PLANNING OF MICRO HYDRO POWER GENERATION

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Venue: REA Board Room

JICA/REA Project for establishment of Rural Electrification model using Renewable energy

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CONTENTS

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Necessity of Off-Grid Mini Hydro

1. Use of underdeveloped energy

- Potential hydro energy resources in the world approximate 14trillion Kwh, 80% of these which are equivalent to energy demand of the whole world are not yet developed.
- About 2/3 of these undeveloped energy resources are in developing world.
- Development of these small hydros would ease the global energy demand.

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Necessity of Off-Grid Mini Hydro Cont.

2. Geo-ecological problem

- Global warming resulting from constant increase in greenhouse gases (CO_2, SO_x, NO_x)

3. Economic development of developing countries

4. Local energy resource

5. Stabilization of electricity rates

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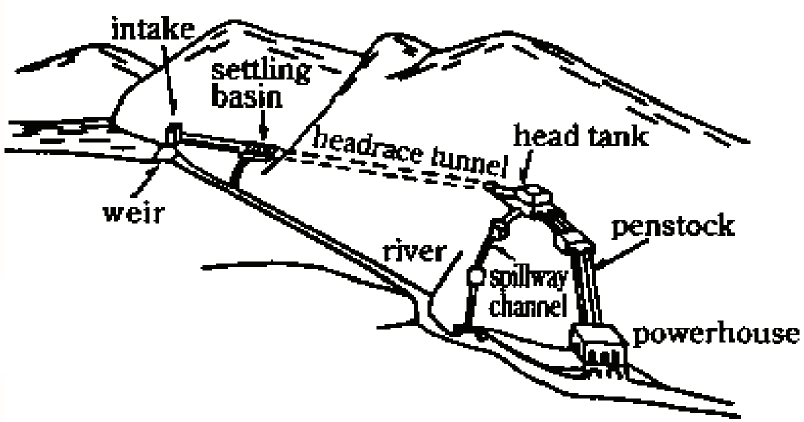
Types of Hydros

- Hydro power generation
  - Conventional type
    - Run-of-river type
    - Reservoir type
    - Pondage type
  - Pumped storage type
    - Pure pumped storage type
    - Pumped and natural flow storage type

Types of Hydros Cont.

- Run of the river
- Reservoir type or pondage
- Waterway type
- Dam type
- Pumped storage type

Run of the river



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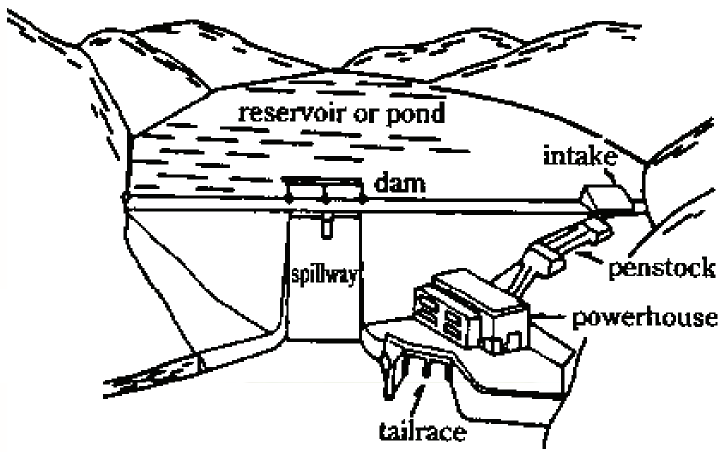
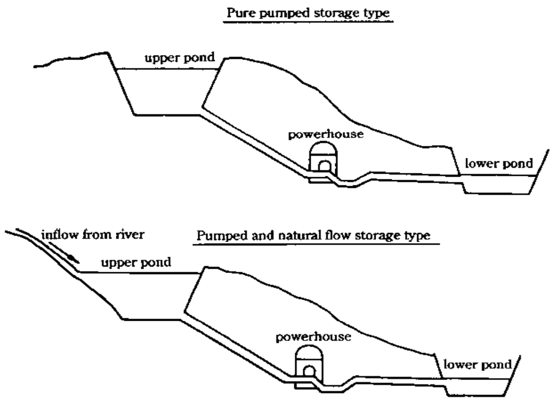
Reservoir type



Attachment K-3-1

Types of Hydros Cont.

- Pumped Storage.



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Identification of the Project

- Project identification determines effective and excellent projects that meets the needs of a particular region.

- Projects preparation and feasibility study examines various technical aspects of the project i.e. Technical, economical, environmental and other factors

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Identification of the Project

- Confirmation of Off-grid Area
- Map Study
- Site Reconnaissance
- Estimation of Power Demand
- Estimation of Hydropower Potential

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Identification of the Project

Confirmation of Off-grid area

The following information needs to be collected in identifying a promising scheme

(a) Electrification condition

- Existing distribution network from REA/KPLC
- Grid extension plan for REA/KPLC
- Low cost development plan from MOE

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Confirmation of Off-Grid Area

(b) Local community population

- Kenya bureau of statistics

(c) Environmental regulations

(d) Physical information

- Topographical maps (1:50 000 or 1:10 000) from land survey
- Hydrological data (Isohyets, rainfall discharge data) from Meteorological department

Preliminary planning of development plan

Site Reconnaissance

- Is a preliminary study
- It is initial stage or individual stage study where topographical maps of 1:50 000 to 1:100 000 are used compared to full feasibility study where topo maps of 1:1 000 to 1: 5 000 are used.
- Feasibility studies are done based on the results of reconnaissance studies.

Preliminary planning of development plan

Site Reconnaissance Cont.

- Preliminary studies are done to upgrade the quality of work gradually while maintaining quality and cost efficiency.

Steps in doing reconnaissance

- Select dam/ weir site and powerhouse site or tailrace site in consideration of flow or river topography
- Determine maximum discharge on the basis of river flow
- Estimate the construction cost of the dam, waterway, powerhouse and other electrical facilities

Micro/mini/ small hydro

Definitions of micro/mini/small hydro are made as follows;

- Micro hydro: Less than 100 kW
- Mini hydro: 100 kW to 1,000 kW (1.0 MW)
- Small hydro: 1,000 kW to 10,000 kW (1.0 MW to 10.0 MW)

Estimation of Power Demand

- Power demand in local community is unknown before electrification
- It is assumed as minimum electric equipments for identification of the promising development schemes.
- Assumptions of percentage share and unit demand of each consumer are made as shown below.

Estimation of power demand

User	Share per household or Number per village	Assumed Demand			
		Lighting	TV	Fan/Heater	Total
Domestic					
Type A	20% of total households	-	-	-	0
Type B	60% of total households	20 W x 2	-	-	40
Type C	20% of total households	20 W x 3	200 W x 1	-	260
Type D	1 per 100 households	40 W x 5	200 W x 1	200 W x 1	600
Public					
Primary School	1 per 100 households	40 W x 15	200 W x 1	200 W x 1	1,000
Community Hall	1 per 100 households	40 W x 15	-	200 W x 2	1,000
Dispensary	1 per village > 100 households	40 W x 5	-	200 W x 2	600
Streetlight	1 per 4 households	40 W x 1	-	-	40
Business					
Battery Carging Station	1 per 50 households	-	-	-	1,200
Posho Mill	1 per 200 households	-	-	-	5,000

Estimation of power demand Cont.

Assumed Number of Consumers (Unit: numbers)

User	Unit Demand	Village Size (Households)								
		700	600	500	400	300	200	100	50	20
Domestic										
Type A	0	140	120	100	80	60	40	20	10	4
Type B	40	420	360	300	240	180	120	60	30	12
Type C	260	140	120	100	80	60	40	20	10	4
Type D	600	7	6	5	4	3	2	1	0	0
Public										
Primary School	1,000	7	6	5	4	3	2	1	0	0
Community Hall	1,000	7	6	5	4	3	2	1	0	0
Dispensary	600	1	1	1	1	1	1	1	0	0
Streetlight	40	175	150	125	100	75	50	25	12	5
Business										
Battery Carging Station	500	14	12	10	8	6	4	2	1	0
Posho Mill	5,000	3	3	2	2	1	1	0	0	0
Total		774	664	553	443	332	222	111	53	21

Assumed power demand

Assumed Power Demand (Unit: kW)

User	Unit Demand	700	600	500	400	300	200	100	50	20
		Village Size (Households)								
Domestic										
Type A	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Type B	40	16.80	14.40	12.00	9.60	7.20	4.80	2.40	1.20	0.48
Type C	260	36.40	31.20	26.00	20.80	15.60	10.40	5.20	2.60	1.04
Type D	600	4.20	3.60	3.00	2.40	1.80	1.20	0.60	0.00	0.00
Public										
Primary School	1,000	7.00	6.00	5.00	4.00	3.00	2.00	1.00	0.00	0.00
Community Hall	1,000	7.00	6.00	5.00	4.00	3.00	2.00	1.00	0.00	0.00
Dispensary	600	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.00	0.00
Streetlight	40	7.00	6.00	5.00	4.00	3.00	2.00	1.00	0.48	0.20
Business										
Battery Carging Station	500	7.00	6.00	5.00	4.00	3.00	2.00	1.00	0.50	0.00
Posho Mill	5,000	15.00	15.00	10.00	10.00	5.00	5.00	0.00	0.00	0.00
Total		101.00	88.80	71.60	59.40	42.20	30.00	12.80	4.78	1.72

Required Generator Output

The required generator output (P_r) to supply for the power demand derived above can be calculated by the following formula:

$$P_r = P_d + w_1 + w_2 = P_d\ (1+0.2+1) = 1.3\ P_d$$

Where,
- P_r: required generator output (kW)
- P_d: power demand (kW)
- w_1: losses for transmission and distribution, assumed to be 20% of P_d
- w_2: reserved power, assumed to be 10% of P_d

Required Generator Capacity by Community Size

For a community size of interest, following formula can be used

$$P_d = 0.2N$$

Where,
- P_g: selected generator capacity (kW)
- N: community size (number of household)

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Required Head and Discharge

When a discharge of Q drops through a head of H, the work done per unit time is called the theoretical hydropower; $P_0 = g \times H \times Q = 9.8HQ$

Where,
- P_0: theoretical hydropower (kW)
- g: gravity acceleration (9.8)
- H: static head (m)
- Q: discharge (m^3/s)

Considering mechanical and electrical energy losses, relationship between theoretical hydropower and required generator capacity can be expressed as;

$$P_i = P_0 \times \eta_G \times \eta_T / (1+m) = 0.57P_0$$

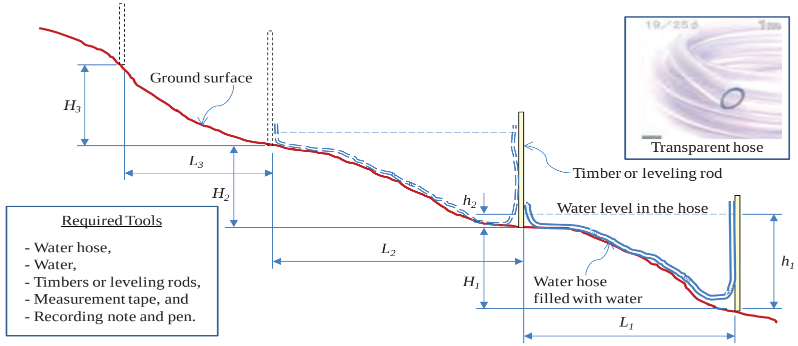
Where,
- P_i: required generator capacity (kW)
- P_0: theoretical hydropower (kW)
- η_G: generator efficiency, assumed to be 90%
- η_T: turbine efficiency, assumed to be 70%
- m: rate of other losses, assumed to be 10%

Attachment K-3-1

Measurement of Head and discharge

Measurement of head

1. Survey Method by using Water Hose

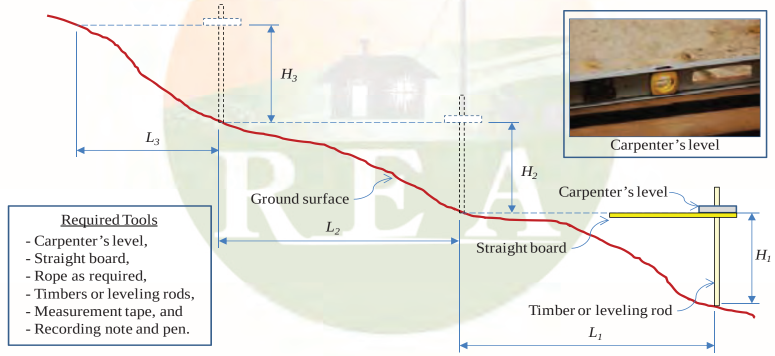


Required Tools

- Water hose,
- Water,
- Timbers or leveling rods,
- Measurement tape, and
- Recording note and pen.

