OCCASION

This publication has been made available to the public on the occasion of the 50th anniversary of the United Nations Industrial Development Organisation.

DISCLAIMER

FAIR USE POLICY

CONTACT

Please contact publications@unido.org for further information concerning UNIDO publications.

For more information about UNIDO, please visit us at www.unido.org

UNITED NATIONS INDUSTRIAL DEVELOPMENT ORGANIZATION
Vienna International Centre, P.O. Box 300, 1400 Vienna, Austria
Tel: (+43-1) 26026-0 · www.unido.org · unido@unido.org

API Renewable Energy LLC

Combining the Energy of Human Hope with New Greener Economy

UNID PROJECT: TF/UGA/08/004

CONTRACT NUMBER: 16002124/RE

FEASIBILITY STUDY FOR ESIA RIVER MINIHYDRO POWER PLANTS TO REDUCE THE VULNERABILITY OF THE POOR POPULATION TO CLIMATE CHANGE IMPACTS BY PROVIDING ECONOMIC EMPOWERMENT

API

2011

API RENEWABLE ENERGY (UGANDA):

PLOT 696/679, BUNGA, GABA ROAD
P.O. BOX 14299, KAMPALA, UGANDA

TEL: +256 414 66 2547

FAX: +256 414 66 2548
MOBILE: +256 715 911 600
Email: info@api.co.ug

API RENEWABLE ENERGY (UGANDA):

BS 8110 HAS BEEN USED FOR ALL CIVIL ENGINEERING STRUCTURAL DESIGN CALCULATIONS

USBR STANDARDS USED FOR HYDRAULIC DESIGN COMPUTATIONS

API 1104 STANDARD IS USED FOR PENSTOCK AND OTHER STEEL STRUCTURAL DESIGNS AND DESIGN VERIFICATIONS

ASME CODE IS USED FOR ELECTRO-MECHANICAL INSTALLATIONS

THE GEOGRAPHICAL POSITIONING ELEVATIONS ARE SHOWN ON THE FLAT PLAN OF THE GENERAL ARRANGEMENT DRAWING. THESE VALUES ARE BASED ON MEAN SEA LEVEL (MSL) IN METERS.

Contents

Table of Figures

LIST OF TABLES

EXECUTIVE SUMMARY

The feasibility analysis of Esia River hydro power projects has been prepared to provide the detailed analysis on the viability of the project including the geographical, geological and hydrological analysis. Upon which its construction methodologies will be assessed and designs has to be carried out and finally the project feasibility will be determined by the project financial analysis.

The proposed mini hydropower project is intended to take water from Esia River, a tributary to the Albert Nile, in Adjumani district in Northern Uganda. Adjumani (whose location is between latitude $2^0$55N and $3^0$35N, longitudes $31^0$25E and $32^0$05E) is located in the north western Uganda bordering the Republic of Sudan in the north, Gulu district in the east and south, Arua district in the West and Moyo district in the North West and North.

Initial site investigations were carried out during the months of December 2010 and March 2011, followed by desk calculations, laboratory investigations, modeling etc prior to the compilation of first draft report. Basic outline of the project includes a diversion weir, headrace channel, forebay tank, penstock, power house with electro-mechanical equipments, transmission and distribution network. The hydrological study on the river flows has been carried out based on rainfall figures obtained from rain gauging stations located within the catchments of river Esia.

A comprehensive geological study has been carried out by a professional geologist, based on the data from field investigations and resulted with observation of the existence of most stable formation of an alternation of granite and gneiss tics granite. This geological study reveals the suitability of the site for the project and recommendations have been provided in order to increase the stability of the soil. A detailed investigation on the catchments erosion and silting has been carried out as river water contain considerable amount of silt due to upstream catchments erosion.

RIVER ESIA MINI HYDRO PROJECT SITE

Figure 1: Uganda showing the position of Adjumani district (River Esia Hydro Project Site)

TOPOGRAPHICAL OVER VIEW OF RIVER ESIA

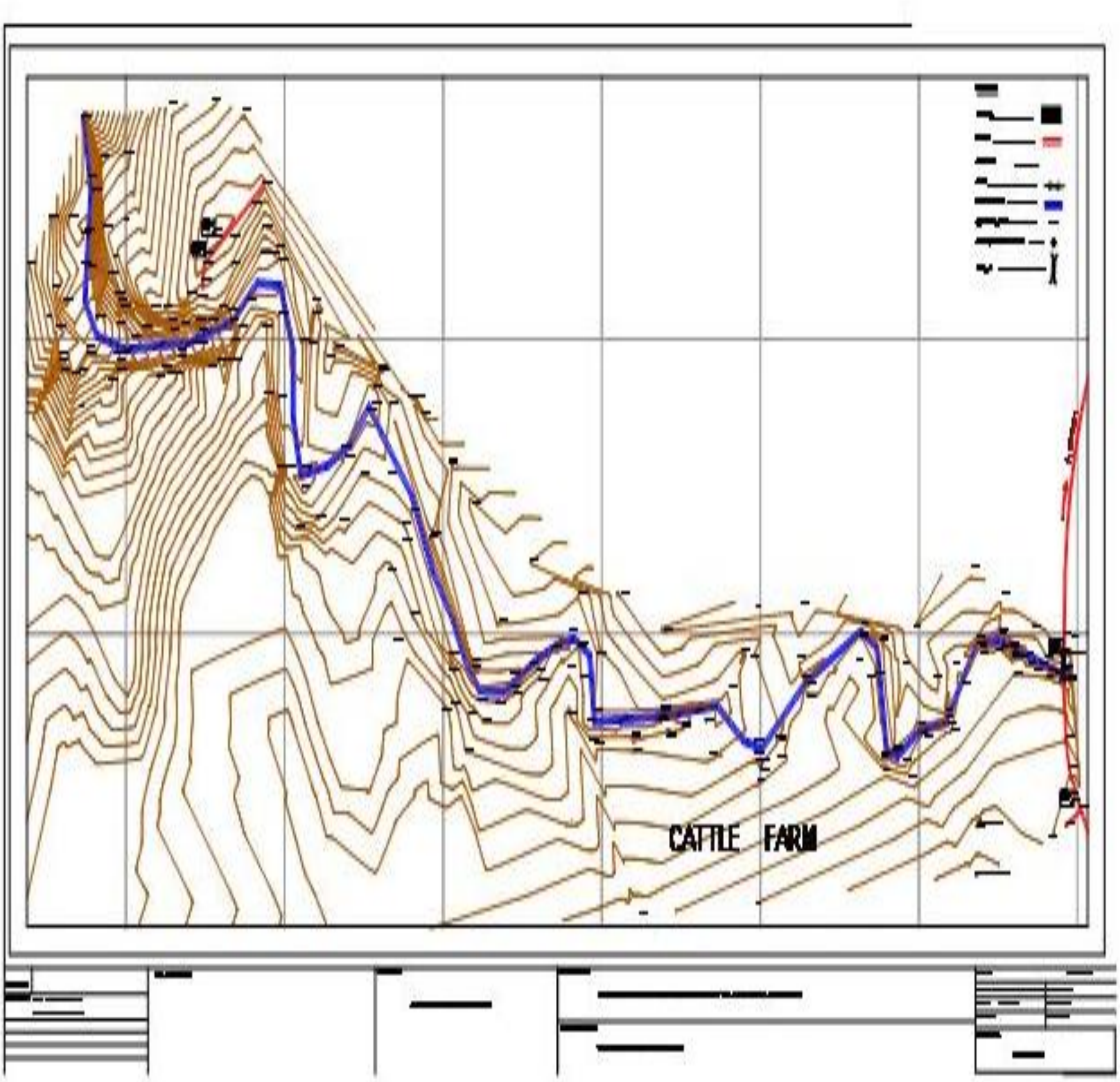


Table 1: Salient Features: Esia River, Adjumani District

Location	
East	:29°16" to 29°32"
South	:2°22" to 2°35"
River	:Esia
Type of project	:Mini Hydro
Hydrology	
Catchment area	:500 acres
Design flow	: $0.5m^3/sec$
Head	
Net head	:31.71 m
Weir	
Type	: Clear overflow
Length	: 20 m
Height	: 2.5 m
Concrete canal	
Type	:Rectangular
Length	:1.8 km
Width	:0.85 m
Height	:0.6 m
Forebay	
Length	:11, 00 m
Width	:1.8, 00 m
Height	:3.2, 00 m
Penstock	
Type	: Steel pipes
Length	: 410 m
Thickness	: 4mm
Diameter	: 610 mm

Power and energy	
Installed capacity	:110 KW
Turbine	
Type	:Francis
Number of unit	:1
Rated capacity	:1,500 trs/min
Generator	
Type	:Synchronous
Number of unit	:1
Rated capacity	:1,500 trs/min
Power transformer	
Type	: Outdoor, oil cooled
Number of unit	: 1
Rated capacity	: 0.40KV/33KV
Financial parameters	
Project cost, USD$	: 2,603,503 (excluding VAT)
Annual energy produced, KWh	: 727,700
Development Cost p/KWh	: $23,668
Financial Project NPV	: ($1,666,253)
Financial Project IRR	: (4.70%)
Economic Return Rate (Annualized)	: 5.07%
Project Payback	: N/A
Recommendation	: Proceed; Co-op format; Off-grid

Salient Features: INFRASTRUCTURE

Power house

Type	:Stone masonry
Floor size	: 84.65 m^2

Access road

Rehabilitation of existing road	: N/A
New access	: 2.25 Km

Substation

Type	:Compact
No of cellule	:7

Transmission line 30 KV **:**18

1.0 HYDROLOGICAL REPORT ON ESIA RIVER MINI HYDRO PROJECT

1.1. Introduction

1.1.1 Background

The amount of electrical energy that can be generated with a hydropower plant is directly proportional to the product of flow and available hydraulic head. This makes the knowledge of river discharge, its annual distribution and long term variability very essential for the planning and design of a hydropower plant. The long term river discharge records are especially very vital for the selection of the design discharge. These discharge records are not always available, however most of the time long term rainfall records may be available within and/or around the catchment of interest. Hydrology present opportunities for modeling the possible river discharge using rainfall records. The modeled discharges can be presented in a form of Flow Duration Curves (FDC) which in turn can be used in selection of the design discharge.

Hydrologic systems are sometimes impacted by extreme events, such as severe storms, floods, and droughts. The magnitude on an extreme event is inversely related to its frequency of occurrence. In general very severe events occur less frequently than more moderate events. The knowledge of flood flow frequency are use for the technical and economic design of engineering structures such dams, weirs, bridges, culverts, flood control structures etc.

The design of a hydropower plant requires the construction of dam and/or weir. One of the basic requirements for the design of dams and weirs is the knowledge of flow exceedence probability that is the maximum flood with which the structure is designed. Hydraulics structures such as dams and weirs are expected to work efficiently without problem of failure or damage for a flood less than or equal to the design flood. In the event of a flood that exceeds the design flow the probability of failure or damage of the structure is possible. The maximum floods that can be expected in a river in the absent of river flow data when the long term rainfall records is available can also be modeled using hydrologic methods. Depending on the method to be use the modeling hydrologic respond of a watershed at the absent of river flow data may require some of the followings: catchment area, rainfall, topography, soil data, and use and land cover etc.

1.2. Objective

This study was carried out with the main objective of determining the basic hydrologic parameters necessary for the design of hydropower plant at River Esia a tributary to the Albert Nile, in Adjumani district in Northern Uganda. This includes the determination of the followings:

a) Analyzing climate and precipitation data
b) Flow measurements
c) Determining the size of the catchment area and assessing its characteristics
d) Analyzing flood patterns
e) Correlating the meteorological data with flow data
f) Obtaining the flow duration curve
g) Investigating the amount of water required for purposes such as irrigation
h) Estimation of the design flow

1.3. Description of the project area

The proposed project site is on River Esia a tributary to the Albert Nile which lies in Adjumani District. Adjumani (whose location is between latitude 2^{0}55N and 3^{0}35N, longitudes 31^{0}25E and 32^{0}05E) is located in the north western Uganda bordering the Republic of Sudan in the north, Gulu district in the east and south, Arua district in the West and Moyo district in the North West and North.

The proposed hydropower project site is located along River Esia a tributary to the Albert Nile, in Adjumani District. The project area is geographically located as shown in the map in figure 2.1 below.

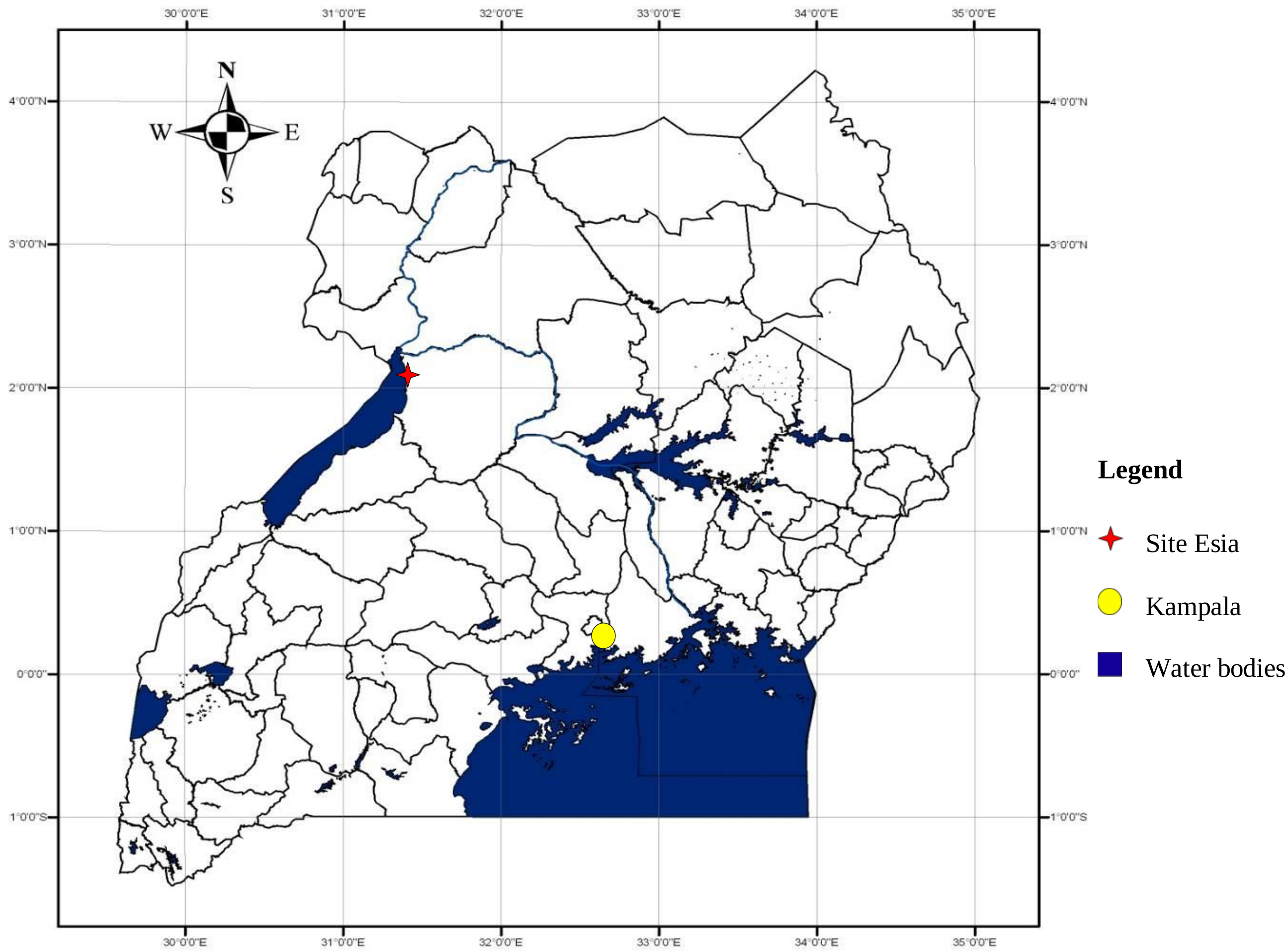


Figure 2: Geographical location of Esia hydropower project.

The proposed site is about 7km southwest of Adjumani town between GPS position 36N0353861, 0366727 (Bridge) and 36N0350894, 6367268. It is characterized by gentle falls over a stretch of a distance of about two kilometers with several visible rapids.


One of the rapids along the Esia River

1.3.1. River description

River Esia can be generally be described as a permanent river (flows throughout the year), although it experiences wide flow variations between the wet and dry seasons. According to information gathered from the locals, the water volume of the river swells to almost 10 times the size it normally holds in the dry season.

The water carried in the river is generally clean with no debris and rotting vegetation. Due to the extensive cultivation along the river banks, it is safe to assume that the river carries a large amount of silt or sediment during the wet season (due to surface runoff and erosion of the river banks and nearby gardens).

In the proposed section of the river for development of the power project, between the proposed intake and tailrace, together with the proposed dam area, the river width varies between 6 and 10 metes while the depth is normally around 0.5 and 1.2 meters.

Figure 3: Pictures from the field showing river Esia

A further assessment of the site shows that, a flood plain of 4 meters width on either side of the river banks, with flood marks at heights ranging between 1 and 1.5 meters were observed. Figure 2.3 below shows the flooding extent of the river.

Figure 4: Picture showing the possible flood extent of the river.

1.3.2. Description of catchment area

The catchment area of river Esia is generally rocky, savanna with shrubs, low density of short trees in an area of approximately 500acres, gently sloping hilly terrain, with a wide River bed (due to large variations in river flow). In addition, deforestation has been carried out to a rather high degree, with few trees observed and more of grassland and bushes. We believe that these two factors contribute greatly to the large variation in river flow between the wet and dry seasons due to the massive evapotranspiration during dry seasons.

Figure 5: Picture of typical rocks found in the catchment area of the River Esia

A rocky catchment ensures a high level of surface runoff and therefore little or no seepage into the ground for storage of under-ground water. When such a scenario prevails, the river generally lacks any replenishment from ground water sources through streams. The lack of vegetative cover ensures that there is little ground water due to the direct heat from the sun.

The catchment can also be described as one with a rather gentle gradient, although there are some rather prevalent declines in some areas.

Figure 6: Picture showing the gentle gradient and the short trees in the Esia catchment

1.4. Available data

1.4.1. Rainfall and temperature data

Rainfall data has been obtained from the Uganda meteorology department. It is important to note that although some data is available, there are some years where gaps (where no data exists) have been noted. Adjumani weather station is the source of the weather data. There are three weather stations and at station 86310010, the available data is from 1943 to 1960 as well as from 1961 to 1965. There is also data for 1971.

Station 86310020 has records from 1940 to 1950. Data obtained from station 86310030 shows rainfall records from 1939 to 1978, although there are gaps in data for the years: 1941, 1952, 1960, 1969, 1970 and 1975.

In order to get a clear picture of the rainfall situation, correlation of the obtained records will be done with data from the neighboring district of Arua. Records from Arua are available from 1943 up to 2008, although there are some gaps as well.

The data shows that, the Esia project area is characterised by one major rainy season that starts from April, peaks in June and ends in august, while the dry season starts in September, peaks in December-February and ends in March.

Temperature data was also obtained. Data is available at station 86300101 in Arua from 1992 to 2008.

1.4.2. River gauging data

There is no gauged data on river Esia, although, the owners of a farm within the proposed project area took it upon themselves to measure the river stage / level at an existing bridge thrice a day from 2005 to 2008. This data has been obtained and it will be used as a check / verification of the flow duration curve that will be derived using the meteorological data. However, this data as it is now cannot be used to derive a flow duration curve.

1.4.3. Topographic maps

Topographic maps of the area at scale 1:50000 are available and have been obtained from the lands and surveys department.

1.4.4. Data analysis

1.4.5. Rainfall and temperature data

Records of the daily maximum and minimum temperature were analyzed. The following average values were obtained for the project area.

Table 2: showing average temperature.

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANNUAL AVERAGE
Daily mean maximum temperature [C]	30.805	32.13106	30.94388	29.22033	28.30963	27.63944	26.681	26.77506	27.87713	27.59747	27.88453	29.16647	28.75258299
Daily mean minimum temperature [C]	17.12594	17.966	18.64538	18.398	18.0175	17.395	16.89488	16.94963	17.00294	17.30213	17.3065	16.89163	17.49129167
Average [C]	23.96547	25.04853	24.79463	23.80917	23.16356	22.51722	21.78794	21.86234	22.44004	22.4498	22.59552	23.02905	23.12193733

Rainfall data was also analyzed. The results were as follows;

	STATION	Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Total
1	86310010	1943	2	18	34	143	191	224	25	142	174	70	13	4	1040
2	86310010	1944	10	5	42	81	313	96	194	118	134	109	135	5	1243
3	86310010	1945	7	0	10	76	160	156	167	185	170	159	90	53	1230
4	86310010	1946	0	0	63	241	369	322	151	163	92	161	56	25	1642
5	86310010	1947	11	18	76	135	143	137	231	154	191	38	35	42	1211
6	86310010	1948	2	24	44	89	122	166	129	285	209	125	53	6	1255
7	86310010	1949	0	1	16	89	112	101	286	197	194	111	15	89	1211
8	86310010	1950	0	7	35	215	168	142	221	168	69	122	15	0	1162
9	86310010	1951	4	0	14	131	135	103	104	90	66	244	223	57	1170
10	86310010	1952	0	35	33	218	143	82	193	205	202	206	48	0	1365
11	86310010	1953	0	10	32	87	138	98	179	264	124	134	23	3	1092
12	86310010	1954	2	55	63	109	162	74	115	152	128	146	3	10	1019
13	86310010	1955	23	1	29	84	40	78	53	250	298	258	141	33	1288
14	86310010	1956	4	23	48	149	129	203	126	83	248	246	51	14	1323
15	86310010	1957	33	13	124	124	267	316	86	202	173	48	29	10	1425
16	86310010	1958	25	138	120	184	90	198	147	79	134	110	68	83	1378
17	86310010	1959	19	0	49	117	50	68	67	182	102	52	87	0	794
			142	348	832	2272	2732	2564	2474	2919	2708	2339	1085	434	
Mean monthly rainfall			8.352941	20.47059	48.94118	133.6471	160.7059	150.8235	145.5294	171.7059	159.2941	137.5882	63.82353	25.52941	1226.411765

Table 3: showing rainfall analysis results at station 86310010

	STATION	Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Total
4	86310030	1943	10	8	26	84	175	225	87	209	125	65	12	21	1047
5	86310030	1944	0	3	45	87	271	131	218	133	174	133	79	31	1305
6	86310030	1945	13	0	36	55	123	63	126	115	235	88	119	0	974
7	86310030	1946	0	0	6	107	240	130	55	239	65	41	14	1	898
8	86310030	1947	0	4	89	135	41	45	168	74	166	48	29	41	839
9	86310030	1948	0	0	78	101	101	137	46	149	138	225	48	3	1026
10	86310030	1949	0	21	22	48	91	102	264	173	73	108	54	43	999
11	86310030	1950	0	6	45	102	123	151	149	131	100	115	13	2	936
12	86310030	1951	0	3	30	79	216	76	91	77	103	259	109	88	1132
14	86310030	1953	3	44	70	58	152	80	168	137	85	130	84	11	1021
15	86310030	1954	0	88	66	160	123	52	122	293	277	83	5	5	1275
16	86310030	1955	15	0	58	79	126	110	152	195	251	148	111	34	1279
17	86310030	1956	10	3	49	223	157	217	136	109	170	123	57	43	1294
18	86310030	1957	14	51	161	98	233	191	71	113	127	120	79	9	1268
19	86310030	1958	0	19	147	213	112	86	105	111	127	73	19	39	1051
20	86310030	1959	16	0	51	78	74	51	109	148	151	104	144	0	925
22	86310030	1961	0	4	200	58	72	124	176	177	341	201	136	30	1518
23	86310030	1962	0	0	47	134	153	94	63	137	153	70	108	64	1023
24	86310030	1963	1	34	31	140	127	75	54	55	61	77	80	15	750
25	86310030	1964	0	0	12	79	114	107	211	171	170	145	3	6	1018
26	86310030	1965	0	1	13	23	79	18	96	65	145	175	38	19	672
27	86310030	1966	5	28	24	106	56	64	148	145	247	122	66	0	1010
28	86310030	1967	0	0	20	44	171	134	45	207	120	171	126	0	1038
29	86310030	1968	0	24	23	109	116	111	112	89	68	173	36	45	904
32	86310030	1971	0	30	32	92	180	82	95	133	58	115	59	0	876
33	86310030	1972	30	41	31	25	119	142	119	224	196	220	33	21	1200
34	86310030	1974	0	11	29	40	146	40	109	69	100	53	7	0	604
36	86310030	1976	0	42	16	65	46	28	116	100	50	106	135	3	706
37	86310030	1977	10	5	36	133	125	94	163	153	92	182	69	8	1070
38	86310030	1978	0	56	53	109	47	142	242	102	150	191	41	21	1154
			127	526	1546	2864	3909	3102	3816	4233	4318	3864	1913	603	
Mean monthly rainfall			4.884615	20.23077	59.46154	110.1538	150.3462	119.3077	146.7692	162.8077	166.0769	148.6154	73.57692	23.19231	1185.423077

Table 4: showing rainfall analysis results at station 86310030

	STATION	Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Total
5	86310020	1943	0	1	32	121	154	152	42	210	95	43	43	0	893
6	86310020	1944	37	0	72	81	205	141	115	170	97	191	77	15	1200
7	86310020	1945	0	0	13	37	58	169	191	44	263	171	52	47	1045
8	86310020	1946	0	0	63	158	127	92	182	436	166	94	210	6	1533
9	86310020	1947	0	14	73	176	11	124	204	106	290	64	54	100	1217
10	86310020	1948	1	40	75	61	88	203	111	181	226	225	43	0	1253
11	86310020	1949	0	6	22	82	42	121	171	120	198	120	18	34	933
12	86310020	1950	0	0	140	120	123	123	142	260	233	128	0	0	1267
			38	61	490	836	808	1125	1158	1527	1568	1036	497	202	
Mean monthly rainfall			4.75	7.625	61.25	104.5	101	140.625	144.75	190.875	196	129.5	62.125	25.25	1168.25

Table 5: showing rainfall analysis results at station 86310020

From above, the area was found to have 1193mm as the average annual rainfall.

