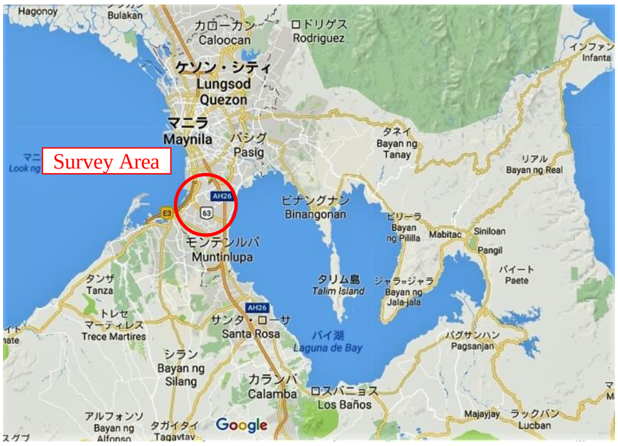
CHAPTER 7. PRE-FEASIBILITY STUDY OF PARAÑAQUE SPILLWAY

7.1 Geological Condition

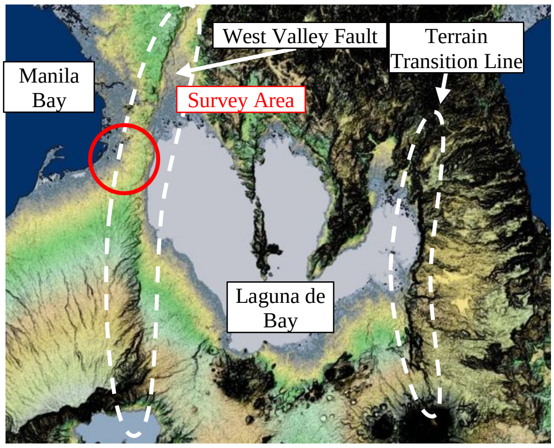
7.1.1 Topography and Geology at the Survey Area

The survey area is at the hilly land, approximately 10 km interval in between Manila Bay and Laguna de Bay at the southern part of Metro Manila as shown in Figure 7.1.1. The topography around the survey area, taken by the Shuttle Radar Topography Mission (SRTM) and published by the National Aeronautics and Space Administration (NASA), shows the terrain transition line in the North-Northeast-South direction as shown in Figure 7.1.2. This line is called “West Valley Fault” in “the Valley Fault System.” Although another terrain transition line can be seen on the east edge of Laguna de Bay, it has not been confirmed as a part of the fault.



Source: JICA Survey Team

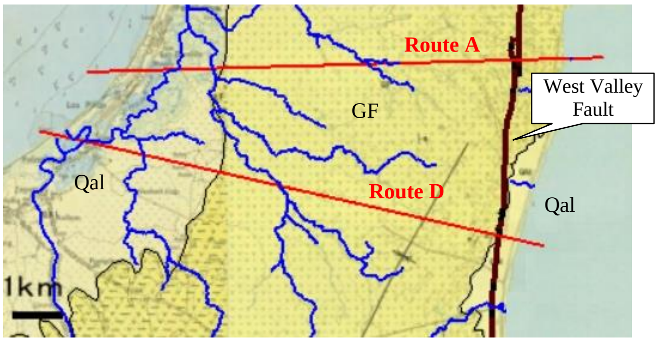
Figure 7.1.1 Location of the Survey Area



Source: SRTM, Visualized by JICA Survey Team

Figure 7.1.2 Topography around the Survey Area

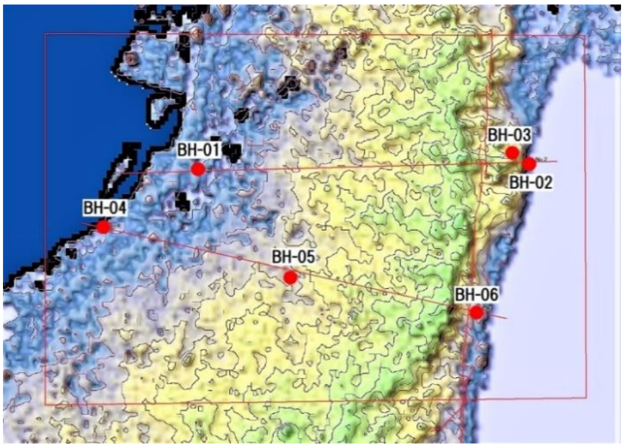
The geological map created by the Geological Survey Division of the Philippine Bureau of Mines and Geo-Sciences is shown in Figure 7.1.3. According to this map, the basement rock in this hilly land is the Pleistocene Guadalupe Formation (GF), mainly composed of volcanic clastic rocks (tuff, lapilli tuff, tuff gravel rock, volcanic ash, silt, etc.), the so-called “soft rocks”. In the lowlands on the western side of the hill and the lowlands along the Laguna de Bay lakeshore area, Holocene Quaternary Alluvium (Qal), unconsolidated deposits such as clay, silt, sand and gravel, cover the basement soft rocks.



Source: Geology of the Philippines, Philippine Bureau Of Mines and Geo-Sciences Geological Survey Division

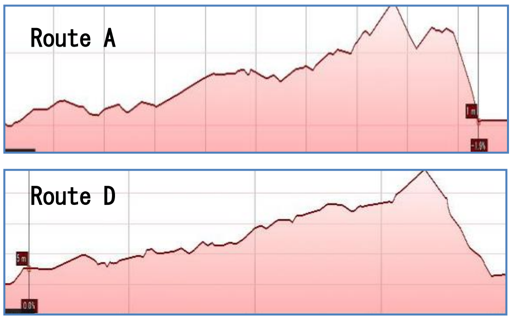
Figure 7.1.3 Geological Map around the Survey Area

In order to study the topography deeply, a contour map based on SRTM was created as shown in Figure 7.1.4. Additionally, the longitudinal sections along the route alignment shown in Figure 7.1.3 are as presented in Figure 7.1.5 based on the elevation data in Google Earth.



Source: SRTM, Visualized by JICA Survey Team

Figure 7.1.4 Topography with the Contour Lines around the Survey Area



Source: Google Earth, Visualized by JICA Survey Team

Figure 7.1.5 Longitudinal Section along Route A and Route D with Google Earth

According to these figures, the hilly land between Manila Bay and Laguna de Bay goes gradually higher and higher toward Laguna de Bay. The elevation of the top of the hilly land is approximately 30 m to 40 m close to Laguna de Bay, then it drops at the lakeshore area. It can, therefore, be assumed that the West Valley Fault appears on the slope of the drop.

Although most of the survey area is covered with dense residential structures; basement rock can be seen at a limited area of the cut slope along roads.

Source: JICA Survey Team

Source: JICA Survey Team

Figure 7.1.6 Exposed Rock Surface in the Survey Area

7.1.2 Geotechnical Investigation with Borehole Drilling

(1) Objective

In order to investigate the underground geological structures, the extent of compaction of materials, underground water level, etc., geotechnical investigation with borehole drilling was conducted. The length of boreholes was set at 70 m to obtain the geological information for tunnel construction and, at the surface area, standard penetration test (SPT) was conducted to obtain the state of compression of the ground surface. Some samples were also taken for the laboratory tests.

(2) Quantity

Table 7.1.1 show the quantities of in-situ tests.

Table 7.1.1 Quantity of In-Situ Tests

Borehole Name	Depth (m)	Ground Elevation (m)	Number of SPTs
BH-01	70.0	14.9	22
BH-02	70.0	12.7	47
BH-03	70.0	35.8	2
BH-04	70.0	11.5	19
BH-05	70.0	19.3	6
BH-06	70.0	17.1	7
Total: 6 holes	420.0	-	103

Source: JICA Survey Team

Table 7.1.2 show the quantities of laboratory tests.

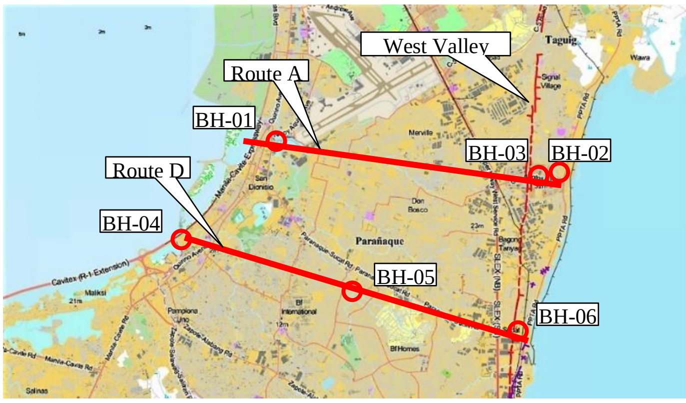
Table 7.1.2 Quantity of Laboratory Tests

Test	Material	Number of Tests
Moisture Content	Sand/Clay, Rock	60, 36
Sieving	Sand/Clay	60
Atterberg Limit	Sand/Clay	60
Specific Gravity	Sand/Clay, Rock	60, 36
Absorption	Rock	36
Uniaxial Strength	Rock	36
Slake Durability	Rock	18

Source: JICA Survey Team

(3) Location of Boreholes

The location of each borehole is as shown in the Figure 7.1.7. Originally, these were planned to be along the two candidate routes of the Parañaque Spillway, i.e., two at the Manila Bay side, two at Laguna de Bay side and two at the hilly land in between them. However, in order to acquire the geological information close to the intake of the Parañaque Spillway, BH-02, which was originally designed at the hilly land, was transferred to the bank of the drainage channel close to Laguna de Bay.



Source: JICA Survey Team

Figure 7.1.7 Location of Boreholes

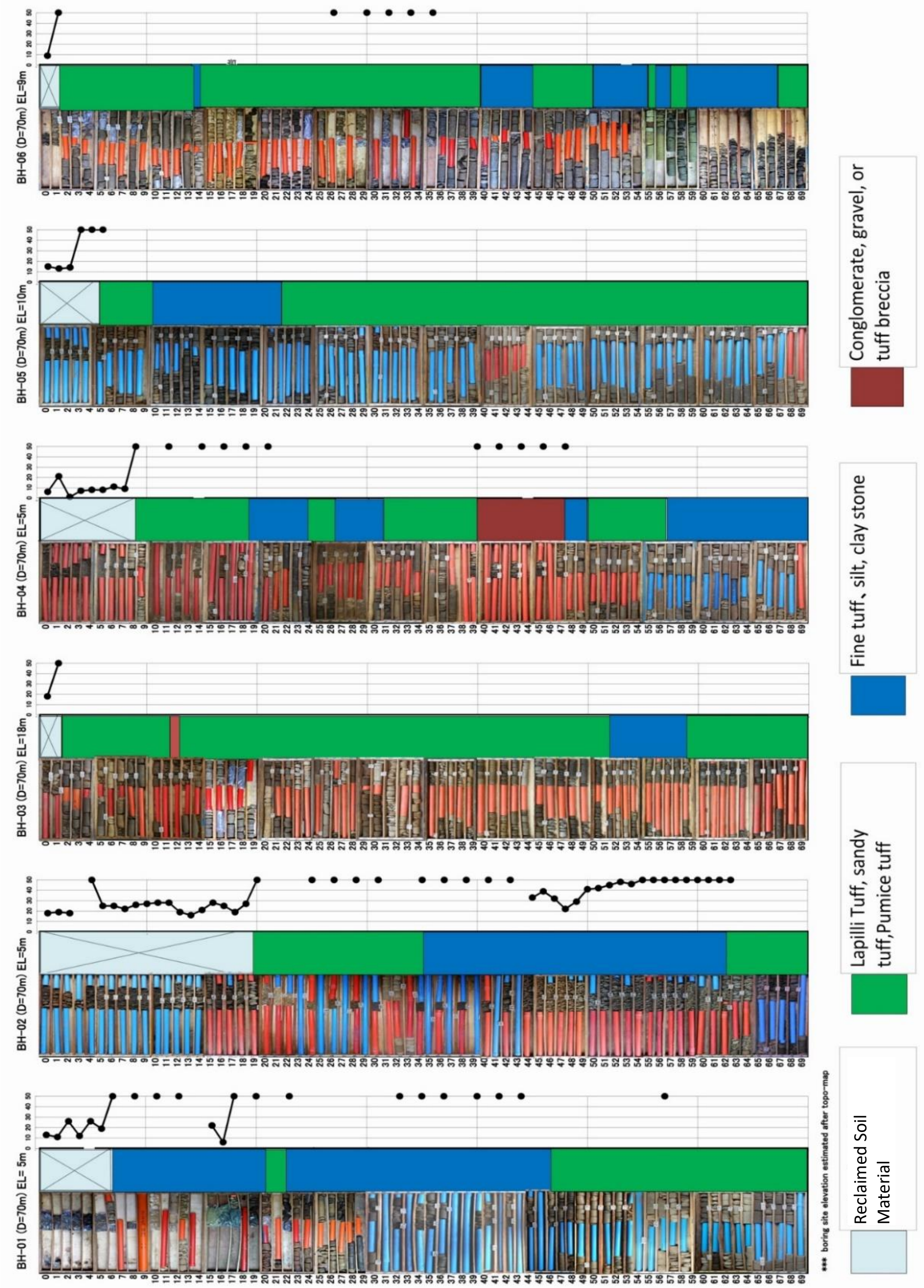
(4) In-Situ Test Results

Table 7.1.3 show the results of boring together with the description of materials, and the core photos and borehole logs are presented in Figure 7.1.8, together with the geological soil classification and N values. As described before, basement rocks in this hilly land consist of Pleistocene Guadalupe Formation (GF), mainly composed of volcanic clastic rocks (tuff, lapilli tuff, tuff gravel rock, volcanic ash, silt, etc.), or the so-called "soft rocks."

Table 7.1.3 Quantity of Laboratory Tests

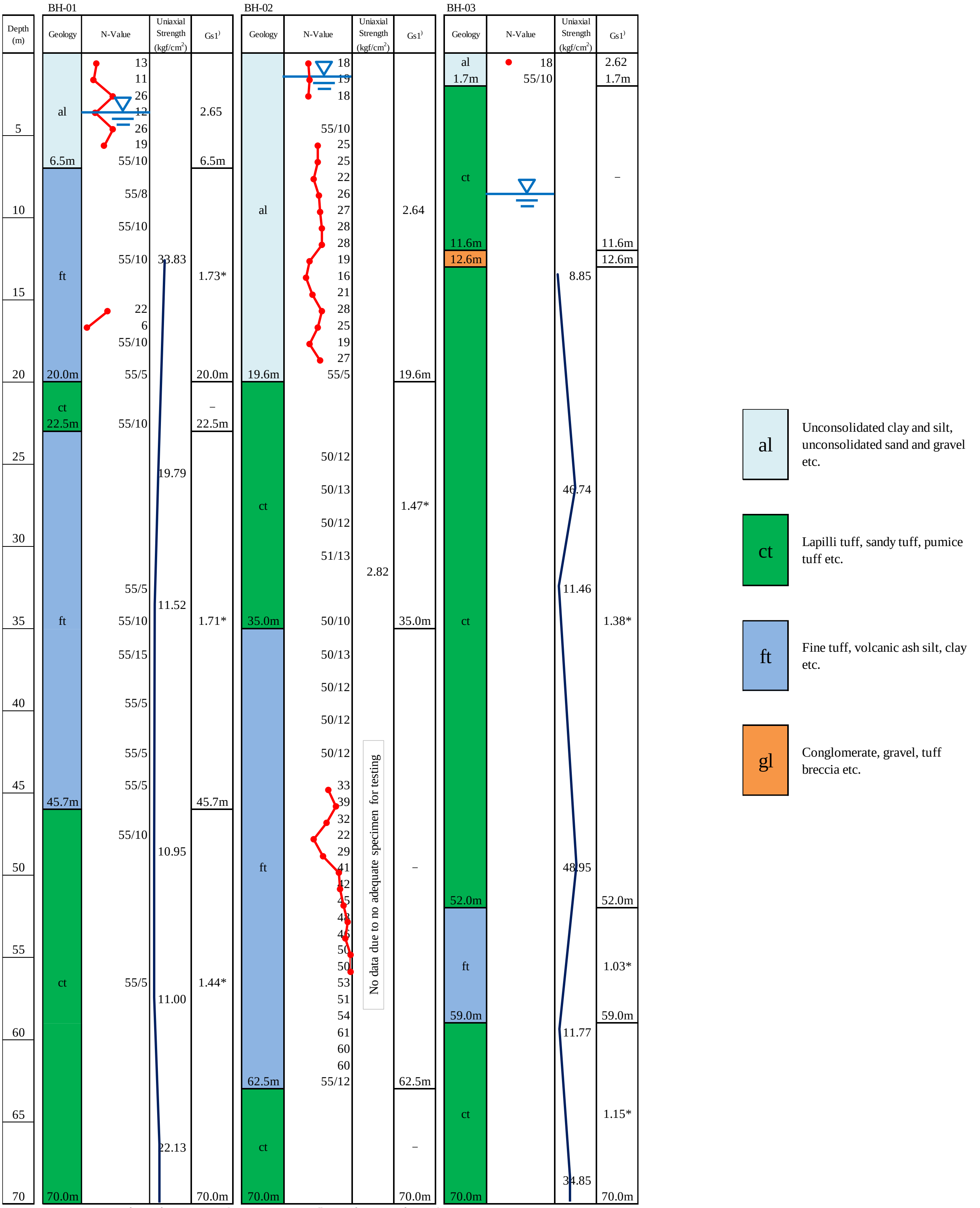
Geological Features		Position of the Layer	Photos
1	Reclaimed Soil Material (Mixed with artifacts such as vinyl)	BH-4: 0～3m	
2	Coarse sand deposits in Laguna de Bay (Contain many white fragments of shells and unconsolidated.)	BH-2: 0～6m	
3	Lapilli Tuff, Sandy Tuff, Pumice Tuff (Contain small rock pieces and well consolidated and permeability is assumed to be low.)	BH-3: 22.6～25m	
	Lapilli Tuff (Contain well consolidated small rock pieces with almost no crack and permeability is low.)	BH-6: 42～45m	
4	Fine tuff, silt, clay (Fine and coarse grained ash are deposited as mutual layers at the water bottom and deposition surfaces can be seen.)	BH-1: 47.6m	
5	Conglomerate (gravel, tuff breccia) (It is assumed that conglomerates with low degree of consolidation are collected in gravel form and permeability is most probably high.)	BH-4: 9.7m	

Source: JICA Survey Team



Source: JICA Survey Team

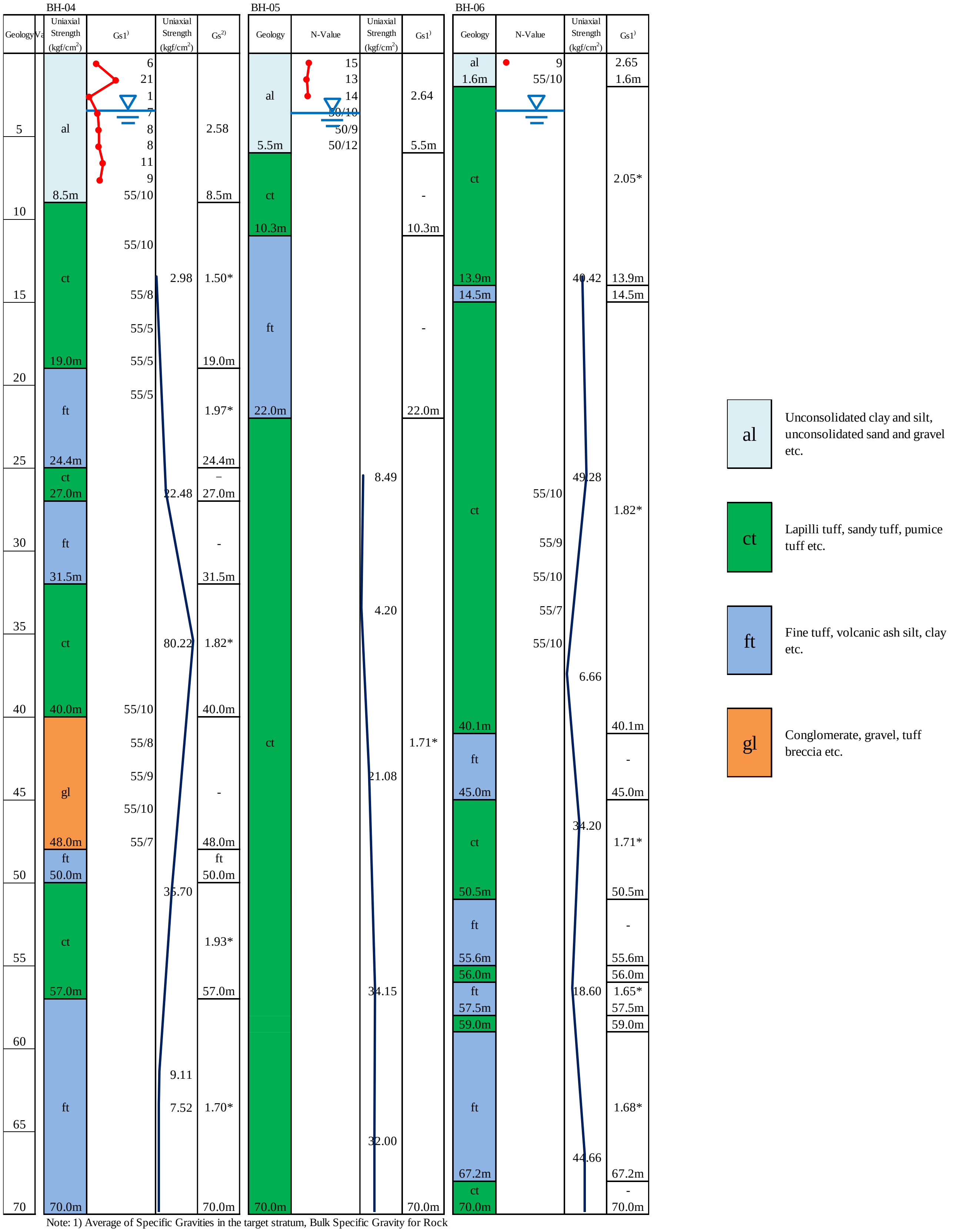
Figure 7.1.8 Borehole Logs



Note: 1) Average of Specific Gravities in the target stratum, Bulk Specific Gravity for Rock

Source: JICA Survey Team

Figure 7.1.9 Borehole Logs (BH-01 to BH-03)

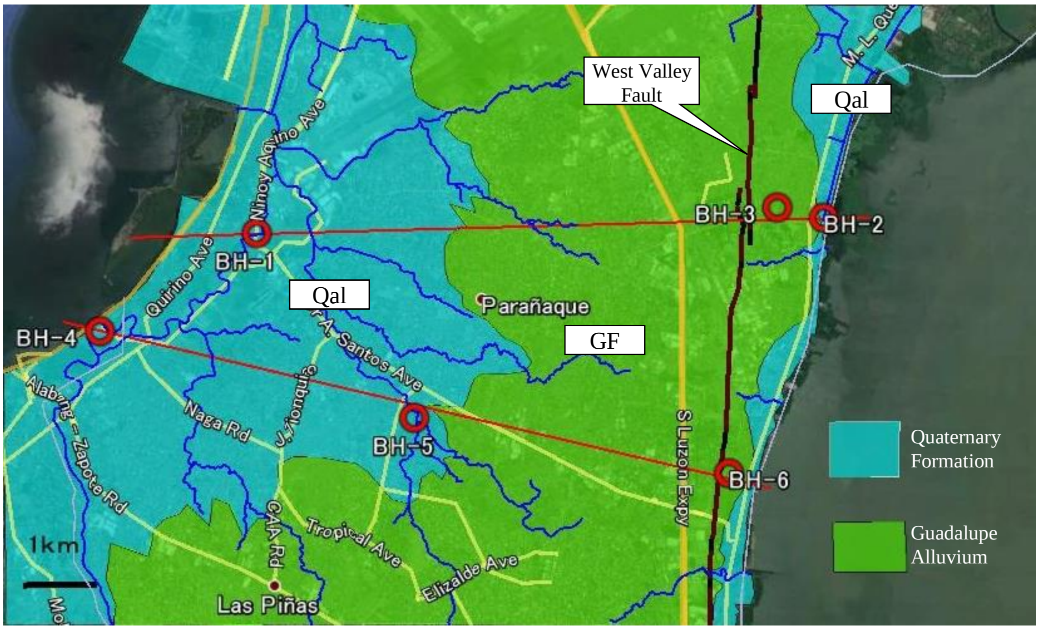


Source: JICA Survey Team

Figure 7.1.10 Borehole Logs (BH-04 to BH-06)

Based on the geological map shown in Figure 7.1.3, another geotechnical map was newly created and shown as Figure 7.1.11, reflecting the information obtained by several site investigations, boring test results and SRTM elevation data. These maps differ as follows:

- Alluvium layer was observed at BH-05 although BH-05 is located at the middle of hilly land. Hence, the area of Qal was expanded toward the hill land area.
- Alluvium layer was also found at BH-02 and its northern area along Laguna de Bay. Hence, the area of Qal along Laguna de Bay was also extended toward north.



Source: JICA Survey Team

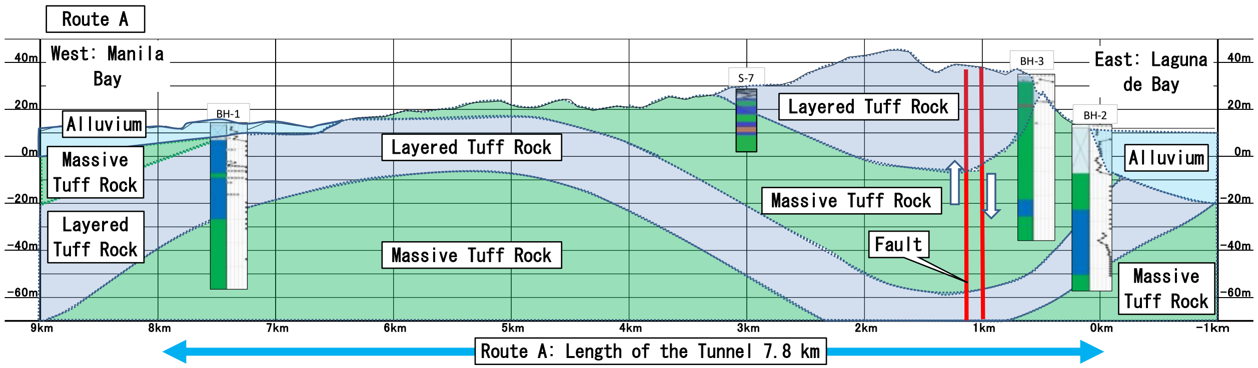
Figure 7.1.11 Geological Map

Legend

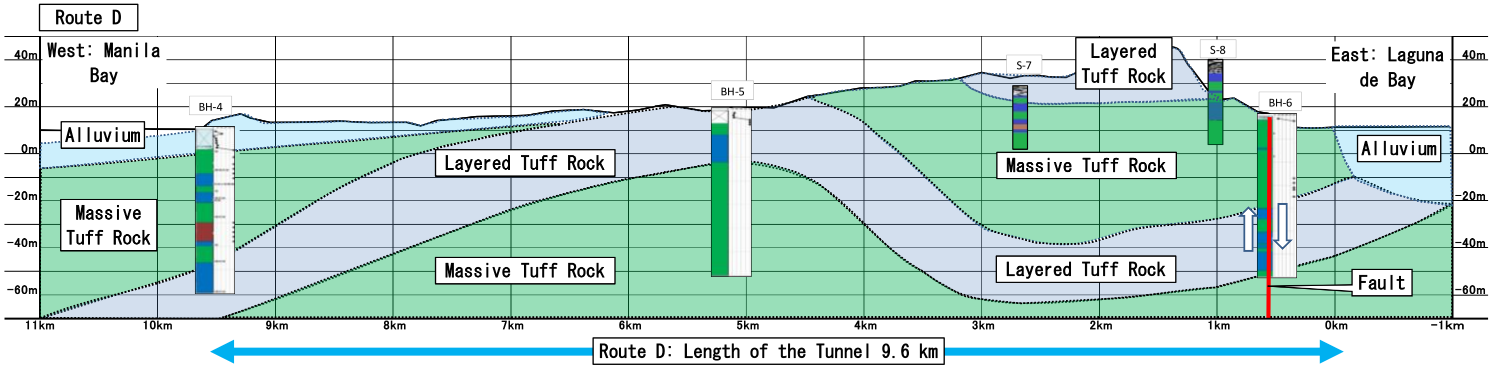
Alluvium
- Unconsolidated clay and silt, unconsolidated sand and gravel etc.

Massive Tuff Rock
- Lapilli tuff, sandy tuff, pumice tuff etc.

Layered Tuff Rock
- Fine tuff, volcanic ash silt, clay etc.



- Alluvium deposits are distributed in the lowland on the western side (Manila Bay side) and east side (Laguna de Bay), and volcanic clastic rocks are distributed in the central hilly land.
- Basement rocks are volcanic clastic rocks (Pleistocene) called Guadalupe Formation (tuff, lapilli tuff, tuff breccia, volcanic ash silt rocks, etc.) and have layered planes within 5 to 6 degrees .
- There are few cracks in the rocks, and the permeability is considered to be small. In addition, the uniaxial compressive strength of the basement rock is about 24.2 Mpa on average.
- The alluvium consists of unconsolidated sand and clay, N value is about 10 to 30, basement rocks excluding weathered part of 1 to 3 m surface layer, N value is more than 50. On the Laguna de Bay Lakeshore area, about 19 m of sand and silt layer with an N value of 10 to 30 has been confirmed.
- From the assumed start point of the Parañaque Spillway, two faults are assumed to be located around 1.0 to 1.1 km, and the altitude of the fault location is around 28 m.



Note:
These drawing are stretched 20 times in lateral direction. Actual strata should be almost horizontally distributed.

- Alluvium deposits are distributed in the lowland on the western side (Manila Bay side) and east side (Laguna de Bay), and volcanic clastic rocks are distributed in the central hilly land.
- Basement rocks are volcanic clastic rocks (Pleistocene) called Guadalupe Formation (tuff, lapilli tuff, tuff breccia, volcanic ash silt rocks, etc.) and have layered planes within 5 to 6 degrees .
- There are few cracks in the rocks, and the permeability is considered to be small. In addition, the uniaxial compressive strength of the basement rock is about 24.2 Mpa on average.
- The alluvium consists of unconsolidated sand and clay, N value is about 10 to 30, basement rocks excluding weathered part of 1 to 3 m surface layer, N value is more than 50. On the Laguna de Bay Lakeshore area, about 19 m of sand and silt layer with an N value of 10 to 30 has been confirmed.
- From the assumed start point of the Parañaque Spillway, single fault is assumed to be located around 0.5 km to 0.6 km, and the altitude of the fault location is around 10 m.

Source: JICA Survey Team

Figure 7.1.12 Geological Longitudinal Section along Route A and D

(5) Laboratory Test Results

The laboratory test results for sand and clay materials are summarised in Table 7.1.4 and Table 7.1.5. The ones for rock materials are similarly done in Table 7.1.6 and Table 7.1.7.

Table 7.1.4 Laboratory Test Result Summary (BH-01 to BH-03, Sand and Clay Material)

Borehole Number	Name of Sample	Depth (m)	Soil Description	N-Value	Moisture Content (%)	PI (%)	Gs	Passing 4.75mm sieve (%)	Passing 0.0075mm sieve (%)
BH-01	SPT-01	0.55-1.00	sandy clay	13	26	17	2.66	79	52
	SPT-02	1.55-2.00	clayey sand	11	34	NP	2.64	80	26
	SPT-03	2.55-3.00	clayey sand, traces of shells and garbage	26	21	NP	2.63	62	17
	SPT-05	4.55-5.00	clayey sand, traces of shells and garbage	26	31	NP	2.66	76	18
	SPT-06	5.55-6.00	clayey sand, traces of shells and garbage, low plasticity	19	36	11	2.67	91	34
	SPT-09	10.55-10.80	sandy silt (ml)	55/10	46	12	2.63	99	53
	SPT-10	12.55-12.65	gravelly clay	55/10	30	40	2.62	75	60
	SPT-11	15.55-16.00	clay (ch)	22	56	46	2.65	99	93
	SPT-12	16.55-17.00	clay (ch), traces of sand	6	81	48	2.67	100	93
	SPT-13	17.55-18.00	sandy silt (mh)	55/10	61	24	2.65	100	71
BH-02	SPT-01	0.55-1.00	light gray, silty sand	18	17	NP	2.65	83	23
	SPT-02	1.55-2.00	light brown, silty sand	19	17	14	2.64	85	42
	SPT-03	2.55-3.00	light brown, silty sand gravel and shells	18	17	8	2.66	69	34
	SPT-05	5.55-6.00	dark gray silty sand	25	27	12	2.64	91	43
	SPT-06	6.55-7.00	dark gray silty sand, non-plastic	22	8	NP	2.63	92	16
	SPT-09	9.55-10.00	dark gray silty sand, non-plastic	27	32	NP	2.63	97	12
	SPT-10	10.55-11.00	dark gray silty sand, non-plastic	28	30	NP	2.64	98	13
	SPT-11	11.55-12.00	dark gray to dark brown, sandy silt	28	63	12	2.63	78	54
	SPT-12	12.55-13.00	dark gray, sandy clay	19	54	52	2.63	95	59
	SPT-14	14.55-15.00	light gray, clay with sand	21	37	46	2.65	95	80
BH-03	SPT-01	0.55-1.00	light gray, sandy clayey silt	18	44	26	2.62	80	53
	SPT-02	1.55-1.65	light gray, silty gravel	55/10	44	NP	-	31	15

Source: JICA Survey Team

Table 7.1.5 Laboratory Test Result Summary (BH-04 to BH-06, Sand and Clay Material)

Borehole Number	Name of Sample	Depth (m)	Soil Description	N-Value	Moisture Content (%)	PI (%)	Gs	Passing 4.75mm sieve (%)	Passing 0.0075mm sieve (%)
BH-04	SPT-01	0.55-1.00	light gray, sandy silt	6	34	18	2.62	92	61
	SPT-02	1.55-2.00	dark gray silty sand, with gravel	21	40	NP	2.36	62	18
	SPT-03	2.55-3.00	dark gray silty sand, with gravel	1	40	NP	2.65	75	16
	SPT-04	3.55-4.00	dark gray, poorly-graded sand	7	35	NP	2.56	72	11
	SPT-05	4.55-5.00	dark gray, well-graded sand	8	23	NP	2.58	91	9
	SPT-06	5.55-6.00	dark gray, well-graded sand	8	28	NP	2.57	95	9
	SPT-07	6.55-7.00	dark gray, silty sand	11	24	NP	2.65	89	15
	SPT-08	7.55-8.00	dark gray, silty sand	9	26	NP	2.65	91	15
	SPT-11	14.55-14.63	yellow brown sandy silt	55/8	61	29	2.64	98	65
	SPT-12	16.55-16.90	light brown, silt with sand	16+ 55/5	28	31	2.60	99	81
BH-05	SPT-01	0.55-1.00	dark brown, clay, with sand	15	38	27	2.65	99	82
	SPT-02	1.55-2.00	dark brown, clay, with sand	13	37	38	2.62	100	85
	SPT-03	2.55-3.00	dark brown, clay, with sand	14	42	31	2.63	100	84
	SPT-04	3.55-4.00	dark gray, clay	50/10	30	23	2.64	99	73
	SPT-05	4.55-5.00	dark brown, sandy silt	50/9	42	14	2.65	100	62
	SPT-06	5.55-5.67	dark brown, sandy silt	50/12	46	15	2.63	100	65
BH-06	SPT-01	0.55-1.00	dark brown, clay with sand	9	26	44	2.65	94	71
	SPT-02	1.55-1.65	dark brown, clay with sand	55/10	30	31	2.64	98	72
	SPT-03	27.55-28.00	dark gray, silty sand	55/10	36	NP	2.66	100	28
	SPT-04	29.55-29.64	dark gray, silty sand	55/9	38	NP	2.65	100	21
	SPT-05	31.55-31.65	dark gray, silty sand	55/10	37	NP	2.65	100	21
	SPT-06	33.55-33.65	dark gray, silty sand	55/7	36	NP	2.64	100	20
	SPT-07	35.55-35.65	dark gray, silty sand	55/10	34	NP	2.67	84	14

Source: JICA Survey Team

Table 7.1.6 Laboratory Test Result Summary (BH-01 to BH-04, Rock Material)

Borehole number	Sample name	Depth (m)	Soil Description	RQD (%)	Uniaxial Strength (kgf/cm²)	Gs (Bulk)	Absorption (%)	Moisture Content (%)	Slaking Durability (%)
BH-01	CS-04	12.65-13.65	brown, tuff rock	33	33.83	1.730	29.20	22.53	-
	CS-09	22.65-23.65	dark gray, tuff rock	0	-	-	-	-	77.8
	CS-12	25.65-26.65	yellowish gray, tuff rock	40	19.79	1.646	38.92	36.42	-
	CS-17	30.65-31.65	dark gray, tuff rock	-	-	1.772	48.02	47.51	
	CS-18	32.60-33.60	dark gray, tuff rock	11	11.52	-	-	-	-
	CS-26	47.65-48.65	yellowish brown, tuff rock	30	10.95	1.552	53.48	50.32	-
	CS-28	49.65-50.65	light gray, tuff rock	20	-	-	-	-	69.8
	CS-34	56.62-57.62	light gray, tuff rock	45	11.00	1.541	35.20	30.09	-
	CS-43	65.62-66.62	gray, tuff rock	40	22.13	1.224	23.36	18.23	-
	CS-44	66.62-67.62	gray, tuff rock	0	-	-	-	-	69.4
BH-02	CS-09	30.68-31.68	dark gray, tuff rock	17	2.82	-	-	-	-
	CS-10	31.68-32.68	dark gray, tuff rock	-	-	1.467	23.31	14.93	-
BH-03	CS-12	12.65-13.65	dark gray, tuff rock	33	8.85	1.256	36.29	30.42	-
	CS-25	25.65-26.67	dark gray, tuff rock	100	46.74	1.395	27.70	21.18	-
	CS-31	31.67-32.67	dark gray, tuff rock	18	11.46	-	-	-	-
	CS-33	33.67-34.67	yellowish brown, tuff rock	20	-	1.480	12.07	3.24	-
	CS-48	48.67-49.67	dark gray, tuff rock	17	48.95	-	-	-	-
	CS-49	49.67-50.67	dark gray, tuff rock	17	-	1.389	30.56	28.74	-
	CS-58	58.67-59.67	greenish gray, tuff rock	20	11.77	1.032	67.06	64.73	-
	CS-67	67.67-68.67	dark gray, tuff rock	36	34.85	1.266	38.05	43.11	-
BH-04	CS-03	11.65-12.65	light gray, tuff rock	11	-	1.502	42.56	34.79	-
	CS-04	12.65-13.65	light gray, tuff rock	11	11.75	-	-	-	-
	CS-11	23.62-24.62	dark gray, tuff rock	56	-	1.968	29.51	28.34	66.7
	CS-13	25.62-26.62	dark gray, tuff rock	36	22.48	-	-	-	-
	CS-22	34.62-35.62	dark gray, tuff rock	90	80.22	-	-	-	-
	CS-24	36.62-37.62	dark gray, tuff rock	50	-	1.795	20.57	18.83	-
	CS-31	38.62-39.62	light gray, tuff rock	25	-	1.853	29.52	28.85	68.9
	CS-32	49.62-50.62	light gray, tuff rock	33	35.70	-	-	-	-
	CS-38	55.62-56.62	light gray, tuff rock	75	-	1.93	25.81	25.18	-
	CS-43	60.62-61.62	light gray, tuff rock	51	9.11	-	-	-	-
	CS-45	62.62-63.62	light gray, tuff rock	56	7.52	1.703	35.76	35.16	71.40

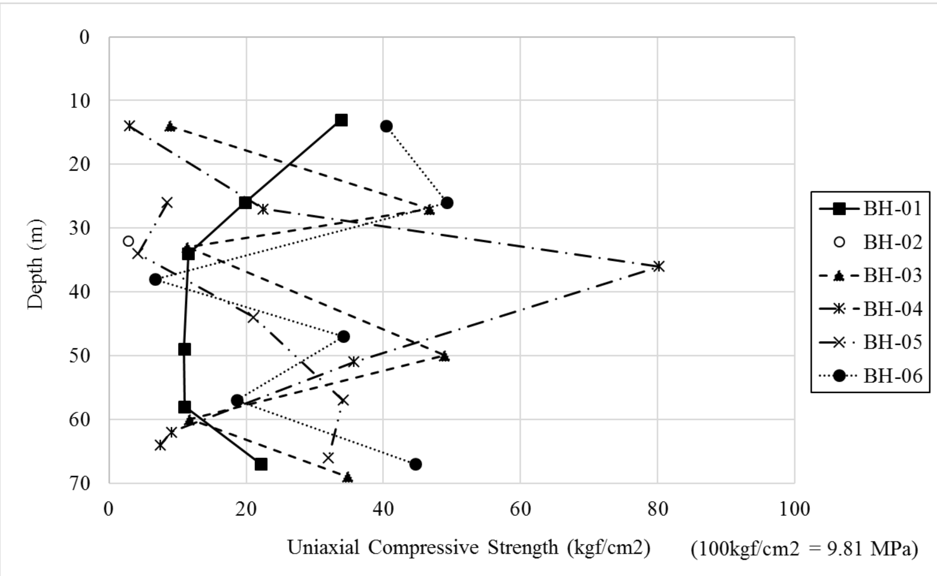
Source: JICA Survey Team

Table 7.1.7 Laboratory Test Result Summary (BH-05 to BH-06, Rock Material)

Borehole Number	Name of Sample	Depth (m)	Soil Description	RQD (%)	Uniaxial Strength (kgf/cm^2)	Gs (Bulk)	Absorption (%)	Moisture Content (%)	Slaking Durability (%)
BH-05	CS-20	24.67-25.67	dark gray, tuff rock	11	8.49	-	-	-	-
	CS-21	25.67-26.67	dark gray, tuff rock	-	-	1.788	33.06	29.34	-
	CS-22	26.67-27.67	dark gray, tuff rock	-	-	-	-	-	71.40
	CS-27	31.67-32.67	dark brown, tuff rock	-	-	1.744	52.34	46.79	-
	CS-28	32.67-33.67	grayish brown, tuff rock	22	4.20	-	-	-	-
	CS-38	42.67-43.67	light gray, tuff rock	66	21.08	1.502	39.76	31.15	-
	CS-39	43.67-44.67	dark gray, tuff rock	-	-	-	-	-	82.00
	CS-51	55.67-56.67	dark gray, tuff rock	63	34.15	1.803	29.14	27.63	-
	CS-60	64.67-65.67	dark gray, tuff rock	30	32.00	1.696	31.73	28.89	-
BH-06	CS-12	12.65-13.65	dark gray, tuff rock	90	40.42	2.049	22.30	21.59	-
	CS-24	24.65-25.65	dark gray, tuff rock	29	49.28	1.830	21.59	20.68	70.6
	CS-31	36.65-37.65	dark gray, tuff rock	50	6.66	1.816	25.49	24.85	-
	CS-40	45.65-46.65	dark gray, tuff rock	80	34.20	1.708	29.54	27.51	66.0
	CS-50	55.65-56.65	yellowish brown, tuff rock	43	18.60	1.652	39.11	37.84	-
	CS-60	65.65-66.65	dark gray, tuff rock	21	44.66	1.676	41.66	41.19	71.4

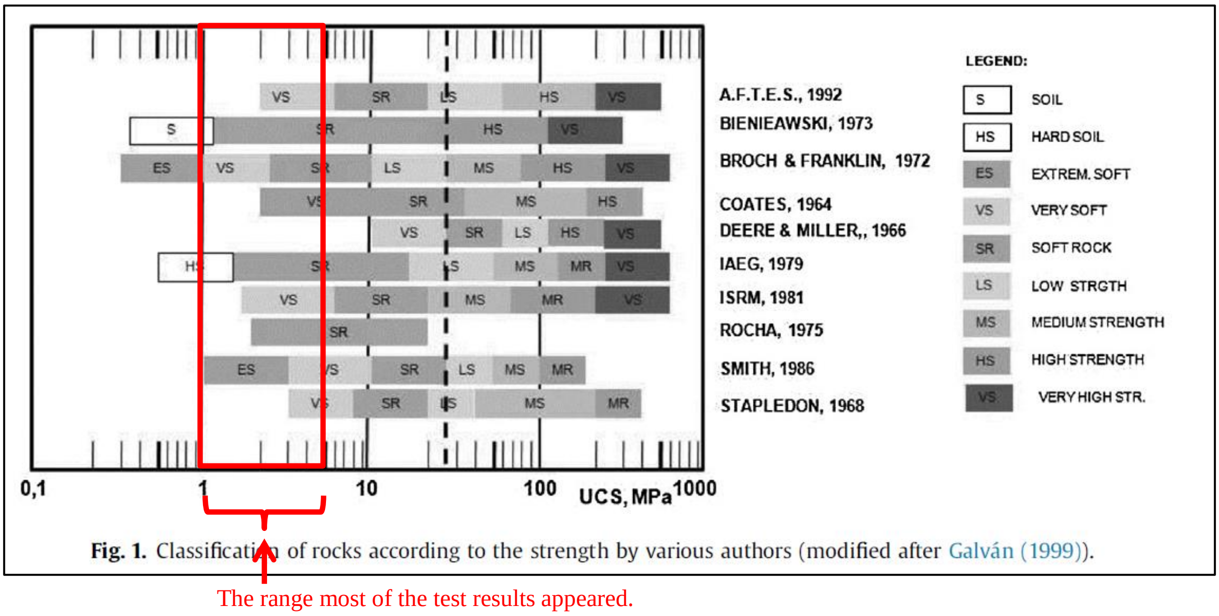
Source: JICA Survey Team

Figure 7.1.13 shows the relation between uniaxial compressive strength of the Guadalupe Formation as well as the depth from the ground surface at each borehole. This figure indicates that most of the uniaxial compressive strength are under 50 kgf/cm^2 (4.91 Mpa). Compared with Figure 7.1.14, the Guadalupe Formation is relatively soft in the soft rock classification.



Source: JICA Survey Team

Figure 7.1.13 Uniaxial Compressive Strength of Rock Specimen



Source: Milton Assis Kanji, Critical Issues in Soft Rocks, Journal of Rock Mechanics and Geotechnical Engineering 6, 2014, p186-p195

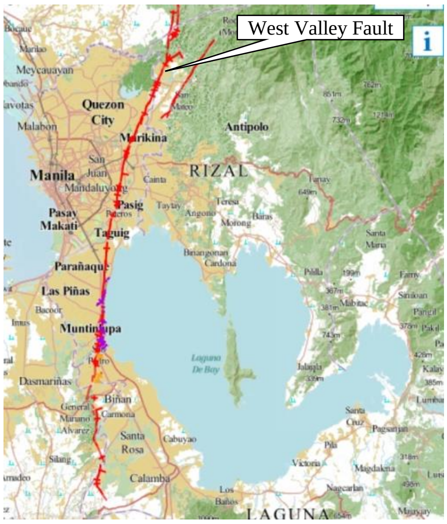
Figure 7.1.14 Comparison between the Test Results and Classification of Rocks

7.1.3 Data Collection on the Activity of Valley Fault System

PHIVOLCS had determined the location of the West Valley Fault of the Valley Fault System as shown in Figure 7.1.15, which is the east side of the hilly land in the survey area. PHIVOLCS had also compiled "The Valley Fault System in Great Metro Manila Area Atlas", which shows the location of the fault in Metro Manila.

JICA also described the Valley Fault System as an active fault with high risk in the "Earthquake Impact Reduction Study for Metropolitan Manila, March 2004".

On the other hand, there are some researches in Japan reporting that the southern area of the West Valley Fault is gradually moving. The following items describe two examples of the researches.



Source: Fault Finder, PHIVOLCS

Figure 7.1.15 Location of West Valley Fault

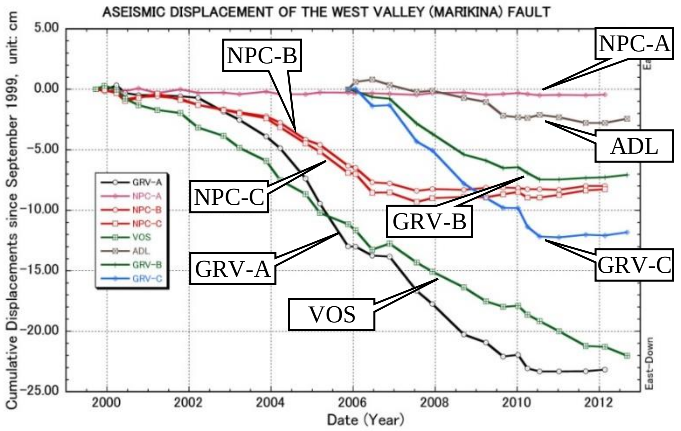
(1) Monitoring of Ground Deformation in South-East Part of Metro Manila, Philippines, by Ground Survey (Tokyo Metropolitan College of Industrial Technology, Research Report 7, p68-p71, March 2013)

Objective: In order to observe the creep movement at the segment in the West Valley Fault, leveling survey was repeatedly conducted at the locations shown in Figure 7.1.16.

Result: The Levelling Survey suggests that the average slip rate is from 1.77 cm/year to 2.45 cm/year between September 1999 and January 2010. Vertical displacement caused by creep in several locations has accelerated since 2004. Most of the movements have ceased since 2010.

Source: Monitoring of Ground Deformation in South-East Part of Metro Manila, Philippines, by Ground Survey

Figure 7.1.16 Location of Ground Deformation Survey Area



Source: Monitoring of Ground Deformation in South-East Part of Metro Manila, Philippines, by Ground Survey (Tokyo Metropolitan College of Industrial Technology, Research Report 7, p68-p71, March 2013)

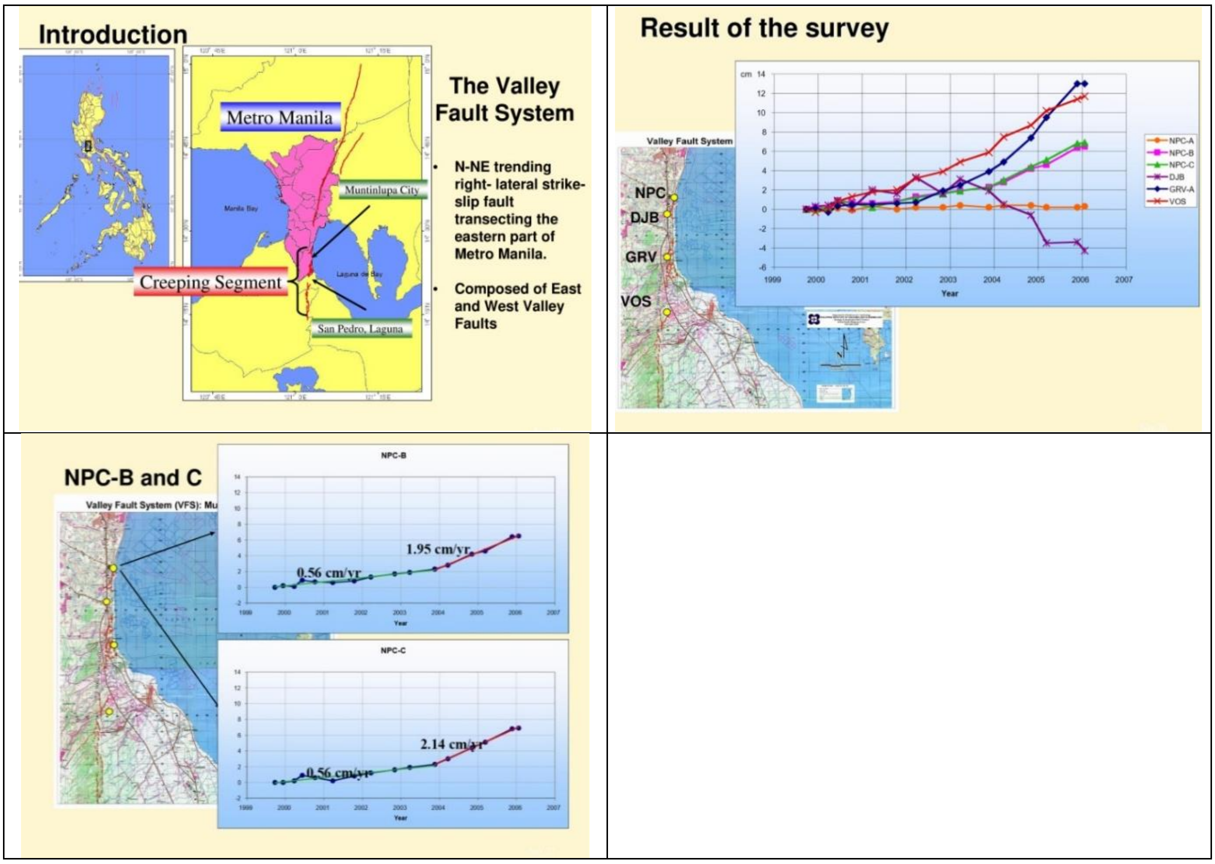
Figure 7.1.17 Movement of the Ground Observed by Repeated Leveling Survey in Metro Manila

(2) Observation of Creep on the Fault in the Philippines by InSAR (Proceedings of 2006 ERI Workshop, "New Generation InSAR" Earthquake Research Institute Joint Usage/Research Program, 2006)

Objective: Utilizing the Japanese Earth Resources Satellite - 1 (JERS-1) data, the movement of the ground was analysed.

Result: Settlement (0.5 cm/year to 2.0 cm/year) was observed in the Laguna de Bay side of the fault at some points.

Recommendation: For the clear explanation of the mechanism of the settlement, a wide range of observation around Metro Manila should be conducted. Other satellite information such as ALOS/PLASAR and simultaneous levelling survey also may help.



Source: InSAR; Observation of Creep on the Fault in the Philippines

Figure 7.1.18 Extracted Presentation Material for the InSAR Workshop

7.1.4 Effect of the Valley Fault System to the Parañaque Spillway Design and Construction

The effects of the Valley Fault System to the Parañaque Spillway design and construction are summarized as follows:

- The Valley Fault System consists of active faults and also probably creeping. In addition, since this fault system is continuously distributed in the north-south direction, it is inevitable that the Parañaque Spillway will cross the fault.
- Looking at the ground surface and the topography of the site, the unevenness appears repeatedly in the east and west direction in the vicinity of the fault position, and there is a possibility that there are two or more faults instead of one fault. (The cut surface along the road, cracks in parallel to the fault were observed.)
- If the underground tunnel structure is located at several tens of meters underground and it is damaged by creep of a fault, it is very difficult to repair it unlike a surface structure. It is, therefore, necessary to construct facilities that can be repaired permanently.
- Based on the current status of data collection, the exact location of the fault creep is unknown. In order to grasp the situation accurately, it is necessary to carry out detailed survey and ground monitoring survey by satellite continuously for at least several years along the east-west direction road. In addition, it is necessary to confirm the continuity at depth of tunnel construction at several tens of meters underground. (However, if it does not move during this survey period, further investigation may be necessary.)

7.2 Basic Design

7.2.1 Design Condition

(1) Hydraulic Condition of Design

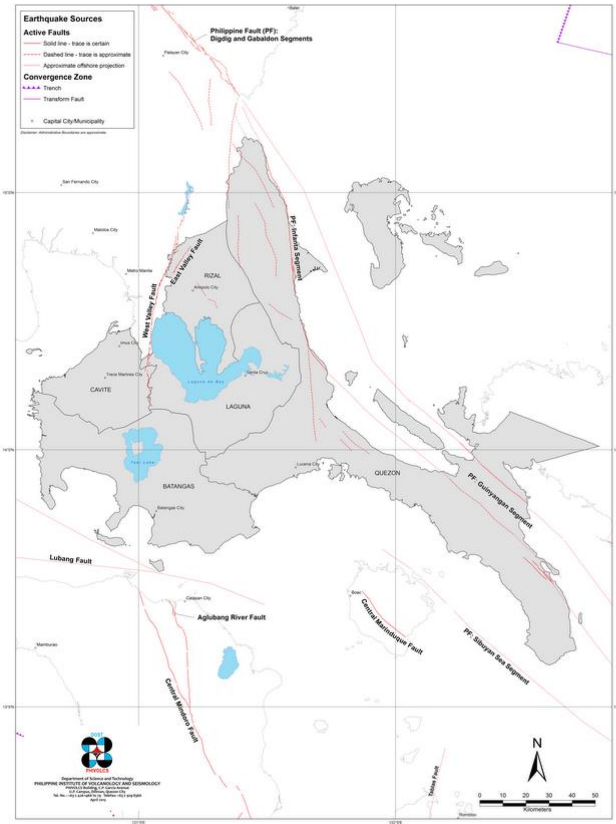
The hydraulic design conditions are based on the Comprehensive Flood Management Plan for Laguna de Bay as follows:

Design Flood Water Level	EL. 14.0 m
Planning Discharge	Qp = 200 m^3/s
Operation Start Water Level	EL. 12.0 m
Operation Start Discharge (Temporarily)*1	Qs = 130 m^3/s

Note) *1: It may be necessary to revise the operation start discharge based on the facility plan of Parañaque Spillway.

(2) Countermeasure Policy for West Valley Geological Fault

The Location Map of Geological Faults in Region IV-A around Laguna de Bay as drawn by the Philippine Institute of Volcanology and Seismology, Department of Science and Technology (DOST), is shown in Figure 7.2.1.



Source: http://202.90.128.67/html/update_GGRDD/Maps/AF-and-Trenches/Regional/Luzon/Region%204A.png

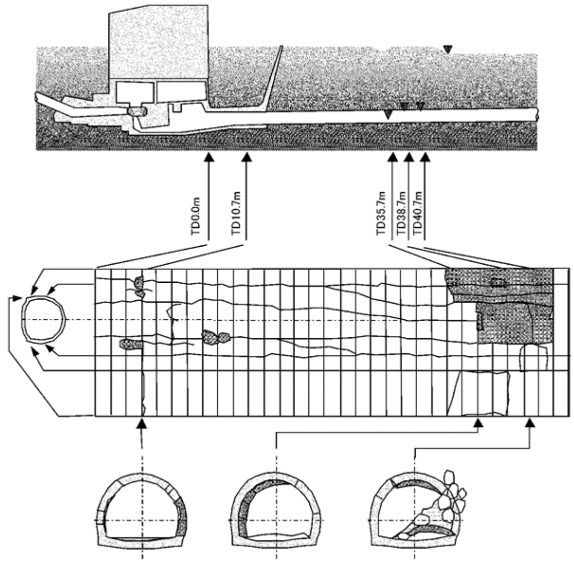
Figure 7.2.1 Location Map of Geological Faults in Region 4-A

The "Earthquake Impact Reduction Study for Metropolitan Manila, Republic of the Philippines (2002)" pointed out the hazards of the West Valley Fault, one of the famous active faults around Metro Manila. Therefore, it is necessary to consider countermeasures for the fault in the design of Parañaque Spillway.

One of the most famous accidents concerning active faults and tunnel construction in Japan involved the "Tanna Tunnel" (Total length: 7,804 m). The long tunnel construction period, which started in 1918 and lasted until 1933, was due to the abnormal spouting of groundwater and the tunnel roof-fall accidents. The North Izu Earthquake (Magnitude 7.0) in 1930 also caused the 2m sliding gap to the north which were repaired during the construction work. Since the activity cycle of the Tanna Fault, the seismic center of the North Izu Earthquake, was estimated to be approximately seven hundred (700) years, the New Tanna Tunnel for the Tokaido Shinkansen which was a high speed railway, was constructed between 1959 and 1964 because of the relatively long term activity cycle.
(Source: https://ja.wikipedia.org/
wiki/%E4%B8%B9%E9%82%A3%E3%83%88%E3%83%B3%E3%83%8D%E3%83%AB)

Another example in recent years was the Inatori Tunnel. The Izu-Oshima Inshore Earthquake (Magnitude 9.0) in 1978 made the 80cm horizontal movement at the eastside mouth and the 20cm horizontal movement at the west-side mouth.
(Source: http://shake.iis.u-tokyo.ac.jp/home-new/projects/2003/report12355020.html)

On the other hand, an example of aqueduct tunnel damage was the Outlet Tunnel of the 2nd Kakkonda Hydropower Station. The Mid-North Iwate Earthquake (Magnitude 6.1-6.2) in 1998, which was a volcanic earthquake, caused the approximately 40m cracks of concrete lining and the falling of part of the lining near the geological fault. In addition, soil and cobblestones, around 30-50cm, flowed into the tunnel, clogging it. The sketch of damaged tunnel of the 2nd Kakkonda Hydropower Station is as shown in Figure 7.2.2.
(Source: http://shake.iis.u-tokyo.ac.jp/home-new/projects/2003/report12355020.html)



Source: Control of Failure Modes of Civil Infrastructures Experiencing Large Soil Deformations Caused by Surface Fault Ruptures

Figure 7.2.2 Sketch of Damaged Tunnel at the Outlet of 2nd Kakkonda Hydropower Station

The activity cycle of the West Valley fault is estimated at approximately 400 years based on the 4-time movement in the recent one thousand four hundred (1,400) years. If the latest movement is assumed as 1658, there is a high possibility that the next movement will be some years later.

(Source: https://www.rappler.com/move-ph/issues/disasters/knowledge-base/93746-maps-west-valley-fault-earthquake-scenarios)

Hence, it is necessary to have countermeasures against the West Valley Fault since the Parañaque Spillway will cross the fault almost vertically. Some countermeasures for the active fault and the comparison of countermeasures for tunnel at West Valley Fault are as shown in Table 7.2.1.

Table 7.2.1 Comparison of Countermeasures for Tunnel at West Valley Fault

Counter-measures	Flexible Structure	Reinforced Lining	Repair after Movement	Open Channel
Summary	Flexible Structure is adopted for the fault in response to the deformation caused by movement, such as flexible joint and elastic coupling.	For soft ground such as alluvial soil, the lining should be designed strong enough to resist deformation even if the fault will move.	Basically, after the movement, tunnel damage should be repaired. The repair shall be conducted inside of the tunnel if it is broken by the fault movement.	Open channel shall be adopted for Laguna de Bay up to the fault because it is easy to be fixed.
Problem	It may be difficult to adjust the deformation if the movement will concentrate at several meters.	It is very difficult for stiff soil such as soft rock, because the load caused by the movement will be huge.	If the fault moves, it will be necessary to restore the other facilities. However, obtaining the repair money might be difficult.	Land acquisition and resettlement are necessary between Laguna de Bay and intake facility.
Construction	Possible O	Possible O	High possibility of collapse and freshet due to the fault Δ	Very easy and no problem ◎
Operation & Maintenance	No Problem O	No Problem O	Very expensive, if the fault moves. Δ	Very easy due to only open channel ◎

Counter-measures	Flexible Structure	Reinforced Lining	Repair after Movement	Open Channel
Social Environment	No Problem O	No Problem O	No Problem O	Land acquisition for open channel area is necessary. ∆
Natural Environment	No Problem O	No Problem O	Influence on the ground water condition at repair. ∆	A few worsening to natural condition due to open channel ∆
Cost	Generally expensive but depend on the allowable deformation ∆	Generally expensive but depend on the structure type ∆	Initial cost is cheaper, but repair cost is expensive ∆	Expensive for land acquisition and resettlement cost ∆
Evaluation	Not impossible but safety is doubtful because of the structural problem.	Not impossible but safety is doubtful because of the structural problem.	Repair budget will be a problem because huge earthquake and damages will be caused by the fault movement.	The most practical measure even if land acquisition and resettlement are necessary.
	∆: Not impossible	∆: Not impossible	∆: Repair Budget	O: Adopted

Legend: ◎ Excellent; ○ Good; △ Not Good/Some Problems; × Difficult/Impossible
Source: JICA Survey Team

According to Table 7.2.1, "Open Channel Structure" is selected because of no structural problem and easy operation and maintenance even if there is land acquisition and resettlement for open channel area. However, if the land acquisition and resettlement for open channel area are impossible because of the social environmental factors, the location of the Intake Facility shall be moved to the landfill area of Laguna de Bay. In this case, the other structural countermeasures, such as the Flexible Structure in the above table, may be adopted for the landfill plan.

(3) Influence on Subway and Railway Projects

The results of the data collection and hearing survey about the subway and railway projects related to Parañaque Spillway are shown in Table 7.2.2.

Table 7.2.2 Hearing Survey Results for Subway and Railway Projects Related to Parañaque Spillway

Type	Project Name	Summary	Hearing Survey Result	Influence Degree
Railway	LTR-1 Cavite Extension	This line will go from center of Manila through Parañaque City to Las Piñas City. Station plans are existing nearby Parañaque River and Zapote River around the location of the drainage facility (outlet).	According to the obtained plan, there is no influence for Parañaque River because the railway will be located at the left side of San Dionisio River. As for Zapote River, the station is planned at left bank side and the rail line is planned at right bank side.	Negotiations and measures about LRT will be necessary if the drain facility is located in Zapote River.
Subway	Mega Manila Subway Project (JICA)	This line will go from the center of Manila through the center of Parañaque City to the center of Las Piñas City. Both viaduct and subway system plans are maintained.	Not clear progress after JICA's Data Collection Survey in 2015. Therefore, both the viaduct and the subway system are still considered at the location of Parañaque Spillway.	Supposedly no influence because of 50m depth spillway plan, inference of neighbouring construction shall be reviewed.

Type	Project Name	Summary	Hearing Survey Result	Influence Degree
Railway	North-South Railway Project South Line	Existing line will be renovated into the viaduct bridge and double lines. The location is west side of SLEX and east side of power plant. Feasibility Study was completed in 2014.	At Lower Bicutan, no problem because of tunnel spillway. At Sucat, Inlet Open Channel will be crossed on the ground (existing rail height).	If the Inlet will locate at Sucat, it will be necessary to make negotiations and measures.

Source: JICA Survey Team

According to the hearing survey results concerning the above-mentioned projects, the construction of LTR-1 Cavite Extension Project started in May 2017 and planned to be completed by 2021. (Source: http://www.interaksyon.com/lrmc-breaks-ground-on-lrt-1-cavite-extension-project/)

However, the other two projects are only at the planning stage and no detailed design has been completed. Therefore, it is necessary to make negotiations and adjustments in line with the progress of Parañaque Spillway. Incidentally, the assumed countermeasures about this matter are as follows:

- The LTR-1 Cavite Extension is planned to locate at the left bank side of San Dionisio River. Therefore, there is no competition problem with regard to the Parañaque River System. As regards Zapote River, there are some competitions, namely; partial overlap with the railway station at the left bank side and with the rail line at the right bank side, as shown in Figure 7.2.3.
- Hence, the drainage facility maybe planned at the right bank side near the river mouth to avoid competition with the LRT facilities.

Note: Produced by the JICA Survey Team based on Data of Google Earth and DOTr.

Figure 7.2.3 Plan of LTR-1 Cavite Extension around Parañaque River System and Zapote River

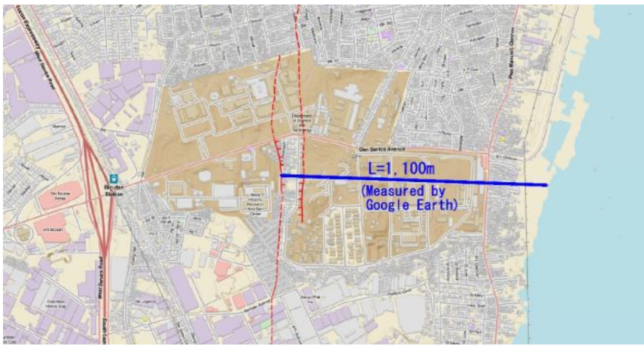
- With regard to the planned Mega Manila Subway Project, neither the viaduct bridge nor the underground subway system at the location of Parañaque Spillway has been decided. However, the height of the railway line is planned to be placed at a relatively shallow depth, approximately EL. -5m, even in the case of a subway system. In this case, the Parañaque Spillway shall be planned 50 meters deeper than the ground level (deeper than EL -34m), under the subway plan, considering the sectional surface rights. The countermeasures about this matter may be possible and not a serious problem because the space between the subway and the spillway is more than two times of tunnel diameter which is generally assumed as the loose soil area caused by the tunnel. However, it shall be supposedly necessary to study on the inference of neighbouring construction and to conduct the adequate measurement survey if the construction of Parañaque Spillway is done after the subway.

- As regards the North-South Railway Project South Line, there will be a competition only if the intake facility is planned at Sucat. The railway is planned to be located at the open channel. Therefore, two solutions are possible. One is the box culvert type instead of the open channel and another is the railway bridge type. Either one should be selected by future negotiation. However, this will not be an issue because it is possible to take care of the measurement by design.

7.2.2 Alignment of Parañaque Spillway

(1) Location of the Intake Facility

According to the results in Section 4.3, "Study on Parañaque Spillway," the possible location of the intake facility is either Lower Bicutan or Sucat in consideration of the spillway length and right-of-way for the construction road. Geological fault maps of Lower Bicutan and Sucat to be utilized for the selection of the intake facility are shown in Figure 7.2.4 and Table 7.2.5, respectively. Since the distances from Laguna de Bay to the West Velley Fault at Lower Bicutan and Sucat are 1,100 m and 500 m, respectively (measured with Google Earth), the lengths of the open channels at Lower Bicutan and Sucat are decided to be L_{O1} = 1,200 m and L_{O2} = 600 m, respectively, to stride over the West Valley Fault. (The detailed fault location must be confirmed by future geological boring survey.)



Source: http://faultfinder.phivolcs.dost.gov.ph/

Figure 7.2.4 Geological Fault Map of Lower Bicutan

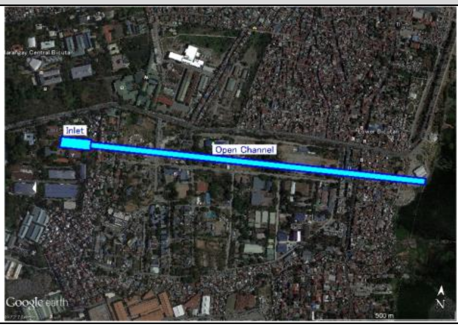
Source: http://faultfinder.phivolcs.dost.gov.ph/

Figure 7.2.5 Geological Fault Map of Sucat

CTI Engineering International Co., Ltd.
Nippon Koei Co., Ltd.
CTI Engineering Co., Ltd.

The comparison of intake facility locations of Parañaque Spillway is shown in Table 7.2.3.

Table 7.2.3 Comparison of Intake Facility Locations of Parañaque Spillway

Place	Lower Bicutan	Sucat
Location of Intake Facility (Google Earth)		
Spillway Length (Measured by Google Earth)	Parañaque River System Lp = 6.0 km Zapote River Lz = 9.1 km Open Channel Length Lo = 1.2 km	Parañaque River System Lp = 6.8 km Zapote River Lz = 8.8 km Open Channel Length Lo = 0.6 km
Site of Intake Facility	It is necessary to relocate large-scale facilities, such as Polytechnic University of Philippines. Δ	Mainly unused ground is widely spaced but adjacent to church. O
Construction	No problem O	No problem O
Operation & Maintenance	No problem O	No problem O
Social Environment	The length of 1200m of the Lower Bicutan Open Channel is longer than Sucat and the land acquisition area is also wider. However, a minimized relocated route may be selected, possibly. Δ	The length of 600m of the Sucat Open Channel is shorter than Lower Bicutan and fewer resettlements is an advantage, except for Laguna de Bay lakeshore area. O
Natural Environment	Disadvantage due to longer open channel length Δ	Advantage due to shorter open channel length O
Effect of Subway & Railway	No problem O	Open channel will cross the railway Δ
Cost	Basically, more economical than the one in Sucat, but depends on the drained river improvement area. O	More expensive because of the longer tunnel length. Δ
Evaluation	Even if it involves a wider land acquisition and many resettlements, it is more economical compared to Sucat.	Even if disadvantageous due to cost and the effect of railway, this is advantageous from the point of view of social environment, especially, land acquisition.
	O: More economical	O: Easy Land Acquisition

Legend: ◎ Excellent; ○ Good; △ Not Good/Some Problems; × Difficult/Impossible

Source: JICA Survey Team

According to Table 7.2.3, about the spillway length, the location of the intake facility is an advantage for the Parañaque River System and a disadvantage for Zapote River. However, the lengths of open channel at Lower Bicutan, approximately 1.2 km, is twice as long as the one in Sucat, which is approximately 0.6 km, and land acquisition is nearly the same. In addition, the number of resettlement houses is large and there are large-scale facilities in Lower Bicutan, so that negotiations will be difficult. Therefore, with respect to social environment and project feasibility, the Lower Bicutan's case has a disadvantage. In conclusion, it is presently difficult to decide on either one of the intake facilities; hence, the Study concentrated on the possibility of both cases.

(2) Location of Drainage Facility

In Section 4.3, "Study on Parañaque Spillway", the several Drainage Systems to Manila de Bay have been reviewed and the direct drainage two (2) method, the Jetty (Seawall) System and the Direct Drainage System, have been denied due to the problems of the land fill of Manila Bay and the river mouth clogging. Therefore, the existing river connection method has been selected including three rivers, namely; the Parañaque River System (South Parañaque River and San Dionisio River) and Zapote River, because of land use, availability of open space at the site and so on.

Regarding the three (3) rivers, the results of the comparison of locations of the drainage facility of the Parañaque Spillway are as shown in Table 7.2.4.

Table 7.2.4 Comparison of Locations of Drainage Facility of Parañaque Spillway

River Name	South Parañaque River	San Dionisio River	Zapote River
Location of Drainage Facility (Google Earth)			
Spillway Length (Measured by Google Eearth)	South Parañaque-Lower Bicutan Spillway Lp = 6.0 km Open Channel Lo = 1.2 km South Parañaque-Sucat Spillway Lp = 6.8 km Open Channel Lo = 0.6 km	San Dionisio-Lower Bicutan Spillway Lp = 6.6 km Open Channel Lo = 1.2 km San Dionisio-Sucat Spillway Lp = 7.2 km Open Channel Lo = 0.6 km	Zapote River-Lower Bicutan Spillway Lp = 9.1 km Open Channel Lo = 1.2 km Zapote River-Sucat Spillway Lp = 8.8 km Open Channel Lo = 0.6 km
Site of Drainage Facility	There is sufficient open space between upstream and downstream, which are the Carlos P. Garcia Avenue exits. O	There is adequate open space between Parañaque Police Center and Premier Medical Center. O	There is substantial open space at the right bank side where there are a few houses even if avoiding the viaduct bridge area. O
River Improvement	The channel width around the drainage facility and up and down stream is not enough, but there are a few houses. In addition, the necessity of the improvement is high due to 270 m^3/s^{*1} in 30-year return period. Δ	The confluences with South Parañaque River and its upstream are narrow and there are many houses. In addition, the present allowable capacity is small due to 30 m^3/s^{*1} in 30-year return period. Δ	There are a few problems on the river improvement because of the wide river channel and near the river mouth. It is desirable that the design discharge is 480 m^3/s^{*1} in 30-year return period O
Construction	No problem O	Vibration noise measures are necessary due to nearby hospital. Δ	No problem O
Operation & Maintenance	No problem O	No problem O	No problem O
Social Environment	No problem due to open green area O	No problem due to unused ground O	No problem due to developed land O
Natural Environment	No problem due to open green area but precious species survey is necessary. O	No problem due to developed land O	No problem due to developed land O
Influence to LPPCHEA	Relatively larger influence than Zapote River case. The final decision should take into account the result of the diffusion analysis of drainage water. Δ	Relatively larger influence than Zapote River case. The final decision should take into account the result of the diffusion analysis of drainage water. Δ	Relatively smaller influence than Parañaque River case. The final decision should take into account the result of the diffusion analysis of drainage water. O

River Name	South Parañaque River	San Dionisio River	Zapote River
Effect of Subway & Railway	No problem O	No problem O	No problem to avoid the site of the railway facility, such as viaduct. O
Cost	The cheapest plan due to the shortest length of spillway O	The intermediate plan among the three. ∆	The most expensive plan among the three. ∆
Evaluation	High possibility for economic reasons, but the feasibility of river improvement and the influence to LPPCHEA still remain as problems.	Good possibility due to the intermediate plan of river improvement of upstream of confluence and drainage facility site.	The most realistic plan despite the relatively expensive cost because the river improvement area is small and the less influence to LPPCHEA.
	O: High Possibility	O: Possible	O: Promising

Legend: ◎ Excellent; ○ Good; △ Not Good/Some Problems; × Difficult/Impossible
Note *1: "The Feasibility Study on Flood Control and Drainage Improvement Project for MIAA Compound and Parañaque-Las Piñas River System in the Republic of the Philippines"
Source: JICA Survey Team

According to Table 7.2.4, it is impossible to select only one plan at this time because the location of the Drainage Facility of Parañaque Spillway must be decided from not only the technical reasons but also the requests and opinions of DPWH and the LGUs. Therefore, the decision will depend on the results of future negotiations. In conclusion, the decision on the location of the Drainage Facility should take into account the following studies and discussions and the results:

- Diffusion analysis of drainage water should be conducted to determine the influence to the LPPCHEA quantitatively in the cases of both Parañaque River Drainage and Zapote River Drainage.
- Collection of current rainfall data/basin data and topographic survey should be conducted for the preparation of river plans including evaluation of the current discharge capacities. After the planning, constituency verification between Parañaque Spillway and drainage river plans should be considered to decide on the appropriate drainage river.
- People residing in the drainage river basin may be anxious about the increase of flooding disasters because of the drainage water from Laguna de Bay. Therefore, to relieve the worries about flooding, a flood mitigation project must be implemented as the incentive to increase the possibility.

(3) Alignment Plan of Spillway

According to the results of the study on the locations of intake and drainage facilities, three alignment plans for Parañaque Spillway are formulated based on the following plan conditions. The alignment plan map and comparison of alignment plans of Parañaque Spillway are shown in Figure 7.2.6 and Table 7.2.5, respectively.[1]

- "No-Bending (Curving) Loss" is desirable to minimize the head loss of water, In addition, interval shaft is not required to intake the floodwaters of the drainage basin. Therefore, straight line alignment should be planned, basically.
- There are two locations for intake facility, Lower Bicutan and Sucat, and three locations for Drainage Facility, South Parañaque River, San Dionisio River and Zapote River. Considering the characteristics of route plans, there are three possible alignments as given below. (The route from Lower Bicutan to Zapote River is disregarded because of longest tunnel length and most expensive cost.)

[1] The routes shown here were discussed at the Pre-F/S level as 'Feasibility Study of Paranaque Spillway'. Routes 1, 2 and 3 are set based on route A, route C and route D shown in fig. 4.3 (2) of 'Chapter 4 Full Menu of Comprehensive Flood Management Plan for Laguna de Bay Lakeshore Area'.

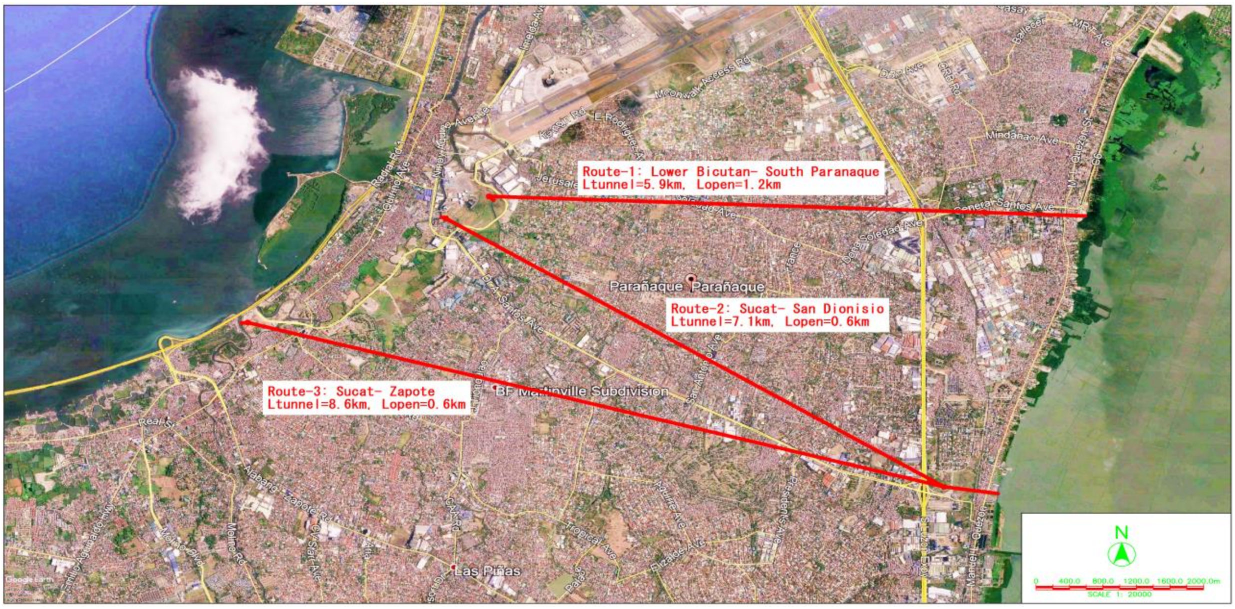
Route-1: Lower Bicutan to South Parañaque River

Route-2: Sucat to San Dionisio River

Route-3: Sucat to Zapote River

Incidentally, the reasons for the selection of different drainage sites for Route-1 and Route-2, which are Lower Bicutan and Sucat, are as follows:

Route-1 is the most economical route with the shortest alignment from Lower Bicutan to South Parañaque River. On the other hand, Route-2 is the most practical alignment considering the connection of the drainage facility to the river, which is San Dionisio River with a wide river channel resulting in smaller scale river improvement works compared to South Parañaque River. In addition, the difference of tunnel length of each route is approximately 400 - 600m, which is not so significant, and it is easy to be converted. Therefore, it is a beneficial decision to make both cases remain as the possible solutions.



