
CHAPTER 5 DESIGN FOR CIVIL STRUCTURES

The main obstacle for a small-scale hydropower plant is the high development cost. In this chapter, element technologies are described assuming the need to reduce the construction cost of civil structures (no description is given for those which equally apply to the design of an ordinary hydropower plant).

5.1 Intake weir

5.1.1 Basic types of intake weirs

There are a number of basic types of intake weirs as listed below.

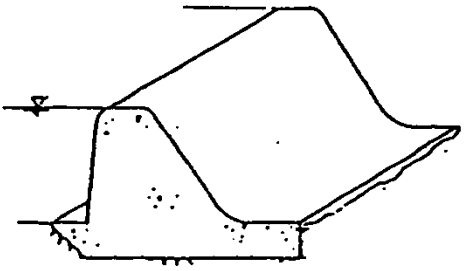
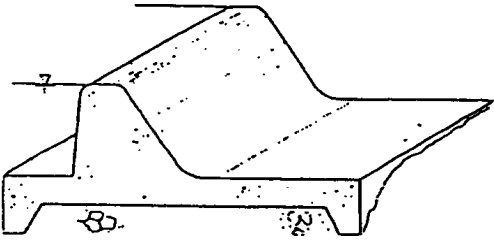
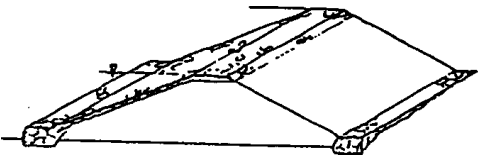
- (1) Concrete gravity dam
- (2) Floating concrete dam
- (3) Earth dam
- (4) Rock-fill dam
- (5) Wet masonry dam
- (6) Gabion dam
- (7) Concrete reinforced gabion dam
- (8) Brushwood dam
- (9) Wooden dam
- (10) Wooden frame with gravel dam

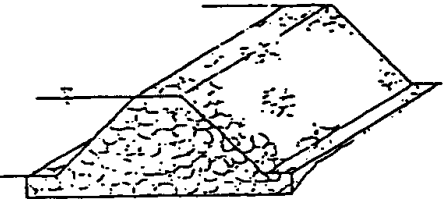
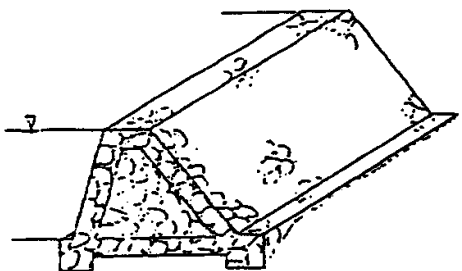
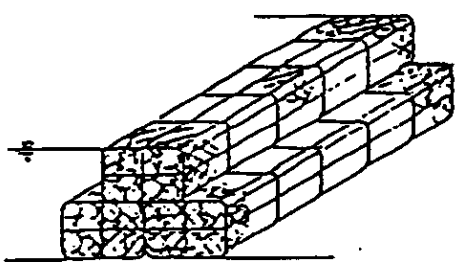
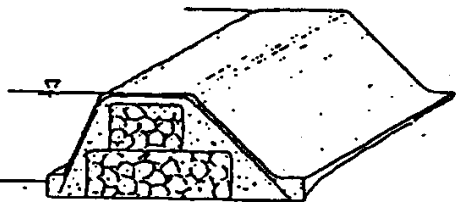


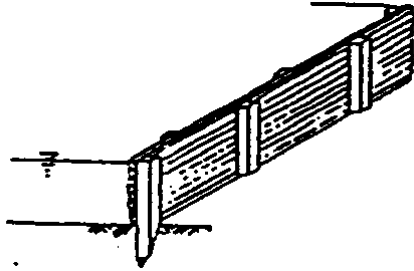
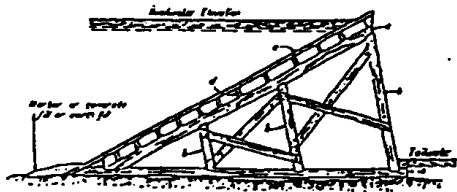
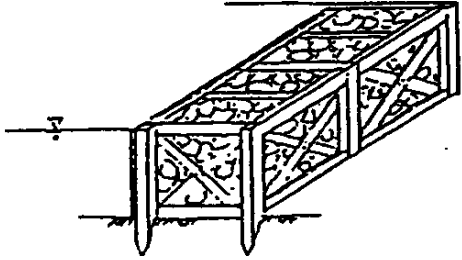
Wet masonry dam

Of the above, the basically flexible rock-fill dam and gabion dam, etc. are popularly used in Southeast Asian countries because of such advantages as (i) little influence by the conditions of the ground base and (ii) relatively easy repair when damaged. However, they can be breached by flooding due to their structure and their application should be preceded by careful examination of the importance of constructing such a civil structure and the conditions of the downstream.

Table 5.1.1 Basic types of intake weirs for small-scale hydropower plant and application conditions

Type	Outline Drawing	Application Conditions
Concrete gravity dam	<p>Concrete is used for the construction of the entire body.</p> 	<p>Foundations: in principle, bedrock River conditions: not affected by the gradient, discharge or level of sediment load Intake conditions: good interception performance and intake efficiency</p>
Floating concrete dam	<p>Lengthened infiltration path of the foundations by means of cut-off, etc. to improve the interception performance</p> 	<p>Foundations: in principle, gravel River conditions: not affected by the gradient, discharge or level of sediment load Intake conditions: good interception performance and intake efficiency</p>
Earth dam	<p>Earth is used as the main material for the body; the introduction of a riprap and core wall may be necessary depending on the situation.</p> 	<p>Foundations: variable from earth to bedrock River conditions: gentle flow and easy to deal with flooding Intake conditions: good intake efficiency because a high interception performance is possible with careful work</p>

Type	Outline Drawing	Application Conditions
Rock-fill dam	<p>Gravel is used as the main material for the body. The introduction of a core wall may be necessary depending on the situation.</p> 	<p>Foundations: various, from earth to bedrock River conditions: river where an earth dam could be washed away by the normal discharge Intake conditions: limited to the partial use of river water due to the low intake efficiency</p>
Wet masonry dam	<p>Filling of the spaces between gravel with mortar, etc.</p> 	<p>Foundations: various, from earth to bedrock River conditions: not affected by the gradient, discharge or level of sediment load Intake conditions: good interception performance and intake efficiency</p>
Gabion dam	<p>Gravel is wrapped by a metal net to improve the integrity.</p> 	<p>Foundations: various, from earth to bedrock River conditions: river where a rock-fill dam could be washed away by the normal discharge Intake conditions: limited to the partial use of river water due to the low intake efficiency</p>
Concrete reinforced gabion dam	<p>Reinforcement of the gabion surface with concrete</p> 	<p>Foundations: various, from earth to bedrock River conditions: river where the metal net could be damaged due to strong flow Intake conditions: applicable when a high intake efficiency is required</p>

Type	Outline Drawing	Application Conditions
Brushwood dam	Simple weir using locally produced tree branches, etc. 	Foundations: various, from earth to gravel layer River conditions: loss due to flooding is assumed Intake conditions: at a site with a low intake volume or intake from a stream to supplement the droughty water flow
Wooden dam	Weir using wood 	Foundations: various, from earth to bedrock River conditions: relatively gentle flow with a low level of sediment transport Intake conditions: a certain level of intake efficiency is secured with a surface coating, etc.
Wooden frame with gravel dam	The inside of the wooden frame is filled with gravel to increase the stability. 	Foundations: various, from earth to bedrock River conditions: river at which a rock-ddddfill dam could be washed away by the normal discharge Intake conditions: limited to the partial use of river water due to the low intake efficiency

5.1.2 Decision on weir height

As the weir volume is proportionate to the square of the height, it is important to decide the weir height in view of its minimisation taking the following conditions into consideration.

- (1) Conditions restricting waterway elevation

To decide the weir height, it is necessary to take the topographical and geological conditions of the waterway route into consideration in addition to the conditions at the weir construction site. Careful examination is particularly necessary at a site where the waterway construction cost accounts for a large proportion of the total construction cost.

The weir height at a site where the waterway is constructed under an existing road is often decided with reference to the elevation of such a road.

(2) Possibility of riverbed rise in downstream

As the weir height for a small-scale hydropower plant is generally low, there is concern that its normal function could be disrupted by a rise of the riverbed in the downstream.

Accordingly, the future riverbed rise should be assumed to decide the weir height if the planned site falls under any of the following cases.

- 1) Gently sloping river with a high level of transported sediment
- 2) Existence of not fully filled check dam, etc. in the downstream of the planned intake weir
- 3) Existence of a collapsing site in the downstream with concern in regard to continuous collapse in the future
- 4) Existence of a narrow section in the downstream which obstructs the flow of sediment and/or driftwood

(3) Conditions to remove sediment from front of weir and settling basin by intake method (Tyrolean intake and side intake)

Under normal circumstances, the weir height should be planned to exceed the calculated value by the following method to ensure the smooth removal of sediment from the front of the weir and the settling basin.