1.5. Derivation of the flow duration curve

In the absence of river gauging data (flow measurements from a gauging station), the consultant felt it wise to carry out spot measurements; and correlate these with calculations from the annual precipitation data.

Having obtained the annual precipitation, the actual annual run off was then derived. However, as stated earlier, the annual run–off is heavily affected by the rate of evapotranspiration and the ground characteristics of the area. The evapotranspiration is heavily dependent on the plant cover, as well as the prevailing temperature in the area. Run-off on the other hand depends on the nature of soils as well as the activities being carried out in the area – urban areas tend to have high levels of run-off due to the high construction levels. Below is the analysis leading to the derivation of the flow duration curve.

Mean Temperature[C]	23.12	Percentage Exceedance	No. of days of the year	Empiric coefficients	Discharge (l/s)	Standard Deviation	+'/'-' (l/s)
Mean Annual Rainfall [mm]	1193.4	5%	18	2.3	1207.20	0.32	167.96
Catchment Area[Sq. km]	183.25	10%	37	1.9	997.25	0.3	157.46
L	999.0334	20%	73	1.39	729.57	0.12	62.98
Annual Evapo-transpiration [mm]	1103.074	30%	110	1.12	587.85	0.09	52.91
Annual Run-off [mm]	90.32628	40%	146	0.92	482.88	0.08	38.63
Mean discharge [l/s]	524.8697	50%	183	0.77	404.15	0.08	41.99
		60%	219	0.64	335.92	0.08	41.99
		70%	256	0.53	278.18	0.09	47.24
		80%	292	0.44	230.94	0.09	47.24
		90%	329	0.35	183.70	0.09	47.24
		100%	365	0.1	52.49	0.09	47.24

Table 6: Flow duration curve analysis table

1.6. Verification of the obtained flow duration curve

In order to fully validate the obtained hydrological analysis / flow duration curve, spot measurements have been carried out using the salt dilution method, at the intake point. The following are the averaged results that have been obtained so far for the period between November 2010 to April 2011.

Period of measurements	Average Flow measured (l/s)	Mean monthly rainfall (mm)
Nov-December 2010	332.6	43.7
Jan-Feb 2011	183.2	6.2
March. 2011	156.3	5.3

Table 7: Spot measurements (averaged) using the salt dilution method at the intake point.

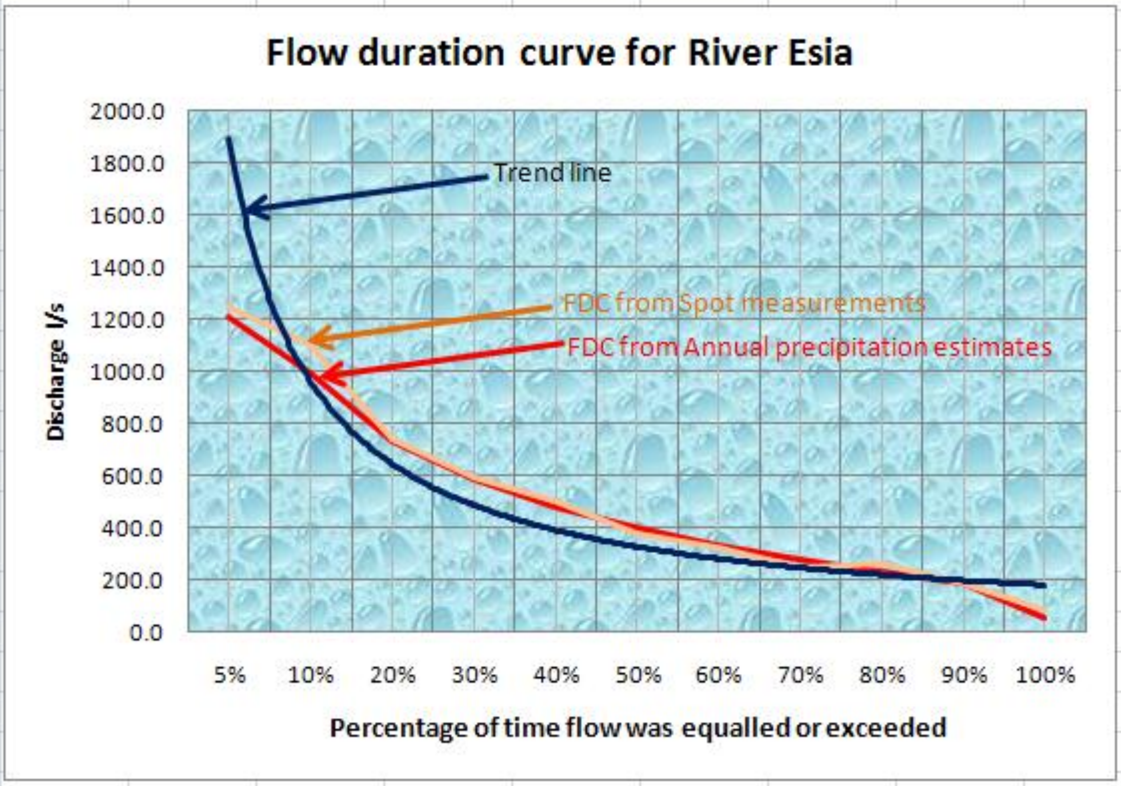


Figure 7: Flow duration curve for River Esia with spot measurements and trend-line.

More spot measurements will be taken during the rainy season so as to have more light into the prevailing hydrological situation. Spot measurements will also be used to obtain a probable rating curve which will be used to analyze the river stage data obtained from the site. Below are pictures showing the present river state (as of 5th April 2011)

Figure 8: Picture showing River Esia (November-March period)

Figure 9: Picture showing River Esia at the bridge (November – March period).

2. GEOLOGY

2.1. EXECUTIVE SUMMARY

The Geotechnical feasibility assessment is targeted at a section of river Esia identified as suitable for setting up a mini-hydro power station. The major objective of the study was to investigate the ground characteristics and profile at the various locations of the proposed

mini-hydro power structures along the Esia river namely the Earth dam area, channel route, forebay, penstock and power house. The soil and rock composition of the project site was a key area although other physical features such as valleys, gullies, seasonal streams and surface deposition were important aspects in this study. A review of the seismic history and analysis was done using datasets acquired from local and international sources.

The Esia river is located in north western Uganda in the district of Adjumani. Its water source originates from mountain ranges within the region and flows in a north westerly direction draining its waters in the greater River Nile.

An integrated approach to the geotechnical study was adopted with a desktop review of datasets and other relevant information relating to the area including topography, soils, geology, and geomorphology. A field study was then followed up with appraisal of the actual site parameters, observation of physical landscape and sampling of soils for laboratory testing of its engineering properties.

The topography indicates that most of area is low-lying in nature with wide valleys and gentle slopes. The river course is characterized by several short intervals of twists and curves. The soils are black/brown clayey sandy of relatively low permeability and are highly susceptible to transportation by run-off. Therefore, all structures constructed in this area have to be laid at/on rock interface foundation. Major rock types in the area are undifferentiated gneisses, granite gneisses and granites of the Basement complex of Uganda. There are intervals of quartzites and quartzitic sandstones which intrude the country rocks. Except at the proposed earth dam location which requires foundation reinforcement, the rocks at the In-take, forebay and power house can provide a suitable foundation for construction. The structural set up indicates a dense network of fault lines which are seismically inactive. The only seismic activity recorded from Earthquakes is approximately 50 km away with magnitudes of 3.0 – 4.0 on the Richter scale whose consequence on structures is very minimal.

The distribution of earthquakes in the region from records of the National seismological center is relatively low and poses no major threat to project structures. The general shear on undisturbed samples posted allowable bearing capacities at the site ranging from 44 kPa to 87kPa.

2.2. Introduction

The geotechnical investigation of the proposed Esia Mini-hydro power project is important in the design of civil and mechanical structures. River Esia is located in north western Uganda approximately 7.0 kilometers from Adjumani township. The proposed Mini-hydropower project is expected to have a considerable structural footprint on the area. All proposed infrastructure have to be designed bearing in mind the soil properties, rock types, depth to the bedrock among other vital parameters. It is therefore important to critically assess the soil engineering properties, profile, potential risks and mitigation measures to be undertaken during design and implementation of project structures. Samples of soils were collected and forwarded to the laboratory for testing of material properties. Locations near or along the project site with suitable construction materials were proposed to avail options to the construction team.

2.3. Objective of Geotechnical Study

The major object of the feasibility study was to investigate the ground characteristics at the various locations of the proposed mini-hydro power structures along the Esia river namely the proposed earth dam site, channel route, forebay, penstock and power house. The soil and rock composition of the project site was a key aspect although other surface phenomenon such as valleys, gullies, seasonal streams and surface deposition were investigated. A review of the seismic history of the area was accomplished during the project.

2.4. Topographical and Physiographical Assessment

The Esia river catchment and vicinity are generally low lying areas with the exception of the forebay and power house area. Regionally the area has numerous fault lines which have correlate to the wide valleys and massive gully structures observed in the field. The deposition of erosion material gradually increases from the source towards the direction where it drains its waters in the river Nile.

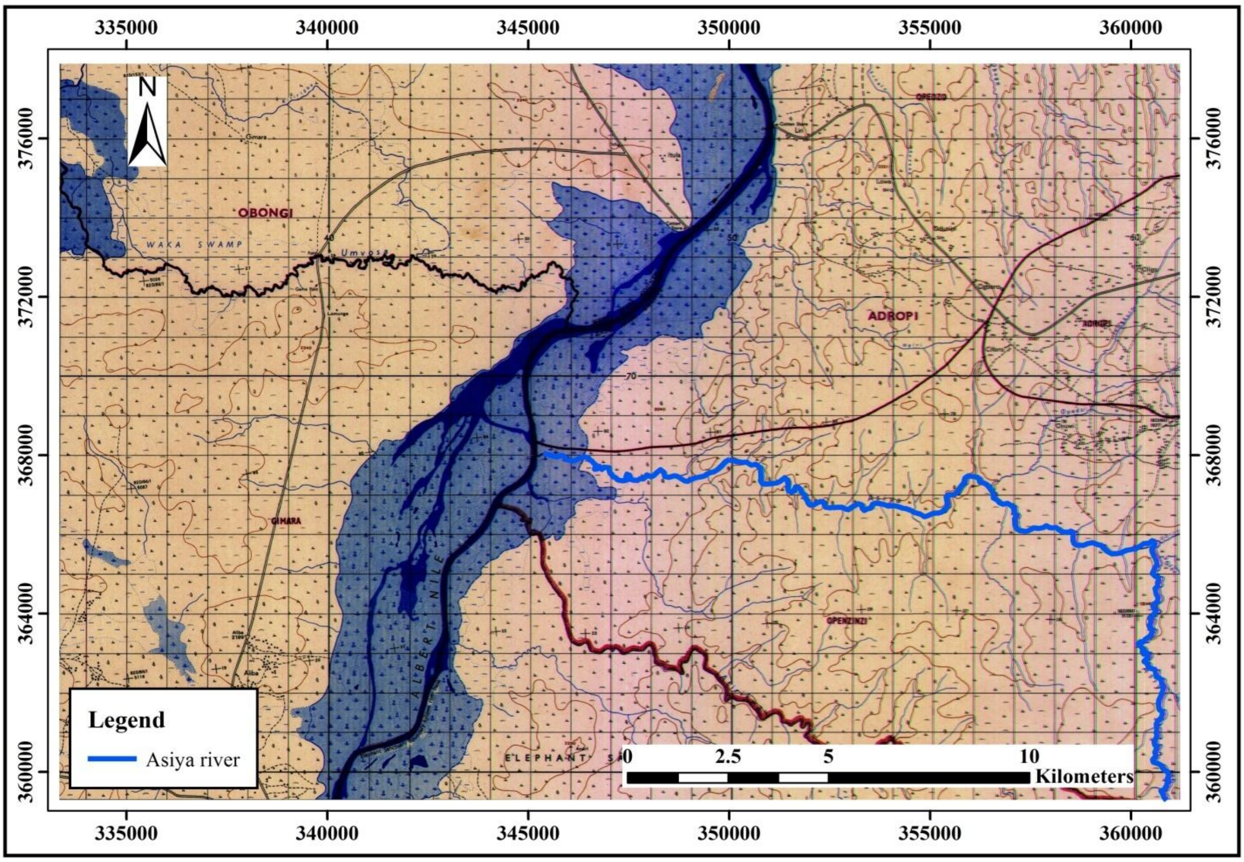
At the In-take area, in-situ rocks and floats cover most of the near surface area. A network of trees hold the rock floats and soils in place, a phenomenon which should be maintained or even improved to minimize siltation of the river. The proposed forebay and power house is located along steep slope with an elevation difference between the two of approximately 30 – 40m equivalent to the head. The penstock route is described as rocky with boulders and floats.

2.5. Soil Composition

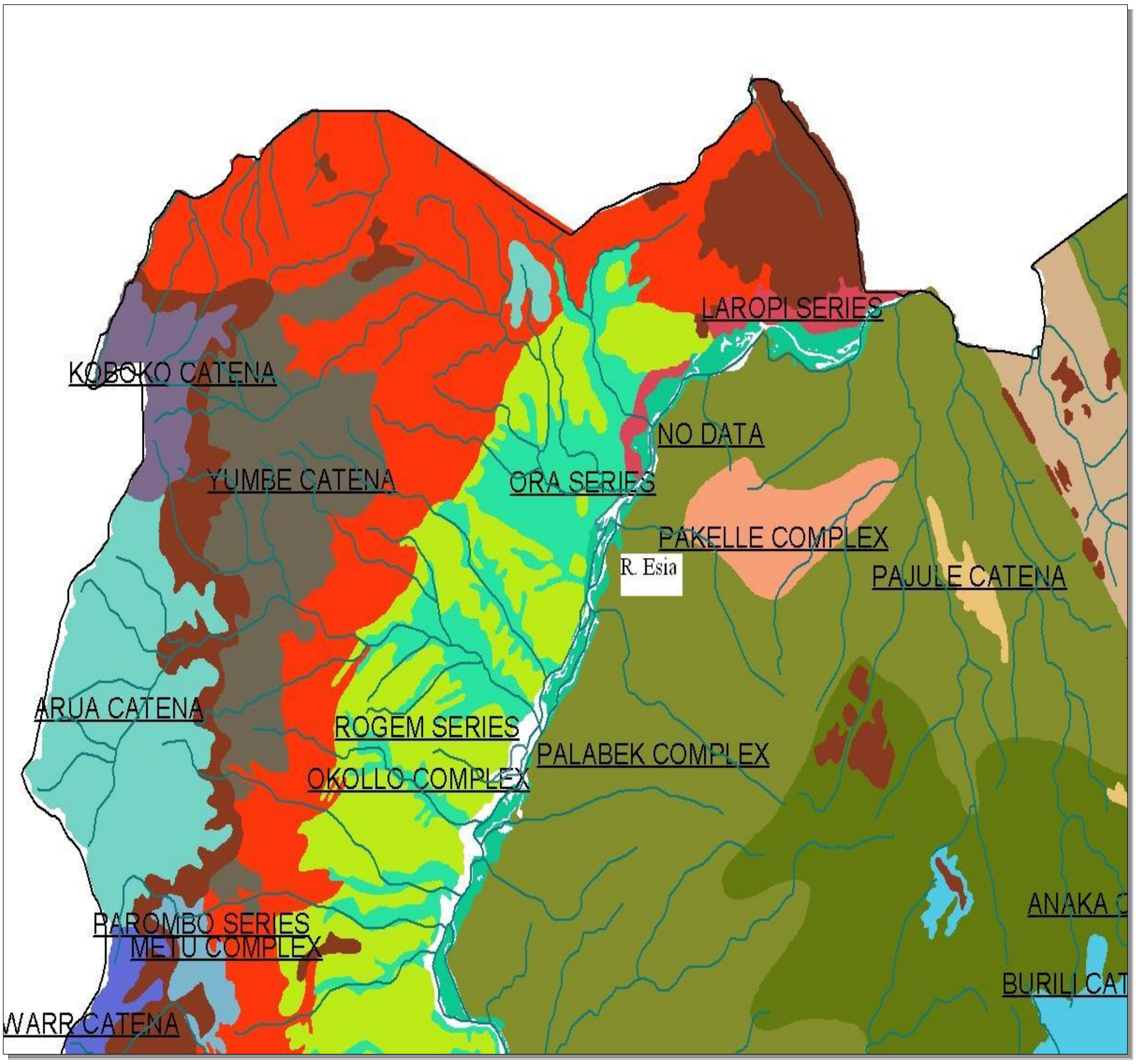
The major soil units identified in this area from desktop review are the Pakelle complex and the Palabek Complex (See Table 1 & map 1.0). During the field excursion, soils of this composition were observed in the area. Black sandy-clayey soils and brown sandy clay soils cover most parts of the project site.

Table 8: Soil units in the project area

MAP UNIT	SOIL TYPE	PARENT ROCK
PAKELLE COMPLEX	Brown sandy clays mottled below and brown sandy loams	Basement complex gneiss and alluvium
PALABEK COMPLEX	Shallow brown/grey sandy loams over old alluvial	Basement complex gneisses
PALABEK COMPLEX	Shallow brown sandy loams over old alluvial	Basement complex gneisses



A map showing the mapped soil units of north western Uganda


LAROPI SERIES
KOBOKO CATENA
NO DATA
YUMBE CATENA
ORA SERIES
PAKELLE COMPLEX
R. Esia
PAJULE CATENA
ARUA CATENA
ROGEM SERIES
PALABEK COMPLEX
OKOLLO COMPLEX
PAROMBO SERIES
METU COMPLEX
ANAKA C
BURILI CAT
WARR CATENA

TP 1 (Bridge Area) Water Table Not encountered in the test pit

DEPTH (m) 0.00	LOG*	SOIL DESCRIPTION	REMARKS
0.35	= = = = = = = = = = = = = = = = = =	Light grey clayey silt	Loose
2.00	* * * * * * * * * * * ** * * * * * * * * * ** * * * * * * * * * ** * * * * * * * * * * * * * * ** *	Yellowish brown silty sand	Medium dense
2.40	« « « « « « « « « « « « « « « «	Rock	Very dense

LOG: Expression of ground Profile with depth*

TP 2 (Proposed Intake) Water Table Not encountered in the test pit

DEPTH (m) 0.00	LOG*	SOIL DESCRIPTION	REMARKS
0.60	= = = = = = = = = = = = = = = = = =	Dark grey clayey silt	Loose
1.05	= =	Light grey, brown sandy silt	Medium dense
1.40	* ** * * * * * * * * * ** * * * *	Yellowish brown - grey consolidated clayey sand (Hard Pan)	Very dense

LOG: Expression of ground Profile with depth*

TP 3 (Forebay) **Water Table** **Not encountered in the test pit**

DEPTH (m) 0.00	LOG*	SOIL DESCRIPTION	REMARKS
0.30	= =	Light grey clayey silt	Loose
	* *	Reddish brown - yellow clayey sand with gravel	Medium dense
0.6 (Hard Pan)	* * * * * * * * * * * * *		

LOG: Expression of ground Profile with depth*

2.5.1 Soil sampling and profiling

Generally, disturbed samples were recovered from the test pits at 1.0m and 2.0m depths from TP1; and from 1.40m and 0.60m depths in TPs 2 & 3 respectively.

2.5.2 Test Pits Log

Figure 10: General Profile of Test pit

2.6. Laboratory Test results

2.6.1 Classification test results

Laboratory analysis of the samples indicated that the soils at the site are generally clayey sands (SC). This classification is based on the Unified Soil Classification System. The key index properties of the material were as summarized in **Table 2** below:

Table 9: Summary of Soils Index Properties

Test pit	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index
TP 1	1.0	29	NP	-
	2.0	27	NP	-
TP 2	1.4	47	15	32
TP 3	0.6	45	20	25

2.6.2 Shear Strength

Table 3 below presents results for shear strength determined using the shear box test, from undisturbed soil samples obtained from the test pits. Detailed results are attached in **Appendix 4**.

Table 10: Summary of Soil Bearing Capacity (General Shear Failure)

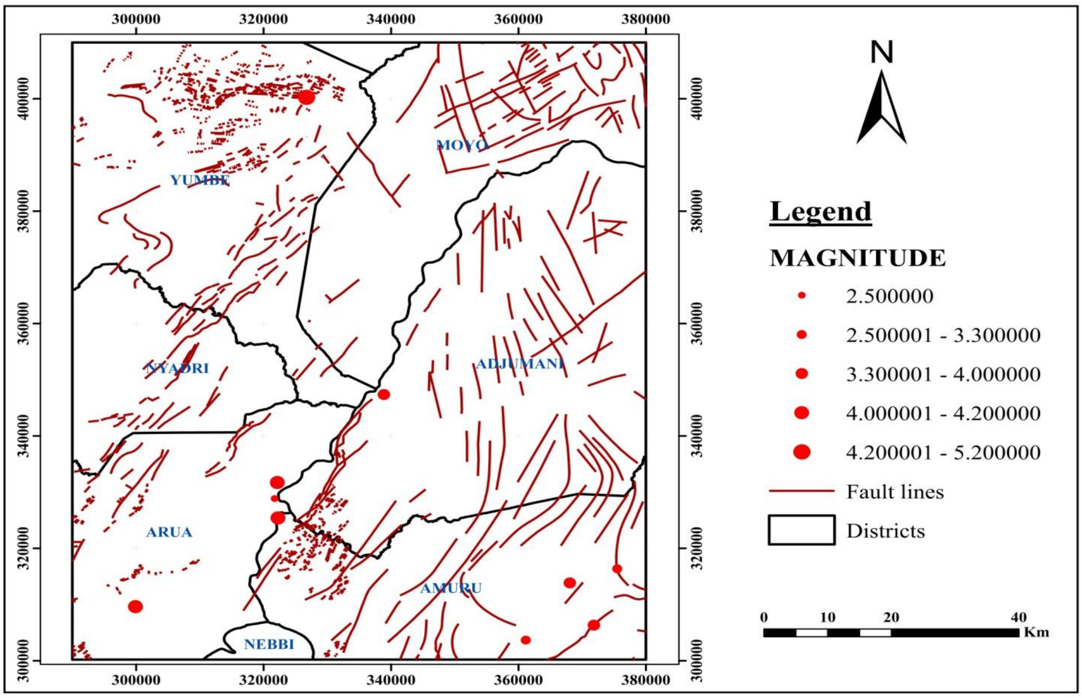
Location	Depth (m)	Angle of friction (Ø′)	Allowable bearing capacity under ultimate load (kPa)
TP 1	1.0	18	44
	2.0	18	87
TP 2	1.4	19	67
TP 3	0.6	23	51

The above evaluation of bearing capacity based on laboratory tests (General Shear Failure) on remolded samples at 95% MDD, OMC posted allowable bearing capacities at the site ranging from 44kPa to 87kPa.

2.7. Seismicity of the Area

The Ugandan plus the Eastern and Southern Africa Regional Seismological Working Group databases were used in this compilation. The distribution of earthquakes from this figure does not present major areas of prominent seismicity. The area of interest can be seen to have numerous fault lines which seem to be inactive in terms of seismic activity (this is clearly shown by the number of events recorded over a period of more than 40 years).

Epicentral Map of part of North Western Uganda for the period 1964- 2008



The lack of a dense network of stations to cover the whole country and the big distances between the existing stations has led to numerous un-locatable events as majority are recorded on less than three stations. This has hampered the size of the database and the quality of research work in terms of valid conclusions. Caution should be taken when drilling or excavating in this area as there are chances that the faults could be activated leading to unstable land.

2.8. Local Geology of the Area

The project comprises of undifferentiated granites and gneisses of the Basement complex. In some locations it is observed that the granites and gneisses have been intruded by quartzites. The processes of metamorphism, hydrothermal alteration and weathering have given rise to other intermediates such as quartzitic sandstones and granite gneisses. Continuous erosion and transportation of material has resulted in the accumulation of alluvium in several locations along the project site but also at the banks of the river.

