3.2 POWER GENERATING PLAN

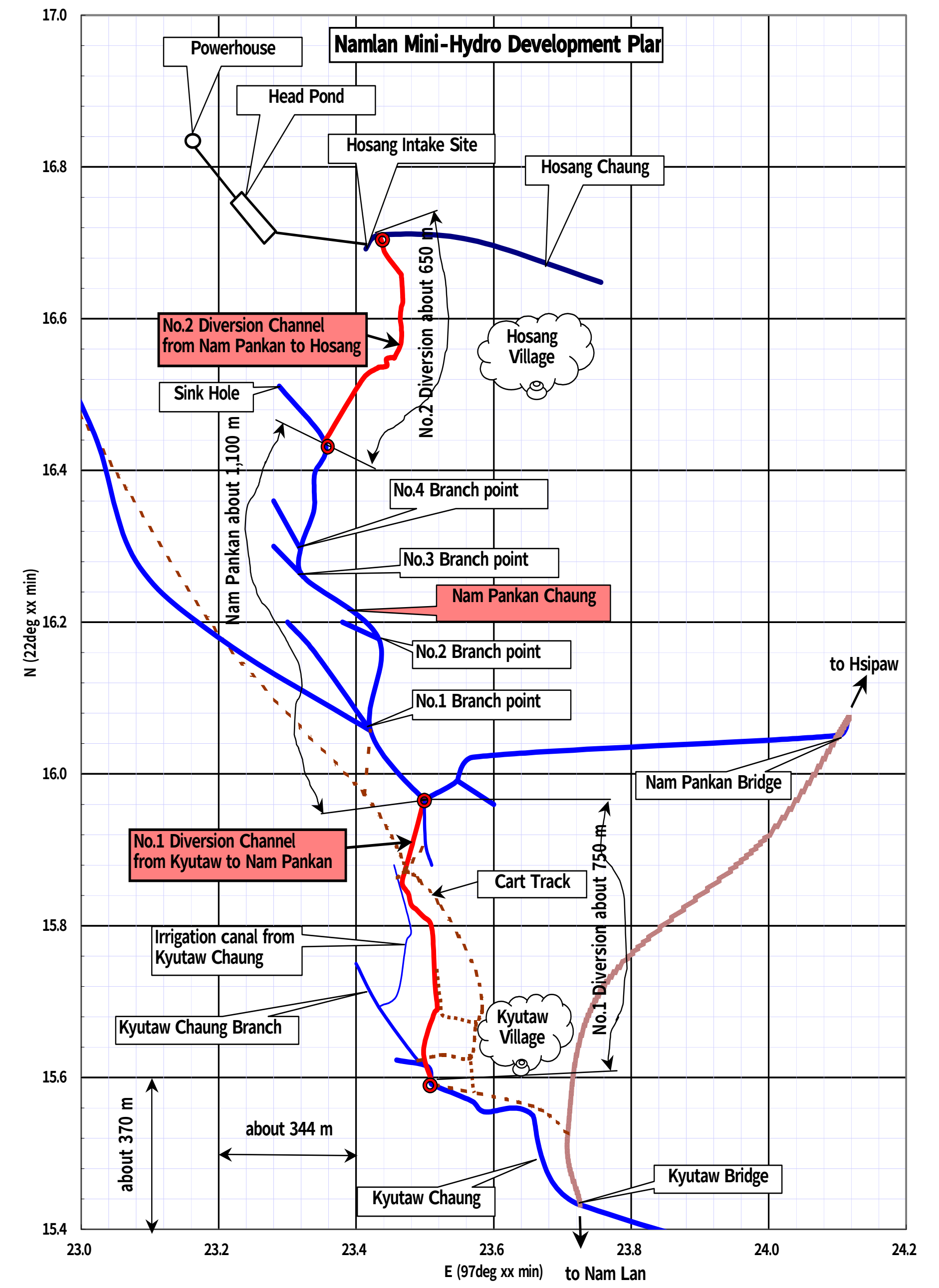
3.2.1 GEOGRAPHICAL AND GEOLOGICAL STUDIES

(1) Topography

The 1/50,000 or 1/100,000 scale topographic map is suitable to identify a site for the hydropower scheme for the initial planning and to determine accessibility from the demand centers.

The portable GPS may be a powerful tool to position easily and accurately the specific points in and around the project area at the initial planning stage. A sample mapping by GPS is shown on the right.

It is essential for the detailed design and construction to map the topography of the anticipated construction areas that will cover the open civil engineering structures such as intake, de-silting basin, head tank, and powerhouse at a scale of 1:500 or larger, based on a topographic survey. As for power canals, the profile and cross sectional surveys may be enough for designing, but further mapping of the areas around the related structures such as cross drains, side spillways, siphons, etc. will be required.



Source: JICA Study Team **Sample of GPS Mapping**

(2) Geology

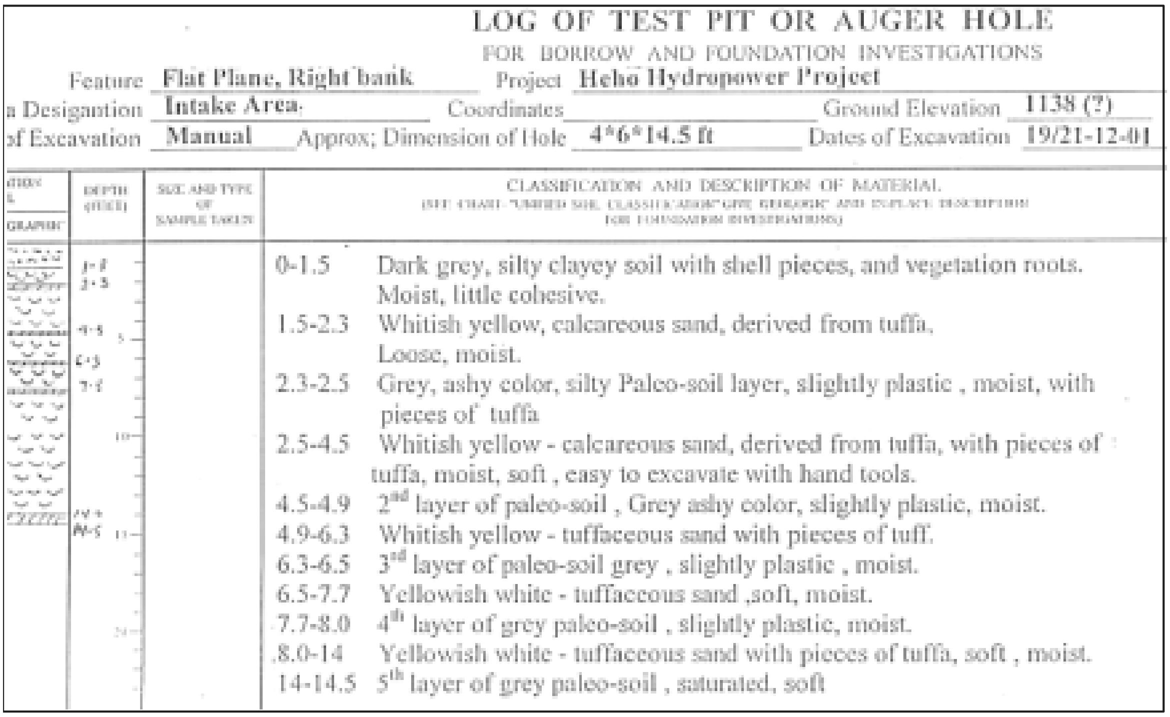
Test pitting is enough to confirm the foundation geology of the key structures for small-hydro schemes. A practical pit size is 1.8 m in length, 1.2 m width and 5.0 m depth. It can be manually dug with scoops and picks, using a rope and bucket to lift up the excavated soil without using any further heavy lifting equipments.

Test Pit

A pit log should be prepared for every test pit, as a report of the test pitting, and should contain the pit number, its location, boundaries and depths, description of soil, groundwater table and bedrock surface, and if any, all other relevant information.

Source: JICA Study Team

Sample Log of Test Pit



LOG OF TEST PIT OR AUGER HOLE

FOR BORROW AND FOUNDATION INVESTIGATIONS

Feature: Flat Plane, Right bank Project: Heha Hydropower Project

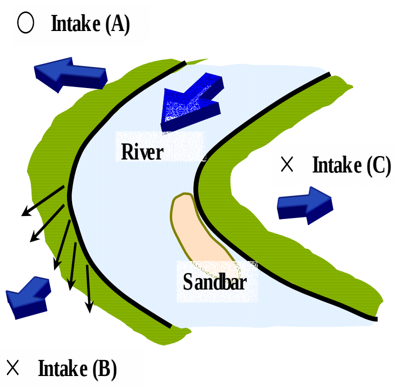
...a Designation: Intake Area Coordinates: Ground Elevation: 1138 (?)

...of Excavation: Manual Approx; Dimension of Hole: 4*6*14.5 ft Dates of Excavation: 19/21-12-01

CLASSIFICATION AND DESCRIPTION OF MATERIAL

Depth	Description
0-1.5	Dark grey, silty clayey soil with shell pieces, and vegetation roots. Moist, little cohesive.
1.5-2.3	Whitish yellow, calcareous sand, derived from tuffa. Loose, moist.
2.3-2.5	Grey, ashy color, silty Paleo-soil layer, slightly plastic , moist, with pieces of tuffa
2.5-4.5	Whitish yellow - calcareous sand, derived from tuffa, with pieces of tuffa, moist, soft , easy to excavate with hand tools.
4.5-4.9	2nd layer of paleo-soil , Grey ashy color, slightly plastic, moist.
4.9-6.3	Whitish yellow - tuffaceous sand with pieces of tuff.
6.3-6.5	3rd layer of paleo-soil grey , slightly plastic , moist.
6.5-7.7	Yellowish white - tuffaceous sand ,soft, moist.
7.7-8.0	4th layer of grey paleo-soil , slightly plastic, moist.
8.0-14	Yellowish white - tuffaceous sand with pieces of tuffa, soft , moist.
14-14.5	5th layer of grey paleo-soil , saturated, soft

3.2.2 SELECTION OF WATER INTAKE LOCATION



Source: JICA Study Team

Location of Intake

This section explains Run-of-River schemes that do not require dam construction but employ a diversion structure or a weir across the river.

One of the most common problems affecting a small/mini/micro hydropower scheme is the damage to the intake caused by floods, and another is sedimentation deposited upstream of the intake or flowing into the waterway. The following points are to be considered in selecting the intake structures:

(i) Intake (A): The best location for the intake is along the relatively straight portion of the stream

(ii) Intake (B): Susceptible to severe damage from floods, debris, and erosion

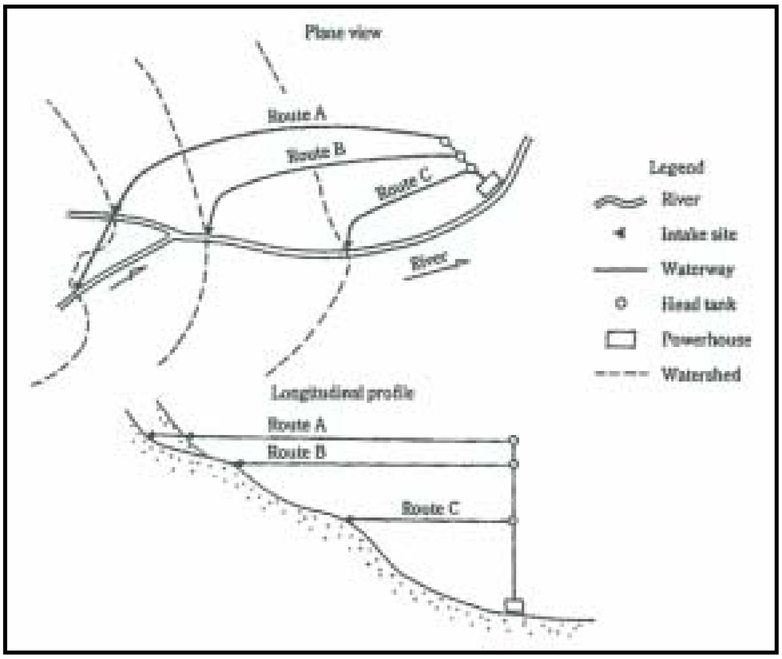
(iii) Intake (C): Sediments tend to accumulate in front of the intake and can enter and/or block the intake

3.2.3 SELECTION OF WATERWAY ROUTE

(1) What is “Waterway”?

The waterway route is a general term given to the route of the headrace, penstock and tailrace. Hydro power projects are advantageous when a large head is obtained with a short waterway. An intake weir is generally constructed in the upper reaches of a river where the river changes from a gentle to a steep gradient, and the powerhouse is constructed where the river changes from a steep to a gentle gradient. If the waterway is a tunnel, a work adit is constructed at intervals of about 3 to 4 km in many cases to curtail the construction period. Generally, the tunnel is aligned at least 30 m below the ground surface (rock cover) for the safety of the tunnel excavation. The power output and energy generation are determined by the product of the available river flow and head. The construction cost is mainly determined by the length of the waterway.

(2) Comparison of Waterway Routes



Source: Guide Manual for Development Aid Programs and Studies of Hydro HPP/ NEF

Comparison of Waterway Routes

Figure on the right shows an example of the comparative study on waterway routes. Waterway routes A, B and C with the location of the power house unchanged and but only the location of the intake weir changed.

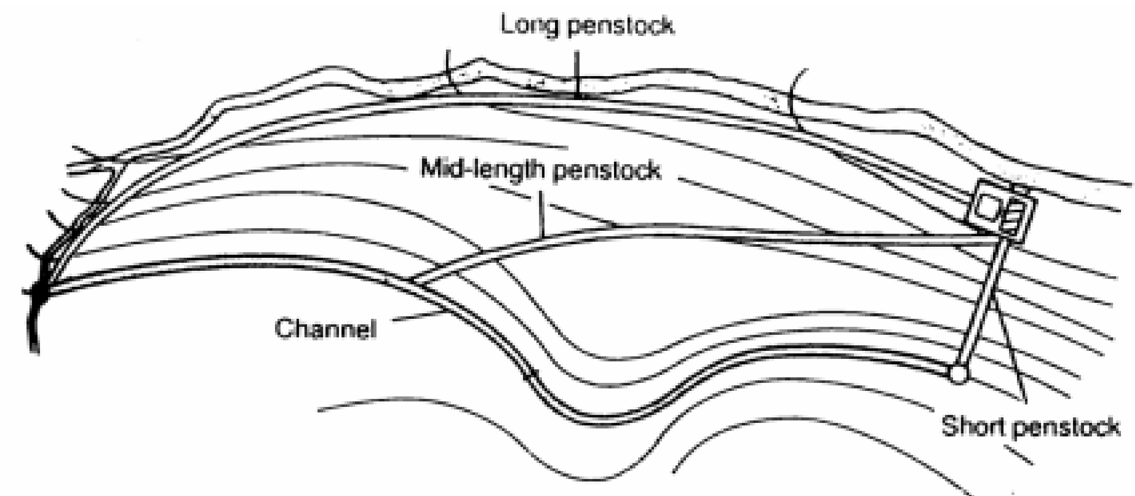
The features of the waterway routes are summarized below. The optimum route is determined by an economic comparison. The sites of the intake weir, powerhouse and other facilities are decided in full consideration of the access roads, and other factors to enable the easy maintenance and administration both during construction and after completion.

Features of Waterway Routes

Route	Catchment Area	River flow	Head	Waterway length
A	Small	Small	Large	Large
B	Medium	Medium	Medium	Medium
C	Large	Large	Small	Small

Source: Guide Manual for Development Aid Programs and Studies of Hydro Electric Power Projects / New Energy Foundation

When both sites for the intake weir and the power house are selected, possible waterway routes are shown below as an example. The 'short penstock' option will in most cases be the most economic scheme, but this is not necessarily the case.

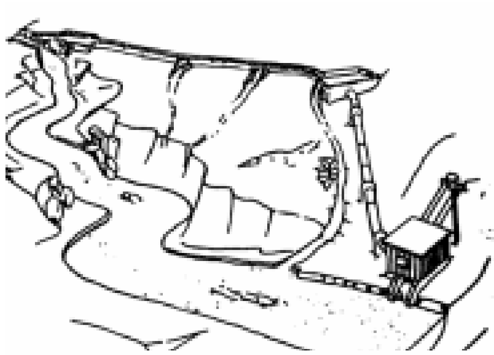


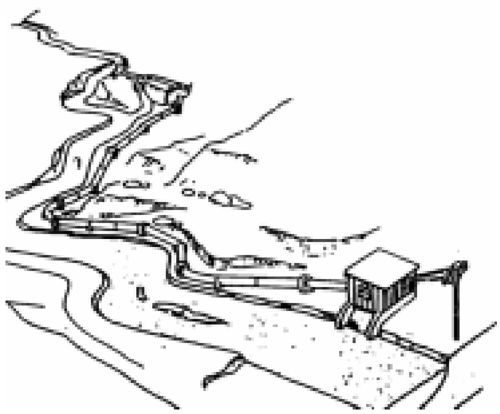
Notice that the channel can be shorted to avoid the risk and expense of construction across a steep slope.

Source: JICA Study Team **Channel and Penstock Option**

<u>**Considering each option in turn**</u>

(i) Short Penstock:

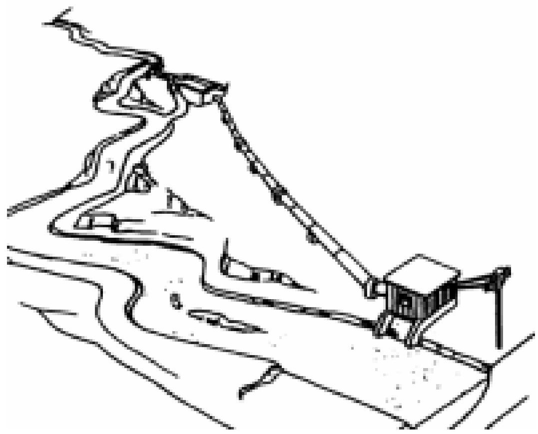




Here the penstock is short but the channel is long. The long channel is exposed to a greater risk of blockage, or of collapse or deterioration as a result of poor maintenance. Installing the channel across a steep slope may be difficult and expensive, or even impossible.

The risk of a steep slope eroding may make the short penstock layout an unacceptable option, because the projected operation and maintenance could be very costly, and outweigh the benefit of the initial purchase cost.

(ii) Long Penstock:



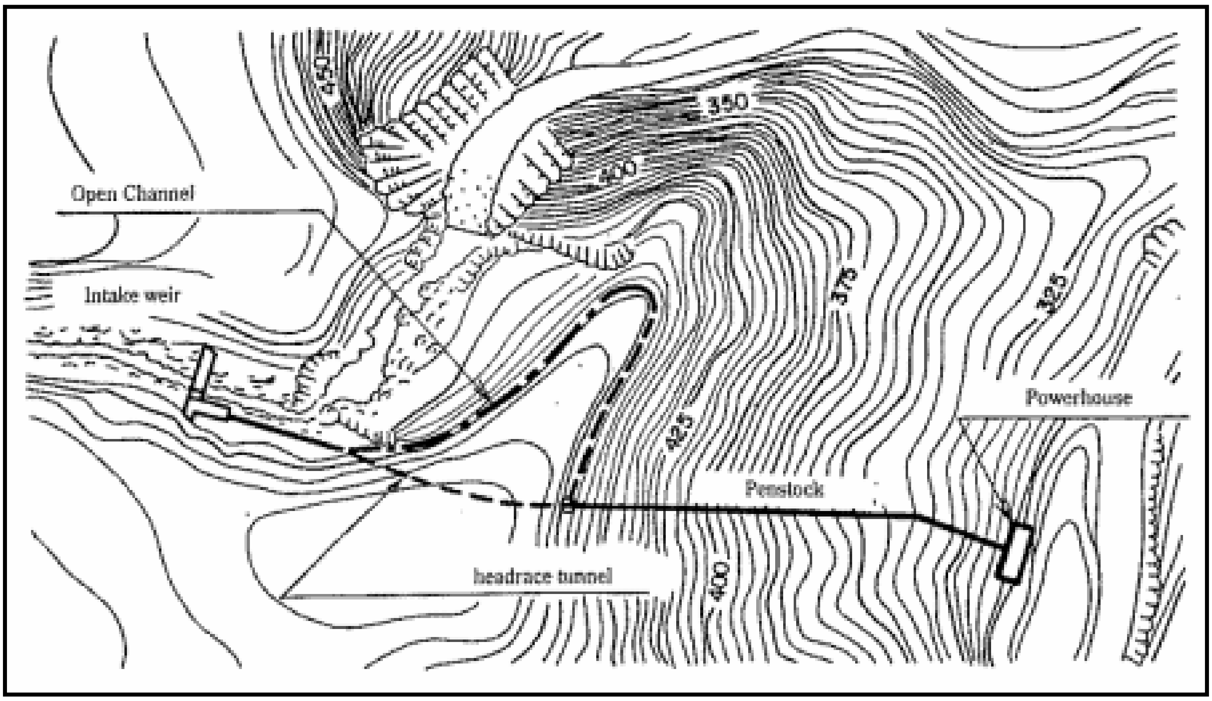
In this case the penstock follows the river. If this layout is necessary because of an impossible terrain to construct a channel, certain precautions must be taken. The most important one is to ensure that the seasonal flooding of the river will not damage or deteriorate the penstock. It is always important to calculate the most economic diameter of the penstock. When it comes to a long penstock, this is particularly important since the cost will be particularly high.

(iii) Mid-length Penstock:

The mid-length penstock will cost more than the short penstock, but the expense of constructing a channel that can safely cross the steep slope will be saved. Even if the initial purchase and construction costs are higher in this case, this option may be preferable if there are signs of instability in the steep slope.

(3) Structure of Waterway

The structure of the waterway is classified into a non-pressure tunnel and an open channel (or canal). As shown in the figure below, the waterway route is aligned on the same contour line as the intake water level when the open channel type is selected. Either the open channel or the tunnel is to be selected in consideration of the topography, geology and construction cost.



Source: Guide Manual for Development Aid Programs and Studies of Hydro Electric Power Projects / New Energy Foundation

Waterway Route

The construction cost of the open channel is significantly less than that of the tunnel of the same length. However, as the open channel is unsuitable in locations where the topography is steep and the geology is unfavorable. A minimum cross section of approximately 1.8 m in height and width is required for tunnel excavation and the discharge capacity is 3 to 4 m^3/s at a gradient of 1:1,000. If the maximum plant discharge is less than this value, a tunnel would be overly expensive while an open channel would be more economical.

(4) Determination of Waterway Route

Topography of route:

As the headrace for a small-scale hydropower plant tends to have an exposed structure, such as an open channel or covered channel, a careful survey of the topography of the passing area is necessary more than in the case of the tunnel-type headrace. When an open channel is constructed on a hillslide slope, proper attention must be paid to its slope gradient of the passing area. If a valley or a ridge exists along the passing area, the actual route should be selected after examining an appropriate way of passing (siphon for the valley section; open excavation or culvert for the elevated ridge section).

Ground stability of passing area:

As there are many incidents of the loss or burial of waterways due to the slope collapse in the case of the ground-type headrace, the ground stability of the passing area must be carefully examined.

Use of existing structures, including road and irrigation channel:

The selection of the headrace route along an existing road or irrigation channel has many advantages for reducing the cost, improving the workability and making it relatively easy to evaluate the slope stability. However, the following points must be taken into account for such existing structures.

(i) Maintenance of road function

(ii) Securing of water quantity for irrigation and water diversion method

3.2.4 SELECTION OF HEAD TANK LOCATION

Because the water tank and the power station are connected by a penstock, their locations should be selected so as to ensure the short horizontal distance of the penstock and a greater head. The location of the water tank at a projecting place such as an area should be avoided.

(1) Topographical and geological conditions

The head tank is often located at a ridge section as the generally high likelihood of the appearance of high stable ground consisting of hard rocks, etc., and the possibility of reducing the amount of excavation work, including for the penstock, offers favorable conditions for selection.

The location of the head tank for a small-scale hydropower plant is basically decided in consideration of such conditions but it must be noted that the distributing of the head tank at the ridge section is not always appropriate in the following cases.

(i) In the case when the level of consolidation is generally low, and the ridge section appears in a relatively shallow area in the more stable ground as a result of the advanced dissection of the valley section.

(ii) When the less sensitive governor is used and the large volume of water is to be used, the small water area of the head tank causes a greater fluctuation of the water level in the tank due to the load fluctuation, possibly resulting in the obstruction of smooth operation. In such a case, it is easier to secure a sufficient water area for the head tank if the tank is located at a dipped site or at a relatively flat site rather than a ridge section.

(2) Ease of dealing with effluents

The spillway for a small-scale hydropower plant may be omitted. However, if the spillway for the head tank is employed, the method for dealing with effluents must be carefully examined. (There have been reports on the ground organism washed away because of the absence of a spillway for the head tank).

The installation of the spillway pile in parallel to the penstock should not cause any major problems but the direct discharge of the surplus water and sediment inside the head tank to a nearby stream or the hillslide slope requires careful examination of the discharge point, profile as well as the cross-sectional alignment and equipment specifications to prevent scouring of the nearby ground due to the discharge of the surplus water or manmade disasters caused by rapid water level changes in the reduced discharge section.

When it comes to small-scale hydropower generation, the combined function of the settling basin and head tank may improve the overall economics of power generation and, therefore, the desirability of introducing a head tank should be carefully examined at the planning stage.

3.2.5 SELECTION OF PENSTOCK ROUTE

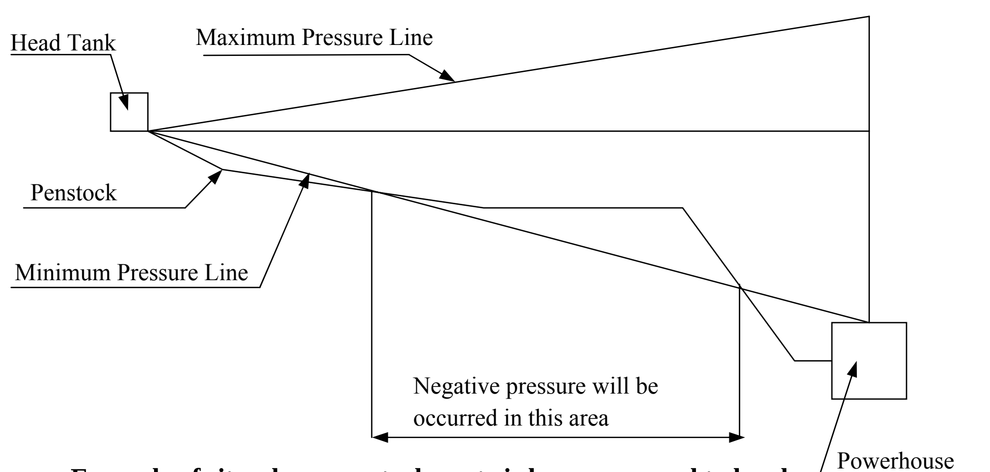
The penstock route should be selected in consideration of the following items.

(i) Hydraulic gradient
(ii) Topography of the passing area
(iii) Ground stability of the passing area
(iv) Use of the existing roads and others

The points to be noted for the selection of the penstock route are basically the same as those for the selection of the headrace route, but its relationship with the hydraulic gradient must be carefully analyzed.

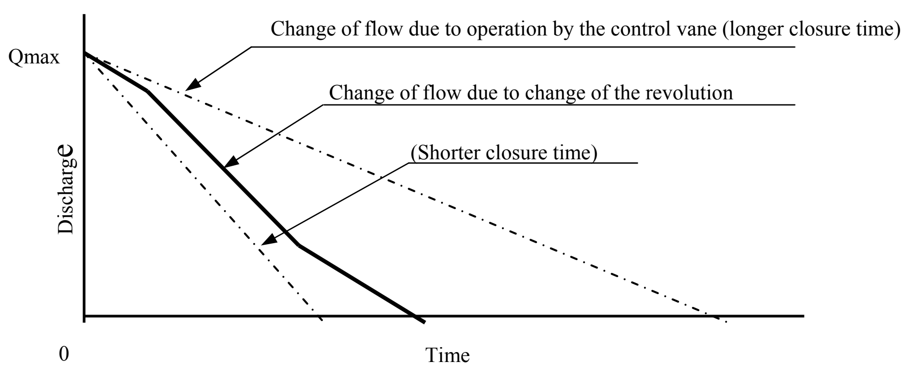
The penstock route must be designed to ensure safety specific internal pressures vis-à-vis external, and it is an absolute condition that the profile of the penstock route must be below the minimum hydraulic gradient line, i.e. minimum pressure line.

