MICRO-HYDROPOWER DESIGN AIDS MANUAL

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Small Hydropower Promotion Project (SHPP/GTZ)	**Mini-Grid Support Programme (MGSP/AEPC-ESAP)**

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NOTICE

These micro-hydropower design aids have been prepared to provide a basis for consultants to undertake calculations and prepare drawings as per the requirements set aside by Alternative Energy promotion Centre (AEPC) of His Majesty's Government of Nepal (HMG/N). It is expected that the use of these design aids will result in a standard approach to carrying out calculations and drawing on Peltric feasibility studies and preliminary and feasibility studies of micro-hydro projects.

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1. INTRODUCTION

1.1 GENERAL

This Micro Hydropower Design Aids Manual covers the procedural guidelines for using Micro Hydro Design Aids prepared under the collaboration between Small Hydropower Promotion Project (SHPP/GTZ) and Alternative Energy promotion Centre (AEPC) of His Majesty's Government of Nepal (HMG/N). The Micro hydro Design Aids, which consists of a set of spreadsheets and drawings, has been developed as a tool to provide quick and reliable means of computing design parameters of different components of the perspective micro hydropower plants in Nepal.

Small Hydropower Promotion Project is a joint project of His Majesty's Government of Nepal (HMG/N), Department of Energy Development (DoED) and German Technical Cooperation (GTZ). Since its establishment in 2000, this project has been providing its services to sustainable development of small hydropower projects in Nepal (100kW to 10MW) leading to public private participation and overall rural development.

Alternative Energy Promotion Centre (AEPC) is a government organization established to promote alternative sources of energy in the rural areas.

Since the nature of small hydropower and micro-hydropower projects are similar in many aspects, SHPP/GTZ signed collaboration with the AEPC to help the sector. During the course of supporting and backstopping micro-hydropower sector as per the scope defined in the collaboration, SHPP helped AEPC upgrading the existing AEPC guidelines, developing a user-friendly micro-hydropower related computational tools and typical drawings. These micro hydropower tools (MHP Design Aids) have been prepared to provide a basis for consultants to undertake calculations and prepare drawings as per the requirements set aside by AEPC. It is expected that the use of these design aids will result in a standard approach to carry out calculations and represent the calculations with the help of standard drawings on Peltric feasibility studies, preliminary feasibility and detailed feasibility studies of micro-hydro projects.

The Design Aids consist of a set of fifteen typical drawings and fourteen typical spreadsheet calculations useful for Nepali micro hydropower projects up to feasibility study level. During the preparation of these design aids, special efforts were made so that the skills and knowledge of the practicing stakeholders such as consultants, manufacturers, inspectors, etc, are not replaced by this design aids. Constant efforts were equally made to make the design aids simple and user friendly. Since most of the stakeholders are familiar with Autodesk AutoCad 2000 and Microsoft Excel XP application software, the design aids were made based on the software.

It is also expected that the stakeholders will be using the electronic version of the design aids. The design aids are distributed in template/read-only forms so that the original copy is always preserved even when the users modify them.

The design aids are specifically prepared for micro hydropower schemes up to 100kW. Although, there are many common approaches and features in all hydropower projects, special care should be taken while using these aids for bigger schemes if the circumstances are unavoidable.

Preparation and use of the design aids is a continuous process. Therefore, valuable suggestions and feedbacks are expected from all the stakeholders/users so that the overall quality of the micro hydro sector is enhanced.

1.2 OBJECTIVES OF THE DESIGN AIDS

The main objective of the design aids is to enhance the quality of the micro hydropower sector in Nepal. The other objectives or function of the design aids are outlined below:

1. It should be able to function as a "Time Saver Kit" for precision and speed (e.g. hydrological calculations based on exact flow measurement date, Q flood off take, friction of penstock, etc.).
2. It should be able to function as references for stakeholders of the micro hydro sector for using and upgrading their skill and creativity. Any external references that may require referring during calculations are minimised by incorporating them in the cell notes, tables, pictures, etc.
3. The presented reports by different consultants are uniform and their data presentations are to the required depth.
4. The design aids should be able to serve as templates and not a readymade design set so that there are sufficient rooms for further creativity and improvement to tailor to incorporate specific needs of particular projects.
5. In addition to the objectives stated above, the design aids should equally be handy and user friendly. The user familiar AutoCad 2000 and MS Excel XP software platforms are used to develop the design aids.

1.3 SOURCES OF THE DESIGN AIDS

Since the Design Aids was made aiming to enhance the overall quality of the micro hydro sector, the prudent materials available and applicable for the specific purposes, studies of different preliminary and feasibility reports of different micro hydro plants, feedbacks suggestions from the stakeholders, etc., are based during the preparation of the design aids. Review of following sources was utilized during the designing of the design aids:

1. Updated AEPC micro hydropower guidelines and standards.
2. Review, assessment and appraisal of the more than 250 preliminary feasibility reports, 150 feasibility and about 50 Peltric micro hydropower project study feasibility reports during the on going SHPP/GTZ-AEPC collaboration.
3. Feedbacks from all the stakeholders such as Reviewers, Consultants, Developers, Manufacturers, Installers, etc.
4. Experience from other micro, small and large hydropower projects.
5. Standard textbooks, guidelines and standards. These sources are listed in the list of references at the end of this manual.

1.4 DESIGN AIDS: TYPICAL DRAWINGS

All together, fifteen AutoCad 2000 drawings are prepared and incorporated in the design aids. Since they are only typical drawings, additions of drawings and their level of details may be changed to fulfil specific needs of a particular project. During the preparation of these drawings, the level of consistency, compatibility and the extent of information in the drawings that are appropriate for micro hydropower plants are considered. Adequate information is incorporated in the drawings so that the presented drawings are complete and all the stakeholders are able to understand and implement the presented content. The main features of the presented drawings are:

1. Minimum required details such as plans and adequate cross sections.
2. Recommended values of elements presented in the drawings such as the minimum thickness of a stone masonry wall, the longitudinal slope of a settling basin, etc.
3. Standard line types and symbols.
4. Elements such as the title box with all the information and controlling signatories; scale; etc.

5. All drawings with standard layouts for a specific printer (may have to be changed as per the actual available printer).

While editing the drawing based on the project needs, it should be noted that the dimensions should be amended along with their geometries. This is especially important so that a 2m long structure is always longer than a 20cm structure.

The presented drawings cover from intake to transmission line. A set of all the drawings are presented in the appendix. For an example, a typical drawing of a gravel trap is presented in Figure 1.1. The drawings that are presented are listed in Table 1.1.

Table 1.1: Summary of Drawings

SN	Drawing Name (*.dwg)	Remarks
1	01 General Layout	General layout showing project components except the transmission and distribution components.
2	02A Side Intake Plan	A general Plan of the headworks including river training, trashrack, intake, gravel trap and spillway.
3	02B Side Intake Sections	A longitudinal section along the water conveyance system from intake to headrace, two cross sections of weir for temporary and permanent weirs respectively and a cross section of spillway.
4	03 Drop Intake Plan	A general plan, a cross section across the permanent weir and a cross section of the drop intake.
5	04 Headrace	A longitudinal headrace profile showing different levels along the chainage.
6	05A Gravel Trap	A plan, a longitudinal section and two cross sections.
7	05B Settling Basin	A plan, a longitudinal section and two cross sections.
8	06 Headrace Canal	Two cross sections for permanent lined and one for temporary unlined canal.
9	07 Forebay	A plan, a longitudinal section, two cross sections and penstock inlet details.
10	08 Penstock Alignment	Longitudinal section of a penstock alignment.
11	09 Anchor & Saddle Blocks	Plans and sections of two different types of anchor blocks and saddle.
12	10 Powerhouse	A plan and a section of a typical powerhouse.
13	11 Machine foundation	A plan and three sections of a typical machine foundation.
14	12 Transmission	A single line diagram if a transmission/distribution system.
15	13 Single line diagram	A single line diagram showing different electrical components.

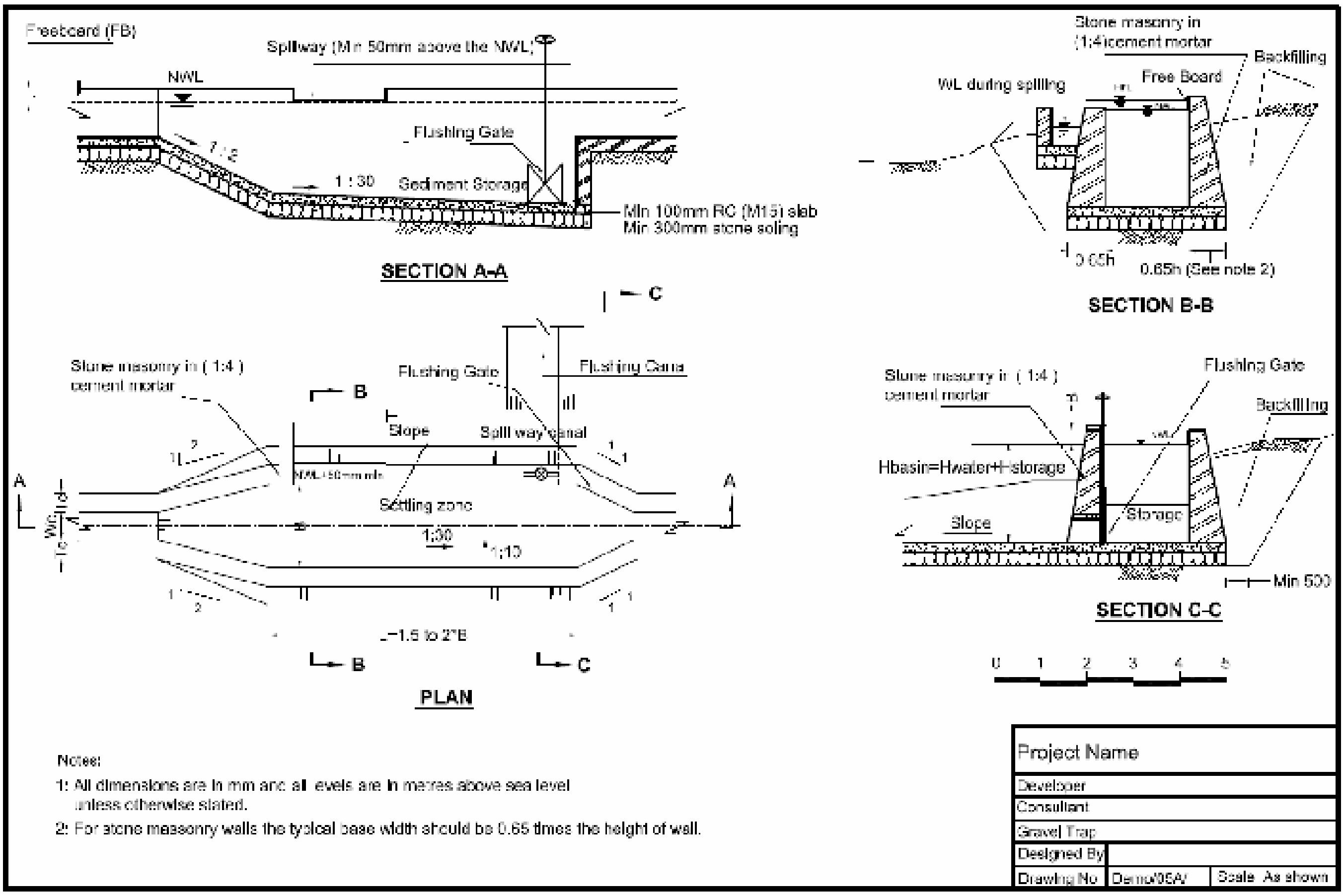


Figure 1.1: A typical Gravel Trap Drawing

1.5 DESIGN AIDS: SPREADSHEETS

As stated earlier, the MS Excel XP is used to develop the spreadsheets. General features of Excel XP are utilized while developing the spreadsheets. There are thirteen main spreadsheets each covering a tool required for covering computations for an element of hydro power schemes. The "Utility" spreadsheet presented at the end of the workbook covers minor calculations such as the uniform depth of water in a rectangular canal, loan payback calculations, etc. The list of the presented spreadsheets and their areas of coverage are presented in Table 1.2.

Table 1.2: Summary of Spreadsheets

SN	Name	Chapter and Area of coverage
1	Discharge	Chapter 2: Computation of river discharge measurement with Salt dilution method.
2	Hydrology	Chapter 3: Hydrological calculations based on MIP and Hydest methods
3	Side Intake	Chapter 4: Design of side intakes including coarse trashrack, flood discharge and spillways.
4	Bottom Intake	Chapter 4: Design of bottom intake including flood bypass.
5	Settling Basin	Chapter 6: Design of settling basins, gravel traps and forebay with spilling and flushing systems with spillways, cones and gates.
6	Canal	Chapter 5: Design of user defined and optimum conveyance canals with multiple profiles and sections.
7	Pipe	Chapter 5: Design of mild steel/HDPE/PVC conveyance pipes.
8	Penstock	Chapter 7: Design of penstock with fine trashrack, expansion joints and power calculations.
9	Turbine	Chapter 8: Selection of turbines based on specific speed and gearing ratios.

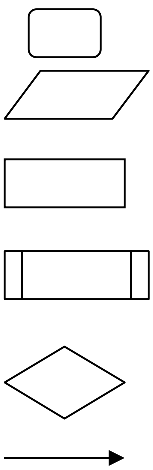
10	Electrical	Chapter 8: Selection of electrical equipment such as different types of generators, cable and other accessories sizing.
11	Transmission	Chapter 9: Transmission / Distribution line calculations with cable estimation.
12	Load Benefit	Chapter 10: Loads and benefit calculations for the first three years and after the first three years of operation.
13	Costing & Financial	Chapter 11: Costing and financial analyses based on the project cost, financing and annual costs and benefits.
14	Utilities	Chapter 12: Utilities such as uniform depth, loan payment calculations, etc.

Since the design aids are aimed at enhancing the quality of micro hydro up to feasibility study level, the financing, manufacturing, etc., part are not covered in greater details.

The spreadsheets not only speed up the computational processes with adequate level of precision but also provide adequate information (printable as well as references such as cell notes, etc). The main features of the presented spreadsheets are:

1.5.1 Flow chart notations

Standard flow chart notations are used describing the flows of programs and procedures. The following notations are mostly used:



Start and End

Input

Processing formulas and output

Processing and output from other sub routine

Conditional branching

Flow direction

1.5.2 Iterative Processes

It saves tedious and long iterative/repetitive processes. Repetitive processes are generally error generating element in human computation and it take considerably longer time. A typical repetitive process is presented in Figure 1.2.

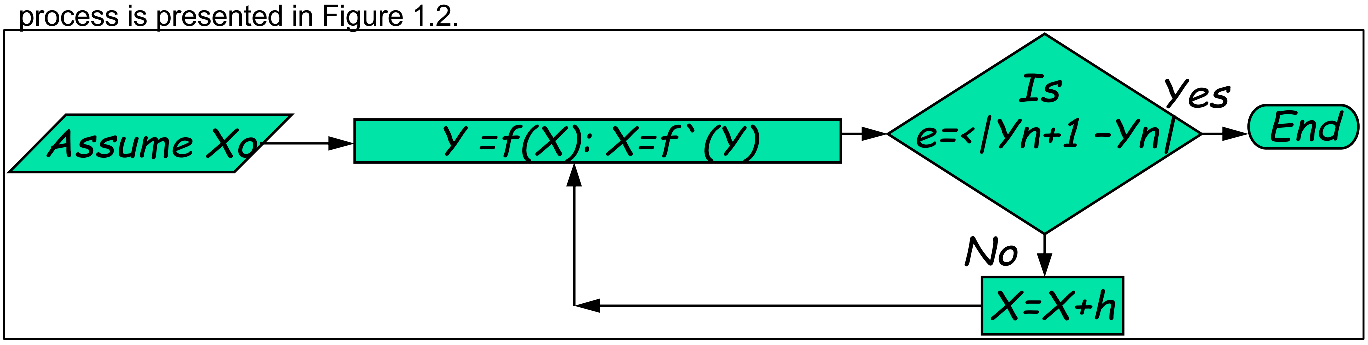


Figure 1.2: Iterative process

As shown in the figure, the assumed value of X_0 is amended until an accepted error limit is reached. By default, Excel does not activate this kind of iterative process and generates Circular Reference Error. The iterative features in Excel can be activated by selecting Calculations tab (Tools->Options->Calculations>Tick Iteration (cycles & h)) and checking the iteration box. This feature is activated in Figure 1.3.

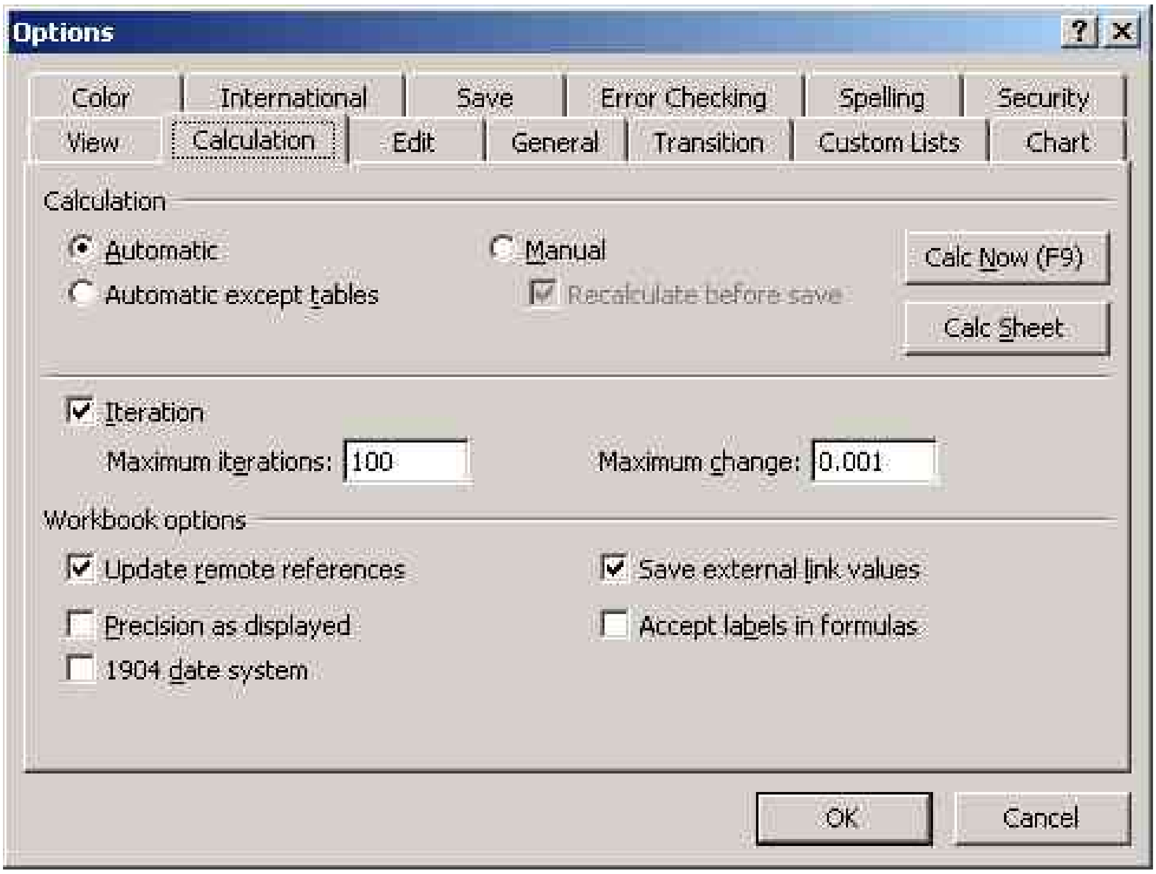


Figure 1.3: Activation of iteration (Tools => Option =>Calculations

1.5.3 Macro Security

The spreadsheet contains some visual basic for application functions and procedures. Because of the safety reasons against viruses, MS Excel disables such VBA functions and procedures by default. For the proper execution, the security level has to be set to medium so that the users will be cautioned and asked whether to enable such Macros. Setting security level to medium (Tools => Macros => Security => Medium) and enabling the macros of the spreadsheets are required for the proper operation. Setting security level to medium and enabling the macros are presented in Figure 1.4.

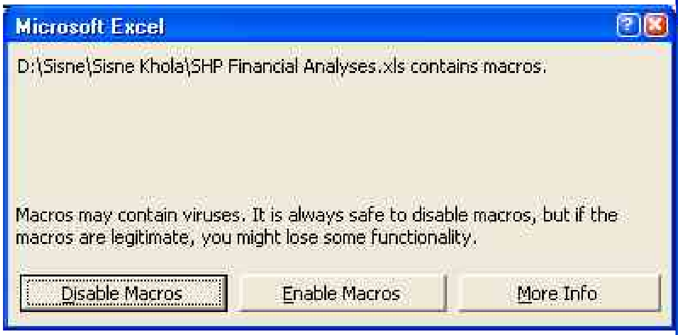


Figure 1.4: Macro security and enabling macros

1.5.4 Individual vs. linked spreadsheets

By default, the spreadsheets are designed in such a way that it can be used as linked spreadsheets for a single project. The user does not have to input the linked data such as the name of the project in every spreadsheet. Some of the other processed data such as the design discharge or flood discharge are also linked by default. However, the users are free to change these values for specific calculations i.e., the spreadsheet can also be used as individual spreadsheets for independent calculations. It is recommended to save an extra copy of the workbook before manipulating such linked cell so that the saved copy can be used as a workbook with linked spreadsheets for a single project.

1.5.5 User specific inputs

Some of the parameters such as the freeboard, width of a canal, factor of safety for a mild steel penstock, etc., have their standard optimum values. However, the user may choose to enter specific values of specific project even if these are not the standard or optimum values. By default, the standard optimum values are computed. However, the user can input the specific non standard values in the spreadsheets.

1.5.6 Interpolated computations

Some of the parameters such as the coefficient of bend, coefficient of gate discharge, etc., have standard proven values for standard conditions. However, if the user input value is not of a standard tabulated value, the interpolated values with the help of curve fitting are used for the calculation. The users are cautioned to check the validity of such values whenever they encounter them.

1.5.7 Errors

Mainly three types of errors are generally expected. One of them is NAME# error which is caused by not functioning the custom functions and procedures present in the spreadsheets. In case such an error occurs, close the workbook, activate the macro security level to medium and enable the macros when opening the workbook again. A typical NAME# error occurs adjacent to the depth of water during flushing yfi (m) and d50f during flushing (mm) in the settling basin spreadsheet.

VALUE# error is the other error that is generated by the malfunctioning of circular references. When such an error occurs, select the error cell, press F2 and press Enter. Q intake Qf cumec in the side intake is such an error.

REF# error in transmission line computation occurs due to the deletion of unnecessary rows in a branch. In such an instance, copy the second cell from the second line of any branch.

1.5.8 Cell notes

Based on the requirement for understanding the computational procedures, understanding used formulas, mandatory requirement set aside by AEPC, references required for deciding proper values and criteria, sufficient cell notes are provided.

A cell containing a cell formula for calculating the specific speed of a turbine is presented in Figure 1.5. Similarly, Figure 1.6 presents a basic table for selecting Manning's coefficient of roughness of a canal.

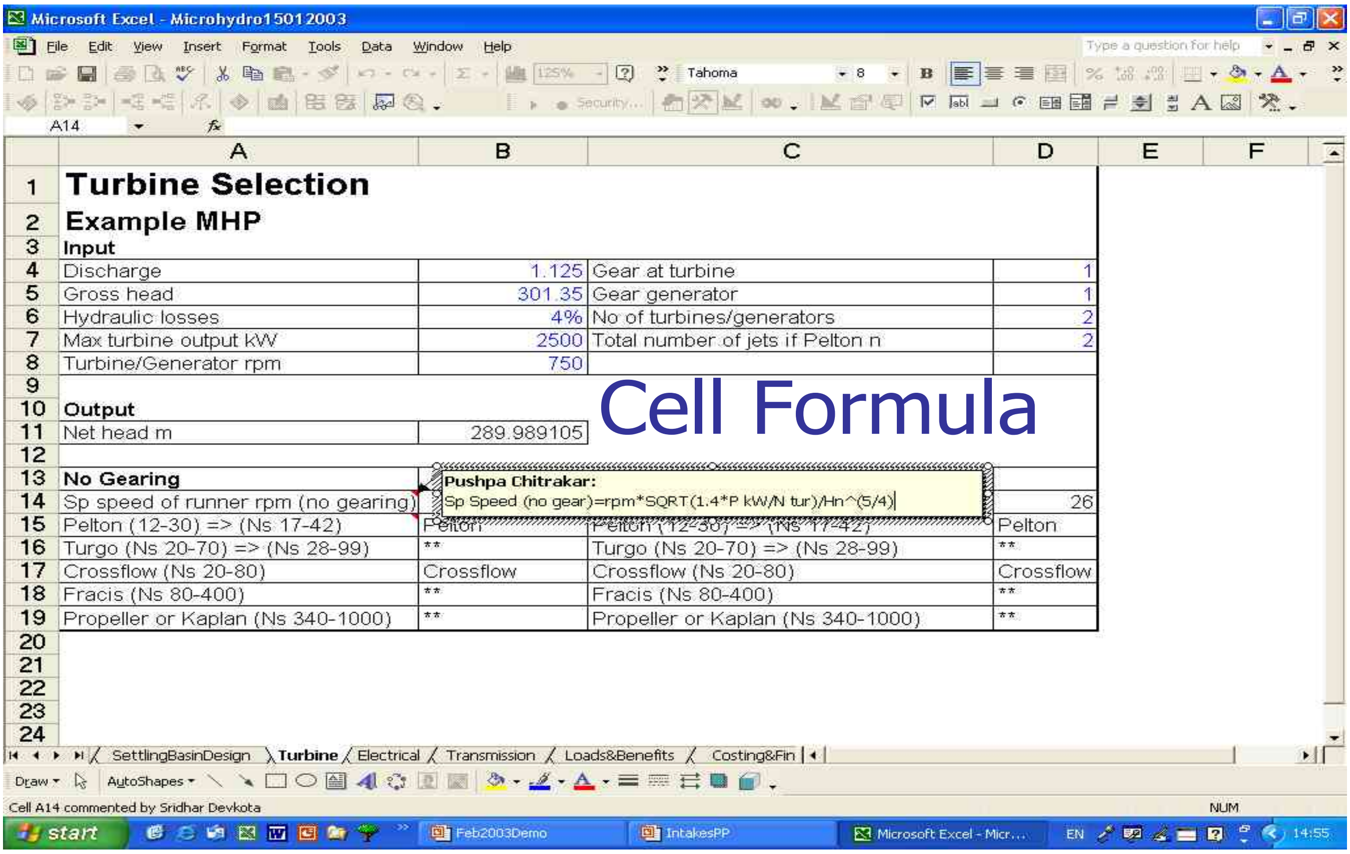


Figure 1.5: Cell formula incorporated in a cell note.

1.5.9 Cell Text Conventions

Three different colours are used to distinct three different types of cells. A typical example of colour coding of cell text is presented in Figure 1.7. The colours and categories of these cells are:

Blue cells: These cells represent the mandatory input cell. The user must input these cells with the correct values of the scheme data for correct output. The mandatory input includes the name of the project, head, discharge, etc.

Red cells: These cells are optional input cells. Standard values are presented in these cells. If a user has sufficient ground to change these values, he or she is allowed to do so by changing these values. It is worth noting that care should be taken while changing these values. The optional values/ input include the density of sediment, sediment swelling factor, temperature of water, etc.

Black cells: The black cells represent the information and or output of the computations. For the sake of protecting accidental and deliberate amendment or change leading to wrong output, these cells are protected.

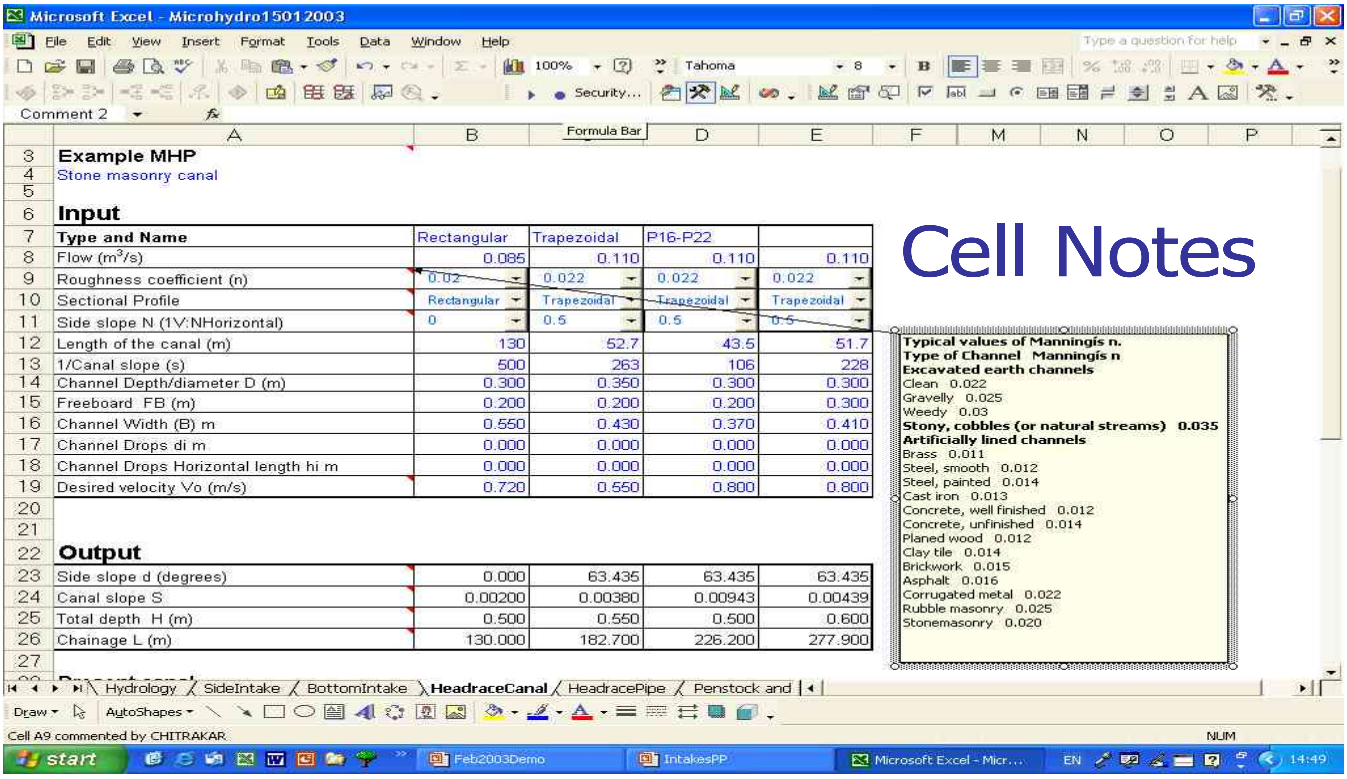


Figure 1.6: A cell note presenting typical values of Manning's n for different surfaces

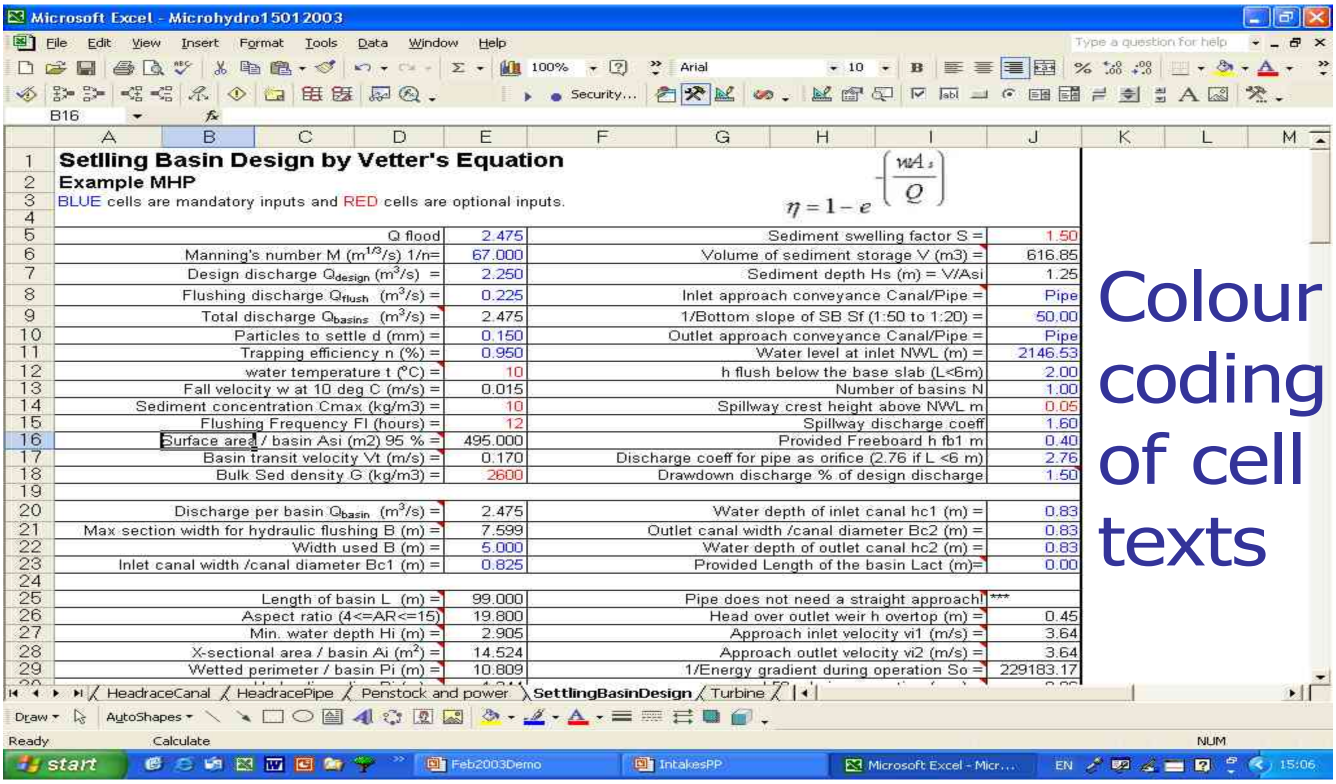


Figure 1. 7: Colour coding of cell texts

1.5.10 Types of input values

Based on the nature of the input, the inputs are categorised in the following three groups:

1. ***User or project specific input***: The inputs variables that totally depend on the user and or the project are categorised as the user or project specific inputs. The programs do not restrict on or validate the values of such inputs. The name of the project, gross head of the projects, etc., are some of the examples that fall on this category. The length of the crest in the example presented in Figure 1.8 can have any value hence it is a user specific input.

2. ***Prescribed Input***: Some of the inputs have some standard values. The programs suggest using such values and giving choices for the user to select. However, the programs do not restrict on or validate such variables. These inputs are termed as prescribed inputs. With the help of a pull down menu, the probable type of the surface is presented for deciding the Manning's coefficient in Figure 1.8. The user does not have to remember the specific values of the type of the surface he is considering. However, he is free to input any values if he has a very good reason to change them.

3. ***Mandatory Input***: Some inputs can have some specific values and the programs need to validate such values for proper computations. These values are termed as mandatory inputs. Since Nepal is divided into seven MIP regions, the MIP region value can have an integer ranging from 1 to 7. In the example presented in Figure 3.1, the MIP region can have values from 1 to 7, the month can have a string from January to December and the date can have an inter ranging from 1 to 31. In case the user enters different values, the program will generate error and prompt for the correct inputs.

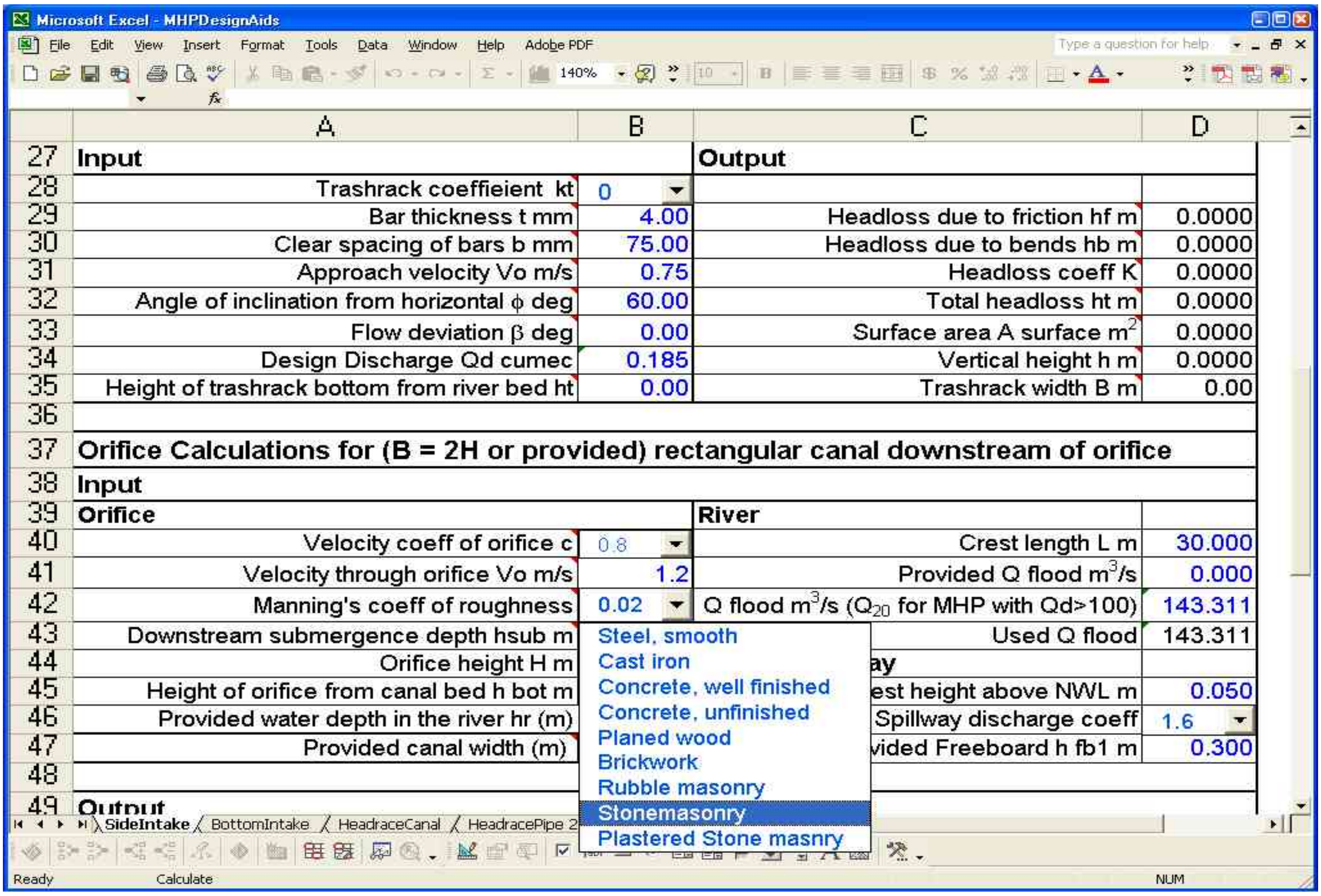


Figure 1.8: Different categories of inputs.

