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-1C
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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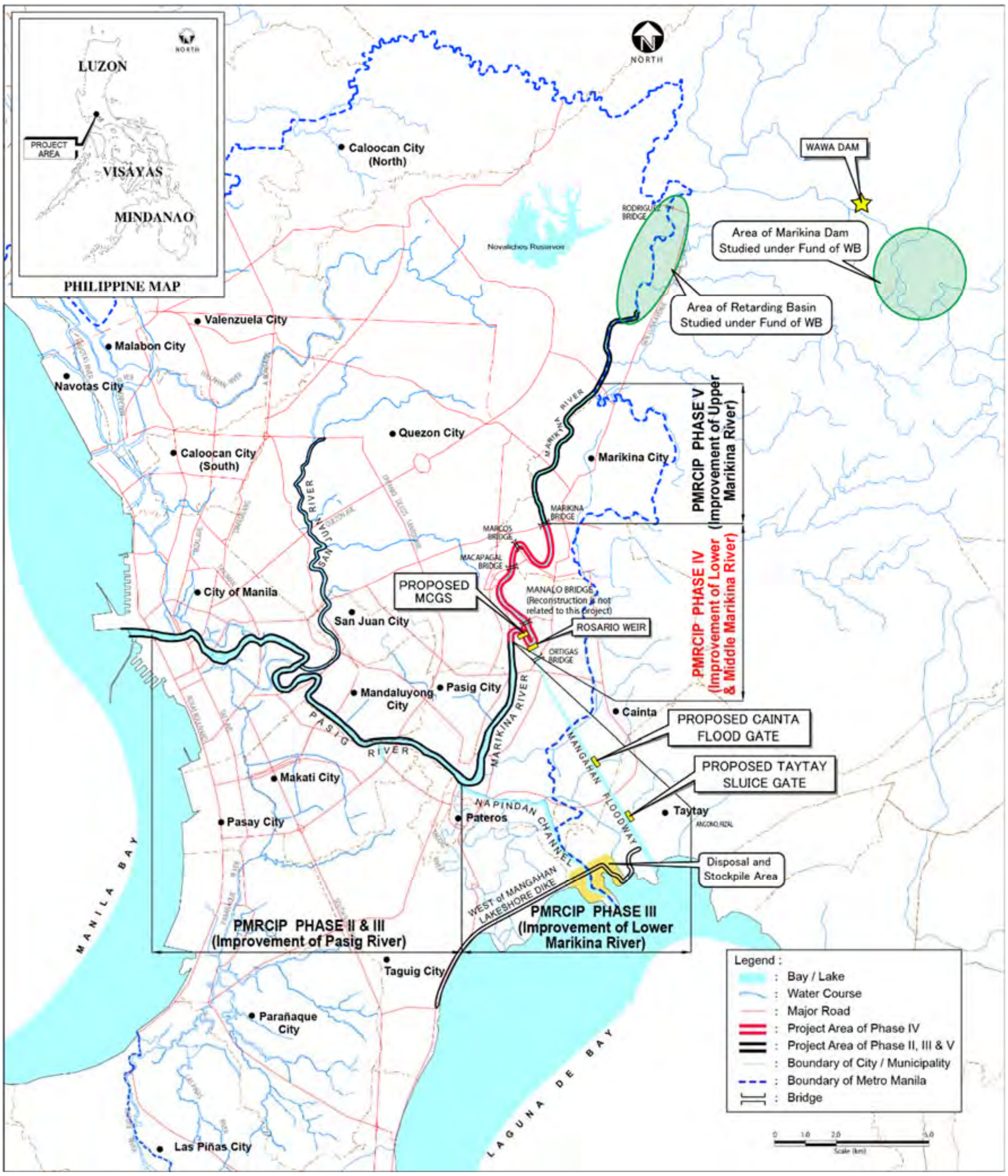

PROJECT LOCATION MAP

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

FINAL REPORT (PRIOR RELEASE VERSION) VOL.-1C MAIN REPORT

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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

7.4 Detailed Design of Cainta Floodgate

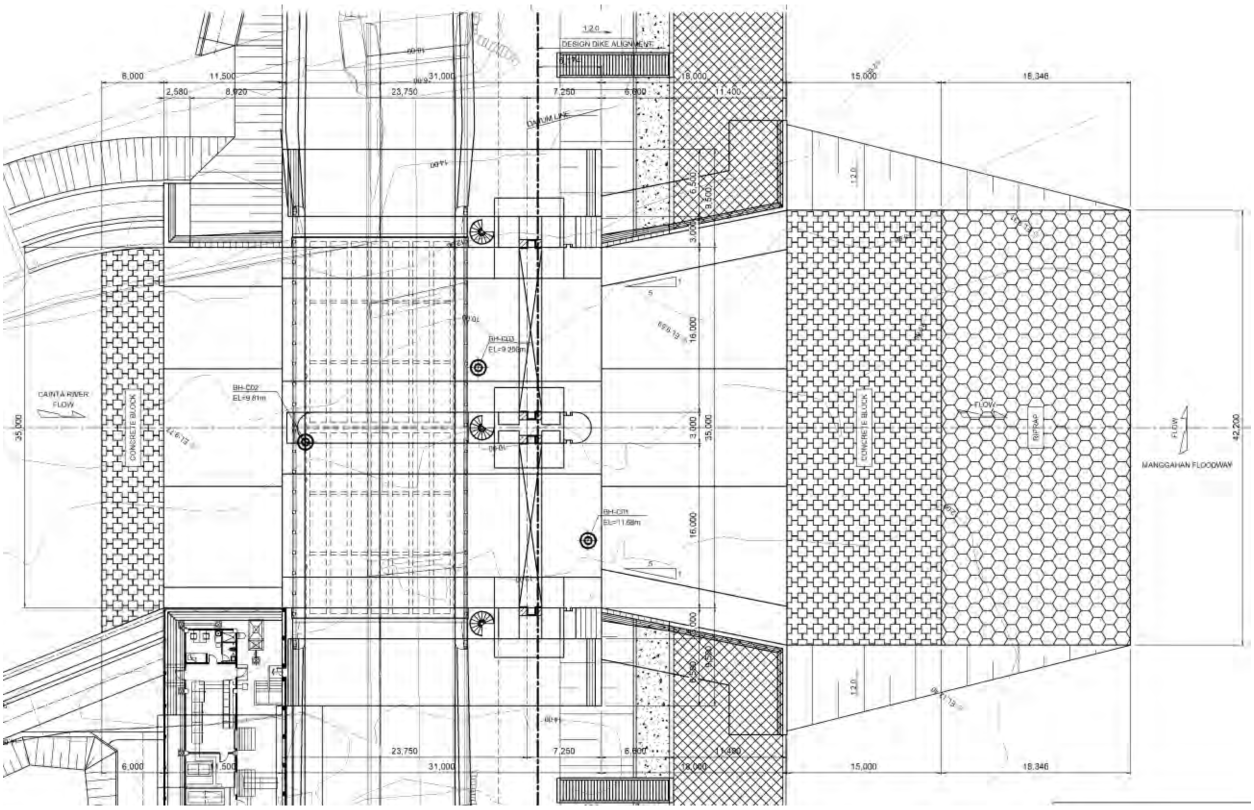
7.4.1 Overview of Detailed Design of Cainta Floodgate

The detailed design was carried out based on the specifications set in **6.4.1 Basic Design of Cainta Floodgate**. In the detailed design, the following examination is carried out.

- Structural design and Level 2 seismic design of civil engineering facilities (Foundation works, man body works, apron, wing walls, etc.)
- Determination of structural design and specifications of gate facility
- Designing temporary equipment (Temporary cofferdam and access slope for construction)
- Detailed examination and specification determination of information and telecommunications facilities and electrical facilities

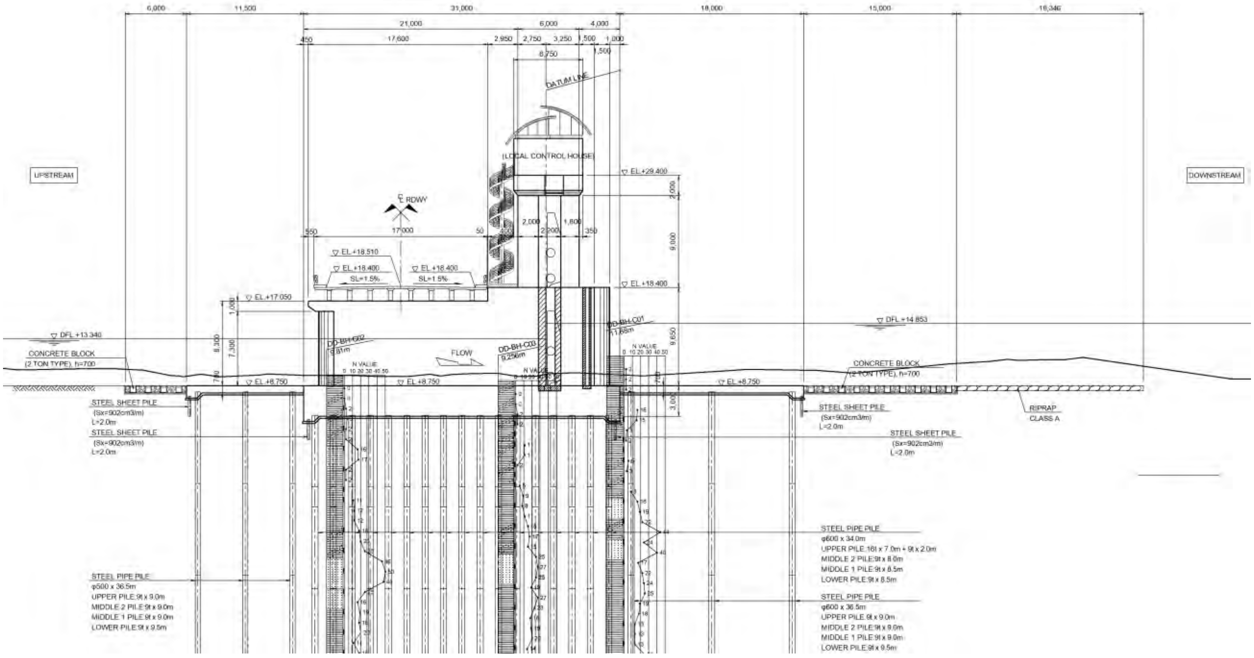
7.4.2 Civil Engineering Design

Civil engineering facility design such as stability calculation of foundation work, structural calculation of main body work, L2 seismic design, etc. is carried out for the determined facility specifications in the basic design of the Cainta Floodgate. From **Figure 7.4.1** to **Figure 7.4.5** shows the structural drawing of the Cainta Floodgate.



