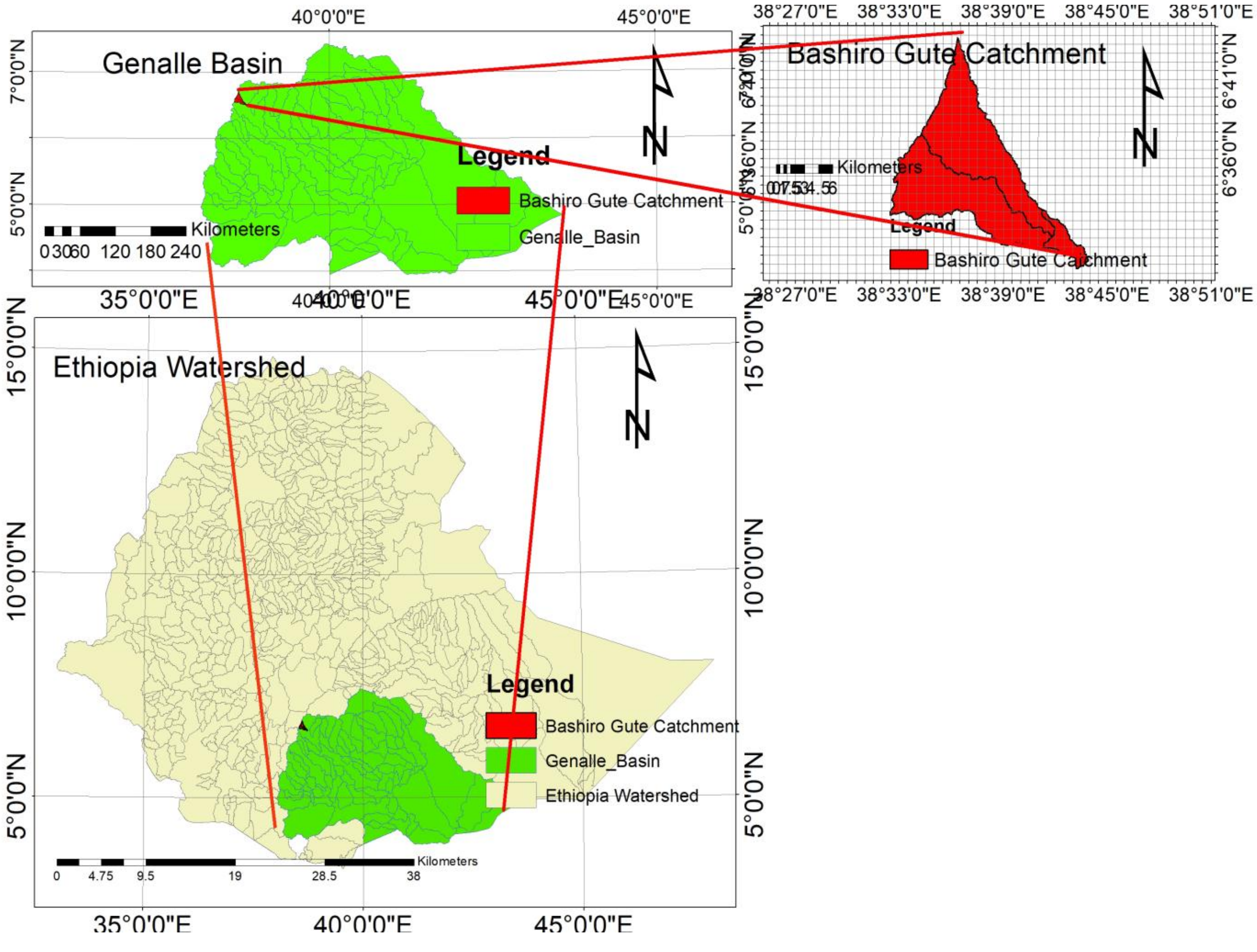


Figure 3. 1. Location of the Study Area

3.1.2. Climate

The study area is ranged in to Woyna-Dega climatic zone since the mean annual temperature ranges between 15.1℃ and22.5℃. The mean annual precipitation is also ranged between 1401mm-600mm. According to ERA, (2013), the precipitation regime of the study area is classified under II-D regime. According to Ethiopian meteorological institute the rain fall distribution is for six months which is from March to September with maximum rainfall occurred between July and August.

3.1.3. Land Use and Soil

The land use land cover of the study area dominantly classified as agriculture, open area, water body and the built-up area according to the report of Sidama Regional State Agriculture and Natural Resource Bureau. The type of land use land cover is kept in changing from time to time for instance, the grazing land, natural forest and fallow lands are decreasing from time to time while cultivated and residential lands are increasing accordingly.

The dominant soil types of the study area are clay, sand, clay-loam, silt-clay-loam, sand-clay, and silt-clay. Land use and soil type have a direct impact on the water flow, structural stability of the construction. According to EMA, (1988) the soil of Ethiopia is commonly categorized under 17 major soil units however the FAO (1998**) s**oil Map classified in to 19 soil units even though which do not coincide spatially with the EMA soil map. The study area soil contains five classifications such as Chromic Luvisols, Chromic Vertisols, Eutric Cambisols, Orthic Luvisols, and Pellic Vertisols.

3.2. Research Design

In this study both quantitative and qualitative approach of research design were applied since the data that were collected include the measured empirical, time series, interview and exiting document reference data. The study relied on both descriptive and explanatory approaches when the examining of the current trend and basic principles in the storm water drainage construction was carried out. These held on by photographing the areas suffering from inadequate storm water drainages, making a basic diagnosis of the storm water drainage problem and defining the solution for the problem. General procedures followed in this thesis have been presented below Figure3 by chart form.

3.3. Data Sources and collection

The required data for this study was obtained from field survey and field observations, USGS website, Ethiopian meteorological Institute, Sidama Regional state Water and Mining, Agriculture and Natural Resource Burea, Sidama, Development Corporation, and from community. The data sources are primary and secondary as it was listed above.

Table 3.1. Data Types and Sources

No	Data type	size	Data Source
1	Stream flow	30	Ministry of Water and Energy
2	Climate (Rainfall & Temperature)	30	Ethiopian Meteorological Institute
3	Hydropower project document	1	Sidama water and Energy Bureau and GIZ
4	Interview	1	

3.3.1. Primary data

Primary data collected for these studies were the existing dimension of weir structure, the interviews opening. The data were obtained from field visit and direct measurements.

3.3.1.1. Field visit and measurement

The data which was diversion weir dimension, head race canal, fore bay and penstock design practice were obtained thorough field visit or measurements. Farmers and professional specialists from the Bona Woredas office of Water and Mines, Sidama Regional State Agriculture and Natural Resource and Water resource and mining Bureau, Sidama development Corporation, community elders and representatives were interviewed.

3.3.2. Secondary Data Collection

The secondary data collected were rainfall data, design document, design specification and design drawing of projects documents, hydrometeorology data and digital elevation models. From these materials detail evaluation on the design development of diversion weir, power canal and settling basin related to hydrological, hydraulic analysis of surface and subsurface flow, dimension of weir and structural analysis were done on the selected schemes. Guideline and manuals design of micro hydropower projects were collected from Sidama region water, Mines and energy Bureau. Deep literature review on the design development of diversion weir on different authors‟ books, journals, modules, lectures and other published and non-published subject area materials were referred.

3.4. Sampling Techniques and Sample size Determination

The Sample frame from which the primary data were going to be selected was the officials from governmental offices such as Sidama Regional State water and mining, Agriculture and Natural Resource Bureau, Kebele Officials and Non-Governmental offices Sidama Development Corporation, GZT coordinate Hawassa Branch. Where the community elders in the project area were the sample frame.

3.4.1. Sampling Techniques

Sampling the respondents for this study were made to get detail information on the hydropower scheme and those who have enough information on the project was selected. Therefore, non-probability sampling method that was purposive sampling was applied to select the study area since the projects were failed to give the operation. The purposive sampling method was applied to decide the interviewees. Nine respondents were selected from the key stakeholders.

3.4.2. Sample Size Determination

The sample for the study as it was described above was officials from governmental offices and non-governmental representatives. The proportion of the sample was tabulated below in the table 3.2.

Table 3.2. The Selected Sample for Interview and focus group Discussion

	Offices	Total	Selected
1	Sidama Water and Mining Bureau	2	1
2	Sidama Environmental Protection Bureau	2	1
3	Sidama Development Corporation Bureau	2	1
4	Bona Woreda Water and Mining Office	2	1
5	Kebele Administration Representative	5	2
6	Community elders	5	2
7	GTZ representative	1	1
		Total	9

3.5. Data Collection Techniques

The data from primary source was collected through field observation/site investigation, surveying and measuring the drainage. The secondary data which were discharge of river, climate (rainfall), land use land cover and soil data was collected from data base and documents.

3.6. Analysis Method

In the analysis of hydrologic and hydraulic parameters were used to analyze the discharge for diversion performance, storage and conveyance to the penstock by comparing the design document with the capacity of the designed structures. For structural analysis the dimensions of structure were determined and compared with past design. The primary data that was collected through interview also analyzed by textual methos. To analyze the causes and way of management of the schemes the following conceptual framework was followed.

