Phase 2 - Ground Based Data Collection

PREFEASIBILITY STUDY OF THE LUEGERE HYDROELECTRIC SCHEME

Renewable Energy Resource Mapping: Small Hydro - Tanzania

December 2017

ABREVIATIONS AND ACRONYMS

ASTER GDEM	Advanced Spaceborne Thermal Emission and Reflection Radiometer Global Digital Elevation Model
CHIRPS	Climate Hazards Group InfradRed Precipitation database
DSM	Digital Surface Model
ESIA	Environmental and Social Impact Assessment
ESMAP	Energy Sector Management Assistance Program
EWURA	Energy and Water Utilities Regulatory Authority
FAO	Food and agricultural organization
GIS	Geographic Information System
GoT	Government of Tanzania
GSHAP	Global Sismic Hazard Assessment
GW	Gigawatt
GWh	Gigawatt hour
IFC	International Finance Corporation
IPP	Independent Power Producers
kW	Kilowatt
kWh	Kilowatt hour
MW	Megawatt
MWh	Megawatt hour
NASA	United States National Aeronautics and Space Administration
OP	Operational Polices
REA	Rural Energy Agency
RE	Renewable Energy
SRTM	Shuttle Radar Topography Mission
TANESCO	Tanzania Electric Supply Company
USACE	United States Army Corps of Engineers
WES	Waterways Experimental Station

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1 EXECUTIVE SUMMARY

The key features of the proposed Luegere hydroelectric scheme are summarized in Table 1 below.

Table 1. Key features of the proposed hydroelectric scheme

FEATURE	PARAMETER	VALUE	UNITS
Location	Region	Kigoma	-
	River	Luegere	-
Hydrology	Catchment area	1,317	km²
	Median streamflow (Q50%)	4.56	m³/s
	Firm streamflow (Q95%)	1.41	m³/s
	Design flow	4.33	m³/s
	Design flood (100 years)	220	m³/s
Diverting structure	Structure type	Gravity weir (Overflowing section : Creager)	-
	Material used	Concrete	-
	Overflowing section crest length	50	m
	Total structure length	70	m
	Overflowing section height	4.50	m
	Non-overflowing section height	7.15	m
	Crest elevation	943.00	masl
	Slab elevation	938.50	masl
Gated flushing channel	Number of bays	2.00	pce
	Gate section	1.4 x 1.5	m x m
Intake	Number of bays	2	pce
	Invert elevation	941.00	masl
	Equipment	Trash rack (manual cleaning)	-
Desilting structure	Yes		
	Number of basins	2.00	pce
	Water level	943.00	masl
Waterway Canal	Headrace canal length	1 420	m
	Headrace canal section	2 x 2.3	m x m
	Average slope	0.001	m / m
Forebay	Yes	-	-
	Water level	941.58	masl
Penstock	Number of penstock(s)	1	pce
	Length	1 110	m
	Diameter	1.20	m
Powerhouse and electrical / electromechanical equipment	Floor elevation	786.00	masl
	Gross head	157.00	m
	Number of units	3	pce
	Turbine type	Pelton	-
	Operating discharge per unit	1.44	m³/s
	Total installed capacity	5 340	kW
	Average annual energy generation	34.40	GWh/year
Access road	Length of road to build	9,000	m

	Length of road to rehabilitate	0	m
Transmission lines	Length	85	km
	Voltage	33	kV
Economic data	CAPEX - without access road and transmission lines	13.14	M$
	LCOE - without access road and transmission lines	0.05	$/kWh
	CAPEX - access road and transmission lines included	24.97	M$
	LCOE - access road and transmission lines included	0.10	$/kWh

2 INTRODUCTION

2.1 OVERVIEW OF THE ESMAP PROGRAM

ESMAP (Energy Sector Management Assistance Program) is a technical assistance program managed by the World Bank and supported by 11 bilateral donors. ESMAP launched in January 2013 an initiative to support the efforts of countries to improve the knowledge of renewable energy (RE) resources, establish appropriate institutional framework for the development of RE and provide "free access" to geospatial resources and data. This initiative will also support the IRENA-GlobalAtlas program by improving data availability and quality, consulted through an interactive atlas.

This "Renewable Energy Mapping: Small Hydro Tanzania" study, is part of a technical assistance project, ESMAP funded, being implemented by Africa Energy Practice 1 (AFTG1) of the World Bank in Tanzania (the 'Client') which aims at supporting resource mapping and geospatial planning for small hydro. It is being undertaken in close coordination with the Rural Energy Agency (REA) of Tanzania, the World Bank's primary Client country counterpart for this study.

The "Provision of Small Hydropower Resource Data and Mapping Services" IDA 8004801 Framework contract was signed on the 29th May 2013, while the specific contract " Renewable Energy Mapping: Small Hydro Tanzania", n. 7169139, is dated 4th of November 2013.

2.2 OBJECTIVES AND PHASING OF THE STUDY

The objectives of the study are:

- To improve the quality and availability of information on Tanzania's small hydropower resources. The project will provide the GoT (Client) and commercial developers with ground-validated maps (at least 70+ sites up to 10 MW) that show the varying levels of hydro potential throughout the country, and highlight several sites most suited for small hydropower projects.

- To contribute to a detailed comprehensive assessment and to a geospatial planning framework of small-hydro resources in Tanzania; (ii) to verify the potential for the most promising sites and prioritized sites (~ 20 prioritized sites) to facilitate new small hydropower projects and ideally to guide private investments into the sector; and (iii) to increase the awareness and knowledge of the Client on RE potential.

The study is delivered in three phases:

PHASE 1: Preliminary resource mapping based on satellite and site visits.

PHASE 2: Ground-based data collection.

PHASE 3: Production of validated resource atlas that combines satellite and ground-based data.

2.3 CONTEXT AND SCOPE OF THE PREFEASIBILITY STUDY

This report is delivered in the context of PHASE 2 (Ground-based data collection). In accordance with our Terms of References (Revised Terms of References for the Phase 2 and 3 of the Project, 30 June 2016), the prefeasibility study covers the following aspects:

- Review of the existing data and GIS information ;
- Additional site visit to the sites and main load centers / national grid connection by relevant sector experts ;
- Additional topographic and geotechnical surveys, update of the hydrology, and assessments of environmental and social impact to reach study results at pre-feasibility level;
- Preparation of a conceptual design and drawings at pre-feasibility level; Schematic Layout of Hydro Powerhouse, weir or dam (when applicable), waterways and Transmission Lines to the main load centers / national grid connection;
- Preparation of a Budgetary Cost Estimate, including costs for environmental and social costs, and Electricity Generation Estimate for a range of installed capacities;
- Preliminary economic analysis.

3 CONTEXT OF THE LUEGUERE HYDROELECTRIC SCHEME

3.1 PROJECT AREA

The Luegere River originates in the Katavi region at elevations over 2,000 m. The Luegere River flows mainly from the Southeast to the Northwest and discharges into the Lake Tanganyika about 10 km downstream to the hydroelectric project. The geographical coordinates (WGS1984) of the proposed weir location are 30.028°East and 5.895°South.

At the proposed intake weir location, the watershed of the Luegere River drains an area of 1317 km². Figure 1 presents the exact location of the proposed site in Tanzania. The administrative and location data are detailed in Table 2 below.

Table 2. Administrative data

Item	Value
Atlas code	SF-022
Site name	Luegere
River	Luegere
Major river basin	Malagarasi and Lake Tanganyika
Region	Kigoma
District	Kigoma Rural
Division	Igalula
Village	Lokoma
Reference topographic map	Topographic map n° 132/3 (scale 1/50,000)

3.2 SITE ACCESS

The proposed site is located 130km South of Kigoma. Access to the site is possible by taking a good dirt road up to the village of Lokoma. Before reaching the village, a dirt road leads to the left bank of the Luegere River. From there, the proposed weir location is accessed with a 3km long track.

Figure 1. Study area

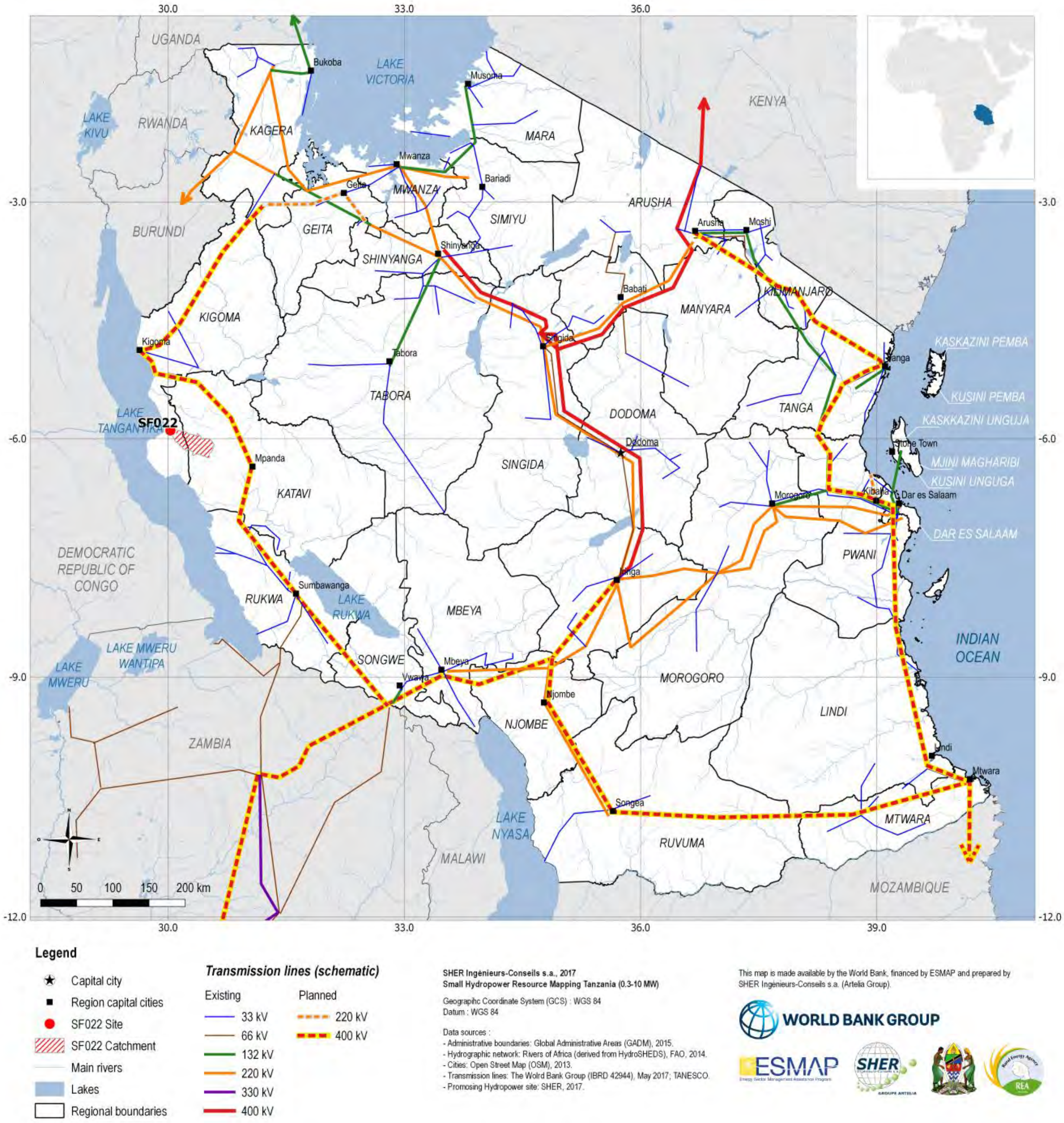
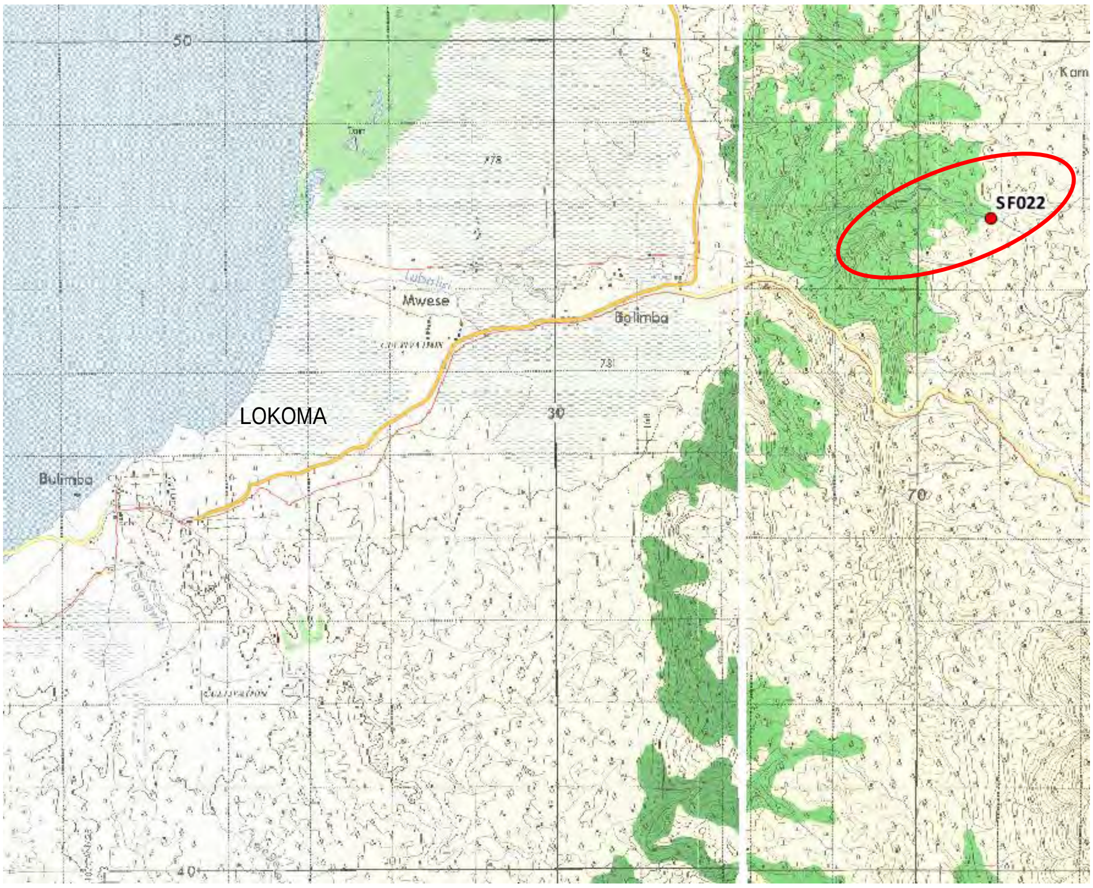


Figure 2. Site access overview

Figure 3. Access to the site (topographic map)



3.3 General Site Description

3.3.1 Site overview

The site is located East of Tanganyika Lake, in a large river (Luegere River) with a large waterfall (Figure 5). Slopes of the right and left valleys are steep. The project would supply the Kigoma mini-grid, towns along the Tanganyika Lake and Mahale National Park facilities.

Figure 4. Site overview (Google Earth)

LOKOMA 🡐

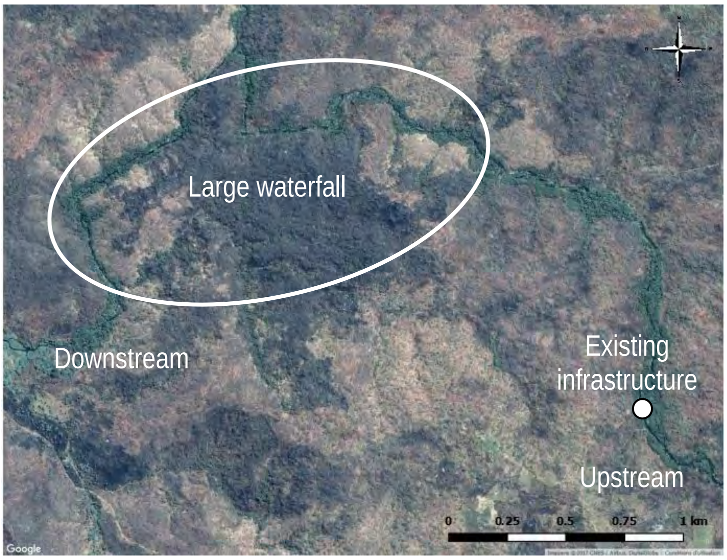
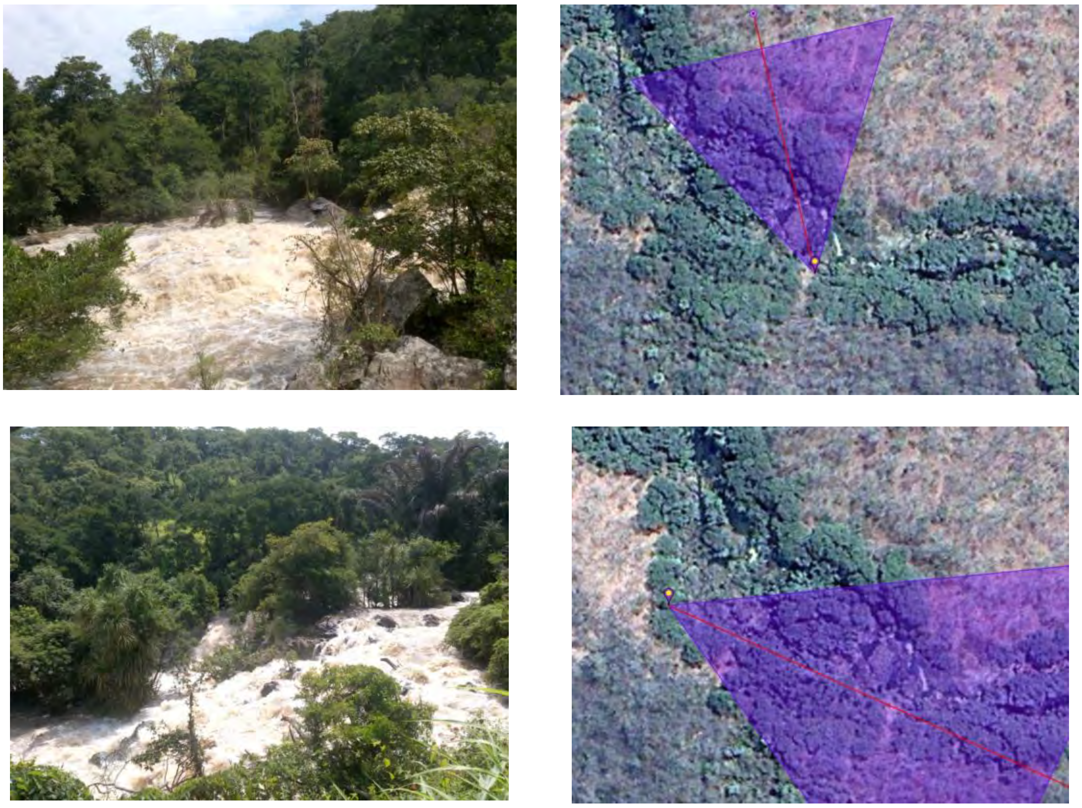


Figure 5. Views of the waterfall during the rainy season (March 2017)



3.3.2 Existing infrastructure

An existing intake structure is located 1.5km upstream the proposed weir location for water supply purposes (Figure 6 and Figure 7)). The infrastructure is not in operation.

Figure 6. Existing infrastructure situated 1.5km upstream the proposed weir location. Water was overflowing both sides of the weir (March 2017)

Figure 7. More details on the existing weir

3.4 Previous Studies

To the best of our knowledge, there are no previous studies of the proposed site.

4 TOPOGRAPHY AND MAPPING

4.1 EXISTING MAPPING

4.1.1 Topographic Mapping

The JPEG format (not georeferenced) 1:50,000 scale topographic maps have been acquired from the Survey and Mapping Department of the Ministry of Land in order to cover the entire study area. The JPEG format (not georeferenced) 1:100,000 scale topographic maps have been also obtained from the Ministry of Land. The 1:50,000 scale map of interest is the sheet 132/3. The contour lines interval is 20m. All the topographic maps have been georeferenced as described in section 4.2.

4.1.2 Thematic Mapping

Thematic maps and their key features, sources and format are presented in Table 3 below.

Table 3. Collected thematic maps

THEMATIC	FORMAT	KEY FEATURES	SOURCES
Administrative boundaries	Vector	Country / Regions / Districts / Divisions	FAO Global Country Boundaries, 2012 REA, 2014
Major cities	Vector	32 cities	Open Street Map, 2014
Topographical maps	Raster	1:250,000 (64 tiles) Full country coverage	Ministry of Land, Survey and Mapping Department
	Raster	1:50,000 (1,333 tiles) Full country coverage	Ministry of Land, Survey and Mapping Department
Digital Elevation Model	Raster	SRTM v4.1 Spatial resolution ~ 90m	NASA, 2014 http://www2.jpl.nasa.gov/srtm/
	Raster	ASTER GDEM v2 Spatial resolution ~ 30m (experimental)	http://www.jspacesystems.or.jp/en_/
Land cover	Vector		
Protected areas	Vector	Protected areas, National Parks and Game reserves	Tourist Board ; Tanzania Conservation Resource Centre ; Ministry of Land ; World Database on Protected Areas ; Protected Planet, 2014
Soil map	Raster	IPCC default soil classes derived from the Harmonized World Soil Data Base (v1.1)	ISRIC-WISE http://www.isric.org
Satellite image	Raster	Image Landsat 2013	Google Earth
Population	Shapefile	Census data at village and region levels	National Bureau of Statistics ; Ministry of Finance, REA
Lakes	Vector	Inland water bodies in Africa	FAO, 2000 http://www.fao.org/geonetwork
River network	Vector	River "flow accumulation" network from the HYDRO1k for Africa	FAO, 2006 http://www.fao.org/geonetwork
Rainfall	Raster	Monthly average rainfall grid Spatial resolution ~ 1km	WorldClim, v1.4 http://www.worldclim.org/
Road network	Vector	National, regional and other roads of Tanzania	World Bank AICD database
Rail network	Vector	Main rail network	World Bank AICD database
Ports	Vector	Major ports	World Bank AICD database
Airports	Vector	Major airports	World Bank AICD database
Power grid	Vector	Existing power grid	IED, 2013 ; REA ; TANESCO

4.1.3 Digital Surface Model

The digital surface model (DSM) used in the hydrological study is based on the "Shuttle Radar Topography Mission" (SRTM, version 1 arc-second). These data were acquired in February 2000 by the United States Space Agency (NASA) through radar measurements from space shuttle Endeavor. These data have a spatial resolution of 1 arc-second (about 30 m at the equator). The DSM of the study area is illustrated in Figure 11 of the chapter describing the Hydrological Study.

4.2 Mapping Carried out as Part of the Study

4.2.1 Digitization and geo-referencing

The 1:50,000 scale topographic maps were geo-referenced using the Quantum GIS software and the following projection parameters:

- Projection Transverse Mercator UTM zone 36S
- Latitude of origin = 0
- Central meridian = 33
- Scale factor = 0.9996
- False Easting = 500,000
- False Northing = 10,000,000
- Datum WGS 1984

4.2.2 Additional surveying

4.2.2.1 Digital surface model

The topographic survey was carried out by remote sensing. An eBee Plus drone equipped with a specific camera designed for photogrammetric mapping was used (Figure 8).

Figure 8. eBee Plus drone equipped with a camera for the topographical survey

Outputs from drone survey are (1) a high-resolution orthophotography (0.10m resolution) and (2) a Digital Surface Model (DSM). The DSM includes the vegetation cover, but it gives an excellent overview of the topographical features of the site of interest. Contour lines are calculated from the DSM. The ortho-photography as well as contour lines deduced from the digital surface model are presented at Figure 10.

Elevations resulting from this topographic survey are relative to each other and have not been linked to the national system. Consequently, the elevations of the works mentioned in this report are not the absolute altitudes of the Tanzanian national system.

4.2.2.2 Digital terrain model

The digital surface models was then post-processed to eliminate the effects of vegetation and hence represent the natural terrain elevation. This has been be achieved by identifying pixels at the natural terrain level (excluding vegetation and other anthropogenic elements) and performing a spatial interpolation of these points in order to obtain a digital terrain model (DTM). At this prefeasibility stage, only the weir/intake and tailwater areas were post-processed to obtain the DTM.

Figure 9. Digital Surface Model (DSM) and orthophotography from drone survey for SF022 site

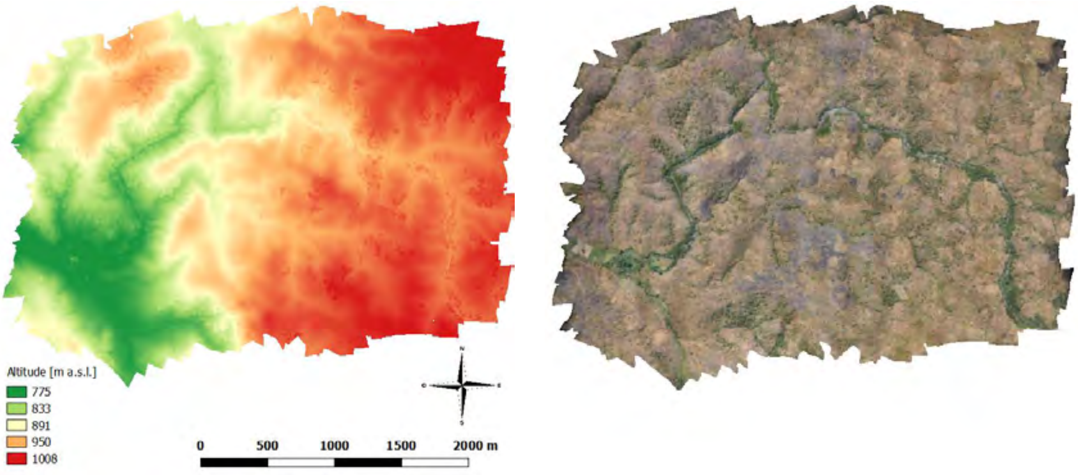
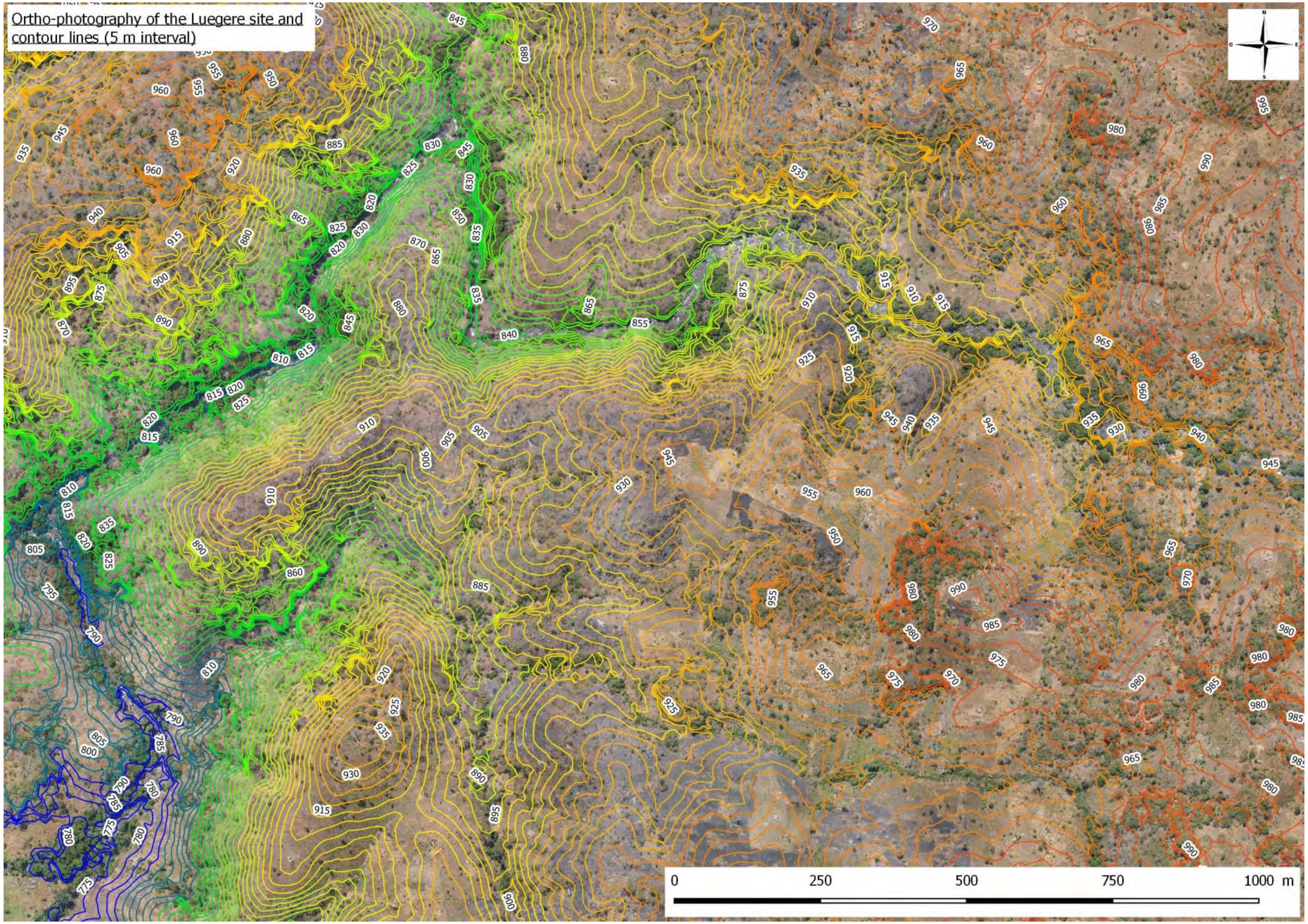


Figure 10. Ortho-photography of the Luegere site and contour lines (5 m interval)



5 HYDROLOGICAL STUDY

5.1 Objectives and Limits

The objective of the hydrological study is to establish and quantify the climatological and hydrological characteristics of the study area in order to determine the hydrological parameters and time series required for the design of the Luegere hydroelectric project as well as for the economic analysis of the pre-feasibility study.

5.2 Description of the Study Area

5.2.1 Physical Context

The Luegere River originates in the Katavi region at elevations over 2,000 m. The Luegere River flows mainly from the Southeast to the Northwest and discharges into the Lake Tanganyika about 10 km downstream to the hydroelectric project.

As shown in Figure 11, the Luegere catchment at the proposed hydroelectric project site features a marked relief with elevations between 2,036 m and 976 m (1,420 m on average). The drainage basin of the Luegere River at the proposed intake site is 1,317 km² (delimitation based on the SRTM DSM of spatial resolution 1 arc-second, i.e. approximately 30 m). The main physical and morphological features of the river catchment are presented in Table 4 below.

The hypsometric curve of the river catchment is shown in Figure 12. This curve shows the percentage of the catchment area above a given elevation. It shows that slopes are important in the upstream part of the catchment and that the rest of the catchment flows on a plateau characterized by a relatively steep slope. This is clearly observed in Figure 12 and Figure 11.

Table 4. Physical and morphological characteristics of the catchment

Parameter	Value	Unit
Area	1,317	km²
Average elevation	1,420	m a.s.l.
Maximum elevation	2,036	m a.s.l.
Maximum elevation (percentile 5%)	1,751	m a.s.l.
Minimum elevation	976	m a.s.l.
Minimum elevation (percentile 95%)	1,105	m a.s.l.
Slope index	8.6	m/km
Elevation range	646	m
Perimeter	271.8	km
Gravelius index	2.10	-

Figure 11. Luegere River catchment and Digital Surface Model

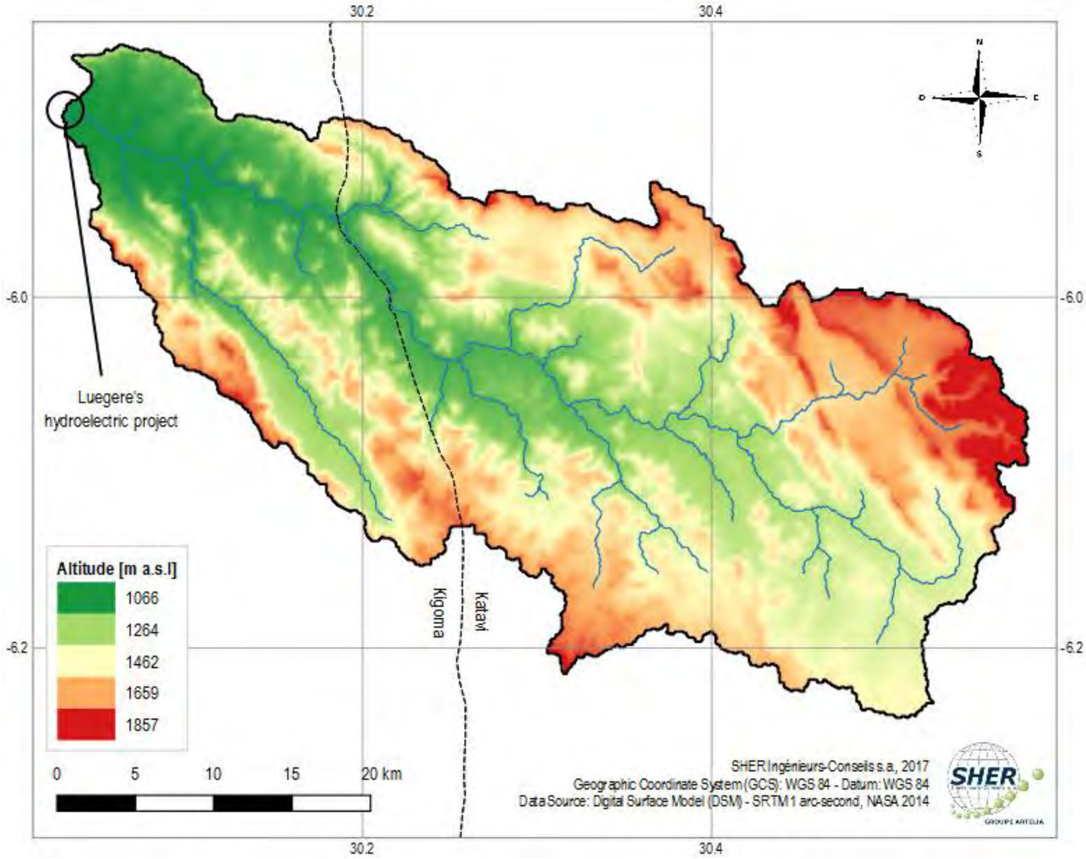
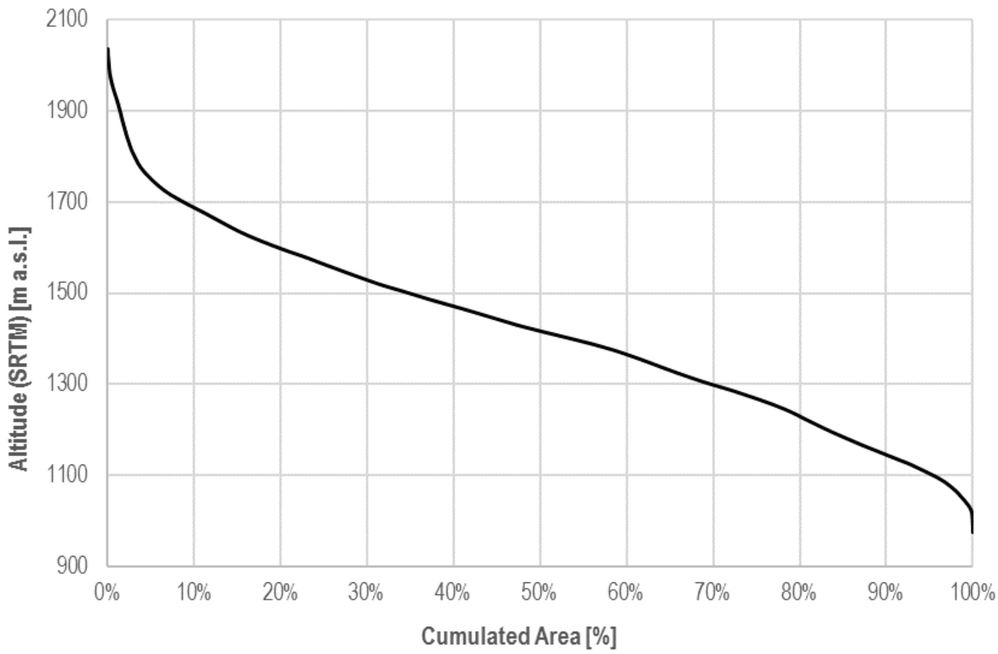


Figure 12. Hypsometric curve of the Luegere River catchment



5.2.2 Land cover and protected areas

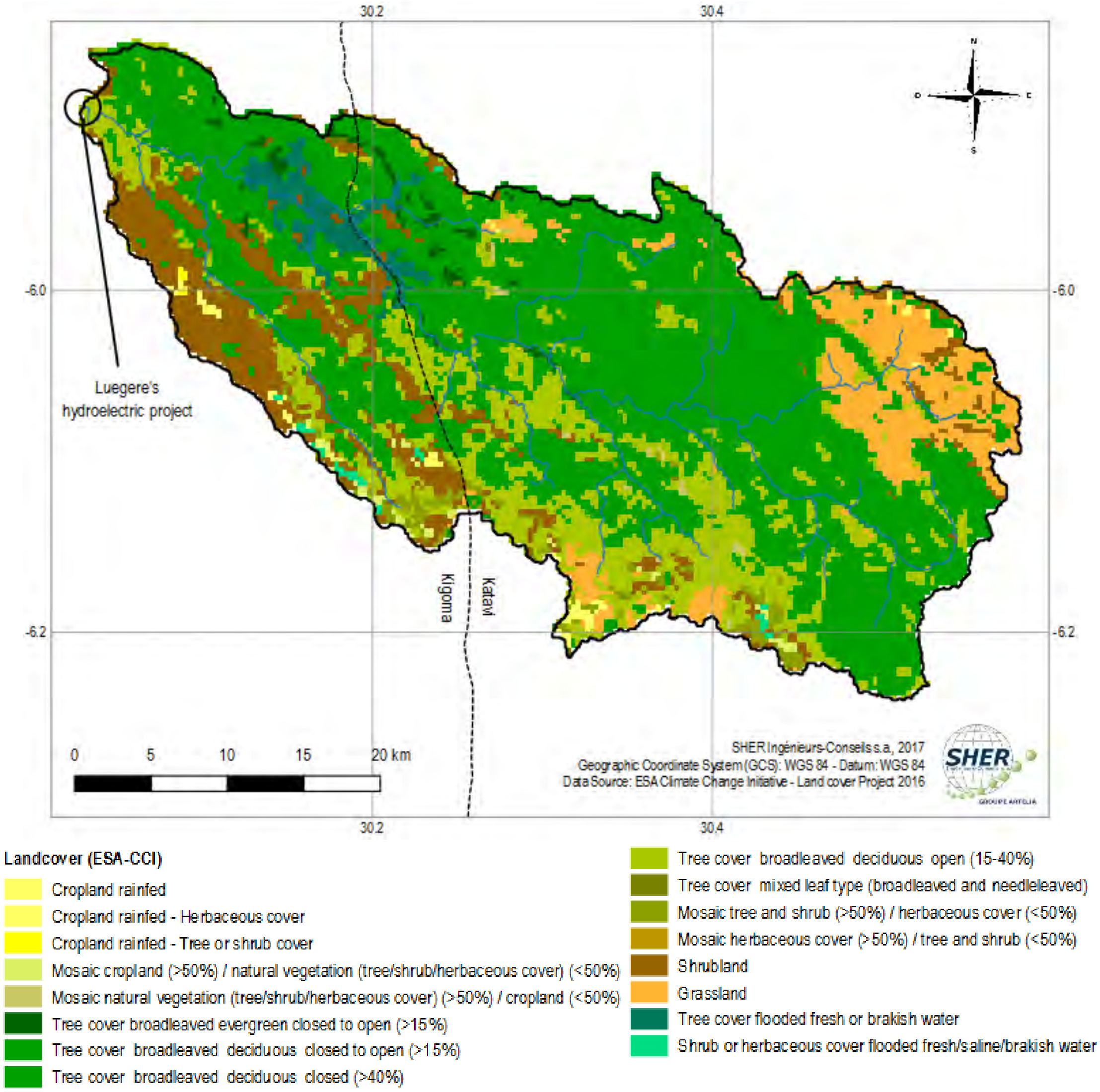
Data from the CCI Land Cover project (© ESA Climate Change Initiative - Land Cover project 2016) is a widely accepted source of information for land use around the world. These data are derived from satellite images acquired by the MERIS instrument of the European Space Agency. The land cover includes 5 years of satellite imagery acquisition between 2008 and 2012. The information is provided in raster format with a spatial resolution of 300m and allows defining the land use classes shown in Figure 13.