This minimum pressure line is determined by considering the internal pressure fluctuation of the penstock at the time of a rapid load shut-down. The range of the pressure fluctuation is larger in the downstream because it is influenced by changes of the discharge from the turbine over time and, therefore, careful attention is necessary at the site where the length of the penstock route is long compared to the head as shown below.



Example of site where penstock route is long compared to head

A careful examination is also required in regard to locating the Francis turbine with a slower specific speed as the range of the pressure fluctuation can be increased due to the occurrence of a phenomenon similar to that caused by the abrupt control vane operation. This is because of the increasing revolution (speed) even if the closure time for the control vane is set up fairly long. With other types of turbines, the closing speed of the control vane is almost in proportion to the speed of discharge reduction. Accordingly, no special problem occurs, provided that an adequate closure time is set.



Change of discharge at rapid load shut-off for Francis turbine with slower specific speed

3.2.6 SELECTION OF POWER STATION LOCATION

The location of the power station should be selected so as to prevent from making the penstock excessively long, and lower the tailwater level as much as practicable, but care must be taken so that the power station is not to be located below the highest recorded flood level. It is also important to consider the nearby roads. Even if the power station is located so as to ensure as large an output as possible, it is sometimes required to newly construct a road over a long section, and so the effect may turn out to be negative.

Careful attention must be paid to the following items in the selection of the powerhouse location:

Access conditions:

It is desirable for the powerhouse to be located at a site with an easy access in view of maintenance after its completion and other reasons.

Conditions of foundations:

The foundations of the powerhouse must be strong enough to withstand the installation of such heavy items as a turbine and generator. In the case of a small-scale hydropower plant, a compacted gravel layer may be sufficient because of the relatively lightweight (approximately 2-3 tons/m^2) of the equipment.

Flood water level:

The location of the powerhouse must avoid a water-hammered section so that scouring due to flooding does not occur. Another required consideration is prevention of inundation of the powerhouse caused by flooding.

In general, the small-scale hydropower station is planned for a small river in a mountainous area and it is often the case that the flood stage is not clearly determined for such a river. In this case, it is necessary to assume the flood water level based on the information listed below and then to decide the ground elevation of the powerhouse which incorporates a sufficient margin.

(i) Information obtained from local residents
(ii) Ground elevation of nearby structures (roads, embankments, bridges, etc.)
(iii) Traces of flooding and vegetation boundaries

Installation conditions for auxiliary facilities:

Space for the installation of an outdoor substation and others is required near the powerhouse, and the sites must be selected considering extension, the direction of the transmission line and other relevant matters.

However, when the transmission voltage is the same as the generating voltage, the size of the required space is small. Accordingly, the space created by the construction of foundations for the powerhouse is often sufficient to accommodate such auxiliary facilities.

3.2.7 SELECTION OF WATER OUTLETS LOCATION

The location of the outlet should be selected so as to avoid damage to the structure or clogging due to the flow or deposited materials, so that the safe discharge could be expected without excessive heaving during floods. Furthermore, in consideration of the intake of the water utilization facilities downstream, it is preferable to locate the outlet in order not to cause any trouble to the intake.

Where the water is discharged to a reservoir, the water level of the reservoir may be below the full water level over a long period and it may be economically advantageous to set the tailwater level below the full water level in consideration of the past operation of the full water level in consideration of the past operation of the reservoir. However, in such a case, the reservoir water level may rise above the outlet water level, and so it is required to plan the outlet and tailrace as a pressure type.

In case of small-scale hydropower plants, the location of the water outlet is determined with the same condition as that of the powerhouse location because it generally located near the powerhouse. In other cases, the location of the water outlet is decided taking the following items into consideration:

Flood water level:

The location of the water outlet is decided in view of the expected flood water level. When the base elevation of the water outlet is planned to be lower than the flood stage of the site, the location and base elevation of the water outlet must be decided in an appropriate manner with these in mind: (i) suitable measures to deal with the inundation or seepage of water into the powerhouse due to flooding and (ii) a method to remove sediment which has flown into the afterbay.

Existence of riverbed fluctuation at water outlet site:

When the riverbed fluctuation is expected to take place in the future, the location of the water outlet must be selected so as to avoid any trouble to its operation due to sedimentation in front of the water outlet.

Possibility of scouring of riverbed and nearby ground due to water discharge:

As the flow velocity is high at the water outlet, careful attention must be paid to avoid the scouring of the riverbed and nearby ground. When the site for the water outlet consists of sediments or soft bedrocks, it must be protected by a gabion or concrete blocks, etc. The selection of the location where such a protective measure can be easily applied is essential.

Flow direction of river water:

The water outlet must be directed (in principle, facing downstream) so as not to disrupt the smooth flow of the river water and the location which allows such direction of the water outlet should be selected.

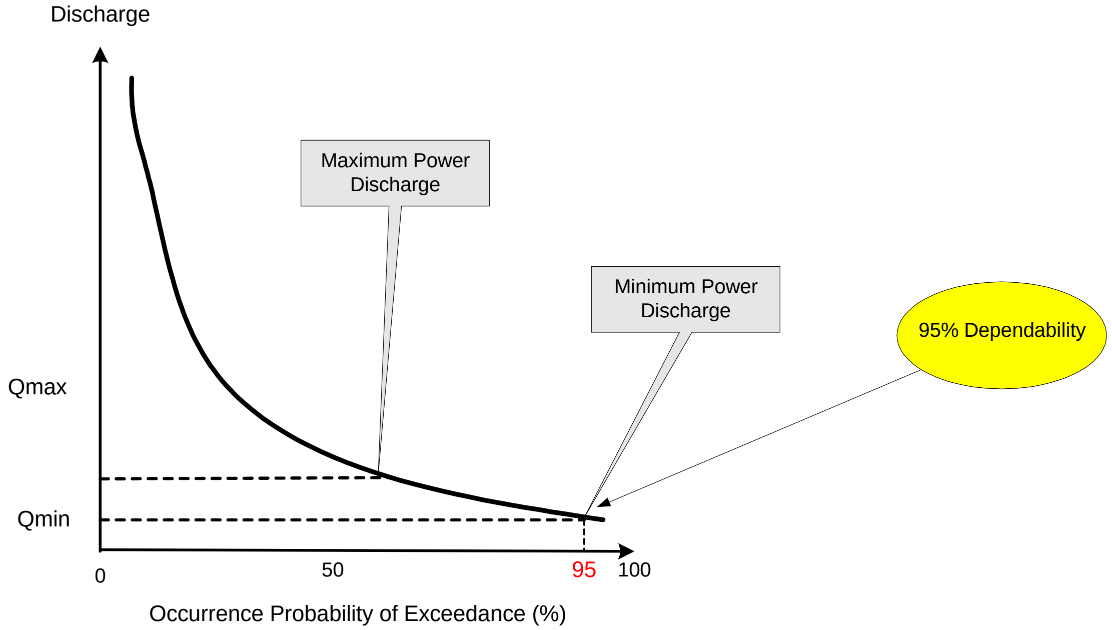
3.2.8 DETERMINATION OF DESIGN POWER DISCHARGE

For the small-hydro station with an isolated grid system, the power to be generated should be above the load demanded when the backup power system cannot be provided. The main points for planning of such small hydro plants are summarized as follows:

(i) determination of the minimum power discharge based on the available minimum discharge for power generation (90 - 95% dependable discharge is a general target)

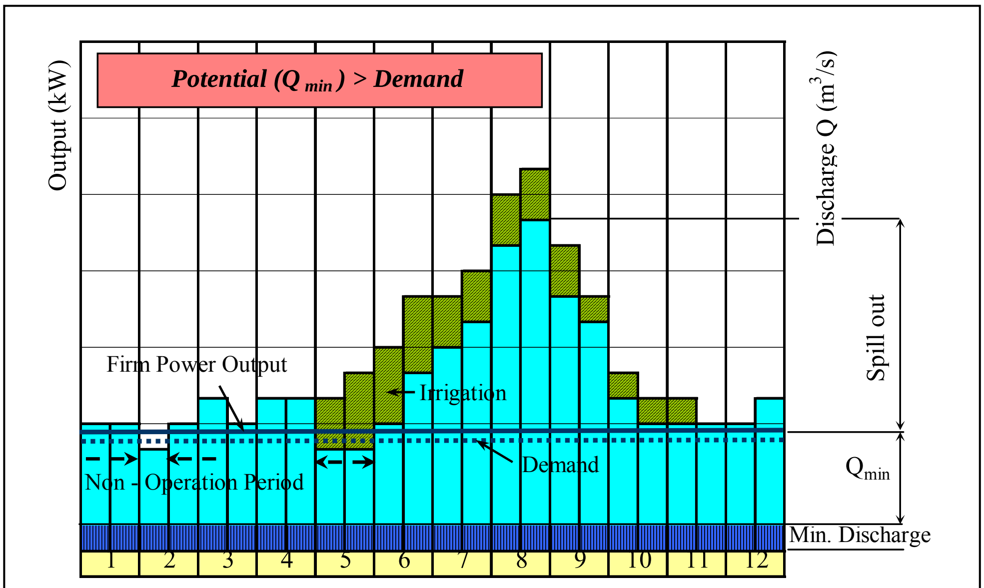
(ii) determination of the maximum power discharge depending on the peak load demand and the available discharge during the rainy season.

The relationship between the minimum and maximum power discharges is illustrated below.

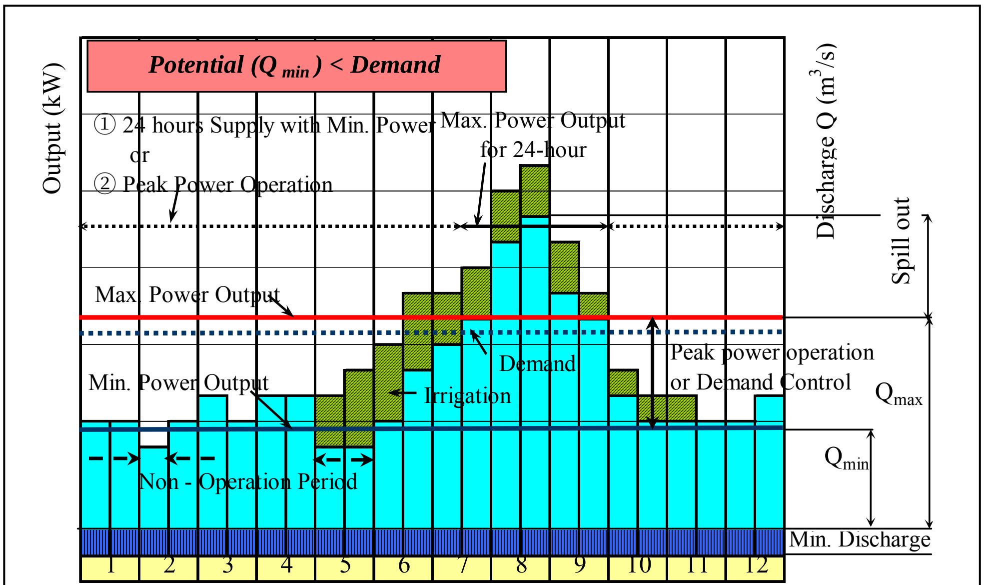


Relationship between Maximum Discharge and Minimum Discharge

If the 95% discharge is adopted to be as a minimum design discharge for power generation, it means that power generation as same as the installed capacity is possible for 347 days (365 x 0.95 = 347) per year on average. However, power generation is likely to be below the installed capacity in around 18 days when the river discharge is smaller than the 95% discharge. In addition, the relationship in the above is explained in terms of monthly mean discharge hydrograph as follows:



Source: JICA Study Team **Small Hydro Development Pattern-1**



Source: JICA Study Team **Small Hydro Development Pattern-2**

Ratios of (minimum turbine discharge)/(maximum turbine discharge) and (minimum efficiency) /(maximum efficiency) are given for typical turbines below:

Minimum Turbine Discharge

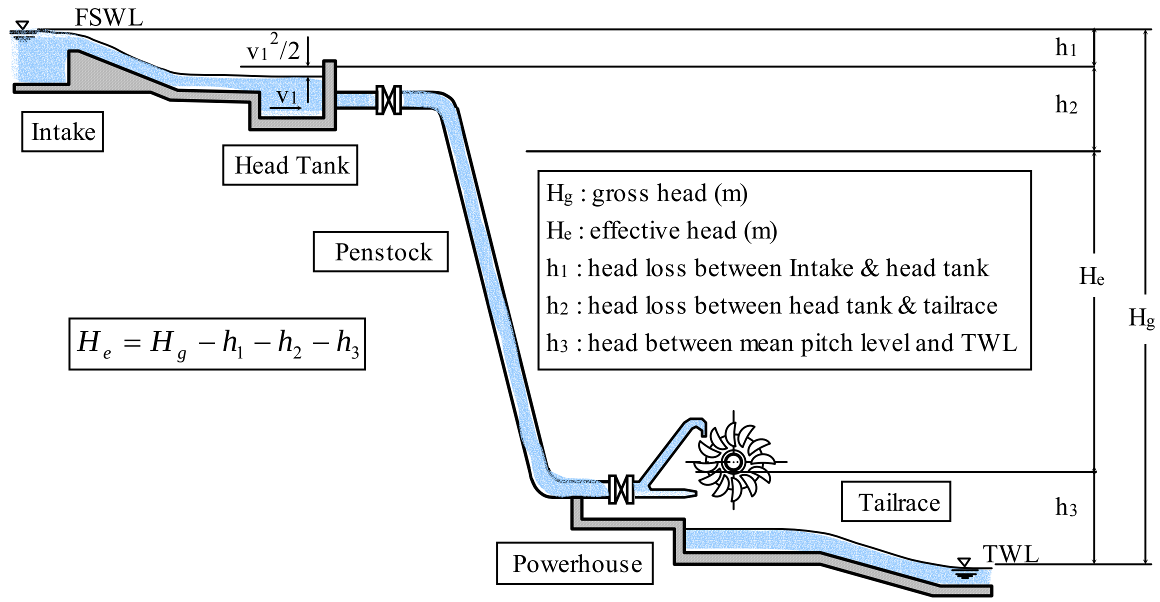
Type	(Q_{min} / Q_{max})	(η_{min} / η_{max})	
Francis with horizontal shaft	30 ~ 40%	0.70	
Pelton with horizontal shaft	15%	0.75	2-nozzle
Pelton with horizontal shaft	30%	0.90	1-nozzle
Cross flow	15%	0.75	guidevane divided
Cross flow	40%	0.75	guidevane not divided
Turgo impulse	10%	0.75	2-nozzle
Turgo impulse	20%	0.75	1-nozzle
Reversed Pump	100%		

Source: Estimation by JICA Study Team

The numbers of turbines for the small-hydro plant are preferably 1 unit, or 2 units to cover the wide range of the discharge fluctuation. When turbines without the discharge control such as Reverse Type are adopted, several units may be installed to respond to available discharges in the rainy and dry seasons. The required number of units is closely related to the selection of turbine types as explained later.

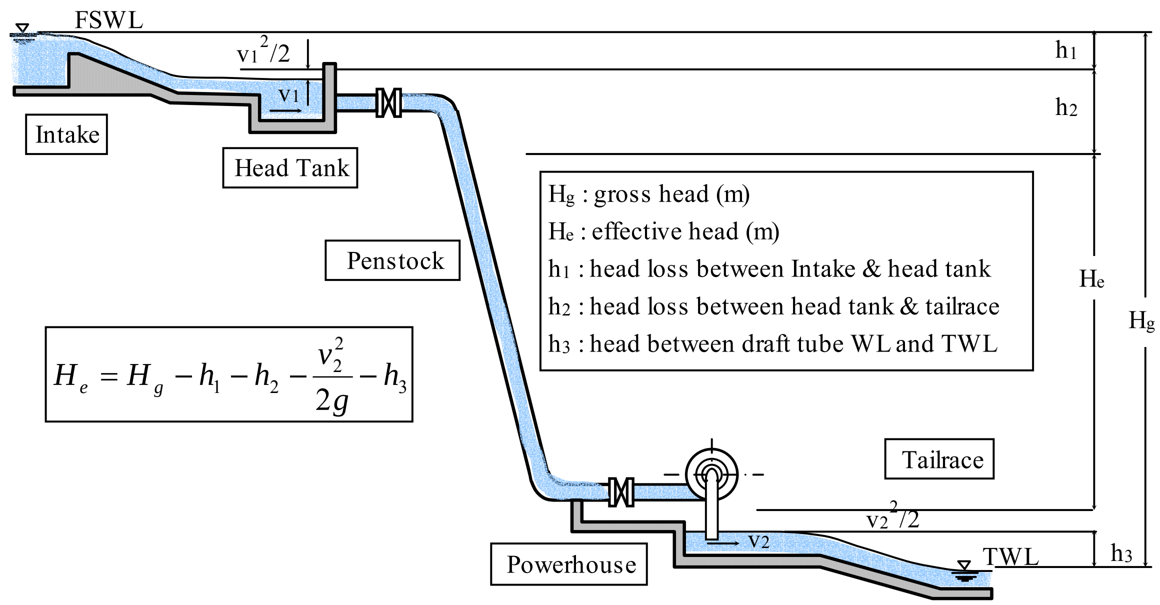
3.2.9 CALCULATING HEAD LOSS AND EFFECTIVE HEAD

The effective head can be calculated by deducting head losses from the gross head between the intake and the tailrace. However, the effective head for impulse turbines (Pelton, Turgo Impulse, Cross Flow) and that for reaction type turbines (Francis, Propeller, Tubular) are calculated differently as shown below:



Source: JICA Study Team

Effective Head for Impulse Turbines



Source: JICA Study Team

Effective Head for Reaction Turbines

3.2.10 CALCULATION OF POWER OUTPUT AND ANUAL ENERGY

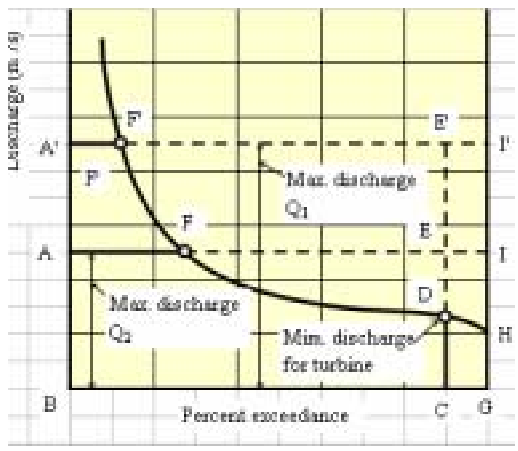
Power output is given by the following formula:

$$\boxed{P = 9.8 \cdot \eta \cdot Q \cdot H}$$

where,

P	:Power output (kW)
η	:combined efficiency for turbine and generator
Q	:power discharge (m^3/s)
H	:effective head (m)

If a Run-of-River scheme requires a flow of more than the minimum river discharge, a flow duration curve is useful to estimate the approximate annual energy as follows:



Source: JICA Study Team

Figure 1.6.5 Flow Duration Curve

For maximum discharge Q_1 :

$$\boxed{PlantFactor(\xi_1) = \frac{area(A'BCDF')}{area(A'BGI')}}$$

$$\boxed{Annual\ Energy\ E_1 = \xi_1 \cdot P \cdot 8{,}760}$$

Where, E_1 : Annual energy (kWh)

P : Max. power output (kW)

For maximum discharge Q2 :

$$\boxed{PlantFactor(\xi_2) = \frac{area(ABCDF)}{area(ABGI)}}$$

$$\boxed{Annual\ Energy\ E_2 = \xi_2 \cdot P \cdot 8{,}760}$$

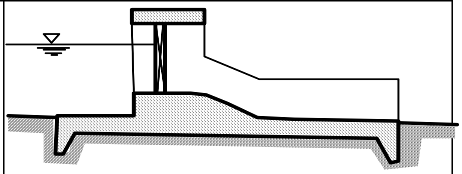
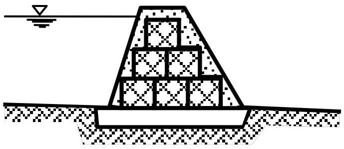
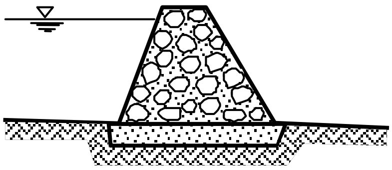
When the bigger discharge (Q1) is selected, a larger scale of power facility with a lower plant factor is required, while the smaller discharge (Q2) gives a smaller plant facility with a higher plant factor. The optimum maximum design discharge needs to be selected finally in consideration of the revenue to be generated, the cost to be incurred in principle, and the power tariff to be properly established.

3.3 CIVIL WORKS LAYOUT-DESIGN OF INTAKE WEIR&DAM

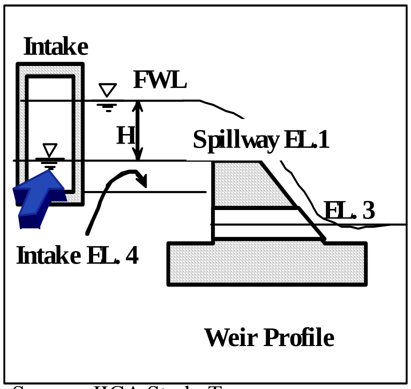
3.3.1 DESIGN OF INTAKE WEIR

(1) General

Types of the weir are summarized in the next table.

Type of Weir	Specific Features	Typical Figure
Concrete gravity	• Applicable on rock foundations • Most commonly used • Durable and impervious • Relatively high cost	
Floating concrete weir	• Applicable on gravel foundations • Need an enough seepage path • Durable • Relatively high cost	
Gabion covered with concrete	• Applicable on gravel foundation • Surface protection by concrete • Relatively low cost	
Gabion	• Applicable on gravel foundation • Flexible • Low cost and easy maintenance	
Stone masonry	• Applicable on gravel foundation • Low cost and easy maintenance	

It should be noted that the types of the weir to be applied should be determined according to the power scale, importance, flood discharge, foundation condition, and maintenance requirements. The use of high quality materials and construction techniques will result in less maintenance and repair work over the life period of the scheme.



Source: JICA Study Team

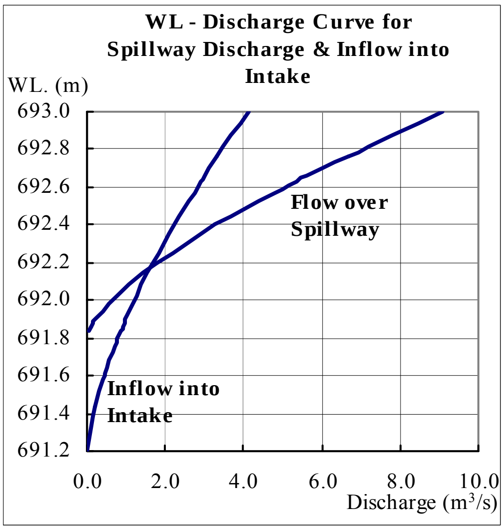
Weir Profile

(i) The weir crest level is normally designed equal to the Full Supply Water Level (FSWL) under the maximum design discharge.

(ii) The hydraulic design of the weir and intake should be made appropriately to take the proper discharge into the waterway. Since the flow taken from the river is not regulated in Run-of-River scheme, any excessive water above the maximum design discharge should be released safely from the spillways. When a weir crest is set equal to the FSWL at the maximum design discharge, the inflow into the intake can be divided into the following cases:

(2) (River flow) < (Maximum design discharge)

- (i) The whole flow enters the intake.
- (ii) The water level varies between FSWL (EL.1) and the intake floor level (EL.2)
- (iii) The maximum design discharge flows into the intake at FSWL.
- (iv) The minimum flow to the downstream basin shall be released from the river outlet at any conditions if needed.



Source: JICA Study Team

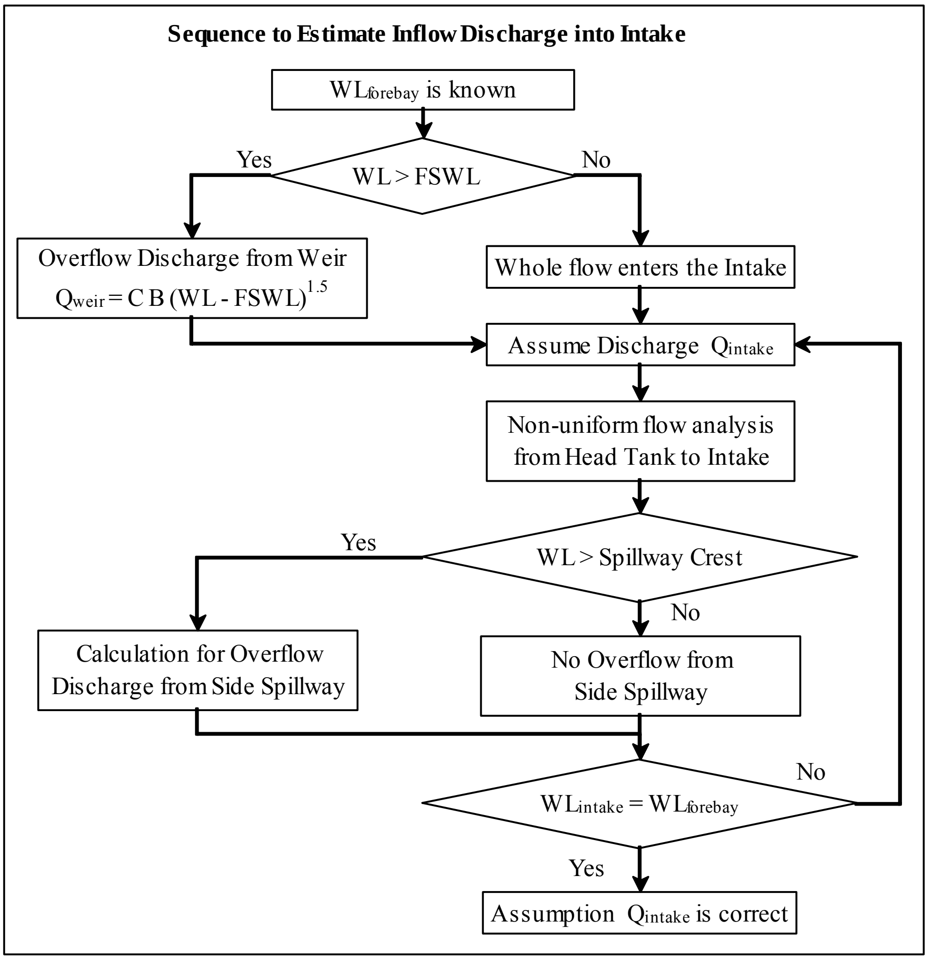
Example of Rating Curve

(3) (River flow) > (Maximum design discharge)

- (i) A water level is above FSWL (EL.1), when part of the discharge is spilt over the weir and the remainder, which exceeds the maximum design discharge, runs in the waterway.
- (ii) Any excessive discharge taken from the intake should be released from the side spillway, which needs to be provided at a suitable location of the waterway.
- (iii) The intake gate should be closed during floods to avoid excessive sediments inflow into the waterway.
- (iv) If the river water level is known from water level gauge provided at the forebay, the

Intake of Gabion covered with concrete: Nam Mong Hydropower Station

discharge entering the waterway can be estimated by the following sequences. Then, a rating curve (WL-Q) at the forebay can be prepared.



Source: JICA Study Team

Flowchart to Estimate Inflow Discharge into Intake

The overflow discharge from the spillway and the outflow discharge through the sand flush gate can be calculated by the following formulas:

Discharge from the weir spillway

$$\boxed{Q_{spill} = 2.1 \cdot B \cdot H^{1.5}}$$

where, Q_{spill} : discharge from spillway (m^3/s)

B : width of spillway (m)

H = WL - Crest Level (m)

Discharge from the sand flush gate

(i) For the orifice flow

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

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

A : Flow area (m^2)

H = WL – Centre level of orifice (m)

(ii) For the pipe flow

$$\boxed{Q = A \cdot \sqrt{\frac{\quad}{\quad + \quad_e}}}$$

f_e : loss coefficient for entrance (0.1 ~ 0.5)

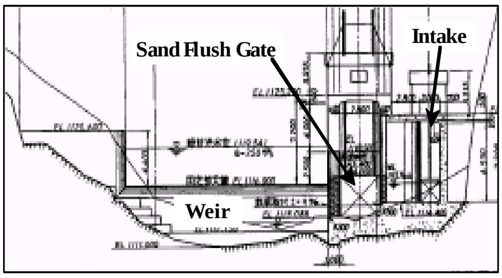
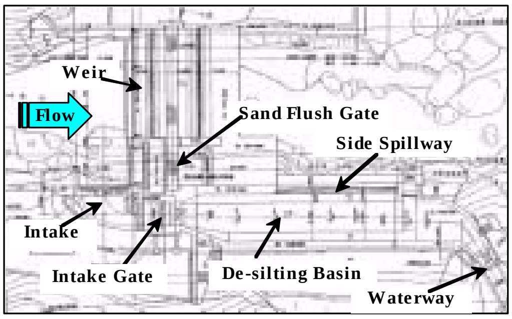
f : loss coefficient for friction = 124.5n^2 L/D$^{(4/3)}$

In order to carry out the peak power generation in the dry season without providing a regulating pond, the river channel storage may be effective if gates are provided on the weir. The gates should be open in the rainy season and be closed in the dry season if floods are not anticipated.