2.8.1 Proposed Bridge Area/Earth dam and Reservoir

This location is mostly underlain by gneissic rocks (gneiss) of the basement complex. These rocks are highly brittle; they are weak under compression forces and can not adequately support the Earth dam structure independently. Owing to the poor integrity of the underlying rock, the Earth dam structure should be reinforced with concrete and steel foundation to a depth of at least 1.5 m. The Black/brown sandy clay soils at this location need to be supported at the river banks with gabions to control siltation minimize erosion deposition and contain any external material from interfering with the reservoir.

Figure 11: Proposed Bridge Area/Earth dam and Reservoir

2.9. Proposed Intake Locations

According to preliminary designs on the survey drawing, three locations were sited as possible intake to the channel. The merits and demerits of these locations in view of ground conditions are given here below.

2.9.1 Intake Area A

- The location is close to the dam and siltation of waters will be minimal since the Earth dam will dredged.
- Overburden to be remove is considerable huge and the cost of excavation may be high.

Close proximity to the dam implies that any uncontrolled process such as siltation will quickly affect the intake.

2.9.2 Intake Area B

- At this location, it was observed that much more excavation is required to create a suitable in-take and accompanying cost implications.
- Due to the rock outcrops visibly close to the surface, blasting would be required.
- Gneisssic rocks are observed which are not suitable for anchoring any structure of worthy integrity.

2.9.3 Intake Area C (Proposed In-take)

- The area has a wider river outlet and flat area which can naturally provide an environment worthy of construction.

- The rocks in this area are granites which are close to the surface and can provide a good foundation base for anchoring structures.
- This location is a summation of river outlets that are flowing into the main river which ensures greater volumes.

2.10. Channel Route/Canal Way

The channel route was inspected to investigate any features that would directly affect the integrity of the structures and establish the possibility of utilizing construction materials along the route.

a) Seasonal streams and gullies have developed in the ground overtime in a few locations like GPS 312 and GPS 313. These seasonal streams carry seasonal water flows towards the main river. Stabilizing the ground conditions with suitable material is necessary to adequately support channel foundation structures.
b) In general, gentle slopes and wide valleys occupy most of the channel route. An attempt to take a linear course will be investigated in detail at advanced stage.(River training)
c) The availability of quartzites forming small rock gravel along the course of the channel which may be utilized for In-fill and reinforcement of fragile ground.

2.11. Key Conclusions on Engineering Properties of Soil

- The site was investigated by excavating 3 No. test pits to a maximum depth of 2.0m; and sampling disturbed samples (2 No. in each) for laboratory testing;
- Ground water table was not encountered in any of the test pits; The average depth to water table was determined at 4.0 – 8.0 meters along the channel route.
- The key index properties of the soil samples from the test pits investigated ranged from: LL = 27 to 47%, PL from NP to 20%. The Natural Moisture Content ranged from 4% to 15%. (See Appendix 7 for detailed laboratory test results);
- Evaluation of bearing capacity based on laboratory tests (General Shear Failure) on undisturbed samples posted allowable bearing capacities at the site ranging from 44kPa to 87kPa at 2.0 depths as per Table 3.0;
- Compaction tests revealed that the MDD ranged from 1.57 to 1.78 Mg/m^3 whereas the OMC ranged from 4 to 13%. The least MDD value was obtained from TP 2 at 1.40m depth and the highest value from TP 3 at 0.6m depth.

- The permeability (k) of the soils ranged from 5.20 x 10-3cm/sec to 6.30x10-5cm/sec implying that it is low to medium. Details are presented in Appendix 7.

3. CONCEPTUAL AND STRUCTURAL DESIGN

SCHEME LAY-OUT

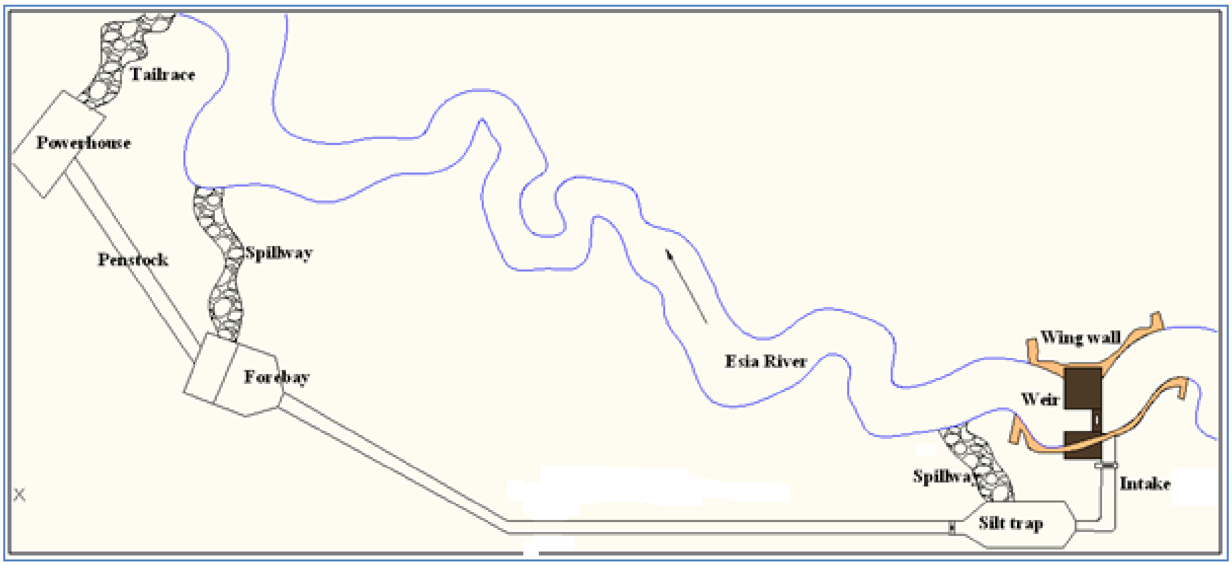


Figure 12: Scheme Lay-out

3.1. Design Flow Rate

3.1.1. ANALYSIS OF THE FDC VALUES

i. The river runs the all year round
ii. River flow rates ranges from 50l/s to about 1500l/s.
iii. The large flow rates (700l/s to 1300l/s) lie on the steeper part of the curve, meaning that they are only experienced for months of the year (3 or 4 months a year).
iv. 400l/s is guaranteed for at least six months of the year.
v. 200l/s is almost guaranteed 11 months of the year.

3.2. Type of the Scheme

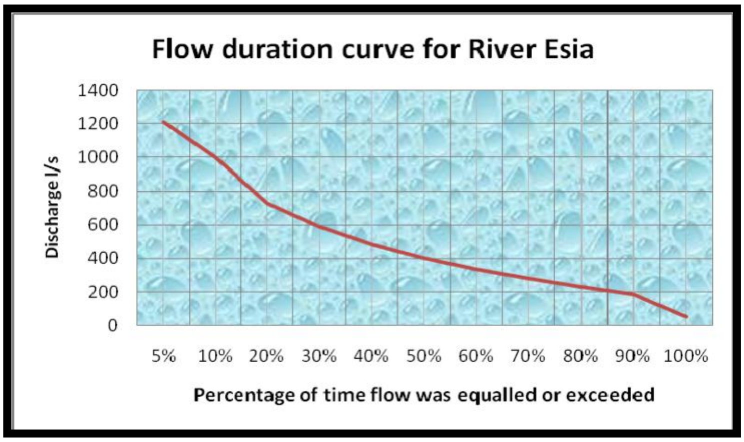
The scheme is designed to be a run-of-river scheme. This means that power generation is only done when the water is available and provided by the river.

Damming is not possible due to the following:

1. The topography of the area is relatively flat making damming disastrous due to the expected widespread flooding of the farmland along the river. This is more evident upstream from the bridge where already large areas of farmland presently get flooded during heavy rains.
2. The effects of damming would greatly affect the bridge which is 1km from the proposed intake site.
3. Esia River has loose soil and short embankments which will not provide a required operational factor of safety in terms of impoundment, leakage and susceptibility to failure.

3.3. Design Flow Rate

Two design flows were considered; 100l/s and 500l/s representing 100% and 45% exceedancies respectively, basing on the established FDC. The idea was to design a system which would operate at both flows. This would require two turbines to enable power production throughout the year. However, the cost of the intake structures, channel shape alteration, penstocks and two turbines outweighed the envisaged advantages. The expected power of 15kw at minimum flow is not justifiable vis-à-vis the incurred capital costs. Therefore, 500l/s was chosen and is the average flow across the entire year.



It is important to note that this design flow is subject to the minimum required environment release of 50l/s as recommended by the Environmentalist. The spill water over the crest of the weir shall supplement the environment release. In the case of discharges greater than 500l/s, the weir shall spill the water downstream.

Table 1: Design flow rate

Type of scheme	Run-of-river
Design flow	500l/s
Minimum Environmental release	50l/s

3.4. Weir

In this design the main water retaining structure is a barrage with a clear overflow type of spillway. The barrage is designed as a gravity type dam (according to design criteria recommended for the Design of small dams United State Bureau of Reclamation - USBR). In general this type of a dam is designed to resist external forces with its own self-weight. However, it is widely recognized that gravity type dams are the most durable, and solid structures that requires minimum maintenance. Since this water-retaining structure is at isolated locations, with difficult access, a dam that requires minimum maintenance is the most suitable. Therefore this barrage is designed as a stone masonry gravity type structure with straight axis. As the ground condition on the barrage axis is of firm bedrock, it is assumed that the structure is constructed on the natural bedrock as its foundation material is strong enough to bear the weight of the structure. An environmental orifice has been inbuilt to offer an environmental flow.

With the abundant availability of sound rocks of granite gneiss, it is recommended that the dam is constructed using stones in 1:4 cement sand mortar. The density of this mass has been taken as 28KN/m3. This type of construction will offer employment to the locals since its execution is simple and well known.

Table 11: Weir and River specifications

Parameter	Value	Unit
Maximum flood, Q_{flood}	20	m^3/s
Weir Breadth, B	15	m
Weir Height, Y	2.5	m
Weir head, Hw	0.075	m
Design Head, H_w	1.5	m
Velocity Head, H_v	0.013	m
Total head	1.588	m
Silt Flushing Head	2.5	m
Silt Flushing Width	0.5	m
Environmental release head	2	m
Environmental release Diameter, $Ø_1$	0.13	m
C_v	0.6	
River Average Velocity	0.5	m/s
Weir Coefficient Cd	0.567	
Maximum flood overtop head	0.89	m
Type of weir	Round overflow	

$$H_v = \frac{V^2}{2g}$$

$$Q = C_d b\sqrt{g}H^{3/2}$$

$$C_d = 0.564 + 0.0846\frac{H_w}{Y}$$

The proposed weir is designed to divert only the needed flow and let out the river permanent flow rate as recommended by the Environmentalist. The proposed length is enough to discharge the flood corresponding to that of return periods of 50 years i.e. $20m^3/s$.

The weir is laid on sound rock foundation of granite gneisses and granites of the basement complex of Uganda. The distribution of earth quakes in the project area obtained from the records of National Seismological center is relatively low and possess no significant threat to the project structures. Therefore, Seismic analysis on the weir is not necessary.

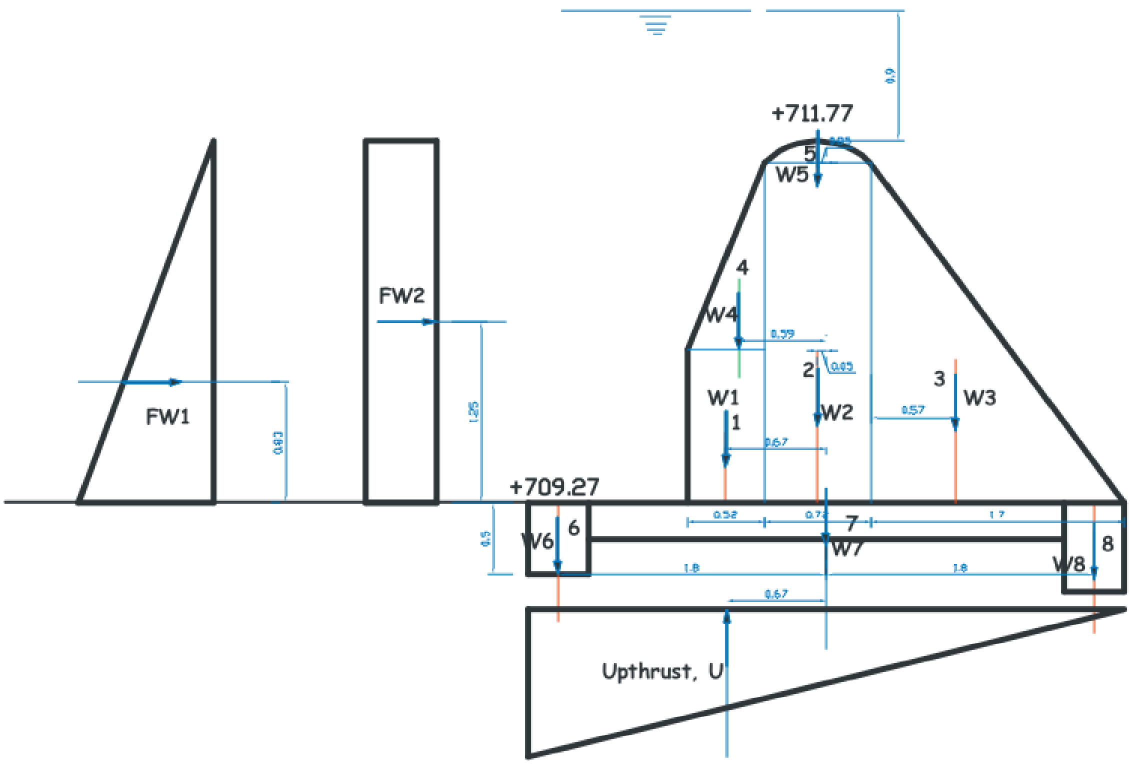


Figure 13: Force analysis on the weir

The overturning moments are taken about the centre of the base.

Table 3: Force analysis on the weir

Force	Volume per m	Force Magnitude		Lever Arm	Moment about centre of base
		Vertical	Horizontal		
	m^3/m	KN	KN	m	KNm
W1	0.545	14.17		-0.67	-9.49
W2	1.676	43.576		-0.05	-2.18
W3	2.006	52.156		0.57	29.73
W4	0.335	8.71		-0.59	-5.14
W5	0.074	1.924		-0.05	-0.10
W6	0.2	5.2		-1.8	-9.36
W7	0.801	20.826		0	0.00
W8	0.247	6.916		1.8	12.45
FW1	3.125		31.25	0.83	0.00
FW2	2.25		22.5	1.25	0.00
U	6	60		0.65	39.00
	17.259	**213.478**	**53.75**		**54.91**

Resultant Moment 54.91KNm

Direct stress at base 53.35 KN/m^2

Bending Stress at the extreme fibres 32.78KN/$m^2$and 55.91 KN/m^2

3.5. Calculating the Weir Flood Flow

Figure 4: Flood Head

Weir parameters

Weir crest length =15m

Y=Weir height

H=Weir head

b=Weir breadth

Using Poleni formula the weir discharge for different crest shapes and for free-flow or submerged conditions can be obtained;

Table 12: Weir flood Head

Maximum flow rate from FDC	2	m^3/s
Worst flood Design flood, Qflood	20	m^3/s
C	1	
μ	0.55	
b	15	m
g	9.81	m/s^2
flood Head, H	0.88	m

$$H = \left(\frac{3Q_{flood}}{2c\mu b\sqrt{2g}} \right)^{2/3}$$

The designed flood is considered to be 10 times the maximum measured flow.
C is the correction factor for submerged orifice and in this case it will be 1
μ is the weir coefficient which vary from 0.49 to 0.79 depending on the cross-sectional shape of the weir.

3.6. Wing Wall Height at the Weir

The wing wall is designed to retain the river water from flooding the adjacent intake structure.
The maximum flow ever recorded approximates to 2m3/s. using the Hydrologist's recommendation; we can say that the worst flood will be=20m3/s

Table 13: Wing wall height

Profile of crest of weir, Cw	1.6	
L_{weir}=b	15	m
hover-top	0.89	m
Height of barrier walls, $H_{barrier}$	3.39	m

$$h_{overtop} = \left(\frac{Q_{flood}}{C_w \cdot L_{weir}} \right)^{1/1.5}$$

The set up of the weir and wing wall shall be able to create a reservoir area of 7 hectares

3.7. Design of the Wing Walls

3.7.1 Reinforced Concrete Wing Wall:

Wing walls are constructed for the following purpose:

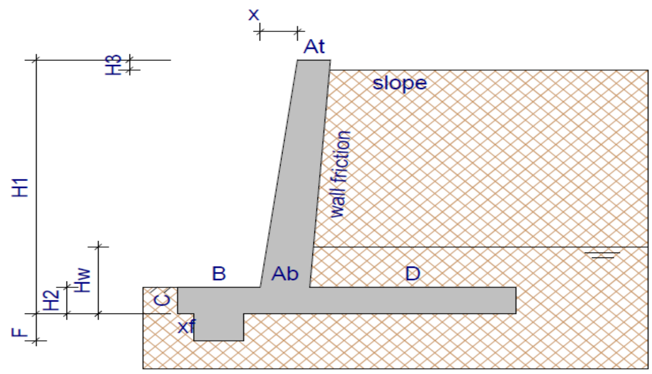
- To retain the water so that the right level is obtained and prevent flood water from damaging the structures at the intake – Reinforced Concrete wall and Gabion wall
- To support the intake structures e.g. orifice structure, Intake and flushing gates – Reinforced Concrete Wall
- To act as an embankment for the created reservoir

Reinforced Concrete Wall:

This will be about 23m on either side of the weir.

Input Data

Figure 5: Wing wall design



Seepage allowed

Active pressure applied on back of shear key for sliding

Theory : Coulomb

Wall type: Cantilever

Table 14: Wing wall Parameters

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :	0.398
Passive Pressure coefficient Kp :	3.525
Base frictional constant μ :	0.364

FORCES ACTING ON THE WALL AT SLS:

All forces/moments are per m width

FORCES (kN) and their LEVER ARMS (m

Description	F Horizontal	Lever arm left (+)	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	56.473	1.350	15.975	1.483
Triangular W-table press Pw	5.792	0.467	0.000	1.405
Hydrostatic pressure on bot of base: uniform portion		-16.088	1.850	
Hydrostatic pressure on bot of base: triangular portion		-12.066	2.533	
Stabilizing forces:				
Passive pressure on base Pp	-12.378	0.267	89.500	1.468
Weight of the wall + base			124.675	2.714
Weight of soil on the base			14.715	2.650
Hydrostatic pressure on top of rear portion of base				
Hydrostatic pressure on top of front portion of base			0.000	0.300

EQUILIBRIUM CALCULATIONS AT SLS

All forces/moments are per m width

1. Moment Equilibrium

Point of rotation: bottom front corner of shear key.

For Overturning moment Mo calculate as follows:

M_o = Sum (hor. forces x l.a.) - Sum (vert. forces x l.a.)

For Stabilizing moment M_r calculate as follows:

M_r = -Sum (hor. forces x l.a.) + Sum (vert. forces x l.a.)

Where l.a. = lever arm of each force.

Stabilizing moment M_r:	512.05 kNm
Destabilizing moment M_o:	115.58 kNm
Safety factor against overturning = M_r/M_o	4.430

2. Force Equilibrium at SLS

Sum of Vertical forces Pv	216.71 kN
Frictional resistance Pfric	78.88 kN
Passive Pressure on shear key	7.93 kN
Passive pressure on base:	12.38 kN
Total Horiz. Resistance Fr	99.18 kN
Horizontal sliding force on wall Fhw :	62.26 kN
Horizontal sliding force on shear key Fht	0.00 kN
Total Horizontal sliding force Fh	62.26 kN
Safety factor against overall sliding = Fr/Fh =	**1.575**

FORCES ACTING ON THE WALL AT ULS:

All forces/moments are per m width

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal	Lever arm left (+)	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	79.062	1.350	22.364	1.483
Triangular W-table press Pw	8.108	0.467	0.000	1.405
Hydrostatic pressure on bot			-22.524	1.850
of base: uniform portion			-16.893	2.533
Hydrostatic pressure on bot				
of base: triangular portion				
Stabilizing forces:				
Passive pressure on base Pp	-11.140	0.267	80.550	1.468
Weight of the wall + base			112.207	2.714
Weight of soil on the base			13.244	2.650
Hydrostatic pressure on top				
of rear portion of base			0.000	0.300
Hydrostatic pressure on top				
of front portion of base				

EQUILIBRIUM CALCULATIONS AT ULS

All forces/moments are per m width

1. Moment Equilibrium

Point of rotation: bottom front corner of shear key.

For Overturning moment Mo calculate as follows:

Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)

For Stabilizing moment Mr calculate as follows:

M_r = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force.

Stabilizing moment Mr:	460.84 kNm
Destabilizing moment Mo:	161.81 kNm
Safety factor against overturning = Mr/Mo	2.848

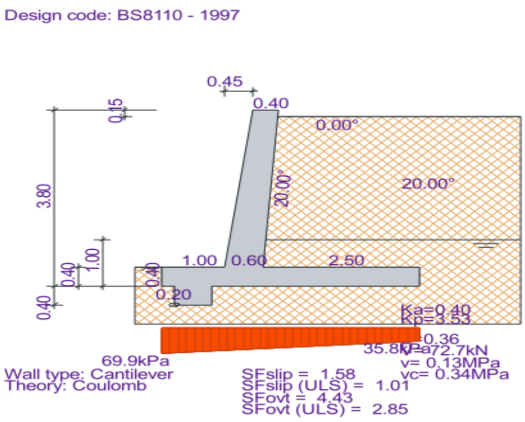
2. Force Equilibrium at ULS

Sum of Vertical forces Pv	195.04 kN
Frictional resistance Pfric	70.99 kN
Passive Pressure on shear key	7.13 kN
Passive pressure on base:	11.14 kN
Total Horiz. Resistance Fr	89.26 kN
Horizontal sliding force on wall Fhw :	87.17 kN
Horizontal sliding force on shear key Fht	0.00 kN
Total Horizontal sliding force Fh	87.17 kN
Safety factor against overall sliding = Fr/Fh =	**1.013**

SOIL PRESSURES UNDER BASE AT SLS

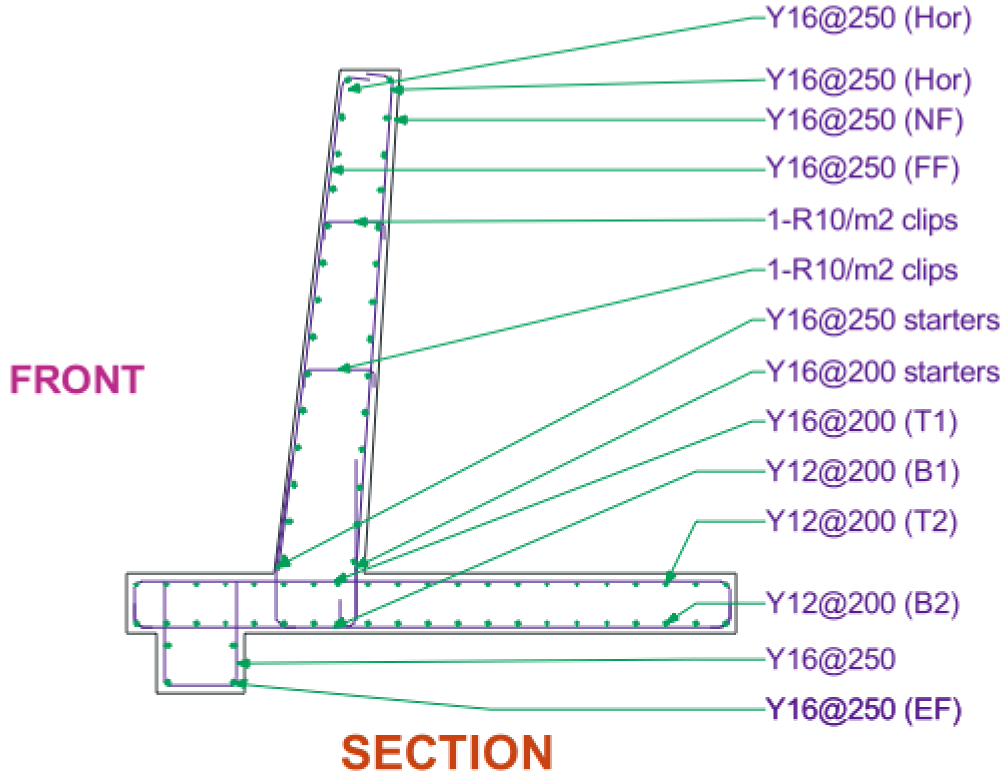
Maximum pressure: 69.91 kPa
Minimum pressure: 35.80 kPa
Maximum pressure occurs at left hand side of base

Figure 6: Wing wall dimensions



Wall Reinforcement

Figure 14: Wing wall Reinforcement



3.7.2 Gabion Wing Wall

The basis of the design of the gabion base should be such that the pressure at the base is less than the anticipated bearing capacity of the material under it, in this case the in-situ rock. The abundant granite gneiss rock shall safely carry the gabion wall. A fill shall be placed behind the gabions and compacted in layers of 300mm thick.

