4 Inspection, Maintenance and Repair for Civil Works

4.1 Monitoring of Hydro-meteorological Conditions and Recording

(1) Objectives of Monitoring

It is essential for the correct operation and maintenance of hydroelectric power stations and related hydraulic structures to grasp the hydro-meteorological conditions surrounding the stations.

The hydro-meteorological information is gathered through the monitoring of rainfall and water levels. The collected data are useful for forecasting water levels of reservoirs and regulating ponds in both normal and flood conditions and the efficient operation of the stations.

- It is recommended that monitoring of the following items be carried out:

  (i) Rainfall at sites surrounding the powerhouse and intake.

  (ii) Water levels of reservoirs (and regulating ponds for run-of-river type station) and also the water levels in the river downstream of the tailrace.

- To accomplish effective operation and maintenance of hydroelectric power stations and their relevant hydraulic structures, the rainfall and water level monitoring results are to be recorded, reported and kept in accordance with the regulations on operation and maintenance.

Source: Manual of Hydrological Analysis, Nippon Koei Co., Ltd.

Figure 4.1.1 Rainfall Gauge

(2) Monitoring of Rainfall

Daily Monitoring of Rainfalls

- The daily monitoring of rainfall is recommended at the sites surrounding the powerhouse and intake.
- Daily observation of rainfall by a rainfall gauge of conventional type is to be carried out at 09:00 a.m. or at some other specified time.

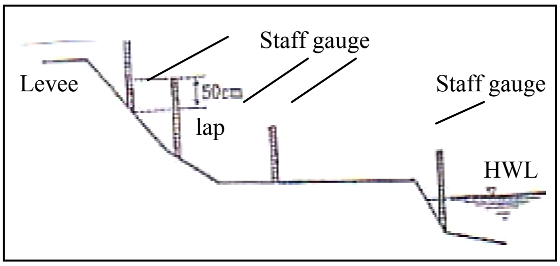
- The volume of rainfall is to be represented at a water depth (unit: mm) which could be obtained by the assumption that rainfall has passed a pan within certain hours and be stored at the horizontal surface of the pan.

- The minimum reading unit is to be 0.1 mm.

Hourly Observation of Rainfalls during Flood

- The hourly observation of rainfall during floods is useful for forecasting water levels of reservoirs and regulating ponds during future floods. However, because the hourly observation of rainfall during floods may endanger the observer, hourly observations should be carried out only using automatic rain gauges.

(3) Monitoring of Water Levels



Source: Manual of Hydrological Analysis, Nippon Koei Co., Ltd.

Figure 4.1.2 Installation of Staff Gauge in River

Daily Observation of Water Levels

- The daily observation of water levels is recommended at both reservoirs and regulating ponds for run-of-river type power stations.

- Also, it is recommended to carry out daily observation of water levels in the river downstream of the tailrace to grasp the volume of river flow.

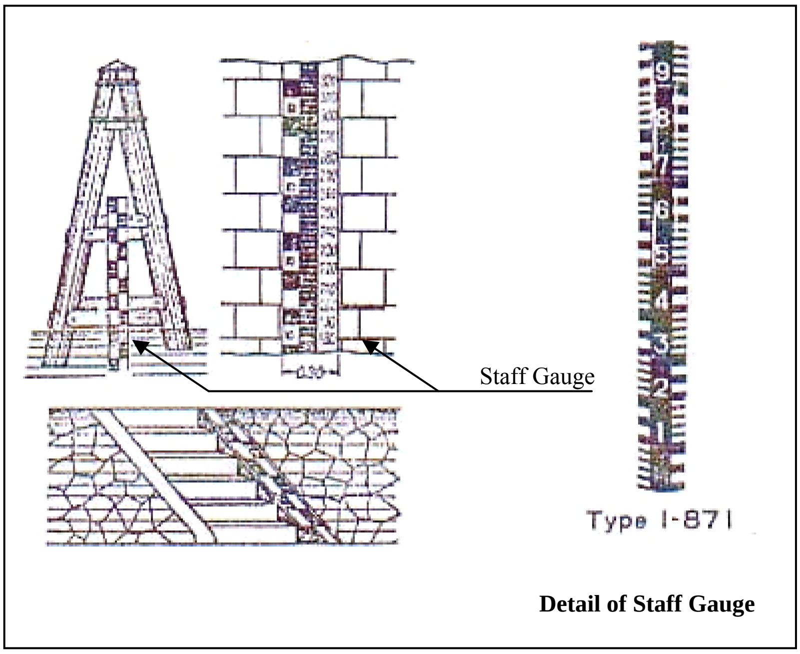
- Staff gauges are to be installed at such locations so as not to be affected by backwater and the water levels are to be observed daily at 10:00 a.m. or at some other specified time.

- The readings of staff gauges should be accurate and recorded to the nearest centimeter.

- An example of a staff gauge is shown below with the specification as follows:

  (i) Minimum reading: 1 cm

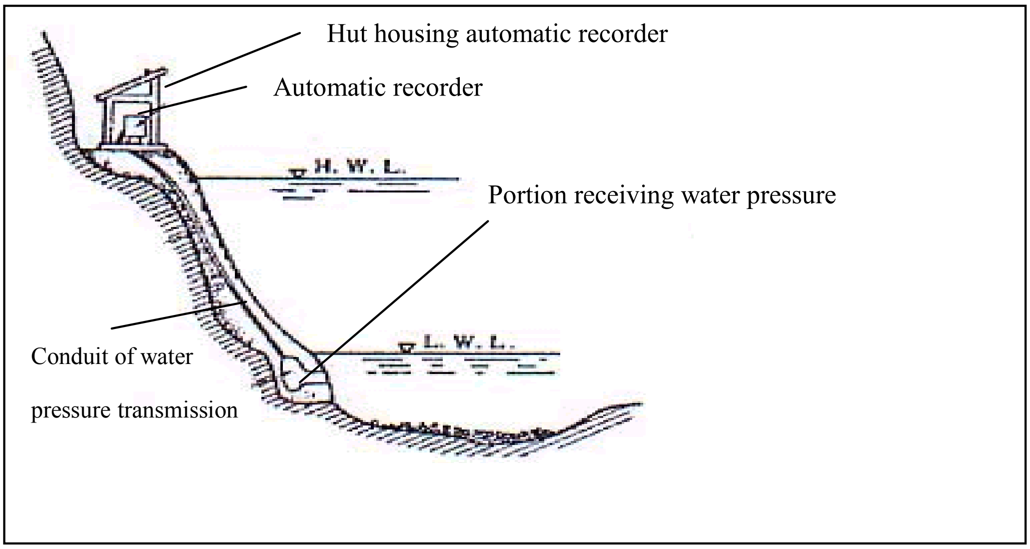
  (ii) Dimensions: length.....1,000 mm, and width.....130 mm



Source: Manual of Hydrological Analysis, Nippon Koei Co., Ltd.

Figure 4.1.3 Detail of Staff Gauge

- The installation of automatic water level gauges is convenient for the observation of water levels in regulating ponds where the water levels are always variable



Source: Manual of Hydrological Analysis, Nippon Koei Co., Ltd.

Figure 4.1.4 Water Level Gauge Housing Automatic Recorder with Water Pressure Type

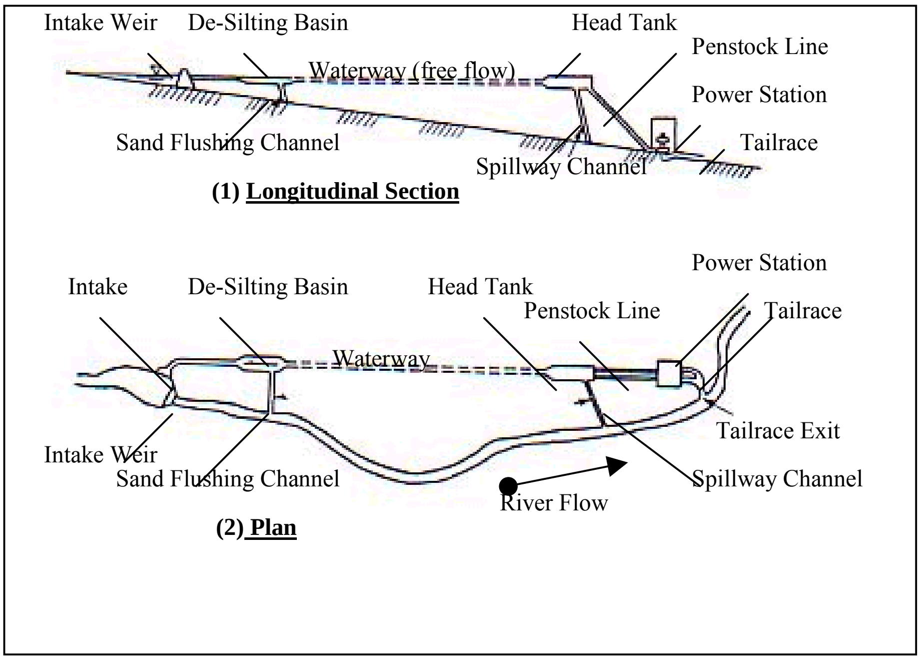
Hourly Observation of Water Levels during Floods

- The hourly observation data of water levels during floods is useful for forecasting water levels of reservoirs and regulating ponds during floods in future. In addition, the hourly observation of water levels in the river downstream of the tailrace during the flood is also useful. However, in due consideration that hourly observation of water levels during floods may endanger the observer, hourly observation is recommended to be carried out only when an automatic water level gauge is installed.

- Further to the recording of water levels observed in the river downstream from the tailrace during floods, the data of the maximum water level will be useful for further confirmation of the safety of hydraulic facilities relevant to the hydroelectric power station.

4.2 Category of Inspection for Hydroelectric Power Facilities

Failure or partial failure of the hydroelectric power facilities (see the following figure) could jeopardise operation of the facilities, endanger the lives and safety of the public and staff or cause substantial property damage.