3.3.2 DESIGN OF INTAKE

Intake

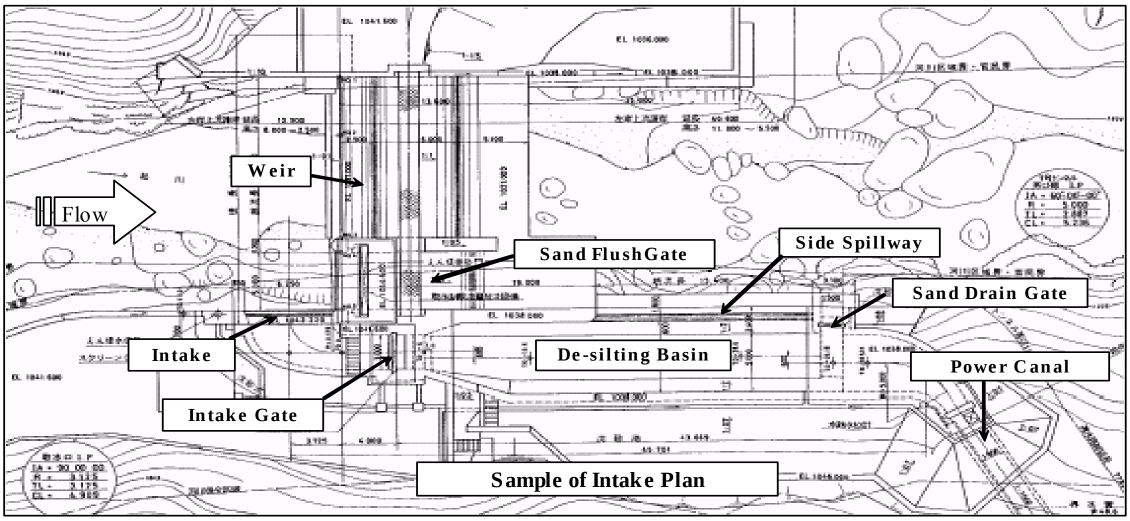
Types of the intake are summarized as follows:

Type of Intake	Specific Features	Typical Figure
Side Intake with Weir	● Most commonly used for Run-of-River Type power schemes ● The sand flush gate is located aside the weir to release sediments deposited upstream of the weir. ● The intake is located at a side of the river just upstream of the weir/sand flush gate. ● The intake gate is provided at the upstream section of the de-silting basin to close during the sand drain operation or maintenance of the waterway.	 
Tyrolean Type Intake	● Suitable for steep rivers containing boulders ● Weir is not necessary ● Necessary to remove driftwoods or leaves on the screen ● Necessary to remove fine sands entering the intake	
Intake to Utilize Pondage	● Applied to natural/artificial ponds to utilize the water for power generation	

Source: JICA Study Team

The site selected for the headworks should be stable and suitable for reliable foundations. All excess water and debris taken from the river need to be minimized by the design of headworks, and those entering with the flood flow need to be returned to the river before flowing into the canal or penstock.

Hydraulic requirements generally applied to the side intake with a concrete weir are summarized as follows:



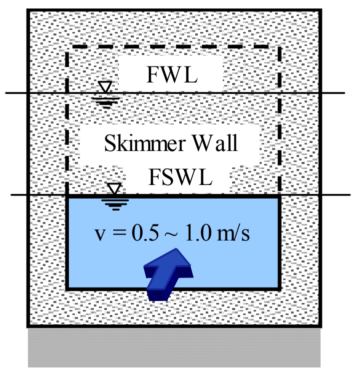
Source: JICA Study Team

Hydraulic Requirements Applied to Side Intake

Item	General Application	Symbol
Crest Level of Intake Weir	= Full Supply Water Level	EL. 1
Sill Level of Sand Flush Gate	= Original River Bed + (0.5m ~ 1.0m)	EL. 2
Floor Level of Intake	= EL.2 + (1.0m ~ 1.5m)	EL. 3
Velocity at Intake	0.5 ~ 1.0 m/sec approximately	
Top of Intake Deck	= Flood Water Level + freeboard (> 1.0m)	EL. 4
Top of Intake Gate	= FSWL	
Velocity at Intake Gate	1.0 ~1.5 m/sec approximately	
Crest of Side Spillway	= FSWL - (0 ~ 10 cm)	EL. 5
Slope of De-silting Basin	1:10 ~ 1:30	
Velocity in De-silting Basin	< 0.3 m/sec	
Length of De-silting Basin	(2 ~ 3) x depth x velocity / sedimentation rate = (2 ~ 3) x depth x 0.3 / 0.1 = (6 ~ 9) x depth	
EL. of Sand Drain	(Sand drain outlet level) > (Water level of the river)	EL. 5
Floor Level of Power Canal	= EL. 3	EL. 7
Slope of Power Canal	1:1,000 ~ 1:2,000	
Velocity in Power Canal	< 2 m/s maximum for lined canal	

Source: JICA Study Team

A skimmer wall at the entrance of the intake will be effective not only to avoid driftwoods entering or debris floating into the intake, but also to restrict an excessive inflow by making an orifice flow when the river water level is higher than the Full Supply Water Level (FSWL) during a flood.

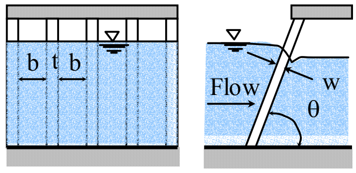


Source: JICA Study Team

Front Elevation of Skimmer Wall at Entrance

The intake gate is provided at the upstream section of the de-silting basin that can be closed during the sand drain operation or maintenance of the waterway. The gate is to be closed during floods to avoid excessive sediment inflows. The velocity through the opening intake gate should be limited to about 1.0 m/s.

Trash racks are installed at the entrance of the intake to prevent trash, leaves, and floating debris from flowing into the waterway. The screen bars are generally arranged with 5-9 mm thick, 50-120 mm bar wide, 100-150 mm intervals, and 60-70º angles to the horizontal.



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

Source: JICA Study Team

Figure Trashracks

Countermeasures against Sedimentation

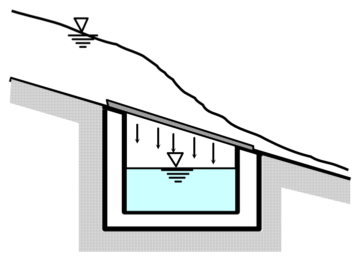
The Tyrolean intake is applicable to mini/micro hydropower stations located on steep rivers containing boulders and pebbles. The characteristics of Tyrolean type intake are as follows:

(i) Intake facilities can be minimized.

(ii) Relatively large amounts of sediments will enter the intake especially during a flood, so the sand drain facility with enough hydraulic gradient and capacity to drain out the sediments is indispensable. Periodical sand draining operations are required.

(iii) Cleaning work for driftwoods or leaves trapped on the screen is necessary.

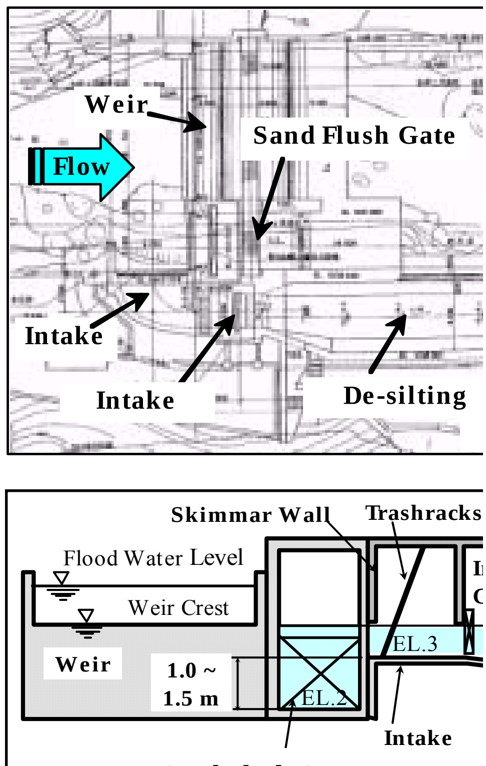
(iv) The intake discharge of 0.1-0.3 $m^3/s/m^2$, the screen slope gentler than 30° and the screen bar interval of 20-30 mm is generally applied.



Source: JICA Study Team

Profile of Tyrolean Intake

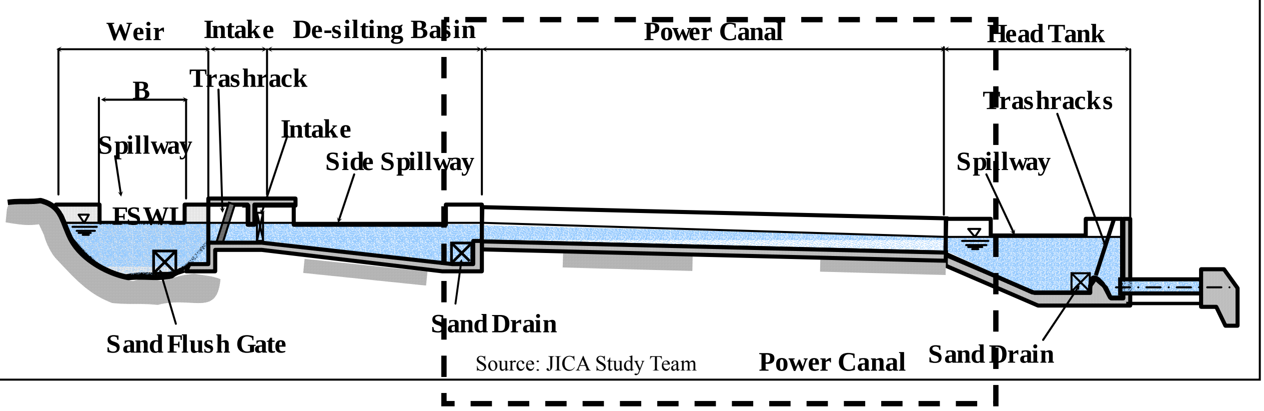
- A sand flush gate should be located to one side of the weir to release sediments deposited upstream of the weir. The intake is located at a side of the river just upstream of the weir and to minimize sand volume entering the intake. The sill level of the sand flush gate is generally set at 0.5-1.0 m higher than the original riverbed level and 1.0-1.5 m lower than the intake floor level.
- The skimmer wall at the entrance of the inlet may be effective to prevent driftwoods or excessive flood flows from entering the intake.
- If the slope failures or sediment yield are confirmed in the upstream basin, protection work such as a gabion wall may be effective to control the sediment outflow.
- The flow velocity at the intake should be limited to 0.5-1.0 m/s to avoid sediments flowing into the waterway.



Source: JICA Study Team

Sand Flush Gate

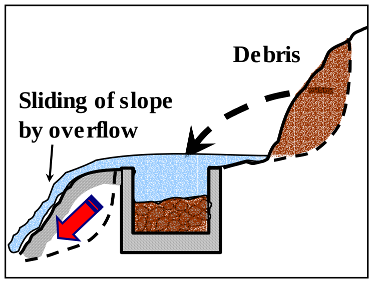
3.3.3 DESIGN OF POWER (HEADRACE) CANAL



Source: JICA Study Team

Route Selection

This section deals with open canals only, which are most commonly applied to small/mini/micro hydropower schemes, especially in Lao PDR.



Sliding may be induced by overflow from a canal in which debris enters the canal. Source: JICA Study Team

Canal and Slope Failure

A route for the power canal needs to be selected after consideration of the topographic features along the canal with the following points:

- (i) Stability against the slope above and/or below the canal
- (ii) Specific conditions such as streams, roads, and the existing structures to be crossed by the canal

The selection of the canal route and the design of canals should be made in consideration of the fact that the water level in the canal may rise for any of several possible reasons:

- (i) When the canal flow is obstructed by a landslide or closure of the downstream gate
- (ii) When the excessive water enters the intake during a flood.
- (iii) When the excessive running water is drained into the canal during heavy rains


Side spillway to overflow excessive inflow Source: JICA Study Team

Side Spillway

The following facilities for the canal may need to be designed to handle the conditions mentioned above:

Facilities for Canal

Potential Landslide	(a) Box culvert or canal cover (concrete/wood) (b) Slope protection by the structural reinforcement of the slope, excavation in a gentler slope, and vegetation such as sodding or planting
Crossing of stream or valley	(a) Aqueduct to by-pass the flows from floods or debris flows (b) Siphon to pass under the stream (c) Drainage facilities to collect the running water in the catchment basin and to release it safely to protect the canal from being damaged or eroded by the drained flow or debris
Crossing of roads or existing structures	(a) Box culvert or bridge to keep connecting the existing road. (b) Steel pipe or concrete conduit embedded under the existing structures.
Excessive inflows	(a) Side spillway to overflow the excessive flow over the max. design discharge. An appropriate protection work against scouring by the overflow is indispensable (b) Drainage facilities to avoid excessive inflows into the canal

Source: JICA Study Team

When selecting the canal route, the existing structures such as foot paths and irrigation channels can be utilized for minimization of the construction cost of the canal as well as for ease of access.

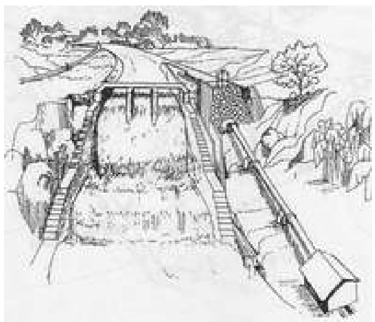
Existing footpaths or irrigation canals may be utilized for the power canal Source: JICA Study Team

Existing Footpath

Depending on the topographic conditions, it may be possible to omit the power canal, and the penstock may be connected directly to the de-silting basin or the head tank.

Canal Dimensions

Power canals are to be designed in consideration of: 1) flow capacity; 2) velocity; 3) roughness; 4) slope; 5) sectional shape; 6) lining (with or without, material); and 5) maintenance.



Omission of canal, and utilization of existing structures

Structure without Canal

The velocity in the canal should be low enough to prevent erosion of the canal, especially if it is unlined, and to keep the effective head as high as possible.

The velocity in the canal should be high enough to prevent sedimentation and the growth of aquatic plants especially for the unlined earth canals.

V_{max} = 0.3 m/s for sedimentation of flows with silts
V_{max} = 0.3 ~ 0.5 m/s for sedimentation of flows with fine sands
V_{max} = 0.7 m/s to prevent aquatic plants to grow

The maximum allowable velocities for the unlined canals to avoid erosion are given as follows:

Velocities for Unlined Canals

Material	*n*	V_{max} (m/s)	Allowable Range ($10^{-6} m^3/s/m^2$)
Fine sand	0.020 – 0.025	0.3 – 0.4	> 8.3
Sandy loam	0.020 – 0.025	0.4 – 0.6	2.8 – 8.3
Clayey loam	0.020 – 0.025	0.6 – 0.8	1.4 – 2.8
Clay	0.020 – 0.025	0.8 – 2.0	0.3 – 1.4

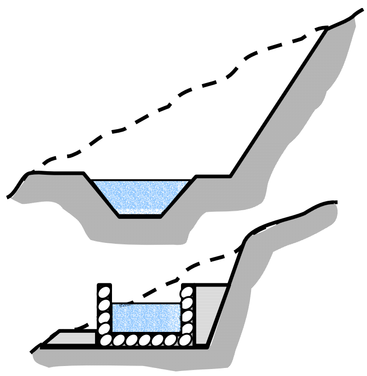
For the lined canals, wear of abrasion sets the upper limit on velocities. Velocities above 10 m/s will not damage concrete lined canals when the water is clear, but velocities above 4 m/s containing sand and gravels may scour the lining.

Source: JICA Study Team

Stone Masonry Canal with Screen

The steeper the slope of the canal is, the smaller the sectional area is to be required, however, the effective head will be decreased. The best combination of a canal size and a slope should be examined within a suitable range of flow velocities.

The maximum velocity in the lined canal is normally smaller than 2.0 m/s.



Properly designed lined canal reduces canal size and the excavation volume to convey the same discharge. Source: JICA Study Team

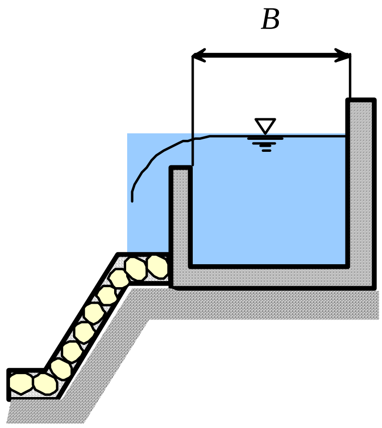
Canal Design

The canal slope, depending on the topographic conditions, is generally as follows:

1/500 ~ 1/1,000	:	to minimize the canal size for the high head plant
1/1,000 ~ 1/1,500	:	general application
1/1,500 ~ 1/2,000	:	to minimize a head drawdown for the low head plant

The roughness coefficient “n” is an empirical measurement of the surface roughness of the waterway. The following values are usually applied:

Steel	:	0.012 ~ 0.013
Concrete	:	0.014 ± 0.001
Stone-masonry	:	0.016 ~ 0.020



Source: JICA Study Team

Side Channel Spillway

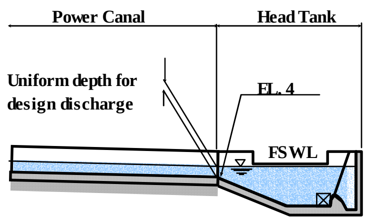
For unlined canals, a trapezoid cross-section is the most common. The side slopes of the canal are 1.0 (V):0.5 (H) for the rock foundation, and 1.0(V):2.0(H) for the sandy loam foundation.

For lined canals, a rectangular or a trapezoid cross-section is commonly used along with the stone masonry lining, and a rectangular section for the concrete lining.

The side channel spillway is generally built at the de-silting basin and the head tank, however, it may be necessary to be designed in a suitable section of the power canal depending on the design conditions. The outflow path needs to be protected against scouring.

Water Surface Profile

The canal floor elevation at the downstream end (EL.4 in the figure) is commonly fixed to provide a uniform flow depth for the maximum design discharge when the water level in the head tank or in the regulating pond is at the Full Supply Water Level (FSWL). In this condition, the flow depth in the canal is uniform over the whole stretch if the canal slope is uniform.



Uniform flow state at the downstream end of the canal at FSWL

Source: JICA Study Team

Water Surface : Uniform Flow

The non-uniform flow analysis should be carried out for the full section of the waterway starting from the head tank or the regulating pond up to the intake with parameters such as discharge, roughness coefficient, and the initial water level at the head tank. The wall height of the canal is to be designed so that the energy line for the maximum inflow into the canal should be lower than the wall crest.

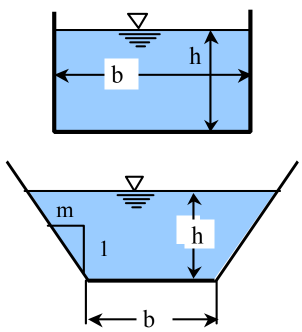
$$Q = \frac{A}{n} R^{\frac{2}{3}} I^{\frac{1}{2}} \qquad Q = V \cdot A$$

For a rectangular section

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

For a triangular section

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



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

Source: JICA Study Team

Discharge Calculation

The uniform flow depth in the canal can be calculated by Manning's Formula: Uniform flow analyses can be made by the computer programs

The non-uniform flow analysis involves in solving the following differential equation:

$$\frac{dh}{dx} = \frac{i + \frac{\alpha Q^2}{gA^3}\frac{\partial A}{\partial b}\frac{\partial b}{\partial x} - \frac{n^2}{R^{4/3}}\left(\frac{Q}{A}\right)^2}{1 - \frac{\alpha Q^2}{gA^3}\frac{\alpha A}{\partial h}}$$

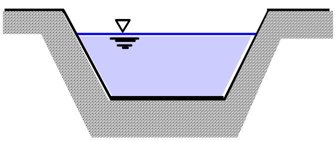
The non-uniform flow analysis can be also made by the computer programs.

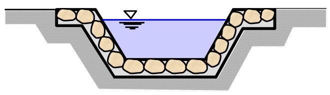
Lining Types

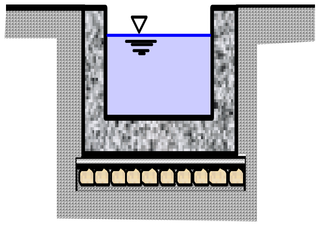
The lining types of earth canals has the following characteristics: (a) easy for construction and maintenance; (b) low cost; (c) not applicable to the pervious and erosive foundation; (c) velocity < 0.3 m/s; (d) roughness coefficient n = 0.014 on average, seepage loss = 1.0 (clay) ~ 8.0 (sand) x 10-6 $m^3/s/m^2$.

The lining type of the stone masonry canal has the following characteristics: (a) easy for construction and maintenance; (b) velocity <1.5 m/s (dry stone masonry) and velocity <2.0 m/s (wet stone masonry); (c) roughness coefficient n = 0.032 (dry stone masonry) and roughness coefficient n = 0.025 (wet stone masonry).

The lining type of the concrete lining canal has the following characteristics: (a) durable; (b) relatively high cost; (c) velocity < 3.0 m/s; (d) roughness coefficient n = 0.015 on average.


Earth Canal


Stone Masonry Canal


Concrete Canal

Source: JICA Study Team

Type of Canal Lining


Stone Masonry Canal

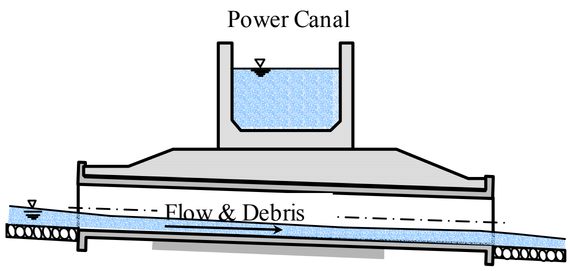

Stone Masonry Canal

Cross Drain

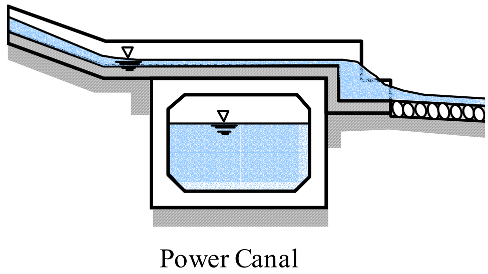
If the power canal passes through valleys with some catchment area, drain facilities that cross under or over the power canal should be installed to protect the canal structure from damages by running water containing debris when or after raining.

- Slope steeper than 1/50
- Size bigger than ϕ60cm
- Enough flow area not to be clogged
- Maintenance for clogging

Box culverts, concrete pipes, polyethylene pipes, etc. are used as underground drains, and open chutes as ground drains. The underground drains need an adequate flow area, since they are likely to be clogged with debris, soil, and so on. The minimum inner space is preferably 60 cm for the manual cleaning.



Source: JICA Study Team
Cross Drain under Power Canal



Source: JICA Study Team
Cross Drain over Power Canal

3.3.4 DESIGN OF DE-SILTING BASIN

The de-silting basin is designed to settle sand which is bigger than 0.5 ~ 1.0 mm in diameter, and the settling velocity corresponds to 0.1 m/s. The average flow velocity in the de-silting basin is generally 0.3 m/s, and the channel slope is 1/10 - 1/30. The length of the de-silting basin is given by the following empirical formula:

Cross Drain over the Power Canal

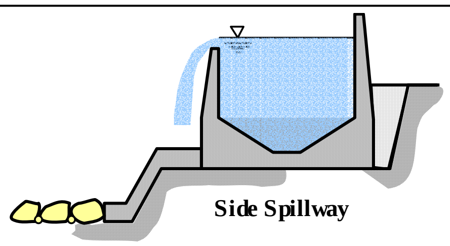
$$L = (2 \sim 3) \cdot \frac{v}{u} h_s$$

where, L : length of de-silting basin (m)

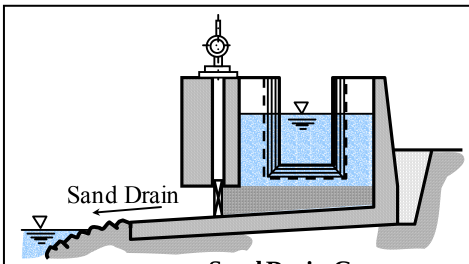
h_s : depth of de-silting basin (m)

v : average velocity in de-silting basin (m/s) = Q / (B x h_s) = 0.3 m/s

u : settling velocity for target sand particle (m/s) = 0.1 m/s for sand grains of 0.5 ~ 1.0 mm

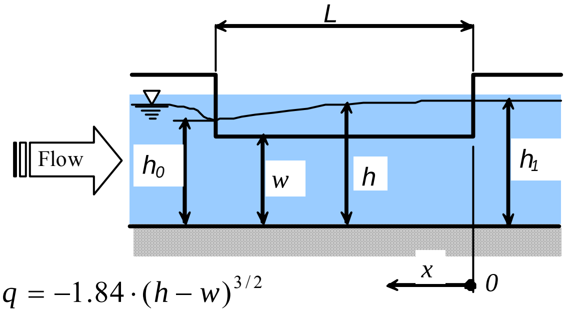


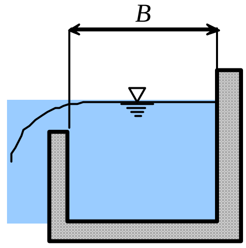
Source: JICA Study Team
Side Spillway



Source: JICA Study Team
Sand Drain Gate

The side spillway should be placed at the de-silting basin to release an excessive inflow during a flood. The length required to overflow the excessive discharge, and the water surface profile can be computed by the following De-Marchi's equations:





$$q = -1.84 \cdot (h - w)^{3/2}$$

$$L = \frac{\sqrt{2g} \cdot B}{1.84} \{\phi(\frac{h_1}{H}) - \phi(\frac{h_0}{H})\}$$

$$\phi = \frac{2H - 3w}{H - w} (\frac{H - h}{h - w})^{1/2} - 3 \tan^{-1} (\frac{H - h}{h - w})^{1/2}$$

Where q : unit overflow discharge (m^3/s/m)
h : depth of flow (m)
B : width of channel
w : height of weir (m)
h_0 : depth at downstream section (m)
h_1 : depth at upstream section (m)
$H = h + Q^2/\{2g\ (B h)^2\}$ (m)

Source: JICA Study Team

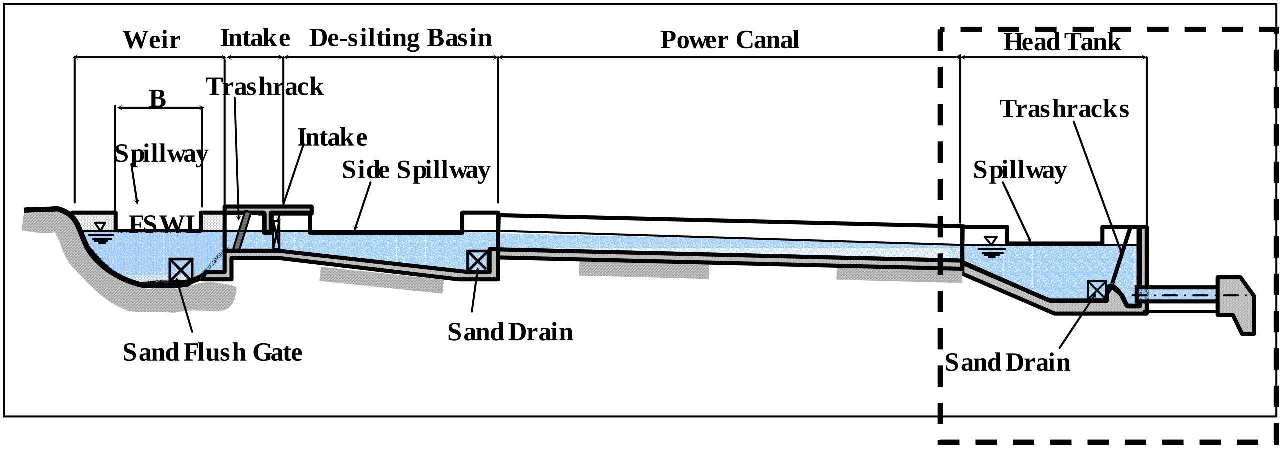
Overflow Discharge and Water Surface Profile in Side Spillway

It is noted that the outflow path needs to be protected against scouring.

