REPUBLIC OF THE PHILIPPINES

DEPARTMENT OF
PUBLIC WORKS AND HIGHWAYS

THE DETAILED DESIGN STUDY FOR THE PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE IV)

FINAL REPORT (PRIOR RELEASE VERSION)

VOLUME-1A
MAIN REPORT

AUGUST 2020

JAPAN INTERNATIONAL COOPERATION AGENCY

CTI ENGINEERING INTERNATIONAL CO., LTD.
JAPAN WATER AGENCY
NIPPON KOEI CO., LTD.
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COMPOSITION OF FINAL REPORT

VOLUME-1A	:	**MAIN REPORT (CHAPTER 1 to 6)**
VOLUME-1B	:	**MAIN REPORT (CHAPTER 7 / 7.1 to 7.3)**
VOLUME-1C	:	**MAIN REPORT (CHAPTER 7 / 7.4 to 7.6)**
VOLUME-1D	:	**MAIN REPORT (CHAPTER 8 to 12)**
VOLUME-2	:	**APPENDIX**
VOLUME-3	:	**APPENDIX FOR GEOTECHNICAL INVESTIGATION**

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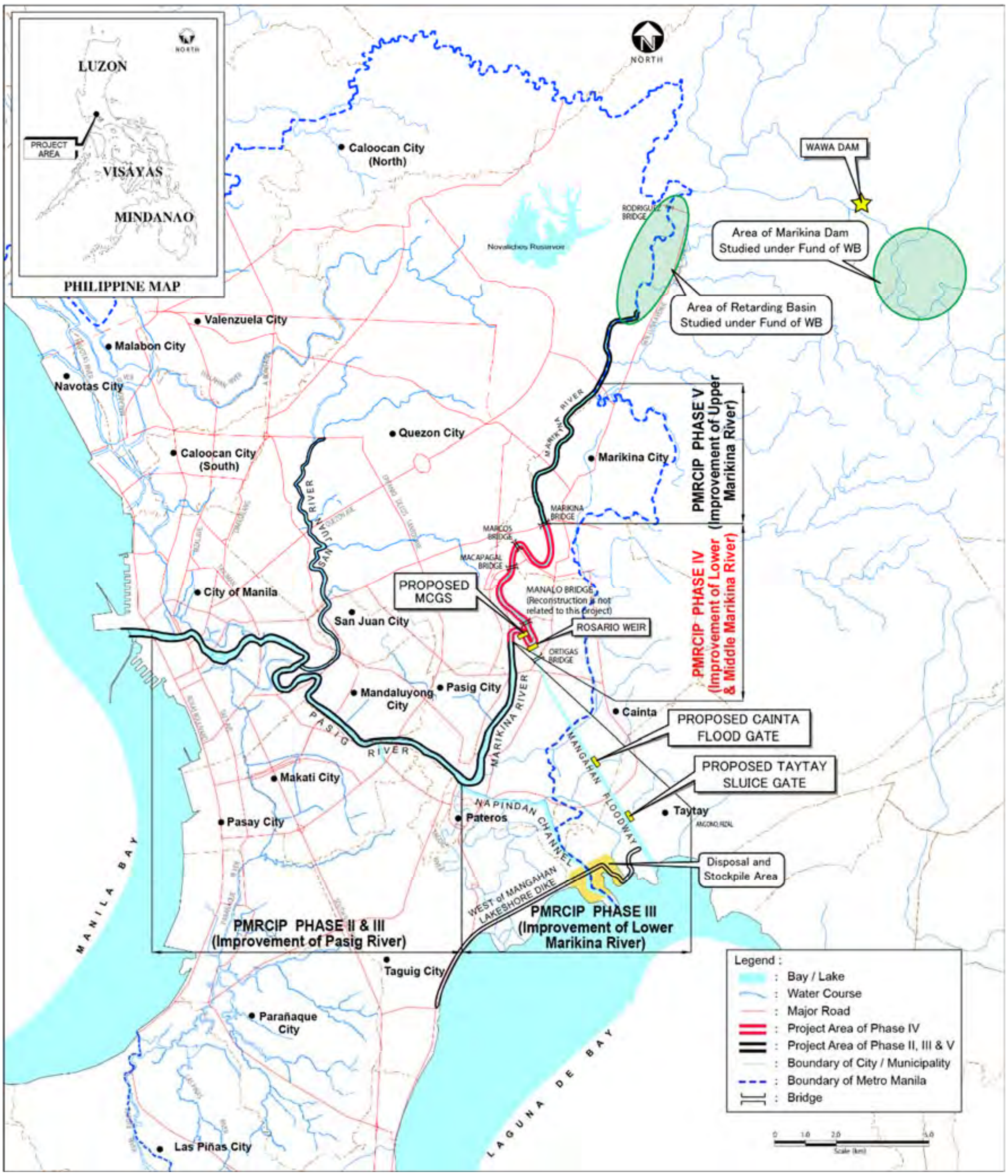

PROJECT LOCATION MAP

THE DETAILED DESIGN STUDY FOR THE PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE IV)

FINAL REPORT (PRIOR RELEASED VERSION) EXECUTIVE SUMMARY

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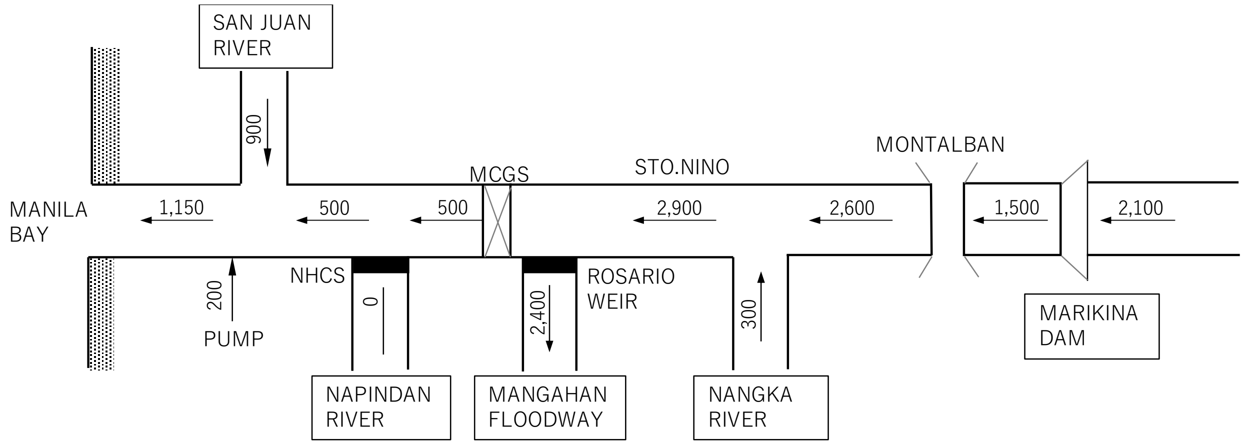
CHAPTER 1 OUTLINE OF THE PROJECT

1.1 Background of the Pasig-Marikina River Channel Improvement Project (PMRCIP)

The Pasig-Marikina River, with a total length of 52.2 km (Manila Bay to Wawa Dam) and a total catchment area of 635 km^2. Metro Manila (also known as Metropolitan Manila or the National Capital Region), through which the Pasig-Marikina River passes, is composed of 16 cities and one municipality. With the population of over 12 million people in 2015, it is the socio-economic and political center of the Philippines.

A flood control plan of the Pasig-Marikina River which included the Metro Manila area was initially formulated in 1952 under the River Control Section of the then Bureau of Public Works, Department of Public Works, Highways and Communications [presently, DPWH-UPMO-FCMC]. In accordance with the plan, several flood control studies and projects, such as construction of the Napindan Hydraulic Control Structure (NHCS) and pumping stations, installation of river walls as well as dredging, were conducted in the 1970's, and the Manggahan Floodway was constructed in 1988 to mitigate flood damage in the downstream areas by diverting floodwater into the Laguna Lake, worsened flooding condition and flood damage, however, was expected due to further urbanization of Metro Manila. Under such circumstances, the Government of the Philippines (GOP) requested the Government of Japan (GOJ), in 1986, to provide technical and financial assistance for flood prevention in Metro Manila.

In response, the GOJ decided to conduct, through JICA, the study from 1988 to 1990 (hereinafter referred to as "JICA1990MP") to formulate a master plan as shown in **Figure 1.1.1** and conduct a feasibility study on the urgent flood control projects selected which include the Pasig-Marikina River Channel Improvement Project (hereinafter "PMRCIP").



Source: JICA1990MP

Figure 1.1.1 Design Flood Discharge Distribution under the JICA1990MP (100-Year Return Period)

However, perennial flooding in Metro Manila continued and the floods in 1998, 2004, 2009, 2012 and 2014 have severely affected the socio-economic condition of Metro Manila. Therefore, the implementation of the PMRCIP has been recognized as essential for the mitigation of flood damage caused by overflow from the Pasig-Marikina river channel. Several follow-up studies and analyses have been undertaken by the GOP and, with the cooperation and assistance of JICA, flood control projects were made in parallel with those undertaken under other international financial institutions.

1.1.1 The Pasig-Marikina River Channel Improvement Project (PMRCIP)

The DPWH, with the support of JICA, embarked on the implementation of "The Pasig-Marikina River Channel Improvement Project" (the "Project") targeted approximately 30 km from the estuary to the Marikina Bridge in Sto. Niño district of Marikina City. The Project was divided into four (4) phases based on the results of the study undertaken with funds from the former Japan Bank for International Cooperation (JBIC: presently, JICA) under the Special Assistance for Project Formation (SAPROF) in 1998 as shown in **Table 1.1.1**.

Table 1.1.1 Phases of the PMRCIP formulated in 1998

Phase	Description of Contents	Project Completion
PMRCIP Phase I	Detailed Engineering Design for Phase II to Phase IV (From Delpan Bridge to Marikina Bridge: 29.7km)	March, 2002
PMRCIP Phase II	River Channel Improvement Works of the Pasig River (From Delpan Bridge to Merging Point with Napindan Channel: 16.4km)	June, 2013
PMRCIP Phase III	River Channel Improvement Works of the Lower Marikina River [From Merging Point with Napindan Channel to Diversion Point of Manggahan Floodway: 7.2km, including the Manggahan Control Gate Structure (MCGS)]	March, 2018
PMRCIP Phase IV	River Channel Improvement of the Middle Marikina River [From Diversion Point of Manggahan Floodway to Marikina Bridge (Sto. Niño): 6.1km]	2026 (present schedule)

Source: Study Team

1.2 PMRCIP Phase IV

JICA and the DPWH signed the LA for the PMRCIP Phase IV as a STEP Loan Project in January 2019.

Prior to LA, JICA and DPWH exchanged the Agreement on the detailed design in October 2018, and decided to carry out the detailed design with JICA funds. This study was conducted based on this agreement.

The contents of the Phase IV Project as a JICA Loan Project under Japanese ODA are as summarized in items (1) to (7) in **Table 1.2.1**.

Table 1.2.1 Outline of the PMRCIP IV Project

No.	Item	Description
(1)	Project Title	Pasig-Marikina River Channel Improvement Project (Phase IV), PMRCIP Phase IV or PMRCIP IV
(2)	Project Objective	To mitigate flood damage in Metro Manila caused by channel overflow of the Pasig-Marikina River, by implementing structural measures together with non-structural measures in consideration of flood management, thereby contributing to the sustainable urban economic development of Metro Manila.
(3)	Date of Signing of LA	January 21, 2019
(4)	Loan Amount	Not Exceeding JPY 37,905 Million
(5)	Contents of the Project	The Measures and Services include: Structural Measures • River Channel Improvement from Sta. 5+400 to Sta. 13+350 (Marikina Bridge at Sto. Niño): About 8 km • Construction of the MCGS: 1 structure • Construction of Floodgates along the Manggahan Floodway: 2 structures (Cainta Floodgate and Taytay Sluicegate) Consulting Services For Structural Measures: • Review of the Detailed Engineering Design • Bid Assistance / Construction Supervision • Support to Environmental Management and Monitoring • Support to Resettlement Actions and Monitoring, etc. For Non-structural Measures: • Formulation of Implementation Plan and Support to Implementation • Analyses to Formulate the Plan
(6)	Target Area	Metro Manila (Marikina River and Manggahan Floodway)
(7)	Implementing Agency	Department of Public Works and Highways (DPWH), GOP
(8)	Agencies/Organizations Concerned	• Metro Manila Development Authority (MMDA) • Local Government Units (LGUs) • The Public Information Agency (PIA) • Department of Environment and Natural Resources (DENR) • Office of Civil Defense (OCD) • Philippine Atmospheric, Geophysical and Astronomical Services Administration (PAGASA) • National Housing Authority: NHA • National Economic and Development Authority (NEDA) • Department of Finance (DOF) • Pasig River Rehabilitation Commission (PRRC) In this DED Study, DPWH as the implementing agency shall coordinate the functions and responsibilities of the above agencies/organizations on matters and issues related to the Project.

Source: Study Team

CHAPTER 2 OUTLINE OF THE DETAILED ENGINEERING DESIGN STUDY

2.1 Objectives of the PMRCIP-IV Detailed Engineering Design Study

As requested by the DPWH, the Detailed Engineering Design (DED) Study has been carried out aiming to prepare the detailed engineering design and the Draft Bidding Documents of PMRCIP-IV.

2.1.1 Basic Concepts and Flood Mitigation Plan of the PMRCIP (Chapter 3)

This Detailed Engineering Design Report presents the "across-the-board" review of the flood control/mitigation plans for the Pasig-Marikina River Basin described in **Chapter 3**. The final flood design distribution against a 100-year return period flood is as shown at the end of **Chapter 3**.

2.1.2 Basic Study and Analysis of River Channel Improvement Plan adopted in PMRCIP-IV (Chapter 4)

The river channel improvement plan for the PMRCIP-IV project was reconfirmed and finalized through the review and verification works under the two previous studies, namely, the 2002DD/PMRCIP-I and the Definitive Plan in 2015. **Chapter 4** presents the processes and results of the review and the finalization.

2.1.3 Survey and Investigation of Present Site Conditions (Chapter 5)

To ensure the necessary accuracy of the DED, topographic and geological surveys as well as the other necessary surveys were carried out through subcontracting to local survey firms. The working processes and results are explained in detail in **Chapter 5**.

2.1.4 Determination of Locations and Dimensions of Target River Structures (Basic Design) (Chapter 6)

The locations and basic dimensions of the MCGS, the Cainta Floodgate and the Taytay Sluicegate, as well as the dike and revetment were reviewed and set on the Basic Design Stage as explained in **Chapter 6**.

2.1.5 Detailed Engineering Design and Design Criteria (Chapter 7 and Chapter 11)

Based on the basic design in **Chapter 6**, the stability analyses, structural calculations and quantity calculations of each member and materials of river structures, and the imperative countermeasures so that all structures to be constructed will function smoothly during their expected lifetime or operating time.

2.1.6 Hydraulic Model Experiment (Chapter 8)

Based on the initial basic design and concepts of the MCGS, hydraulic model experiments were executed to finalize the dimensions of the MCGS through the confirmation of hydraulic condition.

2.1.7 Formulation of Basic Concept of Non-Structural Measures and the Operation and Maintenance Plans after the Completion of PMRCIP-IV (Chapter 9)

The Non-Structural Measures, namely, the Information Campaign and Publicity (ICP) and the Information Provision to enhance the community-based flood mitigation activities, were formulated and proposed. In addition, O&M plan/s for the MCGS, Cainta Floodgate and Taytay Sluicegate, as well as the other river structures such as dikes and revetments to be constructed in the PMRCIP-IV, were also prepared.

2.1.8 Updates and Reviews on Environmental Impact Statement (EIS), Environment Management Plan (EMP), Environment Monitoring Plan (EMoP) and Right-of-Way (ROW) / Resettlement Action Plan (RAP) (Chapter 10)

As for environmental and social considerations, the existing plans have been reviewed. Following two concerns relating with the backfill works for excavated and dredged soils at approximately 1.5 million m^3 were studied to obtain of ECC: (1) if soil materials include hazardous and contaminated materials; and (2) where disposal area/s is/are available and secured for huge amounts of excavated and dredged soils.

2.1.9 Review of Project Evaluation (Chapter 12)

Based on the study results of **Chapter 7**, Construction Plan and Cost Estimate, the project evaluation was confirmed in **Chapter 12** from the relationship between the flood conditions reviewed in **Chapter 3** and the construction costs estimated in this study.

2.2 Summary of Essential Results of the Basic Design and Detailed Engineering Studies to be Considered in the Future

2.2.1 Design Flood Discharge Distribution of the Pasig-Marikina River Basin

The design discharge of 2,900 m^3/s in PMRCIP-IV shall correspond to the 100-year design discharge after the completion of structural measures including the construction of the Marikina Dam and the retarding basins. However, the design discharge of 2,900 m^3/s is only 20 to 30-year return period probability under the existing conditions without the dam and basins. In view of the above, following premises and/or considerations should be taken in the future to establish flood control project at 100-year return period.

1) Flood Control Plan for San Juan River

   the proposed design discharge of the San Juan River is set at 800 m^3/s to conform with the design discharge of San Juan River as proposed at 780 m^3/s in the JICA2014Survey in consideration of actual site status along San Juan River channel although confirmed that the probable discharge of San Juan River at 100-year return period will exceed 1,000 m^3/s. In this connection, more than 200 m^3/s of the peak discharge of San Juan River shall be reduced by the structural measures in the basin.

2) Design Discharge in the Pasig River

   The flow capacity of the Lower Pasig River should be increased from 1,200 m^3/s to 1,400 m^3/s to protect Metro Manila from river floods of 100-year return period. In this case, the dike should be raised at 0.42 m at the confluence with the San Juan River, or dredging of the riverbed is needed from the river mouth of the Pasig River to the confluence (approx. 7 km long) in order to sustain the design flood level.

3) Plans for the Marikina Dam and the Marikina Retarding Basin

   As explained above, the design discharge of 2,900 m^3/s at Sto. Niño should correspond to the flood discharge of 100-year return period. In this connection, the DPWH should harmonize the PMRCIP-IV with the Marikina Dam and the Marikina Retarding Basins being studied by World Bank fund.

2.2.2 Structural Dimensions of the MCGS

The widths of MCGS (w1: 28.3m and w2: 11.7m) have been confirmed through the hydraulic model experiment. The foundation type of the MCGS is amended into Spread Type (without foundation pile) since geological survey clarified that exposed riverbed rock has sufficient strength as the foundation of MCGS.

2.2.3 Structural Dimensions of the Cainta Floodgate

In this Study, the construction point of the floodgate was shifted to the existing San Francisco Bridge to keep it in line with the alignment of the dike crown. As a result, the construction of the floodgate will need relocation of the affected houses and buildings and procure the lots along the Cainta Creek. The existing San Francisco Bridge crossing the Cainta Creek also should be replaced on the structure of the floodgate.

2.2.4 Structural Type of Taytay Floodgate

Taking into consideration conformity with the existing box-culvert, the structural type of the Taytay Floodgate should be the sluicegate.

2.2.5 Draft Bidding Documents

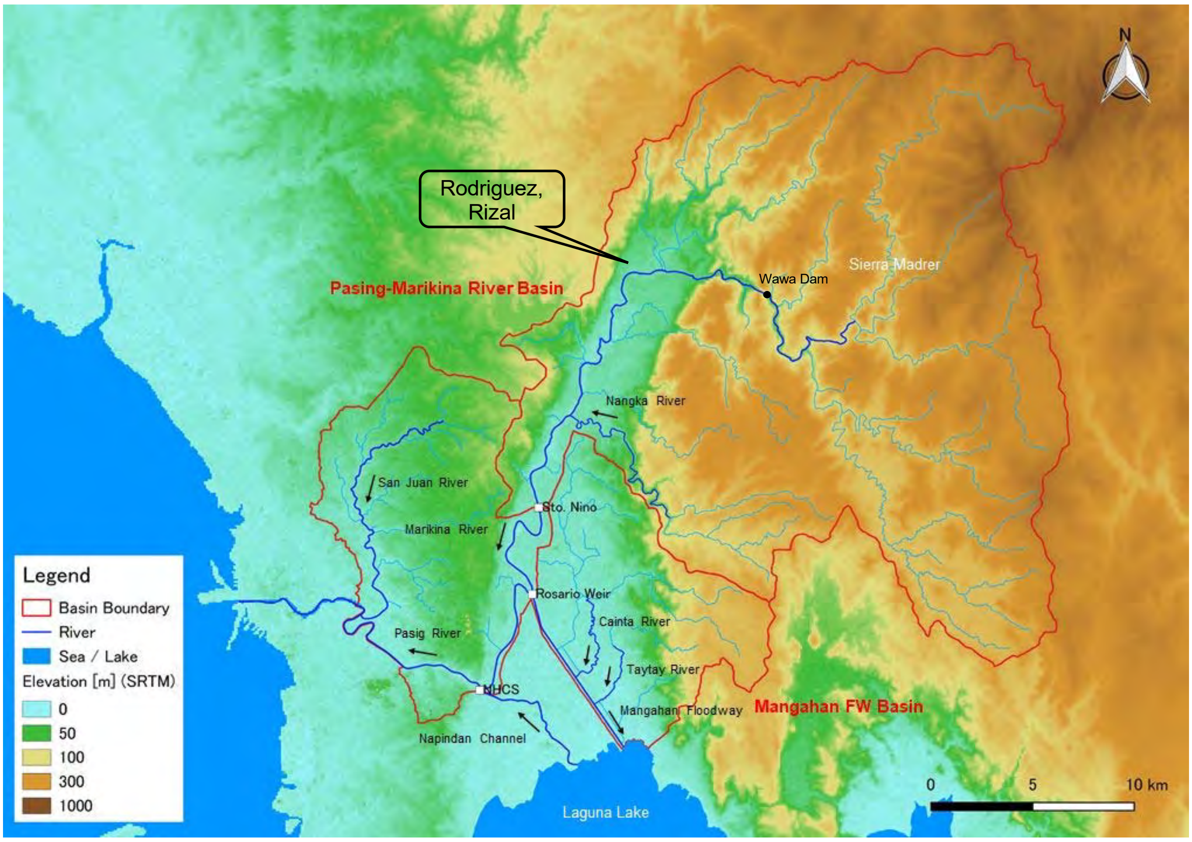
In parallel with the detailed engineering design and cost estimation of each construction package, the Draft Bidding Documents have been prepared in accordance with the "Standard Bidding Document under Japanese ODA Loans" issued in October 2019.

CHAPTER 3 FLOOD MANAGEMENT PLAN FOR PASIG-MARIKINA RIVER

3.1 Current Condition of Pasig-Marikina River Basin

3.1.1 Outline of the River Basin

The Pasig-Marikina River, with a total length of 52.2 km (Manila Bay to Wawa Dam) and a total catchment area of 635 km^2, originates from the southwestern slopes of the Sierra Madre Mountains of which top of elevation is about 1,400 meters (MSL). The river has two major tributaries, namely; the San Juan River which merges at 7.1 km from the river mouth, and the Napindan Channel which merges at 17.1 km from the river mouth, respectively. The Pasig-Marikina River is mainly divided into two (2) sections at 17.1 km from the river mouth. The downstream section is called the Pasig River (from the river mouth to the merging point of the Napindan Channel), and the upstream section is called the Marikina River (upper reach of the river from the merging point of the Napindan Channel). The Pasig-Marikina River also connects with the Laguna de Bay (Laguna Lake) via the Napindan Channel and the Manggahan Floodway. The floodway is manmade and it diverts floodwaters from the Marikina River at 23.8 km from the river mouth. The Location Map is in **Figure 3.1.1**.



Source: Study Team

Figure 3.1.1 Location Map, Pasig-Marikina River Basin

3.1.2 Flow Condition of Marikina River

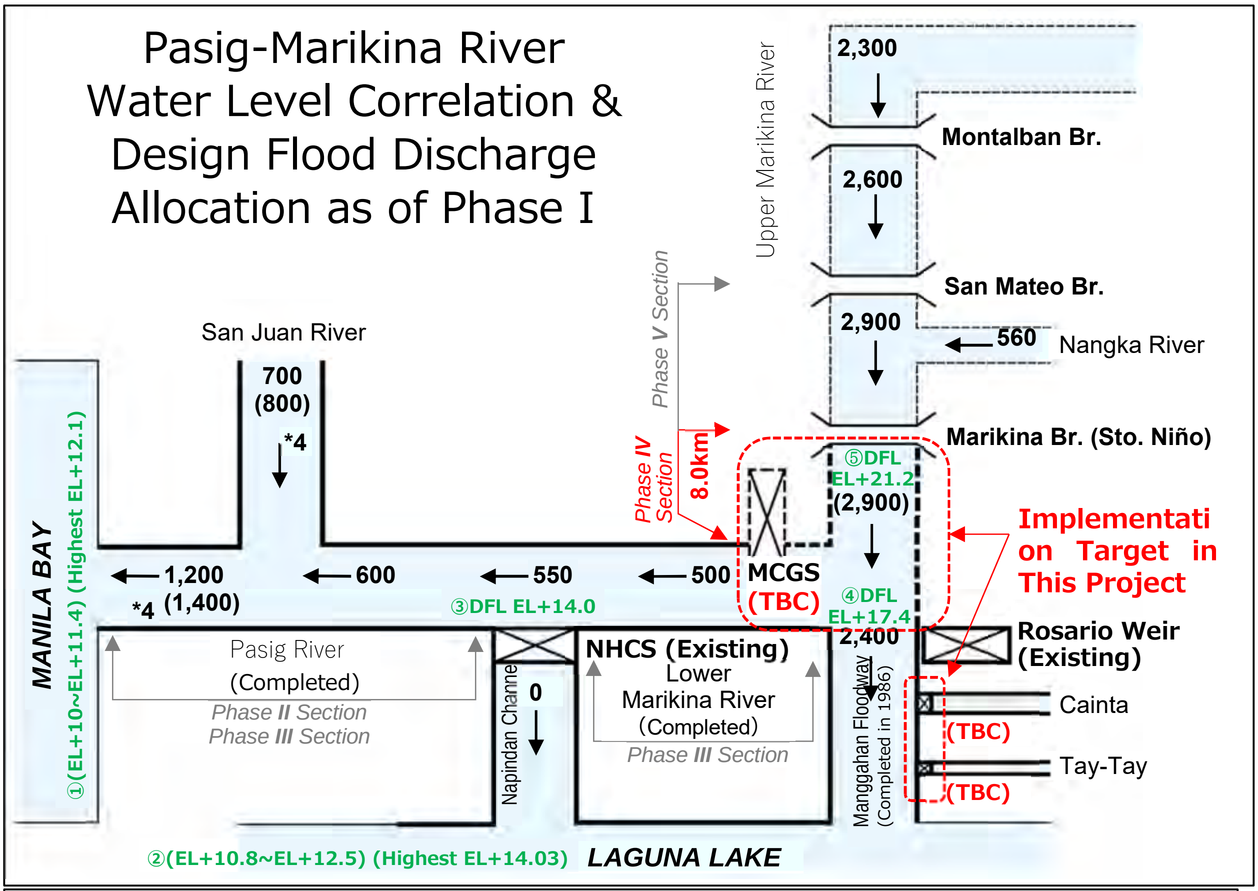
The water levels of average 95-day, 185-day, 275-day and 355-day water level at Sto. Niño in the recent 25 years (1994 to 2018) were 12.63 m, 12.03 m, 11.55 m and 11.27 m, respectively, and the highest water level observed at 5PM during Typhoon Ondoy on the 26th of September 2009 was 22.16 m. Average 95-day, 185-day, 275-day and 355-day discharges were 113.0 m^3/s, 53.0 m^3/s, 22.4 m^3/s and 11.4 m^3/s, respectively. The maximum discharge was 3,480 m^3/s at the highest water level observed in Typhoon Ondoy.

3.1.3 Information on Water Level in the Pasig-Marikina River Basin

The following standard value (elevation) has been used for river structures in the basin.

- Mean Lower Low Water Level (MLLWL) = EL+10.00 m (hereinafter, "DPWH Elevation")

Therefore, information on all the elevations such as the hydraulic analysis to be implemented and examined in this study and the drawings created are represented by this DPWH elevation. The water level and elevation information at each point of this DPWH elevation is as shown in **Figure 3.1.2** below.



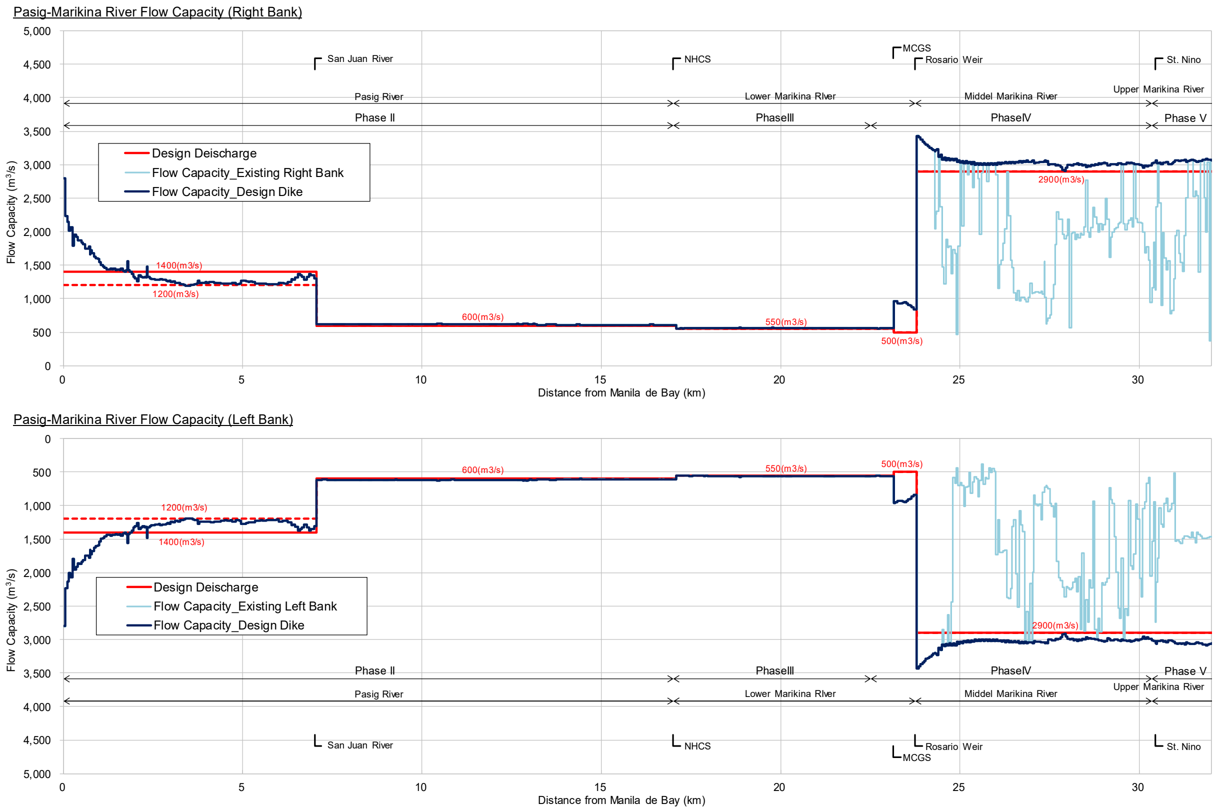
Note *1 : EL: DPWH Elevation (MLLWL＝EL+10.00)
Note *2 : ①: The number in ( ) is fluctuation of water level in Manila Bay
②: The number in ( ) is fluctuation of water level in Laguna Lake
③: DFL at NHCS in Marikina River
④: DFL at Rosario Weir in Marikina River
⑤: DFL at Sto. Nino (Marikina Bridge) in Marikina River
Note * 3 : The number inside river is design flood discharge allocation in each section.
Without parentheses ( ) means improved section/; With ( ) means section to be improved.
Note *4 : The design discharges of the Lower Pasig and San Juan rivers have been reviewed. (See Subsection 3.4.4)

Source: Study Team based on existing information

Figure 3.1.2 Water Level Correlation in Pasig-Marikina River

3.1.4 Current Flow Capacity of Pasig-Marikina River

Current flow capacities of Pasig-Marikina River are as illustrated in **Figure 3.1.3** based on "The Preparatory Study for PMRCIP, Phase III (JICA2011 Preparatory Study)" and the 2015IV&V-FS for this study.



Source: Study Team based on JICA 2011 Preparatory Study and 2015IV&V-FS

Figure 3.1.3 Current Flow Capacity of Pasig-Marikina River

3.2 Existing Flood Management Plan and Related Conceptual Plan

3.2.1 Existing Flood Management Plan

The list of studies on flood management plans is given in **Table 3.2.1**. The river structures and facilities have been planned and designed based on these flood management plans in this Study for Detailed Design.

Table 3.2.1 Past Studies on Flood Management Plan

Project Name	Completion Year	Implementing Agency	Acronym
Formulation of Flood Control Plan in Pasig-Marikina River Basin	1952	Gov't. of the Philippines	1952MP
FS Study and Detailed Design of Manggahan Floodway	1975	USAID	1975FS/DD
The Study on Flood Control and Drainage Project in Metro Manila	1990	JICA	JICA1990MP
Detailed Engineering Design of PMRCIP	2002	DPWH	2002DD
The Preparatory Study on PMRCIP Phase III	2011	JICA	JICA2011 Preparatory Study
Master Plan for Flood Management in Metro Manila and Surrounding Areas	2012	WB	WB2012MP
Data Collection Survey for Flood Management Plan in Metro Manila	2014	JICA	JICA2014 Study
Feasibility Study on PMRCIP Phase IV and V	2015	DPWH	2015IV&V
Feasibility Study and Preparation of Detailed Engineering Design of the Proposed Upper Marikina Dam	2018	WB	WB2018 UMD FS

Source: Study Team

3.2.2 Major Flood Management Projects and River Structures in Pasig-Marikina River Basin

In the Pasig-Marikina River Basin, the river structures and flood forecasting systems shown in **Table 3.2.2** are constructed and/or being installed for flood mitigation/control.

Table 3.2.2 River Structures / Facilities and Systems in the Basin for Flood Control / Mitigation

Purpose	River Structures / System	Completion
Measures for Flood Control against River Overflow Flood	Napindan Hydraulic Control Structure (NHCS)	1982
	Manggahan Floodway	1988
	Rosario Weir	1988
	Dike System along the Pasig Lower Marikina River	2018
Flood Forecasting and Warning System	The Effective Flood Control Operation System (EFCOS) Project (First Phase)	1995
	EFCOS (Rehabilitation by JICA Grant)	2002
	EFCOS (Recovery by JICA Grant for Damaged Facilities by Typhoon Ondoy)	2016
Measures for Flood Control against Inland Flood	12 Pumping Stations along the Pasig River	1970~2000
	5 Pumping Stations in West Manggahan Area	2007
	4 Pumping Stations in KAMANAVA Area 1 Tidal Gate in KAMANAVA Area	2007

Source: Study Team based on references

3.2.3 Flood Control Studies of which the Implementations are Expected in the Basin

(1) Study on Flood Mitigation Project in the East Manggahan Floodway Area (East Manggahan Study)

This study was conducted by the DPWH, aiming at the reduction of frequent inland flood damage in the East Manggahan district (area at the left bank side of the Manggahan Floodway). The main causes of inland flood damage are the water rise of Laguna Lake and the backflow from the Manggahan Floodway to the tributary rivers. In this study, the construction of Cainta and Taytay floodgates, which are included in the Phase IV project, was proposed as a priority project.

(2) New Drainage Project by WB (MM Flood Risk)

Based on the comprehensive flood risk management plan including the drainage proposed in the WB2012MP as listed in **Table 3.2.1** above, the World Bank (WB) begun to support projects to improve the drainage and environment in Metro Manila through co-financing with the AIIB. The project implementation agencies are the DPWH and the MMDA. The project consists of the following four components.

(3) Feasibility Study and Preparation of Detailed Engineering Design of the Proposed Upper Marikina Dam (WB2018UMD)

This WB2018UMD Study was to conduct the FS and DD of for the Upper Marikina Dam which is necessary to complete the whole PMRCIP. The WB2018UMD study was funded by a grant from the WB. The basic objective of the WB2018UMD study was to determine the preferred option for a flood management structure to reduce the water discharge from the Marikina River before it enters Metro Manila through a feasibility study leading to the preparation of detailed designs and tender documents.

3.3 Finalization of Flood Management Plan

The flood management plan in this study basically follows those of the previous 2015IV&V-FS and the JICA2014Study.

3.3.1 Basin Average Probable Rainfall

The basin average probable rainfall in this study follows that of the 2015IV&V-FS as shown in **Table 3.3.1**.

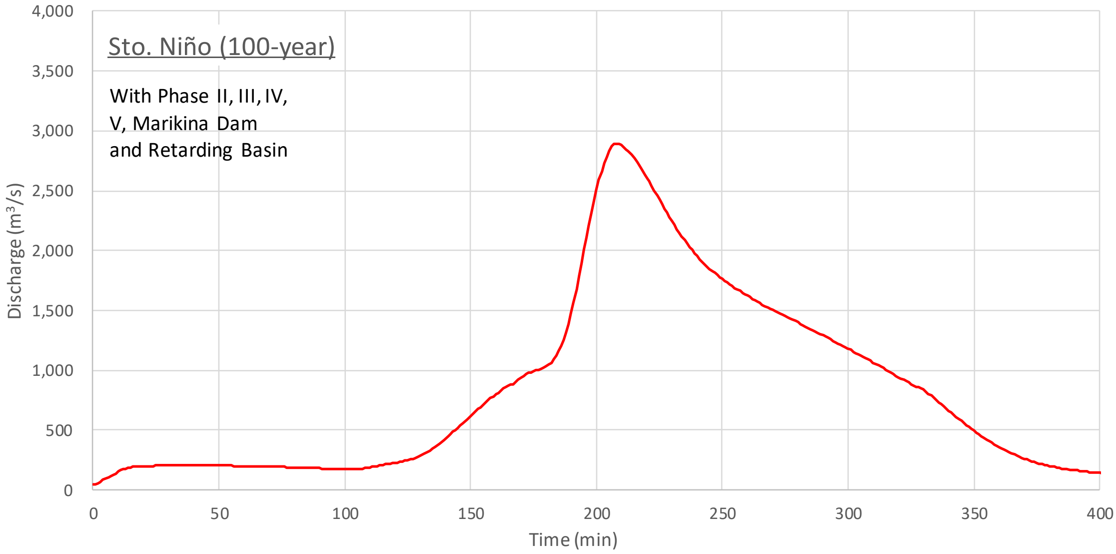
Table 3.3.1 Basin Average Probable Rainfall

Return Period	1-Day Rainfall (mm)
2	122.9
5	172.7
10	205.7
30	**255.5**
50	278.3
100	**309.0**
200	339.6

Source: 2015IV&V-FS

3.3.2 Flood Discharge at Sto. Niño

The design hydrograph at Sto. Niño (100-year return period, with Marikina Dam and retarding basin) is to be finalized under the Retarding Basin Study currently being conducted by the World Bank following the WB2018UMD. For this reason, the anticipated design hydrograph (See **Figure 3.3.1**) which has been estimated in this Study based on the 2018WB UMD FS report, is used to prepare the operation rules for flood control structures. This hydrograph was designed using Typhoon Ondoy type hyetograph, which has the biggest 1-hour rainfall, 1-day rainfall, and the largest peak discharge and rapid water level rise at Sto. Niño.

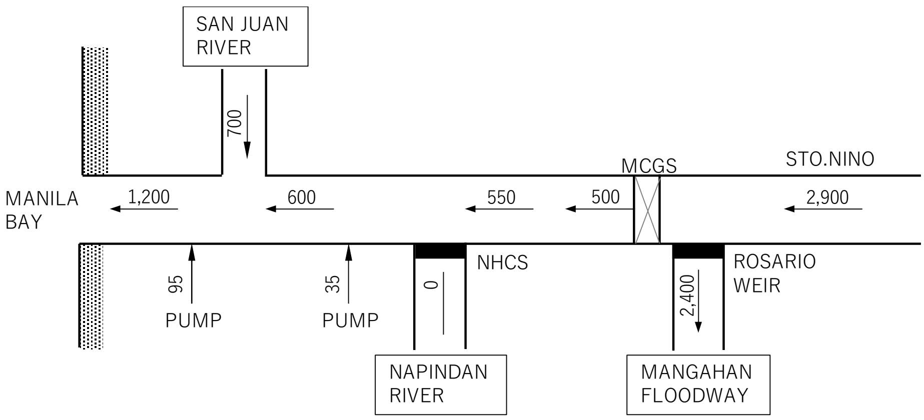


Source: Study Team based on 2018WB UMD FS

Figure 3.3.1 Anticipated Design Hydrograph at Sto. Niño (2,900 m^3/s)

3.3.3 Target Flood Discharge

The Phase II and Phase III sections have been improved based on the immediate target flood discharge (30-year probability) set in the 2002DD as shown in **Figure 3.3.2**.



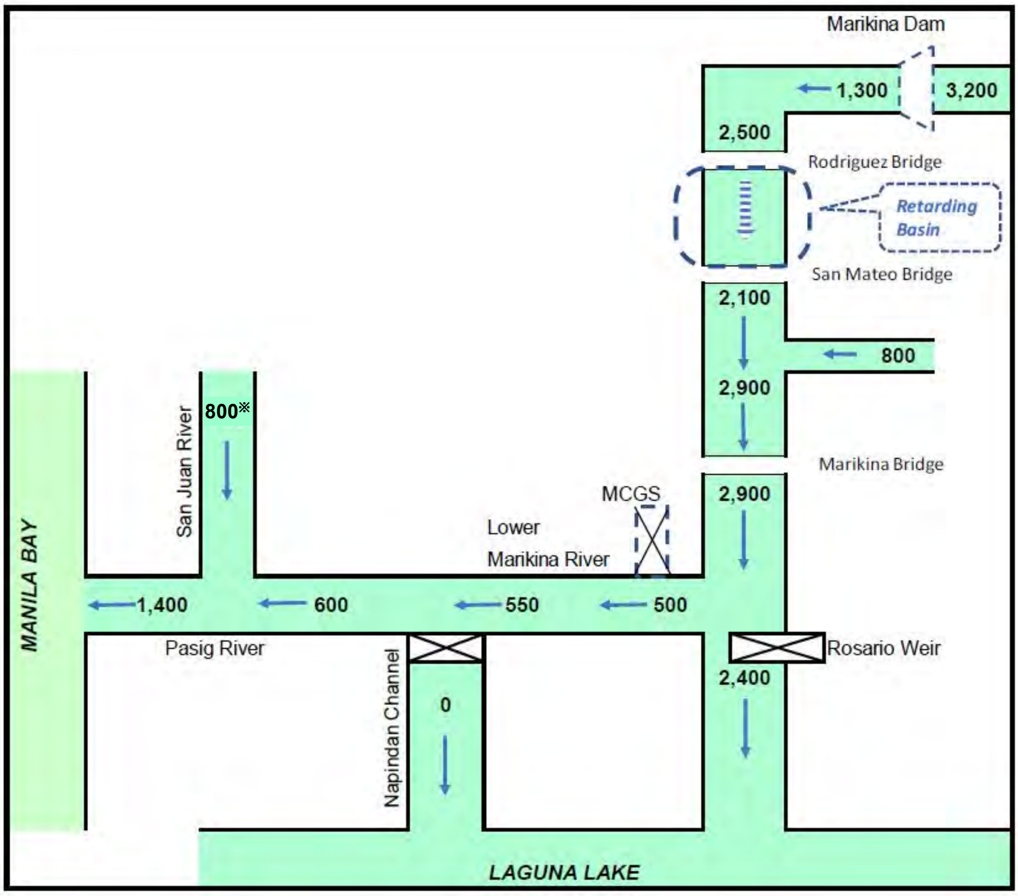
Source: 2002DD

Figure 3.3.2 Immediate Target Flood Discharge Allocation (30-Year Design Flood) (2002DD)

The Phase IV or this detailed design study have followed the plan in which the design discharge at Sto. Niño is of 2,900 m^3/s. In revised hydrological analysis in the recent studies such as the WB2012MP and the JICA2014Study, the estimated discharge probability of 2,900 m^3/s at Sto. Niño is slightly more than a 20-year and lower than a 30-year. Also, 2,900 m^3/s shall be set to be the design flood discharge (100-year) at Sto. Niño after the Marikina Dam and Retarding Basin construction,

3.3.4 Design Flood Discharge Allocation

The design flood discharge allocation of each section is shown in **Figure 3.3.3** as the final design flood discharge allocation.



*Design flood discharge on the assumption that the peak discharge is cut by about 200 m³/s by basin management and so on.
Source: Study Team based on 2015IV&V-FS

Figure 3.3.3 Draft Design Flood Discharge Allocation (100-Year Flood Discharge)

Without the Marikina Dam, about 30-year return period flood can be accommodated with the implementation of the Phase IV Project. No structural measure to resist larger scales of flood will be proposed in the Phase-IV Project since the flood condition will not worsen from the present condition even without the Marikina Dam.

3.3.5 Climate Change Adaptation

The design guideline for water engineering projects in the Philippines (DGCS Volume III) suggests the following allowances for climate change:

- Changes to Extreme Rainfall: Incorporate a 10% increase in rainfall intensity in the design.
- Sea Level Rise: Allow for a 0.3 m sea level rise in the design.

For the Phase IV section, which is the design target section of this study, climate change adaptation has been addressed in the design through the following:

- Design Flood Discharge: 10% increase in rainfall intensity is incorporated in the computation of design flood discharge, which is 2,900 m³/s at Sto. Niño in a 100-year return period. The design flood discharge is the river flow regulated by the Marikina Dam and the retarding basin. Practically, the increased amount of discharge caused by 10% increased design rainfall intensity shall be regulated by the Marikina Dam and the retarding basin.
- Sea Level Rise: Sea level rise does not affect the river water level in the design target section of Phase IV.

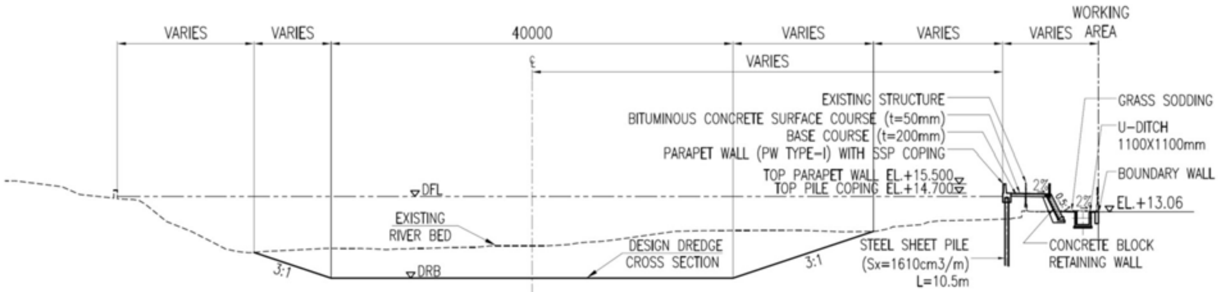
CHAPTER 4 PRECONDITIONS FOR RIVER CHANNEL DESIGN (BASIC DESIGN STAGE)

4.1 Preconditions (Verification of River Channel Planning)

4.1.1 Validation of Past Plans and Determination of Standard Cross Section of Targeted River Stretch

4.1.1.1 Planned Cross Section Downstream of MCGS

The design flood discharge in the downstream of the MCGS is set at 550 m^3/s, and river improvement works have been carried out up to Sta. 5+400 of the Marikina River in the PMRCIP Phase III Project. Excavation and/or dredging of the low water channel has been carried out to satisfy the planned riverbed width of 40 m [Slope: 3:1 (H:V)], while dikes (floodwalls) were constructed where the ground elevation behind the bank was lower than the DFL.



Source: JICA Phase III Detailed Design Report

Figure 4.1.1 Standard Cross Section of Phase III Downstream of the Marikina River Improvement Project

In the PMRCIP-IV Project, river cross sections downstream of the MCGS are set to coincide with the planned and implemented sections for the Phase III Project.

4.1.1.2 Standard Cross Section/s in the Upstream Stretch of the MCGS

(1) Standard Cross-Sections set in River Channel Plan in 2015IV&V

The standard cross sections of the upstream stretch of the MCGS have been set in the DPWH2015IV&V. These standard cross sections have also been designed appropriately for the design discharge of 2,900m^3/s.

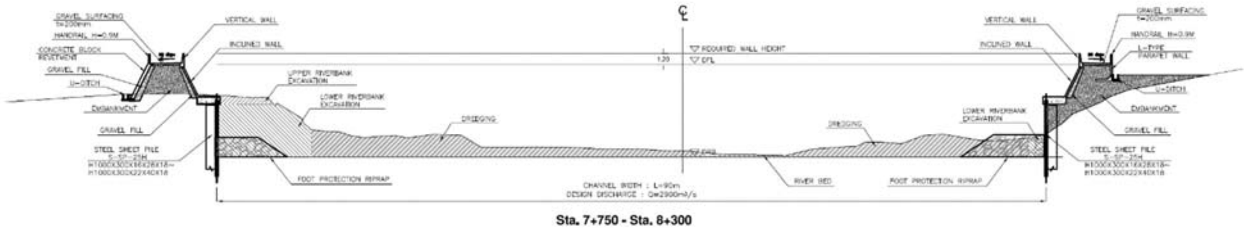
The design conditions in the DPWH2015IV&V are as given below.

Table 4.1.1 Design Conditions in the Definitive Plan (2015)

Items	Design Policy
Design Flow Rate	2,900 m^3/s
Freeboard	1.2 m
Basic Concept of River Alignment to be improved	Fit into the existing channel
Longitudinal Gradient of Design Riverbed	1/4 000 (Rosario Weir – Marikina Bridge)
Low Water Channel Width	Rosario Weir ~ 10+500: 90 m 10+500 ~ 11+000: Widened to 90 m on left side only 11+000 ~ 13+350 (Marikina Bridge): 80 m
Revetment and/or Slope Protection for high water channel	Rosario Weir ~ Sta. 10+500: Inclined Concrete Wall Sta. 10+500 ~ Sta. 12+500: Heightening of Existing Concrete Wall, Construction of new parapet wall Sta. 12+500 ~ Sta. 13+350: No flood protection facility on both banks as requested by the city and the residents. (However, widening of the low water channel will be conducted.)
Maintenance Road Width	3 m macadam pavement

Source: Definitive Plan for PMRCIP Phase IV

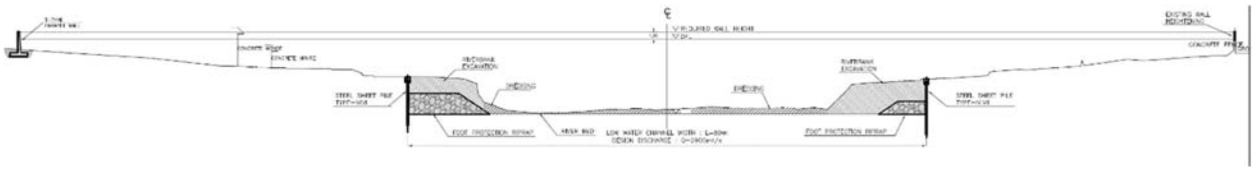
The standard sectional view in the 90m section and the standard sectional view in the 80m section are as shown below.



DFL: EL+17.4 m at Rosario Weir and EL+21.18 m at Sto. Niño

Source: Implementation Program (September 2018, DPWH)

Figure 4.1.2 Standard Section of Renovated 90m Low Channel Section



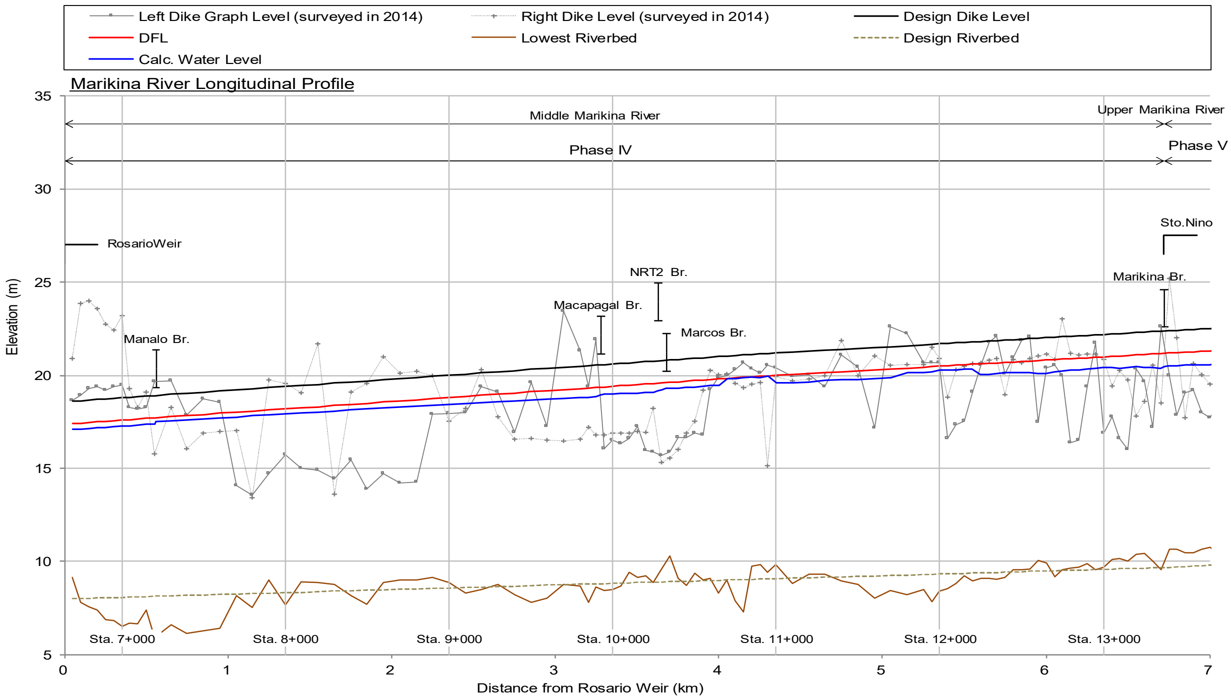
DFL: EL+17.4 m at Rosario Weir and EL+21.18 m at Sto. Niño

Source: Implementation Program (September 2018, DPWH)

Figure 4.1.3 Standard Section of Renovated 80m Low Channel Section

(2) Design Flood Level (DFL)

The DPWH2015IV&V set the Design Flood Level (DFL) in the river channel stretch of Phase IV and Phase V using HEC-RAS in conjunction with the examination of the standard cross sections. However, there is a difference in concept about water level rise to be considered in river channel planning between HEC-RAS and the design criteria of Japan. Therefore, the "difference" is herein verified in this Detailed Design Study. For example, the methods for calculating the water level rise due to bridge pier (⊿h02) and meandering (⊿h03) of the HEC-RAS adopted in the DPWH2015IV&V are different from those of the Japanese "Guideline for River Channel Plans." In this connection, the longitudinal water level calculated at n=0.025 based on the "Guideline for River Channel Plans" and taking into consideration bridge piers and bends of river alignment is as shown in **Figure 4.1.4**. As the result, it was confirmed that the calculated water levels at "bridge piers" and "bend" are within the DFL of the Definitive Plan in 2015 and, therefore, this Detailed Engineering Design has adopted the DFL of the Definitive Plan.



Source: Study Team

Figure 4.1.4 Results of the Evaluation of Flood Water Level Calculation

(3) Exception of Manalo Bridge Replacement Section by DPWH from the PMRCIP IV

At present, the replacement work of the Manalo Bridge located at Sta. 7+210 within the Phase IV project section is being carried out by the DPWH with its own funds. As of January 2020, the construction work at the immediate upstream section of the Bridge on the left bank has started (driving of SSP for low water channel revetment), and the DPWH is also negotiating with the other landowners of the riverside in order to acquire land necessary for the bridge replacement.

4.1.2 Development Status along the River

Almost all lands on both riverbanks in the targeted stretch of Phase IV are fully utilized with no vacant spaces. In the downstream sections from the Marcos Bridge (around Sta. 10+300), land use is dominated mainly by commercial facilities, factories, warehouses, residential areas and others, while the upstream sections are mainly utilized as river parks. In most cases, residential areas exist behind the river parks.

4.1.3 Existing Drainage Channels and Drainage Systems

Along the targeted stretch of the Phase 4 project, there are 290 existing drainage outlets flowing from the residential area into the river channel. These drainage systems are to be integrated, maintaining the existing topography and drainage systems as much as possible. After integrating and reducing the number of outlets of several small drainage systems, drainage outlets are to be installed at the revetment.

4.2 Policy on River Channel Improvement Plan

4.2.1 Basic Policies on River Channel Improvement

The river channel improvement plan in the Definitive Plan (DPWH2015IV&V) has been reviewed according to the current land use, status of land acquisition by the DPWH, and social and environmental conditions in the surrounding area.

According to the Philippine Water Code (PWC), lands of 3 meters in width from both left and right shoulders of existing riverbanks serve as easement for public works. In addition, a law prohibiting development in a 10-m area from the existing riverbank to serve as a natural Environment Protection Area (EPA) is under consideration.

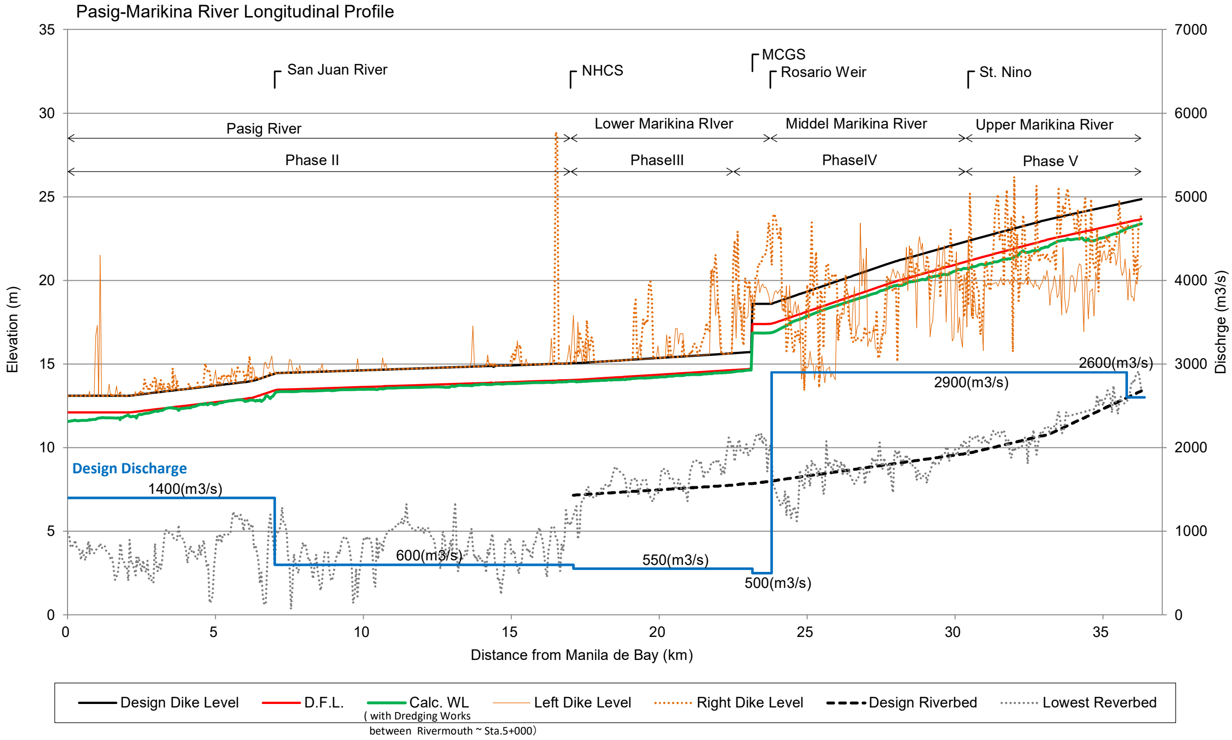
In principle, it is desirable that the centerline of the improved river alignment should be the same as the center of the existing water surface. However, it seems to be difficult to procure land and structures along the riverbanks in almost the entire design section. Therefore, the improved river channel alignment with revetment in each section should be designed and set according to the ease of land acquisition as informed to the Study Team by the DPWH.

There are a number of private lots that may be expropriated for the project according to the river channel alignment appropriately set. The basic principles to fix the alignment of improved river channel is to minimize the land acquisition and compensation for demolished buildings and properties.

Basically, the revetment structure for the low water channel and the highwater channel from Sta. 6+700 to Sta. 10+500 shall be the combination of Steel Sheet Pile (SSP) revetment and the leaning concrete revetment. In the upstream section (from Sta. 10+500 to Sta.13+350), the structure shall be a combination of SSP revetment and parapet wall, or the work shall involve heightening of the existing river wall.

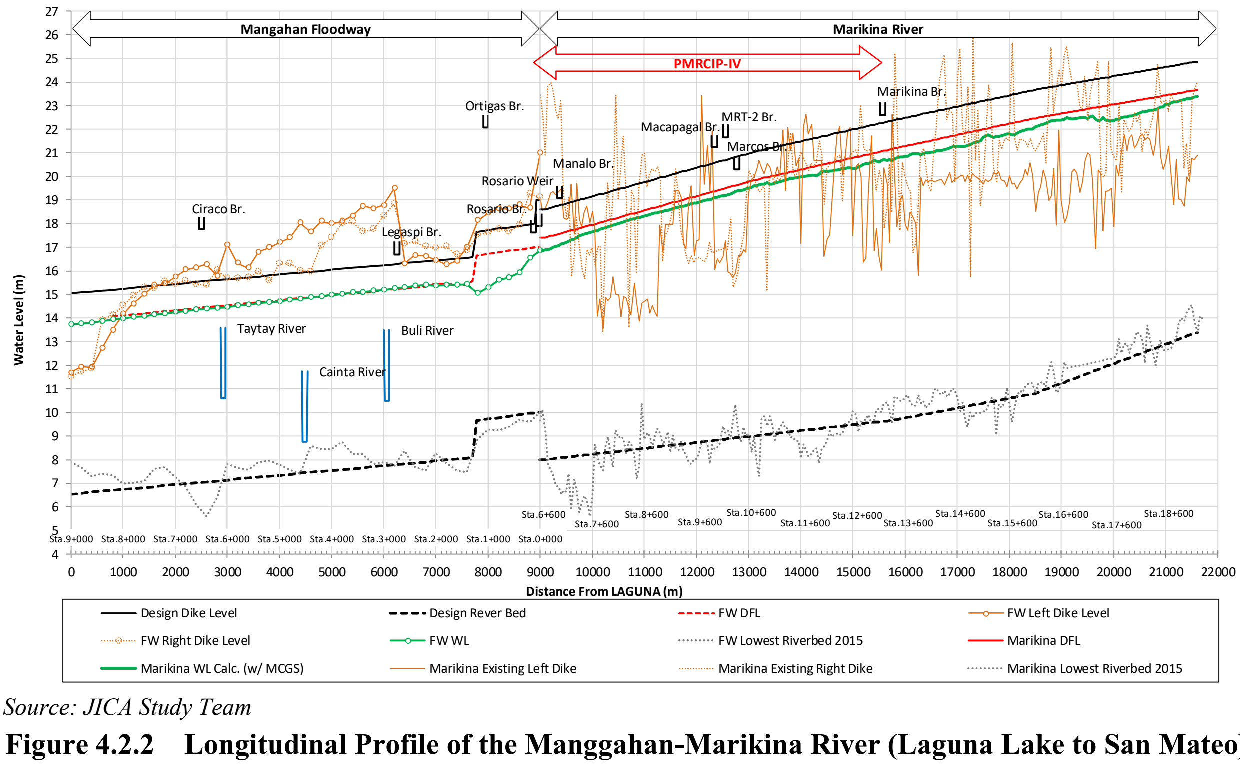
4.2.2 Longitudinal Profile of the Pasig-Marikina River

Figure 4.2.1 and **Figure 4.2.2** show longitudinal profiles of the Pasig-Marikina River and the Manggahan-Marikina River, respectively.



Source: Study Team

Figure 4.2.1 Longitudinal Profile of the Pasig-Marikina River (Manila Bay to San Mateo)



Source: JICA Study Team

Figure 4.2.2 Longitudinal Profile of the Manggahan-Marikina River (Laguna Lake to San Mateo)

CHAPTER 5 NATURAL CONDITION SURVEYS

5.1 Topographic Survey

5.1.1 Objectives and Scope of the Topographic Survey

The main purpose of the topographic survey is to produce the topographic and hydrographic maps with surveys of drainage outlets along river banks for the design and cost estimation, and establishment of concrete control points for reference during the construction stage. The scope of topographic survey included the scope described in **Table 5.1.1**.

Table 5.1.1 Scope of Topographic Survey

Contents	Target	Quantity	Details	Remarks
Topographic Survey (*1)	Marikina River	6 km^2	1:500 Accuracy	Sta.5+400-Sta.13+350
		10 has	1:200 Accuracy	For the MCGS
	Manggahan Floodway	3 has	1:200 Accuracy	For the Cainta Floodgate
		1 ha	1:200 Accuracy	For the Taytay Sluicegate
Hydrographic Survey with River Traversing Survey (Cross-sectional survey)	Marikina River	320 sections	20-m interval	Sta.5+400-Sta.13+350
	Manggahan Floodway	5 sections		For the Cainta Floodgate
		5 sections		For the Taytay Sluicegate
	Drainage channel investigation	500 places	-	Includes location, bed height and cross-section of drainages
Survey for Drainage Outlet	Marikina River	All Outlets		As a result, there are 290 outlets.

**1: with Control points and temporary benchmarks installation (45 points)*

5.1.2 Methodology of the Topographic Survey

5.1.2.1 Hydrographic Survey and Cross Sectional Survey

The hydrographic survey was executed using RTK (Sokkia GRX-2) and an echosounder (Seafloor Hydrolite-TM) mounted on a rubber boat and combined with the DTM from the aerial survey for the final elevation model of the topo area and total station. Cross section survey was carried out every 20 m interval along the PMR alignment as provided by the design team. Two cross section pegs, one on each side of the river, were established by staking the cross sections lines on the ground using RTK (Sokkia GRX-2).

5.1.2.2 Detailed Topographic Surveys

Ground topographic survey at a scale of 1:200 was done on the MCGS, Cainta Creek and Taytay Creek topo areas using RTK and total stations. Data from the cross section and hydrographic surveys of PMR as well as from the drainage inventory survey were also incorporated in this survey.

All prominent structures such as

- Walls, Fences, Piers of Bridges, Culverts and Other Structures
- Edges of Pavements, shoulders of roads, dikes, drainage ditches and facilities
- Electrical lines, water pipes and optical fiber lines
- Trees, Electrical poles, Lightening Poles

were surveyed in the area and appropriately drawn in AutoCAD Civil 3D.

5.1.2.3 Others

The horizontal and vertical coordinates, type, and dimensions of drainages directing flow into the Pasig-Marikina River, Cainta Creek, and Taytay Creek were determined using total stations, and tape. The established GCPs in the areas were used as controls for the inventory survey. A total of 290 drainages were surveyed and measured in the three locations. Drainage pipes inside manholes (CR1.1, CR1.2, CR1.3, and CR1.4) were also surveyed and measured in the Cainta area.

For boreholes made by the Boring Surveyor, horizontal and vertical coordinates were determined using RTK and total stations. Established GCPs and cross section pegs were used as control points in the survey.

5.2 The Geotechnical Investigation

The purpose of the geological survey is to collect data on the ground necessary for the implementation of the detailed design study for the Pasig-Marikina River Channel Improvement Project (Phase IV) and compile it as materials that can be used for the design. The geological survey conducted in this Detailed Design Study is divided into 1) boring survey, 2) soil test, 3) analysis of the survey test results and their summaries. These studies are explained as follows.

- Boring Survey: In the drilling survey, a boring excavation of approximately 20 m was made from the land on the left and right banks of the Marikina River and Pontoon on the river to confirm the stratum and collect samples for soil testing.
- Boring Survey: At the MCGS site, drilling was performed on the left and right banks of the Marikina River and in the center of the river to understand the geological conditions and to confirm the foundation rock for the construction of the weir.
- Boring Survey: At the Cainta river and Taytay river sites on the left bank side of the Manggahan floodway, drilling was performed as a foundation ground survey for the construction of floodgates, and the support layer of the structure was confirmed.
- Soil Test: The soil test was carried out using a soil sample collected by drilling excavation, and a physical test to determine the properties of the soil and a mechanical test to determine the mechanical properties were performed in the soil laboratory.
- Analysis of the survey test results and their summaries: In the analysis and compilation of the results of the geological survey test, a geological cross section required for detailed design of various structures is created from the results of the boring survey, and the soil test results are organized and compiled for each geology to be distributed. proposal was carried out.

The quantity of boring survey is shown in **Table 5.2.1**, and the quantity of soil test is shown in **Table 5.2.2**.

Table 5.2.1 Quantity of Boring Survey

LOCATION	BORING	DEPTH (m)
MARIKINA RIVER	32 HOLES	595.43
MCGS	7 HOLES	56.00
CAINTA / TAYTAY	5 HOLES	167.37
TOTAL	44 HOLES	818.80

Table 5.2.2 Quantity of Soil Test

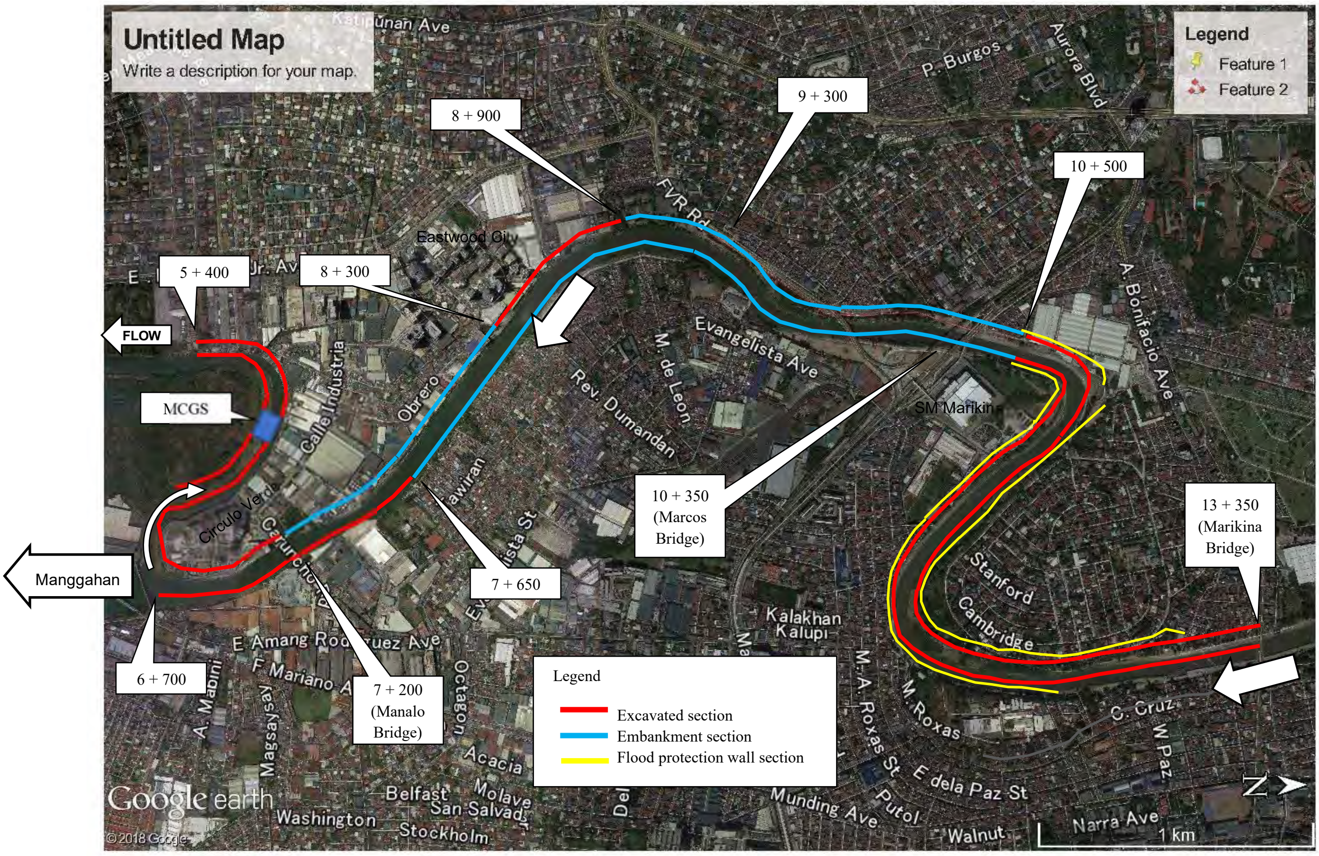
SPT	UDS	Classification	Specific gravity	Moisture Content	Particle Size	Particle Size	Atterberg	Soil Unconfined	Rock Strength	Consolidation
ASTM D1586	-	ASTM D2487	ASTM D854	ASTM D2216	ASTM D422	ASTM E100	ASTM D4318	ASTM D2166	ASTM D2938	ASTM D2435
724	15	366	102	369	366	9	260	5	30	8

CHAPTER 6 BASIC STUDY AND DESIGN OF RIVER STRUCTURES

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

6.1.1 Outline of Basic Design of River Channel

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



Source: Study Team (added on Google map)

Figure 6.1.1 Sections of River Improvement Works in PMRCIP-IV

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

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

Source: Study Team

6.1.2 Basic Design of Revetment for Low Water Channel

6.1.2.1 Type of Revetment for Low Water Channel

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

Table 6.1.2 Type of Revetment for Low Water Channel for Sections

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

Source: Study Team

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

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

According to the comparison of the following 3 alternatives shown, SSP revetment structure would be the self-supporting H-shaped SSP + H-shaped steel. The details of the comparison us shown in **Table 6.1.7 of Main Report.**

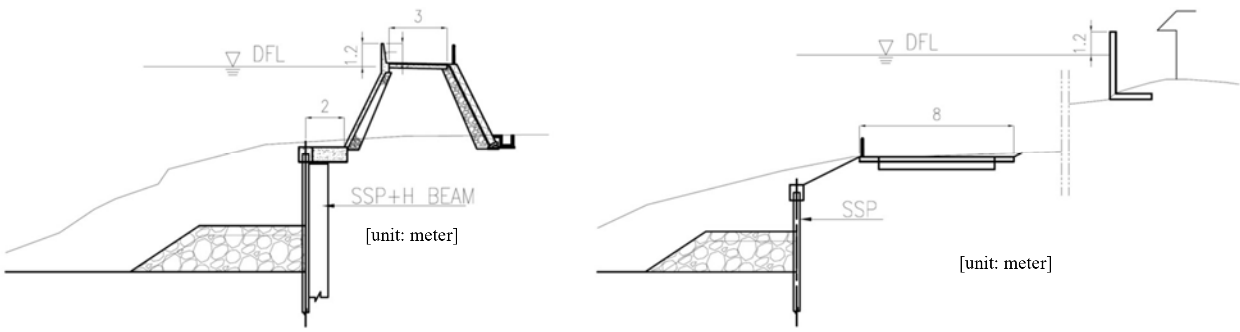
✓ Self-Supporting H-Shaped SSP + H-Shaped Steel -> Adopted

- ✓ Self-Supporting Steel Pipe Sheet Pile(SPSP) -> Less Economical
- ✓ Braces SSP With Tie Rods -> Difficult Due to the Availability of Lands

6.1.2.2 Examination of Steel Sheet Pile Revetment Structure

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

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



Sta. 5+620 to Sta. 10+500
(Embankment Case)

Sta. 10+500 to Sta. 13+350

Source: Study Team

Figure 6.1.2 Standard Revetment Structure

6.1.2.3 Study of Low Water Revetment Consolidation

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

1) Selection of Foot Protection Structure

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

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

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

6.1.2.4 Study on Foot Protection of Bridge Substructure

(1) Target Bridges

As shown in **Figure 6.1.3**, there are six (6) bridges across the Marikina River in the target section of the Project. Among them, the target bridges were set to four (4) bridges: Macapagal Bridge, LRT2 Bridge, Marcos Bridge, SM Marikina Bridge. Manalo Bridge and Marikina Bridge are excluded from this design for the following reasons.

Source: Study Team Added on Google Earth

Figure 6.1.3 Target Bridges

- Manalo Bridge: The foot protection as detailed design of scouring protection is included in the detailed design of the newly rebuilt bridge implemented in Phase II SA2. The foot protection as detailed design of scouring protection is included in the detailed design of the newly rebuilt bridge implemented in Phase II SA2.
- Marikina Bridge: Countermeasures against scour are included in the reinforcement work to be carried out by DPWH-NCR

6.1.3 Design of Dike (Dike Revetment, Parapet Wall)

6.1.3.1 Structure of Dike and Revetment

The sections and their outlines are summarized in **Table 6.1.3**.

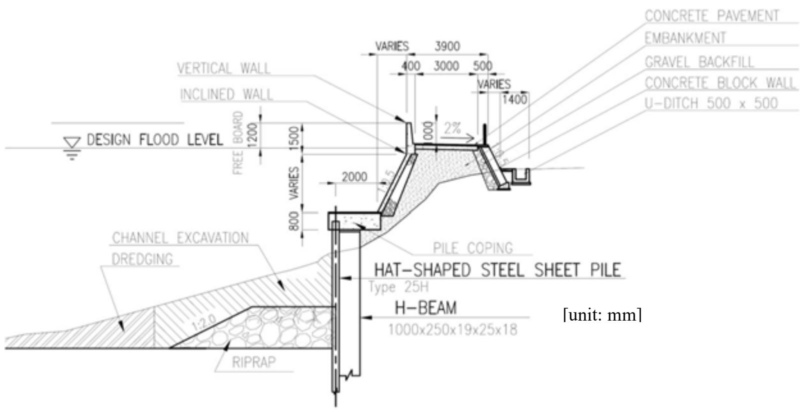
Table 6.1.3 Sections and Outline of Dikes And Revetments

Section		Summary
(1)	Downstream end (Sta.5 + 400) - Rosario weir (Sta.6 + 600)	The structure of downstream of MCGS is a plain soil channel, while directly upstream and downstream of MCGS is SSP and concrete revetment, and the section from MCGS to Rosario Weir is the SSP with concrete revetment.
(2)	Risario weir (Sta.6 + 700) - Marcos Bridge (Sta. 10 + 500	The section will be an excavated channel; only slope protection by revetment will be provided. In the embankment section, the slope protection by embankment and revetment will be constructed. The revetment consists of SSPs and concrete revetment in entire sections.
(3)	Marcos Bridge (Sta. 10 + 500) - Marikina Bridge (Upstream end Sta. 13 + 350)	Since the entire section will be an excavated channel, SSPs at low water channel plus flood protection wall (parapet wall) will be installed. The Definitive Plan proposes only the excavation of the low water channel on the right bank from Sta. 11+000 to Sta. 12+550. To secure enough width of low water channel in the future, the low water channel shall be protected with SSPs for the width of 80m of the river cross section, and roads/sidewalks will be provided on the high-water channel.

Source: Study Team

6.1.3.2 Dike and Revetment Structure

In the stretch from the downstream of MCGS (Sta.5 + 620) to Marcos Bridge (Sta. 10 + 500), the slope protection of the excavated channel and the revetment structure of the dike embankment are summarized in **Table 6.1.4**.



Source: Study Team

Figure 6.1.4 Standard Cross-Section of Revetment Applied to Sta. 6+700 to Sta. 10+500

Table 6.1.4 Outline of Design Specifications of Embankment and Revetment

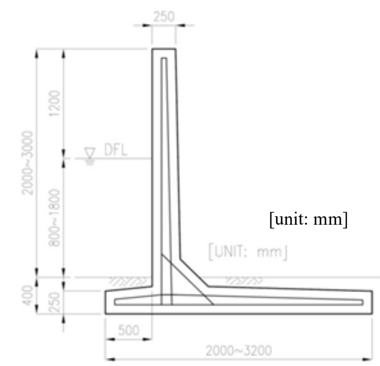
Item	Specification	Remarks
Design Scale	100 year probability (With the dam and Retarding Basin.)	
Design Discharge	· Downstream of Rosario Weir (Sta.6 + 600) : 500 m^3/s · Upstream of Rosario Weir (Sta.6 + 600) : 2,900 m^3/s	
Freeboard	· Downstream of MCGS (Sta.6 + 010) : 1.0 m · MCGS (Sta.6 + 010) ~ Rosario Weir (Sta.6 + 600) : 1.2 m · Upstream of Rosario weir (Sta.6 + 600) : 1.2 m	- Due to the influence of the water level at the diversion point of Manggahan Floodway, the same freeboard as the upstream side of Rosario Weir is considered.
Crest Width of Dike	3.0 m minimum	- Special type of dike of which main part is composed of steel sheet pile and concrete
Width of Maintenance Road	Basically 3.0 m or more	- If there is a suitable path to replace the maintenance road and the site does not have enough space, narrow it to 1.5 m. - 4.0 m is secured in the vicinity of MCGS and EFCOS in consideration of facility maintenance and management.
Side Slope	Steeper gradient than 2.0 :1 However, slope protection by concrete, etc., is installed.	- In case of no slope protection by concrete, etc., 2.0:1 would be applied.
Extra Embankment	20 ~ 45 cm	- Determined considering soil conditions of foundation ground, dike body and embankment height

Source: Study Team

6.1.3.3 Structure of Flood Protection Wall

In the section upstream from the Marcos Bridge (Sta. 10 + 500 ~ 13 + 350), the existing river bank would be used as a flood plain, and the flood protection wall of by parapet wall is installed on the high elevation , or the existing concrete wall is raised to secure the necessary embankment height.

Standard section of the flood protection wall is shown in **Figure 6.1.5**.



Type-T: Applicable for ground elevations between 2 m and 3 m.

Source: Study Team

Figure 6.1.5 Example of flood protection wall applied from Sta. 10 + 500 to Sta. 13 + 350

6.2 Drainage Plan and Design

6.2.1 Summary of Basic Design for Drainage Facility

The target areas of drainage planning are the both bank of the Marikina River from Sta. 5+400 to Sta. 13+350. A total number of existing outlets are 283 locations within these stretches. As a concept of drainage planning, the number of proposed new outlets will be minimized in order to prevent or minimize backflow from these outlets during high flood stages of the Marikina River. The proposed drainage facilities are summarized as shown in **Table 6.2.1**.

Table 6.2.1 The Draft Proposed Drainage Facility

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

Source: Study Team

6.2.2 Drainage Survey and Data Collection

The drainage survey was conducted for 282 locations of existing outlets for the both bank of the Marikina River from Sta. 5+400 to Sta. 13+350. The results are shown in **Table 6.2.2**.

Table 6.2.2 The Summary of Existing Outlets

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

Source: Study Team

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

Furthermore, the latest version of Land Use Map was collected for above three (3) cities.

6.2.3 Drainage Planning

6.2.3.1 Planning Conditions

Conditions are set according to "Design Guidelines, Criteria and Standards: Volume 3 – Water Engineering Projects, 2015" (Hereinafter referred to as "Drainage Facility Design Guidelines") which is an urban drainage plan and design guideline of DPWH. The major conditions are summarized in **Table 6.2.3**.

Table 6.2.3 Summary of Planning Conditions In Drainage Facility Design

Item	Condition	Remarks
Design Scale	25 years Return Period	
Minimum Pipe Diameter	900 mm	As a result of discussions with the DPWH-BOD, the minimum diameter of 900 mm is not applied to rain gutters, domestic wastewater and collector pipes from houses, and the same size as the existing one is adopted for the new sections.
Discharge Calculation	Rational Formula	Estimation of runoff coefficient and time of concentration is established and calculated based on the drainage facility design guidelines.

Source: Study team summarized from the Drainage Facility Design Guideline

6.2.3.2 Planning for Drainage Facility

Drainage facility plan in this project is determined based on drainage plan conditions and basic concept. Drainage facility plan determined is shown in **Sub-section 6.2.3.2 (2) in Main Report**.

6.2.4 Basic Design Condition of Drainage Facility

6.2.4.1 Basic Design of Outlet

Design according to the study method shown in **Table 6.2.4**.

Table 6.2.4 Summary of Study Methods in Design of Drainage Facilities

Item	Study Method	Remarks
Calculation of the Size of The Drainage	Manning's equation	Base on wastewater facility design guidelines
Velocity in the Conduit	0.8 m/sec min Ideally, within 1.0 ~ 1.8 m/sec	Base on wastewater the Drainage Facility Design Guideline Based on the "Technical Standards and Guidelines for Planning and Design, Draft, VOLUME II: URBAN DRAINAGE, March 2002, DPWH and JICA"
Discharge Calculation	Rational Formula	Base on the Drainage Facility Design Guideline
Pipe Connection	Pipe-Top Connection	Same as above
Manhole Placement	Manholes are normally located at the convergence of two or more pipes, at points for maintenance, and at changes in grade or alignment. The maximum spacing of manholes would be adopted at 50m	Same as above

Source: Study Team Summarized

6.2.4.2 Basic Design of Sluiceway

The results of basic design are summarized in the table below.

Table 6.2.5 Summary of Sluiceway Design

Item	Type/specifications	Verification/Remarks
Structure Type	Flexible Sluiceway	Since residual settlement exceeds 5 cm, the structure shall be flexible in accordance with "Guide For Flexible Sluiceway, Japan".
Cross-Sectional Structural Form	Rectangular Cross Section with Cast-In-Place Concrete	· It is more difficult to make cast-in-place concrete circular culvert. · Pre-cast concrete pipes are available but there would be problems on water-tightness on the joints and longitudinal deformation if the bedding is not properly constructed. · In case of steel pipe, welding is needed on the joint.
Gate type	Flap Gate (Hinged Type)	Manual operation is not required, and power-free switching is available

Source: Study Team

6.3 Basic Design of Manggahan Control Gate Structure (MCGS)

6.3.1 Summary of Basic Design of MCGS

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

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

Table 6.3.1 Summary of Basic Design Results (MCGS)

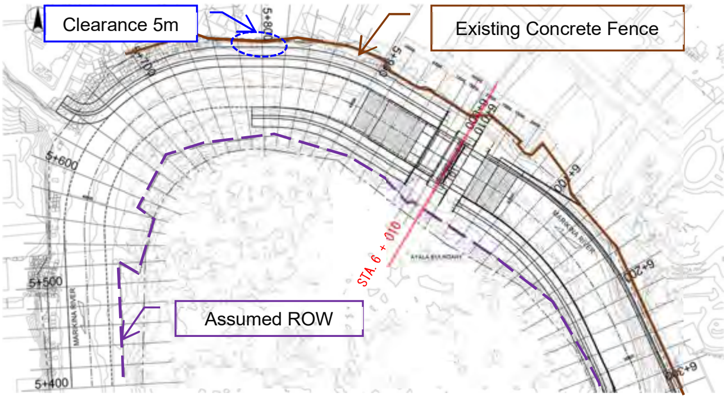
Items	Specifications	Descriptions/Remarks
Structural Category	Movable Weir	- Certainty of diversion considered. - Dewatering from Laguna Lake also considered.
Location	STA.6+010	- Nearer location from EFCOS (STA.6＋550), considering the availability of land and curve of river channel
DFL	Upstream：EL. 17.400 m Downstream：EL. 14.711 m	
Water Level (for Structural Design)	(Flood) Upstream：EL. 17.400 m Downstream ：EL. 13.425 m	- Water level downstream is based on the value of hydraulic model test in this study.
	(Low Water Case) Upstream：EL. 17.400 m Downstream：EL. 11.003 m	- Water level downstream is based on the water level observed at Rosario Weir (Junction Side)
Design Dike Crown	Marikina River：EL.+18.600	- DFL + Freeboard: 1.2m
Number of Gates	2-Span	- Flexibility of operation and redundancy are considered in case of malfunction
Span	31.8m + 15.20 m （Clear Span：28.7m+11.3m）	- Minimum: 12.5 (To avoid closure of water conduction) - 40 m width with 2 gates to secure the water width in ordinary condition - Regulate discharge at 500m^3/s by fully opening only the side which has 11.2 m clear span
Top of The Gate	EL. 17.400 m	- DFL
Sill Elevation	EL. 7.850 m	- Design Riverbed
Energy Dissipator	Stilling Basin L=26.4m、EL.7.050m	- Not to disturb ferry boats passing in ordinary condition and floods flowing smoothly - About 20% of difference of water level between upstream and downstream - Refer to Japanese actual cases[1)]
Length of Main body	20.5m	- The width of maintenance bridge, staircase, column, pier considered
Length of Apron	Upstream：15m	- Refer to Japanese actual cases[1)], 1/2 of downstream side
	Downstream：30m	- Based on the creep distance for seepage control
Length of Bed Protection	Upstream：15m	- Same length as the upstream apron
	Downstream：44m	- Calculated in accordance with “Structural Design Guide for Groundsill”
Top of Main body	EL. 19.0 m	- Finished elevation of revetment (including extra embankment)
Top of Gate Control Structure	EL. 32.05m	- 1.5 m allowance above “Top of Gate” considered
Top of Gate	EL. 17.400 m	- Same as DFL
Type of Gate	Lift Roller Gate	- Selected based on maintenance and economic aspect
Gate Leaf Structure/ Material	Shell Structure/ Alloy Saving Duplex Stainless Steel	- (Structure) Garbage/debris flow, sedimentation and cost efficiency considered - (Material) Lower Lifecycle Cost, applicability under brackish water condition in Pasig Marikina River
Operation	Hoisting Device：Electric motion (commercial power supply) Operation；remote and local control	- Commercial power supply is used with 2 units of generator for backup in case of blackout - In addition to remote and local control, emergency operation panel is installed in generator house
Maintenance Bridge	PC Girder Bridge （Effective Width：4.0m）	- Only maintenance vehicles pass

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

6.3.2 Basic Design of MCGS

6.3.2.1 Study on the Location of MCGS

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



Source: Study Team

Figure 6.3.1 Location of MCGS

6.3.2.2 Study on the Basic Structural Specifications

(1) Study on Type of Weir and Gate

The main purpose of the MCGS, the subject of this design, is discharge regulation. Considering this purpose, the applicable type of weir is selected. The following site conditions and constraints are taken into account to extract the items that may be applicable to this facility.

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

Based on the above consideration, the following four types of comparison are performed. Details of the comparative study are shown in **Table 6.3.20 of Main Report**.

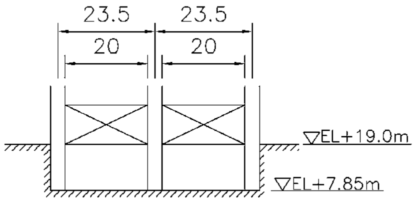
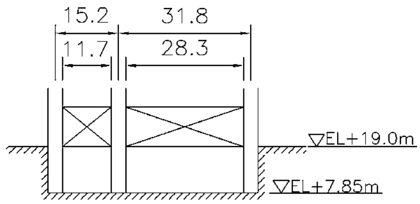
- Alternative 1 : Fixed Wheel Roller Gate -> Adopted
- Alternative 2 : Double Deck Fixed Wheel Roller Gate -> Complex to operate, less economical
- Alternative 3 : Radial Gate -> There is a weak point against overflow, it is not possible to drop by its own weight.
- Alternative 4 : Rising Sector Gate -> Issues in maintenance and management and low economic efficiency

In addition to the ease of maintenance, reliability, and economics, it also takes into consideration that local technicians have sufficient knowledge of operation and maintenance, since same type was adopted in the weir and floodgate nearby. "Alternative 1: Fixed Wheel Roller Gate" is selected.

(2) Span and Span Allocation

When setting the span and its number, "Cabinet Order Concerning Structural Standards for River Administration Facilities, etc. (hereinafter called the "Structural Cabinet Order")" is referred. The two proposals shown in **Table 6.3.2** are extracted. "Alternative 2: 15.2 m + 31.8 m" is adopted as a result of the comparison and examination of the results.

Table 6.3.2 Comparison of Span Allocation

Item	Alternative 1 : 23.5m + 23.5m	Alternative 2 : 15.2m + 31.8m
Figure (During Flood)		
Clear Span	· 20 m + 20 m	· 28.3 m + 11.7 m · (This allocation is determined from the hydraulic model experimental result.)
General	· Proposal in 2002 PMRCIP-I · It satisfies the span length specified in the Structural Cabinet Order, and secure a water surface width of 40 m of the Marikina River in ordinary time between total net lengths	· This alternative narrows the s one side of two gates to regulate the flood discharge. · Referring to "mountainous constriction part" stated in Structural Cabinet Order, the minimum width of the narrower span is set to 12.5 m to avoid driftwoods and garbage blocking the water way.(Since 15.2 m > 12.5 m, this conditions is satisfied.) · Securing the water surface width 40 m of the Marikina River in ordinary time between total net lengths
Discharge Method	· Underflow below the gate leaf	· Overflow on the fixed portion of the weir
Evaluation	· It is difficult to quantitatively evaluate the gate vibration due to underflow discharge and to cope with it after its occurrence.	· There is no concern about gate vibration like Alternative 1, and flexibility is also provided in its gate operation.
		Recommended

Source: Study Team

(1) Study on Maintenance Bridge

The specifications of the management bridge are summarized in **Table 6.3.3**.

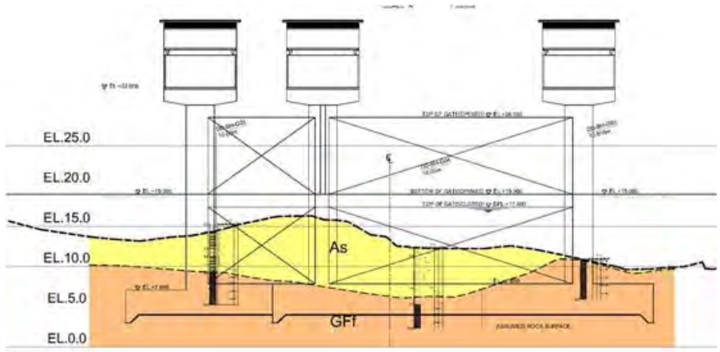
Table 6.3.3 Summary of MCGS Maintenance bridge specifications

	Item		Conditions/specifications, etc.
1	Road Condition		· Maintenance Road · Usage as a public road is not considered
2-1	Bridge Condition	Bridge Length	L1 = 14.55 m (Shorter span), L2 = 31.15 m (Wider span)
2-2		Width Composition	W = 0.500 m (Covering) + 4.000 m (Roadway) + 0.500 m (Covering) = 5.000 m
2-3		Paving	Asphalt Pavement (ACP), t = 50 mm
3	Loading Condition		· Set dead load, live load, fatigue load, and impact load according to DGCS · The maximum design wind speed is set at V = 200 km/h in consideration of consistency with the weir body.
4	Superstructure Type		· PC I girder bridge (AASHTO digit) ➢ Span Length (9.0 m to 42.7 m) ➢ Comparison of economic efficiency with RC slab bridge and steel I-girder bridge

Source: Study Team

(2) Study on Type of Foundation

According to the previous geological investigation around the MCGS site (refer to Figure6.3.7), sand layers with N value of 30 or more, or base rocks with N value of 50 or more, are distributed below approximately EL.7.6m at the MCGS location.(See **Figure 6.3.2**) Since the subgrade has reached the support layer in most parts, the spread foundation type is adopted.



Source: Study Team

Figure 6.3.2 Assumed Geological Cross Section (Weir Position)

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

Dimensions of the main body are set referring to "Technical Standards for River Sand Control (Draft) Design Part I" and ""Design of Weir " (Dam Engineering Center). Here, the set parameters are Organize in **Table 6.3.1**. Furthermore, based on the results of the hydraulic model test, an energy dissipator with an end sill is installed at the downstream of the narrower span gate. Details of the specifications of each main body are shown in the **Sub-section 6.3.3.7 (9) of Main Report**.

6.3.3 Study on Gate Structure and Hoist

The gate structure and hoist of MCGS is determined as shown in **Table 6.3.4**.

Table 6.3.4 Summary of Gate Structure and Hoist of MCGS

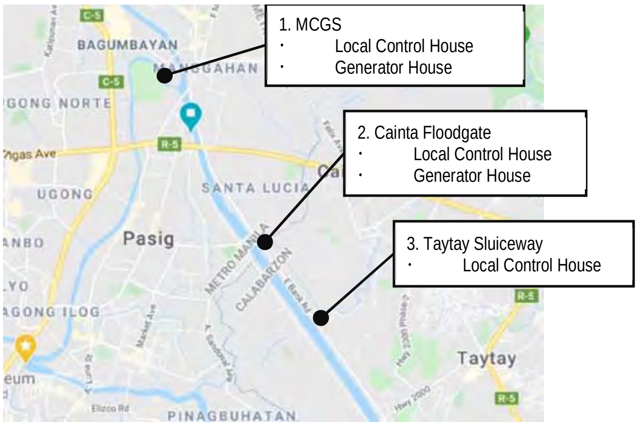
Item	Narrower / Wider Span Gate	Specification	Verification
Gate Leaf Structure	Narrower Gate Span	Plate Girder Structure	· Since the ratio of the gate leaf height to the net diameter is about 1/1.22, the girder structure is adopted from the gate dimension and structural relation diagram shown in Technical Standard for Dam and Weir Facilities(draft).
	Wider Span Gate	Shell Structure	· Since the ratio of the door height to the net diameter is about 1/2.96, the gate is located in the overlapping area in the gate size and structure diagram shown in Technical Standard for Dam and Weir Facilities(draft). · Shell structure is selected because it is economical and Hardly affected by sedimentation and driftwood/garbage.
Gate Material	Both	Alloy saving duplex stainless steel (SUS 323 L)	· Brackish Water Environment · Lifecycle Cost LCC is the lowest. ✓ SM400 ******* PHP (1.00) ✓ SUS 316 ******* PHP (1.06) ✓ SUS 323 L ******* PHP (0.98)
Type of Hoist	Both	Wire Rope Winch Type	· There are a lot of cases in the Philippines. · The structure is simple and easy to maintain. · It is also economical. · No need for control bridges
Type of Wire Rope Winch	Narrower Gate Span	2 Motor 2 Drum Type (2M2D)	· The span length exceeds 25 m
	Wider Span Gate	1 Motor 1 Drum Type (1M1D)	· It does not require an electric shaft and is economical.