Measurement of head and discharge Cont.

2. Survey Method by using Carpenter's Level

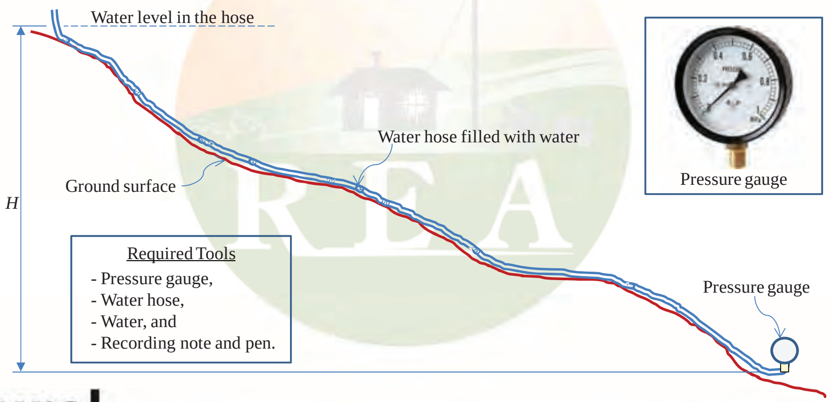


Required Tools

- Carpenter's level,
- Straight board,
- Rope as required,
- Timbers or leveling rods,
- Measurement tape, and
- Recording note and pen.

Measurement of head and Discharge Cont.

3. Survey Method by using Pressure Meter



Required Tools

- Pressure gauge,
- Water hose,
- Water, and
- Recording note and pen.

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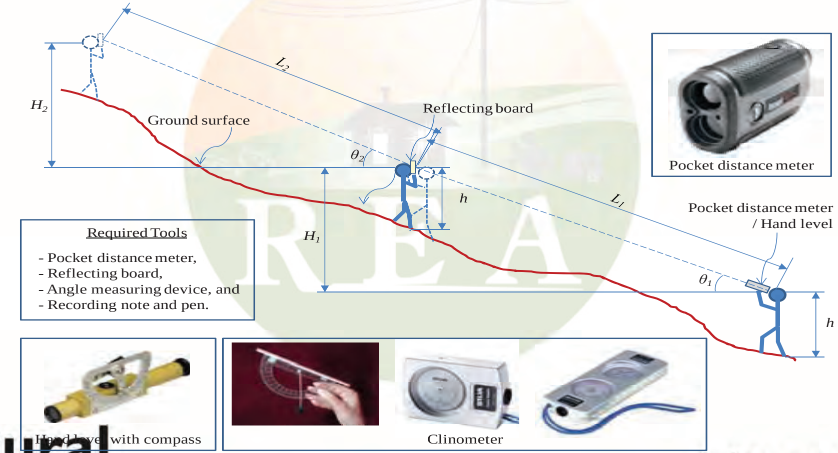
Measurement of head and discharge Cont.

4. Survey Method by using Pocket Distance Meter



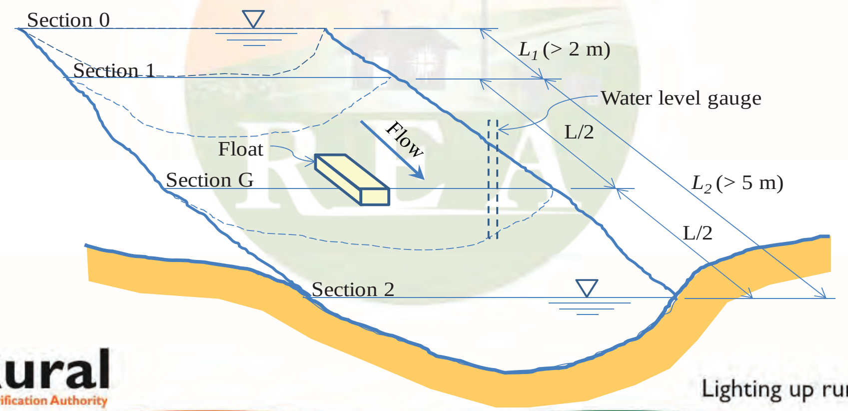
Required Tools

- Pocket distance meter,
- Reflecting board,
- Angle measuring device, and
- Recording note and pen.

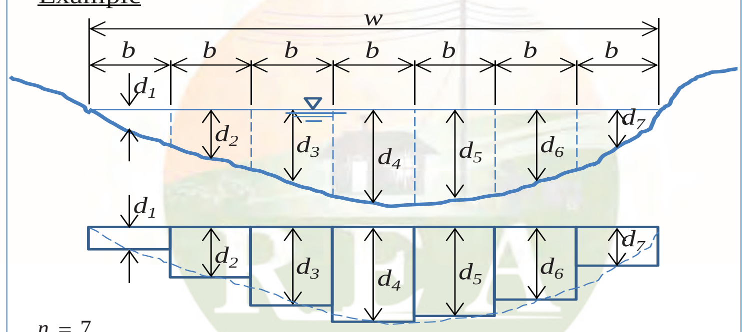
Attachment K-3-1

Measurement of Discharge

1. Measurement by using Float



Measuring x-sectional area



$n = 7$

$\therefore\ b = W/n = W/7$

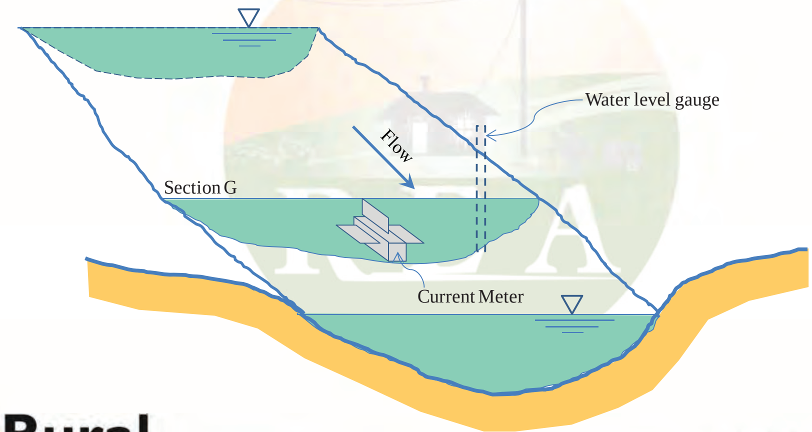
$$A = d_1 \times b + d_2 \times b + d_3 \times b + d_4 \times b + d_5 \times b + d_6 \times b + d_7 \times b$$

$$= b \times (d_1 + d_2 + d_3 + d_4 + d_5 + d_6 + d_7)$$

$$= W/7 \times (d_1 + d_2 + d_3 + d_4 + d_5 + d_6 + d_7)$$

Measurement of discharge

2. Measurement by using Current Meter



END

DESIGN OF MICRO HYDROS

MHP AND BIOGAS SEMINOR

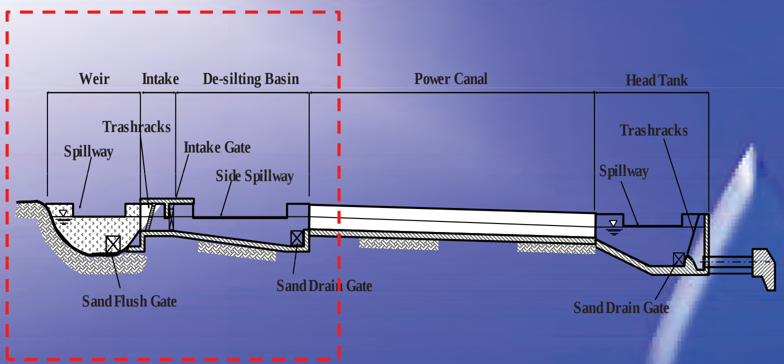
By :JUDITH KIMEU: REA CP
DATE: 25th October
VENUE: REA OFFICES

JICA/REA Project for establishment of rural electrification model using renewable energy.

Design of civil structures

1. Civil work structures in a Mini Hydro Include
   - Head works
   - Power canal
   - Head tank
   - Penstock

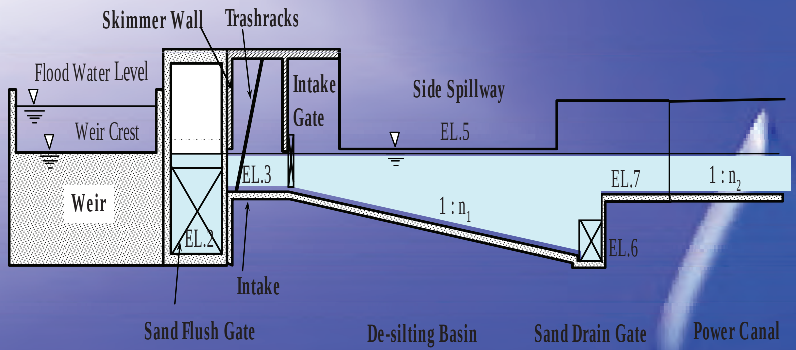
Head works



Head Works :

- The Functions of Head works are
  - ✓Diversion of the required flow from the river into the water conductor system.
  - ✓Control of sediment
  - ✓Flood handling
- Structures to be designed include:
  - Weir
  - Intake Gate
  - Spill way
  - Sand Flush Gate
  - Destilation Basin

Head works Structures



Weir

- Discharge from a weir spillway:

$$Q_{spill} = 1.84 \times B \times H^{1.5}$$

- Where, Q_{spill}:discharge from spillway (m^3/s)
- B: width of spillway (m)
- H: Overflow depth (water level minus crest level of spillway) (m)

Discharge from a sand flush gate:

For orifice flow:

$$Q = 0.6 \times A \times \sqrt{2 \times g \times H}$$

For pipe flow:

$$Q = A \times \sqrt{\frac{2 \times g \times H}{1 + f_e + f}}$$

Where, Q: discharge through the gate (m^3/s)

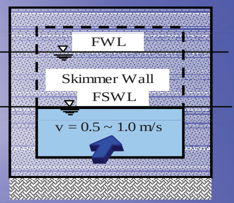
- A:section area of the gate (m^2)
- f_e:loss coefficient for entrance (-) f_e = 0.1 to 0.5
- f: loss coefficient for friction (-) $f = 124.5n^2 L/D^{(4/3)}$

Major Types of Weir and Intake

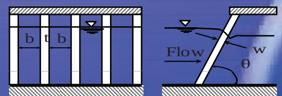
Concrete Gravity	Applicable on rock foundations, Most commonly applied, Durable and impervious, and Relatively high cost.
Floating Concrete Weir	Applicable on gravel foundations, Need an enough seepage path, Durable, and Relatively high cost.
Gabion Covered with Concrete	Applicable on gravel foundation, Surface protection by concrete, and Relatively low cost.
Gabion	Applicable on gravel foundation, Flexible, and Low cost and easy maintenance
Stone Masonry	Applicable on gravel foundation, and Low cost and easy maintenance.

