CHAPTER 9

BASIC DESIGN

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Chapter 9 Basic Design

9.1 General

The design in the Study is carried out at more detailed level than conducted in a feasibility study on a hydropower project, in accordance with S/W for the Study. Main outputs of the basic design are listed below:

- Main features of facilities for the Project,
- General plan and profile of the Project,
- Drawings of plan and profile of main civil structures (headrace tunnel, surge tank, steel penstock, powerhouse, and outlet) and, access adit,
- Plan of layout of main equipment in the powerhouse (those are included in the above drawings)
- Plan of layout of main equipment in the switchyard (those are included in the above drawings,
- Single-line diagram,
- Quantities of main work items of civil works,
- Construction planning for civil works,
- Unit prices of main work items of civil works,
- Construction cost (including cost for environmental mitigation measures) and
- Implementation schedule of the Project.

In this Chapter, the examination results of salient features of civil structures and electromechanical equipment for the optimal development scheme selected in **Chapter 6** are described. Construction planning, estimate of the project cost, and implementation schedule are mentioned in **Chapter 10**.

The proposed expansion project is to connect the existing intake for the expansion and a new powerhouse to be located next to the existing powerhouse with a waterway parallel to the existing waterway. Because the 2 new generating units are to be installed and the existing powerhouse has 3 units, the unit to be installed at the existing powerhouse side is named Unit No. 4 and the other is called Unit No. 5. **Table 9.1-1** shows the salient features of the existing facilities and those of new facilities and electromechanical equipment which are determined based on the results of examinations described in this Chapter.

Table 9.1-1 Salient Features of Victoria Hydropower Expansion Project

Item		Dimension
Reservoir (Existing)	Name of River	Mahaweli river
	Full Supply Level	438.0 m
	Minimum Operation Level	370.0 m
	Available Depth	68.0 m
	Gross Storage Capacity	722×10^6 m^3
	Effective Storage Capacity	688×10^6 m^3
	Design Flood	9,510 m^3/s
Dam (Existing)	Type	Concrete Arch Dam
	Height of Dam	122 m
	Length of Dam Crest	520 m
	Volume of Dam	480×10^3 m^3
Intake for Expansion (Existing)	Number	1
	Type	Inclined Intake
Headrace Tunnel	Number	One (1)
	Inner Diameter	6.6 m
	Total Length	5,003 m
Surge Tank	Type	Restricted Orifice Type
	Diameter	20.0 m (Upper Section) 6.6 m (Lower Section)
	Height	117.0 m (Upper Section) 32.9 m (Lower Section)
Penstock	Type	Tunnel & Open-air
	Number	Tunnel: One (1) Open-air: Two (2)
	Inner Diameter	Tunnel: 6.6 m to 5.6 m Open-air: 3.95 m to 2.85 m
	Length: Tunnel	575 m
	Length Open-air	175 m for Unit 4 160 m for Unit 5
	Total Length	750 m for Unit 4 735 m for Unit 5
Powerhouse	Type	Surface type
	Size	37m wide × 44m high × 69m long
Development Plan	Normal Intake Water level	430.0 m
	Normal Tail Water Level	231.2 m
	Gross Head	199.0 m
	Effective Head	183.3 m
	Maximum Discharge	140 m^3/s
	Number of Unit	Two (2)
	Install Capacity	228 MW (only expansion)
	Peak Duration Time	3 hours
	95% Dependable Capacity	393 MW (with existing)
	Annual Generation Energy	716 GWh (with existing)
	(Firm Energy*)	468 GWh (with existing)
	(Secondary Energy**)	248 GWh (with existing)

Item		Dimension
Turbine	Type	Vertical Shaft, Francis Turbine
	Number	Two (2)
	Rated Output	122 MW per unit
	Revolving Speed	300 r/min
Generator	Type	Three-phases, Synchronous Generator
	Number	Two (2)
	Rated Output	140 MVA per unit
	Frequency	50 Hz
	Voltage	16.5 kV
	Power Factor	0.85 lag
Main Transformer	Type	Outdoor Special Three-phase Type or Outdoor Single Phase Type
	Number	Two (2)
	Capacity	145 MVA per unit
	Voltage	Primary 16.5 kV
		Secondary 220 kV
	Cooling	Natural Convection Oil Forced Air Type
Switchyard	Type	Conventional Type
	Bus System	Double Bus
	Number of Lines Connected	Three (3) cct Transmission Lines
	Voltage	220 kV

*Note: * "Firm energy" means the total of power generated during 3-hour peak duration.*
*** "Secondary energy" means the total of power generated in duration except 3-hour peak time.*

Hydraulic calculations such as head loss calculation, surging analysis and water hammer for penstock have been conducted in the basic design, but dimensions of main structure members are determined with referring to those of similar-size hydropower projects.

Geological drawings of the main structures and design drawings prepared in the basic design are attached in Section **9.8**.

9.2 Maximum Allowable Vibration due to Blasting against Existing Structures

Open-air works and underground works of the Project are to be carried out near the existing Victoria dam, intake facilities, waterway and powerhouse. Hence, vibrations caused by blasting should be controlled to prevent them from being damaged due to the blasting. In this section, the maximum allowable blasting vibration is examined and determined.

9.2.1 Characteristics of Vibration due to Blasting

(1) Comparison of Vibrations due to Earthquakes and Those due to Blasting

Generally, main characteristics of vibration due to blasting are compared with those due to earthquakes as follows:

1) Because vibration energy caused by blasting is small and vibration occurs locally, limited area is affected by the blasting.
2) Frequency of earthquake vibration generally ranges from 1 to 5 Hz, while that of blasting vibration falls in high frequency range from 10 to 200 Hz.
3) Vibrations caused by earthquakes last for several seconds to several minutes, while that due to blasting finish within almost one second.

Table 9.2.1-1 Comparison of Vibrations due to Earthquakes and Blasting

Item	Earthquakes	Blasting
Affected area where vibration is felt	Several 100 km from the hypocenter	At most several 100 m from the blasting point
Frequency of vibration	Around 1 to 5 Hz (depending on characteristics of ground)	10 to 200 Hz or more
Duration of Vibration	Several seconds to several minutes	Within one second

Source: Japan Explosives Industry Association

(2) Units of Vibration

In the case of sine-curve vibration, vibration displacement (Y) at time t is shown by the following equation:

$$Y = A \sin(2f\pi t)$$

Where,

A : Amplitude of displacement
f : Frequency
π : Circle ratio

Vibrations can be indicated by using vibration velocity (V) or acceleration (α), and their maximum values have the following relations;

$$V = 2f\pi A$$
$$\alpha = 2f\pi V = (2f\pi)^2 A$$

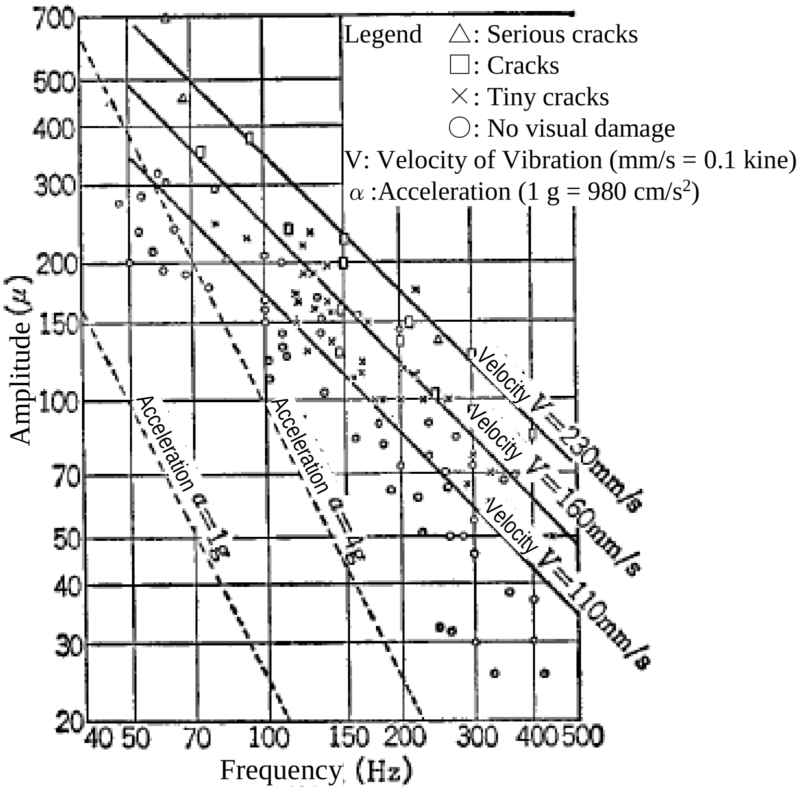
The units mentioned in **Table 9.2.1-2** are used for vibration displacement, velocity and acceleration.

Table 9.2.1-2 Units for Vibration

	Standard Unit	Unit used practically
Displacement	cm	mm, μ (= 0.001 mm)
Velocity	cm/s	kine (cm/s)
Acceleration	cm/s^2	gal (= cm/s^2), g (= 980 cm/s^2)

(3) Damages due to Blasting Vibration

Figure 9.2.1-1 shows relations among ground vibration amplitudes, vibration frequencies and damages caused by blasting observed by Langefors et al.



(by Langefors et. al,. Source: Japan Explosives Industry Association)

Figure 9.2.1-1 Relations between Amplitude and Damages of Buildings due to Blasting Vibration

It is understood according to **Figure 9.2.1-1** that similar damages are caused regardless of displacement or acceleration amplitudes but are caused under almost same velocity amplitude and that there is relation between vibration velocity amplitude and kinds of damages. Therefore, the maximum allowable vibration value is indicated with a unit of vibration velocity. The "kine" is used as a unit of vibration velocity.

9.2.2 Damages of Blasting Vibration to Concrete Structures

(1) Estimate by Three Dimensional Elastic Theory

Stresses caused by vibration in a certain material are calculated with the following equation by using the three dimensional elastic theory:

$$\sigma = \frac{pCV}{g} \times \frac{(1-2\nu)(1+\nu)}{1-\nu} \qquad (1)$$

Where,

σ	:	Stress (kg/cm^2)
p	:	Density of material (g/cm^3)
C	:	Elastic wave velocity of material (m/s)
ν	:	Poisson's ratio of material
g	:	Gravity acceleration (980 cm/s^2)

V : Vibration velocity (cm/s)

Hence, vibration which cause a certain stress is calculated with the equation (1) as follows;

$$V = \frac{\sigma g}{pC} \times \frac{1-\nu}{(1-2\nu)(1+\nu)} \quad \text{.........} (2)$$

In the case that the material is concrete, the following general values can be used.

$p = 2.5\ g/cm^3$
$C = 3{,}000\ m/s = 300{,}000\ cm/s$
$\nu = 0.25$

When concrete tensile strength = 20 kg/cm^2 *[1], the blasting vibration velocity which causes cracks in the concrete is calculated as V = 31.4 kine with the equation (2).

(2) Experiment by Using Actual Tunnel

A few experimental tests by using actual tunnel structures in Japan were conducted, to clarify blasting vibration velocity commencing to cause cracks in the lining concrete. Results are shown in **Table 9.2.2-1**.

Table 9.2.2-1 Experiment Result of Blasting Vibration for Crack Generation

Name of tunnel	Object	Velocity of vibration commencing to cause cracks
Hibi Tunnel	Lining concrete	more than 30 kine
Okitsu Tunnel	Lining concrete	more than 30 to 40 kine
Wakayama Tunnel	Shotcrete	more than 70 kine

Source: Japan Explosives Industry Association

The lowest values in the **Table 9.2.2-1** almost correspond to the calculated value, in (1) above, which commence to cause cracks in the concrete.

9.2.3 Damages against Rock Slopes

Affects caused by blasting vibration to rock slopes by Oriard is shown in **Table 9.2.3-1**;

Table 9.2.3-1 Affects on Rock Slopes due to Blasting Vibration

Vibration Velocity due to Blasting	Affects
5.1 to 10.2 kine	Rock fragments on rock slope fall
12.7 to 38.1 kine	Loose parts of rock slope collapse
63.5 kine or over	Week slopes start to damage

Source: Japan Explosives Industry Association

[1] Lining concrete of the headrace tunnel of the existing Victoria Hydropower Station had a design compressive strength of 200 kg/cm^2. In general, concrete tensile strength is estimated to be around one-tenth of compressive strength. Hence, concrete tensile strength of 20 kg/cm^2 is adopted in this estimate.

9.2.4 Maximum Allowable Vibration Velocity for Existing Underground Structures

(1) Railway Tunnel to be Excavated Near Existing Structure

According to “Manual on Measures for Tunnel Works near Existing Tunnels” prepared by Railway Technical Research Institute in Japan, the maximum value is to be determined in consideration of soundness of existing tunnels. **Table 9.2.4-1** shows classification of soundness of existing tunnels, and **Table 9.2.4-2** shows the maximum allowable vibration velocity.

Table 9.2.4-1 Status (Soundness) of Lining Concrete of Existing Tunnel

Class of Soundness	Affects to Normal Operation	Deformation of Tunnel	Countermeasures for Repair
AA	Dangerous at the present time	Serious	To be taken immediately
A1	To become dangerous in near future	Large deformation and lowering function	To be taken urgently
A2	To become dangerous in future	Deformation is possible to proceed and function may lower	When required, to be taken
B	If worse, to be Classes A	If worse, to be Classes A	To be monitored and to be taken when required
C	No affect at the present time	Slight	To be inspected intensively
S	No affect	No deformation	Not necessary

Source: Japan Explosives Industry Association

Table 9.2.4-2 Maximum Allowable Vibration Velocity

Class of Soundness	Maximum Allowable Vibration Velocity
AA	2 kine
A1, A2	3 kine
B, C, S	4 kine

Source: Japan Explosives Industry Association

It is understood that the maximum allowable velocity of 2 to 4 kine means a safety factor is assumed as around 8 to 15 in comparison with calculated value in **9.2.2 (1)**.

(2) Maximum Allowable Vibration Velocity Limit of Railway and Road Tunnels in Japan

Table 9.2.4-3 shows allowable blasting vibration limits applied to railway and road tunnels which were constricted near the existing structures in Japan.

Table 9.2.4-3
Allowable Blasting Vibration Applied to Railway and Road Tunnel Projects in Japan

Name of Tunnel	Allowable Velocity (kine)	Shortest Distance to Existing Tunnel (m)
Hibi Tunnel (Sanyo Shinkansen)	2.5	11.4
Muikamachi Tunnel (Joetsu Shinkansen)	1.0	1.8
Sasago Tunnel (Chuo Highway)	6.5	17.0
Kinmeiro Tunnel (Sanyo Highway)	5.0	5
Gorigamine Tunnel (Joetsu Highway)	4.0	n.a.
Nagamine Tunnel (Hanwa Highway)	4.0	n.a.
Mihara No.5 Tunnel (Highway)	1.0*	15.5

*Note: *Existing tunnel was classified as AA in the manual mentioned in 9.2.4 (1)*
Source: Japan Explosives Industry Association, etc.

The allowable values range from 2.5 to 6.5 kine except the case that soundness of existing tunnel is very poor and that distance of existing and new tunnels is very small.

(3) Technical Guidelines for Controlled Blasting in India

The following are proposed in “Technical Guidelines for Controlled Blasting” issued by Central Institute of Mining & Fuel Research in India in October 2007.

1) Affects to Rock Mass

Affects to rock mass due to blasting is controlled with strain in the rock mass in the guidelines based on observations by Richard and Moore; “Strain induced by blasting vibration leading to damages is about 10% of the tensile failure strain of the rock and this limit is considered as safe value”. It is understood that the limit has the safety factor of 10.

2) Affects to Mass Concrete

The limit of blasting vibration is controlled with concrete age and distance to the existing structures by Oriard, and indicated with vibration velocity (refer to **Table 9.2.4-4**).

Table 9.2.4-4 Blasting Vibration Limits for Mass Concrete (after Oriad)

Concrete Age	Allowable Velocity from Blasting (kine)	Distance Factor (D.F.)
0 – 4 hrs	10.2 × D.F.	Distance: 0-15 m; D.F. = 1.0 Distance:15-46 m; D.F. = 0.8 Distance:46-76 m; D.F. = 0.7 Distance: >76 m; D.F. = 0.6
4 hrs – 1 day	15.2 × D.F.	
1 day – 3 days	22.9 × D.F.	
3 days – 7 days	30.5 × D.F.	
7 days – 10 days	37.5 × D.F.	
10 days or more	50.8 × D.F.	

Source: Technical Guidelines for Controlled Blasting, Central Institute of Mining & Fuel Research, Oct 2007

The allowable vibration limits in **Table 9.2.4-4** is considered as limits without any safety factor, in comparison with those in (1) and (2) of this Section.

(4) Allowable Blasting Vibration Applied to Hydropower Projects Undertaken by J-Power

Table 9.2.4-5 shows the allowable blasting vibration applied the hydropower expansion projects undertaken by Electric Power Development Co., Ltd. (J-Power). Blasting works were carried out with the allowable vibration of 2 kine against the existing dam, waterway tunnel and powerhouse.