1.5.11 Pull Down menus and data validation

Some of the input cells are equipped with pull down menus to facilitate the users to input standard values related to the input cell. Pull down can have any user specific values than the stated standard values if the data validation for that particular cell is not specified. In Figure 1.8, the pull down menu for Manning's roughness coefficient (n) in cell B42 is activated. Out of the different surface materials are listed in the pull down menu, stone masonry surface is selected and the corresponding standard value of the Manning's coefficient of roughness of 0.02 is substituted in the corresponding cell. Since the value in this cell is not restricted, the user can have any values for this cell.

Some inputs such as the name of the month, MIP hydrological region and dates in Hydrology spreadsheet can have specific values in their respective cells. Since the outcome of the computation will be erroneous if the input data does not match with the desired values, the spreadsheets are designed to reject such an invalid value and flag the error with suggestions. For instance, Nepal is divided into seven MIP hydrological regions. It can have a value from 1 to 7. If a user inputs 8 as the MIP region, the data is rejected and an error message along with suggestions will be flagged. This example is demonstrated in Figure 1.9.

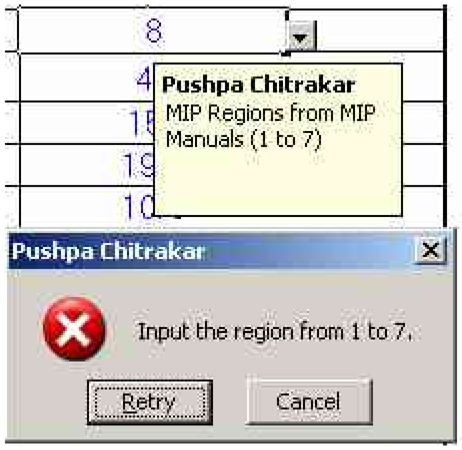


Figure 1.9: Different categories of inputs.

2 DISCHARGE MEASUREMENT

Almost all potential micro hydropower project sites are located in remote areas where there is a complete lack of hydrological information. Tools developed for estimating hydrological parameters are mainly based on regional correlations. The outputs of these tools are close to the actual parameters for rivers having bigger ($100km^2$ or more) catchment areas. Since most of the potential micro hydropower plants have relatively smaller catchment areas, use of these regional regression methods based on at least one set actual site measurement is recommended in the AEPC guidelines for estimating flow parameters. Bucket method for flow up to 10 l/s, weir method for a flow from 10 to 30 l/s and for flows larger than 30l/s salt dilution method (conductivity meter) are recommended.

Since the salt dilution method is quick (less than 10minutes per set of measurement), easier to accomplish and reliable, its accuracy level is relatively higher (less than 7%) and suitable for smaller fast flowing streams (up to 2000 l/s), preference should be given to this method. The change of conductivity levels of the stream due to the pouring of known quantity of predefined diluted salt (50-300gm per 100l/s) are measured with a standardised (conductivity meter with known salt constant, k) conductivity meter at a regular interval (e.g., 5 seconds). For more information, please refer to MGSP Flow Verification Guidelines or Micro Hydro Design Manual (A Harvey).

As per the MGSP guidelines, at least one set of discharge measurement at the proposed intake site should be carried out between November and May. The discharge calculation spreadsheet presented in the Design Aids can handle up to four sets of data and the individual as well as the average discharges are presented as outputs.

The input parameters required for discharge measurement calculation are presented in Table 2.1. The typical input parameters considered in the example are presented in the adjacent column. The first set field readings are presented in Table 2.2.

Table 2.1: Input parameters for Salt Dilution Method

SN	Input parameters	Input for the cited Example
1	Project	Upper Jogmai, Ilam
2	Conductivity Meter	HANNA Instruments HI 933000
3	Date	12-Jan-04
4	Type of Salt	Iyoo Nun
5	Conductivity Constant	1.8 at 15°C
6	Water temp	15°C
7	Time Intervals (dt)	5sec
8	Weights of salt for sets 1 to 4 ()	400g, 1580g and 1795g
9	Readings (μ & $\mu_{baseline}$) for sets 1 to 4	Presented in Table 2.2

Table 2.2: First set conductivity reading for Salt Dilution Method Example

	Time(sec)												
	5	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**	**Sum**
Water Conductivity in µS	25	26	27	28	29	30	31	32	32	33	34	34	361
	34	35	35	35	35	34	34	34	33	33	33	32	407
	32	32	32	31	31	31	31	31	31	30	30	30	372
	30	29	29	29	29	29	29	28	28	28	28	28	344
	28	28	27	27	27	27	27	26	26	26	26	26	321
	26	26	26	26	26	26	26	25	25	25			257
											Total (µS)		**2062**
											Total readings		**70**

With the input parameters, the discharge at the stream is calculated by the following procedures:

Stream Flow, **Q = M x k/A**, where

Q = flow in lit/sec

M = mass of the dried salt in mg (i.e.10^{-6} kg).

k = the salt constant in (μS)/(mg/litre).

A is the area under the graph of conductivity versus time, after excluding the area due to base conductivity. The units for the area under the graph is sec x μS. The area is determined as follows:

Area (A) = ($\Sigma\mu$– nr x $\mu_{baseline}$) * dt

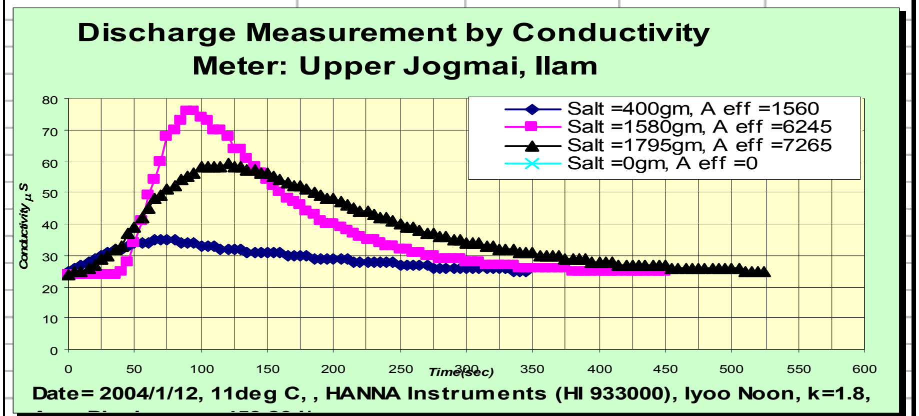
Weighted averages of the individual flows thus calculated are computed. The outputs of the spreadsheet are presented in Figure 2.1. The estimated average discharge will further be used by Medium Irrigation Project Method (MIP) to calculate long term average monthly flows. The calculations procedures for the first set (Set 1) are:

Area (A) = ($\Sigma\mu$– nr x $\mu_{baseline}$) * dt
= (2062-70*25)*5
= 1560 sec x μS

Discharge (Q) = M x k/A
= 400000*1.8/1560
= 461.54 l/s

Discharge Measurement by Conductivity Meter

Project	Upper Jogmai, Ilam			
Meter	HANNA Instruments (HI 933000)			
Salt	Iyoo Noon			
Water temp:	11 deg C	Date:		12-Jan-04
Given k	1.8	Time intervals		5 sec
Salt Const. (k)		1.8000		
Wt. of Salt	400 gm	1580 gm	1795 gm	0 gm
Nr of data	70	91	106	0
Baseline conductivity	25	24	24	0
Sum of readings	2062	3433	3997	0
Effective Area	1560	6245	7265	0
Discharge	462 l/s	455 l/s	445 l/s	0 l/s
			Average Discharve	454 l/s



Time	Reading 1	Reading 2	Reading 3	Reading 4
0	25	24	24	
5	26	24	25	
10	27	24	25	
15	28	24	26	
20	29	24	27	
25	30	24	29	
30	31	24	30	

Figure 2.1: Discharge calculations by salt dilution method

3 HYDROLOGY

3.1 HYDROLOGY AND NEPALI MHP

Hydrology is the science that deals with space-time characteristics of the quantity and quality of the waters of the earth. It is the intricate relationship of water, earth and atmosphere.

As stated earlier, almost all potential micro hydropower project sites are located in remote areas where there is a complete lack of hydrological information. According to the ACPC guidelines, with a set of actual measurement in dry season (November-May), the mean monthly flows for these sites are estimated by Medium Irrigation Project (MIP) method Developed by M. Mac Donald in 1990. The flood hazards are generally critical for a project having a design discharge more than 100 l/s. For such a project the floods should be calculated by using 'methodologies for estimating hydrologic characteristics of ungauged locations in Nepal" WECS/DHM 1990 Study (Hydest) is recommended. These factors are incorporated in "Hydrology" spreadsheet. Brief introduction of these two methods are presented in the subsequent sub-sections.

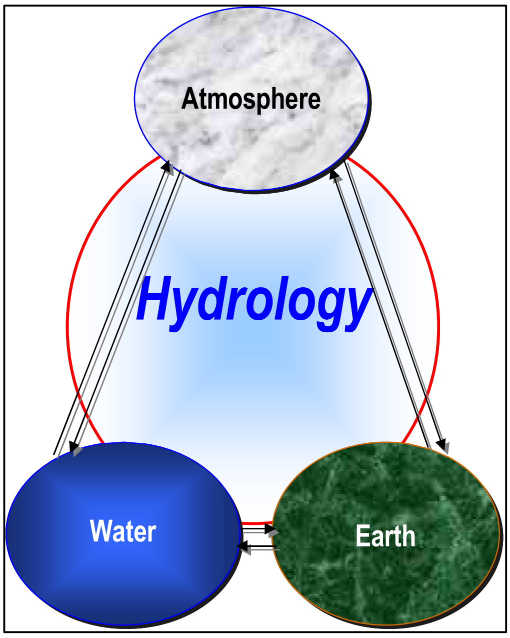


Figure 3.1: Hydrology

3.2 HYDROLOGICAL DATA

As presented in figure 3.2, the hydrological data constitute of stream flow records, precipitation and climatological data, topographical maps, groundwater data, evaporation and transpiration data, soil maps and geologic maps. Large projects may need all the hydrological data. However, only the first three data are sufficient for the estimation of MIP monthly flows and Hydest floods in micro hydropower project.

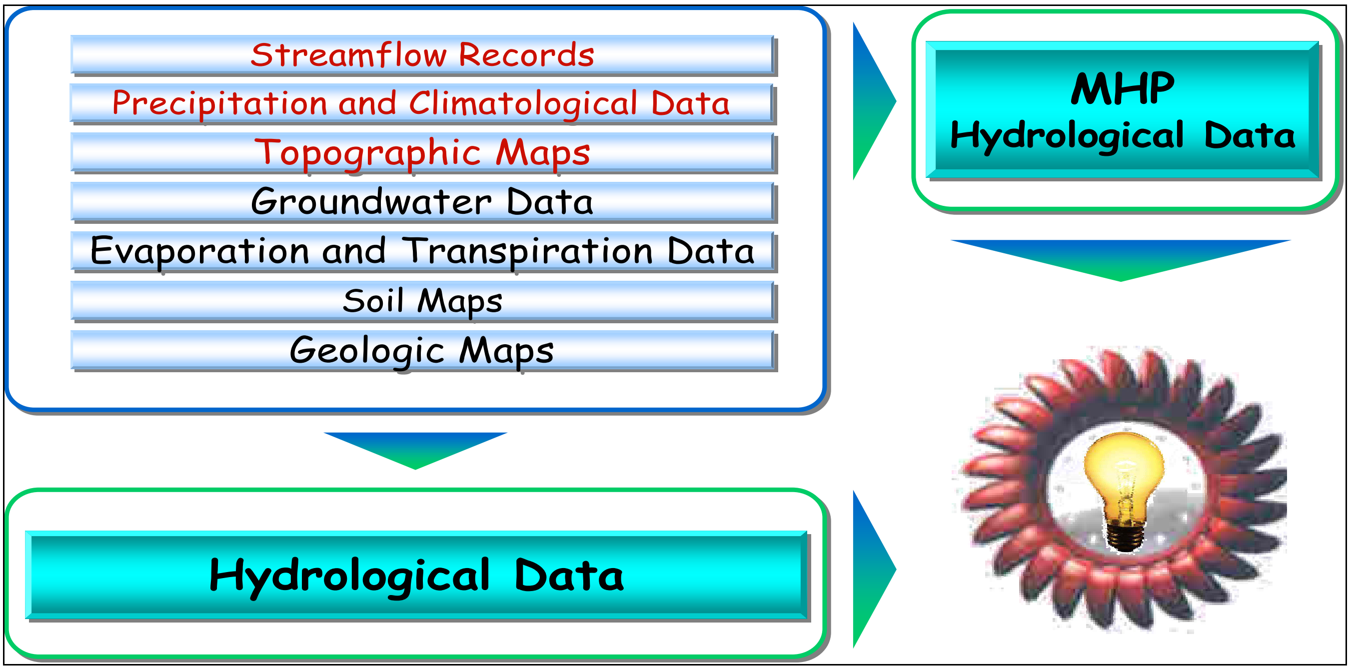


Figure 3.2: Hydrological Data and MHP

3.3 MEDIUM IRRIGATION PROJECT (MIP) METHOD

This method is developed by M. Mac Donald in 1990. According to this method, Nepal is divided into 7 regions. Based on wading measurements by the Department of Hydrology and Meteorology (DHM, HMG/N), non-dimensional regional hydrographs were developed for each region. The month of April was used for non-dimensionalizing. Seven sets of average monthly coefficients for the seven regions for each month were prepared.

The seven regions are graphically shown in Figure 3.3 and the seven sets of monthly coefficients are presented in Table 3.1. It is worth noting that these monthly coefficients have to be interpolated to get the actual monthly coefficients if the flow measurement date is not on the 15th of the month.

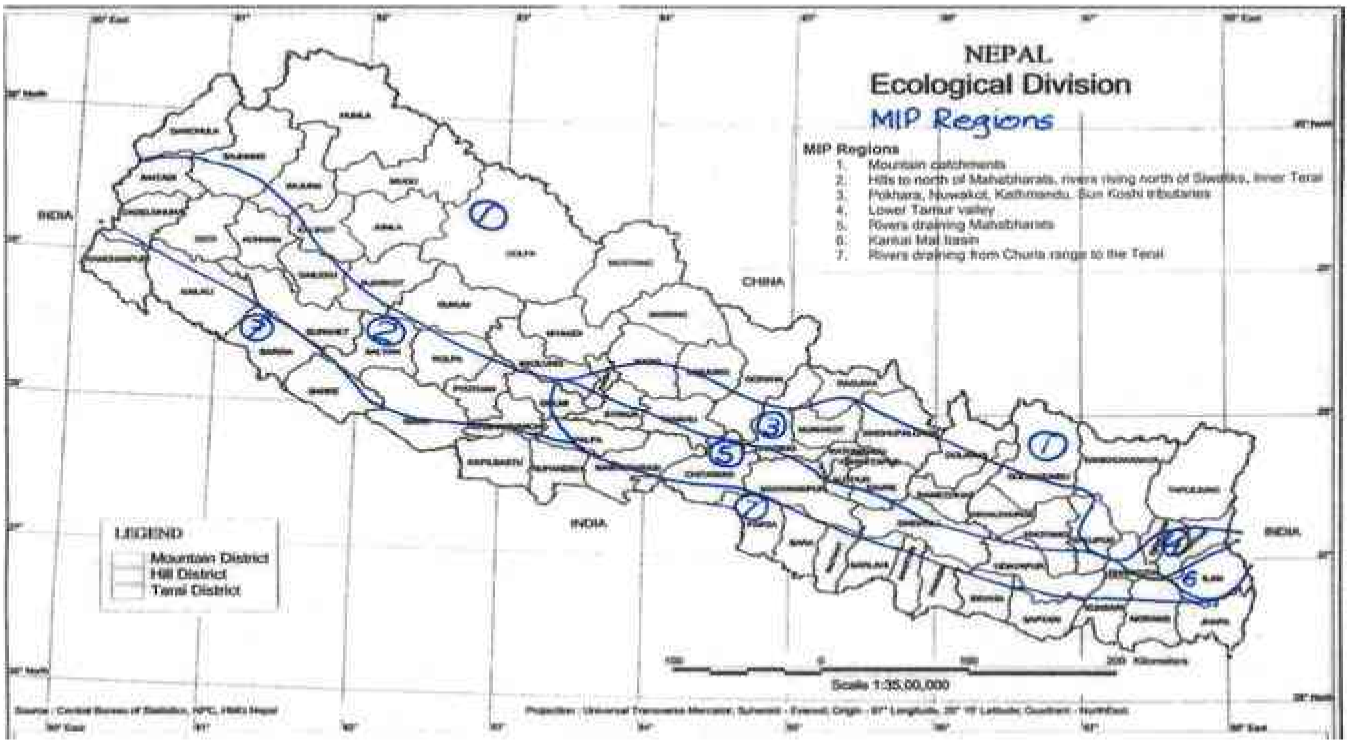


Figure 3.1: MIP Regions

Table 3.1: MIP regional monthly coefficients

Month	Regions						
	1	2	3	4	5	6	7
January	2.40	2.24	2.71	2.59	2.42	2.03	3.30
February	1.80	1.70	1.88	1.88	1.82	1.62	2.20
March	1.30	1.33	1.38	1.38	1.36	1.27	1.40
April	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
May	2.60	1.21	1.88	2.19	0.91	2.57	3.50
June	6.00	7.27	3.13	3.75	2.73	6.08	6.00
July	14.50	18.18	13.54	6.89	11.21	24.32	14.00
August	25.00	27.27	25.00	27.27	13.94	33.78	35.00
September	16.50	20.91	20.83	20.91	10.00	27.03	24.00
October	8.00	9.09	10.42	6.89	6.52	6.08	12.00
November	4.10	3.94	5.00	5.00	4.55	3.38	7.50
December	3.10	3.03	3.75	3.44	3.33	2.57	5.00

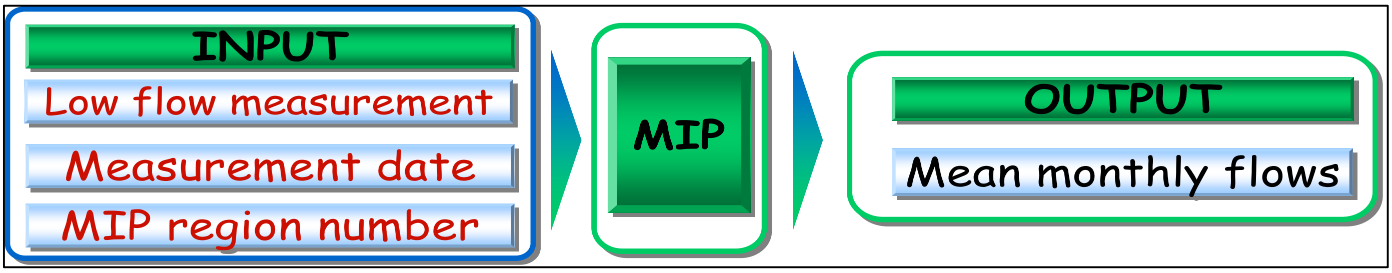
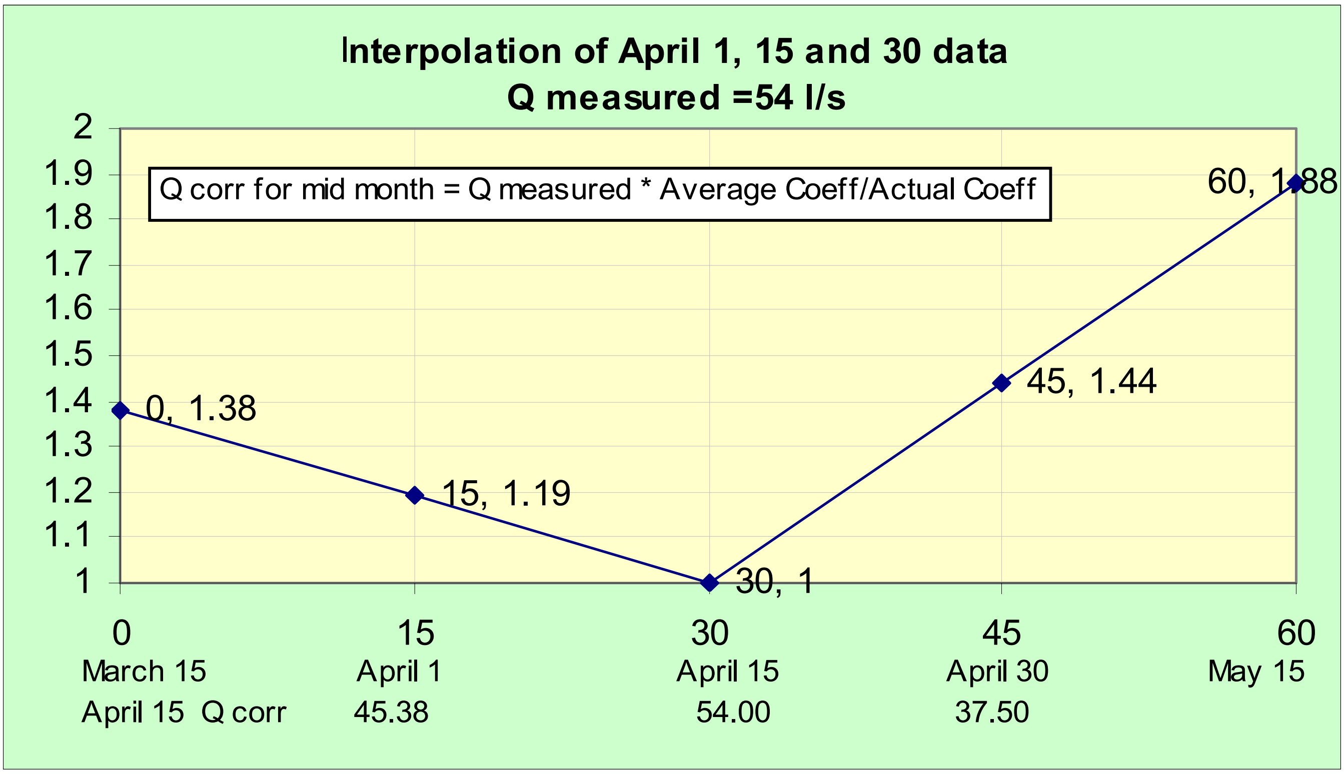


Figure 3.4: MIP model

Figure 3.4 represents the flow chart of the MIP model. As shown in the figure, this model takes low flow measurement, its date and MIP region number as inputs and process them for estimating mean monthly flows for that region. As stated earlier, the actual measurement date plays an important role in computing more realistic mean monthly flows. These mean monthly flows are calculated as:

Mean Coeff. for this month by interpolation if the date is not on 15th
April coef = 1/coeff this month
April flow = April coeff * Q
Monthly flows = April flow * coeffs ($Q_i = Q_{April} * C_i$)

Figure 3.5: Need of interpolation for calculating mean monthly coefficient



The importance of considering the actual date of measurement and the need of calculating actual mean monthly flow of that month are further explained in Figure 3.5. The measured flow is 54 l/s. The corrected flows for April are 45.38 l/s, 54 l/s and 37.5l /s if they are measured on April 1, 15 and 30 respectively. If interpolation was not carried out, a feasible project may fail to prove its viability according to the ACPC criteria or vice-a-versa.

This fact that the mean monthly coefficient calculation plays major role in AEPC acceptance criteria is illustrated by a following example.

Measured flow for MIP method m3/s:	1
MIP region (1 -7):	3

Area of basin below 3000m elevation A3000 km^2: 65
Turbine discharge m^3/s: 1.173
Water losses due to evaporation/flushing %: 15%

Figure 3.6 is the graphical representation of the outcome of the MIP method. The measurement date namely on April 1, 15 and 30 are considered and the errors generated by using different dates are presented. The design flow exceeds 11 months and fulfills the AEPC criteria if it is measured on either April 1st or April 15th. However, the design flow exceeds only 10 months and does not meet AEPC criteria if it is measured on either 1st or 30th of April.

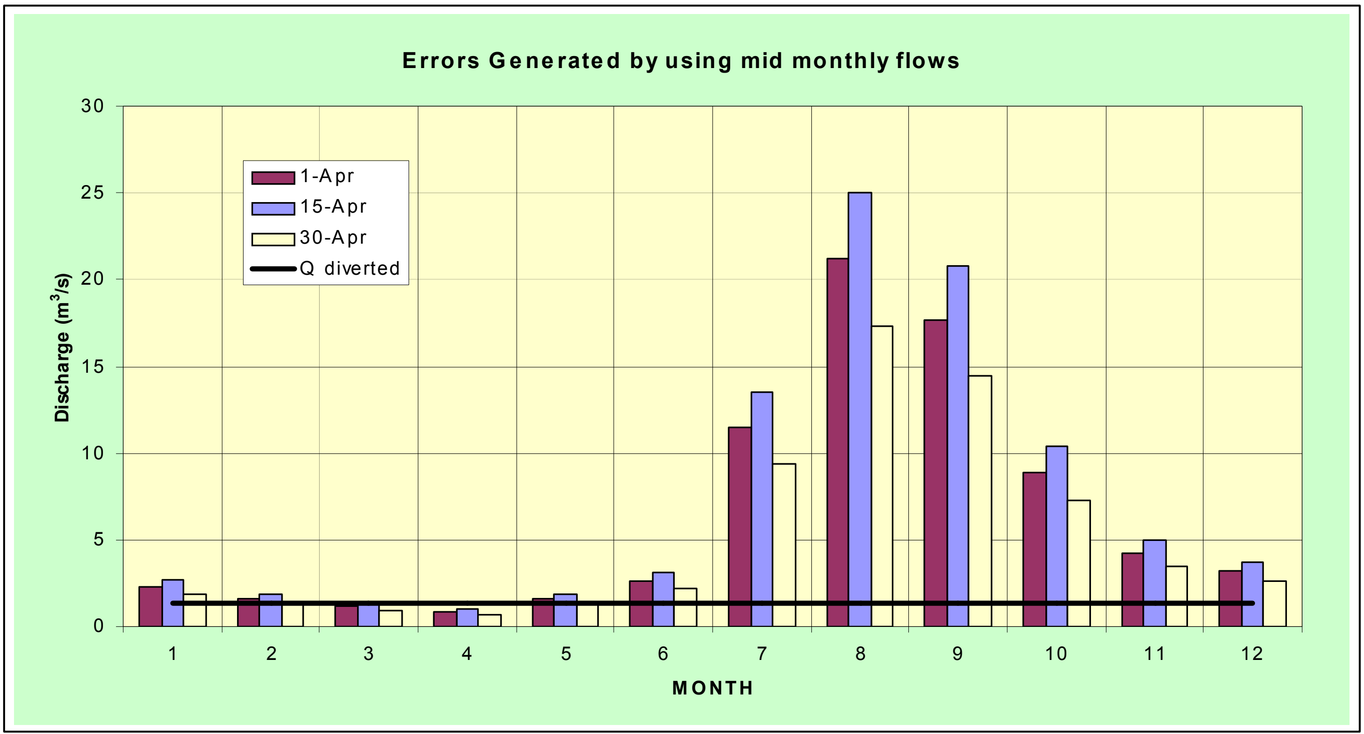


Figure 3.6: Effect of interpolation on mean monthly flows

3.4 WECS/DHM (HYDEST) METHOD

The WECS/DHM (Hydest) Method, which is also known as “Methodologies for estimating hydrologic characteristics of ungauged locations in Nepal” is developed by WECS/DHM in 1990. Long term flow records of DHM stations (33 for flood and 44 for low flow) were used to derive various hydrological parameters such as the monsoon wetness index (June-September mm precipitation). The entire country is considered as a single homogenous region. This method generally estimates reliable results if the basin area is more than 100 km^2 or if the project does not lie within Siwalik or Tarai regions.

Since the instantaneous and daily floods of 20-year return period are recommended to use while designing Nepali micro hydro structures, the flood calculations methods recommended by this method are used in the presented spreadsheet.

3.4.1 Flood Flows:

The catchment area below 3000 m contour line is used for the estimation of floods of various return periods. This elevation of 3000m is believed to be the upper elevation that is influenced by the monsoon precipitation. This method has to be used with caution for catchments that have a significant area above the snowline. The 2-year and 100-year flood can be calculated using following equations:

$$Q_{2\ daily} = 0.8154 \times (A\ below\ 3000\ m + 1)^{0.9527}$$

$Q_{2\ inst}$ = 1.8767 x (A below 3000 m+1)$^{0.8783}$

$Q_{100\ daily}$ =4.144 x (A below 3000 m +1)$^{0.8448}$

$Q_{100\ inst}$ = 14.630 x (A below 3000 m+1)$^{0.7343}$

Flood peak discharge, Q_F, for any other return periods can be calculated using:

$$Q_F = e^{\left(\ln Q_2 + S \cdot \sigma_{\ln QF}\right)}$$

Where, S is the standard normal variant for the chosen return period, from Table 3.2, and

$$\sigma_{\ln QF} = \frac{\ln\left(\frac{Q_{100}}{Q_2}\right)}{2.326}$$

Table 3.2: Standard normal variants for floods

Return period (T) (yrs)	Standard normal variant (S)
2	0
5	0.842
10	1.282
20	1.645
50	2.054
100	2.326

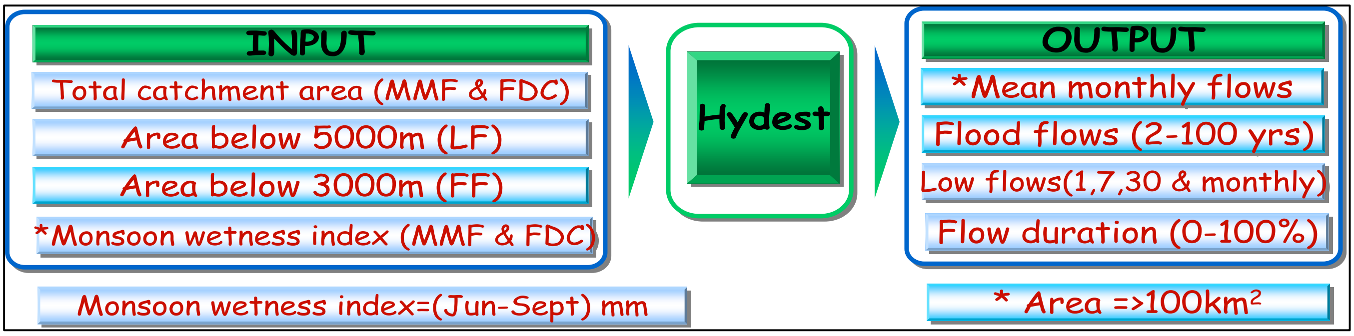


Figure 3.7: Hydest Model

As shown in Figure 3.7, the Hydest method requires different catchment areas and monsoon wetness index as inputs to estimate different hydrological parameters such as the mean monthly flows, floods, low flows and flow duration curve. The hydrological parameters computed by this model are comparable to the actual parameters for a total catchment area close to or more than 100km^2.

3.5 AEPC MGSP/ESAP GUIDELINES & STANDARDS

The guidelines and standards for subsidy approval for the construction of micro hydropower projects in Nepal can be summarised as:

1. Discharge measurement at the proposed intake site should be between November and May.
2. The recommended discharge measurement methods for different discharges are:
    1. Bucket collection < Weir >Salt dilution

2. 10lps < Q >30lps
3. Mean monthly flows should be computed by using MIP method. Alternatively, HYDEST method may be used for catchment area equal to or more than 100 km^2.
4. The design flow should be available at least 11 months in a year (i.e., the probability of exceedance should be 11 months or more). The design flow for sizing installed capacity (Q_d) should not be more than 85% of the 11-month exceedance flow. Loses and environmental releases should also be considered if it exceeds 15% of the 11-month exceedance.
5. There is a provision of ±10% tolerance on Q_d at the time of commissioning a plant.
6. Construction of flood wall against annual flood is recommended if the design flow exceeds 100 l/s.

3.6 PROGRAM BRIEFING & EXAMPLES

As per the standards and guidelines, the proposed spreadsheet is designed to compute MIP mean monthly flows and exceedance of the design flow, Hydest floods and design discharges for different components of a micro hydro schemes. For simplicity, the program considers 30 days a month for all the months. The flow chart for the proposed hydrological calculations are presented in Figure 3.8.

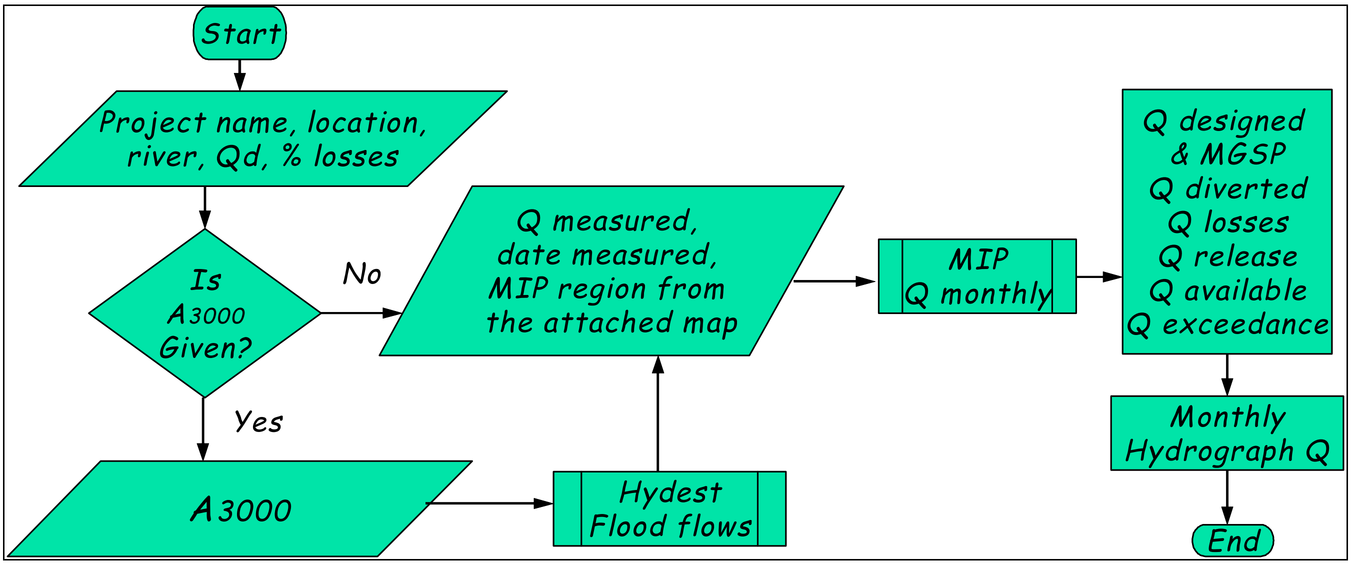


Figure 3.8: Flow chart of Hydrology spreadsheet

A typical example of the spreadsheet including inputs and outputs are presented in Figure 3.9. The information required for computations such as the MIP regions, etc., are included in the spreadsheet. The MIP region map shows that this project lies in MIP region 3. The measured discharge of 80 l/s in March 23 shows that the project is proposed to utilize a small stream. Although the floods are not critical to the project, they are calculated for sizing the floodwall and other structures. Although the estimated design discharge is only 80 l/s, the probability of exceedance for this design discharge is only 10 months and hence does not qualify AEPC acceptance criteria. For AEPC to qualify this project, the turbine design discharge should not exceed 73.389 l/s. The procedures for the calculations are:

MIP mean monthly calculations:

Corrected coefficient and mid month discharges ($Kc_{December}$) for Region 3:
Since the measured date of March 23 lies in between March 15th and April 15th,

K_{March} = 1.38
K_{April} = 1.00

Kc_{March} = K_{March} + (K_{April} - K_{March})*(Date -15)/30
= 1.38 + (1.00-1.38)*(23-15)/30

= 1.2787

Q_{March} = $Q_{measured}$ *K_{March} /Kc_{April}
= 80 *1.38/1.2787
= 86.34 l/s

Q_{April} = Q_{March} /K_{March}
= 86.34/1.38
= 62.57 l/s

Q_{May} = Q_{April} *K_{May}
= 62.57 * 1.88
= 117.62 l/s

Other mid monthly discharges are calculated similar to the discharge calculation for the month of May.

Hydest flood flows:

The 2-year and 100-year floods are:

$Q_{2\ daily}$ = 0.8154 x (A below 3000 m +1) $^{0.9527}$
= 0.8154 * (1.5+1) $^{0.9527}$
= 1.952 m^3/s

$Q_{2\ inst}$ = 1.8767 x (A below 3000 m)$^{0.8783}$
= 1.8767 x (1.5+1)$^{0.8783}$
= 4.197 m^3/s

$Q_{100\ daily}$ =4.144 x (A below 3000 m +1)$^{0.8448}$
=4.144 x (1.5 +1)$^{0.8448}$
= 8.987 m^3/s

$Q_{100\ inst}$ = 14.630 x (A below 3000 m+1)$^{0.7343}$
= 14.630 x (1.5+1)$^{0.7343}$
= 28.669 m^3/s

Peak discharges for other return periods are calculated by using these formulas:

$$\sigma_{lnQF} = \frac{\ln\left(\frac{Q_{100}}{Q_2}\right)}{2.326} \qquad Q_F = e^{\left(\ln Q_2 + S \cdot \sigma_{lnQF}\right)}$$

$Q_{20\ daily}$ = EXP(LN(1.952)+ 1.645*(LN(8.987/1.952)/2.326))
= 5.747 m^3/s

$Q_{20\ inst}$ = EXP(LN(4.197)+ 1.645*(LN(28.669/4.197)/2.326))
= 16.334 m^3/s

Different discharge calculations (as per AEPC criteria are presented for examples):

$Q_{turbine}$ = 85% of the 11 month flow exceedance from the MIP flow if the designed flow is higher or the design flow.
= 73.389 l/s (since the design flow is higher and 10 month exceedance only)

$Q_{diverted}$ = $Q_{turbine}$ / (1-%losses)
= 73.389 / (0.95)

$= 77.252$ l/s

$Q_{losses} = Q_{diverted} - Q_{turbine}$
$= 77.252 - 73.389$
$= 3.863$ l/s

$Q_{release} = Q_{min\ MIP}$ *%release
$= 62.57 * 0.05$
$= 3.128$ l/s

$Q_{required\ at\ river} = Q_{diverted} + Q_{release}$
$= 77.252 + 3.128$
$= 80.380$ l/s

A hydrograph including the design flow, exceedance of the proposed design flow and the flow acceptable for AEPC is presented in Figure 3.9.