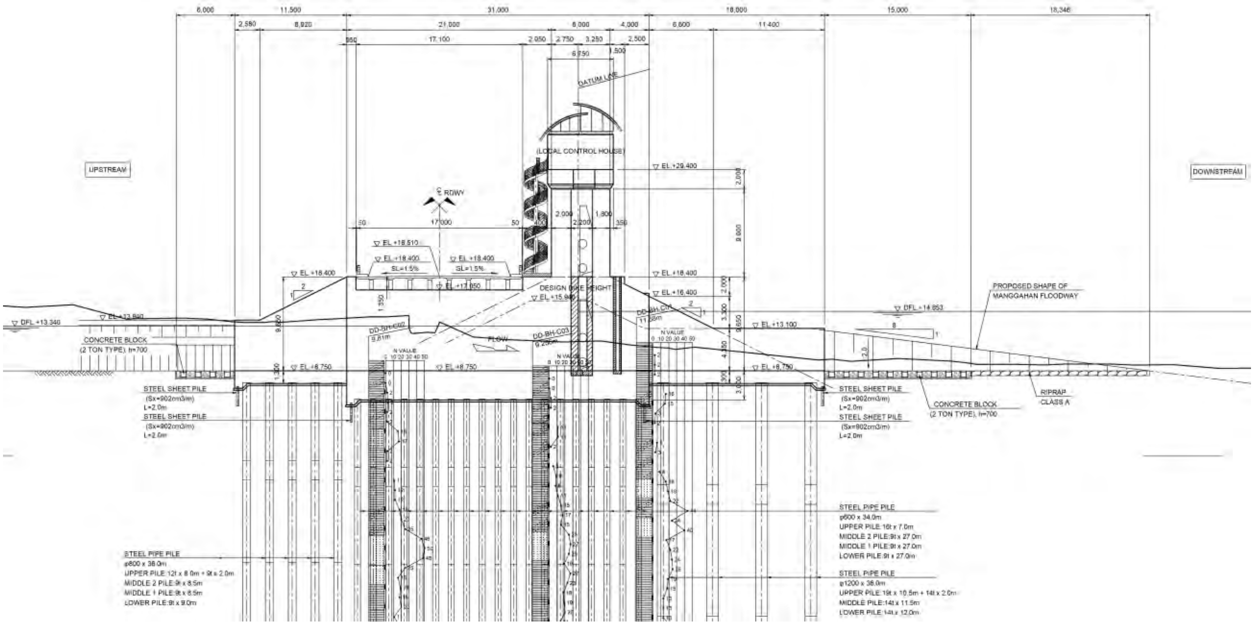
Source: Study team

Figure 7.4.1 Layout Plan



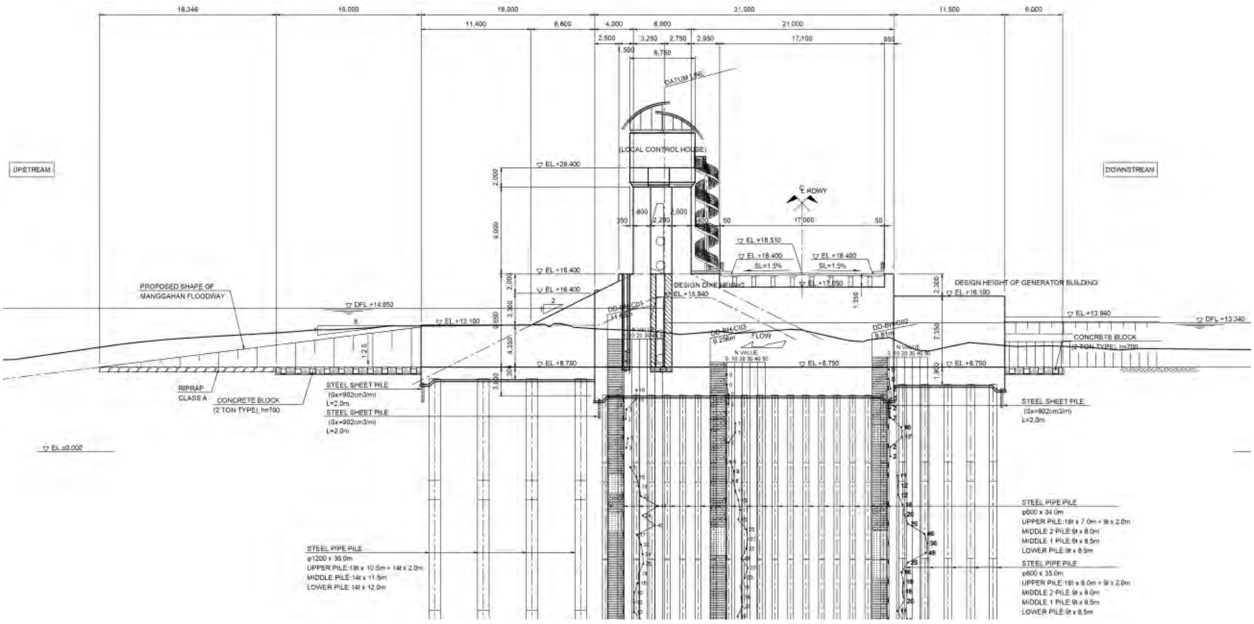
Source: Study team

Figure 7.4.2 Profile (Center pier)



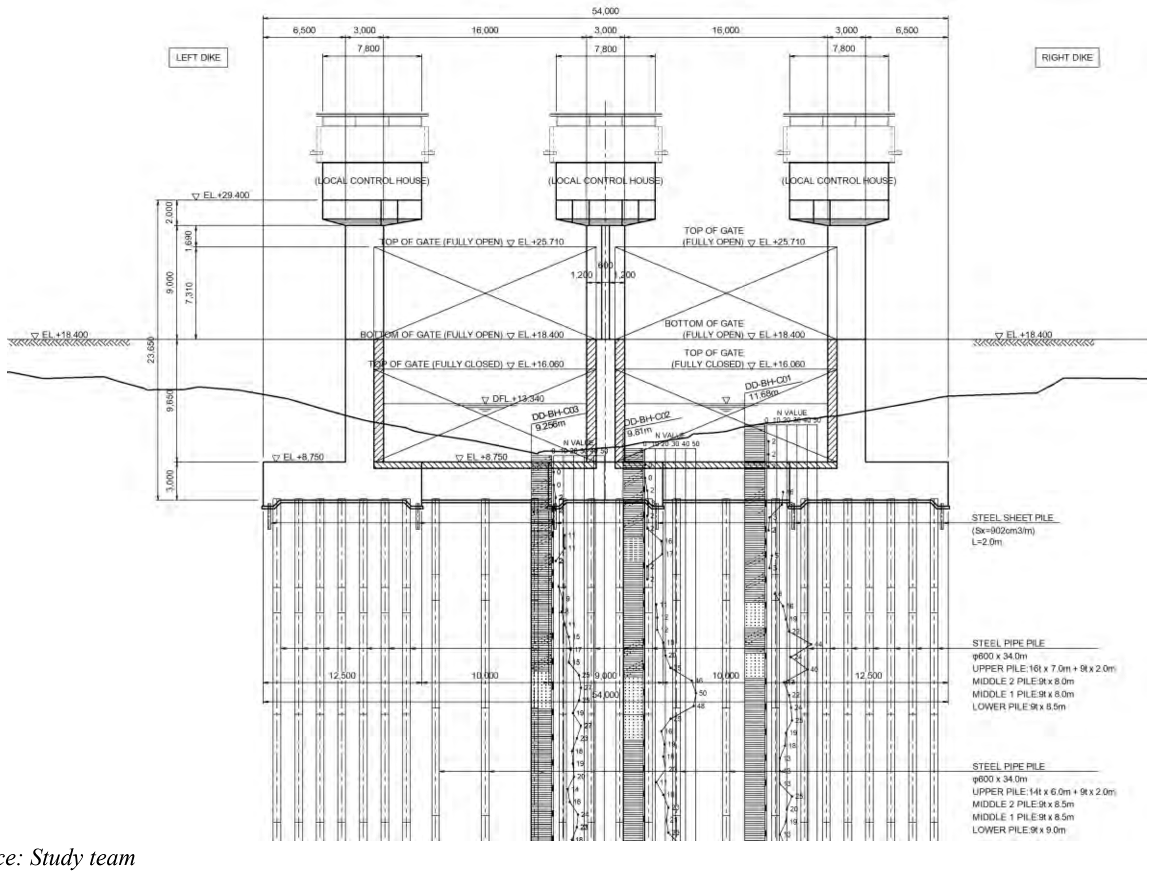
Source: Study team

Figure 7.4.3 Profile (Left bank pier)



Source: Study team

Figure 7.4.4 Profile (Right Bank Pier)



Source: Study team

Figure 7.4.5 Front View

7.4.2.1 Foundation Work

(1) Examination of Consolidation Settlement

1) Calculation Method

The amount of consolidation settlement of the cohesive soil layer is determined by the following equation for each cohesive soil layer divided using the e-log P curve.[1]

$$S_c = \frac{e_0 - e_1}{1 + e_0} \cdot H$$

Here,

S_c : Total consolidation settlement (m)
e_0 : Initial void ratio of cohesive soil
e_1 : Initial void ratio after embankment; it is obtained by applying $p_0 + \Delta p$ of the central depth of the compressive stratum to the e~logp curve obtained in the consolidation examination.
H : Thickness of compressive stratum (m)
p_0 : Effective soil cover pressure before embankment (kN/m2)
Δp : Increased compressive stress (kN/m2)

The increased stress in the ground (Δ p) is calculated with reference to the following standards.

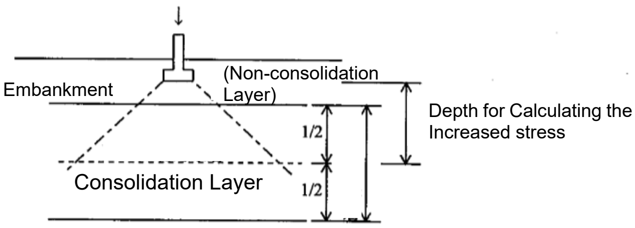


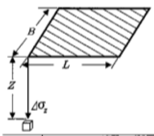
Figure 5.5.6 Depth for Calculating the Increased stress

Source: Small-Scale Building Foundation Design Guidelines, Japan

$$\Delta\sigma = \frac{q}{2\pi}\left\{\frac{mn}{\sqrt{m^2+n^2+1}} \cdot \frac{m^2+n^2+2}{(m^2+1)(n^2+1)} + \sin^{-1}\frac{mn}{\sqrt{(m^2+1)(n^2+1)}}\right\}$$

$$\Delta\sigma = q f_B(m,n) \qquad (5.5.4)$$

Here, Δp : Increased compressive stress (kN/m2)
q : Pressure of ground contact (kN/m2)
$f_B(m,n)$: Function about distribution load(m=B/z, n=L/z, obtained from the following table)



m または n	n または m															
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	2.0	3.0	∞
0.1	0.005	0.009	0.013	0.017	0.020	0.022	0.024	0.026	0.027	0.028	0.029	0.030	0.031	0.031	0.032	0.032
0.2	0.009	0.018	0.026	0.033	0.039	0.043	0.047	0.050	0.053	0.055	0.057	0.059	0.060	0.061	0.062	0.062
0.3	0.013	0.026	0.037	0.047	0.056	0.063	0.069	0.073	0.077	0.079	0.083	0.086	0.087	0.089	0.090	0.090
0.4	0.017	0.033	0.047	0.060	0.071	0.080	0.087	0.093	0.098	0.101	0.106	0.109	0.111	0.113	0.115	0.115
0.5	0.020	0.038	0.056	0.071	0.084	0.095	0.103	0.110	0.116	0.120	0.126	0.130	0.143	0.135	0.137	0.137
0.6	0.022	0.043	0.063	0.080	0.095	0.107	0.117	0.125	0.131	0.136	0.143	0.147	0.150	0.153	0.155	0.156
0.7	0.024	0.047	0.069	0.087	0.103	0.117	0.128	0.137	0.144	0.149	0.157	0.162	0.165	0.169	0.171	0.172
0.8	0.026	0.050	0.073	0.093	0.110	0.125	0.137	0.146	0.154	0.160	0.168	0.174	0.177	0.181	0.184	0.185
0.9	0.027	0.053	0.077	0.098	0.116	0.131	0.144	0.154	0.162	0.168	0.178	0.184	0.187	0.192	0.195	0.196
1.0	0.028	0.055	0.079	0.101	0.120	0.136	0.149	0.160	0.168	0.175	0.185	0.191	0.195	0.200	0.203	0.205
1.2	0.029	0.057	0.083	0.106	0.126	0.143	0.157	0.168	0.178	0.185	0.196	0.203	0.207	0.212	0.216	0.218
1.4	0.030	0.059	0.086	0.109	0.130	0.147	0.162	0.174	0.184	0.191	0.203	0.210	0.215	0.221	0.225	0.227
1.6	0.031	0.060	0.087	0.111	0.132	0.150	0.165	0.177	0.187	0.195	0.207	0.215	0.220	0.226	0.234	0.233
2.0	0.031	0.061	0.089	0.113	0.135	0.153	0.169	0.181	0.192	0.200	0.212	0.221	0.226	0.232	0.238	0.240
3.0	0.032	0.062	0.090	0.115	0.137	0.156	0.174	0.184	0.195	0.203	0.216	0.225	0.231	0.238	0.244	0.247
∞	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196	0.205	0.218	0.227	0.233	0.240	0.247	0.250