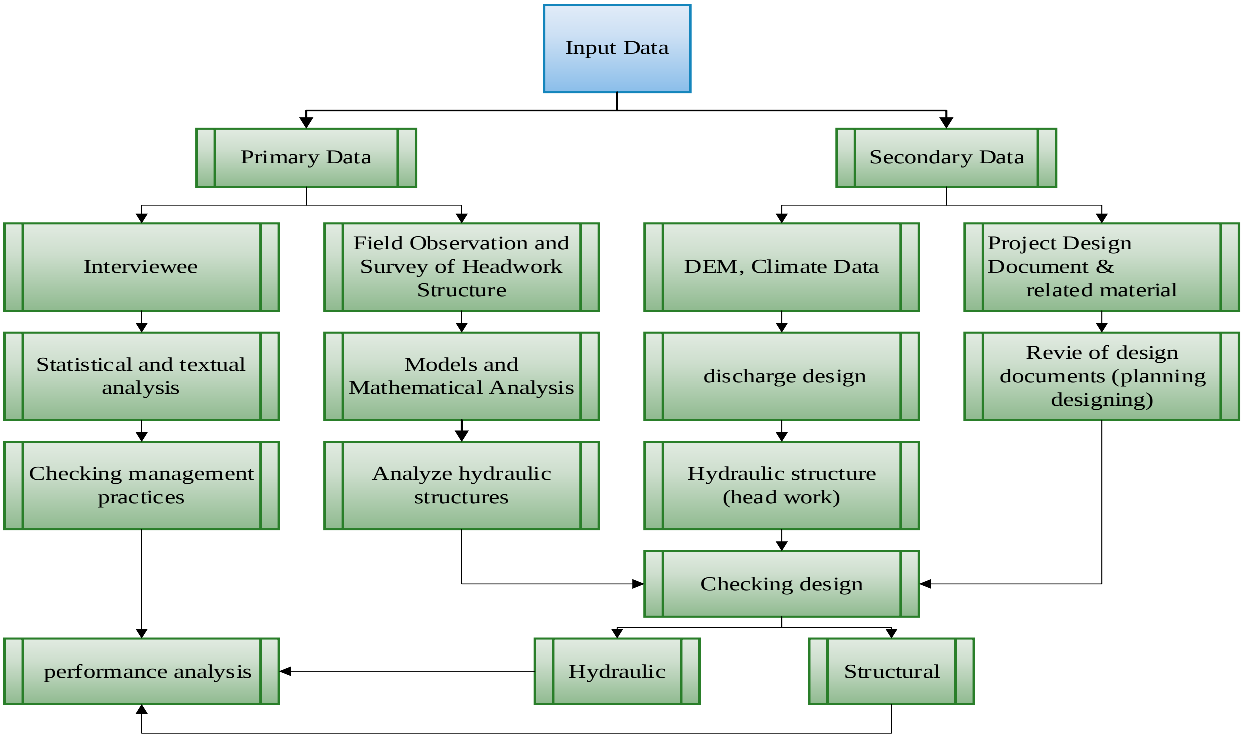


Fig 3.2 Conceptual Framework of the Study

3.6.1. Spatial Data Analysis

The study area elevation, slope and stream flow direction, accumulation and stream order were analyzed using GIS 10.7, and google earth map. The data was obtained from USGS (earth explorer website.

3.6.1.1. Digital Elevation Model

The study area digital elevation was ranged from 1790 m to 3080 m which indicates that the area is mountains. One of the spatial factors which determine the diversion weir site is the topography of the desired site. The figure below 3.3 has been show the elevation difference of the study area location.

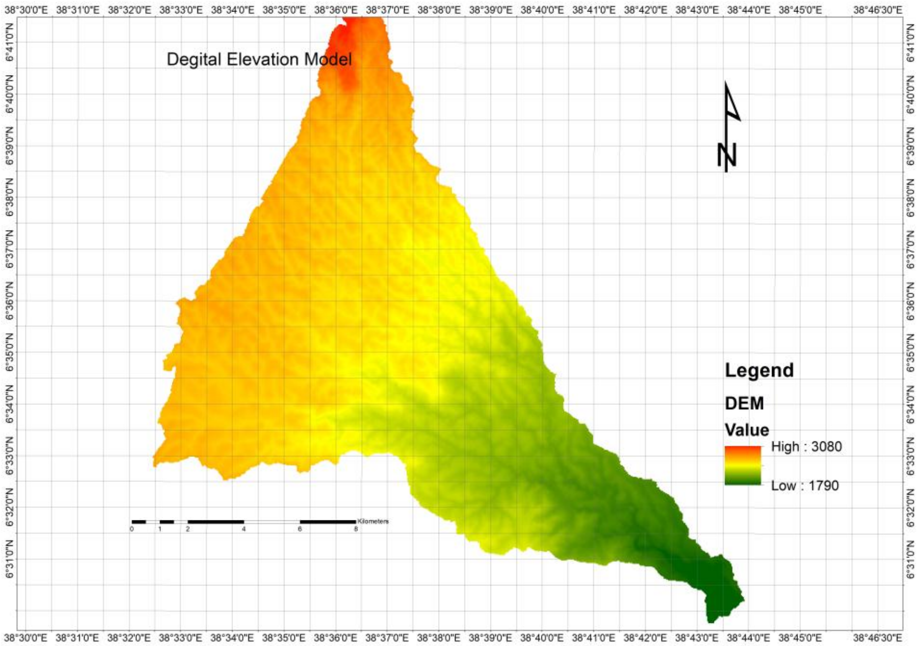


Fig 3.3. Bashiro Gute Sub-watershed DEM

3.6.1.2. Slope of the Study Area

Bashiro Gute catchment slope map was developed using DEM by the aid of ArcGIS software. International Land Reclamation Institute (ILRI, 1994 has classified a slope of an area under five class which are class one indicates flat which is the area under 1%, class two represents slightly sloping area which ranged between 1 to 5% of slope, while class three, four and five represents the area under 5-10%, 10 -20% and more than 20% slopes. The figure 3.4. Indicated that the slope of the study area was ranges to 142.4%. Majority area of the catchment is situated above 5% of slope which indicates the catchment is steep slope.

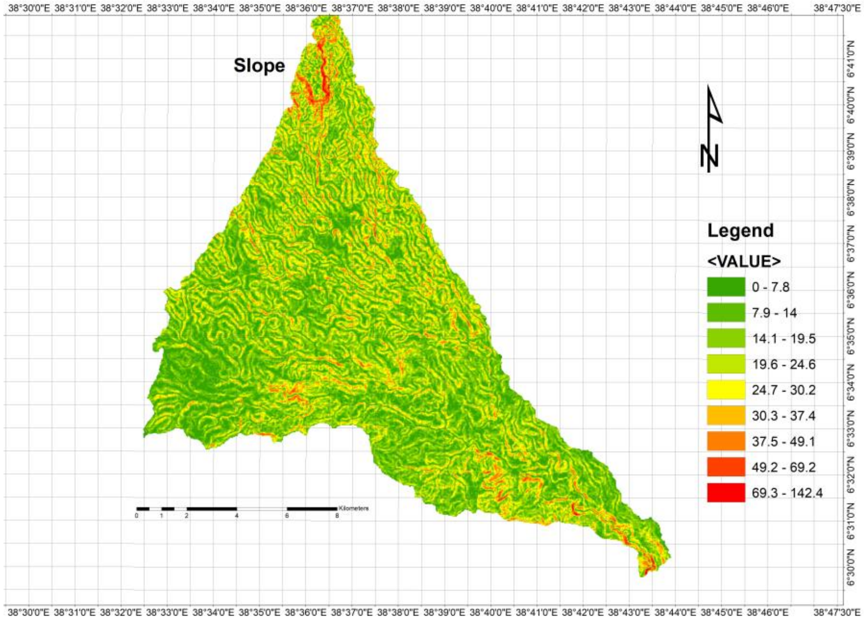


Fig 3.4. Slope of the study Bashiro-Gute Sub watershed

3.6.1.3. Hydrology Map

The study area is located at north end of Genalle River Basin and the stream is drained in to Genalle River. The hydrology of the area which is the stream flow, direction accumulation and the stream order is illustrated in the figure 3.5 below. In the catchment area the stream order is ranged from 1 to four.

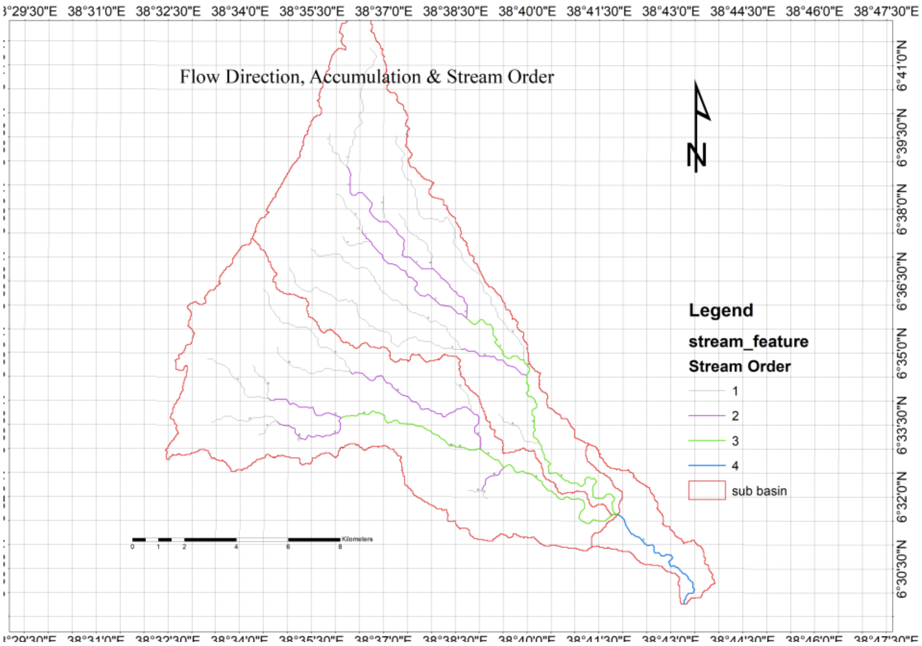


Fig 3.5 Stream Flow Direction, Accumulation & Stream Order

3.1.1. Rainfall Data Analysis

Since the stream of Bisharo Gute is not gauged stream, the design discharges for Bashiro small hydropower was determined based on the rainfall frequency. The average daily data was used to determine design. For this purpose, the development of Intensity Duration Frequency Curve (IDF) was conducted based on the rain gage station data which was “Worancha” rain Gauge station around the schemes. However, the long period of rainfall data is expected to be consistent and homogeneous. To check the data quality filling the missing data, checking the consistency and homogeneity of the neighboring stations had to be done. For such purpose the neighbor Worancha, Bansa Daye, Hagere selam, Aleta wondo, and Yirgalem stations were applied. All the past twenty-seven daily rainfall data of each station were obtained from Ethiopia Meteorological Agency.

Table 3.3. Rainfall Data in the Stations

No.	Stations	Latitude	Longitude	Altitude	Recording year
1	Worancha	6.5	38.66	1975	1994-2021
2	Bansa-Daye	6.42	38.03	1914	1994-2021
3	Hagere-Selam	6.28	38.31	2840	1994-2021
4	Aleta -Wondo	6.35	38.25	1947	1994-2021
5	Yirgalem	6.41	38.8	1786	1994-2021

[Source: NAM, 2022]

3.1.1.1. Estimating Missing Rainfall Data

The past twenty-seven-year from 1994 to 2021 rainfall data of Bona meteorological station was checked using Arithmetic mean method since the normal annual precipitations at surrounding gauges are within the range of 10% of the normal annual precipitation at Worancha station. The arithmetic mean method computed using the formula 3.1 below.

$$Px = \frac{1}{m}(P1 + P2 + P3 \dots \dots \dots Pn) \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots .. eq3.1$$

Where: P_x- missing rainfall data (daily, monthly, or yearly), $P1$, $P2$, -rainfall data at a nearest different station (daily, monthly, yearly), N_x -mean annual rainfall at missed station, n is number of nearest stations

3.1.1.2. Consistency Test

In this study double mass curves method was used to check the consistency of the data. The double mass was developed by plotting the average cumulative precipitation of the neighboring station along the Y-axis and the cumulative precipitation of Worancha along X-axis. The coefficient was computed using the equation 3.2 below.

$$P_a = \frac{S_a}{S_o}(P_o) \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots eq3.2$$

Where$P_a = adjusted\ precipitation,\ P_o = observed\ precipitation$

$S_a = slope\ before\ to\ the\ break\ in\ the\ curve, S_o = slope\ after\ the\ break\ in\ the\ curve$

The result has indicated that there was no any break of slope line in the curve and which indicated that the data didn't violet the consistency assumptions.