Facilities provided in Intake

- Skimmer Wall

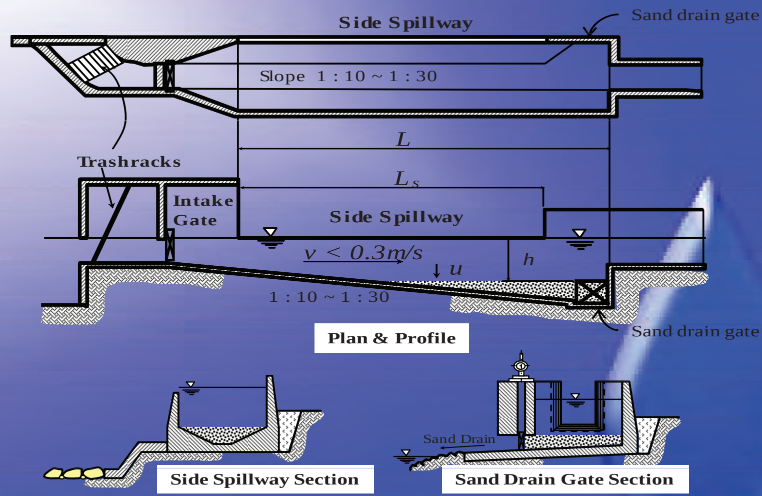


- Intake Gate
- Trash rack



Thickness	t = 5 ~ 9 mm
Width	w = 50 ~ 120 mm
Interval	b = 100 ~ 150 mm
Inclination	θ = 60 ~ 70°

De-silting Basin

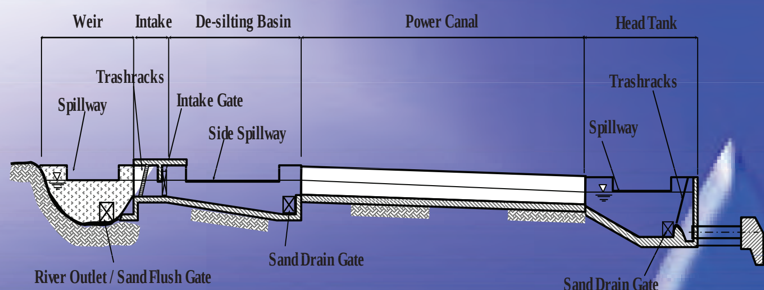


Length of De-silting Basin

- The de-silting basin is designed to settle sands bigger than 0.5 to 1.0 mm diameter of which the settling velocity corresponds to 0.1 m/s. Average flow velocity in a de-silting basin is generally 0.3 m/s, and the channel slope is 1/10 to 1/30.
- The length of de-silting basin is given by the following empirical formula; $L = \alpha \times \frac{v}{u} \times h_s$

- Where, L: length of de-silting basin (m)
- a : coefficient (-) = 2 to 3
- v: average velocity in de-silting basin (m/s) 0.3 m/s
- u: settling velocity for target sand particle (m/s) = 0.1 m/s
- h_s: depth of de-silting basin (m)

Power Canal



To lead the design discharge to the head tank

Route Selection

- Stability against slope above and/or below the canal, and
- Specific conditions such as streams, roads, and the existing structures to be crossed.
- Rise and fall of water in the canal

Major Types of Canal

Earth Canal	•Easy for construction and maintenance, •Low cost, •Not applicable to pervious and erosive foundation, •Velocity < 0.3 m/s, •Roughness coefficient n = 0.014 on an average, •seepage loss = 1.0×10^{-6} $m^3/s/m^2$ (clay) – 8.0 (sand) x 10^{-6} $m^3/s/m^2$
Stone Masonry Canal	Easy for construction and maintenance, Velocity <1.5 m/s (dry stone masonry) and velocity <2.0 m/s (wet stone masonry), Roughness coefficient n = 0.032 (dry stone masonry) and 0.025 (wet stone masonry)
Concrete Canal	Durable, Relatively high cost, Velocity < 3.0 m/s, Roughness coefficient n = 0.015 on an average.

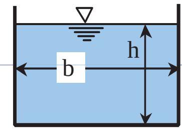
Canal Dimensions

1) flow capacity,
2) velocity,
3) roughness,
4) slope,
5) sectional shape,
6) lining (with or without, material),
5) maintenance

K32-4

Channel Discharge calculation

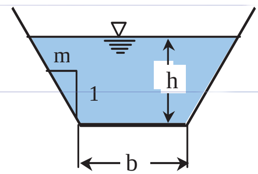
$$Q = \frac{A}{n} R^{\frac{2}{3}} I^{\frac{1}{2}} \qquad Q = V \cdot A$$



For a rectangular section

$$A = b \cdot h \qquad R = \frac{h}{1 + 2h/b}$$

For a triangular section

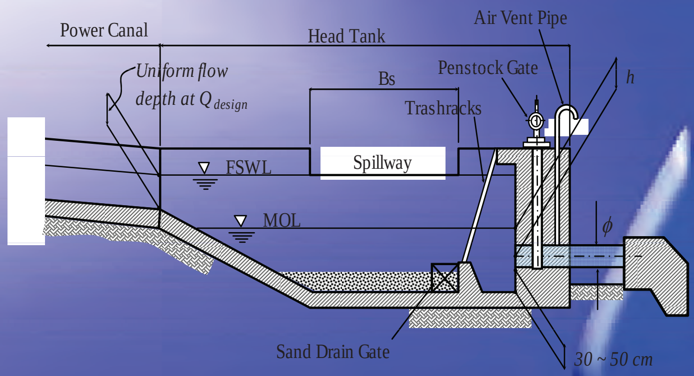


$$A = h(b + mh) \qquad R = \frac{h(b+mh)}{b + 2h\sqrt{1+m^2}}$$

where, Q : discharge (m^3/s), n : roughness coefficient, b : width of canal (m)
h : depth of flow (m), R : hydraulic radius (m), I : slope of canal

Attachment K-3-2

Head Tank



Functions of Head tank

- To materialize smooth transition for the water from free flow in power canal to the pressure flow in penstock.
- To trap flowing debris and leaves by trash racks.
- To deposit sediments (sill elevation of penstock inlet must be higher the floor by 30 cm).
- To release excess water from spillway

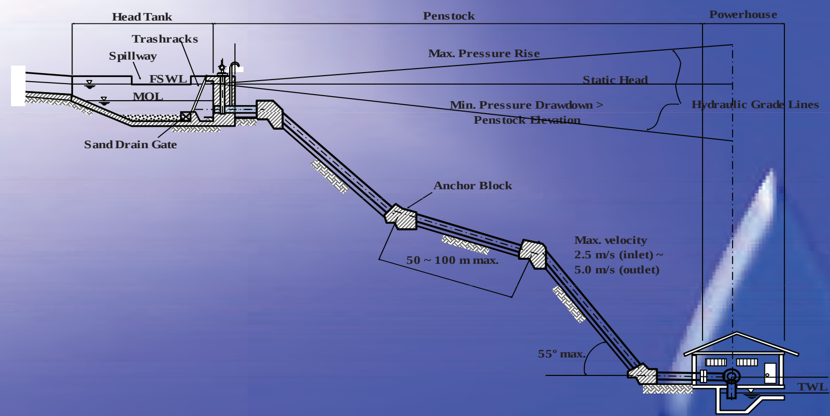
Hydraulic Design

- Required Volume: The capacity of the head tank is determined according to the responsive characteristics of the governors installed in the power plant.
- Discharge from Spillway
- Discharge from Sand Drain Gate
- Minimum Operation Level
- Air Vent Pipe: An air vent pipe is required when the inlet gate is provided on the inlet of the penstock.

The sectional shapes of head tank should be designed to avoid any abrupt changes that can cause the occurrence of a vortex.

An average slope of head tank is 1/15 to 1/50 in order to drain the sediment deposited in the tank through a sand drain gate.

Penstock



To lead the pressured water to the turbine.

Major Types of Penstock

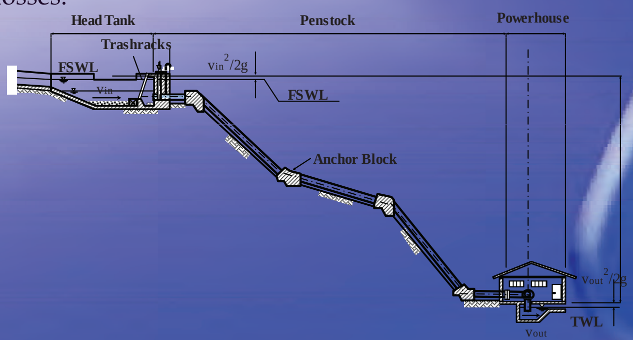
Open Type	**Most commonly applied to small hydro schemes** **Interval of each anchor block should be less than 100 m generally.** **Saddle piers are provided at 6 m interval.** **Maximum angle of pipe inclination should be 55°** **Drainage and slope protection should be considered for the open excavated areas.** **Expansion joints just below the head tank and between each anchor.** **Bitumen between pipes and anchors/saddles to avoid corrosion.**
Embedded Type	Applicable to the following conditions: (a) soft foundations not to support the anchor blocks (b) areas susceptible to attack of landslides or running water (c) gentle slopes to keep the stability of backfill materials Steel pipes should be galvanized, and double coated with either bitumen or high zinc content paint.

Water Hummer

- Discharge in the penstock is changing during the power generation. The discharge is controlled by inlet valve in front of water turbine in the powerhouse. According to the changing of discharge, the velocity in the penstock also changing, and pressure wave occurs at the inlet valve.
- The pressure wave is up and down in the penstock, and pressure wave acts to the penstock. This pressure wave is called as "water hammer".

Head Loss

- In case of MHP scheme, an effective head to be used for the estimate of power output can be obtained from a head difference between Full Supply Water Level (FSWL) at the head tank and Tail Water Level (TWL) at the powerhouse after deducting head losses.



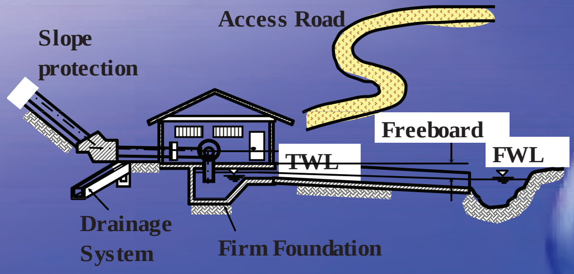
Head Loss Calculation

1) Velocity Head in Head Tank
2) Head Loss at Trashracks
3) Head Loss at Entrance
4) Head Loss due to Friction
5) Head Loss due to Bend
6) Head Loss due to Pipe Reducer
7) Head Loss due to Branch
8) Head Loss due to Inlet Valve
9) Head Loss due to Enlargement at Outlet

The head losses between the head tank and the powerhouse are summation of the above head losses.

Powerhouse

- To protect generating equipment from rainwater.
- To provide shelters for operation.



Tail Water Level

Tail Water Level (TWL) at the powerhouse should be determined so that it will not be affected by the backwater from the river during a flood

DESIGN OF Hydro-Electrical EQUIPMENTs

The main requirements are briefly described below:

- Easy operation
- Durable
- High Availability
- Small Voltage Fluctuation

Equipments

- Turbine
- Generator
- Power Transmission Mechanism
- Control Unit
- Inlet Valve


THANKS

HYDROLOGICAL ANALYSIS FOR MICRO-HYDROPOWER (MHP)

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Abstract
This paper shows the contents of hydrological analysis in "simple pre-feasibility study on a Mini Hydropower MHP (the Study)" as case study of Asurur River in Nandi County in Kenya. Asurur site is located in an off-grid area on tributary of Oroba River. The Study aims to formulate an optimal development plan of rural electrification by MHP. The Study was carried out by a working group of authors together with a JICA expert under the "Project for Establishment of Rural Electrification Model using Renewable Energy in Kenya".

Evaluation of river discharge is vital for the proper planning of hydropower development. River discharge varies daily, monthly and annually, therefore, discharge data with an adequately long period is required for the planning. However, discharge data at candidate sites of MHP are generally not available or limited. In the Study, hydrological analysis was carried out by use of three (3) kinds of data, i.e., i) Discharge data from Water Resource Management Authority (WRMA), ii) Rainfall data from Kenya Metrological department (KMD), and iii), Simulated discharge data by National Water Master Plan 2030 (NWMP 2030). The analyzed results were compared with observed discharge data at Asurur site by the working group, and the design discharge was determined.

It is conceivable that the applied methods in the Study are useful for preliminary evaluation. However, design discharges of MHP shall be determined not only by hydrological analysis but also by observed discharge data at the planning site. Continual observation of river discharge at the planning site for at least 3 years is desirable to formulate MHP planning.

Key words: Kenya, Nandi, Oroba River, MHP planning, hydrological analysis.
Sub-theme: 7. Water, energy, environment and climate change.

1.0 Introduction

Total potential of small, mini, micro and pico hydropower is estimated to be 3,000 MW in Kenya out of which 30 MW has been developed. The upsurge in demand for electric energy since 2004 has revealed an exciting potential for growth in exploitation of the small hydropower subsector. As of 2013, the capacity of the Government-run schemes was generating a total of 15 MW while those by private developers were 10 MW. The development of hydropower has been faced with challenges which include insufficient financial resources, technical personnel and lack of hydrological data for carrying out feasibility studies.

In the Study, hydrological analysis was carried out by the existing available data and results of hydrological analysis in the Study are briefly explained hereinafter.