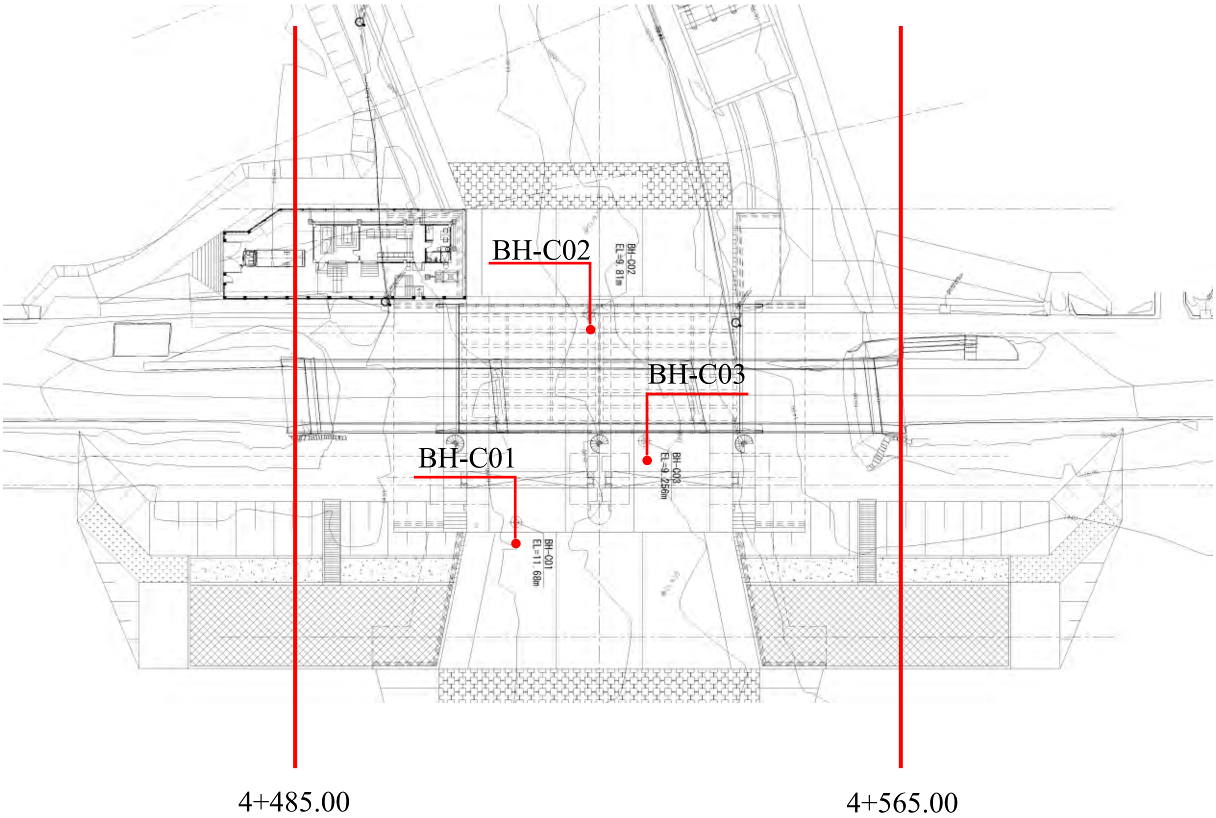
Source: Small-Scale Building Foundation Design Guidelines

[1] River earthwork manual 3.2. 3 Sinking of soft ground

2) Consolidation Settlement

(a) Calculated cross section

For consolidation settlement, the adjacent cross section of the upstream and downstream flow at the floodway installation location is used, and the following two cross sections are used.

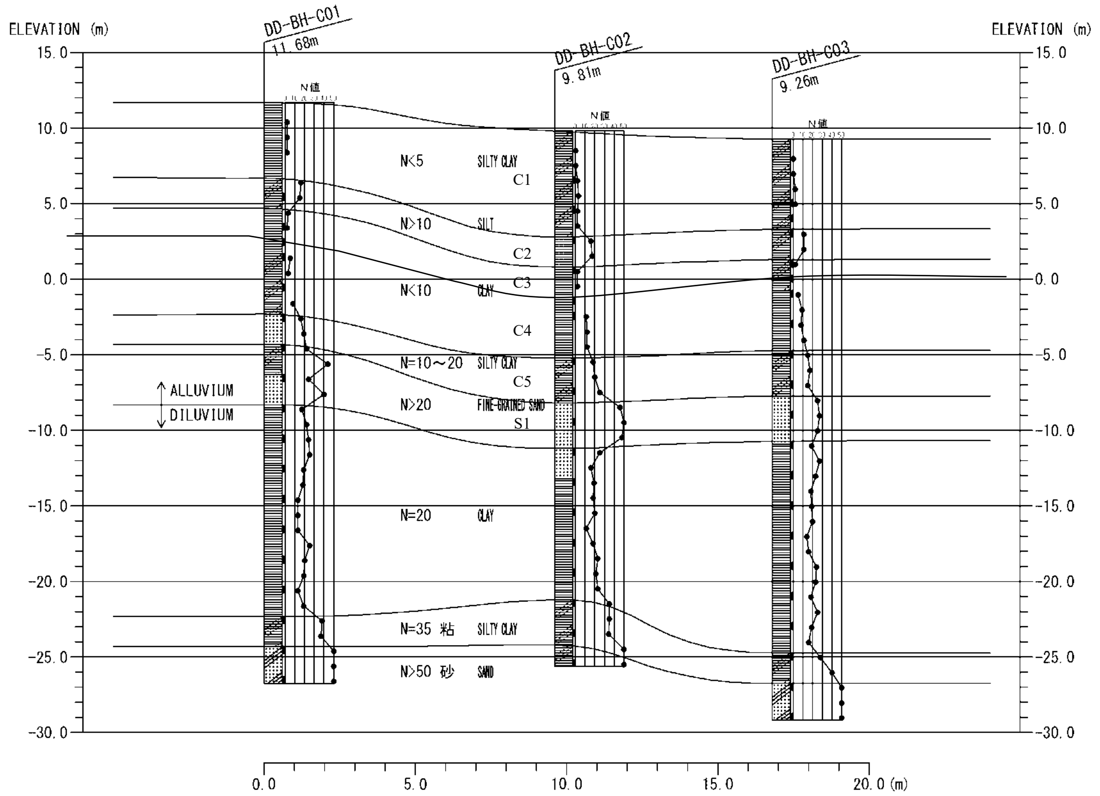


Source: Study team

Figure 7.4.6 Section for Calculation

(b) Soil Condition

As for soil conditions, soil properties set in the basic design are adopted. The soil profile is shown in **Figure 7.4.7**.



Source: Study team

Figure 7.4.7 Soil Profile

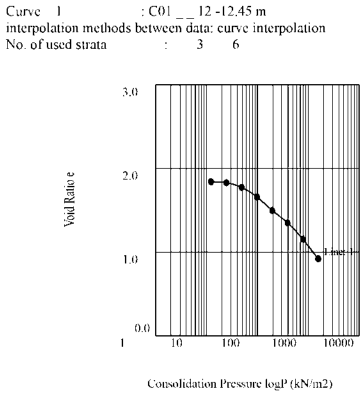
(c) Consolidation Target Layer

For the cohesive soil layer of alluvium is the target, and it is considered that the cohesive soil layer with N ≧ 10 will not settle.[2] The consolidation target layers in the respective calculation cross sections are C1, C3, and C4.

[2] Outline of Sluiceway Design (Draft), Kyushu Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, p. 75

(d) Consolidation Constant

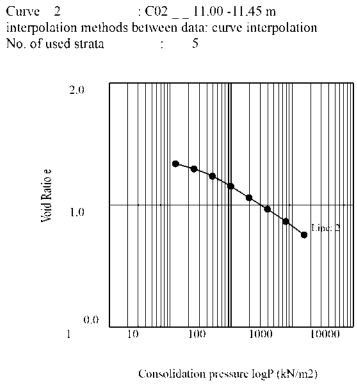
The consolidation constant (e-log P curve) is shown in **Figure 7.4.8** and **Figure 7.4.9**. Test values were used because consolidation tests were conducted on C3 and C4. Since there is no test value for C1, the test result for C3 was adopted in consideration of safety.



No.	1	2	3	4	5	6	7	8
Consolidation pressure	12.25	24.50	49.00	98.00	196.00	392.00	784.00	1568.00
void ratio e	1.841	1.829	1.773	1.658	1.498	1.349	1.153	0.920

Source: Study team

Figure 7.4.8 Consolidation Curve Diagram (C3)



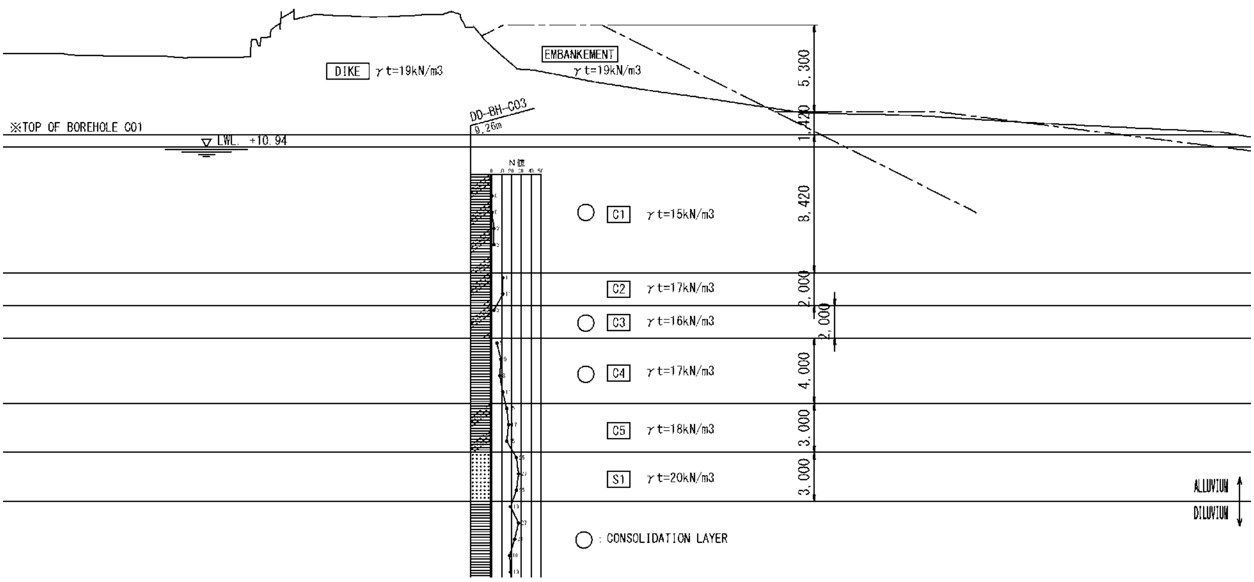
No.	1	2	3	4	5	6	7	8
Consolidation pressure	12.25	24.50	49.00	98.00	196.00	392.00	784.00	1568.00
void ratio e	1.337	1.295	1.236	1.152	1.059	0.965	0.865	0.754

Source: Study team

Figure 7.4.9 Consolidation Curve Diagram (C4)

(e) Computational Model

The calculation model is shown in below.