HYDROLOGICAL CALCULATIONS FOR UNGAUGED MHP RIVERS

Project: ***Chhyota MHP***

INPUT

River name :	Chhyota Khola
Location :	Borang, Sertung VCD-2, Dhading
Measured flow for MIP method l/s:	80
Month and day of flow measurement:	March 23
MIP region (1 -7) :	3
Area of basin below 3000m elevation A_{3000} km^2 :	1.5
Turbine discharge Qd l/s:	80
Water losses due to evaporation/flushing/seepage % :	5%
Downstream water release due to environmental reasons % :	5%

OUTPUT

MIP monthly average discharge

Month	Discharge (l/s)
January	169.55
February	117.62
March	86.34
April	62.57
May	117.62
June	195.83
July	847.13
August	1564.13
September	1303.23
October	651.93
November	312.83
December	234.62
Annual av	**471.950**

Hydest Flood Flows

Return Period (yrs)	Flood Discharge (m^3/s)	
	Daily	Instantaneous
2	1.952	4.197
20	5.747	16.334
100	8.987	28.669

Discharge (l/s)	Designed	As per MGSP
Qturbine	80.000	73.389
Q diverted	84.211	77.252
Q losses	4.211	3.863
Q release	3.128	3.128
Q required	87.339	80.380
Q exceedence (month)	10	11
Q turbine for 11m	77.922	

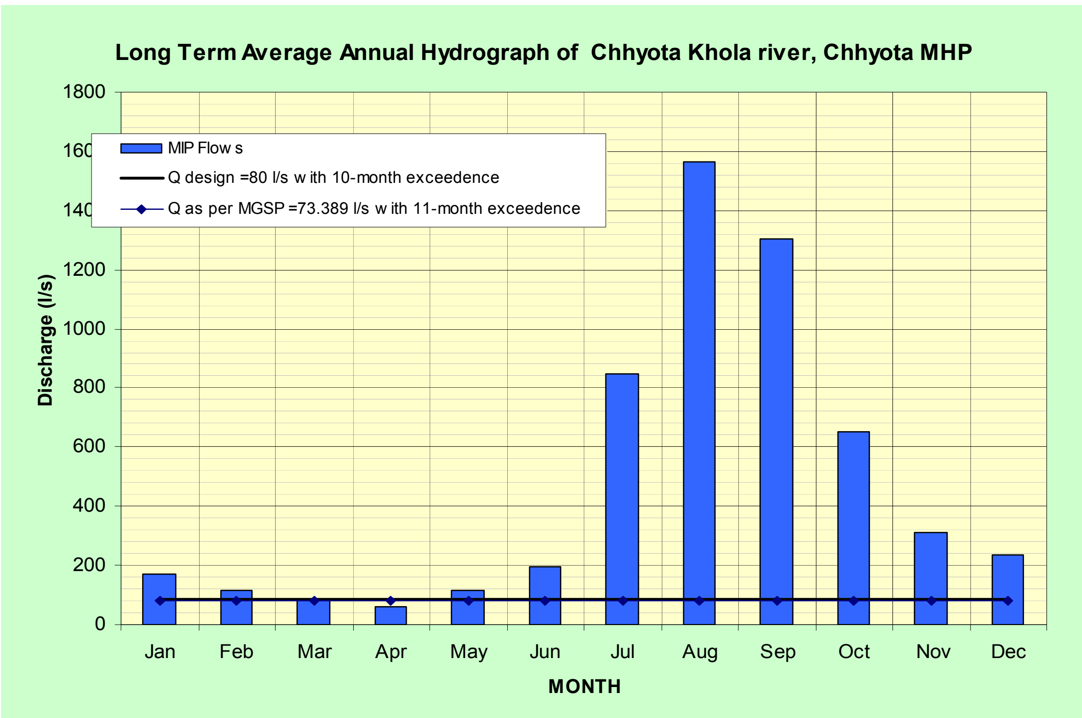


Figure 3.9: Typical example of a hydrological parameters calculation spreadsheet "Hydrology"

4 HEADWORKS

4.1 INTRODUCTION AND DEFINITIONS

Headworks
A headworks consists of all structural components required for safe withdrawal of desired water from the river into a canal/conduit. Intake, weir, protection works, etc., are the main structural components. Indicators of an ideal headworks can be summarized as:

1. Withdrawal of the desired flow (i.e., $Q_{diverted}$ and spilling in case of flood).
2. Sediment bypass of diversion structure (Continued sediment transportation along the river).
3. Debris bypass (Continued debris bypass without any accumulation).
4. Hazard flood bypass with minimum detrimental effects.
5. Sediment control at the intake by blocking/reducing sediment intake into the system).
6. Settling basin control (settling and flushing of finer sediments entered into the system through intakes or open canals).

Intake
An intake can be defined as a structure that diverts water from river or other water course to a conveyance system downstream of the intake. Side intake and bottom intake are the common types of river intakes that are used in Nepali micro hydropower schemes.

Conveyance Intake is an intake which supplies water to a conveyance other than the pressure conduit to the turbine. Power Intake is an intake which supplies water to the pressure conduit to the turbine.

Side Intake
A structure built along a river bank and in front of a canal / conduit end for diverting the required water safely. Side intakes are simple, less expensive, easy to build and maintain.

Bottom/Drop/Tyrolean/Trench Intake
A structure built across and beneath a river for capturing water from the bed of the river and drops it directly in to headrace. These are mainly useful for areas having less sediment movement, steeper gradient, and surplus flow for continual flushing. Inaccessibility of trashrack throughout the monsoon season and exposure of the system to all the bed load even though only a small part of the water is drawn are the common problems/drawbacks of drop intakes.

Weir
A weir is a structure built across a river to raise the river water and store it for diverting a required flow towards the intake.

Protection Works
Protection works are the river protection and river training works to safeguard the headworks against floods, debris and sediments.

Trashrack
A structure placed at an intake mouth to prevent floating logs and boulders entering into headrace. Coarse trashrack and fine trashrack are provided at the river intake and penstock intake respectively.

4.2 AEPC MGSP/ESAP GUIDELINES AND STANDARDS

The requirements set for weir, intake and trashrack are briefly outlines in this section.

4.1.1 Weir

- Type: A weir can be either temporary or permanent in nature. A dry stone or gabion or mud stone masonry can be termed as temporary weir whereas a cement masonry or concrete weir can be termed as a permanent weir.
- Location: It is recommended that the weir should be 5m to 20m d/s of intake. This will assure that water is always available and there is no sediment deposit in front of the intake. A narrow river width with boulders is preferable for weir location.
- Height: The weir should be sufficiently high to create enough submergence and driving head.
- Stability: Permanent weir should be stable against sinking, overturning and sliding even during the designed floods.

4.1.2 Intake

- Type: Side intake is suitable for all types of river categories whereas the drop intake is recommended for rivers having longitudinal slopes more than 10%, less sediment and excess flushing discharge. The side intake is generally is of rectangular orifice type with a minimum of 50mm submergence. The side intake should be at:
    - Straight river u/s & d/s of the intake.
    - Alternatively, on the outer side of the bend to minimize sediment problems and maximise the assured supply of desired water.
    - Relatively permanent river course.
    - By the side of rock outcrops or large boulders for stability and strength.
- Capacity: According to the flushing requirement and tentative losses the intake has to be oversized than the design flow by about 10% to 20% (or $Q_{diverted}$).
- A course trashrack should be provided to prevent big boulders and floating logs from entering into the headrace canal.
- A gate/stop log should be provided to regulate flow (adjust/ close) during operation and maintenance.
- To optimize the downstream canal and other structures, a spillway should be provided close to the intake.

4.1.3 Intake Trashrack

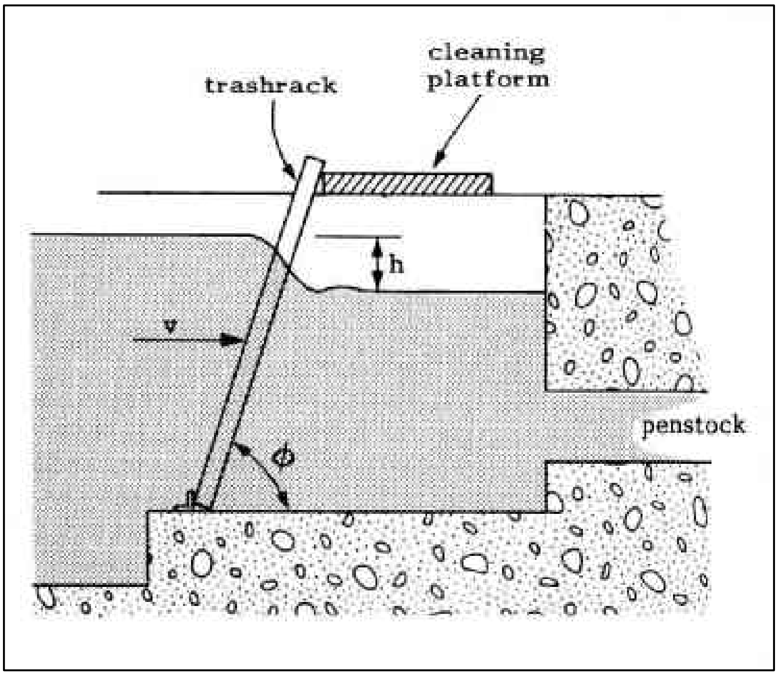
The recommended intake coarse trashrack is made of vertical mild steel strips of 5*40 to 5*75. The clear spacing between these strips should not exceed 75mm. The approach velocity should be less than 1.0m/s. For transportation, the weight of a piece of trashrack should not exceed 60 kg. With respect to the combined effect of racking and hydraulic purposes, placing of trashrack at 3V:1H is considered to be the most optimum option.

4.3 PROGRAM BRIEFINGS AND EXAMPLES

There are two spreadsheets for designing side intake and bottom intake respectively. The first part of the side intake calculates the trashrack parameter while the second part of it calculats side intake parameters including spillways for load rejection and flood discharge off-take. The second spreadsheet calculated the design parameters for a drop intake.

Since most of the program flow chart in this section is self explanatory or the calculation procedures are familiar to most of the practicing stakeholders, only important points are explained.

Figures 4.1 to 4.7 presents the assumptions, flow charts and typical examples for calculating trashrack parameters, side intake and drop intake dimensioning.



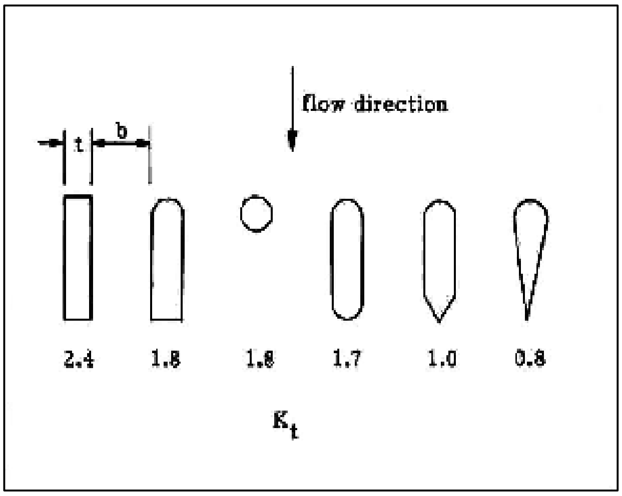


Figure 4.1: Trashrack parameters

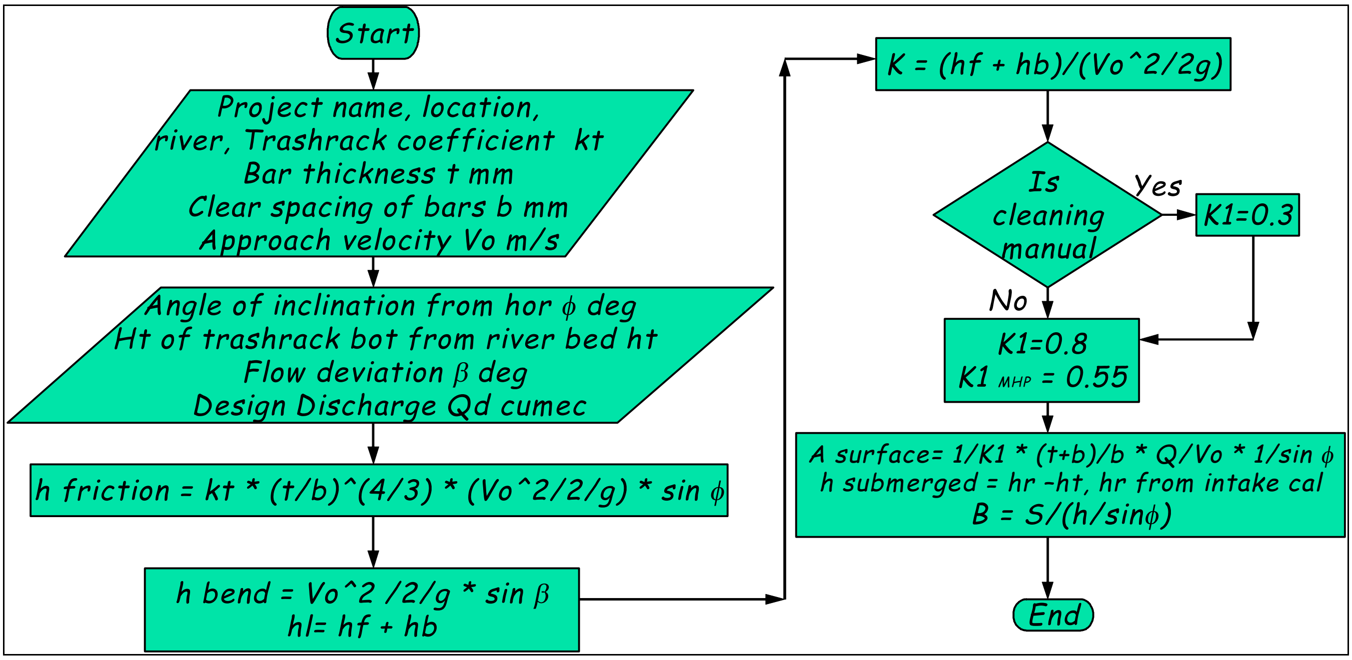


Figure 4.2: Flow chart for trashrack calculations

The trashrack coefficients for different cross section of the bars are presented in the pull down menu. Typical bar thickness, clear spacing and approach velocity are suggested in the respective cell notes.

It can be seen from the flow chart that the trashrack losses constitute of frictional and bend losses. The frictional losses depend on the geometry of trashrack such as the trashrack coefficient, thickness and clear spacing of bars, inclination of the trashrack and the approach velocity. The bend loss depends on the hydraulics of the approaching flow such as the approach velocity and its deviated direction with respect to the normal of the trashrack surface.

The trashrack surface area coefficient K1 for automatic racking is 0.8 whereas it is 0.3 for manual racking suggesting that the racking area for manual operation to recommended surface area is 3.33 times more than the theoretical area. Manual racking is recommended for Nepali MHPs. Since the consequence of temporary reduced trashrack area in micro hydro is not severe and the trashrack sites are generally accessible to operators all the year, the average of automatic and manual racking coefficient is taken for practical and economic reason.

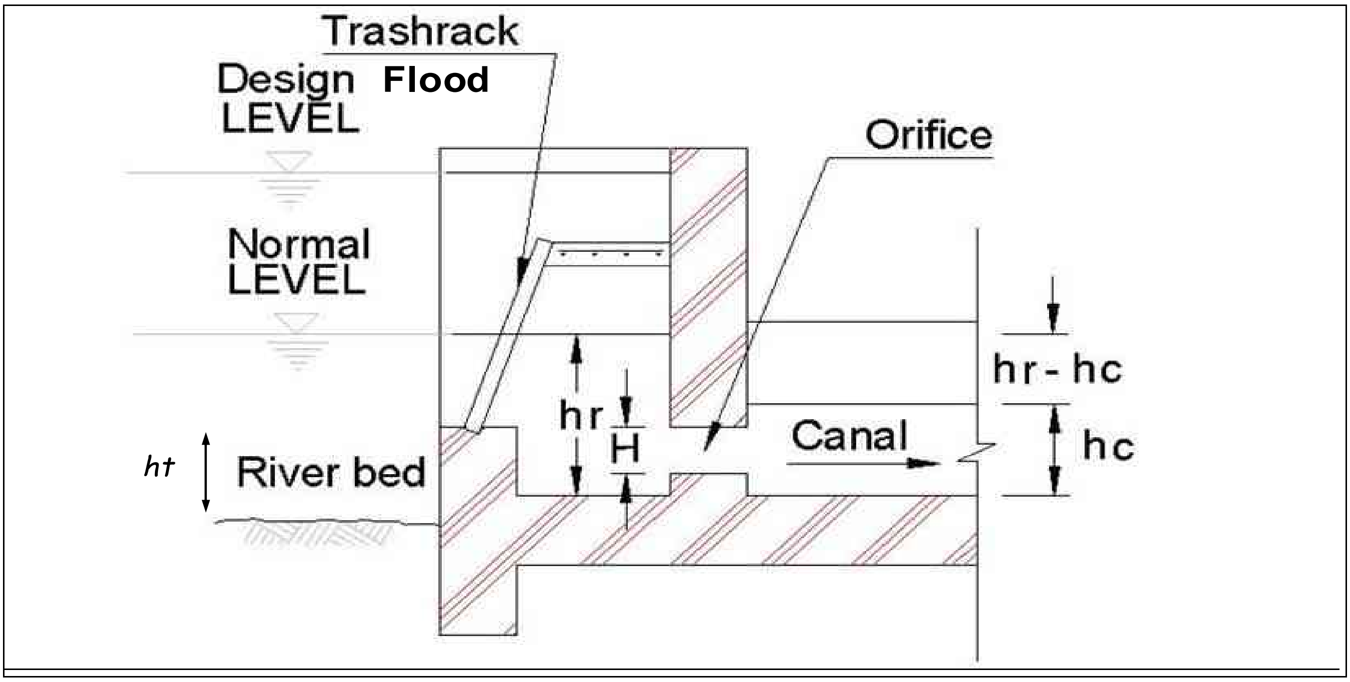


Figure 4.3: Side intake parameters

A typical side intake parameters considered in the spreadsheet are presented in Figure 4.3. The design of side intake parameters are presented in the flow chart in Figure 4.4. An example is presented in Figure 4.6. The design processes for the calculations of a typical side intake are also presented.

Trashrack Design:

Head losses,

h friction $= kt * (t/b)^{(4/3)} * (Vo^2/2g) * \sin \phi$
$= 2.4*(4/25)^{(4/3)} * (0.5^2/2/9.81) * \sin 60^o$
$= 0.0023m$

h bend $= (Vo^2/2g) * \sin \beta$
$= (0.5^2/2/9.81) * \sin 20^o$
$= 0.0044m$

h total = h friction + h bend
$= 0.00232 + 0.0044$
$= 0.0067m$

A surface S $= 1/k1*(t+b)/b * Q/Vo * 1/\sin \phi$
$= 1/0.55*(4+25)/25*0.077*1/\sin 60^o$
$= 0.3763\ m^2$

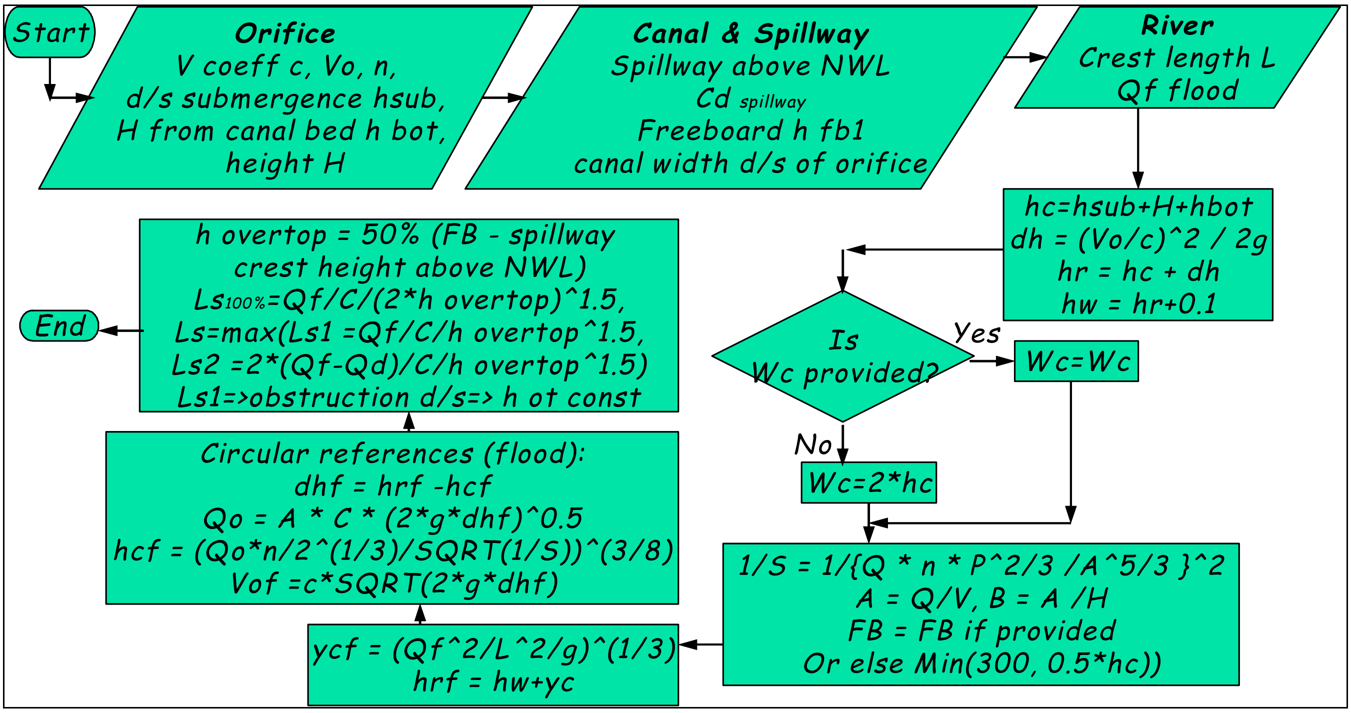


Figure 4.4: Flow chart for side intake calculations

Width B = S/(h/ sin ϕ)
= 0.3763/(.3/ sin 60^{o})
= 1.09 m

Side Intake calculations:

Normal condition:

Depth @ canal (hc) = h submergence + height of orifice + height of orifice sill from bottom of the canal
= 0.05+0.2+0.2
= 0.45m

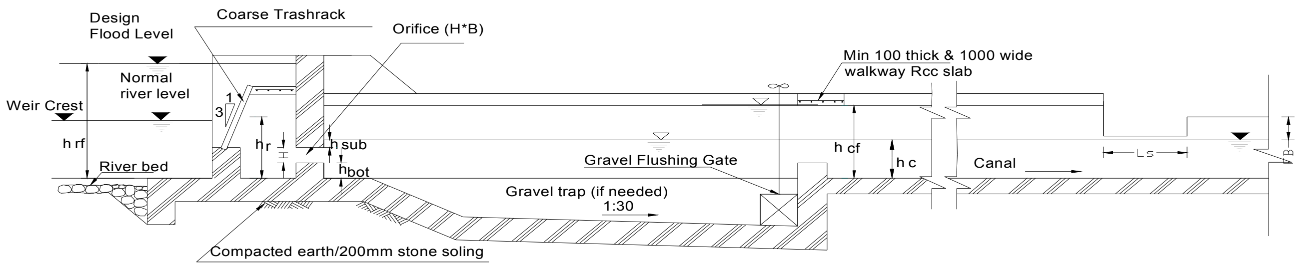
Driving head (dh) = $(Vo/c)^2/2/g$
= $(1.2/0.8)^2/2/9.81$
= 0.115 m

Head at river (hr) = hc+dh (this value can be provided)
= 0.45+0.115
= 0.565 m

Height of weir (hw) = hr +0.1
= 0.565+0.1
= 0.665 m

Side Intake with orifice Design

Chhota MHP



Trashrack calculations

Input		Output	
Trashrack coeffieient kt	2.4		
Bar thickness t mm	4.00	Headloss due to friction hf m	0.0023
Clear spacing of bars b mm	25.00	Headloss due to bends hb m	0.0044
Approach velocity Vo m/s	0.50	Headloss coeff K	0.5226
Angle of inclination from horizontal ϕ deg	60.00	Total headloss ht m	0.0067
Flow deviation β deg	20.00	Surface area A surface m^2	0.3750
Design Discharge Qd cumec	0.077	Vertical height h m	0.3647
Height of trashrack bottom from river bed ht	0.20	Trashrack width B m	0.89

Orifice Calculations for (B = 2H or provided) rectangular canal downstream of orifice

Input

Orifice		River	
Velocity coeff of orifice c	0.8	Crest length L m	5.000
Velocity through orifice Vo m/s	1.2	Provided Q flood m^3/s	10.000
Manning's coeff of roughness	0.02	Q flood m^3/s (Q_{20} for MHP with Qd>100)	16.334
Downstream submergence depth hsub m	0.050	Used Q flood	10.000
Orifice height H m	0.200	**Canal & Spillway**	
Height of orifice from canal bed h bot m	0.200	Spillway crest height above NWL m	0.050
Provided water depth in the river hr (m)	0.000	Spillway discharge coeff	1.6
Provided canal width (m)	0.500	Provided Freeboard h fb1 m	0.300

Output

Normal Condition		Flood	
Canal witdth d/s of orifice	0.500	Critical depth of water at crest yc m	0.742
1/Slope of canal immediately d/s of orifice	1865	Flood head at river hf r = hw+yc m	1.406
Depth of water in canal hc m	0.450	Head difference dhf	0.916
Free board in canal h fb m	0.300	Velocity through orifice Vof m/s	2.544
Area of orifice A m2	0.064	Q intake Qf cumec	0.218
Width of orifice B m	0.321	Depth of water at canal (hc f) m	0.490
Actual velocity through orifice Vo act m/s	1.200		
Canal width Wc m	0.500	**Spillway**	
Water level difference dh m	0.115	Ls for Qf m (d/s Obs & 100% hot -50)	1.521
Water depth in the river hr = hc + dh m	0.565	Length of spillway Ls1 for Qf m (d/s Obs)	3.078
Height of weir (hw = hr+0.1) m	0.665	Length of spillway Ls2 for Qf-Qd m	3.978
Spillway overtopping height h overtop m	0.125	Designed spillway length Ls m	3.978

Figure 4.5: An example of side intake calculations

Orifice area (A) = Q/V
= 0.77/1.2
= 0.064 m^2

Orifice width (B) = A/H
= 0.064/.2
= 0.322 m

Flood:

Critical depth at crest (yc)= $(Qf^2/L^2/g)^{1/3}$
= $(10^2/5^2/9.81)^{1/3}$
= 0.742 m

Head at river (hf r) = hw+yc
= 0.665+0.742
= 1.407 m

Water depth at canal during flood (by equating and iterating flow coming from orifice to that of canal flow) (hcf) = 0.490m

Q intake (Qf) = 0.218 m^3/s

Spillway overtopping height (h overtop) = 50%(Free board –h nwl)
= 0.5*(.3-.05)
= 0.125 m

Spillway length 100% (Ls for Qf) = $Qf/C/(2*h\ ot)^{1.5}$
= $0.218/1.6/(2*0.125)^{1.5}$
= 1.525 m

Spillway length 50% = $Qf/C/(h\ ot)^{1.5}$
= $0.218/1.6/(0.125)^{1.5}$
= 3.078 m

The cell containing the depth of water at canal hcf (m) sometimes can generate VALUE# error. Should such an error occur, select the cell, press F2 key and Enter respectively.

Care should be taken while designing spillway lengths. Ls for Gfm (d/s Obs & 100% hot -50) is only applicable when full downstream obstruction for flood off-take is provided with the help of stop logs or gates. Otherwise, the gradually varying water profile at the spillway has to be considered.

Drop Intake calculations:

The example presented in Figure 4.7 follows the procedures presented in Figure 4.6. Although the calculation procedures for the drop intake are relatively straight forward and simple, it has more restrictions and limitations regarding the stream geometry and operational conditions.

Based on the flow conditions and the slope of rack the flow immediately upstream of the rack may be either critical or sub-critical. Critical depth at the entrance of the rack has to be considered if the rack is steeper (more than 15^o). For more details, please refer to EWI UNIDO Standard.

The main differences for considering critical flow and normal flow condition are presented in the Table 4.1. In the presented spreadsheet, the critical depth of the flow upstream of the intake is calculated and presented if normal flow (sub-critical) is considered.

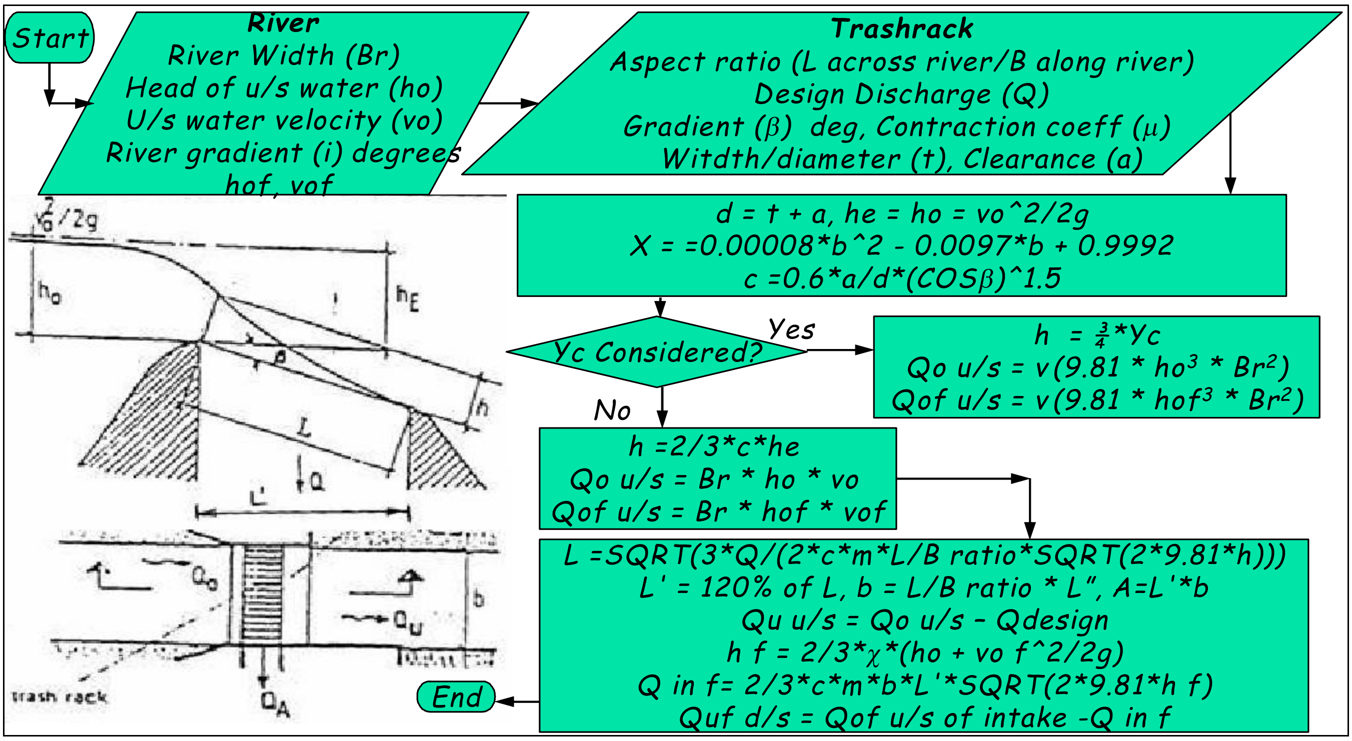


Figure 4.6: Parameters and flow chart of drop intake design

Table 4.1: Drop intake and upstream flow

Parameters	Normal flow	Critical Depth considered
Velocity head (h)	= 2/3 * χ * he	= ¾ * Yc
Qo u/s of intake (m^3/s) normal	= Br * Ho * Vo	= SQRT(9.81*ho ^{3}*Br ^{2})
Qo u/s of intake (m^3/s) flood	= Br * Ho f * Vo	= SQRT(9.81*ho f ^{3}*Br ^{2})

The calculations presented in Figure 4.7 are varified in the following section. In this example the flow upstream of the intake is considered to be of critical.

Normal condition:

c/c distance of trashrack bars d (mm)
= t + a
= 60+30
= 90mm

Kappa (χ) = 0.00008*β^2 - 0.0097*β + 0.9992(by curve fitting)
= 0.00008*36^2 - 0.0097*36 + 0.9992
= 0.749

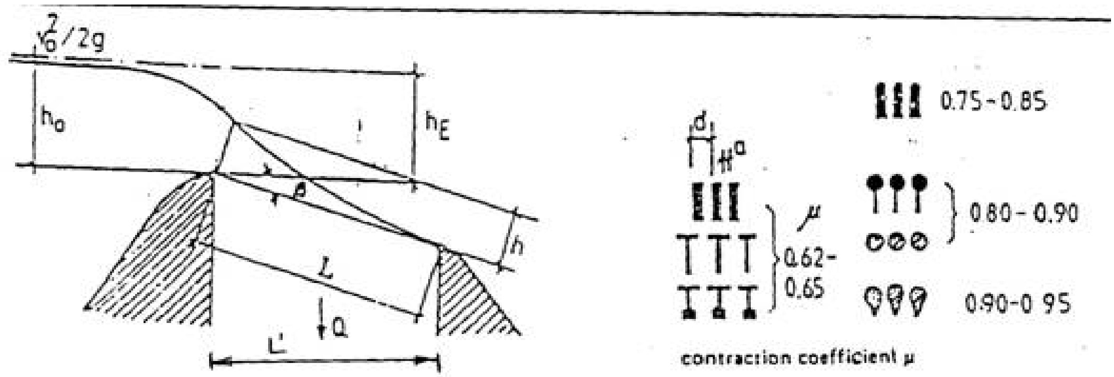
Velocity head (h) m = ¾ * of Yc
= ¾ * 0.226
= 0.170 m

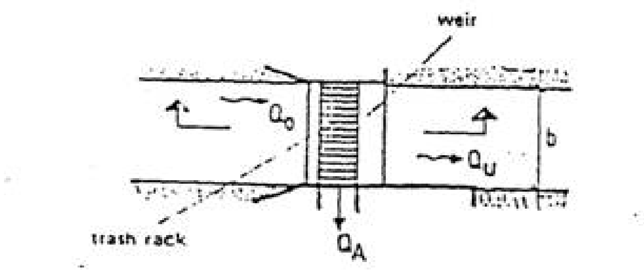
Correction factor (c) = 0.6*a/d*(COSβ)^1.5
= 0.6*30/90*(Cos 36)^1.5
= 0.146

Length of Intake (L) m = SQRT(3*Q/(2*c*μ*L/B ratio*SQRT(2*9.81*h)))
= SQRT(3*2.7/(2*c*0.85*3.546468*SQRT(2*9.81*.170)))
= 2.249 m

Bottom/Drop Intake

Gothi MHP





Input		Critical Depth Considered	1
		River Width flood (Brf) m =	20
River Width (Br) m =	8	ho flood m =	3.000
Head/Critical Depth of u/s water (ho)m =	0.226	vo flood m/s =	4
Upstream water velocity (vo) m/s =	1.494	Design Discharge (Qd), m^3/s =	2.7
River gradient (i) degrees =	9.462	Trashrack witdth/diameter (t) mm =	60
Trashrack gradient (β) deg =	36	Trashrack clearance (a) mm =	30
Contraction coeff (μ) =	0.85		
Aspect ratio (Length across the river/Breadth along the river) =			3.546468
Output			
c/c distance of trash rack bars d mm =	90	Area of intake (A=L' *b) m^2 =	21.522
Total head (he) m =	0.340	Qo u/s of intake (m^3/s) normal =	2.700
kappa (χ) =	0.749	Qu d/s of intake (m^3/s) normal =	0.000
velocity head (h) m =	0.170	h flood =	1.906
Correction factor (c) =	0.146	Qof u/s of intake = Br * hof * vof (m^3/s) =	325.497
Length of intake (L) m =	2.249	Q in flood m^3/s =	10.855
Factored length (L' = 120% of L) m =	2.699	Quf d/s of intake (m^3/s) =	314.641
Intake length across the river (b) m =	7.975		

Figure 4.7: An example of drop intake

Factored length (L') m = 120% of L
= 1.2*2.699
= 2.699 m

Intake length across the river (b) m
= L/B ration * L
= 3.546468*2.249
= 7.975m

Area of intake (A) m^2 = L' * b
= 2.699 * 7.975
= 21.522 m^2

Qo u/s of intake (m^3/s) = SQRT(9.81*ho ^{3}*Br ^{2})
= SQRT(9.81*.226 ^{3}*8 ^{2})
= 2.7 m^3/s

Qu d/s of intake (m^3/s) = Qo u/s -Qd
= 2.7-2.7
= 0 m^3/s

Intake length across the river (b) m
= L/B ration * L
= 3.546468*2.249
= 7.975m

Flood:

h flood (hf) m = 2/3*χ*(ho flood+vo flood^2/2g)
= 2/3*0.749*(3+4^2/2/9.81)
= 1.906 m

Qo u/s of intake (m^3/s) = SQRT(9.81*hof ^{3}*Br ^{2})
= SQRT(9.81*1.906^3*20 ^{2})
= 325.497 m^3/s

Qo in (off-take) (m^3/s) = 2/3*c*μ*b*L' *SQRT(2*9.81*h flood)
= 2/3*0.146*0.85*7.975*2.699*SQRT(2*9.81*1.906)
= 10.855 m^3/s (this discharge can be reduced by introducing a throttling pipe d/s of the intake)

Qu d/s of intake (m^3/s) = Qo u/s -Qd
= 315.497-10.855
= 314.641 m^3/s

5 HEADRACE/TAILRACE

5.1 INTRODUCTION AND DEFINITIONS

A headrace or tailrace can be defined as a conveyance system that conveys designed discharge from one point (e.g. intake) to another (e.g. forebay). Generally a canal is used in all cases whereas a pipe is used for specific e.g. difficult terrain. A canal can be unlined (earthen) or lined (stone masonry or concrete). The typical canal cross sections used in micro hydropower schemes can be rectangular or trapezoidal or triangular or semi-circular in shape. The pipes used in MHP can be of HDPE or mild steel and it can be either open or buried.