Figure 13 and Table 5 show that the Luegere catchment is characterized by a very abundant vegetation cover composed mainly of a forest of deciduous (73.6% of the catchment area, i.e. 969 km²), shrubland (11.2%, i.e. 147 km²) and grassland (8.2%, i.e. 108 km²).

Table 5. Land cover in the Luegere River catchment

Code	Legend	Area	
		[%]	[km²]
10	Cropland rainfed	0.3%	3.40
11	Cropland rainfed - Herbaceous cover	0.4%	5.10
12	Cropland rainfed - Tree or shrub cover	0.1%	0.66
30	Mosaic cropland (>50%) / natural vegetation (tree/shrub/herbaceous cover) (<50%)	0.5%	6.42
40	Mosaic natural vegetation (tree/shrub/herbaceous cover) (>50%) / cropland (<50%)	0.2%	3.12
50	Tree cover broadleaved evergreen closed to open (>15%)	1.0%	12.85
60	Tree cover broadleaved deciduous closed to open (>15%)	53.2%	700.30
61	Tree cover broadleaved deciduous closed (>40%)	2.8%	36.75
62	Tree cover broadleaved deciduous open (15-40%)	17.6%	231.50
90	Tree cover mixed leaf type (broadleaved and needleleaved)	0.1%	1.80
100	Mosaic tree and shrub (>50%) / herbaceous cover (<50%)	1.7%	22.67
110	Mosaic herbaceous cover (>50%) / tree and shrub (<50%)	0.0%	0.19
120	Shrubland	11.2%	146.90
130	Grassland	8.2%	107.90
160	Tree cover flooded fresh or brakish water	2.5%	33.07
180	Shrub or herbaceous cover flooded fresh/saline/brakish water	0.3%	4.06
	TOTAL	100%	1317

Figure 13. Land cover in the Luegere River catchment



5.2.3 Climate

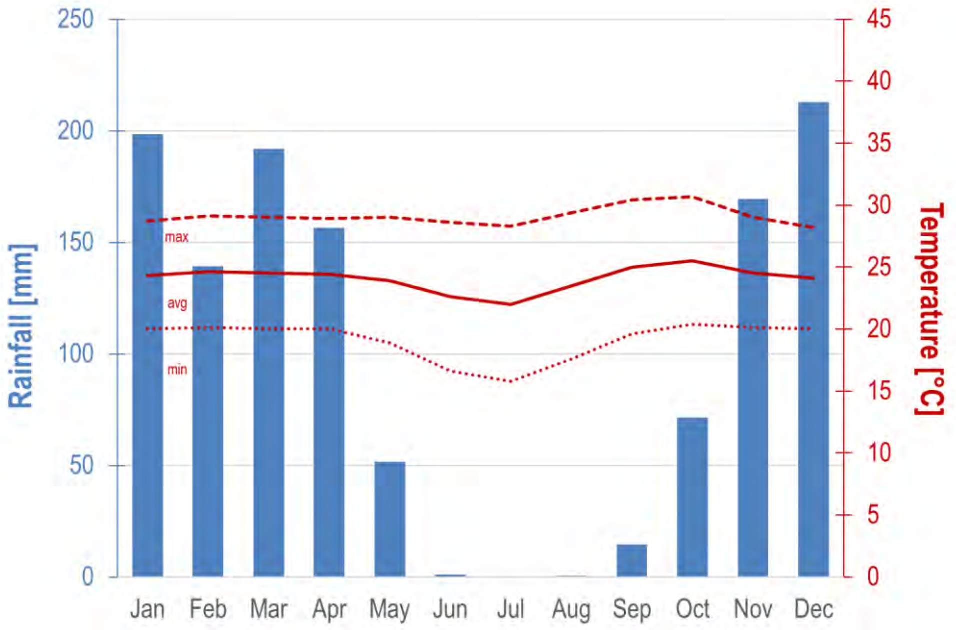
According to the Köppen classification based on rainfall and temperature, the study area (Luegere River catchment) is characterized by a tropical savanna climate with a pronounced dry season and constant high temperatures (*Aw* class). Köppen defines the temperate climate «*A*» by the following characteristics:

- Average temperature of each month of the year > 18 °C ;
- High annual precipitation (greater than annual evaporation) ;
- No winter season.

The rainfall regime « *w* » (dry season in winter) is defined by a savanna climate with a precipitation of the driest winter month < 60 mm and < [100 – (mean annual precipitation) / 25].

Figure 14 shows the climatic diagram as well as the temperature curve for the Luegere River watershed. Precipitations are very low during the dry season (June to September) but significant during the wet season. July is the driest month without precipitation (on average) whereas the wettest month is December with 213 mm on average. The average annual precipitation is 1,208 mm.

Figure 14. Climatic diagram of the Luegere River catchment



It is observed that the average annual temperature is 24.1°C. Temperature does not varies much throughout the year with an average amplitude of 3.5°C. The warmest month is October with 25.5°C and July is the coldest, with an average temperature of 22.0°C.

5.3 HYDRO-METEOROLOGICAL DATABASE

5.3.1 Rainfall and meteorological data

Rainfall data from two sources were used in this study: (i) the WorldClim climate database and (ii) the Climate Hazards Group InfradRed Precipitation database (CHIRPS).

WorldClim is a set of global data representative for the period ~1970-2000 available with a spatial resolution of about 1 km and at a monthly timestep. The spatial resolution is obtained by interpolation of ground-measured data.

Climate Hazards Group InfraRed Precipitation with Station data (CHIRPS) is a 30+ year quasi-global rainfall dataset at a daily timestep. Starting in 1981 to near-present, CHIRPS incorporates 0.05° resolution satellite imagery with in-situ station data to create gridded rainfall time series for trend analysis and seasonal drought monitoring. Values extracted from these satellite images are the means of the precipitation that falls each day on the entire catchment.

5.3.2 Hydrological data

An existing streamflow monitoring station (ref: Luegere River at Lubalisi, 4D1) is located 2 km downstream the hydroelectric project. Data have been collected in the Lake Tanganyika Water Basin Office. The completeness of the time-series (12% of daily data gap, 1975-1988) is not sufficient for a reliable statistical analysis.

To estimate the streamflows of the Luegere River at the hydroelectric project, a method based on the generation of synthetic streamflows on the basis of precipitation data through hydrological modelling was developed and is described in the next section.

5.3.3 Annual and monthly rainfall

5.3.3.1 Annual and monthly distribution

The analysis of the annual distribution of rainfall within the study area is based on the CHIRPS dataset, presented in section 5.3.1. The results are presented monthly in the section 5.2.3, Figure 14.

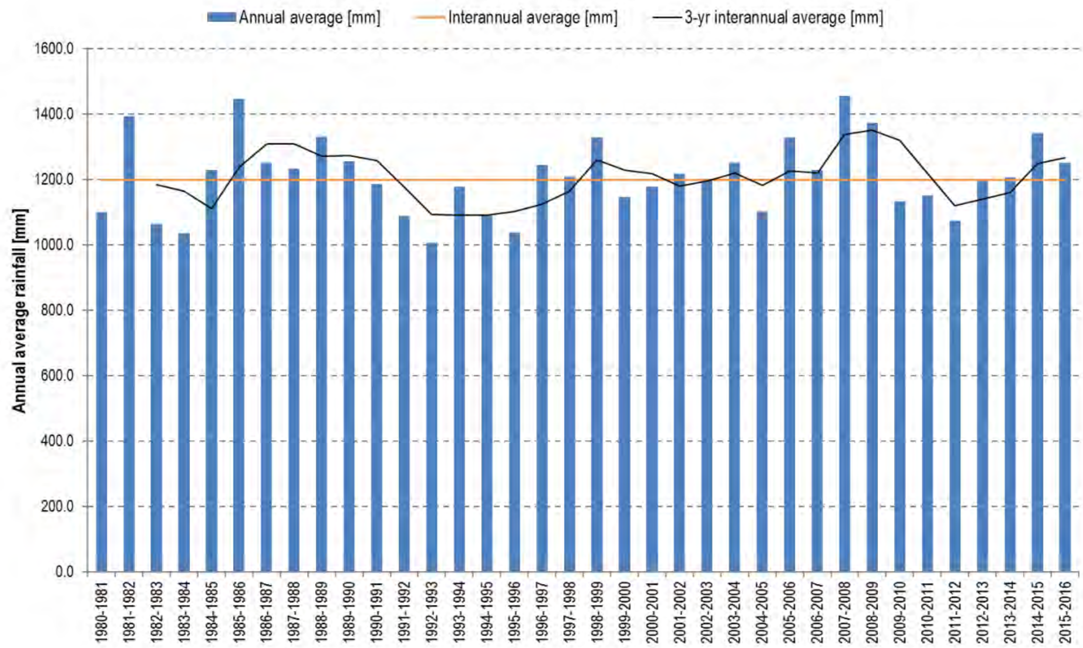
5.3.3.2 Spatial distribution

The analysis of the spatial variation of rainfall within the study area is based on the WorldClim dataset, presented in section 5.3.1. The spatial variation of average annual rainfall within the watershed is significant with a minimum of 1,056 mm in the northwestern part of the catchment and a maximum of 1,283 mm in its northern part. This is illustrated in Figure 15.

5.3.3.3 Temporal variation

The temporal variation in rainfall for the Luegere catchment has been studied from CHIRPS dataset (period 1981-2017) and the results are presented in the graph below. Annual average is fluctuating between 1,000 mm and 1,400 mm but it does not feature any clear trends in annual patterns.

Figure 15. Temporal variation in rainfall for the Luegere catchment



5.3.4 Inflow analysis

5.3.4.1 Methodology

Given the characteristics of the data available (daily datasets with gaps), the hydrological model selected was a daily lumped rainfall-runoff model called GR4J (in French, modèle du Génie Rural à 4 paramètres Journalier, https://webgr.irstea.fr/en/modeles/journalier-gr4j-2/).

First, the model needs three inputs:

- daily precipitation (obtained from CHIRPS satellite imagery)
- daily evapotranspiration (obtained from CLIMWAT agroclimatic stations)

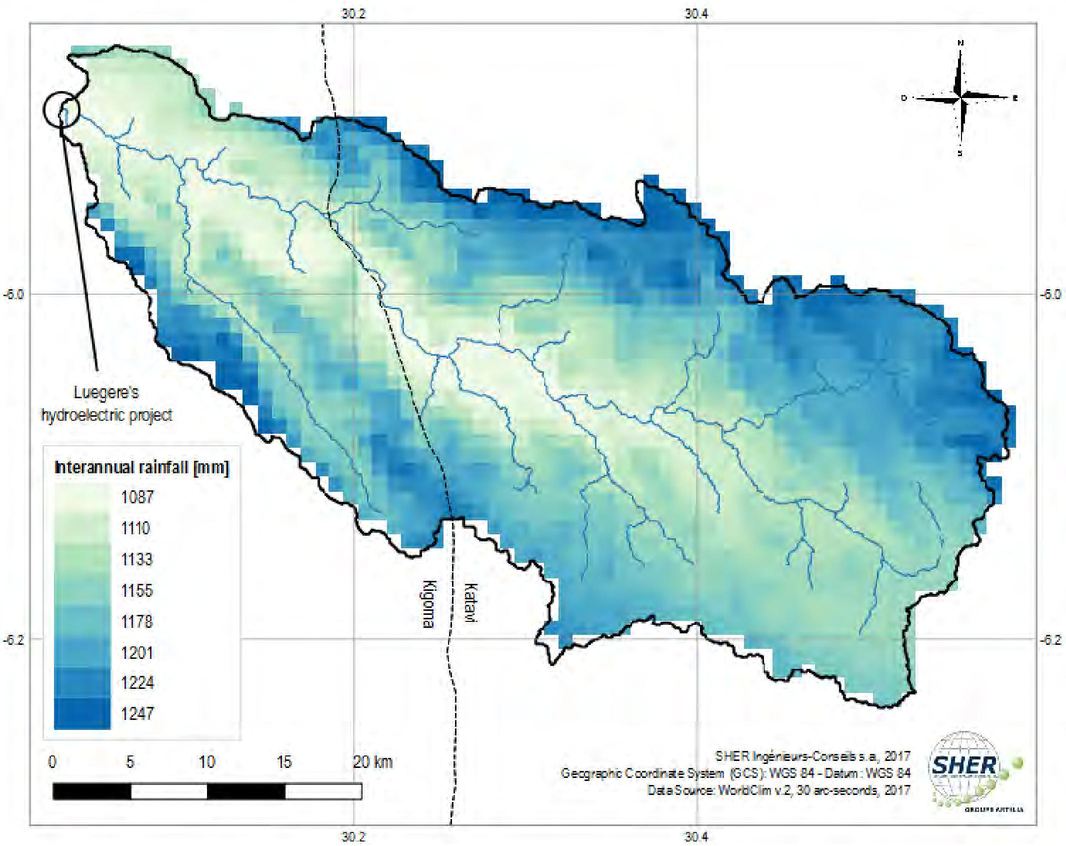
- daily observed streamflow (obtained from data collection in the Lake Tanganyika Water Basin Offices)

From these inputs, the rainfall-runoff relation is established by four parameters. The optimization of these parameters permits to reproduce as much as possible the observed hydrograph.

Once the model (or the rainfall-runoff relation) is optimized, the synthetic streamflows can be generated from the daily precipitation and evapotranspiration data. As these data are available for the period of 1981-2017, the synthetic streamflows can be simulated for the same period.

The hydrological modelling is only applicable if the time-series of observed streamflow period covers at least three consecutive high-quality hydrological years(1) after 1981 (first year of the CHIRPS data).

Figure 16. Spatial Variation of the annual rainfall on the Luegere catchment



5.3.4.2 Flow duration curve

Among the hydrological parameters, the determination of the flow duration curve is essential to know the availability of the flows in the river for the hydroelectric project. Indeed, this curve shows the percentage of time that the streamflow in a river is likely to equal or exceed some specified value of interest.

For the hydrological modelling method used in this study, the flow duration curve is made directly applying a probability function P(X>x) on the time-series of synthetic streamflow data. This function determines the probability of exceedance of each flow reaching the hydroelectric project.

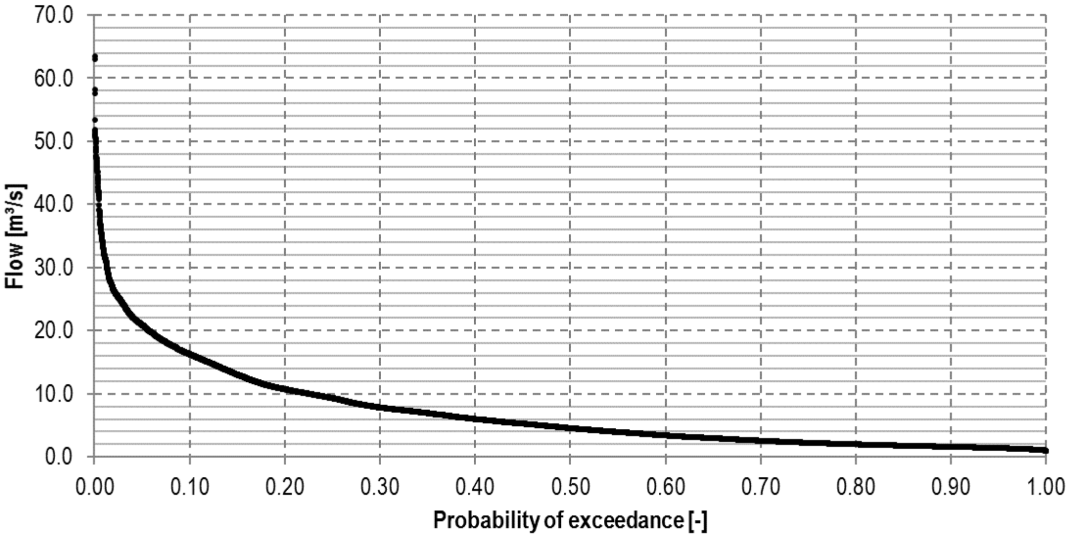
1 A hydrological year is defined as the 12-month period beginning after the dry season. In Tanzania, the hydrological year begins on November 1 and ends on October 31.

Table 6 and Figure 17 show the modelled flow duration curve as well as the main percentiles. The proposed model shows that the streamflow of the Luegere River at the hydroelectric project is less than 12.0 m³/s 50% of the time and that it is higher than 26.8 m³/s only 10% of the time (over a year period). The flow guaranteed 90% of the time (329 days per year) is estimated at 3.4 m³/s.

Table 6. Flow duration curve of the Luegere River at the hydroelectric project

Streamflow		Exceedance Probability
[m³/s]	[L/s/km²]	[-]
1.41	1.07	$Q_{95\%}$
1.59	1.21	$Q_{90\%}$
1.99	1.51	$Q_{80\%}$
2.57	1.95	$Q_{70\%}$
3.39	2.57	$Q_{60\%}$
4.56	3.46	$Q_{50\%}$
5.99	4.55	$Q_{40\%}$
7.83	5.95	$Q_{30\%}$
10.74	8.15	$Q_{20\%}$
16.29	12.37	$Q_{10\%}$
20.91	15.88	$Q_{5\%}$

Figure 17. Modelled flow duration curve of the Luegere River at the hydroelectric project



5.4 Flood Study

5.4.1 Introduction

The flood study is essential for designing structures and equipment such as spillways or floodgates but also for temporary infrastructure such as cofferdams and temporary diversions during the construction period.

The flood study will focus on 10 years and 100 years return period. These floods will be used respectively for the construction and operation phases. A detailed justification for these return periods can be found in section 8.1.4 of this report.

5.4.2 Methodology

Given the lack of observed streamflow data, the methodology used to estimate the floods is a hydrological modelling only based on land features (topography, soil type, and land cover). Hence, the results remain flood estimates and will have to be confirmed at the next stage of the study.

The software used is Hydrological Modelling Software (HEC-HMS v4.2.1) developed by the Hydrologic Engineering Center of the US Army Corps of Engineer. This program is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing.

Hydrological modelling aims to represent the hydrologic response of the watershed for specific rainfall events. Hydrological models are composed by several parameters that can be estimated from land features (topography, soil type, and land cover) influencing the infiltration (production function) and the dynamic of the surface flow (transfer function). These parameters must be calibrated on observed streamflow data in order to establish the best rainfall-runoff relationship to the model. Validated, the model can be used to estimate the hydrographs for extreme rainfall events.

Given the lack of observed streamflow data, it is not possible to calibrate and validate the hydrological model. That is why, at this stage of the study, the results of this hydrological study are indicative only.

5.4.3 Extreme rainfall events estimates

The extreme rainfall events have been determined for 10, 25, 50 and 100 years return period from the CHIRPS dataset by a statistical extrapolation of the observed maximum precipitations (log-normal law[2]). Then, the 24-hr precipitations intensity have been statistically distributed to represent a typical event at the simulation time step. Results are presented in the table below.

Table 7. Extreme rainfall events estimates for the Luegere River watershed

Return period	10 years	25 years	50 years	100 years
24-hr precipitation	50.2 mm	56.9 mm	61.7 mm	66.4 mm

[2] This law is advocated by some hydrologists who justify it by arguing that the appearance of a hydrological event results from the combined action of a large number of factors that multiply. Consequently, the random variable follows a log-normal distribution. Indeed, the product of variables is reduced to the sum of the logarithms of these variables and the central-limit theorem makes it possible to assert the log-normality of the random variable. [Translated from Musy A. (2005). Hydrologie générale. http://echo2.epfl.ch/e-drologie/]

5.4.4 Hydrological parameters estimates

Production function[3]: to estimate the runoff generated for each sub-basin, a "production function" is used. This function evaluates the precipitation amount that does not infiltrated into the soil. The SCS Curve Number method has been selected. The parameter of this method (curve number) is calculated from two land features: (a) the hydrologic soil group (HSG) determined from soil type and (b) the land cover.

Transfer function[4]: to estimate the dynamic of the runoff for each sub-basin, a "transfer function" is used. This function represents how the water coming from the precipitation that is not infiltrated into the soil is moving within each sub-basin to reach the outlet. The SCS Unit Hydrograph method has been selected. The parameter of this method (time of concentration) is calculated from topographic land features: (a) the area and slope of the sub-basin and (b) the length and the slope of the main channel.

5.4.5 Flood estimates

Ten years and hundred years return period flood estimates at the Luegere hydroelectric scheme are presented in the following table.

Table 8. Ten years and hundred years return period flood events

Atlas Code	Site Name	Floods [m³/s]	
		T = 10 years	T = 100 years
SF022	Luegere	91	213

5.5 Key Hydrological Parameters of the Luegere Project

The key hydrological features of the Luegere hydroelectric project on the Luegere River are summarized in Table 9 below.

Table 9. Key hydrological features of the site

Characteristic	Parameter	Value	Unit
Catchment	Area	1,317	km²
	Mean elevation	1,420	m a.s.l.
	Maximum elevation	2,036	m a.s.l.
	Minimum elevation	976	m a.s.l.
	Average slope	8.6	m/km
Rainfall	Long-term average annual (CHIRPS)	1,208	mm/y
Streamflow	Guaranteed ($Q_{90\%}$)	1.6	m³/s
	Median ($Q_{50\%}$)	4.6	m³/s
Flood estimates	10 years	91	m³/s
	100 years	213	m³/s

The study reveals that the Luegere River features a favorable hydrology at the proposed location of the hydroelectric project. However, hydrological uncertainties are important and it is strongly recommended that hydrological monitoring of the river be done beyond this study. This will include:

[3] For more details about SCS Curve Number method: https://www.hydrocad.net/neh/630ch10.pdf
[4] For more details about SCS Unit Hydrograph method: https://www.hydrocad.net/neh/630ch16.pdf

- To continue the measurement of water levels at the automatic station installed downstream the hydroelectric project;
- To continue the gauging operations of this river in order to improve and validate the rating curve.

Beyond the development of the Luegere hydroelectric project, it is strongly recommended that the Government of Tanzania set up a hydrological monitoring network for its rivers with high hydropower potential in order to better understand the available water resources and thus promote the development of hydroelectric projects across the country. It is only in a context of reduced uncertainties through reliable, recent and long-term records (more than 20 years) that technical parameters and economic and financial analyzes of hydroelectric developments can be defined accurately, enabling optimization of their design and their flood control infrastructure (temporary and permanent).

6 GEOLOGY

6.1 INTRODUCTION

The purpose of this chapter is to generate preliminary geological datasets and other important baseline information at the proposed site that will be used for the design of the hydroelectric scheme at the pre-feasibility study level. These data and information will also be used to define the geotechnical investigations that will have to be carried out at next stages of the study.

This study aims to inform about the geological conditions and the types of materials existing in the region, as well as to give an initial overview of the geotechnical properties of these materials. Recommendations are also formulated regarding the need for further studies and investigations if necessary.

6.2 GEOLOGICAL REFERENCE MAP

The geological reference map is sheet 132, Kakungu.

6.3 LOCAL GEOLOGICAL SETTING

6.3.1 Geological context

The geology of the area (Figure 18) is characterized by phyllitic rocks and aphibolitic rocks. These rocks don't seem to be significantly fractured or jointed. A limited number of prominent joints show that one joint set trends NNE-SSW, whereas the prominent foliation is characterized by shallow dipping planes due south (Figure 19).

Figure 18. Location and Geology map of Site SF022. As per this regional map, site lies within plagioclase amphibolites with some amphiboles and acid injection materials). QDS 132, Geological Survey of Tanzania.

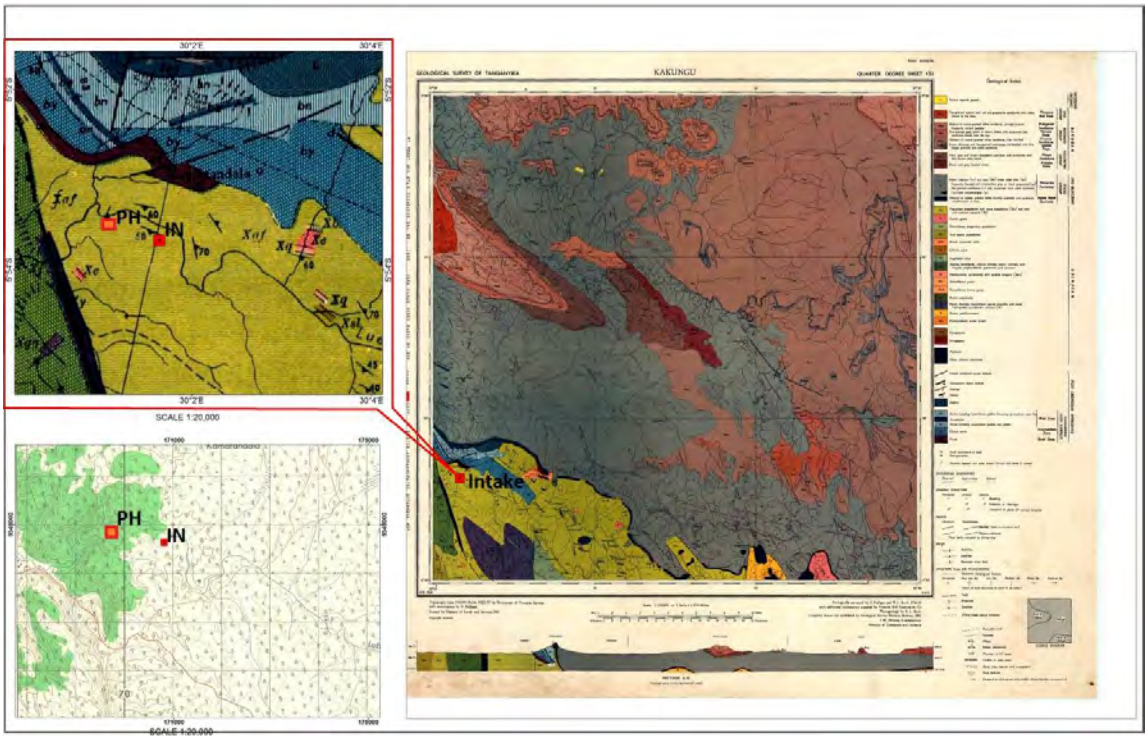
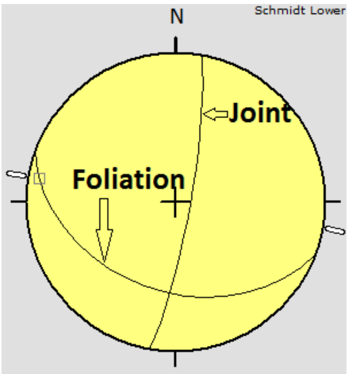


Figure 19. NNE-SSW Prominent joint set, SF022



6.3.2 Technical characteristics

Proposed intake or weir position: At the proposed weir position (Figure 20), the geology looks ideal because there are rocks right from the left bank where the photograph in Figure 20 was taken to the right bank. The rocks seem to be moderately weathered but less fractured / jointed. This implies that the rocks are likely to be suitable for weir construction. Later studies (during dry season) shall point out any element of doubt on this site if any.

Figure 20. Proposed weir position

Left bank support aspect: Rocks are of similar characteristics as those of the proposed weir position.

Right bank support aspect: Same as for left bank support aspect.

Intake and waterway: The water intake and waterway are possible to be set on the left bank. On this side, the topography is relatively smooth and geology suitable.

Powerhouse: The powerhouse platform has to be established by excavating the ground as indicated by the purple circle in Figure 21. Field relationship shows that about 2m or more of loose materials need to be excavated at this point in order to intersect stable rocks, the foliated quartzo-feldsparthic gneiss (Figure 22).

Figure 21. A snap of a topographic map showing the location of the power station

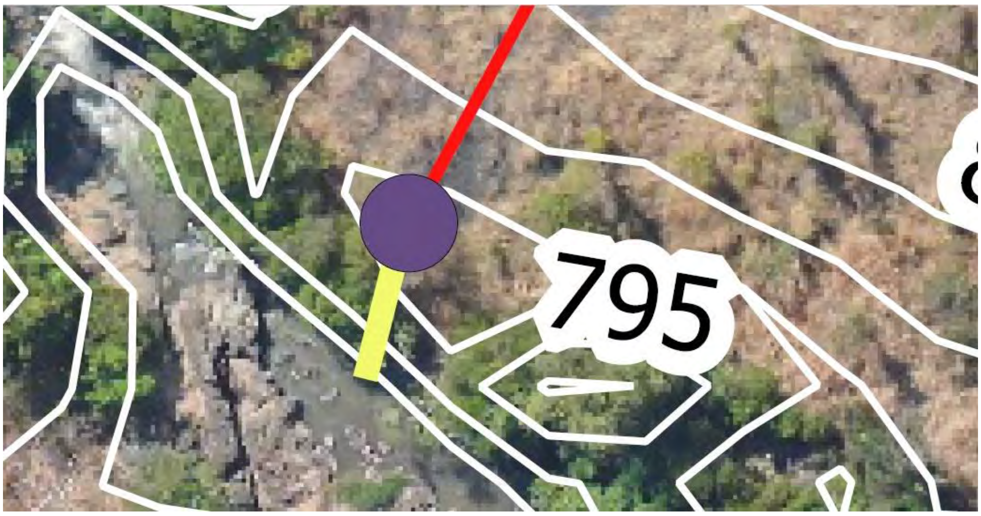


Figure 22. Foliated quartzo-feldsparthic gneiss as observed 10m west of the proposed power station

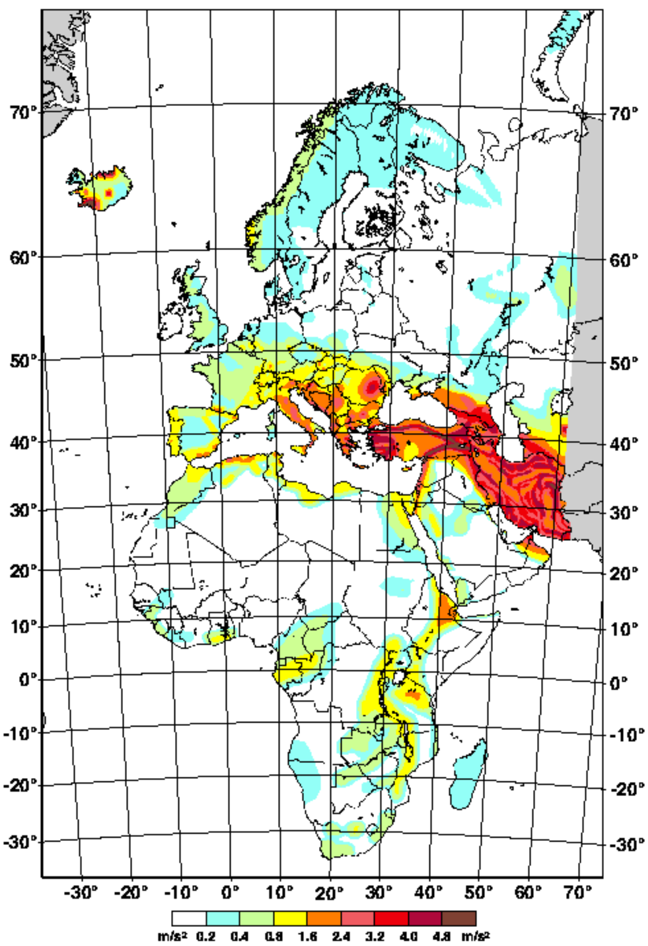
6.4 Seismicity

Tanzania is located along the Great African Rift. Seismicity in this area is relatively unknown, mainly due to the lack of historical data. Within the framework of the Global Seismic Hazard Assessment (GSHAP), the assessment of the seismic hazard in West Africa was carried out based on two data sources:

- The catalog of the British Geological Survey (Musson, 1994), containing quakes of magnitude greater than 4 from 1600-1993 (this is assumed to be complete for magnitudes greater than 5 beyond the year 1950 and for Magnitudes greater than 6 since the beginning of the 20th century),
- The NEIC catalog for more recent events (1993-1998).

A statistical method was used to determine the horizontal acceleration values due to earthquakes. The map below shows the distribution of seismic acceleration coefficients for the entire African continent. It can be seen that the project area is characterized by horizontal accelerations between 0.4 and 1.8 m/s². Those values will of course have to be confirmed by additional studies.

Figure 23. Horizontal acceleration due to seismicity (source: GSHAP)



6.5 Conclusions and Recommendations for Additional Investigations

6.5.1 Conclusion

There are no major geological contraindications to the construction of the Luegere hydroelectric scheme. There is no visible geological risk at this stage of the study.

However, further investigations will have to be carried out during the detailed studies phases in order to confirm the observations (and analysis) made concerning geology and geotechnical characteristics (rock resistance, soil strength, rock compactness, rock permeability, etc.).

A table presented in the following section summarizes the uncertainties to be removed and the type of investigations to be carried out to remove them.

6.5.2 Additional investigations

ELEMENT	UNCERTAINTY TO REMOVE	SURVEY TYPE
Bed at weir	✓ There are limited structural (and lithologic units) data on site due to the fact that almost all the rocks are covered by voluminous flowing water	✓ Undertake detailed studies on structural geology so as to propose the optimal orientation of the weir (Figure 19). This has to be undertaken when the water level is lowest.
Left support at weir	✓ No uncertainty	✓ N/A
Right support at weir	✓ No uncertainty	✓ N/A
Waterway	✓ Lithologic units and soil type along the entire trace of water way, not well established due to accessibility challenges in some parts of the river bank	✓ Visit and study all the areas that shall be proposed after detailed topographic studies. Major aim shall be to study the type of rocks and structures along the length of the water way trace.
Penstock	✓ No earmarked uncertainty	✓ N/A
Powerhouse	✓ Quantity of overburden (boulders and soils).	✓ Excavate and make a platform for the power station. The materials to be excavated shall be ≥2m

6.6 REFERENCES

Compilation of the GSHAP regional seismic hazard for Europe, Africa and the Middle East (http://www.seismo.ethz.ch/static/GSHAP/eu-af-me/euraf.html).

7 PRELIMINARY ENVIRONMENTAL AND SOCIAL IMPACT ANALYSIS

The Environmental and Social Impact Assessment (ESIA) is the procedure for prior analysis of the impacts that a project may have on the environment. It ensures the integration of environmental concerns into project planning and allows for consideration of likely environmental measures from the design stage of the project.

7.1 SOCIO-ENVIRONMENTAL BACKGROUND

The project is located in the Luegere River near the two villages of Mgambazi and Lukoma, in Uvinza District, Kigoma Region.

Figure 24. Proximity of the villages of Mgambazi and Lukoma (IN = intake)

The area is characterized by gentle to steep hills (called Mgambazi hills) with a moderate vegetation cover. The proposed project area is used for agriculture. Most of crops observed during the site visit were maize and beans for food crops and sugarcane and banana for cash crops.

Figure 25. Agricultural plots

The vegetation cover in the area is composed of Miombo woodland dominated by Mibaga, Mikongo, Mininga.

Figure 26. Vegetation cover

Figure 27. Agricultural plots along the access path close to the powerhouse

Mahale National park boundaries are very close to the site. The forests have various valuable tree species such as Pterocarpus angolensis (Mninga), Khaya nyasica (Mkangazi), Afzelia quanzensis (Mkora), Milecea- exelsa (Mvule), Brachystegia spiciformis (Mtundu), and Pterocarpus all species (Mkurungu).

Moreover, some tombs and headstones are present near the site (between 100m and 700m).

7.2 WORLD BANK OPERATIONAL POLICIES AND GUIDELINES

The World Bank has developed a series of operational policies (OP), or safeguards, to help identify, avoid, and minimize social and environmental impacts. These operational polices and safeguards are prerequisites to accessing the World Bank funding assistance to address certain environmental and social risks for specific development projects. There are 11 OPs and associated World Bank procedures that apply to environmental and social risks. Similarly, there are eight IFC performance standards. The details will be provided as part of either the prefeasibility or feasibility studies for each priority projects.

This section summarizes the World Bank's safeguard policies that contribute to the sustainability and effectiveness of development within the World Bank's projects and programs by helping to avoid or mitigate the impacts of these activities on people and society, environment. It ensure potential adverse environmental and social impacts that may result from individual project activity are identified early, and appropriate safeguard measures are prepared prior to implementation to avoid, minimize, mitigate and, in cases where there will be residual impacts, offset or minimize adverse environmental and social impacts.

The following World Bank safeguard policies could be triggered when implementing the Luegere hydroelectric project:

- OP 4.01 – Environmental Assessment (EA): The Bank requires Environmental Assessment (EA) of projects proposed for Bank financing to ensure that they are environmentally sound and sustainable, and thus to improve decision making. However, we can already estimate that the adverse impacts on human populations and environment-linked areas are limited. They are reduced, not irreversible and some measures can prevent, mitigate or minimize them. Moreover, these measures can improve the environmental performance.

- OP 4.11 – Cultural Heritage: There are a few tombs close to the site.

- OP 4.12 – Involuntary Resettlement: The project needs new use of some areas (implementation of the plant, renovation of the access roads to the site…) that can be crop zones.

- OP 4.04 – Natural Habitats: Not applicable.

The projected weir (4.5m high) is classified as a small dam (<15m high); the usual generic safety measures for dams are appropriate and do not need the implementation of OP 4.37 – Safety of Dams (for large dams).

The triggering process of the policies will be completed by the World Bank during dedicated projects appraisal.

7.3 Socio-environmental constraints

Overall, the project does not feature any major environmental or social constraints that cannot be mitigated by appropriate measures and that would jeopardize its development.

Some hamlets are situated in the vicinity of the proposed scheme (weir, canal, penstock pipes and powerhouse). Potential impacts (noise, traffic, atmospheric emissions) on the riparians during the construction phase will have to be mitigated by appropriate measures.

Overall, the development of the project will lead to positive externalities by the use of local labor during the construction phase, increase local skills and bring electricity to local communities that will eventually foster local economic development.

Finally, as mentioned above, a water supply project was intended to be built. However, the project was not completed because of shortage of fund. According to the Village Executive Officer (VEO-Likoma), the fund from the donors were provided to Uvinza District Council, but was missed.