Table 9.2.4-5
Allowable Blasting Vibration Applied to Hydropower Expansion Project by J-Power

Project Name	Allowable Blasting Vibration Velocity (kine)	Remarks
Akiha No. 3	2	Existing structures including concrete gravity dam were located near new structures.
Okutadami Expansion *	2	Expansion of underground type powerhouse. Existing structures including concrete gravity dam, intake facilities, and underground powerhouse were located near new structures..
Ootori Expansion	2	Existing structures including concrete arch dam, intake facilities were located near new structures..

** Blasting work was carried out under operation of the existing generation equipment*
Source;Electric Power Development Co., Ltd. (J-Power)

Table 9.2.4-6 shows allowable vibration against the existing concrete gravity dam due to drilling machines which made a hole for installation of penstock steel pipes.

Table 9.2.4-6
Allowable Vibration due to Drilling Machine to Make Opening in Concrete Gravity Dam

Project Name	Allowable Vibration Velocity (kine) due to Machine	Remarks
Akiha No. 3	2.0	To install steel penstock, make an opening with 6.5 m in diameter
Okutadami Expansion	2.0	To install steel penstock, make an opening with 6.2 m square

Source; Electric Power Development Co., Ltd. (J-Power)

It should be noted that no damages were caused to the existing structures by the above projects..

9.2.5 Maximum Allowable Vibration Velocity for the Victoria Hydropower Expansion Project

The following are noted based on theoretical value, the limits applied to the projects, and guidelines:

1) According to the results of inspections conducted in 2000 for the waterway tunnel of the Victoria Hydropower Station, the class of soundness in **9.2.4 (1)** is estimated as C to S. Hence the maximum blasting vibration velocity for the existing tunnel is considered to be 4 kine.

2) However, stricter limit than 4 kine should be applied to the Project, because blasting works are conducted near the existing arch dam and pressure tunnel.

3) Therefore, 2 kine should be applied to the Project, in consideration of the allowable vibration velocity applied to the J-Power's hydropower expansion projects. The limit of 2 kine means

the safety factor of around 15 is considered against vibration to cause cracks in concrete calculated in **9.2.2 (1)**.

An examination on tunnel alignment, construction planning, preparation of construction schedule, etc. should be carried out in consideration of the limit of 2 kine in the basic design.

Construction methods without blasting will be examined for works near the existing intake facilities for expansion and the existing powerhouse.

9.3 Waterway

9.3.1 Route Setup

Construction of the intake, the headrace tunnel approximately 20 m in length from the intake gate and the access adit, preparation of the open space for the surge tank, and reclamation of the powerhouse space for the Project were already completed at the locations shown in **Figure 9.3.1-1** during construction of the previous project, therefore the new waterway is arranged to connect them.

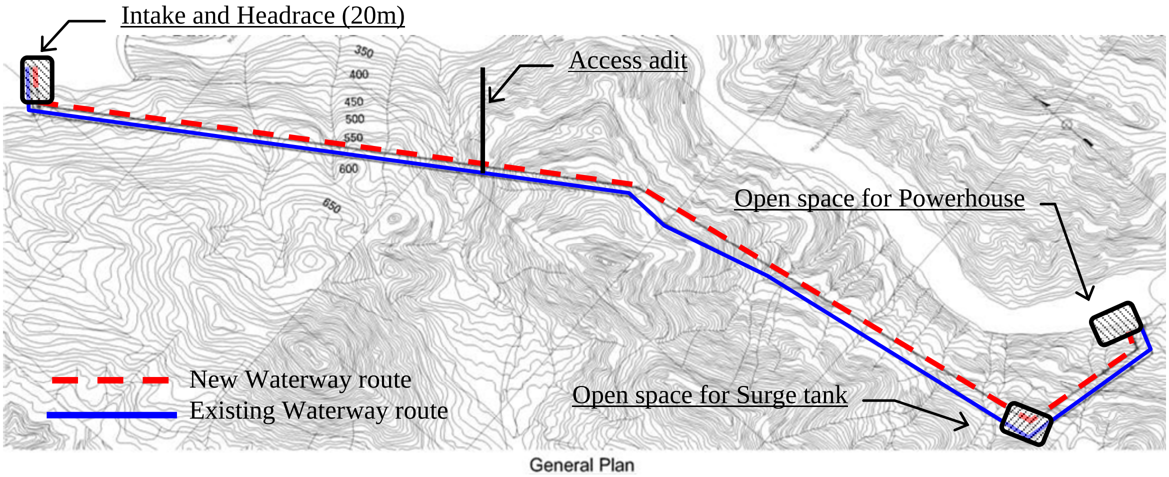


Figure 9.3.1-1 Route of Waterway

Negative impact on the existing headrace structures due to the vibration of blasting during the excavation of the new headrace tunnel shall be avoided. To avoid the negative impact on the existing headrace structure, velocity of vibration on the existing headrace lining concrete shall be limited to 2 cm/s as mentioned in **9.2.5**. The required distance between the center of blasting and the existing headrace lining concrete to satisfy the above mentioned limitation is estimated by the following empirical equation[2].

$$V = K \cdot W^{\frac{2}{3}} \cdot D^{-2}$$

Where,

V : Velocity of vibration (cm/s)

[2] Empirical Equation by ASAHI KASEI, Explosive manufacturer in Japan

K : Coefficient related to blasting conditions
(Center-Cut: 750, Side Hole Blasting 350)

W : Loading of explosive per rotation (kg)

D : Distance from the center of the blasting (m)

In consideration of the total construction schedule, 2.5 m excavation by one blasting is required. Therefore, the distance between the existing and the new headrace tunnels is set to 36 m (43.5 m from the center to the center of the tunnels).

The trial blasting shall be carried out prior to the commencement of the tunnel excavation, to modify the above empirical equation to fit the actual ground condition. The actual blasting shall be controlled based on the result of this trial blasting.

The tunnel shall be stable against internal water pressure in the waterway. To keep the stability, vertical and horizontal depth of the tunnel cover shall satisfy following inequality.

$$Hh < \gamma_{rock} h$$

Where,

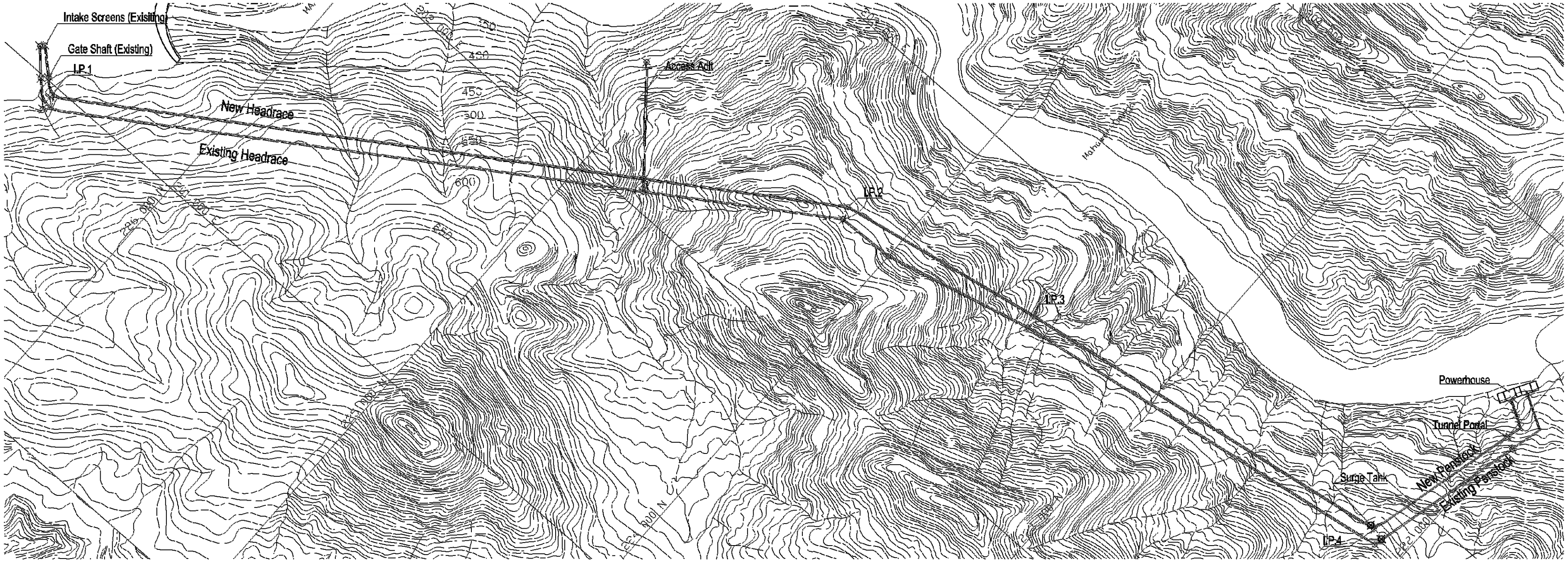
H : Static Water Head (FSL 438 m - elevation of new headrace tunnel)

γ_{rock} : Density of Rock (2.4 t/m^3)

h : Depth of Cover (m)

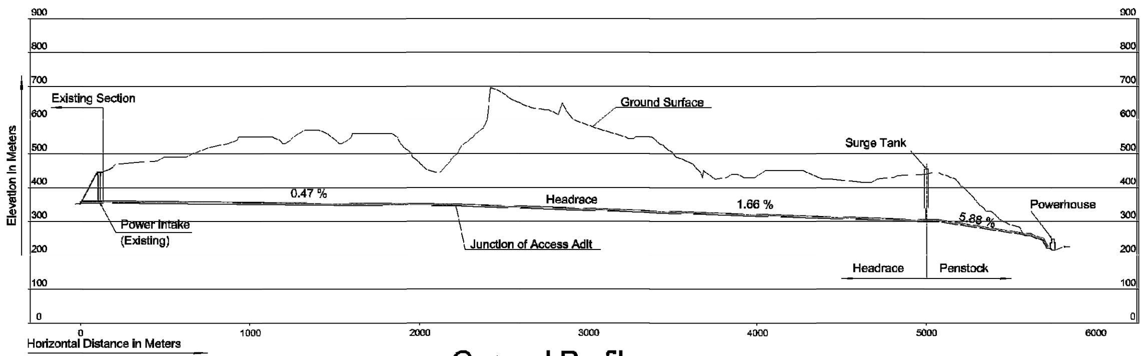
Accordingly, the new waterway route is set as shown in **Figure 9.3.1-2** to satisfy the above mentioned conditions.

For reference, cross sections showing the actual and required depth of cover of the tunnel in low elevation sections are attached into Appendix-II.



General Plan

Scale a



General Profile

Horizonal; scale a. Vertical; scale b

	E	N	Bend Radius (m)	Holizontal Chainage (m)	Center EL. (m)
Intake screen (Existing)	201,081.44	226,643.99		0.000	
Gate shaft (Existing)	201,008.01	226,562.62		109.606	358.100
T.P.1 (A)	-	-		135.512	357.979
I.P.1	200,961.08	226,515.37	50.000	-	-
T.P.1 (B)	-	-		203.821	357.598
Adit junction	202,092.79	224,803.53		2,215.245	348.250
T.P.2 (A)	-	-		2,908.350	336.743
I.P.2	202,482.73	224,213.70	70.000	-	-
T.P.2 (B)	-	-		2,935.920	336.279
I.P.3	202,622.33	223,488.85		-	-
Change in Gradient	202,836.10	222,183.44		4,982.916	302.302
Start of Steel Lining	202,836.10	222,183.44		4,982.916	302.302
T.P4 (A)	-	-		4,987.916	302.302
Surge tank	202,847.41	222,151.53	70.000	5,002.497	302.302
I.P.4	202,844.37	222,132.97	70.000	-	-
T.P.4 (B)	-	-		5,069.580	302.302
Start of Contraction	202,889.16	222,121.87		5,069.580	302.302
End of Contraction	202,894.69	222,120.50		5,069.580	301.967
Tunnel portal	203,381.15	222,000.01		5,576.437	272.496

SCALE a 0 — 2 km

SCALE b 0 — 1 km

Figure 9.3.1-2 Waterway Plan and Profile

9.3.2 Headrace

A 5 km long headrace tunnel is designed as a single line pressure tunnel with a circular section.

The inner diameter of the headrace tunnel is determined in order to minimize the sum of annualized cost of the headrace tunnel (C), which consists of annualized construction and O&M costs, and annual power revenue loss due to the head loss (L) for alternative diameters ranging from 6.0 m to 7.2 m. As shown in **Table 9.3.2-1** and **Figure 9.3.2-1**, C+L takes the minimum value at diameter 6.6 m, therefore the inner diameter of the headrace is determined as 6.6 m.

Table 9.3.2-1 Comparison of the Headrace Diameter

Item	Unit	Alternative Diameter						
Headrace Diameter: D	m	6.0	6.2	6.4	6.6	6.8	7.0	7.2
Cost: C	10^3 USD	5,387	5,744	6,112	6,492	6,883	7,286	7,700
Loss: L	10^3 USD	4,241	3,563	3,010	2,556	2,181	1,870	1,611
C+L	10^3 USD	9,628	9,307	9,122	**9,048**	9,065	9,156	9,311

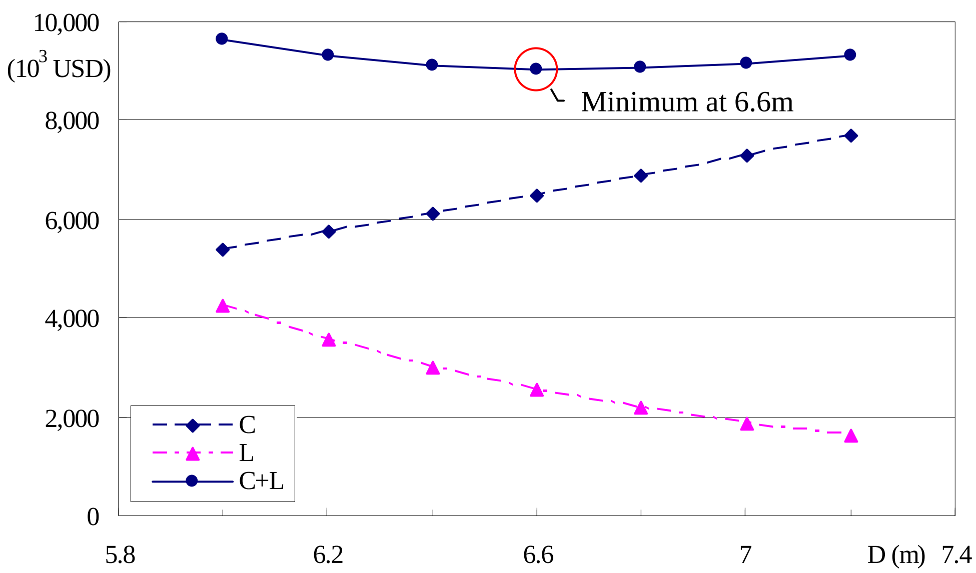


Figure 9.3.2-1 Comparison of the Headrace Diameter

Four types of tunnel support pattern are set depending on the rock condition as shown **Table 9.3.2-2** with reference to the existing headrace tunnel support patterns as mentioned in **7.3.1 (3)**.

Table 9.3.2-2 Headrace Tunnel Support Pattern

Item	Type I	Type II	Type III	Type IV
Shotcrete	Not required	t = 50mm	t = 100mm with wire mesh	t = 100mm with wire mesh
Rock bolt	Not required	L=3m@1.5m	L=4m@1.5m	L=4m@1.5m
Steel Lib	Not required	Not required	Not required	H150×150@1.0m

The reinforced concrete lining will be used for the tunnel, to resist internal water pressure. Thickness of the lining is set 60 cm with reference to the existing pressure tunnels of similar size. Typical sections of the headrace are shown in **Figure 9.3.2-2**.

Contact grouting will be executed between the excavated rock surface and the lining concrete around top portion of the tunnel.

Consolidation grouting of 3 m long with a staggered interval of 1.5 to 3 m will be carried out on entire circumference of the tunnel along its entire length after the strength of the lining concrete reaches the design value.

