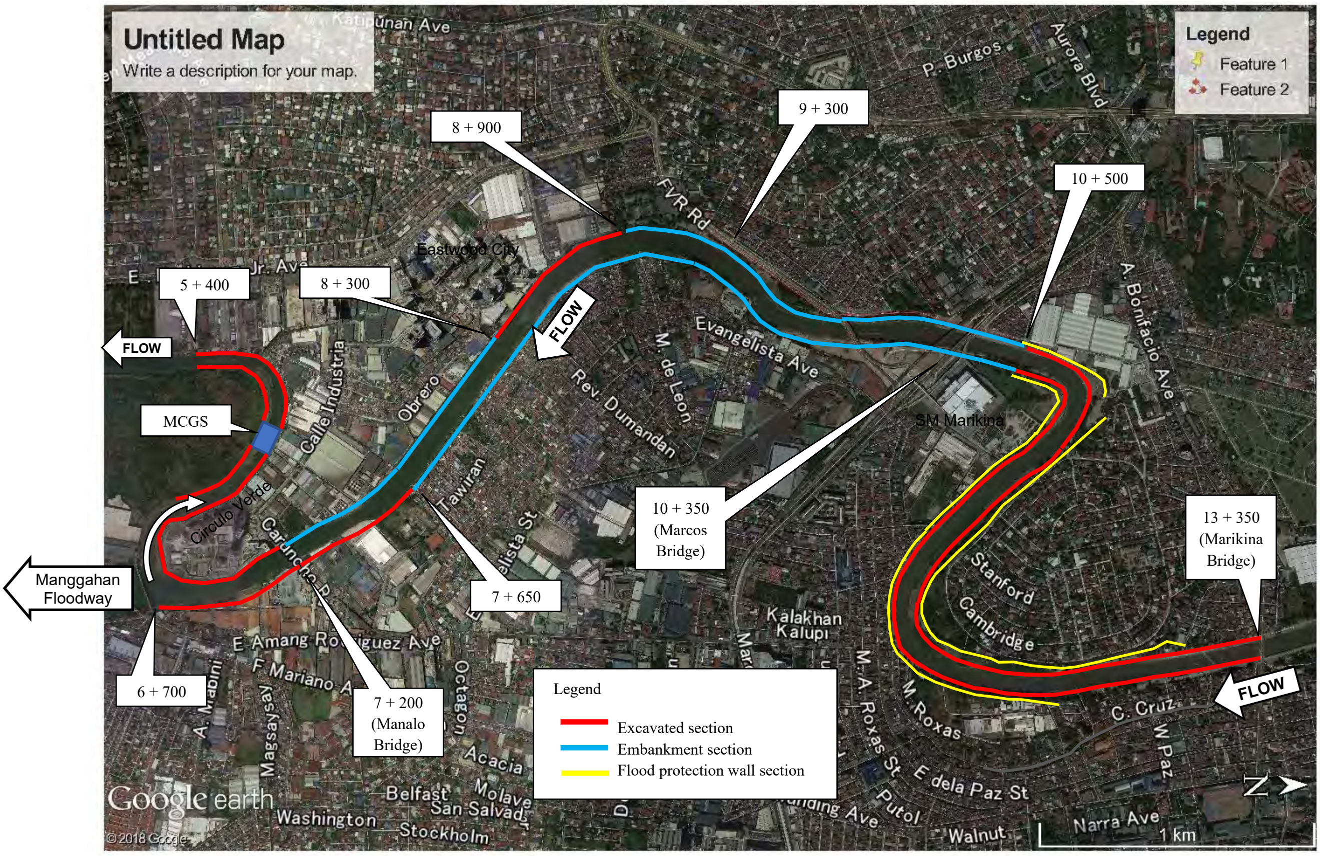
CHAPTER 6 BASIC STUDY AND DESIGN OF RIVER STRUCTURES

6.1 Basic Design of River Channel (Dikes, Revetments, and Revetment for Low Water Channel)

6.1.1 Outline of Basic Design of River Channel

Table 6.1.1 and **Figure 6.1.1** below show the basic design results for river channels. The river channel is divided into two sections: the section requiring reinforcement with embankment and the section in which embankment is not necessary but requires excavation. For both sections, Steel Sheet Piles (SSPs) to protect the low-water channel will be provided. Also, concrete retaining walls will be provided above high-water channels.



Source: Study Team (added on Google map)

Figure 6.1.1 Sections of River Improvement Works in PMRCIP-IV

Table 6.1.1 Basic Design Principles of River Sections, PMRCIP-IV

Station	Riverbed Width	Low-Water Revetment	Structure of Embankments and Revetments	
			Left Bank	Right Bank
Sta. 5+400 to 6+700 Downstream end to Rosario diversion point	40 m	Soil Channel / SSP revetment	Sta. 5+400 to 6 + 350: cutting/concrete dike	Sta. 5+400 to 5+800: cutting
			Sta. 6+350 to 6 + 600: existing revetment	Sta. 5+800 to 6+700: concrete revetment
Sta. 6+700 to 10+500 Rosario diversion point to upstream of Marcos bridge	90 m	SSP revetment	Sta. 6+700 to 7+650: concrete revetment	Sta. 6+700 to 7+200: concrete revetment
				Sta. 7+200 to 8+300: embankment, concrete revetment
			Sta. 7+650 to 10+500: embankment, concrete revetment	Sta. 8+300 to 8+900: concrete revetment
				Sta. 8+00 to Sta. 9+00~10: embankment, concrete revetment
Sta. 10+500 to 13+350 Upper Marcos Bridge to Marikina Bridge	80 m	SSP revetment	Sta. 10+500 to 13+3500: flood protection wall (parapet walls, raising of sting walls)	Sta. 10+500 to 13+3500: flood protection walls (parapet walls, raising of existing walls)

Source: Study Team

6.1.2 Setup of Design Basic Concept

6.1.2.1 Horizontal Layout

The design area of Marikina River locates in the Metro Manila cities of Pasig, Quezon, and Marikina. In the entire riparian area of the design sections, intensive land use driven by urbanization is in progress. Since it is difficult to obtain new land for the river improvement project, a new river channel will be aligned on the original layout, and no drastic change of the channel layout will be made.

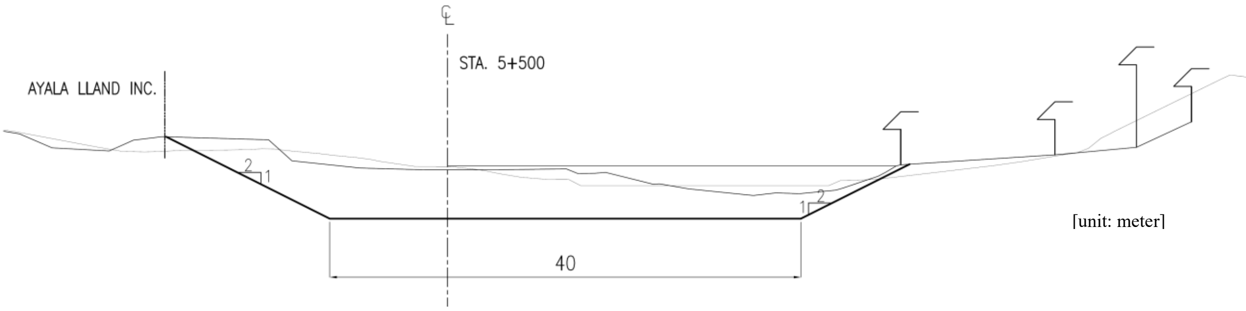
6.1.2.2 Standard Cross Section

(1) Sta. 5+400 to Sta. 6+700 (Downstream of MCGS - MCGS - Rosario Diversion)

Table 6.1.2 Standard Cross Section of Sections

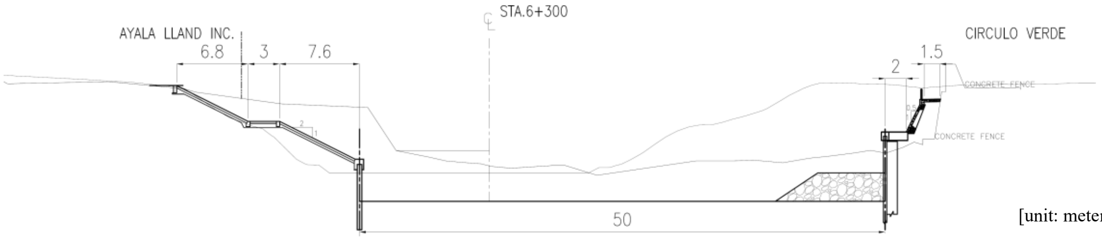
Station	Riverbed Width	Structure of River Channel and Proposed Revetment	
		Left Bank	Right Bank
Sta. 5+400 to 5+800 Downstream design endpoint to the downstream end of the MCGS revetment (**Figure 6.1.2**)	40m	Channel form: Excavated channel	Channel form: Excavated channel
		Revetment structure: Soil channel, slope 1: 2.0	Revetment structure: Soil channel, slope 1: 2.0
Sta. 5+900 to 6+050 MCGS and its upstream and downstream revetments	43.5m	Channel form: Excavated channel	Channel form: Excavated channel
		Revetment structure: SSP+20% concrete revetment	Revetment structure: SSP+20% concrete revetment
Sta. 6+050 to 6+350 Upstream end of MCGS revetment - Rosario Weir (**Figure 6.1.3**)	43.5m~ 50m	Channel form: Excavated channel	Channel form: Excavated channel
		Revetment structure: SSP+20% concrete revetment	Revetment structure: SSP+1:0.5 concrete revetment
Sta. 6+350 to 6+600 Rosario Diversion	50m~ 100m	Channel form: Excavated channel	Channel form: Excavated channel
		revetment structure: Present state (low-water channel excavation)	Revetment structure: SSP+1:0.5 concrete revetment

(Note) "20% concrete revetment" means that the slope of the revetment surface is V: H = 1: 2.0
Source: Study Team



Source: Study Team

Figure 6.1.2 Standard Cross Section between Sta. 5+400 and 5+800 (Sta. 5+500)



Source: Study Team

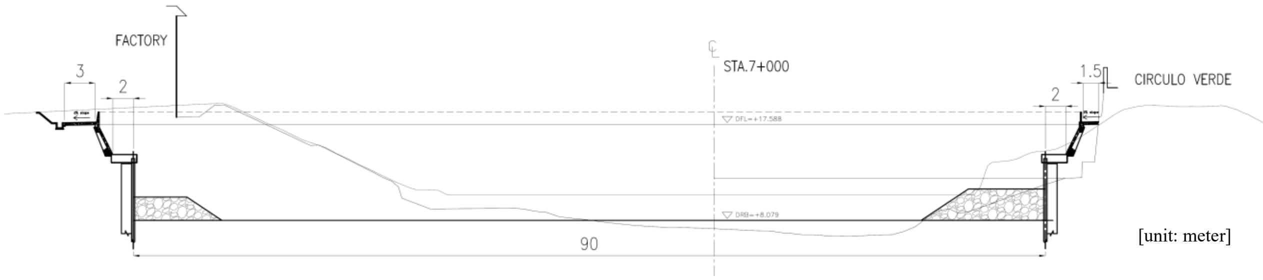
Figure 6.1.3 Standard Cross Section between Sta. 6+050 and 6+600 (Sta. 6+300)

(2) Sta. 6+700 to Sta.10+500 (Rosario Diversion to Marcos Bridge)

Table 6.1.3 Standard Cross Section of Sections

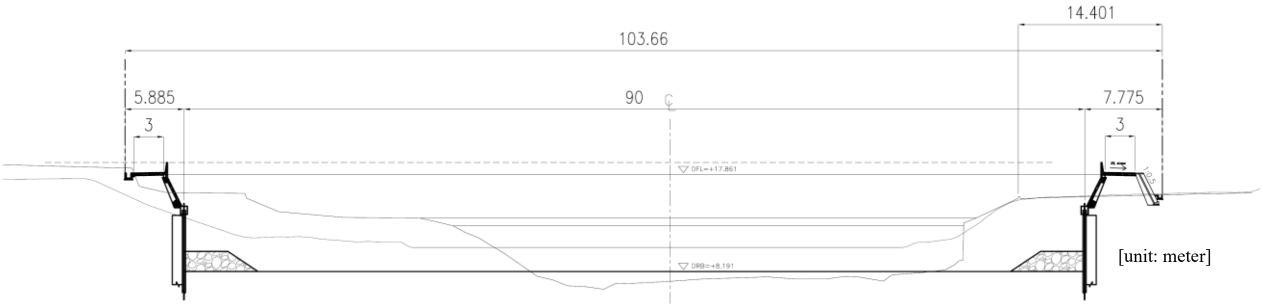
Station	Riverbed Width	Structure of River Channel and Proposed Revetment	
		Left Bank	Right Bank
Sta. 6+700 to 7+200 Rosario Weir to Manalo Bridge (**Figure 6.1.3**)	90 m	Channel form: Excavated channel	Channel form: Excavated channel
		Revetment structure: SSP+1:0.5 concrete revetment	Revetment structure: SSP+1:0.5 concrete revetment
Sta. 7+200 to 7+650 Upstream of the Manalo Bridge No small step due to site restrictions (**Figure 6.1.4**)	90 m	Channel form: Excavated channel	Channel form: Embankment
		Revetment structure: SSP+1:0.5 concrete revetment	Revetment structure: SSP+1:0.5 concrete revetment, Land side: 1:0.5 concrete block retaining wall
Sta. 7+650 to 8+300 Upstream of the Manalo Bridge to Eastwood City	90 m	Channel form: Embankment	Channel form: Embankment
		Revetment structure: SSP+1:0.5 concrete revetment Land side: 1:0.5 concrete block retaining wall	Revetment structure: SSP+1:0.5 concrete revetment, Land side: 1:0.5 concrete block retaining wall
Sta. 8+300 to 8+900 Eastwood City to Camp Atienza River Wall on the left bank is under construction by Pasig City	90 m	Channel form: Embankment	Channel form: Excavated channel
		Revetment structure: SSP+1:0.5 concrete revetment, Land side: 1:0.5 concrete block retaining wall	Revetment structure: SSP+1:0.5 concrete revetment
Sta. 8+900 to 10+500 Camp Atienza to Marcos Bridge (**Figure 6.1.5**)	90 m	Channel form: Embankment	Channel form: Embankment
		Revetment structure: SSP+1:0.5 concrete revetment, land side: 1:0.5 concrete block retaining wall	Revetment structure: SSP+1:0.5 concrete revetment, Land side: 1:0.5 concrete block retaining wall

Source: Study Team



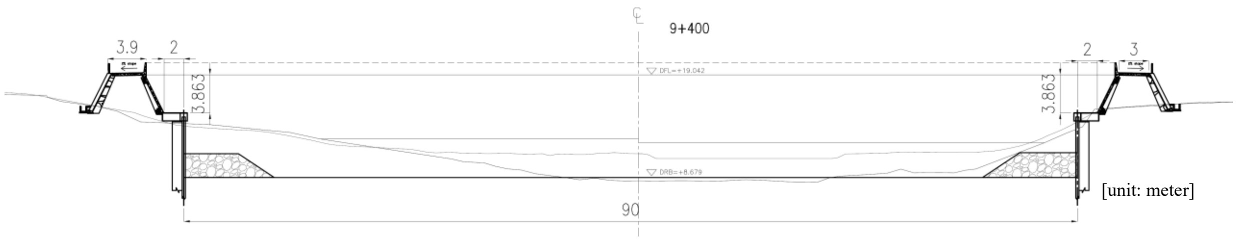
Source: Study Team

Figure 6.1.4 Standard Cross Section between Sta. 6+700 and 7+200 (Sta. 7+000)



Source: Study Team

Figure 6.1.5 Standard Cross Section between Sta. 7+200 and 7+650 (Sta. 7+450)



Source: Study Team

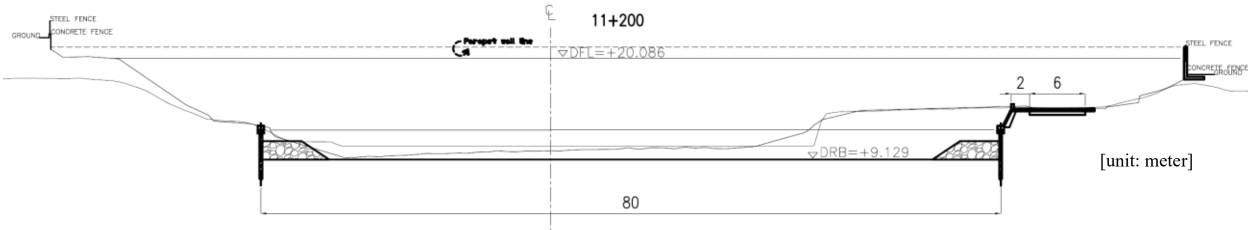
Figure 6.1.6 Standard Cross Section between Sta. 7+650 and 8+300 (Sta. 9+400)

(3) Sta. 10+500 to Sta. 13+350 (Marcos Bridge to Marikina Bridge (Sto. Nino))

Table 6.1.4 Standard Cross Section of Sections

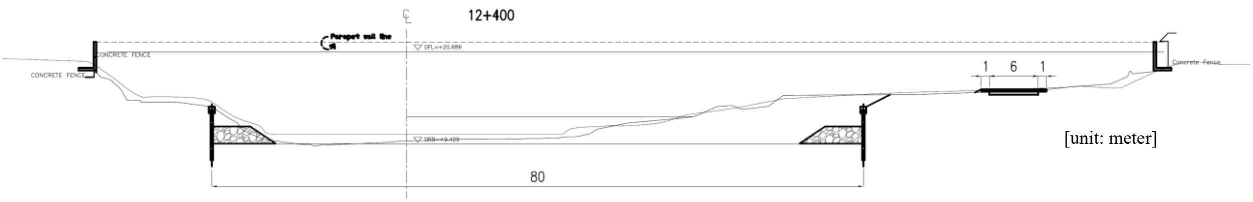
Station	Riverbed Width	Structure of River Channel and Proposed Revetment: Left Bank	Structure of River Channel and Proposed Revetment: Right Bank
Sta. 10+550~11+200, Sta. 11+700~12+000 (**Figure 6.1.6**) Enough land width is not acquirable on the right bank along the river park	80 m	Channel form: excavated channel	Channel form: excavated channel
		Revetment structure: SSP+1:0.5 concrete revetment, parapet wall	Revetment structure: SSP+1:0.5 concrete revetment, river wall raising and riverside road
11+200~11+700,12+000~12+500 (**Figure 6.1.7**) Enough land width is acquirable on the right bank along the river park	80 m	Channel form: excavated channel	Channel form: excavated channel
		Revetment structure: SSPs, parapet walls	Revetment structure: SSP+20% excavation, river wall raising and riverside road
12+500~13+100 (**Figure 6.1.8**) Parapet walls are not required due to enough ground height on the left bank	80 m	Channel form: excavated channel	Channel form: excavated channel
		Revetment structure: SSPs Parapet walls are not necessary since the background is high.	Revetment structure: SSP + 1:0.5 concrete revetment, river wall raising and riverside road
13+100~13+350 (Marikina Bridge) (**Figure 6.1.10**) Parapet walls are not required due to enough ground height on the left bank	80 m	Channel form: excavated channel	Channel form: excavated channel
		Revetment structure: SSPs Parapet walls are not necessary since the background is high.	Revetment structure: SSP + 1:0.5 concrete revetment, riverbank road (Since the ground height is not enough at the 100+13 cross section, additional countermeasures are required.)

(Note) 20% means that the slope of the revetment surface is V: H = 1: 2.0
Source: Study Team



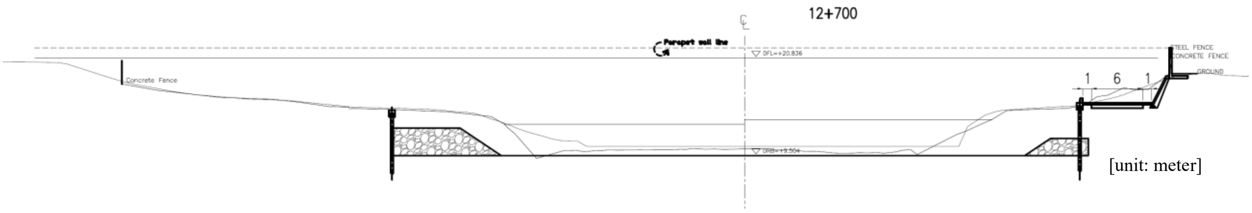
Source: Study Team

Figure 6.1.7 Standard Cross Section between Sta. 10+550 and 11+200 (Sta. 11+200) in Case of Limited Space



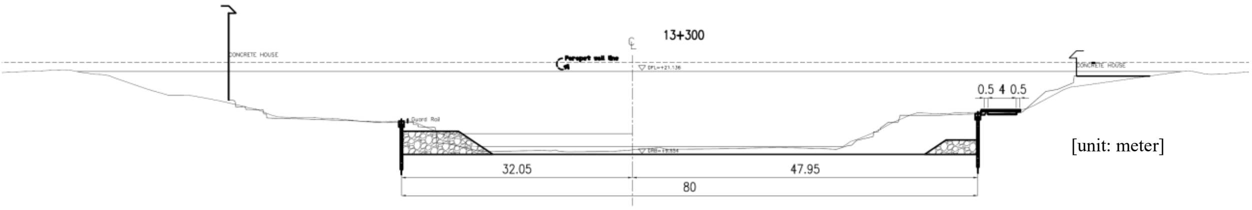
Source: Study Team

Figure 6.1.8 Standard Cross Section between Sta. 12+000 and 12+500 (Sta. 12+400) in Case of Sufficient Space



Source: Study Team

Figure 6.1.9 Standard Cross Section between Sta. 12+500 and 13+100 (Sta. 12+700) in Case Without Wall on the Left Bank



Source: Study Team

Figure 6.1.10 Standard Cross Section between Sta. 13+100 and 13+350 (Sta. 13+300) in Case Without Freeboard

6.1.2.3 Confirmation of Design Floodwater Level (DFL)

The following table shows the DFL and the elevation of the riverbank at the representative cross-section in each section.

Table 6.1.5 DFL and Riverbank Elevation in the Representative Cross-Section of Sections

Station	DFL (EL. m)	Riverbank Elevation (EL. m)		Channel Form		Remarks
		Left Bank	Right Bank	Left Bank	Right Bank	
5 + 400	14.64	17.42	21.11	excavation	excavation	
6 + 050	17.40	19.15	17.65		embankment	The elevation of some section on the right bank is lower/
6 + 300	17.40	19.27	20.94		excavation	
6 + 700	17.41	18.65	20.91			
7 + 000	17.59	19.47	23.2			
7 + 200	17.71	19.66	15.78		embankment	Manalo Bridge
7 + 450	17.86	18.49	15.67			
7 + 650	17.98	17.84	16.25	embankment		
8 + 300	18.38	14.47	13.59			
8 + 900	18.74	17.92	20.0		excavation	
9 + 400	19.04	16.94	16.59			Olandes Sewage Treatment Plant
9 + 600	19.16	17.27	16.52			
9 + 900	19.35	21.93	16.77			Macapagal Bridge
10 + 300	19.59	15.71	15.32			Marcos Bridge
10 + 500	19.71	16.91	17.55			
11 + 200	20.09	20.02	19.82	excavation	excavation	insufficient height will be protected by parapet walls and raising of existing structures
12 + 400	20.69	20.06	18.96			same as above
12 + 700	20.84	20.54	20.84			same as above
13 + 100	21.04	16.62	20.26			Measures other than parapets should be considered.
13 + 300	21.14	17.21	20.52			The background of both the right and left side are higher than the riverbank.
13 + 350	21.16	22.61	18.48			Marikina Bridge

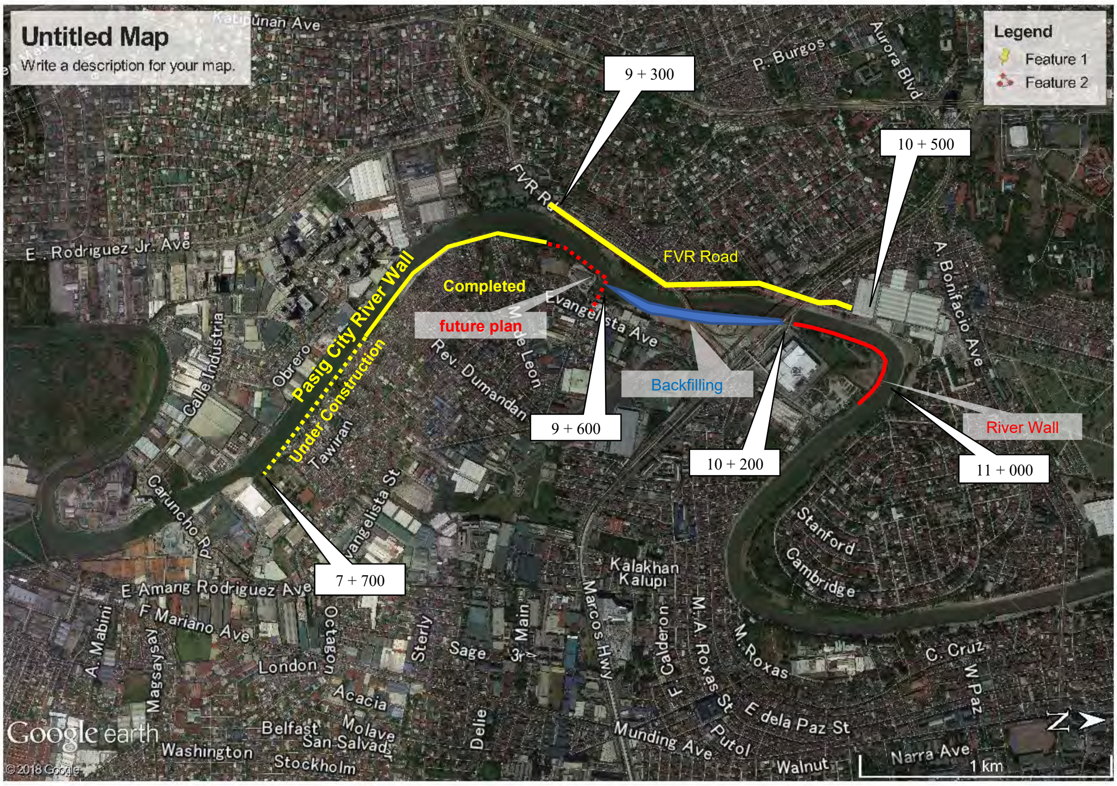
Source: Study Team

For sections where embankment works will be implemented, issues and the concepts for the countermeasures are shown as follows.

The ground height is not sufficient on the right bank from Sta. 9+300 (the downstream of the Olandes Sewage Treatment Plant) to Sta. 10 + 500 (directly upstream of the Marcos Bridge). Thus, embankment is required. The corresponding section is located in Marikina City. Since the city puts a high priority on providing easily approachable rivers, high levees may not be positive options in terms of city planning. Hence, raising the FVR Road would be considered. In this case, additional road improvement works adjacent to the FVR Road will be necessary.

Similarly, the ground height is not sufficient on the left bank from Sta. 7+700 to Sta. 10+500 (directly upstream of the Marcos Bridge). Thus, the embankment of 1~5 m in height is required. Between Sta. 7+700 to Sta. 9+600 in Pasig City, a revetment is being constructed by Pasig City. The crown height of the revetment under construction is only EL. 18.0m. However, the DFL in the section is EL. 18.8m; therefore, the height of the revetment is insufficient, and additional raising of the crown height would be considered.

The left bank between Sta. 9+600 and Sta. 10+200 where land-reclamation development is underway is located in Marikina City. In this section, the width of the river is only approximately 55m at the narrowest cross-section. Since there is no space to widen the right bank from Sta. 9+800 to Sta. 10+000, acquiring sufficient river width by excavating the left bank.



Source: Google Earth postscript

Figure 6.1.11 Current Construction Condition of the Project Area

6.1.3 Basic Design of Revetment for Low Water Channel

6.1.3.1 Type of Revetment for Low Water Channel

Since excavation and widening of the low water channel are necessary in all sections except the downstream section of MCGS, the steel sheet pile revetment will be constructed.

Table 6.1.6 Type of Revetment for Low Water Channel for Sections

Station	Position	Revetment for Low Water Channel
Sta. 5+400 to Sta. 5+800 Left Bank Sta. 5+400 to Sta. 5+620 Right Bank	From downstream design endpoint to the downstream end of the MCGS revetment From downstream design endpoint to the confluence of existing ditch and Marikina River	No revetment Except for the stretch from STA.5+423 to 5-5+581.0, no revetment
Sta. 5+620 to Sta. 5+900 Right Bank	From the confluence of existing ditch and Marikina river to downstream end of MCGS	SSP revetment
Sta. 5+900	MCGS	Concrete revetment or Main body of MCGS
Sta. 6+035 to Sta. 13+350	From upstream end of MCGS revetment to Marikina Bridge	SSP revetment

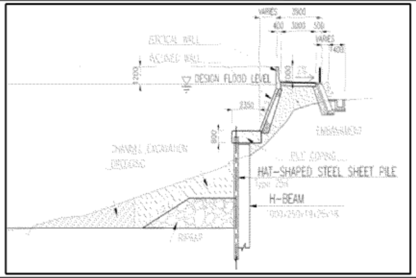
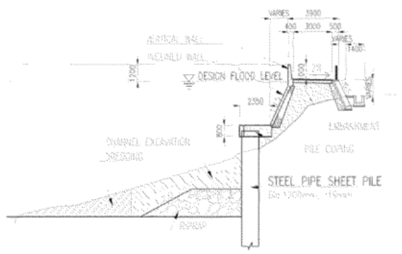
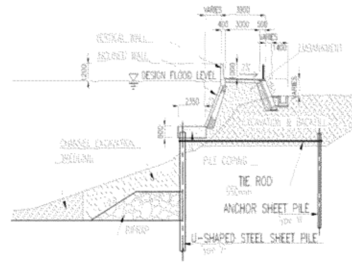
Source: Study Team

The SSP revetment has high flexibility for all places and is suitable as revetment in the following aspects:

- It protects the river area from lateral erosion.
- Temporary works such as coffering, drainage and drying of the construction area are not necessary, so that construction time is shortened.
- By putting concrete coping on the SSP revetment, it is relatively easy to secure the necessary crown height.
- The size and extension of SSPs can be flexibly reviewed according to multiple conditions, such as weak or tough foundation ground as well as securing adequate water depth for ship approach without expanding the construction area.

According to the comparison shown in **Table 6.1.7**, SSP revetment structure will be the self-supporting H-shaped SSP + H-shaped steel.

Table 6.1.7 Comparative Selection Table of Revetments for Low Water Channel

	Case 1: Self-Supporting Hat-Shaped Steel Sheet Pile (SSP)+H-Shaped Steel		Case2: Self-Supporting Steel Pipe Sheet Pile (SPSP)		Case3: Braced SSP with Tie Rods	
Image						
SSP Structure	Hat-Shaped SSP + H-Shaped Steel		Steel Pipe Sheet Pile (SPSP)		SSP+ Counter SSP + Tie Rods	
Construction Results	In the Philippines, Phase III applied this structure. Also, there are several applications in small urban rivers in Japan.	○	Both in the Philippines and Japan, SPSP are frequently used in port & harbor constructions.	○	There are some construction results in the Philippines. In Japan, the structure is applied to revetments for low-water channels in relatively large rivers such as Arakawa River	○
Land Acquisition	Land acquisition is minimum	○	Land acquisition is minimum	○	The land area for acquisition is larger due to the construction of the counter SSPs and tie rods.	×
River Cross-Sectional Area	The maximum cross-sectional area within the current river area can be obtained while ripraps decrease some of the river areas.	○	The maximum cross-sectional area within the current river area can be obtained while ripraps decrease some of the river areas.	○	The maximum cross-sectional area within the current river area can be obtained while ripraps decrease some of the river areas.	○
Maintainability	The front of SSP can be inspected and maintained, while inspection and maintenance behind the SSP and H-shaped steel are difficult.	○	The front of SPSP can be inspected and maintained, while inspection and maintenance behind the SPSP are difficult.	○	The front of SPSP can be inspected and maintained, while inspection and maintenance of tie rods and counter SSPs are extremely difficult.	△
Workability	No underwater construction works needed; good workability	○	No underwater construction works needed; good workability	○	No underwater construction works needed; good workability. However, extensive drilling and backfilling are required for the construction of counter SSPs and tie rods.	△
Economic Efficiency	Construction cost of substructure (SSP Revetment) Hat-Shaped SSP + H-Shaped Steel ****** Peso/m	2	Construction cost of substructure (SPSP Revetment) SPSP ****** Peso/m	3	Construction cost of substructure (SSP Revetment) SSP ****** Counter SSP ****** Tie Rod ****** Earth Works ****** Total ****** Peso/m	1
	Since it is a self-supporting type, the steel material becomes large and the cost of the material becomes high. Since the amount of steel is smaller than that of a steel pipe sheet, it is economically efficient.	△	Since it is a self-supporting type, the steel material becomes large and the cost of the material becomes high.	△	Due to the braced structure, the steel will be smaller than the other two plans, but additional drilling and backfill costs are needed. It is the least expensive of the three options.	○
Judgement	**【Adopted】** It is less economical than Case3 but can be constructed without land acquisition. Since it is more economical than Case 2, it is adopted as a proposal. It also meets the conditions for STEP projects.		【Rejected】 It can constructed without land acquisition, but it is not adopted because it is the most expensive choice.		【Rejected】 It is the most economical choice, but it requires extensive land acquisition behind the embankment for construction. Since land acquisition is very difficult, it is practically impossible to adopt.	

Note: Cost is not presented due to the prior released version.
Source: Study Team

6.1.3.2 Consideration of Liquefaction Risk

The risk of liquefaction in the design section was evaluated. The evaluation method is written in Chapter 11, Section 11.7. In the borings data (DD-BH and DD-BH-G) with geological test results which is necessary to judge liquefaction, soil layers with liquefaction risk were extracted from alluvial deposits (As and Ac layers). In addition, for the extracted soil layer liquefaction risk was evaluated.

The criteria for the layer extraction are ;

1) Saturated soil layer with depth less than 20 m below the ground surface and having ground water level higher than 10 m below the ground surface.
2) Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index, I_P, less than 15, even if FC is larger than 35%.
3) Soil layer having a mean particle size (D_{50}) of less than 10 mm and a particle size at 10% passing (D_{10}) (on the grading curve) is less than 1 mm.

Source: BSDS, DPWH, P6 -3

Table 6.1.8 Extracted Layer for Liquefaction Evaluation (Left Bank)

Borehole No.	Ground Water Depth(m)	Sample No.	Depth (m)	Middl Depth	Geological Classification	Grain size Fc(%)	Ip	D50(mm)	D10(mm)	Saturated Layer	Need Check
BH-G01	3.18	ss-1	1.00 – 1.45	1.23	As	31.0	–	–	–		
		ss-2	2.00 – 2.45	2.23	As	59.0	8	–	–		
		ss-3	3.00 – 3.45	3.23	As	42.0	9	0.16	0.01	○	○
		ss-4	4.00 – 4.45	4.23	As	19.0	–	0.41	<0.1	○	○
		ss-5	5.00 – 5.45	5.23	As	21.0	–	0.41	<0.1	○	○
BH-G04	0.80	ss-1	1.00 – 1.45	1.23	As	36.0	–	–	–		
		ss-2	2.00 – 2.45	2.23	As	17.0	–	0.40	<0.1	○	○
		ss-3	3.00 – 3.45	3.23	As	24.0	–	0.25	<0.1	○	○
		ss-4	4.00 – 4.45	4.23	As	14.0	–	0.40	<0.1	○	○
		ss-5	5.00 – 5.45	5.23	As	14.0	–	0.50	<0.1	○	○
		ss-6	6.00 – 6.45	6.23	As	18.0	–	0.45	<0.1	○	○
BH-G07	2.85	ss-1	1.00 – 1.45	1.23	As	62.0	10	–	–		
		ss-2	2.00 – 2.45	2.23	As	26.0	–	–	–		
		ss-3	3.00 – 3.45	3.23	As	42.0	9	0.15	0.01	○	○
		ss-4	4.00 – 4.45	4.23	As	27.0	8	0.40	<0.1	○	○
		ss-5	5.00 – 5.45	5.23	As	14.0	–	0.50	<0.1	○	○
		ss-6	6.00 – 6.45	6.23	As	24.0	–	0.50	<0.1	○	○
		ss-7	7.00 – 7.30	7.15	As	14.0	–	0.75	<0.1	○	○
BH-L02	5.49	ss-1	1.00 – 1.45	1.23	As	26.0	–	–	–		
		ss-3	3.00 – 3.45	3.23	As	5.0	–	2.00	0.30	○	○
		ss-5	5.00 – 5.45	5.23	As	29.0	–	1.20	<0.1	○	○
		ss-7	7.00 – 7.45	7.23	As	10.0	–	1.50	0.08	○	○
BH-L04	1.97	–	0.00 – 0.50	0.25	As	46.0	10	0.01	0.01		
BH-L07	2.553	ss-2	2.00 – 2.45	2.23	As	49.0	9	–	–		
		ss-3	3.00 – 3.45	3.23	As	6.0	–	5.00	0.30	○	○
		ss-4	4.00 – 4.45	4.23	As	6.0	–	3.20	0.20	○	○
BH-L08	2.7	ss-3	3.00 – 3.45	3.23	As	33.0	23	0.22	<0.1	○	○
BH-L09	2.751	ss-2	2.00 – 2.45	2.23	Ac	82.0	13	–	–		
		ss-4	4.00 – 4.45	4.23	As	49.0	13	0.08	<0.1	○	○
		ss-6	6.00 – 6.45	6.23	Ac	78.0	24	–	–	○	
BH-L10	3.12	ss-1	1.00 – 1.45	1.23	Ac	67.0	10	–	–		
		ss-3	3.00 – 3.45	3.23	As	54.0	9	<0.1	<0.1	○	○
		uds-1	5.00 – 5.45	5.23	Ac	95.0	10	<0.1	<0.1	○	○
		uds-2	7.00 – 7.45	7.23	Ac	92.0	28	–	–	○	
		ss-7	9.00 – 9.45	9.23	Ac	70.0	28	–	–	○	
BH-L11	1.64	ss-1	1.00 – 1.45	1.23	Ac	55.0	9	–	–		
		ss-2	2.00 – 2.45	2.23	As	36.0	–	–	–		
		ss-3	3.00 – 3.45	3.23	Ac	83.0	16	–	–		
		ss-4	4.00 – 4.45	4.23	Ac	60.0	12	0.05	0.02	○	○
		ss-5	5.00 – 5.45	5.23	As	28.0	–	0.35	<0.1	○	○
		ss-6	6.00 – 6.45	6.23	As	30.0	–	0.25	<0.1	○	○
		ss-7	7.00 – 7.45	7.23	As	23.0	8	1.30	<0.1	○	○
BH-L12	2.44	ss-2	3.00 – 3.45	3.23	As	17.0	–	0.50	0.10		○
		ss-4	5.00 – 5.45	5.23	As	10.0	–	2.50	0.08	○	○
BH-L13	3.8	ss-2	2.00 – 2.45	2.23	As	45.0	15	–	–		
		ss-4	4.00 – 4.45	4.23	As	7.0	–	10.90	2.00		
BH-L14	2.71	ss-1	1.00 – 1.45	1.23	Ac	82.0	14	–	–		
		ss-3	3.00 – 3.45	3.23	Ac	46.0	13	0.30	<0.1		○

Source: Study Team

Table 6.1.9 Extracted Layer for Liquefaction Evaluation （Right Bank）

Borehole No.	Ground Water Depth(m)	Sample No.	Depth (m)	Middl Depth	Geological Classification	Grain size Fc(%)	Ip	D50(mm)	D10(mm)	Saturated Layer	Need Check
BH-R03	-0.816	–	0.00 – 0.50	0.25	As	3.0	–	5.00	0.40	○	○
BH-R11	3.918	ss-2	2.00 – 2.45	2.23	As	39.0	–	–	–		
		ss-4	4.00 – 4.45	4.23	As	9.0	–	0.55	0.08	○	○
BH-R12	1.973	ss-2	3.00 – 3.45	3.23	Ac	51.0	11	0.08	0.01	○	○
BH-R14	4.519	ss-1	1.00 – 1.45	1.23	Ac	67.0	7	–	–		
		ss-3	3.00 – 3.45	3.23	As	38.0	–	–	–		
		ss-5	5.00 – 5.45	5.23	As	10.0	–	3.50	0.08	○	○
BH-R15	4.58	ss-2	2.00 – 2.45	2.23	As	36.0	–	–	–		
		ss-4	4.00 – 4.45	4.23	As	13.0	–	1.50	<0.1	○	○
		ss-6	6.00 – 6.45	6.23	As	6.0	–	3.00	0.20	○	○
BH-R16	4.31	ss-1	1.00 – 1.45	1.23	As	8.0	–	1.50	0.15		
		ss-3	3.00 – 3.45	3.23	As	8.0	–	2.00	0.95		
		ss-5	5.00 – 5.45	5.23	As	4.0	–	3.40	1.50		
		ss-7	7.00 – 7.45	7.23	As	0.0	–	1.20	0.90	○	○
BH-R17	4.797	ss-2	2.00 – 2.45	2.23	Ac	97.0	14	–	–		
		ss-4	4.00 – 4.45	4.23	As	32.0	–	0.20	<0.1		
		ss-6	6.00 – 6.45	6.23	As	17.0	–	0.70	<0.1	○	○
		ss-8	8.00 – 8.30	8.15	As	0.0	–	2.50	1.30	○	
BH-R18	3.24	ss-1	1.00 – 1.45	1.23	As	16.0	–	–	–		
		ss-5	5.00 – 5.45	5.23	As	21.0	–	0.70	<0.1	○	○

Source: Study Team

As a result of the layer extraction of liquefaction evaluation, the safety rate to liquefaction (FL value) of some As and Ac layer is less than 1.0. This means there was a spot with a possibility of liquefaction. However, in the case where FL < 1.0 is very limited, such as one meter in the same layer. Therefore, it is overestimated that the entire layer which has extracted layer regards the liquefaction layer. Then, the liquefaction potential index (PL) was used for the evaluation of liquefaction risk for the entire layer.

In the PL method, the FL value is weighted and added in the depth direction to obtain the liquefaction risk of the whole layer.[1,2]。 The relationship between the PL value and the liquefaction risk is shown below.

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5x)dx$$

Here,

P_L : Liquefaction Potential Index

F_L : Safety Rete to Liquefaction (ifF_L≥ 1,F_L=1)

x : Depth from Ground Surface (m)

Table 6.1.10 PL Value and Liquefaction Risk

	PL = 0	0 < PL ≦ 5	5 < PL ≤ 15.	15 < PL
PL Value	The risk of liquefaction is quite low. Detailed investigation on liquefaction is unnecessary.	The risk of liquefaction is low. However, a detailed investigation is needed for important structures.	The risk of liquefaction is high. Important structures require detailed investigation. Moreover, the countermeasures against liquefaction are generally required.	The risk of liquefaction is extremely high. Detailed investigations and countermeasures against liquefaction are inevitable.

Source: Specifications for Highway Bridge Part V Seismic Design

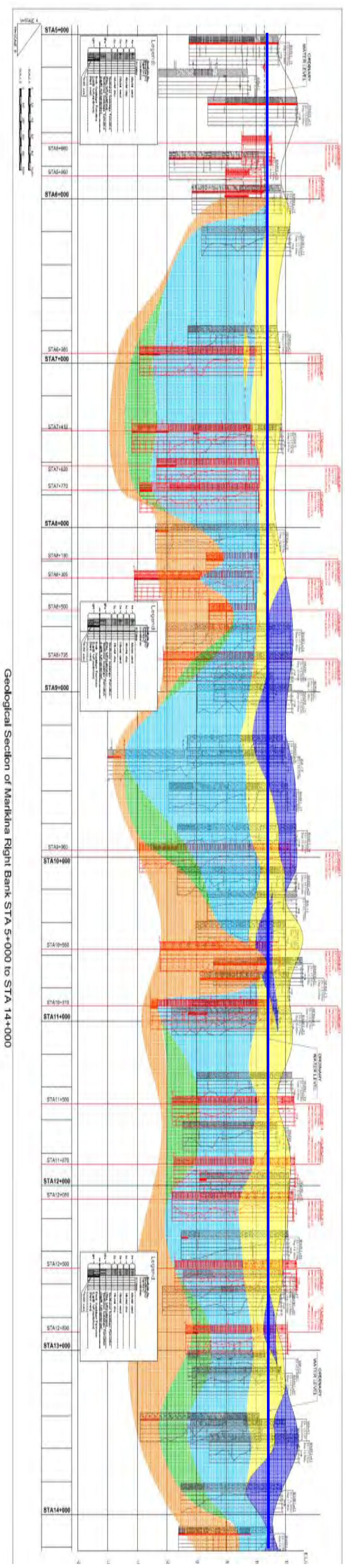
The summary of the result is shown below. (The result of liquefaction assessment at each boring site shall be attached to Structural Calculation.)

[1] Toshio Iwasaki, Fumio Tatsuoka, Kenichi Tomita, Susumu Yasuda, Prediction of the liquefaction during earthquake in 1980, Vol. 28, No. 4, 23 -29.

[2] Specifications for Highway Bridge Part V Seismic Design. 146

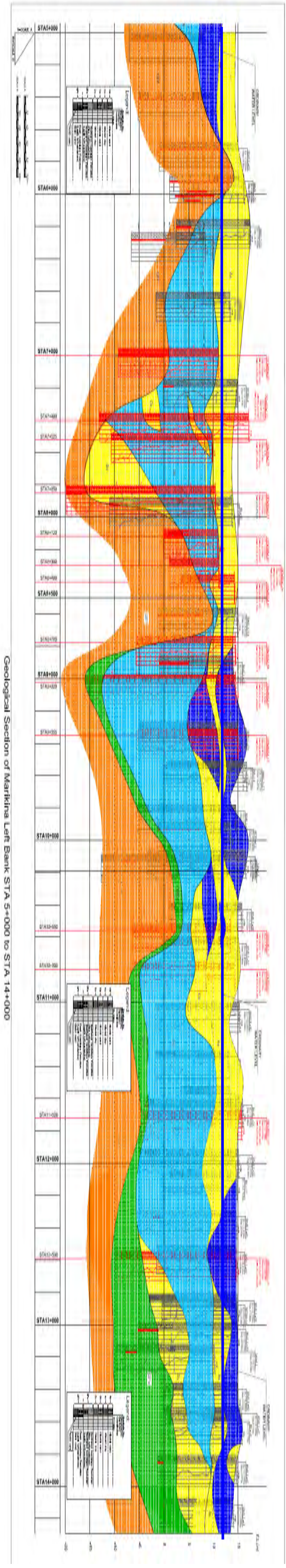
RIGHT BANK

Location	Boring. No	Need Liq. Check	PL	Liq.Soil Layer
6+980	BH-R03	○	5.39	As
7+410	BH-R04	×		
7+620	BH-R05	×		
7+770	BH-R06	×		
8+130	BH-R07	×		
8+305	BH-R08	×		
8+500	BH-R09	×		
8+735	BH-R10	×		
9+960	BH-R11	○	0	
10+560	BH-R12	○	0	
10+910	BH-R13	×		
11+500	BH-R14	○	0	
11+870	BH-R15	○	1.58	As
12+080	BH-R16	○	0.69	As
12+500	BH-R17	○	0	
12+890	BH-R18	○	2.98	Ac



LEFT BANK

Location	Boring. No	Need Liq. Check	PL	Liq.Soil Layer
5+980	BH-G01	○	1.49	As
6+005	BH-G04	○	5.4	As
7+000	BH-L01	×		
7+400	BH-L02	○	0	
7+524	BH-L03	○	0	
7+850	BH-L04	×		
8+120	BH-L05	×		
8+300	BH-L06	×		
8+400	BH-L07	○	0.65	As
8+780	BH-L08	×		
9+020	BH-L09	○	0	
9+355	BH-L10	○	1.32	As
10+550	BH-L11	○	0.07	Ac
10+800	BH-L12	○	0	
11+620	BH-L13	×		
12+590	BH-L14	○	0	



Source: Study Team

Figure 6.1.12 Result of Liquefaction Risk

As a result of the PL evaluation, it was considered that the risk of liquefaction was quite low in the upstream area from Sta. 7+000 on both the left and right banks. Although the PL values at the downstream of Sta.6 + 005 on the left bank and Sta.6 + 980 on the right bank are 5.40 and 5.39, indicating low liquefaction risk, their FL value of the surface As the layer is less than 1.

However, it is considered that the right bank will not be directly affected by liquefaction because steel sheet piles will be placed in front of the existing bank. Since the risk of liquefaction was low, it was judged that countermeasure for liquefaction was unnecessary. On the other hand, for the left bank, because the steel sheet pile was placed in contact with the As layer, it was judged that consideration for liquefaction was necessary. Therefore, in the design of the steel sheet pile upstream from the boring point, the strength of the soil layer shall be decreased by consideration of the liquefaction risk.

Table 6.1.11 Distribution of the FL value (Left Bank BH-G-04 PL = 5.40)

土質記号	深度 (m) D	層厚 (m) h	湿潤重量 (kN/m³) γt	飽和重量 (kN/m³) γsat	判定深度 (m) x	土質区分	土層種類	実測N値 N	有効上載圧 (kN/m²) σv'	標準貫入試験時有効上載圧 (kN/m²) σvb'	細粒分含有率 (%) Fc	平均粒径 (mm) D50	液状化判定を考慮	地震動特性補正係数 CW	繰り返し三軸強度比 RL	動的せん断強度比 R	地震時せん断応力比 L	液状化抵抗率 FL	土質定数低減係数 DE
	1.00	1.00	17.5	18.5	1.0	沖積	砂質土	7.00	15.4	15.4	36.00	0.000		1.00	0.348	0.348	0.227	1.535	1
	2.00	1.00	17.5	18.5	2.0	沖積	砂質土	9.00	23.9	23.9	17.00	0.400		1.00	0.297	0.297	0.294	1.010	1
	3.00	1.00	17.5	18.5	3.0	沖積	砂質土	9.00	32.4	32.4	24.00	0.250		1.00	0.306	0.306	0.322	0.950	1
	4.00	1.00	17.5	18.5	4.0	沖積	砂質土	2.00	40.9	40.9	14.00	0.400		1.00	0.127	0.127	0.336	0.378	1/3
	5.00	1.00	17.5	18.5	5.0	沖積	砂質土	20.00	49.4	49.4	14.00	0.500		1.00	0.923	0.923	0.343	2.688	1
	6.00	1.00	17.5	18.5	6.0	沖積	砂質土	50.00	57.9	57.9	18.00	0.450		1.00	208.239	208.239	0.346	601.205	1
	7.00	1.00	17.0	18.0	7.0	洪積	粘性土	50.00	65.9	65.9	0.00	0.000						**1	1
	8.00	1.00	17.0	18.0	8.0	洪積	粘性土	50.00	73.9	73.9	0.00	0.000						**1	1
	9.00	1.00	17.0	18.0	9.0	洪積	粘性土	50.00	81.9	81.9	0.00	0.000						**1	1
	10.00	1.00	17.0	18.0	10.0	洪積	粘性土	50.00	89.9	89.9	0.00	0.000						**1	1
	11.00	1.00	17.0	18.0	11.0	洪積	粘性土	50.00	97.9	97.9	0.00	0.000						**1	1
	12.00	1.00	17.0	18.0	12.0	洪積	粘性土	50.00	105.9	105.9	0.00	0.000						**1	1
	13.00	1.00	17.0	18.0	13.0	洪積	粘性土	50.00	113.9	113.9	0.00	0.000						**1	1
	14.00	1.00	17.0	18.0	14.0	洪積	粘性土	50.00	121.9	121.9	0.00	0.000						**1	1
	15.00	1.00	17.0	18.0	15.0	洪積	粘性土	50.00	129.9	129.9	0.00	0.000						**1	1
	16.00	1.00	17.0	18.0	16.0	洪積	粘性土	50.00	137.9	137.9	0.00	0.000						**1	1
	17.00	1.00	17.0	18.0	17.0	洪積	粘性土	50.00	145.9	145.9	0.00	0.000						**1	1
	18.00	1.00	17.0	18.0	18.0	洪積	粘性土	50.00	153.9	153.9	0.00	0.000						**1	1
	19.00	1.00	17.0	18.0	19.0	洪積	粘性土	50.00	161.9	161.9	0.00	0.000						**1	1
	20.00	1.00	17.0	18.0	20.0	洪積	粘性土	50.00	169.9	169.9	0.00	0.000						**1	1

Source: Study Team

Table 6.1.12 Distribution of the FL value (Right Bank BH-R-03 PL = 5.39)

土質記号	深度 (m) D	層厚 (m) h	湿潤重量 (kN/m³) γt	飽和重量 (kN/m³) γsat	判定深度 (m) x	土質区分	土層種類	実測N値 N	有効上載圧 (kN/m²) σv'	標準貫入試験時有効上載圧 (kN/m²) σvb'	細粒分含有率 (%) Fc	平均粒径 (mm) D50	液状化判定を考慮	地震動特性補正係数 CW	繰り返し三軸強度比 RL	動的せん断強度比 R	地震時せん断応力比 L	液状化抵抗率 FL	土質定数低減係数 DE
	2.01	2.01	17.5	18.5	2.0	沖積	砂質土	4.00	33.4	33.4	3.00	5.000		1.00	0.173	0.173	0.205	0.846	2/3
	3.01	1.00	17.5	18.5	3.0	沖積	砂質土	4.00	41.9	41.9	3.00	5.000		1.00	0.167	0.167	0.245	0.680	2/3
	4.01	1.00	17.5	18.5	4.0	沖積	砂質土	12.00	50.4	50.4	3.00	5.000		1.00	0.279	0.279	0.270	1.033	1
	5.01	1.00	17.5	18.5	5.0	沖積	砂質土	8.00	58.9	58.9	3.00	5.000		1.00	0.220	0.220	0.285	0.771	2/3
	6.01	1.00	18.0	19.0	6.0	洪積	粘性土	2.00	67.9	67.9	44.00	0.000						**1	1
	7.01	1.00	17.5	18.5	7.0	洪積	粘性土	3.00	76.4	76.4	44.00	0.000						**1	1
	8.01	1.00	18.0	19.0	8.0	洪積	粘性土	3.00	85.4	85.4	96.00	0.100						**1	1
	9.01	1.00	18.0	19.0	9.0	洪積	粘性土	9.00	94.4	94.4	76.00	0.000						**1	1
	10.01	1.00	18.0	19.0	10.0	洪積	粘性土	10.00	103.4	103.4	76.00	0.000						**1	1
	11.01	1.00	18.0	19.0	11.0	洪積	粘性土	13.00	112.4	112.4	83.00	0.000						**1	1
	12.01	1.00	18.0	19.0	12.0	洪積	粘性土	11.00	121.4	121.4	83.00	0.000						**1	1
	13.01	1.00	18.0	19.0	13.0	洪積	粘性土	11.00	130.4	130.4	83.00	0.000						**1	1
	14.01	1.00	18.0	19.0	14.0	洪積	粘性土	20.00	139.4	139.4	83.00	0.000						**1	1
	15.01	1.00	18.0	19.0	15.0	洪積	粘性土	50.00	148.4	148.4	74.00	0.000						**1	1
	16.01	1.00	18.0	19.0	16.0	洪積	粘性土	43.00	157.4	157.4	74.00	0.000						**1	1
	17.01	1.00	18.0	19.0	17.0	洪積	粘性土	22.00	166.4	166.4	29.00	1.200						**1	1
	18.01	1.00	18.0	19.0	18.0	洪積	粘性土	37.00	175.4	175.4	29.00	1.200						**1	1
	19.01	1.00	18.0	19.0	19.0	洪積	粘性土	36.00	184.4	184.4	34.00	0.850						**1	1
	20.01	1.00	18.0	19.0	20.0	洪積	粘性土	50.00	193.4	193.4	34.00	0.850						**1	1

Source: Study Team

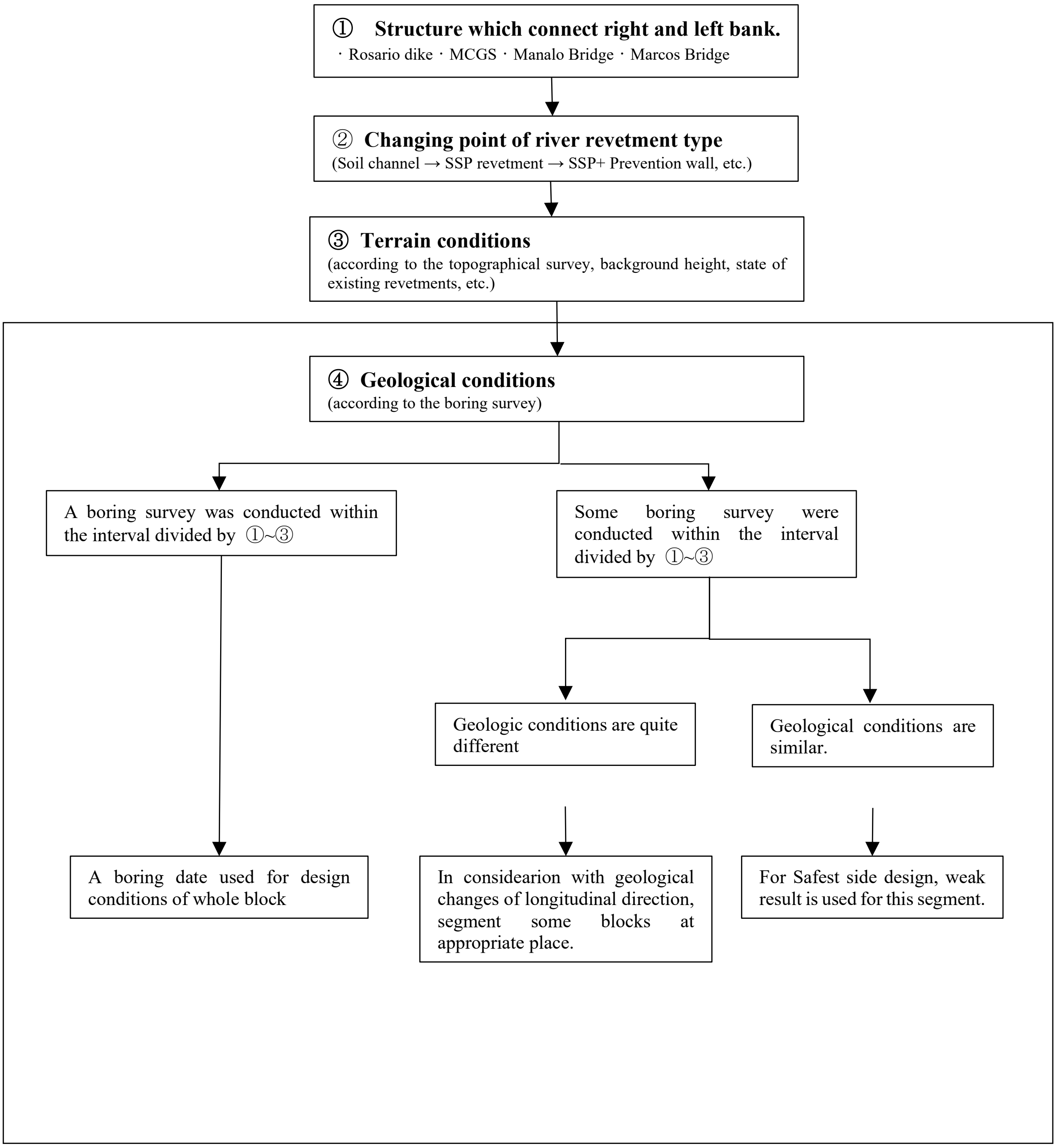
6.1.3.3 Arrangement of Design Conditions for SSP Revetments

The type and scale of SSP revetments in each section shall be appropriately designed based on the conditions such as the load acting on the revetment and the geology of the construction place. Also, design sections will be divided into several block divisions considering that the design conditions of the section vary from place to place. The following conditions shall be considered when setting the block division of the design section:

- Structures that connect right and left banks, such as bridge, dike, and river gate.
- Changing points of river revetment type
- Geological conditions (according to the boring survey)
- Terrain conditions (according to the topographical survey, background height, state of existing revetments, etc.)
- Division into the appropriate number of blocks in a long successive section

Considering the above conditions, the design section of SSP revetments will be divided into an appropriate number of blocks based on the results of each survey.

The flow chart of segmentation is shown in **Figure 6.1.13.** The result of the segmentation are shown in **Table 6.1.13** and **Table 6.1.14**.



Source: Study Team

Figure 6.1.13 Flowchart of Block Segmentation

Table 6.1.13 Block Segmentation for Low Water Revetment (Left Bank)

No.	Sta.			Length (m)
	From	-	To	
L-1	6+035.3	-	6+080	42.80
L-2	6+080	-	6+362.8	266.29
L-3	6+753	-	6+800	61.44
L-4	6+800	-	7+180	409.61
L-5	7+180	-	7+480	302.83
L-6	7+480	-	7+820	359.77
L-7	7+820	-	7+940	120.00
L-8	7+940	-	8+120	180.00
L-9	8+120	-	8+300	180.11
L-10	8+300	-	8+600	293.86
L-11	8+600	-	8+835	211.24
L-12	9+205	-	9+320	107.58
L-13	9+320	-	9+560	230.49
L-14	9+560	-	9+800	267.21
L-15	9+800	-	9+900	100.98
L-16	9+900	-	10+020	120.99
L-17	10+020	-	10+360	332.08
L-18	10+360	-	10+520	160.22
L-19	10+520	-	10+580	60.10
L-20	10+580	-	10+640	60.00
L-21	10+640	-	10+760	86.91
L-22	10+760	-	11+040	230.40
L-23	11+040	-	11+180	140.19
L-24	11+180	-	11+460	282.50
L-25	11+460	-	11+640	182.49
L-26	11+640	-	11+800	164.63
L-27	11+800	-	12+040	266.26
L-28	12+040	-	12+280	260.87
L-29	12+280	-	12+520	249.09
L-30	12+520	-	12+820	304.65
L-31	12+820	-	13+000	179.78
L-32	13+000	-	13+320	322.81
L-33	13+320	-	13+360	31.22

Source: Study Team

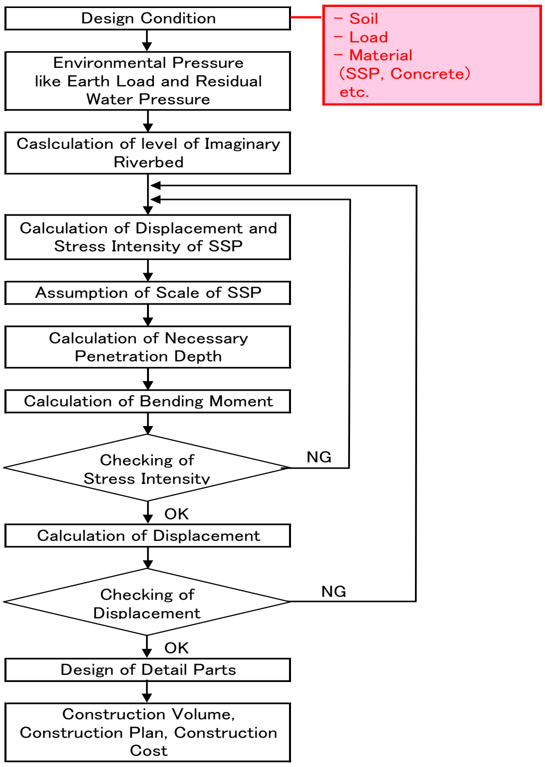
Table 6.1.14 Block Segmentation for Low Water Revetment (Right Bank)

No.	Sta.			Length (m)
	From	-	To	
R-1	5+423	-	5+540	120.61
R-2	5+540	-	5+581.25	49.58
R-3	5+624	-	5+720	125.58
R-4	5+720	-	5+905.80	201.47
R-5	6+035.3	-	6+080	44.51
R-6	6+080	-	6+280	211.53
R-7	6+280	-	6+420	140.28
R-8	6+420	-	6+920	437.46
R-9	6+920	-	7+220	295.15
R-10	7+220	-	7+620	387.86
R-11	7+620	-	7+900	272.03
R-12	7+900	-	8+240	340.06
R-13	8+240	-	8+500	260.19
R-14	8+500	-	8+620	124.41
R-15	8+620	-	8+940	340.55
R-16	8+940	-	9+000	65.45
R-17	9+000	-	9+200	217.04
R-18	9+200	-	9+380	196.62
R-19	9+380	-	9+700	300.75
R-20	9+700	-	9+900	182.87
R-21	9+900	-	10+380	493.71
R-22	10+380	-	10+520	140.72
R-23	10+520	-	10+540	20.67
R-24	10+540	-	10+660	122.88
R-25	10+660	-	10+760	125.77
R-26	10+760	-	10+820	79.86
R-27	10+820	-	10+980	205.01
R-28	10+980	-	11+200	220.61
R-29	11+200	-	11+360	160.68
R-30	11+360	-	11+700	328.31
R-31	11+700	-	11+980	214.88
R-32	11+980	-	12+000	16.30
R-33	12+000	-	12+240	198.95
R-34	12+240	-	12+520	265.59
R-35	12+520	-	12+540	18.97
R-36	12+540	-	12+660	116.78
R-37	12+660	-	12+740	80.13
R-38	12+740	-	12+980	240.44
R-39	12+980	-	13+100	120.17
R-40	13+100	-	13+220	118.37
R-41	13+220	-	13+375	146.11

Source: Study Team

6.1.3.4 Design Calculation of SSP Revetment

The design calculation of SSP revetment is carried out according to the flowchart shown in the following figure.



Source: Study Team

Figure 6.1.14 Design Flow of SSP Revetment

The design conditions of SSP revetments are as shown below.

Table 6.1.15 Design Conditions for SSP Revetment (Materials, Soil Conditions, Water Level, etc.)

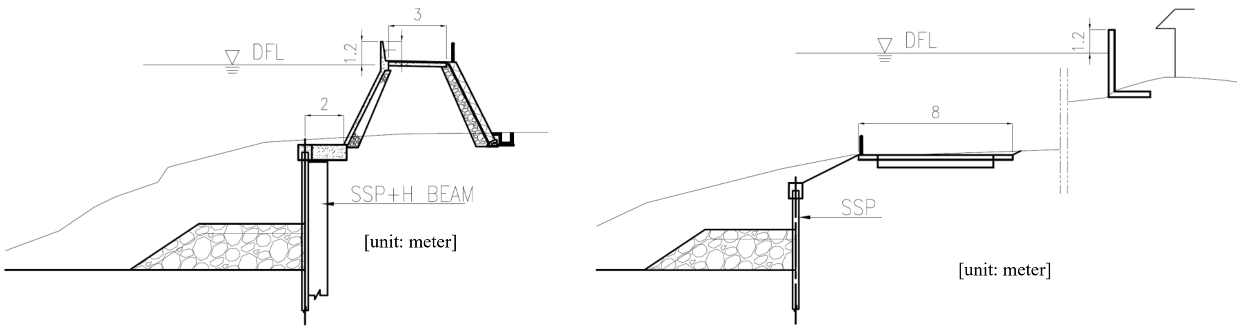
<table>
<tr><th colspan="3">Items</th><th colspan="3">Design Condition</th></tr>
<tr><td rowspan="6">Material Unit Weight</td><td colspan="2">Reinforced concrete</td><td colspan="3">24.0kN/ m^3</td></tr>
<tr><td colspan="2">Plain concrete</td><td colspan="3">23.5kN/ m^3</td></tr>
<tr><td colspan="2">Mortar</td><td colspan="3">21.0kN/ m^3</td></tr>
<tr><td colspan="2">Steel</td><td colspan="3">77.0kN/ m^3</td></tr>
<tr><td colspan="2">Cast iron</td><td colspan="3">71.0kN/ m^3</td></tr>
<tr><td colspan="2">Water</td><td colspan="3">9.8kN/ m^3</td></tr>
<tr><td rowspan="7">Soil Condition</td><td colspan="2">Quality of the back-soil</td><td colspan="3">Loose sandy soil (The soil quality of the backfilled soil is considered because the soil quality test has not yet been completed.)</td></tr>
<tr><td rowspan="3">Unit weight</td><td>Wet condition</td><td colspan="3">18kN/ m^3</td></tr>
<tr><td>Saturation state</td><td colspan="3">20kN/ m^3</td></tr>
<tr><td>Underwater condition</td><td colspan="3">10kN/ m^3</td></tr>
<tr><td colspan="2">N-value</td><td colspan="3">Measured values in standard penetration test</td></tr>
<tr><td colspan="2">Internal friction angle ϕ</td><td colspan="3">$= 15 + \sqrt{15N} \leqq 45$ °, where N ≧ 5</td></tr>
<tr><td colspan="2">Adhesive force C</td><td colspan="3">C = 0</td></tr>
<tr><td rowspan="10">Steel Sheet Pile</td><td colspan="2">Steel grade</td><td colspan="3">SYW 295</td></tr>
<tr><td colspan="2">Allowable bending stress</td><td colspan="3">Regular: 180 N/mm^2 / earthquake: 270 N/mm^2</td></tr>
<tr><td colspan="2">Allowable displacement</td><td colspan="3">Regular: 50 mm / earthquake: 75 mm</td></tr>
<tr><td colspan="2" rowspan="4">Effective rate of cross-sectional performance</td><td rowspan="2">Hat type</td><td>Moment of inertia of area</td><td>100%</td></tr>
<tr><td>Section modulus</td><td>100%</td></tr>
<tr><td rowspan="2">U-Type</td><td>Moment of inertia of area</td><td>100%</td></tr>
<tr><td>Section modulus</td><td>80%</td></tr>
<tr><td colspan="2">Young modulus</td><td colspan="3">2.0 x 105 N/mm^2</td></tr>
<tr><td colspan="2">Corrosion allowance</td><td colspan="3">1mm for both sides (total 2mm)</td></tr>
<tr><td colspan="2">Elevation of concrete coping crown</td><td colspan="3">Equivalent to the DFL</td></tr>
<tr><td colspan="3" rowspan="2">Water Level Condition</td><td colspan="3">[Back water level]
Background elevation ≧ top concrete crown height
Back water level: t top concrete crown height
Background elevation < top concrete crown height
Back water level: background elevation</td></tr>
<tr><td colspan="3">[Front water level]
Normal: drought water level
Low water level at the time of earthquake</td></tr>
<tr><td colspan="3">Horizontal Seismic Coefficient</td><td colspan="3">k = 0.2 (both in air and water)</td></tr>
<tr><td colspan="3">Surcharge</td><td colspan="3">Normal: 10kN/m^2 , seismic: 5kN/m^2</td></tr>
</table>

Source: Study Team

6.1.3.5 Examination of Steel Sheet Pile Revetment Structure

The most critical factor in determining the revetment structure in this design is site restriction. In order to minimize the land acquisition and house relocation, the revetment structure will be basically “the self-supporting steel sheet pile revetment with foot protections" according to the Definitive Plan. The outline of the structure is written below.

- From Sta.5+620 to 6+700 : Sodding or Concrete fencing with the self-supporting SSP
- From Sta.6+700 to 10+500 : Concret fencing with self-supporting SSP
- Upstream from Sta10+500 : SSP as low water chanle and the conclete wall such as prapet wall as flood protection wall.



Sta. 5+620 to Sta. 10+500 (embankment Case)   Sta. 10+500 to Sta. 13+350

Source: Study Team

Figure 6.1.15 Standard Revetment Structure

Source: Study Team

Figure 6.1.16 Example of Standard Revetment Structure Applied in Sta. 6+700 to Sta. 10+500

As the construction of the SSP revetment is conducted throughout the year, the SSP crown height shall be higher than the height that the construction can be possible. Therefore, the SSP crown height is set as shown in below.

- Downsterm of MCGS (Sta. 6+010) : EL. 12.7～12.8m
- From the upstream of MCGS to Sta.10+500 : EL. 14.0～15.0m
- From Sta.10+500 to 13+350 : EL. 12.0～15.0m (Depend on the Exsisting Groung)

6.1.3.6 Determination of Foot Protection for Low Water Channel Revetment

(1) Conditions for Determination

To prevent erosion at the bottom of SSP revetments and to obtain earth pressure at their front side, foot protection shall be installed in front of the SSP revetments.

In the river improvement section of Phase II and Phase III, ripraps have been installed as foot protection in front of SSP revetment. Since the rock-place method (riprap) is a common construction method in the Philippines and there are many domestic examples, the optimum structure will be adopted after examination including the latest construction methods.

1) Determination of Design Velocity

(a) Calculation Method of Design Velocity (V_D)

(i) Upstream of MCGS

Design velocity (V_D) is calculated according to the Japanese technical guideline "Dynamical Design Method of Revetment" and that of DPWH "Technical Standards and Guidelines for Design of Flood Control Structures".

Representative velocity (V0) is set at each survey cross-section along the river where the revetment is to be installed. According to the river profile, such as riverbed condition, substrate type, and meanders, the entire design section is divided into several subsections. In each subsection, a design velocity (V_D) is given as an average of representative velocities (V_0) within the section.

In this design, representative velocity (V_0) was estimated for local scouring, and foot protection works based on mean flow velocity as formula below[3]:

$$V_D = \alpha_1 \cdot \alpha_2 V_m$$

Where,

V_D: Design velocity (m/s)

V_m : Mean velocities of hydraulic analysis (m/s)

α_1 : Coefficient for scouring ($\alpha_1 \leq 2.0$, 2 $\alpha_1 \leq 1.6$)

at straight stretch

$$\alpha_1 = 1 + \frac{\Delta Z}{2H_d} \text{.......} (\alpha_1 \leq 2.0)$$

where,

ΔZ : Maximum scouring depth (m)

H_d : Average design water depth (m)

at bending stretch segment

Inner bank of the bend: $$\alpha_1 = 1 + \frac{B}{2r}$$

Outer bank of the bend: $$\alpha_1 = 1 + \frac{B}{2r} + \frac{\Delta Z}{2H_d}$$

where,

α_1 : Correction coefficient (Segment 2 and 3: $\alpha_1 \leq 1.6$)

B : River width (m)

R : Radius of the bend (m)

ΔZ : Maximum scouring depth (m)

H_d : Average design water depth (m)

α_2 : Coefficient for foot protection work

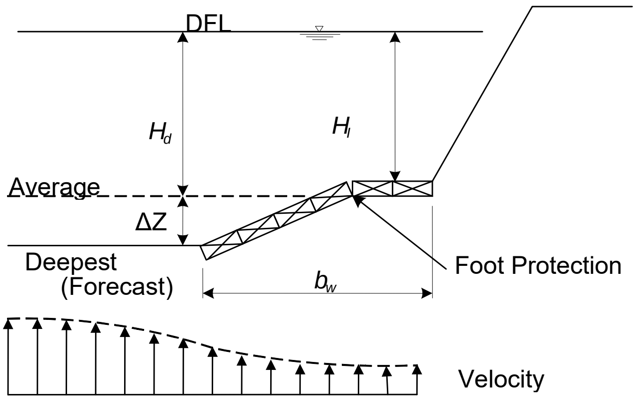
[3] Dynamical Design Method of Revetment, 4-2, DPWH: Technical Standards and Guidelines for Design of Flood Control Structures, 2.3.1

In case structure with the adequate foot protection works (crest width of 2 m or more), the coefficient of α_2 is considered as follows:

$b_w / H_I > 1.0$: $\alpha_2 = 0.9$

$b_w / H_I \leq 1.0$: $\alpha_2 = 1.0$

The average flow velocity V_m uses the flow velocity calculated based on the one-dimensional unequal flow calculation under the design flood.



Source: DPWH: Technical Standards and Guidelines for Design of Flood Control Structures, 2.3.1

Figure 6.1.17 Schematic Layout for Local Scouring and Foot Protection Works

(ii) Downstream of MCGS (5+950～5+400)

In the hydraulic calculation, the influence of the flow contraction caused by the gate operation of MCGS cannot be considered. Thus, the velocity of the near shore measured by the hydraulic model experiment is used as the design velocity. The flow velocity was measured at a water depth of 60%. It is about the same as the average value of the flow velocity in the propulsion direction near the revetment. The measurement of the flow velocity in the experiment is up to 5+800. Since there is no significant change in river width or cross-sectional shape downstream from 5+800, it is estimated that the same flow velocity may be obtained up to the endpoint of the meander (5 + 400). Therefore, the flow velocity in the section from 5+800 to 5 + 400 will refer to the flow velocity at the Station 5+ 800 (2.0 m /s on the left and right shores) of the hydraulic model experiment. (For the measured values of the flow velocity in the hydraulic model experiment, see Chapter 4, Figure 4.2.5).

(b) Summary of Design Velocity

The design velocities at each design section are shown below.

Table 6.1.16 Conditions and Results of Design Velocity (Downstream of Marikina River: Right Bank)

RIVER STA.	Mean Flow Velocity	DFL	Design Riverbed	Type	Coefficient for Scouring	Coefficient for Foot Protection	Represemtative Velocity	Design Velocity
	Vm		Z		α1	α2	V_0	V_D
	(m/s)	(EL+m)	(EL+m)				(m/s)	(m/s)
5+400	1.34	14.53	7.76	Straight	－	－	－	2.0※
5+450	1.36	14.53	7.76	Straight	－	－	－	
5+500	1.27	14.55	7.77	Straight	－	－	－	
5+550	1.28	14.56	7.77	Straight	－	－	－	
5+600	1.25	14.58	7.78	Straight	－	－	－	
5+650	1.26	14.58	7.78	Curve	－	－	－	2.0※
5+700	1.34	14.58	7.79	Curve	－	－	－	
5+750	1.36	14.59	7.79	Curve	－	－	－	
5+800	1.33	14.61	7.80	Straight	－	－	－	3.0※
5+850	1.38	14.61	7.80	Straight	－	－	－	
5+900	1.32	14.63	7.80	Straight	－	－	－	
5+950	1.34	14.63	7.81	Straight	－	－	－	1.5
6+000	1.36	14.64	7.81	Straight	－	－	1.36	
6+050	1.71	14.59	7.82	Straight	－	－	1.71	
6+100	1.54	14.63	7.82	Straight	－	－	1.54	1.7
6+150	1.54	14.65	7.83	Straight	1.1	1.0	1.67	
6+200	1.55	14.66	7.83	Straight	1.1	1.0	1.67	
6+250	1.55	14.67	7.84	Straight	1.1	1.0	1.67	
6+300	1.55	14.68	7.84	Straight	1.2	1.0	1.83	
6+350	1.56	14.69	7.85	Curve	1.2	1.0	1.80	1.6
6+400	1.56	14.70	7.85	Curve	1.1	1.0	1.71	
6+450	1.66	14.70	7.85	Curve	1.1	1.0	1.83	
6+500	1.67	14.71	7.85	Curve	1.2	1.0	1.94	
6+550	1.00	14.81	7.85	Straight	1.1	1.0	1.13	
6+600	1.00	14.81	7.85	Straight	1.5	1.0	1.47	

※Observed Velocity obtained by the Hydraulic Model Calculation

Source: Study Team

Table 6.1.17 Conditions and Results of Design Velocity (Downstream of Marikina River: Left Bank)

RIVER STA.	Mean Flow Velocity	DFL	Design Riverbed	Type	Coefficient for Scouring	Coefficient for Foot Protection	Represemtative Velocity	Design Velocity
	Vm		Z		α1	α2	V_0	V_D
	(m/s)	(EL+m)	(EL+m)				(m/s)	(m/s)
5+400	1.34	14.53	7.76	Straight	–	–	1.34	2.0※
5+450	1.36	14.53	7.76	Straight	–	–	1.36	
5+500	1.27	14.55	7.77	Straight	–	–	1.27	
5+550	1.28	14.56	7.77	Straight	–	–	1.28	
5+600	1.25	14.58	7.78	Straight	–	–	1.25	
5+650	1.26	14.58	7.78	Curve	–	–	1.26	
5+700	1.34	14.58	7.79	Curve	–	–	1.34	
5+750	1.36	14.59	7.79	Curve	–	–	1.36	
5+800	1.33	14.61	7.80	Straight	–	–	1.33	4.0※
5+850	1.38	14.61	7.80	Straight	–	–	1.38	
5+900	1.32	14.63	7.80	Straight	–	–	1.32	
5+950	1.34	14.63	7.81	Straight	–	–	1.34	1.5
6+000	1.36	14.64	7.81	Straight	–	–	1.36	
6+050	1.71	14.59	7.82	Straight	–	–	1.71	
6+100	1.54	14.63	7.82	Straight	–	–	1.54	1.7
6+150	1.54	14.65	7.83	Straight	1.08	1.0	1.67	
6+200	1.55	14.66	7.83	Straight	1.08	1.0	1.67	
6+250	1.55	14.67	7.84	Straight	1.08	1.0	1.67	
6+300	1.55	14.68	7.84	Straight	1.18	1.0	1.83	
6+350	1.56	14.69	7.85	Curve	1.47	1.0	2.29	1.9
6+400	1.56	14.70	7.85	Curve	1.34	1.0	2.09	
6+450	1.66	14.70	7.85	Curve	1.25	1.0	2.07	
6+500	1.67	14.71	7.85	Curve	1.31	1.0	2.19	
6+550	1.00	14.81	7.85	Straight	1.14	1.0	1.13	
6+600	1.00	14.81	7.85	Straight	1.47	1.0	1.47	

※Observed Velocity obtained by the Hydraulic Model Calculation
Source: Study Team

Table 6.1.18 Conditions and Results of Design Velocity (Upstream of Marikina River: Right Bank)

RIVER STA.	Mean Flow Velocity	DFL	Design Riverbed	Type	Coefficient for Scouring	Coefficient for Foot Protection	Represemtative Velocity	Design Velocity
	Vm		Z		α1	α2	V_0	V_D
	(m/s)	(EL+m)	(EL+m)				(m/s)	(m/s)
6+650	5.80	14.38	7.85	Straight	1.2	1.0	6.92	3. 9
6+700	3.69	17.41	8.00	Curve	1.5	1.0	1.46	
6+800	3.67	17.47	8.03	Straight	1.1	1.0	3.89	
6+900	3.64	17.53	8.05	Straight	1.1	1.0	3.85	
7+000	3.62	17.59	8.08	Straight	1.0	1.0	3.79	
7+100	3.60	17.65	8.10	Straight	1.0	1.0	3.73	
7+200	3.58	17.71	8.13	Straight	1.1	1.0	3.78	
7+300	3.52	17.77	8.15	Straight	1.1	1.0	3.80	3. 6
7+400	3.44	17.83	8.18	Straight	1.1	1.0	3.81	
7+500	3.43	17.89	8.20	Straight	1.1	1.0	3.77	
7+600	3.46	17.95	8.23	Straight	1.1	1.0	3.70	
7+700	3.47	18.01	8.25	Straight	1.0	1.0	3.60	
7+800	3.43	18.07	8.28	Straight	1.0	1.0	3.48	
7+900	3.42	18.13	8.30	Straight	1.0	1.0	3.44	
8+000	3.40	18.19	8.33	Straight	1.0	1.0	3.45	
8+100	3.39	18.25	8.35	Straight	1.0	1.0	3.46	
8+200	3.38	18.32	8.38	Straight	1.1	1.0	3.56	
8+300	3.34	18.38	8.40	Straight	1.0	1.0	3.41	
8+400	3.33	18.44	8.43	Straight	1.0	1.0	3.45	
8+500	3.31	18.50	8.45	Straight	1.1	1.0	3.50	
8+600	3.30	18.56	8.48	Straight	1.0	1.0	3.45	
8+700	3.29	18.62	8.50	Curve	1.1	1.0	3.75	3. 7
8+800	3.28	18.68	8.53	Curve	1.1	1.0	3.73	
8+900	3.27	18.74	8.55	Curve	1.1	1.0	3.73	
9+000	3.28	18.80	8.58	Curve	1.1	1.0	3.75	
9+100	3.27	18.86	8.60	Curve	1.2	1.0	3.78	
9+200	3.26	18.92	8.63	Curve	1.1	1.0	3.72	
9+300	3.25	18.98	8.65	Curve	1.1	1.0	3.71	
9+400	3.24	19.04	8.68	Curve	1.1	1.0	3.70	
9+500	3.23	19.10	8.70	Straight	1.1	1.0	3.46	
9+600	3.22	19.16	8.73	Curve	1.2	1.0	3.79	
9+700	3.21	19.22	8.75	Curve	1.2	1.0	3.78	
9+800	3.20	19.28	8.78	Straight	1.0	1.0	3.28	3.2
9+900	3.19	19.35	8.80	Straight	1.0	1.0	3.25	
10+000	3.17	19.41	8.83	Straight	1.0	1.0	3.26	
10+100	3.16	19.47	8.85	Straight	1.0	1.0	3.22	
10+200	3.16	19.53	8.88	Straight	1.0	1.0	3.22	
10+300	3.14	19.59	8.90	Straight	1.0	1.0	3.21	
10+400	3.12	19.65	8.93	Straight	1.0	1.0	3.20	
10+500	3.11	19.71	8.95	Straight	1.0	1.0	3.22	
10+600	3.02	19.77	8.98	Straight	1.0	1.0	3.11	
10+700	2.83	19.83	9.00	Curve	1.3	1.0	3.66	3.7
10+800	2.81	19.89	9.03	Curve	1.3	1.0	3.75	
10+900	2.81	19.94	9.05	Curve	1.3	1.0	3.60	
11+000	2.95	19.99	9.08	Straight	1.0	1.0	3.00	2.9
11+100	3.00	20.04	9.10	Straight	1.0	1.0	3.07	
11+200	2.90	20.09	9.13	Straight	1.0	1.0	3.00	
11+300	2.75	20.14	9.15	Straight	1.0	1.0	2.86	
11+400	2.65	20.19	9.18	Straight	1.0	1.0	2.72	
11+500	2.75	20.24	9.20	Straight	1.1	1.0	2.97	
11+600	2.75	20.29	9.23	Straight	1.1	1.0	2.92	
11+700	2.77	20.34	9.25	Straight	1.1	1.0	2.97	
11+800	2.72	20.39	9.28	Curve	1.2	1.0	3.15	3.2
11+900	2.95	20.44	9.30	Curve	1.2	1.0	3.43	
12+000	2.59	20.49	9.33	Curve	1.2	1.0	3.00	
12+100	2.70	20.54	9.35	Curve	1.2	1.0	3.13	
12+200	2.67	20.59	9.38	Straight	1.0	1.0	2.76	2.7
12+300	2.73	20.64	9.40	Straight	1.0	1.0	2.83	
12+400	2.66	20.69	9.43	Straight	1.0	1.0	2.76	
12+500	2.68	20.74	9.45	Straight	1.0	1.0	2.78	
12+600	2.91	20.79	9.48	Straight	1.0	1.0	2.94	
12+700	2.85	20.84	9.50	Straight	1.0	1.0	2.88	
12+800	2.67	20.89	9.53	Straight	1.0	1.0	2.70	
12+900	2.61	20.94	9.55	Straight	1.0	1.0	2.65	
13+000	2.32	20.99	9.58	Straight	1.0	1.0	2.36	
13+100	2.67	21.04	9.60	Straight	1.0	1.0	2.73	
13+200	2.50	21.09	9.63	Straight	1.0	1.0	2.55	
13+300	2.68	21.14	9.65	Straight	1.0	1.0	2.80	

Source: Study Team

Table 6.1.19 Conditions and Results of Design Velocity (Upstream of Marikina River: Left Bank)

RIVER STA.	Mean Flow Velocity	DFL	Design Riverbed	Type	Coefficient for Scouring	Coefficient for Foot Protection	Represemtative Velocity	Design Velocity
	Vm		Z		α1	α2	V_0	V_D
	(m/s)	(EL+m)	(EL+m)				(m/s)	(m/s)
6+650	5.80	14.38	7.85	Straight	1.19	1.0	6.92	
6+700	3.69	17.41	8.00	Curve	1.58	1.0	5.81	
6+800	3.67	17.47	8.03	Straight	1.06	1.0	3.89	
6+900	3.64	17.53	8.05	Straight	1.06	1.0	3.85	4.5
7+000	3.62	17.59	8.08	Straight	1.05	1.0	3.79	
7+100	3.60	17.65	8.10	Straight	1.04	1.0	3.73	
7+200	3.58	17.71	8.13	Straight	1.06	1.0	3.78	
7+300	3.52	17.77	8.15	Straight	1.08	1.0	3.80	
7+400	3.44	17.83	8.18	Straight	1.11	1.0	3.81	
7+500	3.43	17.89	8.20	Straight	1.10	1.0	3.77	
7+600	3.46	17.95	8.23	Straight	1.07	1.0	3.70	
7+700	3.47	18.01	8.25	Straight	1.04	1.0	3.60	
7+800	3.43	18.07	8.28	Straight	1.01	1.0	3.48	
7+900	3.42	18.13	8.30	Straight	1.01	1.0	3.44	
8+000	3.40	18.19	8.33	Straight	1.01	1.0	3.45	3.6
8+100	3.39	18.25	8.35	Straight	1.02	1.0	3.46	
8+200	3.38	18.32	8.38	Straight	1.05	1.0	3.56	
8+300	3.34	18.38	8.40	Straight	1.02	1.0	3.41	
8+400	3.33	18.44	8.43	Straight	1.04	1.0	3.45	
8+500	3.31	18.50	8.45	Straight	1.06	1.0	3.50	
8+600	3.30	18.56	8.48	Straight	1.05	1.0	3.45	
8+700	3.29	18.62	8.50	Curve	1.09	1.0	3.59	
8+800	3.28	18.68	8.53	Curve	1.09	1.0	3.58	
8+900	3.27	18.74	8.55	Curve	1.09	1.0	3.57	
9+000	3.28	18.80	8.58	Curve	1.09	1.0	3.58	
9+100	3.27	18.86	8.60	Curve	1.09	1.0	3.56	
9+200	3.26	18.92	8.63	Curve	1.09	1.0	3.55	3.7
9+300	3.25	18.98	8.65	Curve	1.09	1.0	3.54	
9+400	3.24	19.04	8.68	Curve	1.09	1.0	3.53	
9+500	3.23	19.10	8.70	Straight	1.07	1.0	3.46	
9+600	3.22	19.16	8.73	Curve	1.32	1.0	4.25	
9+700	3.21	19.22	8.75	Curve	1.32	1.0	4.24	
9+800	3.20	19.28	8.78	Straight	1.03	1.0	3.28	
9+900	3.19	19.35	8.80	Straight	1.02	1.0	3.25	
10+000	3.17	19.41	8.83	Straight	1.03	1.0	3.26	
10+100	3.16	19.47	8.85	Straight	1.02	1.0	3.22	
10+200	3.16	19.53	8.88	Straight	1.02	1.0	3.22	3.2
10+300	3.14	19.59	8.90	Straight	1.02	1.0	3.21	
10+400	3.12	19.65	8.93	Straight	1.02	1.0	3.20	
10+500	3.11	19.71	8.95	Straight	1.03	1.0	3.22	
10+600	3.02	19.77	8.98	Straight	1.03	1.0	3.11	
10+700	2.83	19.83	9.00	Curve	1.27	1.0	3.58	
10+800	2.81	19.89	9.03	Curve	1.27	1.0	3.56	3.6
10+900	2.81	19.94	9.05	Curve	1.27	1.0	3.56	
11+000	2.95	19.99	9.08	Straight	1.02	1.0	3.00	
11+100	3.00	20.04	9.10	Straight	1.02	1.0	3.07	
11+200	2.90	20.09	9.13	Straight	1.04	1.0	3.00	
11+300	2.75	20.14	9.15	Straight	1.04	1.0	2.86	
11+400	2.65	20.19	9.18	Straight	1.02	1.0	2.72	2.9
11+500	2.75	20.24	9.20	Straight	1.08	1.0	2.97	
11+600	2.75	20.29	9.23	Straight	1.06	1.0	2.92	
11+700	2.77	20.34	9.25	Straight	1.07	1.0	2.97	
11+800	2.72	20.39	9.28	Curve	1.23	1.0	3.35	
11+900	2.95	20.44	9.30	Curve	1.23	1.0	3.64	
12+000	2.59	20.49	9.33	Curve	1.23	1.0	3.18	3.4
12+100	2.70	20.54	9.35	Curve	1.21	1.0	3.27	
12+200	2.67	20.59	9.38	Straight	1.04	1.0	2.76	
12+300	2.73	20.64	9.40	Straight	1.04	1.0	2.83	
12+400	2.66	20.69	9.43	Straight	1.04	1.0	2.76	
12+500	2.68	20.74	9.45	Straight	1.04	1.0	2.78	
12+600	2.91	20.79	9.48	Straight	1.01	1.0	2.94	
12+700	2.85	20.84	9.50	Straight	1.01	1.0	2.88	2.7
12+800	2.67	20.89	9.53	Straight	1.01	1.0	2.70	
12+900	2.61	20.94	9.55	Straight	1.02	1.0	2.65	
13+000	2.32	20.99	9.58	Straight	1.02	1.0	2.36	
13+100	2.67	21.04	9.60	Straight	1.02	1.0	2.73	
13+200	2.50	21.09	9.63	Straight	1.02	1.0	2.55	
13+300	2.68	21.14	9.65	Straight	1.04	1.0	2.80	

Source: Study Team

(c) Assessment of Maximum Scouring Depth (ΔZ)[4]

The maximum scouring depth (ΔZ) shall be the larger of the estimated maximum scouring depth (ΔZs) evaluated based on river width, water depth, and riverbed material, or the current maximum scouring depth at the design site (ΔZ_g). However, the maximum scouring depth (ΔZ_g) shall be used to the following cases: 1. the estimated maximum scour depth (ΔZ_s) may be excessive considering the current scouring state. 2. No sand bar exists in the channel (b/Hd ≦ 10, representative particle size: d≦0.2 mm).

(i) Estimated Maximum Scouring Depth at the Straight Channel[5]

In a straight river channel with b / H_d> 10 and representative particle size d> 0.2mm, the maximum scouring depth is strongly affected by the sand bar height (H_s). The wave height of the sand bar is controlled by the channel width (B), the water depth (H_n) of the channel, and the particle diameter d (mm) of the substrate. Figure 9.1.2 shows the relationship between (H_s / H_n) and (d / H_n) obtained by hydraulic model experiments. The water depth ($H_{max\cdot s}$) of the maximum scouring part is evaluated as follows based on past experimental data. The relations of these heights are shown in

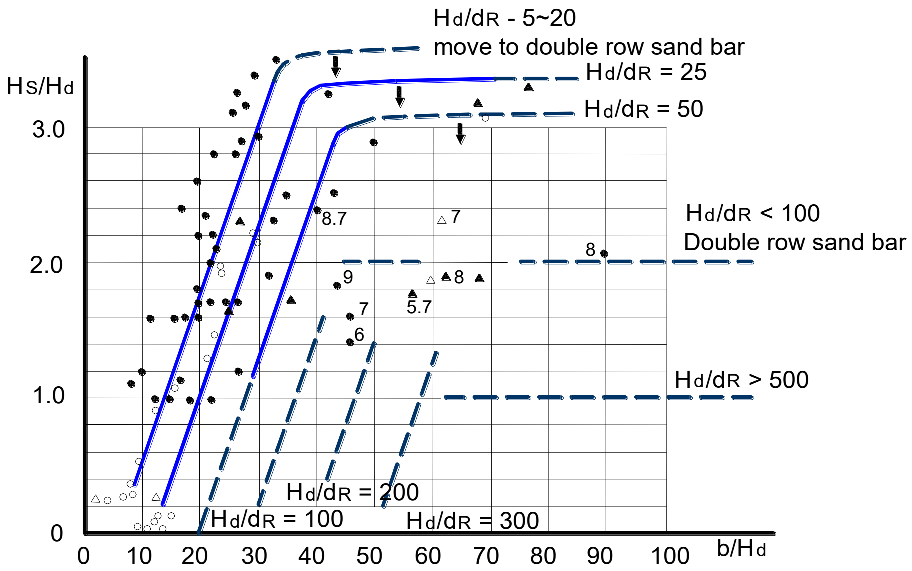
$$H_{max\cdot s} = \{1 + 0.8H_s/H_n\}H_n$$

The bar height at the time of the design flood is able to be evaluated by considering H_n as H_m. Therefore, the scouring depth ΔZ (the difference between the average riverbed height and the deepest riverbed height) of the most scoured part is calculated by the following equation.

$$\Delta Z = H_{max\cdot s} - H_m$$

Here,

ΔZ : Maximum scouring depth(m)
H_{max-s} : Water depth of the most scoured part(m)
H_m : Average water depth during the average annual maximum flow(m)
H_s : Sandbar height(m)

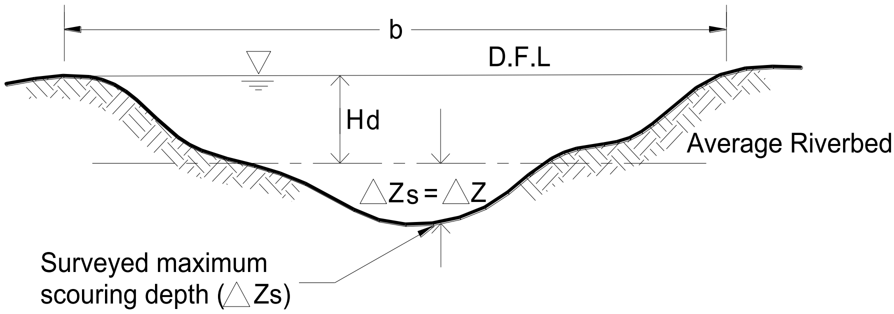


Source: DPWH: Technical Standards And Guidelines For Design Of Flood Control Structures, p39

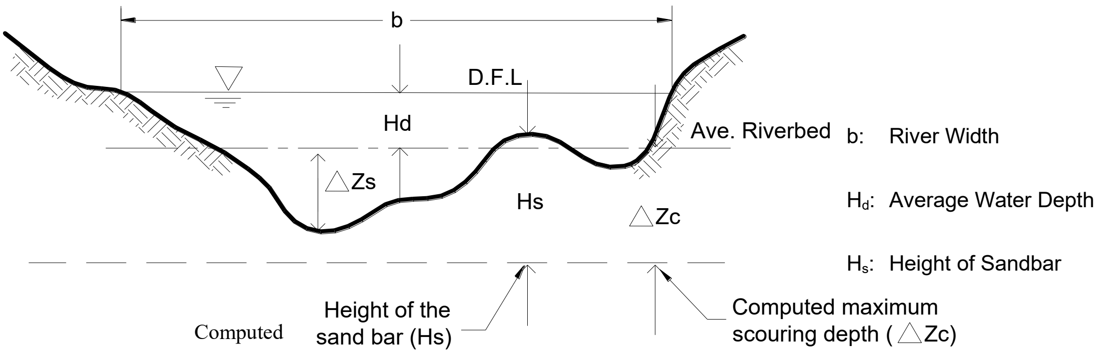
Figure 6.1.18 Relations between H_s/H_d and Hd/d (τ_* : 0.03～0.4)

[4] Dynamical Design Method of Revetment, 4-3, DPWH: Technical Standards and Guidelines for Design of Flood Control Structures, 2.4.3

[5] Dynamical Design Method of Revetment, 4-3, DPWH: Technical Standards and Guidelines for Design of Flood Control Structures, 2.4.3



Deepest Riverbed and Maximum Scouring Depth (ΔZ)
(<u>In case of b/Hd ≦ 10 or dr ≦ 0.2mm</u>)



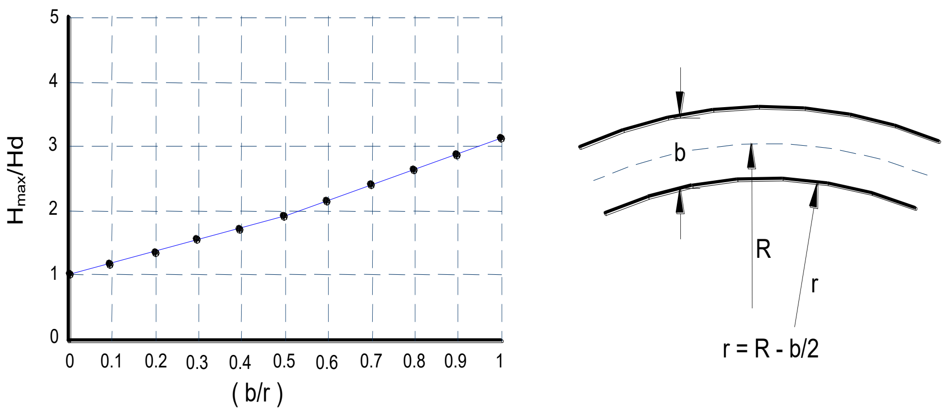
Deepest Riverbed and Maximum Scouring Depth (ΔZ) Influenced by the Height of Sand Bar (<u>In case of b/Hd > 10 or dr > 0.2mm</u>)

Source: DPWH: Technical Standards And Guidelines For Design Of Flood Control Structures, p37, 38

Figure 6.1.19 Illustration of Each Heights

(ii) Estimated Maximum Scouring Depth at the Curved Channel[6]

sentative
from the



Source: Guidelines for Disaster Restoration Works for Conservation of Precious natural surroundings, Japan

Figure 6.1.20 Relationship of H_{max}/H_d and b/r

[6] DPWH:Technical Standards And Guidelines For Design Of Flood Control Structures, 2.4.3,b)

$$\Delta Z = H_{max} - H_d$$

Here,

ΔZ : Maximum scouring depth(m)

H_{max} : Water depth of the most scoured part(m)

H_d : Design Water Depth (m)

(iii) Summary of the Maximum Scouring Depth

The maximum scouring depths of the design section are shown in the following tables.

Table 6.1.20 Maximum Scouring Depth (Downstream of Marikina River: Right Bank)

RIVER STA.	Mean Flow Velocity	SUBSTRATE GRAIN SIZE	Design Riverbed	Present Average River Bed	Present Max. Scour Depth	Estimated Max. Scour Depth	Max. Scour Depth	Lowest Riverbed (①)	Design River Bed −1m (②)	Estimated Lowest Riverbed
	Vm	d	Z	Ha	Zg	Zs	ΔZ	Ha−ΔZ	Z−1	① OR ②
	(m/s)	(mm)	(EL+m)	(EL+m)	(m)	(m)	(m)	(EL+m)	(EL+m)	
5+400	1.34	Rock	7.76	7.76	—	—	—	—	6.76	6.76
5+450	1.36	Rock	7.76	7.76	—	—	—	—	6.76	6.76
5+500	1.27	Rock	7.77	7.77	—	—	—	—	6.77	6.77
5+550	1.28	Rock	7.77	7.77	—	—	—	—	6.77	6.77
5+600	1.25	Rock	7.78	7.78	—	—	—	—	6.78	6.78
5+650	1.26	Rock	7.78	7.78	—	—	—	—	6.78	6.78
5+700	1.34	Rock	7.79	7.79	—	—	—	—	6.79	6.79
5+750	1.36	Rock	7.79	7.79	—	—	—	—	6.79	6.79
5+800	1.33	Rock	7.80	7.80	—	—	—	—	6.80	6.80
5+850	1.38	Rock	7.80	7.80	—	—	—	—	6.80	6.80
5+900	1.32	Rock	7.80	7.80	—	—	—	—	6.80	6.80
5+950	1.34	Rock	7.81	7.81	—	—	—	—	6.81	6.81
6+000	1.36	Rock	7.81	7.81	—	—	—	—	6.81	6.81
6+050	1.71	Rock	7.82	7.82	—	—	—	—	6.82	6.82
6+100	1.54	Rock	7.82	7.82	—	—	—	—	6.82	6.82
6+150	1.54	0.25	7.83	11.64	0.94	1.09	1.09	10.55	6.83	6.83
6+200	1.55	0.25	7.83	11.87	1.01	1.09	1.09	10.77	6.83	6.83
6+250	1.55	0.25	7.84	11.66	1.03	1.09	1.09	10.57	6.84	6.84
6+300	1.55	0.25	7.84	12.03	2.51	1.09	1.09	10.94	6.84	6.84
6+350	1.56	0.25	7.85	13.12	4.26	4.31	4.31	8.81	6.85	6.85
6+400	1.56	0.25	7.85	12.83	3.26	3.29	3.29	9.54	6.85	6.85
6+450	1.66	0.25	7.85	12.30	1.97	1.93	1.93	10.37	6.85	6.85
6+500	1.67	0.25	7.85	12.72	2.35	2.04	2.04	10.68	6.85	6.85
6+550	1.00	0.25	7.85	12.40	1.90	1.11	1.11	11.29	6.85	6.85
6+600	1.00	0.25	7.85	14.12	6.52	2.78	2.78	11.33	6.85	6.85

Source: Study Team

Table 6.1.21 Maximum Scouring Depth (Downstream of Marikina River: Left Bank)

RIVER STA.	Mean Flow Velocity	SUBSTRATE GRAIN SIZE	Design Riverbed	Present Average River Bed	Present Max. Scour Depth	Estimated Max. Scour Depth	Scour Depth	Lowest Riverbed (①)	Design River Bed −1m (②)	Estimated Lowest Riverbed
	Vm	d	Z	Ha	Zg	Zs	ΔZ	Ha−ΔZ	Z−1	① OR ②
	(m/s)	(mm)	(EL+m)	(EL+m)	(m)	(m)	(m)	(EL+m)	(EL+m)	(EL+m)
5+400	1.34	Rock	7.76	7.76	2.29	−	−	−	6.76	6.76
5+450	1.36	Rock	7.76	7.76	1.41	−	−	−	6.76	6.76
5+500	1.27	Rock	7.77	7.77	2.19	−	−	−	6.77	6.77
5+550	1.28	Rock	7.77	7.77	1.13	−	−	−	6.77	6.77
5+600	1.25	Rock	7.78	7.78	1.61	−	−	−	6.78	6.78
5+650	1.26	Rock	7.78	7.78	2.11	−	−	−	6.78	6.78
5+700	1.34	Rock	7.79	7.79	2.02	−	−	−	6.79	6.79
5+750	1.36	Rock	7.79	7.79	2.76	−	−	−	6.79	6.79
5+800	1.33	Rock	7.80	7.80	1.66	−	−	−	6.80	6.80
5+850	1.38	Rock	7.80	7.80	1.66	−	−	−	6.80	6.80
5+900	1.32	Rock	7.80	7.80	2.18	−	−	−	6.80	6.80
5+950	1.34	Rock	7.81	7.81	1.82	−	−	−	6.81	6.81
6+000	1.36	Rock	7.81	7.81	1.83	−	−	−	6.81	6.81
6+050	1.71	Rock	7.82	7.82	1.73	−	−	−	6.82	6.82
6+100	1.54	Rock	7.82	7.82	1.65	−	−	−	6.82	6.82
6+150	1.54	0.25	7.83	11.64	0.94	1.09	1.09	10.55	6.83	6.83
6+200	1.55	0.25	7.83	11.87	1.01	1.09	1.09	10.77	6.83	6.83
6+250	1.55	0.25	7.84	11.66	1.03	1.09	1.09	10.57	6.84	6.84
6+300	1.55	0.25	7.84	12.03	2.51	1.09	1.09	10.94	6.84	6.84
6+350	1.56	0.25	7.85	13.12	4.26	4.31	4.31	8.81	6.85	6.85
6+400	1.56	0.25	7.85	12.83	3.26	3.29	3.29	9.54	6.85	6.85
6+450	1.66	0.25	7.85	12.30	1.97	1.93	1.93	10.37	6.85	6.85
6+500	1.67	0.25	7.85	12.72	2.35	2.04	2.04	10.68	6.85	6.85
6+550	1.00	0.25	7.85	12.40	1.90	1.11	1.11	11.29	6.85	6.85
6+600	1.00	0.25	7.85	14.12	6.52	2.78	2.78	11.33	6.85	6.85

Source: Study Team

Table 6.1.22 Maximum Scouring Depth (Upstream of Marikina River: Right Bank)

RIVER STA.	Mean Flow Velocity	SUBSTRATE GRAIN SIZE	Design Riverbed	Present Average River Bed	Present Max. Scour Depth	Estimated Max. Scour Depth	Max. Scour Depth	Lowest Riverbed (①)	Design River Bed −1m (②)	Estimated Lowest Riverbed
	Vm	d	Z	Ha	Zg	Zs	ΔZ	Ha−ΔZ	Z−1	① OR ②
	(m/s)	(mm)	(EL+m)	(EL+m)	(m)	(m)	(m)	(EL+m)	(EL+m)	(EL+m)
6+650	5.80	0.25	7.85	12.64	2.51	2.09	2.09	10.55	6.85	6.85
6+700	3.69	0.055	8.00	9.63	2.11		0.00	9.63	7.00	7.00
6+800	3.67	0.055	8.03	9.30	1.16	－	1.16	8.14	7.03	7.03
6+900	3.64	0.055	8.05	8.05	1.10	－	1.10	6.96	7.05	6.96
7+000	3.62	0.055	8.08	7.63	0.87	－	0.87	6.76	7.08	6.76
7+100	3.60	0.055	8.10	7.51	0.70	－	0.70	6.81	7.10	6.81
7+200	3.58	0.055	8.13	7.04	1.10	－	1.10	5.93	7.13	5.93
7+300	3.52	0.055	8.15	8.15	1.50	－	1.50	6.65	7.15	6.65
7+400	3.44	0.055	8.18	8.18	2.04	－	2.04	6.14	7.18	6.14
7+500	3.43	0.055	8.20	8.19	1.91	－	1.91	6.28	7.20	6.28
7+600	3.46	0.055	8.23	8.90	1.36	－	1.36	7.54	7.23	7.23
7+700	3.47	0.055	8.25	9.45	0.69	－	0.69	8.76	7.25	7.25
7+800	3.43	0.055	8.28	8.89	0.28	－	0.28	8.61	7.28	7.28
7+900	3.42	0.055	8.30	8.76	0.11	－	0.11	8.65	7.30	7.30
8+000	3.40	0.055	8.33	9.29	0.25	－	0.25	9.05	7.33	7.33
8+100	3.39	0.055	8.35	9.39	0.44	－	0.44	8.95	7.35	7.35
8+200	3.38	0.055	8.38	9.98	1.08	－	1.08	8.90	7.38	7.38
8+300	3.34	0.055	8.40	9.65	0.44	－	0.44	9.21	7.40	7.40
8+400	3.33	0.055	8.43	8.80	0.74	－	0.74	8.06	7.43	7.43
8+500	3.31	0.055	8.45	8.83	1.14	－	1.14	7.70	7.45	7.45
8+600	3.30	0.055	8.48	9.96	0.91	－	0.91	9.05	7.48	7.48
8+700	3.29	0.055	8.50	9.54	0.29	1.10	1.10	8.44	7.50	7.50
8+800	3.28	0.055	8.53	9.90	0.95	1.10	1.10	8.80	7.53	7.53
8+900	3.27	0.055	8.55	9.53	1.02	1.10	1.10	8.43	7.55	7.55
9+000	3.28	0.055	8.58	9.53	1.11	1.10	1.10	8.43	7.58	7.58
9+100	3.27	0.055	8.60	9.97	1.34	1.10	1.10	8.87	7.60	7.60
9+200	3.26	0.055	8.63	9.93	0.90	1.10	1.10	8.83	7.63	7.63
9+300	3.25	0.055	8.65	9.33	0.93	1.10	1.10	8.23	7.65	7.65
9+400	3.24	0.055	8.68	9.53	1.14	1.10	1.10	8.43	7.68	7.68
9+500	3.23	0.055	8.70	9.45	1.50	－	1.50	7.95	7.70	7.70
9+600	3.22	0.055	8.73	9.46	1.52	※	1.52	7.95	7.73	7.73
9+700	3.21	0.055	8.75	9.02	0.67	※	0.67	8.36	7.75	7.75
9+800	3.20	0.055	8.78	9.02	0.57	－	0.57	8.45	7.78	7.78
9+900	3.19	0.055	8.80	8.80	0.42	－	0.42	8.38	7.80	7.80
10+000	3.17	0.055	8.83	9.45	0.61	－	0.61	8.84	7.83	7.83
10+100	3.16	0.055	8.85	10.00	0.40	－	0.40	9.60	7.85	7.85
10+200	3.16	0.055	8.88	9.81	0.41	－	0.41	9.41	7.88	7.88
10+300	3.14	0.055	8.90	9.84	0.45	－	0.45	9.39	7.90	7.90
10+400	3.12	0.055	8.93	9.52	0.52	－	0.52	9.01	7.93	7.93
10+500	3.11	0.055	8.95	9.49	0.73	－	0.73	8.77	7.95	7.95
10+600	3.02	0.055	8.98	9.80	0.63	－	0.63	9.18	7.98	7.98
10+700	2.83	Rock	9.00	9.44	0.59	0.60※	0.60	8.84	8.00	8.00
10+800	2.81	Rock	9.03	9.84	1.47	1.50※	1.50	8.34	8.03	8.03
10+900	2.81	Rock	9.05	9.65	0.30	1.30※	1.30	8.35	8.05	8.05
11+000	2.95	0.055	9.08	9.95	0.41	－	0.41	9.55	8.08	8.08
11+100	3.00	0.055	9.10	9.44	0.49	－	0.49	8.95	8.10	8.10
11+200	2.90	0.055	9.13	9.94	0.81	－	0.81	9.13	8.13	8.13
11+300	2.75	0.055	9.15	10.14	0.88	－	0.88	9.26	8.15	8.15
11+400	2.65	0.055	9.18	9.48	0.55	－	0.55	8.94	8.18	8.18
11+500	2.75	0.055	9.20	9.95	1.75	－	1.75	8.20	8.20	8.20
11+600	2.75	0.055	9.23	9.74	1.31	－	1.31	8.43	8.23	8.23
11+700	2.77	0.055	9.25	9.58	1.56	－	1.56	8.02	8.25	8.02
11+800	2.72	0.055	9.28	9.65	1.57	1.10※	1.10	8.55	8.28	8.28
11+900	2.95	0.055	9.30	9.59	1.62	1.10※	1.10	8.49	8.30	8.30
12+000	2.59	0.055	9.33	9.61	1.54	0.60※	0.60	9.01	8.33	8.33
12+100	2.70	0.055	9.35	9.78	1.18	0.60※	0.60	9.18	8.35	8.35
12+200	2.67	0.055	9.38	9.87	0.79	－	0.79	9.08	8.38	8.38
12+300	2.73	0.055	9.40	10.01	0.85	－	0.85	9.17	8.40	8.40
12+400	2.66	0.055	9.43	10.08	0.81	－	0.81	9.27	8.43	8.43
12+500	2.68	0.055	9.45	10.26	0.79	－	0.79	9.47	8.45	8.45
12+600	2.91	0.055	9.48	10.09	0.27	－	0.27	9.83	8.48	8.48
12+700	2.85	0.055	9.50	10.05	0.27	－	0.27	9.78	8.50	8.50
12+800	2.67	0.055	9.53	10.07	0.32	－	0.32	9.76	8.53	8.53
12+900	2.61	0.055	9.55	10.27	0.44	－	0.44	9.84	8.55	8.55
13+000	2.32	0.055	9.58	10.27	0.38	－	0.38	9.89	8.58	8.58
13+100	2.67	0.055	9.60	10.38	0.51	－	0.51	9.87	8.60	8.60
13+200	2.50	0.055	9.63	10.69	0.54	－	0.54	10.15	8.63	8.63
13+300	2.68	0.055	9.65	10.63	0.99	－	0.99	9.64	8.65	8.65

Source: Study Team

Table 6.1.23 Maximum Scouring Depth (Upstream of Marikina River: Left Bank)

RIVER STA.	Mean Flow Velocity	SUBSTRATE GRAIN SIZE	Design Riverbed	Present Average River Bed	Present Max. Scour Depth	Estimated Max. Scour Depth	Max. Scour Depth	Lowest Riverbed (①)	Design River Bed −1m (②)	Estimated Lowest Riverbed
	Vm	d	Z	Ha	Zg	Zs	ΔZ	Ha−ΔZ	Z−1	① OR ②
	(m/s)	(mm)	(EL+m)	(EL+m)	(m)	(m)	(m)	(EL+m)	(EL+m)	(EL+m)
6+650	5.80	0.25	7.85	12.64	2.51	2.09	2.09	10.55	6.85	6.85
6+700	3.69	0.055	8.00	9.63	2.11	−	2.11	7.52	7.00	7.00
6+800	3.67	0.055	8.03	9.30	1.16	−	1.16	8.14	7.03	7.03
6+900	3.64	0.055	8.05	8.05	1.10	−	1.10	6.96	7.05	6.96
7+000	3.62	0.055	8.08	7.63	0.87	−	0.87	6.76	7.08	6.76
7+100	3.60	0.055	8.10	7.51	0.70	−	0.70	6.81	7.10	6.81
7+200	3.58	0.055	8.13	7.04	1.10	−	1.10	5.93	7.13	5.93
7+300	3.52	0.055	8.15	8.15	1.50	−	1.50	6.65	7.15	6.65
7+400	3.44	0.055	8.18	8.18	2.04	−	2.04	6.14	7.18	6.14
7+500	3.43	0.055	8.20	8.19	1.91	−	1.91	6.28	7.20	6.28
7+600	3.46	0.055	8.23	8.90	1.36	−	1.36	7.54	7.23	7.23
7+700	3.47	0.055	8.25	9.45	0.69	−	0.69	8.76	7.25	7.25
7+800	3.43	0.055	8.28	8.89	0.28	−	0.28	8.61	7.28	7.28
7+900	3.42	0.055	8.30	8.76	0.11	−	0.11	8.65	7.30	7.30
8+000	3.40	0.055	8.33	9.29	0.25	−	0.25	9.05	7.33	7.33
8+100	3.39	0.055	8.35	9.39	0.44	−	0.44	8.95	7.35	7.35
8+200	3.38	0.055	8.38	9.98	1.08	−	1.08	8.90	7.38	7.38
8+300	3.34	0.055	8.40	9.65	0.44	−	0.44	9.21	7.40	7.40
8+400	3.33	0.055	8.43	8.80	0.74	−	0.74	8.06	7.43	7.43
8+500	3.31	0.055	8.45	8.83	1.14	−	1.14	7.70	7.45	7.45
8+600	3.30	0.055	8.48	9.96	0.91	−	0.91	9.05	7.48	7.48
8+700	3.29	0.055	8.50	9.54	0.29	1.01	1.01	8.53	7.50	7.50
8+800	3.28	0.055	8.53	9.90	0.95	1.01	1.01	8.89	7.53	7.53
8+900	3.27	0.055	8.55	9.53	1.02	1.02	1.02	8.51	7.55	7.55
9+000	3.28	0.055	8.58	9.53	1.11	1.02	1.02	8.50	7.58	7.58
9+100	3.27	0.055	8.60	9.97	1.34	1.03	1.03	8.94	7.60	7.60
9+200	3.26	0.055	8.63	9.93	0.90	1.03	1.03	8.90	7.63	7.63
9+300	3.25	0.055	8.65	9.33	0.93	1.03	1.03	8.30	7.65	7.65
9+400	3.24	0.055	8.68	9.53	1.14	1.04	1.04	8.50	7.68	7.68
9+500	3.23	0.055	8.70	9.45	1.50	−	1.50	7.95	7.70	7.70
9+600	3.22	0.055	8.73	9.46	1.52	3.00※	3.00	6.46	7.73	6.46
9+700	3.21	0.055	8.75	9.02	0.67	3.00※	3.00	6.02	7.75	6.02
9+800	3.20	0.055	8.78	9.02	0.57	−	0.57	8.45	7.78	7.78
9+900	3.19	0.055	8.80	8.80	0.42	−	0.42	8.38	7.80	7.80
10+000	3.17	0.055	8.83	9.45	0.61	−	0.61	8.84	7.83	7.83
10+100	3.16	0.055	8.85	10.00	0.40	−	0.40	9.60	7.85	7.85
10+200	3.16	0.055	8.88	9.81	0.41	−	0.41	9.41	7.88	7.88
10+300	3.14	0.055	8.90	9.84	0.45	−	0.45	9.39	7.90	7.90
10+400	3.12	0.055	8.93	9.52	0.52	−	0.52	9.01	7.93	7.93
10+500	3.11	0.055	8.95	9.49	0.73	−	0.73	8.77	7.95	7.95
10+600	3.02	0.055	8.98	9.80	0.63	−	0.63	9.18	7.98	7.98
10+700	2.83	0.055	9.00	9.44	0.59	3.20※	3.20	6.24	8.00	6.24
10+800	2.81	0.055	9.03	9.84	1.47	3.20※	3.20	6.64	8.03	6.64
10+900	2.81	0.055	9.05	9.65	0.30	3.20※	3.20	6.45	8.05	6.45
11+000	2.95	0.055	9.08	9.95	0.41	−	0.41	9.55	8.08	8.08
11+100	3.00	0.055	9.10	9.44	0.49	−	0.49	8.95	8.10	8.10
11+200	2.90	0.055	9.13	9.94	0.81	−	0.81	9.13	8.13	8.13
11+300	2.75	0.055	9.15	10.14	0.88	−	0.88	9.26	8.15	8.15
11+400	2.65	0.055	9.18	9.48	0.55	−	0.55	8.94	8.18	8.18
11+500	2.75	0.055	9.20	9.95	1.75	−	1.75	8.20	8.20	8.20
11+600	2.75	0.055	9.23	9.74	1.31	−	1.31	8.43	8.23	8.23
11+700	2.77	0.055	9.25	9.58	1.56	−	1.56	8.02	8.25	8.02
11+800	2.72	0.055	9.28	9.65	1.57	※	1.57	8.08	8.28	8.08
11+900	2.95	0.055	9.30	9.59	1.62	※	1.62	7.98	8.30	7.98
12+000	2.59	0.055	9.33	9.61	1.54	※	1.54	8.08	8.33	8.08
12+100	2.70	0.055	9.35	9.78	1.18	※	1.18	8.60	8.35	8.35
12+200	2.67	0.055	9.38	9.87	0.79	−	0.79	9.08	8.38	8.38
12+300	2.73	0.055	9.40	10.01	0.85	−	0.85	9.17	8.40	8.40
12+400	2.66	0.055	9.43	10.08	0.81	−	0.81	9.267	8.43	8.43
12+500	2.68	0.055	9.45	10.26	0.79	−	0.79	9.4735	8.45	8.45
12+600	2.91	0.055	9.48	10.09	0.27	−	0.27	9.825	8.48	8.48
12+700	2.85	0.055	9.50	10.05	0.27	−	0.27	9.783	8.50	8.50
12+800	2.67	0.055	9.53	10.07	0.32	−	0.32	9.756	8.53	8.53
12+900	2.61	0.055	9.55	10.27	0.44	−	0.44	9.8395	8.55	8.55
13+000	2.32	0.055	9.58	10.27	0.38	−	0.38	9.8865	8.58	8.58
13+100	2.67	0.055	9.60	10.38	0.51	−	0.51	9.868	8.60	8.60
13+200	2.50	0.055	9.63	10.69	0.54	−	0.54	10.1515	8.63	8.63
13+300	2.68	0.055	9.65	10.63	0.99	−	0.99	9.6425	8.65	8.65

Source: Study Team

※ *When the estimated scour depth is calculated for the surface sandy soil layer, the scour depth is excessively large compared to the actual current scour depth. Thus, the top of the formation was evaluated as the estimated maximum scour depth. The cohesive soil layer was extracted with reference to the representative grain size and soil profile at each depth.*

(2) Study on the Structure of Foot Protection for Low Water Channel

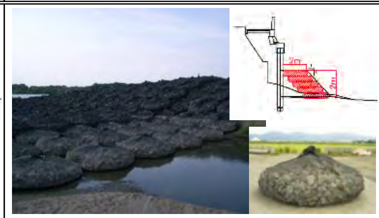
1) Selection of Foot Protection Structure

The following four types of foot protection structures have been compared (**Table 6.1.24**). The results are as follows:

- Case 1: Rock-place method (riprap) – which was applied in Phases II and III
- Case 2: Gabion mattress with anticorrosive treatment
- Case 3: Bag-type foot protection
- Case 4: Geotextile gabion mattress

As a result of the comparison of the four types, Case 1 (Riprap) which has high superiority in terms of the number of applications, ecological considerations, flexibility, maintenance and workability, as well as economic efficiency, will be adopted.

Table 6.1.24 Comparative Selection Table of Foot Protection Structure

Construction Method	Case-1: Riprap (On going Phase II)		Case-2: Galvanized Gabion Mattress		Case-3: Geotextile Gabion Bag for Foot Protection (Bottle Unit)		Case-4: Geotextile Gabion Mattress	
Photography/ General Drawing								
Abstract of the Method	- Dry boulder riprap (300-400mm in diameter) - Standard method of foot protection in Philippines - No serious issue of workability and functionality based on the ongoing Phase II - Relatively expensive because of transportation form other islands		- Cage made of steel wires, which are galvanized because of countermeasures for brackish water - It is desirable not to use this at the places of the strong acidity and high salinity. - Use in the locations where boulders and driftwood must be careful.		- Geotextile gabion bags made of the recycled polyester form PET bottles - Due to the high strength chemical fibers, it is possible to be used at the low pH area and river mouth without rusting. - Use in the locations where boulders and driftwood must be careful.		- Geotextile gabions mainly made form polyethylene - It is used for alternative method in stead of gabion with retaining the characteristics of the gabion. - Due to the high strength chemical fibers, It is possible to be used at the low pH area and river mouth without rusting. - use in the locations where boulders and driftwood must be careful.	
Material	Fieldstone Gravel (Class-B)		Galvanized Wire		Geotextile: Recycled Polyester		Geotextile: Polyethylene	
General Allowable Velocity	Va = 5.0m/s > 3.0m/s (OK)	◎	Va = 6.0m/s > 3.0m/s (OK)	◎	Va = 4.3m/s > 3.0m/s (OK)	◎	Va = 6.0m/s > 3.0m/s (OK)	◎
Adaption for Project	Many projects especially in Philippines	◎	Many in Philippines and Japan, but a few in brackish water	◎/△	Recently increasing in Japan	○	Not many compared with Case-3 due to expensive cost	△
Durability for Brackish Water	- No problems of the fieldstone	◎	- Based on the result of Phase I investigation, the average chloride value of 1617 (ranged from 80 to 6000) mg/L in dry season is higher than 450 mg/L which is allowable content without the wire courting. - Because the salinity varies depending on the sampling depth, deep-water parts still remain of particular concern.	△	- No problems of the fieldstone - No issue about the bags due to the result against 3% sodium chloride test approved by Public Works Research Center	◎	- No problems of the fieldstone - No issue about the bags due to the result of sodium chloride and other tests	○
Environmental Aspect	- No problems of the fieldstone	◎	- For zinc used in rust prevention treatment, report (less than 30 mg/L) has been made of setting environmental standard form the Ministry of the Environment in Japan. - Because of the large mesh size, it is difficult to greening by plants.	△	- Hazardous substances, such as environmental hormones, do not contain because it is made form recycled PET bottles and polyester fibers. - Because of the small mesh size, it is easier than Case-2 to greening by plants.	○	- Hazardous substances, such as environmental hormones, do not contain because It is made form high strength chimerical fibers. - because of the small mesh size, It is easier than Case-2 to greening by plants.	○
Followability against Riverbed Deformation	- It is possible to adjust the deformation of the riverbed.	◎	- It is concerned about mal-function for foot protection due to spaces between each others caused by deformations of riverbed. - The wire diameter of gabion in Philippines is generally thinner than Japan's one, so it is easier to follow the deformation.	△	- It is possible to follow the deformation of the riverbed because of the flexible material bags.	◎	- It is concerned about mal-function for foot protection due to spaces between each others caused by deformations of riverbed.	△
Effect against River Navigation	- The request for changing foot protection method form the river navigation's stakeholders, because barges have been damaged by contact.	△	- Less influences against river navigation due to weak strength of the cage and small size particles of fill materials	◎	- Less influences against river navigation due to flexible bag and small size particles of fill materials	◎	- Less influences against river navigation due to flexible gabion cage and small size particles of fill materials	◎
Maintenance and Repair	- Easy due to dumping and leveling on the top layer	◎	- Relatively not so easy for repair due to dry work needed	△	- easy due to additional setting on the top of geotextile bag	○	- Relatively not so easy for repair due to dry work needed	△
Workability: Site Preparation	- Only rough leveling		- Need for elaborate leveling before construction		- Only rough leveling due to good flexibility		- Need for elaborate leveling before construction	
Workability: Mechanized Construction	- Possible	◎	- Manual set on the site and backfill by the manual and machine	△	- Possible	○	- Manual set on the site and backfill by the manual and machine	△
Workability: Under Water Execution	- Possible		- Basically dry work condition - Coffering or diver is required for some cases.		- Possible		- Basically dry work condition - Coffering or diver is required for some cases.	
Workability: Construction Speed	- Speedy due to dumping and leveling mainly by backhoe		- Relatively slow due to the man-power construction		- Relatively fast due to mainly mechanized construction		- Relatively slow due to the man-power construction	
Cost (Peso/m)	Material Costs ***** Const. Costs ***** **Total Costs** ***** **Peso/m**	1	Material Costs ***** Const. Costs ***** **Total Costs** ***** **Peso/m**	2	Material Costs ***** Const. Costs ***** **Total Costs** ***** **Peso/m**	3	Material Costs ***** Const. Costs ***** **Total Costs** ***** **Peso/m**	4
Evaluation	- The most of the evaluations, especially including the cost, are better than the others and the damages of the barges are not so serious problems.		- The improvement for the river navigation is well, but the cost is greatly more expensive than the Case-1. In addition, the durability against the brackish water is questionable. Therefore, it is difficult to select this method.		- The improvement for the river navigation is well, but the cost is more expensive than the Case-1 and the Case-2. However, it is still possible to be adapted in the partial areas, such as foot protections about the piers because of the characteristics.		- The cost is most expensive than the others. Structure life is longer than the Case-2.	
	◎: Selection		△: Third Place		○: Second Place		△: Fourth Place	

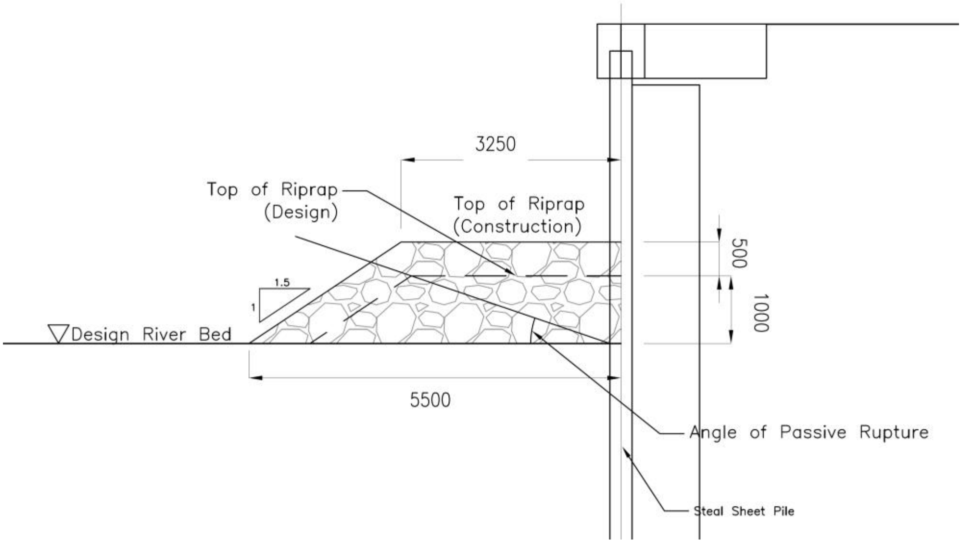
Symbol Legend - ◎:Very Good, ○:Good, △:Fare, ×:Poor or Problem

Note: Cost is not presented due to the prior released version.