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

6.3.4 System Planning

6.3.4.1 Basic Concept for Operation System of the MCGS

The MCGS shall be properly operated in harmonization with other important control gate structures, such as Rosario Weir and two (2) floodgate structures newly constructed along the Manggahan Floodway by MMDA or DPWH (See **Figure 6.3.3** below. In this section, the imperative power units and control system have been discussed for the appropriate operation of the MCGS and integrated control system among other control gate structures.



Source: Study Team

Figure 6.3.3 Location of Three (3) Control Gate Structures to be Operated under Integrated System

6.3.4.2 Basic Design of Power Unit and Control System of the MCGS

(1) Power Unit

Table 6.3.5 shows the summary of power unit facilities. In addition, the basic power plant layout diagram of MCGS is shown in **Figure 6.3.4**.

Table 6.3.5 Summary of power Unit (MCGS)

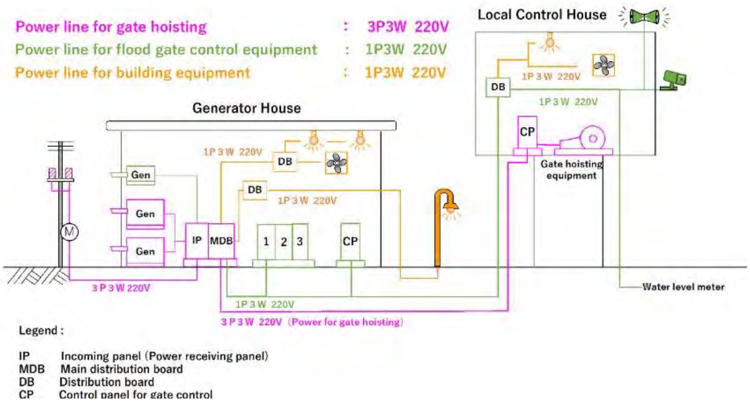
Item	Specification	Remarks
Main Power Unit	Electric Motor	Consideration of the reliability of starting, stability of opening and closing speed, low failure rate, ease of maintenance and remote operability.
Reserve Power Unit	Store Reserve Power Units	Considering flood control safety and cost efficiency.
Power Supply Unit		
a Main Power Supply Unit	The commercial power supply is received in 3φ3 W AC 200V 60Hz and 1φ2 W AC 200V 60Hz directly at the operation panel in the operation room and uses them as power and operation power.	
b Standby Power Supply System	Permanent standby power generation equipment	· Ensure reliable gate operation due to the important gate facility. · Ensure power supply for attached water level gauge equipment, safety equipment, remote control equipment, building equipment, etc.

Source: Study Team

(2) Control System

1) Machine Side Control Panel

A local control panel is installed on the operation deck for operation, periodical maintenance and normal operation. The specific control function of each equipment is included in the



Source: Study Team

Figure 6.3.4 Basic Concept and Layout of Power Unit of MCGS

local control panel. The operation is performed by push-button operation from the local control panel and remote operation from the central control station.

2) Remote Monitoring and Control System

(a) MCGS Operational System Level

The operation of the facility can be generally classified into "Instruction", "Operation" and "Checking and Monitoring", which are combinations of implementation methods for individual functions. The combination summarized in five system levels is shown in **Table 6.3.6**

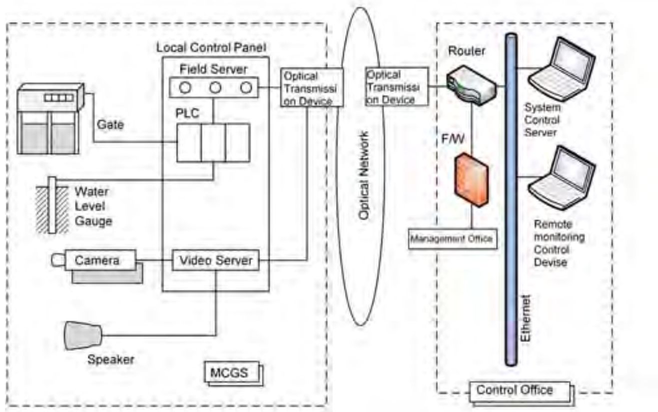
For MCGS operation, system level 5 shall be recommended for the following reasons. A comparison of each system level is shown in **Table 6.3.53 of Main Report**.

Table 6.3.6 System Level in Facility Operation

System Level	Instruction	Operation	Checking and Monitoring
1	Individual Instruction	Field Operation	Confirmation and Record by Manager
2	Simultaneous Instruction	Field Operation	Confirmation and Record by Manager
3	Simultaneous Instruction	Field Operation	Input by Field Operator and Confirmation by Manger
4	Instruction	Field Operation	Automatic Monitoring
5		Remote Operation	Automatic Monitoring

Source: Study Team

- The condition of the water level of the main river, status of inflow from Cainta River and Taytay Creek to Manggahan Floodway and the operation status of related river management facilities (Rosario weir, gates, etc.), which are the information necessary to make decisions, can be observed and reliable and quick operation can be performed.



Source: Study Team

Figure6.3.5 Distributed Web System Configuration

- Reduces workload of operation during flood events and allows the managers to operate facilities by themselves. (Operation of facilities can be performed without outsourcing to local governments, etc.)

(b) Overall System Configuration

As a method for the remote monitoring and control system, "Distributed Web System" is recommended for its versatility and expandability, and the system design would be proceeded under this system. A detailed comparison of each system is shown in **Table 6.3.54 of Main Report.**

3) Update of existing EFCOS system.

According to the field survey conducted prior to this design, many of the existing EFCOS systems were established about 20 years ago and some of them have exceeded their lifetime considering the installation environment. In view of the current status of telecommunications equipment, upgrading equipment that have exceeded their service life shall be considered, together with the MCGS operation system.

6.4 Floodgate to Prevent Backflow

6.4.1 Summary of Basic Design of Floodgates to Prevent Backflow

in this study, based the existing river width of the Cainta River around the floodgate location and the flow capacity of the existing box culvert from Taytay Creek to Manggahan Floodway, each floodgate is being studied.

A list of the specifications established in the above studies is shown as the basic design results.

Table 6.4.1 Summary of Basic Design Results (Cainta Floodgate)

Item	Specifications	Description / Remarks
Function	Water Control Function	· To prevent backflow from Manggahan Floodway during floods
Location	STA. 4 + 525	· Approximate center of the flow of Cainta River
Proposed Discharge	95 m3/s	· Based on the existing river width, this was set in 2008 Pre-F/S
DFL	Floodway Side: EL. 14.853 m Tributary Side: EL. 13.34 m	·
Water Level (for Structural Design)	Floodway Side: EL. 14.853 m Tributary Side: EL. 10.1 m	· Based on the observed lowest water level in Laguna Lake
(for Operation)	Floodway Side: EL. 12.940 m Tributary Side: EL. 13.940 m	· Water level in floodway side is 1 m below the dike crown of tributary 1) · Water level in tributary is dike crown 1)
Dike Crown (Design)	Floodway Side: EL. + 15.940 m Tributary Side: EL. 13.940 m	·
(Existing)	Floodway Side: EL. + 18.00 m Tributary Side: EL. + 13.00 m	· In case of the tributary, the existing ground elevation (After topographic survey, it will be reconfirmed)
Number of Gates	2 Gates	· Considering the redundancy in case of malfunction · Avoiding the size of the gate becomes large
Span	2 spans x 19.0 m (Pure Diameter: 2 x 16.00 m)	· Minimum 15.0m 2) · Based on the existing rive width which is about 35 m (The width of proposed river channel was 34.6 m 2008 Pre-F/S)
Invert Elevation	EL. 8.75 m	· Design riverbed of the tributary
Length of Main body	31.9 m	· Considering the width of maintenance bridge, staircase, column, pier
Length of Apron	Upstream: 11.5 m	· Same as the length of wing wall
	Downstream:18.0 m	· Same as the length of Bed Protection in the Downstream
Length of Bed Protection	Upstream: 5 m	· Approximately same as the water depth at DFL3)
	Downstream side: A: 0.0 m B: 15.0 m	· A: w/o Bed Protection Section A · B: 3-5 times of water depth in the downstream3)
Top of Main Body	EL. 18.4 m	· Finished elevation of the dike (including extra embankment)
Top of Gate Control Structure	EL. 29.4m	· Elevation of the bottom of gate during its maintenance is set to the dike crown EL.+18.0m. · Considering 1.6 m allowance abovementioned elevation

Item	Specifications	Description / Remarks
Top of Gate	EL. 16.060 m (Top of Gate)	・ Rounding the value of DFL in floodway side + Freeboard 1.2 m
Type of Gate	Fixed Wheel Roller Gate	・ Selected based on maintenance and economic aspect
Pier Structure/ Material	Girder Structure/ Alloy-saving Duplex Stainless Steel	・ (Structure) Considering garbage/branches flowing and sedimentation, furthermore, cost efficient ・ (Material) Considering LCC
Operation	Hoist：Electric motion (commercial power supply) Operation；Remote and Local Control	・ Commercial power supply is used with 1 units of generators for backup in case of blackout ・ In addition to remote control and machine side, emergency control panel is installed in generator house
Maintenance Bridge	RC Bridge (Effective width: 7.30 m x 2 or more)	・ Maintenance bridge is open to public. ・ Considering the expansion to 4 lane road

1) *Technical standard for the Facilities of Dams and Weirs*
2) *Cabinet Order Concerning Structural Standards for River Administration Facilities, etc.*
3) *Structural Design Guide for Groundsill*

Source: Study Team

Table 6.4.2 Summary of Basic Design Results (Taytay Sluiceway)

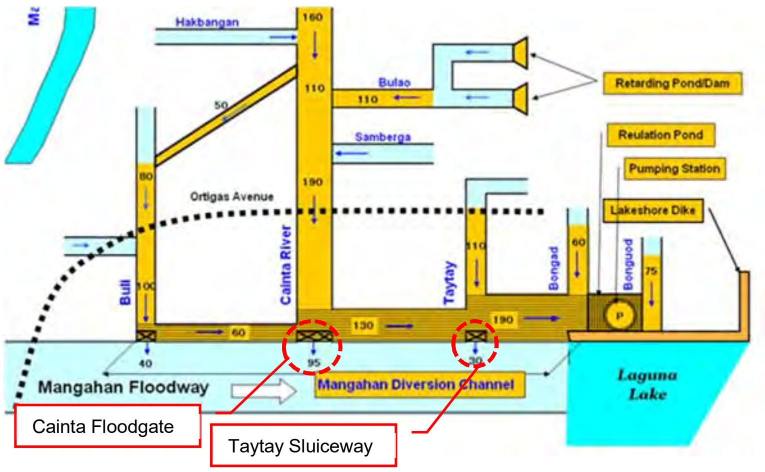
Item	Specifications	Description/ Remarks
Function	Water Control	・ To prevent backflow from Manggahan Floodway during floods
Location	STA.6+090	・ Approximate center position of the existing box culvert
Proposed Discharge	30 m^3/s	・ Based on the existing flow capacity (same as 2008 Pre-F/S)
DFL	Floodway Side：EL.+14.520 m Tributary Side：EL.+13.500 m	・ DFL of the Tributary is at the upstream end of the existing box culvert ・ Bottom side of top slab of the box culvert is EL.+12.4m
Water Level (for Structural Design)	Floodway Side：EL.+14.520 m Tributary Side：EL.+10.100 m	・ Based on the observed lowest water level in Laguna Lake
(for Operation)	Floodway Side：EL.+13.100m Tributary Side：EL.+14.100m	・ Water level in floodway side is 1 m below the dike crown of tributary [1)] ・ Water level in tributary is dike crown [1)]
Dike Crown (Design)	Floodway Side：EL.+15.620m Tributary Side：EL.+14.100m	・ About the tributary side, it is DFL in the upstream side of the existing box culvert
(Existing)	Floodway Side：About EL.+15.6m Tributary Side：About EL.+11.8-13.5m	・ In case of the tributary, the existing ground elevation
Type of Structure	Sluiceway Connecting with the Existing One	・ Considering advantages in the aspect of structure, maintainability for seismic resistance and workability
Number of Gates	3	・ Same as the existing box culvert
Size of Box Culvert	B2.5m x H1.8m x 3 Barrels	・ Same as the existing box culvert ・ Considering residual settlement, the value shall be reviewed in the detailed design stage.
Invert Elevation	EL. +10.600 m	・ Design Riverbed, Invert Elevation of the existing Box Culvert
Length of Box Culvert	Conduit：8.0m	・ Considering the 70 cm height of breast wall, 2.0:1 side slope
Breast Wall	Width：1.0 m	・ To avoid particles of embankment moving and drawing out [1*]
	Height：0.7m	・ Haunch 50 cm +20 cm
Wing Wall	Downstream: Length: 6.0m、Height: 2.1m	・ (Length) Longer than cross-sectional shape of the existing dike [2)]
		・ (Height) considering the height of existing dike
Top of Gate Control Structure	EL. 16.200 m	・ 0.5 m allowance above the top of the opened gate.
Top of Gate	EL.12.4000	・ Bottom elevation of the top slab of the existing box culvert
Type of Gate	Fixed Wheel Roller Gate	・ Due to much garbage, assurance of closure is prioritized.

Item	Specifications	Description/ Remarks
Pier Structure/ Material	Alloy Saving Duplex Stainless Steel	・ Considering less maintenance and technical novelty, stainless is selected ・ Since the life cycle cost is almost the same as the conventional type, high strength and same type as Cainta Floodgate is selected
Operation	Hoist：Electric motion (commercial power supply) Operation；Remote and Local Control	・ Commercial power supply is used with 1 unit of generators for backup in case of blackout ・ Remote control and machine side.
Maintenance Bridge	Steel (width: 1.0m)	・ Access by the operator and administrator

1) Technical standard for the Facilities of Dams and Weirs
2) Guideline for Flexible Sluiceway
Source: Study Team

6.4.2 Background and Purpose of Installation

There are no gates at the Confluence of the Cainta River and the Confluence of the Taytay Creek along the Manggahan Floodway. When the water level of the floodway is high, backflow along the tributaries occurs.



Source: 2008 Pre-F/S

Figure 6.4.1 Distribution of Proposed Discharge

In this connection, the Study on a Flood Mitigation Project in the East Manggahan Floodway Area (hereinafter referred to as 2008 Pre-F/S) was conducted in 2008 to improve drainage in the left side of the floodway to a 10-year return period flood

In the 2008 plan, drainage from the Cainta Floodgate and Taytay Sluiceway was based on the flow capacity of the existing river channels and box culverts.

When the Philippine standard was updated in 2015, the design scale was upgraded to 15 years. The review of the plan based on this new standard will require change to the distribution of discharge in the landside, revision of the river improvement plan, and review of the facility layout plan for the discharge of floodwater to the floodway.

The purpose of installing both a floodgate and a sluiceway in this project is to prevent the backflow of floods from the Manggahan Floodway to the Cainta River and the Taytay Creek, not the drainage improvement in the land side of the dike. In addition, since the drainage channel in the land side has not been improved yet, if the floodgate is installed in accordance with the policy proposed in the 2008 Pre-F/S, it will not become a bottleneck to the current condition and will not have any adverse effect. Therefore, in this project, the floodgates to prevent backflow will be installed based on the following policy.

- ✓ As proposed in the 2008 Pre-F/S, the Cainta Floodgate will be installed on the existing river channel width while the Taytay Sluiceway is on the existing box culverts.

When the drainage plan in the land side is reviewed from the 10-year return period, the flow rate to the floodway in the initial stage of flood (proposed flow rate from both floodgates) is assumed to be maintained at the present rates, i.e., 95 m^3/s and 30 m^3/s, respectively.

6.4.3 Basic design of Cainta Floodgate

6.4.3.1 Location of Floodgate

Table 6.4.3 shows a comparison between the alternatives. Based on this comparison, since the impact on social environmental issues can be minimized and advantages on the access to the floodgate and the installation of dike with the utilization of existing structures are expected, Alternative 2 is recommended.

The river side from the current dike alignment would be excluded from the alternatives, because it interferes with the river area of the Manggahan Floodway. In addition, the center line of the floodgate is set at the position of Sta. 4 + 525 of Manggahan Floodway, considering the point where the center line of the floodgate is approximately at the center of Cainta River.

Table 6.4.3 Comparison of Locations for the Cainta Floodgate

Items	Alternative 1 Landside of the Existing Dike	Alternative 2 Riverside of the Existing Dike
Figure		
General	· The floodgate will be installed at the land side of the existing road crossing the Cainta River and dike embankment connecting to it will be installed. · The existing road bridge will be left as it is.	· The floodgate will be installed along the existing dike alignment, and the existing road bridge will be replaced as a maintenance bridge that will be used also for ordinary traffic.
Evaluation	· There are some advantages on the cost and unnecessity to replace the existing bridge. However, the schedule of the project is probably affected by its social environmental impact.	· The access to the floodgate is good. · Social environmental impact can be minimized.
		Recommended

Source: Study Team

6.4.3.2 Study on the Basic Structural Specifications

(1) Study on Type of Gate

The following site conditions and constraints are taken into account to extract the items that may be applicable to this facility.

- Water Control Function : To prevent floods from the Manggahan Floodway
- Flowing Garbage and Water Plants : There are many garbage and water plants such as water lily and branches, and they tend to accumulate.
- Resistance to Local Climate Condition : High temperature and humidity compared to Japan, and solar radiation heat all year round

- Availability of Land : The site is limited on both sides and the facility is as compact as possible

The following three alternatives are extracted as objects for comparison and examination of gate types.

Alternative 1 : Fixed Wheel Roller Gate -> Adopted
Alternative 3 : Radial Gate -> There is a weak point against overflow, it is not possible to lower by its own weight.
Alternative 4 : Rising Sector Gate -> low economic efficiency

Based on the comparison, Alternative 1 (Fixed Wheel Roller Gate) is selected. In addition to the ease of maintenance, reliability and economy, the knowledge of local technicians on their operation and maintenance was also taken into consideration since the same type was adopted for the nearby weir and Floodgate. A detailed comparative study is presented in **Table 6.4.14 of Main Report**.

(2) Span and Span Allocation

In setting Span and Span Allocation, the following two alternatives are extracted with reference to "Cabinet Order Concerning Structural Standards for River Administration Facilities, etc. (hereinafter so called "Structural Cabinet Order")".

Alternative 1: 1 span (38.0 m) -> The risk of adverse effects on the drainage of the branch river due to gate failure remains.
Alternative 2: 2 spans (19.0 m + 19.0 m) -> Adopted

From the above 2 plans, " Alternative 2: 2 spans (19.0 m + 19.0 m)" is adopted due to its high operational reliability.

(3) Study on the Maintenance Bridge

The specifications of the management bridge are summarized in **Table 6.4.4**.

Table 6.4.4 Summary of Cainta Floodgate Maintenance Bridge Specifications

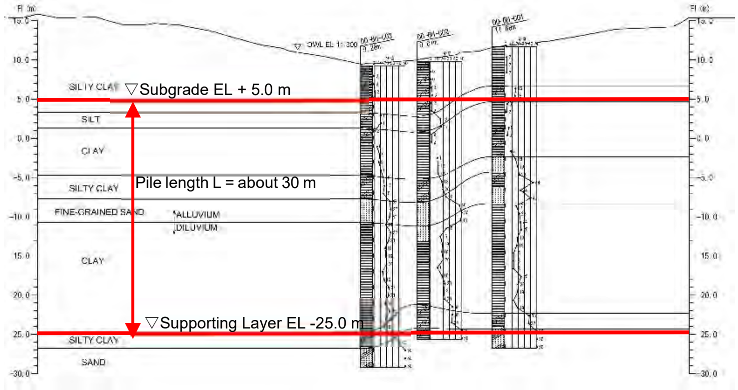
	Item		Conditions/specifications, etc.
1	Road Condition		· Common road (Replacement of existing roads) · 'Urban Road', Design Speed 40 km/h;
2-1	Bridge Condition	Bridge Length	L = 18.50 m
2-2		Width Composition	W = 1.200 m (Sidewalk) + 0.600 m (Shoulder) + 3.350 m * 2 (Roadway) + 0.600 m (Shoulder) + 1.200 m (Sidewalk) = 17.000 m
2-3		Paving	Asphalt Pavement (ACP), t = 50 mm
3	Loading Condition		· Set dead load, live load, fatigue load, and impact load according to DGCS · The maximum design wind speed is set at V = 200 km/h in consideration of consistency with the weir body.
4	Superstructure Type		· RC Floor Slab Bridge ✓ Span Length (13.0 m to 20.0 m) ✓ Comparison of Economic Efficiency with PC I Girder Bridge and Steel I Girder Bridge

Source: Study Team

(4) Study on Foundation Type

1) Supporting Layer

The upper surface of the supporting layer is a SILTY-CLAY layer that a soil layer of N value of 30 or more continues more than 5m. Since the top surface height of the SILTY-CLAY layer differs by up to 4m at each borehole, the support layer is set around the EL -25 m of the deepest DO-BH-C03 hole in this design. Since the elevation of the bottom surface of the floor slab of the main body is EL.+5.75 m, the pile length will be 30m or longer.



Source: Study Team

Figure 6.4.2 Geological Map

2) Type of Foundation

Based on the application examples and interviews with construction companies, the following combinations of pile materials and driving methods were extracted. In this design, the steel pipe pile/driving pile method (vibro-hammer method) is recommended. A comparison of pile materials is shown in **Table 6.4.18 of Main Report**.

- ✓ The impact on the surrounding houses is concerned, because the driving method is used for RC piles.
- ✓ The applicability of cast-in-place piles is low because the groundwater level is assumed to be high at the time of pile driving. After all, the construction by the half river cofferdam will be constructed at this site.

(5) Setting the Specifications of the Main Body of the Floodgate (Section Dimensions)

Dimensions of the main body are set referring to "Technical Standards for River Sand Control (Draft) Design Part I" and ""Design of Weir " (Dam Engineering Center). Here, the set parameters are Organize in **Table 6.4.1**. Details of the specifications of each main body are shown in the **Sub-section 6.4.3.2 (11) of Main Report**.

6.4.3.3 Study on Gate Structure and Hoist

The gate structure and hoist of Cainta Floodgate is determined as shown in **Table 6.3.4**.

Table 6.4.5 Summary of Gate Structure and Hoist of Cainta Floodgate

Item	Specification	Verification
Gate Leaf Structure	Plate Girder Structure	· Since the ratio of the door height to the net diameter is about 1/2.188, the gate is located in the girder structure region according to the gate size and structure related diagram shown in the Technical Specification for Dams and Weirs in Japan (Draft). · It is also more economical than the shell structure (A detailed comparative study is presented in **Table 6.4.27 of Main Report.**).

Item	Specification	Verification
Gate Material	Alloy Saving Duplex Stainless Steel (SUS 821 L1)	· Freshwater Environment · Lifecycle Cost LCC is the lowest. ✓ SM 400 ***** PHP (1.00) ✓ SUS 304 ***** PHP (1.03) ✓ SUS 821 L1 ***** PHP (0.95)
Type of Hoist	Wire Rope Winch Type	· There are a lot of cases in the Philippines. · The structure is simple and easy to maintain. · It is also economical. · No need for connecting bridges
	1 Motor 1 Drum Type (1M1D)	· It does not require an electric shaft, and is advantageous in terms of installation and economical efficiency.

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

6.4.3.4 System Planning

(1) Power Unit

For the power unit, policies and specifications would be same as those for MCGS (see **Table 6.3.5**).

(2) Control System

As well as MCGS, this system is designed to perform machine side operation and remote monitoring control. The control circuit and panel configuration are the same as those of MCGS. Operational monitoring items shall be based on the facility concerned and detailed in **Sub-section 6.3.5.2 (2) of Main Report**. The system level and system configuration in the remote monitoring control are the same as MCGS.

6.4.4 Basic Design of Taytay Sluiceway

6.4.4.1 Study on Layout Location

At the location of the proposed Taytay Sluiceway, there is an existing dike, and although there is a conduit that serves as sluiceway, no gate is installed. As in the case of Cainta Floodgate, in order to prevent the new dike from reduce the flow area of Manggahan Floodway, it is a policy not to place a new dike in the river side of the existing dike.

Alternatives	Contents
Alternative1: Inside of the dike from the existing dike	· The East Bank Road will cross the waterway outside the dike, and a bridge will be required. · The current East Bank Road area needs to be excavated for construction, so it is necessary to move houses that should not be moved.
Alternative 2: existing dike position	· It is necessary to cut the East Bank Road during construction.

In the case of “Alternative 1: Land side of the Existing Dike Alignment”, a new bridge is required to cross the connecting water channel from Taytay Creek, and houses in the landside of the east bank need to be relocated. As a result, the number of houses to be relocated will increase which may affect the progress of the project, and construction costs will also increase. There is no advantage in choosing Alternative 1.

Accordingly, “Alternative 2: Same location as the Existing Dike”, is recommended.

The position where the center line of the Floodgate is approximately located on the center of the existing box culvert is the base and centerline of the Floodgate aligned with Sta.+090 of the Manggahan Floodway.

6.4.4.2 Type of Structure

Elevation: EL.+10.6m+height of gate: 1.8m), the top elevation of the existing dike is EL.+17.0m. When the height of the gate is the same as that of the dike, the weight of the gate becomes heavier and the capacity of its hoist and thickness of the concrete structure also become large. Hence, it is better to install a curtain wall in the case of Cainta Floodgate.

Although the height of the curtain wall becomes more than 3 m, the height of the gate is only 1.8 m. In such a case, since there is enough thickness of dike at the top of box culvert, the sluiceway type can also be considered.

In view of the above situation, we will compare the sluice gate type with the sluice gate type.

Alternative 1: Floodgate Type (Connecting with the Existing One)	->	Poor structural, maintainability and workability
Alternative 2-a: Sluiceway Type (with Total Rehabilitation)	->	The number of relocated houses is large and the social environment is inferior
Alternative 2-b: Sluiceway Type (Connecting with the Existing One)	->	Adopted

From the aspect of structure, maintainability and construction, the sluiceway type is chosen. Additionally, "Alternative 2-b: Sluiceway Type (Connecting with the Existing One)" is recommended to minimize the number of houses to be relocated in the project. The detailed comparative study is presented in **Table 6.4.48 of Main Report**.

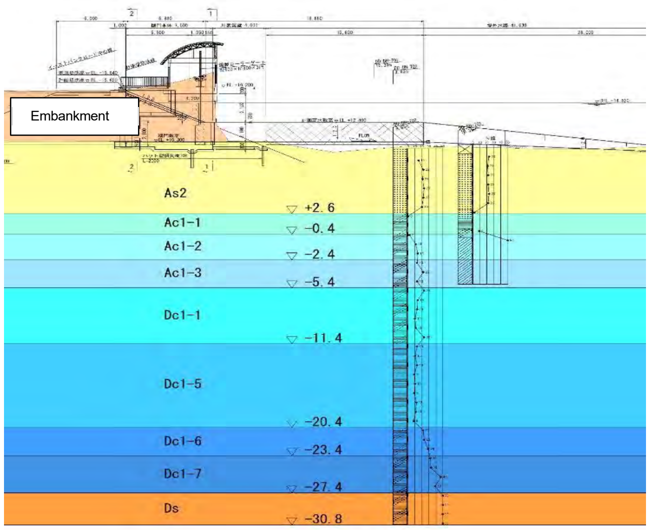
6.4.4.3 Study on Basic Structural Specifications

(1) Invert Elevation and Size of Culvert

The invert elevation of the extension sluiceway is set using the survey result of the existing box culvert. The invert elevation of the existing box culvert at the joint is EL+10.387m. Thus, the height of the connecting sluiceway is set to EL+10.3 m. The size of the box culvert would be the same as that of the existing box culvert and 2.5 m x 1.8 m x 3 barrels.

(2) Box Structure Type

The foundation ground of the Taytay Floodway site is assumed to be a cohesive soil layer, although there is no thick soft layer(See **Figure 6.4.3**). Therefore, residual settlement of 5 cm or more can be expected. This design adopts a flexible type that can follow the settlement of the ground or foundation without using piles, etc., and it does not adversely affect the dike.



Source: Study Team

Figure 6.4.3 Assumed Geological Section of Taytay Sluiceway Gate (Sluiceway Profile)

In Chapter 7, the residual settlement is calculated, and the result is 7.6 cm. Hence, the validity to adopt the flexible type has been confirmed.

(3) Type of Gate

Flap gates are not recommended considering the size of the gate of the Taytay Sluiceway and the social impact of inundation in the land side of dike caused by backflow through the Taytay Creek due to the incomplete closure of gates. Hence, comparative study shall be conducted among the 3 types below that are superior to a flap gate in terms of reliability of closure as follows:

- Fixed Wheel Roller Gate -> Adopted
- Hydraulic Operated Link Mechanism Gate (without Column) -> Less economical
- Flap gate with Balanced Weight (without Column) -> Not suitable for a location with a lot of driftage

Based on the local site condition, fixed wheel roller gate is least affected by drifting objects and is sufficiently reliable. A detailed comparative study is presented in **Table 6.4.49 of Main Report**.

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

Dimensions of the main body are set referring to "Technical Standards for River Sand Control (Draft) Design Part I" and "Guideline for Flexible Sluiceway". Here, the set parameters are Organize in **Table 6.4.2**. Details of the specifications of each main body are shown in the **Sub-section 6.4.4.9 of Main Report**.

6.4.4.4 Study on Gate Structure and Hoist

The gate structure and hoist of Taytay Sluiceway is determined as shown in **Table 6.4.5**.

Table 6.4.6 Summary of Gate Structure and Hoist of Taytay Sluiceway

Item	Specification	Verification
Gate Material	Alloy Saving Duplex Stainless Steel (SUS 821 L1)	· Freshwater Environment · Lifecycle Cost LCC is almost the same. ✓ SM 400 ***** PHP (1.00) ✓ SUS 304 ***** PHP (1.02) ✓ SUS 821 L1 ***** PHP (1.02) · Less maintenance work compared with rolled steel (SM 400) for welded structure
Type of Hoist	Double Pin Rack Type	· It has a lot of good cases in Japan · This type is equipped with a self-weight lowering function as standard equipment, and can be lowered by remote control. · Higher mechanical efficiency and smaller motor capacity than spindle type · Standardized for ease of maintenance

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

6.4.4.5 System Planning

(1) Power Unit

Power Unit is summarized in **Table 6.4.7**

Table 6.4.7 Summary of Power Unit (Taytay Sluiceway)

Item	Specification	Remarks
Main Power Unit	Electric Motor	Same as MCGS
Reserve Power Unit	Manpower	The small gate (Approximately less than 10 m^2 of door area) can be opened and closed manually.
Power Supply Unit		
a Main Power Supply Unit	The commercial power supply is directly received by the gate machine side panel in the operation room at 3 ϕ 3W AC 200 V 60 Hz and 1 ϕ 2W AC 200 V 60 Hz to be used as the power and operation power supply.	Same as MCGS
b Standby Power Supply System	Permanent standby power generation equipment	Same as MCGS

Source: Study Team

(2) Control System

As same as MCGS, this system is designed to perform machine side operation and remote monitoring control. The control circuit and system configuration are the same as those of MCGS. Operational monitoring items shall be based on the facility concerned and detailed in **6.3.5.2 (2) of Main Report**. The system level and system configuration in the remote monitoring control are same as MCGS.

CHAPTER 7 DETAILED DESIGN OF RIVER STRUCTURES

7.1 Detailed Design of River Channel (Dikes, Revetments, and Revetment for Low Water Channel)

7.1.1 Detailed Design of SSP Revetment for Low Water Channel

7.1.1.1 Design Section

(1) List of Design Conditions

The design conditions of the steel sheet pile revetment are as shown in **Table 7.1.1**.

Table 7.1.1 Design Condition of SSP Revetment

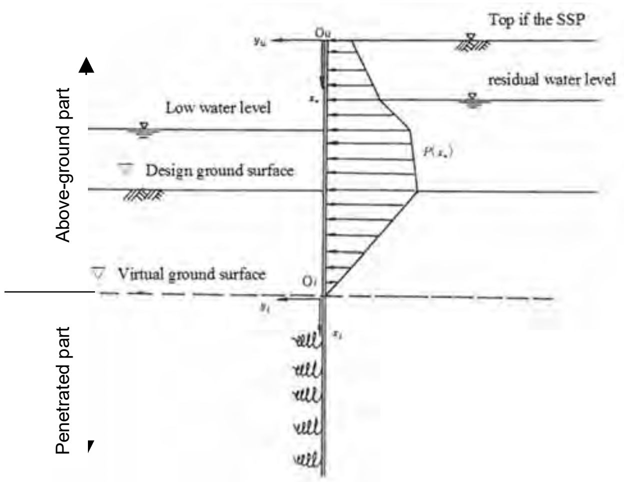
Items				Design Condition	
Soil Condition	Ground			Refer to "Geological conditions to be applied"	
Soil Condition	Back Fill	Soil		Loose Sandy Soil	
		Unit Weight		19 kN/m3	
		N-value		15	
		Internal friction angle ϕ		$= 15 + \sqrt{15N} \leqq 45°$, where $N \geqq 5$	
		Adhesive force C		C = 0	
Steel Sheet Pile	Steel grade			SYW 295	
	Allowable bending stress			Regular: 180 N/mm^2 / earthquake: 270 N/mm^2	
	Allowable displacement			Regular: 50 mm / earthquake: 75 mm	
	Effective rate of cross-sectional performance	Hat type		Moment of inertia of area	100%
				Section modulus	100%
		U-Type		Moment of inertia of area	100%
				Section modulus	80%
	Young modulus			2.0 x 105 N/mm^2	
	Corrosion allowance			1mm for both sides (total 2mm)	
	Elevation of concrete coping crown			Equivalent to the DFL	
Water Level Condition				[Back water level] Background elevation ≧ top concrete crown height Back water level: t top concrete crown height Background elevation < top concrete crown height Back water level: background elevation	
				[Front water level] Normal: drought water level Low water level at the time of earthquake	
Horizontal Seismic Coefficient				k = 0.2 (both in air and water)	
Surcharge				Normal: 10kN/m^2 , seismic: 5kN/m^2	

Source: Study Team

(2) Design Methodology

SSP is designed as a cantilever supported by soil below an imaginary riverbed.

SSP revetment supports lateral load such as earth pressure by lateral support force of the intruded ground and flexural rigidity of the wall. The target external forces are lateral load (horizontal load) such as lateral soil pressure and water pressure. Vertical load will not be considered. Therefore, penetration of SSP is not regarded as a friction pile or support pile, but it shall be designed to withstand lateral load (Calculated as = 3 / β).



Source: Design Guidelines of Disaster Recovery Works Part3 Reference Cahper1 2-8 SSP Revetments

Figure 7.1.1 Location of the virtual ground plane

The Chang's equation will be applied for the structural calculation. In Chang's equation, when the SSP is divided into the upper part and the lower part with the virtual riverbed (A position where the sum of the active earth pressure strength and the residual hydraulic pressure equals the passive earth pressure strength) as the boundary as shown in **Figure 7.1.1**, it is assumed that the ground reaction force of the penetrated part is linear with respect to the displacement amount.

7.1.1.2 Result of Calculation

The SSP specifications and length determined by the structural calculation of SSP are shown in **Table 7.1.2** and **Table 7.1.3**.

Table 7.1.2 List of Specifications of Steel Sheet Piles For Bank Protection(1/2)

Bank	Block No.	Sta.			Length in Longidinal direction	Specification					Length			RIPRAP (SIDE SLOPE: 1.5 :1)		
		From	-	To		Combined					Total	SSP	H-beam	FINISHED HEIGHT FROM DRB	FINISHED TOP WIDTH	CLASS
						No.	Name	Unit Mass	Section Modulus	Moment of Inertia						
					(m)			(kg/m2)	(cm3/m)	(cm4/m)	(m)	(m)	(m)	(m)	(m)	
Left Bank	L-1	6+035.3	-	6+080	42.80	18	10H-750x250x14x19	268	7,090	390,100	15.0	13.0	14.5	-	-	NO RIPRAP (W/ RIVERBED PROTECTION)
	L-2	6+080	-	6+362.8	266.29	10	10H-700x200x12x16	224	5,130	281,600	15.5	13.5	15.0	1.5	3.5	A
	L-3	6+753	-	6+800	61.44	7	10H-550x200x12x16	207	3,900	177,600	15.0	13.0	14.5	1.5	3.5	D
	L-4	6+800	-	7+180	409.61	22	10H-850x250x14x22	293	8,840	532,300	20.5	18.5	20.0	3.5	3.5	D
	L-5	7+180	-	7+480	302.83	27	10H-1000x250x16x25	340	11,980	806,700	20.5	18.5	20.0	3.5	3.5	UP TO 7+300 : D, FROM 7+300: B
	L-6	7+480	-	7+820	359.77	27	10H-1000x250x16x25	340	11,980	806,700	22.5	20.5	22.0	3.5	3.5	B
	L-7	7+820	-	7+940	120.00	28	10H-1000x250x16x28	352	12,670	845,700	22.0	20.0	21.5	2.5	2.5	B
	L-8	7+940	-	8+120	180.00	29	25H-1000x250x16x22	358	12,240	892,400	22.0	20.0	21.5	2.5	2.5	B
	L-9	8+120	-	8+300	180.11	24	10H-900x250x14x25	311	10,120	631,400	19.0	17.0	18.5	1.5	3.5	B
	L-10	8+300	-	8+600	293.86	21	10H-800x250x14x22	287	8,240	471,700	16.5	14.5	16.0	1.5	3.5	B
	L-11	8+600	-	8+835	211.24	28	10H-1000x250x16x28	352	12,670	845,700	20.0	18.0	19.5	1.5	3.5	UP TO 8+700 : B, FROM 8+700: C
	L-12	9+205	-	9+320	107.58	11	10H-700x200x12x19	234	5,540	299,700	15.0	13.0	14.5	2.5	2.5	C
	L-13	9+320	-	9+560	230.49	51	50H-1000x300x19x40	550	21,110	1,543,500	23.0	21.0	22.5	3.5	3.5	C
	L-14	9+560	-	9+800	267.21	25	10H-950x250x16x22	321	10,570	688,700	18.0	16.0	17.5	1.5	3.5	C
	L-15	9+800	-	9+900	100.98	31	25H-1000x250x16x25	370	12,980	935,900	21.0	19.0	20.5	2.5	5.0	B
	L-16	9+900	-	10+020	120.99	19	10H-750x250x14x22	280	7,640	414,700	16.0	14.0	15.5	1.5	3.5	B
	L-17	10+020	-	10+360	332.08	22	10H-850x250x14x22	293	8,840	532,300	19.5	17.5	19.0	1.5	3.5	B
	L-18	10+360	-	10+520	160.22	23	10H-900x250x14x22	299	9,470	597,600	18.0	16.0	17.5	1.5	3.5	B
	L-19	10+520	-	10+580	60.10	19	10H-750x250x14x22	280	7,640	414,700	15.0	13.0	14.5	1.5	3.5	B
	L-20	10+580	-	10+640	60.00	14	10H-700x250x12x19	250	6,270	331,300	13.5	11.5	13.0	1.5	3.5	B
	L-21	10+640	-	10+760	86.91	23	10H-900x250x14x22	299	9,470	597,600	17.5	15.5	17.0	1.5	3.5	B
	L-22	10+760	-	11+040	230.40	22	10H-850x250x14x22	293	8,840	532,300	17.5	15.5	17.0	1.5	3.5	UP TO 11+000 : B, FROM 11+000: A
	L-23	11+040	-	11+180	140.19	1	10H-400x200x9x12	169	2,320	87,800	12.0	10.0	11.5	1.5	3.5	A
	L-24	11+180	-	11+460	282.50	23	10H-900x250x14x22	299	9,470	597,600	20.5	18.5	20.0	1.5	3.5	A
	L-25	11+460	-	11+640	182.49	21	10H-800x250x14x22	287	8,240	471,700	20.5	18.5	20.0	1.5	3.5	A
	L-26	11+640	-	11+800	164.63	8	10H-600x200x12x16	212	4,270	208,200	15.5	13.5	15.0	1.5	3.5	A
	L-27	11+800	-	12+040	266.26	19	10H-750x250x14x22	280	7,640	414,700	17.5	15.5	17.0	1.5	3.5	C
	L-28	12+040	-	12+280	260.87	8	10H-600x200x12x16	212	4,270	208,200	15.0	13.0	14.5	1.5	3.5	UP TO 12+200 : C, FROM 12+200: A
	L-29	12+280	-	12+520	249.09	2	10H-400x200x9x16	182	2,700	98,700	13.0	11.0	12.5	1.5	3.5	A
	L-30	12+520	-	12+820	304.65	1	10H-400x200x9x12	169	2,320	87,800	12.0	10.0	11.5	1.5	3.5	A
	L-31	12+820	-	13+000	179.78	6	10H-500x200x12x16	202	3,540	149,900	14.0	12.0	13.5	1.5	3.5	A
	L-32	13+000	-	13+320	322.81		SP-50H	186	2,760	51,100	12.5	12.5	0.0	1.5	3.5	A
	L-33	13+320	-	13+360	31.22	4	10H-450x200x9x19	196	3,280	128,300	15.0	13.0	14.5	1.5	3.5	A
				Sub Total	**6,569.40**											

Source: Study Team

Table 7.1.3 List of Specifications of Steel Sheet Piles For Bank Protection(2/2)

Bank	Block No.	Sta.			Length in Longidinal direction	Specification					Length			RIPRAP (SIDE SLOPE: 1.5 :1)		
		From	-	To		Combined					Total	SSP	H-beam	FINISHED HEIGHT FROM DRB	FINISHED TOP WIDTH	CLASS
						No.	Name	Unit Mass	Section Modulus	Moment of Inertia						
					(m)			(kg/m2)	(cm3/m)	(cm4/m)	(m)	(m)	(m)	(m)	(m)	
Right Bank	R-1	5+423	-	5+540	120.61	10	10H-700x200x12x16	224	5,130	281,600	14.5	12.5	14	-	-	NO RIPRAP
	R-2	5+540	-	5+581.25	49.58	23	10H-900x250x14x22	299	9,470	597,600	17.0	15	16.5	-	-	NO RIPRAP
	R-3	5+624	-	5+720	125.58		SP-45H	163	2,450	45,000	10.0	10	0	1.5	3.5	A
	R-4	5+720	-	5+905.80	201.47		SP-45H	163	2,450	45,000	10.0	10	0	1.5	3.5	A
	R-5	6+035.3	-	6+080	44.51	26	10H-1000x250x16x22	328	11,270	766,300	17.5	15.5	17	-	-	NO RIPRAP (W/ RIVERBED PROTECTION)
	R-6	6+080	-	6+280	211.53	36	25H-1000x300x16x28	407	15,180	1,064,500	20.0	18	19.5	1.5	3.5	A
	R-7	6+280	-	6+420	140.28	25	10H-950x250x16x22	321	10,570	688,700	18.0	16	17.5	1.5	3.5	A
	R-8	6+420	-	6+920	437.46	32	25H-1000x300x16x22	377	13,420	962,300	23.5	21.5	23	2.5	2.5	UP TO 6+650 : A, FROM 6+650: B
	R-9	6+920	-	7+220	295.15	30	10H-1000x250x16x32	369	13,590	896,800	23.0	21	22.5	2.5	UP TO 7+160 : 2.5, FROM 7+160 : 3.0	B
	R-10	7+220	-	7+620	387.86	38	25H-1000x300x16x32	426	16,330	1,129,800	23.5	21.5	23	3.5	3.5	B
	R-11	7+620	-	7+900	272.03	10	10H-700x200x12x16	224	5,130	281,600	17.5	15.5	17	3.5	3.5	B
	R-12	7+900	-	8+240	340.06	30	10H-1000x250x16x32	369	13,590	896,800	22.0	20	21.5	2.5	2.5	B
	R-13	8+240	-	8+500	260.19	25	10H-950x250x16x22	321	10,570	688,700	19.5	17.5	19	2.5	2.5	B
	R-14	8+500	-	8+620	124.41	25	10H-950x250x16x22	321	10,570	688,700	17.5	15.5	17	1.5	3.5	B
	R-15	8+620	-	8+940	340.55	29	25H-1000x250x16x22	358	12,240	892,400	20.0	18	19.5	3.5	3.5	B
	R-16	8+940	-	9+000	65.45	8	10H-600x200x12x16	212	4,270	208,200	15.0	13	14.5	1.5	3.5	B
	R-17	9+000	-	9+200	217.04	1	10H-400x200x9x12	169	2,320	87,800	12.0	10	11.5	1.5	3.5	B
	R-18	9+200	-	9+380	196.62	38	25H-1000x300x16x32	426	16,330	1,129,800	21.5	19.5	21	3.5	5	B
	R-19	9+380	-	9+700	300.75	32	25H-1000x300x16x22	377	13,420	962,300	22.0	20	21.5	1.5	3.5	B
	R-20	9+700	-	9+900	182.87	11	10H-700x200x12x19	234	5,540	299,700	16.5	14.5	16	1.5	3.5	B
	R-21	9+900	-	10+380	493.71	24	10H-900x250x14x25	311	10,120	631,400	21.5	19.5	21	1.5	3.5	B
	R-22	10+380	-	10+520	140.72	25	10H-950x250x16x22	321	10,570	688,700	19.5	17.5	19	1.5	3.5	B
	R-23	10+520	-	10+540	20.67	8	10H-600x200x12x16	212	4,270	208,200	13.5	11.5	13	1.5	3.5	B
	R-24	10+540	-	10+660	122.88	7	10H-550x200x12x16	207	3,900	177,600	11.5	9.5	11	1.5	3.5	B
	R-25	10+660	-	10+760	125.77	7	10H-550x200x12x16	207	3,900	177,600	11.5	9.5	11	1.5	5.5	B
	R-26	10+760	-	10+820	79.86	1	10H-400x200x9x12	169	2,320	87,800	13.0	11	12.5	1.5	5.5	B
	R-27	10+820	-	10+980	205.01	29	25H-1000x250x16x22	358	12,240	892,400	19.5	17.5	19	1.5	5.5	B
	R-28	10+980	-	11+200	220.61	23	10H-900x250x14x22	299	9,470	597,600	19.0	17	18.5	1.5	3.5	UP TO 11+000 : B FROM 11+000: A
	R-29	11+200	-	11+360	160.68	1	10H-400x200x9x12	169	2,320	87,800	12.5	10.5	12	1.5	3.5	A
	R-30	11+360	-	11+700	328.31	1	10H-400x200x9x12	169	2,320	87,800	14.0	12	13.5	1.5	3.5	A
	R-31	11+700	-	11+980	214.88	27	10H-1000x250x16x25	340	11,980	806,700	21.5	19.5	21	1.5	3.5	A
	R-32	11+980	-	12+000	16.30		SP-50H	186	2,760	51,100	11.0	11	0	1.5	3.5	A
	R-33	12+000	-	12+240	198.95	1	10H-400x200x9x12	169	2,320	87,800	13.0	11	12.5	1.5	3.5	A
	R-34	12+240	-	12+520	265.59	1	10H-400x200x9x12	169	2,320	87,800	12.5	10.5	12	1.5	3.5	A
	R-35	12+520	-	12+540	18.97		SP-25H	126	1,610	24,400	9.0	9	0	1.5	3.5	A
	R-36	12+540	-	12+660	116.78	21	10H-800x250x14x22	287	8,240	471,700	18.5	16.5	18	1.5	3.5	A
	R-37	12+660	-	12+740	80.13	12	10H-700x250x12x16	238	5,740	308,300	16.0	14	15.5	1.5	3.5	A
	R-38	12+740	-	12+980	240.44	15	10H-700x200x12x25	254	6,370	334,800	17.0	15	16.5	1.5	3.5	A
	R-39	12+980	-	13+100	120.17	22	10H-850x250x14x22	293	8,840	532,300	21.5	19.5	21	1.5	3.5	A
	R-40	13+100	-	13+220	118.37	10	10H-700x200x12x16	224	5,130	281,600	18.0	16	17.5	1.5	3.5	A
	R-41	13+220	-	13+375	146.11	24	10H-900x250x14x25	311	10,120	631,400	23.0	21	22.5	2.5	5	A
				Sub Total	**7,748.9**											
				Total	**14,318.3**											

Source: Study Team

7.1.2 Detailed Design of Revetment for Dike

Structural calculations on inclined walls and parapet walls is performed and required cross-sectional dimensions and reinforcement dimensions are determined. The details of the study are stated in the **Sub-section 7.1.2 of Main Report**.

7.1.3 Study on the Material for Embankment and Backfill

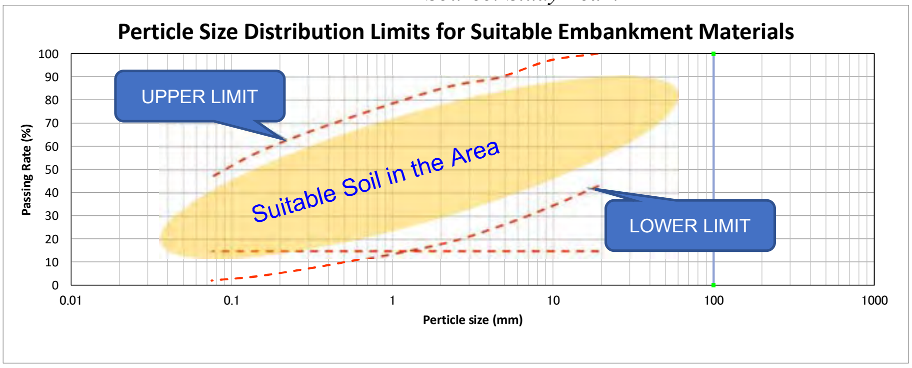
According to "River Earthworks Manual, Japan," the suitable distribution of the particle size for embankment material shown in **Figure 7.1.2** is located in the area surrounded by the red dashed line.

Table 7.1.4 Ratio for Purchased Soil

Package	Ration of Purchased and Mixed Soi
CP -1	30%
CP -2	30%
CP -3	10%

Source: Study Team

On the other hand, the particle size distribution of the soil generated from the excavation work and dredging work exceeds the upper limit line. Therefore, in each package, the embankment material would be prepared from the excavated material with purchasing and mixing 20 ~ 40 mm of gravel (gravel) at the ratio shown in **Table 7.1.4**, to become an appropriate material for embankment.



Source: Study Team

Figure 7.1.2 Particle Size Distribution for Embankment Material

7.1.4 Design of Embankment and Upper Slope of Revetment

7.1.4.1 Normal Condition and Seismic Condition

In this section, stability analysis is carried out at the embankment and slope of the revetment under normal condition and level 1 seismic condition. The analysis was based on the slope stability analysis by the modified Fellenius method described in **Sub-section 11.5. 3 in Main Report**. The seismic coefficient on the horizontal direction is 0.20 (11. 11.4.2.2). As a result of the analysis, when the safety factor is insufficient for the required value, a retaining wall (H = 1.0 m) and a sheet pile foundation (L = 2.0 m) are arranged as countermeasures.

7.1.4.2 During Flood（Slip by Infiltration）

In this section, slip safety by infiltration against the designed flood is examined. The analysis used the modified Ferrenius method, described in **Sub-section 11.5. 3 in Main Report**. As for the target section, all the results satisfy the required safety factor, and no countermeasure is required.

7.2 Detailed Design of Drainage Facilities

7.2.1 Summary

The structure of proposed drainage outlet basically depends on type of river structure and type of existing outlet. The structure of proposed drainage outlet classifies two (2) type of structure, “Outlet” type and “Sluiceway” type. For both types, the manhole will be installed for the connection of existing outlet and new proposed outlet and for maintenance. And also, the collector pipe for minimizing the number of proposed outlets will be connected to manhole. In case that at location of proposed outlet, the existing ground elevation of land side is lower than the design flood water level, in order to prevent the back flow from river, the flap gate would be installed.

7.2.2 Detailed Design of Outlet

7.2.2.1 Summary of Proposed Outlets

The summary of new proposed outlets is tabulated in **Table 7.2.1 of Main Report**. The size of proposed outlet is set in accordance with the basic design conditions and policies described above.

7.2.2.2 Detailed Design of Drainage Outlet Facility

(1) Drain Pipe (Outlet)

The proposed outlet size must have sufficient capacity for the design discharge. Minimum size of drainage pipes would be adopted diameter of 900 mm in accordance with Guideline.

However, in the case of roof and sanitary flows, for new outlet, same as existing size diameter is adopted as the discharge is too small. And also, for collector pipe, it would be adopt the suitable size for design discharge. The structure of outlet is reinforced concrete structure with integrated headwall and coping as well as the outlet of PMRCIP Phase 2 and Phase 3 Project.

(2) Manhole

The manholes will be installed to connect the existing drains to the new outlet and to maintain the facilities. For manholes deeper than 1.0m, a step iron will be installed for entry and exit.

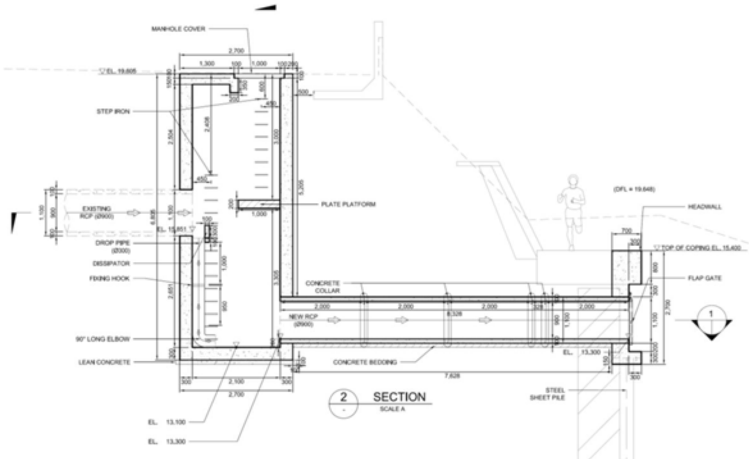
In case that the manhole becomes deeper, a platform will be installed in consideration of the safety aspects of maintenance. In this project, in case that the depth of manhole is more than 5 m, every 3 m the platform will be set up in the manhole.

Moreover, in case that the height difference between the incoming pipe (existing drain) and the outgoing pie (new outlet pipe) is more than 1.2 m, drop pipes will be installed. The drop pipe facilitates make inspection and cleaning work inside the manhole easier, and also has a role to prevent abrasion of the manhole bottom and side walls due to discharging water.

The thickness of a member of each size would be determined by performing structural calculation of the member and stability calculation against uplift force and bearing force. particular structural calculation conditions, methods and results are summarized in the **Sub-section 7.2.2.2 (2) of Main**

Report.

A general drawing of a new drain pipe (Outlet) having the structures described in the above (1) and (2) is shown in **Figure 7.2.1**.



Source: Study Team

Figure 7.2.1 General Drawing of Newly Installed Drain Pipe

7.2.3 Detailed Design of Sluiceways

7.2.3.1 Categorizing of Calculation Type

Sluiceways designed in this project can be categorized into several types of calculation models based on similarity in proportion and soil condition. Following tables explain categorization of all sluiceways.

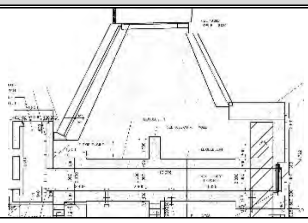
Table 7.2.1 Grouping of Sluiceway and Selection of Calculation Model Type

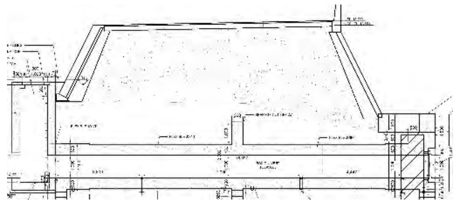
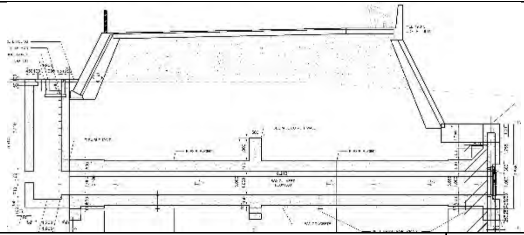
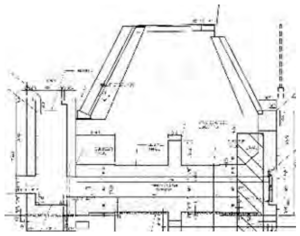
LOCATION			NAME OF SLUICEWAY	Opening (mm)	Length (m)	DFL (m)	EL (m)	GL at Center (m)	DFL-GL at Center (m)	DFL-EL	DFL-Box Top	Thickness of Soil Layers			Structural Type
												As	Ac	Dc	
LEFT BANK	CP2	7+223	ML097A	1000.00	16.280	17.721	10.300	14.351	3.370	7.421	5.971			6.88	TYPE-A
		7+801	ML087A	1000.00	11.007	18.073	10.400	13.993	4.080	7.673	6.223			16.28	TYPE-B
		7+780	ML087B	1000.00	10.777	18.085	10.400	14.525	3.560	7.685	6.235	0.10		14.62	TYPE-B
		7+801	ML091.1	1200.00	10.933	18.000	10.200	13.760	4.240	7.800	6.150	0.25		11.95	TYPE-H
		7+828	ML087.2	1000.00	11.126	18.061	10.400	13.661	4.400	7.661	6.211	0.46		11.41	TYPE-B
		7+856	ML087.1	1000.00	10.146	18.109	11.600	14.819	3.290	6.509	5.059	1.60		10.90	TYPE-B
		7+880	ML087C	1000.00	9.877	18.121	11.400	14.921	3.200	6.721	5.271	1.29		10.53	TYPE-B
		7+980	ML087D	1000.00	9.560	18.182	12.000	14.462	3.720	6.182	4.732	1.11		9.25	TYPE-B
		9+220	ML068A	1000.00	20.189	18.933	10.300	15.996	2.937	8.633	7.183		4.90	1.58	TYPE-C
	CP3	9+320	ML068B	1000.00	17.697	18.659	10.300	15.254	3.405	8.359	6.909		4.99	2.41	TYPE-D
		9+420	ML068C	1000.00	17.477	19.054	10.300	16.954	2.100	8.754	7.304		6.84	0.30	TYPE-D
		9+520	ML068D	1000.00	19.513	19.115	10.400	17.000	2.115	8.715	7.265		1.59	4.77	TYPE-E
		9+527	ML068	1000.00	19.513	19.115	10.400	17.000	2.115	8.715	7.265		1.56	4.79	TYPE-E
RIGHT BANK	CP1	6+060	MR125A	1000.00	17.700	17.400	11.700	12.700	4.700	5.700	4.250			0.26	TYPE-J
	CP2	7+203	MR074	1000.00	9.741	17.709	11.800	14.598	3.111	5.909	4.459	2.04		9.34	TYPE-B
		7+357	MR073	1000.00	7.726	17.806	10.967	15.086	2.720	6.839	5.389	1.35		10.43	TYPE-F
		7+399	MR072.1	1000.00	7.472	17.830	11.363	15.590	2.240	6.467	5.017	1.62		10.88	TYPE-F
		7+509	MR072	1000.00	6.973	17.891	11.402	16.621	1.270	6.489	5.039	0.96		11.25	TYPE-F
		7+840	MR067A	1200.00	10.162	18.097	10.200	14.497	3.600	7.897	6.247			7.81	TYPE-H
	CP3	10+936	MR009A	1000.00	18.375	19.956	10.900	15.576	4.380	9.056	7.606			16.82	TYPE-G
		10+972	MR008	1200.00	19.651	19.966	10.700	14.966	5.000	9.266	7.616			16.51	TYPE-I

Source: Study Team

The selection criteria for each type are described in the table below, together with a cross-sectional view of a typical gutter. Here, only Type-B, D, E, and F are shown. All other types are summarized in **Table 7.2.12 of Main Report**.

Table 7.2.2 Typical Model and Description of Each Type

Structural Type		Description
Type B		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] ✓ Both side of Levee slopes concrete-protected and steep, thus sluiceway is relatively short as 9.9m -10.4m. Elevation of existing ground is relatively low which makes the height of embankment greater. Therefore, greater subsidence due to soil consolidation is expected in this type.

Structural Type	Description	
Type D		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Length of sluiceway is around 17.5m since levee crown is as wide as 10m.
Type E		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Length of sluiceway is around 19.5m since levee crown is as wide as 12m.
Type F		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Sluiceway is the shortest among all types as 7.0m – 7.7m, since coping concrete of SSP retaining wall is narrower than other types in addition to that the both side of Levee slopes are concrete-protected and steep.

Source: Study Team

7.2.3.2 Examination of Settlement and Displacement of Foundation Ground

(1) Residual Settlement of Foundation Ground

Table 7.2.3 Calculation Results of Residual Settlement

Structural Type of Sluiceway	Immediate Settlement (mm)	Consolidation Settlement (mm)	Camber Bedding (mm)	Residual Settlement (mm)		Allowable Settlement (mm)		Foundation Type
TYPE-A	13.0	227.9		240.9	<	300	OK	Flexible
TYPE-B	15.9	304.3	50	270.2	<	300	OK	Flexible
TYPE-C	26.8	125.6		152.4	<	300	OK	Flexible
TYPE-D	42.1	208.8		250.9	<	300	OK	Flexible
TYPE-E	14.3	111.1		125.4	<	300	OK	Flexible
TYPE-F	13.9	122.8		137.0	<	300	OK	Flexible
TYPE-G	14.3	83.3		97.6	<	300	OK	Flexible
TYPE-H	15.0	287.4	50	252.4	<	300	OK	Flexible
TYPE-I	18.2	118.8		137.0	<	300	OK	Flexible
TYPE-J	35.4	135.0		170.4	<	300	OK	Flexible

Source: Study Team

The residual settlement at sluiceway site are obtained as summation of immediate and consolidation settlement. When the residual settlement exceeds 5cm, the sluiceway must be designed as flexible sluiceway to follow the deformation of the ground. Residual settlement is allowed up to 30cm based on capacity of flexible joint. If the settlement exceeds 30cm, camber bedding shall be considered.

Since residual settlement exceeds 5cm in all structural types, flexible type is applied for all sluiceways. Residual settlement of Type B and Type H initially exceed 30cm so that camber bedding are applied for them.

7.2.3.3 Structural Details

As detailed structural study, the structure of seepage control work, flexible joints and SSPs with flexible joint, box culverts, and breath walls are examined, and necessary dimensions are set. The details of the structural study are summarized in the **Sub-section 7.2.3.3 of Main Report**.

7.3 MCGS Detailed Design

7.3.1 Civil Engineering Design

7.3.1.1 Design Conditions

The major structural design conditions are shown in **Table 7.3.1**. Details are provided separately.

Table 7.3.1 List of MCGS Structural Design Conditions

Item	List of conditions		Reason for setting
Compliant Criteria	· DGCS 2015 · Revised Commentary and Order for the Construction of River Management Facilities · Ministry of Construction River Erosion Control Technical Standard (draft) Design Section [I] · weir design · A guide to flexible sluice design · Road bridge specifications IV substructure edition		The basic shape of the weir conforms to the standard on the left.
Material Specification	Concrete	Class A	Use materials from the Philippines
	Rebar	Grade 420	PNS: Philippine National Standard
Physical Constant	Young's modulus	200,000 MPa	Material properties in the Philippines
	Young's modulus ratio	n = 9	〃
	Linear expansion coefficient	10.8 x 10 -6	〃
Allowable Stress	Concrete	fc = 8.28 N/mm2, τ a = 0.36 N/mm2	〃
	Rebar	σ c = 168 N/mm2	〃
	Extra factor	Wind Load 25% Temperature change 25% 33% at the time of earthquake 50% during construction	According to the setting method in the Philippines 40% premium for wind load + temperature change
Minimum Member Thickness	Minimum	0.35 m	Normal
	Round value at premium	0.05 m	
Minimum Rebar Volume	Box lateral direction,	more than 0.2% of A	A: Effective cross-sectional area of concrete
	Longitudinal direction of box	more than 0.3% of A	〃
Rebar specifications	Major bars are set outside, and minor bars are set for each part according to the basic idea in.		General specifications. In order to use rebars as constraining bars for seismic resistance, rebars are arranged in the opposite direction from the left.

Source: Study Team

The design water level of the MCGS is shown in **Table 7.3.2**. The downstream water level at the time of flood used for structural examination is obtained by the hydraulic model experiment.

Table 7.3.2 Design Water Levels of MCGS

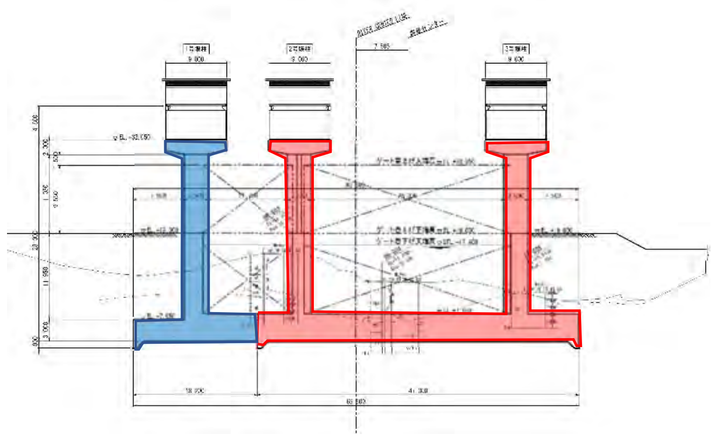
Item	Properties	Remarks
Design High Water Level (DFL)	Upstream: EL. 17.400 m Downstream: EL. 14.711 m	
Water Level	(Flood) Upstream: EL. 17.400 m Downstream: EL. 13.425 m	- The downstream water level is the water level at the time of the hydraulic model experiment.
	(Low water level) Upstream: EL. 17.400 m Downstream: EL. 11.003 m	- Downstream water level is calculated from the observed water level of Rosario dam on the Marikina River side.

Source: Study Team

7.3.1.2 Detailed Design of The Main Body

(1) Stability Calculation

This weir has 2 spans of 11.7 m and 28.3 m in span length. They are an inverse T shape for the end pier and a U shape structure for the end pier + central pier. (See **Figure 7.3.1**).



Source: Study Team

Figure 7.3.1 Structure Type of the Main Body

At each part of the piers (inverted T portion and U-shaped portion) and wing wall, the stability of "Normal Condition", "Seismic Condition" and "During Construction" would be verified, and it shall be checked that required stability is satisfied in overturning, sliding, and bearing. The details of the study are stated in the **Sub-section 7.3.2.3 (1) of Main Report**.

(2) Structural Calculation

Structural calculations would be carried out for the piers (inverted T portion and U-shaped portion), the breast wall, the upstream and downstream aprons, and the upstream and downstream wing wall facilities for “Normal Condition”, “Seismic Condition” and “During Construction”. Then, the necessary cross-sectional dimensions and bar arrangement specifications will be determined. The details of the study are stated in the **Sub-section 7.3.2.3 (2) of Main Report**.

7.3.1.3 L2 Seismic Design of the MCGS Main Body

(1) Seismic Design Condition

Floodgate, sluiceway and weir to be designed in this project are important facilities for flood control; therefore, “Seismic Performance 2” will be applied.

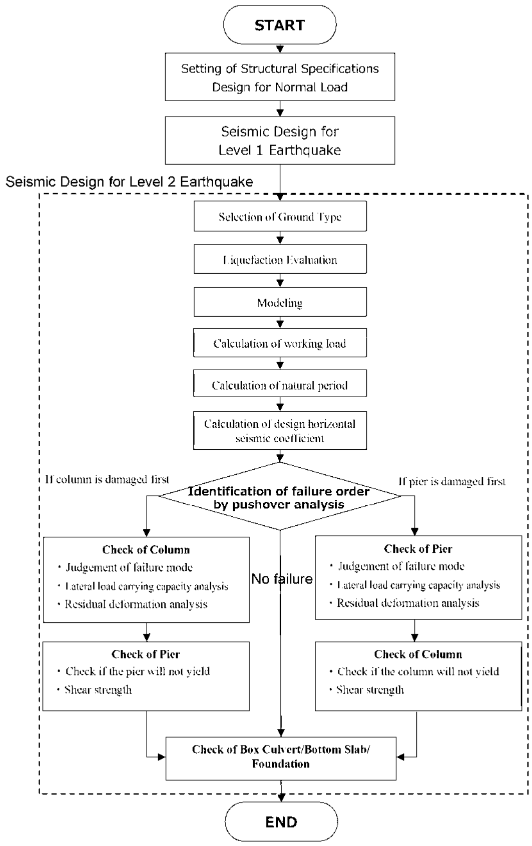
Table 7.3.3 Seismic Performance

Seismic Performance	Performance Required
Seismic Performance 1	Soundness as floodgate, sluiceway or weir is not impaired by an earthquake
Seismic Performance 2	Function as floodgate, sluiceway or weir is maintained even after earthquake
Seismic Performance 3	Damage caused by earthquake is limited, and function can be quickly recovered

Source: Performance Based Seismic Design Criteria for River Structures IV

(1) Analysis Method

The seismic analysis method is summarized in **Figure 7.3.2**.



Source: Study Team

Figure 7.3.2 Flow of Seismic Analysis

(2) Result of Analysis

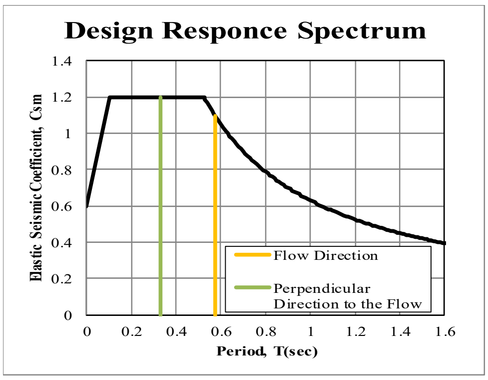
1) Setting of Design Horizontal Seismic Coefficient

Table 7.3.4 describes the horizontal seismic coefficient. These results are illustrated on the acceleration spectrum in **Figure 7.3.3** and **Figure 7.3.4**.

Table 7.3.4 Calculation Result of Design Horizontal Seismic Coefficient

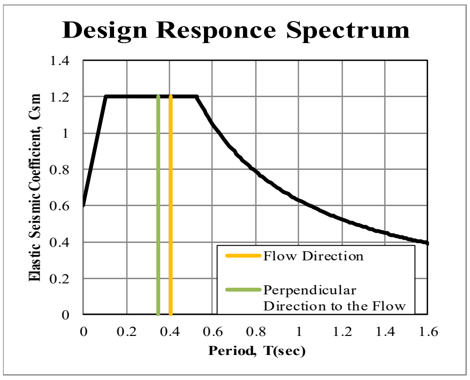
Item		Natural period T (s)	Design Horizontal Seismic Coefficient k_{hc0}
No. 1 Pier	Flow Direction	0.573	1.10
	Direction Perpendicular to The Water Flow	0.334	1.20
No. 2 to No. 3 Pier	Flow Direction	0.351	1.20
	Direction Perpendicular to The Water Flow	0.399	1.20

Source: Study Team



Source: Study Team

Figure 7.3.3 Calculated Design Horizontal Seismic Coefficient of No.1 Pier design No. 1



Source: Study Team

Figure 7.3.4 Calculated Design Horizontal Seismic Coefficient of No. 2 to No. 3 Piers intensity

2) Results of Analysis by the Seismic Horizontal Capacity Method

The analysis by the Seismic Horizontal Capacity Method would be conducted using the two models: No.1 pier, and No.2 to No.3 piers,

As a result, it was confirmed that the required seismic performance could be satisfied with increasing main reinforcement and hoop reinforcement of columns and the base of piers. The results of the analysis and the bar arrangement are summarized in the **Sub-section 7.3.2.4 (7) of Main Report**.

3) Foundation

It has been verified that the strength exceeds the generated value in both bending and shearing by increasing hoop bar and arrangement bar of the floor slab upper end side, and that the seismic performance is satisfied. The results of the analysis and the bar arrangement are summarized in **Sub-section 7.3.2.4 (7) of Main Report.**

7.3.1.4 Maintenance Bridge

The structural calculation have been carried out on main girder and floor slab of short span: L = 14.40 m and long span: L = 31.00 m with the load condition set in the basic design of **Chapter 6** and the determined section respectively, and it has confirmed that the required proof stress can be satisfied. The results of structural calculations are summarized in the **Sub-section 7.3.2.5 of Main Report**.

7.3.2 Gate Facility Design

Design conditions according to the basic design in **Chapter 6** and the following standards is set, and design calculations are performed. The details of each condition are summarized in the **Sub-section 7.3.3.1 of Main Report**, and the design calculation results are summarized in the **Sub-section 7.3.3.2 of Main Report** respectively.

- Technical Specification for Dams and Weirs in Japan (Draft)
- Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,
- Design Guideline for Hoist of Gate (Mechanical)(Draft)

The main specifications of the gate facilities arranged based on the basic design and the study above are shown below.

(1) Gate Facilities (Gate and Guide Frame)

Gate No.	No. 1			No. 2
Gate Type	Roller Gate of Shell Structure Duplex Stainless Steel			Plate Girder Structure Two-Phase Stainless Roller Gate
Pure Span x Effective Height	Clear Span 28.30 m x Effective Height 9.55 m			Clear Span 11.70 m x Effective Height 9.55 m
Number of Gate	1 Gate			1 Gate
Design Depth	(River Side)	EL + 17.400	(Design High Water Level)	
	(Seaward)	EL + 10.794	(Normal Water Level)	
Operating Depth (opening time)	(River Side)	EL + 12.486	(Start water level of -1 m)	
	(Seaward)	EL + 11.486	(Diverting Start Water Level)	
Operating Depth (closing time)	(River Side)	EL + 11.486	(Diverting Start Water Level)	
	(Seaward)	EL + 11.486	(Diverting Start Water Level)	
Bed Height	(Plan)	EL + 7.850		
Water sealing system	3 Way Front Rubber Watertight			
Operation Method	Machine side operation and remote control			

(1) Gate Facilities (Hoist)

Gate No.	No. 1		No. 2
Hoist TYPE	2M2D Wire Rope Winch		1M1D Wire Rope Winch
Rated Opening Capacity	2650 kN		610 kN;
Number of Installations	1		1
Additional Function	Self weight lowering function	Yes	
	Rest hook	Yes	
Normal Lift	Normal H1	11.150 m	
	Dogging H2	11.450 m	
Opening and closing speed	When using an electric motor	0.30 m/min	
	During self-weight descent	1.00 m/min	
Wire Rope	JIS 6 × 37 G Type Plating		
Power	200 VAC - 50 Hz		220 VAC - 60 Hz

(1) Electrical Equipment (Machine Side Control Panel)

Control Panel-Type	Ingate closing Self-Standing Type Steel Plate
Number of Installations	1 Face
Outline Dimensions	Width: 1.000 m x Height: 2.000 m x Depth: 0.500 m

7.3.3 Detailed Design of Information Equipment

7.3.3.1 Design Conditions of Information Equipment

In the information equipment design, the design conditions from the basic design in **Chapter 6** are summarized in **Table 7.3.5**.

Table 7.3.5 Design Conditions

Target Facility	Equipment Classification	Installation Equipment	Design Conditions and Considerations	Installation Quantity
MCGS	Instrumentation Facility	Water Level Gage	Measure the water level upstream and downstream of the gate for accurate gate operation. The water level shall be measured at the level.	4 water level meters 1 observation unit
	Alarm Facility	Siren Loudspeakers Sound Collection Microphone Revolving Light	Install to ensure safety during gate operation	One siren 4 loudspeakers Four sound collection microphones Two rotary lights One control unit
	Monitoring Facility	CCTV Camera	Install camera equipment to check the status of gate opening and closing. Adopt products with a low minimum illuminance of the subject and do not install lighting equipment (floodlight) to enable nighttime monitoring.	Four camera units
	Management Facility	Monitoring and Control Equipment	Monitoring and control are carried out in conjunction with the upstream/downstream water level and CCTV camera images. In addition to monitoring and control in the generator building, information required for monitoring and control is transmitted to EFCOS.	Transmission equipment Network equipment

Source: Study Team

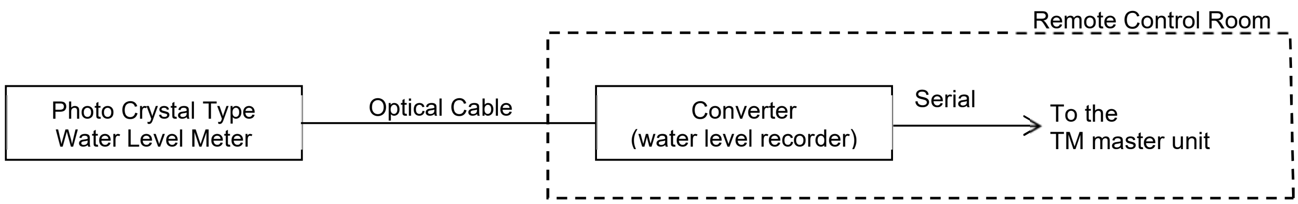
7.3.3.2 Instrumentation (Water Level Observation Equipment) Design

Water level observation system, water level meter layout, and equipment configuration of instrumentation equipment (Water level observation equipment) are studied, and equipment specifications are established. **Table 7.3.6** summarizes the result.

Table 7.3.6 Summary of MCGS Instrumentation

Item	Specifications etc.	Verification
Location of Water Level Gauge	On the revetment part in the upstream side of the weir (floodgate)	Considering workability and easy maintenance
Water Level Observation System	Hydraulic (quartz hydraulic system) water gauge	A float type, reed switch type, hydraulic type (quartz hydraulic system), ultrasonic wave type, and radio wave type that can be installed on the revetment are compared, and the most excellent type in terms of workability and maintenance management is selected.

Source: Study Team



Source: Study Team

Figure 7.3.5 Instrumentation Configuration

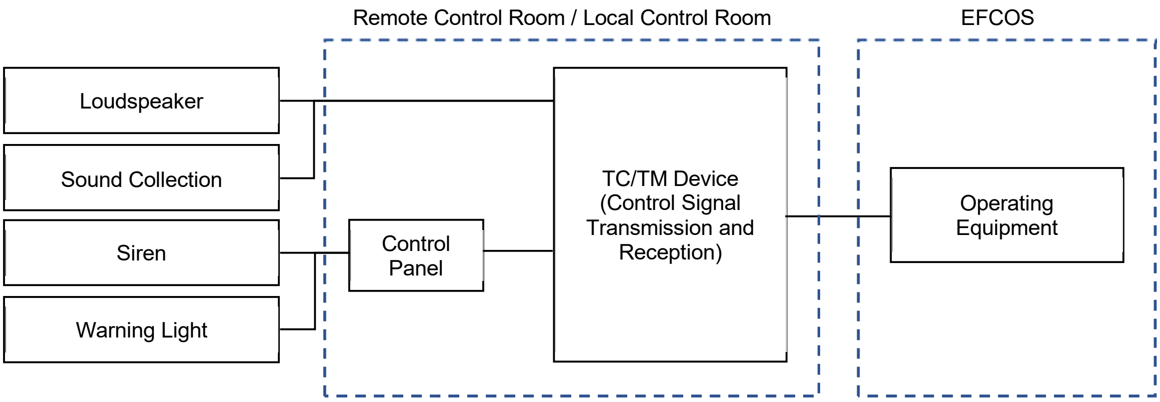
7.3.3.3 Alarm Facility Design

Siren structure, speaker capability, alarm lamp type, etc. are studied and set the equipment specifications. **Table 7.3.7** summarizes the result.

Table 7.3.7 Summary of MCGS Alarm Facilities

Item	Specifications etc.	Verification
Siren	Inverter siren Capacity: 2.2 kW	· Maintainable and lightweight · The sound pressure level shall be such that a linear distance of 570 m from MCGS to EFCOS can be reached.
	Location: Operation Room Rooftop Installation quantity: 1 unit	The sound range of the siren is 800 m, and by placing it on the MCGS operation room, the alarm sound can be reached within a radius of 800 m.
Speaker/Sound Collecting Microphone	Capacity: 50 W	The loudspeaker sound reaches the opposite shore at the maximum, and the sound pressure level reaches a distance of 60 m.
	Installation Location: Upstream/Downstream Side Operation Room (Left and Right Bank) Installation Quantity: 4 Locations	A speaker, a sound collecting microphone and an alarm lamp are arranged in a left bank machine side operation room and a right bank machine side operation room so that they can be blown and turned on in each operation of a small diameter gate and a large span gate. They are arranged on the upstream side and the downstream side of the machine side operation chamber so that they can be known to the upstream/downstream.
Warning Light	LED system Rotating reflector or flashing lamp	Installed to add visual information in addition to audible alarms Uses long-life, power-saving products
	Installation Location: Upstream/Downstream Side Operation Room (Left and Right Bank) Installation Quantity: 4 Locations	Same as speakers and microphone
Operating Equipment	Display Console System EFCOS ONLY	It is superior to the dedicated console in terms of economy and expandability.

Source: Study Team



Source: Study Team

Figure 7.3.6 Configuration of Alarm Facility

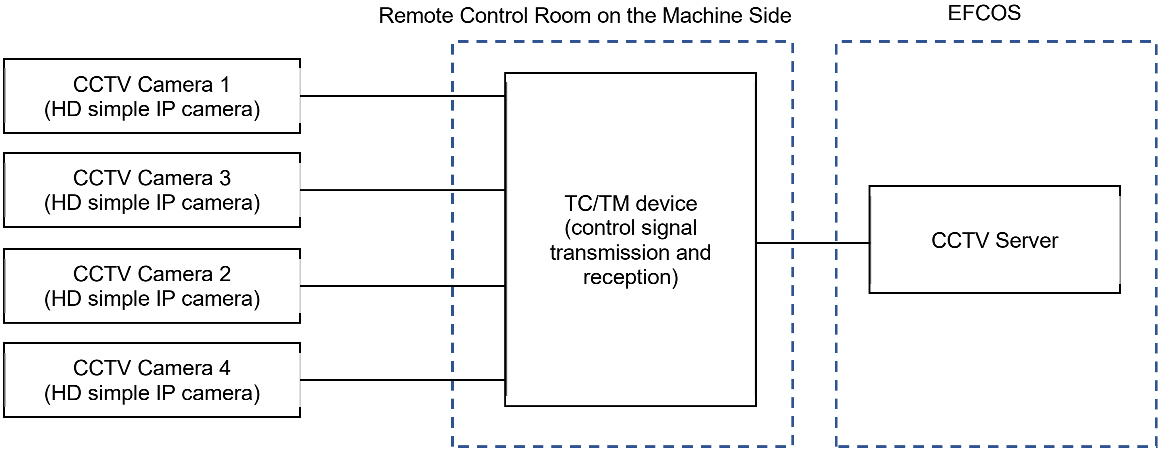
7.3.3.4 Design of Monitoring Equipment (CCTV Camera)

Monitoring system, monitoring equipment layout, and equipment configuration are studied, and establish equipment specifications. **Table 7.3.8** summarizes the results.

Table 7.3.8 Summary of MCGS Monitoring Facility (CCTV Camera)

Item	Specifications etc.	Verification
Monitoring System	HD Simple IP Camera Device	· Superior economics compared to HDIP camera equipment and HDIP camera equipment (high sensitivity) · It can be used in environments with site lighting
Monitoring Equipment Layout	Location: Installed on the left and right banks of the upstream and downstream Installation quantity: 4 units in total	· Located near the gate and inside the fences on the revetment. · Use for spatial monitoring by turning

Source: Study Team



Source: Study Team

Figure 7.3.7 Configuration of Monitoring Equipment

7.3.3.5 Remote Monitoring and Control Facility

Management items, management functions, transmission lines (network), system configuration, etc. are studied, and establish equipment specifications. **Table 7.3.9** summarizes the results. Cainta Floodgate and Taytay Sluiceway are also studied. The overall system diagram is shown in **Figure 7.3.148 of Main Report**.

Table 7.3.9 Summary of Remote Monitoring and Control Facilities

Item	Specifications etc.	Verification
Management Function	System Management, Data Collection, Alarm Determination, Computation, Monitoring Display, Data Storage, Maintenance	Selected necessary functions from management items
Transmission Line	・ Establishment of a Exclusive Line ・ Optical Line	・ For reliable data transmission ・ 120 Mbps transmission bandwidths for MCGS and 270 Mbps transmission bandwidths for Cainta/Taytay
	・ Optical cable specifications (piping, number of lines, buried position, etc.) are determined for each section in accordance with "Optical Fiber Cable Installation Procedure and Explanation, Japan"	・ The remote monitoring and control facilities were installed in the MCGS remote control room and the Cainta Floodgate remote control room starting from the EFCOS. ・ The above section is divided into 8 sections according to the conditions of bridges, roads, etc., and specifications required for each section are established.
Network Equipment	・ L3-SW	In order to avoid bottlenecks in the network, the packet transfer capability is large, and the safety and stability can be improved by duplexing the power supply.
	・ Fixed Type C	Since the L3-SW is a high-standard specification capable of constituting a backbone network, equipment with relatively low specifications in connection port number, data transfer capacity, etc. is selected.

Source: Study Team

7.3.3.6 Electrical Equipment (Emergency Power Supply) Design

Based on the calculation of the capacity of the generators and the selection of motors, the specification is established, and the layout of emergency power generation facilities, etc. are examined. **Table 7.3.10** summarizes the results.

Table 7.3.10 Summary of MCGS Electrical Equipment (Emergency Power Supply)

Item	Specifications etc.	Verification
Expected Operating Time	Gate Equipment: 8 hours	・ It is assumed that the gate is opened and closed more than twice a day during the blackout period.
	Control Equipment: 3 days (72 hours)	・ Considering the situation of existing flood control facilities in Metro Manila and design standards for disaster prevention facilities in Japan
Generator	Horizontal Synchronous Generator	・ Set based on the "Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)"

Item	Specifications etc.	Verification
Generator Capacity	For Gate Equipment: 250 kVA	· Calculating the output of the load, type and starting method, presence or absence of fire-fighting related load, type of motor, etc., and selecting the one that is closest to the standard efficiency table of generators (“Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)”)
	For Control Equipment: 50.0 kVA	· Same as above
Motor	Diesel Engine	· In principle, a diesel engine with a high fuel consumption rate should be used in accordance with the “Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)”.
Motor Output	For Gate Equipment: 225 kW	· Calculating the output of the load, type and starting method, presence or absence of fire-fighting related load, type of prime mover, etc., and selecting the one that is closest to the standard efficiency table of generators (“Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)”)
	For Control Equipment: 48.6 kW	· Same as above
Amount of Fuel Oil Stored	For gate equipment: 627 Liters ->Supplied from the Generator Tank	· Computation in accordance with the Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)”
	For control equipment: 1,353 Liters ->Install a Service Tank in the Outside	· Same as above

Source: Study Team

7.4 Detailed Design of Cainta Floodgate

7.4.1 Civil Engineering Design

7.4.1.1 Design Condition

The basic design conditions are the same as for MCGS and these are shown in **Table 7.3.1**. The design water level used in the design of the Cainta Floodgate is indicated in **Table 7.4.1**.

Table 7.4.1 Design Water Table

No.	Water Level Condition	Design Water Level	Verification
1	Floodway Side, Design Flood Level (DFL)	14.853	Calculated by interpolation from the As-built Drawing2)
2	Tributary Side, Design Flood Level (DFL)	13.340	Cainta River DFL 1)
3	Design Riverbed	8.750	Cainta River STA. 0 + 000, Design Riverbed 1)
4	Floodway Side, OWL	11.30	OWL of Manggahan Floodway
5	Floodway Side, Low Water Level (LWL)	10.94	LWL of Manggahan Floodway
6	Target Water Level for Cofferdam During Construction	14.45	Highest water level of Lake Laguna in the last 5 years (2014 ~ 2018) + 5 cm, taking into account the rise in water level due to cofferdam

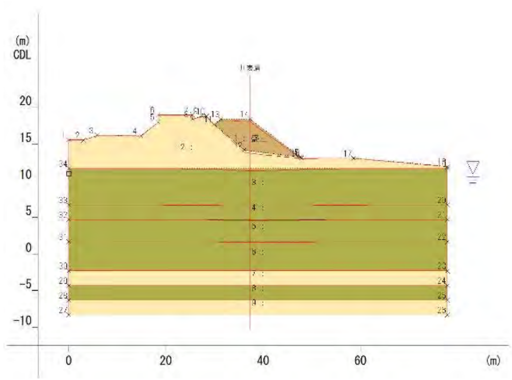
Source: 1) 2008 Pre-F/S
2) Final Report on Consulting Services for Manggahan Floodway Project

7.4.1.2 Foundation Work

(1) Examination of Consolidation Settlement

The calculation results are shown in **Figure 7.4.1**. The amount of consolidation settlement was more than 30 cm in both cases. In the basic design, 40 cm of the extra embankment was considered for the existing levee height EL + 18.00 m, and the following results confirmed the validity of the construction levee height EL + 18.40 m.

Station: 4 + 485 Settlement: 0.34 m

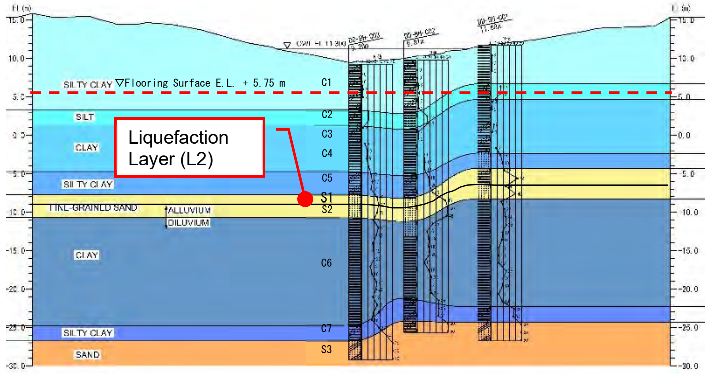


Source: Study Team

Figure 7.4.1 Consolidation Settlement Diagram (STA.4 + 485)

(1) Study of Liquefaction

For each ground of 3 borehole logs around Cainta Floodgate, liquefaction shall be assessed according to the "Performance Based Seismic Design Criteria for River Structures, Japan". Liquefaction was assessed against the design horizontal seismic intensity of L1 and L2 earthquake ground motions



Source: Study Team

Figure 7.4.2 Liquefied Layer

As a result of Liquefaction Analysis, it was evaluated that there is no liquefaction layer against L1 earthquake motion. For the L2 earthquake motion, the S1 layer was evaluated as the liquefaction layer. (See **Figure 7.4.2**)

(2) Design of Foundation Piles

In the design of a foundation pile, under the subjected load conditions, the items listed in **Table 7.4.2** would be checked. if each value does not exceed the allowable values. The most economical pile arrangement among pile arrangements satisfying the allowance would be adopted. The adopted pile arrangement is shown in **Figure 7.4.22 of Main Report**. The results of the study are summarized in the **Sub-section 7.4.2.1 (3) 3) to 10) of Main Report**.

Table 7.4.2 Items to be Checked In Pile Foundation Layout Examination

Item	Checked value	Tolerance	Remarks
Axial push-in bearing force	Maximum value of push-in bearing force Pmax (kn)	Allowable bearing capacity of pile Ra (kn)	
Axial pull-out force	Maximum drawing force Pmin (kn)	Allowable Axial pull-out force Pa (kn)	
Horizontal displacement	Horizontal displacement δx (mm)	Allowable horizontal displacement δxa (mm) = 10 mm	
Pile Body Stress	Pile Body Stress σtc (n/mm2)	Allowable stress σa (n/mm2)	SKK 400

Source: Study Team

7.4.1.3 Main Body Work

(1) Stability Calculation

Since this facility is supported with pile foundations, the stability is verified in the study of pile foundation. Here, the external force used for the verification of the pile is set. External forces are calculated at each facility for the piers (center pier and end pier), the floor slab, the wing wall (upstream left bank, upstream right bank, downstream side), and the apron (upstream and downstream) with respect to the “Normal Condition” “Seismic Condition ” and “During Construction”, and the capacity of piles are verified to determine necessary cross-sectional dimensions. The details of the study are stated in the **Sub-section 7.4.2.2 (1) of Main Report**.

(2) Structural Calculation

Structural calculation is carried out at each facility of the piers (center pier and end pier), floor slab, wing wall (upstream left bank, upstream right bank, downstream side), and apron (upstream and downstream) for "Normal Condition" "Seismic Condition" and "During Construction", and necessary sectional dimensions and reinforcement arrangement dimensions are set. The details of the study are stated in the **Sub-section 7.4.2.2 (2) of Main Report**.

7.4.1.4 Main Body Work (L2 Seismic Design)

(1) Seismic Design Condition

The Cainta Floodgate to be designed in this project are important facilities for flood control; therefore, "Seismic Performance 2" will be applied.

(2) Analysis Method

The examination procedure is the same as MCGS.

(3) Results of Analysis

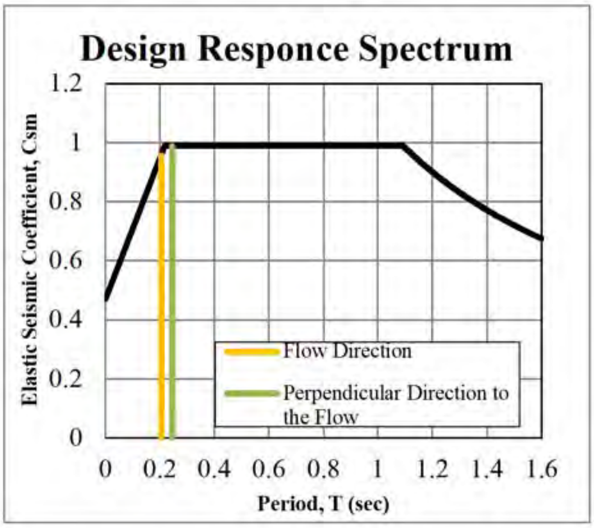
1) Setting of Design Horizontal Seismic Intensity

The calculation results of horizontal seismic intensity is shown in **Table 7.4.3** and **Figure 7.4.3** and **Figure 7.4.4** shows it in the acceleration spectrum.

Table 7.4.3 Calculation Result of Design Horizontal Seismic Intensity

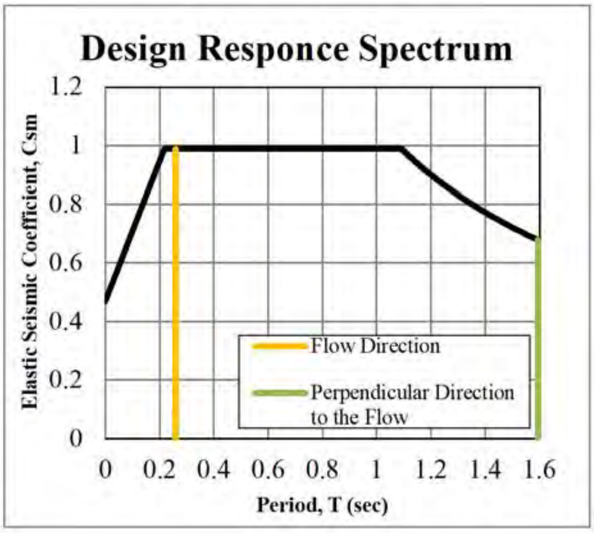
Item		Natural Period T (s)	Design Horizontal Seismic Coefficient khc0
End Pier	Flow Direction	0.203	0.95
	Perpendicular Direction to The Flow	0.246	0.99
Center Pier	Flow Direction	0.257	0.99
	Perpendicular Direction to The Flow	1.592	0.68

Source: Study Team



Source: Study Team

Figure 7.4.3 Calculation Result of Horizontal Seismic Coefficient for End Pier Design



Source: Study Team

Figure 7.4.4 Results of Calculation of Horizontal Seismic Coefficient for Center Pier Design

2) Results of Analysis by the Seismic Horizontal Capacity Method

About the two kinds of models of end pier and center pier, analysis by the seismic horizontal capacity method is performed in the water flow direction and the water flow perpendicular direction. It has been verified that the required seismic performance can be satisfied by the increase of main reinforcement and hoop reinforcement of column and pier. The results of the analysis and the bar arrangement are summarized in the **Sub-section 7.4.2.3 (4) of Main Report**.

3) Foundation

Analysis by the seismic horizontal capacity method is performed on the pile foundation. As a result, it has been verified that the indictment does not yield, and the pile head bending moment is lower than the yield bending moment My of the virtual RC section, and the reinforcement arrangement specifications necessary to satisfy the seismic performance is set. The results of the study and the bar arrangement are summarized in the **Sub-section 7.4.2.3 (4) of Main Report**.

7.4.1.5 Maintenance Bridge

The structural calculation is carried out with the load condition set in the basic design of **Chapter 6** and the determined section, and it has been verified that the required proof stress can be satisfied. The results of structural calculations are summarized in the **Sub-section 7.4.2.4 of Main Report**

7.4.2 Gate Facility Design

Design conditions according to the basic design in **Chapter 6** and the following standards is set, and design calculations are performed. The details of each condition are summarized in the **Sub-section 7.4.3.1 of Main Report**, and the design calculation results are summarized in the **Sub-section 7.4.3.2 of Main Report** respectively.

- Technical Specification for Dams and Weirs in Japan (Draft)
- Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,
- Design Guideline for Hoist of Gate (Mechanical)(Draft)

The main specifications of the gate facilities arranged based on the basic design and the study above are shown below.