2.0 Materials and Methods

2.1 Materials of Hydrological Study

The hydrological analysis was carried out by the following four (4) sets of data;

1) Daily discharge data from Water Resources Management Authority (WRMA),
2) Monthly rainfall data from Kenya Meteorological Department (KMD),
3) Simulated monthly discharge data and regional area flood curve by National Water Master Plan 2030 (NWMP 2030), and
4) Observed discharge data at the site by the working group.

The location of planned Asurur site, related gauging stations and related sub-basin are shown below.

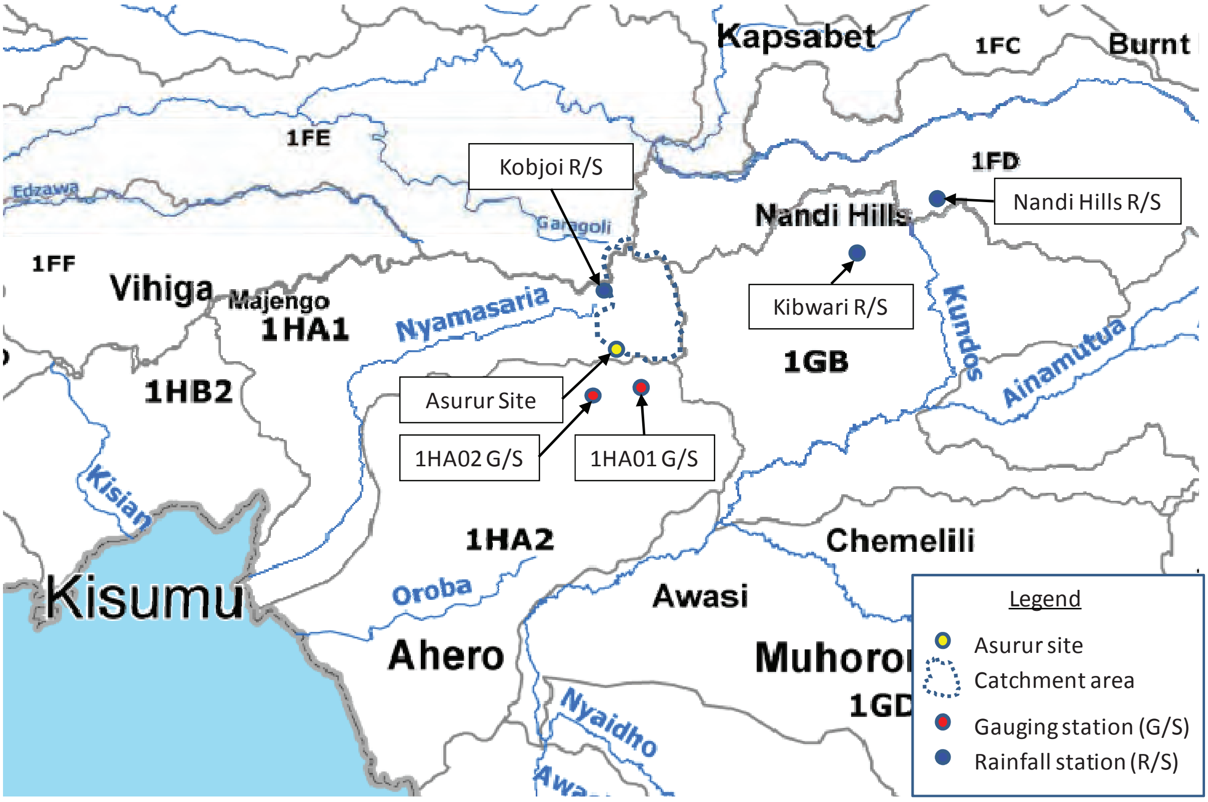

Figure 1: Location Map of Planned Asurur Site and Data Sources

It is noted that the planned Asurur catchment area is situated in Oroba River basin. On the other hand, according to the basin classification of WRMA, Oroba River basin is classified as 1HA2, and Kibos River basin as 1HA1 as shown above. The Asurur catchment area is included in Kibos River basin (1HA1). However, it was confirmed by topographic map with scale of 1: 50,000 that the Asurur catchment area is situated in Oroba River basin. Therefore, the basin boundary of WRMA between 1HA1 and 1HA2 is wrong.

1) WRMA Daily Discharge Data

Daily discharge data at the following two (2) gauging stations were purchased from WRMA.

Table 1: Collected Discharge Data from WRMA

Station ID	Name	Data	Period
1HA01	Great Oroba	Daily discharge	Jan. 1932 – Aug. 1999 (68 years)
1HA02	Little Oroba	Daily discharge	Aug. 1931 – Dec. 2008 (78 years)

2) KMD Monthly Rainfall Data

Monthly rainfall data at the following three (3) rainfall stations were purchased from KMD.

Table 2: Collected Rainfall Data from KMD

Station ID	Name	Data	Period
8934157	Kabujoi Forest Station	Monthly rainfall	Jan. 1993 – Dec. 2012 (10 years)
8935033	Nandi Hills – Savani Estate	Monthly rainfall	Jan. 1993 – Dec. 2012 (10 years)
8935161	Kibweri Tea Estate – Nandi Hills	Monthly rainfall	Jan. 1993 – Dec. 2012 (10 years)

3) Simulated Data by NWMP 2030

Hydrological analysis was conducted in the “Project on Development of the National Water Master Plan 2030 (NWMP 2030)” by JICA in 2013. Results of the following analyses in NWMP 2030 were adopted for the Study.

a) Rainfall-runoff Analysis

Rainfall-runoff analysis was conducted in NWMP 2030 by applying the Similar Hydrological Elements Response model (SHER model) for the whole county. As a result of simulation by SHER model, the data on monthly average naturalized discharge in 204 sub-basins were provided for the period from 1991 to 2010. The naturalized discharge means the river runoff that is not affected by any water use in the catchment area. The monthly average discharge data in sub-basin 1HA1, where the Asurur site is located, were applied.

b) Flood Analysis

The flood analysis was carried out in NWMP 2030. 18 gauging stations which have more than 15 valid years were selected for the frequency analysis from the collected daily discharge data at 47 gauging stations. The valid year was defined as the one with more than 80% data availability. The probable flood discharges of all river reaches in Kenya were evaluated based on the simulated river discharge in the same way as the evaluation based on observed data. Regional area flood curve in Kibos River, where the Asurur site is located, was applied.

4) Observed Discharge Data by the Working Group

River discharge at the planned site was measured in 2012 and 2014 by the Working Group as tabulated below.

Table 3: Discharge Measurement Record at Asurur Site

Measurement Date	1st Measurement (m^3/s)	2nd Measurement (m^3/s)	Daily Average (m^3/s)	Monthly Average (m^3/s)
21 June 2012	1.64	-	1.64	1.71
22 June 2012	1.77	-	1.77	
26 July 2012	1.97	-	1.97	1.97
13 Aug. 2012	1.46	1.59	1.53	1.77
14 Aug. 2012	1.45	1.54	1.50	
27 Aug. 2012	2.11	2.46	2.29	
28 Aug. 2012	2.10	1.87	1.99	
29 Aug. 2012	1.57	1.46	1.52	
10 Sep. 2012	2.26	2.13	2.20	1.79
11 Sep. 2012	1.95	2.03	1.99	
12 Sep. 2012	2.04	1.58	1.81	
23 Sep. 2012	1.46	1.47	1.47	
24 Sep. 2012	1.70	1.66	1.68	
25 Sep. 2012	1.60	1.60	1.60	
31 Jan. 2014	1.33	1.22	1.28	1.28
Maximum	-	-	2.29	1.97
Minimum	-	-	1.28	1.28

Photo 1: Discharge Measurement at Asurur Site

2.2 Methods of Low Flow Analysis

Reliable low flow discharge at the planned Asurur site was estimated based on the following three (3) data. Discharge hydrograph and flow duration curve were created from the estimated daily or monthly average discharges, and the probability was calculated. These results were compared with the observed discharges at the site.

1) Estimation by WRMA Daily Discharge Data

Correlation between the collected WRMA daily discharge data at two (2) stations, named 1HA01 and 1HA02, was checked. The correlation is 0.208, and it is slightly positive, therefore, supplementation of missing data was not executed. Daily discharge data at 1HA01, which is located at downstream of planned Asurur site, was used for the analysis. Daily discharges at the planned Asurur site were estimated by the ratio of catchment area (0.697 = 37.9 km^2 / 54.4 km^2).

2) Estimation by KMD Monthly Rainfall Data

Correlations among the collected KMD monthly rainfall data at three (3) stations were checked. The correlations are 0.658, 0.689 and 0.809, and those are moderate or strongly positive; and, supplementation of missing data was also executed. The monthly rainfall data at Kobjoi Forest station, which is located on the planned Asurur catchment area, was applied for the analysis. Monthly discharges at the planned Asurur site were estimated by the following formula;

$$Q_{ave} = \frac{C \times R \times 10^{-3} \times CA \times 10}{60 \times 60 \times 24 \times D}$$

Where, Q_{ave}: monthly average discharge (m^3/s)

C: runoff ratio (-) = 0.50 (intermediate value of hilly area with forest, land rocky)

Topographic Condition	C
Flat countryside, arable land or sandy soil	0.25 – 0.35
Meadow, smooth slope land	0.35 – 0.45
Hilly area with forest, land rocky	0.45 – 0.55
Mountain area, rock land, frozen surface	0.55 – 0.65

R: monthly rainfall (mm)

CA: catchment area (km^2) = 37.9 (km^2)

D: days of each month (31, 30, 29 or 28 days)

3) Estimation by Simulated Monthly Discharge Data by NWMP 2030

The simulated monthly average discharges in sub-basin named 1HA01, which is included the planned Asurur catchment area, were converted into the discharges at the Asurur site by the ratio of catchment area (0.109 = 37.9 km^2 / 348.8 km^2).

2.3 Methods of Flood Analysis

Probable flood discharge at the planned Asurur site was studied based on the following three (3) data.

1) Estimation by Annual Maximum Discharge in WRMA Daily Discharge Data

Maximum daily discharge in each year between year 1956 and 1988 at the 1HA01 station of WRMA was extracted for the estimation and converted into the discharge at the planned Asurur site by the ratio of catchment area (0.697 = 37.9 km^2 / 54.4 km^2), and these were compared with the average observed discharge of 1.70 m^3/s at the site. However, these annual maximum discharges show relatively small values compared with the observed discharge at the planned site. Annual maximum discharges are distributed within the range of 0.6 to 3.6 times the average observed discharge of 1.70 m^3/s except Year 1988, and 11 values out of 33 values of the annual maximum discharges are smaller than the average observed discharge. Accordingly, it seems that flood discharge data is not indicated in the daily discharge data and this method was not adopted for the probable flood estimation.

2) Estimation by Rational Formula

The rational formula is commonly applied for the estimation of flood discharge at the catchment area less than 200 km^2. The formula is given as follows;

$$Q_P = \frac{1}{3.6} \times f \times R_T \times A \quad \text{[Rational Formula (Lloyd Davies, 1906)]}$$

Where, Q_P: peak discharge (m^3/s)

f: runoff coefficient (-)
= 0.60 (Intermediate value of undulating land and forest).

Land Use Condition	f	Land Use Condition	f
Steep mountainous area	0.75 – 0.90	Irrigated paddy field	0.70 – 0.80
Tertiary mountaious area	0.70 – 0.80	River in highland	0.75 – 0.85
Undulating land and forest	0.50 – 0.75	Small river in plain	0.45 – 0.75
Flat cultivated land	0.45 – 0.60	Large river mostly plain	0.50 – 0.75

R_T: average rainfall intensity in flood duration (mm/hr)

A: catchment area (km^2) = 37.9 (km^2)

$$\therefore Q_P = \frac{1}{3.6} \times 0.6 \times R_T \times 37.9$$

The average rainfall intensity in flood duration (R_T) is estimated based on the monthly rainfall and topographic characteristics in the basin. The average rainfall intensity is estimated by the following two (2) formulae;

$$R_T = \frac{R_{24}}{24} \times \left(\frac{24}{T}\right)^{2/3} \quad \text{[Mononobe Formula]}$$

$$R_T = R_{24} \times \left(\frac{347.1}{t^{1.35}+1{,}502}\right) \quad \text{[Ito Formula]}$$

Where R_T: average rainfall intensity (mm/hr)
R_{24}: probable daily rainfall (mm/day)
T: time to concentration of runoff (hr)
t: time to concentration of runoff (min)

Probable daily rainfall (R_{24}) is assumed to be 10% of monthly rainfall since daily rainfall data is not available. Maximum monthly rainfall in each year at Kobjoi Forest station was extracted from KMD monthly rainfall data, and probability of daily rainfall was calculated based on the above 18 values by the probability distribution function of log Pearson type III, as recommended by the Engineering Manual (EM1110-2-1417: Flood-runoff Analysis) of the US Army Corp of Engineer.