Source: Study Team

2) Cross-section of Riprap

The cross-sectional shape of the riprap is set based on the collapse angle by the passive earth pressure of SSP. The typical cross-section is shown below. The height is 1.5 m, including scouring height, 50 cm.



Source: Study Team

Figure 6.1.21 Typical Cross-section of Riprap（Height 1.5m）

In order not to decrease the cross-sectional area of the river, the riprap should be small. However, in the case, that sufficient stability of the SSP revetment cannot be ensured, or in the case that the scale of the SSP revetment becomes quite large, the riprap is raised partially to expect more resistance of passive earth load.

3) Diameter of Foot Protection

(a) Riprap

To resist the flow velocity during flood, size of riprap shall be determined by the formula of "Traction-Weak Integrity Model" according to "Dynamical Design Method of Revetment ". It is assumed that the tractive force acting on the riprap does not exceed the threshold of riprap movement. The relationship between the representative flow velocity V0 and the riprap size is determined as follows:

A. Riprap on Flat Bed

$$D_m = \frac{1}{E_1^2 \cdot 2g\left[\frac{\rho_s}{\rho_w} - 1\right]} V_0^2$$

Where,

D_m : Average size of riprap (m)
V_0 : Representative flow velocity (m/s)
ρ_s : Density of riprap (kg/m^3)
g : Gravitational acceleration (m/s^2)
ρ_w : Water density (kg/m^3) (ca.2.65as usual)
E_1 : Experimental factor of magnitude of turbulence

As the experimental factor E_1, in the case of a relatively small turbulent flow, E_1 = 1.2 is usually used. In the case of large turbulent flow, the value of E_1 = 0.86 is proposed.

B. Riprap on Slope with Inclination Angle of θ[7]

Diameter of riprap will be decided as the value $K \cdot D_m$, obtained by multiplying particle diameter D_m by slope correction coefficient K.

$$K = \frac{1}{cos\theta\sqrt{1 - \frac{tan^2\theta}{tan^2\varphi}}}$$

Where,

ϕ : ac. 38°for natural rock, ca.41°for macadam.

The design diameter of the material for the riprap uses the value in the following table calculated backward from the reference mass of the Philippines.

Table 6.1.25 Size of Riprap

Unit Weight of Stones = 2.5 t/m3 (g/cm3)

Class	Minimum		Maximum		50% Weighting		
	Weight (kg)	Radius (cm)	Weight (kg)	Radius (cm)	Weight (kg)	Radius (cm)	Diameter (cm)
A	15	11.3	25	13.4	20	12.4	24.8
B	30	14.2	70	17.9	50	16.8	33.7
C	60	17.9	100	21.2	80	19.7	39.4
D	100	21.2	200	26.7	150	24.3	48.6

Source: Study Team

(b) Bag Type Foot Protection Work

The hydraulic limit flow velocity of the bag type foot protection work is shown in **Table 6.1.26** according to the hydraulic model experiments. The method for setting the size of the filling material is the same as that of the above-mentioned riprap.

Table 6.1.26 The Relationship Between Bag-Type Foot Protection Work Weight and Movement Limit Flow Velocity(m/s)

Weight	Single	Group
1t	2.58	3.78
2t	2.90	4.35
3t	3.10	4.65
4t	3.40	5.10

Source: River Bureau-related documents; Advanced Construction Technology Center: Design and construction technical manual for bag-type foot protection work (draft), ACTEC Technical No. 97502, 1996, pp 51-54

(c) Calculation Conditions and Results

The weight and size of the foot protection works were calculated using the design flow velocity. The results are shown in **Table 6.1.27** and **Table 6.1.28**.

[7] U.S. Army Corps of Engineer: Hydraulic design criteria, chart 712-4, 1970

Table 6.1.27 Foot Protection Type (Right Bank)

Section	Bed Protection Type	Design Velocity(m/s)	Dm (m)	Class	Remarks
5+400-5+650	-	2.0	-	-	Riverbed: rock
5+650-5+950	RIPRAP	3.0	0.22	A	
5+950-6+100	-	1.5	-	-	MCGS
6+100-6+350	RIPRAP	1.7	0.07	A	
6+350-6+650	RIPRAP	1.6	0.07	A	
6+650-7+300	RIPRAP	3.9	0.36	B	
7+300-8+700	RIPRAP	3.6	0.30	B	
8+700-9+800	RIPRAP	3.7	0.33	B	
9+800-10+700	RIPRAP	3.2	0.25	B	
10+700-11+000	RIPRAP	3.7	(0.33)	(B)	Riverbed: rock
11+000-11+800	RIPRAP	2.9	0.21	A	
11+800-12+200	RIPRAP	3.2	0.24	A	
12+200-13+400	RIPRAP	2.7	0.18	A	

Source: Study Team

Table 6.1.28 Foot Protection Type (Left Bank)

Section	Bed Protection Type	Design Velocity(m/s)	Dm (m)	Class	Remarks
5+400-5+700	RIPRAP SLOPE: 3:1	2.0	0.1	A	Riverbed: rock
5+700-5+950	-	4.0	-	-	
5+950-6+100	-	1.5	-	-	MCGS
6+100-6+350	RIPRAP	1.7	0.07	A	
6+350-6+650	POLYESTER NET GABION	1.9		1 t	Filler RIPRAP CLASS A
	RIPRAP	1.9	0.09	A	
6+650-7+300	RIPRAP	4.5	0.48	D	
7+300-8+700	RIPRAP	3.6	0.30	B	
8+700-9+800	RIPRAP	3.7	0.34	C	
9+800-10+700	RIPRAP	3.2	0.25	B	
10+700-11+000	RIPRAP	3.6	0.31	B	
11+000-11+800	RIPRAP	2.9	0.21	A	
11+800-12+200	RIPRAP	3.4	0.28	C	

Source: Study Team

6.1.4 Study on Foot Protection of Bridge Substructure

6.1.4.1 Target Bridges

As shown in **Figure 6.1.22**, there are six (6) bridges across the Marikina River in the target section of the Project. Among them, the target bridges were set to four (4) bridges: Macapagal Bridge, LRT2 Bridge, Marcos Bridge, SM Marikina Bridge. Manalo Bridge and Marikina Bridge are excluded from this design for the following reasons.

- Manalo Bridge: The foot protection as detailed design of scouring protection is included in the detailed design of the newly rebuilt bridge implemented in Phase II SA2. The foot protection as detailed design of scouring protection is included in the detailed design of the newly rebuilt bridge implemented in Phase II SA2. The current deepest riverbed in the river section (STA.7+200) in the vicinity of the bridge is deeper than the planned riverbed in some areas, and there is a possibility that the embedment of the bridge foundation turns out to be inadequacy. However, it is not necessary to newly design because the area of the foot protection is set up in a range that can be sufficiently covered even in consideration of this lack of embedment. Before the actual construction, it is recommended to carry out a river crossing survey at the bridge point, and change the scope of protection work and cross-sectional shape if necessary after confirming the current riverbed.
- Marikina Bridge: “Consulting services for the detailed engineering design for retrofitting/strengthening of permanent bridges (Marikina and Mandaluyong bridges)” conducted by DPWH-NCR includes the detailed design of scouring protection

Source: Study Team Added on Google Earth

Figure 6.1.22 Target Bridges

6.1.4.2 Selection of Foot Protection

In order to prevent the stability of the pier from deteriorating due to scouring around the pier, in case the height of the pier's foundation crown is less than 2.0m below the deepest riverbed level of the existing survey section or from the riverbed, foot protection will be installed upstream and downstream of the pier’s foundation. As a first selection, based on the results in **Table 5.1.10 of the Basic Design Report 5.1.3.5 (2) -1)**, " Riprap" and " Geotextile Gabion Bag for Foot Protection (Polyester net gabion)" which are ranked first and second are selected for comparative examination of foot protection. As a secondary selection, compare the economics of applying the two methods to each bridge substructure and adopt the superior method.

6.1.4.3 Examination of the Foot Protection

(1) Riprap

Table 6.1.29 shows the design diameter of the riprap at the bridge position of each bridge calculated by the equation described in **the Basic Design Report 5.1.3.5 (2) -2)**.

Table 6.1.29 Design Diameter of Riprap

Name of Bridge	Representative Flow Velocity (m/s)	K value	Average Size of Riprap (cm)	Class
Macapagal Bridge	3.258	1.368	34.3	C
LRT-2 Bridge	3.218	1.368	33.4	B
Marcos Bridge	3.206	1.368	33.2	B
SM Marikina Bridge	3.206	1.368	33.2	B

Source: Study Team

(2) Polyester Net Gabion for Foot Protection

The polyester net gabion is a method of placing bag materials in a special formwork, putting filled materials into them, binding the mouth by using backhoe, lifting and moving to a required location, and installing. **Table 6.1.30** shows the moving limit flow velocity of each weight of the polyester net gabion. The form in which the polyester net gabion is connected by ropes in the longitudinal and transverse directions is called a gathered type, and it can withstand a flow velocity 1.5 times that of a single type installation. Based on the representative flow velocity at each bridge position shown in **Table 6.1.29**, all bridges can be applied with a weight of 1 ton (gathered type). The output is 1700mm in diameter, 550mm in height and 300mm in outer circumference. Therefore, when installed, the height of one bag will be 425mm on average, and the foot protection will be two layers.

Table 6.1.30 Moving Limit Flow Velocity of Polyester Net Gabion

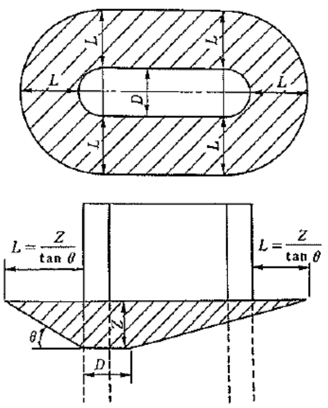
Weight	Moving Limit Flow Velocity (m/s)	
	Single	Gathered
1t	2.58 m/s	3.87 m/s
2t	2.90 m/s	4.35 m/s
3t	3.10 m/s	4.65 m/s
4t	3.40 m/s	5.10 m/s

Source: Study Team

(3) Area of Scouring

The estimation of the scouring area was based on the "Hydraulic Study on Prediction and Countermeasures of Local Scouring Depth by Bridge Pier (*Published by Public Works Research Institute, Japan -1982*)", and the estimation formula shown in Figure 5.22 was used.

Table 6.1.31 shows the installation range of foot protection for each bridge. The area of scouring is approximately 3-7m, and there is a step in the designed riverbed due to foundation and dredging. Considering the above, foot protection are planned to be wider than the following calculation range.



$$\frac{Z}{D} = f\left(\frac{h_0}{D}, \frac{h_0}{d_m}, F_r\right)$$

L : Scour Range (m)
Z : Maximum Scour Depth (m)
θ: Angle of Repose in Water (°)
D : Bridge Pier Width (m)

h_0 : Average Water Depth (m)
d_m : Average Size of Riprap (m)
Fr : Froude Number

Source: "Hydraulic Study on Prediction and Countermeasures of Local Scouring Depth by Bridge Pier (Published by Public Works Research Institute, Japan -1982)"

Figure 6.1.23 Area of Scouring around Pier and Estimated Schematic

Table 6.1.31 Chart of Area of Scouring

Name of Bridge	Construction of Bridge Piers (Number of rows in the vertical direction × Number of row in the transverse direction of the river)	Bridge Pier Width (m)				Area of Scouring (m)		Range of Foot Protection (m)	
		Section Direction ：D		Profile Direction ：H		Maximum Scour Depth ：Z	Scour Range ：L= Z/tanθ		
		Column	Foundation	Column	Foundation				
Macapagal Bridge	4column ×3rows	1.5	-	1.5	-	2.0	3.38	From Pier End	4.0m or more
LRT-2 Bridge	1 column ×4rows	3.0	7.5	3.0	10.0	3.6	6.24	From Pier End	7.0m or more
Marcos Bridge	4column ×4rows	1.3	5.0	1.3	25.0	1.7	2.93	From Foundation End	3.0m or more
SM Marikina Bridge	1column ×4rows	2.0	2.5	2.0	2.5	2.4	4.16	From Pier End	5.0m or more

Source: Study Team

(4) Comparison of Estimated Construction Cost

Table 6.1.32 shows the estimated construction cost for each method.

Table 6.1.32 Estimated Construction Cost

1. Riprap

Name of Bridge	Range of Foot Protection				Unit Cost (peso/m3)	Estimated Construction Cost(peso) per pier
	Transverse Direction (m)	Longitudinal Direction (m)	Height of Installation (m)	Volume (m3)		
Macapagal Bridge	9.50	26.75	1.00	254	*****	*****
LRT-2 Bridge	17.00	17.00	1.00	289	*****	*****
Marcos Bridge	21.96	41.96	1.00	921	*****	*****
SM Marikina Bridge	12.00	12.00	1.00	144	*****	*****

2. Geotextile Gabion Bag for Foot Protection (Polyester net gabion)

Name of Bridge	Range of Foot Protection				Unit Cost (peso/m3)	Estimated Construction Cost(peso) per pier
	Transverse Direction (m)	Longitudinal Direction (m)	Height of Installation (m)	Volume (m3)		
Macapagal Bridge	10.20	27.20	0.85	192	*****	*****
LRT-2 Bridge	17.00	17.00	0.85	200	*****	*****
Marcos Bridge	22.10	42.50	0.85	650	*****	*****
SM Marikina Bridge	13.60	13.60	0.85	128	*****	*****

Note: Cost is not presented due to the prior released version.
Source: Study Team

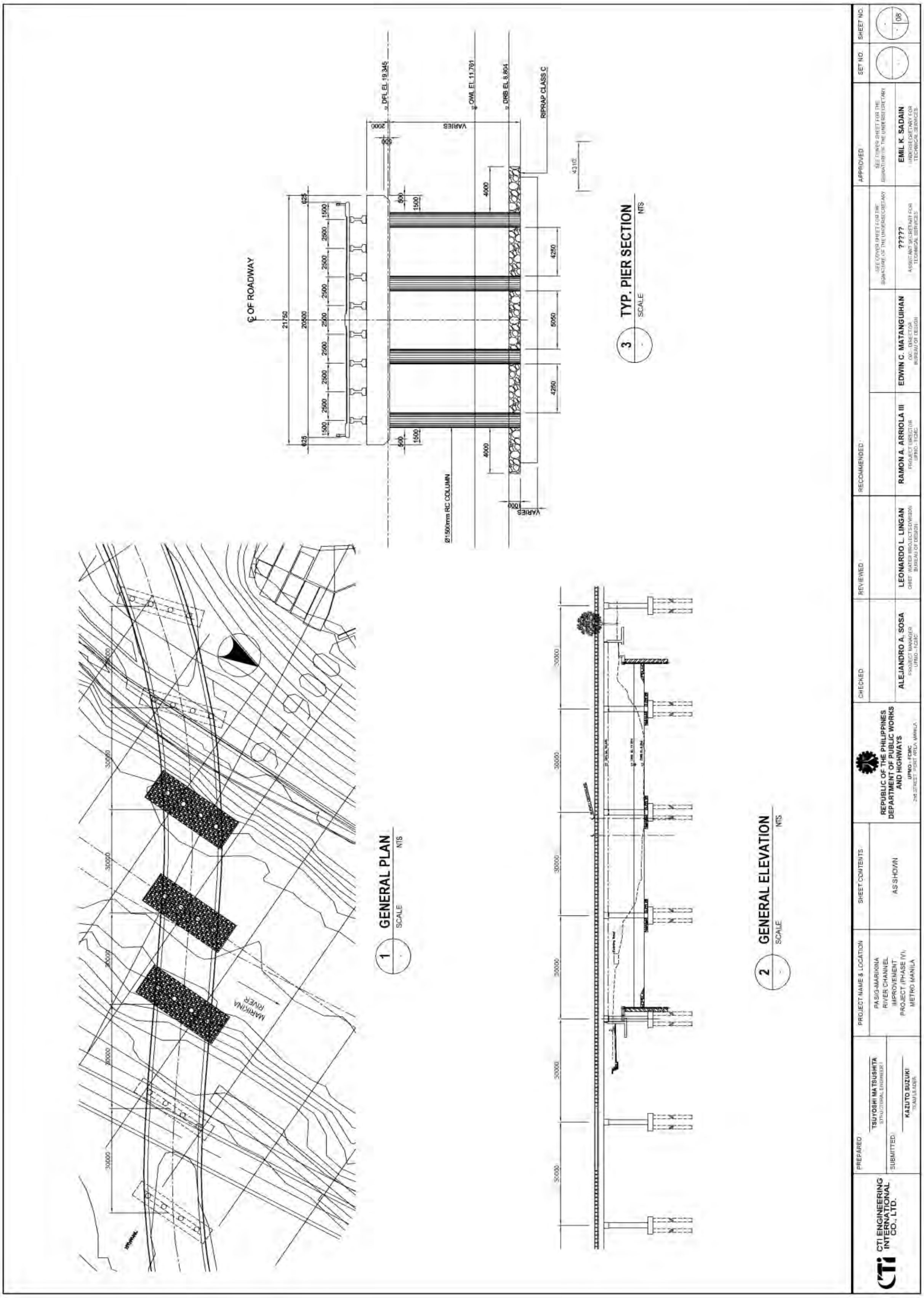
(5) Determination of Foot Protection Method

According to the results in **Table 6.1.32**, Riprap will be applied to all bridges.

6.1.4.4 General Drawings

Figure 6.1.24 to **Figure 6.1.27** show the general drawings of the foot protection for each bridge.

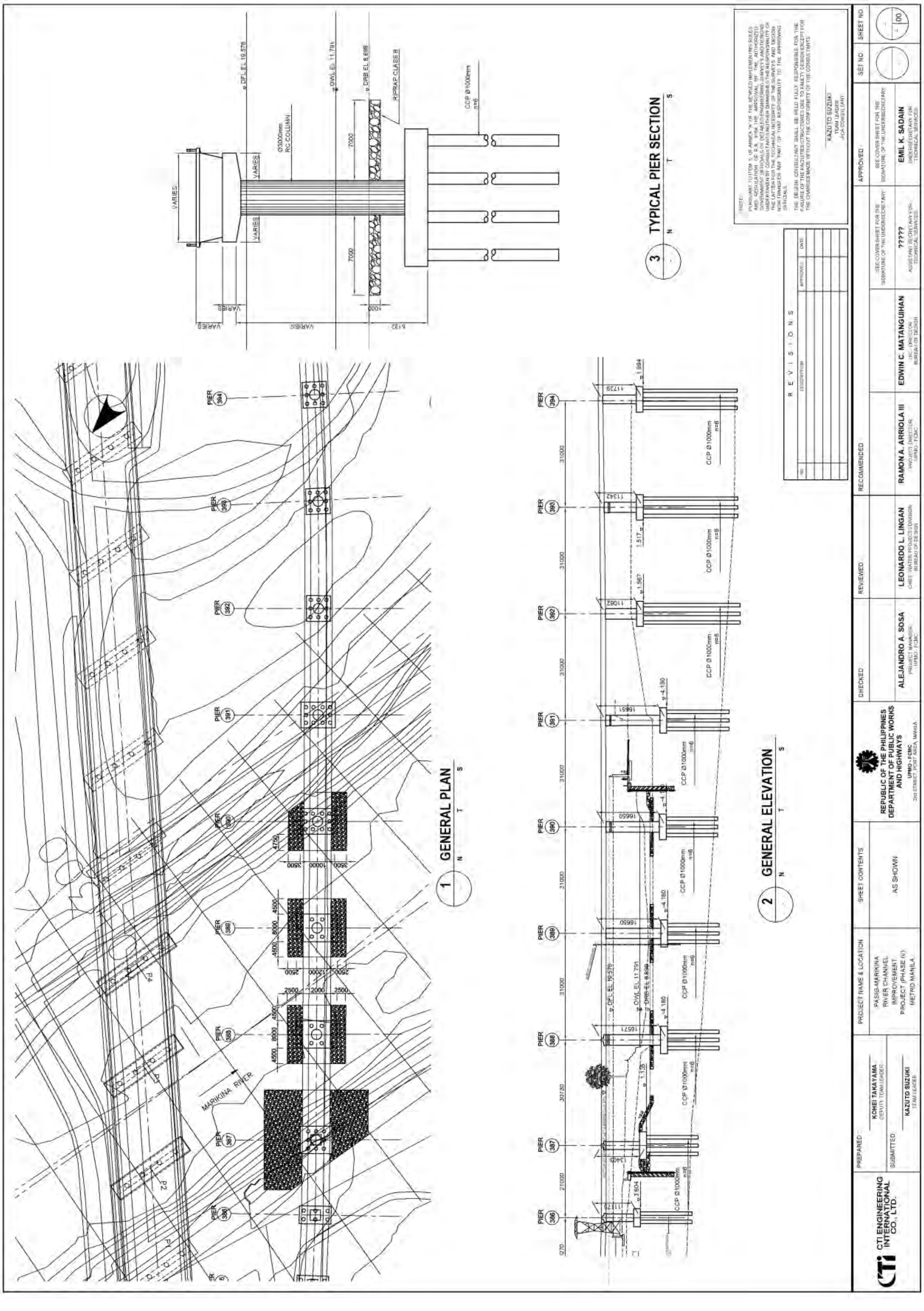
1. Macapagal Bridge



Source: Study Team

Figure 6.1.24 General Drawing of Macapagal Bridge

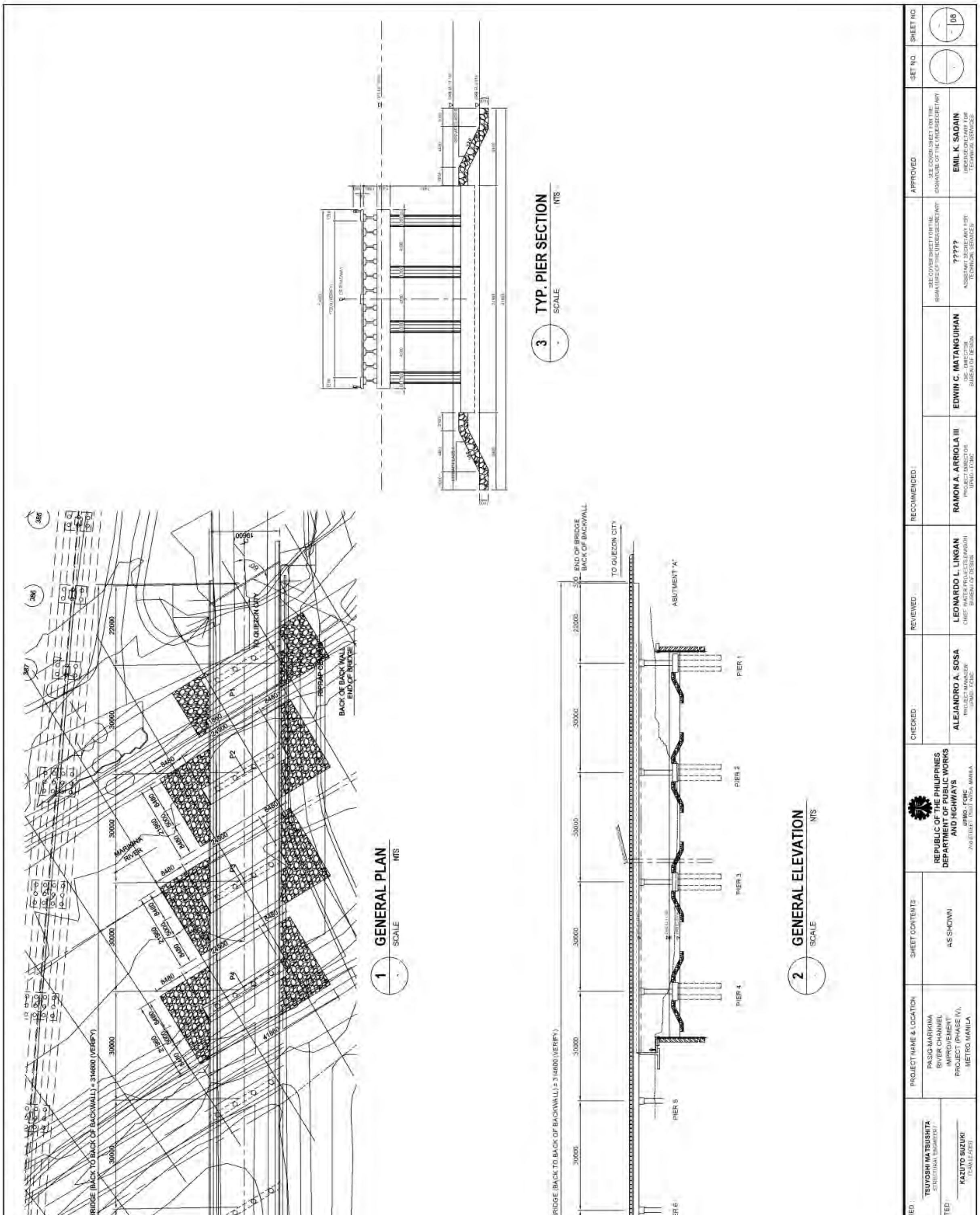
2. LRT-2 Bridge



Source: Study Team

Figure 6.1.25 General Drawing of LRT-2 Bridge

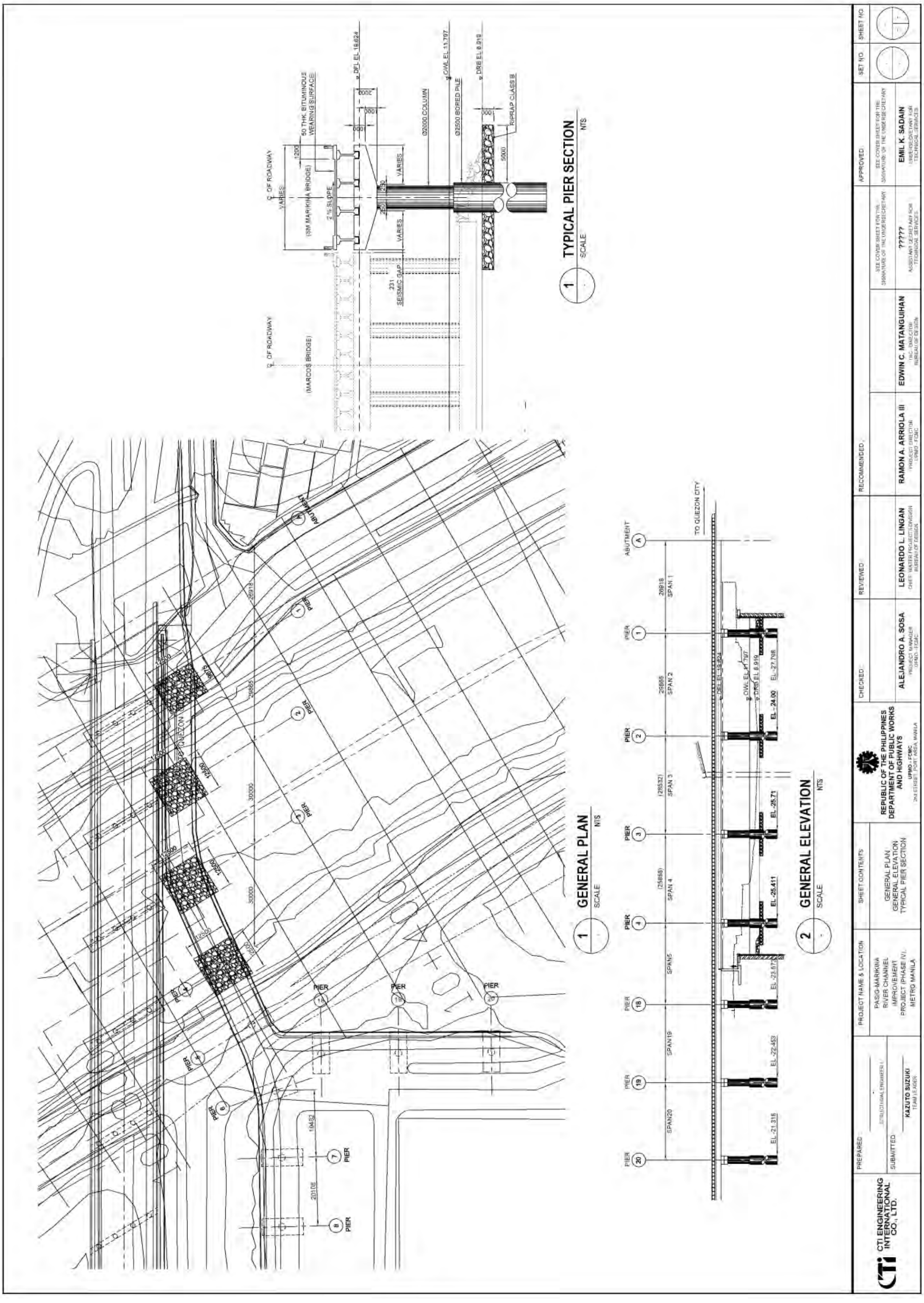
3. Marcos Bridge



Source: Study Team

Figure 6.1.26 General Drawing of Marcos Bridge

4. SM Marikina Bridge



Source: Study Team

Figure 6.1.27 General Drawing of SM Marikina Bridge

6.1.5 Design of Dikes (Dike Protection Works and Non-Soil Levees)

6.1.5.1 Organizing Design Conditions

Basic design conditions are as follows:

- The design shall be based on Philippine standards. If there is no specification under the Philippine standards, the Japanese design standard will be applied.
- The materials used are registered in the "STANDARDIZED PAY ITEMS OF WORK FOR CONSTRUCTION OF INFRASTRUCTURE PROJECTS" in the Philippines.
- The design flow rate of river channel is 500 m3/s in the section from the downstream end (Sta. 5+400) to the Rosario Dam (Sta. 6+600) and 2,900 m3/s in the section from the Rosario Dam (Sta. 6+700) to the Marikina Bridge (Upstream of Sta. 13+350).
- In the river improvement section of this project, the backyard is highly utilized in all sections (factories, commercial facilities, residential areas, etc.). Therefore, the levee structure in the section where embankments are necessary shall be the non-soil levee consisting of SSPs and concrete. The crown width of the levee in this section shall be 3m to ensure the minimum management passage with reference to “Article 31 of the Cabinet Order Concerning Structural Standards for River Administration Facilities, etc. (Exclusion from Application of Provisions on Crown Width).” For embankment structures, safety shall be ensured by confirming consolidation and seepage following the design standards described in Chapter 9.
- Since the design flow velocity during flood is more than 2.0 m/s on the section from the upstream of Rosario Weir (Sta.6+600),(Maximum 4.0m/s at the section from Rosariio weir to Manalo Bridge), a revetment is installed as slope protection. The dimensions such as crown width, slope gradient, and benches are determined to secure resistance against erosion seepage and suction and other risks, as well as to ensure enough stability as a dike.
- In the downstream of Rosario Weir（Sta.6+600）, the flow velocity is less than approximately 2.0m/s, relatively lower than that of the upstream. Therefore, the river protection is installed at only required locations such as the surrounding of river crossing structures, locations where watersheds are formed on the outer shore and where slopes must be established due to site restrictions.
- In the section where the ground height is higher than the DFL (excavated channel section), no embankment will be constructed. However, revetment will be provided to protect shorelines against high flow velocity during flood. In this case, a maintenance road is also provided. The width of the Maintenance Road shall be more than 3m in accordance with “Article 27 (maintenance road for the administration of dike) and Article 15 (maintenance road) of the Cabinet Order concerning Structural Standards for River Administration Facilities, etc.” However, in case it is difficult to expropriate the land because there are factory or commercial facilities, etc., and there is an appropriate passageway which enables access to the river from the area, width of the maintenance road may be reduced to 1.5 m if it is possible to access the river from land.
- In the section upstream from Sta. 10+500, the existing topography will be used as a high water plain. In the section where the ground height is lower than the DFL, the height will be secured by a flood protection wall. Similarly, in the case of a shortage of freeboard, the sufficient height will be achieved by a flood protection wall. However, where the ground height behind the embankment is higher than the DFL, a flood protection wall is not provided to secure the freeboard following “Article 20 (Height) 1 of the Cabinet Order on Structural Standards for River Administration Facilities, etc.”

6.1.5.2 Structure of Dike

(1) Section from the Downstream Design Endpoint (Sta. 5+400) to the Rosario Weir (Sta. 6+600)

The almost watercourse is the excavated channel. Earth channel will be provided in the section where soft rocks are exposed. Similarly, slope protection by revetment will be provided in other sections. As the exisiting ground surface on the right bank from Sta.5+900 to 6+100 is lower relatively, this section is embankment channel.

More specifically, the structure of downstream of MCGS is a plain soil channel, while directly upstream and downstream of MCGS is SSP and concrete revetment, and the section from MCGS to Rosario Weir is the SSP with concrete revetment.

(2) From Rosario Diversion (Sta. 6+700) to Marcos Bridge (Sta. 10+500)

The section will be an excavated channel; only slope protection by revetment will be provided. In the embankment section, the slope protection by embankment and revetment will be constructed. The revetment consists of SSPs and concrete revetment in entire sections.

(3) From Marcos Bridge (Sta. 10+500) to Marikina Bridge (Upstream: Sta. 13+350)

Since the entire section will be an excavated channel, SSPs at low water channel plus flood protection wall (parapet wall) will be installed.

The Definitive Plan proposes only the excavation of the low water channel on the right bank from Sta. 11+000 to Sta. 12+550. However, due to the present construction activities such as the River Wall (**Figure 6.1.5**) which is under construction upstream of SM Marikina and the landfill installed by the developer (an influential Congressman) near Sta. 9+600 to Sta. 9+900 (**Figure 6.1.6**), there is a possibility that the river area will be narrowed in the future. To secure enough width of low water channel in the future, the low water channel shall be protected with SSPs for the width of 80m of the river cross section, and roads/sidewalks will be provided on the high-water channel.

Source: Study Team

Figure 6.1.28 River Wall Constructed near Sta. 10+800

Source: Study Team

Figure 6.1.29 Landfill near Sta. 9+600

6.1.5.3 Revetment Structure

Investigations were carried out to determine the slope protection of the excavated river channel and the revetment structure of river channel with embankment of the section from MCGS (Sta. 6+000) to Marcos Bridge (Sta. 10+500).

(1) Specification

1) Design Flood

The design flood is 100-year flood in the condition with the dam and retarding basins.

The flow volumes of the design flood are as follows.

- Downstream from Rosario weir (Sta. 6 + 600) : 500 m^3/s
- Upstream from Rosario weir (Sta. 6 + 600) : 2,900 m^3/s

2) Freeboard

The top of dike (or the top of the revetment) is calculated by adding the surplus height to the DFL. The necessary freeboard is calculated from the design flow rate at Marikina River as follows. In the section between MCGS (Sta. 6 + 010) and Rosario weir (Sta. 6 + 600), after the division of Manggahan flood control channel, the design flow rate would be 500 m/s. However, during the MCGS operation, the river would become almost flooded and the water level is effected by the water level of the Manggahan weir. Therefore, the freeboard of the downstream from Rosario weir shall be same as that of the upstream.

- Downstrema from MCGS(Sta.6+010) : 1.0 m
- From MCGS(Sta.6+010) to Rosario Weir(Sta.6+600) : 1.2 m
- Upstrema from Rosario Weir(Sta.6+600) : 1.2 m

Table 5-3 Freeboard Allowance for Dikes

Design flood discharge Q (m^3/s)	Freeboard (m)
Less than 200	0.6
200 and less than 500	0.8
500 and less than 2,000	1.0
2,000 and less than 5,000	1.2
5,000 and less than 10,000	1.5
10,000 and over	2.0

Source : DGCS Vol.3, 2015, P5-6

3) Crest Width of Dike

Based on the design flow rate, the required dike crown width is 4 ~ 5 m.

Table 5-4 Recommended Crest Widths for Dikes

Design Flood Discharge, Q (m^3/sec)	Crest Width (m)
Less than 500	3
500 and less than 2,000	4
2,000 and less than 5,000	5
5,000 and less than 10,000	6
10,000 and over	7

Source : DGCS Vol.3, 2015, P5-8

In the river improvement section of this project, as the backyard is highly utilized in all sections (factories, commercial facilities, residential areas, etc.), the dike structure will be SSP and concrete mainly, the non-soil. Therefore, in accordance with "Article 31 of the Order for Structure of River Management Facilities, etc. (Exclusion from Application of Provisions of the crown width)", the crest width of the dike shall be minimum 3 m, which can ensure a management corridor. The width of the dike crown should be more than 3 m if the location has a room at the landside or the location is required wider maintenance road.

4) Width of Maintenance Road

(a) Basic Policy

Width of the maintenance road shall be 3.0m or more, referring to *"Cabinet Order Concerning Structural Standards for River Administration Facilities, etc."* in Japan. However, in the case of the Standing Special dike, if the land on the backland is a factory, commercial facility, etc., where the land acquisition is extremely difficult, and there is an appropriate passage instead of a maintenance road, and the river can be accessed from the land, the width may be narrowed at least 1.5 m.

(b) Width of Maintenance Road between Sta.5+400 to Sta.6+600

The maintenance road in this section connects public roads, the road to MCGS, and EFCOS, and is used for daily river patrols, and for passing construction vehicles such as cranes when repairing facilities. On the left bank side of this section, development is currently underway by developers, who hope for a structure that will access easily to Marikina river after the completion of revetment in this project. Therefore, there is a high possibility that the maintenance road will be opened to river users and used as a walking path.

Therefore, the effective width of the maintenance road in this section is secured to 4.0 m.

5) Slope

The slope of the earth dike is 20% or less. However, it is difficult to install a dike with a gentle slope in all design section because the hinterland is highly developed. Therefore, a slope steeper than 20% is adopted after installing slope protection by concrete or others.

When the height exceeds 5 m, a small step is provided.

5.3.2.4 **Slopes**

The slope of the embankment can be dependent on a number of factors such as the soil type, access arrangements, construction methods and maintenance arrangement. Typically, the soil type is one of the driving factors in determining slope.

The slopes should consider:

- The side slopes should be gentler on both landside and riverside of the embankment than 1V:2H for low embankments (<6.0 m) and 1V:3H for high embankments (>6.0 m).
- A minimum side slope of 1V:4H is typically adopted for embankments consisting of sand and shall be protected by providing a total cover of 300mm thick of a good soil and sodding.
- On the landward side, steeper slopes can be achieved with crib walls or concrete walls where space may be restricted.
- A slope gentler than 1V:4H to 1V:5H should be adopted if maintenance and mowing of the surface is required.
- A steeper slope may be adopted on the riverside where this is protected by a revetment (refer to design of revetments in Section 5-5).

Source: DGCS Vol.3, 2015, P5-9

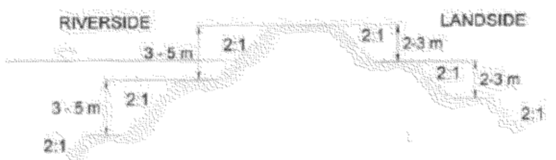
5.3.2.5 **Berms**

Berms are provided for stability, repair and maintenance purposes.

- On the riverbank side, when the crest height from the river bed is more than 6 m, berms shall be provided at every 3 to 5 m. These should have a width of 1 m or more.
- On the landward side, when the crest height from the existing ground is more than 4 m, berms shall be provided at every 2 to 3 m in height with a width of 1 m or more.
- A masonry dike may have a minimum berm width of 1 m when necessary, for stability purposes.

Berms should include swale drains that run parallel to the slope, and aim to reduce the velocity of water running down the slope. These are discussed in more detail in Volume 4: Highway Design.

Figure 5-7 Arrangement of Berm



Source：DGCS Vol.3, 2015, P5-10

6) Extra Embankment

The extra embankment is set considering the settlement after the installation of the dike. The standard height of the extra embankment is shown below.

Table 1.3.8 Standard Value of Extra-embankment Height

Dike Height (m)	Dike Foundation Materials			
	Ordinary Soil		Sand/ Sand & Gravel	
	Extra Embankment Materials			
	Ordinary Soil cm	Sand/ Sand & Gravel cm	Ordinary Soil cm	Sand/ Sand & Gravel cm
≤ 3 m	20	15	15	10
3 m – 5 m	30	25	25	20
5 m – 7 m	40	35	35	30
≥7 m	50	45	45	40

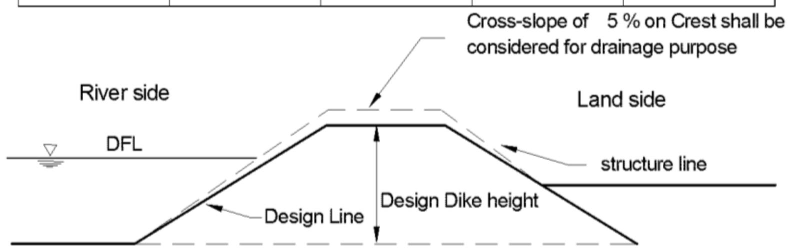


Figure 1.3.8 Extra-embankment Height

Source：“Technical Standards and Guidelines for Design of Flood Control Structures” P8

(a) Section from Sta.5+400 to Rosario Weir（Sta.6+600）

This section is mostly the excavated river channel, but some part is dike section. Therefore the height of dike is decided considering the extra embankment.

(i) From Sta.5+400 to Sta.5+900

The extra embankment of this section is shown in **Table 6.1.33**.

Table 6.1.33 Extra embankment from Sta.5+400 to Downstream of MCGS (Sta.6+010)

	Extra embankment	Reason for setting
Right bank	40 cm	Maximum dike height: 5.5 m (Sta. 5 + 820) Dike material: Ordinary soil, Dike foundation ground: Ordinary soil
Left bank	20 cm	Maximum dike height: 2.5 m (Sta. 5 + 760) Dike material: Ordinary soil, Dike foundation ground: Ordinary soil

Source : Study Team

(ii) From Sta.5+900 to Sta.6+080

The extra embankment of this section is shown in **Table 6.1.34**.

Table 6.1.34 Extra embankment from Sta.5+900 to Sta.6+080

	Extra embankment	Reason for setting
Right bank	40cm	The dike height shall be adjusted to that of left bank due to a management bridge. Maximum dike height: 10.0 m (Sta. 6 + 080) Dike (buried soil) material: Sandy soil, Dike foundation soil: Rock
Left bank	40cm	Maximum Dike height: 5.5 meters (Sta. 6 + 080), Dike foundation Ground: Ordinary soil The elevation matched the existing elevation near EFCOS EL. 19.0 m.

Source : Study Team

(iii) From Sta.6+080 to Sta.6+600

The extra embankment of this section is shown in **Table 6.1.35**.

Table 6.1.35 Extra embankment from Sta.6+080 to Sta.6+600

	Extra embankment	Reason for setting
Right bank	45cm	Maximum Dike height: 8.7 m (Sta. 6 + 160) Dike (buried soil) materials: sandy soil, Dike foundation soil: normal soil
Left bank	40cm	Since the section is mostly cut only, the elevation matches the existing elevation EL. 19.0 near EFCOS.

Source : Study Team

(b) Section from Rosario Weir（Sta.6+600）to Marikina Bridge (Sta.13+450)

In consideration of the consolidated settlement of dike and the ground, it is necessary to increase the extra material as the required surplus height. In order to calculate the height of the extra embankment, consolidation analysis was carried out.

The outline of consolidation analysis condition is as follows.

Calculating of settlement:

Clayer soil	: Δe method
Sandy soil (immediate settlement)	: B.K. Hougu method

Calculating of underground stress : Boussinesq method

Calculating of consolidation coefficient : Average degree of Consolidation

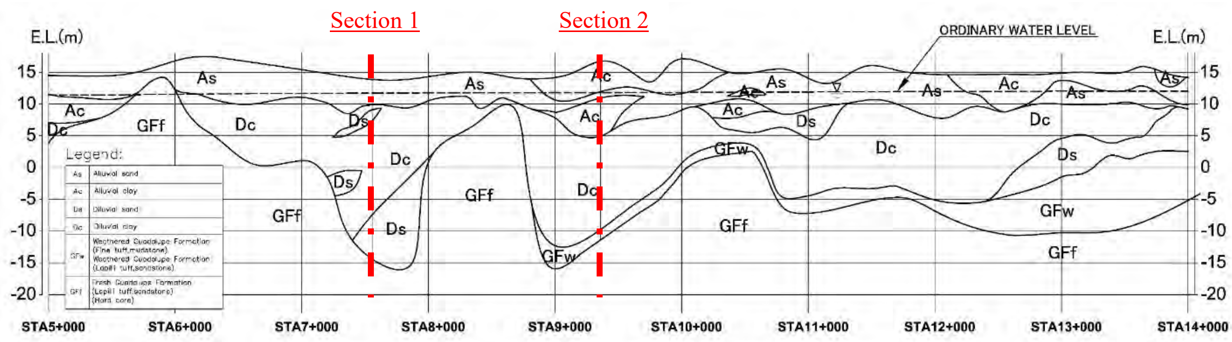
(i) Analysis Condition

A. Cross-section

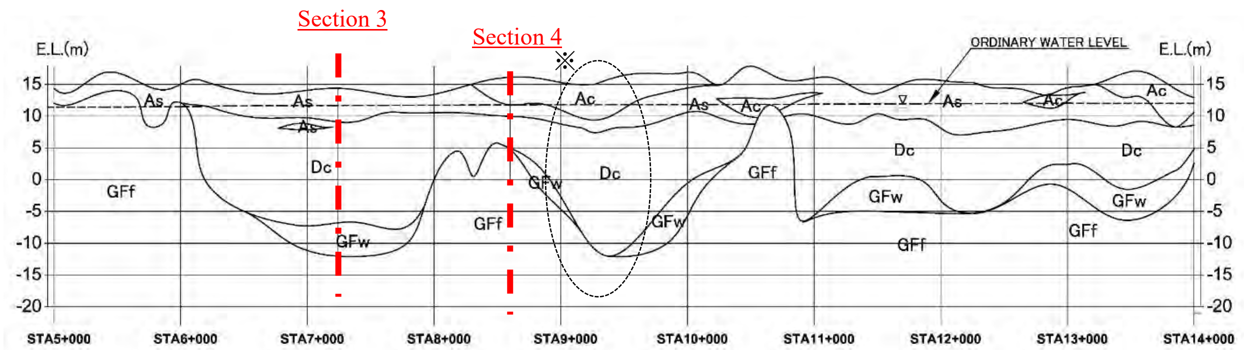
The number of the calculated cross-section was 4 sections, where the amount of consolidation settlement was assumed to be large considering the geological condition and the shape of the hinterland.

1. Left Bank: STA.7+560
   (In left bank, the As layer is deeply deposited from the surface, and the Dc layer is also thickly deposited.)
2. Left Bank STA.9+360
   (In left bank, the Ac layer is deeply deposited from the surface, and the Dc layer is also thickly deposited.)
3. Right Bank STA.7+240
   (In right bank, the As layer is deeply deposited from the surface, and the Dc layer is also thickly deposited.)
4. Right Bank STA.8+600
   (In right bank, the Ac layer is deeply deposited from the surface, and the Dc layer is also thickly deposited.)

This in noted that in the clay layers, the Ac layer and the Dc layer whose N value is less than 20 are modeled, and the Dc layer whose N value is more than 20, the GFw layer, and the GFf layer are not modeled because the ground condition is tight, and the consolidation settlement is ignorable small.



Geological Section of Marikina River Left Bank STA 5+000 to STA 14+000

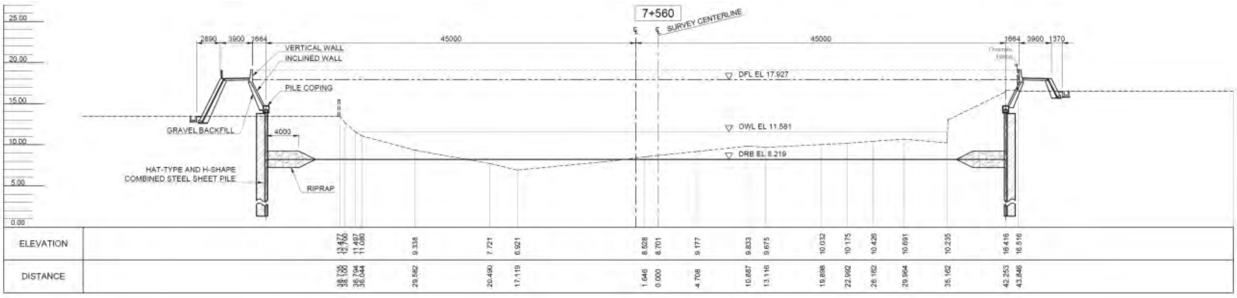


Geological Section of Marikina River Right Bank STA 5+000 to STA 14+000

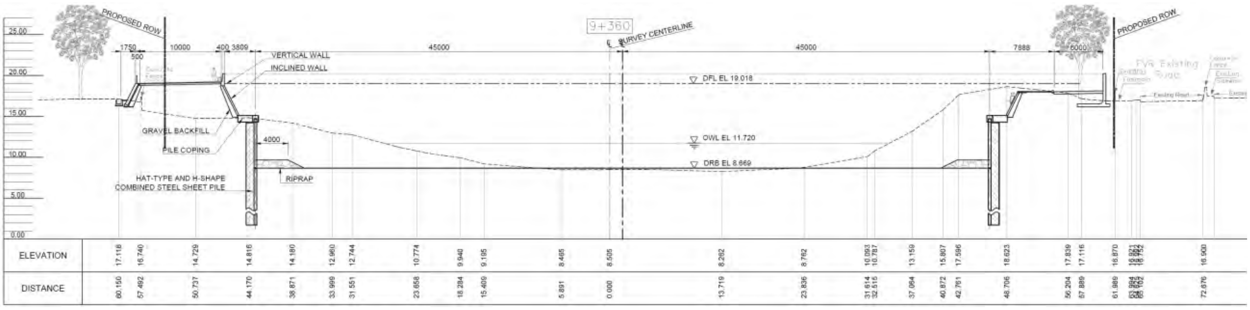
※ Around 9 + 450, the Ac layer and the Dc layer are thickest, but this section isn’t planned to install dike

Source : Study Team

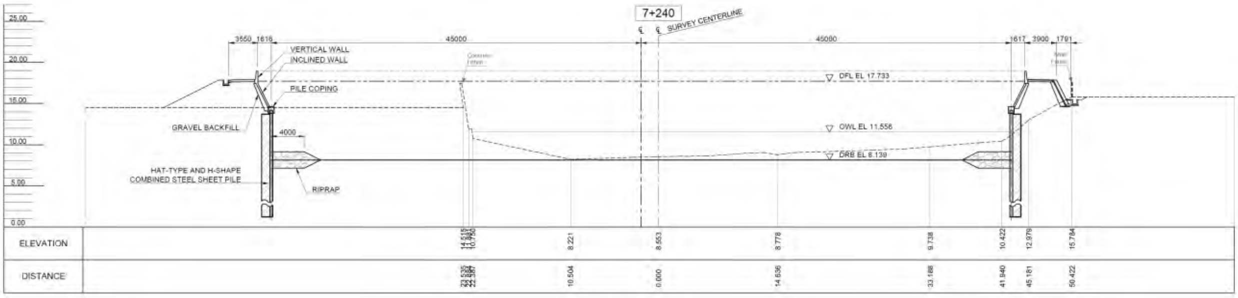
Figure 6.1.30 Location for Cross-section for Consolidation Analysis and Geological Classification



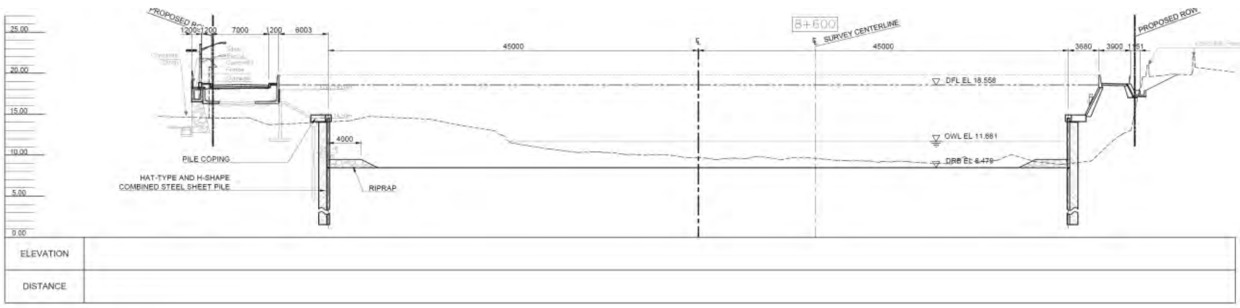
(Sta. 7 + 560, used in calculations for left bank)



(Sta. 9 + 360, used in calculations for left bank)



(Sta. 7 + 240, used in calculations for right bank)



(Sta. 8 + 600, used in calculations for right bank)

Source : Study Team

Figure 6.1.31 Cross-section for Consolidation Analysis

B. Unit Weight

The unit weight is shown in **Table 6.1.36**. As for the GFw and GFe layers, since the converted N value exceeds 50, the consolidation settlement is neglected.

Table 6.1.36 Unit Weight for Consolidation Calculations

Geological Classification		Unit weight of saturated soil (kN/m³)	Unit weight of wet soil (kN/m³)
Name	Symbol		
Field Soil（Embankment）	F	18.5	18.0
Alluvial Sand	As	17.5	17.5
Diluvial Sand	Ds	19.0	19.0
Alluvial Clay	Ac	15.5	15.5
Diluvial Clay	Dc	18.0	18.0

Source : Study Team

C. Consolidation Curve

Clayer Soil

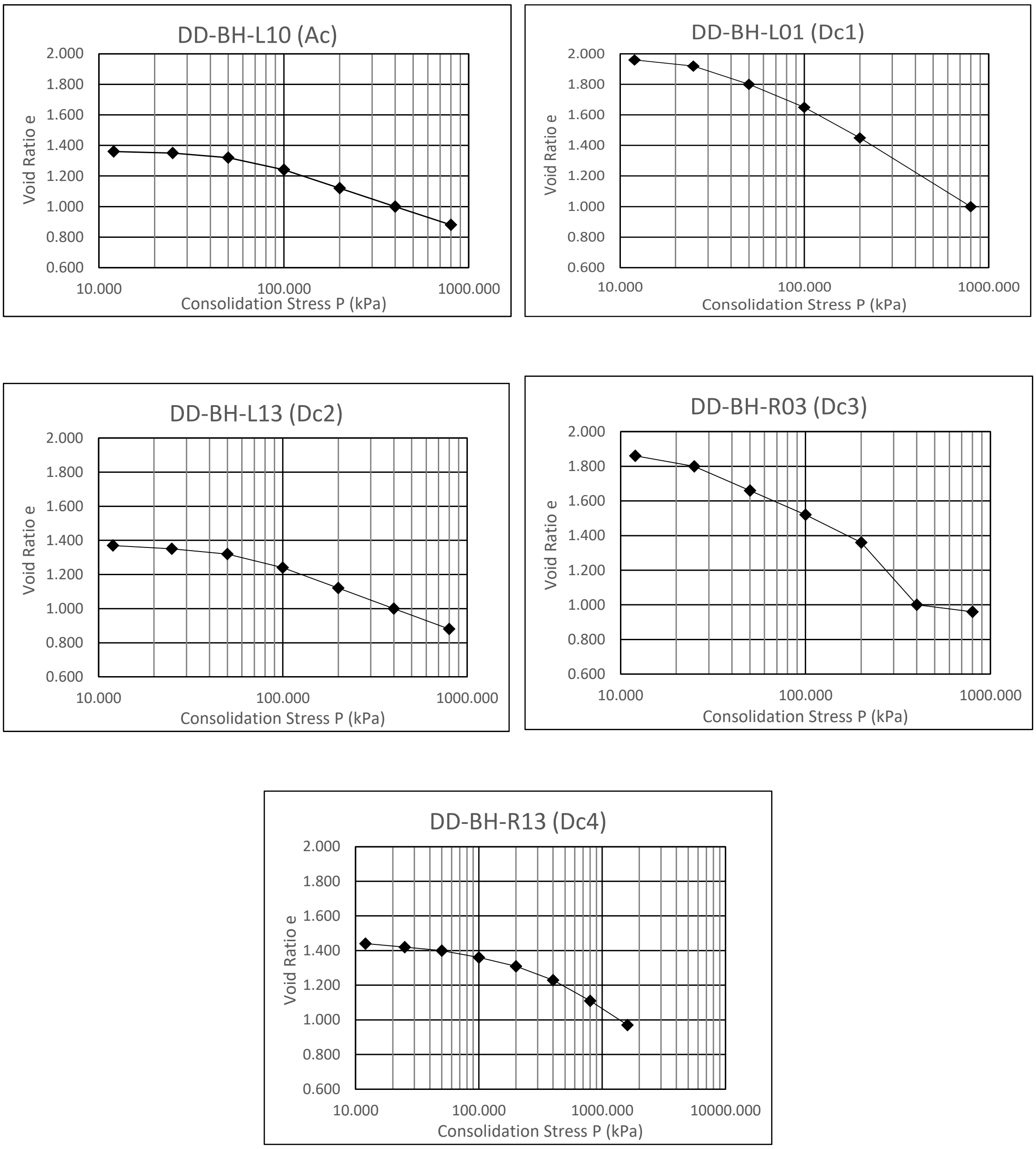
The e-log P and Cv-log P curves used for clayser soil (Ac, Dc) use the results of consolidation tests. In the design section of the Marikina River in this study, five consolidation tests were conducted as shown in **Table 6.1.37**.

Table 6.1.37 Location of Consolidation Test

Location / Station No.		Boring No.	Depth (Elevation)	Geological Classification
Left Bank	STA.7+000	DD-BH-L01	3.00~3.45m (EL.7.61~8.06m)	Diluvial Clay (Dc1)
	STA.9+355	DD-BH-L10	7.00~7.45m (EL.7.48~7.93m)	Alluvial Clay (Ac)
	STA.11+520	DD-BH-L13	8.00~8.45m (EL.7.29~7.74m)	Diluvial Clay (Dc2)
Right Bank	STA.6+980	DD-BH-R03	3.00~3.45m (EL.7.41~7.86m)	Diluvial Clay (Dc3)
	STA.10+910	DD-BH-R13	15.00~15.45m (EL.-3.06~-2.61	Diluvial Clay (Dc4)

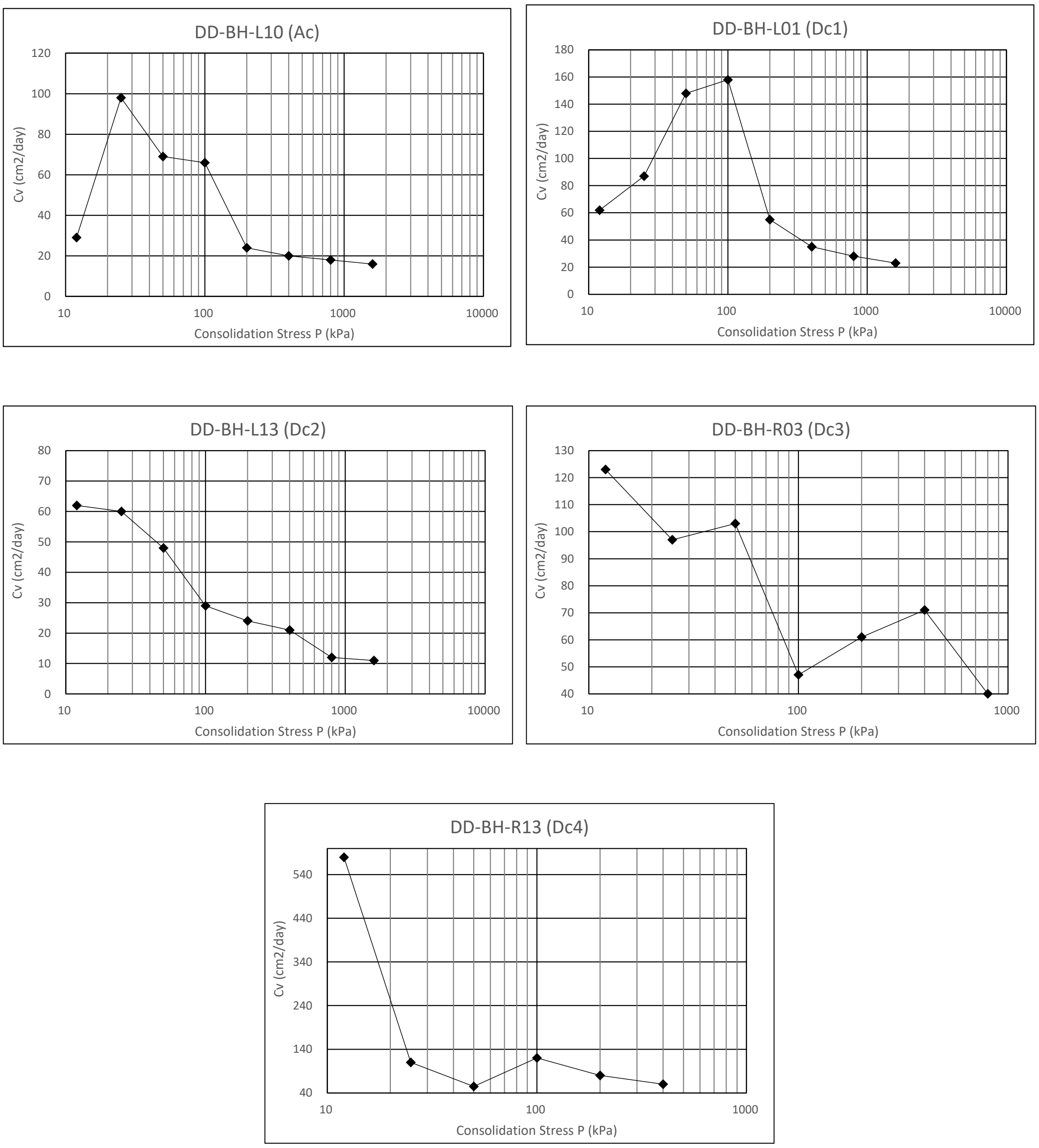
Source : Study Team

The e-log P and Cv-log P curves obtained from the consolidation test are shown below.



Source : Study Team

Figure 6.1.32 e-logP curve for Clayer Soil



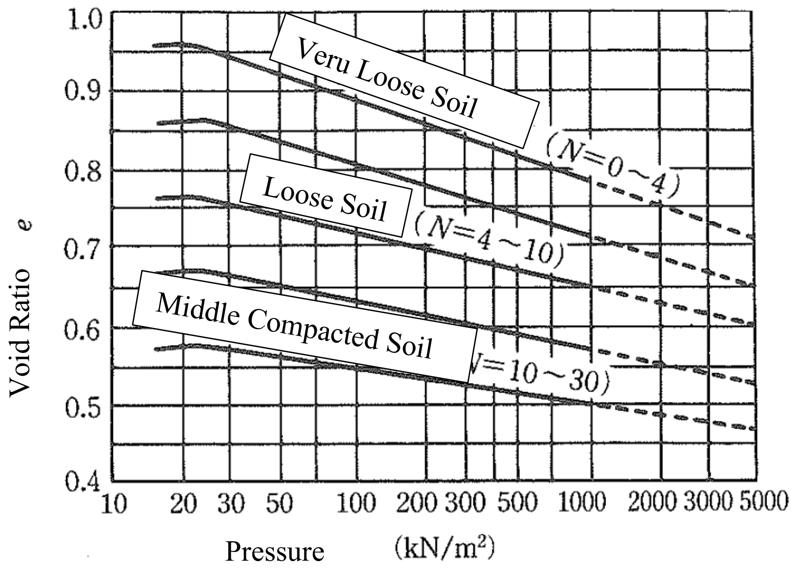
Source : Study Team

Figure 6.1.33 Cv-logP curve

<u>Sandy Soil</u>

For the e-log P-curve of sandy soil (As, Ds), since there are no test results, the e-log P-curve shown in the " Road Earthwork Guideline, Countermeasure for Soft Ground" shall be used.

Since the average N value of As is 9.9, the e-log P curve corresponding to the N value of 4 to 10 "loose sand" is used, and since the average N value of Ds is 21.1, the e-log P curve corresponding to the N value of 10 ~ 30 "Middle compacted sand" is used.



Source: Road Earthwork Guideline, Countermeasure for Soft Ground

Figure 6.1.34 e-logP curve for Sandy Soil

(ii) Calculated Result

The calculation results at the center of the dike are shown below. The amount of immediate settlement means the amount of consolidation settlement at the completion of banking (After 60 days).

The residual settlement obtained by subtracting the immediate settlement from the consolidation settlement is the required height as the extra embankment. The residual settlement reached a maximum of 291.3 mm at Sta. 9 + 360 where the Ac layer was thickly deposited. The average of 4 cross sections was 228.0 mm.

Table 6.1.38 Result of Consolidation Analysis

Location / Station No.		Immediate Settlement (mm)※	Consolidation Settlement (mm)	Residual Settlement (mm)
Left Bank	7+560	185.3	447.7	262.4
	9+360	89.2	380.5	291.3
Right Bank	7+240	98.2	262.3	164.0
	8+600	150.8	439.5	288.7
Average		130.9	382.5	251.6

※ The immediate settlement means the settlement after 60days when the construction finish

Source : Study Team

(iii) Setting of Extra embankment

The extra embankment was set based on the consolidation calculation and standard value shown in **Table 6.1.39**.

As shown in **Table 6.1.40**, the design value as the extra embankment is 300 mm in all sections.

Table 6.1.39 Standard Value of Extra Embankment

Table 1.3.8 Standard Value of Extra-embankment Height

Dike Height (m)	Dike Foundation Materials			
	Ordinary Soil		Sand/ Sand & Gravel	
	Extra Embankment Materials			
	Ordinary Soil cm	Sand/ Sand & Gravel cm	Ordinary Soil cm	Sand/ Sand & Gravel cm
≤ 3 m	20	15	15	10
3 m – 5 m	30	25	25	20
5 m – 7 m	40	35	35	30
≥7 m	50	45	45	40

The embankment height of this project is about 3 ~ 5 m.

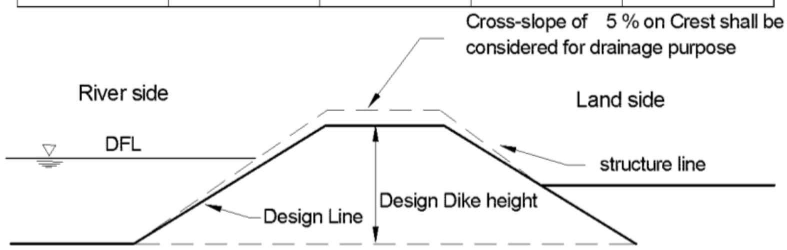


Figure 1.3.8 Extra-embankment Height

Source : "Technical Standards and Guidelines for Design of Flood Control Structures" P8

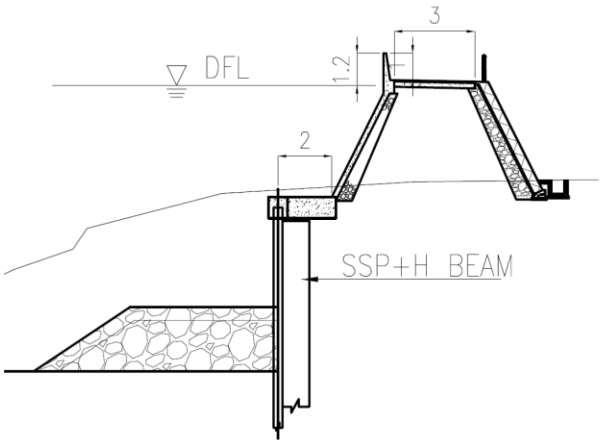
Table 6.1.40 Design Value of Extra Embankment

Location / Station No.		Extra Embankment (mm)	Reason for Setting Numer
Left Bank	Rosario Weir～Sta.8+800	300	The area where the As layer is thickly deposited on the ground surface. The number is set based on the calculation result of Sta. 7 + 560 (Residual settlement 262.4 mm)
	Upstream from Sta.8+800	300	the region where the Ac layer is thickly deposited. The number is set based on the calculation result (Residual settlement 291.3 mm) of Sta. 9 + 360
Right Bank	Rosario Weir～Sta.8+600	300	The area where the As layer is thickly deposited on the ground surface. In the calculation result of Sta. 7 + 240, the residual settlement was the smallest in the cross-section calculated as 164.0 mm. However, in consideration of the geological variation, the standard value of the chapter was adopted.
	Upstream from Sta.8+600	300	the region where the Ac layer is thickly deposited. The number is set based on the calculation result (Residual settlement 288.7 mm) of Sta. 8 + 600

Source : Study Team

(2) Specifications of Dike and Revetment

The standard height and surface slope of the concrete retaining wall installed on the SSP revetment shall be 4m and V:H = 1: 0.5 respectively. The altitude of the crown shall be the design high water level +0.2 m or more. In order to minimize the width, a flood protection wall (Type-IV: Vertical Retaining Wall) with a height of 1.0 m shall be installed on the retaining wall, and extra height (1.2 m) shall be added. The standard revetment section is as shown below. Pursuant to "5.1.4.1 Organizing Design Conditions", the dike crown width of the embankment section shall be 3m; provided, that it is a non-soil levee.



Source: Study Team

Figure 6.1.35 Standard Cross-Section of Revetment Applied to Sta. 6+700 to Sta. 10+500

(3) Basic Structure of Revetment

Coping concrete of SSP revetment shall be made of reinforced concrete and used as the foundation of concrete retaining wall (Seepage cut-off wall).

(4) Structural Details

1) Expansion Joint

Considering the interval of SSPs, an expansion joint made of 10 mm bituminous joint material shall be provided.

2) Drainage Hole

In the concrete revetment of the excavated channel section, a drainage hole (weep hole) with a check valve of about 75 mm in diameter shall be provided for every 2.0 m^2. Behind the embankment (landside), a block retaining wall with one drainage hole for every 2.0 m^2 shall be provided. No drainage hole will be provided on the revetment in front of the embankment (riverside).

6.1.5.4 Design Calculation of Flood Protection Wall

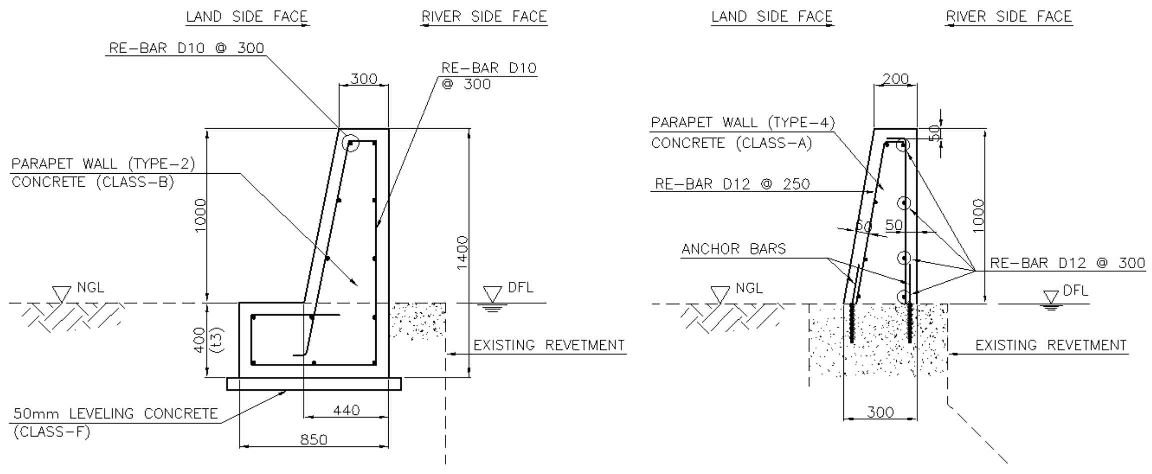
Upstream from the Marcos Bridge (Sta. 10+500 to Sta. 13+350), the existing topography will be used as a high water plain. A flood protection wall for parapet walls will be installed in the high-altitude area, or the existing concrete wall will be raised to secure the necessary height. The type of flood protection wall in this project may be changed depending on the site conditions. The design conditions of the flood protection wall are shown in the following table.

Table 6.1.41 Design Conditions of Flood Protection Wall

Items	Contents	
Case for consideration	Normal, Seismic, Wind load, Flood	
Material unit weight	Same as design conditions for SSP revetment	
Seismic coefficient	k = 0.2	
Wind load	qs= 1.5 kN/m^2	
Flood water level	Design Flood Level + 1m	
Ground friction coefficient	f = 0.6 (installation of leveled concrete is assumed)	
Design calculation condition	Flood protection wall (Type-II, Type-T, and Type-L)	Flood protection wall (Type-IV)
	Overturning: Center of gravity shall act within 1/3 of foundation width Sliding: Sliding force×1.2<sliding resistance force Bearing: Ensure safety factor = 3	Calculate the amount of reinforcing bars required to anchor to an existing concrete wall

Source: Study Team

A cross-sectional view of a typical flood protection wall is shown in the following figure. The optimum form and structure of the flood protection wall shall be determined based on the evaluation of the current status of the revetment.

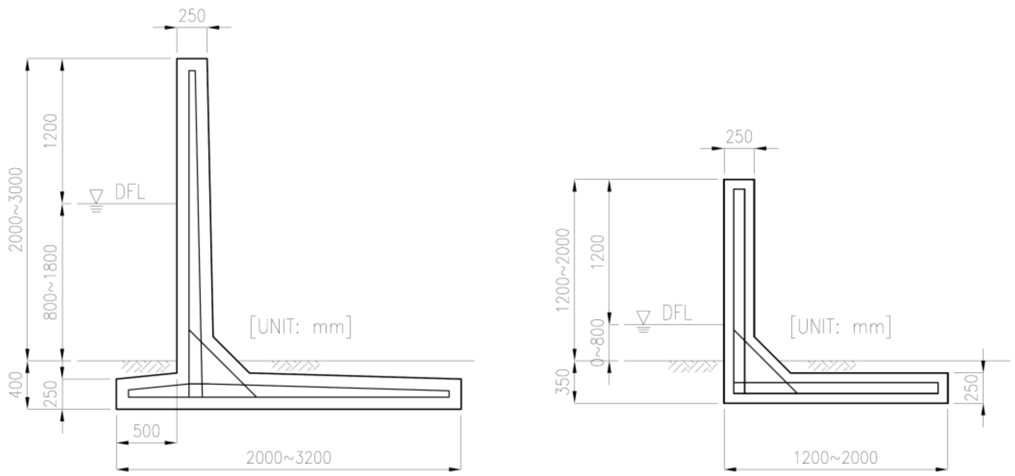


(Type-II Drawing) (Type-IV Drawing)

Flood protection wall (applied when ground height is less than 1.2 m) adopted in phase III

Source: Study Team

Figure 6.1.36 Cross-Sectional View of Flood Protection Wall from Sta. 10+500 to Sta. 13+350



Type-T: Applied to ground height from 2m to 3m Type-L: Applied to ground height from 1.2m to 2m

Source: Study Team

Figure 6.1.37 Cross Sectional View of Flood Protection Wall from Sta. 10+500 to Sta. 13+350

6.1.6 Structure in Other Sections Requiring Particular Consideration

(1) Left Bank from STA.5+400 to 6+340（Along Development Area by the AYALA Land）

In this section, housing land and commercial facilities are being developed by developers (AYALA Land). In discussions with the developers, the developers requested a structure that would allow access from the development area to the river after the river improvement was implemented in this project. Taking this into consideration, the riverbank or revetment structure shown below is adopted.

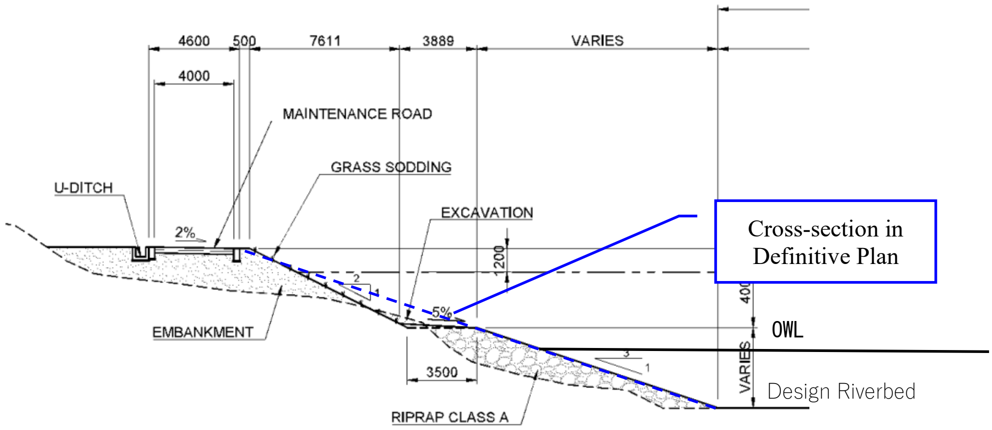
Source : Study Team

Source : Developer

Figure 6.1.38 Developing Area by AYALA Land

1) From STA.5+400 to STA.5+780

The section in the Definitive Plan had a slope of 20% to 30%. In addition, since the design flow rate is less than 2.0 m/s, the installation of a revetment is not required. To allow access to the waterside, a small step is provided at 0.5 m upper elevation from the normal water level. When embankment in the river channel is required to build a small step, embankment with a riprap is installed.

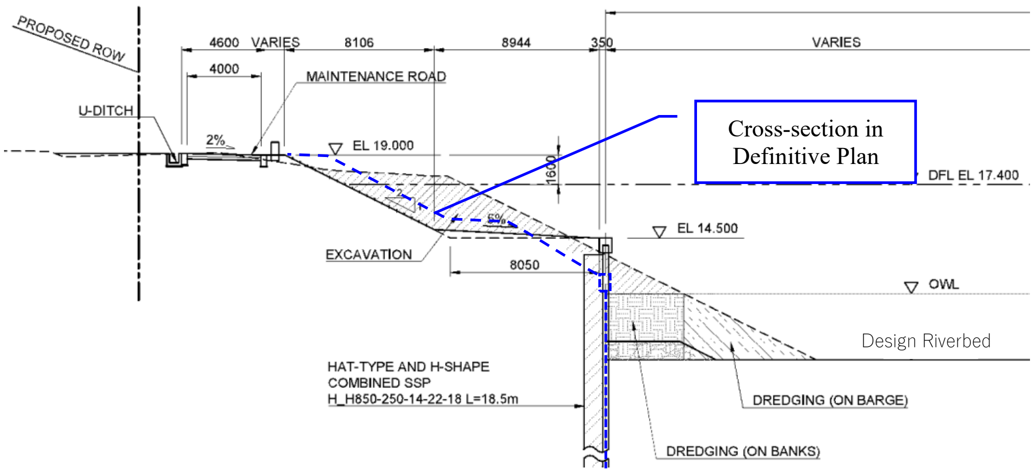


Source: Study Team

Figure 6.1.39 Cross-Section from Sta.5+400 to 5+780

2) From STA.6+035 to STA.6+340

The section in the Definitive Plan had a SSP revetment with 20% slope, and although 20% of the slopes had a small step, there was little space between the SSP revetment and 20% slopes. In this design, the height of the coping concrete in this section is set to EL. 14.5 m to construct this throughout the year. Since this height is about 2.5 m higher than that of the Definitive Plan, a flat area is set up behind the SSP revetment to ensure equivalent river area, and this flat area is used as the space for river uses.



Source: Study Team

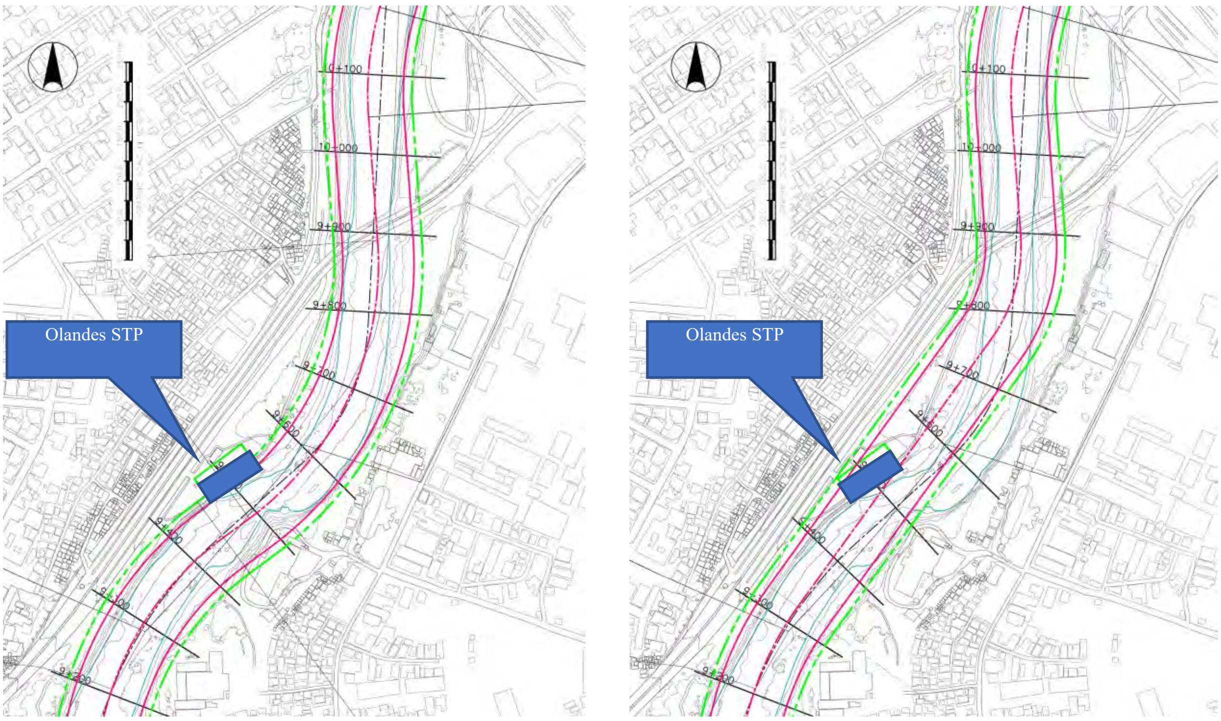
Figure 6.1.40 Cross-Section from Sta.6+035 to 6+340

(2) Olandes Sewage Treatment Plant

The Olandes Sewage Treatment Plant (right bank near Sta.9+500) which was completed 2009 is owned and operated by the Manila Water Company, Inc. Around the Sta. 9+500 where the sewage treatment plant is located, the width of the river surface is only about 60 m.

On the other hand, on the left bank of the opposite shore, land reclamation is in progress at around Sta. 9+550 to Sta. 10+100. Thus, the width of the river channel has become narrower and the width of the water surface is only about 50 m around Sta. 9+700.

There are two possibilities for the layout of the Sta. 9+400 to Sta. 9+800 section: First is to widen the channel by excavating the left bank which includes recently acquired land, and the second is to widen the right bank by removing the Olandes Sewage Treatment Plant. The alignment to be chosen shall be discussed in future.



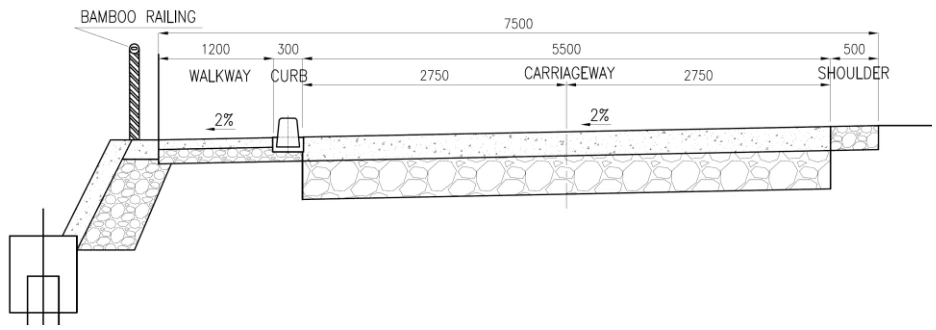
Widen to the left bank Widen to the right bank side

Source: Study Team

Figure 6.1.41 Layout Options between Sta. 9+400 and Sta. 9+800

(3) River-Park Section of Marikina City

A road will be built on the high water plain on the right bank from the Marikina Bridge to the Marcos Bridge. The minimum required width is 2.75 m x 2 lanes. A sidewalk of 1.2 m in width will be provided on the riverside, and concrete curbs will be installed between the pedestrian lane and the roadway. When the sidewalk is adjacent to the river, handrails (Bamboo Railing: Concrete columns imitating bamboo) are to be installed between the sidewalk and the river.

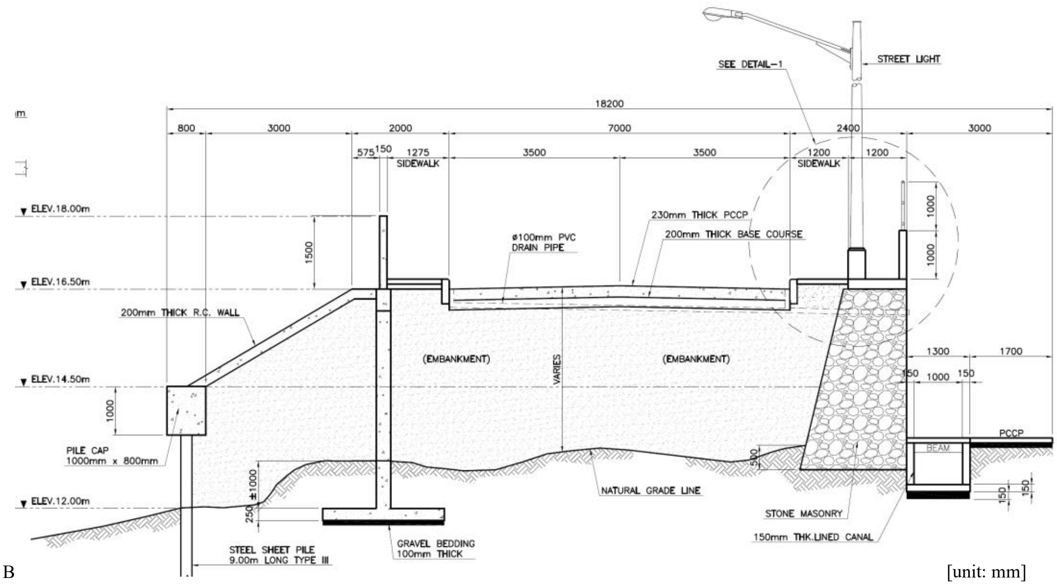


Source: Study Team

Figure 6.1.42 Standard Section of Riverside Road

(4) Pasig City Levee Section

Pasig City is building a dike on the left bank of the Marikina River. The standard section is as shown in the following figure. The crown height of the dike and the concrete wall are EL. 16.5 m and EL. 18.0 m, respectively. On the other hand, the projected DFL in the section is EL. 18m to EL.19m. Compared with the height of the concrete wall, including the freeboard (1.2m), 1~2 m is insufficient. The basic policy to compensate for this height shortage is to raise the dike within the area of the dike currently being constructed by Pasig City.



Note: Drawn by Study Team based on the document collected from Pasig City

Figure 6.1.43 Typical Cross-Section of the Dike Being Built by Pasig City

Table 6.1.42 Project Stage and Current Status of the Pasig City Dike

Station	Development Sequence	Current Status	Points to Remember
Sta. 7+650 ~ Sta. 8+000	Phase 4	Planned/Being Surveyed	Propose unaffected alignments based on survey results in progress
Sta. 8+000 ~ Sta. 8+350	Phase 3	Under Construction	Propose concrete measures based on the survey results under implementation
Sta. 8+350 to Sta. 9+200	Phase 1 & 2	Finished	Propose concrete measures based on the survey results under implementation
Sta. 9+200 to Sta. 9+500	Phase 5	Future Plan	Alignment of the plan needs to be changed because it is planned on the land violating the river area

Note: Summarized by Study Team based on the document collected from Pasig City

6.2 Drainage Plan and Design

6.2.1 Summary of Basic Design for Drainage Facility

The target areas of drainage planning are the both bank of the Marikina River from Sta. 5+400 to Sta. 13+350. A total number of existing outlets are 282 locations within these stretches. As a concept of drainage planning, the number of proposed new outlets will be minimized in order to prevent or minimize backflow from these outlets during high flood stages of the Marikina River. Backflow is prevented by providing flap gate. To reduce the number of outlets, collector pipes parallel to the sheet pile alignment will be provided.

The proposed drainage facilities are summarized as shown in **Table 6.2.1**.

Table 6.2.1 The Draft Proposed Drainage Facility

Proposed Facilities	Quantity	Dimension
New Outlet	In total: 143 locations	
Outlet Type	123 locations	Dia. 910mm – 2000mm
Sluiceway Type	20 locations	1,000 x 1,000 – 2,000 x 2000mm
Collector Pipe	Length=105 m	Dia. 300 - 600mm
Flap Gate	97 unites	Dia. 900mm – 2,000mm (2,000 x 2,000mm)
Creek (width: about 11m) at Right bank, Sta. 8+940		Retain (no drainage works)

Source: Study Team

6.2.2 Drainage Survey and Data Collection

6.2.2.1 Drainage Survey

The drainage survey was conducted for 283 locations of existing outlets for the both bank of the Marikina River from Sta. 5+400 to Sta. 13+350. The survey items are shown as below:

1) Type of outlet (RCP, PVC, Box culvert, etc.)
2) Size (diameter, dimension)
3) Invert Elevation
4) Station number
5) Coordinates

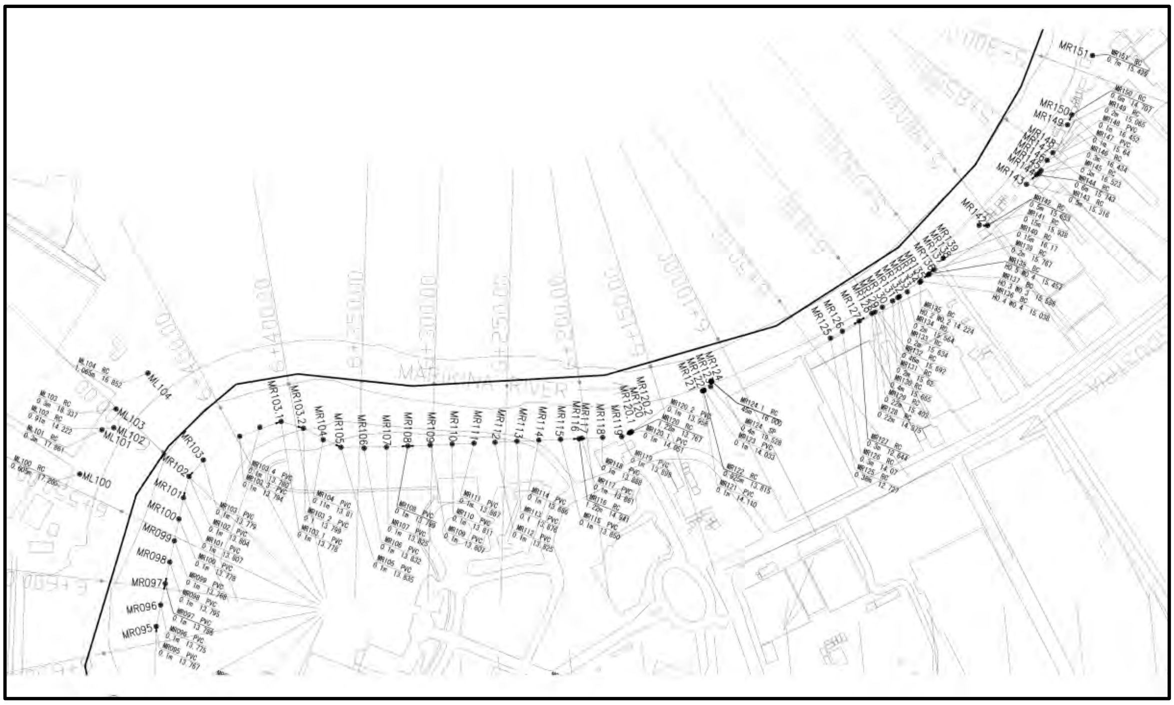
The summary of existing outlets is tabulated as below:

Table 6.2.2 The Summary of Existing Outlets

Location	RC Pipe	Box Culvert	Steel Pipe	Earth Ditch	PVC	Total
Left Bank	57	17	5	1	36	116
Right Bank	96	16	3	0	51	166
Total	153	33	8	1	87	282

Source: Study Team

As a result of drainage survey, the location map of existing outlets and survey data are shown in **Appendix Figure 6.2.1**, **Table 6.2.1** and **Table 6.2.2**, respectively. Sample of each results are as shown below:



Source: Study Team

Figure 6.2.1 The Location Map of Existing Outlets

Table 6.2.3 Site Photo of Existing Outlet

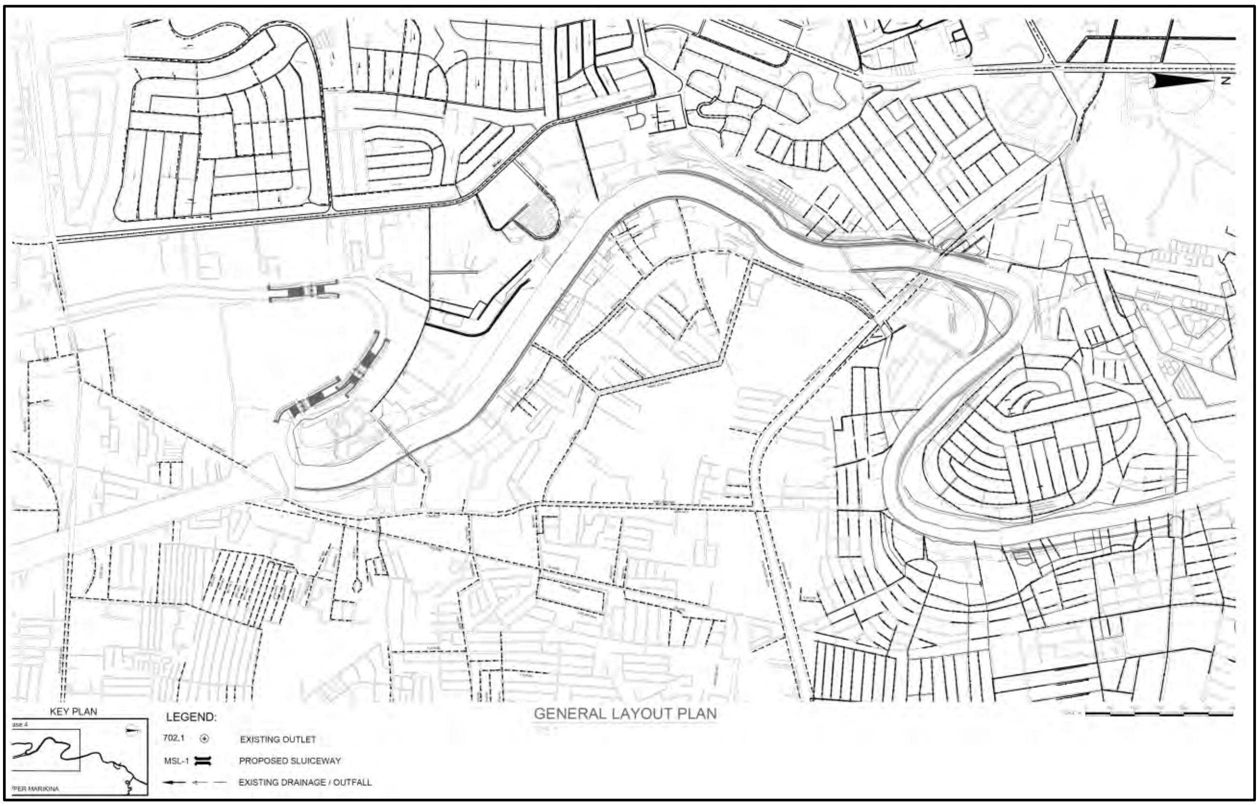
Drainage Outlet Descriptions		
Photo (distant view) 	Photo (short range view)  	Drainage Outlet No.= **ML 017** Northing= 1617891.296 Easting= 510026.016 Elevation= 11.788 Dimension= 920mm Type= RC Date= 814re Station no.= 12+550
Photo (distant view) 	Photo (short range view) 	Drainage Outlet No.= **ML 018** Northing= 1617710.177 Easting= 510010.294 Elevation= 12.063 Dimension= 820mm Type= RC Date= 814re Station no.= 12+375

Source: Study Team

6.2.2.2 Other Data Collection

(1) Existing Drainage Networks

At time of preparation of Definitive Plan, the data of existing drainage networks for Quezon City, Marikina City and Pasig City were collected. The available data was hard copy only and network data did not include the information which were installed by private and some of barangay.

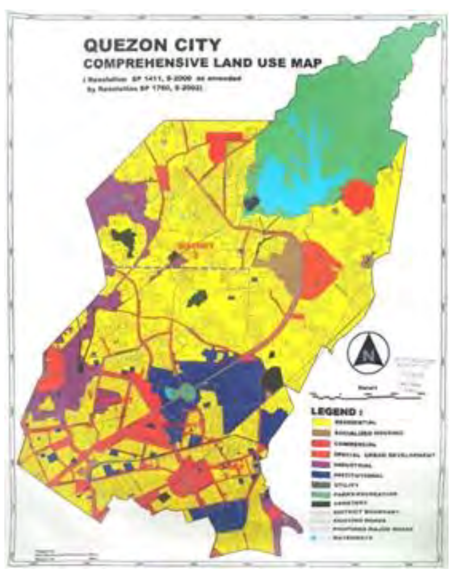


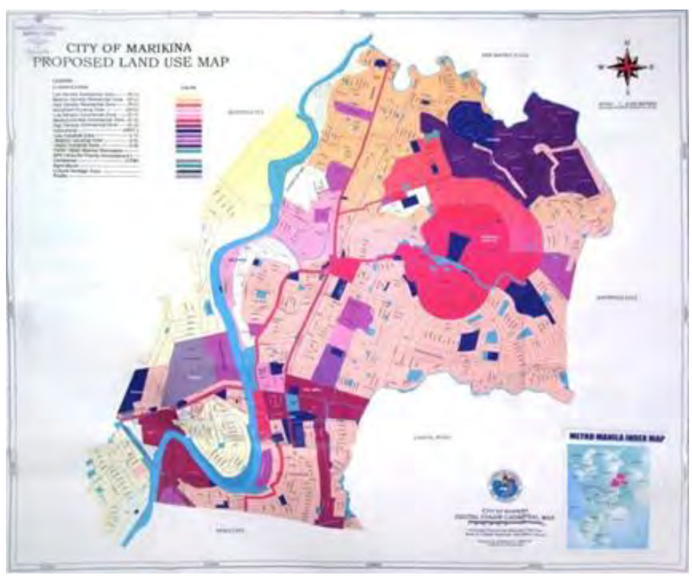
Source: Study Team

Figure 6.2.2 Existing Drainage Networks

(2) Land Use map

Another aspect of the drainage planning is the consideration of land use. The latest version of Land Use Map was collected for above three (3) cities.





Year 2009	Year 2014 (document issued is a Zoning Map)

Source: Quezon City and Marikina City

6.2.3 Drainage Planning

6.2.3.1 Planning Conditions

As a result of the discussion with DPWH-BOD, basically drainage planning in this project is formulated in accordance with “Design Guidelines, Criteria and Standards: Volume 3 – Water Engineering Projects, 2015”, and the planning item which is not including in DGCS will be discussed with DPWH-BOD to set the planning condition.

(1) Design Scale

Design Guidelines, Criteria & Standards, Volume 3: Water Engineering Projects, 2015, DPWH, BOD (referred herein as “Guideline”) sets the design return period for drainage pipes at 25 years. PMRCIP-Phase III comply with this Guideline, therefore, design scale would be adopted at 25 years in this project.

(2) Minimum Size of Drainage Pipes

Minimum size of drainage pipes would be adopted diameter of 900 mm in accordance with Guideline.

(3) Design Discharge

Design Discharge will be calculated in accordance with Guidelines as below method.

1) Rational Formula

Peak discharge is proportional to the runoff coefficient, rainfall intensity and drainage area in accordance with the Rational Formula as shown below

$$Q = \frac{CIA}{360}$$

Where:

Q = Design discharge (m3/sec)

C = Runoff coefficient

I = Rainfall intensity (mm/hr)

A = Catchment area (ha)

2) Runoff Coefficient

The runoff coefficient, C, for various land use types, are shown below table. These were adopted in accordance with the Guidelines.

Table 6.2.4 Runoff Coefficient, C, for Land Use Type

Land Use	Minimum	Maximum
Residential Area - Densely built	0.50	0.75
Residential Area - Not densely built	0.30	0.55
City Business District	0.70	0.95
Light Industrial Areas	0.50	0.80
Heavy Industrial Areas	0.60	0.90
Parks, Playgrounds, Cemeteries, unpaved open spaces and vacant lots	0.20	0.30
Concrete or Asphalt Pavement	0.90	1.00
Gravel Surfaced Road and Shoulder	0.30	0.60
Rocky Surface	0.70	0.90
Bare Clay Surface (faces of slips, etc.)	0.70	0.90
Forested Land (sandy to clay)	0.30	0.50
Flooded or Wet Paddies	0.70	0.80

Source: Guideline

Where multiple land use is found within a drainage area, the runoff coefficient for each sub-area was determined by taking the weighted average C value in proportion to the drainage area size as shown below:

$$C = \frac{A_1C_1 + A_2C_2 + ...A_NC_N}{A_T}$$

Where:

C1~CN = values of C for each sub-catchment area

A1~AN = sub-catchment area

AT = total catchment area

3) Rainfall Intensity

The 25-year rainfall intensity probability have been set in Definitive Plan. Coefficients of the equation for different return periods are shown below table.

$$I = \frac{a}{Tc^n + b}$$

Where:

I = rainfall intensity for given return period (mm/hr)

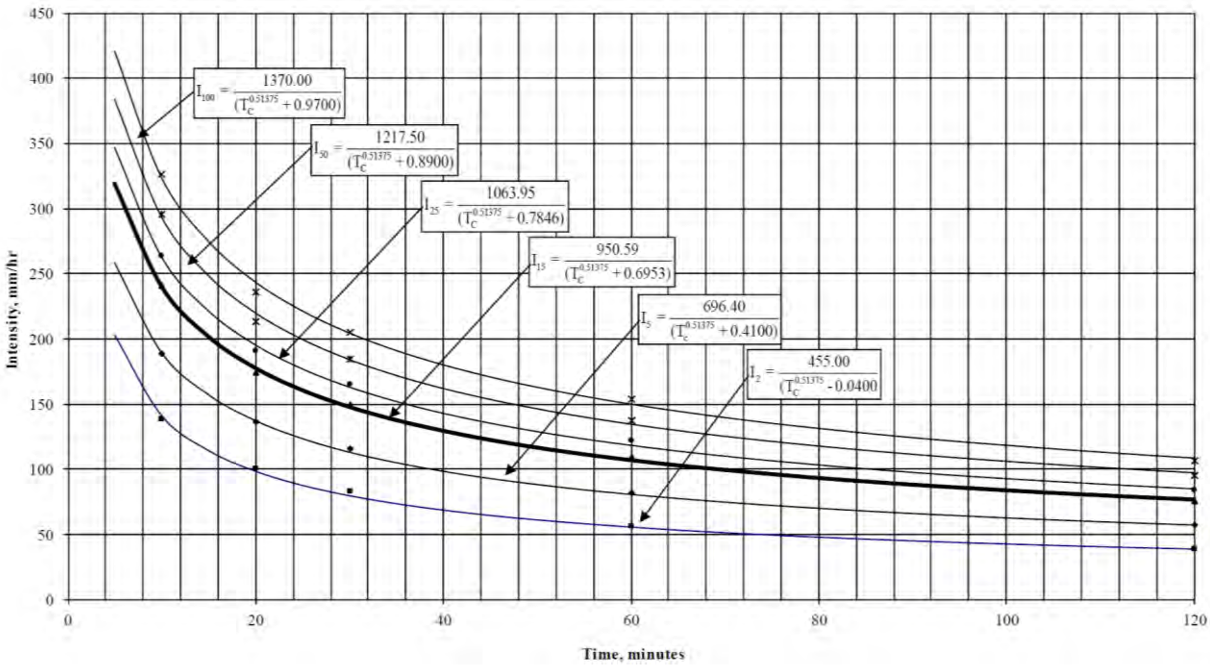
Tc = time of concentration (min)

n, a, b = regression constants

Table 6.2.5 Coefficients for Rainfall Intensity Formula

Return Period	Port Area		
	a	b	n
2	455.00	-0.0400	0.51375
5	696.40	0.4100	0.51375
10	858.40	0.6100	0.51375
15	950.59	0.6953	0.51375
20	1015.44	0.7519	0.51375
25	1063.95	0.7846	0.51375
50	1217.50	0.8900	0.51375
100	1370.00	0.9700	0.51375

Source: Definitive Plan



Source: Definitive Plan

Figure 6.2.3 Rainfall Intensity-Duration-Frequency Curves

In Guideline, it is mentioned that for urban catchments, the minimum time of concentration should be no less than 5 minutes and time of concentration (Tc) will be calculated as below:

Tc = Overland flow (to) + Curb and Gutter flow (tg) + Drain flow (td)

Estimation of these components is provided in below table

Table 6.2.6 Equations for Estimating the Time of Concentration in Urban

Travel Path	Travel Time	Remark
Overflow	$t_o = \frac{107n^*L^{1/3}}{S^{1/5}}$	L = Overland sheet flow path length (m) For steep slopes (> 10%), L ≤ 50 m For moderate slopes (< 5%), L ≤ 100 m For mild slopes (< 1%), L ≤ 200 m n*= Horton's roughness value for the surface S= slope of overland flow surface (%)
Curb and Gutter Flow	$t_g = \frac{L}{40\sqrt{S}}$	L= length of curb gutter flow (m) S= longitudinal slope of gutter (%)
Drain Flow	$t_d = \frac{nL}{60R^{2/3}S^{1/2}}$	n= Manning's roughness coefficient R= Hydraulic Radius (m) S= Friction Slope (m/m) L= Length of Reach (m)

Source: Guideline

Table 6.2.7 Values of Horton's Roughness n*

Land Surface	Horton's Roughness (n*)
Paved	0.0150
Bare Soil	0.0275
Poorly Grassed	0.0350
Average Grassed	0.0450
Densely Grassed	0.0600

Source: Guideline

4) Catchment Area

Each of the sub-drainage areas draining into river is determined from a topographic map. Additional references are considered such as flow directions of existing drains, existing roads as well as other man-made structures that are possible indications of drainage boundaries.

Sub-catchment for each outlet will be determined after completion of detailed survey works.

6.2.3.2 Planning for Drainage Facility

(1) Basic Concept for Planning

The basic concept of the drainage planning is as described below. Basically, it would be adopted same concept as PMRCIP Phase III.

1) Target Area and Facility

The target areas of drainage planning are the both bank of the Marikina River from Sta. 5+400 to Sta. 13+350. All existing outlets within these stretches will be target of drainage planning.

2) Minimize the number of outlets

There are numerous outlets that discharge into the Marikina rivers. As a concept of drainage planning, the number of proposed new outlets will be minimized in order to prevent or minimize backflow from these outlets during high flood stages of the Marikina River, and to reduce the cost for operation and maintenance works. Backflow is prevented by providing flap gate.

3) Design Scale

For proposed new drainage facility, design scale would be adopted at 25 years in accordance with Guidelines.

4) Size of Proposed Outlet

For storm runoff from the catchment, outlet size varies depending on the design discharge. Minimum size of drainage pipes would be adopted diameter of 900 mm in accordance with Guideline.

However, as a result of discussion with DPWH-BOD, in the case of roof and sanitary flows, for new outlet, same as existing size diameter is adopted as the discharge is too small. And also for collector pipe, it would be adopting the suitable size for design discharge.

5) Demolish and clog for non-operational outlets

As far as practicable, all existing usable outlets must be improved or rehabilitated. These outlets are no longer functional but still necessary and must be replaced. In case of replacement or new installation, the design capacity must be recalculated and verified, and must conform to the appropriate design scale. Outlets that are no longer functioning and found to be extraneous should be reconfirmed on site for possible demolition or closure.

6) Retain of existing outlet

At location of existing drainage outlets where no river protection work structure is required and also, where an existing outlet is not affected by the new proposed river protection work structure, there is no recommended drainage improvement plan and the said outlet shall be retained.

7) Prevent back flow from river

In a majority of cases, the existing ground is generally lower than the design flood water level. For such cases, fundamentally, the flap gates must be installed to prevent back flow from the river.

8) Type of Proposed Outlet

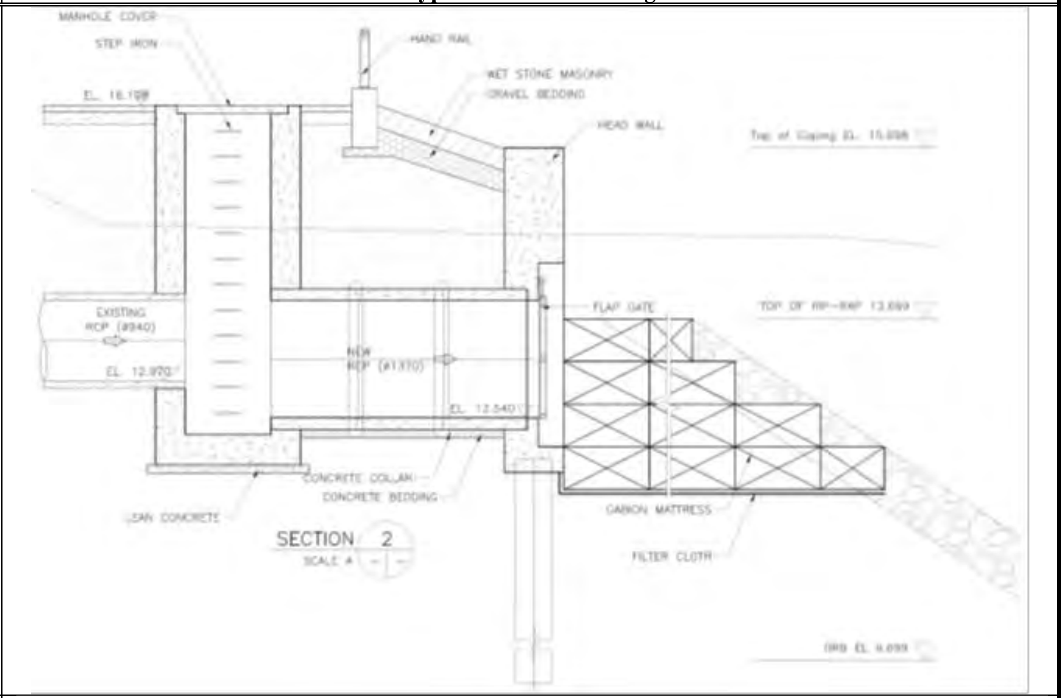
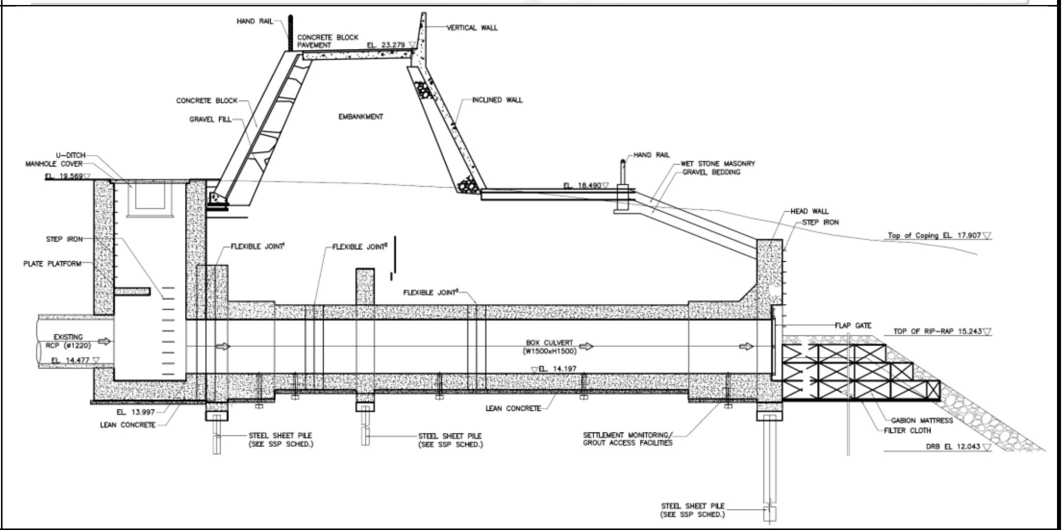
The structure of proposed outlet basically depends on type of river structure and type of existing outlet. The structure of proposed outlet classifies two (2) type of structure as tabulated below.

Table 6.2.8 Type of Proposed Structure and Applicable Case

Type of River Structure / Type	SSP Revetment	Dike
Outlet	○	○*1
Sluiceway		○

*Note: ***1** : in case that Seepage Cut-off Wall will not be required*
Source: Study Team

Typical section for each type is shown in following figure.

Type	Typical Section Drawing
Outlet	
Sluiceway	

Source: Study Team

Figure 6.2.4 Typical Section Drawings for Proposed Structure

(2) Drainage Planning

Based on above discussion, the hydraulic calculation conducted to calculate the design discharge for each outlet. The calculation and result are tabulated in **Appendix Table 6.2.3** and, the proposed drainage works, and facilities are provided for all of existing drainage outlets as shown in following tables.