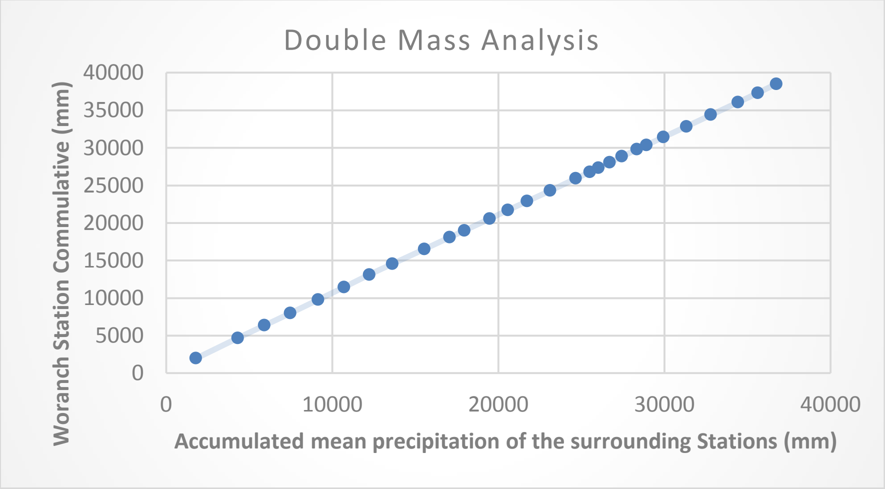


Fig 3.6. Consistency Test for Worancha station

3.1.1.3. Homogeneity Test

In this study homogeneity among the data was tested using XLSTAT software and checked based on the Pettitt's test method. The decision of existence of homogeneity or nor was conducted the calculated p value of the test.

Table 3.4. Homogeneity Test

Tests	p-value (Two-tailed)	alpha
Pettitt's test	0.1372	0.05

According to the table 3.4 Above the calculated p value was greater than the significant alpha level. Therefore, the null hypotheses were rejected there is homogeneity in the data records.

3.1.2. The Rainfall Frequency Analysis

3.1.2.1. Probability Distribution

The Gumbel's distribution and log Pearson's III distributions method was evaluated for the design of hydropower generation since ERA, (2013) recommended the two methods in hydrologic analysis. The selection of the suitable probability distribution model was decided based on goodness of fit tests. The goodness of fit test was conducted using Kolmogorov Simonov and chi-square test methods. Three probability distribution method which are Gumbel's extreme value distribution, Alternative (Frequency Factor) Method, the Log Pearson type-III Distribution was checked to decide the best distribution method.

A. Gumbel's extreme value distribution

Under the Gumbel's extreme value distribution, the probability distribution was calculated using the following empirical equations:

$$X_t = \bar{\mu} + k_t * \sigma \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .. eq3.3.$$

$$k_t = \frac{y_t - y_n)}{S_n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.4$$

$$y_t = yt = ln(ln(\frac{T}{T-1}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.4$$

Where X_T =Maximum rainfall depth in the given return period, μ = Reduced precipitation average, σ = standard deviation of Reduced precipitation, K_T = coefficient factor, Y_T= expected rainfall, Y_n= average of rainfall for the given data period, Sn= standard deviation for the given time period and T= return period

B. The Log Pearson type-III Distribution

The probability under Log Pearson Type III distributions was calculated based on the following steps

- Arrange the rainfall series Xi.
- Calculate the logarithms Yi of X, $y = \log x$

- Calculate the mean $\bar{Y}$, standard deviation S_y and skew coefficient c_s

$$\bar{y} = \frac{\sum_{i=1}^{n} yy_i}{n},\ S_y = \sqrt{\left[\frac{\sum_{i=1}^{n} yy_{i-\underline{y}}}{n-1}\right]}\ , Cv = \frac{\underline{y}}{s_i} = c_s = n\left[\frac{\sum_i^n (y_i - \underline{y}\,)^3}{(n-1)(n-2)s_y} = \right]$$

From the parameter of $y = logx$, x was calculated using the equation:

$$log\ log(y) = \underline{y} + K_t * s_y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\ldots\ldots\ldots\ldots..\ldots\ldots\ldots\ldots\ldots eq3.5$$

The coefficient K_t was calculated using Kitt, 1977 equation:

$K_t = z + (z^2 - 1)k + (z^3 - 6z)k^2 - (z^2 - 1)k^3 + zk^4 + \frac{1}{3}k^5$ Whereas k was computed using the relationship between skewness.

$$k = \frac{s_c}{6}$$

The value of Z corresponding to an exceedance probability of P (P=1/T) can be calculated by finding the value of an intermediate variable w:

$$z\ = w - \frac{2.0515517 + 0.802853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3}$$

$$w\ = [ln\ (\frac{1}{p^2})]^{\frac{1}{2}} \quad 0 \leq p \leq 0.5$$

3.1.2.2. Goodness of Fit

The best fit of the precipitation was decided based on the test values of Kolmogorov-Smirnov and the Chi-Squared tests at the significance level$\propto = 5\% = 0.05$. Under each test the hypothesis Ho and H1 was tested

3.6.2. Hydrologic Analysis

Hydrological data is needed in the design of diversion weir to fix the dimensions of hydraulic structures. To obtain this information for many year flow records were needed and the frequency analysis has to be done. However, the study area no river gauging station at the weir location; the discharge at the weir would be estimated by correcting the rainfall at some known locations similar watershed character. Since the stream is not gauged different method could be applied to assess the potential of the schemes. According to Garba, (2013) the dominant method is analogue methods which is stimulation of long-term flows of the river for the selection of design flow based on the analogue river. After design flow determination the other parameters such as topographic assessment of the site was verified to fix the appropriate site as well as the net available head was also determined. The positions of diversion weir, canals, forebay penstock and the powerhouse was determined based on the slope and topography of the area. Generally, the hydrologic design of the system was conducted by carrying out the following main activities.

3.6.2.1. Watershed Delineation

The watershed delineation was done Based on the digital elevation model of the study area obtaining from the USGS website by ArcGIS10.7. In the process of catchment delineation, the primary activity was filling the sinks in the DEM by using a fill tool under spatial analysis. Filling DEM sinks help to achieve the corrected delineation line and avoid inappropriateness. In such a way the delineations process has been processed by following the steps described below.

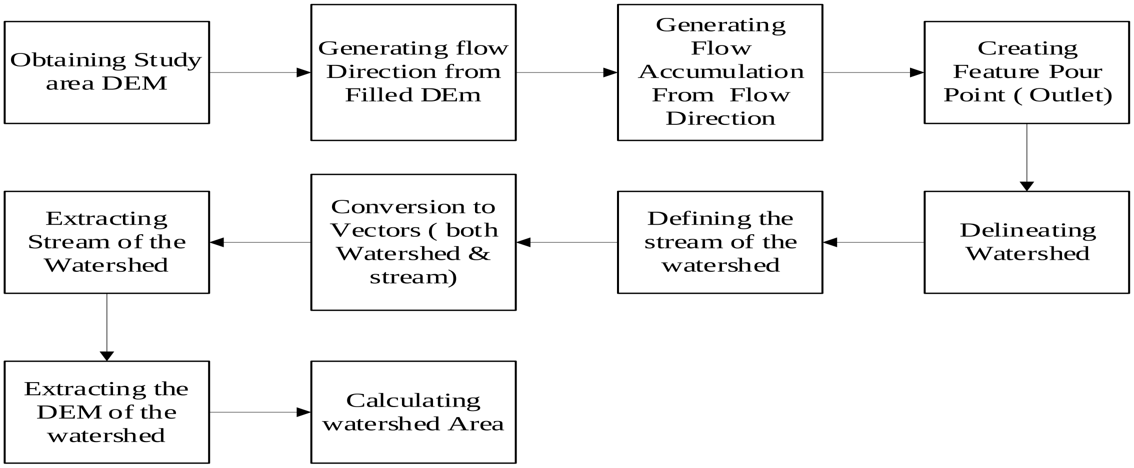


Fig 3.7 Conceptual steps of watershed Delineation

3.6.2.2. Stream Flow Simulation

Scholars used different methods to simulate the stream flow from gauged station to ungauged river stream site by applying different parametric relation. The dominant method that has been used widely was scaling down stream flow data of correspondent gauged site of larger catchment with similar characteristics to the catchment of interest. The catchment area, topography, climatic condition its pattern and distribution soil characteristics are the common characters that was applied to determine the analogue gauged station. In this study this method was applied to simulate the long-term river stream data for the interested site. According to Ahiablame L, et al, (2013), we can apply analogue method for site in interest when the neighboring gauged site is within 30 km radius. In our case Yema catchment and Bashiro Gute catchments are 26 km apart. The relationship between the gauged station and the ungauged station was developed using the equation 3.6

$$\frac{Q_1}{Q_2} = \frac{A_1}{A_2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.6$$

Where, $A1, A2$ =drainage area of the interest area and an analogue site respectively, $Q1, Q2$ represents the discharges at the site of interest and an analogue site (yema Hagereselam).

3.6.2.3. Determination of design flow

The design of hydropower discharge was determined based on the flow duration curve which was developed using the relationship of the gauged and ungauged stations. the gauged station is analog station and Bashiro gute was ungauged therefore the Bashiro gute FDC was developed from the relation. The Flow Duration Curves (FDC) of the proposed diversion sites was developed by arranging the discharge that was simulated from maximum to minimum and ranked. The exceedance and the equal of the time flow were estimated using the equation:

$$p = \frac{R}{N+1} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. eq3.7$$

Where, p= percentage of the time a flow equals or exceeds from R=1 to N

Then the FDC was used to estimate the potential discharge based on the potential power generation classified as minimum, small, median or mean potential power with the exceedance provability.

3.6.2.4. The Potential Head Determination

The potential head was determined based on height differences between headrace and tailrace. It was conducted by from the contour map of the area which was developed using DEM of the drainage area and survey data that was collected during the field survey.

3.6.3. Headwork design

To have sustainable and stable headwork of the diversion structure hydraulic calculation was conducted for fixing different weir structure dimensions. For this purpose, the following data has to be collected. These are maximum flood discharge at the construction site, high flood level before construction, downstream bed level, full supply of canal level which conveys of the water to the penstock, allowable flux and Lacey's silt factor (Garg, 2005).