Source: Study team

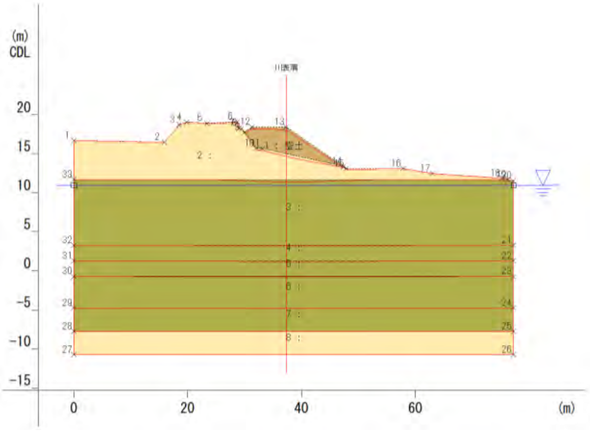
Figure 7.4.10 Calculation Model (4 + 565.00)

(f) Consolidation Settlement

The calculation results are shown below. 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.

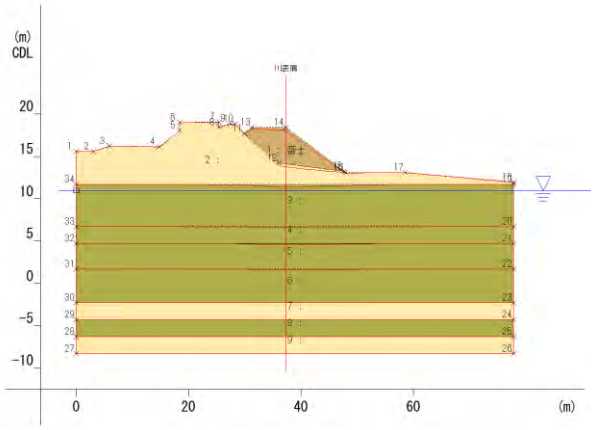
Station: 4 + 565 settlement: 0.33 m



Source: Study team

Figure 7.4.11 Consolidation Settlement Diagram (STA.4 + 565)

Station: 4 + 485 settlement: 0.34 m

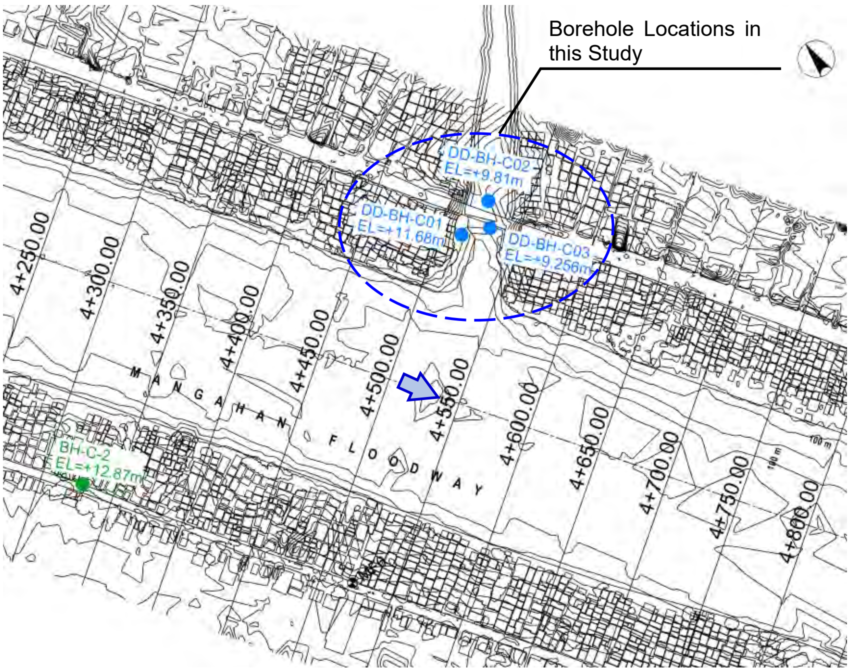


Source: Study team

Figure 7.4.12 Consolidation Settlement Diagram (STA.4 + 485)

(2) Study of Liquefaction

Liquefaction Analysis is carried out for the borehole locations DD-BH-C01, DD-BH-C02 and DD-BH-C03 in the Cainta Floodgate site.



Source: Study team

Figure 7.4.13 Geological Survey Site

1) Foundation Surface and Ground Surface in Seismic design

In the "Performance Based Seismic Design Criteria for River Structures, Japan", the bedrock surface for seismic performance verification is defined as the upper surface of a sufficiently strong soil layer with N value of 25 or more (Shear elastic wave velocity of 300 m/s or more for cohesive soil)[3]。

Since the C7 layer is a strong ground with N value of 25 or more, the upper surface of this layer is set to the foundation surface for seismic performance verification.

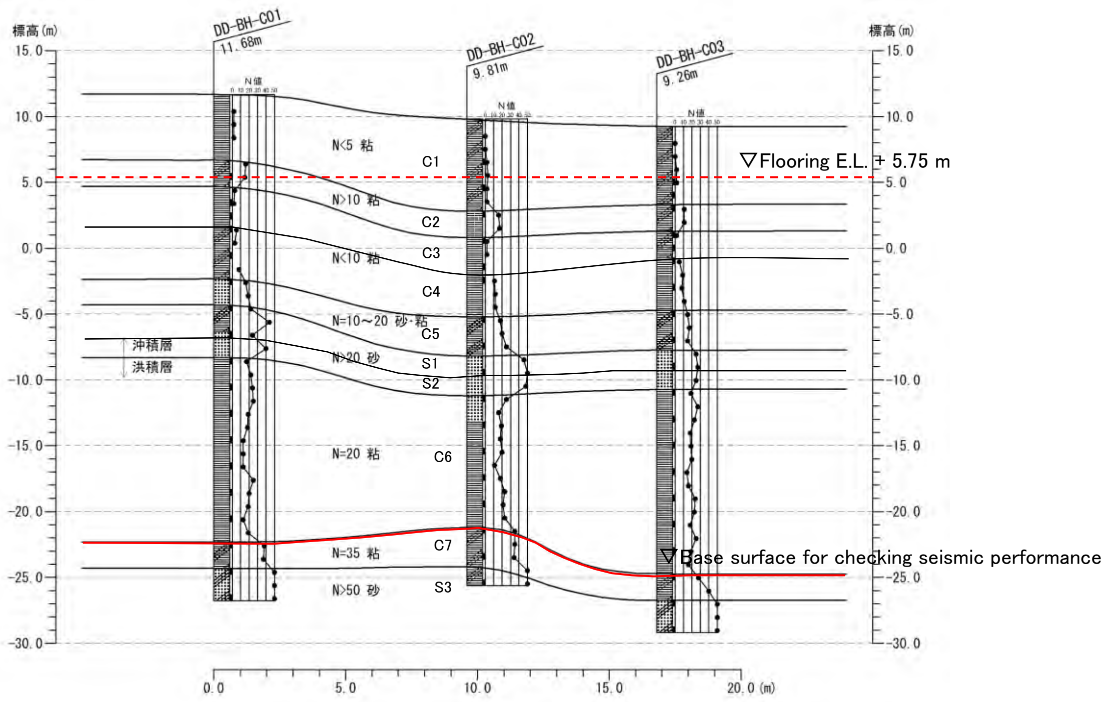


Figure 7.4.14 Geological Cross-Section

2) Design Horizontal Seismic Coefficient

The design horizontal seismic intensity for L1 earthquake ground motion is 0.2. For L2 earthquake ground motion, 0.47 applies.

(a) L1 Earthquake Ground Motion

The design horizontal seismic intensity in the L1 seismic motion is 0.2 from Chapter 6 assuming the case of a medium-scale earthquake. The configuration details are given in 11.4.2.2.

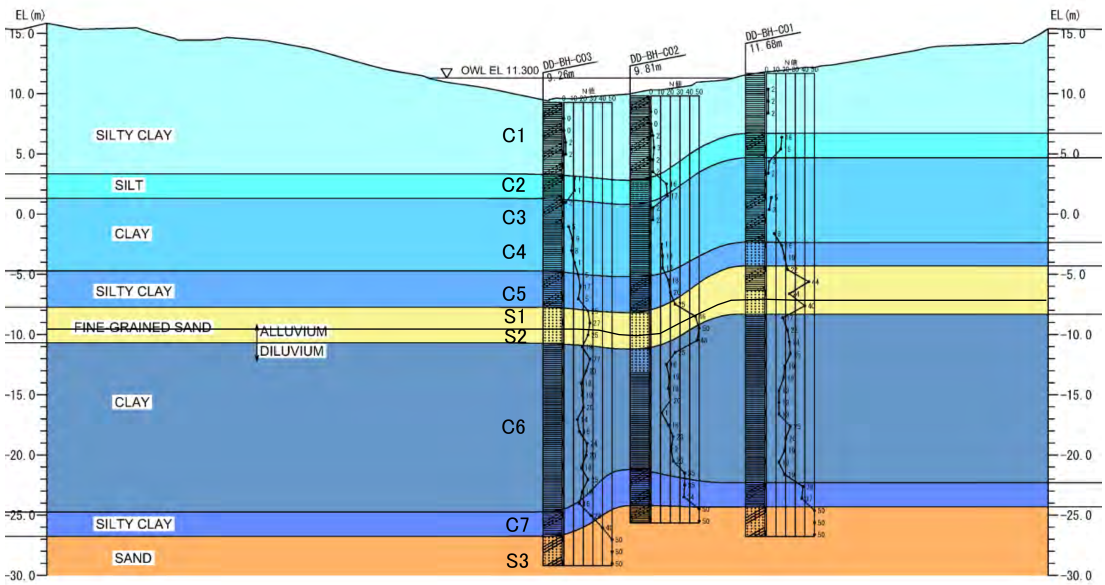
(b) L2 earthquake ground motion

The design horizontal seismic intensity of L2 earthquake ground motion is 0.47. For detailed calculation methods, refer to P 7-659.

3) Soil Condition

In this study, the soil properties arranged in the basic design are adopted based on the results of drilling surveys of DD-BH-C01, DD-BH-C02, and DD-BH-C03. The soil profile is shown in **Figure 7.4.15**.

[3] 2012 Performance Based Seismic Design Criteria for River Structures, Japan I Common Edition



Source: Study team

Figure 7.4.15 Geological Cross-Section

4) Liquefaction Analysis

(a) Extraction of Layer for Liquefaction Analysis

Liquefaction Analysis should be performed for the soils listed below in the BSDS. In this study, Liquefaction Analysis is carried out for the layer which satisfies the following conditions. The target layers that meet the conditions are shown in **Table 7.4.1**. As a result of the determination, the C1, C2, S1, and S2 layers become Liquefaction Analysis target layers.

1) Saturated soil layer with depth less than 20 m below the ground surface and having ground water level higher than 10 m below the ground surface.

2) Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index, I_P, less than 15, even if FC is larger than 35%.

3) Soil layer having a mean particle size (D_{50}) of less than 10 mm and a particle size at 10% passing (D_{10}) (on the grading curve) is less than 1 mm.