The required level on which gabions are to be placed shall be achieved by laying stone material in depressions followed by gabion mattresses and then the proceeding layers placed. The mattresses which are 300mm thick shall be included in the stability calculation.

Table 15: Embankment Specifications

Description	symbols	Amount	units	Notes
Backfill Slope angle	β	0	°	
Angle of Internal Friction	ϕ	19	°	
Angle of wall friction	δ	12	°	
Inclination angle to vertical plane	ω	0	°	
cohesion	c	0	KN/m^2	Ignore Cohesion
Surcharge	q	7	KN/m^2	
Soil Density of fill	γ	17	KN/m^3	
Gabion Density	γ_g	28	KN/m^3	
Height of Wall	H	4.3	m	
Width of base	B	3	m	
Embedment	d	0.3	m	
Allowable Soil Bearing capacity	q_a	800	KN/m^3	Gabions to be laid on In-Situ Rock
Active Earth Pressure coefficient	K_a	0.46		Coulumb's Theory

Below are calculations to check

1. Overturning in which the factor of safety should be greater than 2
2. Sliding where the factor of safety should be greater than 1.5
3. Eccentricity of Resultant Force should be within the middle 3rd of the base

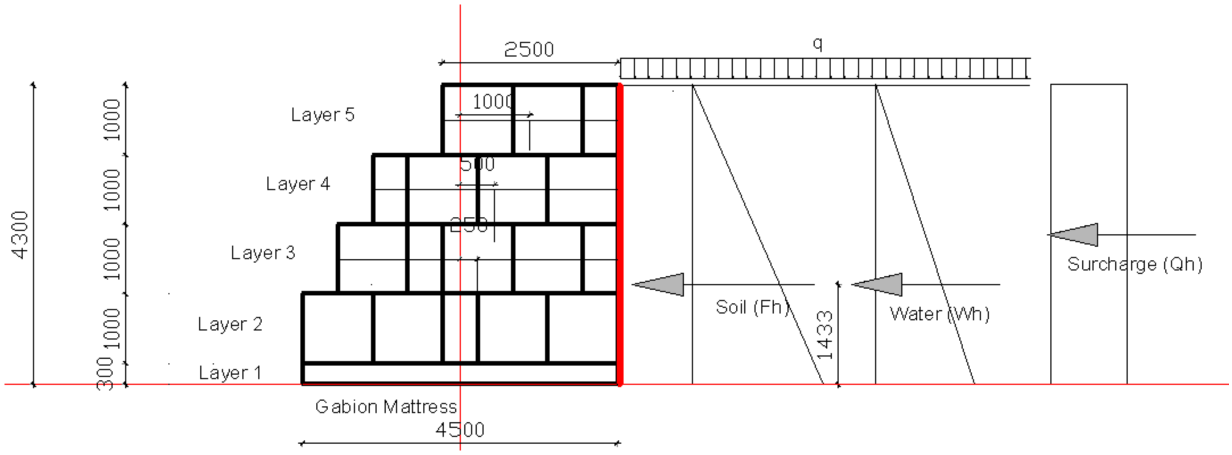


Figure 15: Gabion Alignment

Table 16: Embankment structural analysis

Gabion Row #	Width	Height	Offset from mid base	Offset from toe	Area	Weight	Moment	Moment 2
5	2.5	1	1	3.25	2.5	62.5	62.50	203.125
4	3.5	1	0.5	2.75	3.5	87.5	43.75	240.625
3	4	1	0.25	2.5	4	100	25.00	250
2	4.5	1	0	2.25	4.5	112.5	-	253.125
1	4.5	0.3	0	2.25	1.35	33.75	-	75.9375
Resisting moment (M_R)						**396.25**	**131.25**	**1022.8125**
Overturning moment								
Description	Force	Amount	Moment arm					Moment
Soil Fill	F_h	71.84	1.43					102.73
Water	W_h	92.45	1.43					132.20
Surcharge Force	Q_h	13.75882	2.15					29.58
Total overtuning moment (M_o)								**264.52**
Factor of Safety against overturning								**3.87**

Sliding		
Total vertical weight	396.25	KN/m
Frictional Force	136.44	W.tanΦ
Total horizontal Force	85.60	KN/m
Sliding Factor of Safety	1.59	Ok

Checking The Excentricity of Resultant Force		
Eccentricity (e)	-0.33632	$\frac{\sum M_R - M_o}{\sum W}$
B/6	0.75	

3.8. Backwater Analysis Effect

This analysis is meant to assess the impact of the weir on the upstream bridge during flooding times. All the parameters are designed in reference to the maximum anticipated flood flow rate. The direct step method has been used to compute the river water profile. The river has been assumed to be trapezoidal in shape.

Table 17: Bridge-Weir specifications

Name	Symbol	Unit	Value
Bridge Height	y	m	2.6
Weir Head at Max. flood	Hw	m	0.89
Weir Height	H	m	2.5
flood flow rate	Q	m^3/s	20
Weir crest length	b	m	15
The river slope S	S	%	4/ 1000
Manning's constant n	n		0.035
Total head at the weir	m	m	2.647
Critical depth	y_c	m	0.898
Normal depth	y_n, m	m	0.87
Flood Flow	$Q_{flood,}$	m3/ s	20
Water depth at weir, m		m	3.39
River specifications			
River Bottom Width		m	10
River Side Slope			1 in 0.75
River Bed slope, So	So		0.004
Distance from the weir along river	x	m	
Change in water profile in respect to x	dy	m	0.15

$$Q = C_d b \sqrt{g} H_w^{3/2}$$

$$C_d = 0.564 + 0.0846 \frac{H_w}{Y}$$

$$y_c = \left(\frac{Q^2}{b^2 g} \right)^{1/3}$$

$$Q = \frac{b y_n R_n^{\frac{2}{3}} S_o^{1/2}}{n}$$

$$\frac{dy}{dx} = \frac{S_o - S_f}{1 - Fr^2}$$

$$S_f = \frac{n^2 Q^2 P^{4/3}}{A^{10/3}} = \text{and } Fr^2 = \frac{Q^2 b}{g A^3}$$

Depth of water at the weir at normal flow = $y_0 =$ 0.147+2.5=2.647m

Type of flow is subcritical since $y_0 > y_c$ since the critical depth, y_c=0.898m

The normal depth, y_n =1.35m. Therefore, $y_0 > y_n > y_c$ indicating that the backwater curve away from the weir profile (upstream) has got a positive gradient.

Table 18: Backwater analysis calculations

Water depth	Mean Depth (y_m)	Mean Area $A(y_m)$	Wetted Perimeter (P_m)	Hydraulic Mean Radius (m_m)	Mean Velocity (v_m)	Chezy C ($=m_m^{1/6}/n$)	$j_m=v_m^2/C^2 m_m=S_f$	Mean Top Width (B_m)	Froude No. (F^2_{Nm})	$dy/dx=(S_o-S_{fm})/(1-F^2_{Nm})$	River Profile (m)
3.39											0.0
3.24	3.46	43.59	18.65	2.34	0.46	32.91	0.00008	15.19	0.007	-0.00395	-38.0
3.09	3.31	41.33	18.28	2.26	0.48	32.73	0.00010	14.97	0.009	-0.00394	-76.1
2.94	3.16	39.10	17.90	2.18	0.51	32.54	0.00011	14.74	0.010	-0.00393	-114.3
2.79	3.01	36.90	17.53	2.11	0.54	32.35	0.00013	14.52	0.012	-0.00391	-152.6
2.64	2.86	34.74	17.15	2.03	0.58	32.14	0.00016	14.29	0.014	-0.00390	-191.1
2.49	2.71	32.62	16.78	1.94	0.61	31.92	0.00019	14.07	0.017	-0.00387	-229.9
2.34	2.56	30.52	16.40	1.86	0.66	31.69	0.00023	13.84	0.020	-0.00385	-268.9
2.19	2.41	28.46	16.03	1.78	0.70	31.44	0.00028	13.62	0.024	-0.00381	-308.2
2.04	2.26	26.44	15.65	1.69	0.76	31.18	0.00035	13.39	0.030	-0.00376	-348.1
1.89	2.11	24.45	15.28	1.60	0.82	30.90	0.00044	13.17	0.037	-0.00370	-388.7
1.74	1.96	22.49	14.90	1.51	0.89	30.60	0.00056	12.94	0.046	-0.00361	-430.2
1.59	1.81	20.56	14.53	1.42	0.97	30.28	0.00073	12.72	0.060	-0.00348	-473.4
1.44	1.66	18.67	14.15	1.32	1.07	29.92	0.00097	12.49	0.078	-0.00329	-519.0
1.29	1.51	16.82	13.78	1.22	1.19	29.54	0.00133	12.27	0.105	-0.00299	-569.2
1.14	1.36	14.99	13.40	1.12	1.33	29.11	0.00188	12.04	0.146	-0.00249	-629.6
0.99	1.21	13.20	13.03	1.01	1.51	28.64	0.00276	11.82	0.209	-0.00157	-725.2
0.84	1.06	11.45	12.65	0.90	1.75	28.10	0.00427	11.59	0.315	-0.00039	-1105.1
0.69	0.91	9.73	12.28	0.79	2.06	27.48	0.00706	11.37	0.504	-0.00617	-1129.4

It is important to note that the values of dy/dx in the table above are made negative to give a logical profile starting with the weir at point 0 to the bridge (1 km upstream) as the furthest point (-1000). The values in the table have been plotted to give the water profile from the bridge to the weir (see Fig. 6).

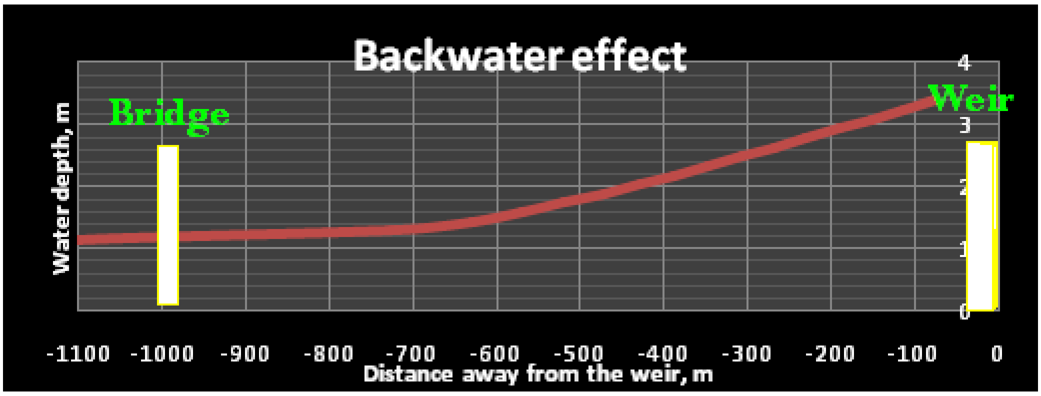


Figure 16: Backwater effect curve

It can be seen from the above profile that the water level at the bridge (1,000m from the weir upstream) is 1.2m as opposed to 2.6m, the existing height of the bridge. Therefore the proposed 2.5m high weir will not affect the bridge.

Observations by the community around the bridge indicate flooding at the bridge during the very heavy rains. This flooding lasts about 2hrs after a heavy downpour. This is due to insufficient size and number of culverts used which make the discharge at the culvert exit too small to cater for the incoming flow.

The effect of the backwater beyond the bridge upstream has not been analyzed.

3.9. Minimum Environmental Water Release Structure

A minimum flow of 50l/s will be released by the 'out let' structure built in the weir. This will be supplemented by the spill water at the weir crest. The structure is a circular orifice shall be elevated at 0.5m from the foot of the weir. As a guarantee for continual environmental minimum flow release, the 'environmental' release orifice has been located below the intake orifice.

From the flow through orifice equation, C_v taken to be 0.6

$$Q = A_{jet} V_{jet} = \frac{\pi D^2}{4} C_v \sqrt{2gH}$$

Table 19: Minimum environmental release structure

Minimum Environmental release	0.05	m^3/s
Submerged head, H	2.00	m
Cv	0.6	
Orifice head	2	m
Jet Velocity	3.76	m/s
Orifice Diameter	130.15	mm

With the silt flushing gate operating below the orifice, orifice being 0.5m from the foot of the weir and with water velocity of 3.76m/s, the orifice will stay clear of any silt and tiny particle blockages.

3.10. Silt Flushing Structure at the Weir

Esia River, just like most of the west Nile Rivers is known for a high silt load. A silt trap alone cannot be reliable and secondly silt accumulation renders blockage of the environment release.

A steel breast wall type roller gate 0.5m wide and 0.5 m long will be built into the weir to allow flushing. This will be operated during de-silting.

The gate dimensions are dependent on the operational head and the design head.

The specifications presented in the table below are from Rodney Hunt company brochure, a Massachusetts' sluice gate manufacturer.

Table 20: Silt sluice gate dimensions

Width × Height	Maximum Design Head	A	B	C	D	E	F	G
mm	m	mm						
500×500	30	762	381	533	908	1054	216	152

The gate of a much higher design head (than what is needed) is used in order to have a bigger flushing area

3.11. Intake Structure

The structure used to take the water from the river into the mini hydropower structure should be able to divert the designed flow, flood flow and also stay clear of any blockage or sedimentation. The intake incorporates a rectangular orifice side intake and a rectangular channel section transporting water into the sedimentation tank.

The intake channel is designed in a way to transport water at a higher speed hence avoiding sedimentation before the sedimentation tanks. This is done by using a relatively high slope, though caution is made that we don't lose so much head that would otherwise be used for electricity generation.

Table 13: Intake orifice and channel dimensions

$$V_i = C_d\sqrt{2g(h_r - h_h)}\, Q = A_i V_i = A_i c_d \sqrt{2g(h_r - h_h)}$$

$$W_h = \frac{Q_{gross}}{v_h h_{h(normal)}}$$

$$S = \left(\frac{n \times v_h}{R^{0.667}}\right)^2$$

$$R = \frac{W_h d}{W_h + 2d}$$

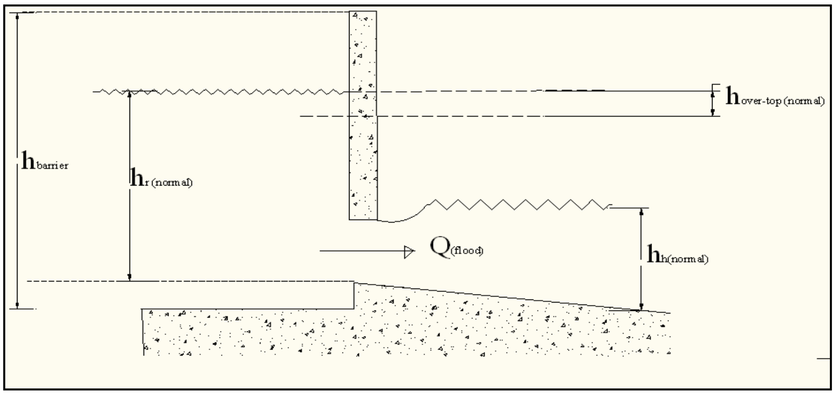


Figure 17: Intake Channel

For a sharp-edged, roughly finished intake we will use Cd=0.6.

We would like to see the surface of the water depth in the headrace () during normal flow conditions at the same level as the upper edge of the intake orifice, then $h_{h(normal)}$=d(the depth of the intake mouth. d is set at 0.5m

Finding the width of the intake mouth, w

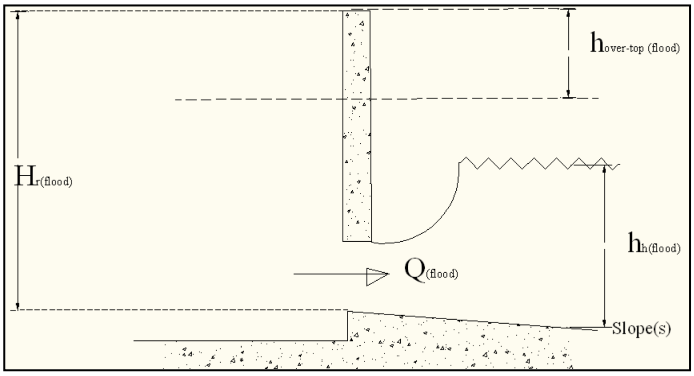


Figure 18: Intake channel under flood flow

1. Headrace flood flow

The water depth in the intake headrace channel will increase during the flood conditions. We use two equations, discharge orifice equation and the manning's equation. The correct depth is known when both the equation gives the same (or almost similar) value.

Equation 1 $$Q_{flood} = d \times w \times c_d \sqrt{2g\left(h_{r(flood)} - h_{h(flood)}\right)}$$

Equation 2 $$Q_{flood} = h_{h(flood)} \times w_h \times \frac{S^{0.5}}{n} \times \left(\frac{w_h h_{h(flood)}}{w_h + 2h_{h(flood)}}\right)^{0.667}$$

By carrying out several assumptions for h_{hflood}, we get equation1 equaling equation 2 when h_{hflood}=0.7503m at Q_{flood}=0.817m^3/s

In order to contain the flood flow of 0.75 depths, the channel walls need to be 0.8m high.

3.12. Minimum Sedimentation Tank Dimensions

The sedimentation tank is designed in a way to sink all particles bigger than 0.2mm.

Note; Wsettling is assumed to be five times the channel width, therefore $L_{settling} = \dfrac{Q}{W_{settling} \times V_o}$, $S_l = Q \times T(s) \times S$, $V = \dfrac{1}{p_d} \times \dfrac{1}{Ds}$, $C = V \times S_l$, $D = \dfrac{C}{W_{settling} \times L_{settling}}$, $D_{settling}$=channel height

Table 21: Sedimentation tank dimensions

Particle size	**0.2**	mm
Vertical velocity, Vo	**0.025**	mm
L_{entry}=$W_{settling}$=L_{exit}	**4.25**	m
$L_{settling}$	4.71	m
Silt quantity ,S	**0.5**	kg/m^3
Cleaning time interval ,T	**12**	hrs
Silt load, S_l	32400	kg
Density of sand, Ds	2600	kg/m^3
Packing density,Pd	**50**	%
Volume Capacity of silt, V	0.00077	m^3/kg
Required collection capacity, C	24.923	m^3
$D_{collection}$, D	0.41	M
$D_{settling}$	0.45	m

A slope of 2% will be used in the sedimentation tank.
A 0.3mX0.3m flushing gates will be used to 'flush' off the accumulated silt.

3.13. Spillway Minimum Dimensions

The minor flood is assumed to be 1.15 the gross flow and also

$H_{minor\ flood}$ =1.15*$H_{spillway}$

$L_{spillway} = \dfrac{Q_{min\,orflow} - Q_{gross}}{C_w \left(h_{min\,orflood} - h_{spillway}\right)^{.5}}$ and minimum freeboard $= h_{minorflood} - h_{spillway}$

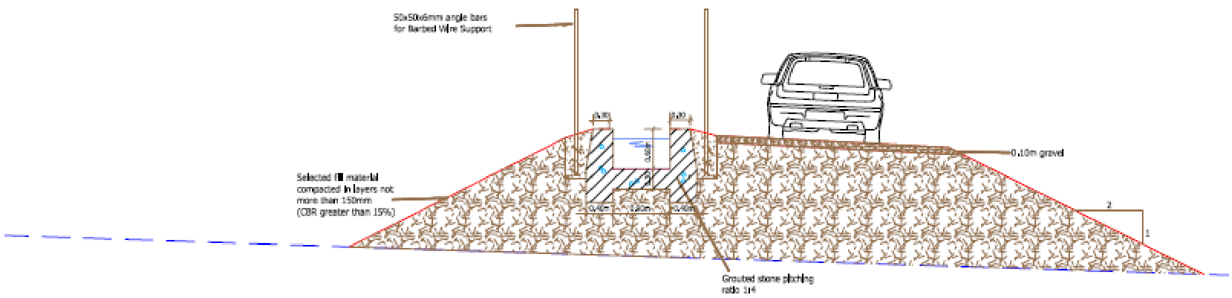
3.14. Headrace Channel

This channel is different from that which joins the intake to the sedimentation tank; this joins the sedimentation tank to the forebay. The selected channel is a rectangular section which is designed to offer optimum channel functioning while minimizing the construction costs.

Different routes for the channel were studied with a view of obtaining the most viable and economical. Whereas the 'other' side of the river has a difficult terrain and the envisaged channel route passes through land owned by many people, the selected side is characterized by gentle slopes and is owned by one person.

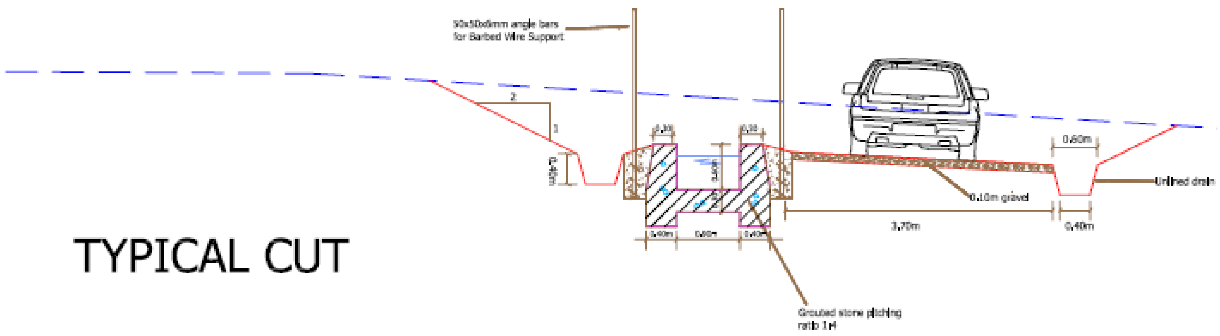
The level of the channel was established by considering a slope of 0.002 and moving in the vicinity of a single contour path to minimize mainly excavations since the soils are weak and when saturated would be unstable. Fills were preferred since in the vicinity of the channel route, there are good reserves of adequate fill material and the resulting borrow pits shall be used to trap water for farm use. The borrow pits would be distributed in such a way that they are used adequately for this purpose.
The Headrace Channel has been designed with a 3.7m wide road to ease the inspection. Culverts have been arranged to drain the water in the cut sections. Below are the typical sections.

Figure 19: Headrace Channel in typical fill



TYPICAL FILL

Figure 20: Headrace Channel in typical cut



Design

The channel slope and the dimensions selected ensure that the water moves at a velocity that prevents concrete scour and the same time permits no sedimentation along the channel. A 'small' slope was considered in order to avoid loss of head and hence optimizing gross head for power generation.

From Manning's formula;

$$Q = \frac{AR_n^{\frac{2}{3}} S^{1/2}}{n} = \frac{A^{5/3} S^{1/2}}{nP^{2/3}}$$

but for optimal dimensioning in respect of highest efficiency, Cross-sectional Area $A=2y^2$, wetted perimeter P=4y, hydraulic radius R=0.5y, channel breadth b=2y, d=y (water depth) and manning's constant n=0.0.12 and the channel slope S=1/500.

We get y=0.43m and b=0.83m, but for ease of construction of the channel and the other structures dependent on it, the dimensions are altered to be y=0.45m and b=0.85m. The channel will be 1.8km.

Table 22: Channel Specification

Desired Flow rate	**0.50**	m^3/s
slope, S	**0.002**	
Water depth y	**0.45**	m
Channel width b	**0.85**	m
hydraulic radius, R	0.225	m
manning's constant	0.012	
Area	0.383	m^2
calculated flow, Q	0.527	m^3/s

Freeboard	0.25	m
Total channel depth	0.70	m
Channel length	1.8	Km

3.15. Forebay Dimensions

Since the de-silting will be done at the beginning and the channel is 'too' long, the forebay will not be made as large as the de-silting tank. That said though, one of the purposes of

The forebay will be to settle sediments intruded into the channel e.g. leaves, stones, grass etc. Secondly the forebay shall provide water to the penstock while eliminating the possibility of vortices' formation at the penstock intake.

The reasons above form the basis for designing the forebay while taking into account the following;

1. The pipe minimum submergence S should not be violated.
   $S > 0.7 \times V \times \sqrt{d}$ Solving for S, S>0.95, therefore S=1.25m
2. A storage depth is provided for possible silt deposition C=0.4
3. A trash rack inclined at 70^{o} and dimensioned in a way to stop relatively big particles from accessing the turbine whereas letting through adequate flow.
4. A dead storage of 0.2m was considered to allow for silt accumulation.