For computing head losses, Manning's equation is used for canal whereas Darcy-Weisbach equation is used for pipe.

5.2 MGSP/ESAP GUIDELINES AND STANDARDS

5.2.1 Canal

a) Capacity: The canal should be able to carry the design flow with adequate freeboard, escapes to discharge excess flow. A canal should generally be able to carry 110 to 120 % the design discharge.

b) Velocity: Self cleaning but non erosive (≥ 0.3m/s).

c) Unlined canal: In stable ground and Q ≤ 30 l/s

d) Lined canal: 1:4 Stone masonry / Concrete (short: crossings or unstable ground). It is recommended to minimize seepage loss and hence minimize the subsequent landslides.

e) Sufficient spillways and escapes as required.

f) Freeboard: Minimum of 300mm or half of water depth.

g) Stability and Safety against rock fall, landslide & storm runoff.

h) Optimum Canal Geometry: Rectangular or trapezoidal section for lined canal and trapezoidal section for unlined canal are recommended.

5.2.2 Pipe

a) PVC/HDPE: Buried at least 1m into ground.

b) Steel/CI: As pipe bridge at short crossings/landslides.

c) Pipe inlet with trashracks for a reach of more than 50m and $1.5*v^2/2g$ submergence.

d) Provision of air valves and wash outs where necessary.

5.3 PROGRAM BRIEFING AND EXAMPLES

5.3.1 Canal

a) Permissible erosion free velocities for different soil conditions:
Fine sand =0.3-0.4
Sandy loam =0.4-0.6
Clayey loam=0.6-0.8
Clay =0.8-2.0
Stone masonry =0.8-2.0

Concrete = 1.0-3.0

b) Sectional profiles:
Semicircular
Rectangular
Triangular
Trapezoidal

c) Two parts of calculations for canal
Evaluation of the design based on the user specified parameters.
Optimum canal parameters computations based on the MHP Sourcebook by Allen R Iversin.

d) Two spreadsheets are included in the Design Aids for:
   a. Canal calculations (Figure 5.1 for flow chart and Figure 5.3 for an illustrative example)
   b. Pipe calculations (Figure 5.4 for flow chart and Figure 5.5 for an illustrative example).

The calculations presented in Figure 5.4 are briefly described in the following section. The rectangular intake canal is considered for the illustration.

Present Canal:

Area A m^2	= D*B = 0.3*0.5 = 0.15 m^2
Top Width T (m)	= B+2*H*N = 0.5+2*0.3*0 = 0.5m
Wetted Perimeter (m)	= 2*D+B = 2*0.3+.5 = 1.1m
Hydraulic Radius r (m)	= A/P = 0.15/1.1 = 0.136 m
Calculated flow (m^3/s)	= $A*r^{2/3}*S^{0.5}/n$ = $0.15*0.136^{2/3}*0.01299^{0.5}/n$ = 0.226 m^3/s
Critical Velocity Vc m/s	= sqrt(A*g/T) = sqrt(0.15*9.81/.5) = 1.72 m/s
Velocity V m/s	= Q/A = 0.185/0.15 = 1.233 m (Okay since it is less than 80% of Vc)
Headloss hl (m)	= S*L + di (drops) = 0.01299*20+0 = 0.260m

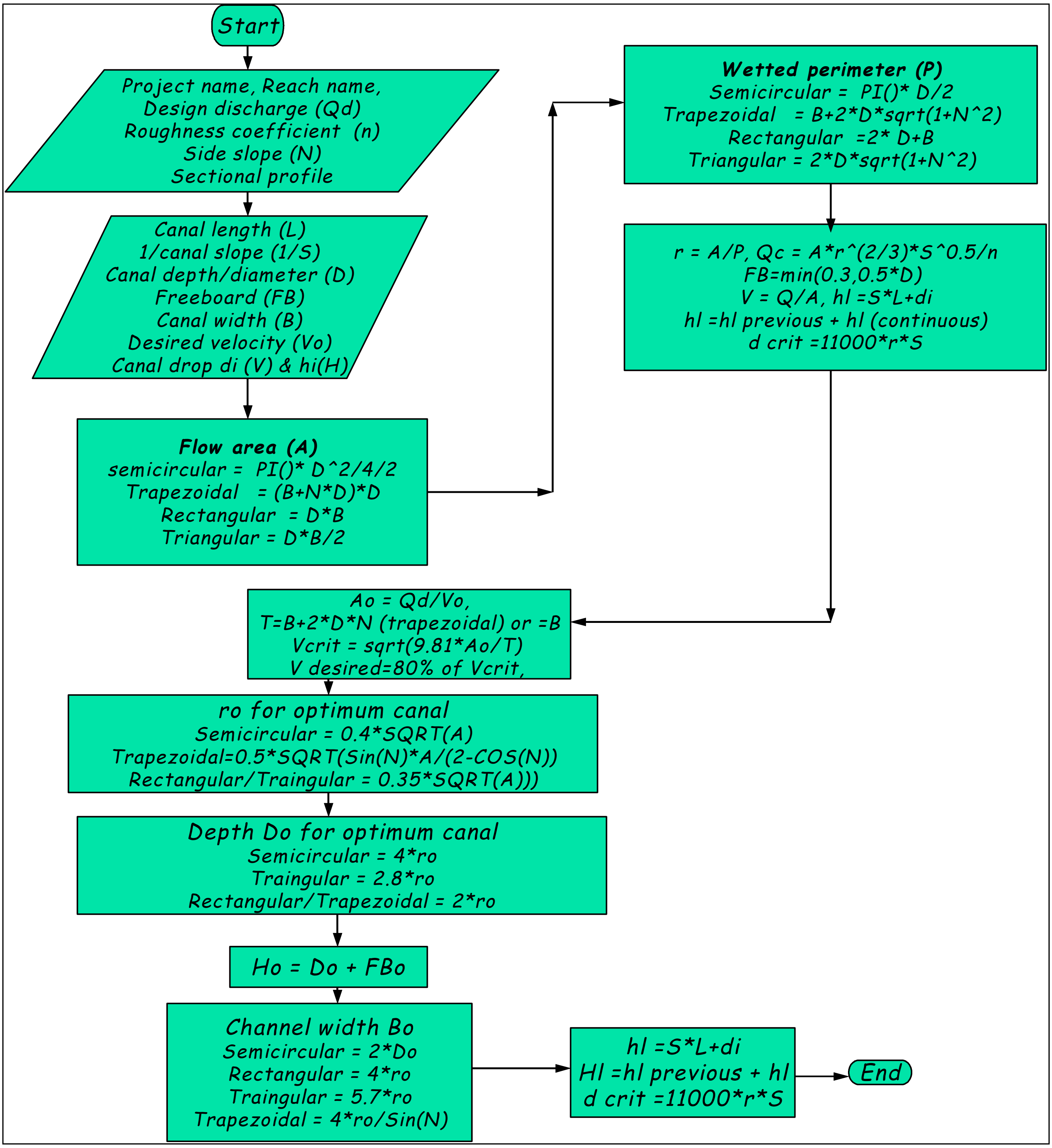


Figure 5.1: Flow chart for canal design

Critical dia of sediment d crit (mm) = 11000*r*S
= 11000*0.136*0.01299
= 19.48mm (i.e., it can self clean sediment of diameter 19.48 or less)

Optimum Canal:

Area A m^2 = Q/v desired
= 0.185/1
= 0.185 m^2

Hydraulic Radius ro (m) = 0.35*SQRT(A)
= 0.35*SQRT(0.185)
= 0.1505 m

Depth Do (m) = 2*ro
= 2*0.1505
= 0.301m

Top Width Bo (m) = 4*ro
= 4*0.1505
= 0.602m

Critical Velocity Vc m/s = sqrt(A*g/T)
= sqrt(0.185*9.81/.602)
= 1.74 m/s (Okay since the desired velocity of 1m/s is less than 80% of Vc)

Headloss hl (m) = S*L + di (drops)
= 0.0050*20+0
= 0.100m

Critical dia of sediment d crit (mm) = 11000*r*S
= 11000*0.136*0.0050
= 8.271mm (i.e., it can self clean sediment of diameter 19.48 or less)

Chhota MHP
Stone masonry canal

Input

Type and Name	Intake Canal	Tailrace	Main2	Main3
Flow (m^3/s)	0.185	0.145	0.145	0.145
Roughness coefficient (n)	0.02	0.017	0.02	0.02
Sectional Profile	Rectangular	Trapezoidal	Semicircular	Triangular
Side slope N (1V:NHorizontal)	0	0.5	0	0.5
Length of the canal (m)	20	40	150	120
1/Canal slope (s)	77	200	30	72
Channel Depth/diameter D (m)	0.300	0.525	0.300	0.300
Freeboard FB (m)	0.300	0.250	0.150	0.150
Channel Width (B) m	0.500	1.000	0.400	0.400
Channel Drops di m	0.000	0.000	0.000	0.000
Channel Drops Horizontal length hi m	0.000	0.000	0.000	0.000
Desired velocity Vo (m/s)	1.000	1.500	1.500	1.500

Output

Side slope d (degrees)	0.000	63.435	0.000	63.435
Canal slope S	0.01299	0.00500	0.03333	0.01389
Total depth H (m)	0.600	0.775	0.450	0.450
Chainage L (m)	20.000	60.000	210.000	330.000

Present canal				
Area A m^2	0.150	0.663	0.035	0.060
Top Width T (m)	0.500	1.525	0.400	0.400
Wetted Perimeter P (m)	1.100	2.174	0.471	0.671
Hydraulic Radius r (m)	0.136	0.305	0.075	0.089
Calculated flow (m^3/s)	0.226	1.249	low 0.057	low 0.071
Comment on freeboard	ok	low	ok	ok
Velocity V m/s	1.233	0.219	4.103	2.417
Critical Velocity Vc m/s & Remarks	1.72 Ok	2.06 Ok	0.93 Not Ok	1.21 Not Ok
Headloss hl (m)	0.260	0.200	5.000	1.667
Total headloss Hl(m)	0.260	0.460	5.460	7.126
Critical dia of sediment d crit (mm)	19.481	16.769	27.500	13.665

Optimum canal				
Area Ao m^2	0.1850	0.0967	0.0967	0.0967
Top Width T (m)	0.6022	0.7636	0.9949	0.4867
Critical Velocity Vc m/s & Remarks	1.74 Ok	1.11 Not Ok	0.98 Not Ok	1.4 Not Ok
Hydraulic Radius ro (m)	0.1505	0.1180	0.1244	0.1088
Channel Depth/diameter Do (m)	0.301	0.236	0.497	0.218
Freeboard Fbo (m)	0.150	0.263	0.150	0.150
Total depth Ho (m)	0.451	0.498	0.647	0.368
Channel Width Bo (m)	0.602	0.528	0.995	0.487
Canal Slope	0.0050	0.0112	0.0145	0.0173
Headloss hlo (m)	0.100	0.449	2.175	2.079
Total headloss Hlo(m)	0.100	0.549	2.724	4.803
Critical dia of sediment d crito (mm)	8.271	14.584	19.834	20.736

Figure 5.2: An example of canal design.

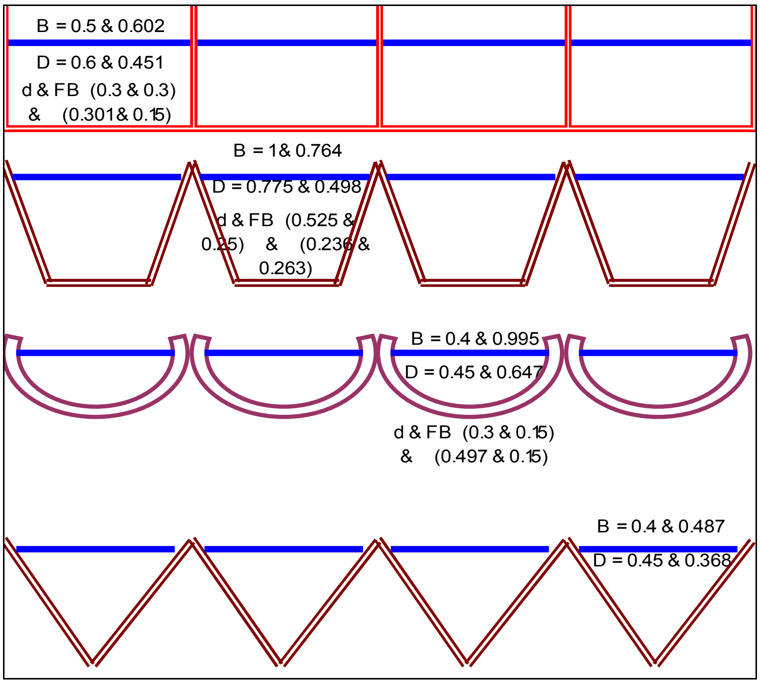


Figure 5.3: Illustrated canal type and their dimensions.

5.3.2 Pipe

The headrace pipe calculations presented in Figure 5.7 are briefly described in the following section. The trashrack calculations are similar to the trashrack calculations presented earlier in the intake design, hence it is not presented in this section. In this example, one HDPE pipe is considered for a design flow of 160 l/s each.

Sizing of headrace pipe

Headloss

HDPE pipe roughness, k =0.06 mm

$$\frac{k}{d} = \frac{0.06\,mm}{260\,mm} = 0.000231$$

$$\frac{1.2Q}{d} = \frac{1.2x0.160}{0.260} = 0.73846$$

From Moody chart (Appendix), f=0.0153. In the spreadsheet program, this friction factor is calculated by the method described in Layman's Guidebook on how to develop a small hydro site, ESHA. In this method f is calculated by iterating following equations:

$$\alpha = \sqrt{1/f} \qquad \alpha = -2\log\left(\frac{e/D}{3.7} + \frac{2.51}{N_R}\alpha\right)$$

Friction loss $= f\dfrac{l}{d}\dfrac{V^2}{2g}$

$$h_{wall\ loss} = \frac{fl0.08Q^2}{d^5} = \frac{0.0153x140x0.08x0.160^2}{0.260^5} = 3.82\,m$$

Turbulent losses considering, Kentrance= 0.8, Kexit=1.0 and Kbends based on the bending angles (see Table in the Appendix)

$$\therefore h_{turbulentlosses} = \left(K_{entrance} + K_{bends} + K_{valve} + K_{others} + K_{exit}\right)\left(\frac{1.5V^2}{2g}\right) = 1.5 * \frac{3.01^2}{2x9.81} = 0.69m$$

Total head loss = 3.82 m + 0.69 m = 4.53 m

Design water level in Gravel trap is 4.53 m (i.e. 22.66% of the available head) below low river level at intake. This is a headrace pipe conveying water from intake to the break pressure tank, therefore headloss more than this may be acceptable if the pipe material is mild steel. The HDPE pipe does not need expansion joints and therefore not calculated.

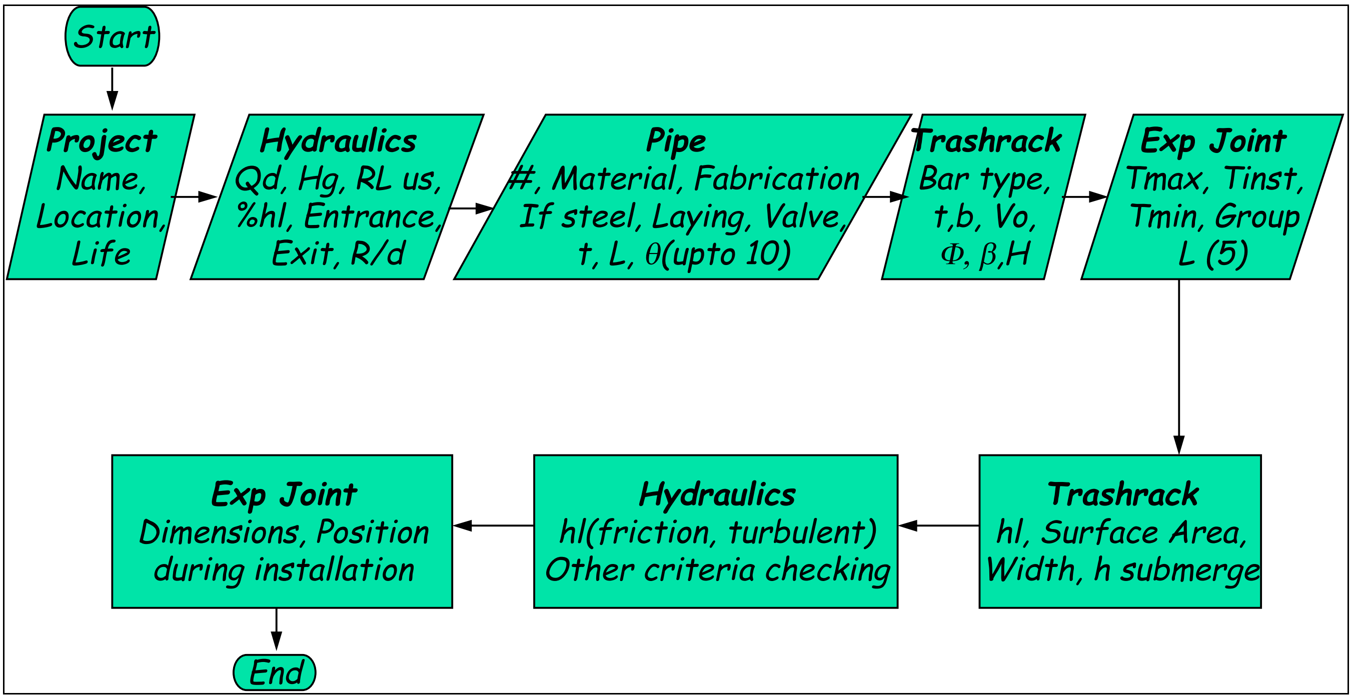


Figure 5.4: Flow chart for pipe design

INPUT			
General:			
Project:	Chhyota MHP		
Location:	Dolakha	Economic life (years)	15
Hydraulics:			
Diversion flow Qd (m^3/s)	0.110	U/S Invert Level (m)	1200.00
Flow in each pipe Qi (m^3/s)	0.110	% head available or headloss hlt (m)	5.00%
Gross headHg (m)	5.000	Entrance Type	Slightly rounded
		Bending radius (r/d)	5
Headrace pipe			
Pipe Material	PVC	Exit (Yes/No)	Yes
Welded / Flat rolled if steel	Welded	No of pipes	1.00
Rolled if steel	Rolled	Bending angle 01	20.00
Type if steel	IS	Bending angle 02	4.00
Burried or exposed	Burried	Bending angle 03	6.00
Type of valve	Gate	Bending angle 04	20.00
Non standard ultimate tensile strength (UTS) N/mm^2	0	Bending angle 05	0
Estimated pipe diameter d(mm)	**245**	Bending angle 06	0.00
Provided pipe diameter d(mm)	400	Bending angle 07	
Min pipe thickness t (mm)	**NA**	Bending angle 08	
Provided pipe thickness t (mm)	3.0	Bending angle 09	
Pipe Length L (m)	100.000	Bending angle 10	

Trashrack

Flat k	t	b	Vo	ϕ	β	Q	H
2.40	6.00	20.00	1.00	60.00	0.00	0.110	3.00

Expansion Joints

Tmax (deg)	T installation	Tmin	1st Pipe length(m)	2nd Pipe L (m)	3rd Pipe L (m)	4th Pipe L (m)	5th Pipe L (m)
40	20	4	50.00	100.00	150.00	200.00	250.00

OUTPUT							
Trashrack							
hf	**hb**	**H coeff**	**H**	**S**	**B**	**Min Submergence**	**CGL=1.5v^2/2g**
0.0213	0.0000	0.4174	0.0213	0.5504	0.16	0.81	0.06

Turbulent loss coefficients					
K inlet	0.20	K bend 05	0.00	K bend 10	0.00
K bend 01	0.16	K bend 06	0.00	K valve	0.10
K bend 02	0.13	K bend 07	0.00	K exit	1.00
K bend 03	0.13	K bend 08	0.00	K others	
K bend 04	0.16	K bend 09	0.00	K Total	1.71

Hydraulics			
Pipe Area A (m^2)	0.126	U/S Invert Level (mAOD)	1200.000
Hydraulic Radius R (m)	0.10	D/S Invert Level (mAOD)	1195.000
Velocity V (m/s)	0.88	Is HL tot < HL available	OKAY
Relative Roughness ks/d	2.500E-05	Friction Losses hf (m)	0.14
Reynolds Number Re = d V /Vk	307018	Fitting Losses hfit (m)	0.07
Type of Flow	Turbulent	Trashracks and intake loss (m)	0.02
Friction Factor f	0.0147	Total Head Loss htot individual (m)	0.23
		% of H.Loss of individual pipe	4.62% Ok

Expansion Joints (mm)	Coeff of linear expansion /deg C			5.4E-05
EJ number	1	2	2	4	5
dL theoretical	97	194	292	389	486
dL recommended	194	389	583	778	972
dL for expansion	108	216	324	432	540
dL for contraction	86	173	259	346	432

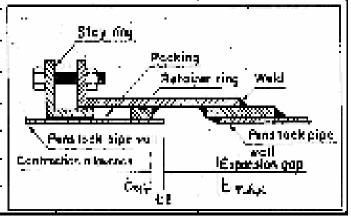


Figure 5.5: An example of pipe design

6 SETTLING BASINS

6.1 INTRODUCTION AND DEFINITIONS

6.1.1 Sediment Settling Basins

A settling basin traps/removes sediment (sand/silt) from water. Since sediment is detrimental to the project civil and mechanical structures and elements, the specific size of the sediment has to be removed to specified percentage. This can only be achieved by reducing turbulence of the sediment carrying water. The turbulence can be reduced by the construction of settling basins along the conveyance system. Since the settling basins are straight and have bigger flow areas, the transit velocity and turbulence are significantly reduced allowing the desired sediments to settle. The sediment thus settled has to be properly flushed back to the natural rivers.

Thus a settling basin:

1. Prevents blocking of headrace and reducing capacity
2. Prevents severe wearing of turbine runner and other parts.
3. Reduces the rate failure and O&M costs.

According to the location and function the settling basin can be of following types:

1. Gravel Trap for settling particles of 2mm diameter or bigger.
2. Settling Basin for settling particles of 0.2mm diameter or bigger.
3. Forebay for settling similar to settling basin (optional) and smooth flow transition from open to closed flow.

Micro hydro settling basins are generally made of stone masonry or concrete with necessary accessories such as spillways, flushing gates, trashracks, etc., as and where necessary. However, functionally, all the settling basins should have following components:

1. Inlet Zone: Gradual expansion.
2. Settling Zone: Settling, deposition, spilling and flushing (and trashrack removal).
3. Outlet Zone: Gradual contraction.

A typical section of a settling basin with all the components (inlet, transition, storage and outlet zones) and accessories (spillway, gate) is presented in Figure 6.1.

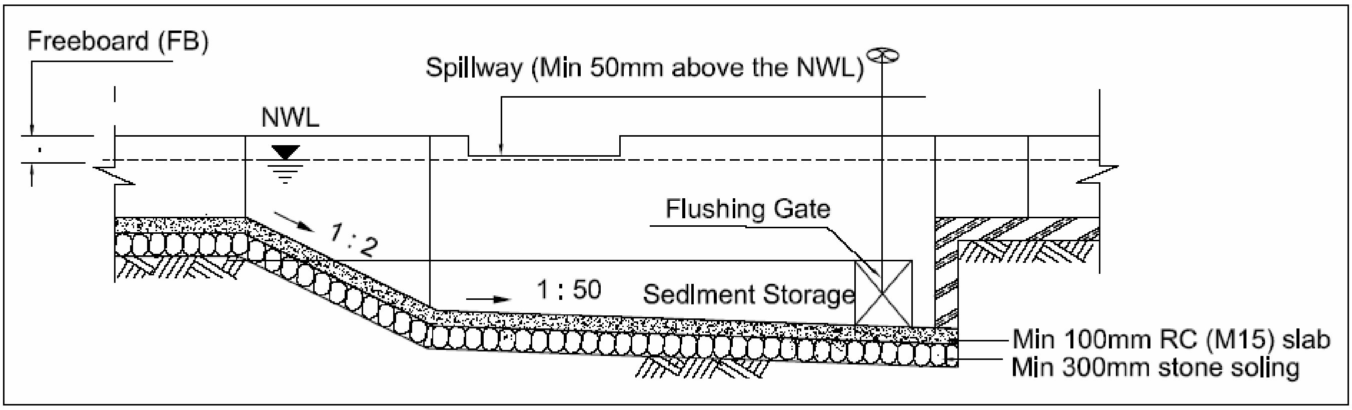


Figure 6.1: Typical section of a settling basin

6.2 SETTLING BASIN THEORY

An ideal settling basin is a basin where the flow is ideal and it flows in a straight line (no turbulence, no eddy current). In practical not a single basin is ideal. For an ideal basin shown in figure 6.3:

a) H/W = L/V => Q/W = L*B = Surface area (i.e., the surface area is directly proportional to the discharge and inversely proportional to the settling velocity/sediment diameter/temperature).

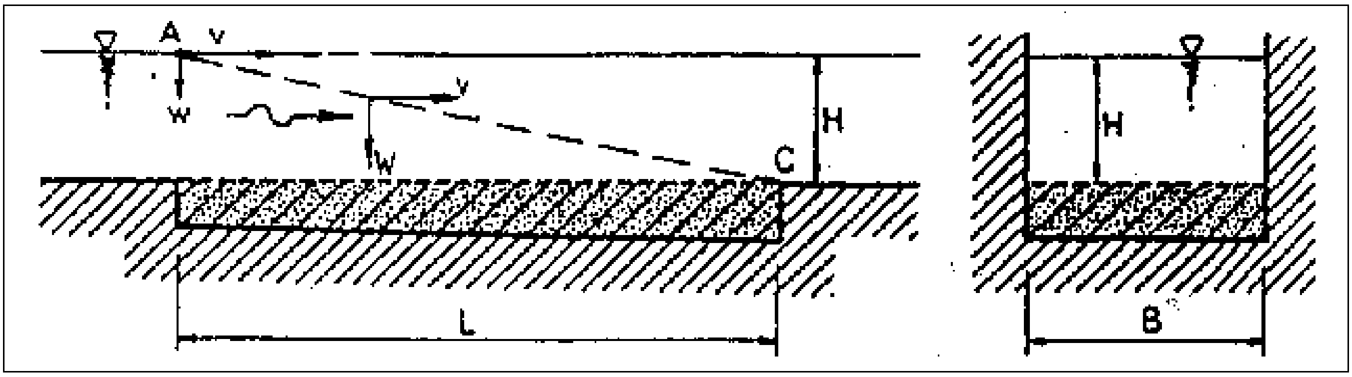


Figure 6.3: An ideal setting basin

As stated earlier a real basin is never ideal and the efficiency of an ideal basin is generally 2 times higher than that of a real basin. This is mainly because of the following factors:

1. Presence of water turbulence in basin.
2. Imperfect flow distribution at entrance.
3. Flow convergence towards exit.

Vetter's Equation takes care of the factors stated above and hence recommended for use in settling basin design.

$$\eta = 1 - e^{-\left(\frac{wA_s}{Q}\right)}$$

6.3 MGSP/ESAP GUIDELINES AND STANDARDS

6.3.1 Gravel Trap

While designing gravel trap, AEPC has outlined following points:

1. Location: Close to intake and safe.
2. Dimensions: Sufficient to settle and flush gravel passing through upstream coarse trashrack.
3. Spilling: Sufficient spillway/vertical flushing pipe.
4. Spilling and flushing: back to the river.
5. Material: 1:4 cement stone masonry with 12mm thick 1:2 cement plastering on the waterside.
6. Recommended settling diameter and trap efficiency are 2mm and 90% respectively.
7. Sediment storage zone: Storage for 12 hours (flushing interval)
8. Drawdown: Capacity of drawdown discharge = 150% of the design discharge.
9. Aspect ratio (straight length to width): 1.5 to 2.

6.3.2 Settling Basin

While designing a settling basin, AEPC has outlined following points:

1. Location: Close to gravel trap/Intake.
2. Dimensions: Sufficient to settle and flush the designed sediment size.
3. Spilling: Sufficient spillway/vertical flushing pipe (layout dependent).
4. Spilling and flushing: back to the river.
5. Material: 1:4 cement stone masonry with 12mm thick 1:2 plastering on the waterside.
6. Recommended settling diameter (trap efficiency) and head
   a. 0.3-0.5mm (90%) – 10m
   b. 0.3mm (90%) ------- 10 to 100m
   c. 0.2mm (95%) -------- more than 100m
7. Sediment storage zone: Storage for 12 hours (flushing interval)
8. Drawdown: Capacity of drawdown discharge = 150% of the design discharge.
9. Aspect ratio (straight length to width): 4 to 10.

6.3.3 Forebay

While designing forebay, AEPC has outlined following points:

1. Dimensions and functions: Similar to SB if u/s is open canal or combined SB+FB.
2. Submergence: Sufficient to prevent vortex (ie 1.5 * $v^2/2g$).
3. Active Storage: 15 sec * Qd.
4. Freeboard: 300mm or half the water depth whichever is less.
5. Drawdown: A drain pipe/Gate.
6. Spilling capacity: Minimum of spilling Qd (load rejection)
7. Fine Trashrack:
   a. At the entrance of the penstock
   b. Inclination: 3V:1H
   c. Bars: Placed along vertical direction (ease of racking).
   d. Clearance: 0.5 * nozzle diameter (Pelton) or half the distance between runner blade (Crossflow)
   e. Velocity: 0.6 to 1m/s
   f. Weight: =< 60kg (porter load)

6.4 PROGRAM BRIEFING AND EXAMPLES

6.4.1 Features of the spreadsheet

1. A single spreadsheet for:
   a. Gravel Trap
   b. Settling Basin (Desilting)
   c. Forebay-cum-Settling Basin
2. Settling of sediment using:
   a. Ideal settling equation

   b. Vetter's equation
3. Flushing of deposited sediment during:
   a. Normal operational hour
   b. Drawn-down condition
4. Sediment flushing with:
   a. Vertical flushing pipe
   b. Gate
   c. Combination of both
5. Spilling of excess flow due to:
   a. Incoming flood
   b. Load rejection
6. Spilling of excess flow with
   a. Spillway
   b. Vertical flushing pipe
   c. Combination of both
7. Drawdown / Dewatering with:
   a. Vertical flushing pipe
   b. Gate
8. Rating curve for the gate: The rating curve computation is based on Norwegian Rules and Regulations of Dam Construction. According to this manual, the flow is of free flow until the gate opening is two third of the water depth behind the gate. Beyond this level (i.e., the gate opening hither than 2/3 of the water depth behind the gate), the flow is a pressure flow.
9. Multiple basins
10. Combination of approach canal / pipe options

6.4.2 Vertical flushing pipe

1. Overflow: Acts as a sharp crested weir.
   $Q_f = \pi * d1 * C_w * hf^{2/3}$: $C_w = 1.6$
   $d_1 = Q_f/(1.6 * \pi * hf^{2/3})$

2. Drawdown / Dewatering
   $1.5*Qd = C*A*(h_b+f_{flush})^{0.5}$: $A=\pi*d21^{2/4}$: C=2.76 for L=<6m
   $d_{21}=(6*Qd/(\pi *C *(h_b+f_{flush})^{0.5})^{0.5}$ @ full
   $d_{22}=(4*Qd/(\pi *C *(f_{flush})^{0.5})^{0.5}$ @ empty

3. Design diameter: Maximum of above (d1, d21,d22)

6.4.3 Spillway at intake

hovertop = 50% of (FB – spillway crest height above NWL)
$Ls1 = Qf/C/h_{overtop}^{1.5}$:=>obstruction d/s=> hot is constant
$Ls2 = 2*Abs(Qf-Qd)/C/h_{overtop}^{1.5})$
$Ls3=Qf/C/(2* h_{overtop} -0.05)^{1.5}$:=>obstruction d/s=> (100%-0.05) hot is constant

6.4.4 Gate

Lifting force F(kg)=W buoyant + 1000*m*Asub*hcg
Gate opening dh=h1-h2

dh<2/3*h1 :=> Pressure flow (as a gate): $Q=C*L*(H1^{1.5}-H2^{1.5})$
dh=>2/3*h1 :=> Open flow (as a spillway): $Q=C*L*H1^{1.5}$
Enter the maximum gate opening in the lowest gate opening cell and press the Calculate Gate Rating Curve button for computing rating curve.

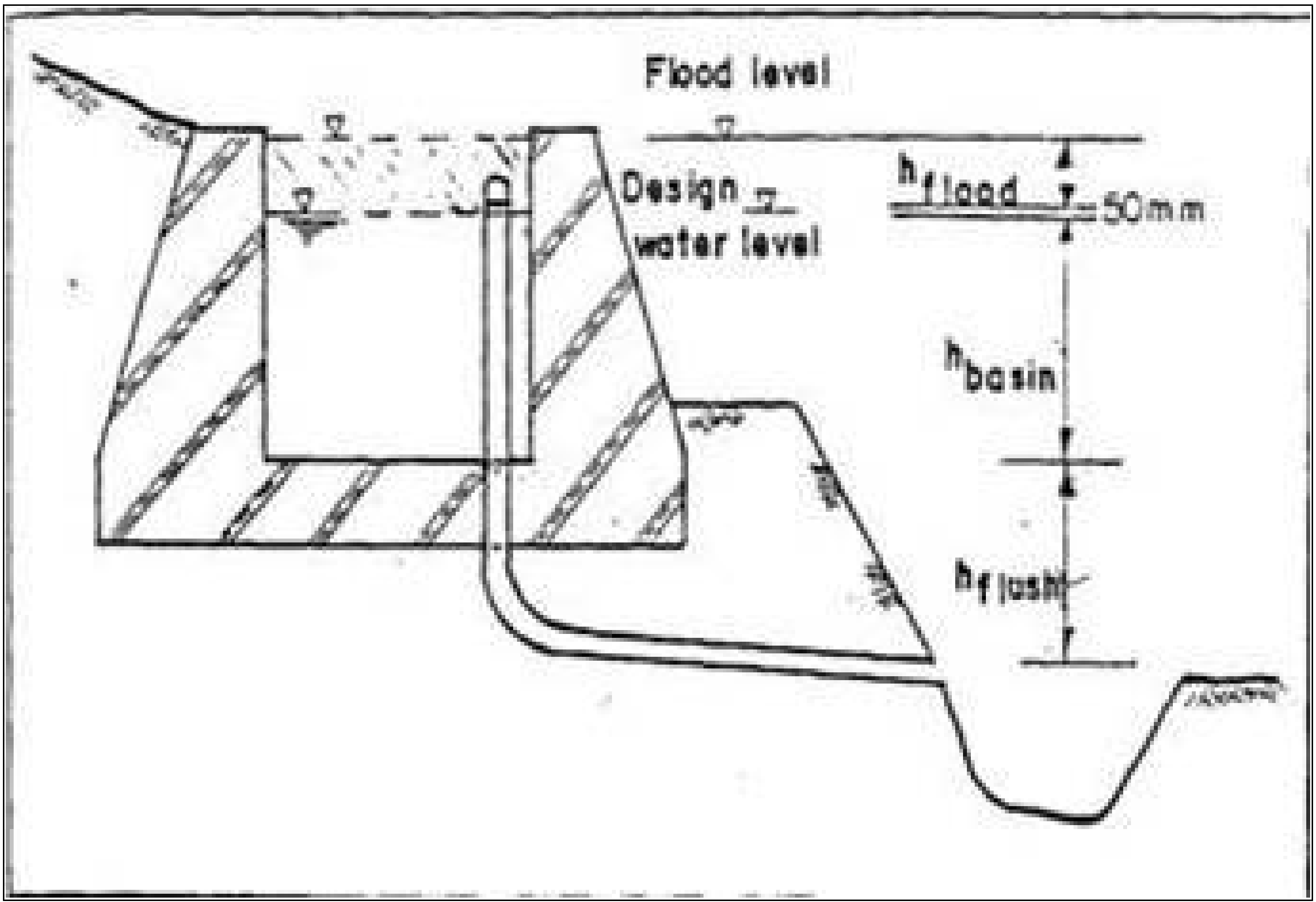


Figure 6.4: Flushing pipe details

Setlling Basin Design by Vetter's Equation

Chhota MHP

$$\eta = 1 - e^{-\left(\frac{wA_s}{Q}\right)}$$

Q flood	0.000	Sediment swelling factor S =	1.50
Manning's number M ($m^{1/3}/s$) 1/n=	50.000	Volume of sediment storage V (m3) =	15.12
Design discharge Q_{design} (m^3/s) =	0.421	Sediment depth Hs (m) = V/Asi	0.60
Flushing discharge Q_{flush} (m^3/s) =	0.034	Inlet approach conveyance Canal/Pipe =	Canal
Total discharge Q_{basins} (m^3/s) =	0.455	1/Bottom slope of SB Sf (1:50 to 1:20) =	50.00
Particles to settle d (mm) =	0.300	Outlet approach conveyance Canal/Pipe =	Pipe
Trapping efficiency n (%) =	85%	Water level at inlet NWL (m) =	1950.00
water temperature t (°C) =	10	h flush below the base slab (L<6m)	1.70
Fall velocity w at 10 deg C (m/s) =	0.035	Number of basins N	1.00
Sediment concentration Cmax (kg/m3) =	2	Spillway crest height above NWL m	0.05
Flushing Frequency Fl (hours) =	8	Spillway discharge coeff	1.60
Surface area / basin Asi (m2) 85 % =	25.000	Provided Freeboard h fb1 m	0.30
Basin transit velocity Vt (m/s) =	0.241	Discharge coeff for pipe as orifice (2.76 if L <6 m)	2.76
Bulk Sed density G (kg/m3) =	2600	Drawdown discharge % of design discharge	1.00
Discharge per basin Q_{basin} (m^3/s) =	0.455	Water depth of inlet canal hc1 (m) =	0.50
Max section width for hydraulic flushing B (m) =	3.258	Outlet canal width /canal diameter Bc2 (m) =	0.50
Width used B (m) =	2.500	Water depth of outlet canal hc2 (m) =	0.30
Inlet canal width /canal diameter Bc1 (m) =	1.000	Provided Length of the basin Lact (m)=	0.00

Length of basin L (m) =	10.000	Pipe does not need a straight approach!	***
Aspect ratio (4<=AR<=15)	4.000	Head over outlet weir h overtop (m) =	0.23
Min. water depth Hi (m) =	0.755	Approach inlet velocity vi1 (m/s) =	0.91
X-sectional area / basin Ai (m^2) =	1.888	Approach outlet velocity vi2 (m/s) =	3.03
Wetted perimeter / basin Pi (m) =	4.010	1/Energy gradient during operation So =	15763.86
Hydraulic radius Ri (m) =	0.471	d 50 during operation (mm) =	0.33
Normal WL @ basin h b m =	1.360	Depth of water during flushing yfi (m) =	0.12
Straight inlet transition length at 1:5 (m) =	3.750	d 50f during flushing (mm) =	49.32
Straight approach canal length (m) =	10.000	Length of an Ideal Basin (m) =	10.00

Spilling of excess water

Vertical Flushing pipe

Diameter for flood d1 m =	0.000	Diameter for load rejection (u/s flood bypass) d1 m =	2 x 0.43

Spillway

Freeboard m	0.300	Spillway length for Qd (under operation)	6.43
Spillway overtopping height h overtop m	0.125	Spillway length for Qd (load rejection & u/s flood bypass)	6.43
Spillway length for Qf (flood and non operational)	0.000	Spillway length for Qd (d/s obstruction & full hovertop-50)	3.18

Combination of vertical flushing pipe and spillway

		Flood and non operational (Qf)	
Vertical flushing pipe diameter d1 m	0.30	Flood discharge passing through vertical pipe	0.000
No of vertical flushing pipe	1.00	Spillway length for the remaining discharge m	1.00
Spillway length used (m)	1.00		

Flood and Under Operation (Qf- Qd)		*Load Rejection (Qd)*	
H overtopping	0.00	H overtopping	0.262
Discharge passing through vertical pipe	0.00	Discharge passing through vertical pipe	0.240
Discharge passing over spillway	0.00	Discharge passing over spillway	0.215

Figure 6.5: Typical example of a settling basin (Settling basin, spilling and flushing).