Source: BSDS, DPWH, P6 -3, provisional translation by the Mission

Table 7.4.1 Extraction of the Liquefaction Analysis Target Layer

Stratum	Geological survey	Fine fraction content fc (%)	Plasticity index Ip	Mean particle size D 50 (mm)	10% particle size D 10 (mm)	Liquefaction Analysis target layer
C1	DD-BH-C01	-	-	-	-	○
	DD-BH-C02	93	15	0.017	0.004	○
	DD-BH-C03	93	21	0.024	0.008	×
C2	DD-BH-C01	85	13	0.02	0.007	○
	DD-BH-C02	83	-	-	-	○
	DD-BH-C03	90	16	-	-	×
C3	DD-BH-C01	80	45	-	-	×
	DD-BH-C02	86	31	0.024	0.011	×
	DD-BH-C03	70	17	-	-	×
C4	DD-BH-C01	89	52	-	-	×
	DD-BH-C02	90	53	-	-	×
	DD-BH-C03	96	65	-	-	×

Stratum	Geological survey	Fine fraction content fc (%)	Plasticity index Ip	Mean particle size D 50 (mm)	10% particle size D 10 (mm)	Liquefaction Analysis target layer
C5	DD-BH-C01	65	25	-	-	×
	DD-BH-C02	77	28	0.28	0.006	×
	DD-BH-C03	60	30	0.18	-	×
C6	DD-BH-C01	90	45	-	-	×
	DD-BH-C02	91	30	-	-	×
	DD-BH-C03	94	45	0.15	-	×
S1	DD-BH-C01	20	-	0.2	-	○
	DD-BH-C02	18	-	-	-	○
	DD-BH-C03	13	-	0.46	-	○
S2	DD-BH-C01	23	-	0.35	0.012	○
	DD-BH-C02	22	-	2.7	-	○
	DD-BH-C03	-	-	-	-	-

: Liquefaction Analysis target layer

Source: Study team

(b) Liquefaction Analysis Formula

Resistivity against liquefaction for soil layers for which liquefaction is evaluated with F_L for each of the Level 1 earthquake ground motions and the Level 2 earthquake ground motions, the value is calculated by the following formula. Soil layers with this value of 1.0 or less shall be deemed to be liquefied.[456]. The determination method is the same for Specifications for Highway Bridges, Japan and BSDS. However, in this study, considering the local ground conditions and past earthquakes, external forces are set according to the BSDS. The external force of Level 2 earthquake ground motion is calculated by k_{hgL}= Fpga PGA in BSDS

$$F_L = R/L$$

$$R = c_w R_L$$

$$L = r_d k_{hg} \sigma_v / \sigma'_v$$

$$r_d = 1.0 - 0.015x$$

$$\sigma_v = r_{t1} h_w + r_{t2}(x - h_w)$$

$$\sigma'_v = r_{t1} h_w + r'_{t2}(x - h_w)$$

$$C_w = \begin{cases} 1.0\ (R_L \le 0.1) \\ 3.3R_L + 0.67\ (0.1 < R_L \le 0.4) \\ 2.0\ (0.4 < R_L) \end{cases}$$

Here,

F_L : Liquefaction resistance factor.
R : Dynamic shear strength ratio.
L : Seismic shear stress ratio.
c_w : Modification factor on earthquake ground motion.
R_L : Cyclic triaxial shear stress ratio to be obtained from Equation below.
r_d : Reduction factor of seismic shear stress ratio, in terms of depth.
k_{hgL} : Design horizontal seismic coefficient at the ground surface for Level 2 EGM.
F_{pga} : Site coefficient for peak ground acceleration specified in Article 3.5.3.
PGA : Peak ground acceleration coefficient on rock, as given in Article 3.6.

[4] Seismic performance evaluation guideline and explanation of river structures I. Common edition p 26 (Note: Only the parts corresponding to Level 2 earthquake ground motions)
[5] Road bridge specifications and explanations V Seismic design section p. 141
[6] BSDS 6.2. 3 Assessment of Soil Liquefaction p6 -2

σ_v : Total overburden pressure, (kN/m2).
σ'_v : Effective overburden pressure, (kN/m2).
x : Depth from the ground surface, (m).
r_{t1} : Unit weight of soil above the ground water level, (kN/m3).
r_{t2} : Unit weight of soil below the ground water level, (kN/m3).
r'_{t2} : Effective unit weight of soil below the ground water level, (kN/m3).
h_w : Depth of the ground water level, (m)

Also, Cyclic triaxial shear stress ratio R_L shall be calculated by the following formula.

$$R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (14 \le N_a) \end{cases}$$

Here,

(For Sandy Soil)

$$N_a = c_1 N_1 + c_2$$

$$N_1 = 170N/(\sigma'_v + 70)$$

$$c_1 = \begin{cases} 1 & (0\% \le FC < 10\%) \\ (FC + 40)/50 & (10\% \le FC < 60\%) \\ FC/20 - 1 & (60\% \le FC) \end{cases}$$

$$c_2 = \begin{cases} 0 & (0\% \le FC < 10\%) \\ (FC - 10)/18 & (10\% \le FC) \end{cases}$$

(For Gravelly Soil)

$$N_a = \{1 - 0.36log_{10}(D_{50}/2)\}N_1$$

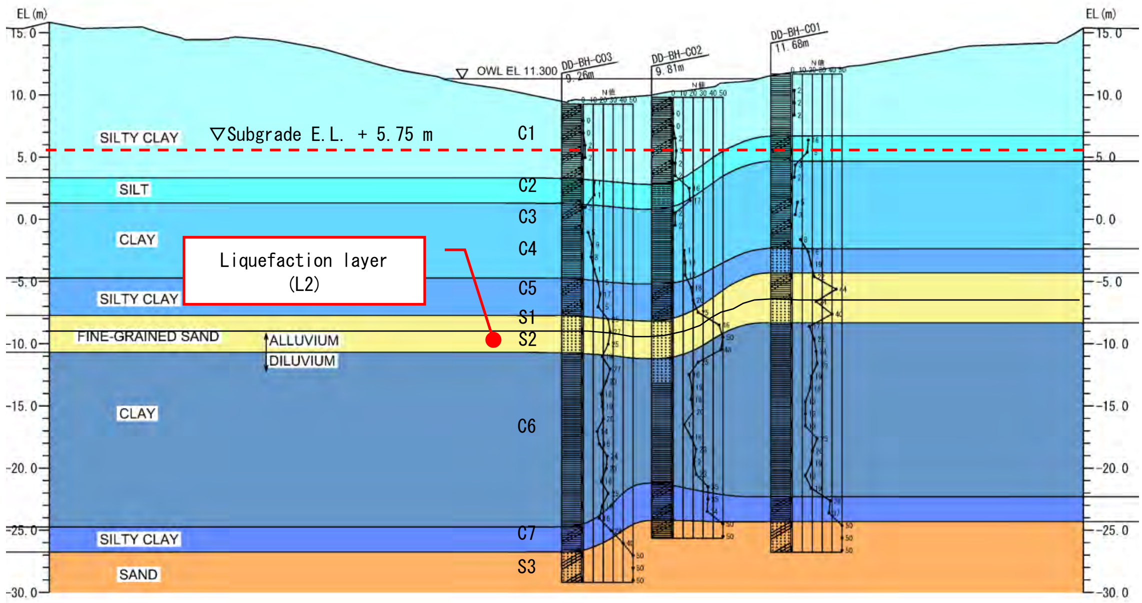
Here,

R_L : Cyclic triaxial shear stress ratio.
N : N-value obtained from the standard penetration test.
N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m2.
N_a : Modified N value considering the effects of grain size.
c_1, c_2 : Modification factors of N value on fine content.
FC : Fine content, (%) (percentage by mass of fine soil passing through the 75 ・m mesh).
D_{50} : Mean grain diameter, (mm).

(c) Liquefaction Analysis Result

For each ground of DD-BH-C01, DD-BH-C02, and DD-BH-C03, 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.

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. The following pages show the details of the Liquefaction Analysis results for each earthquake motion.



Source: Study team

Figure 7.4.16 Liquefied Layer

(d) Liquefaction Analysis Result for L1 Earthquake Ground Motion

From the next page, the results of Liquefaction Analysis for the design horizontal seismic intensity of L1 earthquake ground motion in each stratum and the results of Liquefaction Analysis at each investigation site are shown. The F_L value was calculated using the formula for the L2 -1 earthquake motion shown in "Performance Based Seismic Design Criteria for River Structures, Japan".

List of Liquefaction Analysis results for L1 Earthquake ground motion is shown in **Table 7.4.2**. The results of Liquefaction Analysis at each investigation site are shown in **Table 7.4.3**, **Table 7.4.4** and **Table 7.4.5**. In the C1 layer, the F_L value became 1.0 or less in a part of DD-BH-C02, but no influence is expected because it was located above the subgrade level when the Cainta Floodgate was constructed. Regarding the S1 layer, the F_L value became 1.0 or less at 1 point of DD-BH-C01, but the FL value became 1.0 or more at all other points except for 1 point of the S1 layer of 3 points where the geological survey was conducted. Hence, the S1 layer is not regarded as the liquefaction generation layer. Therefore, the liquefaction layer was not confirmed against the L1 earthquake motion.

Table 7.4.2 Liquefaction Analysis Result List (L1 Earthquake Ground Motion)

Stratum	Geological survey	Depth (m)	N-value	F_L value	Remarks
C1	DD-BH-C01	1.3	2	9.445	Above subgrade level
		2.3	2	7.342	Above subgrade level
		3.3	2	5.834	Above subgrade level
	DD-BH-C02	1.3	0	0.422	Liquefaction occurs, but above the subgrade level
		2.3	0	0.429	Liquefaction occurs, but above the subgrade level
		3.3	2	5.834	Above subgrade level
		4.3	3	26.624	Above subgrade level
		5.3	2	3.909	
		6.3	2	3.289	
	DD-BH-C03	1.3	0	Out of Target	Not calculated due to FC > 35% and Ip > 15

Stratum	Geological survey	Depth (m)	N-value	F_L value	Remarks
		2.3	0	Out of Target	Not calculated due to FC > 35% and Ip > 15
		3.3	2	Out of Target	Not calculated due to FC > 35% and Ip > 15
		4.3	2	Out of Target	Not calculated due to FC > 35% and Ip > 15
C2	DD-BH-C01	5.3	16	23407.1	Above subgrade level
		6.3	15	13040.7	
	DD-BH-C02	7.3	16	13276.6	
		8.3	17	13387.7	
	DD-BH-C03	6.3	11	Out of Target	Not calculated due to FC > 35% and Ip > 15
		7.3	11	Out of Target	Not calculated due to FC > 35% and Ip > 15
S1	DD-BH-C01	14.3	16	0.906	Liquefaction occurs.
		15.3	19	1.206	
	DD-BH-C02	18.3	46	67.559	
		19.3	50	82.347	
		20.3	48	48.824	
	DD-BH-C03	17.3	25	1.224	
		18.3	27	1.363	
		19.3	25	1.048	
S2	DD-BH-C01	18.3	24	2.594	
		19.3	40	40.011	
	DD-BH-C02	21.3	25	Out of Target	Depth is 20 m or more, so it is not evaluated.
		22.3	16	Out of Target	Depth is 20 m or more, so it is not evaluated

Source: Study team

Table 7.4.3 Liquefaction Analysis Result (DD-BH-C01, L1 Earthquake Ground Motion)

Name of the point DD-BH-C01

PL Value 0.215

Unit volume weight of water 10.0 (kN/m3)
Load 0.0 (kN/m2)
Groundwater level 0.00 (m)