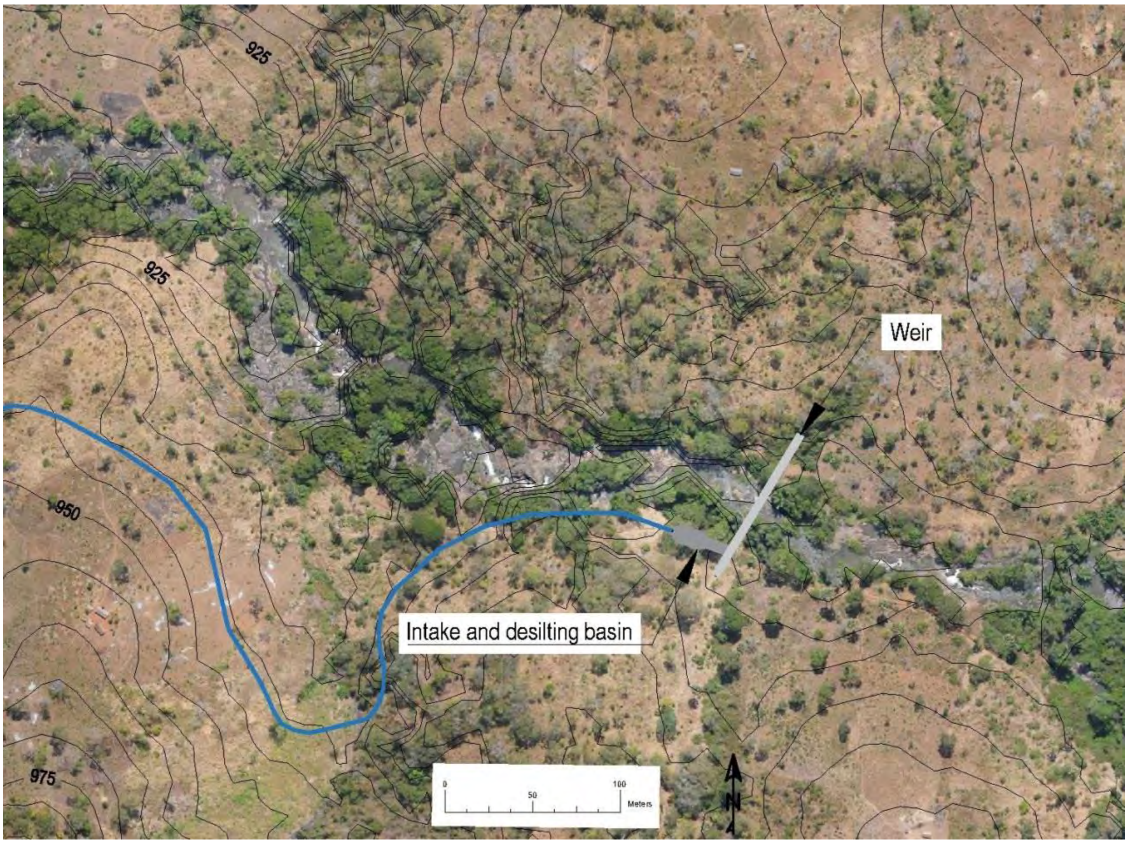
8 PROPOSED SCHEME AND DESIGN

8.1 PROPOSED SCHEME DESCRIPTION

8.1.1 Diverting structure, intake, waterway and powerhouse

As illustrated in Figure 28, the diverting structure (consisting of a weir) is located on the bedrock of the river which is on the riverbed in this section of the river. Moreover, this position allows to set a 50-m long overflowing weir and minimizes the water level over the crest during exceptional floods. It is followed by a small waterfall 100m behind the weir, which should ensure that headrace canal is above flood level. The natural river slope behind the weir also allows desilting basins to flush properly the accumulated sediments.

Figure 28. Weir**, intake and desilting basin's position**

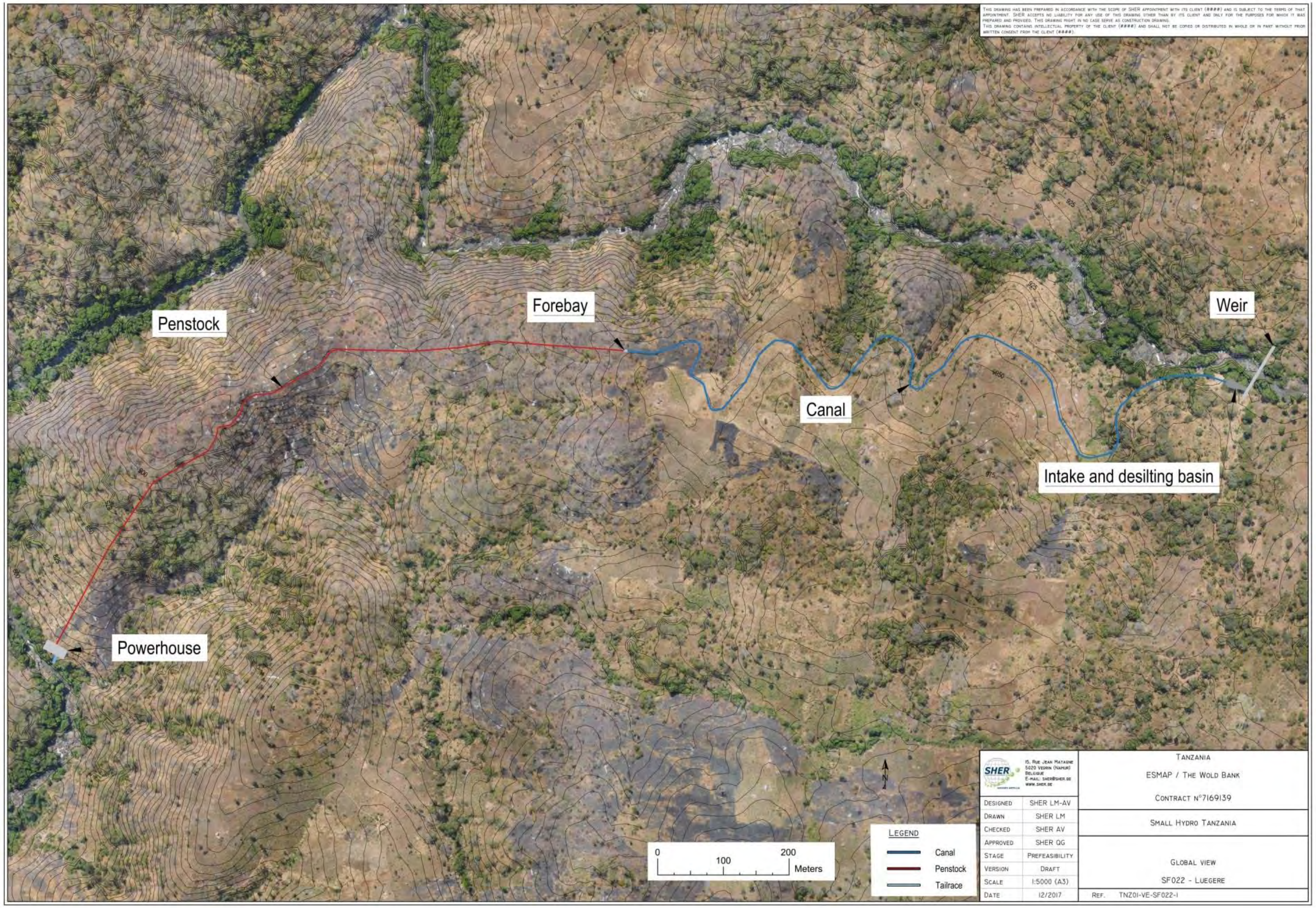


The entire proposed scheme is presented in Figure 29 below and in Appendix 12.1. The intake structure, waterway and power plant will be located on the left bank of the river. The valley features several thalwegs hence civil works will be needed in some places.

The access to the intake and the desilting basin will require a crossing bridge over the weir.

The weir is equipped with a gated flushing channel to prevent the accumulation of sediments in front of the intake and the weir. A 1 420m long headrace canal (rectangular section) will convey the water from the intake structure (including a desilting structure) to the forebay. The intake and the waterway are designed to minimize the head losses. The 1 110m long pressure penstock will convey the water from the forebay to the hydroelectric power plant located on the left bank of the river, above extreme flood level.

Figure 29. Detailed proposed scheme and main components



Geographical coordinates of the main structures are presented in the table below:

Structure	Latitude*	Longitude*
Weir and intake	-5.895	30.028
Hydropower plant	-5.898	30.011

* Decimal degree, WGS1984

8.1.2 Type of scheme

The Luegere project is a run-of-the-river hydropower type of scheme without regulation capacity.

8.1.3 Design flow

The flow duration curve was determined in chapter 5 of this report. It does not however correspond directly to the flow available to the equipment. Indeed, the Luegere River will be by-passed over a length of approximately 3.2 km. An environmental flow guaranteed at all times is required for environmental and ecological reasons.

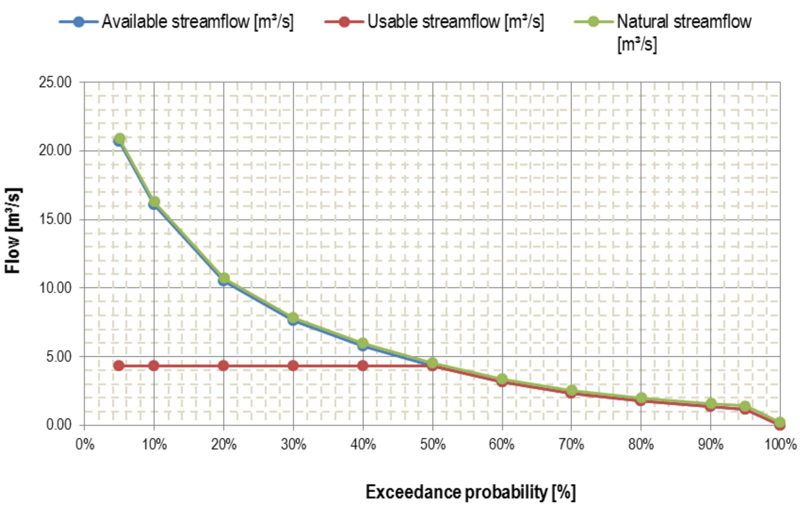
In the absence of commonly agreed international standards and given the uncertainties on the available streamflow of the river, the ecological flow is set at 230 L/s, which corresponds to 5% of the median flow ($Q_{50\%}$) of the river in natural conditions. Since this flow is not available for the turbines, it is necessary to subtract it from the flow duration curve of the river.

The flow duration curve that can actually flow through the turbines is finally obtained by considering the design flow rate of equipment chosen at the pre-feasibility stage, namely 4.33 m³/s. The usable flow duration curve is illustrated in Figure 30 below.

The final choice of design flow will be made at the feasibility study stage based on an economic analysis of alternatives. The flow duration curve must also be validated by the additional hydrological analysis and measurements.

Figure 30. Usable flow duration curve of the Luegere River at the project location

Streamflow [m³/s]		Exceedance Probability
Natural	Available	[-]
1.41	1.18	$Q_{95\%}$
1.59	1.36	$Q_{90\%}$
1.99	1.76	$Q_{80\%}$
2.57	2.34	$Q_{70\%}$
3.39	3.16	$Q_{60\%}$
4.56	4.33	$Q_{50\%}$
5.99	5.76	$Q_{40\%}$
7.83	7.60	$Q_{30\%}$
10.74	10.51	$Q_{20\%}$
16.29	16.06	$Q_{10\%}$



8.1.4 Design Floods

Several national bodies have examined the problem of defining the relevant design flood to be considered for the design of spillway and other associated flood structures. Only US method is developed below.

According to USACE (United States Army Corps of Engineers) in *Recommended guidelines for safety inspection of dams*, dam are classified in accordance with 2 characteristics: (i) the size of the structure and (ii) the potential hazard. The tables below present the classifications.

Table 10. Size classification (USACE)

CATEGORY	STORAGE (AC-FT – HM³)	DAM HEIGHT (FT – M)
Small	< 1000 Ac-ft < 1.2 hm³	< 40 Ft < 12.19 m
Intermediate	> 1000 Ac-ft et < 50 000 Ac-ft >1.2 hm³ et < 61.7 hm³	> 40 Ft et < 100 Ft 12.19 m et < 30.48 m
Large	> 50 000 Ac-ft > 61.7 hm³	> 100 Ft > 30.48 m

In the table above, the height of the dam is calculated from the lowest point of the structure to the maximum level of the reservoir. The category is defined either by the storage capacity of the reservoir or by the height of the dam, depending on the characteristic that classify the dam into the less favorable category.

The proposed weir on the Luegere will be less than 12m high and the storage volume or the reservoir will be less than 1.2 hm³. Therefore, the proposed weir is classified as being *"Small"*.

As far as potential hazard is concerned, it can be considered as *"Low"* according to the table below: there is no risk of loss of human life in the event of failure or misoperation of the diverting structure or appurtenant facilities. There is no significant industry or cultivated area have been identified downstream of the proposed hydropower projet.

Table 11. Hazard potential classification (USACE)

CATEGORY	LOSS OF LIFE (EXTENT OF DEVELOPMENT)	ECONOMIC LOSS (EXTENT OF DEVELOPMENT)
Low	None expected (No permanent structures for human habitation)	Minimal (undeveloped to occasional structures or agriculture)
Significant	Few (No urban development and not more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than a few	Extensive community, industry or agriculture

Table 12 presents the USACE's recommendations for the design flood to be considered as a function of the potential hazard that may occur in the event of failure or misoperation of the diverting structure or appurtenant facilities and the size of the structure. The flood is expressed either by its return period (or frequency) or by the PMF. The PMF (Probable Maximum Flood) is the largest possible flood that can occur through the most severe combination of critical meteorological, geographic, geological and hydrological conditions reasonably possible in a watershed.

Table 12. Recommended spillway design floods (USACE)

HAZARD	SIZE	SPILLWAY DESIGN FLOOD
Low	Small	50 to 100-year frequency
	Intermediate	100-year to ½ PMF
	Large	½ PMF to PMF
Significant	Small	100-year to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
High	Small	½ PMF to PMF
	Intermediate	PMF
	Large	PMF

Following the aforementioned guidelines of the USCA, the recommended design flood for the Luegere hydroelectric scheme is from 50-years to 100-years frequency. The hydrological study presented in chapter 5 estimates the 100-year return period flood to be 220 m³/s.

8.2 STRUCTURES DESIGN

8.2.1 Diverting structure type and characteristics

Given the nature of the foundations as well as the estimated water head on the diverting structure for the design flood, a concrete gravity-overflow weir is the most appropriate structure. A concrete structure is also particularly recommended for submersible structures. This choice is motivated by the following elements:

- The local geology shows that the rock is of good quality, adapted to the foundations of a concrete weir;
- Given the magnitude of the design flood, the weir must be as low as possible in order to minimize the impact of the upstream water level rise;
- An ungated weir/spillway will be easier to build and safer in design since there is no risk of dysfunction or misoperation of the gates, particularly during flood events.

The crest length will be 50m, which limits the water level over the crest during floods. The weir will be equipped with a gated flushing channel on the left bank to flush the sediments that would have accumulated in front of the water intake (see section 8.2.3).

The main function of a spillway is to allow the passage of normal (operational) and exceptional flood flows in a manner that protects the structural integrity of the structures and its foundations.

The overflowing section of the weir will be designed with an ogee-type profile (Creager). The profile of this type of weir is close to the hydraulic profile of the nappe springing freely from a sharp crested weir. The advantage of such a profile is that, at an equivalent discharge, the ogee-type weir is characterized by a lower rise in the water level compared to a broad-crested weir. Similarly, considering the same hydraulic head on the spillway, a longer crest length is required for a broad-crested weir than for an ogee-shaped weir.

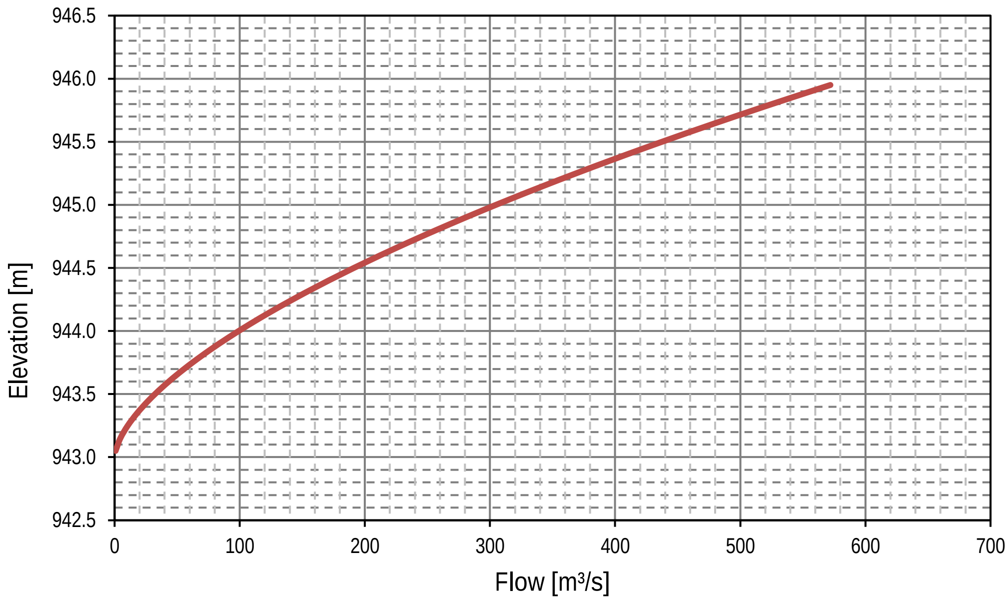
Moreover, this profile also ensures the stability of the weir. The upstream face of the weir will be vertical at this stage of prefeasibility study but will have to be confirmed during the feasibility study based on a more detailed topography. The crest of the weir will have a hydraulic profile that meets the US Army Waterways Experimental Station (WES) standards and recommendations to minimize the risk to the weir structure due to negative pressure and cavitation under the nappe. The discharge flowing over a spillway is calculated based on the following equation:

$$Q = C_d L h^{\frac{3}{2}} \sqrt{2g}$$

Where Q is the discharge [m³/s], C_d the spillway coefficient [-], L the length of the overflow crest [m], h is the total hydraulic head (static and dynamic head) over the crest [m] and g is the gravitational acceleration [m/s²].

Considering an ogee-shaped overflow weir, the hydraulic head over the crest for the design flood (220 m³/s) will be 1.65m. The Creager rating curve is presented in Figure 31 below.

Figure 31. Creager rating curve

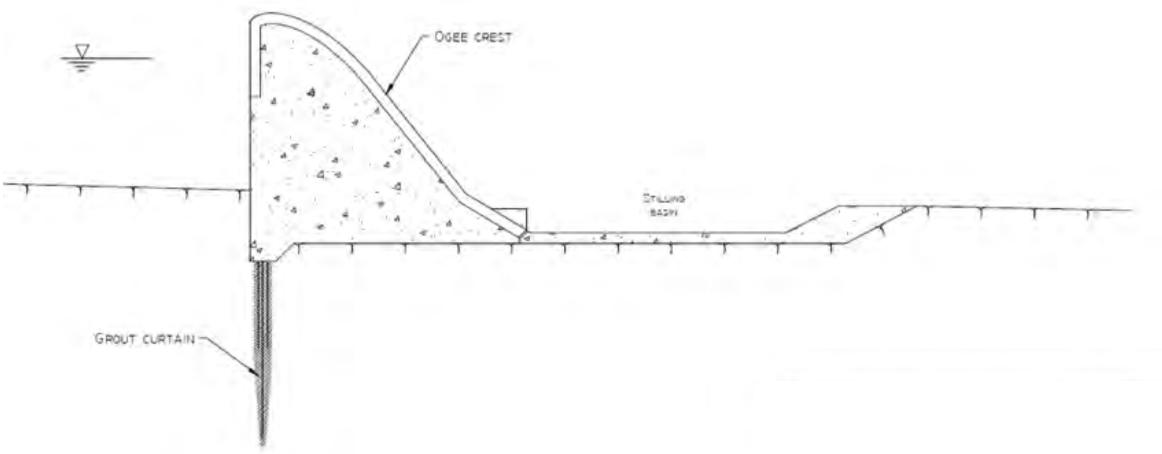


The main features of the weir are presented in Table 13 and a typical cross section of the profile is shown in Figure 32.

Table 13. Weir key features

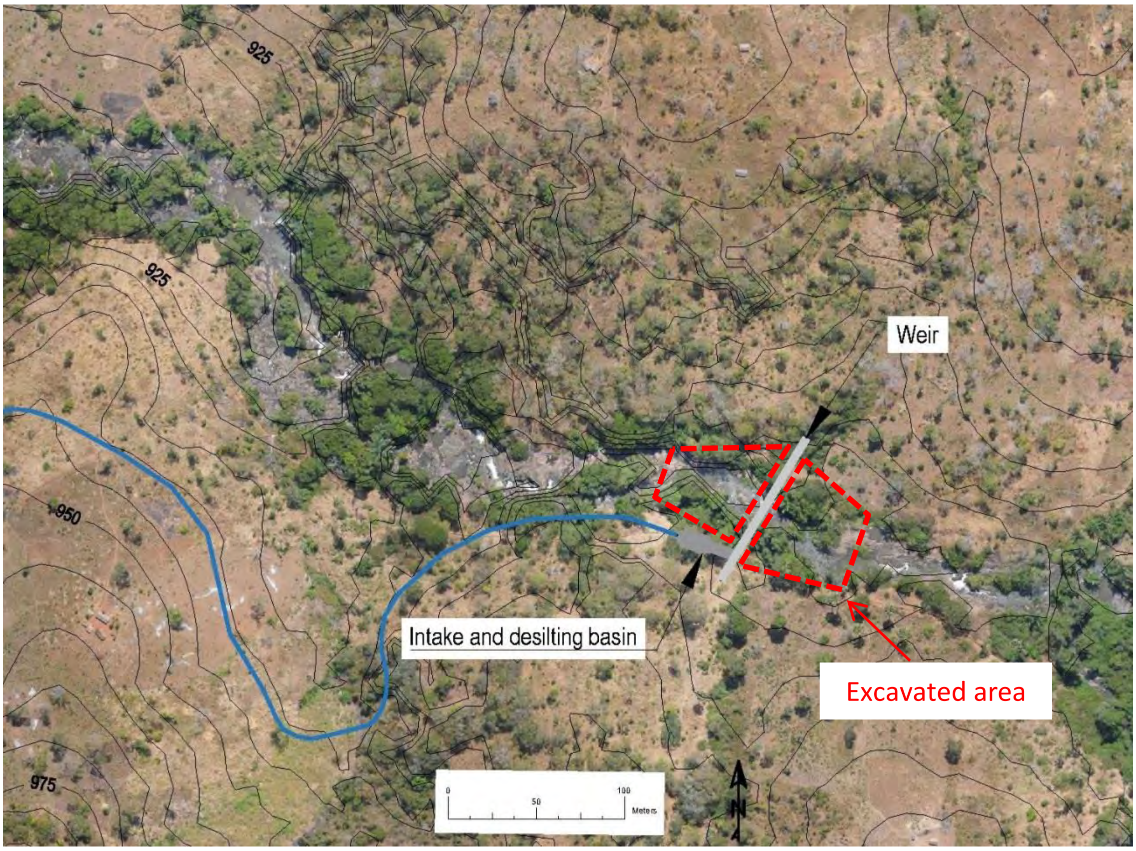
Parameter	Value	Unit
Weir type	Gravity Creager overflowing section	
Material used	Concrete	
Overflowing crest length	50	m
Total weir length	70	m
Overflowing section height	4.50	m
No-overflowing section height	7.15	m
Crest elevation	943.00	masl
Slab elevation	938.50	masl

Figure 32. Typical cross section of a Creager weir



In order to obtain a 50-m overflowing crest length, excavation on both side of the river are needed. It is recommended to reshape the riverbanks upstream and downstream of the weir in order to remove the accumulated sediments and improve the hydraulic conditions for the intake and overflowing weir. The approximate area to be reshaped is illustrated in Figure 33.

Figure 33. Approximate area to be reshaped upstream and downstream the proposed weir location



8.2.2 Temporary diversion

The purpose of the temporary diversion is to dry up part of the river to allow the construction of the weir and appurtenant structures described in the previous section. The temporary diversion will be implemented consecutively on the left bank in order to construct the gated flushing channel and the intake, then on the right bank.

It will consist of a compacted embankment cofferdam or, if the ground conditions are favorable, sheet piles.

8.2.3 Outlet structures

The outlet structure consist in a gated flushing channel. It is designed to allow inspection of the weir and intake. In addition, the outlet structure while open can create a strong current with the effect of flushing the accumulated sediments close to the intake structure.

The flushing channel will be equipped with gates of which the invert is positioned at an elevation close to the elevation of the natural riverbed. The gates will be located on the left side of the weir, next to the intake structure to allow an effective flushing of the accumulated sediments.

The number of bays and their size were calculated to ensure outflow corresponding to twice the $Q_{30\%}$ streamflow of the river (15.66 m³/s). This objective is achieved with the installation of two 1.40m wide and 1.50m high radial gates.

Table 14. Flushing gates characteristics

PARAMETER	UNIT	VALUE
Invert elevation	masl	938.50
Number of bays	-	2
Width	m	1.40
Height	m	1.50

8.2.4 Waterway

8.2.4.1 Intake structure

The intake will be located on the left bank in the continuity of the weir.

The intake will also be equipped with a screen and a manual screen cleaning system upstream of the intake gates, to prevent floating debris or large stones from obstructing the intake gates. The section of the bars and their spacing will be determined at the feasibility study stage.

The intake is designed taking into account the following constraints:

- The invert elevation will be set 2.50m above the invert of the flushing gates;
- The velocity of water at the entrance of the screen should not be greater than 0.7 m/s to minimize turbulence and facilitate screening of debris. That will also minimize head losses.

Hence, the intake will consist of 2 bays of 2.00m wide and 2.00m high, followed by a free inlet that will guide the current lines gradually towards the desilting structure. The invert of the intake will be set at elevation 941m. Details are presented in Table 15 hereafter.

Table 15. Intake characteristics

PARAMETER	SYMBOL	UNIT
Intake invert elevation	masl	941
Intake top elevation	masl	943
Screen inclination	°	15
Design flow	m³/s	4.33
Number of bays	-	2
Bay width	m	2.20
Bay height	m	2.00
Type of gate	-	radial
Flow velocity at intake	m/s	0.7

The free inlet which objective is to allow smooth converging of the current lines to the desilting structure will, for hydraulic reasons, be approximately 2.5 times the width of the intake, i.e. 15m. The feasibility study will analyze the hydraulic behavior of the intake in detail and update its design accordingly.

8.2.4.2 Desilting structure

Solid transport is expected to be high, especially during the wet season. Consequently, the intake and desilting structures must be adequately designed to ensure the removal of the problematic sediment load before entering the headrace canal. It is recommended that the feasibility study include a solid transport study.

Figure 34. Turbidity of the water close to the proposed weir location

If not taken into account at the design stage, it would result in operational and maintenance problem of the hydroelectric plant. The sediments that would accumulate in front of the intake will be flushed by frequent flushing operations using the flushing gates designed for this purpose.

The inlet of the desilting structure will have a sufficient slope in order to guide the solid particles to outlet of the basins. Moreover, the desilting basins will be long enough to ensure particle settling.

The desilting structure is design based on topographic, hydraulic, type of sediments and operation constraints.

At the pre-feasibility study stage, the key features considered for design are presented in the following table:

Table 16. Preliminary design criteria for the desilting basin

PARAMETER	UNIT	VALUE
Invert elevation	m a.s.l.	941
Sediment outlet elevation	m a.s.l.	940
Water outlet elevation	m a.s.l.	941
Design flow	m³/s	4.33
Average solid inflow	kg/m³	0.8
Minimum diameter of the particles	mm	0.3
Maximum flush frequency	hours	24

The width of the desilting basin is determined in such a way that the horizontal water velocity is less than the maximum horizontal speed (which is determined based on the particles diameter). The length of the desilting structure is determined in such a way that a particle located on the surface can be deposited in the reservoir of the desilting structure. The horizontal and vertical velocity ratio is proportional to the ratio of the falling length to the falling height.

The desilting structure will therefore be composed of 2 sub-basins each 4.5m wide and will have a sedimentation length of 19.25m. To this must be added the transition zones upstream and downstream of the settling tank of the desilting structure. The desilting structure will therefore have a total length of 21.75m and a total width of 10.50m and a maximum depth of 3.00m.

The desilting structure will be equipped with a lateral spillway in the event of excessive inflows coming from the intake.

8.2.4.3 Headrace canal

The headrace canal features a rectangular cross section. The slope of the headrace canal is kept below 0.1% in order to minimize the head losses. The canal dimensions are defined on the basis of the uniform flow equation (Manning):

$$\frac{Q}{A} = V = n^{-1} R_h^{\frac{2}{3}} i^{\frac{1}{2}}$$

where *A* is the wetted area [m²], *V* is the mean flow velocity [m/s], *n* is the Manning coefficient, R_h is the hydraulic radius [m] and *i* the slope of the canal [-].

The headrace canal is designed taking into account:

- the average flow velocity is less than 2 m/s in order to avoid erosion of the concrete.
- the cross section is the most economical section: for a given discharge, slope and Manning coefficient, the discharge capacity will be maximum when the hydraulic radius (ratio of the wetted section on the wet perimeter) is maximum.

The canal features a rectangular cross-section of 2.00 m in width for a water height of 2.00 m, to which is added a freeboard of 30cm, which results in a total height of 2.30 m. The headrace canal is 1420m long.

8.2.4.4 Penstock

The headrace canal and the pressure penstock meet at the forebay. The forebay will be equipped with a scour gate in order to drain the channel as well as the particles that would have sedimented in the latter back to the river. The forebay will be equipped with a safety spillway in the event of excessive inflows coming from the headrace canal or allowing the spill of the water in excess during variations flow through the turbines (production decrease, shutdown of a group, etc).

The pressure steel penstock will be overground and 1110m long. The penstock will be supported by reinforced concrete support blocks. At this stage of the study, the distance between two support blocks is 6m. Anchoring blocks will be placed at each elbow to balance the forces related to the change of direction of the flow. A suitable system allowing the thermal expansion of the penstock should be defined at the feasibility study stage.

In order to limit the head losses to a maximum of 8% of the gross head, the penstock will have a diameter of 1.20 m.

8.2.5 Electromechanical Equipment

8.2.5.1 Basic data

The following specific values corresponding to the latitude and elevation of the powerhouse are used for the equipment predesign and calculation:

	Symbol	Unit	Value
Gravity Acceleration	g	m/s^2	9.778
Average temperature of water	T_{water}	°C	20
Density of Water at 20°C	ρ	kg/m^3	998.8

8.2.5.2 Selection of the type of turbine and the number of units

8.2.5.2.1 Methodology

The selection of the turbines type is made on the basis of the sites parameters such as the gross and net heads, and the plant design flow.

With a plant design discharge equal to $Q_{50\%}$, which is guaranteed 50% of the year, unit flexibility is needed to follow the river flow variations all along the year, as the hydropower scheme is a run-of-the river one. Moreover, the Luegere site being connected to the Kigoma mini-grid, flexibility will also be needed to follow the demand during the day.

In order to make a preliminary selection of the most suitable turbine type and of the number of units, a first selection is made according to the available head.

The choice of the number of units is based on several criterion:

- Flexibility and reliability: Even if some turbine types allow a strong flexibility, it is chosen to consider at least two units per site. This choice will prevent possible electricity delivery shortage as, at least, one unit will remain on the grid in case of maintenance or break.
- Standardization or systemization: Considering the expected installed turbine capacities (<10MW per turbine) for ESMAP project, units will be standardized or systemized. In one hand, the best efficiency points will be a little lower than for large units, but, in the other hand, the cost and delivery time will be reduced. Moreover, the maintenance will be easier than for custom made products.
- Access to the site and powerhouse infrastructure: As the site access can be a problem for larges equipment or equipment parts, it can be mandatory to increase the number of unit in order to allow their transport from the nearest harbor. The number of units also has a direct impact on the powerhouse. The greater the number, the bigger the power house, but the

bigger the crane capacity and the unit weight leading to high loads on the power house structure. Finally, the erection of smaller units will be easier than for bigger ones.

The preliminary turbine design is based on statistical values. The detailed analysis of the other alternatives and the optimization of the choice must be made during feasibility study if the site is selected at the end of the prefeasibility phase.

The power and rotation speed of the generators depend on the turbine hydraulic design. The selection process aims in finding the higher rotation speed (which reduces the size of the rotating parts and then price of the unit), taking into consideration hydraulic phenomenon as for instance cavitation.

The efficiency of the generators is assessed according to their power and speed.

At the prefeasibility stage, the power factor of the generators is considered as equal to 0.9.

All the technical data (preliminary dimensions, rotation speeds, efficiency level, etc.) are given for information only. They have to be understood as orders of magnitude and can vary in further studies steps in function of the requested accuracy level.

8.2.5.2.2 Selection process results

According to the net head (~147.6 m) and the available flow, two types of turbines can be considered:

- Two or more vertical Pelton turbines with 4 or more nozzles and rotational speed of 500 or 600 rpm or,
- Two or more high speed Francis turbine with 1’000 or 1’500 rpm.

Considering the installed capacity and the previous criterion, it is better to select at least 2 units to increase the reliability and the availability of the production. Moreover, the rotational speed with one Pelton turbine would be 300 or 333 rpm which is not a good option in terms of equipment availability, weight, cost, transport and maintenance.

A brief comparison of the Francis and Pelton alternatives is given hereafter:

- The flexibility of the Francis is significantly lower than the one of a multi-jet Pelton. However, it can be noticed that the Q_{95} flow is equal to 1.18 m^3/s, corresponding to 54% of the design discharge of one turbine, when selecting two units. So, both Francis and Pelton turbine are able to operate at this Q_{95} discharge with a rather good efficiency.
- At nominal and maximal discharge the efficiency of a Francis turbine is higher than the one of a Pelton turbine. Due to the choice of Q_{50} as design discharge and to the fact that Q_{95} represents 54% of Q_{50}, the preliminary calculation of the potential annual generation leads to a production of 34.4 GWh/yr with 2 Pelton turbines and 35.3 GWh/yr with 2 Francis turbine.
- Due to a higher rotational speed, the generator of the Francis unit will be less expensive than the one of the Pelton unit. Moreover, the casing of the Pelton turbine is roughly two times bigger than the spiral casing of the Francis, what probably increases the cost of the powerhouse.
- The penstock will be protected by a trash rack associated with a manual cleaning. Even with these equipment, it is not possible to exclude that solid materials, vegetal and other floating materials will pass through the rack and reach the turbine. The Francis geometry is more

sensitive to the floating material than Pelton, as it can, for instance, be blocked in the labyrinths, what will block the runner and lead to a runner dismantling.

The Pelton is less sensitive to that kind of floating material. Moreover, it is easier to clean it if needed (easy access by the tailrace for instance).

- The Francis need a watertight shaft seal, which is a wearing part. A vertical axis Pelton does not need one. Then maintenance is reduced and less spare parts are needed.
- The Pelton turbines are equipped with jet deflector that divert the jet from the runner in case of black out, for instance, avoiding that the unit goes to, or remains for a long time at, runaway speed. It is then possible to close the nozzle slowly in order to limit the overpressure in the penstock to a maximum of 1.2 times the static pressure due to the gross head. This is a very efficient protection against water hammer.
- The frequency regulation with a Pelton with 4 or more nozzles will be more accurate than with a Francis turbine. It will then be a strong advantage to control the frequency of the Kigoma mini-grid if the Luegere power plant output is high compared to the grid total power.
- The high inertia of the low speed Pelton turbine will facilitate the follow-up of the load variation, helping to keep the grid frequency constant.
- If the demand from the grid is less than 1MW, it could be a problem to supply it with one Francis unit.

The hereafter table gives an overview of the comparison

Table 17. Comparison between Pelton and Francis turbines

	PELTON	FRANCIS
Answer to discharge variations	Excellent	Medium
Answer to demand variation	Excellent	Medium
Start-up and synchronization	Easy	Easy
Sensitivity to floating material	Low	Medium
Sensitivity to solid material	Low	High
Frequency regulation	Excellent	Good
Watertight shaft seal	No	yes
Runaway risk	No	Yes
Water hammer risk	Low	Medium
Maximum efficiency level	0.89 – 0.92	0.92 – 0.94
Rotational speed in the case of Luegere	Low	High
Size in the case of Luegere	Large	Medium
Weight in the case of Luegere	High	Medium

Considering this short comparison and the prefeasibility study step, the Pelton turbine is selected. Francis alternatives could be considered in a full feasibility study.

With 2 Pelton turbines with 4 nozzles, the size of the casing is 3.6m, which could be a problem for the transportation by road between the harbor the power house. Moreover, the weight of 12 or 14 poles generator of 3’000 kVA is more than 20 t and could also be a problem for the transport and the maintenance operations. Then, for the prefeasibility study, a number of 3 units is considered and the highest rotational speed is preferred (5 nozzles or more).

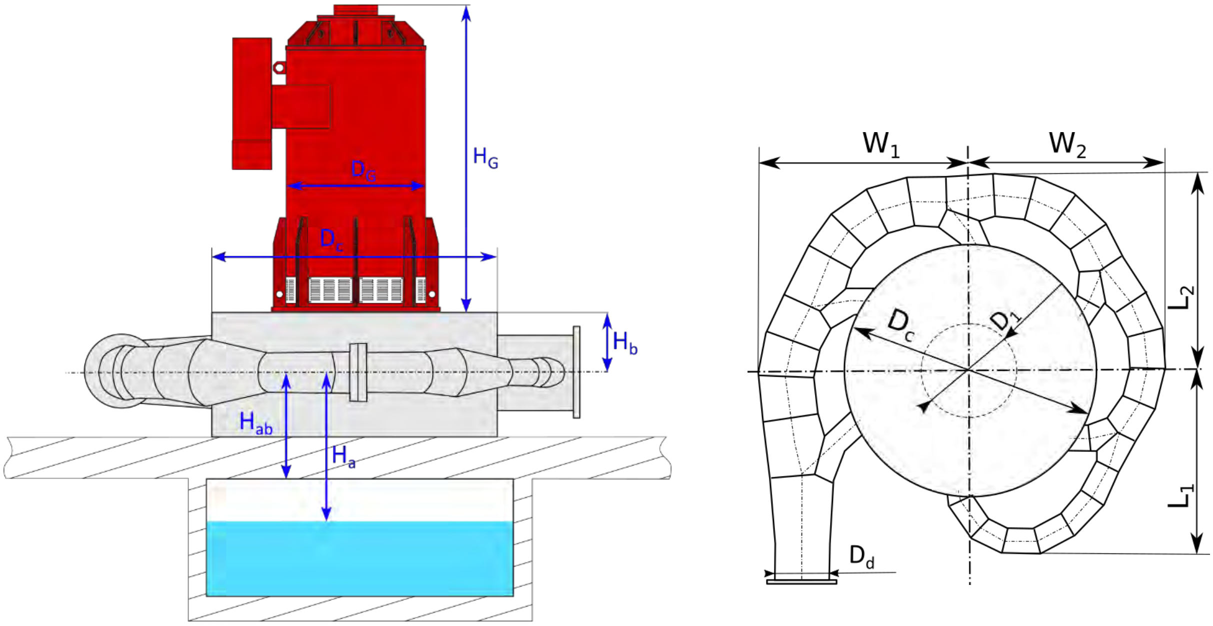
The main characteristics of the equipment are:

Turbine type		Pelton
Number of turbines	(-)	3
Nominal turbine discharge	m^3/s	1.443
Minimal turbine discharge	m^3/s	0.087
Net head (at Q_n and with all the turbines))	m	147.6
Rotation speed	rpm	600
Number of nozzles	-	5 or more
Max. Turbine efficiency	(%)	≈ 90.5%
Max. Generator efficiency	(%)	≈ 94.9%
Power Factor	(-)	0.9
Generator Apparent Power	kVA	≈1’980
Generator Power	kW	≈1’780
Generator voltage	kV	≥ 0.4

The preliminary main dimensions of the 3 Pelton units are:

D_1	Pitch diameter	m	0.81
D_c	Casing diameter	m	2.88
H_a min	Minimal height between the medium plan of the runner and the downstream water level	m	1.85
H_{ab} min	Minimal height between the ceiling of the tailrace channel and the runner axis	m	1.45
H_b min	Minimal height between the medium plan of the runner and the top of the casing	m	0.60
D_D	Diameter at the inlet of the turbine manifold	m	≈0.80
L_1	Length L_1 of the turbine manifold	m	2.7
L_2	Length L_2 of the turbine manifold	m	2.7
W_1	Width W_1 of the turbine manifold	m	2.7
W_2	Width W_2 of the turbine manifold	m	2.4
D_G	Diameter of the generator	m	1.5
H_G	Height of the generator	m	2.5
W_G	Weight of the generator	t	12

Figure 35. Main dimensions of the Pelton unit



8.2.5.3 Hydro and electromechanical equipment of the powerhouse

The plant equipment includes:

- Three security valves, spherical or butterfly type, equipped with counterweight as an emergency closing mechanism in the event of the loss of the grid
- Three Pelton turbines
- Three low voltage synchronous generators
- Three Step Up LV/MV transformers and the connection to the MV switchboards
- The cabinets for control and monitoring systems, included the speed and voltage regulators, metering and relaying panels for each unit
- The power plant control and monitoring cabinet
- The cabinets for Low Voltage distribution
- The Electrical protections and safety systems
- One auxiliary LV transformer
- One DC power supply and an Emergency diesel auxiliary power generator
- Earthing and Lighting system with their protection

The following points should be studied at a later step of the project:

- Sediment issue and the requirement of anti-abrasion coating,
- Need for flywheel (network stability),
- Grid connection voltage

8.2.5.4 Net Head Calculation

The Pelton turbine is an impulse turbine whose main characteristic is to use the velocity of the water to move the runner: The Pelton runner rotates in the air. Consequently the runner must be set above the maximum tailwater level to allow its operation at atmospheric pressure.