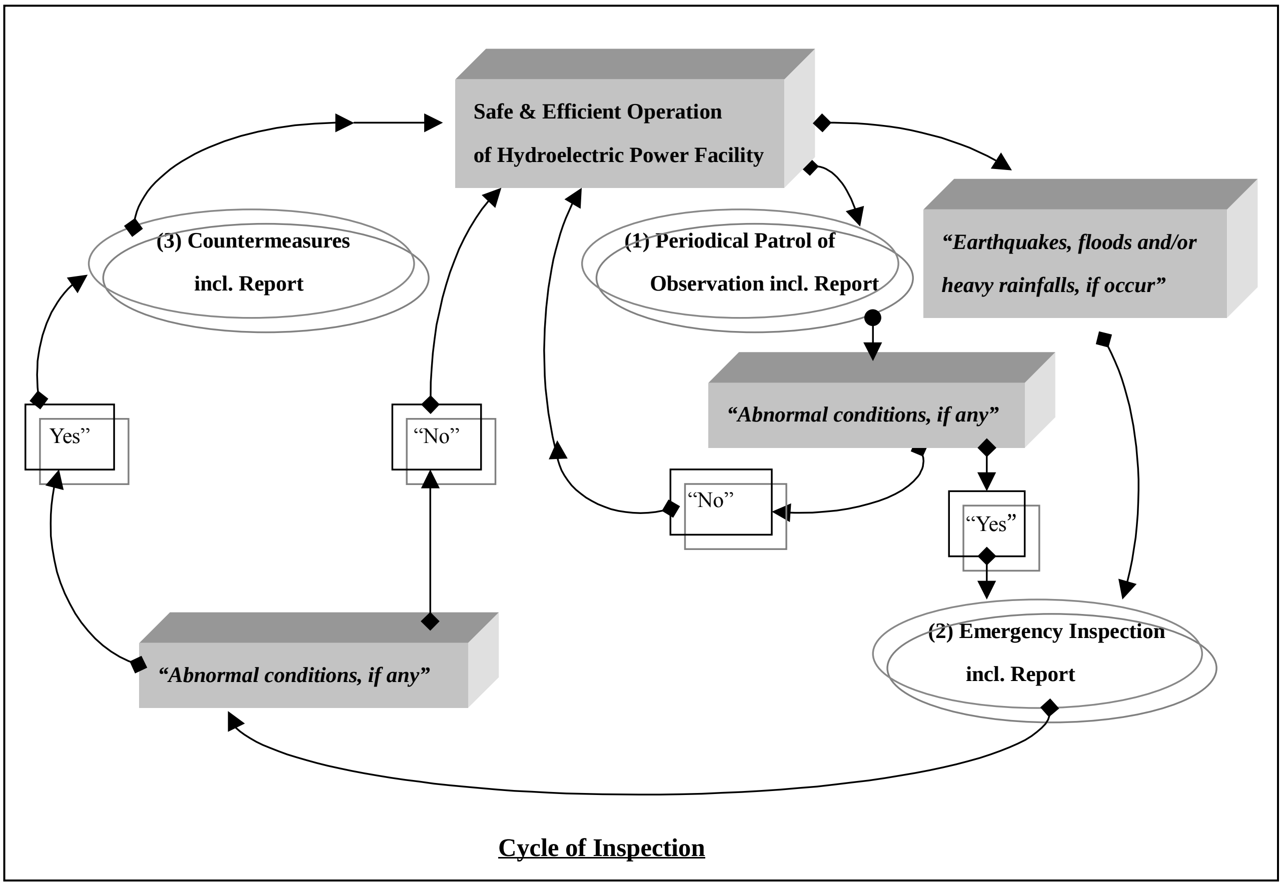


Source: Exercise of Hydro Power, Japan

Figure 4.2.1 Hydroelectric Power Facilities with Run-of-River Type

- It is essential to constantly evaluate the surrounding conditions of the hydroelectric power facility by periodical inspection, i.e. a periodical patrol of observation in order to ensure the stable operations of the plant and ensure its safety and efficient operation.
- In addition, an emergency inspection should be carried out; (i) to confirm the safety of the hydroelectric power facilities and their surroundings, (ii) to maintain stability and safety of each facility, and (iii) to evaluate quantitatively the degree of ageing and deterioration of each facility should the results of periodical inspection show abnormalities in the facilities or after the occurrence of an earthquake, flood and/or heavy rainfall.

The results of these evaluations will be useful for provision of countermeasures inclusive of repairs against the abnormal conditions of facilities in reference to Figure 4.2.2.



Source: JICA Study Team

Figure 4.2.2 Cycle of Inspection

(1) Periodical Patrol Observation

- The periodical patrol observation is aimed to grasp: (i) whether abnormal conditions exist in the hydroelectric power facilities, and (ii) their fulfillment of functions. It is to be carried out periodically once or twice a year in line with the patrol route, and the items to be observed, both of which should be determined in advance.

(2) Emergency Inspection

- The emergency inspection is to be carried out based on the results of periodical patrol observation, if necessary
- In addition, the emergency inspection is to be carried out after earthquakes, floods and/or heavy rainfalls, if necessary.
- The emergency inspection includes inspection and confirmation of the inner conditions of waterway after de-watering as to whether the condition of waterways are abnormal or not, and its fulfillment of functions. It should be carried out once every five years regardless of any other event.

4.3 Civil Works structures

4.3.1 Items to Be Inspected for Civil Works Structures

- The items to be inspected for Civil Works structures are as shown in Table 4.3.1.

4.3.2 Inspection of Civil Works Structures

- Further to the inspection items mentioned above, the procedure of inspection/ monitoring of civil works structures is to be as shown below, which is prepared to grasp the causes for the adverse phenomena identified and identify the countermeasures:

Procedure of Inspection/ Monitoring for Civil Structures:

① To restore a footpath along the headrace canal.

↓

② To identify adverse phenomena and classify them into (i) items related to a decrease in available discharge for power generation, (ii) those related to a decrease in regulation capacity of the head pond, and (iii) those related to a decrease in the safety factor of civil structures.

↓

③ To investigate and examine the adverse phenomena by observation, measurement, topographic survey, hydraulic calculations, etc.

↓

④ To grasp causes of adverse phenomena. → ⑤To identify countermeasures for adverse phenomena.

With respect to each stage (① - ⑤), the detailed discussions are exhibited in Figure 4.3.1(1/5) - 4.3.1(5/5) in reference to the inspection of the civil works structures of Zi Chaung power station which was carried out in the course of both of the 7th and 8th field investigations in Myanmar:

It has been reconfirmed that an inspection of civil works structures is indispensable, irrespective of the importance of an inspection of electro-mechanical works, especially for quantitative confirmation of inflow as to how much volume of inflow is available for power generation. The discharge measurements at the inspection of the civil works structures of Zi Chaung power station showed that the available discharge of power generation has decreased to 69.5% of the inflow at the intake due to leakage of water in the headrace canal (refer to Appendix 1－14 of Part 6-1 in Vol.6 Supporting Report).

① TO RESTORE A FOOTPATH ALONG HEADRACE CANAL

Footpath along Headrace Canal

② TO IDENTIFY ADVERSE PHENOMENA

(i) Items related to a decrease in available discharge for power generation

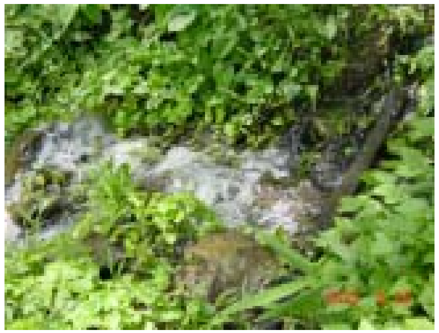
(ii) Items related to a decrease in regulation capacity of the head pond

Sediment Upstream of Intake & Weir

Damaged Screen

Sediment in Headrace Canal

Erosion of Foot of Side Wall



Water Leakage from Headrace

Sediment in Head Pond

1

Figure 4.3.1(1/5) Detailed Procedure of Inspection/ Monitoring

1

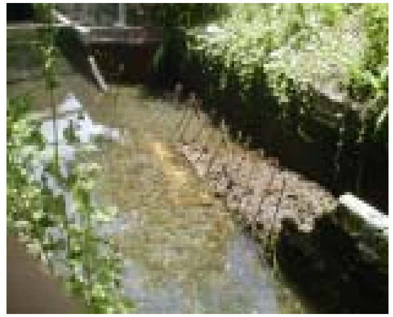
(iii) Items related to a decrease in the safety factor of civil structures

Sediment Upstream of Intake & Weir

Erosion of Weir Body

Different Water Level Caused by Leaves

Damaged Screen



Damaged Side-Spillway

Slope Easy to Slide

Big Stone Fallen from Slope

Debris Once Entered and Removed

2

Figure 4.3.1(2/5) Detailed Procedure of Inspection/ Monitoring

2

③ TO INVESTIGATE AND EXAMINE ADVERSE PHENOMENA BY OBSERVATION, MEAUREMENT, TOPOGRAPHIC SURVEY, CALCULATIONS, ETC.

3

(i) Items related to a decrease in available discharge for power generation

(ii) Items related to a decrease in regulation capacity of the head pond

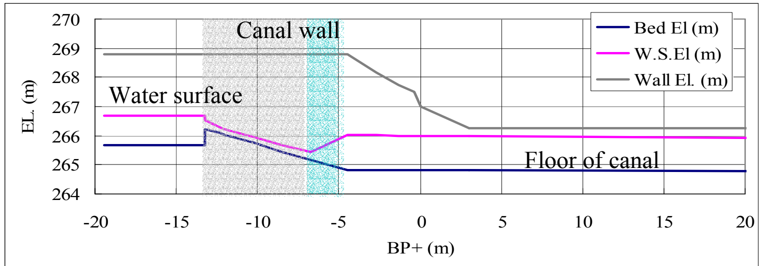
(iii) Items related to a decrease in the safety factor of civil structures


Discharge Measurement


Sediment Thickness


Topographic Survey


Lecture on Leveling Survey



Intake and Water Way Hydraulic Calculations (BP-20m ~ +20m)

Hydraulic Calculations

Figure 4.3.1 (3/5) Detailed Procedure of Inspection/ Monitoring

3

④ **TO GRASP CAUSES OF ADVERSE PHENOMENA**

(i) Items related to a decrease in available discharge for power generation

Causes for Leakage of Water:

① Sediments upstream of weir and intake, ② Inappropriate flow conditions of Intake, ③ Damaged screen, ④ Sediment transport in headrace canal, and ⑤ Erosion of foot of wall in headrace canal.

(ii) Items related to a decrease in regulation capacity of the head pond

Causes for Sediment in Head Pond:

① Sediment in de-silting basin (refer to (5) Damaged Screen and Operation Deck).

(iii) Items related to a decrease in the safety factor of civil structures

Causes for Damaged Side-Spillway: Possibility-a:① Damaged screen, and ② Erosion of foot of side-spillway. Possibility-b: ①Lack of stability in side-spillway due to less thickness of wall.

Causes for Erosion of Downstream Slope of Weir:

① Sediment upstream of weir and intake.

Causes for Debris Once Entered and Removed:

① Slope beside headrace canal, being easy to slide, and ② no concrete covers at the location.

4

Figure 4.3.1 (4/5) Detailed Procedure of Inspection/ Monitoring

4

⑤TO IDENTIFY COUNTERMEASURES FOR ADVERSE PHENOMENA

(i) Items related to a decrease in available discharge for power generation

① Provision of "Gabion Training Walls" for achievement of appropriate flow conditions of intake, to reduce sediment inflow into headrace canal, ② Removal of sediment in headrace canal, and ③ Repair of eroded portion in headrace canal.

(ii) Items related to a decrease in regulation capacity of the head pond

① Periodical removal of sediment in head pond, and ② Removal of sediment in de-silting basin through operation of sand flushing gates.

(iii) Items related to a decrease in the safety factor of civil structures

① Repairing of screen, ② Provision of operation deck at suitable location for removal of drift leaves and driftwoods trapped at screen, and ③ Provision of "Skimmer Walls" in front of intake for avoiding inflow of driftwoods and excessive water into intake.

① Repair and protection of damaged portion in crest and downstream slope of weir.

① Placement of concrete covers over headrace canal in such section where the mountainside slope is found apt to collapse.

Source: JICA Study Team

Figure 4.3.1 (5/5) Detailed Procedure of Inspection/ Monitoring.

Table 4.3.1 Items to Be Inspected for Civil Works structures

Civil Works structures		Locations		Items to Be Inspected							
				Damage	Crack	Water leakage	Displacement	Erosion/ Scoring	Clogging	Sediment	Land slide
1.	Intake weir	(a)	Body of intake weir	○	○	○					
		(b)	Surrounding areas of intake weir		○	○		○			○
		(c)	Related structures		○		○				
		(d)	Other facilities such as water level gauges, staff gauges, safety fences and lighting facilities	○							
2.	Waterway										
	(1) Intake	(a)	Intake body	○	○		○	○			
		(b)	Screen	○					○		
	(2) De-silting basin	(a)	Inside of de-silting basins							○	
	(3) Headrace channel	(a)	Surrounding areas of headrace channel								○
		(b)	Inside of headrace channel		○	○		○		○	
	(4) Head tank and spillway channel	(a)	Body of head tanks/ spillway channels, water level gauges and staff gauges	○	○		○	○			
		(b)	Surrounding areas								○
	(5) Foundations of penstock	(a)	Body of penstock foundations	○			○				
		(b)	Surrounding areas								○
	(6) Power station	(a)	Foundations of powerhouses and related structures		○		○				
	(7) Tailrace	(a)	Body of tailraces, water level gauges and staff gauges	○	○		○	○			
3.	Inspection and insitu roads	(a)	Road surfaces, side ditches, retaining walls, bridges, etc.	○	○		○				

Note: The mark denoted as “ ○ ” means a item to be inspected.