Flushing of water and sediment

Flushing pipe and orifice diameter

d for incoming flow and draw down m	0.35
d for incoming flow only (empty state) m	0.40
ncoming flow only (empty state & with y flushing) m	0.39

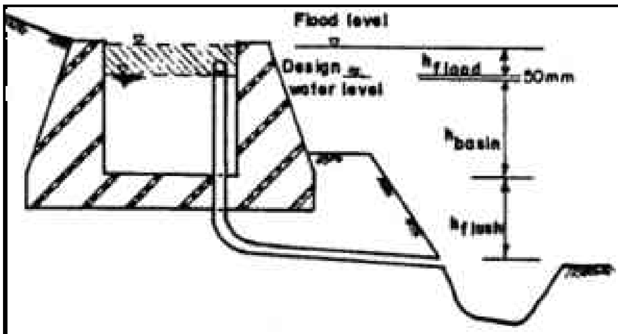
Gate

Buoyance weight of the gate W kgf	300.00
Gate Opening B, (m)	1.00
Gate Opening H (m)	0.50
Submerged area of th gate A m2	0.50
Water surface to cg of submerged area h m	1.11
Coeff of static friction mu	0.90
Lifting force F kgf	799.50
H. of water (H1)	1.36

Calculate Gate Rating Curves

Gate Opening Hg	Relative Gate Openinig Hg/H1	Discharge One basin Q
0.000	0.000	0.000
0.033	0.025	0.127
0.067	0.049	0.249
0.100	0.074	0.366
0.133	0.098	0.479
0.167	0.123	0.591
0.200	0.147	0.700
0.233	0.172	0.807
0.267	0.196	0.912
0.300	0.221	1.016
0.333	0.245	1.117
0.367	0.270	1.216
0.400	0.294	1.312
0.433	0.319	1.406
0.467	0.343	1.497
0.500	0.368	1.586

Figure 6.6: Typical example of a settling basin (Gate and rating curve).



Forebay cum settling basin (for one basin only)

Penstock diameter m	0.41
Penstock velocity m/s	3.45
Submergence depth of penstock pipe m	0.91
Height of pipe above the base slab m	0.30
Min. pond depth m	1.62
Effective thickness of penstock mm	3
FS for air vent (5 burried, 10 exposed)	10
Young's modulus of elasticity E N/mm2	200000
Penstock inlet gate (Yes/No)	No
Air vent diameter mm	Nominal

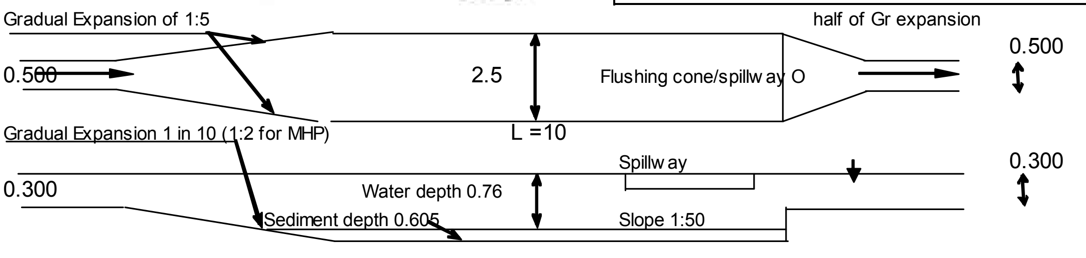


Figure 6.7: Typical example of a settling basin (forebay and dimensioning).

The headrace pipe calculations presented in Figure 6.6 are briefly described in the following section.

Sizing of settling basin

1. Settling of sediment using:

a. Vetter's equation

Surface area of basin = -(Q_{total}l)/w*LN(1-n_{eff})
Asi = -(0.455)/.035* LN(1-n_{eff})
= 25 m^2

Max section with for hydraulic flusing = 4.83*Q^0.5
= 4.83*.455^0.5
= 3.258m

Provided Width B = 2.5m

Length of basin L = Asi/B
= 25/2.5
= 10m, which is 4 times the witdth hence, satisfies the requirement.

Basin transit velocity vt = 0.44* sqrt(d)
= 0.44*sqrt(0.2)
= 0.241m/s

Water depth Hi = Qi/B/vt
= 0.455/2.5/0.241
= 0.755m

Sediment storage volume assuming 100% trap efficiency (conservative side)
V = (Q_{total})*(Flusing intensity in sec)*Concentration max in kg/Bulk Sed. Densigy in kg/m^3/Sed Swelling factor
= (Q_{total})*(FI*3600)*Cmax/G*S
= 0.455*(8*3600)*2/2.6*1.5
= 15.12m^3

Sediment depth Hs = V/Asi
= 15.12/25
= 0.6m

b. Ideal settling equation

Length of an Ideal Basin= Maximum of (4xB and Q/B/w)
= MAX(4*2.5, 0.455/2.5/0.035)
= MAX(10, 5.2)
= 10 m

2. Spilling of excess flow due to load rejection: A combination of a 0.3m diameter vertical pipe and spillway of 1.0m length is used.

H overtopping = h ot =(Q1/(1.9*PI()*n1*d1+Cd*Ls))^(2/3)
= (0.455/(1.9*pi()*1*.3+1.6*1))^(2/3)
= 0.262 m

Q pipe = 1.9*PI()*n1*d1*h ot ^1.5
= 1.9*pi()*1*.3*0.262^1.5
= 0.240 m^3/s

Q spillway = Cd*Ls*h ot ^1.5
= 1.6*1*0.262^1.5
= 0.215 m^3/s

3. Flushing of deposited sediment through the flushing pipe: The pipe diameter will be the biggest of :

a. For incoming flow and draw down:
D1 = (Qflushing%*4*Qi/(PI()*Cd*SQRT(h NWL+h flush)))^0.5
= (100%*4*0.455/(pi()**2.76*SQRT(1.36+1.7)))^0.5
= 0.35m

b. For incoming flow only:
D2 = (4*Qi/(PI()*Cd*SQRT(h flush)))^0.5
= (4*0.455/(pi()**2.76*SQRT(1.7)))^0.5
= 0.4m

In the second case the depth of water during flushing yfi may be added to h flush for higher precision. This is not considered here. The recommended minimum diameter of the flushing pipe diameter is 0.4 m.

The gate curve in the example presented in Figure 6.6 includes the gate dimensions, forces and the rating curve. The rating curve of the gate versus different gate opening can be computed by entering the allowable gate opening at the lowest input cell and clicking “Calculate Gate Rating Curve” button.

The last part the spreadsheet can be used if the considered basin is a settling basin cum forebay. The basic penstock inlet geometry is computed in this section.

7 PENSTOCK AND POWER CALCULATIONS

7.1 INTRODUCTION AND DEFINITIONS

A penstock pipe conveys water from free flow (at a settling basin or a forebay) to pressure flow to powerhouse and converts the potential energy of the flow at the settling basin or forebay to kinetic energy at the turbine.

7.2 MGSP/ESAP GUIDELINES AND STANDARDS

1. Material: Mild steel (exposed and buried) and HDPE (buried) pipes are widely used as penstocks for micro hydro schemes in Nepal.
2. For exposed (i.e., above ground) mild steel penstock alignment, a minimum clearance of 300 mm between the pipe and the ground should be provided for maintenance and to minimise corrosion effects.
3. In case of buried HDPE pipes, they should be buried to a minimum depth of 1 m. Similarly, if mild steel penstock pipes have to be buried, a minimum of 1 m burial depth should be maintained and corrosion protection measures such as high quality bituminous/epoxy paints should be applied. Due to higher risks of leakage, flange connected penstocks are not recommended to be buried.
4. The recommended initial trail internal diameter (D) can be calculated as:

   $D = 41 \times Q^{0.38}$ mm

   Where, Q = Design flow in l/s
5. Total penstock headloss should be limited to 10% of the penstock gross head.
6. Anchor / Thrust block at every horizontal and vertical bends and for every 30m of straight pipe stretch.
7. Expansion joints should be placed immediately downstream of every anchor block for exposed mild steel penstock.
8. Mechanical coupling of last penstock upstream of the turbine is recommended for ease of maintenance and lesser force distribution to the turbine casing..

7.3 PROGRAM BRIEFING AND EXAMPLE

7.3.1 Program Briefing

The design procedure of the penstock is similar to that of the headrace pipes. In this spreadsheet, the penstock is checked against surge / water hammer head propagated due to various closures of the system. Sudden closure of one jet is considered in the calculations.

Power based on the AEPC criteria and actual power generation based on the actual cumulative efficiency of the electro-mechanical are also presented.

Since a provision of maximum of ten bends is generally sufficient for a typical micro hydropower scheme, ten bends are incorporated. However, users can add any cumulative values if there are more than ten bends or other losses due to turbulence in the “K others” cells.

A fine trashrack is always recommended upstream of penstock inlet. A trashrack calculation is also included with the minimum submergence criteria by Gordon and AEPC criteria. AEPC criterion of 150% of the velocity head is enough for micro hydropower project up to 100kW.

User specific factor of safety and ultimate tensile strength are allowed in the spreadsheet.

The friction factor calculation is based on the iteration procedures described in the Layman's Guidebook on how to develop a small hydro site by European Small Hydropower Association.

Design and installation criteria of expansion joints are presented at the end of the spreadsheet.

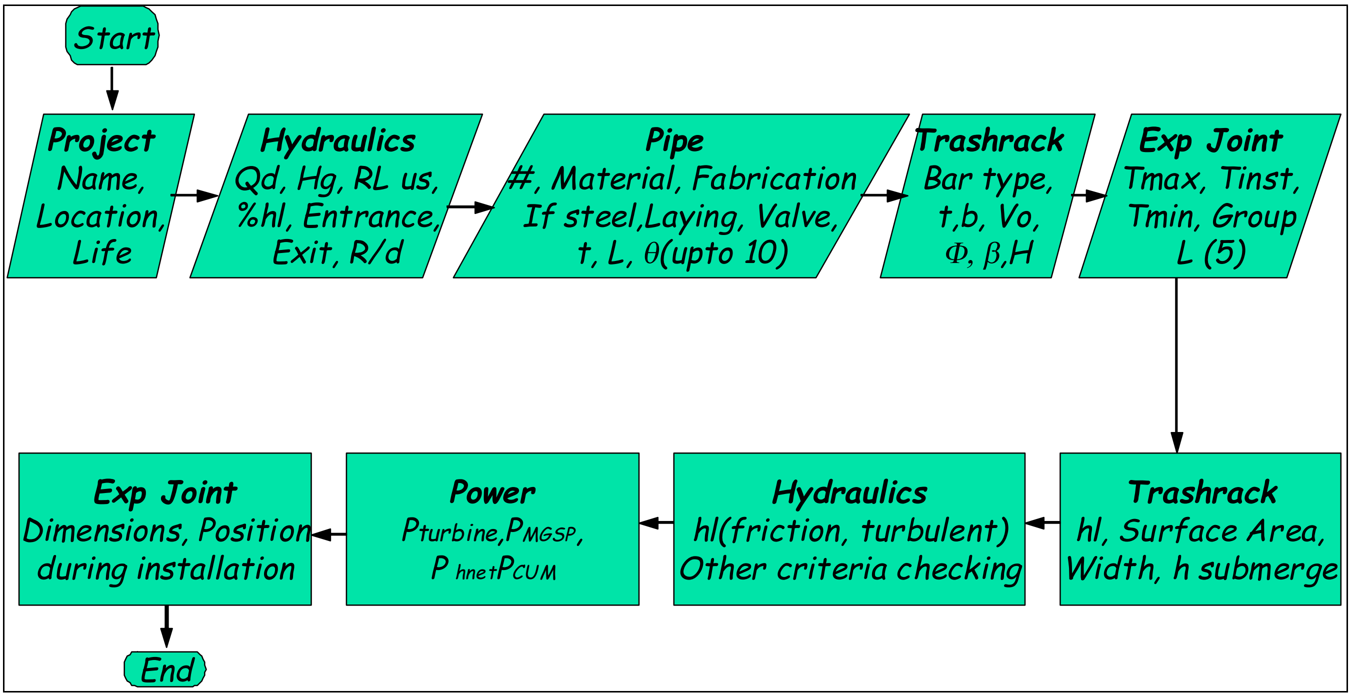


Figure 7.1: Flow diagram of penstock design

7.3.2 Typical example of penstock pipe

Figure 7.1 presents the calculation procedures applied in the example presented in Figures 7.2 and 7.3. A two-nozzle Pelton turbine is selected for the given head and discharge. Since the trashrack and pipe hydraulics are similar to headrace pipe presented earlier, they are not repeated in this section. The steel pipe thickness, expansion joints and power calculations are presented in this section.

Pipe thickness:

It is worth noting that in reality the diameter of penstock pipe is optimized by calculating marginal costs and benefit method. In this method, the incremental cost of annual energy by increasing the pipe diameters and corresponding increase of costs are plotted. The intersecting point reflects the cost of optimum diameter. Alternatively, net present values of these cash/cost flow can be calculated and the net present value (NPV) of marginal benefit from energy gain should be higher than that of the marginal cost of that diameter.

Let's consider 4mm thick 300mm diameter pipe, the wave velocity

$$a = \frac{1440}{\sqrt{1+\left(\dfrac{2.1x10^9\,x\,d}{E\,xt}\right)}} = \frac{1440}{\sqrt{1+\left(\dfrac{2.1\,x\,10^9\,x\,0.300}{200\,x\,10^9\,x\,\dfrac{4}{1000}}\right)}} = 1071.454\,m/s$$

h_{surge} = a * V/(g*njet)
= 1071.454*2.12/(9.81*2)
= 115.84m

h_{total} = h_{gross} + h_{surge}

= 69+115.84
= 184.84 m

t effective = t/(welding factor*rolling) – corrosion factor
= 4/(1.1*1.2) – 1.0
= 2.03mm

Factor of safety (allowable FS = 3.0)

$$S.F. = \frac{t_{effective}\ x\,S}{5\,x\,h_{total}\ x10^{3}\ x\,d} = \frac{(2.03/1000)\,x\,420x10^{6}}{5\,x\,184.84\,x\,10^{3}\ x\,0.300}$$

= 3.0 which does not exceed the allowable FS of 3.0, hence OK

Based on the American Society of Mechanical Engineers (ASME) formula, the allowable thickness is 4.00 mm. The static head for 3mm thick penstock of same diameter can be calculated by similar procedures and found to be 57.91m. This can be found by decreasing the pipe thickness (and diameter too) and noting the value corresponding to “H static capacity of the specified pipe (m)”. The lower portion of the pipe corresponding to the static head of 69-57.91 = 11.09 m should be of 4mm thick 300 diameter pipe. The summary of the output is presented in Figure 7.3.

PENSTOCK AND POWER CALCULATIONS

INPUT			
General:			
Project:	Gothi MHP		
Location:	Gothi Village, Gothi VDC, Humla	Economic life (years)	15
Hydraulics:			
Diversion flow Qd (m3/s)	0.150	WL @ forebay or U/S Invert Level (m)	1920.00
Flow in each pipe Qi (m3/s)	0.150	% head headloss hlt (m)	16.00%
Gross head (from forebay) Hg (m)	69.00	Cumulative known efficiency (g,t,tr,others)	79.38%
Power:			
Turbine type (CROSSFLOW/PELTON)	Pelton	Valves (Sperical/Gate/Butterfly)	Butterfly
No of total jets (nj)	2	Taper (Yes/No)	No
Direct Coupling (Yes/No)	No	Exit (Yes/No)	No
Closure time T sec	30	Non standard ult. tensile strength (UTS) N/mm2	0
Number of units	1		
Penstock pipe:			
Pipe Material (STEEL/HDPE/PVC)	Steel	Safety factor for lower pipes (0 for default)	3
Welded / Flat rolled if steel	Welded	Entrance Type	Slightly rounded
Rolled if steel	Rolled	Entrance with gate and air-vent (Yes/No)	No
Type if steel (UNGRADED/IS)	IS	Bending radius (r/d) (1/2/3/5/1.5)	1.5
Burried or exposed	Exposed	Bending angle 05	22
No of pipes	1.00	Bending angle 06	14.00
Bending angle 01(degrees)	27.00	Bending angle 07	38.00
Bending angle 02	11.00	Bending angle 08	22.00
Bending angle 03	4.00	Bending angle 09	47.00
Bending angle 04	11.00	Bending angle 10	50.00
Penstock diameter d=>d estd, d act (mm)	275 / 300	Pipe thickness t=>t min, t act (mm)	3.0 / 4.0
Pipe Length L (m)	121.000	Roughness coefficient (ks)	0.060

Trashrack

k	t	b	Vo	ϕ	β	Q	H
	6.00	20.00	1.00	71.56	0.00	0.150	0.70

Expansion Joints

Tmax (deg)	T installation	Tmin	1st Pipe length(m)	2nd Pipe L (m)	3rd Pipe L (m)	4th Pipe L (m)	5th Pipe L (m)
40	20	4	10.00	15.00	20.00	25.00	30.00

Figure 7.2: Input required for penstock and power calculations

OUTPUT							
Trashrack							
hf	**hb**	**H coeff**	**H**	**S**	**B**	**Min Submergence**	**CGL=1.5v^2/2g**
0.0000	0.0000	0.0000	0.0000	0.6852	0.93	1.15	0.34
Turbulent loss coefficients	K Total	2.82					
K inlet	0.20		K bend 05	0.24		K bend 10	0.33
K bend 01	0.26		K bend 06	0.22		K valve	0.30
K bend 02	0.21		K bend 07	0.29		K taper	0.00
K bend 03	0.19		K bend 08	0.24		K exit	0.00
K bend 04	0.21		K bend 09	0.32		K others	

Hydraulics			
Pipe Area A (m^2)	0.071	U/S Invert Level (mAOD)	1920.00
Hydraulic Radius R (m)	0.08	D/S Invert Level (mAOD)	1851.00
Velocity V (m/s)	2.12	Is HLtot < HL available	OKAY
Pipe Roughness ks (mm)	0.060	Friction Losses hf (m)	1.41
Relative Roughness ks/d	2.000E-04	Fitting Losses hfit (m)	0.65
Reynolds Number Re = d V /Vk	558214	Trashracks and intake loss (m)	0.00
Type of Flow	Turbulent	Total Head Loss htot individual (m)	2.06
Friction Factor f	0.0153	% of H.Loss of individual pipe	2.99% Ok
Factor of Safety			
Young's modulus of elasticity E N/mm^2	200000	Ultimate tensile strength (UTS) N/mm2	410
Thickness	4.000	H total for one jet closure of Pelton(m)	184.84
Diameter (mm)	300.000	t effective (mm)	2.03
Net Head (m)	66.940	Minimum t effictive for negative pressure (mm)	3.14
Wave Velocity a (m/s)	1071.454	Comment on thickness	NA, No gate
Critical time Tc (sec) *2 = Closing time T	0.23 Ok	Safety Factor (S)	3.00
K if crossflow turbine Kcf	0.00000	Check on Safety Factor	Ok
Hsurge for one jet closure of Pelton(m)	115.840	Air vent diameter d vent (mm)	43.65
Hsurge for instanteneous closure of all unit closure of Pelton (m)	231.680	H total capacity of the specified pipe (m)	184.98
Lengths (max & actual) of the specified pipe (m) & Ok 155.120	121.000	H static capacity of the specified pipe (m)	88.46

Power			
Turbine efficiency as per MGSP	75.00%	Electrical Power as per MGSP GL (kW)	50.77
Available shaft power(kW)	70.18	Electrical Power based on Hnet (kW)	59.10
Reqd.'Turbine Capacity (+10%) (kW)	77.20	Power for known cumulative eff (kW)	78.19

Expansion Joints (mm)		Coeff of linear expansion /deg C		1.2E-05
EJ number	1	2	2	4	5
dL theoretical	4	6	9	11	13
dL recommended	9	13	17	22	26
dL for expansion	5	7	10	12	14
dL for contraction	4	6	8	10	12

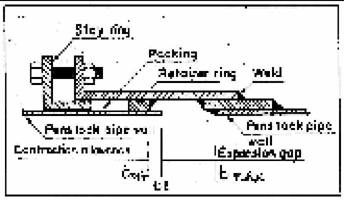


Figure 7.3: Output of penstock and power calculation spreadsheet.

8 TURBINE SELECTION

8.1 INTRODUCTION AND DEFINITIONS

A turbine converts potential energy of water to rotational mechanical energy. Cross-flow and Pelton turbines are the most commonly used turbines in Nepali micro hydropower plants. The size and type of turbine for a particular site depends on the net head and the design flow. Pelton turbines are suitable where the ratio of head to flow is high whereas Cross-flow turbines are suitable for high flow and low head schemes. It should be noted that for certain head and flow ranges both Pelton (multi-jet) and Cross-flow turbines may be appropriate. In such cases, the designer should consult with the manufacturer and make a decision based on availability, efficiency and costs. On a horizontal shaft Pelton turbine the maximum number of jets should be limited to 3 for ease of manufacturing. The number of jets can be higher for vertical shaft Pelton turbines. However, these require higher precision work in mounting the generator vertically on the turbine shaft and furthermore, in case of varying rotational speeds (RPM of the turbine and the generator), the belt drive arrangements (including those for mechanically coupled end uses) will be difficult.

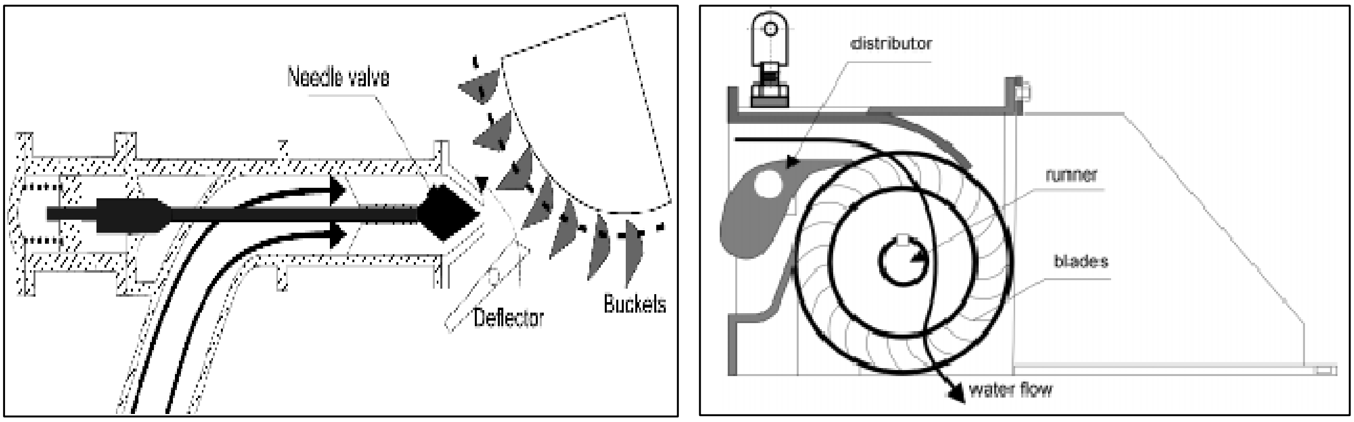


Figure 8.1: A Pelton and a Crossflow Turbines

8.2 MGSP/ESAP GUIDELINES AND STANDARDS

The recommended net heads for maximum rpm and efficiencies for different turbines and turbine specifications are presented in Table 8.1.

Table 8.1: Turbine specifications

Type	Net head (m)	Max RPM	Efficiency (n_t)
Pelton	More than 10m	400 to 1500	70 - 75%
T12 Crossflow	up to 50m	900	60 - 78%
T15 Crossflow	up to 80m	1500	60 - 78%

The type of turbine can be determined by its specific speed given by the following equation:

Sp Speed (no gear) n_s = rpm*$\sqrt{}$(1.4*P kW/N tur)/Hn$^{5/4}$

Sp Speed (gear) n_{sg} = Sp Speed (no gear)* Gear at Turbine/Gear at Generator

The only unknown in the above equation is the turbine rotational speed (N). This depends on the RPM of the generator and the drive system used (e.g., direct or belt drive). The specific speed of a multi-jet Pelton turbine can be computed by multiplying the above n_s by the square root of the number of jets. Because of higher efficiency both in overall and part load, preference should be given to Pelton turbines. If n_s exceeds the range given in Table 8.2, multiple units should be used.

Table 8.2: Turbine type vs. n_s

Turbine types	n_s Ranges
Single Jet Pelton	10 – 30
Double Jet Pelton	30 – 40
Three Jet Pelton	40 – 50
Cross flow	20 – 80

8.3 PROGRAM BRIEFING AND EXAMPLE

The Turbine spreadsheet is presented in Figure 8.2. The specific speeds (n_s) with and without gear are calculated as:

Sp Speed (no gear)= rpm*SQRT(1.4*P kW/N tur)/Hn^(5/4)
= 750*SQRT(1.4*67.89/1)/58^(5/4)
= 46 (Turgo/Crossflow/2-jet Pelton is suitable)

Sp Speed (gear) = Sp Speed (no gear)* Gear at Turbine/Gear at Generator
= 46*1/2
= 23 (Single jet Pelton/Crossflow is suitable)

Specific speed of multi-jet Pelton is computed by multiplying the specific speed of runner by the square root of the number of the jets. The calculations show that for the given parameters in the context of micro hydro plants in Nepal, either a gearless Crossflow turbine or a Pelton/Crossflow with a gear ratio of 1:2 is recommended.

Turbine Selection

Chhota MHP

Input			
Discharge (l/s)	150	Gear ratio at turbine	1
Gross head (m)	69	Gear ratio at generator	2
Hydraulic losses	15.94%	No of turbines/generators	1
Max turbine output kW	67.89	Total number of jets if Pelton n	2
Turbine rpm	750		
Output			
Net head m	58.000	Generator with gearing rpm	1500
No Gearing		**With Gearing**	
Sp speed of runner rpm (no gearing)	46	Sp speed of turbine	23
Pelton (12-30) => (Ns 17-42)	**	Pelton (12-30) => (Ns 17-42)	Pelton
Turgo (Ns 20-70) => (Ns 28-99)	Turgo	Turgo (Ns 20-70) => (Ns 28-99)	**
Crossflow (Ns 20-80)	Crossflow	Crossflow (Ns 20-80)	Crossflow
Fracis (Ns 80-400)	**	Fracis (Ns 80-400)	**
Propeller or Kaplan (Ns 340-1000)	**	Propeller or Kaplan (Ns 340-1000)	**

Figure 8.2: A Typical turbine example.

9 ELECTRICAL EQUIPMENT SELECTION

9.1 INTRODUCTION AND DEFINITIONS

A generator converts mechanical energy to electrical energy. There are two types of generators; namely, synchronous and induction (asynchronous). Induction generators are inexpensive and appropriate for small schemes (up to 15kW). For large schemes (10kW and more), the synchronous generators are suitable. Both synchronous and asynchronous generators are available in single and three phases.

Load controllers are generally used as the governing system in Nepali micro hydro schemes. An Electronic Load Controller (ELC) is used for controlling the power output of a synchronous generator. To control an induction generator an Induction Generator Controller (IGC) is used.

9.2 MGSP/ESAP GUIDELINES AND STANDARDS

9.2.1 Selection of generator size and type

Single Phase Vs. Three Phase

Advantages of Three Phase

- Saving of conductor and machine costs.
- Cheaper above 5 kW.
- Less Weight/size ratio.

Advantage of Single Phase

- Simple wiring.
- Cheaper ELC.
- Load balance not considered.

Induction Vs. Synchronous Generators

Induction Generator

Advantages of Induction Generator:

- Availability
- Cheap, rugged and simple in construction
- Minimum Maintenance

Drawbacks of Induction Generator:

- Problem supplying large inductive loads.
- Less durability of the capacitor bank.
- Poor voltage regulation compared to synchronous generator.

Synchronous Generator

Advantages of Synchronous Generator:

- High quality electrical output.
- Higher efficiency.
- Can start larger motors.

Drawbacks of Synchronous Generator:

- The cost is higher than induction generator for small sizes.

Based on the above assumptions, the general guidelines for selection of phase and type of generator are prepared and summarized in Table 9.1.

Table 9.1: Selection of generator type

Size of scheme	Up to 10 kW	10 to 15 kW	More than 15 kW
Generator	Synchronous/Induction	Synchronous/Induction	Synchronous
Phase	Single or Three Phase	Three Phase	Three Phase

Factors affecting the size of the generator are temperature, altitude, electronic load controller correction factor and power factor of the load. De-rating coefficients to allow for the above-mentioned factors are presented in the Table 9.2.

Table 9.2: Generator rating factors

	Max. Ambient temperature in ^oC	20	25	30	35	40	45	50	55
A	Temperature Factor	1.10	1.08	1.06	1.03	1.00	0.96	0.92	0.88
	Altitudes	1000	1250	1500	1750	2000	2250	2500	2750
B	Altitude Factor	1.00	0.98	0.96	0.945	0.93	0.915	0.90	0.88
	Altitudes	3000	3250	3500	3750	4000	4250	4500	
B	Altitude Factor	0.86	0.845	0.83	0.815	0.8	0.785	0.77	
C	ELC Correction Factor								0.83
D	Power Factor		When load is light bulbs only						1.0
			When load includes tube light and other inductive loads						0.8

9.2.2 Sizing and RPM of Synchronous Generator:

The steps for selecting the size of a synchronous generator are as follows:

1 Take a power factor of 0.8.

2 The size of synchronous generator:

$$\text{Generator kVA} = 1.3 * \frac{\text{Installed Capacity in kW}}{A \times B \times C \times D}$$

Where, A, B, C and D are correction factors from Table 9.2, and 1.3 is the overrating factor (up to 30% recommended) to allow for:

i) Unexpected higher power from turbine.

ii) Handling the starting current if large motors (> 10% of generator size) are supplied from the generator.

iii) The generator running at full load when using an ELC.

3 The synchronous rotational speed:

$$Rotational\ speed(N) = \frac{120f}{P} RPM$$

Where, RPM is revolutions per minute

f is the frequency of the system in Hertz (Hz) (50 Hz in Asia and Europe)

P is the number of poles of the generator (2, 4, 6, etc., in pairs). P for Nepali MHP is generally 4 so that the rotational speed is 1500 RPM.

9.2.3 Sizing and RPM of Induction Generator:

The steps for selecting the size of an induction generator are as follows:

1 The size of induction generator:

$$\text{Induction Generator kW} = 1.3 * \frac{\text{Installed Capacity in kW}}{A \times B}$$

The rating of an induction generator, which is basically a motor, is in kW. Therefore, the ELC factor (C) and the power factor (D) corrections are not applicable. The constant 1.3 is the overrating factor (up to 30% recommended) to serve similar purposes as in the case of a synchronous generator. The generator rating (voltage and current) should not exceed 80% of the electrical motor rating.

2 The rotational speed of induction generator:

$$Rotational\ speed(Ni) = \frac{120f}{P}(1+s)RPM$$

Where,

P and f are as same as for synchronous generator and $s = \frac{Ns - Nr}{Ns}$

Where,
s is the slip of the generator

N_s is the synchronous speed, i.e. $Ns = \frac{120f}{P}RPM$

Nr is the rated rotor speed of the induction motor and Ni always exceeds Ns while acting as a generator.

9.3 PROGRAM BRIEFING AND EXAMPLE

9.3.1 Program Briefing

In addition to calculating electrical parameters stated above, following components are added to the spreadsheet:

1 Computation of excitation capacitance for an induction generator.

2 Sizing of the electrical load controller (ELC) or the induction generator controller (IGC) (equal to the installed capacity).

3 Sizing of ballast (20% higher than the installed capacity). In case the installed capacity exceeds or equal to 50kW, the ballast capacity of ELC-Extension is calculated as:

   Ballast capacity of ELC extension (kW) = 60% * 1.2 * Pe + 40% * Pe

4 Sizing of MCCB/MCB.

5 Sizing of power cables.

a) Sizing of excitation capacitance of an Induction Generator

Excitation capacitance for Delta connection C (цF) = 1/(2*pi()*f*Xc*η_m)

$$\text{Or, C (цF)} = \frac{1000*Pe*\sin(\cos^{-1}(\text{power factor}))}{3*V^2*pf*2*pi()*f*\eta_m}$$

Where,

Xc (Ω) = V / I_m

V (V) = Rated Voltage of the motor (V) (phase to phase voltage 380/400/415)
I_m (A) = Magnetizing Current = I rated at full load current (A) * sin ($\cos^{-1}$ (power factor))
I rated at full load current = Rated power (kW) * 1000/(V*pf)
η_m = rated efficiency of motor at full load
For star connected capacitors, the excitation capacitance is three times that for the Delta connection.

b) Sizing of MCCB/MCB (A) = 1.25*Pe * 1000/(V*pf)

Where,
1.25 = overrating factor by 25%.
Pe (kW)= Installed capacity
V (V) = Rated phase to neutral Voltage (V) (V*√3 for 3-phase)
pf = power factor if induction generator is used

c) Sizing of power cable (A) = 1.7*I

Where,
1.70 = overrating factor by 70%.
I (A) = Current = Generator size/(V *pf if induction generator is used)
V (V) = Rated phase to neutral Voltage (V) (V*√3 for 3-phase)
pf = power factor

9.3.2 Typical example of a 3-phase 60kW synchronous generator

Selection of Electrical Equipment			
Name of the Project:	**Upper Jogmai, Ilam**		
INPUT			
Discharge (m^3/s)	0.204	Power factor	0.8
Gross head (m)	60.000	Safety factor of generator	1.3
Overall plant efficiency (%)	50%	Phase	3-phase
Temperature (°C)	45	Type of Generator	Synchrono
Altitude (m)	1500	Over rating factor of MCCB	1.25
ELC correction factor	0.83	Over rating factor of cable	1.5
Frequency of the system (Hz)	50	No. of poles	4
Capacity of used generator (kVA)	0	Rated rotor speed if induction generator N (rpm)	0
	Delta		89%
OUTPUT			
Pe Electrical output (active power) (kW)	**60.04**	**Ok**	
Generator			
Temp.factor	0.96	Altitude factor	0.96
Capacity (kVA)	**127.70**	**Actual available capacity (kVA)**	140.00
Synchronous rotational speed Ns (rpm)	1500		
ELC capacity (kW)	60.04	**Calculated Ballast capacity 1.2*Pe (kW)**	72.04
		Ballast capacity of ELC-Extention (kW)	67.24
Rated Voltage (V)	400	**I_{rated} for Cable & MCCB (A) at Generator side**	202.08
Rating of MCCB (A)	108.32	**Calculated size of MCCB (A)**	**135.40**
Cable			
Rating (A)	303.12	Size of 4-core cupper armoured cables	**185**

Figure 9.1: Electrical components of a 20kW 3-phase synchronous generator.

The electrical components presented in Figure 9.1 are computed as:

The size of synchronous generator:

$$\text{Generator kVA} = 1.3*\frac{\text{Installed Capacity in kW}}{\text{A x B x C x D}}$$

= 1.3*60.04/(0.96*0.96*0.83*0.8)

= 127.70 kVA

The higher size available in the market of 45kVA is used.

$$Rotational\ speed(N) = \frac{120f}{P} RPM$$

=120*50/4

= 1500 rpm

Since Pe > 50kW, the ballast capacity of ELC extension (kW) = 60% * 1.2 * Pe + 40% * Pe

= 0.6*1.2*60.04 + 0.4*60.04

= 67.24 kW

I_{rated} for Cable & MCCB at Generator side = 1000/ V_{rated} * Generator size /1.732

= 1000 / 400*140/1.732

= 202.08 Amp

Calculated size MCCB/MCB (A) = 1.25*Pe * 1000/(V*pf)

= 1.25*60.04*1000/(400*1.732*0.8)

= 135.40 Amp

Power cable inside the powerhouse

Rating current = 1.5*I_{rated}

= 1.5*202.08

= 303.12 Amp

For this current a 4-core copper armoured cable of ASCR 185mm^2 is chosen.

9.3.3 Typical example of a single phase 20kW induction generator

Figure 9.2 present electrical equipment sizing of the previous project with a single phase induction generator with a rotor speed of 1450rpm.

Since the electrical output is more than 10kW, a reminder error is flagged in the adjacent cell.