1) Side intake

In the case of side intake, the following Case (a) or Case (b), which ever is the higher, is adopted.

- a. Weir height (D_1) determined in relation to the bed elevation of the scour gate of the intake weir

$$D_1 = d_1 + h_i$$

- b. Weir height (D_2) determined by the bed gradient of the settling basin

$$D_2 = d_2 + h_i + L (i_c - i_r)$$

Where,

d_1 : height from the bed of the scour gate to the bed of the inlet (usually 0.5 – 1.0 m)

d_2 : difference between the bed of the scour gate of the settling basin and the riverbed at the same location (usually around 0.5 m)

h_i : water depth of the inlet (usually determined to make the inflow velocity approximately 0.5 – 1.0 m/s)

L : length of the settling basin (see Chapter 5-5.3 and Fig.5.3.1)

i_c : inclination of the settling basin bed (usually around 1/20 – 1/30)

i_r : present inclination of the river

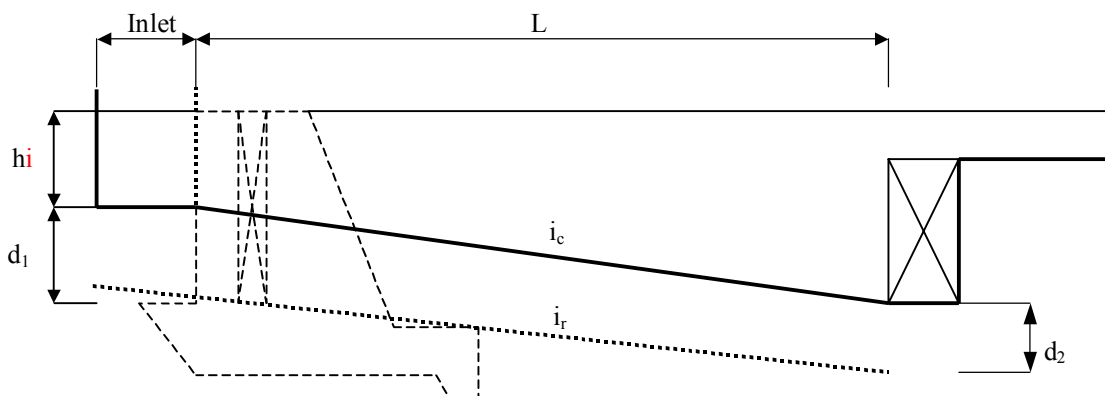


Fig.5.1.1 Sectional view of side intake and weir

2) Tyrolean intake

A Tyrolean intake where water is taken from the bottom assumes that the front of the weir is filled with sediment and, therefore, Case D₂ determines the weir height for side intake.

$$D_2 = d_2 + h_i + L (i_c - i_r)$$

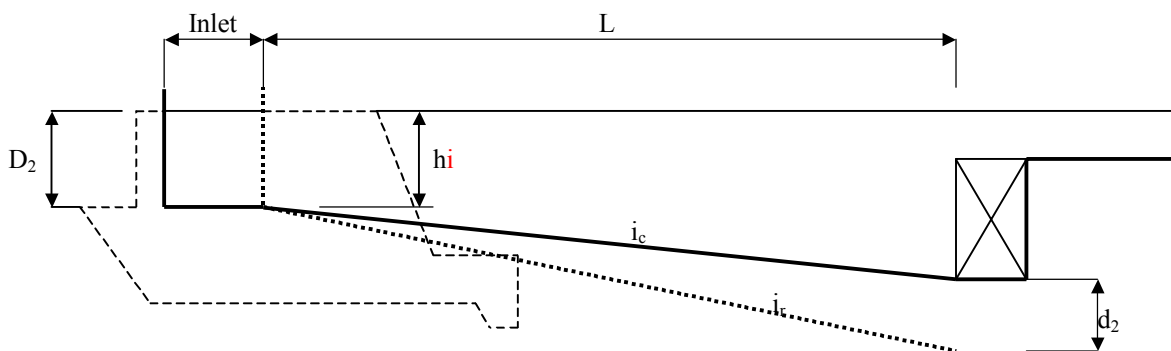


Fig.5.1.2 Sectional view of Tyrolean intake and weir

(4) Influence on electric energy generated

At a site where the usable head is small or where it is planned to secure the necessary head by a weir, the weir height significantly influences the level of generated electric energy. Accordingly, it is necessary to determine the weir height at such a site by comparing the expected changes of both the construction cost and the generated electric energy because of different weir heights.

(5) Influence of back water

When roads, residential land, farmland and bridges, etc. exist in a lower elevation area in the upstream of a planned intake weir site, it is necessary to determine the weir height to prevent flooding due to back water. Particularly at a site with a high weir height, the degree of influence on the above features must be checked by means of back water calculation or other methods.

5.2 Intake

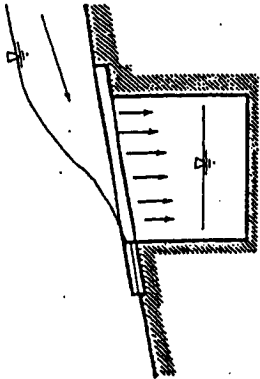
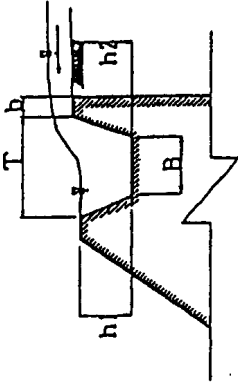
5.2.1 Intake method

Apart from the ordinary side intake method, there are several types of simple intake methods which aim at reducing the weir height and omitting the intake gate (hereinafter referred to as the Tyrolean intake method) for a hydropower plant. Two typical examples are listed below.

- Bar screen type
- Bar-less type

The details of these two types are shown in Table 5.2.1.

Table 5.2.1 Typical examples of Tyrolean intake methods

Intake Method	Outline Drawing	Characteristics	Advantages and Problems Found by Actual Performance Survey
Bar screen type		<ul style="list-style-type: none"> If a screen is installed to cover most of the river channel, it is highly resistant to riverbed fluctuations. A sufficiently wide intake width enables 100% intake of river water. As overflow may occur due to fallen leaves, etc. gathering on the screen surface, the screen width should have a sufficient margin. The sedimentation capacity of the weir to deal with sediment inflow should also be analysed. This type is popularly used and the intake rate is said to be generally $0.1 - 0.3 \text{ m}^3/\text{s}$ per unit width based on a bar installation angle of up to 30°, an inter-bar space of $20 - 30 \text{ mm}$ and a bar length of approximately 1 m. 	<p>< Advantages ></p> <ul style="list-style-type: none"> A scour gate of intake weir can be omitted. A compact intake facility is suitable for a narrow or rapid river. Stable intake is possible despite a change of the riverbed upstream. <p>< Problems ></p> <ul style="list-style-type: none"> At the time of flooding or water discharge, sediment and litter flow into the waterway. A screen which is clogged with gravel, etc. requires much labour for its removal.
Bar-less type		<ul style="list-style-type: none"> The running water usually overflows the fixed weir top and is guided into the settling basin via an intake channel placed across the river channel and along the endsill (deflector). With an increase of the river discharge, the running water overflows the endsill and eventually becomes a rapid flow to fly over the endsill, making intake impossible at the time of flooding. However, sediment deposited in the intake channel is washed away towards the downstream of the endsill, making maintenance of the intake channel easier. While the sectional form of this type is similar to that of the bar screen type, the absence of a screen means a reduction of the maintenance cost and labour related to the screen. 	<p>< Advantage ></p> <ul style="list-style-type: none"> A compact intake facility is suitable for a narrow or rapid river. Stable intake is possible despite a change of the riverbed upstream. Sediment and litter are discharged naturally at the time of flooding. <p>< Problem ></p> <ul style="list-style-type: none"> Plenty of sediment and litter inflow to the waterway. Frequently of scouring of settling basin is required.

5.2.2 Important points for intake design

For the design of the intake for a small-scale hydropower plant, it is necessary to examine the possible omission of an intake gate and others in order to achieve cost reduction.