With a vertical Pelton turbine, the gross head is the difference between the water level in the forebay tank and the runner axis level. The floor elevation of the powerhouse is 786 masl. The height between the runner axis and the tail water will be of the order of 1.6 m. At the prefeasibility step, the tail water and floor levels are not accurately defined. Then the runner axis level is roughly assumed to be equal to the floor elevation.

The water level in the forebay tank is 941.6 masl, and thus the gross head is 156.6 m. Taking into consideration the head losses between the forebay tank and the inlet of the turbine manifold, the net head at nominal discharge is then equal to 147.6 m.

8.2.5.5 Overview of the units operation

The turbine governor will be controlled by the forebay tank level and the frequency measurement. The units operation is as follows:

- If the available discharge is lower than the minimal discharge of one turbine, the plant is in shutdown state;
- As long as the available discharge is between the minimal and maximal discharge of one turbine, only one unit is operating;
- If the available discharge is over the maximal discharge of one turbine and the demand is exceeding one unit capacity, a second unit is started. The discharge of the first turbine is reduced and the discharge of the second one increases until both turbines operate at the same nozzle opening.
- Then, the two turbines can operate with the same nozzle opening until they reach their maximum power. If there is enough water and the demand is exceeding two units capacity, the third turbine is started according to the same starting procedure as for the second unit.
- If the available discharge is larger than the maximal discharge of the hydropower plant, the excess water is released in the river at the intake location.
- If the discharge decreases, the automatic control system reduces the nozzle opening of the turbines in reversed order.

In case of shutdown of one or more turbine, the excess water is released in the river at the intake location.

The frequency regulation is used any time to adapt the production to the demand.

The forebay tank reference water levels to start or stop the units is set to avoid hysteresis.

8.2.5.6 Pelton turbines

The following description and preliminary design presented in chapter 8.2.5.2 are based on the consultant's database. They are given for information and may vary from one manufacturer to another. The turbines performances and characteristics (rotational speed, efficiency guarantees, reliability, etc.) are realistic as long as the turbines are designed and manufactured on the basis of a hydraulic profile issued from laboratory tests and developments.

As the number of nozzle is set to 5 or more, the axis of the turbine is vertical. Moreover, this configuration simplifies the maintenance as the overhung mounting of the runner on the generator shaft does not require shaft alignment and reduces the number of bearings.

The nozzle actuators are preferably hydraulic. In case of emergency, for instance during a load rejection event, deflectors divert the jet from the runner to avoid runaway speed. The actuators of the deflectors must operate in the event of a power failure.

Figure 36. Example of two vertical Pelton turbines with 5 nozzles

8.2.5.8 Generators

The main characteristics of the generator are presented hereafter:

Number of units	3
Type	Three phase, Synchronous
Axis	Vertical with turbine runner overhanging freely
Frequency (Hz)	50
Rated output (kVA)	1’980
Rated Power factor φ (-)	0.9
Rated Voltage (V)	Preferably 690 V or 400 V
Rated speed (rpm)	600
Maximum runaway speed (rpm)	~1’140
Primary coolant	air
Index of protection	IP 23 or above
Insulation class	F (design), operating B class

According to the maximal power, the generator shall be designed with a self-ventilating open air cooling system.

The efficiencies of the generators were assessed from a database collected from recognized generator manufacturers, with a particular emphasis on the rated power and rotational speed parameters.

8.2.5.9 Overhaul and safety valve

Each turbine shall be protected by a safety valve. It could be a DN 900 butterfly or DN 800 spherical valve with PN 25. The minimal discharge of a multi-jet Pelton being low, the cavitation criteria must be carefully checked before the final selection of the valve.

This valve can be used in case of maintenance and as a second safety device in case of emergency shutdown, deflector malfunction or failure. It opens with a hydraulic actuator and closes by counterweight.

8.2.5.10 High Pressure Unit (HPU)

Each unit will have its own High Pressure Unit to drive the nozzles, the deflectors and the safety valve. It will include one hydraulic bladder in case of high pressure pump failure.

8.2.5.11 Control and monitoring system

The plant operation being expected to be entirely automatic, its control and monitoring system has to be as simple as possible, so as to reduce human intervention to a minimum.

The discharge will be controlled by the water level in the forebay tank, which will be measured by mean of a level gauge connected to the plant by optical fiber or other means.

Each unit will have its own control and monitoring cabinet with its own PLC. One additional control and monitoring cabinet and PLC will be installed to control the whole power plant.

It will be possible to operate the units either automatically or manually. In order to prevent untimely operations, manual controls must be locked with a key.

The plant will restart automatically in case of power outage. However, for safety reasons both with regards to the plant’s operation and maintenance staff and to the electric grid, the plant will not restart automatically after an alarm, even if it would disappear without human action.

The electric cabinets will at least include the following elements: safety valve opening, nozzle opening, Power Factor regulation, voltage and frequency controls, and emergency power supply.

The following measurement instruments will be used: Grid and generator voltmeters, wattmeter, frequency meter, power factor measurement, synchroscope, speed sensor, headrace level, hours counter, start-up counter, bearings and alternator coils' temperatures, emergency shut-down, emergency power-supply charge level.

The following alarms will have to be considered: Insufficient water level, insufficient head, too low or too high frequency, alternator overload, overspeed, emergency shut-down, start-up fault, bearing defect, coils defect, current return, battery overload, battery defect.

The plant could be remote-controlled.

8.2.5.12 Emergency power-supply

A 48, or 110 V emergency power supply consisting in batteries, battery chargers, inverters, load indicators, protections, etc., will insure safety in case of power failure. Battery alarms for defects or overloads will be transmitted to the power plant control system. Under normal conditions, the emergency power supply will be powered by the low tension grid. The energy storage must be sufficient to ensure a safe turbine shutdown.

An emergency diesel set or a small Pelton turbine set will maintain power supply to essential feeders of the power house, weir and intake and eventually to enable black start of the HPP (to be studied in the feasibility studies).

8.2.5.13 MV transformer and switchboard

Each generator will be connected to a step up transformer enabling the outlet voltage to be increased to 30 kV. The main specifications of the LV/MV transformer area:

Number of units	3
Type	Dry
Rated Power (kVA)	2'200
Number of phases	3
Primary voltage (V)	400 or 690
Secondary voltage (V)	30'000

On the medium voltage side of the power transformers, a single 30kV/630A circuit breaker will be installed for each generator. This circuit breaker will be strong enough to stand the continuous operating current, as well as the peak short circuits. Its mechanism will allow the interruption of the short circuit to avoid any damage to the transformers, generators, and other electrical equipment.

8.2.5.14 Auxiliary Transformer

Ancillary services of the hydropower plants will be supplied by an auxiliary LV transformer with the following characteristics:

Number of units	1
Rated Power (kVA)	250
Number of phases	3
Primary voltage (V)	30'000
Secondary voltage (V)	400 or 690

A circuit breaker shall be installed to protect the auxiliary equipment.

The own consumption of the powerplant could be estimated roughly to 0.5% of the generated energy. This consumption is not taken into account in the energy production calculation and must be included as an expense in the financial analysis.

8.2.5.15 Overhead travelling crane

The power house will be equipped with an overhead crane that will be able to carry and place turbines, generators and other large devices during construction and maintenance operations.

8.2.5.16 Abrasion

Solid transport is expected to be high, especially during the wet season. The scheme is equipped with a desilting basin to limit or remove most of the sediment. It is recommended that the feasibility study includes a solid transport study. According to the results of this study, especially the composition of the transported particles, the decision to add a protective coating for the critical turbine parts could be adopted.

8.2.6 Power and energy generation performance assessment

The yearly electricity production is calculated by compiling the energy generation according to the flow duration curve and using the following expression:

$$E_{etot} = 10^{-3} \int \rho \; g \; Q_t \; \eta(Q_t) \; H(Q_t) \; dt \quad [\text{kWh/year}]$$

Where E_{etot} = total yearly energy production [kWh/year]

ρ = water specific weight [kg/m^3]

g = acceleration due to gravity [m/s^2]

$\eta(Q_t)$ = Overall unit efficiency, product of turbine, generator and transformer efficiencies, function of discharge [-]

$H(Q_t)$ = Net head, function of global discharge of the power plant [m]

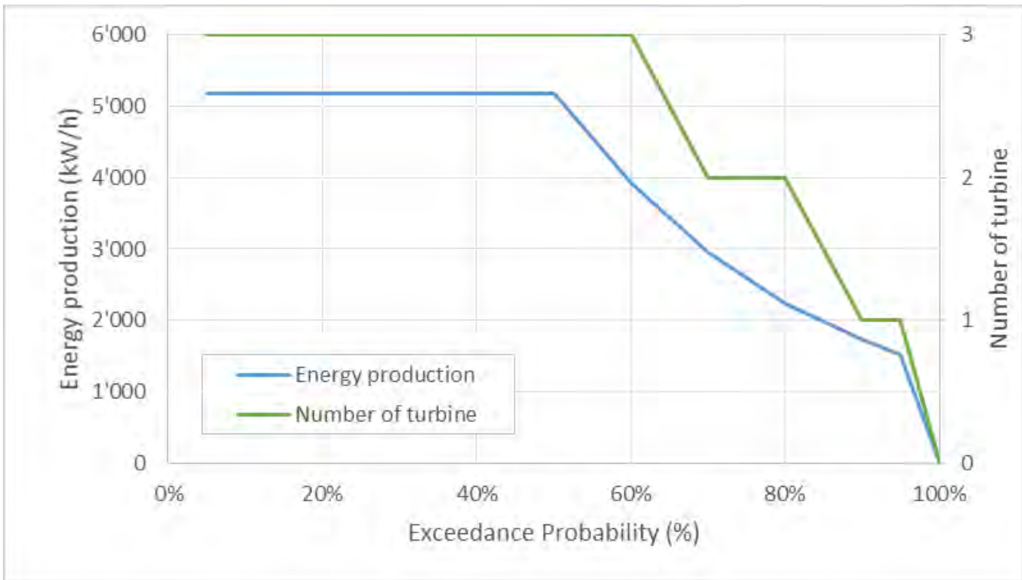
The used turbine efficiencies come from statistical curves based on real turbines of similar type, power and specific speed, taking into account the head and discharge variation.

The used generator efficiencies come from statistical curves based on real generators and taking into account the influence of the generator type, the rated output and the number of poles.

The rated efficiency of the step-up transformers is slightly higher than 99%. The efficiency of the power transformer used in the annual energy generation is considered constant and equal to 99%, independently of the load.

With the high flexibility of 3 multi-jet Pelton turbines, the overall efficiency is high and with small variation. Most of the time 3 units are in operation. It is the result of the design flow at $Q_{50\%}$.

Figure 37. Energy production and number of turbine versus the probability of time



The potential annual energy production is 34.4 GWh per year.

The accuracy of the estimation of the energy production depends mainly from the accuracy of the hydrology.

The optimization of the production must be made in further studies taking into consideration the choice of the design flow, the turbine type and the number of units.

8.2.7 Powerhouse

The hydropower plant will be positioned on the left riverbank. A truck access road should be provided to allow the delivery of the turbine / generator units. A platform will also have to be constructed to allow the maneuvering of long vehicles. Further details are given in section 8.2.9 below.

The power plant floor elevation is chosen so as to ensure that it remains above flood level. The tailrace canal will discharge the turbined outflow to the river downstream of the power station. It will have a length of 10m.

The plant will consist of 3 + 1 bays, one per unit and one bay for assembly / dismantling. One floor is provided for offices, toilets, control room and meeting room. The area under the offices will allow the storage of tools and spare parts. A backup generator will also be placed there. The height of the plant will be governed by the size of the highest of the parts to be handled and by the characteristics of the crane. The dimensions of the plant, estimated at 15m wide, 35m long and 13m high, will have to be refined in subsequent studies.

For safety reasons (fire hazard) the transformers will be positioned in the immediate vicinity of the plant in a separate room.

The characteristics of the plant are given in the following table:

Table 18. Characteristics of the powerhouse

PARAMETER	UNIT	VALUE
Water level in the forebay	m	639.7
Elevation of the power house floor	m	786
Tailwater elevation	m	781
Powerhouse length	m	35
Powerhouse width	m	15
Powerhouse height	m	13
Tailrace canal length	m	10

8.2.8 Transmission line and substation

The mini-grid of Kigoma is currently supplied by a 6.25 MW diesel-fired power station operated by TANESCO. Kigoma power station supplies power to the municipal of Kigoma-Ujiji, the new district of Uvinza and part of the new district of Buhigwe. The construction of the power station started in 2009 and became operational in June 2010. The total installed power is fully available and the maximum load demand recorded is 5.063 MW. Hence, the proposed Luegere hydroelectric project (5.340 MW) is a relevant alternative to the (costly) energy generation by that thermal power station.

The connection of the Luegere hydroelectric project and the mini-grid of Kigoma that extends south through the village of Ilagala on the Malagarasi river would require the construction of an approximately 85km long medium voltage (33kV) transmission line. However, the Power Supply Master Plan (2016) proposes the construction of a 400 kV transmission line between Kigoma and Mpanda at horizon 2020. As a consequence, the required length of the transmission line to evacuate the power generated from the Luegere scheme could be strongly reduced, depending on the feasibility to connect directly to the 400 kV line with a dedicated substation.

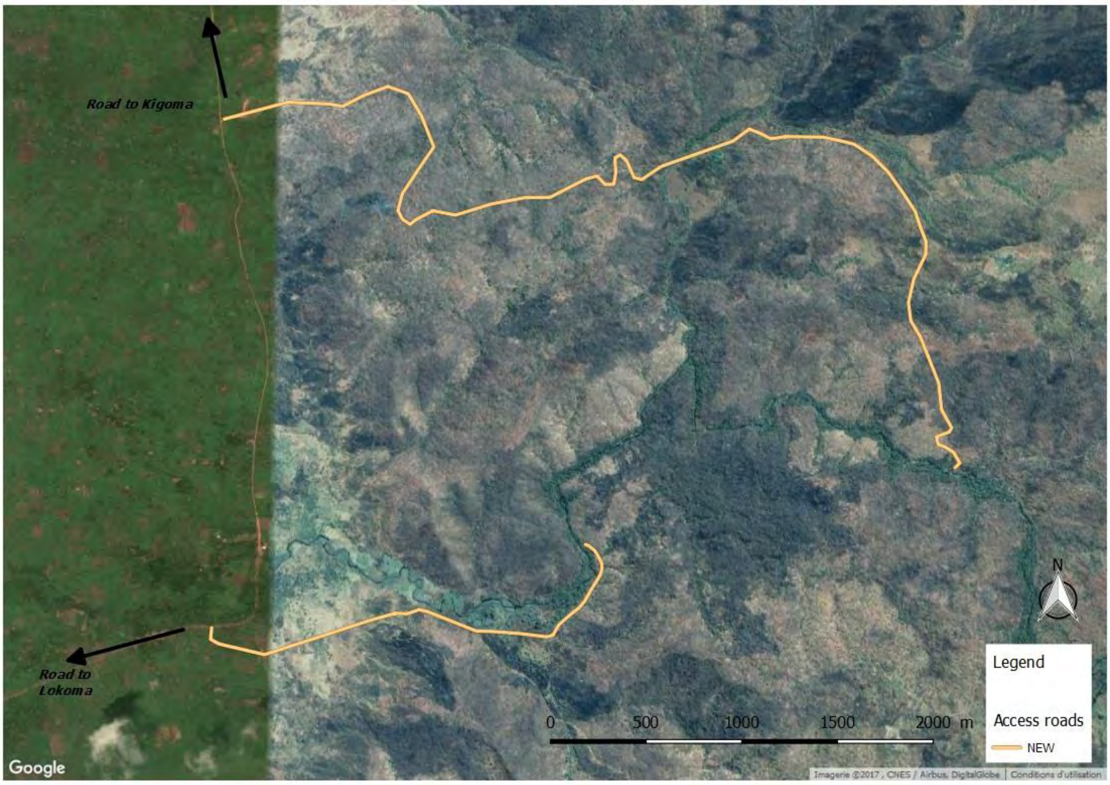
As the surroundings of the proposed project are currently not supplied by the electricity grid, the detailed studies shall analyze the technical and economic feasibility of supplying electricity to neighborhood and villages along the transmission line connecting Kigoma directly from the power plant.

8.2.9 Access

A comprehensive description of existing access is presented and illustrated in Section 3.2 of this report.

For the development of the site, it will be necessary to create 9km of access tracks. Two new tracks are needed. One track will connect, from the north, the existing Lokoma-Kigoma track to the weir, the intake and the desilting structure. A crossing bridge over the weir has to be built to access the intake and desilting basin located on the left riverbank. The other track will connect, from the south, the existing Lokoma-Kigoma track to the powerhouse situated on the left riverbank. These different accesses to create are illustrated in Figure 38 below.

Figure 38. Access to create to access the proposed Luegere hydropower scheme



8.2.10 Temporary infrastructure during the construction period

Temporary infrastructure includes:

- Construction camp.
- Construction works areas (e.g. concrete batching plant, cable crane plant).
- Quarry locations.
- Site access roads

The construction camp is intended to accommodate allochthones workers working on the site. It will consist of accommodations, all the necessary sanitary facilities, a water treatment station and a wastewater treatment plant. This will serve both for the construction camp and for the permanent camp.

8.2.11 Permanent camp

The permanent camp will be located near the power station. It will consist of accommodations for the operators of the power plant as well as for the plant manager. The water treatment plants, constructed for the temporary camp, will also ensure the treatment of the waters of the permanent camp and the power plant.

8.3 KEY PROJECT FEATURES

Table 19 below summarizes the key features of the proposed layout of the Luegere hydroelectric scheme.

Table 19. Key features of the proposed scheme

FEATURE	PARAMETER	VALUE	UNITS
Location	Region	Kigoma	-
	River	Luegere	-
Hydrology	Catchment area	1,317	km²
	Median streamflow (Q50%)	4.56	m³/s
	Firm streamflow (Q95%)	1.41	m³/s
	Design flow	4.33	m³/s
	Design flood (100 years)	220	m³/s
Diverting structure	Structure type	Gravity weir (Overflowing section : Creager)	-
	Material used	Concrete	-
	Overflowing section crest length	50	m
	Total structure length	70	m
	Overflowing section height	4.50	m
	Non-overflowing section height	7.15	m
	Crest elevation	943.00	masl
	Slab elevation	938.50	masl
Gated flushing channel	Number of bays	2.00	pce
	Gate section	1.4 x 1.5	m x m
Intake	Number of bays	2	pce
	Invert elevation	941.00	masl
	Equipment	Trash rack (manual cleaning)	-
Desilting structure	Yes		
	Number of basins	2.00	pce
	Water level	943.00	masl
Waterway Canal	Headrace canal length	1 420	m
	Headrace canal section	2 x 2.3	m x m
	Average slope	0.001	m / m
Forebay	Yes	-	-
	Water level	941.58	masl
Penstock	Number of penstock(s)	1	pce
	Length	1 110	m
	Diameter	1.20	m
Powerhouse and electrical / electromechanical equipment	Floor elevation	786.00	masl
	Gross head	157.00	m
	Number of units	3	pce
	Turbine type	Pelton	-
	Operating discharge per unit	1.44	m³/s
	Total installed capacity	5 340	kW
	Average annual energy generation	34.40	GWh/year
Access road	Length of road to build	9,000	m
	Length of road to rehabilitate	0	m
Transmission lines	Length	85	km
	Voltage	33	kV

Economic data	CAPEX - without access road and transmission lines	13.14	M$
	LCOE - without access road and transmission lines	0.05	$/kWh
	CAPEX - access road and transmission lines included	24.97	M$
	LCOE - access road and transmission lines included	0.10	$/kWh

9 COSTS AND QUANTITIES ESTIMATES

9.1 ASSUMPTIONS

At the prefeasibility study stage of a hydroelectric development, the assumptions detailed in the following paragraphs are commonly accepted.

9.1.1 Unit Costs

The list of unit prices comes from the Consultant's database which includes prices of contractors competent in hydraulic works and which can prove similar works carried out to international standards. This database is based on unit prices valid in Africa for infrastructure projects and updated for Tanzania.

Table 20. Unit prices (2017 USD)

CLASS	DESCRIPTION	UNITS	COST ($)
Excavation	Excavation (rock)	m^3	33.00
	Excavation (diverse)	m^3	17.00
	Excavation (soil)	m^3	6.00
Backfill	Random fill	m^3	9.00
	Compacted earthfill	m^3	13.00
	Rockfill	m^3	55.00
	Sand fill (pipe)	m^3	11.00
Concrete, stone and Masonry	Blinding concrete	m^3	165.00
	Mass concrete	m^3	330.00
	Structural concrete	m^3	550.00
	Concrete for weir	m^3	385.00
	Stone masonry	m^3	127.00
	Stone masonry (weir)	m^3	154.00
	Concrete bloc	m^3	165.00
	Rip-rap	m^3	33.00
Steel	Rebar	Kg	2.00
	Structure	Kg	6.00
	Roof	m^2	17.00
Access road	Access road (new)	m	380.00
	Access road (rehabilitation)	m	101.00
Transmission lines	33 kV Transmission line	km	81 070.00
Miscellaneous	Cofferdam	m^2	110.00
	Finishing (powerhouse)	package	253 110.00
Equipments	Electromechanical equipment	unit	731 500.00
	Penstock	m	1 925.00
	Overhead travelling crane	unit	64 900.00
	Trashrack	unit	14 740.00
	Flush gate (1.4m x 1.5m)	unit	60 500.00
	Intake gate (2m x 2m)	unit	97 900.00
	Drain gate (1m x 1m)	unit	44 770.00
	Desilting isolation gate (2m x 4.5m)	unit	158 180.00
	Isolation gate (2.2m x 2.5m)	unit	110 000.00
	Safety valve	unit	356 400.00
	Electrical equipment	package	653 400.00

9.1.2 Reinforcements and concrete

The reinforcements necessary for the realization of the structural concrete are taken into account in the concrete costs (at 250 kg of steel per m^3). No reinforcement is foreseen in mass concrete (mainly used for the weir).

9.1.3 Hydro and electromechanical equipment costs estimate

The considered equipment are:

- The hydro and electromechanical equipment: turbine, generator, valve, high pressure unit;
- The electrical equipment: power and auxiliary transformers, switchboard, control system and monitoring, power supply, protection system, cables, earthing, cabinets.

Total and unit prices for the main components are indicated in the Project Costs Estimated Table, and are based on recognized cost estimate model (NVE 2016, Electrobras small hydro, Ogayar et al., B. Leyland, in-house model for Pelton turbine). The selection of the appropriate models depends of the type of equipment, rated power, and experience/contract awards for small hydro projects in Africa, and especially in East Africa.

Reference projects in East Africa have been used to adjust the cost estimate.

For different reasons, as for instance, change of prices of raw material (such as steel, copper, etc.) or global small hydro market activity and manufacturing capacities, unexpected deviation from the proposed prices are possible. Nevertheless, cost estimate are taken as up-to-date and reliable enough for the purpose of the present level of the study.

The estimated costs take into account: equipment design and manufacturing, workshop acceptance tests, transport, mobilization, engineering, erection and commissioning; but it does not take into account unforeseen, taxes and duties.

9.1.4 Indirect costs

Indirect costs were estimated using fixed rates applied on different sub-totals of costs, as presented in the table below. Rates applied to Civil Works are higher than rates applied to Electrical and Mechanical Works as more uncertainties remain until the works have started.

Table 21. Indirect costs

Indirect Costs	Applied Rate
Civil works contingencies	20% of civil works costs
Electrical and mechanical works contingencies	10% of E-M costs
Engineering (including ESIA), administration and supervision of works	10% of total costs
Owner’s development costs	2% of total costs

9.1.5 Site facilities costs

Costs for the Contractor site facilities and housing depend on the size of the project. Hence, this cost is taken as 10% of the total civil works costs.

9.1.6 Environmental and Social Impact Assessment Mitigation Costs

At this stage of the study and given the conclusions of the preliminary socio-environmental study, 2% of the total project costs are planned for the Environmental and Social Impact Assessment and mitigation (ESIA costs). This amount shall cover:

- Expropriation costs (compensation or allocation of new land);
- Mitigation cost of environmental impacts.

These costs should be specified in the full Environmental and Social Impact Assessment Study which will be carried out at a later stage of the project development. The costs of this study are taken into account in the indirect engineering costs presented in the previous section (section 9.1.4).

9.2 TOTAL COSTS (CAPEX)

Table 22 below presents a summary of costs for civil works and electromechanical equipment. It also includes indirect costs related to studies, site supervision, project administration and environmental and social mitigation measures.

Table 22. Project costs estimates (2017 US$)

Item	(%)	Costs ($)
Civil Works		16 917 000
Mobilization, installation, demobilization		275 000
Access		2 970 000
Dam/weir, spillway, purge and intake		726 000
Waterway (headrace channel, silting basin, forebay and penstock)		4 537 000
Powerhouse and tailrace channel		1 518 000
Transmission line		6 891 000
Electromechanical equipment		4 244 000
Electromechanical equipment		2 195 000
Hydro mechanical equipment		688 000
Electrical equipment and ancillaries		653 000
Transport	10%	354 000
Installation	10%	354 000
Sub Total (excl. contingencies)		21 161 000
Contingencies		3 809 000
Civil works contingencies	20%	3 384 000
Equipment contingencies	10%	425 000
Total direct project cost (incl. contingencies)		24 970 000
Indirect Costs		3 497 000
Social and environmental mitigation costs	3.0%	500 000
Administration fees	2.0%	500 000
Studies (incl. EIES) and works supervision	10.0%	2 497 000
Total cost of the project		28 467 000

10 ECONOMIC ANALYSIS

10.1 METHODOLOGY

The economic analysis is based on the results of the field investigations and various studies presented in the previous chapters, which includes an estimate of the quantities and the construction costs of the project (Chapter 9) and the definition of the installed capacity and power output. Based on these results, the Consultant has estimated the cost to deliver energy from the development of the Luegere hydroelectric project.

The energy generation alternatives (currently thermal units, fossil fuel-fired) will be compared based on their costs per kWh, the latter being expressed in terms Levelized Cost Of Energy (LCOE) which is a stream of equal payments, normalized over the expected energy production periods that would allow a project owner to recover all costs, an assumed return on investment, over a predetermined life span.

The LCOE is defined from investment costs (CAPEX - Capital Expenditure), operating costs (OPEX - Operational Expenditure) and the expected production of energy.

Investment costs are:

- Study and work supervision costs, hereafter called "Studies and engineering costs" which include:
    - Civil works study and supervision costs
    - Electromechanical works study and supervision costs
    - Owner's development costs
- Civil works and equipment costs, hereafter called "HPP costs"
- Resettlement and environmental impact costs, hereafter called "ESIA costs"

Annual operating costs are:

- Operation and maintenance costs, hereafter called "O&M costs" which include:
    - Fixed operation and maintenance costs (annual scheduled maintenance)
    - Costs related to interim replacement and refurbishments of major items in the course of the project's life
    - Insurance costs

The LCOE is then calculated based on expected production and costs from the following formula:

$$LCOE = \frac{NPV(CAPEX + OPEX)}{NPV(Energy\ production)}$$

Where *NPV* is the Net Present Value which is obtained by: $NPV(value) = \sum_i \frac{valuer_i}{(1+n)^i}$ where *n* is the discount rate.

10.2 Assumptions and Input Data

The main economic assumptions for the economic modeling of the *LCOE* calculation for the Luegere hydroelectric project are presented in Table 23 below.

Table 23. Economic modelling assumptions

Economic lifespan of the project	30 years
Decommissioning cost at the end of the economic life	10% of civils works and equipment costs
Engineering (incl. ESIA) and works supervision **Owner's development costs**	10% of civils works and equipment costs 2% of civils works and equipment costs
Environmental and social impact mitigation costs	3% of civils works and equipment costs
O&M costs Interim replacement Fixed operation costs	 0,25%/year of civils works and equipment costs 10 USD/kW/year
Insurance costs	0,10% of civils works and equipment costs per year
Distribution of costs over the project implementation process	Year -2 = 60% Year -1 = 40% Year 0 = Commissioning
Reference date for economic analysis	2017
Costs are expressed in constants	(2017) USD
Escalation costs (inflation)	No escalation costs were applied to capital costs or operating costs.
Financing costs etc.	Financing costs, tax, duties or other Government levees are ignored at this stage but shall be included in the financial analysis that will be done during the detailed studies.
Discount rate	10%

The economic analysis is carried out by considering that all the energy produced is absorbed by the electricity grid. In other words, the analysis assumes that there is a demand for all the energy generated by the proposed hydroelectric scheme.

10.3 Economic Analysis and Conclusions

Table 24 presents the levelized costs of energy (LCOE) for the Luegere site.

Table 24. Levelized Cost of Energy (LCOE)

	Annual Energy [GWh]	Installed Capacity [MW]	Design Flow [m³/s]	CAPEX [M USD]	LCOE [USD / kWh]
Without Transmission lines and access roads to be rehabilitated	34.40	5.34	4.33	13.14	0.05
With Transmission lines and access roads to be rehabilitated				24.97	0.10

The economic analysis reveals that the proposed Luegere hydroelectric scheme is an economically attractive project with a LCOE of 0.05 $US/kWh (excluding the costs of transmission lines and access roads). Indeed, the LCOE calculated including the cost of transmission lines and access roads will be significantly reduced in the near future with the construction of the 400 kV transmission line between Kigoma and Mpanda at horizon 2020, as proposed in the Power Supply Master Plan (2016).

The mini-grid of Kigoma is currently supplied by a 6.25 MW diesel-fired power station operated by TANESCO. Hence, the LCOE of the Luegere project must be compared with the cost of energy production by the thermal power plants currently in operation since the development of the Luegere hydroelectric project would replace the production of thermal energy by hydroelectricity. The cost of generating thermal power plants depends largely on the fuel costs. As outlined in the SREP-Investment Plan for Tanzania fuel cost from diesel-fired thermal power plant is expected to exceed 0.35 US$/kWh[5].

The LCOE of the proposed Luegere hydroelectric project is attractive when compared to the 0.095 US$/kWh corresponding to the standardized small power projects (SPPs) tariff for hydro between 5MW and 6MW in 2016. The latter is the tariff for SPPs selling bulk power to the national or a regional grid or to DNO-Owned Mini-Grids.

It is important to note that the conclusions of this economic analysis are conditioned to the validation of the flow duration curve estimated in the hydrological study. This validation can only be done by the hydrological monitoring of the Muyovozi River. The hydrological monitoring should include not only the continuous recordings of water levels but also gauging operations of the river for the establishment of validated rating curves.

[5] Source : SREP - Investment plan for Tanzania

11 CONCLUSIONS AND RECOMMANDATIONS

The hydrological study revealed that the Luegere River is characterized by a good guaranteed low-flow that should be confirmed by hydrological monitoring of the River.

The preliminary investigation of the surface geology concludes that from a geological point of view the site is favorable for the construction of the project as long as the appropriate mitigation measures are put in place. The site has no major problems of stability and leakages. Further studies will however have to be undertaken in further studies.

Preliminary socio-environmental studies show that the development of the Luegere project has no major impacts that cannot be mitigated by appropriate measures.

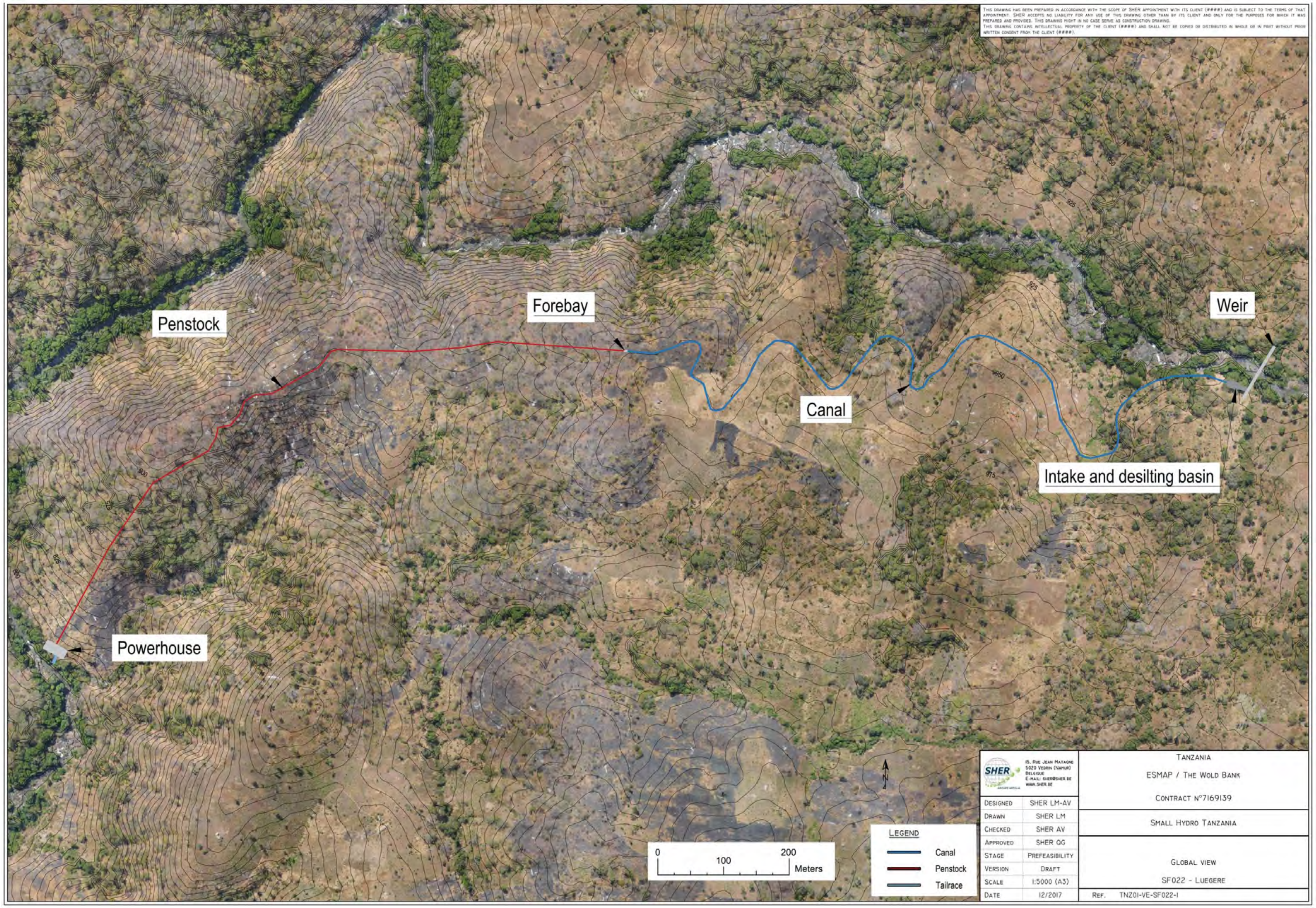
The economic analysis reveals that the construction costs of the 33kV transmission line to the Kigoma mini-grid are high. However, those costs will be significantly reduced in the near future with the construction of the 400kV transmission line between Kigoma and Mpanda at horizon 2020, as proposed in the Power Supply Master Plan (2016). The Luegere hydroelectric project is an economically attractive scheme with a LCOE of 0.05 US$/kWh if the costs of the transmission line costs and access roads are excluded. The Luegere Project features a production costs significantly lower than the standardized small power projects (SPPs) tariff for hydro between 5MW and 6MW, as approved by EWURA in 2016 (0.095 US$/kWh).

It is important to note that the conclusions of this economic analysis are conditioned to the validation of the flow duration curve estimated in the hydrological study. This validation can only be achieved by hydrological monitoring of the Luegere River at the hydrometric station a few kilometers downstream from the proposed project site. This hydrological monitoring should include not only the continuous water level monitoring but also the gauging operations of the river for the establishment of a validated rating curve.

Beyond the development of the Luegere hydroelectric project, it is strongly recommended that the Government of Tanzania further develop the existing hydrological monitoring network for its rivers with high hydropower potential in order to better understand the available water resources and thus promote the development of hydroelectric projects across the country. It is only in a context of reduced uncertainties through reliable, recent and long-term records (more than 20 years) that technical parameters and economic and financial analyzes of hydroelectric developments can be defined accurately, enabling optimization of their design and their flood control infrastructure (temporary and permanent).

12 APPENDICES

12.1 DETAILED PROPOSED SCHEME AND MAIN COMPONENTS



Phase 2 - Ground Based Data Collection

PREFEASIBILITY STUDY OF THE MUYOVOZI HYDROELECTRIC SCHEME

Renewable Energy Resource Mapping: Small Hydro - Tanzania

December 2017

ABREVIATIONS AND ACRONYMS

ASTER GDEM	Advanced Spaceborne Thermal Emission and Reflection Radiometer Global Digital Elevation Model
CHIRPS	Climate Hazards Group InfradRed Precipitation database
DSM	Digital Surface Model
ESIA	Environmental and Social Impact Assessment
ESMAP	Energy Sector Management Assistance Program
EWURA	Energy and Water Utilities Regulatory Authority
FAO	Food and Agricultural Organization
GIS	Geographic Information System
GoT	Government of Tanzania
GSHAP	Global Seismic Hazard Assessment
GW	Gigawatt
GWh	Gigawatt hour
IFC	International Finance Corporation
IPP	Independent Power Producers
kW	Kilowatt
kWh	Kilowatt hour
MW	Megawatt
MWh	Megawatt hour
NASA	United States National Aeronautics and Space Administration
OP	Operational Polices
REA	Rural Energy Agency
RE	Renewable Energy
SRTM	Shuttle Radar Topography Mission
TANESCO	Tanzania Electric Supply Company
USACE	United States Army Corps of Engineers
WES	Waterways Experimental Station

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1 EXECUTIVE SUMMARY

The key features of the Muyovozi hydroelectric scheme are summarized in Table 1 below.