Flood concentration time (*T, t*) is generally estimated by the two times, i.e., overland inflow time from the basin to river and river flow time to the site. Those values are given as follows. The required topographic characteristics in the formulae of the catchment area are assumed from the topographic map with scale of 1: 50,000.

Overland inflow Time

$$T_{OF} = \left(\frac{2}{3} \times 3.28 \times l \times \frac{N}{\sqrt{S}}\right)^{0.467} \quad \text{[Kerby Formula]}$$

Where, T_{OF}: overland inflow time (min)
l: length between basin crest and top of river (m)
N: retardance coefficient (-)

Land Surface Condition	*N*	Land Surface Condition	*N*
Imperious surface	0.02	Forest area with deciduous trees	0.60
Bare ground (grainy surface)	0.10	Forest area with deciduous deposit	0.80
Bare ground (normal surface)	0.20	Forest area with conifer trees	0.80
Rough grass and cultivated fields	0.20	Dense forest area	0.80
Grass farm and normal grass field	0.40	-	-

S: slope of the extreme edge elevation at the most upstream reach (-)

River Flow Time

$$T_{RF} = L/W,\ W = 20 \times \left(h/L\right)^{0.6} \quad \text{[Rziha Formula] (L > 1/20)}$$

Where, T_{RF}: river flow time (s)
L: horizontal length of river channel from top of river to the site (m)
W: flood velocity (m/s)
h: height difference from top of river to the site (m)

3) Estimation by Regional Area Flood Curve by NWMP 2030

Regional area flood curves are given by logarithmic function for each major sub-basin as the results of flood analysis in NWMP 2030. The planned Asurur catchment area is located in Kibos River basin in WRMA's classification, and flood discharges are estimated by the regional area flood curve for Kibos River basin.

4) Confirmation by Creager Envelope Curve

Accuracy of the estimated flood discharges was checked by the Creager envelope curve. The Creager envelope curve is an empirical formula which indicates the relation between catchment area and estimated or recorded flood peak discharge in the form of specific discharge. The Creager envelope curve is given by the following formula:

$$Q_P = 0.503 \times C \times \left(\frac{A}{2.59}\right)^{a-1}$$

$$a = 0.894 \times \left(\frac{A}{2.59}\right)^{-0.048}$$

Where, Q_P: Specific peak discharge (m^3/s/km^2)
C: Creager's coefficient (-)

A: catchment area (km²)

The Creager coefficient "C" indicates regional characteristics of catchment area. Therefore, it is variable depending on the location of concerned basin, but it becomes similar value in nearby basins. The "C" value, therefore, shall be estimated based on the available flood records and/or design flood discharges in the existing projects, and the appropriate value near the concerned basin can be obtained.

3.0 Results

3.1 Low Flow Analysis

Estimated flow duration curves by three (3) kinds of data and observed maximum and minimum discharges are compared as shown below. Range of observed discharges shows relatively low probability in the flow duration curves.

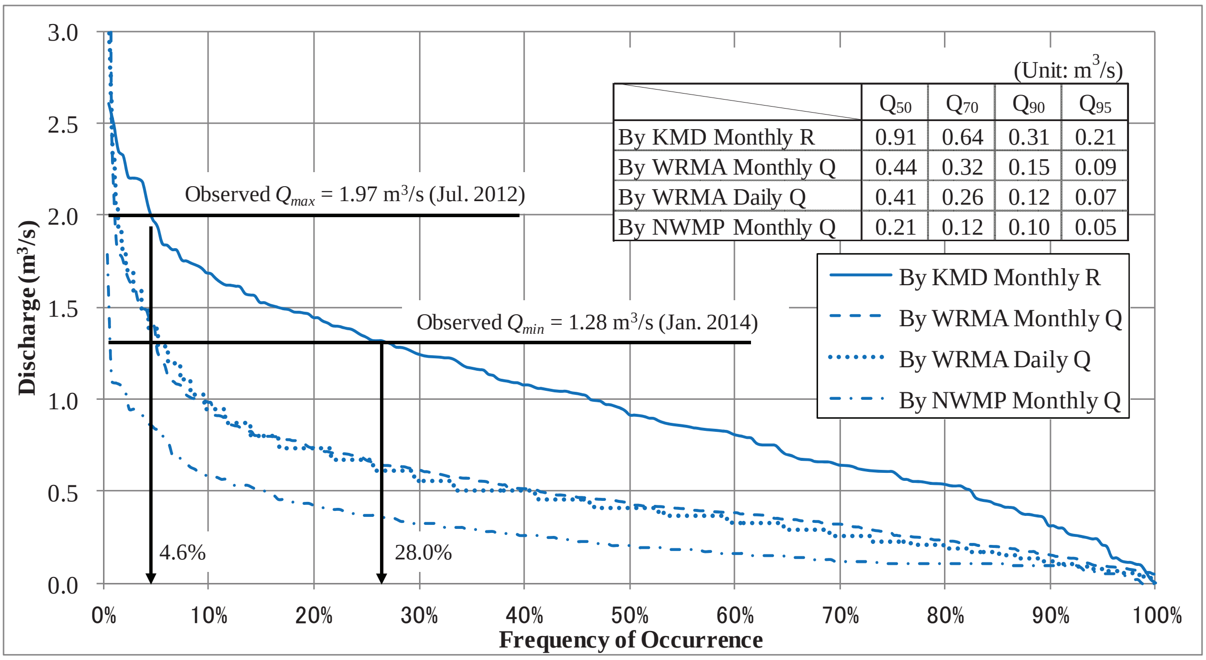

Figure 2: Comparison between Flow Duration Curves and Observed Discharge

Only the rainfall data at Kobujoi Forest Station is available on measurement months among the collected data. Monthly average discharges on measurement months at the planned Asurur site were estimated by available KMD monthly rainfall data in 2012 in the manner described in Section 2.2. 2). As a result, the observed discharges show larger values than the estimated monthly average discharges.

Average annual rainfall at Kobujoi for 18 years (1993 to 2010) is calculated to be 1,661 mm/year. The annual rainfall at Kobujoi in 2012 is not available due to missing data in July and December, but the annual rainfall is assumed to be some 1,360 mm/year based on the monthly average rainfall in July and December from the past 18 years (1993 to 2010). Therefore, annual rainfall in 2012 is smaller than the average. However, the observed discharge shows larger values than the estimated monthly average discharges. Accordingly, it is conceivable that the estimated flow duration curves by three (3) different methods are underestimation.

The following possibilities are conceivable as reasons why the flow duration curves show smaller discharges. Taking into consideration the following possibilities and observed results at the site, the flow duration curve prepared by the KMD monthly rainfall data is applied for the planning of Asurur MHP.

1) WRMA Discharge Data at 1HA01

Asurur River is one of the tributaries of Great Oroba River. The planned Asurur site is located at the shoulder of Nandi Hills and 1HA01 gauging station is located at the foot of Kano Plain as shown below. Accordingly, it is conceivable that infiltration is occurring near 1HA01 gauging station as it is located at the river plain.

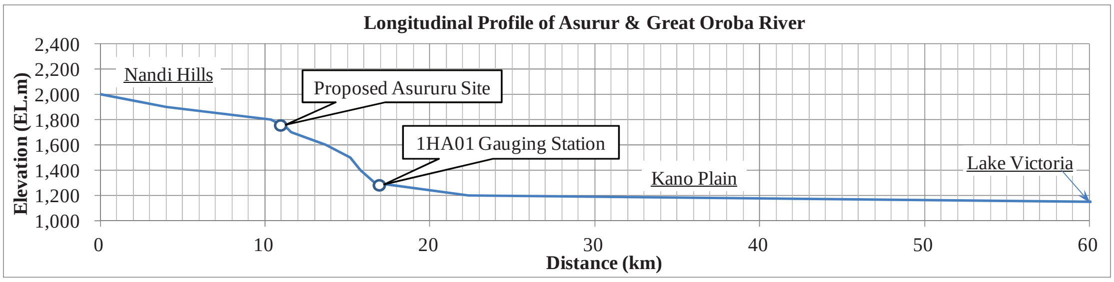

Figure 3: Location of Asurur Site on Longitudinal Profile of Oroba River

2) NWMP 2030 Simulated Discharge Data in Sub-basin 1HA1

The rainfall-runoff analysis was conducted across the nation. The planned Asurur catchment area occupies some 10% of the sub-basin of 1HA1. Accordingly, local characteristics of the Asurur catchment area may not have been captured enough.

3) Overestimation of Discharge Measurement at the Asurur Site

Discharge measurement has some observation errors. It is conceivable that the observed discharge is overestimation.

3.2 Flood Analysis

The probable flood discharges were estimated by two (2) methods; i.e., i) estimation by rational formula and ii) estimation by regional area flood curve in NWMP 2030. The results are explained below.

(1) Estimation by Rational Formula

At first, flood concentration time was estimated as follows;

Overland inflow Time (TOF)

$$T_{OF} = \left(\frac{2}{3} \times 3.28 \times l \times \frac{N}{\sqrt{S}}\right)^{0.467} = \left(\frac{2}{3} \times 3.28 \times 860 \times \frac{0.6}{\sqrt{0.070}}\right)^{0.467} = 49.6\ (\text{min}) = 0.8\ (\text{hr})$$

River Flow Time (TRF)

$$W = 20 \times \left({}^{h}/_{L}\right)^{0.6} = 20 \times \left({}^{320}/_{11{,}270}\right)^{0.6} = 2.36 = 2.4\ (\text{m/s})$$

$$T_{RF} = {}^{L}/_{W} = {}^{11{,}270}/_{2.4} = 4{,}696\ (\text{s}) = 78.3\ (\text{min}) = 1.3\ (\text{hr})$$

Flood Concentration Time (T)

$T = T_{OF} + T_{RF} = 0.8 + 1.3 = 2.1$ (hr)

$t = t_{OF} + t_{RF} = 49.6 + 78.3 = 128$ (min)

Then, the average rainfall intensity was calculated by two (2) formulae, i.e., Mononobe formula and Ito formula, and probable flood discharges are calculated as tabulated below.

Table 4: Estimated Peak Flood Discharge at Asurur Site by Rational Formula

Return Period	(Year)	200	100	50	20	10	5
Excess Probability	(%)	0.5	1.0	2.0	5.0	10.0	20.0
By Mononobe formula							
- Average rainfall intensity (R_T)	(mm/hr)	9.7	9.1	8.5	7.8	7.2	6.5
- Peak flood discharge (Q_p)	(m^3/s)	61.0	57.5	54.0	49.2	45.4	41.3
By Ito formula							
- Average rainfall intensity (R_T)	(mm/hr)	7.4	7.0	6.6	6.0	5.5	5.0
- Peak flood discharge (Q_p)	(m^3/s)	46.9	44.2	41.5	37.9	35.0	31.8

(2) Estimation by Regional Area Flood Curve in NWMP 2030

The peak flood discharges are obtained by plugging in the catchment area into the flood curve as follows;

Table 5: Estimated Peak Flood Discharge at Asurur Site by Rational Formula

Return Period	Regional Area Flood Curve	$log_{10} A$	$log_{10} Q$	Q
10-year	$log_{10} Q = -0.012 + 0.551\ log_{10} A$	1.579	0.857	7.2 m^3/s
25-year	$log_{10} Q = 0.061 + 0.551\ log_{10} A$	1.579	0.937	8.7 m^3/s
50-year	$log_{10} Q = 0.115 + 0.551\ log_{10} A$	1.579	0.991	9.8 m^3/s
100-year	$log_{10} Q = 0.168 + 0.551\ log_{10} A$	1.579	1.040	11.0 m^3/s

(3) Confirmation by Creager Envelope Curve

The flood peak specific discharges and the Creager's coefficient "C" values of hydropower projects in western Kenya and 100-year probable flood peak discharges estimated by two (2) different methods for Asurur MHP were compared as shown below.