Source: JICA Survey Team

Figure 7.2.6 Alignment Plan of Parañaque Spillway

Table 7.2.5 Comparison of Alignment Plans of Parañaque Spillway

Route Name	Route-1 (Lower Bicutan to South Parañaque River)	Route-2 (Sucat to San Dionisio River)	Route-3 (Sucat to Zapote River)
Summary of Spillway Alignment	Basically, straight line between Lower Bicutan and South Parañaque River to minimize the water head loss. (However, the alignment bends upstream of the Outlet Shaft due to the adjustment of inflow angle.)	Basically, straight line between Sucat and San Dionisio River to minimize water head loss. (However, the alignment bends upstream of the Outlet Shaft due to the adjustment of inflow angle.)	Basically, straight line between Sucat and Zapote River to minimize water head loss. (However, the alignment bends upstream of the Outlet Shaft due to the adjustment of inflow angle.)
Spillway Length (Measured by Google Earth)	South Parañaque-Lower Bicutan Spillway Lp = 6.0 km Open Channel Lo = 1.2 km	San Dionisio-Sucat Spillway Lp = 7.2 km Open Channel Lo = 0.6 km	Zapote River-Sucat Spillway Lp = 8.8 km Open Channel Lo = 0.6 km
Site of Intake Facility	It is necessary to relocate large-scale facilities, such as Polytechnic University of Philippines. Δ	Mainly unused ground is widely spaced but adjacent to church. O	

Route Name	Route-1 (Lower Bicutan to South Parañaque River)	Route-2 (Sucat to San Dionisio River)	Route-3 (Sucat to Zapote River)
Site of Drainage Facility	There is sufficient open space between upstream and downstream which are the Carlos P. Garcia Avenue Exits. O	There is adequate open space between Parañaque Police Center and Premier Medical Center. O	There is substantial open space at the right bank side for the viaduct bridge with a few houses avoided. O
River Improvement	Widely required river improvement area due to the narrow existing channel. In addition, it may be necessary to improve the other rivers in the river system. Δ	Required river improvement area is the upstream and downstream of the drainage facility site. In addition, it may necessary to improve the other rivers in the river system. Δ	Required river improvement area is smallest among three rivers because of the wide river channel near the river mouth. O
Construction	No problem O	Vibration noise measures are necessary due to nearby hospital. Δ	No problem O
Operation & Maintenance	No problem O	No problem O	No problem O
Social Environment	The length of 1200m of Open Channel is longer than Sucat and the land acquisition area is also wider. Δ	It is necessary to make resettlement of Laguna de Bay lakeshore area. Δ	It is necessary to make resettlement of Laguna de Bay lakeshore area. Δ
Natural Environment	No problem due to open green area but precious species survey is necessary. O	No problem due to developed land O	No problem due to developed land O
Influence to LPPCHEA	Relatively larger influence than Zapote River Case. The final decision should be considered with the result of the diffusion analysis of drainage water. Δ	Relatively larger influence than Zapote River Case. The final decision should be considered with the result of the diffusion analysis of drainage water. Δ	Relatively smaller influence than Parañaque River Case. The final decision should be considered with the result of the diffusion analysis of drainage water. O
Effect of Subway & Railway	No problem O	Either railway bridge or box culvert is necessary at the Open Channel area. Δ	The same as left at the Open Channel area and the necessity of negotiation of the Drainage Facility site. Δ
Cost	The cheapest plan due to the shortest length of spillway O	More economical than Zapote River Plan due to the short length of tunnel. O	The most expensive plan among the three. Δ
Evaluation	High possibility due to economy but the feasibility of river improvement and the influence to LPPCHEA still remain as problems.	The priority is not so high because of the intermediate plan. It is considered only if both the South Parañaque and Zapote rivers are impossible.	The most realistic plan despite of the relatively expensive cost because the river improvement area is small and the less influence to LPPCHEA.
	O: Possible	Δ	O: Promising

Legend: ◎ Excellent; ○ Good; △ Not Good/Some Problem; × Difficult/Impossible
Source: JICA Survey Team

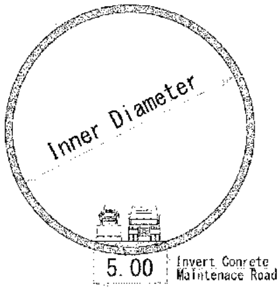
According to Table 7.2.5, it is not desirable to select only one route at this time because all of the three (3) alignment plans have advantages and disadvantages. Especially, the location of drainage facilities require negotiation with the LGUs. Incidentally, there are no river improvement plans and designs because of no detailed study on river drainage in the Survey. In addition, the influence to LPPCHEA should be evaluated because it has been registered under the Ramsar Convention. It is necessary to make the decision quantitatively. Route-3 is tentatively selected for further study because of the less influence to LPPCHEA and small area of the river improvement even though relatively expensive cost. However, it is necessary to study the other two plans as the alternative plans later.

7.2.3 Cross Section Plan and Longitudinal Plan

(1) Cross Section Plan (Inner Section Plan)

(a) Inner Cross Section Plan of Shield Tunneling Method

The cross section of the Shield Tunneling Method is shown in Figure 7.2.7 based on Section 4.3, "Study on Parañaque Spillway." The inner diameter is 12 m and the width of the inner maintenance road is 5 m.



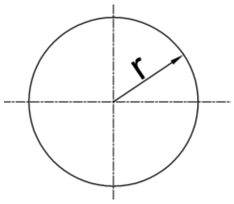
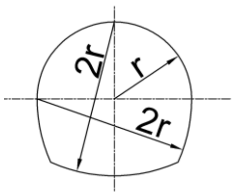
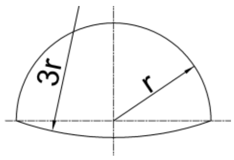
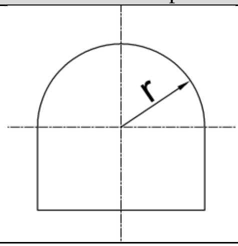
Source: JICA Survey Team

Figure 7.2.7 Cross Section of Shield Tunneling Method

(b) Inner Cross Section Plan of NATM

Comparison of basic shapes for inner cross section of NATM is shown in Table 7.2.6Comparison of Basic Shapes of Inner Cross Section of NATM

Table 7.2.6 Comparison of Basic Shapes of Inner Cross Section of NATM

Shape	Round Shape	Horseshoe Shape	Flat Horseshoe Shape	Hood Shape
Basic Shape				
Characteristics	Standard shape for pressure type tunnel	The most commonly used for open channel tunnel	Advantage for road tunnel and limited water depth tunnel	Adopted since the old times, mainly for stiff rock tunnel.
Advantage	The most economical shape because the stress is sustained by the compression of lining.	Construction of the invert is easy and the stress burden is advantageous due to the almost round shape.	The discharge capacity is quickly increased versus the water depth. Therefore, it is useful for the open channel with a limited water depth.	If there is no lateral pressure, it is economical shape because the vertical load is sustained by the compression of the side walls.
Disadvantage	The construction of invert is relatively difficult. Therefore, another shape is adopted for the excavation figure and then round shape is formulated.	Tension stress at the connection of side wall and invert is almost twice compared with the round shaped pressure pipe tunnel.	Sedimentation increases in the case of small discharge. The stress at the bottom is large and disadvantageous.	The stress is concentrated at the corner of the bottom so that the thickness of wall and deck slab becomes greater.
Evaluation	The round shape is the standard for more than 0.1Mpa (N/mm) of pressure pipe tunnel.	The stress aspect is a disadvantage compared with the round shape. However, it still remains as the discussion subject.	Low selectivity for the pressured pipe tunnel because of the stress disadvantage and low economy.	Low selectivity because of the stress disadvantage and low economy.
	O: Selected	Δ: Discussion Subject	X	X

Legend: ◎ Excellent; ○ Good、△ Not Good/Some Problems、× Difficult/Impossible
Source: JICA Survey Team

In the "Planning & Design Standard for Agricultural Land Improvement Project, Design (Water Tunnel)" (July 2014), the selection of the inner shape for the pressure pipe tunnel is described as follows:

- For the pressure pipe tunnel*1, the inner shape is basically the round shape. However, the standard horseshoe shape is also possible under the water pressure of less than 0.1 Mpa (N/mm) with adequate structural verification.

Note *1: The round shape has more advantages than the standard horseshoe shape for the pressure pipe tunnel because the tensile force of the horseshoe shape caused by the inner water pressure is almost twice of the round shape. Especially, the round shape is desirable for large diameter tunnels. However, the horseshoe shape has, general, advantages for the constructions of excavation and lining more than the round shape. Therefore, under the condition of water pressure less than 0.1 Mpa (N/mm), the tunnel shape should be decided comprehensively taking into account the structural safety, economy, workability and operation & maintenance.

In conclusion, the inner water pressure is 0.54 Mpa and more than 0.10 Mpa (N/mm^2) as shown below. Therefore, the basic shape of the inner section of NATM is decided to be “Round Shape”.

$$(47.0-6.0+14.0) \times 1{,}000 \times 9.81 / 1{,}000{,}000 = 0.54 \text{ Mpa (N/mm}^2\text{)}$$

Incidentally, the abovementioned inner water pressure is offset by the outside water pressure and then it is possible that the difference between both pressures may be less than 0.10Mpa (N/mm^2). However, it is difficult to evaluate the outside water pressure because of the reduction water pressure of the excavated rock layer and the few geotechnical investigations. The “Round Shape” is adequate because it is uncertain that the offset water pressure is less than the limitation. In conclusion, the inner cross section plan of NATM is the same as that of the Shield Tunnelling Method.

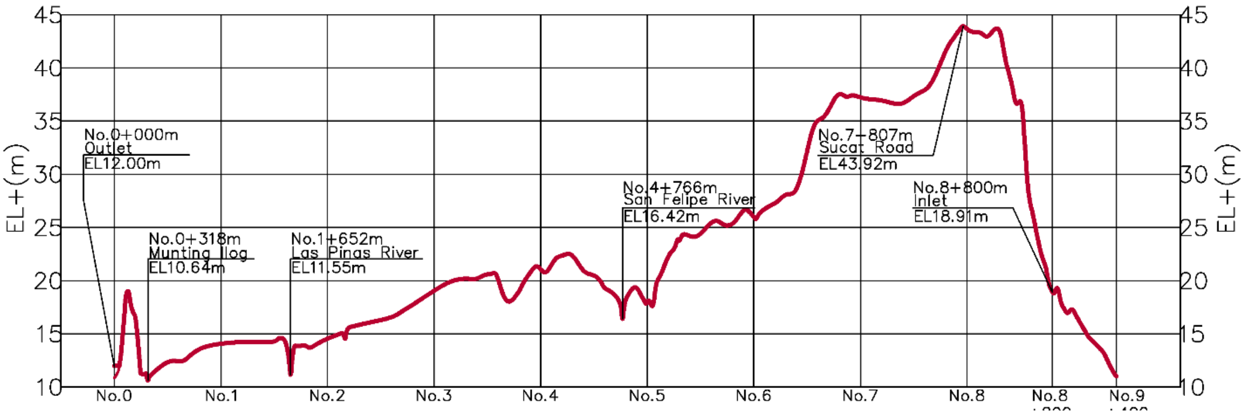
(2) Longitudinal Plan

(a) Basic Longitudinal Plan

According to Section 4.3, “Study on Parañaque Spillway,” the slope of the Basic Longitudinal Plan is “1/1,500” and the direction is “Order Slope” (Inlet to Outlet).

(b) Longitudinal Plan of Shield Tunneling Method

According to Section 11 of the IRR of RA 10752, the depth of the longitudinal plan of Shield Tunneling Method should be more than 50 m to minimize the land acquisition area based on the GIS Data obtained from NAMRIA. The existing ground and the critical point of the longitudinal plan based on GIS Data is shown in Figure 7.2.8.



Source: JICA Survey Team

Figure 7.2.8 Existing Ground Level and Critical Points based on GIS Data

According to Figure 7.2.8, the ground level and longitudinal plan of each critical point of the Shield Tunneling Method are as shown in Table 7.2.7.

Table 7.2.7 Longitudinal Plan of Shield Tunnelling Method

Station	Cumulative Distance (m)	Place	Ground Level EL(m)	Invert Elevation EL(m)	Slope	Soil Cover (m)	Note
No. 0+000	0	Outlet Shaft	+12.00	-52.87	1/1,500	52.27	Complemented Ground
No. 0+318	318	Munting Ilog	+10.64	-52.65		50.69	Critical Point
No. 1+652	1,652	Las Piñas River	+11.55	-51.77		50.72	
No. 4+766	4,766	San Felipe River	+16.42	-49.68		53.50	
No. 7+807	7,807	The Highest Point	+43.92	-47.66		78.98	
No. 8+800	8,800	Inlet Shaft	+18.91	-47.00		53.31	
		Downstream of Open Channel		+10.20	1/2,000	－	Open Channel Section
No. 9+400	9,400	Upstream of Open Channel	+10.99	+10.50		－	

Source: JICA Survey Team

(c) Longitudinal Plan of NATM

For NATM, it is necessary to move the length of tie-rod for NATM construction. As in the Shield Tunnelling Method, according to Section 11 of IRR of RA 10752, the depth of the longitudinal plan of NATM should be more than 50 m to minimize the land acquisition area, and it is 6m deeper than the Shields for the tie rod.

The longitudinal plan of NATM is as shown in Table 7.2.8.

Table 7.2.8 Longitudinal Plan of NATM

Station	Cumulative Distance (m)	Place	Ground Level EL(m)	Invert Elevation EL(m)	Slope	Soil Cover (m)	Note
No.0+000	0	Outlet Shaft	+12.00	-58.87	1/1,500	52.27	Complemented Ground
No. 0+318	318	Munting Ilog	+10.64	-58.65		50.69	Critical Point
No. 1+652	1,652	Las Piñas River	+11.55	-57.77		50.72	
No. 4+766	4,766	San Felipe River	+16.42	-55.68		53.50	
No. 7+807	7,807	The Highest Point	+43.92	-53.66		78.98	
No. 8+800	8,800	Inlet Shaft	+18.91	-53.00		53.31	
		Downstream of Open Channel		+10.20	1/2,000	－	Open Channel Section
No. 9+400	9,400	Upstream of Open Channel	+10.99	+10.50		－	

Source: JICA Survey Team

The above-mentioned longitudinal plans were based on the GIS Data which have a certain amount of error. Therefore, more detailed topographic survey should be necessary, and the plans should be revised in future.

7.2.4 Facility Planning of Spillway

(1) Vertical Shaft

(a) Diameter of Vertical Shaft

In order to plan the diameter of the vertical shaft of Parañaque Spillway, construction examples of the pressure pipe tunnel in Japan are shown in Table 7.2.9.

Table 7.2.9 Construction Examples of Pressure Pipe Tunnels in Japan

Tunnel Name	Design Discharge (m^3/s)	Inner Diameter (m)	Slope	Construction Method	Inner Diameter of Shaft (m)	Note
Metropolitan Area Outer Underground Discharge Channel	200	10.6	1/5000	Slurry Shield	30.0	No.1-3 Vertical Shaft
Loop Road No. 7 Underground Reservoir (I & II)	-	12.5	1/1500	Slurry Shield	26.0	Myousyou-zi River Shaft
Azumagawa Underground River “Plan” (Hyogo Pref.)	171	12.0	1/1300	Slurry Shield	22.0	Diameter was decided by shield departure.
Neya South Underground River	180	9.8	1/1500	Slurry Shield	22.0	Wakae Shaft/ Departure Shaft
Gotanda Discharge Channel	150	8.7	1/1000	Slurry Shield	21.0	Inlet Shaft/ Departure Shaft
Azumagawa Underground River (Saitama Pref.)	63	5.2	1/500	Mud Pressured Shield	13.0	Outlet Shaft
Tsurumi River Onmawashi Park Underground Reservoir	-	16.5m x 15.4m	1/1000	NATM	19.0	Underground Reservoir/ Not Round Shape

Source: JICA Survey Team

According to Table 7.2.9, the diameter of vertical shafts over the inner diameter of more than 10m are widely distributed from 22 m to 30 m. The inner diameter of the vertical shaft should be actually decided by comprehensive study, such as the detailed dimension of shield machine, equipment of departure shaft and hydraulic conditions of Inlet and Outlet shafts. As the facility planning of the Survey, the required inner diameter of Parañaque Spillway is 27m for Shield Tunnelling Method (refer to P7-94) and 25m for NATM (refer to P7-95). In the Survey, the diameter of 30m for Parañaque Spillway, which is the same as the Metropolitan Area Outer Underground Discharge Channel, is considered for later study due to the safety side value.

However, a study on the reduction of diameter of the vertical shafts is necessary to decrease the cost in the future after the decision on the detailed spillway facility, tunnel construction method, detailed construction plan and tunnel hydraulic condition. In addition, each Inlet Shaft and Outlet Shaft should be reviewed separately according to function and construction plan and the adequate diameter should be determined individually.

(b) Vortex Drop Shaft

At the early part of flooding, the water energy of $200m^3/s$ discharge generated by the direct fall from almost 60m is a huge amount of energy that will cause damage to the deck slab of the Inlet Shaft as a waterfall basin. Therefore, a "Vortex Drop Shaft" should be equipped as the energy dissipator in the early part of flooding.

The dimension of the Vortex Drop Shaft should be determined by a more detailed hydraulic study, including a hydraulic model test, because the phenomena of hydraulic condition of the Inlet Shaft are very complicated and totally different. In the Survey, the dimension of the Vortex Drop Shaft is roughly calculated in accordance with the Milwaukee Experiment Report, as follows:

Planning Discharge: Qp = 200 m^3/s

Design Discharge: Qd = 110% x Qp = 1.1 x 200 = 220 m^3/s

Inner Diameter of Vortex Drop Shaft

$$D = \left(\frac{Qd^2}{g}\right)^{0.2} = \left(\frac{220^2}{9.81}\right)^{0.2} = 5.478\text{m} \Rightarrow D = 5.5\text{m}$$

Slit Width: e = D / 4 = 1.375 m

Approach Channel Width: B = 3D / 4 = 3 x 5.5 / 4 = 4.125 m

Assumed Reduction Angle: θ = 16.8 degree

Reduction Channel Length: $L1 = \left(\frac{B-e)}{\tan\theta}\right) = 9.108m \Rightarrow L1 = 10m$

Examples of the Vortex Drop Shaft of the Metropolitan Area Outer Underground Discharge Channel are shown in Figure 7.2.9 (from the Homepage and brochure of the Metropolitan Area Outer Underground Discharge Channel).

a) Explanation of Vortex Drop Shaft; b) Inflow Situation of August 2005 Flooding (No. 3 Vertical Shaft)
Source: http://www.ktr.mlit.go.jp/edogawa/gaikaku/intro/02shuyou/shu-05.html and "Sairyu no Kawa, Metropolitan Area Outer Underground Discharge Channel"

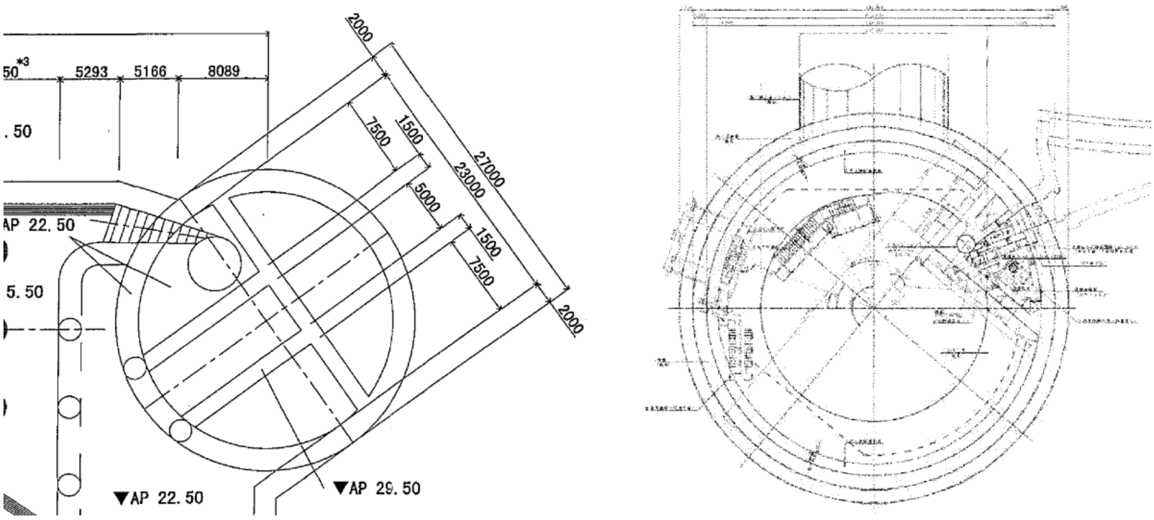
Figure 7.2.9 Vortex Drop Shaft of Metropolitan Area Outer Underground Discharge Channel in Japan

(c) Inlet Vertical Shaft

The residual water of Parañaque Spillway is planned to be drained from the Outlet Vertical Shaft to reduce the damage to the fishery in Laguna de Bay. Therefore, the main units of equipment for Inlet Vertical Shaft are assumed as follows:

- Remaining Water Pump (if necessary, Sand Pump)
- Elevating Equipment (Large-Sized Elevator and Emergency Stairs)
- Equipment for Operation and Maintenance
- Information Management Facility
- Pressure Sealing Door

It is desirable that no equipment of the Inlet Vertical Shaft will maximize the allowable discharge capacity, especially in the pre-discharge stage. However, the above-mentioned equipment to conduct the operation and maintenance is necessary. Inlet vertical shafts of the Gotanda Discharge Channel and the Loop Road No. 7 Underground Reservoir in Japan are as shown in Figure 7.2.10.



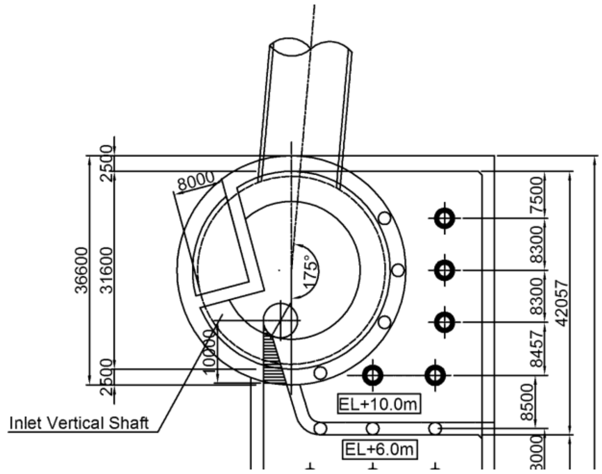
a) Inlet Shaft of Gotanda Discharge Channel; b) Inlet Shaft of Loop Road No. 7 Underground Reservoir

Source: "Final Report on the Design of Fanning Outlet Drainage of Gotanda Discharge Channel" (2000/3) and "Final Report on Construction Work Record No. 2 of Kanda River, Loop Road No. 7 Underground Reservoir" (2008/3) in Japan

Figure 7.2.10 Inlet Vertical Shafts of Gotanda Discharge Channel and Loop Road No. 7 Underground Reservoir as Examples in Japan

The major difference between both vertical shafts is the presence of partition wall in the flow section. The necessity of the partition wall has been decided by hydraulic model test depending on the figures of inflow and water transmission, relative positions and hydraulic condition of inflow. No-partition wall was planned in the Survey because "wider inflow cross section is generally preferable". The detailed dimensions of operation and maintenance facility should be validated in future.

Plan drawing of Inlet Vertical Shaft of Parañaque Spillway, which is based on the past construction examples, is shown in Figure 7.2.11.



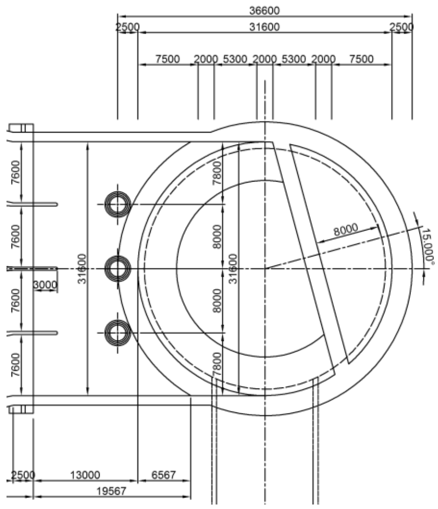
Source: JICA Survey Team

Figure 7.2.11 Plan Drawing of Inlet Vertical Shaft, Parañaque Spillway

The major differences between the former Master Plan and this Pre-Feasibility Study are the alignment angle of the spillway tunnel and the configuration of the dry area. In the latter, the dry area is planned as a half of the former because of no-main drainage pumps and only remaining water pump. Therefore, to minimize the water head loss of the Inlet Vertical Shaft, the inflow section should be minimized as much as possible. This dry area should be reviewed and reduced in the future to adjust the detailed facility plans and the study on operation and maintenance. It is necessary to optimize the dry area and the vortex drop shaft. In addition, the structural design of the Inlet Vertical Shaft, which is taken into account with the earthquake performance examination against the West Valley Fault, should be conducted early because the Frame Beam for Shield Departure is the obstacle to water movement.

(d) Outlet Vertical Shaft

Plan drawing of the Outlet Vertical Shaft of Parañaque Spillway, which is based on the past construction examples, is shown in Figure 7.2.12.



Source: JICA Survey Team

Figure 7.2.12 Plan Drawing of Outlet Vertical Shaft, Parañaque Spillway

Incidentally, the inflow angle of the spillway tunnel is set at the vertical even if a straight line between Inlet Vertical Shaft and Outlet Vertical Shaft is desirable to reduce the tunnel length because the allocation of both the dry area and the shield departure area is difficult due to competition of the areas. Therefore, the Outlet Vertical Shaft is not changed from the Master Plan. However, the modification should be revised in future as same as the Inlet Vertical Shaft to optimize the planning, Especially, the decrease of the dry area and the reduction of the main drainage facility should be considered to linearize the tunnel alignment.

- The width of the existing dry area is 8m considering the main drainage pumps and the operation and maintenance equipment, but it is desirable to reduce this area to maximize the water inflow cross section.
- The inflow angle of the existing spillway tunnel is vertical, but it should be revised. It is, therefore, necessary to conduct a study on dry area re-allocation after the drainage river, drainage facility, drainage location and final alignment of the spillway tunnel are decided.
- As in the Inlet Vertical Shaft, the structural design of the Outlet Vertical Shaft, which shall be taken into account with the earthquake performance examination against the Wet Valley Fault, should be conducted early because the Frame Beam for Shield Departure is an obstacle to water movement.

(2) Intake Facility

(a) Condition of Lake Bottom of Laguna de Bay

The contour drawing of Sucat and Laguna de Bay in Figure 7.2.13 shows the condition of the lake's bottom of Laguna de Bay.

Source: Produced by JICA Survey Team based on Google Earth (Photo) and elevation data from IFSAR and "Laguna de Bay Depth in meter, 2017" by NAMRIA

Figure 7.2.13 Contour Drawing of Sucat and Lake Bottom of Laguna de Bay

According to Figure 7.2.13, the lake's bottom elevation at the west-side lakeshore of Sucat is approximately EL. 11.0 m. However, this elevation is not practicable for the invert elevation of the open channel, because of the assumed wide width of the open channel due to the operation start water level of EL. 12.0 m. The following two cases were then examined in the Survey, but a detailed study should be conducted in future. The topographic sounding survey and review should be re-conducted after making a decision on the location of the intake facility.

- Case 1: EL. 10.5 m (Dredging for approximately 260 m offshore)
- Case 2: EL. 10.0 m (Dredging for approximately 440 m offshore)

(b) Intake Open Channel

The most important condition for the design of the Intake Open Channel is the channel width which is significantly influenced by the channel slope. Especially, it is desirable to minimize the total water head loss for a Gravity Flow Pressure Pipe Tunnel, such as Parañaque Spillway. In this point, the water head loss of the Intake Open Channel is hopefully satisfied within the water head loss on the spillway tunnel. Therefore, the slope of the open channel is calculated in this concept.

(i) Slope of Intake Open Channel

Design conditions for Parañaque Spillway to evaluate the slope of Intake Open channel are defined as follows:

Maximum Total Length of Parañaque Spillway	L = 9,900 m
Offshore Distance of Laguna de Bay	L1 = 500 m
Intake Open Channel Length	L2 = 600 m
Effective Length of Parañaque Spillway	L3 = 8,800 m

<u>Case-A: Equivalent Slope of Spillway Water Head Loss at Operation Start Water Level (EL. 12.0 m)</u>

From Manning's Equation:

$$V = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2}$$

Where V : Average Velocity
n : Manning's Roughness Coefficient
R : Hydraulic Radius (R=d/4 for Round Shape)

Therefore;

$$I = \frac{n^2 \cdot V^2}{R^{4/3}}$$

$$I = 1/\alpha = 1/\left(\frac{R^{\frac{4}{3}}}{n^2 \cdot V^2}\right) = 1/\left(\frac{\left(\frac{11.291}{4}\right)^{\frac{4}{3}}}{0.015^2 \cdot 1.478^2}\right) = 1/8{,}116$$

Case-B: Equivalent Slope of Spillway Water Head Loss at Design High Water Level (EL. 14.0 m) and Design Discharge of 200 m^3/s

$$I = 1/\alpha = 1/(\frac{R^{\frac{4}{3}}}{n^2 \cdot V^2}) = 1/(\frac{\left(\frac{11.291}{4}\right)^{\frac{4}{3}}}{0.015^2 \cdot 1.997^2}) = 1/4{,}445$$

Case-C: Equivalent Slope of Spillway Water Head Loss at Operation Start Water Level (EL. 12.0 m) Considering the Reduction of Spillway Length

$$I = \frac{1}{\alpha} = \frac{1}{8{,}116 \cdot \frac{600}{1100}} = 1/4{,}427$$

Case-D: Equivalent Slope of Spillway Water Head Loss at Design High Water Level (EL. 14.0 m) Considering the Reduction of Spillway Length

$$I = \frac{1}{\alpha} = \frac{1}{4{,}445 \cdot \frac{600}{1100}} = 1/2{,}425$$

Case-E: Equivalent Slope of Spillway Water Head Loss at Operation Start Water Level (EL. 12.0 m) Considering the Reduction of Spillway Length and No Head Loss of Overflow Dike

$$I = \frac{1}{\alpha} = 1/(\frac{600}{\frac{1}{8{,}116} \cdot 1{,}100 + 0.233})) = 1/1{,}628$$

Case-F: Construction Examples of Pasig-Marikina River Improvement Project

Mangahan Floodway	I = 0.0003 = 1 / 3,333
Napindan Channel	I = 0.0007 = 1 / 1,429

A gentle slope for the open channel shall be taken into account together with the hydraulic condition of the total water head loss of the spillway. However, a smaller slope of open channel would require a wider land acquisition. Therefore, it is necessary to decide on the appropriate slope and hydraulic condition of the open channel by considering both aspects.

In the Survey, "Slope I = 1/2,000 = 0.0005" was decided, based on the above-mentioned Case-C to Case-E and Case-F, the construction examples, to minimize the land acquisition area.

(ii) Design Policy of Intake Open Channel

The most important aspect of the design of Open Intake Channel is to minimize land acquisition and resettlement because of the less influence of social environment. Hence, to minimize the land acquisition area, the design policy for Open Intake Channel is as follows:

Fundamental Structure	The most general structure of open channel is, that the river bank is protected by revetment and the riverbed remains as sand and gravel. However, roughness coefficient shall range from 0.020 to 0.030 according to Table 7.2.10, and a wide channel section is necessary.

Therefore, "Concrete 3 Face Revetment Lining" is selected as the fundamental structure of Intake Open Channel.

Roughness Coefficient: n = 0.015 (According to Table 7.2.10, Manning's Roughness Coefficient of Smooth Concrete is 0.014 to 0.018. Generally, the mean value of 0.016 is adopted, but 0.015 is selected as in the spillway tunnel depending on elaborate construction supervision.)

Bank Slope on Channel: 1 : 0.5 (To minimize the land acquisition area, the same bank slope of inclined wall of Pasig-Marikina River Improvement Project is adopted.)

Table 7.2.10 Manning's Roughness Coefficient of Man-Made Channels (Steady Flow)

Table 4-4 Values of Manning's Roughness Coefficient 'n' (Uniform Flow) – Man-made Channels & Ditches

Description	Minimum	Maximum
1. Earth, straight & uniform	0.020	0.025
2. Earth bottom, rubble sides / riprap	0.030	0.035
3. Grass covered	0.035	0.050
4. Dredged	0.028	0.033
5. Stone lined & rock cuts, smooth &uniform	0.030	0.035
6. Stone lined & rock cuts, rough & irregular	0.040	0.045
7. Lined - smooth concrete	0.014	0.018
8. Lined - grouted riprap	0.020	0.030
9. Winding sluggish canals	0.025	0.030
10. Canals with rough stony beds, weeds on earth banks	0.030	0.040

Source: "Design Guidelines, Criteria and Standards", DPWH

(iii) Case-1: Hydraulic Condition of Open Intake Channel for Invert Level of EL. 10.5 m

Upstream Water Level	EL. 12.0 m (EL. 14.0 m)
Design Discharge	Qd = 130 m^3/s (200 m^3/s)
Design Invert Level	EL. 10.5 m
Open Channel Slope	I = 1 / 2,000
Channel Bottom Width	B = 46.0 m
Channel Bank Slope	H : V = 1 : 0.5
Allowable Discharge Capacity	Qc = 132 m^3/s > 130 m^3/s OK (530 m^3/s > 200 m^3/s OK)

Table 7.2.11 Hydraulic Condition of Case-1 (EL. 10.5 m) at Discharge of 130 m³/s and 200 m³/s

Design Discharge Qd=		130.00	m3/s at EL+12.0m	Design Discharge Qd=		200.00	m3/s at EL+14.0m
<Input Data>				<Input Data>			
Bottom Width	B1=	46.000	(m)	Bottom Width	B1=	46.000	(m)
Water Depth	h=	1.50	(m)	Water Depth	h=	3.50	(m)
Slope (Left)	1:	0.50		Slope (Left)	1:	0.50	
Slope (Right)	1:	0.50		Slope (Right)	1:	0.50	
Gradient	I=	0.05000	(%)	Gradient	I=	0.05000	(%)
	= 1/	2,000			= 1/	2,000	
Roughness Coe.	n=	0.015		Roughness Coe.	n=	0.015	
<Output Data>				<Output Data>			
Water Width	B2=	47.500	(m)	Water Width	B2=	49.500	(m)
Flow Area	A=	70.125	(m2)	Flow Area	A=	167.125	(m2)
Wetted Perimeter	P=	49.354	(m)	Wetted Perimeter	P=	53.826	(m)
Hydraulic Radius	R=	A/P		Hydraulic Radius	R=	A/P	
	=	1.421	(m)		=	3.105	(m)
Velocity	V=	1/n*R^(2/3)*I^(1/2)		Velocity	V=	1/n*R^(2/3)*I^(1/2)	
	=	**1.884**	(m/s)		=	**3.173**	(m/s)
Cal. Discharge	Q=	**132.116**	(m3/s)	Cal. Discharge	Q=	**530.288**	(m3/s)
	≧	**130.000**	(m3/S) ---OK		≧	**200.000**	(m3/S) ---OK

Source: JICA Survey Team

(iv) Case-2: Hydraulic Condition of Open Intake Channel for Invert Level of EL. 10.0 m

Upstream Water Level	EL. 12.0 m (EL. 14.0 m)
Design Discharge	Qd=130 m^3/s (200 m^3/s)
Design Invert Level	EL. 10.0 m
Open Channel Slope	I = 1 / 2,000
Channel Bottom Width	B = 29.0 m
Channel Bank Slope	H : V = 1 : 0.5
Allowable Discharge Capacity	Qc = 132 m^3/s > 130 m^3/s OK (407 m^3/s > 200 m^3/s OK)

Table 7.2.12 Hydraulic Condition of Case-1 (EL. 10.0 m) at Discharge of 130 m³/s and 200 m³/s

Design Discharge Qd=		130.00	m3/s at EL+12.0m	Design Discharge Qd=		200.00	m3/s at EL+14.0m
<Input Data>				<Input Data>			
Bottom Width	B1=	29.000	(m)	Bottom Width	B1=	29.000	(m)
Water Depth	h=	2.00	(m)	Water Depth	h=	4.00	(m)
Slope (Left)	1:	0.50		Slope (Left)	1:	0.50	
Slope (Right)	1:	0.50		Slope (Right)	1:	0.50	
Gradient	I=	0.05000	(%)	Gradient	I=	0.05000	(%)
	= 1/	2,000			= 1/	2,000	
Roughness Coe.	n=	0.015		Roughness Coe.	n=	0.015	
<Output Data>				<Output Data>			
Water Width	B2=	31.000	(m)	Water Width	B2=	33.000	(m)
Flow Area	A=	60.000	(m2)	Flow Area	A=	124.000	(m2)
Wetted Perimeter	P=	33.472	(m)	Wetted Perimeter	P=	37.944	(m)
Hydraulic Radius	R=	A/P		Hydraulic Radius	R=	A/P	
	=	1.793	(m)		=	3.268	(m)
Velocity	V=	1/n*R^(2/3)*I^(1/2)		Velocity	V=	1/n*R^(2/3)*I^(1/2)	
	=	**2.200**	(m/s)		=	**3.283**	(m/s)
Cal. Discharge	Q=	**132.000**	(m3/s)	Cal. Discharge	Q=	**407.092**	(m3/s)
	≧	**130.000**	(m3/S) ---OK		≧	**200.000**	(m3/S) ---OK

Source: JICA Survey Team

(v) Decision of Intake Open Channel

Only the land acquisition area and resettlement, Case-2: EL. 10.0 m, has the advantage but the length of the dredging on Laguna de Bay involves approximately 440 m, which is a relatively long distance. The lakeshore of Laguna de Bay is subject to sedimentation because of the soil transported by rivers and the landfill of the lakeshore side. Therefore, frequent operation and maintenance dredging is expected to have a serious influence to the lake fishery. In addition, large dredging would be difficult to negotiate with the LLEDP. In the survey, “Case-1: EL. 10.5 m” is adopted because of the following reasons:

- The frequent dredging for Operation and Maintenance will have a serious influence to fishery, which will cause a social environment problem. In addition, the cost will be expensive.
- To minimize the Operation and Maintenance of the Spillway, transported soil should be limited into the tunnel. Therefore, the lower velocity of Intake Open Channel is desirable because of the enforcement of soil sedimentation through the open channel. In addition, a lower velocity is also preferable to maintain a lower roughness coefficient against friction wear.
- It will be possible to lower the invert level and to narrow the channel width revised by the initial flow discharge, if the negotiation for land acquisition becomes difficult.
- The channel width of 50 m is a sensible range for the floodway channel and not so large. In addition, a certain amount of width is necessary to consider the floating debris from Laguna de Bay, such as drift wood.
- The height of channel bank at the most downstream becomes 5 m. It is possible to be more than 5 m, structurally, but desirable within 5 m height to consider the aseismic design.
- According to “Planing and Design Guidline for Sewage System” (2009/9, Japan Sewage Works Association), allowable maximum velocity of drainage facility is less than va = 3.0 m/s but the calculated velocity in Case-1 at EL. 14.0 m is v=3.173 m/s and a little more than it. However, the actual velocity of the Spillway is only 200 m^3/s controlled by the inflow gates and the velocity of this case is v = 2.207 m/s, as shown in Table 7.2.13, and less than va = 3.0 m/s. In addition, the velocity at the design high water level is V=1.197 m/s and also lower than it.

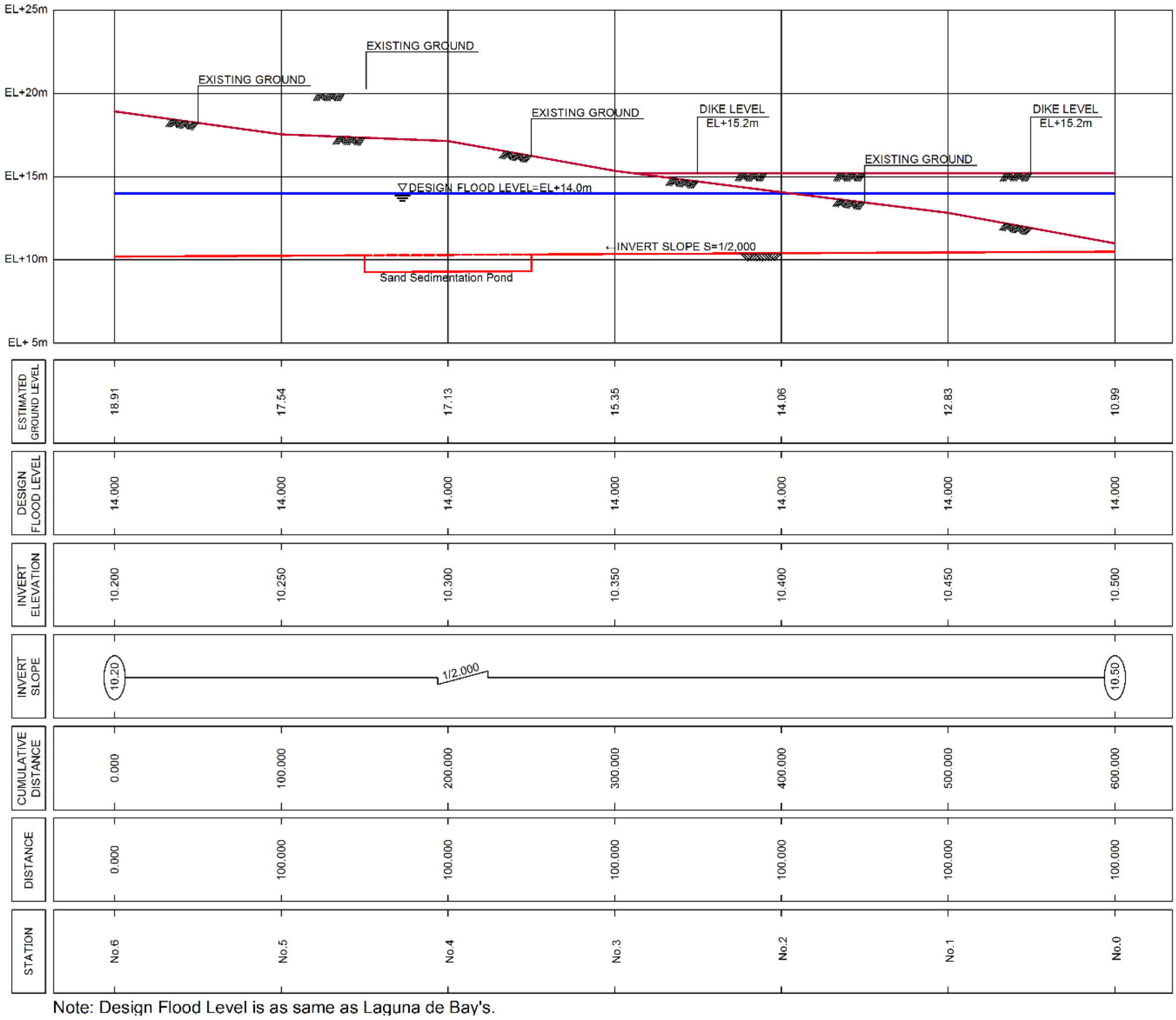
Table 7.2.13 Hydraulic Condition of Open Channel at Design Discharge of and 200m^3/s

Design Discharge Qd= 200.00 m3/s in Case-1				**Design Discharge Qd= 200.00 m3/s in Case-2**			
<Input Data>				**<Input Data>**			
Bottom Width	B1=	46.000	(m)	**Bottom Width**	B1=	29.000	(m)
Water Depth	h=	1.93	(m)	**Water Depth**	h=	2.58	(m)
Slope (Left)	1:	0.50		**Slope (Left)**	1:	0.50	
Slope (Right)	1:	0.50		**Slope (Right)**	1:	0.50	
Gradient	I=	0.05000	(%)	**Gradient**	I=	0.05000	(%)
	= 1/ 2,000				= 1/ 2,000		
Roughness Coe.	n=	0.015		**Roughness Coe.**	n=	0.015	
<Output Data>				**<Output Data>**			
Water Width	B2=	47.930	(m)	**Water Width**	B2=	31.581	(m)
Flow Area	A=	90.621	(m2)	**Flow Area**	A=	78.185	(m2)
Wetted Perimeter	P=	50.315	(m)	**Wetted Perimeter**	P=	34.772	(m)
Hydraulic Radius	R=	A/P		**Hydraulic Radius**	R=	A/P	
	=	1.801	(m)		=	2.249	(m)
Velocity	V=	1/n*R^(2/3)*I^(1/2)		**Velocity**	V=	1/n*R^(2/3)*I^(1/2)	
	=	**2.207**	(m/s)		=	**2.559**	(m/s)
Cal. Discharge	Q=	**200.001**	(m3/s)	**Cal. Discharge**	Q=	**200.075**	(m3/s)
	≧	**200.000**	(m3/S) ---OK		≧	**200.000**	(m3/S) ---OK

Source: JICA Survey Team

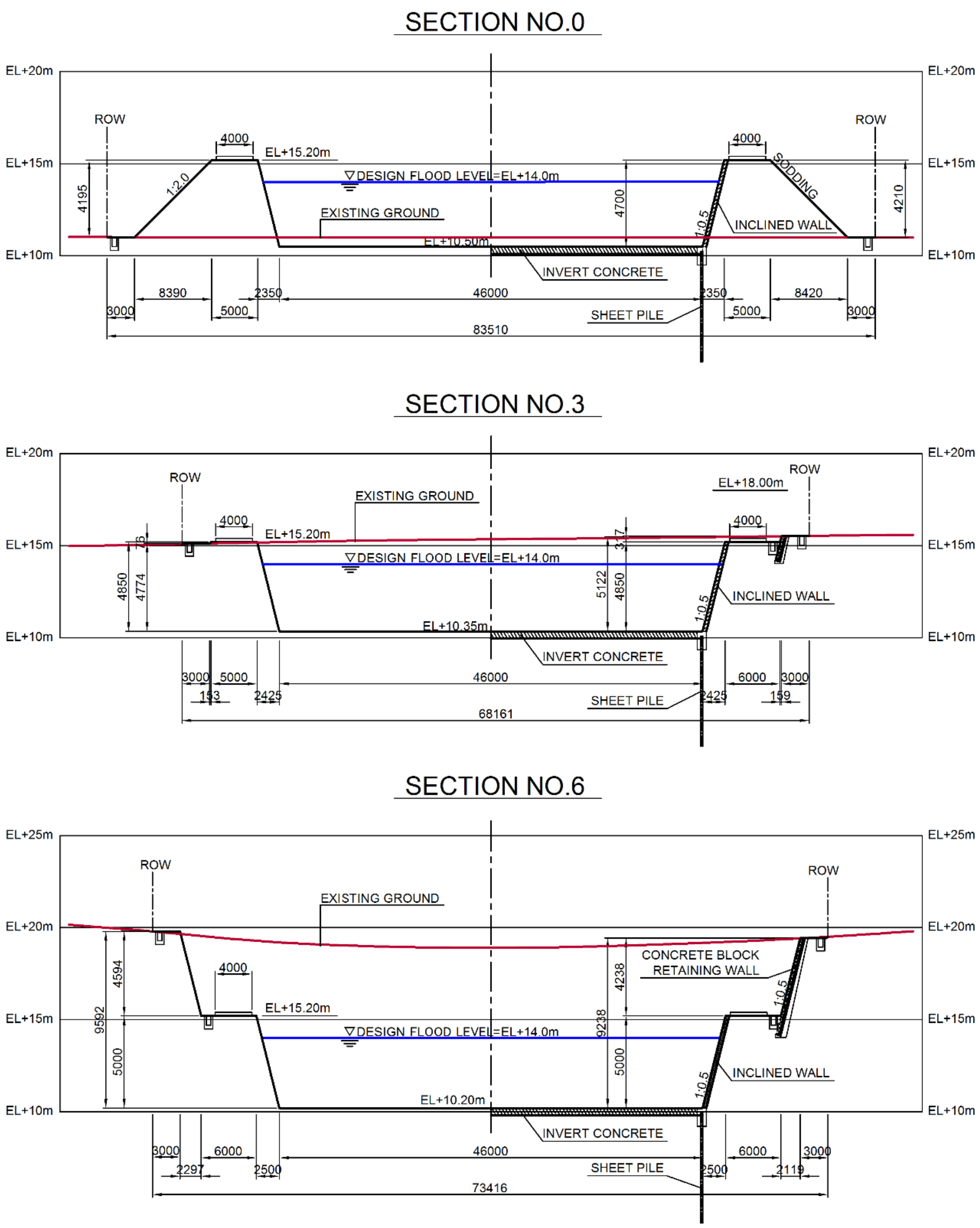
(vi) Plan Drawing of Open Intake Channel

The longitudinal profile and standard cross section of Open Intake Channel are shown in Figure 7.2.14 and Figure 7.2.15, respectively. Incidentally, elevations and location plan drawings are based on the Google Earth and GIS data obtained from NAMRIA. Therefore, there may be some errors. In addition, the topographic sounding survey obtained from LLDA was not verified, precisely. Therefore, it is necessary to review early the planning after the determination of spillway alignment and the topographic survey.



Source: JICA Survey Team

Figure 7.2.14 Longitudinal Profile of Open Intake Channel



Source: JICA Survey Team

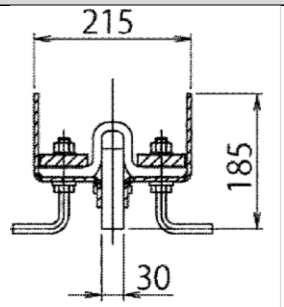
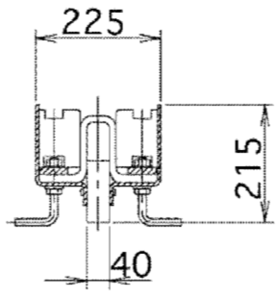
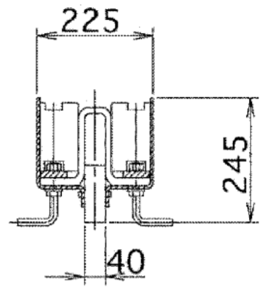
Figure 7.2.15 Standard Cross Section of Open Intake Channel

(vii) Flexible Joint for Deformation Adjustment against West Valley Fault

It is difficult to predict the movement of the West Valley Fault, precisely. Therefore, the design of the Open Intake Channel should consider the deformation to be caused on it. The normal joint of the concrete channel allows only 2 to 5 cm. If the fault moves at 2 cm a year, repair works would be necessary in every 1 or 2 years. Practically, such maintenance repair works are not feasible.

Therefore, a flexible joint should be installed over the fault location to avoid the frequent repair works. Generally, the dimensions of flexible joint for concrete structures are as shown in Table 7.2.14.

Table 7.2.14 Comparison of Dimensions of Flexible Joint for Concrete Structure

Allowable Deformation	10 cm Measure	20 cm Measure	30 cm Measure
Cross Section			
Allowable Settlement	100 mm	200 mm	300 mm
Allowable Length	100 mm	100 mm	120 mm
Allowable Shrinkage	30 mm	40 mm	40 mm
Inside Water Pressure	0.15 Mpa	0.15 Mpa	0.15 Mpa
Outside Water Pressure	0.15 Mpa	0.15 Mpa	0.15 Mpa
Unit Weight	70 kg/m	90 kg/m	100 kg/m
Minimum Thickness	250 mm	300 mm	350 mm

Source: http://www.seibu-p.co.jp/product/construction/bcjoint_u/index.html

In the Survey, the 30cm Measure Type is generally recommmended beccause of the safety side. However, the dimension change of the flexible joits is possible and it is necessary to determined the most adeqate figure and type in the future. Incidentally, as the construction experices in Philippines, it has been used for Pasig- Marikina River Channel Improvement Project and Tagoloan River Flood Risk Managament Project.

Incidentally, there is 70cm defference between end of the open channel at EL.10.2m and bottome of the connnection channel at 9.5m as the transition area. Therefore, if the upheaval of Manila Bay Side is less than 70cm, the reconstruction area shall be only within the transition channel to prevent the adverse slope.

(c) Overflow Dike

To assist in deciding the overflow dike of Parañaque Spillway, the dimensions of overflow dike of river tunnels in Japan are given as examples in Table 7.2.15.

Table 7.2.15 Dimensions of Overflow Dike of River Tunnels in Japan

River Tunnel Name	Location	Overflow Discharge (m^3/s)	Overflow Dike Length (m)	Unit Overflow Discharge ($m^3/s/m$)	Remarks
Metropolitan Area Outer Underground Discharge Channel	Naka River	25	17	1.47	
	Kuramatsu River	100	53	1.89	
	Oootoshi Furutone River	85	33	2.58	
	No. 18 Channel	4.7	4.1	1.14	
	Koumatsu River	6.2	9.0	0.69	

River Tunnel Name	Location	Overflow Discharge (m^3/s)	Overflow Dike Length (m)	Unit Overflow Discharge ($m^3/s/m$)	Remarks
Loop Road No. 7 Underground Reservoir (II)	Myoushouzi River	45 (9.7)	22.5 (11.5)	2.00 (0.84)	・Plan of 104.2m^3/s but with land problem ・Temporary Value in parentheses
Azumagawa Underground River “Plan” (Hyogo Pref.)	Azuma River	47.0	30	1.57	
	Tsumon River	35.0	20	1.75	
Gotanda Discharge Channel	Gotanda River	150	64.25	2.34	
Azumagawa Underground River (Saitama Pref.)	No. 1 Shaft	23	9.5	2.42	
	No. 2 Shaft	10	4.8	2.08	
	No. 3 Shaft	30	8.5	3.53	
Tsurumi River Onmawashi Park Underground Reservoir	Onmawashi Park	33	80	0.41	

Source: JICA Survey Team

In general, the dimensions of overflow dike of river tunnel is decided through the following steps:

Step-1: Case study on several overflow figures and hydraulic condition of target-river

Step-2: Formulation of some overflow plans considering area condition

Step-3: Decision on both height and length of the overflow dike considering frequency and future improvement plan

However, Parañaque Spillway has the following characteristics compared with ordinary rivers:

Characteristic-1: Planning discharge of the spillway is not an absolute number because it is aimed at the reduction of flood damage. (It may be possible to revise it later).

Charactaristic-2: No limitation of overflow discharge because of drainage of floods of Laguna de Bay.

Chractaristic-3: Certain amount of discharge is necessary at the control start water level (EL. 12.0 m)

Generally, it is necessary to decide on the overflow dike of a spillway considering the above-mentioned characteristics. However, the Open Intake Channel of Parañaque Spillway would be the same as ordinary rivers so that overflow dike is not necessary. In addition, it is difficult to obtain enough space that will satisfy the discharge of more than 100 m^3/s, because of the limited land acquisition area and the shallow water depth of Laguna de Bay. In conclusion, no overflow dike of Parañaque Spillway is required.

Incidentally, approximately 1 m depth of sand sedimentation pond is installed in the Open Intake Channel to prevent the invasion of bedload in the spillway. Detailed dimensions of the sand sedimentation pond should be decided by considering the results of riverbed material survey and the Operation and Maintenance Rule for Flood Mitigation in future.

In addition, there is no suggestion of inflow gates in the Survey, but the necessity of those of Laguna de Bay may be supposedly reviewed in future. At present, there is no gate like ordinary rivers. However, water quality in the Open Intake Channel in dry season becomes worse and sedimentation soil caused by normal rain may occur. In this point, it is necessary to install inflow gates even if the cost increases. Furthermore, in addition to dredging of the bottom of Laguna de Bay, the possibility of jetty (seawall) for Laguna de Bay may be supposedly reviewed considering maintenance of the channel bed condition. Those matters shall be studied in the future.

Instead of the overflow dike, the hydraulic conditions of Open Intake Channel are shown below.

(i) Case-1: Hydraulic Condition of Open Intake Channel at Control Start Water Level (EL. 12.0 m)

Upstream Water Level	EL. 12.0 m
Design Discharge	Qd = 130 m^3/s
Spillway Length	L = 8,800 m
Head Loss of Open Channel	hd = 600 m / 2000 = 0.300 m
Discharge Capacity	Qc = 138 m^3/s > Qd = 130 m^3/s (OK)

Table 7.2.16 Hydraulic Condition of Open Intake Channel at Control Start Water Level (EL. 12.0 m)

Water Level at Laguna Lake =	12.0	m	
Water Level at Manila Bay =	10.5	m	Red Letter: Input
Spillway Length =	8,800	m	Blue Letter; for Goal Seaking

1) 10% Reduction

Diameter (m)	Area (m2)	Invert (m)	Angle (Degree)	Invert Area (m2)	10% Reduction Area (m2)	Conversion Diameter (m)	Conversion Area (m2)	Roughness Coefficient	Inlet fe	Outlet fo
12.00	113.097	5.00	24.624	1.836	100.135	11.291	100.135	0.015	0.50	1.00

Velocity *1 v (m/s)	Friction Loss hf (m)	Entarance Loss he (m)	Outflow Loss ho (m)	Screen Loss hs (m)	Overflow Dike Loss hd (m)	Open Channel Loss hc	Total Loss ht (m)	Loss Difference dh (m)	Check <0.01	Calculated Discharge (m/s)
1.386	0.953	0.049	0.098	0.100	0.000	0.300	1.500	0.000	OK	138.790

Note *1: Velocity is Calculated by goal seeking between Velocity and Loss Difference under the condition of dh<0.001.

Source: JICA Survey Team

(ii) <u>Case-2: Hydraulic Condition of Open Intake Channel at Design High Water Level (EL. 14.0 m)</u>

Upstream Water Level	EL. 14.0 m
Design Discharge	Qd = 200 m^3/s
Spillway Length	L = 8,800 m
Head Loss of Open Channel	hd = 600 m / 2000 = 0.300 m
Discharge Capacity	Qc = 232 m^3/s > Qd = 200 m^3/s (OK)

Table 7.2.17 Hydraulic Condition of Open Intake Channel at Design High Water Level (EL. 14.0 m)

Water Level at Laguna Lake = 14.0 m
Water Level at Manila Bay = 10.5 m Red Letter: Input
Spillway Length = 8,800 m Blue Letter; for Goal Seaking

1) 10% Reduction

Diameter (m)	Area (m2)	Invert (m)	Angle (Degree)	Invert Area (m2)	10% Reduction Area (m2)	Conversion Diameter (m)	Conversion Area (m2)	Roughness Coefficient	Inlet fe	Outlet fo
12.00	113.097	5.00	24.624	1.836	100.135	11.291	100.135	0.015	0.50	1.00

Velocity *1 v (m/s)	Friction Loss hf (m)	Entarance Loss he (m)	Outflow Loss ho (m)	Screen Loss hs (m)	Overflow Dike Loss hd (m)	Open Channel Loss hc	Total Loss ht (m)	Loss Difference dh (m)	Check <0.01	Calculated Discharge (m/s)
2.326	2.686	0.138	0.276	0.100	0.000	0.300	3.500	0.000	OK	232.953

Note *1: Velocity is Calculated by goal seeking between Velocity and Loss Difference under the condition of dh<0.001.

Source: JICA Survey Team

(d) Sand Sedimentation Pond

It is necessary to evaluate the function of the sand sedimentation pond, taking into account the riverbed fluctuation including diffused soil caused by disturbed flow. However, there are no riverbed material survey data and hydraulic model test results at present. Therefore, the diameter of sunk soil particle is estimated by sedimentation velocity method as the simple way.

The relationship between the diameter of soil particles and sedimentation velocity is defined by Tsurumi's Equation, "Technical Standards for River and Sabo Works, Planning (1997/9)", as follows:

$D > 0.015$ cm $U = 11{,}940\ d^2$

0.015 cm $< d < 0.11$ cm $U = 171.5\ d$

0.11 cm $< d < 0.58$ cm $U = 81.5\ d^{0.667}$

0.58cm $< d$ $U = 73.2$

Where, U: Sedimentation Velocity (m/s)

D: Diameter (cm)

With regard to the Open Intake Channel Method, the soil sedimentation pond is not planned separately and sediment deposition is expected in the open channel because of the limited land use condition. Therefore, 100 m out of the total 600 m of Open Intake Channel is excavated as the soil sedimentation pond, as follows:

(i) Case-1: Sedimentation Condition of Open Intake Channel at Control Start Water Level (EL. 12.0 m)

Design Discharge $Qd = 130\ m^3/s$

Design Water Depth h = 1.5 m

Invert Level EL. 10.5 m

Average Velocity v = 1.884 m/s from Table 7.2.11

Channel Passing Time t = 100 m / 1.884 m/s = 53.1 s

Diameter d = 0.02 cm U = 171.5 x 0.02 cm = 3.43 cm/s

Sedimentation Time tr = 150 / 3.43 = 43.7 s < t = 53.1 s (OK)

Friction Velocity $u_{*c}^2 = \mathrm{Rig} = 1.421 \times 0.0005 \times 9.81$

$= 0.00697\mathrm{m}^2/\mathrm{s}^2 = 69.7\mathrm{cm}^2/\mathrm{s}^2$

Critical Traction Diameter $d = {u_{*c}^2}/{80.9} = {69.7}/{80.9} = 0.86cm = 8.6mm$

(ii) Case-2: Sedimentation Condition of Open Intake Channel at Control Start Water Level (EL. 12.0 m)

Design Discharge Qd = 200 m^3/s

Design Water Depth h = 1.93 m

Invert Level EL. 10.5 m

Average Velocity v = 2.207 m/s from Table 7.2.13

Channel Passing Time t = 100 m / 2.207 m/s = 45.3 s

Diameter d = 0.03 cm U = 171.5 x 0.03 cm = 5.15 cm/s

Sedimentation Time tr = 193 / 5.15 = 37.4 s < t = 45.3 s (OK)

Friction Velocity $u_{*c}^2 = \mathrm{Rig} = 1.801 \times 0.0005 \times 9.81$

$= 0.00883\mathrm{m}^2/\mathrm{s}^2 = 88.3\mathrm{cm}^2/\mathrm{s}^2$

Critical Traction Diameter $d = {u_{*c}^2}/{80.9} = {88.3}/{80.9} = 1.09\mathrm{cm} = 10.9\mathrm{mm}$

Theoretically, soil particles of less than 1 mm precipitate in the soil sedimentation pond of 100m. Therefore, it is necessary to install a soil sedimentation pond with enough volume for bedload, because soil particles of less than 10 mm are transported through the spillway.

(e) Connection Channel

The Connection Channel is to be installed between the Intake Open Channel and the Inlet Vertical Shaft to connect them together to stabilize the water inflow around the vertical shaft. In general, it is designed to take into account the required hydraulic condition and the allowable land use area. This should be verified by hydraulic model test.

Two possible cases of width of connection channel are as follows, and the comparison of connection channel for the Open Intake Channel Method is as shown in Table 7.2.18.

Case-A: Width of 35 m almost same as Inlet Vertical Shaft

Case-B: Width of 46 m as same as Intake Open Channel

Table 7.2.18 Comparison of Connection Channels of Open Intake Channel Method

Case Name	Case-A: Width = 35 m	Case-B: Width = 46 m	Note
Summary	Transition Channel between the Open Intake Channel and the Connection Channel, Width 46 m = > 35 m, Invert Level EL. 10.2 m => EL. 9.5 m, is necessary.	To minimize the water head loss, the same Width of 46 m and the same Invert Level of EL. 10.2 m are designed.	
Dimension	Width W = 35 m Height H = 5.7 m Required Water Depth h1 = 1.564 m < 2.20 m OK	Width W = 35 m Height H = 5.0 m Required Water Depth h1 = 1.346 m < 1.50 m OK	

Case Name	Case-A: Width = 35 m	Case-B: Width = 46 m	Note
	h2 = 1.982 m < 2.63 m OK	h2 = 1.698 m < 1.93 m OK	
Hydraulic Condition	Average velocities are 2.17 m/s at 200 m^3/s and 1.69 m/s at 130 m^3/s. Invert elevation of open channel is EL. 10.2 m and connection channel is EL. 9.5 m. Therefore, height adjustment is necessary.	Average velocities are 2.25 m/s at 200 m^3/s and 1.88 m/s at 130 m^3/s. There is almost critical water flow around the vertical shaft because of head loss of columns, screens and gates.	It is necessary to consider the slope of connection channel.
Construction	No problem O	No problem O	
Operation & Maintenance	Cleaning area is smaller than Case-B. O	Cleaning area is larger than Case-A by 31.4%. Δ	
Social Environment	Required facility area, approximately 1,000m^2, is smaller than Case-B. O	Required facility area, approximately 1,200m^2, is a little wider than Case-A. Δ	
Natural Environment	No problem O	No problem O	
Others	It is necessary to install transition channel between Open Intake Channel and Connection Channel. Δ	Inflow guide wall may be necessary around Vertical Shaft to stabilize inflow water. Δ	
Cost	Construction and compensation costs are cheaper than Case-B, because the width of the connection channel is reduced by 11m. O	Relatively more expensive cost, because the width of the connection channel is increased by 11m. Δ	
Evaluation	Both required land area and project cost are smaller. In addition, hydraulic condition of connection channel is also better than Case-B. O: Adopted	It is possible to be adopted but wider land area and lower hydraulic condition of connection channel are required. Δ: Possible	

Legend: ◎ Excellent、○ Good、△ Not Good/Some Problem、× Difficult/ Impossible

Source: JICA Survey Team

Results of non-uniform calculation of 35m (Case-A) and 46m (Case-B) width of Connection Channel at Discharge of 130 m^3/s and 200 m^3/s are shown in Table 7.2.19 to Table 7.2.22. The calculations assume that water flow around the Vertical Shaft is the critical water depth.

Table 7.2.19 Non-uniform Calculation of 35m Width of Connection Channel at Discharge of 130m^3/s

Box Dimension	W.L. Height		Width	Freeboard	Inner Height
	2.20	×	35.00	3.50	5.70

＜Upstream＞				＜Downstream＞			
Discharge Q	＝	130.0	(m^3/s)	Discharge Q	＝	130.0	(m^3/s)
Assumed Water Depth H1	＝	1.564	(m)仮定	Calculated Water Depth H2	＝	1.121	(m)･･･Critical Water Depth
Water Area A1	＝	54.727	(m^3)	Water Area A2	＝	39.226	(m^3)
Wetted Perimeter P1	＝	38.127	(m)	Wetted Perimeter P2	＝	37.241	(m)
Hydraulic Radius R1	＝	1.435	(m)	Hydraulic Radius R2	＝	1.053	(m)
Roughness Coefficient n1	＝	0.0150		Roughness Coefficient n2	＝	0.0150	
Velocity V1	＝	2.375	(m/s)	流速 V2	＝	3.314	(m/s)
Length L	＝	110.000	(m)				

$$hf = \frac{130.0^{2} \cdot 110.000}{2} \cdot \left(\frac{0.015^{2}}{1.435^{4/3} \cdot 54.727^{2}} + \frac{0.015^{2}}{1.053^{4/3} \cdot 39^{2}} \right)$$

$$= 929500 \cdot (5E-08 + 1.4E-07) = 0.17$$

Upstream

$$1.563628 + \frac{1.0 \cdot 130.000^{2}}{2g \cdot 54.727^{2}} + 0.000 = 1.852$$

Downstream

$$1.120749 + \frac{1.0 \cdot 130.000^{2}}{2g \cdot 39.226^{2}} + 0.000 + 0.17 = 1.851$$

Energy Error 0.000 (m)

Therefore, the Upstream Water Depth becomes **1.564** (m).

Source: JICA Survey Team

Table 7.2.20 Non-uniform Calculation of 35m Width of Connection Channel at Discharge of 200m^3/s

Box Dimension	W.L. Height		Width	Freeboard	Inner Height
	2.63	×	35.00	3.07	5.70

＜Upstream＞				＜Downstream＞			
Discharge Q	＝	200.0	(m^3/s)	Discharge Q	＝	200.0	(m^3/s)
Assumed Water Depth H1	＝	1.982	(m)仮定	Calculated Water Depth H2	＝	1.494	(m)･･･Critical Water Depth
Water Area A1	＝	69.369	(m^3)	Water Area A2	＝	52.276	(m^3)
Wetted Perimeter P1	＝	38.964	(m)	Wetted Perimeter P2	＝	37.987	(m)
Hydraulic Radius R1	＝	1.780	(m)	Hydraulic Radius R2	＝	1.376	(m)
Roughness Coefficient n1	＝	0.0150		Roughness Coefficient n2	＝	0.0150	
Velocity V1	＝	2.883	(m/s)	流速 V2	＝	3.826	(m/s)
Length L	＝	110.000	(m)				

$$hf = \frac{200.0^{2} \cdot 110.000}{2} \cdot \left(\frac{0.015^{2}}{1.780^{4/3} \cdot 69.369^{2}} + \frac{0.015^{2}}{1.376^{4/3} \cdot 52^{2}} \right)$$

$$= 2200000 \cdot (2E-08 + 5.4E-08) = 0.166$$

Upstream

$$1.981957 + \frac{1.0 \cdot 200.000^{2}}{2g \cdot 69.369^{2}} + 0.000 = 2.406$$

Downstream

$$1.493594 + \frac{1.0 \cdot 200.000^{2}}{2g \cdot 52.276^{2}} + 0.000 + 0.166 = 2.406$$

Energy Error 0.000 (m)

Therefore, the Upstream Water Depth becomes **1.982** (m).

Source: JICA Survey Team

Table 7.2.21 Non-uniform Calculation of 46m Width of Connection Channel at Discharge of 130m³/s

Box Dimension	W.L. Height		Width	Freeboard	Inner Height
	1.50	×	46.00	3.50	5.00

<Upstream>				<Downstream>			
Discharge Q	=	130.0	(m^3/s)	Discharge Q	=	130.0	(m^3/s)
Assumed Water Depth H1	=	1.346	(m)仮定	Calculated Water Depth H2	=	0.934	(m)･･･Critical Water Depth
Water Area A1	=	61.937	(m^3)	Water Area A2	=	42.967	(m^3)
Wetted Perimeter P1	=	48.693	(m)	Wetted Perimeter P2	=	47.868	(m)
Hydraulic Radius R1	=	1.272	(m)	Hydraulic Radius R2	=	0.898	(m)
Roughness Coefficient n1	=	0.0150		Roughness Coefficient n2	=	0.0150	
Velocity V1	=	2.099	(m/s)	流速 V2	=	3.026	(m/s)
Length L	=	110.000	(m)				

$$hf = \frac{130.0^{2} \cdot 110.000}{2} \cdot \left(\frac{0.015^{2}}{1.272^{4/3} \cdot 61.937^{2}} + \frac{0.015^{2}}{0.898^{4/3} \cdot 43^{2}} \right)$$

$$= 929500 \cdot (4E{-}08 + 1.4E{-}07) = 0.17$$

Upstream
$$1.346467 + \frac{1.0 \cdot 130.000^{2}}{2g \cdot 61.937^{2}} + 0.000 = 1.571$$

Downstream
$$0.934075 + \frac{1.0 \cdot 130.000^{2}}{2g \cdot 42.967^{2}} + 0.000 + 0.17 = 1.571$$

Energy Error 0.000 (m)
Therefore, the Upstream Water Depth becomes **1.346** (m).

Source: JICA Survey Team

Table 7.2.22 Non-uniform Calculation of 46m Width of Connection Channel at Discharge of 200m³/s

Box Dimension	W.L. Height		Width	Freeboard	Inner Height
	1.93	×	46.00	3.07	5.00

<Upstream>				<Downstream>			
Discharge Q	=	200.0	(m^3/s)	Discharge Q	=	200.0	(m^3/s)
Assumed Water Depth H1	=	1.698	(m)仮定	Calculated Water Depth H2	=	1.245	(m)･･･Critical Water Depth
Water Area A1	=	78.120	(m^3)	Water Area A2	=	57.262	(m^3)
Wetted Perimeter P1	=	49.397	(m)	Wetted Perimeter P2	=	48.490	(m)
Hydraulic Radius R1	=	1.581	(m)	Hydraulic Radius R2	=	1.181	(m)
Roughness Coefficient n1	=	0.0150		Roughness Coefficient n2	=	0.0150	
Velocity V1	=	2.560	(m/s)	流速 V2	=	3.493	(m/s)
Length L	=	110.000	(m)				

$$hf = \frac{200.0^{2} \cdot 110.000}{2} \cdot \left(\frac{0.015^{2}}{1.581^{4/3} \cdot 78.120^{2}} + \frac{0.015^{2}}{1.181^{4/3} \cdot 57^{2}} \right)$$

$$= 2200000 \cdot (2E{-}08 + 5.5E{-}08) = 0.165$$

Upstream
$$1.69827 + \frac{1.0 \cdot 200.000^{2}}{2g \cdot 78.12^{2}} + 0.000 = 2.033$$

Downstream
$$1.244819 + \frac{1.0 \cdot 200.000^{2}}{2g \cdot 57.262^{2}} + 0.000 + 0.165 = 2.032$$

Energy Error 0.000 (m)
Therefore, the Upstream Water Depth becomes **1.698** (m).

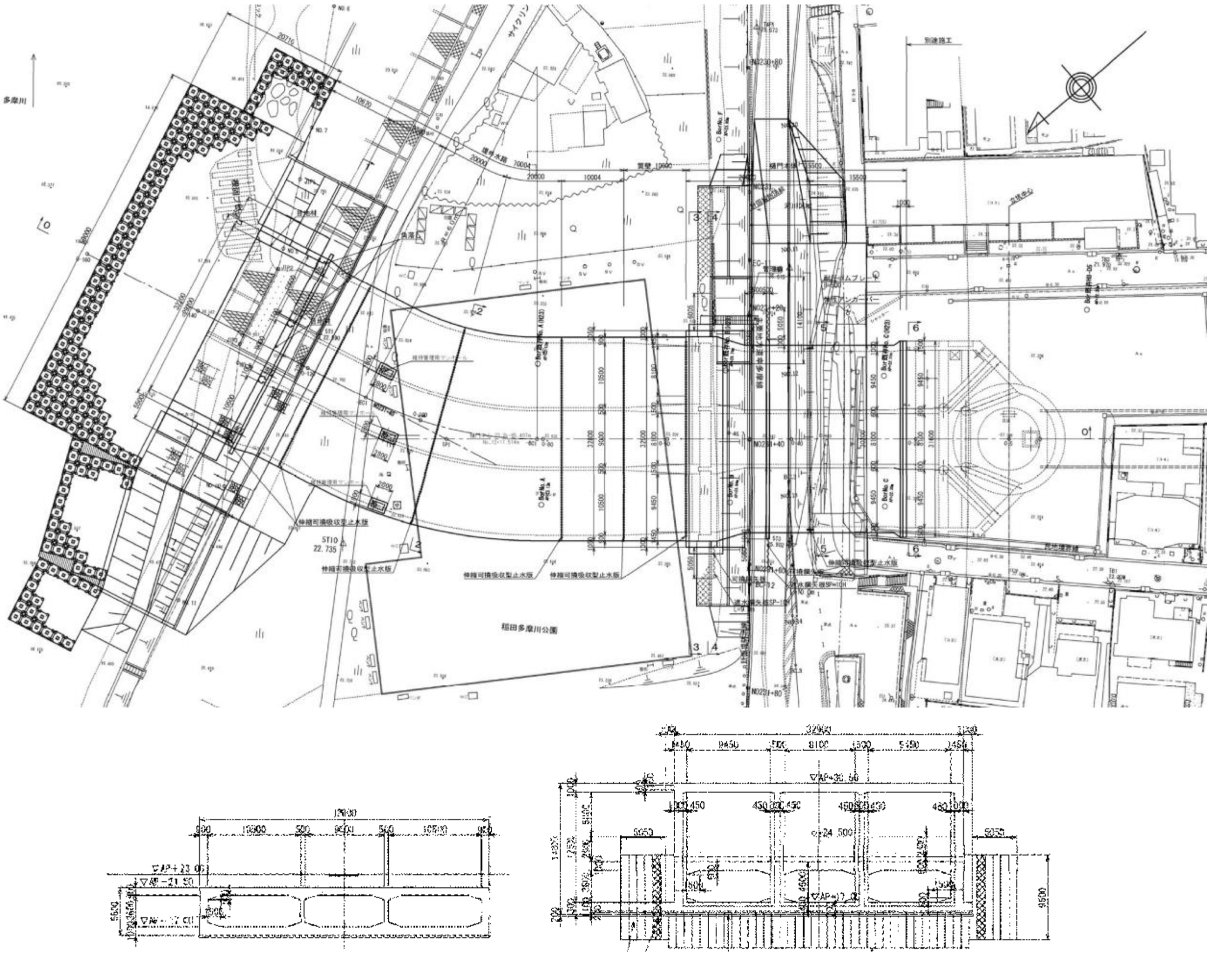
Source: JICA Survey Team

According to Table 7.2.18, "Case-A: Width = 35 m" is adopted in the Survey because of the hydraulic surplus around the Inlet Vertical Shaft and the small required facility area. However, the elevation and dimension of the Connection Channel should be reviewed by certain final conditions, such as available land acquisition area, energy dissipater of Transition Channel, dimension of Vortex Drop Shaft and hydraulic model test result around the Inlet Vertical Shaft.

(3) Drainage Facility

The basic policy on the Drainage Facility (Drainage Sluice) is the same as in Section 4.3, "Study on Parañaque Spillway", and as follows:

- According to Figure 4.3.6 obtained from DPWH, the design water depth is 4m from the normal water level. Therefore, a shallow design water depth is more practical even if dredging is planned additionally. Therefore, the design water depth of Drainage Facility is 4m, more or less.
- As the reference, the outline drawing of drainage facility of Gotanda Discharge Channel in Japan is shown in Figure 7.2.16.



Source: "Final Report for Detailed Design of Drainage Facility of Gotanda Discharge Channel" (2016)

Figure 7.2.16 Outline Drawing of Drainage Facility of Gotanda Discharge Channel in Japan

- According to Figure 4.3.7, Hydraulic Model Test Results of Outlet Drainage of Gotanda Discharge Channel in Japan, the "Wide Figure 3 Boxes (9.45 m + 8.1 m + 9.45 m" for Connection Channel and 10.5 m + 9.0m + 10.5 m for Riverside Channel) are hydraulically better than the 5 Boxes (5 m x 3.6 m x 5 Boxes). Therefore, the fewer box type has a hydraulic advantage. In conclusion, the "Wide Figure 4 Boxes (7.6 m x 3.6 m)" is adopted, with the 5.4 m x 4.2 m x 6 Boxes of the Metropolitan Area Outer Underground Discharge Channel and the 10.0 m x 3.6 m x 3 Boxes of the Gotanda Discharge Channel as reference.

- It is structurally possible to adopt the "Wide Figure 3 Boxes (10.0 m x 3.6 m)" in accordance with the Gotanda Discharge Channel, but the box culvert with 10m width is not economical because the dimensions become large. In addition, according to "Guideline of Earth Works- Culvert Guidance" (2010/3, Japan Road Association), the ordinary standard size of box culvert is approximately 6.5 m x 5.0 m. It is necessary that "adequate and comprehensive design research, which shall take into account all possible force phenomena, should be required if box culvert size, (partially omitted), is larger than the ordinary standard size".
- In addition, the more than 10m width box culvert of drainage facility for Gotanda Discharge Channel is located in a major bed and only sidewalk live load is considered for the design. However, with regard to Parañaque Spillway, the Drainage Facility has high possibility to become the drainage sluice and the design live load is bigger than Gotanda's because operation and maintenance vehicles pass through the river dike.

Therefore, the "Wide figure 4 Boxes (7.6 m x 3.6 m)" size of drainage facility is structurally adopted for Parañaque Spillway . In addition, the tidal level conditions for the hydraulic design are as follows:

(Tidal Level Condition of Manila Bay)

Mean Sea Level (MSL)	EL. 10.50 m
Mean High Water (MHW)	EL. 11.00 m
Mean Low Water (MLW)	EL. 10.00 m

The invert elevation of the drainage facility is set based on enough water depth which is deeper than the critical water depth and more than the Mean Low Water. In addition, the crown elevation of the drainage facility is higher enough than the Mean High Water and more than the freeboard of sluice of 60 cm.

Critical water depth at 7.6 m width and discharge of 200 / 4 = 50 m^3/s is "hc = 1.641 m". Therefore, the invert elevation of the drainage facility is based on the Mean Low Water of EL. 10.0 m, as follows:

Invert Elevation of Drainage Facility, EL. 10.00 m - 1.641 m = 8.359 m, say EL. 8.3 m.

In addition, the Minimum Crown Elevation of the drainage facility is calculated from the Invert Elevation of drainage facility and upstream water depth of h = 2.740 m at Mean High Water (EL. 11.00 m), as follows:

Minimum Crown Elevation of the drainage facility:

EL. 8.3 m + 2.740 m + 0.6 m (freeboard) = EL. 11.64 m.

However, according to Figure 4.3.7, Hydraulic Model Test Results of Outlet Drainage of Gotanda Discharge Channel, the water flow level from the Outlet Vertical Shaft to the Drainage Facility has risen to approximately 20 cm and it is necessary that the phenomenon is confirmed by future hydraulic model test. This height is considered as the margin of 20 cm in the Pre-Feasibility Survey for the crown elevation of the drainage facility as follows. (This should be reviewed according to the result of hydraulic model test in the future.)

Crown Elevation of Drainage Facility:

EL. 11.640 m + 0.20 m (margin) = 11.840 m, say EL. 11.9 m

Results of non-uniform calculation of the drainage facility at Mean Low Water (EL. 10.00 m), Mean Sea Level (EL. 10.50 m) and Mean High Water (EL. 11.00 m) are shown in Table 7.2.23, Table 7.2.24 and Table 7.2.25.

Table 7.2.23 Non-uniform Calculation of Drainage Facility at Mean Low Water (EL. 10.0 m)

Box Dimension	W.L Height		Width	Freeboard	Inner Height
	3.00	×	7.60	0.60	3.60

＜Upstream＞			＜Downstream＞		
Discharge Q	＝	50.0 (m^3/s)	Discharge Q	＝	50.0 (m^3/s)
Assumed Water Depth H1	＝	2.014 (m)仮定	Calculated Water Depth H2	＝	1.700 (m)･･･At Tide Level of EL+10.0m
Water Area A1	＝	15.304 (m^3)	Water Area A2	＝	12.920 (m^3)
Wetted Perimeter P1	＝	11.627 (m)	Wetted Perimeter P2	＝	11.000 (m)
Hydraulic Radius R1	＝	1.316 (m)	Hydraulic Radius R2	＝	1.175 (m)
Roughness Coefficient n1	＝	0.0150	Roughness Coefficient n2	＝	0.0150
Velocity V1	＝	3.267 (m/s)	Velocity V2	＝	3.870 (m/s)
Length L	＝	43.000 (m)			

$$hf = \frac{50.0^{2} \cdot 43.000}{2} \cdot \left(\frac{0.015^{2}}{1.316^{4/3} \cdot 15.304^{2}} + \frac{0.015^{2}}{1.175^{4/3} \cdot 13^{2}} \right)$$

$$= 53750 \cdot (7E{-}07 + 1.1E{-}06) = 0.094$$

Upstream $$2.013683 + \frac{1.0}{2g} \cdot \frac{50.000^{2}}{15.304^{2}} + 0.000 = 2.558$$

Downstream $$1.7 + \frac{1.0}{2g} \cdot \frac{50.000^{2}}{12.92^{2}} + 0.000 + 0.094 = 2.558$$

Energy Error 0.000 (m)

Therefore, the Upstream Water Depth becomes **2.014** (m).