De-silting Basin with Side Spillway (Nam Mong Hydropower Station)

De-silting Basin with Side Spillway (Nam Mong Hydropower Station)

3.3.5 DESIGN OF HEADTANK AND SPILLWAY



Source: JICA Study Team

Head Tank

Site Selection

The head tank is provided between the power canal and the penstock pipe to adjust a turbine discharge corresponding to the load fluctuation, while the surge tank is required when the pressure tunnel or conduit is applied as a headrace. When the penstock pipe is connected directly to the de-silting basin, the de-silting basin may be designed to have functions of a head tank.

The location of the head tank is selected generally to be on a ridge with firm foundations, depending on the topographical and geological conditions.

Source: JICA Study Team

Head Tank with Spillway

The spillway and the sand drain gate should be considered and incorporated into the head tank.

When the spillway is installed (it can be omitted under some conditions), the route of the spillway should be properly designed so as not to cause sliding or erosion of the slope.

Head Tank of Houay Nung Hydropower Station, Nale, Loungnamtha

Intake Gate into Penstock of Houay Nung Hydropower Station

Hydraulic Design

The capacity of the head tank is determined according to the responsive characteristics of the governors installed in the power plant.

(i) Mechanical governors and manual operation

$$\boxed{V > (Q_{max}) \text{ x } (120 - 180)}$$

Where,

V : capacity of tank (m^3)

A : surface area of tank (m^2)

Q_{max} : max.design discharge (m^3/s)

(ii) Electric governor, computer governor and dummy load governor

$$\boxed{V > (Q_{max}) \text{ x } 20 \text{ sec} + A \text{ x } 0.8}$$

The spillway discharge can be calculated as follows:

$$\boxed{Q = 1.84 \cdot Bs \cdot H^{1.5}}$$

Where, Q : spill-out discharge (m^3/s)

Bs : length of spillway (m)

H : overflow depth (m)

The discharge capacity of the sand drain gate is calculated by the following formulas:

(i) For the orifice flow

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

Where,

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

A : Flow area (m^2)

H = WL – Centre level of orifice (m)

(ii) For the pipe flow

$$\boxed{\cdot \sqrt{\frac{\cdot}{+ \quad _e \qquad _b}}}$$

f_e : loss coefficient for entrance (0.1 ~ 0.5)

f_b : loss coefficient for bend
$=\{0.131+0.1632(D/R)^{3.5}\} (\theta/90)^{0.5}$

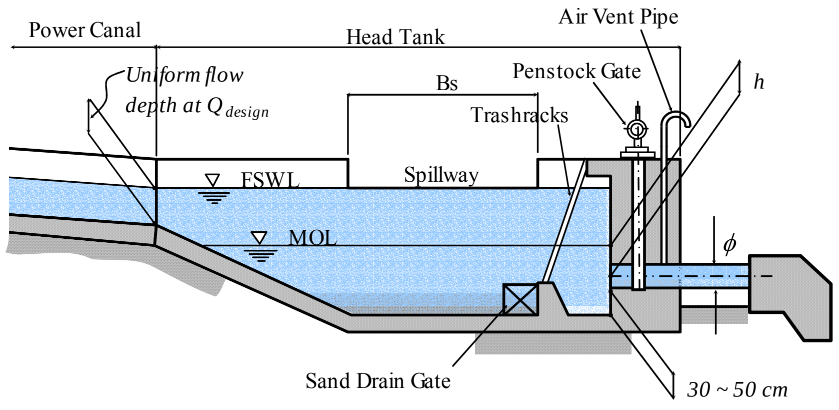
D : pipe diameter (m)

R : radius of curvature (m)

θ : bend angle (º)

f : loss coefficient for friction = $124.5n^2 L/D^{(4/3)}$

L : length of pipe



Source: JICA Study Team

Head Tank

The water depth between the Minimum Operational Level (MOL) and the centre level of the penstock inlet is given as follows:

$$h > \phi \quad (\phi < 1.0\ m)$$
$$h > \phi^2 \quad (\phi > 1.0\ m)$$

Where, h : depth between MOL and pipe centre (m)
ϕ : diameter of penstock pipe (m)

An air vent pipe is required when the inlet gate is provided at the inlet of the penstock. The diameter of the air vent pipe is given by the following empirical formula:

$$\phi = 0.0068\left(\frac{P^2 L}{H}\right)^{0.273}$$

Where,
ϕ : diameter of air vent pipe (m)
P : power output (kW)
L : length of air vent pipe (m)
H : head of penstock (m)

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

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

Omission of Spillway

The spillway of the head tank can be omitted when the discharge is regulated in the intake and the following conditions are applied:

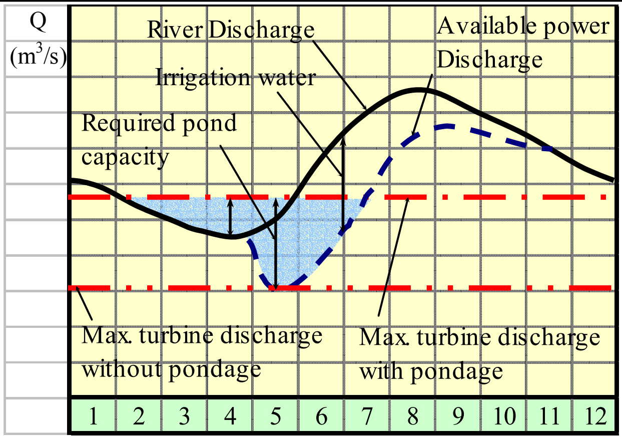
(i) Deflectors are attached to Pelton or Turgo Impulse Type turbines.
(ii) An outlet valve, branched from the penstock pipe, is provided to release the discharge during the load rejection. The valve opening is connected with the closure of the guide vane.

(iii) A dummy load governor, which is applied to mini/micro hydropower schemes smaller than 300 kW, is installed to respond to the load rejection.

3.3.6 DESIGN OF REGULATING POND

The regulating pond is provided for daily peak power generation, and the flat area is selected for its location to accommodate the required pond capacity, which needs to be enough to meet the power demand, especially during the dry season.

The pondage capacity should be determined to allow supply, with a supplementary discharge during a target operation period of time when the available discharge is insufficient for the power demand, while reserving the available water during the rest of the day.



Source: JICA Study Team

Pondage Capacity

The peak power operation can be made by monitoring the water level gauge to be equipped in the pondage. Inflow discharges can be estimated by the following equations:

$$\frac{dV}{dt} = \frac{dV}{dH} \cdot \frac{dH}{dt} = S(H) \cdot \frac{dH}{dt} = (Q_{in} - Q_{out}) \cdot 3{,}600$$

$$\frac{dH}{dt} = \frac{(Q_{in} - Q_{out}) \cdot 3{,}600}{S(H)}$$

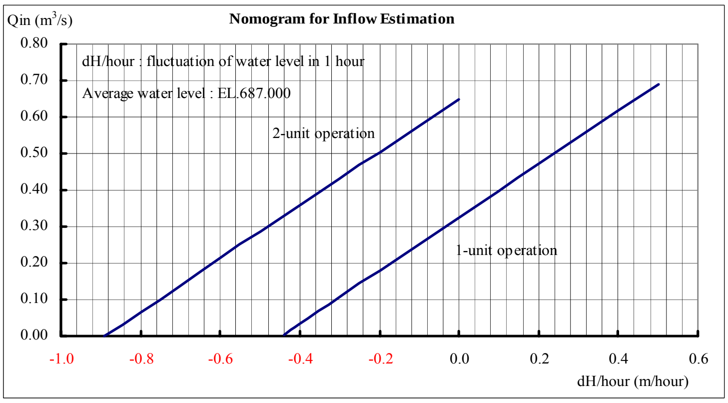
$$Q_{in} = \frac{dH}{dt} \cdot \frac{S(H)}{3{,}600} + Q_{out}$$

Where,
- H : Water level in the pond (m)
- dH/dt : Fluctuation of water level in the pond in one hour (m/hour)
- Q_{in} : Inflow into the pond (m³/s)
- Q_{out} : Turbine discharge (m³/s)
- S(H) : Surface area of the pond at water level of H (m²), which is expressed as ($aH^2 + bH + c$)

The following is an example of inflow estimate:

1) Power operation with 2-unit (320 kW, Qout=0.65 m³/s)
2) Reading of water level in the pond by pressure gauge
3) When fluctuation of water level during 1.0 hour is -0.35m , and average water level is 687.000m under 2-unit operation

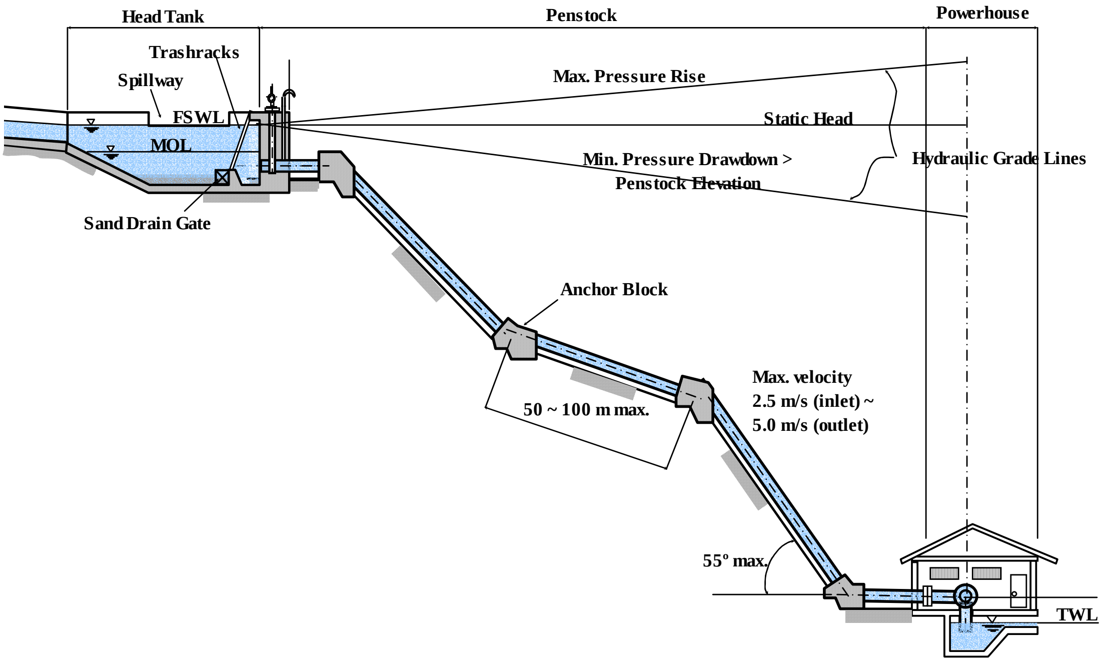
Q_{in} = -0.35 x (687.000 - 661.000) x (687.000 - 586.000) / 3,600 + 0.65 = 0.395 m³/s



The opening degree of the guide-vanes is to be kept constant during the peak time.

3.3.7 DESIGN OF PENSTOCK

The alignment of the penstock should be designed to be located below the minimum hydraulic grade line during the load rejection to avoid negative pressures.

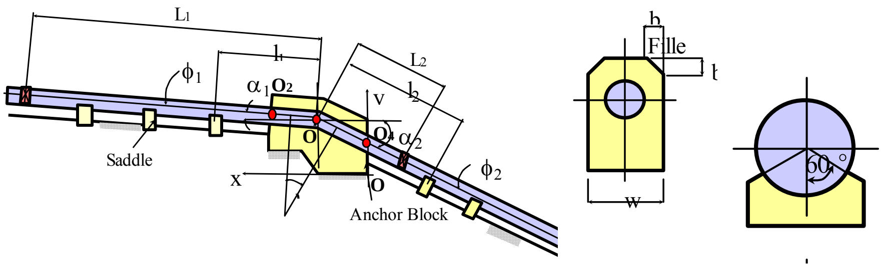


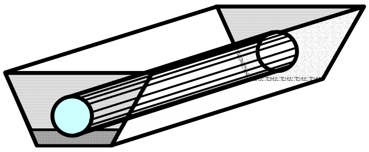
Source: JICA Study Team

Penstock

The types of the penstock are summarized as follows:

Types and Features of Penstock

Type	Features
Open type	 ● Most commonly applied to the small hydro schemes ● The anchor blocks are provided at bend portions, which should be settled on firm foundations enough to support the blocks with penstock pipes against sliding, over turning and bearing. ● The interval of each anchor block should be less than 100 m in general. ● The saddle piers are provided at 6 m intervals. ● The maximum angle of the pipe inclination should be 55° ● The drainage and slope protection should be considered for the open excavated areas. ● Expansion joints to be prepared just below the head tank and between each anchor. ● Bitumen between the pipes and anchors/saddles to avoid corrosion.

<table>
<tr><th>Type</th><th>Features</th></tr>
<tr><td>Buried type</td><td>● Applicable to the following conditions:
(a) soft foundations not to support the anchor blocks
(b) areas susceptible to damages by landslides or running water
(c) gentle slopes to keep the stability of backfill materials
● Steel pipes should be galvanized and double coated with either bitumen or high zinc content paint.
</td></tr>
<tr><td>Tunnel type</td><td>● Generally not applied in small/mini hydropower schemes.</td></tr>
</table>

Source: JICA Study Team

Open Type Penstock

Penstock of Open Type

Penstock of Open Type

Penstock of Open Type

Penstock of Open Type

Water Hammer Analysis

The water hammer can be computed by the Allievi's Equations for the simple penstock pipes without branches. The computer programs and an example are prepared.

Wave Velocity of Water Hammer

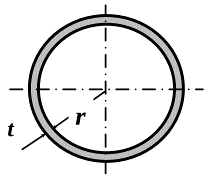
$$a = \frac{1}{\sqrt{\frac{w_0}{g}\left[\frac{1}{K} + \frac{1}{E}\cdot\frac{D}{t}\right]}}$$

Equivalent Sectional Area

$$A_m = \frac{\sum L_i}{\sum (L_i / A_i)}$$

Equivalent Wave Velocity

$$a_m = \frac{\sum L_i}{\sum (L_i / a_i)}$$



Allievi's Equations

(1) 1st phase

$$H_1^{'} - 1 = 2\rho \cdot (1 - \psi_1 \sqrt{H_1^{'}})$$

$$H_1^{'2} - (2 + 4\rho + 4\rho^2 \psi_1^2) H_1^{'} + (1 + 2\rho)^2 = 0$$

$$t_i = t_1 + (i-1)\cdot\mu$$

$$\psi_i = 1 - \frac{i \cdot \mu}{T}$$

$$\mu = \frac{2L}{a}$$

$$\rho = \frac{a \cdot V_0}{2g \cdot H_0}$$

(2) after 2nd phase

$$H_{i-1}^{'} + H_i^{'} - 2 = 2\rho \cdot (\psi_{i-1}\sqrt{H_{i-1}^{'}} - \psi_i \sqrt{H_i^{'}})$$

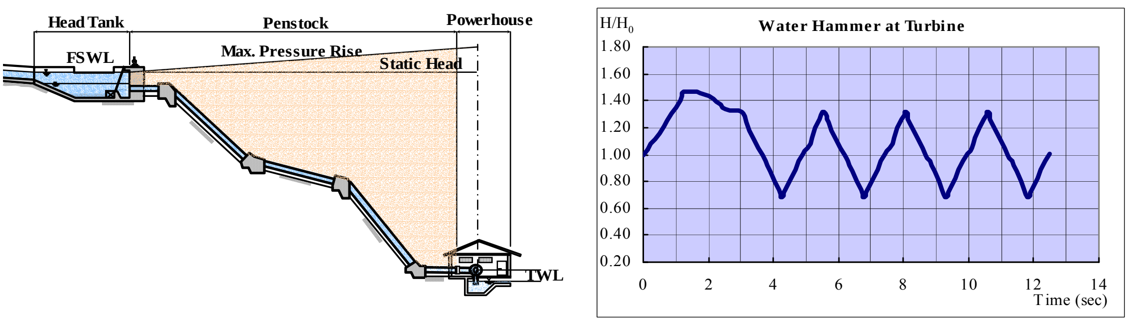
$$\rho = \frac{a \cdot V_0}{2g \cdot H_0}$$

$$H_i^{'2} - (B - 2A)\cdot H_i^{'} + A^2 = 0$$

$$H_i^{'} = \frac{h_i + H_0}{H_0}$$

$$A = H_{i-1}^{'} - 2 - 2\rho\psi_{i-1}\sqrt{H_{i-1}^{'}}$$

$$B = (2\rho\psi_i)^2$$



Source: JICA Study Team

Water Hammer Analysis

Head Loss

An effective head to be used to estimate the power output can be obtained from the head difference between Full Supply Water Level (FSWL) at the head tank and Tail Water Level (TWL) at the powerhouse after deducting head losses. The head losses between the head tank and the powerhouse are expressed as follows:

(i) Velocity Head in Head Tank

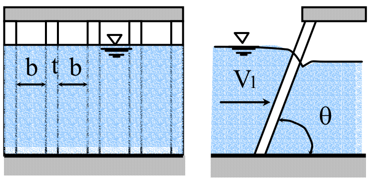
$$\boxed{h_1 = \frac{v_{in}^2}{2g}}$$

V_{in} : velocity in head tank (m/s)

h_2 : head loss at trash racks (m)
f_r : head loss coefficient
V_1 : velocity before trash racks (m/s)
θ : inclination of trash racks (º) $\theta = 60 \sim 70$º
t : width of bar (mm) $t = 5 \sim 9$ mm
b : space between bars (mm) b = 100 ~ 150 mm

(ii) Head Loss at Trash Racks

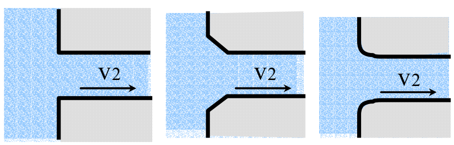
$$\boxed{h_3 = f_e \cdot \frac{v_2^2}{2g}} \qquad \boxed{f_r = 2.34(\sin\theta)(\frac{t}{b})^{4/3}}$$



h_2 : head loss by trash racks (m)
f_r : head loss coefficient
V_1 : velocity before trash racks (m/s)
θ : inclination of trash racks (º) $\theta = 60 \sim 70$º
t : width of bar (mm) $t = 5 \sim 9$ mm
b : space between bars (mm) b = 100 ~ 150 mm

(iii) Head Loss at Penstock Inlet

$$\boxed{h_3 = f_e \cdot \frac{v_2^2}{2g}}$$



$fe = 0.5$ $fe = 0.25$ $fe = 0.2$

h_3 : head loss at entrance (m)
f_e : head loss coefficient of entrance
v_2 : velocity after entrance (m/s)

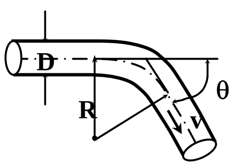
(iv) Head Loss due to Friction in Pipe

$$\boxed{h_4 = \frac{124.5n^2}{D^{4/3}} L \frac{v^2}{2g}}$$

h_4 : head loss due to friction (m)
n : roughness coefficient of pipe ≈ 0.012
D : pipe diameter (m)
L : pipe length (m)
v : velocity in pipe (m/s)

(v) Head Loss due to Bend

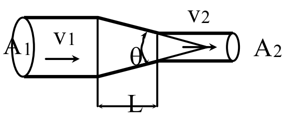
$$\boxed{h_5 = \{0.131 + 0.1632 \cdot (\frac{D}{R})^{3.5}\} \cdot (\frac{\theta}{90})^{0.5} \cdot \frac{v^2}{2g}}$$

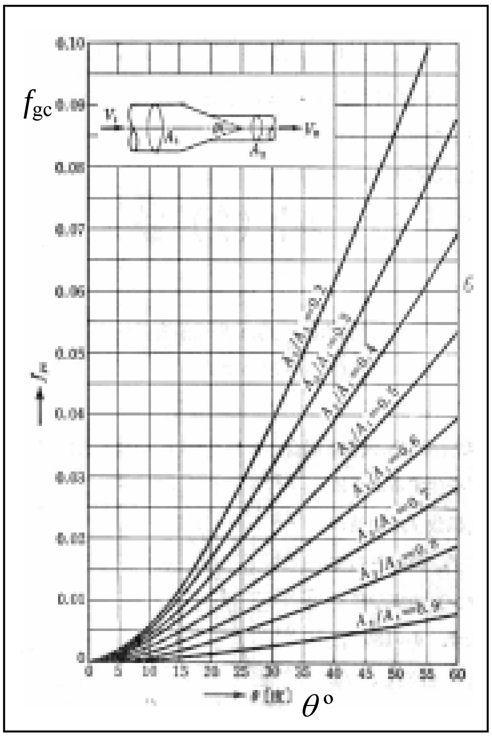


h_5 : head loss due to bend (m)
D : pipe diameter (m)
R : bend radius (m)
θ : bend angle (º)
v : velocity in pipe (m/s)

(vi) Head Loss due to Pipe Reducer

$$h_6 = f_{gc}\frac{v_2^2}{2g}$$

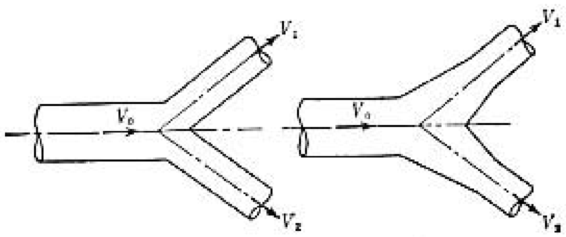




h_6 : head loss due to pipe reducer (m)
f_{gc} : head loss coefficient of reducer
θ : reducer angle (º)
L : reducer length (m)
v_1 : velocity before reducer (m/s)
v_2 : velocity after reducer (m/s)

(vii) Head Loss due to Branch

$$h_7 = f_b \cdot \frac{v_1^2}{2g}$$



h_7 : head loss due to branch (m)
f_b : head loss coefficient of branch
v_1 : velocity before branch (m/s)
(a) : $f_b = 0.75$
(b) : $f_b = 0.50$

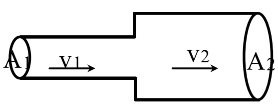
(viii) Head Loss due to Inlet Valve

$$h_8 = f_v \cdot \frac{v^2}{2g}$$

h_8 : head loss due to inlet valve (m)
f_v : head loss coefficient of valve
v : velocity at inlet valve (m/s)
Sluice valve (full open) : $f_v = 0$
Butterfly valve: $f_v = t/d$
t: Thickness of valve circle end
d: Diameter of valve circle

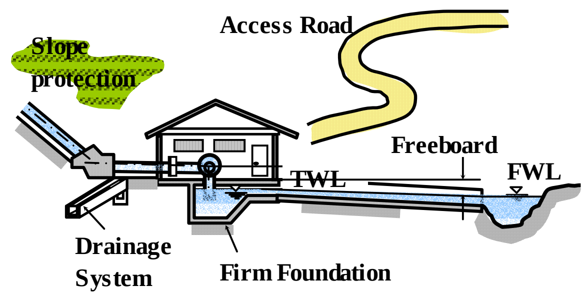
(ix) Enlargement at Outlet

$$h_9 = \{1-(\frac{A_1}{A_2})\}^2 \cdot \frac{v_1^2}{2g}$$



h_9 : head loss due to enlargement (m)
A_1 : flow area before enlargement (m^2)
A_2 : flow area after enlargement (m^2)
v_1 : velocity before enlargement (m/s)

3.3.8 DESIGN OF POWERHOUSE



Source: JICA Study Team

Powerhouse

The location of the powerhouse should be selected considering the following conditions:

(i) Access

An easy access is required for the operation and maintenance after completion.

(ii) Foundation

Rock foundations are preferable, but a well-consolidated foundation will be acceptable which can support the equipment load of 5 ton/m^2.

(iii) Safety against flooding and land sliding

The floor elevation of the powerhouse should be higher than the flood water level of the river downstream, and the slopes surrounding the powerhouse should be stabilized if required.

(iv) Drainage

The drainage facilities around the powerhouse should be properly designed to protect the powerhouse from damages caused by rush flows from the slopes and inundation during heavy rains. The Tail Water Level (TWL) at the powerhouse should be determined so that it will not be affected by the backwater from the river during a flood.

Powerhouse of Nam Mong Hydropower Station

Tailrace of Nam Mong Hydropower Station

Powerhouse of Houay Kaseng Hydropower Station

Tailrace of Houay Kaseng Hydropower Station

Powerhouse of Nam Ham Hydropower Station

Powerhouse and Tailrace

Powerhouse and Tailrace

Powerhouse and Tailrace of Nam Dong Hydropower Station

3.4 ELECTRIC EQUIPMENTS LAYOUT

3.4.1 SELECTION AND DESIGN OF TURBINES

(1) Turbine Types

Turbines are classified into two types according to their water energy utilities:

Impulse Turbines: all the available water energy is converted by a free jet through a nozzle into kinetic energy before water contacts the moving blades (runner peripheral). The energy is then taken from the jet by a suitable flow through the moving vanes. The vanes are partially filled with the jet open to the air throughout its travel to the runner. Losses occur in the flow from the reservoir through the penstock to the base of the nozzle.

Reaction Turbines: a portion of the water energy is converted into kinetic energy by the water passing through adjustable wicket gates before entering the runner, and the remainder of the energy conversion takes place through the runner. All passages are filled completely with water, including the draft tube from the runner to the downstream water surface. The static water pressure occurs on both sides of the vanes and hence does not work. The work is done entirely by the conversion to kinetic energy.

Furthermore, there are many types of turbines according to head, discharge, speed of rotation, structure, etc. The specific speed is a constant widely used in selecting the type of turbines and in the preliminary design.

1) Specific Speed

The specific speed is defined as the speed of rotation of a geometrically similar turbine working under the unit head of pressure exerting a unit power. The specific speed is given by the following formula:

$$n_s = \frac{N\sqrt{P}}{H^{5/4}}$$

n_s: Specific speed (m-kW)

N: Revolution speed (min^{-1})

P: Turbine output (kW)

H: Effective head (m)

There are limitations for n_s depending on the turbine types, and its application range is defined by a parameter of the effective head.

2) Type and Selection of Turbine

The turbines mainly applied for small hydro are categorized in the table below.

Type of Turbines and Range of Application

Type	Form	Applicable range		
		n_s (m-kW)	Effective head (m)	Maximum output (kW)
Impulse Turbine	Pelton turbine	12-25	75-500	300-5,000
	High relative speed Impulse turbine	55-65	40-300	300-5,000
	Crossflow turbine	90-110	8-60	50-1,000
Reaction Turbine	Francis turbine	50-350	20-300	300-5,000
	Package type bulb turbine	600-950	6-18	300-2,000
	S-shaped Tubular Turbine	500-850	3-18	100-3,000

Source: JICA Study Team

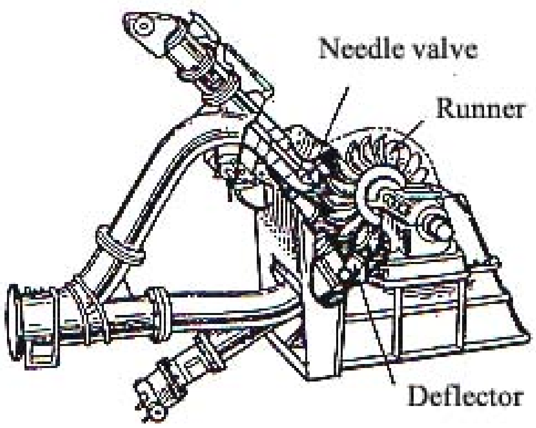
The above table includes only horizontal turbines, which are easy to install, operate, and maintain for small hydro and their output range of application is less than 5,000 kW.

(2) Structure and Features of Turbines

1) Horizontal Axis Pelton Turbine

In this turbine, the free jet from the nozzle strikes double-cupped buckets, which are coupled with the runner. The turned jets over the buckets exert a balanced force that rotates the turbine shaft. There are two types of Pelton turbines, one-nozzle type and two-nozzle type. The figure in the left shows a structure of the two-nozzle type Pelton turbine.