Dpipe	L1	L2	U	W	X	Y=E	B	D	Min Pipe Submergence	Freeboard	A	F
0.610	2.3	8.67	1.8	0.85	1	6.5	1.29	3.75	1.35	0.25	1.7	1.46

All dimensions are in m

4. ELECTRO MECHANICAL EQUIPMENTS

4.1.1. Electromechanical Equipment

4.1.2. Introduction

The electromechanical equipment consists of the penstock and the equipment in the Power House. The penstock conveys water under pressure from the fore-bay into the turbine. The penstock must withstand the pressures, minimize the head loss and be of reasonable cost.

4.1.3. Power House

The role of the powerhouse is to protect the electromechanical equipment that convert the potential energy of water into electricity, from the weather hardships. The number, type and power of the turbo-generators, their configuration, the scheme head and the geomorphology of the site determine the shape and size of the building.

The following equipment will be found in the powerhouse:

Inlet gate or valve
Turbine
Gearbox if required
Generator
Control system
Switchgear
Protection systems
DC emergency supply
Power and current transformers

In low head schemes the intake, trash-rack and weir form part of the power house.

In medium and high head schemes, powerhouses are more conventional with an entrance for the penstock and a tailrace.

4.1.4. Hydraulic turbines

The purpose of a hydraulic turbine is to transform the water potential energy to mechanical rotational energy.

It is appropriate to provide a few criteria to guide the selection of the right turbine for a particular application

4.1.5. Types and configuration

The potential energy in water is converted into mechanical energy in the turbine, by one of two fundamental and basically different mechanisms:

- ❖ The water pressure can apply a force on the face of the runner blades, which decreases as it proceeds through the turbine. Turbines that operate in this way are called reaction turbines. The turbine casing, with the runner fully immersed in water, must be strong enough to withstand the operating pressure. Francis and Kaplan turbines belong to this category.

- ❖ The water pressure is converted into kinetic energy before entering the runner. The kinetic energy is in the form of a high-speed jet that strikes the buckets, mounted on the periphery of the runner. The jet is formed by reducing the diameter of the penstock drastically before it enters the Turbine. Turbines that operate in this way are called impulse turbines. The most usual impulse turbine is the Pelton.

The hydraulic power at disposition of the turbine is given by:

$P_h = \rho Q.gH$ [W]

Where:

ρQ	=	mass flow rate [kg/s]
ρ	=	water specific density [kg/m3]
Q	=	Discharge [m3/s]
gH	=	specific hydraulic energy of machine [J/kg]
g	=	acceleration due to gravity [m/s2]
H	=	"net head" [m]

The mechanical output of the turbine is given by:

P_{mec}= P_h ŋmec h [W]

Ɒ = turbine efficiency

4.1.6. Impulse turbines

4.1.7. Pelton turbines

Pelton turbines are impulse turbines where one or more jets impinge on a wheel carrying on its periphery a large number of buckets. Each jet issues water through a nozzle with a needle valve to control the flow (figure 21). They are suitable for high heads from 60 m to more than 1,000 m. The axes of the nozzles are in the plane of the runner. In case of an emergency stop of the turbine (e.g. in case of load rejection), the jet may be diverted by a deflector so that it does not impinge on the buckets and the runner cannot reach runaway speed. In this way the needle valve can be closed very slowly, so that overpressure surge in the pipeline is kept to an acceptable level (max 1.15 static pressure).

Figure 21: **Pelton turbines**

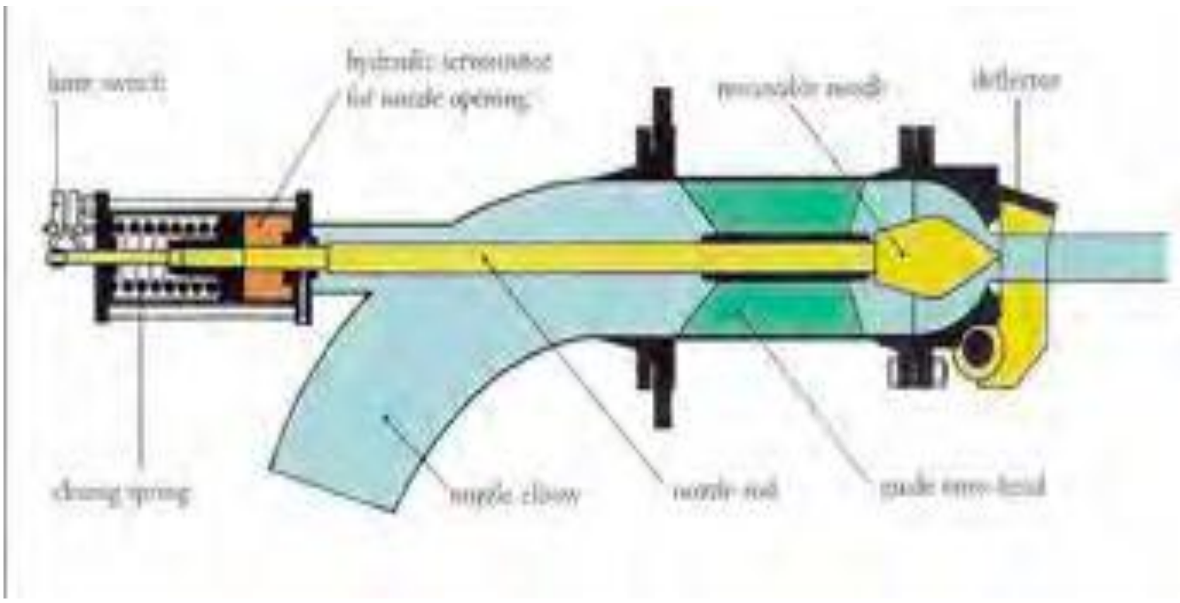


Figure 22: Nozzle

As any kinetic energy leaving the runner is lost, the buckets are designed to keep exit velocities to a minimum.

One or two jet Pelton turbines can have horizontal or vertical axis. Three or more nozzles turbines have vertical axis. The maximum number of nozzles is 6.

The turbine runner is usually directly coupled to the generator shaft. The efficiency of a Pelton is good from 30% to 100% of the maximum discharge for a one-jet turbine and from 10% to 100% for a multi-jet one.

4.1.8. Turgo turbines

The Turgo turbine is also an Impulse turbine and can operate under a head in the range of 50-250 m. Its buckets however are shaped differently from the Pelton and the jet of water strikes the plane of its runner at an angle of 20º. Water enters the runner through one side of the runner disk and emerges from the other side. It can operate between 20% and 100% of the maximal design flow.

The efficiency is lower than for the Pelton and Francis turbines. Compared to the Pelton, a Turgo turbine has a higher rotational speed for the same flow and head. A Turgo can be an alternative to the Francis when the flow strongly varies or in case of long penstocks, as the deflector allows avoidance of runaway speed in the case of load rejection and the resulting water hammer that can occur with a Francis.

4.1.9. Cross-flow turbines

This impulse turbine, also known as Banki-Michell is used for a wide range of heads overlapping those of Kaplan, Francis and Pelton. It can operate with heads between 5 and 200 m.

Water enters the turbine, directed by one or more guide-vanes located upstream of the runner and crosses it two times before leaving the turbine. This simple design makes it cheap and easy to repair in case of runner breaks due to mechanical stresses.

The Cross-flow turbines have low efficiency compared to other turbines and the important loss of head due to the clearance between the runner and the downstream level should be taken into consideration when dealing with low and medium heads. Moreover, high head cross-flow runners may have some troubles with reliability due to high mechanical stress. It is an interesting alternative when one has enough water, defined power needs and low investment possibilities, such as for rural electrification programs.

4.1.10. Reaction turbines

1.16.10.1 Francis turbines

Francis turbines are reaction turbines, with fixed runner blades and adjustable guide vanes, used for medium heads. In this turbine the admission is always radial but the outlet is axial. Figure 23 shows a horizontal axis Francis turbine. Their usual field of application is from 25 to 350 m head.

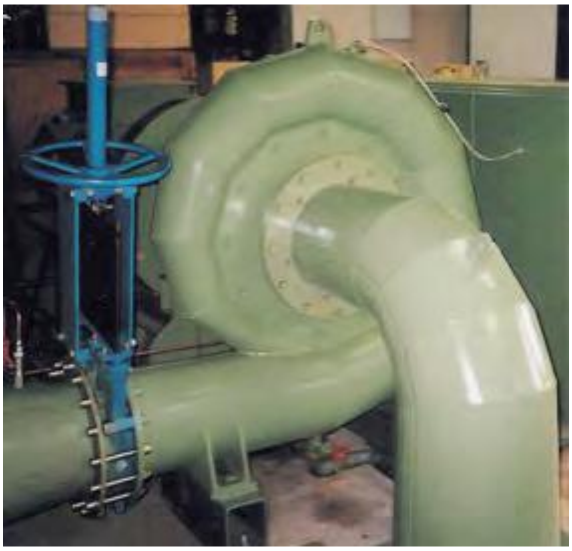


Figure 23: Francis turbine

As with Peltons, Francis turbines can have vertical or horizontal axis.

Francis turbines can be set in an open flume or attached to a penstock. For small heads and power, open flumes were commonly employed, however nowadays the Kaplan turbine provides a better technical and economical solution in such power plants. The water enters the turbine by the spiral case that is designed to keep its tangential velocity constant along the consecutive sections and to distribute it peripherally to the distributor.

As shown in figure 24, this one has mobile guide vanes, whose function is to control the discharge going into the runner and adapt the inlet angle of the flow to the runner blades angles. They rotate around their axes by connecting rods attached to a large ring that synchronize the movement off all vanes. They can be used to shut off the flow to the turbine in emergency situations, although their use does not preclude the installation of a butterfly valve at the entrance to the turbine. The runner transforms the hydraulic energy to mechanical energy and returns it axially to the draft tube.

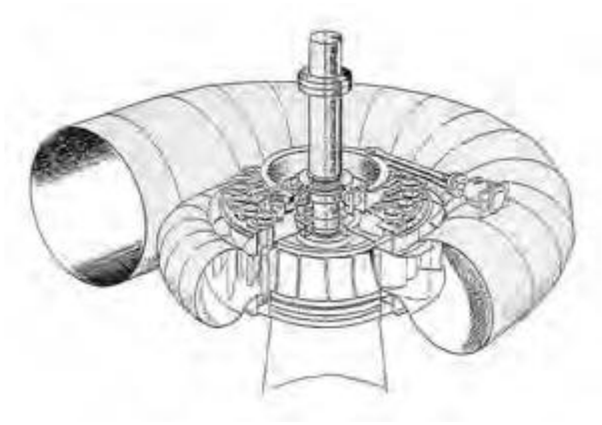


Figure 24:

Small hydro runners are usually made in stainless steel castings. Some manufacturers also use aluminum bronze casting or welded blades, which are generally directly coupled to the generator shaft.

The draft tube of a reaction turbine aims to recover the kinetic energy still remaining in the water leaving the runner. As this energy is proportional to the square of the velocity one of the draft tube objectives is to reduce the turbine outlet velocity. An efficient draft tube would have a conical section but the angle cannot be too large, otherwise flow separation will occur. The optimum angle is 7º but to reduce the draft tube length, and therefore its cost, sometimes angles are increased up to 15º.

The lower head, the more important the draft tube is. As low head generally implies a high nominal discharge, the remaining water speed at the outlet of the runner is quite important. One can easily understand that for a fixed runner diameter, the speed will increase if the flow does. Figure 26 shows the kinetic energy remaining at the runner outlet as a function of the specific speed

1.16.10.2 Kaplan and propeller turbines

Kaplan and propeller turbines are axial-flow reaction turbines; generally used for low heads from 2 to 40 m. The Kaplan turbine has adjustable runner blades and may or may not have adjustable guide- vanes. If both blades and guide-vanes are adjustable it is described as "double-regulated". If the guide-vanes are fixed it is "single-regulated". Fixed runner blade Kaplan turbines are called propeller turbines. They are used when both flow and head remain practically constant, which is a characteristic that makes them unusual in small hydropower schemes.

Figure 25

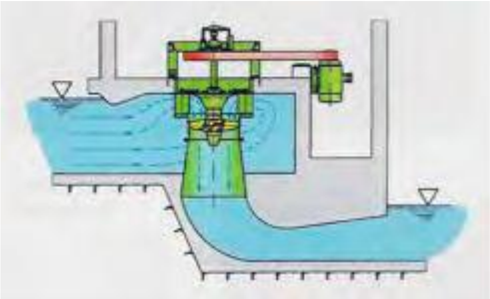


Figure 26

The double regulation allows, at any time, for the adaptation of the runner and guide vanes coupling to any head or discharge variation. It is the most flexible Kaplan turbine that can work between 15% and 100% of the maximum design discharge. Single regulated Kaplan allows a good adaptation to varying available flow but is less flexible in the case of important head variation. They can work between 30% and 100% of the maximum design discharge.

The double-regulated Kaplan illustrated in figure 1.6 is a vertical axis machine with a spiral case and a radial guide vane configuration. The flow enters in a radial manner inward and makes a right angle turn before entering the runner in an axial direction. The control system is designed so that the variation in blade angle is coupled with the guide-vanes setting in order to obtain the best efficiency over a wide range of flows and heads. The blades can rotate with the turbine in operation, through links connected to a vertical rod sliding inside the hollow turbine axis.

Table 23: Range of heads

Turbine type	**Head range in metres**
Kaplan and Propeller	$2 < Hn < 40$
Francis	$25 < Hn < 350$
Pelton	$50 < Hn < 1'300$
Crossflow	$5 < Hn < 200$
Turgo	$50 < Hn < 250$

Discharge

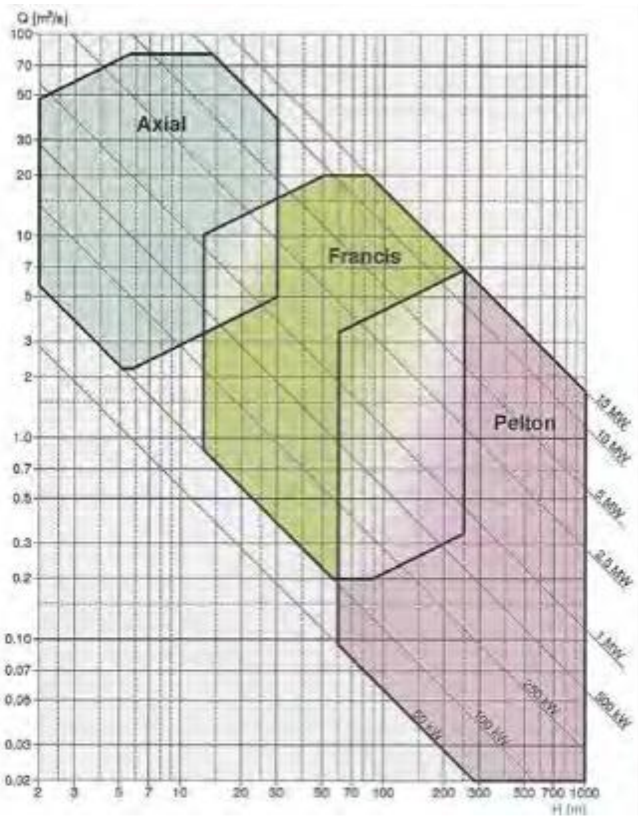


Figure 27: operational envelopes

The rated flow and net head determine the set of turbine types applicable to the site and the flow environment. Suitable turbines are those for which the given rated flow and net head plot within the operational envelopes (figure 27). A point defined as above by the flow and the head will usually plot within several of these envelopes. All of those turbines are appropriate for the job, and it will be necessary to compute installed power and electricity output against costs before making a decision. It should be remembered that the envelopes vary from manufacturer to manufacturer and they should be considered only as a guide.

As a turbine can only accept discharges between the maximal and the practical minimum, it may be advantageous to install several smaller turbines instead of one large turbine. The turbines would be sequentially started, so that all of the turbines in operation, except one, will operate at their nominal discharges and therefore will have a high efficiency. Using two or three smaller turbines will mean a lower unit weight and volume and will facilitate transport and assembly on the site. Sharing the flow between two or more units will also allow for higher rotational speed, which will reduce the need for a speed increaser.

In case of strong flow variation in the range of medium head, a multi-jet Pelton with a low rotational speed will be preferred to a Francis turbine. A similar remark can also be made for Kaplan and Francis in low heads.

The final choice between one or more units or between one type of turbine or another will be the result of an iterative calculation taking into account the investment costs and the yearly production.

Table 24: Flow and head variation acceptance

Turbine Type	Acceptance of Low Variation	Acceptance of Head Variation
Pelton	High	Low
Francis	Medium	Low
Kaplan Double Regulated	High	High
Kaplan Single Regulated	High	Medium
Propeller	Low	Low

Specific speed

The specific speed constitutes a reliable criterion for the selection of the turbine, without any doubt more precise than the conventional enveloping curves, just mentioned. If we wish to produce electricity in a scheme with 100-m net head and 0.9 m3/s, using a turbine directly coupled to a standard 1500-rpm generator we should begin by computing the specific speed according equation (6.5).

nQE 0.135

With this specific speed the only possible selection is a Francis turbine. Otherwise if we accept the possibility of using a lower speed, it could be possible to select, in addition to the Francis, a 4- nozzles Pelton with 600-rpm generator.
If we intend to install a turbine in a 400 m head, 0.42 m3/s scheme, directly coupled to a 1000-rpm generator, we will begin computing the specific speed:

nQE 0.022

Which indicates the 1 jet Pelton option, with a diameter D1 = 0.815 m according to equation (6.15).

A two or more jet Pelton would also be possible if required by a highly variable flow requiring a good efficiency at part load.

As previously explained, the Pelton turbines are generally defined by the D1/B2 ratio rather than by the specific speed. As a general rule, this ratio has to be higher than 2.7. Such a ratio cannot be obtained without model laboratory developments.

Cavitation

When the hydrodynamic pressure in a liquid flow falls below the vapor pressure of the liquid, there is a formation of the vapor phase. This phenomenon induces the formation of small individual bubbles that are carried out of the low-pressure region by the flow and collapse in regions of higher pressure. The formation of these bubbles and their subsequent collapse gives rise to what is called cavitation. Experience shows that these collapsing bubbles create very high impulse pressures accompanied by substantial noise (in fact a turbine undergoing cavitation sounds as though gravel is passing through it). The repetitive action of such collapse in a reaction turbine close to the runner blades or hub for instance results in pitting of the material. With time this pitting degenerates into cracks formed between the pits, and the metal is snatched from the surface. In a relatively short time the turbine is severely damaged and will need to be shut-off and repaired – if possible.

However cavitation is not a fatality. Laboratory developments allow for a proper hydraulic design to be defined and the operating field of the turbines to be fixed, which can both help in avoiding this problem.

It has to be noted that local cavitation can occur on Pelton buckets if the inlet edge is not properly designed or if the laboratory tested shape has not been fully respected during manufacture.

Rotational speed

The rotational speed of a turbine is directly linked to its specific speed, flow and net head. In the small hydro schemes standard generators should be selected such that the generator either coupled directly or through a gearbox to the turbine, should reach the synchronous speed, as given in table 25.

Table 25: Generator synchronization speed

Number of poles	**Frequency 50 Hz**	**60 Hz**
2	3000	3600
4	1500	1800
6	1000	1200
8	750	900
10	600	720
12	500	600

14	428	540
16	375	450
18	333	400
20	300	360
24	250	300
26	231	377
28	214	257

Runaway speed

Each runner profile is characterized by a maximum runaway speed. This is the speed, which the unit can theoretically attain in case of load rejection when the hydraulic power is at its maximum. Depending on the type of turbine, it can attain 2 or 3 times the nominal speed. Table 26 shows this ratio for miscellaneous turbines.

It must be remembered that the cost of both generator and eventual speed increaser may be increased when the runaway speed is higher, since they must be designed to withstand it.

Table 26: Runaway speeds of turbines

Turbine type	**Runaway speed n_{max}/n**
Kaplan single regulated	2.0 - 2.6
Kaplan double regulated	2.8 - 3.2
Francis	1.6 – 2.2
Pelton	1.8 – 1.9
Turgo	1.8 – 1.9

4.1.11. Turbine efficiency

The efficiency a turbine determines the ability of a turbine to exploit a site in an optimal manner. An average turbine efficiency means that the output is low and may also lead to other problems such as cavitation, and vibration which can reduce the annual energy production and damage the turbine.

The Turbine manufacturer can give efficiency guarantees based on Laboratory Test of similar turbines.

When the flow deviates from that nominal discharge so does the turbine's hydraulic efficiency. As the design discharge of reaction turbines is generally chosen to be different from the best efficiency discharge, the efficiencies given in table 27 correspond to best efficiency, but not to efficiency at design or maximum discharge.

Double regulated Kaplan and Pelton turbines can operate satisfactorily over a wide range of flow - upwards from about one fifth of rated discharge. Single regulated Kaplans have acceptable efficiency upward from one-third and Francis turbines from one half of rated discharge. Below 40% of the rated discharge, Francis turbines may show instability resulting in vibration or mechanical shock.

Propeller turbines with fixed guide vanes and blades can operate satisfactorily only over a very limited range close to their rated discharge. It should be noted that single-regulated Kaplan turbines are only efficient if it is the runner that is adjustable.

Table 27: Typical efficiencies of small turbines

Turbine type	**Best efficiency**
Kaplan single regulated	0.91
Kaplan double regulated	0.93
Francis	0.94
Pelton n nozzles	0.90
Pelton 1 nozzle	0.89
Turgo	0.85

4.1.12. Generators

Generators transform mechanical energy into electrical energy. There are two types of generators namely: synchronous and asynchronous generators.

Synchronous generators: They are equipped with a DC electric or permanent magnet excitation system (rotating or static) associated with a voltage regulator to control the output voltage before the generator is connected to the grid or load. They supply the reactive energy required by the power system when the generator is connected to the grid. Synchronous generators can run isolated from the grid and produce power since excitation is not grid-dependent

Asynchronous generators: They are simple squirrel-cage induction motors with no possibility of voltage regulation and running at a speed directly related to system frequency. They draw their excitation current from the grid, absorbing reactive energy by their own magnetism. Adding a bank of capacitors can compensate for the absorbed reactive energy. They cannot normally generate when disconnected from the grid because they are incapable of providing their own excitation current.

However, they are used in very small stand-alone applications as a cheap solution when the required quality of the electricity supply is not very high. In this case the excitation current is supplied from a bank of capacitors.

Below 1 MW, synchronous generators are more expensive than asynchronous generators.

The efficiency of asynchronous generators can be about 95 % for a 100 kW machine and can increase to 97% towards an output power of 1MW. Efficiencies of synchronous generators are slightly higher. In general, when the power exceeds some MVA a synchronous generator is indicated.

The operating voltage of the generator increases with power. The standard generation voltages of 400 V allow for the use of standard distributor transformers as outlet transformers and the use of the generated current to feed into the plant power system. Generators of some MVA are usually designed for higher operating voltages up to some kV and connected to the grid using a customized transformer. In this case an independent transformer HT/LT is necessary for the auxiliary power supply of the power plant.

Table 28: Typical efficiencies of small generators

Rated power [kW]	**Best efficiency**
10	0.910
50	0.940
100	0.950
250	0.955
500	0.960
1000	0.970

4.1.13. Generator configurations

Generators can be manufactured with horizontal or vertical axis, independently of the turbine configuration.

A flywheel is frequently used to smooth-out speed variations and assists the turbine control.

Small generators have an open cooling system, but larger units have a closed cooling circuit provided with air-water heat exchangers.

4.1.14. Exciters

The exciting current for the synchronous generator can be supplied by a small DC generator, known as the exciter, driven from the main shaft. The power absorbed by this DC generator amounts to 0.5% - 1.0% of the total generator power. Modern generators have a static exciter mounted on the shaft.

Rotating exciters.

The field coils of both the main generator and the exciter generator are usually mounted on the main shaft. In larger generators a pilot exciter with permanent magnet excitation is also used. It supplies the exciting current to the main exciter, which in turn supplies the exciting current for the rotor of the generator.

Brushless exciters

A small generator has its field coils on the stator and generates AC current in the rotor windings. A solid state rectifier rotates with the shaft, converting the AC output from the small generator into the DC, which is supplied to the rotating field coils of the main generator without the need for brushes.
The voltage regulation is achieved by controlling the current in the field coils of the small generator.

Static exciters

A static exciter is a grid connected rectifier that provides DC current to the generator field coils instead of the rotating exciter. The voltage and power factor control works in the same way as with the rotating device. Static exciters are robust, easy to maintain and have a high efficiency. The response to the generator voltage oscillations is very good.

4.1.15. Voltage regulation and Synchronization

Asynchronous generators

An asynchronous generator needs to absorb reactive power from the three-phase mains supply to ensure its magnetization is even. The mains supply defines the frequency of the stator rotating flux and hence the synchronous speed above which the rotor shaft must be driven. On start-up, the turbine is accelerated to a speed slightly above the synchronous speed of the generator, when a velocity relay closes the main line switch. From this hyper-synchronized state the generator speed will be reduced to synchronous speed by feeding current into the grid. Speed deviations from synchronous speed will generate a driving or resisting torque that balances in the area of stable operation.