(1) Floodgate Facilities (Gate Leaf and Guide Frame)

Gate Type	Plate Girder Structure Duplex Stainless Roller Gate		
Pure Span X Effective Height	Clear Span 16.00 m X Effective Height 7.31 m		
Number of Gates	Two Gates		
Design Depth	(Floodway Side)	El + 14.853	(DFL)
	(Tributary Side)	El + 10.100	(OWL in Tributary River)
Operating Depth (Opening Time)	(Floodway Side)	El + 12.940	(Design Dike Crown Of Tributary River: -1 M)
	(Tributary Side)	El + 13.940	(Design Dike Crown Of Tributary River)
Operating Depth (Closing Time)	(Floodway Side)	El + 15.940	(Design Dike Crown Of Floodway)
	(Tributary Side)	El + 13.940	(Design Dike Crown Of Tributary River)
Invert Elevation	(Plan)	El + 8.750	
Water Sealing System	Rear Three-Way Rubber Watertight		
Operation Method	Machine Side Operation and Remote Control		

(2) Floodgate Facilities (Hoist)

Hoist Type	1M1D Wire Rope Winch Type	
Rated Opening Capacity	680 KN,	
Number of Installations	Two	
Additional Function	Self-Weight Lowering Function	Yes
	Dogging Device	Yes
Normal Lift	Normal H1	9.650 m
	Dogging H2	9.950 m

Opening And Closing Speed	When Using An Electric Motor	0.30 m/min
	During Self-Weight Lowering	1.00 m/min
Wire Rope	JIS 6 × 37 G type plating	
Power	200 VAC - 50 Hz	

(3) Electrical Equipment (Machine Side Control Panel)

Control Panel Type	Indoor Closing Self-Standing Type Steel Plate
Number of Installations	Two Faces
Outline Dimensions	Width: 1.500 m x Height: 2.000 m x Depth: 0.500 m

7.4.3 Design of Information Facilities

7.4.3.1 Organizing Design Conditions

In the information equipment design, the design conditions are summarized in **Table 7.4.4** from the basic design summarized in **Chapter 6**.

Table 7.4.4 Design Conditions

Facility	Equipment Classification	Installation Equipment	Design Conditions and Considerations	Installation Quantity
Cainta Floodgate	Instrumentation Facility	Water Level Gage	Measure the water level upstream and downstream of the gate for accurate gate operation. The water level shall be measured at the level.	Two Water Level Meters 1 Observation Unit
	Alarm Facility	Siren Speakers Sound Collection Microphone Revolving Light	Install to ensure safety during gate operation	One Siren 2 Speakers Two Sound Collection Microphones Two Rotary Lights One Control Unit
	Monitoring Facility	CCTV Camera	Install camera equipment to check the status of gate opening and closing. Adopt products with a low minimum illuminance of the subject and do not install lighting equipment (floodlight) to enable nighttime monitoring.	Four Camera Units
	Management Facility	Monitoring and Control Equipment	Monitoring and control are carried out in conjunction with the upstream/downstream water level and CCTV camera images. In addition to monitoring and control in the generator building, information required for monitoring and control is transmitted to EFCOS.	Transmission Equipment Network Equipment

Source: Study Team

7.4.3.2 Design of Facilities

Instrumentation (Water level observation equipment) equipment, alarm equipment, monitoring equipment (CCTV cameras) and power supply equipment (emergency power plant) would be designed in the same way as MCGS. The items to be considered, procedures, and equipment configuration are the same as those of MCGS. Summary of the facilities is shown in **Table 7.4.5**. Instrumentation (Water level observation equipment) equipment has the same specifications as MCGS.

Table 7.4.5 Summary of Information/Electrical Facilities of Cainta Floodgates

Item	Specifications etc.	Verification
Equipment Classification	Instrumentation (Water Level Observation Equipment) Equipment	
Location of Water Level Gauge	On the revetment part in the upstream and downstream side of the weir (floodgate)	Considering workability and easy maintenance
Water Level Observation System	Hydraulic (Quartz Hydraulic System) Water Gauge	A float type, reed switch type, hydraulic type (quartz hydraulic system), ultrasonic wave type, and radio wave type that can be installed on the revetment are compared, and the most excellent type in terms of workability and maintenance management is selected.
Equipment Classification	Alarm Facility	
Siren	Inverter Siren Capacity 0.75 Kw	· maintainable and lightweight · Siren capacity may be smaller than MCGS · Sound reach distance of about 500 m
	Location: Operation Room Rooftop Installation Quantity: 1 Unit	Same as MCGS
Speaker/Sound Collecting Microphone	Capacity: 25 W	The loudspeaker sound reaches the opposite shore at the maximum, and the sound pressure level reaches 50 m.
	Location: Left Bank and Right Bank Side of the Control Room Installation Quantity: 4 Places	The loudspeaker, sound collection microphone, and alarm lamp are arranged on the left bank machine side operation room and the right bank machine side operation room so that they can blow and turn on both banks of the Cainta River.
Warning Light	LED System Rotating Reflector or Flashing Lamp	Same as MCGS
	Location: Left Bank and Right Bank Side of the Control Room Installation Quantity: 4 Places	Same as speakers and microphone
Operating Equipment	Display Console System	It is superior to the dedicated console in terms of economy and expandability.
Equipment Classification	Monitoring Equipment (CCTV Camera)	
Monitoring System	HD Simple IP Camera Device	· Same as MCGS
Monitoring Equipment Layout	Location: Left and right bank of the gate and upstream and downstream of the floodgate Installation quantity: 4 units in total	· Install one unit for facility monitoring in the left bank side operation room and the right bank side operation room. · Install one for spatial monitoring the land side in the left bank side operation room · Install one for spatial monitoring in the confluence side and on the right bank revetment.
Equipment Classification	Electrical Equipment (Emergency Power Supply)	
Expected Operating Time	Gate Equipment: 2 Hours	· It is assumed that the gate is opened and closed once a day during the blackout period.
	Control Equipment: 3 Days (72 Hours)	· Same as MCGS
Generator	Horizontal Synchronous Generator	· Same as MCGS
Generator Capacity	For Gate Equipment: 150 KVA	· Calculating the output of the load, type and starting method, presence or absence of fire-fighting related load, type of prime mover, etc., and selecting the one that is closest to the standard efficiency table of generators ("Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)")
	For Control Equipment: 50.0 KVA	· Same as above
Motor	Diesel Engine	· Same as MCGS
Motor Output	For Gate Equipment: 138 Kw	· Calculating the output of the load, type and starting method, presence or absence of fire-fighting related load, type of prime mover, etc., and selecting the one that is closest to the standard efficiency table of generators ("Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)")
	For Control Equipment: 48.6 KW	· ditto
Amount of Fuel Oil Stored	For Gate Equipment: 107 Liters -> Supplied from the Generator Tank	· Computation in accordance with the "Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)"
	For Control Equipment: 1,353 Liters -> Install a Service Tank In the Outside	· Same as above

Source: Study Team

7.5 Detailed Design of Taytay Sluiceway

7.5.1 Civil Engineering Design

7.5.1.1 Design Condition

The basic design conditions are the same as for MCGS and these are shown in **Table 7.3.1** The design water level used in the design of the Taytay Sluiceway is indicated in **Table 7.4.6**. The high water channel bed elevation required for setting the residual water level RWL in structural calculation of wing walls, etc. was set to EL+ 12.40 m from the existing ground height.

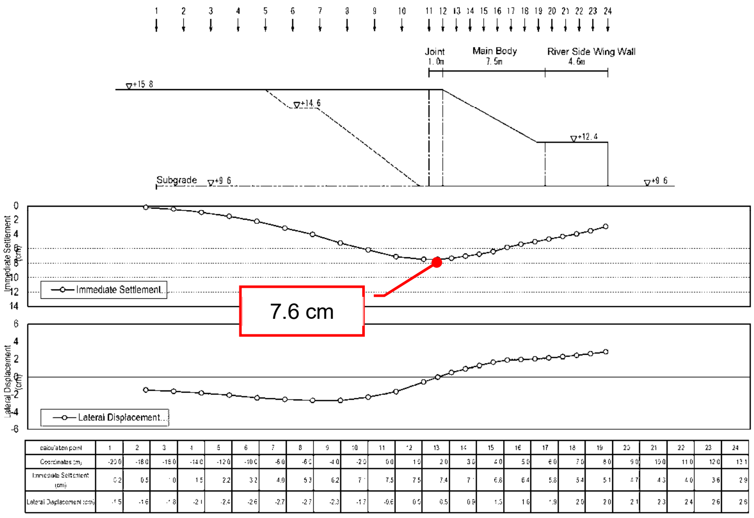
Table 7.4.6 List of Design Water Levels of Taytay Sluiceway

Item	Applied Value	Remarks
DFL	14.52	Calculated by interpolation from the As-built drawing of Manggahan Floodway DFL
Ordinary Water Level OWL	11.30	
Low water Level LWL	10.94	
Groundwater Level GWL	10.94	
Height of flood Plain	12.40	Set arbitrarily based on the current sand bar height

Source: Study Team

7.5.1.2 Foundation Work

As the result of calculation, the residual settlement is 7.6 cm and less than 10 cm, and a spread foundation is used, and the structure would be a flexible sluiceway. From the result of structural calculation in the longitudinal direction of the box culvert, the settlement and opening of the box culvert is 10 cm or less. Hence, the capacity of flexible joint is 10 cm (Minimum Capacity).



Source: Study Team

Figure 7.5.1 Settlement Diagram

7.5.1.3 Main Body Work

(1) Stability Analysis of Main Body

Since the span length of the sluiceway main body is short, the safety against overturning, sliding and bearing capacity is checked. The load acting on the foundation shall be the composited load of the column and the breast wall. Each stability calculation is carried out for "Normal Condition" and "Seismic Condition" , and required cross-sectional dimensions is set. The details of the study are stated in the **Sub-section 7.5.2.4 of Main Report**.

(2) Structural Calculation

In the box culvert (lateral and longitudinal direction), columns, river side breast wall, and river side wing wall (U-shaped and wing section), structural calculation is performed for "Normal Condition" "Seismic Condition" , and necessary cross-sectional dimensions and bar arrangement specifications are set. The details of the study are stated in the **Sub section 7.5.2.4 of Main Report**.

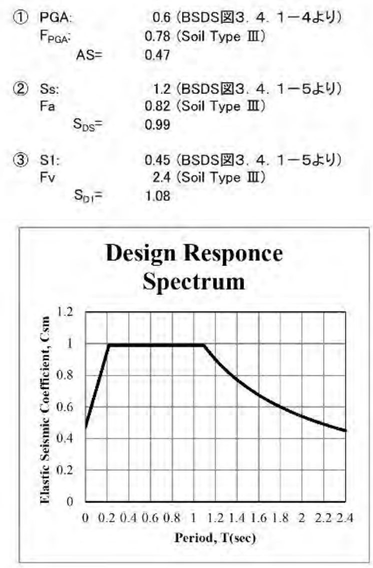
7.5.1.4 Main body Work (L2 Seismic Design)

(1) Seismic Design Condition

1) Seismic Performance

The Taytay Sluiceway to be designed in this project are important facilities for flood control; therefore, "Seismic Performance 2" will be applied.

2) Design Horizontal Seismic Intensity



The design horizontal seismic intensity khgL shall be as follows in accordance with the BSDS, considering that the ground concerned is Class III ground.

Fpga = 0.78, Fa = 0.82, Fv = 2.4

Therefore, the design horizontal seismic intensity khgL is as follows.

khgL = Fpga × PGA
= 0.78 x 0.60 = 0.468
≒ 0.47 (Design Horizontal Seismic Intensity at the Ground Level, Level 2)

(2) Analysis Method

1) Box Culvert (Longitudinal)

The deformation of the bottom of culvert is calculated by static FEM analysis considering liquefaction, and it is verified if the deformation of the joint is within the allowable value.

2) Column

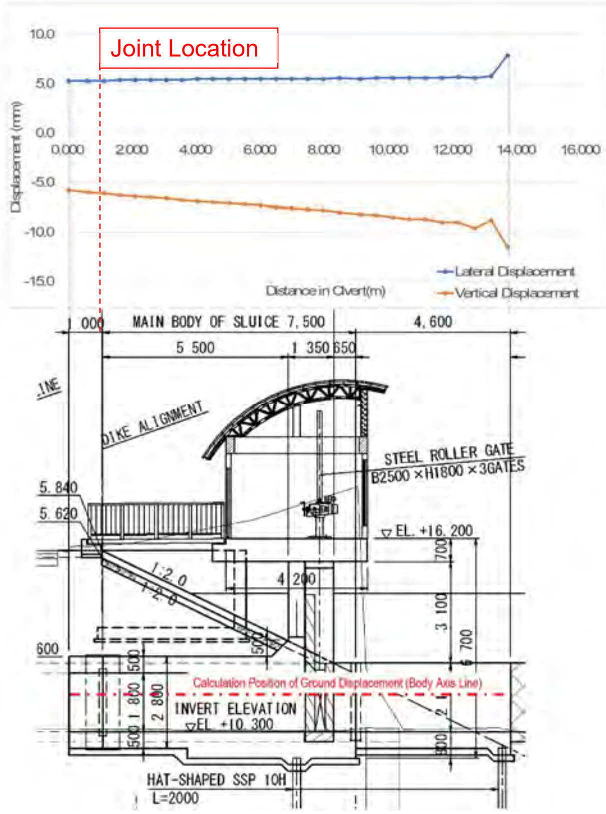
Seismic horizontal capacity and residual displacement at the time of earthquake are verified (Same as MCGS and the Cainta Floodgate). However, the stability of the box culvert against L2 earthquake may not be secured only by this method.

Therefore, the column and the pier are integrated into a three-dimensional model to check the stability against L2 earthquake ground motion.

(3) Results of Analysis

1) Sluiceway Longitudinal Direction

As a result of FEM analysis, the amount of ground deformation caused by the earthquake on the installation axis of the Taytay Sluiceway is calculated. As shown in **Figure 7.5.2**, both horizontal and vertical displacements is about 5 mm at the joint positions between the existing and new.



Source: Study Team

Figure 7.5.2 Ground Deformation at Main Body

The results of verification of the joint openings and calculating the amount of misalignment after the L2 earthquake are shown in **Table 7.5.1**. The calculation results here take into account both immediate settlement and settlement due to liquefaction. Since both of them is below the allowable value of the flexible joint, the joint would not be damaged even after L2 earthquake.

Table 7.5.1 Verification Results of Joint

Item	Calculated Result	Capacity	Calculated Result
Opening	16.5 mm	100 mm	Within the Capacity
False alarm	Start 131.0 mm, End 132.1 mm 1.1 mm gap	200 mm	Within the Capacity

Source: Study Team

2) Verification of Column

The verification of the column is carried out in the direction of the water flow and the direction perpendicular to the flow for two locations of the middle column and the end column. Both the seismic horizontal capacity and residual displacement at the time of the earthquake is below the allowable values, and the failure mode is determined to be bending failure type. In other words, even if L2 earthquake motion occurs with the reinforcement designed for L1 earthquake motion, additional seismic reinforcement is unnecessary because the deformation of the structure is within the elastic region and fatal damage can be avoided.

7.5.2 Gate Facility Design

Design conditions according to the basic design in **Chapter 6** and the following standards is set, and design calculations are performed. The details of each condition are summarized in the **Sub-section 7.5.3.1 of Main Report**, and the design calculation results are summarized in the **Sub-section 7.5.3.2 of Main Report** respectively.

- Technical Specification for Dams and Weirs in Japan (Draft)

- Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,
- Design Guideline for Hoist of Gate (Mechanical)(Draft)

The main specifications of the gate facilities arranged based on the basic design and the study above are shown below.

(1) Gate Facilities (Gate Leaf and Guide Frame)

Gate Type	Plate Girder Structure Duplex Stainless Roller Gate	
Pure Span X Effective Height	Clear Span 2.50 m × Effective Height 1.80 m	
Number of Gates	Three Gates	
Design Depth	(Floodway Side)	EL + 14.520 (DFL)
	(Tributary Side)	EL + 10.600 (OWL in Tributary River)
Operating Depth (Opening Time)	(Floodway Side)	EL + 13.100 (River Bank Elevation of Tributary River: -1 m)
	(Tributary Side)	EL + 14.100 (River Bank Elevation)
Operating Depth (Closing Time)	(Floodway Side)	EL + 15.620 (Design Dike Crown of Floodway)
	(Tributary Side)	EL + 14.100 (Design Dike Crown of Tributary River)
Invert Elevation	(Plan)	EL + 10.300
Water Sealing System	Rear 4-way Rubber Watertight	
Operation Method	Machine Side and Remote Operation	

(2) Gate Facilities (Hoist)

Hoist Type	Double Rack Type	
Rated Opening Capacity	30 KN;	
Number of Installations	3 units	
Additional Function	Self-weight lowering function	Yes
Normal Lift	Normal H1	1.90 m
	Dogging H2	2.20 m
Opening and Closing Speed	When Using an Electric Motor	0.30 m/min
	During Self-Weight Lowering	2.00 m/min
Power	220 VAC – 60 Hz	

(3) Electrical Equipment (Machine Side Control Panel)

Control Panel Type	Switch Mounted Type
Number of Installations	3 Faces
Outline Dimensions	Width: 0.60 m x Height: 0.60 m x Depth: 0.35 m

7.5.3 Information Equipment Design

7.5.3.1 Design Conditions

In the information equipment design, the design conditions are summarized in **Table 7.5.2** from the basic design in **Chapter 6**.

Table 7.5.2 Design Conditions

Target Facility	Equipment Classification	Equipment	Design Conditions and Considerations	Quantity
Taytay Sluiceway	Instrumentation Facility	Water Level Gage	Measure the water level upstream and downstream of the gate for appropriate gate operation. The water level shall be measured with the level.	Two Water Level Meters 1 Observation Unit
	Alarm Facility	Speakers Sound Collection Microphone Revolving Light	Install to ensure safety during gate operation	1 Speaker One Sound Collection Microphone Two Rotary Lights One Control Unit
	Monitoring Facility	CCTV camera	Install camera equipment to check the status of gate opening and closing. Adopt products with a low minimum illuminance of the subject and do not install lighting equipment (floodlight) to enable nighttime monitoring.	Four Camera Units
	Management Facility	Monitoring and Control Equipment	Monitoring and control are carried out together with the upstream/downstream water level and CCTV camera images. In addition to monitoring and control in the local control house, information required for monitoring and control is transmitted to EFCOS.	Transmission Equipment Network Equipment

Source: Study Team

7.5.3.2 Design of Facilities

Instrumentation (Water level observation equipment) equipment, alarm equipment, monitoring equipment (CCTV cameras) and power supply equipment (emergency power plant) would be designed in the same way as MCGS. The items to be considered, procedures, and equipment configuration are the same as those of MCGS. Summary of the facilities is shown in **Table 7.5.3**.

Table 7.5.3 Summary of Information/Electrical Facilities of Taytay Sluiceways

Item	Specifications etc.	Verification
Equipment Classification	Instrumentation (Water Level Observation Equipment) Equipment	
Equipment Classification	On the Revetment Part in the Downstream Side of the Sluiceway Existing Revetment Part Around the Intakes of the Existing Culvert	Considering workability and easy maintenance
Equipment Classification	Hydraulic (Quartz Hydraulic System) Water Gauge	Same as MCGS
Equipment Classification	Alarm Facility	
Siren	Substituted by simulated speaker sound	Due to the limited range to be notified
	Location: Operation Room Rooftop Installation quantity: 1 unit	Same as MCGS
Speaker/Sound Collecting Microphone	Capacity 25 W	The loudspeaker sound reaches the opposite shore at the maximum, and the sound pressure level reaches 15 m.
	Location: Left bank side on the local control house of the sluiceway Installation quantity: 1	Speakers, sound collection microphones, and alarm lights are installed on the left bank of the shed so that they can be blown and turned on to both sides of the Taytay Creek. The warning lamp is installed on the right bank of the shed considering the visual recognition from the right bank direction of Taytay Creek.
Warning Light	LED System Rotating Reflector or Flashing Lamp	Same as MCGS
	Location: Left and right banks of the sluiceway Installation Quantity: 2	The warning lamp is installed on the right bank side of the local control house considering the visual recognition from the right bank direction of Taytay Creek.
Equipment Classification	Monitoring Equipment (CCTV Camera)	
Monitoring System	HD Simple IP Camera Device	· Same as MCGS
Monitoring Equipment Layout	Location: Left and right bank of the gate and upstream and downstream of the sluiceway Installation Quantity: 4 units in total	· Like the Cainta Floodgate.
Equipment Classification	Electrical Equipment (Emergency Power Supply)	
Expected Operating Time	Gate Equipment: 1 Hour	· It is assumed that the gate is opened and closed once a day during the blackout period.
	Control Equipment: Not Considered	· In the event of a power failure, the staff standing by at the Cainta Floodgate generator house perform the operation on the machine side, so that it is not considered to keep the remote control equipment on standby for the entire period during the power failure.
Generator	Horizontal Synchronous Generator	· Same as MCGS
Generator Capacity	37.5 kVA	· Calculating the output of the load, type and starting method, presence or absence of fire-fighting related load, type of prime mover, etc., and selecting the one that is closest to the standard efficiency table of generators ("Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)")
Motor	Diesel Engine	· Same as MCGS
Motor Capacity	37.2 kW	· Calculating the output of the load, type and starting method, presence or absence of fire-fighting related load, type of prime mover, etc., and selecting the one that is closest to the standard efficiency table of generators ("Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)")
Amount of Fuel Oil Stored	14.5 liters -> Supplied from the Generator Tank	· Computation in accordance with the "Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting)"

Source: Study Team

7.6 Building Works Design

7.6.1 Building Structural Design

Here, structural design of local control houses on the operation decks and generator house is performed. In building structural design, structural calculation is carried out considering floor load, wind load, and earthquake load, and necessary cross-sectional dimensions and reinforcement arrangement dimensions are set. For each load condition is set as shown in **Table 7.6.1**.

Table 7.6.1 Loading Conditions in Building Structural Design

Item	Condition			Verification
Floor Live Load	Generator Room Electric Room Staffroom Toilet/Shower Local Control House	Storage-Heavy Storage-Light Office-Offices Restrooms Storage - Light	12.0 kN/m^2 6.0 kN/m^2 2.4 kN/m^2 2.4 kN/m^2 6.0 kN/m^2	Set from the Category indicated in NSCP 2010
Wind Load	Use of simplified wind pressure Reference wind speed: 200 kph (Zone 2)			Set from NSCP 2010
Seismic Load	Use seismic force from static analysis Seismic coefficient Z: 0.4 (Zone 4)			Set from NSCP 2010
Horizontal Seismic Force to the Local Control House on the Operation Deck	The horizontal force acting on the local control house is calculated by the vertical distribution formula in NSCP.			Since seismic design of civil works structure applies static analysis, it is considered separately.

Source: Study Team

7.6.2 Building Service Equipment

7.6.2.1 Plumbing

Plumbing systems are designed by National Plumbing Code of the Philippines and DGCS Volume 6 – “Public Buildings and Other Related Structures”. Here, the water supply equipment to the MCGS generator house without the water supply pipe in the circumference and waste water treatment by the septic tank are examined.

7.6.2.2 Ventilation and Air Conditioning

Location and policy of ventilation and air conditioning on the building works in this project is indicated in **Table 7.6.2**.

Table 7.6.2 Installation Policy of Ventilation and Air Conditioning Equipment in Each Facility

Facilities		Room Name	Design Policy for Ventilation and Air Conditioning
MCGS and Cainta Floodgate	Local Control House	-	Same as existing flood gates in the Philippines, no mechanical ventilation installed, while passive ventilation is considered in architectural design.
	Generator House	Staff Room	Air conditioners will be installed to allow staff to stay long.
		Electrical Room	Ventilation equipment will be installed to keep the room temperature at the same level as outdoors.
		Generator Room	Ventilation equipment will be installed to accommodate generators. (* Exhaust Duct from the generator are installed by mechanical works.)
Taytay Sluiceway	Local Control House	-	Ventilation equipment is provided for generator and electrical equipment.
	Guard House	-	No air conditioning is considered following local practices for other guard houses.

Source: Study Team

In accordance with the National Mechanical Code of the Philippines and DGCS Vol. 6, the ventilation system for the electric room and the generator room has been studied. In addition, for small staff rooms, air conditioning equipment is calculated on the assumption that general-purpose air conditioning equipment for domestic use is installed.

7.6.3 Building Electrical Equipment

Since the Philippine National Standards do not specify the details of lightning protection systems, it is customary to design lightning protection systems by applying the relevant international standards. In this section, a lightning rod is examined with reference to NF C -17 -102 "Early streamer emission lightning protection systems".

Regarding lighting equipment, in accordance with DGCS Vol. 6, illuminance 200 ~ 300 (lux) would be adopted.

7.6.4 Other Details

Other details include the management staircase, the regulations required for the fuel storage room, and the architectural design of the local control house.

CHAPTER 8 HYDRAULIC MODEL EXPERIMENT (ABSTRUCT)

8.1 Outlines of the Hydraulic Model Experiment

8.1.1 Purpose of the Hydraulic Model Test

The purpose of the hydraulic model experiment in the detailed design study for the implementation of Pasig-Marikina River Channel Improvement Project, Phase IV, are as listed below:

- To set the gate opening/width of MCGS that could ensure proper design discharge distribution during floods (To set the proper gate opening);
- To set the optimum river alignment and the gate opening/width of MCGS that could minimize turbulent flow at the upstream and downstream (To set the proper river shape);
- To confirm the relationship between the gate opening of MCGS and the discharge distribution of the Manggahan Floodway and the main river in flood conditions including design flood and excess flood (Confirmation of discharge distribution after construction); and
- To validate the temporary channel that could ensure the safety of the main river during the construction period (Confirmation of flood phenomenon during the construction period).

8.2 Results of Model Experiments

8.2.1 Diversion Characteristics of Existing Channel

The relationship of discharge between Marikina River (under the existing condition) and Manggahan Floodway is summarized in **Table 8.2.1**. As a result, diversion ratio for Marikina River is almost constant at about 20% under the existing condition. Under the existing channel, the diversion discharge to Marikina River, 585m^3/s, exceeds the design discharge, 500m^3/s, when inflow design flood discharge is 2,900m^3/s.

Table 8.2.1 Diversion Ratio of Existing Channel

Inflow Discharge (m^3/s)	Lower Marikina River (m^3/s)	Manggahan Floodway (m^3/s)	Diversion Ratio (%)	Remarks
2,147	437	1,710	20.3%	Maximum in past 5 years
2,900	585	2,315	20.2%	Design Flood Discharge
3,480	711	2,769	20.4%	Typhoon Ondoy (2014 JICA Study)
3,898	791	3,107	20.3%	Bank-full discharge (Upstream end of the model)

Source: Study Team

8.2.2 MCGS Specifications Determined by the Hydraulic Model Experiment

8.2.2.1 Specifications of MCGS Gates

The specifications of MCGS gates to achieve the designed diversion ratio under the condition of design flood discharge were determined as shown in **Table 8.2.2**.

Height of gate was determined as DFL EL.17.4m, in which condition discharge of Lower Marikina River does not increase significantly due to small amount of overflow from fully-closed gate even in excess flood.

Table 8.2.2 Gate Specifications Determined by the Hydraulic Model Experiment

Narrower span gate		Long Span Gate	
Width	Crown Height	Width	Crown Height
11.7m	EL.17.4m（DFL）	28.3m	EL.17.4m（DFL）

Source: Study Team

8.2.2.2 Energy Dissipator and Bed Protection Works

It was confirmed that high flow velocity, about 8m/s, is generated at the downstream of MCGS under the condition of design discharge, 500m^3/s.

As a countermeasure, velocity at Sta.5+950 was reduced to 2 to 3m/s by improving the energy dissipator with the installation of L-type sill, 2.0m in height. Note, however, that this countermeasure restricts ships that could pass through the narrower span gate. Considering that this restriction will not be a great hindrance in navigation due to the wider span gate available for it in ordinary time, the priority shall be given to the improvement of flow condition.

8.2.3 Diversion Characteristics of Planned Channel

Diversion characteristics of the channel after improvement by PMRCIP IV are as shown in **Table 8.2.3**.

Table 8.2.3 Diversion Ratio of Existing Channel Ratio of Planned Channel

Inflow Discharge (m^3/s)	Lower Marikina River (m^3/s)	Manggahan Floodway (m^3/s)	Diversion Ratio (%)	Remarks
2,900	488	2,512	16.8%	Design Flood Discharge
4,000	595	3,405	14.9%	Excess Flood (200-year flood)
4,000	1,140	2,860	28.5%	Only Long Span Gate Fully Opened
4,000	1,236	2,764	30.9%	2 Gates Fully Opened

Source: Study Team

8.2.4 Experiment at the Time of Construction

The water level and flow discharge at the section of the MCGS to be constructed had been confirmed by hydraulic software calculated in the computer. As a result, it has also been confirmed that discharge flow at 440m3/s which corresponding to the maximum discharge for the recent 5 years shall safely flow into the downstream stretch during the construction stage of the MCGS. In addition, it was confirmed that the water level corresponding to 440m3/s was EL+14.0m equivalent to water level at 10-year return period flood.

Table 8.2.4 Construction Steps confirmed by the Hydraulic Model Experiment

Construction Step	Concepts	Plan View of Temporary Channel
STEP1	Flow: Left Bank Construction: P2 and P3	Not Presented due the Closed Information
STEP2	Flow: Right Side between P2 and P3 Construction: P1	Not Presented due the Closed Information

Source: Study Team

In the model experiments, the necessary dimensions of flow areas during construction stage were confirmed. As a result, Water level under the condition of target discharge for temporary cofferdam, $440m^3/s$ in all cases of STEP1, STEP 2 and STEP 2-3, is below EL.14.1m.

Based on those hydraulic experiments results, the temporary cofferdams during the construction phases have been designed.

CHAPTER 9 NON-STRUCTURAL MEASURES AND OPERATION, MAINTENANCE AND MANAGEMENT RULES

9.1 Evaluation of Non-Structural Measures

9.1.1 Evaluation of Non-structural Measures Implemented in Phases II and III

The outline of non-structural measures implemented in Phase II and III and the results of the evaluation of past non-structural measures and requests for future activities are as described below.

9.1.1.1 Non-Structural Measures Implemented in Phases II and III

Table 9.1.1 Non-Structural Measures Implemented in Phase II and III

Phase II and Phase III	
(a)	Development of Information Campaign and Publicity (ICP) plans Review of the existing ICP program prepared by the Consultant for Detailed Design
(b)	Conceptualization of design and preparation of information materials
(c)	Community-based explanatory discussions
(d)	Public hearing survey
(e)	Caravan operation involving schools, government officials, barangay officials
(f)	Development of community-based project motivators
(g)	Establishment of community-based information centers
(h)	Media exposure and public relation activities
(i)	Continuous linkages with national/local government units
Phase III	
(j)	Establishment of Websites
(k)	Elaboration of Hazard Maps

Source: Study Team

9.1.1.2 Evaluation of Implemented Non-structural Measures

(1) Results of Evaluation

72% of respondents are aware of these projects. All of them answered that the projects were beneficial for themselves, their families, and communities. The respondents' impression of each non-structural activity was generally positive. However, many of them associate the project with environmental measures such as water quality improvement and waste measures. Considering this fact, it is necessary to emphasize flood control as the main objective of the project in Phase IV.

(2) Implementation Policy of Non-structural Measures Based on the Survey Results

Based on the above survey results, it is identified that the non-structural measures in Phase IV should be executed according to the following policy.

(a) Scrutinize the results of the questionnaire survey and continue/expand the ICP activities to disseminate information to relevant organizations and residents in the Pasig-Marikina River Basin.

(b) Implement massive and extensive non-structural measures to mitigate flood damage.

(c) Reactivate the Flood Mitigation Committee (FMC) to facilitate consensus-building among the concerned member organizations and implement non-structural measures.

(d) Conduct activities aimed at tapping skills, talents in the spirit of volunteerism and by developing human resources for flood damage mitigation.

9.1.2 Reactivation of Flood Mitigation Committee (FMC)

9.1.2.1 Current Status of FMC

The FMC acts as the coordination body in handling issues relating to the PMRCIP implementation as well as Operation and Maintenance (O&M) of flood control facilities and controlling land encroachment and disorderly land development.

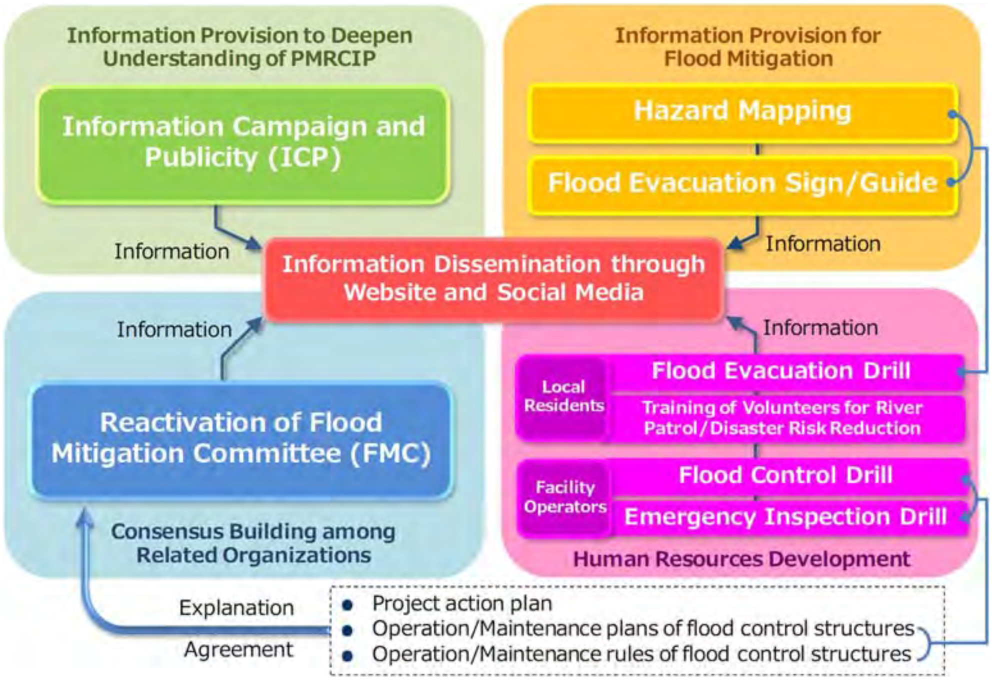
Since the establishment of the FMC in 2013, there has been no record of activities. To implement Phase IV smoothly, measures to reactivate this committee should be taken.

9.1.2.2 Reactivation of the FMC

Since related organizations should approve operation rules and maintenance management plan of flood control facilities and so on which would be prepared in Phase IV before implementing them, it is necessary to reactivate the FMC and facilitate consensus-building among member organizations. Specifically, as stipulated in the MOA at the time of the FMC formation, meetings for project progress briefing and opinion exchange should be convened regularly (at least once every three months) to raise awareness of the FMC and activate its activities.

9.1.3 Concept of Non-Structural Measures in Phase IV

Based on the above, non-structural measures in Phase IV are conceptualized as in **Figure 9.1.1**.



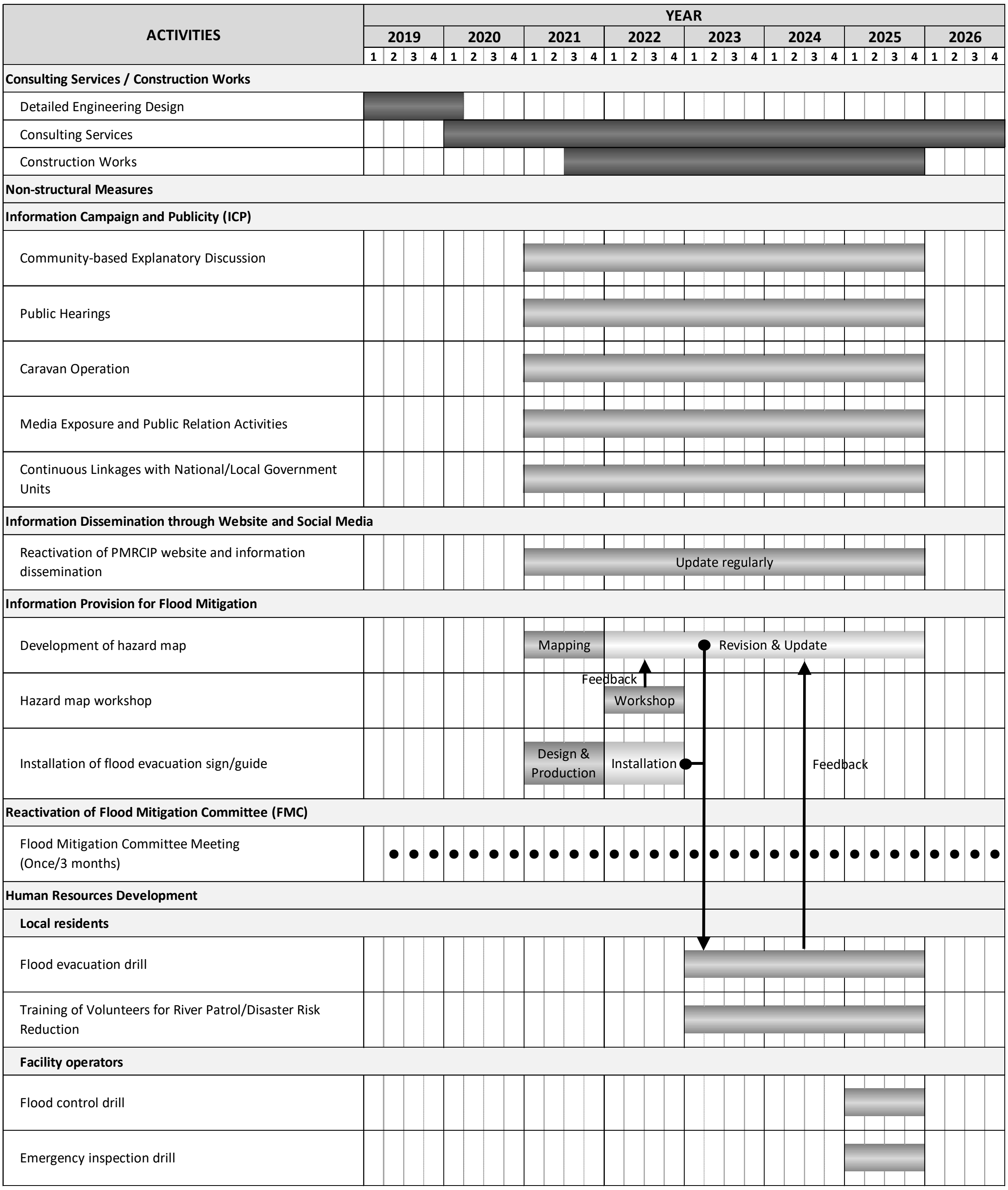
Source: Study Team

Figure 9.1.1 Concept of Non-structural Measures in Phase IV

9.1.4 Action Plan of Non-Structural Measures in Phase IV

Table 9.1.2 shows the timeline of each activity.

Table 9.1.2 Timeline of Each Activity



Source: Study Team

9.2 Operation Rules for Weirs and Watergates

9.2.1 Operation Rules for Existing Structures

9.2.1.1 Rosario Weir and NHCS (Napindan Hydraulic Control Structure)

The operation rules of the Rosario Weir and NHCS are as shown in **Table 9.2.1** and the Operating Rules of Rosario Weir in terms of water level are converted to those in terms of flow rate as shown in **Table 9.2.2**.

Table 9.2.1 Gate Rules for Rosario Weir and NHCS

Condition	Water Level at Sto. Niño	Rosario Weir	NHCS	
While the water level is rising	EL+13.80m	Open Gate No. 4	The main gates of NHCS shall be "closed" as soon as the opening of the Rosario Weir gates is notified.	Basically, the gates must be operated according to the rules on the left column, but there is information that the rules are not strictly followed for the NHCS.
	EL+13.90m	Open Gate No. 5		
	EL+14.0~14.40m	Open Gate No. 3 & 6		
	EL+14.50~15.10m	Open Gate No. 2 & 7		
	EL+15.30m~Up	Open Gate No. 1 & 8		
While the water level is dropping	EL+15.00m	Close Gate No. 1 & 8	The main gates of NHCS shall be "opened" as soon as the end of the gate closing operation of Rosario Weir is notified.	
	EL+14.50m	Close Gate No. 2 & 7		
	EL+14.00m	Close Gate No. 3 & 6		
	EL+13.80m	Close Gate No. 5		
	EL+13.60m	Close Gate No. 4		

Source: Study Team added some information from MMDA-EFCOS Office.

Table 9.2.2 Gate Operation Rules of Rosario Weir in Terms of Flow Rate

Condition	Flow Rate at Sto. Niño	Rosario Weir
While the water level is rising	288 m^3/s	Open Gate No.4
	308 m^3/s	Open Gate No.5
	328 ~ 398 m^3/s	Open Gate No.3 & 6
	419 ~ 552 m^3/s	Open Gate No.2 & 7
	601 m^3/s	Open Gate No.1 & 8
While the water level is dropping	529 m^3/s	Close Gate No.1 & 8
	419 m^3/s	Close Gate No.2 & 7
	328 m^3/s	Close Gate No.3 & 6
	288 m^3/s	Close Gate No.5
	251 m^3/s	Close Gate No.4

Source: Study Team

9.2.1.2 Other Structures

The pumping stations managed by MMDA are installed in Pasig River Basin. Even when the water level in the Pasig River is high, inland water can be drained (unless the facility is submerged). The largest drainage capacity among these pumping stations is 19.0 m^3/s of San Andres Pumping Station installed just before the Pasig River joins the San Juan River, the right tributary.

At each pumping station, the water level at the start and end of operation are defined. It is expected that such pumping stations on the riverside will also be effective in the event of flood. However, no operation is done to stop the drainage pump when the water level of the Pasig River rises up to, e.g., DFL.

9.2.1.3 Evaluation on Operation of the Existing River Structures

(1) Rosario Weir

It is necessary to consider more efficient operation rules that link the two facilities, since the MCGS will be completed through the Phase IV Project.

(2) NHCS

It is necessary to study on the operation of the NHCS because the flow discharge of lower Marikina River will be controlled by the MCGS newly constructed.

(3) Warning Broadcast

In this study, the improvement of warning system has been considered due to the following problems:

- There are no pre-operational broadcasts even though the closing operation causes water level fluctuations upstream and downstream of the weir; and
- There is no broadcast in the event of an excessive flood

(4) Pumping Stations

The pumping stations along the Pasig River are not operated to stop the drainage pump when the main river water level exceeds DFL. Since this operation may lead to increasing the risk of flooding along the riverside, the river water level should be considered:

- Add the river water level as well as the inside one to the reference for stopping the drainage pump.

9.2.2 Basic Concept of Operation Rules for MCGS and Floodgates

9.2.2.1 Fundamental Principles of the Operation

There are no guidelines in the Philippines that illustrate the basic operation rules applicable to the MCGS and the floodgates, etc. Therefore, the fundamental principles of the operation are based on the following Japanese standards and similar examples of diversion weirs and sluices:

- Technical standards for dam and weir facilities (March 2016, MLIT, Japan)
- Standards for the preparation of operation rules for river management facilities (March 9, 2012, Notification by Director, River Environment Division, Water and Disaster Management Bureau, MLIT, Japan)

9.2.2.2 Operational Plan

(1) Operation up to Planned Scale Floods

1) MCGS

It is recommended that the operation rule of the MCGS is shown in the **Table 9.2.3** below in harmonization with that of Rosario Weir.

Table 9.2.3 Proposed Operation Rules of MCGS and Rosario Weir (up to the DFL)

Facilities	Non-Flood Phase	Flood Flow Rate at Sto. Nino (increasing phase)	
		300 m^3/s ~ 600 m^3/s	600m^3/s ~ 2,900m^3/s
MCGS	All gates fully open	All gates fully open	Wider gate fully closed, narrower one fully open (Flow discharge is controlled of up to 500 m^3/s)
Rosario Weir	All gates fully closed	Opening gates sequentially	All gates fully open

Source: Study Team

2) Floodgates to Prevent Backward Flow

The basic operation of the floodgates should be as shown in **Table 9.2.4**.

Table 9.2.4 Proposed Basic Operation Rules for Two Floodgates

Condition	Cainta Floodgate	Taytay Floodgate
Normal	Fully open	Fully open
When the water level of the river/creek and drainage inside the embankment are higher than that of the Manggahan Floodway	Fully open	Fully open
When the water level in the Manggahan Floodway is higher than that of the river/creek and drainage inside the embankment	Fully closed	Fully closed
When the gates are fully closed and the water level inside the embankment becomes higher than that of the Manggahan Floodway	Fully open	Fully open

Source: Study Team

(2) Operation in Excessive Scale Floods

1) MCGS

Based on the comparative study, DPWH has determined that "No Change of the Operation Rule" is the most desirable in excessive floods:

- Operation in Excessive Scale Floods: Not change from the operation rules up to the design flood scale (Wider gate fully closed and Narrower one fully open)

2) Floodgates to Prevent Backward Flow

The operation of the floodgates is basically conducted by the water level in the Manggahan Floodway and that of the river/creek and drainage inside the embankment, so the operation rules are not changed even in the case of an excessive flood.

9.2.2.3 Warning Broadcast

The new contents of the broadcast by the Study Team are proposed below. It is recommended that the same content be broadcasted in Tagalog after English for all broadcasts as it is now.

(1) Rosario Weir

For broadcasting other than during gate operation, the current four types of messages will be reorganized into four types: precaution stage / before opening gates when the river flow increases / emergency stage / before closing gates when the river flow decreases. For the emergency stage, the message will be slightly stronger than others.

(2) MCGS

Unlike the Rosario Weir, since the broadcast is directed toward the Lower Marikina River where there is always water flow, there is no need for a precautionary broadcast. Therefore, there are three types of broadcasting for other than during gate operation: before closing the wider gate when the river flow increases / emergency stage / before opening the wider gate when the river flow decreases. It is the same as the Rosario Weir that a slightly stronger message is broadcasted in the emergency stage.

(3) Floodgates to Prevent Backward Flow

For broadcasts other than during the gate operation, there are two types before the gate closing operation and before the gate opening operation.

9.2.3 Need to Operate the NHCS

9.2.3.1 Policy for Considering the Operation

Normally (in non-flood phase) the MCGS is fully open, and there is no change in the river situation compared with before the construction. Therefore, any change in the operation of the NHCS is not considered necessary.

On the other hand, with the construction of the MCGS, the flow to the Lower Marikina River will be controlled as described in the previous section in the event of flood. Therefore, the necessity of operating the NHCS shall be considered, while there is information that the NHCS has not been actively operated so far.

9.2.3.2 Operational Rule

In the study, the following two cases were considered in order to set the operation plan of the NHCS;

- When the water level in Laguna Lake is higher than that at the main river confluence with Napindan Channel, and
- When the water level in Laguna Lake is lower than that at the main river confluence with Napindan Channel

As a result, the operation rule described in **Table 9.2.5** below is recommended.

Table 9.2.5 Proposed Basic Operation Rules for NHCS

Condition		NHCS
When the water level in Laguna Lake is higher than that at the main river confluence with Napindan Channel	When the water level at Sto. Niño is EL +15.2m or more and there is a necessity to suppress additional inflow from Laguna Lake to the main river	Fully closed
When the water level in Laguna Lake is lower than that at the main river confluence with Napindan Channel	When the water level on the Laguna Lake side reaches EL +11.5m or more and there is a necessity to suppress inflow from the main river	Fully closed
Cases other than the above at the time of flood		Fully open

Source: Study Team

9.2.4 Operation Rules

9.2.4.1 Rosario Weir, MCGS, and NHCS

The concept of operation procedure is shown in **Table 9.2.6**, followed by final draft of operation rules.

Table 9.2.6 Concept of Operation Procedure of Rosario Weir, MCGS, and NHCS

Flow Rate at Sto.Nino	Water Level at Sto.Nino	Operational Phase and Stage	when WL (L: Laguna Lake side) < WL (C: Confluence side)			Other than the Left
			Napindan HCS	Rosario Weir	Manggahan Control Gate Structure (MCGS)	Napindan HCS
Q < 180m^3/s	H < EL+13.0m	Non-Flood Phase		Close all 8 gates	Open all 2 gates	
Q > 180m^3/s	H > EL+13.0m	Flood Phase Precaution Stage		(for the people in the floodway, if necessary) Broadcast R1(15min)		
Q > 300m^3/s	H > EL+13.8m	Caution Stage	Open all 4 gates	Siren and Warning Broadcast R2(30min) Open No.4&5 gates sequentially (Gate operation: 18 minutes +)		Open all 4 gates
Q > 350m^3/s	H > EL+14.0m	Alert Stage	However, Close all 4 gates if WL(L)>EL+11.5m	Warning Broadcast R2(5 min) if any interval Open No.3&6 gates simultaneously (Gate operation: 18 minutes)		
Q > 450m^3/s	H > EL+14.5m	Alarm Stage		Warning Broadcast R2(5 min) if any interval Open No.2&7 gates simultaneously (Gate operation: 18 minutes)		
Q > 600m^3/s	H > EL+15.2m	Critical Stage		Warning Broadcast R2(5 min) if any interval Open No.1&8 gates simultaneously (Gate operation: 18 minutes)	Siren and Warning Broadcast M1(5 min) Close Wider Gate fully (Gate operation: 38 minutes)	Close all 4 gates (32 minutes)
Q > 2,900m^3/s	H > EL+21.17m	Excessive Flood Phase Emergency Stage		Warning Broadcast R3(20 min) Open all 8 gates	Warning Broadcast M2(20 min) Close Wider Gate and Open Narrower Gate	
Q < 2,900m^3/s	H < EL+21.17m	Flood Phase Critical Stage				
Q < 550m^3/s	H < EL+15.0m	Alarm Stage		Warning Broadcast R4(5 min) Close No.1&8 gates simultaneously (Gate operation: 18 minutes)	Siren and Warning Broadcast M3(5 min) Open Wider gate fully (Gate operation: 38 minutes)	Open all 4 gates (32 minutes)
Q < 450m^3/s	H < EL+14.5m	Alert Stage		Warning Broadcast R4(5 min) if any interval Close No.2&7 gates simultaneously (Gate operation: 18 minutes)	Open all 2 gates	
Q < 350m^3/s	H < EL+14.0m	Caution Stage		Warning Broadcast R4(5 min) if any interval Close No.3&6 gates simultaneously (Gate operation: 18 minutes)		
Q < 300m^3/s	H < EL+13.8m	Post Flood Phase		Warning Broadcast R4(5 min) if any interval Close No.5&4 gates sequentially (Gate operation: 18 minutes +)		
Q < 180m^3/s	H < EL+13.0m	Non-Flood Phase		Close all 8 gates		

Note : **Flood Phase** operation starts under the following conditions: if average rainfall (Sto.Nino) > 30 mm/hr, or flow rate (Montalban) > 100 m^3/s
Gate operation velocity = 0.3 m/min, Gate operation time = Lifting height / Operation velocity

Source: Study Team

9.3 Organization and Maintenance Management Plan

9.3.1 Study Policy for Organization and Maintenance Management Plan

9.3.1.1 Need to Draw up Organization and Maintenance Management Plan

To properly fulfill the function of facilities over the long term, it is necessary to ensure the appropriateness, reliability, and safety of operation of each facility such as MCGS and the two floodgates to be constructed in this project as well as the existing structures such as Rosario Weir.

Therefore, it is necessary to make a maintenance plan for each facility including the following items:

- Basic data collection, patrol and inspection methods (monitoring) to grasp the conditions of the facilities on a daily basis, during operation, and when abnormal events occur;
- Methods to maintain the functions of facilities(functional maintenance measures); and
- Recording method of the above activities (maintenance record).

The maintenance management plan shall include not only a maintenance plan described above but also a plan on organizational management structure for determining the size and budget of the organization which implements maintenance management.

9.3.1.2 Standards, Guidelines, etc. to be Applied

The maintenance management plan for the MCGS, the two backflow prevention floodgates, etc. shall be examined by dividing them into civil engineering and building structures such as steel sheet piles and weirs, mechanical facilities such as gates, and electrical and telecommunication facilities necessary to operate them, referring to the following standards and guidelines and similar existing facilities. However, since the terms used in each standard and guideline are very different from each other, the unification of terms in multiple sectors was considered as much as possible when compiling the plan.

9.3.2 Maintenance Management Plan

9.3.2.1 Basic Policy for Maintenance Management

Maintenance management after completion of Phase IV project should be conducted in an appropriate and comprehensive manner by the following **monitoring** and **functional maintenance measures**.

- Conducting data collection and patrols at an appropriate time and frequency commensurate with the content of the completed facilities, and taking necessary measures such as removal of obstacles and dredging to maintain their functions; and
- Carrying out inspections at an appropriate time and frequency commensurate with each component of the completed facility, and taking necessary measures such as maintenance when any deterioration or abnormality such as damage or corrosion is identified.

To achieve the above, the organization responsible for O&M of the facilities shall establish a sufficient system and ensure the necessary budget for the long term.

9.3.2.2 Monitoring

(1) Type of Monitoring

In the maintenance management, it is fundamental to grasp the state of river channels and facilities, and to implement countermeasures according to the results. Items to be carried out to grasp the state of rivers are classified into collection of basic data, river patrols, inspections before the rainy season and after a flood, and inspections of facilities.

(2) Differences between Patrols and Inspections

The purpose of **patrol** is to patrol rivers periodically and systematically and to grasp abnormalities and changes in the round. The purpose of **inspection** is to discover, observe and measure abnormalities and changes in the functions of each facility. Actions requiring prompt initial response upon detection, such as responding to a tort, shall be included in the scope of patrols. Since the contents and accuracy required for grasping the state of rivers are different between river patrols and inspections, it is necessary to carry out appropriately according to the purpose.

(3) Types of Patrol and Inspection

The types of patrols and inspections for civil engineering and building structures, mechanical equipment, and, electrical and telecommunication facilities (electrical equipment and telecommunication one) are summarized in **Table 9.3.1** below.

Table 9.3.1 Types of Patrol and Inspection

Category	Patrol	Periodic Inspection		Operational Check	Extraordinary Inspection
		Monthly	Yearly*		
Civil Engineering and Building Structures	Monthly	-	April	-	After Events (Floods, Earth-quakes, etc.)
Mechanical Equipment	-	Rainy season: Every month Dry season: Every 3 months	April	During Operation	After Events (Floods, Earth-quakes, etc.)
Electrical Equipment	-	Anytime every 3 or 6 months	April	Daily (weekdays)	After Events (Floods, Earth-quakes, etc.)
Telecommunication Equipment			Anytime		

* In principle, Yearly Inspection shall be carried out before Rainy Season except for Telecom Equipment.
Source: Study Team

9.3.2.3 Functional Maintenance Measures

(1) Civil Engineering and Building Structures

Based on the results of periodic or post-flood longitudinal and cross-sectional surveys, or patrols and inspections, if it is judged that changes in the flow capacity and/or the ones in the river bed that affect the safety of the facility may interfere with river management, appropriate measures shall be taken, such as excavation and dredging of the river bed, or construction of a consolidation and filling of scouring areas.

When any abnormalities such as damage or deterioration of the civil engineering or building structure of a facility have been identified through patrols and inspections, the following necessary measures shall be taken to ensure the efficient maintenance and repair of the facility.

(2) Mechanical Equipment

Maintenance shall be carried out in consideration of the function and purpose of the facilities, installation environment, operating conditions, characteristics of the facilities and equipment, etc., and a proper and reasonable maintenance plan shall be formulated for the purpose of maintaining or restoring the functions and ensuring reliability.

Replacement and renewal involves the reinstallation of new equipment or devices in order to ensure the normal function of the facilities in the following cases:

- When it is judged that the reliability and safety cannot be maintained due to deterioration in the functions of the facilities compared to when they were newly installed in spite of proper maintenance management of the facilities; or
- When it is determined that the equipment constituting the facilities has become unable to obtain stable functions and performance due to deterioration such as aging and has reached the end of its life.

(3) Electrical and Telecommunication Facilities

In maintenance, the replacement and adjustment of deteriorated parts specified in advance for each facility shall be systematically carried out, while utilizing the results of inspection and facility diagnosis, and the maintenance and recovery of facility functions shall be attempted by performing prompt and appropriate repairs to any failures or malfunctions that occur.

Many of the facilities and equipment that make up electrical and telecommunication facilities have shorter design life than civil engineering and building structures, and mechanical equipment. Although it is possible to extend the life of the facilities by carrying out thorough maintenance, including the periodical maintenance mentioned above, frequent renewals are required while facilities are operated.

9.3.2.4 Maintenance Record

In order to steadily carry out the maintenance management of facilities, such as the state grasp, analysis and evaluation, and repair and renewal, it is necessary to firstly ensure the preparation and renewal of river ledgers, which are the basic information of the facilities, and at the same time to accurately grasp and record various information concerning the maintenance management of the facilities, such as the inspection results and the evaluation results of soundness, as well as to consolidate important information and advance the creation of a database.

9.3.3 Organizational Management Structures

9.3.3.1 Organizations for Project Implementation and Maintenance

The implementing agency of this project and the manager for each structure are as shown in **Table 9.3.2**.

Table 9.3.2 Proposed Organizations for Project Implementation and Maintenance

Structures	Detailed Design (2019) up to Completion (2026)	Two years after completion (until 2028)	Management Phase (after 2029)
Rosario Weir	MMDA-FCSMO		
NHCS	MMDA-FCSMO		
MCGS	DPWH-UPMO-FCMC	DPWH-UPMO-FCMC	MMDA-FCSMO
Floodgates	DPWH-UPMO-FCMC	DPWH-UPMO-FCMC	DPWH-Region IV-A
Dikes and Revetments	DPWH-UPMO-FCMC	DPWH-UPMO-FCMC	MMDA-FCSMO

Source: Study Team

9.3.3.2 Expansion of Organizational Management Structure

(1) DPWH

With regard to DPWH Region IV-A, which will possess the two floodgates for the prevention of backflow and constructed in this project, a necessary system for the procedures shall be ensured so that the management entrusting to MMDA can proceed smoothly.

(2) MMDA

1) EFCOS Office

To properly operate and maintain the facilities such as the MCGS to be newly constructed in this project, the existing Rosario Weir and NHCS, and the two backflow prevention floodgates which will be entrusted with management from DPWH in cooperation with each other, it is proposed that the operation and maintenance structure of the EFCOS Office of MMDA will be expanded as shown in **Table 9.3.3**.

Table 9.3.3 New Personnel required for MMDA-FCSMO-EFCOS (Clerical and Technical positions)

<table>
<tr><th rowspan="2">Designation</th><th rowspan="2">Common Works</th><th colspan="2">Works Specific to Each Designation</th><th rowspan="2">Current Staff</th><th rowspan="2">Addi-tional Staff</th></tr>
<tr><th>Mainly in the Rainy Season</th><th>Mainly in the Dry Season</th></tr>
<tr><td>Clerk</td><td rowspan="3">O&M, patrol, inspection, (structure)</td><td colspan="2">Budget request and expenditure including Rosario Weir and NHCS</td><td>1</td><td>2</td></tr>
<tr><td>Civil Engr.</td><td>Hydrological analysis etc.</td><td>Maintenance Works (Civil)</td><td>2</td><td>2</td></tr>
<tr><td>Archi. Engr.</td><td>Record of operation, Public relations</td><td>Maintenance Works (buildings and equipment)</td><td>1</td><td>2</td></tr>
<tr><td>Mech. Engr.</td><td rowspan="3">Operation, inspection, simple maintenance</td><td>Troubleshooting (gates)</td><td>Maintenance (gate equi)</td><td>1</td><td>3</td></tr>
<tr><td>Elect. Engr.</td><td>Troubleshooting (electrcl/obsrvtin)</td><td>Maintenance (elect. / observ. equipment)</td><td>2</td><td>2</td></tr>
<tr><td>Tele. Engr.</td><td>Troubleshooting (telecom)</td><td>Maintenance (telecom equip)</td><td>1</td><td>3</td></tr>
</table>

Current staff: The actual number of staff as of 2019
Additional staff: The number of staff to be added to the actual number as of 2019
Source: Study Team

2) First East Metro Manila Flood Control Operation District

In addition to continuing to maintain the river channel with a total length of about 8 km, it is necessary to coordinate and cooperate with local LGUs to newly maintain the revetments and dikes, including steel sheet piles. It is proposed to expand the number of personnel by about three (3).

CHAPTER 10 SOCIO-ENVIRONMENTAL CONSIDERATIONS AND RESETTLEMENT PLANS

10.1 Socio-Environmental Considerations

10.1.1 Review of EIS, EMP and EMoP

In 1998, the DPWH prepared an environmental impact assessment report (EIS) covering all of the Pasig-Marikina River Channel Improvement projects (Phase-II to V) and submitted it to the Department of Environment and Natural Resources (DENR), which issued an environmental conformity certificate (ECC)) in the same year. In Phase-IV, the DPWH prepared a Supplemental EIS (published in August 2018), an updated version of the 1998 EIS, due to the changes in social and environmental conditions around the project site. The Supplemental EIS consisted of an environmental management plan (EMP) and an environmental monitoring plan (EMoP) based on the performance in Phase-III. This was because no major environmental management issue has been reported in Phase-III. The review results on the main items are as summarized below.

(1) Air Quality

There was some concern regarding the concentration of construction dust (TSP), NO_2 and SO_2, but they have not exceeded the standard values in Phase-III. In Phase-IV, the implementation of appropriate mitigation and monitoring plans shall be decided in line with the results of EMP and EMoP.

(2) Water Quality

The effect of turbid water due to dredging is assumed, but the area affected by flowing water is limited, and there will be no effect on the downstream areas. The dredging method was examined to see if dredging can be carried out without diffusion of turbid water. Water pollution by domestic wastewater in the Manggahan Floodway was also confirmed, and there was some concern that soil and sand may flow out, although the scale of construction is small. Consensus must be formed with the Laguna Lake Development Authority (LLDA), which manages the lake located downstream.

(3) Sediment and Soil

The results of the dredged soil sampling survey in Phase-III show that hazardous substances were seldom detected in the sediment of the Marikina River. There were no factories or other facilities that may generate hazardous substances in the area covered by Phase-IV and the upstream area, and no hazardous substance was used in the construction work. For this reason, it is believed that there will be no soil contamination by hazardous substances. To confirm this in Phase-IV, sediment testing for hazardous substances shall be performed (refer to **Section 10.1.3.1** for details).

(4) Wastes

A new disposal site for dredged soil has been planned. Once the possible site is determined, an EIA is to be performed to obtain the ECC. The contamination of dredged soil could be confirmed in the treatment process, but the contamination risk is considered to be low as described in item (3) above (refer to **Section 10.1.3.2** for disposal site).

(5) Noise and Vibration

There were houses and busy roads around the construction area, and the baseline noise has already exceeded the environmental standard value in the country. The impact can be avoided by maintaining a safe distance between the construction sites and the surrounding structures. However, since it is assumed that structures may be located nearby, the impacts shall be confirmed through monitoring and additional measures taken as necessary. During the construction works, the method with small effect of noise and vibration shall be adopted (waterjet method, etc.). As for vibration, since the baseline value was not confirmed in the Supplemental EIS, it should be measured and grasped before the construction. In Phase-III, no complaint due to construction vibration has been reported.

(6) Protected Areas and Ecosystems

The project sites are urbanized, and no protected area, rare terrestrial or aquatic organisms requiring protection has been identified in the EIS. However, in order to properly conserve watershed ecosystems, monitoring shall be carried out, especially for aquatic organisms, including rare and non-rare species. It is considered that most of the trees were planted by the residents, and tree-cutting will be required for bank protection and embankment construction. The number of trees to be felled shall be determined through a tree inventory survey, and greening measures including tree planting shall be implemented in accordance with the laws and regulations of the Philippines, as conducted in Phase-III.

10.1.2 Revision and Update of EIS, EMP and EMoP

Based on the results of the sediment survey, the dredged soil disposal site and the inventory survey for the cutting of trees, the EMP and EMoP shall be updated in the construction stage. In addition, items and activities necessary for mitigation measures and monitoring plans shall be added in a timely manner, taking into account the opinion of stakeholders including the Environmental and Social Safeguard Department (ESSD) which oversees environmental and social considerations of the DPWH, as well as the DENR and relevant local governments.

10.1.3 Support on the Implementation of Socio-Environmental Considerations for Dredged Soil

10.1.3.1 Riverbed Sediment Survey

As a part of the prior monitoring in the project (Phase-IV), in order to investigate and evaluate the toxicity of riverbed sediment dredged or excavated, a verification study on dredged and excavated soil to be treated by dredging was conducted. Thirty-two (32) riverbed sediment samples were collected along the lower reaches of the Marikina River. Sediment and soil samples from 32 sites were analyzed in both leaching tests such as Elutriate and TCLP (Toxicity Characteristic Leaching Procedure) tests. All the substances tested by both methods were undetected or under the standard values of the Philippines. The concentrations of Thirty-two (32) sediment samples collected in the middle- and down-stream sections of Marikina River were lower than the standards set by the Philippine government. This indicates that the sediment in the area is not harmful and may not produce significant levels of toxicity in river water during dredging. In addition, according to the analysis results, the concentrations of heavy metals extracted from the sediment sample in the TCLP test were very small or not detected by the analyzer generally used, that is, the concentrations shall be below the method detection limit. They were not at a significant concentration level exceeding the hazardous waste regulation values (DAO 2004-36/ RA6969). Compared to the regulatory limits for hazardous wastes, the observed concentrations were much lower than the reference values, and it is assumed that those concentrations would give no affections to the planned landfill site and the surrounding environment. In water quality, heavy metals and other harmful inorganic and organic substances were not present at concentrations considered harmful. Therefore, we consider that the dredged sludge (sediment) collected in the middle and downstream sections of Marikina River is safe for use for embankment and landfill purposes.

In addition, the most updated survey of water quality in the Marikina River was also conducted. Three (3) sampling sites for collecting river water are together with those for riverbed sediment survey. A Van Dorn water sampler was used to sample river water. Three river water samples were taken from 3 different locations. The results were compared with the reference values of Class-C specified by DENR. Biological Oxygen Demands, or BODs were above the reference value (7) at two of the three sampling points (upstream and midstream). Dissolved Oxygens, or DOs were also below the reference value (5) at two locations (upstream and downstream). Total Suspended Solids, or TSSs which indicate degrees of turbidity also exceeded the standard value (80) at two locations (upstream and middle). In addition, regarding the number of E. coli, based on DENR Administrative Order No. 34 (1990), the total number of E. coli must not exceed 5,000 MPN/100 ml in the average monitoring period of three months. It greatly exceeded this allowable average value. It should be noted that the total E. coli count results are very high. Bacterial contamination can be caused by domestic household wastewater, commercial wastewater from various tributaries, and even industrial wastewater.

Based on the above considerations and the fact that there would be no source likely to affect the design/ project site of the project and the upstream area, we evaluate that there are also no special environmental considerations in Phase IV.

10.1.3.2 Dredged Soil Disposal Site

The assumed amount of disposed soil to be dredged and excavated due to river channel expansion work in this project is likely about 1.5 million m^3. It is necessary to prepare a specific land to dispose of this large amount of sediment. Therefore, DPWH consulted with the local government and the Laguna Lake Development Authority (LLDA), which has many idle lands, and received a proposal to use a 57 ha of idle land located in the barangay San Juan in Taytay Municipality (refer to **Figure 10.1.1**).

LINES	BEARING	DISTANCE
1-2	S 41d9' E	215.11m
2-3	S 22d12' E	158.55m
3-4	S 30d11' E	24.63m
4-5	N 70d33' E	42.55m
5-6	S 5d9' E	125.63m
6-7	S 15d55' W	448.90
7-8	N 66d5' W	820.62m
8-9	N 77d41' W	402.63
9-10	N 26d22' W	63.67
10-11	N 15d25' E	257.08
11-12	N 0d51' W	47.31
12-13	S 33d21' E	318.93
13-14	N 59d6' E	508.75
14-15	S 56d2' E	105.96
15-16	N 85d7' E	76.61
16-17	N 52d24' E	60.85
17-18	N 26d53' E	43.32
18-19	N 54d0' E	50.49
19-20	S 65d55' E	20.90
20-1	N 42d53' E	85.60

Source: Study Team, based on the data from LLDA

Figure 10.1.1 Potential Landfill Site for Sediment Disposal (LLDA-managed District in Barangay San Juan, Taytay Municipality)

After discussions with LLDA and related organizations, approval was given for conducting an Environmental Impact Assessment (EIA) on the idle land. For this reason, an EIA study has been conducted to obtain an Environmental Compliance Certificate (ECC) for the use of this land as a landfill site from the Department of Environment and Natural Resources (DENR).

As early as possible, the DPWH shall apply for the ECC based on the results of EIA study.

10.1.4 Pre-confirmation of Tree Inventory Survey

In accordance with the Philippine laws and regulations, Pre-confirmation of Tree Inventory Survey was conducted in the Study. The survey started in December 2019 after completion of the basic design and completed in March 2020. The results of the survey were summarized below.

(1) Along Marikina River

A total of 2,066 trees were assessed on both East and West bank of the Marikina River's project area. Specifically, East bank consists of 372 individual tree species while the West bank of the river contains 1,694 trees. In terms of crops, the project area consists of 50 different crop species assessed from 8 farms along both of the riverbanks. Aside from the 15 coconut trees, different crops such as bamboos (84 clumps), banana (50 individual) and various vegetable crops were also noted in the area.

A predicted compensation cost for the affected trees and coconut crops were determined with the guide of relevant policies and regulations. This shows that 736 trees are applicable for earth-balling, 132,250 tree seedlings and 15 coconut seedlings are projected for the replacement on the trees and coconuts to be subjected for cutting. In addition, an amount of Php. 1,500.00 must be provided to the Philippine Coconut Authority as an application fee for the compensation of the affected coconuts. The concerned CENRO (DENR-NCR) will issue the Tree Cutting Permit and/or Earthballing permit which will indicate the exact number of trees to be cut and number of seedlings to compensate based on the analysis of the appropriate infrastructure plan on the result of their ocular inspection.

(2) Along the Manggahan Floodway for Cainta and Taytay Floodgates

A total of 315 trees representing 35 species were surveyed; 121 trees were accounted in Taytay area and 194 trees were in Cainta area.

In terms of crops, the Taytay area consists of 16 different crop species; bananas (147 individuals) and various vegetable crops were noted in the area while Cainta has 18 different species of crops. Bamboo (7 clumps), banana (65 individuals), and other various vegetable crops were accounted in the area. A total of 12 coconut trees were surveyed in both areas; 3 are found in Taytay and 9 are located in Cainta.

Indicative total of 11,300 seedlings will be replaced for the affected 122 trees. Of the 122 trees, 18 are planted trees which corresponds to the replacement of 900 (preferably indigenous species) seedlings while the 104 naturally grown trees and premium species shall be replaced by 10,400 (strictly indigenous species) seedlings. There 193 tees classified as healthy and with DBH<15cm suggested for earth-balling. In terms of coconut, a total 12 coconut trees accounted in both survey areas, hence, estimated 12 seedlings for the replacement. An indicative amount of Php 1,200.00 application fee and additional appropriate costs for processing fee must be provided to the PCA Regional Office.

10.1.5 Capacity Improvement Support Seminar of the DPWH in Environmental and Social Considerations

In order that the DPWH can conduct appropriate monitoring activities (including methods for monitoring terrestrial and aquatic organisms) of the project in line with the EMP and EMoP, half-day training web-seminar (workshop) was conducted for staff members of the ESSD and the UPMO-FCMC of the DPWH, on July 28, 2020.

In the web-seminar, a workshop was also held: (1) to promote the capacity building of DPWH personnel through this seminar with regard to the environmental monitoring report (quarterly in the prescribed format), which is required to be submitted to JICA, and (2) to decide on how to report the status of environmental and social considerations in the Project Status Report.

10.2 Resettlement Plan

The project involves the acquisition of approximately 12.4 ha of land and the relocation of seven (7) utilities, as well as the relocation of 9,327 Informal Settler Families, or ISFs. The procedure for land acquisition and resettlement under the project is based on the resettlement plan prepared in accordance with the domestic procedures and the JICA Guidelines for Environmental and Social Considerations. No particular oppositions to the implementation of the project have been confirmed during the residents' consultations concerning the project.

As of July 2020, in parallel with the detailed design, the Study Team has been supporting the revision of the above-mentioned Resettlement Action Plan (RAP) and annual budgeting scheme developed by DPWH. No specific oppositions to the project have been confirmed at present in the consultations with the residents. The RAPs shall be finalized by the commencement of the construction of the Project.

CHAPTER 11 DESIGN CRITERIA

11.1 Objectives of the Design Criteria

This chapter describes a detailed design approach for river structures along the Pasig-Marikina River. The design and calculation basically conform to the design standards of the Philippines. In case there are no design method guidelines or design criteria, or if it is regarded to be safer or appropriate considering the characteristics of the project site, the globally accepted code and standard will be applied.

For the MCGS, sluiceways and weirs, seismic design for Level 1 and Level 2 ground motions will be applied. Therefore, methodologies and analysis methods regarding seismic design will be described in this chapter.

11.2 Technical Codes and Criteria

This project is implemented based on the Yen Loan Agreement and the international bidding system. Therefore, the materials and construction techniques implemented in this project shall follow the latest version of the Philippines, Japan, and major international codes and standards.

11.3 Basics of Design Method

Structures will be designed with maximum stress generated by the combination of the largest loads that affect structures. Concrete and steel structures are designed by the allowable stress design method (ASD)[1].

11.4 Loads

11.4.1 Load Type

All facilities of the project are designed against combination of the loads enumerated below:

- Dead Load (Dead weight of structures)
- Surcharge (incl. active/collision load and dynamic load from equipment)
- Earth Pressure
- Hydrostatic Pressure
- Uplift
- Seismic Load
- Wind Load
- Thermal Force
- Loads During Construction

11.4.2 Load Combinations and Allowable Stress[2]

11.4.2.1 Load Combinations

All structures shall be designed for the largest stresses resulting from the worst combination of loads that may act on the structure at any given condition. For safety reasons, each component of the structure shall be in proportion to bear the critical combination of these forces:

Group I	: Normal condition	: D + L + I + E + H + U + F + O (+T if consider thermal force)
Group II	: Wind condition I	: D + E + H + U + F + W (+T if consider thermal force)
Group III	: Wind condition II	: Group I + 0.3W + WL + LF (+T if consider thermal force)
Group VII	: Seismic condition	: D + Ee + H + U + V + He

Where,

D : Dead load

---

1 DPWH Design Guidelines Criteria and Standards (Vol. II) 4.1Design Methodology / NSCP Vol. II Bridges (ASD) 8.14.1 Design Methods

2 DPWH Design Guidelines Criteria and Standards (Vol. II) 3.1 Loads

L	: Live load
I	: Impact/ dynamic effect of live load
E	: Earth pressure
H	: Hydrostatic pressure
U	: Uplift
W	: Wind load on structure
WL	: Wind load on live load
LF	: Longitudinal force from live load
V	: Seismic load
F	: Flowing water pressure
Ee	: Earth pressure due to earthquake
He	: Dynamic water pressure due to earthquake
T	: Thermal force
O	: Gate operation load

Source: DPWH Design Guidelines Criteria and Standards Vol. II 3.2.1 Loading Combinations
NSCP Vol. II Bridges, Section 3, 3.22 COMBINATIONS OF LOAD

11.4.2.2 Extra Factors in Allowable Stress

The allowable stress will be increased according to the combination of loads listed above. Regarding the extra factor during earthquakes, the concrete structure shall conform to the Philippine design standard, and thus, 33% of Group IV will be applied. For the SSP revetments, design methods and safety evaluations will be carried out following Japanese standards “Specifications for Highway Bridges” and “Design Guidelines of Disaster Recovery Works”. To maintain the consistency from design to inspection, the extra factor of the Specifications for Highway Bridges shall be used, and thus, it shall be 50% in the seismic case.

The following extra factors in allowable stresses shall be applied to the load combinations listed above.

Table 11.4.1 Extra Factors in Allowable Stress

DPWH Design Guidelines Criteria and Standards Vol. II[3]	Group I	none (25%*)
	Group II	25% (40%*)
	Group III	25% (40%*)
	Group VII	33%
Specification for Highway Bridges, Part IV, Road Association of Japan: Substructure 4.1 General [4]	Normal	none
	Seismic	50%
Road Earthwork Guideline, Temporary Works[5]	Temporary work	50%

* If thermal force is considered to design a structure

Source: Study Team referring to DGCS / Specifications for Highway Bridges

11.5 Stability Analysis

The river structures and facilities have been designed in this Detailed Engineering Design Study in terms of the following phenomena and/or considerations;

- Sliding
- Overturning
- Stability of Slope
- Seepage/Piping
- Consolidation Settlement

[3] DPWH Design Guidelines Criteria and Standards Vol. II 3.1 Loads
[4] Specifications for Highway Bridges IV Substructures 4.1 Common
[5] Road Earthwork Guideline (Temporary Works) 2-6 Allowable Stress

11.6 Material Characteristics

11.6.1 Soil Coefficients/Property

The soil factors have been determined by laboratory tests. If there is no data available, those have been set in accordance with the "Specifications for Highway Bridges in Japan" and "Road Earthwork Guidelines in Japan".

11.6.2 Steel Sheet Pile (SSP)

11.6.2.1 Selection of SSP Type

The type of sheet pile shall be determined in consideration of the stress and displacement in each section not to meet an allowable stress and displacement against outer loads. SSPs consist of Hat-shape SP-10H and SP-25H, 45-H, 50-H and U-shape SP-IA to SP-VIL. In case that the calculated stress and/or displacement SSP revetment are/is not less than the allowable value, combined SSP with H-Beam shall be applied to secure the strength of SSP revetment and not to meet an allowable stress and displacement.

11.6.2.2 Section Efficiency

The 20% reduction of rigidness (e.g., Moment of Inertia of Area: I x 0.8) is applied to U-shape SSP. This reduction is caused by the joint efficiency of U-shape SSP during bending load. On the other hand, there is no reduction of rigidness for Hat-shape SSP due to their connecting structural characteristic between SSPs.

11.6.2.3 Structure

When SSP with H-Beam is employed as SSP revetment, welding structure is to be applied.

11.6.2.4 Types and Properties of SSP and H-Beam

As mentioned previously, SSP should conform to SYW295 specified in JIS A-5523 or equivalent with minimum yield strength (Fy) of 295MPa.

11.6.3 Concrete and Reinforcing Bar

The specifications and features of Concrete and Reinforcing-Bars being utilized in this Study shall be conformed with related Philippine's guidelines and codes, such as DPWH Standard Specifications for Public Works and Highways, and/or Philippine National Standard.

11.6.4 Prestressed Concrete

The specifications and features of Prestressed Concrete being utilized in this Study shall be conformed with related Philippine's guidelines and codes, such as DPWH Standard Specifications for Public Works and Highways, Philippine National Standard, and/or the National Structural Code of the Philippines (NSCP).

11.6.5 Structural Steel

Structural steels with minimum yield strength (Fy) =245/295 MPa are specified in JIS A-5526 (for SHK400/ SHK400M / SHK490M) and in JIS G-3101 (SS 400) respectively. In addition, the allowable stress of structural steel shall conform to that of presented in JIS and other standards.

11.6.6 Bar Arrangement Rules

The river structures and facilities being designed in this Study have a wide variety of structural characteristics and features in terms of the objectives and purposes of the installation.

In this connection, the five patterns of bar arrangements are set in this design. These five types are distinguished considering the characteristics of the location in the structure and the seismic resistance of columns and beams (**Table 11.6.1**).

Table 11.6.1 Standard Bar Arrangements (Five Types)

Pattern	Conditions	Target Part of the Watergates, Sluices, And Weir	Notes
A	Concrete cast against and permanently exposed to earth	Box culvert Breast wall, wing wall, connecting wall (L2 seismic design is not required)	

Pattern	Conditions	Target Part of the Watergates, Sluices, And Weir	Notes
B	Concrete cast against and permanently exposed to earth	Bottom slab of the direct foundation, apron (L2 seismic design is required)	
C	Concrete cast against and permanently exposed to earth ⇒Required Level 2 seismic design	Piers and columns (L2 seismic design is required)	Distribution bars are allocated outside the main bars (tie loop)
D	Concrete exposed to earth or weather	Operation deck	
E	Concrete cast against and permanently exposed to earth	Bottom slab of weir, water gate and pile foundation	

Source: Study Team

11.7 Liquefaction Analysis

The liquefaction which occurs in the saturated sandy soil layer significantly affects the behavior of the structure during an earthquake. Thus, liquefaction assessment will be carried out to identify the risk of liquefaction of the soil layer under (around) the planned structure. The assessment has been conducted using F_L (=resistance Factor against Liquefaction), which is calculated by using the information of grain size test obtained from boring data and cyclic triaxial test.

11.8 Design Methods and Countermeasures against Liquefaction

11.8.1 Embankment

The stability of the embankment against liquefaction is assessed by using the arc slip method considering the excess pore water pressure Δu produced by an earthquake motion.

11.8.2 Sluice

If the foundation ground of a sluice liquefies, the strength and supporting capacity of the foundation ground will decline. It may endanger the stability of the structure. For a sandy layer that is determined to liquefy after the liquefaction assessment, the geotechnical parameters of the layer shall be reduced. The seismic performance of box culvert will use the modified parameters. In addition, suitable countermeasures shall be adopted as necessary.

11.8.3 Floodgate and Weir

When the foundation ground of a floodgate or a weir liquefies, there is a risk that the strength and bearing capacity of the foundation ground declines and may jeopardize the stability of the structure.

For sandy layers that are determined to liquefy by the liquefaction assessment, the changes in the geotechnical parameters of the soil layer shall be appropriately adopted. Based on the changed parameters, the seismic performance of columns, piers and box culverts shall be checked. Countermeasures shall be considered as necessary.

11.8.4 SSP Revetment

For sandy layers that are determined to liquefy by the liquefaction assessment, the changes in the geotechnical parameters of the soil layer shall be appropriately taken into account the instruction. To consider the ground deformation caused by the changed geotechnical parameters due to liquefaction, the deformation of the surrounding ground including the SSP will be statically assessed to calculate deformation and the sectional force of the SSP.

In the seismic performance evaluation of SSP revetments, soil deformation due to liquefaction caused by earth/water pressure or changes in geotechnical properties accompanying liquefaction has been considered.

To calculate SSP deformation due to liquefaction, “a gradual increase component of earth/water pressure” will be considered. Similarly, “a vibration component of earth/water pressure” will be applied to calculate the cross-sectional force of SSP.

11.8.5 Special Levees (Concrete Parapets)

For sandy layers that are determined to liquefy by the liquefaction assessment, the changes in the geotechnical parameters of the soil layer shall be appropriately taken into account the instruction.

The stress generated in the structure shall be checked that is less than the allowable stress. Also, the stress generated in the foundation shall be less than the allowable stress. Also, it shall have enough bearing capacity and be safe against sliding and overturning. The subsidence of the foundation shall be less than the allowable degree.

11.9 Seismic Design

In the Philippines, the seismic design methods for river structures have not been fully established. Therefore, existing Japanese design methods (only for Level 1) have been adopted for the revetment design in Phase-I to Phase-III. However, there is a strong expectation of introducing Japanese latest seismic design methods. On the other hand, "DPWH LRFD Bridge Seismic Design Specifications (after this referred to as BSDS)" was issued in the Philippines in 2013. The BSDS describes the measures against Level 2 earthquake motions. New bridges constructed after the publication of the BSDS shall be designed in consideration of Level 2 earthquake motions.

Considering the above-mentioned background, this project will introduce seismic design that takes into account the Level 2 earthquake motions.

Among the river structures to be designed in this project, river dikes have been historically designed and constructed considering only Level-1 earthquake motion. These are longitudinally continuous structures; in view of integrity of safety level along an entire river, it is not desirable to change safety level in certain river sections. Therefore, this project will put the first priority to complete the dike at a consistent safety level, and thus, they shall be designed to satisfy against the Level 1 earthquakes.

On the other hand, floodgate, sluiceway and weir which will be newly designed in this project, will be designed considering Level 2 earthquake motion, since they are independent structures.

Since BSDS is a basic guideline to which the Japanese latest river structure design is applicable, the design has been conducted based on BSDS and Japanese “Performance Based Seismic Design Criteria for River Structures”.

11.10 Building Works

11.10.1 Building Structures in This Project

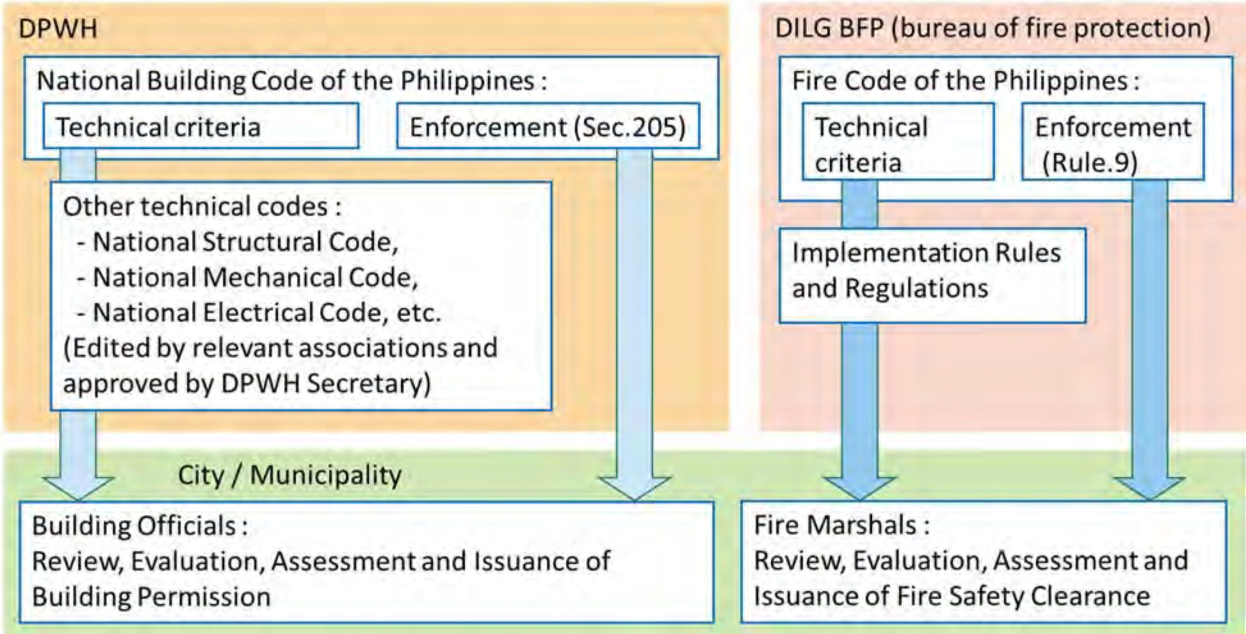
In this project, local control house and generator house of MCGS, Cainta flood gate and Taytay sluiceway are should be designed as building structure. The design of building structures shall conform to the relevant codes and standards used in the Philippines.

11.10.2 Overview of Building Codes and Other Relevant Standards in the Philippines

An overview of building codes and other relevant standards in the Philippines is shown in **Figure 11.10.1**. “National Building Code of the Philippines” (hereafter NBCP), which was authorized by Presidential Decree (Presidential Decree No. 1096), takes an essential role for standard system for building design. DPWH is the ministry responsible for administration and enforcement of NBCP. In addition to NBCP, several technical standards have been established as reference standards (Referral Codes) for NBCPs by relevant professional and authorized under name of DPWH secretary. Some of major ones are;

- ✓ National Structural Code of the Philippines (NSCP),
- ✓ Philippines Electrical Code and
- ✓ Philippine Mechanical Engineering Code.

Besides standard system under NBCP, regulations and technical standards for fire protection are specified in the “Fire Code of the Philippines” (hereafter FC) under Bureau of Fire Protection in Department of Interior and Local Governance.



Source: Survey team

Figure 11.10.1 Outline of the system of technical standards for building structures

Besides standard system under NBCP, regulations and technical standards for fire protection are specified in the "Fire Code of the Philippines" (hereafter FC) under Bureau of Fire Protection in Department of Interior and Local Governance.

CHAPTER 12 PROJECT EVALUATION

In this Chapter, validity of project implementation is evaluated from economic, technical and environmental and social aspects.

12.1 Overall Evaluation of the Project

Economic internal rate of return (EIRR), net present value (NPV) and benefit-costs ratio (BCR) are calculated to evaluate the economic validity of Phase IV projects.

12.1.1 Calculation of Economic Cost

Cost used for economic evaluation is economic cost and transfer items such as taxes are not included. **Table 12.1.1** shows the schedule for disbursement of the economic costs of the project, including the construction of Cainta and Taytay floodgates, and **Table 12.1.1** also shows the economic cost for O&M and replacement.

Table 12.1.1 Financial and Economic Costs for Annual Disbursement and O&M / Replacement

Year	Annual Disbursement (M. P)	
	Financial	Economic
2021	*,***	*,***
2022	*,***	*,***
2023	*,***	*,***
2024	*,***	*,***
2025	*,***	*,***
2026	***	***
2027	0	0
Total	**,***	**,***

Source: Study Team based on Minutes of Technical Discussion on PMRCIP-IV

Year	Economic Cost	Year	Economic Cost	Year	Economic Cost
2026	3	2042	74	2058	45
2027	6	2043	29	2059	63
2028	6	2044	231	2060	21
2029	6	2045	81	2061	83
2030	7	2046	53	2062	71
2031	6	2047	24	2063	243
2032	15	2048	21	2064	106
2033	24	2049	49	2065	69
2034	21	2050	37	2066	10
2035	26	2051	49	2067	8
2036	11	2052	19	2068	18
2037	29	2053	8	2069	94
2038	50	2054	94	2070	325
2039	94	2055	36	2071	385
2040	28	2056	45	2072	414
2041	69	2057	45	2073	325

** The Cost is subject to change after revision of final cost estimation and O&M Plan.*
Source: Study Team based on Minutes of Technical Discussion on PMRCIP-IV

12.1.2 Project Benefits

12.1.2.1 Project Benefits by the Marikina River Improvement

As for estimation of the project benefits, at first the flood damage amounts for each flood probability were estimated. Based on those assumed damages per flood intensity, the annual average damage reduction by the Project was estimated. As a result, annual average damage reduction of Phase IV project is to be 6,682.76 Million Pesos (See **Table 12.1.2**).

Table 12.1.2 Estimated Annual Average Damage Reduction (Phase IV)

Return Period	Flood Damage W/o Project	Flood Damage W/ Project	Reduction	Average (Million P)	Expectation	Benefit (Million P)
2 year	10,710.05	8,836.10	1,873.95			
				4,384.90	0.300	1,315.47
5 year	22,102.87	15,207.02	6,895.85			
				19,202.44	0.100	1,920.24
10 year	50,585.96	19,076.93	31,509.03			
				45,590.56	0.050	2,279.53
20 year	81,967.81	22,295.72	59,672.09			
				70,050.91	0.017	1,167.52
30 year	105,582.70	25,152.97	80,429.73			
					Annual Benefit:	**6,682.76**

Unit: Million *Pesos*
Source: Study Team

12.1.2.2 Project Benefits by the Cainta Floodgate and Taytay Sluicegate

As to the project benefits by the construction of the Cainta Floodgate and Taytay Sluicegate, the those have been estimated in the Implementation Program submitted to NEDA from DPWH in 2008. In this Study, the estimation was reviewed and confirmed by flood simulation analysis with hydrological study.

As a result, annual average damage reduction of Cainta and Taytay Floodgates is to be 360.16 Million Pesos (See **Table 12.1.3**).

Table 12.1.3 Annual Average Damage Reduction (Cainta and Taytay Floodgates)

Return Period	Flood Damage W/o Project	Flood Damage W/ Project	Reduction	Average (Million P)	Expectation	Benefit (Million P)
1.5 year	0.00	0.00	0.00			
				159.70	0.167	26.62
2 year	319.41	0.00	319.41			
				577.22	0.300	173.17
5 year	835.04	0.00	835.04			
				980.45	0.100	98.04
10 year	1,125.85	0.00	1,125.85			
				1,246.70	0.050	62.34
20 year	1,367.55	0.00	1,367.55			
				Annual Benefit:		**360.16**

Source: Study Team

12.1.3 Project Economic Evaluation

Economic internal rate of return (EIRR), net present value (NPV) and benefit-costs ratio (BCR) are calculated using estimated annual average damage reduction of Phase IV project including Cainta and Taytay Floodgates. The result of economic evaluation is as shown in **Table 12.1.4**.

As a result, EIRR became 16.6% which exceeds NEDA's standard of 10%, and it is expected that the Phase IV project is sufficiently effective on flood management in Pasig-Marikina River Basin.

Table 12.1.4 Result of Economic Evaluation (Phase IV Project)

No.	Year	Cost	Benefit	Balance	No.	Year	Cost	Benefit	Balance
-	2021	*,***	0	-*,***	23	2048	21	7,043	7,022
-	2022	*,***	0	-*,***	24	2049	49	7,043	6,994
-	2023	*,***	0	-*,***	25	2050	37	7,043	7,006
-	2024	*,***	0	-*,***	26	2051	49	7,043	6,994
-	2025	*,***	0	-*,***	27	2052	19	7,043	7,024
1	2026	***	7,043	*,***	28	2053	8	7,043	7,035
2	2027	6	7,043	7,037	29	2054	94	7,043	6,949
3	2028	6	7,043	7,037	30	2055	36	7,043	7,007
4	2029	6	7,043	7,037	31	2056	45	7,043	6,998
5	2030	7	7,043	7,036	32	2057	45	7,043	6,998
6	2031	6	7,043	7,037	33	2058	45	7,043	6,998
7	2032	15	7,043	7,028	34	2059	63	7,043	6,980
8	2033	24	7,043	7,019	35	2060	21	7,043	7,022

No.	Year	Cost	Benefit	Balance	No.	Year	Cost	Benefit	Balance
9	2034	21	7,043	7,022	36	2061	83	7,043	6,960
10	2035	26	7,043	7,017	37	2062	71	7,043	6,972
11	2036	11	7,043	7,032	38	2063	243	7,043	6,800
12	2037	29	7,043	7,014	39	2064	106	7,043	6,937
13	2038	50	7,043	6,993	40	2065	69	7,043	6,974
14	2039	94	7,043	6,949	41	2066	10	7,043	7,033
15	2040	28	7,043	7,015	42	2067	8	7,043	7,035
16	2041	69	7,043	6,974	43	2068	18	7,043	7,025
17	2042	74	7,043	6,969	44	2069	94	7,043	6,949
18	2043	29	7,043	7,014	45	2070	325	7,043	6,718
19	2044	231	7,043	6,812	46	2071	385	7,043	6,658
20	2045	81	7,043	6,962	47	2072	414	7,043	6,629
21	2046	53	7,043	6,990	48	2073	325	7,043	6,718
22	2047	24	7,043	7,019					
EIRR =	**16.58%**		**NPV =**	**20,401**		**BCR =**	**1.89**		

12.1.4 Economic Evaluation of Marikina Dam Project

Regarding the economic evaluation of Marikina Dam in the WB2018 UMD FS report, NEDA pointed out that the benefits overlap with the Phase IV projects. For this reason, economic evaluation of Marikina Dam is conducted separately from the Phase IV project to confirm its validity. Therefore, the Economic internal rate of return (EIRR), net present value (NPV) and benefit-costs ratio (BCR) of the Marikina Dam with Retarding Basins are separately calculated using estimated annual average damage reduction of Marikina Dam. As a result, EIRR became 11.8% which slightly less than NEDA's standard of 10%.

12.1.5 Comparison of Economic Evaluation of Phase IV and Marikina Dam

Comparison of economic evaluation of Phase IV and Marikina Dam is as shown in **Table 12.1.5**.

Table 12.1.5 Comparison of Economic Evaluation of Phase IV and Marikina Dam

Item	PMRCIP Phase IV	Marikina Dam
Project Cost (Economic Cost)	**,*** Million Pesos	*,*** Million Pesos
Annual Average Damage Reduction	7,043 Million Pesos	1,034 Million Pesos
EIRR	16.6 %	11.8 %
NPV	20,401 Million Pesos	1,137 Million Pesos
BCR	1.89	1.18

Source: Study Team

12.2 Technical Evaluation of the Project

12.2.1 River Improvement Works

The following works will be implemented in the planned river improvement:

- In the improvement of low water channel, steel sheet pile or HAT-H steel sheet pile which is the reinforced equivalent to steel sheet pile with H-steel is installed by the vibro-hammer method with water jets or the down-the-hole method, and flow area is secured by excavation and dredging in the low water channel.
- The embankment is constructed with banking + revetment or parapet wall, and height with DFL + freeboard and cross section are secured.
- Drainage outlets that flow into the main river are equipped with drains or sluice that enable appropriate wastewater treatment to meet the conditions and prevent backflow during floods.

On the other hand, although there are some points to be noted in the construction of the above structure, the steel sheet pile revetment, flood protection wall (special levee), and drainage work have already been constructed in Phases II and III with no problem. In Phase II and III, the excavated and dredged soil did not contain harmful heavy metals.

12.2.2 MCGS and Cainta and Taytay Floodgates

In the Philippines, many similar structures were built so far such as NHCS and Rosario Weir in the 1980's, locks and floodgates in KAMANAVA region in the 2000's, and there were no problems with construction.
Although there are some points to be noted in the construction of the above structure, there floodgates are structures that can be sufficiently constructed.

12.3 Environmental and Social Evaluation of the Project

The environmental and social evaluation and assessment of the project are described in detail in previous Chapter 10. The outline is as follows.

12.3.1 Environmental Category of the Project

(1) Category Classification and its Basis

The project is categorized as "A" in accordance with the JICA's Environmental and Social criteria.