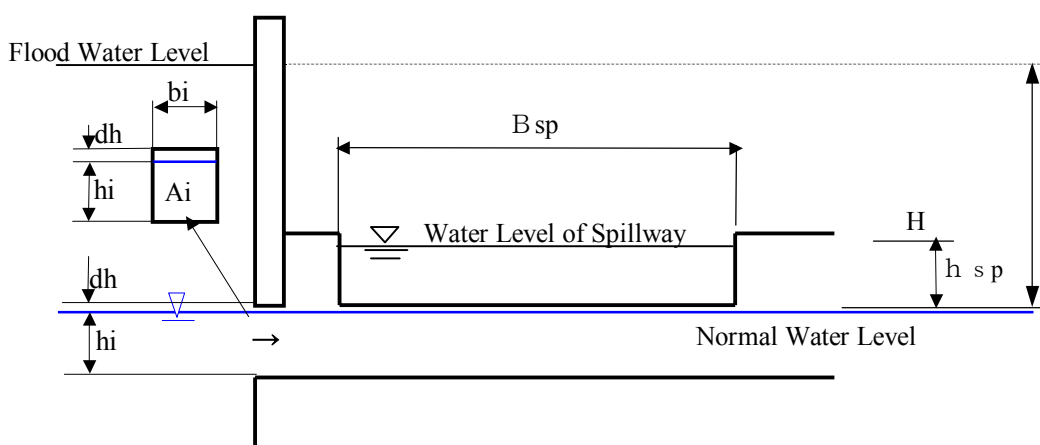
In the case of a small-scale hydropower plant, the headrace tends to be an open channel, a covered channel or a closed conduit. When this type of headrace is employed, it is essential to avoid a water inflow volume which considerably exceeds the design intake volume as it will directly lead to the destruction of the headrace.

Meanwhile, as automatic gate control for a small-scale hydropower plant results in a cost increase, manual control is opted for. In the case of the intake facility for a small-scale hydropower plant being constructed in a remote mountain area, a swift response to flooding is difficult. The following method is, therefore, proposed to control the inflow at the time of flooding without the use of a gate, etc.

(1) Principle

This method intends the design of an intake which becomes an orifice with a rise of the river water level due to flooding and is frequently used for a Tyrolean intake or for small-scale hydropower plants in Indonesia.

The inflow volume in this case is calculated by the formula below.



$$Q_f = A_i \times C_v \times C_a \times (2 \times g \times H)^{0.5}$$

Where,

Q_f : inflow volume of submerged orifice (m³/s)

A_i : area of intake (m²) $A_i = b_i \times (d_h + h_i)$ $d_h = 0.10 \sim 0.15\text{m}$

C_v : coefficient of velocity: $C_v = 1/(1 + f)$

f : coefficient of inflow loss (see next figure)

Bsp, hsp: refer to Chapter 5-5.3 Settling basin

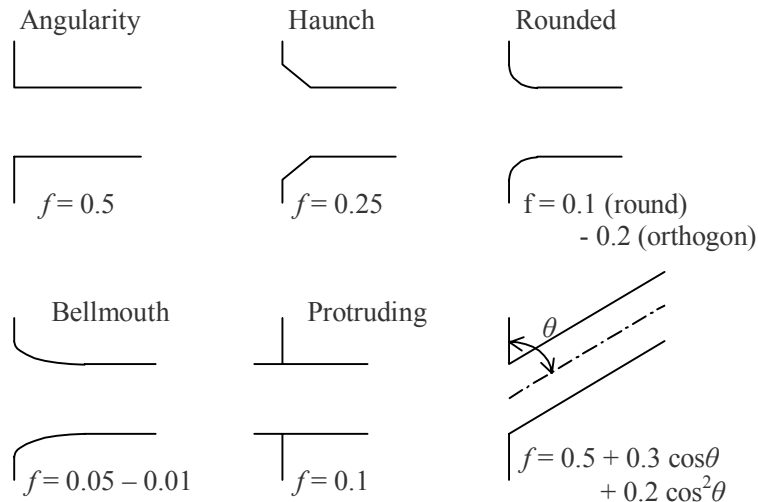


Fig.5.2.1 Coefficient of inflow loss of various inlet shapes

C_a : coefficient of contraction (approximately 0.6; see the following formula)

$$C_a = 0.582 + \frac{0.0418}{1.1 + d/D} \quad (\text{Merriman's formula})$$

where,

D, d : ratio between upstream flow and downstream flow of contracting flow
when, $d \ll D$, $C_a = 0.582$

H : water level difference between upstream and downstream of the orifice (m)

(2) Equipment outline

The important points for design are listed below.

- 1) It is necessary for the intake to have a closed tap instead of an open tap so that it becomes a pressure intake when the river water level rises.
- 2) The intake should be placed at a right angle to the river flow direction where possible so that the head of the approaching velocity at the time of flooding is minimised.
- 3) As water inflow at the time of flooding exceeds the design intake volume, the spillway capacity at the settling basin or starting point of the headrace should be fairly large.

5.3 Settling basin

The settling basin must have not only a structure which is capable of settling and removing sediment which is a size larger than the minimum size which could have an adverse impact on the turbine, etc. but also a spillway to prevent excessive water inflow into the headrace. The basic configuration of a settling basin is illustrated below.

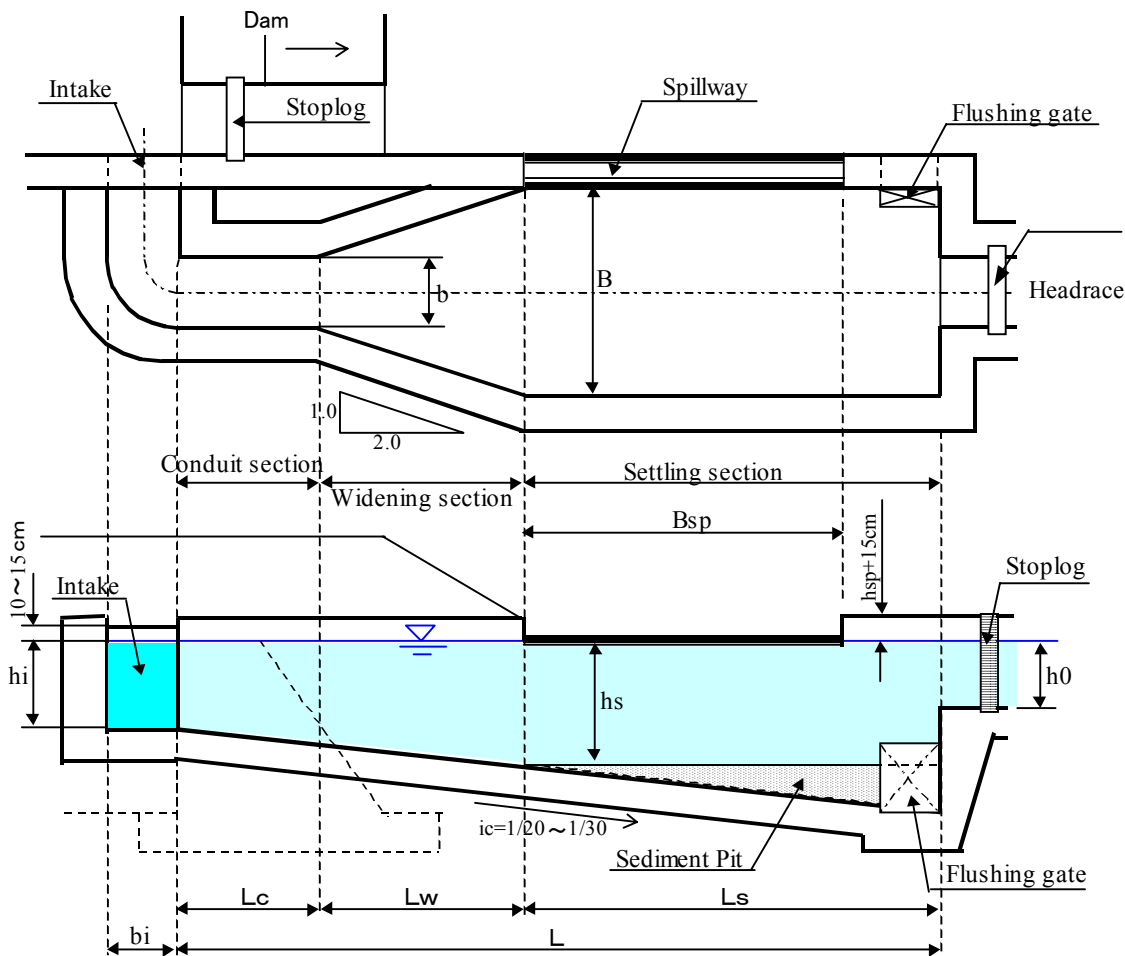


Fig.5.3.1 Basic configuration of settling basin

[Reference]

In oblong section, uniform flow depth: $h_{o1} = H^* \times 0.1 / (SLs)^{0.5}$

H^* : refer to {Ref.5-1}

SLs : slope of top end of the headrace

$$h_{o2} = \{ (\alpha \times Qd^2) / (g \times B^2) \}^{1/3}$$

$\alpha = 1.1$

Qd = Design Discharge (m^3/s)

$g = 9.8$

B: Width of Headrace (m)

if $h_{o1} < h_{o2}$, $h_o = h_{o1}$

if $h_{o1} \leq h_{o2}$, $h_o = h_{o2}$

Each of these sections has the following function.

(1) Conduit section

Conduit section connects the intake with the settling basin. It is necessary that the conduit section should be curtailing its length.

(2) Widening section:

This regulates water flow from the conduit channel to prevent the occurrence of whirlpools and turbulent flow and reduces the flow velocity inside the settling basin to a predetermined velocity.