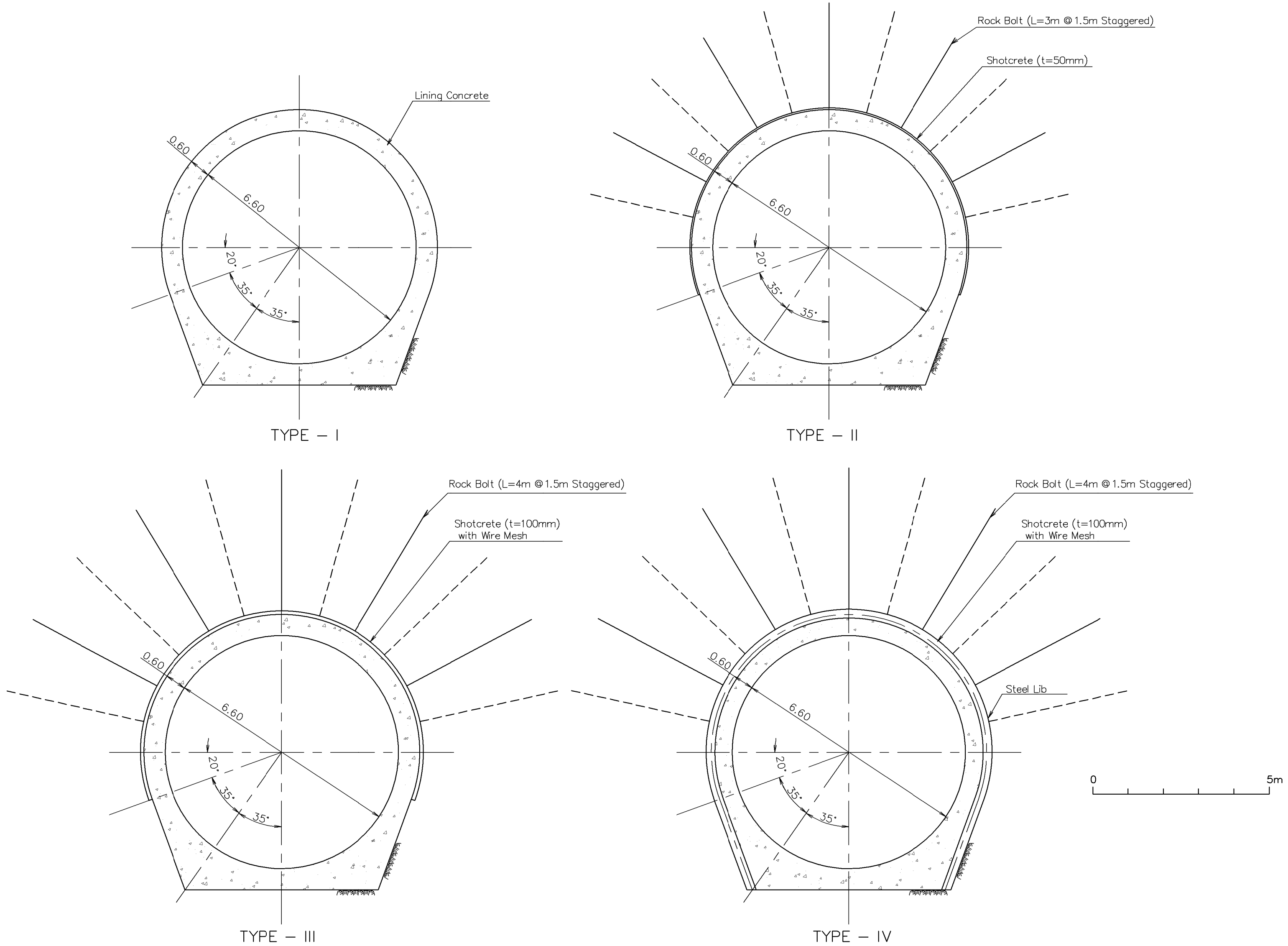


Figure 9.3.2-2 Headrace Typical Section

9.3.3 Penstock

The penstock is designed as a single line tunnel section approximately 575 m long and a two-line open-air section approximately 200 m in length.

It is common in Sri Lanka to install a portal valve immediately downstream of the bifurcation of the open-air section of the penstock, to deal with unexpected contingencies like malfunction of an inlet valve. In the Project, with following this, portal valves are installed immediately downstream of the bifurcation.

The existing penstock in the open-air section has Removable Penstock to carry in the materials and machines for maintenance works, however, it is possible to carry in them which are used in usual maintenance work through the manholes of the open-air penstock. In order to carry in the large-size machines in the waterway, an access manhole is to be installed in the new plug concrete of the existing access adit. Therefore, Removable Penstock is omitted in the new open-air penstock.

The inner diameter of the penstock is determined so as to have the same current velocity upstream and downstream of the bifurcation.

In the same way of the headrace, inner diameter of the penstock upstream of the bifurcation is determined in order to minimize the sum of the annualized cost of the penstock (C) and power revenue loss due to the head loss (L) for alternative diameters ranging from 4.8 m to 6.0 m. As shown in **Table 9.3.3-1** and **Figure 9.3.3-1**, C+L takes the minimum value at diameter 5.6 m, therefore the inner diameter of the penstock in the tunnel section is determined as the 5.6 m, and accordingly inner diameter of the penstock in the open-air section is 3.95 m. Meanwhile, the inner diameter of penstock at the end of inlet valve side is 2.85 m because of the condition of the electromechanical equipment.

Table 9.3.3-1 Comparison of the Penstock Diameter

Item	Unit	Alternative Diameter						
Penstock Diameter: D	m	4.8	5.0	5.2	5.4	5.6	5.8	6.0
Cost: C	10^3 USD	3,095	3,340	3,595	3,848	4,121	4,404	4,696
Loss: L	10^3 USD	3,555	3,021	2,596	2,272	1,998	1,777	1,595
C+L	10^3 USD	6,650	6,361	6,191	6,120	**6,119**	6,180	6,291

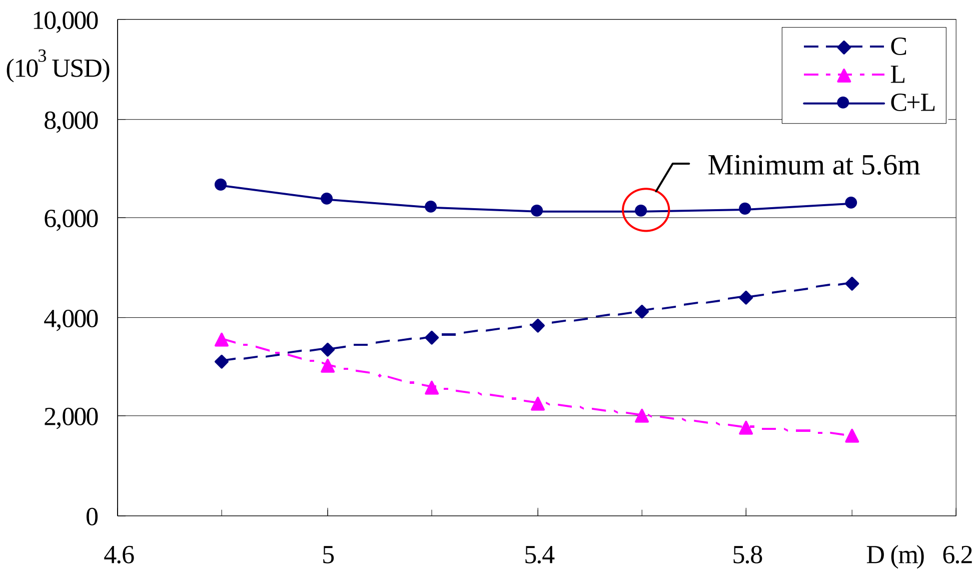


Figure 9.3.3-1 Comparison of the Penstock Diameter

Four types of tunnel support patterns are set depending on the rock condition as shown in **Table 9.3.3-2** with reference to the existing tunnel section penstock support patterns in the same way of the headrace.

Table 9.3.3-2 Penstock Tunnel Support Pattern

Item	Type I	Type II	Type III	Type IV
Shotcrete	Not required	t = 50mm	t = 100mm with wire mesh	t = 100mm with wire mesh
Rock bolt	Not required	L=3m@1.5m	L=4m@1.5m	L=4m@1.5m
Steel Lib	Not required	Not required	Not required	H150×150@1.0m

The working space between the steel pipe and excavated rock surface is 60 cm with reference to the existing tunnel penstock of similar size to secure workability. This space will be filled with backfill concrete after the installation of the steel pipes.

Consolidation grouting of 3 m long with staggered interval of 1.5 to 3 m will be carried out on entire circumference of the tunnel along its entire length prior to the installation of the steel pipe.

Contact grouting will be executed between the excavated rock surface and the backfill concrete around top portion of the tunnel because the gradient of the tunnel is almost level (5.88%).

Concrete anchor blocks will be constructed at the corners and bifurcation of the steel pipe in the open-air section. To avoid harmful impact on the main pressure containing part of steel pipe, concrete saddles will be placed at a proper interval. Details of the anchor block and saddle will be designed during the detailed design stage.

The internal pressure applied to the penstock design is assumed as the sum of the static water pressure and the rising pressure due to water hammer or surging.

The analysis on water hammer and surging was carried out under the condition to cause possible maximum water pressure as shown in **Table 9.3.3-3**. The result of the analysis is shown in **Figure 9.3.3-2** and **Figure 9.3.3-3**.

Table 9.3.3-3 Condition for Water Hammer Analysis

Item		Unit	Down surge
Initial discharge	Q_1	m^3/s	140 (70×2 units)
		%	100
Final discharge	Q_2	m^3/s	0 (0×2 units)
		%	0
Time	T	s	5
Reservoir water level	EL	m	438 (FSL)

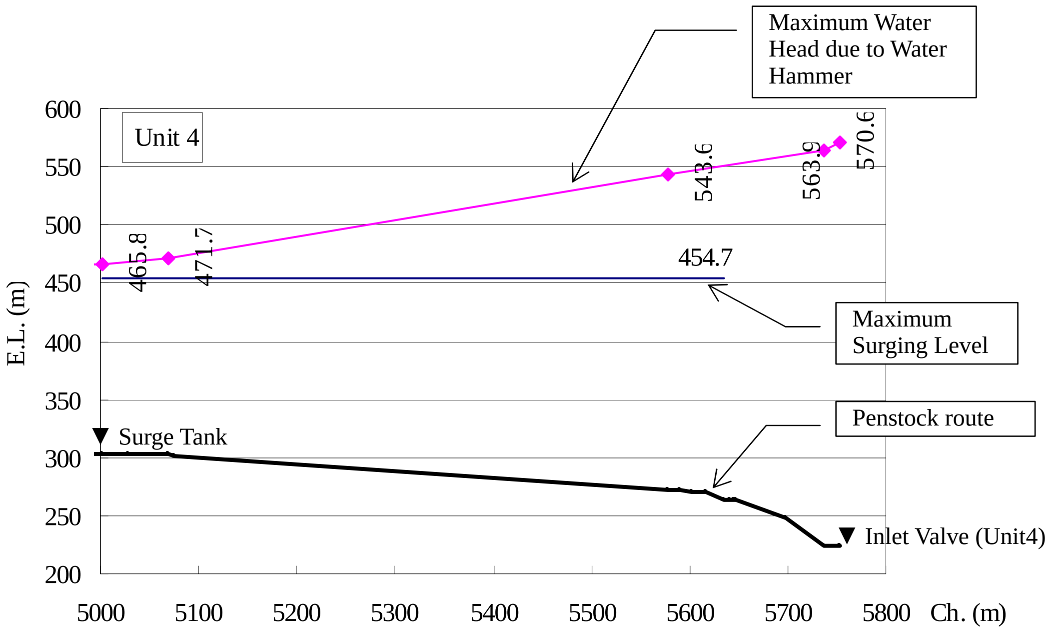


Figure 9.3.3-2 Water Head due to Water Hammer (Unit 4)

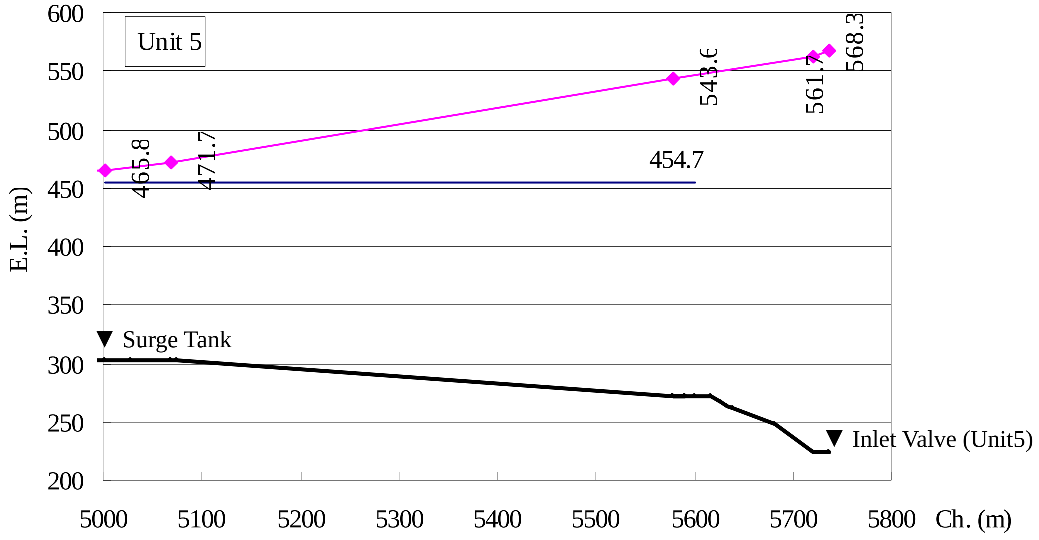


Figure 9.3.3-3 Water Head due to Water Hammer (Unit 5)

The external pressure works on the penstock in the tunnel section is assumed as 0.6 MPa caused by the contact grout pressure.

Typical sections of the penstock in the tunnel section are shown in **Figure 9.3.3-4**. Plan and profiles of the penstock in the open-air section is shown in **Figure 9.3.3-5** and **Figure 9.3.3-6**.

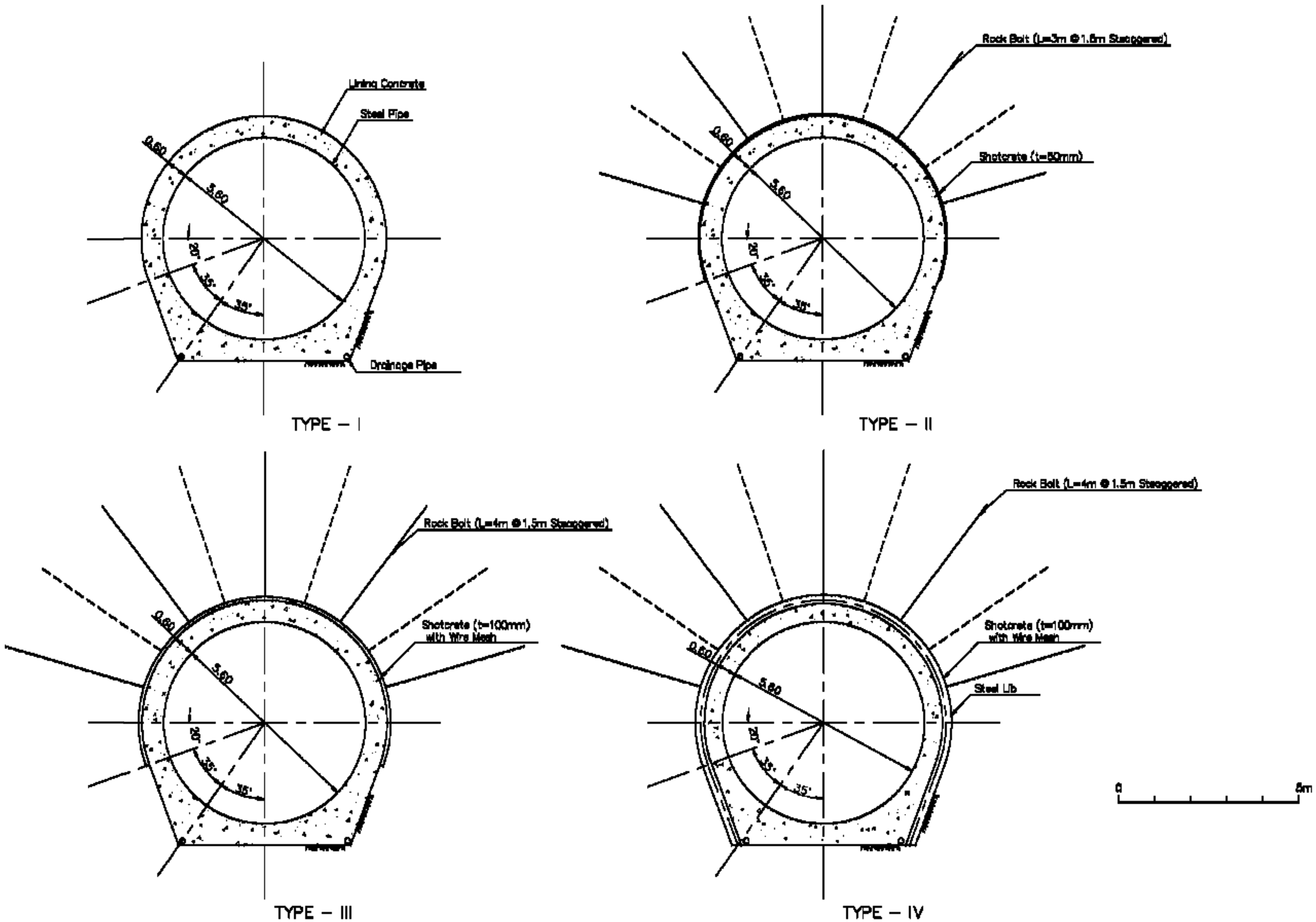


Figure 9.3.3-4 Penstock Typical Section (Tunnel)