Source: JICA Study Team

The major subjects conducted in the aforesaid inspection/ monitoring are discussed below.

(1) Discharge Measurement and Leakage of Water

In advance of the inspection of the headrace canal (6.5'Wx5.5'H, i.e. 1.981 m Wx1.676 m H) about 3.6 km long, a footpath was restored along the headrace canal by clearing bushes and providing temporary bamboo bridges over small streams and handrails along some parts of the concrete walls of the headrace canal. To confirm the discharge available for power generation, discharge measurements by use of an electric preiss current meter were carried out at four (4) locations for the waterway consisting of an intake, a side spillway, a headrace canal, a de-silting basin and a head pond during the 8th field investigation in Myanmar.

The result of discharge measurements is summarized below (refer to Appendix 1-14 of Part 6-1 in Vol.6 Supporting Report).

Table 4.3.2 Discharge Measurement

No.	Location	Discharge (m^3/s)	Leakage (m^3/s)	Percentage (%)
(1)	Downstream face of 2nd concrete cover about 100 m downstream of intake	1.74	-	
(2)	Upstream face of the concrete cover about 200 m downstream of intake	1.74	-	
(3)	Upstream of de-silting basin	1.35	-	
(4)	Just upstream of inlet of head pond	1.21	-	69.5
	(Available discharge of power generation)			
(5)	Water leakage to upstream of de-silting basin	-	0.39	22.4
(6)	Total of water leakage:	-	0.53	30.5
	(Water leakage to just upstream of inlet of head pond)			

Source: Measured by JICA Study Team

As seen from the above table, the discharge available for power generation was 1.21 m^3/s (decreased to 69.5% of the inflow of 1.74 m^3/s) due to the leakage of water from the headrace canal estimated at 0.53 m^3/s (30.5% of inflow of 1.74 m^3/s).

- Such a fact as identified above recommends the following items:
  - A footpath along the headrace canal is to be provided for inspections.
  - The leakage of water means the loss of energy; accordingly, periodical discharge measurement by use of an electric preiss current meter is recommended for quantitative confirmation of inflow as to how much volume of inflow is available for power generation.

(2) Sediment inside the Headrace Canal and Erosion of the Headrace Canal

On May 25, 2003 during the 8th field investigation in Myanmar the intake gates were closed so that the headrace canal was dewatered and inspection of the canal was carried out along the footpath. It revealed much sediment inside the headrace canal beyond expectations as shown in the following photographs. In particular, the sediments just downstream of the side-spillway, for example, measured about 35 cm thick as shown in the following photographs:

Source: JICA Study Team

Figure 4.3.2 Sediment in Headrace Canal

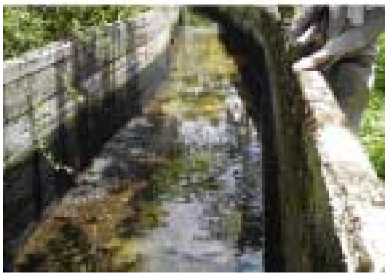
Source: JICA Study Team

Figure 4.3.3 Sediment Measurement

Other conditions of the canal observed during the inspection are summarized below:

Erosion of Foundation

The foundation of the canal seemed to have been eroded by leakage water.



Source: JICA Study Team

Figure 4.3.4 Floor Slab at the Leakage Section

Source: JICA Study Team

Figure 4.3.5 Outside Conditions of Foundation at the Leakage Section

Erosion of Floor Slab and Foots of Side Wall

Erosion of floor slabs and the foot of the side walls was also identified as presented in the photographs below:

Source: JICA Study Team

Figure 4.3.6 Erosion of Floor Slab

Source: JICA Study Team

Figure 4.3.7 Erosion of Foot of Wall

- The following ways are recommended to treat water leakage in a headrace canal as shown in the above photographs:

  - Several countermeasures are to be sought to stop water leakage and the most appropriate countermeasures selected, depending on the cause of water leakage, i.e. erosion of floor slab, erosion of foot of side wall, erosion of foundation of canal due to leakage water, etc.

(3) Topographic Survey and Hydraulic Calculations

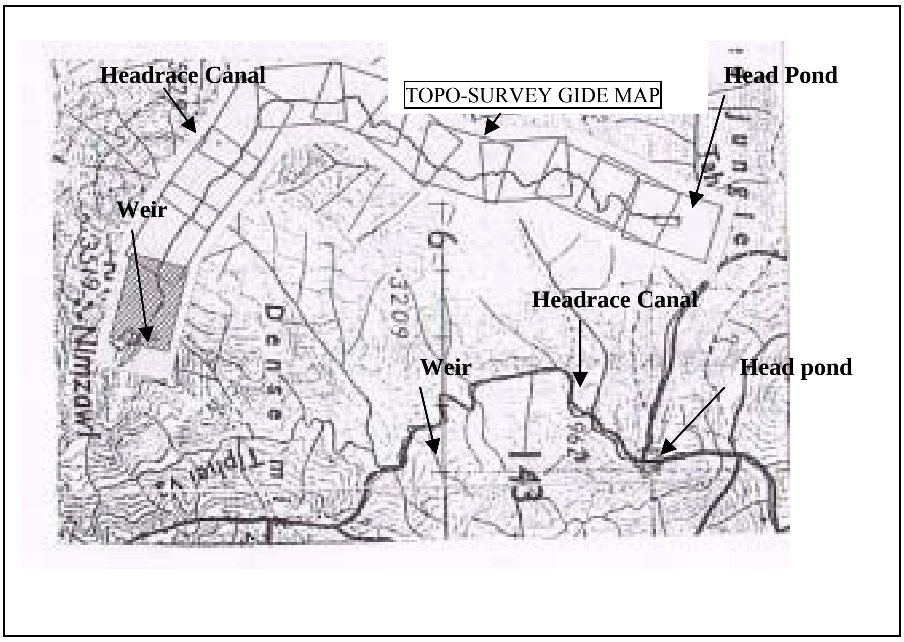
<u>Topographic Survey</u>

In the course of monitoring of the headrace canal, the discharge capacity of the waterway, especially for the section of waterway from the beginning of the intake to the end of the side-spillway, was of great concern to confirm the discharge available for power generation. The following cases were adopted for hydraulic calculations:

(i) In the case of the headrace canal having no sediment, discharge capacity was the same as the design.

(ii) In the case of the headrace canal having sediment, surveyed data were used to calculate discharge capacity.

In advance of hydraulic calculations, the leveling survey was carried out over the whole 3400 m of the headrace canal in order to achieve the canal data for hydraulic calculations.

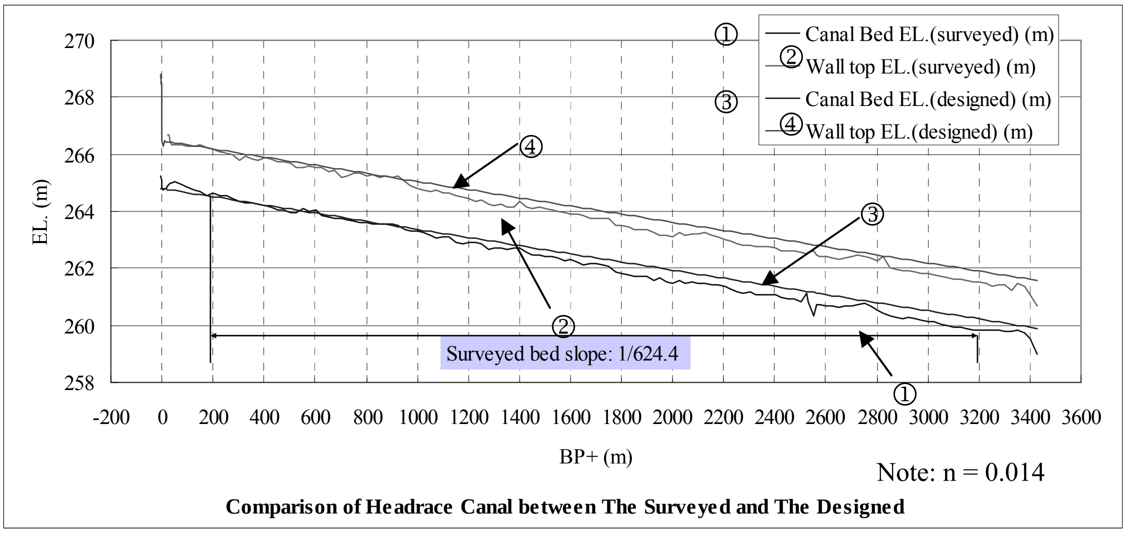


Source: JICA Study Team

Figure 4.3.8 Guide Map of Headrace Canal

Comparison for Elevation of Floor Top between The Designed and The Surveyed

Further, the comparison between the designed profile of the headrace canal (i.e. the top of floor before sedimentation) and the surveyed one (i.e. the top of floor after sedimentation) is as shown below.



Source: JICA Study Team

Figure 4.3.9 Comparison of Headrace Canal between The Surveyed and The Designed

The balance between the designed floor level of the canal and the surveyed one is shown below for the upstream section of the canal from the beginning point to the downstream point 200 m along, which corresponds to the extent of the sediment deposition:

Table 4.3.3 Comparison of Floor Level between The Designed and The Surveyed

Item	Location (Distance from Sta 0)						
	0 m	25 m	50 m	75 m	100 m	150 m	200 m
(i) Designed floor level (EL.. (m))	264.757	264.721	264.685	264.650	264.614	264.542	264.471
(ii) Surveyed floor level (EL.. (m))	264.948	265.017	264.926	264.833	264.764	264.544	264.557
(iii) Balance: (ii)-(i)/ (Corresponding to: Thickness of sediment (m))	0.191	0.296	0.241	0.183	0.150	0.002	0.086

Source: JICA Study Team Note: n = 0.014

Hydraulic Calculations in the Case of Headrace Canal Having No Sediment

In the case of the hydraulic calculations for the headrace canal having no sediment, it was confirmed that the upstream section between the beginning of the intake and the end of the side-spillway has a discharge capacity of 4.0 m^3/s, the same as the design discharge, with a sufficient free board of 30 cm, as shown below.

Table 4.3.4 Hydraulic Calculations in The Case of Headrace Canal Having No Sediment As Same As Design for Design Discharge (Q=4.0 m^3/s) for BP-20 to BP+200

Location (BP+)	Floor Level	Water Surface	Top of Wall	Free Board	Velocity	Fr
(m)	(EL. - m)	(EL. - m)	(EL. - m)	(EL. - m)	(m/s)	
-19.4	265.66	266.69	268.80	2.11	0.5	0.16
-13.2	265.66	266.69	268.80	2.11	0.5	0.16
-4.5	264.80	266.03	268.80	2.77	1.31	0.38
0.0	264.80	265.98	267.00	1.02	1.58	0.46
3.0	264.80	265.97	266.27	0.30	1.77	0.52
25.0	264.76	265.91	266.27	0.36	1.77	0.53
50.0	264.72	265.88	266.40	0.52	1.77	0.52
100.0	264.65	265.81	266.33	0.52	1.77	0.52
200.0	264.51	265.66	266.18	0.52	1.77	0.53

Source: Calculation by JICA Study Team

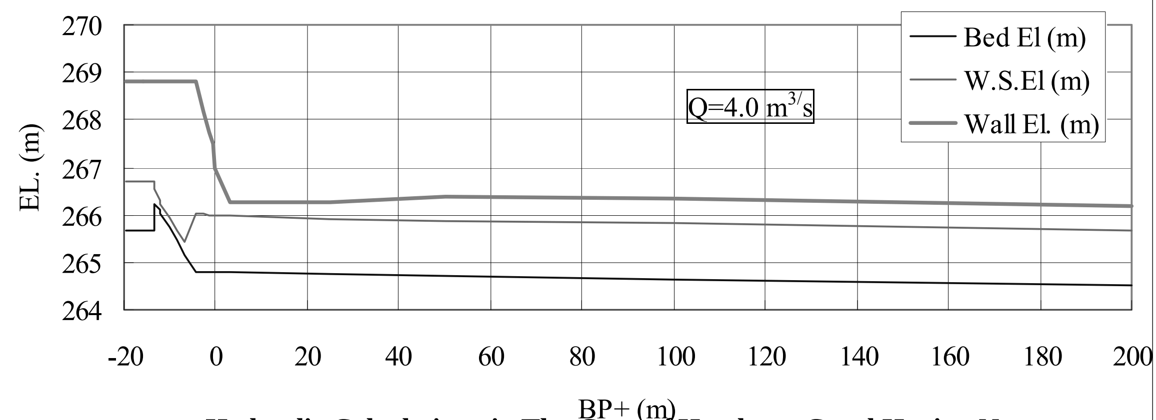


Figure 4.3.10 Hydraulic Calculations in The Case of Headrace Canal Having No Sediment As Same As Design (BP-20 - BP+200)

Source: Calculation by JICA Study Team

Hydraulic Calculations in the Case of Headrace Canal Having Sediment

In the case of the hydraulic calculations for the headrace canal having sediment as surveyed, it was confirmed that the side-spillway started to overflow when the discharge reached 3.8 m^3/s. The discharge capacity of headrace canal is therefore restricted to a discharge capacity of 3.8 m^3/s, i.e. 96% of the design discharge of headrace canal (4.0 m^3/s) as shown below.