(3) Settling section:

This section functions to settle sediment above a certain size and its required length (l) is calculated by the following formula based on the relation between the settling speed, flow velocity in the settling basin and water depth.

The length of the settling basin (L_s) is usually determined so as to incorporate a margin to double the calculated length by the said formula.

$$l \geq \frac{V}{U} \times h_s \quad L_s = 2 \times l$$

Where,

l : minimum length of settling basin (m)

h_s : water depth of settling basin (m) (-see Fig.5.31)

U : marginal settling speed for sediment to be settled (m/s)

usually around 0.1 m/s for a target grain size of 0.5 – 1 mm.

V : mean flow velocity in settling basin (m/s)

usually around 0.3 m/s but up to 0.6 m/s is tolerated in the case where the width of the settling basin is restricted.

$$V = Q_d / (B \times h_s)$$

Q_d: design discharge (m³/s)

B : width of settling basin (m)

(4) Sediment pit:

This is the area in which sediment is deposited.

(5) Spillway

Spillway drains the submerged inflow which flows from the intake. The sizes of spillway will be decided by following equation.

$$Q_f = C \times B_{sp} \times h_{sp}^{1.5} \rightarrow h_{sp} = \{Q_f / (C \times B_{sp})\}^{1/1.5}$$

Where,

Q_f : inflow volume of submerged orifice (m^3/s , see Chapter 5-5.2.2 (1))

C : coefficient =1.80

h_{sp} : water depth at the spillway (m, see Fig 5.3.1)

B_{sp} : width of the spillway (m, see Fig.5.3.1)

5.4 Headrace

5.4.1 Type and Basic Structure of Headrace

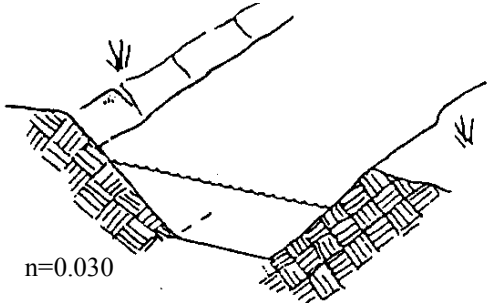
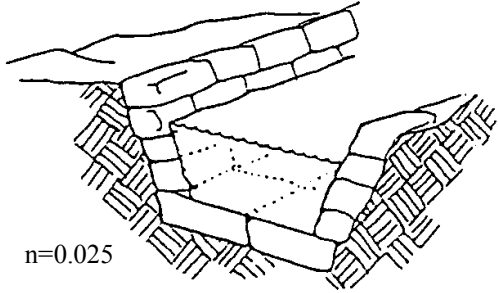
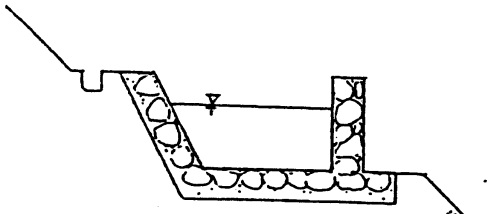
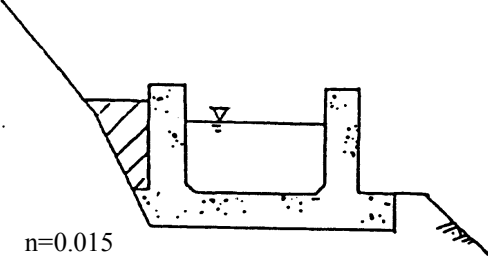
Because of the generally small amount of water conveyance, the headrace for a small-scale hydropower plant basically adopts an exposed structure, such as an open channel or a covered channel, etc. Some examples and their basic structures are given in Table 5.4.1 and Table 5.4.2 respectively.

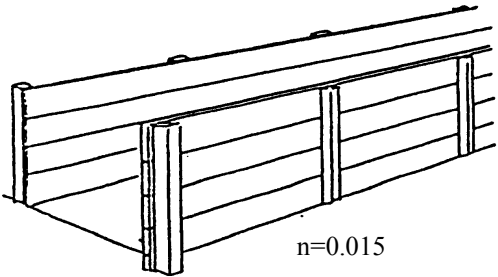
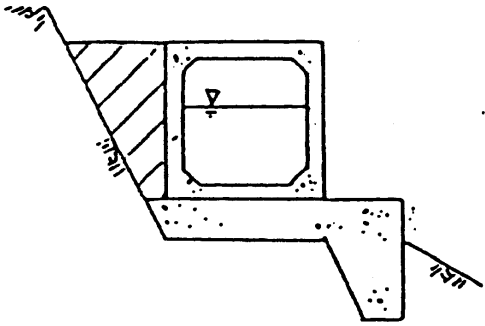
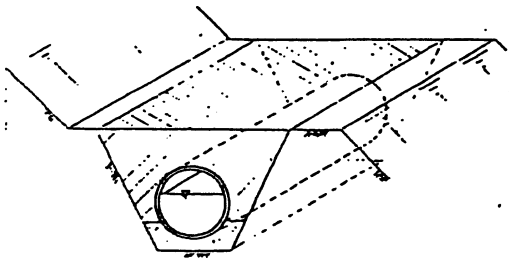


Table 5.4.1 Types of headraces for small-scale hydropower plants

Type	Outline Drawing	Advantages and Problems	Typical Structure
Open channel		<p>< Advantages ></p> <ul style="list-style-type: none"> • Relatively inexpensive • Easy construction <p>< Problems ></p> <ul style="list-style-type: none"> • Possible inflow of sediment from the slope above • High incursion rate of fallen leaves, etc. 	<ul style="list-style-type: none"> • Simple earth channel • Lined channel (dry or wet masonry lining; concrete lining) • Fenced channel (made of wood, concrete or copper) • Sheet-lined channel • Half-tube channel (corrugated piping, etc.)
Closed conduit / Covered channel		<p>< Advantages ></p> <ul style="list-style-type: none"> • Generally large earth work volume • Low incursion rate of sediment and fallen leaves, etc. into the channel <p>< Problems ></p> <ul style="list-style-type: none"> • Less easier channel inspection, maintenance work, including sediment removal, and repair 	<ul style="list-style-type: none"> • Buried tube (Hume, PVC or FRPM) • Box culvert • Fenced channel with cover

Table 5.4.2 Basic structure of headraces for small-scale hydropower plants

Type	Outline Diagram	Advantages and Problems
Simple earth channel	 <p>n=0.030</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • Easy construction • Inexpensive • Easy repair <p>< Problems ></p> <ul style="list-style-type: none"> • Possible scouring or collapse of the walls • Not applicable to highly permeable ground • Difficult to mechanise the sediment removal work
Lined channel (rock and stone)	 <p>n=0.025</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • Relatively easy construction • Can be constructed using only local materials • High resistance to side scouring • Relatively easy repair <p>< Problems ></p> <ul style="list-style-type: none"> • Not applicable to highly permeable ground
Wet masonry channel	 <p>Plastered : n=0.015 Non Plastered : n=0.020</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • Local materials can be used • Strong resistance to back scouring • Can be constructed on relatively high permeable ground. • Easy construction at the curved section due to the non-use of forms <p>< Problems ></p> <ul style="list-style-type: none"> • More expensive than a simple earth channel or dry masonry channel (rock/stone-lined channel) • Relatively takes labour hours.
Concrete channel	 <p>n=0.015</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • High degree of freedom for cross-section design <p>< Problems ></p> <ul style="list-style-type: none"> • Difficult construction when the inner diameter is small • Relatively long construction period

Type	Outline Diagram	Advantages and Problems
Wood fenced channel	 <p style="text-align: center;">n=0.015</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • Less expensive than a concrete channel • Flexible to allow minor ground deformation <p>< Problems ></p> <ul style="list-style-type: none"> • Limited use with earth foundations • Unsuitable for a large cross-section • Difficult to ensure perfect water-tightness • Liable to decay
Box culvert channel	 <p style="text-align: center;">n=0.015</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • Easier construction than a Hume pipe on a slope with a steep cross-sectional gradient • Relatively short construction period and applicable to a small cross-section when ready-made products are used • Rich variety of ready-made products <p>< Problems ></p> <ul style="list-style-type: none"> • Heavy weight and high transportation cost when ready-made products are used • Long construction period when box culverts are made on site
Hume pipe channel	 <p style="text-align: center;">n=0.015</p>	<p>< Advantages ></p> <ul style="list-style-type: none"> • Easy construction on a gently sloping site • Relatively short construction period • High resistance to external pressure • Applicable to a small cross-section • Elevated construction with a short span is possible <p>< Problems ></p> <ul style="list-style-type: none"> • Heavy weight and high transportation cost

5.4.2 Determining the Cross Section and Longitudinal Slope

The size of cross section and slope should be determined in such a manner that the required turbine discharge can be economically guided to the head tank. Generally, the size of cross section is closely related to the slope. The slope of headrace should be made gentler for reducing head loss (difference between water level at intake and at head tank) but this causes a lower velocity and thus a larger cross section. On the contrary, a steeper slope will create a higher velocity and smaller section but also a larger head loss.