3.6.3.1. Hydraulic Calculation

After the data of hydrology typical maximum discharge, hydraulic calculation for the structure was conducted. Since the diversion weir for the hydropower plant was over drop weir type the calculation was conducted based on the theory of Lacy's. Therefore, the various elevations was started to fix using the equation of lacy's:

$$Length\ of\ water\ way\ (L) = 4.75\sqrt{Q} \ldots\ldots\ldots\ldots\ldots\ldots\ldots.\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..eq3.8$$

Where L= length of waterway (m), Q maximum flood discharge (m^3)

According to the Lacey's theory the L may decreased for boulder reach of the river whereas increased by 1.2 to 1.4 time for silt factor 1 (Gorthi, 2020).

Then discharge per meter width of the river was calculated also by applying the following empirical formula:

$$q = \frac{Q}{L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.9$$

Where, q= discharge per meter width (m^3/m),

In third step the Regime scour depth was determined using the equation of Lacy's formula as:

$$R = 1.35 \left[\frac{q^2}{f}\right]^{\frac{1}{3}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.10$$

Where, R= Regime depth (m)

Then from regime depth the regime velocity was calculated by the equation 3.11 below.

$$v = \frac{q}{R} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.11$$

Where, v = Regime velocity (m/s)

Then using the regime velocity and acceleration of gravity the velocity head was calculated using the equation 3.12 below

$$h = \frac{v^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.12$$

Where, h-= velocity head (m)

Then the water level and total energy level such as total energy line (T.E.L) level at downstream (d/s) and upstream (u/s) and upstream high flood levels were computed using the equations 3.13, 3.14 and 3.15 below.

$$d/_S\,(T.E.L) = high\ flood\ level\ before\ construction\ of\ weir\ HFL + \frac{v^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.13$$

$$u/_S\,(T.E.L) = d/_S\,(T.E.L) + Afflux \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.14$$

Then upstream High flood level (HFL) was computed by

$$HFL = {}^{u}/_{S}\,(T.E.L) - \frac{v^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.15$$

Then head over the crest computed from discharge over crest of the weir: -

$$q = 1.7h^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.16$$

Where q= discharge over the crest, h= head over the crest

$$Crest\ level = {}^{u}/_{S}\,(T.E.L) - h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.17$$

$$Pond\ level = full\ supply\ level + loss\ of\ head \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq\ 3.18$$

The protection against scour was also calculated using the equation 3.19

$$Level\ of\ bottom\ of\ {}^{u}/_{S}\ pile = {}^{u}/_{S}\,HFL - 1.5R \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.19$$

$$Level\ of\ bottom\ of\ {}^{d}/_{S}\ pile = {}^{u}/_{S}\,HFL\ after\ retrogration - 2R \ldots\ldots\ldots\ldots\ldots\ldots 3.20$$

3.6.3.2. Weir Design

After fixing the hydraulic elevation the weir well dimensions were calculated using the steps that were describe by Garg, (2005). According to the Garg the top width of the crest, calculated using the formula:

$$a = \frac{h}{\sqrt{\rho}}\ or = \frac{3}{2}\frac{h}{\sqrt{\rho}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.21$$

Where h= height over the crest and $\rho =$ specific gravity of the material of weir.

The bottom width of the weir B, was computed from the stability considerations. Therefore, the base width was taken the largest value of the three.

$$B = \frac{(H + h)}{\sqrt{\rho}}, \frac{(H + h)}{\mu\rho}, or\ a + 0.8H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq3.22$$

3.6.4. Structural Stability Analysis

The weir body is designed to be safe at prevailed critical conditions of its disturbing forces. Taking in to account the geological foundation conditions at the proposed weir site, the stability analysis of the designed weir was carried out by considering the various external forces acting on it such as Self-weight of the structure, external Horizontal water pressure, Water weight and Silt pressure. The stability of diversion weir headwork can be analyzed under pond level/no flow condition (static case) and for high flood level condition (dynamic case) for the following condition

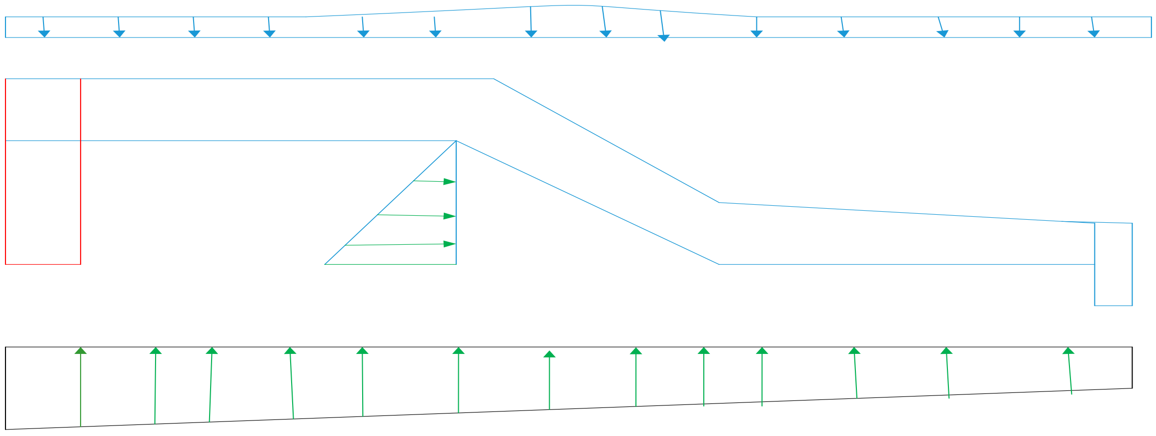
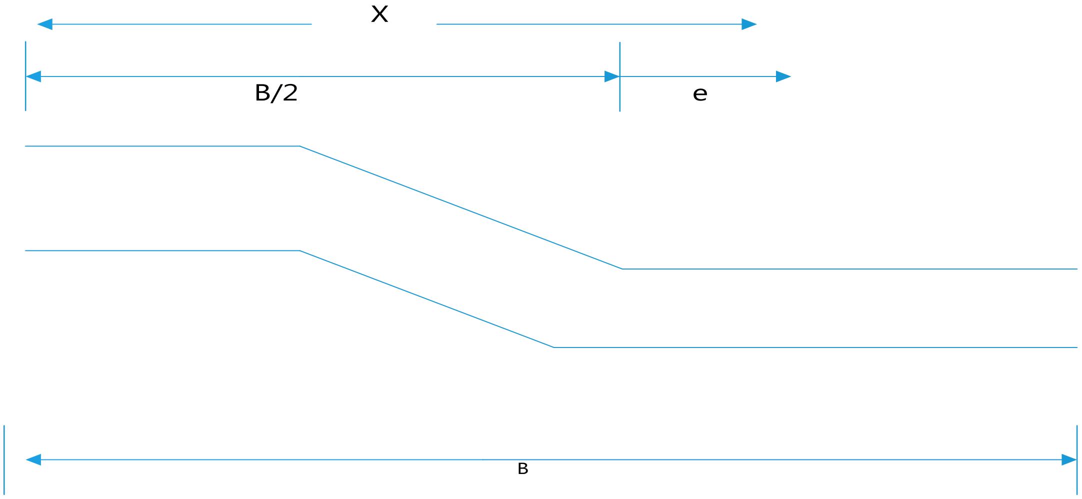


Fig 3.8. Force acting on the weir

3.6.4.1. Stability Against Overturning

The stability issue against overturning is a factor to take into consideration during the weir structure design. The summation of all moments about the structures has to be kept at balancing condition with the vertical forces. According to Baban, (1995) a factor of safety of 1.5 to 2.0 is has to be applied for safety against overturning. Therefore, in order to avoid lifting up the structure’s heel occurrence, the resultant force has to be passing through the middle third of the structure’s base.



$$X = \frac{\sum M}{\sum V_f} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots .. eq3.23$$

$$e < \frac{B}{6} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots eq3.24$$

$$e = \left(\frac{B}{2} - X\right) < \frac{B}{6} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots eq3.25$$

Where, $\sum M$ = all moments at the structures toe, $\sum V_f$= all vertical forces, X= distance of the resultant of the forces from the toe, B= width of the weir base, e= eccentricity

3.6.4.2. Stability Against Sliding

The tackiness of apron at different points along longitudinal section is determined based on the stability against uplift pressure. According to Garg, (2005) the maximum unbalanced head between the uplifting pressure head and depth of surface water above the apron is calculated along the structure based on the scenario of HLF, pond level flow. To carry out the activities the maximum head was calculated and then the necessary thickness of the apron was calculated from the density of apron material using the following equations.

$$t = \frac{h}{G - 1} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots eq3.26$$

Where, t= Thickness of apron at a point h= the unbalanced head between the uplifting pressure head and surface water depth, G= Density of construction material for apron 2.24 for concert

CHAPTER FOUR

4. RESULT AND DISCUSSIONS

Under this part the condition of the hydropower schemes was assessed using the based on the design document and the field observation data. Then the potential of the scheme was examined by following detail designing procedures. Finally, the factor that was contributed to the failures of the scheme was assessed using the technical designing design material and interviews information.

4.1. The Existing Status of Hydropower Schemes

The Bashiro-Gute small hydropower started in 2011 and it was designed for the 25 years according to the information of Water and Mining office of Bona Zuriya woreda. However, it only functioned for two years. The diversion weir, and power canal structure was broken, overturned and totally stopped its function. During field observation the structure of scheme was examined and the picture that captured during the observation indicated this reality. The type of the problem was weir overturned whereas, the canal experiencing seepage problems.

Fig 4.1 Bashiro Gute Headwork present condition

During the field surveying dimension of the weir and canal structure, construction material type, the selection site condition and its appropriateness, the apron position were examined. As it was measured in the field the Weir dimension were 0.9m top width, 2.0 m bottom width, 2.01m crest height and 4.5m crest length and floor thickness was 0.9m. The weir was constructed by masonry with the absence of apron at upstream but only provided at downstream side. The situations led to scouring effect of main weir. Even though Apron was constructed at downstream the dimension was 1m in length. Even though there was diversion weir, the structure have the sealed the head regulator and sluice gate. Hence there was no discharge, silt and other debris regulation and controlling mechanisms.

The power canal dimension that was measured through field surveying were bottom width = 1m, depth = 1.5m, the canal length was 53m. It was constructed by wet masonry and almost it was silted up covered with debris, grasses. Some part of the canal sidewall was collapsed and fallen in to the water way of the canal.

Fig 4.2 Forebay Structure

Even though the scheme has fore bay structure to control controls the water entering to the penstock. It has structures trash rack and totally silted up. This also controls the water entering the penstock.

Fig 4.3 Current Condition of Penstock

According to the picture the penstock was made up of steel with 400mm diameter, 100m length and with 5.5m elevation differences. However, the anchor block which supported the penstock was broken. If the diversion weir and power canal functioned it would give services with small part maintenance.