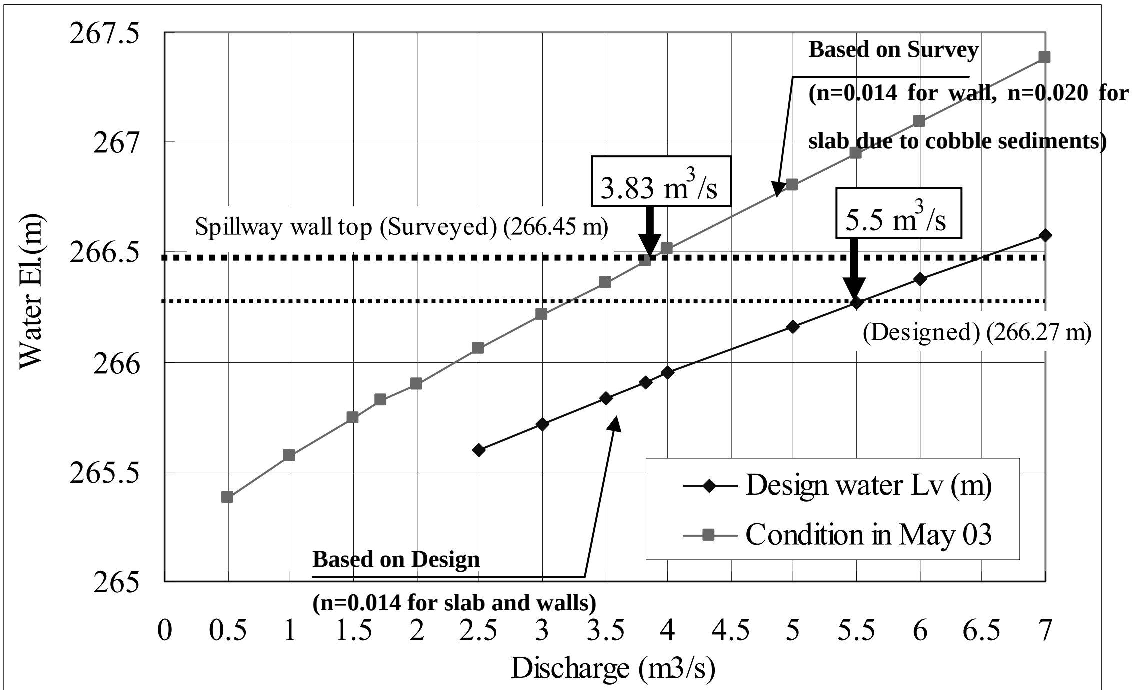


Figure4.3.11 Discharge Rating Curves of Headrace Canal (at BP+2.3 m)

Source: Calculation by JICA Study Team

<u>Consideration of Roughness Coefficient for Hydraulic Calculations and Confirmation of Freeboard for Design of Side-Spillway</u>

In respect of the roughness coefficient of “n” which is an empirical measure of surface roughness for hydraulic calculation, a value of 0.014 was applied for the concrete lined headrace canal, while 0.020 was used where sediment of cobbles was deposited on the concrete lined slab. The n of 0.014 for the slab and walls of the headrace canal without sediment is the same as designed. On the other hand, n of 0.014 and n of 0.020 were applied for the walls and slab respectively where sediments of cobbles existed.

As seen from the above figure, the overflow weir of the side-spillway starts to overflow at a discharge of 5.5 m^3/s in the case of no sediment as designed. In the case of the headrace canal having sediments as surveyed, the overflow weir of side-spillway starts to overflow at a discharge of 3.8 m^3/s, which is less than the design discharge of 4.0 m^3/s. After application of several roughness coefficients for the hydraulic calculations of the headrace canal having no sediment, it was confirmed that application of a roughness coefficient of 0.019 allowed the overflow weir of the side-spillway to start to overflow at the design discharge of 4.0 m^3/s with almost the same freeboard as designed, 0.75 feet (= 22.8 cm).

The roughness coefficient of 0.019 was applied for the hydraulic calculations of Baluchaung hydroelectric power project by reference to its design criteria (refer to

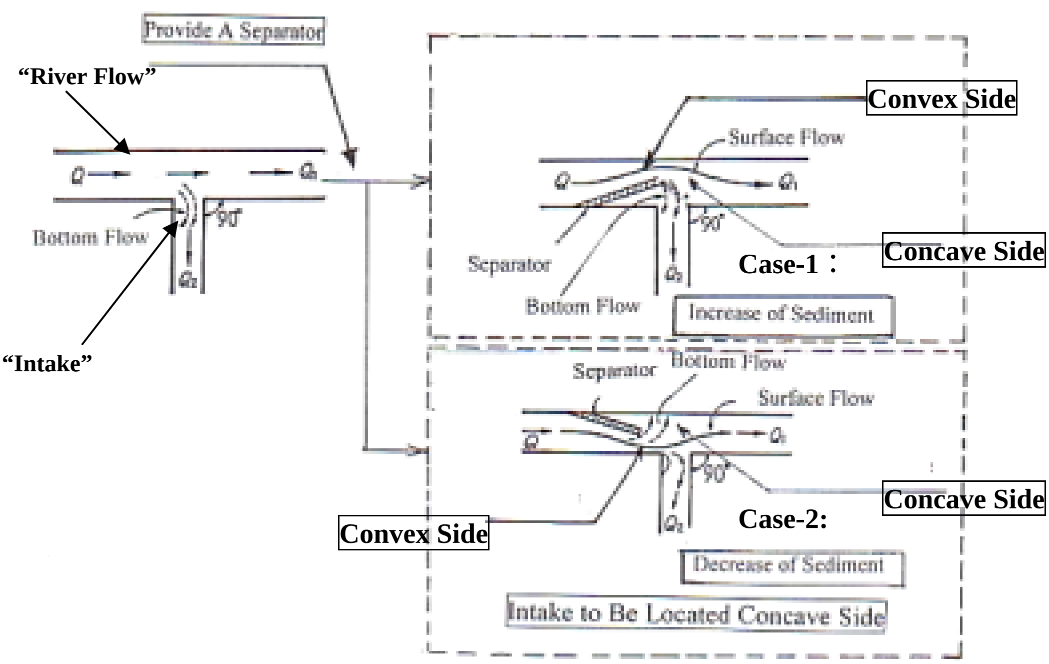
Appendix 3 of Part 6-2 in Vol.6: Supporting Report). Such a roughness coefficient of 0.019 was decided in due consideration of Baluchaung power plant that about 80% of the energy required for the whole of Myanmar would be covered by Baluchaung power plant, which would not allow the Baluchaung power plant to be stopped for dewatering of the headrace canal for removal of sediment after the commencement of power generation.

Consequently, it could be said that application of a roughness coefficient of 0.019 for such a small hydro power project as Zi Chaung power plant would not be reasonable as compared with application of a roughness coefficient of 0.014 which is a standard value of roughness coefficient for the conventional design of hydraulic structures.

Further, it was confirmed that MEPE used the roughness coefficient of 0.018 for the design of Zi Chaung power plant (completed in 1997); however, the Department of Hydroelectric Power (DHP), Ministry of Electric Power has recently used the roughness coefficient of 0.013 for the design of hydraulic structures.

(4) Gabion Training Wall

The intake is located beside the overflow weir and is in the same direction of the river flow as shown in Figure 4.3.7. This location of the intake is judged to be a major cause for sediment from the river entering the headrace canal. Refer to Figure 4.3.12 for a recommendation on the selection of an intake.



Source: JICA Study Team

Figure 4.3.12 Recommendation for Selection of Intake Location

The above figure shows that a branch canal is perpendicularly connected to the flow direction. In spite of such a reasonable direction of the intake, the branch canal is still subject to sediment entering from the river. Under such a condition, an experiment was carried out providing a separator for reduction of sediment entry as shown in case-1 (increase of sediment) and case-2 (decrease of sediment).

- Therefore, it is recommended that the intake be located where the convex shape of river flow exists, for reduction of sediment.

In addition, Figure 4.3.15 shows the profile of the intake in the original design, indicating the permissible height of sediment of 5.974 m (= the intake sill level at EL.265.633 m minus the original river bed at EL.259.659 m). However, the river bed has risen to EL.266.200 m around the weir and several water passages to the intake exist according to the results of topographic survey.

Under such a circumstance, it is suggested to apply a gabion training wall as shown in Figures 4.3.13 and 4.3.14 to achieve an appropriate flow condition to the intake for reduction of sediment.

It is noted that the topographic survey of about 1.5 ha for the upstream of the intake was carried out in advance of this planning of the gabion training wall in the course of the 8^{th} field investigation.