Generally, in the case of small-hydro scheme, the slope of headrace will be determined as 1/500 – 1/1,500. However in the case of micro-hydro scheme, the slope will be determined as 1/50 – 1/500, due to low skill on the survey of levelling and construction by local contractor.

The cross section of headrace is determined by following method.

(1) Method of calculation

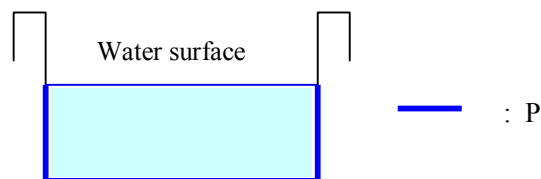
$$Q_d = A \times R^{2/3} \times S_L^{1/2} / n$$

Q_d : design discharge for headrace (m³/s)

A : area of cross section (m²)

R : $R = A / P$ (m)

P : length of wet sides (m) refer to next figure.



S_L : longitudinal slope of headrace (e.g. $S_L = 1/100 = 0.01$)

n : coefficient of roughness (see Table 5.4.2)

For instance, in the case of rectangular cross section, width (B)=0.6m, water depth (h)=0.5m, longitudinal slope (S_L)=1/200=0.005, coefficient of roughness (n)=0.015.

$$A = B \times h = 0.6 \times 0.5 = 0.30 \text{ m}^2$$

$$P = B + 2 \times h = 0.6 + 2 \times 0.5 = 1.60 \text{ m}$$

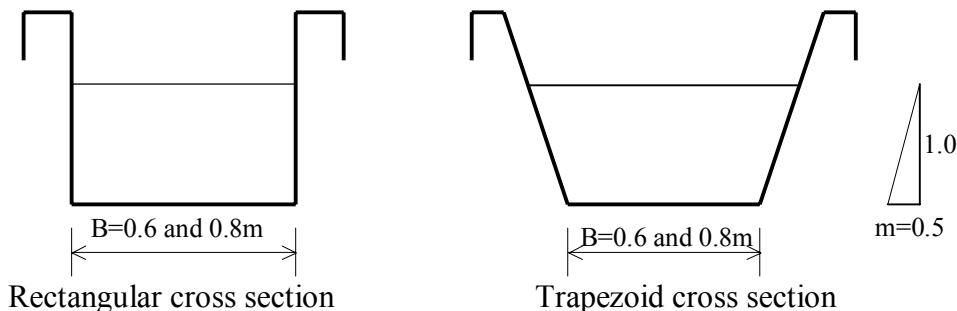
$$R = A / P = 0.30 / 1.60 = 0.188 \text{ m}$$

$$\therefore Q_d = A \times R^{2/3} \times S_L^{1/2} / n = 0.30 \times 1.60^{2/3} \times 0.005^{1/2} / 0.015 = 1.94 \text{ m}^3/\text{s}$$

(2) Simple method

In order to simplify the above method, following method for determining the cross section is used in [Reference 5-1 Simple Method for Determining the Cross Section]

This reference will be used in determination of cross section in the following two sectional forms.



H^* should be calculated on each different slope. For instance, in the case of trapezoid cross section, design discharge (Q)= $0.5\text{m}^3/\text{s}$, width (B)= 0.8m , longitudinal slope ($SL_{A,B,C,D}$)= $1/100, 1/50, 1/100, 1/200$ which is the gentlest position of the headrace, coefficient of roughness (n)= 0.015 .

Water depth (H^*) is approximately 0.3m in Reference 5-1 Fig-4. Therefore actual water depth (H) is

$$H = H^* \times 0.1 / (SL)^{0.5}$$

$$H_{A,C} = H^* \times 0.1 / (SL_{A,C})^{0.5} = 0.3 \times 0.1 / (0.01)^{0.5} = 0.3$$

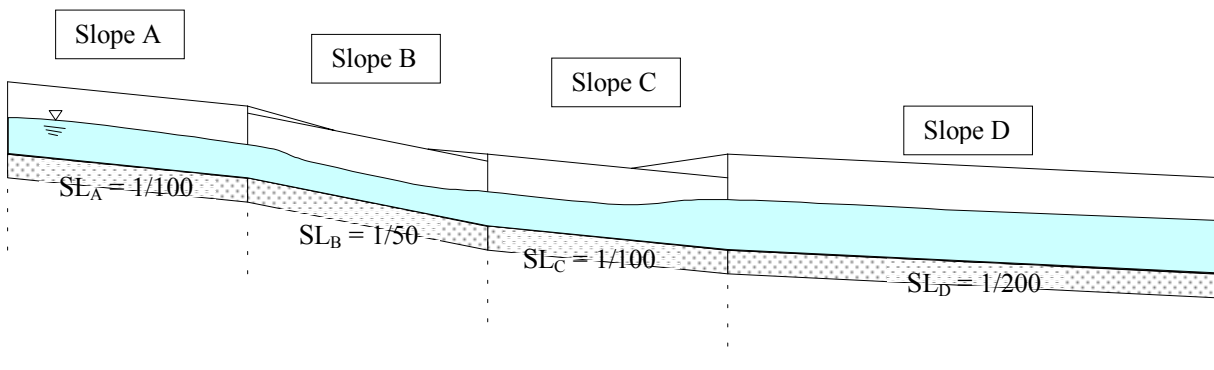
$$H_B = H^* \times 0.1 / (SL_B)^{0.5} = 0.3 \times 0.1 / (0.02)^{0.5} = 0.21$$

$$H_D = H^* \times 0.1 / (SL_D)^{0.5} = 0.3 \times 0.1 / (0.005)^{0.5} = 0.42$$

and height of the cross section of Slope A,C is $0.60\text{m}(0.3+0.2\sim 0.3)$,

height of the cross section of Slope B is $0.55\text{m}(0.21+0.2\sim 0.3)$,

height of the cross section of Slope D is $0.75\text{m}(0.42+0.2\sim 0.3)$.



5.5 Head Tank

5.5.1 Head tank capacity

(1) Function of headtank

The functions of headtank are roughly the following 2 items.

- Control difference of discharge in a penstock and a headrace cause of load fluctuation.
- Finally remove litter (earth and sand, driftwood, etc.) in flowing water

(2) Definition of head tank capacity

The head tank capacity is defined the water depth from h_c to h_0 in the head tank length L as shown in Fig.5.5.1.

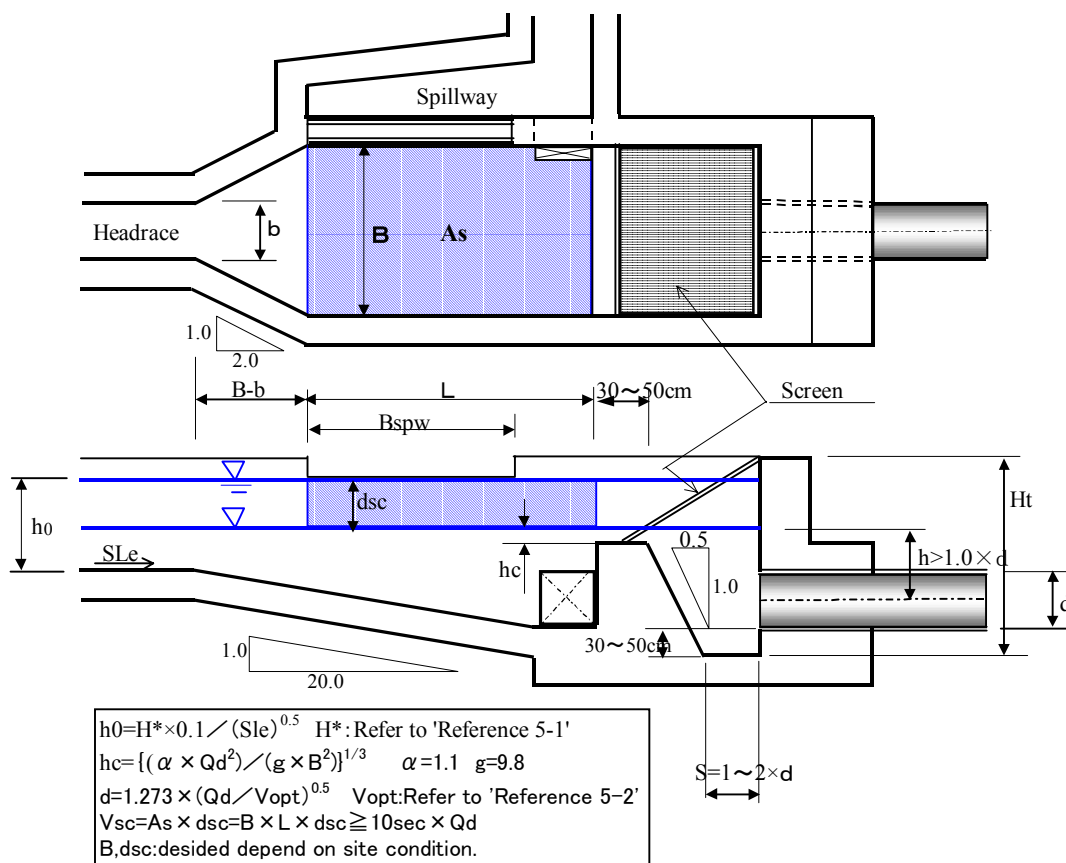


Fig.5.5.1 Picture of headtank capacity

$$\text{Headtank capacity } V_{sc} = A_s \times d_{sc} = B \times L \times d_{sc}$$

where, A_s : area of headtank

B : width of headtank

L : length of headtank

d_{sc} : water depth from uniform flow depth of a headrace when using maximum discharge (h_0) to critical depth from top of a dike for sand trap in a headtank (h_c)