Table 6.2.9 Proposed Drainage Works and Facilities (1/5)

NO.	CODE NO.	EXISTING OUTLET			PROPOSED FACILITIES				Justification of Flap Gate			
		Station No.	Type	Dia. or Size (Height x Width) (mm)	Proposed Works	Dia. or Size (Height x Width) (mm)	Invert Elevation	Flap Gate	Design Flood Level (DFL)		Ground Elevation at Landside	Remarks
1	ML104.5				Additional Outlet for Ayala Property	900	13.47	non	14.64	<	18.09	Ground elevation at landside is higher than DFL
2	ML104.4				Additional Outlet for Ayala Property	900	12.94	non	14.66	<	17.60	Ground elevation at landside is higher than DFL
3	ML104.3				Additional Outlet for Ayala Property	900	13.17	non	14.68	<	16.95	Ground elevation at landside is higher than DFL
4	ML104.2				Additional Outlet for Ayala Property	900	12.40	non	17.40	<	18.78	Ground elevation at landside is higher than DFL
5	ML104.1				Additional Outlet for Ayala Property	900	12.40	non	17.40	<	19.67	Ground elevation at landside is higher than DFL
6	ML104	6+474	RC	D1065mm	Retained							
7	ML103	6+500	RC	D300mm	Provide New Outlet	900	15.59	non	17.40	<	19.78	Ground elevation at landside is higher than DFL
8	ML102	6+509	RC	D910mm	Provide New Outlet	900	13.41	non	17.40	<	19.78	Ground elevation at landside is higher than DFL
9	ML101	6+513	RC	D300mm	Provide New Outlet	900	15.59	non	17.40	<	19.78	Ground elevation at landside is higher than DFL
10	ML100	6+541	RC	D620mm	Provide New Outlet	900	15.59	non	17.40	<	19.35	Ground elevation at landside is higher than DFL
11	ML099.2	6+710	PVC	D100mm	Not affected by Construction							
12	ML099.1	6+711	PVC	D100mm	Not affected by Construction							
13	ML099	6+758	RC	D1840mm	Additional Outlet	1800	9.78	●	17.44	>	15.62	DFL is higher than ground elevation at landside
14	ML098.1	7+061	PVC	D80mm	Demolished due to River Structure							
15	ML098	7+178	RC	D1520mm	Provide New Outlet	1500	10.35	●	17.70	>	17.06	DFL is higher than ground elevation at landside
16	ML097A	7+216	RC	D770mm	Provide New Sluice	1000X1000	10.30	●	17.72	>	14.35	DFL is higher than ground elevation at landside
17	ML097.4	7+406	RC	D1200mm	Provide New Outlet	1200	10.68	non	17.83	<	17.94	Ground elevation at landside is higher than DFL
18	ML096	7+652	PVC	D100mm	Connect to ML097.3 by PVC Pipe							
19	ML097.3	7+658	RC	D1180mm	Provide New Outlet	1200	10.78	non	17.99	<	18.39	Ground elevation at landside is higher than DFL
20	ML087A				Additional New Sluice	1000x1000	10.40	●	18.00	>	13.92	DFL is higher than ground elevation at landside
21	ML087B				Additional New Sluice	1000x1000	10.40	●	18.06	>	14.50	DFL is higher than ground elevation at landside
22	ML092	7+798	PVC	D100mm	Connect to ML091.1 by PVC Pipe							
23	ML091.1	7+800	RC	D1100m	Provide New Sluice	1200X1200	10.20	●	18.07	>	14.66	DFL is higher than ground elevation at landside
24	ML091A	7+805	PVC	D100mm	Connect to ML091.1 by PVC Pipe							
25	ML087.2	7+827	BC	H400mm W400mm	Provide New Sluice	1000X1000	10.40	●	18.09	>	13.85	DFL is higher than ground elevation at landside
26	ML088	7+827	RC	D250mm	Connect to ML087.2 by PVC Pipe							
27	ML087.1	7+857	ED	H300mm W300mm	Provide New Sluice	1000X1000	11.60	●	18.11	>	14.43	DFL is higher than ground elevation at landside
28	ML087.101	7+878	BC	H370mm W310mm	Connect to ML087.1 by RC Collector Pipe							
29	ML087C				Additional New Sluice	1000x1000	11.40	●	18.12	>	14.92	DFL is higher than ground elevation at landside
30	ML087D				Additional New Sluice	1000x1000	12.00	●	18.18	>	15.49	DFL is higher than ground elevation at landside
31	ML073.3	8+137	BC	H1810mm W1430m	Provide New Outlet	1800	11.58	●	18.28	>	14.83	DFL is higher than ground elevation at landside
32	ML073.2	8+245	RC	D900mm	Provide New Outlet	900	12.80	●	18.34	>	16.00	DFL is higher than ground elevation at landside
33	ML073.1	8+341	RC	D900mm	Provide New Outlet	900	12.80	●	18.40	>	14.80	DFL is higher than ground elevation at landside
34	ML073A	8+446	RC	D900mm	Provide New Outlet	900	12.44	●	18.46	>	14.69	DFL is higher than ground elevation at landside
35	ML072A	8+611	RC	D900mm	Provide New Outlet	900	12.80	●	18.57	>	14.65	DFL is higher than ground elevation at landside
36	ML071A	8+791	RC	D600mm	Provide New Outlet	900	12.40	●	18.68	>	14.40	DFL is higher than ground elevation at landside
37	ML070A	8+848	RC	D600mm	Provide New Outlet	600	12.09	non	18.70	<	18.80	Ground elevation at landside is higher than DFL
38	ML069	9+002	BC	H1440mm W1310m	Provide New Outlet	1440X1310	13.13	non	18.80	<	18.98	Ground elevation at landside is higher than DFL
39	ML068.1	9+177	RC	D900mm	Provide New Outlet	900	11.66	non	18.91	<	19.00	Ground elevation at landside is higher than DFL
40	ML068A				Additional New Sluice	1000x1000	10.30	●	18.93	>	17.22	DFL is higher than ground elevation at landside
41	ML068B				Additional New Sluice	1000x1000	10.30	●	18.66	>	16.15	DFL is higher than ground elevation at landside
42	ML068C				Additional New Sluice	1000x1000	10.30	●	19.05	>	16.76	DFL is higher than ground elevation at landside
43	ML068D				Additional New Sluice	1000X1000	10.40	●	19.12	>	17.00	DFL is higher than ground elevation at landside
44	ML068	9+521	BC	H460mm W1000mm	Provide New Sluice	1000X1000	10.40	●	19.12	>	17.00	DFL is higher than ground elevation at landside
45	ML066.4	9+785	RC	D1000mm	Provide New Outlet	1050	12.94	●	19.27	>	18.39	DFL is higher than ground elevation at landside
46	ML066A	9+896	BC	H3150mm W3080m	Provide New Outlet	3150X3080	11.72	non	19.35	<	20.00	Ground elevation at landside is higher than DFL
47	ML066.101	9+899	BC	H760mm W680mm	Provide New Outlet	900	12.40	●	19.35	>	18.03	DFL is higher than ground elevation at landside
48	ML065A	10+064	RC	H1060mm	Provide New Outlet	1050	12.84	●	19.44	>	16.17	DFL is higher than ground elevation at landside
49	ML064.2	10+169	RC	H900mm	Provide New Outlet	900	11.90	●	19.50	>	15.95	DFL is higher than ground elevation at landside
50	ML064.1	10+266	RC	D600mm	Provide New Outlet	900	11.86	●	19.56	>	15.46	DFL is higher than ground elevation at landside
51	ML064	10+292	BC	H1640mm W2750m	Closed							
52	ML062.3D	10+345	RC	D1200mm	Provide New Outlet	1200	12.78	●	19.61	>	15.64	DFL is higher than ground elevation at landside
53	ML062.2C	10+358	RC	D1200mm	Combined with ML062A							
54	ML062.1B	10+359	RC	D1200mm	Combined with ML062A							
55	ML062A	10+361	RC	D1200mm	Provide New Outlet	3X1200	11.87	●	19.62	>	16.44	DFL is higher than ground elevation at landside
56	ML061.4	10+405	RC	D910mm	Provide New Outlet	900	13.30	non	19.65	<	19.70	Ground elevation at landside is higher than DFL
57	ML061.3	10+506	RC	D910mm	Provide New Outlet	900	12.73	non	19.72	<	20.62	Ground elevation at landside is higher than DFL
58	ML061.2	10+607	RC	D900mm	Provide New Outlet	900	11.60	non	19.77	<	20.30	Ground elevation at landside is higher than DFL
59	ML061.1	10+742	RC	D910mm	Retained							
60	ML058.3	11+632	RC	D500mm	Connect to ML058.2 by RC Collector Pipe							
61	ML058.2	11+634	RC	D500mm	Provide New Outlet	900	13.10	●	17.51	>	17.40	DFL is higher than ground elevation at landside
62	ML058.1	11+638	RC	D300mm	Connect to ML058.2 by RC Collector Pipe							
63	ML056	11+741	SP	D120mm	Not affected by Construction							
64	ML055	11+754	PVC	D100mm	Not affected by Construction							
65	ML053	11+763	SP	D90mm	Not affected by Construction							
66	ML054	11+763	PVC	D100mm	Not affected by Construction							
67	ML052	11+771	PVC	D100mm	Not affected by Construction							

Source: Study Team

Table 6.2.10 Proposed Drainage Works and Facilities (2/5)

NO.	CODE NO.	EXISTING OUTLET			PROPOSED FACILITIES				Justification of Flap Gate			
		Station No.	Type	Dia. or Size (Height x Width) (mm)	Proposed Works	Dia. or Size (Height x Width) (mm)	Invert Elevation	Flap Gate	Design Flood Level (DFL)		Ground Elevation at Landside	Remarks
68	ML050.2	11+782	RC	D800mm	Provide New Outlet	900	12.63	non	20.38	<	20.85	Ground elevation at landside is higher than DFL
69	ML050.1	11+786	PVC	D150mm	Not affected by Construction							
70	ML049	11+793	SP	D95mm	Not affected by Construction							
71	ML050	11+793	PVC	D90mm	Not affected by Construction							
72	ML048	11+801	RC	D180mm	Not affected by Construction							
73	ML047.1	11+809	PVC	D180mm	Not affected by Construction							
74	ML047	11+815	PVC	D100mm	Not affected by Construction							
75	ML046	11+822	RC	D800mm	Not affected by Construction							
76	ML046.1	11+823	PVC	D150mm	Not affected by Construction							
77	ML045	11+829	PVC	D80mm	Not affected by Construction							
78	ML044	11+832	PVC	D130mm	Not affected by Construction							
79	ML043	11+843	PVC	D130mm	Not affected by Construction							
80	ML042	11+856	PVC	D100mm	Not affected by Construction							
81	ML041	11+864	PVC	D160mm	Not affected by Construction							
82	ML040	11+867	PVC	D200mm	Not affected by Construction							
83	ML039	11+878	PVC	D140mm	Not affected by Construction							
84	ML038A	11+890	PVC	D120mm	Not affected by Construction							
85	ML037	11+903	PVC	D50mm	Not affected by Construction							
86	ML036	11+906	RC	D150mm	Not affected by Construction							
87	ML035	11+910	PVC	D180mm	Not affected by Construction							
88	ML034.1A	11+924	PVC	D250mm	Not affected by Construction							
89	ML034	11+933	PVC	D250mm	Not affected by Construction							
90	ML032	11+942	SP	D90mm	Not affected by Construction							
91	ML031	11+945	PVC	D100mm	Not affected by Construction							
92	ML030A	11+956	RC	D600mm	Not affected by Construction							
93	ML029A	11+959	PVC	D50mm	Not affected by Construction							
94	ML028	11+968	PVC	D200mm	Not affected by Construction							
95	ML027	11+974	PVC	D180mm	Not affected by Construction							
96	ML026	11+983	PVC	D120mm	Not affected by Construction							
97	ML025	11+989	PVC	D80mm	Not affected by Construction							
98	ML024	11+990	PVC	D120mm	Not affected by Construction							
99	ML023	11+996	PVC	D120mm	Not affected by Construction							
100	ML022A	12+011	RC	D1100mm	Provide New Outlet	1050	12.03	●	20.50	>	18.26	DFL is higher than ground elevation at landside
101	ML021.1	12+052	BC	H1760mm W1800m	Provide New Outlet	1800X1800	12.27	non	20.51	<	20.88	Ground elevation at landside is higher than DFL
102	ML021	12+145	RC	D900mm	Combined with ML020							
103	ML020	12+146	RC	D900mm	Provide New Outlet	2x900	12.40	●	20.56	>	17.06	DFL is higher than ground elevation at landside
104	ML019.1	12+249	RC	D1200mm	Combined with ML019			non	20.62	<	20.89	Ground elevation at landside is higher than DFL
105	ML019	12+251	RC	D1200mm	Provide New Outlet	2x1200	12.83	●	20.62	>	19.08	DFL is higher than ground elevation at landside
106	ML018	12+365	RC	D800mm	Provide New Outlet	900	11.70	●	20.67	>	19.62	DFL is higher than ground elevation at landside
107	ML017.2	12+388	BC	H450mm W550mm	Provide New Outlet	900	12.10	●	20.68	>	19.50	DFL is higher than ground elevation at landside
108	ML017.1	12+539	BC	H2430mm W1950m	Provide New Outlet	2000X2500	10.65	non	20.76	<	20.89	Ground elevation at landside is higher than DFL
109	ML017	12+542	RC	D920mm	Provide New Outlet	900	11.96	●	20.76	>	19.89	DFL is higher than ground elevation at landside
110	ML016	12+572	BC	H630mm W750mm	Provide New Outlet	900	12.10	●	20.77	>	19.08	DFL is higher than ground elevation at landside
111	ML015	12+668	RC	D600mm	Provide New Outlet	900	12.20	●	20.82	>	19.45	DFL is higher than ground elevation at landside
112	ML013	12+764	RC	D900mm	Provide New Outlet	900	12.20	●	20.87	>	14.68	DFL is higher than ground elevation at landside
113	ML012.3	12+823	RC	D200mm	Provide New Outlet	900	12.50	●	20.90	>	16.03	DFL is higher than ground elevation at landside
114	ML012.201	12+878	RC	D490mm	Provide New Outlet	900	11.80	●	20.93	>	14.27	DFL is higher than ground elevation at landside
115	ML012.2	12+942	RC	D400mm	Provide New Outlet	900	12.20	●	20.96	>	16.16	DFL is higher than ground elevation at landside
116	ML012.1	12+989	RC	D700mm	Provide New Outlet	1200	10.36	●	20.98	>	16.34	DFL is higher than ground elevation at landside
117	ML012	13+037	RC	D600mm	Provide New Outlet	900	11.80	●	21.01	>	14.82	DFL is higher than ground elevation at landside
118	ML011.1	13+073	RC	D450mm	Provide New Outlet	900	12.20	●	21.03	>	14.67	DFL is higher than ground elevation at landside
119	ML009.1	13+145	RC	D250mm	Connect to ML009 by RC Collector Pipe							
120	ML009	13+151	BC	H1550mm W1500m	Provide New Outlet	1500X1500	11.55	●	21.07	>	15.05	DFL is higher than ground elevation at landside
121	ML008	13+159	SP	D150mm	Connect to ML009 by PVC Pipe							
122	ML006	13+185	BC	H180mm W250mm	Connect to ML005 by PVC Pipe							
123	ML007A	13+187	PVC	D50mm	Connect to ML006 by PVC pipe							
124	ML005	13+206	BC	H1000mm W720mr	Provide New Outlet	1000X1000	11.76	●	21.09	>	14.80	DFL is higher than ground elevation at landside
125	ML004	13+261	RC	D900mm	Provide New Outlet	900	11.73	●	21.12	>	14.76	DFL is higher than ground elevation at landside
126	ML003	13+280	PVC	D180mm	Connect to ML004 by PVC Pipe							
127	ML002.1	13+312	BC	H2300mm W3100n	Extend Box Culvert to the proposed sheet pile							
128	ML002	13+315	BC	H2300mm W3100n	Extend Box Culvert to the proposed sheet pile							
129	ML001	13+340	RC	D450mm	Retained							
130	MR151C	5+513	RC	D600mm	Provide New Outlet	900	10.40	non	14.65	<	19.57	Ground elevation at landside is higher than DFL
131	MR151B	5+548	RC	D900mm	Retained							
132	MR151A	5+592	RC	D590mm	Retained							
133	MR151D				Additional Outlet in MCGS Area (U-dit	900	10.60	non	14.67		18.62	Ground elevation at landside is higher than DFL
134	MR151	5+788	RC	620mm	Provide New Outlet	900	10.70	non	14.68	<	16.07	Ground elevation at landside is higher than DFL

Source: Study Team

Table 6.2.11 Proposed Drainage Works and Facilities (3/5)

NO.	CODE NO.	EXISTING OUTLET			PROPOSED FACILITIES				Justification of Flap Gate			
		Station No.	Type	Dia. or Size (Height x Width) (mm)	Proposed Works	Dia. or Size (Height x Width) (mm)	Invert Elevation	Flap Gate	Design Flood Level (DFL)		Ground Elevation at Landside	Remarks
135	MR148	5+847	PVC	D100mm	Connect to MR147A by PVC Pipe							
136	MR147A	5+852	PVC	D120mm	Provide New Outlet	300	11.33	non	14.69	<	16.90	Ground elevation at landside is higher than DFL
137	MR146	5+858	RC	D250mm	Connect to U-Ditch	250	15.65					
138	MR145	5+861	RC	D260mm	Connect to U-Ditch	250	15.66					
139	MR144	5+863	RC	D460mm	Provide New Outlet	900	10.70	non	14.69	<	16.90	Ground elevation at landside is higher than DFL
140	MR141	5+907	RC	D150mm	Provide New Outlet	300	11.38	non	14.69	<	16.73	Ground elevation at landside is higher than DFL
141	MR140	5+940	RC	D150mm	Connect to MR139 by PVC Pipe							
142	MR139	5+949	RC	D200mm	Connect to MR137&MR138 by PVC Pipe							
143	MR138	5+954	RC	D250mm	Combined with MR137			non	14.70	<	16.53	Ground elevation at landside is higher than DFL
144	MR137	5+955	BC	H300mm W300mm	Provide New Outlet	900	11.46	non	14.70	<	16.53	Ground elevation at landside is higher than DFL
145	MR136	5+962	BC	H400mm W400mm	Provide New Outlet	900	11.46	non	14.70	<	16.53	Ground elevation at landside is higher than DFL
146	MR135A	5+969	RC	H200mm W200mm	Connect to MR136 by PVC Pipe							
147	MR134	5+974	RC	D200mm	Connect to MR135A by PVC Pipe							
148	MR133	5+980	RC	D200mm	Connect to MR134 by PVC Pipe							
149	MR132B	5+981	PVC	D100mm	Combined with MR132C							
150	MR132C	5+981	PVC	D100mm	Connect to U-Ditch of MCGS	200	15.90					
151	MR132D	5+981	PVC	D100mm	Combined with MR132C							
152	MR131	5+985	RC	D200mm	Connect to MR133 by PVC Pipe	200						
153	MR130A	5+992	RC	D200mm	Connect to U-Ditch of MCGS	200	15.45					
154	MR130B	5+993	PVC	D100mm	Connect to U-Ditch of MCGS	150	15.43					
155	MR130C	5+993	PVC	D100mm	Combined with MR130B							
156	MR129	6+000	RC	D200mm	Connect to MR128 by PVC Pipe							
157	MR128	6+002	RC	D220mm	Connect to MR127 by PVC Pipe							
158	MR127	6+012	RC	D300mm	Connect to MR126 by RC Collector Pipe							
159	MR126	6+025	RC	D300mm	Connect to MR125 by RC Collector Pipe							
160	MR125	6+035	RC	D380mm	Connect to MR125A by RC Collector Pipe							
161	MR125A				Additional Sluice in MCGS Area	1000x1000	11.70	●	17.40	>	14.13	
162	MR124A	6+111	RC	D300mm	Provide New Outlet	900	12.40	●	17.40	>	15.49	DFL is higher than ground elevation at landside
163	MR124	6+114	SP	D400mm	Provide New Outlet	500	14.61	non	17.40	<	18.32	Ground elevation at landside is higher than DFL
164	MR123	6+119	PVC	D100mm	Combined with MR122							
165	MR121	6+120	PVC	D100mm	Combined with MR122							
166	MR122	6+120	RC	D925mm	Provide New Outlet	900	12.40	non	17.40	<	18.32	Ground elevation at landside is higher than DFL
167	MR120.2	6+169	PVC	D100mm	Combined with MR120							
168	MR120	6+170	RC	D1220mm	Provide New Outlet	1200	12.08	non	17.40	<	17.83	Ground elevation at landside is higher than DFL
169	MR120.1	6+170	PVC	D100mm	Combined with MR120							
170	MR119	6+176	PVC	D100mm	Connect to MR120 by PVC Pipe							
171	MR118	6+189	PVC	D100mm	Connect to MR119 by PVC Pipe							
172	MR117	6+200	PVC	D100mm	Connect to MR115 by PVC Pipe							
173	MR116	6+202	RC	D1220mm	Provide New Outlet	1200	12.08	non	17.40	<	17.99	Ground elevation at landside is higher than DFL
174	MR115	6+214	PVC	D100mm	Provide New Outlet	300	13.10	non	17.40	<	17.99	Ground elevation at landside is higher than DFL
175	MR114	6+226	PVC	D100mm	Connect to MR115 by PVC Pipe							
176	MR113	6+241	PVC	D100mm	Connect to MR114 by PVC Pipe							
177	MR090	6+952	RC	D1000mm	Provide New Outlet	1050	12.24	non	17.55	<	20.33	Ground elevation at landside is higher than DFL
178	MR074	7+197	RC	D890mm	Provide New Sluice	1000X1000	11.80	●	17.71	>	16.85	DFL is higher than ground elevation at landside
179	MR073	7+356	RC	D620mm	Provide New Sluice	1000X1000	10.97	●	17.81	>	15.09	DFL is higher than ground elevation at landside
180	MR072.1	7+399	RC	D600mm	Provide New Sluice	1000X1000	11.36	●	17.83	>	15.86	DFL is higher than ground elevation at landside
181	MR072	7+509	RC	D620mm	Provide New Sluice	1000X1000	11.40	●	17.89	>	17.10	DFL is higher than ground elevation at landside
182	MR071	7+523	RC	D620mm	Provide New Outlet	900	11.50	●	17.90	>	17.20	DFL is higher than ground elevation at landside
183	MR069	7+732	RC	D450mm	Provide New Outlet	900	11.10	non	18.04	<	18.20	Ground elevation at landside is higher than DFL
184	MR068	7+740	RC	D200mm	Connect to MR069 by PVC Pipe							
185	MR067.1	7+779	RC	D380mm	Provide New Outlet	900	11.10	non	18.06	<	18.30	Ground elevation at landside is higher than DFL
186	MR067A	7+840	RC	D1200mm	Provide New Sluice	1200X1200	10.20	●	18.10	>	14.40	DFL is higher than ground elevation at landside
187	MR066A	7+872	BC	H400mm W300mm	Connect to U-Ditch	400X300	16.57					
188	MR066B	7+872	RC	D200mm	Connect to U-Ditch	200	16.26					
189	MR065	7+890	RC	D300mm	Provide New Outlet	300	13.21	non	18.12	<	18.40	Ground elevation at landside is higher than DFL
190	MR064	7+912	RC	D300mm	Provide New Outlet	300	13.31	non	18.13	<	18.39	Ground elevation at landside is higher than DFL
191	MR063	7+935	RC	D250mm	Connect to U-Ditch	250	16.47					
192	MR062	7+936	RC	D290mm	Connect to U-Ditch	300	16.80					
193	MR060	7+960	BC	H460mm W310mm	Connect to U-Ditch	460X310	17.03					
194	MR061	7+960	RC	D300mm	Provide New Outlet	300	14.82	non	18.17	<	18.56	Ground elevation at landside is higher than DFL
195	MR059.3	7+968	RC	D100mm	Connect to MR059.2 by PVC Pipe							
196	MR059.2	7+972	RC	D100mm	Connect to MR059.1 by PVC Pipe							
197	MR059.1	7+976	RC	D100mm	Connect to MR059 by PVC Pipe							
198	MR059	7+993	RC	D250mm	Provide New Outlet	900	12.70	●	18.19	>	17.94	DFL is higher than ground elevation at landside
199	MR058.1	8+128	RC	D600mm	Provide New Outlet	900	12.52	●	18.27	>	18.11	DFL is higher than ground elevation at landside
200	MR058	8+196	RC	D500mm	Connect to MR57 by RC Collector Pipe							
201	MR057	8+199	RC	D620mm	Provide New Outlet	900	12.70	●	18.32	>	17.96	DFL is higher than ground elevation at landside

Source: Study Team

Table 6.2.12 Proposed Drainage Works and Facilities (4/5)

NO.	CODE NO.	EXISTING OUTLET			PROPOSED FACILITIES				Justification of Flap Gate			
		Station No.	Type	Dia. or Size (Height x Width) (mm)	Proposed Works	Dia. or Size (Height x Width) (mm)	Invert Elevation	Flap Gate	Design Flood Level (DFL)		Ground Elevation at Landside	Remarks
202	TR001	8+216	RC	D400mm	Connect to MR57 by RC Collector Pipe							
203	MR056	8+245	RC	D290mm	Provide New Outlet	900	12.73	non	18.34	<	19.31	Ground elevation at landside is higher than DFL
204	MR055	8+312	RC	D1470mm	Provide New Outlet	1500	12.15	●	18.39	>	18.17	DFL is higher than ground elevation at landside
205	MR054	8+328	RC	D250mm	Provide New Outlet	900	12.80	●	18.39	>	18.17	DFL is higher than ground elevation at landside
206	MR053	8+443	PVC	D300mm	Provide New Outlet	900	12.80	non	18.46	<	18.99	Ground elevation at landside is higher than DFL
207	MR050	8+448	RC	D160mm	Demolished due to River Structure							
208	MR048.1	8+449	RC	D160mm	Demolished due to River Structure							
209	MR049	8+449	RC	D160mm	Demolished due to River Structure							
210	MR048.2	8+450	RC	D150mm	Demolished due to River Structure							
211	MR048.3	8+450	PVC	D80mm	Demolished due to River Structure							
212	MR051	8+450	RC	D160mm	Demolished due to River Structure							
213	MR051.1	8+450	RC	D160mm	Demolished due to River Structure							
214	MR052	8+450	RC	D150mm	Demolished due to River Structure							
215	MR052.1	8+450	RC	D150mm	Demolished due to River Structure							
216	MR052.2	8+450	RC	D160mm	Demolished due to River Structure							
217	MR052.3	8+450	RC	D160mm	Demolished due to River Structure							
218	MR048	8+492	RC	D630mm	Provide New Outlet	900	12.80	●	18.50	>	17.84	DFL is higher than ground elevation at landside
219	MR047	8+507	RC	D900mm	Provide New Outlet	900	12.80	●	18.50	>	17.84	DFL is higher than ground elevation at landside
220	MR047.1	8+521	RC	D620mm	Provide New Outlet	900	12.80	●	18.51	>	16.56	DFL is higher than ground elevation at landside
221	MR047.2	8+525	PVC	D100mm	Connect to U-Ditch	100	17.04					
222	MR043.39	8+563	RC	D920mm	Provide New Outlet	1200	12.25	non	18.53	<	19.55	Ground elevation at landside is higher than DFL
223	MR047.3	8+597	PVC	D100mm	Connect to MR048 by PVC Pipe							
224	MR032.50	8+603	RC	D1000mm	Provide New Outlet	1200	11.58	non	18.57	<	20.16	Ground elevation at landside is higher than DFL
225	MR032.5	8+607	PVC	D100mm	Demolished due to River Structure							
226	MR032.4	8+612	PVC	D25.4mm	Demolished due to River Structure							
227	MR032.3	8+615	PVC	D25.4mm	Demolished due to River Structure							
228	MR032.2	8+621	PVC	D25.4mm	Demolished due to River Structure							
229	MR032.1	8+646	PVC	D25.4mm	Demolished due to River Structure							
230	MR031.1	8+657	PVC	D25.4mm	Demolished due to River Structure							
231	MR031.2	8+657	PVC	D25.4mm	Demolished due to River Structure							
232	MR030	8+665	RC	D460mm	Provide New Outlet	900	11.30	non	18.59	<	18.78	Ground elevation at landside is higher than DFL
233	MR029	8+671	RC	D920mm	Provide New Outlet	900	11.30	non	18.61	<	18.92	Ground elevation at landside is higher than DFL
234	MR029.2	8+671	PVC	D50mm	Combined with MR029							
235	MR029.1	8+672	PVC	D50mm	Combined with MR029							
236	MR028	8+674	RC	D600mm	Provide New Outlet	900	11.30	non	18.61	<	18.92	Ground elevation at landside is higher than DFL
237	MR029.3	8+680	PVC	D50mm	Connect to MR027 by PVC Pipe							
238	MR027	8+684	PVC	D460mm	Provide New Outlet	900	11.30	non	18.58	<	18.95	Ground elevation at landside is higher than DFL
239	MR026A	8+932	BC	W1950mm H500	Outlet will flow on Opening on SSP							
240	MR026B	8+934	BC	H2900mm W1950m	Outlet will flow on Opening on SSP							
241	MR026.1	8+943	PVC	D290mm	Outlet will flow on Opening on SSP							
242	MR025.1	9+120	BC	H1100mm W1780m	Provide New Outlet	1800	10.38	non	18.87	<	18.98	Ground elevation at landside is higher than DFL
243	MR025	9+202	BC	H2180mm W690mn	Provide New Outlet	2000X2000	10.15	●	18.92	>	18.88	DFL is higher than ground elevation at landside
244	MR024	9+291	BC	H2030mm W2520m	Provide New Outlet	2000X2000	10.15	●	18.97	>	17.81	DFL is higher than ground elevation at landside
245	MR023	9+453	RC	D920mm	Provide New Outlet	900	13.00	●	19.07	>	15.85	DFL is higher than ground elevation at landside
246	MR022	9+471	RC	D350mm	Provide New Outlet	900	12.60	●	19.08	>	15.81	DFL is higher than ground elevation at landside
247	MR021.3	9+569	RC	D900mm	Provide New Outlet	900	12.32	●	19.15	>	15.94	DFL is higher than ground elevation at landside
248	MR021.30	9+570	RC	D390mm	Provide New Outlet	900	10.79	●	19.15	>	15.94	DFL is higher than ground elevation at landside
249	MR021.30	9+571	RC	D900mm	Provide New Outlet	900	12.45	●	19.15	>	15.94	DFL is higher than ground elevation at landside
250	MR021.2	9+638	RC	D800mm	Provide New Outlet	900	12.32	●	19.19	>	17.07	DFL is higher than ground elevation at landside
251	MR021.1	9+774	RC	D1020mm	Provide New Outlet	1050	12.94	●	19.27	>	15.78	DFL is higher than ground elevation at landside
252	MR021	9+840	RC	D760mm	Provide New Outlet	900	11.59	●	19.31	>	16.52	DFL is higher than ground elevation at landside
253	MR020	9+886	RC	D900mm	Provide New Outlet	900	11.78	●	19.33	>	16.43	DFL is higher than ground elevation at landside
254	MR019	9+903	RC	D640mm	Provide New Outlet	900	11.61	●	19.35	>	16.71	DFL is higher than ground elevation at landside
255	MR018	10+221	BC	H4180mm W3900m	Provide New Outlet	3600x3200	12.85	●	19.54	>	16.98	DFL is higher than ground elevation at landside
256	MR017	10+292	RC	D1500mm	Provide New Outlet	1500	11.29	●	19.59	>	16.09	DFL is higher than ground elevation at landside
257	MR016	10+331	BC	H1140mm W2630m	Closed							
258	MR015.2	10+375	RC	D700mm	Provide New Outlet	900	10.71	●	19.64	>	15.66	DFL is higher than ground elevation at landside
259	MR015.1	10+414	RC	D700mm	Provide New Outlet	900	10.64	●	19.66	>	16.22	DFL is higher than ground elevation at landside
260	MR015.10	10+536	RC	D700mm	Provide New Outlet	900	10.36	●	19.73	>	17.23	DFL is higher than ground elevation at landside
261	MR015A	10+569	RC	D700mm	Provide New Outlet	900	11.70	●	19.75	>	18.28	DFL is higher than ground elevation at landside
262	MR014	10+610	BC	H470mm W470mm	Provide New Outlet	900	11.42	non	19.77	<	20.41	Ground elevation at landside is higher than DFL
263	MR013	10+645	PVC	D80mm	Connect to MR014 by PVC Pipe							
264	MR010	10+726	BC	H1360mm W2000m	Provide New Outlet	1360X2000	11.01	non	19.84	<	19.90	Ground elevation at landside is higher than DFL
265	MR009.1	10+804	RC	D1800mm	Provide New Outlet	1800	10.48	●	19.89	>	14.65	DFL is higher than ground elevation at landside
266	MR009A	10+935	RC	D600mm	Provide New Sluice	1000X1000	10.90	●	19.96	>	17.29	DFL is higher than ground elevation at landside
267	MR008	10+964	RC	D1200mm	Provide New Sluice	x1200x1200	12.70	●	19.97	>	15.90	DFL is higher than ground elevation at landside
268	MR007	10+965	RC	D1200mm	Combined with MR008							

Source: Study Team

Table 6.2.13 Proposed Drainage Works and Facilities (5/5)

NO.	CODE NO.	EXISTING OUTLET			PROPOSED FACILITIES				Justification of Flap Gate			
		Station No.	Type	Dia. or Size (Height x Width) (mm)	Proposed Works	Dia. or Size (Height x Width) (mm)	Invert Elevation	Flap Gate	Design Flood Level (DFL)		Ground Elevation at Landside	Remarks
269	MR006.2	10+975	SP	D530mm	Outlet will flow on Opening on SSP							
270	MR006B	10+977	BC	H1500mm W2050m	Outlet will flow on Opening on SSP							
271	MR006A	10+979	BC	H1500mm W2050m	Outlet will flow on Opening on SSP							
272	MR006.1	10+979	SP	D530mm	Outlet will flow on Opening on SSP							
273	MR004.5	12+738	PVC	D50mm	Outlet will flow on Opening on SSP							
274	MR004.4	12+739	PVC	D50mm	Outlet will flow on Opening on SSP							
275	MR005	12+739	BC	W1900mm H2050n	Outlet will flow on Opening on SSP							
276	MR004.3	13+010	RC	D450mm	Provide New Outlet	900	12.16	●	20.99	>	16.28	DFL is higher than ground elevation at landside
277	MR004.2	13+021	PVC	D50mm	Connect to MR004.1 by PVC Pipe							
278	MR004.1	13+033	PVC	D50mm	Provide New Outlet	300	10.96	non	21.00	<	21.10	Ground elevation at landside is higher than DFL
279	MR003.13	13+043	PVC	D50mm	Connect to MR004A by PVC Pipe							
280	MR004A	13+046	RC	D980mm	Provide New Outlet	1200	10.94	●	21.01	>	15.82	DFL is higher than ground elevation at landside
281	MR003.12	13+054	PVC	D50mm	Connect to MR004A by PVC Pipe							
282	MR003.11	13+064	PVC	D30mm	Connect to MR003.12 by PVC Pipe							
283	MR003.105	13+075	PVC	D50mm	Provide New Outlet	300	10.81	non	21.03	<	21.17	Ground elevation at landside is higher than DFL
284	MR003.104	13+085	PVC	D50mm	Connect to MR003.105 by PVC Pipe							
285	MR003.103	13+097	PVC	D50mm	Connect to MR003.104 by PVC Pipe							
286	MR003.102	13+108	PVC	D30mm	Connect to MR003.103 by PVC Pipe							
287	MR003.101	13+119	PVC	D50mm	Connect to MR003.102 by PVC Pipe							
288	MR003.10	13+128	RC	D300mm	Provide New Outlet	900	12.31	●	21.05	>	15.02	DFL is higher than ground elevation at landside
289	MR003.9	13+130	PVC	D50mm	Connect to MR003.8 by PVC Pipe							
290	MR003.8	13+140	PVC	D50mm	Connect to MR003.5 by PVC Pipe							
291	MR003.5	13+158	PVC	D50mm	Provide New Outlet	300	10.50	non	21.07	<	21.23	Ground elevation at landside is higher than DFL
292	MR003.4	13+169	PVC	D50mm	Connect to MR003.5 by PVC Pipe							
293	MR003.3	13+178	PVC	D50mm	Connect to MR003.4 by PVC Pipe							
294	MR003.2	13+188	RC	D50mm	Connect to MR003.3 by PVC Pipe							
295	MR003.1	13+215	RC	D570mm	Provide New Outlet	900	11.92	●	21.10	>	16.28	DFL is higher than ground elevation at landside
296	MR003	13+236	RC	D450mm	Provide New Outlet	900	11.41	●	21.11	>	16.98	DFL is higher than ground elevation at landside
297	MR002A	13+283	RC	D450mm	Provide New Outlet	900	11.86	●	21.13	>	15.42	DFL is higher than ground elevation at landside
298	MR001A	13+327	RC	D450mm	Provide New Outlet	900	11.61	●	21.15	>	18.70	DFL is higher than ground elevation at landside

Source: Study Team

The location of proposed drainage facility in correspondence to above drainage planning are shown in **Appendix Figure 6.2.2**.

6.2.4 Basic Design Condition of Drainage Facility

6.2.4.1 Basic Design of Outlet

(1) Size of Outlet

The Manning's Formula and Continuity Equation, as shown below, were used to calculate the flow capacity of the outlets (in accordance with the Guidelines).

$$V = \frac{1}{n} \times R^{2/3} \times S^{1/2}$$

$$Q = A \times V$$

Where:

Q = Discharge (m3/sec)

V = Velocity (m/sec)

A = area of cross section (m2)

n = Roughness coefficient

R = Hydraulic radius (m)

S = Flow gradient of longitudinal slope

Manning's coefficient of roughness for different materials, as shown below table was applied in accordance with Guidelines.

Table 6.2.14 Roughness Coefficient

Type	Roughness Coefficient (maximum)
PVC Pipe	0.013
RC Pipe	0.013
In-situ Concrete	0.015

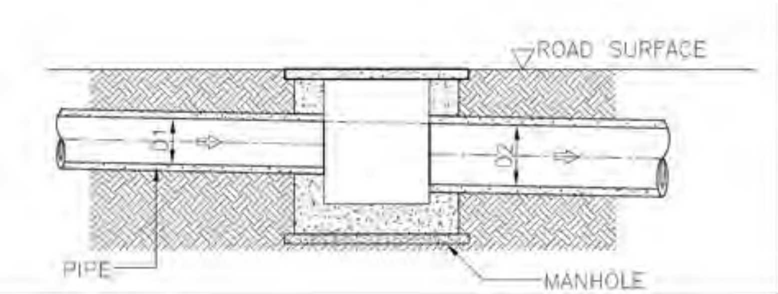
Source: Guideline

(2) Slopes and Velocities

In order to encourage self-cleaning, and minimize sediment build up, pipes should be designed to ensure a minimum flow velocity of 0.8 m/s at pipe full. The maximum velocity to be adopted for piped drainage systems is 5 m/s. The common practice is to use longitudinal slopes that will produce flow velocities within desirable range of 1.0m/sec to 1.8m/sec as mentioned in "Planning and Design Guidelines for Sewerage Facility, 2009, Japan Sewage Works Association" and "Technical Standards and Guidelines for Planning and Design, Volume II: Urban Drainage, March 2002, DPWH, JICA".

(3) Pipe Connection

The pipe-top connection method, as found in the "Technical Standards and Guidelines for Planning and Design, Volume II: Urban Drainage, March 2002, DPWH, JICA", is recommended to adopt for hydraulic reason. In this method, the tops of the upstream and downstream ends of the pipes are aligned as illustrated in following figure.



Source: Study Team

Figure 6.2.5 Pipe-Top Connection Method

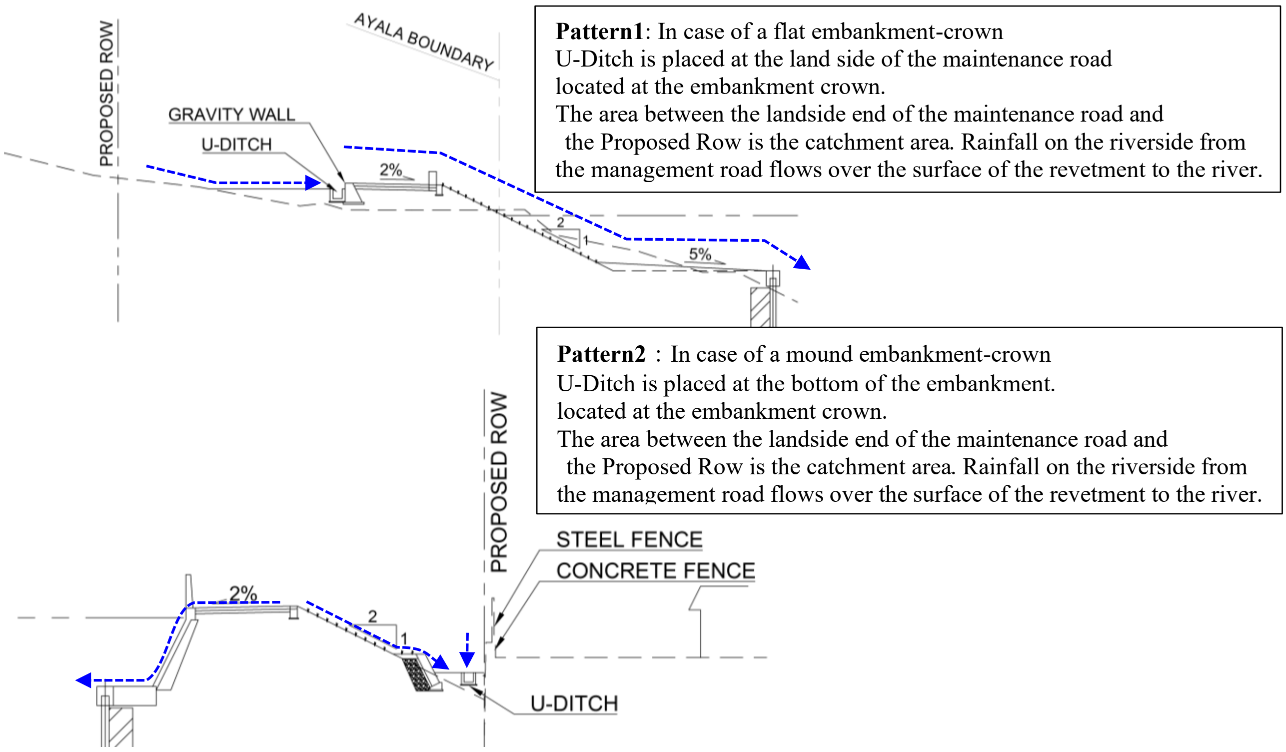
(4) Manhole Spacing

Manholes are normally located at the convergence of two or more pipes, at points for maintenance, and at changes in grade or alignment. The maximum spacing of manholes would be adopted at 50m in accordance with DGCS.

6.2.4.2 Basic Design of Drainage Works Behind the Dike

(1) Allocation of the Drainage Works

Drainage channels will be installed behind new revetment downstream of the Rosario Weir. The drainage channel's purpose is to drain rainwater out of the catchment area which is newly generated by the revetment construction. The type of the drainage channel is a U-shaped ditch. The basic allocation plan of the U-ditch is shown in the following pictures.



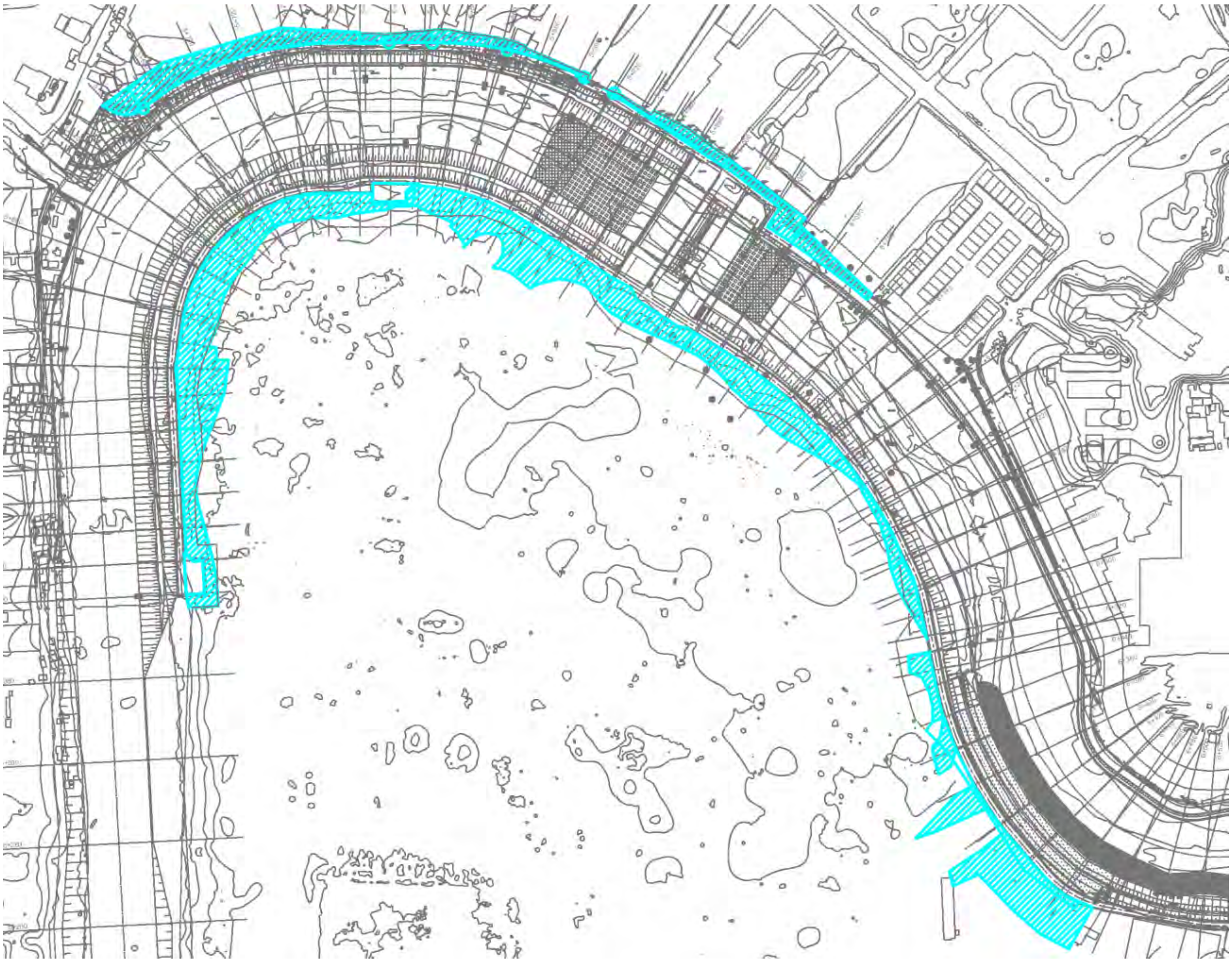
Source: Study Team

Figure 6.2.6 U-Ditch Allocation

(2) Catchment Area

The catchment area of the drainage channel is shown below. In principle, the catchment area is the area created by the construction of a new revetment. On the left bank side, however, a park is planned to be created behind the embankment, and it is expected that some portion of rain in the park will flow the drainage channel.

Therefore, referring to the current park plan, the area on the river side from the park road where the drainage channel will be installed is also included as the catchment area of this drainage channel, even outside the river area.



Source: Study Team

Figure 6.2.7 Catchment Area of the Drainage Works Behind the Dike

(3) Design Discharge and Size of Drainage Works

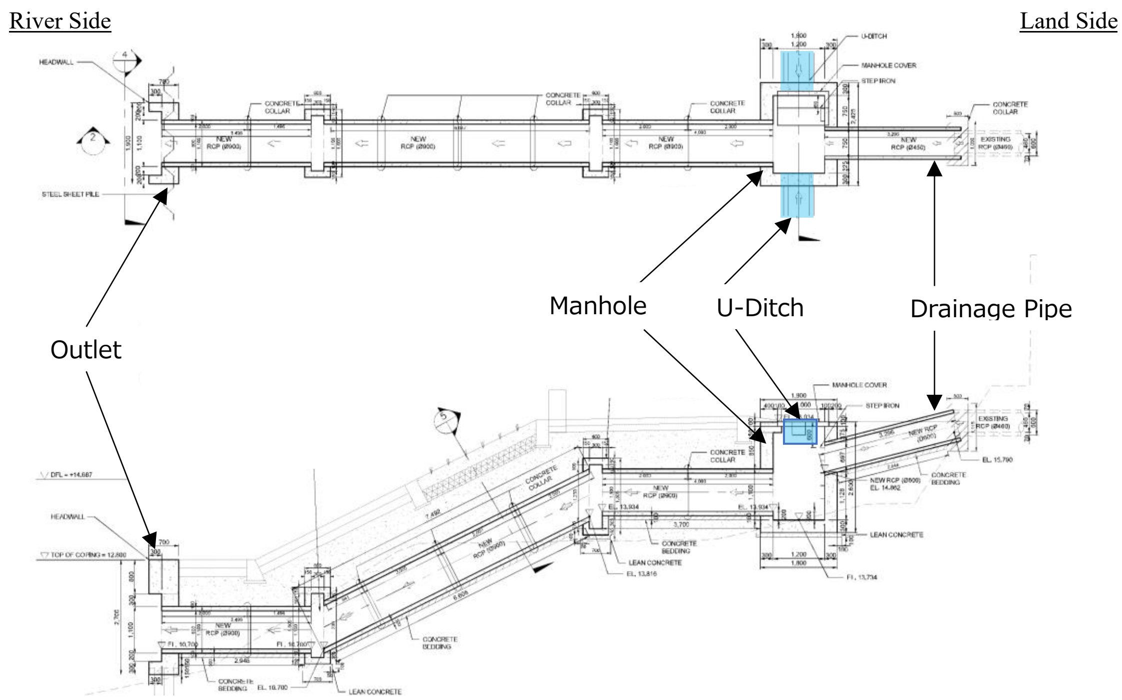
The catchment area is limited to the area near the new embankment, and the target discharge is small. Therefore, the design discharge of the drainage works behind the dike is a 15-year return period (with flow capacity with a freeboard). Also, the flow capacity is confirmed under a 25-year discharge (flow capacity with the full flow without a freeboard). Similar to the drainage facilities in the landside area, the amount of discharge, cross-section of the drainage works is set based on the guideline."

(4) Slopes and Velocities

The flow velocity in the drainage works shall be set at a minimum of 0.8 m / sec following the guideline, considering not to cause sedimentation in the ditch. The vertical gradient shall be designed so that flow velocity does not exceed 1.8 m / sec as a standard while taking into account the embankment layout and ground height.

(5) Allocation of Catch Basin

A catch basin will be installed at the changing point of the channel gradient, the specifications of U-ditch, and the point at which the channel should bend. The drainage will be connected to the manholes provided for landside drainage, as shown in the following figure.



Source: Study Team

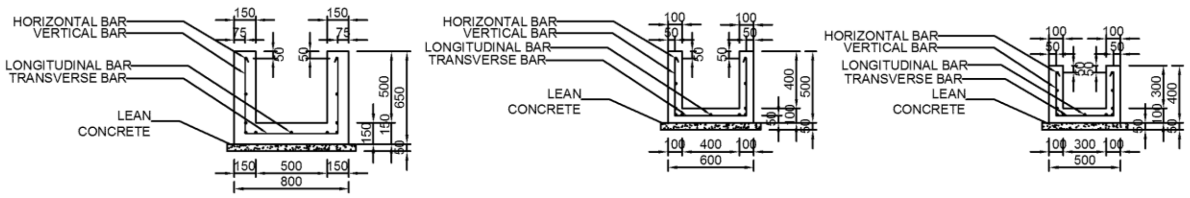
Figure 6.2.8 Connection of Manhole and U-Ditch

(6) Schedule of U-Ditch and Catch Basins

The tables below show the schedule of U-ditch and catch basins behind the dike.

SCHEDULE OF U-DITCH

RIVER	BANK	STATION NO.		LENGTH	TYPE	VERTICAL WALL		BOTTOM SLAB		REMARK
						REINFORCING BAR		REINFORCING BARS		
		START	END			HORIZONTAL BAR	VERTICAL BAR	LONGITUDINAL BAR	TRANSVERSE BAR	
MARIKINA	LEFT	6+460	6+360	135.7	B	D12@250	D12@250	D12@250	D12@250	
		6+360	6+330	35.9	B	D12@250	D12@250	D12@250	D12@250	
		6+330	6+260	57.0	C	D12@250	D12@250	D12@250	D12@250	
		6+260	6+160	85.9	C	D12@250	D12@250	D12@250	D12@250	
		6+160	6+040	99.8	C	D12@250	D12@250	D12@250	D12@250	
		5+980	5+932.3	48.2	C	D12@250	D12@250	D12@250	D12@250	
		5+932.3	5+817.4	86.1	B	D12@250	D12@250	D12@250	D12@250	
		5+817.4	5+809.8	11.6	B	D12@250	D12@250	D12@250	D12@250	
		5+809.8	5+788.7 (a)	20.9	B	D12@250	D12@250	D12@250	D12@250	
		5+7880.7 (a)	5+788.7 (b)	6.5	B	D12@250	D12@250	D12@250	D12@250	
		5+7880.7 (b)	5+700	65.0	B	D12@250	D12@250	D12@250	D12@250	
		5+700	5+500	153.8	A	D12@250	D12@250	D12@250	D12@250	
		5+500	5+420 (b)	86.5	A	D12@250	D12@250	D12@250	D12@250	
		5+420 (b)	5+400 (a)	20.9	A	D12@250	D12@250	D12@250	D12@250	
		5+400 (a)	5+400 (b)	16.2	A	D12@250	D12@250	D12@250	D12@250	
	RIGHT	6+080	6+060	20.8	B	D12@250	D12@250	D12@250	D12@250	
		6+060	6+052(b)	7.6	B	D12@250	D12@250	D12@250	D12@250	
		6+052(b)	6+052(a)	6.9	B	D12@250	D12@250	D12@250	D12@250	
		6+052(a)	6+000	51.8	B	D12@250	D12@250	D12@250	D12@250	
		6+000	5+980	19.3	C	D12@250	D12@250	D12@250	D12@250	
		5+980	5+960	20.0	C	D12@250	D12@250	D12@250	D12@250	
		5+960	5+952	8.0	C	D12@250	D12@250	D12@250	D12@250	
		5+952	5+947	5.0	C	D12@250	D12@250	D12@250	D12@250	
		5+900	5+867.7	47.3	A	D12@250	D12@250	D12@250	D12@250	
		5+867.7	5+854	13.7	A	D12@250	D12@250	D12@250	D12@250	
		5+854	5+783.6	66.0	A	D12@250	D12@250	D12@250	D12@250	
		5+778	5+700	98.5	C	D12@250	D12@250	D12@250	D12@250	
		5+700	5+670	38.1	B	D12@250	D12@250	D12@250	D12@250	
		5+670	5+660	21.8	B	D12@250	D12@250	D12@250	D12@250	



TYPE - A (500 x 500)

TYPE - B (400 x 400)

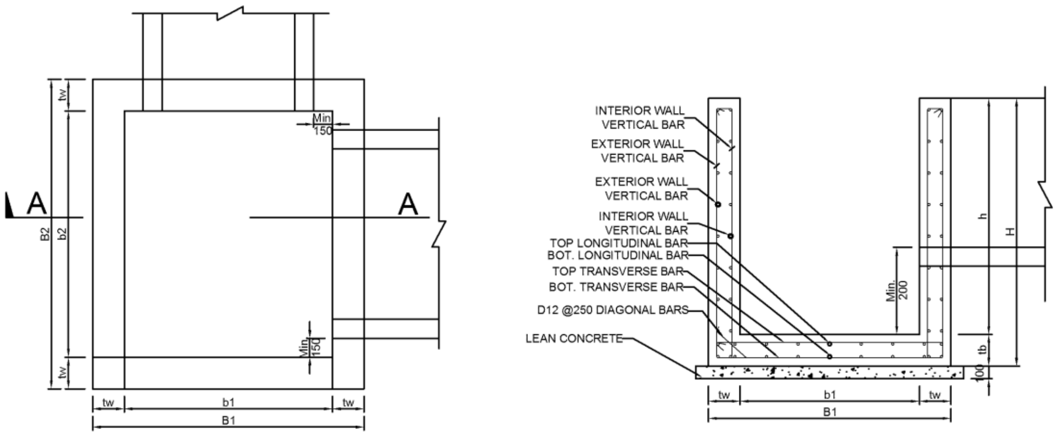
TYPE - C (300 x 300)

Source: Study Team

Figure 6.2.9 Schedule of U-Ditch

SCHEDULE OF CATCH BASIN

RIVER	BANK	STATION	NO		DIMENSIONS								TOP ELEV. (EL.M)	INVET ELEV.	REMARKS
					B1 (mm)	B2 (mm)	H(mm)	b1 (mm)	b2 (mm)	h (mm)	tw (mm)	tb(mm)			
MARIKINA	LEFT	6+360	CB - L	01	1200	1200	750	900	900	600	150	150	19.247	18.547	
		6+260	CB - L	02	1100	1100	650	800	800	500	150	150	19.061	18.361	
		5+932.3	CB - L	03	1200	1200	750	900	900	600	150	150	16.180	15.680	
		5+817.4	CB - L	04	1200	1200	750	900	900	600	150	150	15.965	15.265	
		5+788.7 (a)	CB - L	05	1200	1200	750	900	900	600	150	150	15.867	15.284	
		5+788.7 (b)	CB - L	06	1200	1200	750	900	900	600	150	150	15.867	15.267	
		5+500	CB - L	07	1400	1400	850	1100	1100	700	150	150	15.507	14.807	
		5+420	CB - L	08	1400	1400	850	1100	1100	700	150	150	15.377	14.677	
	RIGHT	6+052 (a)	CB - R	01	1200	1200	750	900	900	600	150	250	15.048	14.458	
		6+052 (b)	CB - R	02	1200	1200	750	900	900	600	150	250	15.068	14.468	
		6+000	CB - R	03	1200	1200	750	900	900	600	150	250	15.324	14.724	
		5+980	CB - R	04	1100	1100	650	800	800	500	150	150	16.190	15.690	
		5+700	CB - R	05	1200	1200	750	900	900	600	150	250	15.178	14.578	



PLAN

SECTION

Source: Study Team

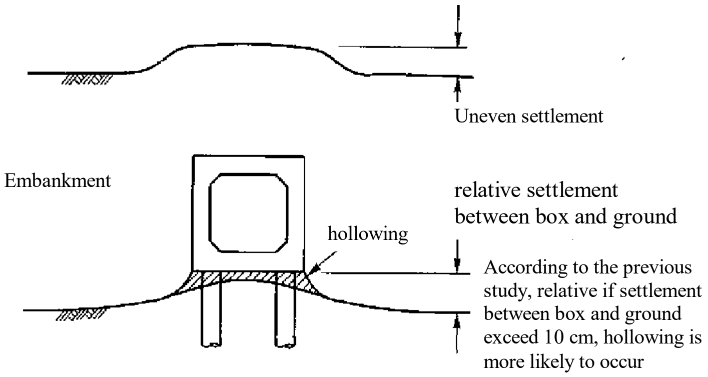
Figure 6.2.10 Schedule of Catch Basin

6.2.4.3 Basic Design of Sluiceway

(1) Type of Sluiceway

As a new outlet in dike area, the sluiceway will be proposed. The soft soil layers underlie the Marikina River area and residual settlement at all sluiceway site exceeds 5 cm. According to “Guideline of flexible Sluiceway” in Japan (November 1998, Japan Institute of Construction Engineering), when residual settlement is more than 5 cm, rigid type sluiceway cannot follow the ground displacement and flexible type should be adopted.

In this design, it cannot avoid the effect of ground deformation especially with the placement of a new embankment dike. In the case where piles are used under the sluiceway, the common observation is that the sluiceway does not settle evenly with the embankment dike. This usually results to a hollowing under the sluiceway, as shown below figure, which gives the dike an uneven, undulating surface. This type of design, therefore, shall not be adopted. Instead, the flexible type sluiceway which can allow the effect of settlement shall be adopted.



Source: Study Team

Figure 6.2.11 Effect of Uneven Settlement with Sluiceway on Pile

(2) Cross-Section Shape

Preferably, a square cross section made of cast-in-place concrete shall be adopted for the following reasons:

- It is more difficult to make cast-in-place concrete circular culvert.
- Pre-cast concrete pipes are available but there would be problems on water-tightness on the joints and longitudinal deformation if the bedding is not properly constructed.
- In case of steel pipe, welding is needed on the joint. This type is inferior to the concrete type on the aspect of construction.

(3) Type of Gate

The floodgates for the Lower Marikina River require the following functions:

- Water-tightness, to avoid backwater entering into land area during flood.
- Drainage, to dispose water from the landside into the river at normal condition

The types of gate that can satisfy these required functions are classified as hinge type and slide type. For the sluiceway, the slide type gate is usually used, as there is less concern about complete gate closure due to accumulation of sediments. However, this type needs manual operation. Some hinge types have automatic operation feature similar to the flap gate, which was used in the Pasig River and Marikina River under PMRCIP Phase II, Phase III and Phase V Project. The flap gate has more advantages over the slide gate on the aspect of operation. Therefore, the Flap Gate would be adopted in this project.

6.3 Basic Design of Manggahan Control Gate Structure (MCGS)

6.3.1 Summary of Basic Design of MCGS

The detailed design of the MCGS was once conducted in the 2002 Detailed Design, Due to the land acquisition issue caused after the PMRCIP-I, the location of the MCGS was reviewed and revised in the 2015 Definitive Plan. In this connection, the study on the detailed design was conducted.

In the basic design study, the following studies were made mainly to:

- ✓ Set a more detailed location based on the results of the 2015 Definitive Plan;
- ✓ Study on the basic specifications (dimensions) of civil structures;
- ✓ Study on the types of gate, structure and material of the gate leaves; and
- ✓ Study on the mechanical and electrical equipment

The specifications determined in the study are shown as the basic design result in the table below.

Table 6.3.1 Summary of Basic Design Results (MCGS)

Items	Specifications	Descriptions/Remarks
Structural Category	Movable Weir	- Certainty of diversion considered. - Dewatering from Laguna Lake also considered.
Location	STA.6+010	- Nearer location from EFCOS (STA.6＋550), considering the availability of land and curve of river channel
DFL	Upstream：EL. 17.400 m Downstream：EL. 14.711 m	
Water Level (for Structural Design)	(Flood) Upstream：EL. 17.400 m Downstream ：EL. 13.425 m	- Water level downstream is based on the value of hydraulic model test in this study.
	(Low Water Case) Upstream：EL. 17.400 m Downstream：EL. 11.003 m	- Water level downstream is based on the water level observed at Rosario Weir (Junction Side)
Design Dike Crown	Marikina River：EL.+18.600	- DFL + Freeboard: 1.2m
Number of Gates	2-Span	- Flexibility of operation and redundancy are considered in case of malfunction
Span	31.8m + 15.20 m （Clear Span：28.7m+11.3m）	- Minimum: 12.5 (To avoid closure of water conduction) - 40 m width with 2 gates to secure the water width in ordinary condition - Regulate discharge at 500m^3/s by fully opening only the side which has 11.2 m clear span
Top of The Gate	EL. 17.400 m	- DFL
Sill Elevation	EL. 7.850 m	- Design Riverbed
Energy Dissipator	Stilling Basin L=26.4m、EL.7.050m	- Not to disturb ferry boats passing in ordinary condition and floods flowing smoothly - About 20% of difference of water level between upstream and downstream - Refer to Japanese actual cases[1)]
Length of Main body	20.5m	- The width of maintenance bridge, staircase, column, pier considered
Length of Apron	Upstream：15m	- Refer to Japanese actual cases[1)], 1/2 of downstream side
	Downstream：30m	- Based on the creep distance for seepage control
Length of Bed Protection	Upstream：15m	- Same length as the upstream apron
	Downstream：44m	- Calculated in accordance with “Structural Design Guide for Groundsill”
Top of Main body	EL. 19.0 m	- Finished elevation of revetment (including extra embankment)
Top of Gate Control Structure	EL. 32.05m	- 1.5 m allowance above “Top of Gate” considered
Top of Gate	EL. 17.400 m	- Same as DFL
Type of Gate	Lift Roller Gate	- Selected based on maintenance and economic aspect

Items	Specifications	Descriptions/Remarks
Gate Leaf Structure/ Material	Shell Structure/ Alloy Saving Duplex Stainless Steel	- (Structure) Garbage/debris flow, sedimentation and cost efficiency considered - (Material) Lower Lifecycle Cost, applicability under brackish water condition in Pasig Marikina River
Operation	Hoisting Device：Electric motion (commercial power supply) Operation；remote and local control	- Commercial power supply is used with 2 units of generator for backup in case of blackout - In addition to remote and local control, emergency operation panel is installed in generator house
Maintenance Bridge	PC Girder Bridge (Effective Width：4.0m)	- Only maintenance vehicles pass

[1] *Design of Weir, Japan Dam Engineering Center*

6.3.2 Summary of the Design in PMRCIP-I and Definitive Plan in 2015

(1) Summary of the Design in PMRCIP-I

A summary of the design in PMRCIP-I is given in **Table 6.3.2**, and the dimensions of major structures are shown in **Figure 6.3.1**.

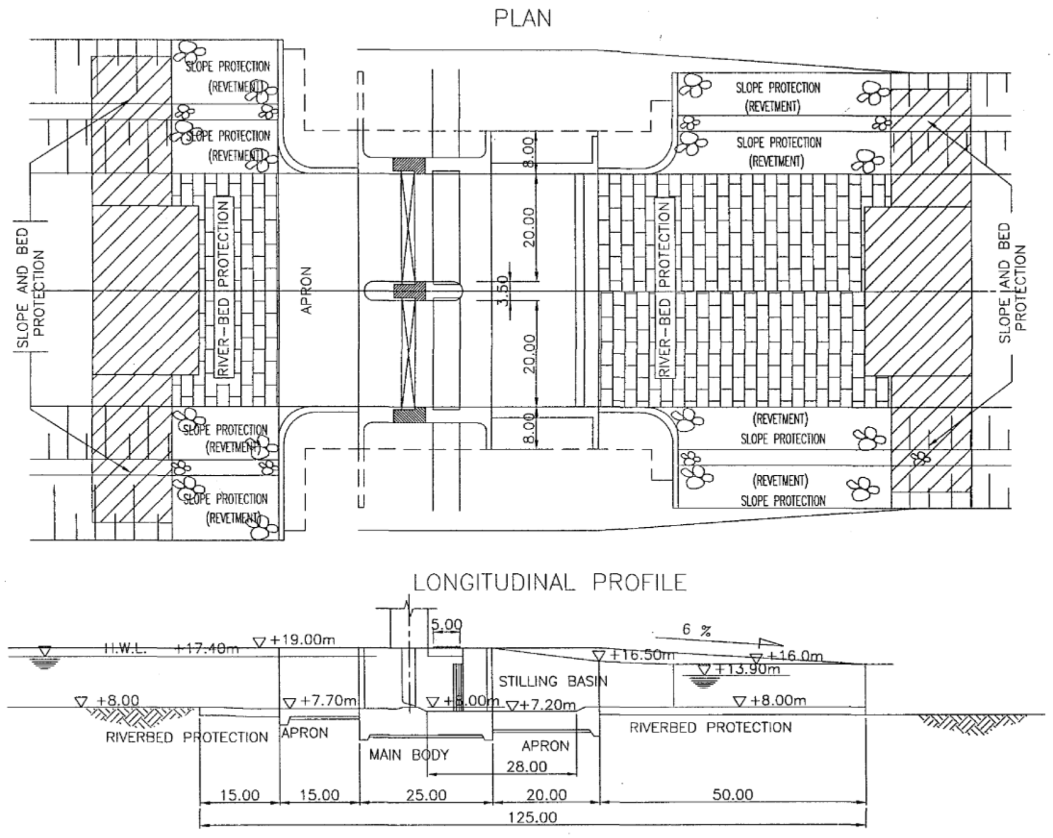
In the detailed design of PMRCIP-I, after the lengths of apron and bed protection works were calculated numerically, these were validated in the hydraulic model test. As a result, the dimensions were finalized.

Table 6.3.2 Summary of Design in the Detailed Design of PMRCIP-I

Items	Specifications	Descriptions / Remarks
Structural Category	Movable Weir	- Certainty of diversion considered - Dewatering from Laguna Lake also considered
Location	STA.6+300	- It can be observed from EFCOS（STA.6＋550） - This location is near the diversion point to Manggahan Floodway; the length of dike considering the backwater is shorter.
DFL	Upstream：EL. 17.400 m Downstream：EL. 14.743 m	
Water Level (for Structural Design)	(Flood) Upstream：EL. 17.40 m Downstream：EL. 14.771 m	- Water level in downstream is based on the value from the hydraulic model test
	(Emergency Upstream：EL. 17.40 m Downstream：EL. 10.000 m	- MLLW of Manila Bay - Considering safety factor which is 1.5 times of it in flood condition
Number of Gates	2 Span	- Considering flexibility of operation and redundancy in case of malfunction
Span	23.5m x 2	- Minimum Width: 12.5 m (To avoid closure of water conduction) - 40 m width with 2 gates to secure the water width in ordinary condition
Top of The Gate	EL. 17.400 m	- DFL
Sill Elevation	EL. 8.000 m	- Design Riverbed - Considering ferry boat passing in the low water level
Energy Dissipator	Stilling Basin L=28.0m、EL.7.2m	- 20% of difference of water level between upstream and downstream - The length is based on the Hydraulic Model Test
Length of Main body	25m	-
Length of Apron	Upstream：15m	- 1.5 times of the water depth in the upstream side
	Downstream：20m	- The length is based on the Hydraulic Model Test
Length of Bed Protection	Upstream：15m	- 1.5 times of the water depth in the upstream side
	Downstream：50m	- The length is based on the Hydraulic Model Test
Top of Gate	EL. 19.00 m	- Same level as the proposed dike
Type of Gate	Lift Roller Gate	- Selected based on maintenance and economic aspect

Items	Specifications	Descriptions / Remarks
Gate Leaf Structure/ Material	Shell Structure/ Steel	- Selected based on maintenance and economic aspect - The hoist load is smaller due to the lower down-pull force
Operation	Hoisting Device：Electric motion (commercial power supply) Operation；remote and local control	- Commercial Power Supply is basically for the equipment excluding mortar. - In addition to remote and local control, emergency operation panel is installed in generator house
Maintenance Bridge	PC Girder Bridge (Effective Width：4.0m)	- In accordance with NSCP, M-18 Load is considered.

Source: Final Report of the 2002DD



Source: Final Report of the 2002DD, Vol. VII, F-57

Figure 6.3.1 Major Dimensions of MCGS in the detailed design of PMRCIP-I

(2) Summary of the Design in Definitive Plan in 2015

1) Change of Location

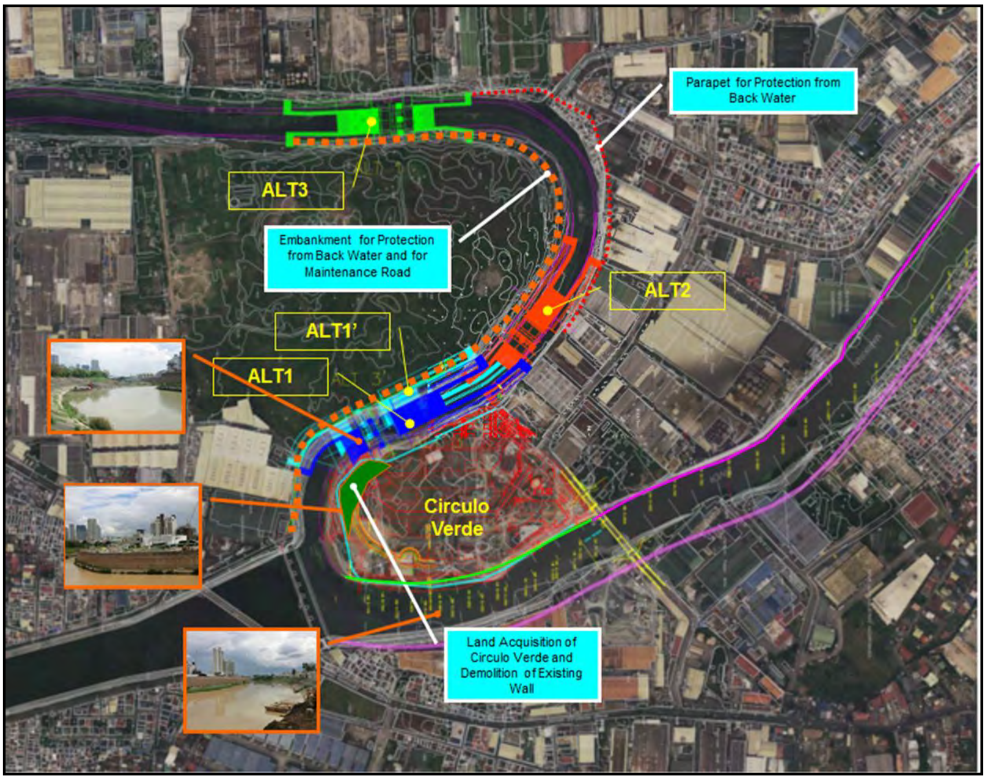
After the detailed design in the 2002DD, the area in the right bank around STA.6+300 was developed. Hence, the location of the proposed MCGS needed to be changed due to the availability of the land. In this connection, the location was changed in the 2015Definitive Plan.

Table 6.3.3 shows the comparison among the location alternatives in the Definitive Plan Study, and the location of each alternative is indicated in **Figure 6.3.2**. Eventually, “Alternative-2, Sta.6+050” was selected considering the economic aspect and easy observation from EFCOS.

Table 6.3.3 Comparison of Construction Location of MCGS

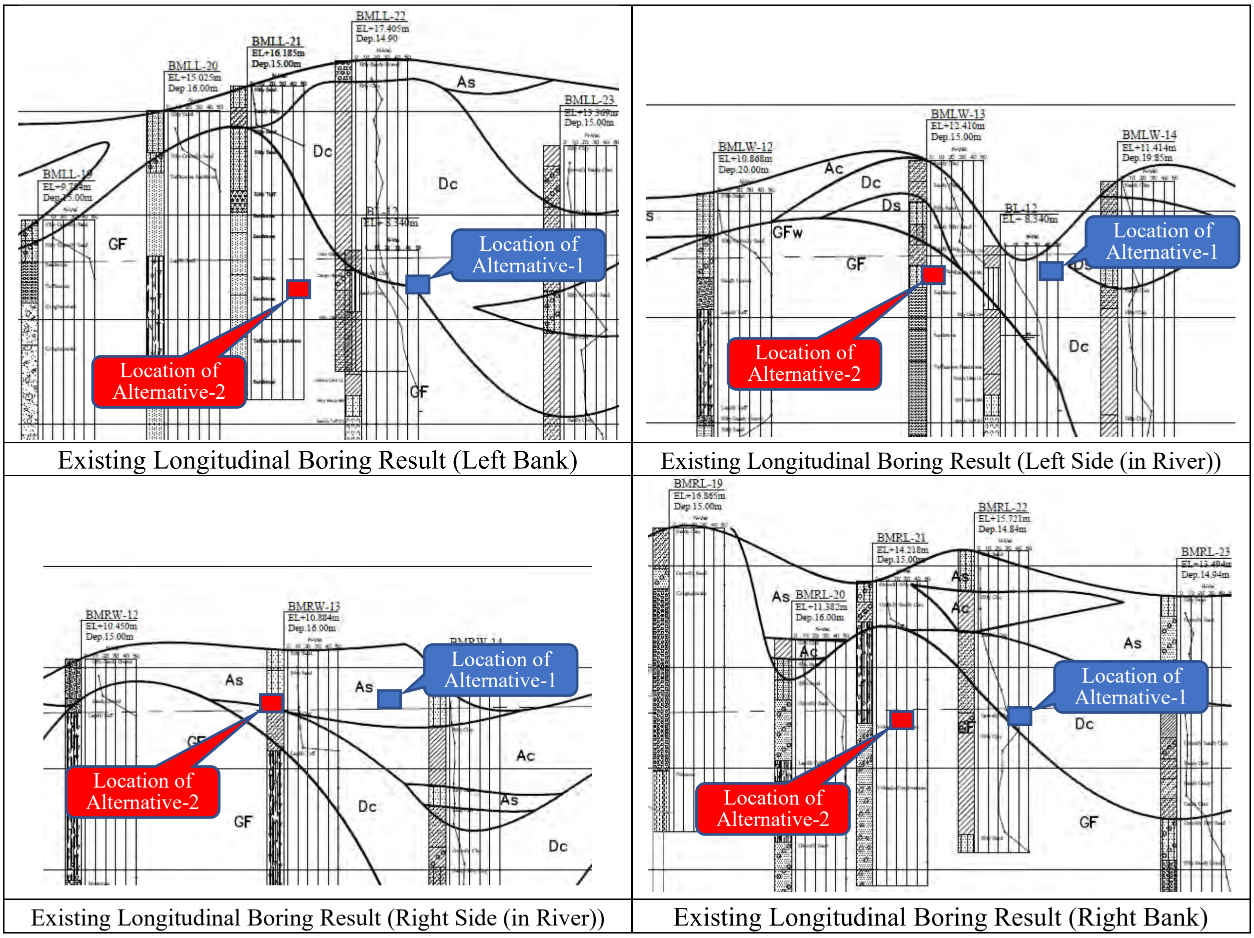
Alternative	Location	Description	Cost Ratio*
Alternative-1	Sta.6+300	- Original location (in 2002/DD). - Necessity of land acquisition of Circulo Verde development area. - Since Rosario Weir and EFCOS Rosario Master Control Station are near, the upstream side of MCGS can be observed easily. - In addition, the operation of MCGS is safer and easier than other alternative locations.	1.00
Alternative-1a	Sta.6+300	- Shifting left side at the location of Alternative-1 to avoid existing newly developed Circulo Verde. - Necessity of land acquisition at the left bank area. - Since Rosario Weir and EFCOS Rosario Master Control Station are near, the upstream side of MCGS can be observed easily. - In addition, the operation of MCGS is safer and easier than other alternatives.	1.03
Alternative-2	Sta.6+050	- 250 m downstream from the original location proposed. - MCGS can be constructed within the river channel, so that land acquisition is minimized. - Since Rosario Weir and EFCOS Rosario Master Control Station are near, the upstream side of MCGS can be observed easily. - In addition, the operation of MCGS is safer and easier than in other alternatives. - The geological condition is almost same as Alternatives 1 and 1a. (See **Figure 6.3.3**)	0.98
Alternative-3	Sta.5+400	- 1.2 km downstream of Rosario Weir (in 1990/FS). - Since Rosario Weir and EFCOS Rosario Master Control Station is 1.2 km away, the operation of MCGS cannot be confirmed visibly. - Floodwall upstream of MCGS is much longer than other alternatives. - Electrical/communication lines to the EFCOS Rosario Master Control Station are much longer than other alternatives. - For the drainage creek joining at 170 m upstream of MCGS, the provision of high walls or pumping station with gate is required.	1.59

Note: Cost consist of direct construction cost and cost for land acquisition.*
Source: Final Report of 2015IV & V-FS, P 6.6



Source : Final Report of The 2015IV&V-FS, F6-10

Figure 6.3.2 Location of Each Alternative



Source : Final Report of The 2015IV&V-FS

Figure 6.3.3 Geological Conditions of Each Alternative

2) Review on the Specifications of Civil Structure

Since the ground condition is not significantly different, the major specifications of civil structures are maintained from the detailed design in the 2002DD. The DFL between MCGS and Rosario Weir is flat. Hence, when the location of MCGS is shifted to the downstream, the DFL at MCGS locations will not be changed.

3) Review on the Gate Facility

As in civil structures, there is no change in the design condition in principle. Nevertheless, the following were stated as items to be required for review during the forthcoming detailed design stage.

(a) Boat Landing/Launching Equipment
- ✓ Power supply system to winch motor
- ✓ Starting method of winch motor

(b) Power Supply and Distribution System
- ✓ Change the specifications and system to minimize voltage drops and reduce cable size
- ✓ Installation of transformer for the power supply from DED to lighting

(c) Communication System in EFCOS
- ✓ Repair works of telemetry system
- ✓ Equipment layout of 3rd floor Rosario master control station

4) Summary of Major Specifications

The major change in the 2015 Definitive Plan was only the location of the weir. However, due to this change, the specification based on the river plan is also updated. **Table 6.3.4** gives a summary of major specifications determined in the 2015 Definitive Plan.

Table 6.3.4 Summary of Design in the 2015 Definitive Plan

Item	Specifications	
	2002 PMRCIP-I	2015 Definitive Plan
Structural Category	Movable Weir	No Update
Location	STA.6+300	STA.6+050
DFL	Upstream：EL. 17.400 m Downstream：EL. 14.743 m	Upstream：EL. 17.400 m Downstream：EL. 14.710 m
Water Level (for Structural Design)	(Flood) Upstream：EL. 17.40 m Downstream：EL. 13.523 m	No Update
	(Emergency Upstream：EL. 17.40 m Downstream：EL. 10.000 m	Same as Above
Number of Gates	2 Span	Same as Above
Span	20m x 2	Same as Above
Top of the Gate	EL.17.400	Same as Above
Sill Elevation	EL. 8.00 m	EL. 7.85 m
Energy Dissipator	Stilling Basin (L=28.0m、EL.7.2m)	No Update
Length of Main body	25m	Same as Above
Length of Apron	Upstream：15m	Same as Above
	Downstream：20m	Same as Above
Length of Bed Protection	Upstream：15m	Same as Above
	Downstream：50m	Same as Above
Top of Gate	EL. 19.00 m	EL.18.85m
Type of Gate	Lift Roller Gate	No Update
Gate Leaf Structure/Material	Shell Structure/ Steel	Same as Above
Operation	Hoisting Device：Electric motion (commercial power supply) Operation；remote and local control	Same as Above
Maintenance Bridge	PC Girder Bridge (width：4.5m)	Same as Above

Source: 2002 PMRCIP-I and 2015IV & V-FS

6.3.3 Basic Design of MCGS

6.3.3.1 Water Level Condition

As is indicated in the 2015 Definitive Plan, DFL between MCGS and Rosario Weir is flat. Hence, the water level condition is set as shown in **Table 6.3.5**.

Table 6.3.5 Water Level Condition at MCGS

Water Level Condition	Upstream	Downstream	Remarks
DFL	EL.17.400 m	EL.14.711 m	
Flood	EL.17.400 m	EL.14.711 m	DFL is used. (Water level downstream is based on the value from the hydraulic model test is reflected in the detailed design)
For Design of Gate Facility	EL.17.400 m	EL.10.794	Water level downstream is based on non-uniform flow calculation in the proposed river channel.

Source: Study Team

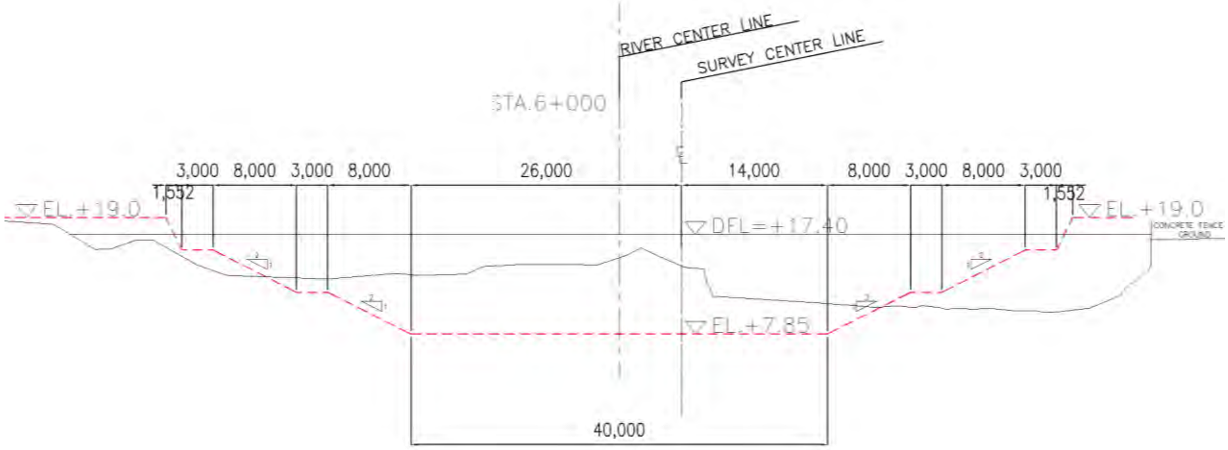
6.3.3.2 Condition of River Channel

Table 6.3.6 gives a summary of the condition of river channel location of MCGS. **Figure 6.3.4** shows the typical cross section around the proposed MCGS. In the right bank there are the existing concrete fences.

Table 6.3.6 Specification of River Channel at MCGS

Item	Specifications	Remarks
Design Discharge	500m^3/s	
DFL	Upstream : EL.+17.400 Downstream : EL.+14.708	
Design Dike Crown	Upstream : EL.+18.600 Downstream : EL.+15.708	
Finished Elevation of Revetment	Upstream : EL.+19.000	Flat up to Rosario Weir Point
Design Riverbed	EL.7.850	
Width of Low Water Channel	Bottom Width : 40m	

Source: Study Team



Source: Study Team

Figure 6.3.4 Typical Cross Section of the River Channel around MCGS

6.3.3.3 Boats/Ships and Other Conditions

(1) Specifications of Boats/Ships Operated in the Target Stretch

In accordance with the report of 2002PMRCIP-I, specifications indicated in **Table 6.3.7** to **Table 6.3.9** will be adopted. The operation of ferry boats has been temporarily stopped, and it is still not operational in June 2019.

Table 6.3.7 Specifications of Boats

Item	Speedboat	Small Boat
Power (HP)	35	12
Weight (kg)	250	-
Passenger	4	4-5
Speed (Maximum)	50-60 km/h (13.9-16.7 m/s)	20 km/h (5.6 m/s)
(Normal)	-	8 km/h (2.2 m/s)

Source: Final Report of 2002 PMRCIP-I Vol. VII, P2-2

Table 6.3.8 Specifications of Ferry Boat

Items	Ferry Boat	
	Type-1	Type-2
Weight (ton)	19.4	16.4
Length (m)	15.1	11.9
Width (m)	5.0	2.5
Required River Depth (m)	1.9	1.3
Vertical Clearance above River Surface (m)	3.0	3.0

Source: Final Report of 2002 PMRCIP-I Vol. VII, P2-2

Table 6.3.9 Specifications of Barge

Items	Tugboat	Barge
Length of Boat (m)	8.0	35.5
Width of Boat (m)	5.0	9.0
Required River Depth (m)	3.0	2.7

Source: Final Report of 2002 PMRCIP-I Vol. VII, P2-2

(2) Required Condition for Boat/Ship Navigation

The required condition for the passage of boats/ships as been determined as shown in **Table 6.3.10** in the report of 2002 PMRCIP-I. The same condition is adopted in this study. Although the operation of ferry boats has been suspended, dredging is currently being implemented and the operation of ferry boats may resume in the future. Accordingly, the design of MCGS shall be conducted considering the passage of ferry boats/ships.

Table 6.3.10 Required Condition for Boat/Ship Navigation

Items	Condition	
	Upstream from Sta. 5+450 (by Ferry Navigation)	Downstream from Sta. 5+450 (by Barge Navigation)
Minimum Width of a Span	10 m	12 m
Minimum Water Depth	2.0 m	2.0 m
Minimum Vertical Clearance	3.0 m	3.0 m

Source: Final Report of 2002 PMRCIP-I Vol. VII, P2-2

(3) Confirmation of Number of Days in which Boats Can Pass Through MCGS

The relationship among 95-day water level, 185-day water level, 275-day water level and 355-day water level at the MCGS location and the design riverbed height (EL.+ 7.850 m) is summarized in **Table 6.3.11**. According to this table, it is possible to secure the depth of about 3 m at the 355-day water level. Since the draft of the ferry boat which is the target ship to be studied is about 2.0 m, there is still a margin of about 1 m. In addition, the gate lower end height at the time of gate opening is EL.+19.000 m (refer to P**6-125**), and there is a sufficient margin for the height (3.0 m) from the water surface of the ferry boat. Accordingly, the riverbed at the MCGS location and the height at the bottom of the gate are not matters of concern in the passage of boats.

Table 6.3.11 Water Depth at MCGS

Items	95-day Water Level	185-day Water Level,	275-day Water Level,	355day Water Level,
Number of days in which the water level is higher than indicated below.	95	185	275	355
Water Level (EL.+m)	11.541	11.085	10.749	10.641
Design Riverbed (EL.+m)	7.850	7.850	7.850	7.850
Water Level (m)	3.691	3.235	2.899	2.791

Note: Water levels have been calculated based on the observed water level.
Source: Study Team

6.3.3.4 Condition with the Existing Structures

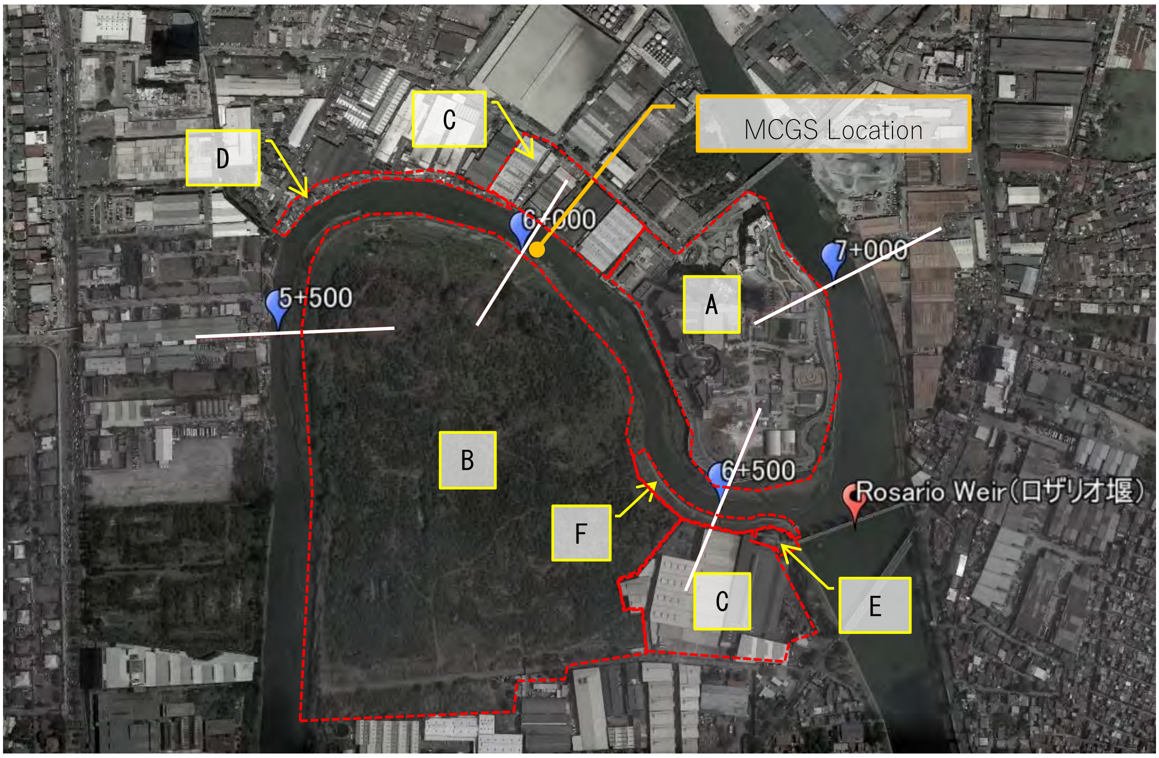
Table 6.3.12 shows the existing major structure which shall be considered during study of layout of facilities and construction plan.

Table 6.3.12 Conditions by the Existing Major Structures

Sybol[1)]	Item	Location	Description
A	Circulo Verde	Sta.6+100 - Sta.6+600 Right Bank	Residential Area and Roads has been implemented. There are flood walls along the river.
B	Development by Ayala	Sta.4+400 - Sta.6+450 Left Bank	As of April in 2019, There is no building. Residential area, roads and parks are planned to be developed. Careful study is needed in the layout of revetment and access road.
C	Large Scale Factories and Warehouse	Sta.5+700 - Sta.6+100 Right Bank Sta.6+450 - Sta.6+600 Left Bank	Especially in the right bank from Sta.5+950 to Sta.6+100 and left bank from Sta.6+500 to Sta.6+550, the existing factory and warehouse stand close to the river. Careful study is needed in the layout of revetment.

Sybol1)	Item	Location	Description
D	Residential area of ISF	Sta.5+650 - Sta.5+900 Right Bank	Careful study is needed in the layout of revetment and access road.
E	EFCOS	Sta.6+600 Right Bank	There is control building of EFCOS and gate console. Careful study is needed in the layout of revetment and access road.
F	Existing Revetment	Sta.6+350 - Sta.6+600 Left Bank	There is the existing revetment. Careful study is needed in the layout of revetment and access road.

1）Symbol indicates the locations of the existing structures shown in ***Figure 6.3.5***
Source: Study Team



Source ：Study Team Added on Google Earth

Figure 6.3.5 Existing Major Structures around MCGS

6.3.3.5 Geotechnical Condition

(1) Past Geotechnical Investigation

The results of the geotechnical investigation in the vicinity of the MCGS site are summarized.

Table 6.3.13 Geotechnical Investigation

Study Name	BHNo.	Remarks
1.2015 IV & V-FS Report	BHMCGS01-02 BHMCGS04-06	Lower Marikina River, only borehole log is available.
2. 2002 Phase I Report	BMRL -21 BMLL -21 BMLW -13 BMRW -13 BMCGS-01, 02	
3. 2019 Phase IV	G01 G02 G03 G04 G05 G06 G07	

Source: Study Team

(2) Geological Overview

The results of geological surveys around the MCGS show that the alluvial sand layer (As) deposits in is a maximum thickness of about 6 to 7 m from the top to. This stratum is dominated by sand but has alternate layers with cohesive soil in some locations. Since even cohesive soil contains sand, the stratum is entirely classified as sandy soil (As).

Weathered sandstone is found below it. The surface layer of the sandstone may become sandy due to strong winds, and the sandstone is collected as gravel or short columnar core during drilling boreholes due to significant weathering overall.

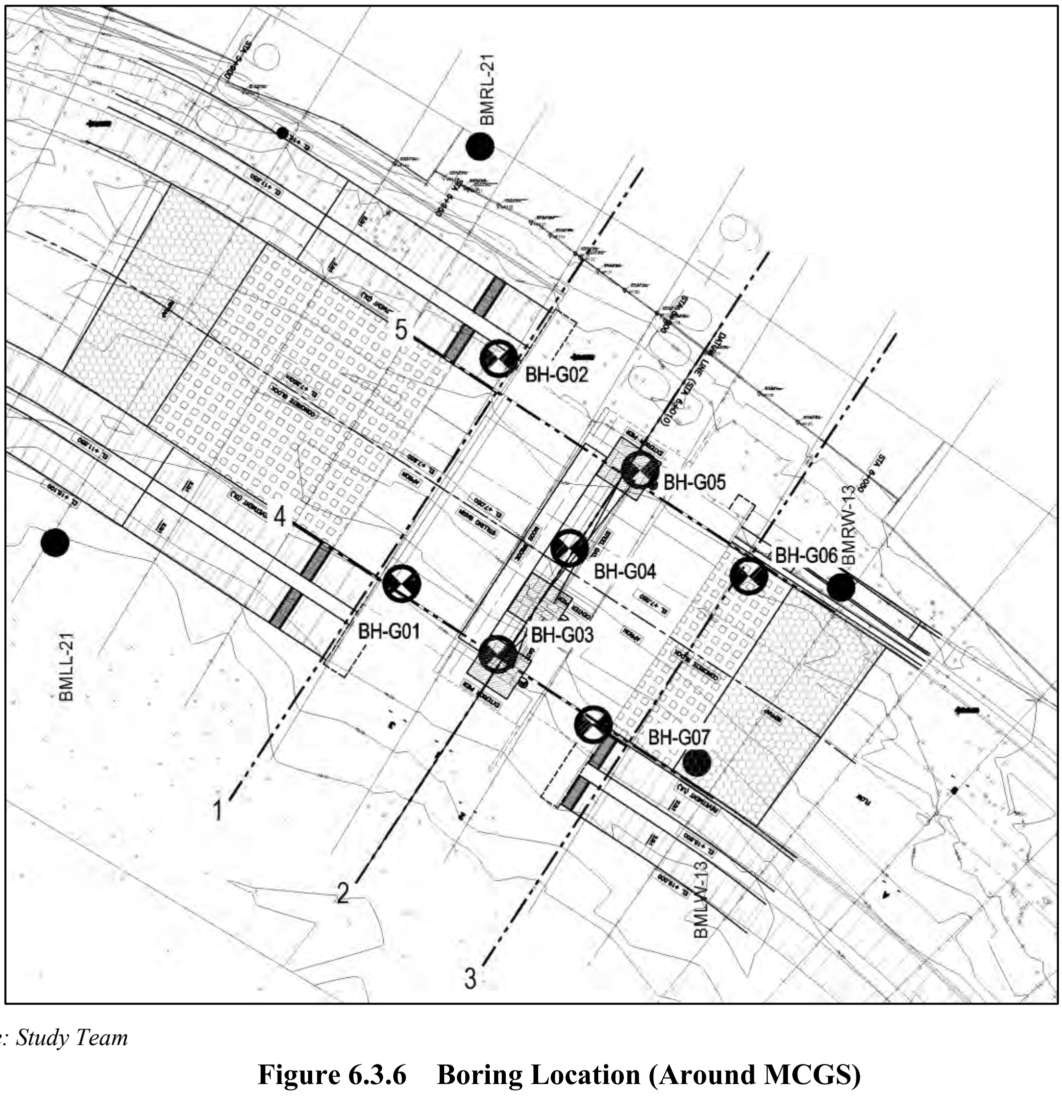
Table6.3.14 Stratification in the Vicinity of the MCGS Site

Geological Age	Symbol	Soil Classification	Features
Alluvial Epoch	As	Sandy Soil	It is alluvial sand and soft with N value of about less than10. fine-grained sand.
Pleistocene Epoch	GFf	Bedrock	Guadalupe layer. It consists of tuff, siltstone, sandstone and conglomerate.

Source: Study Team

(3) Borehole Location Map

The borehole locations around the MCGS is shown below.

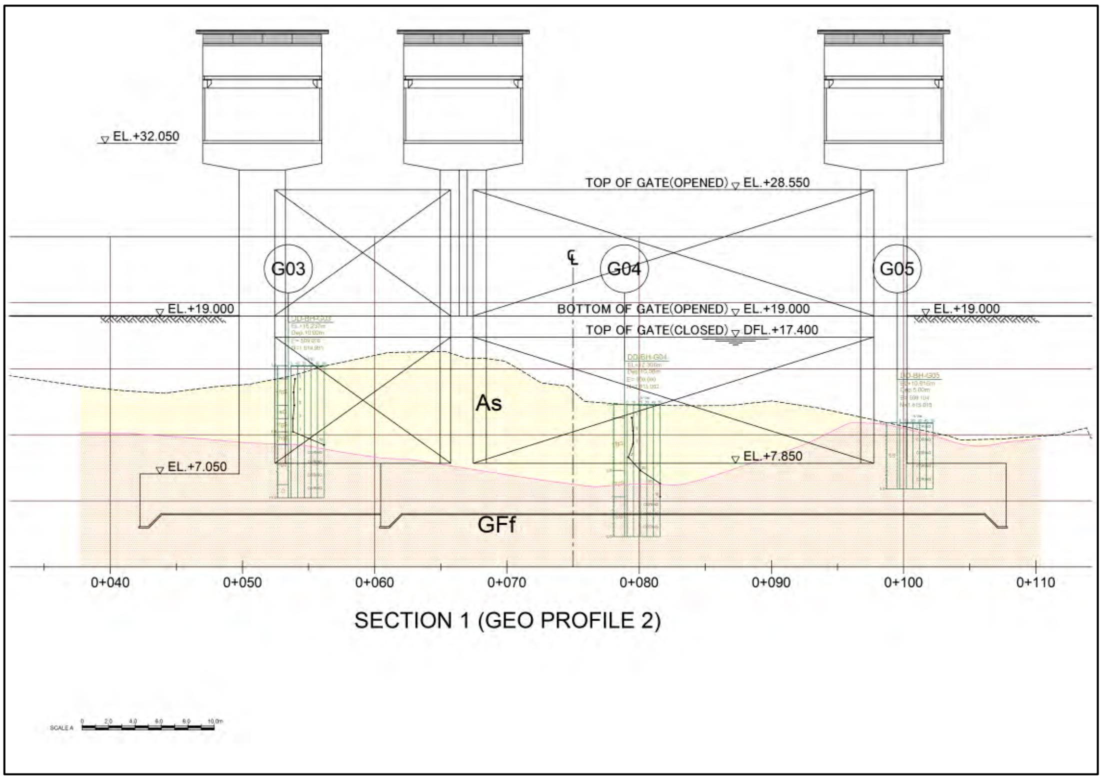


Source: Study Team

Figure 6.3.6 Boring Location (Around MCGS)

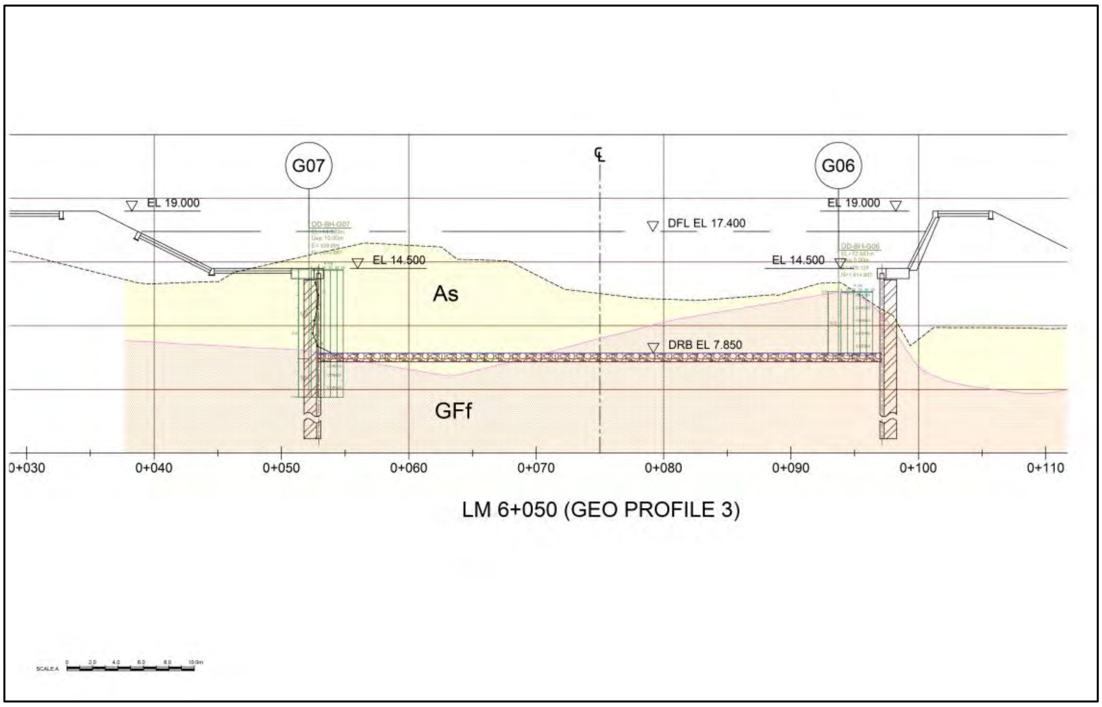
(4) Assumed Geological Cross Section

Assumed geological cross sections around the MCGS is prepared based on the geological survey results in this study.



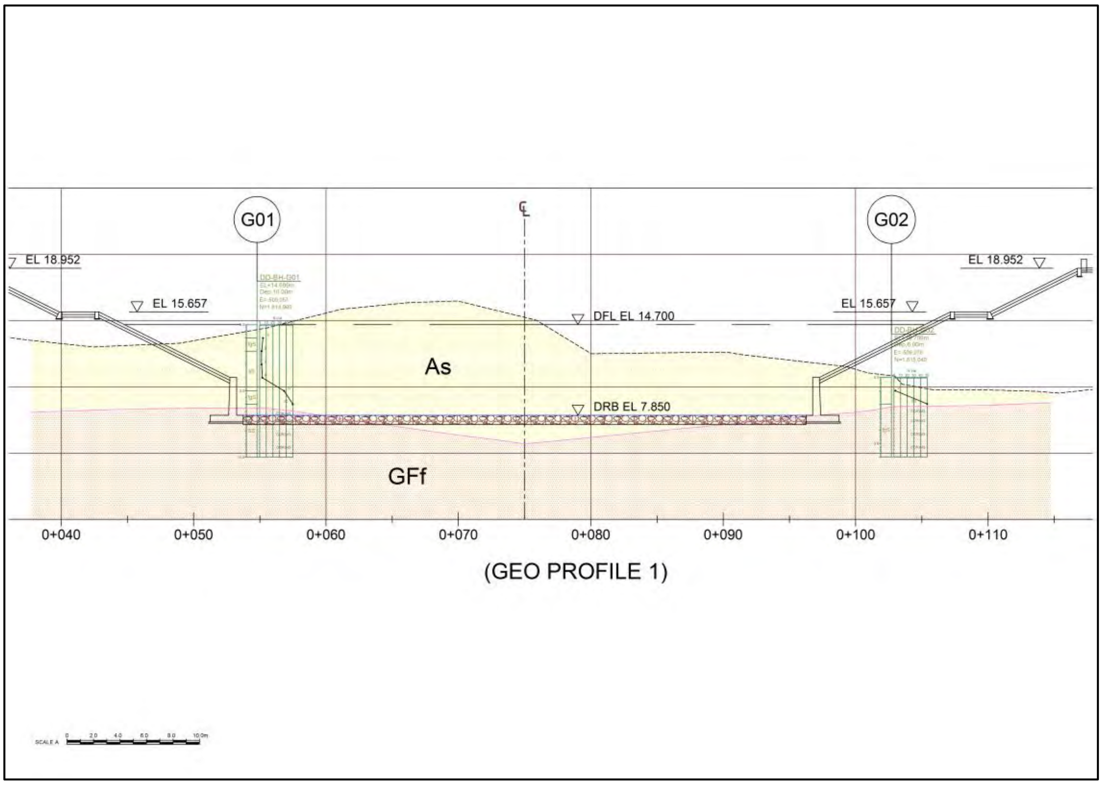
Source: Study Team

Figure6.3.7 Assumed Geological Cross Section (Weir Position)



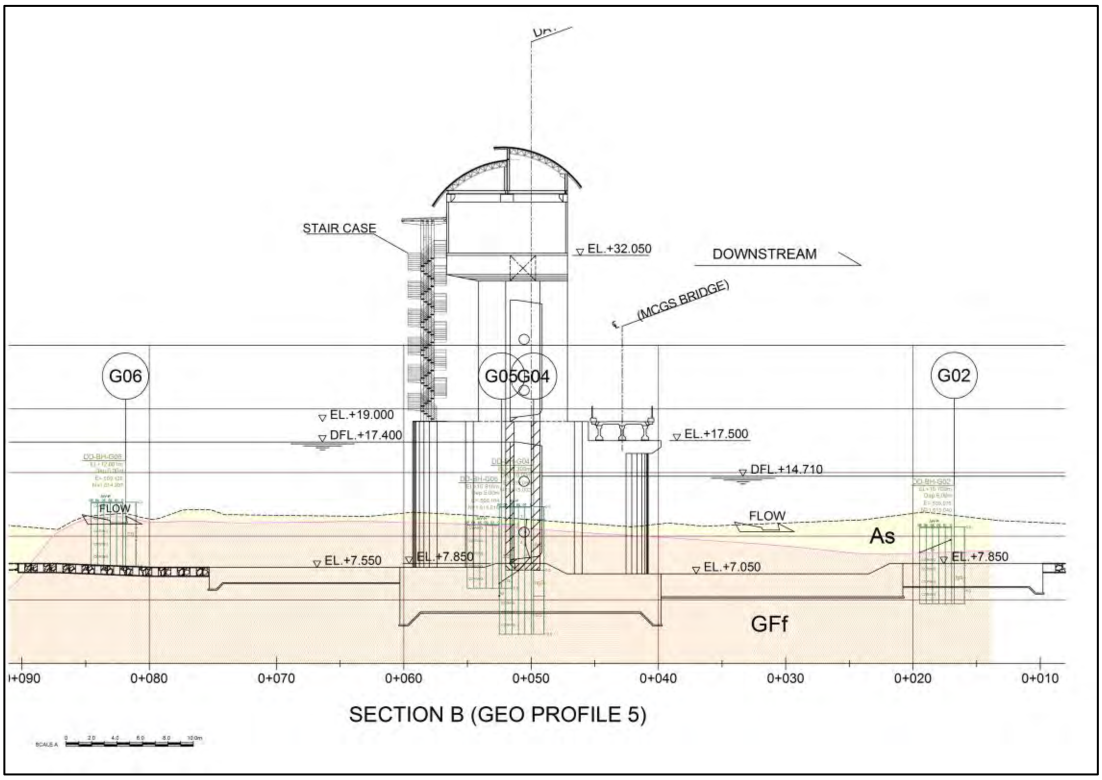
Source: Study Team

Figure6.3.8 Assumed Geological Cross Section (Upstream Side)



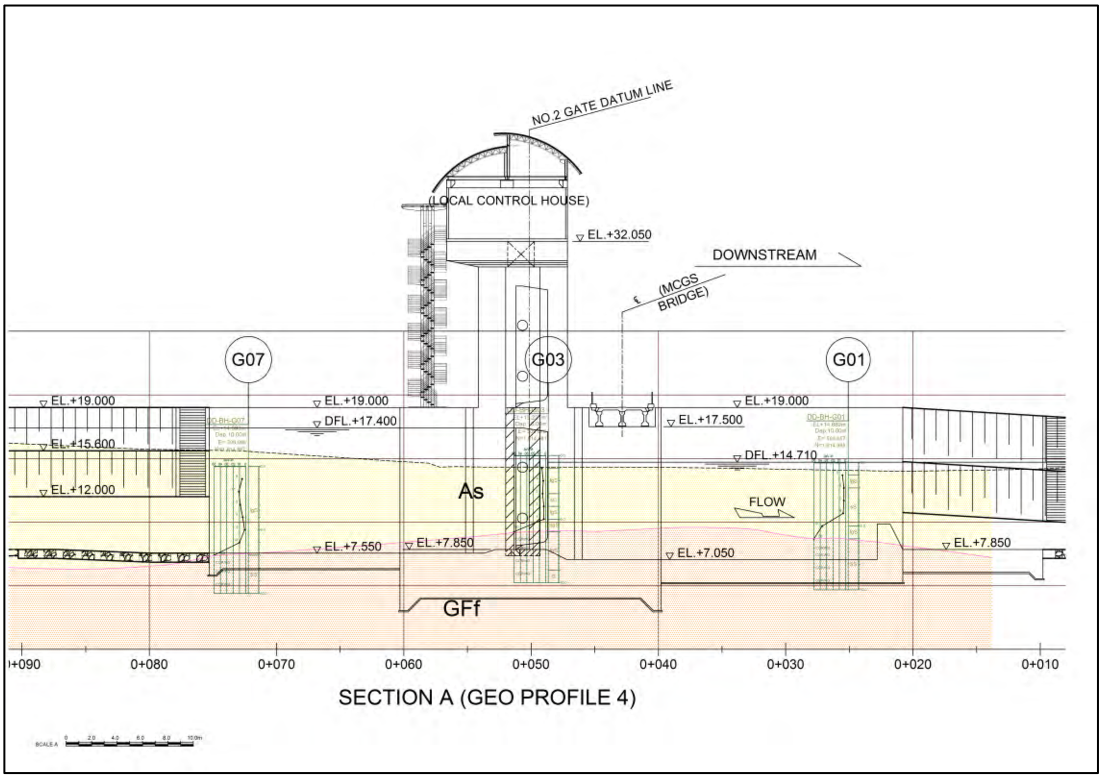
Source: Study Team

Figure6.3.9 Assumed Geological Cross Section (Downstream Side)



Source: Study Team

Figure6.3.10 Assumed Geological Cross Section (Right Bank Side)



Source: Study Team

Figure6.3.11 Assumed Geological Cross Section (Left Bank Side)

(5) Soil Properties

The soil parameters for the design are determined based on the results of the borehole logs and laboratory tests.

1) Setting Policy

Regarding the soil properties, N value, unit volume weight, and shear strength (c, ϕ) required for design are mainly set. At the time of setting each property, the stratum is divided from the borehole log, and these are categorized by the same soil layer. The property of each layer is basically set by laboratory test value.

2) Unit Weight of Soil

Test values is adopted as a principle. For the stratum without test value, it is set referring to the test value of upper and lower layers or the survey result in the vicinity, or "Road Earthwork Guideline, Guideline for embankment works", etc.

3) Shear Strength

The shear strength (c, ϕ) was set as follows.

・As the As layer is sandy soil, c is set to zero, and in accordance with the following equation, the shear strength, ϕ is set in consideration of the N value and effective overloading pressure.

$\phi = 4.8 \times \log N1 + 21,\ N1 = 170 \times N/(\sigma'v + 70)$[1]

・The GFf layer has large dispersion in the depth direction depending on the weathering state, but as average, qu= 7.0 MN/m2, and from c = qu/2, c = 3.5 MN/m2 = 3,500 kN/m2 is set.

[1] Specifications for Highway Bridges IV Substructures

4) Soil Properties

The following soil parameters have been determined based on the results of drilling carried out along the Marikina River, in order to cover approximately 8 km of the Marikina River, which is the target area for revetment design. About the retaining wall on the downstream side of MCGS, it is considered a part of revetment, and the soil properties for revetment design is used considering the consistency of the revetment design along Marikina River.

Table6.3.15 Soil Properties Used in the Design of MCGS Downstream Retaining Walls

Stratum	Soil Type	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γ t (kN/m3)	Cohesion c (kN/m2)	Internal Friction Angle ϕ (°)
As	Sandy Soil	9.9	30	30~20	9	17.5	0	30
GFf	Bed Rock	121.5	8	-	-	17	1,000	30

Source: Study Team

On the other hand, some of the borings has been intensively carried out at the MCGS site, and in the design of the MCGS main body and the breast wall, wing wall, etc., the soil properties set from these boring holes is used.

Table6.3.16 Soil Parameters Used in the Design of MCGS

Stratum	Soil Type	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γ t (kN/m3)	Cohesion c (kN/m2)	Internal Friction Angle ϕ (°)
As	Sandy Soil	5	30	30~20	9	17 (18)	0	30
GFf	Bed Rock	50	8	-	-	17 (18)	3,500	0

Source: Study Team

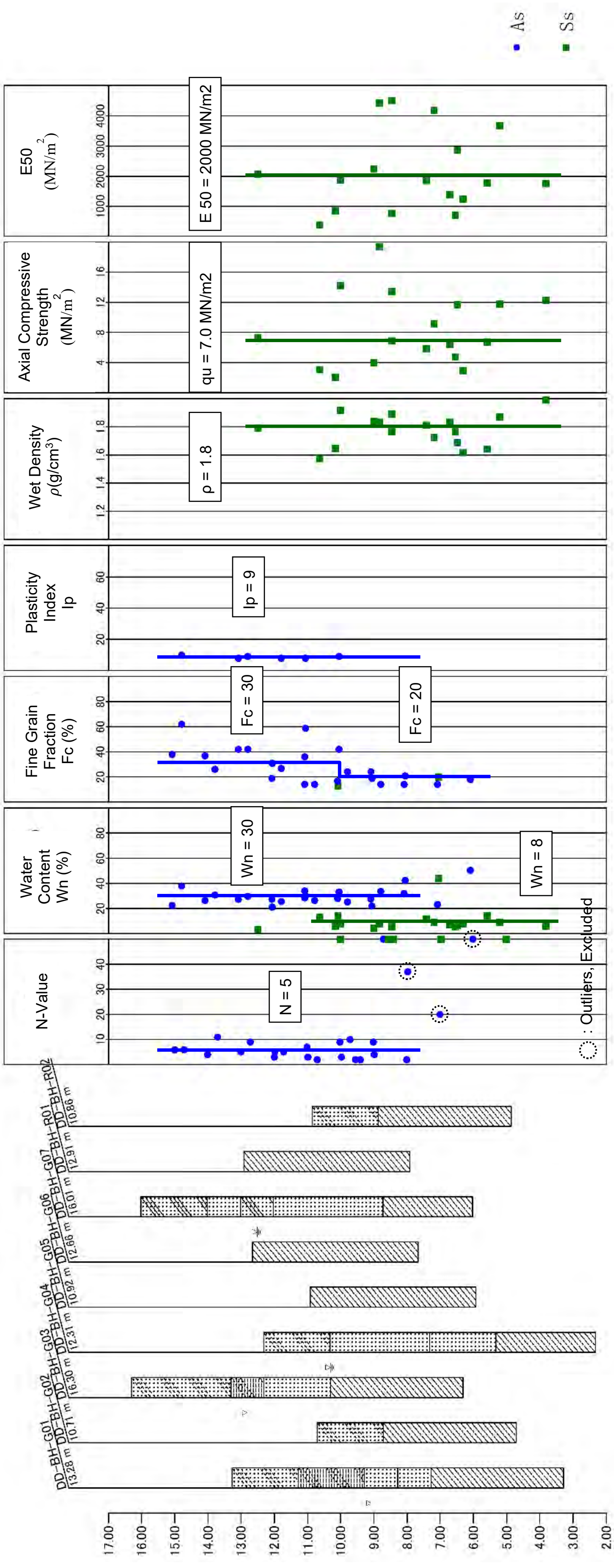


Figure6.3.12 Soil Characteristic Map At MCGS Site

Source: Study Team

(6) Classification of Stratum

The classification of stratum is determined by ground characteristic value T_G. The method indicated below is the same as the Philippine and Japanese standards.

$$T_G = 4\sum_{i-1}^{n}\frac{H_i}{V_{si}}$$

Where, Vsi: Initial Shear Elastic Wave Speed
In case of Sandy Soil Strata: $Vs=80N^{1/3}$,
In case of Cohesive Soil Strata: $Vs=100N^{1/3}$
N: Value/Number of SPT in each Stratum/Layer
i: Number of the i-the Soil Stratum

Table 6.3.17 Classification of stratum

Classification of Stratum	Ground Characteristic Value T_G (s)
Type-I	$T_G \leqq 0.2$
Type-II	$0.2 < T_G \leqq 0.6$
Type-III	$0.6 < T_G$

Source: BSDS

Table 6.3.18 show the calculation results of ground characteristic value TG in the previous boring. Based on the calculation results, the ground at the MCGS is categorized as Type I, also considering that there is the rock foundation below the subgrade of MCGS.

Table 6.3.18 Calculation of Ground Characteristic Value TG (DD-BH-G04)

No.	Stratum/ Layer	Cohesive=2 Sandy=1 Others=3	Thickness of Layer	Depth	Ave. N (SPT)	Elastic Wave Speed Vs	Hi/Vsi
			m	m		m/s	sec
1	As	1	2.00	2.00	7	153.0	0.027
2	As	1	3.00	5.00	6.7	150.6	0.012
3	As	1	2.00	7.00	35.0	261.7	0.006
4	GFf	3	3.00	10.00	50.0		
						T_G=	0.179 s

Source: Study Team

6.3.3.6 Study on the Location of MCGS

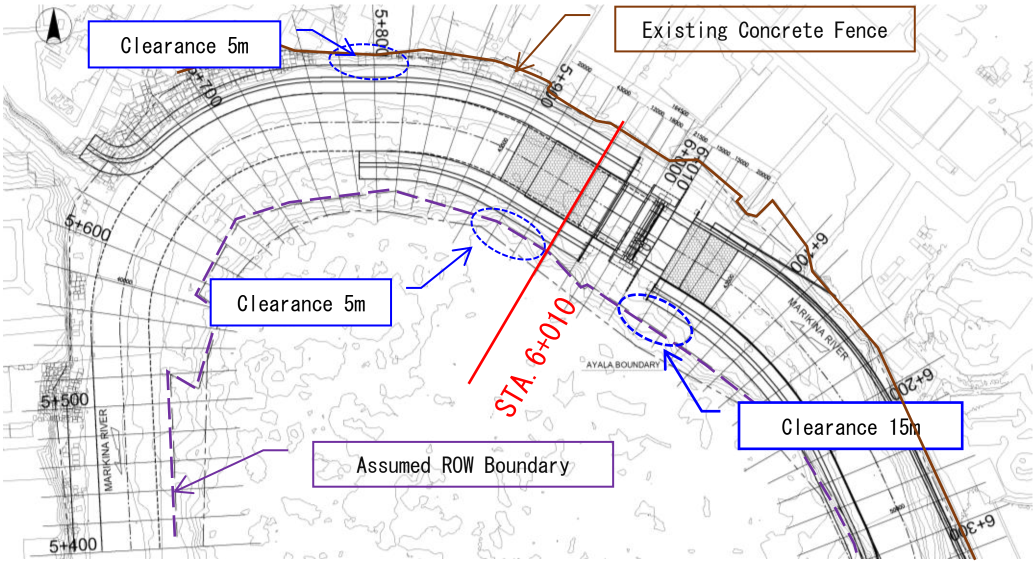
(1) Cross Sectional Location

The central position of the weir in the transverse direction of the channel shall be aligned with the center of the river.

(2) Longitudinal Direction

The location proposed in the 2015IV & V-FS report is Sta.6+050. In this case, however, the distance from the ROW boundary becomes narrow, particularly on the left bank side, and when the channel alignment is moved to the right, aprons in the downstream and upstream are located in the curve. Hence, the weir axis is set on the transverse direction of Sta.6+010.

(3) Location on the Plan



Source: Study Team

Figure 6.3.13 Location of MCGS

6.3.3.7 Study on the Basic Structural Specifications

(1) Study on Type of Weir and Gate

1) Study on Type of Weir

The main purpose of the MCGS, the subject of this design, is discharge regulation. Considering this purpose, the applicable type of weir is selected. In the selection, alternatives are listed with reference to the type of floodgate in the weir as shown in "Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]" (see **Table 6.3.19**).

Table 6.3.19 Type, Location and Purpose of Floodgate

Purpose		Type of Facility (Typical)	Purpose	Type of Floodgate (Typical)
Weir	Diversion / Tide Control / Water Intake	Spillway Gate	Maintain Water Level, Regulate Discharge	Roller, Shell Structure with Roller, Double-Deck Roller, Bear-trap
		Discharge Control Gate	Maintain Water Level, Regulate Discharge	Double Deck Roller, Shell Structure with Roller, Bear-trap
		Flushing Gate	Maintain Water Level, Flush Sediment	Roller, Shell Structure with Roller
		Rock Gate	Maintain Water Level, Shipway	Roller, Shell Structure with Roller
		Fish Course	Maintain Water Level, Regulate Discharge Fish Ascending	Bear-trap, Sector, Lift, Slide
Gate for Repair		Repair	Maintain water level during repair of Gates	Floating, Strut Support, Pier Support, Stop Log, Shield

Note: Translated by Study Team from Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

With reference to **Table 6.3.19** the type of weir can be classified as follows based on with/without gate.

1. Group A: (Fixed Weir) without gate
2. Group B: (Movable Weir) Lift-up Gate Weir
3. Group C: (Movable Weir) Steel Bear-trap Gate Weir
4. Group D: (Movable Weir) Rubber Bear-trap Gate Weir
5. Group E: (Movable Weir) Hinge Type Gate Weir

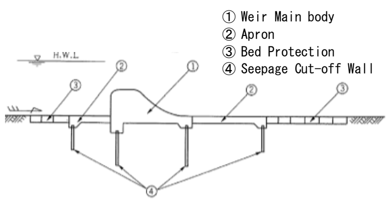
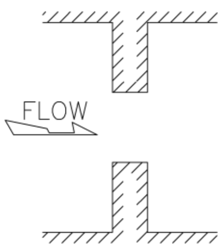
From the above-mentioned groups, the types applicable to this facility is selected considering the site conditions and constraints shown below.

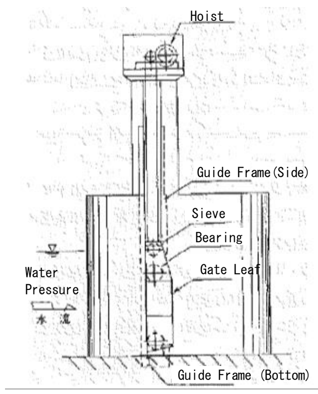
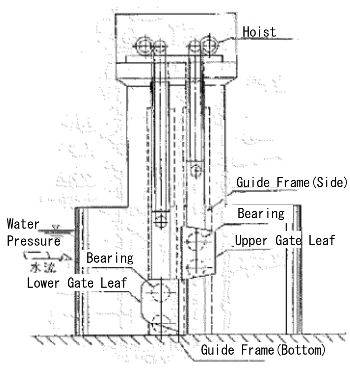
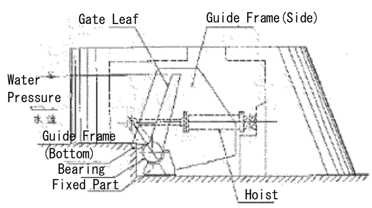
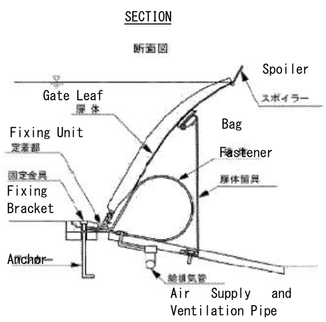
- Function for Regulating Flow: It is possible to flow the discharge 500m^3/s reliably to downstream of the Marikina River at the proposed design scale flood.
- Function for passage of boats in ordinary condition: Ferry navigation is possible under no flood conditions (including the rainy season).
- Propriety of other flow systems than underflow: Concerning occurrence of vibration due to underflow
- Resistance to Local Climate Condition: High temperature and humidity compared to Japan, and solar radiation heat all year round
- Availability of Land: The site is limited on both sides and the facility is as compact as possible.

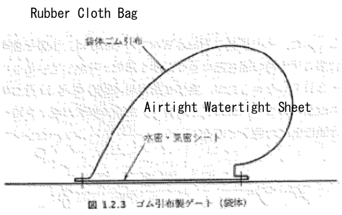
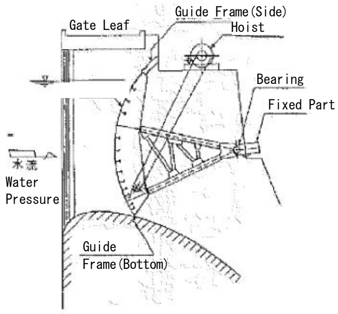
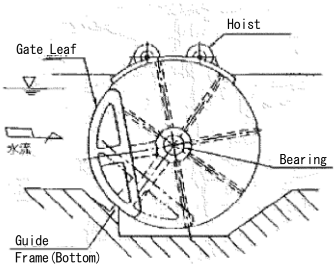
Table 6.3.20 shows a comparison of types of weir. In accordance with **Table 6.3.20**, it is possible to apply the group types classified into Group B and Group E to this facility. From these applicable types, gate type will be chosen. The other three groups are not recommended due to the following issues:

- Group A (Fixed Weir without gate): Ferry Navigation is difficult
- Group C (Movable Weir / Steel Bear-trap Gate Weir): Risk of dysfunction due to sedimentation
- Group D (Movable Weir / Rubber Bear-trap Gate Weir): Not Applicable due to the local climate condition

Table 6.3.20 Comparison of Types of Weir

Group	No.	Type of Weir/Gate	Description of Structure		Reason for Inapplicable	Evaluation
A	1	Fixed Weir (w/o gate)		Method to make the sill higher than the planned riverbed and control the downstream flow at the proposed discharge by the overflow flow	Not applicable due to the prevention of Ferry Navigation also interferes with drainage from the Lake Laguna via the Manggahan drainage channel after the end of the flood	×
A	2	Contracted Channel (w/o Gate)	 PLAN	Method to control the downstream discharge by contracting channel width without raising the riverbed	As the flow rate increases, the flow velocity also increases, and the ferry navigation is affected during normal rainy season.	×

Group	No.	Type of Weir/Gate	Description of Structure		Reason for Inapplicable	Evaluation
B	3	Fixed Wheel Roller Gate		During flood, the door is lifted up above DFL. There is a plate girder structure and a shell structure according to the difference in the gate structure. Overflow type is also possible by set the gate height below DFL.	Nothing	○
B	4	Double Deck Fixed Wheel Roller Gate		It has two stages of gates that can be operated separately, and it is possible to adjust the flow rate with overflow by the adjustment of upper gate.	Nothing	○
C	5	Steel Bear-trap Gate Weir		A type in which the gate leaf rises or falls by a horizontal rotation shaft fixed to the bottom of the channel. It is classified into a torque shaft type, a horizontal main girder type, and a fish flank type according to the span length and the maintenance management.	From the past survey results and interviews with a contractor, the riverbed around the MCGS tends to be deposited, the rotation axis is near the riverbed, and it is always submerged, ensuring the reliability of operation is a problem and not recommended.	×
D	6	SR integrated Bear-trap Gate		The gate body is operated by pressing or removing air or water from the bag-like synthetic rubber-made bag body.	Under severe heat radiation conditions, it is not recommended to use a rubber-bladder bag in the Philippines. Proper maintenance is required so that the rubber bag body does not deteriorate, but it is difficult for the local engineer alone to deal with it.	×

Group	No.	Type of Weir/Gate	Description of Structure		Reason for Inapplicable	Evaluation
D	7	Rubber Bear-trap Gate		The gate body is contoured by pressing or removing air or water into the bag-like synthetic rubber cloth bag body.	There is an issue similar to the above. Although it is possible to adjust the height of the door by controlling the amount of air or water pressed into the bag, it is possible that V-notches may occur, making it difficult to finely adjust the flow rate.	×
E	8	Radial Gate		A gate that opens and closes by rotating the gate around a horizontal rotation axis. Overflow type is also possible by set the gate height below DFL.	nothing	○
E	9	Rising Sector Gate		The door is opened and closed by rotating the end disk. The overflow type is possible, and the vibration can be suppressed even in the underflow type.	Nothing	○

Legend：○...Alternative to be Studied、×...Not Applicable for this Facility
Note: Prepared by Study Team based on the Reference Documents

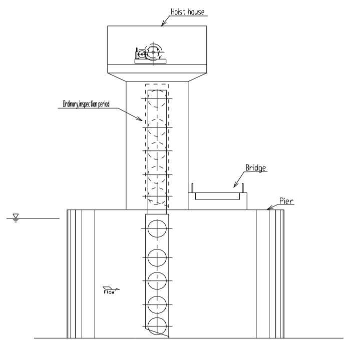
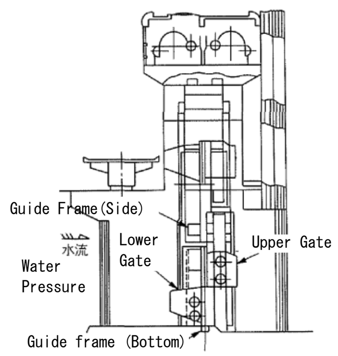
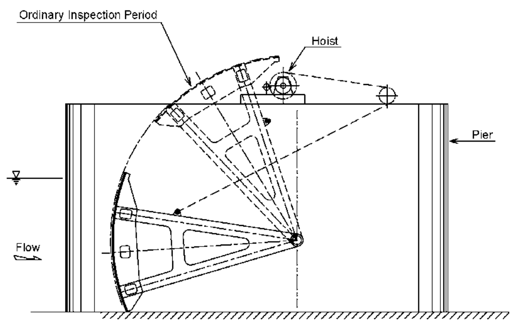
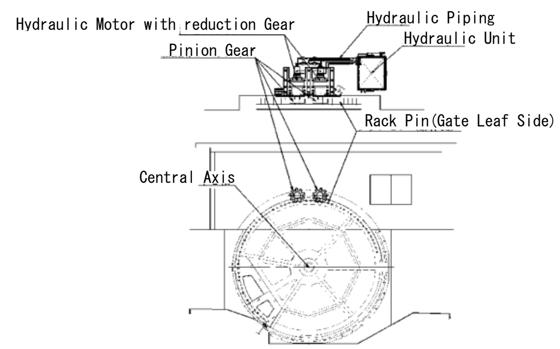
2) Secondary Selection of Type of Gates

Based on the comparison shown in **Table 6.3.20**, the following 4 alternatives are chosen for secondary selection of type of gate.

- Alternative 1：Fixed Wheel Roller Gate
- Alternative 2：Double Deck Fixed Wheel Roller Gate
- Alternative 3：Radial Gate
- Alternative 4：Rising Sector Gate

Table 6.3.21 shows a comparison of the 4 alternatives chosen in the abovementioned primary selection. In addition to the ease of maintenance, reliability, and economics, it also takes into consideration that local technicians have sufficient knowledge of operation and maintenance, since same type was adopted in the weir and floodgate nearby. "Alternative 1: Fixed Wheel Roller Gate" is selected.

Table 6.3.21 Comparison of Gate Type

Item	Alternative 1：Fixed Wheel Roller Gate		Alternative 2：Double Deck Fixed Wheel Roller Gate		Alternative 3：Radial Gate		Alternative 4：Rising Sector Gate	
Figure	 Source：Final Report of 2002 PMRCIP-I		 Source: Design of Weir (Japan Dam Engineering Center)		 Source：Final Report of 2002 PMRCIP-I		 Source：Manufacturer	
General	・ Structure consisting of fixed roller, girder and skin plate attached to the gate leaf The operation of the gate leaf is performed by winding up the wire rope or chain		・ The roller gate on the left is double decked (The upper gate leaf is also flap type.)		・ Type that has an arm next to the gate leaf and consists of skin plate, main girder, sub girder, and beam receiving vertical member ・ Gate opening and closing is done by vertical lift and wire rope ・ A hoist id installed on the top of the pole		・ Both ends of the gate leaf with shell structure are disks, and opening and closing is performed by rotating this disk	
Gate Position	・ Normal and during inspection: Above the water surface ・ During Flood: Below or above the water surface		・ Normal and during inspection: Above the water surface ・ During Flood: Below or above the water surface		Normal and during inspection: Above the water surface During Flood: Below or above the water surface		Normal and during inspection: Above the water surface During Flood: Below the water surface	
Structure	・ Overflow can be dealt with by attaching a spoiler, and there is no structural weakness in particular	○	・ Same as the left	○	・ The pedestal is structurally weak against overflow	△	・ There is no need for spoiler like Alternative 1, and there is no week point in mechanical structure ・ Since the riverbed is lowered, sedimentation occurs easily	△
Maintenance	・ Inspection of each part and replacement of water sealing rubber are easy and maintenance is relatively easy ・ Although many parts are required, few parts of hoist need to be replaced. However, wire rope replacement and grease application on wire rope are necessary	○	・ The basics are the same as the left, However, there are many devices to be maintained and managed due the arrangement of devices and devices in a plane. In addition, the work efficiency is inferior due to closeness of the equipment. There are also many lubrications and refueling points.	△	・ Inspection of each part and replacement of water sealing rubber are easy and maintenance is relatively easy ・ Inspection of the hoist is as easy as fixed wheel type gate	○	・ Both side disks are always submerged, and inspection and maintenance are complicated compared to the fixed wheel roller gate. ・ It is more difficult to remove sediment	△
Reliability	・ If the span is about 15m, the gate leaf will be lowered by its own weight. Hence, the gate can be lowered even in floods, and the operation reliability is high. However, in this case it is impossible to lower by its own weight ・ A lot of sluiceways and floodgate adopted this type aside from weir ・ It is easy to observe the opening and closing status of the gate even from a distance	◎	・ Same as the left, however there is a possibility that reliable gate operation cannot be performed due to the complex gate operation and the lot of facilities to maintain.	○	・ If the span is about 15m, the gate leaf will be lowered by its own weight and the gate can be lowered even in floods, and the operation reliability is high. However, in this case, it is impossible to lower by its own weight ・ There are few cases in weir ・ It is difficult to observe the opening and closing status of the gate from a distance	△	・ Because of the single deck gate, operation is easy and there is little risk of malfunction. ・ Compared with the fixed wheel roller gate, there is less samples cases. Hence the reliability is inferior.	△
Discharge Regulation	・ With the discharge by underflow, the precise flow rate adjustment is possible, however there is a risk that the gate may vibrate depending on the water level conditions of the upstream and downstream ・ If one-sided gate is fully opened during flood by the operation method, the flood control function required for this facility can be satisfied with dealing with the issue of gate vibration.	○	・ By lowering the lower gate and adjusting the opening of the upper gate, it is possible to precisely adjust the flow rate and to cope with the problem of gate vibration due to the underflow	◎	・ Same as Alternative 1	○	・ Since the flow rate can be regulated by the overflow and the falling water is not separated, the vibration of the gate vibration does not occur even without the spoiler.	◎
Landscape	・ As the structure is significantly higher than the riverbank, it is the least scenic alternative.	△	・ Since the column can be lowered than the first plan, it is slightly better on the landscape side	○	・ Since this type does not have tall column and large control house, it is superior in landscape to fixed wheel roller gate.	◎	・ Since this type does not have tall column, it is superior in landscape to fixed wheel roller gate.	◎
Economic	・ Gate Facility : ***** PHP（1.00） ・ Most Economic	◎	・ Gate Facility : ***** PHP（1.40） ・ Most Expensive due to the Complexed Structure	△	・ Gate Facility : ***** PHP（1.15） ・ Middle in Economic	○	・ Gate Facility : ***** PHP（1.40） ・ Middle in Economic	○
Sample in Philippine	・ NHCS(Floodgate)、Rosario Weir etc.	○	・ Never Adopted	△	・ NHCS (Rock Gate)	○	・ Never Adopted	△
Evaluation	Although the column stands out, it is superior in terms of maintenance, management, operation and economy. This is the same type as NHCS and Rosario weir managed by MMDA, and local engineers have enough knowledge on operation and maintenance.		The problem is that the operation is complicated and there are many maintenance and management facilities Most expensive option		It has the same flow rate adjustment function as fixed wheel roller gate, has enough reliability, however, is week against overflow. Lowering by its own weight is impossible and there is not the actual sample adopting this type in weirs It is inferior in terms of economy.		Issues remain in terms of maintenance and performance Unfavorable in economic terms	
	1		4		2		3	

Note: Cost is not presented due to the prior released version. *Legend：◎...Better; ○...No Problem; △...There are issues to be solved* *Source: Study Team*

(2) Invert Elevation

The invert elevation is set to the design riverbed height and the design riverbed of EL. 7.85m at STA.6+010 is adopted.

(3) Position of Gate

The gate position is set at the weir axis, which is STA. 6+010.

(4) Gate Height

At the basic design, in order to reduce the height of the column and the size of the gate leaf, the height of the gate is set at 9.55 m to make the top elevation at the DFL in the fully closed state.

(5) Span and Span Allocation

1) Study Condition

When setting the span and its number, "Cabinet Order Concerning Structural Standards for River Administration Facilities, etc. (hereinafter called the "Structural Cabinet Order")" is referred. The study conditions are as follows:

- ✓ Proposed Discharge: 500m^3/s.
- ✓ At least one gate will be used for ferry navigation during ordinary condition.
- ✓ The span and number correspond to the "mountainous constriction part" in the Structural Cabinet Order from the following reasons:
  - ➢ There is no embankment at the weir location and the ground behind is high.
  - ➢ Even if the MCGS is installed, it does not always close the gate (see **Table 6.3.22** for details). Therefore, when the target flood flows, no impact on flood control in both the upstream and downstream is expected.
- ✓ Structural Cabinet Order mentions that one of the spans can be narrowed when the span is going to be used also for navigation and flushing under the above-mentioned situation. In addition, the cabinet order specifies the minimum span width in accordance with the value of discharge to avoid driftwoods and garbage blocking the water way.
- ✓ As determined in the 2002 PMRCIP-I, secure 40m water surface width in ordinary time.

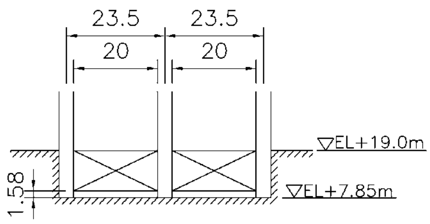
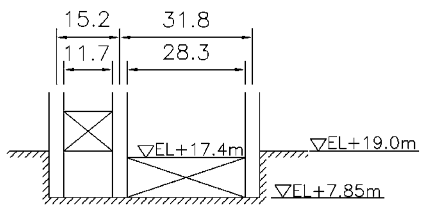
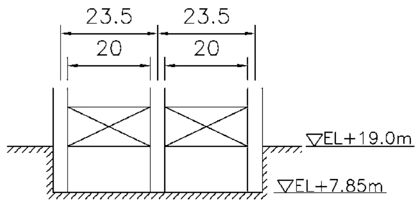
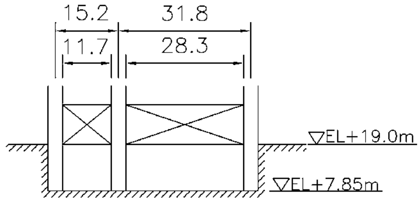
2) Study on Span and Span Allocation

Considering the above-mentioned conditions, the following Alternative 1 and Alternative 2 are compared. As an additional alternative to Alternative 2, there is also an alternative to use three spans, such as 16.1m+16.1m+14.8m; however, the Structural Cabinet Order which specifies that the span length of the wider span gate is more than 20m (flow rate is more than 500m^3/s and below 2,000m^3/s) is not complied. Therefore, 2 spans are needed.

1. Alternative 1 : 23.5m+23.5m (Recommendation in 2002 PMRCIP-I)
2. Alternative 2 : 15.2m+31.8m

Table 6.3.22 shows the comparison of span length. Since there is no concern about gate vibration due to underflow and flexible operation, Alternative 2 (15.2m+31.8m) is adopted. In addition, the clear span (11.3m) of the narrower gate is determined by numerical calculation as shown in the following table, in consideration of the hydraulic model test results.

Table 6.3.22 Comparison of Span Allocation

Item	Alternative 1 : 23.5m + 23.5m	Alternative 2 : 15.2m + 31.8m
Figure (Ordinary Condition)		
(During Flood)		
Clear Span	・ 20m + 20m	・ 28.7m + 11.3m
Type of Gate	・ Fixed Wheel Roller Gate (Single Gate Leaf)	
Blocking Rate	・ 8.0% (= 3.5m/43.5m) * Both sides of the weir become invalid	
General	・ Proposal in 2002 PMRCIP-I ・ It satisfies the span length specified in the Structural Cabinet Order, and secure a water surface width of 40 m of the Marikina River in ordinary time between total net lengths	・ This alternative narrows the s one side of two gates to regulate the flood discharge. ・ Referring to "mountainous constriction part" stated in Structural Cabinet Order, the minimum width of the narrower span is set to 12.5 m to avoid driftwoods and garbage blocking the water way.(Since 15.2 m > 12.5 m, this condition is satisfied.) ・ Securing the water surface width 40 m of the Marikina River in ordinary time between total net lengths
Operation in Ordinary Time	・ Both 2 gates are opened	・ Same as the left
Operation During Flood	・ Both 2 gates are lowered at the middle level depending on the discharge	・ Only the narrower span gate is going to be fully opened and other side is fully closed.
Discharge Method	・ Underflow below the gate leaf	・ Overflow on the fixed portion of the weir
Structure	・ There is a concern that vibration may occur on the gate leaf due to underflow discharge, which may adversely affect the structure and surrounding environment. However, it is difficult to evaluate quantitatively in hydraulic model experiments etc. △	・ Nothing to Concern ○
Maintenance	・ If vibration of the gate occurs and it becomes a problem after completion, it may be difficult for the local facility manager alone to deal with. △	・ Nothing to Concern ○
Evaluation	It is difficult to quantitatively evaluate the gate vibration due to underflow discharge and to cope with it after its occurrence.	There is no concern about gate vibration like Alternative 1, and flexibility is also provided in its gate operation.
		Recommended

Source: Study Team

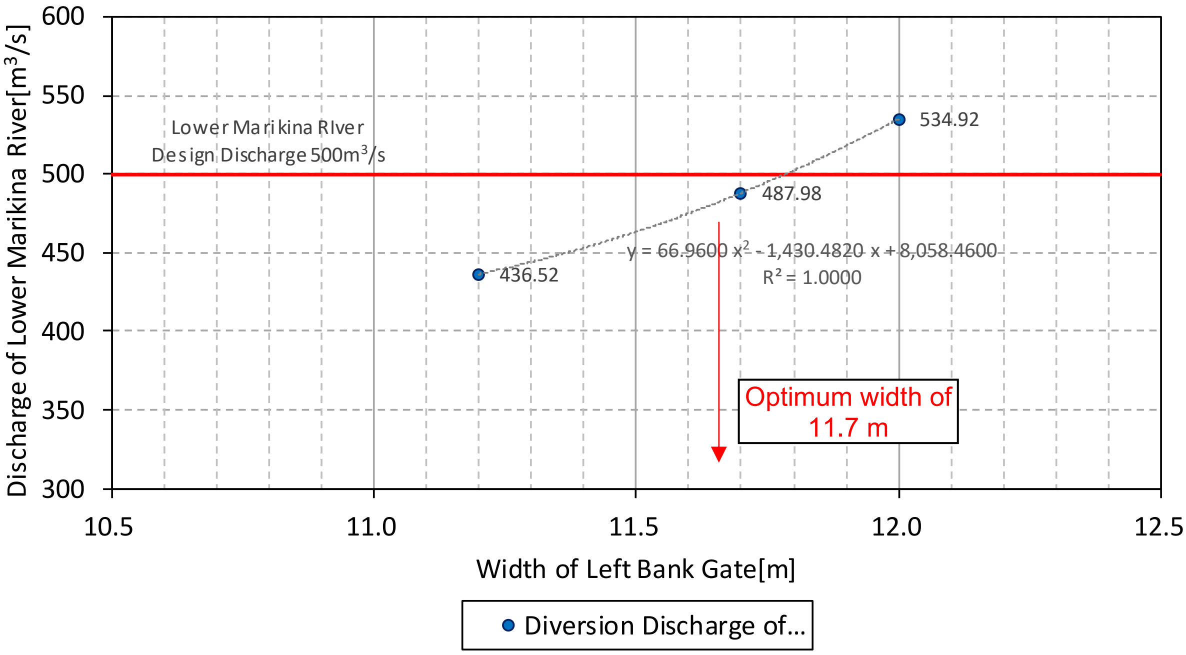
＜Determination of Clear Span of Narrower Span Gate＞

The channel width is set by hydraulic model experiments. (See the **Chapter 8 Hydraulic Model Experiments** for details.)