Table 1. Key features of the proposed hydroelectric scheme

FEATURE	PARAMETER	VALUE	UNITS
Location	Region	Kigoma	-
	River	Muyovozi	-
Hydrology	Watershed area	2 720.00	km²
	Median streamflow (Q50%)	12.04	m³/s
	Firm streamflow (Q95%)	2.12	m³/s
	Design flow	11.44	m³/s
	Design flood (100 years)	624	m³/s
Diverting structure	Structure type	Gravity weir (Overflowing section : Trapezoidal)	-
	Material used	Concrete	-
	Overflowing section crest length	40	m
	Total structure length	85	m
	Overflowing section height	3.00	m
	Non-overflowing section height	8.80	m
	Crest elevation	1 192.00	masl
	Slab elevation	1 189.00	masl
Gated flushing channel	Invert elevation	1 189.00	masl
	Number of bays	2.00	pce
	Gate section	1.6 x 2	m x m
Intake	Number of bays	4	pce
	Invert elevation	1 189.50	masl
	Equipment	Trash rack (manual cleaning)	-
Desilting structure	Yes	-	-
	Number of basins	3.00	pce
	Water level	1 192.00	masl
Waterway		-	-
Canal	Headrace canal length	1 147	m
	Headrace canal section	2.8 x 3.2	m x m
	Average slope	0.001	m / m
Forebay	Yes	-	-
	Water level	1 190.85	masl
Penstock	Number of penstock(s)	1	pce
	Length	226	m
	Diameter	1.70	m
Powerhouse and electrical / electromechanical equipment	Floor elevation	1 165.00	masl
	Gross head	27.00	m
	Number of units	2	pce
	Turbine type	Kaplan	-
	Operating discharge per unit	5.72	m³/s
	Total installed capacity	2 270	kW

	Average annual energy generation	15.00	GWh/year
Access road	Length of road to build	2 200	m
	Length of road to rehabilitate	3 700	m
Transmission lines	Length	60	km
	Voltage	33	kV
Economic data	CAPEX - without access road and transmission lines	9.79	M$
	LCOE - without access road and transmission lines	0.09	$/kWh
	CAPEX - access road and transmission lines included	17.38	M$
	LCOE - access road and transmission lines included	0.16	$/kWh

2 INTRODUCTION

2.1 Overview of the ESMAP Program

ESMAP (Energy Sector Management Assistance Program) is a technical assistance program managed by the World Bank and supported by 11 bilateral donors. ESMAP launched in January 2013 an initiative to support the efforts of countries to improve the knowledge of renewable energy (RE) resources, establish appropriate institutional framework for the development of RE and provide "free access" to geospatial resources and data. This initiative will also support the IRENA-GlobalAtlas program by improving data availability and quality, consulted through an interactive atlas.

This "Renewable Energy Mapping: Small Hydro Tanzania" study, is part of a technical assistance project, ESMAP funded, being implemented by Africa Energy Practice 1 (AFTG1) of the World Bank in Tanzania (the 'Client') which aims at supporting resource mapping and geospatial planning for small hydro. It is being undertaken in close coordination with the Rural Energy Agency (REA) of Tanzania, the World Bank's primary Client country counterpart for this study.

The "Provision of Small Hydropower Resource Data and Mapping Services" IDA 8004801 Framework contract was signed 29th May 2013, while the specific contract " Renewable Energy Mapping: Small Hydro Tanzania", n. 7169139, is dated 4th November 2013.

2.2 Objectives and Phasing of the Study

The objectives of the study are:

- To improve the quality and availability of information on Tanzania's small hydropower resources. The project will provide the GoT (Client) and commercial developers with ground-validated maps (at least 70+ sites up to 10 MW) that show the varying levels of hydro potential throughout the country, and highlight several sites most suited for small hydropower projects.
- To contribute to a detailed comprehensive assessment and to a geospatial planning framework of small-hydro resources in Tanzania; (ii) to verify the potential for the most promising sites and prioritized sites (~ 20 prioritized sites) to facilitate new small hydropower projects and ideally to guide private investments into the sector; and (iii) to increase the awareness and knowledge of the Client on RE potential.

The study is delivered in three phases:

PHASE 1: Preliminary resource mapping based on satellite and site visits.

PHASE 2: Ground-based data collection.

PHASE 3: Production of validated resource atlas that combines satellite and ground-based data.

2.3 Context and Scope of the Prefeasibility Study

This report is delivered in the context of PHASE 2 (Ground-based data collection). In accordance with our Terms of References (Revised Terms of References for the Phase 2 and 3 of the Project, 30 June 2016), the prefeasibility study covers the following aspects:

- Review of the existing data and GIS information ;

- Additional site visit to the sites and main load centers / national grid connection by relevant sector experts ;
- Additional topographic and geotechnical surveys, update of the hydrology, and assessments of environmental and social impact to reach study results at pre-feasibility level;
- Preparation of a conceptual design and drawings at pre-feasibility level; Schematic Layout of Hydro Powerhouse, weir or dam (when applicable), waterways and Transmission Lines to the main load centers / national grid connection;
- Preparation of a Budgetary Cost Estimate, including costs for environmental and social costs, and Electricity Generation Estimate for a range of installed capacities;
- Preliminary economic analysis.

3 CONTEXT OF THE MUYOVOZI HYDROELECTRIC SCHEME

3.1 PROJECT AREA

The Muyovosi project is located on the Muyovosi River approximately 7 km upstream of the confluence with the Muhwazi River. The geographical coordinates (WGS1984) of the proposed weir location are 30.994°East and 3.196°South.

At the proposed intake weir location, the watershed of the Muyovosi River drains an area of 2,720 km². Figure 1 presents the exact location of the proposed site in Tanzania. The administrative and location data are detailed in Table 2 below.

Table 2. Administrative data

Item	Value
Atlas code	SF187
Site name	Muyovozi
River	Muyovozi
Major river basin	Malagarasi and Lake Tanganyika
Region	Kigoma
District	Kibondo
Division	Kakonko / Muhange
Village	Njomulole / Kanyoni
Reference topographic map	Topographic map n° 43/2 and n° 44/1 (scale 1/50,000)

3.2 SITE ACCESS

Access to the site is easy by taking the B8 asphalted road (Nyakanazi – Kibondo stretch, Figure 2) and then taking a good 3km long dirt road. From there, the proposed powerhouse position is accessed from a 1km long track and the surroundings of the proposed weir with a 2.5km long footpath.

Figure 1. Study area

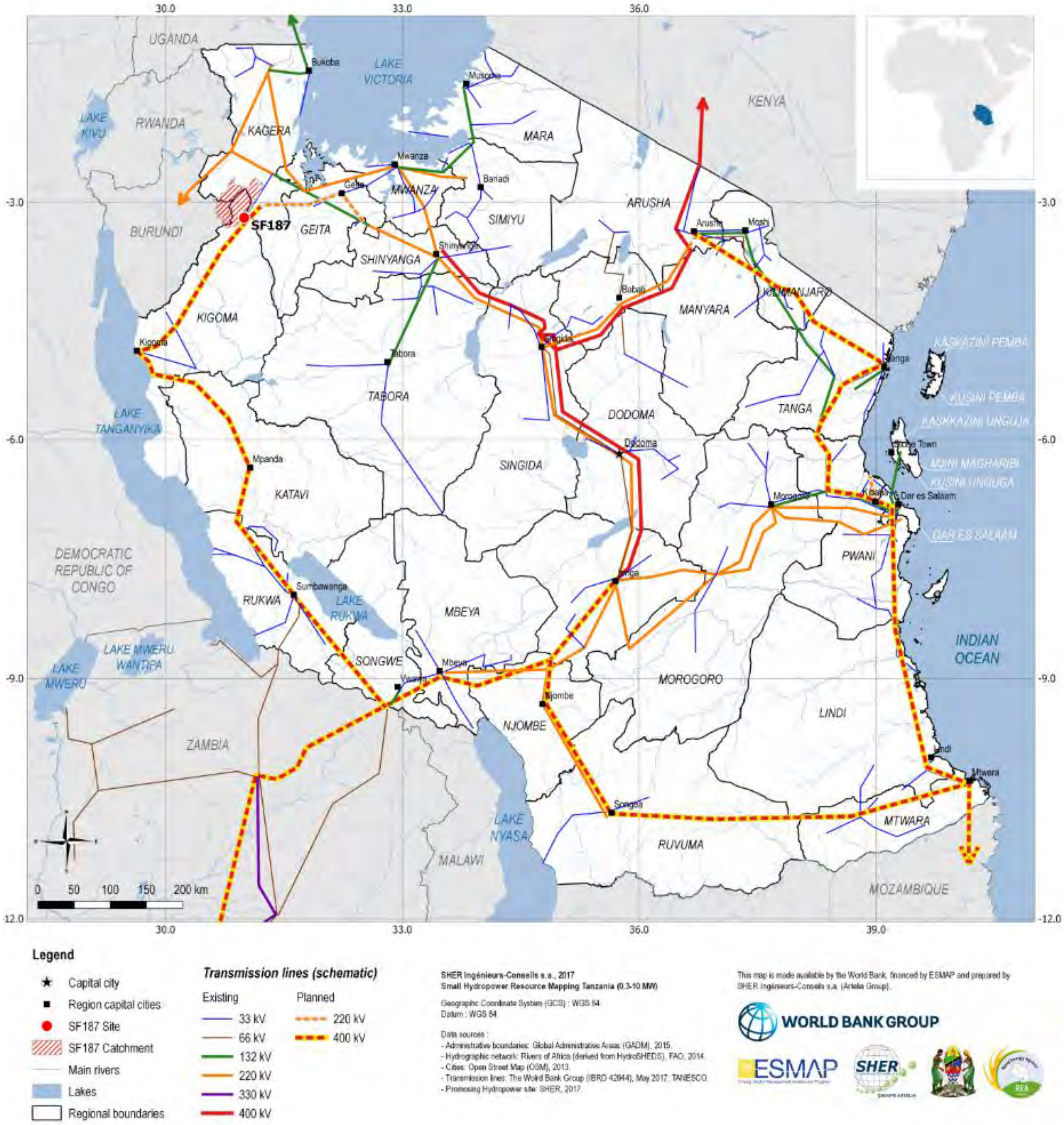


Figure 2. Access to the site

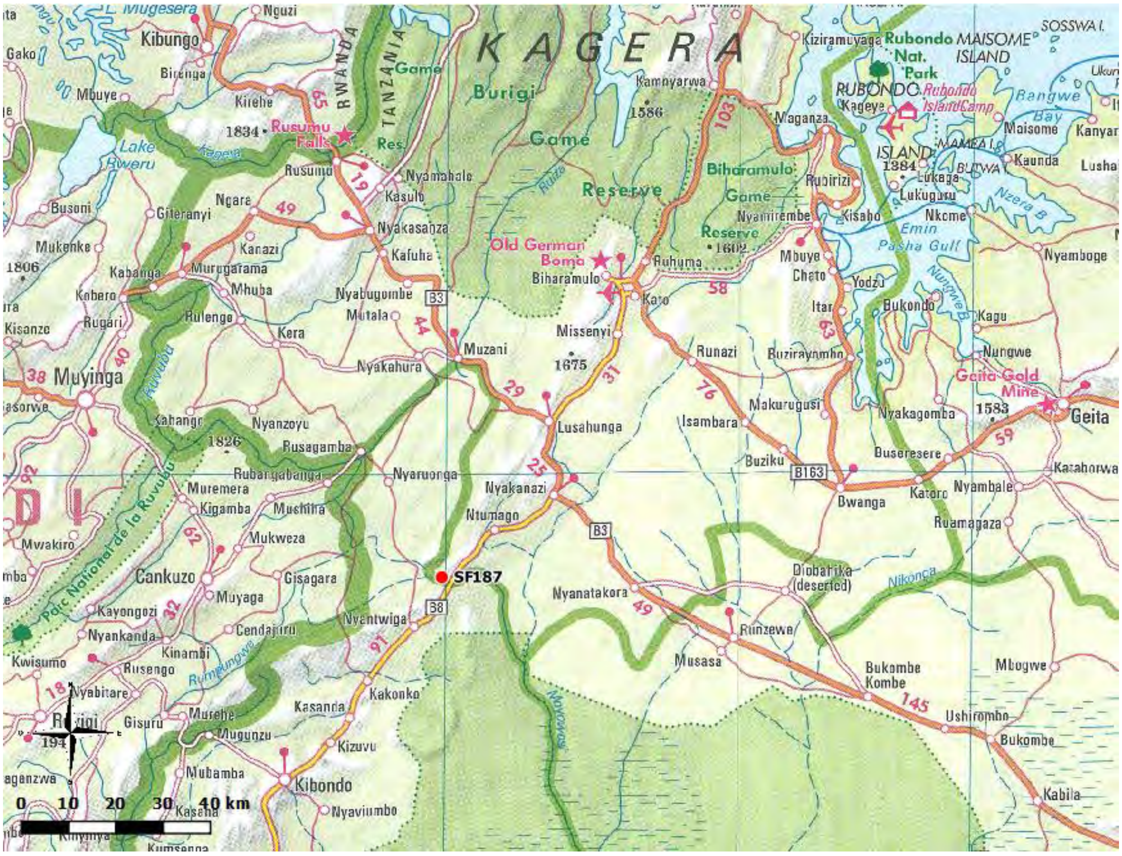
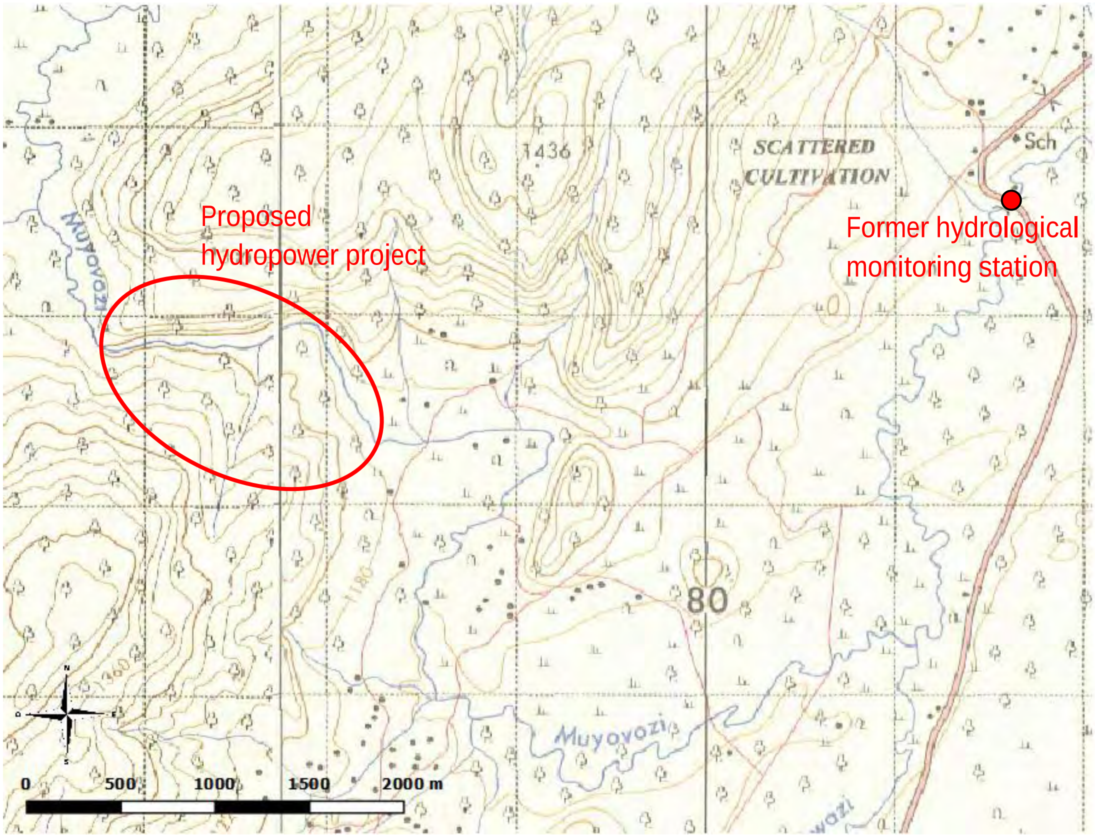


Figure 3. Access to the site (topographic map 1:50,000)



3.3 GENERAL SITE DESCRIPTION

The site is located in a wooded valley where the river follows a long gentle slope (Figure 4). The slopes of valley on the left bank are steeper than on the right bank (Figure 6). Some agricultural activities take place on both banks. Part of the river catchment lies in Burundi.

Figure 4. Overview of the proposed site (Landsat image, Google Earth)

Figure 5. Overview of the river

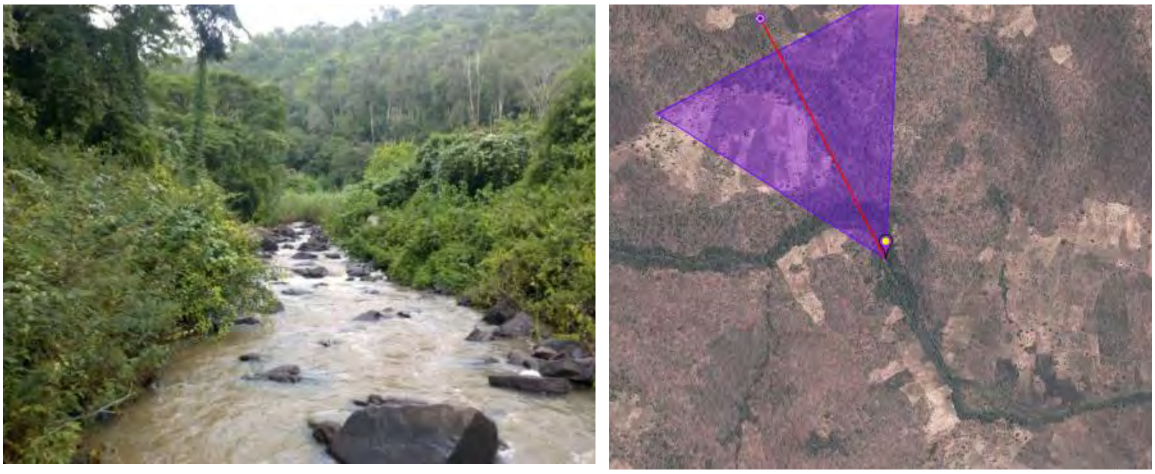


Figure 6. Steep slopes on the left bank valley

Figure 7. Downstream valley

3.4 PREVIOUS STUDIES

To the best of our knowledge, there are no previous studies of the proposed site.

4 TOPOGRAPHY AND MAPPING

4.1 EXISTING MAPPING

4.1.1 Topographic Mapping

The JPEG format (not georeferenced) 1:50,000 scale topographic maps have been acquired from the Survey and Mapping Department of the Ministry of Land in order to cover the entire study area. The JPEG format (not georeferenced) 1:100,000 scale topographic maps have been also obtained from the Ministry of Land. The 1:50,000 scale map of interest is the sheets 43/2 and 44/1. The contour lines interval is 20m. All the topographic maps have been georeferenced as described in section 4.2.

4.1.2 Thematic Mapping

Thematic maps and their key features, sources and format are presented in Table 3 below.

Table 3. Collected thematic maps

THEMATIC	FORMAT	KEY FEATURES	SOURCES
Administrative boundaries	Vector	Country / Regions / Districts / Divisions	FAO Global Country Boundaries, 2012 REA, 2014
Major cities	Vector	32 cities	Open Street Map, 2014
Topographical maps	Raster	1:250,000 (64 tiles) Full country coverage	Ministry of Land, Survey and Mapping Department
	Raster	1:50,000 (1,333 tiles) Full country coverage	Ministry of Land, Survey and Mapping Department
Digital Elevation Model	Raster	SRTM v4.1 Spatial resolution ~ 90m	NASA, 2014 http://www2.jpl.nasa.gov/srtm/
	Raster	ASTER GDEM v2 Spatial resolution ~ 30m (experimental)	http://www.jspacesystems.or.jp/en_/
Land cover	Vector		
Protected areas	Vector	Protected areas, National Parks and Game reserves	Tourist Board ; Tanzania Conservation Resource Centre ; Ministry of Land ; World Database on Protected Areas ; Protected Planet, 2014
Soil map	Raster	IPCC default soil classes derived from the Harmonized World Soil Data Base (v1.1)	ISRIC-WISE http://www.isric.org
Mining activities	Vector	-	Ministry of Energy and Minerals ; World Bank AICD database ; SHER
Satellite image	Raster	Image Landsat 2013	Google Earth
Population	Shapefile	Census data at village and region levels	National Bureau of Statistics ; Ministry of Finance, REA
Lakes	Vector	Inland water bodies in Africa	FAO, 2000 http://www.fao.org/geonetwork
River network	Vector	River "flow accumulation" network from the HYDRO1k for Africa	FAO, 2006 http://www.fao.org/geonetwork
Flow gauging stations	Vector	Location of the YYY gauging	

THEMATIC	FORMAT	KEY FEATURES	SOURCES
Rainfall	Raster	Monthly average rainfall grid Spatial resolution ~ 1km	WorldClim, v1.4 http://www.worldclim.org/
Road network	Vector	National, regional and other roads of Tanzania	World Bank AICD database
Rail network	Vector	Main rail network	World Bank AICD database
Ports	Vector	Major ports	World Bank AICD database
Airports	Vector	Major airports	World Bank AICD database
Power grid	Vector	Existing power grid	IED, 2013 ; REA
Power grid	Vector	Planned expansion of the transmission network	World Bank AICD database
Existing thermal power plants	Vector	34 thermal power plants amongst which 10 connected to the National Power Grid	IED, 2013 ; Power System Master Plan, 2013
Existing hydropower plants	Vector	46 hydropower plants	REA, TANESCO, Ministry of Energy, Diocese, Power System Master Plan, 2013

4.1.3 Digital Surface Model

The digital surface model (DSM) used in the hydrological study is based on the "Shuttle Radar Topography Mission" (SRTM, version 1 arc-second). These data were acquired in February 2000 by the United States Space Agency (NASA) through radar measurements from space shuttle Endeavor. These data have a spatial resolution of 1 arc-second (about 30 m at the equator). The DSM of the study area is illustrated in Figure 11 of the chapter describing the Hydrological Study.

4.2 MAPPING CARRIED OUT AS PART OF THE STUDY

4.2.1 Digitization and geo-referencing

The 1:50,000 scale topographic maps were geo-referenced using the Quantum GIS software and the following projection parameters:

- Projection Transverse Mercator UTM zone 36S
- Latitude of origin = 0
- Central meridian = 33
- Scale factor = 0.9996
- False Easting = 500,000
- False Northing = 10,000,000
- Datum WGS 1984

4.2.2 Additional surveying

4.2.2.1 Digital surface model

The topographic survey was carried out by remote sensing. An eBee Plus drone equipped with a specific camera designed for photogrammetric mapping was used (Figure 8).

Figure 8. eBee Plus drone equipped with a camera for the topographical survey

Outputs from drone survey are (1) a high-resolution orthophotography (0.10m resolution) and (2) a Digital Surface Model (DSM). The DSM includes the vegetation cover, but it gives an excellent overview of the topographical features of the site of interest. Contour lines are calculated from the DSM. The ortho-photography as well as contour lines deduced from the digital surface model are presented at Figure 9 and Figure 10.

Elevations resulting from this topographic survey are relative to each other and have not been linked to the national system. Consequently, the elevations of the works mentioned in this report are not the absolute altitudes of the Tanzanian national system.

4.2.2.2 Digital terrain model

The digital surface models was then post-processed to eliminate the effects of vegetation and hence represent the natural terrain elevation. This has been be achieved by identifying pixels at the natural terrain level (excluding vegetation and other anthropogenic elements) and performing a spatial interpolation of these points in order to obtain a digital terrain model (DTM). At this prefeasibility stage, only the weir/intake and tailwater areas were post-processed to obtain the DTM.

Figure 9. Digital Surface Model (DSM) and orthophotography from drone survey for SF187 site

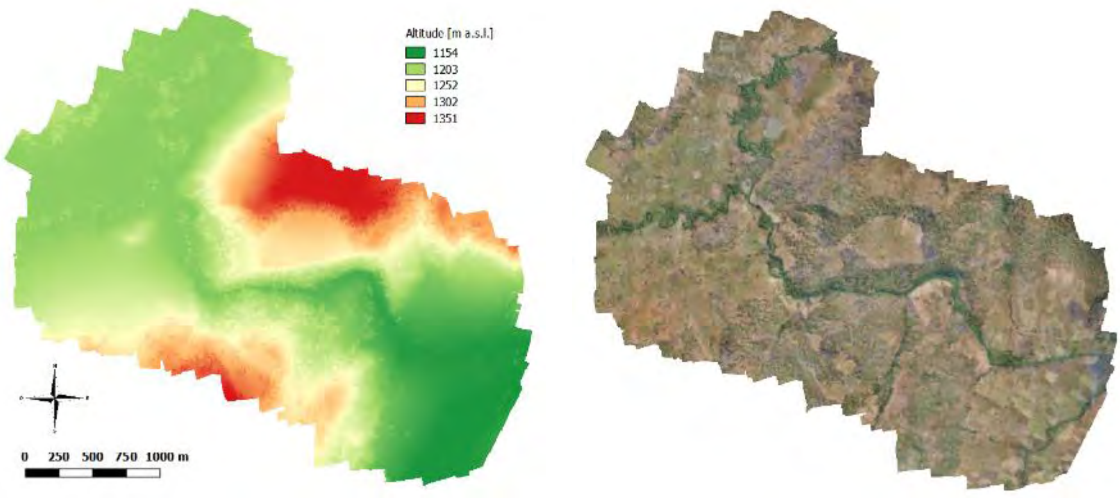
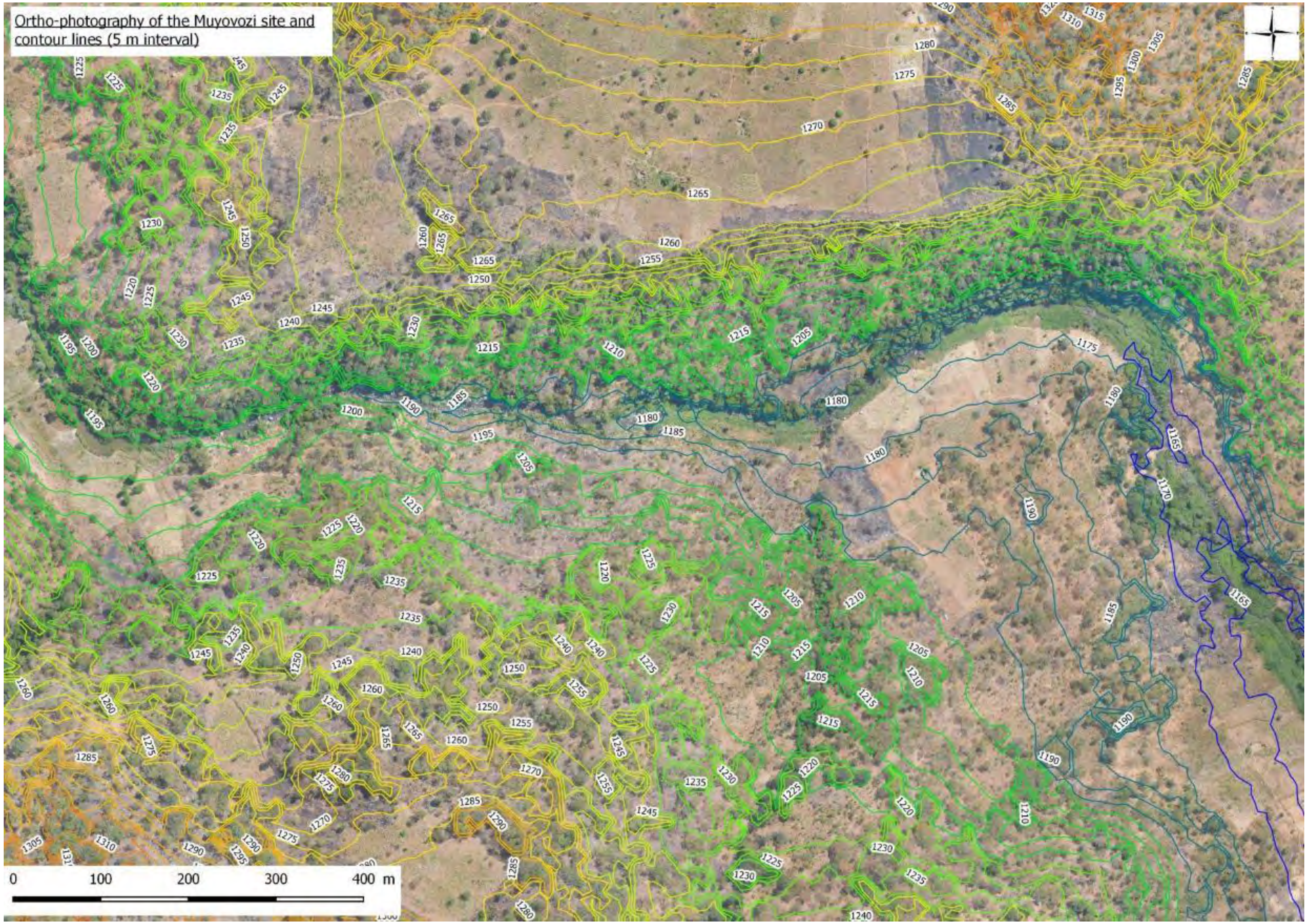


Figure 10. Ortho-photography of the Muyovozi site and contour lines (5 m interval)



5 HYDROLOGICAL STUDY

5.1 Objectives and Limits

The objective of the hydrological study is to establish and quantify the climatological and hydrological characteristics of the study area in order to determine the hydrological parameters and time series required for the design of the Muyovozi hydroelectric project as well as for the economic analysis of the pre-feasibility study.

5.2 Description of the Study Area

5.2.1 Physical Context

The Mwiruzi and Ruhuwiti Rivers merge approximately 1km upstream of the proposed hydroelectric project to create the Muyovozi River. These rivers originate in the Burundian mountainous border at elevations over 1,900m. The Muyovozi River flows mainly from the North to the East and joins the Nikonga and Kigosi Rivers further downstream in a large swamp. The Muyowozi River is part of the Malagarasi River watershed that discharges into the Lake Tanganyika.

As shown in Figure 11, the Muyovozi catchment at the proposed hydroelectric project site features a marked relief with elevations between 1,208m and 1,920m (1,428m on average). The drainage basin of the Muyovozi River at the proposed intake site is 2,720 km² (delimitation based on the SRTM DSM of spatial resolution 1 arc-second, i.e. approximately 30 m). The main physical and morphological features of the river catchment are presented in Table 4 below.

The hypsometric curve of the river catchment is shown in Figure 12. This curve shows the percentage of the catchment area above a given elevation. It shows that slopes are important in the upstream part of the catchment and that 70% of the catchment flows on a plateau characterized by a gentle slope. This is clearly observed in Figure 11 and Figure 12.

Table 4. Physical and morphological characteristics of the catchment

Parameter	Value	Unit
Area	2,720	km²
Average elevation	1,428	m a.s.l.
Maximum elevation	1,920	m a.s.l.
Maximum elevation (percentile 5%)	1,664	m a.s.l.
Minimum elevation	1,208	m a.s.l.
Minimum elevation (percentile 95%)	1,265	m a.s.l.
Slope index	4.0	m/km
Elevation range	399	m
Perimeter	437.5	km
Gravelius index	2.35	-

Figure 11. Muyovozi River catchment and Digital Surface Model

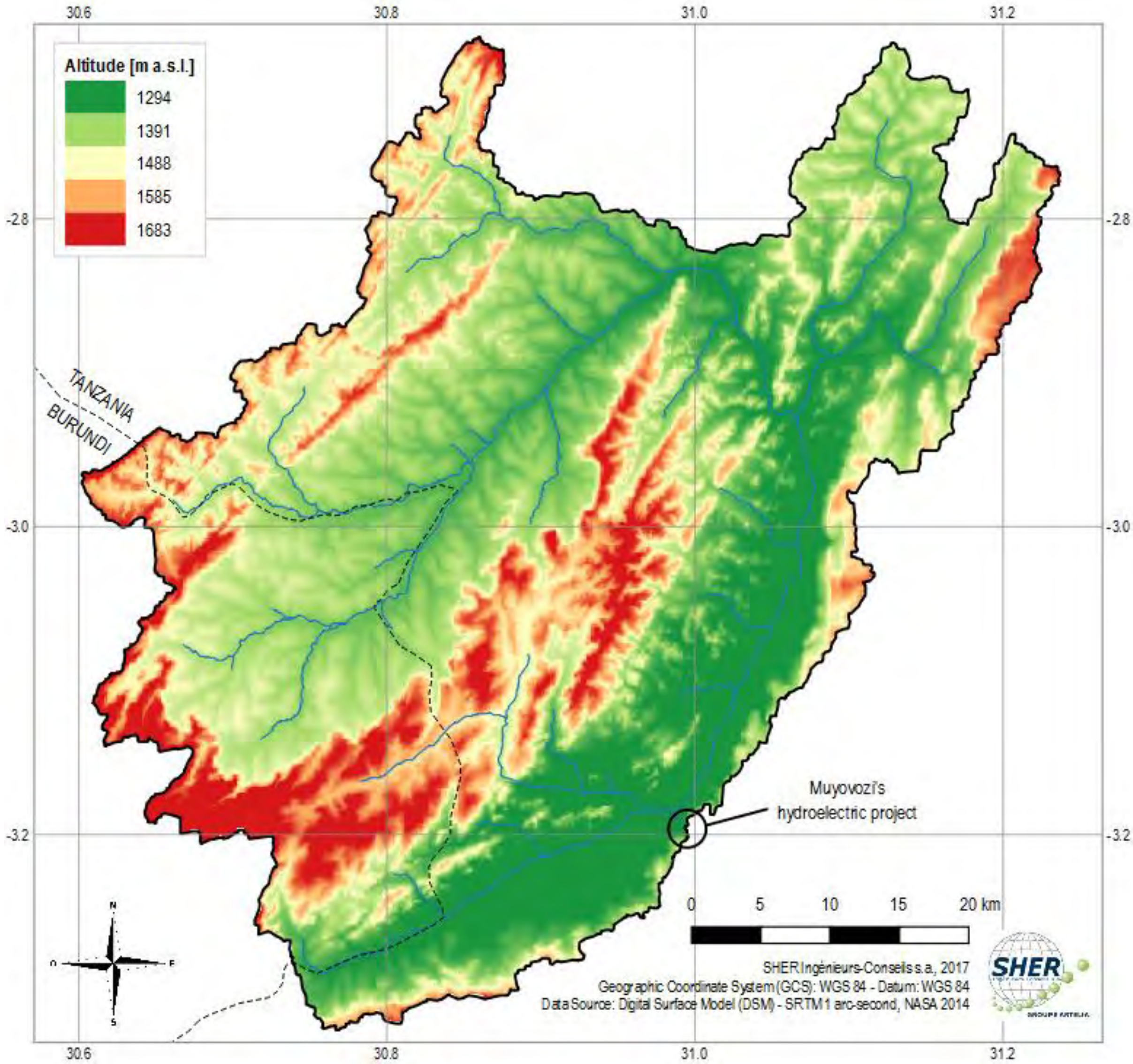
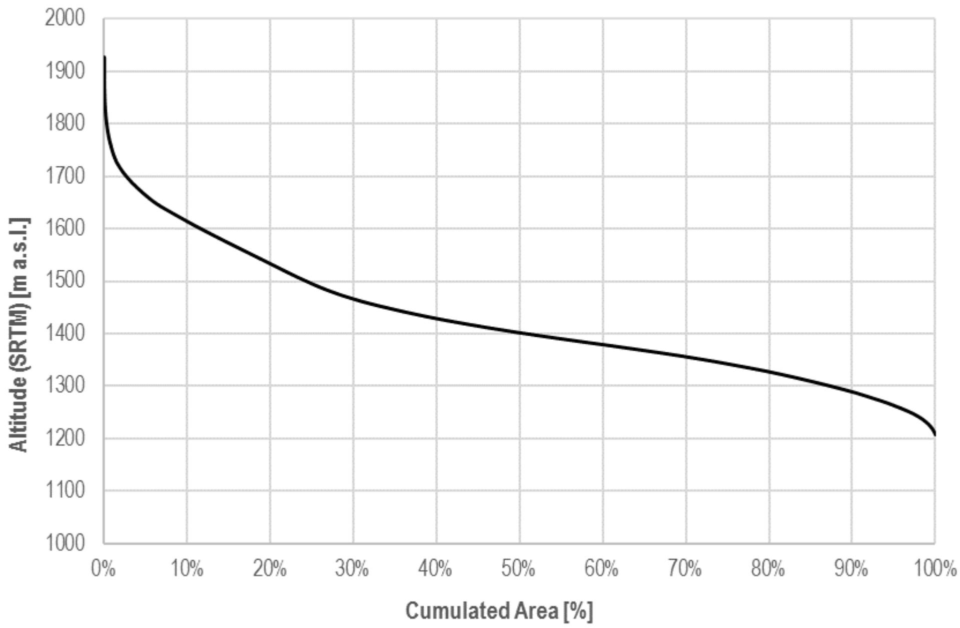


Figure 12. Hypsometric curve of the Muyovozi River catchment



5.2.2 Land cover

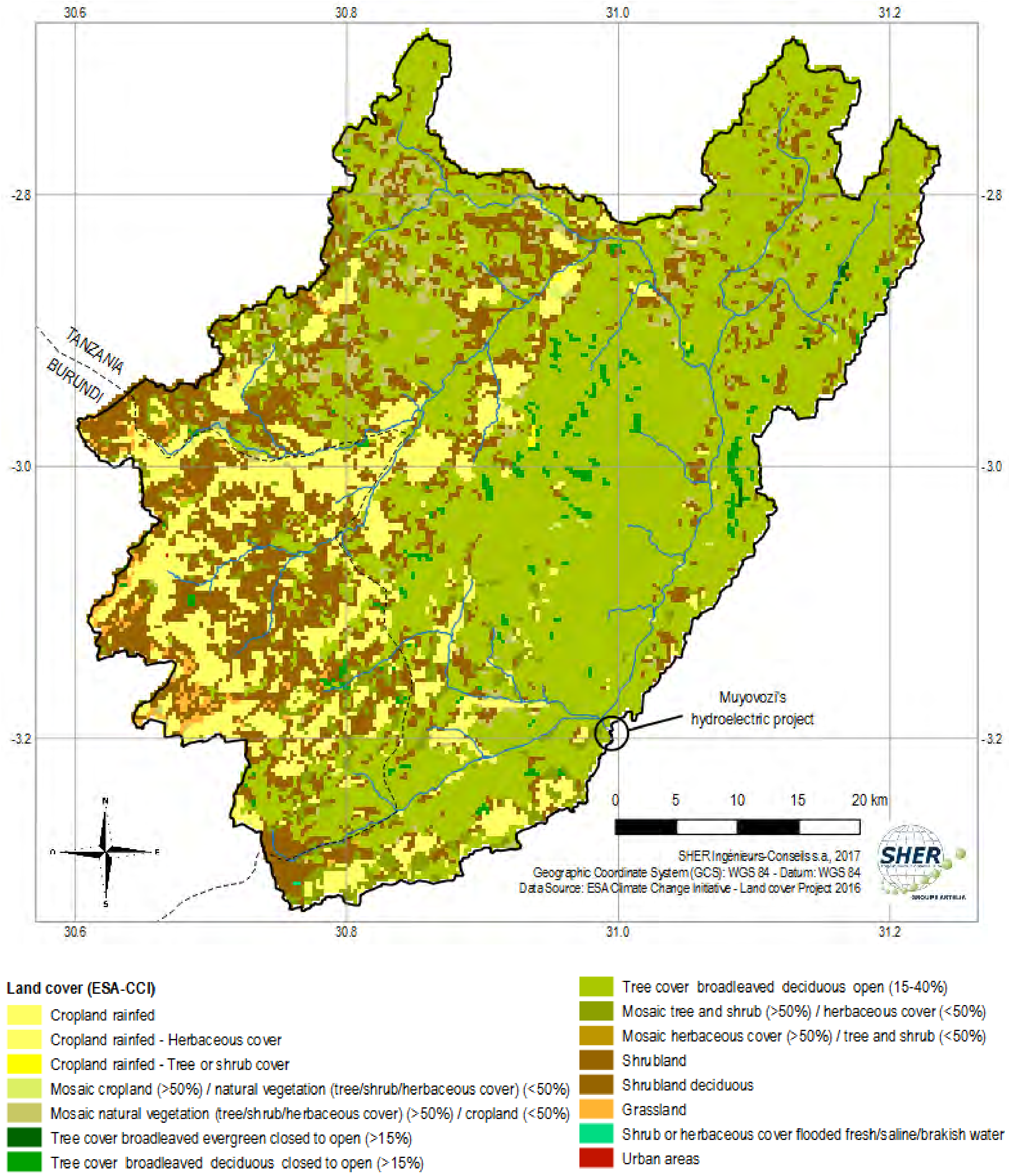
Data from the CCI Land Cover project (© ESA Climate Change Initiative - Land Cover project 2016) is a widely accepted source of information for land use around the world. These data are derived from satellite images acquired by the MERIS instrument of the European Space Agency. The land cover includes 5 years of satellite imagery acquisition between 2008 and 2012. The information is provided in raster format with a spatial resolution of 300m and allows defining the land use classes shown in Figure 13.