[Reference]

In oblong section, uniform flow depth: $h_0 = H^* \times 0.1 / (SLe)^{0.5}$

H^* : refer to {Ref.5-1}

SLe : slope of tail end of the headrace

critical depth: $h_c = \{ (\alpha \times Qd^2) / (g \times B^2) \}^{1/3}$ $\alpha : 1.1$ $g : 9.8$

(3) Determine a headtank capacity

The headtank capacity should be determined in consideration of load control method and discharge method as mentioned below.

a. In case only load is controlled

In the case where only controlled load (demand) fluctuation has been considered, generally a dummy load governor is adopted. A dummy load governor is composed of water-cooled heater or air-cooled heater, difference of electric power between generated in powerhouse and actual load is made to absorb heater. The discharge control is not performed.

The head tank capacity should be secured only to absorb the pulsation from headrace that is about 10 times to 20 times of the design discharge (Q_d).

A view showing a frame format of load controlled by a dummy load governor is shown in Fig.5.5.2.

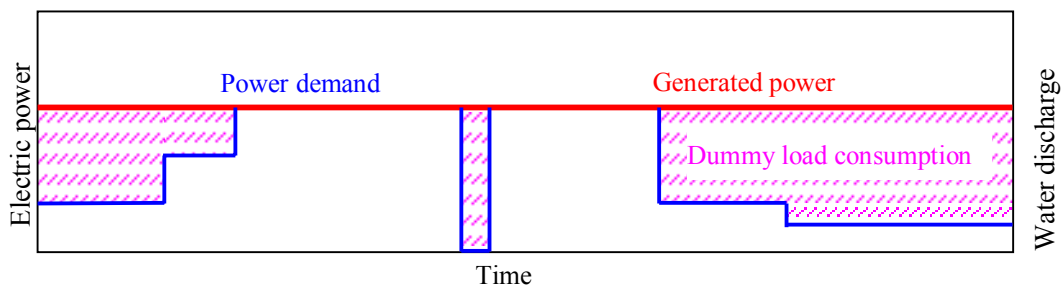


Fig.5.5.2 Pattern diagram of dummy load consumption

b. In case both load and discharge are controlled

In the case of controlled load and discharge, a mechanical governor or electrical governor is used for load control. These governors have the function of controlling vane operation to optimal discharge when electrical load has changed. Generally a mechanical governor is not sensitive to load change, and head tank capacity in this case should be secured 120 times to 180 times of Qd.

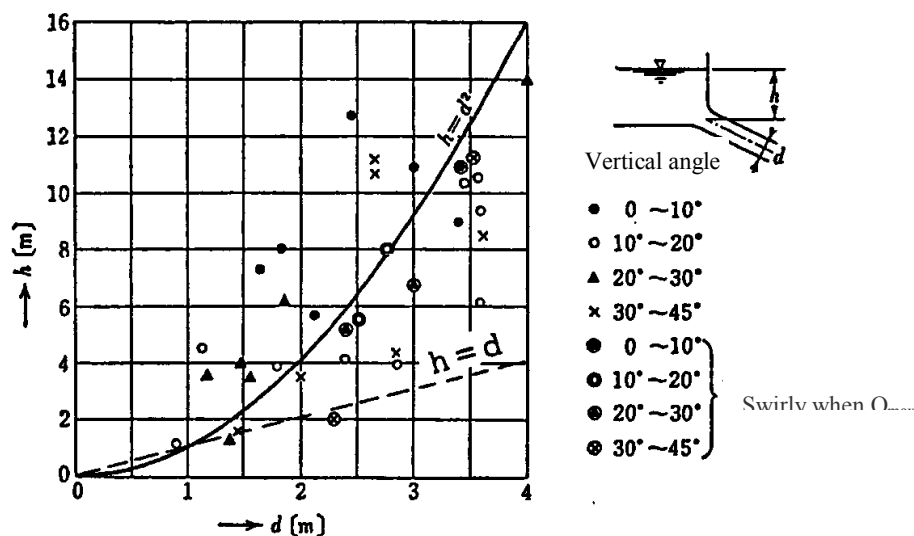
On the other hand, an electrical governor is responsive to load change, therefore head tank capacity is designed about 30 times to 60 times of Qd in many cases.

5.5.2 Important issues for head tank design

The design details for the head tank for a small-scale hydropower plant are basically the same as those for a small to medium-scale hydropower plant and the particularly important issues are discussed below.

(1) Covering water depth and installation height of penstock inlet

As the penstock diameter is generally small (usually 1.0 m or less) in the case of a small-scale hydropower plant, it should be sufficient to secure a covering water depth that is equal to or larger than the inner diameter of the penstock. However, in the case of a channel where both the inner diameter and inclination of the penstock are as large as illustrated below, the occurrence of inflow turbulence has been reported in the past. Accordingly, the covering water depth must be decided with reference to the illustration below when the inner diameter of the penstock exceeds 1.0 m.



$$h = d^2$$

Where,

- h : water depth from the centre of the inlet to the lowest water level of the head tank = covering water depth (m)
- d : inner diameter of the penstock (m)

- Covering Water Depth

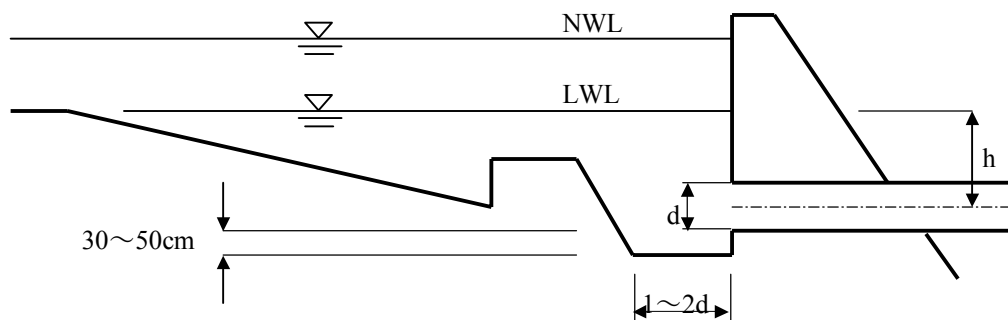
The covering water depth at the penstock inlet must be above the following value to prevent the occurrence of inflow turbulence.

$$d \leq 1.0 \text{ m} \rightarrow h \geq 1.0 d$$

$$d > 1.0 \text{ m} \rightarrow h \geq d^2$$

Where,

- h : water depth from the centre of the inlet to the lowest water level of the head tank = covering water depth (m)
- d : inner diameter of the penstock (m)

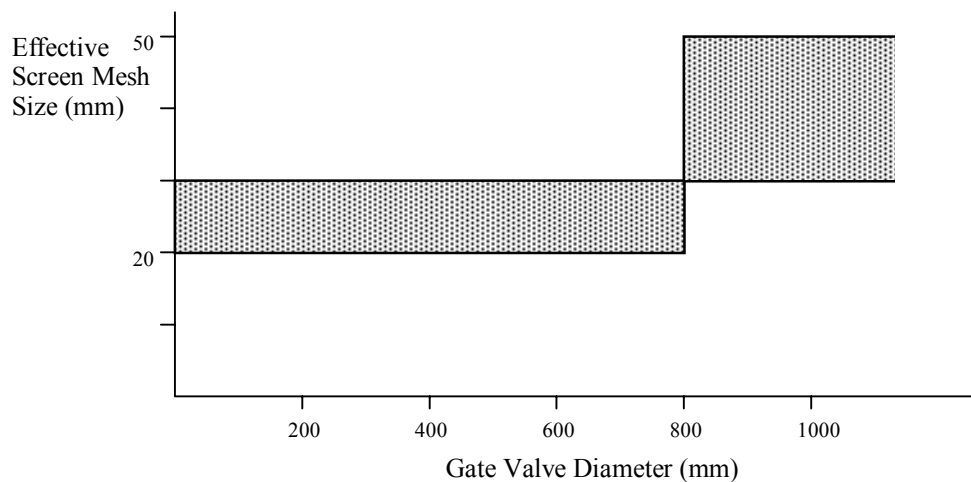


- Installation height of penstock

There are many reports of cases where inappropriate operation has caused the inflow of sediment into the penstock, damage the turbine and other equipment. Accordingly, it is desirable for the inlet bottom of the penstock to be placed slightly higher than the apron of the head tank (some 30 – 50 cm).