The clear span of the narrower gate is set in 3 ways (11.2 meters, 11.7 meters, 12.0 meters), and the relationship between the clear span and the flow rate in Lower Marikina River (see **Figure6.3.14**) and the relationship between the clear span and the water level in the upstream of MCGS(see **Figure6.3.15**) is obtained. The experimental results show that the flow rate of the Lower Marikina River is 488 m^3/s (Proposed Discharge is 500 m^3/s), when the clear span is 11.7 m. In addition, the water level in the upstream of MCGS is below DFL in all cases.

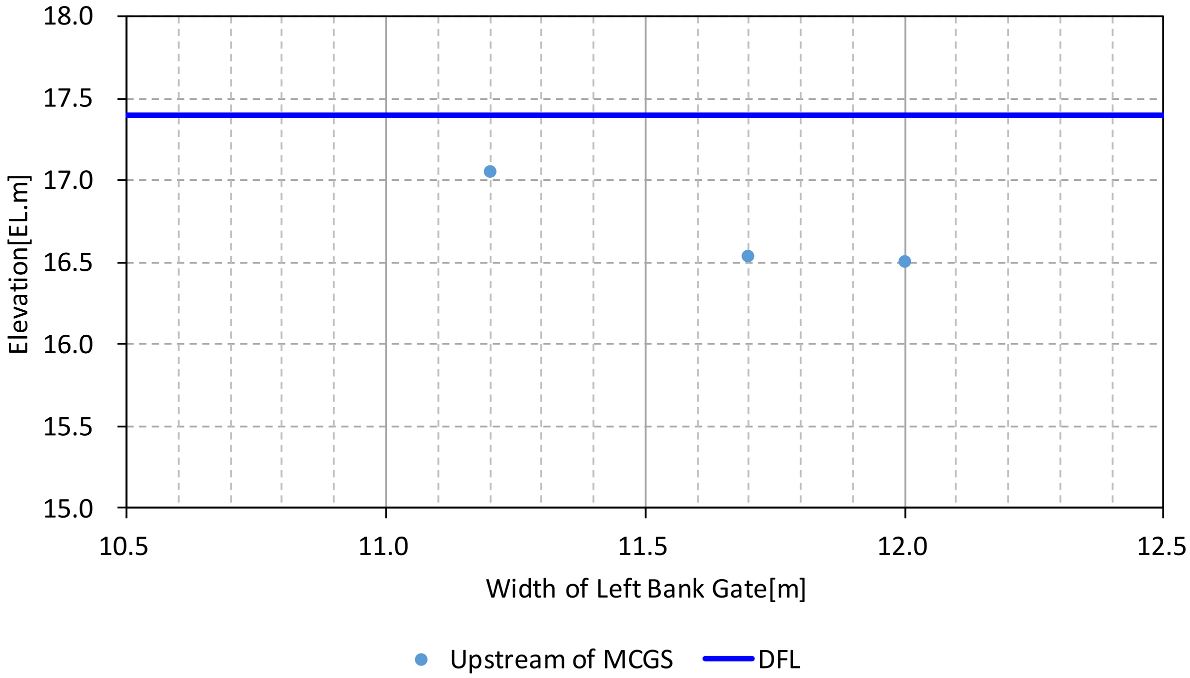
Based on the relationship between the clear span and the flow rate of Lower Marikina River obtained in **Figure6.3.14**, the estimated flow rate of Lower Marikina River with the clear span of 11.8 m exceeds 500 m^3/s, but the optimum gate span length is set to 11.7 m considering the following items.

- The function of the MCGS is to regulate the flow rate of the Lower Marikina River less than 500 m^3/s during the design flood.
- In this hydraulic model experiment, square weir is used as a method of flow rate observation. The flow rate is calculated by the formula of Itaya and Teshima (JIS 8302), and the estimated flow rate by this formula includes a prediction error of 1.4%. Since 1.4% of 500 m3/s is 7 m^3/s, the target flow rate is 490 m^3/s in consideration of safety.



Source: Survey team

Figure6.3.14 Relationships between the Width of Narrower span gate and the Discharge of Lower Marikina River



Source: Survey team

Figure6.3.15 Relationships between the Width of Narrower span gate and the Water Level Upstream of MCGS

3) Position of Narrower Span Gate

At the location of MCGS, from the shape of the river channel, the waterway tends to swerve to the right. In the case of span allocation proposed in **Table 6.3.22**, the Narrower Span Gate is proposed to be placed on the left side for the following reasons:

- ✓ Since the waterway tends to swerve to the right, the riverbed in the left bank at the MCGS site is prone to sediment accumulation. Arranging the narrower span gate on the left bank side can suppress sediment deposition by guiding the flood flow to the left side.
- ✓ The wider span gate needs to be fully closed during flood; hence, making the wider span side waterway in ordinary time suppress sediment deposition as much as possible.
- ✓ If the method such as closing the large span gate during flood is adopted, the mainstream will tend to the left side. Hence, the suppression effect of erosion and scour on the right side can also be expected.

4) Necessity of Narrower Span Gate

In the case of the span allocation selected in **Table 6.3.22,** the narrower span gate is fully opened and not needed during flood with the scale of less than the target return period. In addition, even in the case of excessive flood, when based on no operation during excessive flood and since the narrower span gate may be kept open, there is no need for gate operation.

However, considering the following conditions such that flooding is occurring only in the downstream of the proposed MCGS, it is recommended to install the gate even in the narrower span side to enable flexible gate operation.

When the water level of Laguna Lake is low and flood occurs in the downstream of MCGS; for example, water level of Pasig River rise due to the flood occurring in Sun Juan River, it is necessary to reduce the flow in the Marikina River side with MCGS and to increase the discharge to the Manggahan Floodway to Laguna Lake. Closing the narrower span gate can be considered.

There is also an option to cope with the stop log; however, it is possible to respond more quickly by lowering the gate when manpower and available time resources are limited. In the case of Manila in the Philippines in particular, there is a strong concern about traffic congestion and flooding of roads. These will significantly delay the arrival of materials, equipment and necessary labor for work. In addition, there is a disadvantage that the stop log is only fully closed, and the flow rate cannot be adjusted.

(6) Study on Local Control House

1) Study on Design Conditions and Specifications

Layout planning of local control house shall be based on size of equipment and clearance around equipment. Required clearance for inspection and maintenance works are provided by type of equipment as follows.

- ✓ Clearance around hoisting machine: 80 cm
- ✓ Clearance around other mechanical equipment: 60 cm
- ✓ Clearance in front of control panel: 100 cm
- ✓ Clearance between control panels: 120 cm
- ✓ Clearance at side or back of control panels: 20 cm

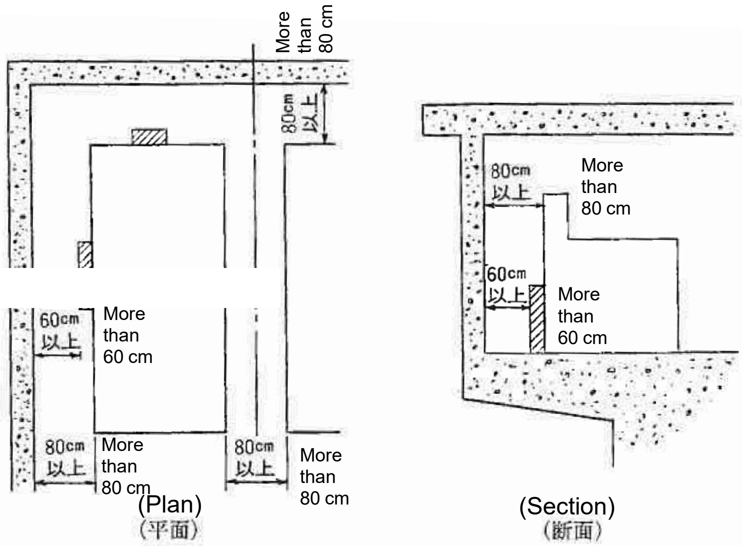


図4.4.2-1 開閉装置の端部から壁までの間隔

Figure 4.4.2-1 Clearance between the Edge of Hoist and Wall

（2） 操作盤と壁および他機器との間隔

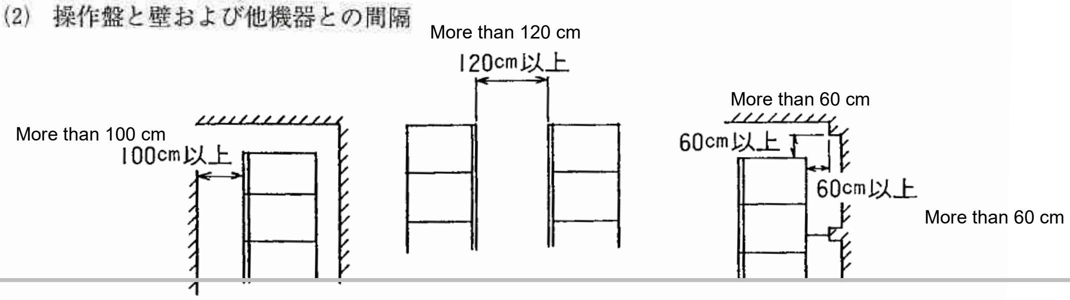


図4.4.2-2 操作盤と壁および他機器との間隔

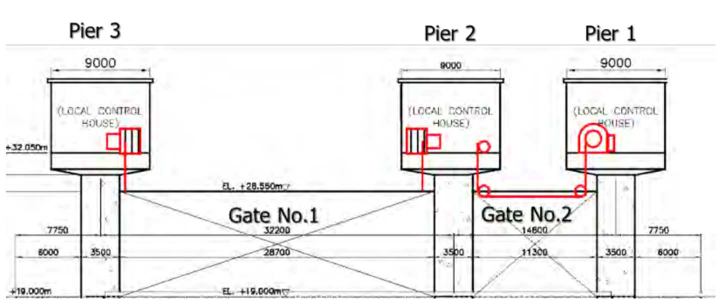
Clearance between the Panel and Wall/Others

Source: Technical Standards for Dam and Weir Facilities (Draft), Explanatory Book and Manual, February 23

Figure 6.3.16 Required Clearance

Major equipment installed in the local control house are listed in **Table 6.3.23**. Generator sets and remote controlling system equipment including telecommunication devices are installed in a separate building on the ground besides the flood gate.

Table 6.3.23 Major equipment installed in MCGS local control house

Location	Equipment	Remarks	Conceptual diagram of equipment configuration
Pier.1 (end post)	Winch Drum	For No. 2 gate	
	Motor	For No. 2 gate	
	Control Panel	For No. 2 gate	
Pier2 (central pillar)	Sheave	For No. 2 gate	
	Winch Drum	For No. 1 gate	
	Motor	For No. 1 gate	
Pier.3 (end post)	Winch Drum	For No. 1 gate	
	Motor	For No. 1 gate	
	Control Panel	For No. 1 gate	

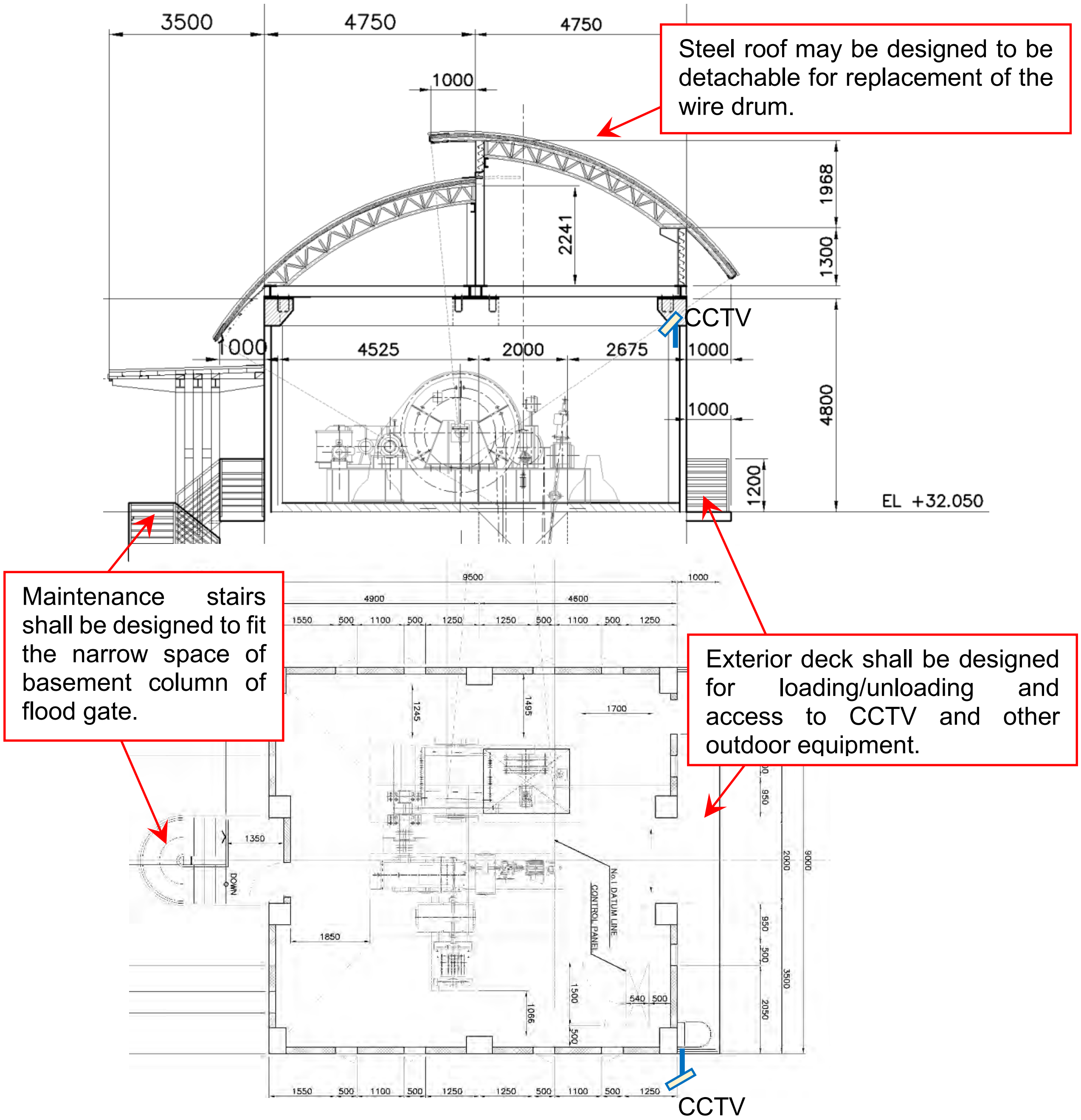
Source: Study Team

In addition, the following conditions shall be considered for ease of maintenance.

- ✓ Steel slide doors and exterior decks are provided for loading and unloading of heavy equipment by crane from the ground.
- ✓ Outdoor spotlights, ITV cameras and speakers shall be accessible from the exterior deck.
- ✓ Steel roof may be designed to be detachable for replacement of the wire drum, which weighs more than 5 tons, instead of installation of roof mounted hoisting crane in the local control house.
- ✓ The ceiling height shall be decided in consideration of necessary vertical clearance for inspection and maintenance of winch drum.
- ✓ The maintenance stairs may not be shall comply the requirement by NCBP comply with requirement specified in the local building standards.

2) Layout Plan and Cross Section of Local Control House

Based on the conditions summarized in the previous page, the plan and cross section of the local control house are shown below.



Source: Study Team

Figure 6.3.17 Plan and Section of MCGS Local Control House

(7) Study on Maintenance Bridge

1) General

With regard to the maintenance bridge, the required width and girder will be studied considering the following conditions:

- ✓ It will not be opened to public
- ✓ Necessity of workspace such as setting of stop log

Since a staircase is to be installed adjacent to maintenance bridge and each column, the installation of a passage (control bridge) connecting the operating decks is not considered. Installing control bridges between operating decks with only one staircase is an option such as the case of NHCS. However, if it is necessary to pass between operating decks under a strong wind and rain condition, it may be unsafe for the operators to pass. To install stairs beside each column instead of the operating bridge is applied.

2) Design Criteria

The bridge and ancillary structures of the bridge in this project will be designed in accordance with the following standards.

- Design Guidelines, Criteria and Standards (DGCS): Volume 5 – Bridges Design
- Design Guidelines, Criteria and Standards (DGCS): Volume 4 – Highway Design
- Design Guidelines, Criteria and Standards (DGCS): Volume 3 – Water Engineering Projects
- DPWH LRFD Bridge Seismic Design Specifications (BSDS)
- DPWH Standard Specification
- Philippine National Standard (PNS)
- American Association of State Highway and Transportation Officials (AASHTO)
- National Structural Code of the Philippines: Volume 1
- National Structural Code of the Philippines: Volume 2

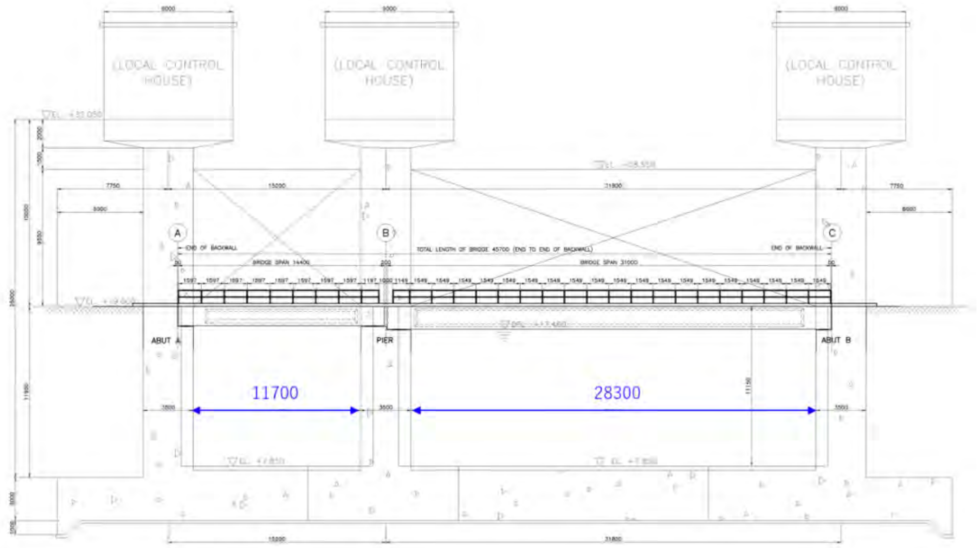
3) Design Conditions

(a) River Condition

(i) Width of Weir

The position of the weir piers and the clear span length that determine the bridge length are as follows.

L1= 11.700m, L2= 28.300m



Source: Study Team

Figure 6.3.18 Position of the Pier and Clear Span of MCGS

(ii) Elevation of Road Surface

The elevation of the road surface shall be the same as the height of the embankment road (EL. +19.000).

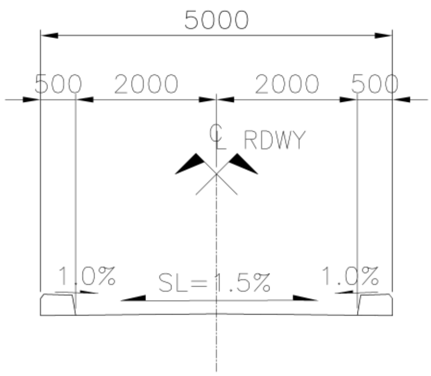
(b) Road Condition

(i) Road Classification

This bridge is a maintenance road for the purpose of access to the control room of MCGS and maintenance of the weir.

(ii) Cross Sections

- Total Road Width : 5.0m
- Effective Road Width : 4.0m
- Number of Lanes : 1 Lane



Source: Study Team

Figure 6.3.19 Cross Sections of MCGS

(iii) Elements of Alignment

- Horizontal Alignment : R=∞
- Vertical Alignment : Level (i = 0.00%)
- Proposed Elevation of Road Surface : EL. +19.000
- Crossfall l: 1.5%
- Skew : None (90° 00’00”)

(c) Bridge Condition

(i) Bridge Length

L1= 14.55m (Short span), L2= 31.15m (Long span)

(ii) Cross Sections

W= 0.500m (Curb) + 4.000m (Carriageway) + 0.500m (Curb) = 5.000m

(iii) Pavement

Asphalt concrete pavement (ACP), 50mm

(d) Loads Condition

(i) Dead Load

The unit weight (kN / m3) of the material is as follows.

Table 6.3.24 Unit Weight of Materials

Materials	Unit Weight	Unit
Concrete: Reinforced /Pre-stressed	24	kN/m³
Concrete: Plain	23.5	kN/m³
Mortar	21	kN/m³
Structural Steel	77	kN/m³
Cast Iron	71	kN/m³
Stone Masonry	22	kN/m³
Timber	8	kN/m³
Water	9.8	kN/m³
Soil : dry (not disturbed)	Ref. **Subsection 9.6.1.1**	kN/m³
: wet (compacted)		kN/m³
: saturated		kN/m³
sand／gravel (compacted)	19	kN/m³

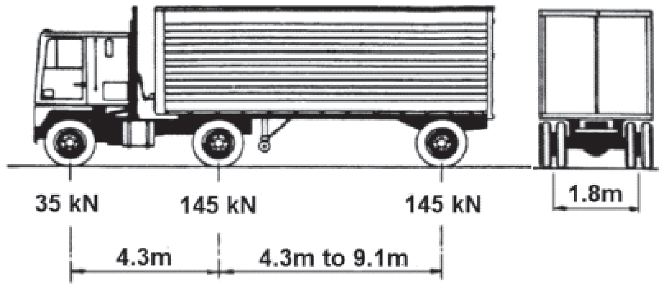
Source: DPWH Design Guidelines Criteria and Standards (Vol. II) 3.11Dead Load, NSCP Vol. II Bridges (ASD) 3.3 Dead Load / Specifications for Highway Bridges I Common 2.2.1 Dead Load

(ii) Live Load

In accordance with DGCS, the followings are considered as live load.

A. Axle Load/ Wheel Load

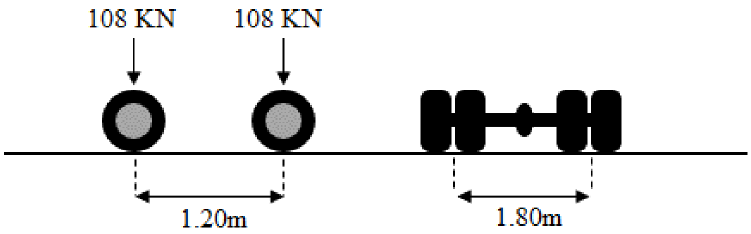
1. Design Truck (HL-93)



Source: DGCS Volume5 10.7.3.1 Design Truck

Figure 6.3.20 Design Truck

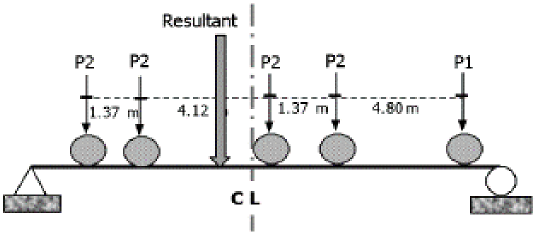
2. Tandem Load



Source: study Team

Figure 6.3.21 Design Tandem

B. Permit Load

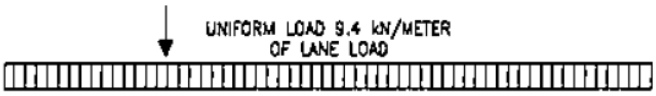


Source: study Team

Figure 6.3.22 Permit Load

(iii) Lane Load

The design lane load shall consist of a load of 9.34kN/m.

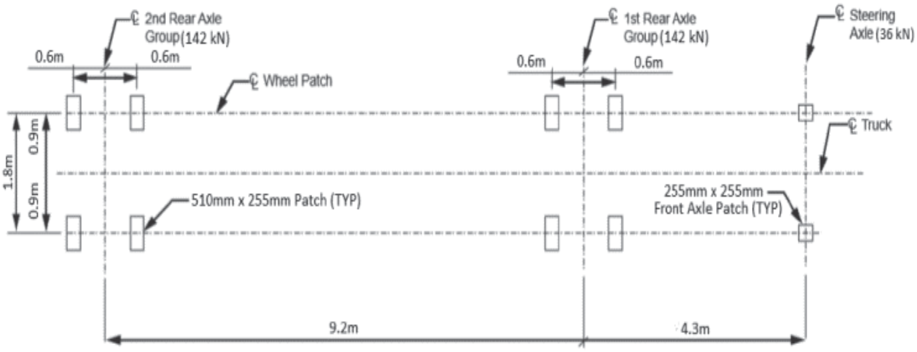


Source: study Team

Figure 6.3.23 Lane Load

(iv) Fatigue Load

In accordance with DGCS, the following is considered as fatigue load.



Source: DGCS Volume5 10.7.5.1 Magnitude and Configuration

Figure 6.3.24 Fatigue Load

(v) Dynamic Load Allowance

In accordance with DGCS, the dynamic load allowance under impact load adopts a constant value of 33% with respect to the strength limit state.

(vi) Wind Load

The DGCS specifies a maximum design wind speed of 160 km/h. However, the design of the control house located above the weir adopts the maximum design wind speed of 200 km/h in accordance with NCSP. Therefore, the bridge design will be based on the control house design, and the maximum design wind speed is V = 200km / h.

(vii) Utilities

No installation

(e) Construction Condition

The construction road will be a maintenance road on the embankment for construction of MCGS.

4) Bridge Planning

(a) Determination of Bridge Length

(i) Seat Width

The seat length of girder is calculated after setting the span length assuming 1/2 of the abutment width as the bridge seat width. If the abutment width is set to 3.500m, the girder end overhang length is set to 350mm, and the expansion gap is set to 50-100mm, the span length will be 14.350m and 30.950m as shown below. In calculating the bridge length, L2 is adopted.

L1= 11.700+1.750*2－(0.35*2+0.05+0.10)= 14.350m

L2= 28.300+1.750*2－(0.35*2+0.05+0.10)= 30.950m **(Selected)**

As shown in the following equations, the seat length of girder and the bearing edge length are approximately 0.855 m and approximately 0.355 m, respectively.

Seat Length of Girder SEM*[1]= 0.7+0.005*L2 = 0.7+0.005*30.950≒ 0.855m

（*[1] Specifications for Highway Bridges in Japan, Volume Ⅴ 16.2 -seat length of girder）

Bearing Edge Length S*[2]= 0.2+0.005*L2 = 0.2+0.005*30.950≒ 0.355m

（*[2] Specifications for Highway Bridges in Japan, Volume Ⅳ 8.6 -design of seat width）

As shown in the following equation, assuming that the standard spacing of the bearing anchor bolts Lb is approximately Lb = 500 mm, the minimum distance Ls from the center of the bearing to the pier edge is 0.605 m.

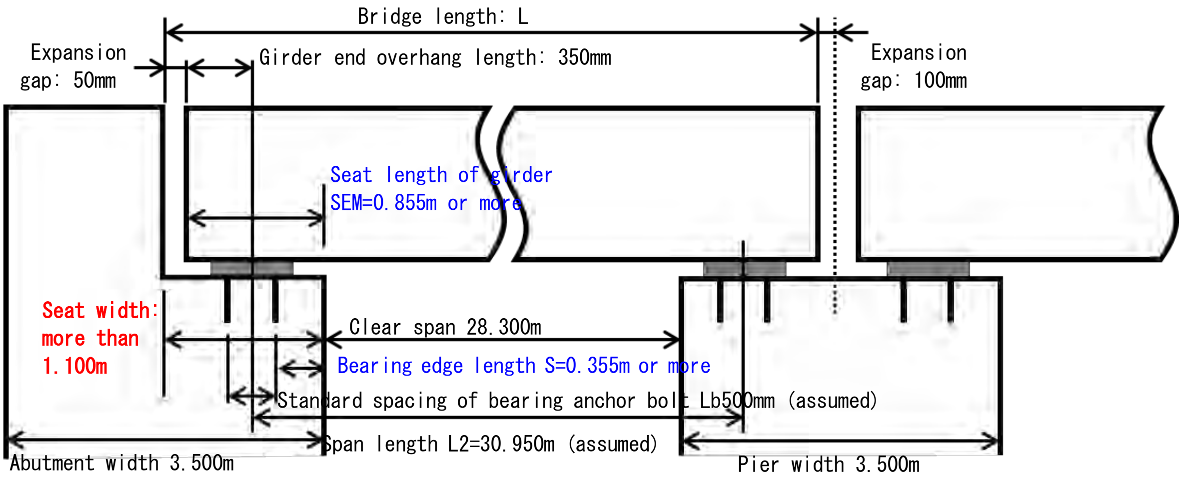
Ls= Lb/2+S= 0.50/2+0.355= 0.605m

The required seat width is B = 1.005m. (refer to the following equation)

B= 0.05+0.855= 0.960m (Calculated from the expansion gap and the seat length of girder)

B= 0.05+0.35+0.605= 1.005m (Calculated from the expansion gap, the girder end overhang length and the minimum distance Ls)

Thus, the bridge seat width is set to **B = 1.100m**. (refer to Figure 1.1.10)



Source: Study Team

Figure 6.3.25 Determination of Bridge Length of MCGS Maintenance Bridge

(ii) Bridge Length

From the above, the bridge length of MCGS maintenance bridge is as follows.

L1= 11.700 + 1.100 + 1.750 = **14.550m**, L2= 28.300 + 1.100 + 1.750 = **31.150m**

(b) Selection of Type of Superstructure

(i) Span Composition/Arrangement

The span composition will be 2 spans according to the position of the piers.

The span lengths are as follows.

L1= 13.700m (Shorter span), L2= 30.300m (Longer span)

(ii) Type of Superstructure

In accordance with DGCS, general type of superstructure and standard applicable span length are as shown below.

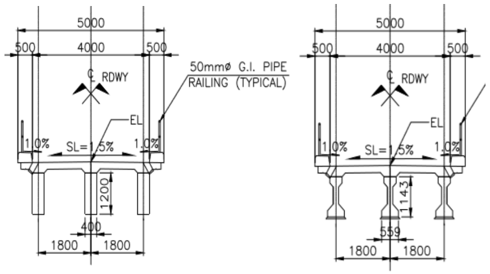
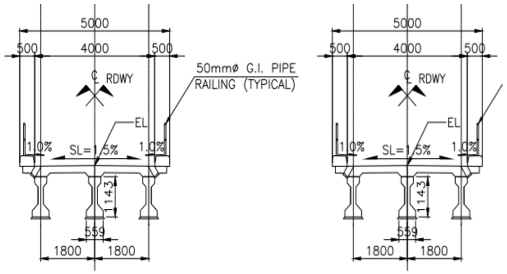
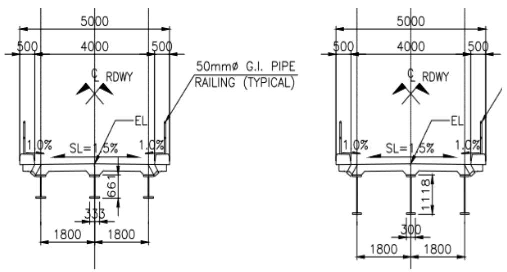
1. Reinforced Concrete Deck Girder (RCDG) : Span Length (13.0m~20.0m)
2. PC-I beam : Span Length (9.0m~42.7m)
3. Steel I-beam : Span Length (15.0m~30.0m)

(iii) Comparison of Type of Superstructure

A comparative study was conducted when the above three types of superstructure were applied to each span. The comparison table is shown on the **Table 6.3.25**

As a result, since it is the most economical and has excellent workability, detailed design will be carried out with **the alternative-2 (PC I-beam for both shorter and longer spans)**

Table 6.3.25 Comparison of Type of Superstructure-MCGS Maintenance Bridge

	Alternative-1 Concrete Deck Girder (RCDG) / PC-I beam		Alternative-2 PC-I beam / PC-I beam		Alternative-3 Steel-I beam / Steel-I beam	
Cross Section (Left; short span, Right; long span)						
Girder Height, Bridge Span	H1=1100mm, L1=14.40m / H2=1143mm, L2=31.00m		H1=913mm, L1=14.40m / H2=1143mm, L2=31.00m		H1=500mm, L1=14.40m / H2=1100mm, L2=31.00m	
Structural Features	For short span; Single-span reinforced concrete deck girder For long span; Single-span prestressed concrete deck girder -Girder weight is heavier than Alt-3	△	Single-span prestressed concrete deck girder -Girder weight is heavier than Alt-3	△	Single-span steel I-shaped girder - lightest girder weight among the alternatives	○
Construction Workability	-Construction period becomes longer due to two different mobilization of the girders are needed	△	-Bents for erection are not needed -The girders can be transported from fabricator's plant	◎	-Bents for erection are needed -Large yard is needed for girder fabrication and assembly	○
Maintainability	-No periodic maintenance works are needed	◎	-No periodic maintenance works are needed	◎	-Periodic repaint/ maintenance cost is needed	△
Construction Cost (Peso/lm)	*****	○	*****	◎	*****	△
(Ratio)	(1.08)		(1.00)		(1.14)	
Total Evaluation	○ : Second Place		◎ : **Selection**		△ : Third Place	

Note: Cost is not presented due to the prior released version.
Source: Study Team

(8) Study on Type of Foundation

1) Sporting Layer

According to the "Technical Criteria for River Works: Practical Guide for Planning [Design] [1]" (Japan Rivers Association), the base ground of the water gate is indicated as follows:

- ✓ In the sand and gravel layers, N-value is almost 30 or more,
- ✓ N-value is almost 20 or more in the cohesive soil layer

According to the previous geological investigation around the MCGS site (refer to **Figure6.3.7**), sand layers with N-value of 30 or more, or base rocks with N-value of 50 or more, are distributed below approximately EL.7.6m at the MCGS location. The design riverbed in the upstream and downstream of the MCGS is EL.+7.85m, and the subgrade of the MCGS body, aprons and e most parts of the connecting retaining wall is located in or on the bed rock. Therefore, the supporting layer of the MCGS is the GF layer, which is the base rock.

2) Type of Foundation

Since the subgrade has reached the support layer in most parts, the spread foundation type is adopted. If the top of the GF layer as the support layer is inclined and it causes the area where the subgrade is located above the GF layer in the same main body or individual structures such as a aprons, the existing ground shall be excavated up to the GF layer and non-reinforced concrete is cast up to the subgrade. Hence, one structure is supported by the same GF layer.

3) Study on Liquefaction

(a) Target Soil Layer

In accordance with the BSDS and the Guideline for Evaluation of Seismic Performance in River Structure (MLIT, Japan), liquefaction analysis is conducted. When liquefaction resistance factor is less than 1.0, the ground shall be liquefied. In both specifications and guideline, the equations for analysis and the criteria are the same. However, seismic force in the BSDS is only $k_{hgL}=F_{pga}PGA$. Hence, the seismic force is set based on this BSDS in this study.

Liquefaction analysis shall be conducted when all the following conditions are satisfied:

1. Saturated soil layer with depth less than 20 m below the ground surface and having ground water level higher than 10 m below the ground surface.
2. Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index, IP, less than 15, even if FC is larger than 35%.
3. Soil layer having a mean particle size (D_{50}) of less than 10 mm and a particle size at 10% passing (D_{10}) (on the grading curve) is less than 1 mm.

Source : BSDS, DPWH, P6-3

Boring surveys in the vicinity of MCGS sites were conducted at a total of seven locations. All the boreholes are composed of 2 layers: sand (As) and rock (GFf). The evaluation is made for G07 which have the thickest sand layer as a target of the liquefaction layer.

The results of laboratory tests at locations near the MCGS installation are summarized in Table 6.3.26.

Table 6.3.26 Liquefaction Analysis Target Layer (G07)

geological name	stratum interface		mean N value	Fine particle content Fc (%)	plasticity index Ip	D 50/D 10	liquefaction study
	EL.m	EL.m					
As	+15.555	+7.395	7.2	30~20	9	-	Target
GF	+7.395	+4.395	50<	-	-	-	Not applicable

Source: Survey team

(b) Calculation of Horizontal Seismic Intensity

Seismic force is calculated in accordance with BSDS, which is the seismic standard in the Philippines. Horizontal seismic Intensity k_{hgL} used for liquefaction analysis is calculated as follows. In addition, external force to be given is assumed to be a level 2 earthquake motion.

$$k_{hgL} = F_{pga} PGA$$

Where,

k_{hgL} : Horizontal seismic intensity

F_{pga} : Regional correction factor (1.0 : Refer to **Table 6.3.27**, Type I)

PGA : Peak Ground Acceleration (0.6: Regional maximum acceleration chart of BSDS (P3B-8))

Table 6.3.27 Regional Correction Factor

Table 3.5.3-1 Values of Site Factor, F_{pga} at Zero-Period on Acceleration Spectrum

Ground Type (Site Class)	Peak Ground Acceleration Coefficient (PGA)[1]					
	$PGA \leq 0.10$	$PGA = 0.20$	$PGA = 0.30$	$PGA = 0.40$	$PGA = 0.50$	$PGA \geq 0.80$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

Note: Type I Ground: PGA: 0.6 at 1.0 (interpolation)

[1] Use straight-line interpolation for intermediate values of PGA.

Source: Study Team added to BSDS P3 -32.

From the above, the horizontal seismic intensity k_{hgL} = 0.6 used for liquefaction determination is as follows.

$$k_{hgL} = F_{pga} PGA = 1.0 \times 0.6 \quad = \boxed{0.6}$$

(c) Liquefaction Analysis

The formula used for liquefaction analysis is basically the same between the "Performance Based Seismic Design Criteria for River Structures" in Japan and the BSDS in the Philippines. However, the following points are different from the Japanese standards. In this case, the evaluation is performed according to the BSDS for Philippines.

- Correction factor c_w by earthquake ground motion characteristics for BSDS in the Philippines shall use the formula in the case of Level 2-2 earthquake ground motions in the Japanese seismic standard.
- The horizontal seismic intensity at the ground surface of Level 2 earthquake ground motion shall be the value obtained by the calculation of the aforementioned horizontal seismic intensity.

As a result of the evaluation, since the resistances FL against liquefaction in the As layer serving as the target layer can be calculated 0.190, it can be evaluated that liquefaction occurs due to Level 2 earthquake ground motions.

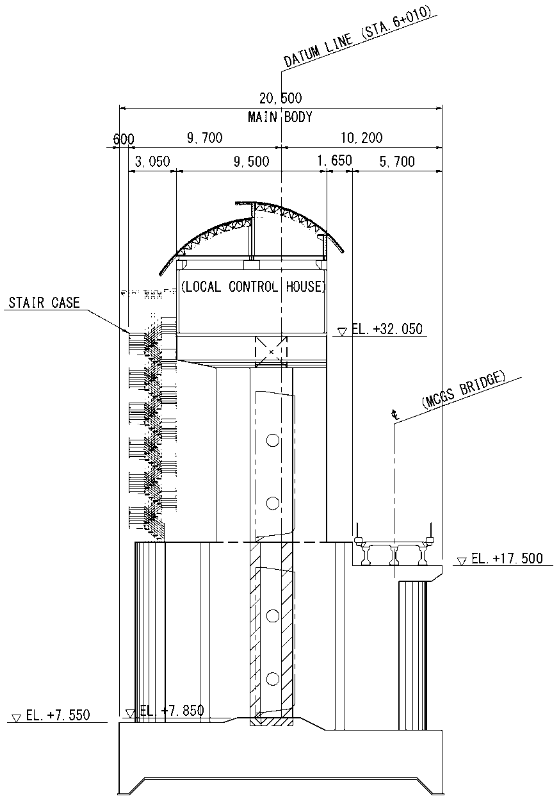
(9) Determination of Main Body Specifications (Section Dimensions)

1) Length of Main Body

The main body length is determined from the dimensions such as the width of a maintenance bridge and the width of a column

I.	Clearance	: 0.60m
II.	Staircase	: 3.05m
III.	Column	: 9.50m
IV.	Clearance	: 1.65m
V.	Maintenance Bridge +Wheel Guard+Clearance* +Extension of Pier	: 5.70m
Total		: 20.5m

*Maintenance Bridge 4.40m+Wheel Guard 0.35m x 2＋Clearance 0.10m+ Extension of Pier0.50m



Source: Study Team

Figure 6.3.26 Length of Main Bod of MCGS

2) Breast Wall

The height and length are set according to the "Technical Standards for River Erosion Control (draft) Design Part I" as follows. The purpose of the parapet is to prevent soil suction and temporary slope failure.

Height:	11.95m	->Adjust to the top of the Revetment height
Length:	6.00m	->More than half chest wall height

3) Width of Weir

According to "design of weir" (Dam Engineering Center), the width of the weir can be calculated based on the span length as follows from past cases.

$t ≒ (1/10 \sim 1/13) \times B = 2.45\text{ m} \sim 3.18\text{m}$

Where, t : Length of Weir (m)

B : Span (m) = Max. 31.8m

However, in "design of weir" (Dam Engineering Center), the above equations are summarized based on the statistics before considering Level 2 earthquake ground motions. Therefore, the maximum value of 3.18 m in the above calculation results is rounded up to 0.5 m, and t = 3.5 m

4) Thickness of Bottom Slab

(a) Pier and Center Slab

In consideration of the fact that Level 2 earthquake ground motion was not taken into account in the statistics of the cases, it is herein set at 3.0 m, with reference to the relationship between thickness of floor slab and span described in "design of weir" (Dam Engineering Center).

The central floor slab supports the gate in combination with the floor slab of the pier. To ensure water tightness, it is necessary to prevent uneven settlement between them and to have a rigidity which can suppress the deflection in the vertical direction of the gate to the allowable value or less. In this case, the thickness is set at 3.0 m, which is the same as that of the floor slab of the pier.

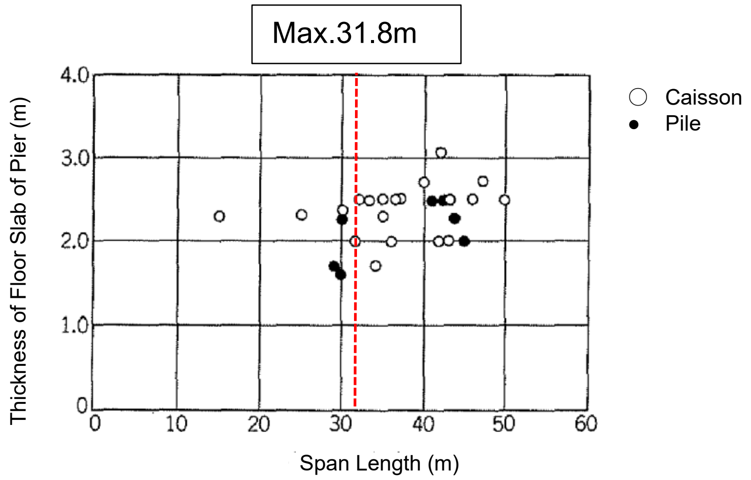


Figure 2.3.45 Span Length and Thickness of Floor Slab of Pier

Source: "Design of Weir", Dam Engineering Center

Figure 6.3.27 Span Length and Thickness of Floor Slab of Pier

(b) Stability against Uplift on Center Floor Slab

The study here checks if the center floor slab satisfies the required stability against uplift. **Table 6.3.28** shows the water level condition.

Table 6.3.28 Water Level Condition

Calculation Conditions		Water Level			Remarks
		Downstream	Upstream	Water Level Difference	
Center Floor Slab	During Flood	13.425	17.400	3.975	

Source: Study Team

(i) Calculation of Uplift

Uplift pressure can be calculated by the following formula.

$$U_{px} = \left(h_2 + \Delta h \frac{\sum l - l_x}{\sum l} \right) \times W_0$$

Where, U_{px} : Uplift (kN/ m²)

h_2 : Depth in Downstream Side (m)

Δh : Water Level Difference (m)

$\sum l$: Total Creep Distance (m)

l_x : Creep Distance from the Upstream End to the Evaluation Point (m)

W_0 : Unit Weight of Water (kN/ m³)

Item	Symbol	Case 1 Flood	Unit	Remarks
Water level in Downstream	h_2	6.375	m	
Difference of Water level	Δh	3.975	m	
Total Creep Distance	$\sum l$	79	m	
Creep Distance from the Upstream edge to the evaluation point	l_x	18	m	
Unit Weight of Water	W_0	9.8	kN/m³	
Uplift at the evaluation point	U_{px}	92.554	kN/m²	

From the above, the uplift is 92.554.kN/m².

(ii) Calculation of the Required Thickness of Slab

The required thickness is calculated by the following formula.

$$t = F_S \frac{u_{pm} - h_2 \cdot W_0}{\gamma_c - W_0}$$

Where,
- t: : Required thickness for apron (m)
- F_S : Safety Factor 4/3
- u_{pm} : the maximum uplift acting on the apron (kN/ m^3)
- γ_c : Unit Volume Weight of Concrete (kN/ m^3)
- h_2 : Depth (m)
- W_0 : Unit volume weight of water (kN/ m^3)

Item	Symbol	Case 1 Flood	Unit	Remarks
Safety Factor	F_S	1.333		
Maximum Uplift acting on Apron	u_{pm}	92.554	kN/m^2	
Unit Weight of Concrete	γ_c	23.5	kN/m^3	
Water Depth	h_2	6.375	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Required Thickness of Apron	t	**2.927**	m	

As described above, since the required thickness is 2.927 m, the 3.0 m thickness of center floor slab satisfies the required stability against floating.

5) Height of Column

(a) Height of the Bottom End of the Gate when Fully Opened

The height of the bottom end of the gate when it is fully opened is set to the higher of the following values:

- Design Dike Crown: DFL + 17.4 m + Freeboard: 1.2 m (Before the Diversion) = EL. + 18.6 m
- Height of the Revetment Determined by the Current Riverbank Height: EL + 19.0 m

From the above, the height of the lower end of the gate when it is fully opened is set to EL + 19.0 m

(b) Height of Column

As for the height of column, in accordance with the "Technical Standards for River Erosion Control (draft) Design Part I", clearance of 1 m or more is secured at the bottom edge of the operation deck. According to the structural detail of gate structure proposed in the 2002DD, since the size of the hoisting sheave at the top of the gate leaf is about 0.9 m, 1.5 m clearance was considered in the 2002DD. The height of the column including the operation deck is calculated as follows:

Height of the bottom of the Gate when it is fully opened	: E.L. + 19.0 m
Gate Height	: 9.55 m
Clearance	: 1.5 m
+ Thickness of Operation Deck Slab	: 2.0 m
Top Height of the Column including the operation Deck	: E.L. + 32.05 m

Therefore, the height of the column including the operating deck is set at EL.+32.05 m.

6) Seepage Cut-off Wall

(a) Location

Seepage cut-off walls are installed to prevent soil particles from flowing below the weir (vertical cut-off wall) and in the weir side face (horizontal cut-off wall), and to prevent soil from being sucked out by scouring. The point of installation is shown **Figure 6.3.28.**

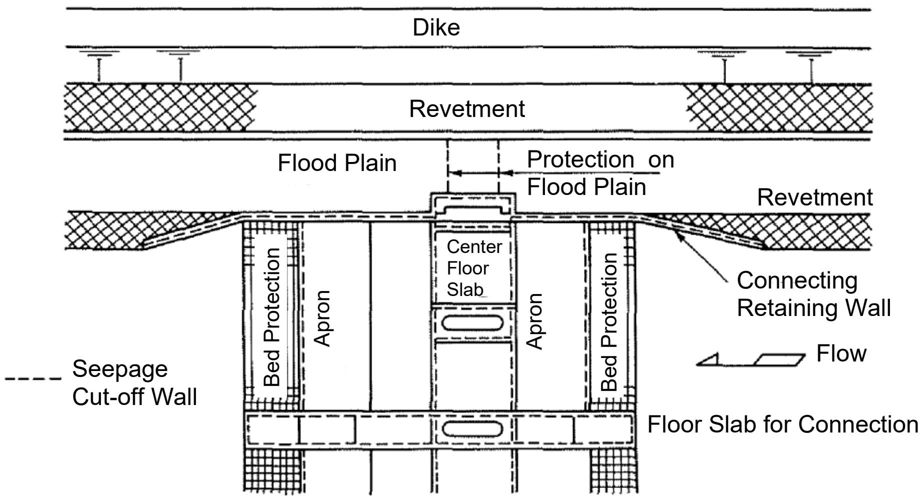


Figure 2.3.91 Layout of Seepage Cut-off Wall

Note: Translated by the Study Team from the "Design of Weir", Dam Engineering Center

Figure 6.3.28 Layout of Seepage Cut-off Walls

(b) Water Barrier Type

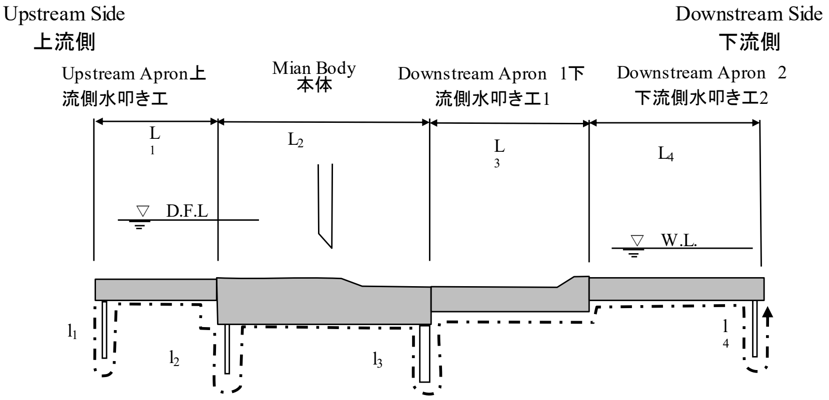
According to the results of previous geotechnical investigation, it can be said that the upper surface of the bedrock appears within several meters from the existing riverbed. Considering that the design riverbed is more than 2 m lower than the existing riverbed and the thickness of the floor slab is 3.0 m, rock surface will appear even at the depth where the seepage cut-off wall is installed. Therefore, the concrete cut-off for seepage cut-off wall is adopted.

(c) Length of Cut-off Wall

The required length of the cut-off wall is calculated from the equation of Lane.

(i) Vertical Direction

The length of the cut-off wall in the vertical direction is examined as follows:



Water Difference ⊿H
水位差 (Unit: EL.m)

	STA	Upstream 上流側	Downstream 下流側	Water Diffrence 水位差	Remarks 備　考
Case1	6+010	17.400	13.425	3.975	

Horizontal Creep Distance L
水平クリープ長 (Unit: m)

	L_1	L_2	L_3	L_4	Total L	Remarks
Case1	15.0	20.5	19.0	11.0	65.5	

Required vertical creep distance is calculated as fllows;
鉛直方向の必要クリープ長は以下の通り算定できる。

$$C \leq \frac{L/3 + \sum l}{\Delta H}$$

Vertical Creep Distance Σl and Creep Distance at Each Seepage Cut off Wall
鉛直クリープ長Σlと各遮水工位置で必要なクリープ長

	Creep Ratio クリープ比	Σl (m)	Reqired Length of Wall at 1 location (m) 1か所あたりの必要長	Number of Walls 遮水工個所数	Type of Ground 土質
Case1	8.5	11.12	1.40	4.00	Very Fine Sand or Silt 極めて細かい砂またはシルト

Case 1 (Unit: m)

	Thickness of Slab	l_1 CC	l_2 CC	l_3 CC	l_4 SSP	Thickness of Slab	Total 合計	Stability 安定性	Ccal. 計算クリープ比
Length 遮水工長さ	3.5	0.5	1.0	1.0	0.5	3.8	10.3	OK	8.86
Side 面数	1.0	2.0	2.0	2.0	2.0	1.0	-		
Distance 距離	3.5	1.0	2.0	2.0	1.0	3.8	13.3		

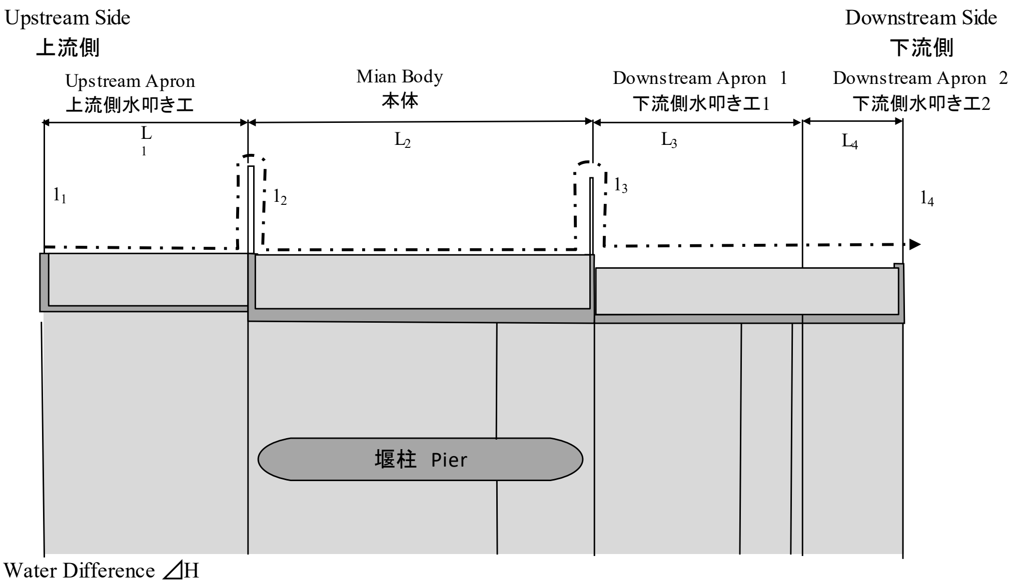
SSP…Steel Sheet Pile(鋼矢板), CC…Concrete Cut off（コンクリートカットオフ）

The creep ratio is set to 8.5, assuming "extremely fine sand or silt" as a safety-side design, considering fine sand backfilled after excavation.

As a result of the above calculation, the required stability can be satisfied by the apron with the concrete cut-off L = 0.5 m and the main body with the concrete cut-off L = 1.0 m.

(ii) Horizontal Direction

The length of the cut-off wall in the horizontal direction is examined as follows in case the length of the cut-off wall in the horizontal direction is assumed to be up to the shoulder of the revetment and to be 7.65 m from the edge of the pier.



Water Difference ⊿H
水位差 (Unit: EL.m)

	STA	Upstream 上流側	Downstream 下流側	Water Diffrence 水位差	Remarks 備　考
Case1	6+010	17.400	13.425	3.975	

Horizontal Creep Distance L
水平クリープ長 (Unit: m)

	L_1	L_2	L_3	L_4	Total L	Remarks
Case1	15	20.5	19	11	65.5	

Required Horizontal creep distance is calculated as fllows;
水平方向の必要クリープ長は以下の通り算定できる。

$$C \leq \frac{L/3 + \sum l}{\Delta H}$$

Horizontal Creep Distance Σl and Length of Each Seepage Cut off Wall
水平クリープ長Σlと各遮水工位置で必要なクリープ長

	Creep Ratio クリープ比	Σl (m)	Reqired Length of Wall at 1 location (m) 1か所あたりの必要長	Number of Walls 遮水工個所数	Type of Ground 土質
Case1	8.5	11.12	5.6	1.0	Very Fine Sand or Silt 極めて細かい砂またはシルト

Case 1 (Unit: m)

	l_1	l_2	l_3	l_4	Total	Stability	Ccal.
Length 遮水工長さ	0.00	7.65	0.00	0.00	7.65	OK	10.97
Side 面数	2.0	2.0	2.0	2.0	-		
Distance 距離	0.00	15.30	0.00	0.00	15.30		

Based on the above calculation results, the required stability can be sufficiently satisfied by installing only the cut-off wall in the upstream side of the main body up to the shoulder of the revetment.

7) Energy Dissipater

Based on the study described later, without considering the energy dissipater, the critical water depth section is more than 70 m from the sill end (end-sill) and hence, protection by concrete floor slab in the corresponding area is required. In order to stabilize the flow condition earlier by suppressing the effect of rapid flow velocity and water jump generated after passage from the weir to the downstream side, and to shorten the protection area by the concrete slab, an energy dissipater is installed.

(a) Type of Energy Dissipater

There are two types of energy dissipaters, one is by the stilling basin and the other is by the baffle pier. To avoid obstructing the flood flow and ferry navigation in ordinary time, a stilling basin shall be basically adopted.

(b) Specifications of the Stilling Basin

Based on the "Design of Weir" (Dam Engineering Center), the following criteria are established based on the previous actual cases.

(i) Invert elevation of Stilling Basin

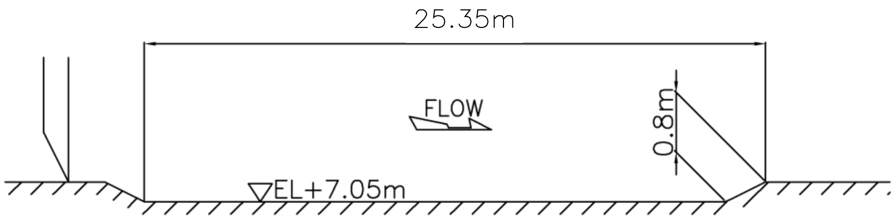
The height of the stilling basin is about 20% of the expected maximum water level difference.

- Decreasing reservoir height, h = (EL + 17.4 m – EL. + 13.425 m) x 20%
  = 0.795 → 0.8 m
- Depth of Stilling Basin = EL. 7.85 m – 0.8 m = EL. + 7.05 m

(ii) Length of the Basin

The length of the basin is about 25 m as a reference. 25.35 m from the bottom of the still is adopted.

From the verification result by the hydraulic model experiment, it was confirmed that there was no problem in the above dimensions (See **Final Report of Hydraulic Model Experiment** for details.). However, according to the results of the hydraulic model experiment, an end-sill shall be installed downstream of the narrower gate to settle the flow conditions of rapid flow.

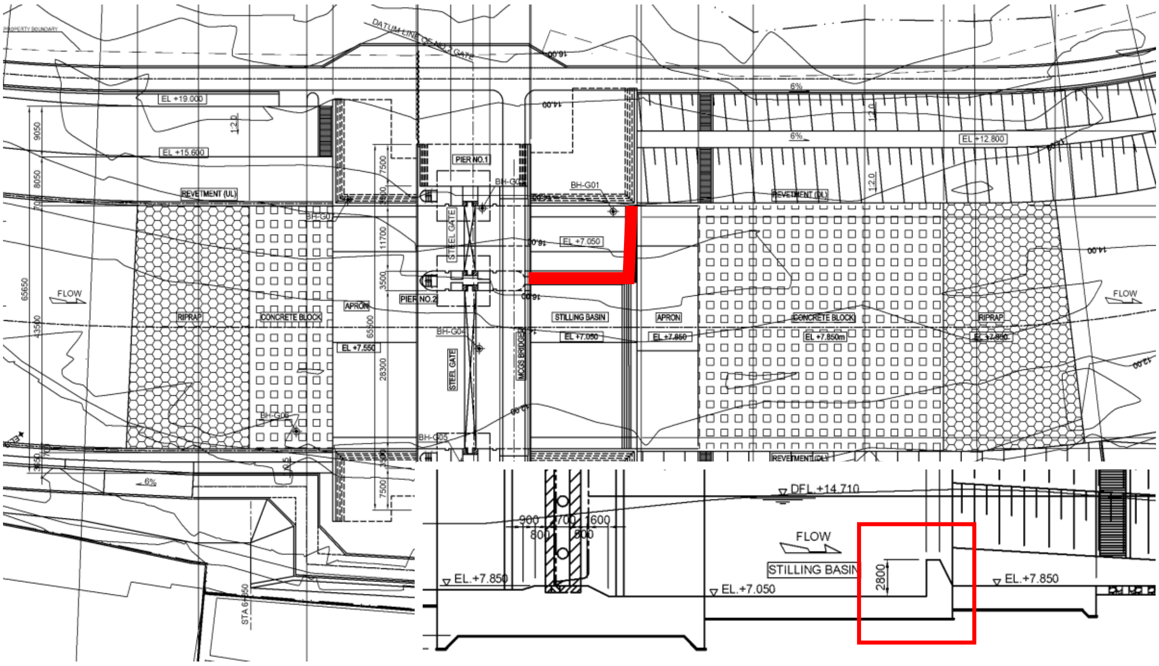


Source: Study Team

Figure 6.3.29 Specification of Stilling Basin

(iii) Specifications of Sil

According to the results of the hydraulic model experiment, the flow velocity on the right bank side after the reduction is suppressed to 2 -3 m, and the effect of dispersing the flow concentration can be sufficiently obtained. Therefore, end-sill with 2.8m height would be placed in an L-shape which surround the stilling basin.



Source: Survey team

Figure6.3.30 L- type End-Sill

(c) Study on the Influence of Sedimentation

Since the end-sill may cause sediment deposition in the stilling basin, the trend of sediment in the stilling basin during flood is confirmed by hydraulic model experiments. The necessity and dimensions of sediment discharge (Slits, etc.) are examined as a countermeasure against sediment siltation in the stilling basin.

(i) Model Experiment Conditions

Table 6.3.29 The Model Experiment Conditions of Examination of the Effect of Sedimentation

Items	Conditions
Target Discharge	Design Discharge at MCGS 500m^3/s Minimum Operational Discharge of Short-Span Gate 288m^3/s
Specifications of Energy Dissipator	L-type End-Sill (Height 2.0m)
How to Pour sand into the Energy Dissipator	Pour sediment at the upstream of MCGS (Initially No Sedimentation) Sediment up to the Weir Height in the Initial State Sediment up to the End-sill Height in the Initial State
Sediment Particle Diameter	About 15mm

Source: Study Team

A. Target Discharge

The target discharge of the experiment are design discharge 500m^3/s and 288m^3/s which is the minimum discharge when operating the narrower span gate of MCGS.

B. Specifications of Energy Dissipator

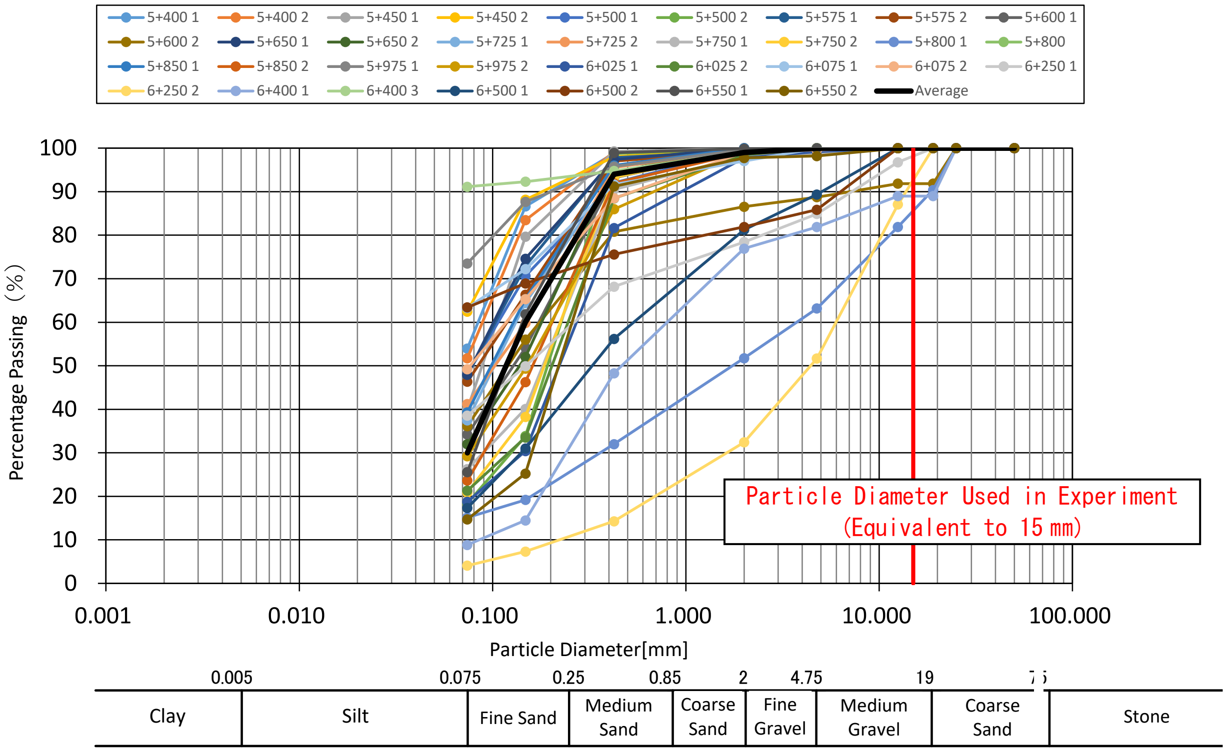
The energy dissipator is L-type end sill with the height of 2.8m, which is final shape obtained in the hydraulic model experiment. No slit is installed on the end sill.

C. How to Pour Sand into Energy Dissipator

At first, assuming the situation where sediment flows into the energy dissipator during flood, the behavior of sediment poured into the upstream of MCGS is examined under the conditions of no sediment in the stilling basin. In addition, assuming a state in which sedimentation in the stilling basin occurs during normal times, the behavior of sediment laid in advance in the stilling basin is also examined.

D. Sediment Particle Diameter

According to the previous boring survey around the planned construction site of MCGS, the representative particle size is about 0.1 to 0.6mm. When this particle size is converted into a model value based on the model scale, 1/50, the particle size of the model is 0.002 to 0.012mm, which is equivalent to cohesive materials such as clay and silt. The use of cohesive materials in the experiment might cause the gap of the phenomenon between in the model and prototype. Therefore, No.6 silica sand, particle size about 0.3mm, which is smallest particle size without cohesiveness shall be used This silica sand is approximately 15mm in prototype scale, which is larger than the sediment on site and is more easily deposited.



Source: Study Team

Figure 6.3.31 Particle Size Distribution of Sand on Site and Sand Used in Experiment

(ii) Conditions and Method of Experiment

The behavior of sediment is observed with Rosario weir fully closed and the narrower span gate fully opened to allow the entire discharge flow to the Marikina River. Conditions and cases of the experiment are shown in **Table 6.3.30** and **Table 6.3.31** respectively.

Table 6.3.30 Experimental Conditions (Effect of Sedimentation)

Conditions	Setting
Target Discharge	Design Discharge at MCGS 500m^3/s Minimum Operational Discharge of Short Span Gate 288m^3/s
Water Level at the Downstream End (Lower Marikina River)	HQ relationship (Sta.4+950)
Energy Dissipator	L-type End sill 2.0m without slit

Source: Study Team

Table 6.3.31 Experimental Cases (Effect fo Sedimentation)

Discharge	Sediment Conditions	Remarks
500m^3/s	Throw into the upstream of MCGS	To confirm sediment inflow and out flow during floods
288m^3/s	Throw into the upstream of MCGS	
288m^3/s	Deposit up to the weir height in the initial state	To confirm the behavior of sediment deposited at normal times
288m^3/s	Deposit up to the end sill height in the initial state	

Source: Study Team

(iii) Results of Experiment

A. Behavior of Sediment Thrown (Design Discharge 500m^3/s)

Assuming the sediment flows from the upstream of MCGS, the behavior of the sediment when the sand was thrown from the upstream of the gate was confirmed. About 300m^3 of sand was thrown from the upstream of the gate under the condition of 500m^3/s without sediment in the stilling basin in the initial state. The results of the experiment are shown below.

- When sediment is thrown away from the gate, the sediment accumulates at that point and does not flow into the energy dissipator.
- When sediment is thrown just upstream of the gate, almost all of the sediment flows downstream of the energy dissipator. There is no accumulation in the stilling basin. Even if there is no slit, it is considered that the sediment flows out downstream of the energy dissipator due to turbulent flow in the stilling basin.

Table 6.3.32 The behavior of Sediment (500m^3/s, Thrown Just Upstream of the Gate)

Conditions	Results
1 minute after sediment was thrown	
2 minutes after sediment was thrown	

Source: Study Team

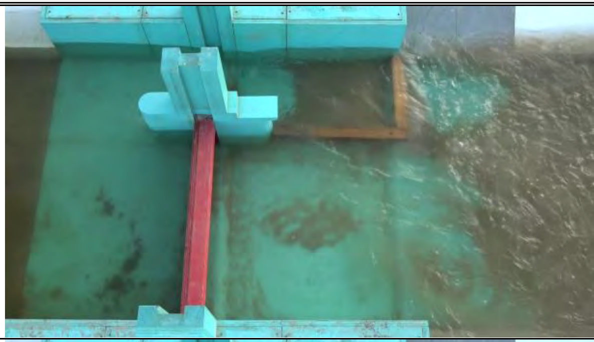
B. Behavior of Sediment Thrown (Minimum Discharge of Short-span Gate 288m^3/s)

Since there is almost no sedimentation in the stilling basin under the design discharge, the behavior of the sediment was confirmed under the minimum discharge of the narrower span gate, 288m^3/s, which is a condition where sedimentation is more likely to occur. About 300m^3 of sand was thrown from just upstream of the gate under the condition of 288m^3/s without sediment in the stilling basin in the initial state. The results of the experiment are shown below.

- When sediment is thrown away from the gate, the sediment accumulates at that point and does not flow into the energy dissipator.

- When sediment is thrown just upstream of the gate, although some of the sediment flows over the end sill, some of the sediment temporarily accumulates in the stilling basin. The deposited sediment flows out downstream of the energy dissipator in about 20 minutes due to turbulent flow.

Table 6.3.33 The behavior of Sediment (288m^3/s, Thrown Just Upstream of the Gate)

Conditions	Results
1 minute after sediment was thrown	
2 minutes after sediment was thrown	
2 minutes after sediment was thrown	

Source: Study Team

C. Behavior of Deposited Sediment

Assuming that sediment was deposited in the stilling basin in normal times, the behavior of the deposited sediment was observed under the condition of the minimum discharge 288m^3/s without sediment supply. The results of the experiment are shown below.

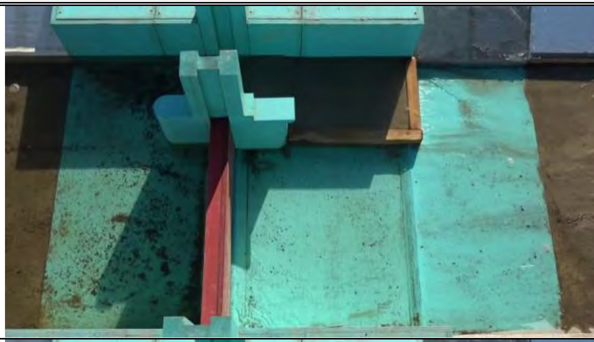
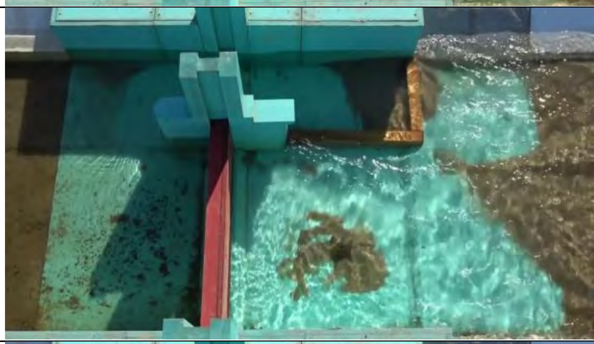
- When sedimentation height is the weir height in the initial state, almost the entire amount of sediment flows out downstream of the energy dissipator in about 140minutes (**Table 6.3.34**).
- When sedimentation height is the end sill height in the initial state, almost the entire amount of sediment flows out downstream of the energy dissipator in about 180minutes (**Table 6.3.35**).
- Considering that sediment is drained in about 3 hours at any deposition height, the necessity of the installation of slit on the end sill is small from the viewpoint of sediment control.

Table 6.3.34 The behavior of Sediment (288m^3/s, Sediment up to the Weir Height)

Conditions	Results
Initial State	
30 minutes after start of water flow	
60 minutes after start of water flow	
120 minutes after start of water flow	
140 minutes after start of water flow	

Source: Study Team

Table 6.3.35 The behavior of Sediment (288m³/s, Sediment up to the End sill Height)

Conditions	Results
Initial State	
30 minutes after start of water flow	
60 minutes after start of water flow	
120 minutes after start of water flow	
180 minutes after start of water flow	

Source: Study Team

(iv) Summary

Based on the above-mentioned results of the experiment, the discussion is organized below.

- Almost all of the sediment with a particle diameter about 15mm deposited in the stilling basin can be discharged to the downstream of the end sill by operating the narrower span gate for about 3 hours during floods even if the end still has not slit.
- Considering that the bed material at the site is about 0.1mm to 6mm according to the on-site drilling survey and that the tractive force is small at the upstream of MCGS due to backwater by the gate, it is presumed that the sediment flowing into the vicinity of MCGS is mostly finer than 15mm. Therefore, it is considered that sediment is more easily discharged on site than in this experiment.

Based on the above, no slit on the end sill is needed to flush sediment because sedimentation problems will rarely occur in the stilling basin.

8) Apron

(a) Length of Apron

(i) Downstream Apron

The length of apron is set considering the length of the creep distance.

From the above-mentioned study, the length of downstream apron, L satisfying the required creep distance can be calculated as follows.

$$C \leq \frac{L/3 + \sum l}{\Delta H}$$

$$\mathrm{C} \leq \frac{(15.0+20.5+L)/3+(1.0+2+2+1.0)+(3.0+3.5)}{3.975} = 8.5 \text{ (Very fine Sand or Silt)}$$

Thus, L≧28.4m

The length is made by rounding up to 5 m, and the length of downstream apron is set at 30 m.

(ii) Upstream Apron

According to "Design of Weir" (Dam Engineering Center), the length of the upstream apron is often set at about 1/2 to 2/3 of the downstream apron. Therefore, the upstream apron is set at L=15.0m which is about 1/2 to 2/3 of the downstream apron. Therefore, the upstream apron is set at L=15.0m.

(b) Thickness of Slab

(i) Downstream Apron

The downstream apron will have to be divided into two parts because it is necessary to install the stilling basin in the middle of 30 m, a length that satisfies the permeation path length.

The thickness of the apron slab is set to be able to resist the assumed uplift by its own weight.

The required thickness shall be calculated based on the formula shown in "Technical Standards for River Erosion Control (draft) Design Part I".

The water level conditions are shown in **Table 6.3.36**.

Table 6.3.36 Water Level Condition

Calculation Conditions		Water Level			Remarks
		Downstream	Upstream	Water Level Difference	
Downstream Apron	During Flood	13.425	17.400	3.975	

Source: Study Team

A. Calculation of Uplift

Uplift pressure can be calculated by the following formula.

$$U_{px} = \left(h_2 + \Delta h \frac{\sum l - l_x}{\sum l} \right) \times W_0$$

Where, U_{px} : Uplift (kN/ m2)
h_2 : Depth in Downstream Side(m)
Δh : Water Level Difference (m)
$\sum l$: Total Creep Distance (m)
l_x : Creep Distance from the Upstream End to the Evaluation Point (m)
W_0 : Unit Weight of Water (kN/ m^3)

Item	Symbol	Case 1 Flood	Unit	Remarks
Water level in Downstream	h_2	6.375	m	
Difference of Water level	Δh	3.975	m	
Total Creep Distance	$\sum l$	79	m	
Creep Distance from the Upstream edge to the evaluation point	l_x	45	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Uplift at the evaluation point		**79.240**	kN/m^2	

From the above, the uplift is 79.240.kN/m^2.

B. Calculation of the Required Thickness of Slab

The required thickness is calculated by the following formula.

$$\mathrm{t} = F_S \frac{u_{pm} - h_2 \cdot W_0}{\gamma_c - W_0}$$

where, t: : Required Thickness for Apron (m)
F_S : Safety Factor 4/3
u_{pm} : The Maximum Uplift Acting on the Apron (kN/ m^3)
γ_c : Unit Volume Weight of Concrete (kN/ m^3)
h_2 : Depth (m)
W_0 : Unit Volume Weight of Water (kN/m^3)

Item	Symbol	Case 1 Flood	Unit	Remarks
Safety Factor	F_s	1.333		
Maximum Uplift acting on Apron	u_{pm}	79.240	kN/m^2	
Unit Weight of Concrete	γ_c	23.5	kN/m^3	
Water Depth	h_2	6.375	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Required Thickness of Apron	t	**1.632**	m	

As described above, since the required floor slab thickness is 1.632m, the floor slab thickness of the downstream aproning work is set to 1.7 m

(ii) Upstream Apron

The slab thickness of the upstream apron is set at 1/2 to 2/3 of the downstream apron thickness.

$$1.8 \times \left(\frac{1}{2} \sim \frac{2}{3} \right) = 0.9m \sim 1.2m$$

From the above, the upstream side water apron floor slab thickness is set to 1.2 m.

9) Bed Protection Work

(a) Bed Protection Length

(i) Downstream

The length of bed protection shall be set in accordance with the "Structural Design Guide for Groundsill" in Japan.

A. Without Energy Dissipator

Without energy dissipator, the length of the Bed Protection A can be calculated as follows.

L = L1 + L2

Where,

L1 : Length of the Section between the points of Water Downflow and the Start of Hydraulic Jump.

L2 : Length of Hydraulic Jump Section

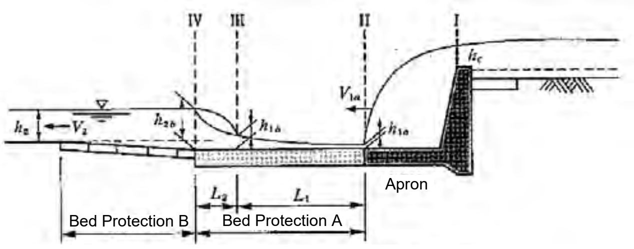


Figure 2-36 General Figure indicating Flow in the Section of Bed Protection A

Source: Structural Design Guide for Groundsill, Japan Institute of Country-ology and Engineering)

Figure 6.3.32 Length of Bed Protection

The following shows the calculation of the length of Bed Protection A.

Study Condition

Discharge Q	500 (m3/s)	Water Level in Down Stream	13.425 (EL.+m)
Width B	11.7 (m)	Critical Death hc	5.710 (m)
Unit Discharge q	42.7 (m3/s/m)	Water Level in Up Stream	13.560 (EL.+m)
Top of Sill	7.850 (EL.+m)		
Top of Apron	7.050 (EL.+m)		

(1) Calculation of Velocity at the Point where Water Falling V1a and Depth h1a

Overflow Point

Depth hc(m)	5.710
Velocity Vc(m/s)	7.484
Froud Number Fc	1.001

→

Point where Water Falling

Depth h1a(m)	4.279
Velocity V1a(m/s)	9.987
Froud Number F1a	1.542

ΔZ(m) 0.800

h1a、V1a is calculated with trial to satisfy Vc^2/2g+ΔZ+hc=V1a^2/2g+h1a

Left Side(Overflow Point)= 9.36785377

Right Side(Point where Water Falling)= 9.36778413 e=Left-Right= 6.96391E-05

(2) Calculation of Conjugate Depth against the Water Level In Down Stream

Water Level at the start of Hydraulic Jump is calculated from the following equation.

$$h_{1b} = \frac{1}{2} \times \left(\sqrt{1 + 8 \cdot F_2^2} - 1 \right) \times h_2$$

(Down Stream Side)

Water Depthh2(m)	6.375
VelocityV2(m/s)	6.704
Froud NumberF2	0.848

→

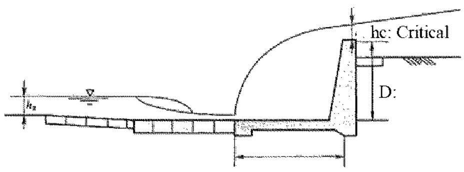
(Point where Hydraulic Jump Stars)

Water Depthh1b(m)	5.096
VelocityV1b(m/s)	8.385
Froud NumberF1b	1.186

(3) Length of Apron

Length of apron is calculated by Rand's Equation.

$$\frac{W}{D} = 4.3 \times \left(\frac{h_c}{D} \right)^{0.81}$$



Where, W : Length of Apron (m)
D : Height (m) → (7.85 - 7.05 = 0.8 m)
hc : Critical Depth)

W= 17.0 m

(4) Length of Bed Protection

1) Length of Critical Depth Section

$$-\frac{q^2}{C^2} x + a = \frac{1}{4} h^4 - h_c^3$$

Where, q : Unit Discharge
C : Chezy' coefficient(=$h^{1/6}$/n)
n : Roughness Coefficient 0.035
x : Section Length(m)
a : Coefficient

When, x=0, h= a= -7.1279E+02

When, h=h_{1b}, the section length is calculated as follows.

-1.30001 x= -67.33657

x= 51.8 =L1

The length of Bed Protection A is the length of the critical flow section when forced hydraulic jump is not considered and needs protection by a concrete slab like apron. Without energy dissipator, along with the length of the apron according to Rand's formula, about 70 m length from the end of the end still must be protected with concrete, which is uneconomical. Therefore, the energy dissipator is installed to generate the forced hydraulic jump and the protection length by the concrete slab shall be shortened.

B. With Energy Dissipator

(4) Length of Bed Protection

1) Calculation of Water Depth h2a at the End of Hydraulic Jump

Water depth at the end of hydraulic jump is calculated from the following equation.

$$h_{2a} = \left(\frac{1}{2}\sqrt{1 + 8 \times Fr_1^2} - 1\right) \cdot h_{1a}$$

Where, h_{2a} : Water Depth at the End of Hydraulic Jump (m)

h_{1a} : Water Level in the Immediate Downstream of Main body (m)

h_{2a} = 5.296 m

2) Length of Bed Protection A

Length of bed protection A is calculated as follows.

L= a x h_{2a}

Where, L : Length of Bed Protection A (m)

a : Coefficient (4.5-6.0)

Coeffient	4.5	**6**
Length of Bed Protection A L(m)	23.9	**31.8**

Since the length is longer, L=31.8m is adopted.

4) Length of Bed Protection B

Length of bed protection B is based on 5 times of the water depth in the down stream.

L_B= 5 x h_2

= 31.9 m

From the above, the length of the bed protection works shall be 44.0 m from the downstream end of the apron as follows:

Bed Protection Length in the Downstream

= (Length of Apron+ Length of Bed Protection A + Length of Bed Protection B) - (Length of Min Body in the downstream Side of End-Sill + Length of Apron)

= （17.0m + 31.8 m +31.9m） －(7.40 m+ 30.0m) = 43.30 m → 44.0m

(b) Study on Concrete Block for Bed Protection

(i) Weight of Bed protection

The block weight is calculated by the following formula of "sliding and tipping - layering" model of the "Dynamic Design Method of Revetment".

$$W = a\left(\frac{\rho_w}{\rho_b - \rho_w}\right)^3 \cdot \frac{\rho_b}{g} \cdot \left(\frac{V_d}{\beta}\right)^6$$

Where, W : Minimum Block Weight to Avoid Moving

V_d : Representative Flow Velocity (m/s)

a, β : Factor Determined by the Block Shape (see **Table 6.3.37**)

ρ_b : Block Density (kgf*s^2/m^4)

ρ_w : Water Density, 102 (kgf*s^2/m^4)

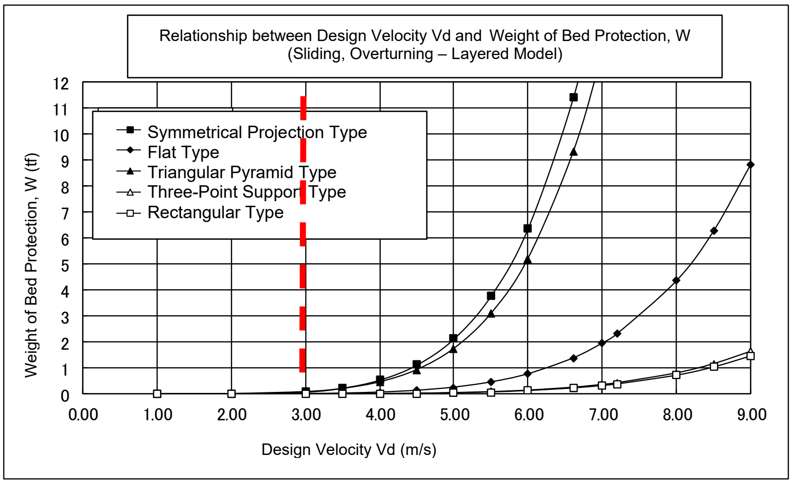
Table 6.3.37 Coefficient α and β of Atypical Concrete Block

Block type	specific gravity of the model block	$\alpha \times 10^{-3}$	β
A: Target Protrusion	ρ_b/ρ_w=2.22	1.2	1.5
B: Planar	ρ_b/ρ_w=2.03	0.54	2.0
C: Triangular	ρ_b/ρ_w=2.35	0.83	1.4
D: three-point support	ρ_b/ρ_w=2.35	0.45	2.3
E - Rectangular	ρ_b/ρ_w=2.09	0.79	2.8

Note: Study Team translated from Dynamic Design Method of Revetment"

The required weight is examined using the flow rate obtained by the hydraulic model experiment. In the hydraulic model experiment, the maximum flow velocity is about 3.0 m/s in the downstream side of the MCGS with end sill.

The target blocks are 5 kinds which are "Symmetrical Projection Type", "Flat Type", "Triangular Pyramid Type", "Three-Point Support Type" and "Rectangular Type". The required weight for each type is calculated using the flow velocity as a parameter and it is shown in **Figure6.3.33**. According to this, in case of the flow velocity of 3.0 m/s, 1 t is sufficient in any block type.



Source: Survey team calculated.

Figure6.3.33 Relationship Between Design Flow velocity and Weight of Bed Protection

(ii) Block Weight Settings

When selecting a block type, a flat block 2t type is selected for the following reasons.

Since the required weight of the bed protection block is proportional to the sixth power of the flow velocity, the block may be flown away due to a slight difference from the experimental value, and it is safer to consider extra allowance. The flat block has a shape on which smaller sweep force acts and can be expected to resist a high flow velocity even if the flow velocity increases. It is also a type introduced in TYPICAL DESIGN DRAWINGS FLOOD CONTROL STRUCTURE (March 2003, DPWH).

(c) Study on Net Gabion for Bed Protection

(i) Study Concept

In order to ease the steep scouring of the riverbed due to the flow and to prevent the settlement of the foundation and wash out of the soil particles from the slope, bed protection with net gabion would be proposed. The design of net gabion is in accordance with “Dynamical Design Method of Revetment, Japan.”

The net gabion is studied considering that “Traction- Gabion” model can be applied.

With respect to the representative flow velocity V_0, movement of the filling material that causes deformation of the basket in the basket shape is not allowed in principle. Therefore, a verification is performed so that the filling material does not move due to the dimensionless tractive force. Here, the required diameter of the material is determined based on the previous experimental results for the dimensionless tractive force.

(ii) Dimensionless Tractive Force

From the results of the University of Colorado experiments, the dimensionless tractive force is:

In case of the deformation of the basket not allowed : $\tau_{*d}=0.10$

In case of the deformation of the basket allowed : $\tau_{*d}=0.12$

The above values are on a horizontal floor. (Even when the basket is piled up in a step shape, the calculation is performed under the condition that the basket is placed flatly.) In case of net gabion, the dimensionless sweeping force that allows some deformation is used.

(iii) Required Weight of Net Gabion

The value of the critical flow velocity for net gabion shown in Table 6.3.37 has been obtained from the past hydraulic model tests.

From the hydraulic model experiment results in this project, the flow velocity in the downstream of the energy dissipating is expected 3.0 m/s or less. Therefore, according to Table 6.3.37, the weight of the net gabion to be used is 2t type.

Table 6.3.38 Critical Flow Velocity for Net Gabion by the Past Hydraulic Model Experiment (m/s)

Weight	Single	Group
1t	2.58	3.78
2t	2.90	4.35
3t	3.10	4.65
4t	3.40	5.10

Source : River Bureau-related documents ; Advanced Construction Technology Center: Design and construction technical manual for bag-type foot protection work (draft), ACTEC Technical No. 97502, 1996, pp 51-54

(iv) Calculation for Size of Filling Material

A. Condition for Stability

About the diameter of the filling material, the following formula is practical.

$$\tau_{*d} = \frac{u_*^{\,2}}{s \cdot g \cdot D_m}$$

$$u_* = \frac{V_0}{\phi}$$

$$\phi = 6.0 + 5.75 \log_{10}\left(\frac{H_d}{k_s}\right)$$

Where,

u_* : Friction Velocity
s : Specific Gravity of Riverbed Material in Water
g : Acceleration of Gravity. (=9.8m/s^2)
D_m : Diameter of Filling Material
V_0 : Representative Velocity
H_d : Design Depth
k_s : Roughness (Assumed as $2.5 \cdot D_m$)

ks is assumed as 2.5・Dm, and the required diameter is obtained by the repeated calculation.

B. Design Calculation

The design condition is set as follows.

Nondimensional Tracie Force	0.12
Representative Velocity	3.00m/s (Based on the Hydraulic Model Test)
Specific Gravity of Riverbed Material in Water	1.65
Design Depth	3.00 m

$$D_m \geqq \frac{V_0^{\,2}}{\left\{6.0 + 5.75\log_{10}\left(\frac{H_d}{k_s}\right)\right\}^2 \cdot \tau_{*d} \cdot s \cdot g}$$

(Left Side) (Right Side)

0.017 ≧ 0.017

Therefore, it is calculated that a filling material having a diameter of 1.7 cm or more could be used. On the other hand, the minimum particle size used as the filling material must be larger than the mesh of the bag to be used, and the particle size of the filling material based on this is 5 cm to 15 cm.

Hence, the diameter of filling material is set to 5 cm to 15 cm.

6.3.4 Study on Gate Structure and Hoist

6.3.4.1 Study on Gate Structure

(1) Gate Structure

The structure of the gate leaf of the fixed wheel roller gate is classified into the following two:

1) Shell Structure

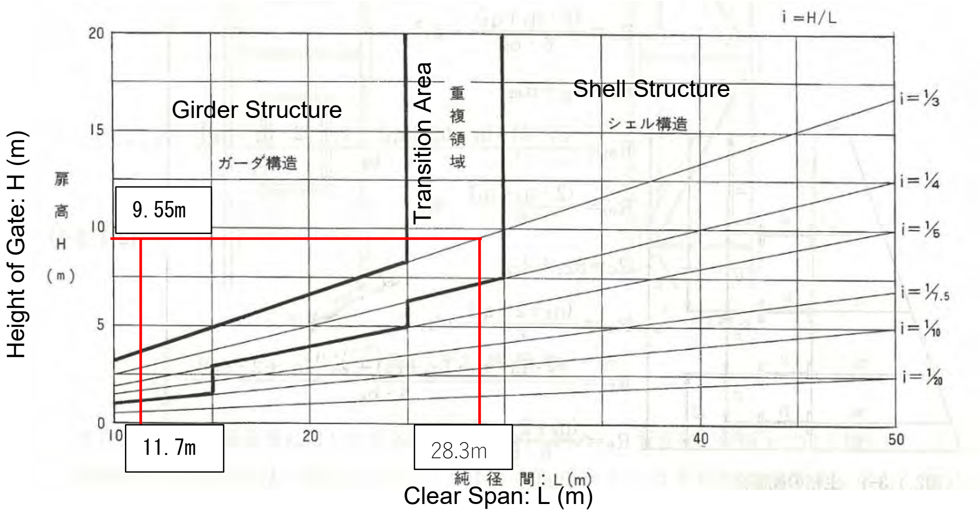
With the entire gate leaf as a shell structure, it resists bending, torsion, shear, deflection, etc., due to water pressure, etc.

2) Plate Girder Structure

The H-section girder (w/o considering support by skin plate), which subdivides the back surface of the waterproof skin plate and arranges it in a grid pattern, resists bending, torsion, shearing, deflection, etc., and is a structure commonly used for river gates between medium and small diameters.

Statistical data of actual values concerning the dimensions of river gates (Clear span, gate height, and ratio between gate height and clear span) and the applicable span, division of gate height, and region of the structure of the gate leaf are shown in **Figure 6.3.34** The ratio between the gate height and the clear span of the gate in case of MCGS is calculated as follows.

- Wider Span Gate : About 1/2.96 (H/L = 9.55/28.3 ≒ 1/2.963)
- Narrower Span Gate : About 1/.22 (H/L = 9.55/11.7 ≒ 1/1.225)



Note: Translated by the Study Team from the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

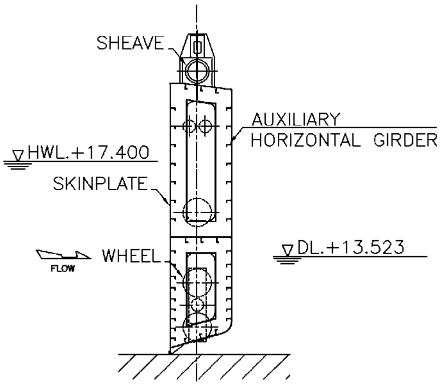
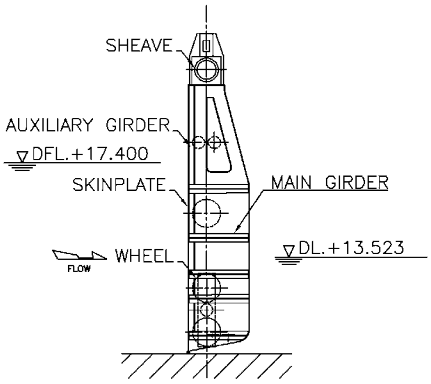
Figure 6.3.34 Gate Dimensions and Structure Diagram

In accordance with **Figure 6.3.34** and based on the actual samples, the wider span gate is located on the overlapping area.

Considering the local site conditions of MCGS, the structures of the wider span gate are compared. **Table 6.3.39** shows the comparison of the gate structures of wide span gate. In accordance with this table, since sedimentation on the girder of the gate leaf is less likely to occur and it is superior in production cost, the shell structure is adopted.

On the other hand, the gate structure of narrower span gate is a girder structure based on the previous samples. Since the narrower span gate is basically fully open both in ordinary time and during flood, sedimentation on the girder of gate leaf is not an issue.

Table 6.3.39 Comparison of Structure of Gate(Wider Span Gate, B28.7 m x H9.55 m)

<table>
<tr><th>Item</th><th colspan="2">Shell Structure</th><th colspan="2">Girder Structure</th></tr>
<tr><td>Figure</td><td colspan="2">
</td><td colspan="2">
</td></tr>
<tr><td>General</td><td colspan="2">・ The cross-section of the gate leaf is a closed shell structure</td><td colspan="2">・ Conventional structural type of fixed roller gate</td></tr>
<tr><td>Structure</td><td>・ Due to high rigidity, it is suitable for long span gates.
・ Since the surface of the gate leaf is less uneven, it is hardly affected by sediment deposits and driftwood/garbage.</td><td>◎</td><td>・ Not used for long span gates since the stiffness is lower than that of shell structures
・ The surface of the gate leaf is uneven, and it is easy to trap soil and driftwood/garbage on the main girder.</td><td>○</td></tr>
<tr><td>Landscape</td><td>・ The surface of the gate leaf is not uneven.</td><td>◎</td><td>・ The girder is exposed on the surface of the gate leaf on the upstream side, giving a mechanical impression with its unevenness.</td><td>○</td></tr>
<tr><td>O&M</td><td>・ Externally visible area is limited
・ Internal inspection is also necessary, and it needs time and labor.</td><td>○</td><td>・ All most all part can be observed by the external inspection.
・ Easy to be inspected.</td><td>◎</td></tr>
<tr><td>Economy</td><td><table><tr><th></th><th>Weight (t)</th><th>Construction Cost (Thousand PHP)</th></tr><tr><td>Gate Lead</td><td>155</td><td>*****</td></tr><tr><td>Guide Frame</td><td>40</td><td>*****</td></tr><tr><td>Hoist</td><td>60</td><td>*****</td></tr><tr><td>Total</td><td>255</td><td>*****</td></tr></table></td><td>1</td><td><table><tr><th></th><th>Weight (t)</th><th>Construction Cost (Thousand PHP)</th></tr><tr><td>Gate Lead</td><td>175</td><td>*****</td></tr><tr><td>Guide Frame</td><td>40</td><td>*****</td></tr><tr><td>Hoist</td><td>67</td><td>*****</td></tr><tr><td>Total</td><td>282</td><td>*****</td></tr></table></td><td>2</td></tr>
<tr><td rowspan="2">Evaluation</td><td colspan="2">Hardly affected by sedimentation and driftwood/garbage, and economically superior .</td><td colspan="2">Easy to be affected by sedimentation and driftwood/garbage
Economically inferior</td></tr>
<tr><td colspan="2">Recommended</td><td colspan="2"></td></tr>
</table>

Note: Construction cost is not presented due to the prior released version.
Source: Study Team

(2) Material of Gate

1) Possibility of Brackish Water

(a) Results of the Previous Water Quality Test

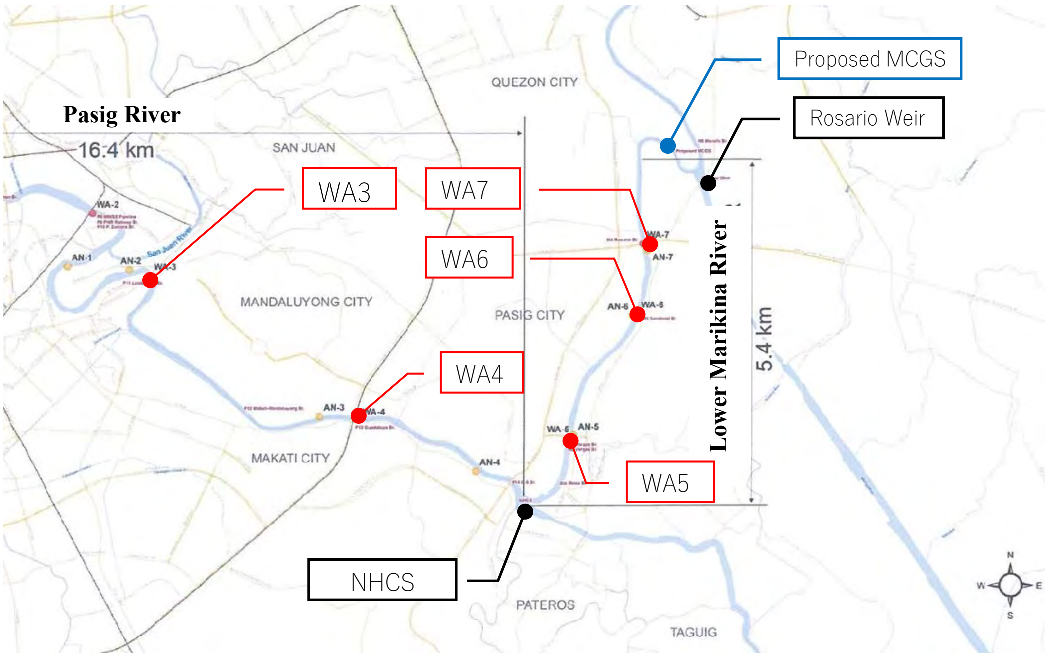
In Lower Marikina River basin near MCGS, a water quality test was conducted during the construction period of PMRCIP-III (**Figure 6.3.35**). **Table 6.3.40** show that the Pasig and Marikina rivers have brackish water condition depending on the season. In particular, Pasig River has a high salinity, and values in brackish water are observed every year. On the other hand, Marikina River, although brackish, has a small value and is not every year.

Table 6.3.40 Salinity in Previous Water Quality Test

River	Sampling Point	Sampling Month	Salinity	Sampling Month	Salinity	Sampling Month	Salinity
Pasig River	WA-3 Lambingan Bridge	Sept. 2014	0.3‰	Feb. 2015	0.3‰	May 2015	**23.33‰**
		Aug. 2015	0.03‰	Nov. 2015	0.2‰	Feb. 2016	0.4‰
		May 2016	**14.4‰**	Aug. 2016	**14.9‰**	Nov. 2016	**11.91‰**
	WA-4 Guadalupe Bridge	Sept. 2014	0.3‰	Feb. 2015	0.3‰	May 2015	**8.99‰**
		Aug. 2015	0.01‰	Nov. 2015	**0.5‰**	Feb. 2016	0.4‰
		May 2016	**11.0‰**	Aug. 2016	**17.3‰**	Nov. 2016	**12.79‰**
Marikina River	WA-5 Vargas Bridge	July 2014	0.4‰	Mar. 2015	0.1‰	Jun. 2015	0.3‰
		Sept. 2015	0.02‰	Jan. 2016	0.1‰	Mar. 2016	0.0‰
		June 2016	0.3‰	Sept. 2016	**3.9‰**	Dec. 2016	**3.9‰**
	WA-6 Alfonso-Sando Bridge	July 2014	0.3‰	Mar. 2015	0.29‰	Jun. 2015	0.3‰
		Sept. 2015	0.0‰	Jan. 2016	0.1‰	Mar. 2016	0.0‰
		June 2016	0.3‰	Sept. 2016	**3.2‰**	Dec. 2016	**3.6‰**
	WA-7 Rosario Bridge	July 2014	0.4‰	Mar. 2015	0.1‰	Jun. 2015	0.3‰
		Sept.2015	0.02‰	Jan. 2016	0.1‰	Mar. 2016	0.0‰
		June 2016	0.3‰	Sept. 2016	**3.4‰**	Dec. 2016	**3.6‰**

: Brackish Water (Salinity of 0.5% to 30%)

Note: Prepared by the Study Team from data in the Annual Monitoring Report (Woodfields Consultants. Inc.)

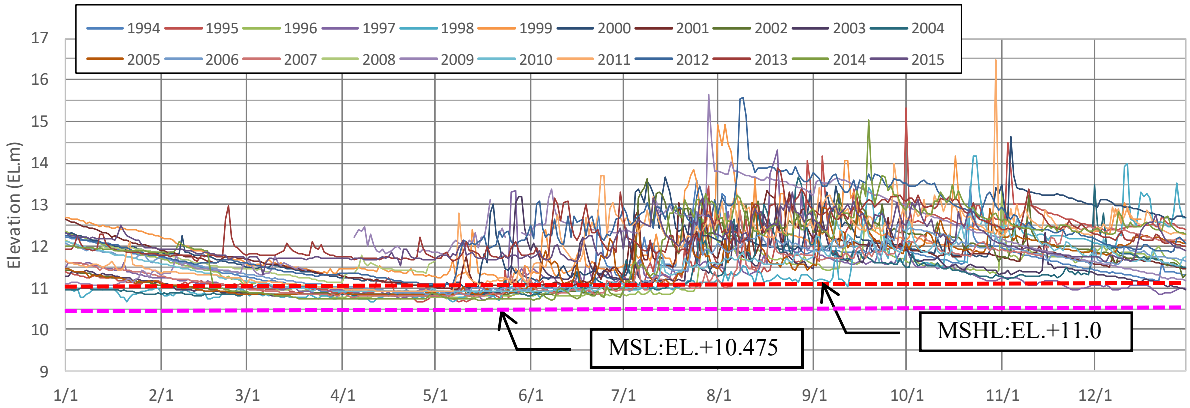


Note: Prepared by the Study Team from data in the Annual Monitoring Report (Woodfield Consultants. Inc.)

Figure 6.3.35 Water Sampling Locations

(b) Past Observed Water Level

Figure 6.3.36 shows the observed water level on the Marikina River side at the Rosario Weir for the past 20 years. Based on **Figure 6.3.36**, the water level is relatively low from December to July every year. In particular, the observed water level from March to mid-July is lower than the average MSHL EL.+11.0m in the Manila Bay in some years. It can be considered that a certain time has lapsed before backflow from the Manila Bay to Rosario Weir and saltwater intrusion occurred.

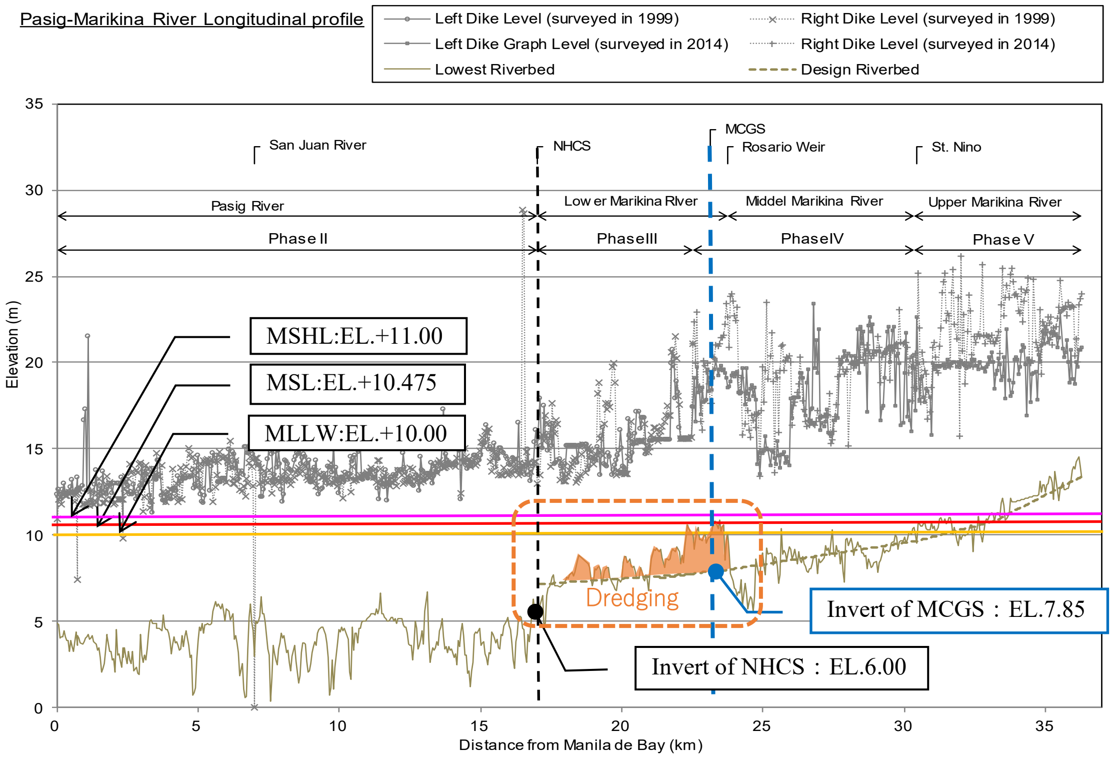


Note: Prepared by the Study Team based on data provided by LLDA.

Figure 6.3.36 Water Level Data at the Rosario Weir, Marikina Side in the past 20 year

(c) Relationship between Riverbed and Sea Level

Figure 6.3.37 shows the riverbed and the mean sea high water level (MSHL), mean sea level (MSL) and mean lower low water level in Manila Bay.



Source: Study Team

Figure 6.3.37 Relation between Riverbed and Sea Water Level

Currently, dredging work is being carried out in the downstream side of Rosario Weir in Marikina River, and when MCGS is installed, the riverbed around MCGS will become EL. +7.85m, which is the same invert elevation as the weir. As shown in **Figure 6.3.37**, the lowest level of the current riverbed height around the MCGS is around EL. +10.0m. However, the riverbed will become lower than the MLLW due to the dredging work, so that the brackish water around the MCGS will definitely become more serious than the present condition.

The NHCS installed at the confluence of the Marikina River and the Napindan Channel has the invert elevation of EL. 6.0 m. The gate leaf has been severely corroded, and it was confirmed that condition of brackish water is very serious[2].

(d) Summary

The situation described above can be summarized as follows:

- Data shows that the MCGS site and the Rosario Weir site contain brackish water, however, it is not every year. As the riverbed in the Lower Marikina River ranges from EL+9.0m to EL+10.0m, it is estimated that there is little saltwater intrusion in the current situation in which dredging work of the Marikina River has not been completed. (According to an EFCOS engineer, saltwater intrusion is sometimes observed in the dry season.)
- After the completion of the dredging work in the Marikina River, the riverbed around the MCGS will be as low as EL+7.85m, and there is a high possibility of salinity intrusion.

From the above, when selecting the material of the gate leaf of MCGS, the brackish water is considered.

2) Comparison of Gate Material

The gate material is selected from the following three materials. Stainless steel of the comparison subject is applicable even in brackish water.

1. Carbon Steel (Rolled steel for welded structures, SM 400)
2. Conventional Stainless Steel (SUS 316)
3. Alloy Saving Duplex Stainless Steel (SUS 323 L)

The comparison of economic efficiency is based on the lifecycle costs (LCC) including maintenance. Also, regarding the initial cost, the design cost by the Japanese company is considered for all materials. In addition, regarding stainless steel, gate fabrication at a factory in the Philippines will be carried out under the supervision of a Japanese company. (The same consideration was adopted in the construction of floodgates in Thailand in 2016.)

From the comparison given in **Table 6.3.41**, the alloy saving duplex stainless steel (SUS 323L), which has no problem in its brackish water applicability and its superior lifecycle cost is recommended

[2] Final Report on the Study on Economic Partnership Projects in Development Countries in FY2017 Study on Floodgates Rehabilitation Project in Metro Manila, Republic of the Philippines February 2018, METI

Table 6.3.41 Comparison of Gate Materials for the MCGS

Item	Carbon Steel（Rolled steel for welded structuresSM400）	Conventional Stainless Steel（SUS316）	Ally Saving Stainless Steel（SUS323L）
General	・ Material commonly used for gate leaf ・ Proof Stress：245 N/mm^2 ・ Tensile Strength: 400～510 N/mm^2	・ Stainless steel commonly used in blackish water ・ Proof Stress：175 N/mm^2 ・ Tensile Strength: 500 N/mm^2	・ Stainless steel with high strength used in blackish water ・ Proof Stress：400 N/mm^2 ・ Tensile Strength: 600 N/mm^2
Weight	Gate Leaf 210+85＝295 t(1.00) Guide Frame 40+20＝60 t(1.00) ＋Hoist 80+35＝115 t(1.00) 470 t(1.00)	Gate Leaf 175+70＝240 t(0.83) Guide Frame 40+20＝60 t(1.00) ＋Hoist 70+20＝90 t(0.78) 395 t(0.84) ・ Since no corrosion margin thickness is required, it is lighter than carbon steel.	Gate Leaf 155+65＝220 t(0.76) Guide Frame 40+20＝60 t(1.00) ＋Hoist 60+25＝85 t(0.74) 365 t(0.77) ・ Due to no corrosion margin and high strength, it is lighter than the conventional stainless steel.
Maintenance	・ Generally, repainting is required at least once every 10 years, however it is considered that repainting is carried out every year based on the past record of the Rosario weir. ・ Repaint Cost:370,000 PHP /times (Calculated from actual results of Rosario Weir)	・ Repainting is not needed	・ Same as the left
Applicability in Blackish Water	・ In the Pasig Marikina River, there is an issue on the resistance of carbon steel to brackish water considering the corrosion of gate leaf of existing NHCS (Invert Elev.: EL. + 6.0) located in the downstream.	No Problem	No problem
LCC (for 50 years) (in Thousand PHP)	Gate Leaf ***** Guide Frame ***** Hoist ***** ＋Repainting ***** Total ***** (1.00)	Gate Leaf ***** Guide Frame ***** Hoist ***** ＋Repainting 0 Total ***** (1.06)	Gate Leaf ***** Guide Frame ***** Hoist ***** ＋Repainting 0 Total ***** (0.98)
Technical Novelty	・ Gate fabrication using carbon steel has also been carried out in the Philippines, and there is little technological novelty.	・ Stainless steel processing is also carried out in the Philippines, however there is no experience of its application to large gates. Hence it is new technology for river structures	・ Same as the left, in addition to the new technology for river structures, the processing of new materials in the Philippines is beneficial for the promotion of industrial production technology in the Philippines.
Evaluation	・ Considering the corrosion conditions of the existing NHCS, it is definitely difficult to use the same gate leaf for 50 years even with proper painting. ・ As a result, lifecycle costs are higher than alloy saving duplex stainless steel	・ It is considered that it can be used in good condition for about 50 years. ・ LCC is highest	・ It is considered that it can be used in good condition for about 50 years. ・ Provide LCC advantages over alternatives
			Recommended

Note: Cost is not presented due to the prior released version. Source: Study Team

(3) Water Sealing System

1) Design Procedure

The design of the water sealing section shall be in accordance with the "Design Guideline for Floodgate and Sluiceway Gate" in Japan.

2) Result of Selection

(a) Selection of Water sealing Method (1)

This gate is a diversion weir and has a front water sealing system for the purpose of blocking the forward flow from the upstream.

(b) Selection of Water sealing Method (2)

In order to shield the upstream water flow, it should be three-way water sealing.

(c) Selection of Water sealing Type

Since it is a weir gate, it is rubber-water sealing which has advantages in water tightness and traceability.

(d) Examination of the Shape of Water sealing Rubber

Since it is a three-way water sealing gate, the side part is L-shaped, and the bottom part is flat.

(e) Examination of Water sealing Rubber Materials

Synthetic rubber with good weather resistance and oil resistance which is common in floodgate gates is used.

6.3.4.2 Study on Type of Hoist

(1) Primary Comparison (Type of Hoist)

Figure 6.3.38 shows the types of hoist commonly used.

Transmission System	Major Equipment of the Drive Unit	Connection with Gate Leaf	Type of Hoist
Mechanical	Drum Drive	Wire Rope	Wire Rope Winch Type
	Spindle Drive	Spindle	Spindle Type
	Rack Drive	Rack Bar	Rack Type
	Chain Drive	Chain	Chain Type
Hydraulic	Hydraulic Cylinder	Piston Rod	Hydraulic Cylinder Type
	Hydraulic Cylinder	Wire Rope	Hydraulic Cylinder Wire Rope Type
	Hydraulic Motor Drum Dive	Wire Rope	Hydraulic Motor Wire Rope Type
	Hydraulic Motor Rack Dive	Rack Bar	Hydraulic Motor Rack Wire Rope Type

Note: Translated by the Study Team from the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

Figure 6.3.38 Types of Hoist

This figure is classified by the combination of the power transmission system to the drive unit, the major equipment of the drive unit, and the connection with the gate leaf.

This is a gate facility for weir, and originally, it is not necessary to select a gate that has a lowering function by its own weight such as a hoist for tsunami floodgate. However, adaptability to remote control and reserve power is important. In addition, considering the local characteristics of Metro Manila and concerns about delays in initial response to floods, it is necessary to consider the possibility of operation by means of a self-weight as an emergency operation.

Considering the above, the type satisfying the functional characteristics, economic efficiency, maintainability, landscape, etc., the hoist is selected by comprehensively evaluating the head, space for installation, environment, etc.

1) Wider Span Gate

The wider span gates are extremely large gates with a clear span of 28.3 m x effective height of 9.55 m, a gate leaf area of 270.3 m^2, and a gate leaf weight of about 170 ton. In the case of such a gate facility, the following four items are generally selected:

- Wire Rope Winch Type
- Chain Type
- Hydraulic Cylinder Type
- Hydraulic Cylinder Wire Rope Type

The above four types can cope with the winding load required in this facility and have sufficient examples and high reliability in closing operation. Therefore, in the study on the type of hoist in this facility, a comparative study is carried out with respect to the above-mentioned four types of hoist types. The major reasons for excluding the other 4 formats (Spindle, rack, hydraulic motor wire rope, hydraulic motor rack) from the items are as follows.

[Reasons for Exclusion]

- Spindle Type Hoist：This type is mainly used for small gates with small hoisting load and equipment for flow rate adjustment with high frequency of use. Not applicable to the size of the facility.
- Rack Type Hoist：It is a type used for small to large gates. However, it cannot be adopted at this scale.
- Hydraulic Motor Rack Type：Not available on this scale.
- Hydraulic Motor Wire Rope Winch Type：Since an extremely large hoisting output can be generated in a relatively small space, this type is mainly used for large gates. However, currently it is rarely used for river gate facilities in Japan.

Table 6.3.42 shows the comparison switches. The wire rope winch type is selected as the hoist type. The reasons are as follows:

- The wire rope winch type has a lot of samples and it is a technically well-established type.
- It has a self-weight lowering function and can be lowered also by remote control.
- Unlike the hydraulic type, it is not necessary to replace a large amount of oil periodically, hence, maintenance is easy.
- Due to its simple structure, it is easy to maintain and also economical.
- Operation Bridge is not needed.

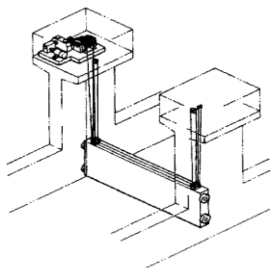
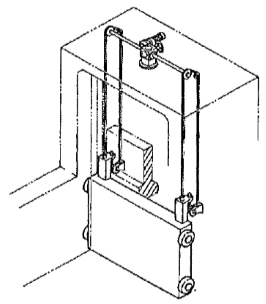
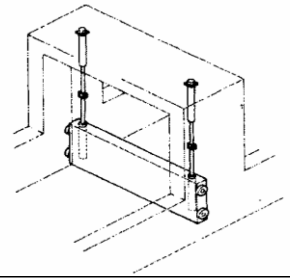
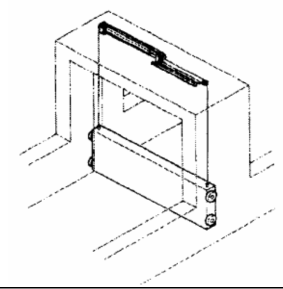
2) Narrower Span Gate

The narrower span gate is classified as a large gate with a clear span of 11.7 m x effective height of 9.55 m, a gate leaf area of 111.7 m^2, and a gate leaf weight of about 60 t. In the case of such gate facility, the above-mentioned four types are selected

According to **Table 6.3.42**, the wire rope winch type is selected as the hoist. The reason is the same as the case of the wider span gate.

- The chain type has less examples; however, there are examples with 1500 kN of hoisting load.
- It has a self-weight lowering function and can be lowered also by remote control.
- Unlike the hydraulic type, it is not necessary to replace a large amount of oil periodically, hence, maintenance is easy.
- Due to its simple structure, it is easy to maintain.

Table 6.3.42 Comparison of Hoist

	Mechanical		Hydraulic	
	Wire Rope Winch Type	Chain Type	Hydraulic Cylinder Type	Hydraulic Cylinder Wire Rope Type
Figure (Extracted from "Floodgate Engineering")	 (1M1D Winch Type)			
General Description	It consists of a wire rope and a winch type hoist, and the hoist force generated by the winch is transmitted to the wire rope to raise and lower the gate leaf.	It consists of a chain and hoist, and hoisting force generated in the hoist body is transmitted to the chain to raise and lower the gate leaf	A structure in which the oil pressure generated by a hydraulic unit is transmitted to a hydraulic cylinder via hydraulic operating oil to expand and contract the hydraulic cylinder.	A type in which wire sheaves are installed at both ends of a hydraulic cylinder, a wire rope is wound between the wire sheaves, and the wire rope is wound using the expansion and contraction of the hydraulic cylinder.
Maji Equipment	Wire Rope Winch Type Hoist Main Motor. Decelerator, Drum, Manual Operation Device, Spare Engine, Switching Device, Hydraulic Push-up Brake, Self-weight Lowering System (with Fan Brake), Wire Rope, Wire Sheave	Chain Type Hoist Motor, Decelerator, Regulator, Manual Operation Device, Opening Meter, Integrated Limit Switch, Chain, Sprocket, gear axis	Hydraulic Cylinder Type Hoist Hydraulic Unit (Driving Source), Hydraulic Cylinder (Elevating) Hydraulic Piping, Hydraulic Fluid, etc.	Hydraulic Cylinder Wire Rope Type Hoist Hydraulic Units (Driving Source), Hydraulic Cylinders (Elevating), Wire Sheaves, Hydraulic Piping, Hydraulic Fluids, and Other Wire Ropes
Installation Space	・ Since the wire winding drum and the decelerator are placed on the deck, the required installation space is increased, however there is no projection upward, and the installation height can be suppressed.	・ Since the integrated hoist and the chain sprocket (transmission gear) are smaller than the wire drum, the required space is small, there is no upward projection, and the installation height can be suppressed. However, since a gear axis is required, a space for that portion is required.	・ The installation space is the smallest when the hydraulic cylinder stands upright, however the upward projection is the largest opening/closing device. ・ A separate hydraulic unit must be installed, and the space for installation must be secured.	・ Only by installing the hydraulic cylinder horizontally, the depth becomes small and the installation space is slightly small. There is no upward projection and the installation height can be reduced. ・ A separate hydraulic unit must be installed, and the space for installation must be secured.
	○	○	○	○
Influence on the substructure work	・ Only the dead weight of the gate leaf is the closing force (compressive stress). ・ Since the hoisting load is transmitted to the substructure on the bottom surface of the operation deck, a large-scaled reinforcement of substructure is unnecessary. ・ The weight of the hoist is slightly heavier than that of the chain type for the drum and the frame.	・ Only the dead weight of the gate leaf is the cofferdam force (compressive stress). ・ Since the hoisting load is transmitted to the substructure on the bottom surface of the operation deck, a large-scaled reinforcement of substructure is unnecessary. ・ The weight of the hoist itself is light ・ In case of independent floodgate structure, due to the operation bridge for the shaft, it becomes heavier.	・ Since the pressing force by the hydraulic cylinder acts, tensile stress as well as compressive stress acts on the substructure. ・ Since stress is generated intensively in the hydraulic cylinder installation part, reinforcement of the substructure may be required. ・ The weight of the hoist itself is light.	・ The closing force (compressive stress) is only the weight of the gate leaf. ・ In some cases, reinforcement of the substructure is required since the load concentrates on the turning part by the wire sheave. ・ The weight of the opening/closing device is slightly heavier than that of the hydraulic cylinder by the wire and sheave.
	○	△	○	○
Response to Earthquakes	・ Since the center of gravity of the hoist is low, it is sufficient to have a bearing part (fixed part) which can withstand the inertial force in the earthquake condition. ・ Operation is possible, since there are many examples at the tsunami floodgate.	・ Since the center of gravity of the hoist is low, it is sufficient to have a bearing part (fixed part) which can withstand the inertial force in the earthquake condition. ・ Aseismatic connector for the operation bridge is needed.	・ Since the hydraulic cylinder is installed in the vertical direction, the center of gravity of the hoist is high, and it is easily affected by the inertial force of earthquake.	・ Since the center of gravity of the hoist itself is low, it is sufficient to have a bearing part (fixed part) which can withstand the inertial force by an earthquake.
	○	△	△	△
Reserve Power	・ The motor is the standard main power. ・ A manual operation handle is normally installed; however, this is not practicable. Hence, a spare engine is also installed. ・ Correspond with a spare generator	・ The motor is the standard main power. ・ A manual operation handle is normally installed; however, this is not practicable. ・ Correspond with a spare generator	・ The motor is the standard main power of the hydraulic unit. ・ Correspond with a spare generator	・ The motor is the standard main power of the hydraulic unit. ・ Correspond with a spare generator
	◎	◎	○	○
Closing Force	・ The closing force depends only on the weight of the gate leaf. ・ When the weight of the gate leaf is light, it is necessary to add weight to it, however it is not necessary in this scale.	・ Same as the left	・ Since the gate leaf can be forcibly pressed to stop water, the closing force is high. ・ Since the forced clamping force of the hydraulic cylinder type is large, sufficient strength checking of the substructure and the gate leaf is required.	・ The closing force depends only on the weight of the door body. ・ When the weight of the gate leaf is light, it is necessary to add weight to it, however it is not necessary in this scale.
	○	○	◎	○
Landscape Character	・ The mechanical portion is not exposed to the outside since it can be placed in the operating room. ・ The position where the hoist is installed can be set to a place other than the slab by extending the wire rope. ・ The wire rope is exposed	・ The mechanical portion is not exposed to the outside , since it can be placed in the operating room. ・ The chain is exposed ・ Operation bridge and the shaft is exposed in the air.	・ Since the hydraulic cylinder stands out significantly , it cannot be said that the landscape is good. However, it is possible to improve the landscape by making it into a monument, as in the case of the Naruka weir. ・ Design on the slab is limited by the hydraulic cylinders standing out.	・ The mechanical portion is not exposed to the outside since it can be placed in the operating room. ・ The position where the switching device is installed can be set to a place other than the slab by extending the wire rope. (Facilitate landscape design) ・ The wire rope is exposed.
	◎	◎	△	○

	Mechanical		Hydraulic	
	Wire Rope Winch Type	Chain Type	Hydraulic Cylinder Type	Hydraulic Cylinder Wire Rope Type
Maintenance	・ The number of machines is the largest among alternatives. However, since the structure is simple, maintenance is relatively easy. ・ Periodic replacement of lubricating oil is necessary, however not as much as in the hydraulic system. ・ It is necessary to apply grease to the wire rope on a regular basis. ・ it is necessary to replace the wire rope about once every 15 years.	・ Since the hoist is standardized, maintenance is easy. ・ It is necessary to change the lubricating oil for gears in the hoist periodically, however the amount is less than it of the other types. ・ Lubricating oil is required for sprockets provided on the gate leaf side. ・ Regular application of grease to the chain is required, however, replacement of the chain is unnecessary.	・ Although the number of single machine parts is small, the parts in each single machine part are very precise and need to be checked carefully. ・ It is necessary to change a lot of hydraulic fluid periodically.	・ Although the number of single machine parts is the second smallest after the hydraulic cylinder type, the parts in each single machine part are very precise and need to be checked carefully. ・ It is necessary to change a lot of hydraulic oil periodically and also to apply grease to the wire rope periodically.
	○	◎	△	△
Actual Examples	・ There are many examples in this scale.	・ This is a relatively new type, Hence, there are not so many examples (released in 1997). ・ Hoisting Capacity is up to 1600 kN.	・ Small Numbers of Examples	・ Small Numbers of Examples
	◎	△	△	△
Production Cost	100%	110% (up to 1600 kN)	145%	115%
	◎	○	△	△
Evaluation	・ The facility is equipped with the necessary functions and can be easily adapted to landscape design. ・ Superior in economics ・ Many examples and reliable	・ The facility is equipped with the necessary functions. However from the aspect of landscape design, slightly inferior to the left due to the operation bridge. ・ In spite of excellent economy and maintainability, the specific expertise is needed. ・ In case of wider span gate, the capacity is insufficient. ・ Not many examples.	・ It has excellent functionality, such as a self-weight lowering function and water tightness by forcing. ・ The length of stroke exceeds more than 10 m, and it is not practical. ・ Less economical and less maintainable. ・ Small Numbers of Examples	・ The facility is equipped with the necessary functions and can be easily adapted to landscape design. ・ The length of stroke exceeds more than 10 m, and it is not practical. ・ Less economical and less maintainable. ・ Small Numbers of Examples
	Recommended	Not Recommended	Not Recommended	Not Recommended

Legend : ◎...Better、○...No Problem、△...There is issues to be solved
Source: Study Team

(2) Secondary Comparison (Wire-Rope Winch Type)

An overview of the structure of each type of wire rope winch shown in **Table 6.3.44**.

The following three types of wiring are considered for the wire rope type winch.

- 1 motor 2 drum winch type (1M2D)
- 1 motor 1 drum winch type (1M1D)
- 2 motor 2 drum winch type (2M2D)

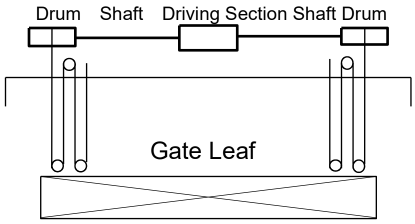
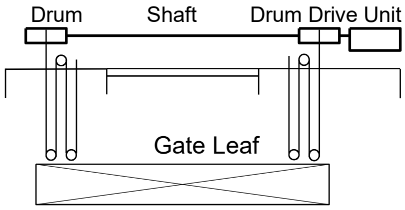
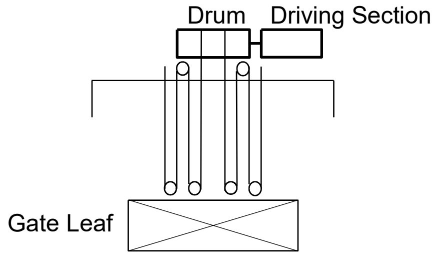
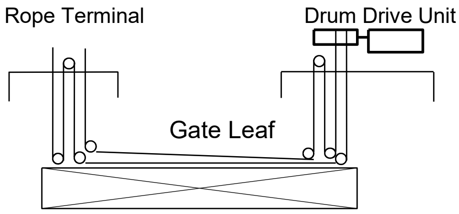
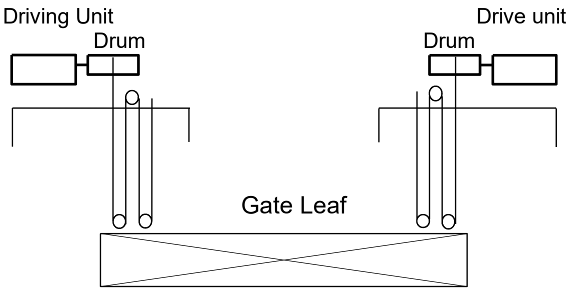
Further, in consideration of the arrangement of a driving unit (motor, reduction gears and other power generators) and a driven unit (Drums, gears, wire ropes, etc.) of a wire rope winch type motor or a drum and the components are classified as follows.

Table 6.3.43 Wire Rope Winch Types and Placement

	Arrangement of Drum And Driving Part	Position of Drum and Drive Unit
1 Motor, 2 Drum Type (1M2D)	Center Drive Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
	Single Drive Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
1 Motor, 1 Drum Type (1M2D)	Central Drum Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
	One Side Drum Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
2 Motor, 2 Drum Type (2M2D)	Double Drum Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column

Source: Study Team

Table 6.3.44 Structure of Wire Rope Winch

Wire Rope Winch Type	Schematic Structural Plan	
1 Motor, 2 Drum Type This type is the most common type in which a drum is arranged at both ends and connected to a power transmission shaft to wind up wire ropes on the left and right sides of a gate leaf. There are two types of drive units, the center drive type and the single drive type. The center drive type is common, but the single drive type is adopted in the independent type of column when the hoisting devices are concentrated on the one side of the columns.	Center Drive Type	
	Single Drive Type	
1 Motor, 1 Drum Type It is a type in which the components of a hoisting device are integrated in one location, and there are a central drum type in which a drum is arranged in the center of the gate leaf, and a one side drum type in which a drum is arranged on one side of the gate leaf. The center drum type is adopted when the lifting span of the gate leaf is narrow (Approx. 10 m or less), but rack type and spindle type are mostly adopted in such scale. In addition, the one side drum type has the advantage that the stage for hoisting device between the columns is not required and the hoisting device can be concentrated on the one-side, and is often used for gates with a span of about 10 m to 25 m.	Central Drum Type	
	One Side Drum Type	
2 Motor, 2 Drum Type Generally, this type is adopted for gates (& weirs) with a span of about 25 m or more, and one motor and one drum type hoisting deice is arranged at both ends of the gate leaf, and the gate leaf is opened and closed with electrically adjusting the opening. Since it is difficult to synchronize mechanically, the risk of lifting on only one-side is higher than other types. Therefore, direct connection of an internal combustion engine or installation of a self-weight closing device shall be avoided. It is more expensive than other types.	Double Drum Type	

Source: study team

1) Wider Span Gate

Since the reliability of complete closure is the most important function of this gate during floods, the type of wire rope winch is also required to have a simple mechanism and high reliability in opening and closing operations.