Figure 14 and Table 5 show that the Muyovozi catchment is characterized by a very abundant vegetation cover composed mainly of a forest of deciduous open (54.1% of the catchment area, i.e. 1454 km²), shrubland (20.5%, i.e. 550 km²) and cropland rainfed (14.4%, i.e. 388 km²).

Table 5. Land cover in the Muyovozi River catchment

Code	Legend	Area	
		[%]	[km²]
10	Cropland rainfed	1.6%	43.34
11	Cropland rainfed - Herbaceous cover	12.8%	343.40
12	Cropland rainfed - Tree or shrub cover	0.1%	1.42
30	Mosaic cropland (>50%) / natural vegetation (tree/shrub/herbaceous cover) (<50%)	0.3%	6.83
40	Mosaic natural vegetation (tree/shrub/herbaceous cover) (>50%) / cropland (<50%)	2.7%	73.04
50	Tree cover broadleaved evergreen closed to open (>15%)	0.1%	2.56
60	Tree cover broadleaved deciduous closed to open (>15%)	1.4%	37.18
62	Tree cover broadleaved deciduous open (15-40%)	54.1%	1454.00
100	Mosaic tree and shrub (>50%) / herbaceous cover (<50%)	5.0%	133.90
110	Mosaic herbaceous cover (>50%) / tree and shrub (<50%)	0.1%	1.90
120	Shrubland	20.5%	549.80
122	Shrubland deciduous	0.6%	16.32
130	Grassland	0.8%	22.48
180	Shrub or herbaceous cover flooded fresh/saline/brakish water	0.0%	0.19
190	Urban areas	0.0%	0.09
	TOTAL	100%	2686

Figure 13. Land cover in the Muyovozi River catchment



5.2.3 Climate

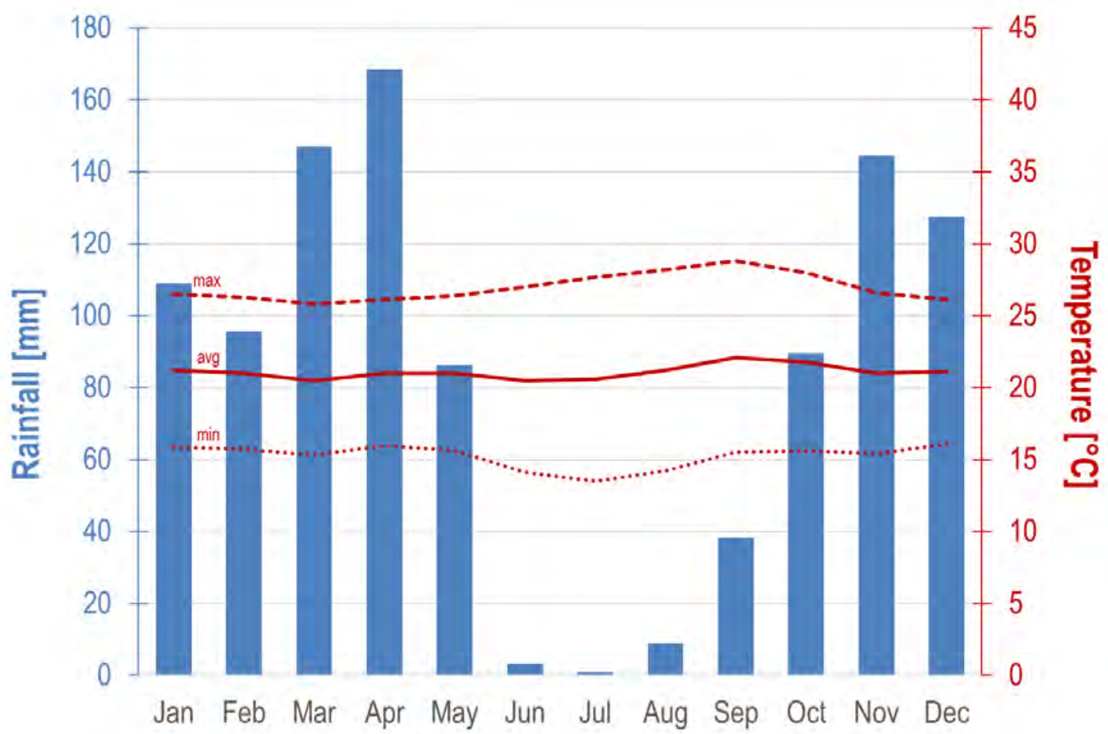
According to the Köppen classification based on rainfall and temperature, the study area (Muyovozi River catchment) is characterized by a tropical savanna climate with a pronounced dry season and constant high temperatures (*Aw* class). Köppen defines the temperate climate «*A*» by the following characteristics:

- Average temperature of each month of the year > 18 °C ;
- High annual precipitation (greater than annual evaporation) ;
- No winter season.

The rainfall regime « *w* » (dry season in winter) is defined by a savanna climate with a precipitation of the driest winter month < 60 mm and < [100 – (mean annual precipitation) / 25].

Figure 14 shows the climatic diagram as well as the temperature curve for the Muyovozi River watershed. Precipitations are very low during the dry season (June to September) but significant during the wet season. July is the driest month with 1 mm of precipitation (on average) whereas the wettest month is April with 169 mm on average. The average annual precipitation is 1,019 mm.

Figure 14. Climatic diagram of the Muyovozi River watershed



It is observed that the average annual temperature is 21.1°C. Temperature does not varies much throughout the year with an average amplitude of 1.6°C. The warmest month is September with 22.1°C and March is the coldest, with an average temperature of 20.5°C.

Figure 14 shows the strong seasonal variability across the year with a dry season from June to August that features monthly precipitations below 10 mm/month.

5.3 HYDRO-METEOROLOGICAL DATABASE

5.3.1 Rainfall and meteorological data

Rainfall data from two sources were used in this study: (i) the WorldClim climate database and (ii) the Climate Hazards Group InfradRed Precipitation database (CHIRPS).

WorldClim is a set of global data representative for the period ~1970-2000 available with a spatial resolution of about 1 km and at a monthly timestep. The spatial resolution is obtained by interpolation of ground-measured data.

Climate Hazards Group InfraRed Precipitation with Station data (CHIRPS) is a 30+ year quasi-global rainfall dataset at a daily timestep. Starting in 1981 to near-present, CHIRPS incorporates 0.05° resolution satellite imagery with in-situ station data to create gridded rainfall time series for trend analysis and seasonal drought monitoring. Values extracted from these satellite images are the means of the precipitation that falls each day on the entire catchment.

5.3.2 Hydrological data

An existing streamflow monitoring station (ref: Muyovozi River at Kanyoni, 4AD2) is located 5km downstream the hydroelectric project just next to the B8 road bridge (Figure 15). Data have been collected in the Lake Tanganyika Water Basin Office but the completeness of the time-series (82% of daily data gap, 1988-2014) is not sufficient for a reliable statistical analysis.

To estimate the streamflows of the Muyovozi River at the hydroelectric project, a method based on the extrapolation of existing hydrological information from similar river catchments was developed and is described in the next section.

Figure 15. Existing streamflow monitoring station (ref: Muyovozi River at Kanyoni, 4AD2)

5.4 RAINFALL AND STREAMFLOW DATA ANALYSIS

5.4.1 Annual and monthly rainfall

5.4.1.1 Annual and monthly distribution

The analysis of the annual distribution of rainfall within the study area is based on the CHIRPS dataset, presented in section 5.3.1. The results are presented monthly in the section 5.2.3, Figure 14.

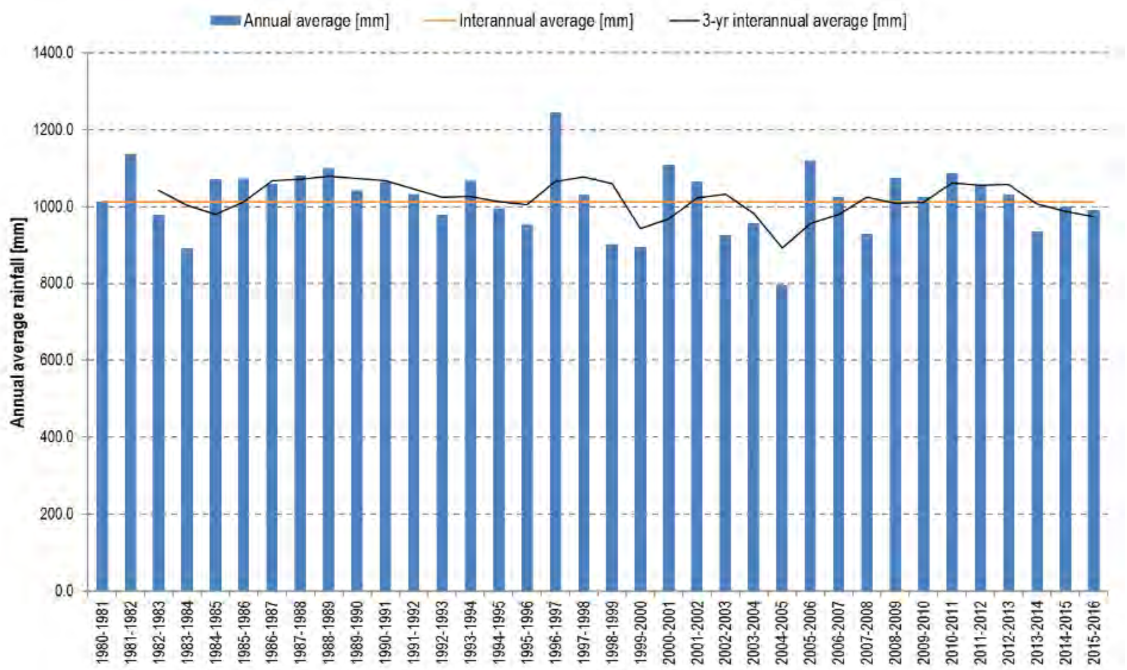
5.4.1.2 Spatial distribution

The analysis of the spatial variation of rainfall within the study area is based on the WorldClim dataset, presented in section 5.3.1. The spatial variation of average annual rainfall within the watershed is significant with a minimum of 887 mm in the northern part of the catchment and a maximum of 1,102 mm in its southwestern part. This is illustrated in Figure 17.

5.4.1.3 Temporal variation

The temporal variation in rainfall for the Muyovozi catchment has been studied from CHIRPS dataset (period 1981-2017) and the results are presented in the graph below. Average data are constant and does not feature any clear trends in annual patterns.

Figure 16. Temporal variation in rainfall for the Muyovozi catchment



5.4.2 Inflow analysis

5.4.2.1 Methodology

To estimate the streamflows of the Muyovozi River at the proposed hydroelectric project, a methodology based on the extrapolation of existing hydrological information from similar river catchments has been developed. This regionalization method was the method used in the Small Hydro Mapping Report delivered in April 2015.

The following two-stage approach was proposed to estimate the key hydrological statistics at the sites of interest:

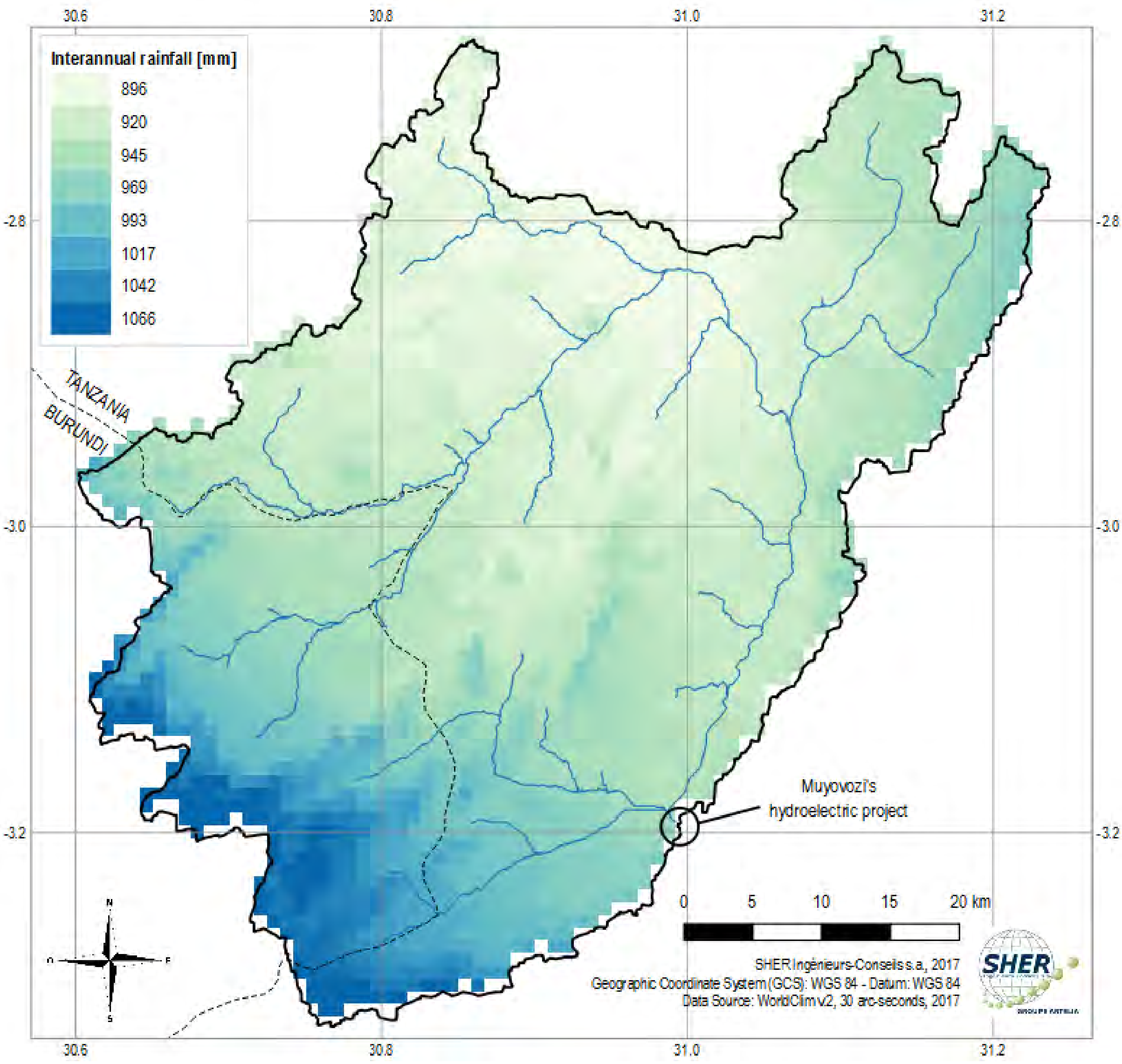
- Stage 1: Models selection and parameterization at gauged sites;
- Stage 2: Actual modeling by extrapolation at ungauged sites.

The objective of this model is to fit a Weibull statistical model for the flow duration curves at the 90 gauged sites using the key features of their watersheds. The Weibull law is characterized by two parameters (shape and scale factors).

The objective of the model parameterization is to build a relationship between the aforementioned two parameters of the Weibull law and the key features of the related gauged sites. These key features include the average annual rainfall, the watershed area, watershed average slope, elevation, etc. (regionalization). This analysis is carried out using the 90 flow gauged sites sample.

Then, the extrapolation to ungauged sites was made applying an extrapolation factor (function of the catchment area ratio or the annual precipitation) on the parameters depending on their locations relative to the existing gauging stations.

Figure 17. Spatial Variation of the annual rainfall on the Muyovozi catchment



5.4.2.2 Flow duration curve

Among the hydrological parameters, the determination of the flow duration curve is essential to know the availability of the flows in the river for the hydroelectric project. Indeed, this curve shows the percentage of time that the streamflow in a river is likely to equal or exceed some specified value of interest.

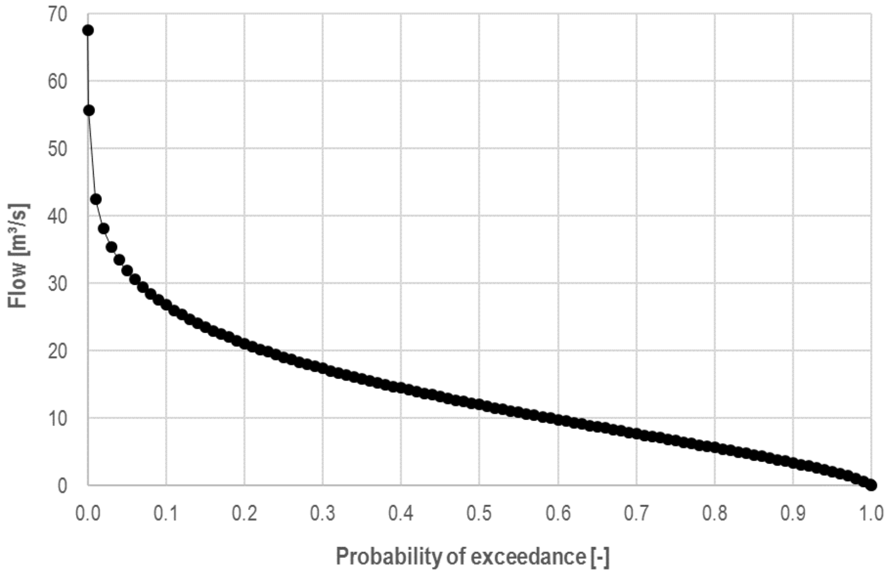
For the regionalization method, the flow duration curve results directly from the extrapolation of the two Weilbull law parameters. This extrapolation depends on the location of the hydroelectric site relative to the existing gauging stations.

Table 6 and Figure 18 show the modelled flow duration curve as well as the main percentiles. The proposed model shows that the streamflow of the Muyovozi River at the hydroelectric project is less than 12.0 m³/s 50% of the time and that it is higher than 26.8 m³/s only 10% of the time (over a year period). The flow guaranteed 90% of the time (329 days per year) is estimated at 3.4 m³/s.

Table 6. Flow duration curve of the Muyovozi River at the hydroelectric project

STREAMFLOW		EXCEEDANCE PROBABILITY
[M³/S]	[L/S/KM²]	[-]
2.1	0.78	$Q_{95\%}$
3.4	1.26	$Q_{90\%}$
5.7	2.08	$Q_{80\%}$
7.7	2.84	$Q_{70\%}$
9.8	3.61	$Q_{60\%}$
12.0	4.43	$Q_{50\%}$
14.5	5.33	$Q_{40\%}$
17.4	6.40	$Q_{30\%}$
21.1	7.76	$Q_{20\%}$
26.8	9.86	$Q_{10\%}$
31.9	11.75	$Q_{5\%}$

Figure 18. Modelled flow duration curve of the Muyovozi River at the hydroelectric project



5.5 FLOOD STUDY

5.5.1 Introduction

The flood study is essential for designing structures and equipment such as spillways or floodgates but also for temporary infrastructure such as cofferdams and temporary diversions during the construction period.

The flood study will focus on 10 years and 100 years return period. These floods will be used respectively for the construction and operation phases. A detailed justification for these return periods can be found in section 8.1.4 of this report.

5.5.2 Methodology

Given the lack of observed streamflow data, the methodology used to estimate the floods is a hydrological modelling only based on land features (topography, soil type, and land cover). Hence, the results remain flood estimates and will have to be confirmed at the next stage of the study.

The software used is Hydrological Modelling Software (HEC-HMS v4.2.1) developed by the Hydrologic Engineering Center of the US Army Corps of Engineer. This program is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing.

Hydrological modelling aims to represent the hydrologic response of the watershed for specific rainfall events. Hydrological models are composed by several parameters that can be estimated from land features (topography, soil type, and land cover) influencing the infiltration (production function) and the dynamic of the surface flow (transfer function). These parameters must be calibrated on observed streamflow data in order to establish the best rainfall-runoff relationship to the model. Validated, the model can be used to estimate the hydrographs for extreme rainfall events.

Given the lack of observed streamflow data, it is not possible to calibrate and validate the hydrological model. That is why, at this stage of the study, the results of this hydrological study are indicative only.

5.5.3 Extreme rainfall events estimates

The extreme rainfall events have been determined for 10, 25, 50 and 100 years return period from the CHIRPS dataset by a statistical extrapolation of the observed maximum precipitations (log-normal law[1]). Then, the 24-hr precipitations intensity have been statistically distributed to represent a typical event at the simulation time step. Results are presented in the table below.

Table 7. Extreme rainfall events estimates for the Muyovozi River watershed

Return period	10 years	25 years	50 years	100 years
24-hr precipitation	48.8 mm	55.0 mm	59.5 mm	63.8 mm

5.5.4 Hydrological parameters estimates

Production function[2]: to estimate the runoff generated for each sub-basin, a “production function” is used. This function evaluates the precipitation amount that does not infiltrated into the soil. The SCS Curve Number

[1] This law is advocated by some hydrologists who justify it by arguing that the appearance of a hydrological event results from the combined action of a large number of factors that multiply. Consequently, the random variable follows a log-normal distribution. Indeed, the product of variables is reduced to the sum of the logarithms of these variables and the central-limit theorem makes it possible to assert the log-normality of the random variable. [Translated from Musy A. (2005). Hydrologie générale. http://echo2.epfl.ch/e-drologie/]

[2] For more details about SCS Curve Number method: https://www.hydrocad.net/neh/630ch10.pdf

method has been selected. The parameter of this method (curve number) is calculated from two land features: (a) the hydrologic soil group (HSG) determined from soil type and (b) the land cover.

Transfer function[3]: to estimate the dynamic of the runoff for each sub-basin, a "transfer function" is used. This function represents how the water coming from the precipitation that is not infiltrated into the soil is moving within each sub-basin to reach the outlet. The SCS Unit Hydrograph method has been selected. The parameter of this method (time of concentration) is calculated from topographic land features: (a) the area and slope of the sub-basin and (b) the length and the slope of the main channel.

5.5.5 Flood estimates

Ten years and hundred years return period flood estimates at the Muyovozi hydroelectric scheme are presented in the following table.

Table 8. Ten years and hundred years return period flood events

Atlas Code	Site Name	Floods [m³/s]	
		T = 10 years	T = 100 years
SF187	Muyovozi	326	624

5.6 Key Hydrological Parameters of the Muyovozi Project

The key hydrological features of the Muyovozi hydroelectric project on the Muyovozi River are summarized in Table 9 below.

Table 9. Key hydrological features of the site

Characteristic	Parameter	Value	Unit
Catchment	Area	2,719.7	km²
	Mean elevation	1,428	m a.s.l.
	Maximum elevation	1,920	m a.s.l.
	Minimum elevation	1,208	m a.s.l.
	Average slope	4.0	m/km
Rainfall	Long-term average annual (CHIRPS)	1,019	mm/y
Streamflow	Guaranteed ($Q_{90\%}$)	3.4	m³/s
	Median ($Q_{50\%}$)	12.0	m³/s
Flood estimates	10 years	326	m³/s
	100 years	624	m³/s

The study reveals that the Muyovozi River features a favorable hydrology at the proposed location of the hydroelectric project. However, hydrological uncertainties are important and it is strongly recommended that hydrological monitoring of the river be done beyond this study. This will include:

- To continue the measurement of water levels at the automatic station installed downstream the hydroelectric project just next to the B8 road bridge;

[3] For more details about SCS Unit Hydrograph method: https://www.hydrocad.net/neh/630ch16.pdf

- To continue the gauging operations of this river in order to improve and validate the rating curve.

Beyond the development of the Muyovozi hydroelectric project, it is strongly recommended that the Government of Tanzania set up a hydrological monitoring network for its rivers with high hydropower potential in order to better understand the available water resources and thus promote the development of hydroelectric projects across the country. It is only in a context of reduced uncertainties through reliable, recent and long-term records (more than 20 years) that technical parameters and economic and financial analyzes of hydroelectric developments can be defined accurately, enabling optimization of their design and their flood control infrastructure (temporary and permanent).

6 GEOLOGY

6.1 INTRODUCTION

The purpose of this chapter is to generate preliminary geological datasets and other important baseline information at the proposed site that will be used for the design of the hydroelectric scheme at the pre-feasibility study level. These data and information will also be used to define the geotechnical investigations that will have to be carried out at next stages of the study.

This study aims to inform about the geological conditions and the types of materials existing in the region, as well as to give an initial overview of the geotechnical properties of these materials. Recommendations are also formulated regarding the need for further studies and investigations if necessary.

6.2 GEOLOGICAL REFERENCE MAP

The area is just east of 10-12km east of the geological sheet QDS43 (not available). However, the geology of the site is the same, i.e. quartzitic sandstone and quartzites.

6.3 GEOLOGICAL SETTING

Site SF-187 is located on the Muyovozi River (Figure 19 and Figure 20) that separates Njomulole village of Kakonko district and Kanyoni village of Biharamulo district is characterized by a whitish medium grained, semi massive quartzite (Figure 19).

Figure 19. Location and Geology map of SF187

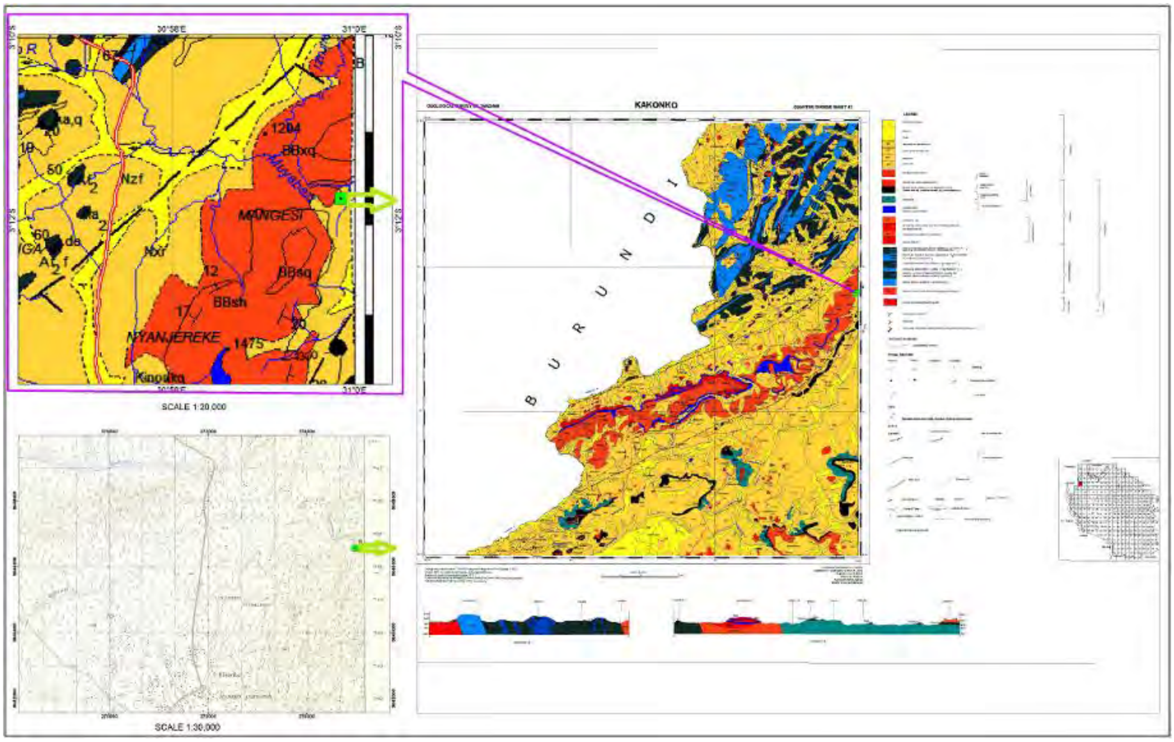


Figure 20. Trace of Intake (IN), canal (in light blue) and penstock and powerhouse (in red) for site SF-187 with topographic contours

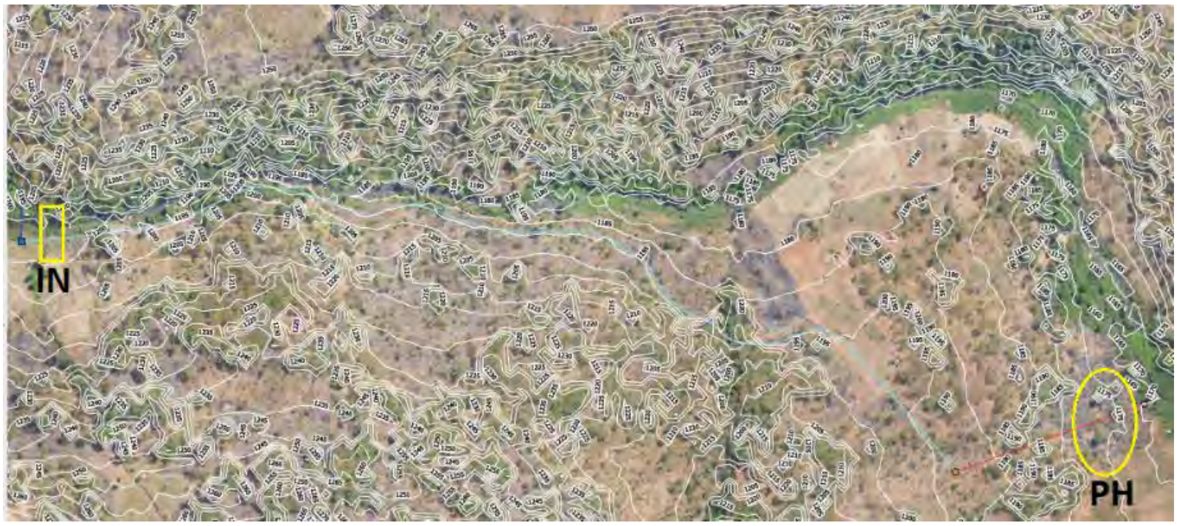
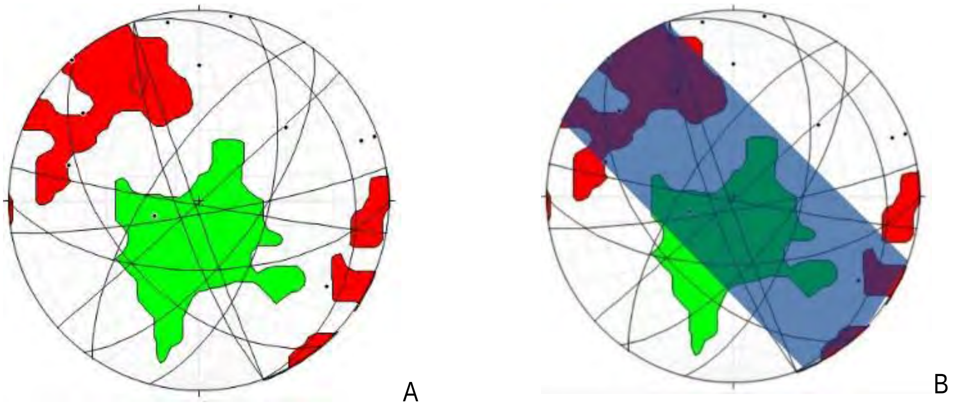


Figure 21. Quartzite outcrop on the left bank, site SF-187

The quartzite is fractured, the fractures being mainly NW-SE and NE-SW. Few E-W structures are also present (Figure 22).

Figure 22. Structural analysis of data from Site SF-187 of Muyovozi river, Kakonko-Biharamulo boarder. Note NW-SE, NE-SW and E-W faults (faults and joints); B: Approximate stability field for the Weir; ~310°/130°



6.4 TECHNICAL CHARACTERISTICS

Intake or Weir position: It is characterized by massive quartzite that is fractured as illustrated in Figure 19. The riverbed at the proposed weir position is 10m wide with 15-20% of rocky bed (Figure 23). Of these rocks, 80-85% are quartzitic boulders whereas the other portion is of in-situ quartzite.

Figure 23. Proposed weir position

Left bank support aspect: Characterized by a steep rocky slope. Rocks are mainly boulders (50-80%). The rocks are also semi-massive, jointed or fractured. This side of the riverbank represents the footwall of a fault (normal / oblique fault). In order to increase the weir stability (anywhere within the > 50m long stretch between water fall to the east and west of this water fall, see red polygon-demarcated area in Figure 24):

Figure 24. Recommended area for weir construction as demarcated by the red polygon

- The Weir location must be located at the position containing minimum amount of boulders and at a location with minimum possibility to have rock falls as indicated in Figure 24, and
- the boulders vertically above the proposed weir position must be either quarried off or reinforced to avoid rock falls.

Discussion on Weir stability

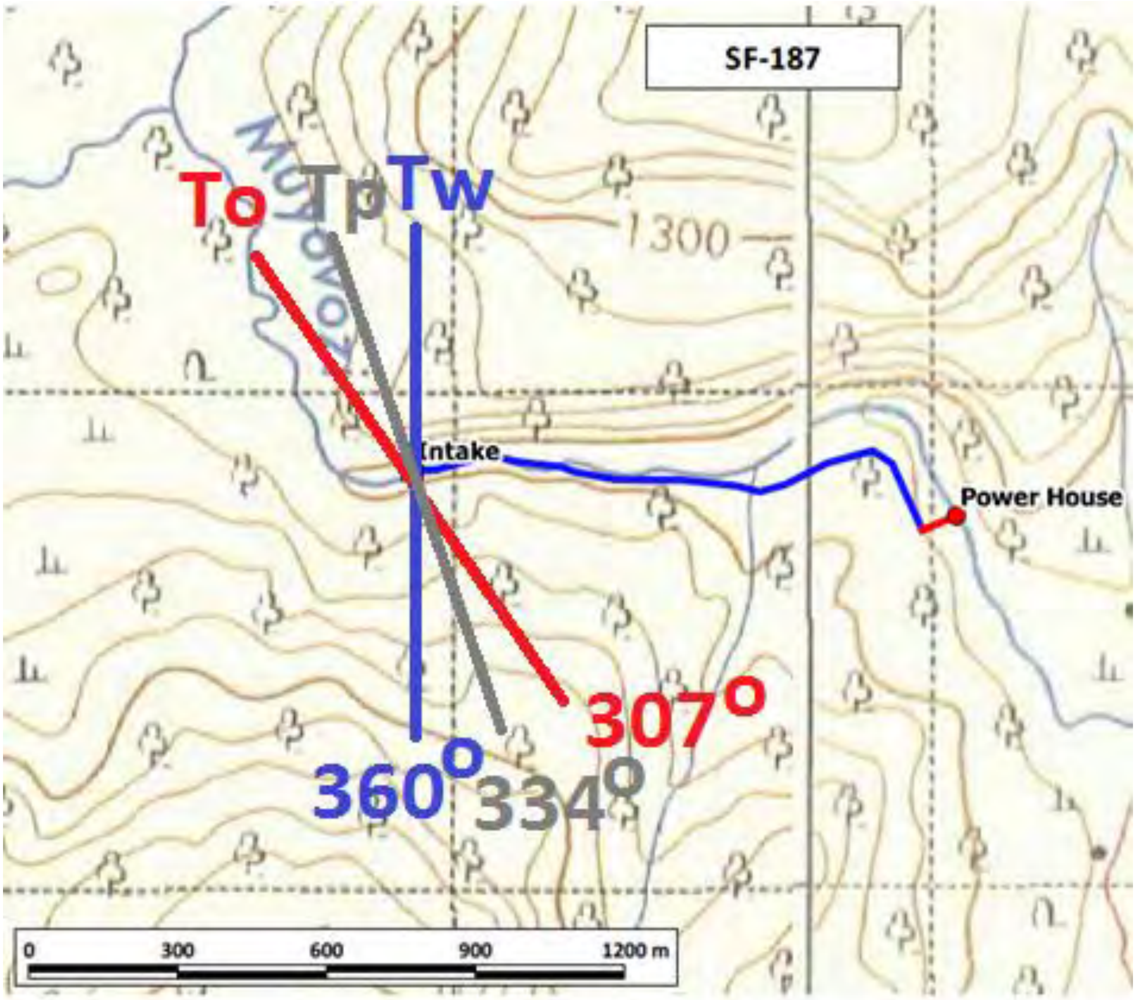
On the tectonic point of view, and based on the limited data available, it can be deduced that:

(i) The rocks around the weir and between the waterway trace dip in two opposing directions (NW and SE with an elevated relief at the middle field). The latter indicates either a domal structure or presence of an asymmetric fold ; difference of dips is of the order of 25° and as per Figures, it is an overall SE shallow dip. The fact that the rocks dip in either direction and that the overall dip is shallow due SE, this indicates that the effect of tectonic joints / fractures is almost minimum as the effect of dips tend to cancel each other. Therefore, the effect of flowing water hold stronger than the effect of tectonics on the optimal direction of the weir at the proposed location.

(ii) Considering Figure 25: Suppose the optimal orientation of the weir for optimal stability due to tectonic activities in the area is to be defined by To, and suppose the optimal orientation of the weir as a function of water forces to be defined by Tw, then the Practical stability orientation would be a point between To and Tw. Assuming that the effects of To and Tw are the same on the weir, then a mid point between them Tp would define the orientation as Tp = To+Tw

However, because To tends to be minimum, then Tp ~Tw. Therefore, during weir construction, the orientation should be perpendicular to the orientation of the riverbed but with allowance in orientation changes towards 334° (Figure 25).