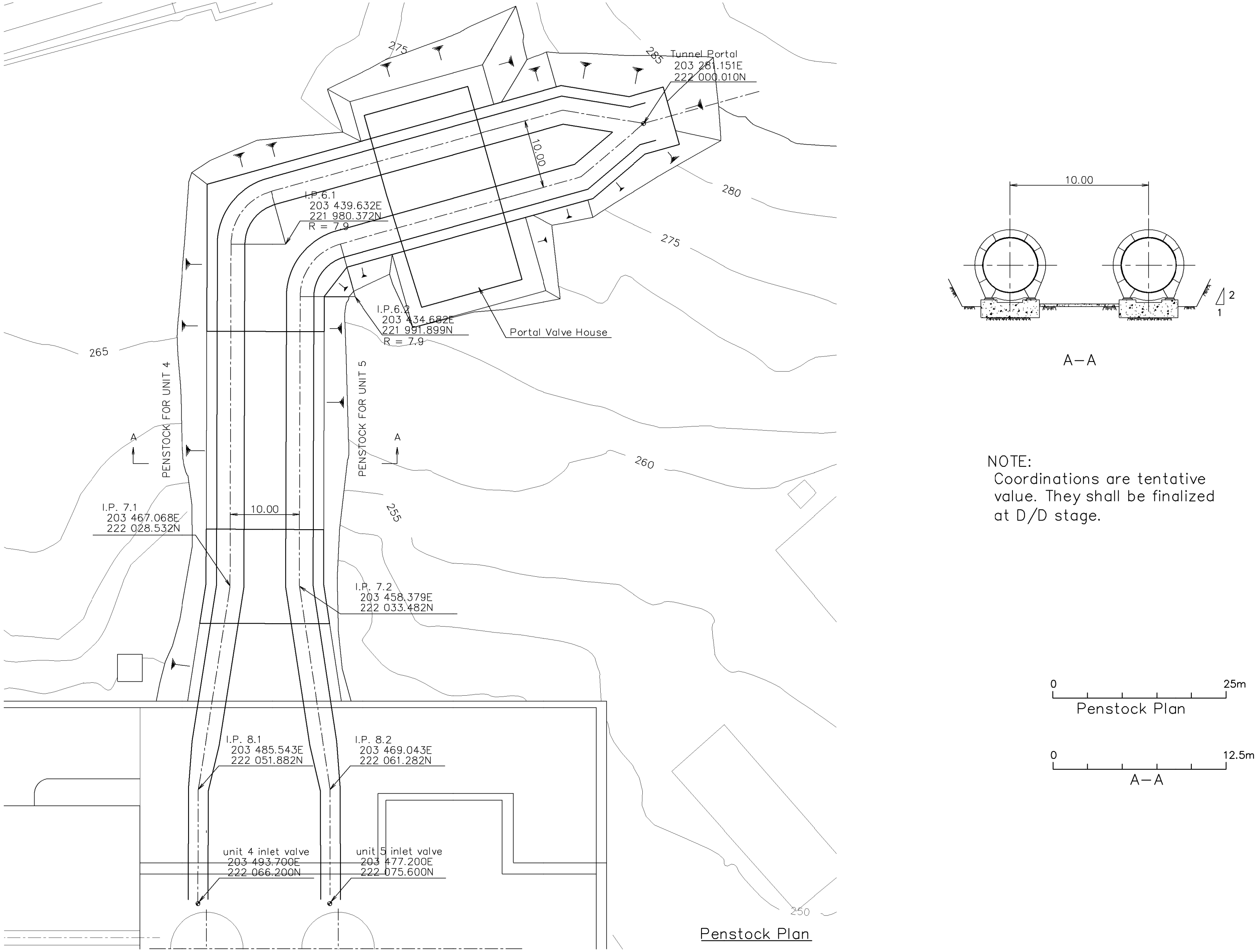


Figure 9.3.3-5 Penstock Plan and Section (Open-Air)

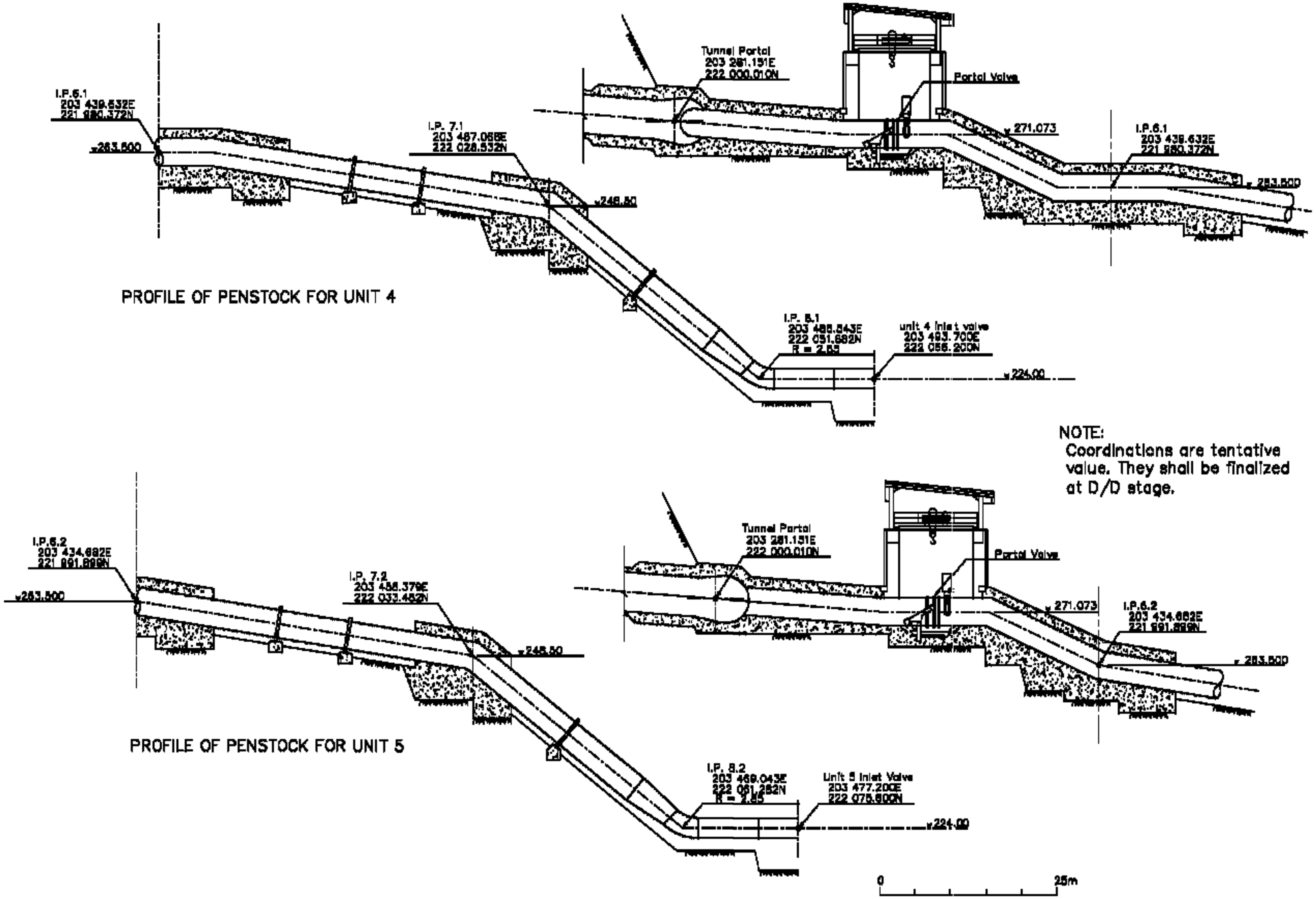


Figure 9.3.3-6 Penstock Profile (Open-Air)

9.3.4 Surge Tank

The new surge tank will be constructed north of the existing surge tank in the open space as shown in **Figure 9.3.4-1** since this is the only available open space around this area. Restricted orifice type, which is of the same type as the existing surge tank, is applied to the new surge tank, to reduce the volume of the surge tank and to secure the damping performance against surging.

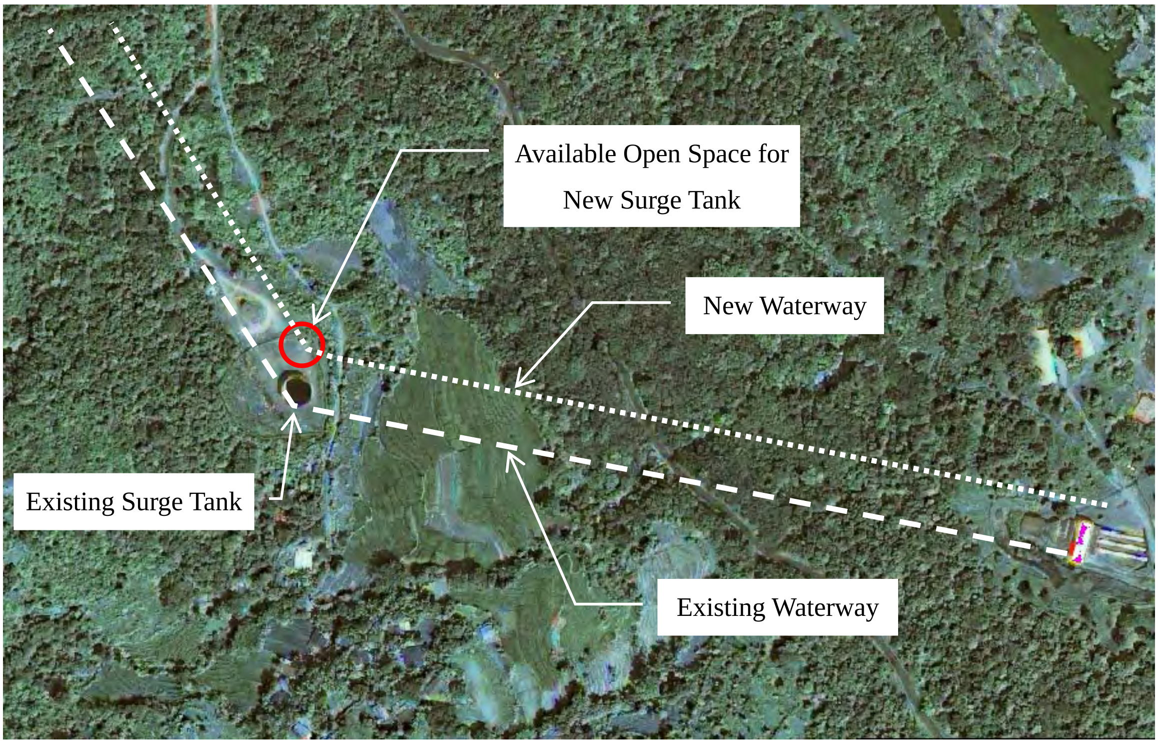


Figure 9.3.4-1 Available Open Space for New Surge Tank

Dimensions of the new surge tank are set with reference to the existing surge tank, and the results of the surging analysis, which was carried out under the condition as shown in **Table 9.3.4-1**, satisfies that the maximum surging level is lower than the top of the surge tank and the minimum surging level is higher than the top of the headrace.

Time series of the water level in the surge tank on the up surge condition and on the down surge condition are shown in **Figure 9.3.4-2** to **Figure 9.3.4-5**.

Table 9.3.4-1 Conditions and Result of Surging Analysis

Item		Unit	Up surge			Down surge
Initial discharge	Q_1	m^3/s	140 (70×2 units)	133 (66.5×2 units)	126 (63×2 units)	70 (35×2 units)
		%	100	95	90	50
Final discharge	Q_2	m^3/s	0 (0×2 units)			140 (100%) (70×2 units)
		%	0			100
Time	T	s	5			5
Reservoir water level	EL	m	438 (FSL)			370 (MOL)
Coefficient of Manning's roughness		$m^{-1/3}s$	Concrete: 0.0115 Steel: 0.011			Concrete: 0.0145 Steel: 0.013
Maximum water level	EL	m	454.7	454.3	453.9	367.7
Minimum water level	EL	m	430.3	430.3	430.4	354.9

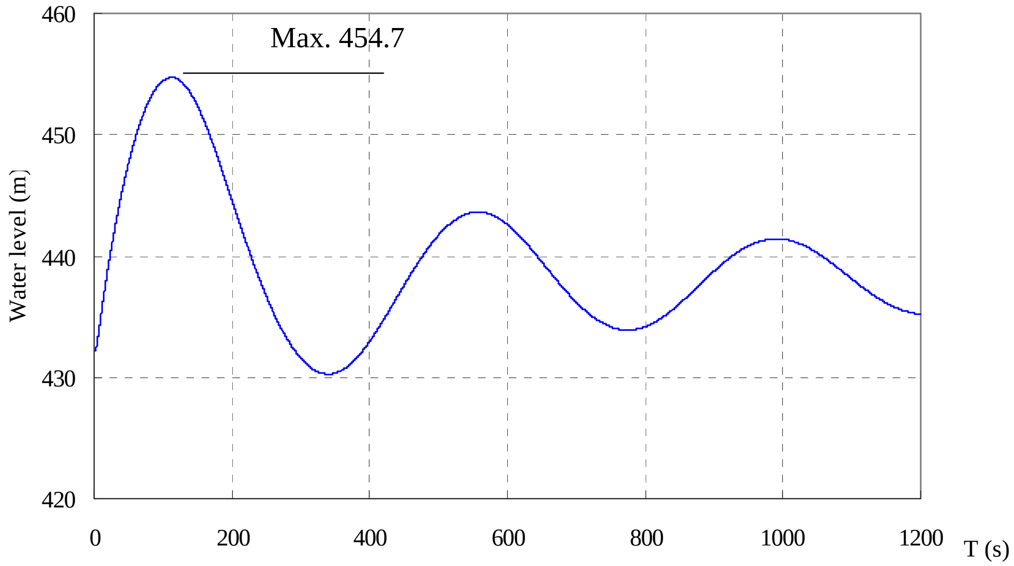


Figure 9.3.4-2 Water Level in the Surge Tank at Up Surge
(Q_1 140m^3/s → Q_2 0m^3/s; T = 5s)

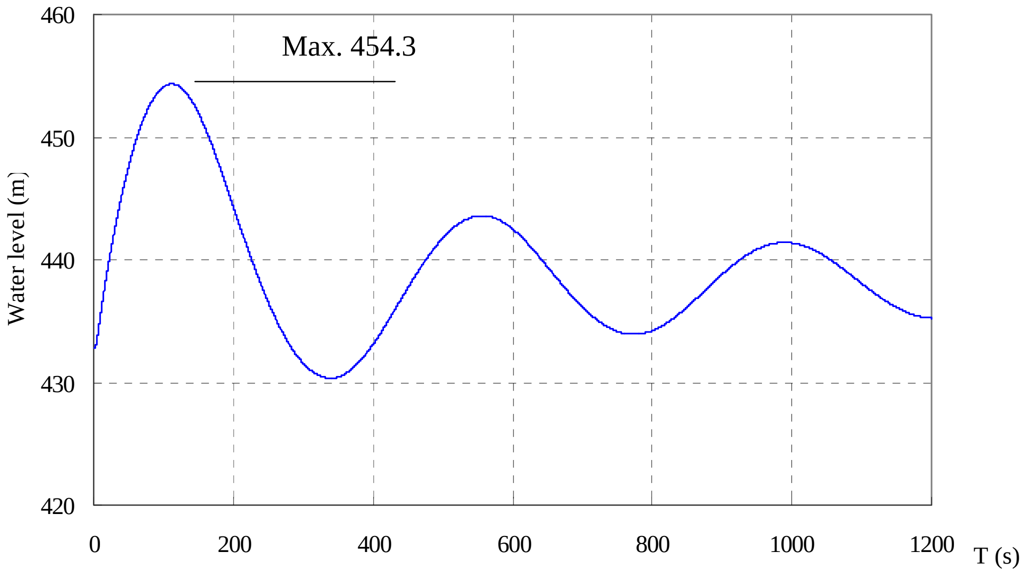


Figure 9.3.4-3 Water Level in the Surge Tank at Up Surge
(Q_1 133m^3/s → Q_2 0m^3/s; T = 5s)

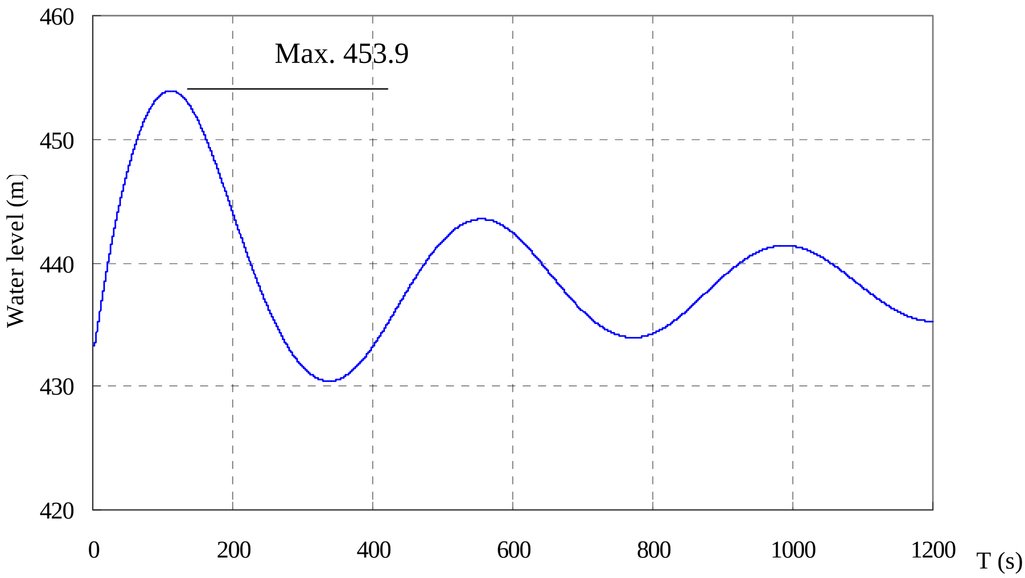


Figure 9.3.4-4 Water Level in the Surge Tank at Up Surge
(Q_1 126m^3/s → Q_2 0m^3/s; T = 5s)