Design horizontal seismic coefficient 0.20

Soil Depth (m) D	Layer Thickness(m) h	Wet Weight (kn/m3) γt	Saturated Weight (kn/m3) γsat	Dynamic Shear Strength Ratio x	Soil Category	Soil Type	Observed N value N	Effective Loading Pressure (kN/m2) σv'	Fine Fraction Content (%) Fc	Particle Size (mm) D50	Target Layer Of Analysis Or Not	Earthquake Dynamic Coefficient Cw	Repeated Triaxial Strength Ratio RL	Dynamic Shear Strength Ratio R	Seismic Response Ratio L	Liquefaction Resistance FL	
				1.3	沖積	粘性土	2.00	6.5	93.00	0.000		1.00	5.557	5.557	0.558	9.415	C1
				2.3	沖積	粘性土	2.00	11.5	93.00	0.000		1.00	4.253	4.253	0.579	7.342	C1
				3.3	沖積	粘性土	2.00	16.5	93.00	0.000		1.00	3.827	3.827	0.570	6.831	C1
5.00	5.00	15.0	15.0	5.3	沖積	粘性土	16.00	27.4	85.00	0.020		1.00			0.540		C2
				6.3	沖積	粘性土	15.00	35.4	86.00	0.020		1.00			0.553		▽Subgrade Level C2
7.00	2.00	18.0	18.0	7.3	沖積	粘性土	2.00	42.0	80.00	0.000		0.00	0.300	0.000	0.000	**2	C3
				8.3	沖積	粘性土	2.00	49.5	80.00	0.000		0.00	0.300	0.000	0.000	**2	C3
10.00	2.00	16.5	16.5	10.3	沖積	粘性土	5.00	62.5	89.00	0.000		0.00	0.300	0.000	0.000	**2	C3
				11.3	沖積	粘性土	3.00	68.9	89.00	0.000		0.00	0.300	0.000	0.000	**2	C4
				13.3	沖積	粘性土	8.00	81.9	89.00	0.000		0.00	0.300	0.000	0.000	**2	C4
14.00	4.00	16.5	16.5	14.3	沖積	砂質土	16.00	89.5	26.60	0.200		1.00	0.370	0.370	0.458	0.806	S1
				15.3	沖積	砂質土	19.00	99.5	26.60	0.200		1.00	0.472	0.472	0.391	1.206	S1
16.00	2.00	20.0	20.0	16.3	沖積	粘性土	22.00	108.9	65.00	0.000		0.00	0.300	0.000	0.000	**2	C5
				17.3	沖積	粘性土	44.00	116.9	65.00	0.000		0.00	0.300	0.000	0.000	**2	C5
18.00	2.00	18.0	18.0	18.3	沖積	砂質土	24.00	125.5	23.00	0.350		1.00	0.925	0.925	0.357	2.594	S2
				19.3	沖積	砂質土	40.00	135.5	23.00	0.350		1.00	13.784	13.784	0.345	40.011	S2
20.00	2.00	20.0	20.0	20.3	洪積	粘性土	17.00	144.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				21.3	洪積	粘性土	22.00	152.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				22.3	洪積	粘性土	24.00	160.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				23.3	洪積	粘性土	25.00	168.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				24.3	洪積	粘性土	19.00	176.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				25.3	洪積	粘性土	18.00	184.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				26.3	洪積	粘性土	13.00	192.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				27.3	洪積	粘性土	13.00	200.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				28.3	洪積	粘性土	13.00	208.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6
				29.3	洪積	粘性土	25.00	216.9	90.00	0.000	しない	0.00	0.300	0.000	0.000	**1	C6

Legend: 沖積…Alluvium, 洪積…Diluvium, 粘性土…Clayey Soil, 砂質土…Sandy Soil, しない…No

Source: Study team

Table 7.4.4 Liquefaction Analysis Result (DD-BH-C02, L1 Earthquake Ground Motion)

Name of the point DD-BH-C02

PL Value 14.784
Unit volume weight of water 10.0 (kN/m3)
Load 0.0 (kN/m2)
Groundwater level 0.00 (m)

Design horizontal seismic coefficient 0.20
Note: :** 1 to * * Z Out of Scope of Liquefaction Analysis

Soil Depth (m)	Soil Property										Target Layer Of Analysis Or Not	Earthquake Dynamic Coefficient	Repeated Triaxial Strength Ratio	Determination Of The Liquid			
	Layer Thickness(m)	Wet Weight (kn/m3)	Saturated Weight (kn/m3)	Dynamic Shear Strength Ratio	Soil Category	Soil Type	Observed N value	Effective Loading Pressure (kN/m2)	Fine Fraction Content (%)	Particle Size (mm)				Dynamic Shear Strength Ratio	Seismic Response Ratio	Liquefaction Resistance	
D	h	γ t	γ sat	x			N	σ v	F c	D 50		C w	R L	R	L	F L	
				1.3	沖積	粘性土	0.00	6.5	93.00	0.017		1.00	0.248	0.248	0.588	0.422	
				2.3	沖積	粘性土	0.00	11.5	93.00	0.017		1.00	0.248	0.248	0.579	0.429	
				3.3	沖積	粘性土	2.00	16.5	93.00	0.017		1.00	3.327	3.327	0.570	5.834	
				4.3	沖積	粘性土	3.00	21.5	93.00	0.017		1.00	14.944	14.944	0.561	26.624	C1
				5.3	沖積	粘性土	2.00	26.5	93.00	0.017		1.00	2.159	2.159	0.552	3.909	▽Subgrade Level
7.00	7.00	15.0	15.0	6.3	沖積	粘性土	2.00	31.5	93.00	0.017		1.00	1.787	1.787	0.543	3.289	
				7.3	沖積	粘性土	16.00	37.4	83.00	0.000		1.00			0.526		
9.00	2.00	18.0	18.0	8.3	沖積	粘性土	17.00	45.4	83.00	0.000		1.00			0.495		C2
				9.3	沖積	粘性土	2.00	52.8	86.00	0.024		0.00	0.000	0.000	0.000	**2	
12.00	3.00	16.0	16.0	10.3	沖積	粘性土	2.00	58.8	86.00	0.024		0.00	0.000	0.000	0.000	**2	C3
				12.3	沖積	粘性土	11.00	71.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	
				13.3	沖積	粘性土	12.00	78.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	
15.00	3.00	17.0	17.0	14.3	沖積	粘性土	12.00	85.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	C4
				15.3	沖積	粘性土	18.00	92.4	77.00	0.540		0.00	0.000	0.000	0.000	**2	
				16.3	沖積	粘性土	20.00	100.4	77.00	0.282		0.00	0.000	0.000	0.000	**2	C5
18.00	3.00	18.0	18.0	17.3	沖積	粘性土	25.00	108.4	77.00	0.023		0.00	0.000	0.000	0.000	**2	
				18.3	沖積	砂質土	46.00	117.3	18.00	0.000		1.00	25.096	25.096	0.371	67.559	
				19.3	沖積	砂質土	50.00	128.3	18.00	0.000		1.00	29.301	29.304	0.356	82.347	S1
21.00	3.00	21.0	21.0	20.3	沖積	砂質土	48.00	139.3	18.00	0.000		1.00	16.688	16.688	0.342	48.824	
				21.3	洪積	砂質土	25.00	150.0	22.00	2.700	しない	0.00	0.000	0.000	0.000	**1	
23.00	2.00	20.0	20.0	22.3	洪積	砂質土	16.00	160.0	22.00	2.700	しない	0.00	0.000	0.000	0.000	**1	S2
				23.3	洪積	粘性土	19.00	169.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
				24.3	洪積	粘性土	18.00	177.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
				25.3	洪積	粘性土	20.00	185.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
				26.3	洪積	粘性土	11.00	193.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	C6
				27.3	洪積	粘性土	18.00	201.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
				28.3	洪積	粘性土	23.00	209.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
				29.3	洪積	粘性土	21.00	217.4	91.00	0.000	しない	0.00	0.000	0.000	0.000	**1	

Legend: 沖積…Alluvium, 洪積…Diluvium, 粘性土…Clayey Soil, 砂質土…Sandy Soil, しない…No

Source: Study team

Table 7.4.5 Liquefaction Analysis Result (DD-BH-C03, L1 Earthquake Ground Motion)

Name of the point DD-BH-C03

PL Value 0.000
Unit volume weight of water 10.0 (kN/m3)
Load 0.0 (kN/m2)
Groundwater level 0.00 (m)

Design horizontal seismic coefficient 0.20
Note: :** 1 to * * Z Out of Scope of Liquefaction Analysis

Soil Depth (m)	Layer Thickness(m)	Wet Weight (kn/m3)	Saturated Weight (kn/m3)	Dynamic Shear Strength Ratio	Soil Category	Soil Type	Observed N value	Effective Loading Pressure (kN/m2)	Fine Fraction Content (%)	Particle Size (mm)	Target Layer Of Analysis Or Not	Earthquake Dynamic Coefficient	Repeated Triaxial Strength Ratio	Dynamic Shear Strength Ratio	Seismic Response Ratio	Liquefaction Resistance	Layer
D	h	γt	γsat	x			N	σv'	Fc	D50		Cw	RL	R	L	FL	
				1.3	沖積	粘性土	0.00	6.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	
				2.3	沖積	粘性土	0.00	11.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	
				3.3	沖積	粘性土	2.00	16.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	
				4.3	沖積	粘性土	2.00	21.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	▽Subgrade Level C1
6.00	6.00	15.0	15.0	6.3	沖積	粘性土	11.00	32.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	
				7.3	沖積	粘性土	11.00	39.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	C2
8.00	2.00	17.0	17.0	8.3	沖積	粘性土	2.00	45.8	70.00	0.000		0.00	0.000	0.000	0.000	**2	
10.00	2.00	16.0	16.0	10.3	沖積	粘性土	5.00	58.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	C3
				11.3	沖積	粘性土	9.00	65.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	
				12.3	沖積	粘性土	8.00	72.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	
				13.3	沖積	粘性土	11.00	79.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	C4
14.00	4.00	17.0	17.0	14.3	沖積	粘性土	15.00	86.4	60.00	0.180		0.00	0.000	0.000	0.000	**2	
				15.3	沖積	粘性土	17.00	94.4	60.00	0.180		0.00	0.000	0.000	0.000	**2	
				16.3	沖積	粘性土	15.00	102.4	60.00	0.180		0.00	0.000	0.000	0.000	**2	C5
17.00	3.00	18.0	18.0	17.3	沖積	砂質土	25.00	111.0	13.00	0.460		1.00	0.464	0.464	0.379	1.224	
				18.3	沖積	砂質土	27.00	121.0	13.00	0.460		1.00	0.497	0.497	0.365	1.363	
				19.3	沖積	砂質土	25.00	131.0	13.00	0.460		1.00	0.368	0.368	0.351	1.018	S1
20.00	3.00	20.0	20.0	20.3	洪積	粘性土	19.00	140.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				21.3	洪積	粘性土	27.00	148.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				22.3	洪積	粘性土	23.00	156.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				23.3	洪積	粘性土	18.00	164.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				24.3	洪積	粘性土	19.00	172.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				25.3	洪積	粘性土	20.00	180.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				26.3	洪積	粘性土	14.00	188.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	C6
				27.3	洪積	粘性土	16.00	196.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				28.3	洪積	粘性土	24.00	204.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				29.3	洪積	粘性土	23.00	212.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	

Legend: 沖積…Alluvium, 洪積…Diluvium, 粘性土…Clayey Soil, 砂質土…Sandy Soil, しない…No

Source: Study team

(e) Liquefaction Analysis result for L2 Earthquake Motion

The following pages show the results of Liquefaction Analysis for the design horizontal seismic intensity of the L2 earthquake in each stratum and the results of Liquefaction Analysis at each investigation site.

A list of Liquefaction Analysis results for L2 earthquake ground motion is shown in **Table 7.4.6**.The results of Liquefaction Analysis at each investigation site are shown in **Table 7.4.7**, **Table 7.4.8**, and **Table 7.4.9**. Also, the locations where the F_L value is 1.0 or less are shown in the geological profile of **Figure 7.4.17**. In the C1 layer, the F_L value became 1.0 or less in a part of DD-BH-C02, but there was no effect because it was located above the subgrade level when the Cainta Floodgate was constructed. On the other hand, for the S1 layer, the F_L value became 1.0 or less at DD-BH-C01 and DD-BH-C03, so the S1 layer is defined as the liquefaction generation layer against the L2 earthquake motion.

Table 7.4.6 Liquefaction Analysis Result List (L2 Earthquake Ground Motion)

Stratum	Geological Survey	Depth (m)	N-value	F_L Value	Remarks
C1	DD-BH-C01	1.3	2	4.019	Above subgrade level
		2.3	2	3.124	Above subgrade level
		3.3	2	2.482	Above subgrade level
	DD-BH-C02	1.3	0	0.180	Liquefaction occurs, but above the subgrade level
		2.3	0	0.182	Liquefaction occurs, but above the subgrade level
		3.3	2	2.482	Above subgrade level
		4.3	3	11.329	Above subgrade level
		5.3	2	1.664	
		6.3	2	1.400	
	DD-BH-C03	1.3	0	Out of Target	Not calculated due to FC > 35% and Ip > 15
		2.3	0	Out of Target	Not calculated due to FC > 35% and Ip > 15
		3.3	2	Out of Target	Not calculated due to FC > 35% and Ip > 15
		4.3	2	Out of Target	Not calculated due to FC > 35% and Ip > 15
C2	DD-BH-C01	5.3	16	9960.5	Above subgrade level
		6.3	15	5549.2	
	DD-BH-C02	7.3	16	5649.6	
		8.3	17	5696.9	
	DD-BH-C03	6.3	11	Out of Target	Not calculated due to FC > 35% and Ip > 15
		7.3	11	Out of Target	Not calculated due to FC > 35% and Ip > 15
S1	DD-BH-C01	14.3	16	0.385	Liquefaction occurs.
		15.3	19	0.513	Liquefaction occurs.
	DD-BH-C02	18.3	46	28.749	
		19.3	50	35.041	
		20.3	48	20.776	
	DD-BH-C03	17.3	25	0.521	Liquefaction occurs.
		18.3	27	0.580	Liquefaction occurs.
		19.3	25	0.446	Liquefaction occurs.
S2	DD-BH-C01	18.3	24	1.104	
		19.3	40	17.026	
	DD-BH-C02	21.3	25	Out of Target	Depth is 20 m or more, so it is Not calculated.
		22.3	16	Out of Target	Depth is 20 m or more, so it is Not calculated.