Table 6: Creager's Coefficient of Hydropower Projects in Western Kenya

Project Name	County	C (km^2)	Q_d (m^3/s)	Q_p ($m^3/s/km^2$)	C (-)	Remarks
Asurur MHP	Nandi	37.9	58	1.517	5.4	100-year flood by Mononobe formula
			44	1.166	4.1	100-year flood by Ito formula
			11	0.290	1.0	100-year flood by NWMP2030
Proposed MHP's						
Greater Oroba	Nandi	50	121	2.407	9.3	Flood during construction
Kiptiget – Itara Conf.	Bomet	186	42	0.227	1.4	Flood during construction
Ainamoi – Kitio	Kericho	333	75	0.227	1.9	Flood during construction
Chemosit – Kabianga	Bomet	1,370	310	0.227	3.8	Flood during construction
Wei Wei – Sighor	Nakuru	203	35	0.172	1.1	Flood during construction
Embobut – Tot	Elegeyo Marakwet	34	496	14.714	50.1	Flood during construction
Large Scale HPP						
Sondu/Miriu	Kisumu	3,345	1,046	0.313	8.6	Probable maximum flood
			749	0.224	6.1	1,000-year flood
Magwagwa	Nyamira	3,160	1,920	0.608	16.1	Probable maximum flood
			1,634	0.517	13.7	1,000-year flood

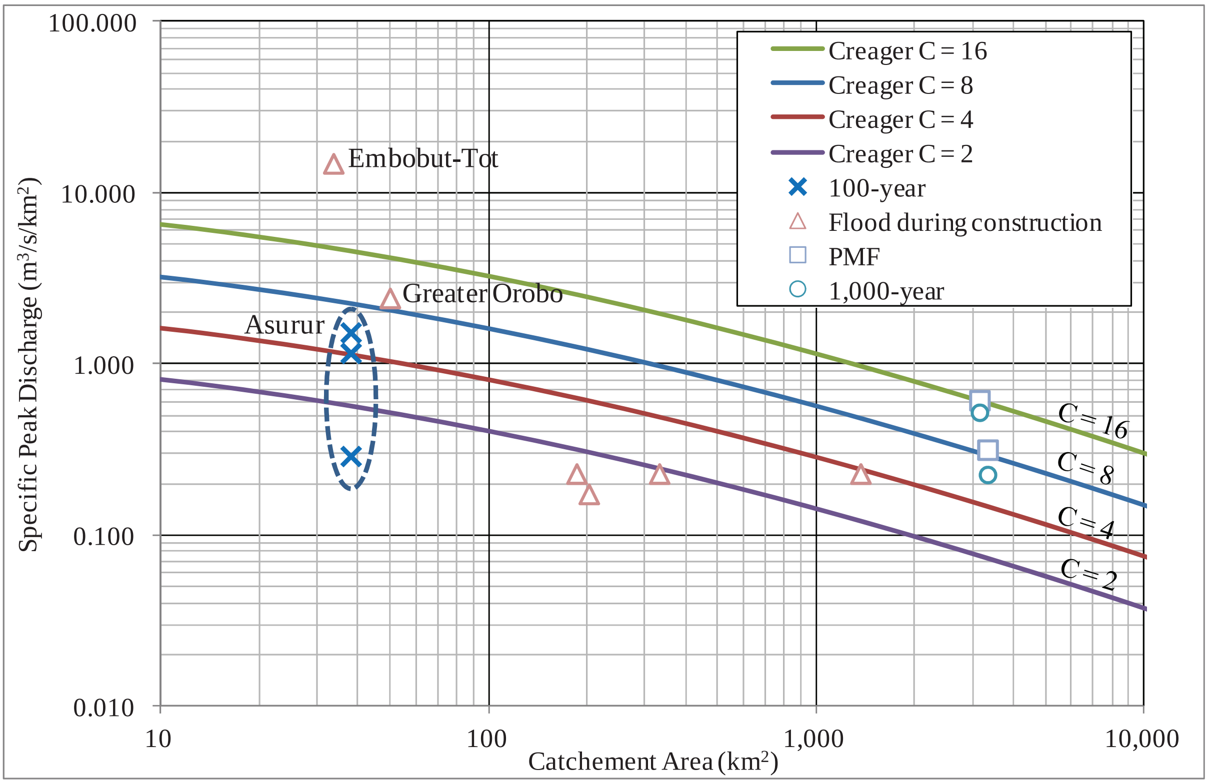

Figure 4: Creager's Coefficient of Hydropower Projects in Western Kenya

The following issues were noted and observed from the comparison;

- ✓ Design flood discharges of probable maximum flood (PMF) and 1,000-year probable flood for Sondu/Miriu HPP and Magwagwa HPP are defined based on the international level of hydrological analysis, and they are regarded as rough indication values,
- ✓ Design flood discharges of the proposed MHP projects are defined as flood during construction, and it is conceivable to be 5-years to 10-years probable flood, however, the “C” values seem relatively large, especially the “C” values for Embobut-Tot and Greater Orobo are remarkably large,
- ✓ The “C” values of assumed 100-year probable flood by Rational Formula for Asurur MHP are smaller than the “C” values of PMF and 1,000-year probable flood and larger than the “C” values of flood during construction, and it seems reasonable, and
- ✓ The “C” value of assumed 100-year probable flood by NWMP 2030 regional area flood curve for Asurur MHP seems to be the smallest among all “C” values, and it seems unreasonable.

Taking the above conditions into consideration, flood discharges estimated by Rational Formula with average rainfall intensity by Ito Formula were adopted for the design discharge of Asurur MHP.

4.0 Conclusions

Lack of available data presented difficulties in the evaluation of the hydropower potential and in the definition of design discharges. It is conceivable that the applied methods in the Study, i.e., i) Discharge data from WRMA, ii) Rainfall data from KMD, and iii) Simulated discharge data and regional area flood curve by NWMP 2030 are useful for preliminary evaluation of river discharges for MHP planning. However, the analyzed results have some discrepancies with observed discharge as shown in the Study.

It is recommended that design discharge of MHP shall be determined not only by hydrological analysis but also observed discharge data at the planning site. Furthermore, continual observation of river discharge at the planning site for at least 3 years is desirable to obtain reliable information to determine the design discharge in order to formulate the reliable MHP planning.

5.0 Acknowledgements

The authors wish to acknowledge JICA and REA for providing this Study opportunity and Asurur Multi-purpose Water Project for cooperating with the Study.

6.0 References

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HYDROLOGICAL ANALYSIS FOR MICRO-HYDROPOWER (MHP)

INTRODUCTION

Total Potential of small, mini, micro and pico hydropower is estimated to be 3,000 MW in Kenya out of which 30 MW has been developed. The upsurge in demand for electric energy since 2004 has revealed an exciting potential for growth in exploitation of the small hydropower subsector.

Evaluation of River Discharge is vital for the proper planning of hydropower development. River discharge varies daily, monthly and annually, therefore, discharge data for an adequately long period is required for the planning. **However**, discharge data at candidate sites of MHP is generally not available or is limited...

REA (Rural Electrification Authority) carried out **the simple pre-feasibility study on Asurur MHP** scheme. The Study aims to formulate an optimal development plan of rural electrification by MHP. The Study was carried out by a working group of authors together with a JICA expert under the “Project for Establishment of Rural Electrification Model using Renewable Energy in Kenya”.

The below contents present the hydrological analysis carried out in the Study.

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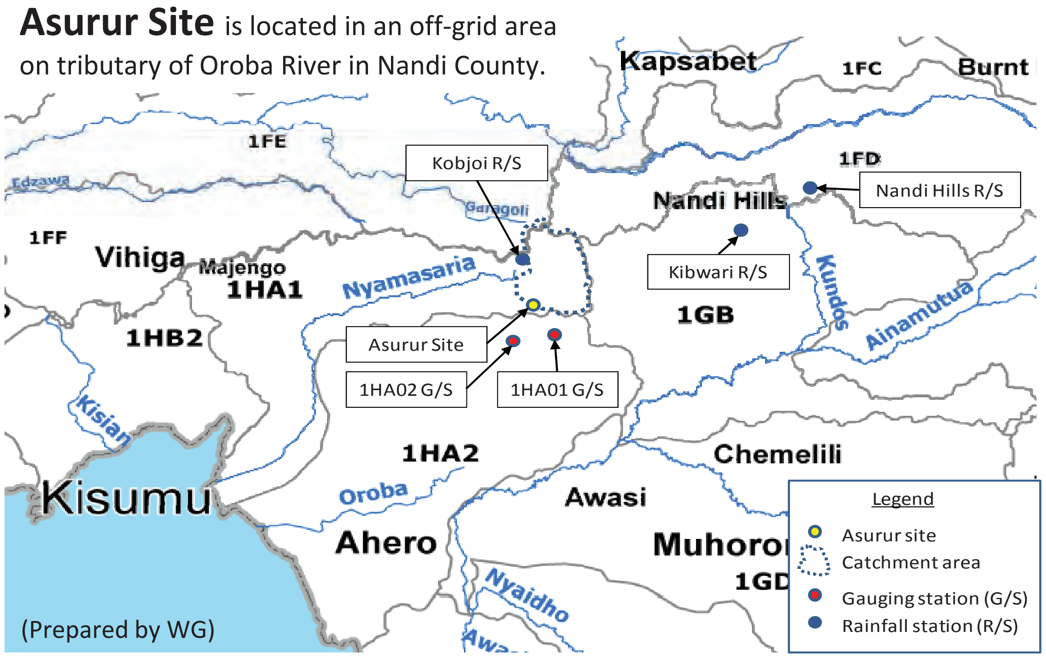
LOCATION MAP

Asurur Site is located in an off-grid area on tributary of Oroba River in Nandi County.



(Prepared by WG)

Overview of Asurur Area (Taken by WG)

APPLIED DATA

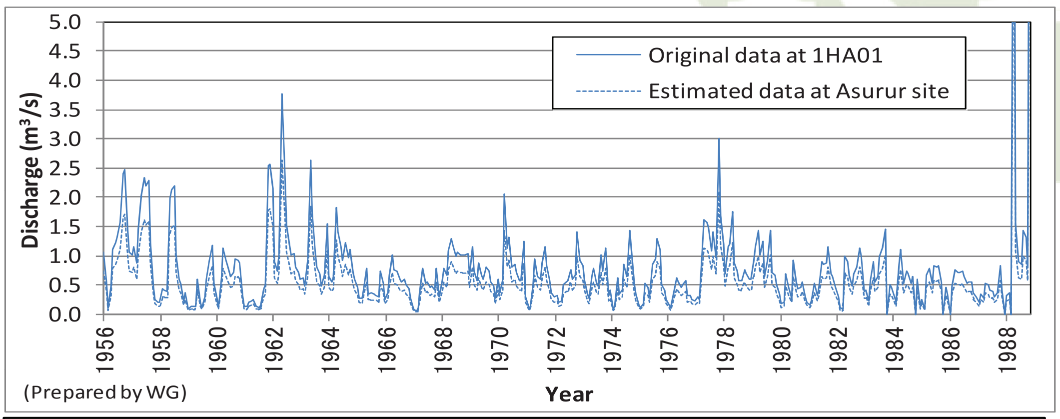
The following data is applied for the analysis.

1) Daily discharge records from WARMA,
2) Monthly rainfall records from KMD,
3) Simulated monthly discharge data from NWMP2030 &
4) Observed discharge data by WG.

Daily Discharge Data from WRMA

(Water Resources Management Authority)

Original data at 1HA01 was applied because correlation is too low. Discharge at Asurur site was estimated by ratio of catchment area.

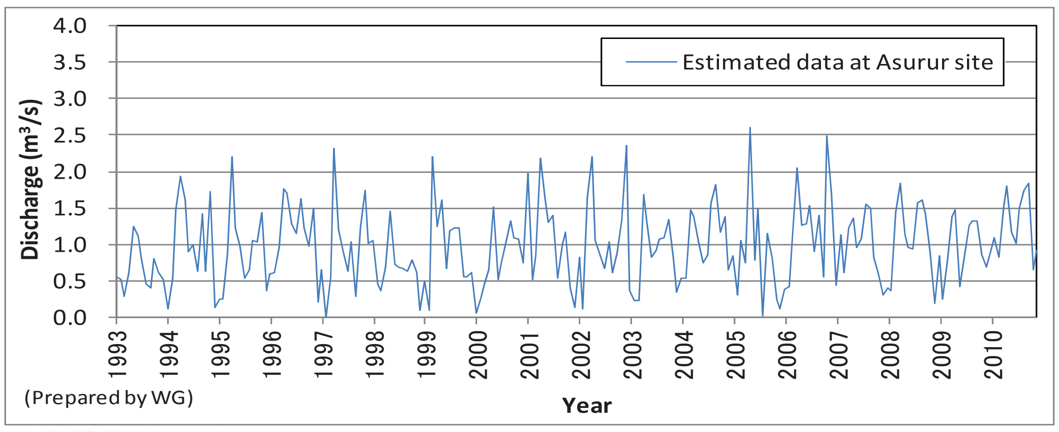


(Prepared by WG)

Monthly Rainfall Data from KMD

(Kenya Meteorological Department)

Supplemented data at Kabujoi was applied because correlations are acceptable. Discharge at Asurur site was estimated by run-off ratio.