(2) Appropriate spacing of screen bars for turbine type, etc.

The spacing of the screen bars (effective screen mesh size) is roughly determined by the gate valve diameter but must be finalised in consideration of the type and dimensions of the turbine and the quantity as well as quality of the litter. The reference value of an effective screen mesh size is shown below.



Effective screen mesh size (reference)

(3) Installation of vent pipe to complement head tank gate

When a head tank gate is installed instead of a gate valve for a power station, it is necessary to install a vent pipe behind the head tank gate to prevent the rupture of the penstock line.

In this case, the following empirical formula is proposed to determine the dimensions of the vent pipe.

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

Where,

d : inner diameter of the vent pipe (m)

P : rated output of the turbine (kW)

L : total length of the vent pipe (m)

H : head (m)

Source: Sarkaria, G.S., "Quick Design of Air Vents for Power Intakes", Proc. A.S.C.E.,
Vol. 85, No. PO.6, Dec., 1959

(4) Spillway at the headtank

Generally, the spillway will be installed at the head tank so that excessive quantity of water is discharged to the river safely when the turbine is stopped.

The size of spillway is decided by following equation.

$$Q_d = C \times B_{spw} \times h_{spw}^{1.5} \rightarrow h_{spw} = \{Q_d / (C \times B_{spw})\}^{1/1.5}$$

Q_d : design discharge (m^3/s)

C : coefficient, usually $C=1.8$

B_{spw} : width of spillway (m, refer to Fig 5.1.1)

h_{spw} : depth at the spillway (m)

5.6 Penstock

5.6.1 Penstock material

At present, the main pipe materials for a penstock are steel piping, ductile iron piping and FRPM (fibre reinforced plastic multi-unit) piping. In the case of a small-scale hydropower plant, the use of hard vinyl chloride piping, Howell piping or spiral welded piping can be considered because of the small diameter and relatively low internal pressure. The characteristics of each pipe material are shown in “Table 5.6.1 – Penstock pipe materials for small-scale hydropower plant”.

5.6.2 Calculation of steel pipe thickness

The minimum thickness of steel pipe of penstock is determined by following formula.

$$t_0 = \frac{P \times d}{2 \times \theta_a \times \eta} + \delta t \text{ (cm)} \quad \text{and } t_0 \geq 0.4 \text{ cm or } t_0 \geq (d+80)/40 \text{ cm}$$

where, t_0 : minimum thickness of pipe

P: design water pressure i.e. hydrostatic pressure + water hammer

(kgf/cm^2), in micro-hydro scheme $P=1.1 \times$ hydrostatic pressure.

for instance, if the head (H_p , refer to following figure) from head tank to turbine is 25m, $P=2.5 \times 1.1=2.75 \text{ kgf/cm}^2$.

d: inside diameter (cm)

θ_a : admissible stress (kgf/cm^2) SS400: 1300 kgf/cm^2

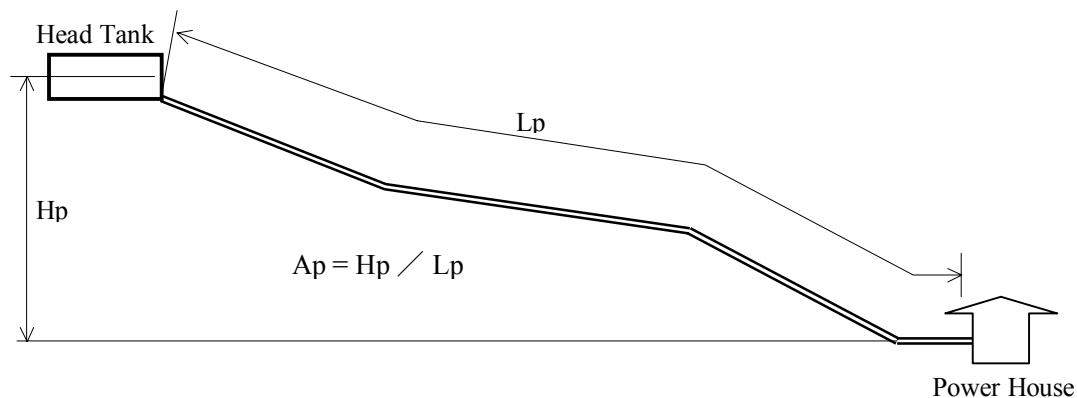
η : welding efficiency (0.85~0.9)

δt : margin (0.15cm in general)

5.6.3 Determining the Diameter of Penstock

Generally the diameter of penstock is determined by comparison between the cost of penstock and head loss at penstock. However a simple method for determining the diameter of penstock is indicated in [Reference 5-2 Simple Method for Determining the Diameter of Penstock].

The diameter of penstock will be determined from “Average angle of Penstock (A_p : see following figure) “ and “Design Discharge (Q_d)”.



For instance, in the case of design discharge (Q_d)=0.50m³/s, length of penstock (L_p)=60m, height from head tank to power house (H_p)=15m, average angle (A_p)=15/60=0.25, the optimum velocity (V_{opt}) is determined as about 2.32 in Reference 5-2. Therefore the diameter of penstock pipe (d) is

$$d = 1.273 \times (Q_d / V_{opt})^{0.5} = 1.273 \times (0.5 / 2.32)^{0.5} = 0.59 \text{ m}$$



Table 5.6.1 Penstock pipe materials for small-scale hydropower plant

	Resin Pipe				Iron Pipe		
	Hard Vinyl Chloride Pipe	Howell Pipe	FRP Pipe	Steel Pipe	Ductile Iron Pipe	Spiral Welded Pipe	
Characteristics	<ul style="list-style-type: none"> • Most popular material for a pipeline as it is frequently used for water supply and sewer lines • Effective for a pipeline with a small discharge • Rich variety of ready-made irregular pipes • Often buried due to weak resistance to impact and large coefficient of linear expansion 	<ul style="list-style-type: none"> • Basically resistant to external pressure but ready-made pipes to resist internal pressure are available • Relatively easy fabrication of irregular pipes due to easy welding • Basically used as a buried pipe 	<ul style="list-style-type: none"> • Plastic pipe reinforced by fibre glass • Used as an exposed pipe and can be made lighter than FRPM pipe with a thinner wall as it is not subject to external load other than snow 	<ul style="list-style-type: none"> • Popular choice to penstock at a hydropower plant • Reliable material due to established design techniques 	<ul style="list-style-type: none"> • Often used for water supply, sewer, irrigation and industrial pipes • Generally used as a buried pipe although exposed use is also possible • High resistance to both external and internal pressure 	<ul style="list-style-type: none"> • Some examples of use for a pipeline • Mainly used as a buried pipe for appearance to hide a spiral welding line • Can be used as steel pipe piles 	
Maximum Pipe Diameter (mm)	Thick pipe: Ø300 Thin pipe: Ø800	Ø2,000	Ø3,000	approx. Ø3,000	Ø2,600	Ø2,500	
Permissible Internal Pressure (kgf/cm ²)	Thick pipe: 10 Thin pipe: 6	2.0 – 3.0	Class A: 22.5	133	approx. 40	15	
Hydraulic Property (n)	0.009 – 0.010	0.010 – 0.011	0.010 – 0.012 (approx. 0.011 in general)	0.010 – 0.014 (approx. 0.012 in general)	0.011 – 0.015 (approx. 0.012 in general)	-	

	Resin Pipe			Iron Pipe		
	Hard Vinyl Chloride Pipe	Howell Pipe	FRP Pipe	Steel Pipe	Ductile Iron Pipe	Spiral Welded Pipe
Workability	<ul style="list-style-type: none"> • Easy design and work due to light weight and rich variety of irregular pipes 	<ul style="list-style-type: none"> • Good workability due to light weight 	<ul style="list-style-type: none"> • Good workability due to light weight and no need for on-site welding as a specially formed rubber ring is used for pipe connection • Steel pipes are used for irregular sections because of the limited availability of irregular FRP pipes 	<ul style="list-style-type: none"> • Inferior workability to FRP pipes 	<ul style="list-style-type: none"> • Inferior workability to FRP pipes 	<ul style="list-style-type: none"> • Inferior workability to FRP pipes
Water-tightness	<ul style="list-style-type: none"> • Good water-tightness as bonding connection is possible 	<ul style="list-style-type: none"> • No problem of water-tightness at the joints 	<ul style="list-style-type: none"> • No problem of water-tightness as the joint connection method is established 	<ul style="list-style-type: none"> • No problem of water-tightness as the joint connection method is established 	<ul style="list-style-type: none"> • Good 	<ul style="list-style-type: none"> • No problems

5.7 Foundation of Powerhouse

Powerhouse can be classified into ‘the above ground type or the semi-underground type’ and ‘the under ground type’. Most small-scale hydro power house are the above ground type.

The dimensions for the floor of powerhouse as well as the layout of main and auxiliary equipment should be determined by taking into account convenience during operation, maintenance and installation work, and the floor area should be effectively utilized.