Runner of Pelton Turbine



Structure of Pelton Turbine

Pelton (H) Turbine at EDL Training Centre in Laos

Pelton (H) Turbine (336 kW x 3) at Nam Dong HEPP in Laos

The one-nozzle type turbine is applied to the high head and low discharge. The two-nozzle turbine is used for the relatively large discharge. In general, two-nozzle type is widely used. The nozzle is a needle valve type. The discharge of the water jet can be adjusted by moving the needle valve.

The efficiency depends on discharge. For one-nozzle Pelton turbine, the efficiency change is about 2-3% against that of around 40% from the needle full-open valve. Thus, it is possible to have a high efficiency operation up to around 20% of the maximum discharge for two-nozzle turbines. The value of n_s for the Pelton turbine is given for a single nozzle.

When the effective head is H m, the maximum power output is P kW, and the speed of rotation is N min^{-1}, the specific speed of the two-nozzle Pelton turbine is given by:

$$n_s = \frac{N\sqrt{P/2}}{H^{5/4}} = \frac{1}{\sqrt{2}} \frac{N\sqrt{P}}{H^{5/4}}$$

A deflector is attached between the top of the nozzle and the buckets, and it enables adjustment of the speed of rotation and sudden intercept of water jet flows into the runner.

The runner of the Pelton turbine needs be positioned high enough so that the runner does not touch the surface of the discharged water. The height between the bottom of the nozzle and the surface of the discharged water is the head loss.

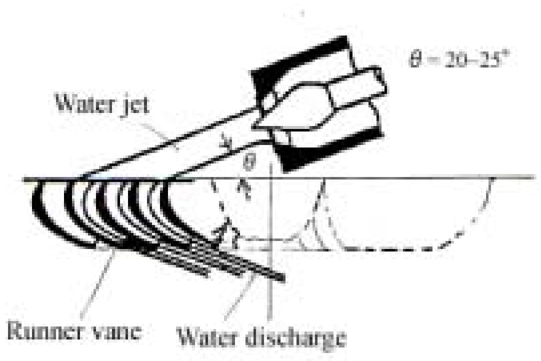
The structure, however, is rather simple. The turbine is suitable for middle and small hydro since the rise in pressure and speed at the load rejection can be controlled to low values by using a deflector.

2) High Specific Speed Impulse Turbine

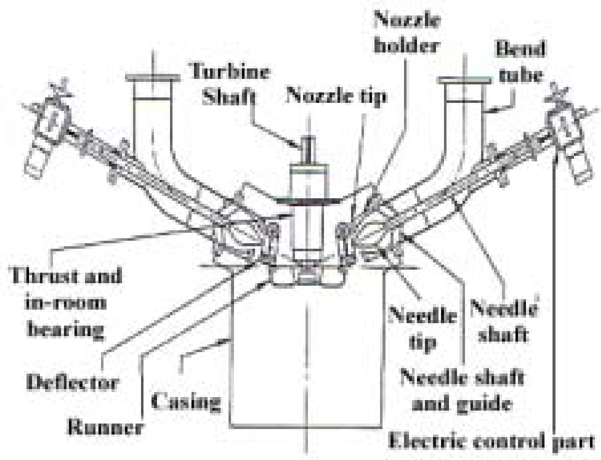
It is known as Turgo Impulse turbine, which is applied for the high specific speed range of 65~55 m-kW as an impulse turbine, so it is applicable to the relatively large discharge against head compared to the Pelton turbine.

As for its structure, it has one runner and one nozzle or two, and is similar to the Pelton turbine. However, the action of the water jet to the runner is quite different.

The figure on the right below shows the structure of a Turgo Impulse turbine with the runner and nozzle inside a casing. The left figure below shows the flow through the Turgo Impulse turbine. The Water jet from the nozzles strikes the runner at an angle of 20-25º at the top of the inlet of horizontal vanes each of three or four runner blades. The water jet is discharged toward about the same direction of the shaft from the outlet side and the opposite to the inlet direction.



Water Flow in Turgo Impulse Turbine



Structure of Turgo Impulse Turbine

Turgo Impulse Turbine in Maguse-Onsen SHP (95 kW)

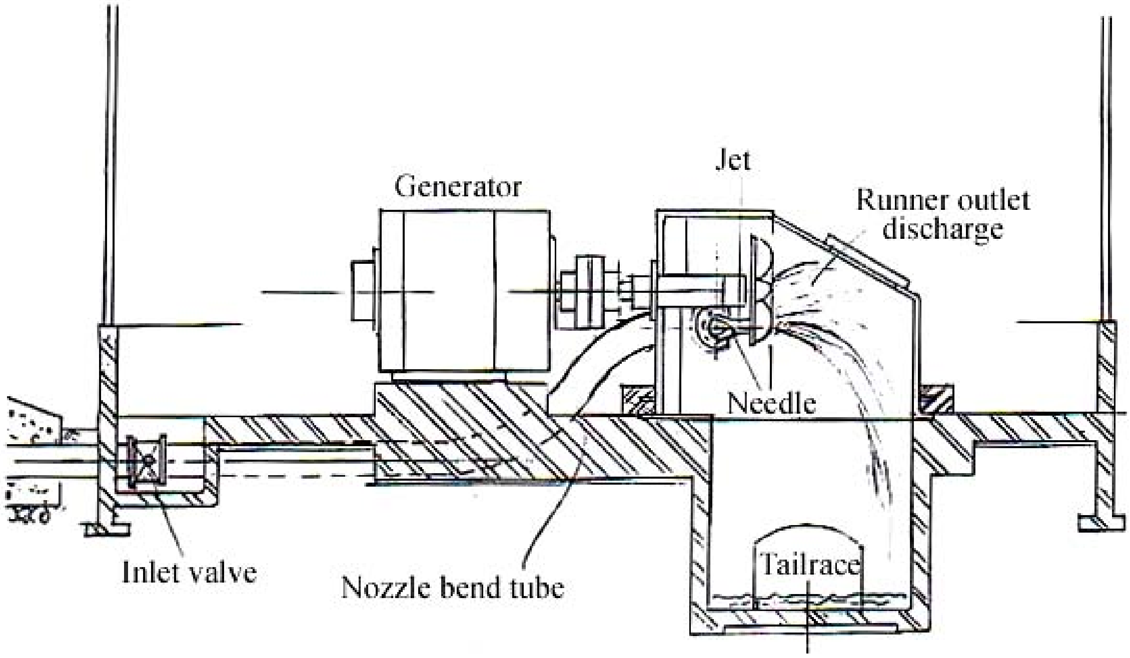
The efficiency characteristics are similar to the Pelton turbine's. However, for a horizontal jet flow, the efficiency decline at the low load is small, but the maximum efficiency is lower than that of the Pelton turbine by 2-3%.

Inner Shape of Turgo Impulse Turbine

The water jet of the Turgo Impulse turbine enters between the outer space of the runner and shaft, while the water jet of the Pelton turbine comes in contact with the outer area of the runner. Thus, the radius of the radical pitch at which the water jet operates is small in the Turgo Impulse turbine. It gains a 20-40% higher speed of rotation than the Pelton at the same head.

The next figure shows the installation of the Turgo Impulse turbine. The nozzle is directly connected to the inlet valve through a bend tube passing through the side of the generator. Thus, part of the pipe is buried so that it does not disturb operation and maintenance.

Aluminum-bronze is generally used for the material of runner. When the head is especially high or sand is expected to be mixed with the flow, it is made of cast stainless steel.



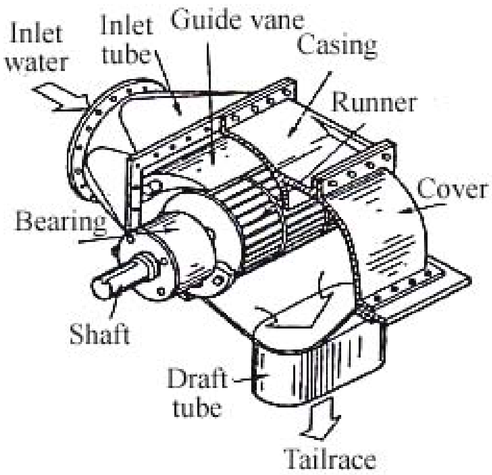
Source: JICA Study Team

Installation of Turgo Impulse Turbine and Tailrace

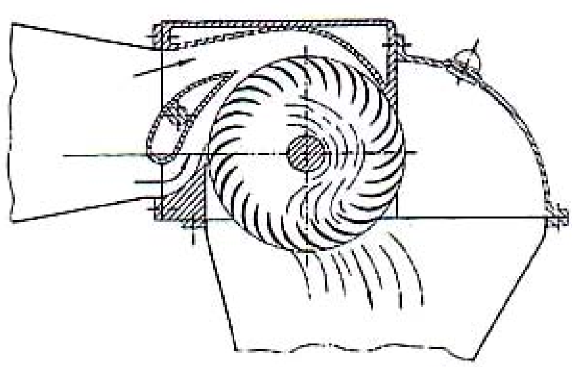
The structure of the deflector for this turbine is simple and durable, which enables the turbine to be stopped by the deflector when the nozzle is fully opened and continuing to discharge. Accordingly, the inlet valve and spillway can be omitted. It can also be applied to the river maintenance discharge.

3) Cross Flow Type Turbine

It is generally called Cross Flow Turbine. The prototype of the cross flow turbine refers to as the Michel turbine or its modification, the Banki turbine, which is applied to the relatively high head. A German manufacturer modified them for low heads, and this design was also applicable for large discharges, named Cross Flow.



Structure of Cross Flow Turbine

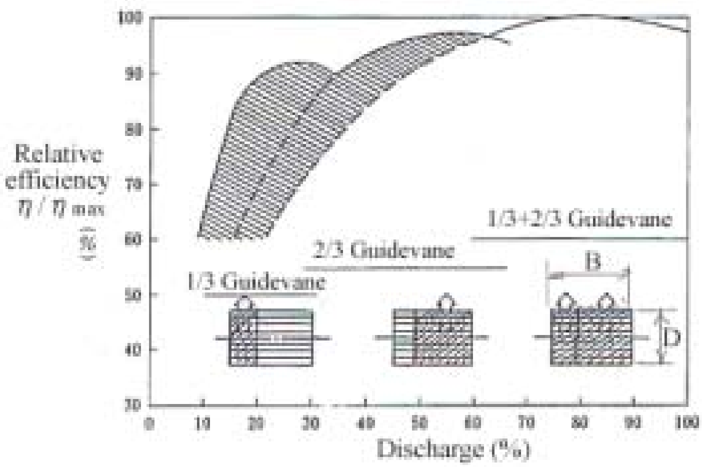


Water Flow in Cross Flow Turbine


Cross Flow Turbine at Hoshino-Onsen SHP (left: No.1, 50 kW / right: No.3, 50 kW)


Cross Flow Turbine (90 kW x 2) at Nam Ham Project

The figure in the above left shows the structure of a Cross Flow turbine. The structure is simple. The main part consists of a runner with slender vanes on the outer area and one guide vanes or two.



The water passage inside the Cross Flow turbine is shown in the figure on the right. The water is led by the guide vane and flows in through the outer side of the runner. It strikes the vanes, passes inside the runner to move the vanes again from inside, and is then discharged.

The runner is cylindrical in shape and long in axis direction. It includes one guide vane or two depending on the inlet width.

A turbine with a large maximum discharge has two guide vanes. The length of each vane is 1/3 and 2/3 of the inflow width, respectively. When the discharge is small, only the shorter vane is used.

When the discharge is more than 1/3 of the maximum discharge, the longer vane is used. If the discharge exceeds 2/3 of the maximum discharge, both vanes are used at the same time to allow a small reduction of efficiency against the change in discharge.

The figure in the previous page shows the characteristics of the Cross Flow turbine.

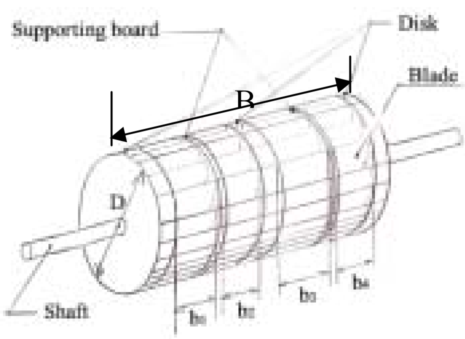
As shown in the figure, a high relative efficiency can be obtained at up to 15% of the rated discharge. This characteristic is similar to that of the low head Kaplan turbine. Therefore, the efficiency is low and smoothness of characteristics is not better than Kaplan. However, its simplicity of the structure allows an easy maintenance, low equipment prices and installation cost, which has resulted in its wide-use in small-hydro generation.

The standard runner diameter lies between 0.30 m and 1.25 m. The maximum width of the guide vane is generally some 3.5 times as big as the runner diameter.

The runner diameter and runner width can be combined at random. Thus, n_s for this turbine is calculated by assuming the unit runner output. The unit runner is defined as a runner for which the ratio of the width and diameter is 1:1, and n_s is given by the following formula:

$$n_s = NP_r^{\frac{1}{2}} D^{\frac{1}{2}} B^{-\frac{1}{2}} H^{-\frac{5}{4}}$$

n_s:	Specific speed	m
N:	Rated revolution speed	min^{-1}
P_r:	Rated output of one turbine	kW
D:	Runner diameter	m
B:	Total width of Guide vane	m
H:	Effective head	m



Source: JICA Study Team

Runner Diameter and Width

The value of n_s is between 90 and 100 m-kW.

$$n_s' = NP_r^{\frac{1}{2}} H^{-\frac{5}{4}}$$

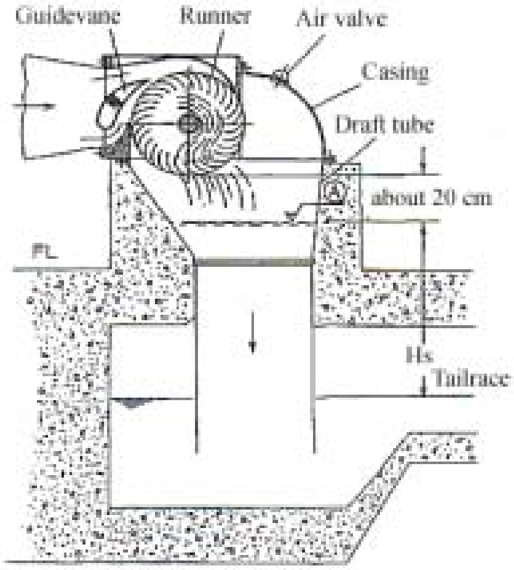
Assuming that the specific speed is n_s', where;

$$\frac{n_s'}{n_s} = \left(\frac{D}{B}\right)^{-\frac{1}{2}}$$

Then;

Thus; $\frac{\quad}{D} \quad \left(\frac{\quad}{n_s}\right)^{2}$, by which the runner width is calculated.

The Cross Flow turbine is an impulse turbine, and thus the space between the runner and the water surface of the tailrace is basically necessary. However, it is possible to utilize the draft head equivalent to 'H_s' as an effective head in attaching such a draft tube as shown in the figure to the Cross flow turbine which can absorb air through the air valve of the casing and cause about 20 cm space between the lower part of runner and the water level inside the draft tube. It is noted that the 80% of H_s is effective for the head in consideration that the draft tube contains air, therefore, the specific gravity of the water inside the draft tube will be reduced.



Source: JICA Study Team

Draft Head of Cross Flow Turbine

In general, the effective head of the Cross Flow turbine is calculated as the head to the turbine center. However, it is possible to increase the power output by adding the draft head described above. This is particularly effective when the head is less than 15 m or when the position of the turbine installation is designed to be high considering flood level fluctuations.

4) Horizontal Shaft Francis Turbine

Francis turbines are applied mostly to middle and small scale hydro schemes. They are applicable to the wide range of heads and capacity with high efficiency, and the gross head is available down to the tailrace level by the draft tube effect that enables a high speed of rotation.

Vertical turbines are mainly applied to medium and small hydro of more than 5,000 kW output. As for the horizontal axis Francis turbine with less than 5,000 kW output, there are many types according to discharges as follows:

(i) Horizontal shaft single runner – single discharge spiral Francis

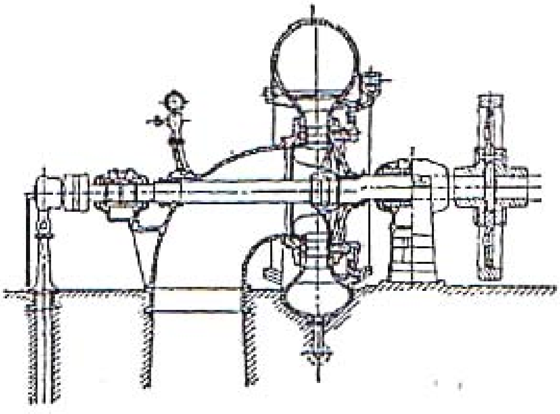
(ii) Horizontal shaft single runner – double discharge spiral Francis

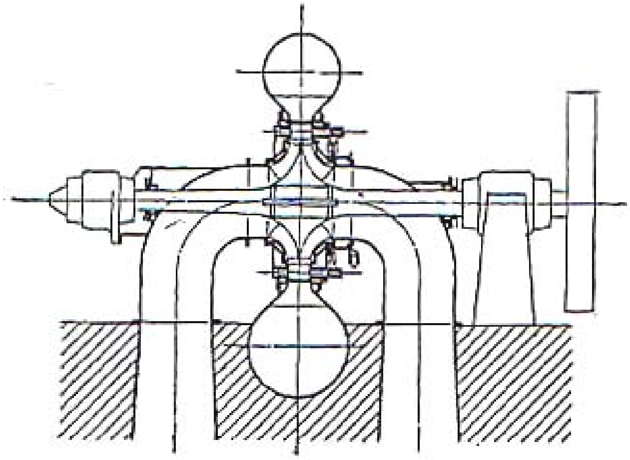
(iii) Horizontal shaft single runner – single discharge front inlet Francis

(iv) Horizontal shaft two runner – single discharge front inlet twin Francis

(v) Horizontal shaft two runner – single discharge exposure twin Francis

Among the above turbines, the most frequently used are single runner – single discharge spiral Francis turbines as shown on the next page (left), and single runner – double discharge spiral Francis turbine (right).





Spiral-Type Francis Turbine with Horizontal Shaft, Single Runner and Single Discharge

Spiral-Type Francis Turbine with Horizontal Shaft, Single Runner and Double Discharge

Francis (H) Turbine (150 kW) at Hoshino-Onsen No.2 SHP in Japan

Francis (H) Turbine (132 kW) at Yamaichi SHP in Japan

Francis (H) Turbine Model Test at EDL Training Center in Laos

Francis (H) Turbine (155 kW) at Houay Kaseng HPP in Laos

Francis (H) Turbine (500 kW x 3) at Nam Ko HPP in Laos

The other Francis turbines mainly work under low heads and large discharges, and are big in size, costly, and difficult in maintenance. Thus, they are rarely adopted recently and are replaced by other turbines such as Kaplan, Tubular, Cross Flow turbines.

Among single runner – single discharge spiral Francis turbines, those of n_s less than 100 m-kW tend to be replaced by Turgo Impulse turbines, which have better characteristics under low loads. Problems such as vibration and cavitation may take place when the load is less than 40%. It should not be selected for powerhouses that will experience long operations under low loads.

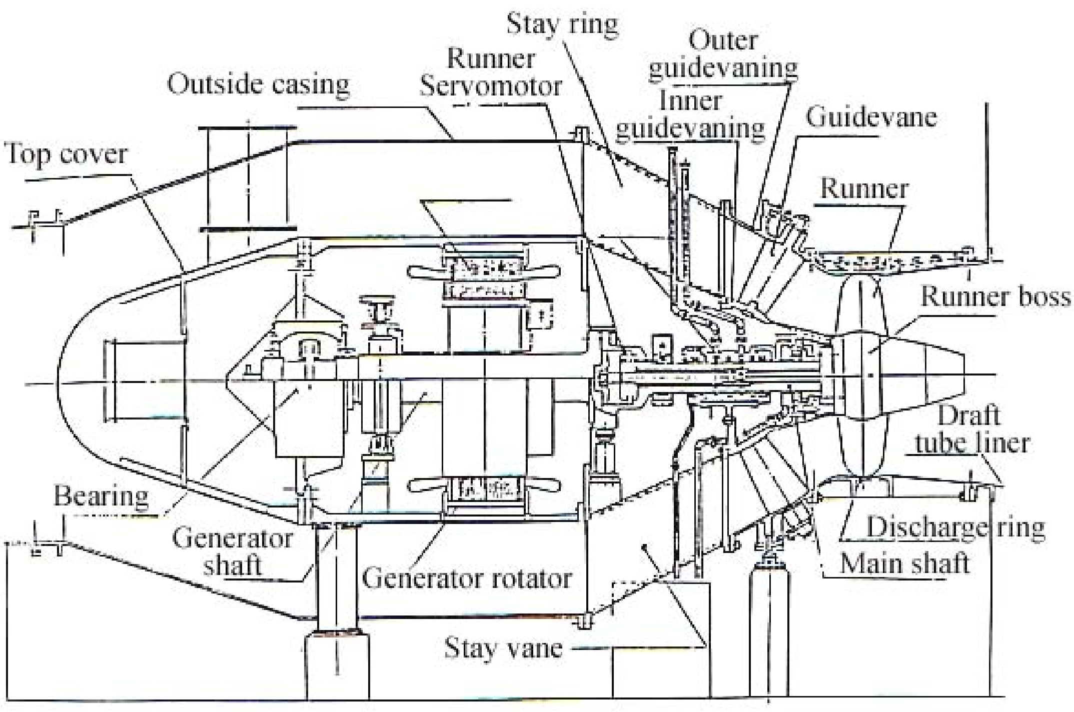
5) Tubular Turbine

The Tubular turbine is a tubular shaped horizontal propeller turbine that is applicable under low heads and relative large discharges.

It can maintain the high efficiency against the change of heads and discharges by rotating both or either runner blades of the guide vane. So far, the vertical Kaplan turbine is used in this range of low heads and large discharges. The horizontal type turbines have advantages in construction, maintenance and operation for small scale hydro and are widely adopted recently.

There are many types of the Tubular turbine. Among them, the following two types are the most popular:

a) Package type Bulb Turbines



Structure of Package-type Bulb Turbine

The structure has a bulb in the flow path, in which the turbine shaft, bearings, and generator are combined together. Because it is inserted in the middle of the turbine pipe, the installation area is small. It is applied to where there is a large fluctuation in head and discharge and to low heads with relatively large discharges utilizing the surplus head in the water supply pipe.

However, there are many issues in design. When the runner diameter is less than 1.0 m, it is not easy to make the runner vane movable, it is difficult to enlarge the diameter of the generator rotating parts, and the flywheel effect GD2 is limited.

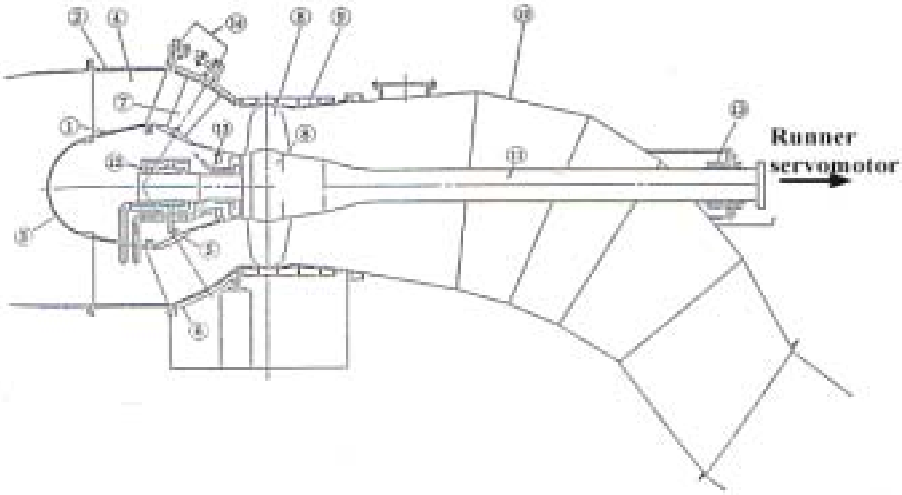
b) S-shaped Tubular Turbine

It is a characteristic of the S-shaped tubular turbine that the turbine shaft of the S-shaped tubular turbine is stretched to the outside part of the draft tube as shown in the next figure, by bending the draft of turbine in the shape of ‘S’.

Basically, the guide vane and runner parts are the same as those for the bulb turbine, but the diameter of the rotating parts can be smaller and thus it is applicable to smaller capacities. It can have a variety of designs such as a combination of fixed guide vanes and movable runner vanes, movable guide vanes and fixed runner vane, or both fixed.

It is possible to install a fly wheel and to use a speed accelerator, especially if the generator is set outside of the flow path. The cost performance is good and the operation is easy.

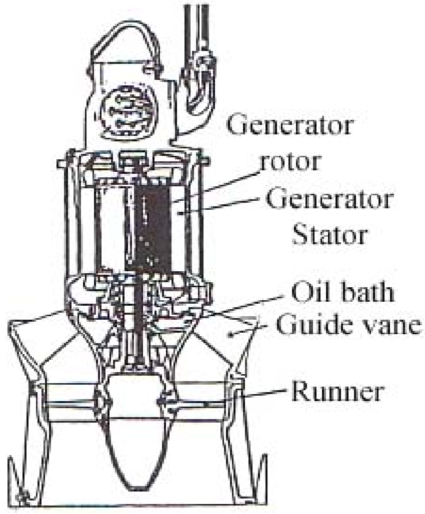
① Inside casing	② Outside casing	③ Top cover
④ Stay vazne	⑤ Inside guidevane ring	⑥ Outside guidevane ring
⑦ Guidevane	⑧ Runner vane, runner boss	⑨ Discharge ring
⑩ Draft tube	⑪ Main shaft	⑫ Main bearing
⑬ Main shaft water sealing	⑭ Guidevane operation system	



Structure of S-shaped Tubular Turbine

6) Reversible Pump Turbine

When water flows from the outlet to the inlet of a pump, the pump wheel rotates and becomes a reaction turbine by the operating water. It can also generate electricity if an induction electric motor is directly coupled with the wheel, and thus operates as an induction generator. This type of pump is called Reversible Pump turbine and is applied to small hydro schemes since the cost is low, operation is simple, and commercially produced pumps can be utilized.



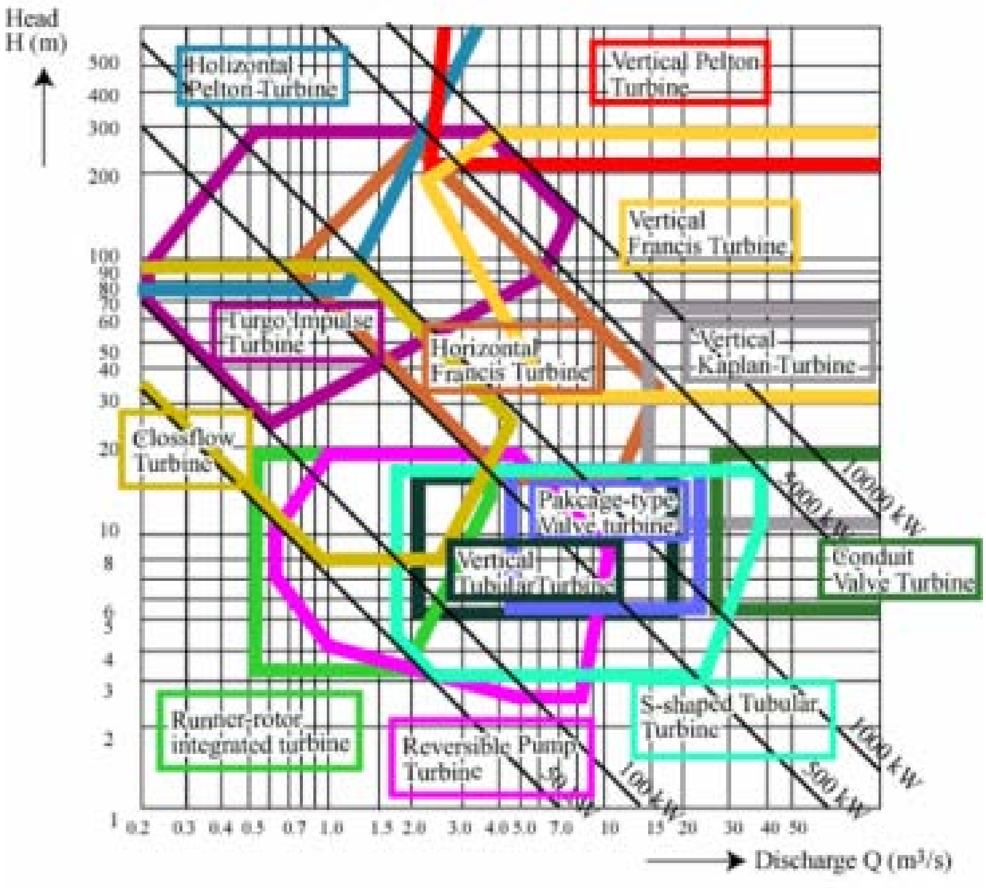
Reversible Pump Turbine

Reversible Pump Turbine (70 kW) at Nam Mong SHP in Laos

The figure above shows the structure of the Reversible Pump turbine working under the water pumped through the vertical shaft axis flow. In this type of turbines, the runner vanes and guide vanes are fixed. One disadvantage of this type of turbines is that the applicable heads and discharges are limited. Besides, it is not originally designed as a turbine. It is difficult to grasp the characteristics when applied, and data have to be determined experimentally.