Source: JICA Survey Team

Table 7.2.24 Non-uniform Calculation of Drainage Facility at Mean Sea Level (EL. 10.5 m)

Box Dimension	W.L Height		Width	Freeboard	Inner Height
	3.00	×	7.60	0.60	3.60

＜Upstream＞			＜Downstream＞		
Discharge Q	＝	50.0 (m^3/s)	Discharge Q	＝	50.0 (m^3/s)
Assumed Water Depth H1	＝	2.287 (m)仮定	Calculated Water Depth H2	＝	2.200 (m)･･･At Tide Level of EL+10.5m
Water Area A1	＝	17.379 (m^3)	Water Area A2	＝	16.720 (m^3)
Wetted Perimeter P1	＝	12.173 (m)	Wetted Perimeter P2	＝	12.000 (m)
Hydraulic Radius R1	＝	1.428 (m)	Hydraulic Radius R2	＝	1.393 (m)
Roughness Coefficient n1	＝	0.0150	Roughness Coefficient n2	＝	0.0150
Velocity V1	＝	2.877 (m/s)	Velocity V2	＝	2.990 (m/s)
Length L	＝	43.000 (m)			

$$hf = \frac{50.0^{2} \cdot 43.000}{2} \cdot \left(\frac{0.015^{2}}{1.428^{4/3} \cdot 17.379^{2}} + \frac{0.015^{2}}{1.393^{4/3} \cdot 17^{2}} \right)$$

$$= 53750 \cdot (5E{-}07 + 5.2E{-}07) = 0.053$$

Upstream $$2.286672 + \frac{1.0}{2g} \cdot \frac{50.000^{2}}{17.379^{2}} + 0.000 = 2.709$$

Downstream $$2.2 + \frac{1.0}{2g} \cdot \frac{50.000^{2}}{16.72^{2}} + 0.000 + 0.053 = 2.709$$

Energy Error 0.000 (m)

Therefore, the Upstream Water Depth becomes **2.287** (m).

Source: JICA Survey Team

Table 7.2.25 Non-uniform Calculation of Drainage Facility at Mean High Water (EL. 11.5 m)

Box Dimension	W.L. Height		Width	Freeboard	Inner Height
	3.00	×	7.60	0.60	3.60

＜Upstream＞				＜Downstream＞			
Discharge Q	＝	50.0	(m^3/s)	Discharge Q	＝	50.0	(m^3/s)
Assumed Water Depth H1	＝	2.740	(m)仮定	Calculated Water Depth H2	＝	2.700	(m)･･･At Tide Level of EL+11.0m
Water Area A1	＝	20.821	(m^3)	Water Area A2	＝	20.520	(m^3)
Wetted Perimeter P1	＝	13.079	(m)	Wetted Perimeter P2	＝	13.000	(m)
Hydraulic Radius R1	＝	1.592	(m)	Hydraulic Radius R2	＝	1.578	(m)
Roughness Coefficient n1	＝	0.0150		Roughness Coefficient n2	＝	0.0150	
Velocity V1	＝	2.401	(m/s)	流速 V2	＝	2.437	(m/s)
Length L	＝	43.000	(m)				

$$hf = \frac{50.0^2 \cdot 43.000}{2} \cdot \left(\frac{0.015^2}{1.592^{4/3} \cdot 20.821^2} + \frac{0.015^2}{1.578^{4/3} \cdot 21^2} \right)$$

$$= 53750 \cdot (3E-07 + 2.9E-07) = 0.031$$

Upstream

$$2.739574 + \frac{1.0 \cdot 50.000^2}{2g \cdot 20.821^2} + 0.000 = 3.034$$

Downstream

$$2.7 + \frac{1.0 \cdot 50.000^2}{2g \cdot 20.52^2} + 0.000 + 0.031 = 3.034$$

Energy Error 0.000 (m)

Therefore, the Upstream Water Depth becomes 2.740 (m).

Source: JICA Survey Team

According to Table 7.2.23, the velocity at Mean Low Water (EL. 10.0 m) is “V = 3.870 m/s” which exceeds the ordinary allowable maximum velocity, Vmax=3m/s, in accordance with “Planning and Design Guideline for Sewage System" (2009/9, Japan Sewage Works Association). However, there is practically no issue about this matter for the following reasons:

- Regulation of the allowable maximum velocity is taken into account together with the wear prevention caused by traction materials such as soil inside drainage box and increase of downstream outflow due to shorter flow time. However, about Parañaque Spillway, traction materials and outflow increase are not expected.

- According to the “Technical Standards for River and Sabo Works” (Ministry of Land, Infrastructure, Transport and Tourism, Japan), the “Velocity of river tunnel is generally less than 7 m/s” and “Regular flow velocity of river tunnel ranges from 2 m/s to 5m/s”. Therefore, the velocity of 3.870 m/s is within these velocities.

- It is possible to conduct countermeasures, such as spray coating for wear prevention at the detailed design stage, if required, because the facility is not huge.

- Velocities at Mean Low Water (EL. 10.0 m) of 3.267 m/s to 3.870 m/s are more than the allowable maximum velocity, Vmax = 3 m/s. However, both velocities at Mean Sea Level (EL. 10.5 m) of 2.877 m/s to 2.990 m/s and velocities at Mean High Water (EL. 11.0 m) of 2.401 m/s to 2.437 m/s are below 3 m/s.

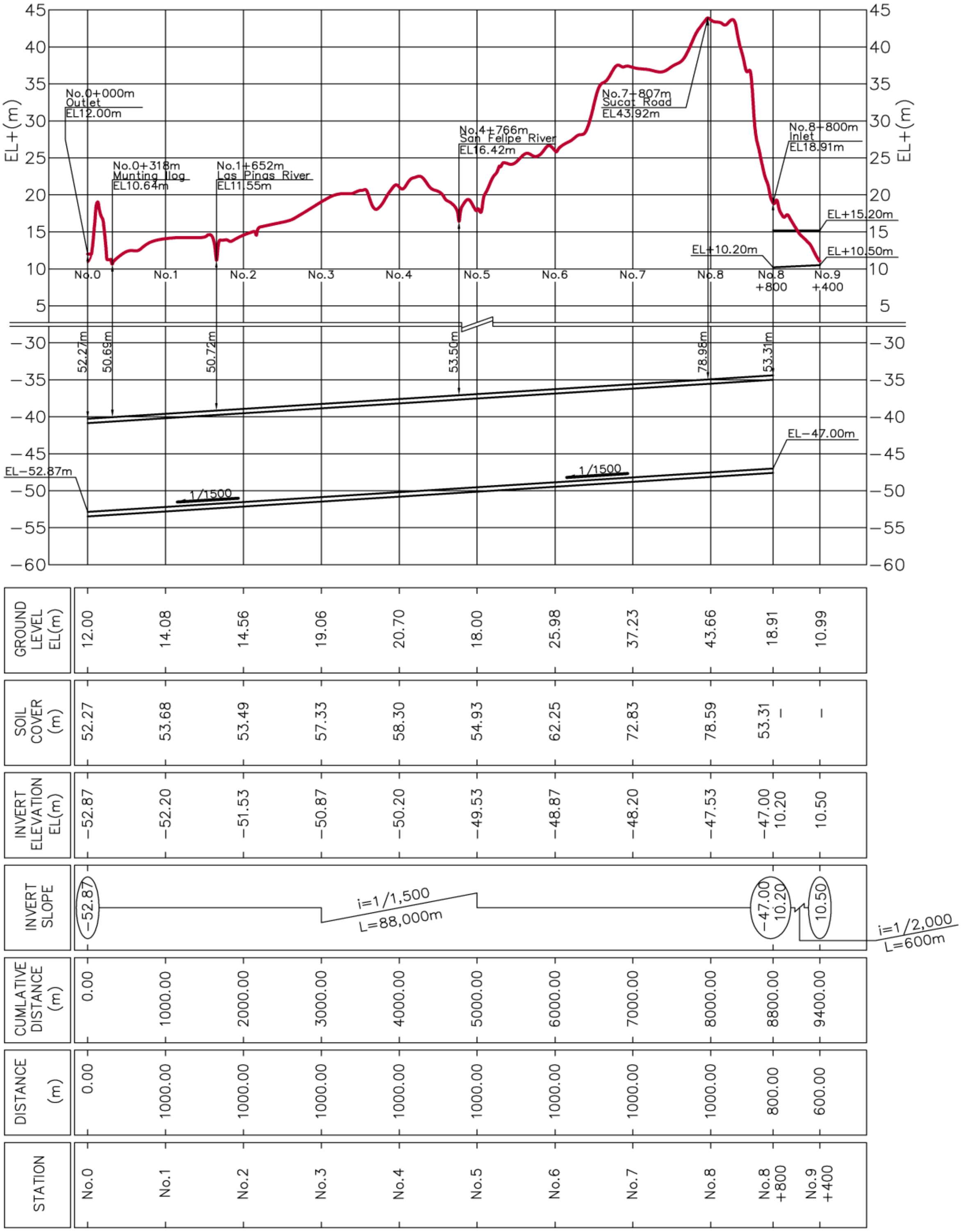
In addition, it is necessary to conduct a design review in the future because of the river plan conditions of river drainage, such as design High Water Level and Riverbed Elevation. At present, four box culverts on the drainage facility is schemed, but other plans, such as 3 box culverts and open channel type, should be also prospected. In respect of the advantage of the hydraulic condition, smaller water head loss, 3 box culvert type is desirable. On the other hand, 4 box culvert type has the advantage of appropriate structural dimension as the drainage facility. This matter shall become the future study theme because more detailed design conditions, such as the drainage facility location, structural type and design conditions, shall be necessary to decide the box culvert type.

In addition, there is no bottom haunch of box culvert in accordance with the Japanese Standard because of difficulty in construction. However, it is necessary to study the necessity of bottom haunches because of structural advantage and workability in Philippines even if the difficulty.

Incidentally, there is no pile foundation because of the sluice structure which is through river dike. However, the structure makes a certain settlement difference at the joint between the drainage facility and the Outlet Vertical Shaft. A flexible joint will be able to handle it, but maintenance is required regularly. In conclusion, to deal with the settlement difference, the possibility of pile foundation should be considered later.

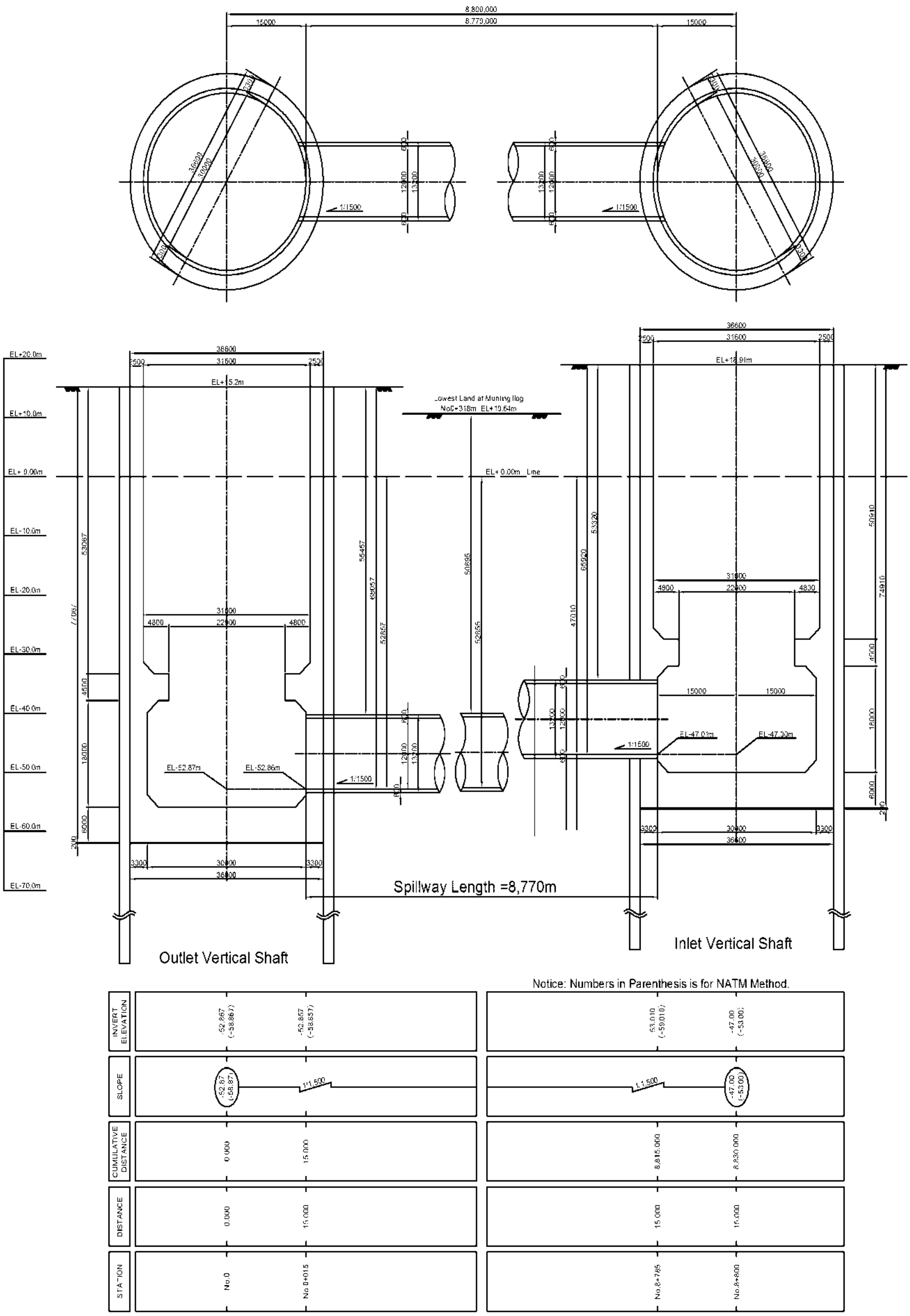
(4) Outline Drawing

Outline drawings such as Longitudinal Profile of Parañaque Spillway, Plan Drawing of Vertical Shaft of Inlet and Outlet, Plan Drawing of the Intake Facility (Inlet), Cross Section Drawing of the Intake Facility, Plan Drawing of Drainage Facility and Cross Section Drawing of Drainage Facility (Outlet) are shown in Figure 7.2.17 to Figure 7.2.22, respectively.



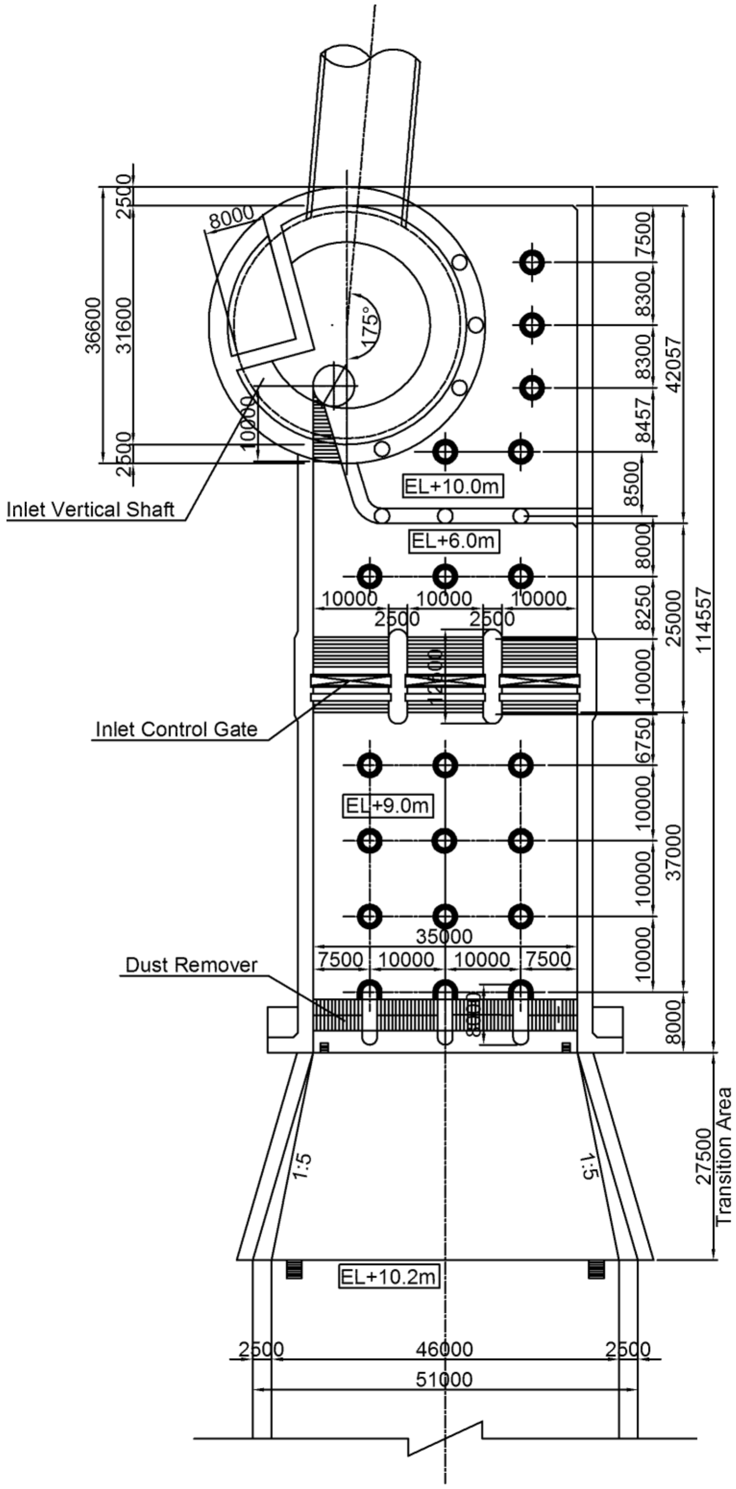
Source: JICA Survey Team

Figure 7.2.17　Longitudinal Profile of Parañaque Spillway



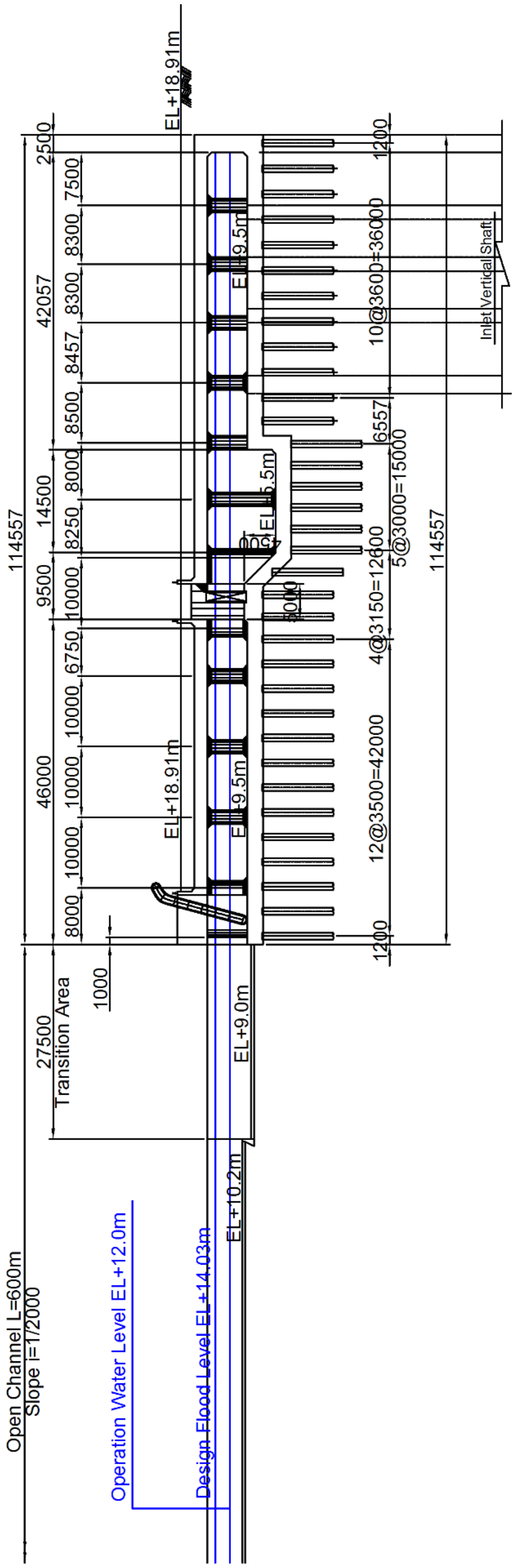
Source: JICA Survey Team

Figure 7.2.18 Plan Drawing of Vertical Shaft of Inlet and Outlet



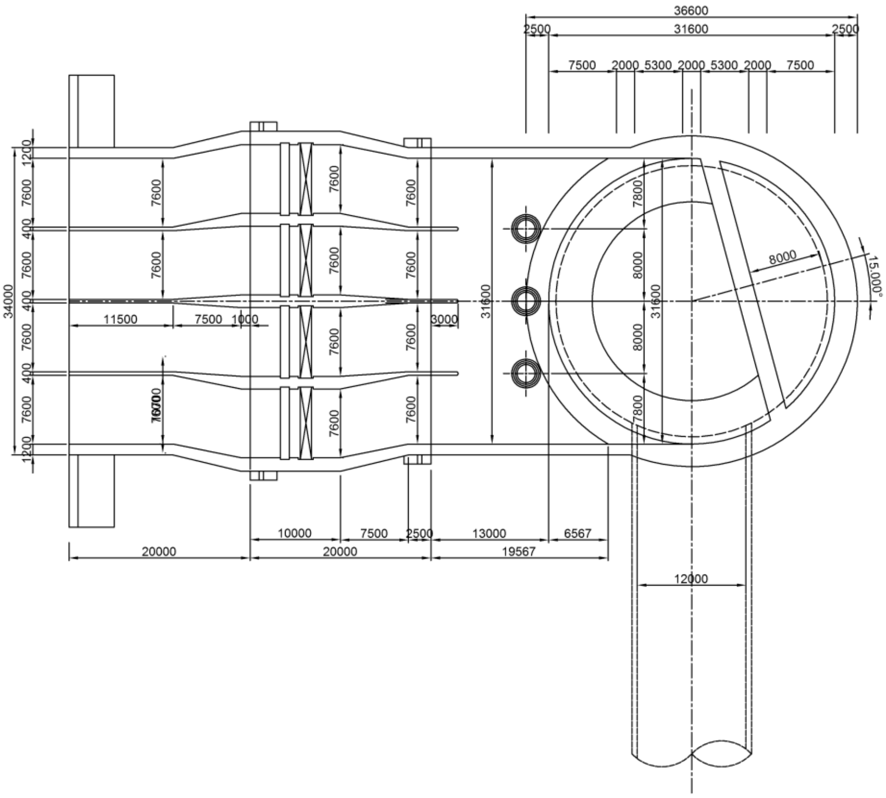
Source: JICA Survey Team

Figure 7.2.19 Plan Drawing of Intake Facility



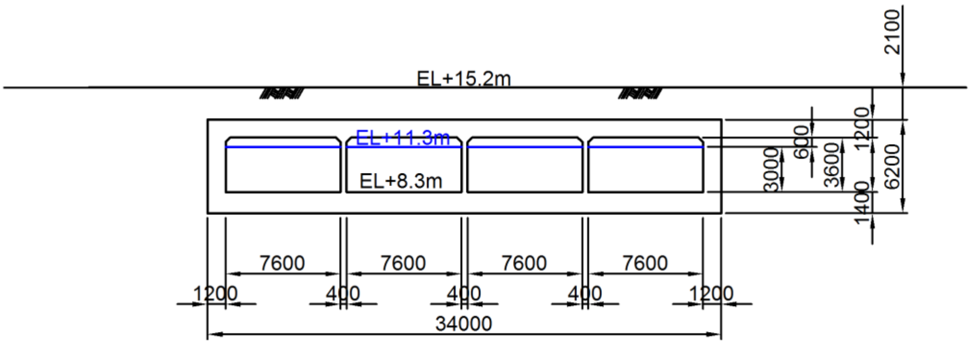
Source: JICA Survey Team

Figure 7.2.20 Cross Section Drawing of Intake Facility



Source: JICA Survey Team

Figure 7.2.21 Plan Drawing of Drainage Facility



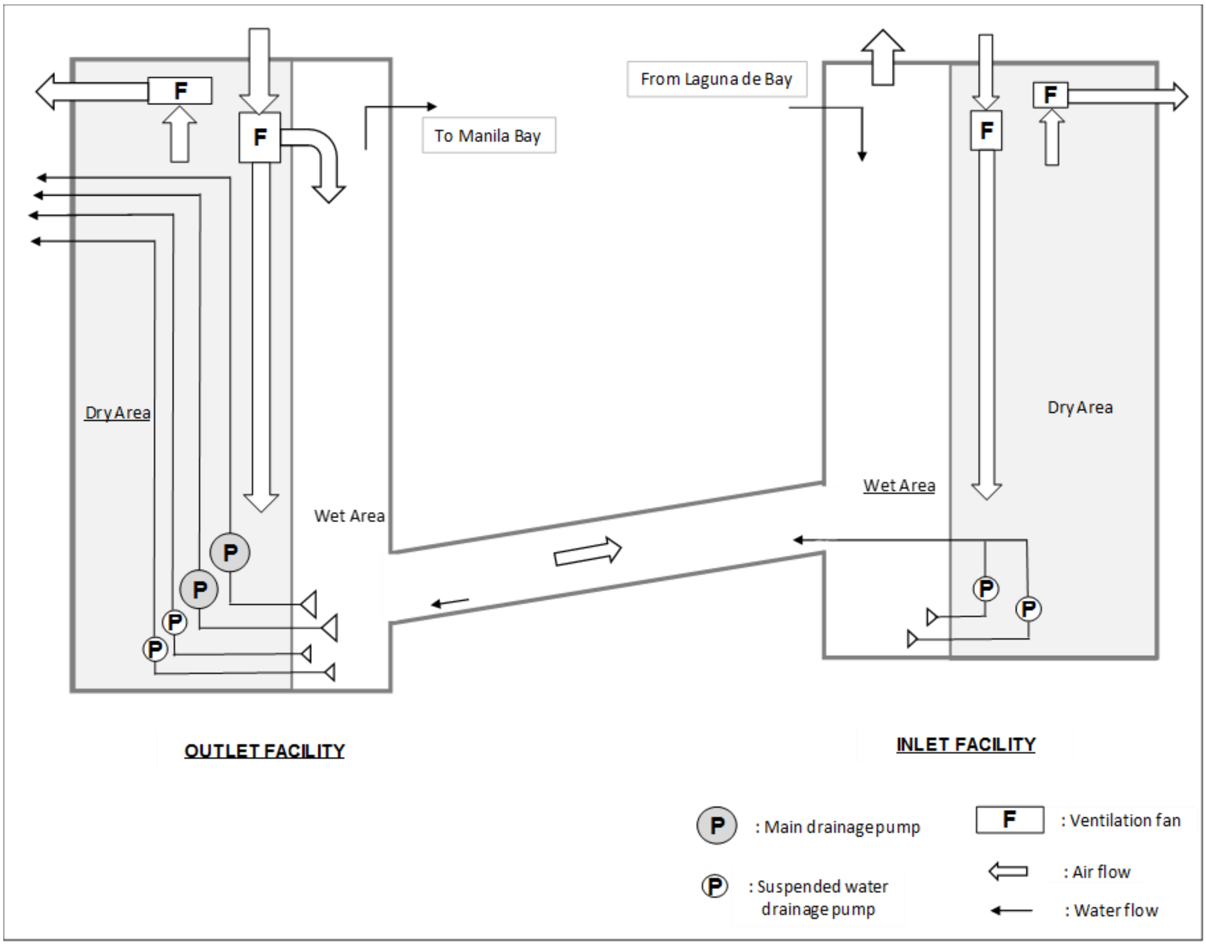
Source: JICA Survey Team

Figure 7.2.22 Cross Section Drawing of Drainage Facility

7.2.5 Design of Pump and Gate

As for the equipment of the Project, trash-rack rakes are employed to remove floating materials such as water hyacinth at the water intake, and drainage pumps are designed to discharge water remaining in the spillway tunnel within five days after stoppage of inflow. Prior to the pumping operation, inlet and outlet gates are designed to block influent from the Laguna de Bay and reverse flow from the connection river to the Manila Bay. For maintenance of the spillway tunnel, it is planned to remove and carry out sediment by using heavy vehicles. Ventilation equipment are designed to maintain a safe working environment in the spillway tunnel and to exhaust heat generated by electric motor of drainage pump in dry area of vertical shaft.

The drainage and ventilation systems are shown in Figure 7.2.23.



Source: JICA Survey Team

Figure 7.2.23 Drainage and Ventilation Systems

(1) Inlet Facility

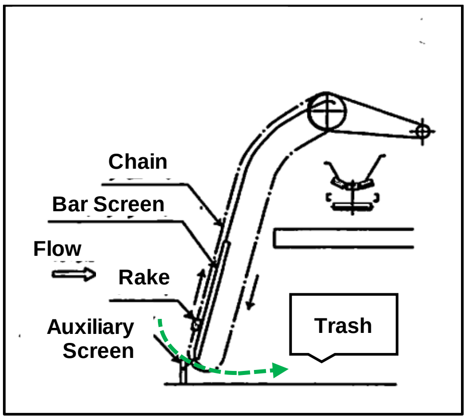
(a) Trash-rack Rake

At the intake facility, a boom is set up to prevent floating materials from entering the spillway tunnel at the intake of Laguna de Bay, and trash-rack rakes are designed at each channel divided into four lanes of canal of 35m in width. A trash-rack rake is equipped with a belt-conveyor and a hopper to load garbage on a dump truck.

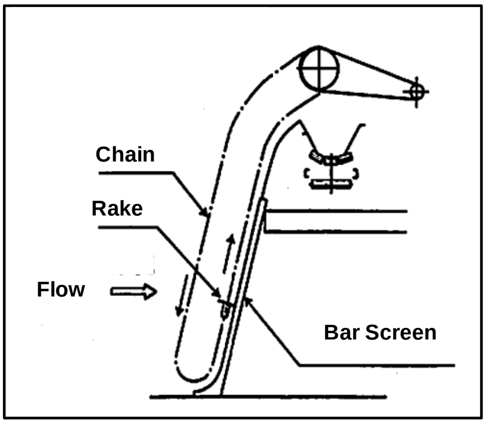
Source: JICA Survey Team

Figure 7.2.24 Water Hyacinth at Suction Side of Pumping Station

Water Hyacinth growing abundantly along the lakeshore of Laguna de Bay and rivers may obstruct the inflow at the intake. Based on the field investigation on drainage pump stations in Metro Manila, a part of floating debris could pass through the trash-rack rake and caught in the bar screen at pump suction side since it is a rear-lowering and front-raising type. In consideration of the present problems, it is recommendable that the trash-rack rake is of the front-lowering and rear-raising type installed in-front of the bar screen as shown in Figure 7.2.25.



FRONT-RISING AND REAR-LOWERING TYPE



FRONT-LOWERING AND REAR-RISING TYPE
(Recommendable)

Source: Technical Guideline of Trash-rack Rake

Figure 7.2.25 Types of Trash-rack Rake

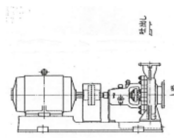
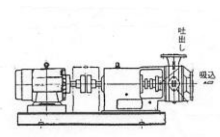
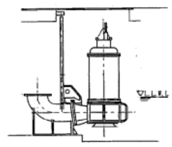
(b) Inlet Control Gate

The inlet control gate is vulnerable to water level fluctuation in Laguna de Bay, because the inlet canal connects with the lake. The gate is closed to ensure safety of work operation at the spillway tunnel and pump drainage operation after discharge, but it is designed to start at the opening water level of 12.0 m. The design head is four meters calculated as 14 m of design maximum water level minus 10.0 m of gate sill level. The gates are designed for installation in three canals 10.0 m in width. The gate material shall be stainless steel to reduce the maintenance cost because painting is not required. For the inspection and repair of gates, stop-logs are designed at the upstream side.

(c) Pump Equipment

Main drainage pump equipment for the spillway tunnel is installed in the outlet facility at the downstream side. Therefore, water will remain up to the invert level of the spillway tunnel in the vertical shaft. The remaining water is discharged into the downstream of the spillway tunnel instead of ground level to save energy consumption. Volume of suspended water remaining in the vertical shaft is approx. 1,800 m^3 estimated as 585 m^2 of wet area in the vertical shaft times 3.0 m of water depth. As the required specifications of suspended water drainage pump, the discharge is designed as 1.3 m^3/min./unit with 7.0 m of total head and two units will be designed for standby duty.

Table 7.2.26 Comparison of Suspended Water Drainage Pump

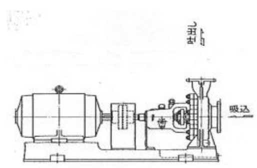
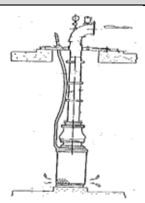
Type of Pump	Single Suction Volute Pump	Screw Type Volute Pump	Submersible Screw Type Volute Pump
Outline*			
Applicable Range*	0.1 - 24 m3/min 2.5 - 140 m Industrial Water, Sea Water	0.24 - 29 m^3/min 3.5 - 100 m Waste Water, Sludge	0.24 - 16 m^3/min 2.5 - 60 m River Water, Rainwater, Waste Water
Condition of Installation	To be installed in dry area.	To be installed in dry area.	To be installed in wet area.
Operation and Maintenance	It is easy to conduct disassembling, inspection and repair of pump at the site.	It is easy to conduct disassembling, inspection and repair of pump at the site.	It is difficult to conduct maintenance of pump installed in wet area of vertical shaft at 50 m underground. It is required to conduct periodical inspection of electric motor against leakage of electricity.
Life Cycle	It is longer life than submersible pump.	It is longer life than submersible pump.	It is shorter life than normal pump.
Evaluation		Adopted: It has advantage of applicable liquid including sand/floating debris and easy maintenance.	

Note: Mark "*" means data source from manufacturer' s catalogue.
Source: JICA Study Team, Manufacturer's catalogue

Screw type volute pump is suggested against clogging of foreign materials and no sealing water system is preferable because of no water feeding system and easy maintenance. The suggested pump material is superior steel to resist abrasion because the water contains suspended solids.

Two units with one as standby submersible motor pump are designed for drainage of dry area in the vertical shaft to ground level. The floor will be able to expand to the opening of sump pit by means of a grating.

Table 7.2.27 Comparison of Sump Pit Drainage Pump

Type of Pump	Horizontal Single Suction Pump	Submersible Motor Pump
Outline*		
Applicable Range*	0.1 ~ 24 m^3/min 2.5 - 140 m	< 14 m^3/min 6 - 80 m
Condition of Installation	To be installed in dry area. It is larger space than the vertical type. It is necessary to make water filling of pump suction pipe prior to pump operation depending on water level.	To be installed in sump pit. It is smaller space than the horizontal type. It is possible to expand the floor to the opening of sump pit by means of a grating.

Type of Pump	Horizontal Single Suction Pump	Submersible Motor Pump
Operation and Maintenance	It is possible to conduct disassembling, inspection and repair of pump at the site.	It is required to lift up pump to floor for inspection.
Life Cycle	It is longer life than submersible pump.	It is shorter life than normal pump.
Evaluation		Adopted: It has advantage of expandable floor.

Note: Mark "*" means data source from manufacturer' s catalogue.
Source: JICA Study Team, Manufacturer's catalogue

(d) Ventilation Equipment in the Inlet Vertical Shaft

As to the amount of ventilation for the dry area in the inlet vertical shaft, the bigger value between the amount of ventilation based on floor area and the required amount of ventilation to exhaust heat generated by equipment is adopted. The ventilation system for dry area in the inlet vertical shaft is composed of air supply fan and exhaust fan.

(i) Amount of ventilation based on floor area

Base amount of ventilation applicable to a windowless factory is 15 $m^3/m^2\cdot hr$ as prescribed by the Japanese Building Standards Law. The amount of ventilation is calculated from the floor area and the number of stories as follows:

$$V_1 = 15\ m^3/m^2\cdot hr \times 72\ m^2 / 60\ min/hr \times 2 = 36\ m^3/min$$

(ii) Amount of ventilation to exhaust heat

Required amount of ventilation in the dry area of vertical shaft is calculated from heat discharge of the suspended water drainage pump as follows:

$$Q_M = P_M \quad (1 - \eta_M) / \eta_M$$

$$= 3.7 \times (1 - 0.9) / 0.9$$
$$= 1.0$$

Where;

Q_M: heat discharge of electric motor (kW)
P_M: output of electric motor (kW)
η_M: efficiency of electric motor

$$V_2 = Q_M / (60\ \rho \cdot C_P\ (t_N - t_O))$$

$$= 1.0 / (60 \times 1.024 \times 0.00028 \times (37 - 30))$$
$$= 8.0 \quad \rightarrow \quad 10\ m^3/min$$

Where;

ρ: air density (= 1.024 kg/m^3)
CP : low pressure specific heat of the air (= 0.00028 kw hr/kg·℃)

t_N : indoor temperature (= 37℃)

t_O : outdoor temperature (=30℃)

From the comparison of these calculations, the amount of ventilation is set at 36 m^3/min.

(2) Outlet Facility

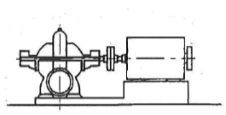
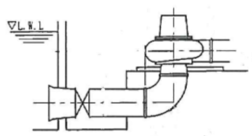
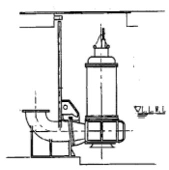
(a) Main Drainage Pump

The pump capacity is designed for dewatering of the tunnel within five days after stoppage of inflow into the spillway tunnel. The volume of water remaining in the tunnel is assumed as 1,200,000 m^3 based on the size of tunnel and vertical shafts. The required drainage capacity is as follows:

$$\text{Drainage capacity: } Q_{out} = 1{,}200{,}000 \text{ m}^3 / (5 \times 24 \times 60 \times 60)$$
$$= 2.77 \text{ m}^3/\text{sec, Say } 2.8 \text{ m}^3/\text{sec}$$

As for the arrangement of equipment in the dry area of outlet vertical shaft, the area should be designed to enable the installation of elevator and staircase, ventilation duct, cable tray, piping and unloading space for equipment in the limited space. The lowest floor should be arranged to accommodate at least two units each of main drainage pump, suspended water drainage pump and bilge pump. In consideration of these conditions, two main drainage pumps are installed for continuous duty without standby unit. According to the design discharge of pumps, two lines of lifting pipe with a diameter of 800mm are installed in the dry area with a bowed height of eight meters. Based on the experience with the Kanda-gawa Underground Reservoir Project in Tokyo, two duty units of drainage pump with the capacity of 0.83m^3/sec each are installed and the lifting pipe of 600mm in diameter in the circular section with a bow height of eight meters has a similar arrangement of equipment.

Table 7.2.28 Comparison of Main Drainage Pump

Type of Pump	Double Suction Volute Pump	Vertical Mixed Flow Volute Pump	Submersible Mixed Flow Volute Pump
Outline			
Applicable Range*	0.6 - 250 m^3/nin 2.2 - 190 m	2.5 - 150 m^3/min 4 - 80 m	1 - 105 m^3/min 2 - 50 m
Condition of Installation	To be installed in dry area. Necessary space is 3 times in comparison with the vertical type.	To be installed in dry area. It is smaller space than the horizontal type.	To be installed in wet area It is the smallest space for installation. Electric motor has a limitation of applicable capacity.

Type of Pump	Double Suction Volute Pump	Vertical Mixed Flow Volute Pump	Submersible Mixed Flow Volute Pump
Operation and Maintenance	It is easy to conduct disassembling, inspection and repair of pump at the site. It is possible to conduct visual inspection of impeller without disassembling at site due to dry area. It is possible to conduct disassembling and inspection of pump without removal of electric motor.	It is easy to conduct disassembling, inspection and repair of pump at the site. It is possible to conduct visual inspection of impeller without disassembling at site due to dry area. To build pump and motor floor separately, it is possible to conduct disassembling and inspection of pump without removal of electric motor.	It is difficult to conduct inspection/repair of pump at site. It is difficult to conduct maintenance of pump installed in wet area of vertical shaft at 50 m underground. It is required to conduct periodical inspection of electric motor against leakage of electricity.
Life Cycle	It is longer life than submersible pump.	It is longer life than submersible pump.	It is shorter life than normal pump.
Evaluation		Adopted: It has advantage of small installation space and easy maintenance.	

Note: Mark "*" means data source from manufacturer' s catalogue.
Source: JICA Study Team, Manufacturer's catalogue

The pump is designed as a vertical mixed flow volute pump due to high pump head, and a double-floor type for convenience in maintenance. The pump is required to be equipped with adequate sealing method instead of a sealing water system for pump shaft. The electric-motor driven prime mover for pump type is selected in terms of easy maintenance and the very limited space of fifty meters underground and simple installation, in comparison with the internal combustion engine without cooling water system, fuel oil piping and air supply/exhaust pipe. The electric motor output is calculated as follows. (Refer to the Draft Technical Guideline of Pumping Equipment, Draft Design Guideline: Supervision of the Ministry of Land, Infrastructure, Transport and Tourism, Japan.)

$$P = \rho \cdot g \cdot Q \cdot H / (1000 \cdot \eta p \cdot \eta g) \times (1 + \alpha)$$

$$= 1000 \times 9.8 \times 1.4 \times 61 / (1000 \times 0.85 \times 0.92) \times (1 + 0.15)$$

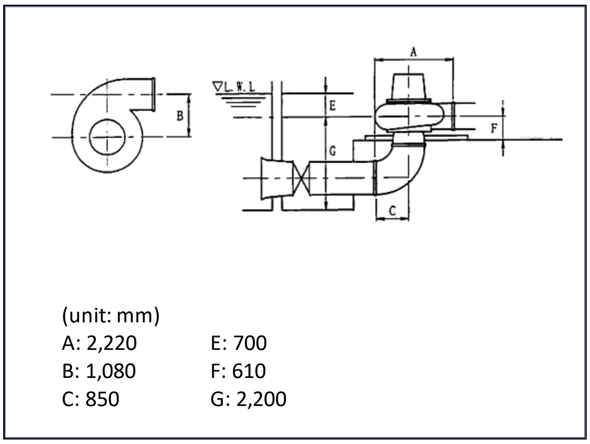
$$= 1{,}229 \text{ kW, Say } 1{,}300 \text{ kW}$$

where;

P : electric motor output (kW)
ρ : water density (1,000 kg/m^3)
g : gravitational acceleration (9.8 m/sec^2)
Q : pump discharge (1.4 m^3/sec)
H : total head (61 m)

As for pump operation of dewatering from upper part of vertical shaft, it is assumed that the pump will be in operation at the excess flow range due to low pump head. Therefore, it is requested to study countermeasures against excess flow as speed control operation, etc., and water-hammering action to the pump discharge pipeline by power failure.

According to the Draft Technical Guideline of Pumping Equipment, pump installation is shown in Figure 7.2.26 in case of vertical mixed flow volute pump with a discharge capacity of 1.4 m^3/sec. Low water level at pump suction chamber is to be set at least at 2,900 mm (‘E’ + ‘G’) above the bottom of the chamber.



Source: Technical Guideline of Pumping Equipment

Figure 7.2.26 Pump Dimension

(b) Suspended Water Drainage Pump

Suspended water remaining in the outlet vertical shaft is designed to discharge into the ground level in the same manner as the main drainage pump. Volume of remaining suspended water is assumed as 1,400m^3 based on the wet area of 504 m^2 times depth of 2.8 m. Discharge capacity of two units of suspended water drainage pump in the outlet facility with one of them as standby unit, is designed at 1.0 m^3/min/unit and pump head is 77 m under the one day drainage condition. The pump is designed as a screw type volute pump because this is clog-less with foreign matters. The pump is required to be equipped with no sealing water system because of easy maintenance and pump material having superior resistance against abrasion shall be selected due the suspended solids in the water. It is required to clarify the necessity of water treatment prior to the discharge of suspended water into the river.

(c) Bilge Pump

For drainage of sump pit for seepage water in the dry area, two units with one of them as standby unit of submersible motor pump are designed to discharge water into the ground. The floor will be able to expand to the opening of sump pit by means of a grating.

(d) Ventilation Equipment in the Outlet Vertical Shaft

Ventilation equipment installed in the outlet vertical shaft send air into both the dry area in vertical shaft and the spillway tunnel. The capacity of ventilation equipment is designed to be bigger in comparison with the required amount of ventilation for the dry area and the tunnel, because operation is not simultaneous.

[Ventilation of dry area in outlet vertical shaft]

In accordance with the calculation procedure of ventilation equipment in the inlet vertical shaft, the amount of ventilation for dry area in the outlet vertical shaft is calculated as follows. The ventilation system for dry area in the outlet vertical shaft is composed of air supply fan and exhaust fan.

(i) Amount of ventilation based on floor area

The applicable base amount of ventilation for a windowless factory is $15 m^3/m^2 \cdot hr$ as prescribed by the Japanese Building Standards Law. The amount of ventilation calculated from the floor area of three stories of $143 m^2$/story is assumed as follows:

$$15\ m^3/m^2 \cdot hr \times 143\ m^2 / 60\ min/hr \times 3 = 107\ m^3/min$$

(ii) Amount of ventilation to exhaust heat

Required amount of ventilation in the dry area of vertical shaft is calculated from heat discharge of the main drainage pumps as follows:

$$Q_{M1} = P_M \cdot (1 - \eta_M) / \eta_M$$
$$= 1{,}300 \times (1 - 0.92) / 0.92$$
$$= 113$$

Where;

Q_{M1}: heat discharge of electric motor (kW)
P_M : output of electric motor (kW)
η_M : efficiency of electric motor

$$V = Q_M / (60\ \rho \cdot C_P\ (t_N - t_O))$$
$$= 226 / (60 \times 1.024 \times 0.00028 \times (37 - 30))$$
$$= 1{,}877,\ \text{Say } 2{,}000\ m^3/min$$

Where;

Q_M : Q_{M1} x 2 units (kW)
ρ : air density (= $1.024 kg/m^3$)
C_P : low pressure specific heat of the air (= 0.00028 kw·hr/kg·℃)
t_N : indoor temperature (= 37℃)
t_O : outdoor temperature (= 30℃)

Hence, the amount of ventilation is designed at $2{,}000 m^3/min$ as the comparison result.

[Ventilation of spillway tunnel]

The ventilation system for spillway tunnel is composed of forced air supply by fan at the outlet vertical shaft and natural air exhaust at the inlet vertical shaft. As experienced in the Kanda-gawa underground reservoir project in Tokyo, the ventilation system was changed to the forced air supply system with natural air exhaust, instead of the forced air supply with forced air exhaust system, according to the recent revision of technical guidelines for ventilation of tunnel.

In accordance with the “Technical Guideline of Ventilation for Construction Work of Tunnel” (Ministry of Health and Welfare, Japan), the amount of ventilation of tunnel is calculated on the basis of two factors: one is the recommended wind velocity of 0.3m/sec times cross-sectional area of tunnel, and the other one is the amount required by operation of heavy-equipment as follows:

(i) Amount of ventilation calculated by wind velocity

Q_1 = (wind velocity) x (cross-sectional area of tunnel)
= $0.3 \times 60 \times \pi \times (12 / 2)^2$
= 2,040 m^3/min

(ii) Amount of ventilation required by heavy-equipment

For cleaning and carrying out sediment inside the tunnel, the amount of ventilation required by operation of heavy equipment is calculated in case of employing a wheel loader with a bucket capacity of 0.3 m^3 and a dump truck with a capacity of 4 tons as follows:

$Q_2 = (H_S \times Q_S \times A) + (H_D \times Q_D \times A) + (H_E \times Q_E \times A_E)$
= (16 x 3.2 x 0.5) + (135 x 3.2 x 0.25)
= 134 m^3/min

Where;

H_S : output of wheel loader (= 16 kW/unit)
Q_S : amount of ventilation required by wheel loader (= 3.2m^3/min-kW)
A_S : load factor of wheel loader (= 0.5)
H_D : output of dump truck (= 135kW/unit)
Q_D : amount of ventilation required by dump truck (= 3.2m^3/min-kW)
A_D : load factor of dump truck (= 0.25)
H_E : output of other machine (= 0)
Q_E : amount of ventilation required by other machine (= 0)
A_E : load factor of other machine (= 0)

(iii) Amount of ventilation of tunnel

$Q_1 + Q_2$ = 2,040 + 134
= 2,174 m^3/min, Say 2,200 m^3/min

Based on the result of calculation above, the amount of ventilation required is 2,000 m^3/min for the dry area of vertical shaft and 2,200 m^3/min for the tunnel. Therefore, the capacity of ventilation equipment is designed at 2,200 m^3/min.

(e) Outlet Gate

The outlet gate is installed at the position always affected by fluctuation of tide level in Manila Bay. The gate is closed to ensure safety of operation of pump drainage and cleaning activities at the time of no water discharging through the spillway tunnel. The gate is designed to open upon water reaches the same water level of the river after filling the tunnel with water incoming from Laguna de Bay. The design head of gate is assumed as 2.7 m based on EL. 11.0 m of mean high-water level in the Manila Bay and EL. 8.3 m of gate sill level, and four units will be installed with channel width of 7.6m. The suggested material of gate is the stainless steel in view of brackish water. Stop logs are designed to conduct inspection and repair works of the gate at the downstream side.

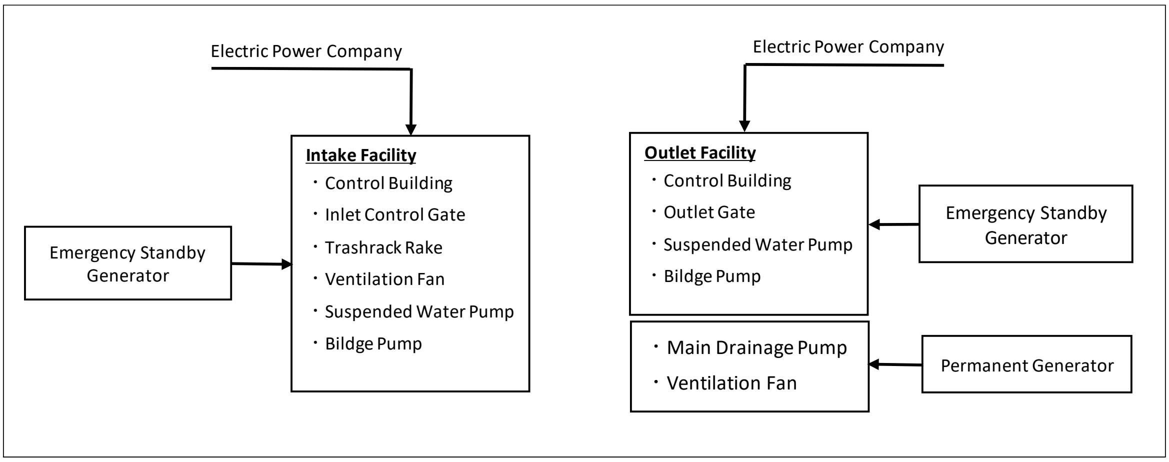
(3) Lifting Equipment

For the installation and inspection/repair works of pump and ventilation equipment, suspended monorail type electric hoists are designed in the inlet facility and the outlet facility. In the outlet facility, the lifting capacity of electric hoist is assumed as approximately 10 tons based on the maximum lifting load of electric motor. At the wet area of both vertical shafts, a gondola is designed to unload heavy equipment, such as wheel loader and dump truck, for inspection/cleaning inside the tunnel. The lifting capacity of gondola is assumed as approximately 10 tons based on 4-ton dump truck with a full load of sediment.

It is conceivable that further study will be conducted to employ additional equipment, such as high-power cleaning truck, high-power vacuum truck and water supply pipe system, for removing/cleaning and carrying out of sediment/mud accumulated in the tunnel.

(4) Electric Power Supply Equipment

As for main drainage pumps and ventilation fan in the outlet facility, electricity supply to the equipment shall be designed to enable supply by in-house power generator instead of the electric power company. Since the equipment is estimated to operate for a short period annually, it is possible to reduce the fixed charges generally imposed for power received from the electric power company. On the other hand, lighting and air-conditioning of office building and auxiliary equipment will be operated by power received from the electric power company. The basic concept of an electric power supply system is shown in Figure 7.2.27.



Souse: JICA Survey Team

Figure 7.2.27 Basic Concept of Electric Power Supply System

(a) Electricity from Power Company

Electricity from the power supply company will be feed to the supervisory control and instrumentation systems, office buildings, trash-rack rakes and inlet/outlet gates, and bilge pumps. As backup against power failure, emergency standby generator unit will be provided to cover the full load capacity of equipment.

(b) In-house Power Generator

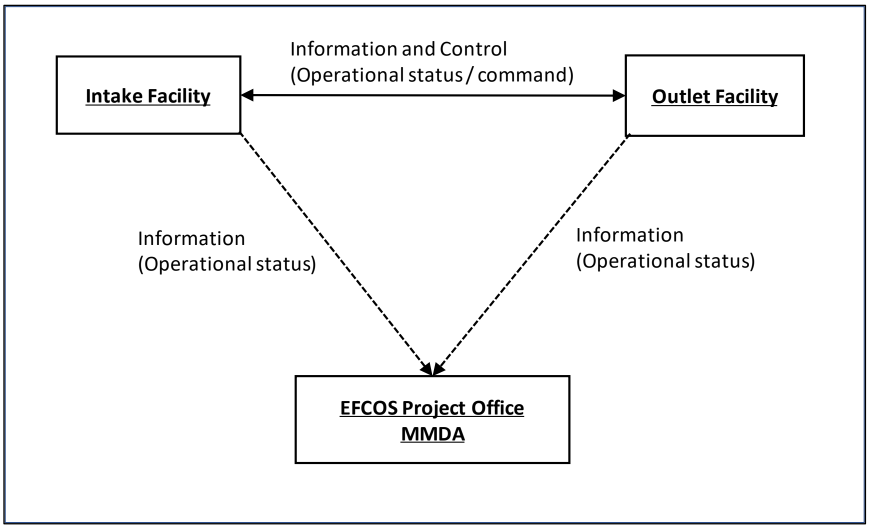
Both main drainage pumps and ventilation fans in the outlet facility will receive electricity from the in-house power generator. Backup against power failure will not be provided as a matter of no urgency.

(5) Supervisory Control Equipment

Gate operation to intake water from the Laguna de Bay and discharge water into the Manila Bay and the pump drainage operation and ventilation are controlled to start and stop individually in both inlet/outlet facilities by means of remote control from the central control panel stationed in the inlet facility. As a backup for emergency, a second central control panel is provided in the outlet facility to ensure remote control of the equipment. Further study is required to set the function of a master station at either inlet/outlet facility. The central control panel is designed to be equipped with status display of the equipment, water level and control/change-over switch. The central control panel will have a monitoring function to confirm the trash-rack rake operation, the inlet/outlet gate operation and the status of inflow/discharge into/from the spillway tunnel, and security in the premises.

For conducting remote control and monitoring of each equipment installed in the inlet/outlet facilities from the central control room, SCADA (Supervisory Control and Data Acquisition) system will be employed together with necessary component to secure proper operation. Video monitoring system will also be employed for monitoring the flow and security. The supervisory control equipment will be equipped with an uninterruptible power supply system or a direct-current power supply system having the capacity of sixty minutes operation to ensure stable operation.

The supervisory control system is formulated as the relationship shown in Figure 7.2.28.



Source: JICA Survey Team

Figure 7.2.28 Supervisory Control System

Interconnection between both facilities is applicable to the radio system or the optical fiber cable to be laid in the tunnel for remote control and data transmission. At present, the allocated frequency range to MMDA may be available for expanding the supervisory control system of the Parañaque Spillway Project in future. (Note: EFCOS: Effective Flood Control Operation System Project Office is located at Rosario Flood Control Gate.)

Source: JICA Survey Team

Figure 7.2.29 EFCOS Project Office

7.3 Study on Rivers in the Downstream Side of Parañaque Spillway

It evaluated the influence the downstream river water level due to drainage of Parañaque Spillway regarding Route 1(Lower Bicutan - South Parañaque R.) and Route 3 (Sucat - Zapote R.), which are evaluated as promising and adoptable as a Parañaque Spillway plan in 7.2.2 (3).

In addition, the drainage discharge will be change due to lake water level of Laguna de Bay and river water level at downstream river channel. However, lake water level is set 14.0m and the drainage discharge is changing due to river water level at downstream river channel in this study.

The analysis model which was modelling in Chapter 3 (3.5 Runoff and Inundation Analysis of Parañaque and Las Piñas Areas) was used to evaluate the influence the downstream river water level. Moreover, the downstream boundary was set design tide level (11.87m).

7.3.1 Route 1 (Lower Bicutan - South Parañaque R.)

The candidate site for outlet facility in Route-1 is around 1.8km on South Parañaque River. The river water level rising was estimated due to the drainage of Parañaque Spillway. The calculation result of river water level and river discharge is shown in Table 7.3.1, Figure 7.3.2 to Figure 7.3.6.

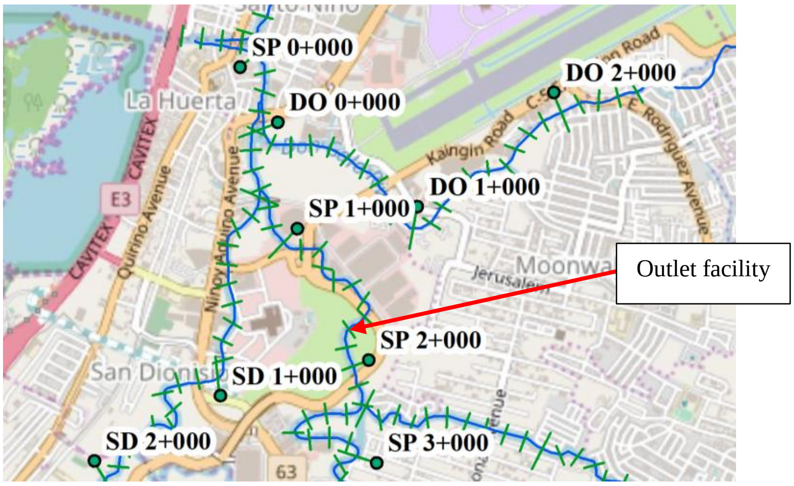


Figure 7.3.1 The Location of Candidate Site of Outlet Facility in Route-1

Table 7.3.1 The River Water Level Change of South Parañaque River (Outlet Facility:SP 1+800)

SP.1+800

Return Period	**without** Parañaque Spillway		**with** Parañaque Spillway				Laguna Lake water level
	WL	River Q	WL	River Q	Outlet Q* Max	Outlet Q* Min	
	(m)	(m^3/S)	(m)	(m^3/S)	(m^3/S)	(m^3/S)	(m)
100	15.0	364.8	-	-	-	-	14.0
50	14.7	315.3	-	-	-	-	14.0
25	14.3	268.5	-	-	-	-	14.0
15	14.1	235.7	-	-	-	-	14.0
10	13.9	210.6	14.0	220.8	124.1	7.9	14.0
5	13.5	168.3	13.8	203.9	124.4	33.1	14.0
2	12.9	110.9	13.6	180.8	124.8	66.8	14.0

* The drainage discharge of Parañaque Spillway is changing due to river water level at downstream river channe.

The river water level changes and drainage discharge of Parañaque Spillway is organized at the cross section (SP.1+800) which is near outlet facility.

The drainage discharge of Parañaque Spillway is changing due to the river water level at downstream river channel (South Parañaque River). In addition, the downstream boundary is set design tide level (11.87m).

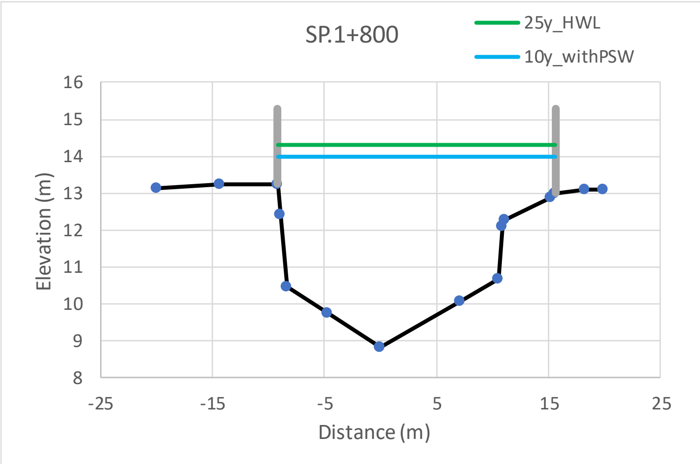


Figure 7.3.2 River Water Level at SP.1+800
(HWL of 25-year Return Period and 10-year Flood + with Parañaque Spillway)

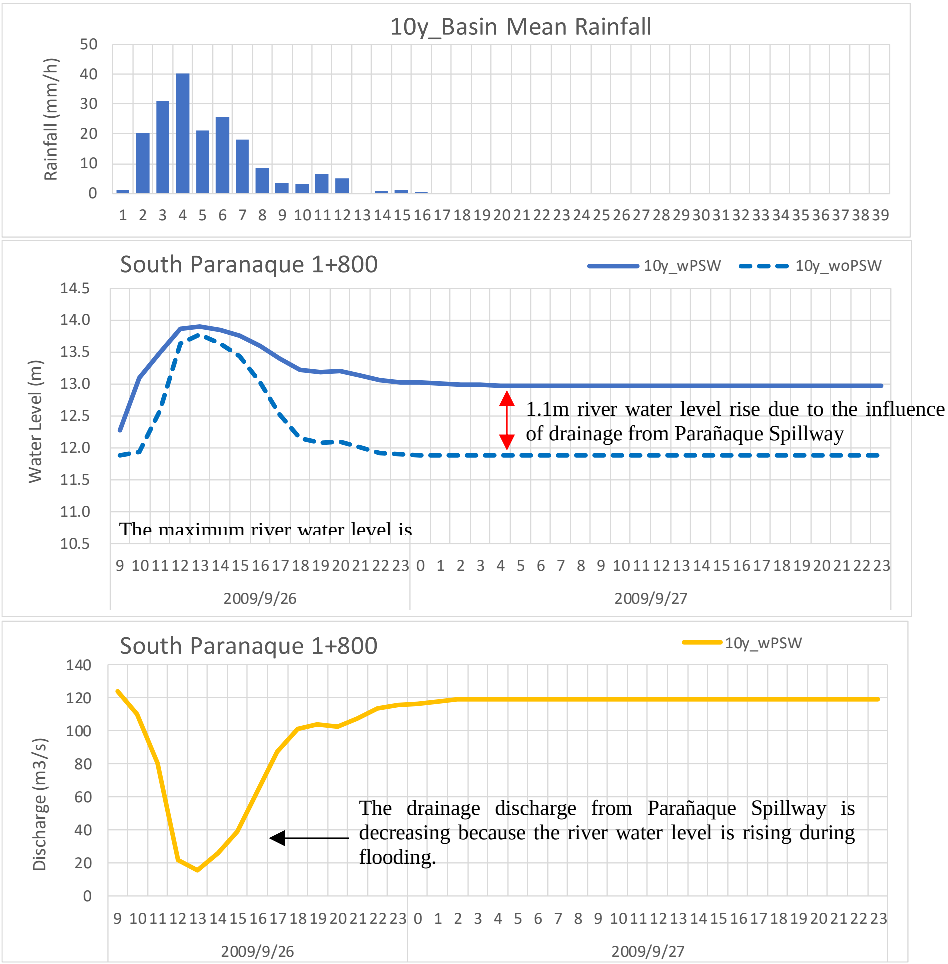


Figure 7.3.3 River Water Level Change at SP.1+800 (With/Without Parañaque Spillway)
Return Preiod:10-year

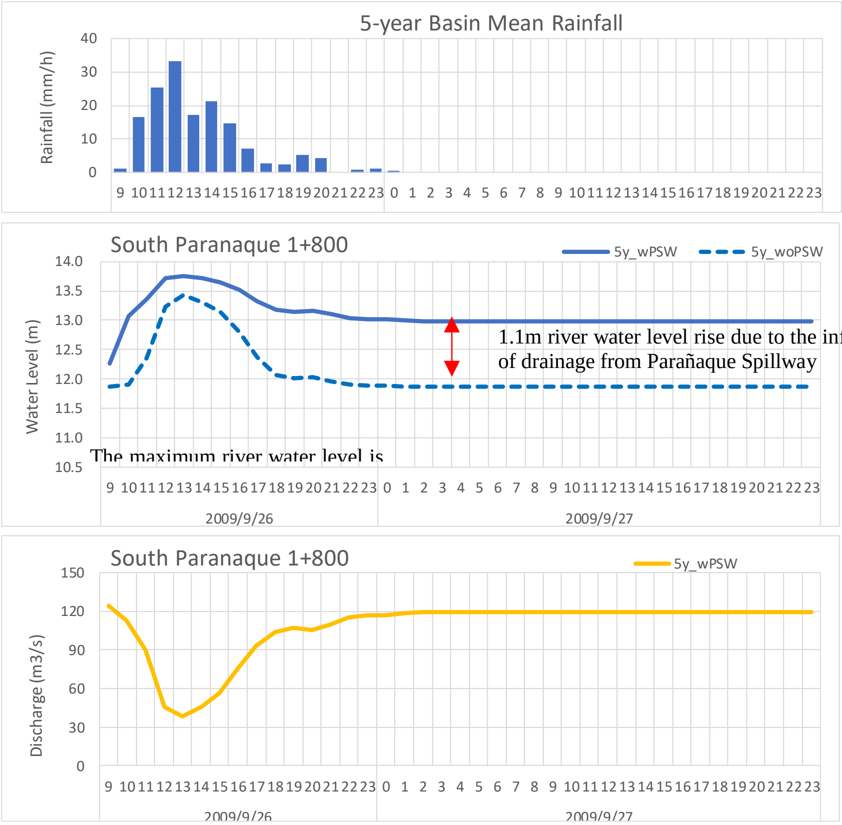


Figure 7.3.4 River Water Level Change at SP.1+800 (With/Without Parañaque Spillway) Return Preiod:5-year

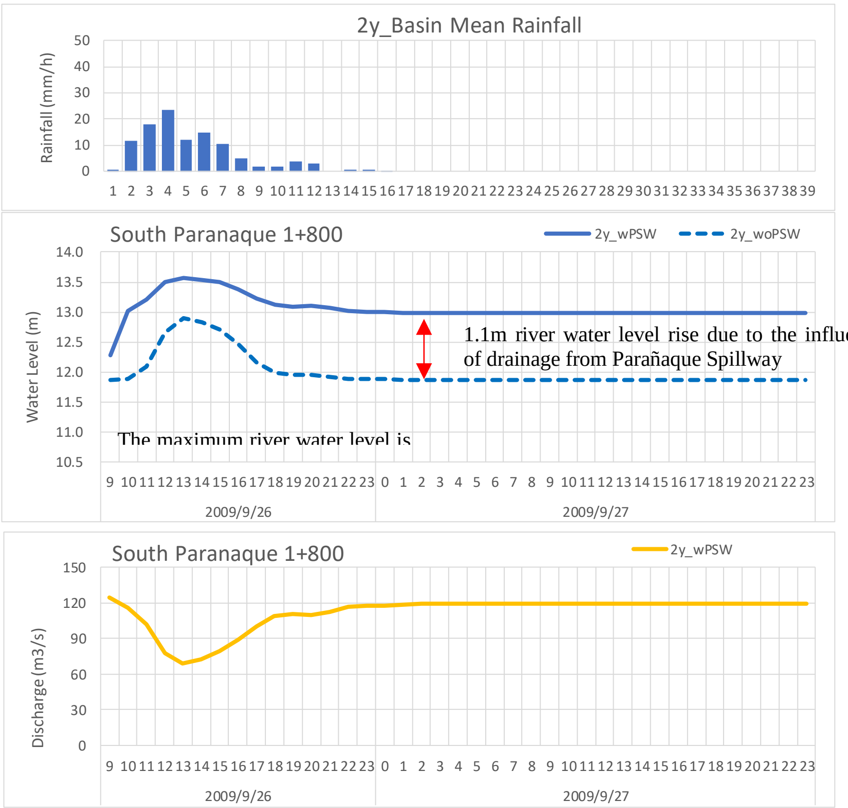


Figure 7.3.5 River Water Level Change at SP.1+800 (With/Without Parañaque Spillway) Return Preiod:2-year

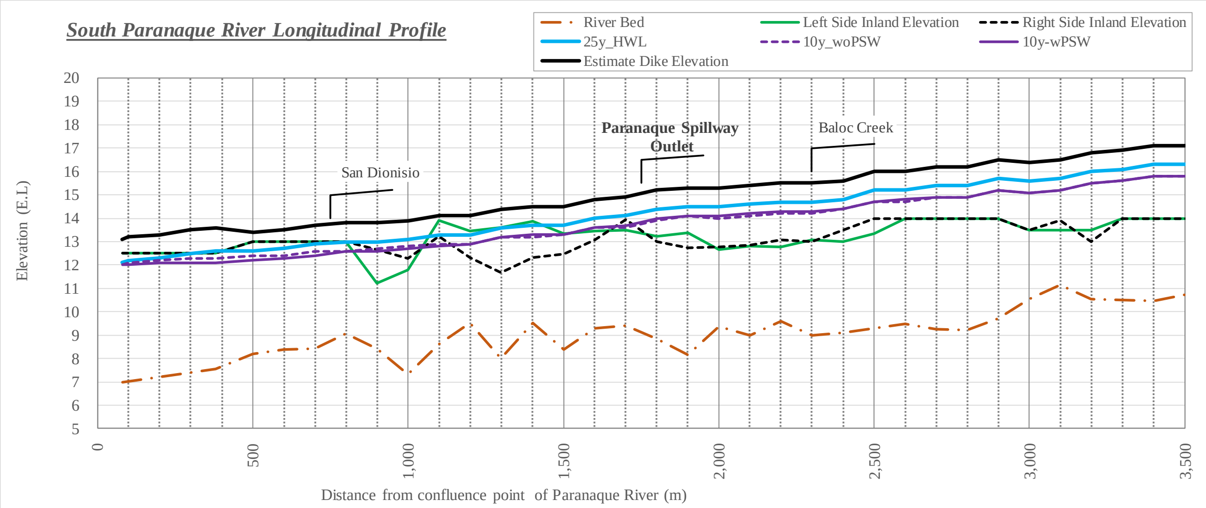


Figure 7.3.6 (1) River Longitudinal Profile of South Parañaque River (Return Period:10-year)

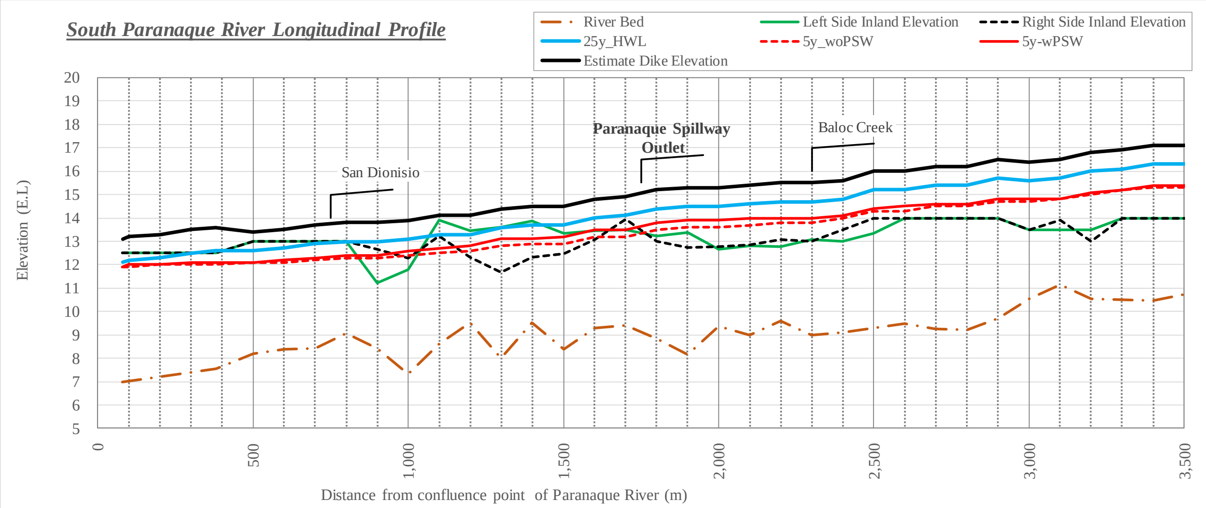


Figure 7.3.6 (2) River Longitudinal Profile of South Parañaque River (Return Period:5-year)

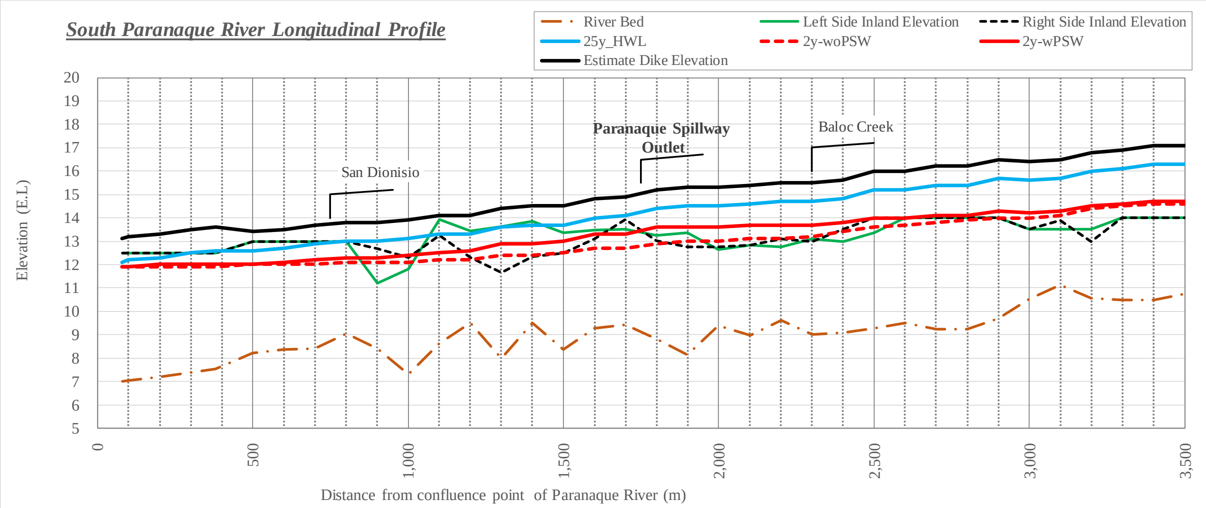


Figure 7.3.6 (3) River Longitudinal Profile of South Parañaque River (Return Period:2-year)

Table 7.3.2 Specification of River Water Level and River Discharge in South Parañaque River

No	STA. No.	距離	単距離	現況河道縦断 河床高	左岸河岸高	右岸河岸高	左岸地盤高	右岸地盤高	Simulated Maximum Water Level (m) Return Period							Simulated Maximum Discharge (m3/s) Return Period						
		(m)	(m)	(E.L.m)	(E.L.m)	(E.L.m)	(E.L.m)	(E.L.m)	100	50	25	15	10	5	2	100	50	25	15	10	5	2
1	SP_0+80	80	80	6.99	12.5	12.5	12.5	12.5	12.4	12.2	12.1	12.1	12.0	11.9	11.9	712.4	617.4	527.0	463.3	414.8	245.2	162.2
2	SP_0+100	100	20	7.03	12.5	12.5	12.5	12.5	12.4	12.3	12.2	12.1	12.1	11.9	11.9	712.4	617.4	527.0	463.3	414.8	245.2	162.3
3	SP_0+200	200	100	7.21	12.5	12.5	12.5	12.5	12.7	12.5	12.3	12.2	12.2	12.0	11.9	712.3	617.3	527.0	463.3	414.7	245.2	162.2
4	SP_0+300	300	100	7.39	12.5	12.5	12.5	12.5	12.9	12.7	12.5	12.4	12.3	12.0	11.9	712.1	617.2	526.9	463.2	414.7	245.2	162.2
5	SP_0+380	380	80	7.54	12.5	12.5	12.5	13.4	13.0	12.8	12.6	12.4	12.3	12.0	11.9	712.0	617.1	526.8	463.1	414.7	245.2	162.2
6	SP_0+500	500	120	8.21	13.0	13.0	13.0	13.5	13.1	12.8	12.6	12.5	12.4	12.1	12.0	481.2	416.2	354.3	310.4	277.1	222.7	148.2
7	SP_0+600	600	100	8.37	13.0	13.0	13.0	13.0	13.2	12.9	12.7	12.5	12.4	12.1	12.0	481.1	416.1	354.2	310.4	277.1	222.7	148.2
8	SP_0+700	700	100	8.42	13.0	13.0	13.0	13.0	13.4	13.1	12.9	12.7	12.6	12.2	12.0	480.5	415.6	353.8	310.1	276.8	222.6	148.0
9	SP_0+800	800	100	9.06	13.0	13.0	13.0	13.0	13.5	13.3	13.0	12.8	12.6	12.3	12.1	479.7	415.0	353.3	309.6	276.4	222.3	147.8
10	SP_0+900	900	100	8.42	11.2	12.7	11.4	12.7	13.5	13.3	13.0	12.8	12.7	12.3	12.1	373.5	322.7	274.4	240.6	214.8	171.3	113.2
11	SP_1+0	1000	100	7.32	11.8	12.3	11.8	12.3	13.6	13.4	13.1	12.9	12.8	12.4	12.1	372.5	321.8	273.7	240.0	214.3	171.0	113.0
12	SP_1+100	1100	100	8.62	13.9	13.2	13.9	13.2	13.8	13.5	13.3	13.0	12.9	12.5	12.2	371.6	321.0	273.1	239.5	213.8	170.6	112.7
13	SP_1+200	1200	100	9.52	13.4	12.3	13.5	12.3	13.9	13.6	13.3	13.1	12.9	12.6	12.2	370.6	320.2	272.4	238.9	213.4	170.3	112.5
14	SP_1+300	1300	100	8.00	13.6	11.7	13.6	11.9	14.2	13.9	13.6	13.4	13.2	12.8	12.4	369.7	319.4	271.8	238.4	212.9	169.9	112.2
15	SP_1+400	1400	100	9.51	13.9	12.3	13.9	12.3	14.3	14.0	13.7	13.4	13.2	12.9	12.4	368.7	318.6	271.1	237.8	212.4	169.6	111.9
16	SP_1+500	1500	100	8.38	13.4	12.5	13.4	12.5	14.3	14.0	13.7	13.5	13.3	12.9	12.5	367.7	317.8	270.4	237.3	212.0	169.3	111.7
17	SP_1+600	1600	100	9.29	13.5	13.1	13.5	13.1	14.7	14.4	14.0	13.8	13.6	13.2	12.7	366.8	317.0	269.8	236.7	211.5	168.9	111.4
18	SP_1+700	1700	100	9.41	13.5	13.9	13.5	13.9	14.6	14.4	14.1	13.8	13.6	13.2	12.7	365.8	316.2	269.1	236.2	211.0	168.6	111.1
19	SP_1+800	1800	100	8.83	13.2	13.0	13.4	13.4	15.0	14.7	14.4	14.1	13.9	13.5	12.9	364.9	315.4	268.5	235.6	210.5	168.3	110.9
20	SP_1+900	1900	100	8.15	13.4	12.7	13.4	14.3	15.2	14.9	14.5	14.3	14.1	13.6	13.0	363.9	314.6	267.8	235.1	210.1	167.9	110.6
21	SP_2+0	2000	100	9.37	12.7	12.8	12.9	13.8	15.2	14.9	14.5	14.3	14.0	13.6	13.0	362.9	313.8	267.1	234.5	209.6	167.6	110.3
22	SP_2+100	2100	100	8.99	12.8	12.8	14.2	13.0	15.3	15.0	14.6	14.4	14.1	13.7	13.1	362.0	313.0	266.5	233.9	209.1	167.3	110.0
23	SP_2+200	2200	100	9.61	12.8	13.1	12.8	13.1	15.4	15.0	14.7	14.4	14.2	13.8	13.1	361.0	312.2	265.8	233.4	208.6	166.9	109.8
24	SP_2+300	2300	100	9.00	13.1	13.0	13.5	13.0	15.4	15.0	14.7	14.4	14.2	13.8	13.2	360.2	311.4	265.2	232.9	208.2	166.6	109.7
25	SP_2+400	2400	100	9.10	13.0	13.5	13.0	13.6	15.5	15.2	14.8	14.6	14.4	14.0	13.4	319.8	276.6	235.7	206.9	185.0	147.9	97.4
26	SP_2+500	2500	100	9.28	13.3	14.0	13.4	14.0	15.8	15.5	15.2	14.9	14.7	14.3	13.6	318.9	275.8	235.0	206.3	184.5	147.6	97.2
27	SP_2+600	2600	100	9.50	14.0	14.0	14.0	14.0	15.9	15.6	15.2	15.0	14.7	14.3	13.7	317.9	275.0	234.3	205.8	184.0	147.2	96.9
28	SP_2+700	2700	100	9.25	14.0	14.0	14.0	14.0	16.0	15.7	15.4	15.1	14.9	14.5	13.8	317.0	274.2	233.6	205.2	183.5	146.9	96.7
29	SP_2+800	2800	100	9.22	14.0	14.0	14.0	14.0	16.0	15.7	15.4	15.1	14.9	14.5	13.9	316.0	273.4	233.0	204.6	183.0	146.5	96.5
30	SP_2+900	2900	100	9.70	14.0	14.0	14.0	14.0	16.4	16.0	15.7	15.4	15.2	14.7	14.0	315.1	272.6	232.3	204.0	182.5	146.1	96.4
31	SP_3+0	3000	100	10.53	13.5	13.5	13.5	13.6	16.2	15.9	15.6	15.3	15.1	14.7	14.0	314.1	271.8	231.6	203.5	182.0	145.8	96.2
32	SP_3+100	3100	100	11.14	13.5	13.9	13.5	13.9	16.4	16.0	15.7	15.4	15.2	14.8	14.1	313.2	271.0	231.0	202.9	181.5	145.4	96.0
33	SP_3+200	3200	100	10.55	13.5	13.0	13.5	13.5	16.7	16.4	16.0	15.7	15.5	15.0	14.4	312.2	270.1	230.3	202.3	181.0	145.0	95.8
34	SP_3+300	3300	100	10.49	14.0	14.0	14.0	14.1	16.9	16.5	16.1	15.8	15.6	15.2	14.5	311.3	269.3	229.6	201.7	180.5	144.6	95.7
35	SP_3+400	3400	100	10.47	14.0	14.0	14.0	14.0	17.1	16.7	16.3	16.0	15.8	15.3	14.6	310.3	268.5	228.9	201.2	180.0	144.3	95.5
36	SP_3+500	3500	100	10.74	14.0	14.0	14.2	14.3	17.1	16.7	16.3	16.0	15.8	15.3	14.6	309.3	267.7	228.2	200.6	179.5	143.9	95.4
37	SP_3+600	3600	100	10.65	14.0	14.0	14.0	14.0	17.3	16.9	16.4	16.1	15.9	15.4	14.6	309.3	267.7	228.2	200.6	179.5	143.9	95.4

No	STA. No.	計画高水位−堤内地盤高 左岸	右岸	余裕高	必要堤防天端標高	Simulated Maximum Water Level (m) with Paranaque Spillway_Route.A Return Period							Simulated Maximum Discharge (m3/s) with Paranaque Spillway_Route.A Return Period						
		(m)	(m)	(m)	(m)	100	50	25	15	10	5	2	100	50	25	15	10	5	2
1	SP_0+80	-0.4	-0.4	1.0	13.1	-	-	-	-	12.0	11.9	11.9	-	-	-	-	317.5	279.9	228.9
2	SP_0+100	-0.3	-0.3	1.0	13.2	-	-	-	-	12.0	12.0	11.9	-	-	-	-	317.5	279.9	228.9
3	SP_0+200	-0.2	-0.2	1.0	13.3	-	-	-	-	12.1	12.0	12.0	-	-	-	-	317.4	279.9	228.9
4	SP_0+300	0.0	0.0	1.0	13.5	-	-	-	-	12.1	12.1	12.0	-	-	-	-	317.4	279.9	228.9
5	SP_0+380	0.1	-0.8	1.0	13.6	-	-	-	-	12.1	12.1	12.0	-	-	-	-	317.4	279.9	228.9
6	SP_0+500	-0.4	-0.9	0.8	13.4	-	-	-	-	12.2	12.1	12.0	-	-	-	-	290.9	257.9	213.2
7	SP_0+600	-0.3	-0.3	0.8	13.5	-	-	-	-	12.3	12.2	12.1	-	-	-	-	290.9	257.8	213.2
8	SP_0+700	-0.1	-0.1	0.8	13.7	-	-	-	-	12.4	12.3	12.2	-	-	-	-	290.6	257.7	213.1
9	SP_0+800	0.0	0.0	0.8	13.8	-	-	-	-	12.6	12.4	12.3	-	-	-	-	290.2	257.3	212.9
10	SP_0+900	1.6	0.3	0.8	13.8	-	-	-	-	12.6	12.4	12.3	-	-	-	-	225.3	207.4	182.9
11	SP_1+0	1.3	0.8	0.8	13.9	-	-	-	-	12.7	12.6	12.4	-	-	-	-	224.9	207.0	182.7
12	SP_1+100	-0.6	0.1	0.8	14.1	-	-	-	-	12.8	12.7	12.5	-	-	-	-	224.4	206.6	182.5
13	SP_1+200	-0.1	1.0	0.8	14.1	-	-	-	-	12.9	12.8	12.6	-	-	-	-	223.9	206.2	182.2
14	SP_1+300	-0.0	1.7	0.8	14.4	-	-	-	-	13.2	13.1	12.9	-	-	-	-	223.4	205.8	182.0
15	SP_1+400	-0.2	1.4	0.8	14.5	-	-	-	-	13.3	13.1	12.9	-	-	-	-	222.8	205.4	181.7
16	SP_1+500	0.4	1.2	0.8	14.5	-	-	-	-	13.3	13.2	13.0	-	-	-	-	222.3	205.0	181.5
17	SP_1+600	0.5	0.9	0.8	14.8	-	-	-	-	13.6	13.5	13.3	-	-	-	-	221.8	204.7	181.2
18	SP_1+700	0.6	0.2	0.8	14.9	-	-	-	-	13.7	13.5	13.3	-	-	-	-	221.3	204.3	181.0
19	SP_1+800	1.0	1.0	0.8	15.2	-	-	-	-	14.0	13.8	13.6	-	-	-	-	220.8	203.9	180.8
20	SP_1+900	1.1	0.2	0.8	15.3	-	-	-	-	14.1	13.9	13.6	-	-	-	-	212.4	170.4	113.7
21	SP_2+0	1.6	0.7	0.8	15.3	-	-	-	-	14.1	13.9	13.6	-	-	-	-	211.9	170.0	113.5
22	SP_2+100	0.4	1.6	0.8	15.4	-	-	-	-	14.2	14.0	13.7	-	-	-	-	211.4	169.6	113.3
23	SP_2+200	1.9	1.6	0.8	15.5	-	-	-	-	14.3	14.0	13.7	-	-	-	-	210.9	169.2	113.0
24	SP_2+300	1.2	1.7	0.8	15.5	-	-	-	-	14.3	14.0	13.7	-	-	-	-	210.4	168.8	112.8
25	SP_2+400	1.8	1.2	0.8	15.6	-	-	-	-	14.4	14.1	13.8	-	-	-	-	186.6	149.6	99.7
26	SP_2+500	1.9	1.2	0.8	16.0	-	-	-	-	14.7	14.4	14.0	-	-	-	-	186.0	149.2	99.4
27	SP_2+600	1.2	1.2	0.8	16.0	-	-	-	-	14.8	14.5	14.0	-	-	-	-	185.5	148.8	99.2
28	SP_2+700	1.4	1.4	0.8	16.2	-	-	-	-	14.9	14.6	14.1	-	-	-	-	185.0	148.4	99.0
29	SP_2+800	1.4	1.4	0.8	16.2	-	-	-	-	14.9	14.6	14.1	-	-	-	-	184.5	148.0	98.7
30	SP_2+900	1.7	1.7	0.8	16.5	-	-	-	-	15.2	14.8	14.3	-	-	-	-	183.9	147.6	98.5
31	SP_3+0	2.1	2.0	0.8	16.4	-	-	-	-	15.1	14.8	14.2	-	-	-	-	183.4	147.2	98.3
32	SP_3+100	2.2	1.8	0.8	16.5	-	-	-	-	15.2	14.8	14.3	-	-	-	-	182.9	146.8	98.0
33	SP_3+200	2.5	2.5	0.8	16.8	-	-	-	-	15.5	15.1	14.5	-	-	-	-	182.4	146.4	97.8
34	SP_3+300	2.1	2.0	0.8	16.9	-	-	-	-	15.6	15.2	14.6	-	-	-	-	181.8	146.0	97.6
35	SP_3+400	2.3	2.3	0.8	17.1	-	-	-	-	15.8	15.4	14.7	-	-	-	-	181.3	145.6	97.4
36	SP_3+500	2.1	2.0	0.8	17.1	-	-	-	-	15.8	15.4	14.7	-	-	-	-	180.8	145.2	97.2
37	SP_3+600	2.4	2.4	0.8	17.2	-	-	-	-	15.9	15.5	14.8	-	-	-	-	180.5	145.0	97.1

: The section where the water level rose due to drainage of Parañaque Spillway

7.3.2 Route 3 (Sucat - Zapote R.)

The candidate site for outlet facility in Route-3 is around 100 m from river mouth on Zapote River. The river water level rising was estimated due to the drainage of Parañaque Spillway.