Various types of foundation for powerhouse can be considered depending on the type of turbine. However the types of foundation for powerhouse can be classified into ‘for Impulse turbine’ (such as Pelton turbine, Turgo turbine and cross flow turbine) and ‘for Reaction turbine’ (Francis turbine, propeller turbine).

5.7.1 Foundation for Impulse Turbine

Fig.5.7.1 shows the foundation for cross flow turbine, which frequently is used in the micro-hydro scheme as an impulse turbine. In case of impulse turbine, the water passing by the runner, is directly discharged into air at tailrace. The water-surface under the turbine will be raged. Therefore the clearance between the slab of powerhouse and water-surface at the afterbay should be kept at least 30-50cm. The water depth (h_c) at the afterbay can be calculated by following equation.

$$h_c = \left(\frac{1.1 \times Qd^2}{9.8 \times b^2} \right)^{1/3}$$

h_c : water depth at afterbay (m)

Qd : design discharge (m^3/s)

b : width of tailrace channel (m)

The water level at afterbay should be set higher than the estimated flood water level.

Then in case of impulse turbine, the head between the center of turbine and water level at the outlet becomes the head-loss (H_{L3} : refer to Ref.5-3).

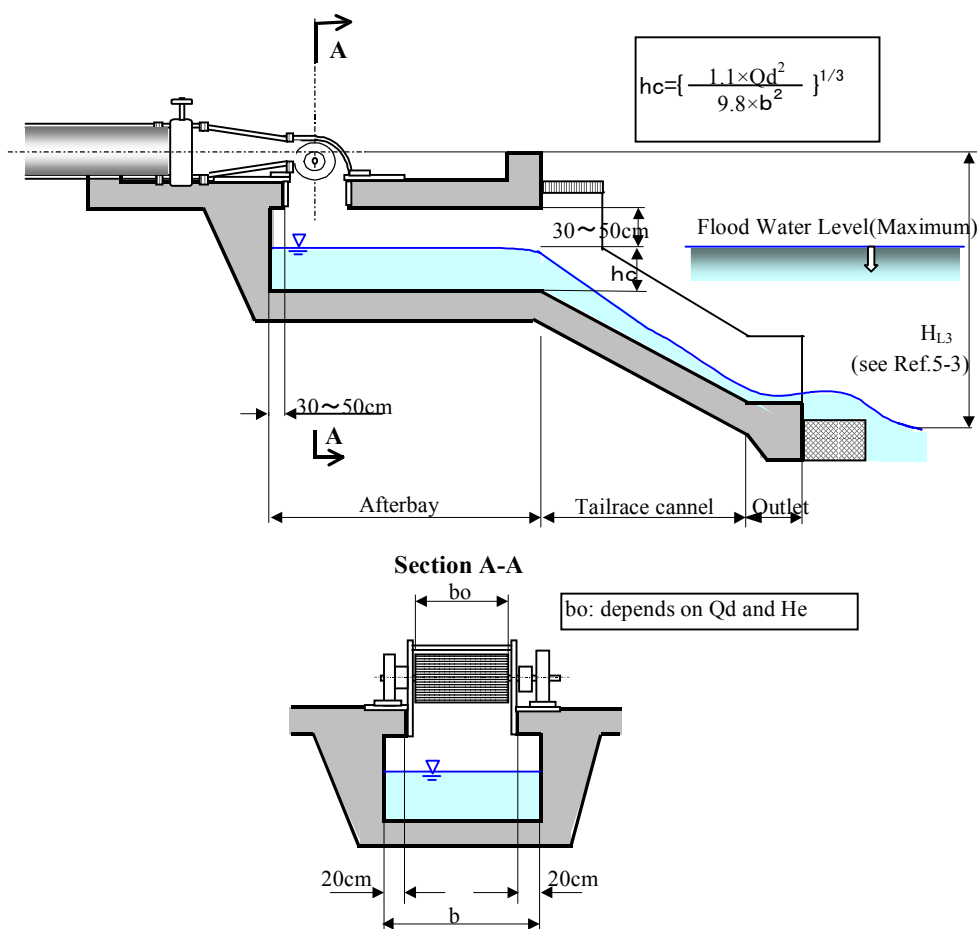


Fig.5.7.1 Foundation of Powerhouse for Impulse Turbine (Crossflow turbine)

5.7.2 Foundation for Reaction Turbine

Fig.5.7.2 (a) shows the foundation for Francis turbine, which is a typical turbine of the reaction turbine. The water is discharged into the afterbay through the turbine.

In case of a reaction turbine, the head between center of turbine and water-level can be used for power generation. Then it is possible that the turbine is installed under the flood water level on condition that the following equipment is provided (see Fig. 5.7.2(b)).

- a. Tailrace gate
- b. Pump at powerhouse

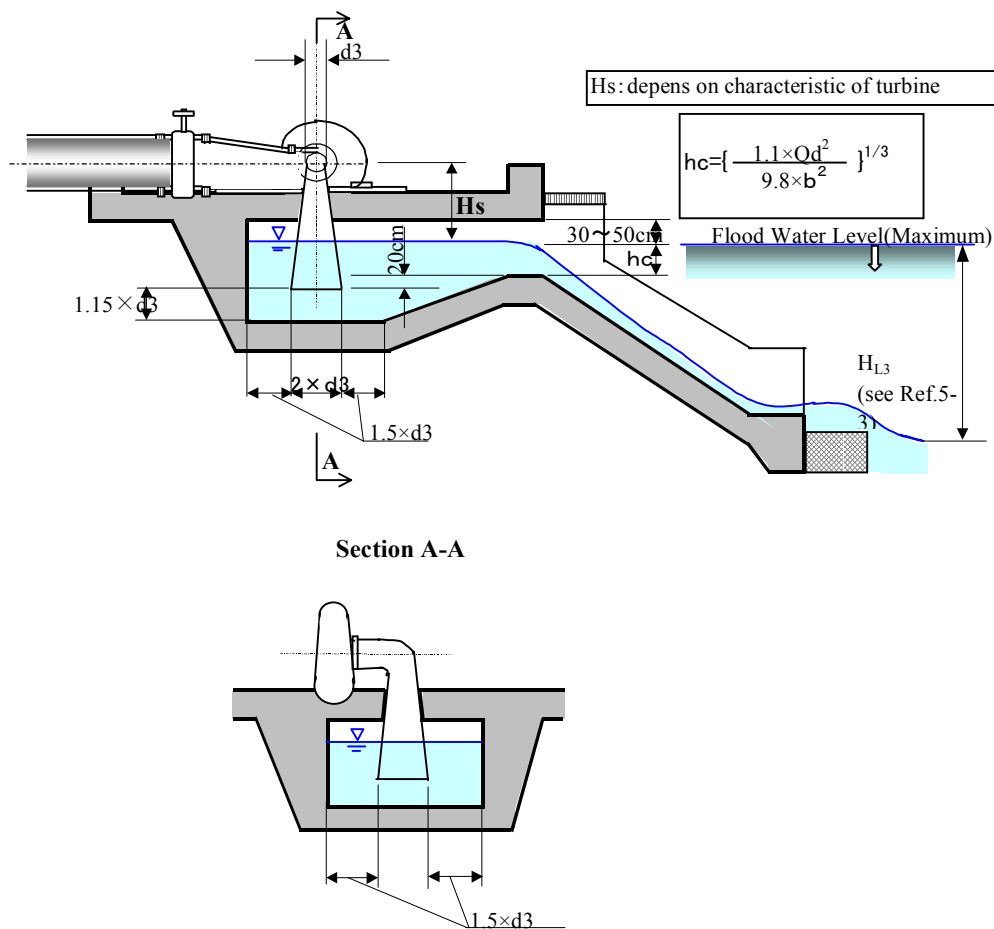


Fig 5.7.2(a) Foundation of powerhouse for Reaction Turbine (Francis turbine)

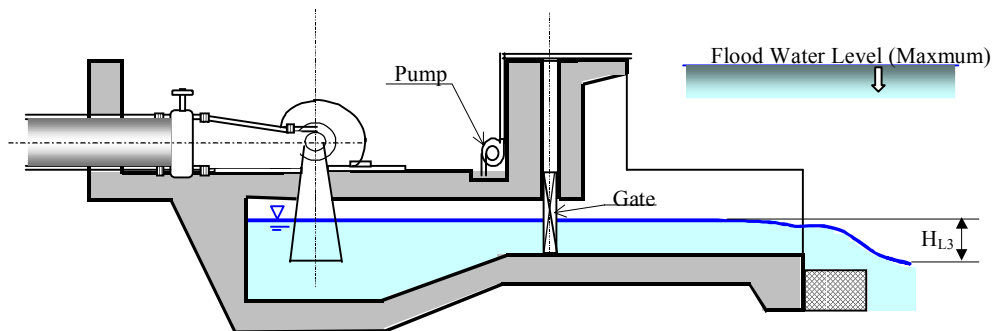


Fig 5.7.2(b) The way of installation to Lower Portion as an Example