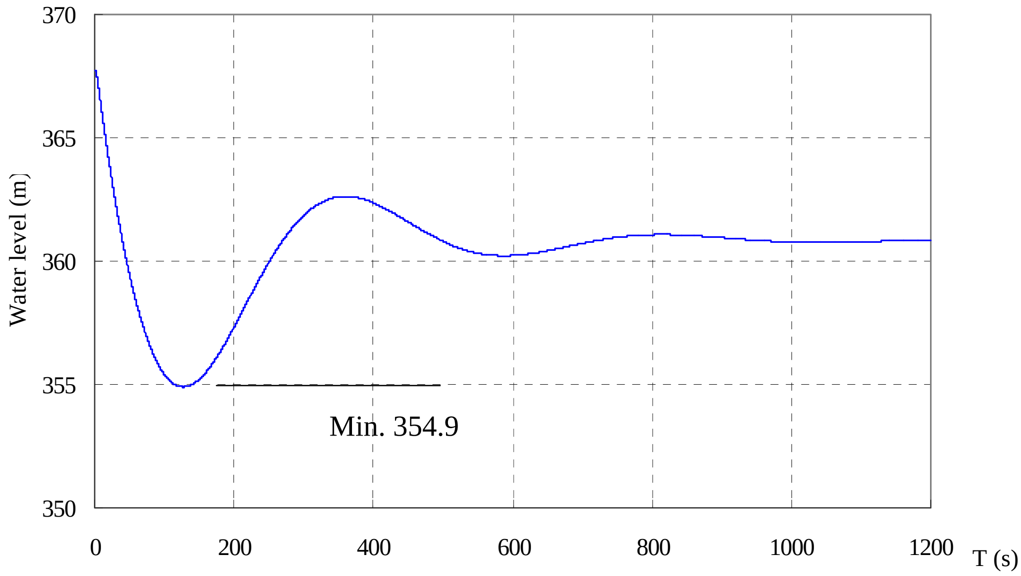


Figure 9.3.4-5 Water Level in the Surge Tank at Down Surge
(Q_1 70m^3/s → Q_2 140m^3/s; T = 5s)

The new surge tank satisfies following Thoma-Schuller's condition of stability, therefore the new surge tank has damping function to make the small water level fluctuation during power operation stable and restore the fluctuation to the balanced condition. And this surge tank also has damping function to prevent the oscillation of the water level from exceeding the designed water level under the condition of overlapped surging.

Thoma-Schuller's condition of stability;

$$h_0 < \frac{H_g}{3} \sim \frac{H_g}{6}$$

Where,

h_0 : Total head loss .... 13.8 m

H_g : Total head.. 206.0 m

→ h_0 < 68.7~34.3 ....OK

$$F > \frac{Lf}{c(1+\eta)gH_g} \sim \frac{Lf}{c(1+\eta)(1+\frac{1-\eta}{1-\eta})g(H_g-z_m)}$$

Where,

F : Area of surge tank.. 314 m^2

L : Length of headrace.. 5,017 m

f : Area of headrace .... 34.2 m^2

c : Coefficient of total head loss ($h_0 = cv^2$, v: velocity in the headrace = Q/f)................ 0.8

k_0 : $1/(2g)(Q_0/(C_dF_p))^2$.. 140 m

η : k_0/h_0.. 2

z_m : High water level.. 22.5 m

→ F > 34.2~58.1 ....OK

In general, a horizontal tunnel section is located 20 m upstream and 20 m downstream of the surge tank to secure workability for its construction.

The horizontal tunnel section will be constructed for the new surge tank 20 m upstream and 67 m downstream of the new surge tank since the new surge tank is located on the bend section of I.P.4. Steel liners will be installed in this horizontal section.

Vertical and horizontal sections of the new surge tank are shown in **Figure 9.3.4-6**.

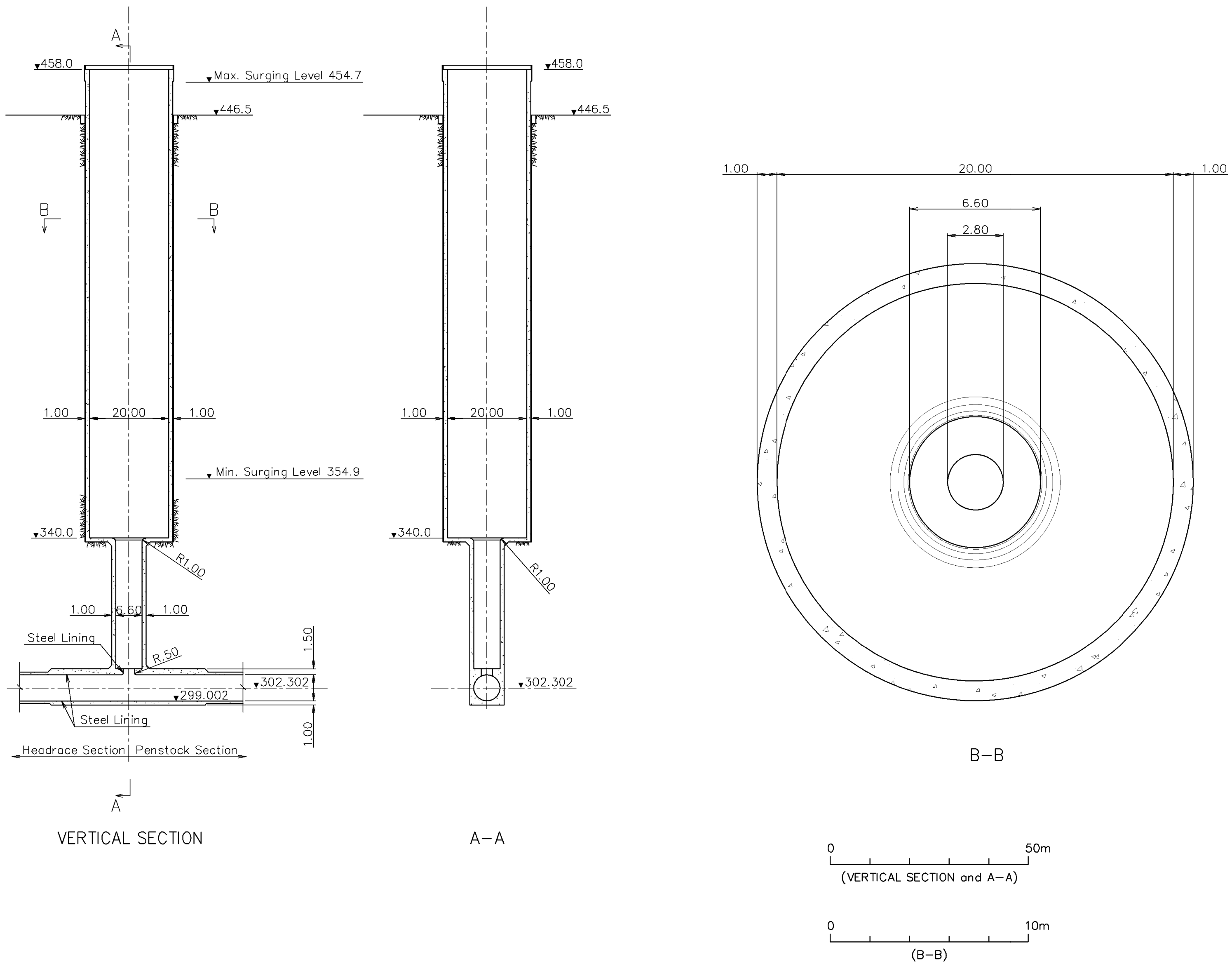


Figure 9.3.4-6 Surge Tank Vertical and Cross Section

[REFERENCE]

Generally, design of a surge tank is carried out under the possible most critical surging condition like the condition in **Table 9.3.4-1**.

In this section, the down surge caused by sudden load rise from 0% to100% as shown in **Table 9.3.4-2** was calculated for reference purpose. Oscillation of water level in the surge tank is shown in **Figure 9.3.4-7**.

However, it is impossible to operate the generator in this pattern for the power station like Victoria Hydropower Station which is connected to the power grid.

Table 9.3.4-2 Conditions and Result of Reference Surging Analysis

Item		Unit	Down surge
Initial discharge	Q_1	m^3/s	0 (0×2 units)
		%	0
Final discharge	Q_2	m^3/s	140 (100%) (70×2 units)
		%	100
Time	T	S	5
Reservoir water level	EL	m	370 (MOL)
Coefficient of Manning's roughness		$m^{-1/3}s$	Concrete: 0.0145 Steel: 0.013
Maximum water level	EL	m	370.0
Minimum water level	EL	m	350.4

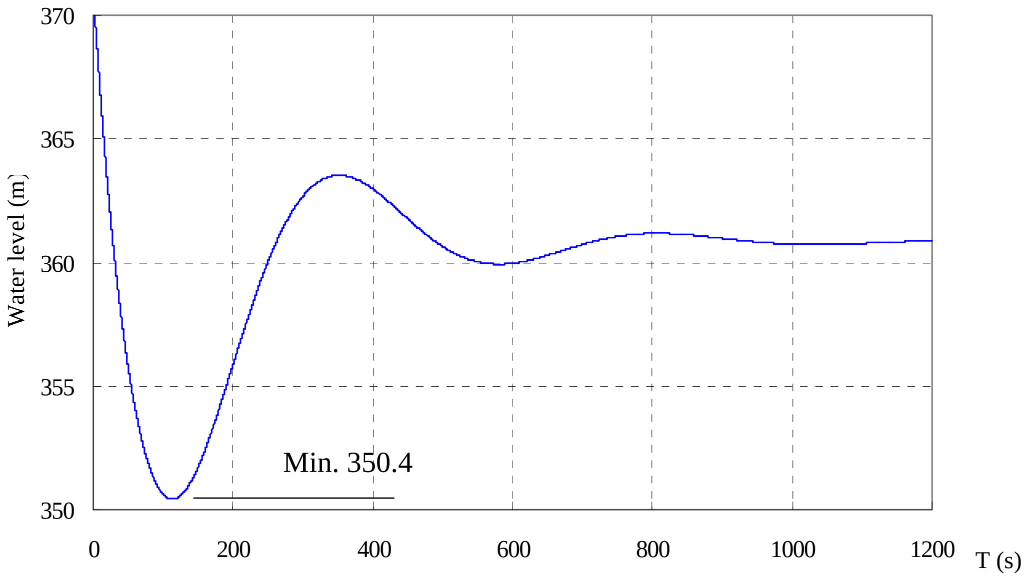


Figure 9.3.4-7 Water Level in the Surge Tank at Down Surge
(Q_1 0m^3/s → Q_2 140m^3/s; T = 5s)

9.3.5 After-bay

The new after-bay consists of as the guide wall and the over-flow weir. Since the maximum discharge of the new power station is the same as that of the existing power station, the dimension of the new after-bay is set at the same dimension as the existing after-bay.

As to the weir, in case of lowering the elevation of the top of weir from EL. 230 m, it is necessary to excavate the riverbed in downstream of the weir. And if the existing power station is operated, the discharged water from the existing power station will flow back to the new after-bay. Hence, the top elevation of the new weir is set on the same elevation of the top of the existing weir of EL.230 m.

Plan and section of the new after-bay is shown in **Drawing 014** and **Draewng 015** of **9.8**.

For reference, the difference of water levels of Mahaweli River in around 1 km section from the new powerhouse to Randenigala Reservoir before and after the expansion is shown in Appendix II.

9.3.6 Access Adit Plug Concrete

The existing access adit has 400 m in length with wagon shape of one circular arc (7.2 m × 7.2 m). This access adit will be used for the Project.

In Sri Lanka, it is common to install an access manhole by using an adit constructed on the middle way of the headrace tunnel for future maintenance works, hence the access manhole with 2 m in inner diameter will be installed in plug concrete of the access adit, to make it possible to carry in large-size machines in the new headrace tunnel.

The plug concrete is designed to resist the internal pressure of the headrace with its shearing force of the bottom face. The length of the plug concrete (L) is set as follow.

$$L = n \frac{P \bullet A}{\tau l}$$

Where,

P : Static water pressure
(= F.S.L. (438.0 m) – Sill E.L. at Plug concrete (345.0 m) = 93 tf/m^2)

A : Sectional area exposed to static water (46.28 m^2)

τ : Shearing strength (70 tf/m^2)

l : Bottom width of tunnel section (7.2 m)

n : Safety factor (4)

→ $L = 15.94 \text{ m} \fallingdotseq 20 \text{ m}$

Plan and profile of the access adit plug concrete is shown in **Figure 9.3.6-1**.

Details of the access manhole will be determined during the detailed design stage.

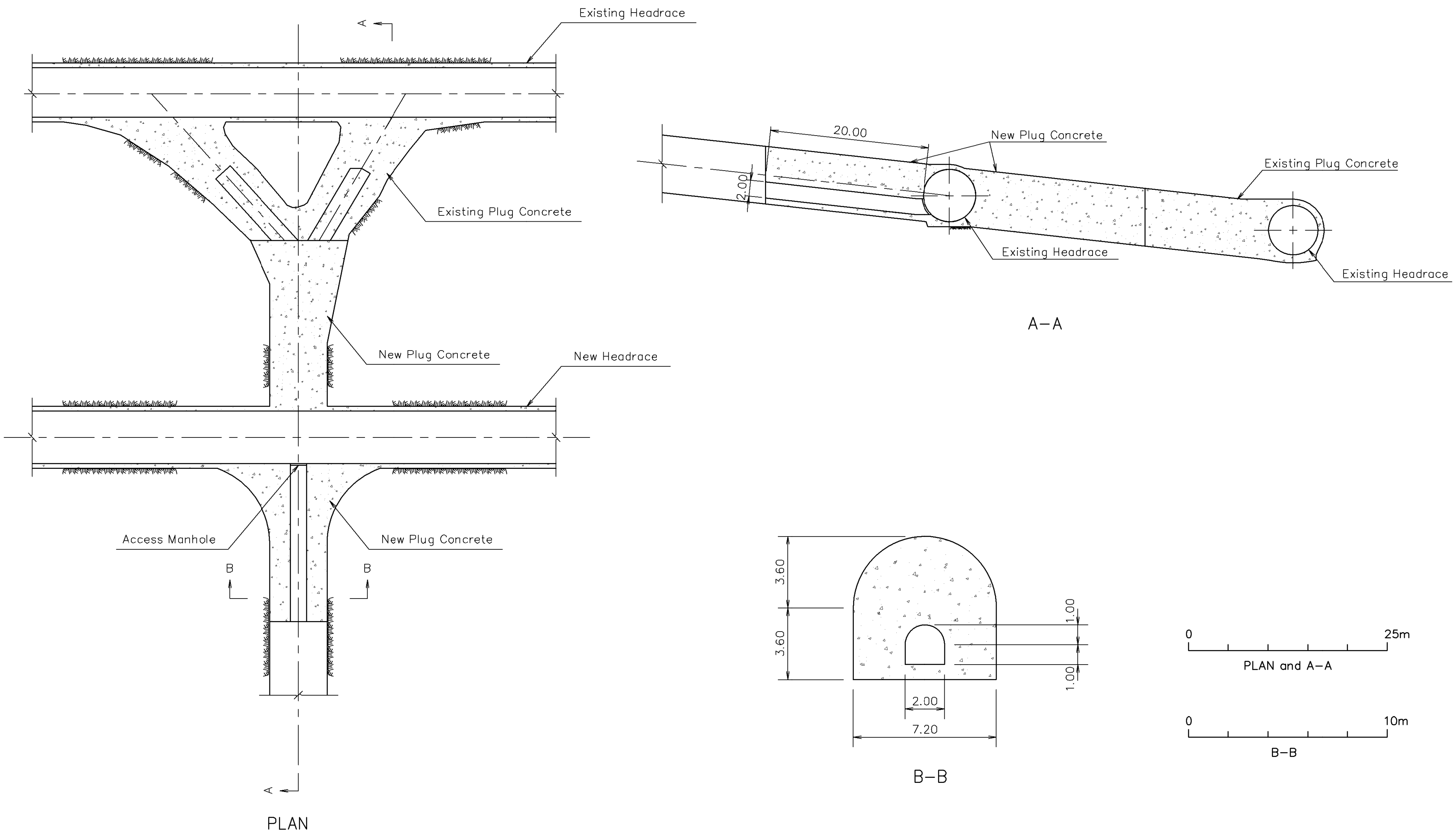


Figure 9.3.6-1 Access Adit Plug Concrete Plan and Section

9.4 Hydromechanical Equipment

9.4.1 Steel Penstock

Steel penstock will be installed from downstream of the surge tank to the inlet valve. The penstock of 575 m long from the surge tank is a single line tunnel section. The steel penstock bifurcates at the tunnel portal and reaches the inlet valves 200 m ahead of the bifurcation in an open-air section. A portal valve will be installed downstream of the bifurcation. The inner diameter of the steel penstock is 6.6 m at the surge tank and contracted to 5.6 m through the tunnel section. Downstream of the bifurcation, the diameter becomes 3.95 m in each line. At the end of the inlet valve side, the diameter is contracted to 2.85 m to connect the inlet valve.