Synchronous generators

The synchronous generator is started before connecting it to the mains by the turbine rotation. By gradually accelerating the turbine, the generator must be synchronized with the mains, regulating the voltage, frequency, phase angle and rotating sense. When all these values are controlled correctly, the generator can be switched to the grid. In the case of an isolated or off grid operation, the voltage controller maintains a predefined constant voltage, independent of the load. In case of the mains supply, the controller maintains the predefined power factor or reactive power.

4.1.16. Turbine control

Turbines are designed for a certain net head and discharge. Any deviation from these parameters must be compensated for by opening or closing the control devices, such as the wicket-gates, vanes, spear nozzles or valves, to keep either the outlet power, the level of the water surface in the intake, or the turbine discharge constant.

In schemes connected to an isolated network, the parameter that needs to be controlled is the turbine speed, which controls the frequency. In an off grid system, if the generator becomes overloaded the turbine slows-down therefore an increase of the flow of water is needed to ensure the turbine does not stall. If there is not enough water to do this then either some of the load must be removed or the turbine will have to be shut down. Conversely if the load decreases then the flow to the turbine is connected to the generator terminals.

In the first approach, speed (frequency) regulation is normally accomplished through flow control; once a gate opening is calculated, the actuator gives the necessary instruction to the servomotor, which results in an extension or retraction of the servo's rod. To ensure that the rod actually reaches the calculated position, feedback is provided to the electronic actuator. These devices are called "speed governors.”.

In the second approach it is assumed that, at full load, constant head and flow, the turbine will operate at design speed, so maintaining full load from the generator; this will run at a constant speed. If the load decreases the turbine will tend to increase its speed. An electronic sensor, measuring the frequency, detects the deviation and a reliable and inexpensive electronic load governor, switches on pre-set resistance and so maintains the system frequency accurately. The controllers that follow the first approach do not have any power limit. The Electronic Load Governors, working according to the second approach rarely exceed 100 kW capacity.

Speed Governors

A governor is a combination of devices and mechanisms, which detect speed deviation and convert it into a change in servomotor position. A speed-sensing element detects the deviation from the set point; this deviation signal is converted and amplified to excite an actuator, hydraulic or electric, that controls the water flow to the turbine. In a Francis turbine, where there is a reduction in water flow you need to rotate the wicket-gates. For this, a powerful governor is required to overcome the hydraulic and frictional forces and to maintain the wicket-gates in a partially closed position or to close them completely.

Several types of governors are available varying from old fashioned purely mechanical to mechanical-hydraulic to electrical-hydraulic and mechanical-electrical. The purely mechanical governor is used with fairly small turbines, because its control valve is easy to operate and does not require a big effort. These governors use a fly-ball mass mechanism driven by the turbine shaft. The output from this device - the fly-ball axis descends or ascends according to the turbine speed - directly drives the valve located at the entrance to the turbine.

In the past, the most commonly used type was the oil-pressure governor (Fig 6.35) that also uses a fly-ball mechanism, which is lighter and more precise than that used in a purely mechanical governor. When the turbine is overloaded, the fly-balls slowdown, the balls drop, and the sleeve of the pilot valve rises to open access to the upper chamber of the servomotor. The oil under pressure enters the upper chamber of the servomotor to rotate the wicket-gates mechanism, increase the flow, and consequently the rotational speed and the frequency.

In a modern electrical-hydraulic governor a sensor located on the generator shaft continuously senses the turbine speed. The input is fed into a summing junction, where it is compared to a speed reference. If the speed sensor signal differs from the reference signal, it emits an error signal (positive or negative) that, once amplified, is sent to the servomotor so this can act in the required sense. In general the actuator is powered by a hydraulic power unit (photo 6.10) consisting of a sump for oil storage, an electric motor operated pump to supply high pressure oil to the system, an accumulator where the oil under pressure is stored, oil control valves and a hydraulic cylinder. All these regulation systems, as have been described, operate by continuously adjusting the wicket-gates position back and forth. To provide quick and stable adjustment of the wicket-gates, and/or of the runner blades, with the least amount of over or under speed deviations during system changes a further device is needed. In oil pressure governors, as may be seen in figure 6.37, this is achieved by interposing a "dash pot" that delays the opening of the pilot valve. In electrical-hydraulic governors the degree of sophistication is much greater, so that the adjustment can be proportional, integral and derivative (PID) giving a minimum variation in the controlling process. An asynchronous generator connected to a stable electric grid, does not need any controller, because its frequency is controlled by the mains. Notwithstanding this, when the generator is disconnected from the mains the turbine accelerates up to runaway speed of the turbine. Generator and speed increaser have to be designed to withstand this speed long enough until the water flow is closed by the controlling system (guide vanes or valve).

To ensure the control of the turbine speed by regulating the water flow, certain inertia of the rotating components is required. Additional inertia can be provided by a flywheel, on the turbine, or the generator shaft. When the main switch disconnects the generator, the power excess accelerates the flywheel; later, when the switch reconnects the load, the deceleration of this inertia flywheel supplies additional power that helps to minimize speed variation. it should not be forgotten that the control of the water flow introduces a new factor: the speed variations on the water column formed by the waterways.

The flywheel effect of the rotating components is stabilizing whereas the water column effect is destabilizing.

To achieve good regulation, it is necessary that Tm/Tw > 4. Realistic water starting times do not exceed 2.5 sec. If it is larger, modification of the water conduits must be considered - either by decreasing the velocity or the length of the conduits by installing a surge tank. The possibility of adding a flywheel to the generator to increase the inertia rotating parts can also be considered. It should be noted that an increase in the inertia of the rotating parts would improve the water-hammer effect and decrease the runaway speed.

4.2. Switchgear equipment

In many countries the electricity supply regulations place a statutory obligation on the electric utilities to maintain the safety and quality of electricity supply within defined limits. The independent producer must operate his plant in such a way that the utility is able to fulfill its obligations. Therefore various associated electrical devices are required inside the powerhouse for the safety and protection of the equipment.

Switchgear must be installed to control the generators and to interface them with the grid or with an isolated load. It must provide protection for the generators, main transformer and station service transformer. The generator breaker, either air, magnetic or vacuum operated, is used to connect or disconnect the generator from the power grid. Instrument transformers, both power transformers (PTs) and current transformers (CTs) are used to transform high voltages and currents down to more manageable levels for metering. The generator control equipment is used to control the generator voltage, power factor and circuit breakers.

The asynchronous generator protection must include, among other devices: a reverse-power relay giving protection against motoring; differential current relays against internal faults in the generator stator winding; a ground-fault relay providing system backup as well as generator ground-fault protection, etc. The power transformer protection includes an instantaneous over-current relay and a timed over-current relay to protect the main transformer when a fault is detected in the bus system or an internal fault in the main power transformer occurs.

The independent producer is responsible for earthing arrangements within his installation. This must be designed in agreement with the public utility. The earthing arrangement will be dependent on the number of units in use and the independent producer's own system configuration and method of operation.

Metering equipment must be installed at the point of supply to record measurements according to the requirements of the electric utility.

Figure 6.38 shows a single-line diagram corresponding to a power plant with a single unit. In the high voltage side there is a line circuit breaker and a line disconnection switch - combined with a grounding switch - to disconnect the power generating unit and main transformer from the transmission line. Metering is achieved through the corresponding P.T and C.T. A generator circuit breaker is included as an extra protection for the generator unit. A transformer provides energy for the operation of intake gates, shutoff valves, servomotors, oil compressors etc. in the station service.

Greater complexity may be expected in multiunit stations where flexibility and continuity of service are important.

4.3. Automatic control

Small hydro schemes are normally unattended and operated through an automatic control system.

Because not all power plants are alike, it is almost impossible to determine the extent of automation that should be included in a given system, but some requirements are of general application:

a) The system must include the necessary relays and devices to detect malfunctioning of a serious nature and then act to bring the unit or the entire plant to a safe de-energized condition.

b) Relevant operational data of the plant should be collected and made readily available for making operating decisions, and stored in a database for later evaluation of plant performance.

c) An intelligent control system should be included to allow for full plant operation in an unattended environment.

d) It must be possible to access the control system from a remote location and override any automatic decisions.

e) The system should be able to communicate with similar units, up and downstream, for the purpose of optimizing operating procedures.

f) Fault anticipation constitutes an enhancement to the control system. Using an expert system, fed with baseline operational data, it is possible to anticipate faults before they occur and take corrective action so that the fault does not occur.

Measurement of water level, wicket-gate position, blade angles, instantaneous power output, temperatures, etc. A digital-to-analogue converter module to drive hydraulic valves, chart recorders, etc. A counter module to count generated kWh pulses, rain gauge pulses, flow pulses, etc. and a "smart" telemetry module providing the interface for offsite communications, via dial-up telephone lines, radio link or other communication technologies. This modular system approach is well suited to the widely varying requirements encountered in hydropower control, and permits both hardware and software to be standardized. Cost reduction can be realized through the use of a standard system and modular software allows for easy maintenance.

Automatic control systems can significantly reduce the cost of energy production by reducing maintenance and increasing reliability, while running the turbines more efficiently and producing more energy from the available water.

With the tremendous development of desktop computers, their prices are now very low. Many manufacturers supply standardized data acquisition systems. New and cheap peripheral equipment, easily connected to a portable computers, are the .“watch-dogs.”- helping to monitor and replace control equipment in the event of failure is available and easy to integrate at low price. Improved graphic programming techniques assist the development of easy to use software with graphic user interfaces. Due to the rapid development of digital technologies, the differences between hardware platforms such as PLCs, micro-controllers and industry PCs, disappear for the operator.

4.4. Ancillary electrical equipment

4.4.1.1. Plant service transformer

Electrical consumption including lighting and station mechanical auxiliaries may require from 1 to 3 percent of the plant capacity; the higher percentage applies to micro hydro (less than 500 kW).
The service transformer must be designed to take these intermittent loads into account. If possible, two alternative supplies, with automatic changeover, should be used to ensure service in an unattended plant.

4.4.1.2. DC control power supply

It is generally recommended that remotely controlled plants are equipped with an emergency 24 V DC back-up power supply from a battery in order to allow plant control for shutdown after a grid failure and communication with the system at any time. The ampere-hour capacity must be such that, on loss of charging current, full control is ensured for as long as it may be required to take corrective action.

4.4.1.3. Headwater and tailwater recorders

In a hydro plant, provisions should be made to record both the headwater and tailwater. The simplest way is to fix, securely in the stream, a board marked with meters and centimeters in the style of a leveling staff, however someone must physically observe and record the measurements.

In powerhouses provided with automatic control the best solution is to use transducers connected to the computer via the data acquisition equipment.

Nowadays measuring units - a sensor - records the measurement variable and converts it into a signal that is transmitted to the processing unit. The measurement sensor must always be installed at the measurement site, where the level has to be measured. - Usually subject to rough environmental well protected environment easily accessible for operation and service.

There is a wide range of sensors each one using a variety of measuring principles. It must be realized that the point of the level measurement needs to be selected carefully in order to represent the whole forebay. According to the Bernoulli principle, a change in the current speed causes a change in the dynamic pressure and consequently in the apparent water level as measured by the pressure sensor. If the measurement site is located in the inflow or outflow structures, where high current velocities can occur, the measurement will give false results.

The level sensor can transmit the signal by using the hydrostatic method or the pneumatic (bubble) method . In the first method care should be taken so that all the tubes for pressure transmission are dimensioned and laid in such a way that they cannot be obstructed nor air allowed accumulating within them. In the second, the sensor orifice is located lower than the corresponding level at the start of the measurement, and no water can penetrate and collect in the lines. floating material can damage the instrument. The best solution is the concealed assembly of all parts together within the wall.

4.4.1.4. Outdoor Substation

The so-called water-to-wire system usually includes the substation. A line breaker must separate the plant including the step-up transformer from the grid in case of faults in the power plant. PTs and CTs for kWh and kW metering are normally mounted at the substation, at the connecting link between the plant-out conductors and the take-off line to the grid. In areas with very high environmental sensitivity the substation is enclosed in the powerhouse, and the transmission cables, leave it along the penstock. Lightning arresters for protection against line surges or lightning strikes in the nearby grid are usually mounted in the substation structure.

5. ECONOMIC ANALYSIS

ESIA RIVER MINI HYDRO PROJECT SCHEME

5.1. Economic Rate of Return (ERR)

The Economic Rate of Return (ERR) is defined as the net benefit to the society as a whole, with respect to the cost and benefit of an investment. It is a broader term than the more narrowly defined term of Financial Internal Rate of Return, that is more widely utilized in traditional financial analysis.

In the case of this particular engagement, Africa Power Initiative Limited has been asked to analyze and review the wider economic benefits of developing and implementing the mini hydro electric power schemes within the context of climate change and its effect on poverty reduction. Therefore, in addition to analyzing the strict financial returns of the three mini hydro schemes, we have also done a broader review of the societal economic impacts and thereafter proceeded to give our recommendations.

The three mini hydro schemes under this engagement were deemed to have differing financial profiles. However, they all did share a central theme in their importance towards alleviating the grinding poverty that is prevalent in the rural Uganda households within which these schemes would be implemented. Specifically, the additional benefits that would accrue t the larger society as a result of these projects include, but are not limited to:

1. Employment opportunities to the local population during the construction as well as post-construction period: During the construction period, there will be the possibility for unskilled as well skilled labor, both manual and technical on the job site. There could also be an additional opportunity for provision of services to the job site employees by the adjacent population. these may include provision of food, shelter, transportation and other related services.

2. Post-construction opportunities include the on-going maintenance and operations of the installed equipment; site security and landscaping maintenance; and any resultant benefits from dual use of the newly installed and operational scheme. Here we have in mind the possibility of having farming activities arising from the potential of site damming, if such a path selected.

It is estimated that just provision of meals to the construction workers during the construction period of approximately 18 months, could result in an additional revenue generation at the local of close to $80-$100,000.

5.2. Net Present Value Analysis

Taking an estimated rate of interest (discount) of 5%, a term of 20 years, and a debt/equity ration of 70/30, the results for Esia River are as follows:

ELECTRICITY PRODUCTION

ESIA RIVER MINI HYDRO

Cost of Project *2	$2,603,503	Loan Interest	5.00%
Head (m)	31.7	Debt	70%
Plant Factor	75%	Term	20
Tariff *1	$0.109	Loan Amt	$1,822,452
Carbon Credit Price	$15	Equity Amt	$781,051
Est. Annual Credits	374	Annual O&M *3	$5,000
Design Flow, m^3/s	0.5	Tax Rate	30%
Installed Capacity, KW	110	Carbon Credits, yrs	10
Annual Output, KWh	722,700		
Annual Revenue *4	$84,384		
Main Cost Summary	**$2,603,503**	NPV, Project	($1,666,253)
Preliminaries	$101,700	Payback Period	NA
Civil Works	$1,621,204	IRR, Project	(4.70%)
Supervisory	$118,073	IRR, Equity - note	
Contingencies	$123,976	***(Note: Equity IRR cannot be***	
Electromechanical	$638,550	***obtained with all negative cash flows)***	
		Cost p/KW	$23,668

Notes:

*1	Uganda REFiT may not apply to projects under 500Kw capacity. if applicable, it is estimated at USD$0.109
*2	Excludes VAT
*3	O&M escalates at annually 7.08%
*4	Includes estimated Carbon Credit sales of $5,610 for 10 years.

As can be seen from the above, the Esia River Mini Hydro project scheme does not have a favorable financial profile when viewed strictly from a traditional financial analysis standpoint. Due to the negative cash flows projected for the project, there is no possibility of calculating the payback period because this point is never attained.

The main reason for this is that the Esia River has a very erratic flow pattern that is very low (almost non-existent) during the dry season. Our spot checks did reveal that there are instances of extremely heavy rainfall that results in flooding, although these instances are not frequent. Additionally, the land around the site is very flat and does not lend itself very well to damming in order to contain the and regulate the water flow.

However, viewed from the broader context of the societal impact and the effects of climate change on poverty reduction, a different and much more favorable view emerges. The implementation of the project will result in avoided costs for:

i) utilization of local biomass such as trees for domestic, and to a limited extend, commercial electrification;

ii) saved foreign exchange that would have had to be spent on the equivalent production of diesel powered generators; and

iii) with sound planning, the local community may implement some limited dual use of the project site by engaging in very limited damming in order to provide a potential for some local farming activities. The effects of this will benefit the environment and help to mitigate the effects of climate change; while any resulting farming produce will go a long way towards reducing poverty.

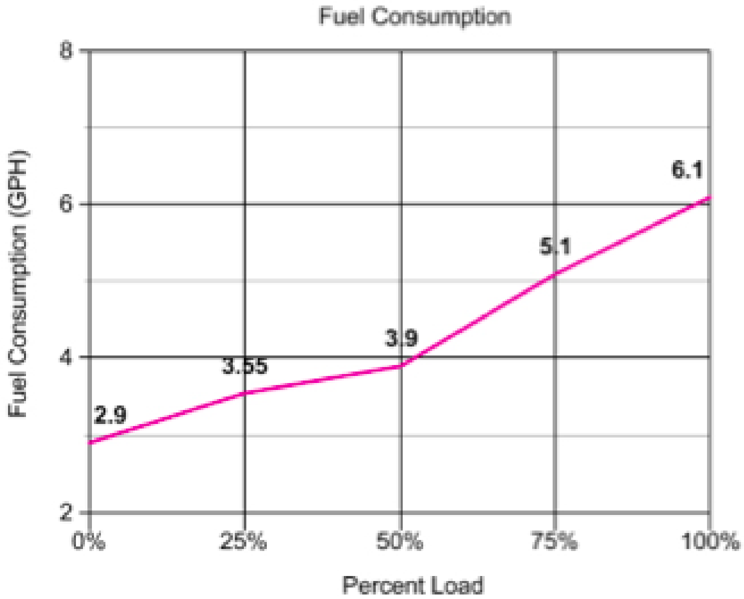
5.3. Avoided Costs Analysis

A typical 100KW Diesel Powered Generator, as shown below will consume approximately 6.1 gallons of fuel per hour at full load. If this machine runs for the equivalent amount of hours per year as the proposed 100KW mini hydro scheme at Esia River, of approximately 6,570 hours per year, it will consume approximately 40,077 gallons (152,292 litres) of diesel fuel per year.

With the current diesel prices in Northern Uganda hovering around UGX 2500-3,000 per litre (i.e. UGX 11,400 p/gallon), then utilizing the generator at equivalent usage would result in a cost of UGX (40,077 * 11,400) = 456,877,800. This is equivalent to approximately USD$207,671 (at USD$1:UGX2,200). There are also the avoided emissions of harmful CO_2 into the atmosphere by running the diesel powered generator for this amount of time per year.

Equally important is the savings of valuable foreign exchange in purchasing 40,077 gallons of diesel fuel annually for the generator.

Perkins 100 kW Diesel Generator



Consumption	
Consumption at 1/2 load	3.9 gallons/hour
Consumption at 3/4 load	5.1 gallons/hour
Consumption at full load load	6.1 gallons/hour

5.4. Sensitivity Analysis

The Esia River mini hydro project scheme is a small project, estimated by the project technical team at approximately 110KW. In conducting the financial and economic analysis, a conservatively low discount (borrowing) rate of 5% was utilized; as well as a proposed debt/equity of 70%/30%. This was done more for illustrative purposes in conducting a "traditional" financial analytical review of the project. Due to its site specific particulars, the installed cost for this project has been estimated at a very high figure of approximately $23,668 per/KW.

In conducting the sensitivity analysis, therefore, the key variable that we felt had the most impact on the project was the construction cost, and its effect on the

Net Present Value of the project. Below is a Sensitivity Matrix illustrating the various points at which the NPV shifts from negative to positive.

Sensitivity Subsidy Amt	Project Cost with subsidy	Installed Cost p/KW	NPV
85%	$390,525	$3,550	$441,345
75%	$650,876	$5,917	$193,392
70%	$781,051	$7,100	$69,416
65%	$911,226	$8,284	($54,561)
60%	$1,041,401	$9,467	($178,537)
50%	$1,301,752	$11,834	($426,490)
40%	$1,562,102	$14,201	($674,442)
30%	$1,822,452	$16,568	($922,395)
0%	$2,603,503	$23,668	($1,666,253)

In performing the above exercise, we presumed that overall project construction costs could be lowered, or otherwise off-set, via a construction subsidy. This subsidy can be obtained from a number of sources, such as the friends and donors to the project for the benefit of the community, and we highly recommend this.

From the above, it can be seen that as the amount of subsidy is increased, the project's financial profile begins to progressively improve. The key inflection point is reach at approximately 70% of subsidy, resulting in a project cost of approximately $780,000 - $800,000, where the NPV becomes positive. We are strongly in support of this project receiving a subsidy as above illustrated.

5.5. Conclusion & Recommendations

A quick overview of the overall societal benefits reveals the following:

		20 Years
1. Project Direct Revenues *1	$ 81,579	$1,631,580
2. Construction Period Revenues *2	$250,000	$ 250,000
3. Worker Labor/Employment *3	$250,000	$ 250,000
4. Misc Other, post construction	$100,000	$2,000,000
5. Avoided Fuel Costs (Savings) *4	$207,671	$4,153,434
Total		**$8,285,015**

Project Cost:	$3,072,134
Economic Benefits:	$8,285,015
Gross Economic Return:	169%
Annualized Economic Return:	5.09%

Notes:
*1: Carbon credit revenue is only for 10 years
*2: Consists of potential provision of meals, transportation and other directly consumable services during construction
*3: Est. $15 p/day; 15 workers; 18 months ($121,500), plus, Supervisory costs ($118,073) per BOQ
*4: (40,077 gallons *UGX11,400 p/gal) * 20

Alternate Scenario:
We can also view the economic benefits that would accrue over a 20 year period in current dollar value, i.e. present value. This is computed by discounting to today's value the future accumulation of the economic benefits. Therefore:

	Annual	PV of 20 Years
1. Direct Revenues, Electric Sales	$78,774	$ 981,700
2. Direct Revenues, Carbon Credit	$ 5,610	$ 43,320
2. Construction Period Revenues	$250,000	$ 250,000
3. Worker Labor/Employment	$250,000	$ 250,000
4. Misc Other, post construction	$100,000	$ 1,246,221
5. Avoided Fuel Costs (Savings)	$207,671	$ 2,588,040
Total		**$ 5,359,278**

Current Project Cost:	$ 2,603,503
Present Value of Economic Benefits:	$ 5,359,278
Net Economic Benefits:	**$ 7,962,781**

Within the above contextual ERR analysis, it is recommended that the project be implemented and developed. It can be observed from the analysis that the annualized economic return over a 20 year period, is essentially equivalent to the estimated discount rate.

The costs of maintaining the status quo and not developing this project are outweighed by the benefits of development. This is clearly demonstrated by the Net Economic Benefits as shown above. If the project is developed as an off-grid application for the benefit of the local community's needs, then the more traditional financial analytical approach would have less of a decision making bearing on the best course of action for this project. Further, the sensitivity analysis also gives guidance on quantifying the amount of "soft" investment (or subsidy) that would be beneficial for the project.

The particular form of development for this project that is recommended is for the community in the area to formulate a joint and mutually agreeable approach and form a cooperative entity that will take the role of project developer. As an added alternative, it could also be considered that this project be developed as an off-grid project to serve the immediate community within which it is located.

This will likely result in a higher benefit from Carbon Credit sales, thus a greater community economic return.