The calculation result of river water level and river discharge is shown in Table 7.3.3, Figure 7.3.8 to Figure 7.3.14.

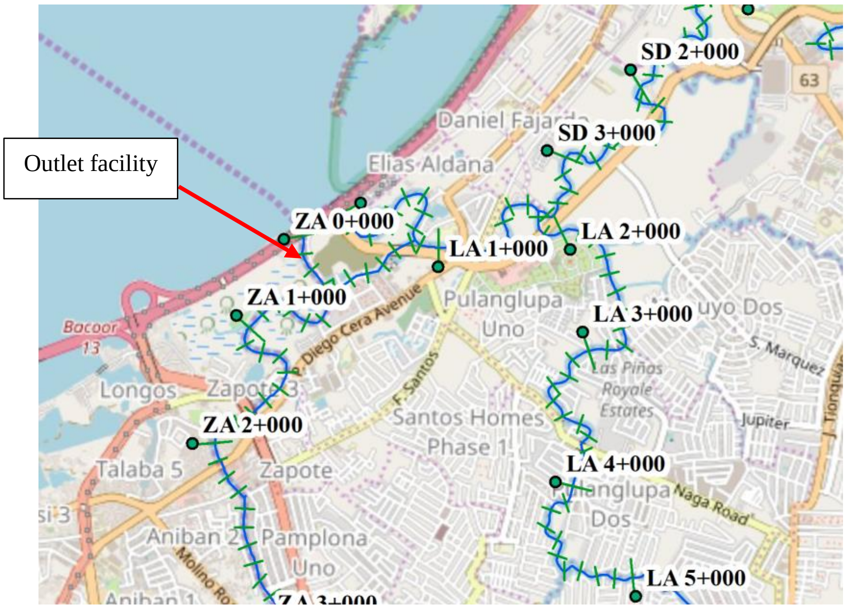


Figure 7.3.7 The Location of Candidate Site of Outlet Facility in Route-3

Table 7.3.3 The River Water Level Change of Zapote River (Outlet Facility:ZA 0+100)

ZA.0+100

Return Period	**<u>without</u>** Parañaque Spillway		**<u>with</u>** Parañaque Spillway				Laguna Lake water level
	WL	River Q	WL	River Q	Outlet Q* Max	Outlet Q* Min	
	(m)	(m^3/S)	(m)	(m^3/S)	(m^3/S)	(m^3/S)	(m)
100	12.2	677.6	12.3	827.8	176.1	155.1	14.0
50	12.1	586.0	12.3	739.8	176.1	159.0	14.0
25	12.0	501.8	12.2	659.2	176.2	162.8	14.0
15	12.0	442.5	12.1	602.5	176.2	165.3	14.0
10	12.0	396.4	12.1	558.4	176.2	167.1	14.0
5	11.9	319.0	12.0	483.8	176.2	169.9	14.0
2	11.9	216.1	12.0	383.7	176.2	172.8	14.0

The river water level changes and drainage discharge of Parañaque Spillway is organized at the cross section (ZA.0+100) which is near outlet facility.

The drainage discharge of Parañaque Spillway is changing due to the river water level at downstream river channel (Zapote River). In addition, the downstream boundary is set design tide level (11.87m).

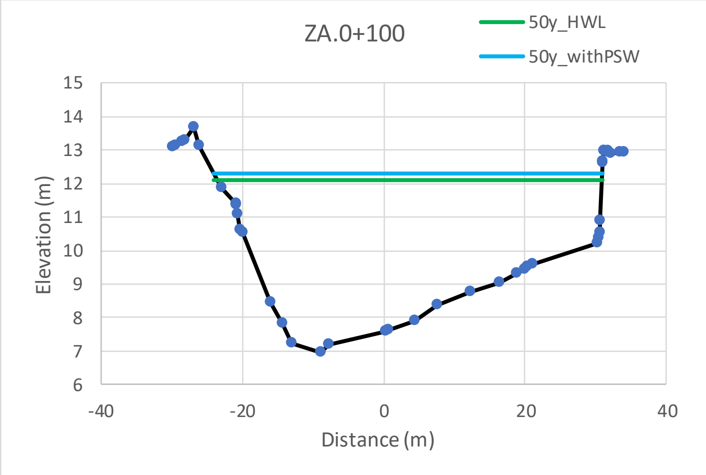


Figure 7.3.8 River Water Level at ZA.0+100
(HWL of 50-year Return Period and 50-year Flood + with Parañaque Spillway)

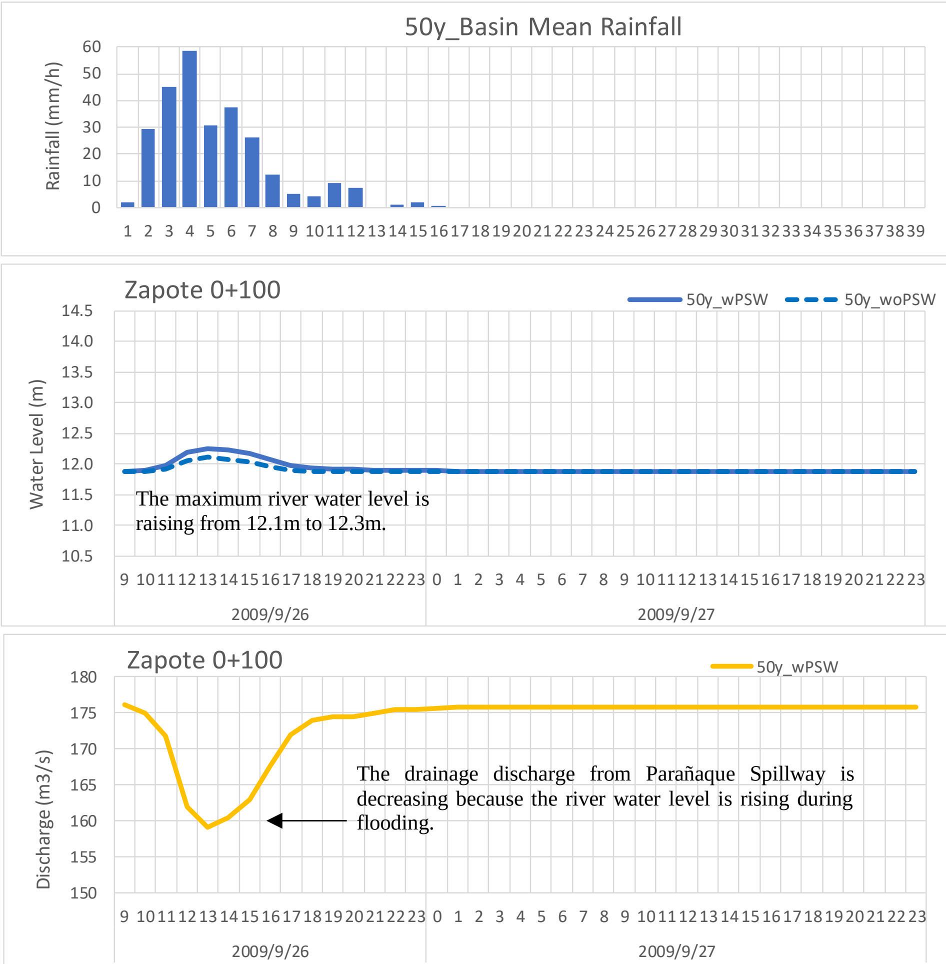


Figure 7.3.9 River Water Level Change at ZA.0+100 (With/Without Parañaque Spillway)
Return Period:50-year

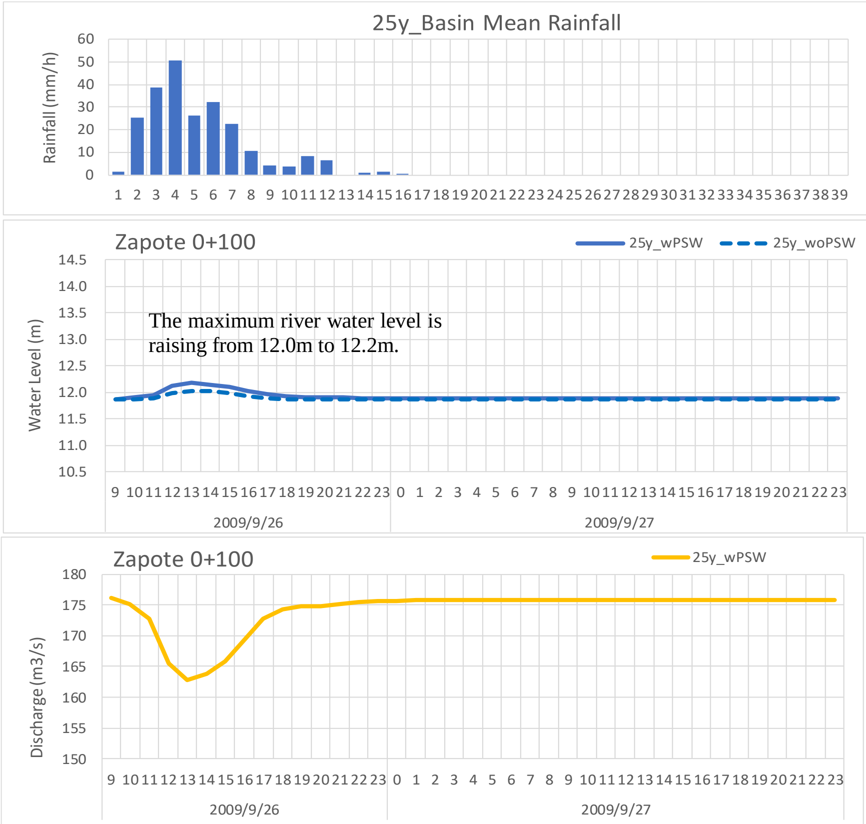


Figure 7.3.10 River Water Level Change at ZA.0+100 (With/Without Parañaque Spillway) Return Period: 25-year

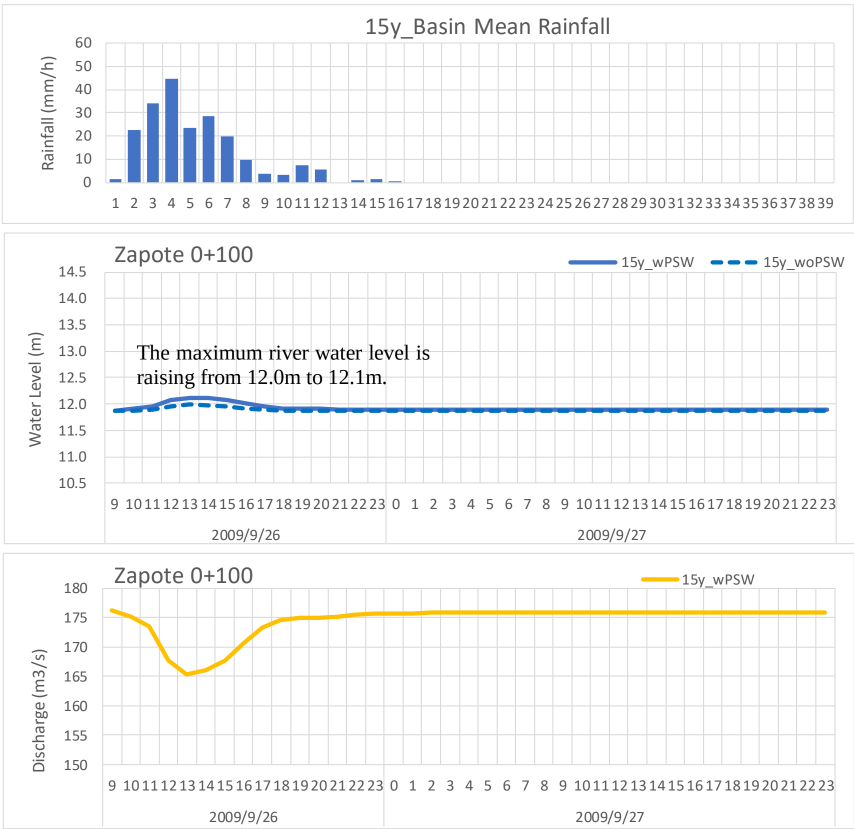


Figure 7.3.11 River Water Level Change at ZA.0+100 (With/Without Parañaque Spillway) Return Preiod:15-year

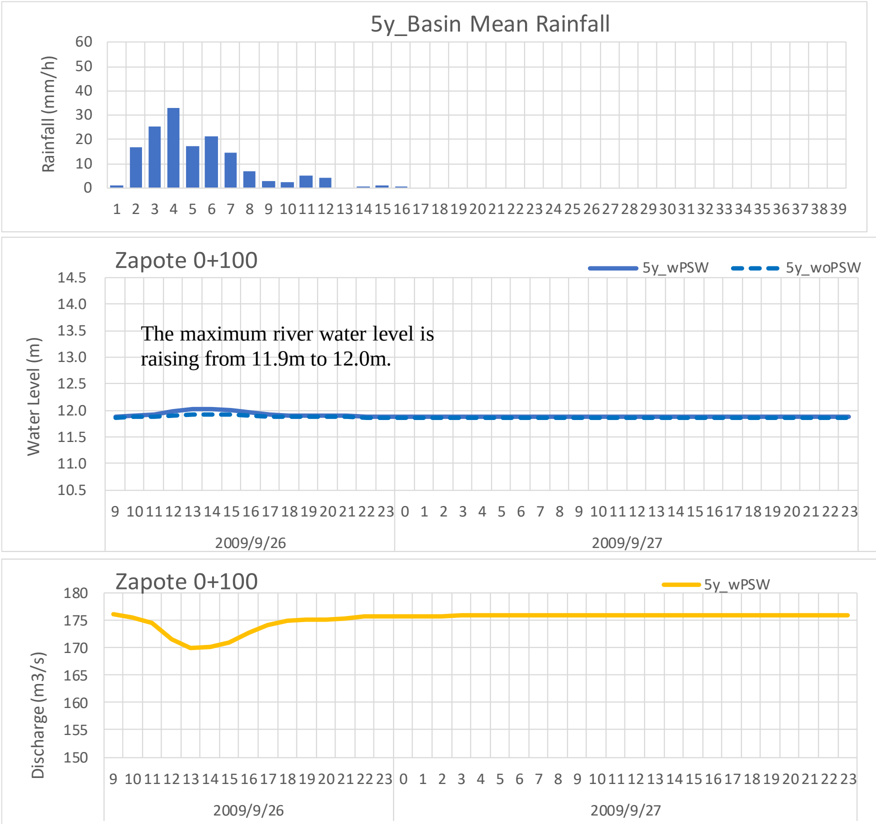


Figure 7.3.12 River Water Level Change at ZA.0+100 (With/Without Parañaque Spillway) Return Preiod:5-year

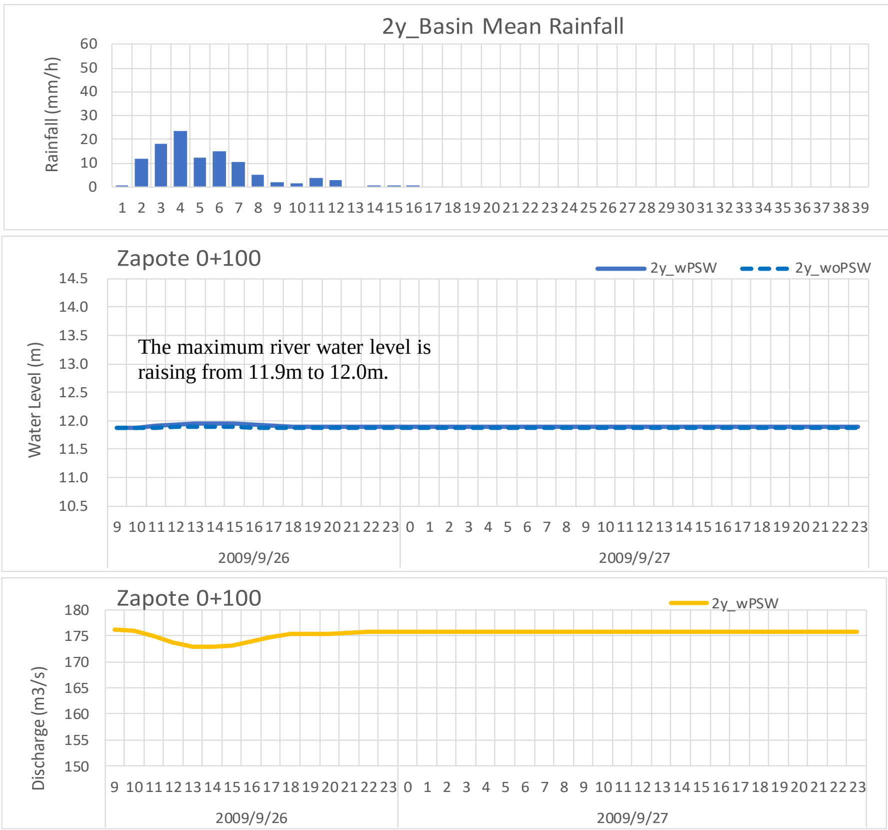


Figure 7.3.13 River Water Level Change at ZA.0+100 (With/Without Parañaque Spillway) Return Preiod:2-year

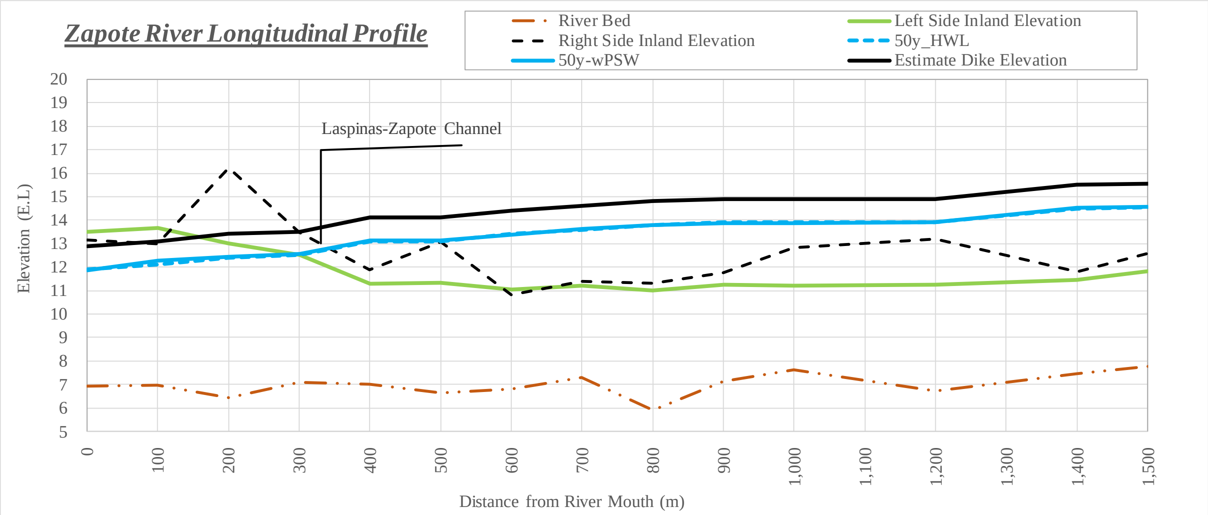


Figure 7.3.14 (1) River Longitudinal Profile of Zapote River (Return Period:50-year)

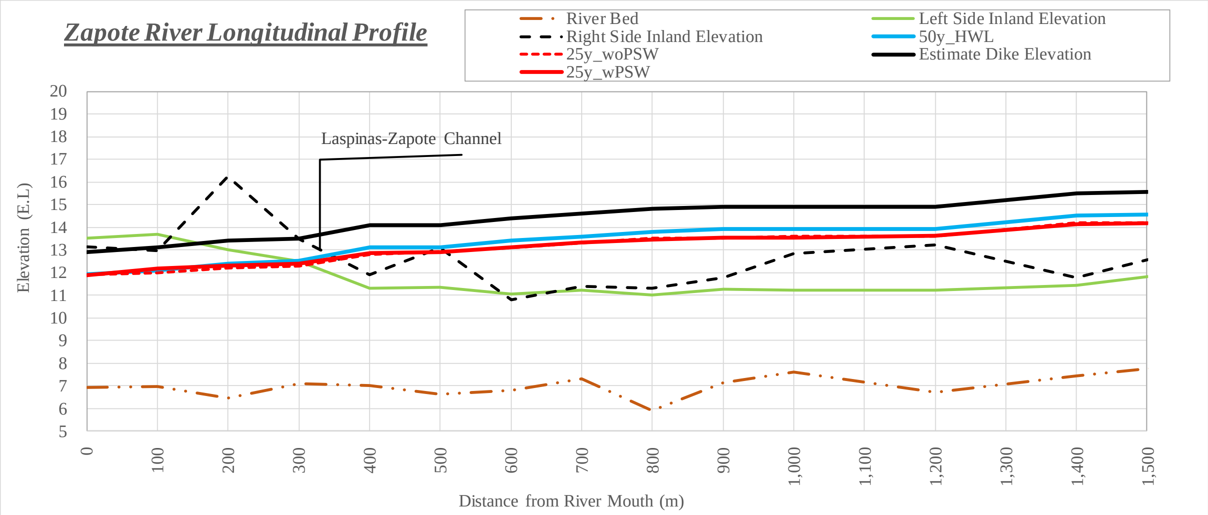


Figure 7.3.14 (2) River Longitudinal Profile of Zapote River (Return Period:25-year)

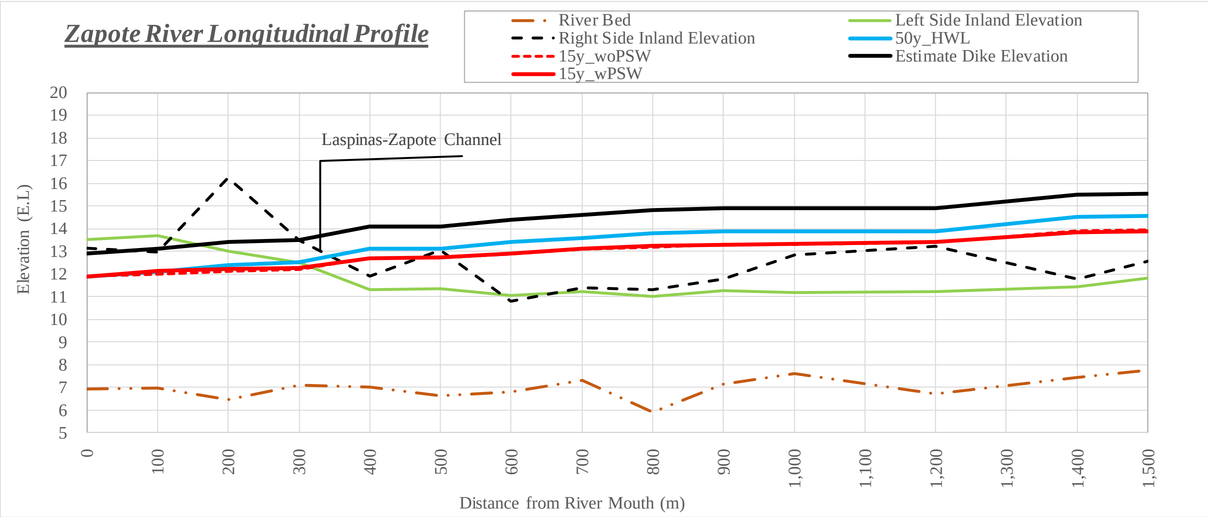


Figure 7.3.14 (3) River Longitudinal Profile of Zapote River (Return Period:15-year)

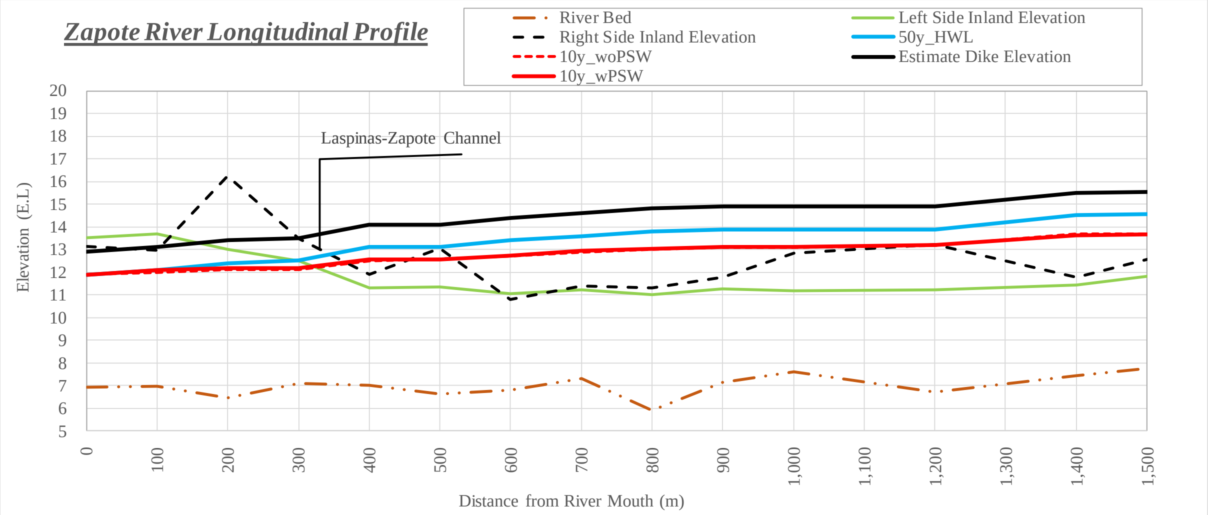


Figure 7.3.14 (4) River Longitudinal Profile of Zapote River (Return Period:10-year)

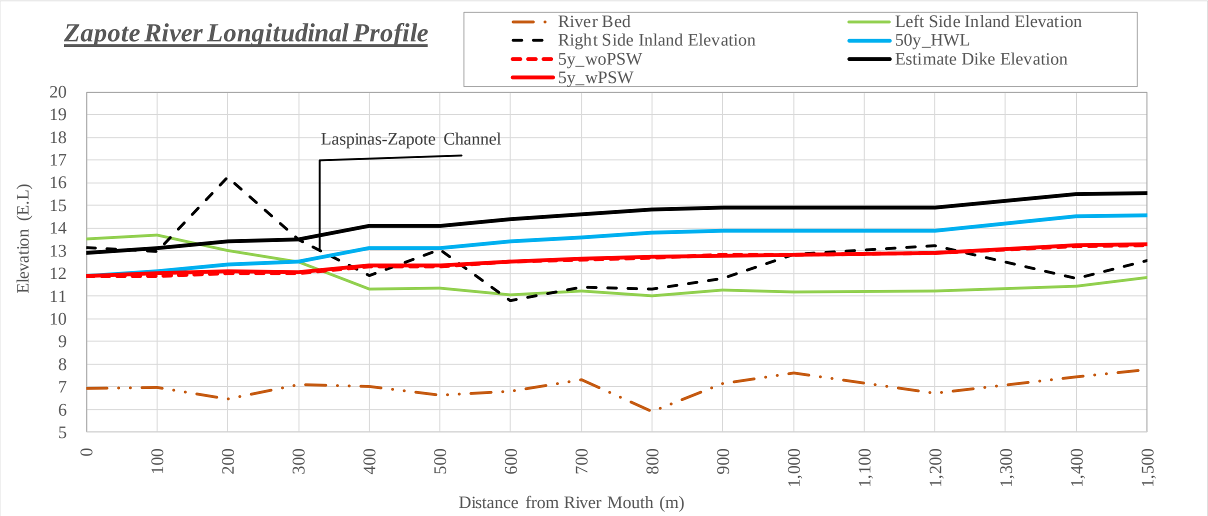


Figure 7.3.14 (5) River Longitudinal Profile of Zapote River (Return Period:5-year)

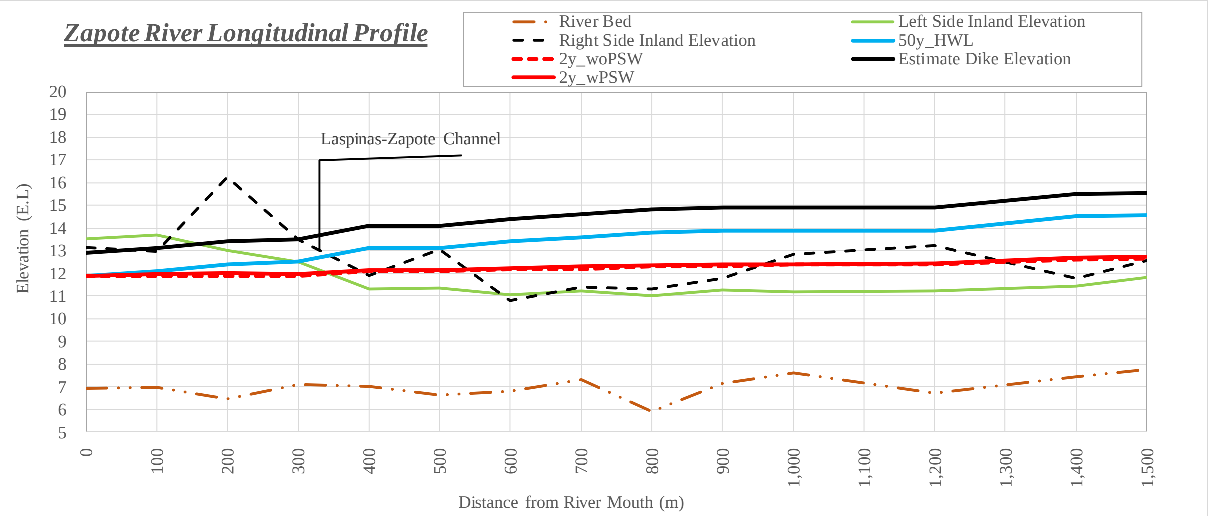


Figure 7.3.14 (6) River Longitudinal Profile of Zapote River (Return Period:2-year)

Table 7.3.4 Specification of River Water Level and River Discharge in Zapote River

No	STA. No.	距離	単距離	現況河道縦断 河床高	左岸河岸高	右岸河岸高	左岸地盤高	右岸地盤高	Simulated Maximum Water Level (m) Return Period 100	50	25	15	10	5	2	Simulated Maximum Discharge (m3/s) Return Period 100	50	25	15	10	5	2
		(m)	(m)	(E.L.m)	(E.L.m)	(E.L.m)	(E.L.m)	(E.L.m)														
1	ZA_0+000	0	0	6.9	13.5	13.0	13.5	13.1	11.9	**11.9**	11.9	11.9	11.9	11.9	11.9	692.1	**598.8**	513.1	453.3	406.9	319.1	216.1
2	ZA_0+100	100	100	7.0	13.7	13.0	13.7	13.0	12.2	**12.1**	12.0	12.0	12.0	11.9	11.9	692.1	**598.8**	513.1	453.3	406.9	319.1	216.1
3	ZA_0+200	200	100	6.4	13.0	13.1	13.0	16.2	12.5	**12.4**	12.2	12.1	12.1	12.0	11.9	692.1	**598.8**	513.1	453.3	406.9	319.1	216.1
4	ZA_0+300	300	100	7.1	12.5	12.7	12.5	13.5	12.8	**12.5**	12.3	12.2	12.1	12.0	11.9	692.1	**598.9**	513.1	453.3	406.9	319.0	216.1
5	ZA_0+400	400	100	7.0	11.3	11.9	11.3	11.9	13.4	**13.1**	12.8	12.7	12.5	12.3	12.1	654.7	**566.9**	483.4	425.1	380.5	298.9	201.3
6	ZA_0+500	500	100	6.6	11.3	13.1	11.3	13.1	13.4	**13.1**	12.9	12.7	12.6	12.3	12.1	654.6	**566.9**	483.4	425.1	380.5	298.9	201.3
7	ZA_0+600	600	100	6.8	11.1	10.5	11.1	10.8	13.6	**13.4**	13.1	12.9	12.7	12.5	12.2	653.9	**566.3**	482.9	424.6	380.1	298.5	201.1
8	ZA_0+700	700	100	7.3	11.2	11.4	11.2	11.4	13.9	**13.6**	13.3	13.1	12.9	12.6	12.2	652.6	**565.2**	481.9	423.8	379.4	297.9	200.6
9	ZA_0+800	800	100	5.9	11.0	11.3	11.0	11.3	14.1	**13.8**	13.5	13.2	13.0	12.7	12.3	651.3	**564.0**	481.0	423.0	378.7	297.1	200.2
10	ZA_0+900	900	100	7.1	11.2	11.8	11.3	11.8	14.2	**13.9**	13.5	13.3	13.1	12.8	12.3	650.0	**562.9**	480.1	422.2	378.0	296.4	199.7
11	ZA_1+000	1000	100	7.6	11.2	12.8	11.2	12.8	14.2	**13.9**	13.6	13.3	13.1	12.8	12.4	648.7	**561.8**	479.1	421.4	377.3	295.7	199.3
12	ZA_1+200	1200	200	6.7	10.0	13.2	11.2	13.2	14.2	**13.9**	13.6	13.4	13.2	12.9	12.4	646.1	**559.6**	477.2	419.8	375.8	294.3	198.4
13	ZA_1+400	1400	200	7.4	11.4	11.8	11.4	11.8	14.9	**14.5**	14.2	13.9	13.7	13.2	12.6	643.5	**557.3**	475.4	418.2	374.4	292.9	197.5
14	ZA_1+600	1600	200	8.1	12.2	13.4	12.2	13.4	15.0	**14.6**	14.2	14.0	13.7	13.3	12.7	640.9	**555.1**	473.5	416.6	373.0	291.5	196.5
15	ZA_1+800	1800	200	7.3	12.4	12.9	12.5	12.9	15.3	**14.9**	14.5	14.3	14.0	13.5	12.9	638.3	**552.9**	471.7	415.0	371.6	290.1	195.6
16	ZA_2+000	2000	200	8.0	12.2	12.4	12.2	12.4	15.6	**15.2**	14.8	14.5	14.3	13.7	13.1	635.8	**550.7**	469.8	413.4	370.2	288.7	194.7
17	ZA_2+200	2200	200	8.9	14.4	13.3	14.4	13.3	15.9	**15.5**	15.1	14.7	14.5	13.9	13.2	633.2	**548.6**	468.0	411.8	368.8	287.7	193.8
18	ZA_2+400	2400	200	8.4	14.0	13.5	14.0	13.5	16.2	**15.8**	15.3	15.0	14.7	14.1	13.3	630.7	**546.4**	466.2	410.2	367.5	286.6	192.8
19	ZA_2+600	2600	200	8.6	14.0	14.0	14.0	14.0	16.3	**15.9**	15.4	15.1	14.8	14.2	13.4	628.2	**544.3**	464.4	408.7	366.1	285.6	191.9
20	ZA_2+800	2800	200	7.3	13.6	14.4	13.6	14.4	16.4	**16.0**	15.5	15.2	14.9	14.3	13.5	625.6	**542.1**	462.6	407.2	364.8	284.6	191.0
21	ZA_3+000	3000	200	8.3	13.5	14.0	13.5	14.0	16.6	**16.2**	15.7	15.3	15.1	14.5	13.6	625.1	**541.6**	462.3	406.9	364.6	284.5	190.7
22	ZA_3+200	3200	200	8.5	14.3	15.1	14.3	15.1	17.0	**16.5**	16.0	15.6	15.3	14.6	13.8	565.3	**489.9**	418.1	368.0	329.7	256.5	171.9
23	ZA_3+400	3400	200	8.4	13.1	14.1	13.1	15.1	16.9	**16.5**	16.0	15.6	15.3	14.7	13.8	562.2	**487.2**	415.8	366.1	328.0	255.2	170.7
24	ZA_3+600	3600	200	8.9	15.2	14.5	15.2	14.7	17.1	**16.6**	16.1	15.7	15.4	14.8	13.9	558.9	**484.4**	413.5	364.1	326.3	253.9	169.4
25	ZA_3+800	3800	200	8.2	13.8	14.4	13.8	14.4	17.2	**16.7**	16.2	15.9	15.6	14.9	14.0	555.8	**481.7**	411.3	362.1	324.6	252.6	168.1
26	ZA_4+000	4000	200	8.5	13.7	13.8	13.7	13.8	17.4	**16.9**	16.3	16.0	15.6	15.0	14.1	552.6	**479.0**	409.0	360.2	322.9	251.3	167.3
27	ZA_4+200	4200	200	8.9	14.6	14.5	14.6	14.5	17.3	**16.8**	16.3	15.9	15.6	15.0	14.1	549.4	**476.3**	406.8	358.2	321.2	250.1	166.6
28	ZA_4+400	4400	200	9.1	14.9	15.7	14.9	15.7	17.5	**17.0**	16.5	16.1	15.8	15.2	14.2	546.2	**473.6**	404.5	356.3	319.5	248.8	165.8
29	ZA_4+600	4600	200	8.3	16.0	16.1	16.0	16.1	17.7	**17.2**	16.7	16.3	16.0	15.3	14.3	543.1	**470.9**	402.2	354.3	317.8	247.5	165.1
30	ZA_4+800	4800	200	8.7	16.1	16.5	16.1	17.1	17.9	**17.4**	16.8	16.4	16.1	15.4	14.4	540.0	**468.3**	400.0	352.5	316.2	246.2	164.4
31	ZA_5+000	5000	200	8.5	16.7	17.2	16.7	17.2	17.9	**17.4**	16.9	16.6	16.2	15.6	14.5	536.9	**465.7**	397.9	350.7	314.6	245.1	163.7
32	ZA_5+200	5200	200	9.4	16.1	18.0	16.1	18.0	18.0	**17.5**	17.0	16.6	16.3	15.6	14.6	533.8	**463.0**	395.7	348.8	313.0	243.8	163.0
33	ZA_5+400	5400	200	10.8	17.7	19.5	17.7	19.5	18.3	**17.7**	17.2	16.8	16.5	15.8	14.7	530.7	**460.4**	393.5	346.9	311.3	242.5	162.3

No	STA. No.	計画高水位−堤内地盤高 左岸	右岸	余裕高	必要堤防天端標高	Simulated Maximum Water Level (m) with Paranaque Spillway_Route.D Return Period 100	50	25	15	10	5	2	Simulated Maximum Discharge (m3/s) with Paranaque Spillway_Route.D Return Period 100	50	25	15	10	5	2
		(m)	(m)	(m)	(m)														
1	ZA_0+000	-1.6	-1.2	1.0	12.9	11.9	11.9	11.9	11.9	11.9	11.9	11.9	827.8	739.8	659.2	602.5	558.4	483.8	383.7
2	ZA_0+100	-1.6	-0.9	1.0	13.1	12.3	12.3	12.2	12.1	12.1	12.0	12.0	**827.8**	**739.8**	**659.2**	**602.5**	**558.4**	**483.8**	**383.7**
3	ZA_0+200	-0.6	-3.8	1.0	13.4	12.6	12.4	12.3	12.2	12.2	12.1	12.0	672.8	580.7	496.4	437.2	391.3	313.9	210.9
4	ZA_0+300	0.0	-1.0	1.0	13.5	12.8	12.6	12.4	12.2	12.2	12.1	12.0	672.8	580.7	496.4	437.2	391.3	313.9	210.9
5	ZA_0+400	1.8	1.2	1.0	14.1	13.4	13.1	12.9	12.7	12.6	12.4	12.1	645.6	557.8	474.3	416.0	371.3	298.9	201.3
6	ZA_0+500	1.8	0.0	1.0	14.1	13.4	13.1	12.9	12.7	12.6	12.4	12.1	645.6	557.8	474.3	416.0	371.3	298.9	201.3
7	ZA_0+600	2.4	2.6	1.0	14.4	13.6	13.4	13.1	12.9	12.8	12.5	12.2	644.9	557.2	473.8	415.6	371.0	298.5	201.1
8	ZA_0+700	2.4	2.2	1.0	14.6	13.9	13.6	13.3	13.1	12.9	12.6	12.3	643.5	556.1	472.9	414.7	370.2	297.9	200.7
9	ZA_0+800	2.8	2.5	1.0	14.8	14.1	13.8	13.5	13.2	13.0	12.7	12.3	642.2	555.0	471.9	413.9	369.5	297.1	200.2
10	ZA_0+900	2.6	2.1	1.0	14.9	14.2	13.9	13.5	13.3	13.1	12.8	12.4	640.9	553.9	471.0	413.1	368.8	296.4	199.7
11	ZA_1+000	2.7	1.1	1.0	14.9	14.2	13.9	13.6	13.3	13.1	12.8	12.4	639.6	552.7	470.1	412.3	368.1	295.7	199.3
12	ZA_1+200	2.7	0.7	1.0	14.9	14.2	13.9	13.6	13.4	13.2	12.9	12.5	637.0	550.5	468.2	410.7	366.7	294.3	198.4
13	ZA_1+400	3.1	2.7	1.0	15.5	14.9	14.5	14.1	13.9	13.6	13.2	12.7	634.4	548.3	466.3	409.1	365.3	292.9	197.5
14	ZA_1+600	2.4	1.2	1.0	15.6	14.9	14.6	14.2	13.9	13.7	13.3	12.8	631.8	546.1	464.5	407.5	363.9	291.5	196.5
15	ZA_1+800	2.5	2.0	1.0	15.9	15.3	14.9	14.5	14.2	14.0	13.6	12.9	629.3	543.9	462.6	405.9	362.5	290.1	195.6
16	ZA_2+000	3.1	2.8	1.0	16.2	15.6	15.2	14.8	14.5	14.2	13.8	13.1	626.7	541.7	460.8	404.3	361.1	288.7	194.7
17	ZA_2+200	1.1	2.2	1.0	16.5	15.9	15.4	15.0	14.7	14.4	13.9	13.2	624.2	539.5	459.0	402.7	359.7	287.7	193.8
18	ZA_2+400	1.8	2.3	1.0	16.8	16.2	15.7	15.3	14.9	14.6	14.1	13.4	621.6	537.4	457.1	401.2	358.3	286.6	192.8
19	ZA_2+600	1.9	1.9	1.0	16.9	16.3	15.8	15.4	15.0	14.7	14.2	13.4	619.1	535.2	455.4	399.6	357.0	285.6	191.9
20	ZA_2+800	2.4	1.6	1.0	17.0	16.4	15.9	15.5	15.1	14.8	14.3	13.5	616.6	533.1	453.6	398.1	355.7	284.6	191.0
21	ZA_3+000	2.7	2.2	1.0	17.2	16.6	16.1	15.6	15.3	15.0	14.5	13.6	616.0	532.6	453.2	397.8	355.5	284.5	190.7
22	ZA_3+200	2.2	1.4	0.8	17.3	16.9	16.4	15.9	15.5	15.2	14.7	13.8	556.3	480.8	409.1	358.9	320.6	256.5	172.0
23	ZA_3+400	3.4	1.4	0.8	17.3	16.9	16.4	15.9	15.5	15.2	14.7	13.8	553.1	478.1	406.8	357.0	319.0	255.2	170.7
24	ZA_3+600	1.5	1.9	0.8	17.4	17.1	16.6	16.0	15.7	15.4	14.8	13.9	549.9	475.4	404.5	355.0	317.2	253.9	169.4
25	ZA_3+800	2.9	2.3	0.8	17.5	17.2	16.7	16.2	15.8	15.5	14.9	14.0	546.7	472.7	402.2	353.1	315.5	252.6	168.1
26	ZA_4+000	3.2	3.1	0.8	17.7	17.3	16.8	16.3	15.9	15.6	15.0	14.1	543.6	470.0	400.0	351.1	313.8	251.3	167.3
27	ZA_4+200	2.3	2.3	0.8	17.6	17.2	16.8	16.3	15.9	15.6	15.0	14.1	540.4	467.3	397.7	349.2	312.2	250.1	166.6
28	ZA_4+400	2.1	1.4	0.8	17.8	17.4	16.9	16.4	16.1	15.7	15.2	14.2	537.2	464.5	395.4	347.2	310.4	248.8	165.8
29	ZA_4+600	1.2	1.1	0.8	18.0	17.6	17.1	16.6	16.2	15.9	15.3	14.3	534.1	461.9	393.2	345.3	308.7	247.5	165.1
30	ZA_4+800	1.3	0.3	0.8	18.2	17.8	17.3	16.8	16.4	16.1	15.4	14.4	531.0	459.2	391.0	343.5	307.1	246.2	164.4
31	ZA_5+000	0.7	0.3	0.8	18.2	17.9	17.4	16.9	16.5	16.2	15.6	14.5	527.9	456.7	388.9	341.7	305.6	245.1	163.7
32	ZA_5+200	1.4	-0.5	0.8	18.3	17.9	17.4	16.9	16.5	16.2	15.6	14.6	524.8	454.0	386.7	339.8	304.0	243.8	163.0
33	ZA_5+400	0.0	-1.8	0.8	18.5	18.2	17.7	17.2	16.8	16.4	15.8	14.7	521.7	451.4	384.5	337.9	302.3	242.5	162.3

: The section where the water level rose due to drainage of Parañaque Spillway

7.3.3 Evaluation on the Influence on Downstream River Channel (Summary)

Effects on downstream river channel due to drainage of Parañaque Spillway can be summarized from the above-mentioned estimation result of downstream river water level rise.

Route	Outlet Facility	Effect to Downstream River Channel
Route-1	South Parañaque R. Location : SP.1+800	● If the plan scale exceeds 10 years, the water level at South Parañaque River, which is the drainage destination, exceeds the Laguna Lake Design Lake Level (14.0 m), so there is time that can not be drainage from the Parañaque Spillway. ● The design scale of South Parañaque is 25-year return period. The water level of South Parañaque will not exceed HWL by drainage of Parañaque Spillway under river improvement commensurate with the planned scale becomes possible. ● However, since the section where the water level rises greatly affects upstream and downstream due to drainage of Parañaque Spillway, the river improvement plan is imperative including not only South Parañaque River but also the upstream branch. ● The South Parañaque river is densely populated on both banks, so it is considered difficult to implement river improvement due to river channel widening, so improvement of embankment is necessary.
Route-3	Zapote R. Location : ZA.0+100	● The location of outlet facility is assumed to be located 100 m from the estuary. The Zapote river water level will not exceed Design Lake Level of Laguna de Bay during flooding. So, there is no duration that can not be drainage from Parañaque Spillway. ● The section where water level rises due to drainage of Parañaque Spillway can be assumed to be around 500 m from the estuary. ● When the probable scale is 25 years or more, there is a section that is over 50 HWL (maximum 20 cm) due to drainage of Parañaque Spillway but lower than the assumed embankment top height (HWL +freeboard). ● Zapote River is connected to Las Piñas River near the mouth of the estuary via a channel and Las Piñas River is also connected to South Parañaque River by San Dionisio River, so the influence of drainage of Parañaque Spillway is not only for Zapote River it also affects the surrounding rivers / channels.

The design scale was set for each river in this project because the flood control plan was not formulated in Las Piñas-Parañaque area. The flood mitigation measures were based on embankment plan and the assumed embankment height was calculated. In addition, river water level rising was evaluated based on river cross section after river improvement.

In future investigation, it is necessary to evaluate the influence of drainage from Parañaque Spillway under flood control plan (design discharge, design cross section, design embankment height and so on) is formulated.

The influence of downstream river was evaluated using by cross section survey results and 5m grid elevation data (IFSAR) in this project. When the study to formulate flood control plan, cross section survey for all rivers and topographic survey is necessary to conduct basic data, to reappear complex river network system in this area and to improve analysis model.

7.4 Non-structural Measures for Parañaque Spillway

The following three items can be considered as non-structural measures for Parañaque Spillway:

1) Water level observation related to the operation of Parañaque Spillway
2) Warning related to the operation of Parañaque Spillway
3) Development of information network related to the operation of Parañaque Spillway

7.4.1 Water Level Observation at the Inlet and Outlet Sides of Parañaque Spillway

If the operation of Parañaque Spillway is conducted based on the water level of Laguna de Bay around the inlet and outlet portions of the spillway (Manila de Bay or rivers), radio telemetric water level gauges (float type or pressure type) are proposed to be installed around these inlet and outlet portions.

7.4.2 Warning System around the Inlet and Outlet Sides of Parañaque Spillway

It is necessary to announce the start of operation through sirens warning the people around the inlet and outlet of the Parañaque Spillway. At the same time, it is necessary to conduct patrols around the inlet and outlet. Information about the operation shall be provided to the City Disaster Risk Reduction and Management Offices (CDRRMOs) responsible for the areas around the inlet and outlet, and it is also necessary to ask cooperation and assistance from the CDRRMOs in alerting and warning the people not to go or roam around the inlet and outlet portions of the Spillway in flood time and during the operation.

In this connection, the Study Team interviewed the CDRRMOs concerned in Route 3 (Inlet side: Muntinlupa City; Outlet side: Las Piñas City and Bacoor City) and Route 1 (Inlet side: Taguig City) and these three CDRRMOs agreed to cooperate in warning and alerting the people around the inlet and outlet of the spillway upon receipt of the operation information from the Spillway management office. Unfortunately, the Study Team was not able to obtain agreement of the Parañaque CDRRMO during the survey due to some other emergency meeting of the Parañaque City side.

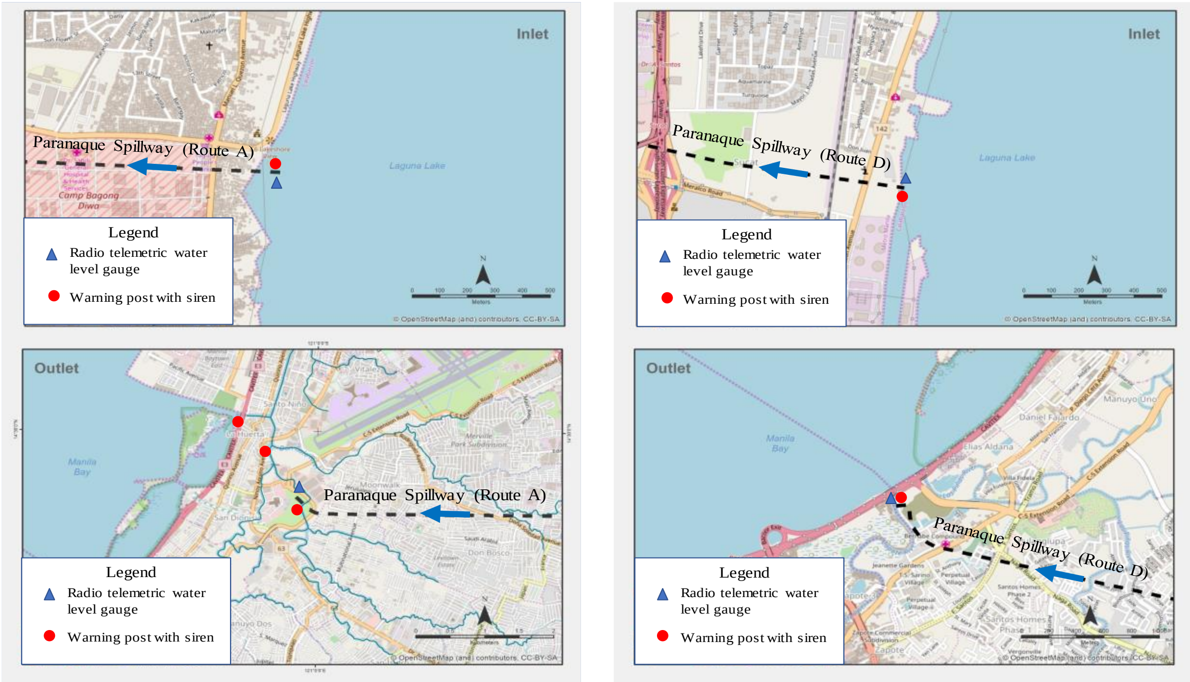
Figure 7.4.1 show the proposed locations of radio telemetric water level gauges and warning posts for alternative Routes 1 and 3 of Parañaque Spillway.

7.4.3 Development of Information Network related to the Operation of Parañaque Spillway

Parañaque Spillway can be positioned as one of the facilities for flood control in the Pasig-Marikina-Laguna River Basin. Therefore, it is proposed that the management of Parañaque Spillway shall be conducted as an extension of that of EFCOS. As for the management office of Parañaque Spillway, a Master Control Station (MCS), which will be the main station (main operating office) of the Spillway, will be installed at the inlet side and a Sub-station (sub-operating office) will be installed at the outlet side of the Spillway. These stations and those of EFCOS, PAGASA, DPWH Head Office and DPWH NCR will be connected by the information network and the warning posts at the inlet and outlet sides will be operated from the MCS for announcing warning to the people around the inlet and outlet sides about the inflow and outflow water from the Spillway. Furthermore, information about the operation of the Spillway will be provided to the

CDRRMOs responsible in the areas around the inlet and outlet sides. Their cooperation also will be requested in warning people not to go nearby the inlet and outlet portions as well as the downstream stretches of rivers.

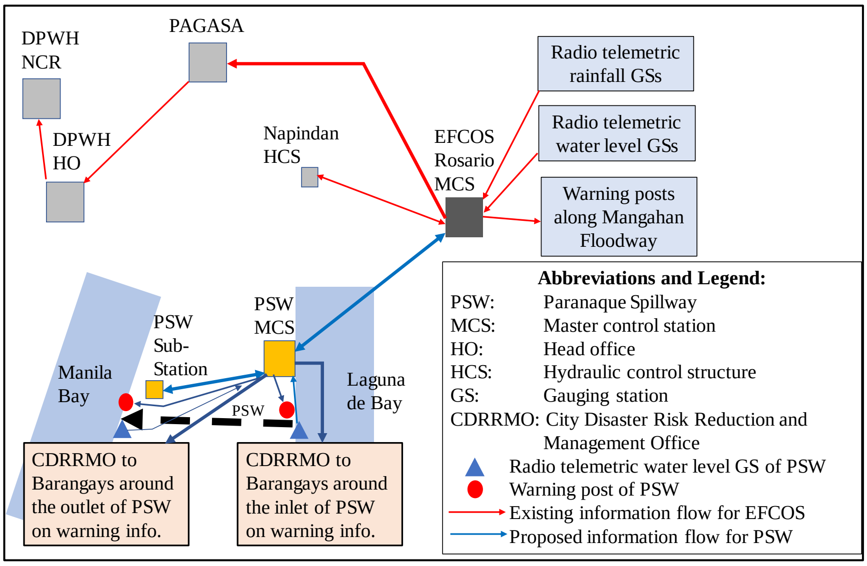
Figure 7.4.2 show images of the information network related to Parañaque Spillway. In the next stage of the Parañaque Spillway Project, necessary devices of the sending and receiving sides of information, as well as connection including frequency, etc., are necessary to be studied.



Route 1 Route 3

Source: JICA Survey Team

Figure 7.4.1 Proposed Telemetric Water Level Gauges and Warning Posts for Parañaque Spillway (Route 1 and 3)



Source: JICA Survey Team

Figure 7.4.2 Proposed Information Network for Parañaque Spillway

7.5 Procurement and Construction Plan

7.5.1 Construction Plan

(1) Study on Tunnel Construction Method

(a) Conditions

Plan A and Plan D, chosen as plans for the route, were studied based on the following conditions.

Table 7.5.1 Conditions of Study for Plan A and Plan D

Item	Condition
Soil	The results of survey conducted at 6 different locations show that there is a tuff layer at 50m in depth where the tunnel is planned. Further and more surveys are necessary for evaluation, but this study assumes that overburden of the tuff layer is sufficient for the tunnel throughout the whole route.
Underground Water Level	The result of the survey shows GL-3m~GL-5m and taking the safe side, this study applies GL.
Tunnel Inner space	Diameter of inner space for circular cross-section is 12m (Approx. 113m^2)
Tunnel Length	Plan A: 6.0km; Plan D: 8.8km
Tunnel Overburden	There is no limitation against overburden under the road area, but 50m or more of overburden should be secured without concerning the surface right, so that the route between Laguna de Bay and Manila Bay can be planned freely.
Middle Manholes	Not considered in this study.

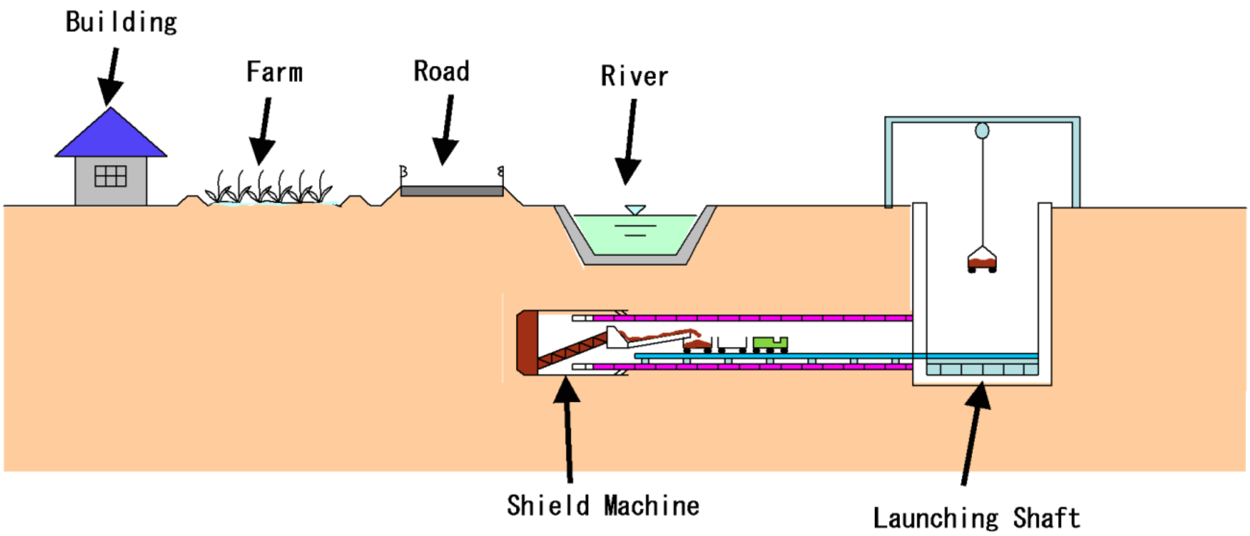
Source: JICA Survey Team

(b) Construction Method

There are four categories of Tunnel Construction Method, namely; Shield Tunneling Method, New Austrian Tunnelling Method (NATM), Cut and Cover Method and Immersed Tube Tunneling Method. The Immersed Tube Tunnelling Method is usually applied when constructing a tunnel at the bottom of rivers, lakes and sea and this is not considered in this study. The Cut and Cover Method is not considered as well since this plan involves the construction of a tunnel under residential areas throughout the whole route and it is not realistic to acquire the land by resettlement. Hence, only the Shield Tunneling Method and the NATM, which are non-open cut methods, are studied.

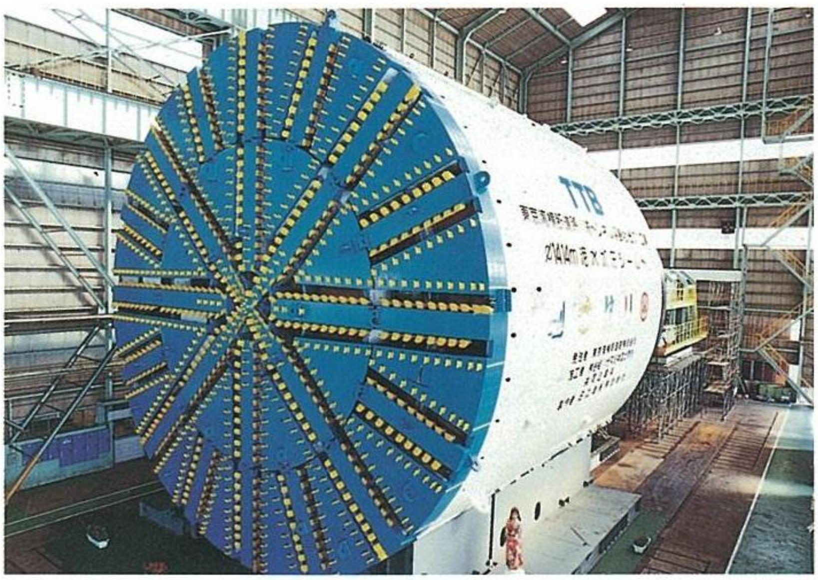
(i) Shield Tunneling Method

The Shield Tunnelling Method is the method of constructing a tunnel by excavating under the ground from a launching shaft to an arrival shaft with a cylindrical machine called shield machine. Excavation and construction of tunnel structure are continuously conducted while cutting through the ground with cutter bits that are attached in front of the machine and assembling segments (exterior wall of tunnel) inside of the machine. This method can be executed without occupying the land aboveground between the launching shaft and the arrival shaft and can be safely applied to soft and hard ground regardless of underground water.



Source: Introduction of Tunnel Construction Method from Japan Water Agency
(https://www.water.go.jp/chubu/kisodo/PDF/d.dousuiro-pdf/tonnerusekouhouhou.pdf)

Figure 7.5.1 Schematic Drawing of Shield Tunneling Method

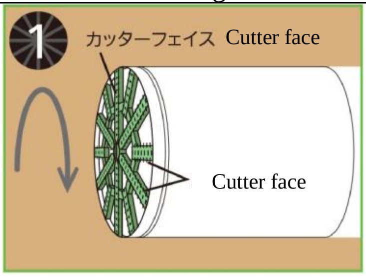
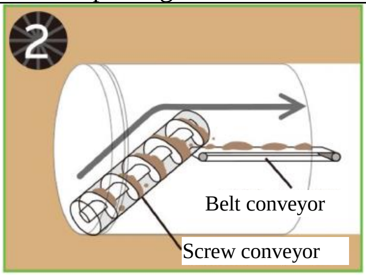
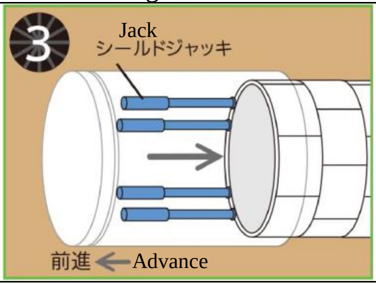
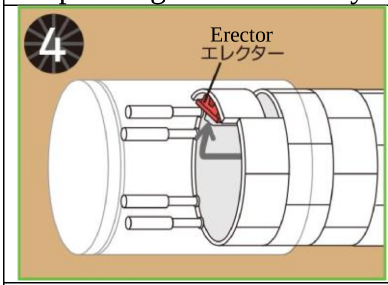


Source: Picture of Road Tunnel from Highway Technology Research Center

Figure 7.5.2 Photo of Shield Machine for the Shield Tunneling Method

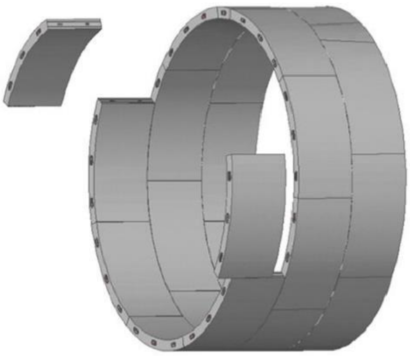
A tunnel is constructed with the Shield Tunnelling Method by repeating the steps shown in Table 7.5.2. Procedures 1 to 3 in Step 1 are simultaneously executed.

Table 7.5.2 Procedure of Shield Tunnelling Method

Step-1: Excavation		
Cutting	Transporting excavated soil	Advancing Shield Machine
		
Revolve the cutter face to cut the ground	Load excavated soil into the machine by a screw conveyor and transport it to the outside	Apply reaction force with jacks installed in the machine against segments and advance the machine accommodating the speed of excavation
Step-2: Segment Assembly		
		
Secure the space to assemble segments by shortening jacks situated at the element to assemble and insert the segment using an erector.		

Source: North Line HP from Metropolitan Expressway Company Limited
(http://www.shutoko.jp/ss/kitasen/yokokan/construction/work02.html)

The tunnel to be constructed for this Project will be a spillway to discharge water from Laguna de Bay to Manila Bay and the environment inside of the tunnel will be corrosive. Therefore, the JICA Survey Team proposes one-pass joint so that steel joint box will not be exposed on the surface of the segment and the inner surface of the tunnel will be smooth as shown in Figure 7.5.3.

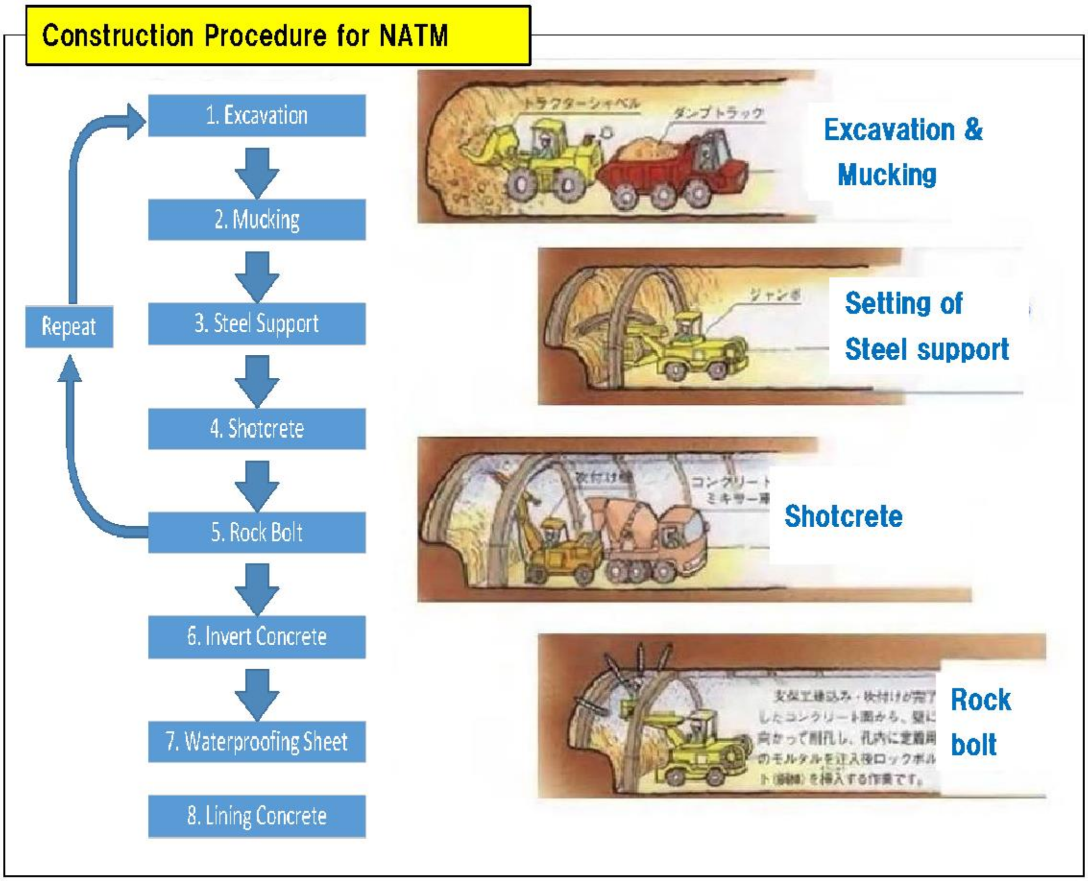


Source: North Line HP from Metropolitan Expressway Company Limited
(http://www.shutoko.jp/ss/kitasen/yokokan/construction/work02.html)

Figure 7.5.3 Image of Segment Assembly

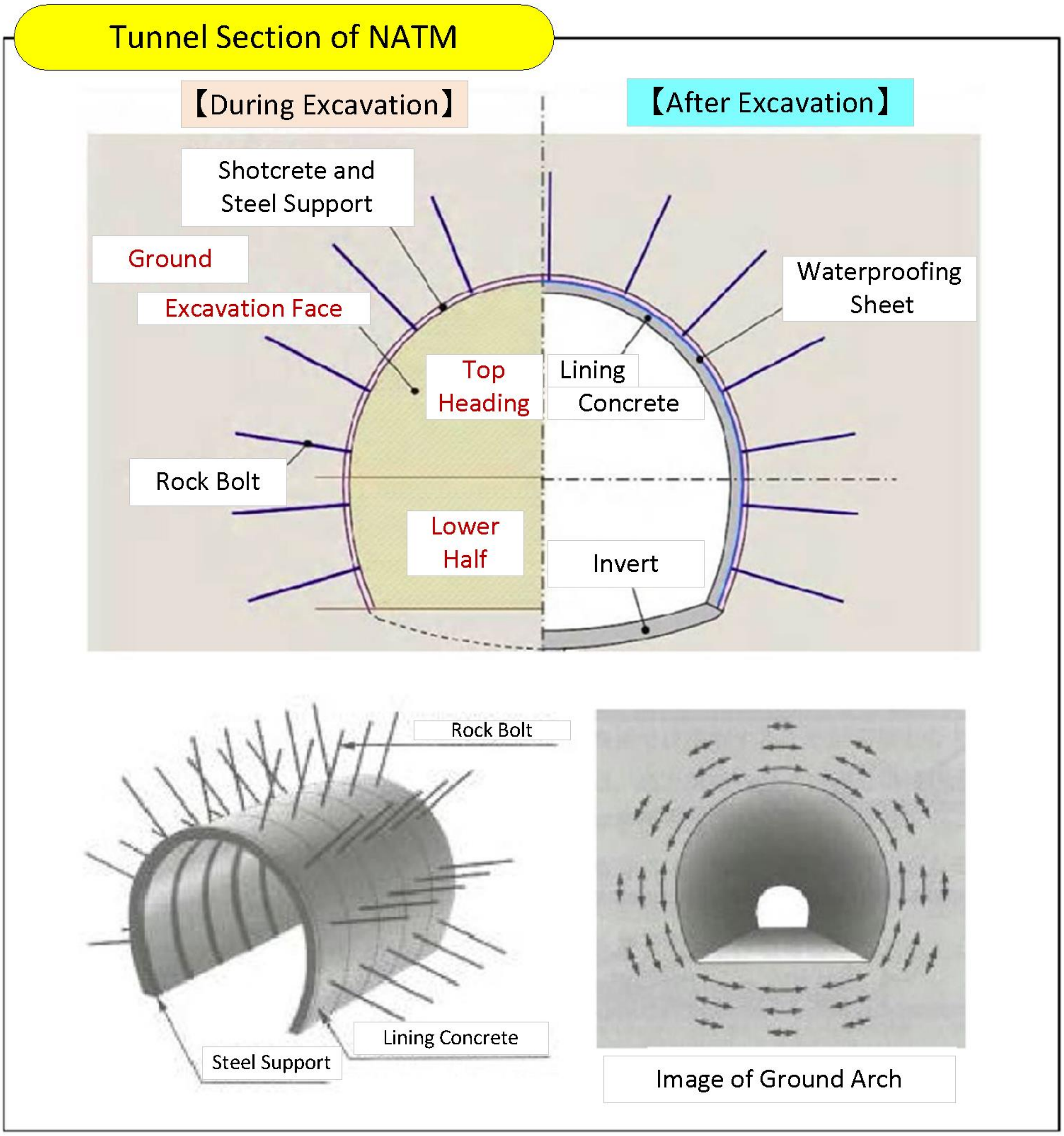
(ii) New Austrian Tunneling Method (NATM)

The New Austrian Tunneling Method (herein called as NATM) utilizes the retention force (ground arch) of the surrounding ground to support the tunnel and hence the excavation proceeds by applying shotcrete as soon as possible, positioning steel support, arranging rock bolts, and securing stability of the ground. Equipment used for the construction is general-purpose heavy equipment such as excavators to excavate the face and dump trucks to transport surplus soil. Compared to the Shield Tunneling Method which uses expensive shield machines, the cost of this method is generally much cheaper. However, in case of leakage in the ground, in order to stop the water, auxiliary measures such as chemical grouting is necessary, but the cost of this method will be relatively expensive. Therefore, when selecting NATM, boring survey should be thoroughly conducted to understand the details of the ground. The ground in which NATM can be applied is hard ground and ground stronger than soft rock, and the face has to be stable.



Source: Design Practice to Understand the Concept Well 7 from Tunnel Designing

Figure 7.5.4 Procedure of NATM



Source: Design Practice to Understand the Concept Well 7 from Tunnel Designing

Figure 7.5.5 Excavation Cross-Section of NATM

(iii) Selection of Construction Method

Table 7.5.3 Summary of Construction Method

Item	Shield Tunneling	NATM
Photo		
Summary of Method	Excavate with the shield machine by stabilizing the face against earth pressure and water pressure using mud pressure or muddy water pressure and assemble segments to retain the ground.	Excavate by stabilizing the ground using shotcrete, rock bolts and steel support, etc., utilizing the support function of surrounding ground. If formation of a ground arch with the surrounding ground and stability of the face when excavating is not ensured, auxiliary measures are necessary.
Ground to apply (Standard application and adaptability to changes such as the ground condition)	Generally applied to the ground with very soft alluvium, diluvium and soft rocks from Neogene period. Relatively easy to adopt to changes of the ground. There are also many cases with hard rocks.	Generally applied to the ground with hard rocks and soft rocks from Neogene period. Depending on the requirements, applicable to unconsolidated ground. The changes of ground can be handled by changing support works, excavation methods, and auxiliary measures.
Advantage	➢ Construction is possible without auxiliary measures, except at the launching and arrival area, by using a closed-face type shield even though leaked water is detected in the ground. ➢ Standard monthly excavation distance is 350m but 500m can be possible if the machine is built for high speed excavation. Compared to NATM, construction speed is much higher.	➢ If auxiliary measures are not necessary; the cost of construction is about half of the cost of Shield Tunnelling method.
Disadvantage	➢ Shield machine and segments are expensive and the total cost for construction can be much higher than that of NATM.	➢ In case of unexpected soft ground or leaked water, auxiliary measures are necessary so that the total cost for construction can be much higher than that of Shield Tunnelling Method. ➢ Monthly excavation distance is about 80m and much slower than that of Shield Tunnelling Method.

Source: JICA Survey Team

At this point in time, since there are no survey results regarding the ground condition at the tunnel excavation point to confirm the presence of leaked water, the JICA Survey Team considers that the Shield Tunneling Method is the better option as it can be applied to both soft and hard ground and can cope with leaked water. Whether NATM is applied or not is judged depending on the results of boring survey thoroughly conducted in future.

Requirements for application of NATM is, that the ground is not soft and leaked water is not detected, since auxiliary measures will not be necessary for the excavation site. In addition, monthly excavation distance of 80m is acceptable.

(2) Study on Construction Method for Vertical Shaft

(a) Requirements for Planning

Requirements for Planning are as follows.

Table 7.5.4 Summary of Construction Method

Condition	Side	Summary
Ground Condition	Manila Bay Side	Silty sand of N-value of 10 is confirmed around GL~GL9m based on BH-01 and BH-04 and Tuff rock is confirmed at deeper than GL9m to GL-70m based on boring survey. This study is conducted based on the result of BH-04.
	Laguna de Bay Side	Silty sand of N-value of 16~28 is confirmed at GL~GL20m in BH-02, which is the nearest from Laguna de Bay, clay of N-value of 22~66 is confirmed at GL-44m~GL62m. However, although 1.65m of weak stratum is confirmed on the surface of the ground at BH-03 which is slightly inland, Tuff rock is confirmed at deeper than GL-62. It is similar at BH-05 located inland and 3.8m of clay is found in the surface layer but Tuff rock is found at deeper than 3.8m. Since a shaft is planned to be constructed inland to avoid Marikina Fault, the study is conducted based on the result of BH-05.
Underground Water Level	Manila Bay Side	It was at GL-3.6m for BH-01 and GL-3.47m for BH-04. Therefore, this study refers to GL-03.
	Laguna de Bay Side	The result of BH-02 is GL-1.42m due to extremely low altitude of the borehole located near Laguna de Bay. On the other hand, the result of BH-03 is GL-8.7m due to extremely high altitude of the borehole. When considering the surrounding area of this borehole, this data is not taken into consideration since it is singular value. It is GL-3.8m at BH-05 and GL-3.4m at BH-06 and by arranging location of the shaft slightly inland of Laguna de Bay, the study will be done for GL-03.
Planned Shafts		2 shafts for Inlet (Slightly inland on Laguna de Bay side) and Outlet (On Manila Bay side）

Source: JICA Survey Team

(b) Study on Scale of Vertical Shaft

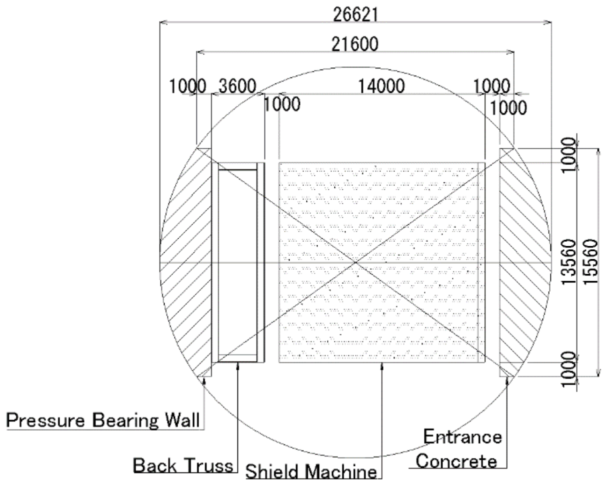
Vertical shafts are used for the launching and arrival of shield machines when constructing, pumping in and out water during the service period, so that the scale of shafts should fulfil these functions. This Spillway is planned to be the siphon type. The planned volume of flow is a dominant factor when determining the scale of the tunnel and the effect on the scale of cross-section for vertical shaft is small. Therefore, the scale necessary for the construction will be the basis of setting the scale of vertical shaft.