Design internal pressure is estimated approximately at 1.6 MPa at the surge tank side and approximately as 3.4 MPa at the inlet valve end. Design external pressure for the steel penstock in the tunnel section is assumed as 0.6 MPa due to contact grouting pressure.

According to the records on the previous construction works, the rock condition in the tunnel section is expected to be good, therefore the rock is expected to bear 20% of the internal pressure of the steel penstock in the section 100 m upstream of the tunnel portal to the surge tank with enough depth of rock cover.

With the above-mentioned conditions, SM570Q in JIS G 3106 is applied to the material of the steel penstock. The thickness of the steel plate is calculated as 19 mm to 34 mm in the tunnel section and as 23 mm to 30 mm in the open-air section.

Stiffening plates will be attached to the steel penstock in the section where the rock is expected to bear the load.

The existing trifurcation is of external reinforced type, but the new bifurcation is determined as internal reinforced Y-shape type to reduce the head loss.

9.4.2 Portal Valve

It is common in Sri Lanka to install a portal valve immediately downstream of the bifurcation of an open-air section of penstock to deal with unexpected contingencies like malfunction of inlet valve. In the Project, in accordance with the above, portal valves are to be installed immediately downstream of the bifurcation in the open-air section.

Expected usage of the portal valve is i) alternative function of the inlet valve during its accidents, and ii) maintenance use with one-unit operation such as change.

9.4.3 Outlet Gate

Outlet gates will be installed between the outlet of the draft tube and the after-bay for maintenance use. Main features of the gate are of slide gate type with 3.7 m in effective width, 3.7 m in effective height and 23.7 m of design water head. Two leaves of gates will be furnished for one draft tube.

Gate slots will be installed in the both sides of piers of the draft tube. Opening and closing operation will be done by the existing gantry crane. Rail tracks for the existing gantry crane will be extended to the new outlet gate location for common use of the existing and new gates.

9.4.4 Access Manhole

The access manhole with inner diameter of 2 m will be installed in plug concrete of the access adit for future maintenance works. The access manhole consists of 10 m long steel liner with 2 m diameter and bulkhead attached by the bolts on flange at the junction of access adit and headrace.

The bulkhead will be opened and closed with a hoist crane suspending from a H-shape steel beam attached to the top of the access adit. As mentioned in **9.3.6**, details of the access manhole will be designed during the detailed design stage.

9.5 Power Station

9.5.1 Civil Structures

(1) Layout of Power Station

As the result of the comparison study on the three alternative options, it becomes obvious that the basic option in which the new powerhouse is to be constructed next to the existing powerhouse is the optimal from the economical and environmental points of view. The new powerhouse will be constructed by unifying it with the existing structure for the convenience of operation and maintenance works.

(2) Salient Feature of Civil Structure

The dimensions of the new powerhouse are determined based on the conditions of electromechanical equipment such as the turbine, generator, overhead traveling crane and so on. The basic design for electromechanical equipment is examined in **9.5.2**.

The structural calculations have not yet been executed as mentioned in **9.1**.

1) Height

a) The elevation of the turbine center is lowered by 4 m from the turbine center of the existing unit due to the increase in draft head.
b) Lifting height of the overhead traveling crane becomes 1 m higher than that of the existing crane due to heavier equipment.
c) Height of the new powerhouse from bottom of the draft to the top of crane becomes 32.7 m which is 5.7 m higher than the existing powerhouse due to the increase in height of the draft tube, turbine and generator.

2) Width (Upstream-Downstream)

The width of the powerhouse is determined as 37 m based on the sizes of the turbine, generator, inlet valve and necessary spaces for the auxiliary equipment. The crane span is 17 m which is 1.7 m wider than the existing due to the larger size and unit capacity.

3) Length

The length of the powerhouse is 69 m based on the necessary distance of each unit and a space for the erection bay. The existing overhead traveling crane and erection bay are not available for the new powerhouse because the crane span is wider than that of the existing crane.

4) Elevation of Each Floor

The elevation of the erection bay is designed at EL.242.00 at the same elevation as the existing, in consideration of the carry-in route for equipment and materials. The new and the existing powerhouses are connected through the tunnel at generator floor (EL.230.25).

The flow chart of the powerhouse design is shown in **Figure 9.5.1-1**.

The comparison of both new and existing powerhouses is shown in **Figure 9.5.1-2** and **Figure 9.5.1-3**.

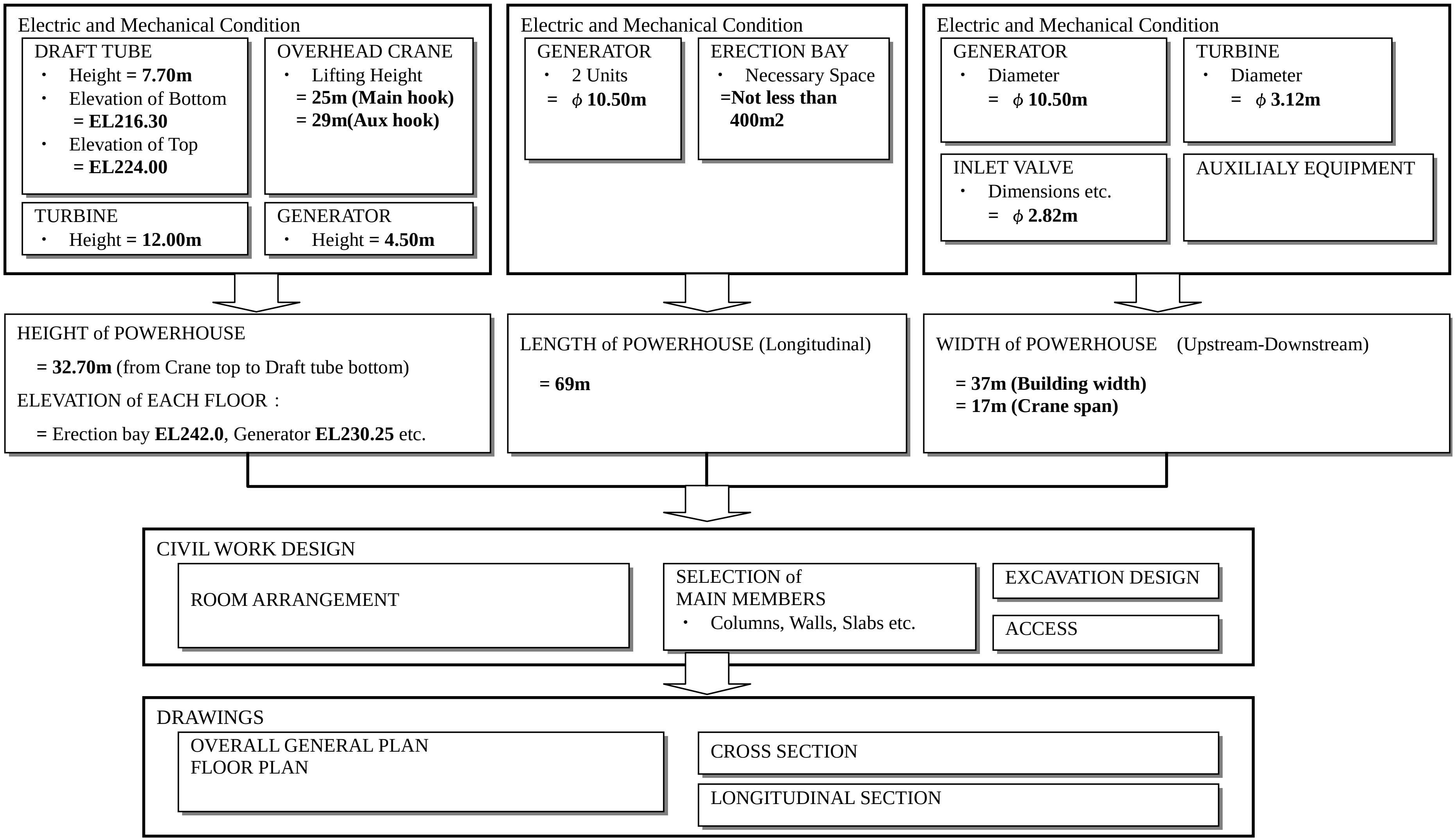


Figure 9.5.1-1 Design Flow

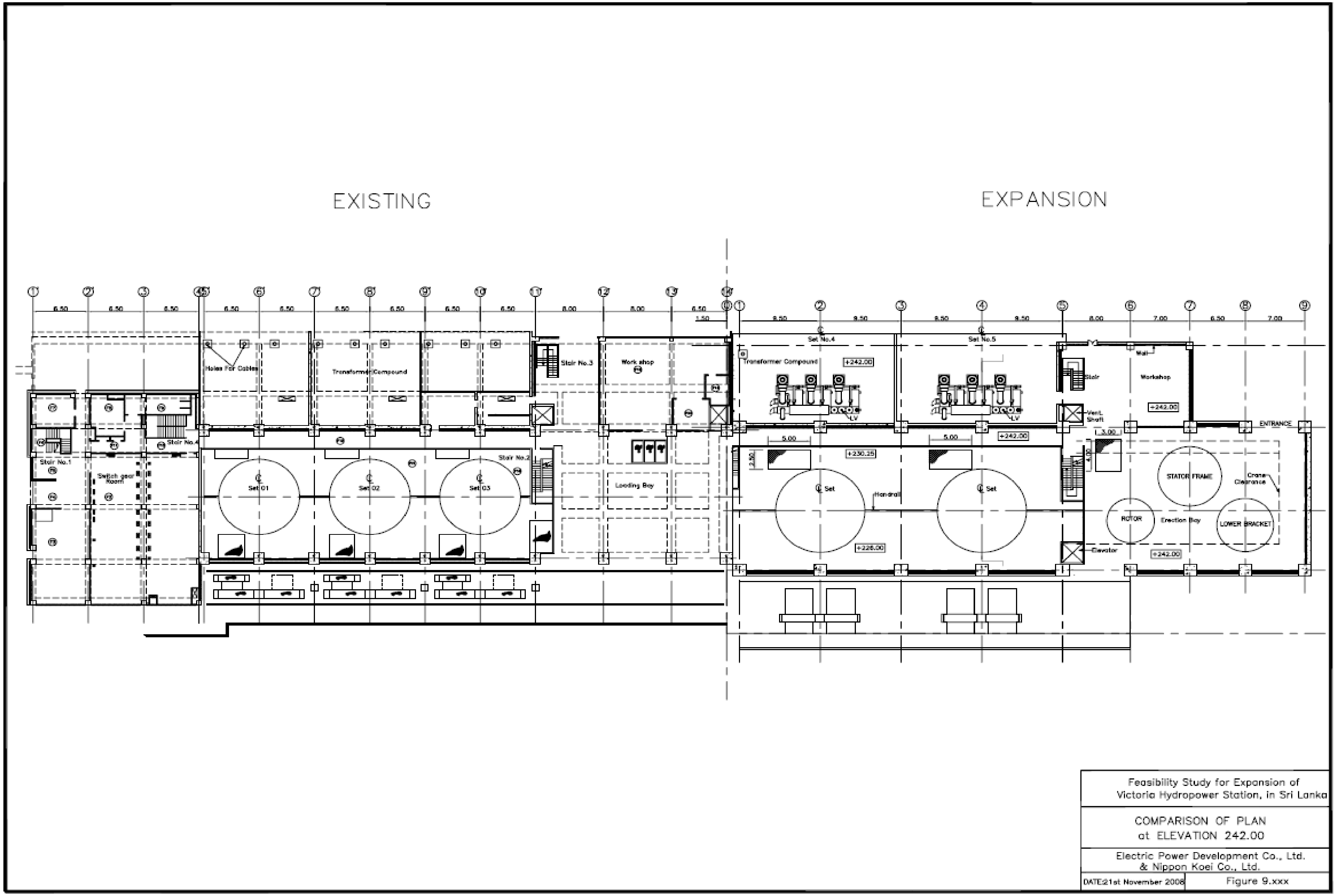


Figure 9.5.1-2 Plan of the Existing and Expansion Powerhouse

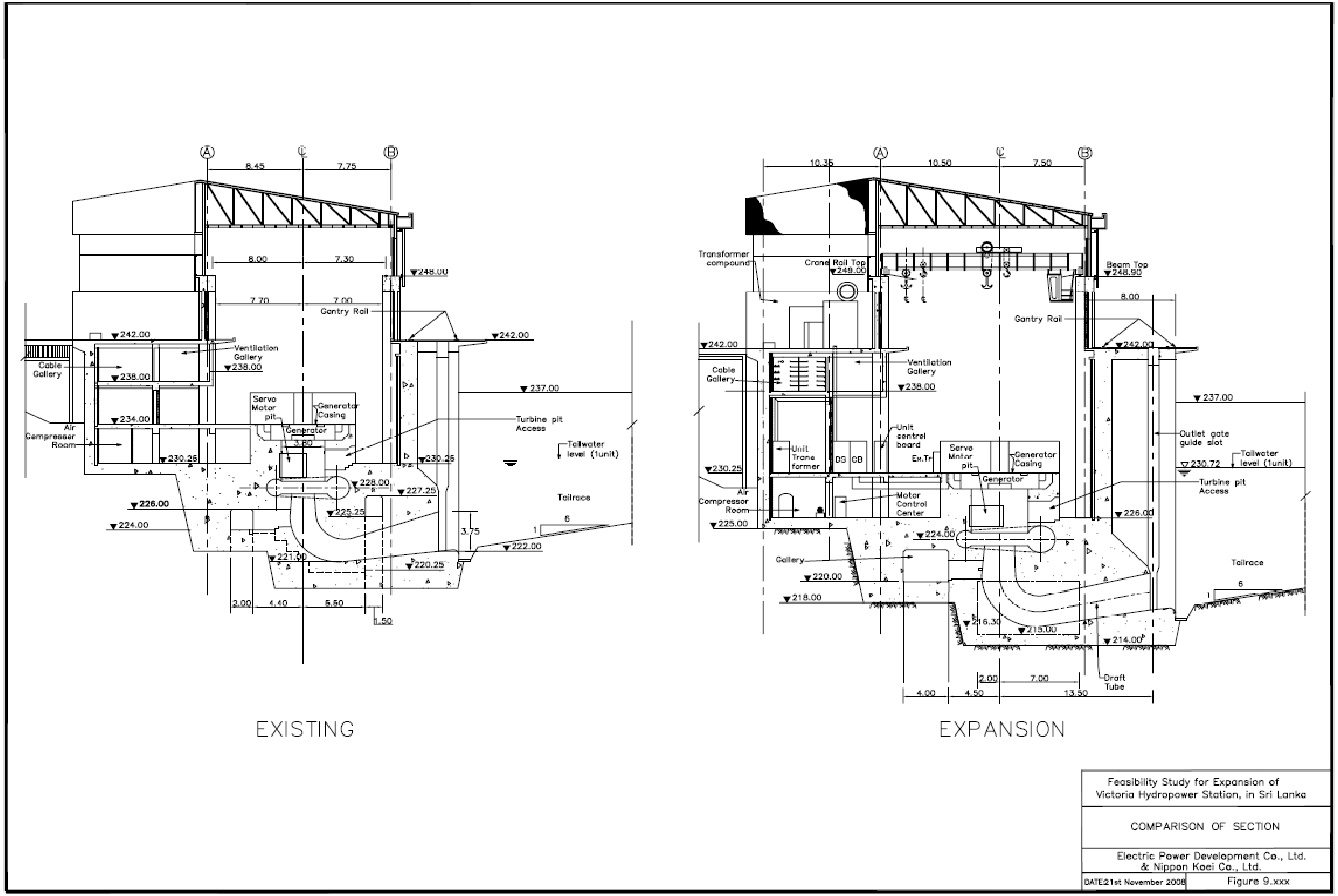


Figure 9.5.1-3 Profile of the Existing and Expansion Powerhouse

(3) Layout of Electromechanical Equipment

Floor elevations of main electromechanical equipment are shown in **Table 9.5.1-1**.