Figure 25. Conceptual model of resolved resultant orientations by tectonic stresses (To), water (Tw) and the resulting practical orientation of the weir at the proposed position, Tp at SF-187



Right bank support aspect: This part of the riverbank is not steep but also contains 50-80% quartzite boulders that are fractured / jointed. The right bank, represents the hanging wall side of a fault structure (normal / oblique fault).

Powerhouse: The power house position has been proposed after detailed topographic studies. At this location (Figure 26), a moderate excavation work is required in order to expose the fresher quartzitic sandstone unit. A platform, about 2 to 2.5 times wider than the proposed powerhouse is required in order to allow other operations around the site to be undertaken safely.

Figure 26. Proposed location for the powerhouse

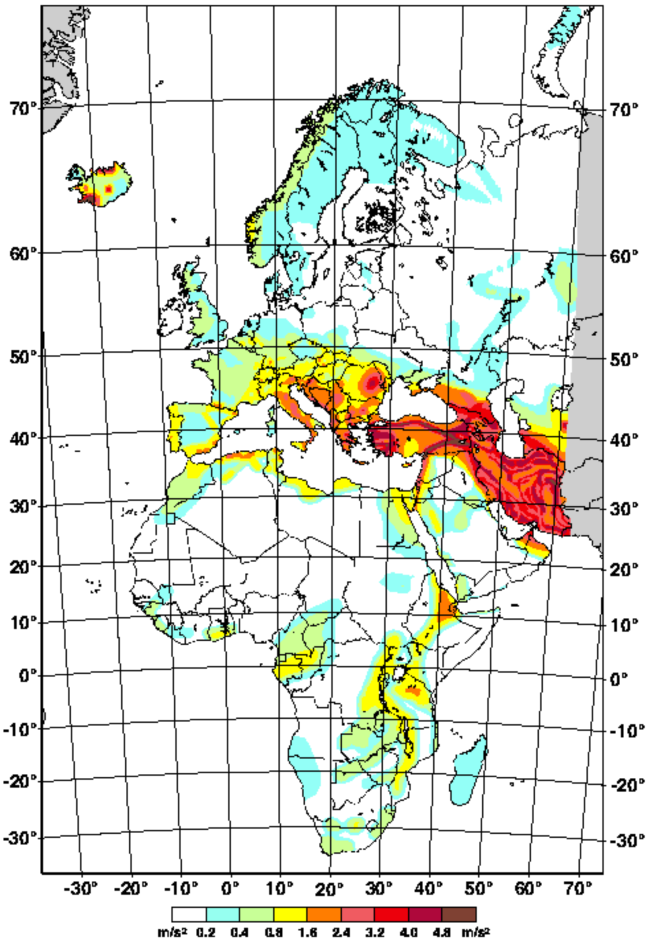
6.5 SEISMICITY

Tanzania is located along the Great African Rift. Seismicity in this area is relatively unknown, mainly due to the lack of historical data. Within the framework of the Global Seismic Hazard Assessment (GSHAP), the assessment of the seismic hazard in West Africa was carried out based on two data sources:

- The catalog of the British Geological Survey (Musson, 1994), containing quakes of magnitude greater than 4 from 1600-1993 (this is assumed to be complete for magnitudes greater than 5 beyond the year 1950 and for Magnitudes greater than 6 since the beginning of the 20th century),
- The NEIC catalog for more recent events (1993-1998).

A statistical method was used to determine the horizontal acceleration values due to earthquakes. The map below shows the distribution of seismic acceleration coefficients for the entire African continent. It can be seen that the project area is characterized by horizontal accelerations between 0.4 and 1.8 m/s². Those values will of course have to be confirmed by additional studies.

Figure 27. Horizontal acceleration due to seismicity (source: GSHAP)



6.6 CONCLUSIONS AND RECOMMENDATIONS FOR ADDITIONAL INVESTIGATIONS

6.6.1 Conclusion

There are no major geological contraindications to the construction of the Muyovozi hydroelectric scheme. However, further investigations will have to be carried out during the detailed studies phases in order to remove various uncertainties concerning geology and geotechnical characteristics (rock resistance, soil strength, rock compactness, rock permeability, etc.).

A table presented in the following section summarizes the uncertainties to be removed and the type of investigations to be carried out to remove them.

6.6.2 Additional investigations

ELEMENT TO BE CONSIDERED FOR RECOMMENDATION	UNCERTAINTIES TO BE CLEARED	RECOMMENDED FUTURE WORK IF ANY
Bed at weir	✓ Quantity of boulders versus in-situ rocks unknown	✓ Amount of boulders and in-situ rocks needs to be determined in order to know how much materials need to be excavated or removed
Left support at weir	✓ Quantity of boulders versus in-situ rocks unknown	✓ Amount of boulders and in-situ rocks needs to be determined in order to know how much materials need to be excavated
Right support at weir	✓ Quantity of boulders versus in-situ rocks unknown	✓ Amount of boulders and in-situ rocks needs to be determined in order to know how much materials need to be excavated
Waterway	✓ NONE	✓ N/A
Penstock	✓ NONE	✓ N/A
Power house	✓ Quantity of boulders to be removed and rock materials to be leveled	✓ Amount of rock materials to be excavated need to be worked out

6.7 REFERENCES

Compilation of the GSHAP regional seismic hazard for Europe, Africa and the Middle East (http://www.seismo.ethz.ch/static/GSHAP/eu-af-me/euraf.html).

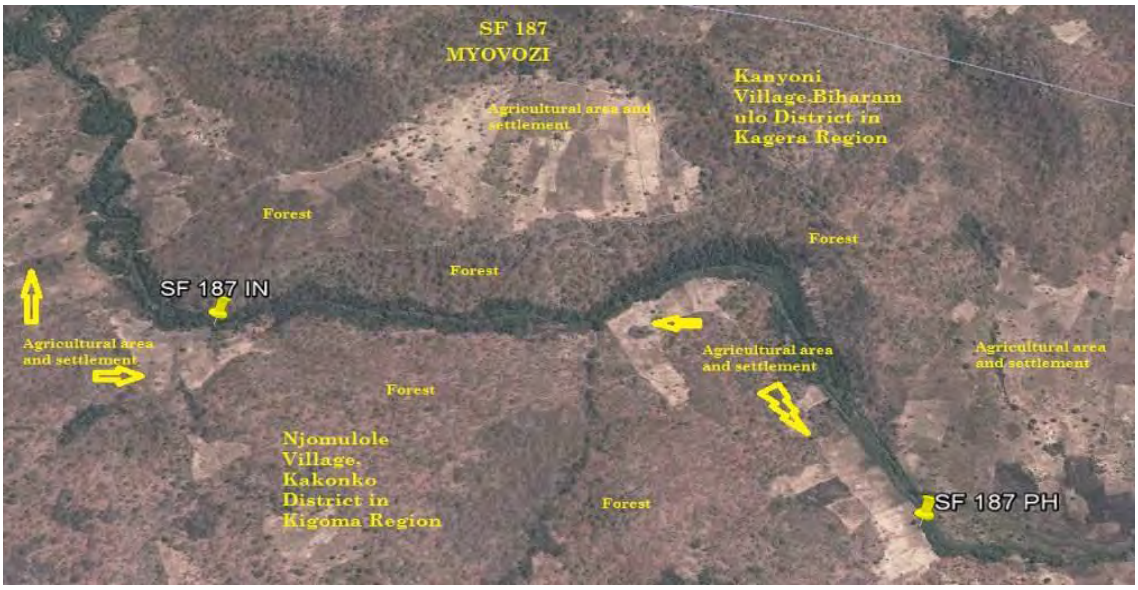
7 PRELIMINARY ENVIRONMENTAL AND SOCIAL IMPACT ANALYSIS

The Environmental and Social Impact Assessment (ESIA) is the procedure for prior analysis of the impacts that a project may have on the environment. It ensures the integration of environmental concerns into project planning and allows for consideration of likely environmental measures from the design stage of the project.

7.1 SOCIO-ENVIRONMENTAL BACKGROUND

The project is located in Njomulole and Kanyoni villages, Kakonko and Biharamulo Districts, Kigoma and Kagera Regions. The site is in Muyovozi River which marks the boundary between the regions. Generally, the area has some areas for human activities and some are still virgin land (Figure 28).

Figure 28. Land use in the project site (IN = intake ; PH = powerhouse)



The area is characterised by a Miombo vegetation cover (Figure 29) dominated by valuable tree species such as Pterocarpus angolensis (Mninga), Khaya nyasica (Mkangazi), Afzelia quanzensis (Mkora), Milecea- exelsa (Mvule), Brachystegia spiciformis (Mtundu), and Pterocarpus all species (Mkurungu).

Figure 29. Vegetation cover in the study area

The area is used for agricultural practice. Most of the crops observed are food (Maize and Beans) and cash crops (Bananas and Sugarcane), Figure 30, Figure 31, Figure 32.

Figure 30. Maize and Banana farm as observed

Figure 31. Agricultural area and settlement close to weir

Figure 32. Agricultural area close to proposed powerhouse area

From the socio-economic perspective, the main Njomulole and Kanyoni Villages are located more than 500m away from the projected site. However, a few houses are found close to the proposed weir and powerhouse as well as on the canal site.

The two villages have no power. Kakonko town (10km to the South) has power (supplied by the Tanzania Rural Agency under Rural electrification program).

7.2 WORLD BANK OPERATIONAL POLICIES AND GUIDELINES

The World Bank has developed a series of operational policies (OP), or safeguards, to help identify, avoid, and minimize social and environmental impacts. These operational polices and safeguards are prerequisites to accessing the World Bank funding assistance to address certain environmental and social risks for specific development projects. There are 11 OPs and associated World Bank procedures that apply to environmental and social risks. Similarly, there are eight IFC performance standards. The details will be provided as part of either the prefeasibility or feasibility studies for each priority projects.

This section summarizes the World Bank's safeguard policies that contribute to the sustainability and effectiveness of development within the World Bank's projects and programs by helping to avoid or mitigate the impacts of these activities on people and society, environment. It ensure potential adverse environmental and social impacts that may result from individual project activity are identified early, and appropriate safeguard measures are prepared prior to implementation to avoid, minimize, mitigate and, in cases where there will be residual impacts, offset or minimize adverse environmental and social impacts.

The following World Bank safeguard policies could be triggered when implementing the Muyovozi hydroelectric project:

- OP 4.01 – Environmental Assessment (EA): The Bank requires Environmental Assessment (EA) of projects proposed for Bank financing to ensure that they are environmentally sound and sustainable, and thus to improve decision making. However, we can already estimate that the adverse impacts on human populations and environment-linked areas are limited. They are reduced, not irreversible and some measures can prevent, mitigate or minimize them. Moreover, these measures can improve the environmental performance.

- OP 4.12 – Involuntary Resettlement: The project needs new use of some areas (implementation of the plant, renovation of the access roads to the site…) that can be crop zones.

The projected weir (3m high) is classified as a small dam (<15m high); the usual generic safety measures for dams are appropriate and do not need the implementation of OP 4.37 – Safety of Dams (for large dams).

The triggering process of the policies will be completed by the World Bank during dedicated projects appraisal.

7.3 SOCIO-ENVIRONMENTAL CONSTRAINTS

Overall, the project does not feature any major environmental or social constraints that cannot be mitigated by appropriate measures and that would jeopardize its development.

Some hamlets are situated in the vicinity of the proposed scheme (weir, canal, penstock pipes and powerhouse). Potential impacts (noise, traffic, atmospheric emissions) on the riparians during the construction phase will have to be mitigated by appropriate measures.

Overall, the development of the project will lead to positive externalities by the use of local labor during the construction phase, increase local skills and bring electricity to local communities that will eventually foster local economic development.

8 PROPOSED SCHEME AND DESIGN

8.1 PROPOSED SCHEME DESCRIPTION

8.1.1 Diverting structure, intake, waterway and powerhouse

As illustrated in Figure 33, two alternatives for the positioning of the diverting and the intake structures were identified. Alternative *"B"* was eventually chosen for the following reasons:

1) As presented in chapter 5, flood events are expected to be large with a design flood (100 years return period) of 624 m³/s. Axis *"B"* allows for increasing the length of the overflowing weir crest and hence reducing the rise of water level upstream the proposed weir location during flood events.

2) Positioning the diverting structure along Axis “*B*” provides for more space for the intake structure including the desilting structure, due to favorable slopes on the right bank of the river.

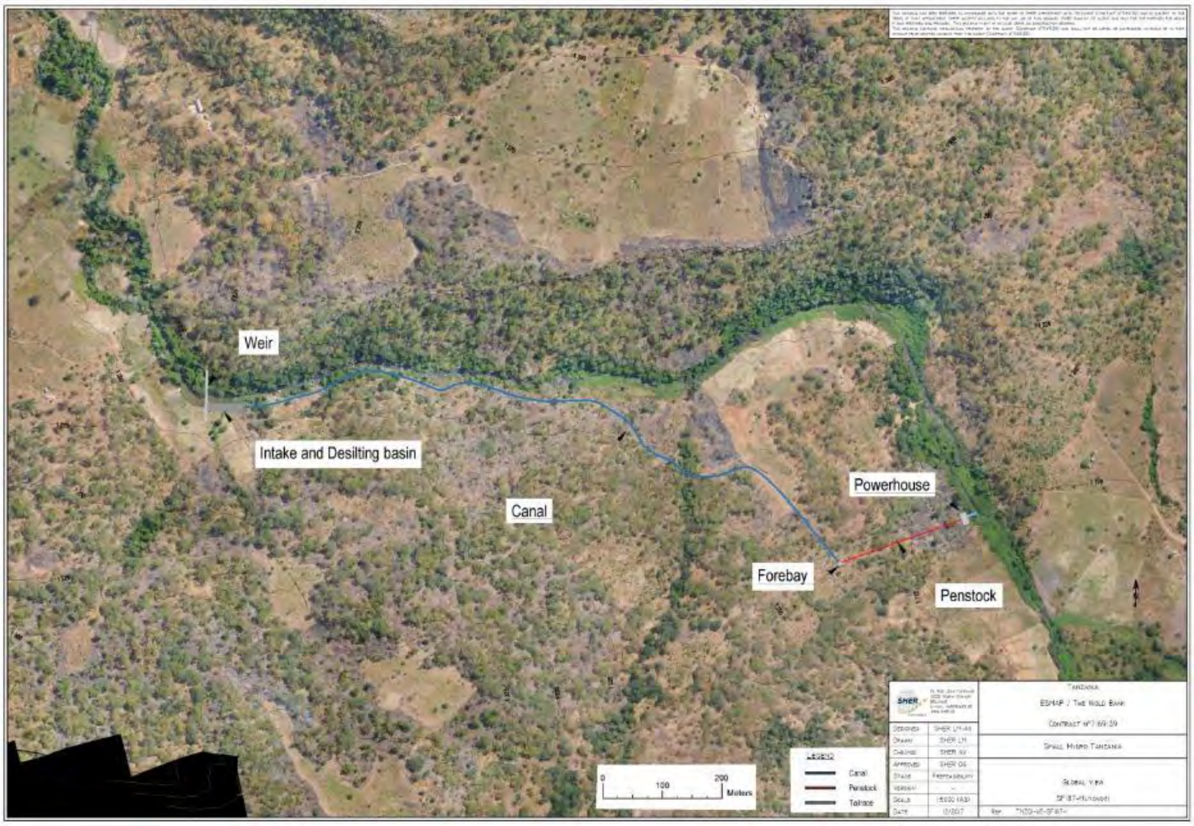
Figure 33. Weir axes alternatives

The entire proposed scheme is presented in Figure 34 below and in the Appendix 12.1. The intake structure, waterway and power plant will be located on the right bank of the river do to its favorable topographic features. The development of the proposed project will require the construction of a bridge crossing the river, near the proposed powerhouse location.

A 1 147m long headrace canal (rectangular section) will convey the water from the intake structure (including a desilting structure) to the forebay. The intake and the waterway are designed to minimize head losses. The 226m long pressure penstock will convey the water from the forebay to the power plant which is located on the right bank of the river, above extreme flood level.

The diverting structure (consisting of a weir) is equipped a gated flushing channel to prevent the accumulation of sediments in front of the intake.

Figure 34. Detailed proposed scheme and main components



Geographical coordinates of the main structures are presented in the table below:

Structure	Latitude*	Longitude*
Weir and intake	-3.196	30.992
Hydropower plant	-3.198	31.003

* Decimal degree, WGS1984

8.1.2 Type of scheme

The Muyovozi project is a run-of-the-river hydropower type of scheme without regulation capacity.

8.1.3 Design flow

The flow duration curve was determined in chapter 5 of this report. It does however not correspond directly to the flow available to the equipment. Indeed, the Muyovozi River will be by-passed over a length of approximately 1.5km. An environmental flow guaranteed at all times is required for environmental and ecological reasons.

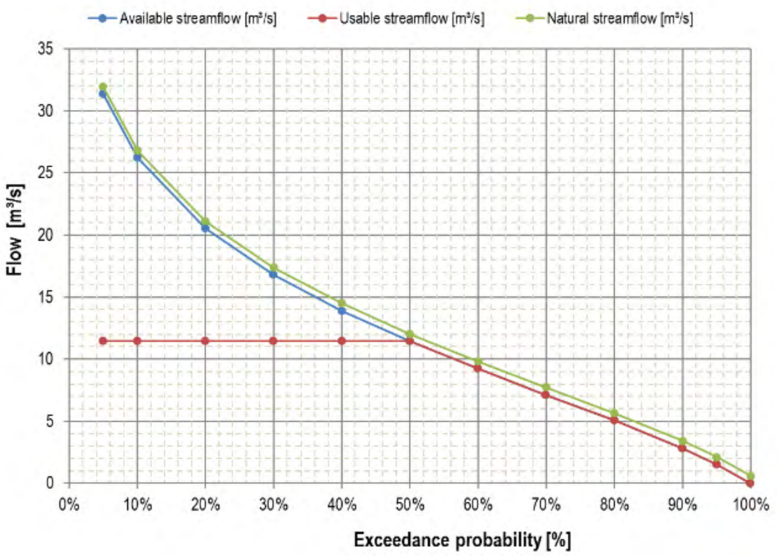
In the absence of commonly agreed international standards and given the uncertainties on the available streamflow of the river, the ecological flow is set at 600 l/s, which corresponds to 5% of the median flow ($Q_{50\%}$) of the river in natural conditions. Since this flow is not available for the turbines, it is necessary to subtract it from the flow duration curve of the river.

The flow duration curve that can actually flow through the turbines is finally obtained by considering the design flow rate of equipment chosen at the pre-feasibility stage, namely 11.44 m³/s. The usable flow duration curve is illustrated in Figure 35 below.

The final choice of design flow will be made at the feasibility study stage based on an economic analysis of alternatives. The flow duration curve must also be validated by additional hydrological analysis and measurements.

Figure 35. Usable flow duration curve of the Muyovozi River at the project location

Streamflow [m³/s]		Exceedance Probability
Natural	Available	[-]
2.1	1.5	$Q_{95\%}$
3.4	2.8	$Q_{90\%}$
5.7	5.1	$Q_{80\%}$
7.7	7.1	$Q_{70\%}$
9.8	9.2	$Q_{60\%}$
12.0	11.4	$Q_{50\%}$
14.5	13.9	$Q_{40\%}$
17.4	16.8	$Q_{30\%}$
21.1	20.5	$Q_{20\%}$
26.8	26.2	$Q_{10\%}$



8.1.4 Design Floods

Several national bodies have examined the problem of defining the relevant design flood to be considered for the design of spillway and other associated flood structures. Only US method is developed below.

According to USACE (United States Army Corps of Engineers) in *Recommended guidelines for safety inspection of dams*, dam are classified in accordance with 2 characteristics: (i) the size of the structure and (ii) the potential hazard. The tables below present the classifications.

Table 10. Size classification (USACE)

Category	Storage (Ac-ft – hm³)	Dam height (Ft – m)
Small	< 1000 Ac-ft < 1.2 hm³	< 40 Ft < 12.19 m
Intermediate	> 1000 Ac-ft et < 50 000 Ac-ft >1.2 hm³ et < 61.7 hm³	> 40 Ft et < 100 Ft 12.19 m et < 30.48 m
Large	> 50 000 Ac-ft > 61.7 hm³	> 100 Ft > 30.48 m

In the table above, the height of the dam is calculated from the lowest point of the structure to the maximum level of the reservoir. The category is defined either by the storage capacity of the reservoir or by the height of the dam, depending on the characteristic that classifies the dam into the less favorable category.

The proposed weir on the Muyovozi will be less than 12m high and the storage volume or the reservoir will be less than 1.2 hm³. Therefore, the proposed weir is classified as being *"Small"*.

As far as potential hazard is concerned, it can be considered as *"Low"* according to the table below: there is no risk of loss of human life in the event of failure or misoperation of the diverting structure or appurtenant facilities. No significant industry or cultivated area have been identified downstream of the proposed hydropower project.

Table 11. Hazard potential classification (USACE)

CATEGORY	LOSS OF LIFE (EXTENT OF DEVELOPMENT)	ECONOMIC LOSS (EXTENT OF DEVELOPMENT)
Low	None expected (No permanent structures for human habitation)	Minimal (undeveloped to occasional structures or agriculture)
Significant	Few (No urban development and not more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than a few	Extensive community, industry or agriculture

Table 12 presents the USACE's recommendations for the design flood to be considered as a function of the potential hazard that may occur in the event of failure or misoperation of the diverting structure or appurtenant facilities and the size of the structure. The flood is expressed either by its return period (or frequency) or by the PMF. The PMF (Probable Maximum Flood) is the largest possible flood that can occur through the most severe combination of critical meteorological, geographic, geological and hydrological conditions reasonably possible in a watershed.

Table 12. Recommended spillway design floods (USACE)

HAZARD	SIZE	SPILLWAY DESIGN FLOOD
Low	Small	50 to 100-year frequency
	Intermediate	100-year to ½ PMF
	Large	½ PMF to PMF
Significant	Small	100-year to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
High	Small	½ PMF to PMF
	Intermediate	PMF
	Large	PMF

Following the aforementioned guidelines of the USACE, the recommended design flood for the Muyovozi hydroelectric scheme is from 50-years to 100-years frequency. The hydrological study presented in chapter 5 estimates the 100-year return period flood to be 624 m³/s.

8.2 STRUCTURES DESIGN

8.2.1 Diverting structure type and characteristics

Given the nature of the foundations as well as the estimated water head on the weir for the design flood, a concrete gravity-overflow weir is the most appropriate structure. It is recommended that the height of the weir minimizes the impact on the upstream water level in order to avoid flooding agricultural lands during extreme flood events. A concrete structure is also particularly recommended for submersible structures. This choice is motivated by the following elements:

- The local geology shows that the rock is of good quality, adapted to the foundations of a concrete weir;

- Given the magnitude of the design flood, the structure must be as low as possible in order to minimize the impact of the upstream water level rise;
- An ungated weir/spillway will be easier to build and safer in design since there is no risk of dysfunction or misoperation of the gates, particularly during flood events.

The crest length will be 40m, which limits the water level over the crest during floods. The diverting structure will be equipped with a gated flushing channel on the right bank to flush the sediments that would have accumulated in front of the water intake (see section 8.2.3).

The main function of a spillway is to allow the passage of normal (operational) and exceptional flood flows in a manner that protects the structural integrity of the structure and its foundations.

The stability of the diverting structure results from its shape. The shape of the overflowing section of the structure will be trapezoidal in order to ensure its stability during extreme flood events characterized by significant water level. The upstream slope of the weir is set as 80° while the downstream slope is 50°. The shape of the weir will have to be confirmed during the feasibility study based on a more detailed topography.

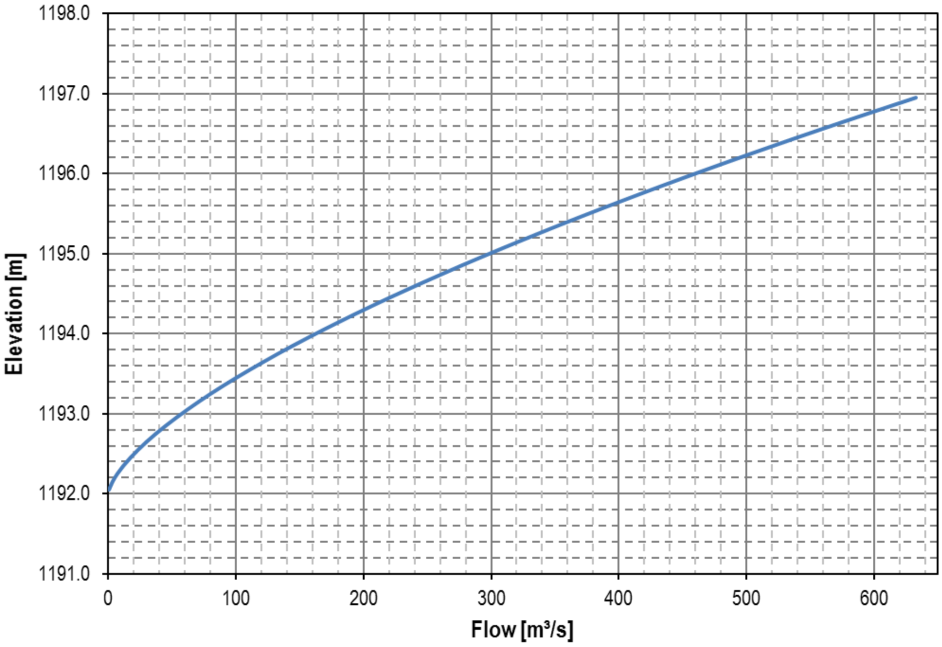
The discharge flowing over a spillway is calculated based on the following equation:

$$Q = C_d L h^{\frac{3}{2}} \sqrt{2g}$$

Where Q is the discharge [m³/s], C_d the spillway coefficient [-], L the length of the overflow crest [m], h is the total hydraulic head (static and dynamic head) over the crest [m] and g is the gravitational acceleration [m/s²].

Considering a broad-crested spillway, the hydraulic head over the crest for the design flood (624 m³/s) will be 4.80m. The spillway rating curve is presented in Figure 36 below.

Figure 36. Spillway rating curve



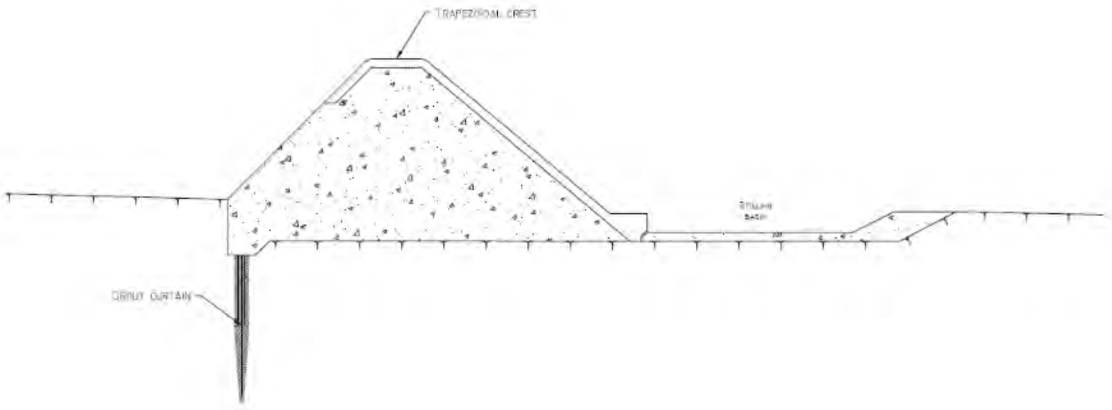
The field reconnaissance revealed the presence of agricultural lands upstream the confluence of the two rivers that form the Muyovozi River. The detailed studies will have to ensure that the water level during extreme flood events will not negatively affect those agricultural lands. The design of the structure might require being adapted accordingly.

The main features of the diverting structure are presented in Table 13 and a typical cross section of the weir is shown in Figure 37.

Table 13. Diverting structure: key features

PARAMETER	VALUE	UNIT
Structure type	Gravity Trapezoidal overflowing section	
Material used	Concrete	
Overflowing crest length	40	m
Total structure length	85	m
Overflowing section height	3.00	m
No-overflowing section height	8.80	m
Crest elevation	1,192.00	masl
Slab elevation	1,189.00	masl

Figure 37. Typical cross section of a trapezoidal broad crested weir



It is recommended to reshape the riverbanks upstream the weir in order to remove the accumulated sediments and improve the hydraulic conditions for the intake and spillway. The approximate area to be reshaped is illustrated in Figure 38.

Figure 38. Approximate area to be reshaped upstream the proposed weir location

8.2.2 Temporary diversion

The purpose of the temporary diversion is to dry up part of the river to allow the construction of the weir and appurtenant structures described in the previous section. The temporary diversion will be implemented consecutively on the right bank in order to construct the gated flushing channel and the intake, then on the left bank.

It will consist of a compacted embankment cofferdam or, if the ground conditions are favorable, sheet piles.

8.2.3 Outlet structure

The outlet structure consist in a gated flushing channel. It is designed to allow inspection of the weir and intake. While open, it can create a strong current which will allow flushing sediments accumulated close to the intake structure.

The flushing channel will be equipped with gates of which the invert is positioned at an elevation close to the elevation of the natural riverbed. The gates will be located on the right side of the weir, next to the intake structure to allow an effective purge of the accumulated sediments.

The number of bays and their size were calculated to ensure outflow corresponding to twice the median streamflow of the river (24 m³/s). This objective is achieved with the installation of two 1.60m wide and 2.00m high radial gates.

Table 14. Gated flushing channel characteristics

PARAMETER	UNIT	VALUE
Invert elevation	m	1189
Number of bays	-	2
Width	m	1.60
Height	m	2.00

8.2.4 Waterway

8.2.4.1 Intake structure

The intake will be located on the right bank in the continuity of the weir.

The intake will be equipped with a screen and an automatic screen cleaning system upstream of the intake gates, to prevent floating debris or large stones from obstructing the intake gates. The section of the bars and their spacing will be determined at the feasibility study stage. A flushing gate will be installed at the end of the transition zone from the intake structure to the desilting structure to allow for the removal of sediments that would have entered the intake.

The intake is designed taking into account the following constraints:

- The invert elevation will be set at least 0.50m above the invert of the flushing channel gates;
- The velocity of water at the entrance of the screen should not be greater than 0.7 m/s to minimize turbulence and facilitate screening of debris. That will also minimize head losses.

Hence, the intake will consist of 4 bays of 2.10m wide and 2.50m high, followed by a free inlet that will guide the current lines gradually towards the desilting structure. The invert of the intake will be set at elevation 1189.50m. Details are presented in Table 15 below.

Table 15. Intake characteristics

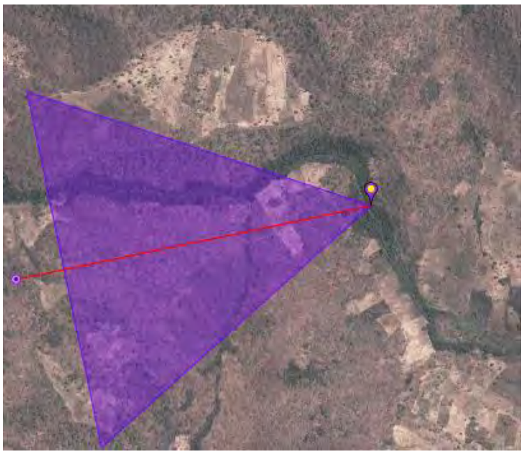
PARAMETER	UNIT	VALUE
Intake invert elevation	m	1189.5
Intake top elevation	m	1193.0
Screen inclination	°	15
Design flow	m³/s	11.4
Number of bays	-	4
Bay width	m	2.10
Bay height	m	2.50
Type of gate	-	radial
Flow velocity at intake	m/s	0.7

The free inlet which objective is to allow smooth converging of the current lines to the desilting structure will, for hydraulic reasons, be approximately 2.5 times the width of the intake, i.e. 25m. The feasibility study will study the hydraulic behavior of the intake in detail and adapt its design accordingly.

8.2.4.2 Desilting structure

Solid transport (sediments in suspension) is expected to be high, especially during the wet season. Consequently, the intake and desilting structures must be adequately designed to ensure the removal of the problematic sediment load before entering the headrace canal.

Figure 39. Turbidity of the river close to the proposed powerhouse location



If not taken into account at the design stage, it would result in operational and maintenance issues of the hydroelectric plant. The sediments that would accumulate in front of the intake will be flushed by frequent flushing operations using the flushing gates designed for this purpose.

The inlet of the desilting structure will have a sufficient slope in order to guide the solid particles to the outlet of the basins. Moreover, the desilting basins will be long enough to ensure particle settling.

The desilting structure is design based on topographic, hydraulic, type of sediments and operation constraints.

At the pre-feasibility study stage, the key features considered for design are presented in the following table:

Table 16. Preliminary design criteria for the desilting structure

Parameter	Unit	Value
Invert elevation	m a.s.l.	1192
Water outlet elevation	m a.s.l.	1192
Design flow	m^3/s	11.4
Average solid inflow	kg/m^3	0.8
Minimum diameter of the particles	mm	0.3
Maximum flush frequency	hours	12

The width of the desilting structure is determined in such a way that the horizontal water velocity is less than the maximum horizontal speed (which is determined based on the particles diameter). The length of the desilting structure is determined in such a way that a particle located on the surface can be deposited in the reservoir of the desilting structure. The horizontal and vertical velocity ratio is proportional to the ratio of the falling length to the falling height.

The desilting structure will therefore be composed of 3 sub-basins each 4m wide and will have a sedimentation length of 38.25m. To this must be added the transition zones upstream and downstream of the settling tank of the desilting structure. The desilting structure will therefore have a total length of 39.85m and a total width of 12.00m and a maximum depth of 4.00m.

The desilting structure will be equipped with a lateral spillway in the event of excessive inflows coming from the intake.

8.2.4.3 Headrace canal

The headrace canal features a rectangular cross section. The slope of the headrace canal is kept below 0.1% in order to minimize the head losses. The canal dimensions are defined on the basis of the uniform flow equation (Manning):

$$\frac{Q}{A} = V = n^{-1} R_h^{\frac{2}{3}} i^{\frac{1}{2}}$$

where A is the wetted area [m²], V is the mean flow velocity [m/s], n is the Manning coefficient, R_h is the hydraulic radius [m] and i the slope of the canal [-].

The headrace canal is designed taking into account:

- the average flow velocity is less than 2 m/s in order to avoid erosion of the concrete.
- the cross section is the most economical section: for a given discharge, slope and Manning coefficient, the discharge capacity will be maximum when the hydraulic radius (ratio of the wetted section on the wet perimeter) is maximum.

The canal features a rectangular cross-section of 2.40 m in width for a water height of 2.90 m, to which is added a freeboard of 30cm, which results in a total height of 3.20 m. The headrace canal is 1 147m long.

8.2.4.4 Penstock

The headrace canal and the pressure penstock meet at the forebay. The forebay will be equipped with a scour gate in order to drain the channel, as well as the particles that would have sedimented in the latter, back to the river. The forebay will be equipped with a safety spillway in the event of excessive inflows coming from the headrace canal or allowing the spill of the water in excess during variations flow through the turbines (production decrease, shutdown of a group, etc).

The pressure penstock will be overground and 226m long. The penstock will be supported by reinforced concrete support blocks. At this stage of the study, the distance between two support blocks is 6m. Anchoring blocks will be placed at each elbow to balance the forces related to the change of direction of the flow. A suitable system allowing the thermal expansion of the penstock should be defined at the feasibility study stage.

In order to limit the head losses to a maximum of 8% of the gross head, the penstock will have a diameter of 1.70 m.

8.2.5 Electromechanical Equipment

8.2.5.1 Basic data

The following specific values corresponding to the latitude and altitude of the power house are used for the equipment predesign and calculation:

Table 17. Basic data for electromechanical equipment

Parameter	Symbol	Unit	Value
Gravity Acceleration	g	m/s²	9.777
Average temperature of water	T_{water}	°C	20
Density of Water at 20°C	ρ	kg/m³	998.8

8.2.5.2 Selection of the type of turbine and the number of units

8.2.5.2.1 Methodology

The selection of the turbines type is made on the basis of the sites parameters such as the gross and net heads, and the plant design flow.

With a plant design discharge equal to $Q_{50\%}$, which is guaranteed 50% of the year, unit flexibility is needed to follow the river flow variations all along the year, as the hydropower scheme is a run-of-the river one. Moreover, the Muyuvozi site being connected to the Kibundo mini-grid, flexibility will also be needed to follow the demand during the day.

In order to make a preliminary selection of the most suitable turbine type and of the number of units, a first selection is made according to the available head.

The choice of the number of units is based on several criterion:

- Flexibility and reliability: Even if some turbine types allow a strong flexibility, it is chosen to consider at least two units per site. This choice will prevent possible electricity delivery shortage as, at least, one unit will remain on the grid in case of maintenance or break.
- Standardization or systemization: Considering the expected installed turbine capacities (<10MW per turbine) for ESMAP project, units will be standardized or systemized. In one hand, the best efficiency points will be a little lower than for large units, but, in the other hand, the cost and delivery time will be reduced. Moreover, the maintenance will be easier than for custom made products.
- Access to the site and powerhouse infrastructure: As the site access can be a problem for larges equipment or equipment parts, it can be mandatory to increase the number of unit in order to allow their transport from the nearest harbor. The number of units also has a direct impact on the powerhouse. The greater the number, the bigger the power house, but the bigger the crane capacity and the unit weight leading to high loads on the power house structure. Finally, the erection of smaller units will be easier than for bigger ones.

The preliminary turbine design is based on statistical values. The detailed analysis of the other alternatives and the optimization of the choice must be made during feasibility study if the site is selected at the end of the prefeasibility phase.

The power and rotation speed of the generators depend on the turbine hydraulic design. The selection process aims in finding the higher rotation speed (which reduces the size of the rotating parts and then price of the unit), taking into consideration hydraulic phenomenon as for instance cavitation.

The efficiency of the generators is assessed according to their power and speed.

At the prefeasibility stage, the power factor of the generators is considered as equal to 0.9.

All the technical data (preliminary dimensions, rotation speeds, efficiency level, etc.) are given for information only. They have to be understood as orders of magnitude and can vary in further studies steps in function of the requested accuracy level.

8.2.5.2.2 Selection process results

According to the net head (~23.8 m) and the available flow, two types of turbines can be considered:

- Two or more Kaplan turbines with a rotational speed of 500 or 600 rpm or,

- Two or more Francis turbine with a rotational speed of 428.57 or 500 rpm.

Considering the installed capacity and the previous criterion, it is better to select at least 2 units to increase the reliability and the availability of the production.

A brief comparison of the Francis and Kaplan alternatives is given hereafter:

- The flexibility of the Francis is significantly lower than the one of a Kaplan. When considering two units, the minimum discharge will be around 2.3 m^3/s for the Francis and 1.4 m^3/s for a Kaplan.
- At nominal and maximal discharge the efficiency of a Francis turbine is of the same order of magnitude than he one of a Kaplan turbine. However, the efficiency at part load is decreasing faster for the Francis than for the Kaplan.
- Due to a higher rotational speed, the generator of the Kaplan unit should be less expensive than the one of the Francis unit. However, the generator shaft for the Kaplan turbine needs to be a hollow shaft to allow the passage of the runner blade control rod, which is more expensive than a standard generator. Finally, the price of this part will be slightly the same for both alternatives.
- The size of the Kaplan and Francis units will be slightly the same.
- The penstock will be protected by a trash rack associated with a manual cleaning. Even with these equipment, it is not possible to exclude that solid materials, vegetal and other floating materials will pass through the rack and reach the turbine. The Francis geometry is more sensitive to the floating material than Kaplan, as it can, for instance, be blocked in the labyrinths, what will block the runner and lead to a runner dismantling.