The operation deck is independent on the left and right sides, and the span is 28.5 m. As shown in , a two-motor, two-drum type (2M2D) is adopted which is usually used in case of a span of 25 m or more.

2) Narrower Span Gate

Since the reliability of complete closure is the most important function of this gate, the type of wire rope winch is also required to have a simple mechanism and high reliability in opening and closing operations.

Of the three types, about the 1M2D wire-rope winch type, since the operation deck is an independent type in the left and right, the single drive type would be adopted.

As for the 1M1D wire rope winch type, in case the operation decks are independent, it is structurally difficult to adopt the central drum type. Hence the one side type is chosen.

Considering these factors, the following two types of winch can be adopted.

1. 1M2D Single Drive
2. 1M1D One Side Drum Type

Table 6.3.45 List of Wire Rope Winch Type Hoist

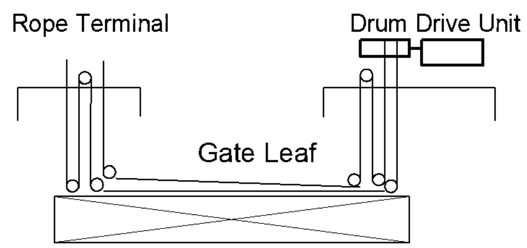
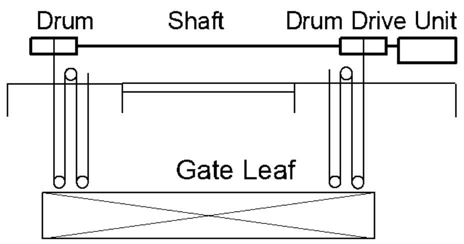
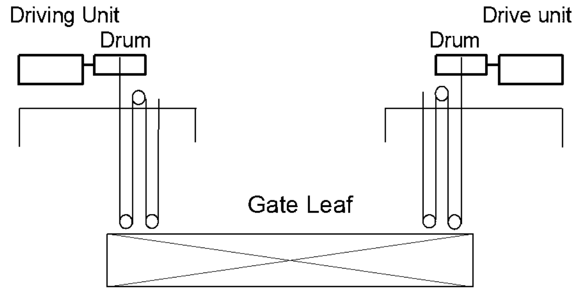
Format	Drum and Placement of Drive	Applicability	
1M2D	Center Drive Type	· Commonly Used Type · Self-Weight Closing is possible. · Lifting on only one side is the least likely to occur, and reliability of Self-weight closing is the highest of the three types.	Inapplicable
	Single Drive Type	· It is commonly adopted in floodgates with a large span and independent column. · Self-Weight Closing is Possible. · Lifting on only one side is the least likely to occur, and reliability of Self-weight closing is the highest of the three types.	○ One of The Alternatives
1M1D	Central Drum Type	· Not used in large gates like the case of this study · Self-weight closing is possible. · Structurally inapplicable to gates with large spans	Inapplicable
	One-Sid Drum Type	· Adopted in large gates · Self-weight closing is possible. · As for securing side-to-side tuning by roping the wires, uncertainty remains in self-weight closing when compared with the mechanical connection method (1M2D Format), and rope adjustment, etc. is difficult and maintenance is inferior.	○ One of The Alternatives
2M2D	Double Drum Type	· Adopted in large gates · Self-weight closing can be performed with a mechanism to synchronize both drums.	Inapplicable (Reference)

Source: Study Team

In the next page, a comparison table of the two types (Also including 2M2D for reference).

Since operation decks are the independent, of the two types, the 1M2D wire rope winch type requires a conduction shaft and the aspect of installation is inferior. Furthermore, the cost becomes also higher than the 1M1D type

Table 6.3.46 Comparison of Wire Rope Winch Type Hoist

Item \ Type	A 1 Motor 1 Drum (1M1D)	B One Side Drive Type 1 Motor 2 Drum (1M2D)	<Reference>: C 2 Motor 2 Drum (2M2D)
outline (positive surface)			
General of Structure	・ Motor is placed on one side ・ Drums and gears are all located on one side ・ One motor and one reduction gear each are installed. (motor output per unit is twice that of “C”.) ・ A wire rope passes through the gate leaf. ・ The smallest number of parts (1/2 times C) ・ Large number of sheaves (More than two ropes than B and C, regardless of the number of ropes)	・ Motor is placed on one side ・ Drums and gears are arranged on both sides. ・ One motor and one reduction gear each are installed. (motor output per unit is twice that of “C”.) ・ The drive transmission shaft crosses between the right and left drums. ・ The transmission shaft must be stiff and durable. ・ The number of parts is between A and C. ・ The number of sheaves used is small.	・ Motor is placed on both sides ・ Drums and gears are located on both sides ・ Motor, reduction gears, etc., all need two. (Output per motor is 1/2 times that of A and B.) ・ No drive transmission shaft is required. ・ It has the largest number of parts. (double A) ・ The number of sheaves used is small.
Operation	・ Slight tendency to become one side lifting ・ No left-right tuning mechanism required ・ Self-weight Closing can be performed.	・ There is no concern of one side lifting. ・ No left-right tuning mechanism required ・ Self-weight closing can be performed.	・ Right and left tuning mechanisms are necessary because there is a tendency of one side lifting due to the uneven rotation speed of the right and left motor. ・ Self-weight closing needs to synchronize left and right with an electric device.
	△	○	△
Installation and Delivery	・ Superior regardless of the gate size due to the smaller number of parts ・ As compared with B and C, parts need to be concentrated on one side, resulting in large blocks, and the necessity of division arises even in relatively small-capacity models.	・ The number of parts is larger due to the intermediate shaft. ・ When the span is long, the transmission shaft becomes long, and it is difficult to install.	・ The number of parts is almost the same as A, and it is advantageous regardless of the size of the span.
	○	△	○
Influence on the Concrete Structure	・ Since the wire rope passes through the inside of the door, a lateral load does not act on the column. ・ One side of the hoist becomes a large block and the operation room becomes large.	・ Since the wire ropes are equally arranged on each side, a lateral load does not act on the Column. ・ Operation deck connecting columns or operation bridge between independent columns is required	・ Since the wire ropes are equally arranged on each side, a lateral load does not act on the Column. ・ Since the left and right blocks are the same, the operation room can be arranged on the same scale.
	△	△	○
Maintenance	・ Since the drive unit is placed on the one side and the numbers of parts are smaller, maintenance is easier. ・ The wire rope in longer and the length passing inside the gate leaf is the longest, it is inferior in the aspect of the maintenance of wire rope.	・ Since the drums and gears are located on both side, maintenance need more manpower and cost due to the larger number of parts. ・ The wire rope can be inspected more easily, it is superior to “A” in the aspect of the maintenance of wire rope.	・ Since driving units are located on both side, maintenance need more manpower and cost due to the larger number of parts ・ The wire rope can be inspected more easily, it is superior to “A” in the aspect of the maintenance of wire rope.
	△	○	○
Landscape Design	・ There is nothing in the span that obstructs the view such as the operation bridge and the operation room, and it is excellent.	・ The operation bridge, the operation room, and other objects in the span obstruct the view.	・ There is nothing in the span that obstructs the view such as the operation bridge and the operation room, and it is excellent.
	○	△	○
Machine Weight	・ The weight of the hoist is the lightest.	・ The weight is increased by the transmission shaft and the operation bridge. Therefore, the larger the span is, the more disadvantageous it is.	・ The steel is heavy. (Increase approximately 20% from A)
	○	△	△
Cost	95%	100%	130%
	1	**2**	**3**
Evaluation	Although it is difficult to maintain wire ropes, this type would be adopted because it is used in many large gates and it does not require a transmission shaft and is inexpensive.	There is no concern of one side lifting, and maintenance of the wire rope is easy, However, the cost is higher than A. It is difficult to install the transmission shaft, it is not adopted.	This type is generally adopted for gates with a span length of 25 m or more. Maintenance and management of wire ropes is easy.
	1	2	**3**

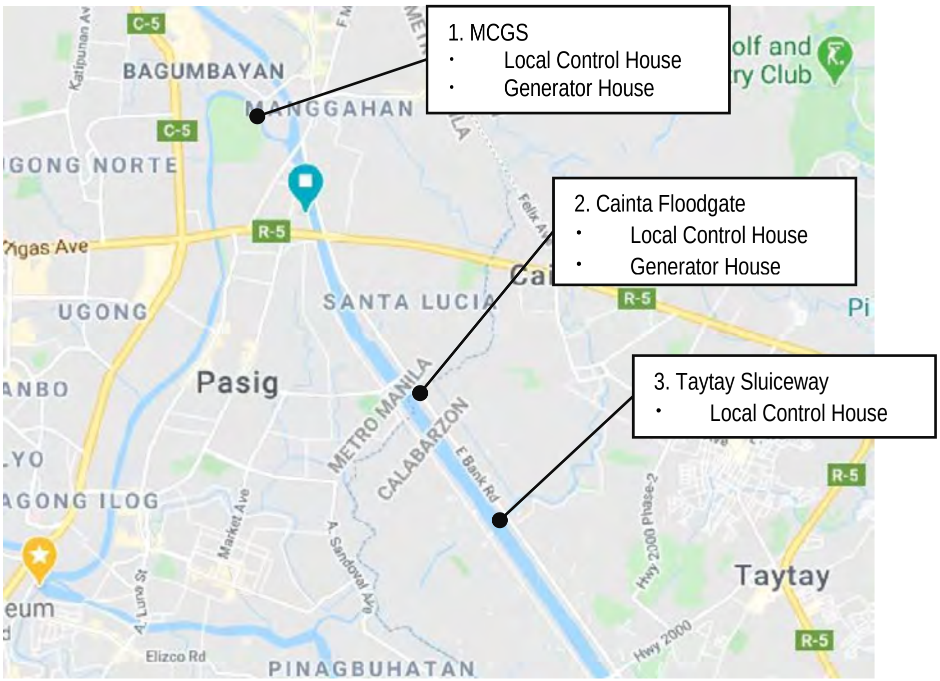
Legend : ◎···Preferable、○···No Problem、△···With Issues to Be Solved、×···Difficult to Adopt

Source: Study Team

6.3.5 System Planning

6.3.5.1 Basic Concept for Operation System of the MCGS

The MCGS shall be properly operated in harmonization with other important control gate structures, such as Rosario Weir and two (2) floodgate structures newly constructed along the Manggahan Floodway by MMDA or DPWH (See **Figure 6.3.39** below. In this section, the imperative power units and control system have been discussed for the appropriate operation of the MCGS and integrated control system among other control gate structures.



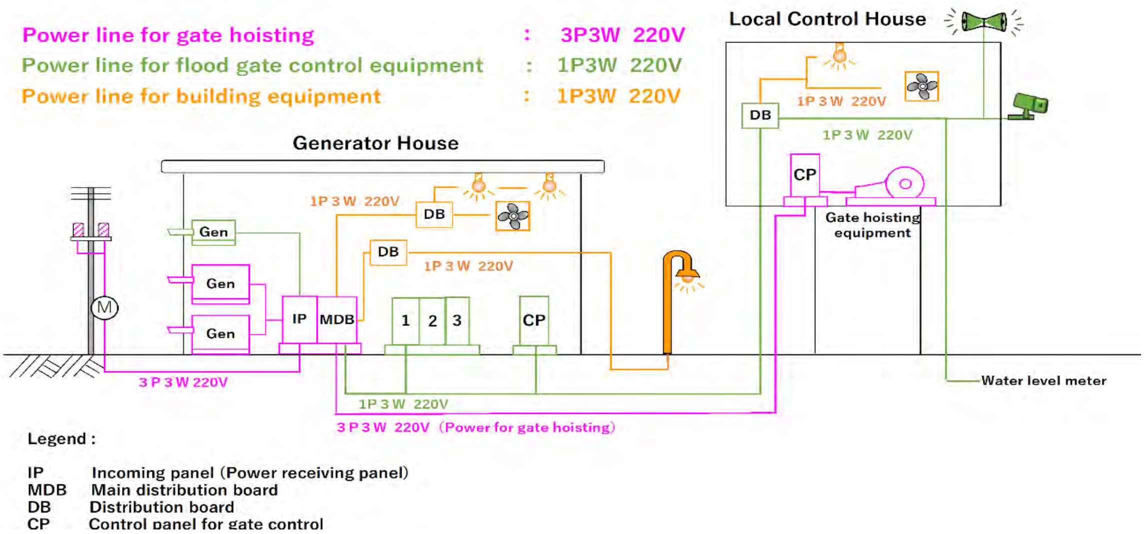
Note: Base Map is quoted from Google Map

Figure 6.3.39 Location of Three (3) Control Gate Structures to be Operated under Integrated System

6.3.5.2 Basic Design of Power Unit and Control System of the MCGS

(1) Power Unit

Basic concept and layout for power units of the MCGS is illustrated in **Figure 6.3.40** below.



Source: Study Team

Figure 6.3.40 Basic Concept and Layout of Power Unit of the MCGS

1) Main Power Unit

There are the electric motors, internal combustion engines, and manpower for opening and closing gates. Since this gate is an important facility for preventing disasters, the main power is an electric motor in consideration of the reliability of starting, stability of opening and closing speed, low failure rate, ease of maintenance and remote operability.

The motor type is a special squirrel-cage three-phase induction motor for floodgates.

2) Reserve Power Unit

As measure against failure of motors which are the main power unit of hoist, reserve power unit shall be provided to ensure necessary functions as important equipment for disaster prevention.

The reserve power for opening and closing includes an electric motor (reserve), an internal combustion engine, and human power.

(a) Motor (reserve)

Usually, a system using an electric motor is adopted due to reliability and easier operation. This system includes the following systems:

1. Always with a spare motor (Fixed installation of spare electric motor)
2. Storage of motor (Storing the same motor as the traction motor and replacing it when the traction motor fails.)

Operation with a spare motor is simple; however, the equipment becomes larger.

Storing a spare electric motor requires more time for replacing the electric motor; however, it does not require the installation of a special device as a spare power, and does not affect the space for installing the hoist.

(b) Internal Combustion Engine

When it is difficult to secure a backup power supply, an internal combustion engine may be employed. In this case, a switch between the electric motor and the internal combustion engine is required, the number of parts increases, and the size of the internal combustion engine itself also influences the size of the equipment. In addition, it takes time and labor for refueling and maintenance.

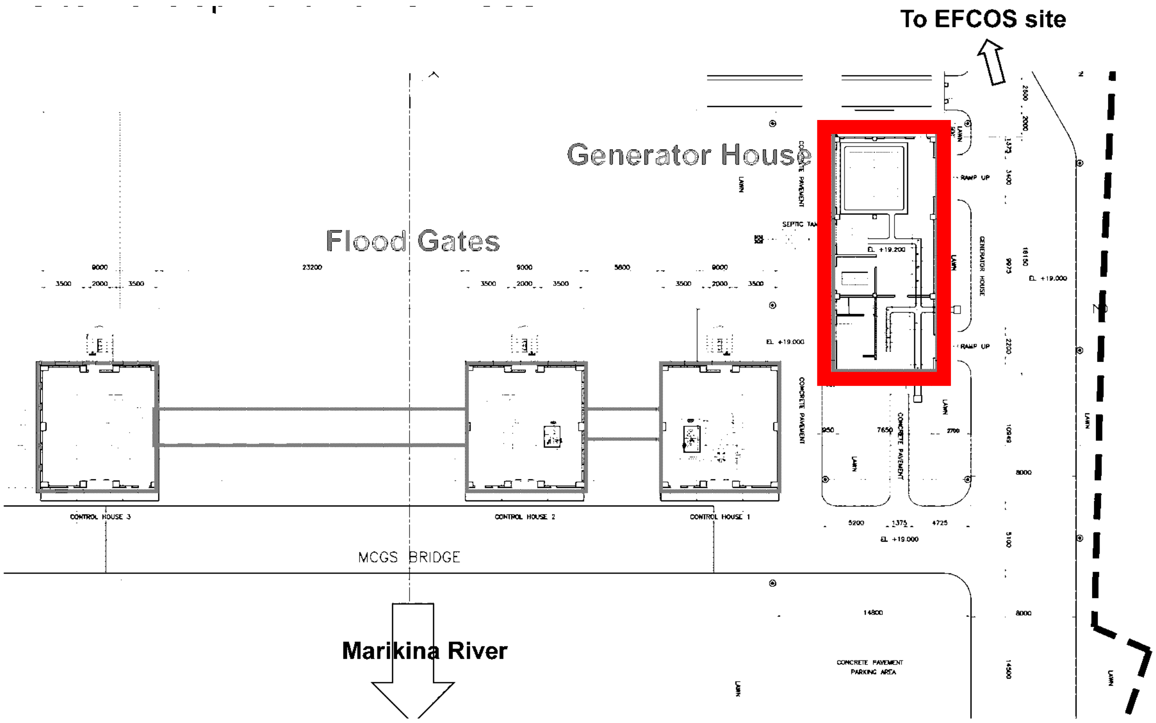
(c) Manpower

Manpower is used when a small-scale facility (approximately less than 10 m^2 of gate leaf area) can be opened and closed by human power. In the case of the gate in this study, it is not practical.

In view of the above, the gate equipment is as follows:

- ✓ This gate is very large (Gate leaf area: 200 m^2 or more), and 2M2D wire rope winch type is applied to the hoist, hence, it is difficult to apply the self-weight lowering device. When the drive motor or power unit fails or the commercial power supply stops, the closing operation becomes impossible. Therefore, a spare switch driver is highly needed.
- ✓ The wire rope winch type hoist can easily have a mechanism in which a spare engine can be incorporated. However, since two sets of reducers are required, installation space and manufacturing cost are increased.
- ✓ If an electric motor is ordered after the failure of the electric motor, the delivery time will be about two months, and it will take time for operation to recover.

Therefore, while emphasizing flood control safety, the reserve power should be "storing a spare motor" considering cost reduction. The Reserve Power Unit should be installed nearby the MCGS. The location of generator house is shown in **Figure 6.3.41** below.



Source: Study Team

Figure 6.3.41 Location of Generator House for Emergency Operation of the MCGS

3) Power Supply Unit

(a) Main Power Supply Unit

The commercial power supply is received in 3φ3 W AC 200V 60Hz and 1φ2 W AC 200V 60Hz directly at the operation panel in the operation room and uses them as power and operation power.

(b) Standby Power Supply System

Since this is an important gate facility, a standby generator will be permanently installed as a backup power for the power unit to ensure the reliability of gate opening and closing.

When the commercial power stops, the power necessary to open and close the gate and the power supply for the incidental water-level gauge, safety equipment, remote control equipment, building equipment, etc., will be secured by the standby power generation unit.

(2) Control System

1) Local Control Panel

(a) Usage

A local control panel is installed on the operation deck for operation, periodical maintenance and normal operation. The specific control function of each equipment is included in the local control panel. The operation is performed by push-button operation from the local control panel and remote operation from the central control station.

(b) Operation Monitoring Item

For gate operation by hoist, in order to ensure safe operation, the local control panel shall be equipped with various monitoring signals and fault indicators as shown in **Table 6.3.47.**

Table 6.3.47 Operation Items and Control Signals

Item	Local Control Panel (Operation)	Control Signal		Remarks
		Presence or Absence of a Signal	Signal Format	
Open (Rise) Operation	○	○	Continuous Output	
Close (Descent) Operation	○	○	Continuous Output	
Stop	○	−		
Alarm Stop	○	-		
Lamp Test	○	-		
Emergency Stop	○	○	Pulse Signal	
Fault Reset	○	-		

Source: Study Team

Status of gates displayed on the local control panel and remote operation console, operation display items, and their monitoring signals are as shown in **Table 6.3.48.**

Table 6.3.48 Gate Operation Display and Monitoring Signal

Item	Local Control Panel(Display)	Monitoring Signal	
		Presence or Absence of a Signal	Signal Format
Power	○	○	Continuous Output
AC Control Power Supply	○	○	Continuous Output
Remote	○	○	Continuous Output
Machine Side	○	○	Continuous Output
Normal Operation	○	○	Continuous Output
Maintenance Operation	○	○	Continuous Output
Normal Circuit	○	○	Continuous Output
Emergency Circuit	○	○	Continuous Output
During Opening (Rise)	○	○	Continuous Output
During Closing (Descent)	○	○	Continuous Output
Stop	○		
Gate Rest	○		
Full Open	○	○	Continuous Output
Full Close	○	○	Continuous Output
hook-out upper limit	○	○	Continuous Output
hook-out lower limit	○	○	Continuous Output
During Gate Operation	○	○	Continuous Output

Source: Study Team

Items to display gate failure on the local control panel are shown in **Table 6.3.49**. Among the items, those which seriously affect operation are indicated as "Serious Failure".

Table 6.3.49 Items to Display Gate Failure and Monitoring Signal

Item	Local Control Panel (Display) Failure Classification		Supervisory Signal	
	Serious Failure	Light Failure	Presence or Absence of a Signal	Signal Format
3E Operation	○		○	Continuous Output
Rope Overload	○		○	Continuous Output
Rope Slack	○		○	Continuous Output
Hydraulic Brake Overload	○		○	Continuous Output
Electromagnetic Brake Overload	○		○	Continuous Output
Emergency Upper Limit	○		○	Continuous Output
Emergency Stop	○		○	Continuous Output
Contact Welding	○		○	Continuous Output
MCCB Trip	○		○	Continuous Output
Leakage		○	○	Continuous Output

Source: Study Team

(c) Panel Configuration

The local control panel consists of a power supply unit for receiving power, an operation unit for operating a gate, a control unit for controlling operation, and a display unit for monitoring the operation status. With regard to the arrangement of equipment on the board and inside the board, equipment necessary for gate operation and monitoring shall be arranged on the operation board on the machine side and equipment not necessary for gate operation and monitoring shall be arranged in the board for easier operation, inspection and maintenance.

(d) Control Circuit

Regarding configuration of the control circuit of the local control panel, there is the method using a contactor relay circuit and the programmable controller (PLC). The advantages and disadvantages of both circuits are as shown in **Table 6.3.50**.

In case of PLC, since it is difficult to cope with failure and there is no need to consider any addition or change on the operation, the relay circuit is adopted.

Table 6.3.50 Advantages and Disadvantages of Contact Relay Circuits and PLC Circuits

Items	Contactor Relay		PLC Circuit	
Reliability	When the circuit becomes complicated, the number of parts increases and the occurrence rate of failure increases.	△	Even if the circuit becomes complicated, the failure rate does not change much.	○
	The influence of failure is almost partial. If the auxiliary relay or timer fails, it can be easily replaced, and the reliability of system can be ensured.	○	The influence of failure is easy to spread. However, if the I/O or the power supply module fails, it can be easily replaced, and reliability of system can be ensured.	△
Repair	It does not require much expertise and can be replaced by an operator if a plug-in type auxiliary relay or compatible timer fails.	○	It is easy to replace the I/O unit. However, when viewing the sequence program, special equipment must be handled. It is not compatible with different manufacturers.	△
Circuit Configuration	The number of parts increases in proportion to the control quantity, since it is composed of an auxiliary relay and a timer, and labor for wiring also increases when the control becomes complicated.	△	Since it consists of the PLC main body and I/O unit, the number of parts does not increase significantly even if the control amount increases. Even if the control becomes complicated, the processing is easy since it is dealt by software.	○
Parts Durability	Although the number of the contact portion is limited, the parts are highly interchangeable can be easily replaced.	○	Since the PLC itself is non-contact, the durability is high; however, the specifications depend on manufacturer, and the parts are not interchangeable.	△
Environment	It is weak in humidity, etc., However, stronger in the operating environment than PLC.	◎	Protective measures against electromagnetic waves such as noise and thunder are necessary.	△
Evaluation	Although the durability is a little low and failure occurrence rate is high, the maintenance is comparatively easy, and it has advantage in environment. Hence, the same system as the one currently used to be adopted.		Although the durability is high and the failure occurrence rate is low, it is not adopted since maintenance of the circuit board requires specialized knowledge and security management is difficult.	
	Recommended			

Legend: ◎... Better, ○... No problem, △... There are issues to be solved.
Source: Study Team

(e) Type of Local Control Panel

Since the local control panel is installed in the operation room, it is an indoor and closed self-standing type with high expandability and easier maintenance and inspection.

2) Remote Monitoring and Control System

(a) Facility Operation Methods and System-Level

(i) Classification of Facility Operation Methods

The MCGS operation is assumed to be performed by a facility manager or a field operator entrusted with the facility operation. With regard to the requirement for facility operation, it is required to open and close the gate safely, quickly and surely. In order to carry out these tasks reliably, in addition to "field operation" which is the most basic method, "remote operation" which is expected to improve the efficiency of facility management will be used. **Table 6.3.51** show the advantages and disadvantages of each method.

Table 6.3.51 Comparison of Operation Techniques

Items	Field Operation	Remote Operation
Advantages	・ Since the field operator operates the MCGS while checking the surrounding conditions at the same time, the safety during operation is excellent.	・ Since there is no need to assign field operators, attention to the safety of operators is not needed. ・ The time from acquisition of weather advisories and warnings to the start of operations is short. ・ Since there is no field operator, instructions to the site are not required, and the task of the manager is reduced. ・ Integrated operation is possible while grasping the situation of the nearby Rosario Weir, the newly proposed floodgates (Cainta and Taytay Floodgate), and the inland water.
Disadvantages	・ Measures to ensure safety and time are necessary for the movement, operation and evacuation of site operators. ・ The MCGS operation takes a relatively long time to start. ・ It is difficult to carry out individual operation while grasping the situation of the nearby Rosario Weir, the newly proposed floodgates (Cainta and Taytay Floodgate), and the inner water area.	・ The facility is needed to be electrically operated, installation and the running costs of facilities for remote monitoring and remote operation are needed. ・ Alarm equipment such as sirens, loudspeakers, and rotating lights should be sufficiently prepared in consideration of the safety of facilities and surroundings. ・ It is necessary to install camera equipment, etc., in order to grasp the condition of facilities and surrounding conditions. ・ Backup measures should be taken when remote operation is not possible.

Source: Study Team

(ii) MCGS Operational System Level

The operation of the facility can be generally classified into "Instruction", "Operation" and "Checking and Monitoring", which are combinations of implementation methods for individual functions.

・ Instruction

Instructions on gate operation to be performed is exchanged between the facility manager and the field operator.

・ Operation

A facility manager or a field operator performs facility operations from the site or from a remote location.

・ Checking and Monitoring

The facility manager understands and records the operation results (Completion of open/close operation, inability to operate, etc.) of the checking and monitoring facility.

The combination summarized in five system levels is shown in **Table 6.3.52** considering practicability and **Figure 6.3.42** shows the combinations.

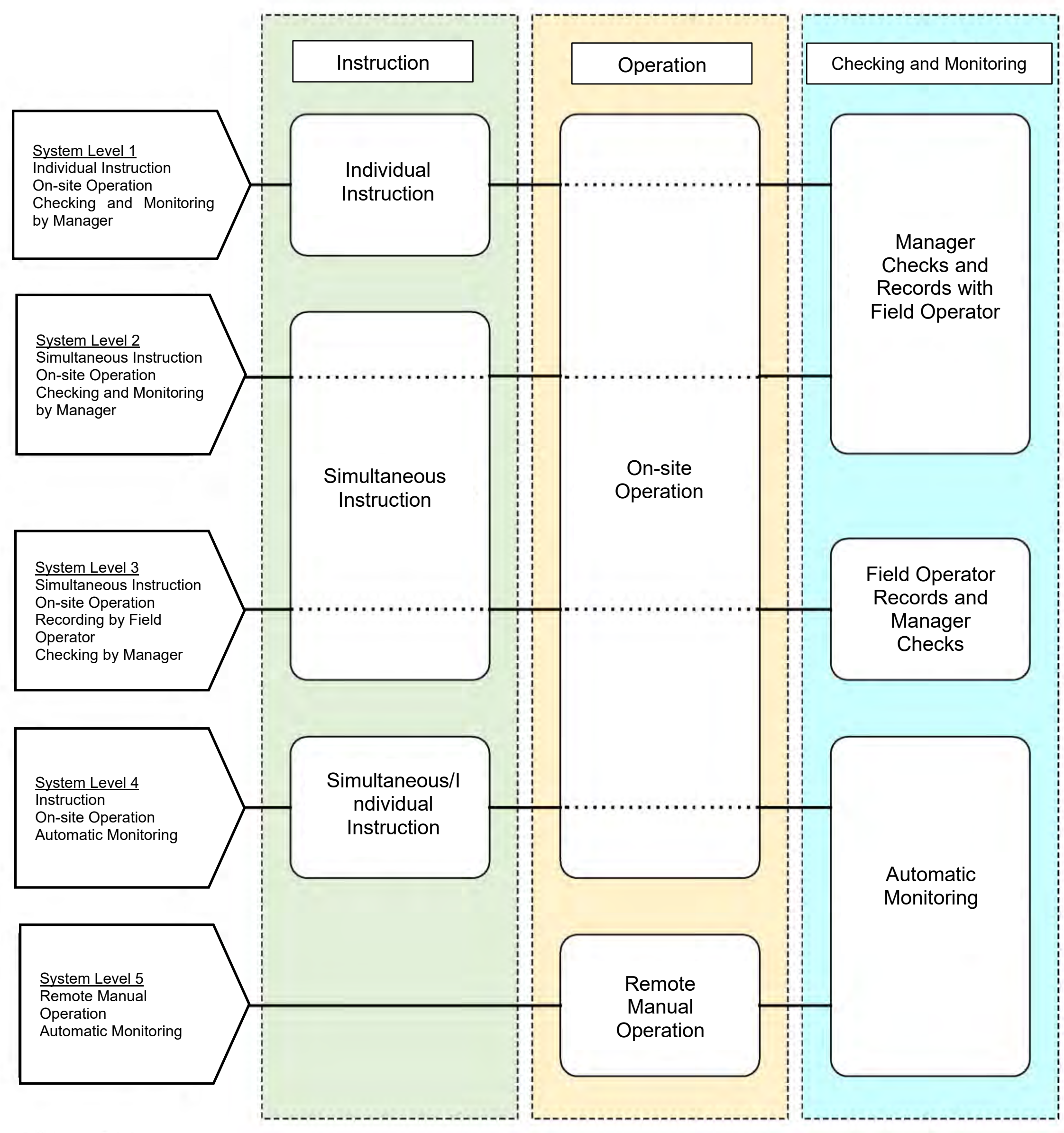
Table 6.3.52 System Level for Facility Operation

System Level	Instruction	Operation	Checking and Monitoring
1	Individual Instruction	Field Operation	Confirmation and Record by Manager
2	Simultaneous Instruction	Field Operation	Confirmation and Record by Manager
3	Simultaneous Instruction	Field Operation	Input by Field Operator and Confirmation by Manger
4	Instruction	Field Operation	Automatic Monitoring
5		Remote Operation	Automatic Monitoring

Source: Study Team

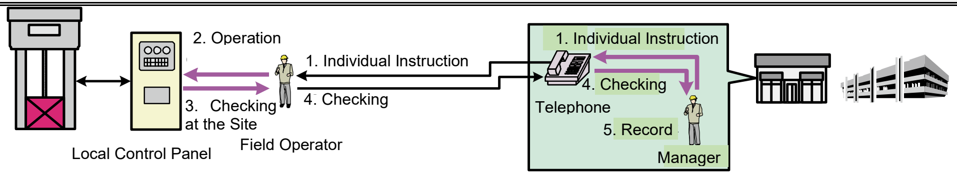
In addition, specific operation management images of managers and field operators in operation monitoring system functions such as MCGS corresponding to these system levels are shown in **Figure6.3.43**.

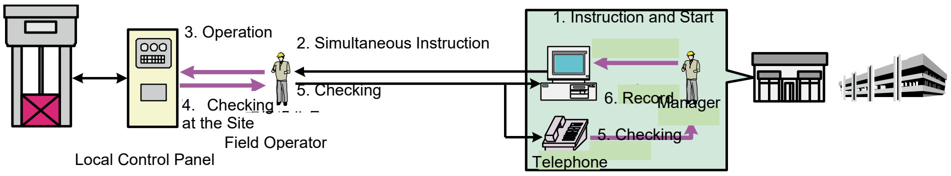
System levels 1 to 4 are the cases with a field operator, and system level 5 is a case with no field operator.

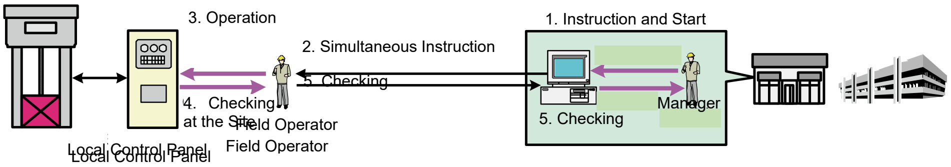


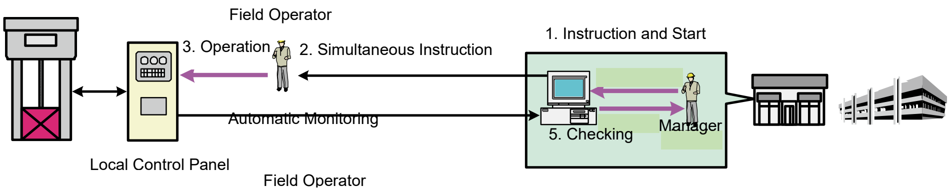
Source: Study Team

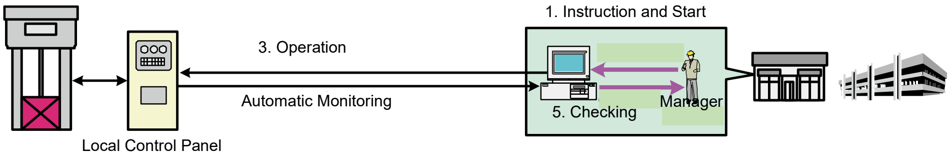
Figure 6.3.42 Conceptual Diagram of System Levels

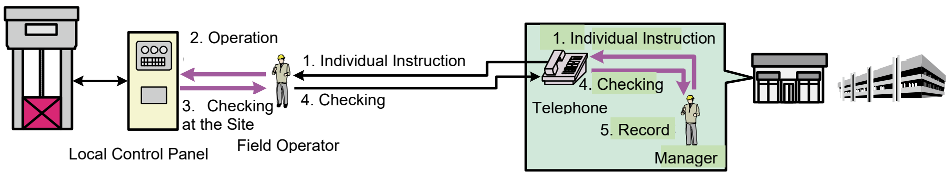
<table>
<tr><th>System Level</th><th>Role of the Field</th><th>Role of a Remote Site</th></tr>
<tr><td>System Level 1
Individual Instruction
Field Operation
Confirmation and Record by Manager</td><td colspan="2">

The manager gives individual instructions to the field operator, and the field operator who receives the instruction operates the facility. After that, the field operator reports the operation result to the manager by telephone or e-mail, and the manager confirms and records the report.</td></tr>
<tr><td>System Level 2
Simultaneous Instruction
Field Operation
Confirmation and Record by Manager</td><td colspan="2">

The manager gives a simultaneous instruction to the field operator by telephone, mail, etc., using a reporting device, and the field operator who receives the instruction operates the facility. After that, the field operator reports the operation result to the manager by telephone or e-mail, and the manager confirms and records the report.</td></tr>
<tr><td>System Level 3
Simultaneous Instruction
Field Operation
Input by Field Operator
Checking by Manager</td><td colspan="2">

The manager gives a simultaneous instruction to the field operator by telephone, mail, etc., using a reporting device, and the field operator who receives the instruction operates the facility. The field operator then enters the operation results from the field into the management system. The manager reviews the input.</td></tr>
<tr><td>System Level 4
Instruction
Field Operation
Automatic Monitoring</td><td colspan="2">

The manager gives instructions to the field operator in any of the above methods, and the field operator who receives the instructions operates the facility. The manager then monitors the automatically entered facility status.</td></tr>
<tr><td>System Level 5
Remote Operation
Automatic Monitoring with Redundancy System</td><td colspan="2">



The manager directly operates the field facility by remote operation. The manager monitors the state of the automatically entered gates.</td></tr>
</table>

: by Manual
: Automatic *Numbers present procedures by the manual.

Source: Study Team

Figure6.3.43 Image of Operation Management for System Levels

Table 6.3.53 shows the comparison of each system level It is necessary to set the applicable system level considering the advantages and disadvantages for a management system from the aspect of the scale of maintenance, maintenance cost, time required for facility operation, work load on managers and field operators, and safety of field operators as shown in this table. In order to secure a reliable operation, it is necessary to enable field operation (local manual operation, manual operation at the machine side, and manual operation by manpower) even in the case of remote operation (System Level 5).

For MCGS operation, it is recommended to implement System Level 5 (remote operation) for the following reasons:

- The condition of the water level of the main river, status of inflow from Cainta River and Taytay Creek to Manggahan Floodway and the operation status of related river management facilities (Rosario weir, gates, etc.), which are the information necessary to make decisions, can be observed and reliable operation can be performed.
- Prompt and timely operations (Shrinking time required for facility operation).
- Reduces workload of operation during flood events and allows the managers to operate facilities by themselves. (Operation of facilities can be performed without outsourcing to local governments, etc.)

Table 6.3.53 Comparison of System Levels

Items \ System Level		System Level 1 (Individual Instruction. Field Operation, Checking and Recording by Manager)	System Level 2 (Simultaneous Instruction, Field Operation, Checking and Recording by Manager)	System Level 3 (Simultaneous Instruction, Field Operation, Input by Field Operator, Checking by Manager)	System Level 4 (Instruction, Field Operation, Automatic Monitoring)	System Level 5 (Remote Operation, Automatic Monitoring)
Scale of Installation		It is often possible to use existing equipment and it is small-scale.	Require simultaneous instruction device and it is slightly small.	A simultaneous instruction device and an input device for field operator are required, and they are medium-sized.	In addition to the device for instruction, a device for remote automatic monitoring is required, and the scale is slightly large.	A remote-control device is required, and it becomes large-scale.
		Small	Still Small	Medium	Medium	Large
Maintenance Cost		Most of them are existing, and maintenance cost is almost unnecessary.	Installation of simple equipment is required. However, maintenance cost is a little small. In many cases, existing equipment can be used.	Installation of new equipment is required.	Installation of new equipment is required.	Installation of new equipment is required. The running cost is also high.
		Small	Somewhat small	Medium	Medium	Large
Time Required for Operation		Long	Medium	Slightly short	Slightly short	Short
Workload of Manager	Directions	Heavy workload due to individual instructions.	Workload is small due to the simultaneous instruction.	Workload is small due to the simultaneous instruction.	Workload depends on the instruction level (Individually or Collectively).	Since there is no instruction to be done, there is no workload.
	Reporting	Much workload to deal with individual reports.	Heavy workload to deal with individual reports.	Since the field operator does the input, the workload is small.	There is no reporting, hence, there is no workload.	There is no reporting, hence, there is no workload.
		Large	Medium	Medium	Medium to moderately small	No Workload
Workload of Field Operator	Directions	Since ordinary communication device is used, not much workload is required to operate the communication system.	The workload is small since there is no need to call the administrator every time.	The workload is small since there is no need to call the administrator every time.	The workload depends on the instruction level (Individually or Collectively).	Since there is no instruction work, there is no workload.
	Reporting	Much workload to report to the manager individually by voice, etc.	The workload is large, since reporting is made individually to the manager by voice, etc.	Since the field operator makes the inputs, there is much workload.	There is no reporting work, so there is no workload.	There is no reporting work, so there is no workload.
		Large	Large	Large	Medium	No Workload
Safety of Field Operator		Need much consideration	Need much consideration	Need much consideration	Need much consideration	Need much consideration

Source: Study Team

(b) Remote Monitoring and Control System

(i) Functions of Remote Monitoring and Control System

The remote monitoring and control system carry out the monitoring and control of equipment close to it or from a remote location (Management Offices, etc.), and consists of A: remote monitoring function, B: remote control function, C: data management function, and D: operation support function. The outline of each function is as follows, and it is constructed according to the scale of the system and the operation management system.

A. Remote Monitoring Function

1. A function is provided for collecting the status of monitoring information by the local control panel.
2. As a means of safety confirmation, a function is provided for video monitoring by a monitoring camera. If necessary, a function to give an alarm or warning to aircrafts is provided.
3. An intercommunication function such as telephone is provided between remote devices.

B. Remote Control Function

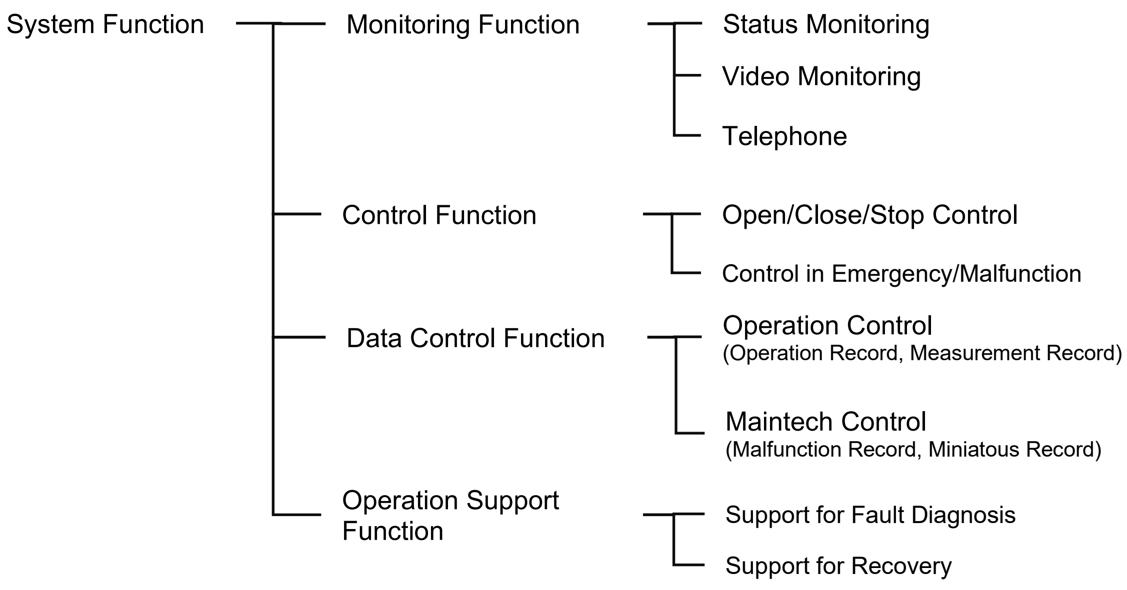
1. A gate opening/closing/stopping function is provided from a distance.
2. An emergency stop function is provided to cope with an emergency situation.
3. A forced operation function is provided for troubleshooting.

C. Data Management Function

Data management functions are provided for operation records, failure records, and inspection and maintenance records for the operation and maintenance of facilities.

D. Driving Support Function

1. As logistical support, failure diagnosis support and recovery support functions are provided.
2. An operation guidance function is provided if necessary.



Source: Study Team

Figure6.3.44 System Function Configuration

(ii) Overall System Configuration

A. Selection of the System Method

The overall system configuration is studied for determination of the remote monitoring control system. There are the following three methods for systematizing a remote monitoring control system. The improvement of each method is as described below. **Table6.3.54** gives a comparison of each improvement method.

I. Client/Server Method (C/S)

The information is handled by a client/server system on an optical network. In this system configuration, the monitoring control program of each facility is provided to the remote monitoring control device of the management office. At the time of facility operation, a monitoring control program corresponding to the target facility is called to perform monitoring control.

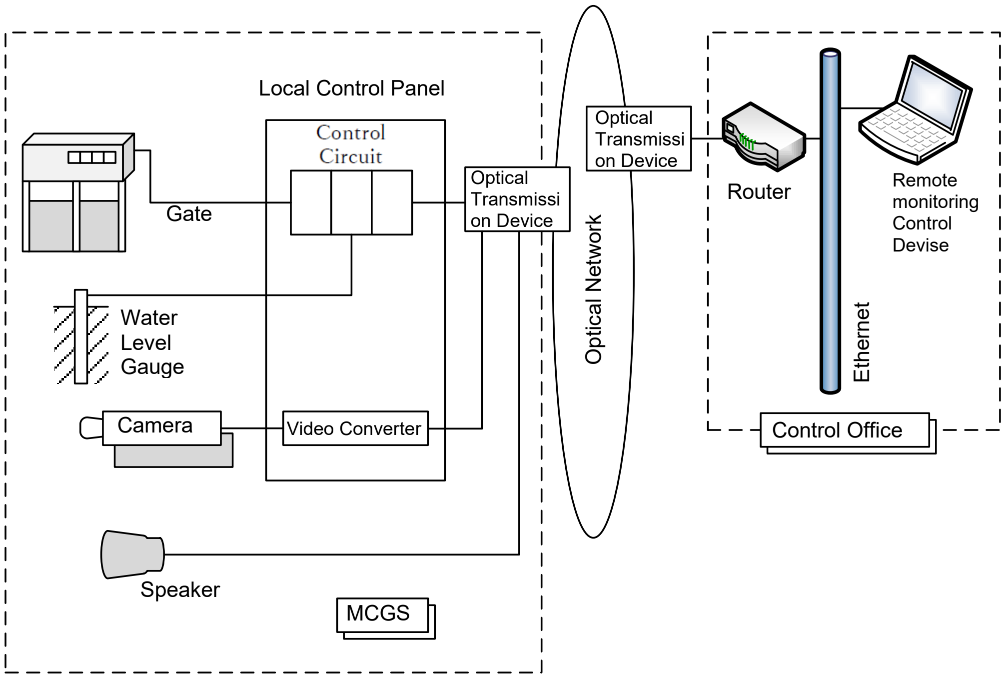


Figure6.3.45 Image of Client/Server System Configuration

II. Centralized Web System

The information is handled by a centralized web system on an optical network. In this system configuration, the monitoring control program of each facility and the transmission program between each facility and the system management server are provided in the system management server. The monitoring control page of each facility is called to the remote monitoring control device by Web from the system management server, and the monitoring control is carried out by the facility specific program.

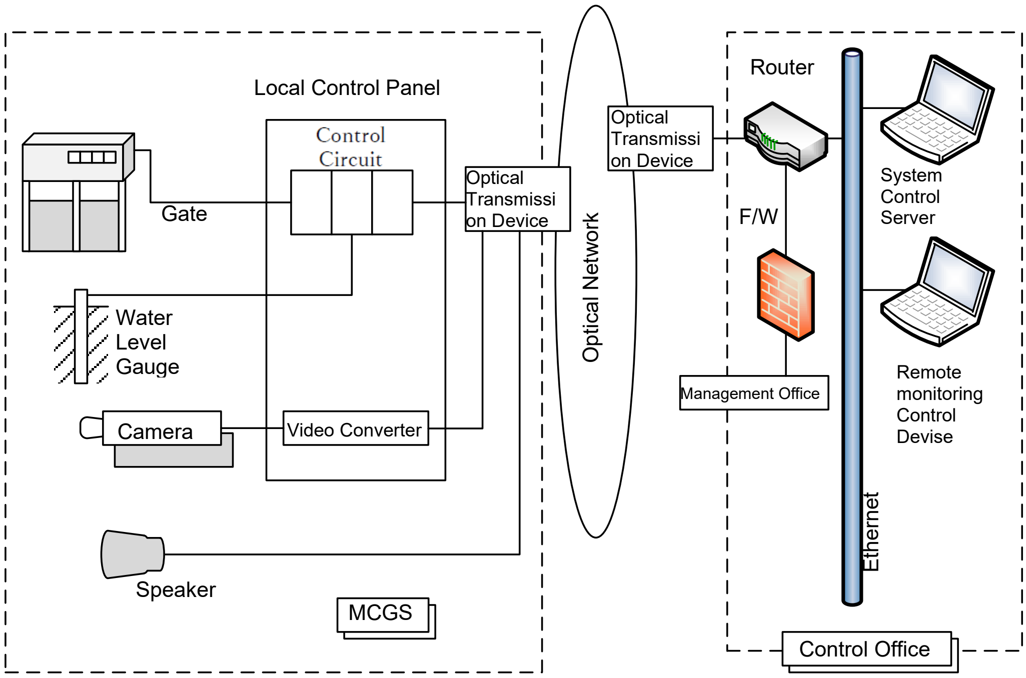


Figure6.3.46 Image of Centralized Web System Configuration Image

III. Distributed Web System

The information is handled by the distributed Web system on the optical network. In this system configuration, the monitoring and control programs of each facility are distributed in the field server. Monitoring and control are performed by calling the monitoring control page of the field server from the remote monitoring control device.

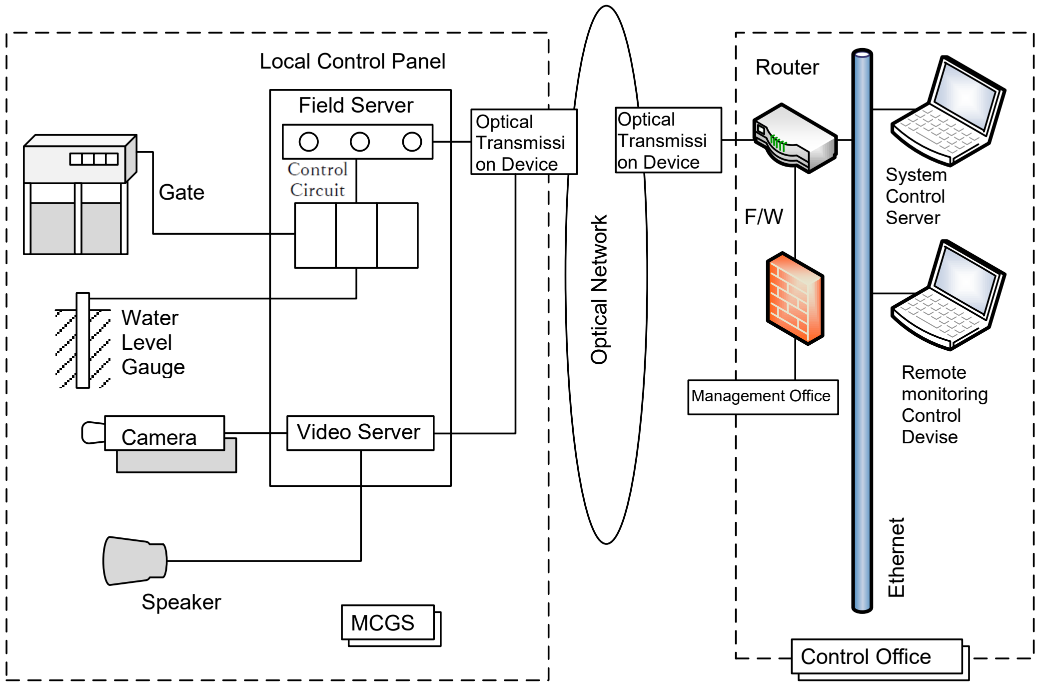
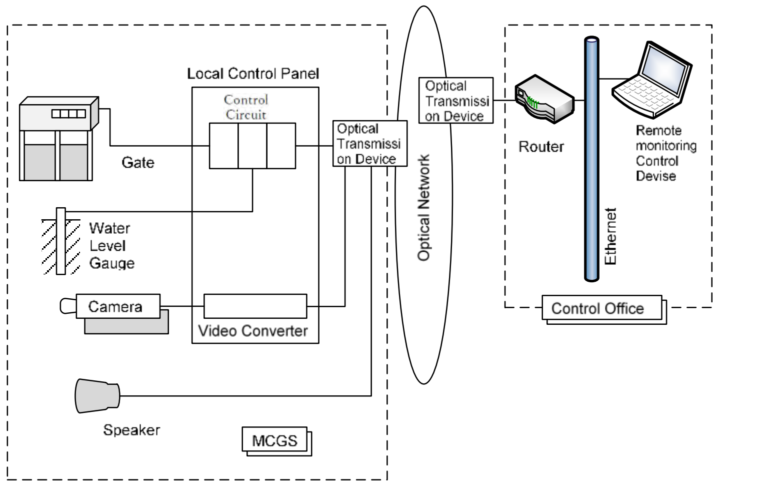
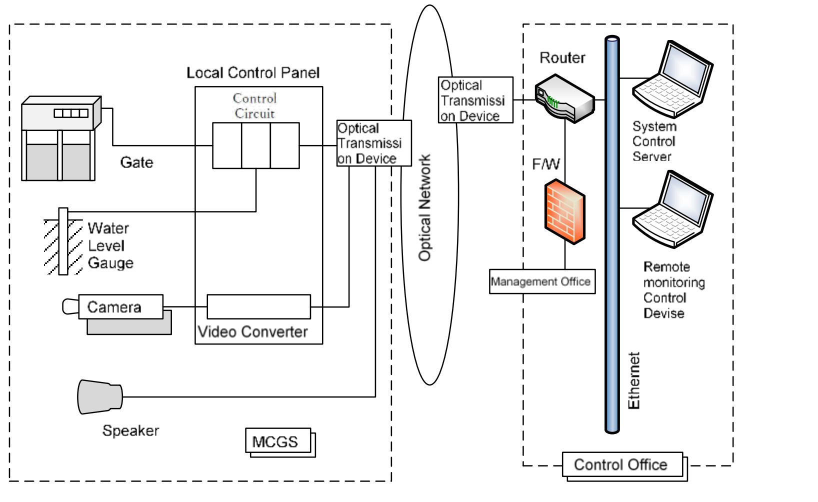
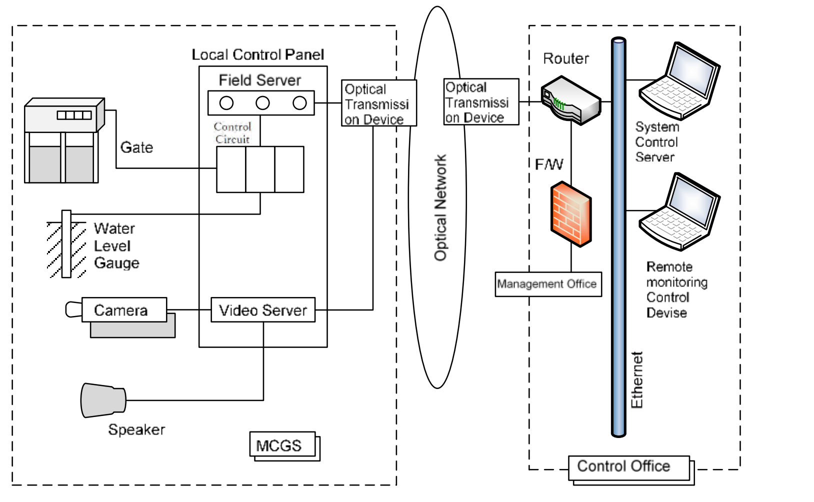


Figure6.3.47 Image of Distributed Web System Configuration

In the MCGS, the "Distributed Web System" is recommended due to the following features, and the system design will be conducted on this system.

1. This system allows anyone to easily view monitoring information.
2. This system can easily introduce measures to enhance reliability such as line monitoring.
3. Widely used programs can be used for a remote monitoring and control device.
4. This system can be easily maintained remotely.
5. This system can cope with simultaneous control of multiple gate facilities. (In addition to MCGS, it can be applied to extend the control system of the Cainta and Taytay floodgates.)
6. Installation of the system is easy, and there is no impact on the operation of the existing system during installation.
7. If the current Rosario Weir Management System is updated in the future, it can be incorporated into the system.
8. It is a load balancing (risk distribution in case of failure) type system of a remote monitoring control device.

Table6.3.54 Comparison of System Configuration

No	Comparison Items	Client/Server Method (C/S)		Centralized Web System		Distributed Web System	
1	System Configuration						
		Monitoring and Control of Major Facilities	Monitoring and Control of Video and other Auxiliary Facilities	Monitoring and control of major facilities	Monitoring and control of video and other auxiliary facilities	Monitoring and control of major facilities	Monitoring and control of video and other auxiliary facilities
2	System Configuration and General Basic Function	・ Client/Server Information Handling over Optical Networks ・ The monitoring control program of each facility is provided to the remote monitoring control device. ・ Monitoring and control are performed by invoking a monitoring and control program for the target facility. ・ In the case of video, monitoring control is performed by selecting a camera at a monitoring location from a remote monitoring control device. ・ Have the line monitoring function between the client and the server (remote monitoring and control unit) in the remote monitoring control device		・ Handle Information in a Centralized Web Method over Optical Networks ・ The system management server has a monitoring control program for each facility and a transmission program between each facility and the system management server. ・ The system management server calls the monitoring control page of each facility on the web to the remote monitoring control device and performs monitoring control using a facility specific program. ・ The system management server calls the camera operation page corresponding to each facility on the web to the remote monitoring control device to control the monitoring camera. ・ Provide remote monitoring control with line monitoring function between client and system management server		・ Handles information over optical networks using distributed Web methods ・ Distribute monitoring and control programs for each facility on a field server ・ The remote monitoring control device calls the monitoring control page of the field server to monitor and control. ・ Access the video server of the target facility from the web base screen and display the video in a separate window. ・ The monitoring camera is controlled by calling up the camera operation page of the video server of the target facility to another window of the remote monitoring control device. ・ Have the system management server function ・ Data of all facilities in the jurisdiction is stored in the system management server.	
3	Load on the Optical Network	・ Have a good response time	・ Have a good response time	・ There is a constant load to send all gate equipment data to the control center. Control load is added during control	・ The load is high because video data is always sent. Need line capacity for load	・ Load is generated according to the number of accesses of remote monitoring control	・ Load is generated according to the number of video accesses, and line capacity is required according to the load
4	Reliability of Control	・ Reliability is enhanced by confirming an abnormality in a transmission path by transmitting a signal at a fixed period from a client to a remote monitoring control device.	・ Enables video monitoring	・ Reliability is enhanced by confirming an abnormality in a transmission path by transmitting a signal at a fixed period from a client to a remote monitoring control device.	・ Enables video monitoring	・ Increase reliability by sending periodic signals between the system management server and the field server to check for abnormalities in the transmission path	・ Enables video monitoring
5	Communication Specifications and Communication Means	・ proprietary protocol		・ TCP/IP, HTTP		・ TCP/IP, HTTP	
6	System Maintenance: Application Version Upgrade	・ All the modification of HTML files will be carried out on system managing server.		・ The modification of HTML files on field server in the machine side, movie server will be carried out in the machine side.		・ HTML files of all field and video servers are changed by the machine (Can also be changed by management).	
	System Maintenance: Change of Operation Registrant	・ Operation registrant of system managing server needs to be changed		・ Change of each field server operation registration is required		・ Individual field server operation registrants must be changed (Can also be changed by the administration office)	
	System Maintenance: Remote Maintenance	・ Remote maintenance is not possible		・ Access the System Management Server to perform maintenance where needed		・ Access individual field servers to perform maintenance where needed	
7	System Architecture	・ The entire program needs to be reviewed every time a facility is delivered.		・ System management server programs must be adjusted each time a facility is delivered		・ The facility supplier can deliver a monitoring and control system that matches the facility, and there is no impact on other existing systems.	
8	Degree of Freedom of Control	・ Parallel control of multiple facilities is also possible.		・ Parallel control of multiple facilities is also possible.		・ Parallel control of multiple facilities is also possible.	
9	System Scalability	・ Clients and remote monitoring and control equipment load increases with each addition.		・ System management server load increases with each addition ・ If the system management server fails, monitoring and control of all facilities cannot be performed.		・ The load of the remote monitoring and control unit does not change even if it is added.	

Source: Study Team

B. System Configuration

The system configuration is based on the "Distributed Web System" as summarized below.

1. Setup of Remote Locations

Remote monitoring and control systems for the MCGS shall be installed at the following locations.

Table6.3.55 Setup of System Location

Classification	River Name	Classification of Facilities	Name of the Facility	Remarks
Remote Control			EFCOS	
Local Control	Marikina River	Weir	MCGS	
	Cainta River	Floodgate	Cainta Floodgate	Refer to Section 6.4 Floodgate to Prevent Backflow
	Taytay Creek	Sluiceway	Taytay Floodgate	Refer to Section 6.4 Floodgate to Prevent Backflow

Source: Study Team

2. Basic Approach to System Configuration

I. The MCGS is remotely monitored and controlled by the classified Web system.
II. An operation screen and a program are equipped in a field server installed on each machine side. The system management server of the EFCOS is a risk classification type system mainly using management data such as a database.
III. A system which secures security by means of a firewall and an authentication function of a server and is capable of remote maintenance and data distribution.
IV. Communication paths between equipment and EFCOS shall have sufficient transmission capacity.
V. The EFCOS and various servers installed on the equipment side are connected to the Ethernet.
VI. An interface such as a router is installed between the optical transmission apparatus and the Ethernet as required.

3. Basic System Configuration

I. The system configuration of the EFCOS consists of the installation of a system management server and a remote monitoring control device. However, in case more than two gates are monitored and controlled simultaneously, a remote monitoring control device is added, if necessary. Information is distributed to related organizations (DPWH, related organizations, LGUs, etc.) through firewalls.
II. Field server and video server are installed on the machine side.
III. Instrumentation equipment, alarm equipment and monitoring equipment are installed for facility control, and they are connected to the field server and video server on the machine side.

4. Main Function of the System

I. The system management server (Installed in EFCOS) has the following functions:

A) Function to store facility layout data for the entire control of EFCOS, facility specifications data of each facility, operation record data, facility delivery data, measurement record data such as water level, etc.
B) Function to process trend graph of measurement record data.
C) Daily and monthly reports processing function of operation management data and maintenance management data.
D) Line diagnostic function to check the line status during monitoring control.
E) Capability of displaying the layout and status of facilities across jurisdictions.
F) Authentication and elimination functions in supervisory control.

G) information distribution function.

II. The remote monitoring control device (Installed in EFCOS) has the following functions:

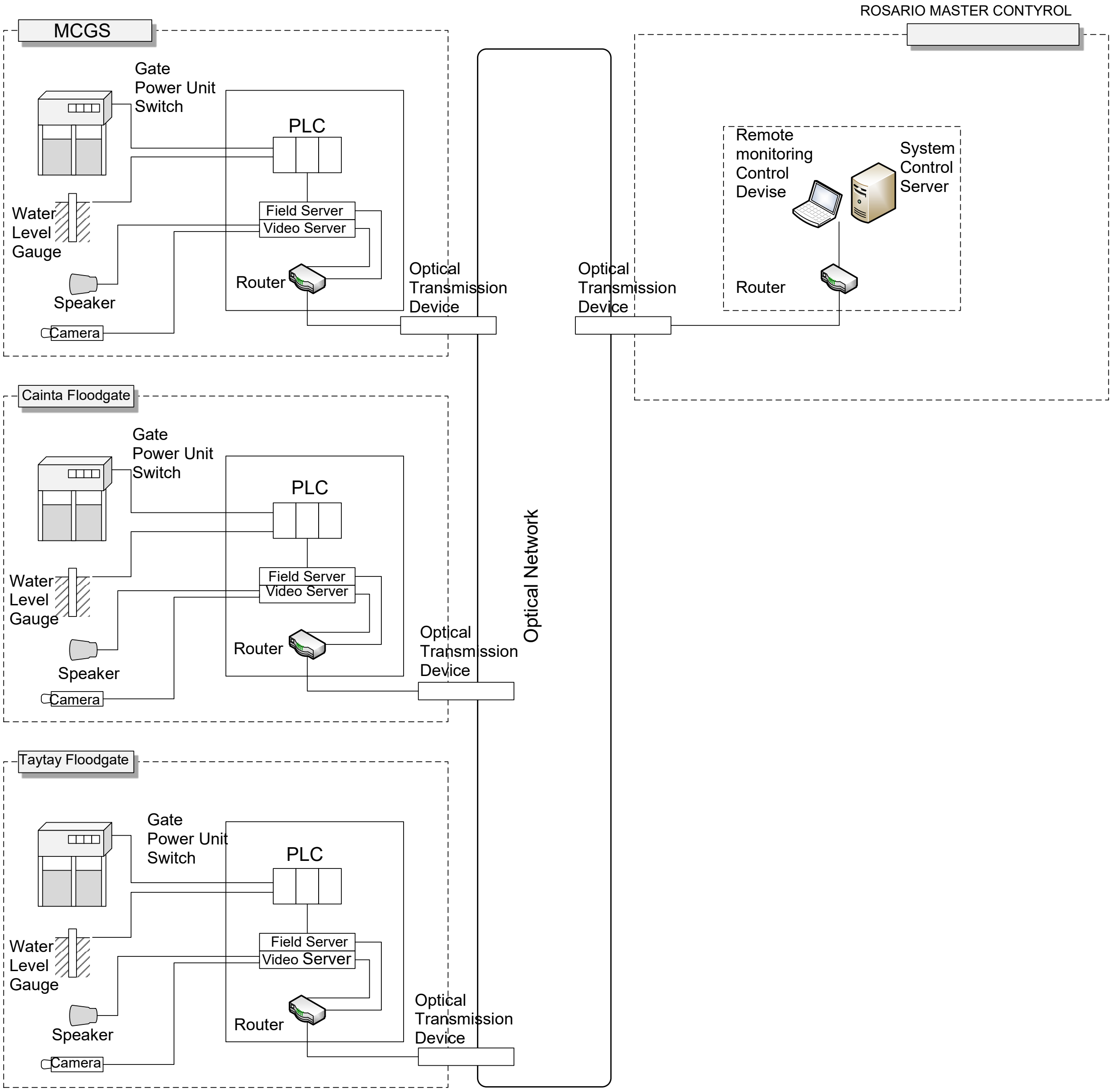
A) A function to access the home page of the field server installed on the machine and remotely monitor and control the target facility using the monitoring screen and operation screen.
B) Function for viewing data stored in the system management server for each facility.
C) A video server installed on the machine is called, and function for image selection by selecting a camera and function for camera control from a distance are provided.
D) Decoder function of video information sent from the video server on the machine side.
E) A function to call up the operation support page from the home page of the field server installed on the machine side, and to support fault diagnosis and recovery.

III. The field server (Installed on the machine side) has the following functions:

A) Website functions for monitoring the facilities and operation screens.
B) Temporary storage of facility operating, measurement and fault records.
C) Processing functions for facility fault diagnosis support and recovery support.
D) Failure diagnosis support and recovery screen homepage function.

IV. The field server (Installed on the machine side) has the following functions:

A) Encoder function that compresses images from surveillance cameras.
B) Home page function for camera monitoring and operation screen installed on the device.



Source: Study Team

Figure 6.3.48 MCGS Remote Monitoring and Control System Configuration including the System for the Cainta and Taytay Floodgates (Draft)

(iii) Configuration of Monitoring and Instrumentation Facilities (MCGS)

The status of remote control and monitoring facilities of the MCGS are as summarized hereinafter. In the 2002 Phase I detailed design, the instrumentation, alarm and monitoring equipment installed on the aircraft side were designed (see **Figure 6.3.49).** For reference, the previous year's design was followed, and the installation equipment was determined.

1. Instrumentation: Used for flood check and facility operation (Gate open/close judgment)
2. Alarm devices (Alarm light, sound collection microphone, and speaker): Used to inform the surrounding people of safety when operating at a distance (Be on alert and alert)
3. It is used to check the status of various facilities by visual inspection of the monitoring equipment and to check the opening and closing of gates.

In Phase IV, the MCGS position will be changed and the gate structure will be reviewed. From this, the installation positions of the instrumentation, alarm and monitoring equipment shall be determined in accordance with the MCGS design.

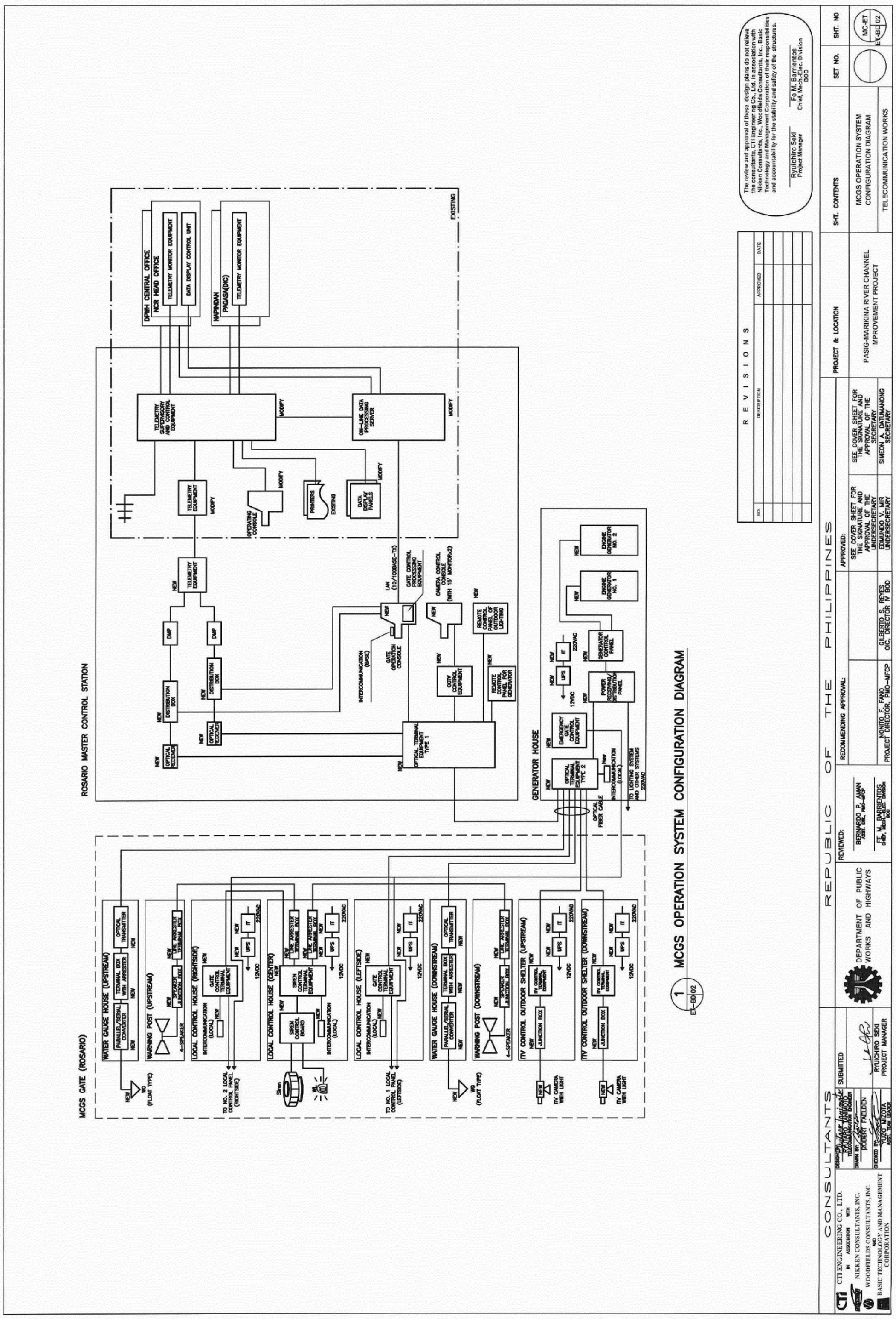
In order to collect observation data and monitoring images, such as the operation of safety awareness facilities and the transmission and reception of remote control signals obtained from each device shown in **Table 6.3.56,** an optical cable on the embankment between the EFCOS and the MCGS shall be installed.

Table 6.3.56 Instrumentation, Alarm and Monitoring Equipment

<table>
<tr><th>River Name</th><th>Facility</th><th>Monitoring and Observation Equipment Installed</th><th>Purpose</th><th>Proposed Location</th></tr>
<tr><td rowspan="5">Marikina</td><td rowspan="5">MCGS</td><td>Water Level Gauge</td><td>Used to grasp the situation of river water level and to judge the opening and closing of gates.</td><td>MCGS Upstream and Downstream, Left Bank of Marikina River</td></tr>
<tr><td>Siren</td><td rowspan="3">Used for safety notification during gate operation</td><td rowspan="2">Gate shed</td></tr>
<tr><td>Warning Light</td></tr>
<tr><td>Speaker and Sound Collection Microphone</td><td>Gate shrouds and sections where alarms are required</td></tr>
<tr><td>Camera</td><td>Used to visually check the status of various facilities, determine gate opening/closing, and check gate opening/closing.</td><td>MCGS Upstream and Downstream, Left Bank of Marikina River, Gate</td></tr>
</table>

Source: Study Team

Figure 6.3.49 Configuration of Remote Monitoring and Control System Proposed in 2002 PMRCIP-I



Source: Final Report of 2002 PMRCIP-I

3) Connection Between Remote Monitoring Control System And Equipment of Machine Side

The remote monitoring and control system needs to transfer a plurality of signals for instrumentation, alarm, and monitoring control. Hence, the connection between the remote monitoring and control equipment and the equipment on the machine side will be based on the connection by the optical cable and the programmable controller (PLC) to simplify the cable connection (reduce the number of cables to be installed) and to cope with the induced lightning.

Here, the equipment on the machine side has a contact relay circuit as a control circuit system. The remote monitoring and control system will be connected to the PLC by a device that converts PLC signals into non-voltage contacts.

4) Updating of Existing EFCOS Systems

According to the field survey conducted prior to this design, many of the existing EFCOS systems were established about 20 years ago and some of them have exceeded their lifetime (service life time with proper inspection and maintenance such as repairing, see **Table 6.3.57**) considering the installation environment. In view of the current status of telecommunications equipment, upgrading equipment that have exceeded their service life shall be considered, together with the MCGS operation system.

Equipment which need replacement considering the site inspection results are shown in **Table 6.3.58** to **Table6.3.65**. For equipment on which the nameplate cannot be identified and those with distribution board (PDB) and terminal board (MDF/IDF) not usable anymore (Equipment marked with "-" for the year of manufacture) the year of manufacture is assumed to be the same as the year of related equipment procurement.

Table 6.3.57 Lifetime of Telecommunication Facilities Considering the Installation Environment

No.	Equipment	Design Lifetime	Service Lifetime Considering Environment	Expected Lifetime with Extension
1	Transforming Equipment	20	30	34
2	Power Generation Equipment	20	25	29
3	Uninterruptible Power Supply (UPS)	15	19	←
4	DC Power Supply	15		
5	CCTV	11	13	16
6	Telemeter Equipment	13	16	←
7	Discharge Warning Equipment	13		
8	Radar Rain Gauge	13	14	←
9	Road Information Display Equipment	15		
10	River Information Display Equipment	15	19	22
11	Emergency Alert Equipment	15		
12	Radio Rebroadcasting Equipment	13	15	←
13	Roadside Communication Equipment	13	18	←
14	Electronic Application Equipment	8	15	←
15	Multiple Wireless Communication Equipment	12	15	←
16	Telephone Exchange Equipment	8	16	←
17	Wired Communication Equipment (*1)	12	12	←
18	Optical Fiber Line Monitoring Equipment	10	10	←
19	Satellite Communication Equipment	13	15	←
20	River Information System	8	10	←
21	Road Information System			
22	Road-vehicle Communication Equipment	7		
23	Dam-weir Information System	8	16	←
24	Network Equipment	5	8	←

Note: Translated from Guidelines for Developing a Telecommunications Facilities Maintenance Plan (draft), March 2018, Telecommunications Office, Ministry of Land, Infrastructure and Transport, Japan

Table 6.3.58 Current Status of Telecommunication Facilities (ROSARIO MASTER CONTROL STATION)

ROSARIO MASTER CONTROL STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
DIESEL ENGINE GENERATOR	1		NIPPON SHARYO, LTD	1982	37
FUEL TANK	2		-	-	-
DATA DISPLAY EQUIPMENT	2	For Alert For Telemeter	JRC	2001	18
OPERATION CONSOLE	2	For Alert For Telemeter	-	-	-
TELEPRINTER	2		JRC	1991	28
TELEPRINTER	1		JRC	-	-
DATA DISPLAY CONTROL UNIT	1		-	-	-
DATA PROCESSING EQUIPMENT	1		-	-	-
DMP READER EQUIPMENT	1		-	-	-
PLASMA DISPLAY	1		-	-	-
MDF	1		Shoden Corporation	1991	28
WARNING EQUIPMENT	1		JRC	-	-
TELEMETRY EQUIPMENT	1		JRC	2001	18
DP-3	1		-	-	-
EP-3	1		-	-	-
AC PDB	1		-	-	-
AVR PDB	1		-	-	-
DC POWER SOURCE	1		Furukawa Battery Co., Ltd.	2015	4
DC12V POWER SUPPLY EQUIPMENT	1		JRC	2015	4
AVR(AUTOMATIC VOLTAGE REGULATOR)	1		Sanken Electric Co., Ltd.	1991	28
ISOLATION TRANSFORMER	1		Sankosha Corporation	2015	4
SIREN CONTROL BOARD	1		JRC	2001	18
DMP RECORDING EQUIPMENT	1		JRC	2001	18
DEHYDRATOR	1		Suzuki Giken Co., Ltd.	2001	18
ANTENNA FILER	2	For Alert For Telemeter	Antenna Giken Co., Ltd.	2015	4
COAXIAL ARRESTER	1		JRC	2015	4
+12 DC POWER SUPPLY	1		Nitto Kogyo Corporation	2001	18
+24 DC POWER SUPPLY	1		Nitto Kogyo Corporation	2001	18
CABLE PROTECTOR	2		Takuwa Corporation	2015	4
IP RADIO	1		NEC Corporation	2015	4
AC/DC CONVERTER	1		Emerson Electric Co.	2015	4
MODEM	1		Cisco Systems, Inc.	-	-
MULTI-FUNCTION UNIT	1		Emerson Electric Co.	-	-
WATER LEVEL CONVERTER	1		Takuwa Corporation	-	-
TM/IP INTERFACE	2		JRC	2015	4
TELEMETRY SUPERVISORY EQUIPMENT	1		JRC	2001	18
RADIO EQUIPMENT	1		JRC	2015	4
WARNING SUPERVISORY AND CONTROL EQUIPMENT	1		JRC	2001	18
RADIO EQUIPMENT	1		JRC	2015	4
GPS RECEIVER	1		JRC	2001	18
NTP SERVER	1		JRC	2001	18
TELEMETRY DATA INPUT INTERFACE EQUIPMENT	1		JRC	2001	18
NETWORK SWITCH(L3)	1		Cisco Systems, Inc.	2015	4
UPS	1		APC by Schneider Electric	2015	4
PDB(FOR TELECOM)	1		-	-	-
SPEAKER CABLE PROTECTOR	1		-	-	-

ROSARIO MASTER CONTROL STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
TTC-3	1		-	-	-
STC-3	1		-	-	-

Source: Study Team

Table 6.3.59 Current Status of Telecommunication Facilities (ANTIPOLO RELAY STATION)

ANTIPOLO RELAY STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
PDB	1		-	-	-
ISOLATION TRANSFORMER	1		Shoden Corporation	1992	27
ISOLATION TRANSFORMER	1		Sankosha Corporation	2015	4
DIESEL ENGINE GENERATOR	1		Tokyo Denki Sangyo Co., Ltd.	-	-
FUEL TANK	1		-	-	-
AUTOMATIC CONTROL PANEL	1		Denyo Co., Ltd.	2001	18
VoIPTEL	1		-	-	-
IP RADIO (ROSARIO)	1		NEC Corporation	2015	4
IP RADIO (MMDA)	1		NEC Corporation	2015	4
IP RADIO (NAPINDAN)	1		NEC Corporation	2015	4
IP RADIO (PAGASA)	1		NEC Corporation	2015	4
UPS	1		APC by Schneider Electric	2015	4
AC/DC CONVERTER	4		Emerson Electric Co.	2015	4
NETWORK SWITCH(L3)	1		Cisco Systems, Inc.	2015	4
REPEATER	1		Hytera	-	-
AUTOMATIC VOLTAGE REGULATOR	1		Akari	-	-
POWER SUPPLY	1		Hytera	-	-
UPS	2		-	-	-

Source: Study Team

Table 6.3.60 Current Status of Telecommunications Facilities (PAGASA SCIENCE GARDEN STATION)

PAGASA (SCIENCE GARDEN) STATION	Q'ty	Remarks	Manufacturer	Year of manufacture	Age
48V DC POWER SUPPLY	1		Suzuki Electric Ind. Co., Ltd.	2008	11
ANTENNA COUPLER	1		JRC	1984	35
DEHYDRATOR	1		JRC	1973	46
AC PDB	1		Toritsu Electric Co., Ltd.	2008	11
DC PDB	1		Toritsu Electric Co., Ltd.	2008	11
12V DC POWER SUPPLY	1		JRC	1991	28
D/C POWER SUPPLY EQUIPMENT	1		Sanken Electric Co., Ltd.	1991	28
COAXIAL ARRESTER	1	For DATE	JRC	-	-
COAXIAL ARRESTER	1	For FAX/TEL	JRC	-	-
COAXIAL ARRESTER	1		JRC	2015	4
MDF	1		Shoden Corporation	1991	28
INDOOR PROTECTOR BOX	1		JRC	1985	34
POWER DISTRIBUTION BOARD	1		JRC	1979	40
MAIN DISTRIBUTION FRAME	1		JRC	1979	40
PULSE TRANSFORMER BOX	1		Shoden Corporation	2001	18
SURGE ABSORB TRANSFORMER	1		JRC	2001	18
M-V REPEATER EQUIPMENT	1		JRC	2001	18
RADIO EQUIPMENT	2		JRC	2015	4
ANTENNA FILTER	1		-	2015	4
VHF/UHF FM REPEATER-BASE UNIT	1		Kenwood	-	-
IP RADIO	1		NEC Corporation	-	-

PAGASA (SCIENCE GARDEN) STATION	Q'ty	Remarks	Manufacturer	Year of manufacture	Age
VoIPTEL	1		—	2015	4
TM/IP INTERFACE	1		JRC	2015	4
AC/DC CONVERTER	4		Emerson Electric Co.	2015	4
NETWORK SWITCH	1		Cisco Systems, Inc.	-	-
UPS	1		-	-	-
VOIP GATEWAY	1		-	-	-
NETWORK SWITCH(L3)	1		-	-	-
CONTACT IP CONVERTER	1		-	-	-
NETWORK SWITCH	1		-	-	-
IP RADIO	1		NEC Corporation	-	-
UPS	1		-	-	-

Source: Study Team

Table 6.3.61 Current Status of Telecommunications Facilities (NAPINDAN HCS MONITOR STATION)

NAPINDAN HCS MONITOR STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
TM/IP INTERFACE	1		JRC	2015	4
NETWORK SWITCH(L3)	1		Cisco Systems, Inc.	2015	4
WATER LEVEL CONVERTER	1		Takuwa Corporation	-	-
DMP RECORDING EQUIPMENT	1		Takuwa Corporation	-	-
TELEMETRY EQUIPMENT	2		JRC	-	-
RADIO EQUIPMENT	1		JRC	2001	18
DC12V POWER SUPPLY EQUIPMENT	1		JRC	2015	4
AVR PDB	1		-	-	-
ISOLATION TRANSFORMER	1		Sankosha Corporation	2015	4
LCD DISPLAY	1		-	-	-
MONITOR TERMINAL	1		-	-	-
PRINTER	1		-	-	-
UPS	1		-	-	-
VoIPTEL	1		-	-	-

Source: Study Team

Table 6.3.62 Current Status of Telecommunications Facilities (DPWH HEAD OFFICE MONITOR STATION)

DPWH HO MONITOR STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
MONITOR TERMINAL	1		-	-	-
AC PDB	1		Toritsu Electric Co., Ltd.	2008	11
PRINTER	1		-	-	-
NETWORK SWITCH	1		-	-	-
CONTACT IP CONVERTER	1		-	-	-
UPS	1		-	-	-
VoIPTEL	1		-	2015	4

Source: Study Team

Table 6.3.63 Current Status of Telecommunications Facilities (MMDA MONITOR STATION)

MMDA MONITOR STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
MONITOR TERMINAL	1		HP Inc.	2015	4
ISOLATION TRANSFORMER	1		Sankosha Corporation	2015	4

Source: Study Team

Table6.3.64 Current Status of Telecommunications Facilities (STO. NIÑO WATER LEVEL GAUGE STATION)

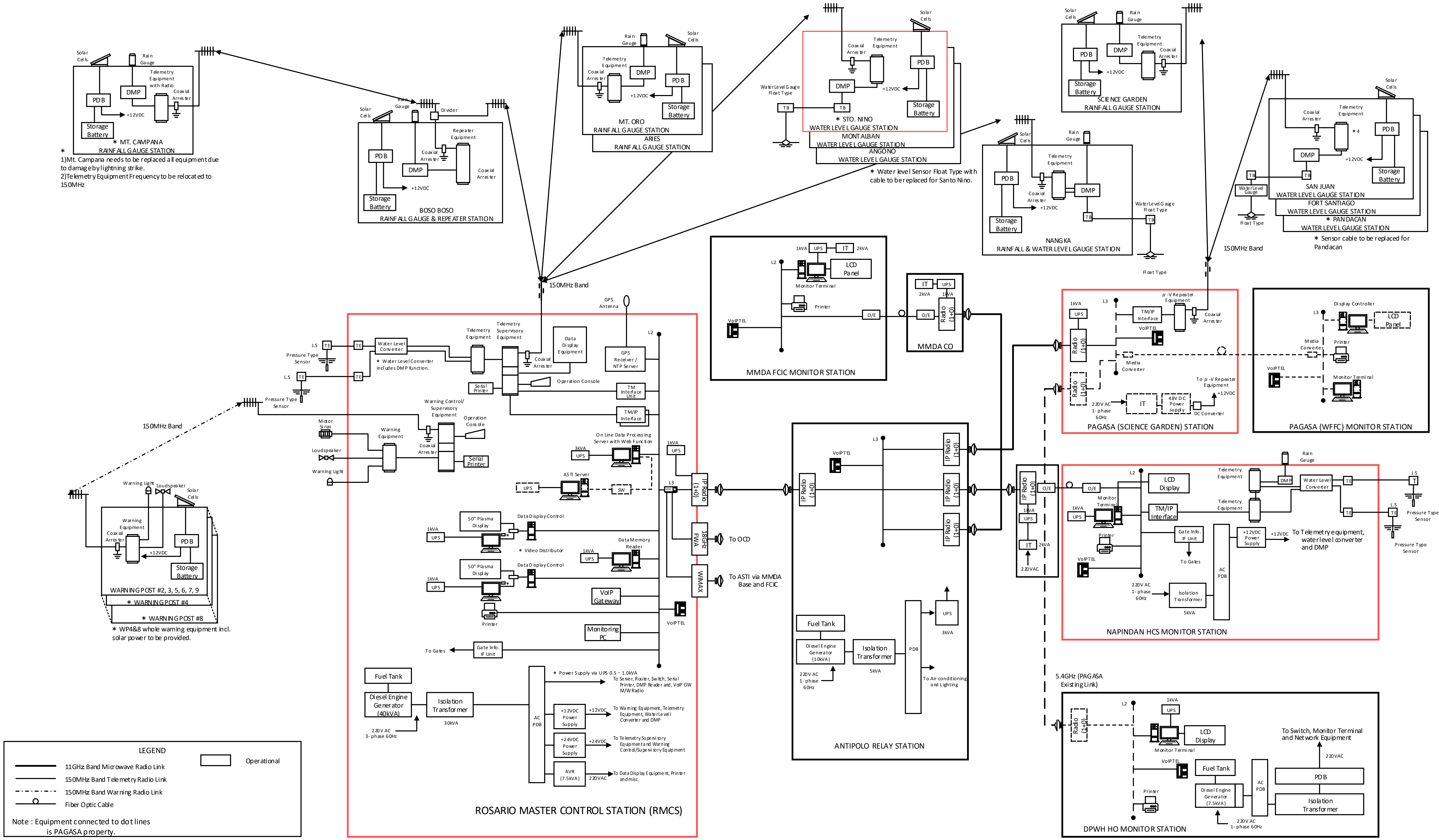
STO. Niño WATER LEVEL GAUGE STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
DMP RECORDING EQUIPMENT	1		Takuwa Corporation	2015	4
TELEMETRY EQUIPMENT	1		JRC	2001	18
RADIO EQUIPMENT	1		JRC	2015	4
PDB	1		JRC	-	-
COAXIAL ARRESTER	1		JRC	2015	4
TERMINAL BOX WITH ARRESTERS	1		JRC	2015	4
STORAGE BATTERY	1		Furukawa Battery Co., Ltd.	2015	4
SOLAR CELL	1		-	-	-
WATER LEVEL RECORDER	1		Ogasawara Keiki Co., Ltd.	2015	4
TERMINAL BOX WITH ARRESTERS	1		JRC	2015	4

Source: Study Team

Table6.3.65 Current Status of Telecommunications Facilities (SCIENCE GARDEN RAINFALL GAUGE STATION)

SCIENCE GARDEN RAINFALL GAUGE STATION	Q'ty	Remarks	Manufacturer	Year of Manufacture	Age
DMP RECORDING EQUIPMENT	1		Takuwa Corporation	2015	4
TELEMETRY EQUIPMENT	1		JRC	2015	4
RADIO EQUIPMENT	1		JRC	2015	4
PDB	1		JRC	-	-
BATTERY	1		Furukawa Battery Co., Ltd.	2015	4

Source: Study Team



Source: Study Team Prepared based on the document by manufacturers

Figure6.3.50 System Configuration Diagram (Renewal of Facilities is Needed in Sites with Red)

6.3.6 Incidental Facility

6.3.6.1 Outline and layout plan of ancillary facilities

Besides the flood gate structures, ancillary facilities listed in **Table 6.3.66** shall be considered.

Table 6.3.66 Facilities attached to MCGS

Facilities	Use And Purpose	Remarks
Installation Revetment	Sliding from Wing Wall to Revetment Shape in Upstream and Downstream Section	
Controlled Road	Access from the EFCOS site	
Parking Space	Parking of managed vehicles	
Generator Building	Installation of emergency generators, installation of monitoring and control panels, and standby areas for operators and security personnel.	Including water supply pipes and sewage treatment facilities
Fence And Gate Door	Prevention of illegal entry and fall from wing walls and weir posts	
Outdoor Lighting	Lighting for managed roads and parking spaces	

Source; Study Team

The layout plan of MCGS ancillary facilities is shown on Figure 6.3.51 in the next page. Following considerations were considered in layout planning ancillary facilities.

[Maintenance roads and parking spaces]

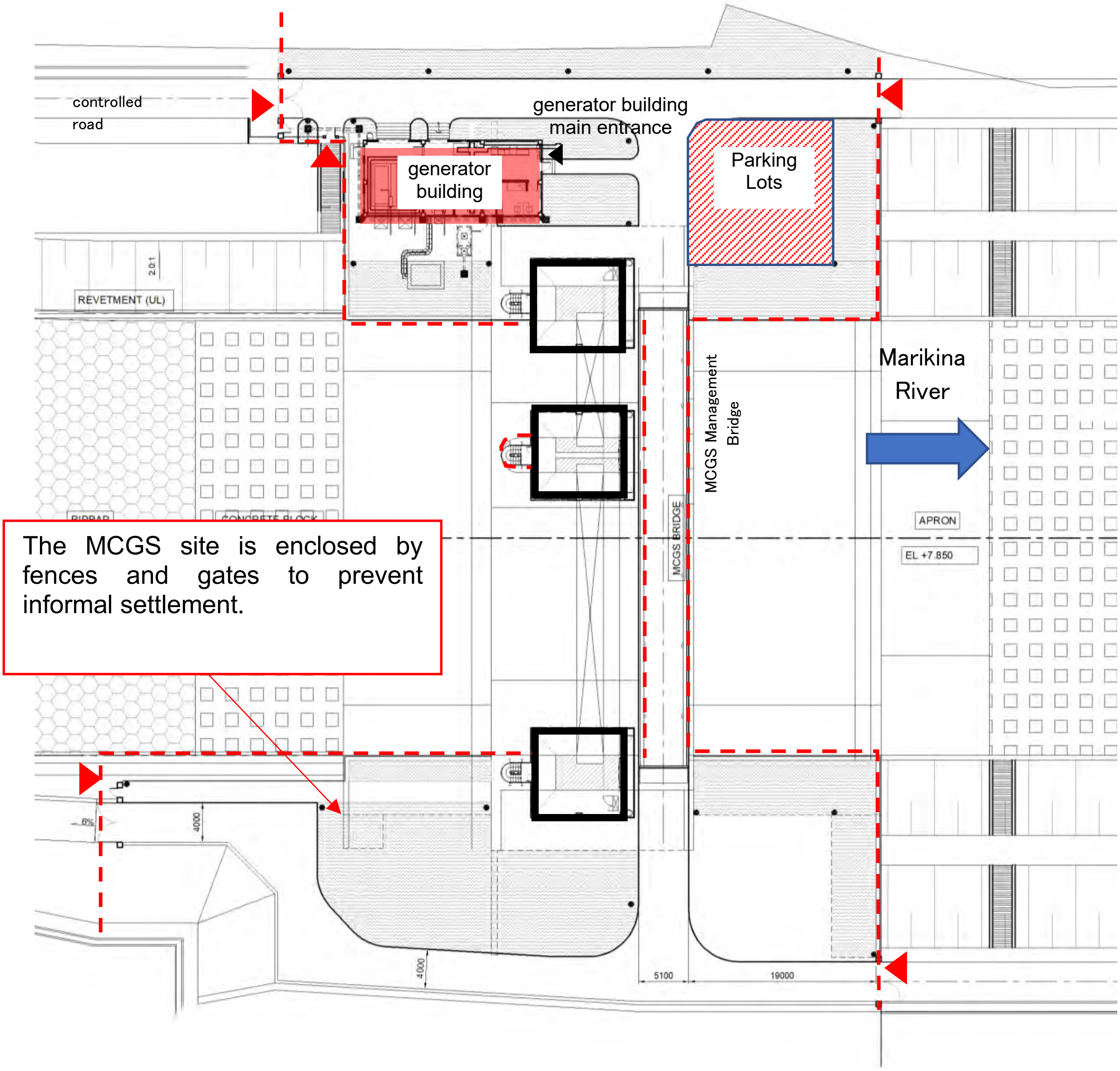
An access road from the EFCOS site to MCGS will be constructed on the left bank of upper stream. Maintenance roads around MCGS shall be designed as extension of the access road and the width is widened to allow vehicles to pass each other. The space between the maintenance road and river on the left bank at downstream side can be used as parking lots.

[generator building]

The generator building shall be located on the left bank where the access road from the EFCOS site will be constructed. The EFCOS site is more than 600 meters away from MCGS, making it impractical to connect to public sewerage systems. Therefore, sewage from the generator building shall be treated on site by a septic tank in accordance with local standards.

[Fence and gate door]

The fences and gate doors should be installed not only to prevent falling to from the structure to the river, but also to prevent informal settlement, illegal dumping of waste and theft of facilities.



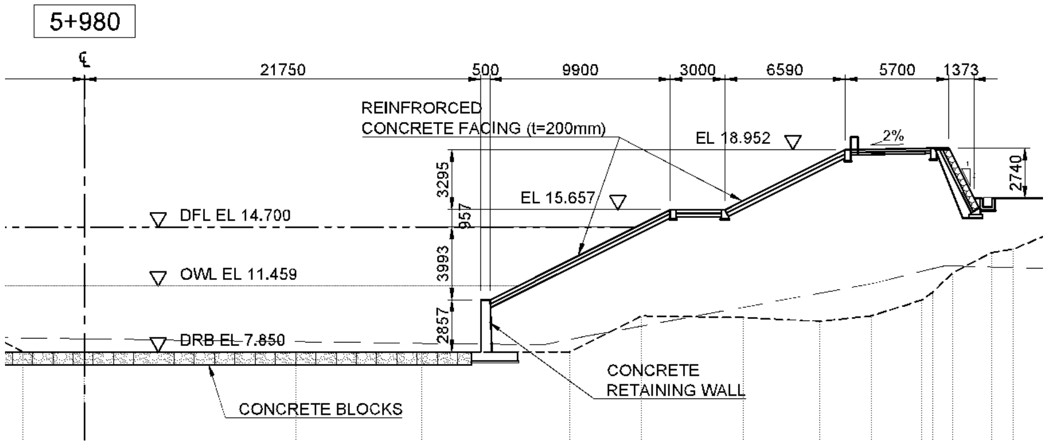
Source: Study Team

Figure 6.3.51 Outline of MCGS Site Development Plan

6.3.6.2 Revetment

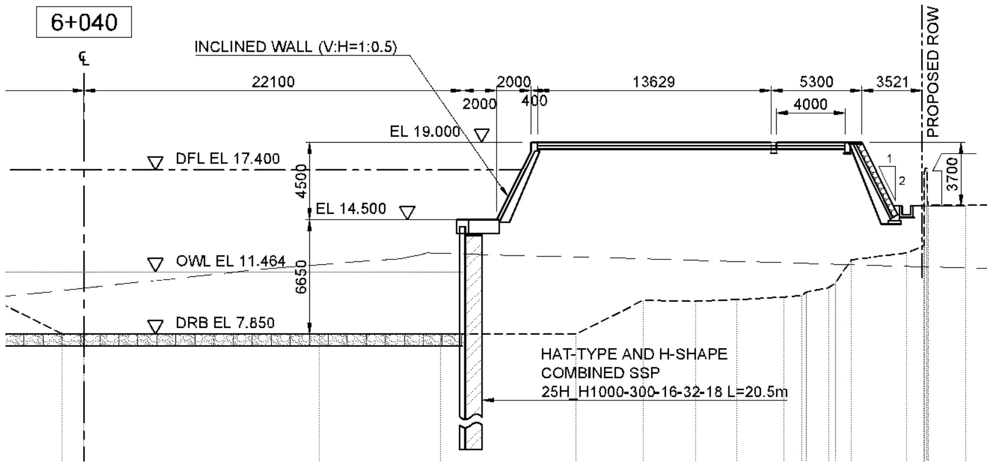
The revetment is installed to make the shape of revetment in the upstream and downstream fit to the structure smoothly. Revetment basic specifications are summarized below.

・ Scope of Maintenance	:	Up To the point of connection with a newly installed revetment
・ Revetment Type	:	(Upstream Right Bank Side) ... Follow the type of revetment in the upstream. Slope ... Inclined Wall Foundation ... Steel sheet Pile Revetment (The other portion) Slope ... Reinforced Concrete Facing Foundation ... Invert T Retaining Wall or Steel Sheet Pile Revetment
・ Slope	:	(Upstream Right Bank Side) 0.5: 1 (The other portion) 2.0: 1
・ Revetment Construction Crown Height	:	Design Dike Crown + Extra Embankment
・ Design Riverbed	:	EL. + 7.850



Source: Survey team

Figure6.3.52 Revetment in the downstream side of MCGS



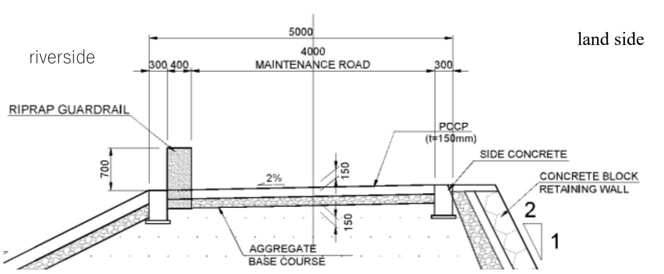
Source: Survey team

Figure6.3.53 Revetment in the upstream side of MCGS

6.3.6.3 Maintenance Road

The purpose of the maintenance road around the MCGS is as follows. In consideration of the following purposes, maintenance road with an effective width of 4.0 m is provided.

- Access to the MCGS for operation
- Passage of construction vehicles such as cranes during repair of facilities in addition to daily river patrols
- Use as a walking path in normal times



Source: Survey team

Figure6.3.54 Standard Cross-Section of the Maintenance Road

6.3.6.4 Generator House

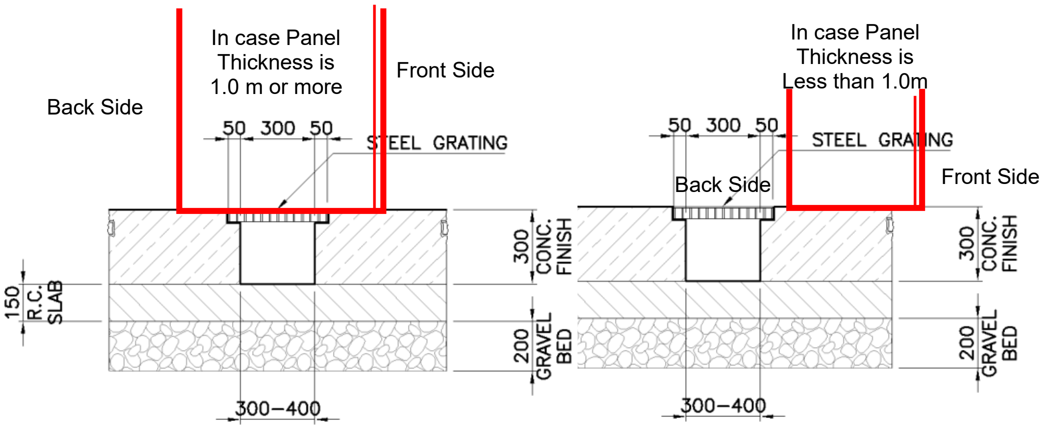
(1) Design Conditions of the Generator Building

The generator house contains emergency generators, control panels and stuff room for operation stuff and guard man to standby. Major equipment installed in the local control house are listed in the table below. The number and dimensions of generators and panels are based on the results of the previous section.

Table 6.3.67 Major equipment installed in the MCGS generator building

Room Name	Equipment or Facilities	Approximate Dimensions (W x L)	Remarks
Generator Room	Generator (For Switching Power) No.1	1,200 x 3,700	
	Generator (For Switching Power) No.2	1,200 x 3,700	
	Generator (For Control Equipment)	1,000 x 2,100	
Electrical Room	Receiving Panel	1,000 x 1,200	
	Main Distribution Board	1,000 x 1,200	
	Uninterruptible Power Supply	700 x 600	
	Lightning Protection Transformer	700 x 600	
	Remote Control System Control Panel	700 x 600	
	Water Level Gauge Control Panel	700 x 600	
	Monitoring Camera Control Panel	700 x 600	
	Simplified Gate Operation Panel	700 x 600	
	Control Panel For Generators (For Switching Power)	1,000 x 1,000	
	Power Storage Panel For Generators (For Switching Power)	1,000 x 1,000	
	Control Panel For Generators (For Control Equipment)	1,000 x 1,000	
	Power Storage Panel For Generators (For Control Equipment)	1,000 x 1,000	
Stuff Room	Desk (For 2 People)		
	Bed (Double)		
	Pantry		Shall follow requirement specified in NBCP
	Toiletry, Shower		Shall follow requirement specified in NBCP

In the generator room and electrical room, cable pit shall be installed as shown below.



Source: Study Team

Figure 6.3.55 Typical Section of Cable Pit for Generator House

The required separation of the generator room is as follows, based on the "Telecommunications Facilities Design Guidelines (electric (al) knitting)".

Layout planning of local control house shall be based on size of equipment and clearance around equipment. Required clearance for inspection and maintenance works are provided by type of equipment as follows.

- ✓ Minimum clearance for operation: 80 cm
- ✓ Minimum clearance for maintenance works: 60 cm
- ✓ Minimum clearance for ventilation: 20 cm

*However, 100 cm or more is required for the side facing transformer, battery or concrete wall.

Table 6.3.68 Minimum Clearance around Generator

Portion To Secure The Separation Distance			Separation Distance
Cubicle Type		Operation Side	More than 1.0m
		Maintenance Side	More than 0.6 m In case of transformers, storage batteries and portion facing to a building, 1.0m
		Ventilation Side	More than 0.2 m
Not Cubicle Type	Generator	Between Generators	More than 1.0 m
		Around Generators	More than 0.6 m
	Generator Control Board	Operation Side	More than 1.0 m (In case of facing each other, 1.2m)
		Maintenance Side	More than 0.6 m (In case of facing each other, 1.0m)
		Ventilation Side	More than 0.2 m
	Fuel Tank Of Small Lots	Internal - combustion engine	More than 0.6 m More than 2.0 m
		Inside surface of Oil Retaining Wall	According to Local Law
	DC Power Supply Equipment	Operation Side	More than 1.0 m
		Maintenance Side	More than 0.6 m

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3-65

(2) Layout of the generator building

The following considerations shall be considered for layout planning of generator house.

[electrical room]

- ✓ Efficiency of cabling layout shall be considered, such as grouping of control panels (panels for gate operation, generator control and power receiving/distribution).
- ✓ Efficiency of flow line of stuff shall be considered, such as good visibility and easy access to each room from the main corridor.
- ✓ A steel shutter shall be provided facing road with sufficient width for control panels to pass through.
- ✓ Handholes shall be provided for cables to local gate control houses and from EFCOS building.

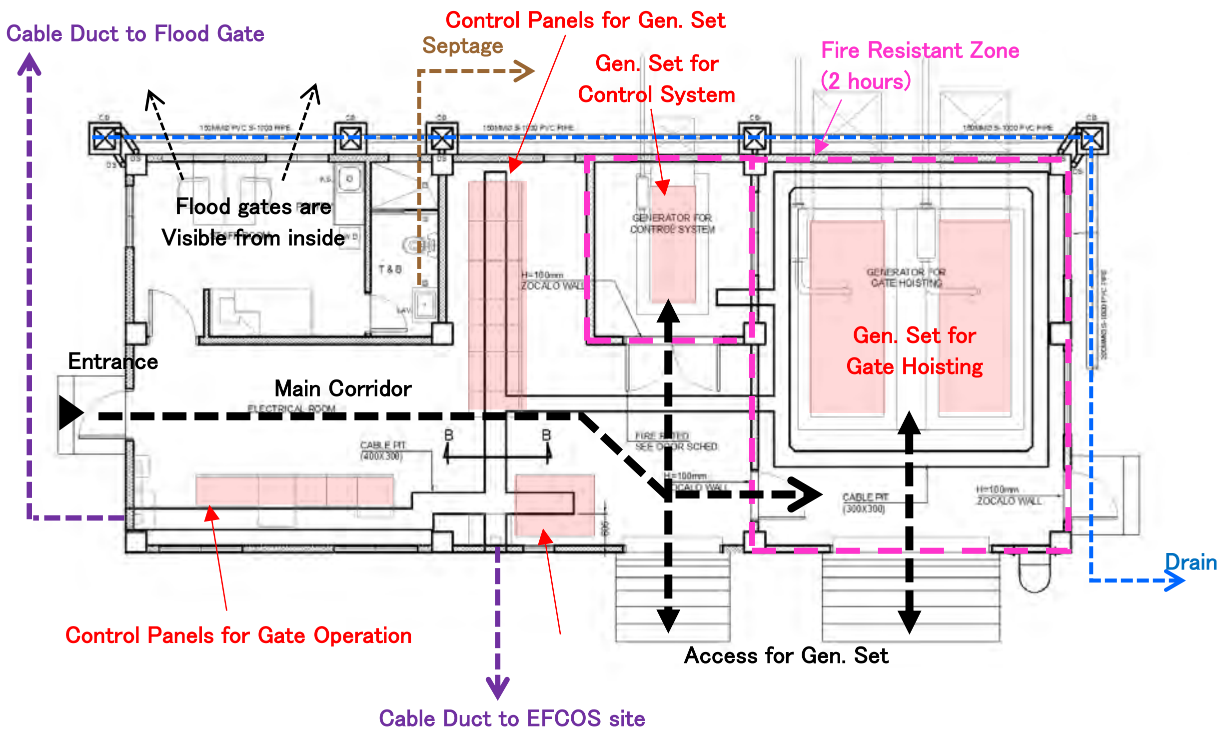
[generator room]

- ✓ Based on local fire codes and enforcement regulations, the maximum area of a room to store fuel is 46.5 m2 under conditions of 2hour fire-rated wall/doors and no sprinkler. Since, it is difficult to house three generators in a same room, generators for gate hoisting and a generator for control system may be planned separately.
- ✓ A steel shutter shall be provided facing road with sufficient width for generators to pass through.

[Other drainage facilities]

- ✓ The stuff room is desirable to be facing the gate structures so that stuff can overlook them easily from inside the room.
- ✓ Rainwater on the roof maybe gathered at river side of the building to avoid interference with the handholes and electrical conduits.
- ✓ Septic tank may be located nearby toilet and bathroom on the river side of buildings that do not interfere with electric wires.

Figure 6.3.56 shows layout plan of MCGS generator house considering the above-mentioned conditions.



Source: Study Team

Figure 6.3.56 Layout Plan of MCGS Local Control House

(3) Cross-sectional Plan of Generator House

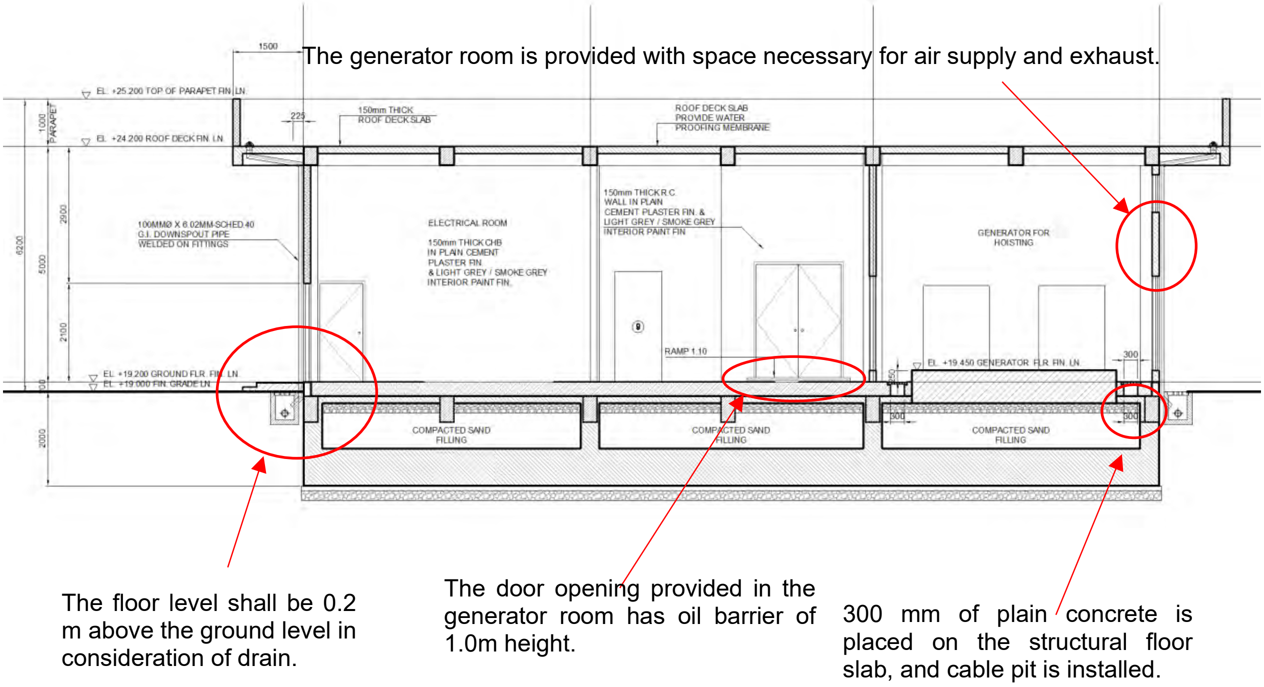
The cross-sectional design of the generator building shall be determined considering the following factors.

[Ceiling Height]

- ✓ The ceiling height of the generator room should be designed to allow sufficient louver height for the supply/exhaust of the generator.

[Floor Height]

- ✓ The floor level is assumed to be 0.2 m above the surrounding ground level in consideration of rain.
- ✓ 300 mm of plain concrete is placed on the structural floor slab, and cable pit is installed. Therefore, the structural floor slab has a floor level of 0.3 m.
- ✓ The door opening installed in the generator room has a lower end height of 0.1 m above the floor level and functions similar to an oil dike in accordance with local fire defense standards and enforcement regulations.



Source: Study Team

Figure 6.3.57 Typical Section of MCGS Generator House

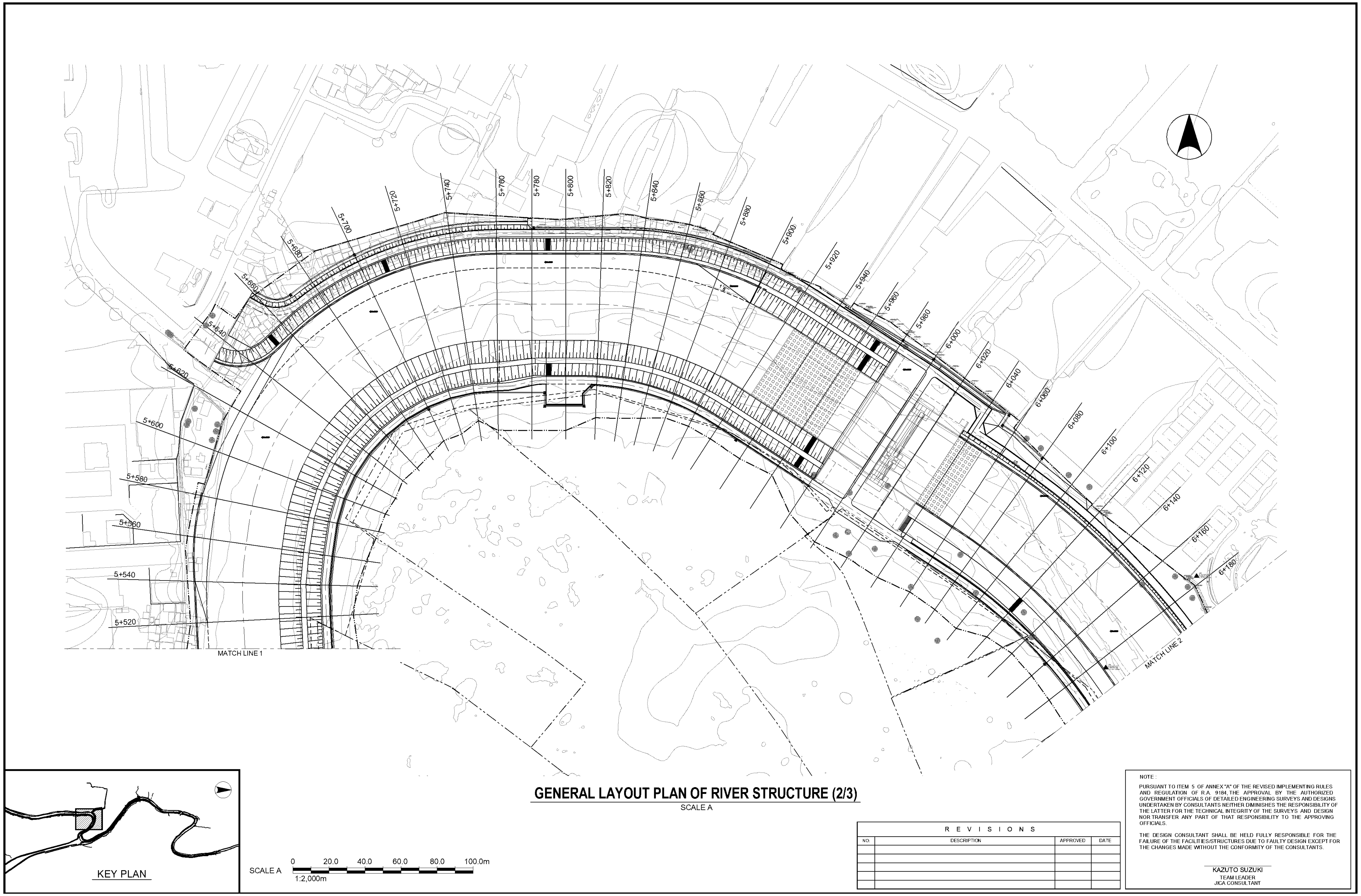
6.3.6.5 Necessity of Spare Gates (Stop Logs)

Considering the following items, spare gates would not be installed but only the supporting frame for stop logs would be installed in MCGS.

- The purpose of the spare gates is to use as a cofferdam during the maintenance around the main gates. Since it is not practical, using instead of the main gate would not be considered.
- In this design, the material of the gate leaves and guide frames is corrosion resistant. Hence, the maintenance with cofferdam around the main gates will not be frequently performed.
- Considering the above 2 items, it is better to prepare the spare gate when it is needed rather than to store the spare gates which needs periodical maintenance. Also, it is superior also in the reliability.

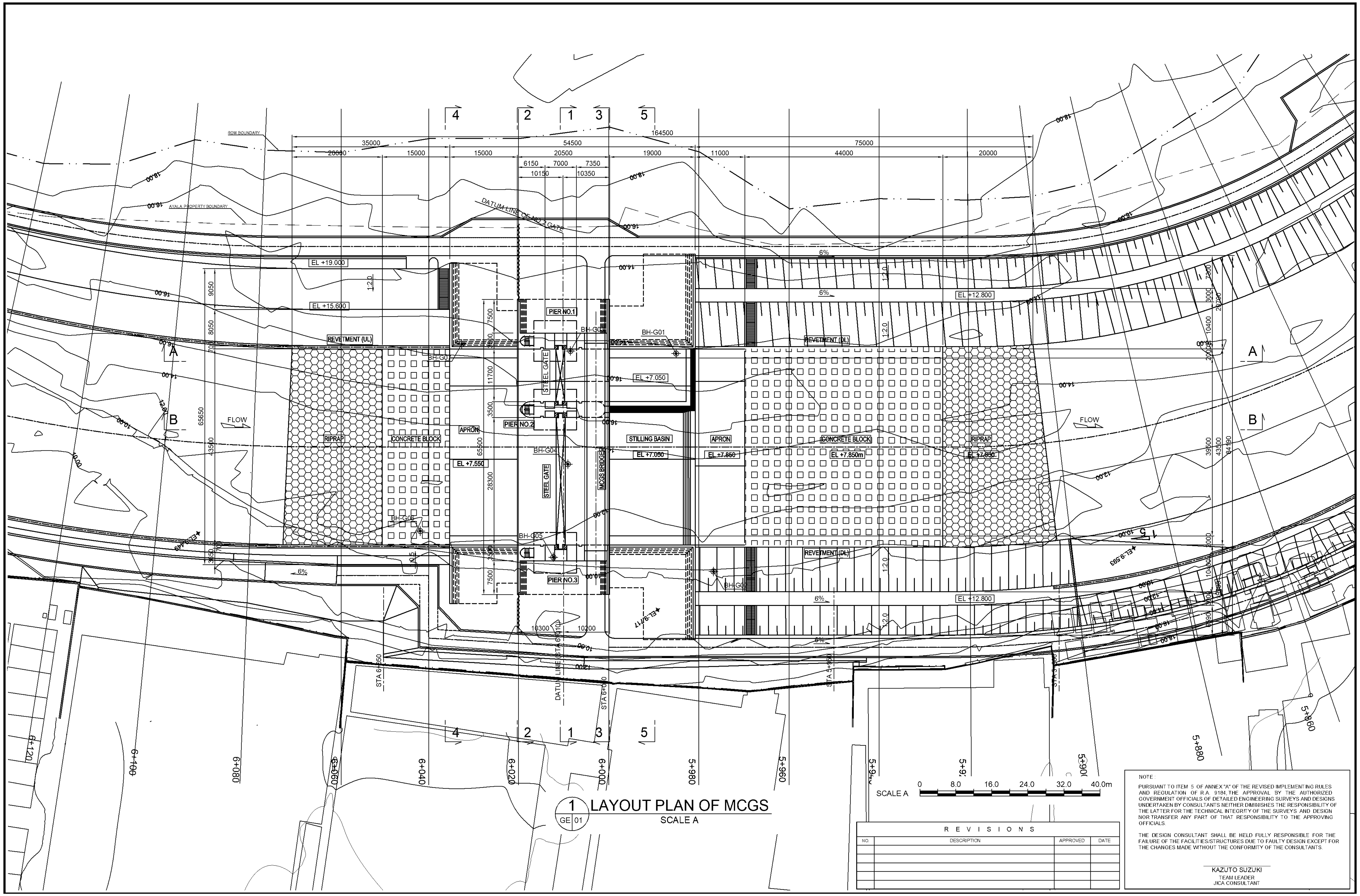
6.3.7 General Drawings

General drawings are prepared based on the above conditions and basic considerations as shown in the following pages.



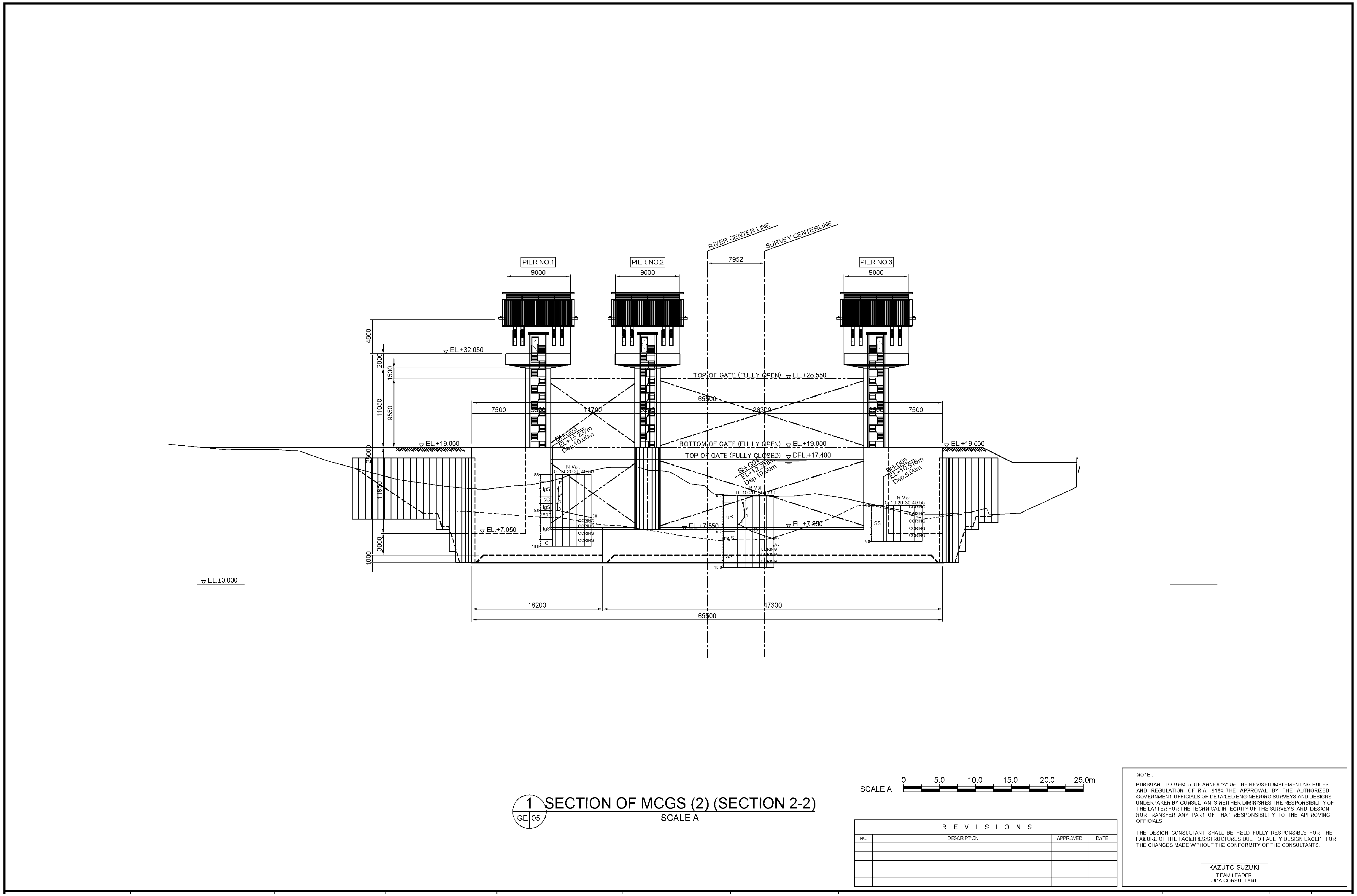
Source: Study Team

Figure 6.3.58 General Layout Plan of MCGS



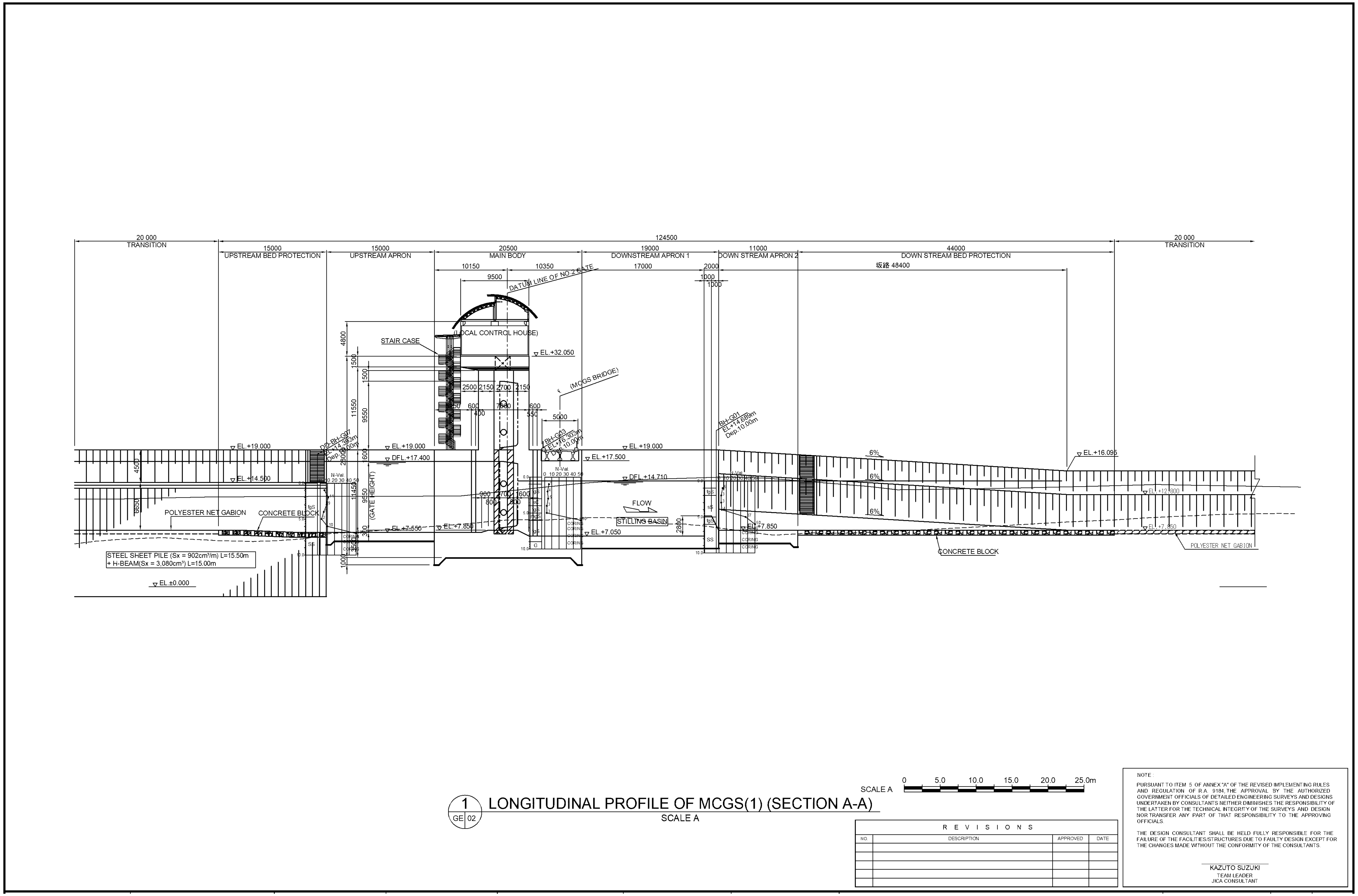
Source: Study Team

Figure 6.3.59 MCGS General Drawings (1)



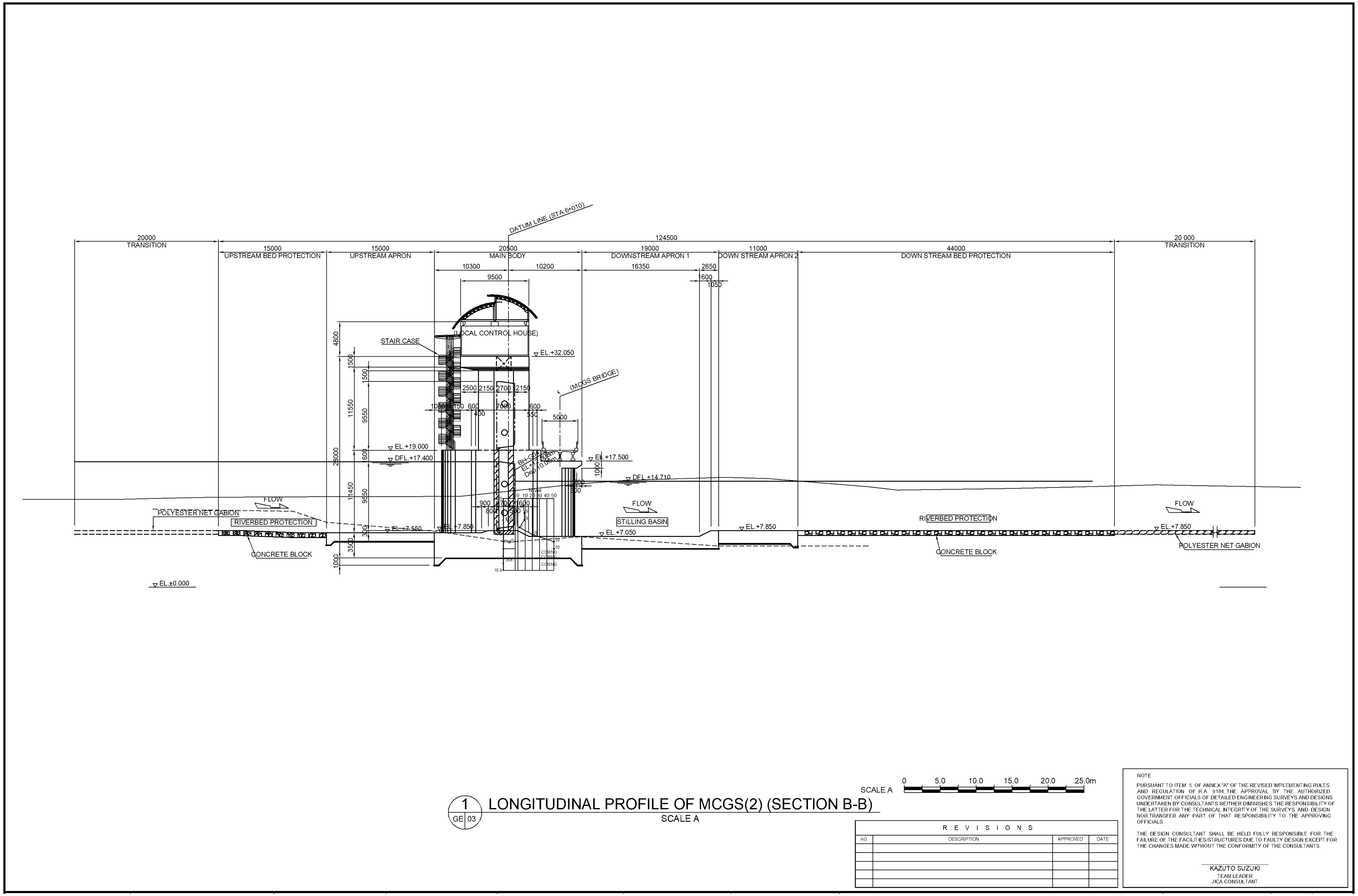
Source: Study Team

Figure 6.3.60 MCGS General Drawings (2)



Source: Study Team

Figure 6.3.61 MCGS General Drawings (3)



Source: Study Team

Figure6.3.62 MCGS General Drawings (4)

6.4 Floodgate to Prevent Backflow

6.4.1 Summary of Basic Design of Floodgates to Prevent Backflow

The Cainta Floodgate and Taytay Sluiceway way along Manggahan Floodway were proposed in the Study on Flood Mitigation Project in the East Manggahan in 2008 (hereinafter referred to as 2008 Pre-F/S) based on the discharge from a tributary river with 10-year return period. In this study, the detailed design of the above two floodgates will be conducted to prevent backflow from the Manggahan Floodway during floods.