Site selection condition of the scheme of diversion structure such as weir, power canal and penstock is determined by the topography of the area. According to the result of the analysis the area that was selected for such structure was steep area. The slope was ranged between 18% to 27% which considered as very steep and class five, according to the classification by International Land Reclamation Institute (ILRI, 1994). In such kind of topography, the diversion structure needs upstream as well as downstream apron with sufficient dimensions. However, there was no apron structure and sufficient retaining wall around the diversion weir.

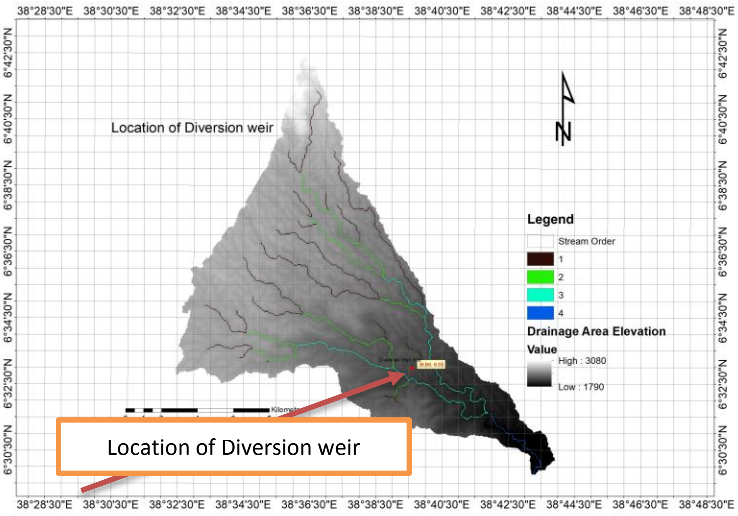


Fig 4.4. Location of Bashiro Gute hydropower Diversion Weir

4.2. Factors of the Scheme Failure

The reason behind the failure of the hydropower scheme was assessed based on the stakeholder interview, by reviewing existing documents and field observations. Therefore, after here the cause of the failure was examined.

4.2.1. Designing Process and Design document Review

The hydropower efficiency is determined by the condition the environment where the scheme is installed, the physical structure suitability and technologies that were used. The development process also determines the high-level design features and its worth. In development process the selection of relevant design features, identification and quantification of stakeholder's interest and the management mechanism has to be accomplished.

In this regard the assessment has indicated that Environmental impact assessment was conducted during the construction time of the project. However, it is hard to find the detail design document of the project, handover and management mechanism. Stakeholders of the scheme such as the German International Cooperation Energy Coordination, Sidama Development Cooperation, the water and mining office of Bunazuria woreda and Sidama regional state bureau assured that no one could provide the detail design of the hydropower features. Except some fact sheet which was indicated in the fig 4.3 below the project lacks reconnaissance, preliminary detailed study and the design decision process. In hydropower scheme development reconnaissance investigation is mandatory since it gives information on the regional geology to identify local rocks, enquiry on maximum flood level and seasonal discharge, type and amount of silt, nearby well is investigated to foundation and water level, reports of finished projects for decision. During preliminary study tentative lay out and preliminary design and estimation for the diversion head work at each site to compare cost is carried out. Whereas the detail investigation is required to do detailed topographical surveys such as contour map include detail section and longitudinal section of the river at upstream and downstream is decided. Moreover, the hydrological investigation such as maximum flood condition is also done during the detail investigation time. However, at the Bashiro Gute hydropower scheme the existence of such important study steps was missed.

AMES-E (Hawassa)
Hydropower service center **BESHURO GUTE(Ererti) Micro Hydropower** **Fact sheet**

MHP demo schemes

The proposed site is situated on Ererti River in SNNPR region in Sidama zone, Beshuro Gute Kebele near Bona town. From Hawassa in direction to Moyale road after 38 km asphalt road, then the all weather gravel road to bona is about 80 km. It is about 5 km to reach Beshuro Gute village.

The GPS coordinates of the weir site is 37 N0463254 UTM 0724661.

The survey site is situated 6km away from the main street of Bona town. The total population of the Kebele is about 5,945. And there are about 563 households around the envisaged hydro power station. The villagers of Beshuro Gute are very cooperative and eager to participate within their possibilities. A cooperative to manage the future power plant is already formed.

The proposed site is accessible. The masonry as well as all excavation will be supplied by the community. A village map based on GPS data is done and it serves as basis for the required grid design of the Beshuro Gute.

There will be three foci. First gtz project will connect public services (school, health centre, church, market.

Second the will be efforts to create local businesses which utilise the available electricity for productive use.

Third it is proposed to draw a power line along the accessible main road, from where private households can take electricity for private use.

Ererti fall

Community meeting

Weir when tested

Community participation

Forebay

Canal

Penstock and power house

General Information

Region/Zone	:	Southern region/Sidama
Woreda/Kebele	:	bona/beshro gute
River	:	Ererti river
Coordinates (weir site)	:	37 N0463254 UTM 0724661
Head/Flow/Pot. capacity	:	16m / 300 l/s / 33 kW
No. households	:	563
Social connections	:	Church, school, commercial centre

Technical Specifications

Civil structure		
Canal	:	Lined masonry/length: 53m
Penstock	:	Ø 400 mm/length: 42m
Head	:	16m
Flow	:	300 l/s
Electro Mechanic		
Turbine	:	T 15, net head 16m, 33 kW
Turbine manufacturer		Selam Plc

gtz

Fig 4.5. Bashiro Gute (Ererti) micro hydropower factsheet

Therefore, one could summarize that the failures of the structure was due to technical feasibility failure of the design.

4.2.2. Interview of the stakeholders

The public and stakeholder participation is a backbone of any project which is aimed to establish at community scale. The community participation and degree of participation Bashiro Gute (Ererti) micro hydropower scheme was assessed using the community and stakeholder interview. During the interview process public participation role in hydropower plant scheme development and management was assessed by provide three questions to the interviewee. The first one was whether they participated or not during the feasibility, detail design and final decision-making phase, if there was participation on the study what was the frequency of the participation, opportunity to forward their opinion. For this question among 9 interviewees 6 (67%) of them answered that they didn't participate at any phased of the study. But three of them said they were participated during feasibility study but not final phase. Those who participated were asked about the frequency of participation whether it was sufficient or not. Almost all (100%) believed that the participation was not sufficient to forward their interest. The degree of ownership on the hydropower project is also determines the effectiveness of the schemes. All interviewee (100%) responded that there was no clear procedure how to own the project and it was not handover to any community and government office legally. In addition to this the regional state water and mining bureau was asked who do govern the schemes. Their response for this question was there is no enough understanding and information from the very begins of the project. However, at current time the bureau has started to plan the maintenance activity and further development options. The problem is no one can find the detail design document and which has a probability to lead wrong conclusions

4.2.3. Field Observations

The current state of hydropower on the Bashiro Gute was stopped to functioning specially the diversion weir and power canal structures were failed. According to the field observation the researcher pinned out the main problem was related with the hydraulic engineering activities. The diversion weir has no apron and the construction material was less quality. In addition to this, the professional has said that as they heard from the community during the construction no follow-up and monitoring from the government

4.3. The hydropower potential Analysis

4.3.1. Hydrologic Analysis

The hydrologic analysis of the hydropower was started by delineation of the catchment area. After delineating the interested watershed fixation of hydrologic parameters was conducted using different empirical methods. Since the stream was not gauged its stream flow curve was developed using the simulated stream flow data from analogue gauged stream. According to

Garba, (2013) the analogue methods is dominant to simulate stream flow for ungauged station from analogues gauged station

4.3.1.1. Watershed Delineation

The watershed delineation was done based on the digital elevation model of the study area by following the conceptual steps that was illustrated in fig 4.6 using ArcGIS10.7. The total watershed of the Bashiro Gute covers 173.2km^2 area and divided in to three sub basins as illustrated in the fig4.6 below. The specific hydropower was located at subsin1 and covers 83.7km^2 drainage area.

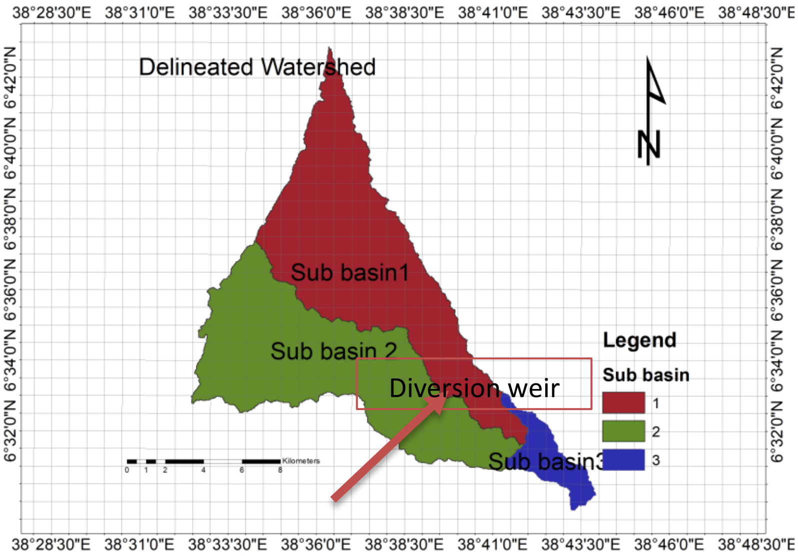


Fig 4.6 watershed of Bashiro Gute (Ererti River)

Since the river was not gauged the flow was simulated from the analog station by using the analogue simulation method.

4.3.1.2. Design Discharge Estimation

The relationship between the gauged station and the ungauged station the flow rate of Bashiro Gute was simulated using the analogue method which was described in equation 3.5.

$\frac{Q_1}{Q_2} = \frac{A_1}{A_2}$, A1 is drainage area Ererti river stream (Bashiro Gute), A_2 is the analogue river stream drainage area, Q_1 represents the Bashiro gute the discharges, Q_2 represents analogue site.

After simulating the stream flow at the interested site, the stream flow duration curve was developed as it was illustrated in the fig 4.7 below. The development method of stream flow curve was described under the methodology part.

Based on the empirical relationship the Ererti River stream flow which was simulated from the analogue station was summarized below in the table 4.1

Table 4.1 Simulated Stream flow of Bashiro Gute (ErertiRiver)

1988	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017
0.69	0.71	0.80	0.58	1.28	1.08	0.95	0.84	1.47	1.11	1.15	0.39	0.66	0.69	0.41	0.56	0.52	0.64	0.90	0.96	0.60	0.57	0.98	0.72	0.53	1.01	1.21	0.73	0.92	1.13

The probability of exceedance was also calculated using the equation which was illustrated in equation 3.6 under methodology part and tabulated in the fig 4.7 below. It was conducted by ordering the flow from maximum to minimum and the detail was attached at the annex.