This project falls under "the trait of being susceptible to influence," listed in the "JICA Guidelines for Environmental and Social Considerations" and is applied for a Category-A project. In particular, more than 10,000 indirectly-affected ISFs currently living within the Manggahan Floodway will need to be relocated, and the detailed design stage will also be required to monitor and support the activities of the DPWH, the project entity, the NHA and the related LGUs.

(2) Environmental Clearance

The Environmental Impact Statement (EIS) of the project was approved by the DENR in June 1998 (refer to Chapter 10). A Supplemental EIS was prepared by the DPWH in August 2018 and will be revised as necessary in this detailed design stage.

12.3.2 Other Assessments

(1) Pollution Control

The impacts by air quality, noise, vibration etc., during construction should be mitigated thorough water sprinkling and dust control measures, periodic maintenance of equipment, installation of temporary walls, etc.

Although muddy water caused by dredging is anticipated to give a limited impact due to flush of running water, the construction is to be considered employing methods such as installing a silt fence mandatory for the construction contractor. The embedded soils were tested and the results revealed that they had no hazardous pollutants. Therefore, they will be re-used as a material for lowland reclamation other than project sites. At present, an EIA survey has been underway to acquire an ECC for approximately 50 has of land under the jurisdiction of LLDA.

(2) Natural Environment

Since the project area does not include a susceptible area such as national parks or their surroundings, adverse impact on the natural environment is assumed to be limited.

(3) Prediction and Assessment of Impacts and Consideration of Mitigation Measures

The EMP and EMoP will be updated based on the results of surveys for riverbed sediment, dredged soil disposal site, and the logged tree inventory as well as the on-going EIA survey for the backfill site and Cainta Floodgate. Necessary items and activities will be reviewed in a timely manner into the EMP and EMoP, based on opinions from the ESSD, DENR and concerned LGUs.

THE DETAILED DESIGN STUDY FOR THE PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE IV)

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ABBREVIATIONS AND ACRONYMS

1952MP	Formulation of Flood Control Plan in Pasig-Marikina River Basin
1975FS/DD	FS Study and Detailed Design for Manggahan Floodway
2002DD	Detailed Engineering Design of PMRCIP
2013III-DD	Detailed Design Study for the Pasig-Marikina River Channel Improvement Project (Phase III)
2015IV&V-FS	Feasibility Study on PMRCIP for Phase IV and V
AASHTO	American Association of State Highway and Transportation Officials
ABC	Approved Budget for the Contract
ACEL	Association of Carriers and Equipment Lessors
ACI	American Concrete Institute
ADB	Asian Development Bank
AIIB	Asian Infrastructure Investment Bank
ASD	Allowable Stress Method
ASDSS	Alloy-Saving Duplex Stainless Steel
ASTM	American Society for Testing and Materials
BAC	Bids and Awards Committee
BC	Box Culvert
B/C	Benefit-Cost Ratio
BDS	Bid Data Sheet
BM	Bench Mark
BOD	Bureau of Design
BOD	Biochemical Oxygen Demand
BOQ	Bill of Quantities
BQ Item	Item of Bill of Quantities
Brgy.	Barangay
BRS	Bureau Research Standards
BSDS	Bridge Seismic Design Specifications
CAAP	Civil Aviation Authority of the Philippines
CRID	Casing Ring bit Inner Drilling Down Hole Hammer
CTIE	CTI Engineering Co., Ltd.
CTII	CTI Engineering International Co., Ltd.
DAO	DENR Administrative Order
DD	Detailed Design
DENR	Department of Environment and Natural Resources
DFL	Design Flood Level
DHWL	Design High Water Level
DFR	Draft Final Report
DGCS	Design Guidelines, Criteria & Standards Volume 3: 'Water Engineering Projects'
DHH	Down-the-Hole-Hammer
DND	Department of National Defense

DO	Department Order
DO	Dissolved Oxygen
D.O.77	Department Order 77
DOF	Department of Finance
DOST-ASTI	Advanced Science and Technology Institute of the Department of Science and Technology
DPWH	Department of Public Works and Highways
DUPA	Detailed Unit Price Analysis
EAM	Equivalent Area Method
ECC	Environment Compliance Certificate
EDC	Estimated Direct Cost
EFCOS	Effective Flood Control Operating System
EIA	Environmental Impact Assessment
EIRR	Economic Internal Rate of Return
EIS	Environmental Impact Statement
EL.	Elevation
ELRD	Environmental Laboratory and Research Division
EMP	Environmental Management Plan
EMoP	Environmental Monitoring Plan
EPA	Environmental Protection Area
ESSD	Environmental Social Safeguards Division
F/C	Foreign Currency
FCIC	Flood Control Information Center
FCMC	Flood Control Management Cluster
FCSMO	Flood Control and Sewerage Management Office
FPM	Flood Plain Management
FMC	Flood Mitigation Committee
FMB	Forest Management Bureau of DENR
FP	Flamework Plan
FRIMP-CTI	Flood Risk Management Project for. Cagayan, Tagaloan and Imus Rivers
FR	Final Report
FS	Feasibility Study
FVR	Fidel Valdez Ramos
GC	General Conditions
GIS	Geographical Information System
GOP	Government of the Philippines
GPS	Global Positioning System
HCDRD	Housing, Community Development and Resettlement Department
HEC-RAS	Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System
ICB	International Competitive Bidding
ICC	Investment Coordination Committee
ICP	Infromation Campaign and Publicity

IR	Inception Report
IEE	Initial Environmental Evaluation
IFB	Invitation for Bids
ISF	Informal Settler Family
ITB	Instructions to Bidders
JBIC	Japan Bank for International Cooperation
JICA	Japan International Cooperation Agency
JICA1990MP	The Study on Flood Control and Drainage Project in Metro Manila
JICA2011 Preparatory Study	The Preparatory Study for Pasig-Marikina River Channel Improvement Project (Phase III)
JICA2014Study	Data Collection Survey on Flood Management Plan in Metro Manila
JIS	Japanese Industrial Standards
JPY	Japanese Yen
JS	Junction Side
JV	Joint Venture
JWA	Japan Water Agency
KOIKA	Korea International Cooperation Agency
LA	Loan Agreement
LARRIPP	Land Acquisition, Resettlement, Rehabilitation and Indigenous Peoples' Policy
L/C	Local Currency
LCC	Life Cycle Cost
LGU	Local Government Unit
LiDAR	Laser Imaging Detection and Ranging
LLDA	Laguna Lake Development Authority
LRA	Land Registration Authority
LRFD	Load and Resistance Factor Design
LRT	Light Rail Transit
MCCB	Molded Case Circuit Breaker; MCCB
MCGS	Manggahan Control Gate Structure
MCM	million cubic meters
MDF/IDF	Main Distributing Frame / Intermediate Distribution Frame
MHHW	Mean Higher High Water Level
MHWL	Mean High Water Level
MLIT	Ministry of Land, Infrastructure, Transport and Tourism, Japan
MLLWL	Mean Lower Low Water Level
MLWL	Mean Low Water Level
MMDA	Metro Manila Development Authority
MOA	Memorandum of Agreement
MP	Master Plan
MRB	Medium Rise Building
MSL	Mean Sea Level

MSHHWL	Mean Spring Higher High Water Level
MSHWL	Mean Spring High Water Level
MWCI	Manila Water Company, Inc.
NCR	National Capital Region
NBCP	National Building Code of the Philippines
NEDA	National Economic and Development Authority
NETIS	New Technology Information System
NGO	Non-Governmental Organization
NHA	National Housing Authority
NHCS	Napindan Hydraulic Control Structure
NK	Nippon Koei Co., Ltd.
NPV	Net Present Value
NSCP	National Structural Code of the Philippines
OC	Open Channel
OCD	Office of Civil Defense
OCM	Overhead, Contingencies and Miscellanous
ODA	Official Development Assistance
OECF	Overseas Economic Cooperation Fund of Japan
OJT	On-the-Job Training
OPP	orthophenyl phenol
PAF	Project Affected Family
PAGASA	Philippine Atmospheric, Geophysical & Astronomical Services Administration
PAP	Project Affected Person
PC	Particular Conditions
PC	Personal Computer
PC	Prestressed Concrete
PCB	Polychlorinated Biphenyl
PD	Presidential Decree
PDB	Power Distribution Box
PLC	Programmable Logic Controller
PMC	Price Monitoring Committee
PR	Public Relations
PVC	Poly Vinyl Chloride
PHIVOLCS	Philippine Institute for Volcanology and Seismology
PHP	Philippine Peso
PIA	Public Information Agency
PMO	Project Management Office
PMRCIP	Pasig-Marikina River Channel Improvement Project
PNS	Philippine National Standard
POW	Program of Works
PRRC	Pasig River Rehabilitation Commission
PSD	Particle Size Distributions

RA	Republic Act
RAP	Resettlement Action Plan
RAM	River Area Management
RCP	Reinforced Concrete Pipe
RD	Record of Discussion
ROW	Right of Way
SAPROF	Special Assistance for Project Formation
SDGs	Sustainable Development Goal
SEA	Strategic Environmental Assessment
SNS	Social Networking Service
SP	Steel Pipe
SPSP	Steel Pipe Sheet Pile
SSP	Steel Sheet Pile
STA. Sta.	Station
STEP	Special Terms for Economic Partnership
STP	Sewerage Treatment Plant
SUS	Steel Special Use Stainless
SYW	Weldable hot rolled steel sheet piles
TCLP	Toxicity Characteristic Leaching Procedure
TDS	Total Dissolved Solids
TSP	Total Suspended Particles
TTS	Telegraphic Transfer Selling
TWG	Technical Working Group
UNDP	United Nations Development Programme
UPAO	The Urban Poor Affairs Office
UPMO	Unified Project Management Office
USACE	United States Army Corps of Engineers
USAID	United States Agency for International Development
USEPA	United States Environmental Protection Agency
VAT	Value Added Tax
WB	World Bank
WB2012MP	Master Plan for Flood Management in Metro Manila and Surrounding Areas
WB2018 UMD FS	Feasibility Study and Preparation of Detailed Engineering Design of the Proposed Upper Marikina Dam
WC	Water Code
WJ	Water Jet
WS	Workshop

Units of Measurement

mm	: millimeter
cm	: centimeter
mm	: millimeter
cm	: centimeter
m	: meter
km	: kilometer
g, gr	: gram
kg	: kilogram
t, ton	: metric ton
m^2	: square meter
ha, has	: hectare, hectares
km^2	: square kilometer
l, lt., ltr	: liter
m^3	: cubic meter
s, sec	: second
m, min.	: minute
h, hr	: hour
y, yr	: year
MW	: megawatt
mm/hr	: millimeter per hour
m/s	: meter per second
km/hr	: kilometer per hour
mg/l	: milligram per liter
m^3/s	: cubic meter per second
$m^3/s/km^2$	: cubic meter per second per square kilometer
%	: percent
ppm	: parts per million
x x	: symbol of multiplication (times)
≤ , ≥	: inequality sign (e.g. A≤B means that value A is less than or equal to value B.)
< , >	: inequality sign (e.g. A<B means that value A is less than value B.)
Y, Y, JPY	: Japanese Yen
P, P, PHP	: Philippine Peso
$	: US Dollar

CHAPTER 1 OUTLINE OF THE PROJECT

1.1 Background of the Pasig-Marikina River Channel Improvement Project (PMRCIP)

The Pasig-Marikina River, with a total length of 52.2 km (Manila Bay to Wawa Dam) and a total catchment area of 635 km^2, originates from the southwestern slopes of the Sierra Madre Mountains. The river initially flows toward the West, converges with several branch river systems and, after changing its flow direction to the south along the West Valley Fault at Rodriguez in Rizal Province, finally pours into the Manila Bay. It, therefore, traverses the entire Metro Manila area or the National Capital Region.

The river has two major tributaries, namely; the San Juan River which merges at 7.1 km from the river mouth, and the Napindan Channel which merges at 17.1 km from the river mouth, respectively.

The Pasig-Marikina River is mainly divided into two (2) sections at 17.1 km from the river mouth. The downstream section is called the Pasig River (from the river mouth to the merging point of the Napindan Channel), and the upstream section is called the Marikina River (upper reach of the river from the merging point of the Napindan Channel).

The Pasig-Marikina River also connects with the Laguna de Bay (Laguna Lake) via the Napindan Channel and the Manggahan Floodway. The floodway is manmade and it diverts floodwaters from the Marikina River at 23.8 km from the river mouth (refer to the **Project Location Map**).

Metro Manila (also known as Metropolitan Manila or the National Capital Region), through which the Pasig-Marikina River passes, is composed of 16 cities and one municipality. With the population of over 12 million people in 2015, it is the socio-economic and political center of the Philippines.

A flood control plan of the Pasig-Marikina River which included the Metro Manila area was initially formulated in 1952 under the River Control Section of the then Bureau of Public Works, Department of Public Works, Highways and Communications [presently, the Flood Control Management Cluster (FCMC) of the Unified Project Management Office (UPMO), Department of Public Works and Highways (DPWH)]. The Marikina River Multipurpose Project followed in 1954.

Following several studies on flood control of the Pasig-Marikina-Laguna Lake Basin, the Napindan Hydraulic Control Structure (NHCS) and the improvement works of the Pasig River which consisted mainly of the installation of river walls and the construction of pumping stations as well as dredging, started to be implemented in the 1970’s. To reduce flood discharge in the downstream stretch of the Marikina and Pasig rivers, and to mitigate flood damage in the downstream areas by diverting floodwater into the Laguna Lake, the Manggahan Floodway was constructed in 1988.

Despite the continuous efforts and large investments on flood control and drainage works, further urbanization of Metro Manila has worsened the flooding condition and flood damage as expected. Under such circumstances, the Government of the Philippines (GOP) requested the Government of Japan (GOJ), in 1986, to provide technical and financial assistance for flood prevention in Metro Manila.

In response, the GOJ decided to conduct, through the Japan International Cooperation Agency (JICA), the “Study on Flood Control and Drainage Project in Metro Manila”, which was carried out from 1988 to 1990 (hereinafter referred to as “JICA1990MP”) to formulate a master plan and conduct a feasibility study on the urgent flood control projects selected which include the Pasig-Marikina River Channel Improvement Project (hereinafter referred to as the “PMRCIP”).

However, perennial flooding in Metro Manila continued and the floods in 1998, 2004, 2009, 2012 and 2014 have severely affected the socio-economic condition of Metro Manila. In particular, the flood brought by Typhoon Ondoy in September 2009 had caused tremendous damage to lives and properties. More than 460 casualties were reported with more than 4.9 million of the population affected.

Therefore, the implementation of the PMRCIP has been recognized as essential for the mitigation of flood damage caused by overflow from the Pasig-Marikina river channel. Several follow-up studies and analyses have been undertaken by the GOP and, with the cooperation and assistance of JICA, actual implementation of flood control projects were made in parallel with those undertaken under other international financial institutions.

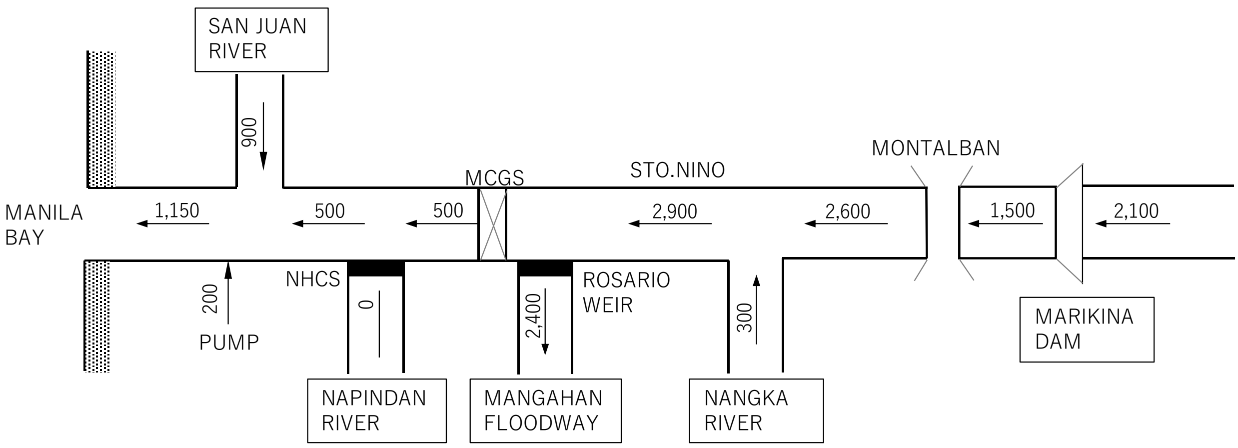
Table 1.1.1 Historical Background of PMRCIP

Year	Name of Study / Project	Description of Contents
1988~1990	(JICA1990MP) Study on Flood Control and Drainage Project in Metro Manila	- To formulate the "Master Plan for Flood Control and Drainage Improvement in Metro Manila" and to conduct a feasibility study on the urgent flood control projects including the Pasig-Marikina River Channel Improvement Project (PMRCIP)
1998	PMRCIP (SAPROF)	- Special Assistance for Project Formation (SAPROF) for "The Pasig-Marikina River Channel Improvement Project (PMRCIP)" - Formulation of the project implementation strategy, dividing the PMRCIP into four phases
2000~2002	PMRCIP Phase I (2002DD)	- Detailed Engineering Design and preparation of bid documents for all phases (Phase I, II, III, IV) - Objectives: river channel improvement from Delpan Bridge to Marikina Bridge, total length=29.7km
2007~2013	PMRCIP Phase II - 2007~2009: Preparatory Stage - 2009~2013: Construction Work Stage (Pasig River)	- Preparatory Stage: Review of detailed design & bid preparation - Construction Work Stage: River channel improvement, Pasig River, from Delpan Bridge to Merging Point with Napindan Channel; Total length of riverbank = 13.1km on each bank
2010~2018	PMRCIP Phase III - 2010~2011: Preparatory Survey - 2013~2014: Detailed Engineering Design - 2014~2018: Construction Work Stage (Pasig River & Lower Marikina River)	- Preparatory Survey: Review of development plan, implementation schedule - Detailed design & bid preparation - Construction Work Stage River channel improvement: (1) Lower Marikina River: from merging point with Napindan Channel to downstream of MCGS; Total length of river channel = 5.4km (2) Pasig River: Remaining section of PMRCIP Phase II, from Delpan Bridge to Merging Point with Napindan Channel; Total length of riverbank = 9.9km on each bank
2019~	PMRCIP Phase IV - 2019~2020: Detailed Engineering Design - (2021~scheduled): Construction Work Stage (Lower/Middle Marikina River)	- Detailed design & bid preparation - Construction Work Stage: (1) River Channel Improvement: Lower/Middle Marikina River: From downstream of MCGS to Marikina Bridge; Total length of river channel = 8.0km (2) Manggahan Control Gate Structure (MCGS) (3) Cainta Floodgate, Taytay Sluicegate

Source: Study Team

1.1.1 Master Plan of Flood Control and Drainage Improvement in Metro Manila

In the JICA1990MP, the target protective level of flood control is set at 100-year return period flood. The proposed structural measures are the river channel improvement works in the Pasig, Marikina and San Juan rivers, including construction of the Manggahan Control Gate Structure (MCGS) at the diversion point of the Manggahan Floodway and the Marikina Multipurpose Dam. The design flood discharge distribution is as shown in **Figure 1.1.1**.



Source: JICA1990MP

Figure 1.1.1 Design Flood Discharge Distribution under the JICA1990MP (100-Year Return Period)

1.1.2 The Pasig-Marikina River Channel Improvement Project (PMRCIP)

In response to the perennial situation of floods hampering the economic and human activities in Metro Manila, the DPWH, with the support of JICA, embarked on the implementation of “The Pasig-Marikina River Channel Improvement Project” (the “Project") targeted approximately 30 km from the estuary to the Marikina Bridge in Sto. Niño district of Marikina City. The Project was divided into four (4) phases based on the results of the study undertaken with funds from the former Japan Bank for International Cooperation (JBIC: presently, JICA) under the Special Assistance for Project Formation (SAPROF) in 1998.

The contents of the four (4) phases are as given in **Table 1.1.2** below.

Table 1.1.2 Phases of the PMRCIP formulated in 1998

Phase	Description of Contents
PMRCIP Phase I	Detailed Engineering Design for Phase II to Phase IV (From Delpan Bridge to Marikina Bridge: 29.7km)
PMRCIP Phase II	River Channel Improvement Works of the Pasig River (From Delpan Bridge to Merging Point with Napindan Channel: 16.4km)
PMRCIP Phase III	River Channel Improvement Works of the Lower Marikina River [From Merging Point with Napindan Channel to Diversion Point of Manggahan Floodway: 7.2km, including the Manggahan Control Gate Structure (MCGS)]
PMRCIP Phase IV	River Channel Improvement of the Middle Marikina River [From Diversion Point of Manggahan Floodway to Marikina Bridge (Sto. Niño): 6.1km]

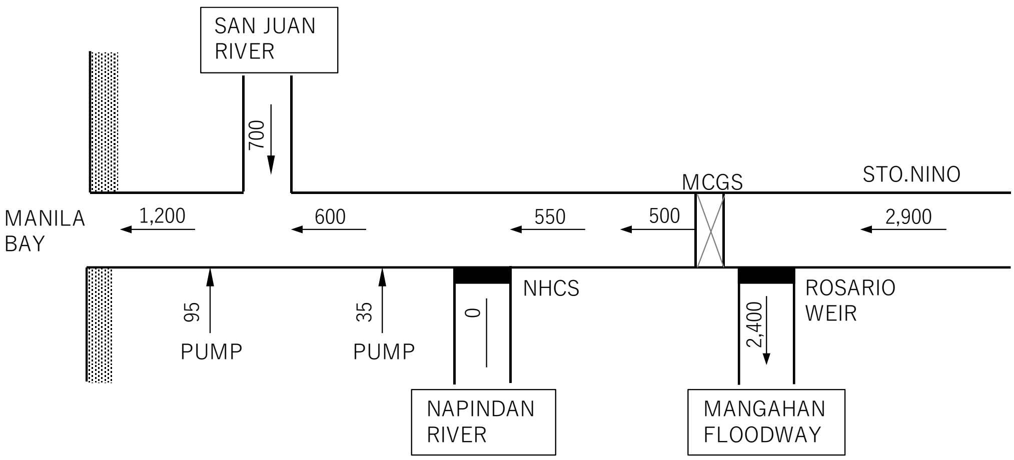
Source: Study Team

1.1.3 PMRCIP Phase I

The PMRCIP Phase I for the detailed engineering design and preparation of bidding documents for all remaining phases (hereinafter referred to as “2002DD”) was implemented from October 2000 to March 2002 based on the Loan Agreement (LA) signed between the GOP and the GOJ in 1999.

In the PMRCIP Phase I, the implementing policies of the PMRCIP were also reviewed. As a result, the target protective level of the PMRCIP was set at a 30-year return period flood without the construction of Marikina Dam.

The provisional design discharge distribution chart set in the PMRCIP Phase I is as illustrated in **Figure 1.1.2**.



Source: Detailed Engineering Design of PMRCIP, Main Report, 2002, DPWH (2002DD)

Figure 1.1.2 Provisional Design Flood Discharge Distribution (30-Year Return Period) set in PMRCIP Phase I

1.1.4 PMRCIP Phase II

The implementation of PMRCIP Phase II commenced in December 2007 under the Special Terms for Economic Partnership (STEP) of Japanese ODA Loans based on the Loan Agreement signed on February 27, 2007.

The PMRCIP Phase II included the review of detailed design due to the changing riverbank conditions, prequalification (PQ) and bidding implementation for the selection of contractor(s) as the preparatory stage, as well as the actual construction work stage.

In addition, a value engineering study was also conducted by the GOP prior to the commencement of actual construction works, under the following reasons:

- Changes of Riparian Conditions due to the development activities undertaken by the Pasig River Rehabilitation Commission (PRRC) and the concerned Local Government Units (LGUs):
    - Revision of Design of Revetment and Drainage System in harmony with the current river parks and riparian conditions.
    - Revision and Confirmation of materials and working methodologies in accordance with the STEP Loan.
- Study on the River Improvement Plan for Upper Marikina River
- Study on the Necessity of Implementation of PMRCIP Phase III and Phase IV, and the Implementation Schedules

After the value engineering and review of design mentioned above, the construction works of the PMRCIP Phase II which consists of two packages, namely, 1-A and 1-B divided by the target stretches, commenced in July 2009, aiming at the completion by 2012. Package 1-B was completed in 2012 while Package 1-A was completed later in June 2013 due to the additional works along the Malacañang Palace sections.

1.1.5 PMRCIP Phase III

Due to the onslaught of Typhoon Ondoy in September 2009, extensive damage was caused in Metro Manila by the overflow from the river channel system of the Pasig-Marikina River. It was then recognized by the DPWH that early implementation of PMRCIP Phase III as one of the urgent measures is essential for the protection of Metro Manila from further flood damage.

Towards this end, a preparatory survey was conducted by JICA from September 2010 to October 2011 on the implementation of the PMRCIP Phase III as a JICA Loan Project. Reviewed in this survey were the implementation plan of all phases of the PMRCIP and the detailed development plan of the target section of PMRCIP III in the Pasig-Marikina River. The main points of review were the most recent riparian and existing revetment conditions affected by land development activities, the recent record of flood damage, and the prediction analysis of flood risk due to climate change. The implementation schedule of the PMRCIP was also reviewed based on the study results, as shown in **Table 1.1.3**.

Table 1.1.3 Preparatory Survey of PMRCIP Implementation Phases (2010-2011)

Implementation Phases	Work Items	Length to be Improved (Design Discharge)
PMRCIP II	Pasig River Channel Improvement (1) (Delpan Bridge to Napindan Channel)	13.1 km on each bank (1,200/600 m^3/s)
PMRCIP III	Lower Marikina River Channel Improvement (Napindan Channel to downstream of MCGS)	5.4 km channel length (500 m^3/s)
	Pasig River Channel Improvement (2) (Remaining Sections between Delpan Bridge and Napindan Channel)	9.9 km on each bank (1,200/600 m^3/s)
PMRCIP IV	Lower/Middle Marikina River & MCGS (Lower Marikina R. (Sta.5+400) - Marikina Bridge)	8.0 km channel length (2,900 m^3/s)
PMRCIP V	Upper Marikina River (Marikina Bridge – San Mateo Bridge)	5.8 km channel length (2,900 m^3/s)

Source: Study Team

The implementation of PMRCIP Phase III was proposed to start as early as possible after the completion of PMRCIP Phase II. The sections to be improved or rehabilitated in PMRCIP Phase III are the sections along the Pasig River where rehabilitation works were not undertaken in PMRCIP Phase II (Total length: 9.9km), as well as the originally targeted sections from the merging point with the Napindan Channel to Sta. 5+400 of the Lower Marikina River (Total length: 5.4km), consisting of the construction of revetment and floodwall and dredging of the riverbed.

The detailed engineering design of the PMRCIP Phase III was conducted under a Grant Aid provided by JICA in 2013 based on the series of discussions between JICA and DPWH during December 5 to 12 in 2011. The Loan Agreement (LA) for the PMRCIP Phase III as a STEP Loan Project was signed on March 30, 2012. The construction works commenced in July 2014 and completed in March 2018.

1.2 PMRCIP Phase IV

1.2.1 Background

The PMRCIP Phase IV include the River Improvement Works in the Middle Stretch of the Marikina River, the Construction of Manggahan Control Gate Structure (MCGS) and the construction of floodgates/sluicegates at the confluence points of the Cainta Creek and Taytay Creek with the Manggahan Floodway.

JICA and the DPWH signed the LA for the PMRCIP Phase IV as a STEP Loan Project in January 2019.

Prior to LA, JICA and DPWH exchanged the Agreement on the detailed design in October 2018, and decided to carry out the detailed design with JICA funds. This study was conducted based on this agreement.

1.2.2 Outline

The contents of the Phase IV Project as a JICA Loan Project under Japanese ODA are as summarized in items (1) to (7) in **Table 1.2.1**.

Table 1.2.1 Outline of the PMRCIP IV Project

No.	Item	Description
(1)	Project Title	Pasig-Marikina River Channel Improvement Project (Phase IV), PMRCIP Phase IV or PMRCIP IV
(2)	Project Objective	To mitigate flood damage in Metro Manila caused by channel overflow of the Pasig-Marikina River, by implementing structural measures together with non-structural measures in consideration of flood management, thereby contributing to the sustainable urban economic development of Metro Manila.
(3)	Date of Signing of LA	January 21, 2019
(4)	Loan Amount	Not Exceeding JPY 37,905 Million
(5)	Contents of the Project	The Measures and Services include: <u>Structural Measures</u> • River Channel Improvement from Sta. 5+400 to Sta. 13+350 (Marikina Bridge at Sto. Niño): About 8 km • Construction of the MCGS: 1 structure • Construction of Floodgates along the Manggahan Floodway: 2 structures (Cainta Floodgate and Taytay Sluicegate) <u>Consulting Services</u> For Structural Measures: • Review of the Detailed Engineering Design • Bid Assistance / Construction Supervision • Support to Environmental Management and Monitoring • Support to Resettlement Actions and Monitoring, etc. For Non-structural Measures: • Formulation of Implementation Plan and Support to Implementation • Analyses to Formulate the Plan
(6)	Target Area	Metro Manila (Marikina River and Manggahan Floodway)

No.	Item	Description
(7)	Implementing Agency	Department of Public Works and Highways (DPWH), GOP
(8)	Agencies/Organizations Concerned	• Metro Manila Development Authority (MMDA) • Local Government Units (LGUs) • The Public Information Agency (PIA) • Department of Environment and Natural Resources (DENR) • Office of Civil Defense (OCD) • Philippine Atmospheric, Geophysical and Astronomical Services Administration (PAGASA) • National Housing Authority: NHA • National Economic and Development Authority (NEDA) • Department of Finance (DOF) • Pasig River Rehabilitation Commission (PRRC) In this DED Study, DPWH as the implementing agency shall coordinate the functions and responsibilities of the above agencies/organizations on matters and issues related to the Project.

Source: Study Team

CHAPTER 2 OUTLINE OF THE DETAILED ENGINEERING DESIGN STUDY

2.1 Objectives of the PMRCIP-IV Detailed Engineering Design Study

As requested by the Government of the Philippines (GOP), particularly, the Department of Public Works and Highways (DPWH), the Detailed Engineering Design Study or the DED Study has been carried out, aiming to prepare the Detailed Engineering Design (DED) and the Draft Bidding Documents of the Pasig-Marikina River Channel Improvement Project, Phase IV (hereinafter referred to as "the Project" or “the Phase IV Project”).

2.2 Outline of the DED Study

The DED Study was in accordance with the agreement between the DPWH and JICA and the internal agreement of JICA in relation to the DED Study in October 2018. The DED study items are as summarized in **Table 2.2.1**.

Table 2.2.1 Summary of Items Studied in the PMRCIP-IV DED Study

No.	DED Study Items of Work
(1)	Collection and arrangement of basic information and data
(2)	Preparation/Explanation of the Inception Report
(3)	Surveys/Investigations of Present Site Conditions
(4)	Basic Design
(5)	Preparation/Explanation of Basic Design Report
(6)	Hydraulic Model Experiment for the Manggahan Control Gate Structure (MCGS)
(7)	Detailed Engineering Design (DED)
(8)	Preparation of Draft Bid Documents
(9)	Verification and Checking of Basic Design and DED
(10)	Assistance/Support on Environmental and Social Considerations for the Project
(11)	Assistance/Support on Resettlement Action Plans for the Project
(12)	Formulation of Non-Structural Measures
(13)	Revitalization of Flood Mitigation Committee
(14)	Formulation of Draft Maintenance Plan for River Structures
(15)	Operation and Maintenance Plans for the MCGS and Floodgates

Source: Study Team

2.3 Designed Target Stretches and Structures

The DED Study target stretches and structures are summarized in **Table 2.3.1**.

Table 2.3.1 DED Study Target Stretches and Structures

Purpose	Objective	Target Stretches / Location	Quantity	Remarks
Detailed Engineering Design for Structures				
	River Improvement	Sta. 5+400~Sta. 13+350	Approx.8 km	Design Discharge The Downstream Section of the MCGS: 500 m^3/s The Upstream Section of the MCGS: 2,900 m^3/s
	Manggahan Control Gate Structure (MCGS)	At Sta. 6+010	1 Structure	Regulation of Discharge toward Downstream Section up to 500 m^3/s
	Cainta Floodgate	At Merging Point between Cainta Creek and Manggahan Floodway (4.55 km downstream from the Rosario Weir)	1 Structure	Design Discharge: 90 m^3/s
	Taytay Floodgate	At Merging Point of Taytay Creek with Manggahan Floodway (6.09 km downstream from the Rosario Weir)	1 Structure	Design Discharge: 28.5 m^3/s (Type: Sluicegate)

Purpose	Objective	Target Stretches / Location	Quantity	Remarks
Formulation of Non-Structural Measures				
	Activities to be taken in PMRCIP IV	Damage Mitigation Activities and Public Information & Educational Campaign	1 L.S.	Based on the Review of Activities in Phase II and Phase III
	Operation and Maintenance Plan	MCGS, Cainta Floodgate and Taytay Floodgate	1 L.S.	Preparation of Integrated Plan with the NHCS and Rosario Weir
Environmental and Social Considerations				
	Review on Existing EMP and EMoP	Proposed Plans and Measures	1 L.S.	Including Soil Quality Analysis and Evaluation by Elutriate and TCLP Test
	Review on Existing RAPS	Proposed Plans and Measures	1 L.S.	Including the EIA and ROW Action Plan for the Cainta Floodgate

Source: Study Team

2.4 Assumed Contents of the Works

The assumed contents and quantities of the PMRCIP-IV works are as given in **Table 2.4.1**, based on the Definitive Plan prepared in 2015 under the PMRCIP-III.

The contents and quantities of each item of work were finalized in the Detailed Design Stage.

Table 2.4.1 Assumed Work Quantities for PMRCIP-IV based on Basic Design

No.	Structures	Descriptions	Assumed Work Quantities
1	SSP Revetment and RC Floodwall	a) HAT-SSP with H-beam	a) 7.1 km
		b) SSP w/o H-beam	b) 3.3 km
		c) Coping Concrete	c) 10.4 km (6,200 m^3)
		d) RC Floodwall	d) 8.4 km (11,800 m^3)
		e) Riprap	e) 10.4 km (203,800 m^3)
2	Reinforcement of Existing Floodwall	a) RC Floodwall	a) 6.1 km (13,000 m^3)
3	Channel Excavation and Dredging	a) Dredging	a) 495,000 m^3
		b) Excavation	b) 1,178,500 m^3
4	Dike/Maintenance Road	a) Embankment	a) 164,000 m^3
		b) Concrete Pavement	b) 8.9km (22,100 m^2)
		c) Concrete Block for Slope	c) 5.4 km
		d) Drainage Ditch	d) 5.9 km
5	Drainage Outlet	a) Box Culvert with Sluice Gate	a) 18 locations
		b) Drainage Outlet with Flap Gate	b) 102 locations
		c) Drainage Outlet w/o Flap Gate	c) 98 locations
6	Bridge Work	a) Construction of New Manalo Bridge (*1)	a) 3 spans (105 m long), PC Girder
		b) Construction of New Cainta Bridge (*2)	b) 2 spans (18 m long), PC Girder
		c) Construction of New Cainta Bridge (*2)	c) 2 spans (12+29 m long), PC Girder
7	MCGS: Roller gate: 2 gates x 20 m (W) x 10 m (H)	a) Foundation Piles	a) 460 piles
		b) RC Works	b) 14,500 m^3
		c) Mechanical & Electrical Works	c) 1 set
8	Cainta Floodgate: Roller gate: 4 gates x7.0 m (H) x 6.0 m (W)	a) Foundation Piles	a) 460 piles
		b) RC Works	b) 14,500 m^3
		c) Mechanical & Electrical Works	c) 1 set
9(*3)	Taytay Floodgates (Sluice): Roller gate: 3 gates x 2.0 m (H) x 2.5 m (W)	a) Foundation Piles	a) 460 piles
		b) RC Works	b) 14,500 m^3
		c) Mechanical & Electrical Works	c) 1 set

*1: Manalo Bridge will be constructed by the DPWH (not included in PMRCIP-IV).
*2: Gate structures play as substructure for bridges
*3: Refer to **Chapter 5**. (In Chapter 5, Sluice Type Structure has been proposed for Taytay Gate.)
Source: Study Team

2.5 Study Policies on the Basic Design and Detailed Engineering Design

The Basic Design was conducted from February 2019 to June 2019. The Detailed Engineering Design, including the studies for the remaining issues on the Basic Design, however, started in July 2017 and was completed in February 2020. The basic and detailed design studies are as briefly described below.

2.5.1 Basic Concepts and Flood Mitigation Plan of the PMRCIP (Chapter 3)

Based on the scheme proposed in the JICA1990MP and with the assistance from JICA, the DPWH will finally complete the river improvement works of the Pasig-Lower Marikina/Middle Marikina River which passes through the core of Metro Manila with the completion of the PMRCIP-IV project works.

The flood mitigation works which started with the construction of the Manggahan Floodway will protect the core of Metro Manila between the Delpan Bridge and the Sto. Niño Bridge from river floods of less than the 30-year return period. The river stretches protected by the dikes and revetments will be 30 km in length after the completion of PMRCIP-IV project (refer to the **Location Map**).

This Detailed Engineering Design Report presents the "across-the-board" review of the flood control/mitigation plans for the Pasig-Marikina River Basin described in **Chapter 3**. The final flood design distribution against a 100-year return period flood is as shown at the end of **Chapter 3**.

2.5.2 Basic Study and Analysis of River Channel Improvement Plan adopted in PMRCIP-IV (Chapter 4)

The river channel improvement plan for the PMRCIP-IV project was initially included and designed under the PMRCIP Phase I Project (2002DD/PMRCIP-I). The plans and designs in the Definitive Plan formulated on the feasibility study level in 2015 under the supplemental work for PMRCIP-III were then reviewed.

Therefore, this "Detailed Design Study for the Pasig-Marikina River Channel Improvement Project, Phase IV" was carried out based on the results of the detailed design undertaken in the 2002DD/PMRCIP-I and the Definitive Plan in 2015. The river channel improvement plan for the PMRCIP-IV project was, therefore, reconfirmed and finalized through the review and verification works under the two previous studies, namely, the 2002DD/PMRCIP-I and the Definitive Plan in 2015. **Chapter 4** presents the processes and results of finalization of the river improvement plans.

2.5.3 Survey and Investigation of Present Site Conditions (Chapter 5)

To ensure the necessary accuracy of the basic and detailed designs, topographic and geological surveys as well as the other necessary surveys were carried out. Both of the topographic survey to prepare topographic maps, cross sections and longitudinal profile and the geological survey to confirm the basic conditions of soil and the ground for stability and structural calculations were subcontracted to a local survey firm. The working processes and results are explained in detail in **Chapter 5**.

2.5.4 Determination of Locations and Dimensions of Target River Structures (Basic Design) (Chapter 6)

The locations and basic dimensions of the MCGS, the Cainta Floodgate and the Taytay Sluicegate, as well as the basic dimensions required for the dike and revetment were reviewed and set on the Basic Design Stage as explained in **Chapter 6**.

As to the MCGS, the hydraulic model experiments were executed based on the results of the first review of the design of MCGS. In the hydraulic model experiments, the effects, widths of gates of the MCGS and the necessary dimensions of the temporary diversion channel during the construction work were also confirmed and finalized. These are described in **Chapter 8**.

2.5.5 Detailed Engineering Design and Design Criteria (Chapter 7 and Chapter 11)

Based on the basic design in **Chapter 6**, the stability analyses, structural calculations of each member of river structures, and the imperative countermeasures to be taken together with the construction works were analyzed and computed so that all structures to be constructed will function smoothly during their expected lifetime or operating time. In addition, quantity calculations for each member and material of the structures were also conducted to estimate the project cost.

The computing processes and results of calculations and analyses are summarized in **Chapter 7**. The detailed computing and calculating processes are given separately in **VOLUME-4**.

The methodologies and criteria of calculations for stability and appropriate design of structures are in **Chapter 11**.

2.5.6 Hydraulic Model Experiment (Chapter 8)

Based on the initial basic design and concepts of the MCGS, hydraulic model experiments were executed to finalize the dimensions of the MCGS through the confirmation of hydraulic condition of upstream and downstream flows. In particular, widths of the two MCGS gates were reviewed and gate operation was simplified to reduce cost thereby maximizing the benefit due to the construction of the MCGS.

The results of the hydraulic model experiments are summarized in **Chapter 8**. The final report on the Hydraulic Model Experiment was submitted to JICA in November 2019.

2.5.7 Formulation of Basic Concept of Non-Structural Measures and the Operation and Maintenance Plans after the Completion of PMRCIP-IV (Chapter 9)

In the Detailed Engineering Design Study, the non-structural measures to be taken during the construction phase of the PMRCIP-IV Project were formulated and proposed. The Non-Structural Measures are divided into two activities, namely, the Information Campaign and Publicity (ICP) and the Information Provision to enhance the community-based flood mitigation activities.

In addition, the operation and maintenance plan/s for the MCGS, Cainta Floodgate and Taytay Sluicegate, as well as the other river structures such as dikes and revetments to be constructed in the PMRCIP-IV, were also prepared. They have been set up based on the review of activities conducted under the PMRCIP-II and III, and through the reactivation of the Flood Mitigation Committee (FMC).

The plans formulated are described in **Chapter 9** together with the deliberation processes in this Study.

2.5.8 Updates and Reviews on Environmental Impact Statement (EIS), Environment Management Plan (EMP), Environment Monitoring Plan (EMoP) and Right-of-Way (ROW) / Resettlement Action Plan (RAP) (Chapter 10)

As for environmental considerations, the existing Environmental Impact Statement (EIS), the Environment Management Plan (EMP), and the Environment Monitoring Plan (EMoP) have been reviewed. In the PMRCIP-IV, the backfill works for excavated and dredged soils at approximately 1.5 million m^3 (MCM) as disposal in the available land should be undertaken under the following two (2) concerns:

- Whether or not disposal materials include hazardous and contaminated materials; and
- Whether or not disposal area/s is/are available and secured for huge amounts of excavated and dredged soils.

In this regard, the "TCLP (Toxicity Characteristic Leaching Procedure) and Elutriate Studies of Disposal Materials; Particle Size Distribution of Disposal Materials; Water Quality surveys" and "Environmental Impact Assessment Study (EIA) for the Backfill Site" were executed by reconsignment work. In the reconsignment for EIA, the survey for the Backfill Site also included assistance activities for obtaining the necessary Environmental Clearance Certificate (ECC) from the DENR.

Furthermore, it was confirmed in the basic design that the construction of the Cainta Floodgate would involve additional resettlement and land acquisition along the Cainta Creek. Hence, the EIA for the Cainta Floodgate was included in the subcontracted works of the EIA Survey for the backfilling site in accordance with the suggestion of the EMB of DENR Region IV-A. These two EIAs were carried out from December 2019 and completed in August 2020.

As for social considerations, two resettlement action plans, namely, those for the Marikina River and the Manggahan Floodway which were formulated in 2018, have also been reviewed and updated taking into account social considerations, the results of detailed engineering design, and the allowable and required budgetary conditions of the DPWH. The activities taken, recognitions observed and draft results on environmental and social considerations on the PMRCIP-IV are described in detail in **Chapter 10**.

2.5.9 Review of Project Evaluation (Chapter 12)

Based on the results of the detailed design with quantity calculation and the construction plan, the project evaluation was also confirmed from the relationship between the conditions reviewed, such as flood simulation analyses before and after the PMRCIP-IV shown in **Chapter 3** and the construction costs newly estimated. Details are given in **Chapter 12**.

2.6 Summary of Essential Results of the Basic Design and Detailed Engineering Studies to be Considered in the Future

As explained in the preceding **Section 2.5**, the following studies and analyses have been conducted, including the preparation of bidding documents:

- Review of Basic Design (**Chapter 3 to Chapter 6**);
- Detailed Engineering Design with Hydraulic Model Experiment (**Chapter 7 and Chapter 8**);
- Formulation of Non-Structural Measures with Setting of Operation Rule of River Structures, and Review of Environmental and Social Considerations (**Chapter 9 and Chapter 10**); and
- Study on Project Evaluation (**Chapter 12**).

Through the results of each study and design, the essential points determined and significant concerns confirmed in this Study are as discussed hereinafter.

2.6.1 Design Flood Discharge Distribution of the Pasig-Marikina River Basin

Flood control plans prepared for the basin have been reviewed in this Study and discussed with the DPWH and the other agencies concerned. The Design Flood Distribution finally proposed are described as follows.

2.6.1.1 Target Flood Protection Scale for the Pasig-Marikina River Basin

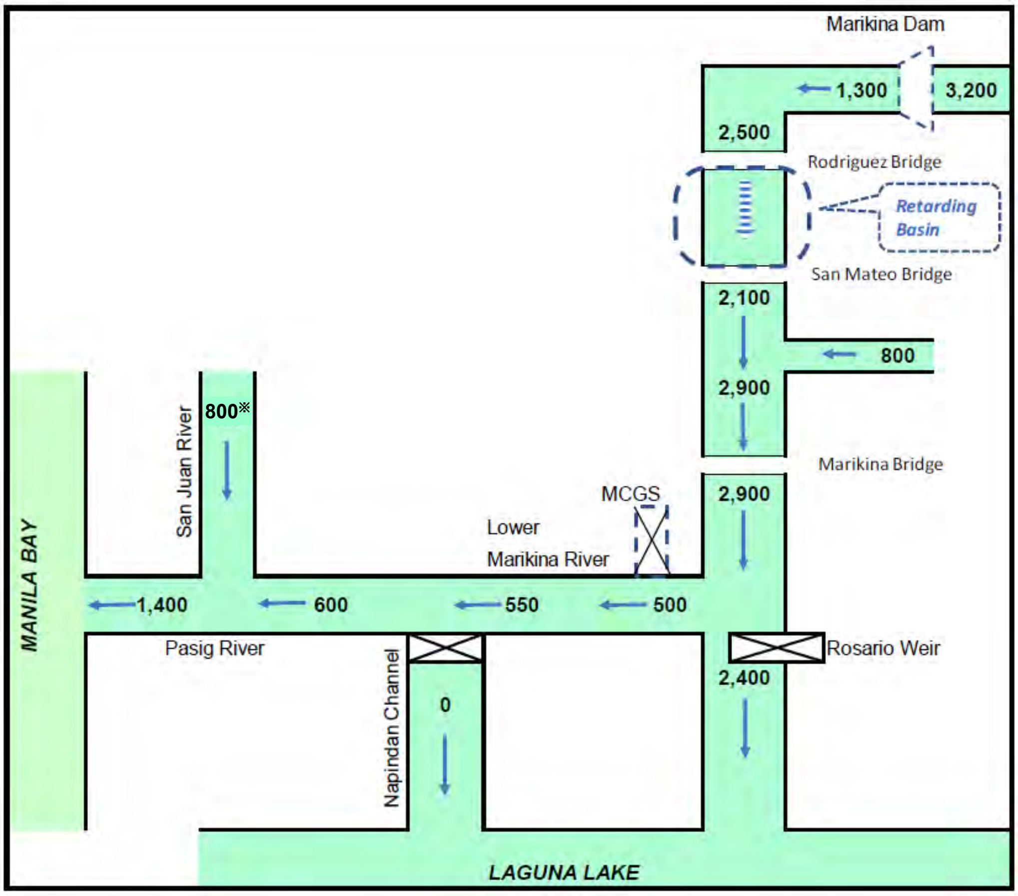
(1) Target Protection Level for the Basin

In line with the Flood Control Master Plan by the World Bank in 2012 and the other review studies after Typhoon Ondoy, it was confirmed that people and properties in the Basin shall basically be protected against a 100-year return period river-overflow flood. Therefore, the design discharge of 2,900 m^3/s for the river channel improvement in PMRCIP-IV shall correspond to the 100-year design discharge after the completion of structural measures proposed in the Basin upstream, such as the construction of the Marikina Dam and the retarding basins. On the other hand, since the design discharge of 2,900 m^3/s is only 20 to 30-year return period probability under the existing condition without the Marikina Dam and the retarding basins, early completion of the Marikina Dam and the retarding basins is desirable.

The Design Discharge Distribution for the whole Basin proposed through the review and verification analyses in this Study is as illustrated in **Figure 2.6.1**.

The design discharge at each stretch have been slightly modified based on those proposed in the Data Collection Survey on Flood Management Plan in Metro Manila (JICA, May 2014; hereinafter referred to as JICA2014Survey) and the 2015IV&V-FS Report (DPWH, March 2015; hereinafter, referred to as 2015IV&V-FS).

The processes and results of each analysis leading to the proposed design discharge distribution are described in detail in **Chapter 3**.



Note: 800*: Proposed Design Discharge of the San Juan River considering the reduction of 200m^3/s by retarding effects in the upper basin (structural measures for peak discharge reduction are discussed in Subsection 3.4.4).

Source: Study Team modifying Figure R 5.2.1, Schematic Diagram of 100-year Flood Discharge Distribution in the 2015IV&V-FS Report (DPWH, March 2015)

Figure 2.6.1 Proposed Design Discharge Distribution (100-year Return Period Flood)

(2) Points of Attention

The proposed design discharge distribution as illustrated in **Figure 2.6.1** shall be established on the following premises and/or considerations that should be taken in the future.

1) Flood Control Plan for San Juan River

In this Basic and Detailed Design, it has been confirmed that the probable discharge of San Juan River at 100-year return period will exceed 1,000 m^3/s as also calculated under the Master Plan for Flood Management in Metro Manila and Surrounding Areas funded by the World Bank (DPWH, June 2012; hereinafter referred to as the "WB2012MP"). On the other hand, DPWH had temporarily set the maximum design discharge after the river improvement work of San Juan River at 700 to 800 m^3/s.

Taking into consideration the conditions mentioned above, the proposed design discharge of the San Juan River is set at 800 m^3/s to conform with the design discharge of San Juan River proposed at 780 m^3/s in the JICA2014Survey. In this connection, more than 200 m^3/s of the peak discharge of San Juan River shall be reduced by structural measures in its basin in the future. Structural measures for peak discharge reduction are discussed in **Subsection 3.4.4** of **Chapter 3**.

2) Design Discharge in the Pasig River

In the PMRCIP Phase II and Phase III, the dikes and revetments along the Pasig River have been constructed on the level of design discharges of 1,200 m^3/s from Delpan Bridge to the Confluence Point with San Juan River and 600 m^3/s from the confluence point with the San Juan River to the upper end of the Pasig River (Sta. 17+100). These design discharges (1,200 $m^3$and 600 m^3/s) correspond to the 30-year return period flood of the basin.

As shown in **Figure 2.6.1**, the DPWH is aiming at the protection of Metro Manila from river floods of 100-year return period. In this regard, the river channel capacity of the Lower Pasig River should be increased from 1,200 m^3/s to 1,400m^3/s.

To attain the upgrade of flood protection level from 30 to 100-year return period, there are two alternatives and one is to raise the top of elevation of flood protection dikes along the Pasig River. In this case, the dike should be raised at 0.42 m at the confluence point with the San Juan River. Another alternative is to dredge the riverbed of the Pasig River by EL+5.00m or lower from the river mouth to the confluence point with the San Juan River (approximately 7 km long) in order to sustain the design flood level of the Pasig River.

3) Plans for the Marikina Dam and the Marikina Retarding Basin

At present, the DPWH is planning to construct the Marikina Dam and the Marikina Retarding Basin, the studies of which have been undertaken with funds from the World Bank in line with the master plan for the flood control of 100-year return period.

As explained in Item (1) above, Target Protection Level for the Basin, the design discharge of 2,900 m^3/s at Sto. Niño for the river channel in PMRCIP-IV should correspond to the flood design discharge of 100-year return period.

In this connection, the DPWH should harmonize the PMRCIP-IV with the studies for the Marikina Dam and the Marikina retarding basins.

2.6.1.2 River Channel Improvement Plan for PMRCIP-IV

As a result of the Study, the significant points modified from the results of the DPWH2015IV&V are as described below (See also **Section 6.1 of Chapter 6**).

(1) Modification of River Alignment

River alignment is slightly modified at some sections from the results of DPWH2015IV&V. As for the following two sections, alternatives for each section are proposed. The appropriate alignment are finalized in the Detailed Design Stage through comparative studies in terms of advantage and disadvantage issues.

1) Section: Manalo Bridge (Sta. 7+200)

Due to the issues on the Right-of-Way (ROW), the alignment around the Manalo Bridge is being considered by DPWH. According to the DPWH, the alignment fixed by DPWH2015IV&V shall be slightly shifted toward the right bank.

In the Detailed Design Stage, the dikes and revetments in this section are designed in line with the alignment fixed by DPWH.

2) Section: Olandes Sewerage Treatment Plan (Olandes STP) (Sta. 9+000 to Sta. 10+000)

The bank at the left side of the Olandes STP has been developed recently by Marikina City and a private development firm. In the DPWH2015IV&V, the alignment is set so as not to affect the facilities of the Olandes STP. However, Marikina City had strongly requested the DPWH to take into account the development at the left bank. In this connection, the alignment of river improvement in this section is shifted slightly toward the right bank.

(2) Change of Type of Low-Water Revetment from Embankment Slope to Steel Sheet Pile Wall

In the basic design of the DPWH2015IV&V, the shape of the low water channel on the right bank at around Sta. 11+500 is the “Embankment Slope (H:V=3:1).” However, the Steel Sheet Pile (SSP) Revetment Type is finally adopted for this section based on the discussions and agreement between the DPWH and Marikina City.

Due to the change of type of revetment for the low-water channel, the berm width shall be made wider for recreational activities in normal time. Furthermore, the type of flood dike around this area can be changed from thin concrete wall to embankment type. As of February 2020, the DEO (First Metropolitan Engineering District) of the DPWH had designed the embankment dike in which the crown can be placed as public service road along this section. Hence, the construction cost for the embankment dike shall be excluded from the cost of the PMRCIP-IV.

2.6.1.3 Structural Dimensions of the MCGS

(1) Modification of the Widths of the MCGS Gates

As discussed in **Section 6.3 of Chapter 6**, the basic study for the MCGS have been conducted as the first step of the design for the MCGS. Through this first step, the widths of the two gates of the MCGS were modified. As the second step of the design, the widths of the two gates have been finalized and confirmed through the hydraulic model experiment. As the final result, the widths of the two gate leaves of the MCGS designed in the 2002DD (20m + 20m) were modified (28.3m + 11.7m).

(2) Modification of the Foundation Type of the MCGS

Through the results of geological surveys illustrated in **Chapter 5**, it was confirmed that the Guadalupe Formation outcropping on the riverbed has sufficient hardness bearing capacity against dead and live loads of the MCGS. In this connection, the foundation type of the MCGS is amended from Pile Type to Spread Type (without foundation pile).

2.6.1.4 Structural Dimensions of the Cainta Floodgate

(1) Construction Point of the Cainta Floodgate

In the DPWH2015FS, the construction point of the Cainta Floodgate is set in alignment with the berm of the dike. It was considered that the construction of the floodgate would not affect the areas outside of the floodway.

In this Study, after the review on the construction point of the floodgate, it was shifted to the existing San Francisco Bridge to keep it in line with the alignment of the dike crown and make the crown appropriately function as a dike of the Manggahan Floodway and should not hamper the flood flow in the floodway. As a result, the construction of the floodgate will need relocation of the affected houses and buildings and procure the lots along the Cainta Creek which was not anticipated by the DPWH. Taking into account the changes in construction conditions, the EIA for the construction of the Cainta Floodgate was executed in this Study.

(2) Temporary Detour Road for the Construction of the Cainta Floodgate

The existing 2-lane San Francisco Bridge running across the Cainta Creek along the dike road should be replaced into a 4-lane bridge on the structure of the Cainta Floodgate that will be constructed.

To ease the concern of the governments of Rizal Province and Cainta Municipality as well as the DPWH about the traffic condition during the construction stage, a temporary detour road is planned and designed so as not to cause heavy traffic at the construction site during the construction stage. (Refer to **Subsection 7.4.5.2 in Chapter 7**.)

2.6.1.5 Structural Type of Taytay Floodgate

Taking into consideration conformity with the existing box-culvert, the structural type of the Taytay Floodgate should be the sluicegate.

Details of the comparative study are given in **Subsection 6.4.4** of **Chapter 6**.

2.6.2 Draft Bidding Documents

In parallel with the detailed engineering design and cost estimation of each construction package, the Draft Bidding Documents have been prepared in accordance with the “Standard Bidding Document under Japanese ODA Loans” issued in October 2019.

In the Draft, the Specification and Pay Items in the Bill of Quantities (BoQ) of the PMRCIP-IV follow the latest edition of the “Standard Specifications for Public Works Structures” issued by the DPWH.

CHAPTER 3 FLOOD MANAGEMENT PLAN FOR PASIG-MARIKINA RIVER

3.1 Current Condition of Pasig-Marikina River Basin

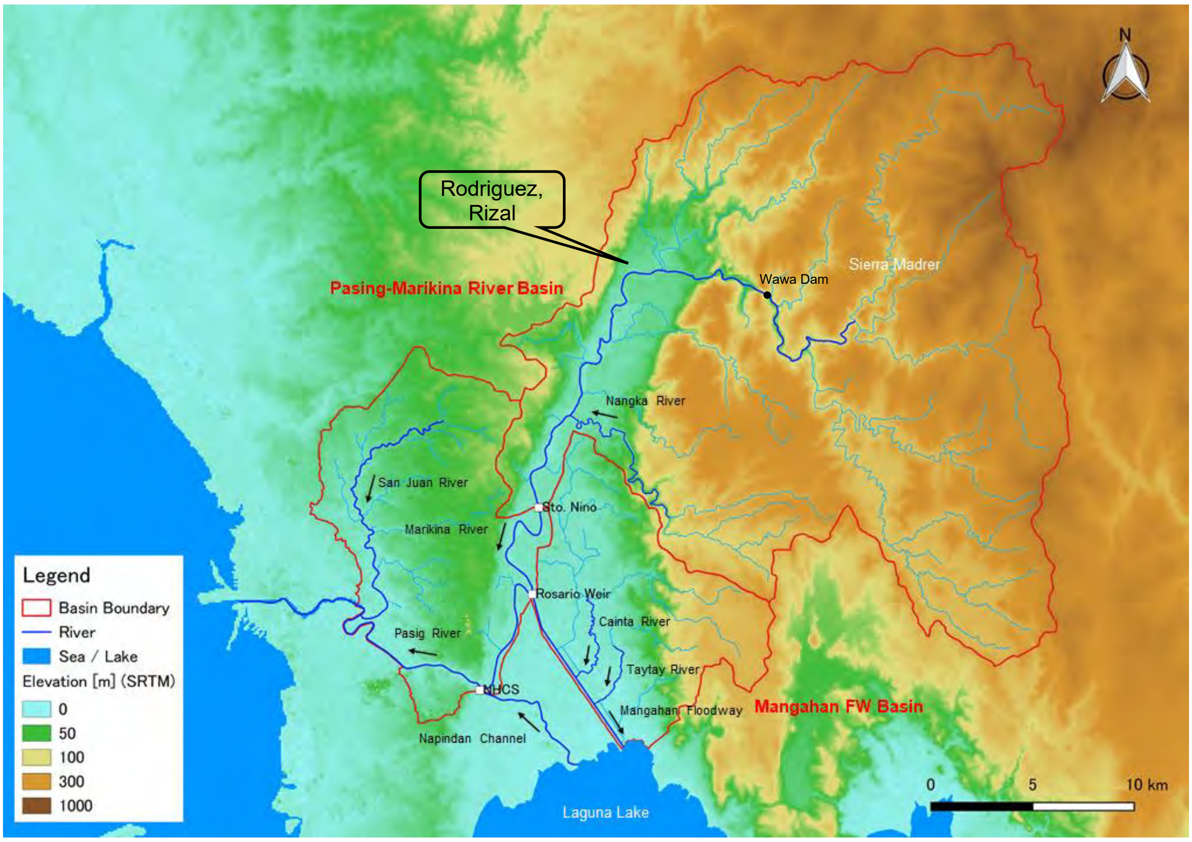
3.1.1 Outline of the River Basin

The Pasig-Marikina River, with a total length of 52.2 km (Manila Bay to Wawa Dam) and a total catchment area of 635 km^2, originates from the southwestern slopes of the Sierra Madre Mountains. The river initially flows toward the West, converges with several branch river systems and, after changing its flow direction to the south along the West Valley Fault at Rodriguez in Rizal Province, finally pours into the Manila Bay. It, therefore, traverses the entire Metro Manila area or the National Capital Region.

The river has two major tributaries, namely; the San Juan River which merges at 7.1 km from the river mouth, and the Napindan Channel which merges at 17.1 km from the river mouth, respectively.

The Pasig-Marikina River is mainly divided into two (2) sections at 17.1 km from the river mouth. The downstream section is called the Pasig River (from the river mouth to the merging point of the Napindan Channel), and the upstream section is called the Marikina River (upper reach of the river from the merging point of the Napindan Channel).

The Pasig-Marikina River also connects with the Laguna de Bay (Laguna Lake) via the Napindan Channel and the Manggahan Floodway. The floodway is manmade and it diverts floodwaters from the Marikina River at 23.8 km from the river mouth. The center of the basin is at 14.5°North latitude and 121°East longitude. The Location Map is in **Figure 3.1.1**.



Source: Study Team

Figure 3.1.1 Location Map, Pasig-Marikina River Basin

The Pasig-Marikina River Basin belongs to the tropical region and the annual mean temperature is 26°C. Climate in the area can be divided into two seasons, the rainy (or wet) season and the dry season. Annual rainfall in the basin is 2,600 mm. About 80% of the total rainfall occur during the rainy season. Weather turbulences which generate precipitation are orographic effects of the southwest monsoon, tropical depression front, and the convective nature of the Inter-Tropical Convergence Zone (ITCZ).[1]

3.1.2 Flow Condition of Marikina River

3.1.2.1 Sto. Niño Station

The water level and flow condition at the Sto. Niño gauging station are as shown in **Table 3.1.1** and **Table 3.1.2**. Average 95-day, 185-day, 275-day and 355-day water level at Sto. Niño in the recent 25 years (1994 to 2018) were 12.63 m, 12.03 m, 11.55 m and 11.27 m, respectively, and the highest water level observed at 5PM during Typhoon Ondoy on the 26th of September 2009 was 22.16 m.

Average 95-day, 185-day, 275-day and 355-day discharges were 113.0 m^3/s, 53.0 m^3/s, 22.4 m^3/s and 11.4 m^3/s, respectively. The maximum discharge was 3,480 m^3/s at the highest water level observed in Typhoon Ondoy.

Table 3.1.1 Water Level Condition Sheet at Sto. Niño (Annual)

Year	Water Level (EL. m)						
	Highest	95-Day	185-Day	275-Day	355-Day	Lowest	Mean
1994	16.33	12.49	11.72	11.17	10.96	10.80	12.20
1995	18.40	13.04	11.44	11.01	10.85	10.80	12.34
1996	16.08	12.44	11.86	11.20	11.09	10.80	12.19
1997	17.16	12.41	11.55	11.15	10.89	10.80	12.07
1998	18.41	12.22	11.55	11.24	10.99	10.80	12.11
1999	18.30	12.67	12.37	11.70	11.40	11.14	12.53
2000	19.02	13.20	12.23	11.71	11.26	11.06	12.82
2001	16.31	12.65	12.03	11.58	11.23	11.09	12.37
2002	17.94	13.12	12.23	11.35	11.04	10.86	12.60
2003	17.76	11.89	11.50	11.12	10.87	10.80	11.98
2004	19.08	11.84	11.39	10.99	10.90	10.80	11.93
2005	16.03	12.31	11.87	11.33	10.95	10.80	12.11
2006	16.37	12.25	11.88	11.47	11.19	10.91	12.16
2007	16.90	12.23	11.72	11.38	11.02	10.86	12.05
2008	16.74	12.13	11.97	11.78	11.49	11.35	12.10
2009	22.16	-	-	-	-	-	-
2010	-	-	-	-	-	-	-
2011	19.13	13.10	12.54	11.47	-	11.21	-
2012	20.42	13.44	12.47	12.02	11.49	11.40	13.05
2013	18.77	13.29	12.62	12.16	11.99	11.79	12.93
2014	19.65	13.00	12.73	12.28	11.65	11.13	12.88
2015	16.73	12.85	12.37	12.04	11.96	11.10	12.61
2016	17.39	12.54	12.10	11.87	-	11.10	-
2017	16.04	12.53	12.27	11.82	11.77	10.84	12.30
2018	20.36	12.79	12.22	11.80	11.75	10.79	12.42
Highest	22.16	13.44	12.73	12.28	11.99	11.79	13.05
Lowest	16.03	11.84	11.39	10.99	10.85	10.79	11.93
Mean	17.98	12.63	12.03	11.55	11.27	11.00	12.37

Note: 2009 and 2010 were excluded since they contain many missing data except for highest water level in 2009.
Source: Study Team based on EFCOS data

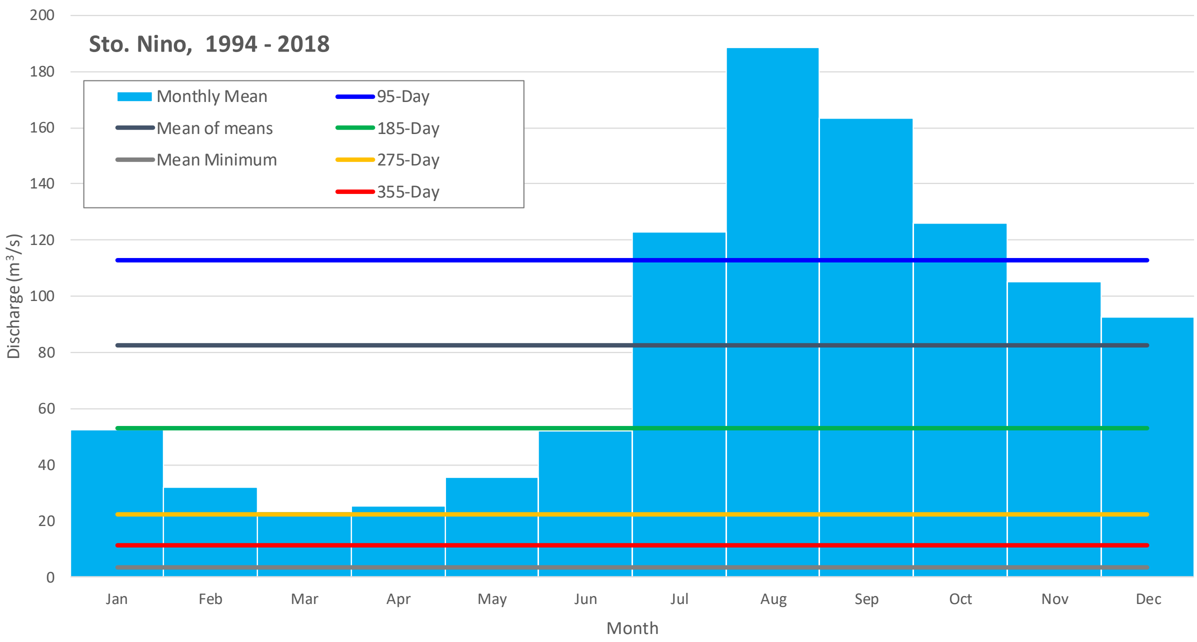
[1] The Study of Water Security Master Plan for Metro Manila and Its Adjoining Areas, Final Report – Stormy Rainfall Analysis in the Pasig-Marikina River Basin, March 2013, JICA

Table 3.1.2 Flow Condition Sheet at Sto. Niño (Basin Area: 496 km²)

Year	Discharge (m^3/s)							
	Maximum	95-Day	185-Day	275-Day	355-Day	Minimum	Mean	Annual Total (Mil. m^3)
1994	883.8	91.0	27.1	4.3	0.8	0.0	62.9	1,984.3
1995	1,617.1	160.6	12.9	1.4	0.1	0.0	75.8	2,390.4
1996	810.1	85.8	36.1	5.1	2.7	0.0	62.2	1,960.4
1997	1,151.4	82.8	17.9	4.0	0.2	0.0	52.0	1,638.9
1998	1,621.1	64.4	18.1	6.2	1.1	0.0	55.1	1,736.8
1999	1,576.6	111.5	78.8	26.0	11.5	3.7	96.4	3,039.0
2000	1,879.5	184.9	65.0	26.5	6.7	2.2	131.2	4,137.7
2001	877.8	109.4	48.1	19.5	6.1	2.7	79.3	2,500.1
2002	1,435.1	172.2	65.3	9.7	1.9	0.1	103.3	3,258.4
2003	1,366.9	37.8	15.5	3.3	0.2	0.0	45.0	1,418.3
2004	1,905.9	34.4	11.0	1.2	0.3	0.0	40.7	1,285.0
2005	795.8	73.3	36.5	9.0	0.7	0.0	55.3	1,743.2
2006	895.9	67.2	37.1	14.3	4.8	0.4	59.7	1,881.8
2007	1,063.8	65.1	27.0	10.8	1.6	0.1	49.7	1,568.2
2008	1,011.6	56.9	43.8	30.7	15.2	9.7	54.4	1,714.0
2009	3,480.0	-	-	-	-	-	-	-
2010	-	-	-	-	-	-	-	-
2011	1,928.1	169.3	97.0	14.6	-	5.4	-	-
2012	2,544.5	223.8	89.0	47.7	15.2	11.5	161.7	5,100.1
2013	1,771.3	198.6	106.0	59.3	45.5	31.4	144.9	4,568.2
2014	2,166.3	155.3	119.4	70.2	23.1	3.5	138.7	4,374.6
2015	1,008.4	134.7	79.2	49.1	43.0	2.9	105.4	3,322.7
2016	1,231.8	97.3	53.8	36.9	-	2.9	-	-
2017	798.6	95.9	69.3	33.3	30.1	0.1	71.6	2,259.2
2018	2,514.0	127.0	64.6	32.0	28.9	0.0	84.3	2,656.9
Maximum	3,480.0	223.8	119.4	70.2	45.5	31.4	161.7	
Minimum	795.8	34.4	11.0	1.2	0.1	0.0	40.7	
Mean	1,514.0	113.0	53.0	22.4	11.4	3.3	82.4	

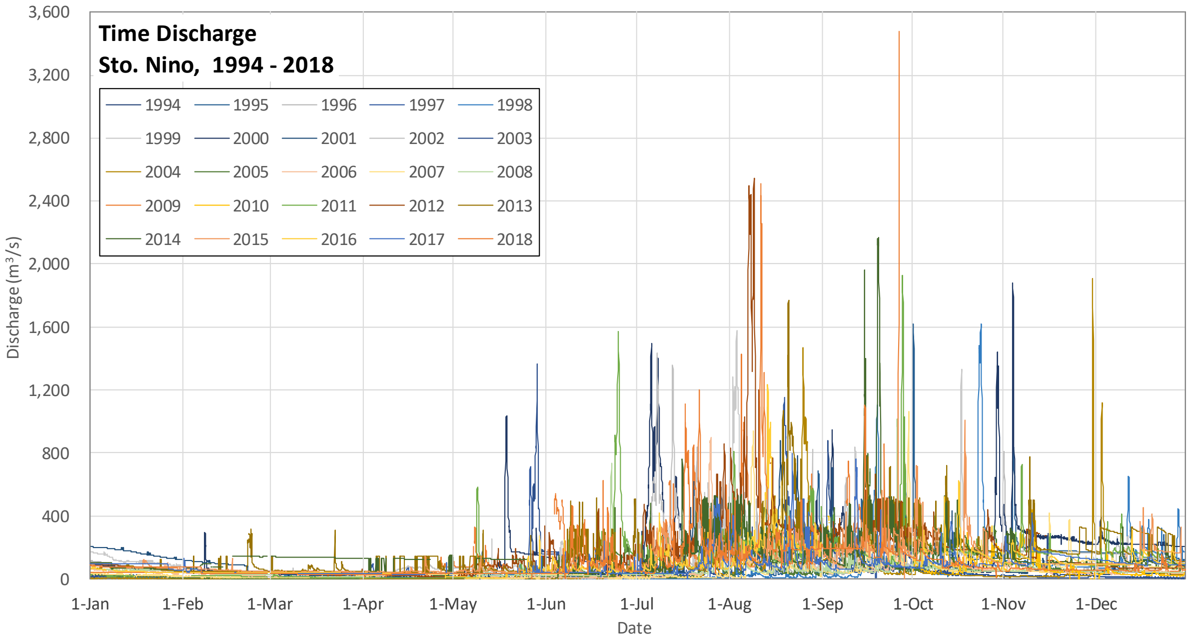
Note: 2009 and 2010 were excluded since they contain many missing data except for maximum discharge in 2009.
Source: Study Team based on EFCOS data

Figure 3.1.2 shows the comparison of the average 95-Day, 185-Day, 275-Day, 355-Day discharge and monthly average discharge for the last 25 years (1994-2018) at Sto. Niño Gauging Station. The average discharge from January to June is below 185-Day discharge. **Figure 3.1.3** shows the time discharge for the last 25 years (1994-2018) at Sto. Niño Gauging Station.



Source: Study Team based on EFCOS data

Figure 3.1.2 Flow Condition at Sto. Niño Gauging Station



Source: Study Team based on EFCOS data

Figure 3.1.3 Time Discharge at Sto. Niño Gauging Station (1994-2018)

Table 3.1.3 shows the water level condition at Sto. Niño gauging station in the rainy season, May to November, which has been referred to in the structural design.

Table 3.1.3 Water Level Condition Sheet at Sto. Niño Gauging Station (Rainy Season)

Year	Water Level (EL. m)						
	Highest	56-Day	108-Day	161-Day	208-Day	Lowest	Mean
1994	16.33	13.00	12.08	11.58	10.96	10.80	12.51
1995	18.40	13.09	12.60	11.25	10.88	10.80	12.63
1996	16.08	13.03	11.99	11.21	11.09	10.80	12.42
1997	17.16	12.80	12.12	11.55	10.87	10.80	12.41
1998	18.41	12.38	11.63	11.25	10.98	10.80	12.22
1999	18.30	12.79	12.47	11.90	11.37	11.14	12.72
2000	19.02	13.40	13.03	12.05	11.31	11.06	13.12
2001	16.31	12.88	12.19	11.73	11.22	11.09	12.48
2002	17.94	13.15	13.10	12.18	11.06	10.86	12.99
2003	17.76	12.52	11.79	11.53	10.88	10.80	12.30
2004	19.08	12.11	11.56	11.35	10.93	10.80	12.13
2005	16.03	12.50	12.11	11.74	11.02	10.80	12.35
2006	16.37	12.66	12.13	11.40	11.18	10.91	12.38
2007	16.90	12.33	11.96	11.45	10.95	10.86	12.20
2008	16.74	12.26	12.04	11.75	11.36	11.35	12.21
2009	22.16	-	-	-	-	-	-
2010	-	-	-	-	-	-	-
2011	19.13	13.32	12.99	12.48	11.28	11.21	13.07
2012	20.42	13.84	13.27	12.62	11.65	11.42	13.55
2013	18.77	13.37	13.12	12.41	12.01	11.89	13.15
2014	19.65	13.38	12.79	12.44	11.81	11.13	13.06
2015	16.73	13.19	12.57	12.20	11.97	11.10	12.81
2016	17.39	12.80	12.44	11.93	-	11.10	-
2017	16.04	12.70	12.38	11.88	11.80	10.84	12.43
2018	20.36	13.19	12.51	12.09	11.78	10.79	12.72
Highest	22.16	13.84	13.27	12.62	12.01	11.89	13.55
Lowest	16.03	12.11	11.56	11.21	10.87	10.79	12.13
Mean	17.98	12.90	12.39	11.82	11.29	11.01	12.63

Note: 2009 and 2010 were excluded since they contain many missing data except for highest water level in 2009.
Source: Study Team based on EFCOS data

3.1.2.2 Rosario Junction Side (JS) Station

Water level condition in annual and rainy season at the Rosario JS station are as shown in **Table 3.1.4** and **Table 3.1.5**. Annual average 95-day, 185-day, 275-day and 355-day water level in the recent 25 years (1994 to 2018) are 12.06m, 11.66m, 11.18m and 10.89m, respectively, and 12.28m, 11.90m, 11.32m and 10.92m, respectively, in the rainy season.

Table 3.1.4 Water Level Condition Sheet at Rosario JS (Annual)

Year	Water Level (EL. m)						
	Highest	95-Day	185-Day	275-Day	355-Day	Lowest	Mean
1994	14.43	11.89	11.55	11.02	10.90	10.86	11.55
1995	15.32	12.38	11.18	10.89	10.56	10.43	11.50
1996	14.11	11.76	11.46	10.84	10.78	10.56	11.40
1997	14.32	11.38	11.06	10.81	10.66	10.48	11.13
1998	14.18	11.62	10.88	10.59	10.53	10.28	11.19
1999	14.94	12.29	11.71	11.32	-	10.90	-
2000	15.87	12.49	11.94	11.41	11.05	10.82	12.03
2001	13.97	11.99	11.71	11.37	11.04	10.70	11.73
2002	14.13	12.26	11.35	10.87	10.84	10.65	11.60
2003	13.94	11.54	11.28	10.89	10.83	10.56	11.35
2004	15.11	11.55	11.08	10.85	10.83	10.79	11.28
2005	13.56	11.99	11.42	10.92	10.83	10.69	11.51
2006	13.49	11.94	11.66	11.07	10.86	10.77	11.57
2007	13.71	11.84	11.29	10.93	10.80	10.76	11.43
2008	-	-	-	-	-	-	-
2009	17.92	11.44	12.83	12.36	-	11.01	-
2010	13.74	11.87	11.57	10.95	10.91	10.49	11.53

Year	Water Level (EL. m)						
	Highest	95-Day	185-Day	275-Day	355-Day	Lowest	Mean
2011	15.25	12.58	12.28	11.25	10.92	10.53	12.00
2012	16.44	12.93	12.22	11.69	11.11	11.02	12.37
2013	15.18	12.61	12.11	11.81	11.70	11.16	12.26
2014	15.83	12.58	11.96	10.81	10.73	10.66	11.86
2015	15.59	12.36	11.95	11.74	11.67	10.76	12.10
2016	14.71	12.15	11.82	11.62	10.59	10.37	11.88
2017	13.41	11.98	11.64	11.06	10.74	10.44	11.56
2018	16.02	12.17	11.87	11.17	10.76	10.07	11.74
Highest	17.92	12.93	12.83	12.36	11.70	11.16	12.37
Lowest	13.41	11.38	10.88	10.59	10.53	10.07	11.13
Mean	14.80	12.06	11.66	11.18	10.89	10.66	11.66

Note: 2008 was excluded since they contain many missing data.
Source: Study Team based on EFCOS data

Table 3.1.5 Water Level Condition Sheet at Rosario JS (Rainy Season)

Year	Water Level (EL. m)						
	Highest	56-Day	108-Day	161-Day	208-Day	Lowest	Mean
1994	14.43	11.95	11.78	11.27	10.90	10.88	11.68
1995	15.32	12.53	11.52	10.88	10.57	10.43	11.68
1996	14.11	11.73	11.48	10.84	10.78	10.56	11.39
1997	14.32	11.48	11.20	10.98	10.65	10.49	11.27
1998	14.18	11.72	11.07	10.81	-	10.28	-
1999	14.94	12.27	12.03	11.24	-	10.90	-
2000	15.87	12.64	12.05	11.53	11.04	10.82	12.16
2001	13.97	12.07	11.80	11.37	11.00	10.70	11.79
2002	14.13	12.47	12.14	10.95	10.84	10.80	11.94
2003	13.94	11.75	11.49	11.27	10.83	10.80	11.58
2004	15.11	11.64	11.31	11.03	10.83	10.81	11.44
2005	13.56	12.08	11.84	11.21	10.84	10.81	11.73
2006	13.49	12.04	11.83	11.07	10.86	10.79	11.66
2007	13.71	11.86	11.69	10.96	10.81	10.78	11.50
2008	-	-	-	-	-	-	-
2009	17.92	13.22	12.35	11.61	-	11.29	-
2010	13.74	12.03	11.67	10.96	10.92	10.67	11.63
2011	15.25	12.73	12.47	12.23	10.92	10.53	12.36
2012	16.44	13.46	12.82	12.35	11.31	11.02	12.88
2013	15.18	12.94	12.50	12.03	11.71	11.25	12.50
2014	15.83	12.82	12.44	10.87	10.72	10.66	12.12
2015	15.59	12.59	12.20	11.81	11.69	11.46	12.25
2016	14.71	12.40	12.05	11.76	10.57	10.37	12.02
2017	13.41	12.04	11.85	11.12	10.72	10.44	11.64
2018	16.02	12.29	12.10	11.53	10.76	10.07	11.93
Highest	17.92	13.46	12.82	12.35	11.71	11.46	12.88
Lowest	13.41	11.48	11.07	10.81	10.57	10.07	11.27
Mean	14.80	12.28	11.90	11.32	10.92	10.73	11.86

Note: 2008 is excluded because there are many missing data.
Source: Study Team based on EFCOS data

3.1.2.3 Napindan Junction Side (JS) Station

Water level condition in annual and rainy season at Napindan JS station located at the Napindan Hydraulic Control Structure (NHCS) are as shown in **Table 3.1.6** and **Table 3.1.7**, respectively. Annual average 95-day, 185-day, 275-day and 355-day water level in the recent 5 years (2014 to 2018) were 11.40m, 11.09m, 10.64m and 10.35m, respectively, and 11.45m, 11.19m, 10.84m and 10.65m, respectively, in the rainy season.

Table 3.1.6 Water Level Condition Sheet at Napindan JS (Annual)

Year	Water Level (EL. m)						
	Highest	95-Day	185-Day	275-Day	355-Day	Lowest	Mean
2014	12.10	11.40	11.20	10.76	10.65	10.31	11.14
2015	11.68	11.18	10.98	10.73	10.64	10.30	10.98
2016	11.70	11.30	10.98	10.73	10.56	10.30	11.02
2017	12.06	11.57	11.13	10.82	10.68	10.42	11.17
2018	12.31	11.57	11.14	10.92	10.67	10.40	11.24
Highest	12.31	11.57	11.20	10.92	10.68	10.42	11.24
Lowest	11.68	11.18	10.98	10.73	10.56	10.30	10.98
Mean	11.97	11.40	11.09	10.79	10.64	10.35	11.11

Source: Study Team based on EFCOS data

Table 3.1.7 Water Level Condition Sheet at Napindan JS (Rainy Season)

Year	Water Level (EL.m)						
	Highest	56-Day	108-Day	161-Day	208-Day	Lowest	Mean
2014	12.10	11.44	11.29	10.79	10.64	10.31	11.21
2015	11.68	11.16	11.00	10.71	10.64	10.30	10.98
2016	11.70	11.31	11.18	10.82	10.60	10.31	11.07
2017	12.06	11.59	11.14	10.83	10.68	10.42	11.19
2018	12.31	11.74	11.32	11.07	10.67	10.40	11.37
Highest	12.31	11.74	11.32	11.07	10.68	10.42	11.37
Lowest	11.68	11.16	11.00	10.71	10.60	10.30	10.98
Mean	11.97	11.45	11.19	10.84	10.65	10.35	11.16

Source: Study Team based on EFCOS data

3.1.3 Information on Water Level in the Pasig-Marikina River Basin

The following standard value (elevation) has been used for river structures in the basin.

- Mean Lower Low Water Level (MLLWL) = EL+10.00 m (hereinafter, "DPWH Elevation")

Therefore, information on all the elevations such as the hydraulic analysis to be implemented and examined in this study and the drawings created are represented by this DPWH elevation. The water level and elevation information at each point of this DPWH elevation is as shown in **Table 3.1.8**, **Figure 3.1.4** and **Figure 3.1.5** below.

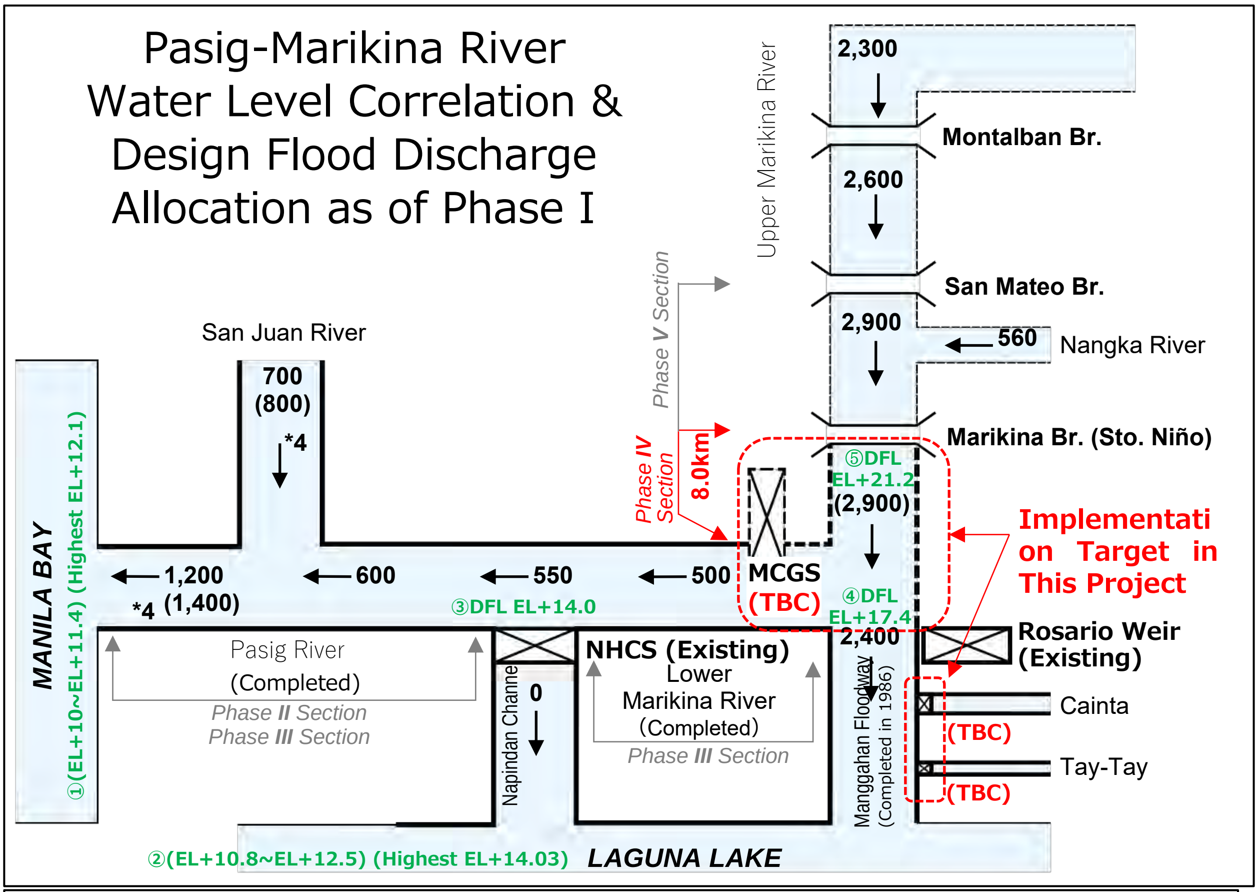
Table 3.1.8 Water Level and Elevation based on DPWH Elevation

Location	Item	Elevation (m)	Remarks
Sea Surface	MLLWL	EL+10.00	
	MSL	EL+10.475	Nominal Value (At present, it is said around EL+10.6m due to land subsidence at control point, periodic sea level rise and so on.)
	MSHWL	EL+11.40	Set in Phase I (EL+11.3m as of JICA1990MP)
	HHWL at Pasig River Mouth	EL+12.10	Recorded Highest High-Water Level in 2000
Laguna Lake	MLWL (Mean Low Water Level)	EL+10.80	Source: LLDA
	MHWL (Mean High Water Level)	EL+12.40	
	HWL in Presidential Decree	EL+12.50	High Water Level Issued in 1975 (P.D.813-1975)
	Historical Highest Water Level	EL+14.03	1972 (Exclude EL+14.62m in 1919 and EL+14.35m in 1943)
	Recorded Level in Typhoon Ondoy	EL+13.85	2009
NHCS	Elevation of Gate Foundation (Sill)	EL+ 6.00	Approximately 17.1km from the River Mouth
	Gate Crest Level	EL+15.50	
	Pasig River DHWL	EL+14.0	Phase II & III
Manggahan Floodway	Elev. of Rosario Weir Foundation	EL+10.50	Constructed in 1988
	Rosario Weir Crest Level	EL+14.00	
Phase IV	Elev. MCGS Foundation	EL+ 7.85	Reconfirmed in this Study
	MCGS Gate Crest Level	EL+19.00	
	DHWL just Downstream of MCGS	EL+15.00	
	DHWL just Upstream of MCGS	EL+17.40	
	DHWL at Sto. Niño Bridge	EL+21.17	Elev. of the bottom of Girder: EL+22.72m

Source: Study Team based on existing reports

Source: Study Team based on existing information

Figure 3.1.4 Water Level Correlation in Pasig-Marikina River (1)



Note *1 :	EL: DPWH Elevation (MLLWL＝EL+10.00)
Note *2 :	①: The number in ( ) is fluctuation of water level in Manila Bay
	②: The number in ( ) is fluctuation of water level in Laguna Lake
	③: DFL at NHCS in Marikina River
	④: DFL at Rosario Weir in Marikina River
	⑤: DFL at Sto. Nino (Marikina Bridge) in Marikina River
Note * 3 :	The number inside river is design flood discharge allocation in each section. Without parentheses ( ) means improved section/; With ( ) means section to be improved.
Note *4 :	The design discharges of the Lower Pasig and San Juan rivers have been reviewed. (See Subsection 3.4.4)

Source: Study Team based on existing information

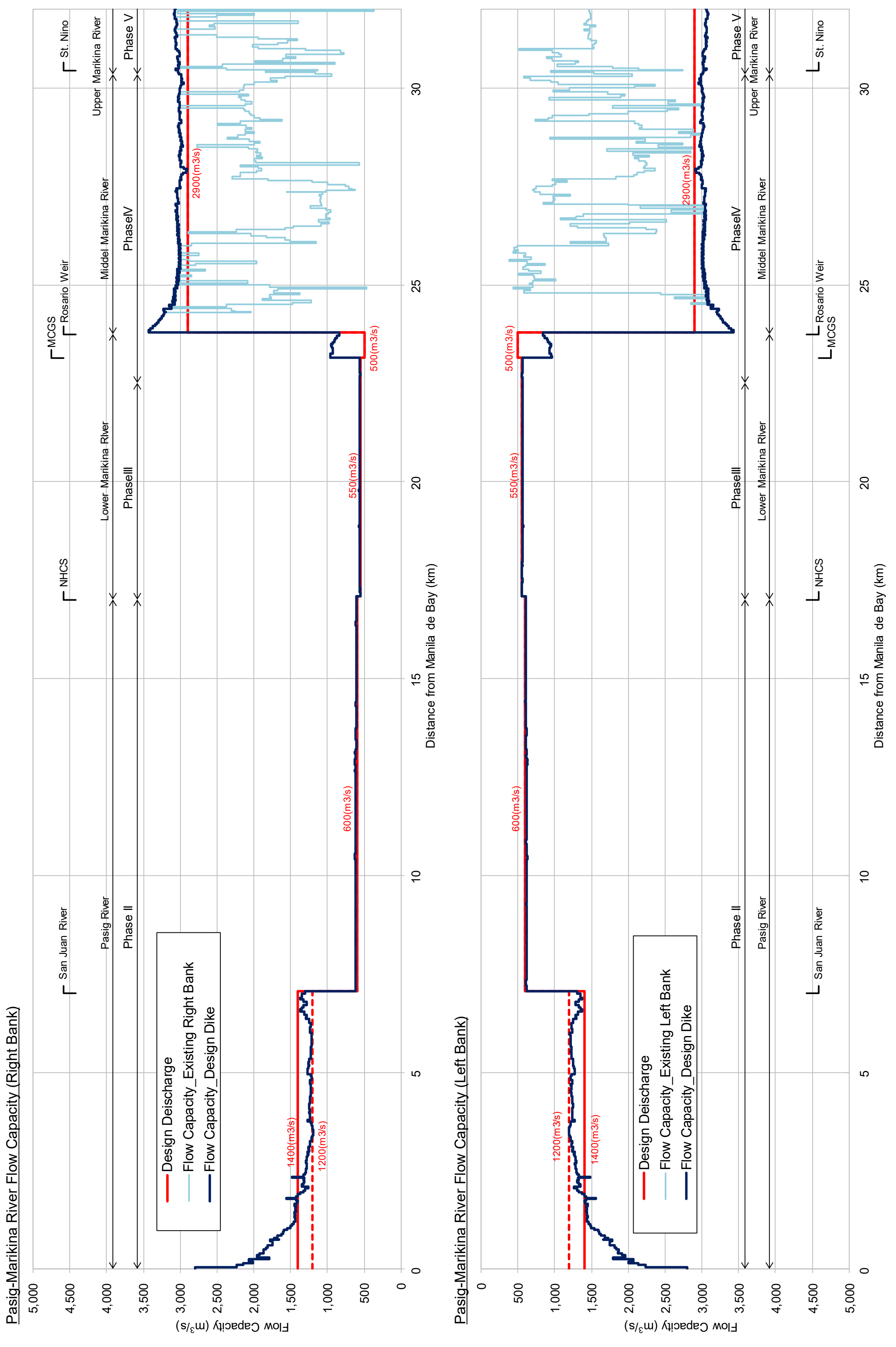
Figure 3.1.5 Water Level Correlation in Pasig-Marikina River (2)

3.1.4 Current Flow Capacity of Pasig-Marikina River

Current flow capacities of Pasig-Marikina River are as illustrated in **Figure 3.1.6**, **Figure 3.1.7** and **Figure 3.1.8** based on "The Preparatory Study for PMRCIP, Phase III (JICA2011 Preparatory Study)" and the 2015IV&V-FS for this study.

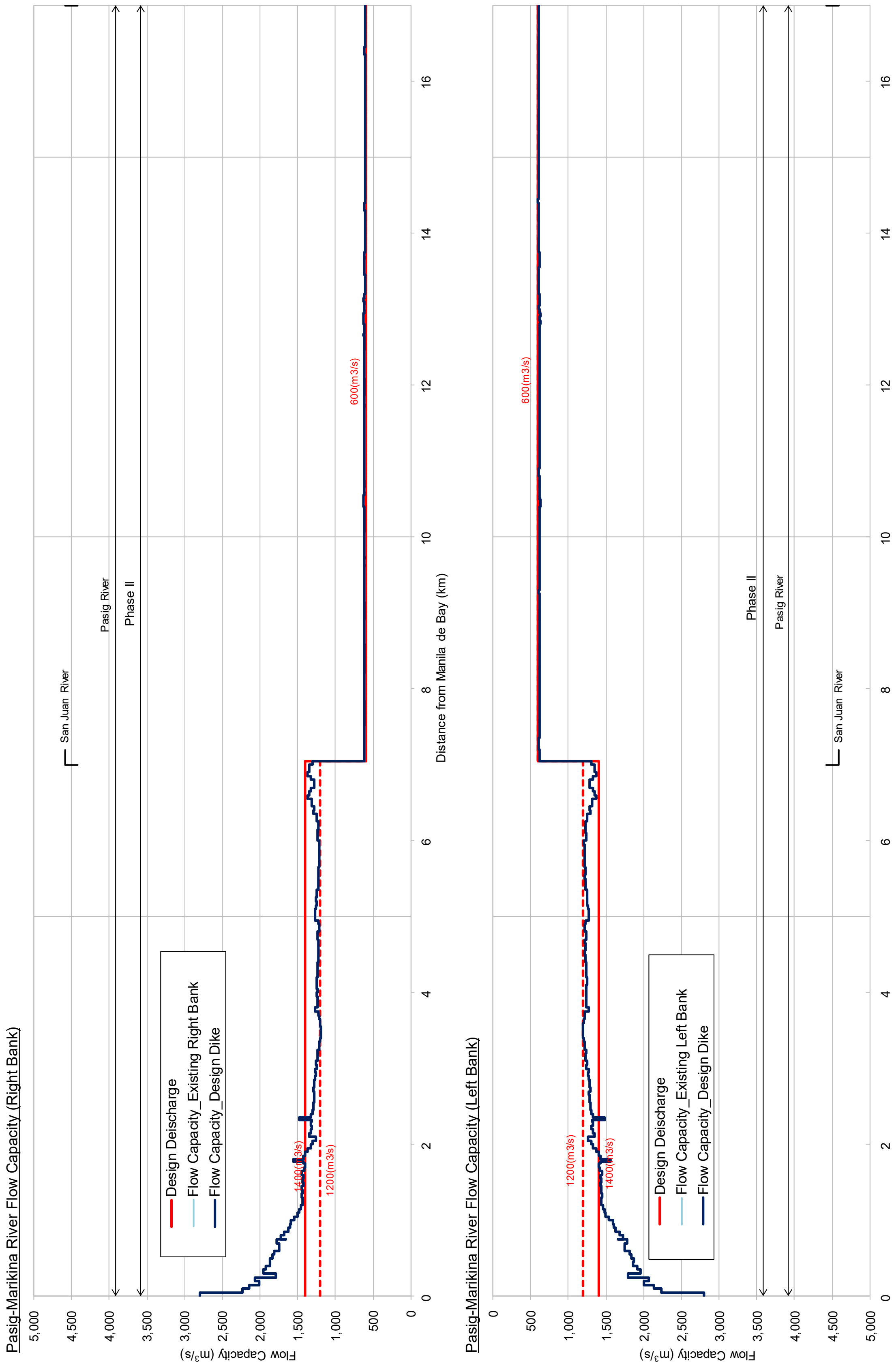
Flow capacities in the Phase II and III sections were mostly bigger than the design flood discharge (1,400 m^3/s and 600m^3/s) since the construction work in the Phase II and III sections have been completed. Although the flow capacity in the whole Phase II section was bigger than 1,200 m^3/s which is the design flood discharge of 20~30-year return period, a section downstream of San Juan River junction is smaller than 1,400 m^3/s which is the tentative design flood discharge of 100-year return period with Marikina Dam in the 2015IV&V-FS and this Study. To meet the tentative design flood discharge, it is necessary to implement river improvement, dredging, floodwall heightening and so on in the section.