Table 9.5.1-1 Floor Arrangements

Main Equipment	Floor Elevation
OHT Crane	EL 249.00
Ventilation Plant	EL 249.00
Transformer	EL 242.00
Erection Bay	EL 242.00
Work Shop	EL 242.00
Cable Gallery	EL 238.00
Ventilation Gallery	EL 238.00
Transformer Oil Water Separation Pit	EL 238.00
Storage Area	EL 238.00
Battery	EL 230.25
AC/DC Control Board	EL 230.25
Unit Control Board	EL 230.25
Low Voltage Cub.	EL 230.25
Air Compressor Room	EL 226.00
Oil Treatment Room	EL 226.00
Motor Control Center	EL 226.00
Governor Oil Pressure Tank	EL 226.00
Turbine Control Board	EL 226.00
Fire Distinguish System	EL 226.00
Governor Cabinet	EL 226.00
G.V.Servo Motor	EL 226.00
Inlet Valve	EL 220.00
Inlet Valve Control Board	EL 220.00
Inlet Valve Oil Pump	EL 220.00
Drainage Pump	EL 220.00
Drainage Pit	EL 220.00

9.5.2 Electromechanical Equipment

(1) General

Main expansion units are composed of 2 units of the hydraulic turbine with rated output of 122 MW at effective head of 191.50 m (at 1-unit operation) and turbine discharge of 70 m^3/s/unit, and the generator with rated capacity of 140 MVA.

The turbines, generators, auxiliary equipment such as cooling water and drainage system, 16.5 kV indoor switchgear equipment such as parallel-in circuit breaker, operation control system and overhead traveling crane are to be installed in the powerhouse. Main transformer will be installed outdoor and connected to the generator through the parallel-in circuit breaker and disconnecting switches. The generator voltage is stepped up from 16.5 kV to 220 kV by the main transformer.

Generated power is transferred to 220 kV outdoor switchyard through 220 kV power cables and to demand sides by the 2-circuit Kotmale and 1-circuit Randenigala transmission lines.

(2) Hydraulic Turbine

1) Turbine Output

Turbine output is calculated with an effective head and water discharge as follows:

$$
\begin{aligned}
Pt &= g \times Q \times H \times \eta t \\
&= 9.8 \times 70 \times 191.5 \times 0.929 \\
&= 122{,}041 \text{ kW} \\
&\fallingdotseq 122{,}000 \text{ kW}
\end{aligned}
$$

Where,

Pt : turbine output (kW)
g : gravity accelerated g ≒ 9.8 m/sec^2
Q : rated discharge (m^3/s)
H : effective head (m)
η : turbine efficiency (estimated value from actual result)

2) Type

Type of the hydraulic turbine is defined by the effective head and water discharge. In general, in case of high effective head, Pelton or Francis type is selected, while in case of low effective head, Kaplan or Tubular with variable blade type for high efficiency operation is selected.

In case of Francis type, applied effective head range is from about 50 to 500 m.

According to the theory of similarity rule, when the specific speed is the same, the turbines have the same characteristics. In case of Francis turbine type, applied ranges of the specific speed are from 70 to 350 m-kW.

The specific speed has limitation empirically to applying each turbine type, and is calculated with the following formula which is standardized by JEC-4001 issued by Japan Electro-technical Committee (JEC). However, it is not so strict, and a design over the value might be adopted upon an economical reason.

$$Nslimit\ (132) \leq 35 + 21{,}000/(H+25)$$

Where,

Nslimit : limited specific speed (m-kW)
H : effective head (m) 191.5 m

According to the above calculation, the specific speed of this plant is estimated at 132(m-kW), and this value is in Francis type selection zone.

Therefore, in consideration of the applying head, specific speed, experiences and examples applied to similar projects, Vertical Francis turbine is selected as the most appropriate type of the turbine.

3) Rated Speed

Based on the equation standardized by JEC-4001 and by using the limited specific speed as a target, revolving speed is calculated, and then the rated speed is determined in consideration of the number of poles with system frequency of 50Hz and economic validity as follows:

$$N = Nslimit \times H^{5/4}/P^{1/2}$$
$$= 132 \times 191.5^{5/4} / 122{,}041^{0.5} = 269\ min^{-1}$$

Therefore, as the nearest value, 300 min^{-1} is adopted based on the **Table 9.5.2-1.**

Table 9.5.2-1   Revolving Speed

Number of Pole	50Hz	60Hz	Number of Pole	50Hz	60Hz
6	1000	1200	28	214	257
8	750	900	32	188	225
10	600	720	36	167	200
12	500	600	40	150	180
14	429	514	48	125	150
16	375	450	56	107	129
18	333	400	64	94	113
20	300	360	72	83	100
24	250	300	80	75	90

Source:JEC-4001: Turbine and Pump-Turbine

4) Runaway Speed

Generally runaway speed of Francis type is estimated 1.6 to 2.2 times of the rated speed.

For specific speed recalculated as ns = 147 (m-kW), 1.83 times of the rated speed is estimated. Hence, under the conditions with the rated speed (n) = 300 min^{-1} and effective head (Hnor) = 191.5 m, the runaway speed (nr) is calculated as follows:

$$nr = 300 \times 1.83 = 549\ (min^{-1})$$

If the operation head (H) is higher than the normal head (Hnor), the runaway speed will be bigger in proportion to square root of the effective head.

Therefore, the maximum runaway speed (n_o) at the maximum effective head (200 m) is estimated as follows:

$$n_o = 549 \times (200/191.5)^{1/2}$$
$$= 561 (min^{-1})$$

5) Fly Wheel Effect (GD^2)

In the case of the load rejection, it is better that input of the runner should be decreased instantaneously, but actually for prevention of high pressure rise and for absorbing of energy of the penstock, adjustment of guide vane closing time and flywheel effect of generator are considered.

The generator originally has inherent flywheel effect, "Inherent GD^2", and it is calculated as follows:

Inherent $GD^2 = 0.6 \times Pg^{1.25}/n^{1.98} \times 10^6$

Where,

Pg : rated capacity (kVA)

n : rated speed (min^{-1})

If the required GD^2 of turbine side is not over Inherent GD^2 of the generator, it is not necessary to take countermeasure any more.

Required GD^2 is calculated with the following formula:

$GD^2 = 364 \times P \times (1+\Delta h'/2)^{3/2} \times (\tau+t/2)/(n2\times\Delta n) \times (n'r-\Delta n^2)/n'r \times K$(t-m^2)

Where,

nr : runaway speed

n'r = (nr-n)/n

t : closing time of GV

τ : dead time of governor

K : coefficient (= 0.85 for Francis turbine)

Δn : momentary speed variation

Δn is calculated with the following formula:

$\Delta n = (Nm-Nl) / n$: (maximum speed – before change the speed) / rated speed

Where,

$Nm = n \times (\Delta n + 1)$

Δn is set as 0.3, 0.35 and 0.4, and the required GD^2 is checked, respectively.

As the result, required GD^2 is estimated 3,800 (t-m^2) under the condition of 5 seconds of closing time of GV and Δn = 0.4.

Therefore necessary GD^2 is estimated as not less than 3,800 (t-m^2), and Δn is estimated as less than 0.4.

6) Speed Governor

The speed governor is of digital PID type. As component of the governor, actuator less type, which is the controller directly positioned on the distributor for keeping high performance, is adopted.

7) Runner

Diameter of the runner is the basic dimension of all design of a hydraulic turbine and calculated based on the characteristic data such as the specific speed, effective head, discharge and rotation speed by using developed model turbines by the manufacturers and/or power companies.

According to the actual data on similar hydropower projects, the maximum diameter and the weight of this turbine is estimated at 3,140 mm and 21 t, respectively. Different figures might be proposed by the contractor’s own experiences, therefore if more advantageous data from the viewpoint of economical reason, performance and quality which is suggested by the contractor, it is worthy to be reconsidered.

Stainless steel anti-corrosion type such as 13 chrome high nickels stainless steel is recommended to be applied to material of the runner in consideration of cavitations.

8) Turbine Center

The turbine center has to be low enough to prevent the runner from cavitations. The appropriate draft head of the turbine is calculated in consideration of tailrace water level, cavitations coefficient and vapor pressure.

In this plant, the draft head is defined as -8.0 m mainly based on the cavitations coefficient of 0.093 which is the standard value recommended by the Institute of Electrical Engineers of Japan.

In consideration of a runner height of 0.85 m (distance between the lower end of the runner and the center of the runner), and the tail race water level of EL.230.72 m which is under 1-unit rated operation, the center of the runner is calculated as EL.224 m as follows:

$$230.72 - 8.0 + 0.85 = 223.57 \fallingdotseq \text{EL.224 m}$$

9) Rotation Direction

Anti-clockwise rotation direction of the turbine-generator is adopted. This direction is the same as that of the existing power plant.

It is noted that there is no standard to determine rotation direction. Generally the rotation direction of a turbine-generator is decided based on relative locations between penstock route and available space for the turbine-generator in a powerhouse for saving spaces of the powerhouse.

(3) Inlet Valve

By-plane type controlled by oil servo motor is adopted for the inlet valve.

This inlet valve shall withstand maximum hydraulic pressure 350 m and full flow shutdown capacity.

(4) Generator

A three (3) phase alternating current synchronous generator with vertical and semi-umbrella type is adopted.

1) Type

Generally the semi- umbrella type is adopted, for the middle class capacity which is over 100 MVA in capacity with the runaway speed ranging between 400 and 700 (min^{-1}).

This generator has middle class of capacity more than 100 MVA with rotation speed of 300 (min^{-1}) and the run away speed is 561 (min^{-1}). Therefore, the semi-umbrella type is adopted.

2) Generator Capacity

Generator capacity is calculated with following formula:

$$
\begin{aligned}
Pg &= Pt \times \eta g \times 1/\cos\varphi \\
&= 122\ MW \times 0.975 \div 0.85 \\
&\fallingdotseq 140\ MVA
\end{aligned}
$$

Where,

Pg	:	Generator capacity
Pt	:	Turbine output
ηg	:	Generator efficiency (estimated with actual data of similar projects)
$\cos\varphi$	:	Power factor = 0.85

From the above calculation, the rated capacity of 140 MVA is adopted.

3) Insulation and Cooling Method

F class is adopted for insulation of the stator and rotor, and enclosed hood, air cooled type with water heat exchanger system is applied to the cooling method.

4) Ratings

The features of the generator examined above are summarized as follows:

- Rotation direction	Counter clockwise from view of generator top
- Rated speed	300 min^{-1}
- Rated capacity	140 MVA
- Rated Power Factor	0.85
- Rated voltage	16.5 kV
- Rated frequency	50 Hz

Thyristor type of excitation system is adopted. For maintaining stability of grid voltage, high voltage side control system in digital AVR is considered.

Regarding bearing system, Polymer Bearing (PEEK) is recommended to be adopted for improving reliability.

(5) Operation Control System

For operation of this power plant, the one-man control system is adopted for the turbine and generator.

Power plant is controlled in the control room in the existing power plant. SCADA system is adopted by using desk-top computers for the control. Transfer speed of the data is recommended to be high (1Gbps or more) for improvement of reliability.

Visual integrated information is to be indicated with a plasma display set on each control desk, for operators to easily watch for monitoring.

The large size plasma display is recommended to be also installed beside the existing control board or appropriate place in the existing control room.

Necessary information to be transferred to the Central Dispatching Center by using communication system will be defined during the detailed design stage.

The generator, main transformer, 220 kV bus, power cables and station service circuit will be protected by the necessary protection relay systems.

(6) Main Transformer

Generated power is paralleled in by the circuit breaker located in the main generator circuit, and the voltage 16.5 kV is stepped up to 220 kV by the main transformer.

The power is transmitted to the 220 kV switchyard using 220 kV high voltage power cables. A location of the main transformer is to be outside next to the powerhouse in consideration of connection from the generator and that to the switchyard. The capacity of the main transformer is defined as 145 MVA in consideration of the generator capacity, reactive power and station service power capacity.

From the viewpoint of transportation limitation, a special three-phase transformer or 3 units of single-phase transformers is recommended to be adopted.

Main specifications of the main transformer are as follows:

- Rated capacity	145 MVA /unit	
- Rated voltage	Primary	16.5 kV
	Secondary	220 kV
- Rated current	Primary	5,100 A
	Secondary	380 A
- Rated frequency	50 Hz	
- Location	Out door type	
- Cooling method	ONAF (Oil natural Air forced)	

The secondary side (220kV) of the transformer is connected to the outdoor switchyard by 220kV power cable. The connection parts of the secondary side of the main transformer (220 kV side) is protected by lightning arrestors.

The isolated phase bus or segregated phase bus is adopted for large current flowing between the generator and the main transformer.

(7) 16.5 kV Indoor Switchgear

Parallel-in circuit breaker, disconnecting switches for 16.5 kV between the generator and the main transformer are installed.

The exciter transformer, surge absorber and power cables for station services are also installed.

(8) 220 kV Outdoor Switchgears

220 kV outdoor switchgears for the expansion power plant are installed in existing switchyard preparing for the expansion equipment.

The 220 kV GCB (gas insulated circuit breaker for A and B bus), DS (disconnecting switches) connected to the bus bar, CT (current transformers) and VT(voltage transformer) for protection and for measurement, support insulator, bus conductor and steel structures are installed in the switchyard. Details of equipment will be defined during the detailed design stage.

220 kV power cables are used for power transmission between the main transformers and outdoor switchyard. The power cables are installed in the existing cable gallery.

(9) Over Head Traveling Crane

The maximum capacity of main hook of the over head traveling crane is defined with the maximum weight of installed equipment parts.

Generally the rotor of the generator is the heaviest equipment part. It is estimated at192 ton in the Study.

The rated capacity of the over head traveling crane includes weights of connection beam, wire ropes and hooks.

The rated capacity is estimated at 230 t (115 t × 2 units) or more, however the maximum weight may be different in the manufacturer’s design, therefore the rating will be reviewed during the detailed design.

(10) Other Necessary Equipment

Other necessary machines in the powerhouse are auxiliary pumps, low voltage power system, DC power supply system and diesel engine generator system for black start.

1) Station Service Circuit

Power for the station service circuit is supplied through the main transformer, generator circuit and station service transformer which has the capacity 3,000 kVA.

The power is distributed to 400 V low voltage circuit connected with auxiliary pumps, lighting system, power sources for switchyard and miscellaneous uses.

A new station service transformer with 11 kV/400 V and 500 kVA is installed in the powerhouse for the power source of the intake gate.

The existing switchgears in station service circuit will be renewed because of modification of the rated values. Control cables, power cables, communication cables and terminal boxes are used for connection between auxiliary pumps and other necessary parts.

2) Emergency Power Source

A diesel engine generator is being used for emergency power for the existing power plant. However the capacity is only 315 kVA and is not enough for the new power plant. Therefore a 500 kVA new diesel engine generator is to be installed.

The new diesel engine generator is to be connected to both existing and new station service circuits without any disturbance for supplying the power as common use.

230 V DC power supply system for control of the power plant and DC 50 V for indication lamps are connected to this station service circuit through a battery system and battery charger board.

The battery type is recommended to be of VRLA (Valve Regulated Lead Acid) type which has advantage on disaster prevention.

3) Water Supply System

Cooling water for each generator supplied with water supply pumps is to be taken from the outlet directly.

An auto-strainer and water supply pumps (regular and stand-by uses) for each unit are installed.

After circulation, the cooling water is discharged to the outlet.

4) Drainage Water Pump

The drainage pit has to have enough capacity against the leakage water in the powerhouse.

The leakage water of the powerhouse is estimated at 0.6 m^3/s. Therefore the required drainage pit capacity is estimated at more than 100 m^3.

The drainage pumps (regular and stand-by uses) are installed as common use for the 2 units.

Jet pump is equipped for emergency use in addition.

5) Compressed Air Supply System

For oil pressure of the inlet valve and speed governor of each unit, and for general uses in powerhouse, compressed air supply systems are equipped.

A high pressure compressed air system (regular, stand-by use) is used for pressure oil system. While, a low pressure compressed air system is used for general use in Powerhouse.

The pressure value will be defined during the detailed design stage.

6) Pressure Oil System

The inlet valve and speed governor for each unit is to be controlled with a pressured oil system.

The system is composed of oil pressure pumps (regular, stand-by use), an oil pressure tank, an oil sump tank, an oil leakage tank, a control board and other necessary parts for each unit.

7) Oil Lubrication System

Lubrication oil system is necessary for bearings of both generator and hydraulic turbine.

The system is composed of a lubrication oil pump for each unit and an oil purifying system (oil tank, filters) for common use.

8) Piping

Water supply, drainage, oil supply and compressed air supply systems are equipped with necessary piping system. The details will be defined during the detailed design stage.

9) Fire Extinguishing System

The fire extinguishing system for the existing plant is of combined system using halogen, water and dry chemical powder type. The same system is to be used for the expansion plant. The details will be defined during the detailed design stage.

(11) Others

Examinations abovementioned are necessary basic items in the basic design.

The machines in the above basic design and not mentioned above shall be specified during the detailed design stage as necessary.

9.6 Annual Energy

9.6.1 Installed Capacity and Annual Energy Based on Basic Design

In this section, the installed capacity and annual energy are calculated based on the results of the basic design examined in **9.3** to **9.5**.