When the Philippine standard was updated in 2015, the design scale was upgraded to 15 years. The review of the plan based on this new standard will require change to the distribution of discharge in the landside, revision of the river improvement plan, and review of the facility layout plan for the discharge of floodwater to the floodway.

However, the drainage distribution from the Cainta Floodgate and Taytay Sluiceway was determined under the existing flow capacity of the river channel or box culvert. Therefore, even if the scale is increased, the current discharge distribution will be maintained.

Therefore, in this study, based the existing river width of the Cainta River around the floodgate location and the flow capacity of the existing box culvert from Taytay Creek to Manggahan Floodway, each floodgate is being studied.

In the basic design, the following studies are mainly carried out:

- ✓ Setting the location of floodgates
- ✓ In case of Taytay Sluiceway, selection of structure type (floodgate or sluiceway.) → select sluiceway
- ✓ Study on the basic specifications of civil structures (Dimensions)
- ✓ Study on the gate type, structure, and material
- ✓ Study on machine and electrical equipment

A list of the specifications established in the above studies is shown as the basic design results.

Table 6.4.1 Summary of Basic Design Results (Cainta Floodgate)

Item	Specifications	Description / Remarks
Function	Water Control Function	・ To prevent backflow from Manggahan Floodway during floods
Location	STA. 4 + 525	・ Approximate center of the flow of Cainta River
Proposed Discharge	95 m^3/s	・ Based on the existing river width, this was set in 2008 Pre-F/S
DFL	Floodway Side: EL. 14.853 m Tributary Side: EL. 13.34 m	
Water Level (for Structural Design)	Floodway Side: EL. 14.853 m Tributary Side: EL. 10.1 m	・ Based on the observed lowest water level in Laguna Lake
(for Operation)	Floodway Side: EL. 12.940 m Tributary Side: EL. 13.940 m	・ Water level in floodway side is 1 m below the dike crown of tributary 1) ・ Water level in tributary is dike crown 1)
Dike Crown (Design)	Floodway Side: EL. + 15.940 m Tributary Side: EL. 13.940 m	
(Existing)	Floodway Side: EL. + 18.00 m Tributary Side: EL. + 13.00 m	・ In case of the tributary, the existing ground elevation (After topographic survey, it will be reconfirmed)
Number of Gates	2 Gates	・ Considering the redundancy in case of malfunction ・ Avoiding the size of the gate becomes large
Span	2 spans x 19.0 m (Pure Diameter: 2 x 16.00 m)	・ Minimum 15.0m 2) ・ Based on the existing rive width which is about 35 m (The width of proposed river channel was 34.6 m 2008 Pre-F/S)
Invert Elevation	EL. 8.75 m	・ Design riverbed of the tributary
Length of Main body	31.9 m	・ Considering the width of maintenance bridge, staircase, column, pier

Item	Specifications	Description / Remarks
Length of Apron	Upstream: 11.5 m	・ Same as the length of wing wall
	Downstream:18.0 m	・ Same as the length of Bed Protection in the Downstream
Length of Bed Protection	Upstream: 5 m	・ Approximately same as the water depth at DFL3）
	Downstream side: A: 0.0 m B: 15.0 m	・ A: w/o Bed Protection Section A ・ B: 3-5 times of water depth in the downstream3）
Top of Main Body	EL. 18.4 m	・ Finished elevation of the dike (including extra embankment)
Top of Gate Control Structure	EL. 29.4m	・ Elevation of the bottom of gate during its maintenance is set to the dike crown EL.+18.0m. ・ Considering 1.6 m allowance abovementioned elevation
Top of Gate	EL. 16.060 m (Top of Gate)	・ Rounding the value of DFL in floodway side + Freeboard 1.2 m
Type of Gate	Fixed Wheel Roller Gate	・ Selected based on maintenance and economic aspect
Pier Structure/ Material	Girder Structure/ Alloy-saving Duplex Stainless Steel	・ (Structure) Considering garbage/branches flowing and sedimentation, furthermore, cost efficient ・ (Material) Considering LCC
Operation	Hoist：Electric motion (commercial power supply) Operation；Remote and Local Control	・ Commercial power supply is used with 1 units of generators for backup in case of blackout ・ In addition to remote control and machine side, emergency control panel is installed in generator house
Maintenance Bridge	RC Bridge (Effective width: 7.30 m x 2 or more)	・ Maintenance bridge is open to public. ・ Considering the expansion to 4 lane road

1) *Technical standard for the Facilities of Dams and Weirs*
2) *Cabinet Order Concerning Structural Standards for River Administration Facilities, etc.*
3) *Structural Design Guide for Groundsill*

Source: Study Team

Table 6.4.2 Summary of Basic Design Results (Taytay Sluiceway)

Item	Specifications	Description/ Remarks
Function	Water Control	・ To prevent backflow from Manggahan Floodway during floods
Location	STA.6+090	・ Approximate center position of the existing box culvert
Proposed Discharge	30 m^3/s	・ Based on the existing flow capacity (same as 2008 Pre-F/S)
DFL	Floodway Side：EL.+14.520 m Tributary Side：EL.+13.500 m	・ DFL of the Tributary is at the upstream end of the existing box culvert ・ Bottom side of top slab of the box culvert is EL.+12.4m
Water Level (for Structural Design)	Floodway Side：EL.+14.520 m Tributary Side：EL.+10.100 m	・ Based on the observed lowest water level in Laguna Lake
(for Operation)	Floodway Side：EL.+13.100m Tributary Side：EL.+14.100m	・ Water level in floodway side is 1 m below the dike crown of tributary [1)] ・ Water level in tributary is dike crown [1)]
Dike Crown (Design)	Floodway Side：EL.+15.620m Tributary Side：EL.+14.100m	・ About the tributary side, it is DFL in the upstream side of the existing box culvert
(Existing)	Floodway Side：About EL.+15.6m Tributary Side：About EL.+11.8-13.5m	・ In case of the tributary, the existing ground elevation
Type of Structure	Sluiceway Connecting with the Existing One	・ Considering advantages in the aspect of structure, maintainability for seismic resistance and workability
Number of Gates	3	・ Same as the existing box culvert
Size of Box Culvert	B2.5m x H1.8m x 3 Barrels	・ Same as the existing box culvert ・ Considering residual settlement, the value shall be reviewed in the detailed design stage.
Invert Elevation	EL. +10.600 m	・ Design Riverbed, Invert Elevation of the existing Box Culvert
Length of Box Culvert	Conduit：8.0m	・ Considering the 70 cm height of breast wall, 2.0:1 side slope

Item	Specifications	Description/ Remarks
Breast Wall	Width：1.0 m	・ To avoid particles of embankment moving and drawing out [1*]
	Height：0.7m	・ Haunch 50 cm +20 cm
Wing Wall	Downstream: Length: 6.0m、Height: 2.1m	・ (Length) Longer than cross-sectional shape of the existing dike [2)]
		・ (Height) considering the height of existing dike
Top of Gate Control Structure	EL. 16.200 m	・ 0.5 m allowance above the top of the opened gate.
Top of Gate	EL.12.4000	・ Bottom elevation of the top slab of the existing box culvert
Type of Gate	Fixed Wheel Roller Gate	・ Due to much garbage, assurance of closure is prioritized.
Pier Structure/ Material	Alloy Saving Duplex Stainless Steel	・ Considering less maintenance and technical novelty, stainless is selected ・ Since the life cycle cost is almost the same as the conventional type, high strength and same type as Cainta Floodgate is selected
Operation	Hoist：Electric motion (commercial power supply) Operation；Remote and Local Control	・ Commercial power supply is used with 1 unit of generators for backup in case of blackout ・ Remote control and machine side.
Maintenance Bridge	Steel (width: 1.0m)	・ Access by the operator and administrator

1) Technical standard for the Facilities of Dams and Weirs
2) Guideline for Flexible Sluiceway
Source: Study Team

6.4.2 Background and Purpose of Installation

6.4.2.1 Background

In this subsection, backflow prevention facilities installed at the confluence point of the Cainta River and the confluence point of the Taytay Creek in the left bank side of Manggahan Floodway are studied.

Manggahan Floodway was constructed in 1988 to discharge some of the floodwater from the Marikina River into Lake Laguna. As of 2019, about 30 years have passed since the start of the operation, and it has had a certain effect on flood damage control in the downstream of the Marikina River and the Pasig River.

On the other hand, there are no gates at the Confluence of the Cainta River and the Confluence of the Taytay Creek along the Manggahan Floodway. When the water level of the floodway is high, backflow along the tributaries occurs.

In this connection, the Study on a Flood Mitigation Project in the East Manggahan Floodway Area (hereinafter referred to as 2008 Pre-F/S) was conducted in 2008 to improve drainage in the left side of the floodway to a 10-year return period flood.

This Pasig Marikina River Channel Improvement Project Phase IV include, together with the countermeasures planned in the above study, facilities to prevent the backflow of floods from the Manggahan Floodway at the confluence of the Cainta River and the confluence of the Taytay Creek. In the 2008 plan, drainage from the Cainta Floodgate and Taytay Sluiceway was based on the flow capacity of the existing river channels and box culverts.

6.4.2.2 Update of the Standard for Drainage Planning

The DPWH standard was updated in 2015. The change increased the design scale of drainage planning as shown in **Table 6.4.3**. According to the current DPWH standard, it is necessary to plan drainage facilities in the land side of the dike at least on a 15-year return period.

Table 6.4.3 Design Scale in Planning of Drainage Facility

Table 6-1 Minimum Capacity of Drainage Infrastructure

Land-use (Note 1)	Minor System		Major Drainage System Drainage Capacity (Note 2)
	Design Capacity	Check Capacity	
Drainage Pipes	15 year flood	25 year flood	100 year flood
Culverts (Note 1)	25 year flood	50 year flood	
Esteros/ creeks/ drainage channels	15 year flood	25 year flood	

Note 1. Refer to Volume 4 for highway cross drainage structure capacities
Note 2. Freeboards for buildings are detailed in Volume 6: Public Buildings and Other Related Structures.

Source: DGCS Vol. 3

6.4.2.3 Purpose and Policy on the Installation of Facilities in PMRCIP-IV

Based on the background mentioned above, it would be appropriate to update the drainage plan, facility layout, and river improvement plan according to the current standards, and then set the necessary width and height of the floodgates. However, as of May 2019, no facility layout and river improvement plans based on the updated drainage plan have been established, and updating of the plans is still scheduled the next year based on the information from DPWH-UPMO-FCMC.

Increasing the design scale of the drainage planning requires the following considerations:

Redesign of Proposed River Cross Section	: Widening of river channel, lowering of design riverbed, or raising of DFL of the tributaries
Reconsideration of the Facility Layout	: Revising the proposed discharge distribution from floodgates and pumping stations in the previous plans or adding drainage facilities.

On the other hand, drainage from the Cainta Floodgate and Taytay Sluiceway, which is the subject of this project, is based on the flow capacity of the existing river channels and box culverts. Therefore, the proposed discharge distribution shall be maintained even if the scale of the project is increased.

The purpose of installing both a floodgate and a sluiceway in this project is to prevent the backflow of floods from the Manggahan Floodway to the Cainta River and the Taytay Creek, not the drainage improvement in the land side of the dike. In addition, since the drainage channel in the land side has not been improved yet, if the floodgate is installed in accordance with the policy proposed in the 2008 Pre-F/S, it will not become a bottleneck to the current condition and will not have any adverse effect. Therefore, in this project, the floodgates to prevent backflow will be installed based on the following policy.

- ✓ As proposed in the 2008 Pre-F/S, the Cainta Floodgate will be installed on the existing river channel width while the Taytay Sluiceway is on the existing box culverts.
- ✓ When the drainage plan in the land side is reviewed from the 10-year return period, the flow rate to the floodway in the initial stage of flood (proposed flow rate from both floodgates) is assumed to be maintained at the present rates, i.e., 95 m^3/s and 30 m^3/s, respectively.

6.4.3 Basic design of Cainta Floodgate

6.4.3.1 Water Level Condition

Water level conditions at Cainta Floodgate are as shown in **Table 6.4.4**.

Table 6.4.4 Water Level Condition of Cainta Floodgate

Water Level Condition	Cainta River (EL.+m)	Manggahan Floodway (EL.+m)	Remarks
DFL	13.340 1*	14.853	DFL of Manggahan Floodway is calculated by interpolation from the completed drawing 2*
During Flood	10.942	14.853	The water level in Cainta River side is based on the LWL of Manggahan Floodway

Source: 1 2008 Pre-F/S*
2 Final Report on Consulting Services for Manggahan Floodway Project*

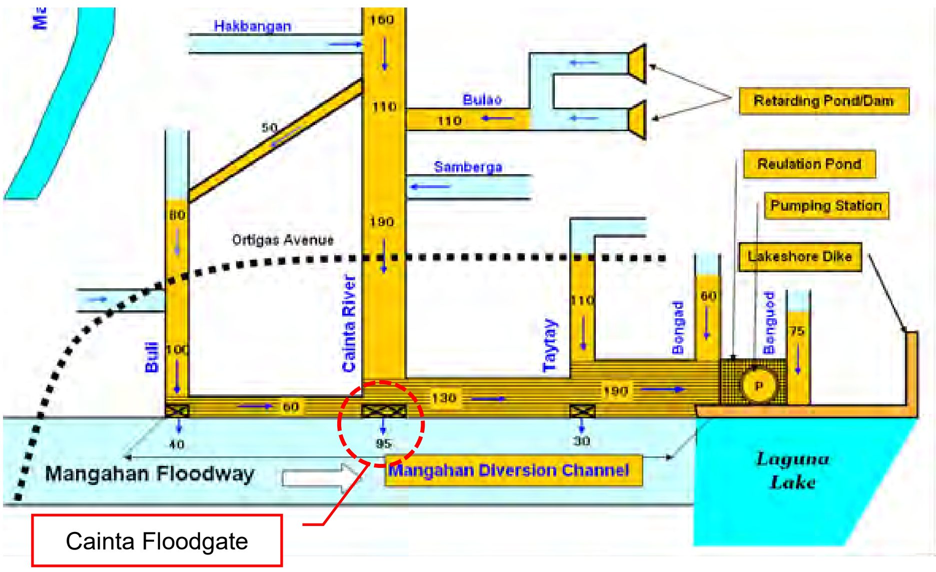
6.4.3.2 Navigation and Other Conditions

At present, there are no boats or ferries operating or crossing the Cainta Floodgate location. Therefore, the conditions for navigation are not considered when studying the height of the lower end of the gate when fully opened.

6.4.3.3 River Condition

(1) Proposed Discharge Distribution

The proposed discharge distribution around the floodgate location is shown in **Figure 6.4.1**. It is set at 95 m^3/s based on the cross section of the existing river channel. **Figure 6.4.1** is the proposed discharge distribution at 10-year return period; however, the proposed discharge from the Cainta Floodgate will be maintained even if the project scale is increased.



Source: The 2008 Pre-F/S

Figure 6.4.1 Distribution of Proposed Discharge

(2) River Channel Condition

Table 6.4.5 shows a list of river channel conditions.

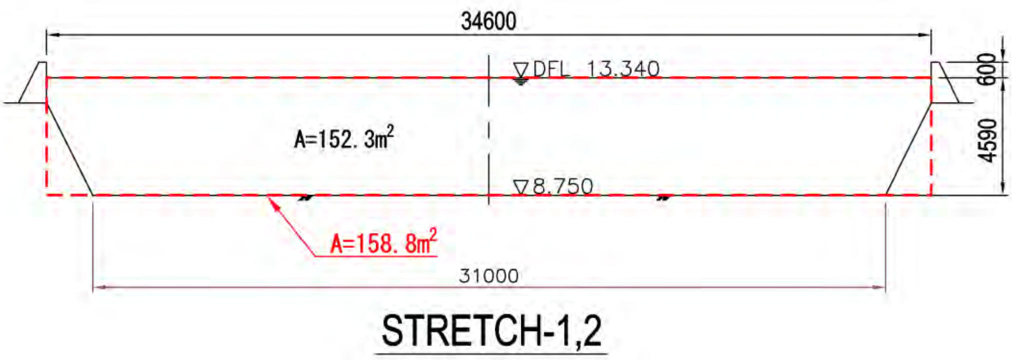
Table 6.4.5 List of River Channel Conditions

Items		Tributary Side	Floodway Side	Remarks
River Name		Cainta	Manggahan Floodway	
Proposed Discharge		95 m^3/s	2,900 m^3/s	
STA.		0 + 000	4 + 530	
DFL		EL. + 13.340 m	EL. + 14.853 m	
Design Dike Crown		EL. + 13.940 m	EL. + 15.940 m *	*Calculated by interpolation based on the as-built drawing
Existing Dike Crown		EL. + 13.0 m	EL. + 18.0 m	
River	Current State	30 to -35 m *	Already the same as	*Refer to **Figure 6.4.2**

Items		Tributary Side	Floodway Side	Remarks
Channel Width			plan	
	Proposed	34.6 m *	254 m	*Section-1 of the Cainta River
Design Dike Width		-	6.0 m *	*From the as-built Drawing of Manggahan Floodway
Design Riverbed Width		31 m *	108 m	*Section-1 of the Cainta River
Design Riverbed		EL. + 8.75 m	EL. + 7.43 m	*Calculated by interpolation based on the as-built drawing

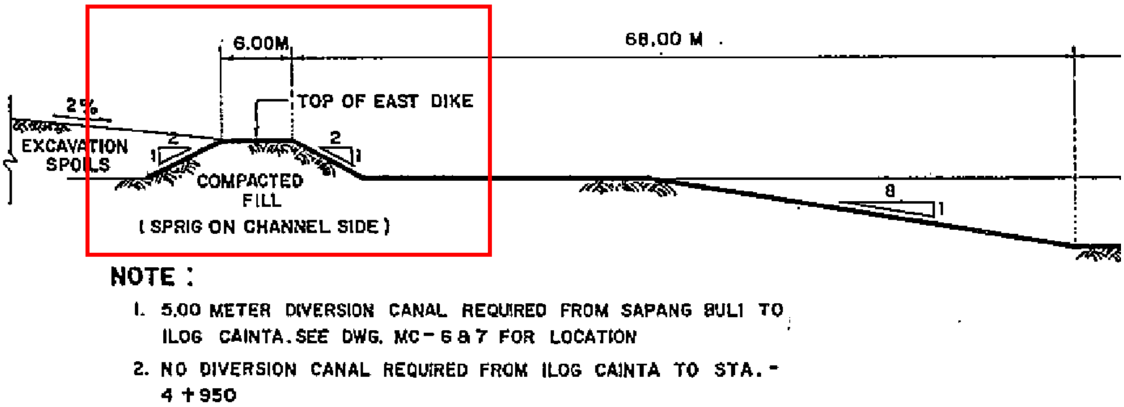
Note: Organized by the Study Team from the following materials:
1) The 2008 Pre-F/S
2) Final Report on Consulting Services for Manggahan Floodway Project

Figure 6.4.2 Current Width around the Confluence of the Cainta River



Note: Added to the 2008 Pre-F/S by the Study Team

Figure 6.4.3 Proposed Cross Section of Cainta River



Source: Final Report on Consulting Services for the Manggahan Floodway Project

Figure 6.4.4 Proposed Dike Shape for Manggahan Floodway

6.4.3.4 Conditions with the Existing Structures

When considering facility layout and construction plans, the conditions due to the major existing structures are shown in **Table 6.4.6**. In the case of Cainta River, although there is a proposed improvement plan based on the 2008 Pre-F/S, no new structure such as revetment has been implemented particularly in the vicinity of the Cainta Floodgate.

Table 6.4.6 Condition of Major Existing Structures

Symbol [1)]	Item	Description
A	Existing Bridge	Since there are existing bridge piers, abutments, superstructures and embankments, it is necessary to pay attention to the influence by the new structure.
B	Houses in the landside	Houses in the land side from the East Bank Road are not the original target of relocation.

[1)] *Symbol indicates location of existing structures, etc., in* ***Figure 6.4.5***
Source: Study Team

Source: Study Team

Figure 6.4.5 Major Existing Structures around the Cainta Floodgate

Photo 6.4.1 shows the existing bridge. The existing bridge has only 2 lanes carriageways and 2 piers in the current river channel. However, the As-built drawings are not available as of this study, the detail of the foundation cannot be grasped. This bridge shall be removed due to the installation of Cainta Floodgate, and the structural detail of it would be assumed based on the standard drawing of bridges and the drawings of bridges which has similar size.

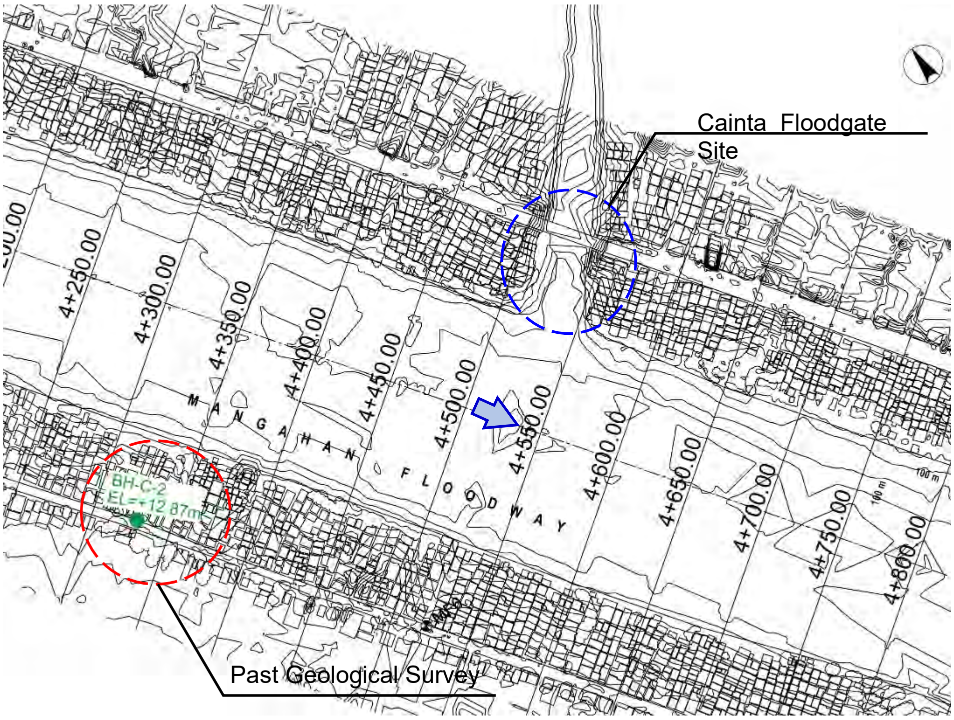
Source: Study Team

Photo 6.4.1 Photo of the Existing Bridge(San Francisco Br.)

6.4.3.5 Geological Conditions

(1) Existing Soil Investigation

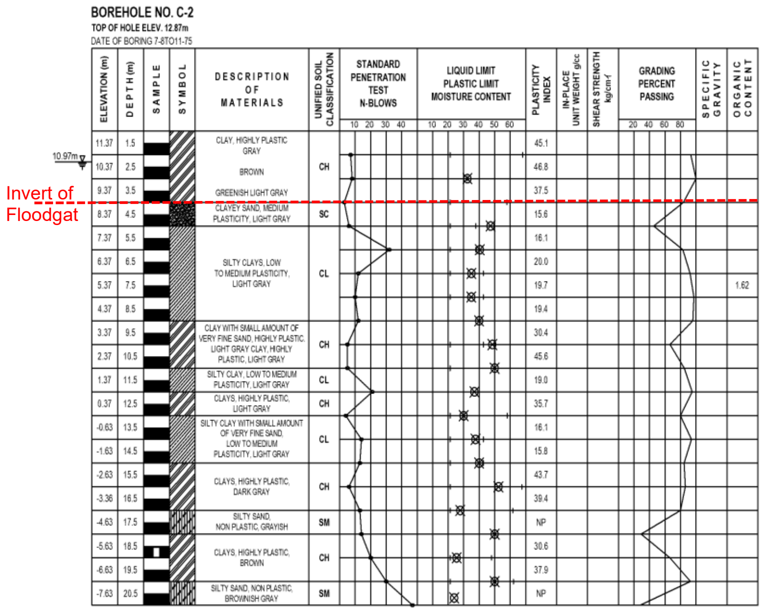
Existing boring in the vicinity is summarized as follows. Although the number of previous boring surveys of the Cainta Floodgate installation site is small, one of the soil surveys in the vicinity of the Cainta Floodgate is a boring survey in the design of Manggahan Floodway. The boring location map of the previous geological survey is shown in **Figure 6.4.6**.



Source: Survey Team added on Construction Drawing Manggahan Floodway Project

Figure 6.4.6 Existing Geological Survey Sites

The borehole log in the past geological survey is shown in **Figure 6.4.7**. According to this, most of the layers are silt and clay. Below the invert elevation of Cainta Floodgate, EL. + 8.75, it is a cohesive soil layer with N value of about 10. Although a sandy soil layer appears at a depth of about 20 m, the N value is 30 or less, so the supporting layer of the floodgate is considered to be located below EL -7.63 m, which is the bottom the existing boring.



Note: Prepared from Construction Drawing of Manggahan Floodway Project

Figure 6.4.7 Previous Borehole Log (No. C -2)

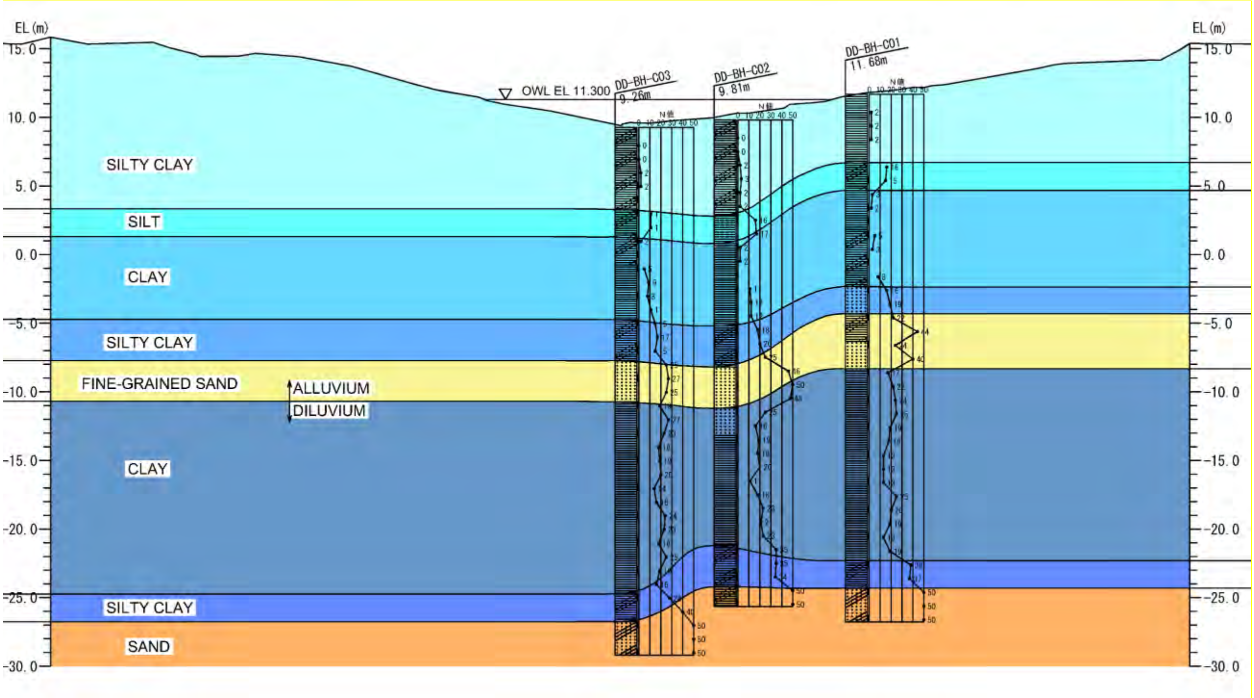
(2) Geological Overview

The geological survey at the Cainta Floodgate shows that, from the surface, the thick alluvial cohesive soil layer is about 15 ~ 20 m, the alluvial sandy soil layer is about 4 m, and the diluvial cohesive soil layer is about 16 m. A diluvial sandy soil layer with an N value of 50 or more appears at the lower part. The alluvial clay layer near the ground surface is a soft ground with an N value of about 0, but the N value tends to increase in the depth direction. The alluvial sand layer has a relatively rigid N value of about 30. The upper part of about 14 m of the diluvial clay layer has an N value of about 20, while ca.2 m from the lower end has an N value of about 30.

The results show that, although the order of the layers is generally the same from DD-BH-C01 to DD-BH-C03, the depth of appearance is different by about 3 m, and the N values are greatly different even in the layers of the same depth.

Regarding the soil condition used for the design, it was decided not to set the soil property uniformly by integrating three boring, but to set the soil property for each individual borehole. In the design, the geological survey which gives the most severe condition for each structure is adopted.

The previous geological survey, BH-C-02, was not used for the design because it was far from the design site.

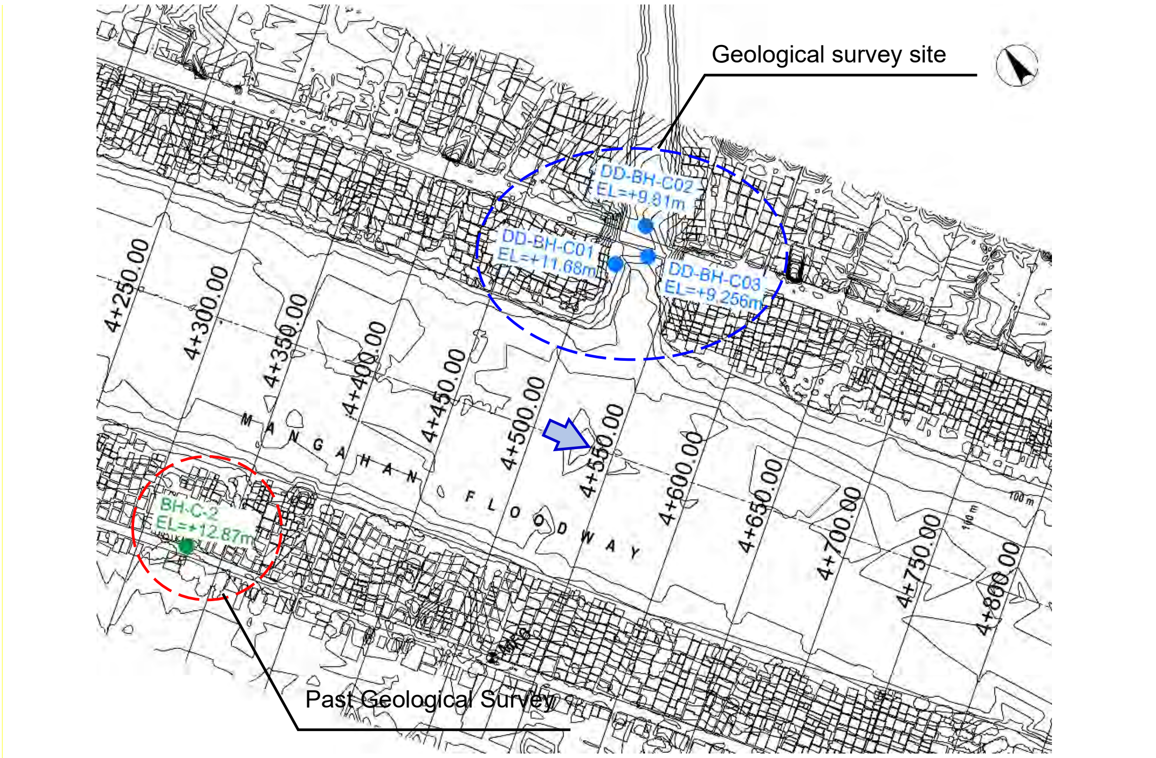


Source: Study Team

Figure 6.4.8 Assumed Geological Cross-Section

(3) Geological Survey Site

The location map of the existing geological survey and those of this time is shown in **Table 6.4.13**.



Source: Study Team

Figure 6.4.9 Geological Survey Site

(4) Soil Property

The soil properties used in the design are determined based on the results of the borehole columns and laboratory tests conducted in this study. The soil constants are set for each boring.

1) Setting Policy

The soil constants are mainly the N values required for the design, the unit weight, and the shear strength (c,ϕ) and consolidation constant. In setting these constants, soil layers were divided from the columnar diagram, and sub-layers with different N values. Similarly, the same soil layer was divided into subdivisions by symbols of cohesive soil (C) and sandy soil (S). In principle, the constant of each layer was set based on the soil test. For untested layers, soil constant was set by quoting the test value of the soil layer which can be judged to be almost the same layer.

2) Unit Volume Weight of Soil

Test values were used as a principle. For the soil layer without test value, it was set referring to the test value of upper and lower layers or the survey result of the vicinity, alternatively, the road earth work dike construction guideline, etc.

3) Shear Strength

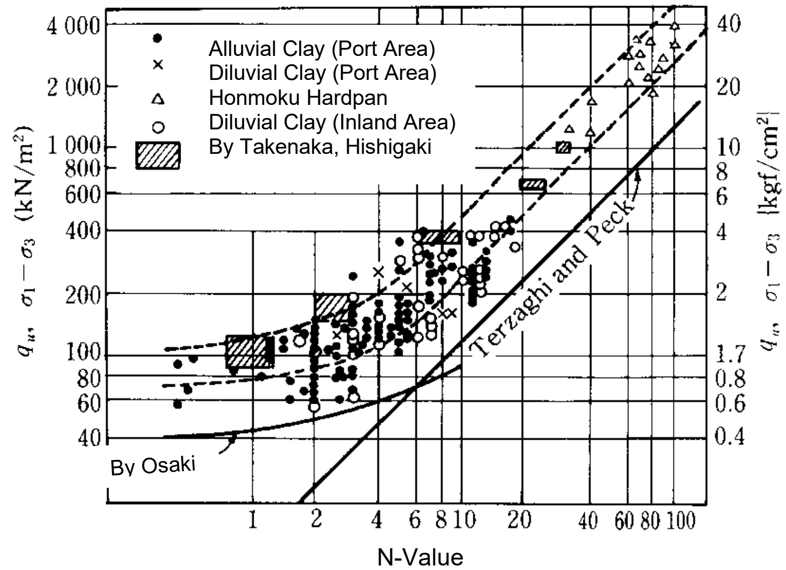
As a general rule, test values were used for shear strength (c, ϕ). For soil layers without test values, the test values of upper and lower layers or the survey results in the vicinity were used as a reference. For sand layers and cohesive soil with a large N value, the estimation was made by the following equation.

・Sandy soil: The following equation was set as a reference considering N value and effective overloading pressure.

$\phi = 4.8 \times \log N1 + 21$, $N1 = 170 \times N/(\phi'v + 70)$

For a sandy soil layer located at the bottom with N = 50, $\varphi = 40°$ was set.

・The cohesive soil was estimated from the relationship (qu = 25 N at lower limit) between the N value and qu shown in the guideline for soft ground countermeasures in the following figure.



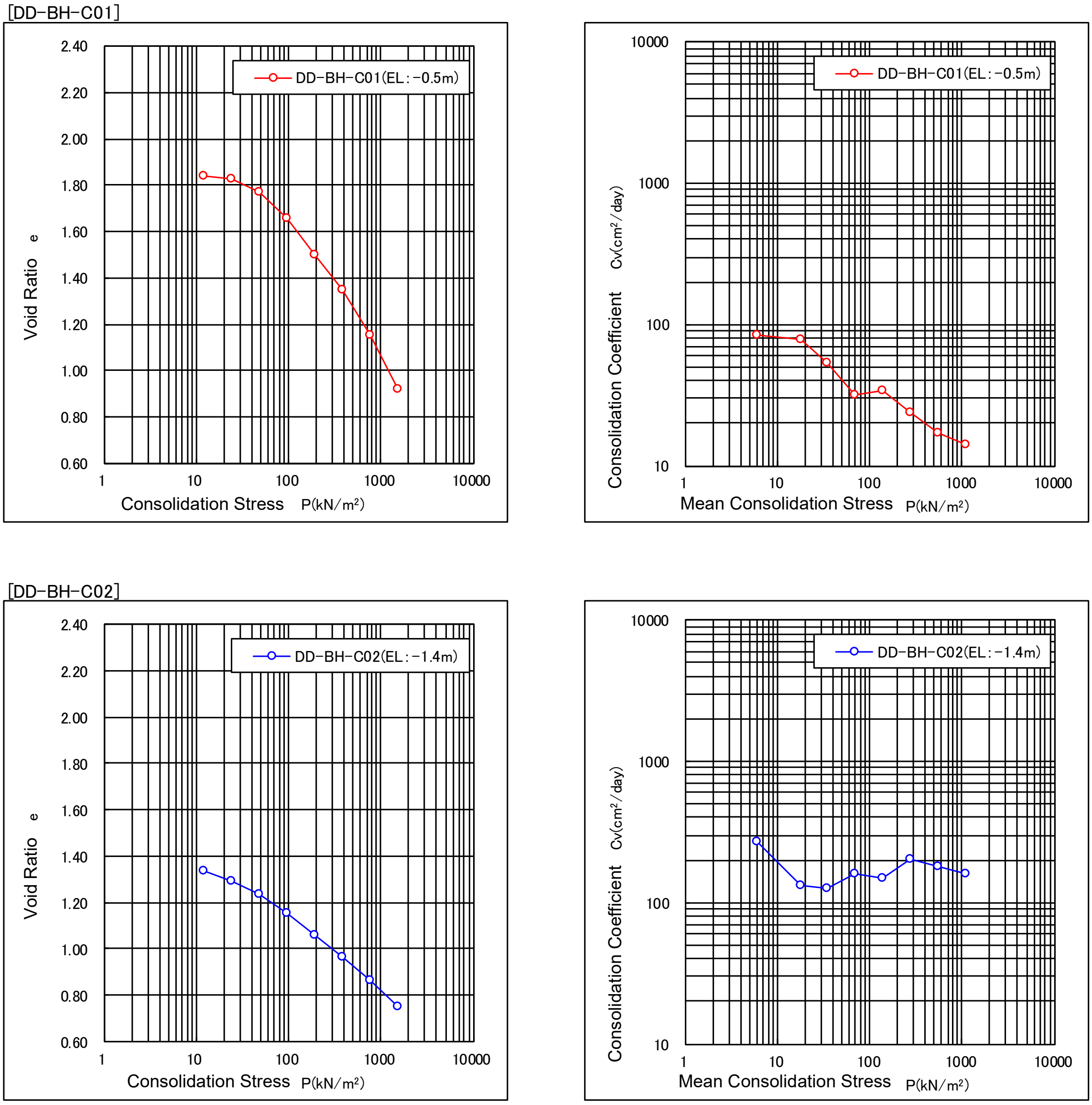
Source: Guidelines for Countermeasures for Weak Ground of Road Soil

Figure 6.4.10 Relationship Between N Value and Uniaxial Compressive Strength

4) Consolidation Constant

The e-log P curve and log Cv-log P curve were arranged from the consolidation test results. The compression index Cc and the swelling index Cs were calculated from these values. Representative values were set by distributing them in the soil property map.

The consolidation curve set to is shown **Figure 6.4.11**



Source: Study Team

Figure 6.4.11 Consolidation Curve

5) Soil Constant

The soil constants are set in the following pages.

Table 6.4.7 List of Soil Constants (DD-BH-C01)

Stratum	Soil Quality	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γt (kN/m3)	Cohesion c (kN/m2)	Shear Resistance Angle ϕ(°)	Deformation Coefficient E 50 (MN/m2)	Consolidation Settlement Target Layer	Compression Coefficient Cc	Swelling Coefficient Cc
C1	Cohesive Soil	2				(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	Cohesive Soil	15	40	85	13	18	180	0				
C3	Cohesive Soil	2	70	80	45	(16.5)	(24)	(0)	(2.0)	○	(0.8)	(0.062)
C4	Cohesive Soil	3	63	89	52	16.5	24	0	2.0	○	0.8	0.062
S1	Sandy Soil	17	30	20		20	0	33				
C5	Cohesive Soil	22	38	65	25	18	270	0				
S2	Sandy Soil	24	20	23		20	0	34				
C6	Cohesive Soil	18	45	90	45	18	220	0				
C7	Cohesive Soil	37	32	52	19	19	460	0				
S3	Sandy Soil	50	26	37	32	20	0	40				

**1 The unit volume weight was all saturated weight.*

**2 Since no mechanical tests have been conducted on the consolidation settlement target layers C1 and C3, the unit volume weight, shear strength, consolidation characteristics, etc., are described as follows:*

C1 is the set value of the boring DD-BH-T01, and C3 is the test value of C4 of the boring.

**3 () is used as a reference for general values, estimates, or test values from another site.*

Source: Study Team

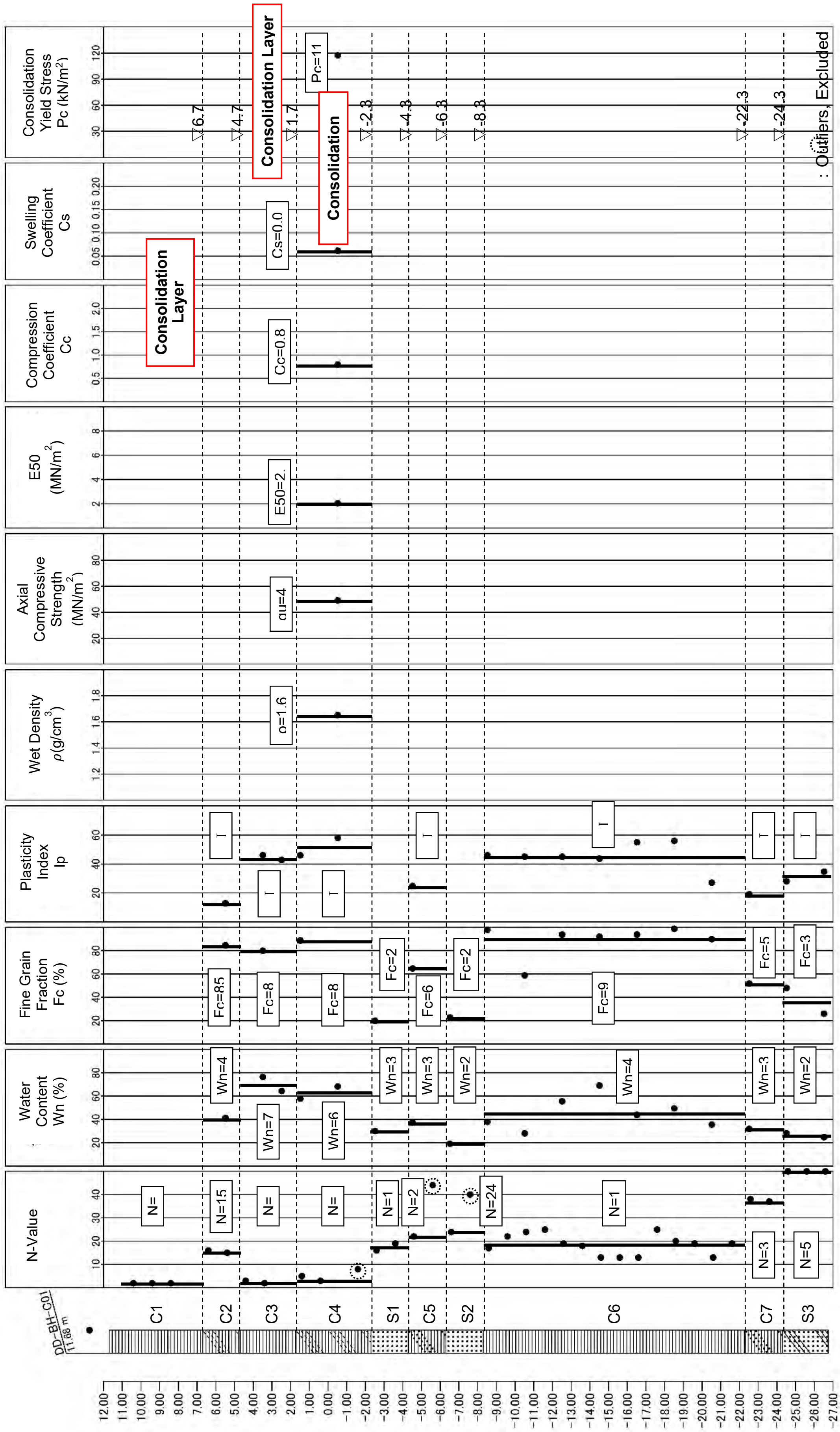


Figure 6.4.12 Soil Characteristics Map (DD-BH-C01)

Source: Study Team

Table 6.4.8 List of Soil Constants (DD-BH-C02)

Stratum	Soil Quality	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γt (kN/m3)	Cohesion c (kN/m2)	Shear Resistance Angle ϕ(°)	Deformation Coefficient E 50 (MN/m2)	Consolidation Settlement Target Layer	Compression Coefficient Cc	Swelling Coefficient Cc
C1	cohesive soil	1	62	93	15	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	cohesive soil	16	44	83		18	200	0				
C3	cohesive soil	2	70	86	31	16	14	0	1.5	○	0.42	0.041
C4	cohesive soil	12	47	90	53	17	150	0				
C5	cohesive soil	20	40	77	28	18	250	0				
S1	sandy soil	48	16	18		21	0	39				
S2	sandy soil	20	22	22		20	0	34				
C6	cohesive soil	20	40	91	30	18	250	0				
C7	cohesive soil	35	27	53	16	19	430	0				
C8	cohesive soil	50	45	86	26	19	620	0				

**1 The unit volume weight was all saturated weight.*

**2 Since a mechanical test has not been performed on the consolidation settlement target layer C1, the weight, shear strength, consolidation characteristics, etc., of a single body are described as follows:*

The set value in boring DD-BH-T01 was adopted.

**3 () is used as a reference for general values, estimates, or test values from another site.*

Source: Study Team

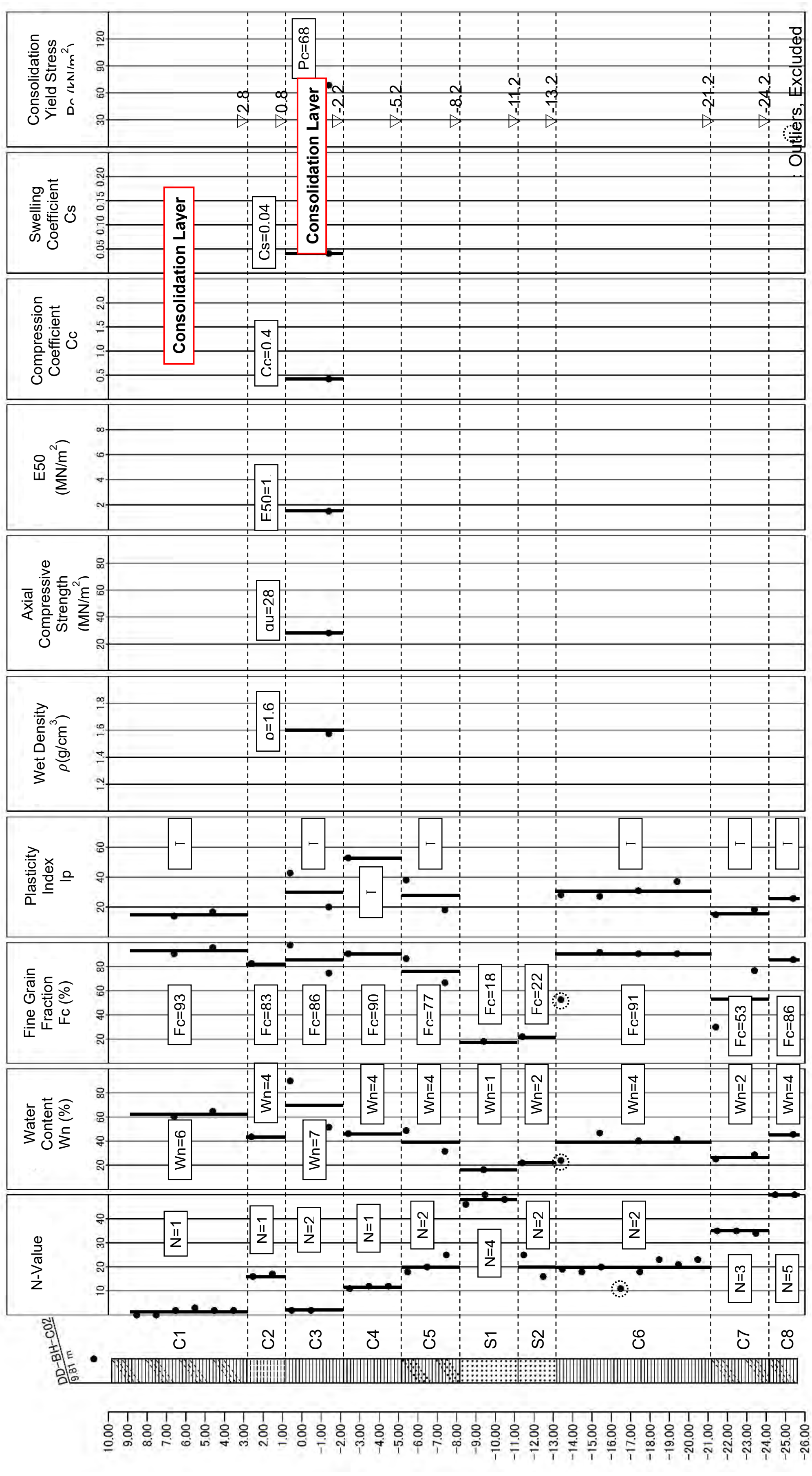


Figure 6.4.13 Soil Characteristics Map (DD-BH-C02)

Source: Study Team

Table 6.4.9 List of Soil Constants (DD-BH-C03)

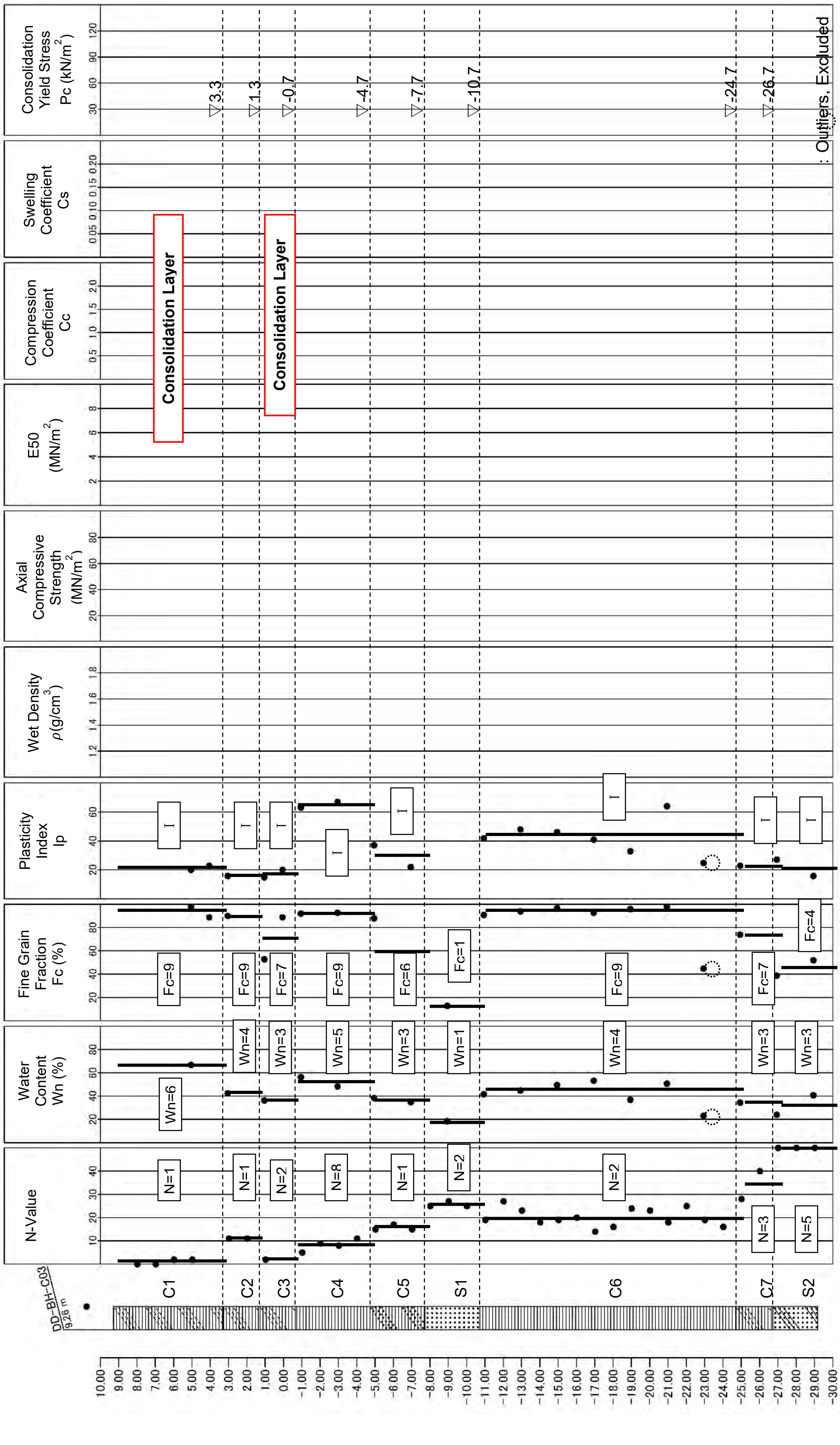
Stratum	Soil Quality	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γt (kN/m3)	Cohesion c (kN/m2)	Shear Resistance Angle ϕ(°)	Deformation Coefficient E 50 (MN/m2)	Consolidation Settlement Target Layer	Compression Coefficient Cc	Swelling Coefficient Cc
C1	cohesive soil	1	67	93	21	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	cohesive soil	11	43	90	16	17	130	0				
C3	cohesive soil	2	37	70	17	(16)	(14)	(0)	(1.5)	○	(0.42)	(0.041)
C4	cohesive soil	8	53	96	65	17	100	0				
C5	cohesive soil	16	37	60	30	18	200	0				
S1	sandy soil	26	19	13		20	0	36				
C6	cohesive soil	20	46	94	45	18	250	0				
C7	cohesive soil	34	35	74	23	19	420	0				
S2	sandy soil	50	33	45	21	21	0	40				

**1 The unit volume weight was all saturated weight.*

**2 Since no mechanical tests have been conducted on the consolidation settlement target layers C1 and C3, the unit volume weight, shear strength, consolidation characteristics, etc. The set value of boring DD-BH-CO2 was adopted.*

**3 () is used as a reference for general values, estimates, or test values from another site.*

Source: Study Team



Source: Study Team

Figure 6.4.14 Soil Characteristics Map (DD-BH-C03)

(5) Type of Ground

The ground classification is determined by the characteristic value TG of the ground.

$$T_G = 4\sum_{i-1}^{n}\frac{H_i}{V_{si}}$$

where: Vsi: Initial shear wave velocity

Sandy soil Vs = 80 N 1/3, cohesive soil Vs = 100 N 1/3

N: N value of each stratum

i: number of the i th soil layer

type of ground	ground characteristic value TG (s)
I species	TG ≦ 0.2
Type II	0.2 < TG ≦ 0.6
Class III	0.6 < TG

Source: BSDS

The ground type is set by DD-BH-C03, which has the lowest appearance depth of the engineering bedrock. The result of calculation of the ground characteristic value TG in DD-BH-C03 is shown in **Table 6.4.10**. From the calculation results, the ground of the Cainta floodgate installation site is set as class III ground.

Table 6.4.10 Calculation of Ground characteristic value TG (DD-BH-C03)

Type Of Ground		Elevation (m)	Thickness (m)	N-value	Vsi (m/s)	Hi/Vsi
Sandy Soil	As1	9.45	3.00	0	50	0.0600
Cohesive Soil	Ac1	6.45	3.19	1	100	0.0319
Cohesive Soil	Ac2	3.26	2.00	11	222	0.0090
Cohesive Soil	Ac2	1.26	2.00	2	126	0.0159
Cohesive Soil	Ac1	-0.74	0.41	8	200	0.0021
Cohesive Soil	Ac1	-1.15	3.59	8	200	0.0180
Cohesive Soil	Ac2	-4.74	3.00	16	252	0.0119
Sandy Soil	As2	-7.74	3.00	26	237	0.0127
Cohesive Soil	Ac2	-10.74	14.00	20	271	0.0517
Cohesive Soil	Ac2	-24.74	2.00	34	324	0.0062
		-26.74				
Subtotal			16.00			
4 Σ H/V						0.8776
Type of Ground					Type III ground	

Source: Study Team

6.4.3.6 Location of Floodgate

(1) Study on Layout Location

Around the location of the Cainta floodgate, currently there is no dike. When a floodgate is installed, it is necessary to set the location of a new dike at this location.

Here, for the following reasons, the new dike shall not be proposed in the river side from the original alignment of the existing embankment as shown in **Figure 6.4.15.**

- ✓ It becomes an obstacle to the water flow during flood of Manggahan Floodway and
- ✓ The dike itself that is shifted towards riverside becomes a water colliding front and it is not preferable location for floodgate structures
- ✓ Allowing to shift alignment of the dike towards floodway side indicates one of validities of the development inside the floodway to other authorities/agencies and private companies.

Source: Study Team

Figure 6.4.15 Image of New Dike Installation on the Riverside

From the above, the following two options can be considered when studying the location of floodgate.

- ✓ Alternative 1: Landside of the Existing Dike
- ✓ Alternative 2: Same Location as the Existing Dike

Table 6.4.11 shows a comparison between the alternatives. Based on this comparison, since the impact on social environmental issues can be minimized and advantages on the access to the floodgate and the installation of dike with the utilization of existing structures are expected, Alternative 2 is recommended.

Table 6.4.11 Comparison of Locations for the Cainta Floodgate

Items	Alternative 1 Landside of the Existing Dike		Alternative 2 Riverside of the Existing Dike	
Figure				
General	・ The floodgate will be installed at the land side of the existing road crossing the Cainta River and dike embankment connecting to it will be installed. ・ The existing road bridge will be left as it is.		・ The floodgate will be installed along the existing dike alignment, and the existing road bridge will be replaced as a maintenance bridge that will be used also for ordinary traffic.	
Hydraulic Aspect	・ Since a pier of floodgate is located near the existing bridge piers, turbulent flow may occur. However, in case there is more clearance than the river width of Cainta River, the influence can be avoided.	○	・ The existing bridge piers will be removed, and a new pier will be installed. Hence, no hydraulic issue due to neighboring piers will occur.	◎
Access to the Floodgate	・ This needs an additional access to the floodgate and its maintenance bridge,	△	・ An additional access is not needed for the floodgate	○

Items	Alternative 1 Landside of the Existing Dike		Alternative 2 Riverside of the Existing Dike	
Construction	・ The existing bridge will remain; no temporary bridge and detour is needed.	○	・ Temporary bridge is needed.	△
Social Environment	・ This alternative requires land acquisition and house relocation in the landside, which will probably affect the progress of the project due to long negotiation with the residents.	△	・ Area to be acquired in the land side can be minimized	○
Cost	・ The construction cost is lower than Alternative 2; however, the numbers of houses to be relocated and the areas to be acquired become large.	1	・ A temporary bridge and rebuilding of the bridge are needed. Hence, the construction cost becomes higher.	2
Evaluation	There are some advantages on the cost and unnecessity to replace the existing bridge. However, the schedule of the project is probably affected by its social environmental impact.		The access to the floodgate is good. Social environmental impact can be minimized.	
			Recommended	

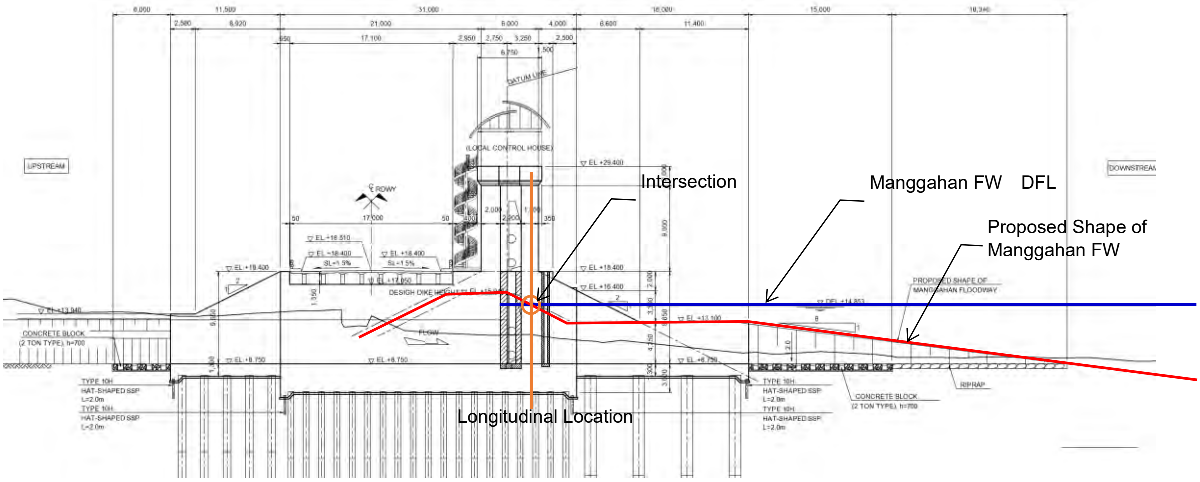
Legend : ◎...Better, ○...No problem, △...It has issues
Source : Study Team

(2) Cross Sectional Location

The position where the center line of the floodgate is approximately located on the center of Cainta River is the base and the centerline of the floodgate is aligned with Sta.4+525 of Manggahan Floodway.

(3) Longitudinal Location

The column and main body shall be within the shoulder of the dike to the intersection between the side slope of the dike and the DFL of Manggahan Floodway. However, it still needs a width for the length of pier in river side and groves for stop logs. Accordingly, the river side edge of the main body shall fit the intersection between the proposed shaped of dike the DFL of Manggahan Floodway and the position is to be determined.



Source : Study Team

Figure 6.4.16 Longitudinal Location of Floodgate

6.4.3.7 Study on the Basic Structural Specifications

(1) Study on Type of Gate

1) Primary Selection of Gate Type

The main purpose of the Cainta Floodgate is to prevent flood backflow from the main river. Considering this purpose, the applicable gate type is selected. For the selection, the following types are options referring to the type of Floodgate as shown in the "Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]" (see **Table 6.4.12**).

Table 6.4.12 Types, Locations and Purpose of Floodgates

Location	Intake Weir and Estuary Weir					Floodgate sluiceway Inverted Siphon	Lock Gate	Tidal Control	Repair
Purpose / Types of Floodgate	Spillway Gate	Flushing Gate	Flow Rate Regulation Gate	Intake Settling Basin Headrace Gate	Fish Course				
Roller Gate	○	○	○	○	○	○	○	○	
Shell Structure Roller Gate	○	○	○			○		○	
Slide Gate		○		○		○			
Bear-trap Gate			○	○	○			○	
Flap Gate				○				○	
Miter Gate						○	○	○	
Sector Gate							○		
Visor gate									
Rolling Gate	○	○							
Stop Log									○
Transverse Gate								○	
Floating Gate									○
Swing Gate						○	○	○	
Shield Gate									○
Rising Sector Gate	○		○		○	○	○	○	

Note: Translated by the Study Team from the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

The following gate types are selected in reference to **Table 6.4.12**.

The slide gate is not applicable since it is a type adopted in the case of the clear span up of about 2.0 m[1].。

- ✓ Roller Gate
- ✓ Miter Gate
- ✓ Swing Gate
- ✓ Radial Gate
- ✓ Rising Sector Gate

Regarding the above types, the following field conditions are considered to select those that may be applicable to this facility.

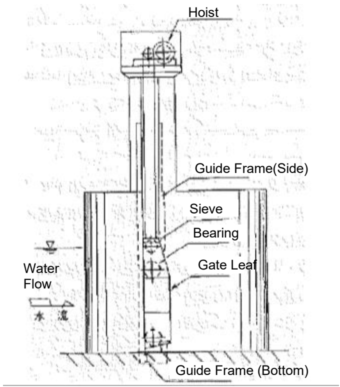
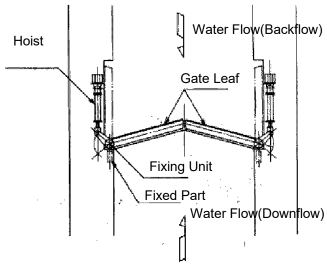
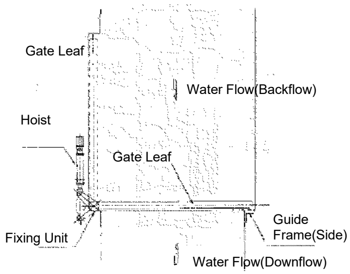
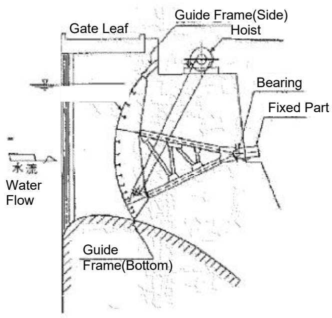
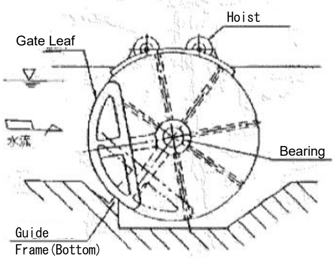
・ Water Control Function	:	To prevent floods from the Manggahan Floodway
・ Flowing Garbage and Water Plants	:	There are many garbage and water plants such as water lily and branches, and they tend to accumulate.
・ Resistance to Local Climate Condition	:	High temperature and humidity compared to Japan, and solar radiation heat all year round
・ Availability of Land	:	The site is limited on both sides and the facility is as compact as possible

The comparison of gate types is shown in **Table 6.4.13**. Based on this table, the following three types can be used for this facility. Further selection from the applicable gate types shall be studied.

1. Fixed Wheel Roller Gate
2. Radial Gate
3. Rising Sector Gate

[1] From the design guidelines (draft) for sluice and sluice gates

Table 6.4.13 Comparison of Gate Types

No.	Type of Weir/Gate	Description of Structure		Reason for Inapplicable	Evaluation
1	Fixed Wheel Roller Gate		During flood, the gate is lifted above DFL. There is a plate girder structure and a shell structure according to the difference in the gate structure.	Nothing	○
2	Miter Gate		It consists of two piers, one on the left and another on the right, for closing the water channel, and it is operated by rotating around the vertical rotation axis.	There is a high possibility of incomplete closure due to entrapment of effluent.	×
3	Swing Gate		Although it has almost the same structure as a Miter gate, it has only one pier.	There is a high possibility of incomplete closure due to entrapment of effluent.	×
4	Radial Gate		A gate that opens and closes by rotating the gate around a horizontal rotation axis.	nothing	○
5	Rising Sector Gate		The gate is opened and closed by rotating the end disk.	Nothing	○

Legend：○...Alternative to be studied、×...Not Applicable for this Facility
Note: Prepared by the Study Team based on the reference documents

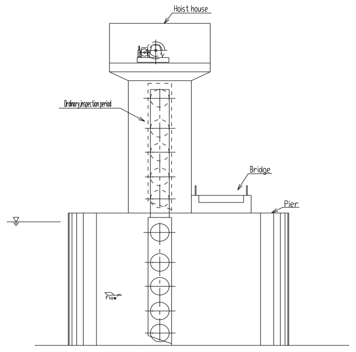
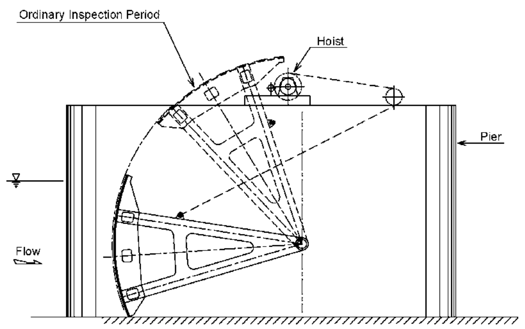
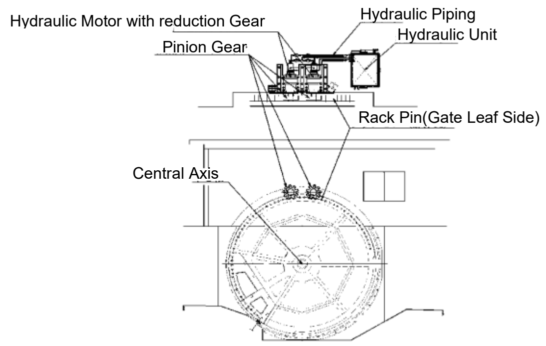
2) Secondary Selection of Gate Types

The following three have been selected as the alternative gate types in the primary selection. A comparison of the three alternatives selected is given in **Table 6.4.12**.

- Alternative 1：Fixed Wheel Roller Gate
- Alternative 2：Radial Gate
- Alternative 3：Rising Sector Gate

Based on the comparison, Alternative 1 (Fixed Wheel Roller Gate) is selected. In addition to the ease of maintenance, reliability and economy, the knowledge of local technicians on their operation and maintenance was also taken into consideration since the same type was adopted for the nearby weir and Floodgate.

Table 6.4.14 Comparison of Gate Types

Item	Alternative 1：Fixed Wheel Roller Gate		Alternative 2：Double Deck Fixed Wheel Roller Gate		Alternative 3：Radial Gate	
Outline Drawing	 *Source：Final Report of 2002 PMRCIP-I*		 *Source：Final Report of 2002 PMRCIP-I*		 *Source：Manufacturer*	
Summary	・ This structure consists of fixed roller, girder and skin plate attached to the pier. ・ Operation of the pier is performed by winding up the wire rope or chain.		・ This is the type that has an arm next to the pier and it consists of skin plate, main girder, sub-girder, and beam receiving vertical member. ・ Gate opening and closing is done by vertical lift and wire rope. ・ A hoist is installed at the top of the pole.		・ Both ends of the pier with shell structure consist of disks, and opening and closing is performed by rotating this disk.	
Gate Location	・ Normal and during inspection: Above the water surface ・ During Flood: Below the water surface		・ Normal and inspection time: Above the water surface ・ Flood: Water		・ Normal and inspection time: Above the water surface ・ Flood: Water	
Maintenance Aspect	・ Inspection of each part and replacement of water sealing rubber are easy and maintenance is relatively easy. ・ Although many parts are required, few parts of hoist need to be replaced. However, wire rope replacement and grease application on wire rope are necessary.	◎	・ Inspection of each part and replacement of water sealing rubber are easy and maintenance is relatively easy. ・ Inspection of the hoist is as easy as fixed wheel type gate.	○	・ Both side disks are always submerged, and inspection and maintenance are complicated compared to the fixed wheel roller gate. ・ It is more difficult to remove sediment.	△
Construction Surface	・ Installation is easier than the other plans.	○	・ If the center of rotation at both ends is not installed with high accuracy, opening and closing will be hindered. Therefore, enough technology and accuracy are required for its construction.	△	・ If the trunnion centers at both ends are not installed with high precision, opening and closing will be hindered. Therefore, enough technology and accuracy are required for its construction.	△
Reliability	・ The pier is lowered by its own weight. Hence, the gate can be lowered even in floods, the operation reliability is high. ・ It is easy to observe the opening and closing status of the gate even from a distance.	◎	・ The pier is lowered by its own weight. Hence, the gate can be lowered even in floods, the operation reliability is high.	○	・ Compared with the fixed wheel roller gate, there is fewer sample cases. Hence, the reliability is inferior.	○
Landscape	・ Since the structure is significantly higher than the riverbank, it is the least scenic alternative.	△	・ Since this type does not have tall column and large control house, it is superior in landscape to fixed wheel roller gate.	○	・ Since this type does not have a tall column, it is superior to the fixed wheel roller gate in terms of landscape.	○
Economy	・ Gate Facility: ***** (1.00) ・ Most economical	1	・ Gate Facility: ***** (1.09) ・ Inferior to the Alternative 1	2	・ Gate Facility: ***** (1.31) ・ Most expensive	3
Example in the Philippines	・ NHCS (Floodgate), Rosario Weir, etc.	○	・ NHCS (Lock Gate)	○	・ No Example	△
Evaluation	Although the column stands out, it is superior in terms of maintenance, management, operation and economy. This is the same type as the NHCS and the Rosario Weir managed by MMDA, and local engineers have enough knowledge in its operation and maintenance.		It has the same function as the fixed wheel roller gate and has enough reliability. However, it is a little inferior in maintenance and construction. Lowering by its own weight is impossible and there is not the actual sample adopting this type in weirs It is inferior in economic terms.		Issues remain in terms of maintenance and performance. The most expensive option.	
	1		2		3	

Note: Cost is not presented due to the prior released version.
Legend：◎...Better、○...No Problem、△...There are issues to be solved.
Source: Study Team

(2) Invert Elevation

The invert elevation is set to the design riverbed of the Cainta River and the EL. 8.75 m at Sta. 0+000 is adopted.

(3) Gate Position

The gate installation position is 6.25m from the river-side surface of the central pier.

(4) Floodgate Width

The width of the Floodgate is set at 35 m based on the width of the existing river channel around the Floodgate location (refer to **Figure 6.4.2**. In addition, the water surface width of the DFL in the proposed section is set as 34.6 m in the 2008 Pre-F/S (refer to **Figure 6.4.3**), almost the same as the current river width.

(5) Span and Span Allocation

1) Study Condition

When setting the span and its allocation, the “Cabinet Order Concerning Structural Standards for River Administration Facilities, etc. (hereinafter so called “Structural Cabinet Order”)” shall be referred. The study conditions are as follows:

1. Proposed Discharge: 95 m^3/s
2. The minimum span as specified in the Structural Cabinet Order is 15 m.

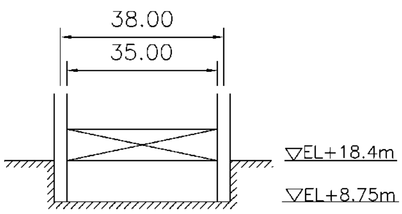
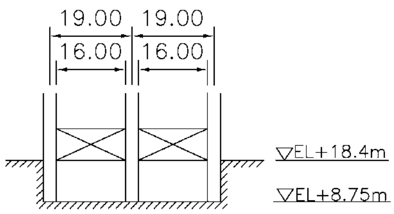
2) Study on Span Length and Span Allocation

Considering the above conditions, the following two alternatives are compared. In the case of three or more spans, the minimum span length will be less than the minimum span length. Hence, it is not considered.

1. Alternative 1: 1 span (38.0 m)
2. Alternative 2: 2 spans (19.0 m + 19.0 m)

Table 6.4.15 shows the comparison of span allocation. “Alternative 2: 2 spans (19.0m+19.0m)" is adopted due to its high operational reliability.

Table 6.4.15 Comparison of Span Allocations

	Alternative 1: 1-span (38.0 m)		Alternative2: 2 spans (19.0 m + 19.0 m)	
Figure				
Clear Span	・ 35.0 m		・ 16.00 m + 16.00 m	
Blocking Rate	・ 0%		・ 8.6%	
Type of Gate	・ Fixed Wheel Roller Gate (Single Pier)			
General	・ The width of the central weir of this type does not obstruct the river channel.		・ This satisfies the minimum span length specified in the Structural Cabinet Order.	
Structure	・ This has a disadvantage by gate deflection due to its long clear span. ・ Since the gate is heavy, the capacity of the hoist is also large.	△	・ Although central pier, columns, and operation deck are required, this has advantage in gate facilities, and the usage of steel materials can be reduced.	○

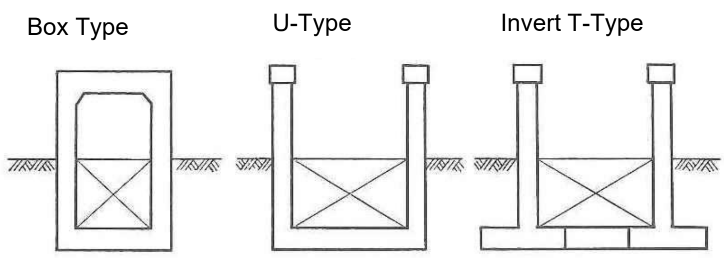
	Alternative 1: 1-span (38.0 m)		Alternative2: 2 spans (19.0 m + 19.0 m)	
Operation	・ If the gate cannot be opened due to failure, the drainage from the tributary river is hindered.	△	・ The operation is not complicated due to the simultaneous operation of two gates. ・ Even if one of the gates malfunctions and cannot be opened, drainage from the tributary river is possible.	○
Maintenance	・ Less maintenance due to less machinery and equipment.	○	・ Although the number of mechanical equipment increases, there is no major issue expected, since knowledge and experience in the maintenance of the fixed wheel gate have been accumulated sufficiently.	○
Evaluation	Since there is no plan to pass large vessels, there is no need to increase the risk of adverse effects on drainage from tributary rivers due to malfunction of the gate.		The use of two spans can increase the redundancy of the facility and enhance its reliability.	
			Recommended	

Legend: ○... No problem, △... There are issues to be solved.
Source: Study Team

(6) Type of Structure

There are three structural types of Floodgates: box type, U-type, and Invert T-type, depending on span length, foundation ground, and construction conditions. The schematic diagram of each type is shown in **Figure 6.4.17**. The classification of types according to span length is also shown.

Since the span length of the Cainta Floodgate is 19.0 m, the U-type Floodgate is applicable. However, since it has 2 spans, a heel slab is required behind the side wall, and the length of the bottom slab will exceed 20 m. Therefore, an inverted T-type is adopted.



*Box Type: up to 12 m
*U-type: 12 to 20 m
*Inverted T-type: More than 20 m

Source: Guideline Checklist for Permitted Structures' Technical Review (MLIT, Japan)

Figure 6.4.17 Types of Main Body of Floodgate

(7) Study on Local Control House

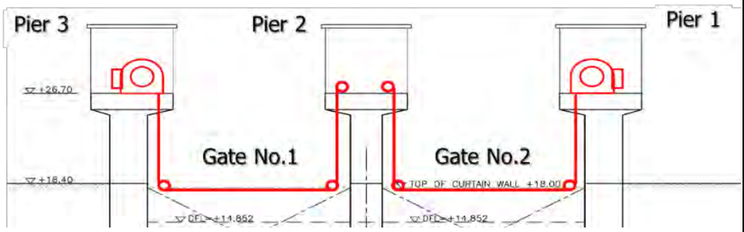
1) Conditions of the local control house

As explained in design of MCGS local control house , required clearance for inspection and maintenance works are provided by type of equipment as follows.

- ✓ Clearance around hoisting machine: 80 cm
- ✓ Clearance around other mechanical equipment: 60 cm
- ✓ Clearance in front of control panel: 100 cm
- ✓ Clearance between control panels: 120 cm
- ✓ Clearance at side or back of control panels: 20 cm

The major equipment installed in the upper control room of the Cainta floodgate is shown in the table below. Emergency generators and control panels (including communication equipment with the EFCOS administration building) shall be installed in a separate building on the ground.

Table 6.4.16 Major Equipment in Cainta Flood Gate Local Control House

Location	Equipment	Remarks	Conceptual diagram of equipment configuration
Pier.1 (end post)	Winch Drum	For No. 1 gate	
	Motor	For No. 1 gate	
	Control Panel	For No. 1 gate	
Pier2 (central pillar)	Sheave	For No. 1 gate	
	Sheave	For No. 2 gate	
No. 3 (end post)	Winch Drum	For No. 2 gate	
	Motor	For No. 2 gate	
	Control Panel	For No. 2 gate	

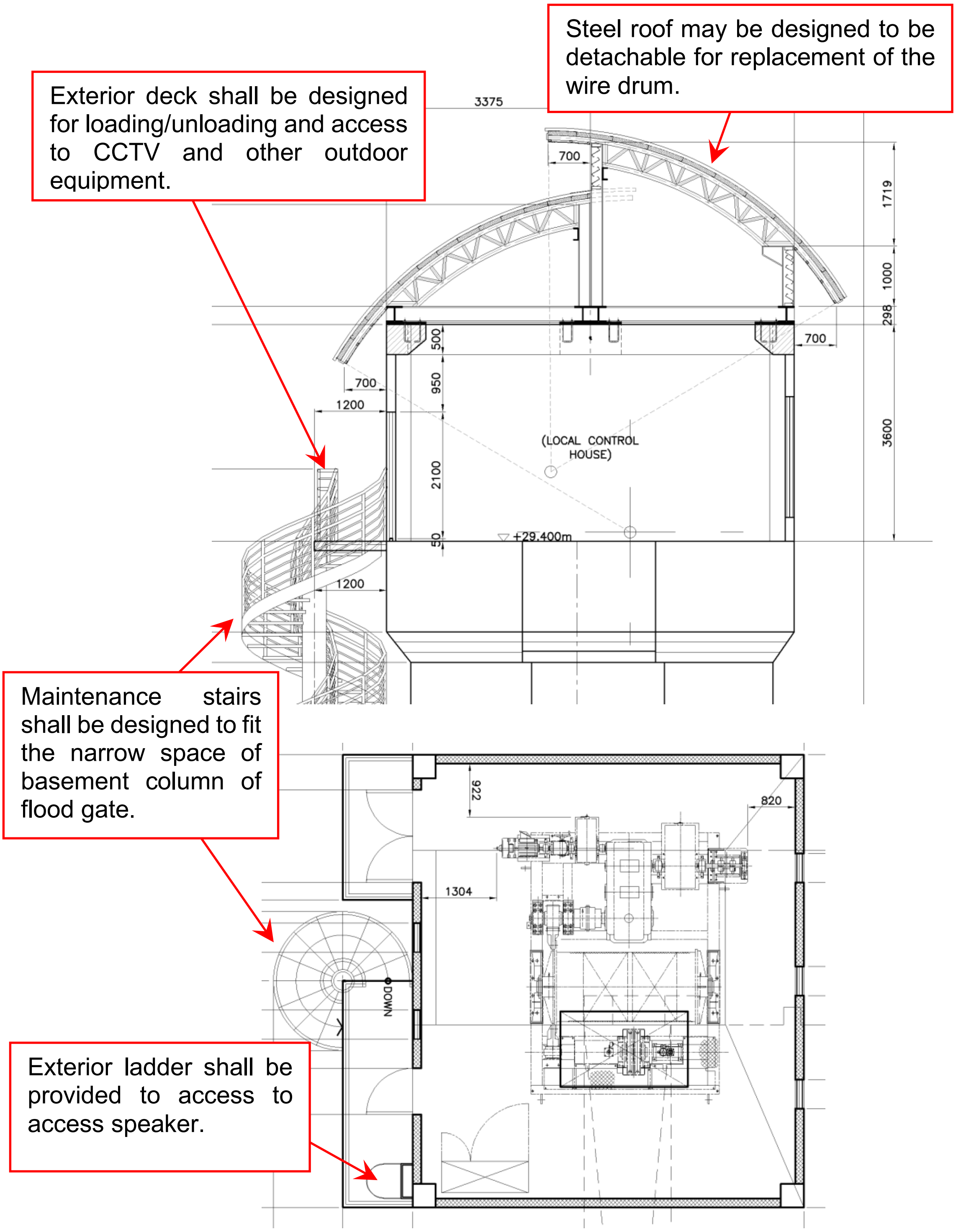
Source: Study Team

In addition, the following conditions shall be considered for ease of maintenance.

- ✓ Steel slide doors and exterior decks are provided for loading and unloading of heavy equipment by crane from the ground.
- ✓ Outdoor spotlights, ITV cameras and speakers shall be accessible from the exterior deck.
- ✓ Steel roof may be designed to be detachable for replacement of the wire drum, which weighs more than 5 tons, instead of installation of roof mounted hoisting crane in the local control house.
- ✓ The ceiling height shall be decided in consideration of necessary vertical clearance for inspection and maintenance of winch drum. Also, clearance of 50 cm or more shall be secured between RC beam and steel door.
- ✓ The maintenance stairs may not be shall comply the requirement by NCBP comply with requirement specified in the local building standards.

2) Layout Plan and Cross Section of Local Control House

Based on the conditions summarized in the previous page, the plan and cross section of the local control house are shown below.



Source: Study Team

Figure 6.4.18 Plan and Section of Cainta Flood Gate Local Control House

(8) Study on the Maintenance Bridge

1) Summary

The necessary width and girder size of the maintenance bridge shall be studied considering the following conditions. The bridge will also function as an existing bridge that needs to be replaced when the Floodgate is installed (For use on public roads).

- ✓ Current width of East Bank Road
- ✓ Necessity of securing workspace such as setting stop log

The existing bridge has one lane on each side, while the upstream and downstream roads have two lanes on each side. In the Philippines, although there are a lot of similar cases, considering the risk of traffic bottlenecks and traffic jams, as well as the higher probability of accidents at the junction before a bridge, it is design to replace the existing bridge with a two-lane on each side similar to the East Bank Road.

In addition, by replacing a bridge with one having two lanes on each side, benefit to road users is expected, and the understanding of residents on the project can be obtained more easily.

2) Design Standards

Bridges and incidental structures of bridges in this project shall be designed in accordance with the following standards.

- ✓ Design Guidelines, Criteria and Standards (DGCS): Volume 5 – Bridges Design
- ✓ Design Guidelines, Criteria and Standards (DGCS): Volume 4 – Highway Design
- ✓ Design Guidelines, Criteria and Standards (DGCS): Volume 3 – Water Engineering Projects
- ✓ DPWH LRFD Bridge Seismic Design Specifications (BSDS)
- ✓ DPWH Standard Specification
- ✓ Philippine National Standard (PNS)
- ✓ American Association of State Highway and Transportation Offices (AASHTO)
- ✓ National Structural Code of the Philippines: Volume 1
- ✓ National Structural Code of the Philippines: Volume 2

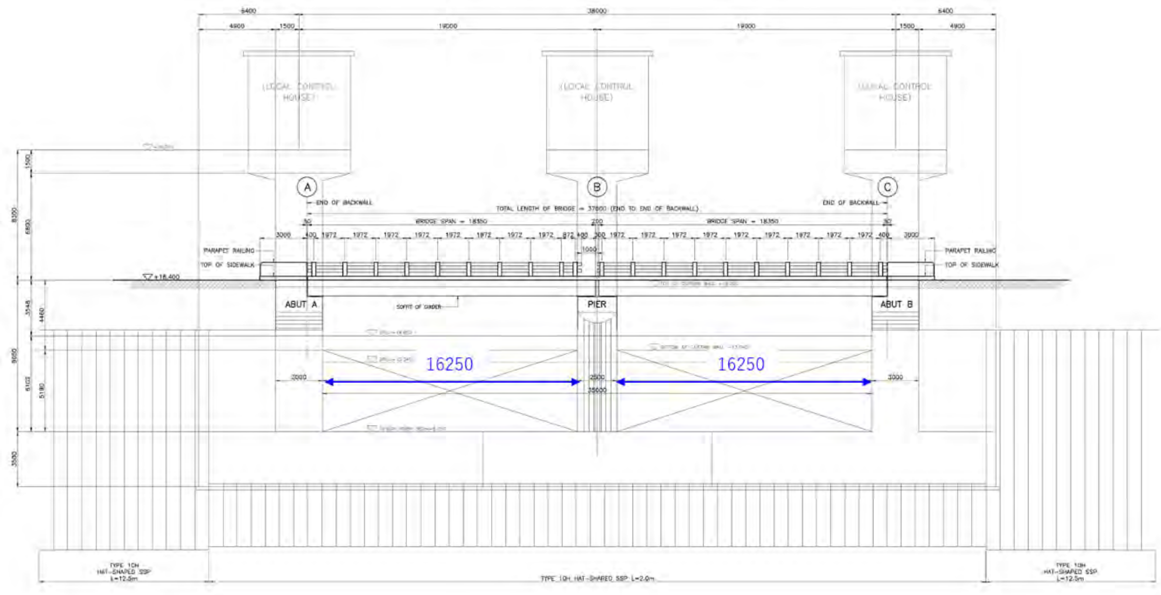
3) Design Condition

(a) River Condition

(i) Width of Floodgate

The position of the weir piers and the clear span length that determine the bridge length are as follows.

L= 16.250m



Source: Study Team

Figure 6.4.19 Position of the Pier and Clear Span of Cainta Floodgate

(ii) Elevation of Road Surface

The elevation of the road surface shall be the same as the height of the existing road (EL. +18.400).

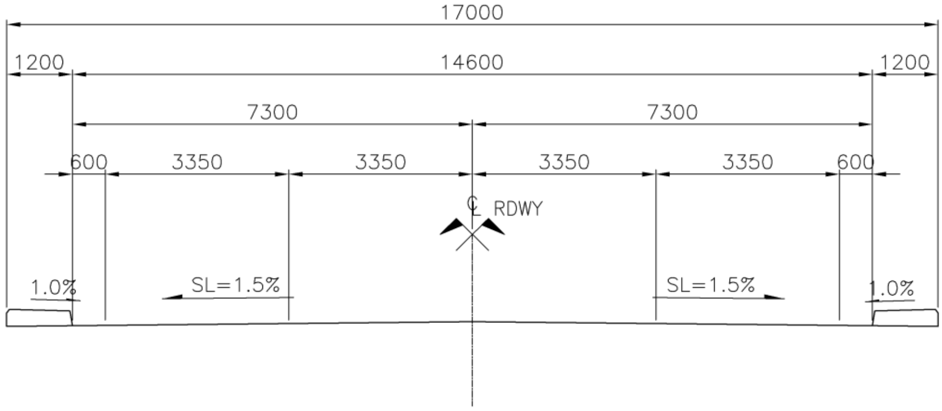
(b) Road Condition

(i) Road Classification

This bridge is a maintenance bridge that also serves as ordinary road, and the road classification shall conform to the existing road at the bridge location on East Bank Road. Existing roads are classified as "urban roads" and the design speed is 40km/h. The existing bridge, San Francisco Bridge (effective width B = 7.3m)), has one lane on each side, but the approach roads behind and ahead of the existing bridge are four lanes, two lanes on each side. Therefore, the new bridge will be widened from the current width of 2 lanes to 4 lanes as the bridge is replaced.

(ii) Cross Sections

- Total Road Width: 17.0m
- Effective Road Width: 14.6m (Carriageway: 3.35m*2 + Shoulder: 0.6m*2)
- Number of Lane: 4 Lanes (2 Lanes on each direction)



Source: Study Team

Figure 6.4.20 Cross Sections of Cainta Floodgate

(iii) Elements of Alignment

- Horizontal Alignment: R=∞
- Vertical Alignment: Level (i = 0.00%)
- Proposed Elevation of Road Surface: EL. +18.400
- Crossfall: Carriageway－1.5%, Sidewalk－2.0%
- Skew: None (90° 00'00")

(c) Bridge Condition

(i) Bridge Length

L= 18.50m

(ii) Cross Sections

W= 1.200m (Sidewalk) + 0.600m (Shoulder) + 3.350m *2(Carriageway) + 0.600m (Shoulder) + 1.200m (Sidewalk) = 17.000m

(iii) Pavement

Asphalt concrete pavement (ACP), 50mm

(iv) Loads

A. Dead Load

The load conditions are the same as for those of MCGS.

(d) Construction Condition

Since the existing bridge will be removed, it will be necessary to shift the existing road during the construction. (refer to **Sub Section 7.4.5.4 Detour Planning During Construction of the Cainta Floodgate**)

4) Bridge Planning

(a) Determination of Bridge Length

(i) Seat Width

The seat length of girder is calculated after setting the span length assuming 1/2 of the abutment width as the bridge seat width. If the abutment width is set to 3.500m, the girder end overhang length is set to 350mm, and the expansion gap is set to 50-100mm, the span length will be 18.150m as shown below.

L= 16.250+1.500+1.250－(0.35*2+0.05+0.10)= 18.150m

As shown in the following equations, the seat length of girder and the bearing edge length are approximately 0.791 m and approximately 0.291 m, respectively.

Seat Length of Girder SEM*[1]= 0.7+0.005*L2 = 0.7+0.005*18.150≒ 0.791m

(*[1] Specifications for Highway Bridges in Japan, Volume Ⅴ 16.2 -seat length of girder)

Bearing Edge Length S*[2]= 0.2+0.005*L2 = 0.2+0.005*18.150≒ 0.291m

(*[2] Specifications for Highway Bridges in Japan, Volume Ⅳ 8.6 -design of seat width)

As shown in the following equation, assuming that the standard spacing of the bearing anchor bolts Lb is approximately Lb = 500 mm, the minimum distance Ls from the center of the bearing to the pier edge is 0.541 m.

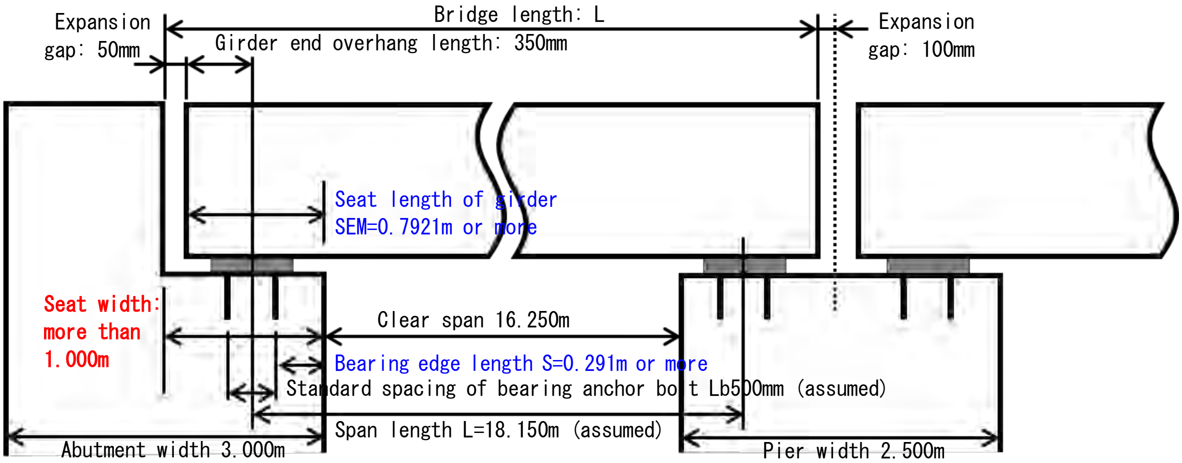
Ls= Lb/2+S= 0.50/2+0.291= 0.541m

The required seat width is B = 1.005m. (refer to the following equation)

B= 0.05+0.791= 0.841m (Calculated from the expansion gap and the seat length of girder)

B= 0.05+0.35+0.541= 0.941m (Calculated from the expansion gap, the girder end overhang length and the minimum distance Ls)

Thus, the bridge seat width is set to **B = 1.000m**. (refer to Figure 1.1.11)



Source: Study Team

Figure 6.4.21 Determination of bridge Length of Cainta Floodgate Maintenance Bridge

(ii) Bridge Length

From the above, the bridge length of Cainta Floodgate maintenance bridge is as follows.

L= 16.250 + 1.000 + 1.250 = **18.500m**

(b) Selection of Type of Superstructure

(i) Span Composition/Arrangement

The span composition will be 2 spans according to the position of the piers.

The span lengths are as follows.

L1,2= 17.650m

(ii) Type of Superstructure

In accordance with DGCS, general type of superstructure and standard applicable span length are as shown below. In addition, continuous girder will be excluded from the study since the maintenance bridge will be a short span in principle from the "Guideline for Flexible Sluiceway, Japan".

1. Reinforced Concrete Deck Girder (RCDG) : Span Length (13.0m~20.0m)
2. PC-I beam : Span Length (9.0m~42.7m)
3. Steel I-beam : Span Length (15.0m~30.0m)

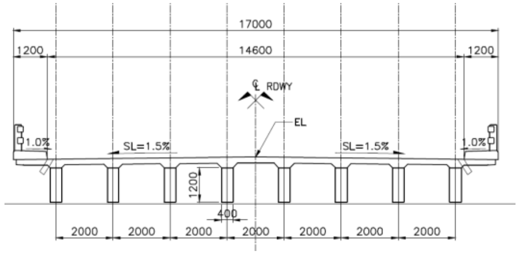
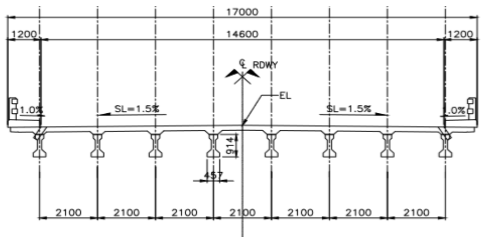
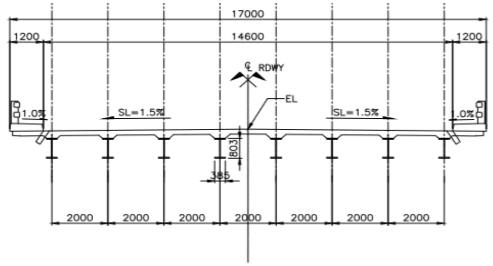
(iii) Comparison of Type of Superstructure

A comparative study was conducted when the above three types of superstructure were applied to each span. The comparison table is shown on the **Figure 6.4.17**

As a result, since it is the most economical and has excellent workability, detailed design will be carried out with **the alternative-1 (RCDG)**.

.

Table 6.4.17 Comparison of Type of Superstructure-Cainta Floodgate Maintenance Bridge

	Alternative-1 Concrete Deck Girder (RCDG)		Alternative-2 PC-I beam		Alternative-3 Steel I-beam	
Cross Section	 		 		 	
Girder Height, Bridge Span	H=1200mm, L=18.350m		H=1143mm, L=18.350m		H=700mm, L=18.350m	
Structural Features	Single-span reinforced concrete deck girder -Girder weight is heavier than Alt-3	△	Single-span Pre-stressed concrete deck girder -Girder weight is heavier than Alt-3	△	Single-span steel I-shaped girder - lightest girder weight among the alternatives	○
Construction Workability	-Bents for erection are needed -Easiest to construct among the alternatives	◎	-Bents for erection are not needed -The girders can be transported from fabricator's plant	◎	-Bents for erection are needed -Large yard is needed for girder fabrication and assembly	△
Maintainability	-No periodic maintenance works are needed	◎	-No periodic maintenance works are needed	◎	-Periodic repaint/ maintenance cost is needed	△
Construction Cost (Peso/lm)	*****	◎	*****	○	*****	△
(Ratio)	(1.00)		(1.14)		(1.30)	
Total Evaluation	**◎ : Selection**		○ : Second Place		△ : Third Place	

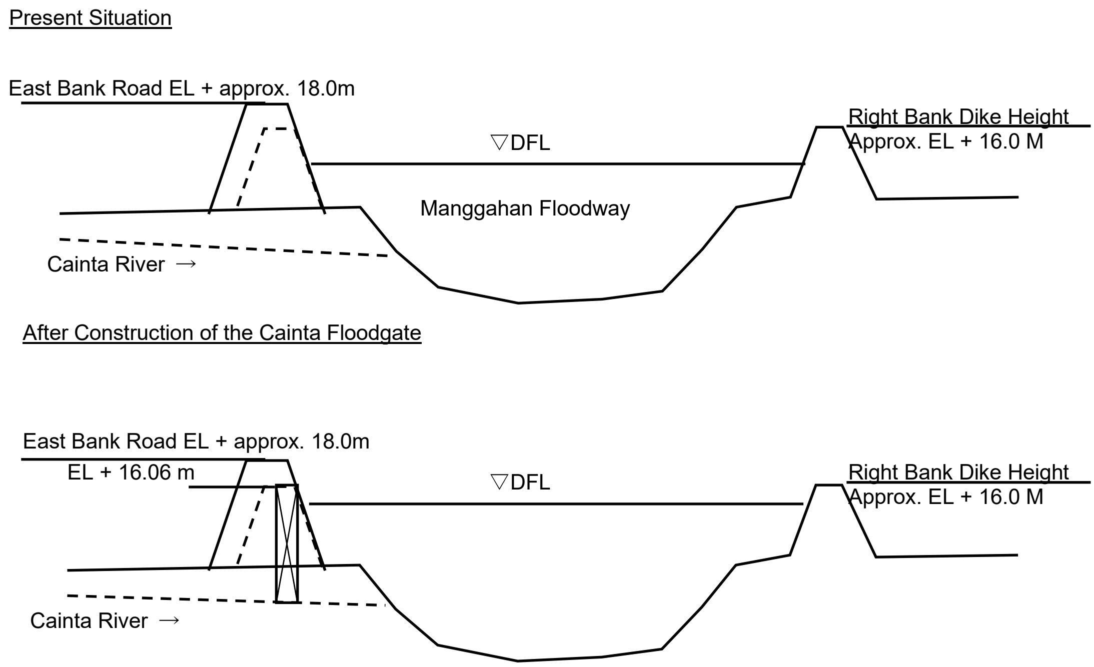
Legend : Good-◎, Fair-○ , Poor-×

Note: Cost is not presented due to the prior released version
Source: Study Team

(9) Study on the Necessity of Curtain Walls

In the Cainta Floodgate, the DFL of the Floodway side is EL + 14.853 m, while the DFL of the Cainta River is EL. + 13.340, Hence, the difference is about 1.5 m.

The design dike height of the Manggahan floodway in this area is EL+15.94 m, while the existing dike height is EL.+18.0 m, which is at least 2m higher. The dike heights on the left and right banks is shown in **Figure 6.4.22**. As described above, it is assumed that the height of the existing dike was raised accompanying the improvement of the East Bank Road. If the floodgates are designed to stop the water flow up to the height of the existing dike, the flood damage may occur on the opposite dike, where the dike height is lower at the time of the excess flood. Consequently, the balance of flood control on the left and right sides of the dike is disrupted from the current state. Thus, in this design, considering the dike height of the opposite bank as the gate height necessary for flood control, EL+16.053 ≒ EL+16.06 m was set as the crown height of the gate part as DFL+1.2 m (= 14.853 m+1.2 m).



Source: Study Team

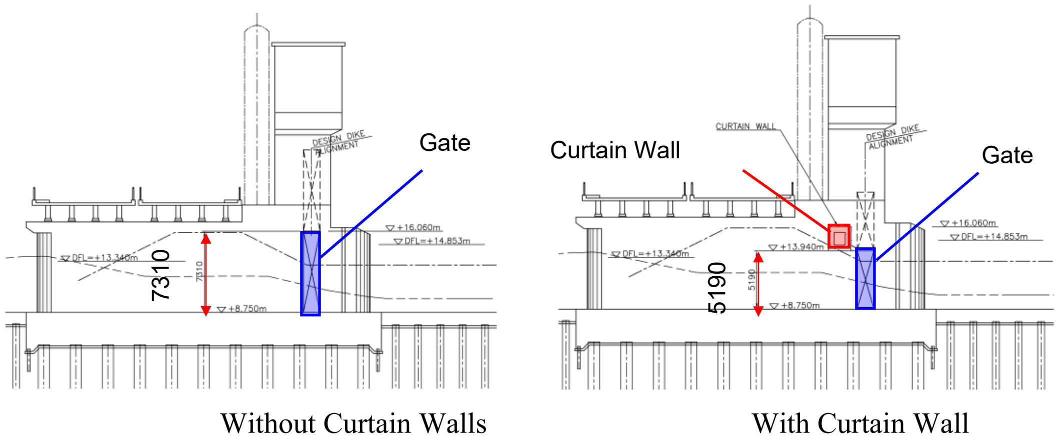
Figure 6.4.22 Dike Height of Manggahan Floodway around the Confluence of Cainta River (Schematic Diagram)

On the other hand, the floodgate is a structure that functions as a dike, the ceiling height of the gate installed in the sluice shall be equal to or higher than the design dike height. In this design, the gate height is about 7.4 m (EL+16.06 - EL + 8.75 = 7.31 m) from the riverbed height.

The height of the proposed dike of the Cainta River is EL + 13.940 m (DFL + Headroom Height: 0.6 m) (see **Figure 6.4.3**). The necessary height of the gate can be suppressed to about 5.2 m (EL 13.94 m - Height of Floodgate EL. 8.75 m = 5.19 m) by making a curtain wall between the current dike crown of the Floodway and the proposed dike crown of the Cainta River.

In this case, while the weight of the steel material can be reduced, a curtain wall (16m) is installed between the piers, and a beam structure (2.4m height) is required. When considering the motion at the time of the L2 earthquake, it is obvious that stress concentrates on the small cross-section of a beam member between piers, and it is difficult to ensure safety from a structural viewpoint. Besides, from the perspective of securing water tightness, it is necessary to integrate the curtain wall into the pier. There is a concern that cracks may occur during curing after casting the main concrete.

In consideration of the above situation, it was decided not to install a curtain wall at the Cainta floodgate.



Source: Study Team

Figure 6.4.23 Difference in Height of Gate Door with and without Curtain Wall

(10) Study on the Type of Foundation

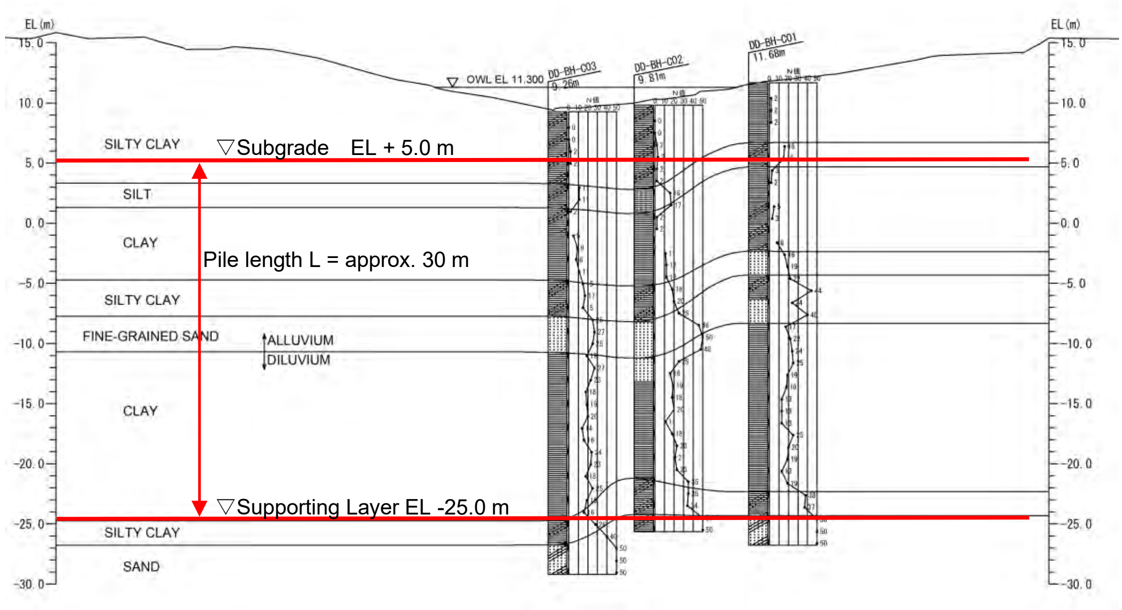
1) Supporting Layer

According to the "Technical Criteria for River Works: Practical Guide for Planning [Design] [1]" (Japan Rivers Association), the base ground of the water gate is indicated as follows:

✓ In the sand and gravel layers, N-value is almost 30 or more,

✓ N-value is almost 20 or more in the cohesive soil layer.

The upper surface of the supporting layer is a SILTY-CLAY layer that a soil layer of N value of 30 or more continues more than 5m. Since the top surface height of the SILTY-CLAY layer differs by up to 4m at each borehole, the support layer is set around the EL -25 m of the deepest DO-BH-C03 hole in this design. Since the elevation of the bottom surface of the floor slab of the main body is EL.+5.75 m, the pile length will be 30m or longer.



Source: Study Team

Figure 6.4.24 Assumed Geological Section

2) Type of Foundation

The "Technical Criteria for River Works: Practical Guide for Planning [Design] [1]" (Japan Rivers Association) lists direct foundations, pile foundations, and caisson foundations as the types of foundations of Floodgates. The applicability of each type is summarized below. The pile foundation type is adopted in this design.

- Direct Foundation → Adopted when enough soil bearing capacity exists and consolidation settlement does not occur in a place where the ground is good such as rock, gravel or sand. Since the cohesive soil layer distributed near the subgrade of Floodgate has N-value < 20 which is not a good supporting layer, the direct foundation is not suitable.
- Pile Foundation → In case of pile foundations, a sandy soil layer with an N-value > 30 or a cohesive soil layer with an N-value > 20 is applicable.
- Caisson Foundation → In the case of the Cainta Floodgate, the applicability is low, since the clear span is 20 m or less and the vertical load is small. This is less economical than the pile foundations.

After soil investigation, liquefaction layer was not confirmed, so liquefaction is not considered in this design.

3) Selection of Materials and Construction Methods

Based on the application examples and interviews with construction companies, the following combinations of pile materials and driving methods were extracted. In this design, the steel pipe pile/driving pile method (vibro-hammer method) is recommended. A comparison of pile materials is shown in **Table 6.4.18**.

- ✓ The impact on the surrounding houses is concerned, because the driving method is used for RC piles.
- ✓ The applicability of cast-in-place piles is low because the groundwater level is assumed to be high at the time of pile driving. After all, the construction by the half river cofferdam will be constructed at this site.

Table 6.4.18 Comparison of Pile Materials

pile material	Casting method	Outline	workability	Influence on the surroundings	Evaluation
RC pile	driving pile method (striking method)	□Method for driving 700 PHC piles by hitting the head of ready-made pile with hydraulic hammer, drop hammer, etc.	○ : No problem with groundwater level No: Difficult to deal with short pile length and high stalls ○: The construction period is short.	No: This product is not used in urban areas because it is generally accompanied by large noise and vibration.	No: Difficulty in applying to the said work from the viewpoint of workability and impact on surrounding areas
cast-in-place pile	all-casing method	Casing tube is embedded, and after drilling in the tube, reinforcing bar is erected, and after pile body is formed by placing concrete, the casing is pulled out and cured.	No: Low applicability in areas with high groundwater levels ○ : Easier to respond to short pile lengths and high stops × : The curing period is necessary and the construction period is long.	○: No problem.	× : Difficult to apply for the construction work in which the groundwater level is high and the construction period is limited
cast-in-place pile	reverse method	Curing after the pile body is formed by placing concrete and erecting the reinforcing rod after the preceding inner	× Low applicability in areas with high groundwater levels ○ : Easier to respond to short pile lengths and high stops	○: No problem.	× : Difficult to apply for the construction work in which the groundwater level is high and the

		moat and excavation to the support layer The pore wall is held by water pressure.	× : The curing period is necessary and the construction period is long.		construction period is limited
steel pipe pile	driving pile method (vibro-hammer method)	A hollow steel pipe pile is driven by a vibro-hammer.	○ : No problem with groundwater level ○ : Easier to respond to short pile lengths and high stops ○: The construction period is short.	○ : Noise and vibration are smaller than those of the striking method, and there are many construction results in urban areas.	○ There are few problems for workability and peripheral effects. ->Recommended

Source: Study Team

Method extracted from Table No. 3.1 of the manual for pile foundation design on page 444

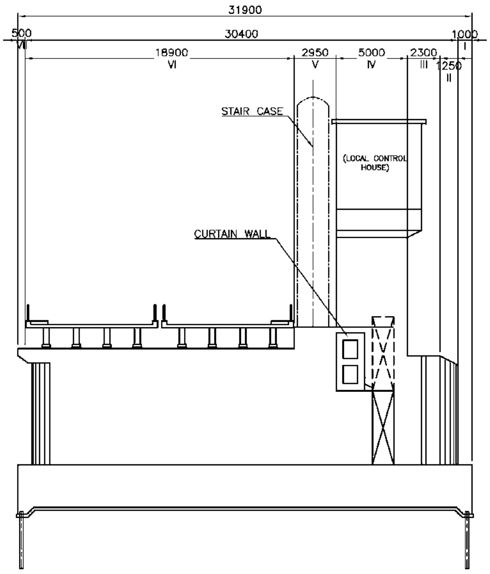
(11) Determination of Main Body Specifications (Section Dimensions)

1) Body Length

The main body length of the Floodgate is determined from the dimensions of the curtain wall width, the pier width, maintenance bridge width, etc.

I. Clearance : 1.0 m
II. End of Center Pier : 1.50 m
III. Groove for Stop Log : 150 m
IV. Width of Column : 6.0 m
V. Staircase : 2.95 m
VI. Maintenance Bridge + Ground Cover + Clearance** : 17.10 m
VII. extension of the weir : 0.45 m
Total length: 31.00 m

* Maintenance bridge 17.00 m + separation between maintenance bridge and body 0.05*2 m)



Source: Study Team

Figure 6.4.25 Length of Main Body (Cainta Floodgate)

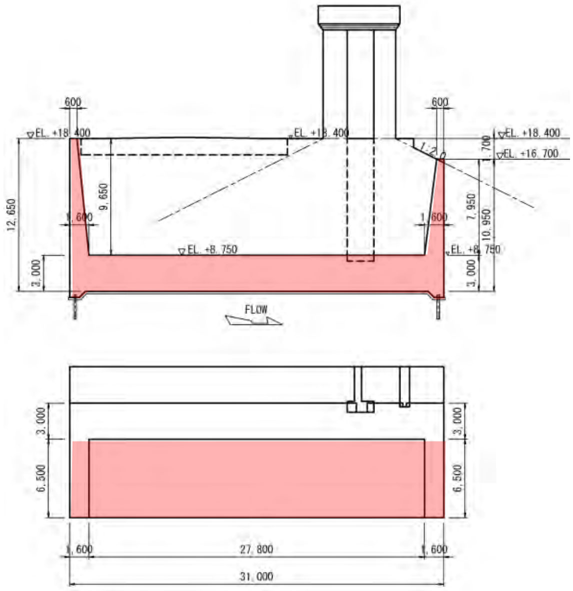
2) Breast Wall

The height and length of breast wall are set according to the "Technical Standards for River Erosion Control (Draft) Design Part I" as follows. The purpose of the breast wall is to prevent soil suction and slope collapse. The structural specifications are shown in **Table 6.4.19** and **Figure 6.4.26**. The locations of the breast walls are shown in **Figure 6.4.27**.

Table 6.4.19 Structure of the Breast wall

Member	Item	Size	Remarks
upstream breast wall	Height:	12.65 m	Match the existing dike height
	Length:	6.5 m	More than half the breast wall height
downstream breast wall	Height:	10.95 m	Match the existing dike section
	Length:	6.5 m	Align with the upstream breast wall

Source: Study Team

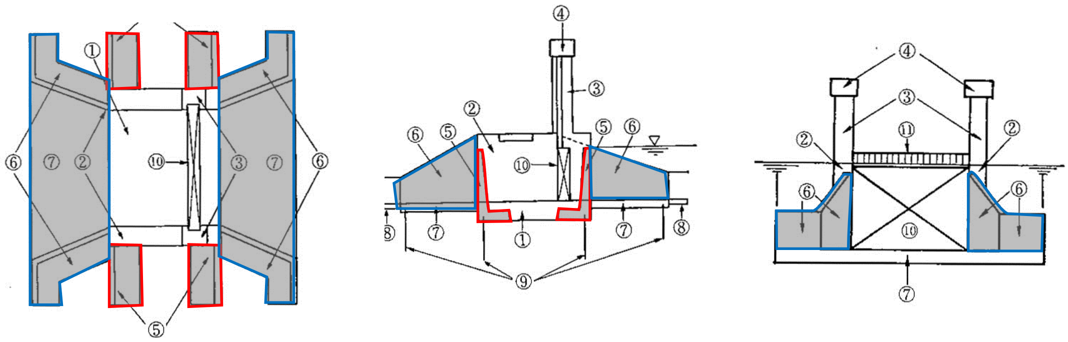


Source: Study Team

Figure 6.4.26 Cainta Floodgate Breast wall Structure

3) Wing wall

Figure 6.4.27 shows the location of the wing wall.



1.床版 Bottom Slab
2.堰柱 Pier
3.門柱 Column
4.操作台 Operation Deck
5.胸壁 Breast Wall
6.翼壁 Wing Wall
7. 水叩き Apron
8. 護床工 Bed Protection
9. 遮水工 Seepage Cut-off Wall
10. ゲート Gate
11. 管理橋 Maintenance Bridge

Source: River erosion control technical standard (draft) Design Part I, English translation added by the investigation team

Figure 6.4.27 Names of the Parts of the Floodgate

The downstream wing wall is gradually expanded at 1: 5.0 so as not to be damaged by vortex flow. The upstream wing wall shall not be gradually expanded because the width of the Watergate is comparable to the design section of the Cainta River.

The wing wall shall be installed within a range not less than the cross-section of the bank (the existing dike sections). For the downstream wing wall, the necessary length is 17.7m, which is larger than the cross-section of the bank. However, since the required installation range of the bed protection work A is 18.0 m, the wing wall-length shall be 18.0 m. For the wing wall at the right upstream bank, the length of the wing wall shall be 11.5 m considering the generator house will be constructed at the site. The left-wing wall on the upstream shall have 11.5 m to be the same as the right side.

The embedded length of the wing wall edge shall be in the range of the wall height + 1.0 m for the upstream right wing wall, and the riverside shall be in the area of the riverside channel (1: 2.0). The upstream left wall on shall be at the height of + 0.5 m from the slope of the dike on the land side connected to the gate.

- ✓ Length of the downstream wing wall : 18.0 m (it shall cover the cross-section of the existing dike and the installation area of the bed protection work A.)
- ✓ Length of the upstream wing wall (right bank) : 11.5 m (the generator house is a critical factor)
- ✓ Length of the upstream wing wall (left bank) : 11.5 m (Same as the right bank)

4) Width of Weir

According to the "design of weir" (Dam Engineering Center), the width of the weir can be calculated based on the span length as follows:

$t = (1/10 \text{ to } 1/13) \times B = 1.46 \text{ m to } 1.9 \text{ m}$

where, t : Width of Pier (m)

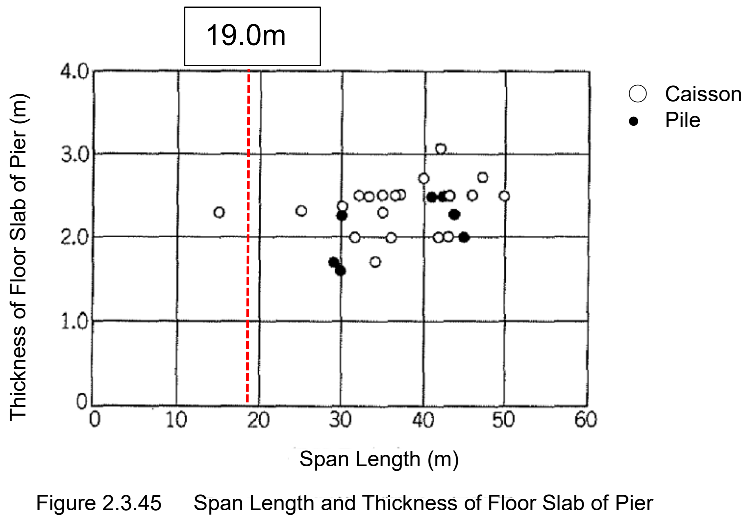
B : Span Length (m) = 19.0 m

However, in the "design of weir" (Dam Engineering Center), the above equations are summarized based on the statistics before considering Level 2 earthquake ground motions. The reference also states that the past cases of the width of the pier are 5.5 m to 2.5 m. Since it is disadvantageous for bending and shearing in the direction perpendicular to the flow direction if the pier width is narrow, it is desirable to widen the pier width as much as possible from the viewpoint of structural stability.

Therefore, in this basic design, the width of the pier is set to 3.0 m and the blocking rate is 8.6% (= 3.0 m/35.0 m).

5) Thickness of Bottom Slab

With reference to the relationship between thickness of floor slab and span described in the "design of weir" (Dam Engineering Center), the thickness of the slabs should be in the range of 3.0 m to 2.0 m. Considering that **Figure 6.4.28** shows the relation in which Level 2 earthquake ground motions are not considered and the balance between the width of pier , the thickness is set to 3.0 m.



Source: "Design of Weir" Dam Engineering Center

Figure 6.4.28 Span Length and Thickness of Floor Slab of Pier

6) Height of Column

(a) Height of the Bottom End of the Gate when Fully Opened

The height of the bottom end of the gate when the gate is fully opened is determined from the height of the design dike crown of the mainstream and the current dike height.

- Design Dike Crown: EL.+15.940 m
- Current Dike Height: Approximately EL.+18.0 m
- Finished Elevation : EL+18.4 m (Considering Extra Embankment)

From the above, the height of the bottom end of the gate when the gate is fully opened is set to EL.+18.4 m.

(b) Height of Column

As for the height of column, in accordance with the " Technical Criteria for River Works: Practical Guide for Planning [Design] [1]", clearance of 1 m or more is secured to the bottom edge of the operation deck. The height of the column including the operating deck is calculated as follows:

Height of the bottom of the Gate when it is fully opened	: EL.+18.0 m
Gate Height	: 7.31 m
Clearance	: 1.60 m
+ Thickness of Operation Deck Slab	: 2.00 m
Top Height of the Column including the operation Deck	: EL.+29.31 m Round(EL.29.40m)

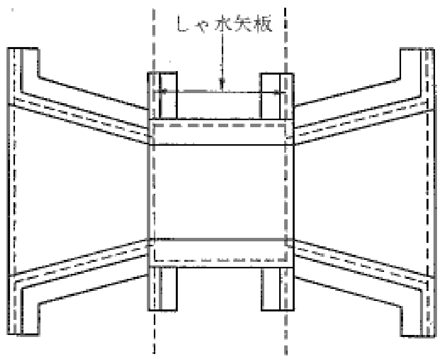
Therefore, the height of the column including the operating deck is set at E.L.+29.40 m.

7) Seepage Cut-off Wall

(a) Location

Seepage cut-off wall is installed to prevent soil particles from flowing below the weir (vertical cut-off wall) and in the weir side face (horizontal cut-off wall). and to prevent soil from being sucked out by scouring. The point of installation is shown **Figure 6.4.29**.

SSP for Seepage Control



Source: Technical Criteria for River Works: Practical Guide for Planning [Design] [1]

Figure 6.4.29 Layout of Seepage Cut-off Wall

(b) Type of Wall

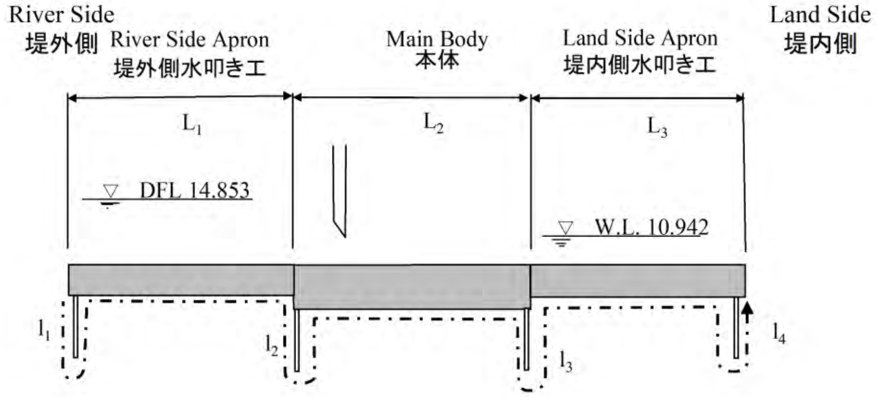
The previous geological investigation in the vicinity has not been confirmed the surface of basement rock at the water gate installation site. Hence, the steel sheet pile is adopted.

(c) Length of Cut-off Wall

The required length of the cut-off wall is calculated from the equation of Lane.

(i) Vertical Direction

The length of the cut-off wall in the vertical direction is examined as follows.



Water Difference ⊿H
水位差

(Unit: EL.m)

	STA	River Side 堤外側	Land Side 堤内側	Water Diffrence 水位差	Remarks 備　考
Case1	4+525	14.853	10.942	3.911	Land Side: Lowest Water Level in Laguna Lake 堤内側:マンガハン放水路低水位

Horizontal Creep Distance L
水平クリープ長

(Unit: m)

	L_1	L_2	L_3	L_4	Total L	Remarks
Case1	18.0	31.0	11.5		60.5	

Required vertical creep distance is calculated as flows;
鉛直方向の必要クリープ長は以下の通り算定できる。

$$C \le \frac{L/3 + \sum l}{\Delta H}$$

Vertical Creep Distance Σl and Creep Distance at Each Seepage Cut off Wall
鉛直クリープ長Σlと各遮水工位置で必要なクリープ長

	Creep Ratio クリープ比	Σl (m)	Number of Walls 遮水工個所数	Type of Ground 土質
Case1	8.5	13.08	4.00	Very Fine Sand or Silt 極めて細かい砂またはシルト

Case 1

(Unit: m)

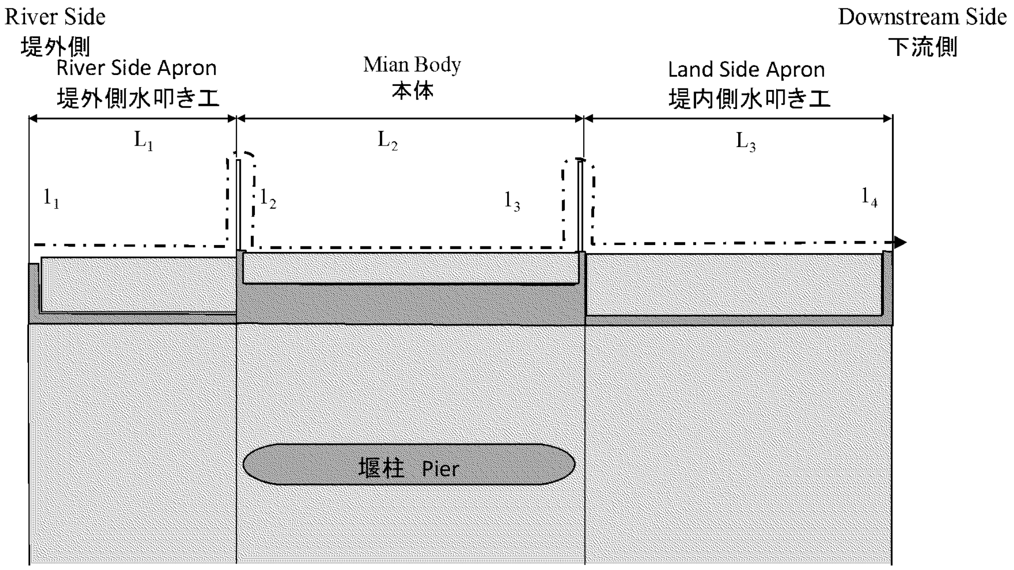
	l_1 SSP	l_2 SSP	l_3 SSP	l_4 SSP		Total 合計	Stability 安定性	Ccal. 計算クリープ比
Length 遮水工長	2.0	2.0	2.0	2.0		8.0	OK	9.25
Side 面数	2.0	2.0	2.0	2.0		-		
Distance 距離	4.0	4.0	4.0	4.0		16.0		

SSP…Steel Sheet Pile(鋼矢板), CC…Concrete Cut off（コンクリートカットオフ）

From the above calculation results, the required stability against seepage can be satisfied with the steel sheet pile L = 2.0 m.