Legend : Liquefaction layer

Source: Study team

Table 7.4.7 Liquefaction Analysis Result (DD-BH-C01, L2 Earthquake Motion)

Name of the point DD-BH-C01
Level 2 Earthquake Ground Motion

PL Value 2.774
Unit volume weight of water 10.0 (kN/m3)
Load 0.0 (kN/m2)
Groundwater level 0.00 (m)

Design horizontal seismic coefficient 0.47
Note: :** 1 to * * Z Out of Scope of Liquefaction Analysis

Soil Depth (m)	Layer (Soil Property)	Thickness(m)	Wet Weight (kn/m3)	Saturated Weight (kn/m3)	Dynamic Shear Strength Ratio	Soil Category	Soil Type	Observed N value	Effective Loading Pressure (kN/m2)	Fine Fraction Content (%)	Particle Size (mm)	Target Layer Of Analysis Or Not	Earthquake Dynamic Coefficient	Repeated Triaxial Strength Ratio	Dynamic Shear Strength Ratio (Determination Of The Liquid)	Seismic Response Ratio	Liquefaction Resistance	
D		H	γt	γsat	x			N	σv'	FC	D50		Cw	RL	R	L	FL	
					1.3	沖積	粘性土	2.00	6.5	93.00	0.000		1.00	5.557	5.557	1.383	4.019	
					2.3	沖積	粘性土	2.00	11.5	93.00	0.000		1.00	4.253	4.253	1.361	3.124	C1
					3.3	沖積	粘性土	2.00	16.5	93.00	0.000		1.00	3.327	3.327	1.340	2.482	
5.00		5.00	15.0	15.0	5.3	沖積	粘性土	16.00	27.4	85.00	0.020		1.00			1.269		
					6.3	沖積	粘性土	15.00	35.4	85.00	0.020		1.00			1.183		▽Subgrade Level C2
7.00		2.00	18.0	18.0	7.3	沖積	粘性土	3.00	43.0	80.00	0.000		0.00	0.000	0.000	0.000	**2	
					8.3	沖積	粘性土	2.00	49.5	80.00	0.000		0.00	0.000	0.000	0.000	**2	C3
10.00		3.00	16.5	16.5	10.3	沖積	粘性土	5.00	62.5	89.00	0.000		0.00	0.000	0.000	0.000	**2	
					11.3	沖積	粘性土	3.00	68.9	89.00	0.000		0.00	0.000	0.000	0.000	**2	C4
					13.3	沖積	粘性土	8.00	81.9	89.00	0.000		0.00	0.000	0.000	0.000	**2	
14.00		4.00	16.5	16.5	14.3	沖積	砂質土	16.00	89.5	20.00	0.200		1.00	0.370	0.370	0.959	0.385	
					15.3	沖積	砂質土	19.00	99.5	20.00	0.200		1.00	0.472	0.472	0.919	0.513	S1
16.00		2.00	20.0	20.0	16.3	沖積	粘性土	22.00	108.9	65.00	0.000		0.00	0.000	0.000	0.000	**2	
					17.3	沖積	粘性土	44.00	116.9	65.00	0.000		0.00	0.000	0.000	0.000	**2	C5
18.00		2.00	18.0	18.0	18.3	沖積	砂質土	24.00	125.5	23.00	0.350		1.00	0.925	0.925	0.838	1.104	
					19.3	沖積	砂質土	40.00	135.5	23.00	0.350		1.00	13.784	13.784	0.810	17.026	S2
20.00		2.00	20.0	20.0	20.3	洪積	粘性土	17.00	144.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					21.3	洪積	粘性土	22.00	152.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					22.3	洪積	粘性土	24.00	160.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					23.3	洪積	粘性土	25.00	168.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					24.3	洪積	粘性土	19.00	176.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	C6
					25.3	洪積	粘性土	18.00	184.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					26.3	洪積	粘性土	13.00	192.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					27.3	洪積	粘性土	13.00	200.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					28.3	洪積	粘性土	13.00	208.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					29.3	洪積	粘性土	25.00	216.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	

Legend: 沖積…Alluvium, 洪積…Diluvium, 粘性土…Clayey Soil, 砂質土…Sandy Soil, しない…No

Source: Study team

Table 7.4.8 Liquefaction Analysis Result (DD-BH-C02, L2 Earthquake Motion)

Name of the point DD-BH-C02
Level 2 Earthquake Ground Motion

PL Value 21.043
Unit volume weight of water 10.0 (kN/m3)
Load 0.0 (kN/m2)
Groundwater level 0.00 (m)

Design horizontal seismic coefficient 0.47
Note: :** 1 to * * Z Out of Scope of Liquefaction Analysis

Soil Depth (m)	Layer Thickness(m)		Wet Weight (kn/m3)	Saturated Weight (kn/m3)	Dynamic Shear Strength Ratio	Soil Category	Soil Type	Observed N value	Effective Loading Pressure (kN/m2)	Fine Fraction Content (%)	Particle Size (mm)	Target Layer Of Analysis Or Not	Earthquake Dynamic Coefficient	Repeated Triaxial Strength Ratio	Dynamic Shear Strength Ratio	Seismic Response Ratio	Liquefaction Resistance	
D	h		γt	γsat	x			N	σv	Fc	D50		Cw	RL	R	L	FL	
					1.3	沖積	粘性土	2.00	6.5	93.00	0.000		1.00	5.557	5.557	1.383	4.019	
					2.3	沖積	粘性土	2.00	11.5	93.00	0.000		1.00	4.253	4.253	1.361	3.124	
					3.3	沖積	粘性土	2.00	16.5	93.00	0.000		1.00	3.327	3.327	1.340	2.482	C1
5.00	5.00		15.0	15.0	5.3	沖積	粘性土	16.00	27.4	85.00	0.020		1.00			1.269		▽Subgrade Level
					6.3	沖積	粘性土	15.00	35.4	85.00	0.020		1.00			1.183		
7.00	2.00		18.0	18.0	7.3	沖積	粘性土	3.00	43.0	80.00	0.000		0.00	0.000	0.000	0.000	**2	
					8.3	沖積	粘性土	2.00	49.5	80.00	0.000		0.00	0.000	0.000	0.000	**2	C2
10.00	3.00		16.5	16.5	10.3	沖積	粘性土	5.00	62.5	89.00	0.000		0.00	0.000	0.000	0.000	**2	
					11.3	沖積	粘性土	3.00	68.9	89.00	0.000		0.00	0.000	0.000	0.000	**2	C3
					13.3	沖積	粘性土	8.00	81.9	89.00	0.000		0.00	0.000	0.000	0.000	**2	
14.00	4.00		16.5	16.5	14.3	沖積	砂質土	16.00	89.5	20.00	0.200		1.00	0.370	0.370	0.959	0.385	C4
					15.3	沖積	砂質土	19.00	99.5	20.00	0.200		1.00	0.472	0.472	0.919	0.513	
16.00	2.00		20.0	20.0	16.3	沖積	粘性土	22.00	108.9	65.00	0.000		0.00	0.000	0.000	0.000	**2	
					17.3	沖積	粘性土	44.00	116.9	65.00	0.000		0.00	0.000	0.000	0.000	**2	C5
18.00	2.00		18.0	18.0	18.3	沖積	砂質土	24.00	125.5	23.00	0.350		1.00	0.925	0.925	0.838	1.104	
					19.3	沖積	砂質土	40.00	135.5	23.00	0.350		1.00	13.784	13.784	0.810	17.026	
20.00	2.00		20.0	20.0	20.3	洪積	粘性土	17.00	144.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	S1
					21.3	洪積	粘性土	22.00	152.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					22.3	洪積	粘性土	24.00	160.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	S2
					23.3	洪積	粘性土	25.00	168.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					24.3	洪積	粘性土	19.00	176.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					25.3	洪積	粘性土	18.00	184.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					26.3	洪積	粘性土	13.00	192.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	C6
					27.3	洪積	粘性土	13.00	200.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					28.3	洪積	粘性土	13.00	208.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	
					29.3	洪積	粘性土	25.00	216.9	90.00	0.000	しない	0.00	0.000	0.000	0.000	**1	

Legend: 沖積…Alluvium, 洪積…Diluvium, 粘性土…Clayey Soil, 砂質土…Sandy Soil, しない…No

Source: Study team

Table 7.4.9 Liquefaction Analysis Result (DD-BH-C03, L2 Earthquake Motion)

Name of the point DD-BH-C03
Level 2 Earthquake Ground Motion

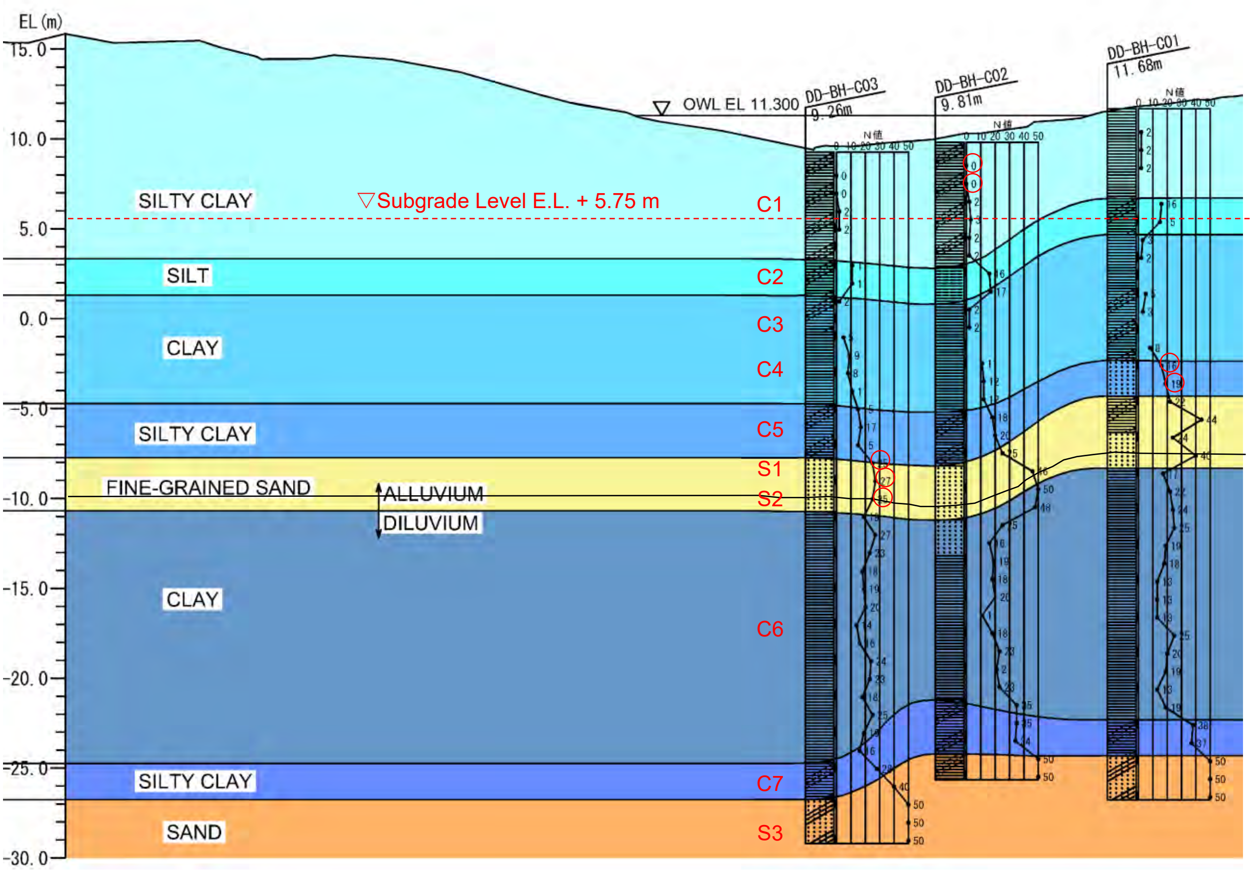
PL Value 1.107
Unit volume weight of water 10.0 (kN/m3)
Load 0.0 (kN/m2)
Groundwater level 0.00 (m)