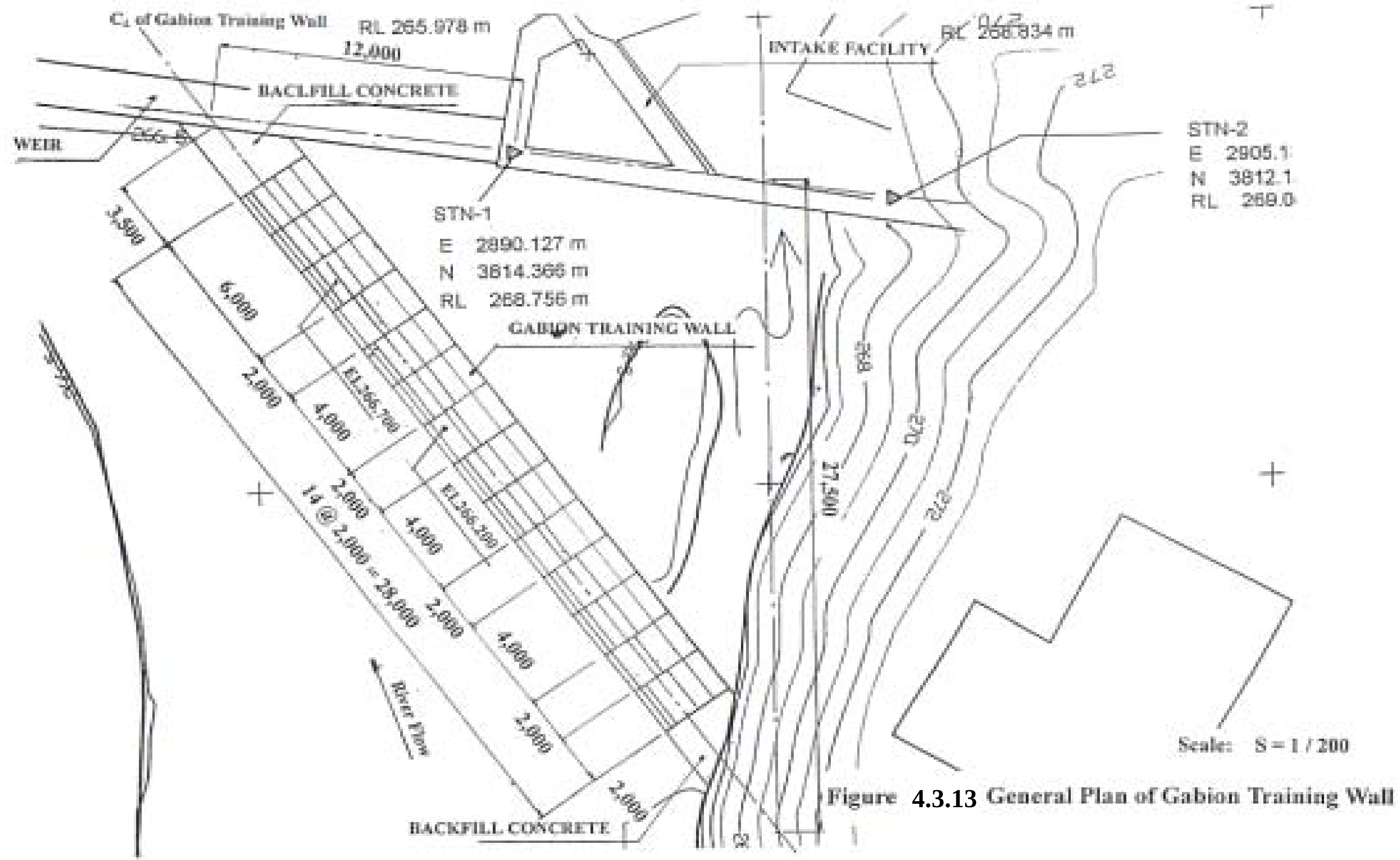


Figure 4.3.13 General Plan of Gabion Training Wall

Source: JICA Study Team

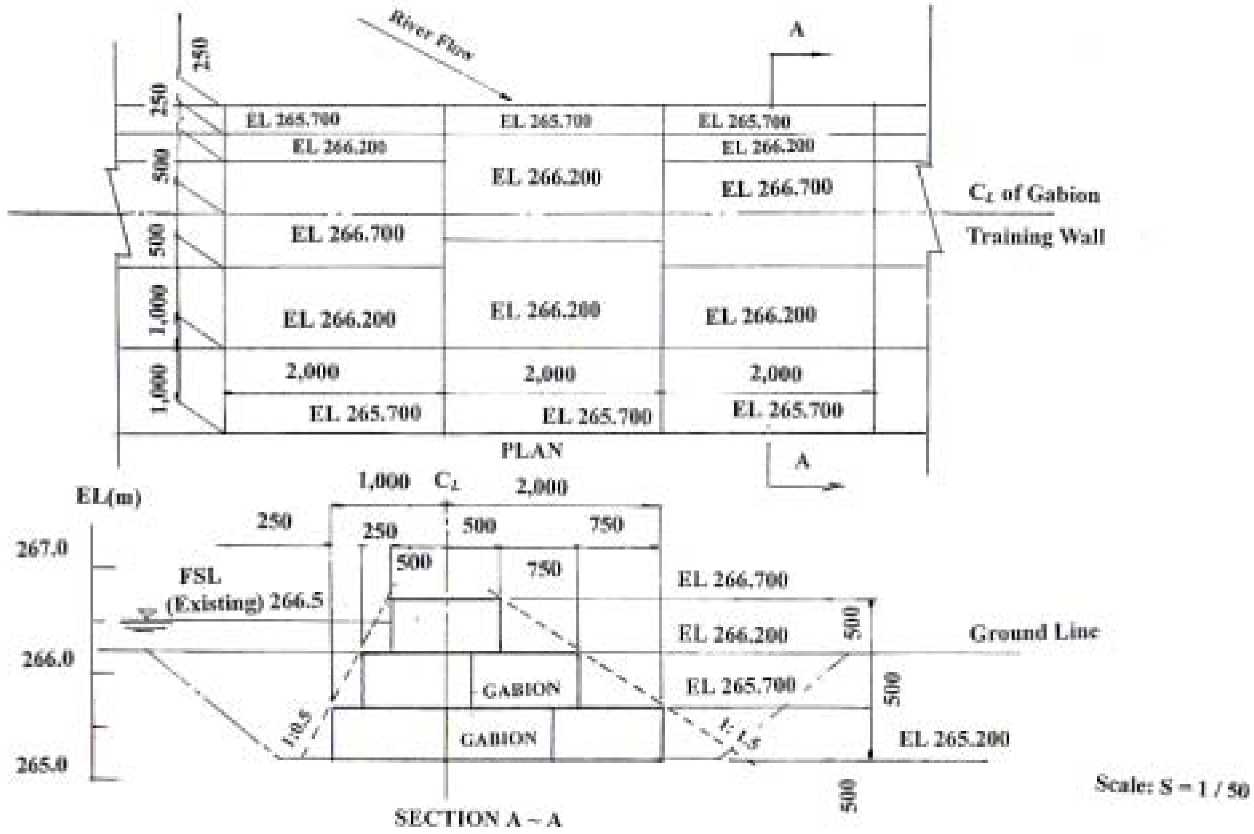


Figure 4.3.14 Detail of Gabion Training Wall

Source: JICA Study Team

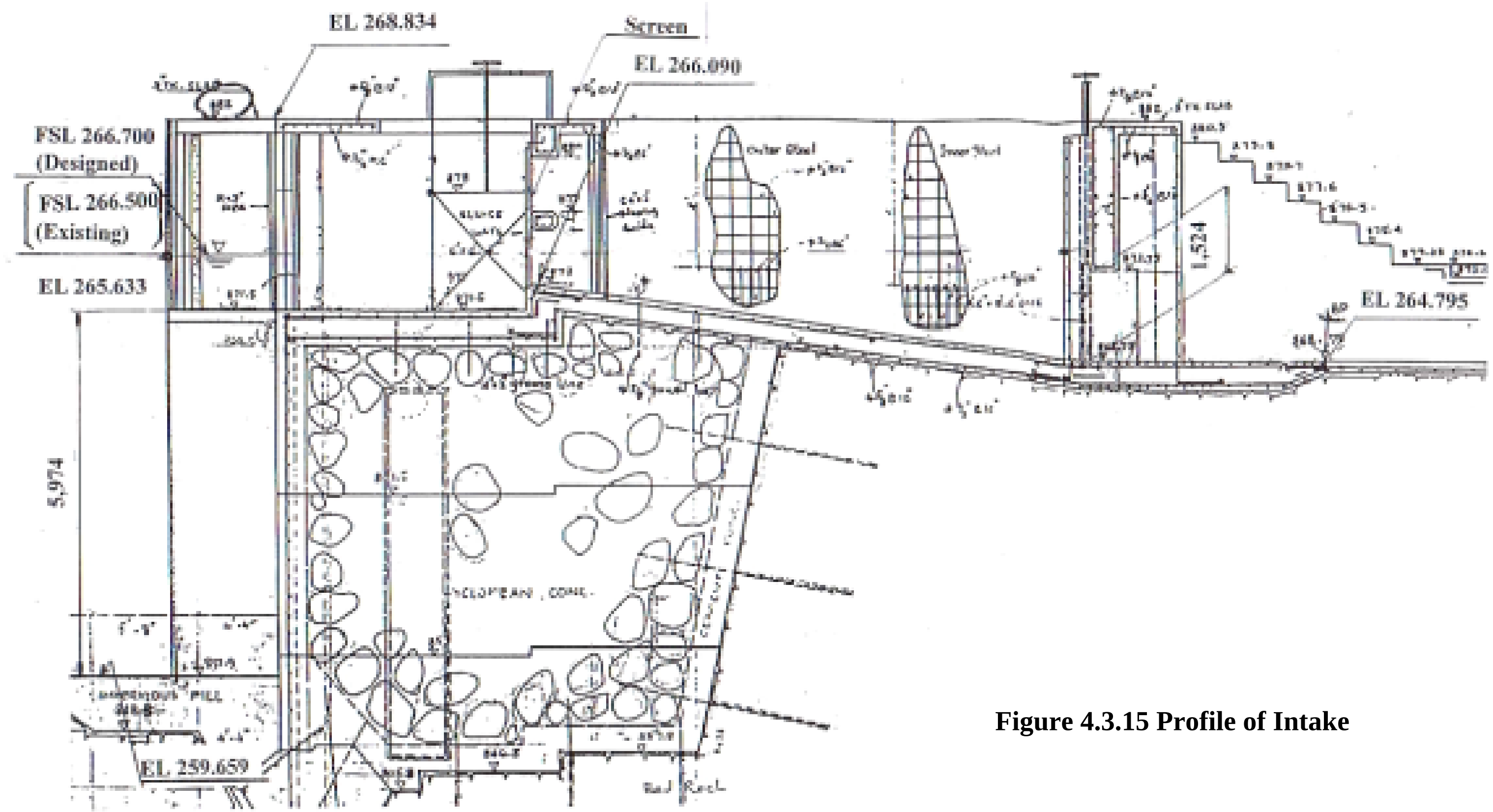


Figure 4.3.15 Profile of Intake

Source: MEPE

(5) Damaged Screen and Operation Deck

<u>Damaged Screen</u>

- Generally speaking, an inclined screen is suitable for removal of drift leaves and driftwoods from screens as compared with a vertical screen.

Nevertheless, the intake of Zie Chaung power station is facilitated with a vertical screen and it is hard to remove drift leaves and driftwoods from the screen. In addition, the operation deck at the site seems impractical for removal of leaves and driftwoods from the vertical screen. Consequently, these conditions prevent removal of drift leaves and driftwoods from the vertical screen. The screen has become damaged due to large water pressure. The damaged screen has then allowed entry of sediment such as cobbles, gravel, sand, etc. into the headrace canal and caused erosion of the headrace canal as illustrated below:

① Vertical screen and inappropriate operation deck.

↓

② No removal of drift leaves and driftwoods from screens.

↓

③ Different water levels between the upstream and downstream of the screen.

↓

④ Large water pressure due to different water levels.

↓

⑤ Screen damaged and eventually collapses of screen.

↓

⑥ Sediment enters headrace canal and cause erosion of the headrace canal and leakage of water.

Operation Deck

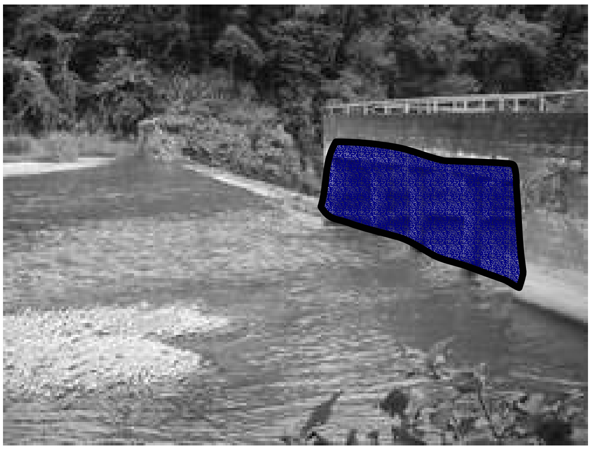
To overcome the inconvenient operation deck, an idea is shown in Figure 4.3.10 modifying the operation deck for easy removal of drift leaves and driftwoods.

Preventive Measures for Driftwoods

In addition to the preventive measures for drift leaves and driftwoods by repair of the screen and modification of the operation deck, one more option is considerable as is shown in Figure 4.3.11, which is to utilize a frames made of reinforcing bars arranged at 50 cm intervals in the gabion training wall.

Skimmer Wall

Another possible preventive measure for driftwoods is to provide a skimmer wall with piers at the inlet for protection of the screen and the sand flushing gates as shown below.