Regarding the Phase IV section downstream of MCGS to Sto. Niño (Marikina Bridge), only the lower section, i.e., the lower end of Phase IV to the upstream of Rosario Weir (22.5km to 24km from river mouth), has been bigger than the design flood discharge while the other sections were smaller. With the implementation of river improvement works such as construction of river wall, river widening and dredging by Phase IV, the whole Phase IV section will meet the design flood discharge of 2,900 m^3/s.



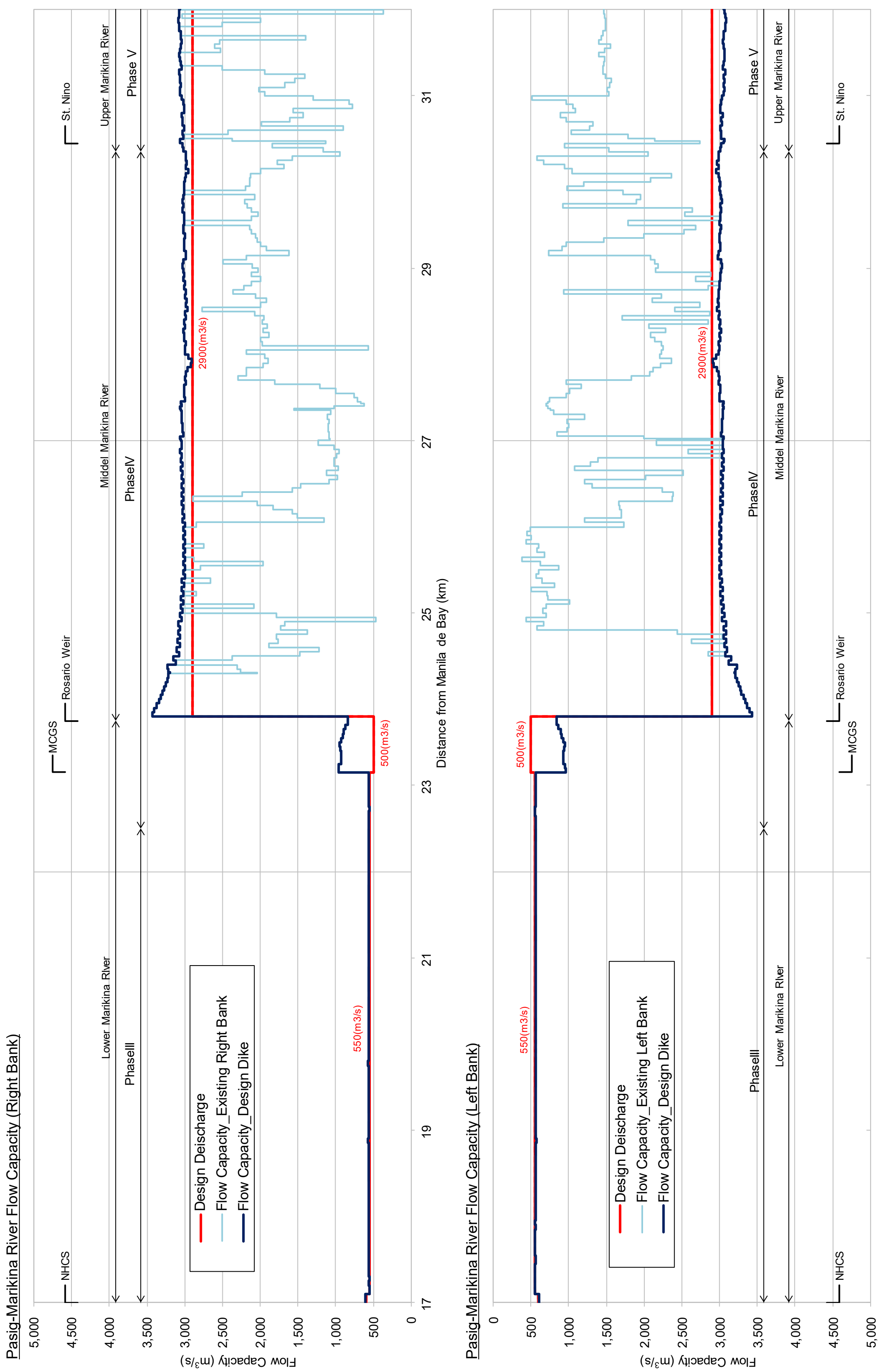
Source: Study Team based on JICA 2011 Preparatory Study and 2015IV&V-FS

Figure 3.1.6 Current Flow Capacity of Pasig-Marikina River



Source: Study Team based on JICA 2011 Preparatory Study and 2015IV&V-FS

Figure 3.1.7 Current Flow Capacity of Pasig River



Source: Study Team based on JICA 2011 Preparatory Study and 2015IV&V-FS

Figure 3.1.8 Current Flow Capacity of Marikina River

3.1.5 Current Operation Manual for Main River Structures

There are two important river structures being operated and managed by MMDA as outlined in **Table 3.1.9**.

Table 3.1.9 Main River Structures located in the Pasig-Marikina River Basin

Structure	Name	Main Dimension		Location
Hydraulic Gate	Napindan Hydraulic Control Structure (NHCS) (Constructed in1982)	Main Gate	Roller Gate: W:15 m× H:9 m×4 Gates	Junction of Pasig River and Napindan Channel (Napindan Channel Side)
		Lock	Radial Gate: W: 19 m× H: 9.59 m×2 Gates Between Locks: 110m	
Weir	Rosario Weir (Movable Weir) (Constructed in 1988)	Gate	Roller Gate (Overflow Type) W: 18.75 m× H:3.5 m×8 Gates	

Source: Study Team based on references

At the above two gates, the operation shown in **Table 3.1.10** is performed based on the water level at the Sto. Niño Bridge (Marikina Bridge: the most upstream point of Phase-IV or the Sto. Nino Station) located at about 6.5 km upstream from the Rosario Weir.

Table 3.1.10 Gate Operation Manual of Rosario Weir and NHCS

Situation	WL at Sto. Niño	Rosario Weir	NHCS	
Water Rising	Normal Condition	All Gates are closed	The main gate of the NHCS is closed as soon as it is informed that the gate opening operation of Rosario Weir has started.	Basically, NHCS must be operated according to the rule on the left, but there is information that is not being strictly followed.
	EL+13.80m	Gate No.4 “Open”		
	EL+13.90m	Gate No.5 “Open”		
	EL+14.0~14.40m	Gate No.3 & 6 “Open”		
	EL+14.50~15.10m	Gate No.2 & 7 “Open”		
	EL+15.30m~Up	Gate No.1 & 8 “Open”		
Water Declining	EL+15.00m	Gate No.1 & 8 “Close”	The main gate of the NHCS is opened as soon as it is informed that the gate closing operation of Rosario Weir has been finished.	
	EL+14.50m	Gate No.2 & 7 “Close”		
	EL+14.00m	Gate No.3 & 6 “Close”		
	EL+13.80m (*1)	Gate No.5 “Close”		
	EL+13.60m (*1)	Gate No.4 “Close”		
	Normal Condition	All Gates are closed		

Source: Study Team based on MMDA EFCOS information

3.2 Existing Flood Management Plan and Related Conceptual Plan

Subsection 3.2.1 summarizes the past flood management plans and flood management related plans in the basin, and **Subsection 3.2.2** summarizes the information on flood control related structures in the basin.

3.2.1 Existing Flood Management Plan

The master plan for flood management of the Pasig Marikina River, including the drainage plan of Metro Manila, was formulated in 1952 and it is the oldest plan on record. Based on this flood management plan, river improvement works have been carried out since the 1980’s.

However, frequent floods in the Pasig Marikina River Basin have not been eliminated. Therefore, in 1986, the Government of the Philippines requested technical cooperation and assistance from the Government of Japan for a study on a flood control plan. Based on this request, JICA implemented the “The Study on Flood Control and Drainage Project in the Metro Manila” and the master plan was formulated in 1990.

Since Typhoon Ondoy caused massive flooding close to 100-year return period in 2009, JICA and the World Bank (WB) started reviewing the flood management plans for the Pasig-Marikina River Basin. The list of studies on flood management plans is given in **Table 3.2.1,** and the outline of each study is presented thereafter.

Table 3.2.1 Past Studies on Flood Management Plan

Project Name	Completion Year	Implementing Agency	Acronym
Formulation of Flood Control Plan in Pasig-Marikina River Basin	1952	Gov't. of the Philippines	1952MP
FS Study and Detailed Design of Manggahan Floodway	1975	USAID	1975FS/DD
The Study on Flood Control and Drainage Project in Metro Manila	1990	JICA	JICA1990MP
Detailed Engineering Design of PMRCIP	2002	DPWH	2002DD
The Preparatory Study on PMRCIP Phase III	2011	JICA	JICA2011 Preparatory Study
Master Plan for Flood Management in Metro Manila and Surrounding Areas	2012	WB	WB2012MP
Data Collection Survey for Flood Management Plan in Metro Manila	2014	JICA	JICA2014 Study
Feasibility Study on PMRCIP Phase IV and V	2015	DPWH	2015IV&V
Feasibility Study and Preparation of Detailed Engineering Design of the Proposed Upper Marikina Dam	2018	WB	WB2018 UMD FS

Source: Study Team

3.2.1.1 Formulation of Flood Control Plan for Pasig-Marikina River Basin, 1952 (1952MP, Government of the Philippines)

(1) Background of the Project

Studies have started in 1943, shortly after the unprecedented flood of November of that year which inundated the city for several days, attaining flood heights higher than any flood previously recorded. A committee of four hydraulic engineers from the then Bureau of Public Works and the Metropolitan Water District was then appointed to study and make a report on the flooding problem, and it submitted an enlightening seven-page report which analyzed briefly the causes of recurrent floods in the city, discussed various schemes for the control of floods, and chartered the course of future investigations for the procurement of data indispensable to the formulation of a definite and workable plan for the solution of the flooding problem. The Liberation cut short further studies on the project and irreplaceable data painstakingly combined in 1944 were all destroyed during the Battle of Manila in World War II.

Investigations were resumed in 1947. Initially the surveys undertaken were primarily for the purpose of controlling the flood flow of the Pasig River. As investigations progressed, however, it became evident that control of the flood flow of the Pasig River alone, would not solve completely the inundation problem in the city. It was said that the disposal of heavy local run-off posed an even more complicated problem, both serious and urgent. In order to solve this problem on river and drainage system, the MP study on drainage measures was to be conducted.

(2) Objectives of the Project

The main objective was to establish master plans of drainage measures for northern and southern Manila. In addition, flood countermeasures for the Pasig Marikina River were studied and proposed.

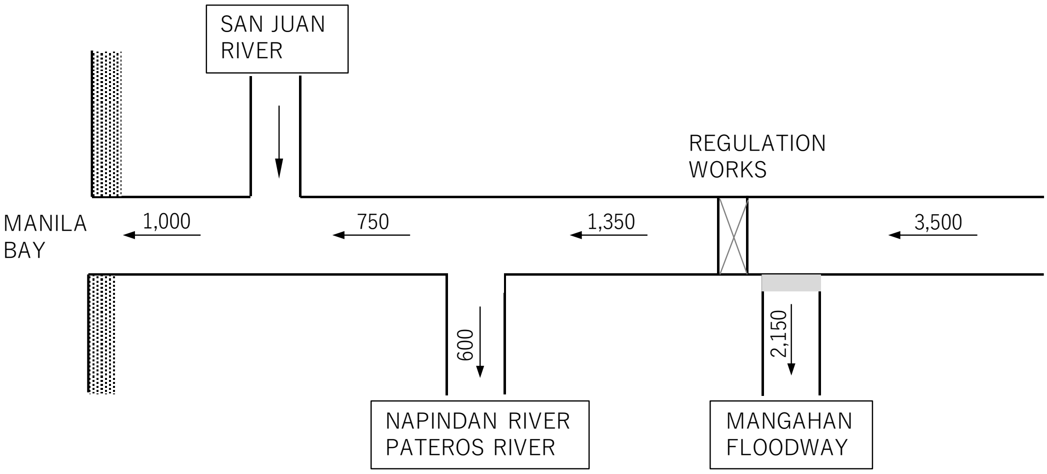
(3) River Improvement Measures and Design Flood Discharge

The main river improvement measures studied and proposed in the 1952MP are as shown in **Table 3.2.2.** The design flood discharge allocation is as shown in **Figure 3.2.1**.

Table 3.2.2 Main River Improvement Works (Targeted the Massive Flood in 1943)

Measures	Contents
River Improvement Works	Pasig River, Marikina River, San Juan River
Marikina Dam (not included in the MP but possibility was studied)	Dam Height: 71.6m
Manggahan Floodway	Floodway Width: 250m, Low Water Channel Width: 100m Bed Level at Inlet is set to +13.0m or Bed Level of Marikina River +3.0m high.
Flood Regulation Works	Equivalent to current MCGS.

Source: 1952MP



Source: Study Team based on 1952 MP

Figure 3.2.1 Design Flood Discharge Allocation (Based on Past Biggest Flood)

3.2.1.2 Feasibility Study and Detailed Design for Manggahan Floodway (1975FS/DD, USAID)

(1) Background of the Project

It has been estimated that an area of 9,480 ha were inundated by the flood that occurred in 1970 in Metro Manila and about 40% of which was damage from the Pasig River while 60% was inland water damage. Before the start of the study, severe flood damage had been experienced 12 times in the past 17 years, affecting important infrastructures. Average annual losses from floods have been estimated at 8.4 million pesos at 1970 prices and 20 million pesos at 1974 prices.

In 1974, the Government of the Philippines requested the Government of Japan to provide assistance for the construction of Manggahan Floodway in order to reduce flood damage along the Pasig River and to speed up the improvement of drainage system against inland water damage.

(2) Objectives of the Study

The objectives of the Study conducted by a US consultant with the support of the USAID were to carry out the feasibility study and detailed design as proposed in the Master Plan of 1952 (1952MP), including the feasibility studies for the Parañaque Spillway which was planned for the drainage of Laguna Lake and the Ring Road (C-6) along the Laguna Lake.

(3) Design Flood Discharge

The design flood discharges of Pasig-Marikina River and the Manggahan Floodway proposed in the Study are as shown in **Table 3.2.3**.

Table 3.2.3 Design Flood Discharge Allocation in 1975FS/DD

Return Period	Discharge at Sto. Niño	Manggahan Floodway	Pasig River
100	3,300	Maximum Design Flood Discharge: 2,400m^3/s	Maximum Design Flood Discharge: 900m^3/s (Increase of flow capacity by construction of Floodwall against the current flow capacity of 600m^3/s at the time was proposed.)
25	3,000		
18	2,900		
10	2,600		
5	2,400		

Source: 1975FS/DD, with further information from the Study Team

(4) Proposed and Designed Structures

Proposed and designed structures in the 1975FS/DD are as shown in **Table 3.2.4**.

Table 3.2.4 Specifications of Manggahan Floodway and Related Structures

Structure	Specifications	Remarks
Marikina River Control Structure (equivalent to current MCGS)	Total Width: 68m (Gate Width: 12m x 5) Total Length: 28m Total Height: 13m (Gate Height: 5m) Elev. of Weir Foundation: EL+10.0m	Gate Type: Radial Orifice Gate Location: Near Sta.+5+400
Rosario Diversion Weir Structure	Type of Fixed Part: Ogee Crest Weir Weir Crest Level: EL+14.0m Weir Foundation Level: Upstream Side: EL+11.5m, Downstream Side: EL+10.0m Total Width: 125m	Concrete Fixed Crest Weir
	Type of Movable Part: Lift Gate Gate Crest Level: EL+14.0m Weir Foundation Level: EL+10.5m Total Width: 10m x 2	Roller Gate
Manggahan Floodway	Length of three-faced Concrete Channel: 1km	Riverbed Width: 80m Face of Slope 2:1(H:V)
	Riprap Channel Length: 1km	Riverbed Width: 80~118m
	Other Length :7km	Riverbed Width: 118m Face of Slope 8:1(H:V)
Others	Bridge	

Source: Study Team based on 1975FS/DD

Based on the design mentioned above, the construction of Manggahan Floodway was initiated using Japanese OECF loan as described in **Subsection 3.2.2.2**.

3.2.1.3 The Study on Flood Control and Drainage Project in Metro Manila, 1990 (JICA1990MP)

(1) Background of the Study

The overall MP for flood protection of the Pasig Marikina River has not been reviewed since 1952 when the works were no longer socio-economic. Under the circumstances, when President Aquino visited Japan in November 1986, technical cooperation was formally requested for "The Study on Flood Control and Drainage Project in Metro Manila." In response to the request, JICA launched the study to improve flood protection and drainage in Metro Manila.

(2) Objectives of the Study

The objectives of the study were to formulate the framework plan (FP) for future comprehensive flood control measures and the MP with the target year 2020, including the feasibility study (FS) for priority areas in Metro Manila and suburban areas.

(3) Flamework Plan (FP)

The proposed river improvement measures in the FP are as shown in **Table 3.2.5**. The design flood discharge allocation is the same as the MP shown in **Figure 3.2.2.**

Table 3.2.5 Main River Improvement Measures of the Framework Plan (100-Year Return Period Flood)

Measures	Contents
River Improvement Works	Pasig River, Marikina River, San Juan River, Napindan River
MCGS	Location: Downstream of Rosario Weir (Marikina River) Gate Type: Roller Gate Weir Height: 15m
Marikina Dam	Location: Montalban Gorge (100m upstream of the existing Wawa Dam) Dam Type: Concrete Gravity Dam Height: 70m
Parañaque Spillway	Design Discharge: 500m^3/s (max) Length: 9,200m
Laguna Ring Dike	Length: 10,700m Crest Level: EL+14.20m, Freeboard: 1.7m

Source: JICA1990MP

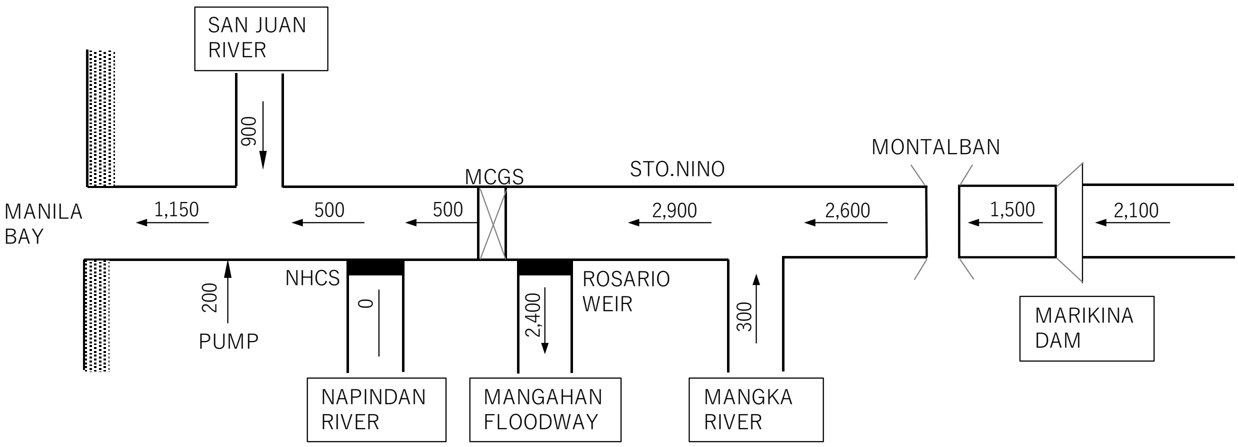
(4) Master Plan (JICA1990MP)

The proposed river improvement measures in the MP are as shown in **Figure 3.2.6**. The design flood discharge allocation (100-year) is as shown in **Figure 3.2.2**.

Table 3.2.6 Main River Improvement Measures in the Master Plan

Measures	Contents
River Improvement Works	Pasig River, Marikina River, San Juan River, Napindan River
MCGS	Location: Downstream of Rosario Weir (Sta. 5+425) (Marikina River) Gate Type: Roller Gate Weir Height: 15m
Marikina Dam	Location: Montalban Gorge (100m upstream of the existing Wawa Dam) Dam Type: Concrete Gravity Dam Height: 70m Flood Control: Natural (No Gate)
Laguna Ring Dike	Length: 10,700m Crest Level: EL+14.20m, Freeboard: 1.7m
Non-structural Measures	Pasig-Marikina River: Effective Flood Control Operating System (EFCOS)

Source: JICA 1990MP



Source: JICA1990MP

Figure 3.2.2 Design Flood Discharge Allocation (100-Year, JICA1990MP)

(5) Feasibility Study (JICA1990FS)

The following priority projects were proposed in the Feasibility Study:

- Drainage improvement in East and West Manggahan
- Drainage improvement in Malabon-Navotas
- River improvement of Pasig-Marikina River (downstream of Manggahan Floodway junction, except for San Juan River)

3.2.1.4 Detailed Engineering Design of PMRCIP (2002DD, DPWH)

(1) Background of the Study

To cope with the frequent floods, DPWH carried out a project review through the Japan's Special Assistance for Project Formation (SAPROF) in 1998, including a review of the FS on the "Pasig Marikina River Improvement Project (PMRCIP)" based on the aforementioned JICA1990MP and the JICA1990FS. As decided, the PMRCIP project is to be implemented in the following four (4) phases with Japanese ODA Loan. (Part of project component changed later.)

Phase I: Detailed Design of Whole Project: Del Pan Bridge to Marikina Bridge (29.7km) (hereinafter, "2002DD")

Phase II: Stage I Construction: River Improvement Works in Pasig River - Del Pan Bridge to Napindan River (16.4km)

Phase III: Stage II Construction: River Improvement Works in Downstream of Marikina River (including construction of MCGS) - Junction with Napindan River to Junction with Manggahan Floodway (7.2km)

Phase IV: Stage III Construction: River Improvement Works from the Upstream of Marikina River: Junction with Manggahan Floodway to Marikina Bridge (6.6km)

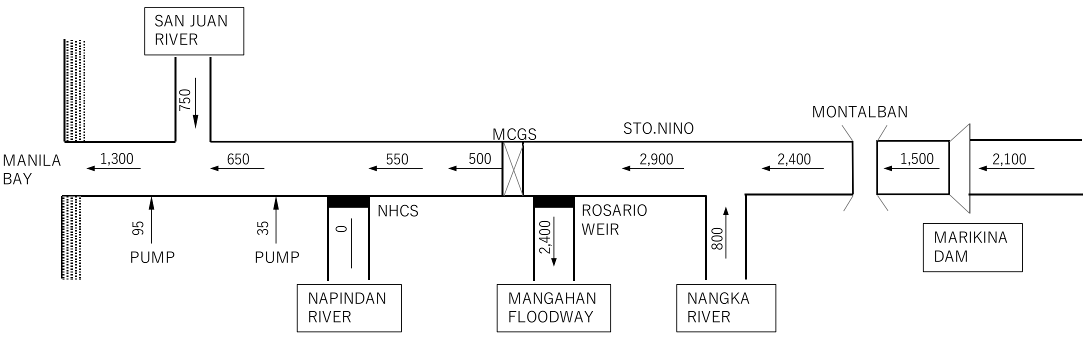
(2) Objectives of the Study

The 2002DD has been positioned as Phase I of the whole PMRCIP in which the detailed design, review of design flood discharge of the JICA 1990 MP, and the setting of immediate target discharges for the present development were carried out. The objectives of the study were as follows:

- To mitigate flood damage caused by channel overflow and spill of the Pasig-Marikina Rive; and
- To enhance the favorable environment and aesthetic view along the riverine areas.

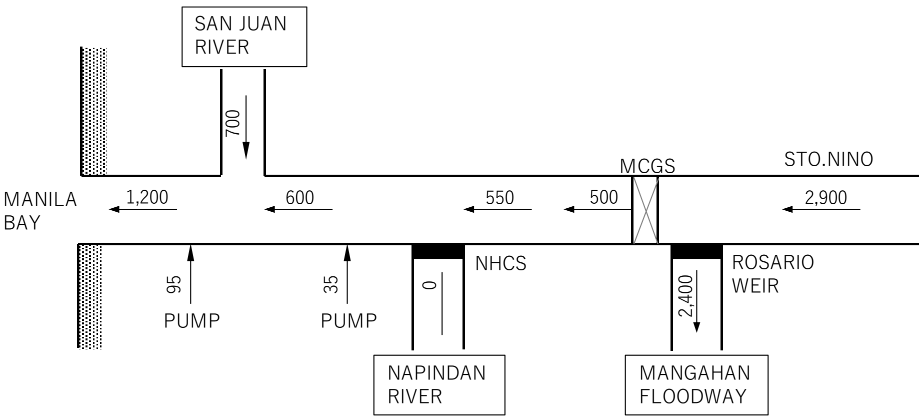
(3) Design Flood Discharge and Immediate Target Flood Discharge

The review of design flood discharge of JICA1990MP and the setting of immediate target flood discharge (30-year return period, without dam) have been carried out. The design flood discharge and the immediate target flood discharge set in the 2002DD are as shown in **Figure 3.2.3** and **Figure 3.2.4**.



Source: 2002DD

Figure 3.2.3 Design Flood Discharge Allocation (100-Year Design Flood, 2002DD)



Source: 2002DD

Figure 3.2.4 Immediate Target Flood Discharge Allocation (30-Year Design Flood, 2002DD)

3.2.1.5 The Preparatory Study for PMRCIP Phase III (JICA2011Study)

(1) Background of the Study

In September 2009, Typhoon Ondoy caused massive flood damage in Metro Manila. It was hence recognized that the early completion of the entire PMRCIP is an urgent task for the protection of Metro Manila from flood damage. In line with this, a preparatory study was conducted by JICA, aiming at the provision of ODA loan for Phase III (September 2010 to October 2011).

(2) Objectives of the Study

The objectives of the JICA2011Study are as recapitulated below:

- To review the then existing Pasig-Marikina River Channel Improvement Project Plan (PMRCIP Plan) focusing on the river improvement stretch covered by Phase III in the course of the study for the whole river improvement stretch (from the river mouth to Marikina Bridge) in the Pasig-Marikina River Basin, including the present river conditions reflecting recent river basin development, recent flood damage conditions, and the impacts to flood damage by future climate change.
- To provide support for the formulation of a Yen-Loan Project as the "Phase III" project, consisting of river channel improvement works, including monitoring, information campaign and publicity towards the local inhabitants, and so on.

(3) Immediate Target Flood Discharge

Although the review of the immediate target flood discharge (30-year design flood) has also been considered, it was decided to follow the discharge allocation set in the 2002DD shown in **Figure 3.2.4**.

3.2.1.6 Master Plan for Flood Management in Metro Manila and Surrounding Areas (WB2012MP,)

(1) Background of the WB2012MP

After the massive flood damage in Metro Manila caused by Typhoon Ondoy in September 2009, this study was conducted to establish the overall vision and road map for sustainable and effective flood risk management (FRM) in Metro Manila and Surrounding Areas.

(2) Objectives of the WB2012MP

The specific objectives of the WB2012MP are as follows:

- To carry out a flood risk assessment study for Metro Manila and Surrounding Areas;
- To prepare a comprehensive flood risk management plan; and
- To propose a set of priority structural and non-structural measures that will provide sustainable flood risk management up to a certain safety level.

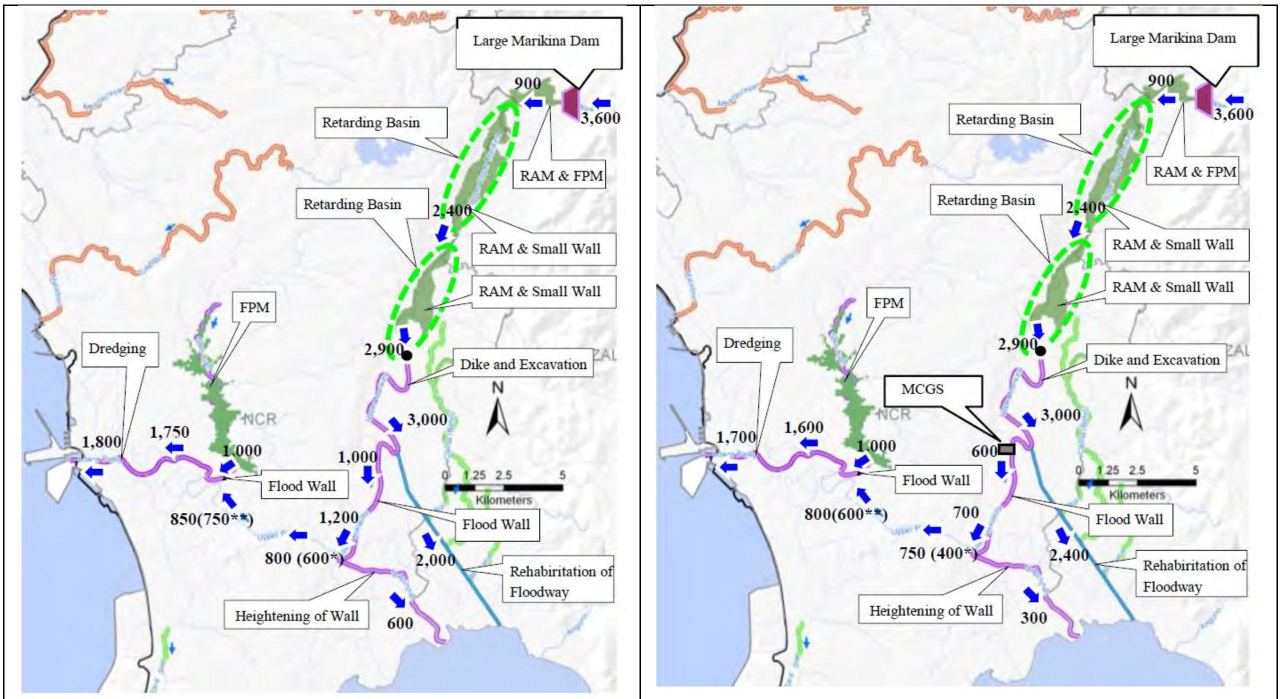
(3) River Improvement Measures and Design Flood Discharge

The WB2012MP has recommended two (2) alternatives. One is "Without MCGS" (Alternative-2) and the other is "With MCGS" (Alternative-3). In addition, it proposes conducting a feasibility study to decide on the necessity of the MCGS. The main river improvement measures and design flood discharge allocation of the proposed alternatives are as shown in **Table 3.2.7** and **Figure 3.2.5**.

Table 3.2.7 Main River Improvement Measures proposed in WB2012MP

Measures	Alternative-2	Alternative-3
River Improvement Works	Pasig River, Marikina River, San Juan River, Napindan Channel, Manggahan Floodway *Dredging of Pasig River, additional works in Phase II, III and IV sections, and embankment heightening of Napindan Channel are required.	Pasig River, Marikina River, San Juan River, Napindan Channel, Manggahan Floodway *Dredging of Pasig River, additional works in Phase II and III sections, and embankment heightening of Napindan Channel are required.
MCGS	Without (No Construction)	With (Construction)
Marikina Dam	Location: 500m Upstream of the existing Wawa Dam Dam Height: 72m Storage Volume: 67.4MCM	Location: 500m Upstream of the existing Wawa Dam Dam Height: 72m Storage Volume: 67.4MCM
Natural Retarding Basin	980ha	980ha

Source: WB2012MP



Alternative-2	Alternative-3

Source: WB2012MP

Figure 3.2.5 Design Flood Discharge Allocation (100-Year, WB2012MP)

3.2.1.7 Data Collection Survey on Flood Management Plan in Metro Manila (JICA2014Study)

(1) Background of the JICA2014Study

The JICA2014Study has been conducted to prepare basic information that could contribute to the development of a more detailed flood control plan through the review of previous survey results (specially design flood discharge in WB2012MP) taking into account climate change in the target areas.

(2) Objectives of the JICA2014Study

The objective of the JICA2014Study was to reexamine the technical validity of the structural measures for the Pasig-Marikina River Basin proposed in the WB2012Study by utilizing the hydrological and hydrodynamic flood simulation model to be refined and updated with appropriately selected datasets in consideration of the future climate change thereby bridging the concept planning and the actual implementation of projects.

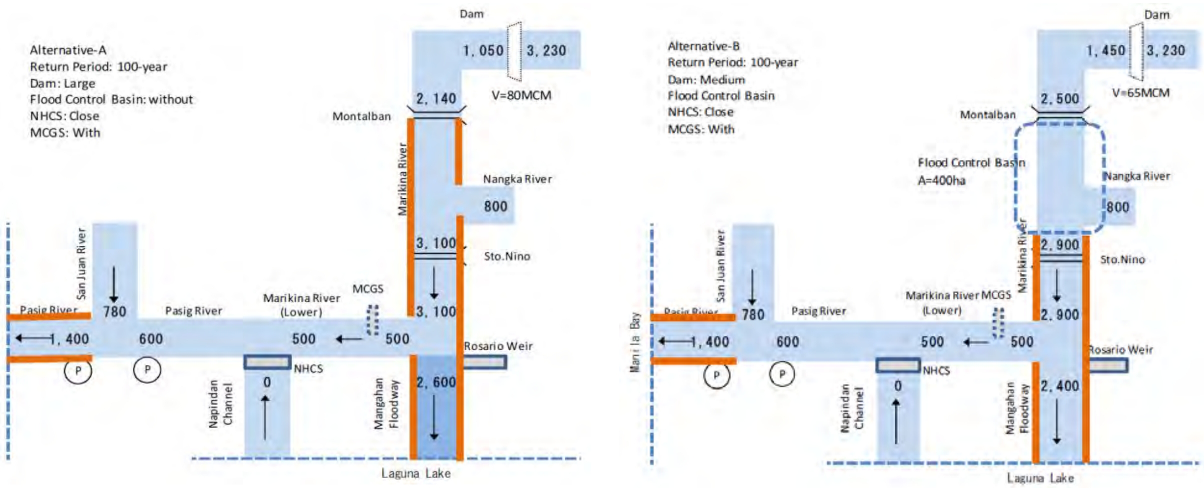
(3) River Improvement Measures and Design Flood Discharge

In the JICA2014Survey, the examination of design flood discharge was also carried out. The river improvement works and design flood discharge allocation of the recommended alternatives are as shown in **Table 3.2.8** and **Figure 3.2.6**.

Table 3.2.8 Main River Improvement Measures considered in JICA2014Study

Measures	A-2-1	B-3
River Improvement Works	Pasig River, Marikina River, San Juan River, Manggahan Floodway *Partial heightening of floodwall in Phase II and IV sections and additional dredging of the Manggahan Floodway are required.	Pasig River, Marikina River, San Juan River, Manggahan Floodway *Partial heightening of floodwall in Phase II is required. There are no additional works in Phases III and IV sections.
MCGS	With (Construction)	With (Construction)
Marikina Dam	Location: 500m Upstream of the existing Wawa Dam Dam Height: 68m Storage Volume: 80MCM	Location: 500m Upstream of the existing Wawa Dam Dam Height: 71m Storage Volume: 65MCM
Retarding Basin	Without (None)	371ha (8 sites)

Source: JICA2014Study



A-2-1 B-3

Source: JICA2014Study

Figure 3.2.6 Design Flood Discharge Allocation (100-Year, JICA2014Study)

3.2.1.8 Feasibility Study of PMRCIP Phases IV and V (DPWH2015IV&V-FS)

(1) Background of the DPWH2015IV&V-FS

In September 2009, Typhoon Ondoy caused severe flood damage. Since then, it was recognized by DPWH that Phase IV needs to be implemented as soon as possible. In addition, to commence Phase IV promptly, relevant organizations have pointed out that the detailed design carried out as Phase I in 2002 needs to be significantly revised due to the developments along the river channel, and that the economic efficiency of the project has changed due to the change in the probable discharge caused by Typhoon Ondoy. Therefore, a reinvestigation equivalent to FS is necessary.

Furthermore, since the upstream section of Phase IV was severely damaged during Typhoon Ondoy, it is necessary to develop the damaged section and conduct a study on the entire PMRCIP, including Marikina Dam and the retarding basin.

Moreover, DPWH extended the river improvement works section from the upstream end of Phase IV (to be supported by JICA) to San Mateo Bridge as the Phase V section.

As a result, the implementation plan of PMRCIP has been revised, as shown in **Table 3.2.9**.

Table 3.2.9 Revised Implementation Plan of PMRCIP under the DPWH2015IV&V-FS

Phase	Revised Project Components	Improvement Section (Design Flood Discharge)
II	Pasig River Channel Improvement (Del Pan Bridge to Napindan River)	Both riverbanks: 13.1km (1,200 / 600 m^3/s)
III	Downstream of Marikina River Channel Improvement (Napindan River to Downstream of MCGS)	5.4km (550 m^3/s)
	Pasig River Channel Improvement (Non-targeted section in Phase II)	Both riverbanks: 9.9km (1,200 / 600 m^3/s)
IV	Middle stream of Marikina River Channel Improvement and Construction of MCGS (MCGS to Marikina Bridge)	8.0km (2,900 m^3/s)
V (Additional)	Upstream of Marikina River Channel Improvement (Marikina Bridge to San Mateo Bridge)	5.8km (2,900 m^3/s)

Source: Study Team

(2) Objectives of the DPWH2015IV&V-FS

The objectives of Phase IV and Phase V as a part of the Pasig-Marikina River Channel Improvement Project confirmed in the DPWH2015IV&V-FS are as follows:

- To mitigate frequent inundation or massive flooding by the overflow of Pasig-Marikina River resulting in severe damage to lives, livestock, properties and infrastructure, aiming at the alleviation of damage to living and sanitary conditions in Metro Manila;
- To create a more dynamic economy by providing a flood-free urban center as an important strategy for furthering national development; and
- To rehabilitate and enhance the environment and aesthetic view along the riverine areas by providing more ecologically stable conditions to arrest the progressive deterioration of environmental conditions, health and sanitation in Metro Manila.

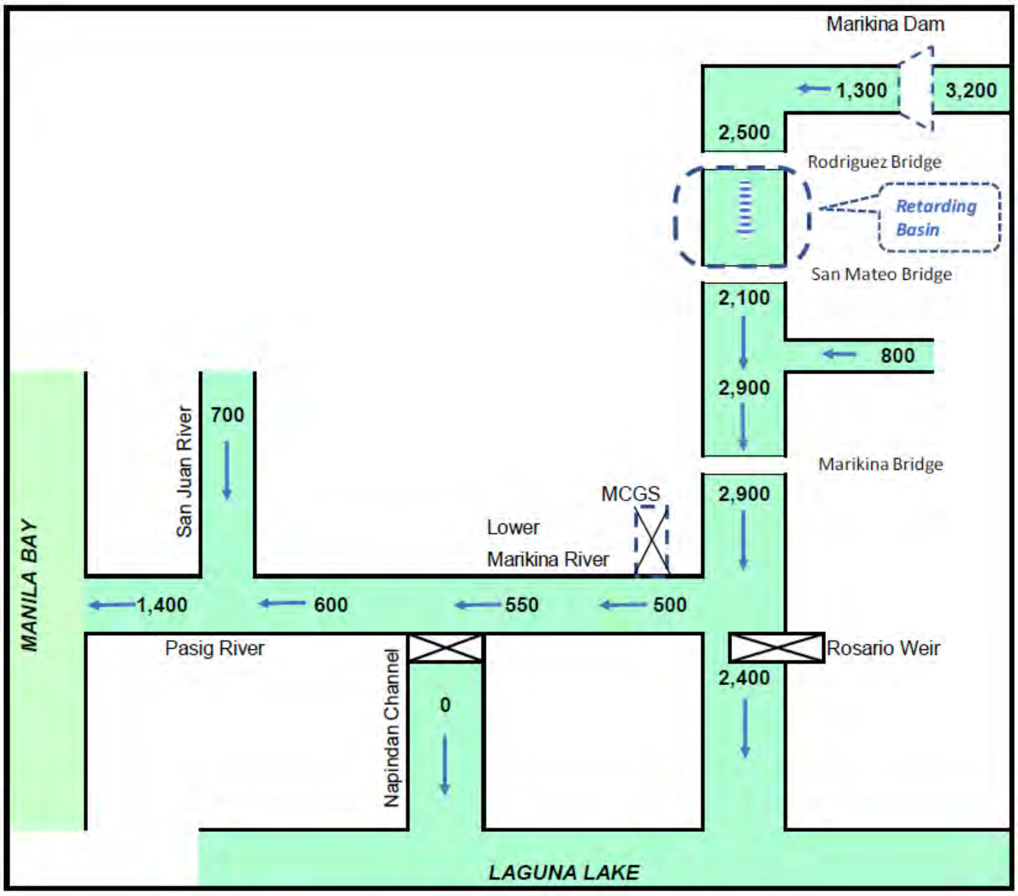
(3) River Improvement Measures and Design Flood Discharge

The design flood discharge has been reviewed based on the changes in location of the retarding basin in accordance with the development of the Phase V section, as shown in **Figure 3.2.7**. This design flood discharge allocation is hereinafter referred to in this report as “design flood discharge allocation set by 2015IV&V-FS”.

Table 3.2.10 Main River Improvement Measures confirmed in the DPWH2015IV&V-FS

Measures	Contents
River Improvement Works	Pasig River, Marikina River, San Juan River, Manggahan Floodway
MCGS	With (to be constructed)
Marikina Dam	Location: Not analyzed Dam Height: 64m Storage Volume: 64.2 MCM
Retarding Basin	337ha (7 sites)

Source: DPWH2015IV&V-FS



Source: 2015IV&V-FS

Figure 3.2.7 Design Flood Discharge Allocation (100-Year, DPWH2015IV&V-FS)

3.2.1.9 Feasibility Study and Preparation of Detailed Engineering Design of the Proposed Upper Marikina Dam (WB2018UMD)

(1) Background of the WB2018UMD Study

This WB2018UMD Study was to conduct the FS and DD of for the Upper Marikina Dam which is necessary to complete the whole PMRCIP. The WB2018UMD study was funded by a grant from the WB.

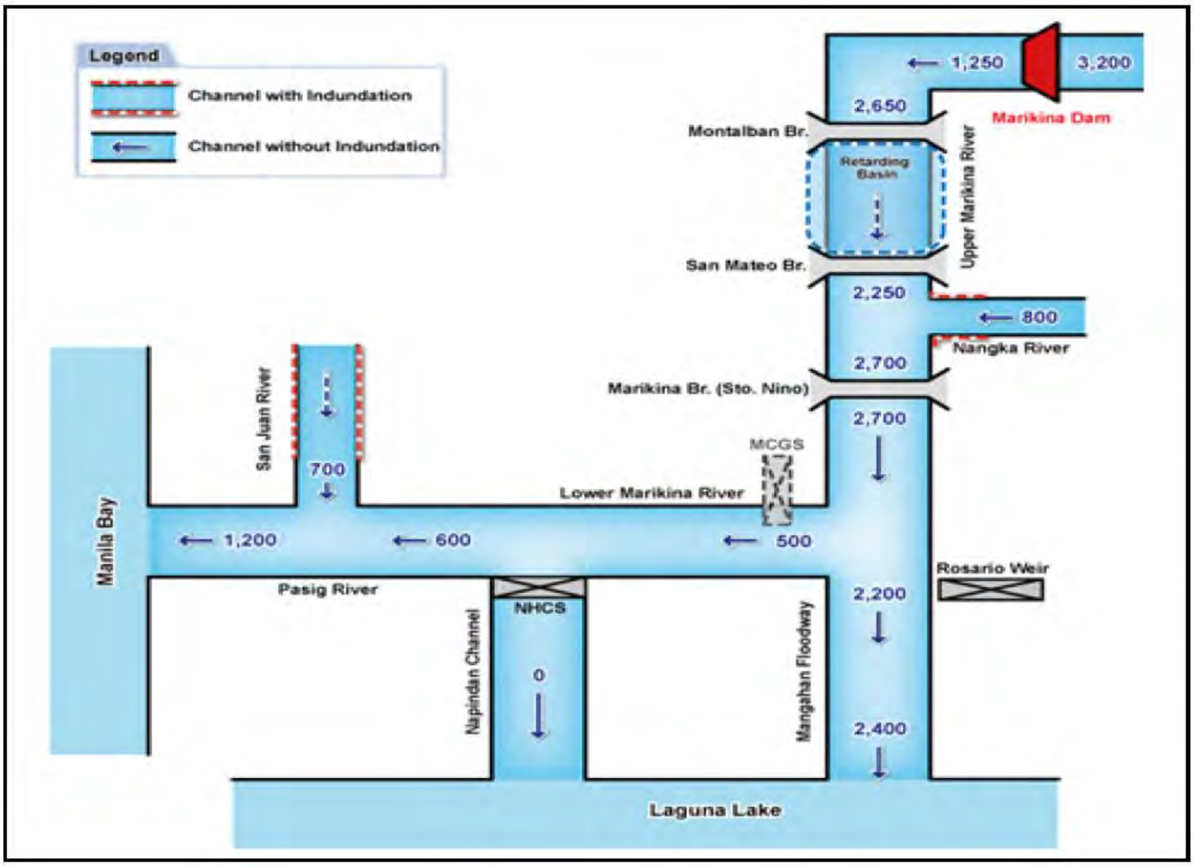
(2) Objectives of the WB2018UMD Study

The basic objective of the WB2018UMD study was to determine the preferred option for a flood management structure to reduce the water discharge from the Marikina River before it enters Metro Manila through a feasibility study leading to the preparation of detailed designs and tender documents.

(3) Design Flood Discharge re-confirmed in the WB2018UMD Study

The design flood discharge has been set based on the hydraulic analysis model made in the WB2012MP.

Based on the hydraulic analysis model of WB2012MP, the design flood discharge allocation was set, considering the Marikina Dam and the retarding basin (See **Figure 3.2.8**). As a result, the design flood discharge at Marikina Bridge (Sto. Niño) became 2,700 m^3/s.



Source: WB2018 UMD FS

Figure 3.2.8 Design Flood Discharge Allocation (100-Year Design Flood, WB2018UMD)

3.2.2 Major Flood Management Projects and River Structures in Pasig-Marikina River Basin

3.2.2.1 Napindan Hydraulic Control Structure (NHCS)

(1) NHCS Construction Project

The Napindan Hydraulic Control Structure (NHCS) is a floodgate built on the Napindan Channel side of the confluence of Pasig River and the Napindan Channel, the only existing natural channel connecting Laguna Lake and the Pasig River (See **Location Map** of the study area).

The NHCS has been proposed for the following purposes by the "Laguna Lake Resources Development Project (Societe Grenobloise d'Études et d'Application Hydrauliques: SOGREAH 1974)", which was jointly formulated by UNDP and ADB in 1974:

- Water resources development in Laguna Lake (Water level control);
- Prevention of saline and polluted water intrusion into Laguna Lake from Pasig River (Desalination);
- Securing boat transport between Manila Bay and Laguna Lake when the gates are closed (Construction of lock gate); and
- Prevention of flooding around Laguna Lake.[2]

The construction of NHCS was completed with ADB loan in 1982. The specifications of NHCS are as shown in **Table 3.2.11.**

[2] Excerpt from "Pasig River Comprehensive Development Plan Study Report (March 1986)" by IDI. However, no information is available on where and how to prevent flooding.

Table 3.2.11 Structural Specifications of NHCS

Item	Contents
Main Gate	Roller Gate: W: 15 m × H:9 m × 4 Gates
Lock Gate	Total Length: 110m (between Lock Gates) Radial Gate (Lock Gate): W: 19 m × H: 9.59 m × 2 Gates
Main Foundation	Pile Foundation Structures

Source: Study Team based on references

(2) Current Condition

1) Operating Condition

Since its completion in 1982, the NHCS has never fully closed its main gates to secure Laguna Lake water resources and for desalination. This is because fishermen operating in Laguna Lake are opposed to the complete closure of the gates, fearing that desalination of Laguna Lake will reduce their catches.

In Phase I of this project, it was proposed that the NHCS should be operated by temporarily closing the gates when Pasig-Marikina River floods occur after the completion of MCGS.

In the "EFCOS Advisor Report (2016)" which will be described later, there is no official record of NHCS operation, and it is a future task.

2) Aging Problem

In February 2018, the Ministry of Economy, Trade and Industry of Japan conducted the "Study on Floodgates Rehabilitation Project in Metro Manila, Republic of the Philippines" (hereinafter, "Study on Floodgates Rehabilitation Project (2018)" which investigated the necessity of rehabilitation due to deterioration of the gate.

In this study, the whole reconstruction with gate replacement of NHCS was studied together with economic evaluation to find a solution to the problems on the deterioration of facilities and the response to the Level 2 earthquake ground motion. As a result, it was proposed that the gate should be rebuilt, subject to further detailed study.

Table 3.2.12 shows the reasons why the gates need to be rebuilt.

Table 3.2.12 Reasons why the NHCS should be rebuilt

Item	Contents
Concrete Structure	Reinforcing bars and auxiliary reinforcing bars used at the time of construction are exposed due to exfoliation of the concrete surface, and there is a possibility that corrosion inside the structure main body is progressing.
Exterior Structure	Corrosion at the Pier-5 stairs is so severe that inspection work cannot be carried out.
Gate Body	The lower part of the gate body is severely damaged, and the paint film has disappeared, and there are through-holes everywhere, which cause a part of the function to be impaired. (There is a low possibility that durability will be improved by repainting/reinforcement, etc.)
Strength against Earthquakes	It is difficult to ensure structural safety when the design load is applied.
Correspondence to New Design Standard	In the Philippines, if the river structure is to be constructed or developed in the future for the "Level-2" design adopted in the bridge design standards, this gate is difficult to be reinforced, and it is desirable to rebuild it.

Source: Study Team based on references

(3) Flood Control Functions and Plans of NHCS

In the case of NHCS, it is possible to reduce the discharge downstream of the main river by opening the gates during floods if the water level of Laguna Lake is low. However, if the water level of Laguna Lake is high, it is impossible to flow a certain discharge into the Napindan Channel.

Therefore, the design flood discharge of Napindan Channel in the flood control plan is zero (0).

3.2.2.2 Manggahan Floodway Construction Project

As described in detail in **Subsection 3.2.1**, the Philippine government and the Japanese government signed an agreement on the construction of Manggahan Floodway with OECF loan in September 1975 based on the results of the FS/DD study on the Manggahan Floodway conducted by DMJM of USA in 1975 and the construction of Manggahan Floodway started with Japanese ODA loan. The outline of the construction project is given below.

(1) Review of FS/DD and Design Changes just before starting Construction

The review on the results of the FS and DD studies started in August 1977. The detailed design of the Manggahan Floodway was completed in November of the same year, and bidding for construction works started.

However, a part of the plans for the Floodway were opposed, and the DD review was implemented by the Task Force created just before execution of the contract.

The major items of the DD review were the following:

1) Cancellation of MCGS Construction

Construction of MCGS was halted because of the significant impact on shipping from Pasig River to Marikina River.

2) Change of Rosario Weir from Fixed Weir to Movable Weir

The design of Rosario Weir was revised in the DD in 1975 from fixed weir to movable weir mainly as a response to floods around the Laguna Lake that occur frequently at that time, such as the largest flood in Laguna Lake in 1972[3].

(2) Outline of the Works

Due to the repeated design changes and delays in land acquisition, the construction of Manggahan Floodway met its loan disbursement deadline on December 31, 1984 with a completion rate of 75%. Therefore, the remaining 25% was completed under the Philippine government's own budget in 1988.

The outline of the project is given in **Table 3.2.13**.

Table 3.2.13 Outline of the Manggahan Floodway Project

<table>
<tr><th>Item</th><th colspan="2">Contents</th></tr>
<tr><td>Outline of Yen Loan</td><td colspan="2">Signing Date: July 4, 1975 (L/A No. PH-P10)
Contract Amount: 2,704 Million Yen
Execution Expenses: 2,073 Million Yen</td></tr>
<tr><td>Construction Period (For 75% completion, except the 25% remaining works)</td><td colspan="2">May 1980 to April 1985</td></tr>
<tr><td rowspan="7">Contents of the Project</td><td rowspan="3">Floodway
(Total Length: 9.00km)</td><td>• Concrete-lined Channel: 1.15km</td></tr>
<tr><td>• Stone-Lined Channel (Transition Segment): 1.00km</td></tr>
<tr><td>• Sand Channel (Width of Low Water Channel: 118m / Embankment Slope: 1:8): 6.85km</td></tr>
<tr><td>Rosario Weir</td><td>Roller Gate (Overflow Type):
W: 18.75 m × H:3.5 m × 8 Gates</td></tr>
<tr><td>Ortigas Bridge</td><td>Width: 23.614m (6 Lanes)
Span: 32.10m (4 Spans)</td></tr>
<tr><td>Bank Road</td><td>Width of East Bank Road (Left Bank Crest): 13.7m (4 Lanes)
Width of West Bank Road (Right Bank Crest): 7.0m (2 Lanes)</td></tr>
<tr><td>Transfer</td><td>Implemented relocation of approximately 450 ISFs</td></tr>
</table>

Source: Study Team based on references

(3) Flood Control Functions and Plans of Manggahan Floodway

As for the Manggahan Floodway, flowing the design flood discharge of 2,400 m^3/s safely into the Laguna Lake and maintaining the functions of the conventional flood control plan have been the basis of the flood control plan.

[3] The objective is to lower the water level of Laguna Lake as early as possible by opening the movable Rosario Weir when the water level of Laguna Lake is high.

3.2.2.3 The Effective Flood Control Operation System (EFCOS) Project

(1) Beginning of EFCOS Project: System Construction by Yen Loan Project

1) Background of the Project

This EFCOS project has been one of the projects related to the Manggahan Floodway that was constructed for the flood control of Metro Manila based on the "Pasig River Flood Control Project" in 1952 as described in **Subsection 3.2.1.1**. In the Manggahan Floodway construction project, the Rosario Weir which is the water inlet of Manggahan Floodway was initially planned as a fixed weir.

However, after the project started, extensive flood damage occurred around the Laguna Lake, the outlet of the Manggahan Floodway. Therefore, as one of the reasons for the reduction of flood damage around the Laguna Lake, the weir was redesigned into a movable weir which can drain floodwaters through the Manggahan Floodway when the water level of Laguna Lake is high.

The MCGS was, however, not constructed after the completion of Manggahan Floodway (June 1984), but the gate opening operation of the Rosario Weir during flood was started for the following purposes:

- To reduce the water level upstream of the Marikina River; and
- To reduce the discharge into the Pasig River.

As a result, gate operation made a rapid change of discharge in the floodway, so that a warning system for the protection of human lives and properties and a telemetry system for the early and effective operation of the weir became indispensable.

2) Objectives of the EFCOS Project

The objectives of this EFCOS project were to install a flood warning system for the Manggahan Floodway to prevent the damage of human lives and properties from the sudden change of flood flow in the floodway caused by the operation of the weir, and to make effective operation of the weir by the telemetry system.

3) Contents of the EFCOS at Initial Stage

The EFCOS project was implemented as the Yen Loan project concluded in 1983. The contents are as summarized in the following table.

Table 3.2.14 Outline of the EFCOS Project

Item	Contents
Implementing Agency	DPWH
Accepted Amount / Implemented Amount	1,140 Million Yen / 1,036 Million Yen
Implementation Period	December 1984 to October 1995
Rainfall Station	2 Stations (Mt. Oro, Boso-Boso)
Gauging Station	5 Telemetry Stations (Montalban / Sto. Niño / Angono / Pandacan / Fort Santiago) 4 Other Stations (Rosario Weir Upstream / Downstream / NHCS Pasig Side / Napindan Side)
Relay Station	PAGASA, Antipolo
Operation and Monitoring Post	Rosario (Main Post) and 3 Other Posts (DPWH-NCR Head Office / DPWH Central Office / NHCS)
Warning/Siren Post	Rosario (Main Post) 8 Other Posts (along Manggahan Floodway)

Source: Study Team based on references

Figure 3.2.9 shows the initial system configuration of EFCOS.

Source: EFCOS Office

Figure 3.2.9 System Configuration of EFCOS (Phase 1)

(2) EFCOS Rehabilitation Project by JICA Grant

1) Background of the Grant Project

In 2000, it became difficult for the system constructed with ODA loan to predict floods with high accuracy, and it was not able to respond adequately to small and medium-sized floods caused by urbanization and population concentration in the basin. In addition, the spread of mobile phones and the development of communication devices has caused frequent interference in radio communications, resulting in problems with radio-wave management. In view of this situation, it has been required to carry out accurate flood forecasting by first reviewing the entire EFCOS system.

2) Objectives of the Grant Project

The objectives of the grant project was to review the whole system of EFCOS, improve the system for more accurate flood forecasting, and procure equipment necessary for functional enhancement.

3) Project Contents

In this grant project, hydrological stations were added, the telemeter system was digitized, computers became on-line, and the system for automatic collection and processing of hydrological data was constructed. In addition, a flood prediction system was introduced to read these data, and the results were converted into images and displayed on TV screens. Moreover, a wireless communication system was installed, interconnecting drainage pump stations along the Pasig River and in local government offices to enable utilization of the predicted information for flood planning and implementing countermeasures. Regarding the operation of the system after equipment procurement, technical guidance was provided with the introduction of non-structural components.

The outline of the grant aid rehabilitation project is given in **Table 3.2.15**.

Table 3.2.15 Outline of the EFCOS Rehabilitation Project

Item	Contents	
Implementing Agency	DPWH-PMO (Current DPWH-UPMO-FCMC)	
Project Operation and Management	DPWH-NCR EFCOS Office (Currently, MMDA EFCOS Office)	
Project Contents	Improvement and Development of Hydrological Observation System	2 Water Gauge Stations (New) 5 Rain Gauge Stations (New)
	Improvement of Telecommunication System	Analog System: Upgraded to Digital Telemetry System Telemetry System: Changed to 424.750 MHz Dam Release Warning System: Changed to 424.900 MHz Multiple Communication System: Changed to 7.5 GHz, 22 GHz
	Improvement of Data Processing System	• Automatic online data collection of hydrological and gate information; storage in database. • Conversion of collected and processed data into images and their transmission to the monitoring stations such as DPWH and DPWH-NCR. • Introduction of flood forecasting system and water level forecasting for gate operation of the weir. • Installation of Backup System.
	Installation of Emergency Radio System	Flood forecast information is transmitted with PAGASA as the relay station: • Transmission of flood forecasting information to 11 pumping stations along Pasig River to increase pump operation efficiency • Installation of radio system for flood control at 27 locations targeting metropolitan governments, DPWH-NCR, regional offices and related organizations, etc.
	Non-structural Components	• Technical Assistance (TA): For modification and updating of flood forecasting model (evaluation of model error) and development of technical guidelines. • Operation Assistance: Updating of EFCOS system manual created in 1993 and preparation of instruction manual for overall operation of the entire system.
Total Project Cost	1,200 Million Yen (Grant: 1,100 Million Yen, DPWH-PMO: 100 Million Yen)	

Source: Study Team based on references

Figure 3.2.10 shows the system configuration of the EFCOS that was improved under the EFCOS Rehabilitation Project.

Source: EFCOS Office

Figure 3.2.10 Improved System Configuration of EFCOS (Phase 2)

4) Transfer of EFCOS Project

After the implementation of the above-mentioned grant projects, the operation of EFCOS was transferred by the DPWH to the MMDA in 2002.

(3) Improvement/Restoration of Telemetry Equipment of Effective Flood Control Operation System (EFCOS)

1) Background of the Project

Due to the repeated onslaught of large-scale typhoons/tropical storms in and nearby Metro Manila such as the typhoons Milenyo in September 2006 and Ondoy in September 2009, the EFCOS facilities including the monitoring and telecommunication systems were heavily damaged, resulting to the malfunction of the whole system. The Government of the Philippines (GOP), through the Metropolitan Manila Development Authority (MMDA), exerted efforts to repair EFCOS using its own budget; however, complete rehabilitation with its own budget has been very difficult since the damaged equipment/system included major and expensive parts which also require technical examination for rehabilitation.

In recent years, floods brought by typhoons and monsoonal storms have been occurring in Metro Manila more frequently than before. Therefore, rehabilitation of the EFCOS has become an urgent and critical issue. Under the situation, the GOP requested assistance from the GOJ for the "Project for the Improvement/Restoration of Telemetry Equipment of the Effective Flood Control Operation System" (hereinafter, the "EFCOS Project"), aiming to restore the function of EFCOS by rehabilitating the damaged facilities/equipment.

In response to this request, JICA provided support for part of the project and provided post-assistance advisors to assist in the appropriate management and maintenance of the EFCOS.

2) Outline of the EFCOS Project

As mentioned above, the project was implemented partly through JICA's support to the Philippine government. The details of the project are given in **Table 3.2.16**.

Table 3.2.16 Outline of the EFCOS Project by the Government of the Philippines

Particulars	Contents	
Agreement Date on Project Implementation between JICA and GOP	October 2014	
Implementation Period	Civil Works (GOP): October 2014 to January 2016 Equipment Installation Works: June 2015 to February 2016	
Contents of the Project	Civil Works (MMDA)	• Heightening of Nangka Station
		• Heightening of Warning Post (Warning Post No. 8)
		• Improvement of EFCOS Project Room in Napindan Operation Building
		• Construction of Transmission and Reception Tower to NHCS
		• Installation of Air-Conditioning Equipment for Antipolo Relay Station
	Equipment Installation Works (Supported by Japan: About 129 Million Yen)	Installation of Equipment for Improving EFCOS Project • Additional Equipment for Rainfall and Water Level Data Reception and Warning Device Operation • IP Radio Usable Equipment • Equipment for Receiving/Transmission of Data in each Monitor Base

Source: Study Team based on references

(4) Current Challenges of the EFCOS Project

After the EFCOS Project, KOICA installed additional rain and water level observatories in the Pasig-Marikina River Basin. The Pasig-Marikina River Basin has thus been congested with rain and water gauging and monitoring systems with the system developed by DOST in collaboration with the University of the Philippines (Project NOAH), as well as the early warning systems of PAGASA supported by KOICA.

According to the "Advisory Services for Flood Management on Project for Improvement/Restoration of Telemetry Equipment of Effective Flood Control Operation System (EFCOS): Services Completion Report [hereinafter "EFCOS Advisor Report (2016)"] conducted in February 2016 and the field and hearing survey conducted by a staff of this project, the following are the challenges facing the current EFCOS project:

- It has been confirmed that EFCOS data is being transmitted with the monitoring screen in PAGASA, and EFCOS also intends to incorporate EFCOS data into the large-screen monitor installed by the KOICA project. Currently, however, the integration is not working.
- It is still unclear whether the EFCOS Project or PAGASA will implement the Pasig-Marikina River Flood Forecasting and Warning System.
- Although there is congestion between donor supported systems as described above, it appears that the information on the PAGASA website, which displays the flood warning level and real-time water level in three stages, led by PAGASA, is now being used primarily for breaking news on TV and for public awareness. Judging from this, the EFCOS project is likely to be under the forecasting and warning system of PAGASA with support from KOICA.
- Issues in operation, maintenance and management:
  - EFCOS office is responsible only for the collection and arrangement of transmitted observation data, transmission of raw data to relevant organizations, and gate operation of Rosario Weir, excluding flood forecasting and warning operations.
  - Therefore, the flood information transmitted from the EFCOS office to the Flood Control Information Center (FCIC), which is also an organization inside the MMDA, is only hourly

rainfall and hourly river water level. Since there is no flood warning, there is still insufficient information transmission from the FCIC to the residents.

3.2.2.4 Drainage Project

(1) Pasig River Basin

Along the Pasig River, there are 12 large pumping stations managed by the MMDA as listed in **Table 3.2.17**. Many of the pumping stations constructed with Japanese ODA loan between 1970 and 1980 are being gradually replaced with larger capacity pumps[4].

Table 3.2.17 Basic Information on Existing Drainage Pump Stations along the Pasig River

Bank	Name	Location	Drainage	Size and Type of Pump (*1)	Number and Capacity of Pump (m^3/s)	Completion Year
Right (North)	Binondo	Junction with Estero de Binondo	Binondo, La Reina	4x185kW SP	4x3.63=14.52	1985/07
	Escolta	Muelle del Banco Nacional, Sta. Cruz, Manila	-	3x650mm SP	3x0.5=1.5	1983
	Quiapo	Elizondo St. Quiapo, Manila	Quiapo, San Miguel	4x185kW SP	4x3.625=14.52	1976/07
	Uli-uli	Junction with Estero de Uli-uli	Uli-uli	N/A	2x3=6.0	2012
	Aviles	Junction with Estero de Sampaloc	Aviles, Sampaloc	4x287kW SP	4x4.53=18,12	1976/07
	Valencia	Junction with Estero de Valencia	Valencia	4x231kW SP	4x3.5=14.00	1976/07
Left (South)	Balete	Romualdez St., Ermita, Manila	-	3x90kW SP; 1x1000mmSP	3x1.0+1x(0.8+1.0)=4.8	1989/10
	Paco	Junction with Estero de Paco	Paco, Concordia, Pandacan	3x1000mm VAF	3x2.53=7.59	1977/07
	Pandacan	Jesus St., Pandacan, Manila	Pandacan	2x160kW VAF	2x2.75=5.5	1976/06
	San Andres	Dr. M. L. Carreon St., San Andres, Manila	Tripa de Gallina	4x1500mm HAF	4x4.75=19.0	1998/02
	Sta. Clara	Junction with Estero de Sta. Clara	Sta. Clara	2x1000mm VAF	2x2.65=5.3	1977/-7
	Makati	Corner Zobel Street and Osmeña St.	-	2x1200mm VAF	2x3.5=7.0	1982/07

*1: VAF: Vertical Axial Flow Pump; HAF: Horizontal Axial Flow Pump; SP: Submergible Pump
Source: Study Team based on 2002DD (added some information from MMDA Website)

All large pumping stations are equipped with gates that can drain water to the Pasig River when the water level is low. In addition to the 12 pumping stations mentioned above, there are floodgates at the confluence of three drainage channels. The outline of these specifications is given in **Table 3.2.18**.

Except the steel through gate type of the Pandacan Gate, all other gates are the steel roller gate type.

Table 3.2.18 Basic information of Existing Floodgate along Pasig River

Name of Gate		Name of Pumping Station	Peak Discharge (m^3/s)	Bottom Elevation (EL+ m)	Top of Revetment (EL+ m)	Width (m)	Gate Dimension B x H x Nos.
Ancillary Floodgates	Binondo	Binondo	30.50	8.35	13.20	26.0	6.0 x 4.65 x 1
	Escolta	-		7.72	13.32	17.0	4.0 x 5.4 x 2
	Quiapo	Quiapo	33.50	7.40	13.60	21.0	4.0 x 6.3 x 2
	Uli-uli	Uli-uli	N/A	N/A	N/A	N/A	N/A
	Aviles	Aviles	47.90	8.00	14.20	17.0	4.0 x 6.0 x 2

[4] Rehabilitation / Upgrading of Twelve (12) Pumping Stations in Metro Manila
http://www.mmda.gov.ph/images/Home/flood-Control/rehabilitation-of-12-PS-in-MM.pdf

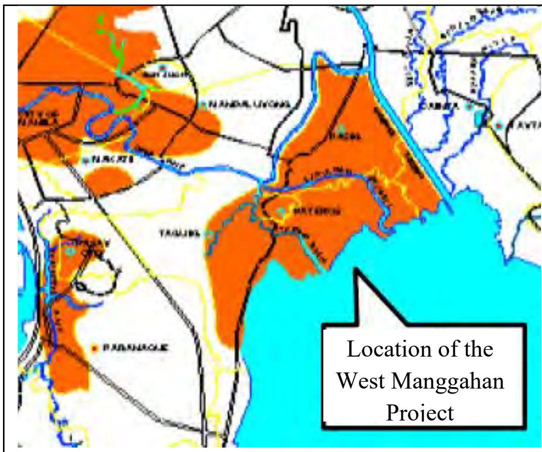
Name of Gate		Name of Pumping Station	Peak Discharge (m^3/s)	Bottom Elevation (EL+ m)	Top of Revetment (EL+ m)	Width (m)	Gate Dimension B x H x Nos.
	-Sampaloc						
	Valencia	Valencia	38.30	8.00	14.40	10.0	4.0 x 6.2 x 2
	Balete	Balete	8.40	8.70	13.80	14.0	4.0 x 4.9 x 2
	Paco	Paco	23.60	7.20	13.90	22.0	14.0 x 6.5 x 1
	Pandacan	Pandacan	22.10	8.75	14.40	12.0	5.0 x 5.45 x 1
	San Andres	San Andres	N/A	N/A	N/A	N/A	N/A
	Sta. Clara	Sta. Clara	20.10	9.85	15.20	7.0	5.0 x 5.17 x 1
	Makati	Makati	24.10	10.10	15.40	-	5.0 x 5.1 x 1
Independent	Beata	-	8.10	9.85	14.80	7.0	4.0 x 4.75 x 1
	Santebanez	-	-	8.30	14.00	31.0	10.0 x 5.5 x 1

Source: Study Team based on 2002DD

In addition to the above, it has been confirmed that there are six (6) small-scale pumping stations on the Pasig River. It has been said that there are 57 pumping stations managed by MMDA in Metro Manila and 36 of them will be renovated with loans from the WB and the AIIB. Details are given in **Item (5) of Subsection 3.2.2.4**.

(2) Metro Manila Flood Control Project - West of Manggahan (West Manggahan Project)

This project has been proposed based on the JICA1990MP/FS described in detail in **Subsection 3.2.1**. It is intended to protect the low-lying areas around the Laguna Lake from flooding and to reduce inland flood damage in the target area (See **Figure 3.2.11**). The Project is as briefly described below.



Source: JICA

Figure 3.2.11 Location of West Manggahan Project

1) Objectives of the Project

The objectives of this project were to prevent floods in the West Manggahan Area of Metro Manila at the northern side of Laguna Lake and to improve the living environment of residents with the construction of ring dike, pumping stations and bridges.

2) Project Contents

The project has been implemented in four contract packages as shown in **Table 3.2.19**.

Table 3.2.19 Outline of West Manggahan Project

Item	Contents	Remarks
Accepted Amount / Implemented Amount	9,411 Million Yen / 8,958 Million Yen	
Loan Agreement	March, 1997	Completed in August, 2007
Construction Package 1	Laguna Ring Dike	Length: 10.8km Crest Level: 15.0m (Partly EL+14.0m)
	Bridge	Napindan Channel Bridge
	Flood Control Reservoir [Storage Volume (Design)]	Tapayan [119,000m^3 (141,000m^3)] Labasan [80,000m^3 (80,000m^3)] Taguig [101,000m^3 (99,000m^3)] Hagonoy [58,000 m^3 (58,000 m^3)]
Construction Package 2	Napindan Channel Embankment (Sand Bank: EL+14.6m) (Concrete Parapet Wall: EL+14.1m)	Sand Bank 0.12km (Right Bank) 0.1km (Left Bank) Parapet Wall: 5.16km
	Sluice	4
Construction Package 3	Tapayan Pumping Station Labasan Pumping Station	Submerged Pump 3m^3/sx3=9m^3/s Submerged Pump 3m^3/sx3=9m^3/s
	Sluice	2
	Wharf	1 (Additional Scope)
Construction Package 4	Taguig Pumping Station Hagonoy Pumping Station	Submerged Pump 3m^3/s x 4=12m^3/s Submerged Pump 3m^3/s x 2=6m^3/s
	Sluice	2
	Additional Pumping Station	San Agustin Pumping Station

Source: DPWH

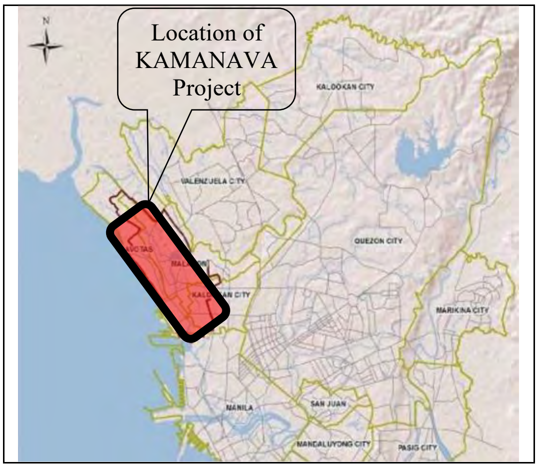
(3) KAMANAVA Area Flood Control and Drainage System Improvement Project (KAMANAVA Project)

This project was also proposed based on the JICA1990MP/FS described in detail in **Subsection 3.2.1**. Like the West Manggahan Project that was completed in August 2007, it is a project to reduce inland flood damage in the target area (See **Figure 3.2.12**). The outline of this project is given below.

1) Objectives of the Project

The Project covered the areas along Malabon and Tullahan rivers in the cities of Malabon, Navotas and Caloocan in Metro Manila. Together with Valenzuela, the area is called the KAMANAVA region. Since this area is low-lying at 0 to 1.5 meters above sea level and is particularly vulnerable to flood damage, it was necessary to implement flood countermeasures immediately.

The objectives of the project were to improve the living and sanitary environment in the region and to develop the local economy by reducing flood damage through the repair of dikes, improvement and construction of floodgates, improvement and construction of navigation gates, and procurement of hydro-meteorological observation equipment.



Source: JICA

Figure 3.2.12 Location Map, KAMANAVA Project

2) Project Contents

The contents of KAMANAVA Project are given in **Table 3.2.20.**

Table 3.2.20 Outline of KAMANAVA Project

Items	Contents	Remarks
Accepted Amount / Implemented Amount	8,929 Million Yen / 8,786 Million Yen	
LA	April 2000	Completed in January 2012
Contents of the Project	Polder Dike	Total Length: 8.6km
	River wall heightening	Malabon River: 10.5km (Malabon River: 6.6km, Marala River: 3.9km)
	Independent Floodgates	6 Gates
	Control Gates	None
	Pumping stations with ancillary flood gates	4 Stations
	Improvement of Existing Drainage Channel	5.6 km (funded by GOP)
	Construction of New Drainage Channel	2.1km

Source: DPWH

(4) Study on Flood Mitigation Project in the East Manggahan Floodway Area (East Manggahan Study)

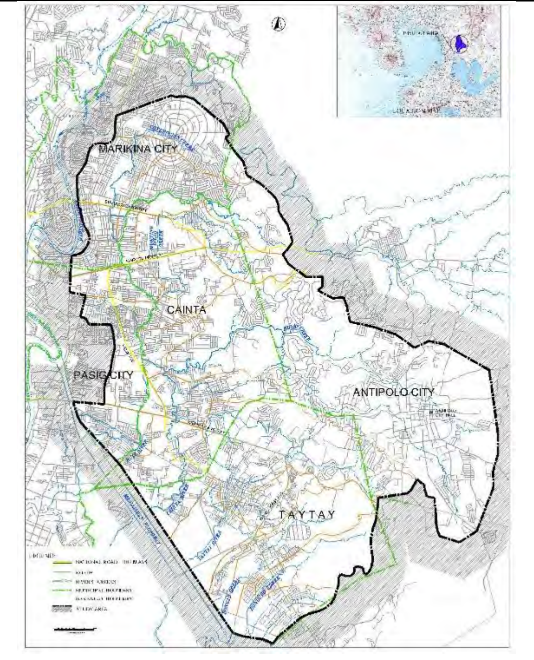
This study was conducted by the DPWH, aiming at the reduction of frequent inland flood damage in the East Manggahan district (area at the left bank side of the Manggahan Floodway). The main causes of inland flood damage are the water rise of Laguna Lake and the backflow from the Manggahan Floodway to the tributary rivers. In this study, the construction of Cainta and Taytay floodgates, which are included in the Phase IV project, was proposed as a priority project. The outline of the study is given below.

1) Objectives of the Project

The objectives of the project were to formulate a flood protection plan, including long-term countermeasures, and to propose emergency and priority projects in the target area (See **Figure 3.2.13**).

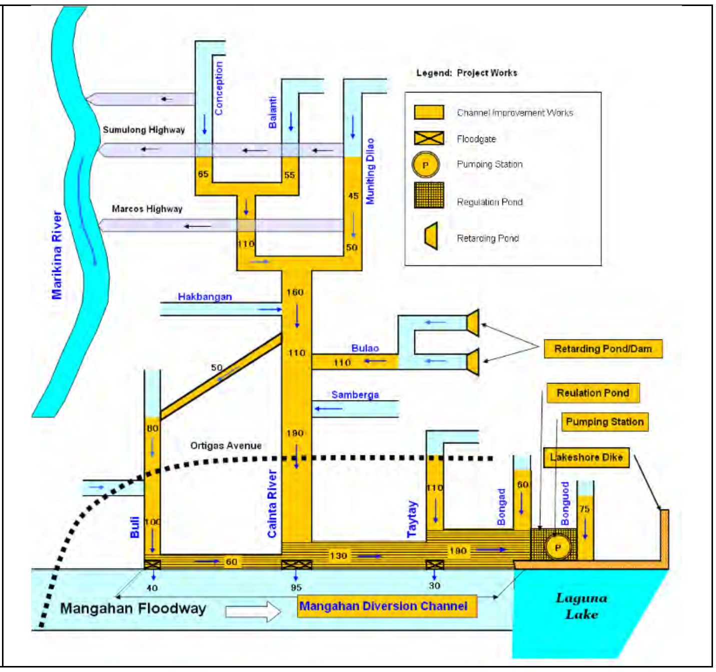
2) Project Contents

A flood protection plan was formulated, and the stage 1 projects shown in **Table 3.2.21** were proposed as projects that should be implemented immediately. The Buli Floodgate proposed in this project has already been constructed by Pasig City, and the Cainta and Taytay floodgates will be constructed in the Phase IV project.



Source : DPWH

Figure 3.2.13 Project Location, East Manggahan



Source : DPWH

Figure 3.2.14 Proposed Project and Design Flood Discharge Allocation

Table 3.2.21 Results of East Manggahan Study (Implementation Plan)

Stages	1	2	3
Components	(1) Lakeshore Dike (2) Taytay, Cainta and Buli Floodgates (Buli has been constructed by Pasig City) (3) Taytay Pumping Station	(1) Channel Improvement and Bridge Replaced in the downstream (2) Manggahan Diversion Channel	(1) Channel Improvement and Bridge Replacement in the Upstream (2) Retarding Basin/Dam

Source: DPWH

Table 3.2.22 Specifications of Proposed Floodgates, East Manggahan

No.	Name	Gate Size (m) (No. of Units x H x W)	Design Condition
1	Taytay	3 x 2.0 x 2.5	Connected channel width
2	Cainta	4 x 7.3 x 6.0	Connected channel width
3	Buli	2 x 6.0 x 6.0	Connected channel width

Source: DPWH

(5) New Drainage Project by WB (MM Flood Risk)

Based on the comprehensive flood risk management plan including the drainage proposed in the WB2012MP as described in **Subsection 3.2.1.6** above, the World Bank (WB) begun to support projects to improve the drainage and environment in Metro Manila through co-financing with the AIIB. The project implementation agencies are the DPWH and the MMDA. The project consists of the following four components.

1) Component 1 – Modernization of Drainage Areas (US$375.2 million)

Component 1 is the component to improve the functions of 36 large and small drainage pump stations in Metro Manila and surrounding areas, and to construct new drainage pump stations in 20 districts where there are no drainage pump stations at present. DPWH will be the project implementation agency, and maintenance will be transferred to the MMDA after completion.

2) Component 2 – Minimization of Solid Waste in Waterways (US$48 million)

It is said that the drainage capacity of drainage channels in Metro Manila and its surrounding areas has been reduced due to solid wastes. Therefore, Component 2 is the component to improve the collection and treatment of solid wastes. Residents' educational campaign activities such as not throwing garbage into channels are also carried out. The MMDA is the project implementation agency.

3) Component 3 - Participatory Housing and Resettlement (US$55.75 million)

This component involves the relocation of ISFs with the renovation and construction of drainage pump stations and drainage channels. At present, ISFs are estimated to be about 2,500, but the exact number of relocated ISFs will be confirmed in the project. The DPWH is the project implementation agency.

4) Component 4 - Project Management and Coordination (US$20.0 million)

This component will manage the whole project and consists of management and consulting services provided in coordination with Component 1 through Component 3. The project implementation agency will be both the DPWH and the MMDA.

5) Progress of the Project

There is information that the submission and evaluation of Expressions of Interest (EOI) for the Consulting Services for the DPWH Part (Components 1 and 3) have been completed. The DPWH and the World Bank are presently discussing the results of the evaluation.

6) Drainage Issues and Drainage Volume under the Pasig-Marikina River Flood Management Plan

According to the operation rules of all drainage pump stations built along the Pasig River and draining into the Pasig River, operation is decided only by the water level of landside water. However, if this operation is carried out, the operation will continue even if Pasig River reaches the Design Flood Level (DFL) and the possibility of overtopping of dikes becomes very high. In fact, during Typhoon Ondoy in 2009, pumps continued to run after the Pasig River reached DFL, increasing the risk of overflow outside the dike.

In the flood management plan of this study, if the Pasig-Marikina River reaches DFL, an operation to stop drainage from the pumping station along the river shall be proposed and the discharge of drainage shall not be included in the design flood discharge.

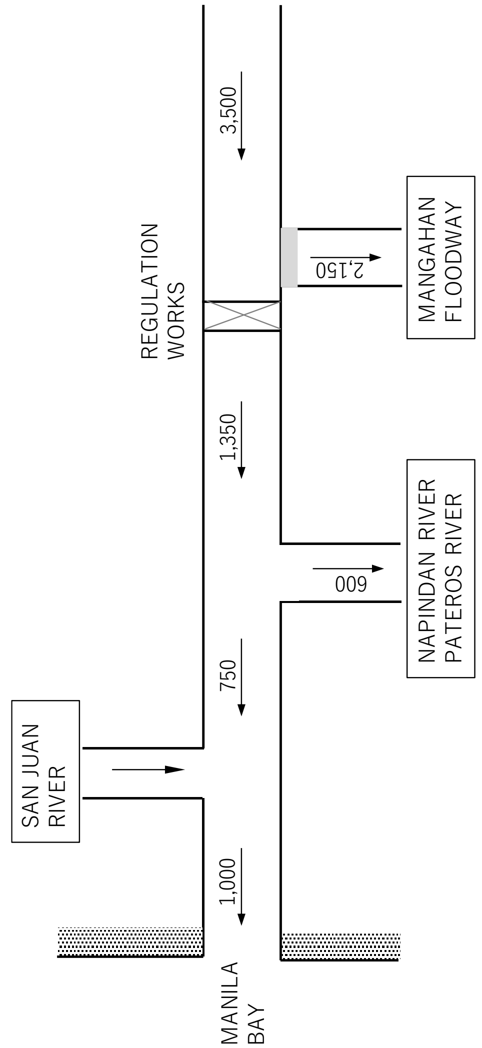
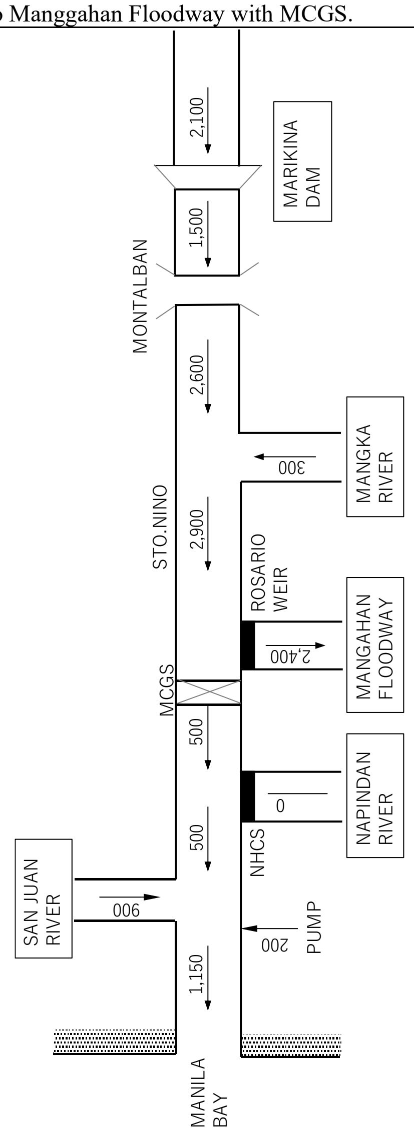
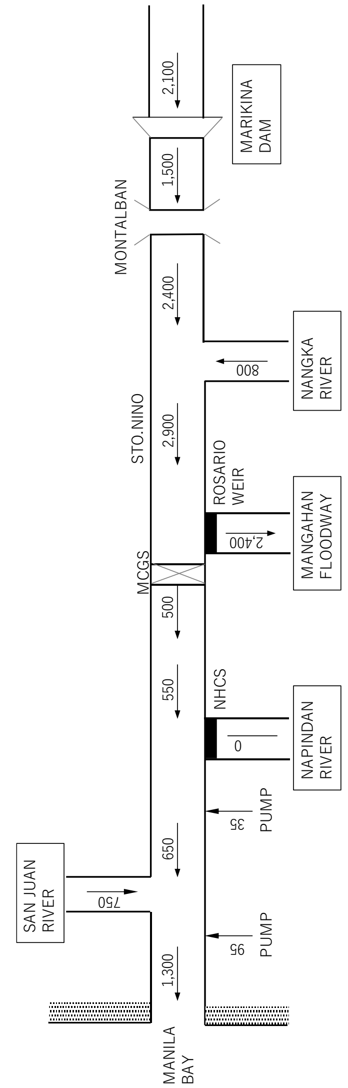
3.3 Comparison of Past Study's Contents

As for the above-mentioned past study contents related to the flood management plan, the following items are as compiled in **Table 3.3.1**, **Table 3.3.2** and **Table 3.3.3**. **Table 3.3.4** and **Table 3.3.5** give a comparison of detailed study contents.

- Background and Objectives of the Project
- Basic Concept of Design Flood Discharge Allocation Setting and its Changes
- Reason for Setting Immediate Target Flood Discharge
- Basic Concept of River Improvement (Proposed River Structures)
- Response Policy after Typhoon Ondoy Flood (2009)

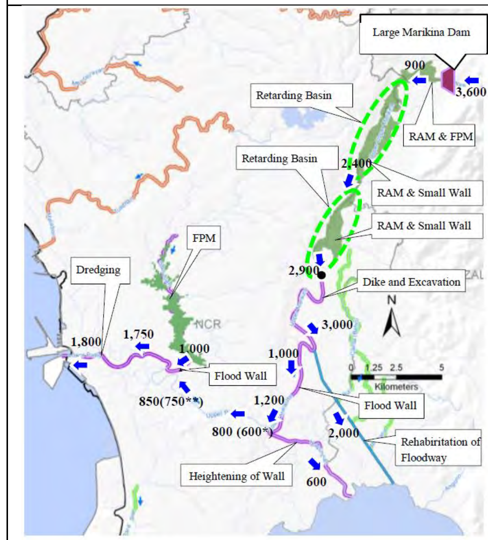
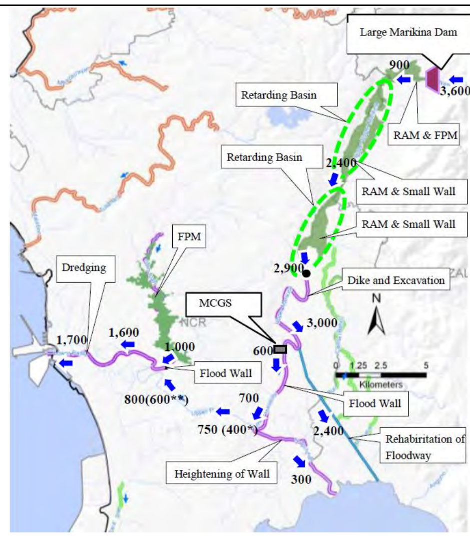
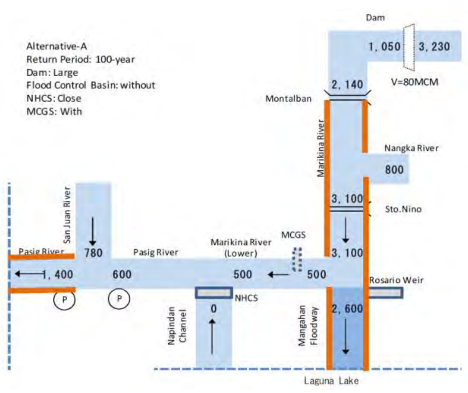
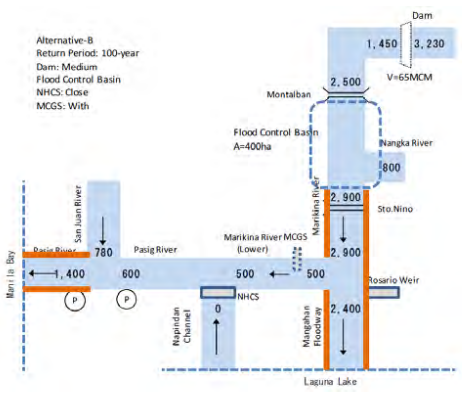
In addition, the comparison of 100-year design flood discharge allocation and specifications of Marikina Dam are shown in **Table 3.3.6** and **Table 3.3.7,** respectively.

Table 3.3.1 Comparison of Past Flood Management Studies (1)

Item		1952MP	1975FS/DD	JICA1990MP	2002DD
Background and Objectives of the Project		• The studies were started in 1943, shortly after the unprecedented flood of November of that year, which inundated the city for several days. • Main objective is to establish master plans for drainage measures in northern Manila and southern Manila. • Flood countermeasures for the Pasig Marikina River was also studied and proposed.	• The study was carried out with the massive flood in 1970. • FS and DD of the Manggahan Floodway proposed in the MP in 1952 and FS of the Parañaque Spillway which was planned for the drainage of Laguna Lake were carried out.	• President Aquino visited Japan in November 1986, and technical cooperation was formally requested for "The Study on Flood Control and Drainage Project in Metro Manila". In response to this request, JICA had launched a study to improve flood protection and drainage in Metro Manila. • Study on FP, MP and FS for propriety areas was conducted.	• In order to cope with frequent floods, DPWH decided to implement PMRCIP based on JICA1990MP/FS. • 2002DD was positioned as Phase I of the PMRCIP, and the detailed design of overall plan, review of design flood discharge of JICA1990MP, and the setting of immediate target discharge for the present development were carried out.
Design Flood Discharge Allocation	Basic Concept of Setting	• Targeted on massive flood in 1943 (without dam)	• Maximum Design Flood Discharge of Manggahan Floodway: 2,400m^3/s • Maximum Design Flood Discharge of Pasig River: 900m^3/s	• Design flood discharge (100-year) at Sto. Niño is 2,900m^3/s with Marikina Dam. • Divert 500m^3/s to downstream of Marikina River and 2,400 m3/s to Manggahan Floodway with MCGS.	• Design flood discharge (100-year) at Sto. Niño is 2,900m^3/s with Marikina Dam. • Divert 500m^3/s to downstream of Marikina River and 2,400m^3/s to Manggahan Floodway with MCGS.
	Changes		Not Analyzed		
Reason for Setting Immediate Target Flood Discharge		Not Analyzed	Not Analyzed	Not Analyzed	• For the primary river improvement, immediate target flood discharge (30-year probability, without dam) was set in this study.
Basic Concept of River Improvement (Proposed River Structures)		[River Improvement Measures] • Pasig River, Marikina River, San Juan River • Manggahan Floodway • Regulation Works): It is equivalent to current MCGS	[River Improvement Measures] • MCGS • Rosario Weir • Manggahan Floodway	[River Improvement Measures (MP)] • Pasig River, Marikina River, San Juan River, Napindan River • Marikina Dam • MCGS • Laguna Ring Dike • Non-structural Measures: Pasig-Marikina River: Effective Flood Control Operating System	[River Improvement Measures] • Pasig River, Marikina River, San Juan River • Marikina Dam • MCGS

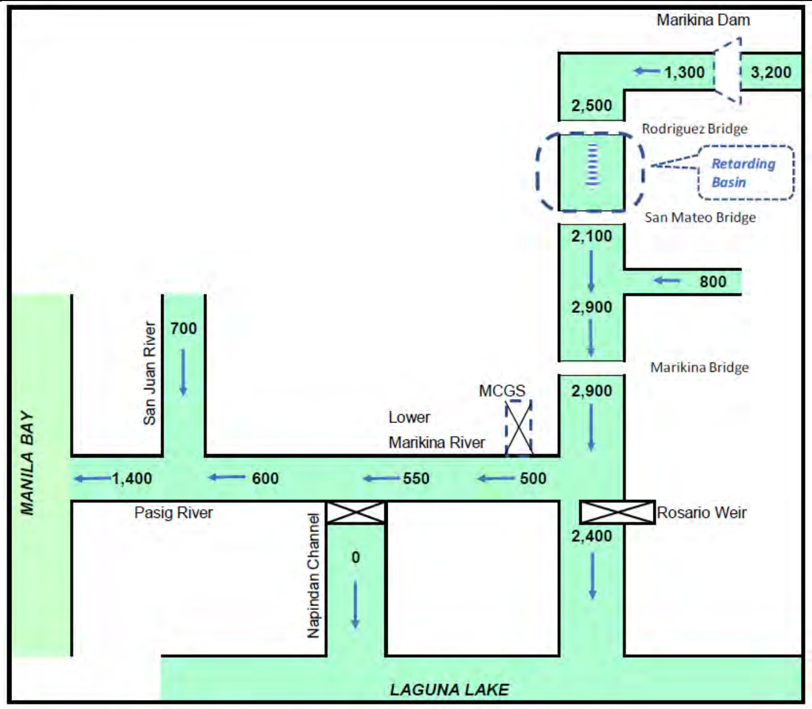
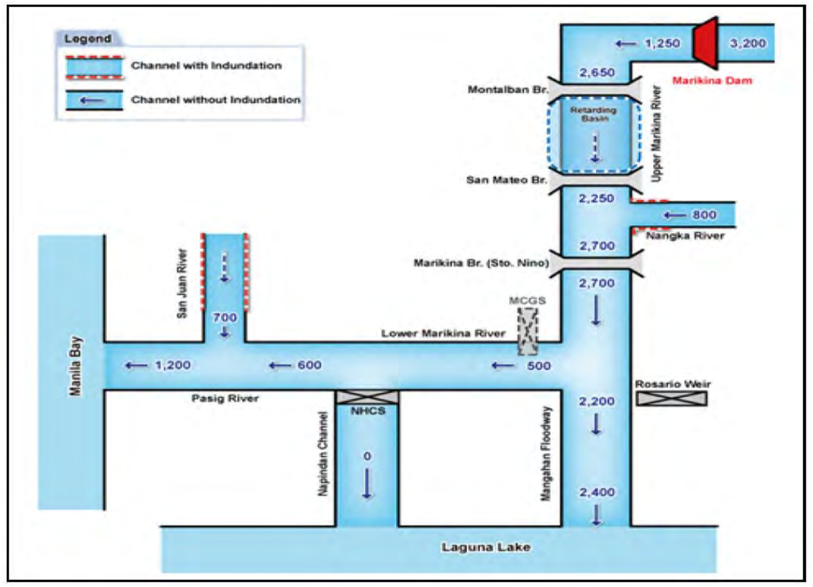
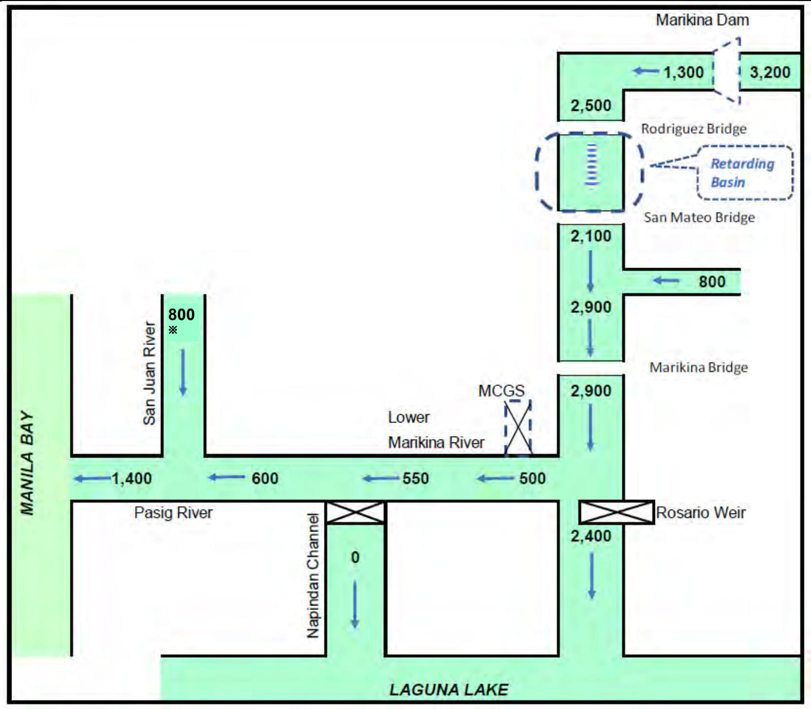
Source: Study Team

Table 3.3.2 Comparison of Past Flood Management Studies (1)

<table>
<tr><th colspan="2">Item</th><th>JICA2011 Preparatory Study</th><th colspan="3">WB2012MP</th><th colspan="3">JICA2014Study</th></tr>
<tr><td colspan="2">Background and Objectives of the Project</td><td>• In September 2009, Typhoon Ondoy caused massive flood damage in Metro Manila. The early completion of the entire PMRCIP was therefore an urgent task to prevent further flood damage.
• To review the existing PMRCIP, focusing on the river improvement section covered by Phase III including the following items: present river conditions reflecting recent river basin development, recent flood damage conditions, and impacts to flood damage by future climate change.</td><td colspan="3">• Following the massive flood damage in Metro Manila caused by Typhoon Ondoy in September 2009, this study was conducted to establish the overall vision and road map for a sustainable and effective flood risk management (FRM) in Metro Manila and Surrounding Areas.
• One of the specific objectives of the project is to prepare a comprehensive flood risk management plan.</td><td colspan="3">• The study was conducted to prepare basic information that would contribute to the development of a more detailed flood control plan by reviewing the previous survey results (specially design flood discharge in WB2012MP) taking into account climate change in the target areas.
• The objective is to reexamine the technical validity of the proposed structural measures in Pasig-Marikina River Basin under the WB Study.</td></tr>
<tr><td rowspan="3">Design Flood Discharge Allocation</td><td>Basic Concept of Setting</td><td>• Design flood discharge (100-year) at Sto. Niño is 2,900m^3/s with Marikina Dam.
• Divert 500m^3/s to downstream of Marikina River and 2,400 m^3/s to Manggahan Floodway with MCGS.</td><td colspan="3">• Design flood discharge (100-year) at Sto. Niño is 2,900m^3/s with Marikina Dam and natural retarding basin.
• Design flood discharge (100-year) were set under two (2) alternatives with/without MCGS.</td><td colspan="3">Discharge allocation was set under the flowing two (2) cases:
• Design flood discharge (100-year) at Sto. Niño is 2,900m^3/s with Marikina Dam and retarding basin.
• Design flood discharge (100-year) at Sto. Niño is 3,100m^3/s with Marikina Dam without retarding basin.
• In both cases, divert 500 m^3/s to downstream of Marikina River with MCGS.</td></tr>
<tr><td rowspan="2">Changes</td><td rowspan="2">Not Analyzed</td><td colspan="2">Alternative-2</td><td>Alternative-3</td><td colspan="2">A-2-1</td><td>B-3</td></tr>
<tr><td colspan="2"></td><td></td><td colspan="2"></td><td></td></tr>
<tr><td colspan="2">Reason for Setting Immediate Target Flood Discharge</td><td>• Although the review of the immediate target flood discharge (30-year design flood) was also considered, it was decided to follow the discharge allocation set in 2002DD.</td><td colspan="3">Not Analyzed</td><td colspan="3">• 30-year design flood discharge at this time was estimated including Typhoon Ondoy.</td></tr>
<tr><td colspan="2" rowspan="5">Basic Concept of River Improvement (Proposed River Structures)</td><td rowspan="5">Not Analyzed</td><td>Measures</td><td>Alternative-2</td><td>Alternative-3</td><td>Measures</td><td>A-2-1</td><td>B-3</td></tr>
<tr><td>River Improvement</td><td>Pasig River, Marikina River, San Juan River, Napindan Channel, Manggahan Floodway
*Dredging of Pasig River, additional works in Phase II, III and IV sections, and embank heightening of Napindan Channel are required.</td><td>Pasig River, Marikina River, San Juan River, Napindan Channel, Manggahan Floodway
*Dredging of Pasig River, additional works in Phase II and III sections, and embank heightening of Napindan Channel are required.</td><td>River Improvement Works</td><td>Pasig River, Marikina River, San Juan River, Manggahan Floodway
*Partial heightening of floodwall in Phase II and IV sections and additional dredging of the Manggahan Floodway are required.</td><td>Pasig River, Marikina River, San Juan River, Manggahan Floodway
*Partial heightening of floodwall in Phase II is required. There is no additional works in Phase III and IV sections.</td></tr>
<tr><td>MCGS</td><td>Yes</td><td>Yes</td><td>MCGS</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Marikina Dam</td><td>Location: 500m Upstream of the existing Wawa Dam
Dam Height: 72m
Storage Volume: 67.4MCM</td><td>Location: 500m Upstream of the existing Wawa Dam
Dam Height: 72m
Storage Volume: 67.4MCM</td><td>Marikina Dam</td><td>Location: 500m Upstream of the existing Wawa Dam
Dam Height: 68m
Storage Volume: 80MCM</td><td>Location: 500m Upstream of the existing Wawa Dam
Dam Height: 71m
Storage Volume: 65MCM</td></tr>
<tr><td>Natural Retarding Basin</td><td>980ha</td><td>980ha</td><td>Retarding Basin</td><td>No</td><td>371ha (8 sites)</td></tr>
<tr><td colspan="2">Response Policy after Typhoon Ondoy Flood (2009)</td><td>• The early completion of the entire PMRCIP was an urgent task to prevent further flood damage. In line with this, preparatory study was conducted by JICA with a view to extending ODA loans to Phase III.</td><td colspan="3">• The study was conducted to establish the overall vision and road map for a sustainable and effective flood risk management (FRM) in Metro Manila and Surrounding Areas.</td><td colspan="3">-</td></tr>
</table>

Source: Study Team

Table 3.3.3 Comparison of Past Flood Management Studies (1)

Item		2015IV & V	WB2018 UMD FS	Final Proposal in DED for PMRCIP IV
Background and Objectives of the Project		• Since Typhoon Ondoy caused severe flood damage in 2009, it has been recognized by DPWH that Phase IV needs to be implemented as soon as possible. • The objectives of the study are to conduct FS for Phase IV section and FS/DD for Phase V section.	• This study is to conduct FS and DD of Marikina Dam that is necessary for the completion of the whole PMRCIP • The basic objective of the project is to determine the preferred option for a flood management structure to reduce the water discharge from the Marikina River before it enters Metro Manila through a feasibility study before preparation of detailed designs and tender documents.	• Detailed engineering design project for PMRCIP IV (Detailed engineering design, preparation of bidding documents) • Final confirmation of river basin flood control plan to implement detailed engineering design
Design Flood Discharge Allocation	Basic Concept of Setting	• Design flood discharge (100-year) at Sto. Niño is 2,900m^3/s with Marikina Dam and retarding basin. • Divert 500m^3/s to downstream of Marikina River and 2,400 m^3/s to Manggahan Floodway with MCGS.	• Peak discharge of 100-year design flood at Sto. Niño will be cut by Marikina Dam and retarding basin. • Divert 500m^3/s to downstream of Marikina River with MCGS.	• Based on the DGCS of DPWH revised in 2015, it will be 100-year probable flood (rainfall). • Basically follow 2015 IV&V, and it will be reconfirmed/calculated within the survey. • If the idea of 2015 IV&V plan is dangerous side on the flood control plan from the result of reconfirmation within the survey, the final plan will be proposed considering the proposal of WB2012MP and JICA2014study after Typhoon Ondoy.
	Changes			
Reason for Setting Immediate Target Flood Discharge		Not Analyzed	Not Analyzed	• The design flood discharge distribution for 100-year probable flood was reconfirmed in the project, and the results of the 2015IV&V were proposed for Pasig-Marikina River. • However, as for design flood discharge of San Juan River, basic design discharge exceeded 1,000 m^3/s in the recalculation in the project. • Therefore, design flood discharge of 800 m^3/s proposed in the JICA2014Study which is bigger than 2015IV&V is proposed in the project on the premise of conducting a study to reduce flood discharge in San Juan River basin in future.
Basic Concept of River Improvement (Proposed River Structures)		[River Improvement Measures] • Pasig River, Marikina River, San Juan River, Manggahan Floodway *Partial heightening of floodwall in Phase II is required. There are no additional works in Phase III and IV sections. • Marikina Dam • MCGS • Retarding Basin (337ha)	Not Analyzed	[River Improvement Measures] • River Improvement: Pasig River, Marikina River, San Juan River * It was confirmed that the existing DFL could be maintained by dredging of Pasig River after the confluence of the San Juan River where the section already rehabilitated by Phase II and III. • Maintenance: Manggahan Floodway • MCGS • Marikina Dam (Proposal by WB2018 UMD FS) • Retarding Basin (Proposal by WB2018 UMD FS)
Response Policy after Typhoon Ondoy Flood (2009)		-	-	Confirmed to make planning using the rainfall reviewed after Typhoon Ondoy.