(3) Selection of Turbines

The turbine type is selected according to the operating head and maximum discharge considering the range of the specific speed of each type of turbine. The figure below shows a diagram for selection of applicable turbine types for given discharges and heads. In the diagram, many types of applicable turbines may be selected for the same discharge and head. The most appropriate turbine will be selected according to the conditions of the location in consideration of size, efficiency and comparison of characteristics.



3.4.2 PRINCIPAL OF GENERATOR

There are two types of generators for hydropower, synchronous and induction. The synchronous generators are widely used to generate the three-phase alternating current with low-voltage terminal voltage for a small capacity, but in case of more than 1,000 kVA capacity, 11,000 V voltage might be applicable.

(1) Synchronous Generator

This generator type induces a voltage in armature coils by rotating magnetic poles. There are several types of exciter system such as Separate Excitation type, Static Excitation type, and Alternate Current Excitation Brushless type. The Brushless type generators are often employed in small hydro plants because they are easy to maintain.

The rotation speed of the generator is determined by the following equation based on the rated frequency of generated electricity and number of poles.

$$N = 120\, f / P$$

N : rated revolution speed (min^{-1}), f: rated frequency (Hz)

P : number of poles

Since it is desirable to directly couple a turbine and a generator, the turbine speed will be selected to be equal to that of the generator as much as practically possible. However, when it is very difficult to select the same speed between the turbine and generator or when the turbine speed is very low to require many magnetic poles to the generator (requires to obtain rated frequency at the low speed given, and will be complicated, expensive, and not readily available in the market), a speed-up gear system is employed to couple the turbine and generator. The speed-up gear is referred to as a gear system in this manual (in some references it is also referred to as a speed increaser).

For small hydro, the number of poles used is generally up to 12. For more than 12 poles, the generator is generally larger and more expensive, therefore, application of speed-up gears is necessary for increasing the turbine speed together with application of a 4-6 pole high-speed generator. Those are necessary especially for the Cross Flow turbine.

The following items constitute the specifications of a synchronous generator.

Direction of shaft	Horizontal
Capacity	kVA
Voltage	*V*
Current	*A*
Power factor	(0.8～0.5)
Frequency	Hz
Revolution speed	min^{-1}
Insulation class	(F class)
Type of bearing	(thrust bearing required or not ?)
Type of excitation	
Short circuit ratio	(specified in case of large size)

(2) Induction Generator

The Induction Generator has a rotating structure composed of a primary and a secondary winding, and electricity is generated through electromagnetic induction between the windings.

It is applied to the powerhouse which is less than 1,000 kW, connected to the power grid in parallel. The generator structure can be simple and keep cost low by applying a squirrel cage type secondary winding. Generally, this type of generator cannot generate independently. Operations must be established by supplying an excitation current to the primary winding from another power source. In addition, the generator causes such a rush current which corresponds to the rated current several times when it is connected to the power system on null voltage.

However, it tends to be applied to small hydro because of its low cost, simple maintenance, and easy operation and control. Induction motors are applied to the generators at a low cost. In this case, it should be noted that the induction motor is not able to withstand the over speed condition.

The following items comprise the specification of the induction generator.

Axis direction	Horizontal
Rated	Continuous
Output	kW
Voltage	*V*
Current	*A*
Power factor	(Manufacturer's specification)
Frequency	Hz
Revolution speed	m^{-1}
Slip	%
Insulation class	(F class)

Since the induction generator is operated by an excitation current from the connected power grid, the isolated operation and power factor adjustment are not possible.

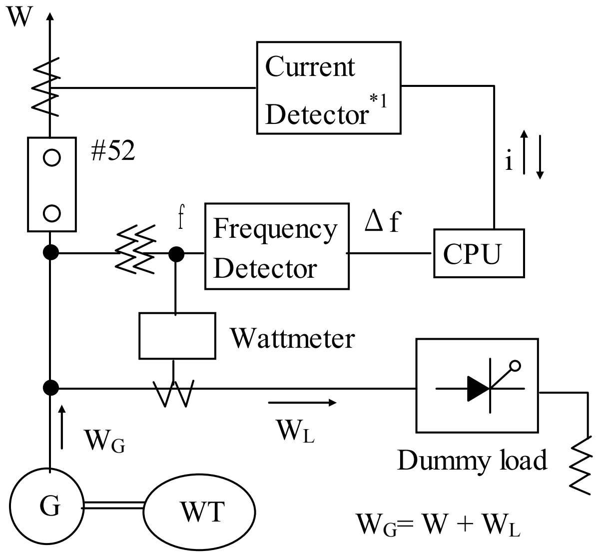
To improve the power factor, a parallel condenser is connected to the generator. When the load shedding occurs with a large condenser in place, it is required to pay attentions to the following fact: As the rotational frequency of the generator increases, the generator is excited by the leading current in the condenser, which eventually causes a self-excitation phenomena in a high voltage.

3.4.3 CONTROL UNIT

(1) Governor

The governor adjusts the water inflow mechanism such as guide vanes, needle valves and deflectors, and controls the water inflow, turbine rotation speed, and output. As the parts that directly adjust the water inflow require a large force, a hydraulic servomotor is used for medium-small scale hydro.

For small-hydro, an electric servomotor is applied due to its accurate control as well as easy maintenance and inspection. The CPU and electronic circuit detect control parameters (such as speed, water level, discharge, and output), compute the required range of control, and transmit the control signal to the servomotor. The CPU tends to be included in an integrated control unit with another operation control unit. For units less than 200 kW, the Dummy Load Governor may be applied.



*1 detect current and its director

The Dummy Load Governor is illustrated on the right. A dummy load is connected in parallel with the demand load. This keeps the frequency constant by adjusting the dummy load automatically so that power generation and the load become the same by detecting the generator output, demand load change and frequency change to balance the total of the dummy load and the demand load with the generator power output. This governor does not involve mechanical operations like a servomotor.

Accordingly, the control characteristics are fine and it can respond to sudden output change even from minimum to maximum. This governor is applicable for turbines without a water inlet adjusting mechanism, such as the Reversible Pump turbine or isolated operation in a small grid.

Hydraulic Servomotor

(2) Integrated Control Panel

An integrated control panel uses the CPU to perform integrated operations of control, operations of protective equipment, and storage of operation records for the powerhouse. It is contained in a small board.

Examples of the functions included in the integrated control panel are as follows:

Control functions	Operation sequence, starting speed control, voltage control, power factor control, frequency control, load control, water level control, discharge control, program operation control, etc.
Protective functions	Over current, over voltage, under voltage, bus ground fault, lack of phase fault
Display functions	Voltage, current, power output, power factor, frequency, water level, opening, bearing temperature, faults, etc.
Recording functions	kWh, discharge, historical record of fault , daily operation report, monthly report, annual salary, testing record, etc.

(3) Direct Current Power Source Unit

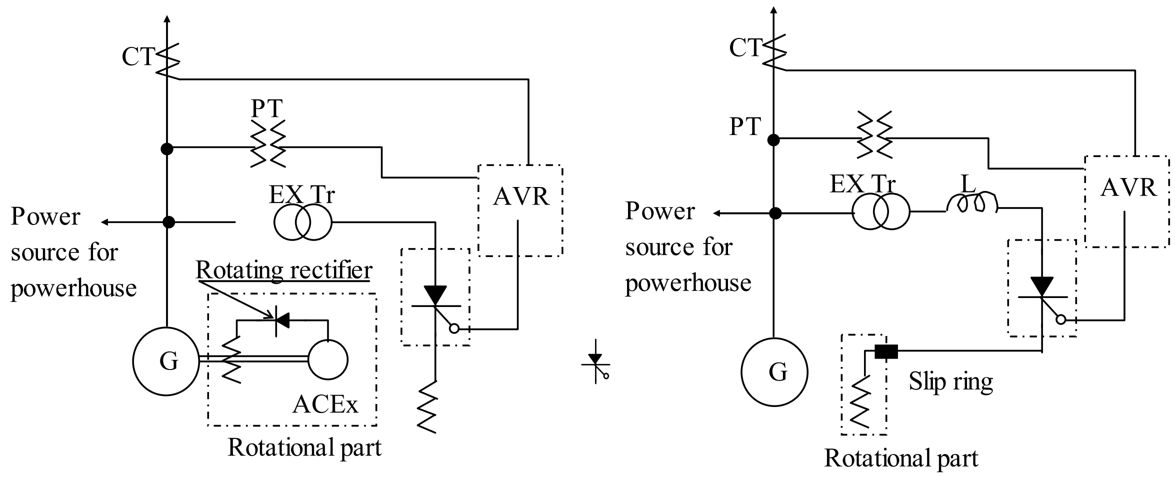
Batteries with chargers are used as a power source for the operation control and protective relay. Commonly, 100 V is used, but 24 V may be applicable for small capacity power station.

There are two types of batteries, lead acid and alkali. Alkali batteries adopted more commonly these days because of its simple maintenance.

(4) AVR

This device controls power generation by adjusting the excitation current in the Synchronous Generator. The following figures show two types of AVR application.

In both types of AVR, the AVR detects a voltage change in the generator bus and adjusts the excitation current by controlling the SCR gate. The Brushless Excitation AVR has a small excitation current and is easy to control. The Static Excitation AVR requires a large current and the time constant of the magnetic field circuit is rather long, so they need careful attention to control.



AVR	: Automatic voltage regulator	CT	:Current Tanasformer
ACEx	: Alternating current exciter	Ex/Tr	:Transformer for excitation
G	: Generator	L	:Reactor
PT	: Potential Transformer		:Thyristor rectifier

(a) Brushless excitation type (b) Static excitation type

Source: JICA Study Team

Excitating Circuit with AVR

Both AVRs of brushless excitation and static excitation use the exciting current in the generator. Therefore, such a way can compensate a voltage drop caused by the load current, which is so called "compound-wound characteristics".

The power factor in the isolated operation is given by the load power factor, which can not be improved by adjusting the excitation current. In the grid parallel operation, some AVRs are equipped with a power factor control unit and the reactive power adjustment by the excitation current.

3.4.4 INLET VALVES

The inlet valve is installed near the turbine inlet at the end of the penstock. The usage and purposes are as follows:

(i) Shutting off a flow path when the turbine is stopped.

(ii) In this case, the water inflow to the runner is closed off by the preceding operation of the guide vane or the needle.

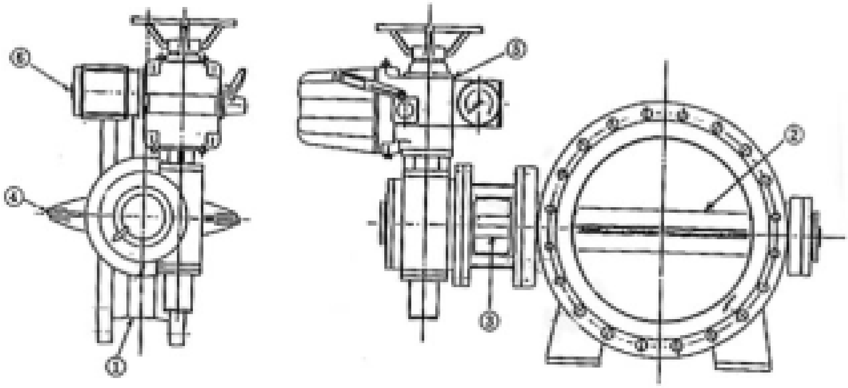
(iii) Shutting off the water flow when the guide vanes and/or needles cannot be controlled.

(iv) Stopping water flow during the turbine inspection.

(v) When the diameter of the inlet valve is large and there is a low head and large discharge, the inlet valve may be omitted by providing a regulating gate at the inlet of the penstock in the head pond.

(vi) Butterfly Valves, Double Leaf Valves, and Sluice Valves are applied as inlet valves.

(1) Butterfly valve



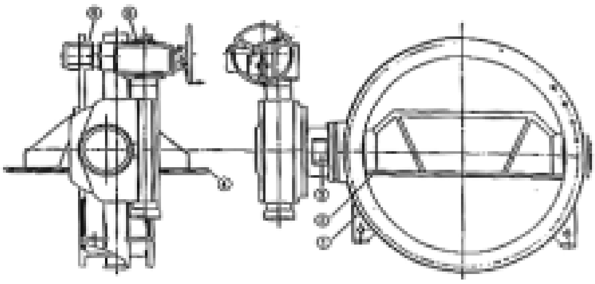
No.	Items	Material
①	Valve Frame	Pressed and stretched steel plate for general structure
②	Valve Body	Ditto
③	Valve Axis	Stainless steel, Cast carbon steel, or carbon steel for mechanical structure
④	Valve Sheet	Rubber
⑤	Speed Reducer	-
⑥	Electric Motor	-

Structure of Butterfly Valve

Butterfly Valve in Yukawa No.2 SHP in Japan

The Butterfly Valve is applied for heads of less than 150 m. The structure is shown in the figure above. There are the horizontal axis type and the vertical axis type valves as shown in the table. The horizontal axis type valves have the advantage that the weight of the valve itself is easy to support.

The valve diameter is generally 1.1 to 1.2 times the turbine inlet diameter. The structure is simple. The sealing method is good in the rubber seal type valves. The head loss is rather large compared to other valve types, but this valve type is still applied to the inlets with relatively small diameters because of its low cost.

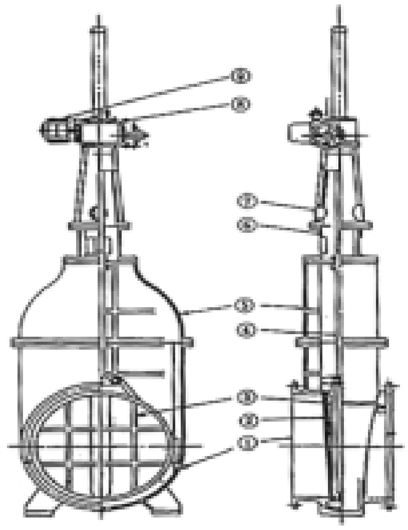


No.	Items	Material
①	Valve Frame	Pressed and stretched steel plate for general structure
②	Valve Body	Ditto
③	Valve Axis	Stainless steel, Cast carbon steel, or carbon steel for mechanical structure
④	Valve Sheet	Rubber
⑤	Speed Reducer	-
⑥	Electric Motor	-

Structure of Through-flow Valve

The Double Leaf Valve is suitable for heads of less than 200 m. The structure is the same as the Butterfly Valve, but the valve body is thin and obtains strength by combining two pieces in order to reduce head losses. The structure is shown in figure above.

The valve diameter is 1.1 to 1.2 times the turbine inlet pipe diameter, similar to the Butterfly Valve.



No.	Item	Material
①	Valve Frame	Cast carbon steel
②	Valve Sheet	Bronze
③	Valve Body	Cast carbon steel
④	Axis of Valve	Stainless steel or carbon steel for mechanical structure
⑤	Cover	Cast carbon steel
⑥	Stand	Cast iron
⑦	Stand	Ditto
⑧	Speed Reducer	-
⑨	Electric Motor	-

Structure of Sluice Valve

Sluice Valve (left: Trongsa SHP in Bhutan, right: Maguse-Onsen SHP)

The Sluice Valve is applied to high heads and small discharges. It has small head losses. The structure is shown in the figure above.

There are two types of spindles that operate the valve electrically. One is the inner screw type with a female screw attached to the valve body and a male screw on a spindle is slewed for the up-down movement. The other type, the outer screw, slews a female screw at the top of the spindle. The former is applied to the inlets with relatively large diameters, but the latter has the advantage in safety and is applied for hydro more frequently.

(2) Types of Operation and Drive

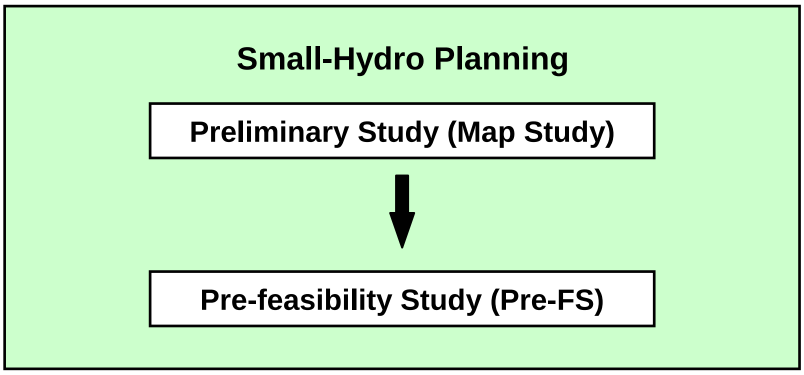
In the case of automatic operations, there are two types of drives: the motor driven and hydraulic oil driven types.

The hydraulic type drive is used as a large diameter application, such as the Tubular turbine. Generally, the motor drive is widely adopted. The power source for the driving motor is basically the direct current, but the alternate current may be applied. It is said that opening and closing time is set at shorter than 180 seconds.

3.5 COST ESTIMATE

3.5.1 LEVELS OF PRELIMINARY COST ESTIMATE

A small-hydro planning is conducted usually in two (2) steps; a **Preliminary Study** (**Map Stud**y), and a **Pre-feasibility Study** on the selected schemes. The objective of these studies above is to identify technically and economically sound small-hydro sites close to load centers.



Because of the difference of the study level (depth) on the sites (more detailed information and data with the field investigations such as topographic survey, geological survey, etc. is available at the pre-feasibility study level), the approach for estimating the project cost differs each other.

3.5.2 COST ESTIMATE AT THE MAP STUDY LEVEL

The main purpose of the Map Study is to identify hydropower potential sites and to select favorable schemes for pre-feasibility study as a next step. The Map Study has the following limitations for planning:

- Information on the topographical features and river conditions is only from the available topographical maps with a 1:100,000 scale (contour interval is from 20 to 40 m depending on maps).

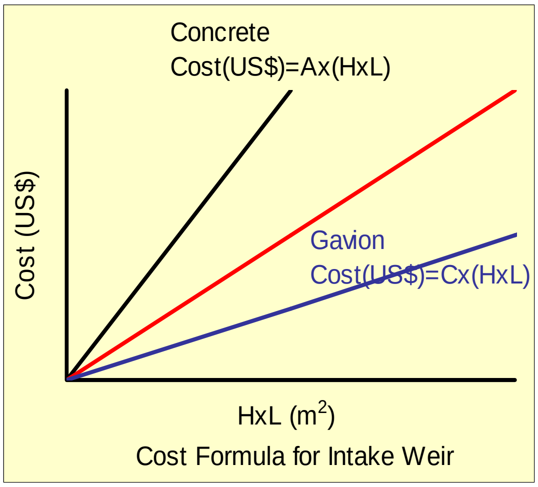
- Assessment of the hydropower potential (power discharge, head, installed capacity, etc.) is very rough containing unavoidable error to some extent.

- Layout of the scheme is very rough and preliminary. No layout drawings are made for each scheme.

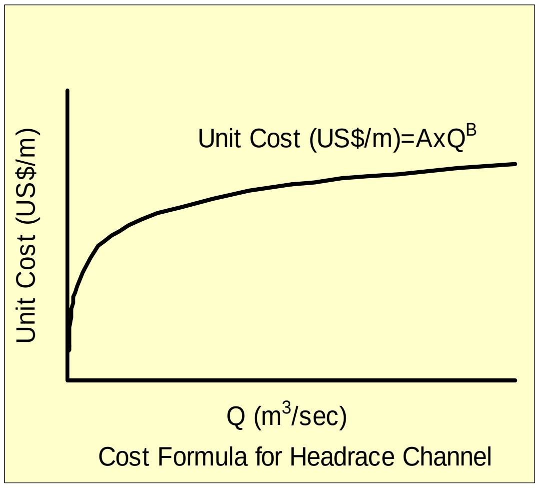
Thus the cost estimate at the Map Study level is very preliminary with a margin for error.

In practice, the cost estimate is conducted by applying the various Cost Formulas for quick cost estimate on the many identified schemes. The cost formulas are established based on the construction cost data obtained in various projects in Lao PDR. The estimated cost shall be expressed in US$ at the current price level. The old cost shall be updated to the current price level before establishment of the formulas.

Table below is an example of the cost formulas for the intake weir and power canal (headrace channel):



Cost Formula for Intake Weir



Cost Formula for Headrace Channel

Under the Map Study, the preliminary cost estimate for the direct construction cost was made for the following main cost item by means of its key parameter:

Main Cost Item	Key Parameter	Cost Formula
Hydropower Generation Plan	Installed Capacity (kW) Design Discharge (m^3/sec) Head (m)	-
Intake Weir	Height of Weir (m), Length of Weir (m)	4,000 US$/$m^2$
Headrace Channel	Design Discharge (m^3/sec), Length of Headrace (m)	Formulas including channel excavation and concrete lining
Head Tank	Design Discharge (m^3/sec)	Formulas including channel excavation, wet masonry and concrete lining
Penstock	Design Discharge (m^3/sec), Length of Penstock (m)	Formulas including concrete works and penstock weight
Powerhouse	Installed Capacity (kW)	40 US$/kW
Turbine and Generator	Installed Capacity (kW)	400 US$/kW
22 kV Transmission Line	Length of Transmission Line (km)	10,000 US$/km
Transformer	No. of Village to be Electrified	6,000 US$/unit
Access Road	Length of Access Road (km)	50,000 US$/km

For the estimation of the total project cost, the indirect costs shall be added to the estimated direct construction cost above. The indirect costs are; i) Miscellaneous cost (10% of the direct construction cost) and ii) Contingency (20% of the direct construction cost and miscellaneous cost).

Reference on the kW Cost for Very Quick Cost Estimate:

Table blows shows the installed capacity and kW cost of the existing 10 small-hydro projects in Lao PDR for reference. As seen below, the kW costs of both Nam Peun and Nam Mong plants are considerably high compared to other plants. This is because these plants were respectively constructed as a pilot scheme by the Governments of Japan and Germany. **All of other plants are made in China. It is found that the kW cost made in China varies 3,000 to 6,000 US$/kW.**

Existing Small-Hydro Projects in Lao PDR（10 Sites）

No.	Project	Installed Cap. (kW)	Number of Turbine	Cost (US$)	Unit Cost (US$/kW)	Year	District	Province	Turbine Made in
1	Nam Ko	1,500	3x500	9,815,071	6,543	1996	Xai	Oudomxai	China
2	Nam Sam	110	2x55	678,000	6,163	1995	Xamtai	Huaphan	China
3	Nam Peun	60	1x60	1,791,000	29,850	1986	Huamuang	Huaphan	Germany
4	Nam Sipkha	55	1x55	220,030	4,000	-	Kham	Xieng Khouang	China
5	Nam Tien	75	1x75	227,661	3,035	-	Kham	Xieng Khouang	China
6	Nam Chat	100	1x100	366,451	3,665	-	Mot	Xieng Khouang	China
7	Ban Nong	40	1x40	166,467	4,162	1995	Phaxai	Xieng Khouang	China
8	Nam Ka	81	55+26	312,285	3,855	1995	Phaxai	Xieng Khouang	China
9	Houay Kasen	155	155	758,000	4,890	2002	Pakbeng	Oudomxai	China
10	Nam Mong	70	1x70	820,000	11,714	2000	Nam Bak	Louang Prabang	Japan

The location map of the existing small-hydro plants is shown in the next page.

3.5.3 COST ESTIMATE AT THE PRE-FEASIBILITY STUDY LEVEL

The cost estimate at the pre-feasibility study level is generally conducted by applying the Unit Price Method.

The direct construction cost of the small-hydro project generally consists of the cost components for the following major works:

(i) Civil Works

(ii) Electro-Mechanical Works

For the construction works of the small-hydro project, the above works comprises the following major Work Items:

Components	Work Item
Civil Works (Intake Weir, Intake, Headrace Channel /Tunnel, Head Tank, Penstock, Powerhouse, Tailrace, etc.)	Excavation-common
	Excavation-rock
	Excavation-channel
	Excavation-tunnel
	Concrete
	Gabion
	Wet Masonry
	Miscellaneous
Electro-Mechanical Works (Metal Work, Distribution Work, etc.)	Steel Penstock
	Gate and Trashracks
	Turbine and Generator
	Distribution line
	Transformer and Switchgears
	Miscellaneous

There are various types of structures in each component such as Intake, Headrace Channel / Tunnel, Head Tank, Penstock, Powerhouse and Tailrace.

Furthermore, each Structure comprises the various Work Items. For example, the construction work of the intake weir mainly consists of the following works items; excavation, concrete, gabion, reinforcing bar and other works. Each work quantity is usually obtained by the empirical formula or directly from the layout drawings. It is necessary to calculate the work quantity for each work items based on the design. Thus the unit price shall be also prepared for Works Item.

The cost of work item is thus estimated multiplying the **Work Quantity** by its **Unit Price** as given below.

Cost of Work Item = Unit Price x Work Quantity

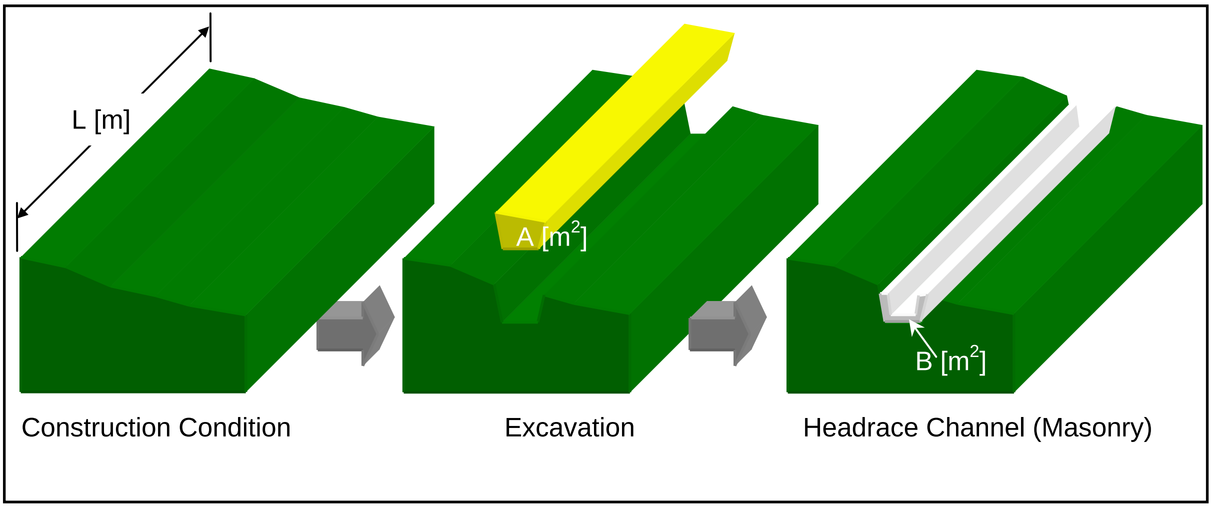
The total direct construction cost is estimated by summing up all the cost of Work Items.

In order to estimate various unit prices necessary for the cost estimate, the cost estimates undertaken by other studies/projects in Lao PDR shall be analyzed. Several unit prices for the civil works are also available at PDIH. Table below shows the major **Unit Prices of Civil Works** to be applied under the Master Plan Study.

Unit Price Table for the M/P Study on Small-Hydro in Northern Laos

Work Item	Unit Price Table			Remarks
Excavation-common	V_E	US$/m^3	1.50	
Excavation-rock	V_E	US$/m^3	4.50	
Excavation-channel	V_E	US$/m^3	2.00	
Excavation-tunnel	V_E	US$/m^3	50.00	
Concrete	V_C	US$/m^3	220.00	Incl. Re-bar & Form
Wet Masonry	V_C	US$/m^3	70.00	
Gabion	V_C	US$/m^3	70.00	
Gate	W_G	US$/ton	6,000.00	
Screen	W_S	US$/ton	3,000.00	
Penstock	W_P	US$/ton	4,000.00	
Turbine & Generator	E	US$/ton	4,000.00	

A breakdown of the construction cost for the civil works under the Master Plan study is as per attached. This is the same approach of cost estimation for other structures, and it should take care that there is no oversight in the work item that should be listed up based on the layout drawings.