Source: JICA Study Team

Figure 4.3.16 Skimmer Wall at Inlet

(6) Protection of Weir Crest and Downstream Slope

- With regard to determining the method for protection of the weir crest and the downstream slope against erosion by rocks, stones, cobbles, etc. transported by overflow water, a yardstick is whether the damaged portion can be repaired or not in future.

For example, Figure 4.3.19 shows a lining (SUS 304) applied for dams in Japan to protect a sand flushing facility from sediment. The lining is planned to be durable for 100 years as it is located upstream of the dam body making it difficult to repair damaged portions of the sand flushing facility in future.

In addition to repairing the damaged portion, the following options are considerable for protection of the weir crest and the downstream slope of Zi Chaung power station:

- Construction of check dams by gabions similar to the gabion training wall (refer to Figure 4.3.14), steel frame gabions (refer to Figure 4.3.21), concreted gabions, etc. to reduce the amount of gravel and sediment transported by overflow water.

- Placement of an additional reinforced concrete to the weir to allow for erosion of concrete in the weir body (refer to Figure 4.3.20).

In establishment of the rehabilitation plan and design for the weir of Zi Chaung power station, it is suggested that MEPE consult with the concerned staff of the Department of Hydroelectric Power (DHP), Ministry of Electric Power.

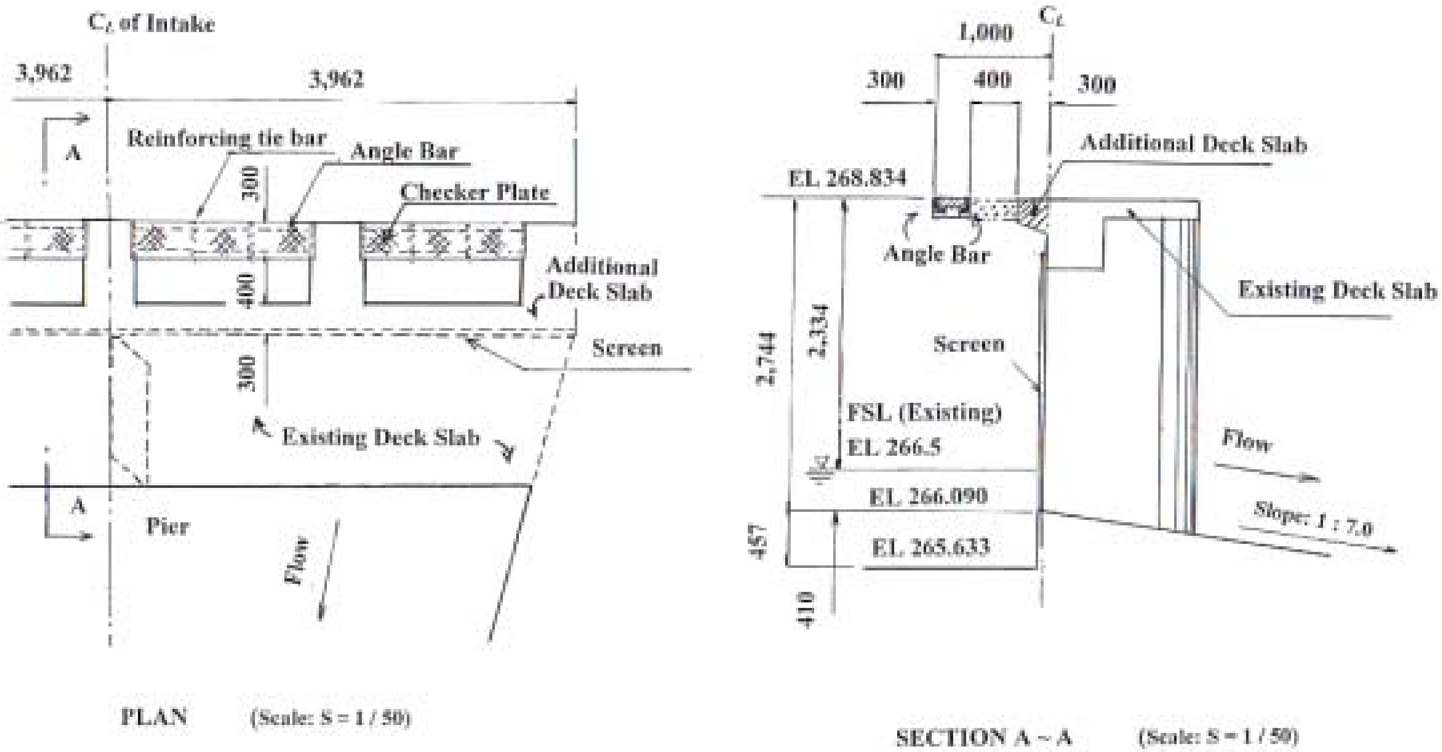


Figure 4.3.17 Plan of Operation Deck

Source: JICA Study Team

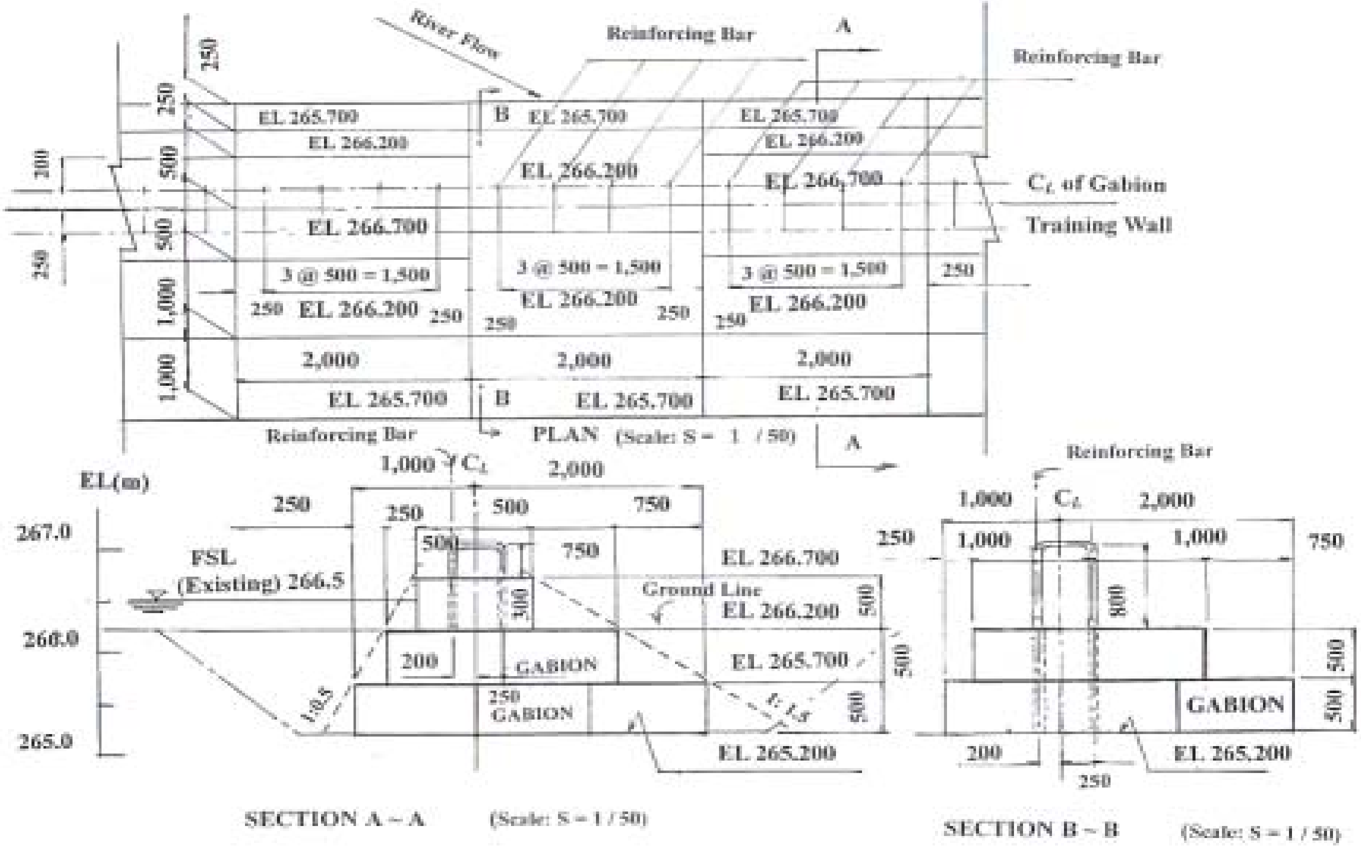


Figure 4.3.18 Plan of Preventive Measures for Driftwoods

Source: JICA Study Team

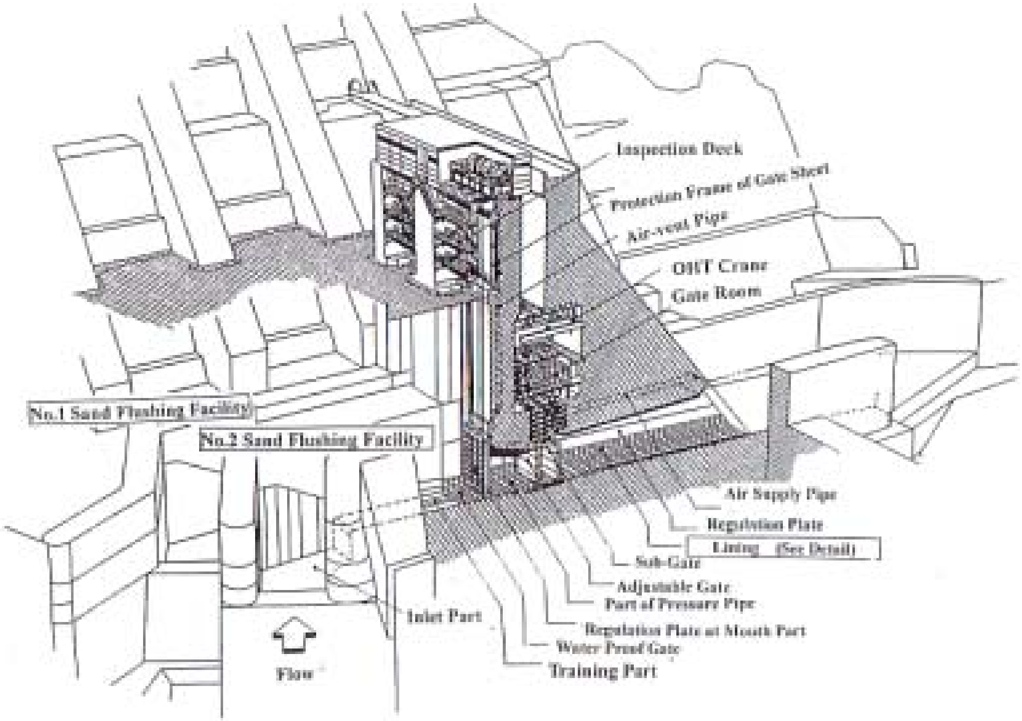


Figure Bird's-eye View of Sand Flushing Facility for Unaduki Dam, Japan

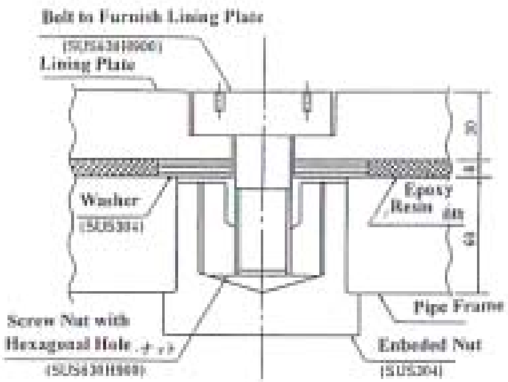
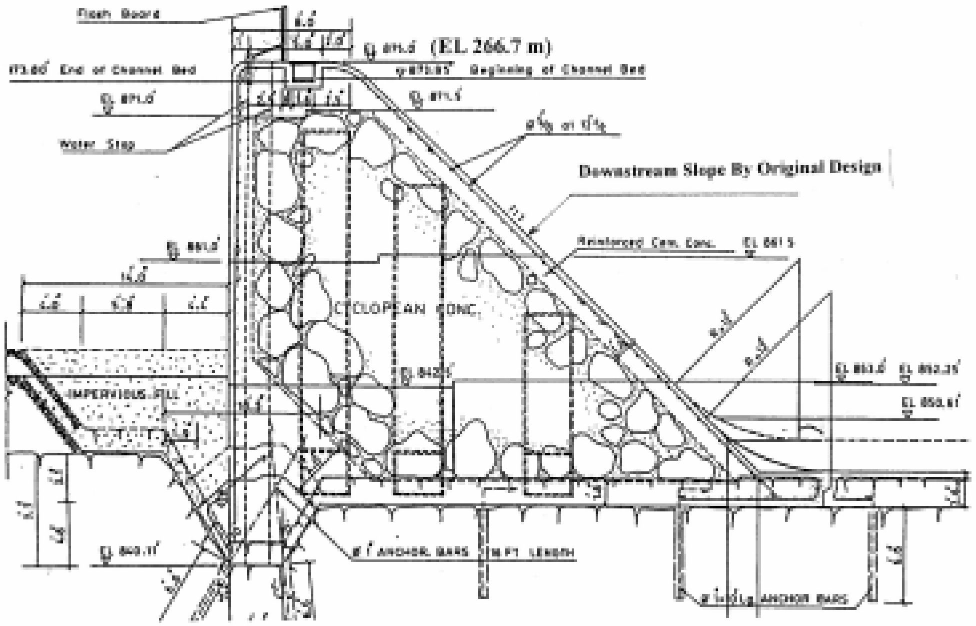
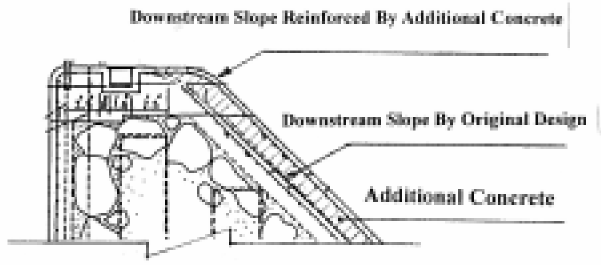


Figure Detail of Furnishing of Lining

Figure 4.3.19 Lining Plate for Protection of Concrete Surface

Source: Dam Engineering No.132 (1997.9), Japan





Weir Body Reinforced By Additional Concrete

Profile of Weir By Original Design

Figure 4.3.20 Weir Body Reinforced By Additional Concrete

Source: MEPE

Source: JICA Study Team

Figure 4.3.21 Steel Frame Gabion

4.4 Gates, Trashracks and Penstocks

4.4.1 Items to Be Inspected for Gates, Trashracks and Penstocks

- The items to be inspected for gates, trashracks and penstocks are shown in Table 4.4.1.

4.4.2 Operational Check Points of Discharge Control Gates for Intake and Head Pond

The operational check points of discharge control gates for the intake and head pond, when provided, are as follows:

(1) Normal Conditions

- Gate leaves are to be in complete contact with the gate sheets to secure water tightness and appropriate countermeasures are to be taken for the prevention of water leakage, if any.

Table 4.4.1 Items to Be Inspected for Gate, Trashrack and Penstock

Steel structures		Locations		Items to Be Inspected						
				Damage	Deformation	Abrasion	Oiling	Painting	Leakage water	Vibration
1.	Gates and hoist	(a)	Gate sheet	○	○					
		(b)	Gate leaf and hoist	○	○	○	○	○		
2.	Trashracks	(a)	Screen	○	○			○		
3.	Penstock	(a)	Pipe	○	○			○	○	○
		(b)	Bearing	○	○			○		○
4.	Others	(a)	Steel ladder, steel cover, etc.	○	○			○		

Source: JICA Study Team

Note: The mark denoted as “ ○ ” means a item to be inspected.

- Each lifting location of the control gates for both the intake and head tank is to be determined to ensure the lowest point of the gate leaves does not touch the water surface.