(Prepared by WG)

ASURUR MULTI-PURPOSE WATER PROJECT

Asurur MHP development was originaly proposed by the **"Asurur Multi-purpose Water Project"**, which is a registered self-help group in Nandi South District.

The working group discussed with them to confirm the social economic conditions, willingness for electrification, plan of electricity utilization, etc. As well, they assisted the working group on the discharge measurement.

(Taken by WG)

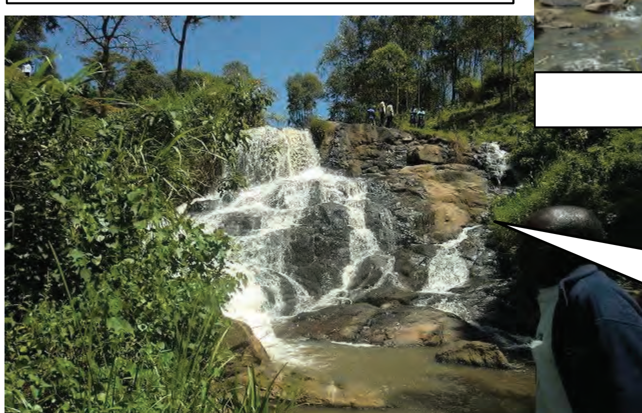
Consultation Meeting

(Taken by WG)

Discharge Measurement

(Taken by WG)

Discharge Measurement



(Taken by WG)

Asurur Waterfall

Head of some 30 m &
Discharge of some 1.0 m^3/s

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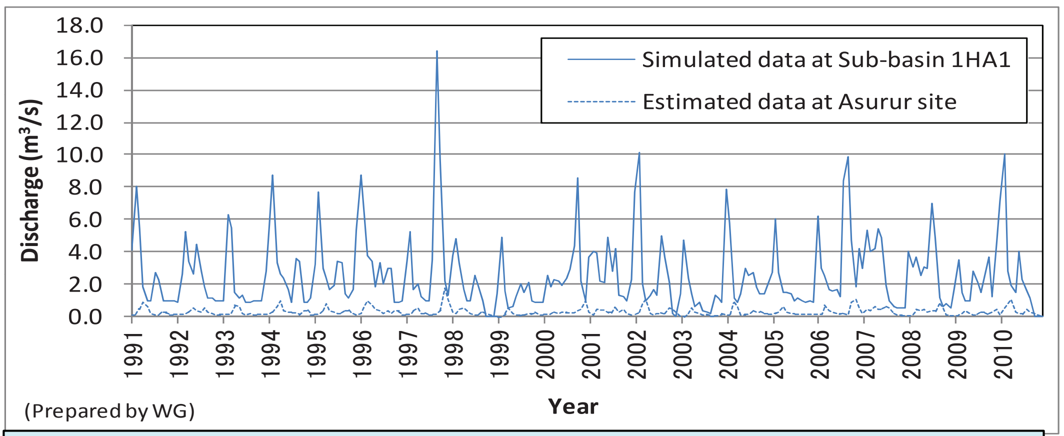
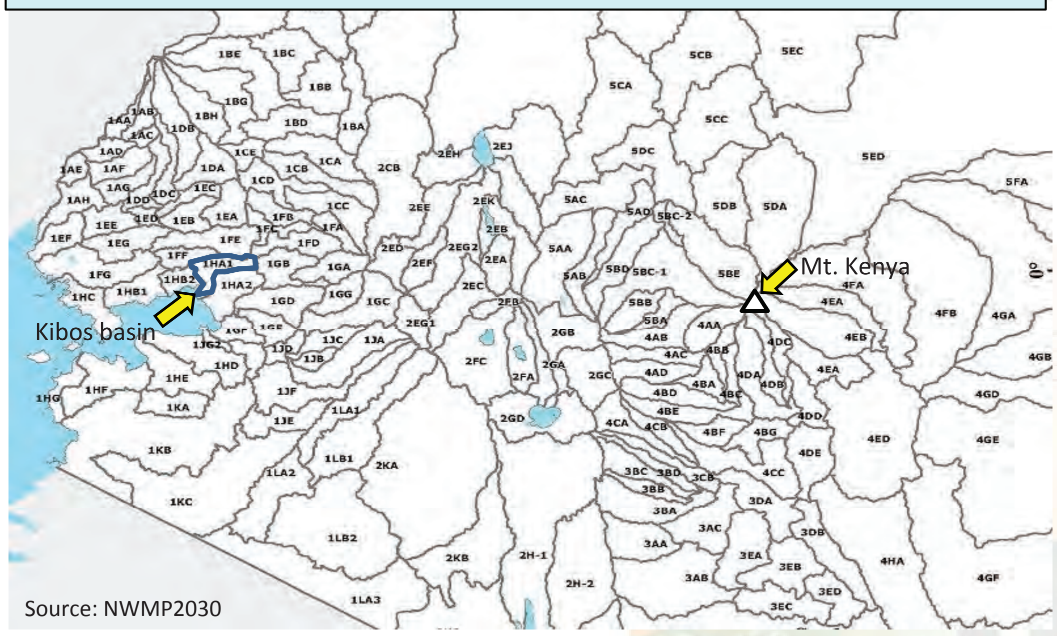
HYDROLOGICAL ANALYSIS FOR MICRO-HYDROPOWER (MHP)

Simulated Monthly Discharge Data from NWMP2030 (National Water Master Plan 2030)

Simulated monthly discharges were provided by NWMP2030 for 203 sub-basins. Discharge at Asurur site was estimated by ratio of catchment area.



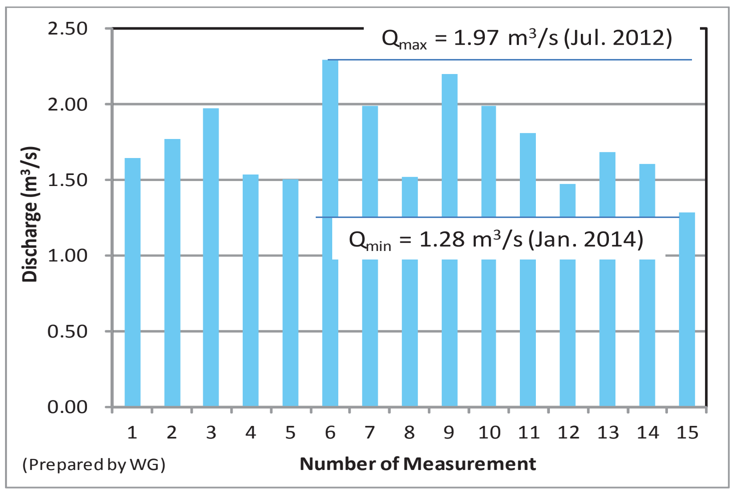
Sub-basins surrounding Potential Area of Hydropower Development



Observed Discharge Data by Working Group

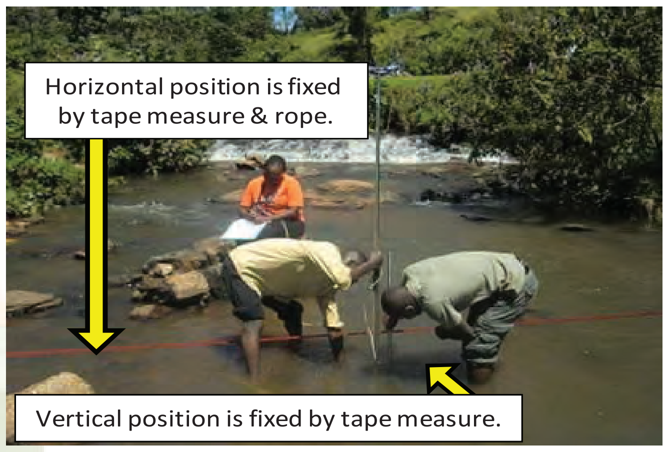
The discharge measurements were carried out 15 times in total during June to September 2012 and January 2014 as shown below.

No.	Date	Discharge (m^3/s)
1	21-Jun-12	1.64
2	22-Jun-12	1.77
3	26-Jul-12	1.97
4	13-Aug-12	1.53
5	14-Aug-12	1.50
6	27-Aug-12	2.29
7	28-Aug-12	1.99
8	29-Aug-12	1.52
9	10-Sep-12	2.20
10	11-Sep-12	1.99
11	12-Sep-12	1.81
12	23-Sep-12	1.47
13	24-Sep-12	1.68
14	25-Sep-12	1.60
15	31-Jan-14	1.28



Continuous observation is desirable, but the observation was limited due to project management issues.

- ✓ The measurement had been done twice a day, and the average was adopted as a discharge of the day,
- ✓ The width of river section is approximately 8 m, and measurement was done for 1 m interval.



How to Observe Discharges by Current Meter?

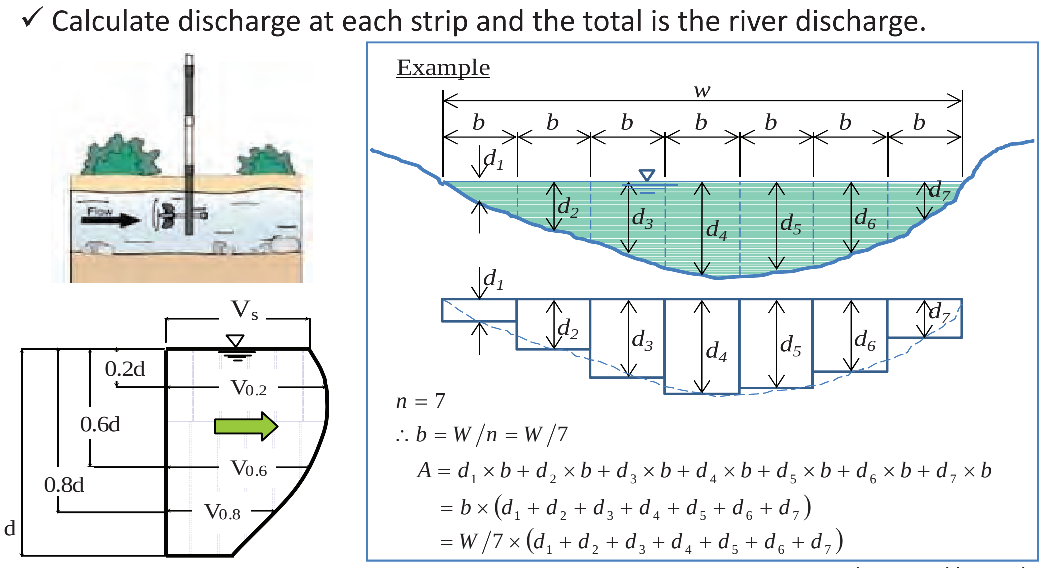
Current meter is a device that measures velocity of river flow. The river discharge is obtained as follows:

$$Q = A \times V$$

"A" is a sectional area of flow, and
"V" is a velocity to be obtained by the measurement.

The procedure of discharge observation is as follows:

- ✓ Divide the river width (w) into n (> 2) uniform strips,
- ✓ Measure water depth (d_i) at the center of each strip,
- ✓ Adjust the current meter at the center of each strip,
- ✓ Measure the velocity at least twice at same position,
- ✓ Calculate discharge at each strip and the total is the river discharge.