CARBON MARKET INITIATIVE

This project would benefit greatly from the utilization of the Carbon Market Initiative (CMI). In Uganda there is an incumbent organization named the "Uganda Carbon Bureau" which the project developer is highly recommended to make contact with. They can be reached at Telephone +256 414 200988 Plot 47 Lubowa Estate, Clocktower, Kampala, Uganda. Their web site is: www.ugandacarbon.org

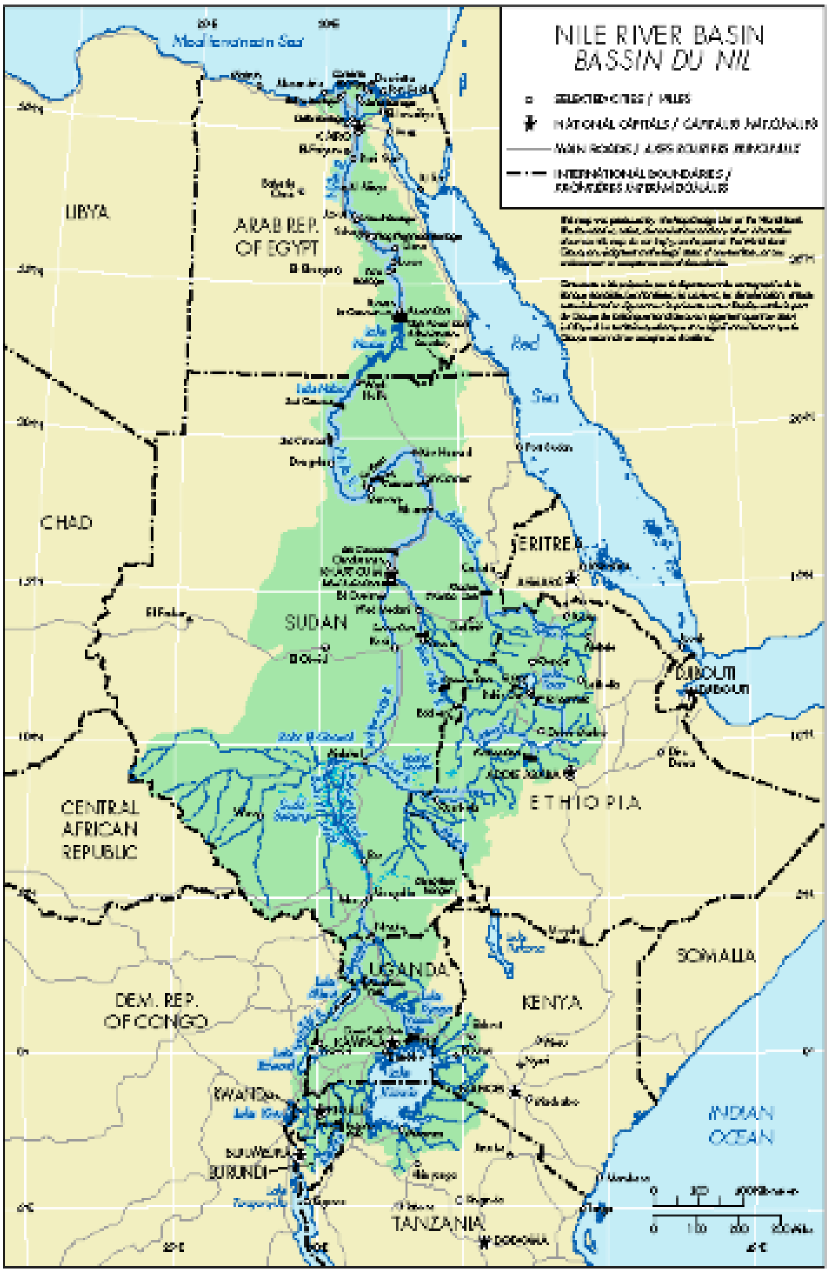
At the currently estimated electricity production rate, the Esia River mini hydro power scheme can receive approximately $5,600 per year for 10 years of carbon credit revenues. This estimate is based on receipt of approximately 374 credits, priced t roughy $15 per credit ton offset, per year.

In addition, several of the multilateral agency donors with offices in Uganda have recently began staffing their local secretariats with renewable energy experts, whose function is to disseminate and identify projects in which their governments can invest in. This is driven primarily by the desire of these "carbon credit buyers" to off-set their polluting industrial complexes in their home nations.

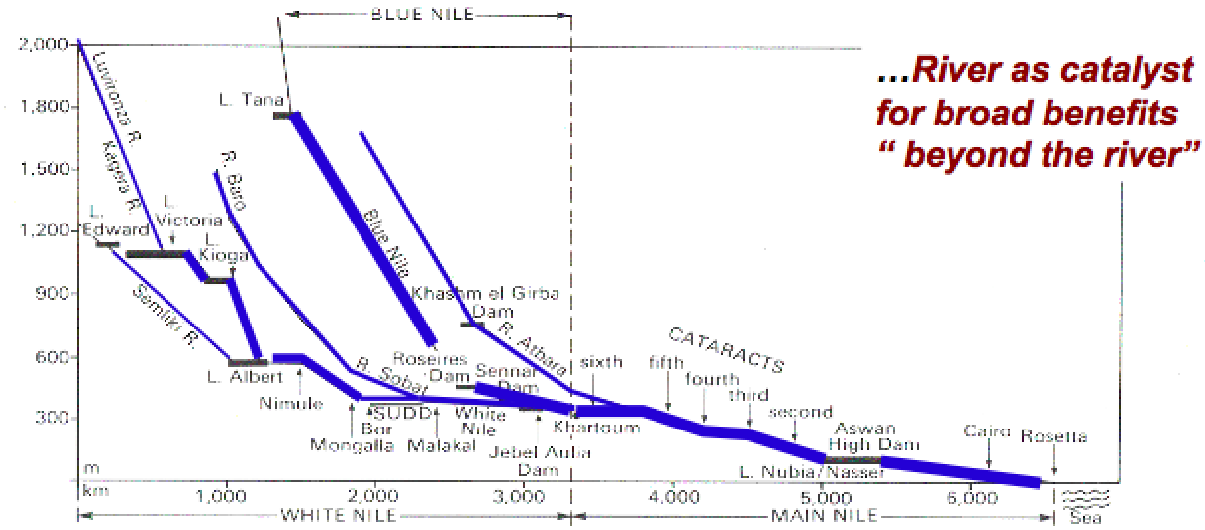
Esia River hydro scheme can therefore take advantage of this situation by leveraging the scheme's renewable energy credentials in exchange for financial benefits under the carbon credit market initiative.

NILE BASIN INITIATIVE

According to the publicly available information, the Nile Basin Initiative (NBI) is an inter-governmental organization dedicated to equitable and sustainable management and development of the shared water resources of the Nile Basin. NBI Member States include Burundi, Democratic Republic of Congo, Egypt, Ethiopia, Kenya, Rwanda, Sudan, Tanzania and Uganda. Eritrea is as an observer.



The NBI was established on February 22, 1999 in Dar es Salaam, by Ministers responsible for Water Affairs of each of the nine Member States. The Nile Council of Ministers (Nile-COM) agreed on a Shared Vision which states: ‘to achieve sustainable socio-economic development through the equitable utilization of and benefit from the common Nile Basin water resources’.



The NBI also has an investment arm named the 'Subsidiary Action Program' (SAP) to guide Nile cooperation. The SAP is the investment arm of NBI focusing on preparation of investment projects that are trans-boundary in nature.

Due to the fact the Esia River project is very close to the border with South Sudan, the project developer is advised to liaise with the NBI secretariat in Entebbe in order to explore the potential to be included among the selected project by NBI for further investment review. If the Esia project can further offset its cost structure with non-debt, public sector financing, then the project's impact at the local level will be greatly beneficial.

6. APPENDICES

6.1. APPENDIX 1: BILL OF QUANTITIES

a) MOBILIZATION

MOBILIZATION					
This includes the mobilization of all forces and equipment during the period of the Contract, except for the mobilization of forces and equipment for Foundation Drilling and Grouting they will be compensated for under grouting works Bid Items. The contractor shall include in this appendix full breakdown and details of this item along with price details , if this appendix left blank , this cost of mobilization is deemed to be included in the contract amount . and no separate payment will be made to the contractor in this respect .					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
A	allow a provisional sum of USD 45,000(Forty Five Thousand United States Dollars) for demobilisation of works	PS	1.00	45,000.00	45,000.00
	Total Amount to be carried to Item 1.1 of BOQ No. 1				45,000.00

b) **DEMOBILIZATION**

DEMOBILIZATION					
This includes the demobilization of all forces and equipment during the period of the Contract, except for the demobilization of forces and equipment for Foundation Drilling and Grouting they will be compensated for under grouting works Bid items. The contractor shall include in this appendix full breakdown and details of this item along with price details , if this appendix left blank , this cost of demobilization is deemed to be included in the contract amount . and no separate payment will be made to the contractor in this respect					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
A	allow a provisional sum of USD 40,000(Forty Thousand United States Dollars) for demobilisation of works	PS	1.00	40,000.00	40,000.00
	Total Amount to be carried to Item 1.1 of BOQ No. 1				40,000.00

c) **Preparatory Works and Facilities**

BILL NO. 1 : Preparatory Works and Facilities					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	BILL NO. 1				
	PRELIMINARIES				
	-				
	-				
A	Mobilization Carried from Appendix 1	LS	1.00	45,000.00	45,000.00
					-
B	Demobilization Carried from Appendix 2	LS	1.00	40,000.00	40,000.00
					-
C	Construction Surveys and setting out	LS	1.00	5,000.00	5,000.00
					-
D	Supply erection and maintenance of a permanent building for the field office (300 m2)and laboratory (400 m2) including all equipment and furniture	LS	1.00	6,250.00	6,250.00
					-
E	Supply, Installation and complete chain link fence for the office and laboratory site	LM	60.00	7.50	450.00
					-
F	Contractor's Quality Control	LS	1.00	5,000.00	5,000.00
Total for Bill No. 1 Carried to summary					101,700.00

d) **BILL NO. 2 DIVERSION AND ACCESS ROADS**

BILL NO. 2 DIVERSION AND ACCESS ROADS					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	BILL NO. 2				
	CIVIL WORKS				
	ELEMENT NO.1				
	DIVERSION AND ACCESS ROADS				
	Earth works and site clearance				
A	Clear site of all trees, bushes, shrubs and under growth including grubbing up roots and removing away from site	SM	28,577	1.25	35,721.25
				-	-
B	Excavate average 200mm deep to remove top soil: remove from site	SM	28,577	1.67	47,628.33
				-	-
C	Allow for maintaining and upholding sides of excavation : clear off all fallen material, rubbish.	ITEM	1	1,250.00	1,250.00
D	Allow for keeping the whole of the excavation free from general water.	ITEM	1	1,250.00	1,250.00
				-	-
	CART AWAY AND FILLING			-	-
				-	-
E	filling with stablised marrum to make up levels in layers of 250mm thick including compacting	CM	26,364	10.42	274,625.00
F	cut to reduce levels to formation level of the diversion	CM	1,085	6.25	6,781.25
G	Extra over excavation for excavating in rock.	CM	7,144	50.00	357,212.50
	CUTTING TREES			-	-

H	Cut down trees and grub up roots and chop-up and remove all arising from site : large trees girth not exceeding 600mm girth	**NO**	368	83.33	30,666.67
I	Ditto : girth 600 - 900mm girth.	**NO**	184	125.00	23,000.00
				-	-
	CHANEL FOUNDATION AND MASONRY WALLS			-	-
J	provide and place stone masonary works to foundations of the weir and abutments average thickness 400mm in cement sand mortar mix 1:4	CM	885	83.33	73,748.88
K	masonry stone work to bases and walls in cement sand mortar mix 1:4	CM	1,991	83.33	165,934.98
				-	-
	PLASER TO CHANEL WALLS			-	-
L	12mm thick cement sand plaser to masonry surfaces of weir and abutments mix 1:4	SM	7,559	6.25	47,245.38
Total for Element No. 1 Carried to collection					1,065,064.24
					2/1/1
	FENCING			-	-
M	barbed wire fencing 2000mm high comprising 65mm x 65mm x 6mm angle line poles, straining poles and corner poles all cast in concrete bases, barbed wire guage 10 in three lines at 600mm c/c including foundation excavation and back filling. Fenced on	LM	3,687	7.50	27,655.83

	either sides of the channel				
				-	-
N	Extra for corner posts with 2no. 65 x65mm x6mm straining posts 3240mm long on including 65x65x6mm angle struts including concrete grade 25 on bases	No	120	20.83	2,500.00
				-	-
O	Extra for intermidiate posts with 2no. 65 x65mm x6mm straining posts 3240mm long on including 65x65x6mm angle struts including concrete grade 25 on bases	No	1,229	18.75	23,043.75
	TRASH SCREEN				-
P	supply and install steel trash screen for the intake shaft complete including all embedded parts and all requirements as per engineer's drawingings	SM	2	400.00	876.00
					-
Total for Element No. 1 Carried to collection					54,075.58
	Element 1. collection				
	Page No. 2/1/1				1,065,064.24
	Page No. 2/1/2				54,075.58
Total for Element No. 1 Carried to Bill ! Summary					1,119,139.82

					2/1/2
BILL NO. 3. The Weir and the Aburtments					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	<u>BILL NO. 2</u>				
	<u>CIVIL WORKS</u>				
	<u>ELEMENT NO.2</u>				
	<u>WEIR AND ABUTMENTS</u>				
	<u>Earth works and site clearance</u>				
A	Clear site of all trees, bushes, shrubs and under growth including grubbing up roots and removing away from site	SM	88	1.25	110.38
					-
B	Excavate average 200mm deep to remove top soil: remove from site	SM	88	1.67	147.17
C	Mass excavation to reduce levels not exceeding 2.0m commencing from stripped level; deposit in spoil heaps where directed on site	CM	177	6.25	1,103.75
D	Extra over excavation for excavating in rock.	CM	119	50.00	5,960.25
					-
E	Allow for maintaining and upholding sides of excavation : clear off all fallen material, rubbish.	ITEM	1	1,250.00	1,250.00
F	Allow for keeping the whole of the excavation free from general water.	ITEM	1	1,250.00	1,250.00
					-
	CART AWAY AND FILLING				-
G	Load from spoil heaps and cart away excavated material from site.	CM	296	6.25	1,848.78

					-
	CUTTING TREES				-
H	Cut down trees and grub up roots and chop-up and remove all arising from site : large trees girth not exceeding 600mm girth	NO	3	83.33	250.00
I	Ditto : girth 600 - 900mm girth.	NO	2	125.00	250.00
	WEIR & ABUTMENT FOUNDATION				-
J	provide and place stone masonary works to foundations of the weir and abutments average thickness 400mm in cement sand mortar mix 1:4	CM	6	83.33	476.33
	WEIR & ABUTMENT BASES AND WALLS				-
K	masonry stone work to weir and abutment bases in cement sand mortar mix 1:4	CM	6	83.33	476.33
L	Ditto to abutment walls	CM	50	83.33	4,166.67
M	Ditto to weir	CM	180	83.33	14,983.33
	GABIONS				-
N	Provide and place galvanised gabion boxes, size 2x1x1 inclusive of rockfill as specified or directed by the Engineer	CM	17	50.00	850.00
Total for Element No. 2 Carried to collection					33,122.99
					2/2/3
					-
	PLASER TO WEIR & ABUTMENT				-
O	12mm thick cement sand plaser to masonry surfaces of weir and abutments	SM	561	6.25	3,503.75

	mix 1:4				
	FENCING				-
P	2400mm high steel grill built around the weir structure including the abutments comprising of pointed tops in steel hollow sections of 50x 50mm welded and built into boundary wall pre painted before brought to site	LM	120	104.17	12,500.00
					-
Q	Extra for corner posts with 2no. 65 x65mm x6mm straining posts 3240mm long on including 65x65x6mm angle struts including concrete grade 25 on bases	No	12	20.83	250.00
					-
R	Mild steel gate size2000mm wide X 2400mm high overall comprising steel hollow section frame and 16mm steel rails spaced at 150mm centre to centre including covering bottom half with ironmongery and painting all exposed steel surfaces.	No	1	833.33	833.33
	CONCRETE TO WEIR				
S	Supply all materials placing and finishing of C25 reinforced concrete for theflushing gate slab, including steel reinforcement, joints and all requirements	CM	1	250.00	250.00
	GATES				-

T	allow a provisional sum of US$ 5,000 (Five Thousand US Dollars) for supply and install intake gate facility including all necessary accessories as specified	SUM	1	5,000.00	5,000.00
U	allow a provisional sum of US$ 5,000 (Five Thousand US Dollars) for supply and install Flushing gate facility including all necessary accessories as specified	SUM	1	5,000.00	5,000.00
	ENVIRONMENTAL REALEASE PIPE				-
T	supply and install environmental release pipe including all necissarry fittings and valves as specified	LM	15	35.00	525.00
					-
Total for Element No. 2 Carried to summary					27,862.08
	Element 2. collection				
	Page No. 2/2/3				33,122.99
	Page No. 2/2/4				27,862.08
Total for Element No. 2 Carried to Bill 1 Summary					60,985.07
					2/2/4
BILL NO. DISILTING TANK					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	BILL NO. 2				
	CIVIL WORKS				
	ELEMENT NO.3				

	<u>**DISILTING TANK**</u>				
	<u>**Earth works and site clearance**</u>				
A	Clear site of all trees, bushes, shrubs and under growth including grubbing up roots and removing away from site	SM	80	1.25	100.00
B	Excavate average 200mm deep to remove top soil: remove from site	SM	80	1.67	133.33
C	Mass excavation to reduce levels not exceeding 2.0m commencing from stripped level; deposit in spoil heaps where directed on site	CM	160	6.25	1,000.00
D	Extra over excavation for excavating in rock.	CM	108	50.00	5,400.00
					-
E	Allow for maintaining and upholding sides of excavation : clear off all fallen material, rubbish.	ITEM	1	1,250.00	1,250.00
F	Allow for keeping the whole of the excavation free from general water.	ITEM	1	1,250.00	1,250.00
	CART AWAY AND FILLING				-
G	Load from spoil heaps and cart away excavated material from site.	CM	268	6.25	1,675.00
	CUTTING TREES				-
H	Cut down trees and grub up roots and chop-up and remove all arising from site : large trees girth not exceeding 600mm girth	NO	4	83.33	333.33
I	Ditto : girth 600 - 900mm girth.	NO	2	125.00	250.00
	TANK FOUNDATION				-

	AND MASONRY WALLS				
J	provide and place stone masonary works to foundations of the tank average thickness 400mm in cement sand mortar mix 1:4	CM	32	83.33	2,666.67
K	masonry stone work to bases in cement sand mortar mix 1:4	CM	28	83.33	2,353.33
L	Ditto to walls	CM	21	83.33	1,743.33
	PLASER TO DISILTING TANK				-
M	12mm thick cement sand plaser to masonry surfaces mix 1:4	SM	84	6.25	525.00
Total for Element No. 3 Carried to collection					18,680.00
					2/3/5
	FLUSHING PIPE				-
N	supply and install 200mm Dia. PVC flushing pipe including all necissarry fittings and valves	LM	8	35.00	280.00
	FENCING				-
O	2400mm high steel grill built around the weir structure including the abutments comprising of pointed tops in steel hollow sections of 50x 50mm welded and built into boundary wall pre painted before brought to site	LM	35	104.17	3,645.83
					-
P	Extra for corner posts with 2no. 65 x65mm x6mm straining posts 3240mm long on including 65x65x6mm angle struts including concrete grade 25 on	No	6	20.83	125.00

	bases				
					-
Q	Mild steel gate size2000mm wide X 2400mm high overall comprising steel hollow section frame and 16mm steel rails spaced at 150mm centre to centre including covering bottom half with ironmongery and painting all exposed steel surfaces.	No	1	833.33	833.33
	TRASH SCREEN				-
R	supply and install steel trash screen for the intake shaft complete including all embedded parts and all requirements as per engineer's drawingings	SM	2	400.00	876.00
Total for Element No. 3 Carried to summary					5,760.17
	Element 3. collection				
	Page No. 2/3/5				18,680.00
	Page No. 2/3/6				5,760.17
Total for Element No. 3 Carried to Bill 1 Summary					24,440.17
					2/3/6
BILL NO. FOREBAY					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**

	BILL NO. 2				
	CIVIL WORKS				
	ELEMENT NO.4				
	FOREBAY				
	Earth works and site clearance				
A	Clear site of all trees, bushes, shrubs and under growth including grubbing up roots and removing away from site	SM	52	1.25	65.38
					-
B	Excavate average 200mm deep to remove top soil: remove from site	SM	52	1.67	87.17
					-
C	Mass excavation to reduce levels not exceeding 2.0m commencing from stripped level; deposit in spoil heaps where directed on site	CM	105	6.25	653.75
					-
D	Extra over excavation for excavating in rock.	CM	71	50.00	3,530.25
					-
E	Allow for maintaining and upholding sides of excavation : clear off all fallen material, rubbish.	ITEM	1	1,250.00	1,250.00
F	Allow for keeping the whole of the excavation free from general water.	ITEM	1	1,250.00	1,250.00
					-
	CART AWAY AND FILLING				-
G	Load from spoil heaps and cart away excavated material from site.	CM	175	6.25	1,095.03
					-
	CUTTING TREES				-

H	Cut down trees and grub up roots and chop-up and remove all arising from site : large trees girth not exceeding 600mm girth	NO	3	83.33	250.00
I	Ditto : girth 600 - 900mm girth.	NO	3	125.00	375.00
	FOREBAY FOUNDATION AND MASONRY WALLS				-
J	provide and place stone masonary works to foundations of the forebay average thickness 400mm in cement sand mortar mix 1:4	CM	21	83.33	1,743.33
	masonry stone work to bases in cement sand mortar mix 1:4	CM	28	83.33	2,346.67
K	Ditto to walls	CM	23	83.33	1,916.67
	PLASER TO FOREBAY WALLS AND BASE				-
L	12mm thick cement sand plaser to masonry surfaces mix 1:4	SM	94	6.25	587.50
Total for Element No. 4 Carried to collection					15,150.74
					2/4/7
	FENCING				-
M	2400mm high steel grill built around the weir structure including the abutments comprising of pointed tops in steel hollow sections of 50x 50mm welded and built into boundary wall pre painted before brought to site	LM	60	104.17	6,250.00
					-

N	Extra for corner posts with 2no. 65 x65mm x6mm straining posts 3240mm long on including 65x65x6mm angle struts including concrete grade 25 on bases	No	7	20.83	145.83
					-
O	Mild steel gate size2000mm wide X 2400mm high overall comprising steel hollow section frame and 16mm steel rails spaced at 150mm centre to centre including covering bottom half with ironmongery and painting all exposed steel surfaces.	No	12	833.33	10,000.00
					-
	CONCRETE TO FOREBAY				
P	Supply all materials placing and finishing of C25 reinforced concrete for the tank, including steel reinforcement, joints and all requirements	CM	9	250.00	2,187.50
	FLUSHING PIPE				-
Q	supply and install 200mm Dia. PVC flushing pipe including all necissarry fittings and valves	LM	5	35.00	175.00
Total for Element No. 4 Carried to summary					18,758.33
	Element 4. collection				
	Page No. 2/4/7				15,150.74
	Page No. 2/4/8				18,758.33

	Total for Element No. 4 Carried to Bill 1 Summary				33,909.07
					2/4/8
	BILL NO. 3. The Penstock				
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	BILL NO. 2				
	CIVIL WORKS				
	ELEMENT NO.5				
	PENSTOCK				
	Earth works and site clearance				
A	Clear site of all trees, bushes, shrubs and under growth including grubbing up roots and removing away from site	SM	808	1.25	1,010.00
					-
B	Excavate average 200mm deep to remove top soil: remove from site	SM	808	1.67	1,346.67
					-
C	Mass excavation to reduce levels not exceeding 2.0m commencing from stripped level; deposit in spoil heaps where directed on site	CM	1,616	6.25	10,100.00
					-
D	Extra over excavation for excavating in rock.	CM	1,091	50.00	54,540.00
					-
E	Allow for maintaining and upholding sides of excavation : clear off all fallen material, rubbish.	ITEM	1	1,250.00	1,250.00
F	Allow for keeping the whole of the excavation free from general water.	ITEM	1	1,250.00	1,250.00
	CART AWAY AND FILLING				-

G	Load from spoil heaps and cart away excavated material from site.	CM	2,707	6.25	16,917.50
					-
	CUTTING TREES				-
H	Cut down trees and grub up roots and chop-up and remove all arising from site : large trees girth not exceeding 600mm girth	NO	18	83.33	1,500.00
I	Ditto : girth 600 - 900mm girth.	NO	12	125.00	1,500.00
	PENSTOCK FOUNDATION AND PIPE WORK				-
K	supply and install DN610 3.65mm thick spiral welded steel pipe epoxy lined complete in 12m lengths	No	36	2,400.00	86,400.00
L	Jointing	No	32	300.00	9,600.00
M	Flexible joint	No	4	1,200.00	4,800.00
N	Anchor blocks	No	4	1,800.00	7,200.00
O	Support Blocks	No	140	300.00	42,000.00
Total for Element No. 5 Carried to collection					239,414.17
					2/5/9
	PAINTINTING TO PENSTOCK PIPE				-
L	prepare and apply three coats of specified gloss pait to penstock pipe and stands as per engineers recommendations	SM	2,056	3.75	7,711.50
	FENCING			-	-

M	barbed wire fencing 2000mm high comprising 65mm x 65mm x 6mm angle line poles, straining poles and corner poles all cast in concrete bases, barbed wire guage 10 in three lines at 600mm c/c including foundation excavation and back filling. Fenced on either sides of the channel	LM	808	7.50	6,060.00
Total for Element No. 5 Carried to summary					13,771.50
	Element 5. collection				
	Page No. 2/5/9				239,414.17
	Page No. 2/5/10				13,771.50
Total for Element No. 5 Carried to Bill 1 Summary					253,185.67
					2/5/10
BILL NO. 3. The Spillway Canal from forebay					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	BILL NO. 2				
	CIVIL WORKS				
	ELEMENT NO.6				
	SPILLWAY				
	Earth works and site clearance				
A	Clear site of all trees, bushes, shrubs and under growth including grubbing up roots and removing	SM	25	1.25	31.25

	away from site				
					-
B	Excavate average 200mm deep to remove top soil: remove from site	SM	25	1.67	41.67
					-
C	Mass excavation to reduce levels not exceeding 2.0m commencing from stripped level; deposit in spoil heaps where directed on site	CM	50	6.25	312.50
					-
D	Extra over excavation for excavating in rock.	CM	34	50.00	1,687.50
					-
E	Allow for maintaining and upholding sides of excavation : clear off all fallen material, rubbish.	ITEM	1	1,250.00	1,250.00
F	Allow for keeping the whole of the excavation free from general water.	ITEM	1	1,250.00	1,250.00
					-
	CART AWAY AND FILLING				-
G	Load from spoil heaps and cart away excavated material from site.	CM	84	6.25	523.44
					-
	CUTTING TREES				-
H	Cut down trees and grub up roots and chop-up and remove all arising from site : large trees girth not exceeding 600mm girth	NO	15	83.33	1,250.00
I	Ditto : girth 600 - 900mm girth.	NO	5	125.00	625.00
	FOUNDATION				-