Illustrator of Work Items for Headrace Channel Construction

The Work Item of the Headrace Channel is Excavation and Masonry.

The quantity of Excavation is the cross sectional area multiplied by its length, and the construction cost is quantity multiplied by its unit price. The quantity of Masonry is the cross sectional area multiplied by its length, and the cost is quantity multiplied by its unit price. The headrace channel construction cost is summing the cost of excavation and masonry.

Then, calculate the each construction items in case of intake, headrace channel, head tank, powerhouse and tailrace.

Total Project Cost is calculated by summing up each construction items as following example

BILL OF QUANTITY

No.	Work Item	unit	Q'ty	unit price (US$)	Amount (US$)
1.	**Civil Works**				
1.1	**Intake Weir**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Gabion	m^3		70.00	
	Sub-total				
1.2	**Intake**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Sub-total				
1.3	**De-silting Basin**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Sub-total				
1.4	**Headrace Channel or Tunnel**				
	Tunnel Excavation	m^3		50.00	
	Channel Excavation	m^3		2.00	
	Concrete	m^3		220.00	
	Wet Masonry	m^3		70.00	
	Sub-total				
1.5	**Head Tank (Surge Tank)**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Sub-total				
1.6	**Spillway**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Sub-total				
1.7	**Penstock**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Sub-total				
1.8	**Powerhouse**				
	Excavation-common	m^3		1.50	
	Excavation-rock	m^3		4.50	
	Concrete	m^3		220.00	
	Sub-total				
1.9	**Tailrace**				
	Excavation-channel	m^3		2.00	
	Concrete	m^3		220.00	
	Wet Masonry	m^3		70.00	
	Sub-total				
1.10	**Access Road**	km		10,000.00	
1.11	**Miscellaneous**	%	30		
	Total of Civil Works				
2.	**Steel Penstock**	ton		3,000.00	
3.	**Gate and Trashracks**	ton		1,500.00	
4.	**Turbine & Generator**	L.S.			
5.	**Transformer and Switchgears**	L.S.			
6.	**Distribution Lines**	km			
7.	**E&M Miscellaneous (2 ~ 6)**	%	10		
	Total of E&M Works				
8.	**Administration & Engineering Fee**	%	15		
	GRAND TOTAL				

3.6 ECONOMIC AND FINANCIAL EVALUATION OF PROJECT

3.6.1 GENERAL

Evaluation of an efficiency of small-hydro project is based on the following four aspects; i) technically sound, ii) socially and environmentally acceptable, iii) economically efficient, and iv) financially viable. The latter two aspects are evaluated on the basis of an economic analysis and financial analysis, respectively.

The economic analysis is the way to evaluate the project by comparing the social and economic benefit with the construction cost from the viewpoints of public interest or national economy. On the other hand, the financial analysis is the way to evaluate the profitability of project comparing the revenues to investment cost from the standpoint of business enterprise.

3.6.2 KEY COMPARATIVE INDICATORS FOR EVALUATION

Key comparative indicators for evaluating economical and financial viability of the project are as follows:

- i) Present Value (PV)
- ii) Net Present Value (NPV) (B-C)
- iii) Cost-Benefit Ratio (B/C)
- iv) Internal Rate of Return (IRR)
- v) Unit Energy Cost (kWh Cost)

(1) Present Value (PV)

In view of the time value of money, table below shows an example of compound amount calculation of bank account under the following conditions:

Present value (deposit money) : P or Kip 1,000 (table on the right)

Compound Interest Rate : i or 0.1 (10%) (table on the right)

Deposit period (years in the future) : 10 years

Example of Bank Account

Year End	Interest	Total Amount		Interest (Kip)	Compound Amount (Kip)
1st	P x i	P(1+i)		1,000 x 0.1=100	1,100
2nd	P(1+i) x i	$P(1+i)^2$		1,000(1+0.1) x 0.1=110	1,210
3rd	$P(1+i)^2$ x i	$P(1+i)^3$		$1,000(1+0.1)^2$ x 0.1=121	1,331
....	....	....		....	....
10-th	$P(1+i)^9$ x i	$P(1+i)^{10}$		$1,000(1+0.1)^9$ x 0.1=236	2,594

As seen above, the present value (PV) and compound amount (CA) in 10 years' time are summarized as follows:

$$PV = \text{Kip } 1{,}000 = \text{Kip } 2{,}594 / (1 + 0.1)^{10} = CA / (1 + 0.1)^{10}$$

$$CA = \text{Kip } 2{,}594 = \text{Kip } 1{,}000 \text{ x } (1 + 0.1)^{10} = PV \text{ x } (1 + 0.1)^{10}$$

It is seen that money (the present value of Kip. 1,000) is expected to arise Kip 2,594 in 10 years in the future at the interest rate of 10%. Future money of Kip 2,594 is discounted in value to be the present value of Kip. 1,000. The future value is therefore considered to be equivalent to a smaller value at present. On the contrary this means that a sum in hand now (money at present) is equivalent to a larger sum (compound amount) in the future. In other words, a present value has an increased future value, the increase being the "interest". The word "interest" therefore expresses the same idea as the word "discount". Just like a discount rate, a market interest rate takes both inflation and the cost of capital into account. A bank will quote a market rate of interest, because it expects repayments which compensate it for lending the money, as well as to compensate it for the devaluation due to inflation.

In summary, relationship between the present value (PV) and future value (FV) is expressed in terms of the following general formula for discounting:

$$PV = FV \frac{1}{(1+r)^n}$$

$$FV = PV \text{ x } (1 + r)^n$$

where, r : Discount rate

n : years in the future

Table below shows an example of present value calculation of annual cost under the following conditions:

Annual Cost : Pi or constant Kip 1,000 (table on the right)

Discount Rate : r or 0.1 (10%) (table on the right)

Discounted Period : 10 years

Example of PV of Annual Cost

Year End	Cost	Present Value		Cost (Kip)	Present Value (Kip)
1st	P_1	$P_1 / (1+r)$		1,000	1,000 / (1+0.1) = 909
2nd	P_2	$P_2 / (1+r)^2$		1,000	1,000 / $(1+0.1)^2$ = 826
3rd	P_3	$P_3 / (1+r)^3$		1,000	1,000 / $(1+0.1)^3$= 751
4th	P_4	$P_4 / (1+r)^4$		1,000	1,000 / $(1+0.1)^4$= 683
5th	P_5	$P_5 / (1+r)^5$		1,000	1,000 / $(1+0.1)^5$= 621
6th	P_6	$P_6 / (1+r)^6$		1,000	1,000 / $(1+0.1)^6$= 564
7th	P_7	$P_7 / (1+r)^7$		1,000	1,000 / $(1+0.1)^7$= 513
8th	P_8	$P_8 / (1+r)^8$		1,000	1,000 / $(1+0.1)^8$= 467
9th	P_9	$P_9 / (1+r)^9$		1,000	1,000 / $(1+0.1)^9$= 424
10-th	P_{10}	$P_{10} / (1+r)^{10}$		1,000	1,000 / $(1+0.1)^{10}$ = 386

(2) Net Present Value (NPV) (B-C)

A small-hydro project is expected to bring in revenue in future years after its completion, and also to incur running costs. The Net Present Value (NPV) is simply the present value (PV) of all revenues (benefit) minus the present value of all running and capital costs. It is always expressed together with the discount rate as follows:

$$NPV = \sum_{t=1}^{n} \frac{B_t - C_t}{(1+r)^n}$$

where, Bt : Benefit in t-th year

Ct : Cost in t-th year

n : Number of years for evaluation (project life)

t : t-th year counted from the base year

r : Discount rate (%)

Table below shows an example calculation of NPV of small-hydro project under the following conditions:

Capital cost : US$ 8,000

Running cost : US$ 1,000 per year

Revenue (Benefit) : US$ 2,400 per year

Project life : 15 years

Base year : 2006

Cost and Benefit Stream of Small Hydropower Project

Discout rate = 10% Unit : US$

No.	Year	Capital Cost	Running Cost	Total Cost	Benefit	B-C	Present Value
0	2005	8,000		8,000	0	-8,000	-8,000
1	2006		1,000	1,000	2,400	1,400	1,273
2	2007		1,000	1,000	2,400	1,400	1,157
3	2008		1,000	1,000	2,400	1,400	1,052
4	2009		1,000	1,000	2,400	1,400	956
5	2010		1,000	1,000	2,400	1,400	869
6	2011		1,000	1,000	2,400	1,400	790
7	2012		1,000	1,000	2,400	1,400	718
8	2013		1,000	1,000	2,400	1,400	653
9	2014		1,000	1,000	2,400	1,400	594
10	2015		1,000	1,000	2,400	1,400	540
11	2016		1,000	1,000	2,400	1,400	491
12	2017		1,000	1,000	2,400	1,400	446
13	2018		1,000	1,000	2,400	1,400	406
14	2019		1,000	1,000	2,400	1,400	369
15	2020		1,000	1,000	2,400	1,400	335
Total		**8,000**	**15,000**	**23,000**	**36,000**	**13,000**	***2,649***

The “B-C” above is discounted for each year to its present value (PV), and all the present value are finally added together. The NPV is estimated US$ 2,649. The result is the total present value of the project earnings (net benefit of B-C). If the NPV is positive (B-C＞0), it means that the project will earn more than it costs.

(3) Cost-Benefit Ratio (B/C)

The Cost-Benefit Ratio (B/C) that is expressed by the next equation is one of indicators to show the efficiency of project. If the B/C is greater than 1.0, it means that the project will earn more than it costs.

$$B/C = \frac{\sum_{t=1}^{n} \frac{B_t}{(1+r)^n}}{\sum_{t=1}^{n} \frac{C_t}{(1+r)^n}}$$

Below is an example calculation of B/C of small-hydro project. The same conditions as the NPV calculation above are applied.

Cost and Benefit Stream of Small Hydropower Project

Discout rate = 10% Unit : US$

No.	Year	Capital Cost	Running Cost	Total Cost	PV of Cost	Benefit	PV of Benefit
0	2005	8,000		8,000	8,000	0	0
1	2006		1,000	1,000	909	2,400	2,182
2	2007		1,000	1,000	826	2,400	1,983
3	2008		1,000	1,000	751	2,400	1,803
4	2009		1,000	1,000	683	2,400	1,639
5	2010		1,000	1,000	621	2,400	1,490
6	2011		1,000	1,000	564	2,400	1,355
7	2012		1,000	1,000	513	2,400	1,232
8	2013		1,000	1,000	467	2,400	1,120
9	2014		1,000	1,000	424	2,400	1,018
10	2015		1,000	1,000	386	2,400	925
11	2016		1,000	1,000	350	2,400	841
12	2017		1,000	1,000	319	2,400	765
13	2018		1,000	1,000	290	2,400	695
14	2019		1,000	1,000	263	2,400	632
15	2020		1,000	1,000	239	2,400	575
Total		**8,000**	**15,000**	**23,000**	**15,606**	**36,000**	**18,255**

The estimated net present values of cost and benefit and are US$ 15,606 and US$ 18,255, respectively. Therefore the B/C of project is estimated as follows:

$$B/C = \frac{18{,}255}{15{,}606} = 1.17$$

(4) Internal Rate of Return (IRR)

The Internal Rate of Return (IRR) is the discount rate at which NPV=0. In other words, the IRR is the discount rate to equalize the NPV of benefit and cost as expressed below.

$$\sum_{t=1}^{n} \frac{C_t}{(1+r)^t} = \sum_{t=1}^{n} \frac{B_t}{(1+r)^t}$$

The IRR is graphically shown below. In this illustration, cost and benefit curves are given as follows:

$$C = \sum_{t=1}^{n} \frac{C_t}{(1+r)^t}$$

$$B = \sum_{t=1}^{n} \frac{B_t}{(1+r)^t}$$

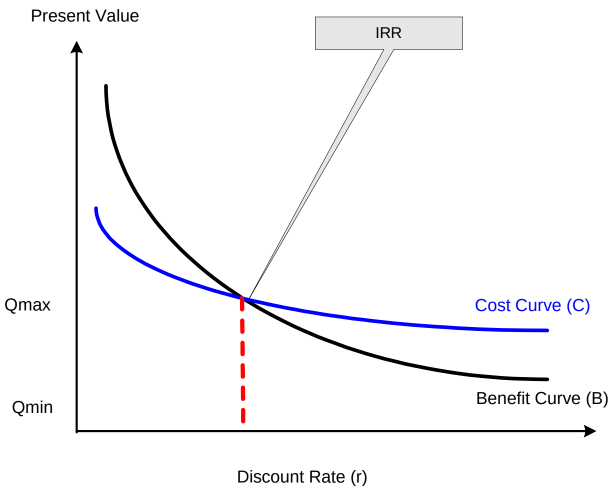


Illustration of IRR Calculation

Below is a calculation of PV at different discount rates for the cost and benefit curves of the same small-hydro project.

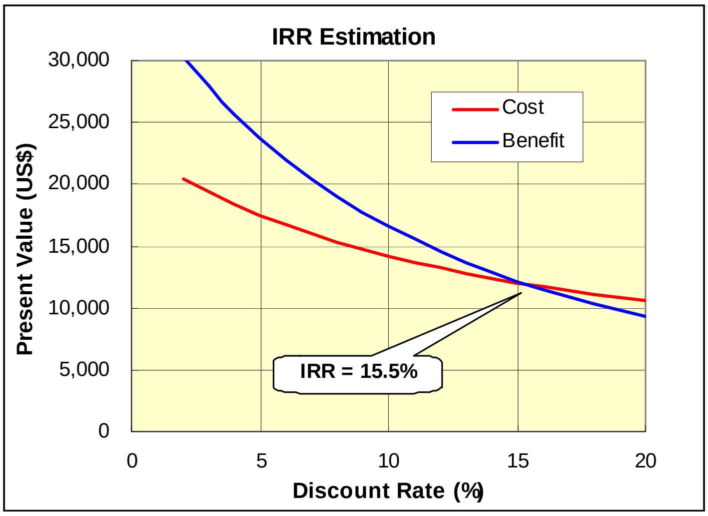
Cost Curve:

No.	Year	Cost	Discount Rate (%)									
			2	4	6	8	10	12	14	16	18	20
0	2005	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000	8,000
1	2006	1,000	980	962	943	926	909	893	877	862	847	833
2	2007	1,000	961	925	890	857	826	797	769	743	718	694
3	2008	1,000	942	889	840	794	751	712	675	641	609	579
4	2009	1,000	924	855	792	735	683	636	592	552	516	482
5	2010	1,000	906	822	747	681	621	567	519	476	437	402
6	2011	1,000	888	790	705	630	564	507	456	410	370	335
7	2012	1,000	871	760	665	583	513	452	400	354	314	279
8	2013	1,000	853	731	627	540	467	404	351	305	266	233
9	2014	1,000	837	703	592	500	424	361	308	263	225	194
10	2015	1,000	820	676	558	463	386	322	270	227	191	162
11	2016	1,000	804	650	527	429	350	287	237	195	162	135
12	2017	1,000	788	625	497	397	319	257	208	168	137	112
13	2018	1,000	773	601	469	368	290	229	182	145	116	93
14	2019	1,000	758	577	442	340	263	205	160	125	99	78
15	2020	1,000	743	555	417	315	239	183	140	108	84	65
	PV	**23,000**	**20,849**	**19,118**	**17,712**	**16,559**	**15,606**	**14,811**	**14,142**	**13,575**	**13,092**	**12,675**

Benefit Curve:

No.	Year	Benefit	Discount Rate (%)									
			2	4	6	8	10	12	14	16	18	20
0	2005	0	0	0	0	0	0	0	0	0	0	0
1	2006	2,400	2,353	2,308	2,264	2,222	2,182	2,143	2,105	2,069	2,034	2,000
2	2007	2,400	2,307	2,219	2,136	2,058	1,983	1,913	1,847	1,784	1,724	1,667
3	2008	2,400	2,262	2,134	2,015	1,905	1,803	1,708	1,620	1,538	1,461	1,389
4	2009	2,400	2,217	2,052	1,901	1,764	1,639	1,525	1,421	1,325	1,238	1,157
5	2010	2,400	2,174	1,973	1,793	1,633	1,490	1,362	1,246	1,143	1,049	965
6	2011	2,400	2,131	1,897	1,692	1,512	1,355	1,216	1,093	985	889	804
7	2012	2,400	2,089	1,824	1,596	1,400	1,232	1,086	959	849	753	670
8	2013	2,400	2,048	1,754	1,506	1,297	1,120	969	841	732	638	558
9	2014	2,400	2,008	1,686	1,421	1,201	1,018	865	738	631	541	465
10	2015	2,400	1,969	1,621	1,340	1,112	925	773	647	544	459	388
11	2016	2,400	1,930	1,559	1,264	1,029	841	690	568	469	389	323
12	2017	2,400	1,892	1,499	1,193	953	765	616	498	404	329	269
13	2018	2,400	1,855	1,441	1,125	882	695	550	437	349	279	224
14	2019	2,400	1,819	1,386	1,062	817	632	491	383	300	237	187
15	2020	2,400	1,783	1,333	1,001	757	575	438	336	259	200	156
	PV	**36,000**	**30,838**	**26,684**	**23,309**	**20,543**	**18,255**	**16,346**	**14,741**	**13,381**	**12,220**	**11,221**

As shown below, IRR is calculated 15.5% from the intersection of both curves.



(5) Unit Energy Cost (kWh Cost)

Unit Energy (Generation) Cost (kWh Cost) is an important indicator for financial viability of the small-hydro project. This indicator will provide a guideline for determining selling price of energy. The energy cost is calculated by the following equation:

Energy Cost = (Ca + Cr) / E

where, Ca : Annualized construction cost (US$ or Kip)

Cr : Annual running (O&M) cost (US$ or Kip)

E : Annual energy output (kWh)

It is noted that the annualized construction cost "Ca" shall be given in terms of a constant annual sum throughout the life of the project. Therefore the unit energy cost varies by discount rate applied and the project life. The annuity equation provides a simple way of converting the initial construction cost into an annual cost.

$Ca = C \times C_f$

$$C_f = \frac{r(1+r)^n}{(1+r)^n - 1}$$

where, C_f : Capital recovery factor

r : Discount rate

n : Project life (Service life)

The construction cost "C" is equivalent to a sum of the present value of "Ca" in each year for the life of the project (service life). The relationship between "C" and "Ca" is explained below.

Discount rate : 10%
Project Life : 15 years
Unit : US$

$$C_f = \frac{0.1x(1+0.1)^{15}}{(1+0.1)^{15}-1} = \frac{0.4177}{3.1772} = 0.1315$$

No.	Year	Construction Cost	Annualized Cost (Ca)	Present Value (PV) of Annualized Cost
0	2005	**8,000**		
1	2006		1,052	956
2	2007		1,052	869
3	2008		1,052	790
4	2009		1,052	719
5	2010		1,052	653
6	2011		1,052	594
7	2012		1,052	540
8	2013		1,052	491
9	2014		1,052	446
10	2015		1,052	406
11	2016		1,052	369
12	2017		1,052	335
13	2018		1,052	305
14	2019		1,052	277
15	2020		1,052	252
		Total	**15,780**	**8,000**

As shown above, the annualized construction cost is estimated to be US$ 1,052 multiplying the construction cost US$ 8,000 by the capital recovery factor of 0.1315. Then, the sum of present value (base year: 2006) of annualized construction cost throughout the project life amounts to US$ 8,000.

On the other hand, the annual energy output is estimated by means of the installed capacity (P) and plant factor (PF) as follows:

Annual energy (kWh) = P (kW) x 24 (hours) x 365 (days) x PF
= P (kW) x 8,760 (hours) x PF

Table below shows various calculations of unit energy cost of several small-hydro project under the following conditions:

Discount rate : 10%

Project life : 20 years

O&M cost ： 2% of the construction cost

Capital recovery factor: 0.1175

Discount Rate : 10%
Project Life : 20 years
O & M Cost : 2% of Construction Cost

$$C_f = \frac{0.1x(1+0.1)^{20}}{(1+0.1)^{20}-1} = \frac{0.6727}{5.7275} = 0.1175$$

Installed Capacity (kW)	Plant Factor (%)	Annual Energy (kWh)	Construction Cost (US$)	Annualized Cost (US$)	O&M Cost (US$)	Unit Energy Cost (US$/kWh)	Unit Energy Cost (UScent/kWh
400	80	2,803,200	500,000	58,750	10,000	0.0245	2.45
500	70	3,066,000	600,000	70,500	12,000	0.0269	2.69
600	60	3,153,600	700,000	82,250	14,000	0.0305	3.05
1,000	60	5,256,000	1,700,000	199,750	34,000	0.0445	4.45
1,000	70	6,132,000	1,700,000	199,750	34,000	0.0381	3.81
1,000	80	7,008,000	1,700,000	199,750	34,000	0.0334	3.34
2,000	60	10,512,000	3,000,000	352,500	60,000	0.0392	3.92
2,000	70	12,264,000	3,000,000	352,500	60,000	0.0336	3.36
2,000	80	14,016,000	3,000,000	352,500	60,000	0.0294	2.94
3,000	60	15,768,000	6,000,000	705,000	120,000	0.0523	5.23
3,000	70	18,396,000	6,000,000	705,000	120,000	0.0448	4.48
3,000	80	21,024,000	6,000,000	705,000	120,000	0.0392	3.92

3.6.3 ECONOMIC EVALUATION

(1) Economic Project Benefit Estimate

Economic analysis evaluates the economic viability of the project, taking into account the effects of the project on the entire national economy or society as a whole. In this sense, the economic benefit shall be estimated from the viewpoint of contribution to the national income.

Past experiences indicates that when non-electrified village is electrified, a number of productive opportunities have been identified and initiated by village entrepreneurs. To cite a few examples:

i) Weaving and basket making in the evening

ii) Sewing business

iii) Poultry rearing with electric bulbs

iv) Fruit processing and other processing tasks

v) Rice milling and furniture production

These activities undoubtedly increase the production level of the country, and will be added as economic benefits, if ever possible to identify. In broader perspective, socio-economic benefits of electricity include improved education (longer book reading time, etc.), easier household routine tasks particularly for women, better entertainment and updated information and improved health. These benefits are generally difficult to measure and quantify. However they can be referred to in narrative section of economic analysis.

In practice two methods are applied to estimate the power benefit from economic viewpoints:

i) Alternative facility cost method

ii) Consumer's willingness to pay for electricity

An alternative facility cost method is often applied to estimate the benefits of medium to large hydropower projects. By this method, the benefit of hydropower project is defined as the cost of the least-cost thermal plant alternative having the comparable electric supply performance as the hydropower that would be saved for its implementation. It shall be considered to save the cost of second best alternative under the fixed demand condition. Under this method, the capacity and energy values of hydropower are calculated.

For mini to small-hydro projects, ***consumer's willingness-to-pay (WTP) method*** is often applied. Under the current Master Plan Study, a village socio-economic survey was conducted in November-December 2004 in order to elicit information about WTP for electricity in the northern Laos. Two WTP curves were established for the connection fee and electricity tariff. From the estimated WTP curves, the Master Plan Study recommends the average WTP for connection fee of US$ 101.9 and electricity tariff US¢ 27.07 per kWh for economic benefit estimation.

(2) Economic Project Cost Estimate

Financial costs are easy to identify, as they are basically paid in cash. Economic costs need to be adjusted for market distortions caused by government intervention; import duties, sales tax, price control, and transfer payments (taxes and subsidies) for many cost items and have to be evaluated with the concept of the opportunity cost.

For economic evaluation, all costs involved the project have to be measured as economic costs, i.e. the real costs incurred from the viewpoint of nation's economy. In other words, economic cost estimate is a modification of the project construction cost estimate reflecting an adjustment, to the local currency portion of the project cost from the national economic point of view.

The economic cost is measured as the value in case that resources employed to the project are used for other activities, known as the "shadow" price. The shadow price is represented as the value of the local component of the project cost which could be earned foreign exchange if sold abroad or as the value to the economy of local labor drawn from other sector, primarily agriculture. Estimate of the shadow prices for all the cost items is a difficult task, depending upon a myriad of assumptions about the nature of the labor pool drawn and the market conditions for each category of material. For the purposes of the pre-feasibility or feasibility estimate, a simplified procedure is adopted.

In Laos, the internal transfer portion is approximately 10 % on an average; in other words, the shadow price is 90 % of the financial cost. Based on these approximate guidelines, an overall adjustment factor for the economic value to the nation of local currency portion of the project cost is estimated to be approximately 90 % of the financial cost. On the other hand, all equipment and materials to be newly imported for the project that is the foreign currency portion of the project can be used as an economic cost without any conversion.

Project cost (financial cost) consists of i) local currency portion and ii) foreign currency portion. The economic project cost is estimated as follows:

Economic Project Cost = Local Currency Portion x 0.9 + Foreign Currency Portion

(3) Economic Evaluation

Economic viability of the project is usually assessed in terms of the economic internal rate of return (EIRR) and the net present value (NPV) by use of the estimated economic project benefits and costs. It has been standard practice for major donors such as the World Bank and ADB to use an EIRR for evaluation of economic viability of projects:

i) Project with an EIRR of at least 12% can be considered acceptable.

ii) Projects with an EIRR between 10 and 12% may be accepted if additional unvalued (unquantifiable) benefits are sufficiently demonstrated.

iii) Projects with an EIRR below 10% are not accepted.

(4) Example of Economic Analysis of Small-hydro Project

The Economic Internal Rate of Return (EIRR) for 5 on-grid projects and 6 off-grid projects are calculated for a 30 year and 20 year period, respectively. Economic analyses of the candidate projects are undertaken in real terms using constant 2004 prices. All cost and benefit stream are expressed in US dollar[*1]. In the JICA Study, economic analysis was made for 11 candidate hydropower sites. Of these analyses, the Nam Ou Neua project (off-grid type) is instantiated hereafter for example.

a) Economic Costs

Economic costs of the Nam Ou Neua projects are broadly divided in to 5 groups; i) construction costs of hydropower station, 22 kV transmission line and related substations, ii) construction cost of 400 kV distribution line, iii) installation costs of house wring and electric meter for each customer, iv) operation and maintenance cost, and v) rehabilitation

[1] Exchange rate used for financial and economic analyses are US$1.0= Kip 98,000 (as of November 1st 2004)

cost.

Costs of civil works are assumed to be 100% of local portion, while costs of electronic and mechanical equipments are assumed to be 100% of foreign portion. In calculating economic price, the foreign costs are valued at CIF (cost, insurance and freight) price, and local costs (non-tradable costs) are converted into boarder price using the Standard Conversion Factor (SCF) of 0.9[*2].

House wiring and electric meter costs are assumed at US$ 60.0 per connection, which consists of electric meter (US$ 50.0) and low voltage distribution line (US$ 10.0). It is assumed that 80% of household (1,239 H/H) in the supply area will apply electricity service. As a result, costs for house wiring and electric are computed US$ 74,340.

Annual O&M costs are assumed to be 2.0% of capital costs (US$ 31,757 per year), which covering salary of staffs, spare parts for routine maintenance, and consumables for operation. Rehabilitation is supposed to be executed 10 years after the completion. Rehabilitation costs (US$ 461,788) are assumed to be 40% of electronic and mechanical equipments cost and 20% of civil work cost.

b) Economic Benefits

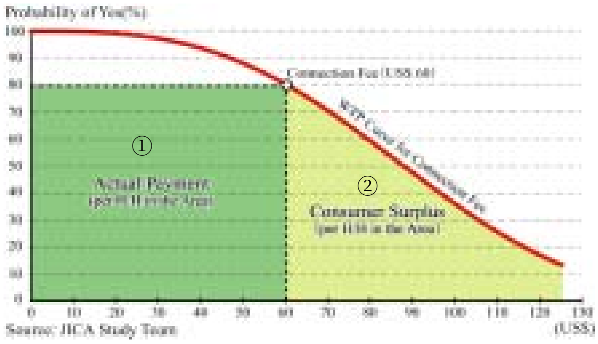
Economic benefits are calculated consumer's average WTP for connection fee and for electricity tariff. These benefits are calculated as following formulas;

Benefit from Connection = No. of H/H x Application Ratio x Average WTP for Connection Fee

Benefit from Electricity Consumption = Energy Consumption x Average WTP for Electricity Tariff

Differences of WTP used for financial analysis and economic analysis are explained as follows. For example, when setting connection fee as US$ X, only household having more then US$ X of WTP will apply the service. WTP adopted for financial analysis is determined to satisfy WTP of 80% of households. And the financial benefits were computed based on the actual payment from consumers, and can be illustrate as the quadrangle① in the following figure.