Source: Study Team

Table 3.3.4 Comparison of the Content of Past Studies (1)

Item		JICA1990MP	2002DD	JICA2011 Preparatory Study	WB2012MP
Design Hyetograph	Objective	Important item for estimation of peak discharge and scale of storage facility (capacity)			
	Applied Hyetograph	Middle-peak Fictional Hyetograph • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area	Middle-peak Fictional Hyetograph • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area	Middle-peak Fictional Hyetograph • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area	Type 1: Typhoon Ondoy Type • Observed Hyetograph Type 2: Middle-peak Fictional Hyetograph • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area Typhoon Ondoy Type is adopted.
Estimation of Basin Average Rainfall	Objective	Important Item to estimate probable rainfall, peak discharge and scale of storage facility (capacity) consequently			
	Estimation Method of Basin Average Rainfall	Rainfall at Port Area x Rainfall Adjustment Coefficient • Estimated as uniform rainfall in whole area	Rainfall at Port Area x Rainfall Adjustment Coefficient • Estimated as uniform rainfall in whole area	Rainfall at Port Area x Rainfall Adjustment Coefficient • Estimated as uniform rainfall in whole area	Type 1: Typhoon Ondoy Type • Thiessen Method and Adjustment by IDW Method • Estimated each 34 Thiessen Polygon Type 2: Middle-peak Fictional Hyetograph • IDW Method • Estimated for 3 Sub-basins
Design Rainfall Duration	Objective	Item which affect peak discharge and scale of storage facility (capacity), depending on basin characteristics such as basin area, slope, land-use and so on.			
	Applied Design Rainfall Duration	2 days rainfall is applied to cover observed rainfall duration of past floods.	2 days rainfall is applied to cover observed rainfall duration of past floods.	2 days rainfall is applied to cover observed rainfall duration of past floods.	2 days rainfall is applied to cover observed rainfall duration of past floods.
Basin Average Probable Rainfall	Objective	Important to estimate runoff discharge for each probable year.			
	Applied Basin Average Probable Rainfall	Whole Basin (2-day) • 30-year: 540mm • 100-year: 660mm	Whole Basin (2-day) • 30-year: 244.5mm • 100-year: 300.7mm	Whole Basin (2-day) • 30-year: 392.3mm • 100-year: 445.8mm	Whole Basin (2-day) • 30-year: 367mm • 100-year: 439mm
Flood Discharge at Sto.Niño	Objective	Discharge at Sto. Niño is important to calibrate model constants which is the key of accuracy of runoff model. Since it is difficult of continuous observation of discharge directory, H-Q equation to convert from water level to discharge is applied.			
	Applied Estimation of Flood Discharge	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ H < 17.0 • $Q = 17.49 \times (H-8.61)^2$ H > 17.0 H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986.	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ H < 17.0 • $Q = 17.49 \times (H-8.61)^2$ H > 17.0 H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986.	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ H < 17.0 • $Q = 17.49 \times (H-8.61)^2$ H > 17.0 H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986. Max. discharge of Typhoon Ondoy (2009) • 3,211m^3/sec	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 31.44 \times (H-10.96)^2$ H > 13.0 H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986, and estimated discharge by uniform flow and observed water level in 1994-2009. Max. discharge of Typhoon Ondoy (2009) • 3,950m^3/sec
Runoff Analysis	Objective	It is the analysis to convert rainfall to discharge, and accuracy of analysis effect peak discharge and scale of storage facility.			
	Runoff Analysis Method	Rainfall-Runoff Model • Storage Function Method: Mountainous Area • Quasi-linier Storage Type: General value was applied for constants of concentration time.	Rainfall-Runoff Model • Storage Function Method: Mountainous Area • Quasi-linier Storage Type: General value was applied for constants of concentration time.	Rainfall-Runoff Model • Storage Function Method: Mountainous Area • Quasi-linier Storage Type: Urbanized Area Calibration and Verification of Model Parameters • 2 floods in 2004 was reproduced. • Model parameters were calibrated to conform calculated hydrograph to observed discharge. • Parameters for Storage Function Method (delay factors) were determined based on previous model.	Integrated Analysis model of basin, river and flood plain. • Basin: Rainfall-Runoff Model (SCS Unit Hydrograph Method) • River: One-dimensional Unsteady Flow Model • Flood Plain: Two-dimensional Unsteady Flow Model Calibration and Verification of Model Parameters • Flood by Ondoy Typhoon was reproduced. • Model parameters were calibrated to conform calculated hydrograph to observed peak discharge and water level. • Model was verified by reproducing 2004 flood and 1998 flood.
Water Level in Laguna Lake	Objective	Water level in Laguna Lake effect discharge of Manggahan Floodway and water level of Marikina River upstream of Manggahan Floodway.			
	Applied Water Level in Laguna Lake	Average Annual Maximum Water Level: 12.5m	12.0/12.5/13.0/13.5, etc., were applied for Non-uniform Flow calculation in Manggahan Floodway. (Average Annual Maximum Water Level: 12.34m) (Record Highest Water Level: 14.03m (1972))	Average Water Level during Flood: 12.2m	Observed Water Level during Ondoy Typhoon: 12.78-13.85m
Inundation Analysis	Objective	It is important for evaluation of planned flood control facilities.			
	Applied Inundation Analysis	(None)	Inundation Analysis Method • River: One-dimensional Non-uniform Flow Model • Flood Plain: Leveled Inundation Method	Inundation Analysis Model • River: One-dimensional Unsteady Flow Model • Flood Plain: Two-dimensional Unsteady Flow Model • Flood by Typhoon Ondoy was reproduced. • Simulation results were well conformed with interview survey results.	Integrated Analysis model of basin, river and flood plain. • Flood by Typhoon Ondoy was reproduced. • Simulation results were well conformed with inundation map based on flood damage survey.

Item		JICA1990MP	2002DD	JICA2011 Preparatory Study	WB2012MP
Basic Design Discharge	Without Inundation	Probable Discharge at Sto. Niño • 30-year: 2,900 m^3/sec • 100-year: 3,500 m^3/sec	Probable Discharge at Sto. Niño • 30-year: 2,900 m^3/sec • 100-year: 3,430 m^3/sec	Probable Discharge at Sto. Niño • 30-year: 2,740 m^3/sec • 100-year: 3,210 m^3/sec	(Not analyzed)
	With Inundation	(Not analyzed)	(Not analyzed)	(Not analyzed)	Probable Discharge at Sto. Niño 30-year: 3,600 m^3/sec 100-year: 4,100 m^3/sec Large scale inundation at left side between confluence of Nangka River and Rosario Weir by dyke break
Design Flood Discharge Allocation	30-year	Estimated as a reference Case 1: without Marikina Dam and MCGS Case 2: with Marikina Dam and MCGS (see table JICA1990MP 30-year)	Countermeasures against 30-year probable flood: • River Improvement • MCGS (see table 2002DD 30-year)	Previous discharge allocation is applied. Countermeasures against 30-year probable flood: • River Improvement • MCGS (see table JICA2011 30-year)	(None)
	100-year	Countermeasures against 100-year probable flood: • River Improvement • MCGS • Marikina Dam (see table JICA1990MP 100-year)	Design Flood Discharge in JICA1990MP is revised. Countermeasures against 100-year probable flood: • River Improvement • MCGS • Marikina Dam (see table 2002DD 100-year)	Estimated as a reference with the following measures; • River Improvement • MCGS (see table JICA2011 100-year)	Alternative 2 and 3 including the following measures is recommended. Alt-2: River Improvement, Marikina Dam, Retarding Basin and Non-structural Measures Alt-3: River Improvement, MCGS, Marikina Dam, Retarding Basin and Non-structural Measures (see table WB2012MP 100-year)

JICA1990MP 30-year — 30-year Discharge Unit: Q(m^3/s)

Section	Case 1	Case 2
Wawa	1,700	1,700
Montalban Bridge	2,250	1,800
Before Nangka River	2,550	2,200
Nangka River		
Marikina Bridge (Sto. Niño)	2,900	2,500
Manggahan Floodway	1,850	2,200
Lower Marikina River		
Napindan Channel	0	0
Pasig River	1,000	100
San Juan River	850	850
Pasig River - Manila Bay	1,200	800

2002DD 30-year — 30-year Design Discharge (with MCGS)

Section	Q(m^3/s)
Wawa	(1,740)
Montalban Bridge	(2,230)
Before Nangka River	(2,520)
Nangka River	(690)
Marikina Bridge (Sto. Niño)	2,900
Manggahan Floodway	2,400
Lower Marikina River	500
Napindan Channel	0
Pasig River	600
San Juan River	700
Pasig River - Manila Bay	1,200

JICA2011 30-year — 30-year Design Discharge (with MCGS) Unit:Q(m^3/s)

Section	This Study	Final
Wawa	1,590	
Montalban Bridge	2,110	
Before Nangka River	2,420	
Nangka River	640	
Marikina Bridge (Sto. Niño)	2,740	2,900
Manggahan Floodway	2,230	2,400
Lower Marikina River	500	500
Napindan Channel	0	0
Pasig River	575	600
San Juan River	690	700
Pasig River - Manila Bay	1,160	1,200

JICA1990MP 100-year — 100-year Design Discharge (with MCGS)

Section	Q(m^3/s)
Wawa	2,100
Marikina Dam	1,500
Montalban Bridge	2,600
(Retarding Basin)	
Nangka River	300
Marikina Bridge (Sto. Niño)	2,900
Manggahan Floodway	2,400
Lower Marikina River	500
Napindan Channel	0
Pasig River	500
San Juan River	900
Pasig River - Manila Bay	1,150

2002DD 100-year — 100-year Design Discharge (with MCGS)

Section	Q(m^3/s)
Wawa	2,100
Marikina Dam	1,500
Montalban Bridge	2,400
(Retarding Basin)	
Nangka River	800
Marikina Bridge (Sto. Niño)	2,900
Manggahan Floodway	2,400
Lower Marikina River	500
Napindan Channel	0
Pasig River	650
San Juan River	750
Pasig River - Manila Bay	1,300

JICA2011 100-year — 100-year Design Discharge (with MCGS)

Section	Q(m^3/s)
Wawa	1,890
Marikina Dam (without)	
Montalban Bridge	2,500
(Retarding Basin)	
Nangka River	730
Marikina Bridge (Sto. Niño)	3,210
Manggahan Floodway	2,720
Lower Marikina River	500
Napindan Channel	0
Pasig River	585
San Juan River	770
Pasig River - Manila Bay	1,310

WB2012MP 100-year — 100-year Design Discharge Unit: Q(m^3/s)

Section	Alt-2	Alt-3
Wawa	3,600	3,600
Marikina Dam	900	900
Montalban Bridge	2,400	2,400
(Retarding Basin)		
Nangka River		
Marikina Bridge (Sto. Niño)	2,900	2,900
Manggahan Floodway	2,000	2,400
Lower Marikina River	1,000	600
Napindan Channel	600	300
Pasig River	850	800
San Juan River	1,000	1,000
Pasig River - Manila Bay	1,800	1,700

Source: Study Team based on JICA2014 Study

Table 3.3.5 Comparison of the Content of Past Studies (2)

Item		JICA2014 Study	2015IV&V	2018WB UMD FS	Final Proposal in DED for PMRCIP IV
Design Hyetograph	Objective	Important item for estimation of peak discharge and scale of storage facility (capacity)			
	Applied Hyetograph	Observed 7 Hyetographs + Middle-peak Fictional Hyetograph • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area • Typhoon Ondoy Type is adopted.	Observed 10 Hyetographs • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area • Typhoon Ondoy Type is adopted.	• Typhoon Ondoy Type is adopted.	<Follow 2015IV&V and reconfirm> Observed 10 Hyetographs • Hyetograph based on probable rainfall intensities by rainfall durations of Port Area • Typhoon Ondoy Type is adopted.
Estimation of Basin Average Rainfall	Objective	Important Item to estimate probable rainfall, peak discharge and scale of storage facility (capacity) consequently			
	Estimation Method of Basin Average Rainfall	Adjusted by Thiessen Method and IDW Method based on observed rainfall	Adjusted by Thiessen Method and IDW Method based on observed rainfall	Adjusted by Thiessen Method based on observed rainfall	<Follow 2015IV&V and reconfirm> Adjusted by Thiessen Method and IDW Method based on observed rainfall
Design Rainfall Duration	Objective	Item which affect peak discharge and scale of storage facility (capacity), depending on basin characteristics such as basin area, slope, land-use and so on.			
	Applied Design Rainfall Duration	Relation between rainfall and water level is analyzed. Due to its high correlation, 1-day rainfall is applied.	Relation between rainfall and water level is analyzed. Due to its high correlation, 1-day rainfall is applied.	Relation between rainfall and water level is analyzed. Due to its high correlation, 1-day rainfall is applied.	<Follow 2015IV&V and reconfirm> Relation between rainfall and water level is analyzed. Due to its high correlation, 1-day rainfall is applied.
Basin Average Probable Rainfall	Objective	Important to estimate runoff discharge for each probable year.			
	Applied Basin Average Probable Rainfall	Whole Basin (1-day) • 30-year: 232.4mm 100-year: 285.5mm	Whole Basin (1-day) • 30-year: 255.5mm • 100-year: 309.0mm	Whole Basin (1-day) • 30-year: 299mm • 100-year: 359mm	<Follow 2015IV&V and reconfirm> Whole Basin (1-day) • 30-year: 255.5mm • 100-year: 309.0mm
Flood Discharge at Sto. Niño	Objective	Discharge at Sto. Niño is important to calibrate model constants which is the key of accuracy of runoff model. Since it is difficult of continuous observation of discharge directory, H-Q equation to convert from water level to discharge is applied.			
	Applied Estimation of Flood Discharge	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ $H < 14.0$ • $Q = 25.65 \times (H-10.46)^2$ $H > 14.0$ H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986, and estimated discharge by non-uniform flow and observed water level in 1994-2009. Max. discharge of Typhoon Ondoy (2009) • 3,480m^3/sec	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ $H < 14.0$ • $Q = 25.65 \times (H-10.46)^2$ $H > 14.0$ H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986, and estimated discharge by non-uniform flow and observed water level in 1994-2009. Max. discharge of Typhoon Ondoy (2009) • 3,480m^3/sec	Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ $H < 14.0$ • $Q = 25.65 \times (H-10.46)^2$ $H > 14.0$ H-Q equation was made based on observed discharge and water level data in 1958-77 and 1986, and estimated discharge by non-uniform flow and observed water level in 1994-2009. Max. discharge of Typhoon Ondoy (2009) • 3,480m^3/sec	<Follow 2015IV&V and reconfirm> Annual highest water level in 1958-77, 1986, and 1994-2009 were converted to discharge. H-Q Equation: • $Q = 32.03 \times (H-10.80)^2$ $H < 14.0$ • $Q = 25.65 \times (H-10.46)^2$ $H > 14.0$ Max. discharge of Typhoon Ondoy (2009) • 3,480m^3/sec
Runoff Analysis	Objective	It is the analysis to convert rainfall to discharge, and accuracy of analysis effect peak discharge and scale of storage facility.			
	Runoff Analysis Method	Integrated Analysis model of basin, river and flood plain. • Basin: Rainfall-Runoff Model (WEB-DHM Model) • River Course: One-dimensional Non-uniform Flow Model • Flood Plain: Two-dimensional Unsteady Flow Analysis Model Calibration and Verification of Model Parameters • Flood by Ondoy Typhoon was reproduced. • Model parameters were calibrated to conform calculated hydrograph to observed hydrograph as well as peak discharge and water level. • Model was verified by reproducing 2004 flood and 2012 flood.	Integrated Analysis model of basin, river and flood plain. • Basin: Rainfall-Runoff Model (NAM Model) • River Course: One-dimensional Non-uniform Flow Model • Flood Plain: Two-dimensional Unsteady Flow Analysis Model Calibration and Verification of Model Parameters • Flood by Ondoy Typhoon was reproduced. • Model parameters were calibrated to conform calculated hydrograph to observed hydrograph as well as peak discharge and water level. • Model was verified by reproducing 2004 flood and 2014 flood.	Integrated Analysis model of basin, river and flood plain. • Basin: Rainfall-Runoff Model (SCS Unit Hydrograph Method) • River: One-dimensional Unsteady Flow Model • Flood Plain: Two-dimensional Unsteady Flow Model Calibration and Verification of Model Parameters • Flood by Typhoon Ondoy was reproduced. • Model parameters were calibrated to conform calculated hydrograph to observed peak discharge and water level. • Model was verified by reproducing 2004 flood and 2012 flood.	<Follow 2015IV&V and reconfirm> Integrated Analysis model of basin, river and flood plain. • Basin: Rainfall-Runoff Model (NAM Model) • River Course: One-dimensional Non-uniform Flow Model • Flood Plain: Two-dimensional Unsteady Flow Analysis Model Calibration and Verification of Model Parameters • Flood by Ondoy Typhoon was reproduced. • Model parameters were calibrated to conform calculated hydrograph to observed hydrograph as well as peak discharge and water level. • Model was verified by reproducing 2004 flood and 2014 flood.
Water Level in Laguna Lake	Objective	Water level in Laguna Lake effect discharge of Manggahan Floodway and water level of Marikina River upstream of Manggahan Floodway.			
	Applied Water Level in Laguna Lake	Past Highest Water Level after Manggahan Floodway Constructed: 13.90m	Past Highest Water Level after Manggahan Floodway Constructed: 13.90m	Past Highest Water Level after Manggahan Floodway Construction: 13.90m	<Follow 2015IV&V and reconfirm> Past Highest Water Level after Manggahan Floodway Construction: 13.90m
Inundation Analysis	Objective	It is important for evaluation of planned flood control facilities.			
	Applied Inundation Analysis	Integrated Analysis model of basin, river and flood plain. • Flood by Typhoon Ondoy was reproduced. • Simulation results were well conformed with inundation map based on flood damage survey.	Integrated Analysis model of basin, river and flood plain. • Flood by Typhoon Ondoy was reproduced. • Simulation results were well conformed with inundation map based on flood damage survey.	Integrated Analysis model of basin, river and flood plain. • Flood by Typhoon Ondoy was reproduced. • Simulation results were well conformed with inundation map based on flood damage survey.	<Follow 2015IV&V and reconfirm> Integrated Analysis model of basin, river and flood plain. • Flood by Typhoon Ondoy was reproduced. • Simulation results were well conformed with inundation map based on flood damage survey.
	Without Inundation	Probable Discharge at Sto. Niño • 30-year: 3,990 m^3/sec	Probable Discharge at Sto. Niño (Not analyzed)	Probable Discharge at Sto. Niño (Not analyzed)	<Follow 2015IV&V and reconfirm>

Item		JICA2014 Study	2015IV&V	2018WB UMD FS	Final Proposal in DED for PMRCIP IV
Basic Design Discharge		• 100-year: 4,980 m^3/sec			Probable Discharge at Sto. Niño (Without inundation also confirmed) • 30-year: 4,300 m^3/sec • 100-year: 5,200 m^3/sec
	With Inundation	• 30-year: 3,030 m^3/sec • 100-year: 3,580 m^3/sec	• 30-year: 3,200 m^3/sec • 100-year: 3,600 m^3/sec	• 30-year: 2,950 m^3/sec • 100-year: 3,700 m^3/sec • With Retarding Basin	<Follow 2015IV&V and reconfirm> • 30-year: 3,200 m^3/sec • 100-year: 3,600 m^3/sec
Design Flood Discharge Allocation	30-year	The following counter measure are recommended. Countermeasures against 30-year probable flood: Alternative-A: River Improvement, MCGS, Improvement of Manggahan Floodway Alternative-B: River Improvement, MCGS, Construction of Retarding Basin	Estimated as a reference with the following conditions; • Current River Cross Section • Without inundation from downstream of San Mateo Bridge • With inundation from San Juan River	Estimated as a reference with the following conditions; • Based on 2015IV&V • With Retarding Basin	<Follow 2015IV&V and reconfirm> Estimated as a reference with the following conditions; • Current River Cross Section • Without inundation from downstream of San Mateo Bridge • With inundation from San Juan River
	100-year	Alternative A-2-1 and B-3 are shown in below. Countermeasures against 100-year probable flood: Alernative-A-2-1; • River Improvement • MCGS • Improvement of Manggahan Floodway • Marikina Dam Alernative-B-3; • River Improvement • MCGS • Marikina Dam • Retarding Basin	Countermeasures against 100-year probable flood: • River Improvement • MCGS • Marikina Dam • Retarding Basin	Countermeasures against 100-year probable flood: • River Improvement • MCGS • Marikina Dam • Retarding Basin	<Follow 2015IV&V and reconfirm> Countermeasures against 100-year probable flood: • River Improvement • MCGS • Marikina Dam • Retarding Basin

JICA2014 Study — 30-year Design Discharge (with MCGS) Unit: Q(m^3/s)

Section	A	B
Wawa	2,720	2,720
Montalban Bridge	3,560	3,560
(Retarding Basin)		
Nangka River		
Marikina Bridge (Sto. Niño)	3,100	2,900
Manggahan Floodway	2,600	2,400
Lower Marikina River	500	500
Napindan Channel	0	0
Pasig River	600	600
San Juan River	700	700
Pasig River - Manila Bay	1,300	1,300

2015IV&V — 30-year Design Discharge (without MCGS)

Section	Q(m^3/s)
Wawa	2,600
Montalban Bridge	3,600
(Retarding Basin)	
Nangka River	540
Marikina Bridge (Sto. Niño)	3,200
Manggahan Floodway	2,100
Lower Marikina River	1,100
Napindan Channel	500
Pasig River	550
San Juan River	550
Pasig River - Manila Bay	1,100

2018WB UMD FS — 30-year Design Discharge (without MCGS)

Section	Q(m^3/s)
Wawa	2,650
Montalban Bridge	3,850
(Retarding Basin)	
Nangka River	650
Marikina Bridge (Sto. Niño)	2,950
Manggahan Floodway	2,000
Lower Marikina River	1,050
Napindan Channel	0
Pasig River	1,100
San Juan River	650
Pasig River - Manila Bay	1,600

Final Proposal in DED for PMRCIP IV — 30-year Design Discharge (without MCGS)

Section	Q(m^3/s)
Wawa	2,600
Montalban Bridge	3,600
(Retarding Basin)	
Nangka River	540
Marikina Bridge (Sto. Niño)	3,200
Manggahan Floodway	2,100
Lower Marikina River	1,100
Napindan Channel	500
Pasig River	550
San Juan River	550
Pasig River - Manila Bay	1,100

JICA2014 Study — 100-year Design Discharge Unit: Q(m^3/s)

Section	A-2-1	B-3
Wawa	3,230	3,230
Marikina Dam	1,260	1,260
Montalban Bridge	2,140	2,500
(Retarding Basin)		
Nangka River	800	800
Marikina Bridge (Sto. Niño)	3,100	2,900
Manggahan Floodway	2,400	2,400
Lower Marikina River	500	500
Napindan Channel	0	0
Pasig River	600	600
San Juan River	780	780
Pasig River - Manila Bay	1,400	1,400

2015IV&V — 100-year Design Discharge (with MCGS)

Section	Q(m^3/s)
Wawa	3,200
Marikina Dam	1,300
Montalban Bridge	2,500
(Retarding Basin)	
Nangka River	800
Marikina Bridge (Sto. Niño)	2,900
Manggahan Floodway	2,400
Lower Marikina River	500
Napindan Channel	0
Pasig River	600
San Juan River	700
Pasig River - Manila Bay	1,400

2018WB UMD FS — 100-year Design Discharge (with MCGS)

Section	Q(m^3/s)
Wawa	3,200
Marikina Dam	1,250
Montalban Bridge	2,650
(Retarding Basin)	
Nangka River	800
Marikina Bridge (Sto. Niño)	2,700
Manggahan Floodway	2,200
Lower Marikina River	500
Napindan Channel	0
Pasig River	600
San Juan River	700
Pasig River - Manila Bay	1,200

Final Proposal in DED for PMRCIP IV — 100-year Design Discharge (with MCGS)

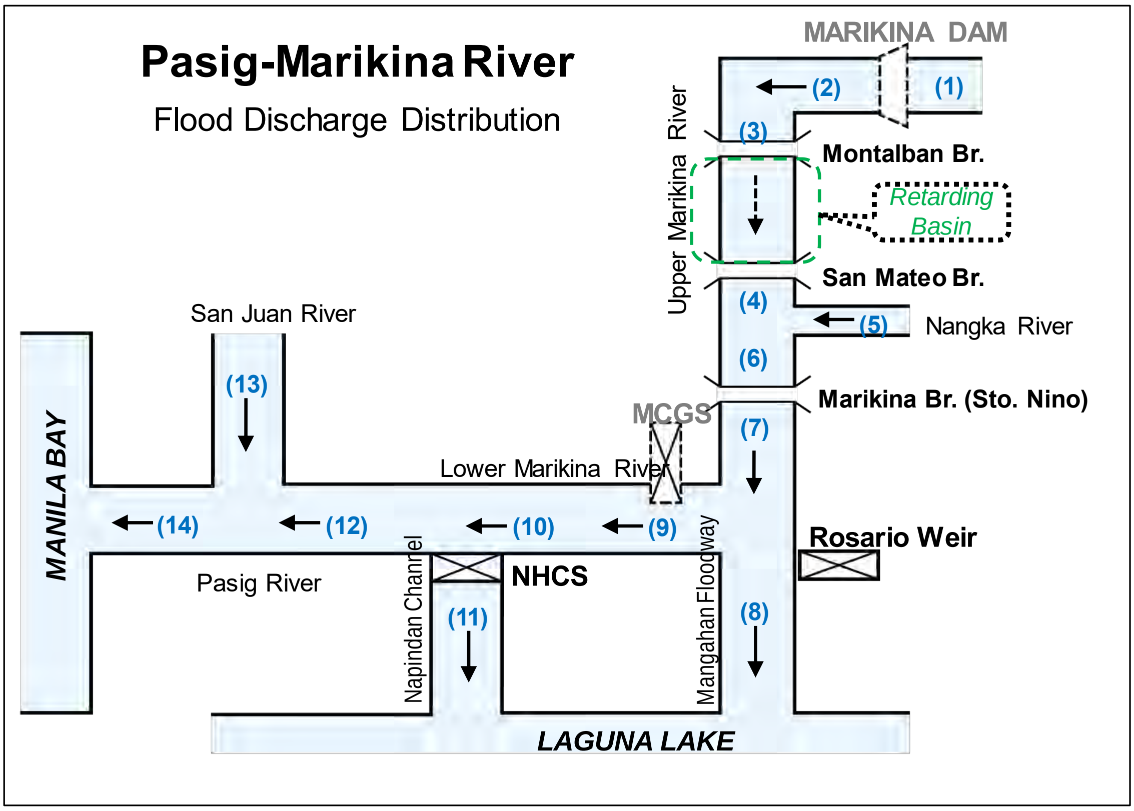
Section	Q(m^3/s)
Wawa	3,200
Marikina Dam	1,300
Montalban Bridge	2,500
(Retarding Basin)	
Nangka River	800
Marikina Bridge (Sto. Niño)	2,900
Manggahan Floodway	2,400
Lower Marikina River	500
Napindan Channel	0
Pasig River	600
San Juan River	800
Pasig River - Manila Bay	1,400

Source: Study Team based on JICA2014 Study

Table 3.3.6 Comparison of 100-Year Design Flood Discharge Allocations

Point No. and Name		Probable Discharge of 100-year Design Flood (m^3/s)					
		JICA 1990MP	2002DD	WB 2012MP	JICA 2014MP	2015 IV&$V	WB 2018FS
(1)	Wawa	2,100	2,100	3,600	3,230	3,200	3,200
(2)	Marikina Dam	1,500	1,500	900	1,450	1,300	1,250
(3)	Montalban Bridge	-	-	2,400	2,500	2,500	2,650
(4)	San Mateo Bridge	2,600	2,400	-	-	2,100	2,250
(5)	Nangka River	300	800	-	800	800	800
(6)	After Nangka River	2,900	2,900	2,900	2,900	2,900	2,700
(7)	Marikina Bridge	2,900	2,900	3,000	2,900	2,900	2,700
(8)	Manggahan Floodway	2,400	2,400	2,000	2,400	2,400	2,200
(9)	Lower Marikina I (MCGS)	500	500	1,000	500	500	500
(10)	Lower Marikina II	500	550	1,200	500	550	500
(11)	Napindan Channel	0	0	800	0	0	0
(12)	Before San Juan River	500	650	850	600	600	600
(13)	San Juan River	900	700	1,000	780	700	700
(14)	River Mouth	1,150	1,300	1,800	1,400	1,400	1,200

Source: Study Team



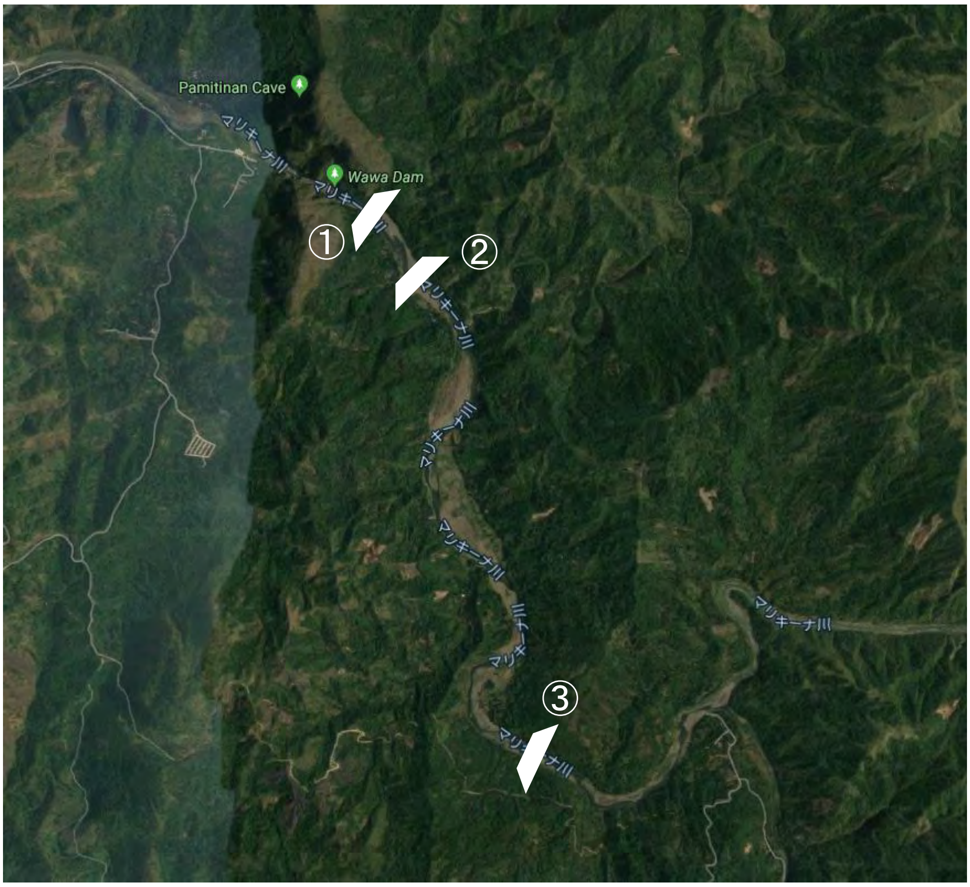
Source: Study Team

Figure 3.3.1 Design Flood Discharge Allocation

Table 3.3.7 Comparation of Specifications of Marikina Dam

Item	JICA 1990MP	2002DD	WB 2012MP	JICA 2014MP	2015 IV&V	WB 2018FS
Location	①100m Upstream of the existing Wawa Dam	①100m Upstream of the existing Wawa Dam	②500m Upstream of the existing Wawa Dam	②500m Upstream of the existing Wawa Dam	Not Analyzed	③3km Upstream of the existing Wawa Dam
Height (m)	70	70	72	71	64	72
Storage Volume (MCM)	Not Analyzed	Not Analyzed	67.4	65	64.2	63.5
Inflow at Design Flood Discharge (m^3/s)	2,100	2,100	3,600	3,230	3,200	3,200
Outflow at Design Flood Discharge (m^3/s)	1,500	1,500	900	1,450	1,300	1,250

Source: Study Team



Source: Study Team based on Google Map

Figure 3.3.2 Location of Proposed Marikina Dam

3.4 Finalization of Flood Management Plan

The flood management plan in this study basically follows those of the previous 2015IV&V-FS and the JICA2014Study.

3.4.1 Basin Average Probable Rainfall

The basin average probable rainfall in this study follows that of the 2015IV&V-FS as shown in **Table 3.4.1**.

This basin average probable rainfall was calculated using Typhoon Ondoy type hyetograph which recorded the largest basin average 1-hour rainfall, 1-day rainfall and peak discharge at Sto. Niño. For the design rainfall duration, the relation between rainfall and water level was analyzed and 1-day rainfall was applied due to its high correlation with water level compared to 2-day rainfall. The JICA2014Study and the 2018WB/UMD/FS also used the Typhoon Ondoy type and 1-day rainfall.

Table 3.4.1 Basin Average Probable Rainfall

Return Period	1-Day Rainfall (mm)
2	122.9
3	146.4
5	172.7
10	205.7
20	237.3
30	**255.5**
50	278.3
80	299.1
100	**309.0**
150	326.9
200	339.6
400	370.1

Source: 2015IV&V-FS

3.4.2 Flood Discharge at Sto. Niño

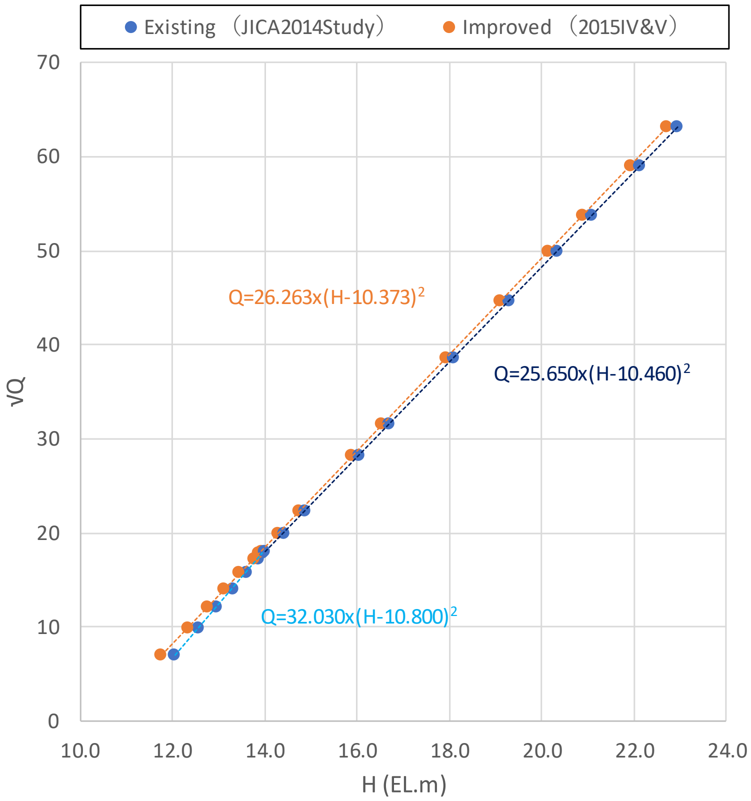
In this study, the H-Q equation in the existing river channel condition at Sto. Niño follows the H-Q equation made in the JICA2014Study as shown below. The equation was made based on observed discharges and water level data in 1958-77 and 1986, as well as the discharge estimated by non-uniform flow and observed water level in 1994-2009.

- $Q = 32.030 \times (H-10.800)^2 \quad H < 14.0$
- $Q = 25.650 \times (H-10.460)^2 \quad H > 14.0$

The H-Q equation in the improved river channel condition at Sto. Niño follows the following H-Q equation made under the 2015IV&V-FS:

- $Q = 26.263 \times (H-10.373)^2$

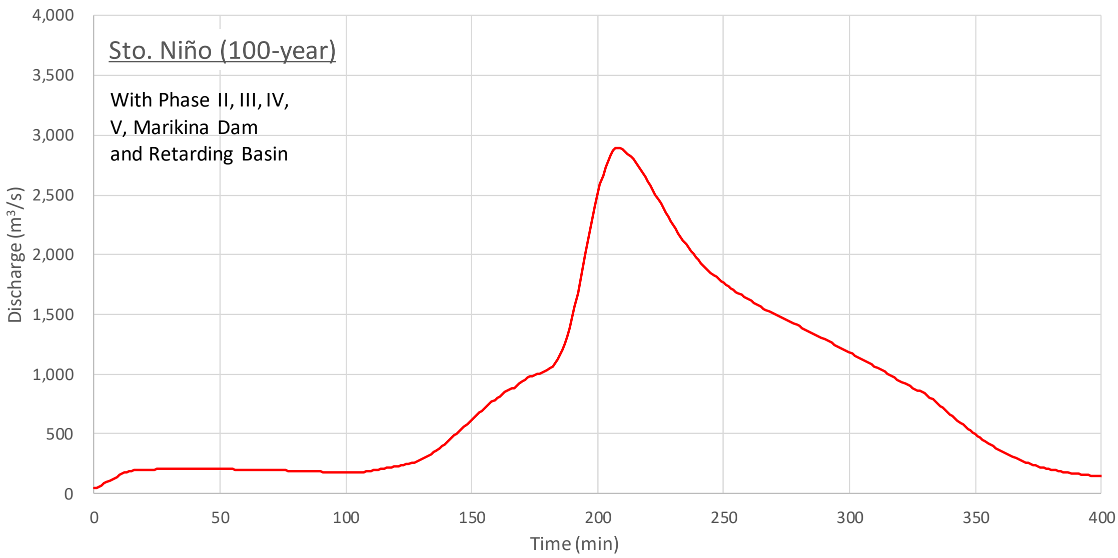
Figure 3.4.1 gives a comparison of H-Q equations at Sto. Niño in the existing and improved river channel conditions. The water level at Sto. Niño will decrease to about 20cm under the design flood discharge (2,900 m^3/s) due to the river channel improvement to be undertaken in the Phase-IV Project.



Source: Study Team based on JICA2014Study and 2015IV&V-FS

Figure 3.4.1 Comparison of H-Q Equation at Sto. Niño in the Existing and Improved Conditions

The design hydrograph at Sto. Niño (100-year return period, with Marikina Dam and retarding basin) is to be finalized under the Retarding Basin Study currently being conducted by the World Bank following the WB2018UMD. For this reason, the anticipated design hydrograph (See **Figure 3.4.2**) which has been estimated in this Study based on the 2018WB UMD FS report, is used to prepare the operation rules for flood control structures. This hydrograph was designed using Typhoon Ondoy type hyetograph, which has the biggest 1-hour rainfall, 1-day rainfall, and the largest peak discharge and rapid water level rise at Sto. Niño.

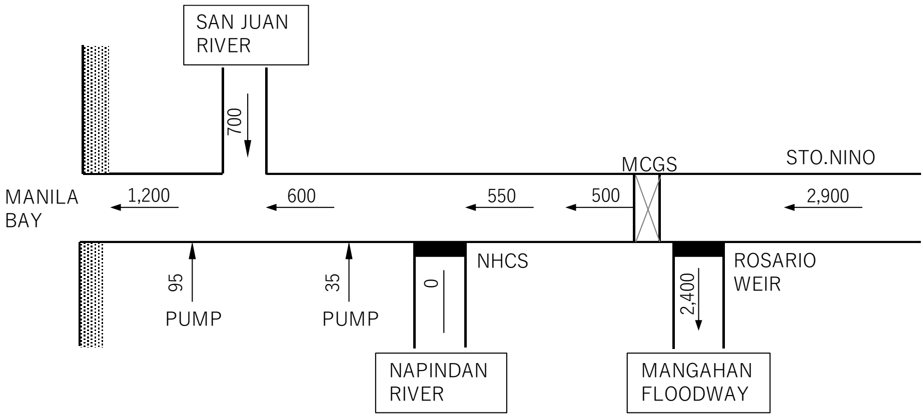


Source: Study Team based on 2018WB UMD FS

Figure 3.4.2 Anticipated Design Hydrograph at Sto. Niño (2,900 m^3/s)

3.4.3 Immediate Target Flood Discharge

The Phase II and Phase III sections have been improved based on the immediate target flood discharge (30-year probability) set in the 2002DD as shown in **Table 3.4.2**.



Source: 2002DD

Figure 3.4.3 Immediate Target Flood Discharge Allocation (30-Year Design Flood) (2002DD)

The design flood discharge of 2,900 m^3/s at Sto. Niño was a 30-year probability at the time of the 2002DD, but the probability scale was later revised under the WB2012MP and the JICA2014Study. According to the JICA2014Study, the 30-year flood discharge was about 3,100 m^3/s which is larger than the 2,900 m^3/s, as a result of the review on probable rainfall based on the recent floods including Typhoon Ondoy in 2009. **Table 3.4.2** shows the probable discharge at Sto. Niño with inundation upstream in the JICA2014Study. The estimated discharge probability of 2,900 m^3/s at Sto. Niño is slightly more than a 20-year.

This project follows the JICA2014Study, and the design flood discharge of 2,900 m^3/s at Sto. Niño is to be treated as approximately 20-year to 30-year in terms of flood probability scale. However, this probability scale will change whenever the data is extended and the design rainfall is revised. The immediate target discharge and design flood discharge at Sto. Niño are 2,900 m^3/s regardless of probable rainfall. Also, 2,900 m^3/s was originally assumed to be the design flood discharge (100-year) at Sto. Niño after the Marikina Dam construction, and 30-year flood probability was estimated using the rainfall data before Typhoon Ondoy occurred; that is, the 30-year flood probability of 2,900 m^3/s at Sto. Niño will be a tentative target discharge until the completion of Marikina Dam and the retarding basins.

It is desirable to express the future project objectives as the "immediate target discharge", and it is proposed to continue the implementation as a flood control project in response to the 100-year flood, including the construction plan of Marikina Dam and retarding basin to be constructed upstream.

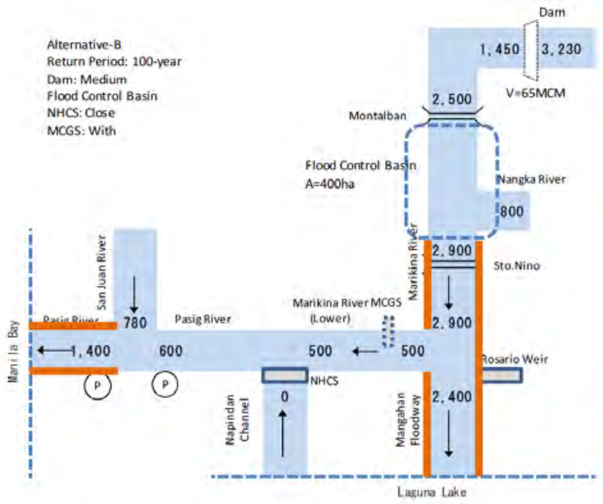
Table 3.4.2 Probable Discharge at Sto. Niño

Return Period [year]	Sto. Niño (With Inundation) [m^3/s]
2	1,620
5	2,290
10	2,670
20	2,860
30	3,030
50	3,220
100	3,580

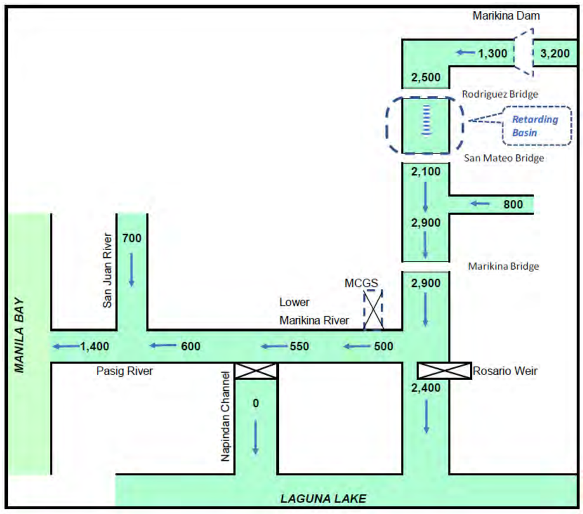
Source: JICA2014Study

3.4.4 Design Flood Discharge

The design flood discharge has been reviewed and revised many times. As the proposed design flood discharge allocation at present, there are the discharge allocations of the JICA2014Study and 2015IV&V-FS formulated by the DPWH, the implementing agency for this project (See **Figure 3.4.4**).



Source: JICA2014Study



Source: 2015IV&V-FS

Figure 3.4.4 Comparation of Design Flood Discharge Allocations (100-year Design Flood)

3.4.4.1 Upstream Section of Sto. Niño

It is necessary to finalize the design discharge based on the study results of Marikina Dam and retarding basins conducted by the WB. At present, it follows the design flood discharge allocation of the 2015IV&V-FS, which has been referred to in the World Bank study. The design flood discharge allocation at Sto. Niño was the same at 2,900 m^3/s in both the JICA2014Study and the 2015IV&V-FS.

3.4.4.2 Phase IV Section

The JICA2014Study and the 2015IV&V-FS employ the same design flood discharge of 2,900 m^3/s at Sto. Niño, and 500 m^3/s is diverted into the downstream of Marikina River and 2,400 m^3/s to the Manggahan Floodway by the MCGS. In addition, the World Bank Study Team for retarding basin has a common recognition of design flood discharge at Sto. Niño of 2,900 m^3/s. Therefore, this project will follow the discharge allocation in the JICA2014Study and the 2015IV&V-FS, and the Phase IV section will be developed at the design flood discharge of 2,900 m^3/s.

3.4.4.3 MCGS - Junction with San Juan River

There are differences in the discharge allocation between the downstream of MCGS and the near NHCS. As shown in **Figure 3.4.4**, in the projects of Phase II and III, the junction of San Juan River (upper ends of Pasig River section) was improved at the target discharge of 600 m^3/s, and the upper ends of Pasig River (MCGS section) was improved at the target discharge of 550 m^3/s. Therefore, the design flood discharge of this section follows the 2015IV&V-FS, namely, 600 m^3/s and 550 m^3/s respectively.

3.4.4.4 Downstream Ends of Pasig River

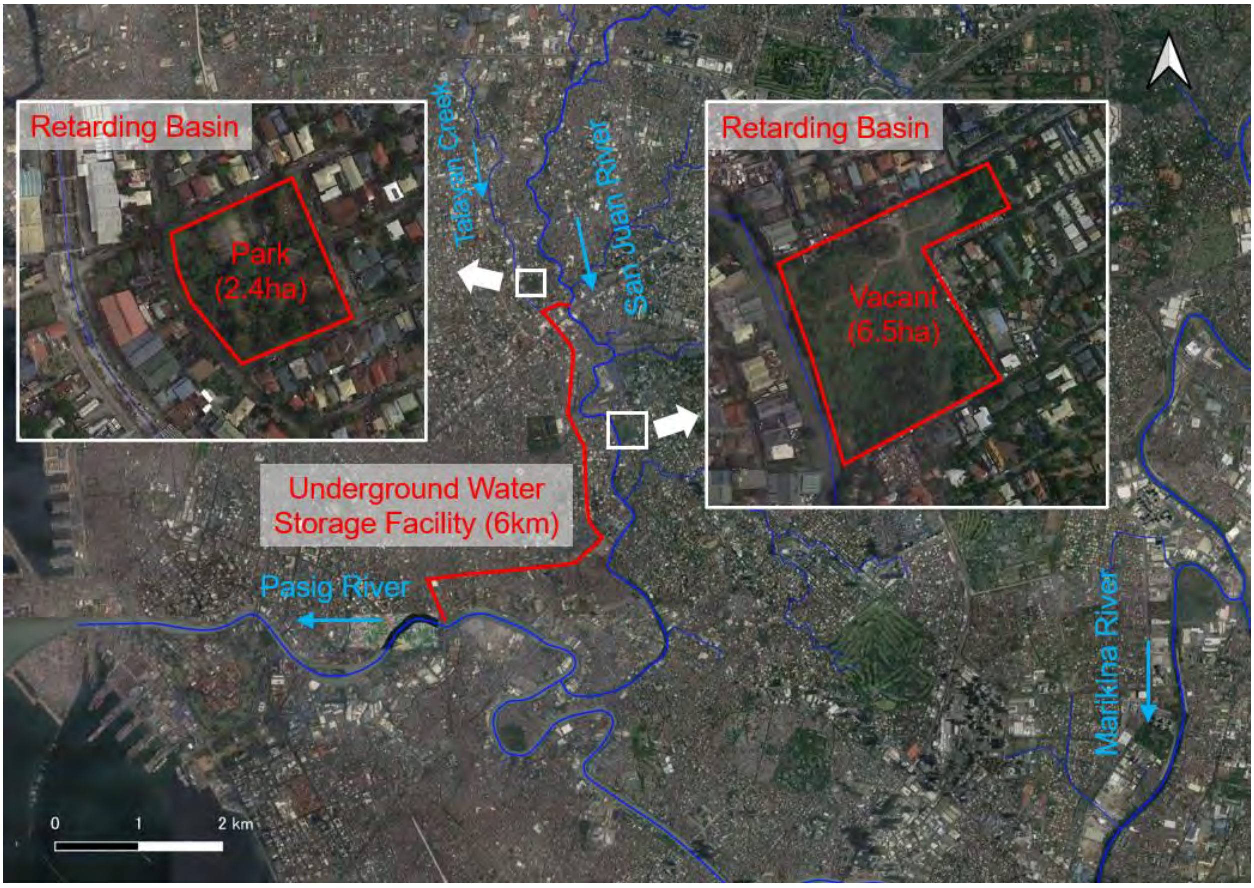
Both are 1,400 m^3/s. However, this section has already been improved with a target discharge of 1,200 m^3/s, and it is necessary to raise the height of the parapet wall or dredge the river channel in order to cope with the increase of 200 m^3/s when the design flood discharge is 1,400 m^3/s.

3.4.4.5 San Juan River

In the 2015IV&V-FS, the design flood discharge of 700 m^3/s follows the immediate target flood discharge, while in the JICA2014Study, it is 780 m^3/s based on the current DPWH plan. However, the basic flood discharge of San Juan River in 100-year design flood exceeds 1,000 m^3/s in both the WB2012MP and the recalculation made in this study.

As mentioned above, the downstream section of Pasig River has already been improved, and additional improvement works is required if discharge from the tributary increases. In order to minimize this as much as possible, the design flood discharge shall be 800 m^3/s, which is rounded up from the 780 m^3/s in the JICA2014Study, and the difference of about 200 m^3/s will be reduced by watershed management, etc. The following are possible measures to reduce discharge from the San Juan River.

- The vacant land (6.5ha) on the left bank of the San Juan River located in the Damayang Lagi district of Quezon City and the park (2.4ha) on the left bank of the tributary Talayan Creek would be used as retarding basins.
- Underground water storage facility (about 6km) can be proposed on the right bank of the San Juan River which connects the Araneta Road to the Magsaysay Road and connects to the Pasig River near the Mabini Bridge.

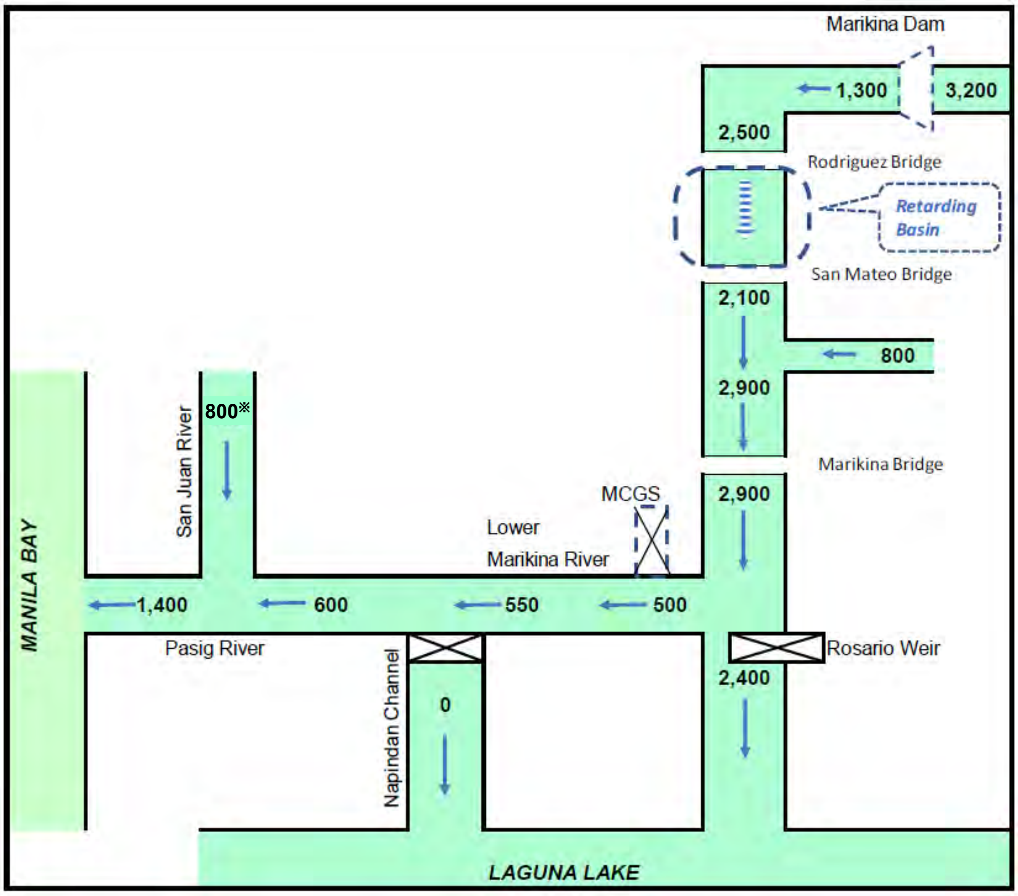


Source: Study Team

Figure 3.4.5 Possible Measures to reduce discharge from San Juan River

3.4.4.6 Draft Design Flood Discharge Allocation

The design flood discharge allocation of each section mentioned above is shown in **Figure 3.4.6** as the tentative design flood discharge allocation.



*Design flood discharge on the assumption that the peak discharge is cut by about 200 m^3/s by basin management and so on.
Source: Study Team based on 2015IV&V-FS

Figure 3.4.6 Draft Design Flood Discharge Allocation (100-Year Flood Discharge)

Without the Marikina Dam, about 20~30-year return period flood can be accommodated with the implementation of the Phase IV Project.

The study for Marikina Dam and retarding basins has already been started to aim the safety level of 100-year probability against the river floods. Additional structural measures to be taken before the construction of Marikina Dam will be meaningless after the completion of Marikina Dam. Therefore, Non-structural measures or the installation of temporary floodwalls are more preferable as tentative countermeasures for flood before construction of the Marikina Dam.

3.4.5 Climate Change Adaptation

The design guideline for water engineering projects in the Philippines (DGCS Volume III) suggests the following allowances for climate change:

- Changes to Extreme Rainfall: Incorporate a 10% increase in rainfall intensity in the design.
- Sea Level Rise: Allow for a 0.3 m sea level rise in the design.

For the Phase IV section, which is the design target section of this study, climate change adaptation has been addressed in the design through the following:

- Design Flood Discharge: 10% increase in rainfall intensity is incorporated in the computation of design flood discharge, which is 2,900 m^3/s at Sto. Niño in a 100-year return period. The design flood discharge is the river flow regulated by the Marikina Dam and the retarding basin. Practically, the increased amount of discharge caused by 10% increased design rainfall intensity shall be regulated by the Marikina Dam and the retarding basin.
- Sea Level Rise: Sea level rise does not affect the river water level in the design target section of Phase IV.

CHAPTER 4 PRECONDITIONS FOR RIVER CHANNEL DESIGN (BASIC DESIGN STAGE)

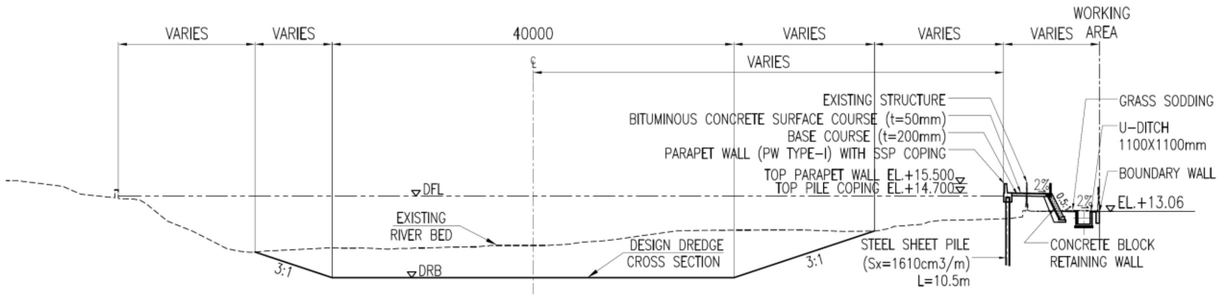
4.1 Preconditions (Verification of River Channel Planning)

4.1.1 Validation of Past Plans and Determination of Standard Cross Section of Targeted River Stretch

Past river channel plans for the stretch targeted in this detailed design study are as described below, including the concepts for the determination of standard cross sections of the PMRCIP Phase IV Project.

4.1.1.1 Planned Cross Section Downstream of MCGS

The design flood discharge in the downstream of the MCGS is set at 550 m^3/s, and river improvement works have been carried out up to Sta. 5+400 of the Marikina River in the PMRCIP Phase III Project. Excavation and/or dredging of the low water channel has been carried out to satisfy the planned riverbed width of 40 m [Slope: 3:1 (H:V)], while dikes (floodwalls) were constructed where the ground elevation behind the bank was lower than the DFL.



Source: JICA Phase III Detailed Design Report

Figure 4.1.1 Standard Cross Section of Phase III Downstream of the Marikina River Improvement Project

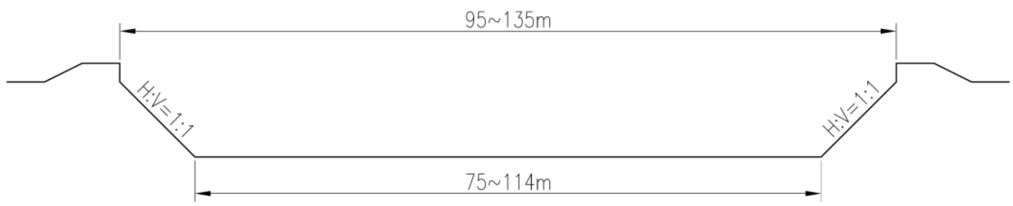
In the PMRCIP-IV Project, river cross sections downstream of the MCGS are set to coincide with the plan for the Phase III Project.

4.1.1.2 Standard Cross Section/s in the Upstream Stretch of the MCGS

The design flood discharge in the upstream stretch of the MCGS is set at 2,900 m^3/s, as presented in the foregoing **Subsection 3.2.1.8**. As for the basic design, the transition of the design standard cross section proposed in the past studies from the JICA1990MP to the DPWH2015IV&V-FS are as discussed below.

(1) Standard Cross Section proposed in JICA1990MP

In the JICA's Master Plan in 1990 (JICA1990MP), the following cross section is proposed.



Source: JICA 1990 MP Report

Figure 4.1.2 Cross Section of Phase IV of the Marikina River Improvement Project Proposed in JICA1990MP (Sta. 5+425/Sta. 13+060)

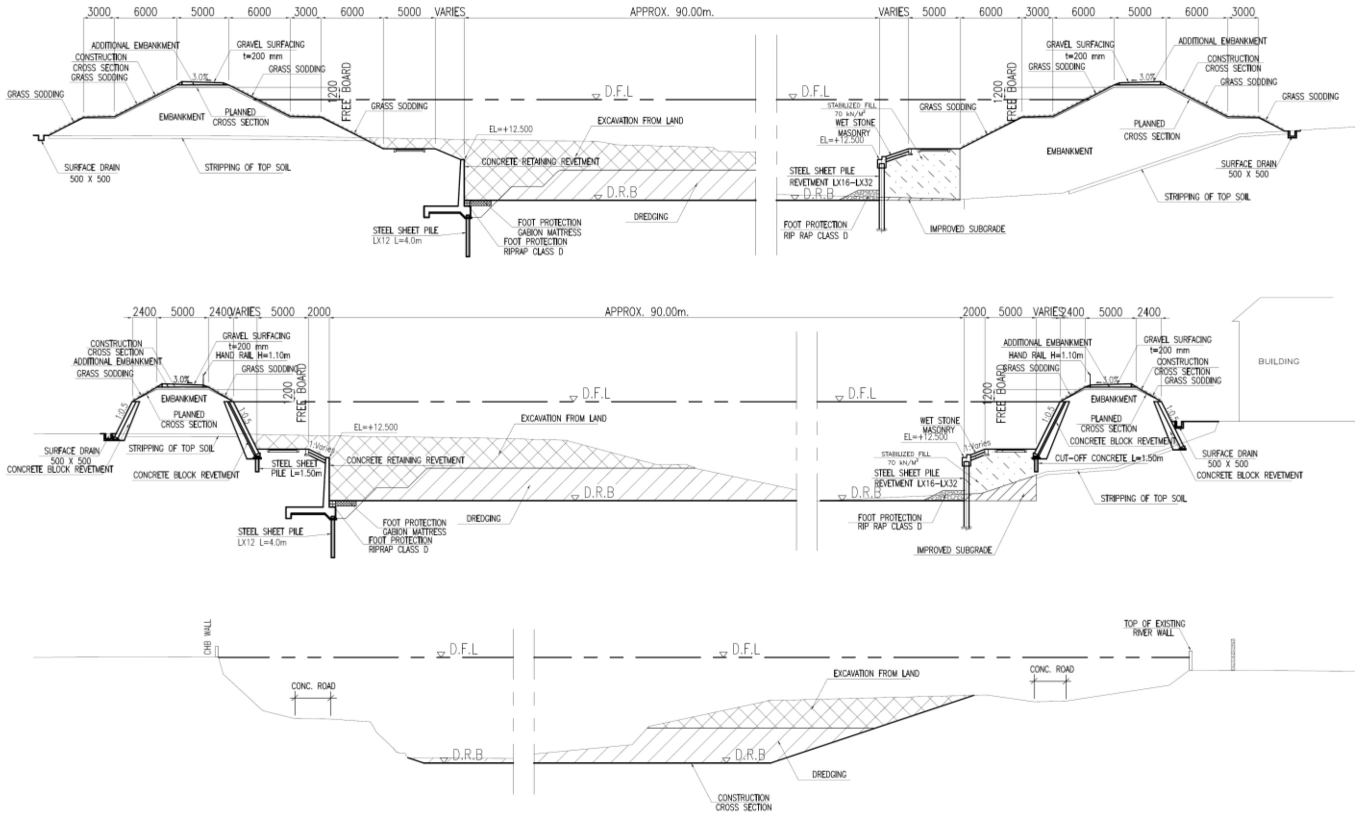
In the JICA1990MP, the design flood water level (DFL) directly upstream of the Rosario Weir is EL+16.8 m, and that at Sto. Niño is EL+18.9 m.

(2) Standard Cross Section proposed in DPWH2002DD

In the detailed design of the PMRCIP Phase I in 2002, further development of the riverside conditions in 1990 were considered and the roughness coefficient for the Manggahan Floodway concrete-lined channel section (n=0.014 to 0.021) was reviewed.

In addition, standard cross sections were designed for the following three different longitudinal sections:

(a) Downstream Section: Low water channel (excavation/vertical wall revetment (Steel Sheet Pile/Inverse-T Revetment)) + Embankment Section
(b) Intermediate Section: Low water channel (excavation/vertical wall revetment (Steel Sheet Pile/ Inverse-T Revetment)) + Embankment Dike with Slope Protection
(c) Upstream Section: Low water channel (excavation with non-protected slope) + Non-soil levee (parapet concrete wall)



Source: 2002 DD Report

Figure 4.1.3 Standard Cross Sections Proposed in the 2002DD for the Phase IV Marikina River Improvement Project

In the 2002DD, the design flood water level (DFL) directly upstream of the Rosario Weir is EL+17.4 m, and that of Sto. Niño is EL+20.472 m.

(3) River Channel Plan in 2015IV&V

1) Standard Cross Section proposed in 2015IV&V

The river channel layout and cross sections of the Definitive Plan implemented in 2015IV&V were established after discussions between the DPWH and the concerned parties including JICA. Regarding the determination of river channel alignment, a comparative study was conducted on the river channel width of 80m (in order to minimize social impact) as Plan-1 recommended by the DPWH, as well as 90m (Plan-3) and 80 ~ 90m (partial widening) as Plan-2. These three options were compared as to the hydraulic conditions, compensation for obstacles and structures along riverbanks, and overall project costs. The details of comparison for each option are shown in **Table 4.1.1**.

Plan-2 (partial widening) is the most economical option. However, it has the most significant impact on land, and the social impact is substantial. In Plan-1 (80m), the average flow velocity exceeds 4m/s, which is critical in terms of revetment durability and riverbed scouring. Therefore, Plan-3 (90m) (river channel width: 90m up to Sta. 10+500 and 80m for the upstream) was finally recommended.

Table 4.1.1 Comparison of River Channel Layout Options

Item		2002 D/D	Alternative Plans*			Remarks
			Plan 1: Riverbed Width With 80m	Plan 2: Riverbed Width with Partial Widening (90-115-100m)	Plan 3: Riverbed Width with 90m	
Technical Aspect	Clearance on Marcos Bridge	Maintain	Secured by Reconstruction (0.36m)	Maintain	Secured by Reconstruction (0.89m)	Design Code : more than 1.2m, ( ) shows the clearance in case of No Reconstruction.
	Average Dike Height (Difference between Ground Elevation in the Bank and Top Elevation of the Dike)	1.50m	2.02m (+0.52m)	1.59m (+0.09m)	1.65m (+0.15m)	( ) shows the Increase on average of Water Level at the section of Sta.6+700- Sta.10+500 compared with 2002 D/D.
	Maximum Mean Flow Velocity (near Rosario Weir)	3.5m/s	4.2m/s	3.7m/s	3.7m/s	4.0m/s is assumed as the Flow Velocity to increase the Scouring Risk for the River Channel Improvement Plan
Social Aspect	Circulo Verde and Olandes STP	Partial Acquisition/ Compensation	Maintain	Maintain	Maintain	Constructed after 2002.
	SM-Marikina	Maintain	Maintain	Maintain	Maintain	Constructed after 2002. Compensation of the Part of Access Road is necessary for all Plans.
	Number Establishment under Operation for Relocation and Compensation	12(7)	6(4)	13(8)	7(5)	( ) shows the Number of Acquisition and/or Compensation of Buildings.
	Flood Wall by Pasig City LGU	Removal	Maintain	Removal	Maintain	Constructed after 2002. Coordination for 1,100m Future Plan is required with Pasig City LGU in all Plans.
	Number of Affected Houses	1,200 (in 2002)	39	936	170	Including ISFs.
	Number of Affected Areas (m^2)	314,100	92,000	180,400	122,900	Areas between the Alignment of River Channel and River Bank.
Project Scale	Construction and Compensation Cost (Mil Pesos)	---	16,070	15,960	16,430	
	Breakdown of Construction Cost of River Channel Improvement for Phase IV Section (Mil. Pesos)	---	13,560	14,760	13,680	Excluding the Construction Cost of MCGS.
	Breakdown of Reconstruction of Marcos Bridge	---	1,790	---	1,790	Traffic Volume: 70,000 Vehicle/Day
	Breakdown of Land Acquisition Cost (Mil Pesos)	---	720	1,200	960	Related to Item ⑦Number of Affected Areas

Note) *: Figures in the table above do not present the finalized ones but those as of Jan. 2015.

Source: Definitive Plan for PMRCIP Phase IV

According to the study result of the DPWH2015IV&V, the clearance at the bottom girder of Marcos Bridge above the Design Flood Level (DFL) is insufficient based on the DPWH Design Guidelines (DGCS). Thus, it is necessary to replace the Marcos Bridge, taking flood risk into account. Therefore, the elevation of new Marcos Bridge has to be higher when the old existing bridge becomes deteriorated and needs replacement in the future.

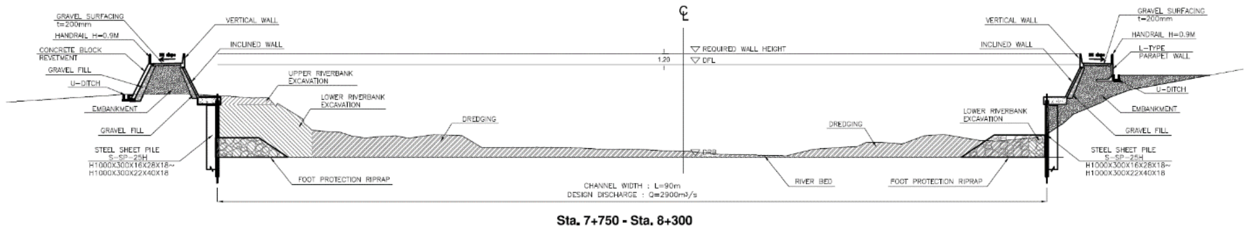
The design conditions in the DPWH2015IV&V are as given below.

Table 4.1.2 Design Conditions in the Definitive Plan (2015)

Items	Design Policy
Design Flow Rate	2,900 m^3/s
Freeboard	1.2 m
Basic Concept of River Alignment to be improved	Fit into the existing channel
Longitudinal Gradient of Design Riverbed	1/4 000 (Rosario Weir – Marikina Bridge)
Low Water Channel Width	Rosario Weir ~ 10+500: 90 m 10+500 ~ 11+000: Widened to 90 m on left side only 11+000 ~ 13+350 (Marikina Bridge): 80 m
Revetment and/or Slope Protection for high water channel	Rosario Weir ~ Sta. 10+500: Inclined Concrete Wall Sta. 10+500 ~ Sta. 12+500: Heightening of Existing Concrete Wall, Construction of new parapet wall Sta. 12+500 ~ Sta. 13+350: No flood protection facility on both banks as requested by the city and the residents. (However, widening of the low water channel will be conducted.)
Maintenance Road Width	3 m macadam pavement

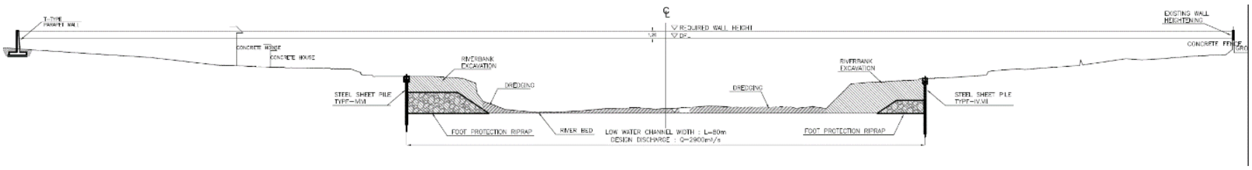
Source: Definitive Plan for PMRCIP Phase IV

The standard sectional view in the 90m section and the standard sectional view in the 80m section are as shown below.



DFL: EL+17.4 m at Rosario Weir and EL+21.18 m at Sto. Niño
Source: Implementation Program (September 2018, DPWH)

Figure 4.1.4 Standard Section of Renovated 90m Low Channel Section



DFL: EL+17.4 m at Rosario Weir and EL+21.18 m at Sto. Niño
Source: Implementation Program (September 2018, DPWH)

Figure 4.1.5 Standard Section of Renovated 80m Low Channel Section

2) Design Floodwater Level (DFL)

The DPWH2015IV&V set the DFL in the river channel stretch of Phase IV and Phase V using HEC-RAS in conjunction with the examination of the standard cross sections described above.

HEC-RAS is a hydraulic analysis software developed by the US Army Corps of Engineers (USACE) and commonly used worldwide. The guideline for river planning in the Philippines (DGCS, Volume III) recommends HEC-RAS as one of the hydraulic analyses software to be used.

However, there is a difference in concept about water level rise to be considered in river channel planning between HEC-RAS and the design criteria of Japan. Therefore, the "difference" is herein verified.

The methods for calculating the water level rise due to bridge pier (⊿h02) and meandering (⊿h03) of the HEC-RAS adopted in the DPWH2015IV&V are different from those of the Japanese "Guideline for River Channel Plans," as shown in **Figure 4.1.3**.

Table 4.1.3 Difference in Calculation Methods of Water Level Rise due to Pier and Meander

Water Level Rise Item	DPWH2015IV&V (HEC-RAS)	Japanese Guideline for River Channel Plans
Water level rise due to bridge pier (⊿h02)	Yarnell Formula	D'Aubuisson Formula
Water level rise due to meander (⊿h03)	Reflect in roughness coefficient (n = 0.028)	Estimate using the simplified formula

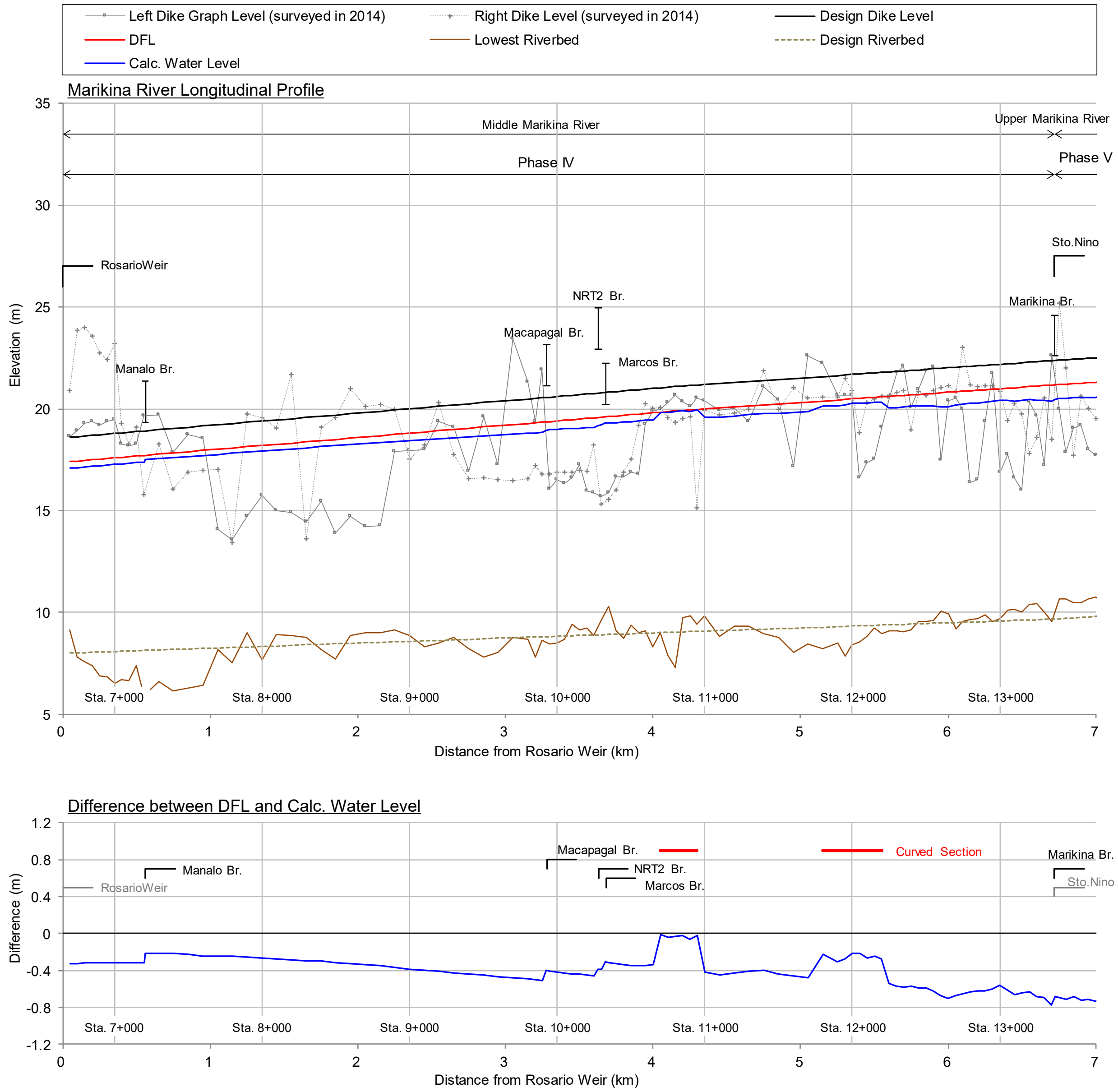
Source: Study Team

In calculating ⊿h02 due to bridge pier, the d'Aubuisson Formula is used for river channel planning in Japan, while the Yarnell Empirical Formula is used for the Definitive Plan (DPWH2015IV&V). According to Kawai and Matsumoto (1979), the calculation results by the d'Aubuisson Formula" is generally said to be overestimated (safety side)."[1] In the Definitive Plan, ⊿h03 is considered for the roughness coefficient. This is because the roughness coefficient (n=0.028) is on the safety side

[1] On the raising backwater by bridge piers- official criticism of d'Aubuisson formula and proposal of a practical formula- Toru Kawai and Yoshio Matsumoto, Journal of Japan Society of Agricultural Civil Engineering 47 (7), 1979

taking into account the effect of meandering to the roughness coefficient (n=0.025) of the designed channel with a corrugated steel sidewall as used in the Pasig-Marikina River Channel Plan.[2]

In this connection, the longitudinal water level calculated at n=0.025 based on the "Guideline for River Channel Plans" and taking into consideration bridge piers and bends of river alignment is as shown in **Figure 4.1.6**. As the result, it was confirmed that the calculated water levels at "bridge piers" and "bend" are within the DFL of the Definitive Plan in 2015 and, therefore, this Detailed Engineering Design has adopted the DFL of the Definitive Plan.



Source: Study Team

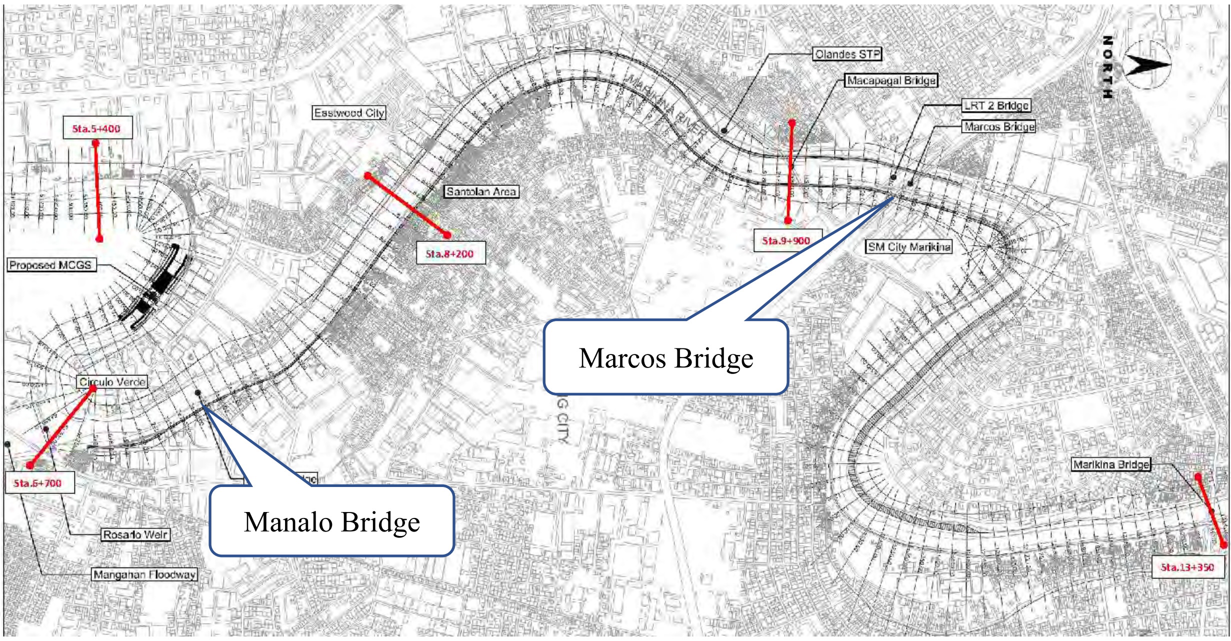
Figure 4.1.6 Results of Water Level Calculation

(4) Manalo Bridge Replacement Section by DPWH

At present, the replacement work of the Manalo Bridge located at Sta. 7+210 within the Phase IV project section is being carried out by the DPWH with its own funds (**Figure 4.1.7**). As of January 2020, the construction work at the immediate upstream section of the Bridge on the left bank has

[2] Open Channel Hydraulics, Ven Te Chow (1959)

started (driving of SSP for low water channel revetment), and the DPWH is also negotiating with the other landowners of the riverside in order to acquire land necessary for the bridge replacement.



Source: Implementation Program (September 2018, DPWH)

Figure 4.1.7 Location Map of Manalo Bridge and Marcos Bridge

(5) Basic River Plan Design Policy in this Design

As described in items (1) to (3) of this section, a lot of studies have been conducted for the design of the target sections of this basic design work. In DPWH2015IV&V, the former Secretary of DPWH also participated in the discussions for the river channel plan.

Since 2015, many development activities have been planned along the riverside of the design section, and some projects have encroached the low water channel of the existing river channel.

In this Detailed Engineering Design, the basic channel plan determined in DPWH2015 IV&V is not changed to "further narrow the width" for the following reasons:

- Even in the present river channel plan, the bottom girder of the Marcos Bridge located at Sta. 10+330 does not satisfy the clearance of elevation, so that future replacement is indispensable. No further change that would affect other infrastructures will be made to the river channel plan.
- The current DFL is as high as the relatively high sections in the existing riverbank. If the DFL is further raised, the banks at many sections of the drainage system may suffer during river floods.
- The results of public hearing and consultations confirmed that many residents feel negative about high levees.

4.1.2 Additional Hydraulic Investigation

4.1.2.1 Investigation of Effect of Rising Water in Upstream Channel due to MCGS Construction

Some stakeholders in the Philippines are concerned that:

- Construction of the MCGS will significantly raise the water level of the upstream channel; or
- Lowering of the Floodway DFL at the Rosario Weir is the most effective measure for flood control in the upper reaches of the Marikina River.

To avoid misunderstandings, hydraulic analysis was conducted to check the effect of water level of the upstream channel section with the MCGS, as described below.

(1) Method of Hydraulic Analysis

Due to the construction of the MCGS, there is concern that the water level of the upstream channel will rise if the flow rate to the lower reaches of the Marikina River is controlled to the maximum of 500 m^3/s. The influence of this water rise has been confirmed using the non-uniform flow calculation as well as the hydraulic model experiment as explained in **Chapter 8**. A calculation model was constructed to reproduce the calculated water level by HEC-RAS as applied in the Definitive Plan (DPWH2015IV&V).

(2) Study Cases and Water Level Calculation Conditions

Water levels of the Marikina River with and without the MCGS have been calculated under the two river channel conditions, namely, the present condition and the improved section, and the phenomena of water rising was confirmed under the two conditions. The study cases and conditions are shown in **Table 4.1.4**, and the non-uniform flow calculation conditions are shown in **Table 4.1.5**.

Table 4.1.4 Cases for Consideration and River Channel Conditions

Case	Flow Rate	No.	River Channel	MCGS	Remarks
1	Design Flood (2,900 m^3/s)	1-1	Present river channel	Yes	River Condition • Existing cross section (before Phase IV) • Improved Cross Section (after Phase IV)
		1-2	Present river channel	None	
		1-3	Planned channel	Yes	
		1-4	Planned channel	None	
2	Basic Flood (Flood without dam and retarding basin: 100-year flood with 3,600m^3/s)	2-1	Present river channel	Yes	
		2-2	Present river channel	None	
		2-3	Planned channel	Yes	
		2-4	Planned channel	None	

Source: Study Team

Table 4.1.5 Conditions for Non-Uniform Flow Calculation (Marikina River)

River Name	Marikina River	
Cross Sections	Sta. 6+700 to Sta. 19+250 (Manggahan Floodway to San Mateo Bridge)	
Calculation Method	Non-uniform flow calculation	
Consideration of ⊿h	Water level rise due to confluence (⊿h01): Nangka River Water level rise due to structures (⊿h02): Manalo Bridge, Macapagal Bridge, Bridge for LRT-2, Marcos Bridge, Marikina Bridge, Tumana Bridge, and San Mateo Bridge (Yarnell formula)	
Cross section used for calculation	2015IV&V present cross section, and 2015IV&V planned cross section	
Roughness Coefficient	Sta. 6+700 to Sta. 19+250: 0.028	
Flow Condition	a. Design Flood (return period: 1/100) (with dam and retarding basin) Sta. 6+700 to Sta. 18+650: 2,900 m^3/s Sta. 18+700 to Sta. 19+250: 2,600 m^3/s	
	b. Basic Flood (return period: 1/100) (without dam nor retarding basin) Sta. 6+700 to Sta. 18+650: 3,600 m^3/s Sta. 18+700 to Sta. 19+250: 3,200 m^3/s	
Initial Water Level	The water level at the upstream end of Manggahan Floodway (calculation condition: **Table 4.1.6**)	
	1. During Design Flood With MCGS: 16.84 m Without MCGS: 16.24 m	2. During Basic Flood With/Without MCGS: 16.84 m

Source: Study Team

The water level of the immediate upstream section of Manggahan Floodway was given as the water level of Marikina River (Manggahan Floodway) at Sta. 6+700. **Table 4.1.6** shows the conditions for calculating the water level of the Manggahan Floodway.

Table 4.1.6 Conditions of Non-Uniform Flow Calculation (Manggahan Floodway)

Name of Waterway	Manggahan Floodway
Target Section	Sta. 0+000 to Sta. 9+000 (Rosario Weir to Laguna Lake) * Upstream side is Sta. 0+000
Calculation Method	Non-uniform Flow Calculation
Cross-section used for calculation	Sta. 0 +000 to Sta. 1+200: Present cross section (by 2016 survey) Sta. 1+400 to Sta. 9+000: Planned cross section
Roughness Coefficient	Sta. 0+000 to Sta. 1+200: 0.021 (Concrete-Lined Sections) Sta. 1+400 to Sta. 9+000: 0.030 (No Revetment Sections)
Flow Condition	1. Design Flood in Marikina River With MCGS: 2,400 m^3/s Without MCGS: 2,000 m^3/s (The discharge of 2,000m^3/s in the floodway is assumed by Hydraulic Model Experiment when flood of 2,900 m^3/s flows down to Sto. Niño.)
	2. Basic Flood in Marikina River With/Without MCGS: 2,400 m^3/s (The amount of flow exceeding the design flood of the Floodway is assumed to flow into the Marikina River)
Initial Water Level	DFL of Lake Laguna: 13.80 m

Source: Study Team

(3) Result of Water Level Calculation

Water level at several points were calculated based on the above calculation conditions. The results are shown in **Table 4.1.7** and **Table 4.1.8**. To confirm the effects of the Phase IV project, the water level before and after the Phase IV project are compared for the Design Flood (**Figure 4.1.8**) and the Basic Flood (**Figure 4.1.9**), respectively.

- <u>In case of Design Flood</u>

A comparison between the water level of the existing channel condition (without MCGS) and the water level after the project (with MCGS) shows that the water level at Rosario Weir after the project with the MCGS is 0.6 m higher. However, the relationship of the two water levels (before/after the project) reversed at the immediate upstream section of the Manalo Bridge. At Sto. Niño, the water level after the project remained constant at one meter lower than the existing channel.

- ✓ Rosario Weir~Sta.7+500: the water level of the existing channel condition (without MCGS) is lower than that of after the project (with MCGS)
- ✓ Upstream Sections from Sta.7+500: the water level after the project (with MCGS) is lower than that of the existing channel condition (without MCGS)

- <u>In case of Basic Flood</u>

The amount of flow exceeding the design flood of the Floodway (2,400 m^3/s) has been assumed to flow into the Lower Marikina River. Thus, the initial water level (at the diversion point of Manggahan Floodway) of Marikina River, which provides the water level at the upstream end of the Manggahan Floodway, is 16.84 m regardless of the MCGS. Therefore, the initial water level at the immediate upstream section of the floodway (EL+16.84m) is the same under both the with and without MCGS conditions.