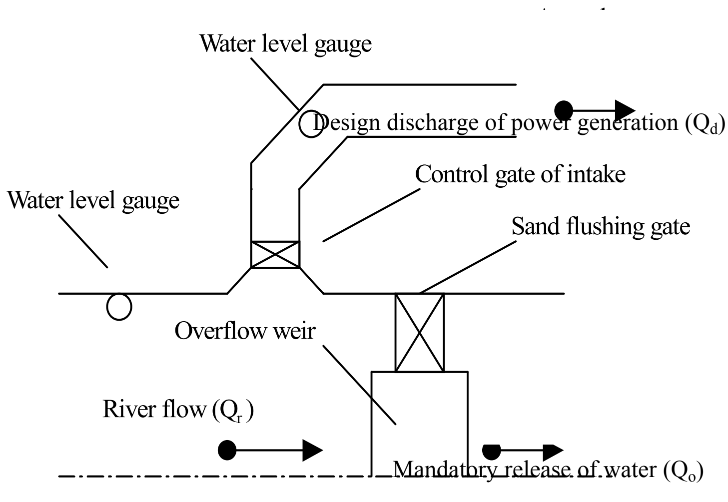
(2) Flood Conditions

- During floods, it is recommended to stop power generation and to close the control gates of the intake to prevent sediment and drift wood from affecting the associated structures.

4.4.3 Gates Operation in Intake

Operations of the control gate of the intake are generally required for: (i) the mandatory release of water for irrigation and environmental requirements in downstream reaches, even with the reduction of discharge for power generation, (ii) water filling and de-watering for the purpose of inspection and repairs of the waterway structures, and (iii) prevention of waterway structures from sediment and drift wood during flood conditions.

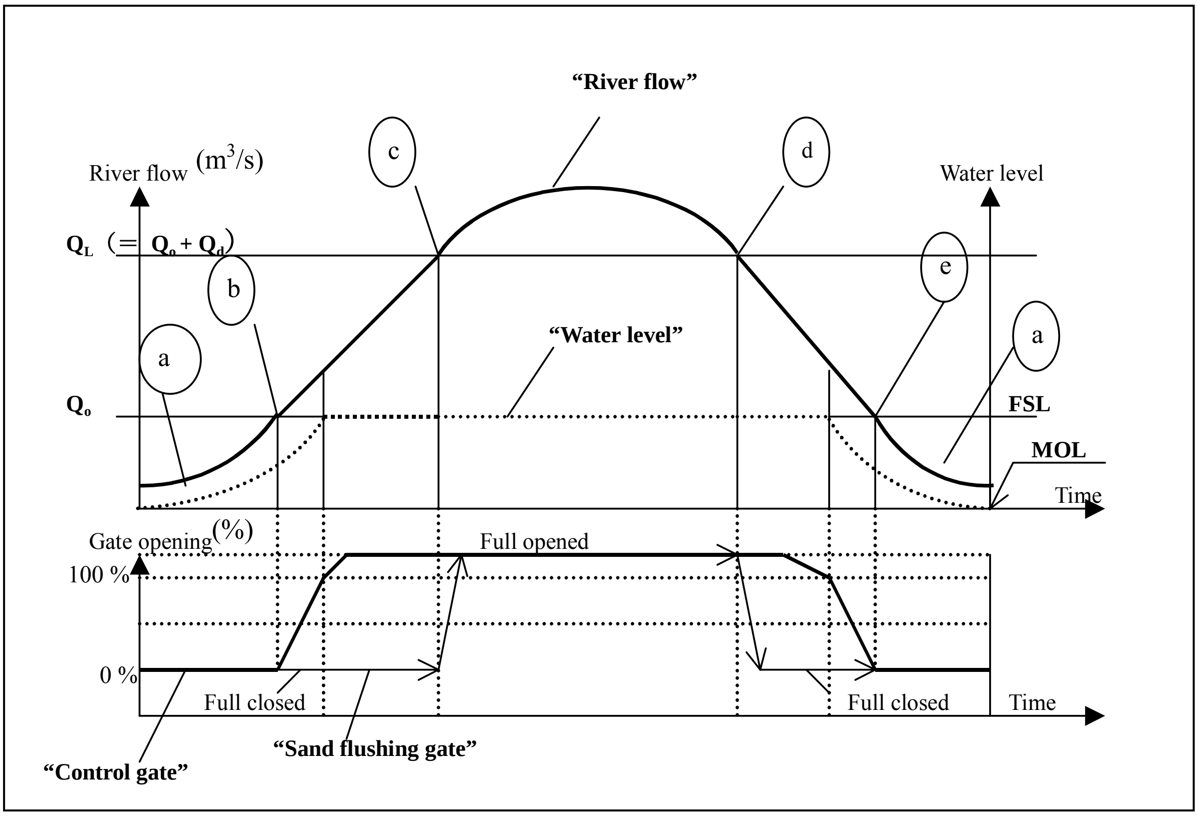
In Japan, it has been recently required to achieve appropriate and precise gate operations for the water intake and the mandatory release of water, being subject to the quantity of river flow and the regulations for utilization of river flow. In reference to the run-of-river-type hydroelectric power stations in Japan, the control of water intake and the mandatory release of water are explained as follows:



Gates and Water Level Gauges in Intake

Source: Compiled by JICA Study Team

Figure 4.4.1 Gates and Water Level Gauge in Intake



Source: JICA Study Team

Figure 4.4.2 Operation of Control Gate and Sand Flushing Gate

It is noted that in respect of the gate operation for the intake gate of Zi Chaung power station, which is located about 3.4 km from the powerhouse, the daily gate operation is not necessary as there are no requirements for mandatory release of water and there are a side-spillway just downstream of the intake gate and another side-spillway in the regulating pond neighboring the powerhouse.

Procedure of Gate Operation:

Step-1 (See a in Figure 4.4.2)

Operation of Control Gate to Secure Mandatory Release of Water for Irrigation and Environmental Requirements:

In the case when the river flow (Q_r) is smaller than the mandatory water for irrigation and environmental requirements (Q_o), the control gate of the intake should be operated to secure the mandatory release of water for irrigation and environmental requirements and to head for achievement of the FSL (Full Supply Water Level) in the upstream of the overflow weir.

$\boxed{\text{Step-2}}$ (See b in Figure 4.4.2)

Operation of Control Gate to Maintain FSL:

In the case when the river flow (Q_r) is greater than the mandatory water for irrigation and environmental requirements (Q_o) and it is smaller than the total of the mandatory water for irrigation and environmental requirements (Q_o) and the design discharge of waterway for power generation (Q_d), the control gate of the intake will be gradually opened to the full opening and maintain the FSL.

$\boxed{\text{Step-3}}$ (See c in Figure 4.4.2)

Opening of Sand Flushing Gate:

In the case when the river flow (Q_r) becomes greater than the discharge limit (Q_L), i.e. the total of the mandatory water for irrigation and environmental requirements (Q_o) and the design discharge of waterway (Q_d), the full opening of the control gate will continue to keep the FSL and opening of the sand flushing gate will start for the purpose of sand flushing.

$\boxed{\text{Step-4}}$ (See d in Figure 4.4.2)

Closing of Sand Flushing Gate:

In the case when the river flow (Q_r) becomes less than the discharge limit ($Q_L = Q_o + Q_d$), the sand flushing gate should be fully closed and the control gate of the intake should be gradually closed.