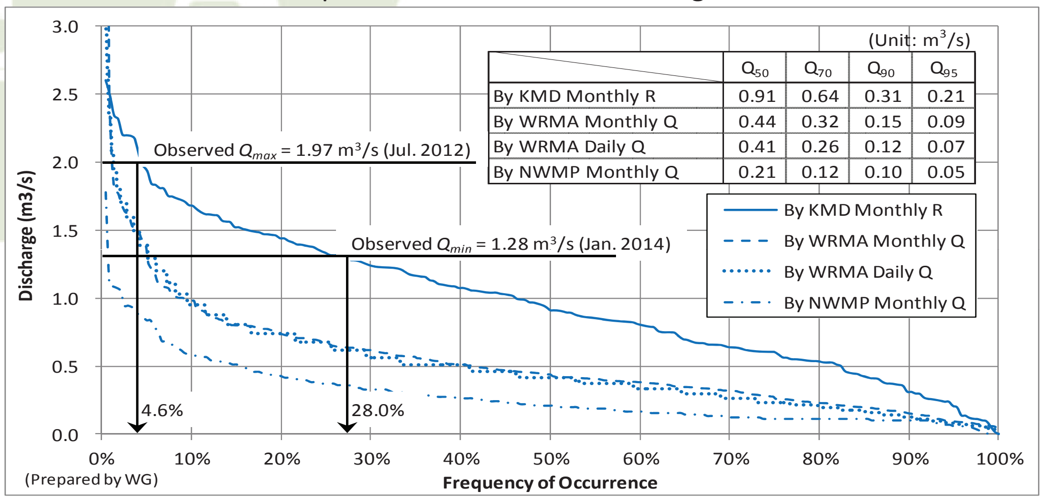
(Prepared by WG)

Flow velocity is distributed as shown above. Therefore, vertical position is variable depending on the river depth:

- ✓ In case that the river is deep enough, measure the velocity at 0.2d & 0.8d,
- ✓ In case that the river is shallow, measure the velocity at 0.6d only.

COMPARISON OF LOW FLOW ANALYSIS

The estimated hydrographs by three (3) kinds of data is converted into flow duration curves and compared with observed discharges as shown below.



	Q_{50}	Q_{70}	Q_{90}	Q_{95}
By KMD Monthly R	0.91	0.64	0.31	0.21
By WRMA Monthly Q	0.44	0.32	0.15	0.09
By WRMA Daily Q	0.41	0.26	0.12	0.07
By NWMP Monthly Q	0.21	0.12	0.10	0.05

Observed discharges show larger values than other estimated discharges. The following posibilities are conceivable as reasons why the flow duration curves show smaller discharges:

- ✓ The Asurur site is located at the shoulder of Nandi Hills and WRMA 1HA01 gauging station is located at the foot of Kano Plain. Accordingly, infiltration is occuring,
- ✓ The rainfall-runoff analysis in NWMP2030 was conducted across the nation. Accordingly, local characteristics may not have been captured enough, and
- ✓ Discharge measurment has some observation errors, and the observed discharge is overestimation.

⇨ **Continuous observation at the site is essential.**

As a result, the design plant discharges are defined as follows:

Minimum discharge: **0.1 m^3/s** ($Q_{90\%}$ - $Q_{95\%}$)
Maximum discharge: **0.7 m^3/s** ($Q_{50\%}$ - $Q_{95\%}$)

* $Q_{95\%}$ is the amount of the reserve for ecological and basic human needs in accordance with "Guidelines for Water Allocation (WRMA, 2010)".

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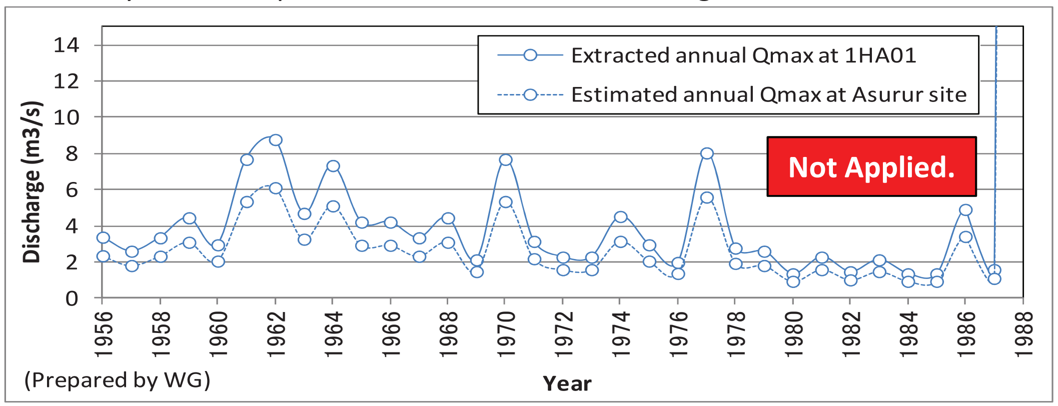
FLOOD ANALYSIS

Estimation by Daily Discharge Data from WRMA

Annual maximum discharges at 1HA01 were extracted and converted to Asurur site by ratio of catchment area. However, these discharges are relatively small compared with the observed discharges as shown below.



Estimation by Monthly Rainfall Data from KMD

From the monthly rainfall data, average rainfall intensity was estimated by two methods, and peak discharge was estimated by rational formula:

$$Q_P = \frac{1}{3.6} \times f \times R_T \times A$$

Where, Q_p: peak discharge (m³/s)
f: runoff coefficient (-)
R_T: average rainfall intensity (mm/hr)
A: catchment area (km²)

Return Period	(Year)	200	100	50	20	10	5
Excess Probability	(%)	0.5	1.0	2.0	5.0	10.0	20.0
Peak Flood Discharge							
- Method 1 (Mononobe)	(m^3/s)	61.0	57.5	54.0	49.2	45.4	41.3
- Method 2 (Ito)	(m^3/s)	46.9	44.2	41.5	37.9	35.0	31.8

(Prepared by WG)

The rational formula is commonly applied for the estimation of flood discharge at the catchment area less than 200 km².
For details, see the full paper.

Regional Area Flood Curve from NWMP2030

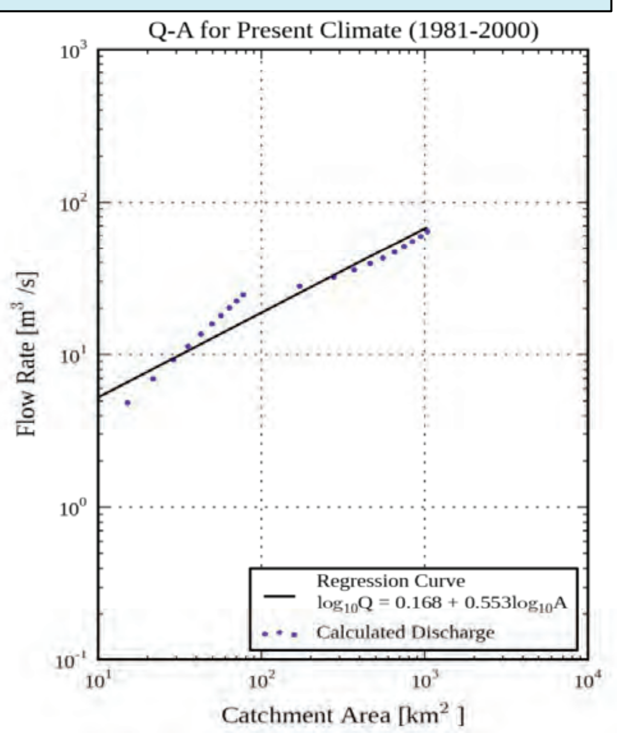
(National Water Master Plan 2030)

Regional area flood curve by logarithmic function are provided by NWMP2030 based on the simulated river discharge.

10-year, 25-year, 50-year & 100-year probable flood curves are analysed for 21 major basins listed on the right.

Sample of Frequency Analysis

(LVS-14 Kibos, 100-year return period)



Source: NWMP2030

Lake Victoria North
- LVN-2 Yala
- LVN-3 Nzoia
- LVN-6 Malikisi
- LVN-7 Malaba

Lake Victoria South
- LVS-2 Asure
- LVS-5 Mara
- LVS-6 Migori
- LVS-10 Kabondo Awach
- LVS-11 Sondu
- LVS-12 Nyando
- LVS-13 Oruba
- LVS-14 Kibos
- LVS-16 Awach Seme

Rift Valley
- RV-2 Ewaso Ngiro (South)
- RV-12 Kedong
- RV-14 Lake Naivasha
- RV-19 Lake Bogoria
- RV-20 Lake Baringo

Athi AT-1 Athi

Tana TN-1 Tana

Ewaso Ngiro
- EN-1 Ewaso Ngiro

Estimation by Regional Flood Curve from NWMP2030

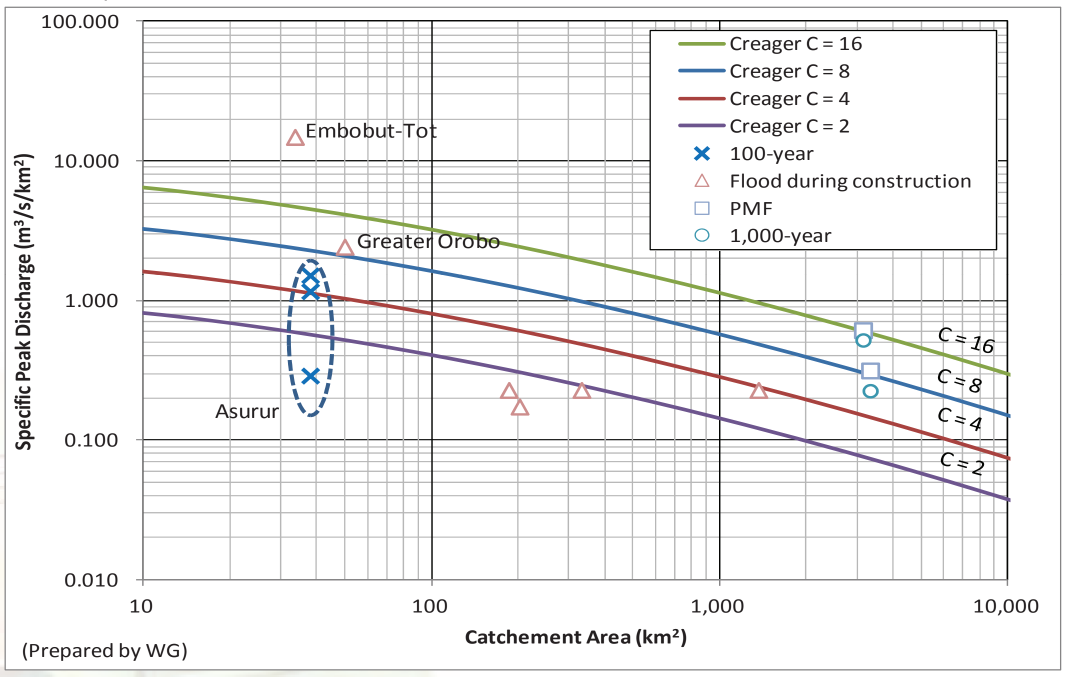
Asurur site is situated in Kibos basin in accordance with WRMA classification. The peak flood discharges were obtained by plugging in the catchment area into the regional area flood curve as follows:

Return Period	Regional Area Flood Curve	$log_{10} A$	$log_{10} Q$	Q (m^3/s)
10-year	$log_{10} Q = -0.012 + 0.551 log_{10} A$	1.579	0.857	7.2
25-year	$log_{10} Q = 0.061 + 0.551 log_{10} A$	1.579	0.937	8.7
50-year	$log_{10} Q = 0.115 + 0.551 log_{10} A$	1.579	0.991	9.8
100-year	$log_{10} Q = 0.168 + 0.551 log_{10} A$	1.579	1.04	11.0

(Prepared by WG)

Confirmation by Creager Envelope Curve

Accuracy of the estimated flood discharges was checked by the Creager envelope curve as shown below:



As a result, the design flood discharge is defined as follows:
Q = 42.0 m³/s (50-year probable flood)

(Taken by WG)

CONCLUSION

Lack of available data presented difficulties in the evaluation of the hydropower potential and in the definition of design discharges.
The following points are recommended to formulate the reliable MHP planning in view of effective utilization of valuable natural resources as the conclusion of our experiences:

✓ It is conceivable that the **applied methods in the Study are useful** for preliminary evaluation of river discharges for MHP planning,
✓ It is recommended that design discharge of MHP shall be determined **not only by hydrological analysis but also by observed discharge data** at the planning site, and
✓ It is desirable that **continual observation** of river discharge at the planning site for at least 3 years be carried out to obtain reliable information to determine the design discharge.

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