- ✓ All Sections from Rosario Weir: the water level after the project (with MCGS) is lower than that of the existing channel condition (without MCGS)

- <u>Conclusion of the Comparative Study</u>

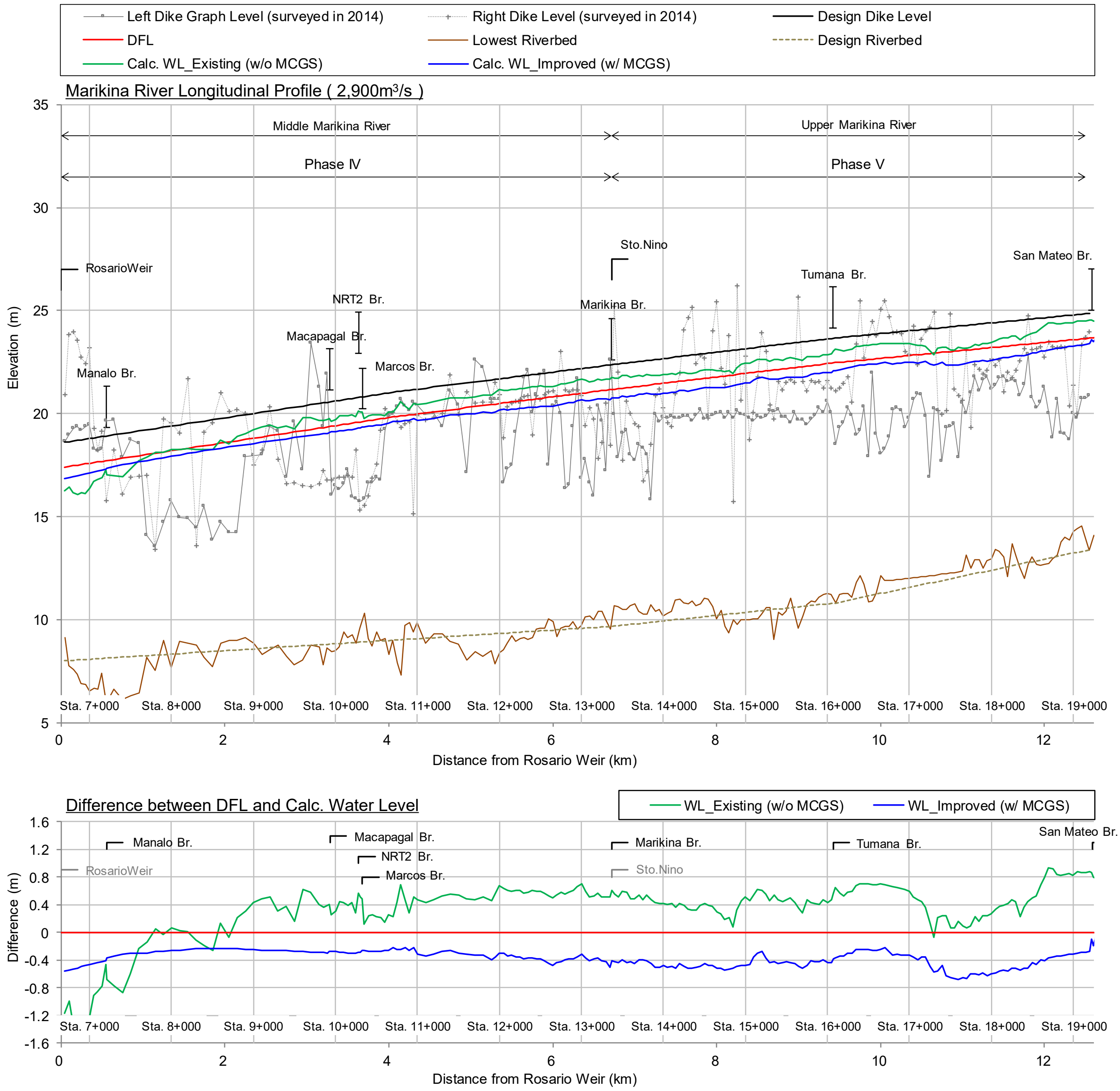
As shown in **Figure 4.1.8** and **Figure 4.1.9**, a comparison between the water level of the existing channel condition (without MCGS) and the water level after the project (with MCGS) shows that the water level at Rosario Weir is the same as explained above. However, the water level after the project is 0.2 meter lower at the Manalo Bridge, and one meter lower at Sto. Niño.

In conclusion, it can be said that the water level rise at the upstream channel due to the MCGS construction is very limited and the effect of lowering the water level due to the river channel improvement is more significant.

Table 4.1.7 Results of Water Level Calculation (Case 1: Design Flood)

No.	Case		Water Level (EL. m)			
			Rosario Weir	Manalo Bridge	Sto. Niño	San Mateo
1	Existing	w/o MCGS	16.24	17.03	21.78	24.53
		w/ MCGS	16.84	17.39	21.82	24.54
2	Improved	w/o MCGS	16.24	16.87	20.64	23.54
		w/ MCGS	16.84	17.34	20.76	23.57

Source: Study Team



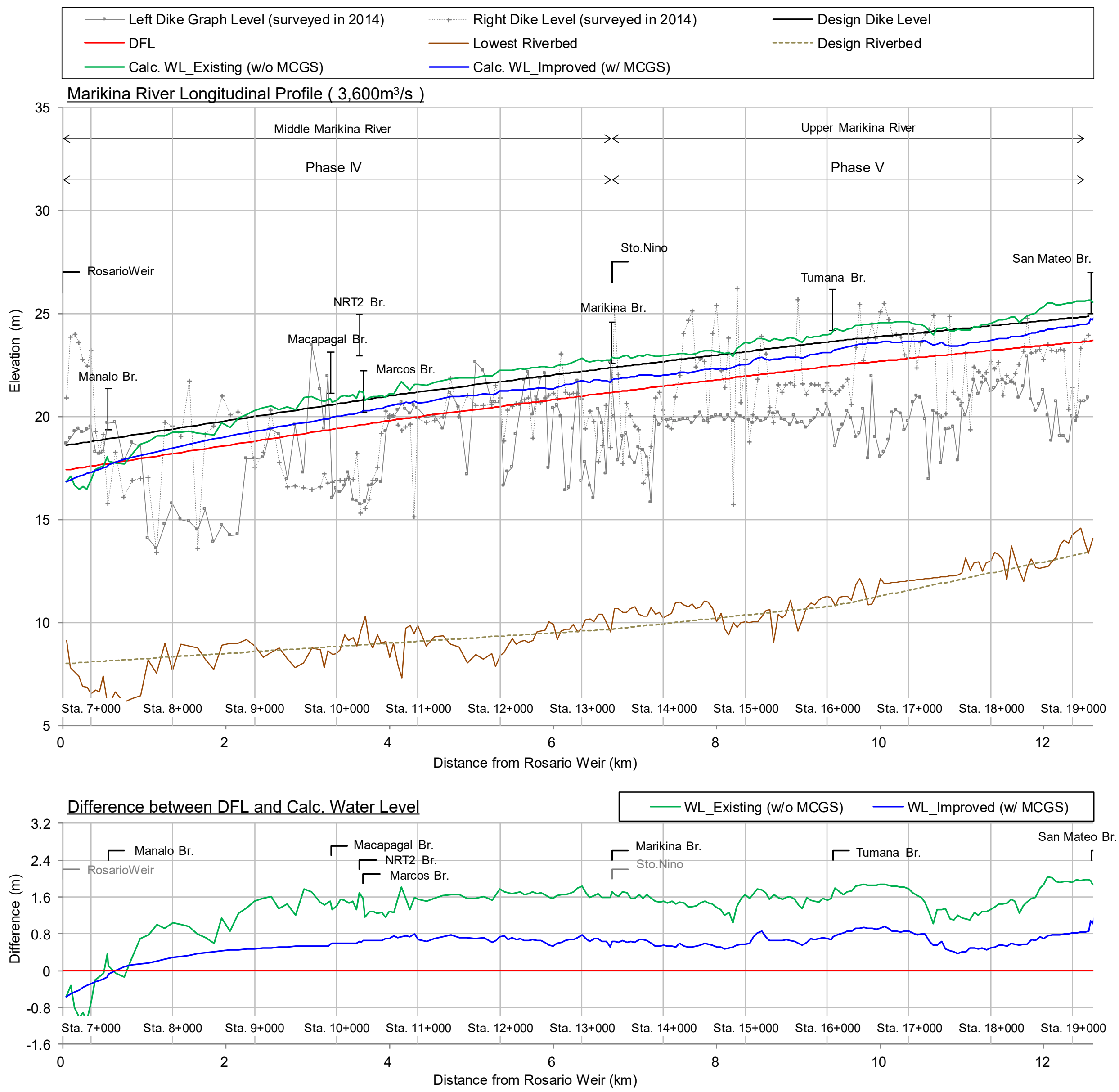
Source: Study Team

Figure 4.1.8 Longitudinal Elevation (Design Flood, 2,900 m³/s)

Table 4.1.8 Results of Water Level Calculation (Case 2: Basic Flood)

No.	Case		Water Level (EL. m)			
			Rosario Weir	Manalo Bridge	Sto. Niño	San Mateo
1	Existing	w/o MCGS	16.84	17.82	22.89	25.63
		w/ MCGS	16.84	17.82	22.89	25.63
2	Improved	w/o MCGS	16.84	17.64	21.8	24.74
		w/ MCGS	16.84	17.64	21.8	24.74

Source: Study Team



Source: Study Team

Figure 4.1.9 Longitudinal Elevation (Basic Flood, 3,600 m³/s)

4.1.3 Development Status along the River

Almost all lands on both riverbanks in the targeted stretch of Phase IV are fully utilized with no vacant spaces. In the downstream sections from the Marcos Bridge (around Sta. 10+300), land use is dominated mainly by commercial facilities, factories, warehouses, residential areas and others, while the upstream sections are mainly utilized as river parks. In most cases, residential areas exist behind the river parks.

Table 4.1.9 Development Status along Rivers

Station	Left Bank	Right Bank
5+400 ~ 6+200 Downstream end to MCGS	Land development by Ayala Corporation	Factories and warehouses are densely built. There are many ISAFs.
6+200 ~ 6+600 MCGS - Rosario Weir	Land development by Ayala Corporation 6+300 ~ 6+600: Existing revetment	Property of Circulo Verde 
6+600 ~ 6+700	Rosario Weir	
6+700 ~ 7+200 Rosario Weir – Manalo Bridge	Factories and warehouses are densely built. 6+700 ~ 7+050: Existing revetment	
7+200 ~ 7+600 Manalo Bridge	Factories	Factories 7+350 ~ 600 with existing revetment 
7+600 ~ 8+300	Residential areas (low-income group) 8+450 ~ 9+200: Revetment and riverside road dike by Pasig City is in progress	Factories to Eastwood (up to 8+300)
8+300 ~ 8+600		Commercial area (Eastwood City)
8+600 ~ 8+900		Factories and warehouses
8+900 ~ 9+200		8+900 ~ 9+100: Corps Base (Camp Atienza)
9+200 ~ 9+400	Cockpit	Undeveloped land (private land) Upstream of 9+300: Marikina City
9+400 ~ 9+900 Pasig/Marikina Macapagal Bridge	Cement plant and event facilities upstream of 9+600: Marikina City at 9+520 ~ 9+900, landfill is in progress (by a member of Batasang Pambansa (Former Mayor of Marikina) 	Olandes Sewage Treatment Plant  Riverside green space
9+900 ~ 10+300 near the Marcos Bridge	Land development by commercial facilities is in progress. 10+100: SM Marikina	Riverside green space 10+200 ~ 10+900: existing revetment
10+300 ~ 10+900	River park 10+100 ~ 10+500: SM Marikina 10+800: Riverbanks Convention Center	Commercial facilities (Marikina Riverbank Center) river park (Riverbanks) 10+200 ~ 10+900: Existing revetment
10+900 ~ 13+100	10+900 ~ 11+250: River park 11+250 ~ 11+550: Factories 11 + 550 ~: Residential area 11+500 ~ Marikina Bridge: Road 11+400 ~ 12+050: Existing revetment 12+080: Kalumpang Gymnasium 12+300 ~ 500: Factories	Riverside green areas and river parks 10+550 ~ Marikina Bridge: Road 10+900 ~ 13+100: River Wall
13+100 ~ 13+350 Marikina Bridge	River Park Steel sheet pile revetment	River Park Steel sheet pile revetment

Source: Field Survey Results by JICA Study Team

4.1.4 Existing Drainage Channels and Drainage Systems

Along the targeted stretch of the Phase 4 project, there are 290 existing drainage outlets flowing from the residential area into the river channel. These drainage systems are to be integrated, maintaining the existing topography and drainage systems as much as possible. After integrating and reducing the number of outlets of several small drainage systems, drainage outlets are to be installed at the revetment. The design concept and results are explained in detailed in **Section 6.2 of Chapter 6**.

4.2 Policy on River Channel Improvement Plan

4.2.1 Basic Policies on River Channel Improvement

The river channel improvement plan in the Definitive Plan (DPWH2015IV&V) has been reviewed according to the current land use, status of land acquisition by the DPWH, and social and environmental conditions in the surrounding area.

According to the Philippine Water Code (PWC), lands of 3 meters in width from both left and right shoulders of existing riverbanks serve as easement for public works. In addition, a law prohibiting development in a 10-m area from the existing riverbank to serve as a natural Environment Protection Area (EPA) is under consideration.

In principle, it is desirable that the centerline of the improved river alignment should be the same as the center of the existing water surface. However, it seems to be difficult to procure land and structures along the riverbanks in almost the entire design section. Therefore, the improved river channel alignment with revetment in each section should be designed and set according to the ease of land acquisition as informed to the Study Team by the DPWH.

The basic principles to fix the alignment of improved river channel and minimize the land acquisition and compensation for demolished buildings and properties are as explained in **Table 4.2.1**. There are a number of private lots that may be expropriated for the project according to the river channel alignment appropriately set as shown in **Table 4.2.1**.

Basically, the revetment structure for the low water channel and the highwater channel from Sta. 6+700 to Sta. 10+500 shall be the combination of Steel Sheet Pile (SSP) revetment and the leaning concrete revetment. In the upstream section (from Sta. 10+500 to Sta.13+350), the structure shall be a combination of SSP revetment and parapet wall, or the work shall involve heightening of the existing river wall.

The details of the standard cross section and the specifications of each structure are given in **Chapter 6**.

Table 4.2.1 Design Policy for Each Section of River Improvement based on the Basic Design

Station	Structure of Water Channel	ROW Line to be Set	Concerns
5+400 ~ 5+800 Downstream design endpoint to the downstream of the MCGS revetment	Gentle Slope Channel (Bottom width: 40m) Slope: 1:3.0 (V:H) Centerline should be set not to invade the ROW on both sides.	Left Bank: Top of existing slope Right Bank: Road at the edge of wall	There are many ISFs on the right bank.
5+800 ~ 6+200 MCGS and adjacent revetments	5+800 ~ 5+950: SSP + concrete revetment 5+950 ~ 6+110: Weir and Apron + Concrete revetment 6+110 ~ 6+200: SSP + Adjacent concrete revetment of weir	Left Bank: Top of existing slope Right Bank: Road at the edge of cliff, a wall	There are many ISFs on the right bank.
6+200 ~ 6+600 Upstream of MCGS revetment to Rosario Weir	Left Bank: SSP + 1:2 revetment Upstream from 6+350: Existing revetment Right Bank: SSP + 1:0.5 revetment River layout will be determined by land boundaries of the right bank. Distance between SSPs: 50 m	Left Bank: Top of existing slope Right Bank: Wall of Circulo Verde	
6+600 ~ 6+700 Rosario Weir	Left Bank: Present structures will remain (with excavation of the low water channel) Right Bank: SSP + 1:0.5 revetment River layout will be determined by land boundaries of the right bank	Left Bank: None Right Bank: Wall of Circulo Verde	
6+700 ~ 7+200	Left Bank: SSP + 1:0.5 revetment (Existing revetment will be removed)	Left Bank: 8 ~ 9m from the SSP	Acquisition of factory land (or

Station	Structure of Water Channel	ROW Line to be Set	Concerns
Rosario Weir to Manalo Bridge Distance between SSPs at the low water channel: 90m	Right Bank: SSP + 1:0.5 revetment River layout will be determined by land boundaries of the right bank.	Right Bank: Wall of Circulo Verde	warehouse) on the left bank is needed.
7+200 ~ 7+600 Upper part of Manalo Bridge Distance between SSPs at the low water channel: 90m	Left Bank: SSP + 1:0.5 revetment (Without small steps) Right Bank: SSP + 1:0.5 revetment (Without small steps; existing revetment will be removed.) Determine river alignment from left bank right-of-way. River layout will be determined by land boundaries of the left bank. As a result of the land acquisition negotiation by DPWH, there was an instruction to move the boundary set in the Definitive Plan (2015) to the right bank for a maximum of 14.5 m.	Left Bank: Boundary specified by DPWH Right Bank: ROW of the left bank is shifted to the right bank by 104 m.	The present river width is very narrow. At around 7+450 ~ 550, land acquisition of more than 10 m on the right bank will be needed.
7+600 ~ 7+750 Distance between SSPs at the low water channel: 90m	Left Bank: SSP + 1:0.5 revetment (With small steps; the width of steps ranges between 600 ~ 650.) Same centerline as the Definitive Plan Right Bank: 1. Inverse T-shaped retaining walls: (The existing revetments will be removed; connecting structure with SSP revetment should be examined.) (2) SSP + 1:0.5 revetment (With small steps; the width of the steps range between 600 ~ 650; existing revetments will be removed.)	ROW required width: 111 m Left Bank: 53 m from the centerline Right Bank: 58 m from the centerline	To reduce construction cost, inverse T-shaped retaining wall revetment is preferable. However, it is necessary to examine the connection with SSP revetment of upstream and downstream. (It will be defined at the detailed design.)
7+750 ~ 8+150	Left and right bank: SSP + 1:0.5 revetment (with small steps.) Distance between SSPs at the low water channel: 90 m Same centerline as Definitive Plan The river layout was determined so as not to affect the factory lot on the right bank.	ROW required width: 109 m Left Bank: 56 m from centerline Right bank: 53m from centerline, top of slope	There are many APs on the left bank side: Need negotiation with Pasig City
8+150 ~ 8+450	Left and Right banks: SSP + 1:0.5 revetment (with small steps.) Distance between SSPs at the low water channel: 90 m River layout determined by land acquisition restrictions on the right bank.	ROW required width: 109 m Left Bank: Determined by distance from right bank Right Bank: Top of slope	At 8+300 ~ 450, land of Eastwood City covers the river area. The present river width is 82 m. Need land acquisition
8+450 ~ 8+900	Left Bank: SSP + 1:0.5 revetment (with small steps), or existing revetment Right Bank: SSP + 1:0.5 revetment (with small steps.) Distance between SSPs at the low water channel: 90 m In the Definitive Plan, the river layout was determined not to affect the structure on the right bank; however, a new revetment was built on the left bank.	ROW required width: 109 m Left Bank: Existing revetment? Right Bank: Top of slope	At 8+450 ~ 8+800, the distance between existing revetments is only 95 m: Need land acquisition At 8+850 ~ 8+900, the distance between existing revetment is only 102 m: Need land acquisition
8+900 ~ 9+200	Left Bank: SSP + 1:0.5 revetment (with small steps), or existing revetment Right bank: SSP + 1:0.5 revetment (with small steps), need embankment.	ROW required width: 112 m Left Bank: Existing revetment? Right Bank: Top of slope	Available land width is ca.110m: Need land acquisition

Station	Structure of Water Channel	ROW Line to be Set	Concerns
	Distance between SSPs at the low water channel: 90 m In the Definitive Plan, the river layout was determined not to affect the structure on the right bank; however, a new revetment was built on the left bank.		
9+200 ~ 9+400 (upstream from 9+300 on the right bank is the area of Marikina City)	Left/right bank: SSP + 1:0.5 revetment (with small steps.) need embankment Distance between SSPs at the low water channel: 90 m Minimizing the impact on the structures of the left bank, the watercourse will be excavated to the extent that does not affect the road on the right bank.	ROW required width: 112m Left Bank: TBD Right Bank: road	Present river width is ca.70m: Need land acquisition
9+400 ~ 9+800 (upstream from 9+600 on the left bank “directly upstream of the STP)” is the area of Marikina City)	Left/right bank: SSP + 1:0.5 revetment (with small steps): need embankment Distance between SSPs at the low water channel: 90 m In the Definitive Plan, left bank will be excavated in order not to affect the Olandes STP; however, there are local objections about the excavation on the left bank.	ROW required width: 112 m Left Bank: TBD Right Bank: TBD, STP?	Olandes Sewage Treatment Plant is on the right bank and a cement plant is on the left bank; Need land acquisition of one or the other
9+800 ~ 10+500	Left/Right Bank: SSP + 1:0.5 revetment (with small steps): Need embankment Distance between SSPs at the low water channel: 90m In the Definitive Plan, the left bank is planned to be excavated to build the low water channel; however, land development is progressing on the left bank. 10+100 ~ 10+500 is SM Marikina	ROW required width: 112 m Left Bank: TBD, road? Right Bank: Existing revetment and road	Width of present river is narrow (ca. 50m in water surface width at 10+000). Need land acquisition of one or the other
10+500 ~ 10+850	Left Bank: SSP, parapet walls Right Bank: SSP, parapet walls Distance between SSPs at the low water channel: 80 m (SSP on both sides) In the Definitive Plan, a parapet wall was to be built at the top of the existing revetment at the right bank, and the SSP + parapet was to be constructed after excavating the left bank. However, to construct a low water channel, SSPs are to be installed at both banks instead. At the left bank, the River Wall (10+700 ~ 900) is under construction. If the height of the wall is sufficient, only the SSP at the low water channel will be provided.	Left Bank: TBD Location of existing revetment has been identified. Right Bank: Existing revetment and road	There is a road on the right bank from 10+550 to Marikina Bridge: Need design considerations
10+900 ~ 11+100	Left Bank: SSPs at the bottom edge of existing revetment, and parapet walls at the top. Right Bank: SSP will be installed at 80 m from the left bank SSP (with low water channel excavation) Current structure + River Wall raising or parapet wall Distance between SSPs at the low water channel: 80 m (SSP on both sides)	Left Bank: Existing revetment Right Bank: River Wall	Need check at the detailed design if the river walls on the right bank from 10+900 ~ 13+100 are used.
11+150 ~ 12+050	Left Bank: SSPs at the bottom edge of existing revetment (no parapet wall) Right Bank: SSP will be installed at 80m from the left bank SSP. (with low water channel excavation) Current structure + River Wall raising or parapet wall, Distance between SSPs at the low water channel: 80 m (SSP on both sides)	Left Bank: Existing revetment Right Bank: River Wall	There is a road on the left bank from 11+500 to Marikina Bridge: Need design considerations

Station	Structure of Water Channel	ROW Line to be Set	Concerns
12+100 ~ 12+500	Left Bank: SSP at the top of the low water channel slope, River wall raising, or parapet wall Right Bank: SSP will be installed at 80 m from the left bank SSP. (with low water channel excavation) Current structure + River Wall raising or parapet wall Distance between SSPs at the low water channel: 80 m (SSP on both sides)	Left Bank: River Wall Right Bank: River Wall	Construction is possible within the existing river width.
12+500 ~ 12+550	Smoothing section		
12+550 ~ 13+100	Left Bank: Low water channel is SSP, River Wall raising or parapet wall (Need to check the elevation of the road 100m to 200m east from the riverbank. If it is high enough, dike raising is not necessary.) Right Bank: Low water channel is SSP, river wall raising or parapet wall Distance between SSPs at the low water channel: 80 m The crown height of the SSP is the existing ground height. The centerline is the center of the current river.	Left Bank: River Wall Right Bank: River Wall	Mayor of Marikina City and residents have an opinion that high dike is not necessary: Need consideration in embankment design
13+100 ~ 13+350	Left Bank: Low water channel is SSP (no embankment) Right Bank: Low water channel is SSP, parapet wall Distance between SSPs at the low water channel: 80 m Crown height of the SSP is the existing ground height. Need to check the elevation of the land behind the banks. If it is high enough, it is not necessary to raise the existing dike. The centerline of upstream of the Marikina Bridge is close to the right bank, so the centerline shall be shifted toward the right bank.	Left Bank: Top Slope of the river terrace Right Bank: Shoulder of river terrace	Previous investigations had confirmed that no embankment will be constructed at the left bank. Need further investigation for the right bank.

Source: Study Team

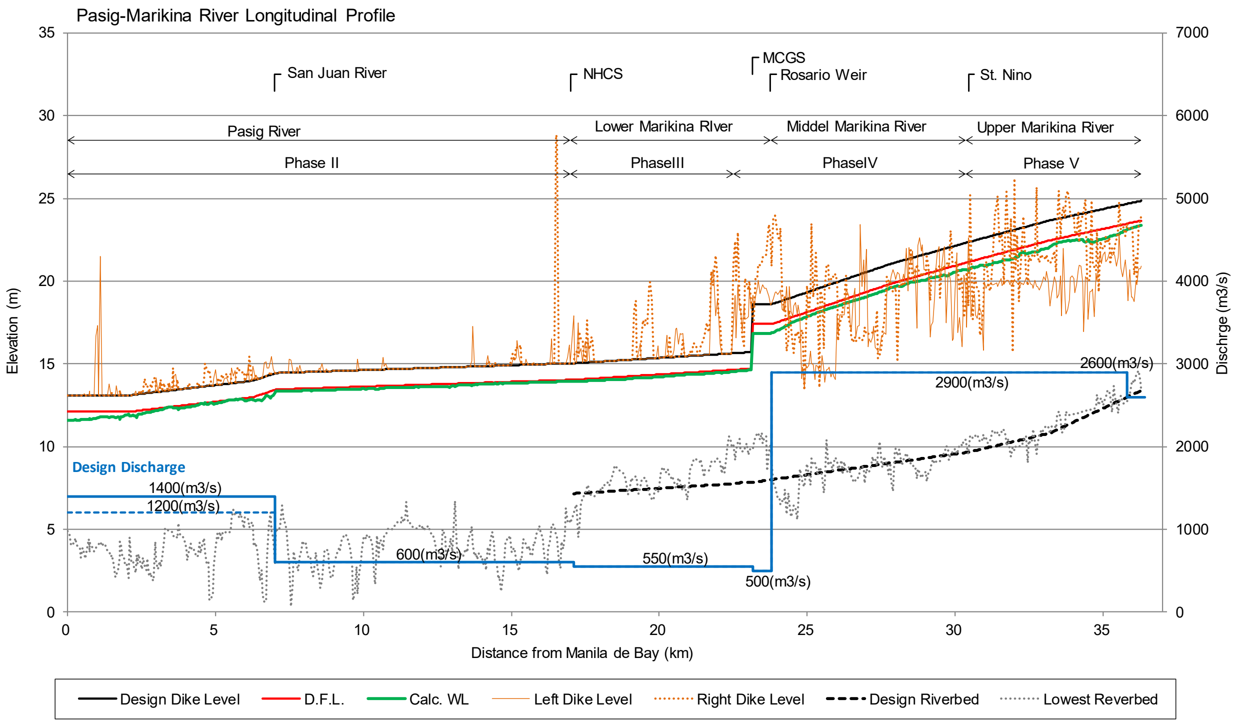
4.2.2 Longitudinal Profile of the Pasig-Marikina River

Figure 4.2.1 and **Figure 4.2.2** show longitudinal profiles of the Pasig-Marikina River and the Manggahan-Marikina River, respectively.

4.2.2.1 Longitudinal Profile from Rivermouth

As shown in **Figure 4.2.1**, the elevation of river banks (orange-colored lines) and riverbed (brown-colored dash line) in the Pasig River of which the length is about 17km is almost flat for whole of stretches. Therefore, the river water flows to downstream by the gradient of river water surface.

In addition, the riverbed (brown-colored dash line) in the Lower Marikina River (target section of Phase III) is annually rising trend due to sedimentation. In this connection, periodical dredging works will be required in order to sustain the elevation of riverbed by the design riverbed elevation.



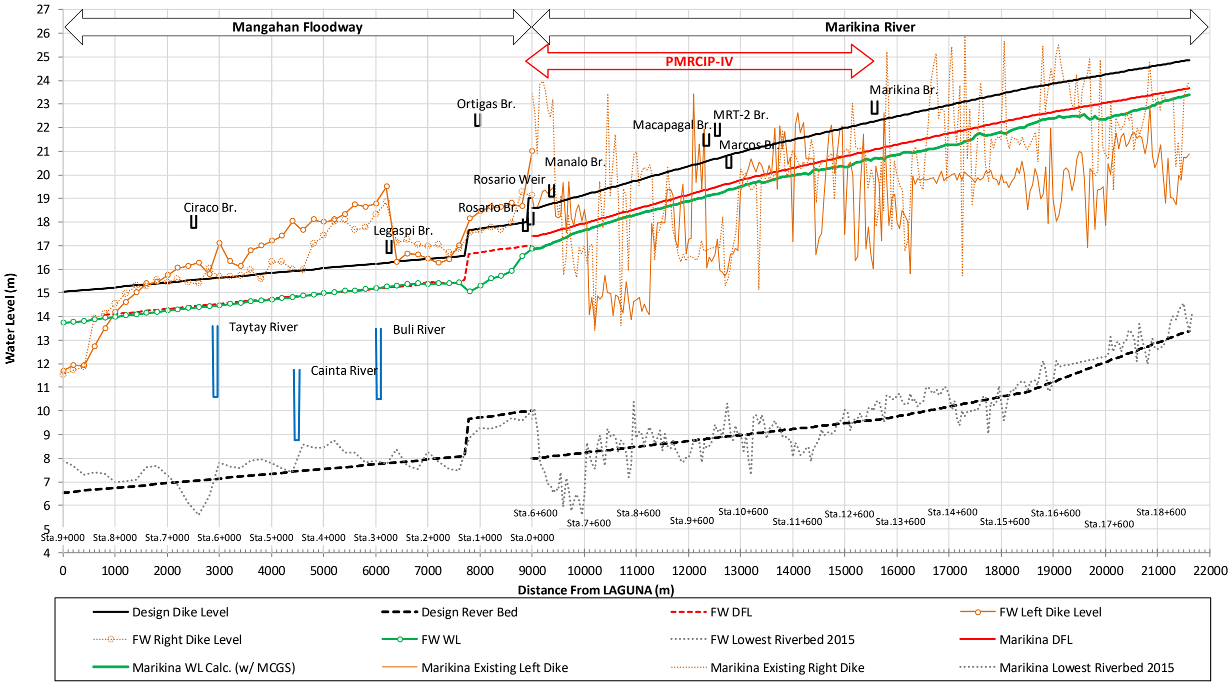
Source: Study Team

Figure 4.2.1 Longitudinal Profile of the Pasig-Marikina River (Manila Bay to San Mateo)

4.2.2.2 Longitudinal Profile from Laguna Lake

As shown in **Figure 4.2.2**, the sections between 7,800~9,000m from the Laguna Lake in the Manggahan Floodway of which the length is almost 9km in total are constructed by reinforced concrete and the elevation of bed (black-colored dash line) has been raised so that the flow discharge of the Floodway is not affected by the water level of the Laguna Lake.

The fluctuation tendency of the riverbed in the Marikina River as target section of the Phase IV has been less and the equilibrium situation has been continued taking into account the riverbed surveys in the past, such as 1988 (purple-colored dash line), 2001 (aqua-colored dash line) and 2015 (brown-colored dash line).



Source: JICA Study Team

Figure 4.2.2 Longitudinal Profile of the Manggahan-Marikina River (Laguna Lake to San Mateo)

CHAPTER 5 NATURAL CONDITION SURVEYS

5.1 Topographic Survey

5.1.1 Objectives and Scope of the Topographic Survey

The main purpose of the topographic survey is to produce the topographic and hydrographic maps with surveys of drainage outlets along river banks for the design and cost estimation, and establishment of concrete control points for reference during the construction stage.

5.1.2 Scope of Works

In order to obtain the data for the objectives mentioned in Section 5.1.1, the scope of topographic survey included the scope described in **Table 5.1.1** and targeted areas are illustrated in **Figure 5.1.1**.

Table 5.1.1 Scope of Topographic Survey

Contents	Target	Quantity	Details	Remarks
Topographic Survey (*1)	Marikina River	6 km^2	1:500 Accuracy	Sta.5+400-Sta.13+350
		10 has	1:200 Accuracy	For the MCGS
	Manggahan Floodway	3 has	1:200 Accuracy	For the Cainta Floodgate
		1 ha	1:200 Accuracy	For the Taytay Sluicegate
Hydrographic Survey with River Traversing Survey (Cross-sectional survey)	Marikina River	320 sections	20-m interval	Sta.5+400-Sta.13+350
	Manggahan Floodway	5 sections		For the Cainta Floodgate
		5 sections		For the Taytay Sluicegate
	Drainage channel investigation	500 places	-	Includes location, bed height and cross-section of drainages
Survey for Drainage Outlet	Marikina River	All Outlets		As a result, there are 289 outlets.

**1: with Control points and temporary benchmarks installation (45 points)*
Source: JICA Study Team

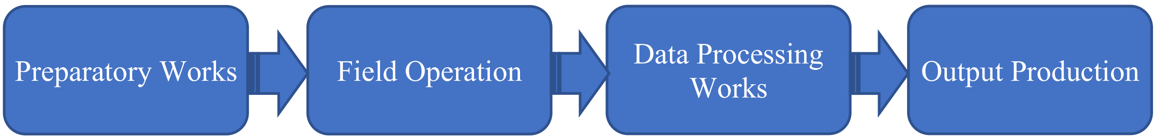
Source: JICA Study Team

Figure 5.1.1 Areas for Topographic Survey

5.1.3 Methodology of the Topographic Survey

5.1.3.1 Flow and Process of Survey Works

Topographic survey were conducted by sub-contract and the flow of works were executed in accordance with **Figure 5.1.2** shown below.



Source: JICA Study Team

Figure 5.1.2 Work Flow of Topographic Survey

5.1.3.2 Preparatory Works

As preparatory works, the following activities were conducted.

- Kick off meetings
- Flight Planning
- Securing of Permits
- Reconnaissance and GCP Marking
- NAMRIA GCP and Research Benchmark

5.1.3.3 Filed Operation / Works

As field operation and works, the following activities were undertaken.

- Horizontal and Vertical Control Survey
- Aerial Survey
- Topographic and Hydrographic Surveys
- Drainage Inventory Survey
- Borehole Survey

5.1.3.4 Data Processing Works

As data processing works, the following calculations and/or analyses were conducted.

- Ground Control Point Computation
- Image Processing
- Topographic Data Computation
- Data Plotting and Layout
- Data Correction

5.1.3.5 Production of Outputs

As the works for production of outputs, the following works were undertaken.

- Preparation of Ground Control Descriptions and Documentation
- Production of Surfaces, Profiles, Cross Sections and other drawings
- Production of drainage inventory

5.1.4 Survey Results

The all outputs by topographic surveys have been utilized in the Drawings of the Bidding Document as bases of plans, cross-sections and detailed location of exiting/planned structures.

In this sub-section 5.1.4, the main productions by topographic surveys are described hereinafter.

5.1.4.1 Establishment of Control Points

Control points were preliminarily and basically marked on roads using concrete nails and washers or marked on the ground using nails on wooden stakes. For adequate visibility and identification in aerial photos, all control points were marked using paint or white sacks forming an "L" shape with the point at the vertex. Control points that were also used for ground surveys were permanently marked with a concrete monuments or cement putty as shown in **Figure 5.1.3**.

Source: JICA Study Team (Sub-Contractor)

Figure 5.1.3 Examples of Control Points Established in this Detailed Design

A total of forty-seven (49) permanent or temporary GCPs (Ground Control Stations) were installed in the study areas. Forty-five (45) were installed in the Pasig-Marikina River topo area, two (2) in the Cainta Creek topo area, and two (2) in the Taytay Creek topo area. Of the forty-five in the Pasig-Marikina River topo area, six (6) were also in the MCGS topo area. (See **Figure 5.1.4** below.)

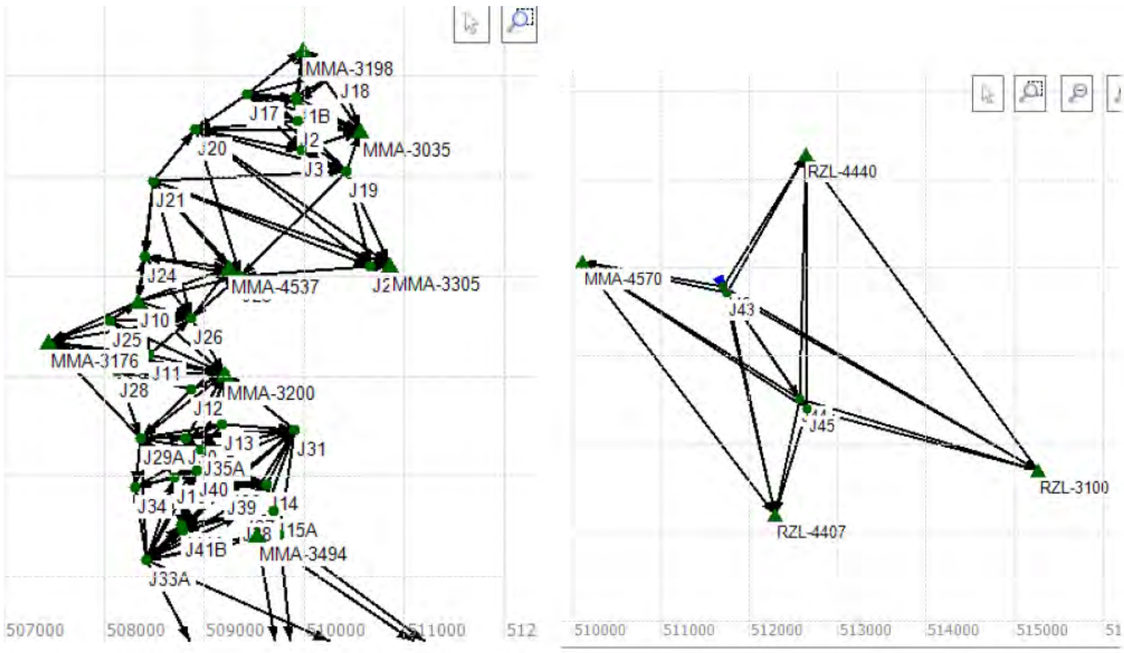
Source: JICA Study Team (Sub-Contractor)

Figure 5.1.4 GCP Locations

5.1.4.2 Horizontal Control Survey

(1) Polygonal Surveying for Control Points

As polygonal surveying for control points, GNSS (Global Navigation Satellite System) survey was conducted to obtain the horizontal coordinates of control points using survey grade GNSS receivers (Sokkia GRX-2, Sokkia GCX-2). Length of GNSS observations and network design were made as shown in **Figure 5.1.5**.



Source: JICA Study Team (Sub-Contractor)

Figure 5.1.5 Pasig-Marikina River (left) and Rizal (right) GNSS Network

(2) Vertical Control Survey

A forward and backward closed loop leveling method was used for the Vertical Control Survey to obtain the elevations of the GCPs. Leveling started from a certified NAMRIA benchmark (MM-310 & MM-103), connected to unknown points (temporary benchmarks & GCPs), and ended at the same or other certified NAMRIA benchmarks. Leveling was conducted using digital levels (Leica Sprinter 250). All leveling lines complied with the required DPWH accuracy, $10mm\sqrt{k}$ where k is the length of the leveling line in km.

Results are tabulated in **Table 5.1.2** and **Table 5.1.3**.

Table 5.1.2 Leveling Routes and Accuracies

LEVEL RUN	CONTROLS	Error of Closure (mm)	Allowable EOC (mm)	Loop Length (m)
1	MM310-J11-J12-J27-J31-J15-MM103	0.943	19.251	3706.197
2	MM310-J23-J22-J19-TBM5-J1-J2-J11-MM310	-6.925	32.689	10685.496
3	J11-J10-J25-J28-J29-J30J35-TBM7	6.707	21.062	4436.236
4	J8-JM24-J7	0.671	7.965	634.383
5	MM310-J26	0.159	7.105	504.747
6	J19-J34-J33	-4.919	16.607	2757.84
7	TBM51-J36-J37	2.31	9.091	826.473
8	TBM3-J1	-2.472	4.956	245.644
9	JIA-J1B	0.243	3.282	107.741
10	TBM5-J18	-2.819	7.444	554.125
11	TBM359-J17	-2.565	13.332	1777.55
12	TBM8-J20	1.4	10.527	1108.276
13	J8-J21	3.101	8.035	645.559
14	J9-J24	-0.212	11.076	1226.68
15	J15-J14	-0.619	8.909	793.78
16	J15-J32	-0.35	7.872	619.731
17	MM103-J41A-J41B-J40	-5.852	18.417	3391.696
18	TBM35-J39	-2.362	7.235	523.497
19	J29-J29A	0.128	1.755	30.806
20	J30-J13	-2.648	13.108	1718.121
21	J15-J38	-0.048	8.842	781.758
22	J15-J15A	-0.452	4.333	187.784
23	TBM37-J16	-1.145	9.059	820.722
24	J15-J42-J43-J44-J45	-8.479	34.952	12216.28
25	J14-MT11	1	4.509	203.28

Source: JICA Study Team (Sub-Contractor)

Table 5.1.3 Control Survey Results

GCP	N	E	Z		GCP	N	E	Z
J1A	1618731	509934.5	5.732		J24	1617199	508404.6	6.379
J1B	1618783	509922.3	5.933		J25	1616545	508065.7	12.027
J2	1618539	509943.5	3.748		J26	1616575	508876.7	9.440
J3	1618245	509984.2	4.214		J27	1616017	509208.9	8.412
J4	1617723	510016.4	4.459		J28	1616046	508128	10.701
J5	1617350	509752.9	5.451		J29A	1615377	508369.1	7.628
J6	1617600	509336.7	5.326		J30	1615368	508817.9	9.085
J7	1618042	509074.6	3.211		J31	1615458	509905.6	8.914
J8	1617724	508702.9	3.009		J32A	1614412	509752.7	7.095
J9	1617098	508598.6	5.105		J33A	1614159	508420.2	8.001
J10	1616740	508346	5.540		J34	1614878	508313.2	12.877
J11	1616208	508450.4	5.935		J35A	1615262	508964.8	7.555
J12	1615864	508874.1	5.670		J36	1614925	509253.6	7.342
J13	1615506	509179.3	5.368		J37	1614679	509387	8.497
J14	1614916	509625.2	8.798		J38	1614597	509363.1	4.958
J15A	1614645	509693.3	8.925		J39	1614875	509203.3	4.039
J16	1614980	508702.9	3.099		J40	1615052	508928	6.018
J17	1618804	509433.2	11.237		J41A	1614503	508780	4.092
J18	1619033	510341.2	11.108		J41B	1614448	508784.2	6.738
J19	1618046	510418.6	9.793		J42	1610781	511705.4	8.000
J20	1618459	508907.7	10.178		J43	1610699	511751.2	8.868
J21	1617954	508495.2	12.201		J44	1609506	512577.6	5.527
J22A	1617091	510665.7	8.808		J45	1609385	512661.6	5.413
J23	1616992	509366.5	10.207					

Source: JICA Study Team (Sub-Contractor)

5.1.4.3 Aerial Survey

An aerial photogrammetry survey was done for the Pasig-Marikina River topo area. The survey produced orthomosaic, DSM, and DTM for the area. Suitable established GCPs were marked to be used as constraints in the image processing or checks to the produced Orthomosaic, DSM, and DTM for the outputs to be suitable to be used as supplements to the ground survey.

A combination of manned two types of unmanned aerial vehicles (Sensefly eBee X & DJI Phantom 4 Pro v2) were used to capture aerial imagery of the project area (See **Figure 5.1.6**). All image acquisition was done at a flying height of 120m as per the awarded CAAP flying permit.

The parts of the topo area north of the Marcos Highway bridge (Riverbanks to Sto Nino area) was covered by the eBee X for faster acquisition. The rest of the area was covered by the Phantom 4 Pro v2 for its maneuverability in areas with high-rise buildings (Eastwood) exceeding the allowed flying height. The eBee X captured 2888 photos while the Phantom 4 Pro v2 captured 7981 photos (See **Figure 5.1.7**).

Source: JICA Study Team (Sub-Contractor)

Figure 5.1.6 Aircrafts used for Aerial Survey (Left: Sensefly eBee X, Right: Phantom 4 Pro v2)

Source: JICA Study Team (Sub-Contractor)

Figure 5.1.7 Photos taken by eBee X (left), Photos taken by Phantom 4 Pro v2 (right)

5.1.4.4 Hydrographic Survey and Cross Sectional Survey

The hydrographic survey was done using RTK (Sokkia GRX-2) and an echosounder (Seafloor Hydrolite-TM) mounted on a rubber boat. GCPs established near the river were used as base for the RTK and echosounder, and as control checks to correct the gathered data on the field. Bar checks were done at the start and end of each survey to calibrate the echosounder readings and RTK control checks were done for every control point that the boat passes by. Water levels were also measured using the RTK on the boat and checked by the RTK on the ground in between control checks. The survey covered around 8km of the Pasig-Marikina River, from station 5+400 to station 13+400. Hydrographic survey data was used to provide ground elevation data of the riverbed for the profile and cross sections, and to be combined with the DTM from the aerial survey for the final elevation model of the topo area.

Cross section survey was carried out every 20m along the PMR alignment as provided by the design team. Two cross section pegs, one on each side of the river, were established by staking the cross sections lines on the ground using RTK (Sokkia GRX-2).

5.1.4.5 Detailed Topographic Surveys

Ground topographic survey at a scale of 1:200 was done on the MCGS, Cainta Creek and Taytay Creek topo areas using RTK and total stations. Established GCPs and cross section pegs were used as control points for the survey. Data from the cross section and hydrographic surveys of PMR as well as from the drainage inventory survey were also incorporated in this survey.

All prominent structures such as

- Walls, Fences, Piers of Bridges, Culverts and Other Structures
- Edges of Pavements, shoulders of roads, dikes, drainage ditches and facilities
- Electrical lines, water pipes and optical fiber lines
- Trees, Electrical poles, Lightening Poles

were surveyed in the area and appropriately drawn in AutoCAD Civil 3D.

5.1.4.6 Others

(1) Drainage Inventory Survey

The horizontal and vertical coordinates, type, and dimensions of drainages directing flow into the Pasig-Marikina River, Cainta Creek, and Taytay Creek were determined using total stations, and tape. The established GCPs in the areas were used as controls for the inventory survey.

A total of 290 drainages were surveyed and measured in the three locations. Drainage pipes inside manholes (CR1.1, CR1.2, CR1.3, and CR1.4) were also surveyed and measured in the Cainta area.

(See **Figure 5.1.8** and **Table 5.1.4** below.)

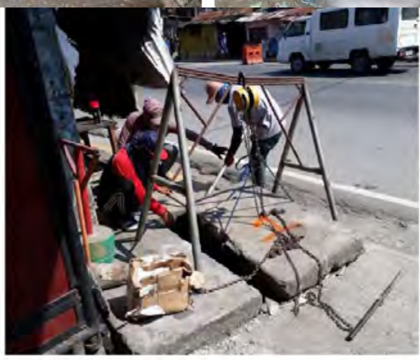


Figure 29: Opening of Manholes in Cainta

Drainage Outlet Descriptions — Upper Marikina River Channel Improvement Works (PMRCIP Phase IV & V)

Photo (distant view) | Photo (short range view)

Drainage Outlet No.=	ML049
Northing=	1617362.446
Easting=	509600.139
Elevation=	19.844
Dimension=	D100mm
Type=	PVC
Date=	6/13/2019
Station no.=	11+793

Source: JICA Study Team (Sub-Contractor)

Figure 5.1.8 Location of Reference Points for the coordinates of Drainages

Table 5.1.4 Drainage Outlets confirmed in the Topographic Survey

Location	The Number Surveyed
Marikina River (Left Bank)	116
Marikina River (Right Bank)	166
Cainta Creek	7
Taytay Creek	1
Total	290

Source: JICA Study Team

(2) Boreholes Survey

Horizontal and vertical coordinates were determined for boreholes made by the Boring Surveyor using RTK and total stations. Established GCPs and cross section pegs were used as control points in the survey. Boreholes were located through turnover from a representative from geological surveyor or through photos taken during the boring. Onshore boreholes that were still located during the conduct of the ground survey were surveyed exactly on the borehole while those that could no longer be located and those offshore were surveyed on estimated locations based on the photos and advice from the geological surveyor's representative.

Source: JICA Study Team (Sub-Contractor)

Figure 5.1.9 Borehole Located During Ground Survey (Left), Borehole Marked and Documented by AGES (right)

5.1.4.7 Quality Assurance

Accuracy of data was checked by connecting with GPS survey and total station traverse survey for horizontal controls, and by comparing with multiple leveling observations for vertical controls. Mutual accuracy was confirmed by connecting the given points of this survey with those of the previous Pasig-Marikina River Channel Improvement Project (Phase III). Reference back target check for topographic survey and drainage outlet survey were carried out at appropriate intervals to confirm the stability of total station and accuracy of the observation. Leveling observations in this survey were also checked against data acquired from the tidal observatory.

5.2 The Geotechnical Investigation

5.2.1 Overview

5.2.1.1 Purposes of the Geotechnical Investigation

The purpose of the geological survey is to collect data on the ground necessary for the implementation of the detailed design study for the Pasig-Marikina River Channel Improvement Project (Phase IV) and compile it as materials that can be used for the design.

5.2.1.2 Overview of Geotechnical Investigation

The geological survey conducted in this Detailed Design Study is divided into 1) boring survey, 2) soil test, 3) analysis of the survey test results and their summaries. These studies are explained as follows.

- Boring Survey: In the drilling survey, a boring excavation of approximately 20 m was made from the land on the left and right banks of the Marikina River and Pontoon on the river to confirm the stratum and collect samples for soil testing.
- Boring Survey: At the MCGS site, drilling was performed on the left and right banks of the Marikina River and in the center of the river to understand the geological conditions and to confirm the foundation rock for the construction of the weir.
- Boring Survey: At the Cainta river and Taytay river sites on the left bank side of the Manggahan floodway, drilling was performed as a foundation ground survey for the construction of floodgates, and the support layer of the structure was confirmed.
- Soil Test: The soil test was carried out using a soil sample collected by drilling excavation, and a physical test to determine the properties of the soil and a mechanical test to determine the mechanical properties were performed in the soil laboratory.
- Analysis of the survey test results and their summaries: In the analysis and compilation of the results of the geological survey test, a geological cross section required for detailed design of various structures is created from the results of the boring survey, and the soil test results are organized and compiled for each geology to be distributed. proposal was carried out.

5.2.2 Geotechnical Investigation Implementation Method

5.2.2.1 Geotechnical Investigation

(1) Overview of Geotechnical Investigation Implementation

The implementation of the geotechnical investigation is divided into the following seven stages.

1) Boring Survey: Confirmation of existing geotechnical investigation data

   Using the geological cross-section along the Marikina River created in Phase II, this includes the extraction of locations where the geological survey is not sufficient or where the geological structure is unknown.

2) Common: Site geological survey

   In addition to conducting on-site topographical and geological surveys, this includes the possibility of entry for boring surveys, the availability of work sites, and the study of alternative sites.

3) Boring Survey: Preparation of drilling survey plan

   Based on the results of the site survey, a drilling survey plan was created.

4) Soil Test: Study preparation of soil test implementation plan

   A soil test plan was created based on the results of the previous boring survey.

5) Boring Survey: Conduct a boring survey

   A boring survey was conducted with the consent of the landowner and related organizations, and the completion of the excavation was confirmed by constantly contacting the operation staff.

6) Soil Test: Conduct of soil test

   The soil samples collected by the boring were transported to the laboratory for necessary soil tests.

7) Analysis of the survey test results and their summaries: Analyze and summarize survey results

   The results of the boring survey, the results of the soil test, and the compilation were compiled, and a geological section map and report were prepared.

   The quantity of boring survey is shown in **Table 5.2.1**, and the quantity of soil test is shown in **Table 5.2.2**.

Table 5.2.1 Quantity of boring survey

LOCATION	BORING	DEPTH(m)
MARIKINA RIVER	32 HOLES	595.43
MCGS	7 HOLES	56.00
CAINTA / TAYTAY	5 HOLES	167.37
TOTAL	44 HOLES	818.80

Source: Study Team

Table 5.2.2 Quantity of soil test

SPT	UDS	Classification	Specific gravity	Moisture Content	Particle Size	Particle Size	Atterberg	Soil Unconfined	Rock Strength	Consolidation
ASTM D1586	-	ASTM D2487	ASTM D854	ASTM D2216	ASTM D422	ASTM E100	ASTM D4318	ASTM D2166	ASTM D2938	ASTM D2435
724	15	366	102	369	366	9	260	5	30	8

Source: Study Team

(2) Methodology of Survey

The survey was conducted as follows.

1) In order to propose the necessary soil modulus for the detailed design, boring surveys were conducted on the left and right banks of STA5 + 400 to STA13 + 350 along the Marikina River, and a geological cross and longitudinal sections were prepared for the revetment design and collected. Soil tests using the soil sample followed.

2) Boring surveys were conducted to clarify the geological distribution and geological structure of the planned MCGS site near STA6 + 000.

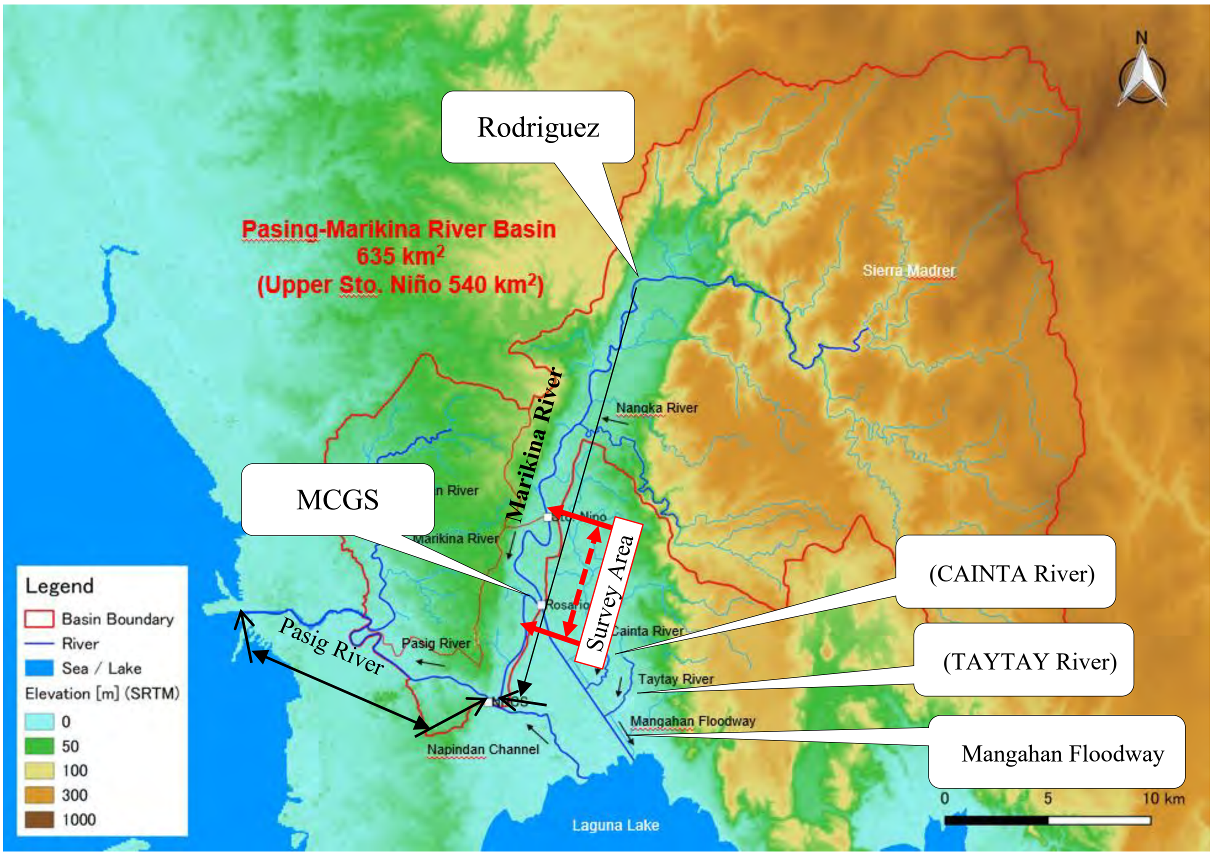
3) In order to clarify the geological distribution and structure of the planned floodgate and sluice gate with the surrounding area, and to prepare basic soil characteristics for detailed design of the structures, at the locations on the left bank of Manggahan Floodway STA4 + 550 (Cainta Floodgate) and STA6 + 100 (Taytay Sluice Gate), drilling surveys were also conducted.

5.2.3 Survey Results

(1) Topographic Conditions

The regional topographic map of the study area is shown in Figure 5.2.1.

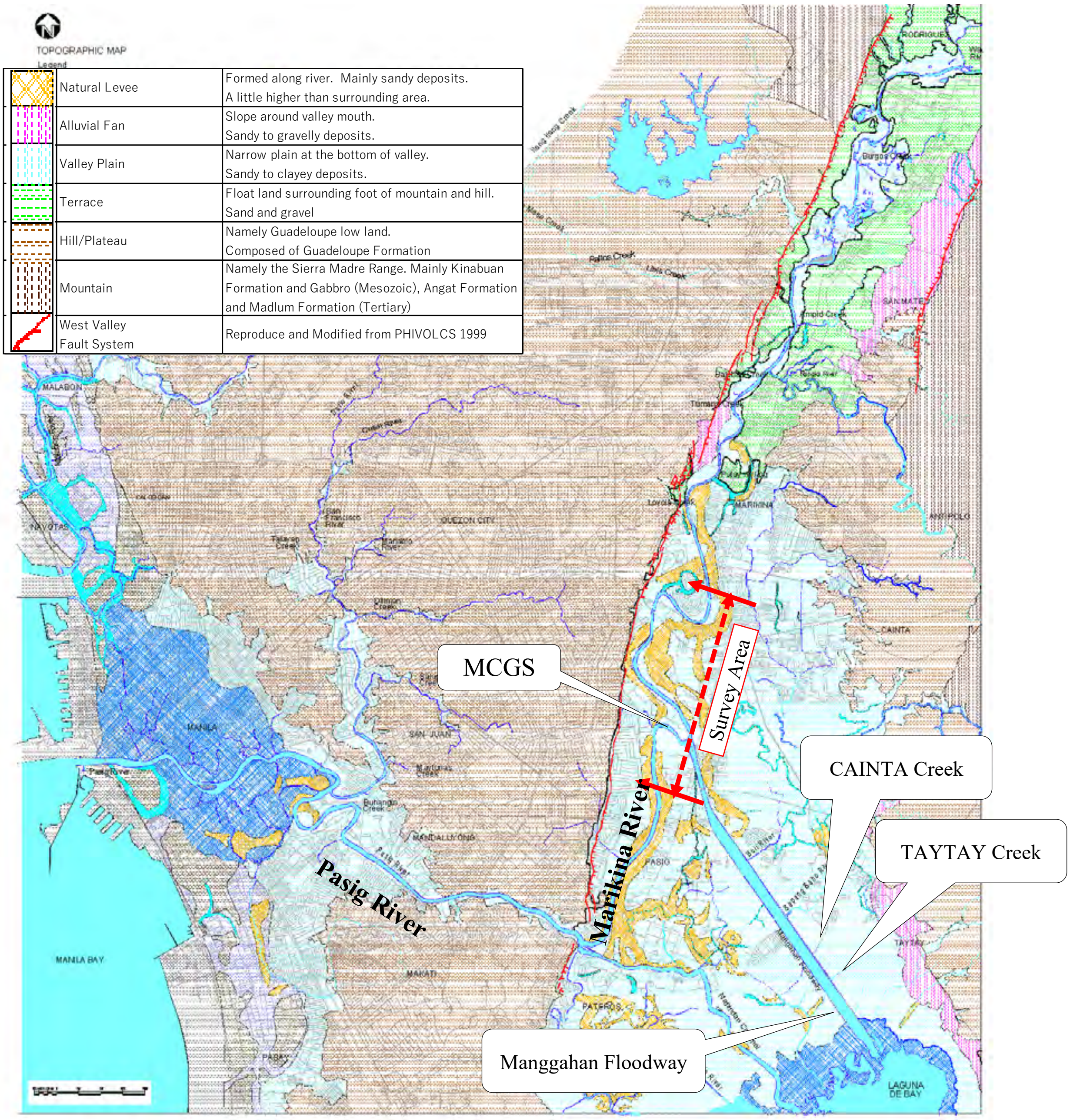
According to this, the Marikina River originated in the Sierra Madre Mountains at an altitude of about 1400 m in the northeast, flowed westward while merging many tributaries, then turned south in the town of Rodriguez, Rizal Province, and then joined the Napindan Channel. From this point, it flows west, changing its name to Pasig River. The Pasig River flows through Manila Metropolitan Area and pours into Manila Bay with a basin area of 635 km2.



Source: Final Report (Pasig-Marikina River Rainfall Analysis), Basic Information Collection Survey on Water Resources Development Plans in Metro Manila and Surrounding Areas in the Philippines (Water Balance Analysis, etc.), March 2013, JICA Study Team Addition

Figure 5.2.1 Topographic Map of the Study Area

Figure 5.2.2 shows the topographic classification map to give an overview of the topography of the study area.



Source : PHIVOLCS

Figure 5.2.2 Topographic Classification Map

According to this, the topography of the survey area is divided into a lowland where the Marikina River flows on the east side and a plateau on the west side with the West Valley Fault System distributed on the west side of the Marikina River.

The Marikina River runs from the north to the south along the fault on the west side of the lowland.

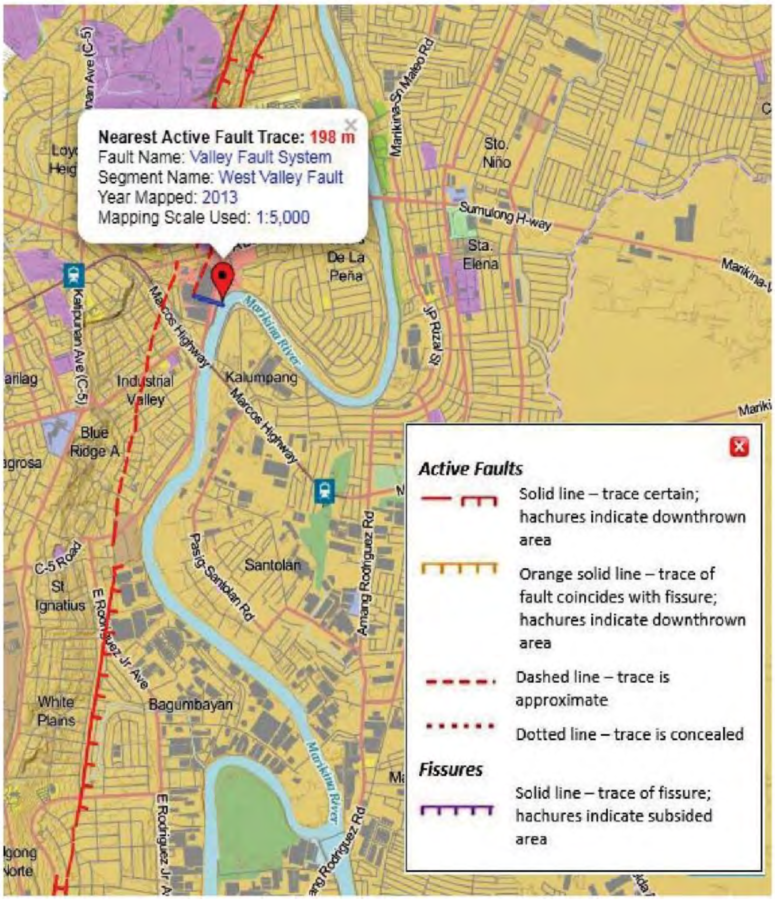
The west plateau has a gentle terrain at an altitude of about 20-50m, and the Guadalupe bedrock is distributed in some places.

The eastern lowland of Metro Manila along the left and right banks of the Marikina River is a lowland without large undulations, but according to **Figure 5.2.2** Topographic Classification Map, there is partly natural embankment areas in this lowland area. In the foothills of the eastern side, alluvial fans are also developed (see **Figure 5.2.3**).

Source : Study Team

Figure 5.2.3 (Photo) Lowland along the Marikina River

Along the right bank of the Marikina River, an active fault called the West Valley Fault extends continuously from north to south. The movement of the fault is a right-lateral strike-slip fault, whose activity is said to take place once every 200-400 years. However, according to PHIVOLCS 1997, the exact interval of huge earthquakes is uncertain. **Figure 5.2.4** shows the alignment map of the faults near the survey area.

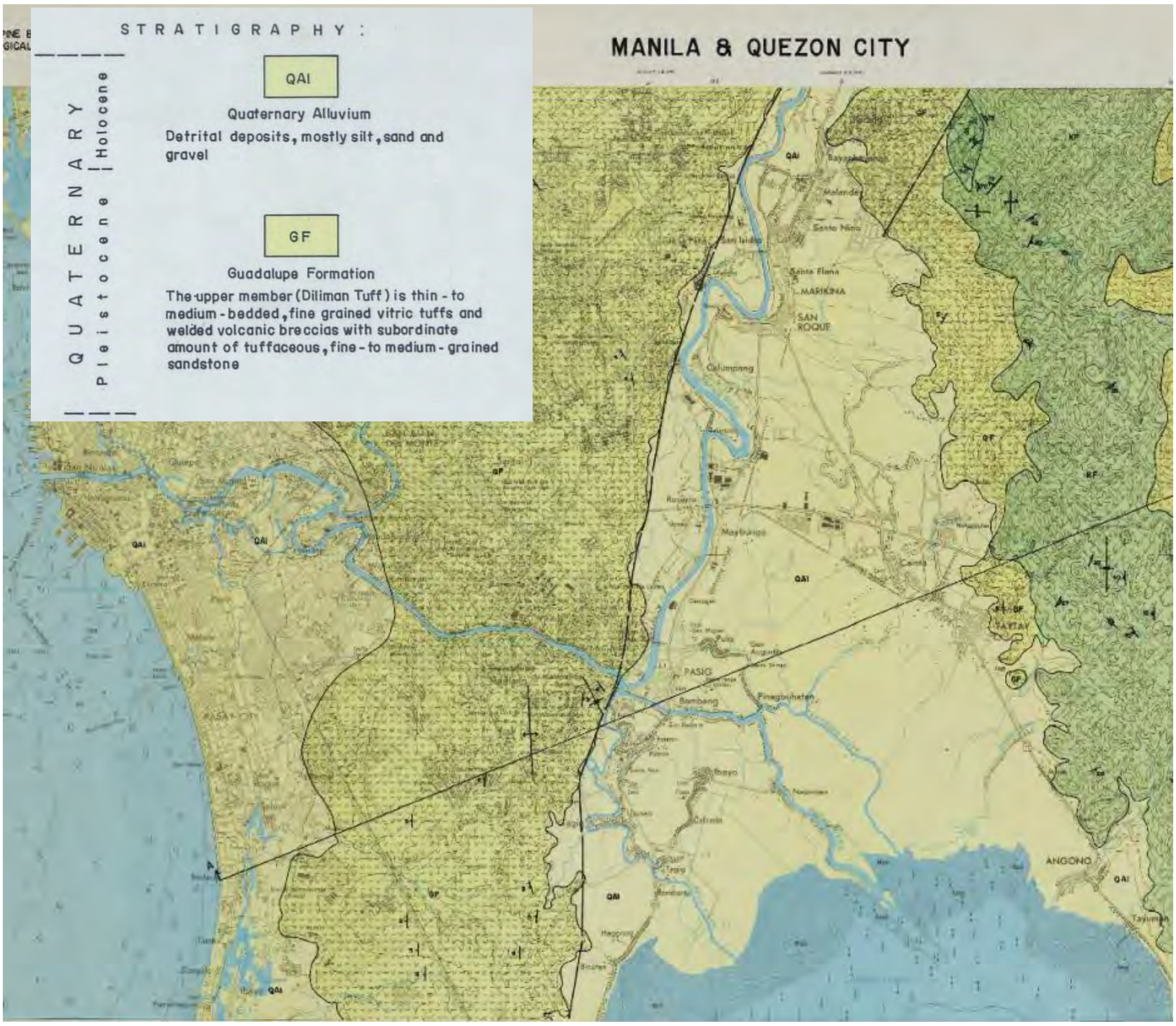


Source : PHIVOLCS Fault Finder 2018

Figure 5.2.4 West Valley Fault System

(2) Geology

Figure 5.2.5 shows the geological map (part) of the Manila & Quezon area.



Source : PHILIPPINE BUREAU OF MINES AND GEO-SCIENCES GEOLOGICAL SURVEY DIVISION

Figure 5.2.5 Geological Map of Manila and Quezon City

According to this, the geology of the Marikina River basin is distributed by the GF layer (Guadalupe Formation) in the western upland area, bordering on the West Valley Fault System that runs along the right bank of the Marikina River.

In the lowlands along the Marikina River on the eastern side of the fault, unconsolidated alluvial deposits of QAL (Quaternary Alluvium) are distributed on the surface.

The Guadalupe Formation consists of Pleistocene volcaniclastic rocks and is further divided into Alat conglomerate, Diliman tuff, and Antipolo basalt.

According to Geology and Mineral Resources of the Philippines, Guadalupe Formation is summarized as follows (see **Table 5.2.3**).

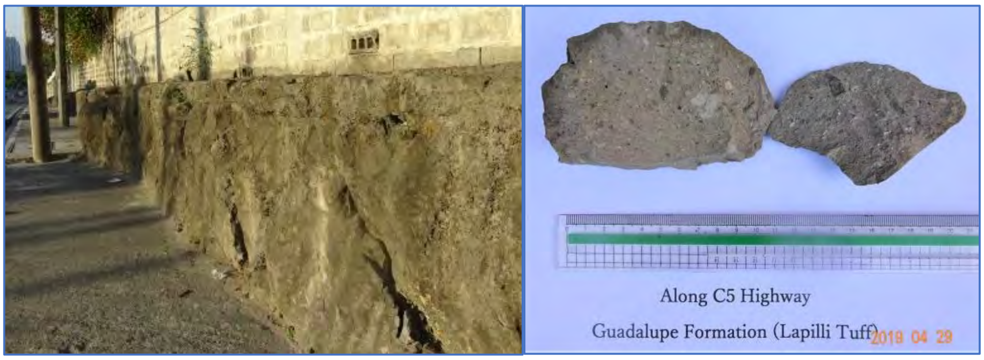
Table 5.2.3 General characteristics of Guadalupe Formation

Item	Detail
Lithology	Alat Conglomerate – conglomerate, silty mudstone, tuffaceous sandstone
	Diliman Tuff – vitric tuff, ignimbrite, volcanic breccia
Stratigraphic relations	Unconformable over Miocene rocks
Distribution	Quezon City, Pasig, Makati; southern Rizal; eastern Bulacan; southeastern Nueva Ecija
Age	Pleistocene
Thickness	1,500 – 2,000 m

Source: Geology and Mineral Resources of the Philippines

The Guadalupe Formation in the study area has almost horizontal stratified tuff, as shown in **Figure**

5.2.6 to **Figure 5.2.7**, along the road on the western plateau of the fault and on the riverbed of the Marikina River. According to the geological map, the Guadalupe Formation is inclined to the west by about 5 °.



Source : Study Team

Figure 5.2.6 (Photo) Guadalupe Formation along the C5 Highway (Lapilli tuff)

Source : Study Team

Figure 5.2.7 (Photo) Guadalupe Formation exposed along the Marikina River

In the lowlands along the Marikina River on the east side of the fault, unconsolidated alluvial deposits composed of clay, silt, sand, and gravel are distributed.

According to boring surveys, unconsolidated flood deposits consisting of clay, silt, sand, gravel, etc. are distributed over the alluvial deposits, and their thickness reaches 30 m. The Pliocene Guadalupe Formation has been identified in some areas below the dip.

Figure 5.2.8 shows the alluvial lowland along the Marikina River. Sand layer on the left bank of Marikina River. **Figure 5.2.9** shows a cohesive soil layer including waste on the left bank of the Marikina River.

Source : Study Team

Figure 5.2.8 (Photo) Sand layer on the left bank of the Marikina River

Source : Study Team

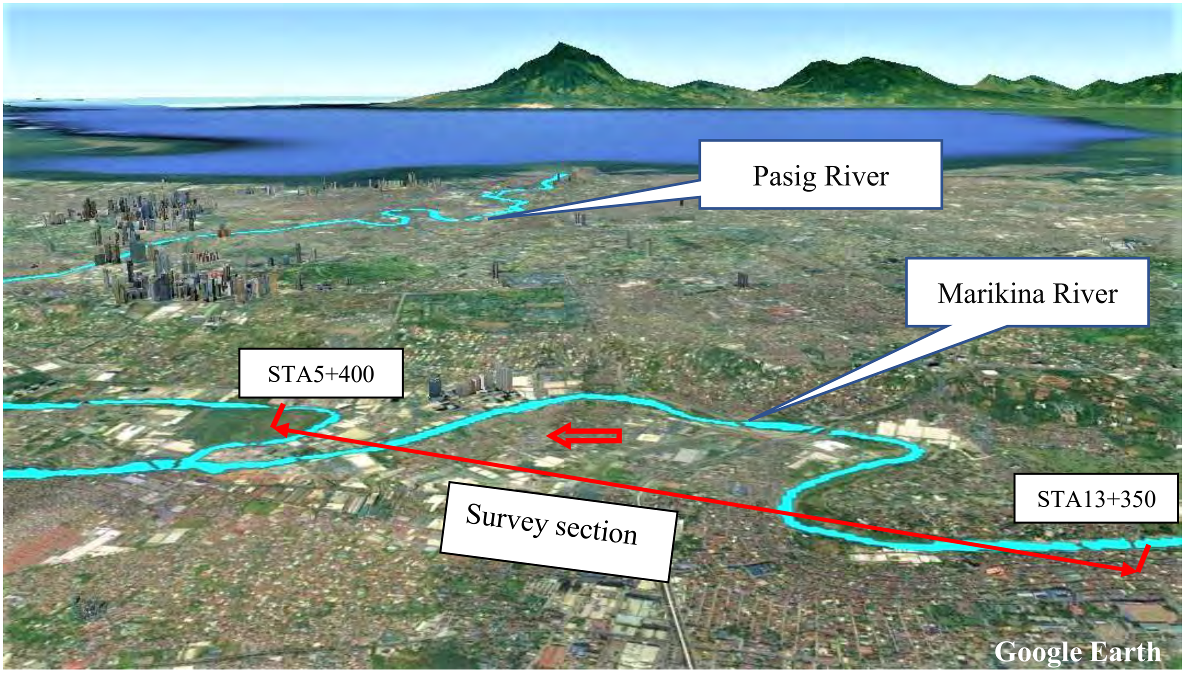
Figure 5.2.9 (Photo) Cohesive soil layer on the left bank of the Lower Marikina River

5.2.3.2 Boring Survey Results

(1) Boring Survey along the Pasig-Marikina river channel

The scope of this survey is about 8 km from STA5 + 400 to STA13 + 350 along the Marikina River. (See **Figure 5.2.11** for the location of the boring survey point)

Figure 5.2.10 shows the area of the survey.



Source : Study Team based on the Google Earth

Figure 5.2.10 Target Stretch of River Improvement and Boring Survey

The points of bore holes were determined so that the interval between each drilling survey including the bore holes executed in Phase I was approximately every 200 m.

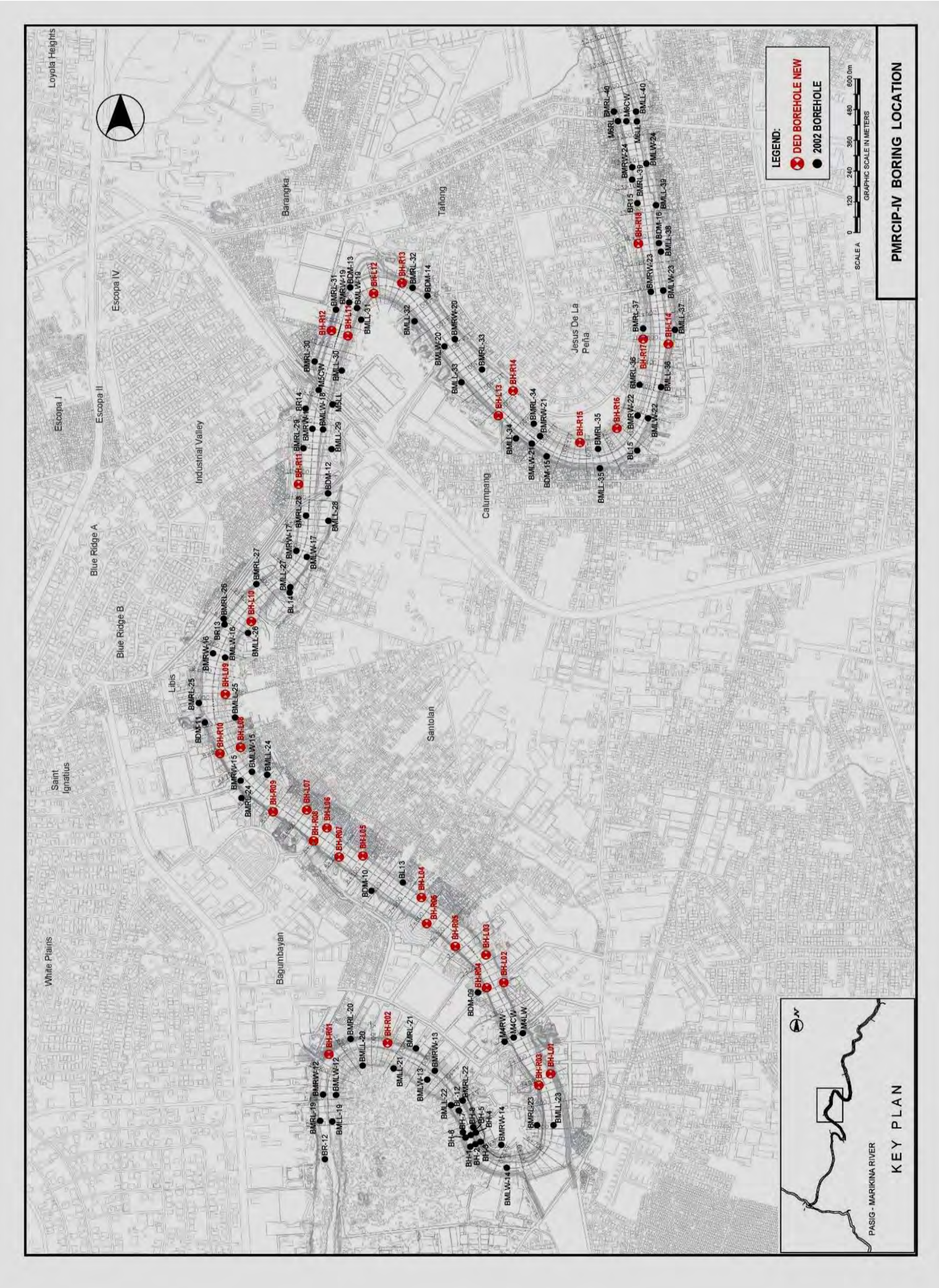
The borings were executed from off-shore or on-shore based on the site conditions As a result, a total of 32 borings were drilled along the Marikina River except for the MCGS Site, 18 holes on land and 14 on water respectively. The drilling depth was basically 20m, the longest was 30.45m and the shortest was 5.0m depending on rock conditions.

The drilling quantity is shown in **Table 5.2.4**. This list shows the total lengths of borings executed in this detailed design study including the boring for the MCGS, Cainta Floodgate and Taytay Sluicegate.

Table 5.2.4 List of Boring Survey and their Quantities

River	Bank	Hole No.	Depth	Water Leve	By land	Offshore	Station	Northing	Easting	Elev_DPWH	Location
Marikina	Left	DD-BH-L01	20.25			○	STA 7+000	1,614,982	509,582	11.060	Sta. Lucia
		DD-BH-L02	30.25	6.50	○		STA 7+400	1,615,338	509,405	17.187	Sta. Lucia
		DD-BH-L03	20.45			○	STA 7+525	1,615,446	509,338	9.810	Sta. Lucia
		DD-BH-L04	30.25			○	STA 7+850	1,615,670	509,093	10.431	Sta. Lucia
		DD-BH-L05	11.00			○	STA 8+120	1,615,833	508,872	10.932	Sta. Lucia
		DD-BH-L06	10.00			○	STA 8+300	1,615,942	508,736	11.185	Sta. Lucia
		DD-BH-L07	11.00	2.70	○		STA 8+400	1,616,012	508,660	14.308	Santolan
		DD-BH-L08	20.05	3.00	○		STA 8+780	1,616,256	508,409	14.466	Santolan
		DD-BH-L09	30.45	3.50	○		STA 9+020	1,616,464	508,351	14.539	Santolan
		DD-BH-L10	20.45	5.00	○		STA 9+355	1,616,748	508,450	14.928	Santolan
		DD-BH-L11	20.00	2.00	○		STA 10+560	1,617,863	508,815	13.514	Calumpang
		DD-BH-L12	20.35	3.00	○		STA 10+800	1,618,028	508,913	14.326	Calumpang
		DD-BH-L13	20.45	4.00	○		STA 11+520	1,617,551	509,385	15.741	Calumpang
		DD-BH-L14	24.45	3.10	○		STA 12+590	1,617,831	510,028	14.649	Calumpang
	Right	DD-BH-R01	5.00			○	STA 5+660	1,615,058	508,743	12.912	
		DD-BH-R02	6.00			○	STA 5+860	1,615,103	508,965	10.858	
		DD-BH-R03	20.45			○	STA 6+980	1,614,938	509,538	10.862	Bagumbayan
		DD-BH-R04	20.13			○	STA 7+410	1,615,319	509,340	9.224	Bagumbayan
		DD-BH-R05	17.30			○	STA 7+620	1,615,479	509,221	10.551	Bagumbayan
		DD-BH-R06	20.10			○	STA 7+770	1,615,568	509,113	10.465	Bagumbayan
		DD-BH-R07	9.00			○	STA 8+190	1,615,815	508,776	10.545	Bagumbayan
		DD-BH-R08	20.10	0.00	○		STA 8+305	1,615,891	508,686	9.558	Bagumbayan
		DD-BH-R09	8.00			○	STA 8+500	1,616,005	508,533	10.075	Bagumbayan
		DD-BH-R10	15.25			○	STA 8+795	1,616,232	508,331	9.651	Santolan
		DD-BH-R11	25.40	5.00	○		STA 9+960	1,617,283	508,629	15.757	Jesus dela Peña
		DD-BH-R12	20.10	2.00	○		STA 10+560	1,617,883	508,754	13.846	Jesus dela Peña
		DD-BH-R13	20.20	3.60	○		STA 10+910	1,618,069	509,020	12.392	Jesus dela Peña
		DD-BH-R14	20.45	3.20	○		STA 11+500	1,617,648	509,440	16.404	Jesus dela Peña
		DD-BH-R15	20.45	3.60	○		STA 11+870	1,617,446	509,693	16.526	Jesus dela Peña
		DD-BH-R16	20.45	5.50	○		STA 12+080	1,617,502	509,833	16.261	Jesus dela Peña
		DD-BH-R17	20.45	4.00	○		STA 12+500	1,617,850	509,932	16.774	Jesus dela Peña
		DD-BH-R18	17.20	3.50	○		STA 12+890	1,618,223	509,914	15.290	Jesus dela Peña
Subtotal		32hole	595.43		○						
MCGS	Left	DD-BH-G01	10.00	4.20	○		STA 5+980	1,614,995	509,057	14.689	MCGS
	Right	DD-BH-G02	6.00	-		○	STA 5+978	1,615,040	509,076	10.709	MCGS
	Left	DD-BH-G03	10.00	3.50	○		STA 6+005	1,614,981	509,076	15.237	MCGS
	Center	DD-BH-G04	10.00	2.00	○		STA 6+009	1,615,002	509,090	12.308	MCGS
	Right	DD-BH-G05	5.00	-		○	STA 6+013	1,615,018	509,104	10.916	MCGS
	Right	DD-BH-G06	5.00	-		○	STA 6+042	1,614,997	509,126	12.661	MCGS
	Left	DD-BH-G07	10.00	3.50	○		STA 6+036	1,614,967	509,095	14.393	MCGS
Subtotal		7hole	56.00								
Taytay	Right	DD-BH-T01	15.00			○	STA 0+029	1,609,438	512,598	10.385	Taytay
	Left	DD-BH-T02	40.32			○	STA 0+028	1,609,434	512,599	9.631	Taytay
Cainta	Right	DD-BH-C01	38.45			○	STA 0-014	1,610,733	511,713	11.680	Cainta
	Center	DD-BH-C02	35.45			○	STA 0+011	1,610,741	511,741	9.810	Cainta
	Left	DD-BH-C03	38.15			○	STA 0-007	1,610,726	511,731	9.256	Cainta
Subtotal		5hole	167.37								
Total		44hole	818.80								

Source: Study Team



Source : Study Team

Figure 5.2.11 Boring survey Points along the Marikina River

The boring survey results are summarized in the appendix including boring logs and the geological section of Marikina River which shows cross-sectional views of the left bank and right bank.

According to the boring survey results, the geology distributed along the Marikina River is largely divided into alluvial and diluvial.

Alluvium is further divided into alluvial clay layer (Ac) and alluvial sand layer (As), and diluvial layer is further divided into diluvial clay layer (Dc) and diluvial sand layer (Ds). As covered with these deposition layers, weathering rock layer (GFw) and fresh rock layer (GFf) are distributed as base. The GFw and GFf are diluvial deposit categorized as "Tuff".

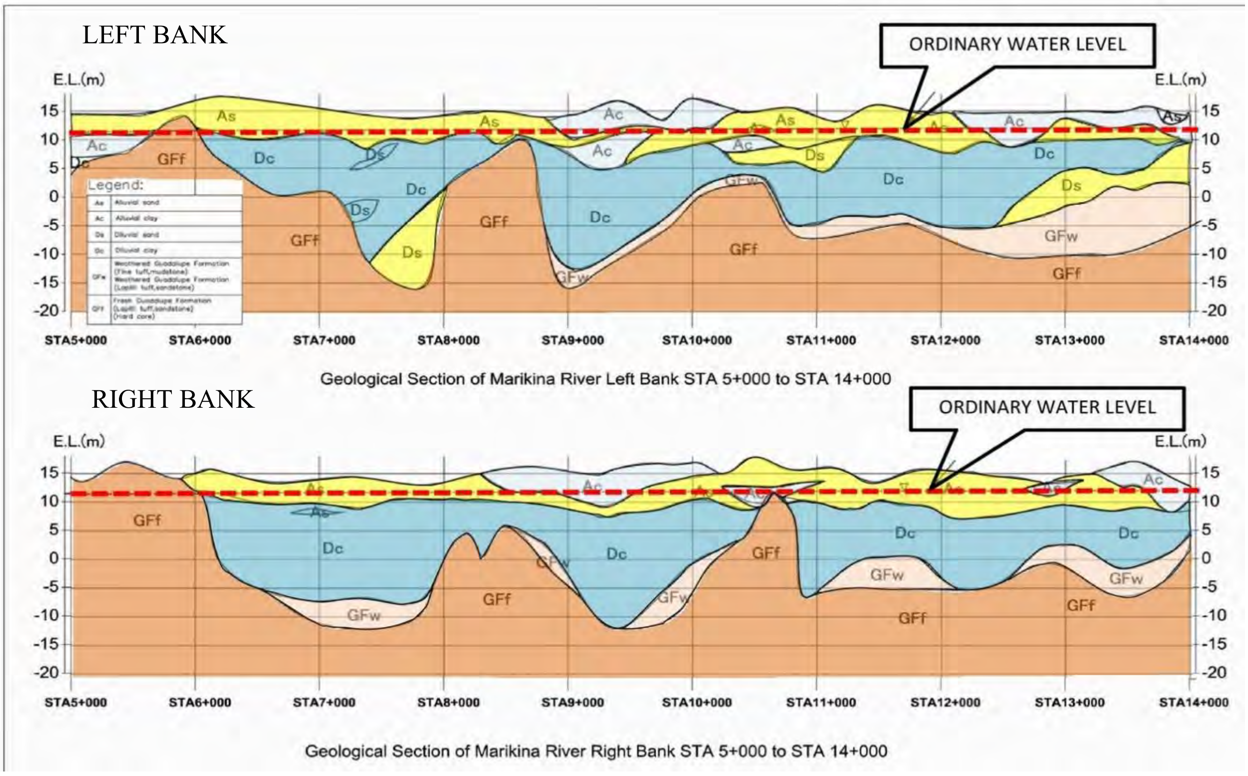
Table 5.2.5 summarizes the photographs and characteristics of the representative local strata.

Table 5.2.5 Strata and their characteristics observed along the Marikina River

Age		Geological Symbol / Photo	average N value	Thickness (m))	Characteristic
Quaternary	Holocene (Alluvium)	Ac 	6	5	Distributed in the middle to upper surface of the study area. Gray to dark brown clayey soil containing sand.
		As 	10	5-10	From the downstream end of the study area to the upstream end, it is partially covered by the Ac layer and distributed. Dark brown to brown, fine to coarse sand, the lower part consisting of gravel.
	Pleistocene (Diluvium)	Dc 	19	10-20	It fills the valley of the Marikina River and is widely distributed. Cohesive soil with brown, grey, dark blue-gray colors and high sand content. The surface layer is a little hard about 10m, but it is very hard at depths.
		Ds 	19	5-10	It is covered with Dc on the left bank of the Marikina River, and consists of dark blue - gray to brown coarse sand. The relative density is medium-dense.
		GFw 	50＜	5-10	Surface weathering of basement rock. The distribution is local and judged as gravel soil. Shows brown to gray.
		GFf 	50＜	10m<	Basement rocks in the study area. It consists of white to gray tuff, lapilli tuff, tuffaceous sandstone, conglomerate. Fresh parts are collected in the core.

Source : Study Team

Figure 5.2.12 shows a schematic cross-sectional view of the distribution of these strata.



Source : Study Team

Figure 5.2.12 Schematic Geological profile

The strata distributed below the left and right banks of the Marikina River show similar tendencies, such as the shape of basement rocks, the distribution of strata, and the thickness.

Alluvial layers, Ac and As layers, are distributed on the left and right banks of which the lower ends are generally higher than the riverbed elevation (about EL+10 m). Below the riverbed elevation, the Dc layer formed by diluvial clay is thickly distributed, with a maximum of 20 m or more. The Ds layer is partially being formed by diluvial sand. The characteristics of the stratum distribution are described below.

1) Characteristics of geological distribution

(i) The As layer is distributed at a thickness of about 5 to 10 m from the riverbed on the left and right banks of the Marikina River to the elevation of about EL+15 m. N value is about 10. The lower part may be accompanied by sand and gravel. Based on the topographic classification map shown in **Figure 5.2.2**, these are considered to correspond to natural levees formed in the Alluvial Age.

(ii) The Ac layer is distributed around the STA9 + 000 and STA13 + 000 with a thickness of about 5 m being covering As layer. The N value is as small as about 6. The Ac layer is thought to correspond to the oxbow lake sediment.

(iii) The Dc layer is widely and thickly distributed under the riverbed on both the left and right banks. The average thickness of the stratum is about 23m between ranges from EL+10m to EL-13m. The N value is 19 on average, but tends to be larger by depth, such as N=20 or less at 5-10 m from the surface of the stratum and N=20-30 at lower parts. The Dc layer is a sediment when the water level of the Marikina River was quite low during the Pleistocene.

(iv) The Ds layer is found on the left bank and is distributed between EL+0m and EL-15m at Sta.7 + 700. In addition, it is also found around Sta.12 + 500 to Sta.14 + 000. The layer thickness is about 5 to 15 m, and the N value is 19 on average. The Ds layer is considered to be a sand layer deposited during the flood in the Pleistocene.

(v) There are three (3) locations where the surface of Guadalupe Formation upthrusts near the riverbed elevation. On the other hand, the sections where the surface of Guadalupe Foundation is assumed to be distributed at EL+0 m, which is about 10 m deep from the riverbed are as follows;

Right bank: Sta.5 + 000 - 6 + 100, Sta.8 + 000 - 8 + 800, Sta.10 + 200 - 10 + 850, Sta. 13 + 950-14 + 200,
Left bank: Sta.5 + 000 - 7 + 150, Sta.8 + 000 - 8 + 750, Sta. 9 + 960 - 10 + 650

From the above, the origin of the Marikina River flowed through a valley of which the elevation of riverbed was about 20 m lower than the existing riverbed in the Pleistocene. At that time, the bedrock of the Guadalupe Formation was exposed on the riverbed and the right bank. It is also probable that the Marikina River was flowing into Laguna Lake while depositing clay and/or sand on the valley. It is further probable that the water in the Laguna Lake flowed out from the current Napindan Channel into Manila Bay through a valley along the Pasig River. Although the details are unclear (when it started), the sedimentation of the Marikina River progressed and the flow into the Laguna Lake was disturbed by sedimented deposits, the Marikina River flowed toward west over the saddle of the Guadalupe Formation, which was located between the current Circulo Verde and the Rosario Weir. It is thought that this saddle was eroded year by year and the current lower Marikina River was formed. These assumptions are assumed by the fact that the Guadalupe Formation has been exposed on only the riverbed from the vicinity of Circulo Verde (Sta.5+800 ~ Sta.6+500) of the Marikina River.

2) Water Levels in Boreholes

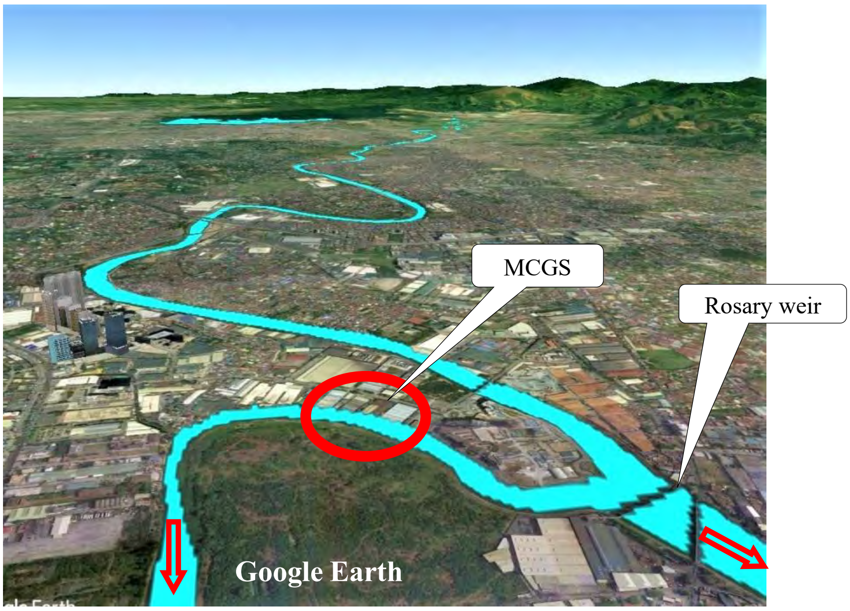
The borehole water levels were measured at 22 holes drilled on land. The borehole water levels are shown in **Table 5.2.4**.

The water levels in each borehole are almost similar to the adjacent river water levels.

From those observation results, it is assumed that there is no confined groundwater layer around the Project Site.

(2) Boring Survey for the MCGS

Figure 5.2.13 shows the site condition around the MCGS of which the location is near STA6 + 000 of the Marikina River at about 0.5 km downstream of the Rosario Weir.



Source : Study Team based on the Google Earth

Figure 5.2.13 Site of the MCGS

At the periphery of the MCGS construction site, some industrial factories are located on the right bank, on the other hand, vacant lots covered with shrubs and roves are spreading on the left bank.

The water depth of the Marikina River is shallow near this area, and the basement rock Guadalupe is exposed in part of the riverbed.

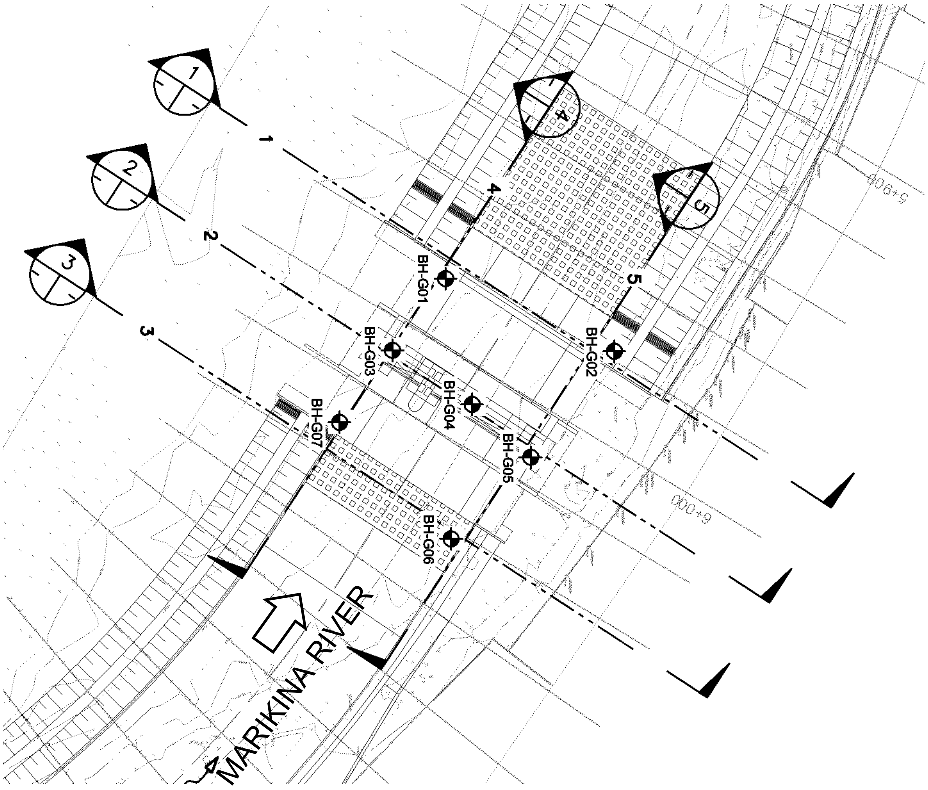
Seven (7) borings conducted at the planned MCGS site. The list of boring quantities is shown in

Table 5.2.6 and the location of boring survey is shown in **Figure 5.2.14**.