On the other hand, even if a household having US$ 90 of WTP, they will pay US$ 60. In this case, remaining US$ 30 is regarded as consumer surplus of the household. Economic benefit includes not only actual payment of US$ 60 but also consumer surplus of US$ 30. Actual payment① and consumer surplus② can be



Actual Payment and Consumer Surplus

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[2]Same SCF was applied for "Northern Area Rural Power Distribution Project", which is financed by Asian Development Bank.

illustrated as the figure below. Following formula was prepared base on the WTP curve prepared in the socio-economic survey, and was used in calculating the average WTP.

$$\mathrm{WTP}_{CF}=\left\{\int_{T=t_1}^{\infty}\exp\left\{-\exp\left(\frac{\ln T-13.58}{0.332}\right)\right\}dT+t_1\times 80\%\right\}\div 0.8$$

Where:
WTP_{CF} = Consumer's average WTP for Connection Fee
T= Connection Fee (variable)
t_1= Actual Connection Fee Levied on Consumers

$$\mathrm{WTP}_{ET}=\left\{\int_{X=x_1}^{\infty}\left\{\frac{1}{1+\exp(17.07+2.286\ln X)}\right\}dX+x_1\times 80\%\right\}\div 0.8$$

Where:
WTP_{CF} = Consumer's average WTP for Electricity Tariff
T= Electricity Tariff (variable)
t_1= Actual Electricity Tariff Levied on Consumers

Formula to Calculate Consumer's Average WTP for Connection Fee and Electricity Tariff

Using the methodology, consumer's average WTP for connection fee and electricity tariff, and economic benefits is calculated as follows;

Average WTP for Electricity Tariff and Connection Fee of the Applicants and Economic Benefit (Nam Ou Neua)

Economic Benefit from Connection			Economic Benefit from Sales of Electricity		
Average WTP	Number of Applicants	Economic Benefit	Average WTP	Annual Energy Sales	Annual Economic Benefit
US$ 97.74	1,239	US$ 121,100	US¢ 27.32 /kWh	1,033 MWh	US$ 282,215

Source: JICA Study Team

c) Calculation of Economic Internal Rate of Return (EIRR)

Calculated EIRR, net present value and benefit-cost ratio, using discount rate of 10%, for Nam Ou Neua project is summarized as following tables.

EIRR of these projects are exceeding cost of capital in Lao PDR (10%). Accordingly, Nam Ou Neua project is confirmed its economic viability.

Calculation of the Economic Internal Rate of Return (Unit: US$)

	Capital Cost	House Wiring	O&M Cost	Cost Total	Electricity Tariff	Connection Charge	Total Benefit	Net Benefit
0	600,475.0			600,475.0			0.0	-600,475.0
1	900,712.5	74,340.0	30,023.8	1,005,076.3	133,434.7	121,099.9	254,534.5	-750,541.8
2			30,023.8	30,023.8	157,498.3		157,498.3	127,474.5
3			30,023.8	30,023.8	181,561.9		181,561.9	151,538.1
4			30,023.8	30,023.8	205,625.5		205,625.5	175,601.7
5			30,023.8	30,023.8	229,689.1		229,689.1	199,665.3
6			30,023.8	30,023.8	253,752.7		253,752.7	223,728.9
7			30,023.8	30,023.8	277,816.3		277,816.3	247,792.5
8			30,023.8	30,023.8	282,215.2		282,215.2	252,191.4
9			30,023.8	30,023.8	282,215.2		282,215.2	252,191.4
10	444,452.5		30,023.8	474,476.3	282,215.2		282,215.2	-192,261.1
11			30,023.8	30,023.8	282,215.2		282,215.2	252,191.4
|			|	|	|		|	|
20			30,023.8	30,023.8	282,215.2		282,215.2	252,191.4

Present Value (@10.0%)			Benefit Cost Ratio	EIRR
Cost	Benefit	NPV		
1,739,865	1,905,044	165,179	1.095	11.77%

3.6.4 FINANCIAL EVALUATION

(1) Project Costs

A small-hydro project is expensive. It is also a risk because most of the cost must be met at the start of the project. The investor (a private individual, prospective owner, funding agency, or rural development bank) will need to be convinced that such a major investment is safe. It is necessary to convince the investor that the project will produce financially viable results, and must be identified which proposed schemes are likely fail and warn the investor of poor financial potential.

Financial costs shall be estimated based on the current price level. Price escalation shall be taken into account applying annual inflation rates. The O&M cost also shall be inflated to the current price level.

(2) Project Revenue

Financial revenue is a total amount of electricity tariffs to be collected end-users. The electricity tariff and connection fee for revenue estimation are proposed to be US¢ 11.80 per kWh and US$ 77.8 respectively that were estimated based on the developed WTP curves under the current Master Plan Study.

(3) Financial Evaluation

Financial analysis examines the profitability of project to a operating entity. Financial viability of the project is usually assessed in terms of the financial rate of return (FIRR) and the net present value (NPV) by use of the estimated project cost and revenue.

(4) Example of Financial Analysis of Small-hydro Project

The Financial Internal Rate of Return (FIRR) for on-grid projects and off-grid projects are calculated for a 30 year and 20 year period, respectively. All cost and benefit stream are expressed in real terms using constant 2004 prices and are expressed in US dollar[*3]. Of financial analysis of the 11 candidate sites, the Nam Ou Neua project (off-grid type) is instantiated hereafter for example.

a) Costs of the Project

Costs of the project are broadly divided in to 5 groups; i) construction costs of hydropower station, 22 kV transmission line and related substations, ii) construction cost of 400 kV distribution line, iii)

[3] Exchange Rate Used: US.$ 1.08= Kip 10,376.5

installation costs of house wring and electric meter for each customer, iv) operation and maintenance cost, and v) rehabilitation cost.

Construction cost of the power station was estimated at US$ 1,065,891, and is consists of civil work (US$ 866,792) and electronic and mechanical equipments (US$ 199,099). Construction cost of 22 kV transmission line and related facilities were estimated US$ 521,976.

House wiring and electric meter costs as well as annual O&M costs are assumed as same as economic analysis (please refer to 3.6.3 (4)).

b) Benefits of the Project

Benefits of the Off-grid type projects are determined as the total expected revenue from sales of energy and revenue from connection fee. The Nam Ou Nuea is expected to provide electricity for 15 villages (1,549 households). 80% of household (1,239 households) are assumed to be received electricity from the power station.

Revenue from Connection Fee= No. of H/H in the villages x application ratio x connection fee
Revenue from Electricity Tariff= Generation Volume x T/D Loss x Electricity Tariff

i. Amount of Energy Sold

Amount of energy sold to village people is calculated as which ever is smaller of a) available energy, and b) energy demand. Application Ratio, total energy loss ratio, and load factor are assumed as 80%, 10%, and 50%, respectively. These values are assumed to be common among the sites. Peak demand per household is assumed to be 90W at the inception of electrification and be progressively increased and then catch up with national average by 2015. Routing of distribution lines, electrification targeted villages, are determined using GIS database. Number of customers is estimated based on 80% of application ratio and number of households in the target villages.

Energy Demand= No. of H/H x Application Ratio x Peak Demand per H/H x Load Factor x 8,760 hrs.
= 1,549 H/H x 80% x 90 Watt (in 2007) - 304 Watt (in 2026) x 50% x 8,760 hrs.
= 488.4 – 1,651.5 MWh per year

Available Energy = Generation Volume x (1 – Total Loss Ratio)
=1,147.8 MWh per year x (1 – 10%) = 1,033.0 MWh per year

From 2015 onwards, energy demand will execced supply capacity and thus energy sales will remain unchange after that.

ii. Setting of Connection Fee and Electricity Tariff

As mentioned, sales volume of energy is calculated based on the 80% of application ratio. Thus, connection fee and electricity tariff used for financial analysis should be set so as to be agreed by 80% of household. In this study, connection fee and electricity tariff are decided based on the WTP (willingness to pay) curves, which are estimated through village socio-economic survey. As a result, connection fee and electricity tariff in the Nam Ou Neua are decided as follows;

Connection Fee= US$ 62.10

Electricity Tariff= US¢ 9.01 per kWh

c) Calculation of Financial Internal Rate of Return (FIRR)

FIRR, net present value and benefit-cost ratio, using discount rate of 10%, are calculated. Unit generation costs are computed based on cost and generation stream with discount rate of 10.0%. The table below summarized the results of the financial analysis.

As same as other off-grid projects in the developing countries, all the financial indicators show very bad performance, and accordingly the Nam Ou Neua projects are judged to be financially not viable.

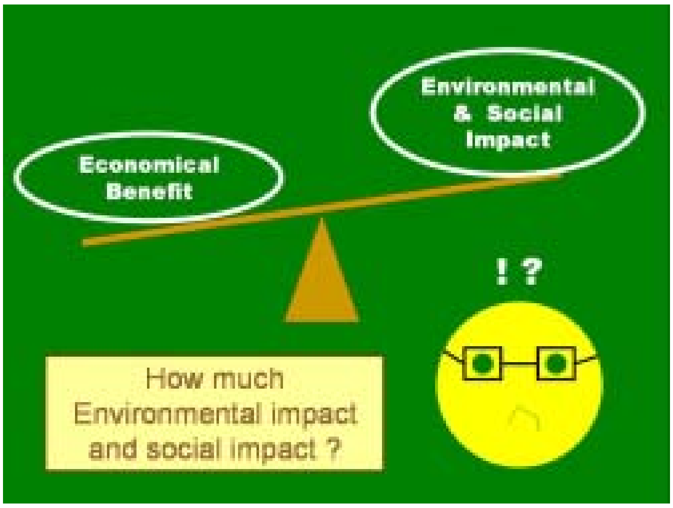
Calculation of the Financial Internal Rate of Return (Unit: US$)

	Capital Cost	House Wiring	O&M Cost	Cost Total	Electricity Tariff	Connection Charge	Total Benefit	Net Benefit
0	635,146.7			635,146.7			0.0	-635,146.7
1	952,720.0	74,340.0	31,757.3	1,058,817.4	44,006.1	76,941.9	120,948.0	-937,869.4
2			31,757.3	31,757.3	51,942.1		51,942.1	20,184.8
3			31,757.3	31,757.3	59,878.2		59,878.2	28,120.9
4			31,757.3	31,757.3	67,814.3		67,814.3	36,056.9
5			31,757.3	31,757.3	75,750.3		75,750.3	43,993.0
6			31,757.3	31,757.3	83,686.4		83,686.4	51,929.0
7			31,757.3	31,757.3	91,622.4		91,622.4	59,865.1
8			31,757.3	31,757.3	93,073.2		93,073.2	61,315.8
9			31,757.3	31,757.3	93,073.2		93,073.2	61,315.8
10	461,788.4		31,757.3	493,545.7	93,073.2		93,073.2	-400,472.6
11			31,757.3	31,757.3	93,073.2		93,073.2	61,315.8
|			|	|	|		|	|
20			31,757.3	31,757.3	93,073.2		93,073.2	61,315.8

Present Value (@10.0%)			Benefit Cost Ratio	FIRR
Cost	Benefit	NPV		
1,833,859	658,856	-1,175,004	0.359	-6.87%

3.7 ENVIRONMENTAL ASSESSMENT (EA)

3.7.1 WHAT IS EA? AND WHY EA IS NECESSARY?



What is EA?

An Environmental Assessment (EA) is the entire process accompanying a development project proposal that determines the likely environmental impacts due to construction, operation, and closing the project.

Why EA is Necessary?

Because….. In order to make a decision whether the planned project shall be implemented or not, we should know in advance how much environmental impacts are expected to occur due to its implementation.

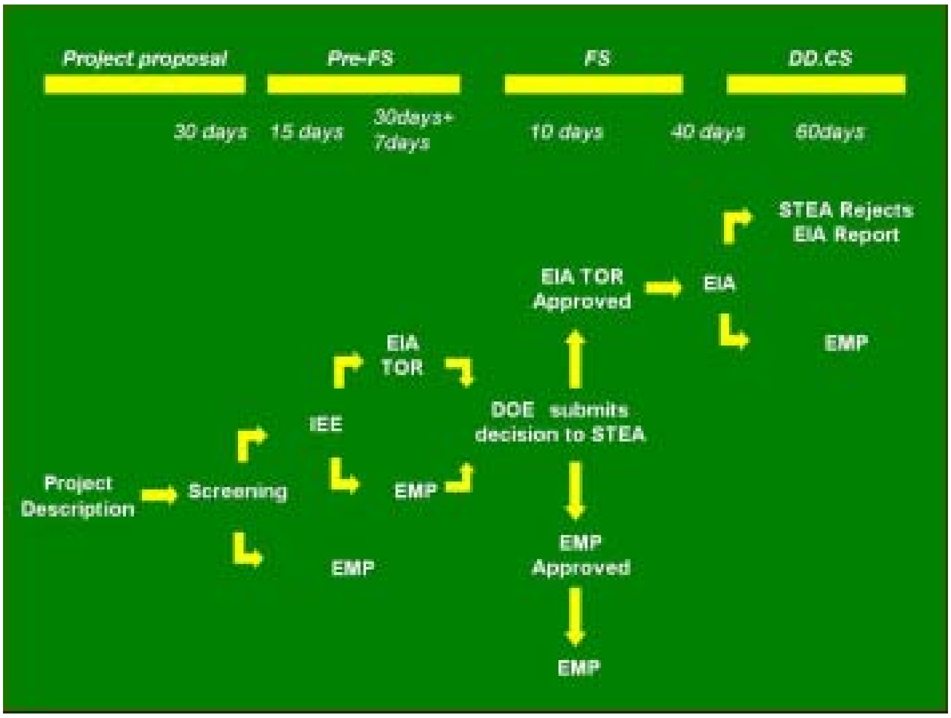
3.7.2 ENVIRONMENTAL ASSESSMENT GUIDELINES IN LAO PDR

The procedure of the Environmental Assessment in Lao PDR should follow the Environmental Assessment Guidelines of Lao PDR. The Environmental Assessment guidelines for the energy development project are as follows:

- **Environmental Protection Law (2001)**
- **Decree on the Implementation of the Environmental Protection Law (2002)**
- **Regulation on Implementing the Environmental Assessment for the Electricity Projects in Lao PDR (2001)**
- **Environmental Management Standard for Electricity Projects (2003)**

3.7.3 ENVIRONMENTAL ASSESSMENT PROCEDURE FOR SMALL-HYDRO PROJECT

According to the EA guidelines of Lao PDR, the Environmental Assessment shall comprise Project Description, Review of Project Description, Project Screening, Review and Approve of Project Screening , IEE (Initial Environmental Examination), EIA (Environmental Impact Assessment) and EMP (Environmental Management Plan). The procedural flow of the EA in Lao PDR is illustrated below.



Source: Figure 1, Page 7 of 25, Regulation on Implementing the Environmental Assessment for Electricity Projects in Lao PDR, 2002

Environmental Assessment Process for Electricity Sector in Lao PDR

3.7.4 EXAMPLES OF ENVIRONMENTAL ASSESSMENT FOR SMALL-HYDRO PROJECT

(1) Project Description

The Project Description is required for the Screening. The Project Description is the initial process of EA as shown in the illustration above. In order to ensure an appropriate and well informed screening process, full information should be provided in the Project Description that enables the Project Review team to determine whether the further EA is required or not. The Project Description should include a summary and details of the project as listed below.

- Project Owner
- Project Type
- Project Size
- Project Objective
- Project Location
- Materials to be used in construction and operation
- Estimate of the quantity and quality of any solid, liquid or air-borne wastes resulting from construction and operation
- Projects intended work force for both construction and operation (number and origin)
- Anticipated positive and negative environmental (physical, biological, social, cultural and economic) impacts
- Proposed environmental management measures that will be implemented through all stages of the project
- Information used and assumptions made when determining the anticipated impacts

Required Information in the Project Description

An example of the Project Description is also shown below.

Example of Project Description

<table>
<tr><td>Project Name</td><td colspan="4">Nam Ou Neau</td></tr>
<tr><td>Project Owner</td><td colspan="4">DOE/MIH</td></tr>
<tr><td>Project Type</td><td colspan="4">Hydro Power Project</td></tr>
<tr><td>Project Size</td><td colspan="4">Power Output:
Installed Capacity 383kW
Design Discharged 1.6 m^3/s
Access Road:0 km
22kV Transmission Line: 90 km</td></tr>
<tr><td>Project Objective</td><td colspan="4">Improvement of the electrification level, the poverty alleviation, and economic growth targeting Gnod district</td></tr>
<tr><td>Project Location</td><td colspan="4">Province: Phongsaly
District: Gnod
Village of Dam site: Nagnao
River: Nam Ou</td></tr>
<tr><td rowspan="5">Materials to be used in construction and operation</td><td rowspan="2">Weir</td><td rowspan="2">Concrete</td><td>Height(m)</td><td>6</td></tr>
<tr><td>Crest Length(m)</td><td>50</td></tr>
<tr><td rowspan="3">Waterway</td><td>Open Channel</td><td>Length(m)</td><td>2,300</td></tr>
<tr><td>Head tank</td><td>Length(m)</td><td>9</td></tr>
<tr><td>Penstock</td><td>Length(m)</td><td>100</td></tr>
<tr><td>Estimate of the quantity and quality of any solid, liquid or air-borne wastes resulting from construction and operation</td><td colspan="4">Solid waste: Cut Trees (construction phase)
Liquid waste: Murky Waters (construction phase)</td></tr>
<tr><td>Projects intended work force for both construction and operation (number and origin)</td><td colspan="4">Not decided</td></tr>
<tr><td>Anticipated positive and negative environmental (physical, biological, social, cultural and economic) impacts</td><td colspan="4">Positive impact:
・ Improvement of the electrification level
Negative impact:
・ Impact on fishing and irrigation in law water section caused by reduction of stream flow
・ The paddy area on the left bank of intake will be inundated partly or fully, depending on the design of the project. Its social impact as well as it compensation needs careful consideration.</td></tr>
<tr><td>Proposed environmental management measures that will be implemented through all stages of the project</td><td colspan="4">Not decided</td></tr>
<tr><td>Information used and assumptions made when determining the anticipated impacts</td><td colspan="4">・ Information collected by site visiting</td></tr>
</table>

	• Project design • GIS data and map

(2) Project Summary

For the hydropower development project that has an installed capacity greater than two thousand kilowatts (2,000 kW), the Project Summary in the Project Description shall contain the information on the following:

- Objective/Purpose of project
- Location
- Key Stakeholders and details of the PI activities undertaken to date
- Institutional Framework
- Project Area Main settlements including villages, households and estimated number of inhabitants and ethnic groups
  - Environmentally Sensitive Areas
  - Main vegetation and land use type
  - River/s, stream/s including discharge average and minimum, total head other major geographical features and their characteristics
  - Main commercial activities
  - Main subsistence activities
  - Main community infrastructure, services and facilities
- Generation Type Hydro/Thermal/Other
  - Installed Capacity
  - Source of Fuel
  - Fuel, Chemical Storage
  - Outputs/Wastes
- Other project Features such as
  - Power Station and Sub Stations
  - Generators and Turbines(type, capacity, output)
  - Dam weir including size (height, length), materials
  - Reservoir including Surface area, maximum depth storage volume at FSL
  - Transmission line including size, length and proposed route
  - Access Road size, length and proposed route
  - Head race, Penstock, Tailrace (Type and length)
  - Construction Workforce
  - Pollution treatment
- Source of information and include whether it is measured or estimated, assumptions made
- Anticipated positive and negative environmental (physical, biological, social, cultural and economic) impacts
- Attach maps and other visual presentations eg. Photo, videos

Required Information in the Project Summary

An example of the Project Summary is shown on next page.

Example of Project Summary Sheet for the Project with a Capacity Greater than 2000kW

Project Name	Nam Boun2	
Objective/Purpose of project	Improvement of the electrification level, the poverty alleviation, and economic growth targeting district	
Location	Province	Phomngsaly
	District	
	Village of Dam site	Sentham
Key Stakeholders and details of the PI activities undertaken to date	The Workshop was held for three (3) days during March 4 to 6, 2004 at an EDL hall in	

		Vientiane, inviting participants from DOE, PDIH of the northern provinces and other concerned parties.		
Institutional Framework		**Project Owner**: DOE/MIH		
Project Area Main settlements including villages, households and estimated number of inhabitants and ethnic groups	**Environmentally Sensitive Areas**	No environmental sensitive area		
	Main vegetation and land use type	Forest and Paddy field		
	River/s, stream/s including discharge average and minimum, total head other major geographical features and their characteristics	River	Nam Boun2	
		Specific Discharge in Dry Season	-- litre/s/km^2	
		Discharge in Dry Season	-- m^3/s	
		Effected Head	-- m	
	Main commercial activities	--		
	Main subsistence activities	Agriculture		
	Main community infrastructure, services and facilities	Road Check dam in Nam Hoy river		
Generation Type Hydro/Thermal/Other	**Installed Capacity**	850-3400kW		
	Source of Fuel	Hydro Power		
	Fuel, Chemical Storage	No fuel and chemical storage		
		Design Discharge: 1-4 m^3/s		
Other project Features	**Power Station and Sub Stations**	Power station will be constructed. Detail design has not decided yet.		
	Generators and Turbines(type, capacity, output)	Installed Capacity: 850-3400kW Design Discharge: 1-4 m^3/s		
	Dam weir including size (height, length), materials	Material	Concrete	
		Height	6 m	
		Crest Length	40 m	
	Reservoir including Surface area, maximum depth storage volume at FSL	--		
	Transmission line including size, length and proposed route	-- km		
	Access Road size, length and proposed route	-- km		
	Head race, Penstock, Tailrace (Type and length)	Open Channel	Length(m)	5,500
		Head tank	Length(m)	--
		Penstock	Length(m)	--
	Construction Workforce	--		

(3) Screening

A Screening is a preliminary assessment of project's potential impacts. It is normally completed at a project's identification stage. The Screening is used to decide whether project's impacts are of significant nature to warrant further environmental assessment or not. To assist the Screening process, the electricity development project shall be categorized into one of three project types. These types are listed below.

Three Project Types of Electricity Development Project

Project Type	Screening
Electricity project with an installed capacity of less than one hundred (100) Kilowatts	No further EA procedure
Electricity project with an installed capacity of more than one hundred (100) – two thousand (2,000) Kilowatts	Screening by the project description REVIEW
Electricity project with an installed capacity of more than two thousand (2,000) Kilowatts	Screening by VISITING the project site

(4) IEE (Initial Environmental Examination)

An initial environmental examination (IEE) should be prepared for all the projects that are determined to require further EA. The IEE should be a brief study to identify and describe the environmental and social issues of the project, and the impacts that the project will probably cause without environmental protection measures. The IEE should consider the data that is available from the government, published data from similar projects in similar locations, general field observations, and other available data. The result of IEE should be summarized into an IEE report. An example of TOC (Table of Content) of the IEE report is as follows:

Initial Environmental Examination

1) Introduction
2) Institutional Framework
3) Description of the Environment
4) Potential Impacts
5) Public Involvement
6) Environmental Management
7) Conclusion

The examination of potential impacts is the most important in the IEE. The potential impacts shall be examined by using impact matrix as shown below.

Impact Matrix for Scoping

Items	No.	Likely Impacts	Overall Rating	Planning Ph e	Constructio n Ph e	Operation Ph e
Social Environment. *Regarding Right”,	1	Involuntary Resettlement				
	2	livelihood, etc.				
	3	resources				
		Social institutions such as social infrastructure and local decision-making institutions				
		Existing social infrastructures and services				
	6	The poor, indigenous and ethnic people				
	7	Misdistribution of benefit and damage				
	8	Cultural heritage				
	9	Local conflict of interests				
	10	Water Usage or Water Rights and Rights of Common				
	11	Sanitation				
	12	Hazards (Risk) Infectious diseases such as HIV/AIDS				
Natural Environ	13	Topography and Geographical features				
	14	Groundwater				
	15	Soil Erosion				
	16	Hydrological Situation				
	17	Coastal Zone				
	18	Flora, Fauna and Biodiversity				
	19	Meteorology				
	20	Landscape				
	21	Global Warming				
Pollution	22	Air Pollution				
	23	Water Pollution				
	24	Soil Contamination				
	25	Waste				
	26	Noise and Vibration				
	27	Ground Subsidence				
	28	Offensive Odor				
	29	Bottom sediment				
	30	Accidents				

Rating:
A: Serious impact is expected.
B: Some impact is expected.
C: Extent of impact is unknown
No Mark: No impact is expected. EIA is not necessary.

An example of the impact matrix is shown on next page.

Sample of Impact Matrix for Scoping (Nam Boun2 Project)

Items	No.	Phase				Op Ph		
		Likely Impacts	**waterway a facility**	**ission line**	**access road**	**Water intake**	**Water discharge**	**O of power station**
Social Environment: *Re “Chi criteria of Social Environment.		**Involuntary Resettlement**						
	2	**Local economy such as employment and livelihood, etc.**				C	A	
	3	**resources**				C	A	
	4	**infrastructure and local decision-making institutions**						
		Existing social infrastructures and services						
	6	**The poor, indigenous and ethnic people**						
	7	**Misdistribution of benefit and damage**				B	B	B
	8	**Cultural heritage**	C	C	C			
	9	**Local conflict of interests**						
	10	**Water Usage or Water Rights and Rights of Common**				A		
	11	**Sanitation**	B	B	B			
	12	**Hazards (Risk) Infectious diseases such as HIV/AIDS**	B		B			
Nat	13	**Topography and Geographical features**						
	14	**Groundwater**	C					
	15	**Soil Erosion**	B	B	B			
	16	**Hydrological Situation**	B			A		
	17	**Coastal Zone**						
	18	**Flora, Fauna and Biodiversity**	B		B	A		
	19	**Meteorology**						
	20	**Landscape**						
	21	**Global Warming**						
Po	22	**Air Pollution**						
	23	**Water Pollution**	B		B		B	
	24	**Soil Contamination**						
	25	**Waste**	B		B			
	26	**Noise and Vibration**						
	27	**Ground Subsidence**						
	28	**Offensive Odor**						
	29	**Bottom sediment**	B	B	B		B	
	30	**Accidents**	B	B	B			

Rating:
A: Serious impact is expected.
B: Some impact is expected.
C: Extent of impact is unknown
No Mark: No impact is expected. EIA is not necessary.

(5) EIA (Environmental Impact Assessment)

The EIA is the detailed assessment of the environmental impacts that could be caused by a project and alternatives to the project along with an EMP (Environmental Monitoring Plan) to prevent, reduce, and monitor impacts. The EIA and its report for Electricity Projects in Lao PDR must include and address the following requirements and components:

Required Items for EIA

No	Requirement	Example of EIA report
1	Table of Contents	
2	Terms of Reference	
3	Executive Summary	
4	Introduction	
5	Institutional Framework(Including Policy, Legal and Administrative)	
6	Description of the Environment (Baseline condition: Physical, Biological, Social, Cultural and Economic Environment)	
	6.1 General	
	6.2 Environmental Study Area	
	6.3 Baseline Information	
	6.4 Visual Presentation	
7	Study of Alternative	
8	Environmental Impacts (Physical, Biological, Social, Cultural and Economic Impacts)	
9	Public Involvement	
10	Description of the Chosen Alternative	
11	Environmental Management Plan	
12	Conclusion	
13	Additional Information	
14	Annexes	
15	Glossary of Terms, Abbreviations and Acronyms	
16	References	

(6) EMP (Environmental Management Plan)

The EMP is a part of the IEE and the EIA reports that identifies:

(i) Environmental protection measures that are to be implemented during construction, operation, and closure of the project to prevent or reduce significant environmental impacts

(ii) Institutional arrangements, responsibilities, and schedule for implementing the EMP

(iii) Monitoring and evaluation program to be followed during project construction, operation, and closure, and proposed budget for implementing the EMP

The EMP and its report for Electricity Projects in Lao PDR must provide and address the following requirements:

Required Items for EMP

No	Requirement
1	Table of Contents
2	Executive Summary
3	Introduction
4	Institutional Framework(Including Policy, Legal and Administrative)
5	Management Arrangements
6	Environmental Management Measure
7	Monitoring
8	Contractor’s Environmental Management Plan (CEMP)
9	Corrective Action
10	Public Involvement
11	Implementation Schedule
12	Costing
13	Glossary of Terms, Abbreviations and Acronyms
14	Review of EMP
15	References