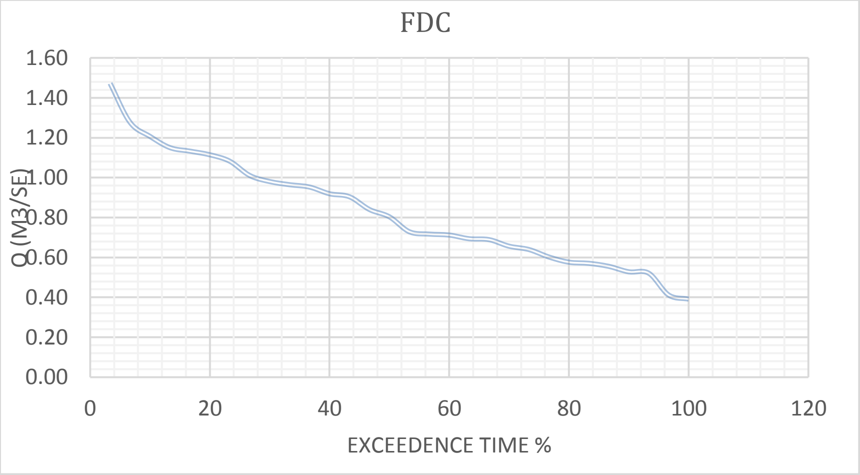


Fig 4.7 Flow Duration Curve

Based on the flow duration curve the hydropower potential of Bashiro-Gute was selected for mean power generation which was Q_{50} exceedance time discharge. From the fig 4.7 the discharge equals to 50% of time is $0.8m^3/s$. The hydropower could give mean power corresponding to the discharge rate at Q_{50} or exceedance time. For the existing hydropower scheme $300\ l/sec$ discharge was selected as a discharge design however nobody could know whether it is for minimum, small, median or mean potential power generation or other else since the minimum potential power computed from the minimum flow available for 100 % of the time, small from the flow value of 95%, and mean is from 50% respectively.

The available geometric head which was the difference between the head race and tailrace of the scheme was 29m as measured during the filed survey but for existing hydropower they adopted 16m as design head. Therefore, based on these geometric head and the FDC curve value different potential of power which could be generated from the Bashrio Gute River was calculated using:

$$p = \eta\rho gHQ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots eq4.1.$$

Where P= potential power generation potentials, ρ= density of water =1000 kg.m-3, g= gravitational acceleration m/s^2, Q= design flow (discharge) m3/s, H=gross head, m, and η= overall efficiency of the plant

Table 4.2 Theoretical Hydropower Potential of Bashiro Gute (Ererti River)

Power	discharge from FDC (	Geometric head (m)	g	ρ	η	Kw
Minimum potential power (Q100%)	0.39	29	9.8	1	0.7	110.838
Small potential power (95%)	0.45	29	9.8	1	0.7	127.89
Average potential power (50%)	0.8	29	9.8	1	0.7	227.36

According to the field record the geometric potential from the powerhouse to the diversions out was 29 m. However, the fact sheet that was collected from the woreda has indicated that the head of the hydropower was 16m. Based on these heads the power potential and technical available power was illustrated in the table below corresponding to the flow time.

Table 4.3 Actual Hydropower potential on the Bashiro Gute River

Power type	Discharge (m3/s)	Net head	g	ρ	η	Kw
Minimum potential power (100%)	0.39	27.98	9.8	1	0.7	74.9
Small potential power (95%)	0.45	27.98	9.8	1	0.7	86.4
Average potential power (50%)	0.8	27.98	9.8	1	0.7	153.6

The existing designed hydropower of the Bashiro Gute has indicated that scheme installed to generate 33kw even though there was detail study of the project and the discharge that was taken as a discharge was$300\, {}^{l}/_{s}$. However according to this detail study has indicated the river could generate 74.9kw, 86.4kw, and 153.6kw actual power at $0.4\, {}^{m^3}/_{s}$,$0.5\, {}^{m^3}/_{s}$, and $0.8\, {}^{m^3}/_{s}$ respectively. The hydropower at average potential accepted as a suitable commonly for hydropower generations. Therefore $0.8\, {}^{m^3}/_{s}$ discharge at 50% exceedance time was taken for this study.

4.3.2. Headwork design

To have sustainable and stable headwork of the diversion structure hydraulic calculation was conducted for fixing different weir structure elevations. For this purpose, the following data has to be collected. These are maximum flood discharge at the construction site, high flood level before construction, downstream bed level, full supply of canal level which take of the water to the penstock, allowable flux and Lacey’s silt factor (Garg, 2005).

4.3.2.1. The hydraulic Calculation of the Scheme

Based on the river stream record the maximum flood discharge was determined to fix various hydraulic elevations on the headwork of the scheme. So that at the Bashiro Gute or Ererti river the maximum flood discharge in past thirty year as 1.47m^3/s. Using this input data, the length of waterway was calculated using the lacy's equation which was described in the equation 3.1 under methodology part.

$$Length\ of\ water\ way\ (L) = \ 4.75\sqrt{Q} = 4.75\sqrt{1.47} = 3.6m$$

From the length of waterway, the discharge per meter width of the river was calculated equation 3.2 above, and which was

$q = \frac{1.47}{3.6} = 0.4\,{m^3}/{s}$,

In the hydraulic elevation fix the third step was determination of the Regime scour depth which was determined using the equation of Lacy's formula at equation 3.3

$R = 1.35\left[\frac{q^2}{f}\right]^{\frac{1}{3}} = 1.35\left[\frac{0.4^2}{1}\right]^{\frac{1}{3}} = 0.75m$, using the Regime depth Regime velocity was calculated as:

$v = \frac{q}{R} = \frac{0.4}{0.75} = 0.55\,{m}/{s}$,

Then velocity head $h_v$was calculated using the regime velocity and acceleration of using the equation 3.5 above in the methodology part

$h_v = \frac{v^2}{2g} = h = \frac{0.55^2}{2*9.81} = 0.02m$

Then the water level and total energy level such as total energy line (T.E.L) level at downstream (d/s) and upstream (u/s) and upstream high flood levels were computed using the equations 3.6, 3.7 and 3.8 under methodology part.

Since the high flood level (HFL) before construction was 2274m, and the river bed level was 2272m, the downstream total energy level was calculated as:

$^{d}/_{S}\,(T.E.L) = HFL + \frac{v^2}{2g} = 2274m + 0.02m = 2274.02m$, whereas the upper stream total energy level (U/S T.E.L) was computed as;

Since the afflux due to the abstraction on the river was assumed as, 1m

$^{u}/_{S}\,(T.E.L.) = {}^{d}/_{S}\,(T.E.L) + Afflux = 2274.02m + 1m = 2275.02m$, based on this result the upstream High flood level (HFL) was computed by

$U/sHFL = {}^{u}/_{S}\,(T.E.L) - \frac{v^2}{2g} = 2275.04m - 0.02m = 2275.02,$

$D/sHFL = 2274.02 - 0.02 = 2274,$

Then head over the crest computed from discharge over crest of the weir using the equation of 3.9

$q = 1.7h^{\frac{3}{2}} \Rightarrow h^{3/2} = \left(\frac{q}{1.7}\right) \Rightarrow h = \left(\frac{q}{1.7}\right)^{\frac{2}{3}} = \left[\frac{0.4}{1.7}\right]^{\frac{2}{3}} = 0.4m,$

The crest level then calculated as the equation at 3.10

$Crest\ level = {}^{u}/_{S}\,(T.E.L) - h = 2275.04m - 0.4m = 2274.64m,$

The protection against scour was also calculated using the equation at 3.12

$Level\ of\ bottom\ of\ {}^{u}/_{S}\ pile(d1) = {}^{u}/_{S}\,HFL - 1.5R = 2275.02 - 1.5 * 0.75m = 2273.91m,$

The depth of upstream pile has to be $2275.02 - 2273.91m = 1.1m$ whereas, the downstream pile depth also has to be

$$Level\ of\ bottom\ of\ {}^{d}/_{s}\ pile = {}^{u}/_{s}\ HFL\ after\ retrogration - 2R = 2275.02m - 2 * 0.75$$

$2275.02m - 2 * 0.75$ =2273.5= 1.5m

The length of U/S horizontal launching apron=1.5d1to 2d1=2*1.1=2.2m with 0.6m up to 1.0m thickness provided even if zero uplift pressure

From the field measurements, information gathered from Woreda, regional bureau and fact sheet there was no Upstream and downstream cutoff provided for the scheme as well as upstream horizontal floor and only 1m downstream apron had provided. But in actual design upstream cutoff=1.1m, Downstream cutoff=1.5m, upstream apron=2.2m and downstream apron=9m

So, these also shows big differences in previous design and actual one

4.3.2.2. Diversion Weir Design

After fixing the hydraulic elevation the weir well dimensions were calculated using the steps that was describe by Garg, (2005). The top width of the crest was calculated using the formula:

$a = \frac{h}{\sqrt{\rho}}\ or = \frac{3}{2}\frac{h}{\sqrt{\rho}}$, the height over the crest = 0.4m and specific gravity of the material (ρ) masonry assumed to be 2.24kg/m3

Therefore, $a = \frac{0.4}{\sqrt{2.24}} = \frac{0.4}{0.5} = 0.8m$

Assuming the safety factor is $\frac{3}{2} \Longrightarrow a = 0.8m * \frac{3}{2} = \frac{2.4}{2} = 1.2m.$

The bottom width of the weir B was computed from the stability considerations. Therefore, the base width was taken the largest value which calculated using the formula below:

The bottom width of the weir was calculated using the overturning moment and resisting moment relationships.

$$m_0 = \frac{w(H+S)}{6}\ overturning\ momentum$$

$$m_r = \frac{wh\rho\ (b^2+ab-a^2)}{12} = Resisting\ momentum$$

Since there was no shutter therefor only the heigh of the crest was implemented. The crest heigh calculated from the crest level and the stream level which was 2274.65-2272=2.65m=H

$$\frac{w(H+S)}{6} = \frac{wh\rho\ (b^2+ab-a^2)}{12} = \frac{(1)}{6} = \frac{2.24\ (b^2+0.8b-6.4)}{12}$$

$\frac{2.24\ (b^2+0.8b-6.4)}{2} = 1.12(b^2+0.8b-6.4) = 1$, $(b^2+0.8b-6.4) = 0.89$,

$(b^2+0.8b-6.4) = 0.89 = b^2+0.8b = 6.4+0.89 = 7.29$, $b^2+0.8b = 7.29$ then the b Value was accepted which equates the equations. Therefore b=2.3m

The actual dimension of diversion weir computed are, crest top width = 0.8m, bottom width 2.3m, and height was 2.65m.But, the existing structure that was measured and surveyed in field has indicated that top width=0.9m, bottom width=2.0m and height was 2.01 m.