The electrical components presented in Figure 9.1 are computed as:

The size of synchronous generator:

$$\text{Generator kW} = 1.3*\frac{\text{Installed Capacity in kW}}{\text{A x B}}$$

= 1.3*20/(0.96*0.96)

= 28.25 kW

The higher size available in the market of 30kW is used.

$$Rotational\ speed(N) = \frac{120f}{P}RPM$$

=120*50/4
= 1500 rpm

Rotational speed of a generator = Ns*(1+(Ns-N)/Ns)
= 1500*(1+(1500-1450)/1500)
= 1550 rpm

Selection of Electrical Equipment			
Name of the Project:	**Upper Jogmai, Ilam**		
INPUT			
Discharge (m^3/s)	0.08	Power factor	0.8
Gross head (m)	50.968	Safety factor of generator	1.3
Overall plant efficiency (%)	50%	Phase	1-phase
Temperature (^oC)	45	Type of Generator	Induction
Altitude (m)	1500	Over rating factor of MCCB	1.25
ELC correction factor	0.83	Over rating factor of cable	1.5
Frequency of the system (Hz)	50	No. of poles	4
Capacity of used generator (kW)	0	Rated rotor speed if induction generator N (rpm)	1450
Capacitor configuration	Delta	Efficiency of motor at full load	89%
OUTPUT			
Pe Electrical output (active power) (kW)	**20.00**	**Use of 3-phase generator is mandatory**	
Generator			
Temp.factor	0.96	Altitude factor	0.96
Capacity (kW)	**28.25**	**Actual available capacity (kW)**	30.00
Synchronous rotational speed Ns (rpm)	1500	Rotational speed of the generator (rpm)	1550
IGC capacity (kW)	20.00	**Calculated Ballast capacity 1.2*Pe (kW)**	24.00
		Excitation Capacitance (micro F)	123.16
Rated Voltage (V)	220	**I_{rated} for Cable & MCCB (A) at Generator side**	170.45
Rating of MCCB (A)	113.64	**Calculated size of MCCB (A)**	**142.04**
Cable			
Rating (A)	255.68	Size of 2-core cupper armoured cables	**150**

Figure 9.2: Electrical components of a 20kW 1-phase induction generator.

Excitation capacitance

$$C\ (\text{цF}) = \frac{1000 \cdot Pe \cdot \sin(\cos^{-1}(\text{power factor}))}{3 \cdot V^2 \cdot pf \cdot 2 \cdot pi() \cdot f \cdot \eta_m}$$

$$C\ (\text{цF}) = \frac{1000 \cdot 20 \cdot \sin(\cos^{-1}(0.8))}{3 \cdot 400^2 \cdot 0.8 \cdot 2 \cdot pi() \cdot 50 \cdot 0.89}$$

= 123.16 цF

I_{rated} for Cable & MCCB at Generator side = 1000/ V_{rated} * Generator size /pf
= 1000 / 220*30/0.8
= 170.45 Amp

MCCB/MCB (A) = 1.25*Pe * 1000/(V)
= 1.25*20*1000/(220)
= 142.05 Amp

Power cable inside the powerhouse

Rating current = 1.5*I_{rated}
= 1.5*170.45
= 255.68 Amp

For this current a 2-core copper armoured cable of ASCR 185mm^2 is chosen.

TRANSMISSION AND DISTRIBUTION

9.4 INTRODUCTION AND DEFINITIONS

Power generated at a powerhouse is evacuated to load centres with the help of transmission and distribution lines. According to the Nepal Standard, 400/230V is used for distribution system whereas 400/11000V is used for transmission system. Use of these standard voltages is recommended so that the power can be easily synchronised and evacuated to grid in future.

9.5 MGSP/ESAP GUIDELINES AND STANDARDS

1 Type: Buried or suspended on wooden or steel or concrete poles.

2 Permissible Voltage drop: 10% of nominal value.

3 Conductor: ACSR or Arial Bundled Cable (ABC)

4 The ASCR specifications are presented in Table 10.1.

Table 10.1: ASCR specifications

ACSR Code number	Type of ACSR	Resistance Ohm/km	Current rating max Amps	Equivalent Copper area mm^2	Impedance Ohm/km	Sp. Weight (kg/km)	Sp. Cost (Rs/km)
1	**Squirrel**	**1.374**	76	**13**	**0.3013**	**80**	**13000**
2	**Gopher**	**1.098**	85	**16**	**0.294**	**106**	**14500**
3	**Weasel**	**0.9116**	95	**20**	**0.288**	**128**	**15500**
4	**Rabbit**	**0.5449**	135	**30**	**0.2723**	**214**	**25750**
5	**Otter**	**0.3434**	185	**50**	**0.257**		
6	**Dog**	**0.2745**	205	**65**	**0.25**	**394**	**52000**

9.6 PROGRAM BRIEFING AND EXAMPLE

9.6.1 Program Briefing

1 The presented spreadsheet is designed to calculated transmission parameters for three phase 11kV and 400V and single phase 230V transmission and distribution lines.

2 Balanced load is considered, i.e., neutral does not carries any current.

3 The rated voltage of 400V for three-phase and 230V for single phase are considered in the design.

4 With a power factor of 0.8, the rated current and voltage drop are calculated as:

Phase	Current (A)	Voltage drop (cV)
3-phase	Power*1000/(1.732*V* power factor)	1.732*I*Z*L
1-phase	Power*1000/(V*power factor)	2*I*Z*L

5 Impedance (Z) = √(Resistance^2+Reactance^2).

6 Voltage at node (Vi)

Phase	Voltage at node (Vi)
Single to single phase or 3 to 3 phase	Vprevious - dV
Three to single phase	Vprevious/1.732 - dV

7 ASCR Specifications:

ACSR Code number	Type of ACSR	Resistance Ohm/km	Current rating max Amps	Equivalent Cu area mm^2	Impedance Ohm/km	Sp. Weight (kg/km)	Sp. Cost (Rs/km)
1	Squirrel	1.374	76	13	0.3013	80	13000
2	Gopher	1.098	85	16	0.294	106	14500
3	Weasel	0.9116	95	20	0.288	128	15500
4	Rabbit	0.5449	135	30	0.2723	214	25750
5	Otter	0.3434	185	50	0.257		
6	Dog	0.2745	205	65	0.25	394	52000

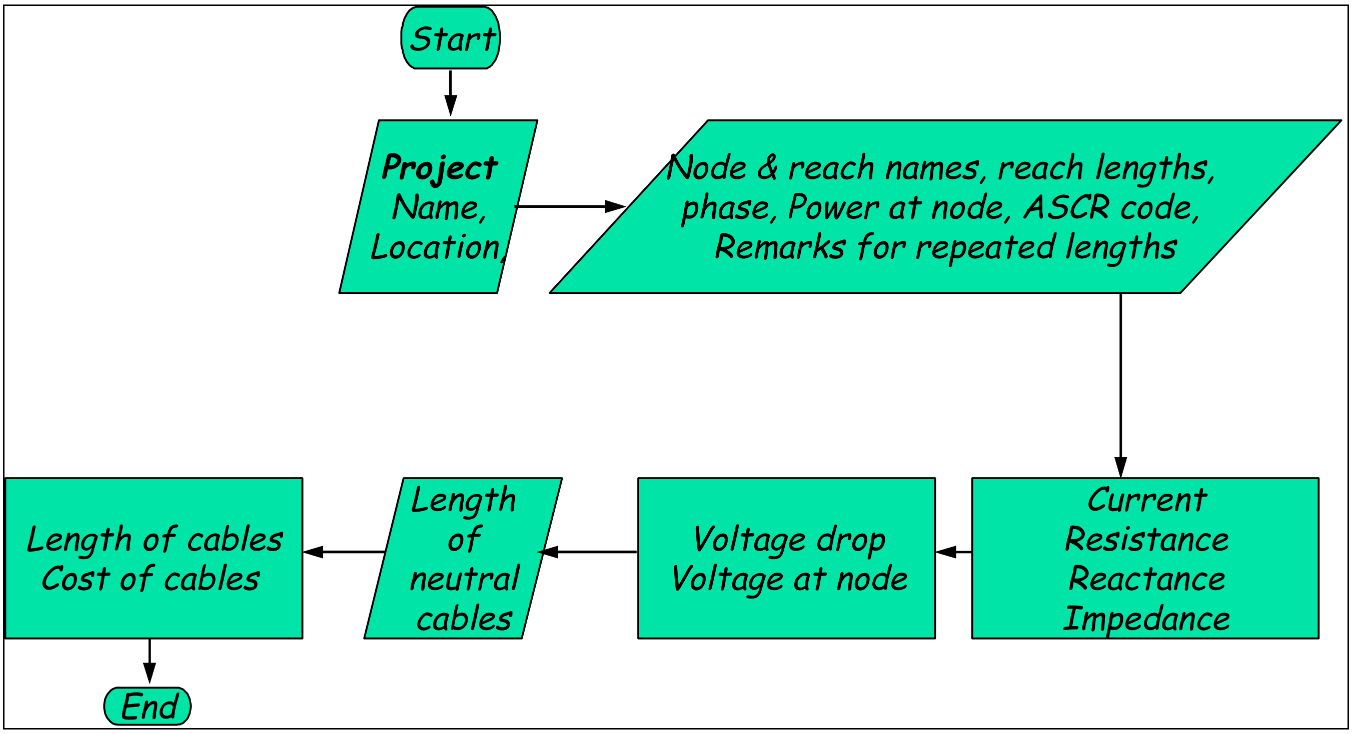


Figure 10.1: Flow chart of transmission and distribution line computation.

The grid and load presented in Figure 10.2 are used for the calculations presented in Figure 10.3.

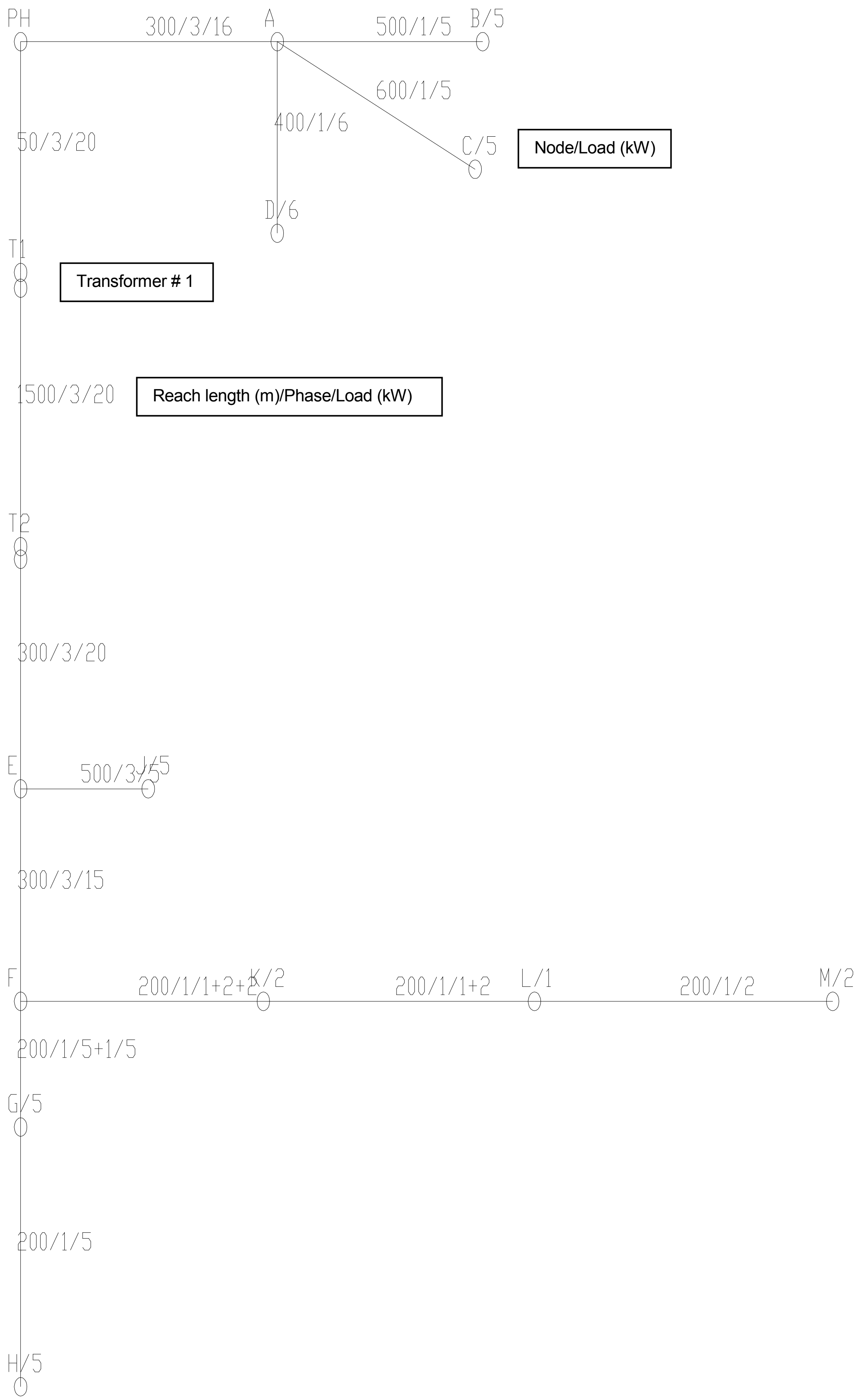


Figure 10.2: Transmission line and load used for the example.

Transmission and Distribution System:

Name of the project:			Upper Jogmai, Ilam				
Node name		Reach name	Reach Length (km)	Phase 1,3,11	Power at next node (kW)	ACSR type	Vrated @ node & Current (A)
PH-A-B-C-D				3			400.00
PH	A	PHA	0.450	3	16	Dog	28.87
A	B	AB	0.660	1	5	Rabbit	27.64
	C	AC	0.090	1	5	Rabbit	30.65
	D	AD	0.090	1	6	Rabbit	33.68
PH-T1				3			400.00
PH	T1	PHT1	0.050	3	20	Otter	36.08
T1-T2				11			11000.00
T1	T2	T1 T2	1.500	11	20	Squirrel	1.31
T2-E				3			400.00
T2	E	T2 E	0.300	3	20	Dog	36.08
E-J (r)				1			226.90
T2	J	T2 J	0.300	1	20	Dog	110.18
E-H (y)				1			226.90
E	F	EF	0.300	1	6	Otter	33.05
F	H	FH	0.400	1	5	Otter	28.62
E-G (b)				1			226.90
E	F	E F	0.300	1	7	Rabbit	38.56
F	G	FG	0.200	1	5	Rabbit	29.37
E-M (r)				1			226.90
E	F	EF	0.300	1	1	Squirrel	5.51
F	M	FM	0.600	1	1	Squirrel	5.62
F-K (y)				1			218.40
F	K	F K	0.180	1	2	Squirrel	11.45
F-L(b)				1			212.80
F	L	F L	0.180	1	1	Squirrel	5.87

Total length of cables (km)			7.02	0.00	10.00	2.68	1.55	2.85
Length of neutral cables (km)					10			
Cost of cables(Rs.)		517720.00						
Reach Voltage drop (V)	**Volt at node branch (V)**	**% voltag drop**	**Squirrel**	**Gopher**	**Weasel**	**Rabbiit**	**Otter**	**Dog**
	400.00							
8.40	391.60	2.10						1.35
22.20	203.89	13.45				1.32		
3.40	222.69	3.18				0.18		
3.70	222.39	3.31				0.18		
	400.00							
1.30	398.70	0.32					0.15	
	11000.00							
4.70	10995.30	0.04	4.50					
	400.00							
7.00	393.00	1.75						0.90
	226.90							
24.50	202.40	12.00						0.60
	226.90							
8.50	218.40	5.04					0.60	
9.80	208.60	14.35					0.80	
	226.90							
14.10	212.80	7.48				0.60		
7.20	205.60	18.09				0.40		
	226.90							
4.60	222.30	3.35	0.60					
9.30	213.00	10.74	1.20					
	218.40							
5.70	212.70	7.52	0.36					
	212.80							
2.90	209.90	8.74	0.36					

Figure 10. 3: Typical example of a low voltage transmission line.

10 LOADS AND BENEFITS

10.1 INTRODUCTION AND DEFINITIONS

By allocating different loads in different time slots, a micro hydro plant can be optimised resulting in the maximum benefits. The presented spreadsheet may help to fulfil this objective.

10.2 MGSP/ESAP GUIDELINES AND STANDARDS

13. Maximum subscription wattage should not exceed 120W per household.

13. Minimum of 10% productive end use is mandatory.

13. Multipurpose scheme is preferable.

10.3 PROGRAM BRIEFING AND EXAMPLE

10.3.1 Program Briefing

The flow chart of the load and benefits analyses used in the spreadsheet is presented in Figure 11.1. An example is presented in the in Figure 11.2. The main features and assumptions are:

1. For the first three years of operation, one set of domestic and five different end uses can be defined in five different time slots in the 24-hour load duration curve.
2. Probable business load after three years of operation can defined based on the AEPC requirements.
3. Annual available energy, annual load, productive end use load factor and annual total income are calculated and subsequently used in the financial analyses.
4. A load duration chart for the first three years of operation is presented at the end of the spreadsheet. This chart is very helpful in planning and allocating different load so that the benefits are maximized.

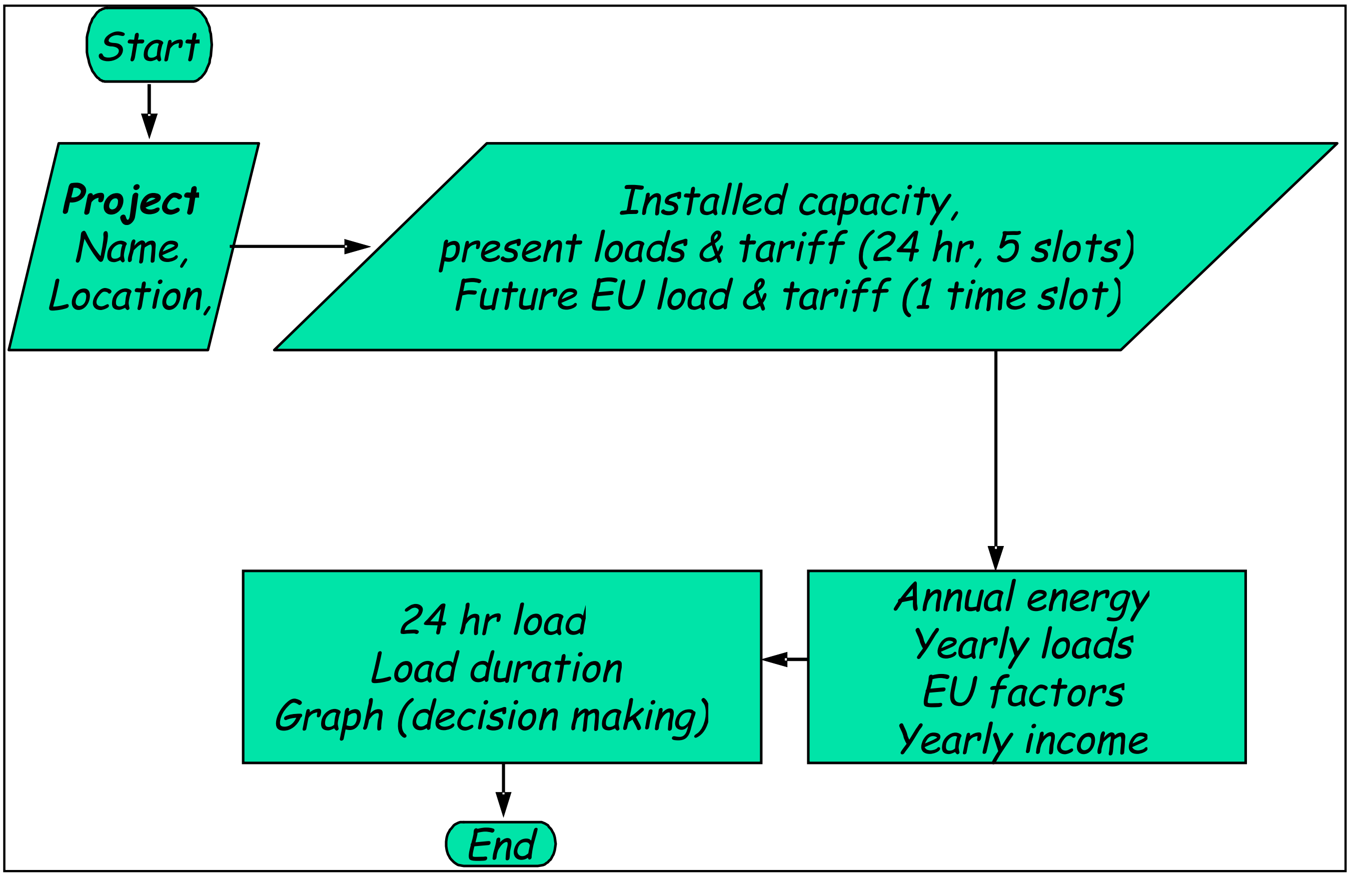


Figure 11.1: Flow chart of the load and benefits calculation spreadsheet.

10.3.2 Typical example of loads and benefits

LOADS AND BENEFITS

INPUT

General

Power Output (kW)	96.1
Name of the Source	Gadi Gad Khola
Location	Ladagada VDC, Doti
Beneficiary HH (nos.)	1035
Plant's operating days	330

Domestic lighting

Average subscription/household (W/HH)			80		
System loss	10%				
time	0	5	8	18	20
load	91.08	0	0	91.1	0

Loads (kWh or W/m	Operating	Tariff (Rs)
Domestic	330	1
Agro-processing	330	6
Bakery	320	6.00
Saw Mill	300	5.00
Herbs Processing	180	5.50
load 5	330	
load 6	330	

Probable Business Load Expected after 3 years

	Operatir	Tariff (Rs)	Load	From	To (hr)
Metal Workshop	330	6.00	10	12	16
Photo studio	320	6.00	1	8	20
Dairy Processing	320	6.00	8	8	18
Cold store	310	6.00	6	8	18
load 5					
load 6					

Proposed end uses and operting hours

time (hr)	0	4	12	16	22
Agro-processing	0	0	22.5	0	0
time (hr)	0	8	14	18	22
Bakery	0	9	0	0	0
time (hr)	0	3	16	18	22
Saw Mill	0	0	10	0	0
time (hr)	0	3	12	17	22
Herbs Processing	0	0	25	0	0
time (hr)	0	6	8	15	22
load 5	15	0	0	0	0
time (hr)	0	3	5	10	22
load 6	12	0	0	0	0

OUTPUT

Summary

Annual Available kWh	761112	
	First 3 years	**After 3 years**
Yearly load (kWh)	117060	147680
Productive end use load factor (%)	15.38	19.40
Annual total (domestic + end uses) Income (R	1,429,230	1,612,950

End Use	Load (kW)	Operation Period Hours/day	Days/year	Yearly Load kWh	LF (%)	**Annual Income (Rs)**
Domestic Lighting	91.08	7	330	210394.8	27.64	**993,600**
Existing/Committed Business Load						
Agro-processing	22.5	4	330	29700	3.90	178200
Bakery	9	6	320	17280	2.27	103680
Saw Mill	10	2	300	6000	0.79	30000
Herbs Processing	25	5	180	22500	2.96	123750
load 5	15	6	330	29700	3.90	0
load 6	12	3	330	11880	1.56	0
Total				**117060**	**15.38**	**435,630**
Total Annual Income from sales of electricity						**1,429,230**
Probable Business Load after 3 years						
Metal Workshop	10	4	330	13200	1.73	79200
Photo studio	1	12	320	3840	0.50	23040
Dairy Processing	8	10	320	25600	3.36	153600
Cold store	6	10	310	18600	2.44	111600
load 5	0	0	0	0	0.00	0
load 6	0	0	0	0	0.00	0
Total additional annual income after 3 years				61240	8.05	**183,720**
Productive End Use (	19.40					

Figure 11.2: An example of load and benefits calculation.

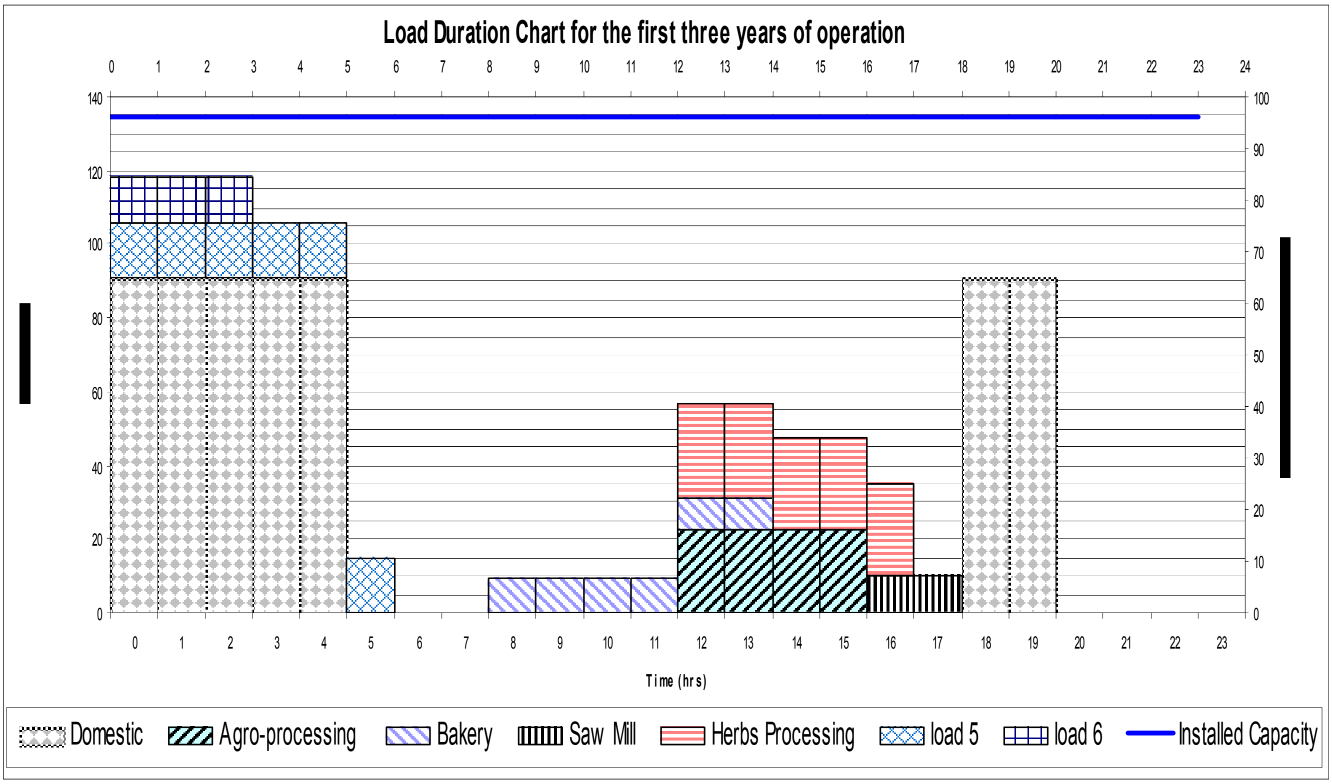


Figure 11.3: Load duration curve

It can be seen from the load duration curve presented in Figure 11.3 that the project is mainly dominated by domestic load. Other end uses can be incorporated within 05 to 16 hours. If the project has to share water with other existing water utilities such as irrigation system, etc. This can be arranged during the non-operating hours or during partial load period. This load duration curve can also be used to maximize benefits even at lower tariff during such hours.

13. COSTING AND FINANCIAL ANALYSES

13.1. INTRODUCTION AND DEFINITIONS

This spreadsheet tests the financial viability of the project based on the guidelines and standards set aside by AEPC.

13.2. MGSP/ESAP GUIDELINES AND STANDARDS

1. 15 years as the economic life span of the project for calculating financial parameters.
2. Total cost of the project including subsidy should be limited to

Table 12.1: Per kilowatt subsidy and cost ceiling as per AEPC

Walking distance	Subsidy	Ceiling
less than 2 days walking distance	70000	150000
2-5 days walking distance	78750	158750
more than 5 days walking distance	91500	171500

3. Net present value of equity investment at a discount rate of 4% should be positive.

13.3. PROGRAM BRIEFING AND EXAMPLE

13.3.1. Program Briefing

The spreadsheet presented takes the total costs, financing of the project and annual cost as inputs to calculate the financial parameters such as the net present value, cost per kilowatt, etc. The flow chart on which the spreadsheet is based is presented in Figure 12.1. Annual cash flows for the stated planning horizon is presented and used to calculate different financial parameters.

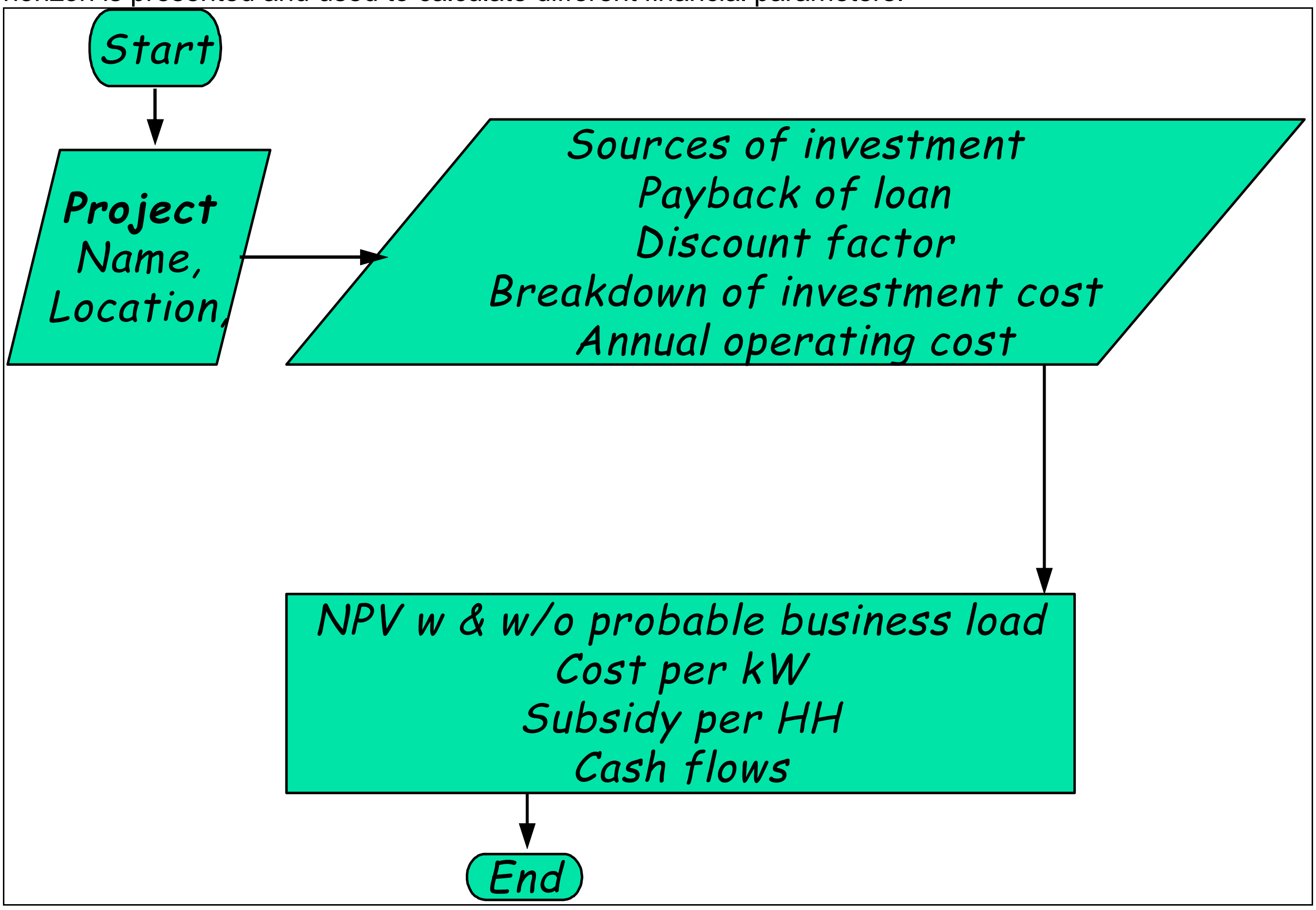


Figure 12.1: Flow chart for Project costing and financial analyses.

13.3.2. Typical example of costing and financial analyses

A typical example of costing and financial analyses based on the flow chart presented in Figure 12.1 is presented in Figure 12.2.

Project Costing and Financial Analyses

INPUT

Total Project Cost (Rs.)		**8,516,715**					
more than 5 days walking distance	Subsidy/kW	Total subsidy	Bank loan	Other loan	Cash equity	Kind equity	Others
	91,500	4575000	1,890,044		1,200,000	851,671	0
Interest rate (%)			3%				
Payback period (yr)			7				
Plant life (yr)		15					
Discount Rate (%)		4%					

Investment Cost (Rs)	**8,516,715**		
Mechanical components	999,040	Installation	232,500
Electrical component	2,061,717	Commissioning	0
Civil component	1,363,497	VAT	623,611
Spare parts & tools	57,550	Contingencies	0
Transport.	3,178,800	Others	

Annual Operating Cost (Rs)	**305,004**				
Salary	Spares	Maintenance	Office expenses	Miscellaneous	Others
114,000	0	171,000		20,004	

Summary

Project cost (Rs)	8,516,715
Annual Operation, Maintenance and other Costs (Rs)	305,004
Annual Income without probable business loads (Rs)	826230
Annual Income with probable business loads (Rs)	855180
Annual installment for Bank loan	303364
Annual installment for other loan	NA
NPV on equity without probable business load (Rs)+ve	2,667,678
NPV equity with probable business load (Rs)+ve	2,899,926
Cost/Kw =>>Ok	170,334
Subsidy/HH	9,713

NPV on ====>>>	Without probable business load (Rs)	With probable business load (Rs)
Total cost	(4,367,625)	(4,135,377)
Total Cost -Subsidy	31,413	263,662
Equity	2,667,678	2,899,926
After Loan Repayment	3,509,282	3,704,195

Cash Flows

Year =>>	0	1	2	3
Annual income without probable business loads		826230	826230	826230
Annual income with probable business loads		826230	826230	826230
Total Equity	1200000			
Annual O & M costs		305,004	305,004	305,004
Loan repayment		303364	303364	303364
Cash flow without probable business load	-1200000	217862	217862	217862
Cash flow with probable business load	-1200000	217862	217862	217862

4	5	6	7	8	9	10	11	12	13	14	15
826230	826230	826230	826230	826230	826230	826230	826230	826230	826230	826230	826230
855180	855180	855180	855180	855180	855180	855180	855180	855180	855180	855180	855180
305,004	305,004	305,004	305,004	305,004	305,004	305,004	305,004	305,004	305,004	305,004	305,004
303364	303364	303364	303364	0	0	0	0	0	0	0	0
217862	217862	217862	217862	521226	521226	521226	521226	521226	521226	521226	521226
246812	246812	246812	246812	550176	550176	550176	550176	550176	550176	550176	550176

Figure 12.2: A typical example of project costing and financial analyses.