(i) Plane Scale

The Shield Tunnelling Method and the NATM Method have different shaft sizes which should be decided.

a) Shaft for Shield Launching

The inner plane scale is decided from the launching space for Shield Machine. As a result, in order to fulfil the 21.6m x 16.56m indicated in Figure 7.5.6, the diameter of inner plane for vertical shaft is 30m in this study. Reasons for determination are as shown in Table 7.5.5.



Source: JICA Survey Team

Figure 7.5.6 Minimum Space Necessary for Launching Shaft of Shield Tunnelling Method

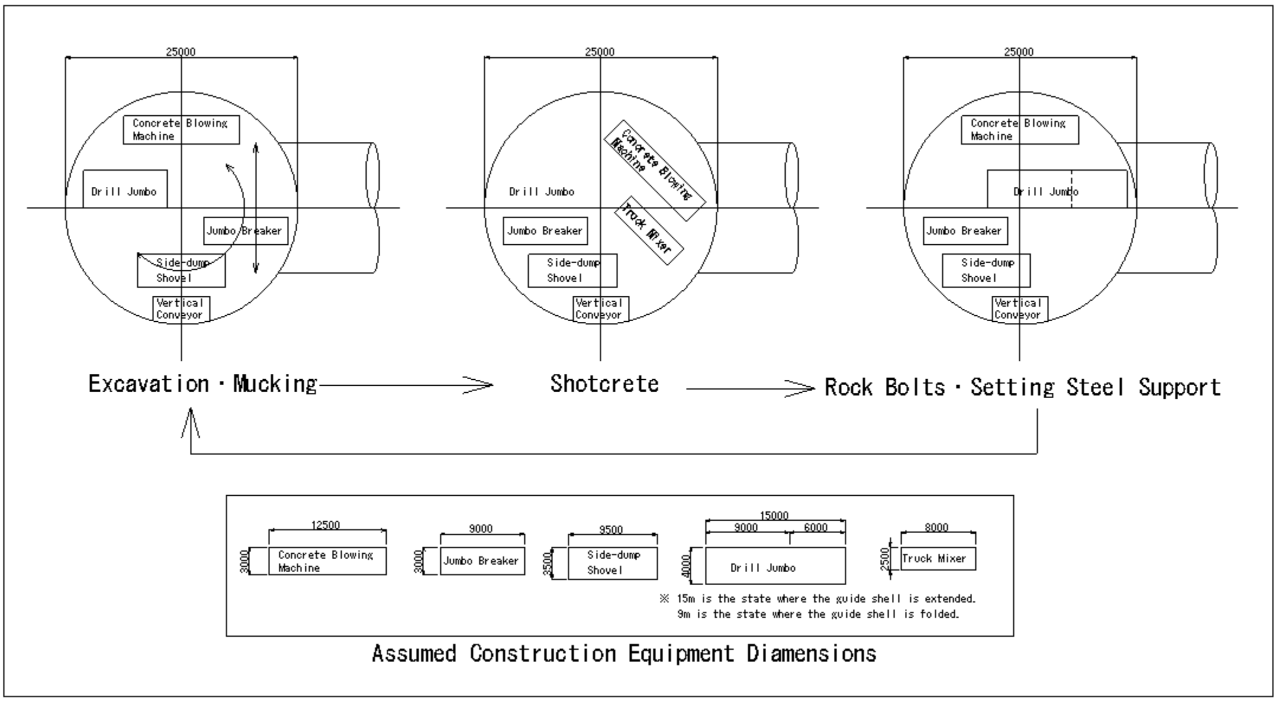
Table 7.5.5 Reasons for Determination of Launching Shaft of Shield Tunnelling Method

Items	Reasons
Launching Portal	Thickness is 1 m due to the scale of shield machine. Width is 15.56 m including 1m of entrance packing space on both sides which is placed outside of the machine.
Distance between launching portal and Shield Machine	Distance is 1 m for the space of Breaking of Tunnel Entrance.
Length of Shield Machine	Length is 14 m due to the previous cases of similar construction in the past.
Outer diameter of Shield Machine	Outer diameter is 13.56 m considering segment outer diameter of 13.2 m and 180 mm of space on both sides of the ring, assuming that it is excavated in hard ground and pipes for simultaneous backfill grouting is set inside tail.
Distance between Shield Machine and temporary support	Thickness is 1 m due to the scale of shield machine.
Width of temporary support	Length of segment is 1.6m, allowance for lifting segments is 500mm in front and back, and H – 500 x 500 of steel temporary support is 3.6 m.
Bearing wall	Thickness is 1 m due to the scale of shield machine.
Work space for shield machine installation	Clearance is 1 m each due to the scale of shield machine.

Source: JICA Survey Team

b) Shaft for NATM Method

Since this planning tunnel is presupposed to secure overburden of 50 m or more, the construction will be carried out of the tunnel excavation by hanging down the construction equipment into the shaft about GL-70 m from ground level. The NATM separate the construction steps for “Excavation and Mucking”, “Shotcrete” and “Rock bolts and Setting of steel support” and each step is carried out by different construction equipment. Therefore, tunnel construction is carried out by replacing the construction equipment at each step. However, considering the depth of shaft, there is the problem of cycle time and safety. For this reason, the plane scale for shaft is decided at φ25 m from minimum space of arrangement for the construction equipment for first excavation and the rearrangement. The basis for setting up the shaft of the NATM is shown in Figure 7.5.7.



Source: JICA Survey Team

Figure 7.5.7 Minimum Size for NATM Shaft

(ii) Tunneling Method and Depth of Vertical Shaft

This project requires the overburden of the tunnel to be 50 m or more, and large uplifting pressure will apply to the bottom slab since the vertical shaft will be located in great depth. Internal construction of vertical shaft does not happen for a long period of time since the shaft is used for transporting tunnel equipment and excavated soil. Therefore, it is necessary to secure weight of frame sufficient for safety against uplifting in this situation.

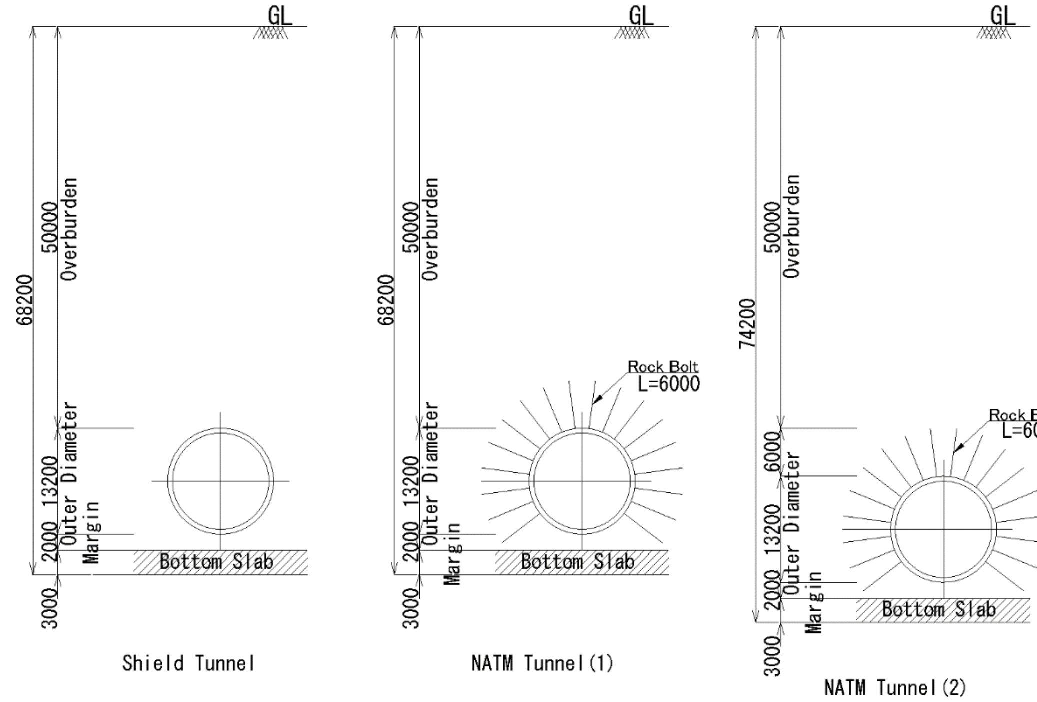
Depth of vertical shaft is a very important element for the structure of the shaft since the scale of uplifting increases in accordance with depth of vertical shaft. Depth of vertical shaft varies depending on tunneling methods and concept of tunnel overburden and it is shown in Table 7.5.6 and Figure 7.5.8. The length of rock bolts for NATM uses 4.0 m ~ 6.0 m of standard cross-section for 2-lane road tunnel as a reference and the longest length of 6.0 m is applied.

Whether or not to consider rock bolts as a part of the structure will be discussed and clarified later.

Table 7.5.6 Details of Vertical Shaft Depth for Each Tunneling Method

	Shield Tunnelling	NATM (1) When rock bolts are not a part of tunnel structure	NATM (2) When rock bolts are a part of tunnel structure
Overburden	50 m (Common)	50 m (Common)	50 m (Common)
Tunnel Outer Diameter (height)	13.2 m	12.4 m	12.4 + 6.0 = 18.4 m
Height of Allowance	5.0 m (Common)	5.0 m (Common)	5.0 m (Common)
Thickness of Bottom Slab	5.0 m (Common)	5.0 m (Common)	5.0 m (Common)
Total Depth	73.2 m	72.4 m	78.4 m

Source: JICA Survey Team



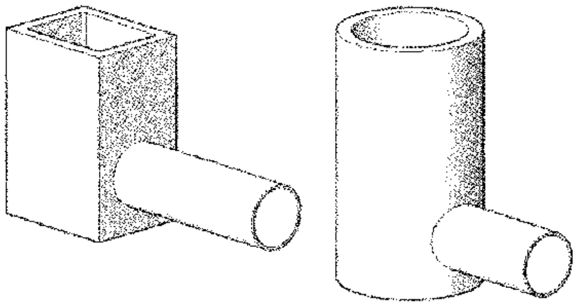
Source: JICA Survey Team

Figure 7.5.8 Depth Plans of Bottom Slab

(c) Shape of Vertical Shaft

Currently, vertical shafts for pumping in and out are planned to be constructed on both sides of Manila Bay and Laguna de Bay and during construction, these shafts will be used for launching and arrival shafts of shield machines. The shape of these shafts is discussed below.

The vertical shaft is generally classified into two shapes: rectangular and circular. Based on the requirement of vertical shafts for the project, which is vertical shafts for launching and arrival of shield machine in great depth, properties of both rectangular and circular shafts are compared. Based on the results, the shape of vertical shaft is planned to be circular which is confirmed to be more beneficial. Results of comparison between rectangular shape and circular shape are shown in Table 7.5.7.



Rectangular shape shaft Circular shape shaft

Source: Japan Society of Civil Engineers: Design of Vertical Shaft for Shield Construction

Figure 7.5.9 Image of Rectangular and Circular Shafts

Table 7.5.7 Comparison of Plane Shapes of Vertical Shaft

Property and Shape	Rectangular	Circular
Planar arrangement	In case of launching shaft, this is more beneficial since there is no useless space. ◎	In case of launching shaft, there will be extra space on the side. However, the shaft planned in this project might require an elevator and the extra space can be used for that. ○
Flexibility of tunnel installation	Generally, there is restriction on 4 directions. △	Tunnel can be installed in any direction. Restriction on plan for inlet and discharge sewers can be reduced. ◎
Coping of structure against depth	In great depth, cross-section force applied on the side walls due to lateral pressure and it is necessary to thicken wall thickness or to arrange beams, center walls and middle plates finely. △	When lateral pressure is large since the vertical shaft is located in depth, this is beneficial in terms of structure as axial force excels (thickness of wall is thin). In addition, center walls and middle plates for structure will not be necessary. ◎
Space during construction	Earth retaining works such as brace or stile beams are necessary and there is a restriction at opening. △	Brace and others are not necessary and workability of tunnel shaft is good. ◎
Evaluation	△	◎

Legend; ◎ Very Good、○ Good/Possible、△ Not Good/Some Problems

Source: JICA Survey Team

(d) Construction Method

Diaphragm wall method and Caisson method are currently the only two methods that can cope with 30 m of excavation cross-section and 70 m or more excavation depth.

Diaphragm wall method is categorized into RC diaphragm wall method and Steel diaphragm wall method. Steel diaphragm wall method is usually applied in constructions under the condition in which lifting of heavy load, such as reinforcement cages, is difficult in overhead space, and under the special condition in which thickness of the wall needs to be thin due to the restriction of land acquisition. Generally, the cost of steel diaphragm wall is higher than that of RC diaphragm wall. This study applies the RC diaphragm wall since these restrictions do not exist.

Caisson method is classified into open-caisson method and pneumatic caisson method. In grounds with high underground water level and high permeability, pneumatic caisson method allows excavation in a dry environment by pumping compressed air, which is compatible with underground water pressure, into the work space located at lower part of caisson to prevent penetration of underground water. For this method, equipment such as air supply equipment, man lock, material lock, and ground remote control system is necessary, and excavation is remotely controlled and conducted by mechanical drilling since excavation is executed under high pressure. As a consequence, special equipment and operation are required for this method and the cost of the excavation is expensive. Therefore, the JICA Survey Team has decided that the open-caisson method is more suitable for this project since it is possible to prevent leakage, which would be an obstacle to the construction, with bulkhead using steel sheet piles, even though the ground to be excavated contains a water-bearing layer on the upper part. For this reason and the economical perspective, the open-caisson method is to be applied in this project.

(i) RC Diaphragm Wall Method

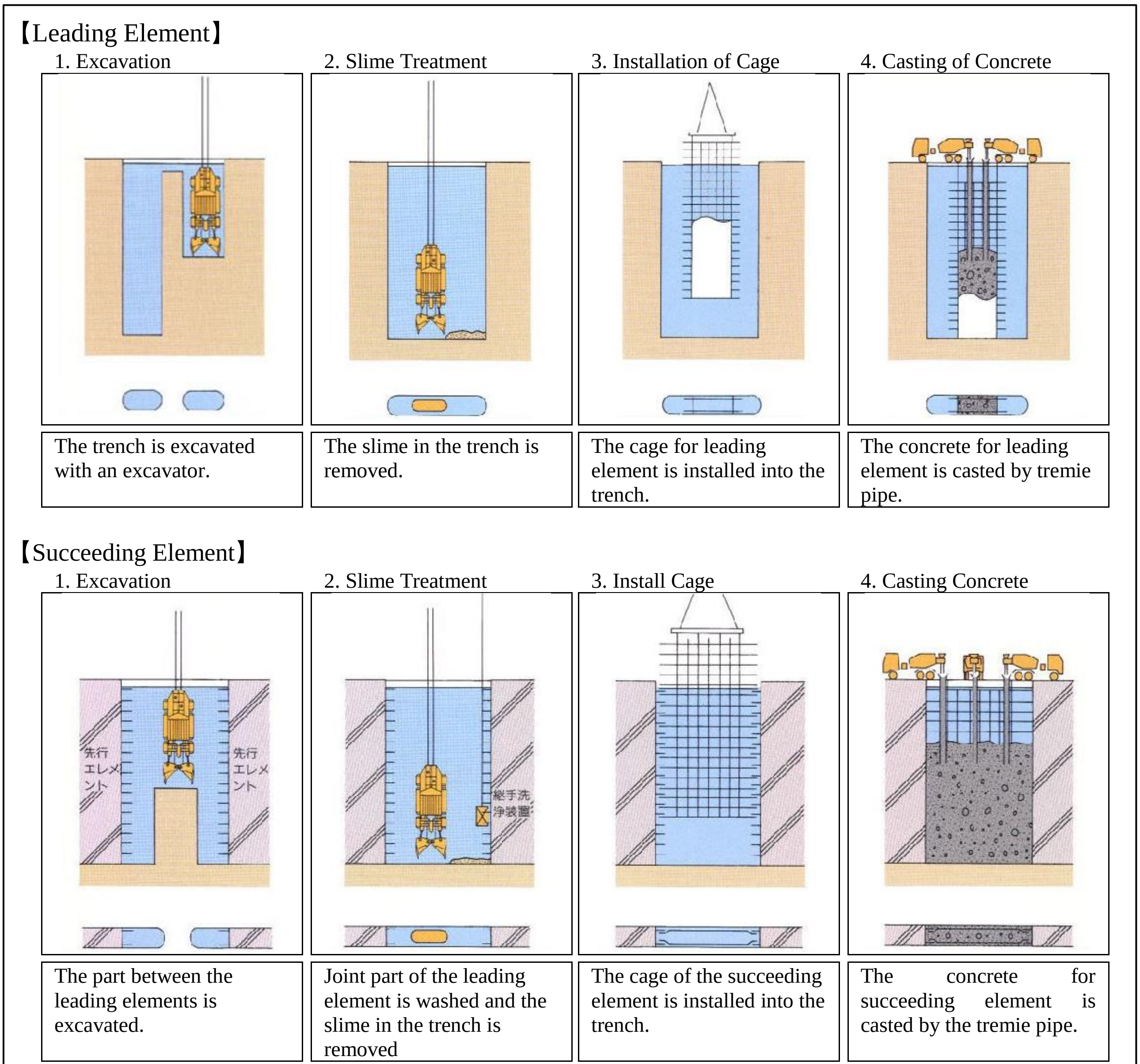
The RC diaphragm wall is constructed by inserting reinforcement cages into drilling grooves excavated using slurry and replacing slurry with concrete to cast continuous RC wall. Maximum depth of 140m and maximum wall thickness of 3.0 m are possible in this construction. This can be applied to any ground condition including soft ground and rock ground.

Source: Bauer Foundation HP

Figure 7.5.10 Case of RC Diaphragm Wall Circular Shaft

a) Construction Procedure

The leading element is constructed first and then the succeeding element is constructed for walls to be continuous.

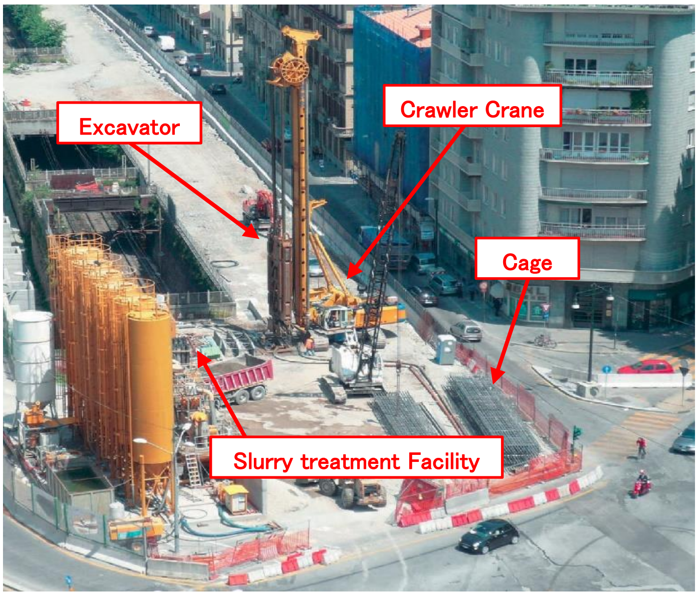


Source: Diaphragm Wall Association (http://www7b.biglobe.ne.jp/~renpeki/index.html)

Figure 7.5.11 RC Diaphragm Wall Construction Procedure

b) Construction Overview

The RC diaphragm wall is constructed by inserting reinforcement cages into drilling grooves excavated using slurry and replacing slurry with concrete to cast continuous RC wall. Excavators, cages, crawler crane to install cages, slurry treatment facility are required for the construction. This method enables excavation without using short strut and wailing by making the shape of shaft circular so that the structure can be stabilized with axial compressive force acting in the horizontal direction.



Source：Bauer Trench Cutter System HP
(http://www.bauerpileco.com/export/shared/documents/pdf/bma/datenblatter/BC_Trench_Cutter_Systems_905-679-2_EN_01-16.pdf)

Figure 7.5.12 Photo of RC Diaphragm Wall Construction

(ii) Open-Caisson Method

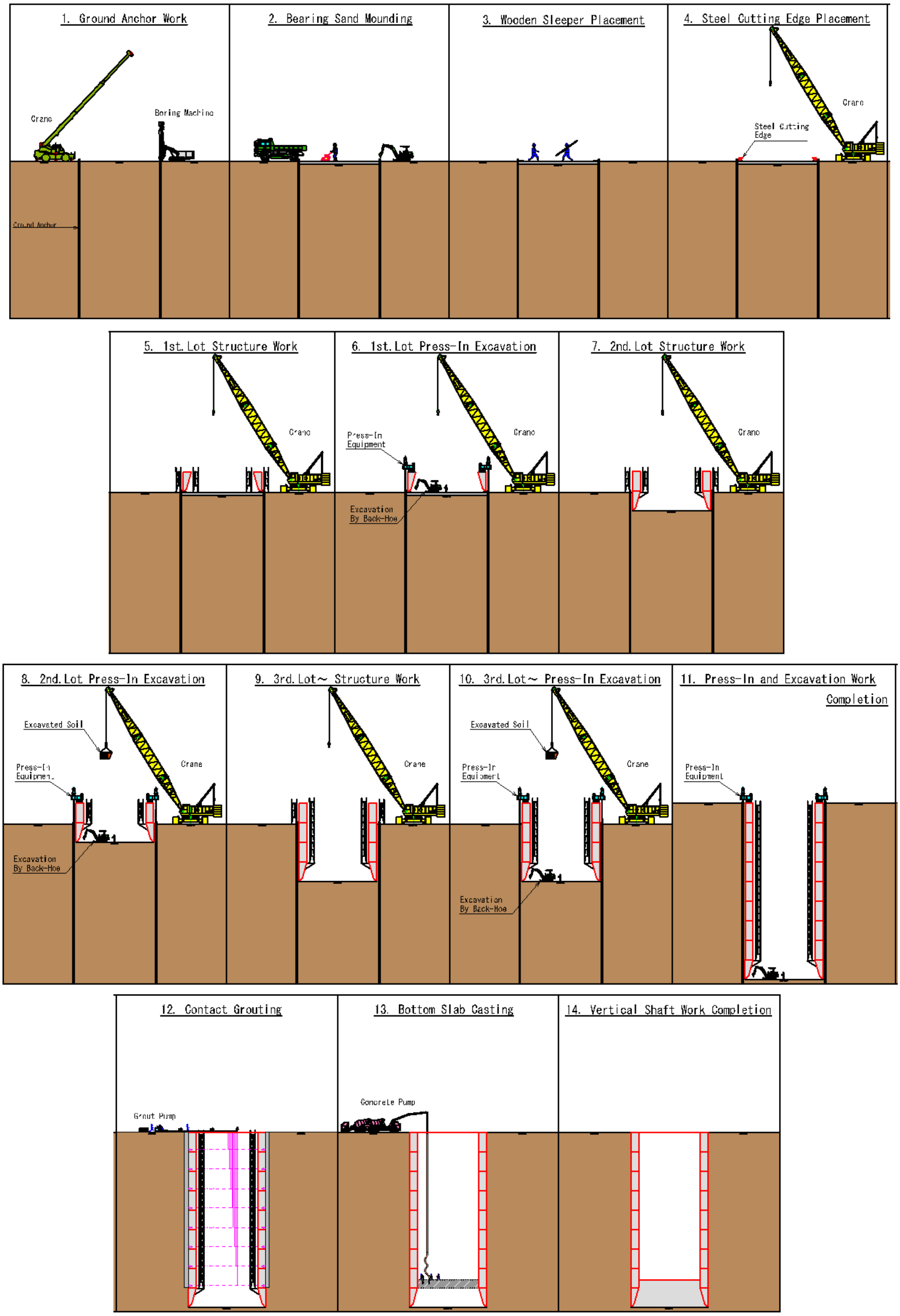
In this method, the main frame of caisson is constructed with reinforced concrete on the ground and it is installed into the ground by using reaction force of dead weight and ground anchors while proceeding with the excavation of the ground at the bottom surface of the caisson. There seems no sump water in the area planned for construction and even if there is, sump, treatment will be enough to cope with the sump water. Excavation will be done with backhoes and breakers and soil is gathered with a bucket-type machine

Source：Construction Plaza (http://www.kensetsu-plaza.com/kiji/post/6751)

Figure 7.5.13 Photo of Press Caisson Construction

a) Construction Procedure

The construction procedure for press caisson in a general case of shaft excavation without problem of leaked water is shown in Figure 7.5.14. If the result of boring survey shows that the strength of excavation ground is high at the location where the shaft is planned, countermeasures such as replacing the ground with sand at the cutting-edge part prior to the excavation are necessary.

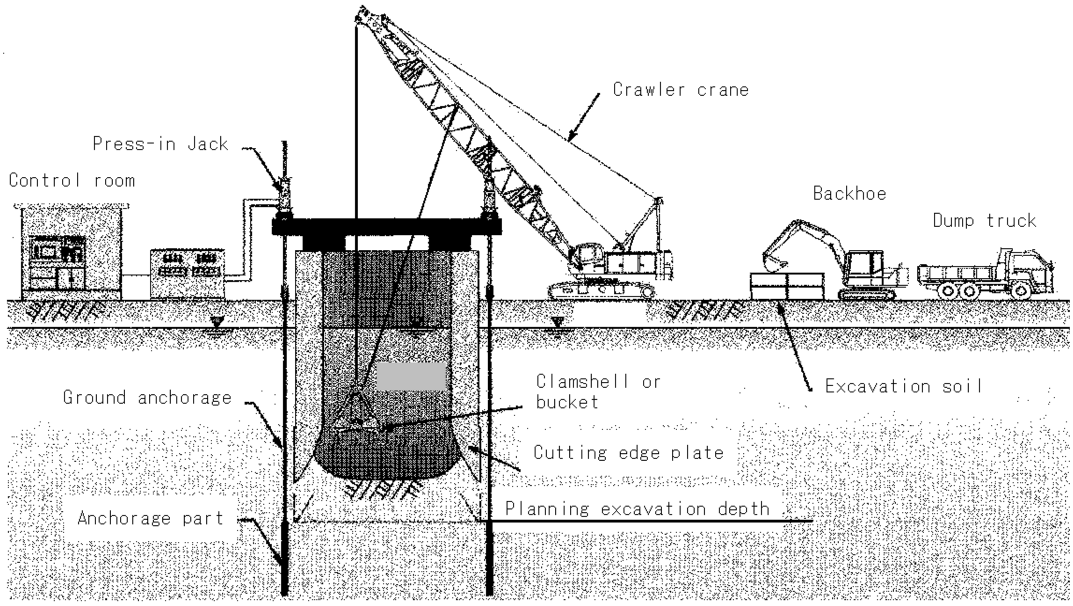


Source: JICA Survey Team

Figure 7.5.14 Overview of Construction Procedure for Press Caisson

The construction of press caisson is generally conducted with equipment and arrangement shown in Figure 7.5.15. There is concern that dead weight of cutting-edge cannot be supported since the surface layer is weak. In this case, formation level should be built after excavating to the layer with sufficient soil bearing capacity. In addition, countermeasures such as soil improvement to secure sufficient soil bearing capacity are necessary.

Furthermore, anchorage zone for reaction force is to be installed at the place deeper than the depth of caisson floor. In this case, since the anchor is longer than 80m and it is difficult to make it removable, upper part of anchor tendon should be cut and left at the position.



Source: Japan Society of Civil Engineers: Design of Vertical Shaft for Shield Construction

Figure 7.5.15 Overview of Press Caisson Construction

(iii) Selection of Construction Method

Table 7.5.8 Comparison of Construction Methods of Vertical Shaft

		RC Diaphragm Wall	Open Caisson
Requirements for Plan		1. Inner space for Shaft: φ30 m, 2. Depth of shaft floor：Max. 74.2 m, 3. Overburden, 50 m, and opening for shield launching, φ13.6 m	
Application Scale	Inner Space	No limitation	6 m to 20 m (30 m)
	Floor Depth	Penetration length: 140 m	30 m (0 m)
	Wall thickness	3 m	No limitation since it is constructed cast-in-place on the ground.
Applicability		Since internal water pressure acts on the shaft and the frame should be constructed inside of RC wall, inner diameter of the wall would be bigger and wall dimension would be larger. In addition, construction period would be extended to construct the inner frame.	If rock bolts for NATM are considered as a part of the structure and 50m of overburden is secured, the depth will be the greatest among any previous constructions. The structure can be reasonable considering the opening for shield machine which is used for installing the frame perfected on the ground.

	RC Diaphragm Wall	Open Caisson
Construction Period	RC wall：24 months Excavation：5 months Inner frame：15 months Total：44 months	Advance drilling (replacing with sand)：16 months Caisson (Inclusive of Excavation, Inner frame)：33 months Total：49 months
Construction Cost	RC wall ：3.4 billion JPY Excavation：0.6 billion JPY Inner frame：2.3 billion JPY Total：6.3 billion JPY	Advance drilling (replacing with sand)：2.0 billion JPY Caisson (Inclusive of Excavation, Inner frame)：2.8 billion JPY Total：4.8 billion JPY
Evaluation	△	○

Legend: ○ good/possible, △ Not good/some problems

Source: JICA Survey Team

(3) Study on the Construction Method of Open Channel

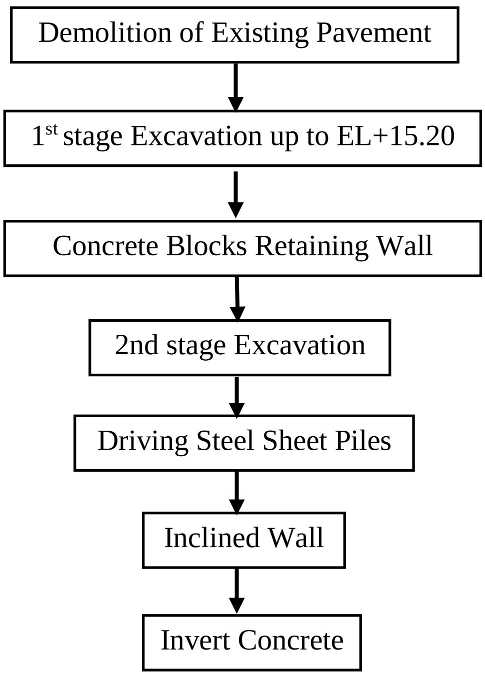
(a) Work Procedure

(i) Relation with Other Works

After the commencement of the Project, the construction of open-channel which does not have any conflict with other works can proceed concurrently with the construction of launching shaft. However, it is necessary to keep the route for the disposal of excavated surplus soil from the construction of shafts and tunnel and also for the transportation of materials and equipment.

(ii) Work Procedure for Open-Channel

The work procedure for open-channel is as follows:



Source: JICA Survey Team

Figure 7.5.16 Work Flow of Open Channel Construction

(b) Equipment Required

The equipment required for open channel construction is as shown below. Judging from the results of soil investigation, the appearance of soft rock is assumed. In this case, excavation by using bulldozer with ripper will be necessary.

Table 7.5.9 Equipment for Procurement from Foreign Country

Work Item	Function	Equipment Required
Earthwork	Demolition of Existing Pavement	Giant Breaker (Base Machine; Excavator)
	Excavation (Soil)	Excavator, Bulldozer
	Excavation (Rock)	Bulldozer with Ripper Giant Breaker
	Loading	Excavator, Pay loader
	Transportation	Dump Truck
	Levelling	Bulldozer
	Compaction	Vibratory Roller, Tire Roller, Plate Compactor, Tamping Rammer, Bulldozer
	Trimming of Slope	Excavator
	Lifting of Reinforcement, Formwork	Rough Terrain Crane
Concrete Structure Works	Compaction of Ready-Mixed Concrete	Concrete Vibrator
	Transportation of Ready-Mixed Concrete	Concrete Agitator Car
	Casting of Ready-Mixed Concrete	Concrete Pump, Rough Terrain Crane with Hopper
	Driving of Steel Sheet Piles	Vibro-hammer (Base Machine; Excavator)

Source: JICA Survey Team

(4) Plan for Transportation and Disposal of Surplus Soil

(a) Transportation Plan for Surplus Soil

The handling of surplus soil generated from the construction is a special concern in this project, so that the construction plan takes the location of surplus soil disposal into consideration.

When selecting the Shield Tunnelling Method for the tunnel assuming that one shield machine is used for the excavation, excavation has to start from either Manila Bay or Laguna de Bay. Since landfill is very difficult because Manila Bay is protected under the Ramsar Convention and the development of seaside districts has advanced, it is extremely difficult to secure the land for landfill. On the other hand, there is not much restriction on landfill or temporary storage and realization is higher than that of Manila Bay, so that the launching shaft which handles muck is planned to be constructed at the Laguna de Bay side.

When NATM is selected for the tunnel part, excavation starts from both the Laguna de Bay and the Manila Bay sides and connects underground. Therefore, muck will be generated from both sides. However, since the excavation speed of the NATM is slower than that of the shield tunnelling method, the amount of muck generated daily is not as much as the shield tunnelling method. Muck can be stored on the site during heavy traffic and transported to the disposal site located near the Laguna de Bay by dump trucks at night.

(i) Maximum Muck generated per day by Shield Tunneling Method

Assuming that the monthly excavation distance is 350 m in the shield tunnelling method, 50% is assumed and added to round up the maximum monthly excavation distance to 525 m per month, based on the past record of Shield Tunnelling Projects in Japan and Southeast Asia

13.56 x 13.56 π / 4 x 525 / 30 = 2,527 m^3/day

2,527 m^3/day x 1.3 = 3,285 m^3 (consider volume rate of change for soil soft rock I: 1.3)

3,285 m^3 / 5 m^3 / truck = 657 trucks/day

657 trucks/day / 24h = 27.4 trucks/h ⇒ One dump truck transports muck every 2 min and 10 sec.

(Note: Under the condition that truck transportation can be executed without any limitation for 24 hours.)

(ii) Maximum Muck generated per day per Shaft in NATM

Assuming that the monthly excavation distance is 80 m for NATM, 50% is assumed and added to round up the maximum monthly excavation distance to 120 m per month, based on the past record of NATM Projects in Japan and Southeast Asia.

13.56 x 13.56 π / 4 x 1 20 / 30 = 578 m^3/day

578 m^3/day x 1.3 = 752 m^3 (consider volume rate of change for soil soft rock I: 1.3)

752 m^3 / 5 m^3/truck = 151 trucks/day

151 trucks/day / 6h = 25.2 trucks/h ⇒ One dump truck transport muck every 2 min and 10 sec.

(Note: Under the condition that truck transportation can be executed for 6 hours from 22:00 to 04:00.)

(b) Plan of Surplus Soil Disposal

(i) Generation Amount of Surplus Soil

Generation amount of surplus soil is calculated for each route (1 or 3) and excavation method (Shield or NATM).

Route 1: Lower Bicutan to South Parañaque River

Route 3: Sucat to Zapote River

Soft Rock I of Estimated Standards, October 2016 Version (Ministry of Land, Infrastructure, Transport and Tourism, Japan), is adopted as the bulking factor of soil.

Table 7.5.10 Generation Amount of Surplus Soil

【Conditions】：Route-A, Sheild

Tunnel	Excavation diameter	Φ13.56 m
	Excavation length	6 km
Inlet shaft	Excavation diameter	37 m
	Excavation depth	75 m
Outlet shaft	Excavation diameter	37 m
	Excavation depth	82 m

【Soil Volume】

Type of Works	Excavation	Transportation	Back-fillinmg
	1.0	1.3	1.2
Tunnel	866,484 m^3	1,126,429 m^3	996,457 m^3
Inlet shaft	80,641 m^3	104,833 m^3	92,737 m^3
Out let shaft	88,167 m^3	114,617 m^3	101,392 m^3
Open channel	440,633 m^3	572,823 m^3	506,728 m^3
Total	1,475,925 m^3	1,918,702 m^3	1,697,314 m^3

【Conditions】：Route-D, Sheild

Tunnel	Excavation diameter	Φ13.56 m
	Excavation length	8.6 km
Inlet shaft	Excavation diameter	37 m
	Excavation depth	75 m
Outlet shaft	Excavation diameter	37 m
	Excavation depth	82 m

【Soil Volume】

Type of Works	Excavation	Transportation	Back-fillinmg
	1.0	1.3	1.2
Tunnel	1,241,960 m^3	1,614,548 m^3	1,428,254 m^3
Inlet shaft	80,641 m^3	104,833 m^3	92,737 m^3
Out let shaft	88,167 m^3	114,617 m^3	101,392 m^3
Open channel	152,577 m^3	198,350 m^3	175,464 m^3
Total	1,563,345 m^3	2,032,349 m^3	1,797,847 m^3

【Conditions】：Route-A, NATM

Tunnel	Excavation diameter	Φ13.56 m
	Excavation length	6 km
Inlet shaft	Excavation diameter	32 m
	Excavation depth	75 m
Outlet shaft	Excavation diameter	32 m
	Excavation depth	82 m

【Soil Volume】

Type of Works	Excavation	Transportation	Back-fillinmg
	1.0	1.3	1.2
Tunnel	866,484 m^3	1,126,429 m^3	996,457 m^3
Inlet shaft	60,319 m^3	78,414 m^3	69,366 m^3
Out let shaft	65,948 m^3	85,733 m^3	75,841 m^3
Open channel	440,633 m^3	572,823 m^3	506,728 m^3
Total	1,433,384 m^3	1,863,399 m^3	1,648,391 m^3

【Conditions】：Route-D, NATM

Tunnel	Excavation diameter	Φ13.56 m
	Excavation length	8.6 km
Inlet shaft	Excavation diameter	32 m
	Excavation depth	75 m
Outlet shaft	Excavation diameter	32 m
	Excavation depth	82 m

【Soil Volume】

Type of Works	Excavation	Transportation	Back-fillinmg
	1.0	1.3	1.2
Tunnel	1,241,960 m^3	1,614,548 m^3	1,428,254 m^3
Inlet shaft	60,319 m^3	78,414 m^3	69,366 m^3
Out let shaft	65,948 m^3	85,733 m^3	75,841 m^3
Open channel	152,577 m^3	198,350 m^3	175,464 m^3
Total	1,520,804 m^3	1,977,045 m^3	1,748,925 m^3

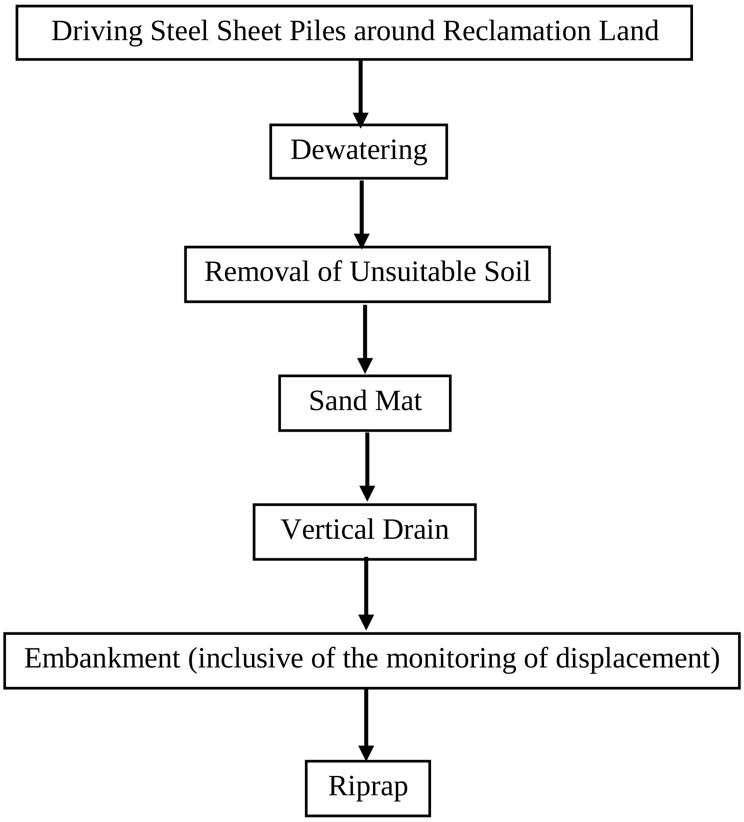
Source: JICA Survey Team

(ii) Plan for Surplus Soil Disposal

Surplus soil should be transported and disposed by dump trucks to the soil disposal site of Laguna de Bay secured within 10km from the launching shaft, i.e., the area adjacent to Metro Manila.

(iii) Reuse of Excavated Material

As stated above, in this survey, the plan for disposal of excavated surplus soil considers up to transportation and subsequent disposal into the designated disposal yard for Laguna de Bay. However, there is another option where excess excavated soil is reused for the reclamation of Laguna de Bay to create new land area. In this case, it is necessary to consider countermeasures to minimize the consequence such as settlement and horizontal displacement of land that will be newly created. Working procedure for the reuse of excavated material is as follows.



Source: JICA Survey Team

Figure 7.5.17 Work Flow of Re-use of Excavated Material

The estimated cost for re-use of excavated material is about 2,300 million pesos which is based on Option 3 (Route 3, Shield Tunnelling Method).

Quantity of embankment is as follows:

Area: 45 ha, Height: 4 m, Volume: 1.8 million m^3

7.5.2 Procurement Plan

(1) General Circumstances on the Procurement of Materials, Equipment and Labour in Metro Manila

Parañaque City, located inside Metro Manila, does not have any problem on the procurement of materials, labour, equipment for the construction of general concrete works, earth works and foundation works, etc.

(a) Procurement of Materials

(i) Steel Materials

Steel materials such as reinforcement, steel sheet piles and H-beams, etc., can be procured from local suppliers. However, steel sheet piles exceeding Type IV and H-beam larger than H-400×400 need to be imported from overseas countries.

(ii) Ready-Mixed Concrete

There are many commercial batching plants in Metro Manila. However, there are cases where onsite batching plant is set for the purpose of quality and schedule control.

(iii) Borrow Pit

There are borrow pits in Antipolo and Angono, Province of Rizal. In the case of “Metro Manila Flood Control Project - West Mangahan” implemented during the period from 2000 to 2007, Fort Bonifacio was used for borrow pit. However, Fort Bonifacio is currently not available because the development of a city in this area has already been done.

(iv) Aggregates

There are quarries in the provinces of Pampanga and Rizal (Antipolo, San Mateo, etc.)

(b) Procurement of Labor

Generally, local contractors keep enough Pilipino skilled labourers.

(c) Procurement of Equipment

Local contractors keep common equipment such as excavators, bulldozers, dump trucks, etc. In recent years the number of rental companies for construction equipment have increased and even uncommon equipment like 500-ton crane, etc., can be rented.

(2) Policy on the Procurement of Contractor

The construction of the large-scale and deep tunnel structure of Parañaque Spillway is unprecedented in the Philippines. Therefore, it is necessary to choose international prime contractors who have a wide experience in similar works by ICB (International Competitive Bidding). At first, PQ (Pre-qualification) of contractors is necessary to evaluate experiences, past records, capabilities, etc.

(3) Procurement from Overseas Countries for the Construction of Parañaque Spillway

As stated above, there are many past records on the construction of common concrete and earth structures in Metro Manila. Therefore, there is no problem in the procurement for the construction of said structures. However, the construction of Parañaque Spillway involves the construction of shafts at great depths and large tunnel works by either the Shield Tunnelling Method or the NATM which are unprecedented works in the Philippines.

Therefore, the procurement from overseas countries is necessary in the following items. In addition, the procurement of main materials for the shield method need lots of labour and cost in case of procurement and transportation from foreign countries. Therefore, local fabrication is considered realistic subject to the guidance and supervision from the foreign supplier.

Table 7.5.11 Items for Procurement from Foreign Countries

Works	Items for Procurement		Remarks
Tunnel Works (Shield Tunneling Method)	Material	Backfilling Material	
		Additive (Mud Additive)	
	Equipment	Shield Machine	Candidate Countries; Japan, Western Countries (Germany, America, etc.)
		Plant	Candidate Countries; Japan, Western Countries, Singapore, China, etc.
		Segment Lifter	Unloading of Segments into Shafts
	Labor	Technical Staff	Overall Tunnelling Work
		Maintenance Staff	Repair and Maintenance of the above Machineries
Tunnel Works (NATM)	Material	Support Materials (Rock Bolt, Steel Support)	
	Temporary Material	Movable Formwork for Lining Concrete	
	Equipment	Drill Jumbo	Hydraulic Drifter (Making Narrow Holes by Rock Bolt)
		Shotcrete Machine	
		Brower	
		Measurement Apparatus	
	Labor	Technical Staff	Overall Tunnelling Works
		Maintenance Staff	Repair and Maintenance on the above Machineries
Construction of Shafts (Open Caisson Method)	Equipment	Jacking Apparatus ・Boring Machine ・Anchor for Reaction Force ・Press-in Jacks ・Press-in Beams ・Reinforcing Steel Plate of Cutting Edge	Procurement from Japan (the team for the construction of shafts)
	Labor	Technical Advisor	
Construction of Shafts (Diaphragm Wall)	Equipment	Trencher	Candidate Countries; Japan, Western Countries, Singapore, etc.
		Muddy Water Treatment Plant	Ditto
	Labor	Technical Advisor	Ditto
M&E	Equipment	Pump, Ventilation Fan, Screening Equipment, Gate, Control Panel, etc.	Ditto

Source: JICA Survey Team

7.6 Plan of Operation, Maintenance and Management

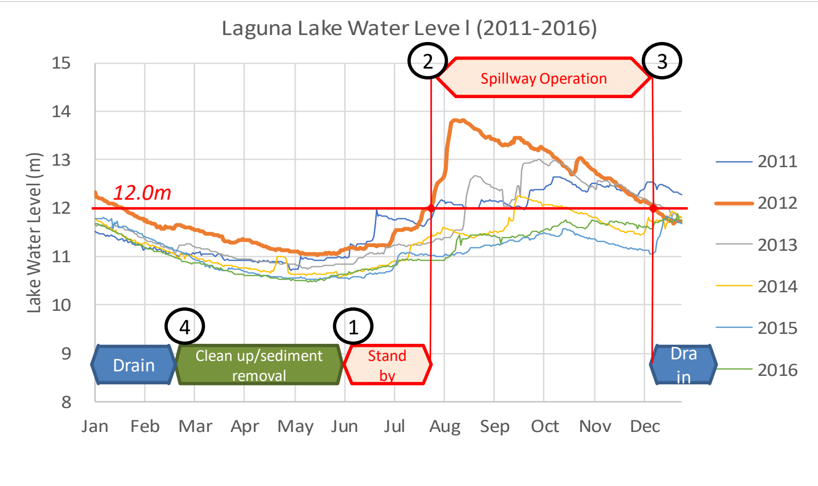
The plan of operation, maintenance and management for the proposed priority project of Parañaque Spillway were studied through assessment of requirements for operation, maintenance and management of the structure, facilities and equipment. In addition, interview and data collection were conducted to study the present conditions of operation and maintenance of the existing drainage channels and pumping stations, and to estimate the operation and maintenance cost. Consequently, issues and consideration are summarized for the additional study and the project implementation in the future.

7.6.1 Outline of Plan of Operation, Maintenance and Management

(1) Work Flow of Operation, Maintenance and Management

The work flow of operation, maintenance and management of the proposed Parañaque Spillway is shown in Figure 7.6.1. Lake-water level of the Laguna de Bay varies in a year; i.e., the lowest in the end of dry season from April to May and the highest in the latter half of the wet season from September to January. The proposed facilities will be used as flood spillway from a large reservoir such as the Laguna de Bay, and to be operated aiming at reducing lake-water level rising in the wet season as the target for the long-term lake water level variation.

Based on the results of hydraulic and hydrological analyses and the basic design of the proposed Parañaque Spillway, spillway operation is started when the lake water level rises above EL. 12.00 m, and ended after the lake water level has subsided below EL. 12.00 m. After the stop of spillway operation, water in the underground tunnel is drained, then cleaning and inspection for the tunnel is conducted to prepare for the operation in the next wet season.



① Stand-by
② Start spillway operation (when Lake WL >12.0m)
③ Finish spillway operation (when Lake WL <12.0m), and start drain from tunnel
④ Start clean up of underground tunnel after completion of drainage
⑤ Stand by

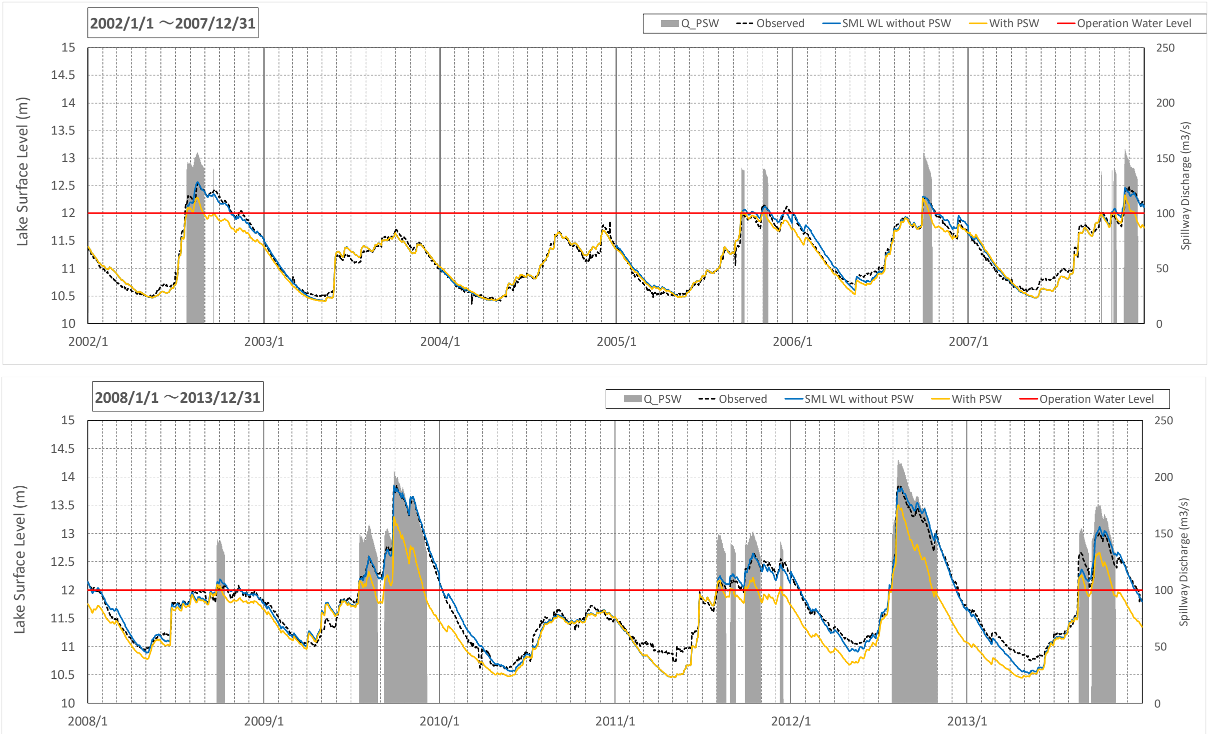
Source: JICA Survey Team

Figure 7.6.1 Example of General Schedule of Operation and Maintenance of Parañaque Spillway

(2) Results of Long-Term Simulation of Parañaque Spillway

Based on the results of the long-term reproduction calculation of the Laguna de Bay lake water level change from 2002 to 2013 as mentioned in Subsection 3.4.1, basic data of the spillway operation is assessed. The results of calculation are presented in Figure 7.6.2.

Paranaque Spill Way control level=12.0m



Source: JICA Survey Team

Figure 7.6.2 Results of Long-Term Reproduction Calculation of the Laguna de Bay Lake Water Level Change from 2002 to 2013

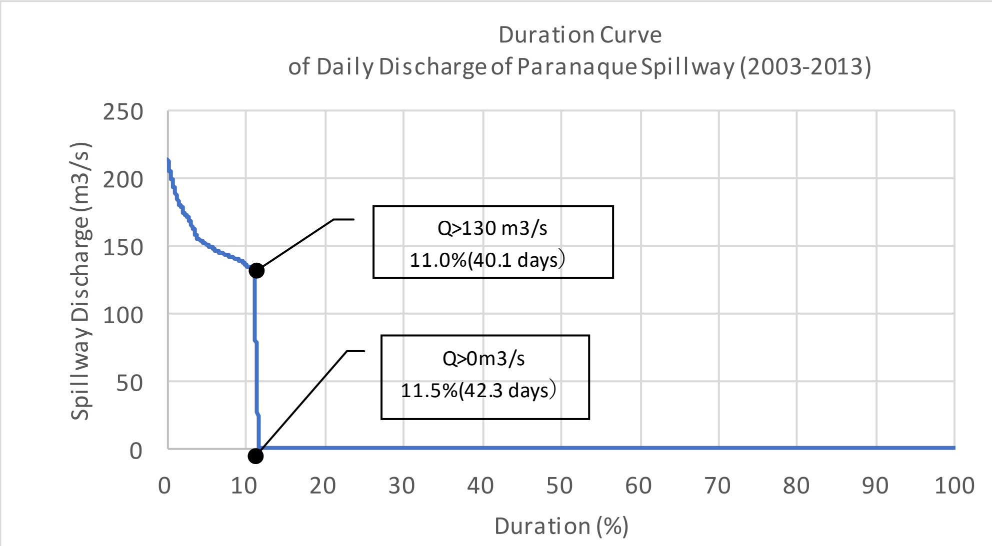
As the result of the calculation, the data of operation period, number of operation dates, operation frequency and total discharge volume of the spillway is estimated for each year as shown in Table 7.6.1. There were three years in which no spillway operation was conducted. For the operation period, the spillway operation was started from July in the earliest case, and the spillway operation was ended by December in the latest case. In 2009, which was the case of wet year, the spillway operation continued for a half-year from July to December.

Table 7.6.1 Basic Information of the Parañaque Spillway Operation

Year	Operation Period	Number of Operation Days	Operation Frequency (times)	Total Spillway Discharge (million m^3)
2002	Jul. – Sep.	39	3	483
2003	—	0	0	0
2004	—	0	0	0
2005	Sep.－Nov.	19	2	209
2006	Oct.	21	1	251
2007	Oct. – Dec.	41	4	481
2008	Sep. – Oct.	17	1	205
2009	Jul. – Dec.	129	2	1,785
2010	—	0	0	0
2011	Aug. – Dec.	73	4	846
2012	Jul. – Oct.	95	1	1,444
2013	Aug. – Nov.	74	2	963
Min.	-	0	0	0
Max.	-	129	4	1,785
Average	-	42.3	1.7	556

Source: JICA Survey Team

Figure 7.6.3 shows the duration curve of daily discharge of the spillway based on the Reproduction Calculation. The number of operation dates (Q > 0 m^3/s) is 42.3 days per year which is 11.5% in frequency. As shown in the diagram, there is a break point at the spillway discharge of 127 m^3/s, because the Parañaque Spillway adopts the operation rule of the control water level (operation starting water level of EL. 12.0 m) to open/start the spillway operation. At the control water level, the spillway discharge is designed at approximately 130 m^3/s. The duration of this discharge is 40.4 days per year which is 11.1% in frequency. It is said that most of the time of spillway operation, the spillway discharge is not less than 130 m^3/s, and therefore, the general flow velocity of the spillway tunnel is estimated at around 1.2 m/s in the condition of the discharge of 130 m^3/s and tunnel inner diameter of 12.0 m.

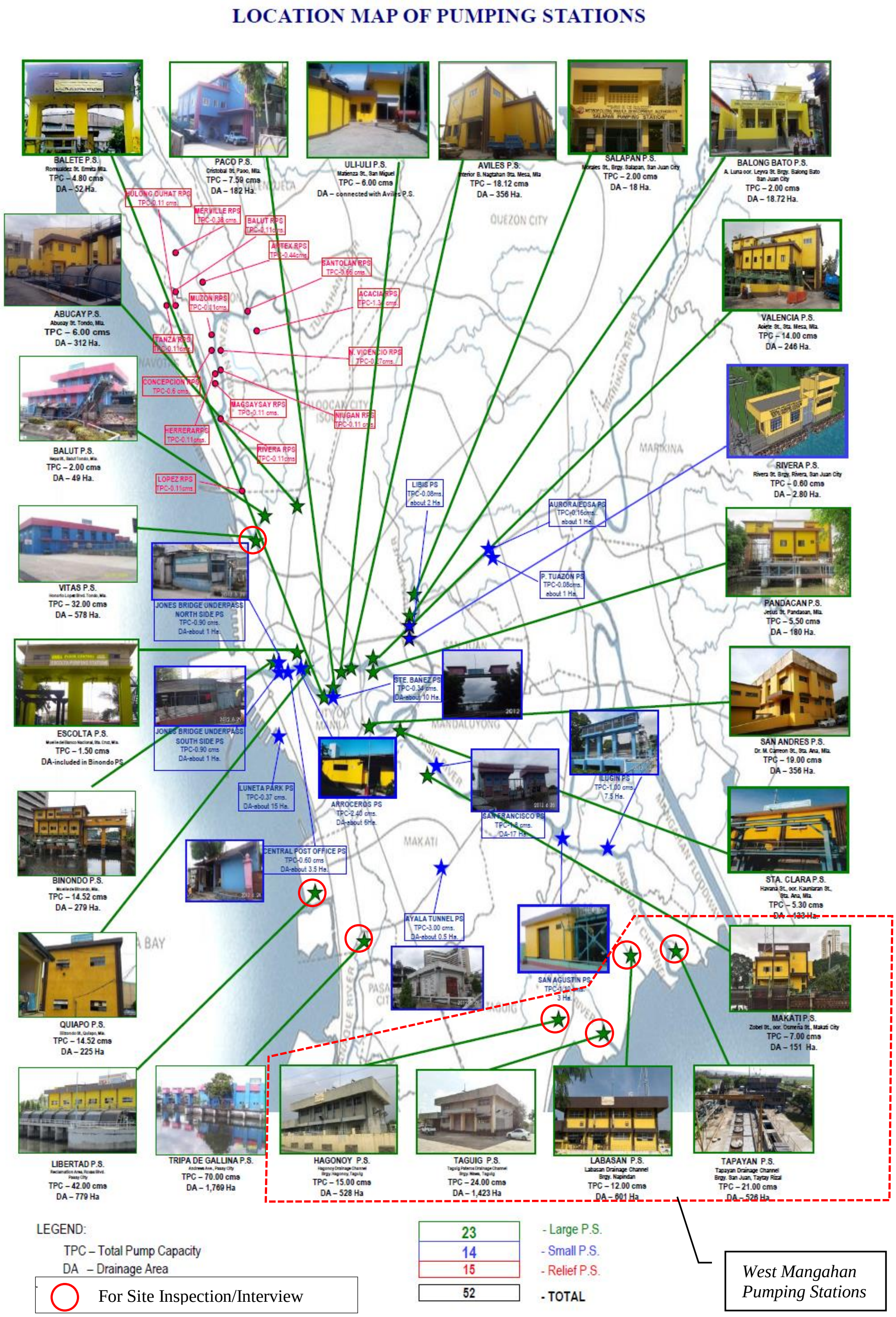


Source: JICA Survey Team

Figure 7.6.3 Duration Curve of Daily Discharge of Parañaque Spillway

(3) Study on Present Condition of Operation, Maintenance and Management of Existing Drainage Pumping Stations in the Philippines

Drainage canals and pumping stations in Metro Manila have been managed by MMDA since 2002. Location map of existing pumping stations is presented in Figure 7.6.4. In order to study the present conditions of operation and maintenance for existing pumping stations, site inspection of existing pumping stations and interview survey with the engineers of MMDA-Flood Control and Sewerage Management Office (FCSMO) and technical staff in the pumping stations were conducted. The result is summarized as follows.



Source: MMDA

Figure 7.6.4 Location Map of MMDA Pumping Stations in Metro Manila

- The operation and maintenance of drainage pumping stations are undertaken in 3 shifts of 24 hours. Overtime work is done especially during the flood season to increase manpower.

- An operation and maintenance manual is prepared during the construction of each pumping station, but no update is undertaken.
- An operation rule is established for the pumping system of each station, defining the pump starting water level and pump stopping water level to enable the site engineer to directly control the pumping system by himself. The central office does not give any instruction, but just confirm the operation condition of each pumping station.
- A backup system with diesel power generator is installed in each pumping station for continuous operation in case of commercial power failure.
- Site staff in the pumping station carry out regular maintenance of mechanical equipment, such as changing of lubricating oil and tightening of bolts. If large-scale maintenance is necessary, it is implemented by the technical staff and budget of the MMDA head office.
- There are lots of sediment, garbage and water-hyacinth in the storage pond of drainage pumping stations. The removal and cleaning of sediment and garbage in drainage canals and storage ponds are done mainly by the DPWH. Cleaning work is basically done once a year, but it depends on the situation.
- An automatic dust removal machine and trash rack are installed at the intake of each pumping station. A large amount of floating garbage and water hyacinth are removed manually or by the dust removal machine every day in the rainy season. The daily average generated garbage volume is estimated at 3-6 m^3 for the pumping station along the Laguna de Bay, and 18 m^3 for the pumping station in the densely populated area. In the Vitas Pumping Station, Tondo, Manila, daily generated garbage was reduced from 30 m^3 to 18 m^3 after the implementation of resettlement of informal settlers along the river bank in association with river improvement works.
- Removed garbage at the pumping station is collected by LGUs, and transported to the waste treatment plant.
- Each pumping station has a general information board, a location map of local drainage system, an inventory sheet of equipment and a work-shift schedule with organization chart. Site staff members maintain a diary of actual daily operation results showing operation hours, fuel consumption and electricity consumption of each pump and generator, and removed volume of garbage.
- MMDA has 35 flood control facilities composed of drainage pump stations, floodgates and warehouses in Metro Manila. The annual budget for operation and maintenance of these facilities from 2014 to 2017 (only 6 months from January to June for 2017) is given in Table 7.6.2. The annual budgets in the past 4 years were kept at almost the same amount of around 100 million to 120 million pesos. The major pay items are fuel cost, electricity cost and costs for labour/manpower. The share of these items in the total cost is 21%, 26% and 47%, respectively.

Table 7.6.2 Annual Budget for Existing Pumping Stations of MMDA (2014-2017)

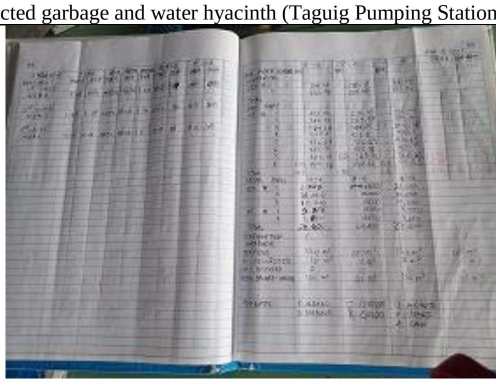
Year	2014	2015	2016	2017.1-6	Average(2014-2016)	
Operating Hour (hour)	23,045	13,421	18,472	3,896	18,313	
Fuel Consumption (ltr)	928,105	537,055	435,968	69,621	633,709	
Cost (PHP)						
Fuel	39,878,540	18,956,680	13,691,488	2,218,341	24,175,569	21.9%
Electricity	15,595,996	25,006,306	45,954,338	17,061,832	28,852,213	26.2%
Water service	3,102,130	3,553,912	3,706,934	1,344,384	3,454,325	3.1%
Telephon	101,531	101,531	101,531	69,097	101,531	0.1%
Labor/Manpower	49,995,156	51,754,937	52,729,491	28,727,213	51,493,194	46.7%
Micellaneous	1,889,817	1,281,567	3,115,101	2,046,550	2,095,495	1.9%
Total	**110,563,170**	**100,654,933**	**119,298,882**	**51,467,417**	**110,172,328**	100.0%

Source: MMDA-FCSMO


Dust and garbage removal at screen (Vitas Pumping Station)


Collected garbage and water hyacinth (Taguig Pumping Station)


Maintenance works of drainage pump (Hagonoy Pumping Station, Bulacan)


Daily report of pump operation (Tipra Pumping Station)

Source: JICA Survey Team

Figure 7.6.5 Photos of Operation and Maintenance at MMDA Pumping Stations in Metro Manila

(4) Study on Present Condition of Operation and Maintenance of Underground River Tunnel in Japan

Data collection and interview survey on the operation and maintenance works of existing underground river tunnels, namely; i) the Kandagawa River Ring Road No. 7 Underground Regulating Reservoir, and ii) the Metropolitan Area Outer Underground Discharge Channel, both in Japan, were conducted in the course of the study. The results are summarized in Table 7.6.3 and Table 7.6.4.

Table 7.6.3 General Information on Underground River Tunnels in Japan

Item		i) Kandagawa River Ring Road No. 7 Underground Regulating Reservoir	ii) Metropolitan Area Outer Underground Discharge Channel
Basic Information	Completion year	May 2008	2006
	Storage volume	540,000 m^3	670,000 m^3
	Tunnel Inner diameter	12.5 m	10.6 m
	Length	4.0 km	6.3 km
	Number of shafts	7	5
	Flood drainage pump	None	3,000 m^3/min x 4 units (gas turbine)
	Tunnel drainage pump	50m^3/min x 4 units	80 m^3/min x 2 units
	Project cost	103.0 Billion Yen	231.0 Billion Yen *[1]

*[1] Includes land acquisition cost of approx. 10% and staff salary of approx. 10%.

Table 7.6.4 Operation and Maintenance of Underground Tunnel River in Japan

Item		i) Kandagawa River Ring Road No. 7 Underground Regulating Reservoir		ii) Metropolitan Area Outer Underground Discharge Channel	
Operation and Maintenance Plan	Drainage days	within 2 days		within 5 days	
	Frequency of maintenance of pump/gate	Monthly		Daily for gas turbine	
	Frequency of cleaning and sediment removal of tunnel	Every year		Once in 3–4 years	
Annual Operation and Maintenance Cost (Mil. Yen)	Cleaning and sediment removal	98.0*[1]	43.6%	70.0*[2]	10.4%
	Maintenance of equipment	63.0	28.0%	230.0	34.3%
	Repair	-	-	280.0	41.8%
	Fuel and electricity cost	64.0	28.4%	70.0	10.4%
	Grass cutting, etc.	-	-	20.0	3.0%
	Total	225.0	100%	670.0	100%
		(0.22% of Project cost)		(0.36%of Project cost)	

*[1] In the Kandagawa River Ring Road No. 7 Underground Regulating Reservoir, cleaning and sediment removal of tunnel is conducted every year. The average volume of sediment removal is approximately 1,539 ton/year.

*[2] In the Metropolitan Area Outer Underground Discharge Channel, cleaning and sediment removal of tunnel is conducted once in every 4 years. In 2016, the volume of sediment removal for the past 3 years was approximately 3,900 m^3. Assuming that the unit weight of the sediment is 1.5 ton/m^3, .it is estimated at 1,950 ton/year (=3,900 / 3 x 1.5).

Source: JICA Survey Team

- Major pay items of operation and maintenance cost are cleaning and removal of sediment from the tunnel, maintenance and repair of equipment, and fuel and electricity cost.

- Annual operation and maintenance cost is 225.0 million yen for the Kandagawa River Ring Road No. 7 Underground Regulating Reservoir and 670.0 million yen for the Metropolitan Area Outer Underground Discharge Channel. These are 0.22% and 0.36% of the total project cost, respectively.
- The proposed Parañaque Spillway is designed to be approx. 930,000 to 1,150,000 m^3 of storage volume, which is almost the same as the above two examples of existing underground tunnels. It is roughly estimated that the Parañaque Spillway requires almost the same cost for its operation and maintenance.
- The unit cost of cleaning and sediment removal is estimated at JPY 36,000–64,000/ton based on the actual cost. This cost include the costs for cleaning the underground tunnel by high-pressure cleaning car, drainage by volume car at the vertical shaft, and final treatment of removed sediment and sludge.

Kandagawa River Ring Road No. 7 Underground Regulating Reservoir	JPY 98,000,000 / 1,539 ton = JPY 64,000 / ton or PHP 16,419 / ton
Metropolitan Area Outer Underground Discharge Channel	JPY 70,000,000 / 1,950 ton = JPY 36,000 /ton or PHP 29,317 / ton)

(5) Operation and Maintenance Plan of Parañaque Spillway

(a) Proposed Facilities and Equipment for Operation and Maintenance

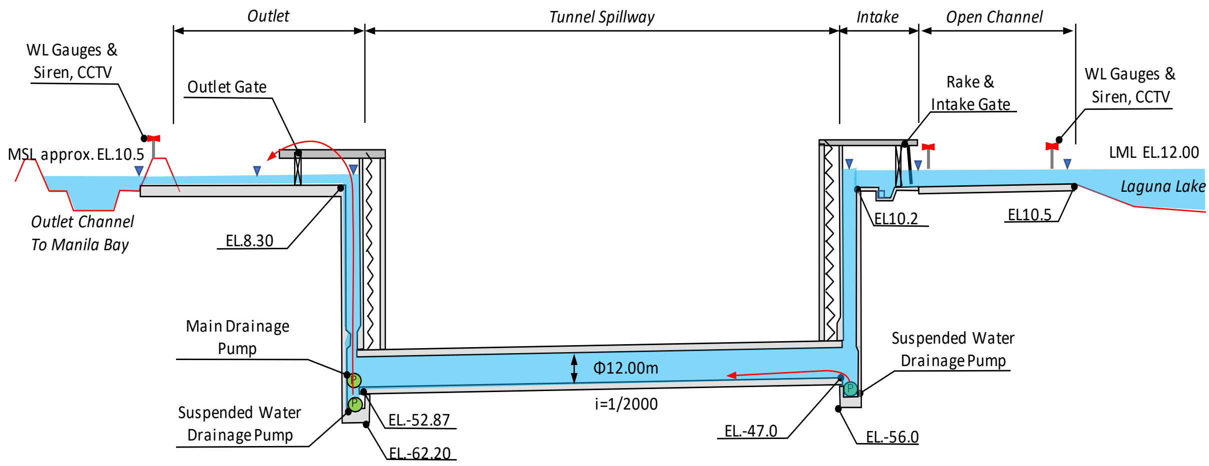
Facilities and equipment proposed for the operation and maintenance of Parañaque Spillway are as listed in Table 7.6.5.

Table 7.6.5 Target Facilities and Equipment for the Operation and Maintenance of Parañaque Spillway

Name of Facility	Work Components	Main Facilities and Equipment
Intake Facility (Inlet)	Civil structures	Intake, Intake open channel, Sand sedimentation pond, Inlet vertical shaft, Building of Control Center
	Dust removal system	Boom (Floating weed trap), Screen (Trach rack), Dust remover
	Pumping system	Suspended water drainage pump, Operation system
	Ventilation system	Ventilation fan
	Lifting system	Elevator, Stair, Gondola, Hoist crane
	Power supply system	Generator, Transformer, Fuel tank
	Water level and discharge monitoring system	Water level gauge, Remote monitoring device, Siren
	Water level and discharge control system	Inlet control gate, Stop-log
Drainage Facility (Outlet)	Civil structures	Outlet vertical shaft, outlet, outlet channel, Building of control center
	Pumping system	Main drainage pump, Suspended water drainage pump, Transformer, Generator, Operation system
	Ventilation system	Ventilation fan
	Lifting system	Elevator, Stair, Gondola, Hoist crane, Pressure door
	Power supply system	Generator, Transformer, Fuel tank
	Water level and discharge monitoring	Water level gauge, Remote monitoring device, Siren,

Name of Facility	Work Components	Main Facilities and Equipment
	system	
	Water level and discharge control system	Outlet gate, Stop-log
Spillway (Underground Tunnel Type)	Civil structures	Underground tunnel
	Cleaning and sediment removal equipment	Manual cleaning machine, High-pressure cleaning car, Wheel loader, Vacuum car
	Ventilation equipment	Ventilation fan

Source: JICA Survey Team



Source: The JICA Survey Team

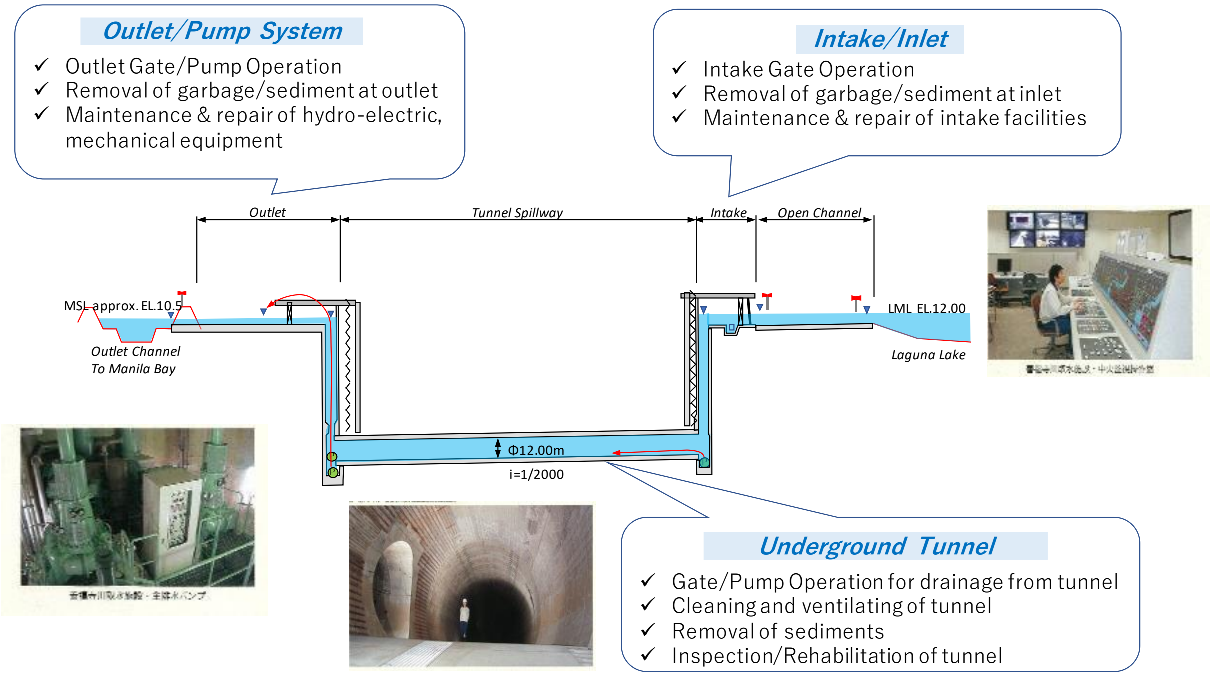
Figure 7.6.6 Configuration of Proposed Facilities and Equipment for the Operation and Maintenance of Parañaque Spillway

(b) Operation and Maintenance

The operation and maintenance work of the proposed Parañaque Spillway is divided into (A) Operation during flood season, and (B) Maintenance during normal condition, as shown below:

<A: Operation during flood season>
(A-1) Preparation and stand-by for spillway operation
(A-2) Spillway operation
(A-3) Drainage, ventilating, cleaning and sediment removal of spillway tunnel after spillway operation
<B: Maintenance during normal condition>

(B-1) Maintenance and repair works:



Source: Pamphlet of Kandagawa River Ring Road No. 7 Underground Regulating Reservoir

Figure 7.6.7 Outline of Operation and Maintenance System of Parañaque Spillway

(i) Operation and Maintenance in Flood Season

The outline of operation and maintenance during flood season, from preparation before flood, spillway operation, and the drainage and cleaning works of tunnel after the spillway operation, is given in Table 7.6.6.

Table 7.6.6 Work Items of Operation and Maintenance of Parañaque Spillway during Flood Season

Items	Works	Detail of Works	Implementing Agency/Frequency
(1) Preparation for spillway operation	Monitoring of lake water level, and preparation for operation	• Contiguous monitoring of lake-water level. • Both inlet and outlet gates are closed. • Carrying out of preparation works so that all facilities and equipment will work properly and timely when the lake water level rises above the operation starting water level (EL. 12.0 m).	Facility Management Office
(2) Spillway operation	Gate opening operation (Start spillway operation)	• In the rainy season, when the lake water level has risen above EL. 12.0 m, the inlet control gate is opened to start intake operation. • The outlet gate is closed for a while to prevent inverse flow from the drainage river to the outlet. • After the water level at the drainage river and the outlet has stabilized, the outlet gate is opened and spillway operation is started.	Facility Management Office
	Monitoring of Intake Facilities	• Before starting intake operation, safety in the adjacent area of intake in Laguna de Bay, open channel and intake should be confirmed, and notice of starting spillway operation is disseminated to concerned agencies and LGUs. • During spillway operation, the operator should continuously monitor inflow condition at intake	Facility Management Office

Items	Works	Detail of Works	Implementing Agency/Frequency
		by ocular inspection and remote monitoring devise to confirm the smooth inflow without any disturbance of clogging of intake with floating garbage.	
	Removal of garbage at intake screen	• Floating garbage trapped at the intake screen should be timely removed manually and by dust remover.	Facility Management Office/LGUs
	Monitoring of Drainage Facilities	• Before starting spillway operation, safety in the adjacent area of drainage-river and outlet should be confirmed and notice of starting spillway operation is disseminated to concerned agencies and LGUs. • During the spillway operation, the operator should continuously monitor the outflow condition at the outlet by ocular inspection and remote monitoring devise to confirm smooth drainage without any rapid increase of discharge and rise of water level.	Facility Management Office
	Gate closing operation (Stop spillway operation)	• When the lake water level has subsided below EL. 12.0 m, the inlet control gate and outlet gate are closed to stop the spillway operation.	Facility Management Office
(3) Drainage, ventilation, cleaning and sediment removal after spillway operation	Drainage of tunnel	• In the latter half of the wet season when lake water level drops down below EL. 12.0 m and in case it is not predicted that the next flood would come soon referring to the long-term meteorological information, etc., storage water in the tunnel should be drained by the main drainage pump installed in the outlet shaft. • Duration of drainage operation is assumed to be within 5 days. • After that, water remaining at the bottom of the inlet shaft is drained into the tunnel by the suspended water drainage pump. • Lastly, the water remaining at the bottom of the outlet shaft is drained to the outlet river by the suspended water drainage pump.	Facility Management Office
	Ventilating and cleaning of tunnel	• Ventilation of tunnel is necessary before maintenance staff enter the tunnel. After securing enough concentration of Oxidant by forced ventilation, cleaning of tunnel can be started. • Garbage and sludge on the wall and invert of the tunnel are cleaned up manually or by using a high-pressure cleaning car.	Facility Management Office
	Removal of sediment and garbage from the tunnel	• Thrown garbage and sludge from the wall are collected into a drain ditch of the invert and transported to the outlet shaft through the ditch. • Liquids are drained out by the suspended water drainage pump, and solids are packed into a box or container, then picked out from the shaft by lifting equipment.	LGUs; Facility Management Office/LGUs

Source: JICA Survey Team

(ii) Operation and Maintenance in Normal Condition

Work items and work flow for the operation and maintenance in normal condition is presented in Table 7.6.7.

Table 7.6.7 Items of Operation and Maintenance Works of Parañaque Spillway during Flood Season

Items	Works	Items of Work	Implementing Agency
Operation and Maintenance in Normal Condition	Periodical cleaning and dredging along the lakeshore	• Cleaning and removal of floating garbage and water hyacinth along the lakeshore adjacent to the intake. • In case massive sediment deposition is observed in the area, they are removed by heavy equipment.	LLDA; LGUs
	Removal of sediment in the intake facilitates	• In case massive sediment deposition is observed in the intake facility, they are removed by manpower or heavy equipment. Removed sediment istransported to the designated disposal area.	LGUs; Facility Management Office/LGUs
	Inspection and measurement in Tunnel	• Ocular inspection is conducted through all tunnel area to check if there are deformations, cracks, and water leakage. • In case any abnormal condition is found, detailed measurement and investigation should be conducted to identify the reason, and countermeasures should be carried out as required.	Facility Management Office
	Inspection and maintenance of civil structures and other facilities	• Ocular inspection is conducted for all civil structures. • Operational condition of drainage pumps, gates and other facilities should be confirmed. • In case any problem is found, repair and adjustment should be conducted as required, and reconfirm the structure’s operational conditions.	Facility Management Office

Source: JICA Survey Team

(6) Operation and Maintenance Organization

At present, DPWH conducts the operation and maintenance of large-scale flood control projects. In the Metro Manila area, MMDA is the agency responsible for the operation and maintenance of drainage canals and drainage pumping stations.

The proposed Parañaque Spillway is located within the Metro Manila area under the jurisdiction of MMDA, but the target area to be affected by the project operation and the project benefits cover the surrounding area of the Laguna de Bay, which include the provinces of Laguna and Rizal outside of the MMDA’s jurisdiction. It is, therefore, considered appropriate to establish a project implementation/operation and maintenance system by positioning DPWH at the center, taking into account of the coverage of project benefits as well as financial capability and number of technical staff.

Since MMDA has abundant experience and technical knowledge on the actual operation and maintenance works of drainage facilities, it is recommended that actual tasks for operation and maintenance are shared with MMDA by effectively using their human resources, facilities and equipment. It is also considered that a new facility management office/agency composed of both DPWH and MMDA staff is established for the operation and maintenance at the initial stage, then the completed facilities are later transferred to MMDA which will ultimately be responsible for the operation and maintenance of the Parañaque Spillway.

It is necessary to study and coordinate with both DPWH and MMDA about the demarcation of roles and responsibilities for the operation, maintenance and management of the Parañaque Spillway in the course of project implementation planning in the future.

(7) Operation and Maintenance Cost

Operation and maintenance cost of the Parañaque Spillway is mainly divided into the following three (3) categories:

1. Operation cost for drainage pump
2. Costs for cleaning and removal of sediment in the tunnel
3. Maintenance cost for facility and equipment

(a) Operation cost for drainage pump

(i) Estimation of Drainage Pump Capacity

As mentioned in Subsection 3.4.1, drainage operation of the Parañaque Spillway is not carried out after every small-scale flood event, but just once after a long-term spillway operation during wet season when the lake water level rises above EL. 12.0 m.

In general, an underground flood control facility constructed for regulating a flood event need to drain the stored water immediately after a flood in preparation for the next rainfall event. For example, in Japan, there are many cases that duration of drainage is set within 12 to 24 hours after every flood control operation.

On the other hand, the Parañaque Spillway is not required to drain the stored water after a short period of time based on its operation procedure. The duration of drainage of the Parañaque Spillway can be established considering the required size of pumping facility and the flow capacity of the drainage river. In this study, it is assumed that the duration of drainage from the underground tunnel is within five days, taking into account the necessity of drainage in case of emergency inspection after the occurrence of a large earthquake, and to avoid deposition and consolidation of suspended sediment in the stored water. The number of pumps is designed at two units considering the malfunction and alternate operation.

For the two cases of routes of the Parañaque Spillway, the required pump capacity is estimated as below.