(ii) Horizontal

The length of the seepage cut-off wall in the horizontal direction has been studied as follows:



Water Difference ⊿H
水位差 (Unit: EL.m)

	STA	Upstream 上流側	Downstream 下流側	Water Diffrence 水位差	Remarks 備　考
Case1	4+525	14.853	10.942	3.911	Land Side: Lowest Water Level in Laguna Lake 堤内側:マンガハン放水路低水位

Horizontal Creep Distance L
水平クリープ長 (Unit: m)

	L_1	L_2	L_3	L_4	Total L	Remarks
Case1	18.0	31.0	11.5	0	60.5	

Required Horizontal creep distance is calculated as fllows;
水平方向の必要クリープ長は以下の通り算定できる。

$$C \leq \frac{L/3 + \sum l}{\Delta H}$$

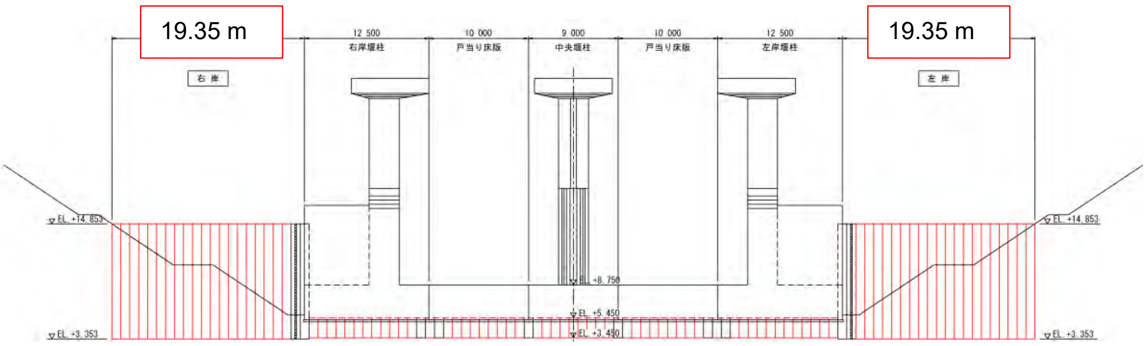
Horizontal Creep Distance Σl and Length of Each Seepage Cut off Wall
水平クリープ長Σlと各遮水工位置で必要なクリープ長

	Creep Ratio クリープ比	Σl (m)	Reqired Length of Wall at 1 location (m) 1か所あたりの必要長	Number of Walls 遮水工個所数	Type of Ground 土質
Case1	8.5	13.08	3.3	2.0	Very Fine Sand or Silt 極めて細かい砂またはシルト

Case 1 (Unit: m)

	l_1	l_2	l_3	l_4	Total	Stability	Ccal.
Length 遮水工長	0.0	4.0	4.0	0.0	8.0		
Side 面数	0.0	2.0	2.0	0.0	-	OK	9.25
Distance 距離	0.0	8.0	8.0	0.0	16.0		

From the calculation results, the required stability is sufficiently satisfied by installing the cut-off wall up to 4.0m from the end pier. On the other hand, the width of the existing ground open-cut surface up to EL+14.853 of the design high-water level is as follows: As shown in **Figure 6.4.30,** the range is wider than the above range. Therefore, a horizontal cut-off wall is installed within a range of 19.35 m from the end pier.



Source: Study Team

Figure 6.4.30 Installation Range of Cut-off Walls

8) Apron

(a) Water Level Condition

The water level condition for the study on length and thickness of apron are shown in **Table 6.4.20**.

Table 6.4.20 Water Level Conditions

Calculation Conditions		water level			Remarks
		River Side	Land Side	water level difference	
Downstream Apron	DFL in Floodway	10.94 1)	13.340	2.400	
Upstream Apron	DFL in Tributary	14.853	10.94 1)	3.913	

1) Set from the observed minimum water level of Lake Laguna

Source: Study Team

(b) Apron length

The length of apron shall be the same as the length of wing wall and shall be set as follows:

- Length of Upstream Apron : 18.0 m
- Length of Downstream Apron : 11.5 m

(c) Thickness of Slab

(i) Riverside Apron

The thickness of the apron slab is set to be able to resist the assumed uplift by its own weight. The required thickness shall be calculated based on the formula shown in the "Technical Standards for River Erosion Control (Draft) Design Part I".

A. Calculation of Uplift

Uplift pressure can be calculated by the following formula:

$$U_{px} = \left(h_2 + \Delta h \frac{\sum l - l_x}{\sum l}\right) \times W_0$$

Where,
U_{px} : Uplift (kN/ m^2)
h_2 : Depth in Downstream Side(m)
Δh : Water Level Difference (m)
$\sum l$: Total Creep Distance (m)
l_x : Creep Distance from the Upstream End to the Evaluation Point (m)
W_0 : Unit Weight of Water (kN/ m^3)

B. Calculation of the Required Thickness of Slab

The required thickness is calculated by the following formula:

$$t = F_S \frac{u_{pm} - h_2 \cdot W_0}{\gamma_c - W_0}$$

where, t: : Required Thickness for Apron (m)
F_S : Safety Factor 4/3
u_{pm} : The Maximum Uplift Acting on the Apron (kN/m^3)
γ_c : Unit Volume Weight of Concrete (kN/m^3)
h_2 : Depth (m)
W_0 : Unit Volume Weight of Water (kN/m^3)

Item	Symbol	Case 1 Flood	Unit	Remarks
Water level in Floodway Side	h_2	2.192	m	Lowest Water Level of Laguna Lake
Difference of Water level	Δh	3.91	m	DFL of Floodway Side
Total Creep Distance	$\sum l$	82.65	m	
Creep Distance from the Upstream edge to the evaluation point	l_x	68.45	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Uplift at the evaluation point	U_{px}	**28.065**	kN/m^2	

Item	Symbol	Case 1 Flood	Unit	Remarks
Safety Factor	F_s	1.333		
Maximum Uplift acting on Apron	u_{pm}	28.065	kN/m^2	
Unit Weight of Concrete	γ_c	23.5	kN/m^3	
Water Depth	h_2	2.192	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Required Thickness of Apron	t	**0.641**	m	

As described above, since the required floor slab thickness is 0.641 m, the floor slab thickness of the downstream aproning work is set to 0.7 m

(ii) Upstream Apron

Uplift pressure and necessary floor slab thickness are calculated by the same method as the riverside apron.

Item	Symbol	Case 1 Flood	Unit	Remarks
Water level in Floodway Side	h_2	2.192	m	Lowest Water Level of Laguna Lake
Difference of Water level	Δh	2.398	m	DFL of Floodway Side
Total Creep Distance	$\sum l$	82.65	m	
Creep Distance from the Upstream edge to the evaluation point	l_x	60.25	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Uplift at the evaluation point	U_{px}	**27.851**	kN/m^2	

Item	Symbol	Case 1 Flood	Unit	Remarks
Safety Factor	F_s	1.333		
Maximum Uplift acting on Apron	u_{pm}	27.851	kN/m^2	
Unit Weight of Concrete	γ_c	23.5	kN/m^3	
Water Depth	h_2	2.192	m	
Unit Weight of Water	W_0	9.8	kN/m^3	
Required Thickness of Apron	t	**0.620**	m	

As described above, since the required floor slab thickness is 0.620 m, the floor slab thickness of the downstream aproning work is set to 0.7 m

9) Bed Protection work

(a) Bed protection Length

(i) Method of Study

The installation range of the bed protection is set by the following method:

- The length of the downstream protection shall be long enough to prevent the riverbed from being scoured by underflow from the bottom of the gate when the gate is opened.
- The length of the upstream protection should be long enough to prevent local scouring.

(ii) Study Condition

Table 6.4.21 shows a list of study conditions.

Table 6.4.21 List of Study Conditions

Condition	Items	Condition	Remarks
Floodgate Condition	Water Flow Width	32.0 m	
	Invert Elevation	EL. +8.75	
	Opening Speed	0.3 m/min	
Water Level Condition	Water Level in landside	EL. +13.940	Full Water Level of the Tributary Operating Water Level of Floodgate 1)
(Cainta River)	water level in Riverside	EL. +12.940	Full Water Level of the Tributary -1.0 m. Operating water level of Floodgate 1)

1) Set based on the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

(iii) Length of Downstream Bed Protection

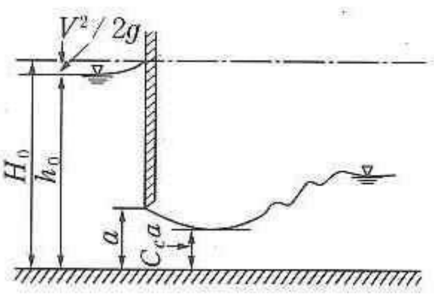
A. Outflow at Gate Opening

The free runoff from the gate of the horizontal channel floor slab is determined by the following equation with reference to the free runoff of the horizontal channel floor gate (Refer to "Hydraulic Formulas, Japan" page 254). Flow coefficient C and contraction coefficient C_c is from the following figures.

$$Q = CaB\sqrt{2gh_0}$$

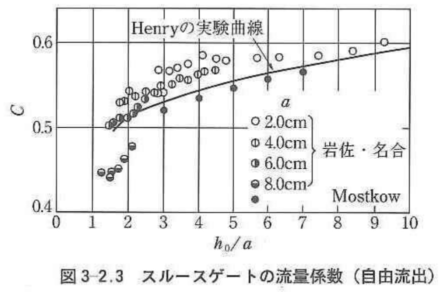
$$Q = C_c aB\sqrt{\frac{2g(h_0 - C_c a)}{1-(C_c a/h_0)^2}}$$

Where, Q : Discharge
a : Gate Opening
B : Outflow Width
h_0 : Water Depth in the Upstream
C : Flow coefficient
C_c : Shrinkage Factor



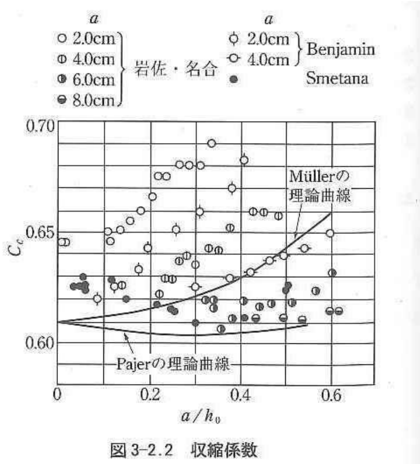
Source: Hydraulic formulae, Japan

Figure 6.4.31 Free Discharge from Sluice Gate



Source: Hydraulic formulae, Japan

Figure 6.4.32 Sluice Gate Flow Coefficient



Source: Hydraulic formulae, Japan

Figure 6.4.33 Shrinkage Factor

The discharge from below the gate is calculated as shown in **Table 6.4.22**.

Table 6.4.22 Free Discharge from the Gate

Land Side: EL. 13.940 m
Opening of Gate: 0.30 m/min
River SIde: EL. 12.940 m
Number of Gates: 2
Width of Flood Gate: 16.00 m
Invert Elevation: EL. 8.750 m

T(hr)	T(min)	T(sec)	h0	Opening of Gate a(m)	h0/a	a/h0	Cc	C	Discharge Q (m3/s)	h1a(Cca)	V1a	Water Level at Downstream h2(m)
0.000	0	0	5.2	0.00					0.0	0.00		4.19
0.017	1	60	5.2	0.30	17.30	0.06	0.61	0.60	58.1	0.18	9.92	4.19
0.033	2	120	5.2	0.60	8.65	0.12	0.61	0.58	112.3	0.37	9.59	4.19
0.050	3	180	5.2	0.90	5.77	0.17	0.61	0.56	162.7	0.55	9.26	4.19
0.067	4	240	5.2	1.20	4.33	0.23	0.61	0.55	213.0	0.73	9.09	4.19
0.083	5	300	5.2	1.50	3.46	0.29	0.61	0.53	256.6	0.92	8.76	4.19
0.100	6	360	5.2	1.80	2.88	0.35	0.61	0.52	302.1	1.10	8.60	4.19
0.117	7	420	5.2	2.10	2.47	0.40	0.61	0.52	352.4	1.28	8.60	4.19
0.133	8	480	5.2	2.40	2.16	0.46	0.61	0.51	395.0	1.46	8.43	4.19
0.150	9	540	5.2	2.70	1.92	0.52	0.61	0.51	444.4	1.65	8.43	4.19
0.167	10	600	5.2	3.00	1.73	0.58	0.61	0.51	493.8	1.83	8.43	4.19
0.183	11	660	5.2	3.30	1.57	0.64	0.61	0.50	532.5	2.01	8.27	4.19
0.200	12	720	5.2	3.60	1.44	0.69	0.61	0.50	580.9	2.20	8.27	4.19
0.217	13	780	5.2	3.90	1.33	0.75	0.61	0.50	629.4	2.38	8.27	4.19
0.233	14	840	5.2	4.20	1.24	0.81	0.61	0.50	677.8	2.56	8.27	4.19
0.250	15	900	5.2	4.50	1.15	0.87	0.61	0.50	726.2	2.75	8.27	4.19
0.267	16	960	5.2	4.80	1.08	0.92	0.61	0.50	774.6	2.93	8.27	4.19
0.283	17	1020	5.2	5.10	1.02	0.98	0.61	0.50	823.0	3.11	8.27	4.19
0.288	17.30	1038	5.2	5.19	1.00	1.00	0.61	0.50	837.5	3.17	8.27	4.19

Source: Study Team

B. Downstream Velocity V_2

Based on the calculated discharge at each opening of the gate, uniform flow calculation (**Table 6.4.23**) is conducted and the downstream velocity V_2 is estimated. Since the downstream water depth is set to upstream water level -1.0m, h_2 = 4.19 is estimated.

Table 6.4.23 Estimation of Downstream Velocity V_2

T(sec)	Discharge Q (m3/s)	Flow Area A(m2)	Channel Width B(m)	Water Level at Downstream h2(m)	Side Slope m	Flow Velocity v(m/s)
0	0.0	146.7	35	4.19	0.0	0.000
60	58.1	146.7	35	4.19	0.0	0.396
120	112.3	146.7	35	4.19	0.0	0.766
180	162.7	146.7	35	4.19	0.0	1.109
240	213.0	146.7	35	4.19	0.0	1.453
300	256.6	146.7	35	4.19	0.0	1.750
360	302.1	146.7	35	4.19	0.0	2.060
420	352.4	146.7	35	4.19	0.0	2.403
480	395.0	146.7	35	4.19	0.0	2.694
540	444.4	146.7	35	4.19	0.0	3.030
600	493.8	146.7	35	4.19	0.0	3.367
660	532.5	146.7	35	4.19	0.0	3.631
720	580.9	146.7	35	4.19	0.0	3.961
780	629.4	146.7	35	4.19	0.0	4.292
840	677.8	146.7	35	4.19	0.0	4.622
900	726.2	146.7	35	4.19	0.0	4.952
960	774.6	146.7	35	4.19	0.0	5.282
1020	823.0	146.7	35	4.19	0.0	5.612
1038	837.5	146.7	35	4.19	0.0	5.711

Source: Study Team

C. Calculation of Length of the Exposed Supercritical Flow Section (L1) and the Hydraulic Jump Section (L2)

The length of the exposed supercritical follow section (L_1) and the hydraulic jump section (L_2) are determined by the following equations. The calculation results are shown on the next page.

The length of the exposed super critical follow section L_1 is obtained by substituting the initial water depth h = h1a (x = 0) to obtain constant a and then substituting h with h1b to obtain x (L_1) in the following equation.

$$-\frac{q^2}{C^2}\,x + a = \frac{1}{4}h^4 - h_c^3 \cdot h$$

where, q : Unit Flow Rate(m^3/s/m)
C : Chezy's Coefficient (= h 1/6/n)
x : Interval Length
a : Constant
n : Roughness Coefficient (= 0.035)

The length of hydraulic jump section is estimated as follows:

L2 = (4.5 ~ 6.0) x h2

In this study, L2 = 6.0 x h2 is adopted.

Table 6.4.24 Length of Hydraulic Jump Section (L2) and the Exposed Supercritical Flow Section (L1) Calculation Results

T(sec)	Opening of Gate a(m)	Discharge Q(m3/s)	h1a(Cca)	V1a	Water Level at Downstream h2(m)	Flow Velocity at Downstream v2(m)	Fruid Numberat Downstream F2	Critical Depth hc(m)	Wate Level at the start of Hydraulic Jump h1b(m)	Unit Discharge q(m3/s/m)	Coefficient a	x=L1 (m)	L2 (m)	L1+L2 (m)
0	0.0	0.0	0.0	0.0	4.19	0.00	–	–	–	–	–	0.00	0.00	0.00
60	0.3	59.0	0.2	9.9	4.19	0.40	0.06	0.66	0.03	1.69	−0.05	0.00	25.14	25.14
120	0.6	114.1	0.4	9.6	4.19	0.78	0.12	1.03	0.12	3.26	−0.40	0.00	25.14	25.14
180	0.9	165.2	0.5	9.3	4.19	1.13	0.18	1.31	0.24	4.72	−1.21	0.00	25.14	25.14
240	1.2	216.3	0.7	9.1	4.19	1.48	0.23	1.57	0.40	6.18	−2.76	0.00	25.14	25.14
300	1.5	260.6	0.9	8.8	4.19	1.78	0.28	1.78	0.57	7.45	−4.99	0.00	25.14	25.14
360	1.8	306.8	1.1	8.6	4.19	2.09	0.33	1.99	0.76	8.77	−8.29	0.00	25.14	25.14
420	2.1	357.9	1.3	8.6	4.19	2.44	0.38	2.20	0.98	10.23	−12.97	0.00	25.14	25.14
480	2.4	401.2	1.5	8.4	4.19	2.74	0.43	2.37	1.19	11.46	−18.34	0.00	25.14	25.14
540	2.7	451.4	1.6	8.4	4.19	3.08	0.48	2.57	1.44	12.90	−26.12	0.00	25.14	25.14
600	3.0	501.5	1.8	8.4	4.19	3.42	0.53	2.76	1.70	14.33	−35.67	0.00	25.14	25.14
660	3.3	540.9	2.0	8.3	4.19	3.69	0.58	2.90	1.91	15.45	−44.99	0.00	25.14	25.14
720	3.6	590.0	2.2	8.3	4.19	4.02	0.63	3.07	2.17	16.86	−57.73	0.00	25.14	25.14

Source: Study Team

From the above calculation results, Section A of the bed protection construction is as follows:

- Length of Hydraulic Jump Section $L_1 = 0.0$ m
- the Exposed Super Critical Follow $L_2 = 25.14$ m

Bed Protection A $(L_1 + L_2) = 25.14$ m

However, the area of the bed Protection A includes the lower apron. Since the length of the floor slab downstream of the Gate is 7.25 m, the protection work A can be omitted if the length of the apron of the downward flowing water is 18.0 m. Since the length of the apron (wing wall length) required for the section above the cross-section of the dike is 17.7 m, the range of the apron in section A of the protection works is extended to 18.0 m.

Downstream Bed Protection A (determination length)
= None
The length of Bed Protection B is calculated as follows:
$L = (3 \sim 5) \times h2 = 15 \rightarrow 12.57 \sim 20.95$ m

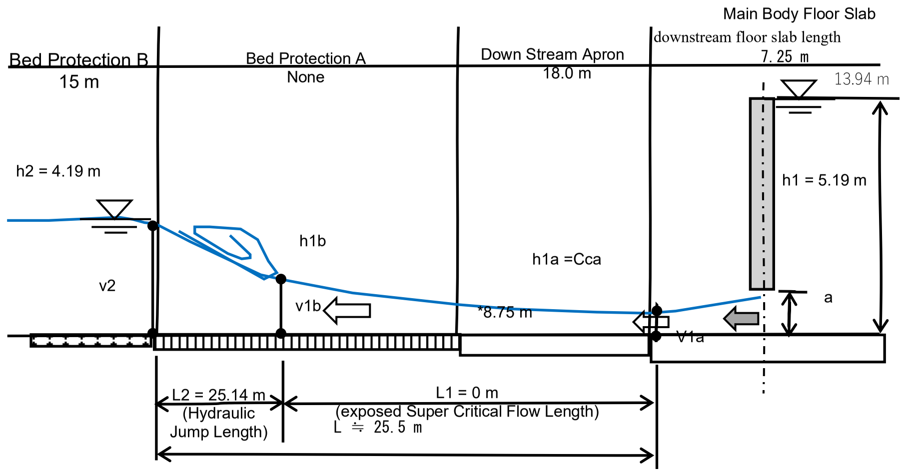


Figure 6.4.34 Length of Bed Protection

(iv) Upstream Protection

Upstream bed protection is determined based on the estimated depth of water and in accordance with the "Structural Design Guide for Groundsill, Japan", which states as follows: “According to hydraulic model test and past cases, the local scouring which is caused by the formation of sandbars in the upstream riverbed and the vortex generated by the lowering of the upstream riverbed height is about the water depth. Based on the above, a rectangular block of 1.5 m square will be installed as the bed protection. In consideration of block allocation, the length of the upstream bed protection shall be 6.0 m.

13.34 - 8.75 = 4.59 → 6.0 m

(b) Weight of Bed Protection

The block weight is calculated by the following formula ("Sliding and Overturning - Layered" model of the "Dynamic Design Method of Revetment").

$$W = a\left(\frac{\rho_w}{\rho_b - \rho_w}\right)^3 \cdot \frac{\rho_b}{g} \cdot \left(\frac{V_d}{\beta}\right)^6$$

Where, W : Minimum Block Weight to Avoid Moving
V_d : Representative Flow Velocity (m/s)
a, β : Factor Determined by the Block Shape (**Table 6.4.25**)
ρ_b : Block Density (kgf*s^2/m^4)
ρ_w : Water Density, 102 (kgf*s^2/m^4)

Table 6.4.25 Coefficient a and β of Atypical Concrete Block

Block Type	Specific Gravity of the Model Block	$a \times 10^{-3}$	β
A: Target Protrusion	ρ_b/ρ_w=2.22	1.2	1.5
B: Planar	ρ_b/ρ_w=2.03	0.54	2.0
C: Triangular	ρ_b/ρ_w=2.35	0.83	1.4
D: three-point support	ρ_b/ρ_w=2.35	0.45	2.3
E - Rectangular	ρ_b/ρ_w=2.09	0.79	2.8

Note: Translated by the JICA Study Team from the Dynamic Design Method of Revetment"

Bed protection A corresponds to the range of the apron, the block weight is not set.

Required weight of bed Protection B is required referring to the maximum downstream flow velocity V_2 of 5.8 m/s (**Table 6.4.23**).

The blocks are supposed to be connected by reinforcing bars, and the weight of the blocks should be set in consideration of the following reduction rate in accordance with the method of "Test report on hydraulic characteristics of revetment blocks, Japan Civil Engineering Research Center".

Reduction rate $= \frac{V_{SC} + 2V_{gc}}{3V_{gc}}$ [2]

Here,

V_{SC} : Unitary critical flow velocity (m/s)

V_{gc} : Limit velocity of moving body time (m/s)

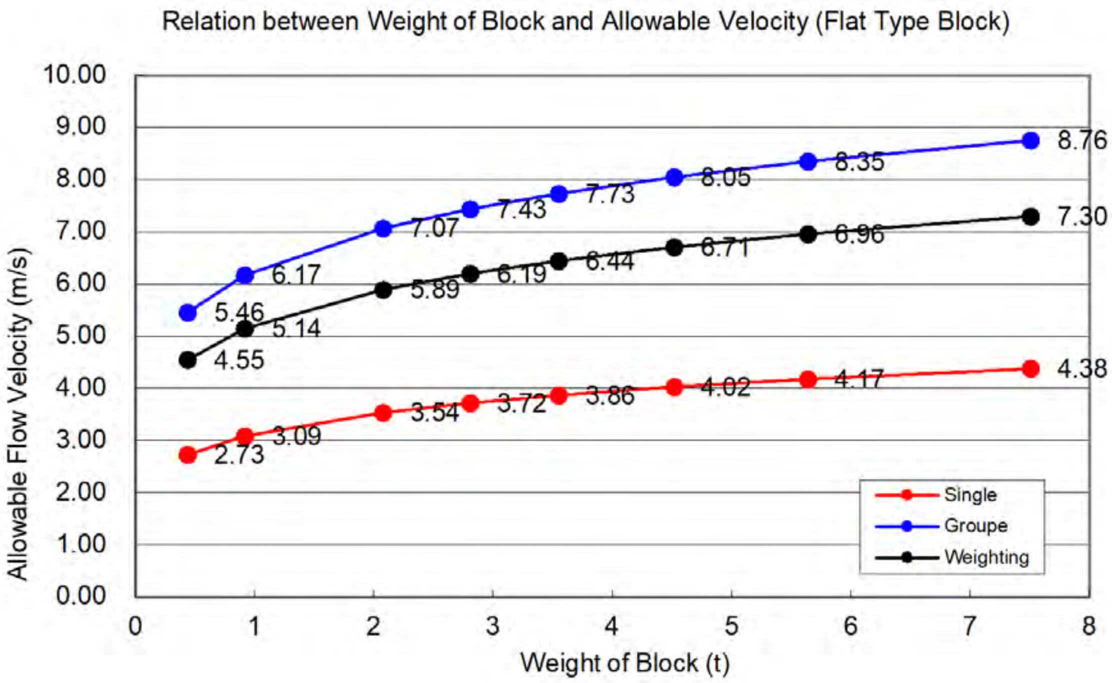
[2] Test report on hydraulic characteristics of revetment block

Table 6.4.26 Calculation of Block Weight in Section of the Bed Protection Work B

Type	Weight	Corrected Critical Flow Rate (m/s)			
		Single	Groupe	Reduction Factor	Weighting
0.5	0.44	2.73	5.46	83%	4.55
1.0	0.92	3.09	6.17	83%	5.14
2.0	2.08	3.54	7.07	83%	5.89
3.0	2.81	3.72	7.43	83%	6.19
4.0	3.55	3.86	7.73	83%	6.44
5.0	4.52	4.02	8.05	83%	6.71
6.0	5.64	4.17	8.35	83%	6.96
8.0	7.51	4.38	8.76	83%	7.30

*Single body is group/ β

Source: Study Team



Source: Study Team

Figure 6.4.35 Relationship between Block Weight and Allowable Flow Velocity

As a result of the examination, the block weight of the bed protection work B is 2t class.

6.4.3.8 Study on Gate Structure and Hoist

(1) Study on Gate Structure

1) Gate Structure

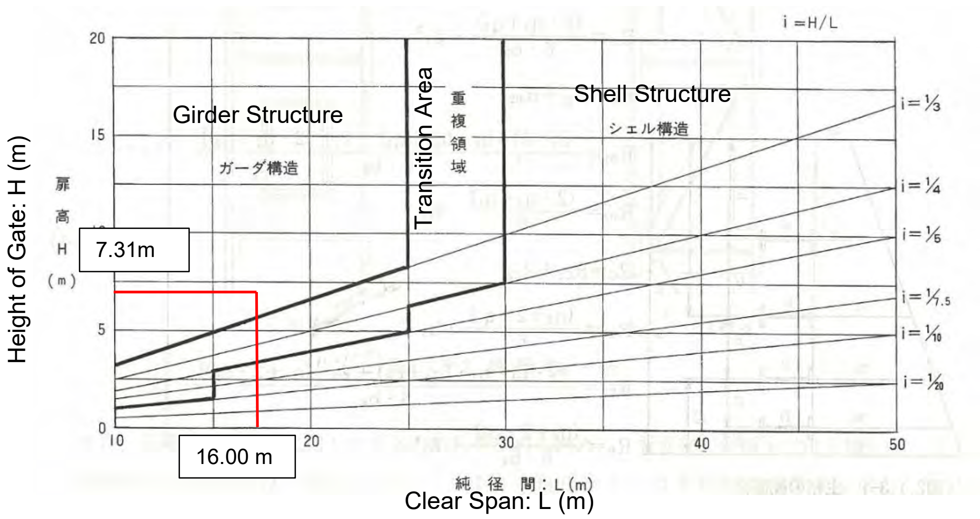
The section of the gate leaf of the fixed wheel roller gate is classified into the following two:

1. Shell Structure Gate
2. Girder Structure

Statistical data of actual values concerning the dimensions of river gates (Clear span, gate height, and ratio between gate height and clear span) and the applicable span, division of gate height, and region of the structure of the gate leaf are shown in **Figure 6.4.36**.

The ratio between the gate height and the clear span of the gate in the case of Cainta Floodgate is calculated as follows:

Approx. 1/2. 199 (H/L = 7.31/16.0 ≒ 1/2. 188)



•

Note: Translated by the JICA Study Team from the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

Figure 6.4.36 Gate Dimensions and Structure Diagram

Figure 6.4.36 illustrates that the actual value falls within in the girder structure domain.

Therefore, the gate structure of the Cainta Floodgate is determined by comparing the gate structure in consideration of the local conditions and economic efficiency.

Table 6.4.27 shows the comparison of gate structures. According to **Table 6.4.27**, the girder structure is recommended and is more economical.

Table 6.4.27 Comparison of Gate Structures

<table>
<tr><th>Item</th><th colspan="2">Shell Structure</th><th colspan="2">Girder Structure</th></tr>
<tr><td>General</td><td colspan="2">・ The cross-section of the gate leaf is a closed shell structure</td><td colspan="2">・ Conventional structural type of fixed roller gate</td></tr>
<tr><td>Structure</td><td>・ Since the surface of the gate leaf is less uneven, it is hardly affected by sediment deposits and driftwood/garbage.</td><td>◎</td><td>・ The surface of the gate leaf is uneven, and it is easy to trap soil and driftwood/garbage on the main girder.</td><td>○</td></tr>
<tr><td>Landscape</td><td>・ The surface of the gate leaf is not uneven.</td><td>◎</td><td>・ The girder is exposed on the surface of the gate leaf on the upstream side, giving a mechanical impression with its unevenness.</td><td>○</td></tr>
<tr><td>O&M</td><td>・ Externally visible area is limited
・ Internal inspection is also necessary, and it needs time and labor.</td><td>○</td><td>・ All most all part can be observed by the external inspection.
・ Easy to be inspected.</td><td>◎</td></tr>
<tr><td>Economic</td><td><table><tr><th></th><th>Weight (t)</th><th>Construction Cost (1000 PHP)</th></tr><tr><td>Gate Leaf</td><td>105</td><td>*****</td></tr><tr><td>Guide Frame</td><td>30</td><td>*****</td></tr><tr><td>Hoist</td><td>35</td><td>*****</td></tr><tr><td>Total</td><td>170</td><td>*****</td></tr></table></td><td>2</td><td><table><tr><th></th><th>Weight (t)</th><th>Construction Cost (1000 PHP)</th></tr><tr><td>Gate Leaf</td><td>115</td><td>*****</td></tr><tr><td>Guide Frame</td><td>30</td><td>*****</td></tr><tr><td>Hoist</td><td>40</td><td>*****</td></tr><tr><td>Total</td><td>185</td><td>*****</td></tr></table></td><td>1</td></tr>
<tr><td rowspan="2">Evaluation</td><td colspan="2">Less economical</td><td colspan="2">It is economically superior. Furthermore, it is easy to inspect.</td></tr>
<tr><td colspan="2"></td><td colspan="2">Recommended</td></tr>
</table>

Note: Cost is not presented due to the prior released version.
Legend : ◎...Better、○...No Problem、△...There is issues to be solved.
Source: Study Team

2) Material of Gate

(a) Possibility of Brackish Water

(i) Results of the Previous Water Quality Test

Table 6.4.28 shows the results of water quality tests carried out with samples taken from Laguna Lake and the Manggahan Floodway. Based on the results shown in **Table 6.4.28,** recent records show that no brackish water has been observed in Manggahan Floodway. Laguna Lake, on the other hand, is brackish. However, the values are low and is not observed every year. Salinity in the Laguna Lake is lower than that of the Marikina River (refer to **Table 6.4.28**).

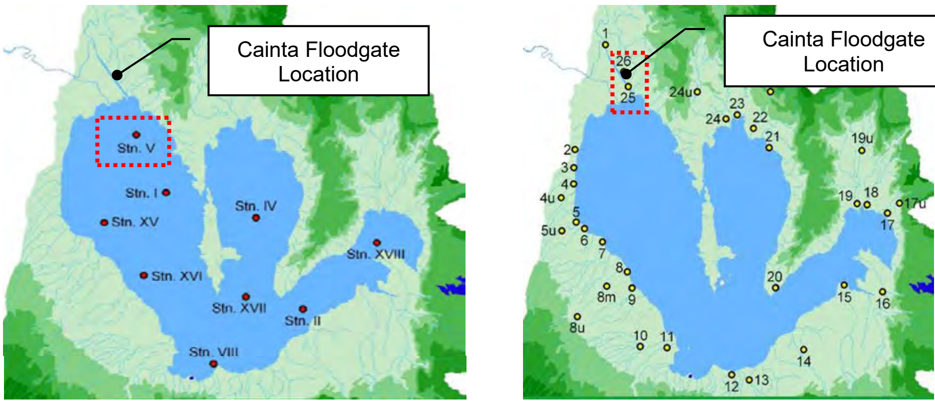
However, according to the previous water quality test reported by LLDA, the water depth is about 0.5 m for the sampling points in Laguna Lake. Also, at the sampling point along the Manggahan Floodway, the depth of water sampling is expected to be shallow (see **Figure 6.4.37**).

Table 6.4.28 Salinity in Previous Water Quality Test

River	Sampling Point	Sampling Month	Salinity	Sampling Month	Salinity	Sampling Month	Salinity
Manggahan Floodway	Sapong Baho River (Cainta)	January 2013	0.25 ‰	February 2013	0.25 ‰	March 2013	0.27 ‰
		April 2013	0.25 ‰	May 2013	0.19 ‰	June 2013	0.18 ‰
		July 2013	-	August 2013	0.18 ‰	September 2013	0.23 ‰
		October 2013	0.18 ‰	November 2013	0.2 ‰	December 2013	-
		January 2014	0.23 ‰	April 2014	0.25 ‰		
		October 2015	0.19 ‰	November 2015	0.23 ‰	December 2015	0.25 ‰
	Manggahan Floodway (Taytay)	January 2013	0.27 ‰	February 2013	0.29 ‰	March 2013	0.28 ‰
		April 2013	0.28 ‰	May 2013	0.19 ‰	June 2013	0.24 ‰
		July 2013	0.22 ‰	August 2013	0.16 ‰	September 2013	0.24 ‰
		October 2013	0.15 ‰	November 2013	0.11 ‰	December 2013	0.24 ‰
		January 2014	0.28 ‰	April 2014	0.3 ‰		
		October 2015	0.16 ‰	November 2015	0.27 ‰	December 2015	0.31 ‰
Laguna Lake	Stn. V Northern West Bay	January 2013	0.11 ‰	February 2013	0.1 ‰	March 2013	0.1 ‰
		April 2013	0.11 ‰	May 2013	0.17 ‰	June 2013	**0.85 ‰**
		July 2013	0.2 ‰	August 2013	0.17 ‰	September 2013	0.12 ‰
		October 2013	0.1 ‰	November 2013	0.13 ‰	December 2013	0.13 ‰
		January 2014	0.13 ‰	April 2014	0.1 ‰		
		October 2015	0.28 ‰	November 2015	**0.51 ‰**	December 2015	0.43 ‰

: Brackish Water (Salinity is 0.5‰ to 30‰)

Note: Prepared by the Study Team based on the data in the Annual Water Quality Report (ELRD, LLDA)

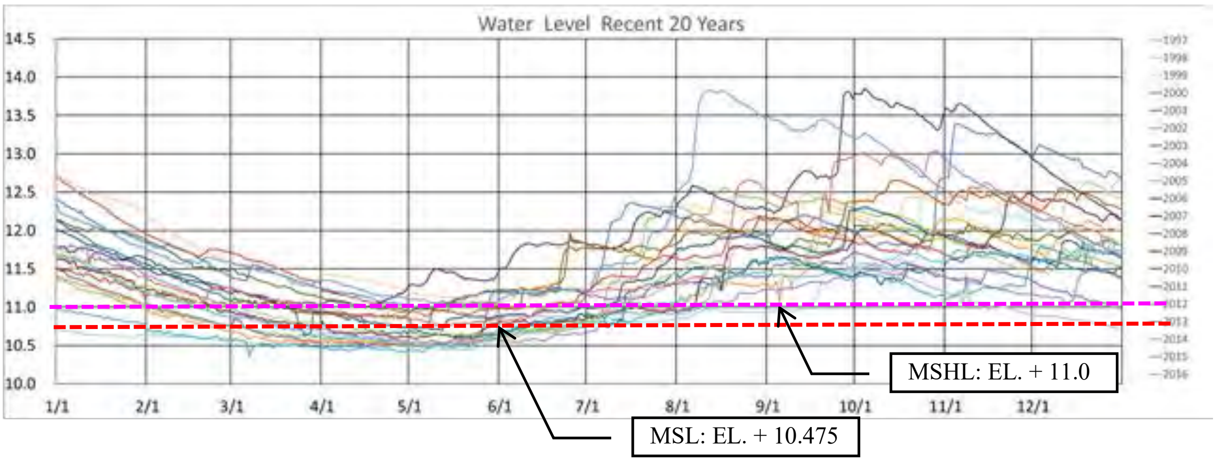


Note: Added on the Annual Water Quality Report (ELRD, LLDA) by the Study Team

Figure 6.4.37 Water Sampling Locations

(ii) Past Observed Water Level

Figure 6.4.38 shows the water level in Laguna Lake for the past 20 years. From the Mean Sea Level EL + 10.5 m, Mean Sea High Water Level EL + 11.0 m, and Mean Lower Low Water Level EL + 10.0 m in Manila Bay, it can be estimated that backflow (saltwater intrusion) from Manila Bay to Laguna Lake occurs during February and June (mainly in the dry season) when Lake Laguna is low.



Note: Prepared by the Study Team based on data provided by LLDA.

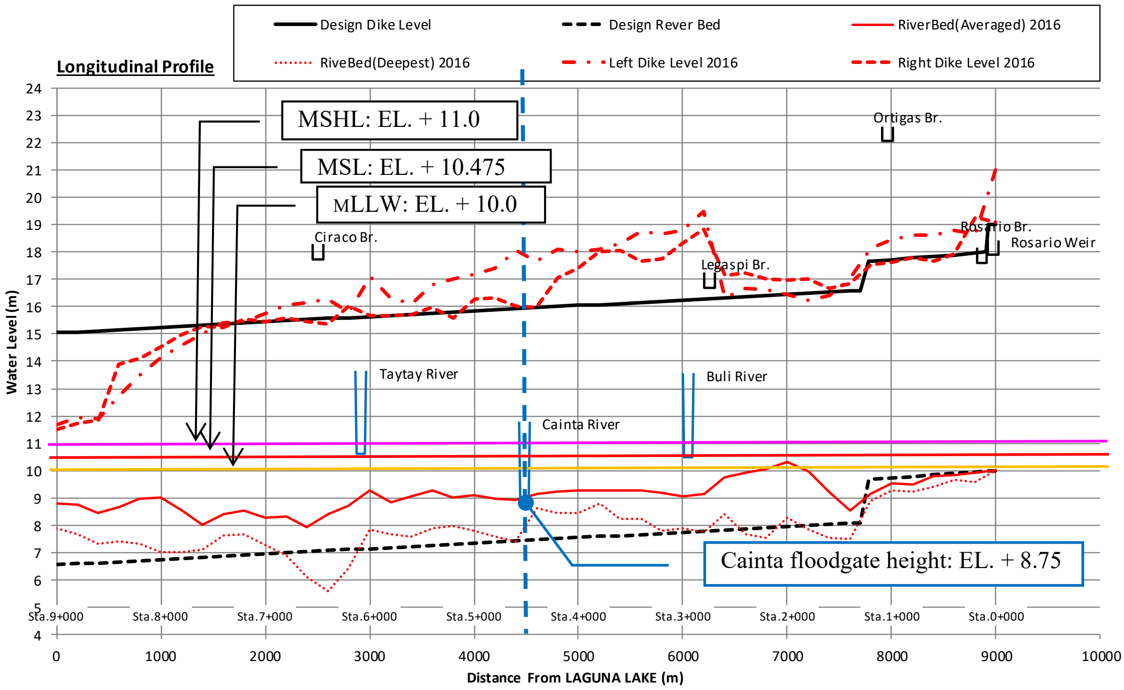
Figure 6.4.38 Water Level Data of Laguna Lake in the Past 20 Years

(iii) Relationship between Riverbed and Sea Level

Figure 6.4.30 shows the riverbed height and mean spring tide level (MSHL), mean tide level (MSL) and mean low tide level (MLLW) of Manila Bay.

The current Manggahan floodway has been filled with sediments since its completion. Once the MCGS is in place and the design flow distribution is ensured, the Manggahan floodway should be periodically excavated to maintain the flow capacity. The design bed height of the Manggahan floodway and the bed height of the Cainta floodgate are lower than the average low tide level (MLLW) of Manila Bay. Therefore, if the bed height of the Manggahan floodway is maintained appropriately, salinity intrusion may occur.

Accordingly, since the riverbed around the Cainta Floodgate location is lower than the Mean Lower low Water Level (MLLW), it is expected that the condition of brackish water will be more serious than the current condition by maintaining the riverbed of the Floodway.



Source: Study Team

Figure 6.4.39 Relation between Riverbed and Sea Water Level

(iv) Summary

The situation described above can be summarized as follows:

- At the Cainta Floodgate site, the results of water quality tests conducted with samples around the water surface cannot confirm the data indicating brackish water. Laguna Lake, on the other hand, is brackish once every few years. The comparison of salinity levels between Laguna Lake and the Marikina River suggests that the condition of brackish water of Laguna Lake is less severe than that of the Marikina River.
- After installation of the MCGS and when the Floodway bed is maintained by regular dredging, salt intrusion can be expected, considering relation among the Mean Lower Low Water Level in Manila Bay (MLLW), the height of the riverbed, and the invert of the Cainta Floodgate.

Basically, the Cainta Floodgate is to be closed only when flood flows in the Manggahan Floodway. On the other hand, in the case of MCGS, it is assumed that the gates on one side are fully closed for a certain period in the rainy season. Therefore, it can be said that it is hardly affected by brackish water. Therefore, when selecting the gate material for the Cainta Floodgate, installing the gate in the freshwater area is considered.

(b) Comparative Study of Gate Materials

The gate material is selected from the following three materials. Based on the comparison, stainless steel is applicable in freshwater.

1. Carbon Steel (Rolled steel for welded structures) (SM 400)
2. Conventional stainless steel (SUS 304)
3. Alloy saving duplex stainless steel (SUS 821 L)

The comparison of economic efficiency is based on the lifecycle costs (LCC) including maintenance. Also, regarding the initial cost, the design cost by a Japanese company is considered for all materials. In addition, regarding stainless steel, gate fabrication at a factory in the Philippines will be carried out under the supervision of a Japanese company

Based on the comparison in **Table 6.4.29**, alloy saving duplex stainless steel is recommended, since it is the most economical in terms of lifecycle cost compared to the other alternatives.

Table 6.4.29 Comparison of Materials for the Cainta Floodgate

Item	Carbon Steel(Rolled steel for welded structures) (SM400)	Conventional Stainless Steel (SUS304)	Alloy-Saving Stainless Steel (SUS821L)
Summary	· Most commonly used ordinary steel for door bodies · Proof Stress: 245 N/mm² · Tensile strength: 400 ~ 510 N/mm²	· Stainless steel commonly used in brackish water · Proof Stress: 205 N/mm² · Tensile strength: 500 N/mm²	· Stainless steel commonly used in brackish water · Proof Stress: 400 N/mm² · Tensile strength: 600 N/mm²
Weight	Gate Leaf 135 t (1.0) Guide Frame 30 t (1.0) +Hoist 55 t (1.0) 220 t (1.0)	Gate Leaf 125 t (0.93) Guide Frame 30 t (1.0) +Hoist 45 t (0.82) 200 t (0.91) · Since no corrosion margin thickness is required, it is lighter than carbon steel.	Gate Leaf 115 t (0.85) Guide Frame 30 t (1.0) +Hoist 40 t (0.82) 185 t (0.84) · Due to no corrosion margin and high strength, it is lighter than the conventional stainless steel.
Maintenance	· Generally, repainting is required at least once every 10 years, however, it is considered that repainting is carried out every year based on the record of the Rosario Weir. · Painting cost: 160,000 PHP/batch (Calculated from actual results of Rosario Weir)	· Repainting is not needed	· Same as the left
LCC (For 50 Years, Thousand PHP)	Gate Leaf ***** Guide Frame ***** Hoist ***** + Repainting ***** Total ***** (1.0)	Gate Leaf ***** Guide Frame ***** Hoist ***** + Repainting 0 Total ***** (1.03)	Gate Leaf ***** Guide Frame ***** Hoist ***** + Repainting 0 Total ***** (0.95)
Technical Novelty	· Gate fabrication using carbon steel has also been carried out in the Philippines, and there is little technological novelty.	· Stainless steel processing is also carried out in the Philippines, however, there is no experience on its application to large gates. Hence, it is a new technology for river structures	· Same as the left. In addition to the new technology for river structures, the processing of new materials in the Philippines is beneficial for the promotion of industrial production technology in the Philippines.
Evaluation	Lifecycle cost (LCC) is higher than alloy saving duplex stainless steel.	LCC is highest	It has the lowest lifecycle cost and can save on maintenance. It is also beneficial in terms of technical novelty. **Recommended**

Note: Cost is not presented due to the prior released version.
Source: Study Team

3) Water sealing System

(a) Water sealing System

Since this gate is Floodgates to prevent backflow, the rear three-side water sealing system is water sealing.

(b) Water sealing Form

A rubber water sealing type with excellent water tightness and traceability that is commonly used for river gates

(c) Water sealing Rubber Shape

L-type shall be used for the side and upper and flat bottom parts, which are generally used in three-side water sealing system construction.

(d) Water sealing Rubber Material

Synthetic rubber with excellent weather resistance and oil resistance is generally used in Floodgates.

(2) Study on Type of Hoist

1) Primary Comparison (Type of Hoist)

The types of hoist commonly used is shown below.

Transmission System	Major Equipment of the Drive Unit	Connection with Gate Leaf	Type of Hoist
Mechanical	Drum Drive	Wire Rope	Wire Rope Winch Type
	Spindle Drive	Spindle	Spindle Type
	Rack Drive	Rack Bar	Rack Type
	Chain Drive	Chain	Chain Type
Hydraulic	Hydraulic Cylinder	Piston Rod	Hydraulic Cylinder Type
	Hydraulic Cylinder	Wire Rope	Hydraulic Cylinder Wire Rope Type
	Hydraulic Motor Drum Dive	Wire Rope	Hydraulic Motor Wire Rope Type
	Hydraulic Motor Rack Dive	Rack Bar	Hydraulic Motor Rack Wire Rope Type

Note: Translated by the Study Team from the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

Figure 6.4.40 Types of Hoist

This figure is classified by the combination of the power transmission system to the drive unit, the major equipment of the drive unit, and the connection with the gate leaf.

This facility is for backflow prevention and disaster management. It is desirable to equip a self-weight lowering function without power in order to cope with power interruption that are likely to occur in disasters. The adaptability to remote control and reserve power is also important.

Considering the above, a type satisfying the functional characteristics, economic efficiency, maintainability, landscape, etc., the hoist is selected by comprehensively evaluating the head, space for installation, environment, etc.

The gate belongs to the classification of large gates with a clear span of 16.00 m x effective height of 7.31 m, a gate leaf area of 116.91 m^2, and a gate leaf weight of about 11585 t. In case of such gate facility, the following two items are generally selected:

- Wire Rope Winch Type
- Chain Type

These two types can cope with the hoisting load of about 1000 kN which is required for this Floodgate and have a lot of past cases and high reliability for closing operation.

Therefore, for the hoist type in this facility, a comparative study on the above two types is to be conducted.

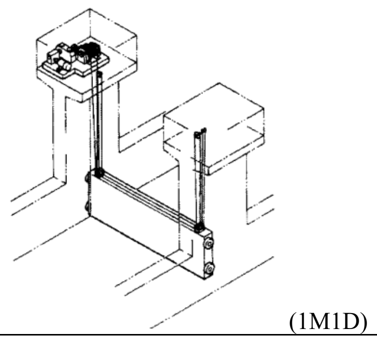
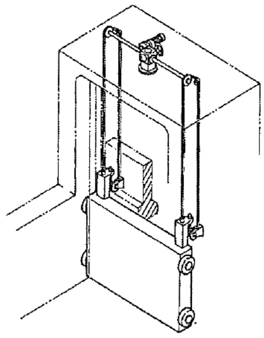
The major reasons for excluding the other 6 types from the alternatives are as follows:

1. The spindle type cannot by lowered by its own weight due to the arrangement of reduction gear. This type is used mainly for small gates with small hoisting loads, and for equipment to adjust flow rate frequently. It is not applicable to a large gate such as this facility.
2. The rack type can lower by its own weight. The hoisting load is applied up to about 500 kN. Since the hoisting load of this floodgate is 1000 kN, it is not applicable.
3. Although the hydraulic hoist can make the hoist compact, a separate hydraulic unit is required, and the installation space becomes larger. Also, it is not often used in river gate facilities in Japan due to the concern about hydraulic oil leakage, the necessity of replacing a large amount of oil regularly, and the need for maintenance. Therefore, it is not adopted in this facility.

Table 6.4.30 shows a comparison of the above two types. The chain type is selected as the hoist type suitable for this facility. The reasons are as follows:

1. In the Philippines, this type is often used.
2. The self-weight lowering function is possible, and self-weight lowering by remote control is also possible.
3. In the present span, an interlocking shaft and an operation bridge is necessary for the chain type, but the wire rope type is not necessary
4. It is a simple structure and maintenance and management are easy.

Table 6.4.30 Comparison of Hoists (Cainta Floodgate)

	Mechanical	
	Wire Rope Winch Type	Chain Type
Figure (Extracted from "Floodgate Engineering")	 (1M1D)	
General Description	・ It consists of a wire rope and a winch type hoist, and the hoist force generated by the winch is transmitted to the wire rope to raise and lower the gate leaf.	・ It consists of a chain and a hoist, and hoisting force generated in the hoist body is transmitted to the chain to raise and lower the gate leaf

	Mechanical	
	Wire Rope Winch Type	Chain Type
Major Equipment	· Wire Rope Winch Type Hoist · Main Motor, Decelerator, Drum, Manual Operation Device, Spare Engine, Switching Device, Hydraulic Push-up Brake, Self-weight Lowering System (with Fan Brake), Wire Rope, Wire Sheave	· Chain Type Hoist · Motor, Decelerator, Regulator, Manual Operation Device, Opening Meter, Integrated Limit Switch, Chain, Sprocket, Interlocking shaft and operation bridge
Installation Space	· Since the wire winding drum and the decelerator are placed on the deck, the required installation space is increased; however, there is no projection upward, and the installation height can be suppressed.	· Since the integrated hoist and the chain sprocket (transmission gear) are smaller than the wire drum, the required space is small, there is no upward projection, and the installation height can be suppressed. However, since a gear axis is required, a space for that portion is required.
	○	○
Influence on the Substructure Works	· The only dead weight of the gate leaf is the closing force (compressive stress).. · Since the hoisting load is transmitted to the substructure on the bottom surface of the operation deck, a large-scaled reinforcement of substructure is unnecessary. · The weight of the hoist is slightly heavier than that of the chain type for the drum and the frame.	· The only dead weight of the gate leaf is the cofferdam force (compressive stress). · Since the hoisting load is transmitted to the substructure on the bottom surface of the operation deck, a large-scaled reinforcement of substructure is unnecessary. · The weight of the hoist itself is light · The weight of the operation bridge acts on the substructure.
	○	○
Response to Earthquakes	· Since the center of gravity of the hoist is low, it is enough to have a bearing part (fixed part) which can withstand inertial force in the earthquake condition. · Operation is possible, since there are many examples of the tsunami Floodgate.	· Since the center of gravity of the hoist is low, it is enough to have a bearing part (fixed part) that can withstand inertial force in the earthquake condition.
	○	○
Reserve Power	· The motor is the standard main power. · Manual operation handle is normally installed; however, it is not practical. Hence, a spare engine is also installed. · Correspond to a spare generator	· The motor is the standard main power. · Manual operation handle is normally installed; however, it is not practicable. · Correspond to a spare generator
	◎	○
Self-Weight Lowering	· The self-weight lowering device can be an external device such as a fan brake. · The speed of self-weight lowering can be adjusted to 4.0 m/min or less.	· The self-weight lowering device is incorporated in the main body of the switch and is possible. · The speed of self-weight lowering can be adjusted to 4.0 m/min or less.
	○	◎
Closing Force	· The closing force depends only on the weight of the gate leaf. · When the weight of the gate leaf is light, it is necessary to add weight to it, however, it is not necessary in this scale.	· Same as the left
	○	○
Landscape	· The mechanical portion is not exposed to the outside, since it can be placed in the operating room. · The position where the hoist is installed can be set at a place other than the slab by extending the wire rope. · The wire rope is exposed.	· The mechanical portion is not exposed to the outside, since it can be placed in the operating room. · The chain is exposed · Interlocking shaft and operation bridge are exposed.
	○	○

	Mechanical	
	Wire Rope Winch Type	Chain Type
Maintenance	・ The number of machines is the largest among alternatives. However, since the structure is simple, maintenance is relatively easy. ・ Periodic replacement of lubricating oil is necessary; however, not as much as the hydraulic system. ・ It is necessary to apply grease to the wire rope on a regular basis. ・ It is necessary to replace the wire rope about once every 15 years.	・ Since the hoist is standardized, maintenance is easy, though, technical skill is required. ・ It is necessary to change the lubricating oil for gears in the hoist periodically, however the amount is less than it of the other types. ・ Lubricating oil is required for sprockets provided on the gate leaf side. ・ Regular application of grease to the chain is required; however, replacement of the chain is unnecessary.
	○	△
Actual Examples	・ There are many examples in this scale.	・ This is a relatively new type, hence, there are not so many examples (released in 1997).
	◎	△
Production Cost	1.0	about 1.1
	◎	◎
Evaluation	The facility is equipped with the necessary functions and can be easily adapted to landscape design. The maintainability is inferior to the chain type. Many examples and reliable.	The facility is equipped with the necessary functions and can be easily adapted to landscape design. Excellent maintainability. Similar economics. Not many examples.
	Recommended	Applicable

Legend : ◎...Better、○...No Problem、△...There are issues to be solved.
Source: Study Team

2) Secondary Comparison (Wire-Rope Winch Type)

An overview of the structure of each type of wire rope winch shown in **Table 6.4.31** and

Table 6.4.32.

The following three types of wiring are considered for the wire rope type winch.

- 1 motor 2 drum winch type (1M2D)
- 1 motor 1 drum winch type (1M1D)
- 2 motor 2 drum winch type (2M2D)

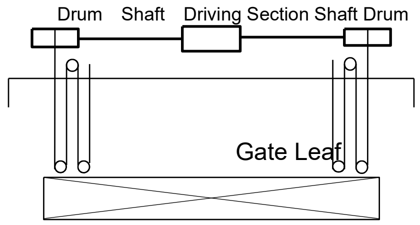
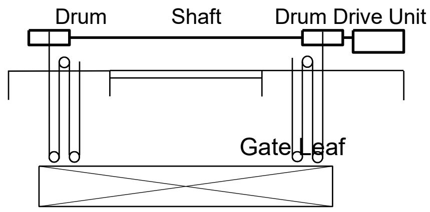
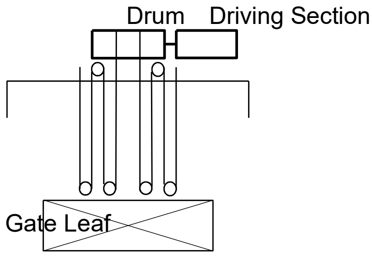
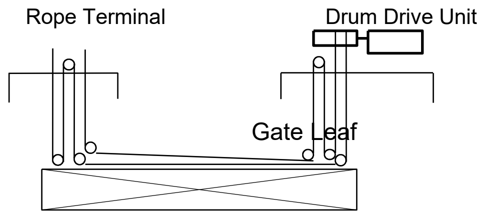
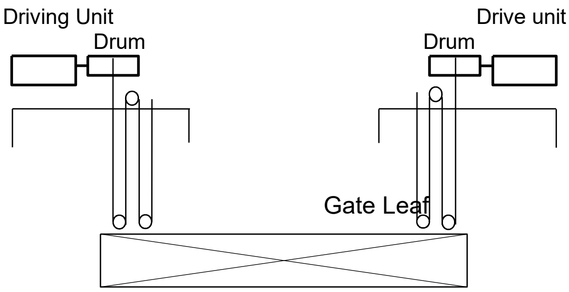
Further, in consideration of the arrangement of a driving unit (motor, reduction gears and other power generators) and a driven unit (Drums, gears, wire ropes, etc.) of a wire rope winch type motor or a drum and the components are classified as follows.

Table 6.4.31 Wire Rope Winch Types and Placement

	Arrangement of Drum And Driving Part	Position of Drum and Drive Unit
1 Motor, 2 Drum Type (1M2D)	Center Drive Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
	Single Drive Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
1 Motor, 1 Drum Type (1M2D)	Central Drum Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
	One Side Drum Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column
2 Motor, 2 Drum Type (2M2D)	Double Drum Type	Direct Rope Hoisting, Drum and Drive Unit Placed on the Top of Column

Source: Study Team

Table 6.4.32 Structures of Wire Rope Winch Type

Wire Rope Winch Type	Schematic Structural Plan	
<u>1 Motor, 2 Drum Type</u> This type is the most common type in which a drum is arranged at both ends and connected to a power transmission shaft to wind up wire ropes on the left and right sides of a gate leaf. There are two types of drive units, the center drive type and the single drive type. The center drive type is common, but the single drive type is adopted in the independent type of column when the hoisting devices are concentrated on the one side of the columns.	Center Drive Type	
	Single Drive Type	
<u>1 Motor, 1 Drum Type</u> It is a type in which the components of a hoisting device are integrated in one location, and there are a central drum type in which a drum is arranged in the center of the gate leaf, and a one side drum type in which a drum is arranged on one side of the gate leaf. The center drum type is adopted when the lifting span of the gate leaf is narrow (Approx. 10 m or less), but rack type and spindle type are mostly adopted in such scale. In addition, the one side drum type has the advantage that the stage for hoisting device between the columns is not required and the hoisting device can be concentrated on the one-side, and is often used for gates with a span of about 10 m to 25 m.	Central Drum Type	
	One Side Drum Type	
<u>2 Motor, 2 Drum Type</u> Generally, this type is adopted for gates (& weirs) with a span of about 25 m or more, and one motor and one drum type hoisting deice is arranged at both ends of the gate leaf, and the gate leaf is opened and closed with electrically adjusting the opening. Since it is difficult to synchronize mechanically, the risk of lifting on only one-side is higher than other types. Therefore, direct connection of an internal combustion engine or installation of a self-weight closing device shall be avoided. It is more expensive than other types.	Double Drum Type	

Source: study team

Since the reliability of complete closure is the most important function of this gate, the type of wire rope winch is also required to have a simple mechanism and high reliability in opening and closing operations.

Of the three types, the 2M2D wire-rope winch type is usually adopted when a span is 25 m or more and the cost is also high.

About 1M2D wire rope winch type, since the operation deck is an independent type in the left and right, the single drive type would be adopted.

As for the 1M1D wire rope winch type, in case the operation decks are independent, it is structurally difficult to adopt the central drum type. Hence the one side type is chosen.

Considering these factors, the following two types of winch can be adopted.

1. 1M2D Single Drive
2. 1M1D One Side Drum Type

Table 6.4.33 List of Wire Rope Winch Type Hoist

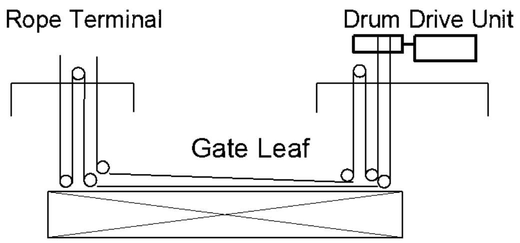
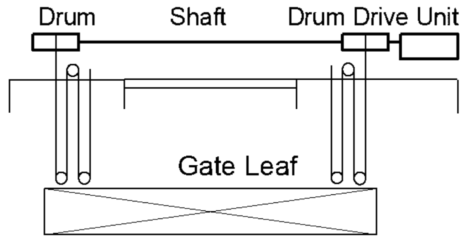
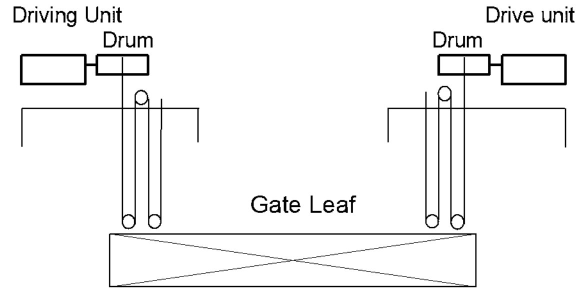
Format	Drum and Placement of Drive	Applicability	
1M2D	Center Drive Type	・ Commonly Used Type ・ Self-Weight Closing is possible. ・ Lifting on only one side is the least likely to occur, and reliability of Self-weight closing is the highest of the three types.	Inapplicable
	Single Drive Type	・ It is commonly adopted in floodgates with a large span and independent column. ・ Self-Weight Closing is Possible. ・ Lifting on only one side is the least likely to occur, and reliability of Self-weight closing is the highest of the three types.	○ One of The Alternatives
1M1D	Central Drum Type	・ Not used in large gates like the case of this study ・ Self-weight closing is possible. ・ Structurally inapplicable to gates with large spans	Inapplicable
	One-Sid Drum Type	・ Adopted in large gates ・ Self-weight closing is possible. ・ As for securing side-to-side tuning by roping the wires, uncertainty remains in self-weight closing when compared with the mechanical connection method (1M2D Format), and rope adjustment, etc. is difficult and maintenance is inferior.	○ One of The Alternatives
2M2D	Double Drum Type	・ Adopted in large gates ・ Self-weight closing can be performed with a mechanism to synchronize both drums.	Inapplicable (Reference)

Source: Study Team

In the next page, a comparison table of the two types (Also including 2M2D for reference).

Since operation decks are the independent, of the two types, the 1M2D wire rope winch type requires a conduction shaft and the aspect of installation is inferior. Furthermore, the cost becomes also higher than the 1M1D type. Hence, 1M1D type is recommended.

Table 6.4.34 Comparison of Wire Rope Winch Type Hoist

Type / Item	A 1 Motor 1 Drum (1M1D)	B One Side Drive Type 1 Motor 2 Drum (1M2D)	<Reference>: C 2 Motor 2 Drum (2M2D)
outline (positive surface)			
General of Structure	・ Motor is placed on one side ・ Drums and gears are all located on one side ・ One motor and one reduction gear each are installed. (motor output per unit is twice that of “C”.) ・ A wire rope passes through the gate leaf. ・ The smallest number of parts (1/2 times C) ・ Large number of sheaves (More than two ropes than B and C, regardless of the number of ropes)	・ Motor is placed on one side ・ Drums and gears are arranged on both sides. ・ One motor and one reduction gear each are installed. (motor output per unit is twice that of “C”.) ・ The drive transmission shaft crosses between the right and left drums. ・ The transmission shaft must be stiff and durable. ・ The number of parts is between A and C. ・ The number of sheaves used is small.	・ Motor is placed on both sides ・ Drums and gears are located on both sides ・ Motor, reduction gears, etc., all need two. (Output per motor is 1/2 times that of A and B.) ・ No drive transmission shaft is required. ・ It has the largest number of parts. (double A) ・ The number of sheaves used is small.
Operation	・ Slight tendency to become one side lifting ・ No left-right tuning mechanism required ・ Self-weight Closing can be performed.	・ There is no concern of one side lifting. ・ No left-right tuning mechanism required ・ Self-weight closing can be performed.	・ Right and left tuning mechanisms are necessary because there is a tendency of one side lifting due to the uneven rotation speed of the right and left motor. ・ Self-weight closing needs to synchronize left and right with an electric device.
	△	○	△
Installation and Delivery	・ Superior regardless of the gate size due to the smaller number of parts ・ As compared with B and C, parts need to be concentrated on one side, resulting in large blocks, and the necessity of division arises even in relatively small-capacity models.	・ The number of parts is larger due to the intermediate shaft. ・ When the span is long, the transmission shaft becomes long, and it is difficult to install.	・ The number of parts is almost the same as A, and it is advantageous regardless of the size of the span.
	○	△	○
Influence on the Concrete Structure	・ Since the wire rope passes through the inside of the door, a lateral load does not act on the column. ・ One side of the hoist becomes a large block and the operation room becomes large.	・ Since the wire ropes are equally arranged on each side, a lateral load does not act on the Column. ・ Operation deck connecting columns or operation bridge between independent columns is required	・ Since the wire ropes are equally arranged on each side, a lateral load does not act on the Column. ・ Since the left and right blocks are the same, the operation room can be arranged on the same scale.
	△	△	○
Maintenance	・ Since the drive unit is placed on the one side and the numbers of parts are smaller, maintenance is easier. ・ The wire rope in longer and the length passing inside the gate leaf is the longest, it is inferior in the aspect of the maintenance of wire rope.	・ Since the drums and gears are located on both side, maintenance need more manpower and cost due to the larger number of parts. ・ The wire rope can be inspected more easily, it is superior to “A” in the aspect of the maintenance of wire rope.	・ Since driving units are located on both side, maintenance need more manpower and cost due to the larger number of parts ・ The wire rope can be inspected more easily, it is superior to “A” in the aspect of the maintenance of wire rope.
	△	○	○
Landscape Design	・ There is nothing in the span that obstructs the view such as the operation bridge and the operation room, and it is excellent.	・ The operation bridge, the operation room, and other objects in the span obstruct the view.	・ There is nothing in the span that obstructs the view such as the operation bridge and the operation room, and it is excellent.
	○	△	○
Machine Weight	・ The weight of the hoist is the lightest.	・ The weight is increased by the transmission shaft and the operation bridge. Therefore, the larger the span is, the more disadvantageous it is.	・ The steel is heavy. (Increase approximately 20% from A)
	○	△	△
Cost	95%	100%	130%
	1	**2**	**3**
Evaluation	Although it is difficult to maintain wire ropes, this type would be adopted because it is used in many large gates and it does not require a transmission shaft and is inexpensive.	There is no concern of one side lifting, and maintenance of the wire rope is easy, However, the cost is higher than A. It is difficult to install the transmission shaft, it is not adopted.	This type is generally adopted for gates with a span length of 25 m or more. Maintenance and management of wire ropes is easy.
	1	2	**3**

Legend：◎…Preferable、○…No Problem、△…With Issues to Be Solved、×…Difficult to Adopt

Source: Study Team

6.4.3.9 System Planning

(1) Power Unit

1) Main Power Unit

There are electric motors, internal combustion engines, and manpower for opening and closing gates. Since this gate is an important facility for preventing disasters, the main power is an electric motor in consideration of the starting reliability, opening stability and closing speed, low failure rate, ease of maintenance and remote operability.

The motor type is a special squirrel-cage three-phase induction motor for Floodgates.

2) Reserve Power Unit

As a measure against failure of motors which are the main power unit of hoist, reserve power unit shall be provided to ensure necessary functions as important equipment for disaster prevention.

The reserve power for opening and closing includes an electric motor (reserve), an internal combustion engine, and human power.

(a) Motor (Reserve)

Usually, a system using an electric motor is adopted due to reliability and easier operation. This system includes the following systems:

1. Always with a spare motor (Fixed installation of spare electric motor)
2. Storage of spare motor (the same type of motor as the traction motor is stored as replacement when the traction motor fails.)

It is simple to operate the equipment with a spare motor; however, the equipment becomes larger. Storage of spare electric motor requires time for replacing the motor, but the installation of a special device as the spare power is not required and the electric motor does not affect the space for installing the hoist.

(b) Internal Combustion Engine

When it is difficult to secure a backup power supply, an internal combustion engine may be employed. In this case, a switch between the electric motor and the internal combustion engine is required, the number of parts increases, and the size of the internal combustion engine itself also influences the size of the equipment. In addition, it takes time and labor for refueling and maintenance.

(c) Manpower

- ✓ This gate has a span length of more than 16m (door body area is more than 200m^2), and a 1M1D2M2D wire-rope-winch type can be applied as the switchgear. Therefore, the closing operation at the time of failure of the drive motor or at the time of the commercial power failure becomes impossible. Thus, the necessity for spare of the switch drive is low.
- ✓ This gate is very large (Gate leaf area: 200 m^2 or more), and 2M2D wire rope winch type is employed for the hoist. Hence, it is difficult to apply a self-weight lowering device. Besides, when the drive motor or power unit fails or the commercial power supply stops, the closing operation becomes impossible. Therefore, a spare switch driver is highly needed.
- ✓ The wire rope winch type hoist can easily have a mechanism in which a spare engine can be incorporated. However, since two sets of reducers are required, installation space and manufacturing cost increase.
- ✓ If an electric motor is ordered after the failure of the installed electric motor, the delivery time will be about two months and thus take time to resume operation.
- ✓ Therefore, while emphasizing flood control safety, "storing a spare motor" is advantageous considering cost reduction.

(2) Power Supply Unit

1) Main Power Supply Unit

The commercial power supply is received in 3φ3 W AC 200V 60Hz and 1φ2 W AC 200V 60Hz directly at the control panel in the operation room and used as the operation power.

2) Standby Power Supply System

Since this is an important gate facility, a standby generator will be permanently installed as a backup power for the power unit to ensure the reliability of gate opening and closing.

When the commercial power stops, the power necessary to open and close the gate and the power supply for the incidental water-level gauge, safety equipment, remote control equipment, building equipment, etc., could be secured through the standby power generation unit.

(3) Control System

1) Local Control Panel

(a) Usage

The local control panel is installed on the operation deck for periodical maintenance and normal operation. The specific control function of each equipment is included in the local control panel. The operation is performed by push-button operation from the local control panel and remote operation from the central control station.

(b) Operation Monitoring Item

For gate operation by hoist, in order to ensure safe operation, the local control panel shall be equipped with various monitoring signals and fault indicators as shown in **Table 6.4.35**.

Table 6.4.35 Operation Items and Control Signals

Item	Local Control Panel (Operation)	Control Signal		Remarks
		Presence or Absence of Signal	Signal Format	
Open (Rise) Operation	○	○	Continuous Output	Gate Open Command
Close (Descent) Operation	○	○	Continuous Output	Gate Closing Command
Stop	○	–		Gate Stop Command
Alarm Stop	○	-		
Lamp Test	○	-		
Emergency Stop	○	○	Pulse Signal	Gate Emergency Stop Command
Fault Reset	○	-		

Source: Study Team

The status of gate is displayed on the local control panel and remote operation console. Operation display items and their monitoring signals are as shown in **Table 6.4.36**.

Table 6.4.36 Gate Status, Items to Display Operation and Monitoring Signals

Item	Local Control Panel (Display)	Monitoring Signal	
		Presence or Absence of a Signal	Signal Format
Power	○	○	Continuous Output
AC Control Power Supply	○	○	Continuous Output
Remote	○	○	Continuous Output
Machine Side	○	○	Continuous Output
Normal Operation	○	○	Continuous Output
Maintenance Operation	○	○	Continuous Output
Normal Circuit	○	○	Continuous Output
Emergency Circuit	○	○	Continuous Output
During Opening (Rise)	○	○	Continuous Output
During Closing (Descent)	○	○	Continuous Output
Stop	○		

Item	Local Control Panel (Display)	Monitoring Signal	
		Presence or Absence of a Signal	Signal Format
Gate Rest	○		
Full Open	○	○	Continuous Output
Full Close	○	○	Continuous Output
Hook-out Upper Limit	○	○	Continuous Output
Hook-out Lower Limit	○	○	Continuous Output
During Gate Operation	○	○	Continuous Output

Source: Study Team

Items to display gate failure on the local control panel are as shown in **Table 6.4.37**. Among the items, those which seriously affect operation are indicated as "Serious Failure".

Table 6.4.37 Items to Display Gate Failure and Monitoring Signal

Item	Local Control Panel (Display) Failure Classification		Supervisory Signal	
	Serious Failure	Light Failure	Presence or Absence of a Signal	Signal Format
3E Operation	○		○	Continuous Output
Rope Overload	○		○	Continuous Output
Rope Slack	○		○	Continuous Output
Hydraulic Brake Overload	○		○	Continuous Output
Electromagnetic Brake Overload	○		○	Continuous Output
Emergency Upper Limit	○		○	Continuous Output
Emergency Stop	○		○	Continuous Output
Contact Welding	○		○	Continuous Output
MCCB Trip	○		○	Continuous Output
Leakage		○	○	Continuous Output

Source: Study Team

(c) Panel Configuration

The local control panel consists of a power supply unit for receiving power, an operation unit for operating the gate, a control unit for controlling operation, and a display unit for monitoring the operation status. As to the arrangement of equipment on and inside the board, equipment necessary for gate operation and monitoring shall be arranged on the machine side of the operation board, and equipment not necessary for gate operation and monitoring shall be arranged in the board for easier and precise operation, inspection and maintenance.

(d) Control Circuit

Regarding the configuration of the control circuit of the local control panel, there are the method using a contactor relay circuit and the programmable controller (PLC). The advantages and disadvantages of both circuits are as shown in **Table 6.4.38**.

The relay circuit is adopted since PLC is difficult to cope with failure and there is no need to consider any addition or change in operation.

Table 6.4.38 Advantages and Disadvantages of Contact Relay Circuits and PLC Circuits

Items	Contact Relay		PLC Circuit	
Reliability	If the circuit is complicated, the number of parts increases and the occurrence of failure increases.	△	Even if the circuit becomes complicated, the failure rate does not change much.	○
	The influence of failure is almost partial. If the auxiliary relay or timer fails, it can be easily replaced, and system reliability is ensured.	○	The influence of failure is easy to spread. However, if the I/O or the power supply module fails, it can be easily replaced, and system reliability is ensured.	△
Repairability	It does not require much expertise (technique) and can be replaced by an operator if a plug-in type auxiliary relay or timer fails. (Compatible.)	○	It is easy to replace the I/O unit. However, when viewing the program in sequence, a special equipment must be handled. It is not compatible with equipment from different manufacturers.	△

Items	Contact Relay		PLC Circuit	
Circuit Configuration	The number of parts increases in proportion to the control quantity, since it is composed of an auxiliary relay and a timer, and the labor for wiring also increases if the control is complicated.	△	Since it consists of the PLC main body and I/O unit, the number of parts does not increase significantly even if the control amount increases. Even if the control becomes complicated, the processing is easy since it is dealt by software.	○
Parts Durability	Although the number of contact portion is limited, the parts are highly interchangeable and can be easily replaced.	○	Since the PLC itself is non-contact, the durability is high; however, the specifications depend on the manufacturer, and the parts are not interchangeable.	△
Environment	It is weak in humidity, etc. However, it is stronger in the operating environment than the PLC.	◎	Protective measures against electromagnetic waves such as noise and thunder are necessary.	△
Evaluation	Although durability is a little low and failure occurrence rate is high, maintenance is comparatively easier, and it has an advantage environmentally. Hence, the same system as the one currently shall be adopted.		Although the durability is high and failure occurrence rate is low, it is not adopted since maintenance of the circuit board requires specialized knowledge and security management is difficult.	
	Recommended			

Legend: ◎... Better, ○... No problem, △... There are issues to be solved.
Source: Study Team

(e) Type of Machine Side Panel

Since the local control panel is installed in the operation room, it is an indoor type and closed self-standing type with high expandability and easy maintenance and inspection.

(4) Remote Monitoring and Control System Level

1) Method For Remote Monitoring And Control And System Level

For the operating system of the Cainta Floodgate, the introduction of system level 5 (remote control) is recommended as in the MCGS in view of the following:

- ✓ It is necessary to carry out reliable operation while monitoring the conditions such as the water level of the Marikina River, operation of the MCGS and the Rosario Weir, and the inflow from the Cainta River and Taytay Creek.
- ✓ Prompt and timely operation (minimizes time required for facility operation).
- ✓ Reduces workload of operation during flood events and allows managers to operate facilities by themselves. (Operation of facilities can be performed without outsourcing to local governments, etc.)

2) Monitoring and Control System

(a) Functions of Monitoring and Control System

The remote monitoring and control system carry out the monitoring and control of equipment from a remote location (management offices, etc.), and consists of the (i) remote monitoring function; (ii) remote control function; (iii) data management function; and (iv) operation support function. The outline of each function is described below and constructed according to the scale and operation management of the system.

(b) Overall System Configuration

The overall system configuration will be integrated with the MCGS remote monitoring and control system, and systematization will be attempted by the "Distributed Web System".

(c) Configuration of Monitoring and Instrumentation Facilities

For remote monitoring and control of the Cainta Floodgate, the facilities (instrumentation, alarm and monitoring equipment) for status monitoring and video monitoring together with their monitoring functions are summarized below.

(i) Instrumentation

Used to check flooding condition and to operate facilities (Gate open/close judgment)

(ii) Alarm devices (Alarm light, sound collection microphone and speaker)

Used to inform the surrounding people of safety during remote operation (Be on alert and alert)

(iii) Monitoring Equipment

Used to visually check the status of various facilities and to check the opening and closing of gates.

In order to collect observation data and monitoring images obtained from the devices shown in **Table 6.4.39**, to operate the facilities for safety notification, and to transmit and receive remote control signals, optical cables from the EFCOS to the Cainta Floodgate will be installed by overhead or an underground line along the East Bank Road.

Table 6.4.39 Instrumentation, Alarm and Monitoring Equipment

River Name	Facility	Monitoring and Observation Equipment	Purpose	Location (Tentative)
Manggahan Floodway	Cainta Floodgate	Water Level Gauge	Used to grasp the situation of river water level and to judge the opening and closing of gates.	Upstream and downstream of Cainta Floodgate
		Siren	Used for safety notification during gate operation	Local Control House
		Warning Light		
		Speaker and Sound Collection Microphone		Local control houses and sections where alarms are required
		Camera	Used to visually check the status of various facilities, determine gate opening/closing, and check gate opening/closing.	Local Control House

Source: Study Team

6.4.3.10 Incidental facility

(1) Conditional Revetment

In the floodgate, a conditional revetment is installed to prevent the scouring of a riverbank caused by the flow alteration.

1) Area to be Protected

For the Cainta River, the design river improvement has not yet implemented. Therefore, no revetments would be proposed in the upstream of the existing revetment of the tributary except for the portion to fit to the existing revetment.

in the downstream of the tributary (Main river), a revetment is provided considering the following points.

- 10m from the end of the breast wall or wing wall
- Dike excavation width during construction

Since this facility is a floodgate and there is no area under the maintenance bridge, the installation height of the revetment must be up to DFL = 14.853.

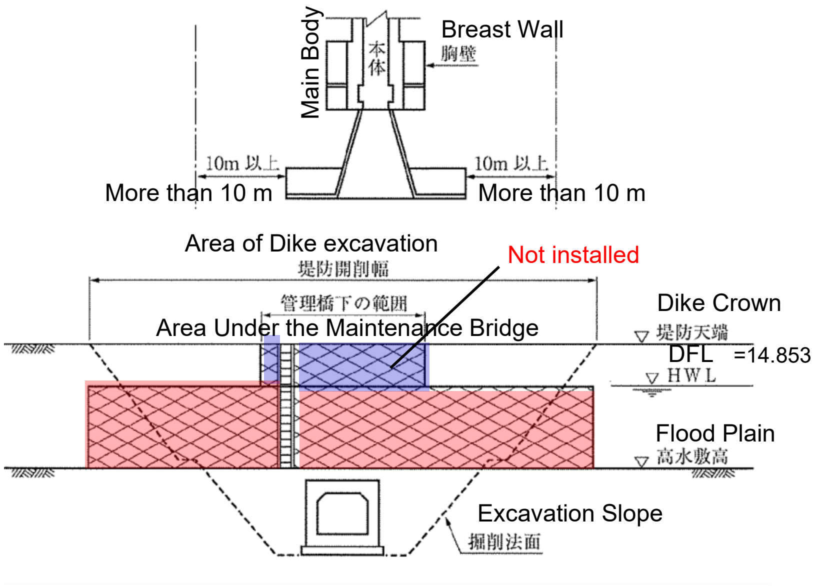
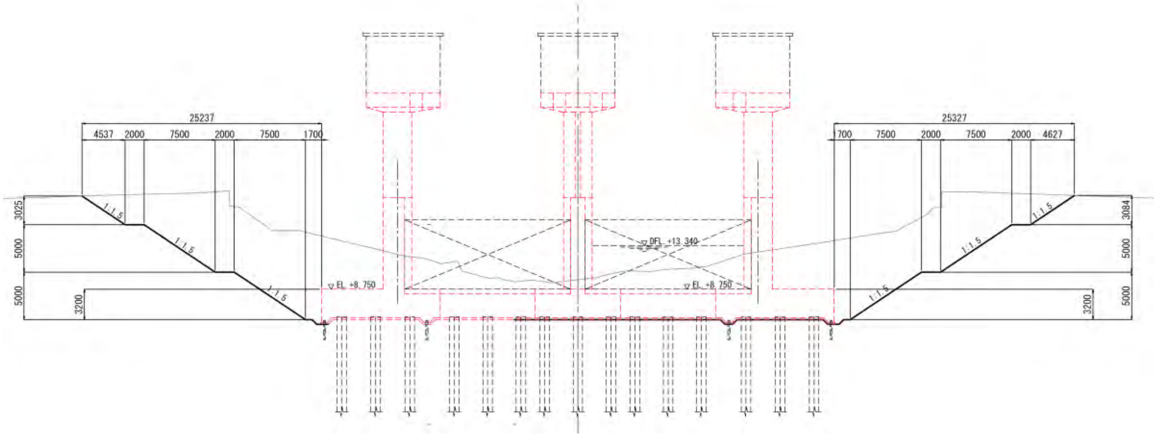


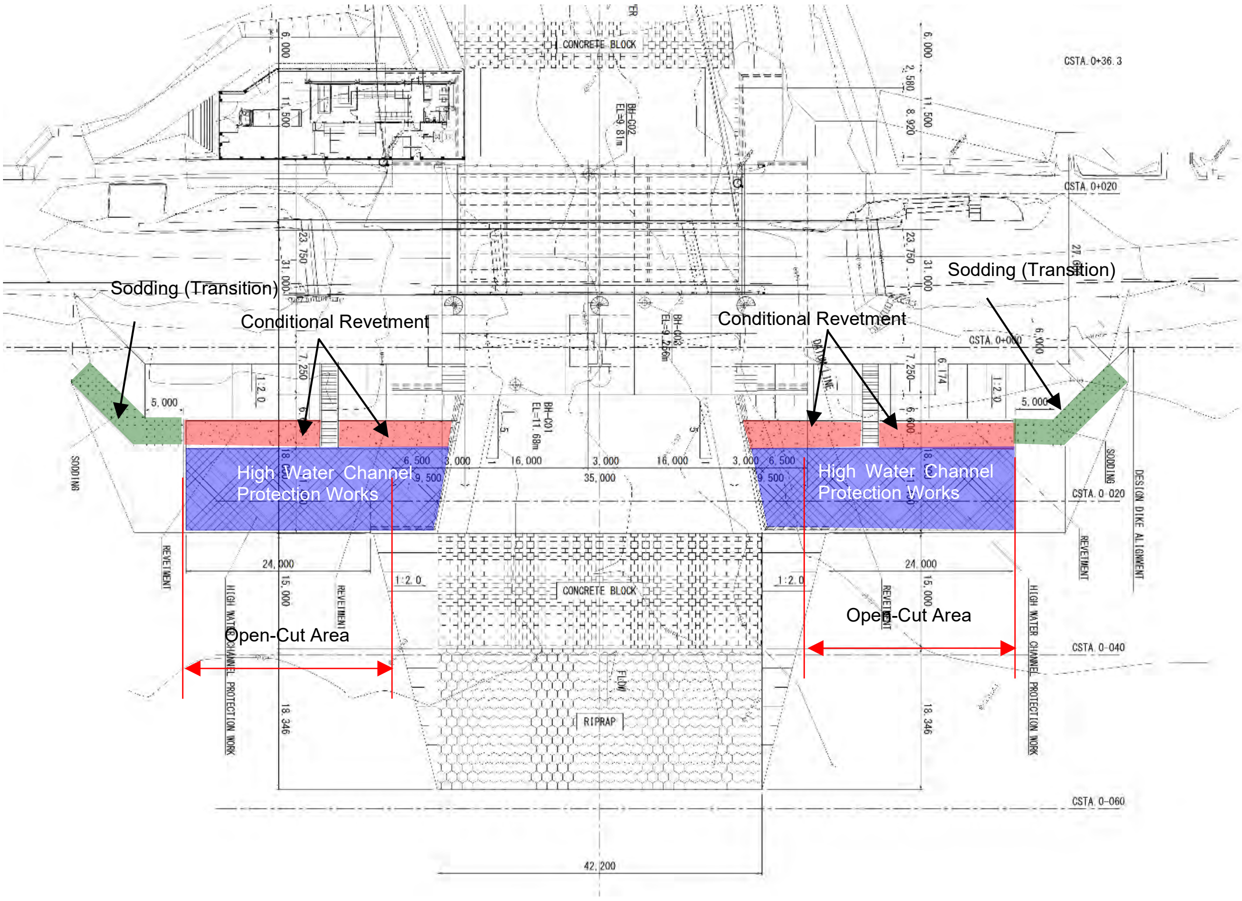
Figure 6.4.41 Area of the Bank Revetment

The excavation width is shown in **Figure 6.4.42**. From the excavation width, the range of the conditional revetment is set as shown in **Figure 6.4.43**. As for the installation of the upstream and downstream of the floodgate to the existing dike, a 5m transition section is secured and set at 45 °. The slope protection of this part is vegetation.



Source: Study Team

Figure 6.4.42 Excavation width



Source: Study Team

Figure 6.4.43 The Extent of Connecting Revetments

2) Revetment Structure

One-dimensional non-uniform flow calculations for the Manggahan floodway show that the average flow velocity at the Cainta Floodgate is between 1.6 m/s and 1.7 m/s. The slope protection is sodding from DGCS Volume 3.

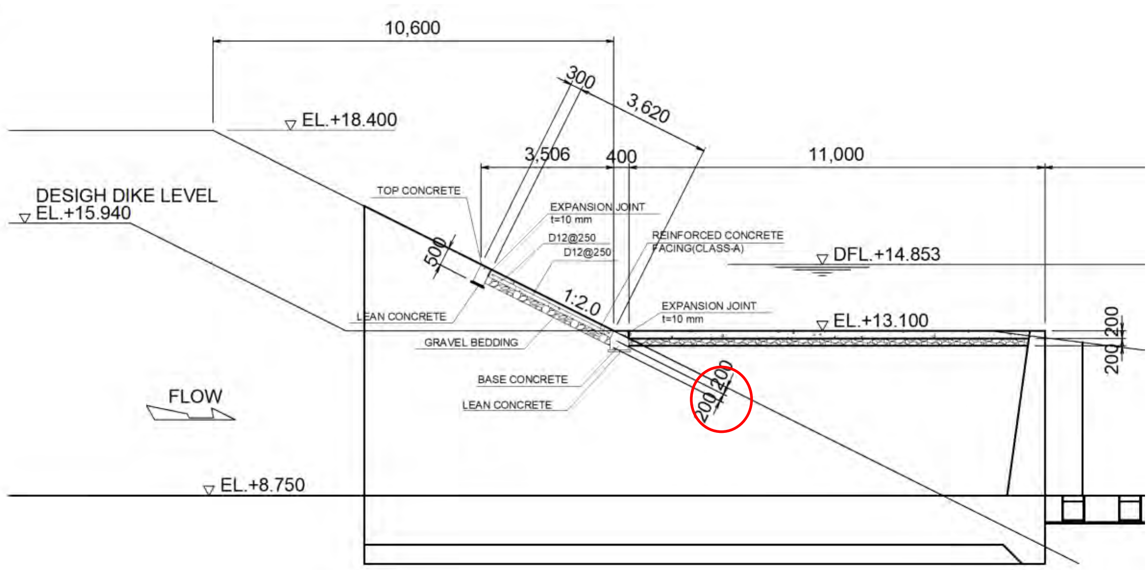
However, since the revetment around the floodgate is reinforced concrete facing would be adopted so that it does not become a weak point of the dike.

Table 6.4.40 Revetment Structure in accordance with the Flow Velocity

Table 5-5 Overview of Different Slope Protection Works & Considerations

	Indicative Maximum Design Velocity (m/s)	Slope (V/H)	Remarks
1. Sodded Riverbank with Pile Fence	2.0	Milder than 1:2	Not applicable for places near roads and houses Diameter and length of wooden pile shall be determined considering past construction records. Note that this is not a common technique used for revetments.
2. Dry Boulder Riprap	3.0 to 4.0	Milder than 1:2	Diameter of boulder shall be determined using Table 5-7. Height of generally less than 3 to 5 m.
3. Grouted Riprap (Spread Type)	5.0	Milder than 1:1.5	Use Class "A" boulders for grouted riprap and loose boulder apron.
4. Grouted Riprap (Wall Type)	5.0	1:1.5 to 1:0.5	Use class "A" boulder for grouted riprap.
5. Gabion (Mattress or Spread Type)	5.0	Milder than 1:1.5	Not advisable in rivers affected by saline water intrusion. Not applicable in rivers where diameter of boulders present is greater than 20 cm.
6. Gabion (Pile-up type) – Gabion Wall	6.5	1:1.5 to 1:0.5	Not advisable in rivers affected by saline water intrusion. Not applicable in rivers where diameter of boulders present is greater than 20 cm.
7. Rubble Concrete (spread type)		Milder than 1:1.5	
8. Rubble Concrete (Wall Type)			
9. Reinforced Concrete			Minimum thickness of 20cm
10. Gravity Wall			
11. Sheet Pile		Vertical	In cases where ordinary water level is very high
12. Vegetation and Reinforced Grass/ TRM		Milder than 1:4	Typically for the upper section of the protection, where the velocities of flow are lower. Should be located above the ordinary water level to ensure only irregular inundation. Refer to Section 5.5.3.7.

Source: DGCS Volume 3



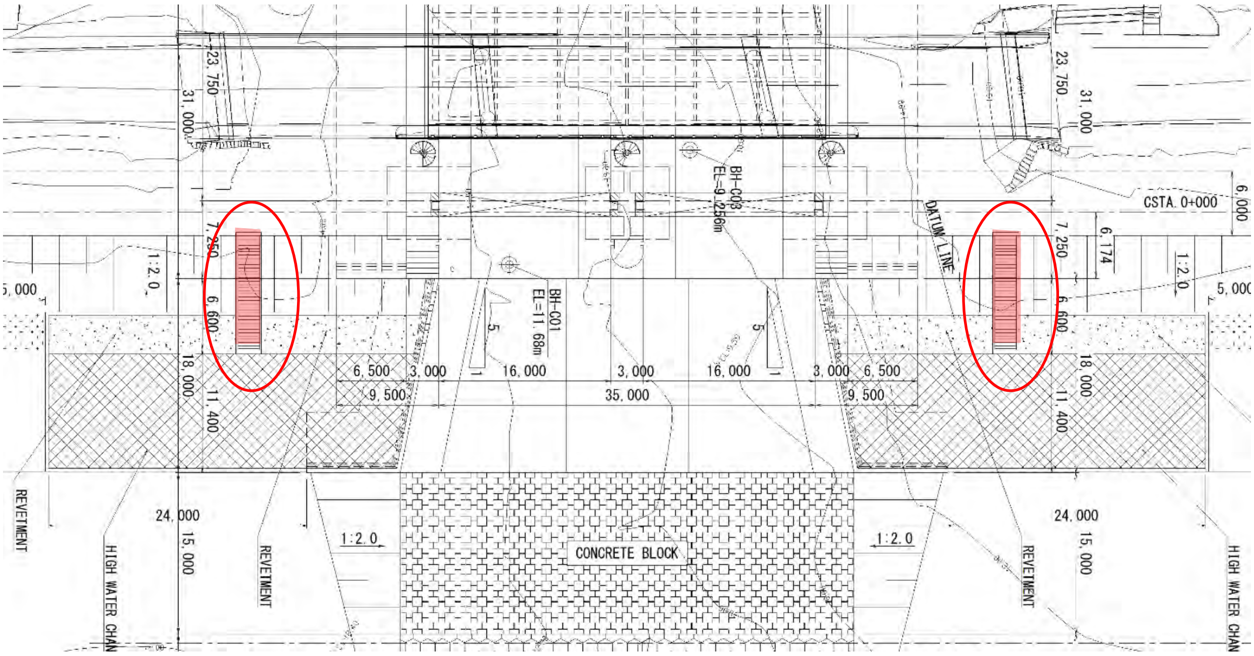
Source: Study Team

Figure 6.4.44 Revetment Structure

(2) Stairs

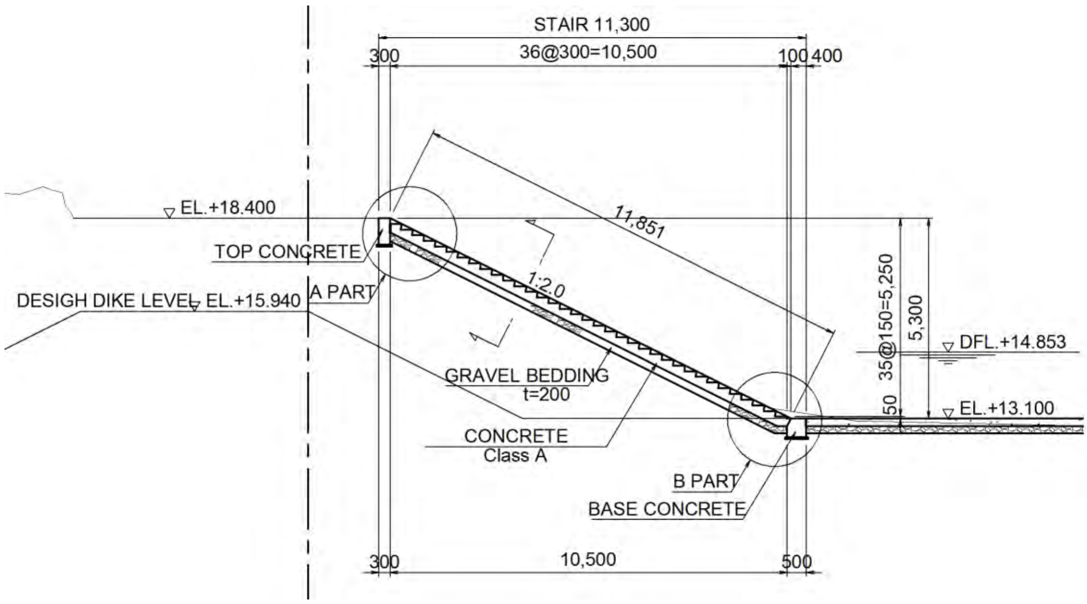
Stairs for maintenance are installed on the dike slope in the floodway side. The stairs would be constructed of concrete with an effective width of 1m or more and a height of 15 ~ 20 cm per step. The stairs should be placed both in the upper and lower side of the floodgate.

The plan of the stairs are illustrated in **Figure 6.4.45** and **Figure 6.4.42**.



Source: Study Team

Figure 6.4.45 Stair Plan (1)



Source: Study Team

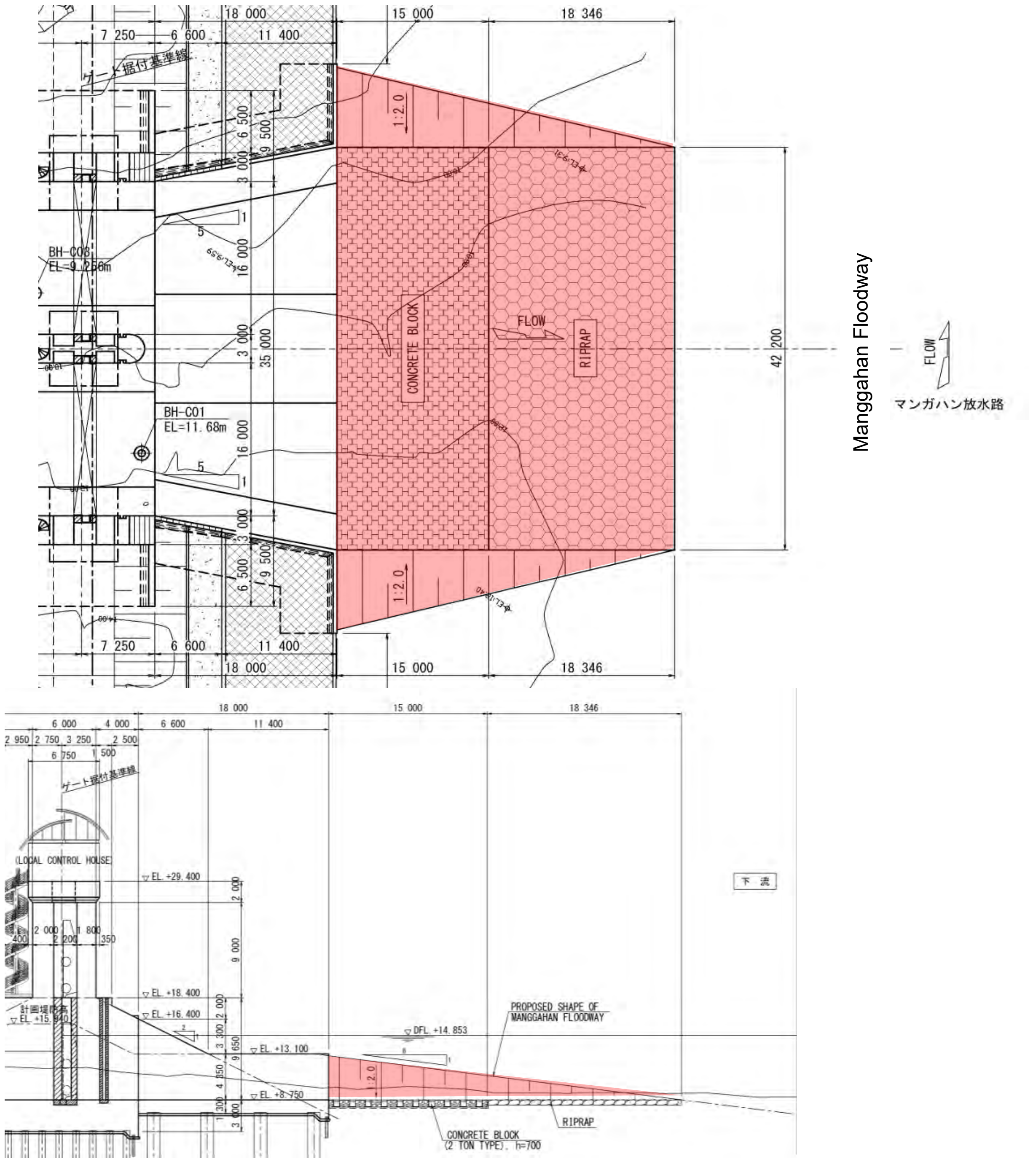
Figure 6.4.46 Stair Plan (2)

(3) Connecting Water Channel

A water channel be installed to connect the downstream wing wall and Manggahan floodway.

1) Location

As shown in the figure below, the location is from the end of the downstream wing wall to the point where the design riverbed height EL+8.75 of the Cainta River crosses the proposed shaped of Manggahan Floodway.



Source: Study Team

Figure 6.4.47 Connecting Water Channel

2) Channel structure

The connecting water channel shall be provided in a direction perpendicular to flow of the floodway not to accelerate the scouring of dike during flood. The side slope of the channel shall be 1: 2.0. The channel structure is designed with reference to **Figure 6.4.48**. Regarding the revetment of the channel, the " Guideline for Flexible Sluiceway " states that it should be installed up to the riverbank protection line (**Figure 6.4.48**) However, in the case of this facility, the wing wall will be installed up to the high water channel of the Manggahan Floodway, and the water channel is located in the low water channel of Manggahan Floodway. Therefore, the any revetment would be placed on the slope of connecting water channel of this facility.

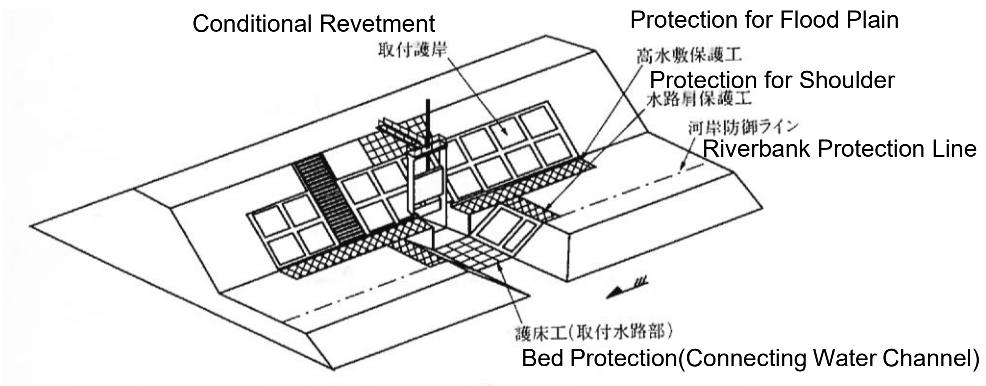


図 1-6-16 取付水路の護岸の例

Figure 1-6-16 Example of connecting water channel revetment

Source: Guideline for Flexible Sluiceway

Figure 6.4.48 Revetment of Connecting Water Channel

Ripraps shall be installed on the riverbed of water connecting channel in the floodway side to prevent it from being scoured. Since the flow velocity of the Manggahan floodway in this section is about 1.7 m/s, the type of RIPRAP is Class A.

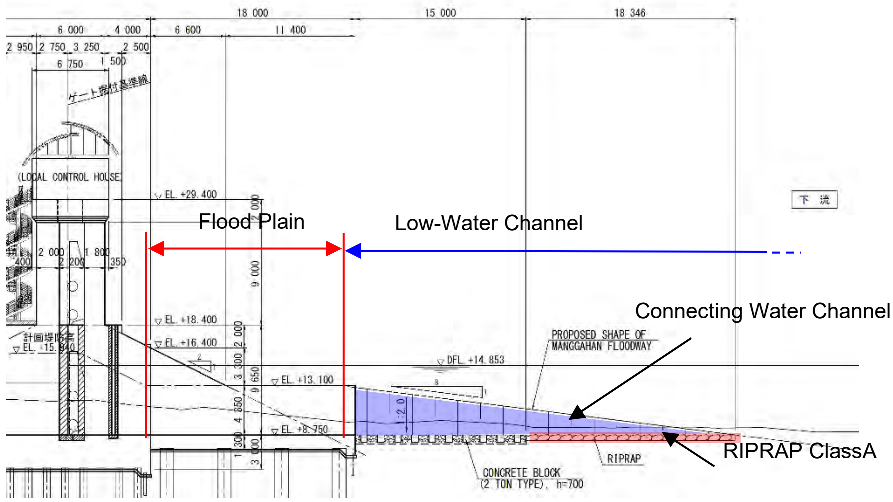
Table 6.4.41 RIPRAP Class and Flow Velocity

Unit Weight of Stones = 2.5 t/m3 (g/cm3)

Class	Minimum		Maximum		50% Weighting			Maximum Limit
	Weight (kg)	Radius (cm)	Weight (kg)	Radius (cm)	Weight (kg)	Radius (cm)	Diameter (cm)	
A	15	11.3	25	13.4	20	12.4	24.8	2.2 m/s
B	30	14.2	70	17.9	50	16.8	33.7	2.6 m/s
C	60	17.9	100	21.2	80	19.7	39.4	2.9 m/s
D	100	21.2	200	26.7	150	24.3	48.6	3.2 m/s

Note: E1 = 0.86 (Strongly Turbulent Flow) is considered.

Source: Study Team calculation



Source: Study Team

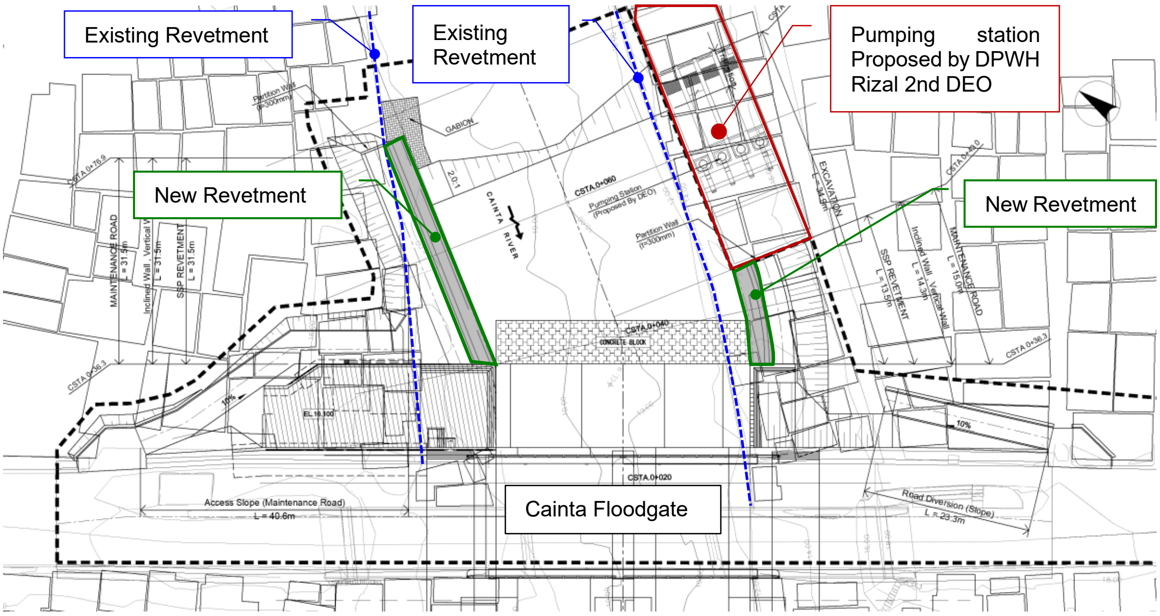
Figure 6.4.49 Section of Manggahan Floodway

(4) Cainta River Revetment

1) Summary

Although there are plans to improve the Cainta River in the future, specific improvement plan has not been determined. Therefore, in this design, only the revetment for connection with the existing revetment is designed maintaining the river channel width at the floodgate section. In the left bank side, the DPWH Rizal 2nd District Office is proposing to install a drainage pumping station, and the civil engineering works for the pumping station is scheduled to be completed before the construction of the Cainta Floodgate. Since the pumping station is designed to be installed near the existing riverbank of the Cainta River, the left bank revetment of the Cainta River is designed to be connected to the wall of the pump station.

• Stretch	:	(Right Bank) Up to the existing revetment (Left Bank) Connect with the wall of the pumping station proposed by DPWH Rizal 2nd DEO
• Revetment Type	:	Slope ...Inclined Wall Foundation ... Steel Sheet Pile Revetment
• Slope	:	0.5:1
• Revetment Construction Crown Height	:	Design Dike Level (DFL + Freeboard)



Source: Study Team

Figure 6.4.50 Installation Stretch of Cainta River Revetment

2) Revetment Type

In consideration of the followings, revetment with steel sheet pile revetment and inclined wall is recommended, as is the case with the Marikina River.

Since the revetment is connected to the floodway, it is necessary to have a durable revetment type so as not to make a weak point.

The design riverbed is EL 8.750 m and the total height exceeds 5 m from the two of revetment EL 13.940 m.

As well as Marikina River, there are a lot of houses along both riverbanks, and it is necessary to make the width of the structure as narrow as possible by making the slope steeper, etc.

The ordinary water level is EL 11.30 m, and the water depth is usually about 1.5 m at present, so the workability of the steel sheet pile revetment is better.

3) Revetment Crown Height

Revetment crown height is calculated by adding freeboard to the DFL of the Cainta River. In this section, the ground level in the landside is higher than that DFL, and since it is the excavated river channel, the minimum value of 0.6 m is set as the free board.

4) Extra Embankment

Cainta River is the river are excavated river channels in the both the left and right banks, and the banks of the river, including sections in the upstream side that will be extended in the future, are generally improved by cutting, so extra embankment is not considered. However, sandy soil (borrow material) shall be used in the portion between existing revetments and new revetments to be filled to minimize the amount of subsidence.

5) Maintenance Road

A maintenance road is provided on the top of the revetment. Considering the improvement of the Cainta River in the future, the road width shall be set to 3 m so that vehicles can pass through. In addition, the revetment to be installed in this project is only a short section of about 70 m from the Floodgate to the existing revetment, and it is assumed that only the maintenance vehicle passes through the maintenance road. Therefore, only gravel pavement is installed.

6) Foot Protection Work

In order to maintain the design riverbed and prevent the scouring in front of the steel sheet pile revetment, a foot protection is installed. The type is riprap as well as Marikina River, and the width would be 4 m based on the following calculation.

$B = L_n + \Delta Z / \sin\theta$ [3]

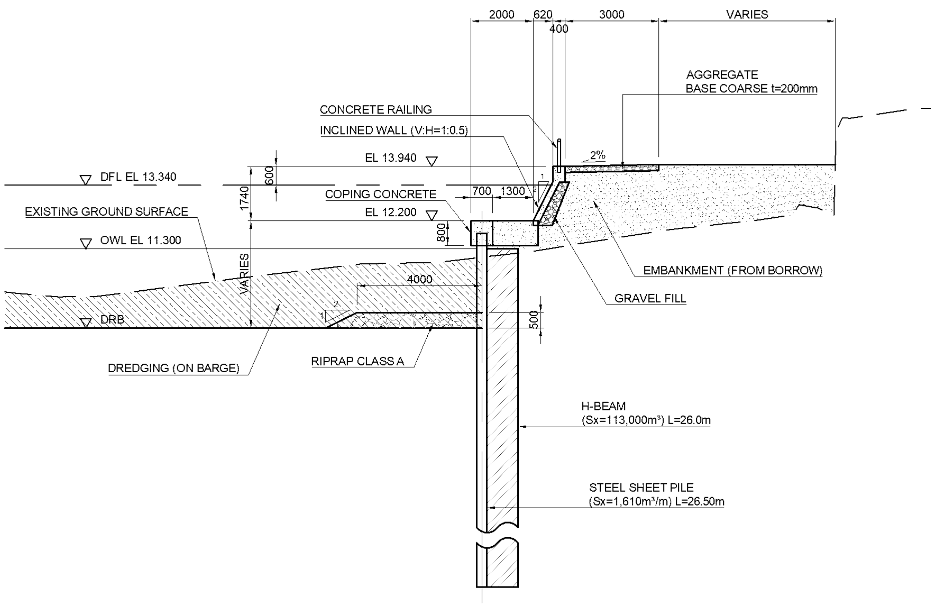
= 2 + 1/sin 30 ° = 4.0 m

where:

Ln : Horizontal Width (= 2m)
$\sin\theta$: Scouring angle (= 30 °)
ΔZ : The height from the riverbed after scouring to the design riverbed (= assumed to be 1 m).

7) Standard Cross-Section of Revetment

A standard cross-section of the Cainta River revetment is shown below.



Source: Study Team

Figure 6.4.51 Standard Cross-Section of the Cainta River Revetment

[3] DGCS vol. 3, 2015, P5 -47

(5) Approach Road

An approach road is installed between an existing east bank road (two lanes on each side) and a new bridge (two lanes on each side).

1) Width

The width of the carriageway shall be 14.6 m, which is equal to that of the bridge section, and curb and gutter of 0.67 m shall be provided at both ends of the roadway. About sidewalk, the minimum width of 1.2 m specified in DCGS is applied as well as the bridge section. The width of the sidewalk of the existing bridge is about 1.2 meters.

The width of the roadway of the existing road near the edge of the approach road is about 12.5 m, which is narrower than the 14.6 m of the proposed approach road. Therefore, transitions are provided at the edge.

2) Pavement Structure

Although it was confirmed at the site that the pavement of the existing road was concrete pavement covered with asphalt, it was not possible to obtain detailed data on the pavement structure of the existing road. Since asphalt covering of concrete pavement is not recommended by DPWH D.O. No. 11 of 2015, only concrete pavement is installed in this design. The pavement thickness is set to 300 mm referring to the past similar case of existing bridge and approach road. Since the required performance is equivalent to that of the existing East Bank Road, it is recommended to change the same pavement thickness as the existing road after the confirmation the pavement thickness of the existing road during construction.

3) Cross Sectional Gradient

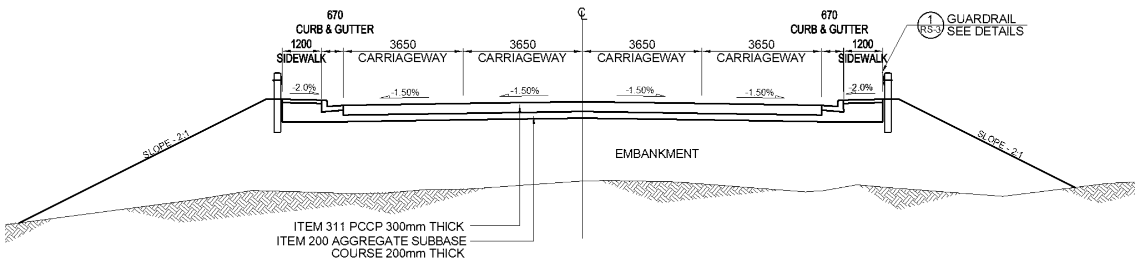
1.5% cross slope is provided as well as the bridge section.

4) Longitudinal Gradient

Basically, no longitudinal slope on the approach road is set. However, while the elevation of the bridge surface on the centerline of the bridge is EL. 18.51 m, the elevation of the existing road near the edge of the approach road on the centerline is EL. 18.00. Therefore, longitudinal transition would be provided between the existing and the new section. The longitudinal gradient shall be approximately 2% on the right bank side of Cainta River and 1.5% on the left bank side of Cainta River considering of the longitudinal gradient of the existing road.

5) Standard Cross-Section

Standard cross section of the approach road is shown in **Figure 6.4.52**.



Source: Study Team

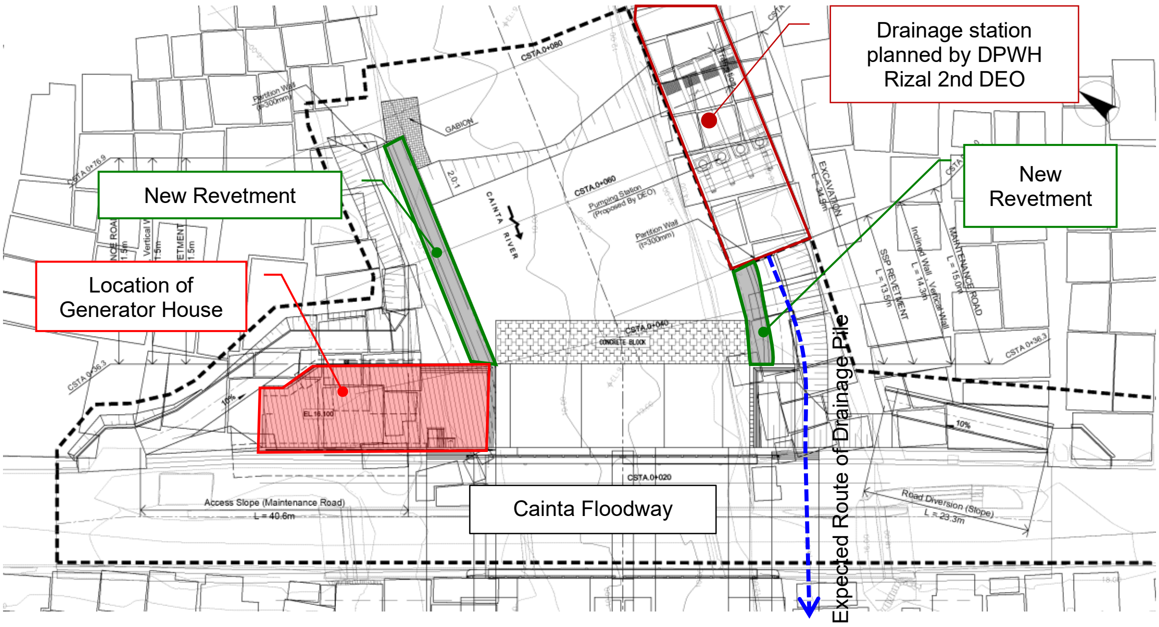
Figure 6.4.52 Standard Cross Section of Mounted Road

(6) Ground for Generator House

1) Location

The Cainta Floodgate will have a separate generator house, as well as the case of MCGS and Rosario weir. In addition to the generator, an emergency operation panel and an office space for operators are provided in the generator house.

A pumping station proposed by DPWH Rizai 2nd DEO will be installed on the left bank side of the Cainta River, and plumbing will be installed to connect the pumping station to the Manggahan Floodway. Therefore, the right bank side of the Cainta River should be avoided, and the left bank side should be used for the location of generator house. Furthermore, considering the necessity of securing the space by providing retaining walls in the floodway side and avoiding installation on the upstream side of the floodway, the generator house is installed on the top of the wing wall in the right bank upstream side of the Cainta River. The location of the generator house is shown in **Figure 6.4.53**.



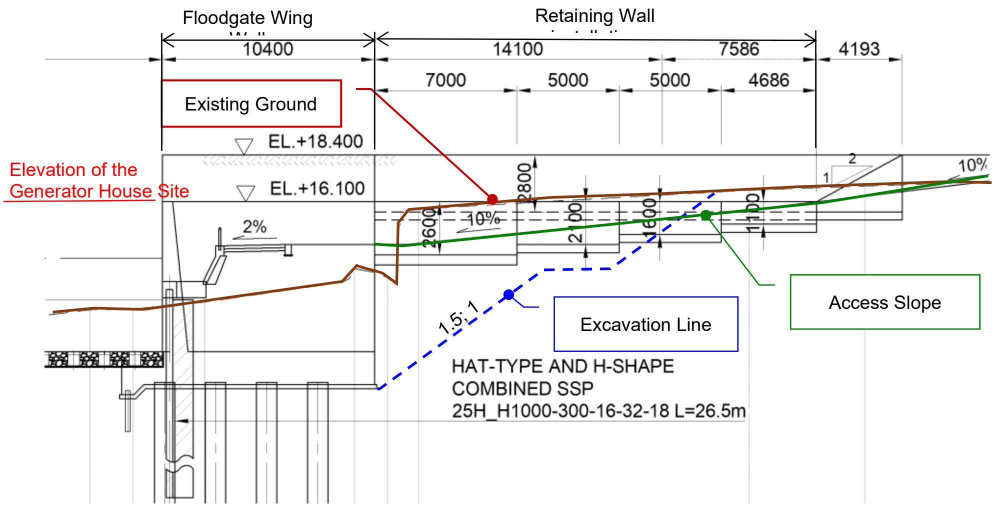
Source: Study Team

Figure 6.4.53 Location of the Generator House

2) Elevation of Ground

The elevation of the ground shall be equal to or higher than the design dike crown of the Manggahan Floodway considering a case the floodway cannot be closed and set to EL. 16.100. Extra embankment is not considered for the following reasons.

- The floodway wing wall is supported by pile foundations, so there is little concern of settlement. Borrowed Material (sandy soil) is used as the filling material behind the vertical wall.
- The present ground elevation in the land side from the floodway wing wall is about EL. 16.0 or more, and the effective vertical pressure hardly increases. Rear side of the vertical wall of the floodway wing wall would be backfilled with borrowed Material (sandy soil).



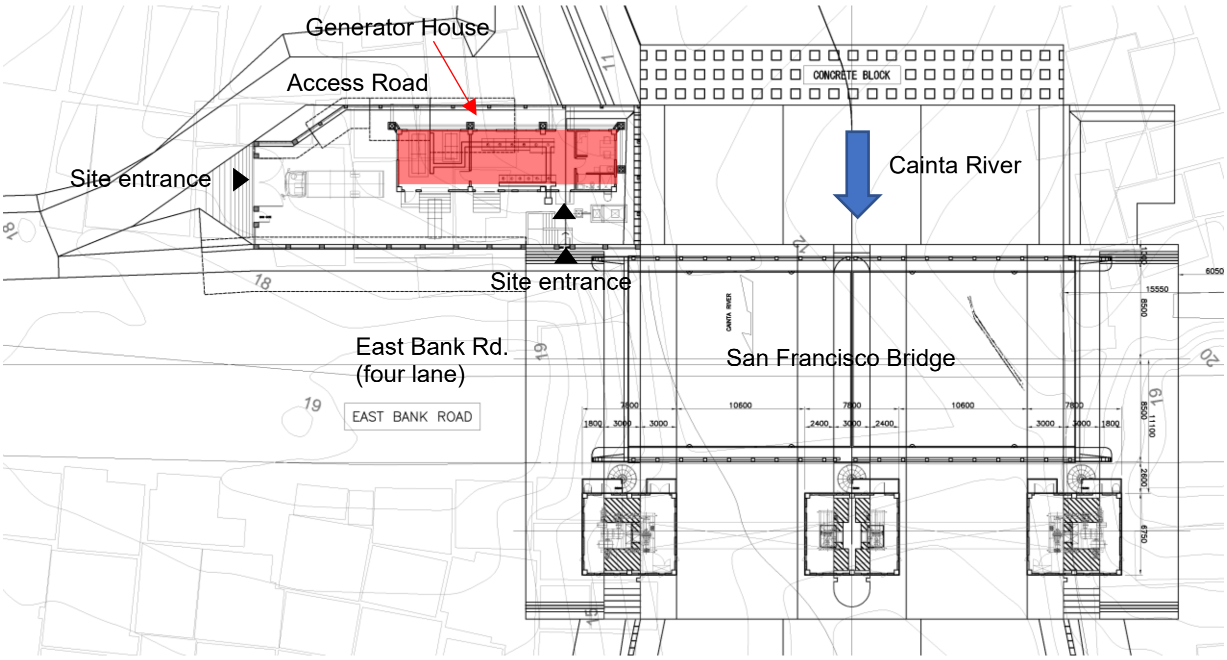
Source: Study Team

Figure 6.4.54 Relationships Between the Elevation of Generator House and the Ground of the Surrounding Area

(7) Generator House

1) Outline and layout plan of the generator building

The generator house contains emergency generators, control panels and stuff room for operation stuff and guard man to standby. Generator house shall be located on land side of East bank Rd. not to invade cross section area of Manggahan floodway.



Source: Study Team

Figure 6.4.55 Outline of Cainta Flood Gate Site Development Plan

The following points should be considered in relation to the location of the entrance to the generator building and the surrounding structures.

- ✓ The space between the access road and East bank Rd. at can be used as parking lot. The site elevation will be +16.1 m corresponds to the elevation of access road in front of the entrance.
- ✓ Since site elevation is 2.3 m below the elevation of East Bank Rd., it is not practical to connect it to public sewerage. Sewage from the generator building shall be treated on site by a septic tank in accordance with local standards. Rainwater can be drained to Cainta Creek throughout drainage under the access road.

2) Basic plan for the generator building

(a) Generator House

(i) Design Conditions of the Generator House

The generator house contains emergency generators, control panels and stuff room for operation stuff and guard man to standby. Major equipment installed in the local control house are listed in the table below. The number and dimensions of generators and panels are based on the results of the previous section.

Table 6.4.42 Major Equipment Installed in Generator Building

Room Name	Equipment	Remarks
Generator room	Generator (for switching power)	
	Generator (for control equipment)	
Electrical room	receiving panel	
	main distribution board	
	uninterruptible power supply	
	lightning protection transformer	
	remote control system control panel	
	water level gauge control panel	
	monitoring camera control panel	
	simplified gate operation panel	
	Control panel for generators (for switching power)	
	Power storage panel for generators (for switching power)	
	Control panel for generators (for control equipment)	
	Power storage panel for generators (for control equipment)	
Stuff Room	Desk (For 2 people)	
	Bed (Double)	
	Pantry	Shall follow requirement specified in NBCP
	Toiletry, Shower	Shall follow requirement specified in NBCP

Source: Study Team

The minimum separation of panels and generators and the arrangement of cable pits shall be in accordance with the conditions specified in the MCGS generator building.

(ii) Plan of the Generator Building

The followings shall be considered for layout planning of generator house.

[Electrical Room]

- ✓ Efficiency of cabling layout shall be considered, such as grouping of control panels (panels for gate operation, generator control and power receiving/distribution).
- ✓ Efficiency of flow line of stuff shall be considered, such as good visibility and easy access to each room from the main corridor.
- ✓ A steel shutter shall be provided facing road with sufficient width for control panels to pass through.
- ✓ Handholes shall be provided for cables to local gate control houses.

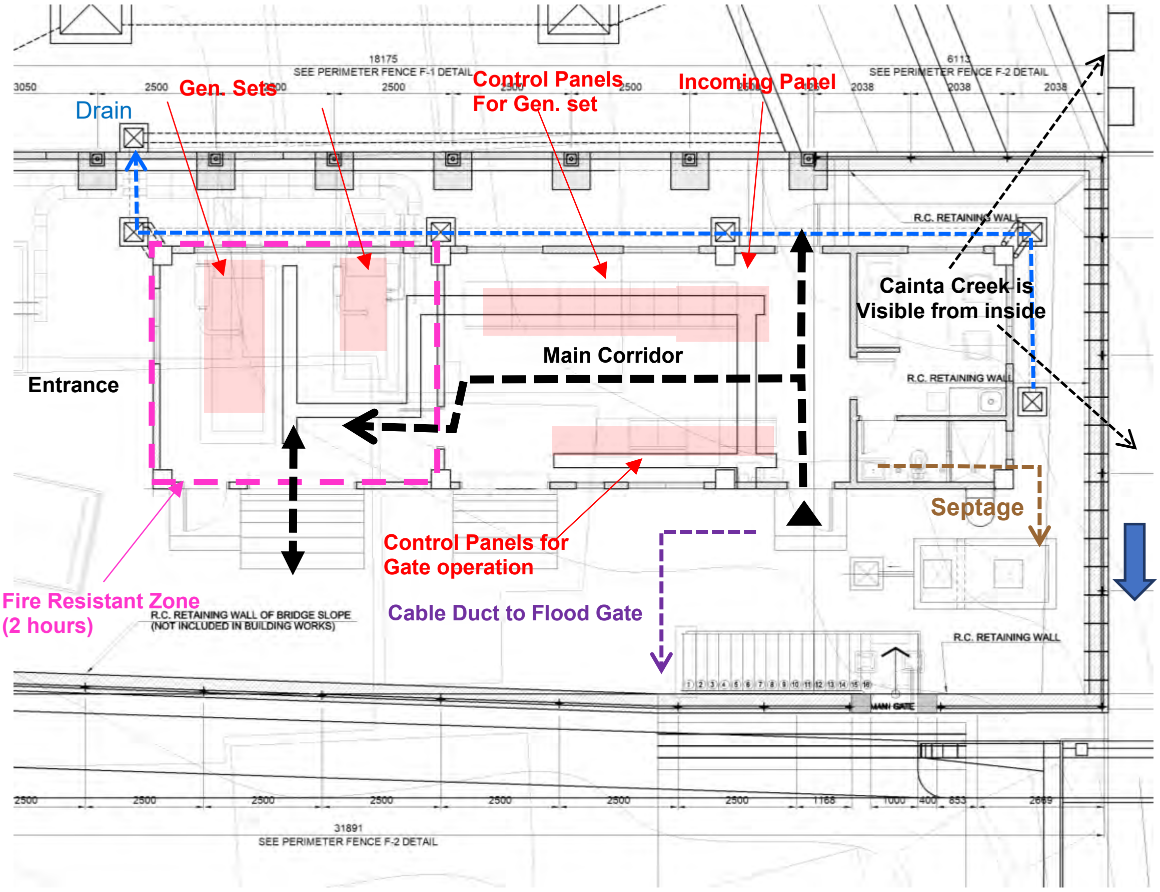
[Generator Room]

- ✓ Based on local fire codes and enforcement regulations, the maximum area of a room to store fuel is 46.5 m2 under conditions of 2hour fire-rated wall/doors and no sprinkler. generators for gate hoisting and a generator for control system can be installed in a same room.
- ✓ A steel shutter shall be provided facing road with sufficient width for generators to pass through.

[Other Drainage Facilities]

- ✓ The stuff room is desirable to be facing the gate structures so that stuff can overlook them easily from inside the room.
- ✓ Rainwater on the roof maybe gathered at river side of the building to avoid interference with the handholes and electrical conduits.
- ✓ Septic tank may be located nearby toilet and bathroom close to the river so that it does not interfere with electric wires.

Figure 6.4.56 shows layout plan of MCGS generator house considering the above mentioned conditions.



Source: Study Team

Figure 6.4.56 Layout Plan of MCGS Local Control House

(b) Cross-sectional Plan of Generator House

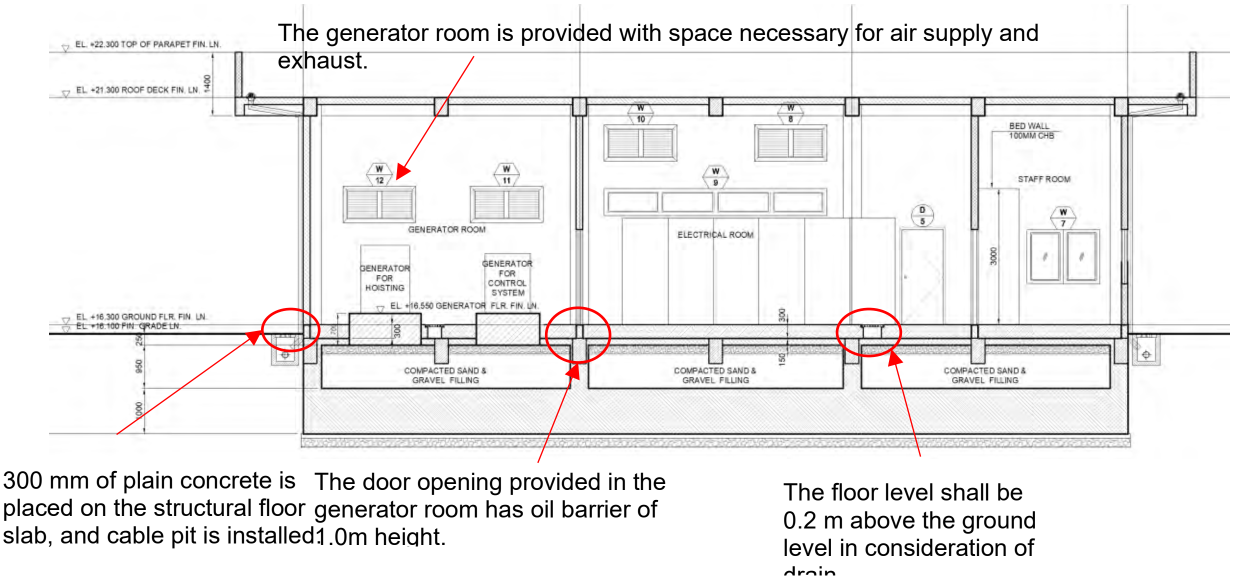
The cross-sectional design of the generator building shall be determined considering the following factors.