This dimension was varied from the dimension that was measured during surveying time of the existing structures.

4.3.2.3. Canal Dimensions

The canal dimensions and design are governed by various factor such as capacity, velocity of the water, slope of the side, head loss and seepage and the type of sediment disposition in the canal. For canal design the first step one should follow is the determination of the type of the canal construction depending on the location of the site. In this regard the suitable construction type that selected was one masonry with cement lined. The type of lining were determined due to the characteristics of the soil type which is porous induce a probability of seepage. According to

Kunwor, (2012), for stone massonary type canal the recommended side slope and roughness coefficients of different types were summarized in the table 4.3.

Table 4.3 Recommended sideslope, maximum velocity for Headwork Canal

Material used in the Canal	Side Slope(N=h/v)	Maximum recommended velocity for canals (V)	
		<0.3m depth	<1m depth
Stone masonry with mud mortar	0.5-1.0	1.0	1.0
Stone masonry with cement mor-tar	0-1.5	1.5	1.5

[Source Kunwor, 2012]

The recommended Roughness Coefficients for masonry canals was indicated in the table 4.4 below.

Table 4.4. Roughness coefficient for masonry canal

Brickwork	Roughness coefficient “n” =0.015
Normal Masonry with cement mortar	0.017
Coarse rubble masonry	0.020

[Source Kunwor, 2012]

Using the recommended parameters value in the table 4.3 and 4.4, further dimension design was conducted. So that according to the table the roughness coefficient for the canal was easily determined as: n= 0.017, the side slope of the canal also estimated as 1:2 which was 0.5. therefore, using this information, the dimension of the canal was calculated as

$Q = Av$, where, A is cross sectional area of the canal (m^2) , Q is the design discharge $0.8\frac{m^3}{s}$, and maximum permissible velocity was selected as $1.5\,{}^{m}/_{s}$ from the above table. Therefore, the dimension of the canal has to be

$A = \frac{Q}{V} = \frac{0.8\frac{m^3}{s}}{1.5 m/_s} = 0.5m^2$, then the optimum height of the canal bottom and top width was calculated based on the economical dimension concepts.

Cross sectional Area (A) = width *depth= b*d,

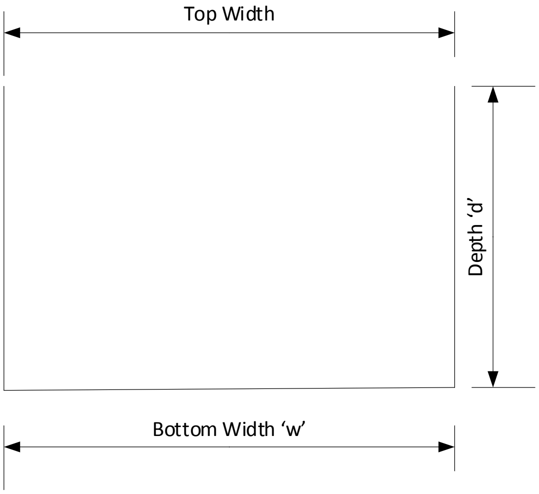
For economical dimension, b=2d; for the case of rectangle shape canal,

Cross sectional A= 2d*d= $2d^2$

Then $A = 2d * d = 2d^2 \Longrightarrow 0.5m^2 = 2d^2 \Longrightarrow d^2 = \frac{0.5}{2}m^2 = 0.5$

$$b = 2d = 2 * 0.5 = 1m$$

wetted perimeter as $R = \frac{cross\ sectional\ area\ of\ flow}{wetted\ perimer} = \frac{A}{P} = \frac{2d^2}{4d} = 0.5d = 0.5 * 0.5 = 0.25m$



Then the “bed slope” “S” was calculated by using the Manning’s equation. Manning’s equation actually relates the flow and velocity of water as $v = \frac{R^{\frac{2}{3}} * S^{\frac{1}{2}}}{n}$

$v = \frac{R^{\frac{2}{3}} * S^{\frac{1}{2}}}{n} \Longrightarrow S^{1/2} = \frac{vn}{R^{2/3}} = \frac{1.5 m/_s * 0.017}{0.25m^{2/3}} = 0.0015$

since the length of the canal from the diversion weir is 100m the drop would be 1m and it was calculated using the formula:

head loss in the canal (h)=SL=0.07*100=1m

Therefore, the power canal has to constructed rectangular section with the dimension of width= 0.8m, depth=0.4m with the free board =0.3, wetted perimeter =0.2m and bed slop 1 to 0.01m. But the existing power canal dimension measured was, bottom width = 1m, top width=2m, depth = 1.5m, the canal length was 53mand bed slope 1to 0.001m

4.3.2.4. Design of Penstock

The main purpose of the penstock is to convey water from the forebay to the turbine. Since the velocity and head loss in penstock determined by the diameter of the penstock. The first step of penstock design is to choosing the materials. Therefore, for this hydropower the chosen penstock was steel. According to the Hydropower course material of Hawassa university in 2022, the maximum lit for steel type penstock is 5 to 8 m/s. therefore the maximum velocity limit 8m/s was determined to calculate the diameter of the penstock.

The economic size for steel penstocks used in small hydropower installations according to Graphical method was calculated using the relationship

$$Q = vA = V * \pi * \frac{D^2}{4} \Longrightarrow D = \sqrt{\left(\frac{Q*4}{v*\pi}\right)} = 0.262m = 262mm \approx 300mm$$

The diameter of the penstock was 400mm that used to convey the required water to the power house. But, actual economic diameter penstocks for this scheme were 300mm.

From the diameter the friction loss was calculated in the penstock according to the formula that was indicated (Fox, R. W. ;McDonald, A. T. ;& Pritchard, P. J. , 2004).

$$h_f = \frac{fLQ^2}{12.1D^2}$$

whereas f= friction factor for pipe material, dimension less, 0.0014 from (from Moody Chart prided in annex

the length of penstock =L=30m, D=0.3m, Q=0.8m^3/s

$$h_f = \frac{fLQ^2}{12.1D^2} = \frac{0.0014 * 30m * 0.8^2 m^3/s}{12.1 * 0.3^2} = 0.02m$$

Then the net head loss was calculated using the velocity head los at the head work, the head loss at the canal and friction loss at the penstock.

$$total\ (h_{ne}) = velocity\ head\ loss - head\ loss\ at\ canal - frictionloss\ at\ penstok$$

$$h_{ne} = 0.02m + 1\text{m} + \ 0.02m = 1.02m$$

Therefore, the net head for micro hydropower generation on the Bashiro Gute was calculate using the relationship

$$Net\ head\ available\ H = H_g - h_{total\ loss} = 29 - 1.02m = 27.98m$$

CHAPTER FIVE

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

The micro hydropower of the Bashiro Gute on the Ererti River is one of the four micro hydropower that was installed by German International Cooperation Energy Coordination GIZ in collaborations with Sidama Developmental Association (SDA) in Sidama Regional state. However, at current time the scheme was failed to provide the services for the community at Bashiro gute kebel.

The Diversion weir and the Power canal were overturned and broken as it was seen during the field surveying time. The Diversion weir was constructed by dry masonry and had not sufficient apron at the downstream side to dissipate energy of overflowing water, upstream and downstream piles was absent which has opened a loop for the seepage problem and facilitates the overturning of the structure. During the field surveying dimension of the weir were measured and it was 0.9m top width, 2.0m bottom width, 2.01m crest height and 4.5m crest length and floor thickness was 0.9m. Even though Apron was constructed at downstream which had dimension 1m in length. The diversion weir was vertical and overflowing type however the structure has the head regulator and sluice gate but it was sealed. The power canal was constructed by masonry and it was lined by cements however almost it was cracked, silted up covered with debris, grasses. The dimension of the structure has shown that bottom width = 1m, depth = 1.5m, the canal length was 53m. The scheme has fore bay structure with escaping spillway and trash rack to regulate and controls the water entering to the penstock even though those structures also stolen and silted. The existing penstock was made up of steel with 400mm diameter, 42m length. However, the Anchor block which supported the penstock was broken by community.

The selected site of the diversion structure slope was varied with 18% to 27%. This slope is classified as steep and class five according to Class classification by International Land Reclamation Institute (ILRI, 1994). In such kind of topography, the diversion structure needs upstream and downstream apron with sufficient dimensions.

The current status of the hydropower assessment report has indicated that the scheme has been stopped working. One of the factors which has led to the failure was the project no fulfill all the planning principles and requirements. The document review result has indicated this reality since there was no plan and sufficient design documents. The participants on the project such as Sidama Development Cooperation, water and mining office of Bona Zuria woreda and Sidama regional state bureau of Water, Mines and Energy assured that no one could provide the detail design of the hydropower features. According to the interview of the stakeholders about (67%) didn't participate in any phase of the study but 33% participated during feasibility study but not at the final phase. This has indicated that the failure of the project was caused by lack of the sense of ownership since they didn't participate as well. Almost all interviewee (100%) responded that there was no clear procedure how to own the project and it was not still handover either to community or the government.

As it was indicated in under the status analysis of the hydropower the pervious designed hydropower aimed to generate 33 kw. However, whether this design was feasible or not nobody could explain. However according to this detail study from the river 74.9kw, 86.4kw, and

153.6kw actual power could be generated from $0.4\,{m^3}/{s}$, $0.5\,{m^3}/{s}$, and $0.8\,{m^3}/{s}$ discharge design at 100, 95 and 50% exceedance time respectively. The hydropower at average potential which was $0.8\,{m^3}/{s}$ at 50% was accepted as a suitable for the hydropower generations for this study. The proposed crest dimension which expected to be constructed were top width 0.8m, bottom width 2.3m, and height 2.65m. However, this differed the dimension previously constructed crest dimensions since it was top width=0.9m, bottom width=2.0m and height was 2.01 m. The power canal that proposed to pass the discharge that was fixed to generate the estimated power was width= 0.8m, depth=0.4m with the free board 0.3m, wetted perimeter =0.2m and bed slop 1 to 0.01m with the head loss at power canal was calculate 1m. The $300mm$ diameter of penstock is required and the head loss at penstock is 0.02m. Generally, the total head loss in the generation of hydropower is 1.02m and the net head required is 27.98m to generate 153.6.

5.2. Recommendations

This result has revealed that the hydropower study and installment lack the public participations and consultation. However, the consulting and hearing the intentions of the community is the backbone for the effectiveness of a project. The other main which could be ignored is technical feasibility study since it clearly determines the efficiency and stability of any project. The legal binding document and designing procedure also give detail information for anyone who want to deal with any issues regarding to any project. Therefore, having this information, the following main issues was recommended with regard to the Bashiro Gute hydropower scheme.