Table 7.6.8 Required Capacity of Drainage Pump

Item		Unit	Route 1	Route 3
Tunnel	Inner Diameter	m	12.0	12.0
	Length	m	7,800	9,600
	Volume*2	m^3	931,600	1,135,200
Drainage Duration		day	5.0	5.0
Required Drainage Capacity		m^3/s	2.04	2.16
Drainage Pump	Discharge	—	1.2 m^3/s x 2 units	1.4 m^3/s x 2 units
	Head	m	67.7	67.7
	Install Capacity	kW	1,000	1,200

*1 Refer to Table 7.4.5 for the name of each route; *2 Includes volume of vertical shafts (inlet/outlet)

Source: JICA Survey Team

(ii) Operation Cost for Drainage Pump

Operation cost for the drainage pump is estimated by the following two methods:

1) Estimation by Cost Function Curve prepared in Japan

2) Estimation based on actual operation cost of existing drainage pumping stations

1) Estimation of Operation Cost by Cost Function Curve

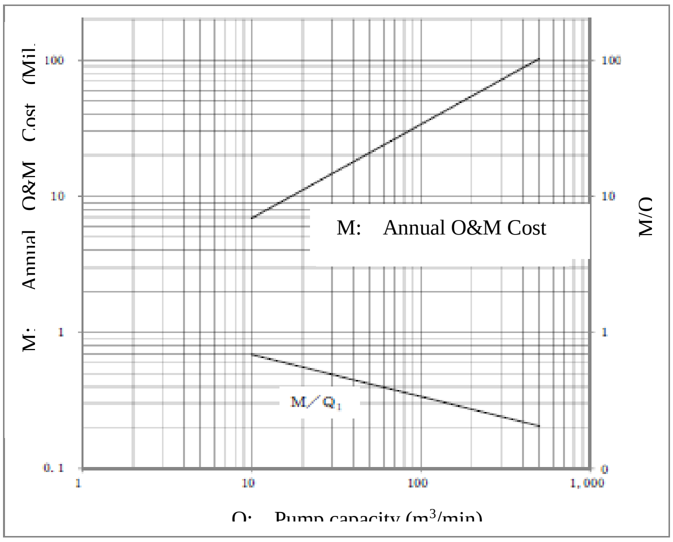
In Japan, cost function curves are established for construction cost and operation and maintenance cost of pumping station of general sewerage facility as below.

(Annual Operation and Maintenance Cost) $M = 1.00 \times Q^{0.69} \times (109.9 / 78.1)$

Where;

M: Annual operation and maintenance cost of pumping station (million yen/year)

Q: Hourly Maximum Pump Capacity (m^3/min)



Source: Guideline of Comprehensive Basin-wide Planning of Sewerage System 2015

Figure 7.6.8 Operation Cost by Cost Function Curve in Japan

Based on the above cost function curve and the design pump discharge of Q = 2.4 to 2.8 m^3/s (144 to168 m^3/min), the annual pump operation cost is estimated at 43.4 to 48.3 million yen/year. This cost include pump operation cost and maintenance cost of civil facilities and mechanical equipment, but not including the cost for cleaning and sediment removal from the tunnel.

Table 7.6.9 Operation and Maintenance Cost of Drainage Pumping Station by Cost Functioning Curve prepared in Japan

Item	Unit	Route 1	Route 3
Design Pump Capacity	m^3/s	2.4	2.8
	m^3/min	144	168
Annual operation and maintenance cost	Million yen/year	43.4	48.3
	PHP Million /year	19.9	22.1

Source: Estimation by JICA Survey Team referring to "Guideline of Comprehensive Basin-wide Planning of Sewerage System 2015"

2) Estimation based on Actual Operation Cost of Existing Drainage Pumping Stations

Actual operation record of four MMDA pumping stations located along the Laguna de Bay in West Mangahan is presented in Table 7.6.10. Referring to this record, operation cost of existing drainage pumping station is estimated.

In the above four pumping stations, the type of pump is the vertical axis flow pump with discharge capacity of 3.0 m^3/s per unit. Average annual operating hours of the four pumping stations was 3,152 hours for three years from 2014 to 2016, and the annual power consumption was approximately 2,560,200 kWh.

To keep the above operation level, the annual average budget for the operation and maintenance was PHP 14,054,200. This can be converted to the unit rate per kWh as PHP 5.49 /kWh (= 14,054,200 ÷ 2,560,200).

Table 7.6.10 Operation Record of MMDA Pumping Stations in West Mangahan Area (Average of 2014-2016)

Name of Pumping Station	Basic Information			Operating Hours		Annual Power Consumption	Total OM Budget	Unit Cost of Total/kWh
	Discharge	Drain Area	Capacity	Generator	Pump			
	m^3/s	ha	kW	Hour	Hour	kWh	Peso	Peso/kWh
Tapayan P.S.	21	526	930	792	1,384	1,286,705	4,397,450	3.42
Labasan P.S.	12	601	530	566	1,000	530,216	3,528,571	6.65
Taguig P.S.	24	1,423	1,060	649	586	621,061	3,323,539	5.35
Hagonoy P.S.	15	528	670	195	182	122,143	2,804,602	22.96
Total	**72**	**3,078**	**3,190**	**2,201**	**3,152**	**2,560,124**	**14,054,162**	**5.49**

Source: MMDA

Since the design pump head is 70 m and the design pump discharge is 2.4 to 2.8 m^3/s, the total output of the drainage pump is estimated at 2,000 to 2,400 kW. By assuming the drainage duration of 5 days (120 hours) and adopting the above unit cost per kWh, the required operation cost of the Parañaque Spillway is estimated as follows.

Table 7.6.11 Operation Cost based on Actual Operation Cost of Existing Drainage Pumping Stations

Item	Unit	Route 1	Route 3
Drainage duration per year	day	5.0	5.0
Total output of drainage pumps	kW	2,000	2,400
Annual power consumption	kWh	240,000	288,000
Unit operation cost per kWh (actual base)	Peso/kWh	5.49	5.49
Annual Operation Cost	Peso	1,317,600	1,581,120

PHP 1.0 = JPY 2.183
Source: JICA Survey Team

(b) Costs for cleaning and removal of sediment from the tunnel

(i) Sediment inflow into Underground Tunnel

From the viewpoint of maintenance of underground tunnel, it is appropriate not to make sediment enter and deposit in the underground tunnel. For this reason, the following measures are applied to reduce sediment deposition volume in the tunnel in this study.

<u>Measures for bed load</u>

The inlet facility is designed to have the capacity to trap sediment bed load before it enters the underground tunnel.

<u>Measures for suspended sediment and wash load</u>

For suspended sediment, it is expected that the sediment materials with a diameter of 0.02 cm can be deposited and removed due to trapping efficiency of the inlet facility as mentioned before. However, sediment materials finer than the above and wash load cannot be trapped completely before they enter the underground tunnel.

During normal operation of the Parañaque Spillway, the flow velocity in the tunnel is relatively fast at 1.2 m/s as mentioned before. Under this condition, most of the finer suspended sediment materials and wash load are not deposited in the channel but directly discharged from the outlet. On the other hand, when the spillway operation is stopped, the sediment materials in the stored water will no longer be suspended but start deposition in the tunnel. It is considered that some of these deposited sediment materials could be transported by the flushing effect when spillway operation is re-started, but re-suspension and transportation of consolidated sediment deposits will be very limited. There will be periodical need to mechanically remove the deposited sediment from the underground tunnel.

(ii) Estimation of Volume of Sediment Deposits in the Underground Tunnel

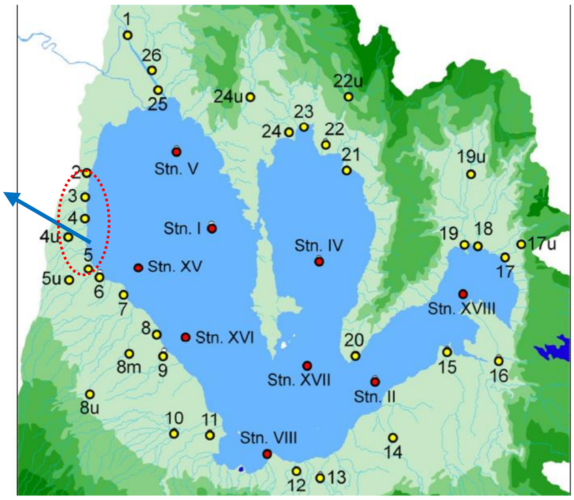
To estimate the volume of sediment deposits in the underground tunnel more accurately, it is necessary to conduct sediment hydraulic simulation. For this simulation, a more detailed survey of sediment concentration variation in the Laguna de Bay and riverbed materials is necessary, but so far no sufficient information is available. In this study, by assuming the sediment concentration of flood in the tunnel according to the available data, sediment deposition volume in the tunnel is roughly estimated as follows:

Concentration of Sediment Inflow into Underground Tunnel

Sediment transported with floodwater is categorized into two kinds, namely; suspended load and bed load, and as mentioned before, most of the sediment inflow to the tunnel is suspended load while bed load is trapped in the lake or in the inlet facilities. DENR conducts periodical monitoring of water quality including Total Suspended Solids (TSS) at 9 locations in the Laguna de Bay and 26 locations in the tributaries.

There are three monitoring stations (No. 2, 3 and 4) in areas around the proposed intake of the Parañaque Spillway. The result of previous monthly monitoring at the three stations from 2009 to 2015 is shown in Figure 7.6.9. The TSS varies by month and location. The maximum value of TSS among the data was recorded at 512 ppm (mg/ltr).

It is said, in general, that concentration of suspended sediment in floodwater has a correlation with flood discharge. Therefore, the concentration varies hourly or daily. However, the available data collected during the study is only monthly data, and not continuous. In this study, the concentration of sediment inflow is assumed at 520 ppm as the past recorded monthly maximum value of the suspended sediment. In order to make more accurate estimation of sediment deposition in the underground tunnel of the Parañaque Spillway, it is necessary to carry out a more detailed time scale and special scale monitoring of suspended sediment at the proposed inlet in the Laguna de Bay.



Source: LLDA

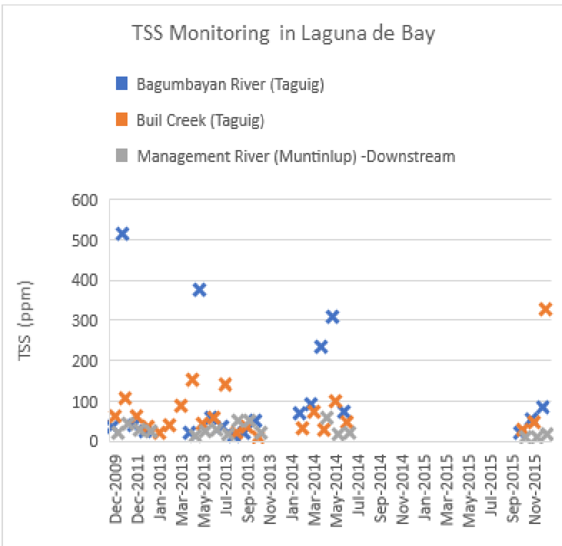


Figure 7.6.9 Location Map of Water Quality Monitoring Stations in Laguna de Bay and Tributaries, and Time Series Variation of Monitoring Data of TSS

Sediment Deposition in Tunnel

It is said that wash load does not settle in the condition that flow velocity is faster than 0.3 m/s. In the plan of the Parañaque Spillway, the design velocity of the tunnel is 3.6 m/s at the design discharge of 200 m^3/s, and 1.2 m/s even at the operation starting discharge of 130 m^3/s. It is appropriate to estimate that the settling volume of the suspended sediment is small at the time of the spillway operation during flood season.

On the other hand, after stopping spillway operation, water in the tunnel becomes stagnant until the next spillway operation or drainage operation by pump is started. In this study, it is assumed that 100% of suspended sediment in the whole stretch of the tunnel settle in this period. This assumption which estimates that sediment deposition volume is larger, is conservative.

In addition, it is considered that the settling of suspended sediment occurs 1.7 times per year, which is estimated based on the result of spillway operation simulation as described in Subsection 3.4.1.

By applying the above conditions, sediment deposition in the underground tunnel of the Parañaque Spillway is as estimated by the following equation.

$$Vs = Vt \times D \times r \times f$$

Where;

Vs : Annual sediment deposit volume in tunnel
D : Concentration of suspended sediment inflow (ton/m^3)
r : Ratio of sediment deposition after stopping of spillway operation (%/times)
f : Operation frequency of spillway per year (times/year)

The annual cost for cleaning and sediment removal from the tunnel is estimated by the above annual sediment deposit volume multiplied by the unit price (PHP 16,491/ton) that is estimated in the previous section, referring to the actual costs for operation and maintenance in tunnel spillways in Japan.

Table 7.6.12 Estimation of Sediment Deposition Volume in Underground Tunnel and Cost for Cleaning and Sediment Removal from Tunnel

Item	Unit	Route 1	Route 3
Length of Tunnel	km	5.9	8.6
Concentration of suspended sediment inflow (D)	ppm	520	520
	ton/m^3	0.00052	0.00052
Volume of tunnel (Vt)	m^3	931,600	1,135,200
Ratio of sediment deposition after stopping of spillway operation (r)	%	100	100
Operation frequency of spillway per year (f)	times/year	1.7	1.7
Annual sediments deposit volume in tunnel (Vs = D x Vt x r x f)	ton/year	824	1,004
Annual cost for cleaning and sediment removal of the tunnel	PHP million/year	13.6	16.6

Source: JICA Survey Team

(c) Operation and maintenance cost

Operation and maintenance cost of the proposed Parañaque Spillway, which is composed of operation cost for drainage pumps (fuel, manpower), maintenance cost of hydro-mechanical facilities (repair and replacement), and maintenance cost of underground tunnels (inspection and repairs), is estimated at 0.5% of construction cost and 1.0% of procurement cost of hydro-mechanical facilities. Costs for sediment removal from tunnels and cleaning of tunnels are added, referring to the actual costs for operation and maintenance in tunnel spillways in Japan.

Table 7.6.13 Operation and Maintenance Cost for the Priority Project (Parañaque Spillway)

Item	Cost Items	O&M Cost (PHP million)		Reference
		Route 1	Route 3	
Parañaque Spillway	Operation cost of drainage pump	1.3	1.6	Table 7.6.11
	Maintenance cost of hydro-mechanical facilities	17.9	17.9	1.0% of Mechanical Cost
	Maintenance cost of underground tunnels	142.9	201.8	0.5% of Civil Works Cost
	Sediment removal and cleaning of spillway tunnel	13.6	16.6	Table 7.6.12
	Sub-Total	175.7	237.9	

Source: JICA Survey Team

(8) Frequency of Operation and Maintenance

The frequency of spillway operation is estimated at 1.7 times per year based on the result of spillway operation simulation described in Subsection 3.4.1. On the other hand, drainage, sediment removal and cleaning of the spillway tunnel is assumed to be once per year after the end of rainy season. These works is estimated to take around three (3) months.

In addition, during the spillway operation, i) continuous monitoring of water level and flow condition at inlet facilities and outlet facilities, and ii) periodical removal of garbage at inlet facility are necessary. For the gate opening/closing operation, this is needed at the time of start and end of the spillway operation. The inlet control gate can be operated for discharge control as required.

7.6.2 Monitoring and Measurement

The following monitoring and measurements should be conducted to properly maintain the spillway facilities and to sustain the effects of the project.

[Monitoring]

i) Water level and flow conditions at Laguna de Bay, inlet facility (intake, open channel and intake shaft), outlet facility (outlet shaft, outlet channel, outlet), drainage river channel and river mouth at Manila Bay)

ii) Safety condition of overall facilities and management areas

[Measurement]

iii) Sedimentation in inlet facility (intake, open channel and intake shaft), spillway tunnel, outlet facility (outlet shaft, outlet channel, outlet), drainage / river

iv) Condition of spillway tunnel such as internal water pressure, displacement of inner space of tunnel, internal stress of tunnel lining

v) Condition of vertical shafts such as displacement of inner space of shaft, internal stress of lining wall, settlement

vi) Condition of tunnel route such as settlement of ground, ground water level and replacement of important adjacent structures

7.6.3 Issues and Considerations on Operation, Maintenance and Management

Based on the results of study for operation plan preparation, maintenance and management of the priority project, issues and considerations on the operation, maintenance and management are summarized below:

[Operational Issues and Considerations]

i) Establishment of organization system of operation, maintenance and management

ii) Securing budgetary allocation

iii) Securing human resources

iv) Coordination and cooperation with LLDA and LGUs

[Technical Issues and Considerations]

i) Establishment of methodology and procedure and operation and maintenance

ii) Countermeasures for garbage and sedimentation

iii) Establishment of monitoring and measurement system

iv) Social and environmental considerations on operation, maintenance and management

v) Study on cost reduction of operation, maintenance and management

(1) Establishment of Organization System of Operation, Maintenance and Management

DPWH oversees the planning, designing and construction of large-scale flood control projects in the Metro Manila area. The completed flood control facilities are later transferred to MMDA which conduct the operation and maintenance.

The target area of this project covers the Metro Manila area under the jurisdiction of MMDA and the provinces of Laguna and Rizal outside of the MMDA’s jurisdiction. Therefore, the responsibility for operation and maintenance is shared among several organizations, which is not always effective. In addition, since the proposed measures are large-scale structures, it is but appropriate to establish the project implementation/operation and maintenance system by positioning DPWH at the center.

(2) Securing Budgetary Allocation

For operation and maintenance of the spillway tunnel, costs are needed for pump operation, maintenance and repair of facilities and equipment, and periodical cleaning and removal of sediment from the underground tunnel. The operation and maintenance cost are estimated to be as high as PHP 180 to 240 million per year.

To sustain the function of facilities and equipment and to operate them properly, the budget for operation and maintenance should be secured appropriately. At present, DPWH allocates a special budget for completed foreign-funded projects (around 20 million to 200 million pesos per year). For the proposed Parañaque Spillway Project, the required operation and maintenance cost is considered to be secured under this budgetary framework of the DPWH.

(3) Securing Human Resources

As for human resources for the operation and maintenance of the spillway tunnel, around 10 to 15 technical staff for the continuous operation and maintenance of the inlet and outlet facilities and pumping systems are needed. In addition, special staff and equipment are necessary for periodical cleaning, sediment removal and inspection of the tunnel.

Aside from the budgetary requirement, appropriate technical and skilled staff should be secured. For the operation of the drainage pumping stations, DPWH and MMDA has a lot of staff with abundant experience and knowledge for the existing facilities. It is considered appropriate to designate these skilled staff and to establish the necessary organization.

(4) Coordination and Cooperation with LLDA and LGUs

To carry out the cleaning of shore areas along the Laguna de Bay and Manila Bay, disposal works for removed sediment and garbage, and proper land management in the areas surrounding the facilities, it is essential to coordinate and cooperate with the LLDA and the LGUs. It is likewise appropriate to forge a Memorandum of Agreement describing the duties, responsibilities and budgetary allocation among the facility management office, LLDA and the LGUs.

(5) Establishment of Procedure and Methodology of Operation and Maintenance

This Parañaque Spillway Project will be the first challenge to operate and maintain an underground spillway tunnel in the Philippines. It is, therefore, necessary to establish in detail the methodology, procedures and staff arrangements for gate operation at starting and stopping of the spillway operation; monitoring and measurement during the spillway operation; operation of drainage pumps, and cleaning and sediment removal after the spillway operation.

It would be necessary to transfer Japanese technology on operation and maintenance which have been established through long-term experience with such underground tunnel works and storage facilities in Japan. It is likewise necessary to prepare an operation and maintenance manual and to have Japanese technical advisors to periodically assist in the operation and maintenance works at the site.

(6) Countermeasures for Garbage and Sedimentation

It has been confirmed through the interview survey on the existing pumping stations and data collection survey on tunnel/rivers in Japan that a lot of human resources and costs were required to remove garbage and sediment deposits.

Since it was also confirmed that many garbage and water hyacinths are transported into the proposed intake site, dust remover and trash rack at the intake facility are proposed to be installed to prevent garbage inflow. It is important to conduct periodical cleaning along the shore areas of the Laguna de Bay, and to enable effective removal works at intake facilities.

In addition, another important issue is the removal/reduction of sediment inflow into the spillway tunnel. The proposed dust remover and screen at the intake facility could stop garbage inflow, but not sedimentation. Once sediment enter and deposited in the underground tunnel, the removal cost would be significantly high because of the siphon shape of the tunnel spillway.

(7) Establishment of Monitoring and Measurements System

Continuous monitoring of water level and discharge at each control point will be necessary for operation and maintenance of the proposed spillway facilities. In addition, it is important to periodically conduct measurement to investigate soundness of the structures and stability of the surrounding areas, and to assess profiles of sediment deposition in the intake facilities and tunnel spillway. At the time of the detailed design stage, it is necessary to identify which item should be monitored and measured, and to study the monitoring and measurement systems and the method of recording and assessment/analysis of collected data.

(8) Social and Environmental Consideration on Operation, Maintenance and Management

There is a possibility that water quality in the spillway tunnel would get worse because of inflow of dirty and polluted intake water, or contamination due to long term stagnant water in the tunnel. A study on the treatment of turbid water during drainage operation and deodorization treatment during the ventilation operation will be necessary.

In addition, noise and vibration generated during drainage pump operation and sediment removal would be a serious problem. An operation and maintenance plan should be prepared taking into consideration the social and environmental conditions in the surrounding areas.

(9) Study on Cost Reduction of Operation, Maintenance and Management

As mentioned above, a huge cost will be required for the operation and maintenance of the underground tunnel. The policy on facility design and investment plan in the future should be directed towards cost reduction for operation and maintenance works, such as:

- Optimization of capacity of drainage pump and size of inner space of vertical shaft by detailed study for pump drainage operation.

- Optimization of maintenance frequency of civil facility, mechanical and electrical equipment.
- Study on the automation/labour-saving of works for cleaning and sediment removal from tunnel, tunnel inspection method (development of underwater inspection, etc.).
- Study on the improvement of efficiency of removal of garbage and water hyacinth at inlet facilities.
- Study on the improvement of efficiency of lifting system for removal of sediment at vertical shafts.
- Study on the life extension works of mechanical and electrical equipment.
- Study on the multipurpose use of stored water in the tunnel spillway (supplying water for extinguishing fires or environmental flow, etc.).

7.7 Preliminary Cost Estimate

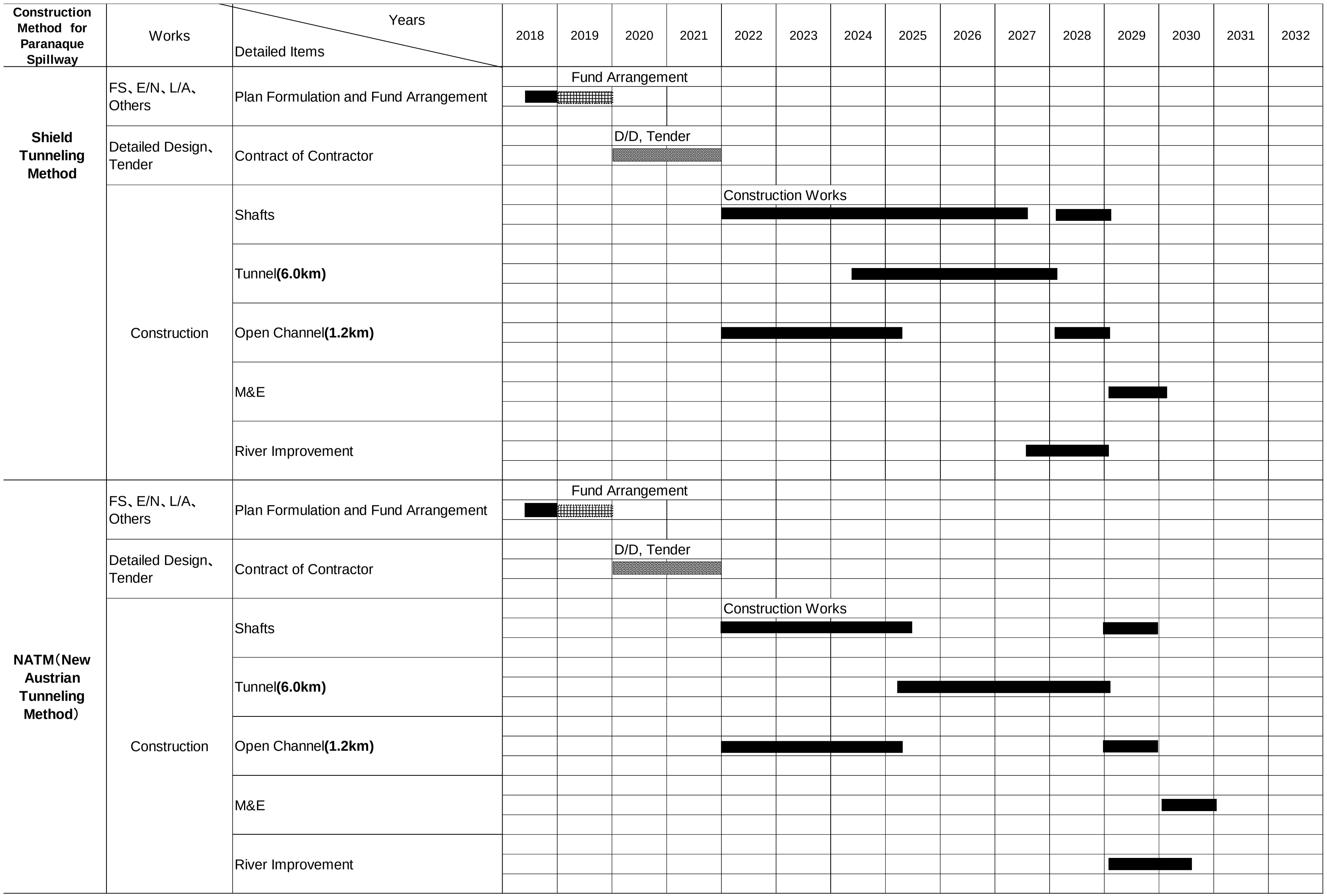
7.7.1 Implementation Schedule

The project implementation schedule for the construction of Parañaque Spillway which is the target structure for the Pre-Feasibility Study is shown in Figure 7.7.1 and Figure 7.7.2. Parañaque Spillway has four options as stated below and the schedules are shown for each route[2].

- The detailed design and procurement of Consultant will proceed concurrently after obtaining the ICC, signing of the Exchange of Notes (E/N) and Loan Agreement (L/A), while procurement of the Contractor will be scheduled in 2021.
- Construction Project: Parañaque Spillway

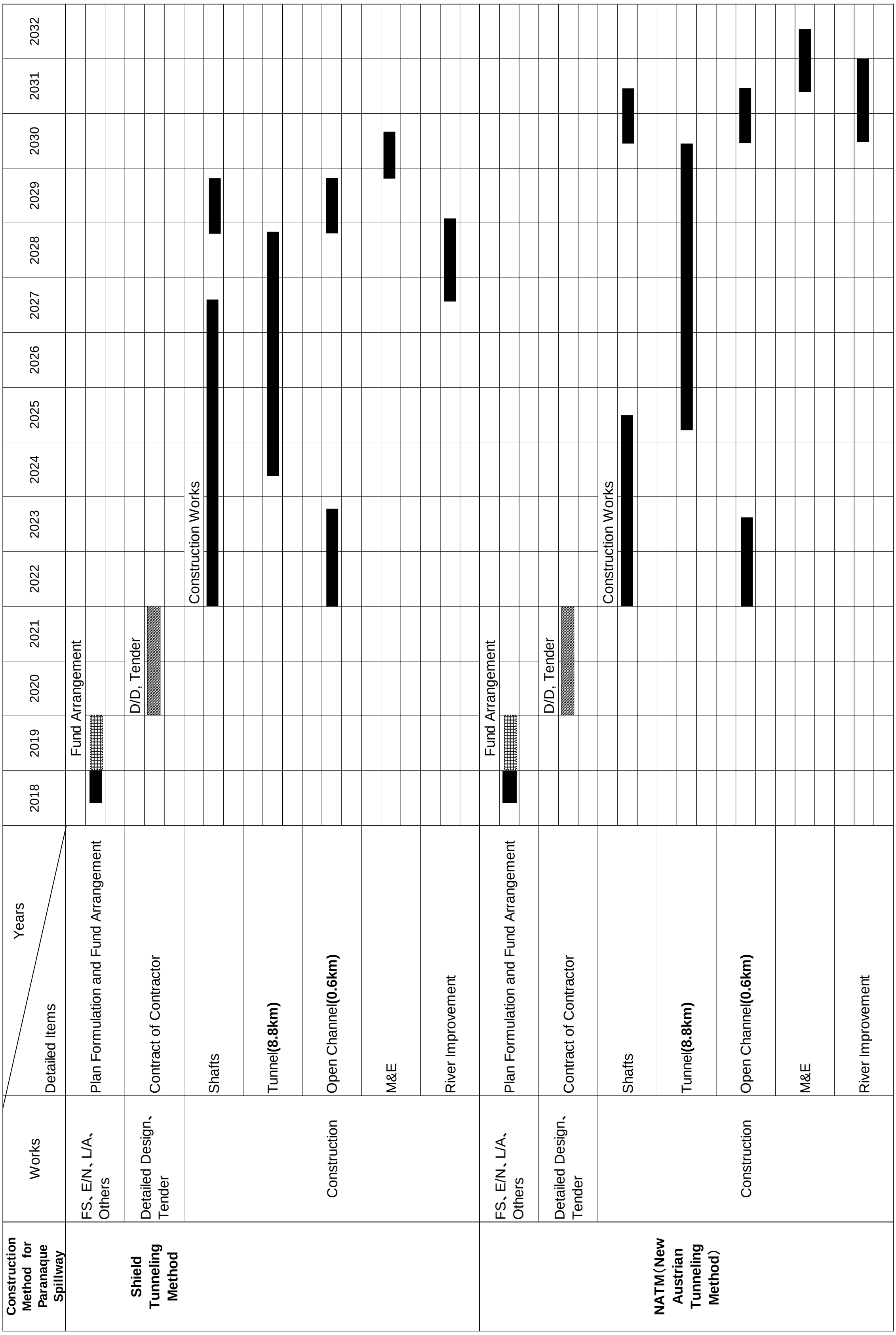
Option 1 : Route 1, Shield Tunnelling Method: January to February 2030
Option 2 : Route 1, NATM: January 2022 to January 2031
Option 3 : Route 3, Shield Tunnelling Method: January 2022 to August 2030
Option 4 : Route 3, NATM: January 2022 to June 2032

[2] The implementation schedule shown here was considered as Pre-F/S level of Paranaque Spillway and differs from the schedule shown in ‘Chapter 5 Comprehensive Flood Management Plan for Laguna de Bay Lakeshore Area’.



Source: JICA Survey Team

Figure 7.7.1 Project Implementation Schedule (Parañaque Spillway：Route 1)



Source: JICA Survey Team

Figure 7.7.2 Project Implementation Schedule (Parañaque Spillway : Route 3)

7.7.2 Preliminary Cost Estimate

(1) Project Cost Items

Items of project cost are as follows:

- Construction Cost
- Engineering Cost (the cost for consulting services)
- Price Escalation
- Contingency

The following are non-eligible items for loan:

- Land acquisition and compensation
- Project administration cost by project implementation body
- TAX (VAT)

(2) Policy on the calculation of construction cost

There are no past experiences on large tunnelling projects in the Philippines. Therefore, the cost estimation will be done by referring to the examples in other countries, including Japan, and the information obtained by hearing survey with Japanese Contractors and Specialist Contractors.

(3) Calculation Conditions of Project Cost

The following conditions were applied to calculate Project Cost.

Table 7.7.1 Calculation Conditions of Project Cost

Items	Conditions	Remarks
Base Year of Cost Estimate	September 2017	
Exchange Rate	1 USD= 110.96 JPY; 1 USD = 50.84 PHP, 1 PHP = 2.183 JPY	Refer to the Exchange Rates from the IMF homepage (Average rate from July 2017 to September 2017)
Engineering Cost	10% of Construction Cost	
Price Escalation	Price Escalation regarding Construction Cost, Engineering Cost: Foreign Currency (F/C): 0.8%, Local Currency (L/C): 1.8%	Refer to the “World Economy Outlook” published in the IMF homepage
Contingency	10% of total amount of construction cost, engineering cost and price escalation	
Land Acquisition and Compensation	Detailed calculation for land acquisition and compensation for building removal (Inclusive of price escalation of 1.8% for LC and also contingency of 10%)	
Administration Cost for Project Implementation Body	2% of total amount of construction cost, engineering cost and cost for land acquisition and compensation	
VAT	12.0%	

Source: JICA Survey Team

(4) Calculation of Project Cost

Project Cost based on the above policy and condition are shown in Table 7.7.2 to Table 7.7.5[3].

As stated in the planning condition of implementation schedules, the following four options for Parañaque Spillway were applied:

Option 1: Route 1, Shield Tunnelling Method
Option 2: Route 1, NATM
Option 3: Route 3, Shield Tunnelling Method
Option 4: Route 3, NATM

Table 7.7.2 Project Cost (Option 1)

Cost Items	Work Items	F/C (million Pesos)	L/C (million Pesos)	Total (million Pesos)
Construction Cost (Route 1, Shield Tunnelling Method)	Tunnel	7,674	10,205	17,879
	Vertical Shafts	8,054	3,886	11,940
	Open Channel	0	4,544	4,544
	River Improvement	0	2,382	2,382
	Surplus Soil Disposal	0	1,828	1,828
	Sub-Total	15,728	22,845	38,573
Engineering Cost		1,929	1,929	3,857
Price Escalation		951	3,070	4,022
Contingency		1,861	2,784	4,645
Land Acquisition and Compensation		0	1,352	1,352
Project Administration Cost		0	1,049	1,049
VAT		0	6,294	6,294
Total (million pesos)		20,469	39,324	59,792

Source: JICA Survey Team

Table 7.7.3 Project Cost (Option 2)

Cost Item	Work Item	F/C (million Pesos)	L/C (million Pesos)	Total (million Pesos)
Construction Cost (Route 1, NATM)	Tunnel	4,204	7,503	11,707
	Vertical Shafts	6,013	3,886	9,899
	Open Channel	0	4,544	4,544
	River Improvement	0	2,382	2,382
	Surplus Soil Disposal	0	1,828	1,828
	Sub-Total	10,217	20,143	30,360
Engineering Cost		1,518	1,518	3,036
Price Escalation		669	2,976	3,645
Contingency		1,240	2,464	3,704
Land Acquisition and Compensation		0	1,352	1,352
Project Administration Cost		0	842	842
VAT		0	5,052	5,052
Total (million Pesos)		13,645	34,346	47,991

Source: JICA Survey Team

[3] The project cost shown here was considered as Pre-F/S level of Paranaque Spillway and differs from the project cost shown in 'Chapter 5 Comprehensive Flood Management Plan for Laguna de Bay Lakeshore Area'.

Table 7.7.4 Project Cost (Option 3)

Cost Item	Work Item	F/C (million Pesos)	L/C (million Pesos)	Total (million Pesos)
Construction Cost (Route 3, Shield Tunnelling Method)	Tunnel	9,295	14,963	24,258
	Vertical Shafts	8,054	3,886	11,940
	Open Channel	0	3,412	3,412
	River Improvement	0	596	596
	Surplus Soil Disposal	0	1,937	1,937
	Sub-Total	17,349	24,794	42,143
Engineering Cost		2,107	2,107	4,214
Price Escalation		1,079	3,460	4,359
Contingency		2,054	3,036	5,090
Land Acquisition and Compensation		0	1,316	1,316
Project Administration Cost		0	1,146	1,146
VAT		0	6,876	6,876
Total (million Pesos)		22,589	42,735	65,324

Source: JICA Survey Team

Table 7.7.5 Project Cost (Option 4)

Cost Item	Work Item	F/C (million Pesos)	L/C (million Pesos)	Total (million Pesos)
Construction Cost (Route 3, NATM)	Tunnel	5,834	11,005	16,839
	Vertical Shafts	6,013	3,886	9,899
	Open Channel	0	3,412	3,412
	River Improvement	0	596	596
	Surplus Soil Disposal	0	1,937	1,937
	Sub-Total	11,847	20,836	32,683
Engineering Cost		1,634	1,634	3,268
Price Escalation		849	3,370	4,218
Contingency		1,433	2,584	4,017
Land Acquisition and Compensation		0	1,316	1,316
Project Administration Cost		0	910	910
VAT		0	5,460	5,460
Total (million Pesos)		15,763	36,110	51,873

Source: JICA Survey Team

(5) Cost Disbursement Schedule

Cost Disbursement Schedules were considered based on the implementation schedule (four options) from 2020.

Table 7.7.6 Cost Disbursement Schedule (Option 1, Breakdown of Construction Cost)

(Unit: Million of PHP)

Year	Tunnel (Route A Shield)			Vertical Shafts			Open Channel			River Improvement			Surplus Soil Disposal			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2021	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2022	0	0	0	1,164	512	1,677	0	1,049	1,049	0	0	0	0	258	258	1,164	1,819	2,984
2023	0	0	0	1,164	512	1,677	0	1,049	1,049	0	0	0	0	258	258	1,164	1,819	2,984
2024	1,194	1,587	2,781	388	512	901	0	1,049	1,049	0	0	0	0	258	258	1,582	3,407	4,988
2025	2,046	2,721	4,768	1,164	512	1,677	0	350	350	0	0	0	0	258	258	3,211	3,841	7,052
2026	2,046	2,721	4,768	1,164	512	1,677	0	0	0	0	0	0	0	258	258	3,211	3,492	6,703
2027	2,046	2,721	4,768	679	299	978	0	0	0	0	662	662	0	258	258	2,726	3,940	6,666
2028	341	454	795	970	427	1,397	0	874	874	0	1,588	1,588	0	258	258	1,311	3,601	4,912
2029	0	0	0	1,164	512	1,677	0	175	175	0	132	132	0	22	22	1,164	841	2,005
2030	0	0	0	194	85	279	0	0	0	0	0	0	0	0	0	194	85	279
2031	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2032	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Cost	7,674	10,205	17,879	8,054	3,886	11,940	0	4,544	4,544	0	2,382	2,382	0	1,828	1,828	**15,728**	**22,845**	**38,573**

Source: JICA Survey Tea

Table 7.7.7 Cost Disbursement Schedule (Option 1)

(Unit: Million of PHP)

Year	Construction Works (Route A Shield)			Engineering Services			Physical Contingency			Price Escalation			Land Acquisition			Administration Cost			VAT			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	190	190	379	19	19	38	2	3	5	0	670	670	0	22	22	0	131	131	210	1,036	1,246
2021	0	0	0	190	190	379	19	20	39	3	7	10	0	682	682	0	22	22	0	133	133	212	1,054	1,266
2022	1,164	1,819	2,984	190	190	379	139	212	351	33	110	143	0	0	0	0	77	77	0	463	463	1,526	2,871	4,397
2023	1,164	1,819	2,984	190	190	379	140	216	356	44	149	192	0	0	0	0	78	78	0	469	469	1,538	2,921	4,458
2024	1,582	3,407	4,988	190	190	379	184	393	578	72	336	408	0	0	0	0	127	127	0	762	762	2,028	5,214	7,242
2025	3,211	3,841	7,052	190	190	379	357	449	805	167	455	622	0	0	0	0	177	177	0	1,063	1,063	3,924	6,175	10,099
2026	3,211	3,492	6,703	190	190	379	360	417	777	195	490	685	0	0	0	0	171	171	0	1,025	1,025	3,955	5,784	9,740
2027	2,726	3,940	6,666	190	190	379	311	476	787	192	634	825	0	0	0	0	173	173	0	1,039	1,039	3,418	6,452	9,870
2028	1,311	3,601	4,912	190	190	379	161	445	606	112	660	772	0	0	0	0	133	133	0	800	800	1,774	5,829	7,603
2029	1,164	841	2,005	190	190	379	147	123	270	112	201	314	0	0	0	0	59	59	0	356	356	1,613	1,771	3,384
2030	194	85	279	32	32	63	25	14	39	21	25	46	0	0	0	0	9	9	0	51	51	271	217	488
2031	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2032	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Cost	15,728	22,845	38,573	1,929	1,929	3,857	1,861	2,784	4,645	951	3,070	4,022	0	1,352	1,352	0	1,049	1,049	0	6,294	6,294	**20,469**	**39,324**	**59,792**

Source: JICA Survey Tea

Table 7.7.8 Cost Disbursement Schedule (Option 2, Breakdown of Construction Cost)

(Unit: Million of PHP)

Year	Tunnel (Route A NATM)			Vertical Shafts			Open Channel			River Improvement			Surplus Soil Disposal			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2021	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2022	0	0	0	1,077	696	1,773	0	1,049	1,049	0	0	0	0	213	213	1,077	1,958	3,035
2023	0	0	0	1,077	696	1,773	0	1,049	1,049	0	0	0	0	213	213	1,077	1,958	3,035
2024	0	0	0	1,077	696	1,773	0	1,049	1,049	0	0	0	0	213	213	1,077	1,958	3,035
2025	747	1,334	2,081	538	348	886	0	350	350	0	0	0	0	213	213	1,286	2,244	3,530
2026	1,121	2,001	3,122	0	0	0	0	0	0	0	0	0	0	213	213	1,121	2,214	3,335
2027	1,121	2,001	3,122	0	0	0	0	0	0	0	0	0	0	213	213	1,121	2,214	3,335
2028	1,121	2,001	3,122	0	0	0	0	0	0	0	0	0	0	213	213	1,121	2,214	3,335
2029	93	167	260	1,077	696	1,773	0	1,049	1,049	0	1,456	1,456	0	213	213	1,170	3,580	4,750
2030	0	0	0	1,077	696	1,773	0	0	0	0	926	926	0	124	124	1,077	1,747	2,824
2031	0	0	0	90	58	148	0	0	0	0	0	0	0	0	0	90	58	148
2032	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Cost	4,204	7,503	11,707	6,013	3,886	9,899	0	4,544	4,544	0	2,382	2,382	0	1,828	1,828	**10,217**	**20,143**	**30,360**

Source: JICA Survey Tea

Table 7.7.9 Cost Disbursement Schedule (Option 2)

(Unit: Million of PHP)

Year	Construction Works (RouteA NATM)			Engineering Services			Physical Contingency			Price Escalation			Land Acquisition			Administration Cost			VAT			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	137	137	274	14	14	28	1	2	4	0	670	670	0	20	20	0	117	117	152	960	1,112
2021	0	0	0	137	137	274	14	14	28	2	5	7	0	682	682	0	20	20	0	119	119	153	977	1,130
2022	1,077	1,958	3,035	137	137	274	124	221	345	29	115	145	0	0	0	0	76	76	0	456	456	1,368	2,962	4,330
2023	1,077	1,958	3,035	137	137	274	125	225	350	39	155	194	0	0	0	0	77	77	0	462	462	1,379	3,014	4,392
2024	1,077	1,958	3,035	137	137	274	126	229	355	49	195	245	0	0	0	0	78	78	0	469	469	1,390	3,066	4,456
2025	1,286	2,244	3,530	137	137	274	149	265	414	70	269	339	0	0	0	0	91	91	0	547	547	1,642	3,553	5,195
2026	1,121	2,214	3,335	137	137	274	133	266	399	72	313	385	0	0	0	0	88	88	0	527	527	1,463	3,545	5,008
2027	1,121	2,214	3,335	137	137	274	134	271	405	83	361	443	0	0	0	0	89	89	0	535	535	1,475	3,607	5,081
2028	1,121	2,214	3,335	137	137	274	135	276	411	94	409	503	0	0	0	0	90	90	0	543	543	1,487	3,669	5,156
2029	1,170	3,580	4,750	137	137	274	142	444	586	108	726	834	0	0	0	0	129	129	0	773	773	1,557	5,789	7,347
2030	1,077	1,747	2,824	137	137	274	133	229	362	111	408	520	0	0	0	0	80	80	0	477	477	1,458	3,078	4,536
2031	90	58	148	11	11	23	11	9	20	10	17	27	0	0	0	0	4	4	0	26	26	122	125	247
2032	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Cost	10,217	20,143	30,360	1,518	1,518	3,036	1,240	2,464	3,704	669	2,976	3,645	0	1,352	1,352	0	842	842	0	5,052	5,052	**13,645**	**34,346**	**47,991**

Source: JICA Survey Tea

Table 7.7.10 Cost Disbursement Schedule (Option 3, Breakdown of Construction Cost)

(Unit: Million of PHP)

Year	Tunnel (Route D Shield)			Vertical Shafts			Open Channel			River Improvement			Surplus Soil Disposal			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2021	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2022	0	0	0	1,062	512	1,575	0	1,280	1,280	0	0	0	0	250	250	1,062	2,042	3,104
2023	0	0	0	1,062	512	1,575	0	853	853	0	0	0	0	250	250	1,062	1,615	2,677
2024	1,251	2,014	3,266	1,062	512	1,575	0	0	0	0	0	0	0	250	250	2,313	2,777	5,090
2025	2,145	3,453	5,598	1,062	512	1,575	0	0	0	0	0	0	0	250	250	3,207	4,215	7,422
2026	2,145	3,453	5,598	1,062	512	1,575	0	0	0	0	0	0	0	250	250	3,207	4,215	7,422
2027	2,145	3,453	5,598	620	299	918	0	0	0	0	166	166	0	250	250	2,765	4,167	6,932
2028	1,609	2,590	4,199	266	128	394	0	320	320	0	397	397	0	250	250	1,874	3,685	5,559
2029	0	0	0	1,062	512	1,575	0	960	960	0	33	33	0	187	187	1,062	1,693	2,755
2030	0	0	0	797	384	1,181	0	0	0	0	0	0	0	0	0	797	384	1,181
2031	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2032	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Cost	9,295	14,963	24,258	8,054	3,886	11,940	0	3,412	3,412	0	596	596	0	1,937	1,937	**17,349**	**24,794**	**42,143**

Source: JICA Survey Tea

Table 7.7.11 Cost Disbursement Schedule (Option 3)

(Unit: Million of PHP)

Year	Construction Works (RouteD Shield)			Engineering Services			Physical Contingency			Price Escalation			Land Acquisition			Administration Cost			VAT			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	196	196	392	20	20	40	2	4	5	0	652	652	0	22	22	0	131	131	217	1,024	1,242
2021	0	0	0	196	196	392	20	20	40	3	7	10	0	664	664	0	22	22	0	133	133	219	1,042	1,261
2022	1,062	2,042	3,104	196	196	392	129	236	365	30	123	153	0	0	0	0	80	80	0	482	482	1,417	3,159	4,576
2023	1,062	1,615	2,677	196	196	392	130	195	324	41	134	175	0	0	0	0	71	71	0	428	428	1,429	2,640	4,068
2024	2,313	2,777	5,090	196	196	392	261	325	586	102	277	379	0	0	0	0	129	129	0	774	774	2,872	4,478	7,350
2025	3,207	4,215	7,422	196	196	392	357	491	848	167	498	665	0	0	0	0	187	187	0	1,119	1,119	3,927	6,707	10,633
2026	3,207	4,215	7,422	196	196	392	360	500	860	195	587	782	0	0	0	0	189	189	0	1,135	1,135	3,958	6,822	10,780
2027	2,765	4,167	6,932	196	196	392	316	503	819	195	669	864	0	0	0	0	180	180	0	1,081	1,081	3,471	6,797	10,268
2028	1,874	3,685	5,559	196	196	392	222	456	678	154	676	830	0	0	0	0	149	149	0	895	895	2,447	6,057	8,504
2029	1,062	1,693	2,755	196	196	392	136	226	362	104	369	473	0	0	0	0	80	80	0	478	478	1,499	3,041	4,539
2030	797	384	1,181	147	147	294	103	65	168	86	115	202	0	0	0	0	37	37	0	221	221	1,133	969	2,102
2031	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2032	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Cost	17,349	24,794	42,143	2,107	2,107	4,214	2,054	3,036	5,090	1,079	3,460	4,539	0	1,316	1,316	0	1,146	1,146	0	6,876	6,876	**22,589**	**42,735**	**65,324**

Source: JICA Survey Tea

Table 7.7.12 Cost Disbursement Schedule (Option 4, Breakdown of Construction Cost)

(Unit: Million of PHP)

Year	Tunnel(Route D NATM)			Vertical Shafts			Open Channel			River Improvement			Surplus Soil Disposal			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2021	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2022	0	0	0	1,093	707	1,800	0	1,280	1,280	0	0	0	0	204	204	1,093	2,190	3,283
2023	0	0	0	1,093	707	1,800	0	853	853	0	0	0	0	204	204	1,093	1,763	2,857
2024	0	0	0	1,093	707	1,800	0	0	0	0	0	0	0	204	204	1,093	910	2,004
2025	753	1,420	2,173	547	353	900	0	0	0	0	0	0	0	204	204	1,299	1,977	3,277
2026	1,129	2,130	3,259	0	0	0	0	0	0	0	0	0	0	204	204	1,129	2,334	3,463
2027	1,129	2,130	3,259	0	0	0	0	0	0	0	0	0	0	204	204	1,129	2,334	3,463
2028	1,129	2,130	3,259	0	0	0	0	0	0	0	0	0	0	204	204	1,129	2,334	3,463
2029	1,129	2,130	3,259	0	0	0	0	0	0	0	0	0	0	204	204	1,129	2,334	3,463
2030	565	1,065	1,630	547	353	900	0	640	640	0	199	199	0	204	204	1,111	2,461	3,572
2031	0	0	0	1,093	707	1,800	0	640	640	0	397	397	0	102	102	1,093	1,846	2,939
2032	0	0	0	547	353	900	0	0	0	0	0	0	0	0	0	547	353	900
Total Cost	5,834	11,005	16,839	6,013	3,886	9,899	0	3,412	3,412	0	596	596	0	1,937	1,937	**11,847**	**20,836**	**32,683**

Source: JICA Survey Tea

Table 7.7.13 Cost Disbursement Schedule (Option 4)

(Unit: Million of PHP)

Year	Construction Works (RouteD NATM)			Engineering Services			Physical Contingency			Price Escalation			Land Acquisition			Administration Cost			VAT			Total		
	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total	F.C.	L.C.	Sub-Total
2020	0	0	0	131	131	261	13	13	26	1	2	3	0	652	652	0	19	19	0	113	113	145	931	1,076
2021	0	0	0	131	131	261	13	14	27	2	5	7	0	664	664	0	19	19	0	115	115	146	947	1,093
2022	1,093	2,190	3,283	131	131	261	125	245	370	30	128	157	0	0	0	0	81	81	0	489	489	1,379	3,263	4,642
2023	1,093	1,763	2,857	131	131	261	126	203	330	40	140	180	0	0	0	0	73	73	0	435	435	1,390	2,746	4,136
2024	1,093	910	2,004	131	131	261	127	114	241	50	97	147	0	0	0	0	53	53	0	318	318	1,401	1,624	3,025
2025	1,299	1,977	3,277	131	131	261	150	235	385	70	238	308	0	0	0	0	85	85	0	508	508	1,650	3,173	4,823
2026	1,129	2,334	3,463	131	131	261	133	279	412	72	328	400	0	0	0	0	91	91	0	544	544	1,465	3,707	5,172
2027	1,129	2,334	3,463	131	131	261	134	284	419	83	378	461	0	0	0	0	92	92	0	552	552	1,477	3,772	5,249
2028	1,129	2,334	3,463	131	131	261	135	289	425	94	429	523	0	0	0	0	93	93	0	561	561	1,489	3,837	5,326
2029	1,129	2,334	3,463	131	131	261	136	295	431	104	481	586	0	0	0	0	95	95	0	569	569	1,501	3,904	5,405
2030	1,111	2,461	3,572	131	131	261	136	315	451	114	562	676	0	0	0	0	99	99	0	595	595	1,491	4,163	5,654
2031	1,093	1,846	2,939	131	131	261	135	245	379	123	472	595	0	0	0	0	83	83	0	501	501	1,482	3,277	4,759
2032	547	353	900	65	65	131	68	53	121	67	109	176	0	0	0	0	27	27	0	159	159	747	767	1,513
Total Cost	11,847	20,836	32,683	1,634	1,634	3,268	1,433	2,584	4,017	849	3,370	4,218	0	1,316	1,316	0	910	910	0	5,460	5,460	**15,763**	**36,110**	**51,873**

Source: JICA Survey Tea

7.8 Economic Evaluation and Verification of the Project

Economic analysis is implemented to judge the economic viability of the Project. The qualitative benefits which could not be economically quantified, and the operation and effect indicators of the Project are also explained in this Section.

7.8.1 Quantitative Effect (Economic Internal Rate of Return)

(1) General Assumptions of Economic Analysis

General assumptions for the economic analysis are as follows:

- Evaluation period is 60 years, including 10 years of construction (depend on case 8.2 ~ 10.5 years) and 50 years of O&M period.
- Discount Rate (target of EIRR) is set at 10% in accordance with the ICC guideline.
- Costs and benefits are calculated based on the prices in 2017.

(2) Outline of Quantified Costs and Benefits

Main quantified costs and benefits are summarized in Table 7.8.1.

Table 7.8.1 Economic Costs and Economic Benefits

Project Cost	Economic Benefits
(1) Initial Construction Cost (2) O&M Cost / Major Rehabilitation Cost	(1) Reduced Economic Damage induced by Inundation (household assets, commercial/industrial assets, agriculture crops, suspension of economic activities) (2) Increase of Land Price※1 (3) Re-use of Surplus Soil※1

Source: JICA Survey Team
※1：calculate as a reference value

The annual average value of “reduced economic damage induced by inundation” is calculated by multiplying the “avoided damage of assets under different return period cases (2, 3, 5, 10, 20, 30, 50, 100, 200 years)” and “occurrence rate of each case per year”. Reduced damage of more than 200 years of return period is neglected since the value reduces as the probability and damage value become smaller. The benefits of 'increase of land price' and 'Re-use of surplus soil' are calculated as reference values. The O&M costs and economic benefits are assumed to start in the following year after the completion of the construction works.

(3) Economic Cost

The following Standard Conversion Factor and Shadow Wage Rate is used to estimate the economic cost of the Project:

- Standard Conversion Factor (SCF): 0.833 = 1 / Shadow Exchange Rate (1.2)
- Shadow Wage Rate (SWR) for Non-Skilled Labour: 0.6

Based on the above assumptions, economic costs of initial construction cost and O&M cost are estimated in the following items.

(a) Initial Construction Cost

To estimate the economic cost, price escalation and TAX are excluded from the project cost items shown in Subsection 7.7.2.

Labour cost is assumed at 10% of the local currency portion of the Project Cost for spillway. The cost for skilled labour takes 70% of the total labour cost, and the un-skilled labour cost takes the rest (30% of the labour cost).

In conclusion, the disbursement schedule of financial cost and economic cost are as shown in Table 7.8.2.

Table 7.8.2 Financial Costs and Economic Costs under Each Option

Year	Financial Cost (in million Pesos, incl. Price Escalation and TAX)				Economic Cost (in million Pesos)			
	1: Route 1 Shield	2: Route 1 NATM	3: Route 3 Shield	4: Route 3 NATM	1: Route 1 Shield	2: Route 1 NATM	3: Route 3 Shield	4: Route 3 NATM
2020	1,246	1,112	1,242	1,076	964	855	930	827
2021	1,266	1,130	1,261	1,093	975	866	944	837
2022	4,397	4,330	4,576	4,642	3,418	3,342	3,334	3,568
2023	4,458	4,392	4,068	4,136	3,424	3,348	2,989	3,169
2024	7,242	4,456	7,350	3,025	5,406	3,353	5,443	2,364
2025	10,099	5,195	10,633	4,823	7,660	3,866	7,840	3,614
2026	9,740	5,008	10,780	5,172	7,338	3,658	7,945	3,769
2027	9,870	5,081	10,268	5,249	7,231	3,664	7,524	3,775
2028	7,603	5,156	8,504	5,326	5,323	3,670	6,168	3,781
2029	3,384	7,347	4,539	5,405	2,518	5,044	3,321	3,788
2030	488	4,536	2,102	5,654	367	3,184	1,603	3,896
2031	0	247	0	4,759	0	181	0	3,290
2032	0	0	0	1,513	0	0	0	1,097
Total	59,792	47,991	65,324	51,873	44,624	35,031	48,041	37,775

Note: Cost for improvement of downstream river such as Parañaque River is not included
Source: JICA Survey Team

(b) O&M Cost / Rehabilitation Cost

Annual O&M cost including rehabilitation cost is estimated as in Table 7.8.2. The economic cost is obtained by multiplying the average SCF at 0.838 to the financial cost.

Table 7.8.3 Annual O&M Costs of Each Option (Economic Cost)

Item	Cost Items	Financial Cost (PHP million)		Economic Cost (PHP million)	
		Route 1	Route 3	Route 1	Route 3
Parañaque Spillway	Operation cost of drainage pump	1.3	1.6	1.1	1.3
	Maintenance cost of hydro-mechanical facilities	17.9	17.9	15.0	15.0
	Maintenance cost of underground tunnels	142.9	201.8	119.7	169.2
	Sediment removal and cleaning of spillway tunnel	13.6	16.6	11.4	13.9
	Sub-Total	175.7	237.9	147.2	199.4

Source: JICA Survey Team

(4) Quantified Economic Benefits

The economic benefits induced by the Project's implementation are quantified as described below. The same benefit amounts are used under each of the four option cases.

(a) Reduced Economic Damage induced by Inundation

In relation to the calculation of flood damage, there are no guidelines or past detailed damage data in the Philippines. Therefore, the calculation of flood damage is made based on the methodology used in the "Manual for Economic Analysis for Flood Control Project in Japan", issued by the Ministry of Infrastructure, Land and Transportation, Japan, in 2005.

GIS is used for analysing the flood area to count the number of inundated households and enterprises. The base map data for GIS analysis was originally taken from the Landsat 8 Satellite Image data, and the built-up and agricultural areas are recognized automatically by image analysis of 100 meters mesh. The land level data was taken from the IFSAR data provided by NAMRIA. Annual average reduced damage value was calculated utilizing such geographical data, as well as the census data for population and enterprises, estimated inundation depth in each return period, average asset value of households and enterprises, economic value of agricultural field, etc. The data source and detailed methodology are as explained below.

The 31 LGUs, which are located around Laguna Lake and have legislative territory below 14.7 m of land level (maximum water level under 200 years return period), are selected to calculate the economic damage caused by inundation. Lists of target LGUs is attached as Appendix 3-1.

(i) Damage of Household Buildings and Household Assets

"Damage of Household Building" = "Number of Affected Household (affected population / average household size)" x "Value of Household Assets" x "Damage Rate" x 1.2 (including indirect damage)

"Damage of Household Assets" = "Number of Affected Household (affected population / average household size)" x (30% of "Value of Household Assets") x "Damage Rate" x 1.2 (including indirect damage)

Economic damages of household building and assets are estimated by multiplying the number of affected household, analysed by GIS analysis, asset value per household building/assets and assumed damage rate. In addition, as considering the cleaning and rehabilitation works after the inundation, 20% of damaged asset value, which is the commonly adopted percentage for economic analysis, is added as the indirect damage. The calculated economic damage in each LGU under different water level is shown in the Appendix 3-2.

[Number of Affected Households]

For estimating the number of affected households, population living at the land level of 12.5 m to

14.7 m above sea level is calculated by GIS analysis for every 10cm in height. The population data is quoted from the 2015 census at barangay level, and the one in 2017 is assumed by using the projected population growth rate per province provided by the Philippine Statistics Authority (PSA). For the calculation, the population is assumed to live in the built-up area in each barangay at the same density. The built-up area is recognized by image analysis of Landsat 8 Satellite Image data. The calculated population in every 10 cm land height is divided by the average number of household members of each region (NCR: 4.4; Region IV-A: 4.1) to estimate the affected household number per LGU.

[Value of Household Building and Household Assets]

Value of household asset is estimated by analysing survey result of Consumer Finance Survey issued by Bangko Sentral ng Pilipinas in 2014. Building values in target 31 LGUs around Laguna Lake are selected from interview samples and average value is calculated. The annual CPI is applied on the estimated value to convert them into the price in 2017. The value of household assets are assumd to be 30% of household building considering the assumption used in two studies of “Preparatory survey for Cavite industrial area flood risk management project (2017)”and “Pasig-Marikina river channel improvement project, Phase IV (2017)”

Table 7.8.4 Estimated Value of Household Building and Household Assets (PHP)

Area	Number of Samples	Average Value of Household Building (PHP)		Value of Household Assets (PHP)
		Price in 2013	Price in 2017	Price in 2017
NCR	245	776,862	834,837	250,451
Laguna	267	529,166	568,656	170,597
Rizal	227	459,195	493,464	148,039

Source: Consumer Finance Survey, 2014, Bangko Sentral ng Pilipinas

[Damage Rate of Household Buildings]

Damage rate referred from the Japanese manual as the one in the Philippines is not available. In the manual, different damage rates are set depending on the inclination angle. The lowest rates, which are given on lower than 1/1000 of inclination angle, are used for the calculation based on the principle of conservatism for economic analysis.

Table 7.8.5 Damage Rate of Household Building and Household Assets

Inundation Depth	0.15 m-0.5 m	0.5 m - 1.0 m	1.0 m - 2.0 m	2.0 m - 3.0 m	> 3.0 m
Household Building	0.092	0.119	0.266	0.580	0.834
Household Assets	0.145	0.326	0.508	0.928	0.991

Source: Manual for Economic Analysis for Flood Control Projects in Japan, Ministry of Infrastructure, Land and Transportation, 2005s

(ii) Damage of Industrial and Commercial Assets

“Number of Affected Enterprises” x “Value of Industrial/Commercial Assets” x “Damage Rate” x 1.2 (including indirect damage)

Economic damage of industrial and commercial assets is obtained by multiplying the number of affected enterprise, asset value per enterprise, and damage rate. Moreover, considering the damage of cleaning and rehabilitation activities, the indirect cost, 20% of asset damage is added. The estimated damage value of industrial and commercial assets of different water level in each LGU is shown in the Appendix 3-3.

[Number of Affected Enterprises]

For estimating the number of affected enterprises, area of built-up area is analysed for every 10 cm from 12.5 m to 14.7 m above sea level. The built-up area is made by image analysis of Landsat 8 Satellite Image data. Number of enterprises per industrial category is quoted from the Annual Survey of the Philippines Business and Industry, 2015 (PSA), and these enterprises are assumed to locate over the built-up area in each LGU at the same density. Counted number of enterprises per industrial category are shown in Appendix 3-4.

[Value of Commercial Assets]

Three kinds of asset values of enterprises, value of building, depreciable asset and stocks, were quoted from the Annual Survey of Philippine Business and Industry, 2014 (PSA). The price is converted to the value in 2017 reflecting the past CPI.

Table 7.8.6 Average Asset Value of Enterprise per Industrial Category

(Unit: PHP)

Category	Price in 2014				Price in 2017
	Building	Depreciable	Stock	Total	Total
Manufacturing	13,639,250	25,984,837	39,126,669	78,750,756	94,815,910
Constructions	6,475,804	26,772,455	16,445,488	49,693,747	59,831,271
Wholesale and Retail Trade	95,867	99,239	6,134,050	6,329,156	7,620,304
Transportation and Storage	8,475,507	5,278,713	2,781,415	16,535,635	19,908,905
Accommodation and Food Service Activities	245,604	199,888	592,347	1,037,839	1,249,558
Financial and Communication	1,478,329	1,975,329	6,615,410	10,069,068	12,123,158
Real Estate Activities	6,403,978	869,013	86,511,338	93,784,329	112,916,332
Education	557,312	268,954	85,348	911,614	1,097,583
Human Health and Social Work Activity	881,656	843,548	1,222,959	2,948,163	3,549,588
Other Servicer Activities	18,713	20,079	194,245	233,037	280,577

Source: Annual Survey of Philippine Business and Industry, 2014 (PSA)

[Damage Rate]

Damage rate quoted from the Japanese manual as the data in the Philippines is not available. The lowest damage rate is chosen for estimating the damage amount of building asset, which varies depending on the inclination angle of the location.

Table 7.8.7 Damage Rate of Enterprises

Inundation Depth	0.15 - 0.5m	0.5 - 1.0m	1.0 - 2.0m	2.0 - 3.0m	> 3.0m
Damage Rate of Building	0.092	0.119	0.266	0.580	0.834
Damage Rate of Depreciable Asset	0.232	0.453	0.789	0.966	0.995
Damage Rate of Stocks	0.128	0.267	0.586	0.897	0.982

Source: Manual for Economic Analysis for Flood Control Projects, Ministry of Infrastructure, Land and Transportation, Japan (2005)

(iii) Damage of Infrastructure Facilities

"Damage of Infrastructure Facilities" = 65% x "Damage of Household building, household assets and commercial assets"

The past damage data of infrastructure facilities caused by inundations in the Philippines is not sufficient to estimate the economic damage. In the Japanese manual, the economic damage ratio compared to the direct damage of general assets is estimated based on the historical damage values caused by inundation in Japan. The economic values of damaged infrastructures of roads, bridges, sewerage and urban facilities corresponds to 65.4% (61.1%, 3.7%, 0.4% and 0.2%, respectively) of direct damage of general assets. Assuming the situation is similar in the Philippines, the economic damage of infrastructure facilities is estimated at 65% of direct damage of household building, household assets and commercial assets.

(iv) Damage to Agricultural Crops

Damage to agricultural crops (Paddy, Maize, commercial crops) is estimated as follows:

"Damage of Agricultural Crops" = "Affected Agricultural Area" x "Economic Value of Agricultural Crops per m2" x "Damage Rate"

[Affected Agricultural Area]

Affected agricultural area in each LGU is estimated by GIS analysis from 12.5 m to 14.7 m above sea level by every 10 cm of height. The agricultural land is recognized automatically by image analysis of the Landsat 8 Satellite Image. There is a difference between the agricultural land in the GIS and statistical data, so that the total area recognized by GIS analysis is adjusted to match the total area of the statistical data in the calculation.

Produced crops are assumed to be paddy, maize and 11 kinds of other major commercial crops (coconut, coffee, banana, calamansi (lemon), mango, pineapple, sweet potatoes, cassava, eggplant, peanut, tomato). Cultivated area and total yield of each crop are referred from the agricultural census data called "Major Crops Statistics of the Philippines, 2010-2014", as of year 2014, as shown in Appendix 3-5. In the statistical data, there is no agricultural area in NCR; therefore, benefit is not added in the NCR region.

[Economic Value of Agricultural Crops per "m^2"]

Using the mentioned statistical data, the total produced value is calculated by multiplying the total yield and economic value of crops. The average economic value of agricultural land is estimated by dividing the said total value by total agricultural area.

The future crop price is quoted from the most reliable projection data called "Commodity Market Outlook" issued by the World Bank as of April 2017. In the document, price of paddy and maize is forecast as of the year 2030. The economic values of paddy and maize are calculated by assuming that the crops are imported to the Philippines, considering the transportation cost and margins of wholesale companies, etc. As a result, economic costs of paddy and maize become 10.46 peso/kg and 9.32 peso/kg, respectively. The calculation process of these prices is shown in Appendix 3-6.

Since there is no reliable projected future prices of 11 commercial crops, the current farm-gate price of each crop in 2017 is quoted from the Homepage of PSA[4]. The quoted values are shown in Appendix 3-5.

In conclusion, the economic value of agricultural land including paddy, maize and 11 other commercial crops became 2.59 Peso/m^2 and 3.49 Peso/m^2 in Laguna Province and Rizal Province, respectively.

[Damage Rate]

In the Japanese manual, the damage rate of agricultural crops is determined by inundation depth (less than 0.5 m, 0.5 to 0.99 m, more than 1.0 m) and inundation period (1 to 2 days, 3 to 4 days, 4 to 6 days, more than 7 days). The rate of more than 7 days inundated is set at 0.74 to 1.00. It is difficult to closely estimate the inundated depth and period by GIS analysis. Also, the inundated period in the study area generally lasts for 1 week to several months as described in Section 3.4. Therefore, the damage rate of 1.0 is used for the calculation.

(v) Avoided Economic Loss of Suspended Business Activities

The economic loss of With-Project and Without-Project cases are calculated under each return period by the formula below, and the difference is considered as the economic benefit.

"Economic Loss of Suspended Business Activities" = "Number of Affected Enterprises" x "Period of Suspension" x "Average Daily Added Value per Enterprise"

[Number of Affected Enterprises]

The number of affected enterprises is calculated for the estimation of damage to commercial assets by inundation, and the same number is used. Estimation of suspended period of economic activities

4 http://countrystat.psa.gov.ph/?cont=12&pageid=59555B5A1D737166767A78671D006D67661D066C727560797576607718026B7273667F707566721D4C595E

under several return periods are made for every 50 cm, and the number of enterprises are counted for the water levels of 12.5 m, 13.0 m, 13.5 m, 14.0 m and 14.5 m.

[Period of Suspension]

As stipulated in Section 3.4, “Runoff and Inundation Analysis and Laguna de Bay Water Level Fluctuation Analysis”, water level is predicted in With- and Without-Project cases under several return period situations. The difference of suspended period is estimated for every 50 cm of water level as shown in Table 7.8.8, Table 7.8.9 and Table 7.8.10.

Table 7.8.8 Suspended Period of Business Activities under Without-Project Situation

(Unit: days)

Water Level	Return Period						
(EL. m)	5 years	10 years	20 years	30 years	50 years	100 years	200 years
>14.5	0	0	0	0	0	0	19
>14.0	0	0	0	0	2	28	66
>13.5	0	0	16	21	63	71	89
>13.0	0	23	68	71	88	99	114
>12.5	62	74	97	103	116	124	141

Source: JICA Survey Team

Table 7.8.9 Suspended Period of Business Activities under With-Project Situation

(Unit: days)

Water Level	Return Period						
(EL. m)	5 years	10 years	20 years	30 years	50 years	100 years	200 years
>14.5	0	0	0	0	0	0	0
>14.0	0	0	0	0	0	0	17
>13.5	0	0	0	0	13	23	43
>13.0	0	2	20	23	33	53	68
>12.5	18	26	49	53	70	79	93

Source: JICA Survey Team

Table 7.8.10 Reduced Suspended Period of Business Activities by the Project

(Unit: days)

Water Level	Return Period						
(EL. m)	5 years	10 years	20 years	30 years	50 years	100 years	200 years
>14.5	0	0	0	0	0	0	19
>14.0	0	0	0	0	2	28	49
>13.5	0	0	16	21	50	48	46
>13.0	0	21	48	48	55	46	46
>12.5	44	48	48	50	46	45	48

Source: JICA Survey Team

[Average Daily Added-Value per Enterprise]

The average daily added-value per enterprise in several sectors is quoted from the national average figures of “2014, Annual Survey of Philippine Business and Industry”, issued by PSA, and converted to the price in 2017 by multiplying with the GDP growth rate.

Table 7.8.11 Average Daily Added-Value per Industrial Category

(Unit: peso/days)

Category	Price in 2014	Price in 2017
Manufacturing	126,408	152,195
Constructions	200,416	241,301
Wholesale and Retail Trade	9,813	11,815
Transportation and Storage	166,252	200,167
Accommodation and Food Service Activities	13,203	15,896
Financial and Communication	199,751	240,500
Real Estate Activities	97,425	117,300
Education	22,951	27,633
Human Health and Social Work Activity	26,772	32,233
Other Servicer Activities	4,345	5,231

Source: Annual Survey of Philippine Business and Industry, (PSA, 2014)

(vi) Calculation of Annual Average Reduced Damage

As previously explained in this section, the damage due to the inundation of household assets, commercial assets, agricultural crops and the suspension of business are estimated separately for each LGU under different water levels of 12.5 m to 14.7 m above sea level. In connection with Section 3.4, "Runoff and Inundation Analysis and Laguna de Bay Water Level Fluctuation Analysis", the water level of Laguna de Bay under With-Project and Without-Project situations of 2, 3, 5, 10, 20, 30, 50, 100 and 200-year return periods are as shown in Table 7.8.12. The summed damage value corresponding to the water level is shown in the same table. The calculated Annual Average Reduced Damage in Taytay (urban area), Lumban (rural area) and the Total Value in 31 LGUs are shown in the tables below.

The difference between the damage value under With-Project and Without-Project situations is multiplied with the probability, and the Annual Average Benefit Amount in 31 LGUs becomes 3,314 million pesos.

Table 7.8.12 Calculation of Annual Average Reduced Damage (Taytay)

(Unit: peso)

Return Period	Water Level (m)			Damage Value			Suspention of Business (b)	Total Economic Loss (c)=(a)+(b)	Probability (d)	Probability between two cases (e)	Average Damage of two cases(f)	Annual Economic Loss (e) x (f)
	Without	With	Difference	Without	With	Difference (a)						
200	14.7	14.3	0.4	18,011,585,999	11,165,777,475	6,845,808,524	1,240,383,132	8,086,191,656	0.005	0.00500	6,937,765,845	34,688,829
100	14.3	13.9	0.4	12,147,435,209	7,287,660,072	4,859,775,136	929,564,898	5,789,340,035	0.010	0.01000	5,042,174,371	50,421,744
50	14.0	13.7	0.3	8,292,745,276	4,792,869,127	3,499,876,149	795,132,558	4,295,008,707	0.020	0.01333	3,677,581,127	49,034,415
30	13.7	13.4	0.3	5,440,492,323	2,899,495,002	2,540,997,321	519,156,225	3,060,153,547	0.033	0.01667	2,740,970,480	45,682,841
20	13.6	13.4	0.2	4,399,441,131	2,447,691,137	1,951,749,993	470,037,420	2,421,787,413	0.050	0.05000	1,829,964,530	91,498,227
10	13.2	13.0	0.2	1,791,289,492	784,978,854	1,006,310,638	231,831,010	1,238,141,647	0.100	0.10000	794,837,186	79,483,719
5	12.9	12.8	0.1	584,328,737	372,999,453	211,329,284	140,203,440	351,532,725	0.200	0.13333	251,418,699	33,522,493
3	12.6	12.5	0.1	151,304,673		151,304,673	0	151,304,673	0.333	0.16667	75,652,336	12,608,723
2	12.3	12.3	0.0					0	0.500	0.50000	0	0
												396,940,990

Source: JICA Survey Team

Table 7.8.13 Calculation of Annual Average Reduced Damage (Lumban)

(Unit: peso)

Return Period	Water Level			Damage Value			Suspention of Business (b)	Total Economic Loss (c)=(a)+(b)	Probability (d)	Probability between two cases (e)	Average Damage of two cases(f)	Annual Economic Loss (e) x (f)
	Without	With	Difference	Without	With	Difference (a)						
200	14.7	14.3	0.4	3,613,359,884	2,106,178,335	1,507,181,550	202,427,517	1,709,609,066	0.005	0.00500	1,485,961,667	7,429,808
100	14.3	13.9	0.4	2,336,788,193	1,194,738,516	1,142,049,677	120,264,590	1,262,314,267	0.010	0.01000	1,044,673,584	10,446,736
50	14.0	13.7	0.3	1,416,853,030	660,170,751	756,682,279	70,350,622	827,032,901	0.020	0.01333	622,027,481	8,293,700
30	13.7	13.4	0.3	761,468,040	385,595,695	375,872,345	41,149,716	417,022,061	0.033	0.01667	372,188,993	6,203,150
20	13.6	13.4	0.2	592,376,786	301,511,336	290,865,449	36,490,476	327,355,926	0.050	0.05000	223,583,359	11,179,168
10	13.2	13.0	0.2	194,795,328	91,476,074	103,319,254	16,491,538	119,810,792	0.100	0.10000	83,509,805	8,350,980
5	12.9	12.8	0.1	68,791,439	31,991,857	36,799,582	10,409,235	47,208,817	0.200	0.13333	29,032,482	3,870,998
3	12.6	12.5	0.1	10,856,147		10,856,147	0	10,856,147	0.333	0.16667	5,428,074	904,679
2	12.3	12.3	0.0					0	0.500	0.50000	0	0
												56,679,219

Source: JICA Survey Team

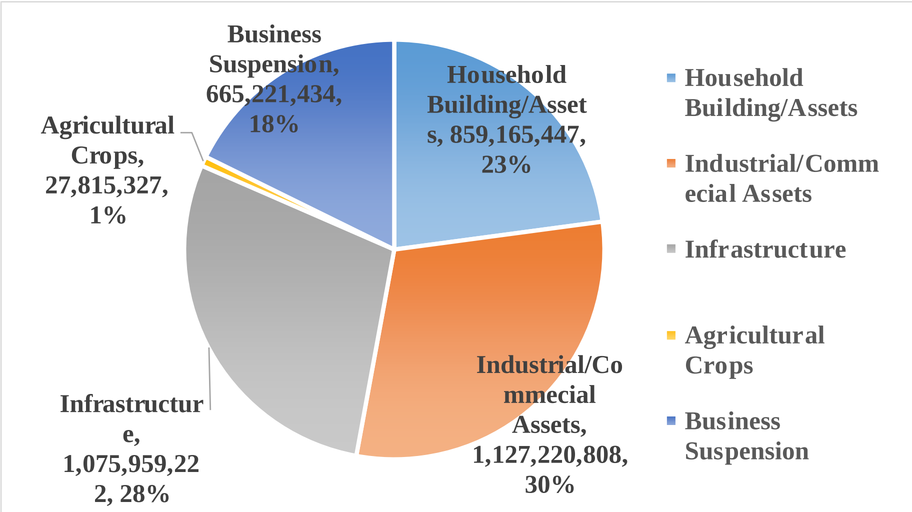
Table 7.8.14 Calculation of Annual Average Reduced Damage in 31 Target LGUs

(Unit: peso)

Return Period	Water Level (m)			Damage Value			Suspention of Business (b)	Total Economic Loss (c)=(a)+(b)	Probability (d)	Probability between two cases (e)	Average Damage of two cases(f)	Annual Economic Loss (e) x (f)
	Without	With	Difference	Without	With	Difference (a)						
200	14.7	14.3	0.4	171,900,856,031	109,151,662,797	62,749,193,234	11,139,603,814	73,888,797,048	0.005	0.00500	63,782,454,775	318,912,274
100	14.3	13.9	0.4	118,748,667,980	73,389,780,852	45,358,887,128	8,317,225,375	53,676,112,502	0.010	0.01000	49,105,402,872	491,054,029
50	14.0	13.7	0.3	84,024,543,627	46,380,838,792	37,643,704,835	6,890,988,407	44,534,693,242	0.020	0.01333	36,333,566,423	484,447,552
30	13.7	13.4	0.3	52,754,246,724	29,281,697,620	23,472,549,104	4,659,890,500	28,132,439,604	0.033	0.01667	25,111,316,517	418,521,942
20	13.6	13.4	0.2	42,533,334,981	24,731,047,902	17,802,287,079	4,287,906,351	22,090,193,430	0.050	0.05000	16,573,312,168	828,665,608
10	13.2	13.0	0.2	18,139,247,230	9,166,508,290	8,972,738,939	2,083,691,967	11,056,430,906	0.100	0.10000	7,585,214,628	758,521,463
5	12.9	12.8	0.1	6,721,906,318	3,683,561,418	3,038,344,900	1,075,653,450	4,113,998,350	0.200	0.13333	2,660,308,555	354,707,807
3	12.6	12.5	0.1	1,206,618,760	0	1,206,618,760	0	1,206,618,760	0.333	0.16667	603,309,380	100,551,563
2	12.3	12.3	0.0	0	0	0	0	0	0.500	0.50000	0	0
												3,755,382,239

Source: JICA Survey Team

Breakdown of the annual average reduced damage amount in the study area for 31 LGUs is shown in the figure below. Out of total value, damage to household building and assets takes 23%, damage to industrial asset takes 30%, damage to infrastructure facilities takes 28%, damage to agricultural crops takes 1%, and the rest 18% is the economic loss caused by the suspension of businesses. The benefit amounts per LGU and compositions of each benefit item are shown in Table 7.8.15.



Source: JICA Survey Team

Figure 7.8.1 Composition of Average Annual Damage Reduction

Table 7.8.15 Composition of Average Annual Damage Reduction of 31 LGUs

No	Province	LGU	Household Building/ Assets	Industrial/ Commecial Assets	Infrastructure	Agricultural Crops	Business Suspension	Total
1	Metro Manila	Taguig	99,350,735	1,774,335	54,776,079	0	1,094,167	156,995,316
2	Metro Manila	Muntinlupa	80,334,164	87,202,387	90,748,965	0	54,654,272	312,939,788
3	Laguna	San Pedro	42,602,467	56,464,823	53,661,449	2,830	24,954,162	177,685,731
4	Laguna	Binan	119,466,935	100,139,612	118,953,546	128,943	48,639,920	387,328,958
5	Laguna	Santa Rosa	41,772,358	35,631,759	41,927,230	172,600	18,565,151	138,069,098
6	Laguna	Cabuyao	66,892,144	124,702,667	103,780,523	545,025	56,734,848	352,655,208
7	Laguna	Calamba	35,587,483	45,094,008	43,702,474	2,400,612	19,707,653	146,492,232
8	Laguna	Los Banos	15,504,072	23,855,050	21,319,524	245,211	24,928,487	85,852,344
9	Laguna	Bay	20,987,869	42,185,169	34,218,729	1,513,622	44,947,470	143,852,858
10	Laguna	Calauan	287,423	493,915	423,225	1,042,017	168,656	2,415,235
11	Laguna	Victoria	22,338,743	24,830,091	25,549,785	2,565,220	15,377,353	90,661,191
12	Laguna	Pila	14,841,293	34,332,938	26,636,042	2,494,467	10,015,140	88,319,880
13	Laguna	Santa Cruz	56,225,646	65,559,841	65,967,138	3,366,134	46,814,696	237,933,455
14	Laguna	Pagsanjan	1,943,227	5,056,144	3,791,326	489,936	2,466,196	13,746,828
15	Laguna	Lumban	13,148,126	17,346,479	16,517,911	3,153,020	6,513,682	56,679,219
16	Laguna	Kalayaan	388,074	705,656	592,437	557,724	136,736	2,380,627
17	Laguna	Paete	15,255,641	68,462,406	45,347,275	362,161	33,484,839	162,912,322
18	Laguna	Pakil	2,391,071	7,353,755	5,278,447	292,370	5,994,161	21,309,804
19	Laguna	Pangil	5,480,874	5,348,696	5,866,017	1,199,618	2,276,844	20,172,048
20	Laguna	Siniloan	9,689,913	35,367,969	24,406,353	2,258,473	14,027,221	85,749,929
21	Laguna	Famy	1,157,775	620,714	963,348	616,353	245,169	3,603,360
22	Laguna	Mabitac	4,896,231	2,821,100	4,180,221	2,778,116	1,301,779	15,977,445
23	Rizal	Jalajala	7,819,132	7,775,085	8,446,868	95,566	7,887,622	32,024,274
24	Rizal	Pililia	6,223,938	19,061,025	13,696,021	359,971	11,879,502	51,220,457
25	Rizal	Tanay	30,401,465	31,296,416	33,419,685	197,078	21,818,151	117,132,795
26	Rizal	Baras	5,642,459	4,186,599	5,324,073	177,774	1,320,982	16,651,887
27	Rizal	Morong	7,063,277	22,335,464	15,924,318	416,849	14,993,833	60,733,742
28	Rizal	Cardona	5,134,727	13,149,604	9,904,012	167,133	13,112,357	41,467,832
29	Rizal	Binangonan	39,124,649	45,065,152	45,602,809	124,838	58,961,528	188,878,976
30	Rizal	Angono	27,420,415	51,032,614	42,495,391	29	25,649,963	146,598,412
31	Rizal	Taytay	59,793,123	147,969,337	112,537,999	91,638	76,548,892	396,940,990
Total			859,165,447	1,127,220,808	1,075,959,222	27,815,327	665,221,434	3,755,382,239
Share			23%	30%	29%	1%	18%	100%

Source: JICA Survey Team

The value is calculated by the price level in 2017. On the assumption that the number of households and enterprises increase faster than the growth rate of population, the estimated benefit is assumed to increase in relation to population growth rate per province during the evaluation period. The assumed growth rate per province and projected annual benefit amount per year is shown in the Appendix 3-7.