[Ceiling Height]

- ✓ The ceiling height of the generator room should be designed to allow sufficient louver height for the supply/exhaust of the generator.

[Floor Height]

- ✓ The floor level is assumed to be 0.2 m above the surrounding ground level in consideration of rain.
- ✓ 300 mm of plain concrete is placed on the structural floor slab, and cable pit is installed. Therefore, the structural floor slab has a floor level of 0.3 m.
- ✓ The door opening installed in the generator room has a lower end height of 0.1 m above the floor level and functions similar to an oil dike in accordance with local fire defense standards and enforcement regulations.



Source: Study Team

Figure 6.4.57 Typical Section of Cainta Flood Gate Generator House

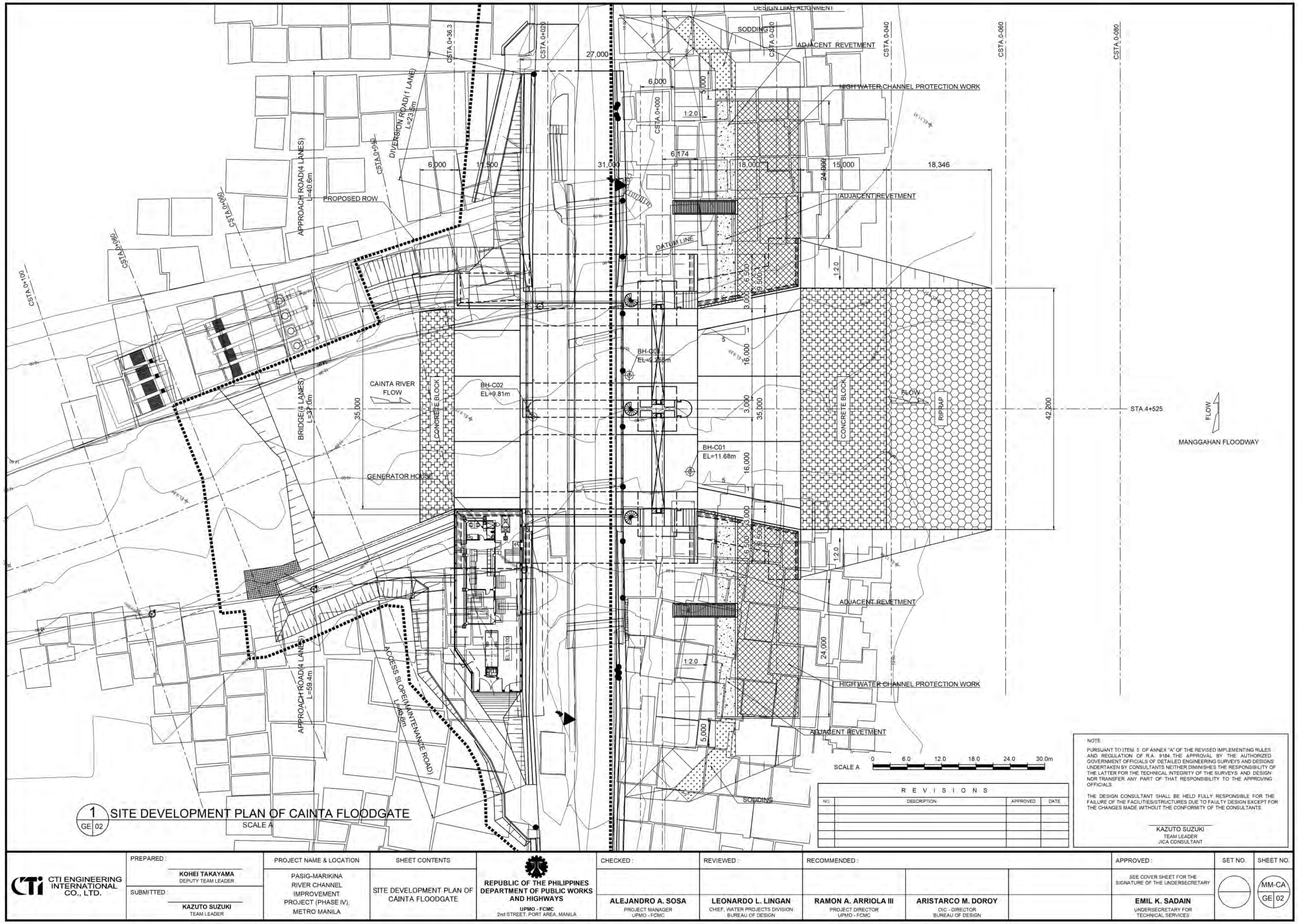
(8) Necessity of Spare Gates (Stop Logs)

Considering the following items, spare gates would not be installed but only the supporting frame for stop logs would be installed in Cainta Floodgate.

- The purpose of the spare gates is to use as a cofferdam during the maintenance around the main gates. Since it is not practical, using instead of the main gate would not be considered.
- In this design, the material of the gate leaves and guide frames is corrosion resistant. Hence, the maintenance with cofferdam around the main gates will not be frequently performed.
- Considering the above 2 items, it is better to prepare the spare gate when it is needed rather that to store the spare gates which needs periodical maintenance. Also, it is superior also in the reliability.

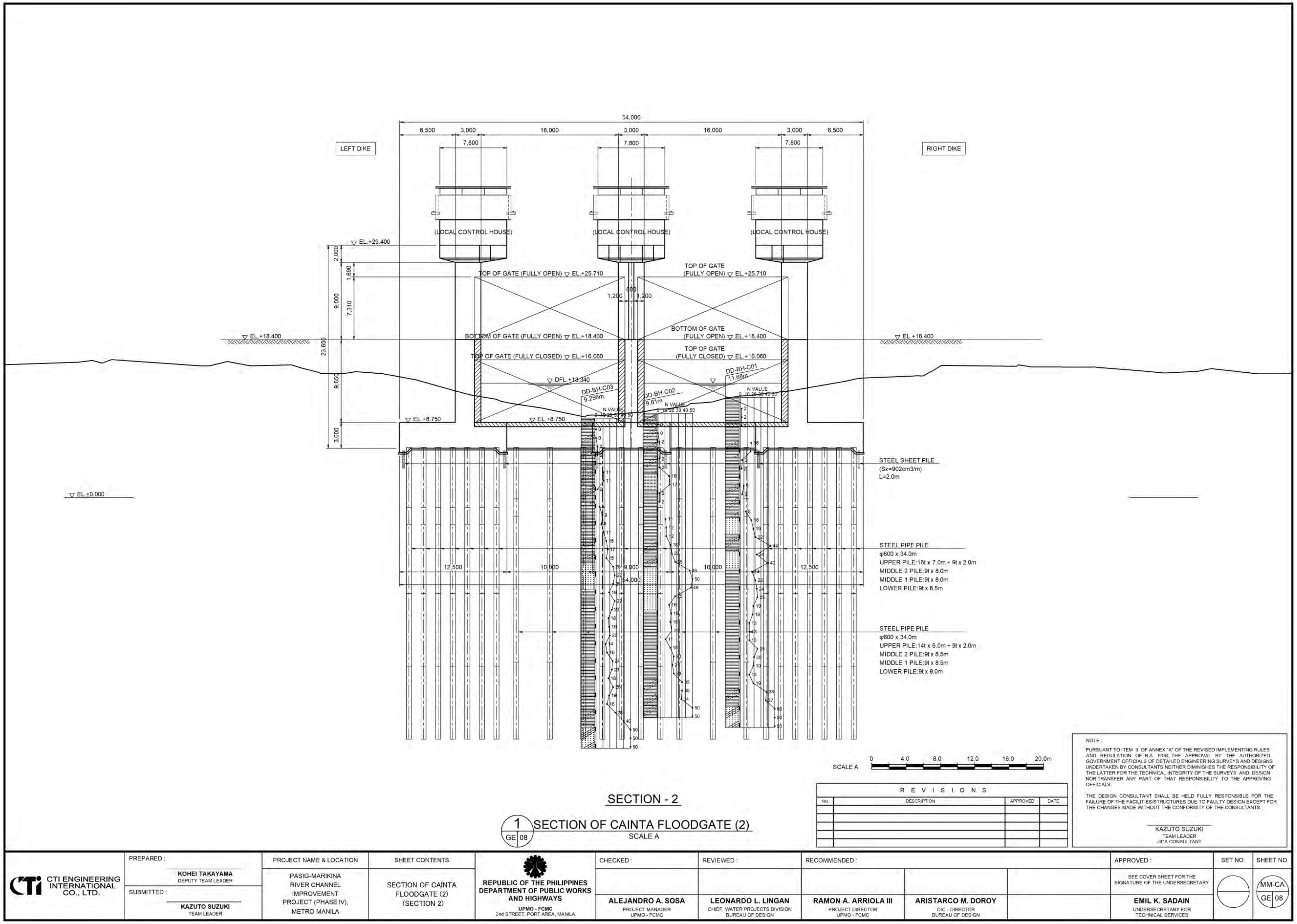
6.4.3.11 General Drawings

The following pages show the general drawings prepared based on the above conditions and basic considerations.



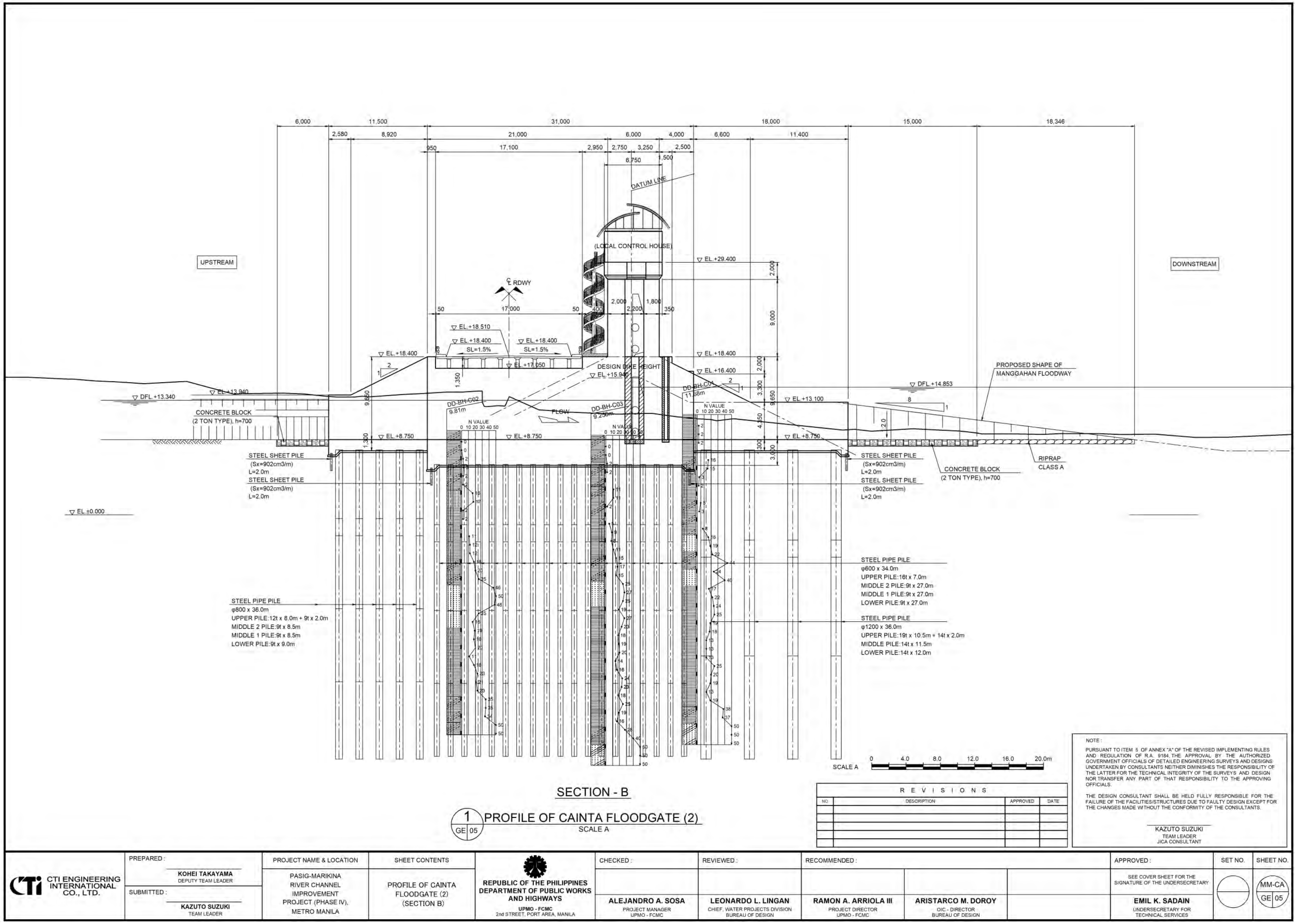
Source: Study Team

Figure 6.4.58 General Drawings of Cainta Floodgate (1)



Source: Study Team

Figure 6.4.59 General Drawings of Cainta Floodgate (2)



Source: Study Team

Figure 6.4.60 General Drawings of Cainta Floodgate (3)

6.4.4 Taytay Sluiceway Basic Design

6.4.4.1 Water Level Condition

Water level conditions at the site of the Taytay Sluiceway are as shown in **Table 6.4.43**.

Table 6.4.43 Taytay Sluiceway Water Level Conditions

Water Level Condition	Taytay Creek (EL.+m)	Manggahan Floodway (EL.+m)	Remarks
DFL	13.500[1)]	14.520	DFL of Manggahan Floodway is calculated by interpolation from the completed drawing [2)] The water level at the Taytay Creek side is DFL at the upstream side of the existing box culvert.
During Flood	10.600	14.520	Invert Elevation of the existing Box Culvert [2)]

Source: [1)] 2008 Pre-F/S; [2)] Final Report on the Consulting Services for the Manggahan Floodway Project

6.4.4.2 Navigation and Other Conditions

(1) Condition of Navigation

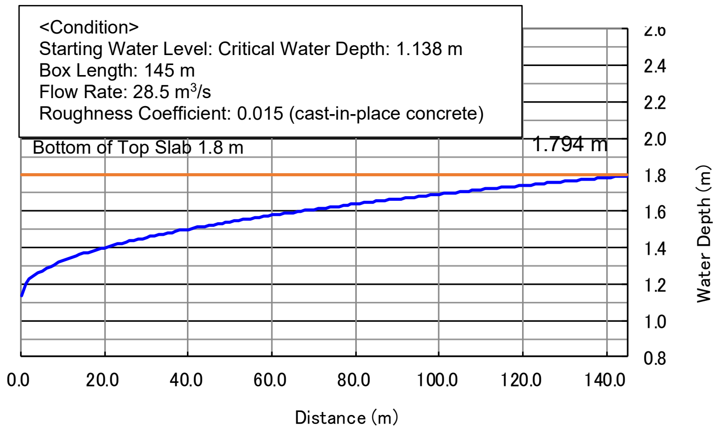
At present, no boat or ferry is operating in the Cainta Floodgate location. Therefore, ship navigation is not considered in the study on the lower end height of the gate when fully open.

(2) Condition of Existing Box Culvert

1) Flow Capacity

In the discharge distribution proposed in the 2008 Pre-F/S, the drainage volume of 30 m^3/s from the Taytay Sluiceway is set based on the flow capacity of the existing box culvert. The capacity of the existing box culvert was checked in the study.

The calculation result of one-dimensional non-uniform flow in the existing box culvert is as shown in **Figure 6.4.61**. According to this figure, the flow rate when the culvert is full becomes 28.5 m^3/s, which is almost equal to the discharge distribution of 30 m^3/s proposed in the previous plan.



Source: Study Team

Figure 6.4.61 One-Dimensional Non-Uniform Flow Calculation Results for Box Culvert

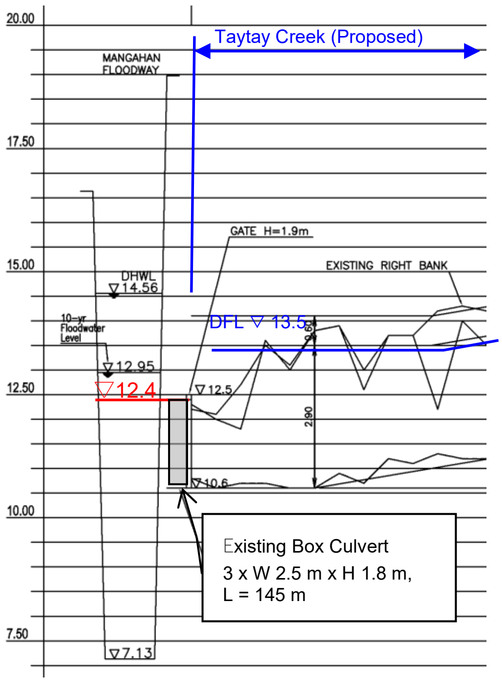
According to the 2008 Pre-F/S, the bottom of top slab of the existing box culvert is EL. 12.4 m, while the DFL of Taytay Creek at the connecting point with the existing box culvert is set to EL.+13.5m (see **Figure6.4.62**). Based on this information, it can be said that in the 2008 Pre-F/S, the flow capacity of the existing box culvert was evaluated under the full flow condition of the culvert.

Accordingly, the flow capacity of the existing box culvert is considered as 28.5 m^3/s in the full flow condition.

2) Top of Gate and Width of Water Path

There is a dense residential area above the existing box culvert. Therefore, it is unlikely that the section of the existing box culvert is demolished, and the flow area of the box culvert is expanded.

Therefore, it is considered that the lower end of the gate opening does not need to be raised higher than the required height which is the bottom of the top slab of the existing box culvert. In addition, it is considered unnecessary to expand the width of the waterway of the Floodgate beyond that of the existing box culvert, since the flow capacity of the Floodgate is limited by the existing box culvert.



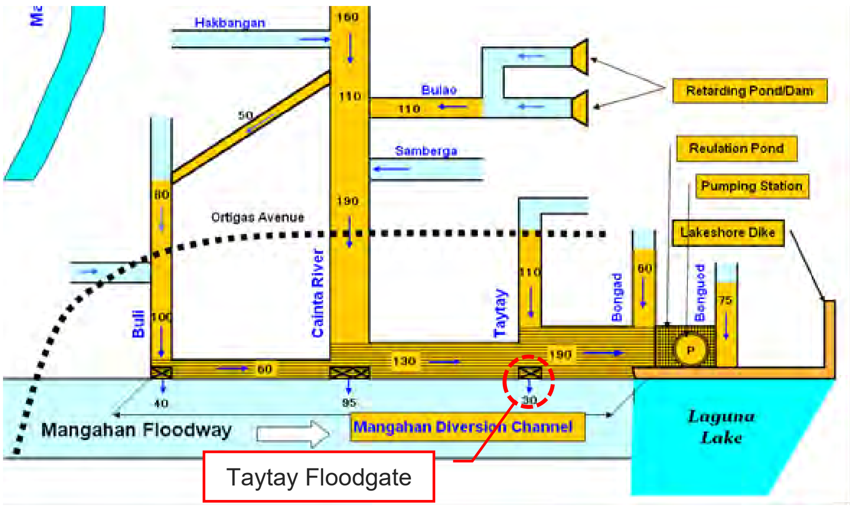
Note: Added by the JICA Study Team to the 2008 Pre-F/S Study result.

Figure6.4.62 Taytay Creek Proposed Profile

6.4.4.3 River Condition

(1) Proposed Discharge Distribution

The proposed discharge distribution around the Floodgate location is shown in **Figure 6.4.63**. The 2008 Pre-F/S plan has a 10-year probability scale. As mentioned above, the proposed discharge from the Taytay Sluiceway to Manggahan Floodway is set based on the capacity of the existing box culvert. Therefore, the original plan of 30 m^3/s is adopted even if the design scale of the drainage plan in the land side is updated and increased.



Source: The 2008 Pre-F/S

Figure 6.4.63 Distribution of Proposed Discharge

(2) River Channel Condition

Table 6.4.44 shows a list of river channel conditions.

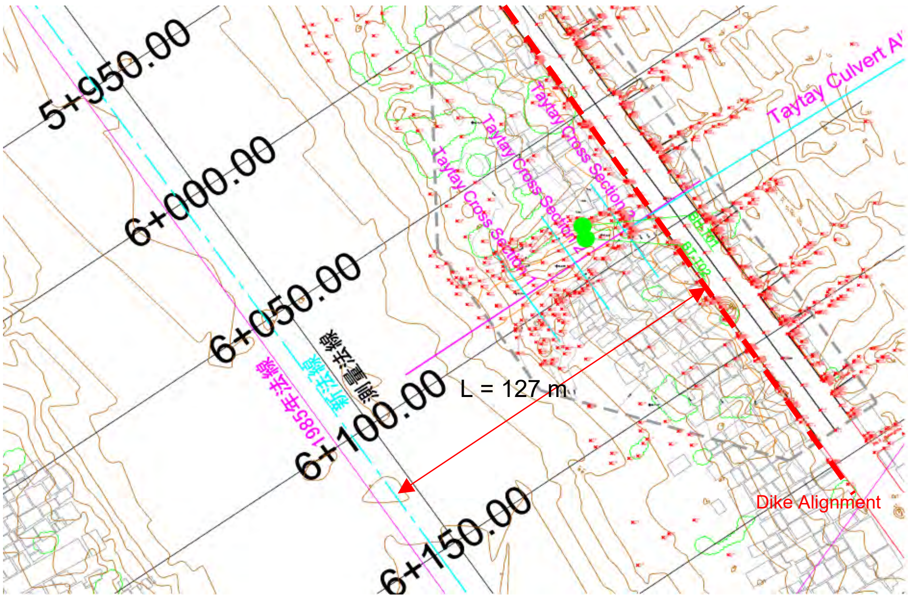
Table 6.4.44 List of River Channel Conditions

Items		Tributary Side	Floodway Side	Remarks
River Name		Taytay Creek	Manggahan Floodway	
Proposed Discharge		30 m^3/s	2,900 m^3/s	
STA.		0 -145 *	6 + 090	*The upstream end of the existing box culvert is assumed to be 0 + 000.
DFL		EL. + 13.500 m *	EL. + 14.520 m**	*Value of the upstream end of the existing box culvert **Calculated by interpolation based on the as-built drawing
Design Dike Crown		EL. + 14.100 m *	EL. + 15.620m * *	Same as above
Existing Dike Crown		EL. 11.8 - EL. 13.5 *	EL: + approx. 15.8 m	Height of the existing road center from the survey results
river channel width	Current State	15 to -20 m Duct: 3 x 2.5 m	Already the same as plan	
	Proposed	26.1 m	254.0 m	
Design Dike Width		-	6.0 m *	*From the as-built Drawing of Manggahan Floodway
Design Riverbed Width		23.0 m	108.0 m *	*Same as above
Design Riverbed		EL. + 10.600 m	EL. + 7. 120 *	*Calculated by interpolation based on the as-built drawing

Note: Arranged by the JICA Study Team from the following materials:
1) The 2008 Pre-F/S:
2) Final Report on Consulting Services for Manggahan Floodway Project

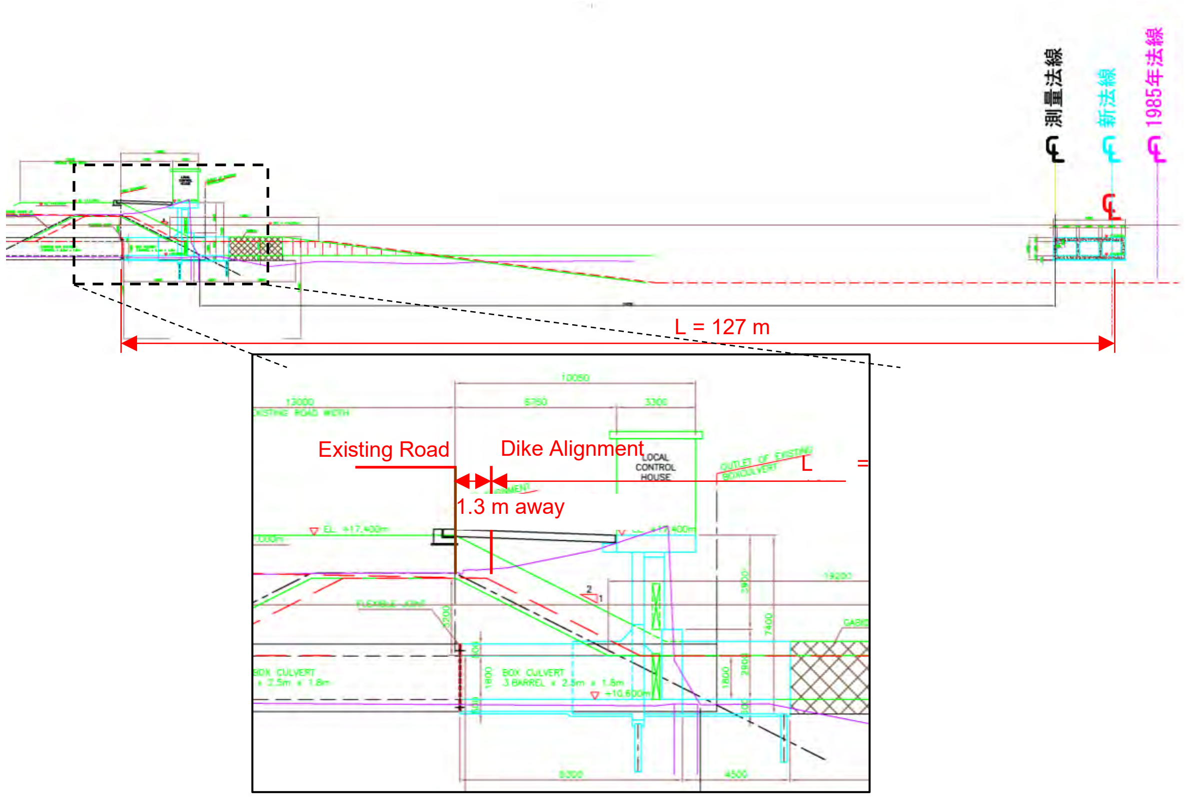
(3) Dike Alignment

The distance between the dike alignment in both banks is 254 m. A position where the distance from the river center line is 127 m in the left bank side is set to the dike alignment in the left bank side. As a result, as shown in the figure below, it is about 1.3 m away from the existing road dike.



Source: Study Team

Figure 6.4.64 River center Line and Dike Alignment



Source: Study Team

Figure 6.4.65 Dike Alignment

6.4.4.4 Condition due to Existing Structures

When considering facility layout and construction plans, the conditions due to the major existing structures are shown in **Table 6.4.45**.

Table 6.4.45 Conditions Due to Major Existing Structures

Symbol 1)	Item	Description
A1	Existing Box Culvert	A box culvert of about 145 m in length has been installed. Considering the N-value according to the past boring data, it can be assumed that it has pile foundations. (The As-built drawing is not available.).
A2	Existing Outlet of Box Culvert	End of the Box Culvert Wing walls are installed on the right and left sides of the existing box culvert, and earth retaining walls are installed on the top slab.
B	Houses in the landside	Houses in the land side from the East Bank Road are not the original targets of relocation.

1) "Symbol" indicates the location of existing structures, etc., in ***Figure 6.4.66.***
Source: Study Team.

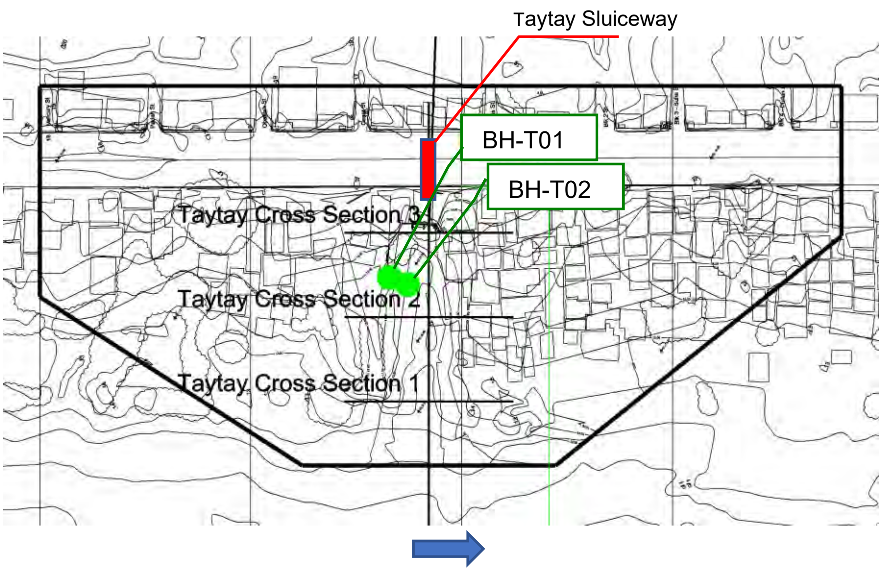
Source: Study Team

Figure 6.4.66 Major Existing Structures around the Taytay Sluiceway

6.4.4.5 Geological Condition

(1) Previous Geotechnical Investigations

The location map of the existing soil investigation and this investigation is shown in **Figure 6.4.67** See.

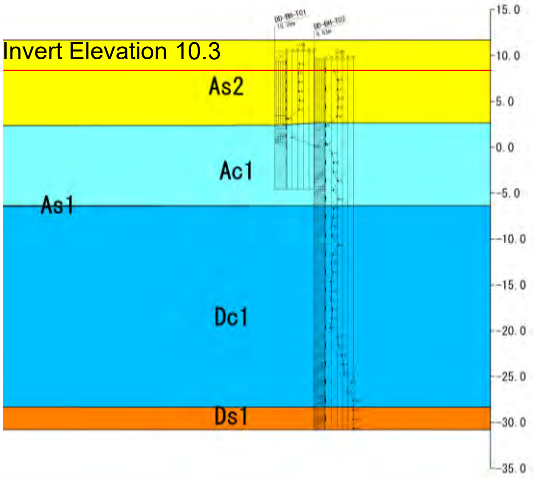


Source: Study Team

Figure 6.4.67 Geological Survey of Taytay Sluiceway Gates

(2) Geological Overview

The geological survey at the site of the Taytay Sluiceway revealed that the alluvial sandy soil layer (As2), alluvial cohesive soil layer (Ac1) and diluvial cohesive soil layer (Dc1) were approximately 8 m, 20 m and 8 m, respectively, from the top. The diluvial sandy soil layer (Ds1) appears in the lower part. Considering the continuity of the upstream and downstream flows, the Ds1 layer is positioned as the base layer. The alluvial sand layer has a relatively small N value of about 20. The N value of the alluvial cohesive soil layer at the boundary with the sand layer is as weak as about 0 ~ 2, but it tends to increase gradually downward. The N value of the upper part of 15 the diluvial clay layer is about 10 m, while the N value of the lower part tends to increase in the depth direction. In DD-BH-T01, mudstone solidified at a depth of about 10 m was confirmed, but the results of geological survey of DD-BH-T02 and Cainta Floodgate were summarized as not having longitudinal continuity.



Source: Study Team

Figure 6.4.68 Geological Profile of Taytay Sluiceway Gates (Excerpt from the Vicinity of Taytay)

(3) Assumed Geological Section

A geological map of the Taytay Sluiceway is shown. **Figure 6.4.69** See. DD-BH-T01 confirmed the solidified mudstone with N value of 50 or more at the depth of 10 m, but the continuity could not be confirmed as the result of adjacent boring at 4.6 m away (DD-BHT02) indicated that the stratum with the same depth had N value of about 10 ~ 20. Therefore, it was decided that the stratum composition of DD-BHT02 would be used for the safety side. The geological structure of DD-BHT02 was set to be horizontal, because geological surveys of the inside of the dike and the river side were not carried out.

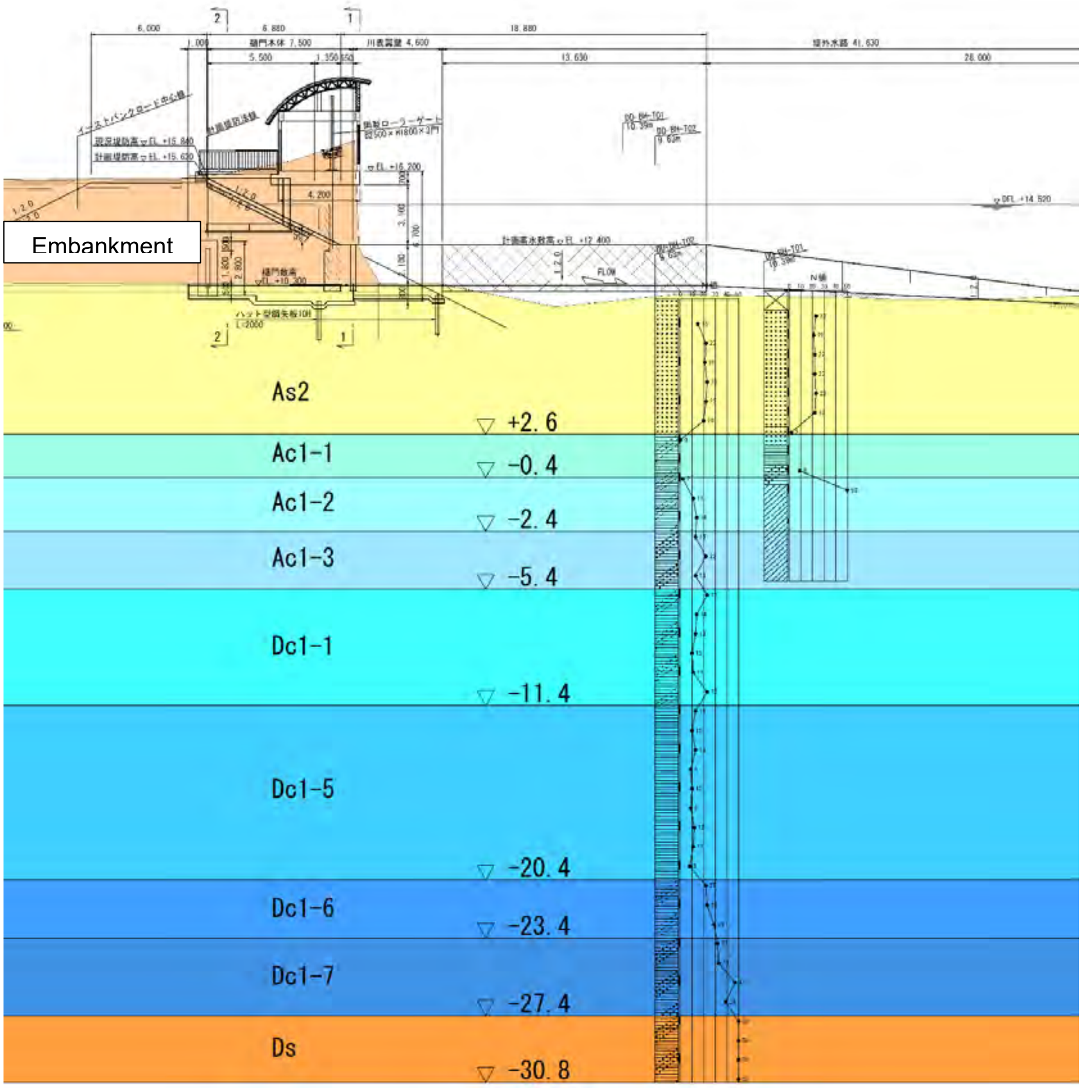


Figure 6.4.69 Assumed Geological Section of Taytay Sluiceway Gate (Sluiceway Profile)

(4) Soil Property

The soil parameters used in the design are determined based on the results of the borehole columns and laboratory tests conducted in this soil survey.

1) Setting Policy

The soil parameters were mainly set as N value, single body weight, shear strength (c, ϕ) and consolidation constant required for design. In this constant setting, stratum division was carried out from the columnar diagram, and strata with different N value in the same soil layer were made to be a fine division, and it was divided by the symbol of cohesive soil (C) and sandy soil (S). In principle, the constant of each layer was set by soil test values, but for untested layers, it was set by quoting test values of strata which can be judged to be almost the same layer.

2) Unit Weight of Soil

The test value was adopted in principle. The stratum without the test value was set referring to the test value of upper and lower layers or the survey result in the vicinity, or referring to the road earth work dike construction guideline, etc.

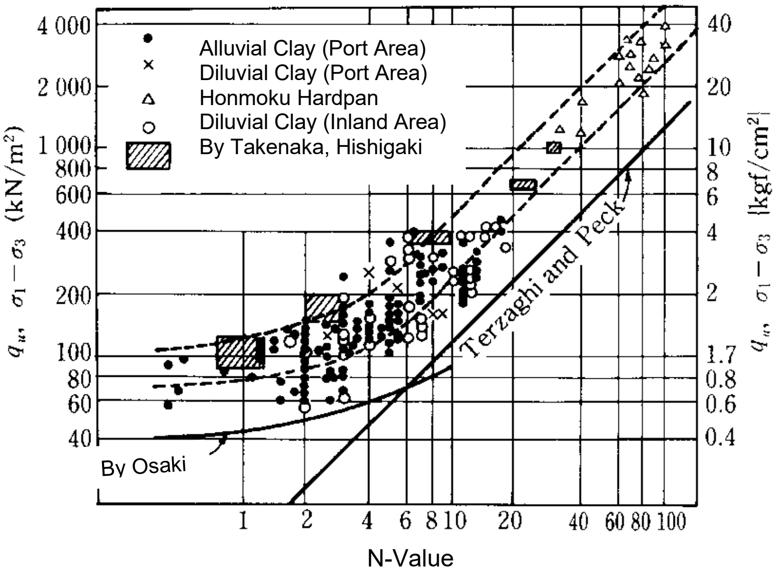
3) Shear Strength c, ϕ

Test values were adopted as a principle. For strata without test values, test values of upper and lower layers or investigation results in the vicinity were used as a reference, but for sand layers and cohesive soil with large N value, it was estimated by the following equation.

Sandy soil: N value and effective overloading pressure were considered, and the calculation formula of road bridge specifications was set as a reference. For the sandy soil layer which can be judged as a base layer at N = 50 in the lowest part, $\phi = 40°$ was set.

$$\varphi = 4.8 \times \log N1 + 21,\ N1 = 170 \times N/(\sigma'v + 70)$$

The cohesive soil was estimated from the relationship (pt = 25 N at lower limit) between the N value and qu shown in the guideline for soft ground countermeasures in the following figure.



Source: Guidelines for Countermeasures for Weak Ground of Road Soil (2012.8)

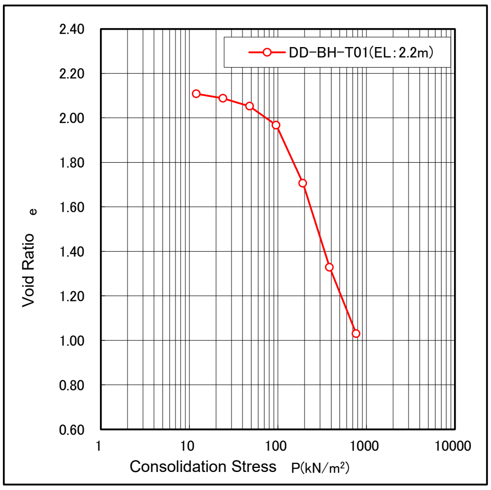
Figure 6.4.70 Relationship Between N value and Uniaxial Compressive Strength

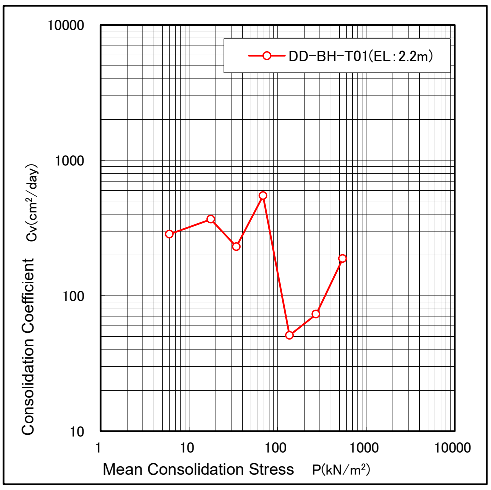
4) Consolidation Constant

The e-log P curve and logCv-log P curve were arranged from the consolidation test results. The compression index Cc and the swelling index Cs were calculated from these values, and representative values were set by showing the depth distribution in the soil property map.

・e-log P curve

・logCv to logP curve





Source: Study Team

Figure 6.4.71 Consolidation Curve

5) Setting Constant

The soil constant was set as follows. A constant is set for each boring, but a set value of DD-BH-T02 is adopted for the design.

- The unit weight was all saturated weight.
- Since no mechanical test has been conducted on the consolidation settlement target layer C1, the set values of the boring DD-BH-T01 were adopted for the weight of the single body, shear strength, consolidation characteristics, etc.

Table 6.4.46 List of Soil Constants (DD-BH-C01)

Stratum	Soil Quality	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γt (kN/m3)	Cohesion c (kN/m2)	Shear Resistance Angle ϕ(°)	Deformation Coefficient E 50 (MN/m2)	Consolidation Settlement Target Layer	Compression Coefficient Cc	Swelling Coefficient Cc
As2	Sandy Soil	21	17	4		20	0	37				(0.056)
Ac1 -1	Cohesive Soil	1	57	80	27	(15)	(14)	(0)	(1.5)	○	(1.17)	
Ac1 -2	Cohesive Soil	12	33	75	41	17	150	0				
Ac1 -3	Cohesive Soil	13	35	62	15	17	160	0				
Dc1 -1	Cohesive Soil	12	40	85	25	17	150	0				
Dc1 -2	Cohesive Soil	10	60	88	33	17	120	0				
Dc1 -3	Cohesive Soil	25	27	82	21	18	310	0				
Dc1 -4	Cohesive Soil	35	32	64	21	19	430	0				
Dc1 -5	Cohesive Soil	50	50	60	37	19	620	0				

Source: Study Team

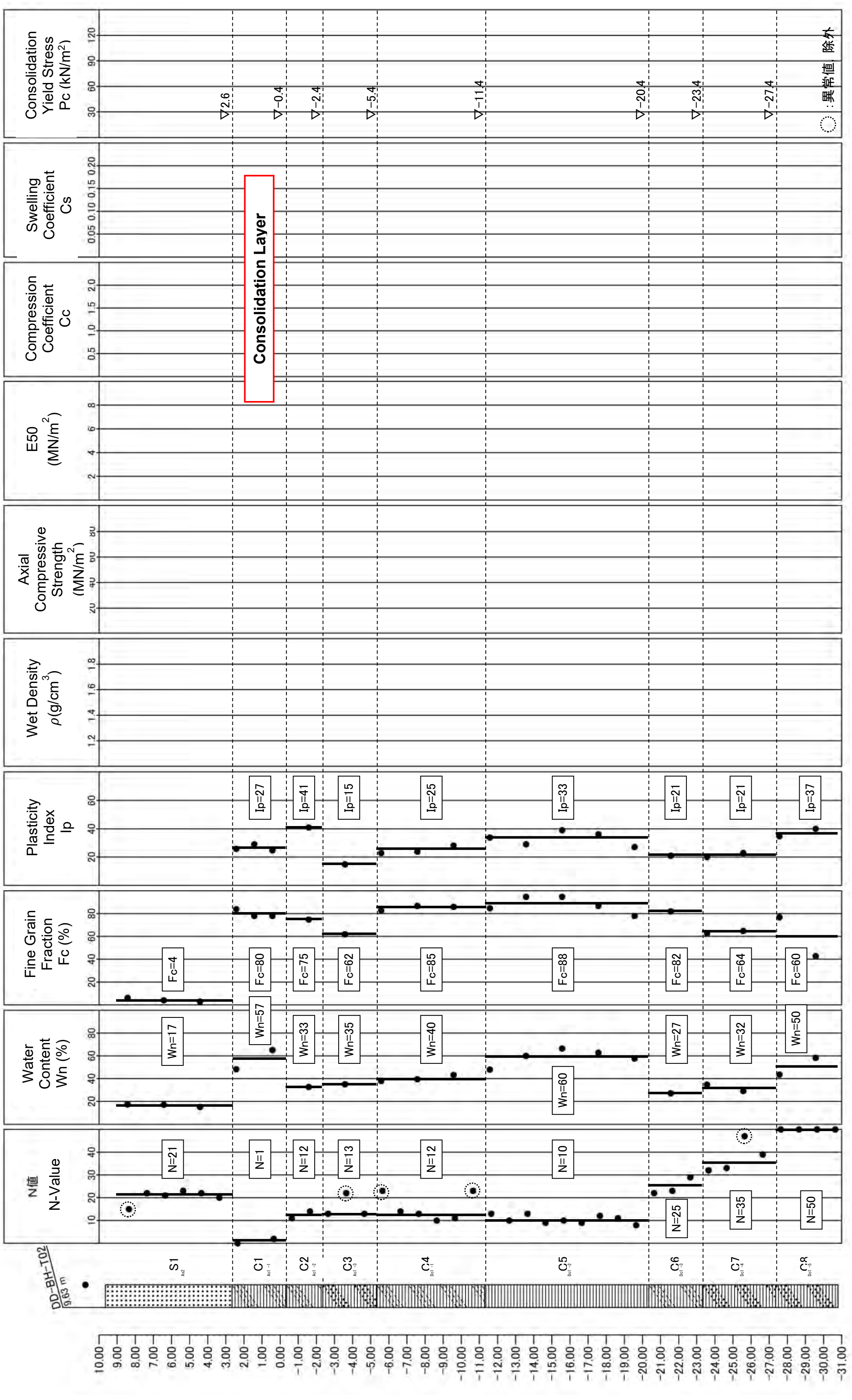


Figure 6.4.72 Soil Characteristics Map (DD-BH-C01)

Source: Study Team

(5) Type of Ground

The ground type is evaluated by the characteristic value TG of the ground, as follows:

$$T_G = 4\sum_{i-1}^{n}\frac{H_i}{V_{si}}$$

Where, Vsi : Initial Shear Elastic Wave Speed
In case of Sandy Soil Strata: $Vs=80N^{1/3}$, In case of Cohesive Soil Strata: $Vs=100N^{1/3}$
N : Value/Number of SPT in each Stratum/Layer
i : Number of the i-the Soil Stratum

Classification of Stratum	Ground Characteristic Value T_G (s)
Type-I	$T_G \leqq 0.2$
Type-II	$0.2 < T_G \leqq 0.6$
Type-III	$0.6 < T_G$

Source: BSDS

The calculation result of ground characteristic value TG in the vicinity boring (DD-BH-T02) is shown below. The ground of the Taytay Sluiceway gate installation site was class III ground.

Table 6.4.47 Calculation of Ground Characteristic Value T_G (DD-BH-T02)

No.	Stratum / Layer	Cohesive＝2 Sandy＝1 Others＝3	Boundary Surface		Thickness of Layer	N (SPT)	Elastic Wave Speed Vs	Hi/Vsi
			EL.m	EL.m	m		m/s	sec
1	As1	1	+10.39	+2.60	7.79	21	220.7	0.035
2	Ac1 -1	2	+2.60	-0.40	3.00	1	100.0	0.030
3	Ac1 -2	2	-0.40	-2.40	2.00	12	228.9	0.009
4	Ac1 -3	2	-2.40	-5.40	3.00	13	235.1	0.013
5	Dc1 -1	2	-5.40	-11.40	6.00	12	228.9	0.026
6	Dc1 -2	2	-11.40	-20.40	9.00	10	215.4	0.042
7	Dc1 -3	2	-20.40	-23.40	3.00	25	292.4	0.010
8	Dc1 -4	2	-23.40	-27.40	4.00	35	327.1	0.012
							T_G=	0.710 s

Source: Study Team

6.4.4.6 Study of Floodgate

(1) Study on Layout Location

At the location of the proposed Taytay Sluiceway, there is an existing dike, and although there is a conduit that serves as sluiceway, no gate is installed. As in the case of Cainta Floodgate, it is a policy not to place a dike in the river side of the existing dike for the following reasons:

- The dike becomes an obstacle to the water flow during floods in the Manggahan Floodway.
- The dike if shifted to the riverside becomes a water colliding front so that the shifted location is not preferable for Floodgate structures; and
- Shifting the dike alignment towards the Floodway side is one of the valid developments inside the Floodway to other authorities/agencies and private companies.

Therefore, when comparing the location of Floodgates, the following two alternatives can be cited:

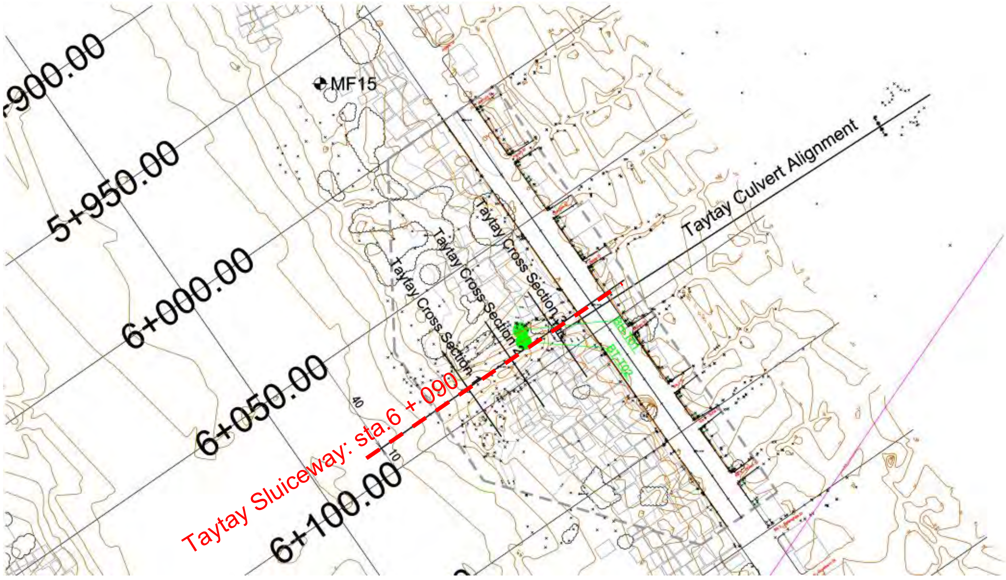
Alternatives	Contents
Alternative1: Inside of the dike from the existing dike	✓ The East Bank Road will cross the waterway outside the dike, and a bridge will be required. ✓ The current East Bank Road area needs to be excavated for construction, so it is necessary to move houses that should not be moved.
Alternative 2: existing dike position	✓ It is necessary to cut the East Bank Road during construction.

In the case of "Alternative 1: Land side of the Existing Dike Alignment", a new bridge is required to cross the connecting water channel from Taytay Creek, and houses in the landside of the east bank need to be relocated. As a result, the number of houses to be relocated will increase which may affect the progress of the project, and construction costs will also increase. There is no advantage in choosing Alternative 1.

Accordingly, "Alternative 2: Same location as the Existing Dike", is recommended.

(2) Cross Sectional Location

The position where the center line of the Floodgate is approximately located on the center of the existing box culvert is the base and centerline of the Floodgate aligned with Sta.+090 of the Manggahan Floodway.



Source: Study Team

Figure 6.4.73 Location of Taytay Sluiceway Gate

(3) Longitudinal Direction

As to the Floodgate type, the column and main body shall be contained from the shoulder of the dike up to the intersection between the side slope of the dike and the DFL of Manggahan Floodway.

However, from the results of the study described below, the sluiceway connecting with the existing box culvert is recommended as the structural type of the Taytay Sluiceway. As to sluiceway type, the longitudinal position is set as described below.

1) Position of the Outlet of Box Culvert

The longitudinal position of the outlet of the box culvert is set by the position of the riverside slope and the breast wall, maintaining the alignment of the existing dike.

2) Joint between the Existing and New Box Culverts

Since the thickness of dike on the box culvert changes at the alignment of the dike (riverside shoulder of dike), the residual settlement of the subgrade of the box culvert changes. Considering the above, the joint between the existing and new box culvert is set based on the position of alignment of the dike.

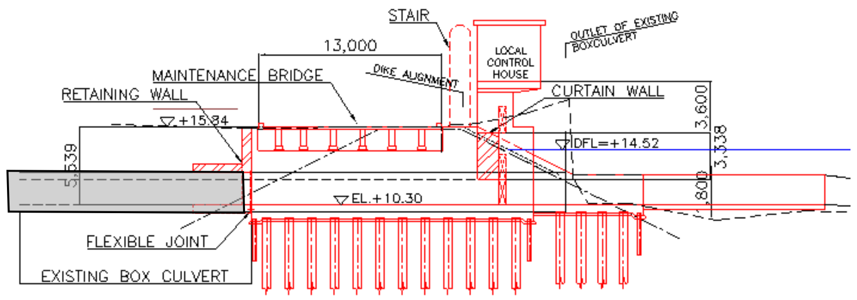
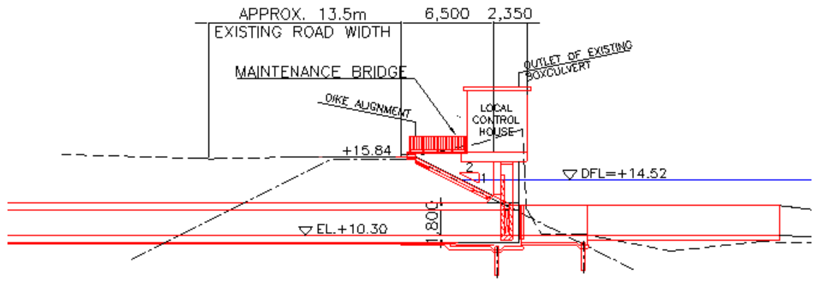
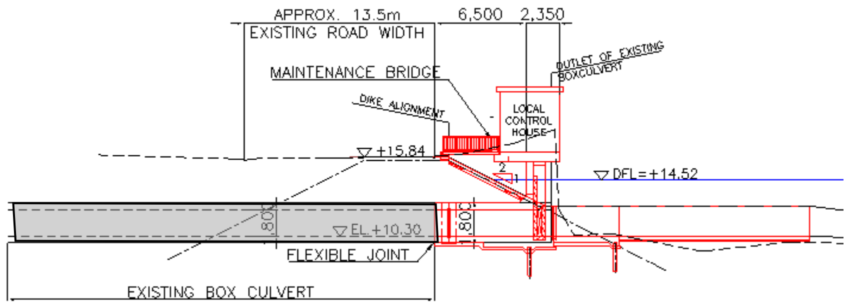
6.4.4.7 Type of Structure

In the Taytay Sluiceway, while the bottom elevation of the gate fully opened is EL.+12.4m (=Invert Elevation: EL.+10.6m+height of gate: 1.8m), the top elevation of the existing dike is EL.+17.0m. When the height of the gate is the same as that of the dike, the weight of the gate becomes heavier and the capacity of its hoist and thickness of the concrete structure also become large. Hence, it is better to install a curtain wall in the case of Cainta Floodgate.

Although the height of the curtain wall becomes more than 5 m, the height of the gate is only 1.8 m. In such a case, since there is enough thickness of dike at the top of box culvert, the sluiceway type can also be considered.

Accordingly, types of structures such as Floodgate and sluiceway were compared, as shown in **Table 6.4.48** and from the aspect of structure, maintainability and construction, the sluiceway type is chosen. “Alternative 2-b: Sluiceway Type (Connecting with the Existing One)” is recommended to minimize the number of houses to be relocated in the project.

Table 6.4.48 Comparison of Types of Structures

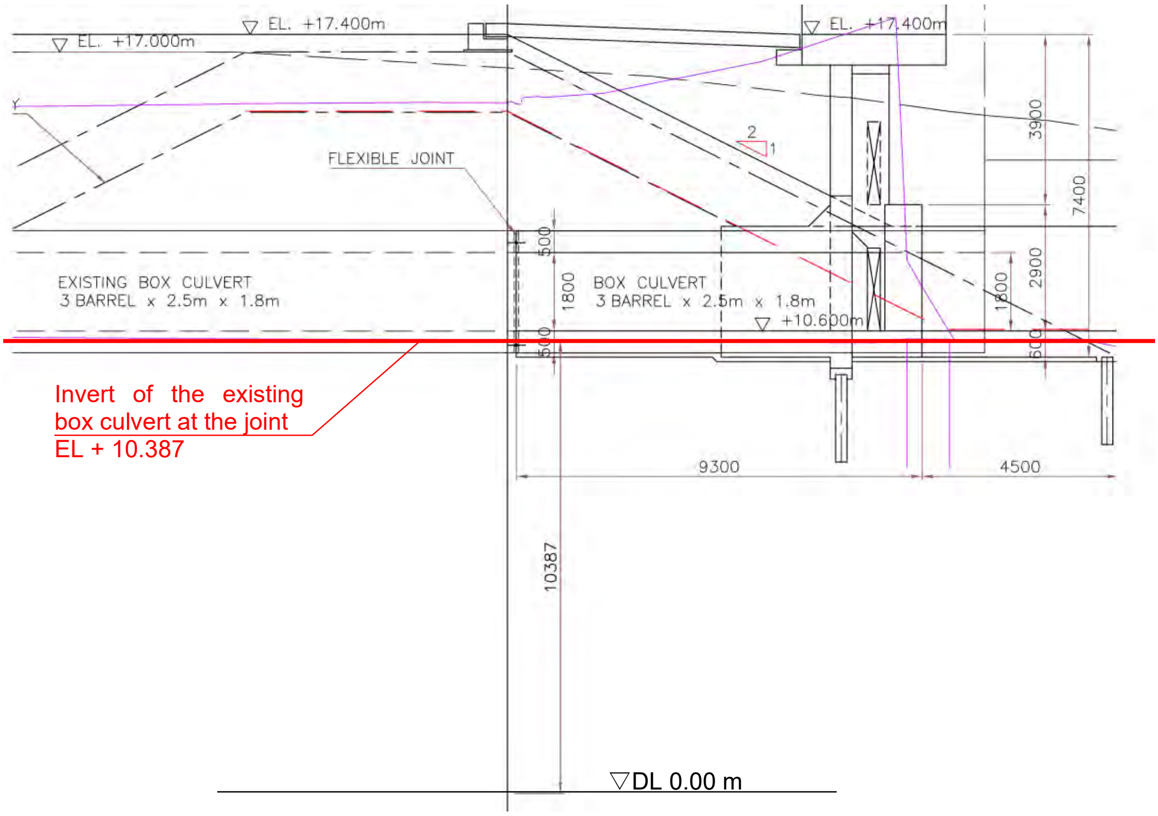
	Alternative 1: Floodgate Type (Connecting with the Existing One)		Alternative 2-a: Sluiceway Type (with Total Rehabilitation)		Alternative 2-b: Sluiceway Type (Connecting with the Existing One)	
Figure						
Specification	· Type of Structure: Floodgate · Design Discharge Allocated: 30m³/s (Existing: 28.5 m³/s) · Pier: Alloy-Saving Stainless Steel · Roller Gate: W8.5m x H1.8m x 1 Gate · Curtain Wall: W8.5m x H4.6m · Hoist: Wire Rope Winch Type · Foundation: Pile L≒30m		· Type of Structure: Sluiceway · Design Discharge Allocated: 30m³/s (The Existing: 28.5 m³/s) · Pier: Alloy-Saving Stainless Steel · Roller Gate: W2.5m x H1.8m x 3 Gate · Hoist: Electoral Rack Type · Foundation: Spread Foundation (Flexible)		· Type of Structure: Sluiceway Connecting to the Existing One · Design Discharge Allocated: 30m³/s (The Existing: 28.5 m³/s) · Pier: Alloy-Saving Stainless Steel · Roller Gate W2.5m x H1.8m x 3 Gate · Hoist: Electoral Rack Type · Foundation: Spread Foundation (Flexible)	
Hydraulics	· Design discharge allocated is based on the capacity of the existing box culvert. Hence, it will not be affected by the raising of design scale of the drainage plan in land side.	○	· The existing box culvert is totally rebuilt. Hence, it is possible to raise the design discharge of the proposed sluiceway in accordance with the update of the drainage plan in the land side. (It will also cause more socio-environmental issues.	○	· Same as Alternative 1.	○
Influence of Debris and Garbage	· Due to its only one span, less blocking by debris and garbage flowing on the river will be expected.	◎	· Since this has the same 3 barrels as the existing box culvert, the situation will not change.	○	· Same as the Left	○
Structure	· Curtain wall is high. · The length of foundation pile is about 30 m. · This type needs a bridge both for maintenance and public highway. · Retaining wall is also needed to reduce the number of houses to be relocated.	△	· Since the height of dike is about 5m above the top of box culvert, there is no structural issue. · This can be an option when the existing structure has critical structural damage.	◎	· Same as the Left · Gap of the residual settlement between the existing and new culverts is expected. Hence, countermeasures for the settlement and flow area after the settlement shall be carefully studied.	○
Maintainability (After Earthquake)	· In case piles of the Floodgate are damaged due to earthquake, total rehabilitation with cutting and opening the existing dike is needed.	○	· Flexible sluiceways can follow the ground deformation, and less damage to structures due to stress concentration is expected even in earthquakes · Since the major damage is deformation and gap of joints, repairing is easier than the Floodgate type.	◎	· The new portion has the same advantage as Alternative 2. · The structural difference between the new and existing portion may cause damage on the joints and the body.	◎
Construction	· Installation of the foundation piles takes time. · Piling work with avoiding the position of existing piles is difficult and needs more time.	△	· Based on the previous borehole log, a liquefaction layer was not confirmed. Hence, large-scale countermeasure such as soil stabilization up to the diluvial formation is not needed. (This is to be verified again after the geotechnical investigation).	○	· Same as the Left	○
Operation and Maintenance	· Easier since there is only 1 gate.	◎	· Since this type has 3 gates, it becomes more complicated.	○	· Same as the Left	○
Socio-Environment	· Relocation of houses in the landside of East Bank Road is needed.	2	· A lot of houses up to the area around the intake of the existing box culvert need to be relocated and this will affect the progress of the project.	3	· Relocation of houses in the land side of East Bank Road is not needed.	1
Cost	· This alternative needs a bridge and the scale of the gate and civil structure becomes large. Hence, the Cost is higher than Alternative 3.	2	· This alternative need total replacement of the existing box culvert which is 145 m in length, and the cost is the highest.	3	· The cost is lower than Alternative 1. (Large-scale countermeasures for the existing ground are not needed based on the existing borehole log. (This concept will be verified again after the geotechnical investigation.)	1
Evaluation	This alternative has a disadvantage in the structural aspect, maintainability and construction.		From the structural aspect, maintainability and construction, this alternative is superior to Alternative 1. However, a lot of houses need to be relocated and this will significantly affect the progress of the project.		Relocation of the houses in the landside of East bank Road is not needed. From the structural aspect, maintainability and construction, this alternative is superior to Alternative 1.	
	Not Recommended		An Option in Case the Existing Box Culvert has Critical Damage		Recommended	

Legend : ◎...Better、○...No Problem、△...It has issues to be solved.
Source: Study Team

6.4.4.8 Study on Basic Structural Specifications

(1) Invert Elevation

The invert elevation of the extension sluiceway is set using the survey result of the existing box culvert. The invert elevation of the existing box culvert at the joint is EL+10.387m. Thus, the height of the connecting sluiceway is set to EL+10.3 m.



Source: Study Team

Figure 6.4.74 Invert of the Box Culvert at the Joint

(2) Span and Span Allocation

The span length and its allocation shall be the same as the existing box culvert, and the width shall be 2.5 m x 1.8 m x 3 barrels.

(3) Inside Height of the Box Culvert

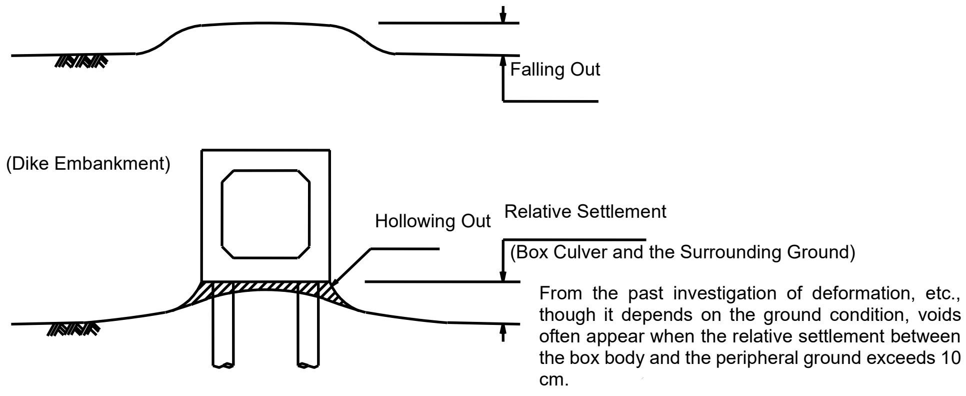
Inside height of the box culvert is set at 1.8 m..

(4) Box Structure Type

According to the "Guideline for Flexible Sluiceway", when the residual settlement exceeds 5cm, a flexible type that can follow the ground settlement shall be adopted, since a rigid type such as a pile foundation cannot cope with it. Specifically, in the case of the type using pile foundations, the box culvert does not follow the settlement around the sluiceway. As shown in **Figure 6.4.75**, there will be voids between the bottom of the sluiceway and the subgrade, and this situation gives an adverse effect on the dike.

The foundation ground of the Taytay Floodway site is assumed to be a cohesive soil layer, although there is no thick soft layer. Therefore, residual settlement of 5 cm or more can be expected. This design adopts a flexible type that can follow the settlement of the ground or foundation without using piles, etc., and it does not adversely affect the dike.

In Chapter 7, the residual settlement is calculated, and the result is 7.6 cm. Hence, the validity to adopt the flexible type has been confirmed.



Source: Guideline for Flexible Sluiceway

Figure 6.4.75 Hollowing Phenomenon under the Bottom Slab of Box Culvert with Pile Foundations

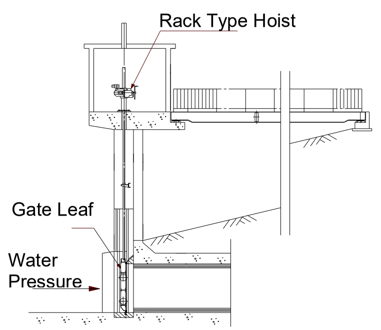
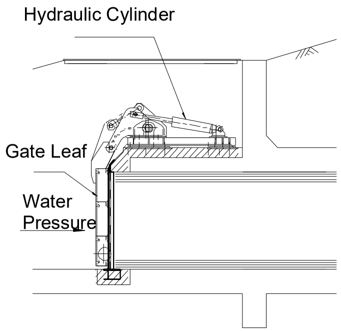
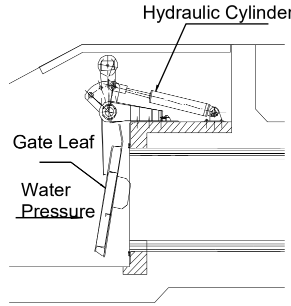
(5) Type of Gate

The following three gate types are adoptable. Flap gates are not recommended considering the size of the gate of the Taytay Sluiceway and the social impact of inundation in the land side of dike caused by backflow through the Taytay Creek due to the incomplete closure of gates. Hence, comparative study shall be conducted among the types below that are superior to a flap gate in terms of reliability of closure as follows:

- Fixed Wheel Roller Gate
- Hydraulic Operated Link Mechanism Gate (without Column)
- Flap gate with Balanced Weight (without Column)

The comparison table of the gate type is shown in **Table 6.4.49**. From the point of view of local site condition, the Fixed Wheel Roller Gate is recommended, since it is least affected by garbage and water plants with enough reliability. There is a large amount of floating garbage and water plants, and a large amount of aquatic plants flow in, covering the water surface when the flow rate is small like in the dry season.

Table 6.4.49 Comparison of Gate Types

	Alternative 1: Fixed Wheel Roller Gate (w/ Column)		Alternative 2: Hydraulic Operated Link Mechanism Gate (w/o Column)		Alternative 3: Flap gate with Balanced Weigh (w/o Column)	
Figure						
General	· This is a type of gate in which the roller moves up and down along the gate groove of the column and is opened and closed by a rack-type hoist on the operation deck on the column.		· This is a type of gate without column in which an opening/closing operation of the gate leaf is performed by a hydraulic hoisting device (Piping is installed on the side slope of dike.). The upper half is operated by rotating motion and the lower half is operated by lifting/lowering motion.		· This is a type of gate without column that automatically opens and closes without power by the rotational movement of the gate leaf in accordance with the difference of water level inside and outside with the effect of the balance weight attached to the upper part of the gate leaf and the float on the back.	
Influence due to Garbage and water plant	· Since the gate leaf is opened and closed by lifting it, garbage and water plants are hardly caught in the gate leaf when closed, and the number of actual samples are enough.	◎	· Since the lower half of the gate leaf is opened and closed by a lifting system, garbage and water plants are less likely to be caught by the gate compared with Alternative 3.	○	· Since the gate opens and closes by rotating motion, garbage and water plants are easily caught in the gate when the gate is closed.	△
Response in Case of Failure	· Self-weight lowering is possible · If the motor fails, it can be manually operated.	○	· Self-weight lowering is possible. · It can be opened and closed without power due to the difference in water level, because it is in a flap state after the gate is lowered. · When the motor is out of order, it can be manually operated with oil pressure.	○	· It can always be opened and closed without power by the water level difference. · Hydraulic or manual operation can be performed.	○
Maintainability	· Good maintainability, since it is always accessible above the gate	○	· Gate and hoist are not accessible when water level is high	△	· Same as the left column	△
maintenance	· It needs replacement due to abrasion, oil supply and adjustment for each part of the hoist. · Due to wear of water sealing rubber, the service life is shorter than the other types.	3	· Each part of the hoist needs replacement due to abrasion, oil supply and adjustment. · Long service life because water sealing rubber and guide frame do not slide during opening and closing operation.	2	· Since the main power is automatic, there are fewer parts to be inspected every year and replaced than other the alternatives. · Long service life because water sealing rubber and guide frame do not slide during opening and closing operation.	1
economy (Three Gates)	***** PHP (1.0) *Include a Maintech Bridge	1	***** PHP (1.98) *Including Cover and Handrails	3	***** PHP (1.56) * Including Cover and Handrails	2
Evaluation	The most reliable type considering the current site condition where there is a lot of garbage and water plants. Most economical type Recommended		It is inferior to Alternative 1 in terms of influence of garbage and water plants and is also inferior in terms of maintainability.		Although it is superior in maintenance and operation, it is not suitable for use in a place where there are a lot of garbage and water plants.	

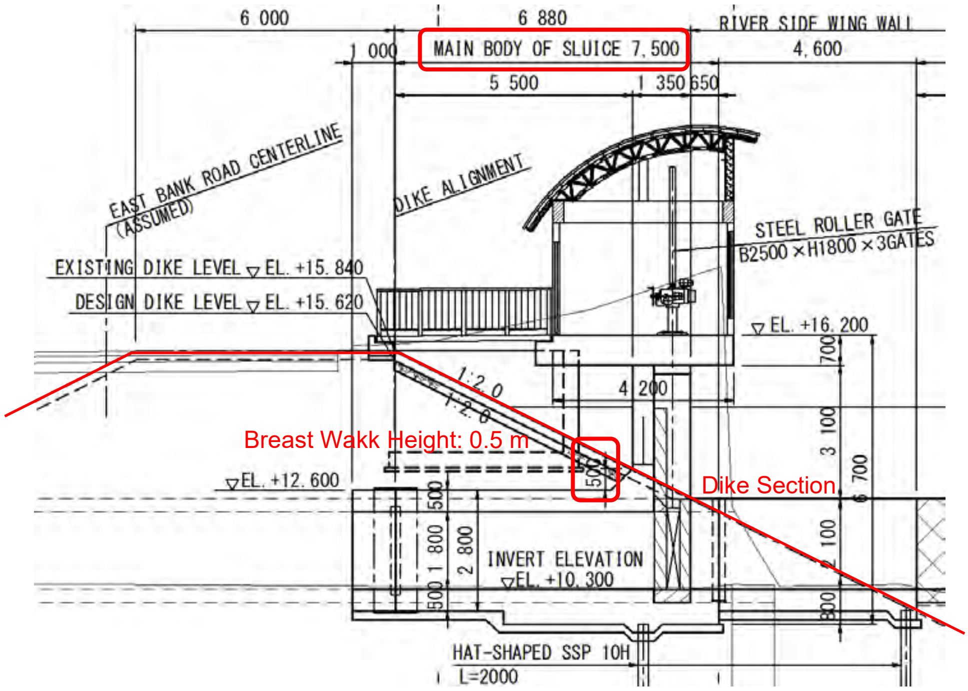
Note: Cost is not presented due to the prior released version.
Legend: ◎... Better ○... No problem, △... There are issues to be solved.
Source: Study Team

(6) Determination of Main Body Specifications (Section Dimensions)

1) Length of Box Culvert

Since it may cause water leakage, It should be avoided that the main body significantly cut in the shape of the dike. Therefore, as a general rule, the main body length shall be set to reach the slope of the river side and the toe of the slope in the land side of the dike.

The length of the main body is 7.5 m when the breast wall in the river side is set to 0.5 m to minimize the cut the cross section shape of the dike.



Source: Study Team

Figure 6.4.76 Length of Sluiceway

2) Span Allocation

The maximum length of 1 span is set to 20 m in "Guideline For Flexible Sluiceway, Japan". Since the total length of the Taytay Sluiceway is 7.5 m and less than 20 m, only 1 span is adopted.

3) Seepage Cut-off wall

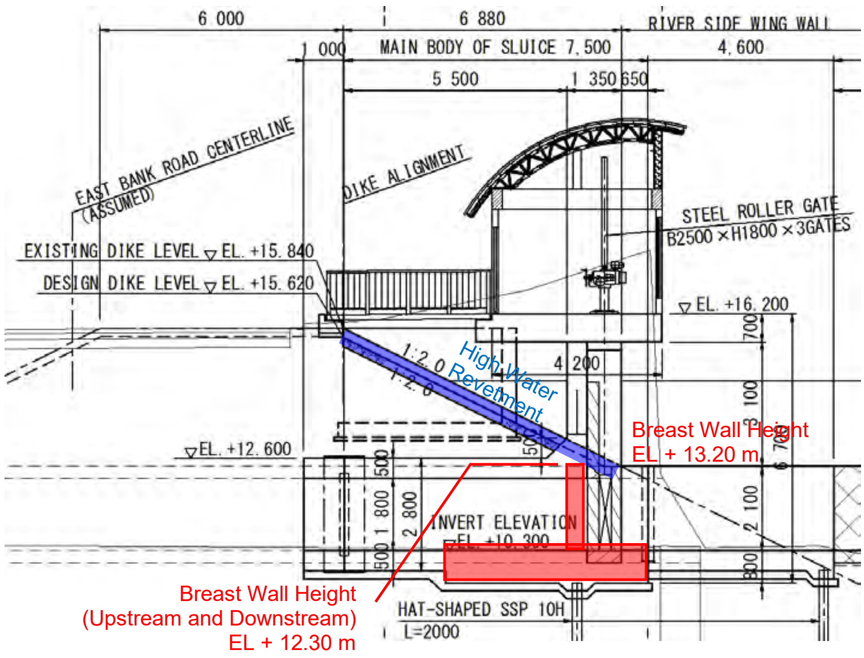
The cut-off wall is installed to prevent piping around the box due to seepage flow. One or two locations are provided near the center of the main body of the conduit pipe.

In this study, the length of the extension sluiceway is as short as 7.5 m, and the existing culvert has 145 m length, and it is excavated river channel considering the ground elevation in the land side. The piping phenomenon is not expected. Hence, the seepage cut-off wall is unnecessary.

4) Breast Wall

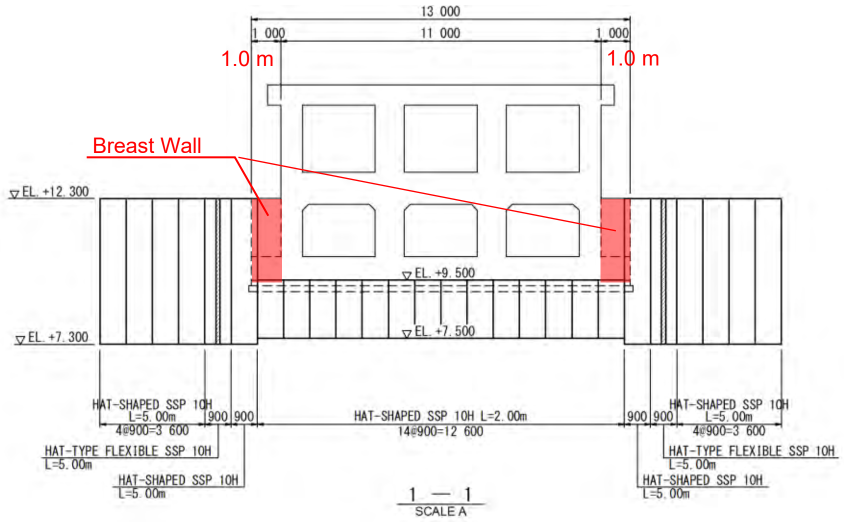
Breast wall is an integral structure with the main body aiming at preventing the movement or suction of particles at the contact portion between the body and the dike.

The crown height is set to EL + 13.20 m, which minimizes the cut in the cross section shape of the dike. The crown height of the overhanging part on the main body would be EL + 12.30 m considering the relation with the high-water revetment. The breast wall width is set to 1.0 m.



Source: Study Team

Figure 6.4.77 Crown Height of Breast wall

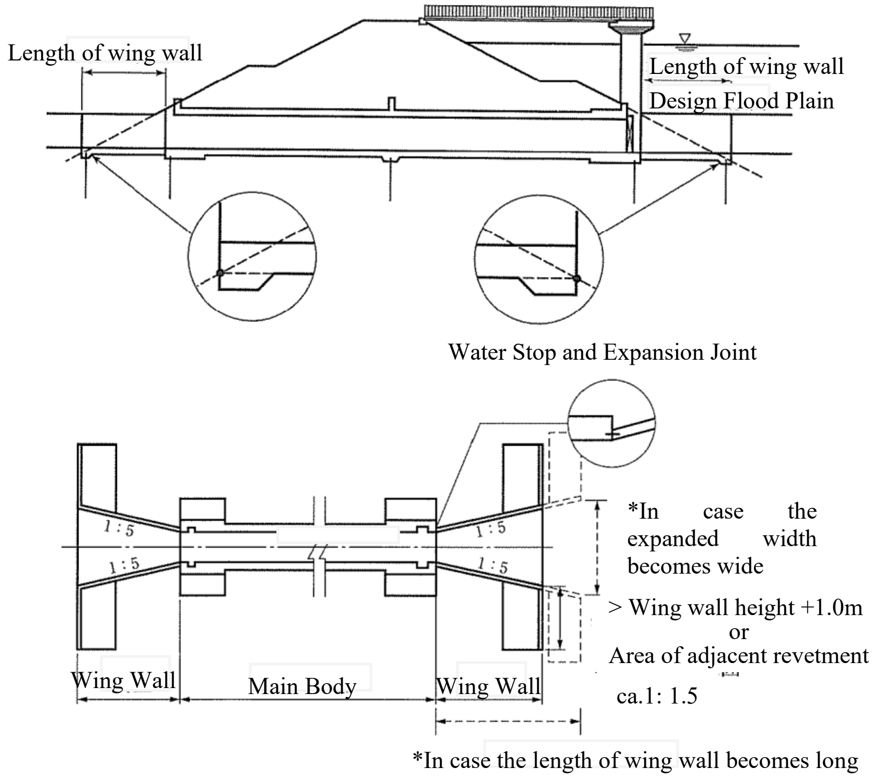


Source: Study Team

Figure 6.4.78 Width of Breast Wall

5) Wing Wall

The wing wall shall be designed as an independent structure separated from the main body. The concept of dimension settings is shown in **Figure 6.4.79**. The Taytay Sluiceway is the extended sluiceway, and only the riverside is arranged.

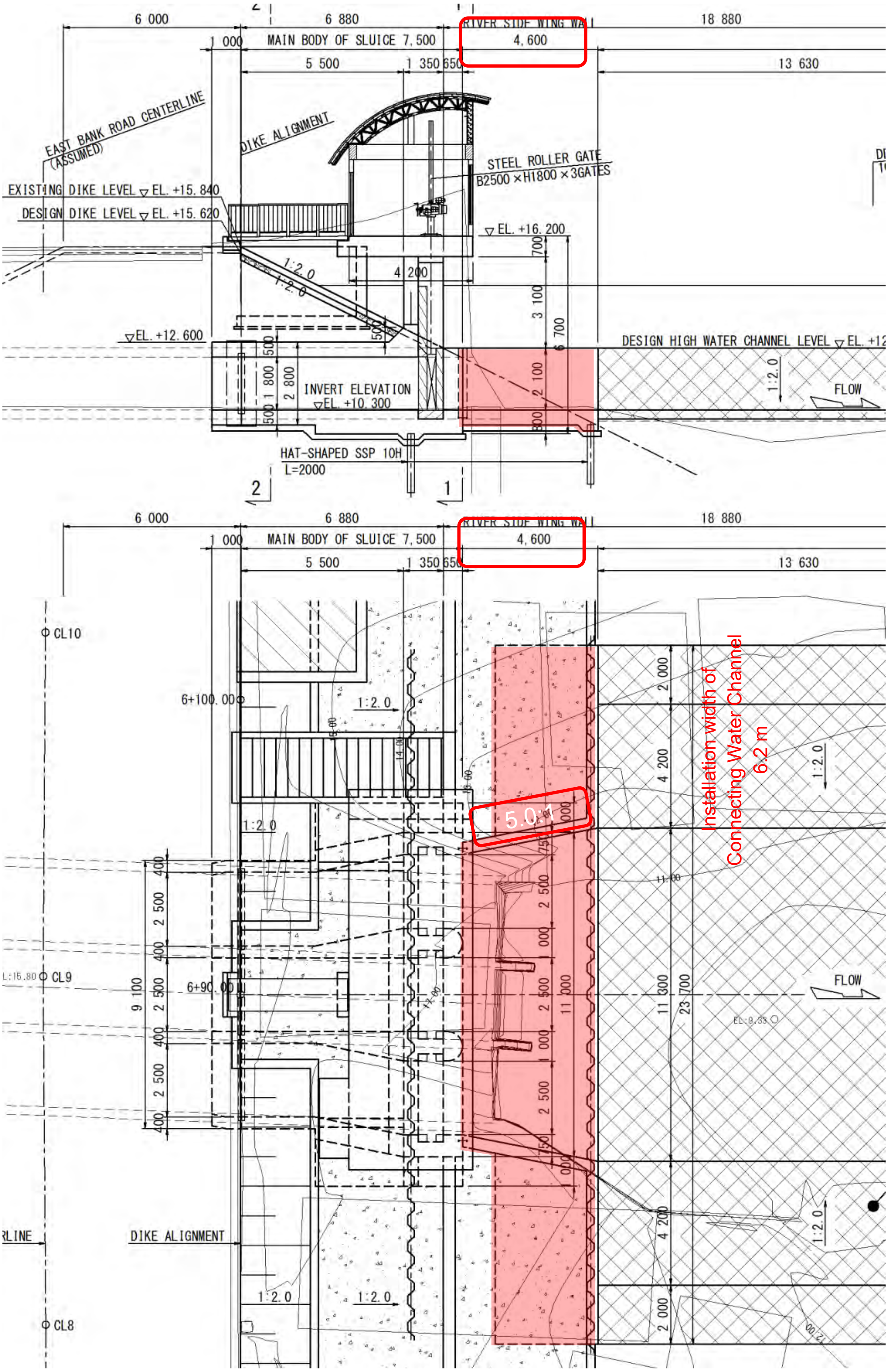


Source: Guideline for Flexible Sluiceway

Figure 6.4.79 Concept of Wing Wall Length And Layout

The major dimensions of the wing wall are shown in **Figure 6.4.80** and these are determined based on the following concept.

- The length of the wing wall is set to 4.6 m based on the intersection of the cross-section shape of the dike and the bottom slab of wing wall.
- The plane shape of the side wall of the wing wall is made to expand at 5.0: 1.
- The length of the end section of wing wall in the is set considering the shape of connecting water channel.



Source: Study Team

Figure 6.4.80 Wing Wall of the River Side

6) Seepage Control Work

Seepage control works are installed to reduce the hydraulic gradient of seepage water due to the difference in water level between inside and outside, and to prevent soil and sand from being sucked out by soil and sand flow and scouring. When the total box length is 145 m including the existing box culvert, the creep ratio is calculated as follows:

$$C = \frac{L/3 + \sum l}{\Delta H}$$

Where.

C : Creep Ratio

L : Box Length (= 145 m)

$\sum l$: Contact Length between Ground and Structure in the transverse direction to the box culvert (Set to 0)

ΔH : Water Level Difference (= 14.52 – 10.6 = 3.92 m)

$$C = \frac{145/3 + 0}{3.920} = 12.3 \quad >8.5 \text{ (very fine sand or silt)}$$

As described above, even when the seepage cut-off wall is not provided in the newly installed box body, enough creep length can be ensured in calculation. Therefore, the minimum length of seepage control works necessary for the installation of sluiceway would be installed.

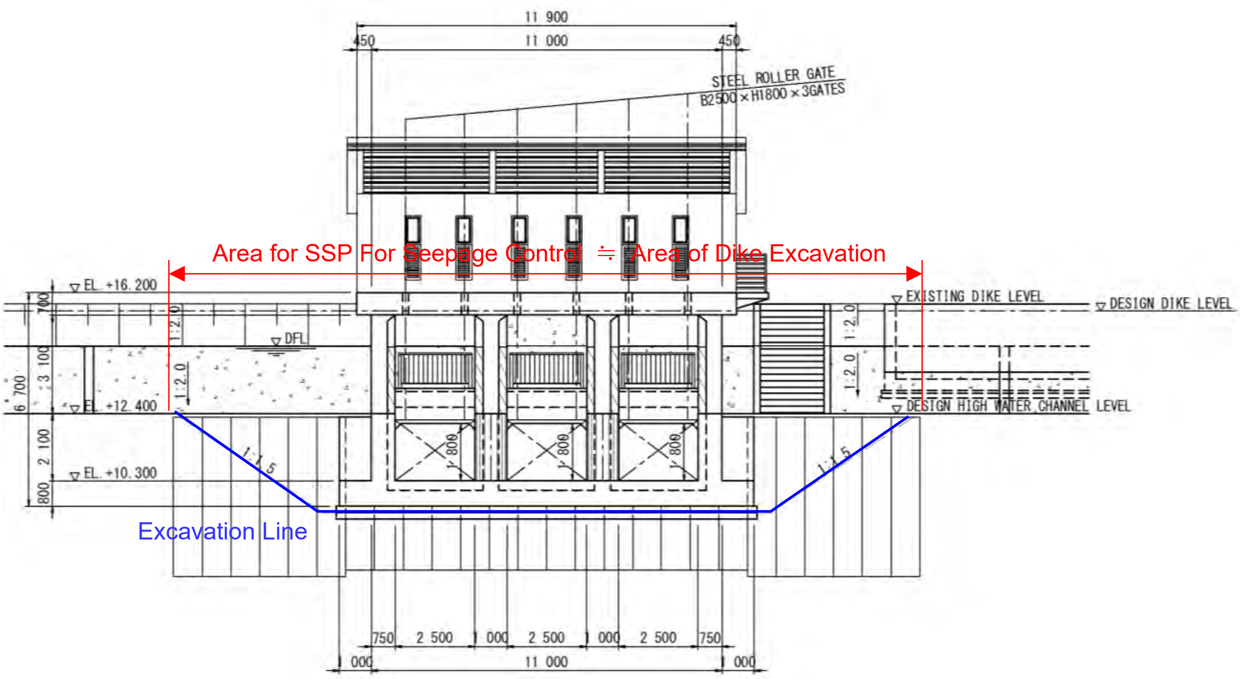
(a) Vertical Direction

Considering the previous borehole log that the bedrock layer does not appear below the subgrade of the sluiceway, 2.0 m seepage cut-off wall using steel sheet pile is installed.

(b) Horizontal

Since the Taytay Sluiceway is located along the excavated water channel part, and it will connect with the existing box culvert which has 145 m length, the hydraulic gradient is very small, and the safety against the movement of soil particles is not needed to concern. On the other hand, in order to install Taytay Sluiceway, the existing dike will be excavated and embanked again. Since the embankment area is artificially compacted, it is inferior to the natural ground in terms of the safety against the movement of soil particles immediately after construction. Hence, steel sheet pile for seepage control would be installed to cover the dike excavation area(See **Figure 6.4.81**)..

About the type of SSP, since the width of 1 piece is wide and it is more economical, Hat-shaped steel sheet pile Type -10H would be applied. The length of SSP is set to 2.0 m which is the minimum length.



Source: Study Team

Figure 6.4.81 Relationship between Dike Excavation and Seepage Control Works

7) Column

When he top elevation of the column is determined, It is necessary to consider the aspect of flood control, maintenance and manageability of the gate, etc. In the “Guideline for Flexible Sluiceway”, it is mentioned "As for the top of the column, the height below the maintenance bridge girder should be higher than the design dike height, and the height of the bottom surface of operation deck shall be determined considering the operation of the gate and the management of the gate is determined."

The upper surface of the operation deck is set to EL + 16.20 m based on the following arrangement.

The top of operation deck would be determined from the higher value obtained from the following methods.

(A) The bottom surface of girder of the maintenance bridge shall be higher than the design dike crown.

(B) The bottom surface of operation deck (including haunch) is set from the higher value obtained from the followings.

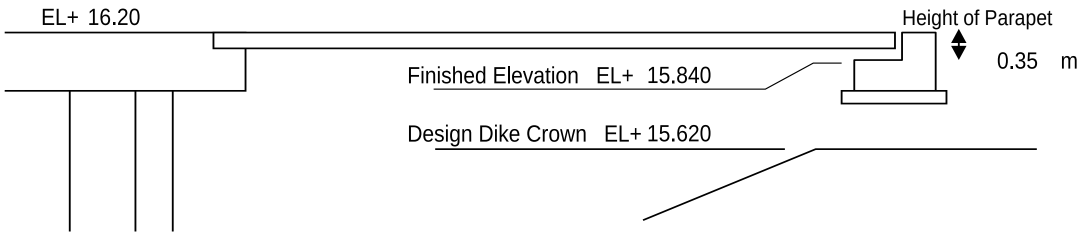
① {Bottom of Gate(Fully Opened)}+(Height of Gate)+(Allowance for full open)
※Allowance for full open is 50 cm or more.

② (Top of Guide Frame) +(Height of Gate)+(Allowance for installation and removal)
※Allowance for Removal is 50 cm or more.

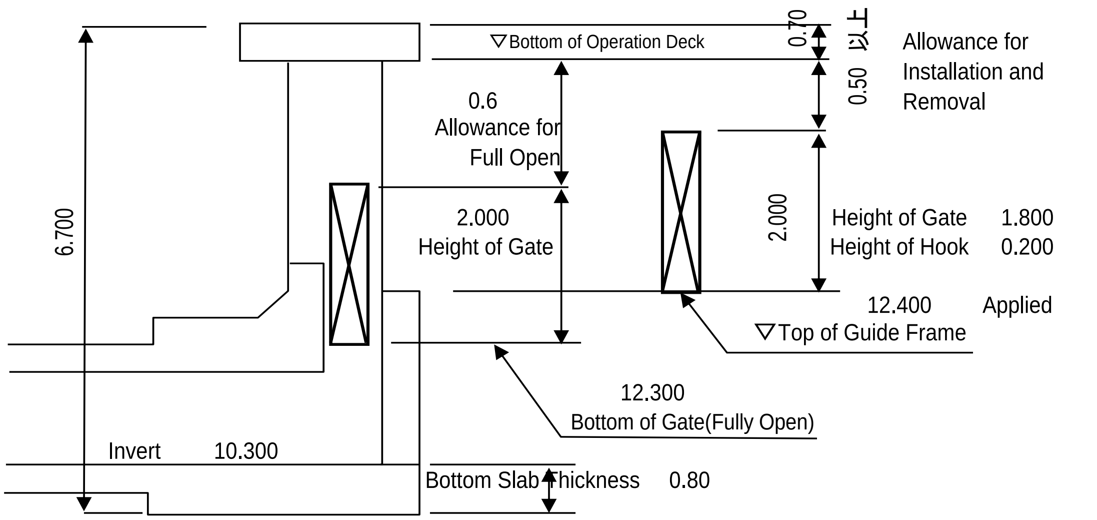
A Method

Top of Operation Deck＝ Top of Maintenance Bridge Surface ＝ Finished Elevation of Dike + Height of Parapet of Abutment

＝	15.840	+	0.350	＝	16.200	m
	Existing Dike Level		Height of Parapet			



B Method



Top of Operation Deck＝ Top of Guide Frame + Height of Gate + Allowance for Installation/Removal +Thichness of Operation Deck

＝	12.400	+	2.000	+	0.500	+	0.700	
	Top of Guide Frame		Height of Gate		Allowance		Thickness of Operation Deck	
＝	15.600	m	<	16.200	m			

From the above, A Method is applied and the top of operation deck is se 16.200 .

Bottom of Column＝	10.300	-	0.800	＝	9.500	m
	Invert		Thickness of bottom Slab			
Total height of Column	16.200	-	9.500	＝	6.700	m
			Top of Operation Deck	＝	16.200	m

6.4.4.9 Study on Local Control House

(1) Conditions for Studying Local Control House

1) Study of Local Control House

(a) Conditions of the local control house

As explained in design of MCGS local control house , required clearance for inspection and maintenance works are provided by type of equipment as follows.

- ✓ Clearance around hoisting machine: 80 cm
- ✓ Clearance around other mechanical equipment: 60 cm
- ✓ Clearance in front of control panel: 100 cm
- ✓ Clearance between control panels: 120 cm
- ✓ Clearance at side or back of control panels: 20 cm

The major equipment installed in the local control house of Taytay sluiceway is shown in the table below.

Table 6.4.50 Major Equipment in Taytay Sluicewaye Local Control House

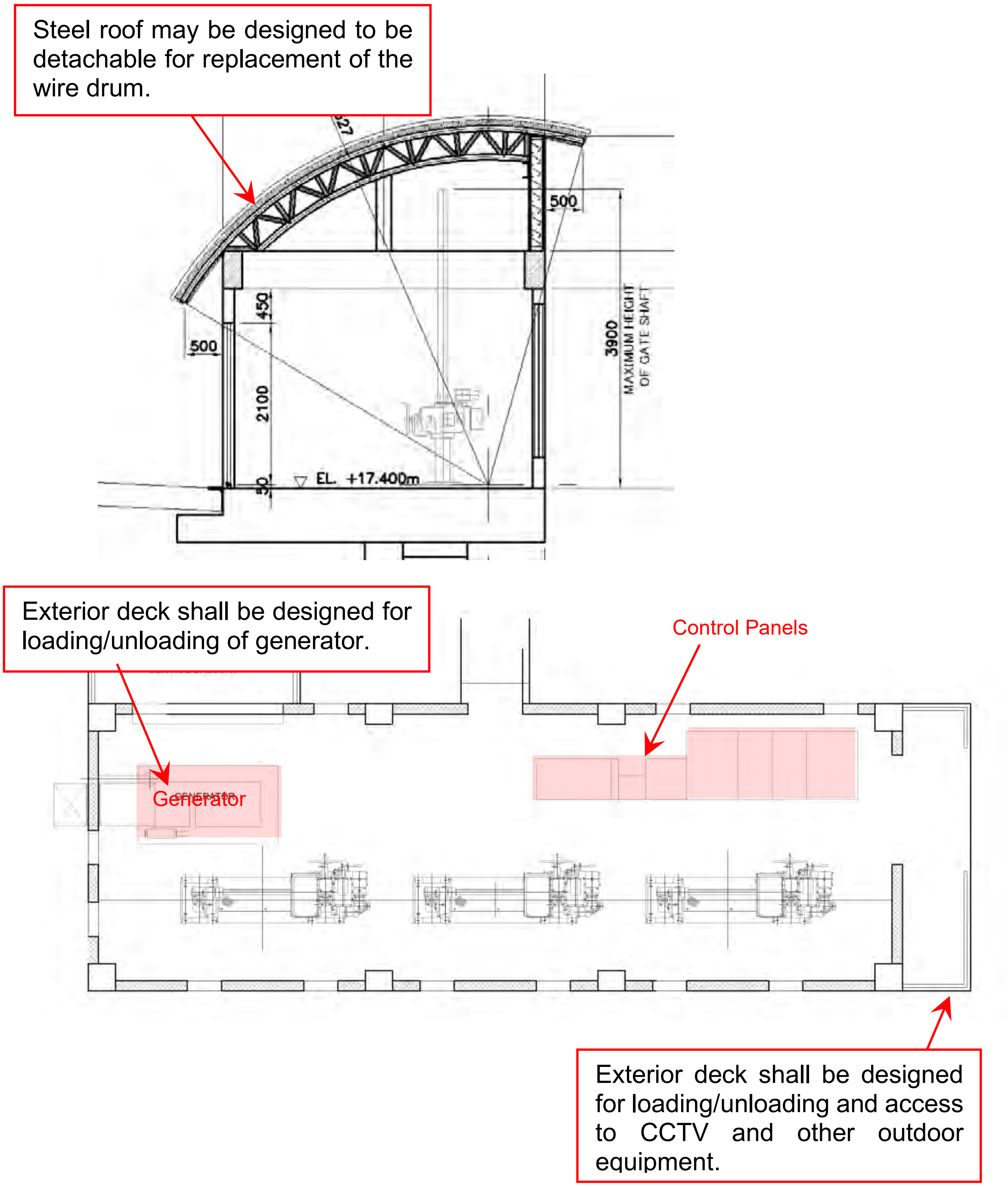
Name	Equipment	Remarks
Local Control House	Receiving Panel	
	Main Distribution Board	
	Uninterruptible Power Supply	
	Lightning Protection Transformer	
	Remote Control System Control Panel	
	Water Level Gauge Control Panel	
	Monitoring Camera Control Panel	
	Simplified Gate Operation Panel	
	Generator	

In addition, the following conditions shall be considered for ease of maintenance.

- ✓ Steel slide doors and exterior decks are provided for loading and unloading of heavy equipment by crane from the ground.
- ✓ Outdoor spotlights, ITV cameras and speakers shall be accessible from the exterior deck.
- ✓ Steel roof may be designed to be detachable for replacement of the wire drum, which weighs more than 5 tons, instead of installation of roof mounted hoisting crane in the local control house.
- ✓ The ceiling height shall be decided in consideration of necessary vertical clearance for inspection and maintenance of winch drum. Also, clearance of 50 cm or more shall be secured between RC beam and steel door.
- ✓ The maintenance stairs may not be shall comply the requirement by NCBP comply with requirement specified in the local building standards.

(b) Plan and cross-sectional plan of local control house and gate control room

Based on the conditions summarized in the previous page, the plan and cross section of the local control house are shown below



Source: Study Team

Figure 6.4.82 Plan and Section of Cainta Flood Gate Local Control House

6.4.4.10 Study on Gate Structure and Hoist

(1) Material of Gate

1) Possibility of Brackish Water

(a) Results of Previous Water Quality Test

Table 6.4.28 show the results of the past water quality test carried out in the floodway of Laguna Lake as part of the study on gate material for the Cainta Floodgate. According to the results, no brackish water was observed in the Manggahan Floodway.

In Laguna Lake, on the other hand, brackish water has low values and it is not observed every year. **Table 6.4.28** shows that the condition of brackish water in the Laguna Lake is lower than that of the Marikina River.

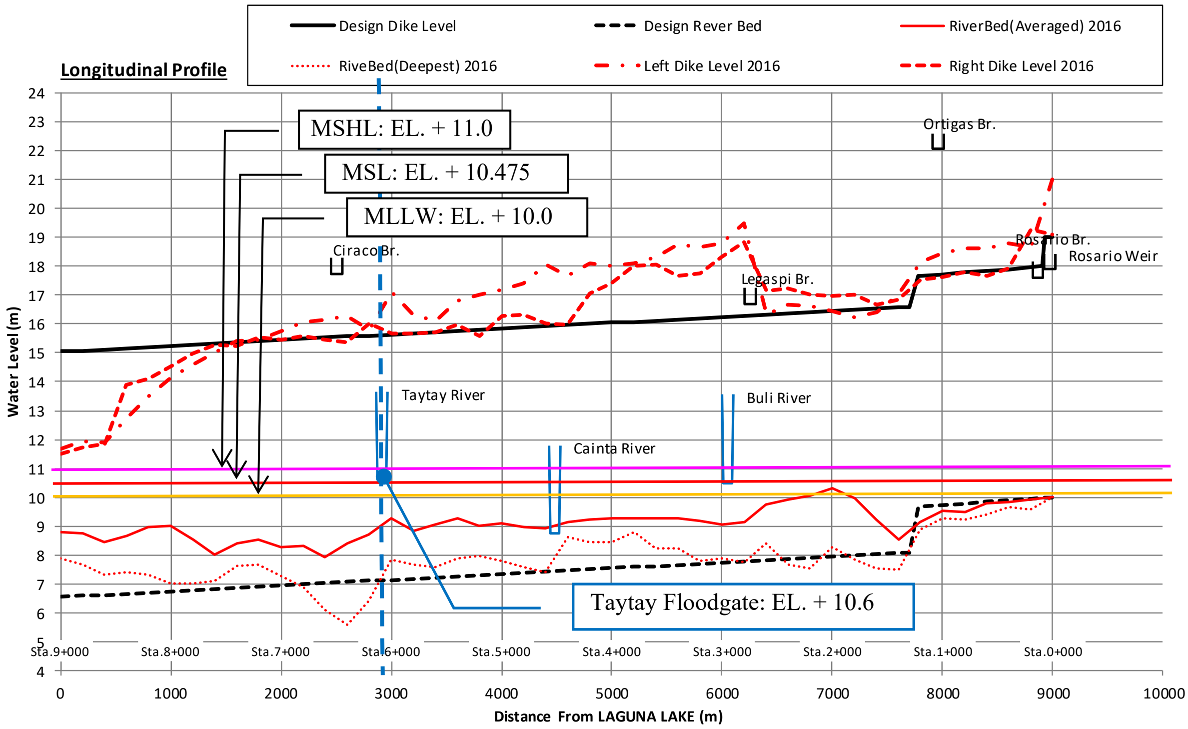
(b) Past Observed Water Level

The water level in the Laguna Lake in the recent 20 years for the study on gate materials for the Cainta Floodgate is as shown in **Figure 6.4.38**. From the Mean Sea Level of EL.+10.5 m, Mean Sea High Water Level of EL+11.0 m, and the Mean Lower Low Water Level of EL+10.0 m in the Manila Bay, it is estimated that the backflow (saltwater intrusion) from the Manila Bay to the Laguna Lake occurs during February and June (mainly in the dry season) when water in the Laguna Lake is low.

(c) Relationship between the Riverbed and Sea Water Level

Figure 6.4.83 shows the riverbed and the Mean Sea High Water Level (MSHL), Mean Sea Level (MSL) and Mean Lower Low Water Level in the Manila Bay.

Since the completion of the floodway, sediment has accumulated on the riverbed and the riverbank. Once the MCGS is installed, and the flow distribution based on the plan is achieved, periodic riverbed excavation in the Manggahan Floodway is required to maintain its flow capacity. The proposed riverbed of the floodway and the invert elevation of the Taytay Floodgate are the Mean Sea High Water Level (MSHL) in the Manila Bay, but higher than the Mean Sea Level (MSL) and the Mean Lower Low Water Level (MLLW). Even if the riverbed height of the floodway is maintained and salt run-up occurs, the condition of brackish water may be milder than that of the Cainta Floodgate.



Source: Study Team

Figure 6.4.83 Relationship between Riverbed and Sea Water Level

(d) Summary

The situation described above can be summarized as follows:

- Brackish water has not been observed based on the results of water quality tests conducted with samples taken near the water surface around the Taytay Sluiceway site. The Laguna Lake, on the other hand, is brackish once in every few years. The comparison of salinity level between the Laguna Lake and the Marikina River suggests that the condition of brackish water of the Laguna Lake is less severe than that of the Marikina River.
- After the installation of the MCGS and even if the Floodway bed is maintained by regular dredging, salt run-up may still occur considering the Mean Lower Low Water Level in the Manila Bay (MLLW) and the design riverbed of the Floodway.
- A comparison of the invert elevation of EL.+10.30 m of the Taytay Sluiceway with the tide level of the Manila Bay shows that the invert elevation is approximately the same as the MSL EL.+0.475 m of the Manila Bay. It is then considered that water in the area becomes brackish depending on the season, but this will hardly affect the gate.

Likewise, as in the case of Cainta Floodgate, basically, the gates are closed only during floods, and the possibility of areas being affected by brackish water is low.

Accordingly, when selecting the gate leaf material for the Taytay Sluiceway, installing the gate in freshwater is considered.

2) Comparative Study of Gate Materials

The gate material is selected from the following three types. Stainless steel is applicable in freshwater.

- Carbon Steel (SM 400)
- Conventional stainless steel (SUS 304)
- Alloy-saving duplex stainless steel (SUS 821 L)

The comparison of economic efficiency is based on the lifecycle costs (LCC), including maintenance. Also, regarding the initial cost, the design cost by a Japanese company is considered for all materials. In addition, regarding stainless steel, gate fabrication at a factory in the Philippines is to be carried out under the supervision of a Japanese company.

From the comparison shown in **Table 6.4.51**, although the lifecycle cost is slightly higher than that of carbon steel (about 350,000 PHP, 2% difference), the alloy-saving duplex stainless steel is recommended due to less maintenance and the introduction of new technology useful for promoting industrial production in the Philippines.

Table 6.4.51 Comparison of Gate Materials for the Taytay Sluiceway

Item	Carbon Steel (SM400)	Conventional Stainless Steel (SUS304)	Alloy Saving Stainless Steel (SUS821L)
Summary	・ Material commonly used for gate leaf ・ Proof Stress: 245 N/mm^2 ・ Tensile strength: 400 ~ 510 N/mm^2	・ Stainless steel commonly used in fresh water ・ Proof Stress: 205 N/mm^2 ・ Tensile strength: 500 N/mm^2	・ Stainless steel with high strength used in freshwater ・ Proof Stress: 400 N/mm^2 ・ Tensile strength: 600 N/mm^2
Weight	Gate Leaf 5.1 t (1.0) Guide Frame 2.5 t (1.0) +Hoist 3.0 t (1.0) 10.6 t (1.0)	Gate Leaf 4,3t (0.85) Guide Frame 2.5 t (1.0) +Hoist 3.0 t (1.0) 9.8 t (0.92)	Gate Leaf 4.3 t (0.85) Guide Frame 2.5 t (1.0) +Hoist 3.0 t (1.0) 9.8 t (0.92)
		・ It is lighter than ordinary steel because no corrosion margin thickness is required.	・ Same as the left column
Maintenance	・ Generally, repainting is required at least once every 10 years. However, it is considered that repainting is carried out every year based on the past record of the Rosario Weir. ・ Painting Cost: 15,000 PHP/batch (Calculated from actual results of Rosario Weir)	・ There is no need for painting.	・ Same as the left column
LCC (For 50 Years, in Thousand PHP)	Gate Leaf ***** Guide Frame ***** Hoist ***** + Repainting ***** Total ***** (1.0)	Gate Leaf ***** Guide Frame ***** Hoist ***** + Repainting 0 Total ***** (1.02)	Gate Leaf ***** Guide Frame ***** Hoist ***** + Repainting 0 Total ***** (1.02)
Technical Novelty	・ Gate fabrication using carbon steel has been carried out in the Philippines, and there is little technological novelty.	・ Stainless steel processing is also carried out in the Philippines; however, there is no experience on its application to large gates. Hence, it is a new technology for river structures	・ Same as the left. In addition to the new technology for river structures, the processing of new materials in the Philippines is beneficial for the promotion of industrial production technology in the Philippines.
Evaluation	・ Although the lifecycle cost is slightly lower than that of Alloy-Saving Stainless Steel, maintenance work is required.	・ Although the lifecycle cost is equivalent to or slightly higher than that of carbon steel, less maintenance is expected.	・ Less maintenance. ・ The lifecycle cost is the same as that of conventional stainless steel. ・ There is a technical novelty. ・ Same kind of steel as the gate leaf of the Cainta Floodgate
			Recommended

Note: Cost is not presented due to the prior released version.
Source: Study Team

(2) Water Sealing System

1) Water Sealing System

Since this type of gate is for Floodgates with a curtain wall to prevent backflow, the rear surface is water sealing.

2) Water Sealing Form

This is a rubber water sealing type of gate with excellent water tightness and traceability that is commonly used for river gates.

3) Water Sealing Rubber Shape

The P-type is used for the side, upper and flat bottom parts. This is generally used in four-way water sealing construction.

4) Water Sealing Rubber Material

Synthetic rubber with excellent weather and oil resistance is generally used in Floodgate gates.

(3) Study on Type of Hoist

Figure 6.4.84 show the types of hoist commonly used.

Transmission System	Major Equipment of the Drive Unit	Connection with Gate Leaf	Type of Hoist
Mechanical	Drum Drive	Wire Rope	Wire Rope Winch Type
	Spindle Drive	Spindle	Spindle Type
	Rack Drive	Rack Bar	Rack Type
	Chain Drive	Chain	Chain Type
Hydraulic	Hydraulic Cylinder	Piston Rod	Hydraulic Cylinder Type
	Hydraulic Cylinder	Wire Rope	Hydraulic Cylinder Wire Rope Type
	Hydraulic Motor Drum Dive	Wire Rope	Hydraulic Motor Wire Rope Type
	Hydraulic Motor Rack Dive	Rack Bar	Hydraulic Motor Rack Wire Rope Type

Note: Translated by the Study Team from the Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

Figure 6.4.84 Types of Hoist

This figure is classified by the combination of power transmission system to the drive unit, the major equipment of the drive unit, and the connection with the gate leaf.

This facility is for backflow prevention and disaster management. It is desirable to equip a self-weight lowering function without power in order to cope with power interruptions that will likely occur during disasters. Adaptability to remote control and reserve power is also important.

Considering the above, the hoist satisfying the functional characteristics, economic efficiency, maintainability, landscape, etc., is selected with comprehensive evaluation of head, space for installation, environment, etc.

The gate belongs to small gates with a clear span of 2.5 m x effective height of 1.8 m, a gate leaf area of 4.5 m2, and a gate leaf weight of about 1.4 tons. In case of such a gate facility, the following two types are generally selected:

- Rack Type
- Spindle Type

These two types can cope with the hoisting load of about 20 kN which is required for this facility. The rack type is selected as the type of opening/closing device suitable for this facility from the following reasons:

1. The rack type has many examples and is technically established.
2. It is equipped with a self-weight lowering function as standard equipment, and it can be lowered by remote control.
3. Higher mechanical efficiency and smaller motor capacity than the spindle type.
4. Standardized for easy maintenance.

The main reasons for excluding the other types are as follows:

1. Wire winch type is used mainly for medium and large gates with large hoisting loads. It is not for a small gate such as this facility and is not applicable.
2. Chain type switching devices are used mainly for medium and large gates with large switching loads. It is not for a small gate such as this facility and is not applicable.
3. Although the hydraulic hoist is compact, a separate hydraulic unit is required, and the installation space becomes larger. Also, it is not often used in river gate facilities in Japan due to the concern about hydraulic oil leakage, the necessity of replacing a large amount of oil regularly, and the need for maintenance. Therefore, it is not adopted for this facility

6.4.4.11 System Planning

(1) Power Unit

1) Main Power Unit

There are electric motors, internal combustion engines, and manpower for opening and closing gates. Since this gate is an important facility for preventing disasters, the main power shall be an electric motor in consideration of starting reliability, opening and closing speed, stability, low failure rate, ease of maintenance and remote operability.

The motor type is a special squirrel-cage, three-phase, induction motor for Floodgates.

2) Reserve Power Plant

In the case of rack type, a mechanism for installing a spare motor is not provided as countermeasure against failure of the motor of the power source. Therefore, alternate power for opening and closing is supplied by human power.

In such a small gate (approximately less than 10 m^2 of gate leaf area), it is sufficiently possible to open and close by human power.

3) Motor (Reserve)

When the electric motor fails and needs replacement, it will take about two months for the delivery of a new one which is quite long. Hence, a spare electric motor of the same type has to be stored together with the operating motor for ready replacement.

(2) Power Supply Unit

1) Main Power Supply Unit

Commercial power supply is received in 3φ3 W AC 200V 60Hz and 1φ2 W AC 200V 60Hz directly at the control panel in the operation room and used as the operation power.

2) Standby Power Supply System

Since this is an important gate facility, a standby generator is permanently installed as backup for the power unit to ensure the reliability of gate opening and closing.

When the commercial power fails, the power necessary to open and close the gates and the power supply for the incidental water-level gauge, safety equipment, remote control equipment, building equipment, etc. is secured by the standby power generation unit.

(3) Control System

1) Local Control Panel

(a) Usage

A local control panel is installed on the operation deck for periodical maintenance and normal operation. The specific control function of each equipment is included in the local control panel. The operation is performed by push-button operation from the local control panel and by remote operation from the central control station.

(b) Operation Monitoring Item

For gate operation by hoist, in order to ensure safe operation, the local control panel shall be equipped with various monitoring signals and fault indicators as shown in **Table 6.4.52**.

Table 6.4.52 Operation Items and Control Signals

Item	Local Control Panel (Operation)	Control Signal		Remarks
		Presence or Absence of Signal	Signal Format	
Open (Rise) Operation	○	○	Continuous Output	Gate Open Command
Close (Descent) Operation	○	○	Continuous Output	Gate Closing Command
Stop	○	-		Gate Stop Command
Alarm Stop	○	-		
Lamp Test	○	-		
Emergency Stop	○	○	Pulse Signal	Gate Emergency Stop Command
Fault Reset	○	-		

Source: Study Team

Table 6.4.53 show the status of gates displayed on the local control panel and remote operation console, operation display items, and their monitoring signals.

Table 6.4.53 Gate Status, Operation and Monitoring Signals Displayed on the Local Control Console and Remote Operation Console

Item	Local Control Panel (Display)	Monitoring Signal	
		Presence or Absence of Signal	Signal Format
Power	○	○	Continuous Output
Control Power Supply	○	○	Continuous Output
Remote	○	○	Continuous Output
Machine Side	○	○	Continuous Output
Normal Operation	○	○	Continuous Output
Maintenance Operation	○	○	Continuous Output
Normal Circuit	○	○	Continuous Output
Limit of Inspection	○	○	Continuous Output
Full Open	○	○	Continuous Output
Full Close	○	○	Continuous Output

Source: Study Team

Table 6.4.54 show the items that indicate gate failure on the local control panel. Among these items, those which seriously affect operation are indicated as "Serious Failure".

Table 6.4.54 Gate Failure and Monitoring Signal Items Displayed on the Local Control Panel

Item	Local Control Panel (Display) Failure Classification		Supervisory Signal	
	Serious Failure	Light Failure	Presence or Absence of Signal	Serious Failure
3E Operation	○		○	Continuous Output
Emergency Stop	○		○	Continuous Output
Contact Welding	○		○	Continuous Output
Opening Torque	○		○	Continuous Output
Closing Torque	○		○	Continuous Output
MCCB Trip	○		○	Continuous Output
Leakage		○	○	Continuous Output

Source: Study Team

(c) Panel Configuration

The local control panel consists of a power supply unit for receiving power, an operation unit for operating a gate, a control unit for controlling operation, and a display unit for monitoring the operation status. With regard to the arrangement of equipment on the board and inside the board, equipment necessary for gate operation and monitoring are arranged on the machine side of the operation board, and the equipment not necessary for gate operation and monitoring are arranged in the board for easier and precise operation and maintenance inspection.

(d) Control Circuit

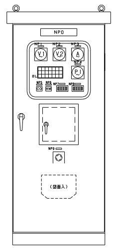
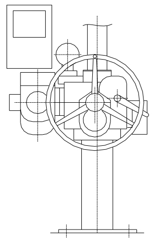
As to the configuration of the local control panel, there are the method using a contactor relay circuit and the programmable controller (PLC). The advantages and disadvantages of each circuit are the same as those shown in **Table 6.4.55** of the study for the Cainta Floodgate. As in the case of the Cainta Floodgate, the relay circuit is adopted, since it is difficult to cope with failure and there is no additional or change in operation.

(e) Type of Local Control Panel

For the local control panel of the rack type of hoist, there are the switch-mounted type and the self-standing type. The comparison of both types is shown in **Table 6.4.55**.

Since it is excellent in economy, can secure a larger space around the switchgear than the present one, can save space, and can improve maintainability, the switchgear mounting type is adopted.

Table 6.4.55 Advantage and Disadvantage of Local Control Panel Type

Item	Self-Standing Control Panel		Switch-Mounted Control Panel	
Figure				
General	The operation unit, display unit and control unit necessary for gate operation are appropriately arranged on a self-standing panel with a closed structure considering operability, safety, etc.		It was developed for cost and space saving, and the control panel is mounted on the rack type of hoist. It is functionally the same as the self-standing type.	
Installation Space	It is necessary to secure an appropriate space in front of the control panel for operation and inspection.	△	Since it is mounted on the hoisting device, no space for a control panel is required. However, the position of the board is constrained.	○
Wiring Work	Wiring work is necessary from the control panel to the power receiving section of the switching device.	△	It can be connected directly from the power receiving panel to the control panel installed with the hoisting device.	○
Functionality	Normal opening, closing and stopping operation can be easily performed. There are few restrictions on the surface of the board, and it is possible to display easily. This type has many examples and reliable.	○	Normal operation of opening, closing and stopping can be easily performed. Due to the compact surface of the board, there are restrictions on the display, but there is no problem with the rack type. This type also has many examples.	○
Serviceability	The arrangement in the board is clear, and the maintenance is comparatively easy.	○	Skill is necessary for maintenance, since the equipment in the board is complicated.	△
Economy	Expensive compared to the Mounted Type (100%)	2	Cheaper than the Self-standing Type (75%)	1
Evaluation	It has many good examples and high reliability; however, it is inferior in cost and space.		Although maintenance and inspection require skill, it is excellent in space saving and cost saving.	
			Recommended	

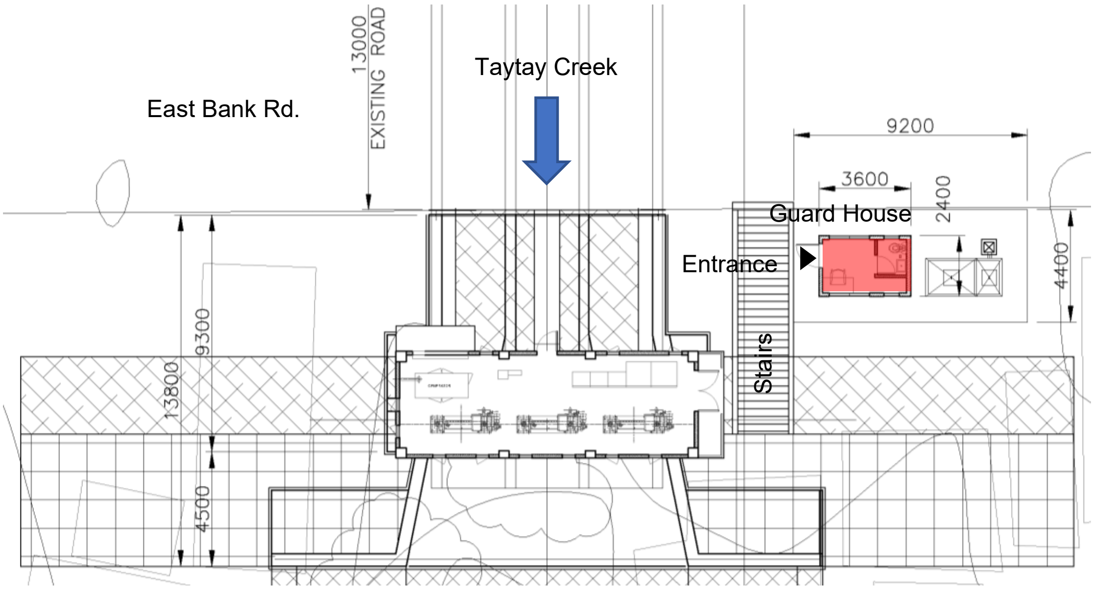
Legend: ○... No problem, △... There are issues to be solved.
Source: Study Team

(4) Remote Monitoring System

The study procedure is the same as in the Cainta Floodgate.

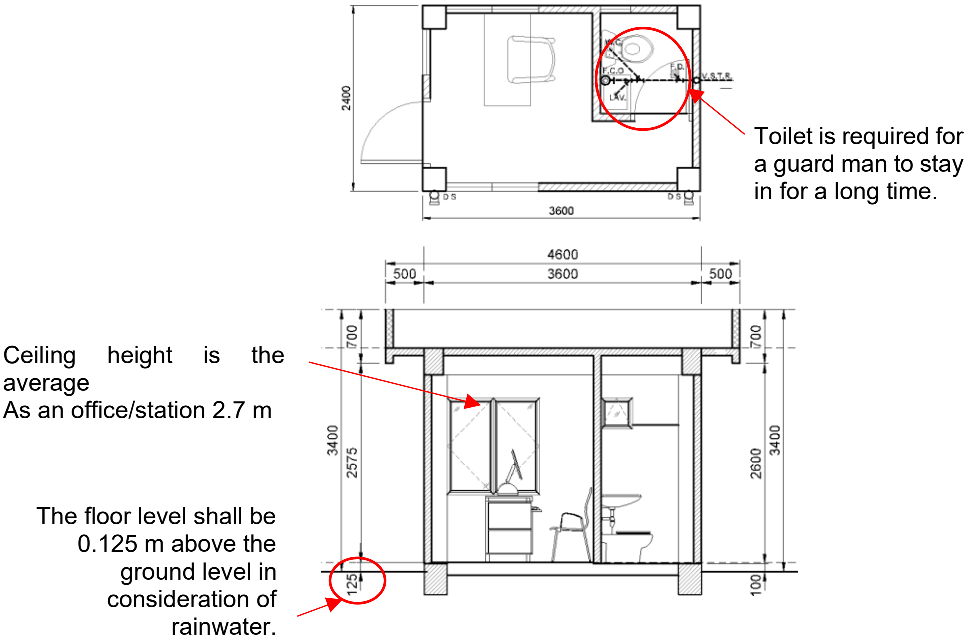
6.4.4.12 Incidental Facility

As incidental facilities of the Taytay Sluiceway, a management stair for accessing the river side end of the sluiceway and a guard house where security guards are stationed is provided. In addition, MMDA, which will be the manager of this sluiceway, is recommending providing with a guard house based on their experience that expensive equipment was stolen in other pumping stations.



Source: Study Team

Figure 6.4.85 Plans for Layout of Incidental Facilities of Taytay Sluiceway Gates

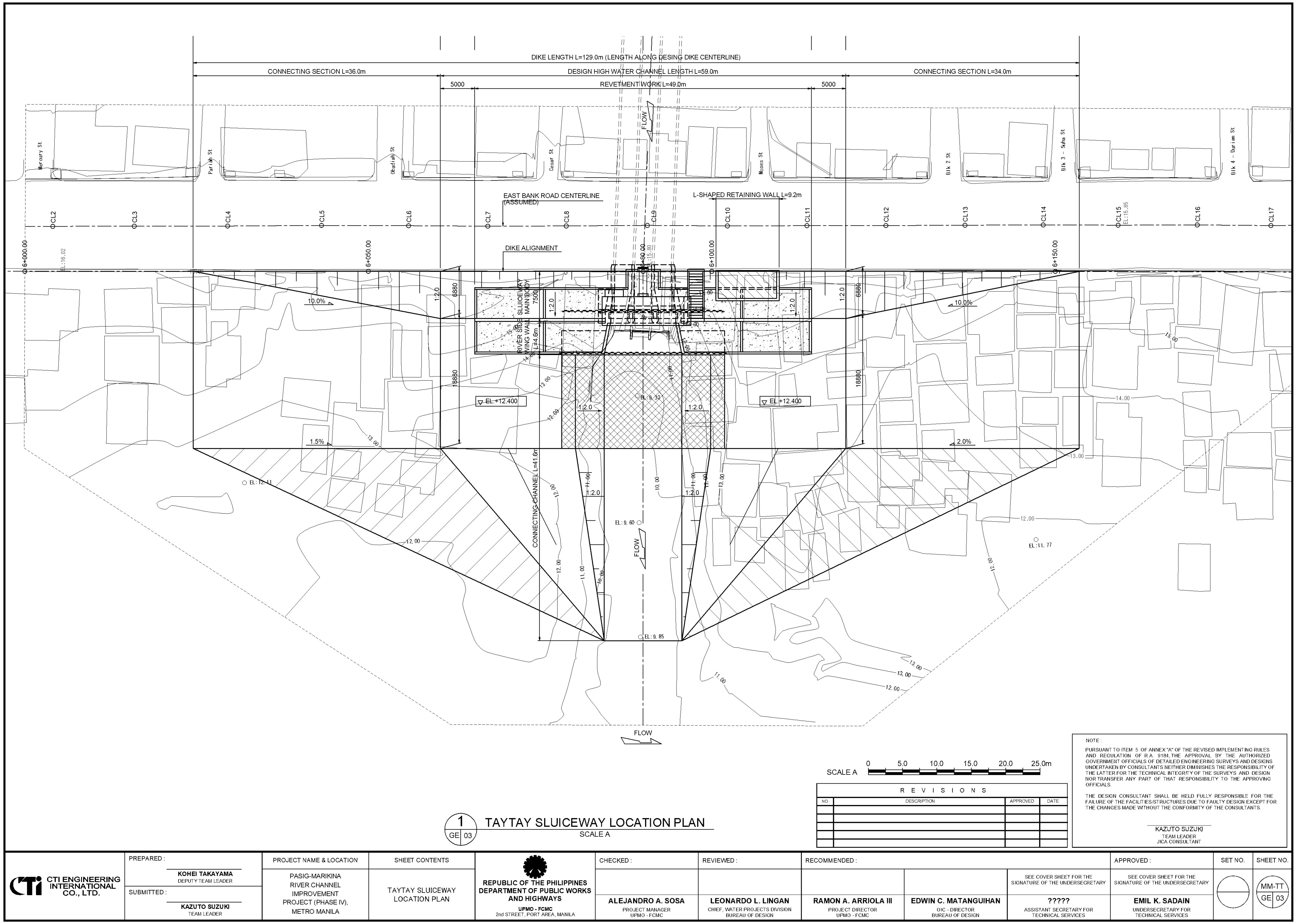


Source: Study Team

Figure 6.4.86 Plan and Cross Section of Taytay Guard House

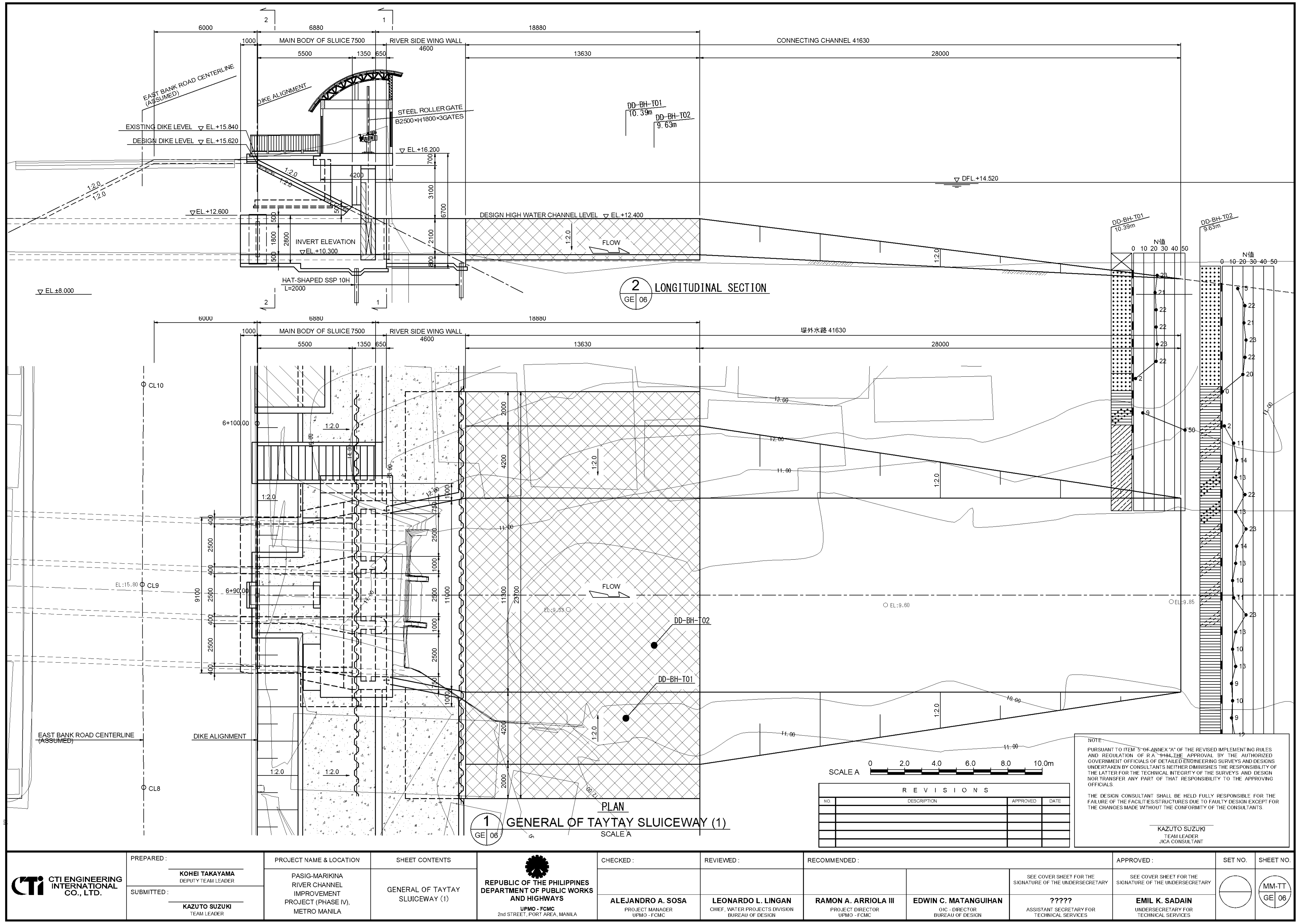
6.4.4.13 General Drawings

General drawings based on the above conditions and considerations are attached below.



Source: Study Team

Figure 6.4.87 General Drawing of Taytay Sluiceway Gate (1)



Source: Study Team

Figure 6.4.88 General Drawing of Taytay Sluiceway Gate (2)