- Steering committee has to be established at regional level from Water and energy Bureau, Electric utility service, Sidama Development Associations and Other non-governmental Organizations to plan the upgrading
- The rural hydropower development strategies and implementation manual has to be developed at regional level
- To fix the problems that was seen at the project detail consultation with the community, stakeholders have to be conducted based on the findings of this study.
- Based on this finding the detail discussion with professionals, decision making body, and financial supporter has to be made to upgrade the scheme.
- Strict and coordinated monitoring has to be implemented to solve any defect on the technical efficiency.

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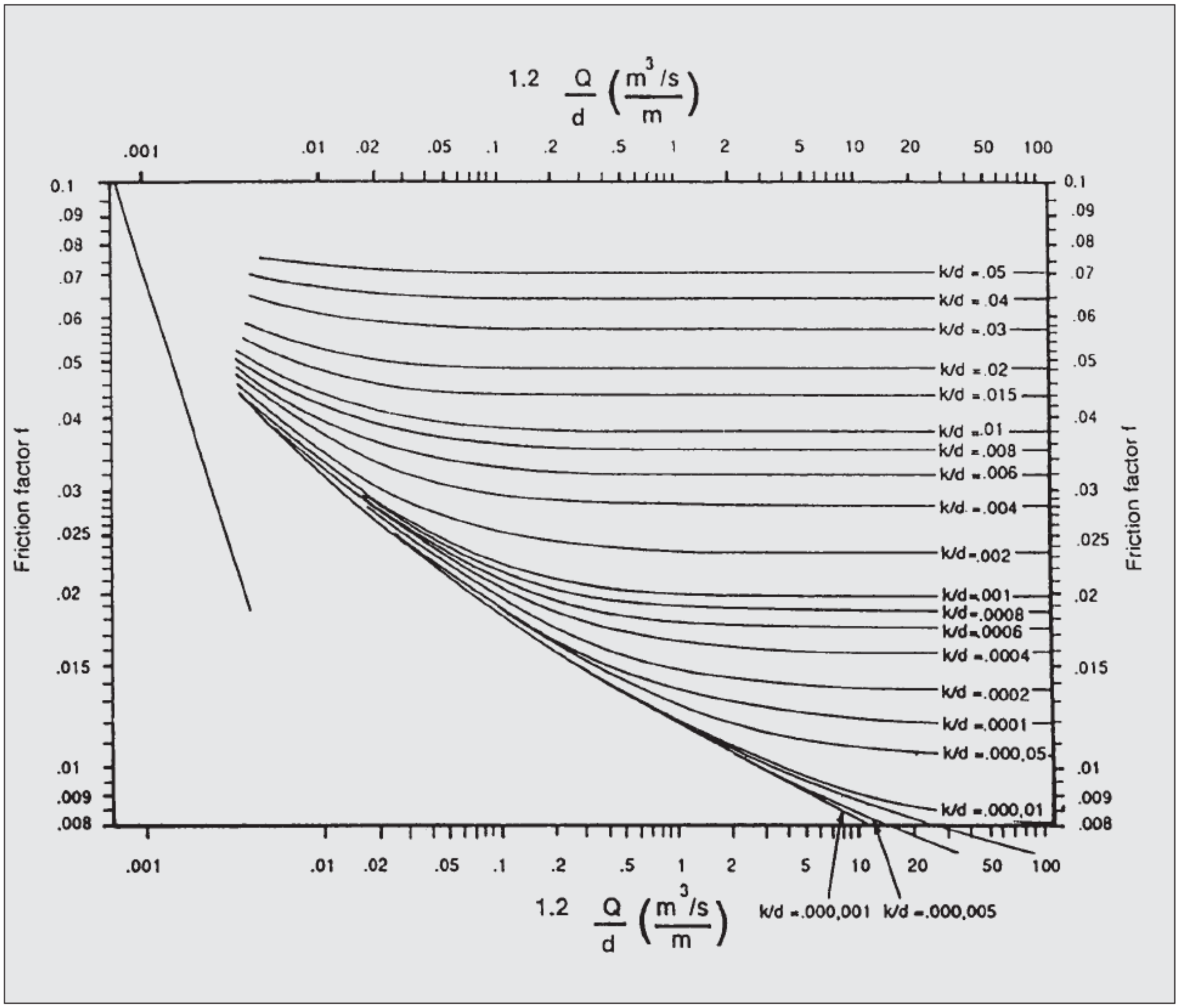
APPENDIX

A. Simulated Stream flow Of Bashiro Gute from the analog Station(yema)

year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Q2	A2	A1	Q1
1988	6.3	19.0	42.5	136.6	82.6	138.6	206.0	355.2	322.6	385.3	135.7	68.4	5.3	636.2	83.7	0.7
1989	80.4	64.4	61.2	148.6	124.4	175.5	178.5	111.5	350.0	385.0	138.2	131.7	5.4	636.2	83.7	0.7
1990	74.7	106.5	187.6	302.9	328.1	187.7	195.0	186.4	204.5	215.2	112.4	98.7	6.1	636.2	83.7	0.8
1991	98.8	96.4	116.2	138.0	144.2	101.1	149.5	134.8	310.7	169.2	63.2	54.8	4.4	636.2	83.7	0.6
1992	35.9	45.0	37.3	199.0	201.9	151.2	221.4	707.1	584.4	851.3	322.2	138.5	9.7	636.2	83.7	1.3
1993	90.4	120.8	72.2	167.2	506.0	496.9	239.9	169.0	279.3	510.6	226.1	83.7	8.2	636.2	83.7	1.1
1994	56.0	40.6	53.3	94.0	303.5	206.0	496.5	548.3	376.9	197.5	159.4	75.7	7.2	636.2	83.7	1.0
1995	50.7	46.8	57.4	285.3	281.2	126.8	147.6	202.5	508.6	386.6	115.1	91.0	6.4	636.2	83.7	0.8
1996	88.7	54.6	185.2	338.8	459.6	480.5	493.6	564.7	563.5	596.1	123.9	80.5	11.2	636.2	83.7	1.47
1997	65.8	43.0	48.2	161.4	250.6	174.1	313.7	401.6	244.8	663.7	502.9	180.7	8.5	636.2	83.7	1.1
1998	63.9	29.4	37.0	100.1	313.1	134.4	234.9	717.5	327.8	922.2	183.2	84.7	8.7	636.2	83.7	1.2
1999	67.3	48.8	59.3	57.2	113.7	74.1	76.6	95.0	118.5	211.9	89.8	58.7	3.0	636.2	83.7	0.4
2000	43.6	37.8	39.4	58.9	152.2	90.5	77.9	242.7	216.1	572.8	188.2	76.1	5.0	636.2	83.7	0.7
2001	69.5	50.8	59.0	83.2	174.4	250.7	122.0	273.2	297.6	304.9	124.1	76.9	5.2	636.2	83.7	0.7
2002	62.3	43.6	63.3	87.9	122.1	158.6	84.1	113.4	146.8	109.3	65.6	69.5	3.1	636.2	83.7	0.4
2003	57.1	41.1	53.0	120.5	92.6	72.1	93.2	203.1	145.1	350.5	200.4	91.0	4.2	636.2	83.7	0.6
2004	72.1	57.8	74.7	129.4	205.4	97.3	86.5	124.6	186.4	246.2	78.5	67.3	4.0	636.2	83.7	0.5
2005	55.0	38.4	53.3	85.2	326.4	202.8	172.5	186.4	263.4	194.3	110.0	62.7	4.9	636.2	83.7	0.6
2006	51.0	46.7	85.1	136.9	286.8	158.0	339.9	431.8	266.0	340.5	191.0	140.6	6.9	636.2	83.7	0.9
2007	94.2	70.9	70.8	148.8	308.8	318.5	221.9	518.4	348.6	338.3	122.4	77.8	7.3	636.2	83.7	1.0
2008	62.2	50.1	47.4	60.2	106.3	121.7	134.5	169.7	186.0	193.8	425.2	90.8	4.6	636.2	83.7	0.6
2009	73.2	62.0	74.2	160.9	238.4	152.3	84.1	111.5	150.9	255.3	89.8	108.8	4.3	636.2	83.7	0.6
2010	105.7	78.2	178.0	328.0	415.6	185.9	257.0	296.1	415.1	243.6	105.3	74.6	7.5	636.2	83.7	1.0
2011	64.1	48.7	55.5	58.2	165.0	152.1	179.7	290.6	416.9	286.6	144.5	100.4	5.5	636.2	83.7	0.7
2012	67.5	53.5	53.9	69.8	93.0	86.3	168.7	242.8	251.6	178.4	103.2	76.3	4.0	636.2	83.7	0.5
2013	60.6	44.0	65.6	126.6	305.1	202.1	401.8	571.5	317.3	412.5	166.5	92.4	7.7	636.2	83.7	1.0
2014	71.3	69.9	128.7	381.1	395.5	285.6	310.9	455.5	348.6	536.6	213.9	104.9	9.2	636.2	83.7	1.2
2015	64.0	48.6	63.0	208.5	245.6	224.1	134.5	313.8	186.0	255.3	122.4	128.8	5.5	636.2	83.7	0.7
2016	66.0	46.6	58.2	164.4	525.2	230.6	219.3	240.1	431.6	346.1	106.0	81.5	7.0	636.2	83.7	0.9
2017	49.6	48.4	69.0	133.5	280.6	778.0	351.6	259.2	554.1	381.3	110.3	84.5	8.6	636.2	83.7	1.1

B. **Moody Chart**



C. Determined levels of Hydraulic

D.

L= length of water way	3.5625m
Q= maximum Flood Discharge =	1.47m3/s
q= discharge Perimeter	0.41m3/s
R= Regime score depth	0.75m
v= Regime Velocity	0.55m/s
H_V= velocity head	0.02m
D/s TEL= total energy level	2274.02m
U/s TEL= total energy level	2275.04m
HFL=high flood level	2275.02m
V_H = Velocity head	0.39m
Crest level	2274.65m
U/s Pile level	2273.91m
D/spilled level	2273.52m
f= lacy factor	1
afflux m	1m
HFL=before con m	2274m
Design discharge	0.8m3/s
Net head	27.98m

E. Proposed Crest Dimensions

Crest Dimension	
Top width	0.8m
Bottom width	2.3m
Height	2.65m

F. Proposed Canal Dimensions

Canal Dimension	
width	0.8m
Bottom width	0.4m
Side slope	1:2
Wetted perimeter (w)	0.2m
Bed slope	1:0.01
Permissible velocity	1.5m/s
Manning coefficient	0.017

G. Penstock Dimension

Penstock Dimension	
Diameter	300mm
length	30m