(b) Benefit of Increase of Land Price

Future land price increase caused by the Project's implementation is predicted as follows. Since the benefit of increase of land prices has a wide range of assumptions, it is calculated here as a reference value.

"Benefit of Increase of Land Price" = "Influenced Area" x "Current Market Value of Land" x "Increase Rate of Land Value"

[Influenced Area]

In consideration of the predicted inundation area and the opinion of real estate companies, the land price in half of the area between the land elevation of 12.5m and 14.7m, which is 38.2 km^2, is assumed to increase.

[Current Market Value of Land]

The average zonal value in target areas in Laguna, Rizal and NCR are 1,514, 1,223 and 25,740 pesos/m^2, respectively, with reference to the Zonal Value data of the Bureau of Internal Revenue (BIR). Market values are estimated at 120% of Average Zonal Value, which are 1,817, 1,467 and 30,888 pesos/m^2 in Laguna, Rizal and NCR provinces, respectively.

[Increase Rate of Land Value]

In consideration of the opinion of real estate companies and the past report assumptions of the World Bank, the value of land protected by the Parañaque Spillway is considered to increase by 15% during the first 10 years.

(c) Re-use of Surplus Soil

As described in Subsection 4.2.1, Procurement Plan, there is an optional plan to reuse the excavated soil for the reclamation of Laguna de Bay to create a new land. The economic value of the reclaimed land is estimated at 9,000 million pesos by the formula described below. The additional financial cost is estimated at 2,300 million pesos, which corresponds to 1,930 million pesos of economic cost.

"Benefit of Reuse of Surplus Soil" = "Area of Reclamation" x "Market Value of Reclamation Land"

[Area of Reclamation]

As stipulated in Subsection 4.2.1, the area of new reclamation land is estimated at 45 hectares (450,000 m^2).

[Market Value of Reclamation Land]

As prescribed, the Average Zonal Value in NCR is 25,740 pesos/m^2. The market value is assumed at

120% of the Zonal Value for the estimation, which is 30,888 pesos/m^2. Taking this figure into account, the economic value of reclamation land is set at 20,000 pesos/m^2 on the conservative side.

(5) Results of Economic Analysis

The above mentioned economic costs and economic benefits are recorded in the calculation table, and the EIRR, B/C, NPV are calculated as summarized in the Table 7.8.16. Calculation tables under each case is added in Appendix 3-8.

The economic costs vary under each four cost options.

Cost Option 1: Spillway (Route1, Shield)
Cost Option 2: Spillway (Route1, NATM)
Cost Option 3: Spillway (Route3, Shield)
Cost Option 4: Spillway (Route3, NATM)

As described, the following economic benefits are quantified.

Benefit (1): Reduced Economic Damage induced by Inundation
Benefit (2): Increase of Land Price (Reference Value)
Benefit (3): Re-use of Surplus Soil (Reference Value)

Out of the above three benefits, it is not easy to judge if the benefit (2), future land price increase in the target area, is really made by the effect of the project or other reasons. Also, the method of disposal of surplus soil is not determined, and it is unknown if benefit (3) could be made. Therefore, the economic analysis is made under different 3 benefit cases depending on which benefit is included in the calculation.

Benefit Case 1: Includes only Benefit (1)
Benefit Case 2: Includes Benefit (1) and Benefit (2)
Benefit Case 3: Benefit (1), Benefit (2) and Benefit (3)

Table 7.8.16 Result of Economic Analysis under Each Cost Option and Benefit Case

Benefit Case	Cost Option	EIRR	B/C	NPV (PHP million)
Benefit Case 1	Option 1 (Route 1, Shield)	9.1%	0.87	-3,181
	Option 2 (Route 1, NATM)	10.4%	1.06	1,109
	Option 3 (Route 3, Shield)	8.3%	0.76	-6,279
	Option 4 (Route 3, NATM)	9.6%	0.95	-1,062
Benefit Case 2 (Reference Value)	Option 1 (Route 1, Shield)	10.1%	1.02	420
	Option 2 (Route 1, NATM)	11.5%	1.23	4,383
	Option 3 (Route 3, Shield)	9.2%	0.89	-3,005
	Option 4 (Route 3, NATM)	10.6%	1.10	1,914
Benefit Case 3 (Reference Value)	Option 1 (Route 1, Shield)	10.7%	1.11	2,511
	Option 2 (Route 1, NATM)	12.3%	1.38	6,474
	Option 3 (Route 3, Shield)	9.7%	0.96	-914
	Option 4 (Route 3, NATM)	11.4%	1.22	4,005

Source: JICA Study Team

Under benefit case1 where only the reduced damage of assets by inundation is included, the EIRR value under Cost Option 2 surpasses 10%. Under benefit case 2 and 3, EIRR values of Cost Option1, Option2 and Option4 become higher than 10%, which is the criteria of justification of economic viability of the Project.

Considering the other qualitative benefits which the impact on the society is not quantified in economic value (summarized in the Subsection 7.8.2), the project is thought to be at the economically viable level under every option.

(6) Sensitivity Analysis

To understand the impact of external factors on the economic conditions of the Project, the sensitivity analysis is conducted by assuming the following three cases.

Sensitivity Case

Sensitivity Case1: 10% increase of the Project cost (initial construction cost, O&M cost)
Sensitivity Case2: 10% decrease of the economic benefits
Sensitivity Case3: Occurrence of both Sensitivity Case1 (Economic Cost +10%) and Sensitivity Case2 (Economic Benefit -10%)

The result of sensitivity analysis is shown below. Detail calculation sheet is attached at Appendix 3-8.

10% cost overrun (sensitivity case1) make the EIRR figure lower by 0.5% to 0.8%. 10% reduction of economic benefits (sensitivity case2) decreases EIRR by 0.6% to 0.9%. The impact of the both cases (under sensitivity case3) is around 1.1% to 1.7% decrease of EIRR value.

As the EIRR values are close to 10%, which is the criteria to be considered economic viable, the prudent control of cost and benefit during construction and O&M phase is expected.

Table 7.8.17 Result of Sensitivity Analysis

Benefit Case	Cost Option	Base Case	Sensitivity Case1: Project Cost +10%	Sensitivity Case2: Economic Benefits －10%	Sensitivity Case3: Cost +10% benefit－10%
Benefit Case 1	Option1 (Route 1, Shield)	9.1%	8.5%	8.4%	7.8%
	Option2 (Route 1, NATM)	10.4%	9.7%	9.7%	9.1%
	Option3 (Route 3, Shield)	8.3%	7.7%	7.7%	7.2%
	Option4 (Route 3, NATM)	9.6%	9.1%	9.0%	8.4%
	Difference between Base Case	-	－0.5~－0.7%	－0.6~－0.7%	－1.1~－1.3%
Benefit Case 2 (Reference Value)	Option1 (Route 1, Shield)	10.1%	9.4%	9.4%	8.7%
	Option2 (Route 1, NATM)	11.5%	10.8%	10.7%	10.0%
	Option3 (Route 3, Shield)	9.2%	8.5%	8.5%	7.9%
	Option4 (Route 3, NATM)	10.6%	10.0%	9.9%	9.3%
	Difference between Base Case	-	－0.6~－0.7%	－ 0.7~－0.8%	－1.3~－1.5%

Benefit Case	Cost Option	Base Case	Sensitivity Case1: Project Cost +10%	Sensitivity Case2: Economic Benefits −10%	Sensitivity Case3: Cost +10% benefit−10%
Benefit Case 3 (Reference Value)	Option1 (Route 1, Shield)	10.7%	10.0%	9.9%	9.1%
	Option2 (Route 1, NATM)	12.3%	11.5%	11.4%	10.6%
	Option3 (Route 3, Shield)	9.7%	9.0%	9.0%	8.3%
	Option4 (Route 3, NATM)	11.4%	10.6%	10.5%	9.8%
	Difference between Base Case	-	−0.7~−0.8%	−0.7~−0.9%	−1.4~−1.7%

Source: JICA Study Team

7.8.2 Economic Evaluation of the Project with Pasig - Marikina Flood Mitigation Project

(1) General

During flood of Pasig - Marikina River Basin, a flood flow of up to 2,400 m^3/s flows into Laguna de Bay through Mangahan Floodway. Peak discharge of Pasig - Marikina River is reduced by this diversion through Mangahan Floodway. It is the most important measure of the Pasig - Marikina River flood mitigation plan. Meanwhile, lakeshore area of Laguna de Bay is inundated due to flood flow from Marikina River. The flood management plan of lakeshore area of Laguna de Bay consisting of Parañaque Spillway should be dealt with Pasig - Marikina River flood mitigation project.

In addition, in 1975, Mangahan Floodway, which aims to divert flood flow of Marikina River, and Parañaque Spillway, which aims to discharge flood flow of the Marikina River to Manila Bay, are proposed in the set.

Both projects are composed of the following structure;

<u>Pasig - Marikina River Flood Mitigation Project</u>

1) River Improvement Project

- River Improvement Project Phase II (Completed)
- River Improvement Project Phase III (Completed)
- River Improvement Project Phase IV (Planed)

2) Marikina Dam Project

3) Retarding Basin Project

<u>Comprehensive Flood Management Project of Laguna de Bay Lakeshore Area</u>

1) Parañaque Spillway Project

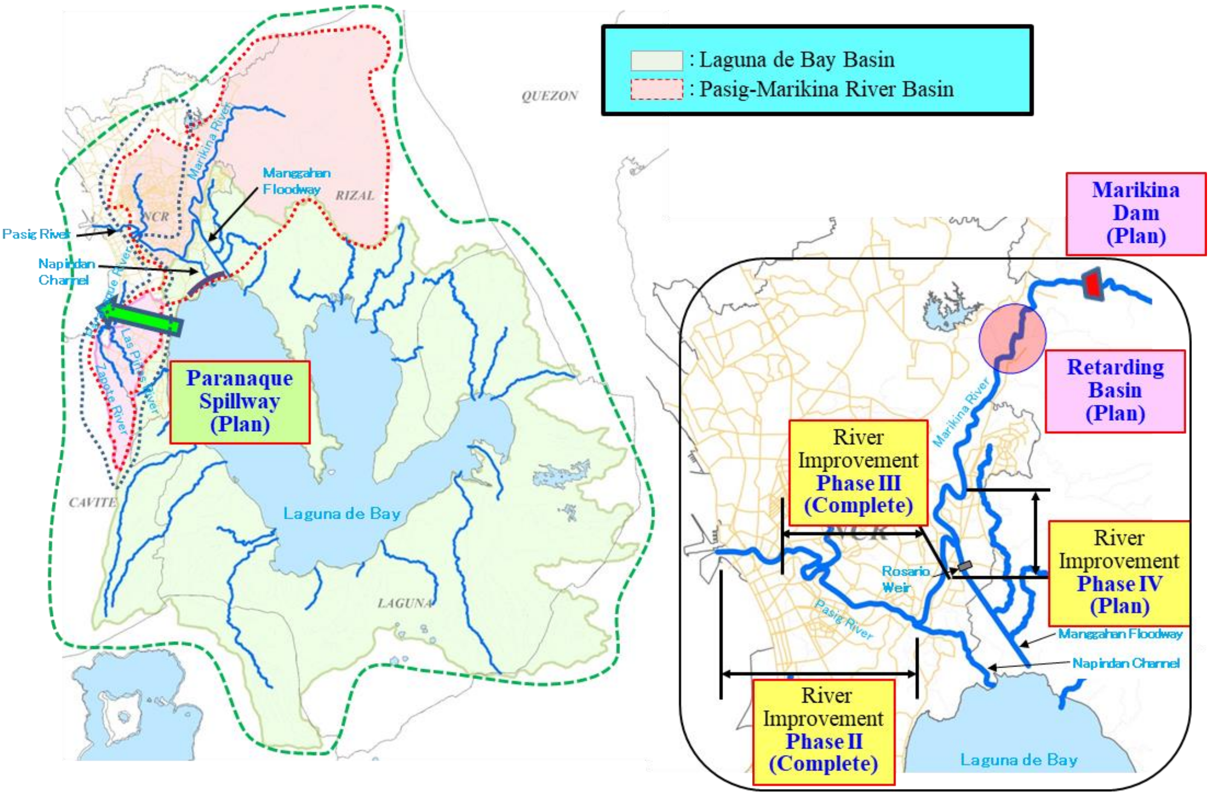


Figure 7.8.2 Pasig - Marikina Flood Mitigation Project and Comprehensive Flood Management Project of Laguna Lakeshore Area

(2) Results of Economic Analysis

Economic analysis on the river improvement project of Pasig - Marikina River has been executed in "The Preparatory Survey for Pasig - Marikina River Channel Improvement Project (Phase III), October 2011". In this survey, economic analysis is carried out by updating the cost and benefit of this study.

The cost and benefit of the river improvement project shown in the above report, and the assumed cost and benefit of the on-going Marikina Dam and Retarding Basin Projects, as well as the cost and benefit of the Parañaque Spillway estimated in this survey can be used. The results of the analysis are shown in Table 7.8.18. Breakdown of each case is given in Table 7.8.19, Table 7.8.20 and Table 7.8.21. The case of the Parañaque Spillway alone is as shown in Table 7.8.16.

Table 7.8.18 Result of Economic Analysis for Pasig - Marikina Flood Mitigation Project and Comprehensive Flood Management Project of Laguna Lakeshore Area

Combination of Project	EIRR	B/C	NPV (PHP million)
1. Pasig - Marikina River Improvement Phase II, III, IV	28.6%	4.5	27,391
2. Pasig - Marikina River Improvement Phase II, III, IV + Parañaque Spillway	26.8%	3.1	27,708
3. Pasig - Marikina River Improvement Phase II, III, IV + Marikina Dam and Retarding Basin + Parañaque Spillway	26.1%	2.8	28,285

Table 7.8.19 Result of Economic Analysis for Pasig - Marikina Flood Mitigation Project and Comprehensive Flood Management Project of Laguna Lakeshore Area (1/3)

(1. River Improvement Phase II, III, IV)

Year	Cost							Benefit				Net Benefit
	River Improvement			Marikina Dam	Retarding Basin	Paranaque Spillway	Total	River Improvement + Retarding basin +Dam		Paranaque Spillway	Total	
	Phase II	Phase III	Phase IV					30-year Return Period	30 to 100-year Return Period			
2007	133	0	0	0	0	0	133	0	0	0	0	-133
2008	418	0	0	0	0	0	418	0	0	0	0	-418
2009	564	0	0	0	0	0	564	0	0	0	0	-564
2010	919	0	0	0	0	0	919	0	0	0	0	-919
2011	1,105	0	0	0	0	0	1,105	0	0	0	0	-1,105
2012	1,400	166	0	0	0	0	1,566	0	0	0	0	-1,566
2013	12	1,174	0	0	0	0	1,186	1,470	0	0	1,470	284
2014	12	2,111	0	0	0	0	2,123	1,470	0	0	1,470	-653
2015	12	2,111	0	0	0	0	2,123	1,470	0	0	1,470	-653
2016	12	984	0	0	0	0	995	1,470	0	0	1,470	474
2017	12	15	63	0	0	0	89	5,653	0	0	5,653	5,563
2018	12	15	1,103	0	0	0	1,130	5,653	0	0	5,653	4,523
2019	12	15	1,605	0	0	0	1,632	5,653	0	0	5,653	4,021
2020	12	15	1,605	0	0	0	1,632	5,653	0	0	5,653	4,021
2021	12	15	1,641	0	0	0	1,668	5,653	0	0	5,653	3,985
2022	12	15	536	0	0	0	563	5,653	0	0	5,653	5,090
2023	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2024	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2025	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2026	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2027	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2028	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2029	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2030	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2031	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2032	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2033	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2034	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2035	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2036	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2037	12	15	325	0	0	0	351	10,692	0	0	10,692	10,340
2038	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2039	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2040	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2041	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2042	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2043	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2044	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2045	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2046	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2047	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2048	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2049	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2050	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2051	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2052	12	15	325	0	0	0	351	10,692	0	0	10,692	10,340
2053	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2054	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2055	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2056	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2057	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2058	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2059	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2060	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2061	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2062	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2063	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2064	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2065	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2066	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2067	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2068	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2069	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2070	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2071	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2072	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2073	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2074	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2075	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2076	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2077	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2078	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650
2079	12	15	15	0	0	0	42	10,692	0	0	10,692	10,650

Discount Rate	10.0%
EIRR	28.6%
NPV	27,391
B/C	4.5

Table 7.8.20 Result of Economic Analysis for Pasig - Marikina Flood Mitigation Project and Comprehensive Flood Management Project of Laguna Lakeshore Area (2/3)

(2. River Improvement Phase II, III, IV + Parañaque Spillway)

Year	Cost							Benefit				Net Benefit
	River Improvement			Marikina Dam	Retarding Basin	Paranaque Spillway	Total	River Improvement + Retarding basin +Dam		Paranaque Spillway	Total	
	Phase II	Phase III	Phase IV					30-year Return Period	30 to 100-year Return Period			
2007	133	0	0	0	0	0	133	0	0	0	0	-133
2008	418	0	0	0	0	0	418	0	0	0	0	-418
2009	564	0	0	0	0	0	564	0	0	0	0	-564
2010	919	0	0	0	0	0	919	0	0	0	0	-919
2011	1,105	0	0	0	0	0	1,105	0	0	0	0	-1,105
2012	1,400	166	0	0	0	0	1,566	0	0	0	0	-1,566
2013	12	1,174	0	0	0	0	1,186	1,470	0	0	1,470	284
2014	12	2,111	0	0	0	0	2,123	1,470	0	0	1,470	-653
2015	12	2,111	0	0	0	0	2,123	1,470	0	0	1,470	-653
2016	12	984	0	0	0	0	995	1,470	0	0	1,470	474
2017	12	15	63	0	0	0	89	5,653	0	0	5,653	5,563
2018	12	15	1,103	0	0	0	1,130	5,653	0	0	5,653	4,523
2019	12	15	1,605	0	0	0	1,632	5,653	0	0	5,653	4,021
2020	12	15	1,605	0	0	856	2,488	5,653	0	0	5,653	3,165
2021	12	15	1,641	0	0	867	2,535	5,653	0	0	5,653	3,118
2022	12	15	536	0	0	3,345	3,907	5,653	0	0	5,653	1,745
2023	12	15	15	0	0	3,350	3,392	10,692	0	0	10,692	7,300
2024	12	15	15	0	0	3,355	3,397	10,692	0	0	10,692	7,295
2025	12	15	15	0	0	3,869	3,911	10,692	0	0	10,692	6,781
2026	12	15	15	0	0	3,660	3,702	10,692	0	0	10,692	6,989
2027	12	15	15	0	0	3,667	3,708	10,692	0	0	10,692	6,983
2028	12	15	15	0	0	3,673	3,715	10,692	0	0	10,692	6,977
2029	12	15	15	0	0	5,048	5,090	10,692	0	0	10,692	5,602
2030	12	15	15	0	0	3,186	3,228	10,692	0	0	10,692	7,464
2031	12	15	15	0	0	330	371	10,692	0	4,967	15,658	15,287
2032	12	15	15	0	0	148	190	10,692	0	5,055	15,746	15,556
2033	12	15	15	0	0	148	190	10,692	0	5,144	15,836	15,646
2034	12	15	15	0	0	148	190	10,692	0	5,235	15,927	15,737
2035	12	15	15	0	0	148	190	10,692	0	5,329	16,020	15,830
2036	12	15	15	0	0	148	190	10,692	0	5,418	16,110	15,920
2037	12	15	325	0	0	148	500	10,692	0	5,509	16,200	15,701
2038	12	15	15	0	0	148	190	10,692	0	5,601	16,293	16,103
2039	12	15	15	0	0	148	190	10,692	0	5,696	16,387	16,197
2040	12	15	15	0	0	148	190	10,692	0	5,791	16,483	16,293
2041	12	15	15	0	0	148	190	10,692	0	5,882	16,573	16,383
2042	12	15	15	0	0	148	190	10,692	0	5,974	16,665	16,475
2043	12	15	15	0	0	148	190	10,692	0	6,067	16,759	16,569
2044	12	15	15	0	0	148	190	10,692	0	6,162	16,854	16,664
2045	12	15	15	0	0	148	190	10,692	0	6,259	16,950	16,760
2046	12	15	15	0	0	148	190	10,692	0	6,357	17,049	16,859
2047	12	15	15	0	0	148	190	10,692	0	6,457	17,149	16,959
2048	12	15	15	0	0	148	190	10,692	0	6,559	17,251	17,061
2049	12	15	15	0	0	148	190	10,692	0	6,662	17,354	17,164
2050	12	15	15	0	0	148	190	10,692	0	6,768	17,459	17,269
2051	12	15	15	0	0	148	190	10,692	0	6,875	17,566	17,376
2052	12	15	325	0	0	148	500	10,692	0	6,984	17,675	17,176
2053	12	15	15	0	0	148	190	10,692	0	7,094	17,786	17,596
2054	12	15	15	0	0	148	190	10,692	0	7,207	17,899	17,709
2055	12	15	15	0	0	148	190	10,692	0	7,322	18,013	17,823
2056	12	15	15	0	0	148	190	10,692	0	7,438	18,130	17,940
2057	12	15	15	0	0	148	190	10,692	0	7,557	18,249	18,059
2058	12	15	15	0	0	148	190	10,692	0	7,677	18,369	18,179
2059	12	15	15	0	0	148	190	10,692	0	7,800	18,492	18,302
2060	12	15	15	0	0	148	190	10,692	0	7,925	18,617	18,427
2061	12	15	15	0	0	148	190	10,692	0	8,052	18,744	18,554
2062	12	15	15	0	0	148	190	10,692	0	8,181	18,873	18,683
2063	12	15	15	0	0	148	190	10,692	0	8,312	19,004	18,814
2064	12	15	15	0	0	148	190	10,692	0	8,446	19,138	18,948
2065	12	15	15	0	0	148	190	10,692	0	8,582	19,274	19,084
2066	12	15	15	0	0	148	190	10,692	0	8,720	19,412	19,222
2067	12	15	15	0	0	148	190	10,692	0	8,861	19,552	19,362
2068	12	15	15	0	0	148	190	10,692	0	9,004	19,695	19,505
2069	12	15	15	0	0	148	190	10,692	0	9,149	19,841	19,651
2070	12	15	15	0	0	148	190	10,692	0	9,297	19,989	19,799
2071	12	15	15	0	0	148	190	10,692	0	9,448	20,139	19,949
2072	12	15	15	0	0	148	190	10,692	0	9,601	20,292	20,102
2073	12	15	15	0	0	148	190	10,692	0	9,756	20,448	20,258
2074	12	15	15	0	0	148	190	10,692	0	9,915	20,607	20,417
2075	12	15	15	0	0	148	190	10,692	0	10,076	20,768	20,578
2076	12	15	15	0	0	148	190	10,692	0	10,240	20,932	20,742
2077	12	15	15	0	0	148	190	10,692	0	10,407	21,098	20,908
2078	12	15	15	0	0	148	190	10,692	0	10,576	21,268	21,078
2079	12	15	15	0	0	148	190	10,692	0	10,749	21,440	21,250

Discount Rate	10.0%
EIRR	26.8%
NPV	27,708
B/C	3.1

Table 7.8.21 Result of Economic Analysis for Pasig - Marikina Flood Mitigation Project and Comprehensive Flood Management Project of Laguna Lakeshore Area (3/3)

(3. River Improvement Phase II, III, IV + Marikina Dam and Retarding Basin + Parañaque Spillway)

Year	Cost							Benefit				Net Benefit
	River Improvement			Marikina Dam	Retarding Basin	Paranaque Spillway	Total	River Improvement + Retarding basin +Dam		Paranaque Spillway	Total	
	Phase II	Phase III	Phase IV					30-year Return Period	30 to 100-year Return Period			
2007	133	0	0	0	0	0	133	0	0	0	0	-133
2008	418	0	0	0	0	0	418	0	0	0	0	-418
2009	564	0	0	0	0	0	564	0	0	0	0	-564
2010	919	0	0	0	0	0	919	0	0	0	0	-919
2011	1,105	0	0	0	0	0	1,105	0	0	0	0	-1,105
2012	1,400	166	0	0	0	0	1,566	0	0	0	0	-1,566
2013	12	1,174	0	0	0	0	1,186	1,470	0	0	1,470	284
2014	12	2,111	0	0	0	0	2,123	1,470	0	0	1,470	-653
2015	12	2,111	0	0	0	0	2,123	1,470	0	0	1,470	-653
2016	12	984	0	0	0	0	995	1,470	0	0	1,470	474
2017	12	15	63	0	0	0	89	5,653	0	0	5,653	5,563
2018	12	15	1,103	0	0	0	1,130	5,653	0	0	5,653	4,523
2019	12	15	1,605	0	0	0	1,632	5,653	0	0	5,653	4,021
2020	12	15	1,605	0	0	856	2,488	5,653	0	0	5,653	3,165
2021	12	15	1,641	0	0	867	2,535	5,653	0	0	5,653	3,118
2022	12	15	536	1,633	1,500	3,345	7,041	5,653	0	0	5,653	-1,388
2023	12	15	15	1,633	1,500	3,350	6,525	10,692	0	0	10,692	4,167
2024	12	15	15	1,633	1,500	3,355	6,530	10,692	0	0	10,692	4,161
2025	12	15	15	1,633	1,500	3,869	7,044	10,692	0	0	10,692	3,648
2026	12	15	15	33	30	3,660	3,765	10,692	1,880	0	12,572	8,807
2027	12	15	15	33	30	3,667	3,771	10,692	1,880	0	12,572	8,801
2028	12	15	15	33	30	3,673	3,777	10,692	1,880	0	12,572	8,795
2029	12	15	15	33	30	5,048	5,153	10,692	1,880	0	12,572	7,419
2030	12	15	15	33	30	3,186	3,290	10,692	1,880	0	12,572	9,282
2031	12	15	15	33	30	330	434	10,692	1,880	4,967	17,539	17,105
2032	12	15	15	33	30	148	253	10,692	1,880	5,055	17,627	17,374
2033	12	15	15	33	30	148	253	10,692	1,880	5,144	17,716	17,463
2034	12	15	15	33	30	148	253	10,692	1,880	5,235	17,808	17,555
2035	12	15	15	33	30	148	253	10,692	1,880	5,329	17,901	17,648
2036	12	15	15	33	30	148	253	10,692	1,880	5,418	17,990	17,737
2037	12	15	325	33	30	148	562	10,692	1,880	5,509	18,081	17,518
2038	12	15	15	33	30	148	253	10,692	1,880	5,601	18,173	17,921
2039	12	15	15	33	30	148	253	10,692	1,880	5,696	18,268	18,015
2040	12	15	15	33	30	148	253	10,692	1,880	5,791	18,364	18,111
2041	12	15	15	33	30	148	253	10,692	1,880	5,882	18,454	18,201
2042	12	15	15	33	30	148	253	10,692	1,880	5,974	18,546	18,293
2043	12	15	15	33	30	148	253	10,692	1,880	6,067	18,639	18,386
2044	12	15	15	33	30	148	253	10,692	1,880	6,162	18,734	18,481
2045	12	15	15	33	30	148	253	10,692	1,880	6,259	18,831	18,578
2046	12	15	15	33	30	148	253	10,692	1,880	6,357	18,929	18,676
2047	12	15	15	33	30	148	253	10,692	1,880	6,457	19,029	18,776
2048	12	15	15	33	30	148	253	10,692	1,880	6,559	19,131	18,878
2049	12	15	15	33	30	148	253	10,692	1,880	6,662	19,234	18,982
2050	12	15	15	33	30	148	253	10,692	1,880	6,768	19,340	19,087
2051	12	15	15	33	30	148	253	10,692	1,880	6,875	19,447	19,194
2052	12	15	325	33	30	148	562	10,692	1,880	6,984	19,556	18,993
2053	12	15	15	33	30	148	253	10,692	1,880	7,094	19,666	19,414
2054	12	15	15	33	30	148	253	10,692	1,880	7,207	19,779	19,526
2055	12	15	15	33	30	148	253	10,692	1,880	7,322	19,894	19,641
2056	12	15	15	33	30	148	253	10,692	1,880	7,438	20,010	19,758
2057	12	15	15	33	30	148	253	10,692	1,880	7,557	20,129	19,876
2058	12	15	15	33	30	148	253	10,692	1,880	7,677	20,250	19,997
2059	12	15	15	33	30	148	253	10,692	1,880	7,800	20,372	20,120
2060	12	15	15	33	30	148	253	10,692	1,880	7,925	20,497	20,244
2061	12	15	15	33	30	148	253	10,692	1,880	8,052	20,624	20,371
2062	12	15	15	33	30	148	253	10,692	1,880	8,181	20,753	20,500
2063	12	15	15	33	30	148	253	10,692	1,880	8,312	20,884	20,632
2064	12	15	15	33	30	148	253	10,692	1,880	8,446	21,018	20,765
2065	12	15	15	33	30	148	253	10,692	1,880	8,582	21,154	20,901
2066	12	15	15	33	30	148	253	10,692	1,880	8,720	21,292	21,039
2067	12	15	15	33	30	148	253	10,692	1,880	8,861	21,433	21,180
2068	12	15	15	33	30	148	253	10,692	1,880	9,004	21,576	21,323
2069	12	15	15	33	30	148	253	10,692	1,880	9,149	21,721	21,469
2070	12	15	15	33	30	148	253	10,692	1,880	9,297	21,869	21,616
2071	12	15	15	33	30	148	253	10,692	1,880	9,448	22,020	21,767
2072	12	15	15	33	30	148	253	10,692	1,880	9,601	22,173	21,920
2073	12	15	15	33	30	148	253	10,692	1,880	9,756	22,329	22,076
2074	12	15	15	33	30	148	253	10,692	1,880	9,915	22,487	22,234
2075	12	15	15	33	30	148	253	10,692	1,880	10,076	22,648	22,395
2076	12	15	15	33	30	148	253	10,692	1,880	10,240	22,812	22,559
2077	12	15	15	33	30	148	253	10,692	1,880	10,407	22,979	22,726
2078	12	15	15	33	30	148	253	10,692	1,880	10,576	23,148	22,895
2079	12	15	15	33	30	148	253	10,692	1,880	10,749	23,321	23,068

Discount Rate	10.0%
EIRR	26.1%
NPV	28,285
B/C	2.8

7.8.3 Qualitative Effect

The economic benefit which could be quantified in economic value is as prescribed in Subsection 7.8.1. The other qualitative benefit, which is difficult to be quantified, is summarized in this Subsection. People living in the beneficiary area take some of the qualitative benefits more important than quantitative benefits, and these sentiments need to be carefully considered in implementing the project.

(1) Reduction of Economic Damage by Inundation

In Subsection 7.8.1, economic damage of household assets, commercial assets, agricultural crops and suspensions of business are quantified. The other major damages caused by inundation are the damage to infrastructure facilities, economic loss of human life and economic cost of disruption of traffic, etc. Those items are explained below.

[Damage of Infrastructure Asset]

As described in Subsection 2.2.3, Typhoon Ondoy caused the damage of 11.2 million pesos, in total, on facilities of electricity, water supply, sanitation, and flood management and transportation sectors. It is quite difficult to estimate the future economic damage amount of infrastructure facilities, but the large amount of economic damage on basic utilities is avoided by implementing the Project, which will make positive effect on the social economy.

[Loss of Human Resources]

As explained in Subsection 2.2.3, human loss from Typhoon Ondoy and Typhoon Pepeng is more than 500 (casualties and missing people). Referring to Table 2.2.7, number of casualties caused by Typhoon Ondoy in the study area tolls up to 133. Those number could be reduced by the implementation of the Project.

Quantification of human life is one of the issues related to economic analysis. There are not many reports which include the value of human loss in the benefits. One exception is the "Economic Impact of Sanitation in the Philippines (World Bank/USAID, 2008)" where the value of human life is calculated at 28,700 U.S. dollar/person based on the average lifetime earnings in the Philippines. Applying the GDP growth rate, the value of human loss becomes around 50,000 U.S. dollar/person as of 2017. Furthermore, it is well known that the value of human life would be several times higher than the value calculated by lifetime earnings, if it is estimated by the "willingness to pay" survey. The economic value is significant, and this benefit should be considered well before the implementation.

[Economic Loss of Disruption of Traffic]

When the water level rises, some road routes are interrupted by inundation, and the transportation time of people and commercial products in the surrounding area gets much longer. It is difficult to analyse the traffic network in the study area, and to evaluate the benefit of reducing the disruption time. However, the inundation period lasts for several weeks to a few months in the area, and the benefit has significant impact on society.

(2) Development in the Target Area

Land price increase is quantified and included in the economic analysis. However, implementation of the Project is believed to make more positive impacts on society. The beneficiary area is located in the commuting area to Metro Manila, and the population is supposed to increase if the risk of inundation reduces. Along with the population growth, the other business activities, such as manufacturing, commercial malls, restaurants and retail shops, are supposed to grow, which contributes to enhance the local economy.

(3) Improvement of Employment and Technical Knowledge by the Project

In case the Project is implemented, the project cost of around 48 to 65 billion pesos would be spent in the study area during 10 years of designing and construction period. In addition, around 0.2 to 0.3 billion pesos would be spent on the O&M works during the O&M phase. Such large amount of budget is spent for construction and consulting companies, and it contributes to improve the employment conditions in the area.

Furthermore, the technology level used to make the spillway is quite new and sophisticated, and such technology and experience could be transferred to the local engineers and counterparts along with the progress of project implementation. Such knowledge surely contributes to enhancing the technology of the Philippines in the long term.

7.9 Environment and Social Considerations

7.9.1 Existing Condition of Target Areas

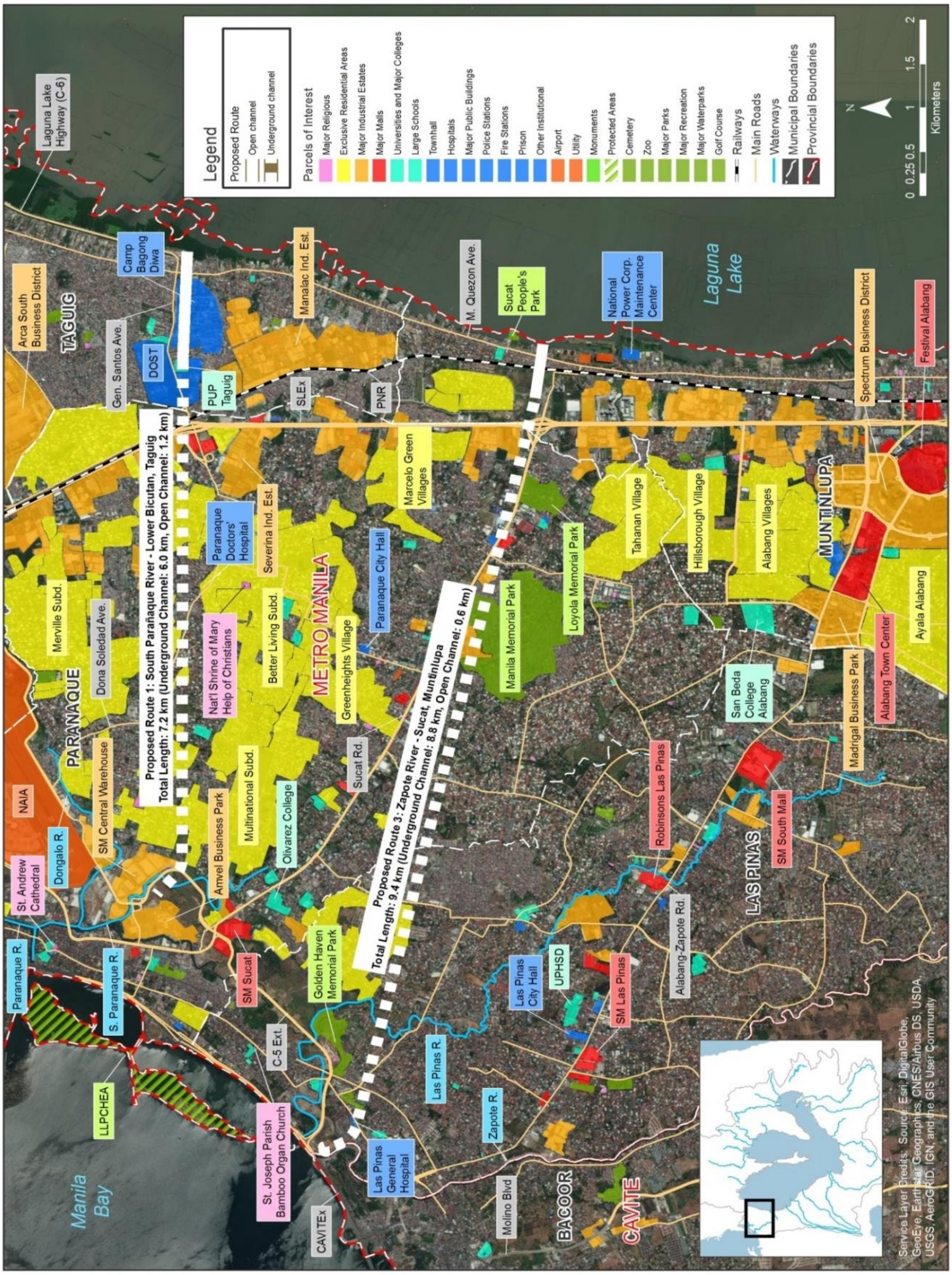
(1) Land Use and Existing Facilities along the Candidate Sites of Parañaque Spillway

(a) Current Status along the Candidate Routes of Parañaque Spillway

As discussed in Section 7.2, there are three candidate routes of Parañaque Spillway (Figure 7.2.6), where current land use is described as below, focusing on the prospective two routes (Table 7.2.5).

As shown in Figure 7.9.1, land use around the candidate location of the open channel along Route 1 with the length of 1.2 km includes a residential area, a police facility (Camp Bagong Diwa), a school (Polytechnic University of the Philippines), and a government facility (Department of Science and Technology: DOST). To the west beyond the Philippine National Railway (PNR) and SLEX (Southern Luzon Expressway) are residential subdivision areas. Land use near the candidate drainage facility located in Barangay San Dionisio, Parañaque, is a warehouse complex and business district (Amel Business Park).

On the other hand, land use along the candidate location of open channel along Route 2 with the length of 0.6 km includes a residential area along M. Quezon Avenue, PNR and SLEX westward. There is an open space located in-between the PNR and SLEX. Further westward, there are built-up areas and residential areas, as well as cemeteries (Loyola Memorial Park, Manila Memorial Park). Land use near the candidate drainage facility along the Manila Bay is an open space (property of Las Piñas City) and a former garbage dumping site (currently, private property). A hospital (Las Piñas General Hospital) is located in the south.



Source: JICA Survey Team

Figure 7.9.1 Land Use and Existing Facilities along the Candidate Sites of Parañaque Spillway

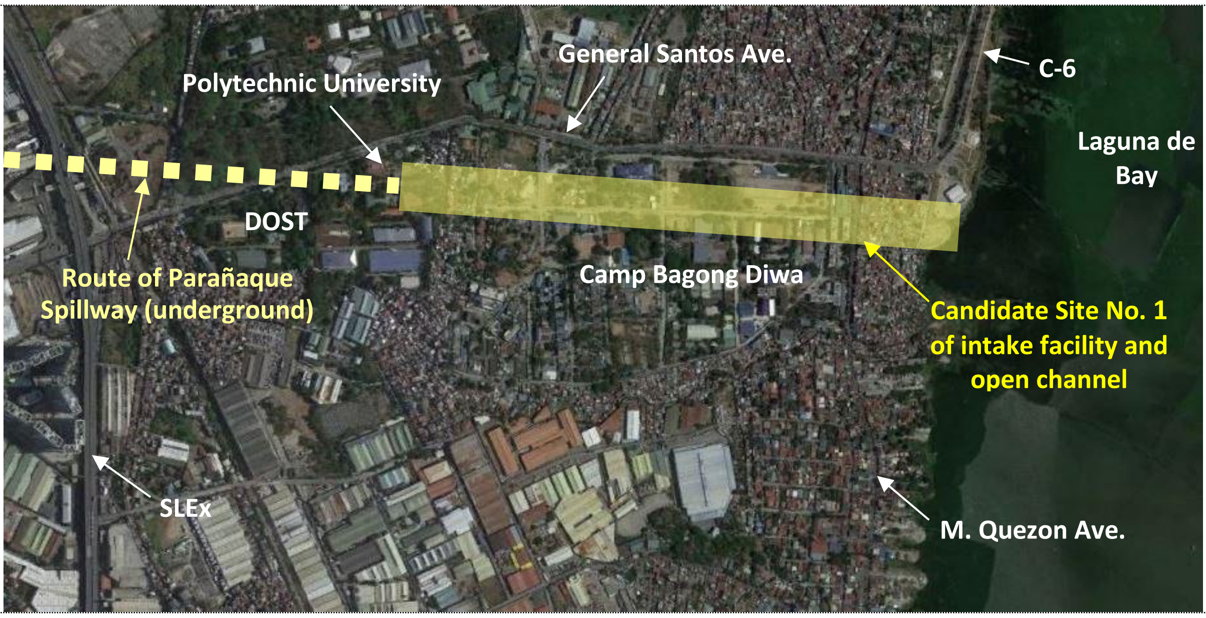
(b) Current Status near the Candidate Sites of Intake Facility and Open Channel

There are two candidate sites of intake facility and open channel as shown in Section 7.2 (Figure 7.2.4, Figure 7.2.5 and Table 7.2.3). One is located in Barangay Lower Bicutan, Taguig City, and the other is in Barangay Sucat, Muntinlupa City. Current environmental status near these candidate sites are as shown in Table 7.9.1 and Figure 7.9.2.

Table 7.9.1 Current Status near the Candidate Sites of Intake Facility and Open Channel

Candidate Site	Current Status near Candidate Site
Candidate Site No. 1: Barangay Lower Bicutan, Taguig City	Traffic Condition: There is the Laguna Lake Highway (C-6) along Laguna de Bay. M. Quezon Avenue is located south of C-6 in north-south direction. General Santos Avenue runs in east-west direction along the candidate site of open channel. M. Quezon Avenue, a two-lane road, is often overcrowded. Land Use/Existing Facility: There is the Taguig Lakeshore Hall and its park area along Laguna de Bay. There are the residential area, police facility (Camp Bagong Diwa), university (Polytechnic University of the Philippines), and government facility (Department of Science and Technology: DOST) along the candidate site of open channel. Natural Environment: Vegetation in the residential area is limited. Although there are vegetated areas in the police facility, they are not natural but affected by human activities as terrestrial ecosystem. Aquatic ecosystem along Laguna de Bay is developed in marshy land including such plants as water hyacinth (*Eichhornia crassipes*) and kangkong (*Ipomea aquatic*). Water Area Use: Fish cages exist offshore in Laguna de Bay at the distance of several hundred meters from the shoreline. Several mooring facilities also exist for boats along the lakeshore.
Candidate Site No. 2: Barangay Sucat, Muntinlupa City	Traffic Condition: M. Quezon Avenue and the Philippine National Railways (PNR) run in north-south direction and Sucat Road runs in east-west direction. Land Use/Existing Facility: Residential areas exist along both sides of M. Quezon Avenue. There is the real-estate property of Vista Land and Lifescapes, Inc. west of PNR. The current status of the estate is open space. Natural Environment: Vegetation in the residential area is limited. The open space owned by the real estate company is occupied with grasses and bushes. This Candidate Site No. 2 is covered with aquatic plants in Laguna de Bay as with Candidate Site No. 1. Water Area Use: Aquatic culture facilities (fish pens and fish cages) exist offshore in the Laguna de Bay at the distance of several hundred meters from the shoreline.

Source: JICA Survey Team



Aero Photo 1 : Candidate Site No. 1 of Intake Facility and Open Channel (Barangay Lower Bicutan, Taguig)

Aero Photo 2 : Candidate Site No. 2 of Intake Facility and Open Channel (Barangay Sucat, Muntinlupa)

Figure 7.9.2 Current Status along the Candidate Sites of Intake Facility and Open Channel

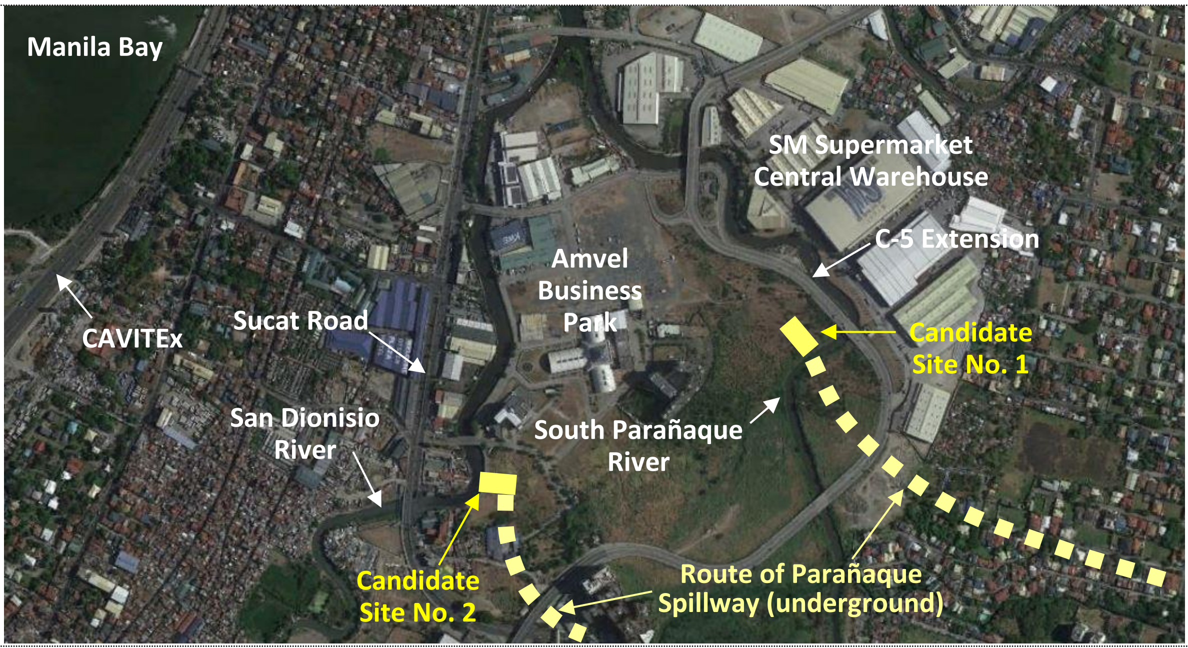
(c) Current Status near the Candidate Sites of Drainage Facility

There are three candidate sites of drainage facility as discussed in Section 7.2, and two of which are located at the lower section of the Parañaque River System (Barangay San Dionisio, Parañaque City) and the other is located at the lowermost section of the Zapote River (Barangay Pulang Lupa Uno, Las Piñas City). Current status near the candidate sites are as shown in Table 7.9.2 and Figure 7.9.3.

Table 7.9.2 Current Status near the Candidate Sites of Drainage Facility

Candidate Site	Current Status near the Candidate Site
Candidate site No. 1 and No. 2: Located at the lower section of the Parañaque River system (Barangay Lower Bicutan, Taguig City)	Traffic Condition: There is Carlos P. Garcia Avenue (C-5 Extension) running from north to south sides of the candidate sites with enclosing them. Sucat Road runs on the west side of Candidate Site No. 2 in north-south direction. Land Use/Existing Facility: Candidate Sites No. 1 and 2, located along the Parañaque River system, are currently vast vegetated lands. There is the business district (Amvel Business Park) and the warehouse near the sites. Natural Environment: Terrestrial ecosystem around the candidate sites is not natural but affected by human activities. The sites are open land and covered with dense vegetation such as grass and bush.
Candidate site No. 3: Located at the lowermost section of the Zapote River (Barangay Sucat, Muntinlupa City)	Traffic Condition: CAVITEX runs along Manila Bay. Carlos P. Garcia Avenue (C-5) runs east of the candidate site in east-west direction. Land Use/Existing Facility: Right bank side of the candidate site is a motor pool area of Las Piñas City. The south side of it was formerly used as garbage dumping site and is currently a private lot (according to Las Piñas City) where there are many Informal Settler Families (ISFs). The left bank side, on the other hand, is the City of Bacoor, Cavite Province, where a lot of ISFs dwell. The south side of the former dumping site is a residential area, in which a hospital and a college are located. Natural Environment: Vegetation in the private lot of the Zapote River is mainly composed of bush. The left bank side of the river is a vast vegetated marsh land.

Source: JICA Survey Team



Aero Photo 1 : Location of Candidate Sites No. 1 and 2 of Drainage Facility
(South Parañaque River and San Dionisio River)

Aero Photo 2 : Location of Candidate Site No. 3 of Drainage Facility (Zapote River)

Figure 7.9.3 Current Status along the Candidate Sites of Drainage Facility

(2) Fishery and Water Transportation around Candidate Sites of Water Intake Facility

Interview with the Environmental Regulations Department of LLDA was carried out to collect information on fishing activities, water transportation, etc., in Laguna de Bay, especially, those near the candidate site of intake facilities of the Project. The results of the interview are summarized as follows:

- Aquaculture using fish pen and fish cage is done near Candidate Site No. 1 and No. 2. In addition, fish traps are installed near the lakeshore. Several piers and mooring facilities are installed at intermediate locations of the two candidate sites in Barangay Bagumbayan, Taguig City (Photo 1,

Figure 7.9.4). These facilities are used for transportation between lakeshore and aquaculture facilities.

- Aquaculture facilities such as fish pens and fish cages are administered by LLDA and they are required to obtain permits for installation. These facilities should be installed offshore farther than 200 m from the shoreline of the lake while no permit is required for the installation of fish traps.
- There are two associations of fisher-folks/companies who are conducting commercial fishing in Laguna de Bay, and these are administered by LLDA. Other than these, there are fisher-folk associations administered by the Fishery and Aquatic Resources Management Council based on RA No. 8550.
- Besides fishing, cultivation and harvesting of water plants, i.e., kangkong (*Ipomoea aquatica*) is done in the lake (Photo 2, Figure 7.9.4).
- Status of water transportation in Laguna de Bay is discussed in Subsection 2.4.1. There was a water transportation route south of Candidate Site No. 2 when the Sucat Thermal Power Plant was still operational. Oil for the thermal plant is transported along the transportation route. At present, however, the transportation route is not being used because of stoppage of operation of the thermal plant. The disposal process of the thermal plant to another potential operator is being made according to press news, and therefore, the transportation route might re-function in the future when the plant is re-operated.
- Regarding the existing water intake from Laguna de Bay, there is no intake point near the candidate sites. The nearest water intake point which is operated by NIA (National Irrigation Administration) is located in Barangay Putatan, at approximately 6 km south from Candidate Site No. 2, in Muntinlupa City.

Photo 1: Fishing boats and mooring facilities
(Barangay Bagumbayan, Taguig City)

Photo 2: Cultivation and harvesting of water plants
(Kangkong)
(Barangay Lower Bicutan, Taguig City)

Source: JICA Survey Team

Figure 7.9.4 Status of Mooring Facilities and Cultivation of Water Plants in Laguna de Bay near the Candidate Sites of Intake Facility of the Parañaque Spillway Project

(3) ISFs along the Candidate Sites of Parañaque Spillway

(a) Candidate Sites of the Intake Facility and Open Channel

Information on ISFs were gathered through interview with concerned agencies (UPAO: Urban Poor Affairs Office) of Taguig and Muntinlupa cities. Site reconnaissance was conducted too for confirming the situation onsite. Results of the survey are summarized in Table 7.9.3.

Table 7.9.3 ISFs along the Candidate Sites of the Intake Facility and Open Channel

Candidate Site	Interview Result	Result of Site Reconnaissance
Candidate No. 1 (Barangay Lower Bicutan, Taguig City)	UPAO (Urban Poor Affairs Office), Taguig City: • There are ISF settlements in the west side of C-6 Road in Barangay Lower Bicutan. No ISFs are seen along the lakeshore of Laguna de Bay at the south of Taguig Lakeshore Hall. • There may be some ISFs in the premises of the police facility (Camp Bagong Diwa) and university (Polytechnic University of the Philippines).	It was observed that the lakeshore of Laguna de Bay is encroached by houses at the south of Taguig Lakeshore Hall. Houses are equipped with an elevated floor to avoid inundation even during high water level. (Photo 1, Figure 7.9.5)
Candidate No. 2 (Barangay Sucat, Muntinlupa City)	UPAO (Urban Poor Affairs Office), Muntinlupa City: • Census conducted under Oplan Likas Project in 2015 revealed the following number of ISFs in the five (5) "puroks" (villages) where Candidate Site No. 2 is located: 1) Samahang Purok 4 Aplaya: 127 ISFs 2) Purok 2 and Purok 3 Lakeside: 59 ISFs 3) Purok 4 Sunrise Aplaya: 49 ISFs, villages 4) Block 3 Bagong Silang Tabing Ilog: 33 ISFs 5) Purok 1 Sucat: 14 ISFs	It was confirmed that the lakeshore of Laguna de Bay is encroached by houses which have elevated floors to avoid inundation even during floods (Photo 2, Figure 7.9.5).

Source: JICA Survey Team (Developed based on the results of interview with LGUs and site reconnaissance)

Photo 1: Housing Situation along Lakeshore
(Barangay Lower Bicutan, Taguig City)

Photo 2: Housing Situation along Lakeshore
(Barangay Sucat, Muntinlupa City)

Source: JICA Survey Team

Figure 7.9.5 Housing Situation at the Lakeshore of Laguna de Bay near Candidate Sites

(b) Candidate Sites of Drainage Facility

Information on ISFs were gathered through interview with concerned agencies (UMADO: Urban Mission Area Development Office and UPAO: Urban Poor Affairs Office) of Parañaque and Las Piñas cities where drainage facility is to be located. Site reconnaissance was also conducted to confirm the situations onsite. Results of the survey are summarized in Table 7.9.4.

Table 7.9.4 ISFs near the Candidate Sites of Drainage Facility

Candidate Sites	Interview Results	Results of Site Reconnaissance
Candidate Site No. 1 and No. 2, including downstream areas (Barangay San Dionisio and La Huerta, Parañaque City)	Urban Mission Area Development Office (UMADO), Parañaque City: ISFs identified through a census conducted in the downstream area of candidate sites (the area around La Huerta Elementary School on the left bank side) are as follows: 1) Lopez Jaena Extension: 175 ISFs 2) Christian Muslim Area: 60 ISFs 3) Back of La Huerta Elementary School: 20 ISFs	No ISF dwells in Candidate Sites No. 1 and No. 2 (Photos 1 and 2, Figure 7.9.6) ISFs exists along the downstream river section (Photo 3, Figure 7.9.6).
Candidate Site No. 3: (Barangay Pulang Lupa Uno, Las Piñas City)	Urban Poor Affairs Office (UPAO), Las Piñas City: Candidate site, located in the property of Las Piñas City, is currently used as a motor pool. It was formerly occupied by many ISFs who were relocated in 2014 and no ISF is found in the area although ISFs are seen in its southern portion which used to be a garbage dumping site and is currently a private property.	It was observed that solid wastes are transported into the area of Candidate Site No. 3 (Photo No. 4, Figure 7.9.6). ISFs dwelling in the southern portion can be observed on site.

Source: JICA Survey Team (Developed based on the results of interview with LGUs and site reconnaissance)

Photo 1: Candidate Site No. 1 of Drainage Facility
(Barangay San Dionisio, Parañaque City)

Photo 2: Candidate Site No. 2 of Drainage Facility
(Barangay San Dionisio, Parañaque City)

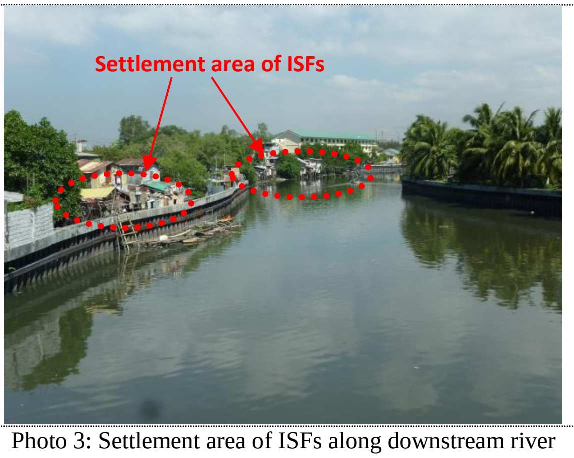


Photo 3: Settlement area of ISFs along downstream river section of Candidate Site No. 1 and 2
(Barangay La Huerta, Parañaque City)

Photo 4: Candidate Site No. 3 of Drainage Facility
(Barangay Pulang Lupa Uno, Las Piñas City)

Source: JICA Survey Team

Figure 7.9.6 Current Situation of Candidate Sites of Drainage Facility

(4) Underground Structures and Water Use along the Candidate Routes of Parañaque Spillway

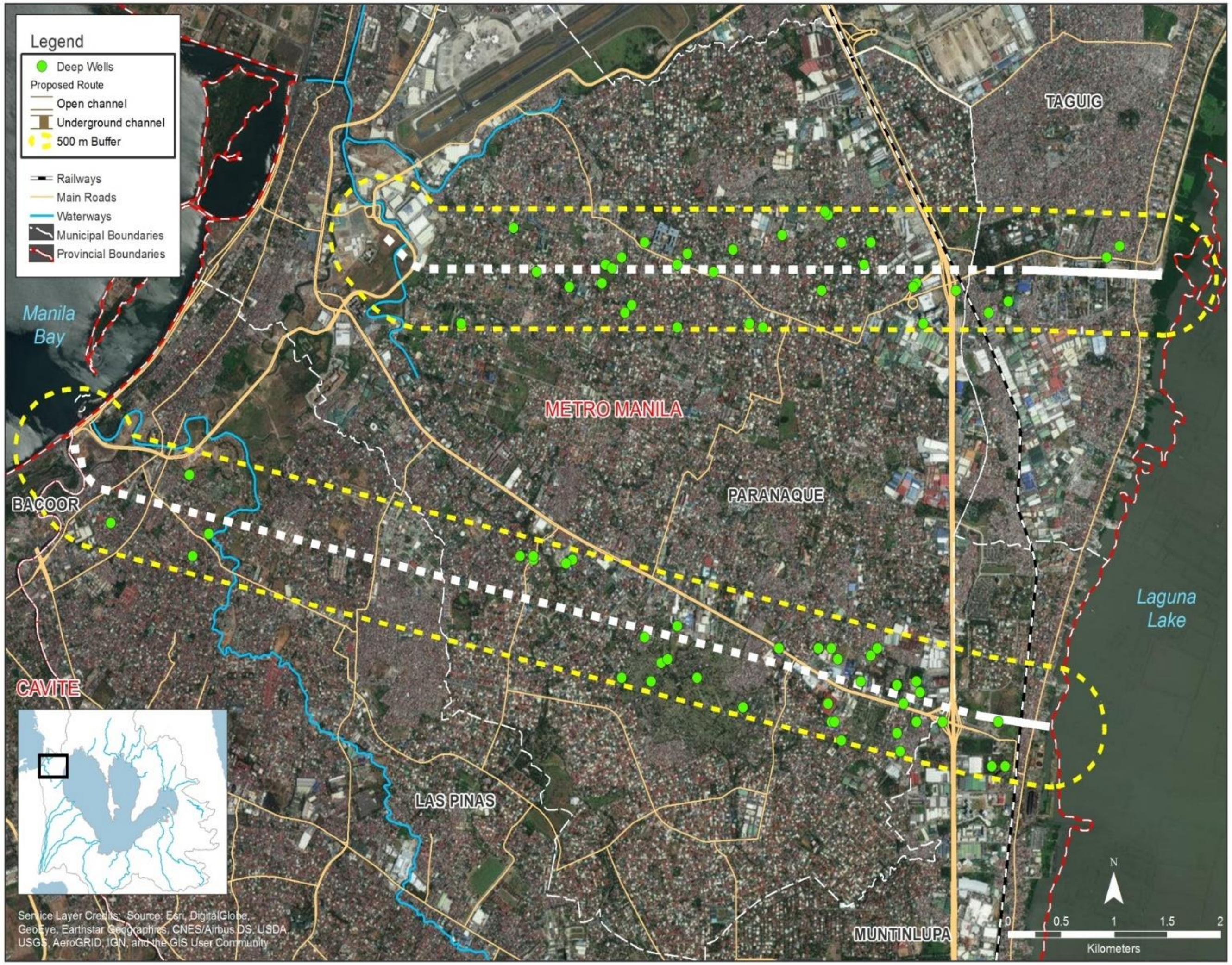
Underground structures along Candidate Route No. 1 and No. 3 of Parañaque Spillway include foundations of the elevated sections of the South Luzon Expressway (SLEX), high-rise buildings, etc. In addition, there will be foundations of the subway and railway planned near the candidate routes of Parañaque Spillway as described in Section 7.2 (Table 7.2.2).

The number of water permits granted within the area until approximately 500 m away from the candidate routes of Parañaque Spillway are shown in Table 7.9.5 and Figure 7.9.7. It is revealed that there are 35 and 40 water rights along Route 1 and Route 3, respectively. Most of them are deep wells, but there are 2 cases of water permit of surface water use within the jurisdiction of Parañaque City along Route 3.

Table 7.9.5 Number of Water Permits Granted WithinAreas 500m Away from Parañaque Spillway

Location		Route 1			Route 3		
Province	City / Municipality	Groundwater	Surface Water	Total	Groundwater	Surface Water	Total
Metro Manila	Parañaque	29	0	29	30	2	32
	Las Piñas	-	-	-	4	0	4
	Taguig	6	0	6	-	-	-
	Muntinlupa	-	-	-	4	0	4
Cavite	Bacoor	-	-	-	0	0	0
Total		**35**	**0**	**35**	**38**	**2**	**40**

Note: Locations of granted water permits are shown in Figure 7.9.7.
Source: JICA Survey Team (Developed from the data of National Water Resources Board (NWRB), 2017)



Note：There are some locations which water permits are superimposed and therefore the number of them in Table 7.9.5 and those plotted in the figure are not always consistent.
Source：JICA Survey Team (Developed from the data of National Water Resources Board (NWRB), 2017)

Figure 7.9.7 Location Map of Water Permits Granted within the Area 500 Away from Parañaque Spillway

(5) Water Quality

To check the impact of drainage from Laguna de Bay to Manila Bay through the Parañaque Spillway on the water quality of Manila Bay, the water quality of both Laguna de Bay and Manila Bay were studied using the existing water quality data observed by DENR and LLDA. Based on the water quality comparison study, the negative impact on the water quality of Manila Bay is considered small.

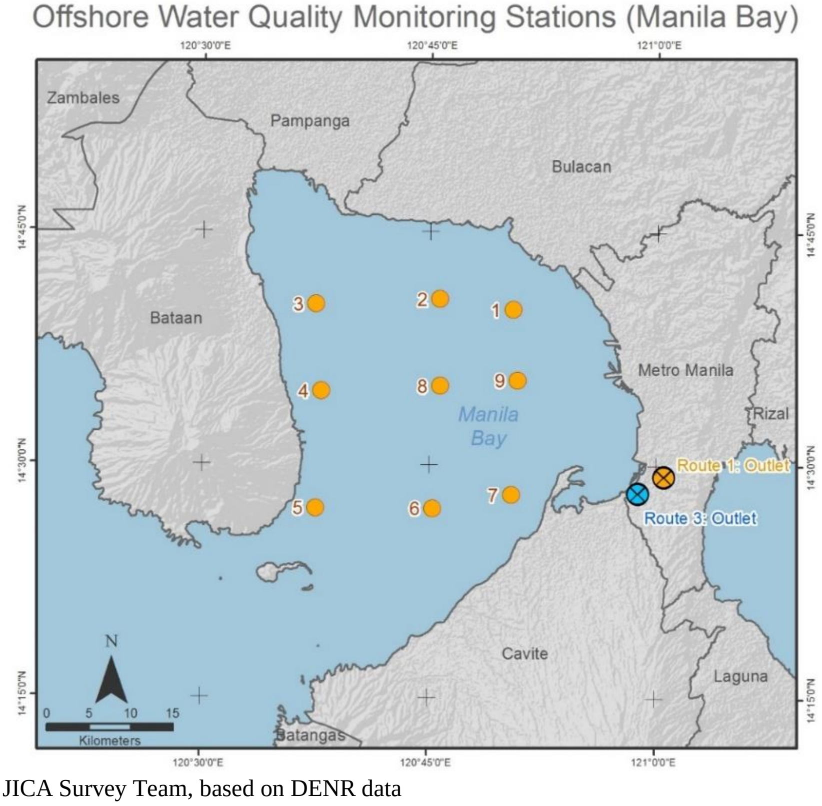
(a) Water Quality of Manila Bay and Laguna de Bay

It is difficult to estimate future water quality from past data because water quality depends on human activities. If the drainage law changes, the water quality will change drastically. If urbanization happens and population increases, water quality changes. Therefore, regarding water quality, the study focussed on the data of recent years because it represents the results of recent human activities. In this study, the data from 2013 to 2017 are analysed. The details are explained in the following items.

(i) Water Quality of Manila Bay

Water quality of the whole Manila Bay

There are nine (9) monitoring stations in Manila Bay (Figure 7.9.8). Monitoring is carried out monthly, and the objective water quality items are Dissolved Oxygen (DO), pH, salinity, water temperature and phosphate. At each point, samples are taken at three depths, surface, middle and bottom.



Source: JICA Survey Team, based on DENR data

Figure 7.9.8 Offshore Water Quality Monitoring Stations in Manila Bay

In DAO 2016-08, the water quality of saltwater is sorted in the order of excellence into Class SA, SB, SC or SD. Class SA to SC is appropriate for fishery as described in Section 6.3 (5). The annual average water quality of 2014 was obtained from DENR (Table 7.9.6).

Table 7.9.6 Annual Average Water Quality of Manila Bay in 2014

Station		DO (mg/L)	pH	Salinity (%)	Temperature (°C)	Phosphate (mg/L)
1	*Surface*	8.57	8.43	2	29.5	1
	Mid	6.89	8.83	2.33	29.1	1.05
	Bottom	3.63	8.69	2.45	29.6	1.08
2	*Surface*	8.79	8.82	1.76	29.2	0.91
	Mid	7.46	8.96	2.38	29.1	1.05
	Bottom	4.73	8.76	2.47	29.4	1.08
3	*Surface*	10.77	8.72	0.71	29	0.97
	Mid	6.85	8.89	2.41	29.2	1.08
	Bottom	5.84	8.59	2.39	29.3	1.1
4	*Surface*	9.61	8.9	1.27	29.4	1.01
	Mid	784	8.95	2.44	29.4	0.94
	Bottom	6.39	8.82	2.49	29.5	1.09
5	*Surface*	8.92	9.04	2.61	31.3	1.01
	Mid	7.14	8.82	2.66	29.4	0.91
	Bottom	4.77	8.75	2.66	29.2	1.1
6	*Surface*	8.81	9.08	2.6	31.4	1.02
	Mid	7.18	8.96	2.59	31.2	0.88
	Bottom	5.99	8.8	2.67	29.6	8.98
7	*Surface*	10.25	9.22	2.58	32.2	0.92
	Mid	8.12	8.99	2.57	30.4	0.87
	Bottom	4.54	8.84	2.64	30	1.02
8	*Surface*	8.4	9.03	2.2	28.5	1.08
	Mid	7.01	8.94	2.44	29.2	1.09
	Bottom	7.48	8.95	2.46	29.1	1.12
9	Surface	7.82	8.7	1.63	28.8	0.86
	Mid	5.71	8.7	2.47	29.3	0.81
	Bottom	4.64	8.59	2.46	29.3	1.01

Source: Manila Bay Area Environmental Atlas 2nd Edition

Table 7.9.7 Annual Average Water Quality of Manila Bay in 2014

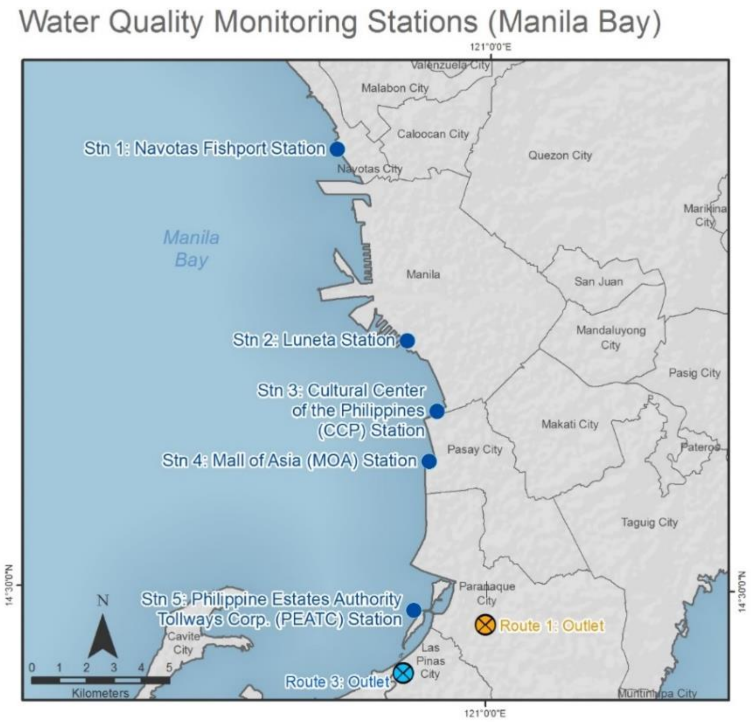
Item	Evaluation	Note
DO	Surface SA Mid SA Bottom SA to SC	Overall, the surface and mid-layer satisfy Class SA. The DO is enough for fishes, although 4 stations fail Class SC (5 mg/L) at bottom layer (Stations 1, 2, 5, 7 and 9). Compared with the DO in coastal areas near the outfalls of the Parañaque River System (South Parañaque River and San Dionisio River) and the Zapote River, which are mentioned in the next section, the DO of the offshore is better. This is because the offshore monitoring stations are far from the river outfalls where organic matters flow in and depth is deeper. The concentration of organic matters is diluted in the offshore and the consumption of oxygen is lower than in the coastal area.
pH	SD	The pH is relatively higher (8.5–9.22) than the standard on the whole. According to the “Manila Bay Area Environmental Atlas, 2nd Edition”, this is attributed to low carbon dioxide concentration due to the photosynthesis by phytoplankton. The deeper it gets the lower pH becomes, because sunlight decreases with water depth and photosynthesis becomes less active.

Item	Evaluation	Note
Salinity	N/A	There is no standard for salinity of saltwater. Salinity is the lowest (0.71%) in the surface layer of Monitoring Station 3. This is because this station is located near the outfall of Pampanga River, which occupies about 50% of freshwater that enters Manila Bay. The influence of freshwater is mainly seen on the surface layer, since freshwater is lighter than saltwater. The influence of freshwater inflow is also seen in the surface layer of Stations 4 and 9. At the other monitoring station, salinity ranges from 2.33% to 2.66%. It is smaller than the open ocean average (3.4-4%).
Temperature	SA	The water temperature of the surface layer near the mouth of the bay is relatively higher (31.3-32.2 degree Celcius). The other stations satisfy Class SA.
Phosphate	SD	Phosphate is uniform in Manila Bay and it fails Class SC. Compared with the phosphate concentration in the coastal area near the outfalls of the Parañaque River System (South Parañaque River and San Dionisio River) and the Zapote River, which are mentioned in the next section, that of the offshore is about a quarter to a half lower. The contamination level is lower due to dilution by saltwater. In 2017, the phosphate passed the SA standard in the coastal area, and that of offshore is also expected to pass.

Source: JICA Survey Team

Water Quality of Coastal Area

There are five (5) water quality monitoring stations near the outfall of the candidate river outlets of Parañaque Spillway, which are the Parañaque River System (South Parañaque River and San Dionisio River) and the Zapote River. The nearest one is Station No. 5 (PEATC). The locations are shown in Figure 7.9.9.

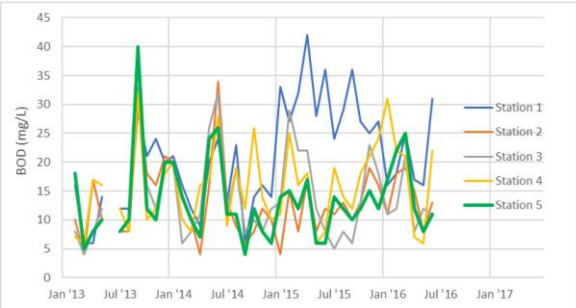


Source: JICA Survey Team based on DENR data

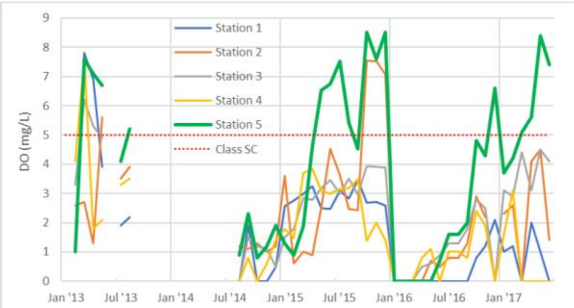
Figure 7.9.9 Coastal Water Quality Monitoring Stations in the National Capital Region

Other than those 5 stations, there are 7 stations on the outfalls. In this study, however, those 7 stations are excluded from the water quality comparison between Manila Bay and Laguna de Bay because they are instantaneous values of flowing water, while the observed values in the bay represents the accumulation of those inflow. Therefore, the data of Manila Bay are appropriate to compare the water quality of Manila Bay and Laguna de Bay and analyse the impact of inflow from Laguna de Bay through Parañaque Spillway.

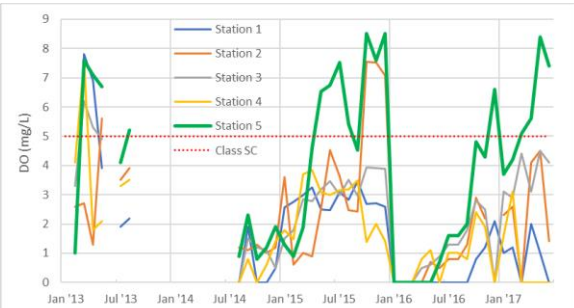
The monitoring data of 5 stations from 2013 to 2017 are as graphically shown in Figure 7.9.10 by water quality item. Class SC which is the lowest standard appropriate for fishery, is shown in red dotted lines. Except for DO, the values above the red dotted line fails Class SC. Regarding DO, the values lower than the line fail Class SC. BOD does not have any standard. The breaks of the lines show missing data. No difference between wet and dry seasons are seen.



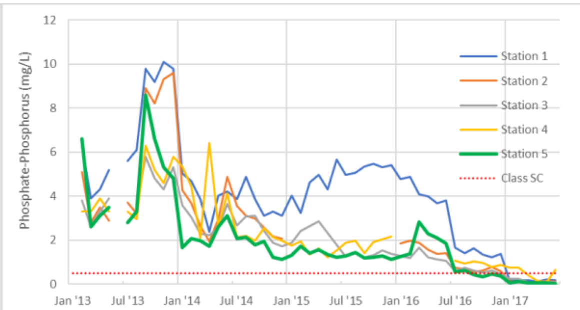
Biochemical Oxygen Demand (BOD, mg/L)



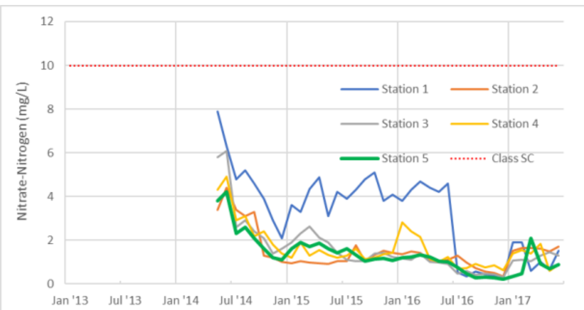
Dissolved Oxygen (DO, mg/l)



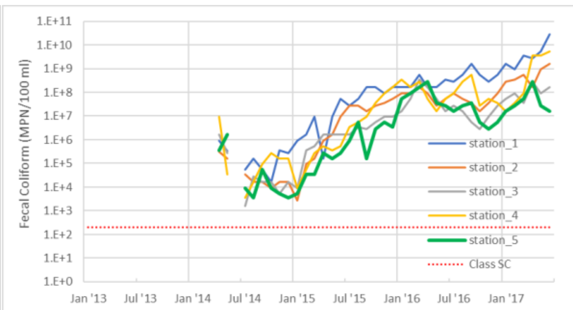
Total Suspended Solids (TSS, mg/L)



Phosphate Phosphorus (mg/L)



Nitrate Nitrogen (mg/L)



Fecal Coliform (MPN/100mL)

Source: JICA Survey Team based on DENR data

Figure 7.9.10 Water Quality of Manila Bay Coastal Area, 2013 to 2017

Table 7.9.8 Water Quality of Manila Bay Coastal Area, 2013 to 2017

Item	Evaluation	Note
BOD	N/A	BOD ranges between 5 and 40 mg/L. No seasonal trend is seen. There is no BOD standard for saltwater, but compared with that of freshwater Class C of 7 mg/L and Class D of 15 mg/L, most of the stations fail Class C and some fail Class D. This shows contamination with organic matter or ammonia. Among the coastal stations, Station 1 is the worst (consistently fail Class D).
DO	Fails SD (with exception)	DO increases or decreases periodically, but it does not depend on the change in season. DO ranges from 0 to 8 mg/L. Oxygen is almost zero from February to April in 2016 and no fish can survive there. It starts to recover in May 2016, but only Station 3 and 5 pass the Class SC of 5 mg/L. The best station is Station 5 near the LPPCHEA. It passes Class SC to SA for about half a year.
TSS	SA	Although there is temporary exceedance, TSS satisfy SA of 25 mg/L at all monitoring stations.
Fecal Coliform	Fails SD	Fecal coliform has been increasing at an exponential rate since 2014 and deteriorating the natural environment of the coastal area. The worst value is a million times larger than the Class SC of 200 MPN/100mL observed at Station 1. The best is Station 5, but still it is 100 thousand times larger than Class SC. The informal settlers and the coastal urban area are considered as the main sources of contamination.
Phosphate Phosphorus	SA	Owning to the effluent control by DENR, Phosphate Phosphorus concentration has been improving since 2013. It satisfied Class SA in early 2017.
Nitrate Nitrogen	SA	Owning to the effluent control by DENR, Nitrate Nitrogen concentration has been improving since 2013. It satisfied Class SA in early 2017.

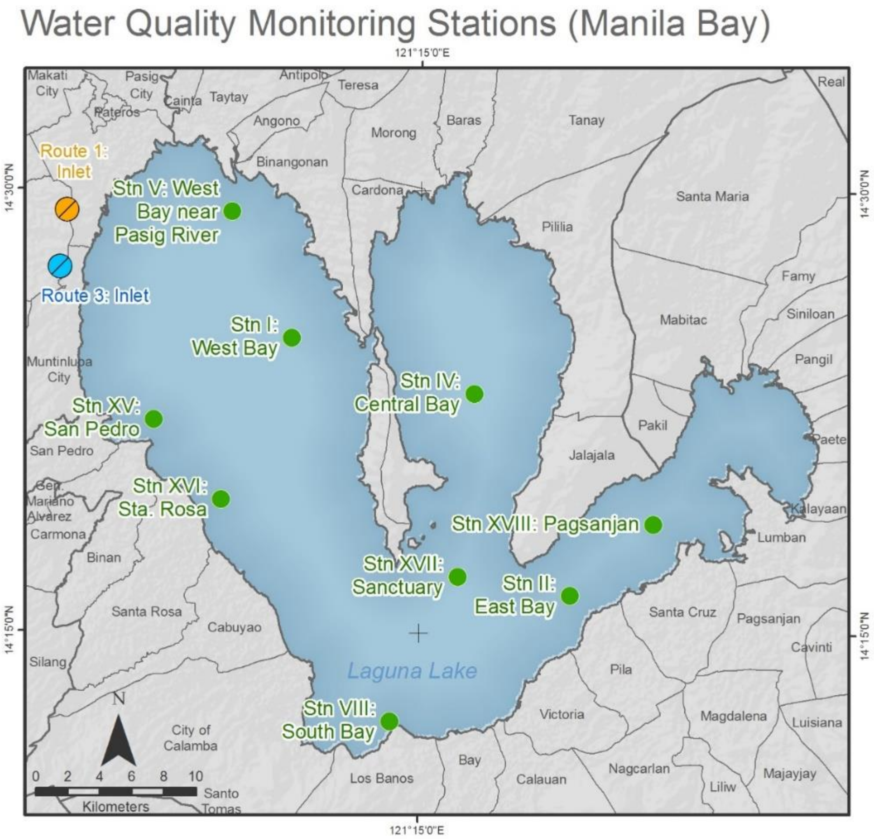
Note: Evaluation standard is DAO 2016-08, DENR.
Source: JICA Survey Team

In addition to the table above, oil and grease, chromium, and lead exceed the standards around LPPCHEA, according to the survey in May 2017 by DENR [Subsection 6.1.1(3)].

(ii) Water Quality of Laguna de Bay

According to the water quality data provided by LLDA, almost all of the water quality items of the Laguna de Bay passed Class C standard consistently from 2013 to 2017, with exception of temporary exceedance and a few water quality items: Ammonia, Oil and Grease and pH. In DAO 2016-08, the water quality of freshwater is sorted in order of excellence from AA, A, B, C and D. Class AA to Class C are considered suitable for fishery.

There are 9 stations in Laguna de Bay. Their locations are shown in Figure 7.9.11.

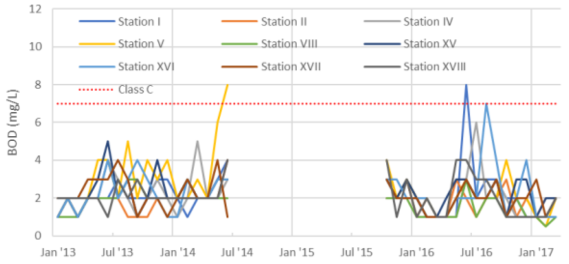


Source: JICA Survey Team based on LLDA data

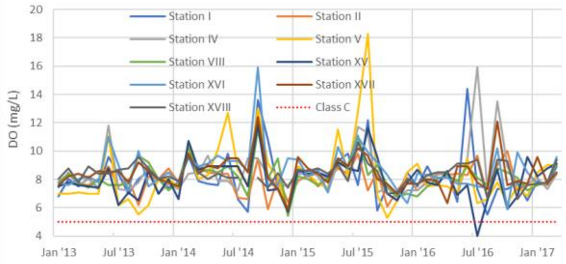
Figure 7.9.11 Water Quality Monitoring Stations in Laguna de Bay

Other than those 9 stations, there are 36 stations on the river mouths and rivers. In this study, however, those 36 stations are excluded from the water quality comparison between Manila Bay and Laguna de Bay because they are instantaneous values of flowing water, while the observed values in the lake represents the accumulation of those inflow. Therefore, the data of Laguna de Bay are appropriate to compare the water quality of Manila Bay and Laguna de Bay, and analyse the impact of Parañaque Spillway on the water quality of Manila Bay.