  The Kaplan, having less blades than a Francis is less sensitive to that kind of floating material.
- The frequency regulation with a double regulated Kaplan will be more accurate than with a Francis turbine. It will then be a strong advantage to control the frequency of the grid if the Muyovozi power plant output is high compared to the grid total power.
- If the demand from the grid is low, it could be a problem to supply it with one Francis unit.
- The Kaplan turbine, with its double regulation and complex runner is more expensive and more complicated to operate and maintain than a Francis.

The hereafter table gives an overview of the comparison

Table 18. Comparison between Kaplan and Francis turbines

	Kaplan	Francis
Answer to discharge variations	Excellent	Medium
Answer to demand variation	Excellent	Medium
Start-up and synchronization	Easy	Easy
Sensitivity to floating material	Low to medium	Medium
Sensitivity to solid material	Low to medium	High
Frequency regulation	Excellent	Good

Watertight shaft seal	yes	yes
Runaway risk	yes	Yes
Water hammer risk	Medium to high	Medium
Maximum efficiency level	0.90 – 0.93	0.92 – 0.94
Rotational speed in the case of Muyovizi	Medium	Medium

Considering this short comparison and the prefeasibility study step, Kaplan turbines are selected. Francis alternatives could be considered in a full feasibility study.

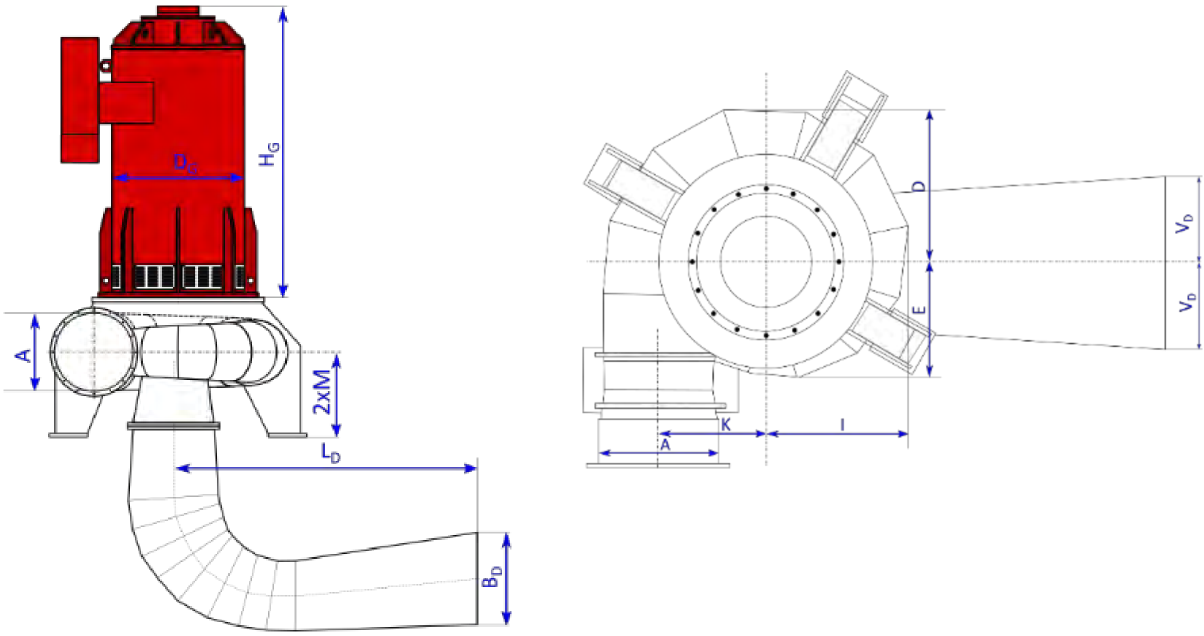
The main characteristics of the equipment are:

Turbine type		Kaplan
Number of turbines	(-)	2
Nominal turbine discharge	m3/s	5.7
Minimal turbine discharge	m3/s	1.43
Net head (at Qn and with all the turbines))	m	23.6
Rotation speed	rpm	600
Max. Turbine efficiency	(%)	≈ 91.4%
Max. Generator efficiency	(%)	≈ 94.6%
Power Factor	(-)	0.9
Generator Apparent Power	kVA	≈1’270
Generator Power	kW	≈1’135
Generator voltage	kV	0.4 or 0.69

The preliminary main dimensions of the 2 Kaplan units are:

	Spiral casing		
A	A (inlet diameter)	m	1.3
D	Length 1	m	1.8
E	Length 2	m	1.35
I	Width 1	m	1.6
K	Width 2	m	1.45
	Draft tube		
L_D	Length	m	6
B_D	Height of the exit	m	1.3
V_D	½ Width of the exit	m	1.12
	Generator		
D_G	Diameter of the generator	m	1.3
H_G	Height of the generator	m	2.2
W_G	Weight of the generator	t	8

Figure 40. Main dimensions of the Kaplan unit



8.2.5.3 Hydro and electromechanical equipment of the powerhouse

The plant equipment includes:

- Two security valves, butterfly type, equipped with counterweight as an emergency closing mechanism in the event of the loss of the grid
- Two double regulated Kaplan turbines
- Two low voltage synchronous generators
- Two Step Up LV/MV transformers and the connection to the MV switchboards
- The cabinets for control and monitoring systems, included the speed and voltage regulators, metering and relaying panels for each unit
- The power plant control and monitoring cabinet
- The cabinets for Low Voltage distribution
- The Electrical protections and safety systems
- One auxiliary LV transformer
- One DC power supply and an Emergency diesel auxiliary power generator
- Earthing and Lighting system with their protection

The following points should be studied at a later step of the project:

- Sediment issue and the requirement of anti-abrasion coating,
- Need for flywheel (network stability),
- Grid connection voltage

8.2.5.4 Net Head Calculation

The net head in the case of a Kaplan turbine must take into account the loss of energy corresponding to the remaining kinetic energy at the outlet of the draft tube. That speed depends on each manufacturer's design. It is in general close to 2 m/s at nominal discharge, value which we use in this study.

The net head can thus be expressed as follows:

$$H(Q) = \Delta Z - H_f(Q) - \frac{v^2}{2g} \qquad \text{[m]}$$

With:

- H(Q) : net head, function of the turbines' discharge [m]
- ΔZ : gross head [m]
- H_f (Q) : penstock frictions losses [m]
- v: speed at the outlet of the draft tube, [m]
- g: acceleration due to gravity [m/s^2]

The choice of 2 m/s as exit speed is a compromise between the kinetic energy recovered in the draft tube and flow conditions at turbine exit. At this prefeasibility stage, we will moreover consider that kinetic loss remains constant in the draft tube, whatever the turbine's discharge.

The Kaplan turbine is a reaction turbine, then no dewatering of the runner is needed. The level to be considered for the suction height is the tail water level.

The gross head is the difference between the upstream water level in the forebay tank and the downstream water lever in the tailrace channel. The floor elevation of the powerhouse is 1'165 masl. At the prefeasibility step, the tail race water and floor levels are not accurately defined. Then the tail water level is considered as equal to the powerhouse floor level.

The water level in the forebay tank is 1'190.9 masl, and thus the gross head is 25.9 m. Taking into consideration the head losses between the forebay tank and the inlet of the turbine spiral case and the kinetic energy losses at the outlet of the draft tube, the net head at nominal discharge is then equal to 23.6 m.

8.2.5.5 Overview of the units operation

The turbine governor will be controlled by the forebay tank level and the frequency measurement. The units operation is as follows:

- If the available discharge is lower than the minimal discharge of one turbine, the plant is in shutdown state;
- As long as the available discharge is between the minimal and maximal discharge of one turbine, only one unit is operating;
- If the available discharge is over the maximal discharge of one turbine and the demand is exceeding one unit capacity, a second unit is started. The discharge of the first turbine is reduced and the discharge of the second one increases until both turbines operate at the same opening.
- Then, the two turbines can operate in parallel with the same opening until they reach their maximum power.
- If the available discharge is larger than the maximal discharge of the hydropower plant, the excess water is released in the river at the intake location.
- If the discharge decreases, the automatic control system reduces the opening of the turbines in reversed order.

In case of shutdown of one or more turbine, the excess water is released in the river at the intake location.

The frequency regulation is used any time to adapt the production to the demand.

The forebay tank reference water levels to start or stop the units is set to avoid hysteresis.

8.2.5.6 Kaplan turbines

The preliminary design presented in chapter 8.2.5.2 is based on the consultant's database. It is given for information only and may vary from one manufacturer to another. The turbines performances and characteristics (rotational speed, efficiency guarantees, reliability, etc.) are realistic as long as the turbines are designed and manufactured on the basis of a hydraulic profile issued from laboratory tests and developments.

Considering the head, a spiral case configuration with vertical axis is chosen. The runner blades and turbine guide vanes actuators are preferably hydraulic. In case of emergency, for instance during a load rejection event, the runner blades are fully opened when the guide vanes are closing. The actuators of the runner blades and guide vanes must operate in the event of a power failure.

8.2.5.7 Generators

The main characteristics of the generator are presented hereafter:

Parameter	Value
Number of units	2
Type	Three phase, Synchronous
Axis	Vertical
Frequency (Hz)	50
Rated output (kVA)	1'270
Rated Power factor (-)	0.9
Rated Voltage (V)	Preferably 690 V or 400 V
Rated speed (rpm)	600
Maximum runaway speed (rpm)	~1'700
Primary coolant	Air
Index of protection	IP 23 or above
Insulation class	F (design), operating B class

According to the maximal power, the generator shall be designed with a self-ventilating open air cooling system.

The efficiencies of the generators were assessed from a data base collected from recognized generator manufacturers, with a particular emphasis on the rated power and rotational speed parameters.

As standard generator are made for a setting up to 1000 masl, particular attention should be paid to the **altitude of the powerhouse (1'165 masl) in further studies and possibl**e tender documents preparation.

8.2.5.8 Overhaul and safety valve

Each turbine shall be protected by a safety valve. It could be a DN 1300 butterfly valve with PN 6 or PN 10.

This valve can be used in case of maintenance and as a safety device in case of emergency shutdown. It opens with a hydraulic actuator and closes by counterweight.

8.2.5.9 High Pressure Unit (HPU)

Each unit will have its own High Pressure Unit to drive the guide vanes, the runner blades and the safety valve. It will include one hydraulic bladder in case of high pressure pump failure.

8.2.5.10 Control and monitoring system

The plant operation being expected to be entirely automatic, its control and monitoring system has to be as simple as possible, so as to reduce human intervention to a minimum.

The discharge will be controlled by the water level in the forebay tank, which will be measured by mean of a level gauge connected to the plant by optical fiber or other means.

Each unit will have its own control and monitoring cabinet with its own PLC. One additional control and monitoring cabinet and PLC will be installed to control the whole power plant.

It will be possible to operate the units either automatically or manually. In order to prevent untimely operations, manual controls must be locked with a key.

The plant will restart automatically in case of power outage. However, for safety reasons both with regards to the plant's operation and maintenance staff and to the electric grid, the plant will not restart automatically after an alarm, even if it would disappear without human action.

The electric cabinets will at least include the following elements: runner blade and guide vanes, opening, safety valve opening, Power Factor regulation, voltage and frequency controls, and emergency power supply.

The following measurement instruments will be used: Grid and generator voltmeters, wattmeter, frequency meter, power factor measurement, synchroscope, speed sensor, headrace level, hours counter, start-up counter, bearings and alternator coils' temperatures, emergency shut-down, emergency power-supply charge level.

The following alarms will have to be considered: Insufficient water level, insufficient head, too low or too high frequency, alternator overload, overspeed, emergency shut-down, start-up fault, bearing defect, coils defect, current return, battery overload, battery defect.

The plant could be remote-controlled.

8.2.5.11 Emergency power-supply

A 48, or 110 V emergency power supply consisting in batteries, battery chargers, inverters, load indicators, protections, etc., will insure safety in case of power failure. Battery alarms for defects or overloads will be transmitted to the power plant control system. Under normal conditions, the emergency power supply will be powered by the low tension grid. The energy storage must be sufficient to ensure a safe turbine shutdown.

An emergency diesel set will maintain power supply to essential feeders of the power house, weir and intake and eventually to enable black start of the HPP (to be studied in the feasibility studies).

8.2.5.12 MV transformer and switchboard

Each generator will be connected to a step up transformer enabling the outlet voltage to be increased to 30 kV. The main specifications of the LV/MV transformer area:

Number of units	2
Type	Dry
Rated Power (kVA)	1'400
Number of phases	3
Primary voltage (V)	400 or 690
Secondary voltage (V)	30'000

On the medium voltage side of the power transformers, a single 30kV/630A circuit breaker will be installed for each generator. This circuit breaker will be strong enough to stand the continuous operating current, as well as the peak short circuits. Its mechanism will allow the interruption of the short circuit to avoid any damage to the transformers, generators, and other electrical equipment.

8.2.5.13 Auxiliary Transformer

Ancillary services of the hydropower plants will be supplied by an auxiliary LV transformer with the following characteristics:

Number of units	1
Rated Power (kVA)	250
Number of phases	3
Primary voltage (V)	30'000
Secondary voltage (V)	400 or 690

A circuit breaker shall be installed to protect the auxiliary equipment.

The own consumption of the powerplant could be estimated roughly to 0.5% of the generated energy. This consumption is not taken into account in the energy production calculation and must be included as an expense in the financial analysis.

8.2.5.14 Overhead travelling crane

The power house will be equipped with an overhead crane that will be able to carry and place turbines, generators and other large devices during construction and maintenance operations.

8.2.5.15 Abrasion

Solid transport is expected to be high, especially during the wet season. The scheme is equipped with a desilting structure to limit or remove most of the sediment. It is recommended that the feasibility study includes a solid transport study. According to the results of this study, especially the composition of the transported particles, the decision to add a protective coating for the critical turbine parts could be adopted.

8.2.6 Power and energy generation performance assessment

The yearly electricity production is calculated by compiling the energy generation according to the flow duration curve and using the following expression:

$$E_{etot} = 10^{-3} \int \rho \; g \; Q_t \; \eta(Q_t) \; H(Q_t) \; dt \quad \text{[kWh/year]}$$

Where E_{etot} = total yearly energy production [kWh/year]

ρ = water specific weight [kg/m^3]

g = acceleration due to gravity [m/s^2]

$\eta(Q_t)$ = Overall unit efficiency, product of turbine, generator and transformer efficiencies, function of discharge [-]

$H(Q_t)$ = Net head, function of global discharge of the power plant [m]

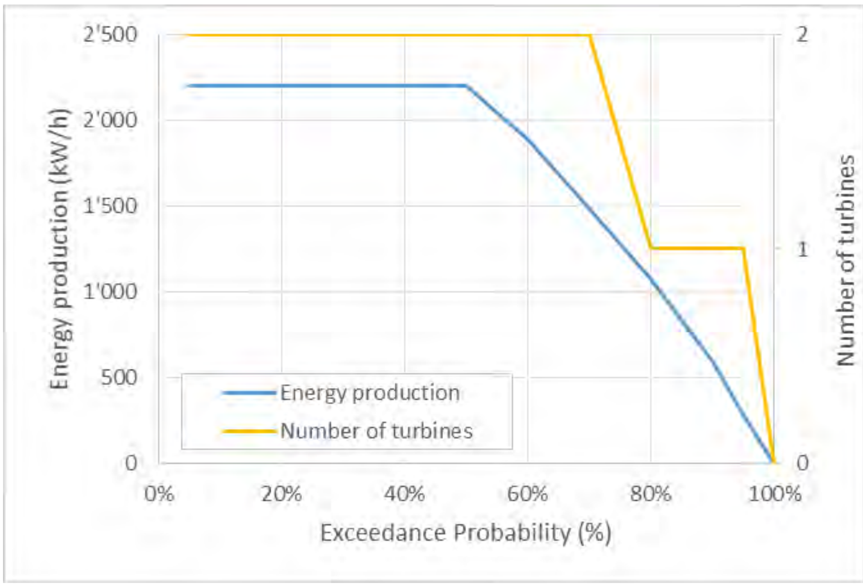
The used turbine efficiencies come from statistical curves based on real turbines of similar type, power and specific speed, taking into account the head and discharge variation.

The used generator efficiencies come from statistical curves based on real generators and taking into account the influence of the generator type, the rated output and the number of poles.

The rated efficiency of the step-up transformers is slightly higher than 99%. The efficiency of the power transformer used in the annual energy generation is considered constant and equal to 99%, independently of the load.

With the high flexibility of 2 double regulated Kaplan units, the overall efficiency is high and with small variation. Most of the time 2 units are in operation. It is the result of the design flow at $Q_{50\%}$.

Figure 41. Energy production and number of turbine versus the probability of time



The potential annual energy production is 15 GWh per year.

The accuracy of the estimation of the energy production depends mainly from the accuracy of the hydrology.

The optimization of the production must be made in further studies taking into consideration the choice of the design flow, the turbine type and the number of units.

8.2.7 Powerhouse

The hydropower plant will be positioned on the right riverbank. A truck access road should be built to allow the delivery of the turbine / generator units. A platform will also have to be constructed to allow the maneuvering of long vehicles. For that purpose, a bridge crossing the Muyovozi River will be required. Further details are given in section 8.2.9 below.

The power plant floor elevation is determined so as to ensure that it remains above flood level. However, the equipment requires a minimum downstream level to ensure their operation. The tailrace canal will discharge the turbined outflow to the river downstream of the power station. It will have a length of 10m.

The plant will consist of 2 + 1 bays, one per unit and one bay for assembly / dismantling. One floor is provided for offices, toilets, control room and meeting room. The area under the offices will allow the storage of tools and spare parts. A backup generator will also be placed there. The height of the plant will be governed by the size of the highest of the parts to be handled and by the characteristics of the crane. The dimensions of the plant, estimated at 10m wide, 20m long and 8m high, will have to be refined in subsequent studies.

For safety reasons (fire hazard) the transformers will be positioned in the immediate vicinity of the plant in a separate room.

The characteristics of the plant are given in the following table:

Table 19. Characteristics of the powerhouse

PARAMETER	UNIT	VALUE
Water level in the forebay	m	1190.85
Elevation of the power house floor	m	1165.0
Tailwater elevation	m	1160.0
Powerhouse length	m	20
Powerhouse width	m	10
Powerhouse height	m	8
Tailrace canal length	m	10.0

Given the site configuration at the tailrace area, it may be worthwhile to recalibrate the riverbed as shown in Figure 42 in order to:

1) Facilitate flowing at the tailwater;

2) Facilitate the flow of the river in case of flood and consequently decrease the level of water in the river;

3) Avoid the accumulation of rocks and other solid debris carried by the river during flood events at tailwater zone.

Figure 42. Tailwater zone to be reshaped

8.2.8 Transmission line and substation

The mini-grid of Kibondo is currently supplied by a 2.5 MW diesel-fired power station operated by TANESCO. Hence, the proposed Muyovozi hydroelectric project (2.27 MW) is a relevant alternative to the (costly) energy generation by that thermal power station.

The connection of the Muyovozi hydroelectric project and Kibondo would require the construction of an approximately 60km long high voltage (33kV) transmission line.

However, the Power Supply Master Plan (2016) proposes the construction of a 400 kV transmission line between Nyakanazi and Kigoma at horizon 2020. As a consequence, the required length of the transmission line to evacuate the power generated from the Muyovozi scheme could be much lower than 60 km, depending on the feasibility to connect directly to the 400 kV line with a dedicated substation.

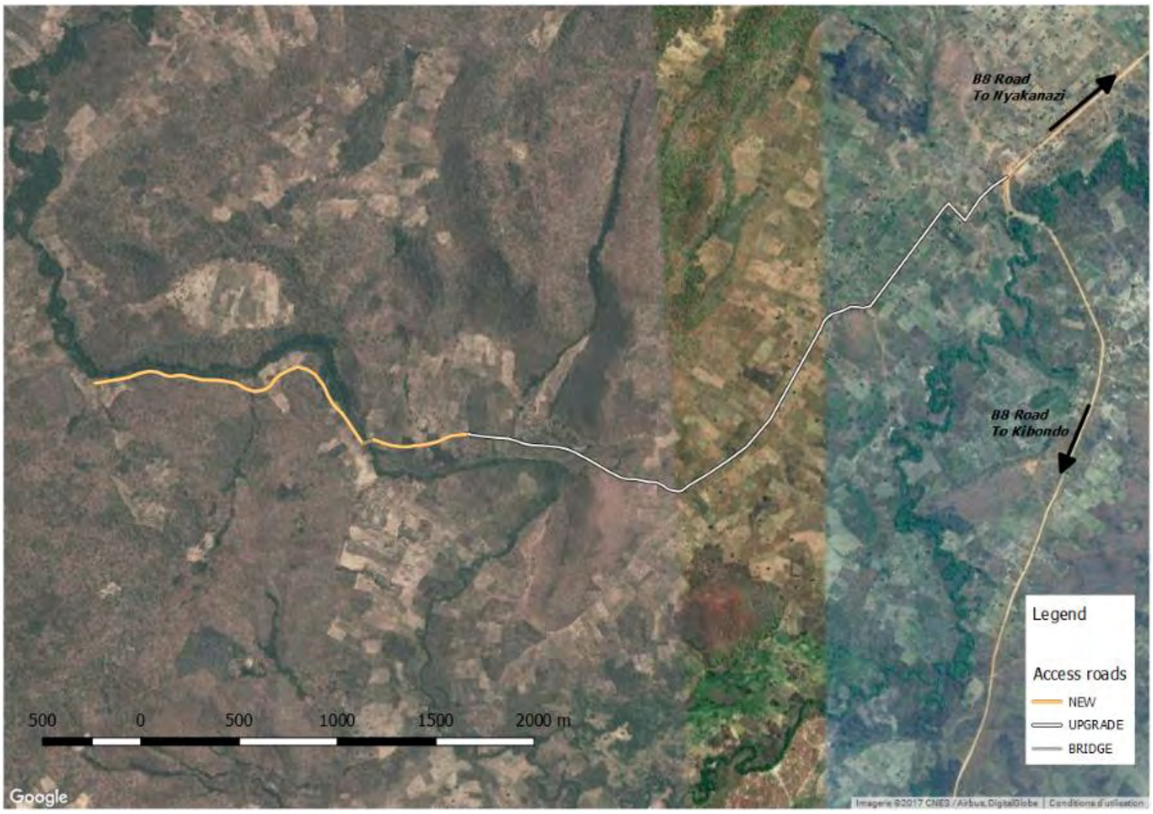
As the surroundings of the proposed project are currently not supplied by the electricity grid, the detailed studies shall analyze the technical and economic feasibility of supplying electricity to those villages directly from the power plant.

8.2.9 Access

A comprehensive description of existing access is presented and illustrated in Section 3.2 of this report.

For the development of the site, it will be necessary to create 2.2km of access track and a bridge over the river to access the proposed scheme located on the right riverbank. The proposed new track will connect the existing track, the powerhouse and the weir, following the headrace canal. It will also be necessary to rehabilitate the track between the B8 Road (currently under rehabilitation) and the proposed new access track. The different accesses to be rehabilitated and to create are illustrated in Figure 43 below.

Figure 43. Access to create and rehabilitate to access the proposed Muyovozi hydropower scheme



8.2.10 Temporary infrastructure during the construction period

Temporary infrastructure includes:

- Construction camp.
- Construction works areas (e.g. concrete batching plant, cable crane plant).
- Quarry locations.
- Site access roads

The construction camp is intended to accommodate allochthones workers working on the site. It will consist of accommodations, all the necessary sanitary facilities, a water treatment station and a wastewater treatment plant. This will serve both for the construction camp and for the permanent camp.

8.2.11 Permanent camp

The permanent camp will be located near the power station. It will consist of accommodations for the operators of the power plant as well as for the plant manager. The water treatment plants, constructed for the temporary camp, will also ensure the treatment of the waters of the permanent camp and the power plant.

8.3 KEY PROJECT FEATURES

Table 20 below summarizes the key features of the proposed layout of the Muyovozi hydroelectric scheme.

Table 20. Key features of the proposed scheme

Feature	Parameter	Value	Units
Location	Region	Kigoma	-
	River	Muyovozi	-
Hydrology	Watershed area	2 720.00	km²
	Median streamflow (Q50%)	12.04	m³/s
	Firm streamflow (Q95%)	2.12	m³/s
	Design flow	11.44	m³/s
	Design flood (100 years)	624	m³/s
Diverting structure	Structure type	Gravity weir (Overflowing section : Trapezoidal)	-
	Material used	Concrete	-
	Overflowing section crest length	40	m
	Total structure length	85	m
	Overflowing section height	3.00	m
	Non-overflowing section height	8.80	m
	Crest elevation	1 192.00	masl
	Slab elevation	1 189.00	masl
Gated flushing channel	Invert elevation	1 189.00	masl
	Number of bays	2.00	pce
	Gate section	1.6 x 2	m x m
Intake	Number of bays	4	pce
	Invert elevation	1 189.50	masl
	Equipment	Trash rack (manual cleaning)	-
Desilting structure	Yes		
	Number of basins	3.00	pce
	Water level	1 192.00	masl

Waterway		-	-
Canal	Headrace canal length	1 147	m
	Headrace canal section	2.8 x 3.2	m x m
	Average slope	0.001	m / m
Forebay	Yes	-	-
	Water level	1 190.85	masl
Penstock	Number of penstock(s)	1	pce
	Length	226	m
	Diameter	1.70	m
Powerhouse and electrical / mechanical works	Floor elevation	1 165.00	masl
	Gross head	27.00	m
	Number of units	2	pce
	Turbine type	Kaplan	-
	Operating discharge per unit	5.72	m³/s
	Total installed capacity	2 270	kW
	Average annual energy generation	15.00	GWh/year
Access road	Length of road to build	2 200	m
	Length of road to renovate	3 700	m
Transmission lines	Length	60	km
	Voltage	33	kV
Economic data	CAPEX - without access road and transmission lines	9.79	M$
	LCOE - without access road and transmission lines	0.09	$/kWh
	CAPEX - access road and transmission lines included	17.38	M$
	LCOE - access road and transmission lines included	0.16	$/kWh

9 COSTS AND QUANTITIES ESTIMATES

9.1 ASSUMPTIONS

At the prefeasibility study stage of a hydroelectric development, the assumptions detailed in the following paragraphs are commonly accepted.

9.1.1 Unit Costs

The list of unit prices comes from the Consultant's database which includes prices of contractors competent in hydraulic works and which can prove similar works carried out to international standards. This database is based on unit prices valid in Africa for infrastructure projects and updated for Tanzania.

Table 21. Unit prices (2017 USD)

CLASS	DESCRIPTION	UNITS	COST ($)
Excavation	Excavation (rock)	m^3	33.00
	Excavation (diverse)	m^3	17.00
	Excavation (soil)	m^3	6.00
Backfill	Random fill	m^3	9.00
	Compacted earthfill	m^3	13.00
	Rockfill	m^3	55.00
	Sand fill (pipe)	m^3	11.00
Concrete, stone and Masonry	Blinding concrete	m^3	165.00
	Mass concrete	m^3	330.00
	Structural concrete	m^3	550.00
	Concrete for weir	m^3	385.00
	Stone masonry	m^3	127.00
	Stone masonry (weir)	m^3	154.00
	Concrete bloc	m^3	165.00
	Rip-rap	m^3	33.00
Steel	Rebar	kg	2.00
	Structure	kg	6.00
	Roof	m^2	17.00
Access road	Access road (new)	m	380.00
	Access road (rehabilitation)	m	101.00
Transmission lines	33 kV Transmission line	km	81 070.00
Miscellaneous	Cofferdam	m^2	110.00
	Finishing (powerhouse)	package	95 370.00
Equipment	Electromechanical equipment	unit	798 600.00
	Penstock	m	1 711.00
	Overhead travelling crane	unit	74 800.00
	Trash rack	unit	31 680.00
	Flush gate (1.6m x 2m)	unit	77 110.00
	Intake gate (2.1m x 2.5m)	unit	115 500.00
	Drain gate (1m x 1m)	unit	44 770.00
	Desilting isolation gate (2m x 4m)	unit	169 620.00
	Isolation gate (2.6m x 2.8m)	unit	133 100.00
	Safety valve	unit	260 700.00
	Electrical equipment	package	407 000.00

9.1.2 Reinforcements and concrete

The reinforcements necessary for the realization of the structural concrete are taken into account in the concrete costs (at 250 kg of steel per m³). No reinforcement is foreseen in mass concrete (mainly used for the spillway).

9.1.3 Hydro and electromechanical equipment costs estimate

The considered equipment are:

- The hydro and electromechanical equipment: turbine, generator, valve, high pressure unit;
- The electrical equipment: power and auxiliary transformers, switchboard, control system and monitoring, power supply, protection system, cables, earthing, cabinets.

Global prices for the equipment are indicated in the Project costs estimates table, and are based on recognized cost estimate model (NVE 2016, Electrobras small hydro, Ogayar et al., B. Leyland). The selection of the appropriate models depends of the type of equipment, rated power, and experience/contract awards for small hydro projects in Africa, and especially in East Africa.

Reference projects in East Africa have been used to adjust the cost estimate.

For different reasons, as for instance, change of prices of raw material (such as steel, copper, etc.) or global small hydro market activity and manufacturing capacities, unexpected deviation from the proposed prices are possible. Nevertheless, cost estimate are taken as up-to-date and reliable enough for the purpose of the present level of the study.

The estimated costs take into account: equipment design and manufacturing, workshop acceptance tests, transport, mobilization, engineering, erection and commissioning; but it does not take into account unforeseen, taxes and duties.

9.1.4 Indirect costs

Indirect costs were estimated using fixed rates applied on different sub-totals of costs, as presented in the table below. Rates applied to Civil Works are higher than rates applied to Electrical and Mechanical Works as more uncertainties remain until the works have started.

Table 22. Indirect costs

INDIRECT COSTS	APPLIED RATE
Civil works contingencies	20% of civil works costs
Electrical and mechanical works contingencies	10% of E-M costs
Engineering (including ESIA), administration and supervision of works	10% of total costs
Owner’s development costs	2% of total costs

9.1.5 Site facilities costs

Costs for the Contractor site facilities and housing depend on the size of the project. Hence, this cost is taken as 10% of the total civil works costs.

9.1.6 Environmental and Social Impact Assessment Mitigation Costs

At this stage of the study and given the conclusions of the preliminary socio-environmental study, 3% of the total project costs are planned for the Environmental and Social Impact Assessment and mitigation (ESIA costs). This amount shall cover:

- Expropriation costs (compensation or allocation of new land);
- Mitigation cost of environmental impacts.

These costs should be specified in the full Environmental and Social Impact Assessment Study which will be carried out at a later stage of the project development. The costs of this study are taken into account in the indirect engineering costs presented in the previous section (section 9.1.4).

9.2 TOTAL COSTS (CAPEX)

Table 23 below presents a summary of costs for civil works and electromechanical equipment. It also includes indirect costs related to studies, site supervision, project administration and environmental and social mitigation measures.

Table 23. Project costs estimates (2017 US$)

Item	(%)	Costs ($)
Civil Works		11 279 000
Mobilization, installation, demobilization		275 000
Access		1 465 000
Dam/weir, spillway, purge and intake		805 000
Waterway (headrace channel, silting basin, forebay and penstock)		3 298 000
Powerhouse and tailrace channel		572 000
Transmission line		4 864 000
Electromechanical equipment		3 495 000
Electromechanical equipment		1 597 000
Hydro mechanical equipment		909 000
Electrical equipment and ancillaries		407 000
Transport	10%	291 000
Installation	10%	291 000
Sub Total (excl. contingencies)		14 774 000
Contingencies		2 606 000
Civil works contingencies	20%	2 256 000
Equipment contingencies	10%	350 000
Total direct project cost (incl. contingencies)		17 380 000
Indirect Costs		2 608 000
Social and environmental mitigation costs	3.0%	522 000
Administration fees	2.0%	348 000
Studies (incl. EIES) and works supervision	10.0%	1 738 000
Total cost of the project		19 988 000

10 ECONOMIC ANALYSIS

10.1 METHODOLOGY

The economic analysis is based on the results of the field investigations and various studies presented in the previous chapters, which includes an estimate of the quantities and the construction costs of the project (Chapter 9) and the definition of the installed capacity and power output. Based on these results, the Consultant has estimated the cost to deliver energy from the development of the Muyovozi hydroelectric project.

The energy generation alternatives (currently thermal units, fossil fuel-fired) will be compared based on their costs per kWh, the latter being expressed in terms Levelized Cost Of Energy (LCOE) which is a stream of equal payments, normalized over the expected energy production periods that would allow a project owner to recover all costs, an assumed return on investment, over a predetermined life span.

The LCOE is defined from investment costs (CAPEX - Capital Expenditure), operating costs (OPEX - Operational Expenditure) and the expected production of energy.

Investment costs are:

- Study and work supervision costs, hereafter called “Studies and engineering costs” which include:
    - Civil works study and supervision costs
    - Electromechanical works study and supervision costs
    - Owner’s development costs
- Civil works and equipment costs, hereafter called “HPP costs”
- Resettlement and environmental impact costs, hereafter called “ESIA costs”

Annual operating costs are:

- Operation and maintenance costs, hereafter called “O&M costs” which include:
    - Fixed operation and maintenance costs (annual scheduled maintenance)
    - Costs related to interim replacement and refurbishments of major items in the course of the project’s life
    - Insurance costs

The LCOE is then calculated based on expected production and costs from the following formula:

$$LCOE = \frac{NPV(CAPEX + OPEX)}{NPV(Energy\ production)}$$

Where *NPV* is the Net Present Value which is obtained by: $NPV(value) = \sum_i \frac{valuer_i}{(1+n)^i}$ where *n* is the discount rate.

10.2 ASSUMPTIONS AND INPUT DATA

The main economic assumptions for the economic modeling of the *LCOE* calculation for the Muyovozi hydroelectric project are presented in Table 24 below.

Table 24. Economic modelling assumptions

Parameter	Value
Economic lifespan of the project	30 years
Decommissioning cost at the end of the economic life	10% of civils works and equipment costs
Engineering (incl. ESIA) and works supervision **Owner's development costs**	10% of civils works and equipment costs 2% of civils works and equipment costs
Environmental and social impact mitigation costs	3% of civils works and equipment costs
O&M costs Interim replacement Fixed operation costs	 0,25%/year of civils works and equipment costs 10 USD/kW/year
Insurance costs	0,10% of civils works and equipment costs per year
Distribution of costs over the project implementation process	Year -2 = 60% Year -1 = 40% Year 0 = Commissioning
Reference date for economic analysis	2017
Costs are expressed in constants	(2017) USD
Escalation costs (inflation)	No escalation costs were applied to capital costs or operating costs.
Financing costs etc.	Financing costs, tax, duties or other Government levees are ignored at this stage but shall be included in the financial analysis that will be done during the detailed studies.
Discount rate	10%

The economic analysis is carried out by considering that all the energy produced is absorbed by the electricity grid. In other words, the analysis assumes that there is a demand for all the energy generated by the proposed hydroelectric scheme.

10.3 Economic Analysis and Conclusions

Table 25 presents the levelized costs of energy (LCOE) for the Muyovozi site.

Table 25. Levelized Cost of Energy (LCOE)

	Annual Energy [GWh]	Installed Capacity [MW]	Design Flow [m³/s]	CAPEX [M USD]	LCOE [USD / kWh]
Without Transmission lines and access roads to be rehabilitated	15.0	2.27	11.4	9.79	0.09
With Transmission lines and access roads to be rehabilitated				17.38	0.16

The economic analysis reveals that the proposed Muyovozi hydroelectric scheme is an economically attractive project with a LCOE of 0.0897 $US/kWh (excluding the costs of transmission lines and access roads). Indeed, the costs of transmission lines will be significantly reduced in the near future with the construction of the 400kV transmission line between Nyakanazi and Kigoma at horizon 2020, as proposed in the Power Supply Master Plan (2016).

The mini-grid of Kibondo is currently supplied by a 2.5 MW diesel-fired power station operated by TANESCO. Hence, the LCOE of the Muyovozi project must be compared with the cost of energy production from the thermal power plant currently in operation since the Muyovozi project would replace the production of thermal energy by hydroelectricity. The energy generation cost from thermal power plants depends largely on the fuel costs. As outlined in the SREP-Investment Plan for Tanzania fuel cost from diesel-fired thermal power plant is expected to exceed 0.35 US$/kWh[1].

The LCOE of the proposed Muyovozi hydroelectric project is attractive when compared to the 0.108 US$/kWh corresponding to the standardized small power projects (SPPs) tariff for hydro between 2MW and 3MW in 2016. The latter is the tariff for SPPs selling bulk power to the national or a regional grid or to DNO-Owned Mini-Grids.

It is important to note that the conclusions of this economic analysis are conditioned to the validation of the flow duration curve estimated in the hydrological study. This validation can only be done by the hydrological monitoring of the Muyovozi River. The hydrological monitoring should include not only the continuous recordings of water levels but also gauging operations of the river for the establishment of validated rating curves.

[1] Source : SREP - Investment plan for Tanzania

11 CONCLUSIONS AND RECOMMANDATIONS

The hydrological study revealed that the Muyovozi River is characterized by a good guaranteed low-flow which should be confirmed by hydrological monitoring of the River.

The preliminary investigation of the surface geology concludes that the site is favorable for the construction of the project as long as appropriate mitigation measures are put in place. The site has no major problems of stability and leakages. Further investigations will however have to be undertaken in further studies.

Preliminary socio-environmental studies show that the development of the Muyovozi project has no major impacts that cannot be mitigated by appropriate measures.

The economic analysis reveals that the construction costs of the 33kV transmission line to Kokonko mini-grid are high. However, those costs will be significantly reduced in the near future with the construction of the 400kV transmission line between Nyakanazi and Kigoma at horizon 2020, as proposed in the Power Supply Master Plan (2016). The Muyovozi hydroelectric project is an economically attractive scheme with a LCOE of 0.0897 US$/kWh if the costs of the costs of the transmission line and access roads are excluded. The Muyovozi Project features a production costs significantly lower than the standardized small power projects (SPPs) tariff for hydro between 2MW and 3MW, as approved by EWURA in 2016 (0.108 US$/kWh).

It is important to note that the conclusions of this economic analysis are conditioned to the validation of the flow duration curve estimated in the hydrological study. This validation can only be achieved by hydrological monitoring of the Muyovozi River at the hydrometric station a few kilometers downstream from the proposed project site. This hydrological monitoring should include not only the continuous water level monitoring but also the gauging operations of the river for the establishment of a validated rating curve.

Beyond the development of the Muyovozi hydroelectric project, it is strongly recommended that the Government of Tanzania further develop the existing hydrological monitoring network for its rivers with high hydropower potential in order to better understand the available water resources and thus promote the development of hydroelectric projects across the country. It is only in a context of reduced uncertainties through reliable, recent and long-term records (more than 20 years) that technical parameters and economic and financial analyzes of hydroelectric developments can be defined accurately, enabling optimization of their design and their flood control infrastructure (temporary and permanent).

12 APPENDICES

12.1 DETAILED PROPOSED SCHEME AND MAIN COMPONENTS