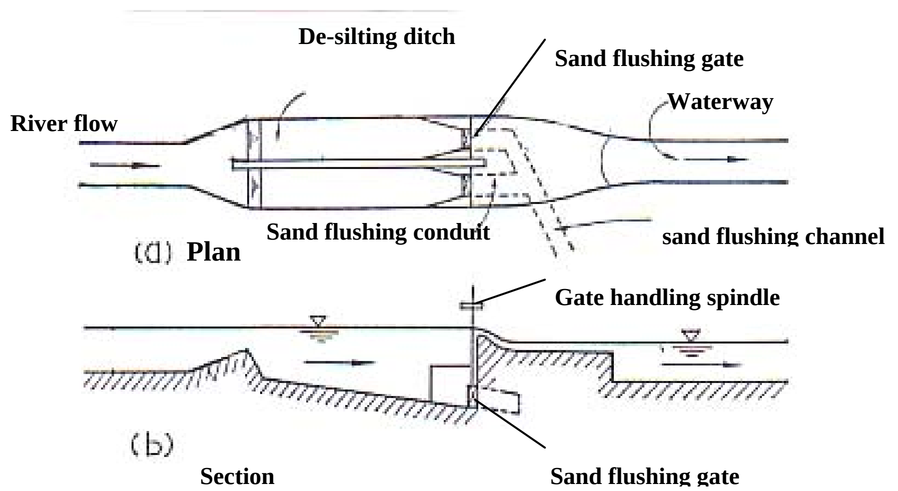
$\boxed{\text{Step-5}}$ (See e in Figure 4.4.2)

Operation of Control Gate to Secure Mandatory Release of Water for Irrigation and Environmental Requirements :

In the case when the river flow (Q_r) becomes less than the mandatory water for irrigation and environmental requirements (Q_o), the control gate of the intake should be fully closed for achievement of (Q_o).

4.5 Operation for Sediment Flushing

De-silting basins are provided for the settling and flushing of sediment particles to reduce abrasion of the waterway, penstock pipe, hydraulic turbines, etc., and eventually to achieve stable and efficient operation of the hydroelectric power plant.



Source: Compiled by JICA Study Team

Figure 4.5.1 De-silting Basin

(1) Operational Check Points of Sand Flushing Gates

<u>For Normal Conditions</u>

- Sand flushing gates are provided for flushing out sediments.
- Upon observation of sediment deposits, sand-flushing gates are to be opened for flushing of sediment deposits.
- No operation of sand-flushing gates for a long time may cause serious sediment deposits which may not be able to be removed by operation of the sand-flushing gates. It is essential to carry out periodical sand flushing.

<u>For Flood Conditions</u>

- During and after floods, the sand flushing gates are to be opened for flushing of sediments deposits.
- Also during and after floods, drift leaves and driftwoods may cause clogging of the screen, and eventually may cause crushing of the screens due to the water pressure acting on the upstream surface of the screens resulting from the raised water level

by clogging. Accordingly, together with operation of the sand flushing gates it is very important to remove such drift leaves and driftwoods from the screen.

4.6 Countermeasures against Sedimentation

The application of reliable and efficient methods of sediment control and removing sediment from waterways is one of the most important considerations for the successful utilization of water resources in a hydroelectric power scheme. It is generally expected that appropriate operations of both the sand-flushing gates installed in neighboring dams or weirs and the sand flushing gates in the de-silting basin will effectively return sediment back to the river.

However, it is a fact that run-of-river projects and storage schemes with large dams for hydroelectric power projects have suffered from serious sediment deposits that have not been removed by the sand-flushing gates.

To overcome the severe sediment transportation problems, a recent study* demonstrated sediment control methods utilizing a suction head, i.e. (i) a sand flushing through the use of a *'Saxophone' suction head** and (ii) sand flushing through a *Slotted pipe sediment sluicer**. According to the introduction of the method, it is said that sediment control can be accomplished without interruption of the normal operation of reservoirs and de-silting facilities. In addition, these sand flushing techniques do not cause interruption of the power generation since de-watering of the de-silting basin is not necessary.

(1) Sand Flushing through 'Saxophone' Suction Head

- The 'Saxophone' suction head has open slots in the invert portion of the pipes.
- Upon opening the valve, a suction head caused by the water head between the Pond and outlet will give rise to a velocity of water inside the pipe. The velocity of water will give the inside of the pipe negative pressures to absorb sediment and it will transport the sediment to the outside of the pipe.
- Once absorption of sediment starts, collapse of the sediment deposits will occur automatically and it will supply sediment to the open slots, which will cause repeated absorption of sediment and its transportation.
- In the case when the open slots are provided in the crown portion of pipes, the open slots will be plugged by sediment during collapse of sediment deposits. Accordingly, the open slots are to be provided in the invert portion of pipes.
- It is a special property of the 'Saxophone' suction head that sediment will not cause plugging of the absorption pipe even with the occurrence of settlement of the 'Saxophone' suction head.

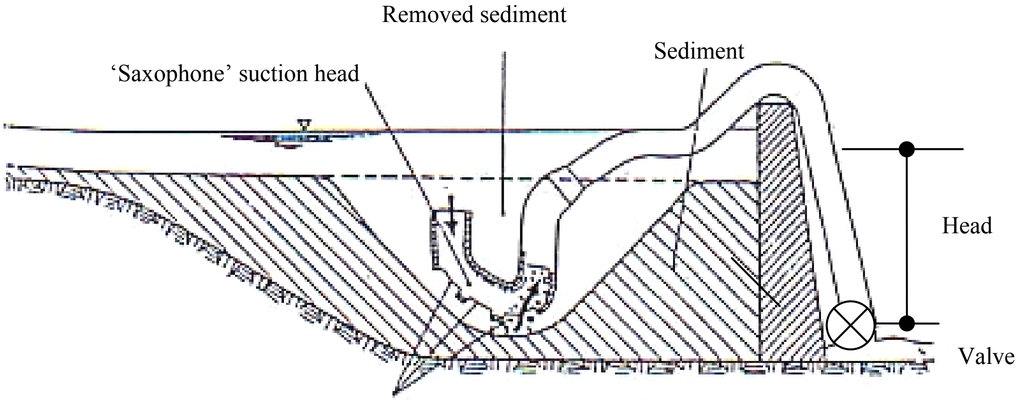


Figure 4.6.1 "Saxophone" Suction Head

(2) Sand Flushing through Slotted Pipe Sediment Sluicer

- Open slots are provided along the invert portion of pipes.

- The sand flushing through the slotted pipe sediment sluicer is a fixed sluicing system to utilize the water head between the pond and the outlet.

- Upon opening the valve, a suction head caused by water head between the pond and outlet will give rise to a velocity of water inside the pipe. The velocity of water will give the inside of the pipe negative pressures to absorb sediment and it will transport the sediment to the outside of pipe.

- Once absorption of sediment starts, collapse of the sediment deposits will occur automatically and will supply sediment to the open slots, which will cause repeated absorption of sediment and its transportation.

- When the open slots are provided in the crown portion of pipes, the open slots will be plugged by sediment during collapse of sediment deposits. Accordingly, the open slots are to be provided in the invert portion of pipes.

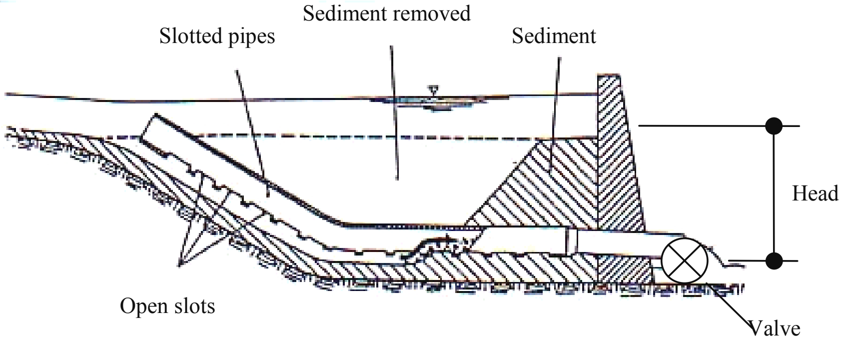


Figure 4.6.2 "Flute" Pipe

Source: See foot notes

(3) Calculation for Sand Flushing Capacity of 'Saxophone' Suction Head

An example of calculating the sand flushing capacity of a 'Saxophone' Suction Head is shown below.

Example of a Calculation of the Sand Flushing Capacity of 'Saxophone' Suction Head

(i) Pipe ϕ = 15 cm, L = 20 m

$$\boxed{V = \sqrt{\frac{2g \cdot H}{1 + f_e + N \cdot f_b + f}}}$$

f_e = 1.00 (inlet loss), $N \cdot f_b$ = 0.40 (bend loss), $f = f_r \cdot L/\phi$ = 4.5 (friction loss)

where $f_r = 12.7g \cdot n^2 \cdot \phi^{1/3}$ = 0.03373 and n = 0.012 (roughness coefficient)

When H (head) = 1.5 m, V (velocity) = 2.06 m/s

(ii) Pipe ϕ = 15 cm, L = 20 m, H = 1.5 m, V = 2.06 m/s

Q = 0.036 m^3/s (= 2.19 m^3/min = 131 m^3/hr): discharge flushed

Sand = 131 m^3/hr x 3%*[1] = 3.9 m^3/hr (= 31 m^3/day*[2]): sand volume flushed

Note; *[1]: 3% of the discharge is applied in reference to the results of test operation for 'Saxophone' Suction Head as shown in Table A1.14.9 of Appendix 1-14 in Vol.6 Supporting Report.

*[2]: 8 hours is applied for the working hour.

In addition, the sand flushing capacities of a 'Saxophone' Suction Head for several diameters of pipe and several suction heads are calculated and shown in Table 4.6.1.

Notes; Source: "Sediment control: recent developments for headworks" by D.K. Lysne, N.R.B.Olsen, H.St∅le and T.Jacobsen, The Norwegian Institute of Technology, Trondhelm, Norway, pp.46 – 49 of Hydropower and Dam March 1995 (See Appendix 1-12 of Supporting Report in Volume 6).

Table 4.6.1 Sand Flushing Capacity of 'Saxophone' Suction Head

Dia. (m)	L (m)	H (m)	V (m/s)	Q (m^3/s)	Q (m^3/hr)	Sand (m^3/hr)	Sand (m^3/day)
0.1	10	1.5	2.17	0.017	61	1.83	15
0.1	10	3	3.06	0.024	86	2.58	21
0.1	20	1.5	1.7	0.013	47	1.41	11
0.1	20	3	2.41	0.019	68	2.04	16
0.1	30	1.5	1.45	0.011	40	1.2	10
0.1	30	3	2.05	0.016	58	1.74	14
0.1	40	1.5	1.28	0.01	36	1.08	9
0.1	40	3	1.82	0.014	50	1.5	12
0.1	50	1.5	1.16	0.009	32	0.96	8
0.1	50	3	1.65	0.013	47	1.41	11
0.1	60	1.5	1.07	0.008	29	0.87	7
0.1	60	3	1.52	0.012	43	1.29	10
0.15	10	1.5	2.51	0.044	158	4.74	38
0.15	10	3	3.56	0.063	227	6.81	54
0.15	20	1.5	2.06	0.036	130	3.9	31
0.15	20	3	2.92	0.052	187	5.61	45
0.15	30	1.5	1.79	0.032	115	3.45	28
0.15	30	3	2.54	0.045	162	4.86	39
0.15	40	1.5	1.61	0.028	101	3.03	24
0.15	40	3	2.27	0.04	144	4.32	35
0.15	50	1.5	1.47	0.026	94	2.82	23
0.15	50	3	2.08	0.037	133	3.99	32
0.15	60	1.5	1.36	0.024	86	2.58	21
0.15	60	3	1.92	0.034	122	3.66	29

Source: Calculation by JICA Study Team

(4) Field Tests of 'Saxophone' Suction Head and 'Flute' Pipe

The hydraulic functions of the 'Saxophone' suction head and 'Flute' pipe were confirmed in the 7^{th} and 8^{th} field investigations in Myanmar using trial devices made in the field. However, the sand flushing efficiency of 'Saxophone' suction head was at about 3% of the discharge in reference to Table A1.14.9 of Appendix 1-14 in Vol.6 Supporting Report.

Consequently, it was confirmed that further field tests would be necessary to achieve practicable flushing devices based on these suction head concepts.