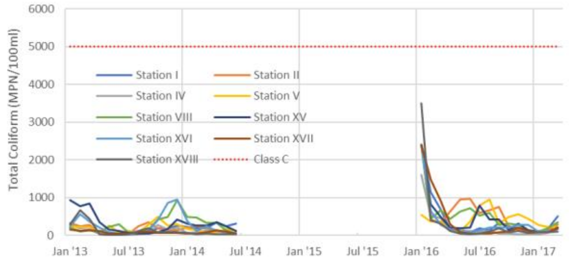
The monitoring data of 9 stations from 2013 to 2017 are as graphically shown in Figure 7.9.12 by water quality item. Class SC which is the lowest standard appropriate for fishery, is shown in red dotted lines. Except for DO, the values above the red dotted line fail Class SC. Regarding DO, the values lower than the line fail Class SC. pH need to be between two lines to pass Class C. The breaks of the lines show missing data. No difference between wet and dry seasons is seen.



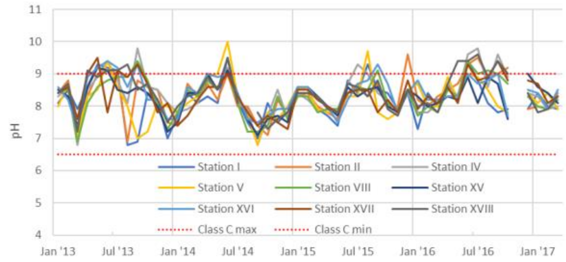
Biochemical Oxygen Demand (BOD, mg/L)



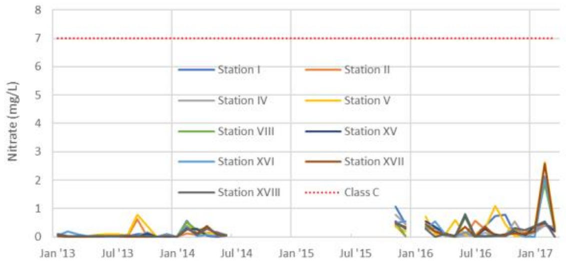
Dissolved Oxygen (DO, mg/l)



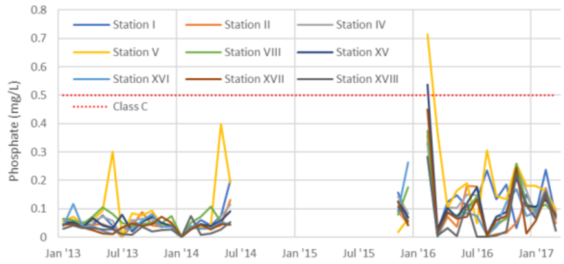
Total Coliform (MPN/100mL)



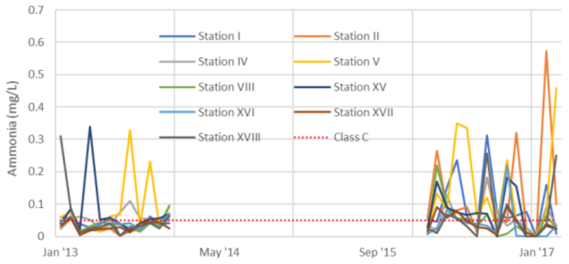
pH



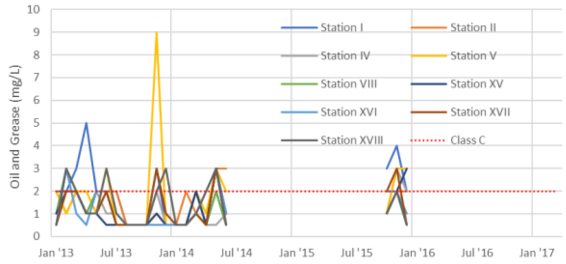
Nitrate (mg/L)



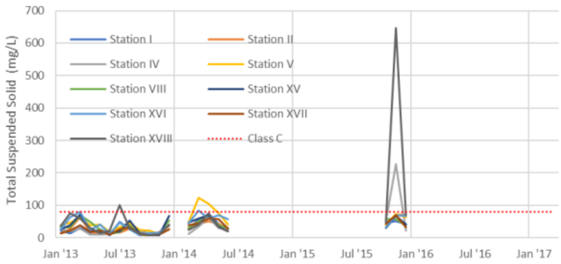
Inorganic Phosphate (mg/L)



Ammonia (mg/L)



Oil and Grease (mg/L)



Total Suspended Solids (TSS, mg/L)

Source: JICA Survey Team, based on LLDA data.

Figure 7.9.12 Water Quality of Laguna de Bay, 2013 to 2017

Table 7.9.9 Water Quality of Laguna de Bay, 2013 to 2017

Item	Evaluation	Note
BOD	AA – B	BOD of Laguna de Bay is 1 to 4 mg/L. It is the range from AA to B in DAO 2016-08 (suitable for swimming). Although Station I and V failed Class C in the past, the failure was temporary and it recovered in following month.
DO	AA	DO is rich at all the stations and passes Class AA (standards for Class AA to C are the same), which is attributed to the active photosynthesis of algae. The only failure occurred at Station XV in July 2016 and the value was 4 mg/L (Class C standard is 5 mg/L), but it recovered the following month.
Total Coliform	A – C	The number of coliform is less than 1000 MPN/100mL at all the stations in most period. When it is low, the number is less than 100 MPN/100 mL. This item is not included in DAO 2016-08. Therefore, the previous standard DAO No. 34 was referred. Based on the standard, Class A and B is less than 1000 MPN/ 100 mL and Class C is less than 5000 MPN/100mL. The water quality of Laguna de Bay passed Class C in the worst period.
pH	C – D	pH is 7.5 – 9.5 at all the stations. It is relatively higher than Class C standard (6.5 – 9.0). The factor that influences the rise of pH is expected to be the active utilization of carbon dioxide by algae at times when the condition in the lake favours their growth and reproduction.
Nitrate	AA	Nitrate has been increasing from 2013. It is relatively higher at Station V, XVI and XVII in February 2017, and they exceeded 2.0 mg/L. It is still OK because Class AA is less than 7.0 mg/L, but the increasing trend is not good for water quality.
Inorganic Phosphate	A	Inorganic phosphate has been increasing from 2013. The value was high (0.3 to 0.7 mg/L) at all the stations in February 2016, but in the other period, it is 0.01 to 0.3 mg/L and passed the Class A standard of 0.5 mg/L (Class A to C have the same standard value).
Ammonia	D	Ammonia failed Class C (0.05 mg/L) at various stations. In addition, Ammonia has been increasing from 2013. The contamination level at Stations V, I, XV and II is relatively higher than those of the other stations.
Oil & Grease	AA – D	Oil and Grease were not monitored from 2016 to 2017. The observed values were 0.5 to 4 mg/L from 2013 to 2015. The relatively higher values of 5 mg/L and 9 mg/L were observed at Stations I and V, respectively.
TSS	A – C	TSS was not monitored from 2016 to 2017. From 2013 to 2015, the range of TSS was 30 to 70 mg/L, though temporary steep surge were observed at Stations IV and XVIII. A relatively high TSS was monitored in the central bay and eastern bay, respectively, while in the western bay where the inlet construction site of the Parañaque Spillway is planned, TSS was relatively smaller because the watershed is urban area and sediment sources are few.

Note: Evaluation standard is DAO 2016-08, DENR.
Source: JICA Survey Team

<u>Contamination Source of Laguna de Bay</u>

The major contamination sources are as summarized in the Laguna de Bay 2013 Ecosystem Health Report Card and shown as Figure 7.9.13. This figure shows the inflow of nutrients, which come mainly from urban/industrial areas located in north-western to western side of Laguna de Bay. Nutrient also flows in from the livestock facility located in the south-western side of the lake. In addition, sediment enters the central bay from the mining site on the north side.

The water quality monitoring result described in Table 7.9.9 is consistent with the figure. At the monitoring stations located in the west bay, BOD is relatively higher and DO is lower and the concentration of Ammonia, and Oil & Grease is higher. Ammonia is also relatively higher in the east bay due to inflow from the livestock farming area. Regarding TSS, it is relatively higher in the central bay where sediment enters from the mining site, and the east bay which is surrounded by mountains.

Source: Laguna de Bay 2013 Ecosystem Health Report Card

Figure 7.9.13 Natural Resource Values and Human Activity Threats

(b) Comparison of Water Quality between Manila Bay and Laguna de Bay

(i) Water Quality Comparison between Offshore of Manila Bay and Laguna de Bay

As mentioned in Section 6.3(3), the impact of drainage on the water quality of Manila Bay from Parañaque Spillway seems to be small because compared with the freshwater supply from the Pampanga River, the amount of freshwater that enters Manila Bay through Parañaque Spillway is relatively smaller and it lasts only from 1 to 3 months. The water quality of Laguna de Bay is better than that of Manila Bay (at offshore). This fact also supports the above conclusion. The comparison of the annual average water quality in 2014 is summarized in Table 7.9.10. It is noted that, phosphate data in Laguna de Bay include a lot of missing data and it is impossible to obtain the annual average. Therefore, the average phosphate data of 2016 was used for the comparison. Compared with that of 2013, phosphate in 2016 is 2.5 times larger.

Table 7.9.10 Water Quality Comparison between Manila Bay (Offshore) and Laguna de Bay

Item	Manila Bay Offshore Evaluation	Laguna de Bay Evaluation	Comparison
DO	SA 7.19 mg/L	AA 8.54 mg/L	Both Manila Bay and Laguna de Bay are rich in oxygen and are appropriate for fishes.
pH	SD 8.84	AA 8.13	pH is higher in Manila Bay. It is attributed to photosynthesis by phytoplankton and photosynthetic micro-organisms. Laguna de Bay is better in terms of pH.
Phosphate	SD 1.3 mg/L	A 0.123 mg/L	Laguna de Bay satisfies Class A; Manila Bay (offshore) fails Class SC.
Salinity	– 2.31%	AA 0.02%	The salinity of Laguna de Bay is normally almost zero. When saltwater intrusion occurs, it increases up to about 0.18%. The salinity of Manila Bay is lower than the average salinity of open sea of 3.5 – 4%. The reason is, that the Manila Bay is an inner bay and the tidal current speed is relatively slow.

Note: Evaluation standard is DA O2016-08, DENR.
Source: JICA Survey Team

(ii) Water Quality Comparison between Coast of Manila Bay and Laguna de Bay

To check the impact on the water quality around LPPCHEA, the water quality of the coastal areas of Manila Bay (see Figure 7.9.9) and Laguna de Bay were compared. The compared value is the half-year average of 2016 (July to December). The period from July to December was chosen because the water quality changes with season and the annual average does not represent water quality in the period when Parañaque Spillway is operated. The period was decided based on the long-term simulation (see the Section 3.4.1 (8)).

It is noted that the BOD and pH are of 2015, because data in Manila Bay in 2016 was not available. The result is shown in Table 7.9.20.

Table 7.9.11 Water Quality Comparison between Coast of Manila Bay and Laguna de Bay

Item	Manila Bay Coast Evaluation	Laguna de Bay Evaluation	Comparison
BOD	N/A 16.9 mg/L	A 3.61 mg/L	BOD of saltwater cannot be evaluated because the standard is not defined. However, compared with the standard for freshwater, the BOD of Manila Bay fails Class D, while that of Laguna de bay passes Class A.

Item	Manila Bay Coast Evaluation	Laguna de Bay Evaluation	Comparison
DO	Fails D 1.71 mg/L	A 8.01 mg/L	The dissolved oxygen of Laguna de Bay is enough for fishes, while that of the coast of Manila Bay is too small for fishes to survive.
Fecal coliform	Fails SD 180 Million MPN/100mL	—	Fecal coliform is not monitored in Laguna de Bay. Fecal coliform in the coast of Manila Bay is 100 thousand to 1 million times larger than the standard. It has been getting worse.
Total coliform	—	OK 262 MPN/100mL	This item is not monitored in the coast of Manila Bay. The total coliform of Laguna de Bay passes Class A (<1000 MPN/100mL). The evaluation was done under DAO No. 34, because DAO 2016-08 does not include a standard for total coliform.
pH	SD 6.3	AA 8.42	The pH of the coast of Manila Bay is lower than Class C; The pH of Laguna de Bay is within Class AA.
Nitrogen	SA 0.55 mg/L	AA 0.17 mg/L	Both the coasts of Manila Bay and Laguna de Bay are top rated.
Phosphorus	SD 0.8 mg/L	A 0.105 mg/L	Phosphorus of Laguna de Bay passes Class A. Phosphorus of the coast of Manila Bay is Class SD, but it is presently improving.
Ammonia	—	D 0.07 mg/L	Ammonia is not monitored in the coast of Manila Bay, but it is assumed to be high, because a lot of fecal coliform implies the inflow of human waste. Ammonia of Laguna de Bay fails Class C of 0.05 mg/L.
TSS	SA 13.1 mg/L	—	TSS of the coast of Manila Bay passes top rating of SA. TSS has not been measured in Laguna de Bay since 2015 and in 2015 monitoring was carried out in October to December only. Only the data of the objective period in 2013 is available, and it is about 24 mg/L.

Note: Evaluation standard is DAO 2016-08 of DENR, but total coliform was evaluated with DAO No. 34.
Source: JICA Survey Team

As a result of comparison, it was found that the water quality in Laguna de Bay is better than in the coast of Manila Bay. Especially, the difference in DO and fecal coliform is significant. The DO in Laguna de Bay is rich, while that of the coast of Manila Bay is so small that fishes asphyxiate. The total coliform of Laguna de Bay passes the standard, but that of the coast of Manila Bay is 100 thousand to 1 million times larger than the standard and it has been deteriorating. TSS in the coast of Manila Bay is better than that of Laguna de Bay.

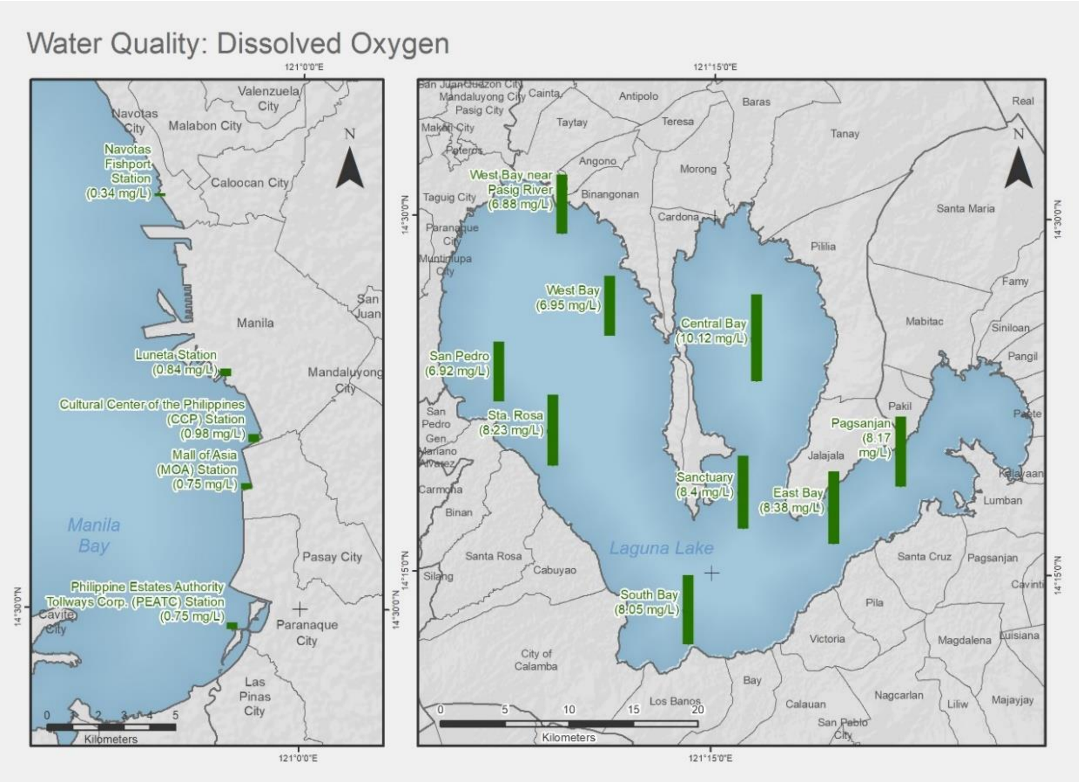
Accordingly, the inflow from Laguna de Bay has positive impact on the water quality of Manila Bay. TSS might have a negative impact. However, considering the settling basin function of Laguna de Bay, the TSS constitutes wash load. Therefore, it is expected to be washed away to the offshore with the momentum of drainage and does not accumulate in the coastal area of Manila Bay.

The spatial distribution of DO and coliform is shown in Figure 7.9.14.

Spatial Distribution of DO

Average DO (JUL-DEC 2016) in the coast of Manila Bay and Laguna de Bay is shown in Figure 7.9.14. DO in the coast of Manila Bay is less than one-tenth of the average of Laguna de Bay. The lowest station is the Navotas Fishport Station. In Laguna de Bay, DO of the central bay is the richest (10.1 mg/L), the second is the east and south bays (8 to 8.4 mg/L). The lowest is the north-western area (6.8 – 7 mg/L) due to nutrient inflow from urban and industrial areas, but still passes the Class AA standard of 5 mg/L.

Based on the survey result, the drainage from Laguna de Bay is expected to improve DO of the coast of Manila Bay.

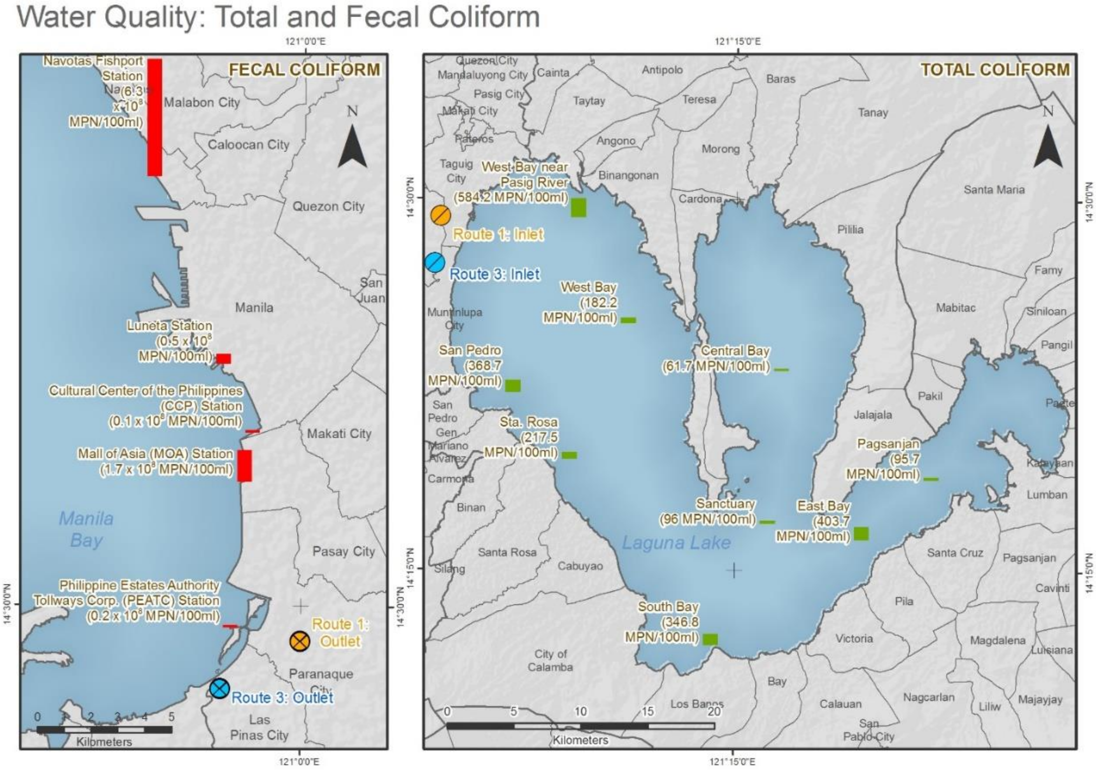


Source: JICA Survey Team

Figure 7.9.14 Comparison of DO in Laguna de Bay and Coast of Manila Bay

Spatial Distribution of Coliform

The average fecal coliform of the coast of Manila Bay and total coliform of Laguna de Bay for the period from July to December 2016 are shown in Figure 7.9.15. Fecal coliform is a group in the total coliform that is directly related to human or animal waste. In the coast of Manila Bay, fecal coliform is 100 thousand to 1 million times larger than the standard. The largest is found at the Navotas Fishport Station which is 1 million times larger than the standard. The smallest is at PETAC Station, but it is still 100 thousand times larger than the standard. On the other hand, the total coliform of Laguna de Bay passes the standard. Relatively larger values are observed at two areas, north-western area and south-eastern area. The contaminated water from the urban area is expected to be the cause of the larger value in the north-western area while the animal waste inflow from the livestock farming area is expected to be the cause in the south-eastern area.



Source: JICA Survey Team

Figure 7.9.15 Total Coliform in Laguna de Bay and Fecal Coliform in the Coast of Manila Bay

(iii) Impact on Salinity

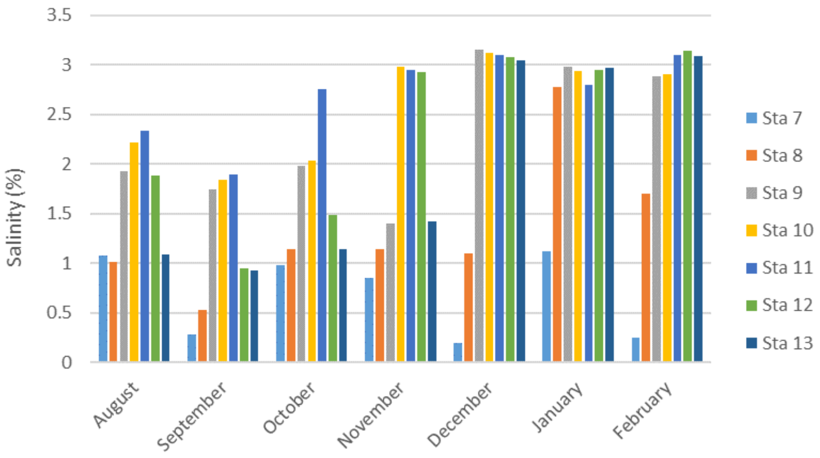
The drainage through Parañaque Spillway will increase the freshwater amount entering the Manila Bay and decrease salinity near the river mouth. LPPCHEA is located near the river mouth of the Parañaque or Zapote River, which are the candidate river outlets of Parañaque Spillway, so that the impact of decline of salinity on the LPPCHEA should be considered. To check the impact, it is necessary to understand the current salinity around LPPCHEA, especially when freshwater flows into the area.

However, salinity is not monitored around LPPCHEA and no observation data exists. Therefore, the JICA Survey Team carried out the salinity measurement around LPPCHEA monthly from August 2017 to February 2018. The device used for the measurement was LAQUAact74D with pH electrode 9625-10D and electric conductivity cell 9382-10D from HORIBA Ltd.

The JICA Survey Team also conducted salinity survey near the planned intake site of Parañaque Spillway. The observed salinity was consistently about 0.02%, which passes the Class AA standard for freshwater.

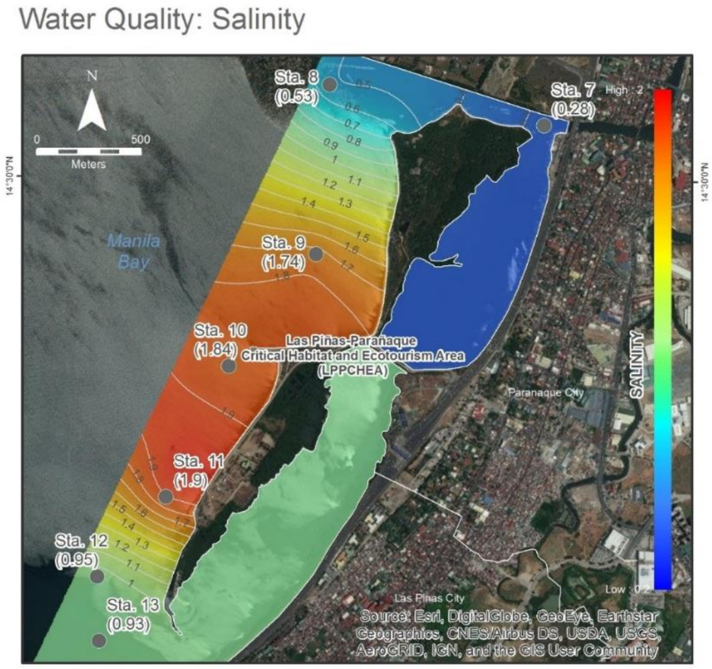
Source: JICA Survey Team

Figure 7.9.16 Measurement Device (Left) and Measurement (Right)



Note: The location of each station is shown in Figure 7.9.18.
Source: JICA Survey Team

Figure 7.9.17 Observed Salinity around LPPCHEA



Source: JICA Survey Team

Figure 7.9.18 Salinity around LPPCHEA in September 2017

Salinity is relatively lower at the outfalls where freshwater flows in. Salinity decreases to 0.2% at the outfall of the Parañaque River in September while the salinity at the outfall of the Zapote River is 0.93%. It is higher than that at the outfall of the Parañaque River. The difference may have been caused by the geographic difference. The river mouth of the Parañaque River is surrounded by the reclaimed area and LPPCHEA, which increases the concentration of freshwater.

The amount of drainage water to decrease the salinity around LPPCHEA through Parañaque Spillway is not clear, but it was found that the decrease in salinity is a normal phenomenon around LPPCHEA. Therefore, drainage through Parañaque Spillway is not expected to devastate the environment around LPPCHEA.

(6) Natural Environment (Details of LPPCHEA)

The overview and history of LPPCHEA are described in Section 6.2 (4). In this section, flora and fauna and their habitat is explained in the order of plant species, Macro-invertebrate and fishes and avian species.

(a) Plants

The plant species in the LPPCHEA are shown in Table 7.9.12. Among them mangroves are particularly important to preserve the natural environment of LPPCHEA. Mangroves comprise 36 ha of LPPCHEA, which is about 18% of the island. LPPCHEA's mangrove forest is known as the thickest and most diverse among the remaining mangrove areas in Manila Bay. In the past, a lot of mangrove areas existed in Manila Bay, but they were lost with the development of the country. LPPCHEA has been drawing attention as the last remaining thick and diverse mangrove area.

Mangroves are salt-tolerant trees adapted to living in salt and brackish water conditions. Mangroves do not need saltwater to live, but they need freshwater like other plant species. Mangroves just have an ability to remove salt from saltwater. The reason why the mangroves are not seen in forests on lands is a natural selection. The other plant species are superior to them in freshwater conditions. On the other hand, in salt and brackish water conditions, only mangroves can survive and mangrove forests develop therein.

The increase of freshwater entering near LPPCHEA due to the Parañaque Spillway is favourable for mangroves because they can save energy to remove salt from saltwater. Moreover, if the salinity decreases temporarily due to freshwater inflow, there will be no room for other plant spices to intrude, because salt will be unlimitedly provided from the ocean, and the salinity will recover after the drainage from Parañaque Spillway. Therefore, the inflow from Parañaque Spillway is expected to have no negative impact on the mangroves in LPPCHEA. The plant species in LPPCHEA are shown in Table 7.9.21.

Table 7.9.12 Plant Species in LPPCHEA

PLANT SPECIES	Total Number of Individuals (n)		Total
	Free Island	Long Island	
Mangrove Species			
Bungalon *(Avicennia marina)*	516	1,589	2,105
Kulasi *(Lumnitzera racemose)*	39	13	52
Pagatpat *(Sonneratia alba)*	98	76	174
Bakauan *(Rhizophora spp.)*	-	681	681
Pototan *(Bruguiera sexangula)*	-	1	1
Nipa *(Nypa fruticans)*			2
Mangrove-associated species			
Banalo *(Thespesia populnea)*	6	3	9
Bangkoro *(Morinda citrofolia)*	43	-	43
Buta-buta *(Excoecarcia agallocha)*	-	7	7
Beach type species			
Alagau *(Premna odorata)*	1	-	1
Aroma *(Acacia fernesiana)*	65	65	130
Talisai *(Terminalia catappa)*	12	2	14
Other plant species			
American kapok *(Ceiba pentandra)*	1	-	1
Atis *(Annona squamosa)*	1	-	1
Aure *(Acacia auricouliformis)*	1	-	1
Castor Oil *(Ricinus communis)*	3	-	3
Datiles *(Muntingia calabura)*	60	1	61
Guava *(Psidium guajava)*	6	-	6
Ipil-ipil *(Leucaena leucocephala)*	262	49	311
Jathropa *(Jathropa curcas)*	4	-	4
Malungai *(Moringa oleifera*	4	-	4
Kamachile *(Pithecellobium dulce)*	-	2	2
Sampaloc *(Tamarindus indica)*	-	1	1

Source: JICA Survey Team

(b) Macro-invertebrate and Fish Species

LPPCHEA has 114 hectares of mudflat which serve as feeding grounds for shore birds. In the mudflats macro-invertebrates and fish species live. Macro-invertebrates include polychaetes represented by mud worms, crustaceans and molluscs. Molluscs are the most abundant, and they include 23 species of bivalves and 14 species of gastropods.

Eight (8) species of juvenile to sub-adult are also found near LPPCHEA, which indicates the significant function of mangroves as spawning grounds, nursery, feeding and temporary shelter. The list of Macro-invertebrates and fish species in LPPCHEA is given in Table 7.9.22.

Table 7.9.13 Macro-invertebrates and Fish Species in LPPCHEA

MOLLUSCS			
BIVALES		GASTROPODS	
Scientific Name	English Name	Scientific Name	English Name
Anadara antiquata	AntiqueArk	*Cantharus (pollia) fumosus*	Smoky Goblet
Anadara granosa	Blood Cockle	*Cerithium sp*	Cerith
Andara maculosa	Ark Shell	*Clypeomorus batillariaeformis*	Necklace Cerith
Arca navicularis	Indo-Pacific ark	*Cronia margariticola*	Pearl-Shell-Inhabiting Murex
Arca ventricosa	Ventricose Ark	*Euchelus atratus*	Euchelus
Babatia foliate	Decussate Ark	*Monodonta labiate*	Monodont
Chama sp.	Jewel Box	*Nassarius olivaceous*	Mud Snail
Crassostrea glomerate	Auckland Oyster	*Nassarius pullus*	Nassa
Crassostrea iredalei	Philippine Cupped Oyster	*Pyrene scripta*	Dotted Dove Shell
Culcullea labiata	Culcullea	*Strombus canarium*	Dog Conch
Gafrarium pectinatum	Comb Venus	*Strombus urceus*	Little Pitcher Conch
Gafrarium tumidum	Tumid Venus	*Tonna sulcosa*	Banded Tun
Gari togate	Courtesan Sunset Clam	*Umbonium moniliferum*	Costate Button Top
Gloriopallium pallium	Royal Cloak Scallop		
Katelysia hiantina	Hiant Venus		
Lioconcha castrensis	Camp Pitar Venus		
Perna virdis	Asian Brown Mussel		
Pinotada margaritifera	Pacific Pearl-Oyster		
Placeman calophylla	Woodcarving Cake		
Spondylus squamosus	Ducal Thorny Oyster		
Tellina staurella	Cross Tellin		
Vepricardium multispinosum	Many-spined Heart Cockle		
CRUSTACEANS		POLYCHAETES	
Scientific Name	English Name	Scientific Name	English Name
Amphibalanus Amphitrite	Striped Barnacle	*Nereiid polychaete*	Rag Worms

Source: JICA Survey Team

(c) Avian Species

Owning to the 114 hectares of mudflats that are abundant in bird food, molluscs and other bottom-dwelling and small aquatic animals, LPPCHEA is a good habitat for avian species. In addition, a lot of migratory birds visit LPPCHEA as an overwintering site from August to April and the number of birds reaches 5,000.

Based on the survey by DENR in 2004 to 2008, 44 species of birds roosted in LPPCHEA. Twenty-nine (29) of these species were migratory birds and include Egretta eulophotes, an endangered species. The other 15 species were resident birds that also include an endangered species, the Philippine Duck.

There are a lot of shore birds in LPPCHEA. Among them the largest population species is the Black-winged Stilts, and its population was 1,000 heads. This makes LPPCHEA a wetland of global ecological importance following the RAMSAR convention. The bird species in LPPCHEA is summarized in Table 7.9.23.

Table 7.9.14 Macro-invertebrates and Avian Species in LPPCHEA

Scientific Name	English Name	Scientific Name	English Name
Phaethon rubricauda	Red-tailed Tropobird	*Tringa glareola*	Wood Sandpiper
Ardea cinerea	Grey Heron	*Tringa stagnatilis*	Marsh Sandpiper
Ardea purpurea	Purple Heron	*Actitis hypoleucos*	Common Sandpiper
Ardea alba	Great Egret	*Heteroscelus brevipes*	Grey-tailed Tattler
Egretta intermedia	Intermediate Egret	*Arenaria interpes*	Ruddy Turnstone
Egretta eulophotes	Chinese Egret	*Calidris ruficollis*	Rufous-necked Stint (Red-necked Stint)
Egretta garzetta	Little Egret	*Calidris subminuta*	Long-toed Stint
Egretta sacra	Pacific Reef Egret	*Calidris acuminata*	Sharp-tailed Sandpiper
Butorides striata	Little Heron (Striated Heron)	*Philomachus pugnax*	Ruff (Reeve)
Nycticorax nycticorax	Black-crowed Night Heron	*Glareola maldivarum*	Oriental Pratincole
Nycticorax caledonicus	Rufuos Night-Heron	*Hiamantopus himantopus*	Black-winged Stilt
Ixobrychus cinnamomeus	Cinnamon Bittern	*Recurvirostra avosetta*	Pied Avocet
Ixobrychus sinensis	Yellow Bittern	*Larus ridibundus*	Black-headed Gull (Common Black-headed Gull)
Anas luzonica	Philippine Duck	*Sterna hirundo*	Common Tern
Anas clypeata	Northern Shovoler	*Chlidonias leucopterus*	White-winged Tern (White -winged Black Tern)
Aythya fuligula	Tufted Duck	*Chlidonias hybridus*	Whiskered Tern
Pandion haliaetus	Osprey	*Streptopelia bitorquata*	Island Collared-Dove
Haliastur indus	Brahminy Kite	*Streptopelia chinensis*	Spotted Dove (Spotted-necked Dove)
Falco tinnunculus	Eurasian Kestrel (Common Kestrel)	*Geopelia Striata*	Zebra Dove
Gallirallus torquatus	Barred Rail	*Macropygia tenuirostris*	Philippine Cuckoo-Dove
Porzana cinerea	White-browed crake	*Loriculus philippensis*	Colasisi
Amauromis phoenicurus	White-breasted-Waterhen (White-breasted Bush-hen)	*Centropus bengalensis*	Lesser Coucal
Gallinula chloropus	Common Moorhen	*Caprimulgus affinis*	Savanna Nightjar
Pluvialis fulva	Asian Golden-Plover (Pacific Gloden-Plover)	*Alcedo atthis*	Common Kingfisher
Charadrius dubius	Little Ringed-Plover	*Halcyon smyrnensis*	White-throated Kingfisher
Charadrius alexandrines	Kentish Plover	*Todirhampus chloris*	White-collared Kingfisher (Collared Kingfisher)
Charadrius mongolus	Lesser Sand-Plover (Mongolian Plover)	*Hirundo rustica*	Barn Swallow
Numenius phaeopus	Whimbrel	*Hirundo tahitica*	Pacific Swallow
Limosa lapponica	Bar-tailed Godwit	*Pycnonotus goiavier*	Yellow-vented Bulbul
Tringa tetanus	Common Redshank	*Ixos Philippinus*	Philippine Bulbul
Tringa nebularia	Common Greenshank	*Oriolus Chinensis*	Blacke-naped Oriole
Luscinia calliope	Siberian Rubythroat	*Rhipidura javanica*	Pied Fantail
Gerygone sulphurea	Golden-bellied Flyeater (Golden-bellied	*Motacilla cinereal*	Grey Wagtail

Scientific Name	English Name	Scientific Name	English Name
	Gerygone)		
Phulloscopus borealis	Artic Wrabler	*Motacilla flava*	Yellow Wigtail
Acrocephalus Stentoreus	Clamorous Reed-Warbler	*Lanius cristatus*	Brown Shrike
Acrocephalis orientalis	Oriental Reed-Wabler	*Aplonis panayensis*	Asian Glossy Starling
Megalurus palustris	Striated Grassbird	*Acridotheres cristatellus*	Crested Mynah
Locustella ochotensis	Middendorff's Grasshopper-Wabler (Middendorff's Warbler)	*Passer montanus*	Eurasian Tree Sparrow
Cisticola exilis	Bright-capped Cisticola (Golden-headed Cisticola)	*Lonchura punctulate*	Scaly-brested Munia
Cisticola juncidis	Zitting Cisticola (Fan-ailed Cisticola)	*Lonchura malacca*	Chestnut Munia
Eudynamys Scolopacea	Common Koel	*Phalacrocorax carbo*	Great Cormorant
Terpsiphone atrocaudata	Japanese Paradise Flycatcher		

Source: JICA Survey Team

7.9.2 Confirmation of Law and System of Environmental Assessment

(1) Screening of Project Facilities based on PEISS

The construction of Parañaque Spillway, including intake and drainage facilities and open channel in this project would cause significant impacts on the environment and socio-economy. As described in Section 6.2, screening, i.e., application of PEISS to these facilities, was carried out based on Annex A, EMB MC 2014-005. These structures/facilities fall under "3.1, Dams, Water Supply and Flood Control Project of Clause 3. Infrastructure Projects" of Annex A.

However, there is no exact conformance between the type of structures/facilities listed in Annex A and the structures/facilities proposed in the project. The JICA Survey Team, therefore, explained these points to DENR-EMB, a competent agency of PEISS, and eventually obtained their opinion on the EIS requirements of the proposed structures/facilities as follows:

"The project is considered to be an environment enhancement project, which is to be categorized as C (in Section 6.2). However, based on the project scale and size of the structures/facilities and taking into account that similar projects were required to conduct EIA to secure an ECC, it is natural to require the EIA study for this project. It is, therefore, necessary for the Proponent (DPWH) to submit a Project Description (PD) to the competent authority (DENR-EMB) in advance for the determination of EIS requirements."

Table 7.9.15 Screening of Flood Control Facilities based on PEISS

Projects/ Description	Covered (Required to secure ECC)			Not Covered	Project Size Parameters / Remarks
	Category A: ECP*	Category B: Non-ECP		Category D	
	EIS	EIS	IEE Checklist	PD	
3. INFRASTRUCTURE PROJECTS					
3.1 Dams, Water Supply and Flood Control Project					
3.1.1 DAMS (including those for irrigation, flood control, water source and hydropower projects), including run-of-river type	≥ 25 ha, OR ≥ 20 million m^3	> 5 ha, but < 25 ha, OR > 5 million m^3, but < 20 million m^3	≤ 5 ha AND ≤ 5 million m^3	None	Reservoir flooded/ inundated area and/or water storage capacity

ECP*: Environmentally Critical Project
Source：Annex A: Project Thresholds for Coverage Screening and Categorization, EMB-MC 2014-005

(2) The Act Facilitating the Acquisition of Right-of-Way Site or Location for National Government Infrastructure Projects (RA No. 10752-2016) and its IRR, as well as DPWH-DO NO. 203-2016

Land acquisition and resettlement necessary for infrastructure development projects should be carried out pursuant to the legal framework shown in Section 6.2. As for the land acquisition and resettlement to be caused by the construction of open channel and drainage facilities under this project, necessary procedures should be securely performed in accordance with the relevant laws and regulations, especially, RA No. 10752 of 2016, An Act Facilitating the Acquisition of Right-of-Way Site or Location for National Government Infrastructure Projects, and its IRR, which was promulgated through the drastic amendment of the former Act, RA 8974 of 2000, in order to facilitate the effective and expeditious implementation of infrastructure projects. The following are the important points of RA No. 10752 in case of land acquisition in this Project:

- The current market value of the land for ROW of infrastructure projects should be offered as compensation for the acquisition [Section 5 of RA No. 10752 and Section 6 of its IRR)].
- The government or any of its authorized representatives should not be prevented from entry into and use of the subsurface or subterranean portions of such private and government lands by surface owners or occupants, if such entry and use are made more than fifty (50) meters away from the surface for infrastructures such as subways, tunnels, underpasses, waterways, floodways, etc. (Section 4 of RA No. 10752 and Section 11 of its IRR.)

7.9.3 Natural Environment

(1) Impact on Manila Bay

As mentioned in Section 6.3 (3), the impact on Manila Bay seems to be small. There are three reasons as given below.

(a) Amount of Freshwater

Pampanga River contributes approximately 50% of all freshwater that enters Manila Bay. Compared to the water from Pampanga River, the increase in flow rate by the Parañaque Spillway is smaller, and the total amount of freshwater does not change. Therefore, decrease in the density of chloride in Manila Bay is unlikely.

(b) Water Quality

Owning to the control by LLDA, the water quality of Laguna de Bay is better than that of Manila Bay.

(c) Sediment

Sediment concentration of the water discharged through the spillway is expected to be small because Laguna de Bay will work as a settling basin. In addition, the tributaries which are the main sediment sources enter the central and eastern parts of the lake, but the intake of the spillway will be constructed in the western part of the lake. Considering the low current velocity in the lake, the transport of sediment to the intake is also unlikely.

In addition, as mentioned in Subsection 7.6.1(5), the water quality of Laguna de Bay is better than that of Manila Bay (offshore stations). The fact also supports that the impact on Manila Bay is small. The regional impact near the outfall of drainage-river is described in the next section.

(2) Impact on LPPCHEA

Based on the survey result, the drainage through Parañaque Spillway is not likely to have a negative impact on LPPCHEA but seems to have a positive impact. There are three reasons, as below.

(a) Water Quality

According to the water quality data provided by LLDA and DENR, the water quality of Laguna de Bay is better than that around LPPCHEA. The inflow of oxygen-rich water will improve the low dissolved oxygen of the water around LPCHEA and the significantly high number of fecal coliform will be diluted with the water from Laguna de Bay. Although the TSS of Laguna de Bay is a little bit higher than that of the coast of Manila Bay, it will be washed away with the momentum of drainage and not likely to dwell in that area, because it is expected to consist of relatively fine sediments due to settlement in Laguna de Bay.

(b) Impact of Freshwater

If the Parañaque Spillway increases the amount of freshwater entering the area near LPPCHEA, the mangroves will not be devastated because they do not need saltwater to survive. If the mangroves survive, the ecosystem fishes, birds, etc., will be preserved.

(c) Temporary Event

The drainage through the spillway is a temporary event that lasts from 1 to 3 months. After drainage finishes, the environment restores to its normal state. The salinity also rises to its normal level and it maintains the environment that is suitable for mangroves.

For these three reasons, the negative impact on the LPPCHEA is less likely to happen. However, the impact of the increased inflow by Parañaque Spillway was not quantitatively assessed because the flow regimes of the Zapote and Parañaque rivers were not surveyed in this study. A flow regime survey should be done in the next study.

(3) Water Quality Simulation

(a) Objective

The objective of the water quality simulation is to quantitatively understand the impact of drainage on LPPCHEA through Parañaque Spillway. In order to understand and achieve this objective, it is necessary to compute water quality changes, extents and durations with a water quality simulation model. The quantitative result is important to explain the impact on LPPCHEA to the conservation groups.

(b) Analysis Method and Study Items

An overview of the analysis method and study items is given below.

1. Modelling Area	• Whole Manila Bay (to set boundary conditions at the mouth of the bay) • 15 major rivers that enter Manila Bay (water quality and flow regime) • Sewage plant that discharge to Manila Bay (amount of effluent)
2. Simulation period and Computation time steps	• Computation period is before draining to the period when the salinity of the coast of Manila Bay becomes normal level (July to January or February seem to be enough). • Computation time step is a minute to consider tide. • Input data hydrological data, weather data are hourly
3. Mesh size	• Mesh sizes are 100 m near the outfall. • Mesh sizes become larger with the distance from the outfall.
4. Water quality item to be modelled	• Select items which pose big impact from the existing data. (e.g., water temperature, salinity, DO, coliform, phosphate, nitrate, zooplankton, phytoplankton, TSS, etc.)
5. Input data	• Seabed topography • The water quality of Manila Bay (offshore) • The water quality and discharge of the 15 major rivers that enter Manila Bay • Water quality of Laguna de Bay. • Tide level at mouth of the bay and near LPPCHEA • Bottom sediment data (sediment diameter distribution and amount of organic materials) • Effluent from large sewage plants and factories and their water quality.
6. Flora and Fauna	• Only include planktons and exclude other animals. • The impact on other animals will be considered based on the simulation result
7. Flushing by Drainage	• Modeling the movement of bottom sediment is costly and takes a long time. Therefore, bottom sediment will be considered based on

the simulation result, current velocity, tractive force and diameter of sediment.

- The impact on the roots of mangroves will be considered in the same manner as above.
- Rising of heavy metals in the bottom sediment will also be considered with the simulation result.

(c) Additional Survey

Water quality data and seabed topography were obtained in this study from DENR, LLDA and NAMRIA. There are still some data that need to be collected or surveyed to carry out water quality simulation. The necessary additional surveys will be as follows:

- Topographic Survey of the mudflat area near the outfalls of Parañaque or Zapote river at 50 meters interval (half length of the mesh sizes)
- Survey of bottom sediment near the outfalls of Parañaque and Zapote rivers (organic material, COD, Nitrate, Phosphate and particle size distribution)
- Survey of planktons in water in both Manila Bay and Laguna de Bay (number of zooplankton and phytoplankton)
- Survey/collection of latest water quality data of the offshore of Manila Bay to calibrate the water quality simulation model
- Survey of organic materials in water of both Manila Bay and Laguna de Bay

7.9.4 Social Environment Consideration

(1) Impacts of Construction of the Intake Facility and Necessary Considerations

Major potential impacts of construction of intake facility will be the following four:

- Impacts on fishery (open lake fishing and aquaculture) in Laguna de Bay;
- Impacts on water transportation and navigation routes in Laguna de Bay;
- Impacts on cultivation and harvesting of water plants in Laguna de Bay; and
- Impacts on water use (irrigation, domestic water supply, etc.) in Laguna de Bay.

The details of these potential impacts and necessary considerations are shown in Table 7.9.16.

Table 7.9.16 Potential Impacts of Construction of the Intake Facility and Necessary Consideration

Potential Impacts	Description of Impacts	Consideration
1. Impact on fishery	<u>Area around Candidate Site No. 1</u>: Aquaculture has developed in Laguna de Bay at offshore of Barangay Lower Bicutan, Taguig City. Although the distance between fish pens/fish cages and shoreline is more than 200m (in accordance with the rule of LLDA), these fishing facilities might be affected by the construction of intake facility of the project. There are mooring facilities used for fishery south of the candidate site. These facilities might be affected depending on the construction site of the intake facility. <u>Area around Candidate Site No. 2:</u> Fish traps are distributed at about 250m offshore and fish pens/fish cages are at about 450m to 500m in Laguna de Bay. There facilities might be affected by the construction of intake facility. No mooring facility is seen near the candidate site.	Selection/determination of location of intake facility of the project and construction planning should be done considering the avoidance/minimization of the impact on fishery. In case the impact on fishery is inevitable, proper compensation should be provided through coordination with relevant fisher folks/associations, mutual agreement on compensation amount and the timing of provision of compensation, namely; before construction of the project facility, etc.
2. Impact on water transportation	There was a water transportation route at the south of Candidate Site No. 2 to transport oil used in the Sucat Thermal Power Plant. At present, however, the transportation route is not being used because of the stoppage of operation of the thermal power plant. The transportation route might be reused in the future and there might then be an impact of the project on the transportation route.	Coordination with relevant organizations of the thermal power plant should be made regarding location of intake points of the project to avoid impact on water transportation.
3. Impact of cultivation and harvesting of water plants	Cultivation and harvesting of water plants being implemented in Laguna de Bay at around Taguig Lakeside Hall might be affected.	In case the impact on cultivation and harvesting of water plants is inevitable, proper compensation should be provided with the same process such as coordination with LGUs and other relevant organization as in the case of compensation of affected fishery mentioned above.
4. Impact on water use	Currently, no water permit for taking water from Laguna de Bay is granted near the candidate sites of intake facility of the project. Therefore, there is no impact under the current situation.	It is necessary to watch/check if a new water permit will be issued near the proposed location of intake facility of the project through coordination with NWRB.

Source: JICA Survey Team

(2) Impacts of Construction of Open Channel and Necessary Consideration

Major potential impacts of open channel construction will be the following three:

- Land acquisition of project site for the construction of open channel;
- Involuntary resettlement due to land acquisition and impacts on livelihood of project-affected persons (PAPs); and
- Impacts of land acquisition on existing facilities and structures.

The details of these potential impacts and necessary considerations are shown in Table 7.9.17 and summarized as follows:

In the case of Candidate Site No. 1, necessary land acquisition area for open channel is estimated at approximately 10.3 ha. If the area for drainage facility (approximately 0.1 ha) is added, the total area of land acquisition will be approximately 10.4 ha. The number of affected buildings and PAPs is estimated at approximately 280 and 860, respectively. Compensation costs for land acquisition and replacement cost for affected buildings are estimated to be 979 million pesos, and 218 million pesos, respectively, amounting to 1,197 million pesos in total (refer to Appendix 2-3 for the details). The number of ISFs included in the PAPs is not clear because the details of the settlement of ISFs were not yet surveyed.

In the case of Candidate Site No. 2, the land acquisition area necessary for open channel is estimated at approximately 5.4 ha. If the area for drainage facility (approximately 0.1 ha) is added, the total area of land acquisition will be approximately 5.5 ha. The number of affected buildings and PAPs is estimated at approximately 290 and 1,190, respectively. Compensation costs for land acquisition and replacement cost for affected buildings are estimated to be 939 million and 225 million pesos, respectively, amounting to 1,165 million pesos in total (refer to Appendix 2-3 for the details). The details of ISFs are not clear as in the case of Candidate Site No. 1.

Table 7.9.17 Potential Impacts of Construction of Open Channel and Necessary Considerations

Potential Impact	Description of Impact	Consideration
1. Land Acquisition	Candidate Site No. 1 (Table 7.9.18): Necessary area of land acquisition will be approximately 10.3 ha. (Total area will be approximately 10.4 ha when including that of drainage facility). The necessary area includes not only that for open channel (width: approx. 71m, length: approx. 1.2km), but also additional area (6m wide marginal area along both sides of the open channel) necessary for access roads, for construction works and marginal area. As for the construction of underground spillway, no land acquisition will be required in accordance with RA No. 10752 because the depth of the spillway is planned deeper than 50m below the ground surface. Candidate Site No. 2 (Table 7.9.18): Necessary area of land acquisition will be approximately 5.4 ha (Total area will be approximately 5.5 ha when including that of drainage facility). The difference with Candidate Site No. 1 is attributed to the length of open channel. As in Candidate Site No. 1, no land acquisition is required for the underground spillway.	Consideration should be given in planning the alignment of open channel to minimize the area of land acquisition. Land acquisition and compensation should be done in accordance with relevant laws and regulations, and in particular, compensation cost should be negotiated with the landowner pursuant to RA No. 10752 as follows: Current market value of the land; Replacement cost of structures and improvements therein; and Current market value of crops and trees therein.
2. Involuntary resettlement and impacts on livelihood of PAPs	Candidate Site No. 1 (Table 7.9.18): The number of affected buildings will be approximately 280, out of which residential buildings account for approximately 210 by deducting those located in the premises of the police facility (Camp Bagong Diwa) and the university (Polytechnic University of the Philippines), etc. The project affected persons (PAPs) therein will be required to resettle. Candidate site No. 2 (Table 7.9.18): The number of affected buildings will be approximately 290, almost all of which are residential buildings and the PAPs therein will also be required to resettle.	Compensation should be given in accordance with the provision of relevant laws and regulations as mentioned above. Resettlement and livelihood rehabilitation of PAPs should be performed pursuant to RA No. 7279 and DPWH Land Acquisition, Resettlement, Rehabilitation and Indigenous People Policy (LARRIP), etc., considering eligibility for resettlement, severity of impact, entitlement, and public consultation and participation, as well as ensuring necessary procedures.

Potential Impact	Description of Impact	Consideration
3. Impacts on existing facilities and structures	Candidate Site No. 1 (Table 7.9.18): Existing facilities such as police facility (Camp Bagong Diwa), university (Polytechnic University of the Philippines), government facility (DOST: Department of Science and Technology), etc., will be affected. As for the police facility and the university, the proposed site of open channel is located in the center of their premises, and therefore, their relocation is required partially or totally. Candidate Site No. 2 (Table 7.9.18): Other than residential buildings, there is the property of a private company (developer), which is currently an open space, and the railway of PNR (Philippine National Railways), which will be affected by the project.	Consideration should be given in planning the alignment of the spillway/location of open channel to minimize the impacts on existing facilities and structures. Compensation should be done in accordance with the provision of relevant law and regulations (RA No. 10752 and its IRR) as mentioned above.

Source: JICA Survey Team

Table 7.9.18 Estimation of Compensation Cost for Affected Lands and Buildings

Candidate Site	Facility	Barangay, Municipality /City	Magnitude of Impact			Compensation Cost		
			Area of Land Acquisition (ha)	Affected Buildings (No.)	Project-Affected Persons (No.)	Land (million Pesos)	Building (million Pesos)	Total (million Pesos)
Candidate Site No. 1	Open channel (including departing shaft)	Lower Bicutan, Taguig City	10.3	280	860	970	218	1,188
	Drainage Facility (Arrival shaft)	San Dionisio, Parañaque City	0.1	0	0	9	0	9
	Total	-	**10.4**	**280**	**860**	**979**	**218**	**1,197**
Candidate Site No. 2	Open channel (including departing shaft)	Sucat, Muntinlupa City	5.4	290	1,190	935	225	1,161
	Drainage Facility (Arrival shaft)	Pulang Lupa Uno, Las Piñas City	0.1	0	0	4	0	4
	Total	-	**5.5**	**290**	**1,190**	**939**	**225**	**1,165**

Source: JICA Survey Team

(3) Impacts of Construction of Underground Spillway and Necessary Considerations

Major potential impacts of construction of underground spillway will be the following two:

- Impacts of underground excavation and tunnelling works on existing underground structures; and
- Impacts of underground excavation and tunnelling works on groundwater.

The details of these potential impacts and necessary considerations are shown in Table 7.9.19. With regard to the land acquisition for underground spillway, the position of the spillway is proposed to be deeper than 50m from the ground surface and, therefore, land acquisition and compensation for surface owners or occupants will not be required in accordance with RA No. 10752 and its IRR as aforementioned. Thus, impact on social environment related to land acquisition will not be a matter of concern.

Table 7.9.19 Potential Impacts of Construction of Underground Spillway and Necessary Considerations

Potential Impact	Description of Impact	Consideration
1. Impact on underground structures	Underground structures such as foundation of high-rise buildings and urban infrastructures including water supply and sewerage system as well as the foundations of elevated road (SLEX) and the planned subway system and railway projects are likely to be affected by the construction of underground spillway. However, the possibility of impact will be minor because depth of the proposed underground spillway under this project is more than 50m from the ground surface.	Clarification of the depth of foundations along the candidate site of underground spillway should be made. Consideration by adjustment of the alignment of the spillway is required when necessary to avoid the impact.
2. Impact on groundwater	Candidate Route No. 1: There are 35 deep wells within the area until 500m distant from the proposed route of the spillway (Table 7.9.5). Water use of the deep wells might be affected by the spillway depending on geological and aquifer conditions. Candidate Route No. 3: There are 38 deep wells and two (2) surface water intake points within the area until 500m distant from the proposed route of the spillway (Table 7.9.5). Water use of the deep wells and surface water intake points might be affected by the spillway depending on geological and aquifer conditions.	Clarification of hydrogeological conditions around the deep wells and surface water intake points should be made through geological and hydrological surveys. In addition, an investigation of actual conditions of water use at the deep wells should be carried out, and based on which mitigation measures should be figured out to avoid/minimize the impacts on water use.

Source: JICA Survey Team

(4) Impacts of Construction of Drainage Facility and Necessary Considerations

Major potential impacts of construction of drainage facility will be the following three:

- Land acquisition of project site for the construction of drainage facility;
- Involuntary resettlement due to land acquisition and impacts on livelihood of project affected persons (PAPs); and
- Impacts on the residents and existing facilities in the downstream area.

The details of these potential impacts and necessary considerations are shown in Table 7.9.20.

Table 7.9.20 Potential Impacts of Construction of Drainage Facility and Necessary Considerations

Potential Impact	Description of Impact	Consideration
1. Land acquisition for construction of drainage facility	Candidate sites for the construction of drainage facility are located on the left bank side of the South Parañaque River (Candidate Site No. 1), on the right bank side of the San Dionisio River (Candidate Site No. 2), and on the right bank side of the Zapote River (Candidate Site No. 2) as shown in Figure 7.9.3. Necessary area of land acquisition for the construction of drainage facility is estimated at approximately 1,000 m^2. Compensation costs of land acquisition and buildings are shown in Table 7.9.18.	Consideration to minimize the area of land acquisition should be given in the selection of location of the drainage facility and in layout planning of the facility. Land acquisition and compensation should be done in accordance with relevant laws and regulations, and in particular, compensation cost should be negotiated with the landowner pursuant to RA No. 10752 as follows: • Current market value of the land; • Replacement cost of structures and improvements therein; and • Current market value of crops and trees therein.
2. Involuntary resettlement and impacts on livelihood of PAPs	There is no formal and/or informal settler family dwelling in any of the candidate sites, and therefore, resettlement of residents is not necessary.	Resettlement and livelihood rehabilitation of PAPs, when necessary, should be performed in accordance with RA No. 7279 and the DPWH Land Acquisition, Resettlement, Rehabilitation and Indigenous People Policy (LARRIP), etc., considering eligibility for resettlement, severity of impact, entitlement, and public consultation and participation, as well as ensuring necessary procedures.
3. Impacts on residents and existing facilities in the downstream area	Candidate Sites No. 1 and 2: There might be a necessity of river improvement such as widening, dredging, etc., of the downstream river section of the South Parañaque River (Candidate Site No. 1) or San Dionisio River (Candidate Site No. 2) to meet the necessary drainage capacity of the river (Section 7.2). In such a case, land acquisition and resettlement of riparian people might be required. Candidate Site No. 3: It is estimated that river improvement work will not be needed for the Zapote River because its width is rather wide in terms of necessary drainage capacity of the river (Section 7.2). However, ISFs on the opposite side of the river (Municipality of Bacoor, Cavite) might be affected psychologically (such as generation of a sense of victimization as PAPs due to the project).	In case of the necessity of river improvement for Candidate Site No. 1 and No. 2, consideration for minimizing the area of land acquisition and resettlement should be given. Land acquisition and compensation, as well as resettlement of PAPs, should be performed in accordance with relevant laws and regulations (as mentioned in Impact Case No. 1 above). In case of Candidate Site No. 3, it is necessary to encourage the Municipality of Bacoor, Cavite, to facilitate relocation of the ISFs on the left bank side of the Zapote River.

Source: JICA Survey Team

(5) Impacts of Disposal of Excavated Materials and Necessary Consideration

Major potential impacts of disposal of excavated materials from tunnelling works will be the following two:

- Impacts of development of disposal site of excavated materials, and
- Impacts of transportation of excavated materials on road traffic.

The details of these potential impacts and necessary considerations are shown in Table 7.9.21.

Table 7.9.21 Potential Impacts of Disposal of Excavated Materials and Necessary Consideration

Potential Impact	Description of Impact	Consideration
1. Impacts of development of disposal site	Disposal site of the excavated materials has yet to be fixed at the moment. There will be various types of impacts in case the disposal site is to be developed in the lakeshore area of Laguna de Bay as the potential site as described in Section 7.5, including those on fishery, water use, water transportation, and impacts on aquatic ecosystem, etc.	Investigation on fishing ground, aquaculture (fish pen and fish cage,) fish sanctuaries and mooring facilities, etc., should be conducted focusing on those located along the lakeshore. Based on the investigation results, concrete mitigation measures should be formulated. In case of difficulty to avoid or minimize the adverse impact on these economic activities, it is necessary to compensate for the impact upon consultation/coordination with relevant fisher folks, organizations and LGUs.
2. Impacts of transportation of the excavated materials on road traffic	Refer to "(6), Impacts of Construction Works and Necessary Consideration."	Refer to "(6), Impacts of Construction Works and Necessary Consideration."

Source: JICA Survey Team

(6) Impacts of Construction Works and Necessary Considerations

Major potential impacts of construction works of project facilities (intake and drainage facility, open channel, underground spillway, etc.) will be the following three:

- Generation of public pollution (air pollution, noise pollution, generation of low-frequency sound, water pollution, etc.) due to construction works;
- Impacts of solid wastes to be generated by demolition of existing structures/facilities; and
- Impacts of transportation of construction equipment and materials, and excavated materials on road traffic.

The details of these potential impacts and necessary considerations are shown in Section 6.3. In this section, the important points to be carefully considered are described in Table 7.9.22.

Table 7.9.22 Potential Impacts of Construction Works and Necessary Considerations

Potential Impacts	Description of Impacts	Consideration
1. Generation of public pollution due to construction works	Air pollution (dust and emission gas) due to operation of construction equipment and vehicles.	Watering during dry period, thorough maintenance of construction equipment and vehicles, idling stop, consideration in driving and operation of vehicles and equipment, Information, Education and Communication (IEC) for the dissemination of information on the project and the implementation schedule of construction works, etc.
	Noise pollution, generation of vibration and low-frequency sound due to operation of construction equipment and vehicles	Thorough maintenance of construction equipment and vehicles, consideration in driving and operation of vehicles and equipment, introduction of low-noise and

Potential Impacts	Description of Impacts	Consideration
		low-vibration type equipment, adjustment of working time, IEC, etc.
	Discharge of earth materials in Laguna de Bay during rains, generation of turbid water and oil and their diffusion, generation of high alkali water, and impacts on fishing activities in the lake.	Installation of sedimentation pond, drainage channel, installation of diffusion prevention curtain/fence, IEC, etc.
	Ground movements, groundwater discharge and drawdown of groundwater level during excavation works for construction of open channel and tunnelling works.	Implementation of enough supporting work for prevention of ground movements, investigation of hydrogeological conditions and actual condition of water use at the deep wells, and formulation of mitigation measures based on the investigation results.
2. Impacts of solid wastes to be generated by demolition of existing structures/ facilities	Solid wastes to be generated by implementation of the project including debris of demolished structures/facilities for construction of intake facility and open channel, and solid wastes to be generated from construction yards, liquid wastes (effluent) to be generated from base camp, etc. The volume of these wastes is estimated to be enormous.	Preparation of waste management plan by the Construction Contractor(s) and thorough implementation of management items such as reduction of waste generation, segregation, reuse, recycling, etc. In this regard, solid wastes should be treated based on RA No. 9003, which is the basic policy. Reuse and recycling of the demolished structures/facilities should be facilitated in collaboration with LGUs utilizing MRFs through thorough segregation. Harmful wastes should be properly treated based on RA No. 6969. In addition, disposal site should be developed through coordination with relevant GA (including LLDA) and LGUs.
3. Impacts of transportation of construction equipment and materials, and excavated materials	Generation of traffic for transportation of segments for shield tunnelling of the underground spillway (in case the spillway is conducted in shield tunnelling method.): 37 times of transportation by 27-ton trailer/day	Investigation of existing traffic conditions around the planned transportation routes, and formulation of traffic management plan by the Construction Contractor(s), including such management measures as consideration of transportation route and time, prevention of traffic accident in collaboration with police authorities, appointment of traffic control person(s), public relations/dissemination campaign about the project and traffic control.
	Generation volume (volume of earth to be transported) of excavated materials from tunnelling works will be estimated depending on the tunnelling methods and routes as follows (Section 7.5 for details): Shield Tunnelling Method, Route 1 : approx. 1,919,000m^2 Shield Tunnelling Method, Route 3 : approx. 2,032,000m^2 NATM, Route 1 : approx. 1,863,000m^2 NATM, Route 3 : approx. 1,977,000m^2 Transportation of the excavated materials will be done by dump truck. The number of transportation times is estimated based on the condition that transportation by dump truck is limited to the period from 10 pm to 4 am as follows: In case of shield tunnelling method: 657 times/day (= 28 times/hour); In case of NATM: 151 times/day (= 26 times/hour).	Ditto

Source: JICA Survey Team

7.9.5 Project Categorization based on JICA Guidelines and Preliminary Scoping

(1) Project Categorization based on JICA Guidelines for Environmental and Social Considerations

Analyses and discussions so far made reveal that the project would cause various types of environmental and social impacts. It is anticipated that the construction of project facilities on the ground surface including intake facility, open channel, drainage facility, etc., will require land acquisition and resettlement amounting to 300 households at the maximum, although construction of the underground spillway proposed at the depth of more than 50m from the ground surface will not require any land acquisition and compensation in accordance with the legislation of the Philippines (RA No. 10752).

Generation of solid wastes is estimated to be enormous, consisting of debris of demolished structures and facilities for the construction of project facilities. The volume of excavated materials from tunnelling works for underground spillway is anticipated at 2 million cubic meters at the maximum. Thus, it is indispensable to pay attention to these potential impacts to conduct necessary mitigation measures though the formulation of an Environmental Management Plan (EMP).

Accordingly, it is proposed that the project is classified as Category A in accordance with the JICA Guidelines for Environmental and Social Considerations.

(2) Preliminary Scoping and Necessary Study and Analysis in Feasibility Study Stage

Based on the results of study and discussion in Section 6.1 to 6.3 and Subsection 7.9.1 to 7.9.4, preliminary scoping and the necessary study and analysis in the Feasibility Study stage are as summarized in Table 7.9.23.

Table 7.9.23 Preliminary Scoping and Necessary Study and Analysis in the Feasibility Study Stage

Environmental Elements		Evaluation		Description of Evaluation	Study and Analysis in F/S Stage
		Planning/ Construction	Operation		
Pollution	Air pollution	B-	D	Air pollution due to emission gas and dust generation caused by construction equipment and vehicles during earth work, etc., is anticipated.	Survey for primary data on baseline condition of ambient air quality in the project area, and impact prediction of emission gas by the implementation of the project
	Water pollution	B-	D	If sediment in the construction sites of inlet or outlet contains toxic substances (e.g. heavy metals), they might stir up sediment during the construction and contaminate the water. The discharge through Parañaque Spillway is not likely to affect the water quality of Manila Bay.	Collect and confirm the latest water quality survey result of Manila Bay and Laguna de Bay. Survey the sediment of the inlet site in Laguna de Bay and the outlet site on the Zapote River or Parañaque River to check whether it contains toxic substances.
	Wastes	A-	D	Solid wastes will be generated by implementation of the project including debris of demolished structures/facilities. In addition, generation of excavated materials due to tunnelling works is anticipated with the volume of 200 million m^3 at the maximum.	Prediction of the volume of construction wastes including excavated materials, as well as preparation of waste management policy including collection, recycling, treatment and disposal of the wastes.

Environmental Elements		Evaluation: Planning/ Construction	Evaluation: Operation	Description of Evaluation	Study and Analysis in F/S Stage
	Soil contamination	C-	C-	There will be a possibility of soil contamination in case the excavated materials are contaminated with hazardous substances (heavy metals) with concentration exceeding the Philippine criteria (DAO No. 2013-22).	Laboratory analysis on chemical characteristics of earth (excavated materials) by TCLP and elutriate tests to identify soil contamination and its degree.
	Noise and vibration	B-	D	There will be generation of noise and vibration due to construction works on the ground such as construction of intake facility, open channel, drainage facility and vertical shaft, and those due to transportation by vehicles. Low frequency sound due to tunnelling work (shield tunnelling) also may be anticipated.	Baseline survey for primary data on ambient noise and vibration around the construction work sites on the ground, prediction of the degree of noise and vibration, low frequency sound, etc.
	Ground movement	C-	D	There will be a possibility of ground movements due to tunnelling work, and the possibility of affecting the existing underground structures such as foundations.	Ground survey by means of borehole tests and geotechnical tests, inventory of underground structures, as well as analysis on the possibility of ground movements.
	Offensive odor	B-	C-	There will be a possibility to generate offensive odour during the construction work for intake and drainage facilities, especially due to dredging works in Laguna de Bay and rivers/creeks. In the operation stage, offensive odour may be emitted from drainage facility during draining floodwater of Laguna de Bay.	Examination of the possibility of offensive odor through site survey on baseline condition and analysis of similar cases of spillway operation.
Natural Environment	Topography and geology	B-	B-	There will be topographical and geological alteration due to construction work for open channel, tunnelling work, etc.	Ground survey by means of borehole tests and geotechnical tests, and description of the degree of topographical and geological alteration.
Natural Environment	Groundwater	C-	C-	There will be a possibility of impacts on groundwater level and flow.	Survey on groundwater level by means of borehole tests and secondary data collection, inventory of deep wells and survey on groundwater use.
Natural Environment	Water regime	D	A+	Flood risk will be alleviated around Laguna de Bay in operation stage in consequence of the implementation of the project.	Change in water level of Laguna de Bay and degree of the positive effect by the implementation of the project.
Natural Environment	Terrestrial flora and fauna	C-	D	There will be a possibility of impacts of clearing of vegetation and disturbance of habitats of wildlife on terrestrial flora and fauna and protected species, if any.	Inventory of flora and fauna in the area of project sites, especially in case the project site covered by vegetation is modified.
Natural Environment	Aquatic biota	C-	D	In case the excavated materials from tunnelling works are to be disposed in Laguna de Bay, there will be an impact on aquatic biota in the lake.	Inventory of aquatic biota in Laguna de Bay, as well as coordination with relevant organizations (LLDA, etc.) on identification/development of disposal site of the excavated materials.
Natural Environment	Protected area (LPPCHEA)	D	C	LPPCHEA locates near the outfalls of the Zapote River and Parañaque River. The drainage through spillway increases the river discharge, which may pose negative impact on the area such as scouring. On the other hand, the drainage might improve the water quality around LPPCHEA (e.g. increase of DO).	Carry out water quality simulation to evaluate the impact of drainage through Parañaque Spillway. Based on the result above, study the impact on LPPCHEA.
Social Environment	Land acquisition/ involuntary resettlement	A-	C-	Land acquisition for the project sites will be required. Involuntary resettlement will also be required since there are residential areas including ISFs in the project sites. Existing structures and facilities will be	Confirmation of necessary land acquisition based on the facility plan of the project, inventory of ISFs, preparation of resettlement action plan (RAP), including socio-economic survey for PAPs, survey

Environmental Elements		Evaluation		Description of Evaluation	Study and Analysis in F/S Stage
		Planning/ Construction	Operation		
				affected, too. The number of affected buildings will be approximately 280 to 290.	on replacement costs for affected buildings, improvements and structures, and survey on market prices of affected lands, trees and crops, etc.
	Land use	D	B-	Existing land use will be drastically modified in and around the project sites.	Confirmation of comprehensive land use plan (CLUP) of concerned LGUs and conformance of the development plan under the project within the CLUPs.
	Economic activity/ employment/ livelihood	B-	C-	There will be a possibility of adverse impact on fishery, water transportation, cultivation and harvesting of aquatic plants, etc., in Laguna de Bay. Livelihood of PAPs, including employment conditions will be affected due to resettlement caused by the project.	Baseline survey on fishing activities, mooring facilities, water transportation, water intake and cultivation of water plants, etc., near the project sites in Laguna de Bay; Socio-economic survey targeted for PAPs and preparation of RAP, including livelihood rehabilitation programs.
	Cultural heritage	C-	D	There are four (4) registered heritage sites near the candidate sites of the project (Figure 6.1.10). These might be affected depending on the location/determination of project site.	Baseline survey on existing heritage sites near the project sites.
	Water use	C-	C-	There are many cases of water use on which water permit is granted by NWRB around the proposed project sites (Figure 7.9.7). There will be a possibility of impact on the water use.	Survey on hydrogeological condition, groundwater use (deep well survey), and analysis on possibility of impact on water use based on the survey results, as well as formulation of mitigation measures.
	Traffic	B-	D	There will be impacts on road traffic caused by project-related transportation of construction materials and equipment, excavated materials, etc.	Investigation of existing traffic conditions around the planned transportation routes, prediction of traffic volume of project-related vehicles, and formulation of traffic management policy.
	Other Elements on Social Impacts	C-	C-	Sufficient data or information for anticipation of social impacts has yet to be gathered.	Baseline survey, impact prediction regarding social elements based on project plan.

A+/-: Significant positive/negative impact is expected.

B+/-: Positive/negative impact is expected to some extent.

C+/-: Possibility of impact and its magnitude are unknown. (Further examination is needed, and the impact could be clarified as the study progresses.)

D: No impact is expected.

Source: JICA Survey Team

CHAPTER 8. CONCLUSION AND RECOMMENDATION

8.1 Conclusion

Since August 2017, "Data Collection Survey on Parañaque Spillway in Metro Manila" has been implemented by JICA as Japanese ODA. The comprehensive flood management plan of Laguna de Bay Lakeshore Area and Preliminary Feasibility Study (Pre-F/S) of Parañaque Spillway, which was approved as a priority project by the 4th Steering Committee Meeting held on 23 January 2018, has been executed. In this survey, there were three (3) routes for the Parañaque Spillway and two (2) types of tunnel construction methods for the underground spillway (Shield Tunneling Method and NATM).

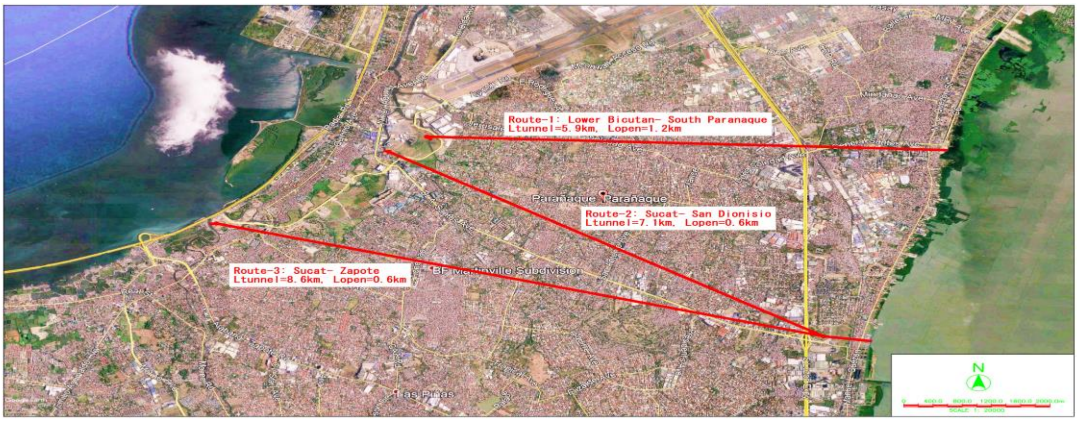


Figure 8.1.1 Alignment Alternatives of Parañaque Spillway

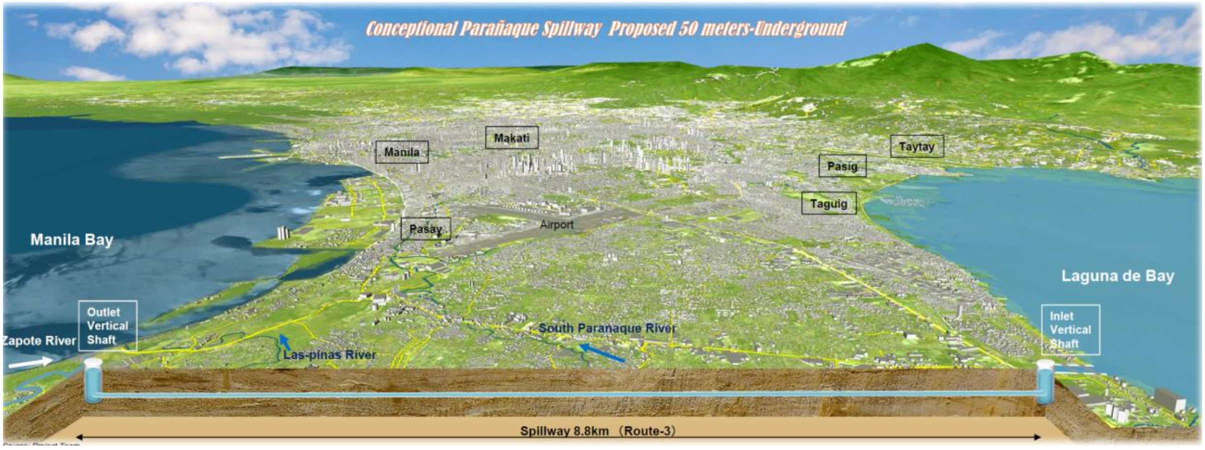


Figure 8.1.2 3-Dimensional Image of Parañaque Spillway

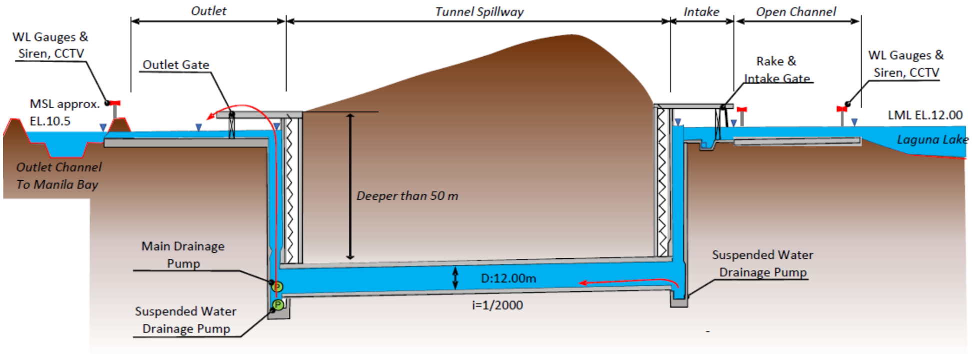


Figure 8.1.3 Schematic View of Parañaque Spillway

The results of the Pre-F/S show that EIRR is more than 10% for Parañaque Spillway alone, and EIRR is more than 20% if combined with flood mitigation project of Pasig-Marikina River Basin. The results suggest that Parañaque Spillway project is feasible.

8.2 Recommedation

However, this project was conducted in a short period of time (Approximately 9 months: August 2017 to April 2018) in which formulated comprehensive flood management plan and implemented Pre-F/S of Parañaque Spillway in limitation of the time. Therefore, Parañaque Spillway project was recommended to be feasible in Pre-F/S, but F/S is urgently needed based on the additional investigation shown in below.

＜Contents to be included in the F/S ＞

1. **Topographic Survey**

   In the Pre-F/S, existing terrain data (IFSAR data; 5m grid elevation data, NAMRIA) was utilized and the longitudinal gradient of spillway and designed intake facility (vertical shaft, water intake and drainage facilities) were examined. Since there is an error in the grid elevation data and actual elevation, it is necessary to carry out a detailed topographic survey and review the consideration in the F/S.

2. **Sounding Survey (Laguna de Bay)**

   In the Pre-F/S, the condition of bottom of Laguna de Bay was studied by using existing data of NAMRIA and examined the water intake facility. However, there is an inaccurate data in the actual bottom elevation/situation and existing data, so the necessary dredging quantity of Laguna de Bay for the placement of water intake facilities cannot be accurately estimated. Therefore, it is necessary to review the design of the open channel section of water intake facility by conducting sounding survey and accurately grasping the current lake bottom situation.

3. **Longitudinal and Cross-sectional River Survey and Evaluate effective to Downstream River**

   In the Pre-F/S, the effects of downstream river due to drainage of Parañaque Spillway were evaluated. The design scale was set for each river in the Pre-F/S because the flood control plan has not been formulated in Las-Pinas and Parañaque area. Evaluated downstream river water level raising due to drainage of Parañaque Spillway and river improvement plan based on embankment.

   Rivers in Las-Pinas and Parañaque district are connected by channels and they present a complex river network. The effect of drainage of Parañaque Spillway is not only for the downstream river, but also for other rivers which are connected by channels. In the F/S, additional river survey is necessary where there is no survey data in order to improve analysis model conducted by Pre-F/S then it is necessary to evaluate the effect of downstream river.

   In addition, it was found that the flooding in Las-Pinas and Parañaque district was caused by overflow from the river. It is also necessary to plan and formulate flood control plan at the same time as the F/S study.

4. Borehole Drilling Survey

In the Pre-F/S, six (6) boring surveys were conducted to grasp the approximate geological composition and groundwater level. The excavation depth of boring was set to 70 m, and the geological structure in deep underground was grasped, and Shield Tunneling Method and NATM were proposed as construction methods. However, the results of the drilling survey is not sufficient geological information for the 10km extension of structure, so it is necessary to conduct additional investigations. In particular, the effect of the Valley Fault System, located around intake facility of Parañaque Spillway, on the design and construction of the drainage channel has not been sufficiently grasped. At least 20 additional borehole drilling survey should be conducted to examine the construction method and to examine the design of underground spillway structure.

5. Hydraulic Model Experiment

In the Pre-F/S, the drainage facilities were examined and designed based on the existing study (The Metropolitan Area Outer Underground Discharge Channel and so on) of Japan. However, in the F/S, hydraulic model experiments should be conducted, and detailed drainage facilities (drop shaft) should be examined and designed, and the hydraulic specifications of tunnels should be examined.

6. Diffusion analysis of discharge from Parañaque Spillway

In the Pre-F/S, three (3) locations have been proposed as drainage facility, but in selecting the location of drainage facilities, it is necessary to determine the local LGU's opinions and the environmental impact. In the F/S survey, water diffusion analysis should be conducted to examine the effect of Laguna de Bay water on Manila Bay and quantitatively assess the effect on LPPCHEA.

7. Environment Impact Assessment (EIA) and Preparation of the Resettlement Action Plan (RAP)

In the Pre-F/S, the Parañaque Spillway was planned to be deeper than 50 meters underground. Therefore, there is no need to compensate land owners on the ground. However, it is assumed that land acquisition, house evacuation, and environmental impact of construction of water intake facilities, open channels and drainage facilities on the ground will occur. In the F/S, environmental impact assessment (EIA) and resettlement action plan (RAP) should be prepared and these issues should be thoroughly studied.

As described above, based on the results of the pre-F/S, the contents to be included in the F/S as the next step for implementation of Paranaque Spillway Project has been shown.

The Laguna de Bay Lakeshore Area has been developed year by year and the flood disaster potential has increased. The comprehensive flood countermeasures are an important task for DPWH, which is a desire for local residents and local governments. JICA technical assistance is recommended for the implementation of the F/S.

JICA