J	provide and place stone masonary works to foundations of the weir and abutments average thickness 400mm in cement sand mortar mix 1:4	CM	10	83.33	833.33
	FOUNDATION AND MASONRY WALLS				-
K	provide and place stone masonary works to foundations average thickness 400mm in cement sand mortar mix 1:4	CM	10	83.33	833.33
L	masonry stone work to bases and walls in cement sand mortar mix 1:4	CM	9	83.33	750.00
Total for Element No. 6 Carried to collection					9,388.02
					2/6/11
					-
	PLASER TO WEIR & ABUTMENT				-
M	12mm thick cement sand plaster to masonry surfaces mix 1:4	SM	25	6.25	156.25
					-
Total for Element No. 6 Carried to summary					156.25
	Element 6. collection				
	Page No. 2/6/11				9,388.02
	Page No. 2/6/12				156.25
Total for Element No. 6 Carried to Bill 1Summary					9,544.27
BILL NO. 4. Power House					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**

	BILL NO. 2				
	CIVIL WORKS				
	ELEMENT NO.7				
	POWER HOUSE				
A	allow a provisional sum of USD 120,000 (One hundred Twenty Thousand United States Dollars) for construction of the Power House as designed by the Engineer	No	1	120,000.00	120,000.00
Total forElement No. 7 Carried to Bill 1Summary					120,000.00

e) SUMMARY OF CIVIL WORKS

ELEMENT	***DESCRIPTION***	***PAGE NO.***	***AMOUNT (USD)***
	BILL NO. 2		
	CIVIL WORKS		
1	DIVERSION AND ACCESS ROADS	2/1/2	1,119,140
2	WEIR AND ABUTMENTS	2/2/4	60,985
3	DISILTING TANK	2/3/6	24,440
4	FOREBAY	2/4/8	33,909
5	PENSTOCK	2/5/10	253,186
6	SPILLWAY	2/6/12	9,544
7	POWER HOUSE	2/7/13	120,000
	TOTAL BILL NO.3 CARRIED TO MAIN SUMMARY		**1,621,204**

f)

g) **BILL NO. 3. ELECTROMECHANICAL BILL OF QUANTITIES**

BILL NO. 3. ELECTROMECHANICAL					
ITEM	**DESCRIPTION**	**UNIT**	**QTY**	**RATE (USD)**	**AMOUNT (USD)**
	BILL NO. 3				
	ELECTROMECHANICAL				
	Turbine				
A	Micro-hydro turbine of the Francis type, rated at 160KW, Speed 1500 RPM	NO	1.00	105,000.00	105,000.00
B	DN600 PN 16 inlet Butterfly valve	NO	1.00	650.00	650.00
C	Turbine Governor including dummy load.	NO	1.00	12,350.00	12,350.00
D	Flywheel	NO	1.00	4,850.00	4,850.00
	Generator				-
E	Horizontal shaft, three phase AC Synchronous Generator, 4 pole, speed 1500 RPM, Frequency 50Hz, capacity 150KVA, rated voltage 400V, rated current 222A, Efficiency 91% (minimum), power factor 0.8, Air cooled, with self exciter and brushless.	NO	1.00	55,000.00	55,000.00
F	Lightning Protection	NO	1.00	3,000.00	3,000.00
G	Earthing	NO	1.00	800.00	800.00
	Control Panel				-
H	Generator control panel free standing.	NO	1.00	25,000.00	25,000.00
I	Governor control panel	NO	1.00	15,000.00	15,000.00
	Transformer				-
J	Step up Transformer 200KVA, 400V/33KV, 50Hz, Dyn11	NO	1.00	20,000.00	20,000.00
K	Step down Transformer 200KVA, 33KV/11KV/415V, 50Hz, Dyn11	NO	1.00	20,000.00	20,000.00
L	Protection Equipment	ITEM	1.00	22,500.00	22,500.00
	Transmission Line				-

M	33KV Overhead transmission line with 3nox50mm2 AAAC flat formation conductors complete with GS cross-arms, pin type insulators and all accessories.	KM	18.00	18,000.00	324,000.00
N	10KVA standby generator	NO	1.00	3,500.00	3,500.00
					-
O	Wiring and electrical accessories for Power house	NO	1.00	1,900.00	1,900.00
					-
P	Spares and Tools	ITEM	1.00	25,000.00	25,000.00
Total BILL NO.3 Carried to MAIN Summary					638,550.00

h) **MAIN SUMMARY**

BILL NO.	DESCRIPTION	PAGE NO.	AMOUNT (USD.)
	CIVIL WORKS		
1	PRELIMINARIES	1/S	101,700
2	CIVIL WORKS	2/S	1,621,204
3	ELECTROMECHANICAL	3/S	638,550
			2,361,454
	ADD		
	5% supervisory costs		**118,073**
	SUB-TOTAL (USD.)		**2,479,527**
	ADD: 5% CONTINGENCIES		123,976
	SUB-TOTAL (USD.)		**2,603,503**
	ADD: 18% VAT		468,631
	TOTAL AMOUNT (USD.)		**3,072,134**

APPENDIX 2: SITE GPS CO-ORDINATES

SITE COORDINATES		
Grid	UTM	
Datum	ARC 1960	
Name	Name on the map	Position
Channel point	6	36 N 352845 366557
Channel point	7	36 N 352807 366557
Channel point	8	36 N 352769 366557
Channel point	9	36 N 352734 366557
Channel point	10	36 N 352691 366557
Channel point	11	36 N 352649 366557
Channel point	12	36 N 352590 366557
Channel point	13	36 N 352541 366557
Channel point	14	36 N 352491 366557
Channel point	15	36 N 352437 366557
Channel point	16	36 N 352381 366557
Channel point	17	36 N 352331 366557
Channel point	18	36 N 352291 366557
Channel point	19	36 N 352260 366557
Channel point	20	36 N 352217 366557
Channel point	21	36 N 352187 366557
Channel point	22	36 N 352157 366557

Channel point	23	36 N 352114 366557
Channel point	24	36 N 352086 366569
Channel point	25	36 N 352051 366584
Channel point	26	36 N 352025 366595
Channel point	27	36 N 351985 366613
Channel point	28	36 N 351957 366625
Channel point	29	36 N 351935 366635
Channel point	30	36 N 351904 366648
Channel point	31	36 N 351875 366661
Channel point	32	36 N 351847 366673
Channel point	33	36 N 351817 366686

Channel point	34	36 N 351789 366698
Channel point	35	36 N 351753 366714
Channel point	36	36 N 351724 366726
Channel point	37	36 N 351681 366745
Channel point	38	36 N 351655 366756
Channel point	39	36 N 351618 366772
Channel point	40	36 N 351584 366787
Channel point	41	36 N 351548 366803
Channel point	42	36 N 351508 366820
Channel point	43	36 N 351474 366835
Channel point	44	36 N 351435 366852

Channel point	45	36 N 351401 366866
Channel point	46	36 N 351363 366883
Channel point	47	36 N 351331 366897
Channel point	48	36 N 351302 366910
Channel point	49	36 N 351272 366922
Channel point	50	36 N 351228 366942
Channel point	51	36 N 351197 366955
Channel point	52	36 N 351161 366971
Waypoint	Fall	36 N 350797 367281
Waypoint	Fill Material	36 N 351919 366710
Waypoint	Flood Zone	36 N 350845 367252

Waypoint	FOREBAY	36 N 351162 366970
Waypoint	Galley 1	36 N 352734 366604
Waypoint	Galley 2	36 N 352707 366613
Waypoint	Galley 3	36 N 352137 366542
Waypoint	Galley 5	36 N 351518 366743
Waypoint	Island Weir Pt	36 N 352879 366591
Waypoint	Powerhouse A	36 N 350928 367218
Waypoint	Powerhouse B	36 N 350866 367248
Intake Channel point	R1	36 N 352858 366558
Intake Channel point	R2	36 N 352845 366555
Waypoint	R7	36 N 351779 366735

Waypoint	Real Intake	36 N 352876 366578
Waypoint	Rocky Area	36 N 351093 367058
Waypoint	Rocky Diversion	36 N 352883 366615
Waypoint	Stone Material	36 N 351962 366650
Waypoint	Stream2	36 N 350842 367199
Waypoint	Stream3	36 N 350759 367244
Penstock point	P2	36 N 350906 367228
Penstock point	P3	36 N 350945 367200
Penstock point	P4	36 N 350967 367175
Penstock point	P5	36 N 350990 367153
Penstock point	P6	36 N 351014 367129

Penstock point	P7	36 N 351036 367104
Penstock point	P8	36 N 351056 367083
Penstock point	P9	36 N 351076 367060
Penstock point	P10	36 N 351099 367040
Penstock point	P11	36 N 351115 367021
Penstock point	P12	36 N 351140 366997

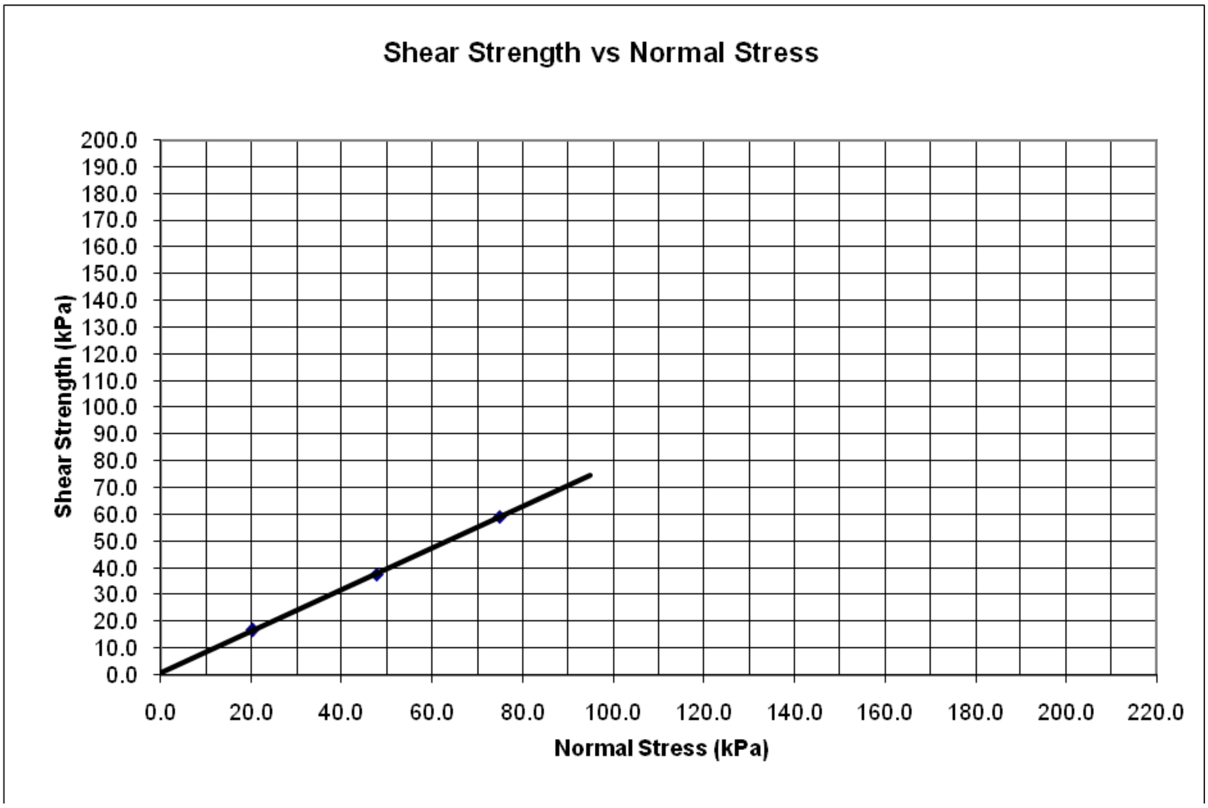
APPENDIX 3: GEOLOGY

a)

SHEAR BOX TEST OF SOILS

Project : Asiya River Mini - Electrical Power in Adjuman District	**SAMPLE No: TP 1**
Client :M/S Africa Power Initiative	
Date : April 2011	**DEPTH (m): 1.0**

Bulk Density γ_b Mg/m^3	**Normal Stress** δ_n kPa	**Shear Strength** τ_s kPa	**Cohesion** C kPa	**Angle of Internal Friction** ϕ (Degree)
1.59	20.3	14.3	2	18
	47.6	29.3		
	74.8	46.4		

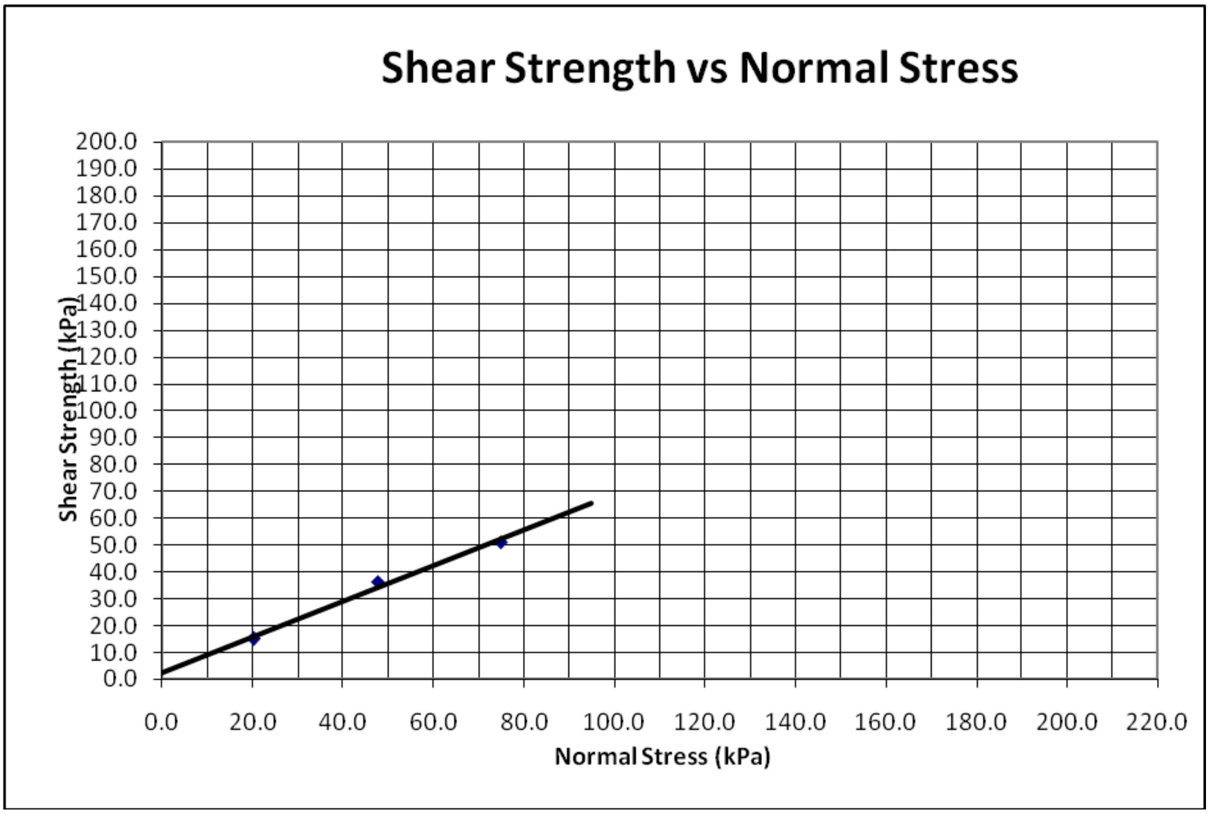


b)

SHEAR BOX TEST OF SOILS

Project : Esiya River Mini - Hydro Power in Adjumani District	**SAMPLE No: TP 2**
Client :M/S Africa Power Initiative	
Date : April 2011	**DEPTH (m): 1.4**

Bulk Density γ_b Mg/m^3	Normal Stress δ_n kPa	Shear Strength τ_s kPa	Cohesion C kPa	Angle of Internal Friction ϕ (Degree)
1.69	20.3	15.0	3	19
	47.6	36.3		
	74.8	51.2		

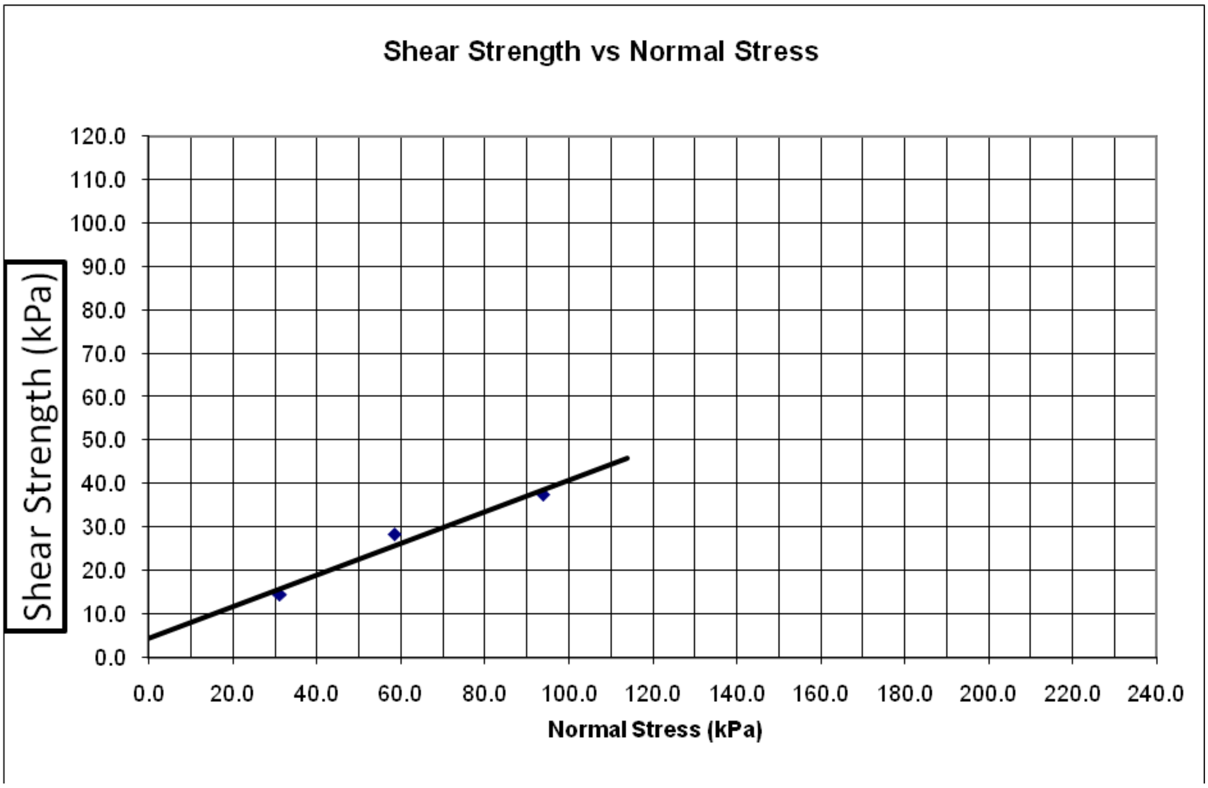


c)

SHEAR BOX TEST OF SOILS

Project : Asiya River Mini - Hydro Power in Adjuman District	**SAMPLE No: TP 1**
Client :M/S Africa Power Initiative	
Date : April 2011	**DEPTH (m): 2.0**

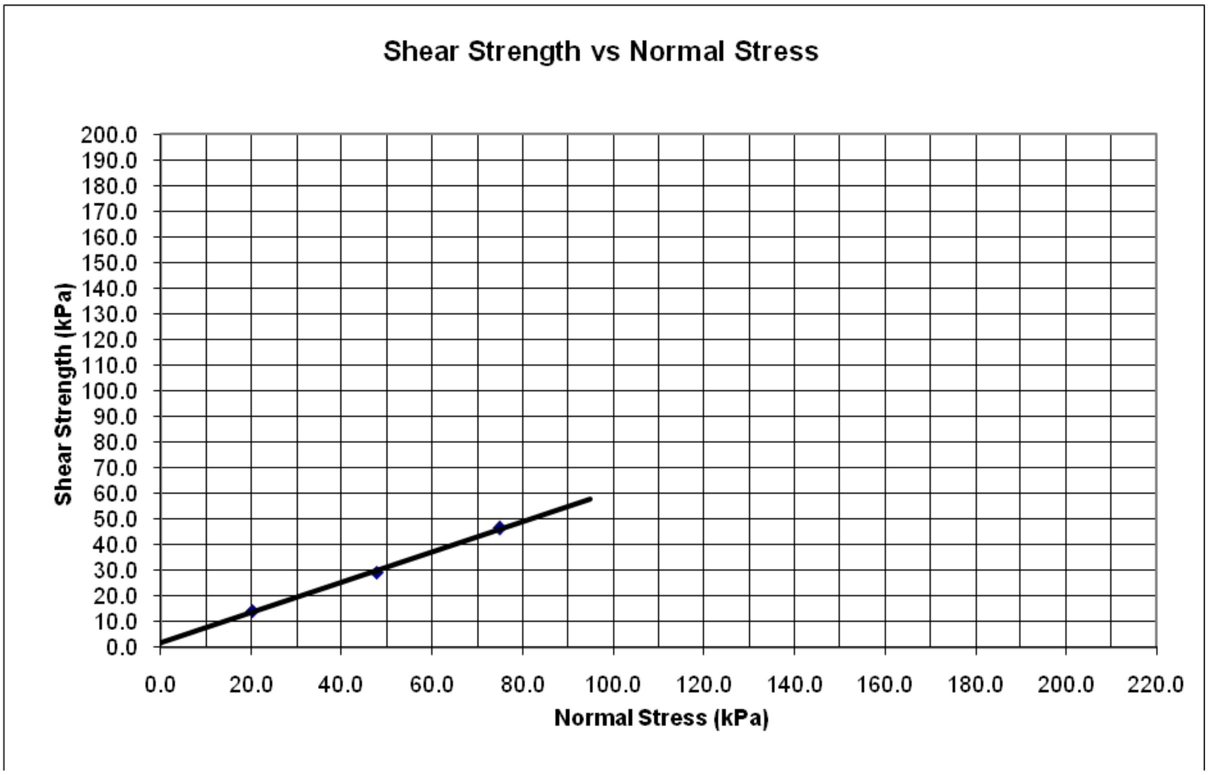
Bulk Density γ_b Mg/m^3	**Normal Stress** δ_n kPa	**Shear Strength** τ_s kPa	**Cohesion** C kPa	**Angle of Internal Friction** ϕ (Degree)
1.60	31.2	14.5	5	18
	58.5	28.5		
	93.9	37.6		



d)

SHEAR BOX TEST OF SOILS	
Project : Esia River Mini - Electrical Power in Adjuman District	**SAMPLE No: TP 1**
Client :M/S Africa Power Initiative	
Date : April 2011	**DEPTH (m): 1.0**

Bulk Density γ_b Mg/m^3	**Normal Stress** δ_n kPa	**Shear Strength** τ_s kPa	**Cohesion** C kPa	**Angle of Internal Friction** ϕ (Degree)
1.59	20.3	14.3	2	18
	47.6	29.3		
	74.8	46.4		



APPENDIX 4: EVALUATION OF BEARING CAPACITY BASED ON TERZAGHI'S MODEL (GENERAL SHEAR FAILURE)

SAMPLE	DEPTH, D	WIDTH, B	BULK DENSITY, γ	COHESION	ANGLE OF FRICTION	BEARING CAPACITY FACTORS			ULTIMATE BEARING CAPACITY	SAFETY FACTOR	EBEARING CAPACITY
No.	(m)	(m)	$(Mg/m^3)x10$	C (KPa)	ϕ (Degrees)	Nc	Nq	N_γ	q_{ult} (KPa)	(F)	q_{all} (KPa)
TP 1	1.0	1.0	15.9	2	18	13.1	5.3	2.1	131	3	44
	2.0	1.0	16.0	5	18	13.1	5.3	2.1	262	3	87
TP 2	1.4	1.0	16.9	3	19	13.9	5.8	2.5	201	3	67
TP 3	0.6	1.0	18.2	1	23	18.1	8.7	4.9	152	3	51

$q_{ult} = CN_C sc + q_o N_q + ½\gamma BN_\gamma s_\gamma$

Where: $q_o = \gamma D$

$q_{all} = q_{ult}/F$

For :	strip	round	square
sc	1	1.3	1.3
s_γ	1	0.6	0.8