13. UTILITIES

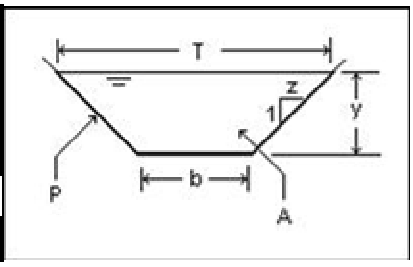
13.1. INTRODUCTION

In this spreadsheet small tools for independent calculations are presented. These calculations are similar to the calculations presented in other spreadsheet for individual components. The main aim of these small calculations is to provide quick and handy computation of:

1. Uniform depth of a rectangular or trapezoidal canal.

Uniform Depth of a Trapezoidal Canal (Y-m)

Design Discharge (l/s):	200.000
1/Mannings Coeff (M):	65.0000
1/Canal Slope (S):	50
Width of Canal (b-m):	0.500
Unlined firm/gravelyl/clay/side hill c/s in average loam cut (z m)	1
Uniform Depth (Y-m)	**0.150**



2. Payment of loan for different periods such as monthly, quarterly and yearly.

Payment of a loan

Loan amount:	1,800,000
Interest rate (APR):	6.00%
Monthly payments and No	12
Monthly Payment	**154,919.57**

3. Power calculations

Power MGSP-ESAP (Pe-kW)

Discharge (l/s):	160
Cumulative efficiency(n%)	65.00%
Head (H-m)	27.50
Actual Power (Pact-kW)	**28.06**
Power MGSP-ESAP (Pe-kW)	**21.58**

4. Spillway sizing.

Spillway Lengths (m)

Flood discharge (l/s):	540
Design discharge (l/s):	126
Overtopping height (ho) mm:	150
Spillway discharge coeff	2.1
L spillway min for Qf m & full height	**4.43**
Length of spillway Ls1 for Qf m & half height	**9.60**

5. Voltage drops of transmission line.

Voltage Drop

Reach length (km)	7.000
Voltage at 1st node (V)	11,000
Power (kW)	190
ASCR type	Squirrel
Phase at 1st node (1/2/11(for 11kV or above)	11
Phase at 2nd node (1/2/11(for 11kV or above)	11
Current (A)	**12.47**
Impedence Ohm/km	**1.3803**
Voltage at 2nd node (kV) 208.6V ,1.9%	**10,791.40**

6. Pipe friction factor.

Friction Factor (f) & Net head

Discharge (m3/s)	0.160
Gross head (m)	20
Pipe roughness ks (mm)	0.060
Pipe diameter (m)	260.00
Pipe Length (m)	140
Turbulent headloss factor (K)	1.50
Friction factor f	**0.0153**
Net Head (m), hl=4.52m 22.59%	**15.48**

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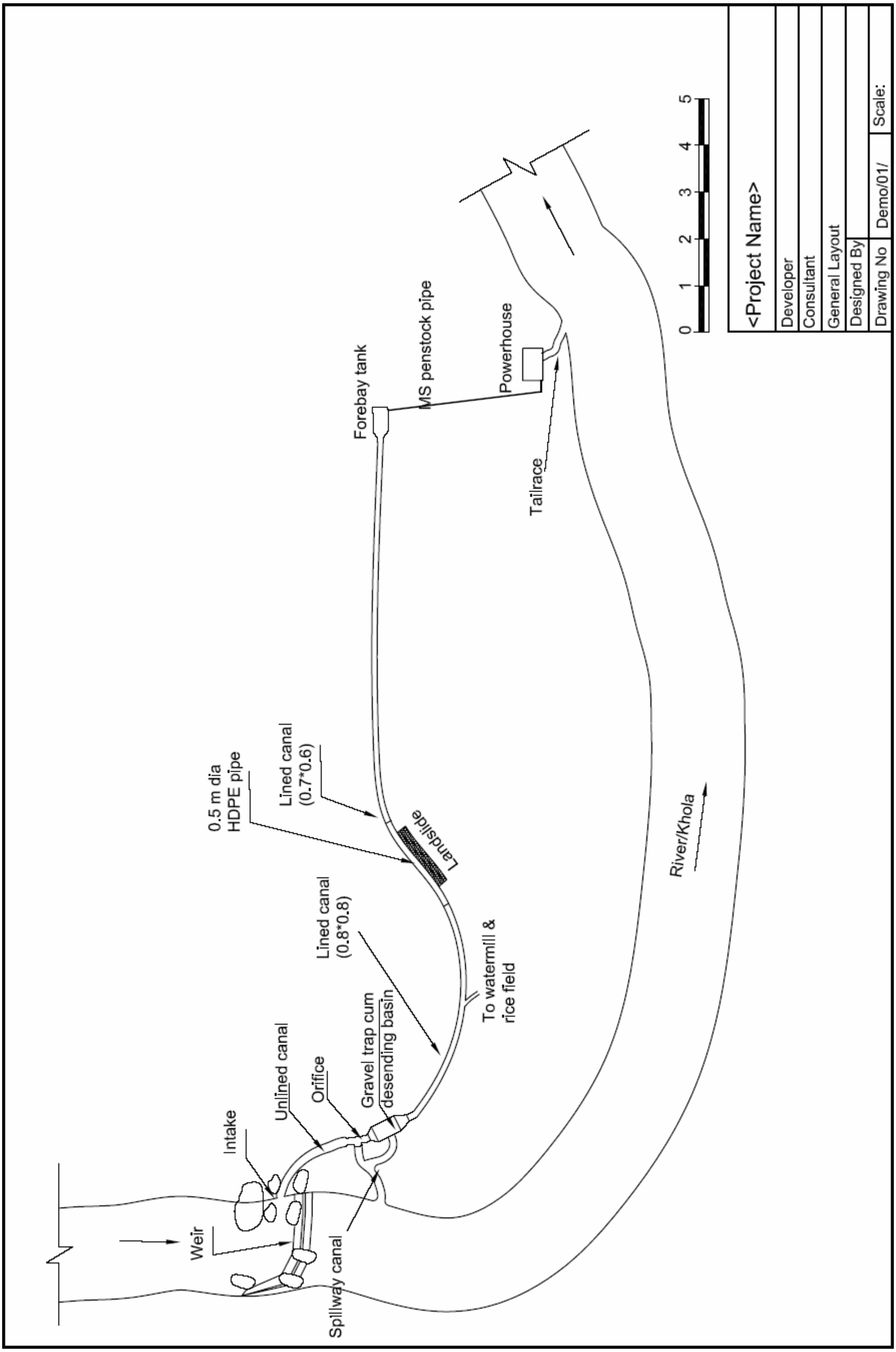
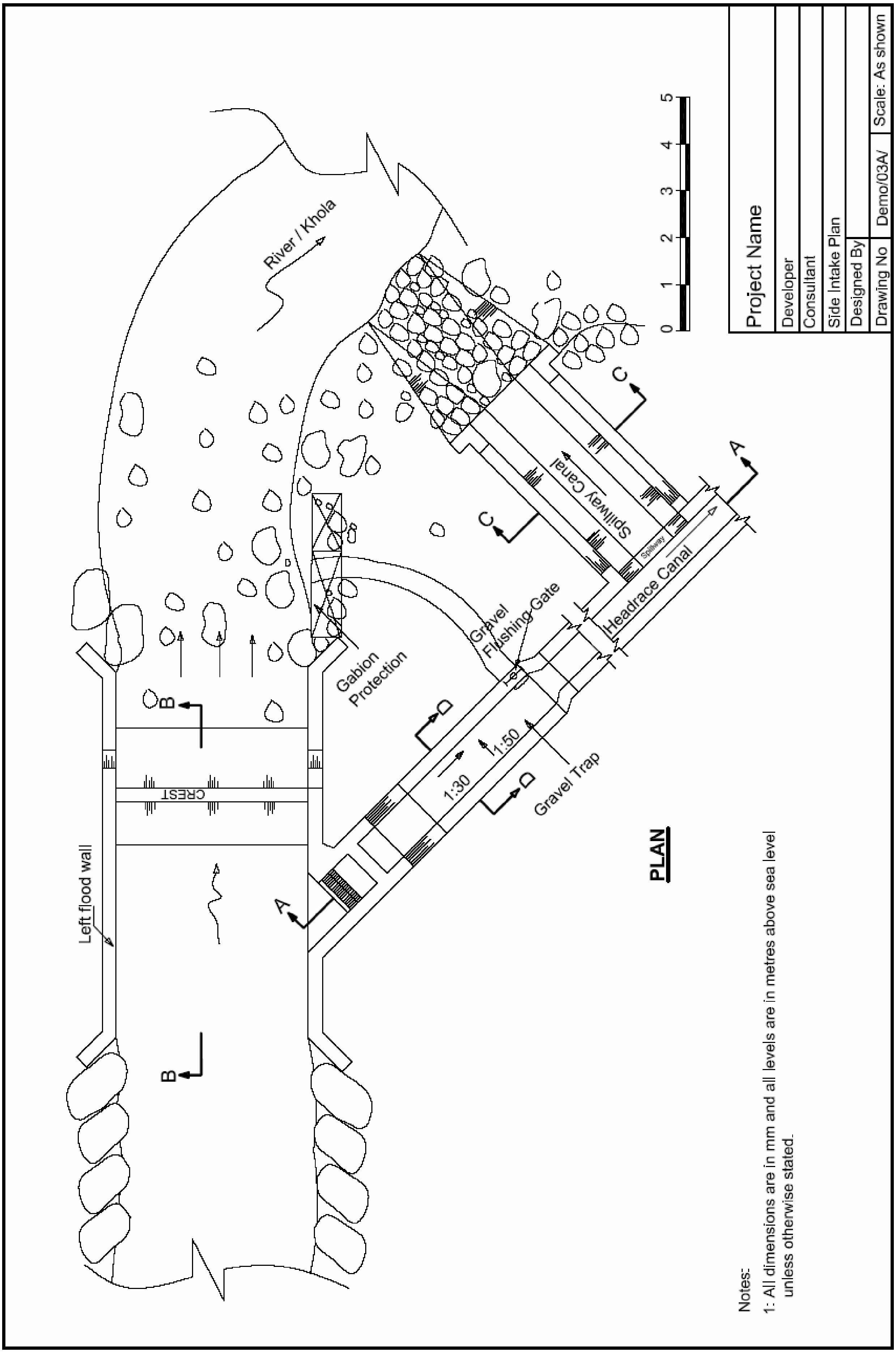
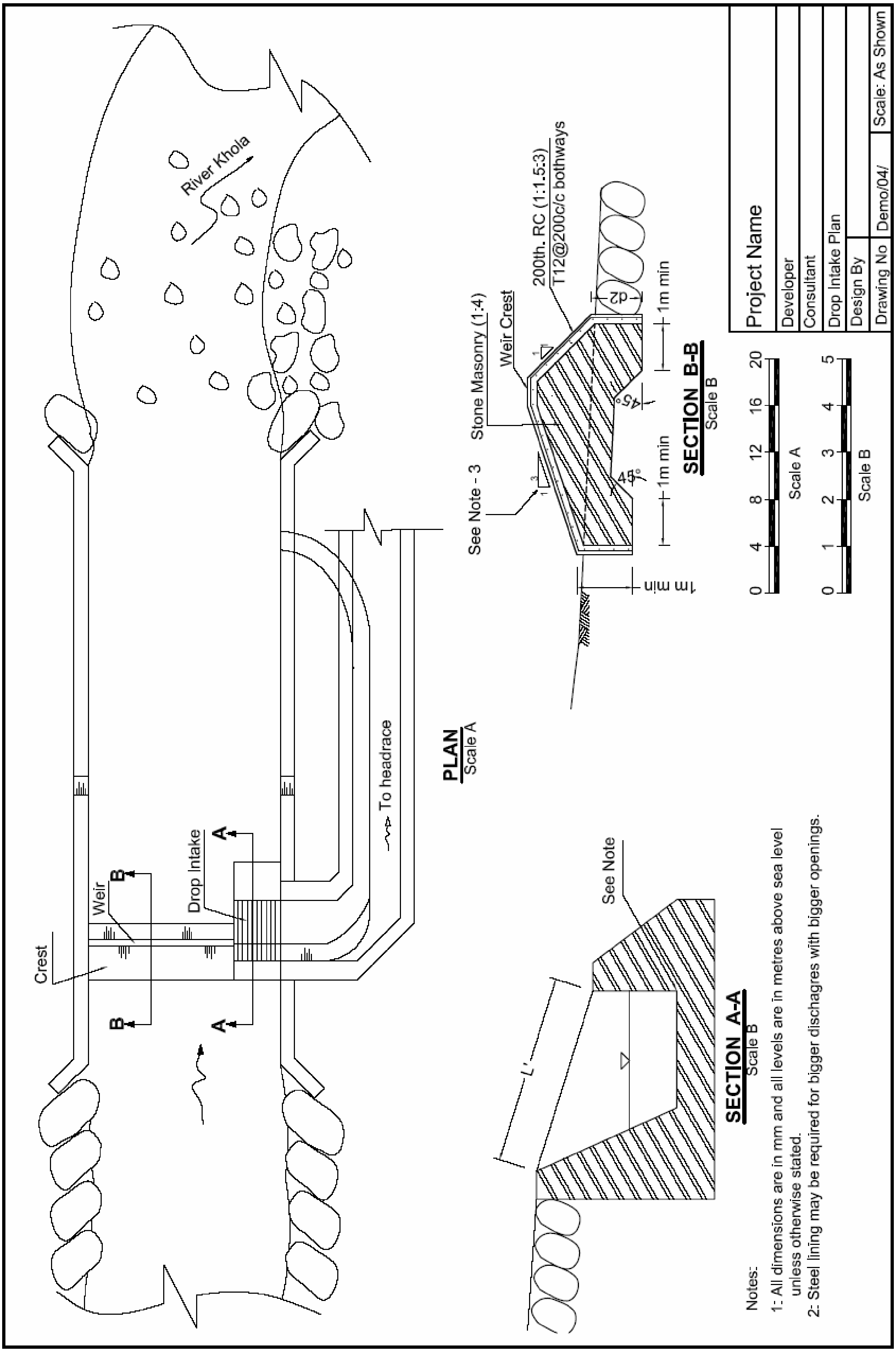
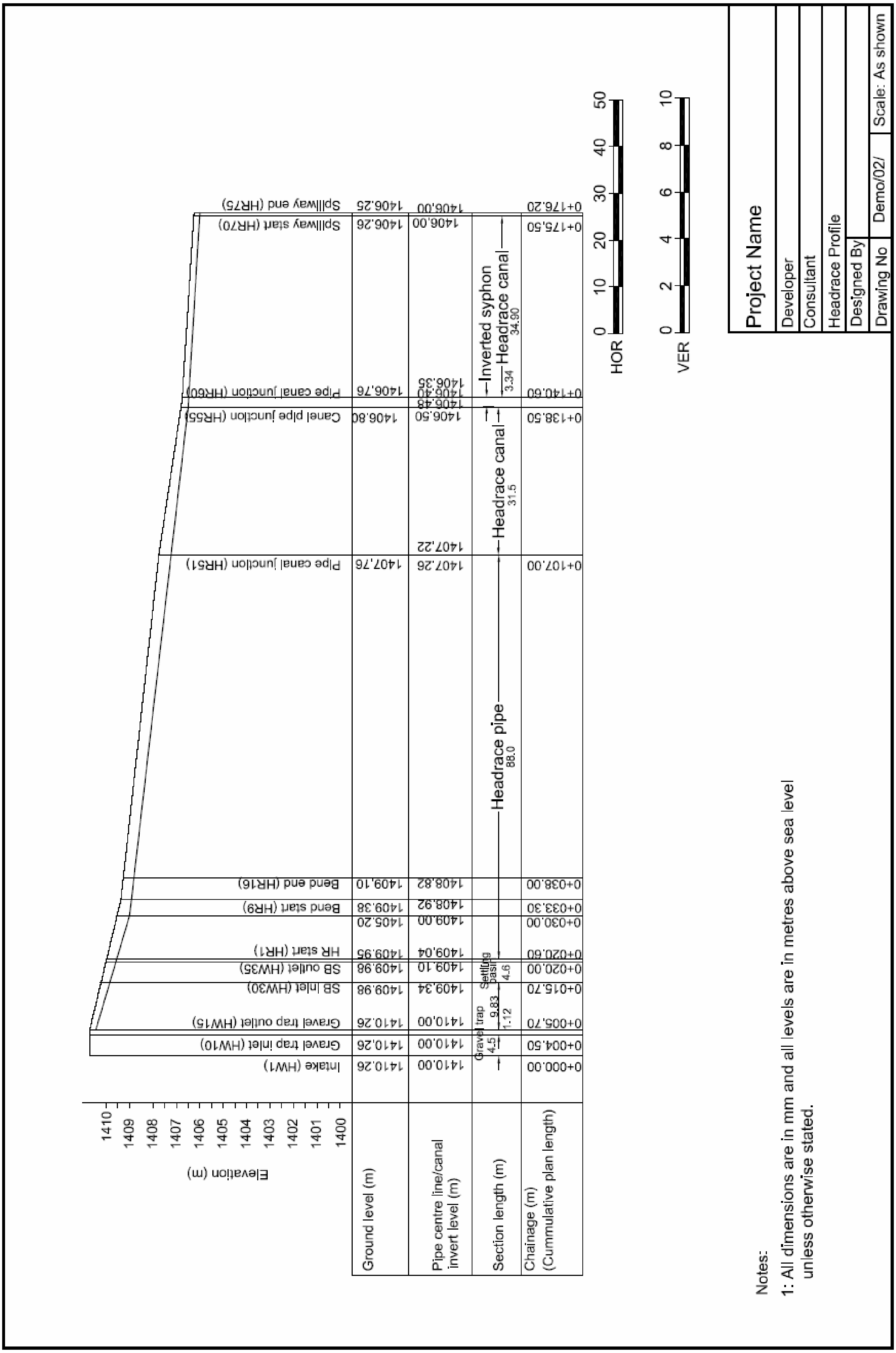
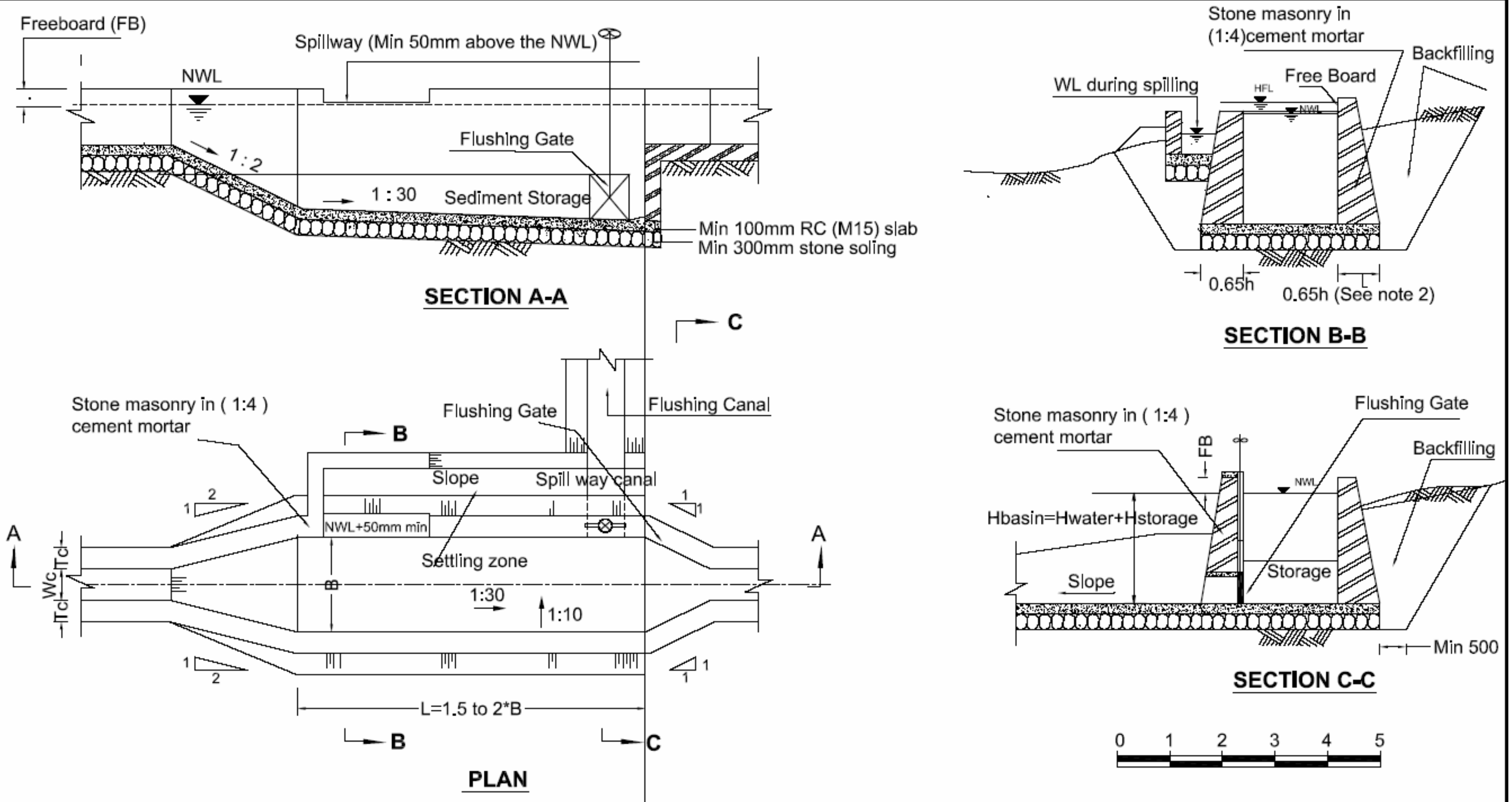
APPENDICES


Weir
Intake
Unlined canal
Orifice
Gravel trap cum desending basin
Spillway canal
Lined canal (0.8*0.8)
To watermill & rice field
0.5 m dia HDPE pipe
Landslide
Lined canal (0.7*0.6)
Forebay tank
MS penstock pipe
Powerhouse
Tailrace
River/Khola
0 1 2 3 4 5
<Project Name>
Developer
Consultant
General Layout
Designed By
Drawing No
Demo/01/
Scale:


River / Khola
Left flood wall
CREST
Gabion Protection
Gravel Flushing Gate
Gravel Trap
1:30
1:50
Headrace Canal
Spillway Canal
Spillway
A
B
C
D
0
1
2
3
4
5
PLAN
Notes:
1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.
Project Name
Developer
Consultant
Side Intake Plan
Designed By
Drawing No
Demo/03A/
Scale: As shown


River Khola
Crest
Weir
Drop Intake
B
A
To headrace
PLAN
Scale A
See Note - 3
Stone Masonry (1:4)
Weir Crest
200th. RC (1:1.5:3)
T12@200c/c bothways
d2
45°
1m min
SECTION B-B
Scale B
See Note
L'
SECTION A-A
Scale B
Notes:
1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.
2: Steel lining may be required for bigger dischagres with bigger openings.
0 4 8 12 16 20
Scale A
0 1 2 3 4 5
Scale B
Project Name
Developer
Consultant
Drop Intake Plan
Design By
Drawing No
Demo/04/
Scale: As Shown


Elevation (m)
1410
1409
1408
1407
1406
1405
1404
1403
1402
1401
1400
Ground level (m)
Pipe centre line/canal invert level (m)
Section length (m)
Chainage (m) (Cummulative plan length)
Intake (HW1)
1410.26
1410.00
0+000.00
Gravel trap inlet (HW10)
1410.26
1410.00
0+004.50
Gravel trap outlet (HW15)
1410.26
1410.00
0+005.70
SB inlet (HW30)
1409.98
1409.34
0+015.70
SB outlet (HW35)
1409.98
1409.10
0+020.00
HR start (HR1)
1409.95
1409.04
0+020.60
1405.20
1409.00
0+030.00
Bend start (HR9)
1409.38
1408.92
0+033.30
Bend end (HR16)
1409.10
1408.82
0+038.00
Pipe canal junction (HR51)
1407.76
1407.26
1407.22
0+107.00
Canal pipe junction (HR55)
1406.80
1406.50
0+138.50
Pipe canal junction (HR60)
1406.76
1406.48
1406.40
1406.35
0+140.60
Spillway start (HR70)
1406.26
1406.00
0+175.50
Spillway end (HR75)
1406.25
1406.00
0+176.20
Gravel trap
4.5
1.12
9.83
Settling basin
4.6
Headrace pipe
88.0
Headrace canal
31.5
Inverted syphon
3.34
Headrace canal
34.90
HOR
0
10
20
30
40
50
VER
0
2
4
6
8
10
Project Name
Developer
Consultant
Headrace Profile
Designed By
Drawing No
Demo/02/
Scale: As shown
Notes:
1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.

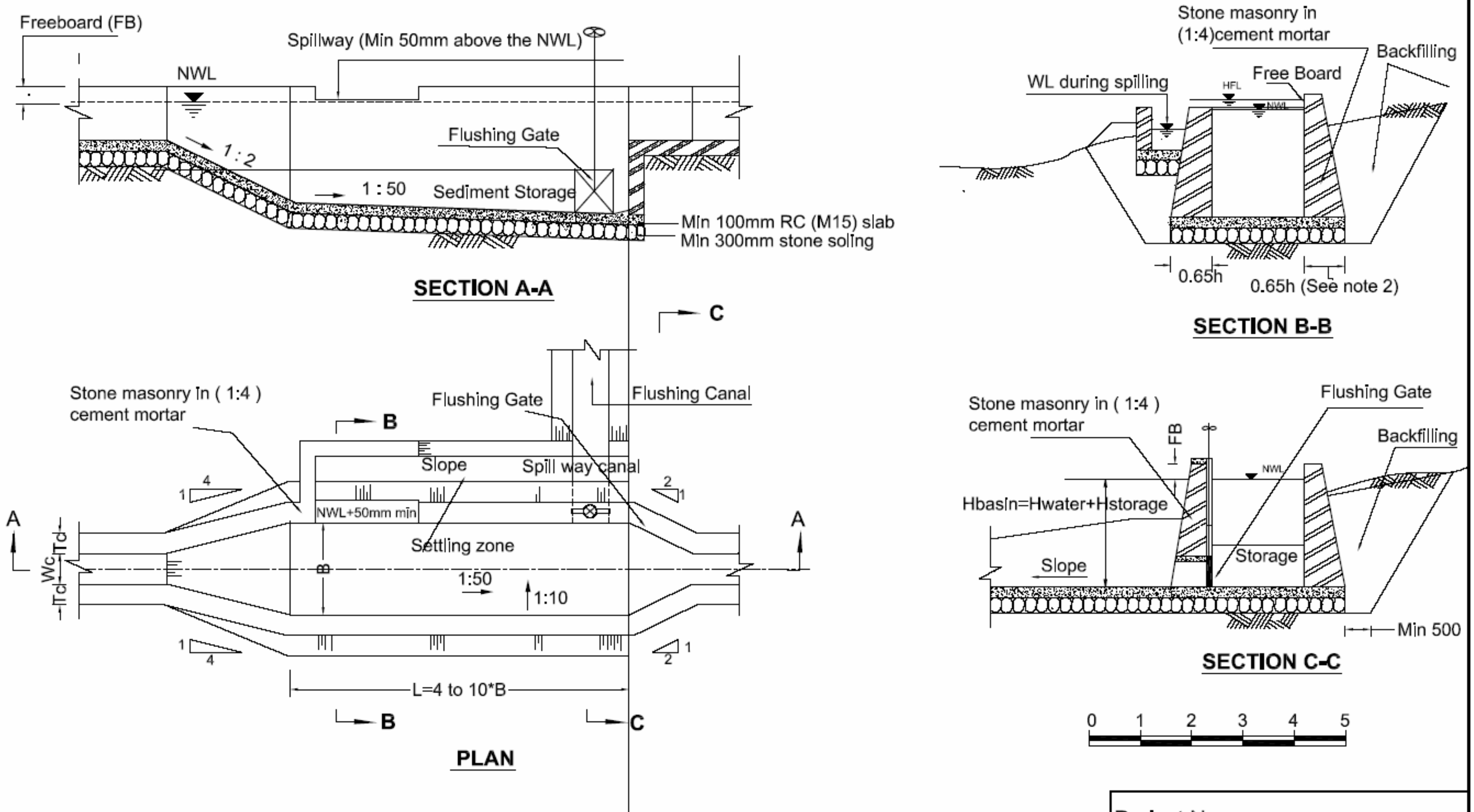


Notes:

1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.

2: For stone massonry walls the typical base width should be 0.65 times the height of wall.

Project Name		
Developer		
Consultant		
Gravel Trap		
Designed By		
Drawing No	Demo/05A/	Scale: As shown

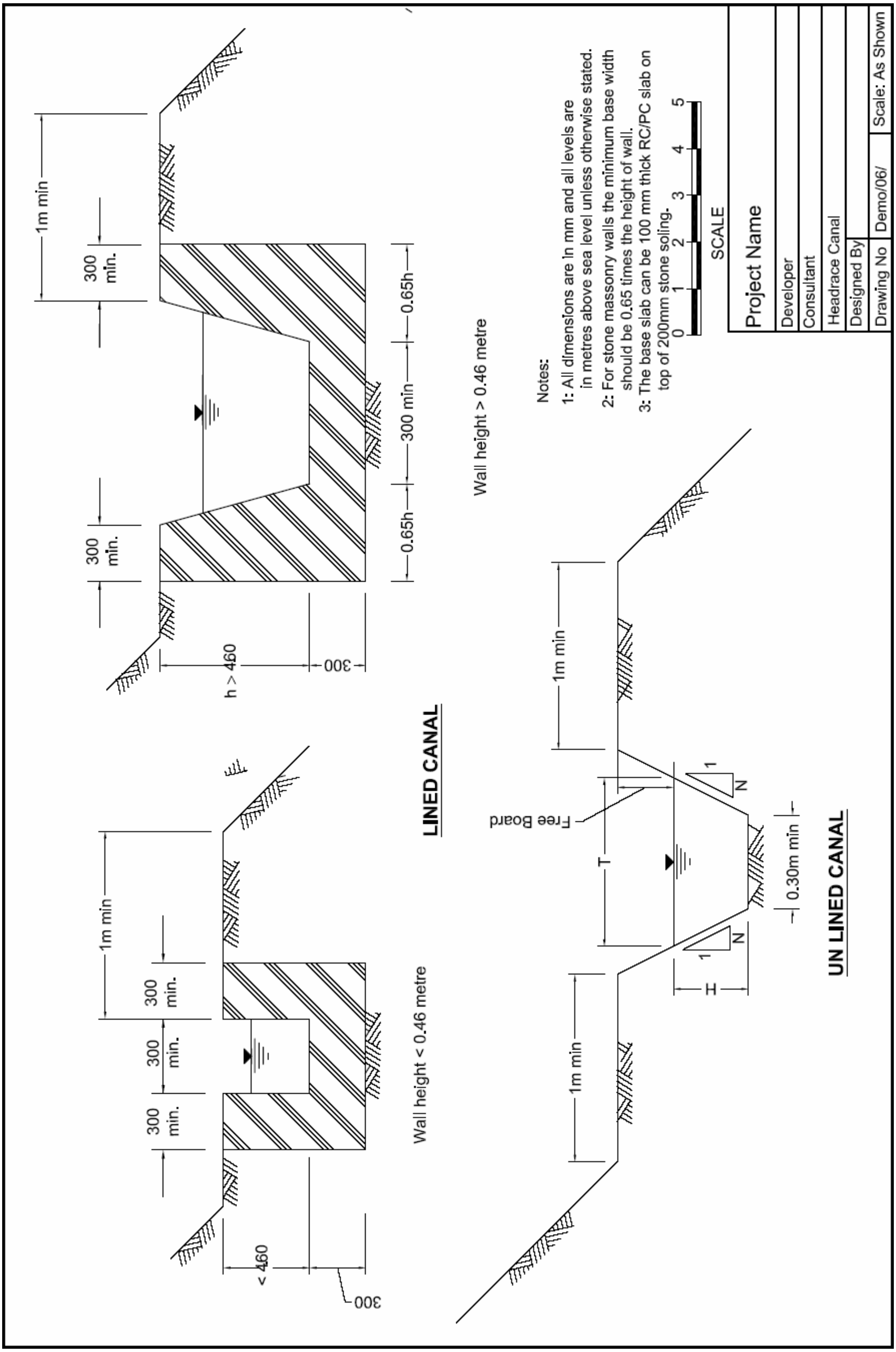


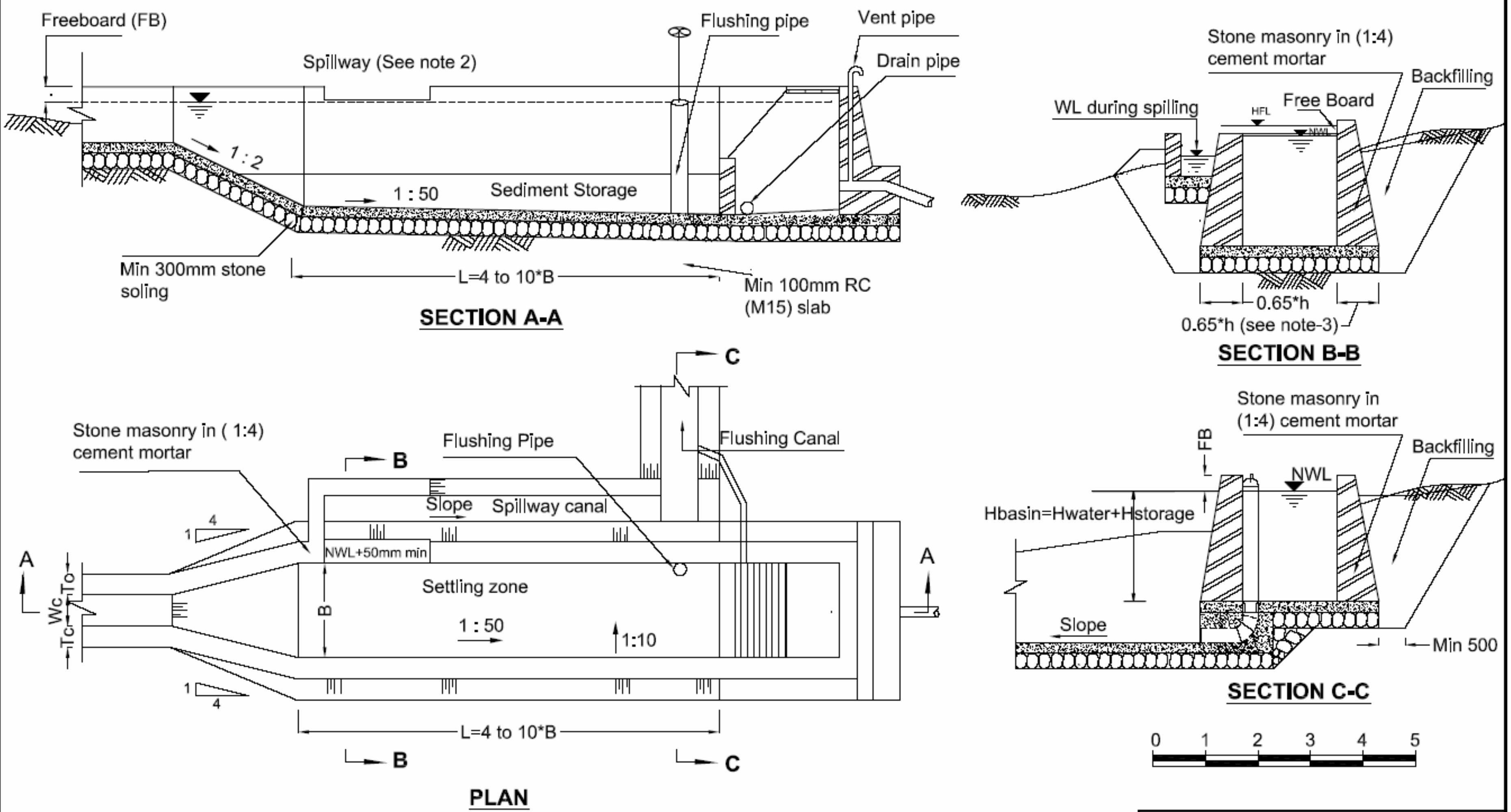
Notes:

1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.

2: For stone massonry walls the typical base width should be 0.65 times the height of wall.

Project Name			
Developer			
Consultant			
Settling Basin			
Designed By			
Drawing No	Demo/05B/	Scale: As shown	


LINED CANAL
Wall height < 0.46 metre
1m min
300 min.
300 min.
300 min.
< 460
300
Wall height > 0.46 metre
1m min
300 min.
300 min.
h > 460
300
0.65h
300 min
0.65h
UN LINED CANAL
1m min
1m min
Free Board
T
H
N
1
N
1
0.30m min
Notes:
1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.
2: For stone massonry walls the minimum base width should be 0.65 times the height of wall.
3: The base slab can be 100 mm thick RC/PC slab on top of 200mm stone soling.
0 1 2 3 4 5
SCALE
Project Name
Developer
Consultant
Headrace Canal
Designed By
Drawing No Demo/06/
Scale: As Shown



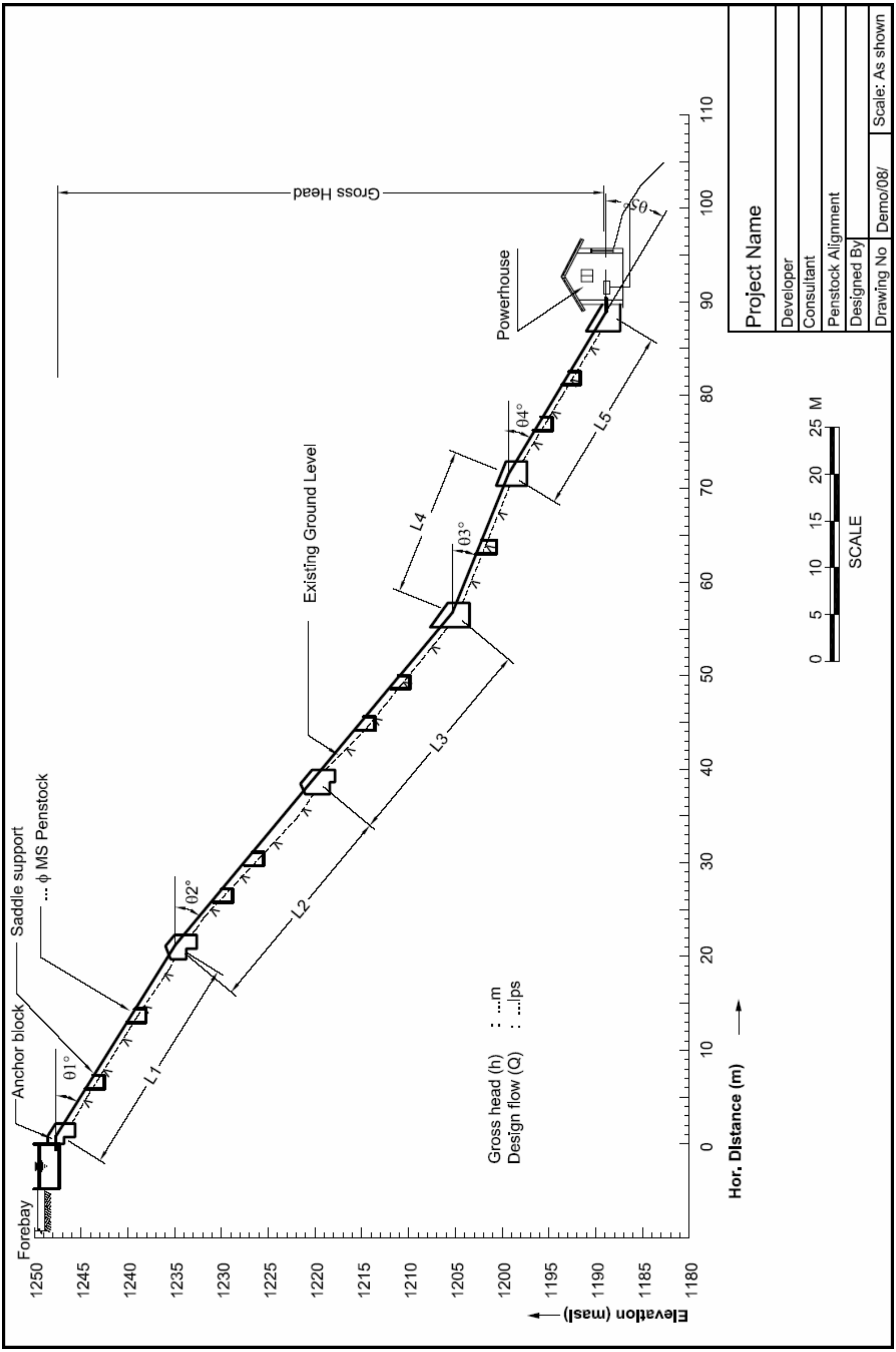
Notes:

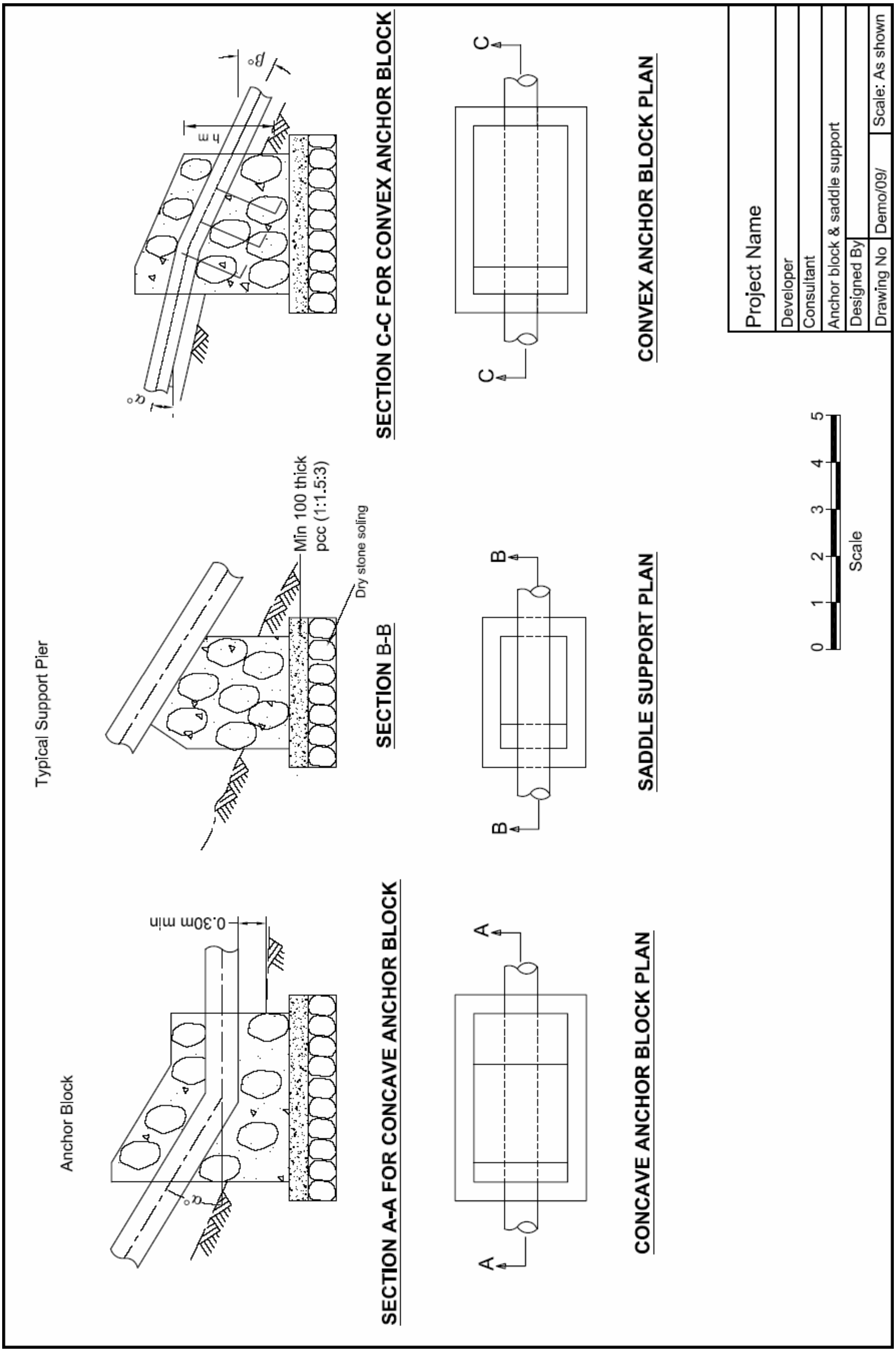
1: All dimensions are in mm and all levels are in metres above sea level unless otherwise stated.