Based on the internal diameters determined in **9.3**, head loss is calculated (refer to Appendix II). The normal intake water level is EL. 430 m as determined in **Chapter 6**, and tail water level with the maximum discharge of 140 m^3/s is EL. 231.04 m which is elevation of water overflowing the weir of the after-bay. The installed capacity is calculated as 228,000 kW, and the unit capacity is 114,000 kW as shown in **Table 9.6.1-1**.

Table 9.6.1-1 Calculation of Installed Capacity

Item	Unit	Figure
Maximum Discharge	m3/s	140
Normal Intake Water Level	m	430
Normal Tail Water Level	m	231.04
Gross Head	m	198.96
Head Loss	m	15.57
Effective Head	m	183.30
Efficiency of Turbine ηt =		0.929
Efficiency of Generator ηg =		0.975
Installed capacity	kW	**228,000**

The power generation for the existing plant and for both existing and expansion plants is simulated by using the inflow (See **Table 5.4-1**) including spillout discharge and excluding discharge from bottom outlet for sediment flushing. The result is shown in **Table 9.6.1-2**. Output sheets of annual energy simulation are attached to Appendix II.

Table 9.6.1-2 Anneal Energy Based on Basic Design

	Unit	Existing	Existing & Expansion
Installed Capacity	MW	210	438
Annual Energy	GWh	704.6	715.9
Firm*	GWh	229.8	468.2
Secondary**	GWh	474.9	247.7
95% Dependable Capacity	MW	210	393

*Note: * "Firm energy" means the total of power generated during 3-hour peak duration.*
*** "Secondary energy" means the total of power generated in duration except 3-hour peak time.*

9.6.2 Additional Energy Calculations for Project Evaluation

In this section, annual energy is additionally calculated for use of the project evaluation described in **Chapter 11**. Output sheets of annual energy simulation of each case are attached to Appendix II.

(1) Case for Increase in Diversion Volume at Polgolla Weir

As mentioned in **5.4**, the average annual volume diverted from the Polgalla weir from 1985 to 2006 is 878 MCM, based on the diversion policy indicating the annual diversion volume of 875 MCM decided in 1985. MASL, however, insists that the diversion volume should increase, because i) the diversion volume prepared in previous studies is larger than 875 MCM, and ii) increasing demand on irrigation water is forecast in future.

At the initial stage of the Study, MASL commented that the Study on the expansion should be carried out based on the annual diversion volume of 1,270 MCM which was estimated during the design stage of the weir. CEB replied to MASL that i) CEB would prepare future plans based on the present diversion policy of 875 MCM until a new policy is prepared in DSWRPP, but ii) the affect to the expansion would be considered in the case that the diversion volume is increased to 1,270 MCM.

In light of the forgoing, affect to the expansion in the said case is to be examined. In the Study, economic validity is studied by using annual energy calculated with the annual diversion volume of 1,270 MCM. It is noted that diversion volume increased to 1,270 MCM means that inflow to the Victoria reservoir decrease from 1,532 MCM to 1,206 MCM.

The energy values before and after expansion are shown in **Table 9.6.2-1**.

Table 9.6.2-1 Anneal Energy in Case of Increase in Diversion Volume

	Unit	Existing	Existing & Expansion
Installed Capacity	MW	210	438
Annual Energy	GWh	572	572
Firm*	GWh	227	399
Secondary**	GWh	346	173
95% Dependable Capacity	MW	207	352

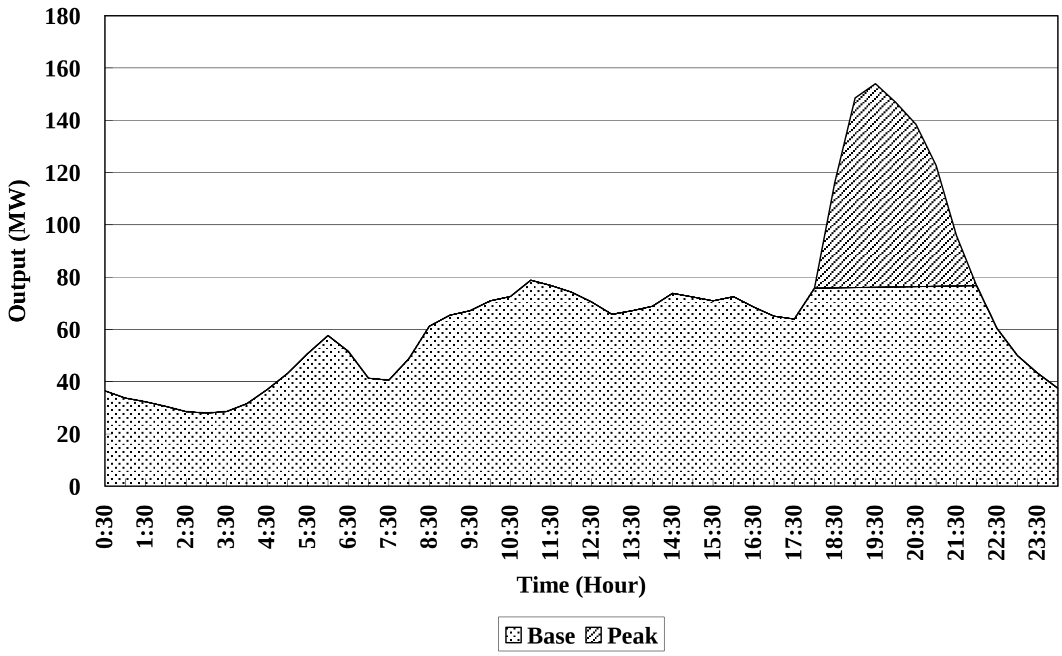
*Note: * "Firm energy" means the total of power generated during 3-hour peak duration.*
*** "Secondary energy" means the total of power generated in duration except 3-hour peak time.*

In comparison with the result in **9.6.1**, the annual generating energy and 95% dependable capacity decrease by 20% and by 10%, respectively. The economic evaluation result of the case is described in "**11.1.5 Sensitivity Analysis**".

(2) Case of Using as Base Demand Power Source

The Study is carried out based on using the Hydropower Station as power source for peak demand. However, it is assumed the case that the hydropower station after the expansion will be used for power source for peak as well as base/middle demand as operated presently for delay in development of base demand power sources for some reasons. Hence, economic validity is examined in the case that the present operation pattern is continued after the expansion.

To clarify operation pattern of the existing plant, annual average of generation records in MW every 30 minutes are calculated. The result is shown **Figure 9.6.2-1**.



Source: CEB

Figure 9.6.2-1 Hourly Output of Annual Average in 2007

The ratio of energy for base and for peak in **Figure 9.6.2-1** is 86:14. That means 86% of water is used for base demand and 14% for peak demand during one day. It is considered that around 85% of water will be used for base demand, in the case that the hydropower station has a role for base and peak demand power sources.

By using reservoir operation rule established in **6.1.6 (3) 9)** (re-numbered as **Figure 9.6.2-2**), the operation pattern is assumed as follows:

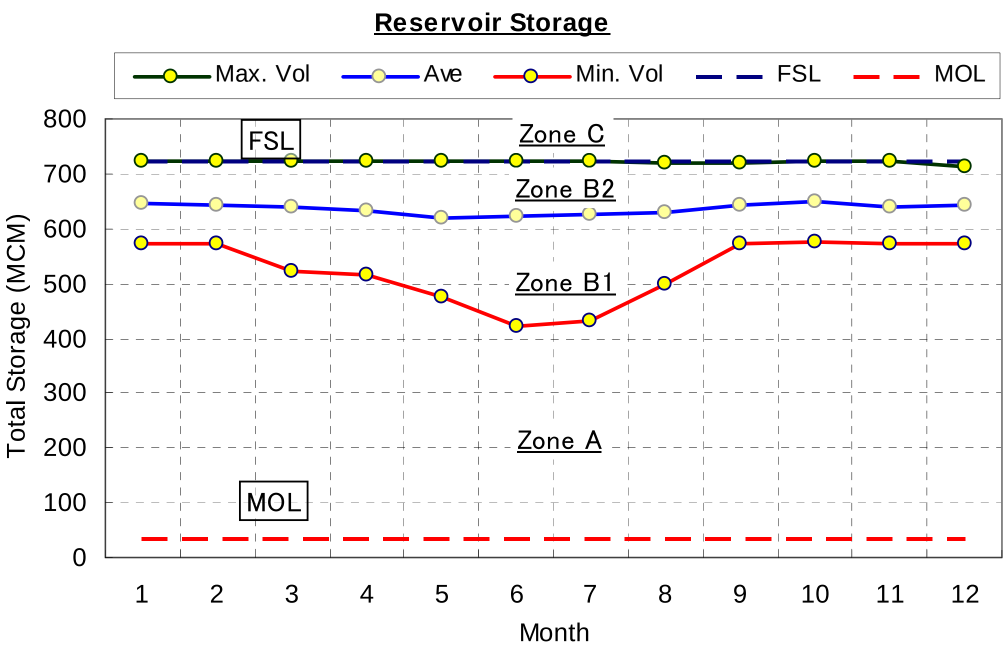


Figure 9.6.2-2 Storage Zone of the Victoria Reservoir

1) 85% of 35 m^3/s, 95% firm discharge calculated in **6.1.4 (3)**, equivalent to 30 m^3/s (86%) is used for base demand power source through a year.

2) Zone A

The power station is used for only base demand power source.

3) Zone B1

The power station is used for base demand power source during off peak time and used for peak demand power source for peak duration (3hours) with 70 m^3/s in maximum by the existing units before expansion and by the expansion units after expansion.

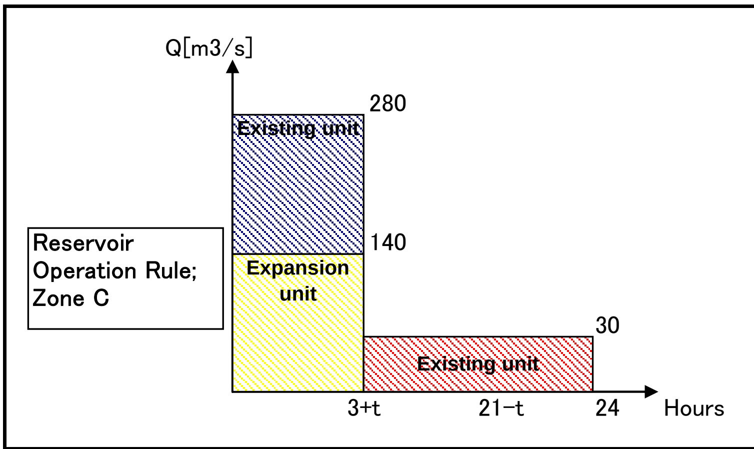
4) Zone B2

The power station is used for base demand power source during off peak time and used for peak demand power source for peak duration with the maximum of 140 m^3/s by the existing units before expansion and with the maximum of 280 m^3/s by both existing and expansion units after expansion.

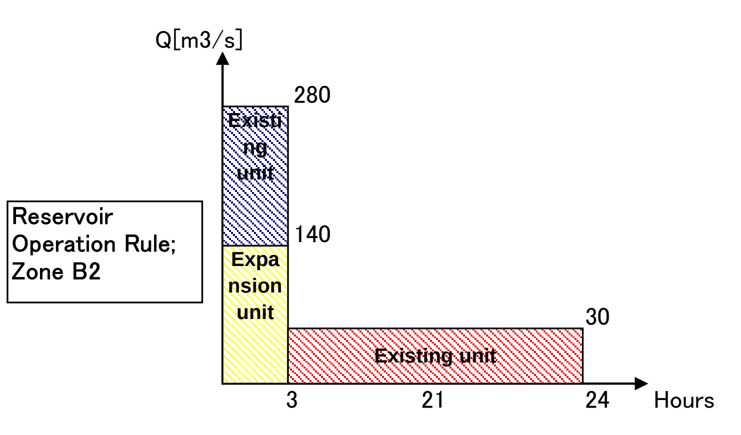
5) Zone C

The existing units with 140 m^3/s before expansion and both existing and expansion units with 280 m^3/s after expansion are operated as long as possible, and operated as base power source in the remaining time.

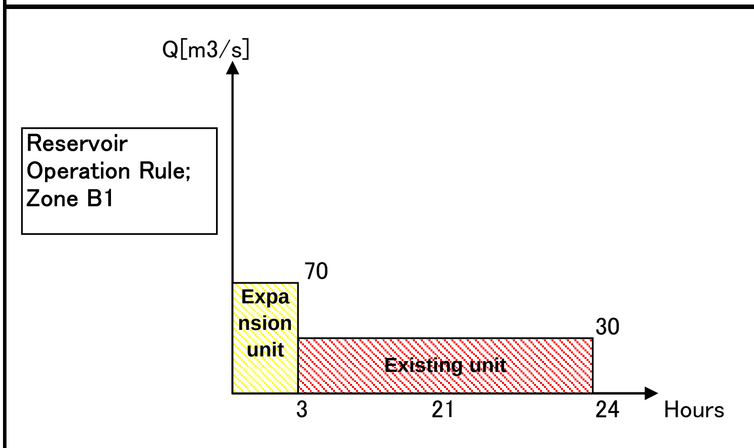
The operation rule is shown in **Figure 9.6.2-3**.



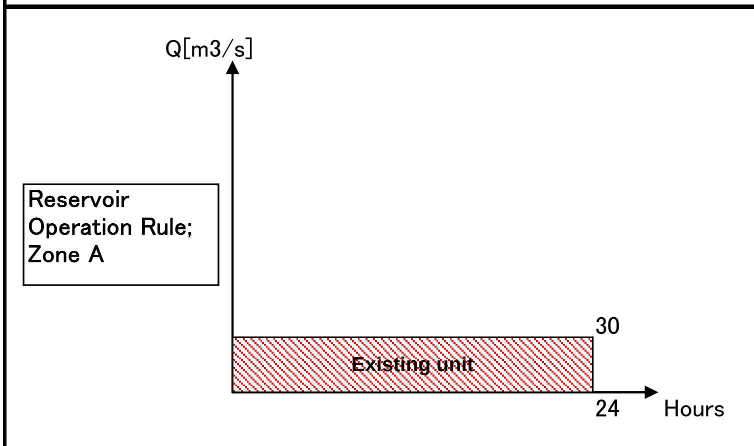
Base	Q	hours	No. of unit
Existing	30	21−t	1
Expansion	0	0	0
Peak	**Q**	**hours**	**No. of unit**
Existing	140	3+t	3
Expansion	140	3+t	2



Base	Q	hours	No. of unit
Existing	30	21	1
Expansion	0	0	0
Peak	**Q**	**hours**	**No. of unit**
Existing	140	3	3
Expansion	140	3	2



Base	Q	hours	No. of unit
Existing	30	21	1
Expansion	0	0	0
Peak	**Q**	**hours**	**No. of unit**
Existing	0	3	0
Expansion	70	3	1



Base	Q	hours	No. of unit
Existing	1	24	1
Expansion	0	0	0
Peak	**Q**	**hours**	**No. of unit**
Existing	0	0	0
Expansion	0	0	0

Figure 9.6.2-3 Assumed Operation Mode

The result is shown in **Table 9.6.2-2**.

Table 9.6.2-2 Annual Energy (Peak: Base = 14:86)

	Unit	Existing	Existing & Expansion
Installed Capacity	MW	210	438
Annual Energy	GWh	709	731
Firm*	GWh	135	172
Secondary**	GWh	575	558
95% Dependable Capacity	MW	49	49

*Note: * "Firm energy" means the total of power generated during 3-hour peak duration.*
*** "Secondary energy" means the total of power generated in duration except 3-hour peak time.*

In comparison with the result in **9.6.1**, annual energy increases by around 2%, but 95% dependable capacity decrease to 1/8. The economic evaluation result of the case is described in Section "**11.1.5 Sensitivity Analysis**".

9.7 Power System Analysis

The following power system analysis was carried out in order to evaluate the influence of Expansion of Victoria Hydro Power Station.

9.7.1 Conditions of Power System Analysis

(1) Year of Commissioning : 2015

(2) No. of Units : 2

(3) Rated Output : 114 MW per unit (228 MW in total)

(4) Interconnection for Power System
The expansion generators would be connected to the existing 220kV buses.

(5) Operating for the Victoria P/S
The expansion and the existing generators will be operated as Peaking Units. Therefore in the Study each generation of Victoria would be operated with its Rated Output for the Night Peak.

9.7.2 Conclusion of Power System Analysis

Following analyses were carried out for the scenarios of Hydro Maximum Night Peak and Thermal Maximum Night Peak. Details of the analysis result are attached in Appendix II.

(1) Power Flow Analysis
(2) Short Circuit Analysis
(3) Transient Stability Analysis

Many over-loaded transformers would be observed in the power system less than 132 kV, and low voltage of the buses would be observed because most of small hydro-power plants would be out of service in the scenario of Thermal-Maximum. However these problems would not depend on

Victoria Expansion and the result of the analysis shows that any problems of the power system would not be occurred by the expansion.

And the brief study for the expansion was carried out by CEB Transmission Planning Branch. No technical limitation would be found in the study.