Table 5.2.6 List of Boring Survey and their Quantities for the MCGS

River	Bank	Hole No.	Depth	Water Leve	By land	Offshore	Station	Northing	Easting	Elev_DPWH	Location
MCGS	Left	DD-BH-G01	10.00	4.20	○		STA 5+980	1,614,995	509,057	14.689	MCGS
	Right	DD-BH-G02	6.00	-		○	STA 5+978	1,615,040	509,076	10.709	MCGS
	Left	DD-BH-G03	10.00	3.50	○		STA 6+005	1,614,981	509,076	15.237	MCGS
	Center	DD-BH-G04	10.00	2.00	○		STA 6+009	1,615,002	509,090	12.308	MCGS
	Right	DD-BH-G05	5.00	-		○	STA 6+013	1,615,018	509,104	10.916	MCGS
	Right	DD-BH-G06	5.00	-		○	STA 6+042	1,614,997	509,126	12.661	MCGS
	Left	DD-BH-G07	10.00	3.50	○		STA 6+036	1,614,967	509,095	14.393	MCGS
Subtotal		7hole	56.00								

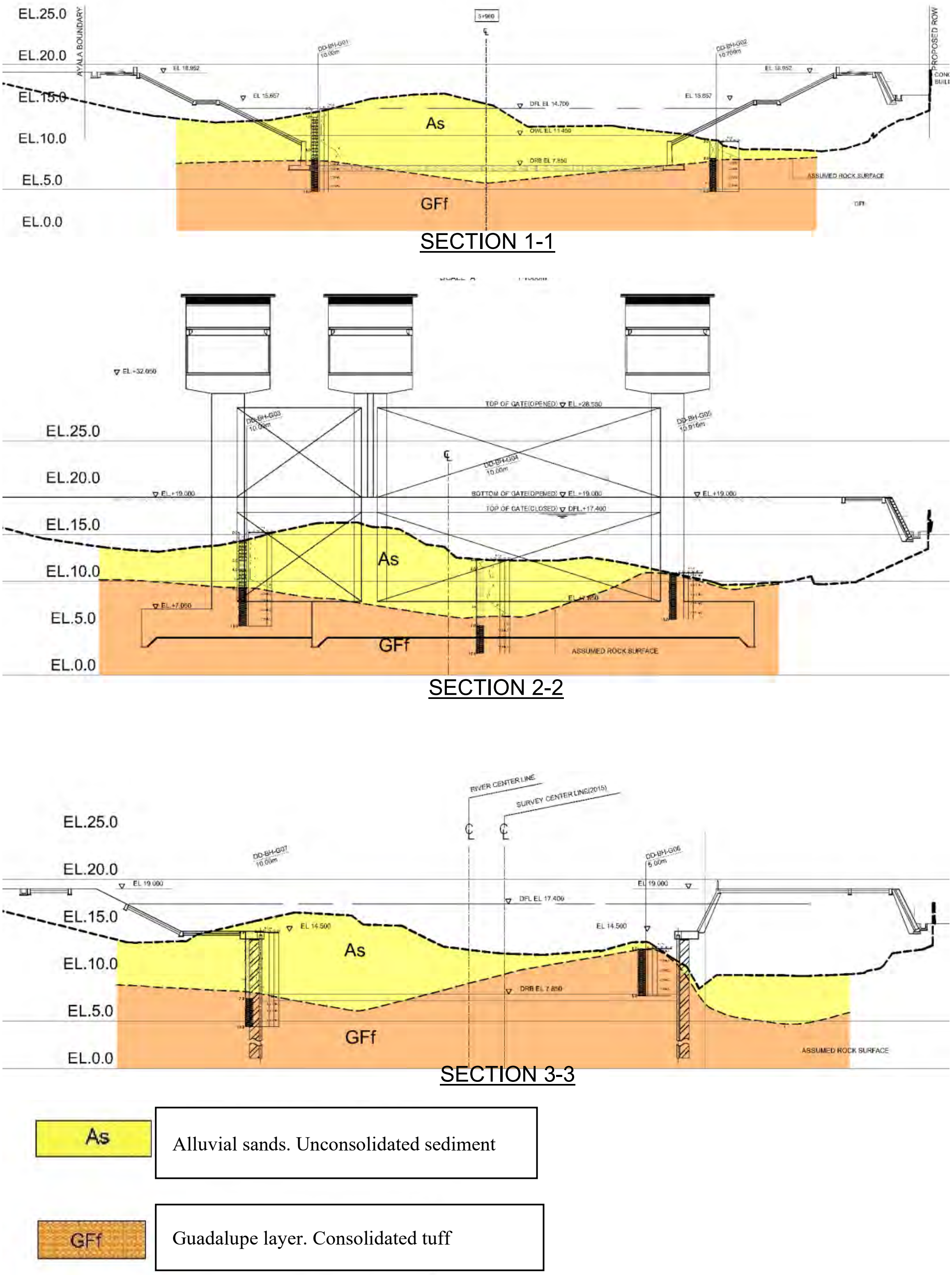
Source: Study Team



Source : Study Team

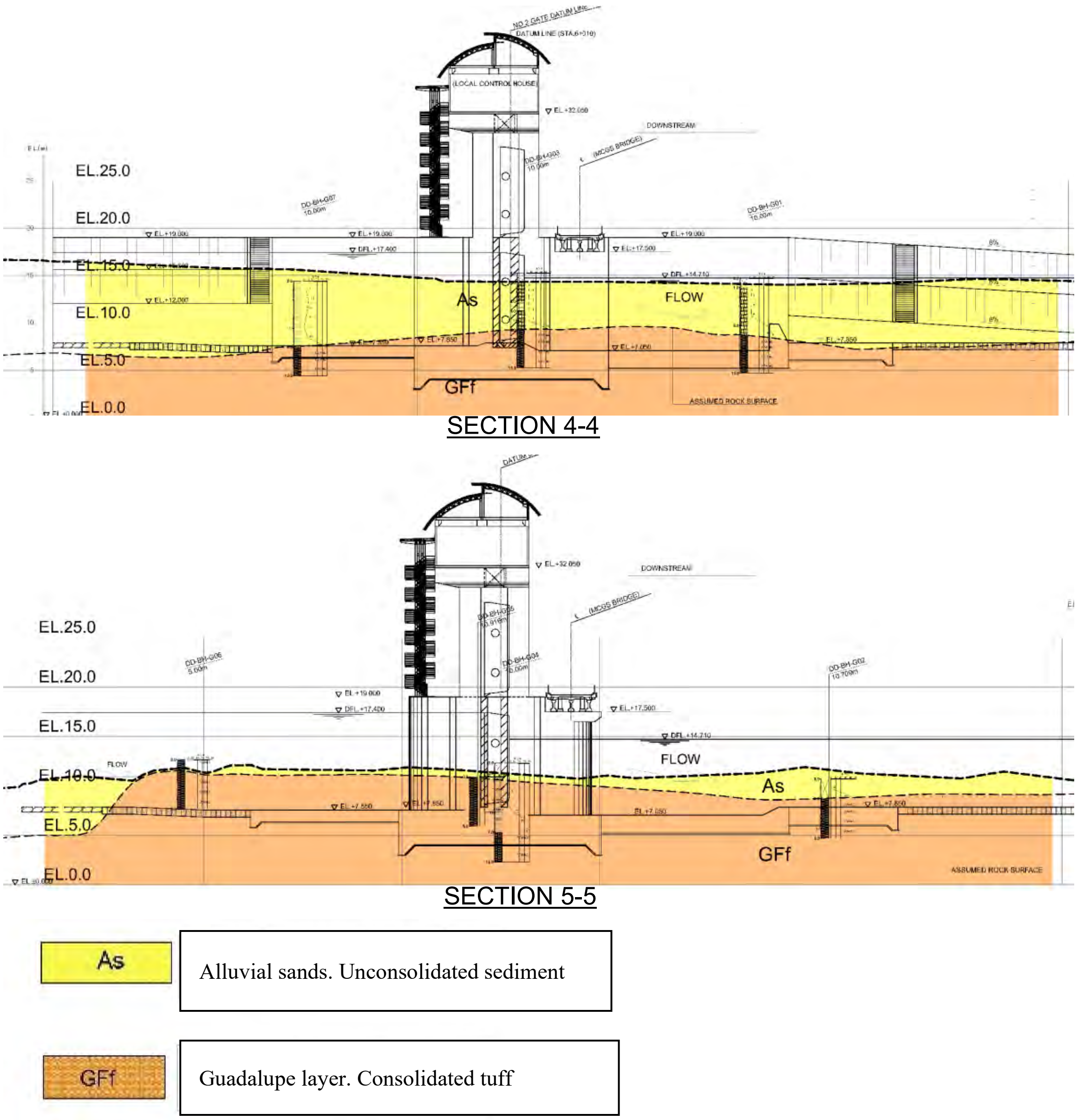
Figure 5.2.14 Location of Boreholes surveyed for the MCGS

The results of the boring survey are summarized in the appendix as a boring log, and a geological cross section is shown in **Figure 5.2.15** and **Figure 5.2.16**.



See **Figure 5.2.14** *for the Number of Each Section*
Source : Study Team

Figure 5.2.15 Geological Condition around the MCGS



See **Figure 5.2.14** *for the Number of Each Section*
Source : Study Team

Figure 5.2.16 Geological Condition around the MCGS

The following is a description of the geological features of the MCGS construction site.

(i) In the surface layer on the left bank side of the riverbed, unconsolidated alluvial sand, gravel and cohesive soil layers with a thickness of several meters are distributed.
(ii) The basement rock, Guadalupe, is located at the shallow depth of the riverbed just below the alluvial deposit and is partially exposed.
(iii) The Guadalupe Formation consists of soft rocks such as tuff and tuffaceous sandstone, mainly lapilli tuff. The Guadalupe Formation is a bedrock that is fresh from the surface layer with almost no weathered layer on the bedrock layer.
(iv) The Guadalupe Formation is a bedrock that is fresh from the surface layer with almost no weathered layer on the bedrock layer.
(v) Guadalupe Formation is almost horizontal, the bedrock is massive and almost no cracks are seen.
(vi) The distribution of the Guadalupe Formation tends to be deeper upstream of the planned MCGS site.

Figure 5.2.17 shows a photograph of a boring core sampled at the MCGS survey site.

Source : Study Team

Figure 5.2.17 (Photo) 0-5m core of BH-G-05 hole (Red part is tuff gray part is lapilli tuff)

At the time of the field survey, the riverbed was being excavated. Photographs of riverbed conditions and bedrock at this time are shown in **Figure 5.2.18** and **Figure 5.2.19**.

Source : Study Team

Figure 5.2.18 (Photo) Riverbed excavation

Source : Study Team

Figure 5.2.19 (Photo) Excavated rock composed of fresh tuff

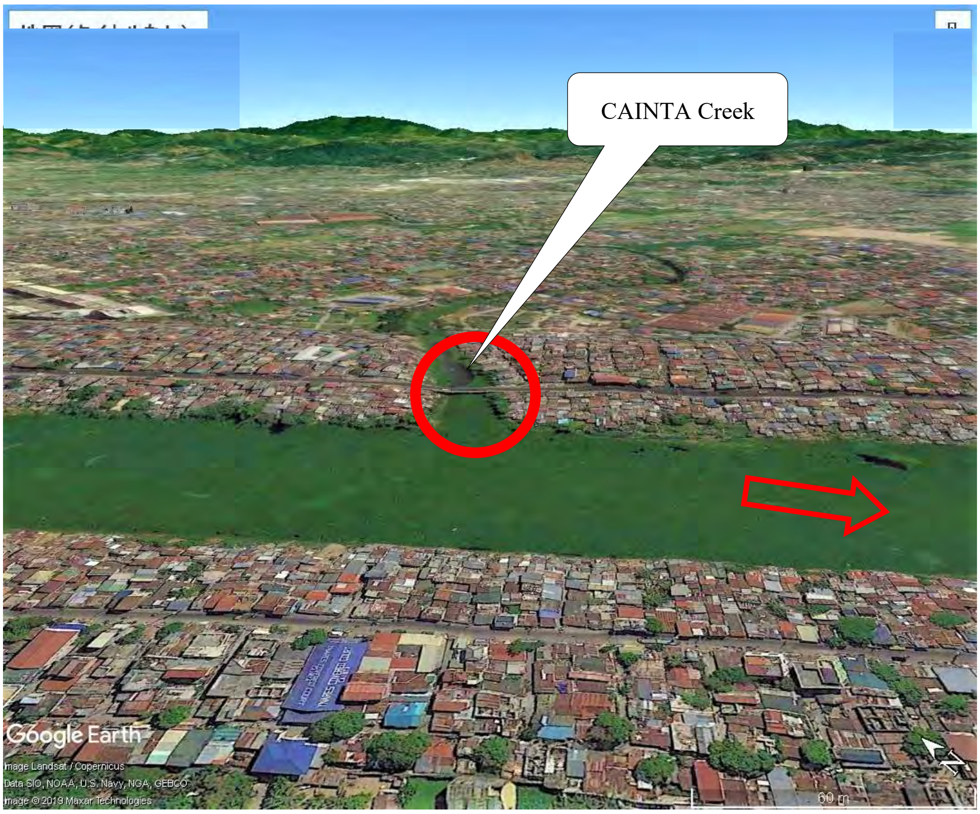
5.2.3.3 Cainta / Taytay Flood Gate boring survey

Downstream of Manggahan Floodway, Cainta Creek on the left bank of Sta.4 + 550 and Taytay Creek on the left bank of STA6 + 100, it is planned to install backflow prevention gates.

Figure 5.2.20 shows the situation at the junction of the Cainta River and **Figure 5.2.21** shows the situation at the junction of the Taytay River.

The Cainta Creek flows from east to west and flows into the Manggahan Floodway.

In the lower Cainta creek, a bridge is built in front of the spillway. The villages are densely located around the road along the spillway and around the slope of the spillway. Furthermore, many settlements have developed in the middle basin of the Cainta Creek, but the ground is several meters below the road.

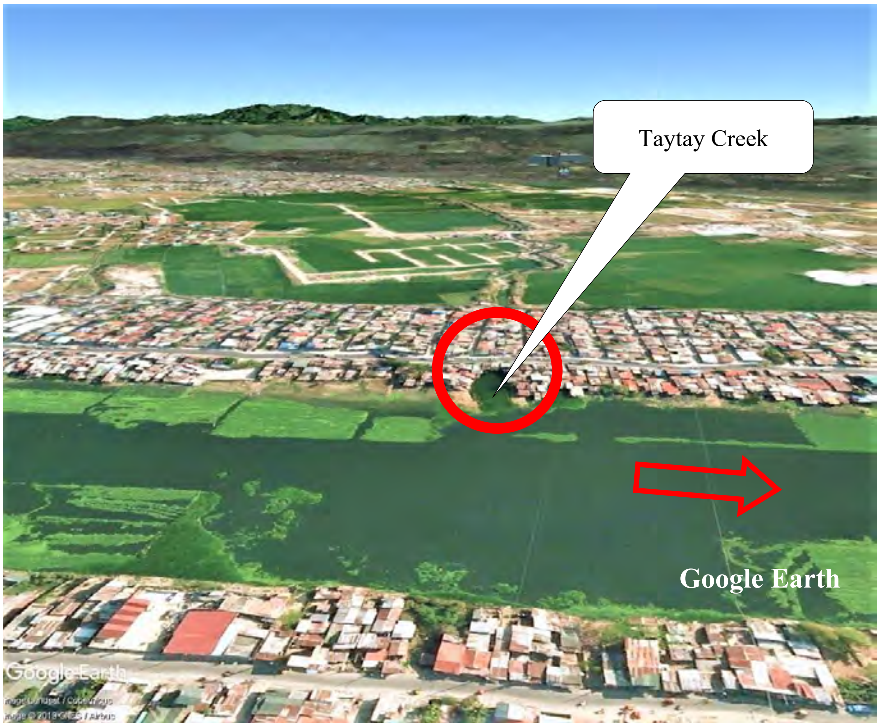


Source : Study Team based on Google Earth

Figure 5.2.20 Current Situation around the Cainta Floodgate Proposed

The Taytay Creek also flows from east to west and flows into the Manggahan floodway.

The downstream end of the Taytay Creek is a culvert about 150m long. In those areas, there are densely populated villages along the dike road and the slope / berm of the floodway. The areas of middle to upper reaches of the Taytay Creek are under development. The elevation of those developed areas is lower than dike road at 5~10 meters.



Source : Study Team based on Google Earth

Figure 5.2.21 Current Situation around the Taytay Sluicegate Proposed

A total of five (5) drilling surveys (three (3) at Cainta and two (2) at Taytay) were conducted.

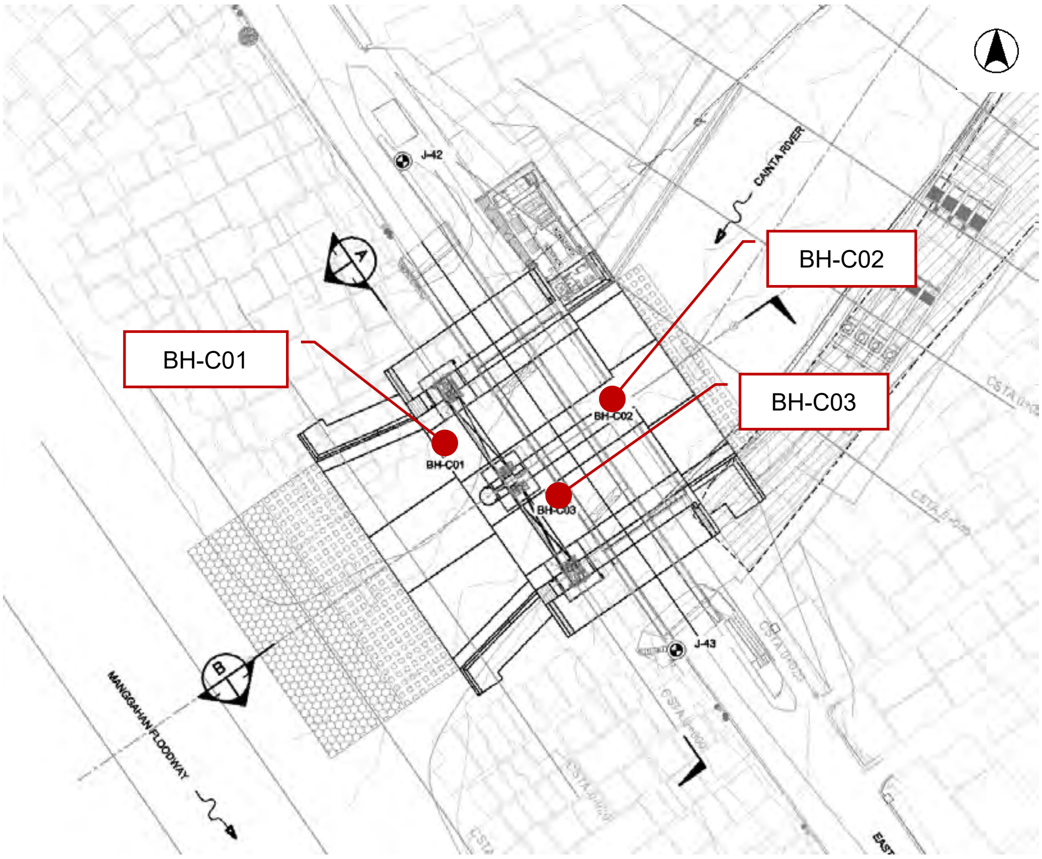
In all cases, there was no open space for boring machines installed on land, therefore boring machines were carried on barges.

The boring survey quantity is shown in **Table 5.2.7**, and the survey location is shown in **Figure 5.2.22** and **Figure 5.2.23**.

Table 5.2.7 List of boring survey quantities (Cainta / Taytay)

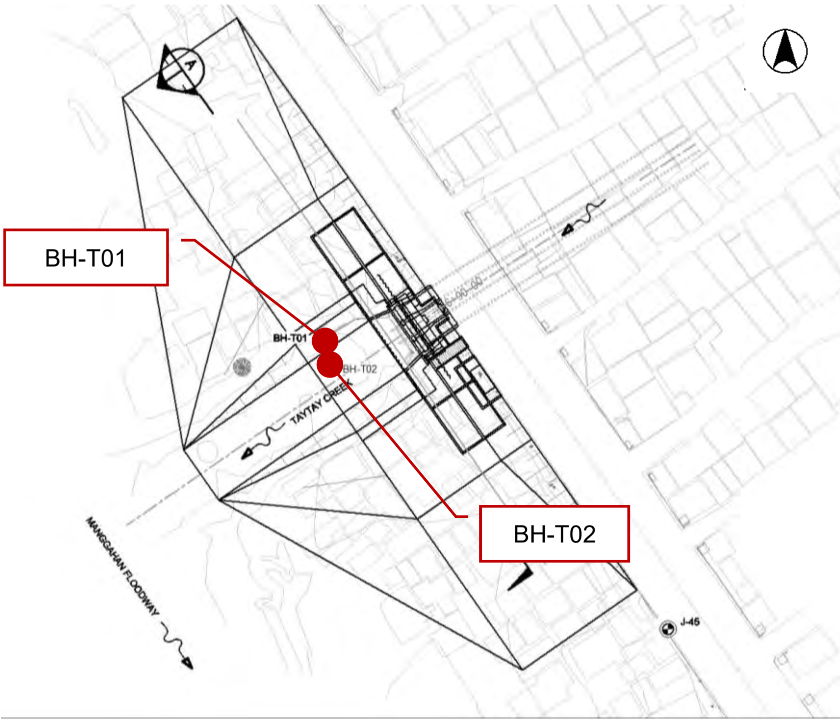
River	Bank (L / R)	Hole No.	Depth	Station	Northing	Easting	Elev_DPWH	Location
Taytay	Right	DD-BH-T01	15.00	STA 0+029	1,609,438	512,598	10.385	Taytay
	Left	DD-BH-T02	40.32	STA 0+028	1,609,434	512,599	9.631	Taytay
Cainta	Right	DD-BH-C01	38.45	STA 0-014	1,610,733	511,713	11.680	Cainta
	Center	DD-BH-C02	35.45	STA 0+011	1,610,741	511,741	9.810	Cainta
	Left	DD-BH-C03	38.15	STA 0-007	1,610,726	511,731	9.256	Cainta
Total	-	-	**167.37**	-	-	-	-	-

Source: Study Team



Source : Study Team

Figure 5.2.22 Location Map of Boreholes for the Cainta Floodgate



Source : Study Team

Figure 5.2.23 Location Map of Boreholes for the Taytay Sluicegate

The drilling depth at the Taytay site was 15-40 m to confirm the support layer of the sluice facility.

At Cainta and Taytay, the survey was for the support layer of the weir. For this reason, the geological stratification was re-divided into four layers of alluvium (As2 layer, Ac2 layer, Ac1 layer, As1 layer) and two layers of diluvial layer (Dc1 layer, Ds1 layer).

At these points, the Guadalupe Formation, the bedrock of the study area, could not be identified.

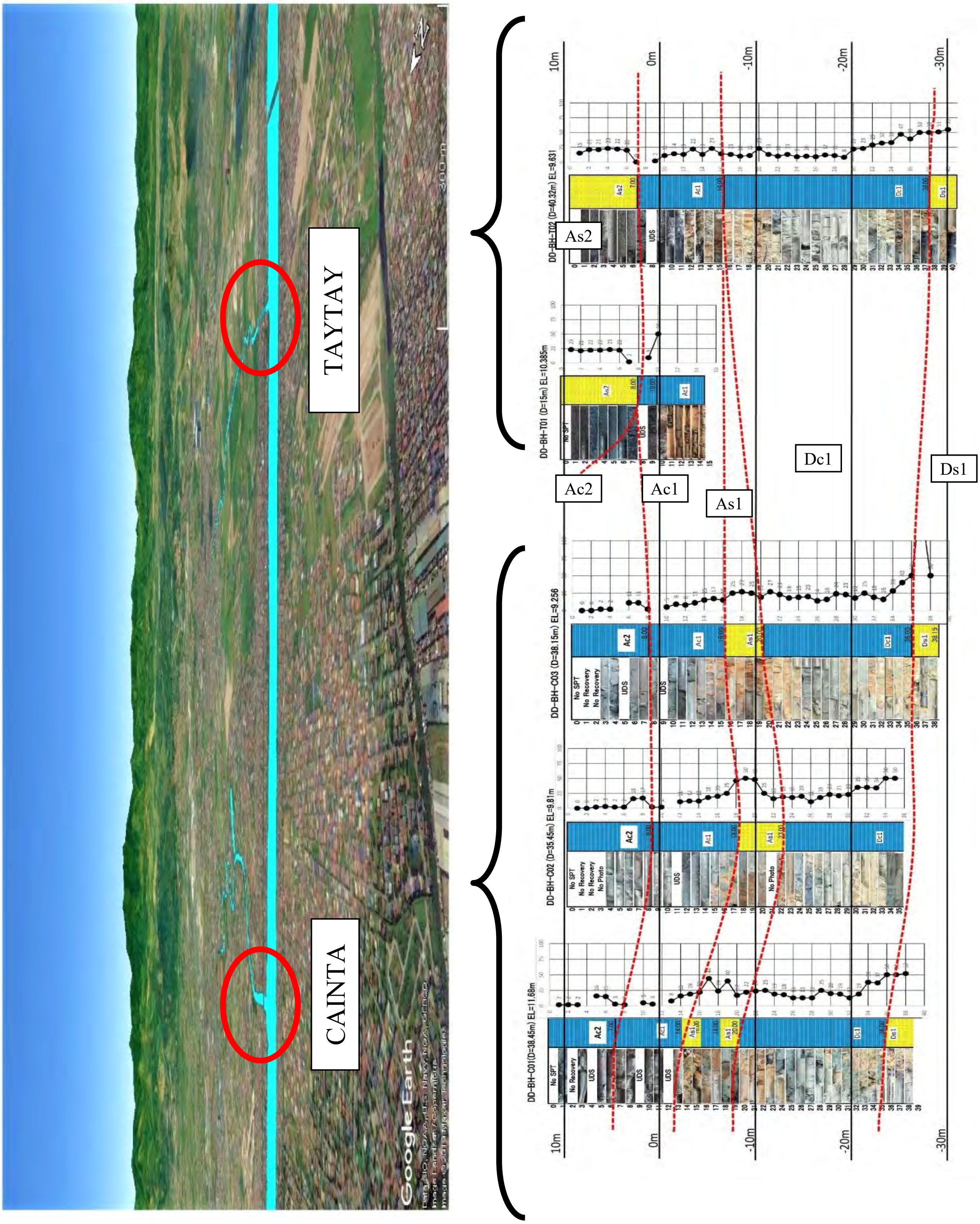
Table 5.2.8 summarizes the photos and characteristics of representative strata.

Table 5.2.8 List of boring survey quantities (Cainta / Taytay)

Age		Geological Symbol / Photo	average N value	Thickness (m)	Characteristic
Cainta					
Quaternary	Holocene (Alluvium)	**Ac2** 	6	7m	Clay layer, a soft clay distributed on the surface layer at the CAINTA point. N value increases with sand at the bottom
		Ac1 	13	10m	Clay layer. Distribution at both CAINTA and TAYTAY sites, N value increases at the bottom.
		As1 	29	5m	Sand layer. Confirmed at CAINTA location, oxidation progressed and turned brown. Judged as a permeable stratum
	Pleistocene (Diluvium)	**Dc1** 	22	16m	Clay layer. It is distributed at a depth of 16-20m or less, and the N value is stable at 15-20, but sand is mixed in the lower part, and the N value is as large as 30 or more.
		Ds1 	50	More than 3m	Sand layer. Confirmed at CAINTA, TAYTAY at a depth of 36-38m or less. It is a very dense sand layer. The thickness of this layer is unknown, indicating a light brown color.
Taytay					
Quaternary	Holocene (Alluvium)	**As2** 	20	6m	Sand layer distributed on the surface layer at TAYTAY point, not distributed at CAINTA point on the upstream side. Often contains shell shards and has a relative density of medium.
		Ac2 	6	7m	Clay layer, a soft clay distributed on the surface layer at the CAINTA point. N value increases with sand at the bottom
		Ac1 	13	10m	Clay layer. Distribution at both CAINTA and TAYTAY sites, N value increases at the bottom. At TAYTAY, at a depth of 10 m, N value exceeded 50, coring occurred, but it could not be confirmed by adjacent boring. Local phenomenon and judgment.
	Pleistocene (Diluvium)	**Dc1** 	22	16m	Clay layer. It is distributed at a depth of 16-20m or less, and the N value is stable at 15-20, but sand is mixed in the lower part, and the N value is as large as 30 or more.
		Ds1 	50	More than 3m	Sand layer. Confirmed at CAINTA, TAYTAY at a depth of 36-38m or less. It is a very dense sand layer. The thickness of this layer is unknown, indicating a light brown color.

Source: Study Team

In addition, the results of the boring survey are organized as geological log in the appendix. **Figure 5.2.24** shows the geological cross section connecting Cainta and Taytay site.



Source : Study Team

Figure 5.2.24 Geological Section of Cainta / Taytay

The survey points of the Cainta and Taytay creeks are about 1.5 km apart. Both are considered to be the places where sediment from Marikina River was deposited, the continuity of the stratum is good, and the N value shows a similar tendency.

The characteristics of the stratum distribution are described below.

1) Geological characteristics of Cainta and Taytay is divided into an upper portion of alluvium and bottom of diluvial. Each layer thickness is about 20m.

2) In the deepest part, the diluvial sand layer (Ds1) distributed at EL-25m to EL-30m at DPWH Datum Elevation system is continuously covered from Cainta to Taytay.

3) Diluvial clay layer (Dc1) of thickness of about 10-20m from EL-10m to EL-25m is distributed.

4) An alluvial sand layer (As1) with a thickness of about 5 m can be confirmed EL-2 m to EL- 12 m. This layer has disappeared near Taytay.

5) In the alluvium, an alluvial clay layer (Ac1) with a thickness of about 8 m is continuous around EL+0m. In the Cainta area, an alluvial clay layer (Ac2) with a thickness of about 10 m is distributed. These boundaries are distinguished by the depth of change of the N value.

6) At the DD-BH-T01 hole at the Taytay area, solidified mudstone with an N value of 50 or more was confirmed at a depth of 10 m, and coring was performed at 5 m, but the same depth was used at an adjacent boring hole 4.6 m away. Since the N value was 11-23 in the stratum, excavation was performed to a depth of 40m to confirm the N value of 50 or more to ensure the confirmation of the support layer.

7) A sand layer (As2) with a thickness of about 8m is distributed on the surface of the Taytay area. Since this layer is not found in the upstream Cainta area, it is considered that it was not transported from the Marikina River, but rather from the Taytay Creek.

8) In the Cainta / Taytay area, the N value of each layer tends to increase as the depth increases. Furthermore, there is a tendency for the N value to increase in places including the sand layer.

5.2.3.4 Results of Soil Tests

(1) The Pasig-Marikina River Channel and MCGS

1) Results of Soil Tests

The following laboratory soil tests were conducted to investigate the physical and mechanical properties of the revetment foundation ground. The samples used for the test were a perturbed sample collected during a standard penetration test and an undisturbed sample collected by a shelby tube. The test items and quantity show the soil test results summarized in **Table 5.2.9** and **Table 5.2.10**.

Table 5.2.9 List of soil test quantities (Marikina River channel Improvement Study / MCGS)

Hole No.	Depth (m)	SPT ASTM D1586	UDS		Classification ASTM D2487	Specific gravity ASTM D854	Moisture Content ASTM D2216	Particle Size ASTM D422	Particle Size ASTM E100	Attenberg ASTM D4318	Soil Unconfined ASTM D2166	Rock Strength ASTM D2938	Consolidation ASTM D2435
DD-BH-L01	20.25	20	1	-	12	3	11	12	0	7	0	0	1
DD-BH-L02	30.25	30	0	-	15	3	10	15	0	6	0	0	0
DD-BH-L03	20.45	19	1	4.00-4.45	11	3	11	11	0	5	0	0	0
DD-BH-L04	30.25	30	0	-	16	3	16	16	0	8	0	0	0
DD-BH-L05	11.00	7	0	-	8	2	8	8	0	6	0	2	0
DD-BH-L06	10.00	5	0	-	6	2	6	6	0	6	0	2	0
DD-BH-L07	11.00	7	0	-	4	2	3	4	0	2	0	2	0
DD-BH-L08	20.05	20	0	-	9	3	9	9	0	7	0	0	0
DD-BH-L09	30.45	30	0	-	15	3	15	15	0	13	0	0	0
DD-BH-L10	20.45	19	1	5-5.45	10	3	10	10	0	11	0	0	1
DD-BH-L11	20.00	11	0	-	10	4	11	10	0	7	0	1	0
DD-BH-L12	20.35	19	0	-	8	3	8	8	0	6	0	0	0
DD-BH-L13	20.45	19	1	8-8.45	10	3	8	10	0	7	0	0	1
DD-BH-L14	24.45	24	0	-	12	3	12	12	0	10	0	0	0
DD-BH-R01	5.00	0	0	-	0	0	0	0	0	0	0	2	0
DD-BH-R02	6.00	2	0	-	0	0	0	0	0	0	0	2	0
DD-BH-R03	20.45	19	1	3-3.45	12	6	11	12	0	9	1	0	1
DD-BH-R04	20.13	20	0	-	11	3	11	11	0	9	0	0	0
DD-BH-R05	17.30	16	1	2-2.45	9	3	8	9	0	6	1	0	0
DD-BH-R06	20.10	20	0	-	11	3	11	11	0	8	0	0	0
DD-BH-R07	9.00	6	0	-	6	6	6	6	0	6	0	3	0
DD-BH-R08	20.10	20	0	-	11	3	11	11	0	4	0	0	0
DD-BH-R09	8.00	4	0	-	5	6	5	5	0	4	0	3	0
DD-BH-R10	15.25	15	0	-	9	3	19	9	0	4	0	0	0
DD-BH-R11	25.40	25	0	-	12	3	15	12	0	8	0	0	0
DD-BH-R12	20.10	20	0	-	8	3	8	8	0	3	0	0	0
DD-BH-R13	20.20	19	1	15-15.45	11	3	10	11	0	7	0	0	1
DD-BH-R14	20.45	20	0	-	10	3	10	10	0	7	0	0	0
DD-BH-R15	20.45	20	0	-	10	3	10	10	0	5	0	0	0
DD-BH-R16	20.45	20	0	-	10	3	10	10	0	6	0	0	0
DD-BH-R17	20.45	20	0	-	10	3	10	10	0	11	0	0	0
DD-BH-R18	17.20	17	0	-	8	2	8	8	0	3	0	0	0
DD-BH-G01	10.00	5	0	-	0	0	0	0	0	0	0	2	0
DD-BH-G02	6.00	2	0	-	0	0	0	0	0	0	0	2	0
DD-BH-G03	10.00	6	0	-	0	0	0	0	0	0	0	2	0
DD-BH-G04	10.00	7	0	-	0	0	0	0	0	0	0	2	0
DD-BH-G05	5.00	0	0	-	0	0	0	0	0	0	0	2	0
DD-BH-G06	5.00	0	0	-	0	0	0	0	0	0	0	2	0
DD-BH-G07	10.00	7	0	-	0	0	0	0	0	0	0	1	0
DD-BH-T01	15.00	9	1	8-8.45	10	4	9	10	0	3	1	0	1
DD-BH-T02	40.32	39	1	8-8.45	21	0	20	21	0	10	0	0	0
DD-BH-C01	38.45	35	3	4.00-4.45 9.00-9.45 12.00-12.45	18	2	16	18	2	16	1	0	1
DD-BH-C02	35.45	34	1	11-11.45	16	0	15	16	3	13	1	0	1
DD-BH-C03	38.15	37	2	5.00-5.45 9.00-9.45	2	0	18	2	4	17	0	0	0
Total	818.8	724	15	-	366	102	379	366	9	260	5	30	8

Source: Study Team

Table 5.2.10 Summary Table of Soil Test Results

Geological age		Holocene		Pleistocene			
Layer		As	Ac	Ds	Dc	GFw	GFf
N value		9.9	7.9	21.1	22.6	63.5	121.5
		0.0 - 30.0	0.0 - 44.0	3.0 - 48.0	0.0 - 75.0	12.0 - 150.0	12.0 - 300.0
Physical Properties	Specific gravity	2.63	2.63	2.68	2.59	2.63	2.48
	(g/cm^3)	2.55 - 2.71	2.60 - 2.65	2.66 - 2.69	2.02 - 2.72	2.53 - 2.72	2.26 - 2.69
	NMC(%)	23.19	37.89	30.26	40.73	27.59	33.50
		5.8 - 58.5	22.0 - 66.2	18.6 - 52.7	13.3 - 94.1	9.8 - 56.6	7.0 - 59.3
	Fc(%)	24.12	73.21	39.47	81.35	-	-
		0.0 - 62.0	46.0 - 97.0	15.0 - 96.0	16.0 - 100.0	-	-
	Liquid Limit	29.6	35.2	37.3	51.6	44.5	45.0
	(%)	26.0 - 44.0	27.0 - 53.0	28.0 - 52.0	29.0 - 86.0	29.0 - 69.0	32.0 - 65.0
	Plastic Limit	18.9	20.3	21.3	23.1	22.5	23.3
	(%)	16.0 - 22.0	18.0 - 25.0	17.0 - 28.0	13.0 - 32.0	18.0 - 31.0	20.0 - 25.0
	Plasticity Index	10.7	14.9	16.0	28.5	22.0	21.7
	(%)	8.0 - 23.0	7.0 - 28.0	11.0 - 24.0	10.0 - 60.0	10.0 - 38.0	12.0 - 41.0
	Unit weight	-	-	-	1.54	-	-
	γw (g/cm^3)	-	-	-	1.46 - 1.59	-	-
Mechanical Properties	qu	-	-	-	26.7	-	6,643
	(kN/m^2)	-	-	-	18.8 - 34.7	-	474.7 - 19,461
	Pc	-	-	-	100.9	-	-
	(kg/cm^2)	-	-	-	49.1 - 196.2	-	-
	Cc	-	-	-	0.62	-	-
		-	-	-	0.46 - 0.87	-	-

Upper Value: Representative of Layer / Lower Value: Range between lowest and highest values observed
Source: Study Team

The results of each soil test are described below.

(a) Standard Penetration Test (SPT)

Table 5.2.11 shows the results of the standard penetration test. **Table 5.2.12** summarizes the results of unconsolidated layers (Ac, As, Dc, Ds), including the results of standard penetration tests for existing boring. Of the test results for rock masses (GFw, GFf), converted N values were calculated for N> 50.

For unconsolidated layers (Ac, As, Dc, Ds), there is no significant difference between the N value including the results of this survey and the results of previous surveys. Although the N values of all the formations have large variations, the N values of the Dc layer and the Ds layer are more than twice as large as the Ac layer and the As layer in terms of the average value.

Table 5.2.11 Results of standard penetration test (results of this survey)

N-value \ Simbol	Ac	As	Dc	Ds	GFw	GFf
AVERAGE	6.4	10.1	19.1	19.3	63.5	121.5
MAX	17	26	50	48	150	300
MIN	0	0	0	3	12	12
MODE	2	16	12	24	150	150
S_DEVIATION	5.1	6.2	10.6	9.4	44.4	65.6
Data	25	59	301	36	41	71

Source: Study Team

Table 5.2.12 Results of Standard Penetration Test (excluding GFw and GFf, including existing boring data)

Simbol / N-value	Ac	As	Dc	Ds
AVERAGE	7.9	9.9	22.6	21.1
MAX	44	30	75	48
MIN	0	0	0	3
MODE	3	2	21	24
S_DEVIATION	7.9	7.2	12.7	10.0
Data	113	220	698	62

Source: Study Team

(b) Undisturbed sampling

Undisturbed samples were collected basically after confirming that a soft stratum with an N value is about 4 or less, and then injecting a Shelby tube into the lower stratum to collect undisturbed samples. The number of samples collected was 15 in total because the number of target soft layers was less. The collected samples were transported to the laboratory and soil tests were conducted.

(c) Specific gravity and rock density test

The average value of the soil particle specific gravity of the Ac layer and the As layer is the same. Although the number of tests on the Ds layer is small, it is larger than the soil specific gravity of the Dc layer. The soil specific gravity of the Ac layer is higher than that of general alluvial cohesive soil and higher than that of the Dc layer (**Table 5.2.13** and **Table 5.2.14**). This is probably because the Ac layer contains a little more sand. The soil specific gravity of the Dc and Ds layers indicate the soil specific gravity of general diluvial cohesive and diluvial sandy soils.

Table 5.2.13 Specific gravity of soil particles and rock density (natural water content)

Simbol / Value type	Ac	As	Dc	Ds	GFw	GFf
AVERAGE	2.63	2.63	2.59	2.68	2.63	2.48
MAX	2.65	2.71	2.72	2.69	2.72	2.69
MIN	2.60	2.55	2.02	2.66	2.53	2.26
MODE	2.65	2.67	2.55	-	-	-
S_DEVIATION	0.02	0.05	0.10	0.01	0.07	0.15
Data	7	23	50	3	6	12

Source: Study Team

Table 5.2.14 Densities of major minerals and soil particles (Japan)

Soil name	Density ρs (g/cm³)
Alluvial sandy soil	2.6 ～ 2.8
Alluvial clayey soil	2.50～2.75
Diluvial sandy soil	2.6 ～ 2.8
Diluvial clayey soil	2.50～2.75
Peat	1.4 ～ 2.3
Black soil (Kuroboku)	2.3 ～ 2.6

Source: "Japan Geotechnical Society; Methods and explanations for soil testing ... P.58"

(d) Natural water content

The water content of the cohesive soil layers (Ac, Dc) is higher than that of the sandy soil layers (As, Ds) (See **Table 5.2.15**). The water content of the Dc layer is indicated as that of "Holocene cohesive soil" as shown in **Table 5.2.16**. The water content of the As and Ds layers is almost same value as common "Masa d o" shown in **Table 5.2.16**. The water content of the Ac layer is lower than that of the alluvial clayey soil shown in **Table 5.2.16**. This is probably because the Ac layer contains a little more sand.

Table 5.2.15 Natural Water Contents

Value type \ Simbol	Ac	As	Dc	Ds	GFw	GFf
AVERAGE	37.89	23.19	40.73	30.26	27.59	33.50
MAX	66.17	58.52	94.13	52.74	56.61	59.33
MIN	22.00	5.77	13.27	18.55	9.81	7.04
MODE	-	28.13	38.85	-	-	-
S_DEVIATION	13.89	10.29	13.05	9.34	11.47	13.62
data	14	59	174	14	25	35

Source: Study Team

Table 5.2.16 Common Water Content in Each Soil Type (in Japan)

Soil name	Area found (in Japan)	Water content (%)
Alluvial clay	Tokyo	50 ～ 80
Diluvial clay	Tokyo	30 ～ 60
Loam Soil	Kanto	80 ～ 150
Decomposed granite soil (Masado)	Cyugoku	6 ～ 30
Deposits of volcanic ash and sand (Shirasu)	South kyusyu	15 ～ 33
Andosol (Kuroboku)	kyusyu	30 ～ 270
Peat	Ishikari	110 ～ 1300

Source: "Japan Geotechnical Society; Methods and explanations for soil testing"

(e) Fine Particle Contents

Table 5.2.17 shows the fine particle content based on the results of the particle size analysis. The fine particle content of Dc layer is higher than that of the Ac layer. For this reason, the specific gravity of Dc layer is smaller than that of the Ac layer. The fines content of the Ds layer is larger than that of the As layer. For this reason, the water content ratio of the Ds layer is larger than that of As layer.

Table 5.2.17 Fine particle Contents

Value type \ Simbol	Ac	As	Dc	Ds
AVERAGE	73.21	24.12	81.35	39.47
MAX	97.00	62.00	100.00	96.00
MIN	46.00	0.00	16.00	15.00
MODE	82.00	14.00	96.00	96.00
S_DEVIATION	16.40	15.59	18.71	24.32
data	14	59	171	15

Source: Study Team

(f) Liquid limit / plastic limit

The average value of the liquid limit depends on the soil layer, but the average value of the plastic limit is around 20%. According to the plasticity index and the plasticity diagram, the Dc layer is evaluated as a highly compressible and highly plastic clay.

Table 5.2.18 Fine particle Contents

【Liquid limit】

Value type \ Simbol	Ac	As	Dc	Ds
AVERAGE	35.21	29.62	51.58	37.33
MAX	53.00	44.00	86.00	52.00
MIN	27.00	26.00	29.00	28.00
MODE	32.00	27.00	43.00	-
S_DEVIATION	8.18	5.16	12.18	8.33
data	14	13	167	6

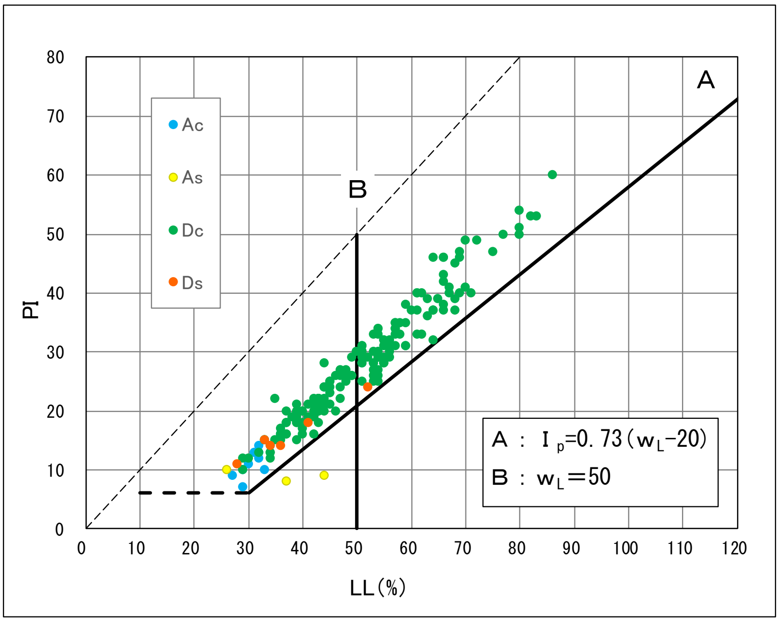
【Plastic limit】

Value type \ Simbol	Ac	As	Dc	Ds
AVERAGE	20.29	18.92	23.07	21.33
MAX	25.00	22.00	32.00	28.00
MIN	18.00	16.00	13.00	17.00
MODE	20.00	18.00	23.00	-
S_DEVIATION	2.16	1.66	3.35	3.98
data	14	13	167	6

【Plasticity index】

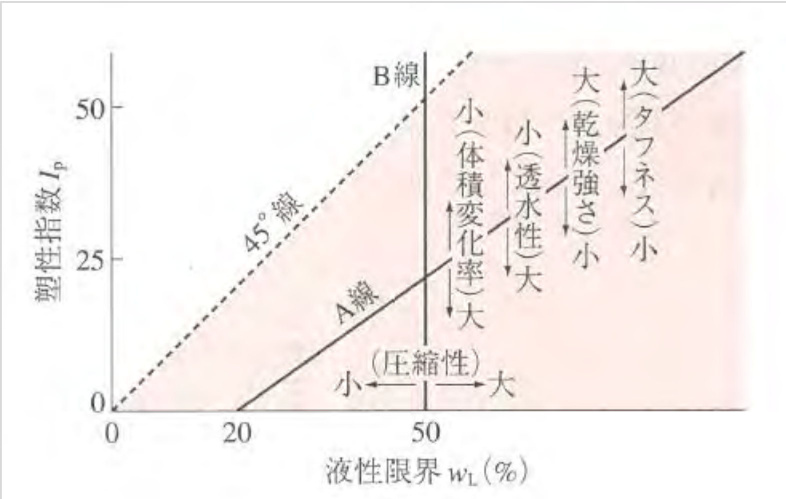
Value type \ Simbol	Ac	As	Dc	Ds
AVERAGE	14.93	10.69	28.51	16.00
MAX	28.00	23.00	60.00	24.00
MIN	7.00	8.00	10.00	11.00
MODE	13.00	8.00	20.00	14.00
S_DEVIATION	6.82	4.25	10.25	4.52
data	14	13	167	6

Source: Study Team



Source : Study Team

Figure 5.2.25 Plasticity Diagram of Soil along Marikina River



Source : "Soil Testing Basics and Guide P.45: Geotechnical Society"

Figure 5.2.26 Mechanical properties of cohesive soil based on plasticity diagram

(g) Uniaxial compression test of soil

A uniaxial compression test was performed on the soft part (N ≦ 4) of the Dc layer. Although the number of tests is small, the average value is 27 kPa, indicating a value of c = 6 × N (kPa).

Table 5.2.19 Uniaxial compressive strength of soil (Dc layer)

Borehole No.	Sample No.	Sampling Depth (m)	Symbol	Wet Density (Kg/m3)	Compressive Strength (KPa)
BH-R03	uds-1	3.00-3.45	Dc	12,550	35
BH-R05	uds-1	2.00-2.45	Dc	14,520	19
All	Average			13,540	27

Source: Study Team

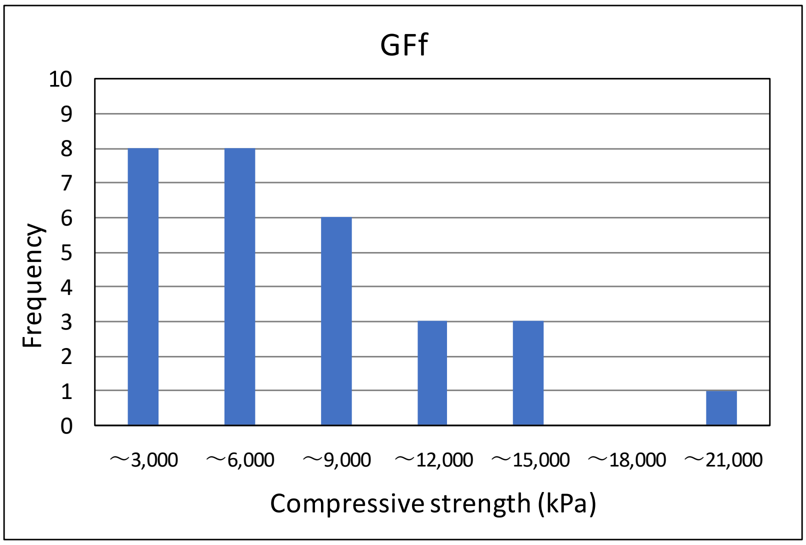
(h) Uniaxial compression test of rock

A uniaxial compression test (29 samples) was performed on GFf. The unconfined compressive strength is 6,643 kPa on average, and the wet density of the test sample is 17.16 kN / m3 on average (**Table 5.2.20** and **Figure 5.2.27**).

Table 5.2.20 Result of Uniaxial Compression Test of Rock

Borehole No.	Sample No.	Sampling Depth (m)	Symbol	Wet Density (Kg/m3)	Compressive Strength (KPa)
BH-G01	cs-1	6.70-6.90	GFf	16,910	11,685
	cs-2	7.60-7.80	GFf	16,450	6,765
BH-G02	cs-2	3.20-3.40	GFf	18,110	5,933
	cs-4	4.30-4.31	GFf	16,200	2,982
BH-G03	cs-1	6.20-6.40	GFf	19,180	14,240
	cs-2	7.20-7.40	GFf	18,410	4,037
BH-G04	cs-1	7.00-7.20	GFf	18,730	11,807
	cs-2	8.40-8.60	GFf	19,940	12,282
BH-G05	cs-1	0.20-0.38	GFf	15,750	3,085
	cs-2	4.30-4.45	GFf	17,680	4,747
BH-G06	cs-2	2.40-2.60	GFf	16,500	2,112
	cs-5	4.10-4.30	GFf	18,950	6,882
BH-G07	cs-2	8.75-8.90	GFf	17,250	9,194
BH-L05	cs-1	7.60-7.72	GFf	16,770	3,204
	cs-2	8.20-8.34	GFf	17,600	2,017
BH-L06	cs-4	8.20-8.44	GFf	17,240	4,925
	cs-5	9.25-9.45	GFf	16,590	2,172
BH-L07	cs-2	8.27-8.50	GFf	15,300	3,790
BH-L11	cs-9	16.00-16.20	GFf	14,390	1,802
BH-R01	cs-1	0.33-0.50	GFf	17,900	7,393
	cs-5	4.00-4.15	GFf	18,340	19,461
BH-R02	cs-1	2.30-2.50	GFf	17,690	13,528
	cs-3	4.00-4.30	GFf	18,350	6,408
BH-R07	cs-1	6.27-6.40	GFf	18,300	1,246
	cs-2	7.23-7.42	GFf	16,500	475
	cs-3	8.00-8.20	GFf	15,560	8,449
BH-R09	cs-1	4.25-4.42	GFf	16,670	5,755
	cs-2	5.25-5.40	GFf	15,110	6,882
	cs-3	6.20-6.40	GFf	15,290	2,753
All	Maximum			19,940	19,461
	Minimum			14,390	475
	Average			17,160	6,643

Source: Study Team



Source : Study Team

Figure 5.2.27 Histogram of uniaxial compressive strength of rock

(i) Soil consolidation test

A consolidation test was performed on the cohesive soil layers (Ac layer, Dc layer). In some samples, the consolidation yield stress (Pc) and effective overburden pressure (P0) show Pc / P0> 1, indicating that they are slightly over-consolidated. The consolidation index (Cc) tends to increase as the liquidity limit (LL) increases.

Table 5.2.21 Result of soil consolidation test (Part 1)

Borehole No.	Sample No.	Sampling Depth (m)	Simbol	LL (%)	PL (%)	PI (%)	Specific gravity (g/cm^3)	Precon Pressure Pc (kPa)	Effective overburden pressure★ P_0 (kPa)	Pc/P_0	Compression Index Cc
BH-L10	uds-1	7.00 - 7.45	Ac	41	25	16	2.63	58.86	112.69	0.52	0.50

Borehole No.	Sample No.	Sampling Depth (m)	Simbol	LL (%)	PL (%)	PI (%)	Specific gravity (g/cm^3)	Precon Pressure Pc (kPa)	Effective overburden pressure★ P_0 (kPa)	Pc/P_0	Compression Index Cc
BH-L01	uds-1	3.00 - 3.45	Dc	66	28	38	2.65	68.67	47.14	1.46	0.87
BH-L13	uds-1	8.00 - 8.45	Dc	54	20	34	2.67	88.29	128.29	0.69	0.46
BH-R03	uds-1	3.00 - 3.45	Dc	52	28	24	2.61	49.05	46.19	1.06	0.74
BH-R13	uds-2	15.00 - 15.45	Dc	43	21	22	2.60	196.20	237.48	0.83	0.47
Maximam				66	28	38	2.67	-	-	-	0.87
Minimam				43	20	22	2.60	-	-	-	0.46
Average				54	24	30	2.63	-	-	-	0.65

★Effective overburden pressure = (Wet unit wt) x (Sampling depth average)

Source: Study Team

Table 5.2.22 Result of soil consolidation test (Part 2)

Borehole No.	Sample No.	Sampling Depth (m)	Simbol	Water content (%)		Wet unit wt (g/cm^3)		Void ratio (%)		Saturation (%)	
				Initial	Final	Initial	Final	Initial	Final	Initial	Final
BH-L10	uds-1	7.00 - 7.45	Ac	53.67	26.14	1.59	1.98	1.42	0.68	99.28	101.39

Borehole No.	Sample No.	Sampling Depth (m)	Simbol	Water content (%)		Wet unit wt (g/cm^3)		Void ratio (%)		Saturation (%)	
				Initial	Final	Initial	Final	Initial	Final	Initial	Final
BH-L01	uds-1	3.00 - 3.45	Dc	73.61	30.08	1.49	1.92	1.97	0.80	99.18	100.19
BH-L13	uds-1	8.00 - 8.45	Dc	51.18	28.43	1.59	1.96	1.37	0.75	99.84	100.75
BH-R03	uds-1	3.00 - 3.45	Dc	71.81	30.76	1.46	1.91	1.89	0.79	99.12	101.98
BH-R13	uds-2	15.00 - 15.45	Dc	54.20	37.86	1.59	1.82	1.42	0.97	99.16	101.86
Maximam				73.61	37.86	1.59	1.96	1.97	0.97	99.84	101.98
Minimam				51.18	28.43	1.46	1.82	1.37	0.75	99.12	100.19
Average				62.60	32.24	1.53	1.90	1.67	0.84	99.38	101.16

Source: Study Team

2) Proposed Soil Modulus

Based on the previous survey results and the current survey results, the soil modulus shown in **Table 5.2.23** are proposed. The grounds for setting each constant are described below.

Table 5.2.23 Proposed Soil Modulus

Geological Classification		Unit Weight of Wet Soil	Unit Weight of Submerged Soil	Angle of Internal Friction	Effective Cohesion	Reference
		γt	γw	ϕ	c	*Average N Value*
Name	Symbol	(kN/m3)	(kN/m3)	(degree)	(kN/m2)	-
Field Soil	F	18.0	8.0	30	0	15
Alluvial Sand	As	17.5	7.5	For River Structure N<9: 27 9 ≦ N: $15+\sqrt{15N}$ (Max 45) For MCGS $\emptyset = 4.8 \times \log N_0$ N_1 $= 170\ x\ N/(\delta, +70)$	0	10
Diluvial Sand	Ds	19.0	9.0	N<9: 27 9 < N: $15+\sqrt{15N}$ (Max 45)	0	21
Alluvial Clay	Ac	15.5	5.5	0	N= 6 x N 4<N≦8: 25 8<N≦15: 50 15<N≦30: 100 30<N : 200	8
Diluvial Clay	Dc	18.0	8.0	0	N= 8 x N 4<N≦8: 50 8<N≦15: 100 15<N≦30: 180 30<N : 250	23
Guadalupe Formation	GFw	16.5	6.5	20	200	64
	GFf	17.0	7.0	30	For River Structure and MCGS Retaining Wall 1,000 For MCGS Main Body 3,500	122

Source: Study Team

(a) Unit Weight

In this survey, the unit volume weight of the Ac layer and the Dc layer of about N ≦ 4 was measured in the uniaxial compression test and the consolidation test, but the number is small. For this reason, the unit volume weight of the unconsolidated layer was set from the experiences in Phase I (**Table 5.2.24**) and experiences in Japan (**Table 5.2.25**). On the other hand, for GFf, the unit volume weight of 29 samples was measured, and was set based on the result.

Table 5.2.24 Example of unit weight of soil (Based on Japanese Experiences)

Ground	Soil	Unit weight of soil (tf/m3)	
		Loose	Dense
Natural ground	Sand and gravel	1.8	2
	Sandy soil	1.7	1.9
	Cohesive soil	1.4	1.8
Embankment	Sand and gravel	2	
	Sandy soil	1.9	
	Cohesive soil	1.8	

Source: Japan Highway Association Road Bridge Specification (I Common Edition) / Commentary (1996)

Table 5.2.25 Soil modulus (Properties) at Phase 1

Geological Age		Recent	Holocene		Pleistocene			
Layer		F	Ac	As	Dc1	Dc2	GFw	GF
N-Value		0~31	0~5	0~11	5~28	15~29	30~	
Physical Properties	NMC (%)	19~73	37~75	17~38	26~82	25~77	17~55	5~37
	Liquid Limit (%)	43~71	45~72	-	46~84	44~89	45~78	-
	Plastic Limit (%)	19~25	20~26	-	21~26	21~33	21~30	-
	Plasticity Index (%)	22~49	35~50	-	22~60	20~60	17~52	-
	Gravel (%)	0~68	0~5	0~23	0~(76)	0~10	0~37	-
	Sand (%)	2~94	1~22	55~94	2~(54)	1~18	4~78	-
	Fines (%)	1~95	78~98	6~42	(5)~97	91~99	1~91	-
	Unit Weight (g/cm^3)	1.80	1.49~1.82 1.55	1.75	1.76~1.9 1.80	1.80	1.65	1.65
Mechanical Properties	ϕ (degree)	25	0	25	0	0	40**	0
	qu (kgf/cm^2)	0	0.53~1.60 0.9	-	1.47 1.5	2.5	7.5*	7~205 70
	Cu (kgf/cm^2)	0	0.45	-	0.36~0.51 0.50	1.25	3.3*	35
	Pc (kgf/cm^2)	0	1.25~1.70 1.5	-	2.14~2.50 2.0	4.2	11.0*	-
	Cc	0	0.35~0.50 0.5	-	0.26~0.52 0.5	0.5	0.4*	-

Note: *) Sandy Portion
**) Clayey Portion

Source: Study Team

(b) Soil adhesion and internal friction angle

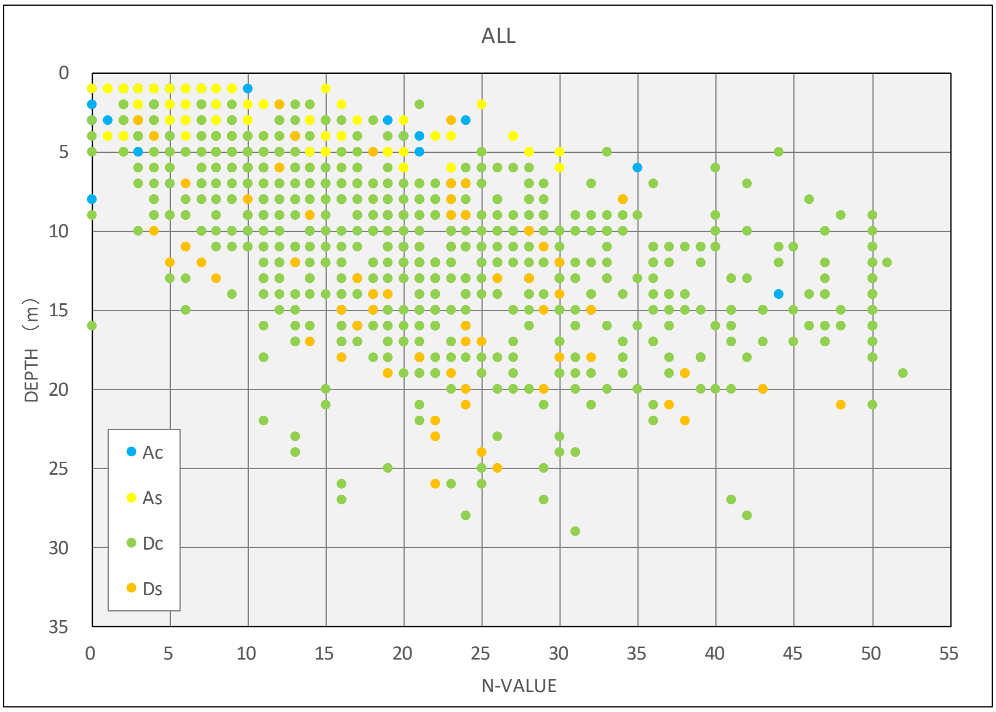
Sandy soil contains a certain amount of fine particles, while clayey soil contains a certain amount of sand, but basically the adhesiveness (c) of sandy soil is zero and the internal friction angle (φ) of cohesive soil is zero as general consideration of the soil modulus. For the following reasons, the internal friction angle (φ) of sandy soil and adhesive strength (c) of cohesive soil were set based on the N value as shown in **Table 5.2.23** in this detailed design.

- The N value of each soil layer has varied widely.
- The N value is dependent on depth (**Figure 5.2.28**).
- As the angle of internal friction of As layer, for river structures, the same value at Phase 1~3

is used, and, for MCGS, the below equation is used in consideration with the relation N value and effective overloading pressure.

$$\phi = 4.8 \times \log N_1 + 21、N_1 = 170 \times N / (\sigma'_v + 70)$$ [1]

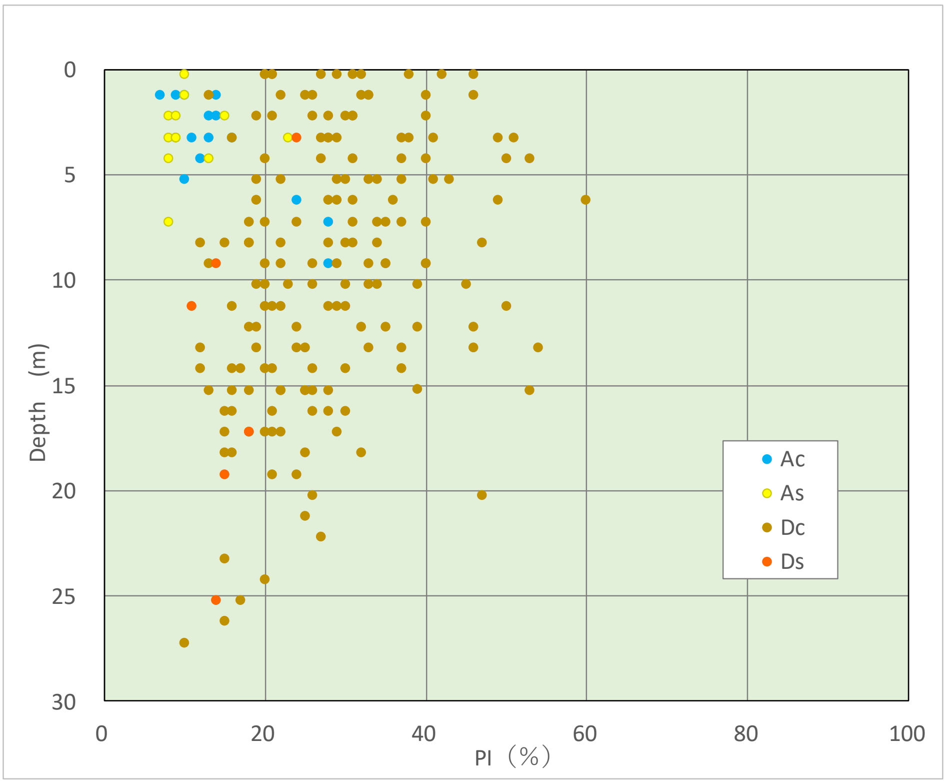
- The adhesive strength of the Ac layer was set based on qu = 12.5N: Terzaghi and Peck. Also, relationship between c and qu ($c = {}^{q_u}/_2$) when qu was examined by laboratory test. Therefore, c = 6.25N is adopted.
- Since the average value of the Dc layer is more than twice that of the Ac layer and the plasticity index (PI) is quite high. In this connection it was set based on c = 8.35N (qu = 16.7N: Peck) (**Figure 5.2.29** and **Figure 5.2.30**).



Source : Study Team

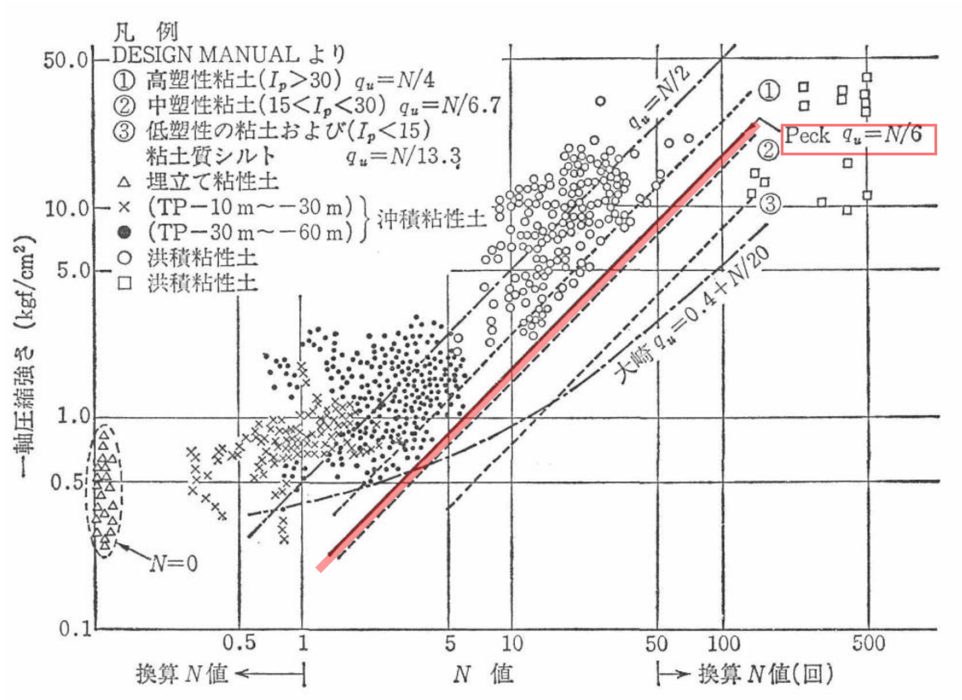
Figure 5.2.28 Relationship between depth and N value

[1] Specifications for Highway Bridges IV Substructures



Source : Study Team

Figure 5.2.29 Relationship between depth and N value



Source : (public) Japan Society of Civil Engineers Strength constants in design - C, φ, N values [1988]

Figure 5.2.30 Relationship between uniaxial compressive strength (qu) and N value

(c) Rock adhesive strength and internal friction angle

For GFw and GFf, the intensity (c, φ) can be estimated from the reduced N value using the estimation formula shown in **Table 5.2.26**. However, for GFf, since a uniaxial compression test was performed on 29 samples, the average value (6,643 kPa) was used to calculate the compressive strength (qu) and shear strength (adhesive strength) shown below. Thus, the adhesive strength (c) was estimated. For the internal friction angle (φ) of GFf, the lower limit of CL class rock was adopted from the previous general values (**Figure 5.2.31**) based on the results of rock evaluation using a boring core.

Log(c)=0.9144 log(qu)-0.6106 （kgf/cm^2）

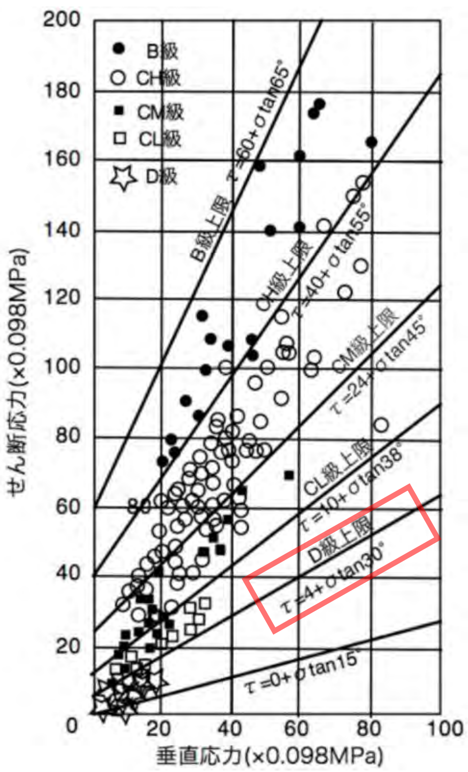
(Source: Correlation equation between compressive strength and adhesive strength "rock classification and its application)

As for the internal friction angle used for the main body of the MCGS, since the boring was carried out intensively, the angle established from nearby borings is used.

Table 5.2.26 Estimation of Rock Mass Strength Using Converted N Value

		Sandstone and conglomerate plutonic rocks	Andesite	Mudstone and tuff, tuff breccia	Remarks
adhesive (kN/m2)	Relation between adhesive N value and average N value	$15.2N^{0.327}$	$25.3N^{0.334}$	$16.2N^{0.606}$	
	Standard deviation	0.218	0.384	0.464	Value on Log axis
Shear resistance angle	Relation between adhesive N value and average N value	5.10LogN +29.3	6.82LogN +21.5	0.888LogN +19.3	
	Standard deviation	4.4	7.85	9.78	

Source: East Japan Expressway Co., Ltd. Design Guidelines



Source : "Rock classification and its application

Figure 5.2.31 Relationship between rock mass class and in-situ test results for massive rock mass

3) Consideration of Driving SSP into Hard Strata (Soft-rock)

In the study area, the Guadalupe bedrock is distributed in some places. The surface of the Guadalupe bedrock is rugged. Those strengths are widely observed from 0.5MPa to 20MPa as shown in Figure 5.2.27 and Table 5.2.20.

Most of sections of river channel will be widened and all of channel slopes will be protected by Self-Supporting Steel Sheet Pile Revetment (SSP Revetment).

In general, the design of self-supporting SSP Revetment has been executed utilizing Formula of “Chang” provided that the SSP has semi-infinitely long. Those SSP Revetment will be driven by Vibro-hammer with Waterjet Machine. Vibro-Hammer with Waterjet Method will be adopted when the strength of rock is lower than 3~4MPa. Therefore, the SSP Revetment may not be driven up to designed depth. In that case, the design of SSP Revetment may be modified as pile having finite length.

To finalize the design of SSP Revetment, the further boring data with strength of Guadalupe bedrock are required in the construction stage.

(2) Cainta Floodgate / Taytay Sluicegate

1) Soil test results

The following laboratory soil tests were conducted for the borehole at the proposed sites of Cainta / Taytay Gate structures. Samples used for the test were both of the undisturbed samples collected by shelby Tube and disrupting samples taken after SPT. Test items and quantities are shown in **Table 5.2.27**. **Table 5.2.28** shows a summary of the soil test results including in-situ tests.

Table 5.2.27 List of Boring Survey and their Quantities for the Cainta Floodgate and Taytay Sluicegate

Hole No.	Depth (m)	SPT	UDS	Classification	Specific gravity	Moisture Content	Particle Size	Particle Size	Attenberg	Soil Unconfined	Rock Strength	Consolidation
		ASTM D1586		ASTM D2487	ASTM D854	ASTM D2216	ASTM D422	ASTM E100	ASTM D4318	ASTM D2166	ASTM D2938	ASTM D2435
DD-BH-T01	15.00	9	1	10	4	9	10	0	3	1	0	1
DD-BH-T02	40.32	39	1	21	0	20	21	0	10	0	0	0
DD-BH-C01	38.45	35	3	18	2	16	18	2	16	1	0	1
DD-BH-C02	35.45	34	1	16	0	15	16	3	13	1	0	1
DD-BH-C03	38.15	37	2	2	0	18	2	4	17	0	0	0
Total	167.37	154	8	67	6	78	67	9	59	3	0	3

Source: Study Team

Table 5.2.28 Summary of Soil Test Results for the Cainta Floodgate and Taytay Sluicegate

Geological age		Holocene				Pleistocene	
Layer		As2	As1	Ac2	Ac1	Ds1	Dc1
N value		19.8	28.9	5.5	12.9	50.0	22.1
		0.0 - 23.0	0.0 - 50.0	0.0 - 17.0	0.0 - 50.0	0.0 - 50.0	0.0 - 50.0
Physical Properties	Specific gravity	2.64	2.68	2.61	2.60	-	2.67
	(g/cm^3)	2.60 - 2.67	2.66 - 2.70	2.58 - 2.65	2.60 - 2.60	-	2.67 - 2.67
	NMC(%)	25.2	27.0	56.9	49.3	35.3	43.1
		13.8 - 79.2	18.7 - 34.9	41.4 - 66.9	31.9 - 76.4	24.1 - 58.4	23.6 - 69.5
	Liquid Limit	59.0	42.0	40.2	59.8	52.0	58.6
	(%)	59.0 - 59.0	42.0 - 42.0	36.0 - 47.0	34.0 - 103.0	38.0 - 67.0	34.0 - 91.0
	Plastic Limit	23.0	19.0	23.0	24.7	23.0	24.5
	(%)	29.0 - 29.0	19.0 - 19.0	21.0 - 24.0	19.0 - 36.0	19.0 - 27.0	18.0 - 35.0
	Plasticity Index	30.0	23.0	17.2	35.1	29.0	34.0
	(%)	30.0 - 30.0	23.0 - 23.0	13.0 - 23.0	15.0 - 67.0	16.0 - 40.0	15.0 - 64.0
	Unit weight	-	-	-	1.59	-	-
	γw (g/cm^3)	-	-	-	1.49 - 1.69	-	-
Mechanical Properties	qu	-	-	-	38.5	-	-
	(kN/m^2)	-	-	-	28.0 - 49.0	-	-
	Pc	-	-	-	94.83	-	-
	(kg/cm^2)	-	-	-	68.7 - 117.7	-	-
	Cc	-	-	-	0.80	-	-
		-	-	-	0.42 - 1.17	-	-

Source: Study Team

The results of each soil test are described below.

(a) Standard Penetration Test

Table 5.2.29 shows the results of the standard penetration test (SPT). As for unconsolidated layers, the converted N value was not determined, and N = 50 was set as the upper limit. Except for the Ac2 and the As2 layer, the average N-value of the Dc1 and the Ds1 layer is about twice or more compared to that of the Ac1 and Ac2 layer, and that of the As1 and As2 layer although the actual N-Value numbers have varied widely.

Table 5.2.29 Results of SPT of the Cainta Floodgate and Taytay Sluicegate

Simbol / N-value	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	12.9	5.5	28.9	19.8	22.1	50.0
MAX	50.0	17.0	50.0	23.0	50.0	50.0
MIN	0.0	0.0	0.0	0.0	0.0	0.0
MODE	2.0	2.0	25.0	22.0	19.0	50.0
S_DEVIATION	11.4	6.4	12.7	5.7	10.5	0.0
Data	32	17	5	13	67	9

Source: Study Team

(b) Collection of Undisturbed Samples

At the Cainta / Taytay sites, eight (8) undisturbed samples were collected using a shelby tube. However, mechanical tests could be conducted for only three (3) samples because one sample were dropped off during hauling it and obtained disturbed soils from remaining four (4) samples were insufficient for the conduct of mechanical tests.

(c) Specific gravity of soil particles and density test of rock

The particle specific gravity of each soil layer indicates a common value in the range of general alluvial cohesive soil, alluvial sandy soil and diluvial cohesive soil. Since the specific gravity of soil particles of Dc1 layer is relatively large compared to common value of same classification of soil, it is considered that Dc1 layer contains a little sand.

Table 5.2.30 Specific gravity of soil particles

Simbol / Value type	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	2.60	2.61	2.68	2.64	2.67	-
MAX	2.60	2.65	2.70	2.67	2.67	-
MIN	2.60	2.58	2.66	2.60	2.67	-
Data	1	2	2	3	1	0

Source: Study Team

(d) Natural Water Contents

The natural moisture content of each soil layer indicates values in the range of general alluvial cohesive soil and diluvial cohesive soil.

Table 5.2.31 Natural Water Content

Simbol / Value type	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	49.26	56.90	27.05	25.19	43.13	35.34
MAX	76.41	66.92	34.86	79.24	69.48	58.43
MIN	31.86	41.38	18.66	13.84	23.56	24.09
MODE	-	64.92	-	-	-	-
S_DEVIATION	12.72	11.71	7.45	17.41	11.97	14.61
data	22	6	5	12	34	5

Source: Study Team

(e) Fine Particle Content

Each soil layer has the characteristics of cohesive soil and sandy soil, but As2 layer has a large variation in the fine particle content and contains some cohesive soil. On the other hand, the content of fine particles in Ac2 layer is large, and almost no sand is contained.

Table 5.2.32 Fine Particle Contents

Simbol / Value type	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	78.80	91.50	22.20	16.33	81.94	41.60
MAX	98.00	98.00	37.00	93.00	99.00	52.00
MIN	53.00	85.00	13.00	3.00	30.00	26.00
MODE	78.00	-	-	6.00	94.00	-
S_DEVIATION	12.34	4.76	9.04	25.16	17.26	10.01
data	20	6	5	12	34	5

Source: Study Team

(f) Liquid limit / plastic limit

The average value of the liquid limit depends on the soil layer, but the average value of the plastic limit is around 20%. According to the plasticity index and the plasticity diagram, Ac1 layer and Dc1 layer are evaluated as highly compressible and highly plastic clay.

Table 5.2.33 Liquidity limit and plastic limit

【LL(%)】

Simbol / Value type	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	59.78	40.17	42.00	59.00	58.56	52.00
MAX	103.00	47.00	42.00	59.00	91.00	67.00
MIN	34.00	36.00	42.00	59.00	34.00	38.00
MODE	44.00	36.00	-	-	72.00	-
S_DEVIATION	18.69	4.62	-	-	14.48	11.14
data	23	6	1	1	34	5

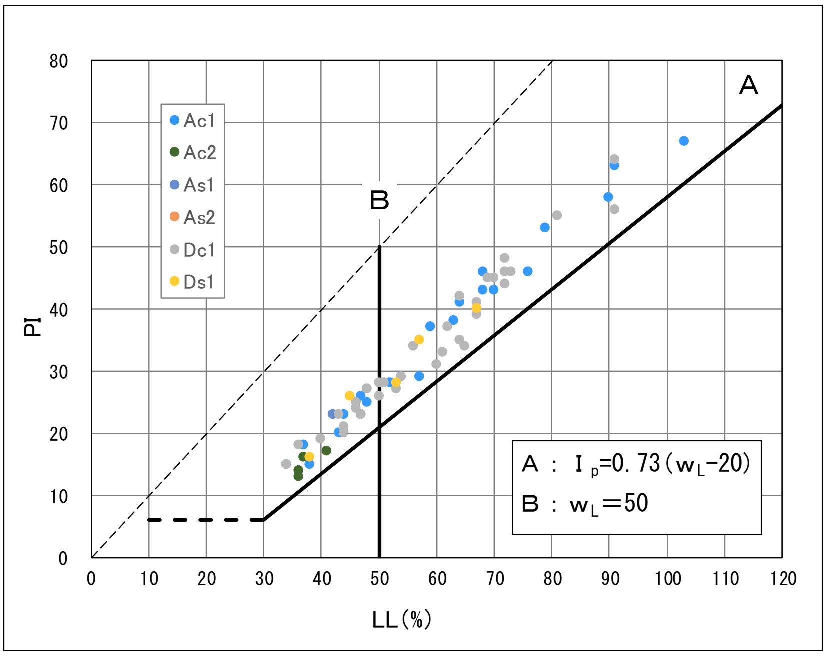
【PL(%)】

Simbol / Value type	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	24.65	23.00	19.00	29.00	24.53	23.00
MAX	36.00	24.00	19.00	29.00	35.00	27.00
MIN	19.00	21.00	19.00	29.00	18.00	19.00
MODE	23.00	24.00	-	-	24.00	22.00
S_DEVIATION	4.11	1.26	-	-	3.49	3.08
data	23	6	1	1	34	5

【PI(%)】

Simbol / Value type	Ac1	Ac2	As1	As2	Dc1	Ds1
AVERAGE	35.13	17.17	23.00	30.00	33.97	29.00
MAX	67.00	23.00	23.00	30.00	64.00	40.00
MIN	15.00	13.00	23.00	30.00	15.00	16.00
MODE	29.00	-	-	-	27.00	-
S_DEVIATION	15.32	3.76	-	-	11.99	9.17
data	23	6	1	1	34	5

Source: Study Team



Source : Study Team

Figure 5.2.32 Plasticity Diagram for Cainta and Taytay Sites

(g) Uniaxial compression test of soil

In Ac1 layer, samples were taken at three locations, and a uniaxial compressive strength test was performed. In the sample (cs-2) collected from the section of DD-BH-T01 hole where N> 50, uniaxial compressive strength comparable to that of rock was obtained. , N-value of about 11 to 23 is confirmed. Therefore, the average value of 38.50 kN / m2 obtained by rejecting the above values was evaluated as the uniaxial compressive strength of Ac1 layer.

Table 5.2.34 Uniaxial compressive strength of soil (Ac1 layer)

Borehole No.	Sample No.	Sampling Depth (m)			Simbol	Wet Density (g/cm^3)	Compressive Strength (kPa)
BH-T01	cs-2	11.80	-	12.00	Ac1	17.65	5,102.50
BH-C01	uds-3	12.30	-	12.45	Ac1	16.57	48.96
BH-C02	uds-1	11.20	-	11.30	Ac1	15.77	28.05
	Average					16.66	38.50

Source: Study Team

(h) Soil Consolidation Test

A consolidation test was performed on the Ac1 layer. All samples show a normal consolidation state, and the consolidation index (Cc) tends to increase as the liquid limit (LL) increases.

Table 5.2.35 Result of soil consolidation test (Part 1)

Borehole No.	Sample No.	Sampling Depth	Simbol	LL	PL	PI	Specific gravity	Precon Pressure	Effective overburden pressure★	Pc/P_0	Compression Index
		(m)		(%)	(%)	(%)	(g/cm^3)	Pc (kPa)	P_0 (kPa)		Cc
BH-T01	uds-1	8.00 - 8.45	Ac1	59	29	30	2.60	98.10	120.22	0.82	1.17
BH-C01	uds-3	12.0 - 12.5	Ac1	90	32	58	2.67	117.72	189.49	0.62	0.80
BH-C02	uds-1	11.0 - 11.5	Ac1	44	24	20	2.64	68.67	186.10	0.37	0.42
	Average			64	28	36	2.64	-	-	-	0.80

★Effective overburden pressure = (Wet unit wt) x (Sampling depth average)

Source: Study Team

Table 5.2.36 Result of soil consolidation test (Part 2)

Borehole No.	Sample No.	Sampling Depth	Simbol	Water content (%)		Wet unit wt (g/cc)		Void ratio (%)		Saturation (%)	
		(m)		Initial	Final	Initial	Final	Initial	Final	Initial	Final
BH-T01	uds-1	8.00 - 8.45	Ac1	79.24	40.88	1.49	1.79	2.11	1.05	97.43	101.46
BH-C01	uds-3	12.0 - 12.5	Ac1	68.68	34.84	1.58	1.87	1.85	0.92	99.19	101.13
BH-C02	uds-1	11.0 - 11.5	Ac1	51.96	28.84	1.69	1.94	1.38	0.75	99.54	100.96
	Average			66.63	34.85	1.59	1.87	1.78	0.91	98.72	27.24

Source: Study Team

2) Proposed Soil Modulus

Based on the results of the geological survey mentioned above, the soil modulus shown in **Table 5.2.37** are proposed. Since the Cainta gate / Taytay sluicegate are not a continuous linear structure like a seawall, it is considered appropriate to set the soil modulus of the relevant location individually from the N value in each boring. The detailed soil modulus being utilized in this detailed design are explained in Chapter 6.

Table 5.2.37 Soil Modulus to be Utilized in this Detailed Design

Geological Classification		Unit Weight of Wet Soil	Unit Weight of Submerged Soil	Angle of Internal Friction	Effective Cohesion	Reference
		γt	γw	ϕ	c	*Average N Value*
Name	Symbol	(kN/m3)	(kN/m3)	(degree)	(kN/m2)	-
Field Soil	F	18.0	8.0	30	0	15
Alluvial Sand	As2	17.0 – 20.0	7.0 – 10.0	25 – 38 $\emptyset = 4.8 \times \log N_0$ $N_1 = 170\ x\ N/(\delta, +70)$	0	20
	As1	20.0 – 21.0	10.0 – 11.0	33 – 39 $\emptyset = 4.8 \times \log N_0$ $N_1 = 170\ x\ N/(\delta, +70)$	0	29
Diluvial Sand	Ds1	19.0 – 21.0	9.0 – 11.0	40	0	50
Alluvial Clay	Ac2	15.0 – 18.0	5.0 – 8.0	0	N =1, 2 : 14 N=3 : 24 N ≧ 4 : 130 – 200	6
	Ac1	15.0 – 18.0	5.0 – 8.0	0	N =1~2 : 14 N=3 : 24 N ≧ 4 100 – 250	13
Diluvial Clay	Dc1	18.0 - 19.0	8.0 – 9.0	0	220 – 620	22

Source: Study Team

The grounds for setting each constant are described below.

(a) Unit Weight

In this study, the unit volume weight in the Ac1 layer was measured in the uniaxial compression test and the consolidation test, but there was no measurement in other formations. For this reason, the unit volume weight of the unconsolidated layer was set based on the set values (**Table 5.2.25**), N values, and general values (**Table 5.2.24**) in Phase 1.

Table 5.2.38 Example of soil constant used in design

<table>
<tr><th colspan="2">Classification</th><th colspan="2">Sample condition</th><th>Wet density (k N/m3)</th><th>φ (dgree)</th><th>c (kN/m2)</th><th>Geotechnical Society Standard※</th></tr>
<tr><td rowspan="6">Embankment</td><td>Gravel and sand mixed with gravel</td><td colspan="2">Compacted</td><td>20</td><td>40</td><td>0</td><td>(G)</td></tr>
<tr><td rowspan="2">Sand</td><td rowspan="2">Compacted</td><td>Wide particle size range</td><td>20</td><td>35</td><td>0</td><td rowspan="2">(S)</td></tr>
<tr><td>Well-classified particles</td><td>19</td><td>30</td><td>0</td></tr>
<tr><td>Sandy soil</td><td colspan="2">Compacted</td><td>19</td><td>25</td><td>30 or less</td><td>(SF)</td></tr>
<tr><td>Cohesive soil</td><td colspan="2">Compacted</td><td>18</td><td>15</td><td>50 or less</td><td>(M,C)</td></tr>
<tr><td>Kanto Loam</td><td colspan="2">Compacted</td><td>14</td><td>20</td><td>10 or less</td><td>(V)</td></tr>
<tr><td rowspan="15">Natural ground</td><td rowspan="2">Gravel</td><td colspan="2">Dense or Wide particle size range</td><td>20</td><td>40</td><td>0</td><td rowspan="2">(G)</td></tr>
<tr><td colspan="2">Not dense or well classified particles</td><td>18</td><td>35</td><td>0</td></tr>
<tr><td rowspan="2">Sand with gravel</td><td colspan="2">Dense</td><td>21</td><td>40</td><td>0</td><td rowspan="2">(G)</td></tr>
<tr><td colspan="2">Not dense</td><td>19</td><td>35</td><td>0</td></tr>
<tr><td rowspan="2">Sand</td><td colspan="2">Dense or wide particle size range</td><td>20</td><td>35</td><td>0</td><td rowspan="2">(S)</td></tr>
<tr><td colspan="2">Not dense or well-classified particles</td><td>18</td><td>30</td><td>0</td></tr>
<tr><td rowspan="2">Sandy soil</td><td colspan="2">Dense</td><td>19</td><td>30</td><td>30 or less</td><td rowspan="2">(SF)</td></tr>
<tr><td colspan="2">Not dense</td><td>17</td><td>25</td><td>0</td></tr>
<tr><td rowspan="3">Cohesive soil</td><td colspan="2">Stiff</td><td>18</td><td>25</td><td>50 or less</td><td rowspan="3">(M,C)</td></tr>
<tr><td colspan="2">Medium stiff</td><td>17</td><td>20</td><td>30 or less</td></tr>
<tr><td colspan="2">Soft</td><td>16</td><td>15</td><td>15 or less</td></tr>
<tr><td rowspan="3">Clay and silt</td><td colspan="2">Stiff</td><td>17</td><td>20</td><td>50 or less</td><td rowspan="3">(M,C)</td></tr>
<tr><td colspan="2">Medium stiff</td><td>16</td><td>15</td><td>30 or less</td></tr>
<tr><td colspan="2">Soft</td><td>14</td><td>10</td><td>15 or less</td></tr>
<tr><td>Kanto Loam</td><td colspan="2"></td><td>14</td><td>5（φu）</td><td>30</td><td>(V)</td></tr>
<tr><td></td><td colspan="7">Estimated N value：Dense（N=8～15）,Medium dense（N=４～8）,Soft（N=2～4）</td></tr>
<tr><td></td><td colspan="7">※is a guideline</td></tr>
</table>

Source: Guideline for Earth Works in Road Project, Japan

(b) Soil adhesion and internal friction angle

Basically, the soil modulus was set as follows, assuming that the adhesive strength (c) of the sandy soil was zero and the internal friction angle (φ) of the cohesive soil was zero.

Sandy soil: Estimated by the following equation using N value and effective loading pressure as parameters. However, φ = 40 ° is set for the lowermost Ds1 layer with N> 50.

$\varphi = 4.8 \times \ln N_1 + 21$ （$N>5$）， $N_1 = 170 \times N / (\sigma' v + 70)$， $\sigma'_v = \gamma_{t1} h_w + \gamma'_{t2} (x - h_w)$

where,

σ'v: The value at the time when the standard penetration test was performed at the effective loading pressure (kN / m2)

N1: N value converted to an effective loading pressure of 100 kN / m2. However, when σ′v at the original position is σ′v <50 kN / m2, calculation is performed as σ′v = 50 kN / m2.

N: N value obtained from standard penetration test

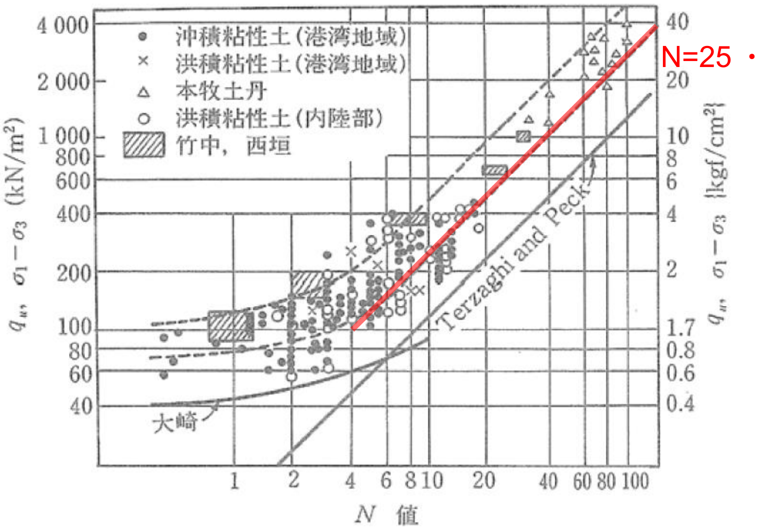
γt1: Unit volume weight of soil at a position shallower than the groundwater table (kN / m3)

γ ″ t2: Unit volume weight of soil at a position deeper than the groundwater table (kN / m3)

x: Depth from the ground surface (m)

hw: Depth of groundwater level (m)

Clay soil: c = 14 kN / m2 for N = 1 and 2, for which uniaxial compression test results are obtained, and c = 24 kN / m2 for N = 3. For other than N>4, it is estimated from the relationship between the N value and qu indicated in the N Soft Ground Measures Guidelines.



q_uと N 値との関係（奥村16)に加筆修正）

Source : Guideline for Design of Soft Soil for Earth Works in Road Project, Japan

Figure 5.2.33 Relationship between qu and N value

(3) Soil Modulus

The soil modulus in the Marikina river channel /MCGS and CAINTA/TAYTAY are shown in Table 5.2.39. Marikina River channel is linear structures with limited density of borings, but for MCGS, Cainta floodgate and Taytay Sluicegate, the boring test was conducted at the exact locations. The proposed unit volume weight is comparable between Marikina river channel and three gates, but as for the effective cohesion of the clay soil and the internal friction angle of the sandy soil, different estimation methods are used.

Effective cohesion of clay:

<u>Marikina River Channel</u>

Based on the results of the current and previous geological surveys, the same method which is used in the previous phase (Phase 1~3) is adopted (similar to the Terzaghi-Peck equation).

<u>MCGS, Cainta floodgate and Taytay sluicegate</u>

The empirical equation between qu and N value is used (c=25*N). (see Figure 5.2.33).

Internal friction angle of sandy soil:

<u>Marikina River Channel</u>

Based on the results of the present and previous geological surveys, the same method which is used in the previous phase (Phase 1~3) is adopted ($\phi = \sqrt{15*N} + 15$).

<u>MCGS, Cainta floodgate and Taytay sluicegate</u>

The latest equation is adopted. This equation takes into account the effect of effective loading pressure to N-value.

Table 5.2.39 Soil Modulus being Utilized

Structure	Geological Classification		Unit Weight of Wet Soil	Unit Weight of Submerged Soil	Angle of Internal Friction	Effective Cohesion	Reference
			γt	γw	ϕ	c	*Average N Value*
Name	Name	Symbol	(kN/m3)	(kN/m3)	(degree)	(kN/m2)	-
	Field Soil	F	18.0	8.0	30	0	15
River Structure	Alluvial Sand	As	17.5	7.5	N<9: 27 9 ≦ N: $15+\sqrt{15N}$ (Max 45)	0	10
	Diluvial Sand	Ds	19.0	9.0	N<9: 27 9 < N: $15+\sqrt{15N}$ (Max 45)	0	21
	Alluvial Clay	Ac	15.5	5.5	0	N= 6 x N 4<N≦8: 25 8<N≦15: 50 15<N≦30: 100 30<N : 200	8
	Diluvial Clay	Dc	18.0	8.0	0	N= 8 x N 4<N≦8: 50 8<N≦15: 100 15<N≦30: 180 30<N : 250	23
	Guadalupe Formation	GFw	16.5	6.5	20	200	64
		GFf	17.0	7.0	30	1,000	
MCGS	Alluvial Sand	As	17.5	7.5	$\emptyset = 4.8 \times \log N_0$ $N_1 = 170\,x\,N/(\delta, +70)$	0	10
	Guadalupe Formation	GFf	17.0	7.0	30	For Retaining Wall 1000 For Main Body 3,500	122
Cainta and Taytay	Field Soil	F	18.0	8.0	30	0	15
	Alluvial Sand	As2	17.0 – 20.0	7.0 – 10.0	25 – 38 $\emptyset = 4.8 \times \log N_0$ $N_1 = 170\,x\,N/(\delta, +70)$	0	20
		As1	20.0 – 21.0	10.0 – 11.0	33 – 39 $\emptyset = 4.8 \times \log N_0$ $N_1 = 170\,x\,N/(\delta, +70)$	0	29
	Diluvial Sand	Ds1	19.0 – 21.0	9.0 – 11.0	40	0	50
	Alluvial Clay	Ac2	15.0 – 18.0	5.0 – 8.0	0	N =1, 2 : 14 N=3 : 24 N ≧ 4 : 130 – 200	6
		Ac1	15.0 – 18.0	5.0 – 8.0	0	N = 1~2 : 14 N= 3 : 24 N ≧ 4 100 – 250	13
	Diluvial Clay	Dc1	18.0 - 19.0	8.0 – 9.0	0	220 – 620	22

Source: Study Team

5.2.4 Appendix

The following materials are shown at the end of this report as Appendix.

Table 5.2.40 List of Documents shown in Appendix

Number	Document Title
5-1.	LOCATION MAP OF BORINGS
5-2.	GEOLOGICAL SECTION OF MARIKINA RIVER
5-3.	BORING LOGS
5-3-1.	BORING LOGS (PHASE Ⅳ)
5-3-2.	BORING LOGS (PHASE Ⅳ) WITH CORE PHOTOS
5-3-3.	BORING LOGS（PHASE Ⅰ）
5-4.	N-value and geological Tables
5-4-1.	N-value and geological Tables (PHASE Ⅳ)
5-4-2.	N-value and geological Tables (PHASE Ⅰ)
5-5.	IN-SITU SURVEY PHOTO

Source: Study Team