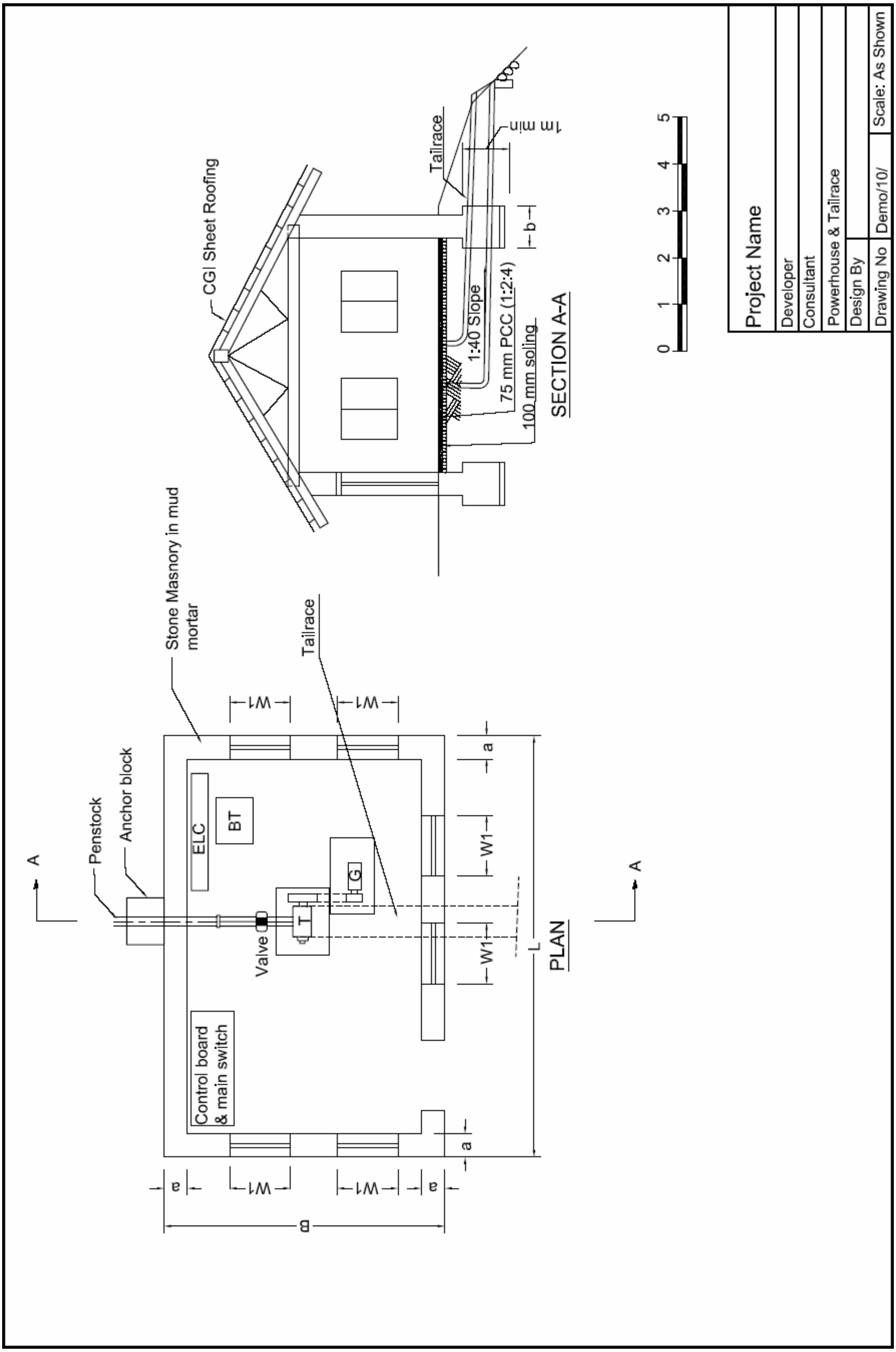
2: The spillway is not be necessary if
   a) The flushing pipe is capable of spilling the desired discharge or
   b) Spilling mechanism is present in GT & FB.

3: For stone massonry walls the minimum base width should be 0.65 times the height of wall.

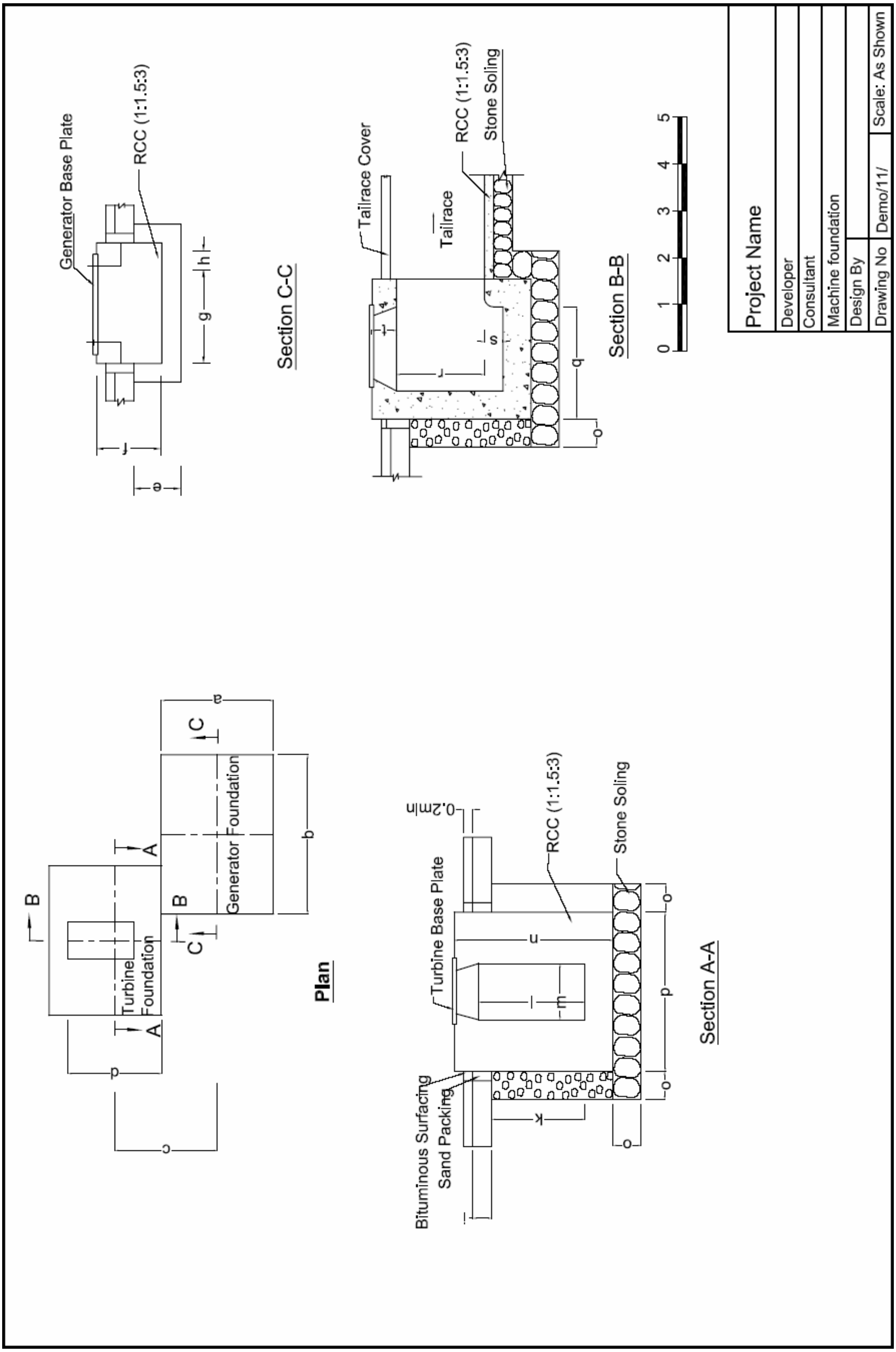
Project Name		
Developer		
Consultant		
Forebay		
Designed By		
Drawing No	Demo/07/	Scale: As shown


Forebay
Anchor block
Saddle support
... φ MS Penstock
θ1°
θ2°
θ3°
θ4°
θ5°
L1
L2
L3
L4
L5
Existing Ground Level
Powerhouse
Gross Head
Gross head (h) : ...m
Design flow (Q) : ...lps
Hor. Distance (m)
Elevation (masl)
SCALE
0 5 10 15 20 25 M
Project Name
Developer
Consultant
Penstock Alignment
Designed By
Drawing No
Demo/08/
Scale: As shown

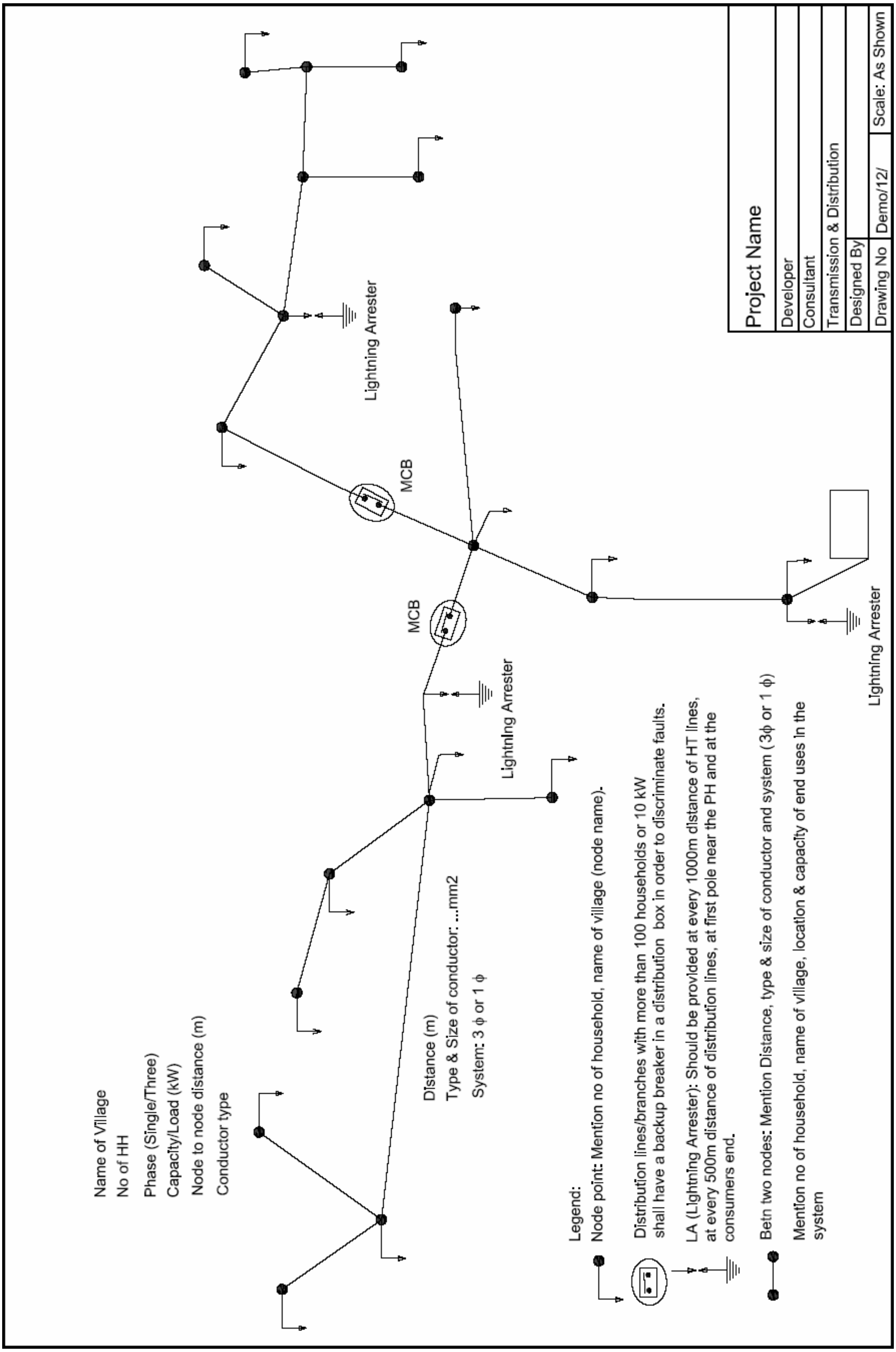

Anchor Block
Typical Support Pier
0.30m min
α°
β°
h m
Min 100 thick
pcc (1:1.5:3)
Dry stone soling
A
B
C
SECTION A-A FOR CONCAVE ANCHOR BLOCK
SECTION B-B
SECTION C-C FOR CONVEX ANCHOR BLOCK
CONCAVE ANCHOR BLOCK PLAN
SADDLE SUPPORT PLAN
CONVEX ANCHOR BLOCK PLAN
0
1
2
3
4
5
Scale
Project Name
Developer
Consultant
Anchor block & saddle support
Designed By
Drawing No
Demo/09/
Scale: As shown

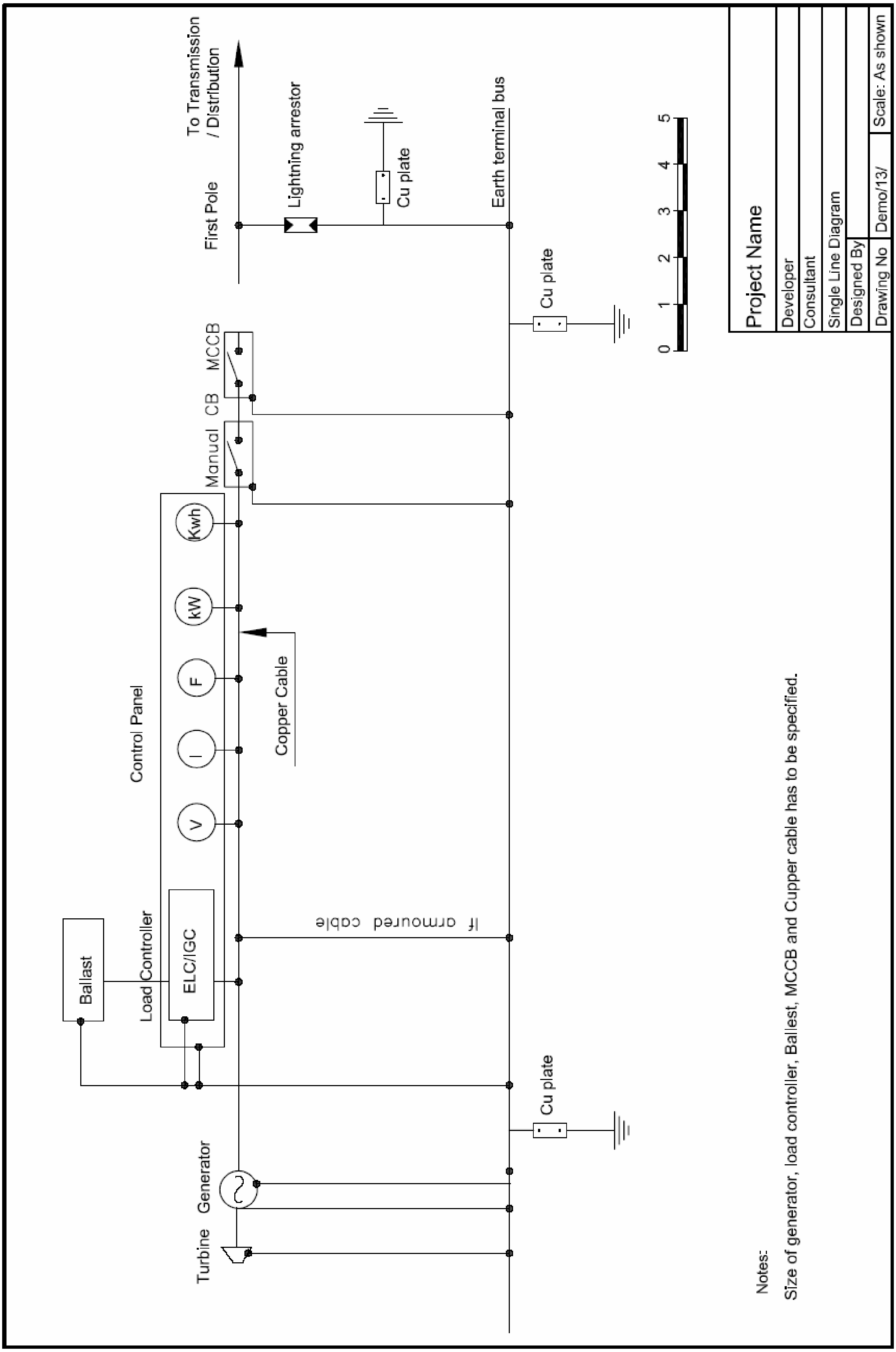


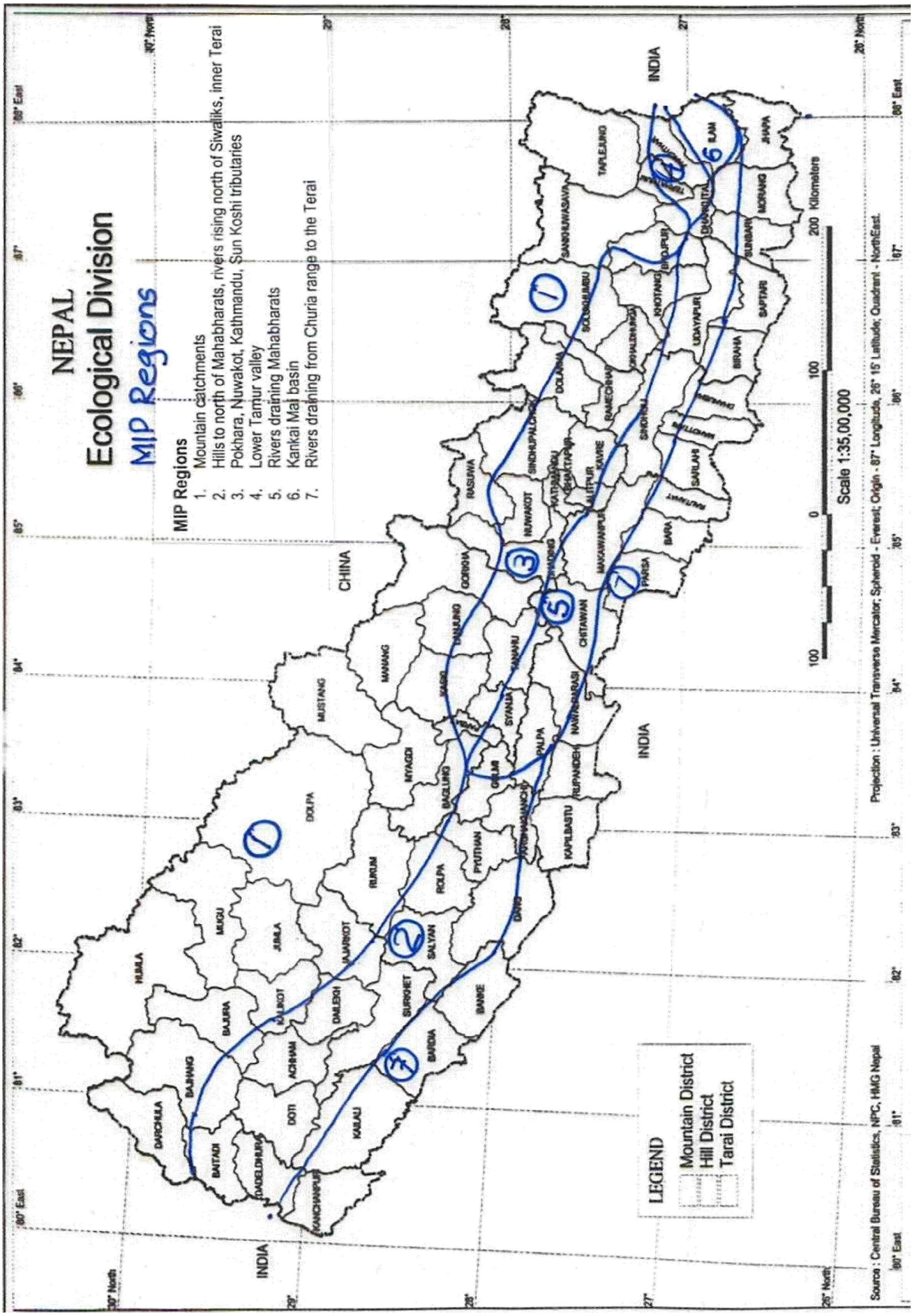
Penstock
Anchor block
Stone Masnory in mud mortar
Tailrace
ELC
BT
G
T
Valve
Control board & main switch
W1
a
B
L
A
PLAN
CGI Sheet Roofing
Tailrace
1m min
b
1:40 Slope
75 mm PCC (1:2:4)
100 mm soling
SECTION A-A
0 1 2 3 4 5
Project Name
Developer
Consultant
Powerhouse & Tailrace
Design By
Drawing No
Demo/10/
Scale: As Shown

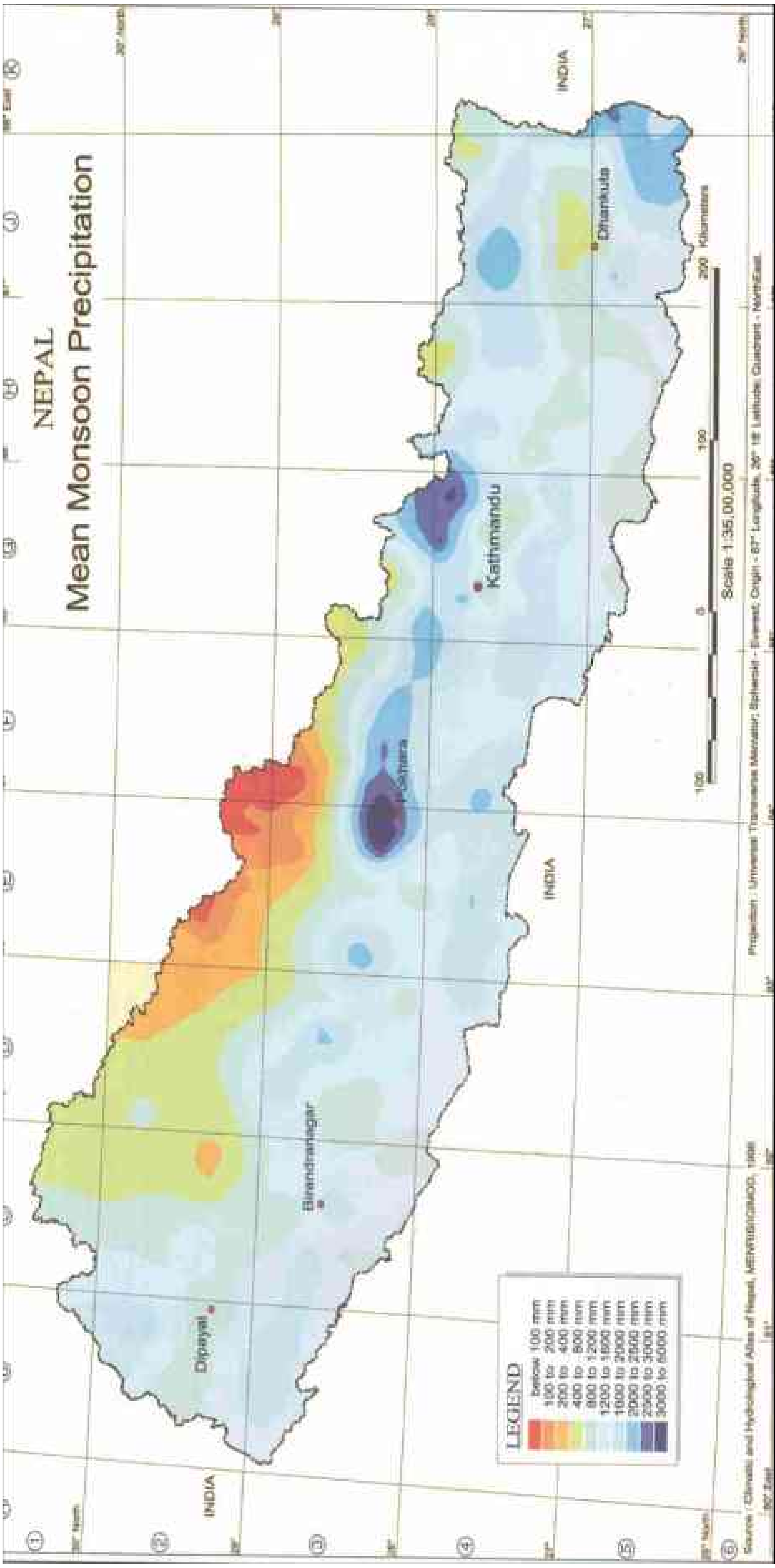


Generator Base Plate
RCC (1:1.5:3)
Section C-C
Tailrace Cover
Tailrace
RCC (1:1.5:3)
Stone Soling
Section B-B
0 1 2 3 4 5
Project Name
Developer
Consultant
Machine foundation
Design By
Drawing No Demo/11/
Scale: As Shown
Turbine Foundation
Generator Foundation
Plan
Turbine Base Plate
RCC (1:1.5:3)
Stone Soling
0.2m
Bituminous Surfacing
Sand Packing
Section A-A


Name of Village
No of HH
Phase (Single/Three)
Capacity/Load (kW)
Node to node distance (m)
Conductor type
Distance (m)
Type & Size of conductor: ...mm2
System: 3 ϕ or 1 ϕ
MCB
MCB
Lightning Arrester
Lightning Arrester
Lightning Arrester
Legend:
Node point: Mention no of household, name of village (node name).
Distribution lines/branches with more than 100 households or 10 kW shall have a backup breaker in a distribution box in order to discriminate faults.
LA (Lighting Arrester): Should be provided at every 1000m distance of HT lines, at every 500m distance of distribution lines, at first pole near the PH and at the consumers end.
Betn two nodes: Mention Distance, type & size of conductor and system (3ϕ or 1 ϕ)
Mention no of household, name of village, location & capacity of end uses in the system
Project Name
Developer
Consultant
Transmission & Distribution
Designed By
Drawing No
Demo/12/
Scale: As Shown


To Transmission / Distribution
First Pole
Lightning arrestor
Cu plate
Earth terminal bus
MCCB
Manual CB
Control Panel
Kwh
kW
F
I
V
Copper Cable
Load Controller
ELC/IGC
Ballast
If armoured cable
Cu plate
Cu plate
Generator
Turbine
0 1 2 3 4 5
Project Name
Developer
Consultant
Single Line Diagram
Designed By
Drawing No Demo/13/
Scale: As shown
Notes:
Size of generator, load controller, Ballest, MCCB and Cupper cable has to be specified.


NEPAL
Ecological Division
MIP Regions
MIP Regions
1. Mountain catchments
2. Hills to north of Mahabharats, rivers rising north of Siwaliks, inner Terai
3. Pokhara, Nuwakot, Kathmandu, Sun Koshi tributaries
4. Lower Tamur valley
5. Rivers draining Mahabharats
6. Kankai Mai basin
7. Rivers draining from Churia range to the Terai
CHINA
INDIA
LEGEND
Mountain District
Hill District
Tarai District
Scale 1:35,00,000
100 0 100 200 Kilometers
Projection : Universal Transverse Mercator; Spheroid - Everest; Origin - 87° Longitude, 26° 15' Latitude; Quadrant - NorthEast.
Source : Central Bureau of Statistics, NPC, HMG Nepal

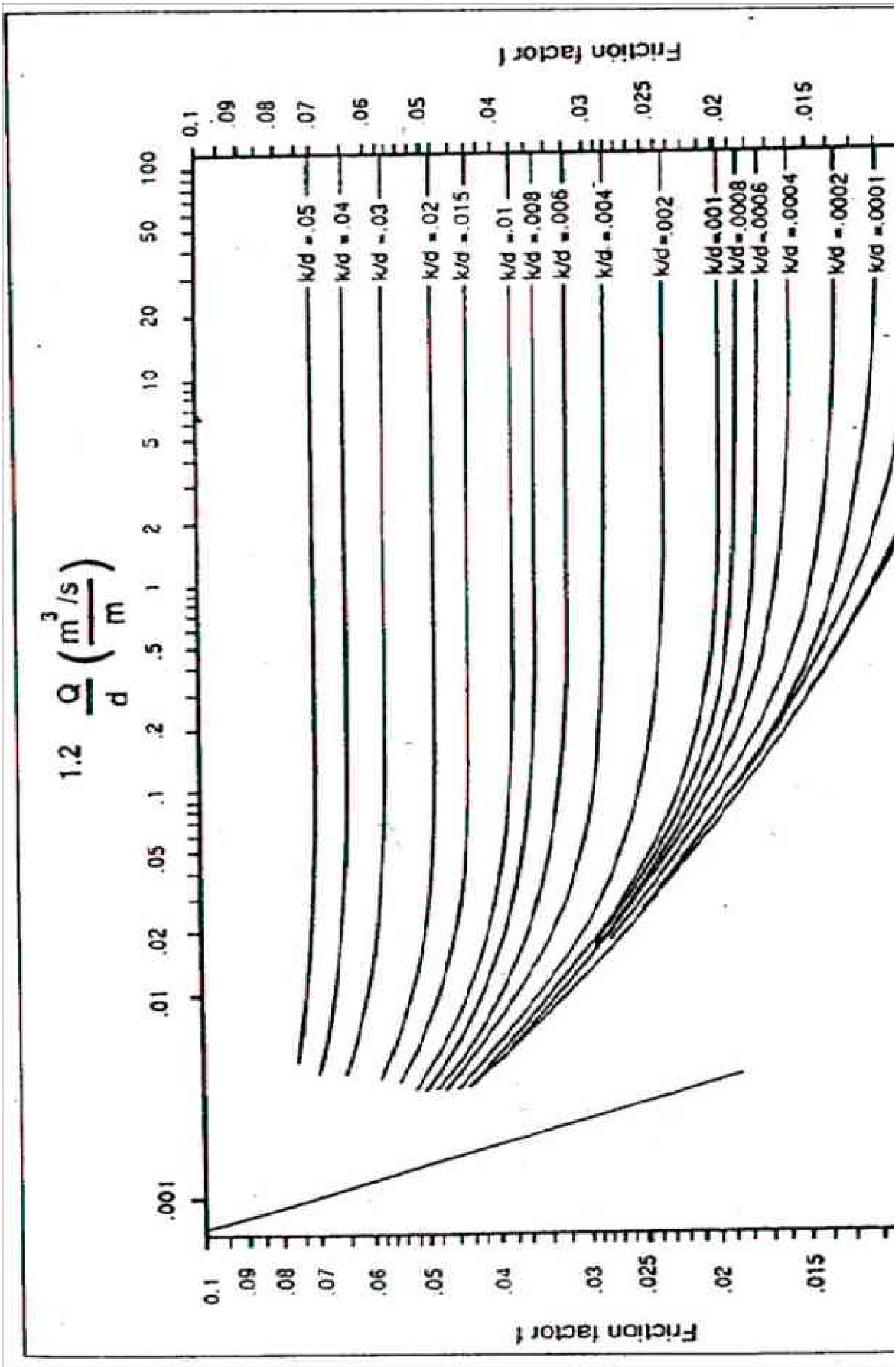

NEPAL
Mean Monsoon Precipitation
LEGEND
Dipayal
Birendranagar
Pokhara
Kathmandu
Dhankuta
INDIA
Scale 1:35,00,000
Kilometers

Month	Regions						
	1	2	3	4	5	6	7
January	2.40	2.24	2.71	2.59	2.42	2.03	3.30
February	1.80	1.70	1.88	1.88	1.82	1.62	2.20
March	1.30	1.33	1.38	1.38	1.36	1.27	1.40
April	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
May	2.60	1.21	1.88	2.19	0.91	2.57	3.50
June	6.00	7.27	3.13	3.75	2.73	6.08	6.00
July	14.50	18.18	13.54	6.89	11.21	24.32	14.00
August	25.00	27.27	25.00	27.27	13.94	33.78	35.00
September	16.50	20.91	20.83	20.91	10.00	27.03	24.00
October	8.00	9.09	10.42	6.89	6.52	6.08	12.00
November	4.10	3.94	5.00	5.00	4.55	3.38	7.50
December	3.10	3.03	3.75	3.44	3.33	2.57	5.00

Roughness values, k mm

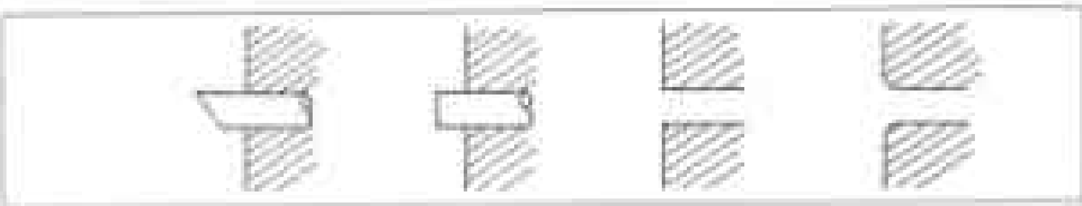
Use **'normal condition'** for design purposes

Material		**Age/condition**		
		Good (< 5 years)	Normal (5-15 years)	Poor (>15 years)
Smooth pipes PVC, HDPE, MDPE, Glass fibre		0.003	0.01	0.05
Concrete		0.06	0.15	1.5
Mild steel	- Uncoated	0.01	0.1	0.5
	- Galvanized	0.06	0.15	0.3
Cast iron New		0.15	0.3	0.6
Old	- Slight corrosion	0.6	1.5	3.0
	- Moderate corrosion	1.5	3.0	6.0
	- Severe corrosion	6.0	10.0	20.0


1.2 $\frac{Q}{d}\left(\frac{m^3/s}{m}\right)$
.001 .01 .02 .05 .1 .2 .5 1 2 5 10 20 50 100
Friction factor f
0.1 .09 .08 .07 .06 .05 .04 .03 .025 .02 .015
k/d = .05
k/d = .04
k/d = .03
k/d = .02
k/d = .015
k/d = .01
k/d = .008
k/d = .006
k/d = .004
k/d = .002
k/d = .001
k/d = .0008
k/d = .0006
k/d = .0004
k/d = .0002
k/d = .0001

Head loss coefficient for intakes ($K_{entrance}$)

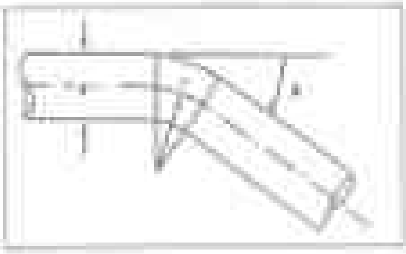
Entrance profile



$K_{entrance}$	1.0	0.8	0.5	0.2

Head loss coefficients for bends (K_{bend})

bend profile

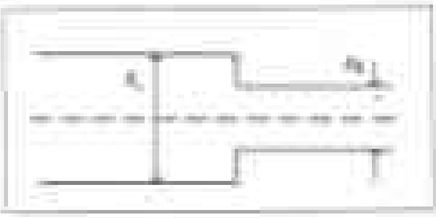


r/d	1	2	3	5	MITRED*
K_{bend} (θ = 20°)	0.20	0.15	0.12	0.10	0.10
K_{bend} (θ = 45°)	0.40	0.30	0.25	0.20	0.22
K_{bend} (θ = 90°)	0.75	0.50	0.40	0.30	0.45

*Mitred bends with r/d = 1.5, maximum 15° per mitred joint.

Head loss coefficients for sudden contractions ($K_{contraction}$)

Contraction profile



d_1/d_2	1.0	1.5	2.0	2.5	5.0
$K_{contraction}$	0	0.25	0.35	0.40	0.50

Head loss coefficients for valves (K_{valve})

TYPE OF VALVE	SPHERICAL	GATE	BUTTERFLY
K_{valve}	0	0.1	0.3