Design horizontal seismic coefficient 0.47
Note: :** 1 to * * Z Out of Scope of Liquefaction Analysis

Soil Depth (m) D	Layer Thickness(m) h	Wet Weight (kn/m3) γt	Saturated Weight (kn/m3) γsat	Dynamic Shear Strength Ratio x	Soil Category	Soil Type	Observed N value N	Effective Loading Pressure (kN/m2) σv	Fine Fraction Content (%) Fc	Particle Size (mm) D50	Target Layer Of Analysis Or Not	Earthquake Dynamic Coefficient Cw	Repeated Triaxial Strength Ratio RL	Dynamic Shear Strength Ratio R	Seismic Response Ratio L	Liquefaction Resistance FL	
				1.3	沖積	粘性土	0.00	6.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	
				2.3	沖積	粘性土	0.00	11.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	
				3.3	沖積	粘性土	2.00	16.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	C1
				4.3	沖積	粘性土	2.00	21.5	93.00	0.024		0.00	0.000	0.000	0.000	**2	▽Subgrade Level
6.00	6.00	15.0	15.0	6.3	沖積	粘性土	11.00	32.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	
				7.3	沖積	粘性土	11.00	39.1	90.00	0.000		0.00	0.000	0.000	0.000	**2	C2
8.00	2.00	17.0	17.0	8.3	沖積	粘性土	2.00	45.8	70.00	0.000		0.00	0.000	0.000	0.000	**2	C3
10.00	2.00	16.0	16.0	10.3	沖積	粘性土	5.00	58.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	
				11.3	沖積	粘性土	9.00	65.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	
				12.3	沖積	粘性土	8.00	72.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	C4
				13.3	沖積	粘性土	11.00	79.1	96.00	0.000		0.00	0.000	0.000	0.000	**2	
14.00	4.00	17.0	17.0	14.3	沖積	粘性土	15.00	86.4	60.00	0.180		0.00	0.000	0.000	0.000	**2	
				15.3	沖積	粘性土	17.00	94.4	60.00	0.180		0.00	0.000	0.000	0.000	**2	C5
				16.3	沖積	粘性土	15.00	102.4	60.00	0.180		0.00	0.000	0.000	0.000	**2	
17.00	3.00	18.0	18.0	17.3	沖積	砂質土	25.00	111.0	13.00	0.460		1.00	0.464	0.464	0.890	0.521	
				18.3	沖積	砂質土	27.00	121.0	13.00	0.460		1.00	0.497	0.497	0.857	0.580	S1
				19.3	沖積	砂質土	25.00	131.0	13.00	0.460		1.00	0.368	0.368	0.826	0.446	
20.00	3.00	20.0	20.0	20.3	洪積	粘性土	19.00	140.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				21.3	洪積	粘性土	27.00	148.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				22.3	洪積	粘性土	23.00	156.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				23.3	洪積	粘性土	18.00	164.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				24.3	洪積	粘性土	19.00	172.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	C6
				25.3	洪積	粘性土	20.00	180.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				26.3	洪積	粘性土	14.00	188.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				27.3	洪積	粘性土	16.00	196.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				28.3	洪積	粘性土	24.00	204.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	
				29.3	洪積	粘性土	23.00	212.4	94.00	0.150	しない	0.00	0.000	0.000	0.000	**1	

Legend: 沖積…Alluvium, 洪積…Diluvium, 粘性土…Clayey Soil, 砂質土…Sandy Soil, しない…No

Source: Study team



○: F_L value of 1.0 or less for L2 earthquake motion

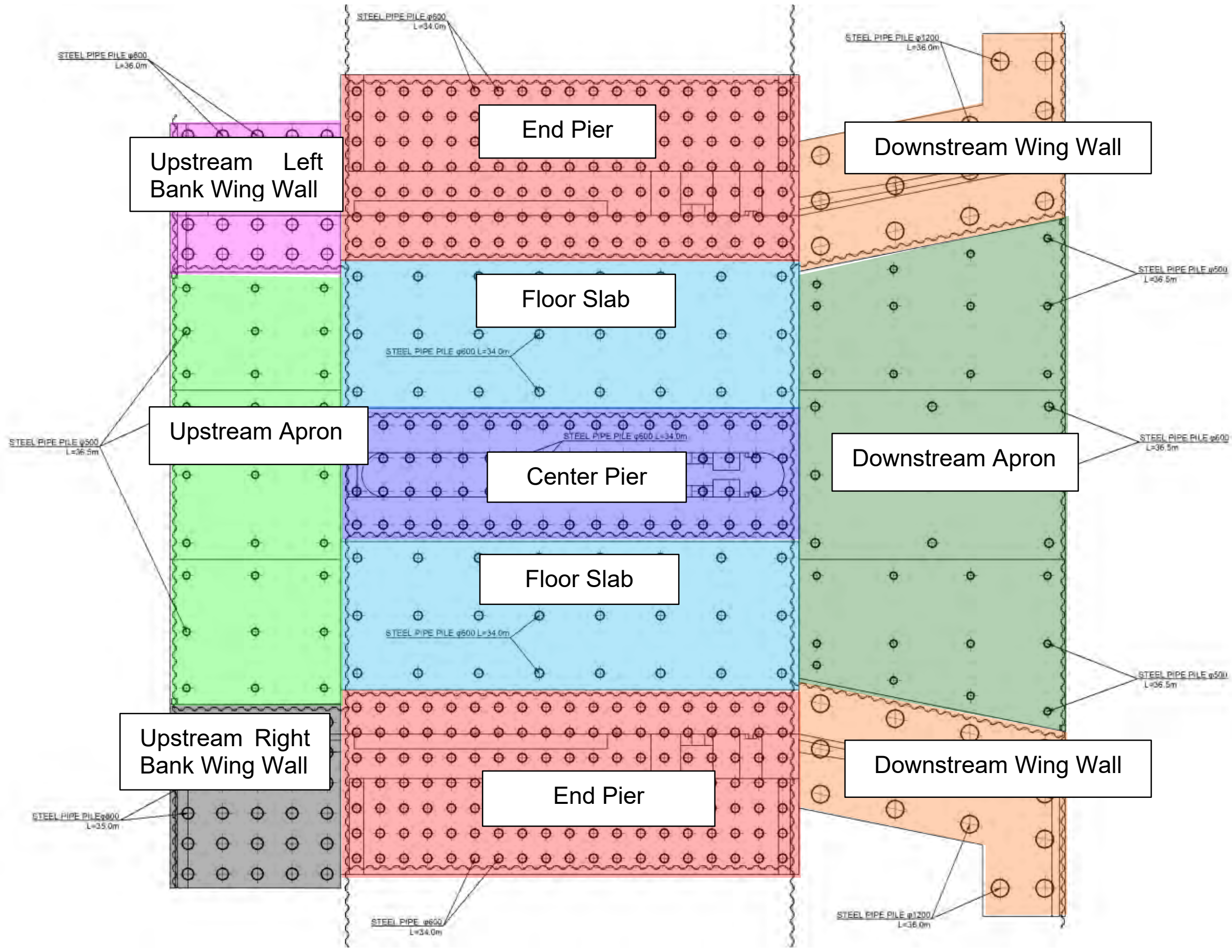
Source: Study team

Figure 7.4.17 Geological Profile

(3) Design of Foundation Piles

1) Study Policy

Design the floodgate main body, the wing wall and the foundation pile of the apron. The floodgate main body is composed of three piers of a center pier, a left pier and a right pier, and a floor slab. Regarding the calculation, since the piers on the left and right sides are the object structure, 2 piers (Center pier and end pier), floor slab, wing wall, and foundation pile for apron are calculated.



Source: Study team

Figure 7.4.18 Study Member for Foundation Pile

In the design of foundation piles, pile arrangements that satisfy the allowable values of axial push-in bearing capacity, axial pull-out capacity, and horizontal displacement are selected for the applied load conditions. The list of check items is shown in **Table 7.4.10**. It should be checked that the values in the table do not exceed the allowable values.

Table 7.4.10 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

To adopt the most economical pile arrangement among pile arrangements satisfying the above checking.

(a) Application criteria and references

The standards and references applied to the design of pile foundations are shown below.

Specifications for Highway Bridges I, Common (March 2012)
Specifications for Highway Bridges III Concrete Bridge (March 2012)
Specifications for Highway Bridges IV Substructure (March 2012)
Specifications for Highway Bridges V Seismic Design (March 2012)
Pile Foundation Design Manual (2015)
Seismic Design of Road Bridges (March 1997)

(b) Pile Type

Regarding pile types and driving methods, steel pipe piles by the vibro hammer method are selected from the effects on surrounding houses and workability.

(c) Materials used and allowable stress

The material used for the steel pipe pile is SKK 400. The allowable stress of SKK 400 is shown in the table below.

Table 7.4.11 Allowable Stress in Steel Pipe Piles (N/mm2)

No	Extra factor	Allowable bending compressive stress σ ca		Allowable bending tensile stress σ ta		allowable shear stress τa	
		SKK 400	SKK 490	SKK 400	SKK 490	SKK 400	SKK 490
1	1.00	140.00	185.00	140.00	185.00	80.00	105.00
2	1.33	186.20	246.05	186.20	246.05	106.40	139.65
3	1.50	210.00	277.00	210.00	277.00	120.00	157.00

Source: Specifications for Highway Bridges IV Substructure Compilation Table -4.4 .1

(d) Pile Diameter and Plate Thickness

The pile diameter and plate thickness of steel pipe piles shall be within the following ranges.

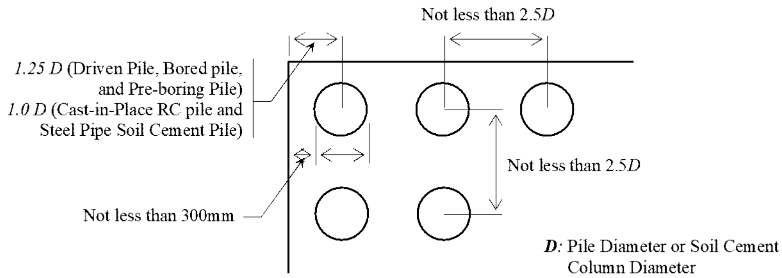
Table 7.4.12 Range of Diameter and Thickness of Steel Pipe Pile Used for Hammering Method

Nominal Diameter (mm)	Range of Plate Thickness)
400	9- 12
500	9 - 14
600 – 800	9 – 16
900 -1100	12 -19
1200 – 1400	14 – 22
1500 – 1600	16 – 25
1800 - 2000	19 - 25

Source: Specifications for Highway Bridges IV Substructure Compilation - Solution 12.11 .1

(e) Pile Arrangement

The minimum center distance and footing edge distance of the piles shall be as shown in the figure below.



Source: BSDS, 5-16

Figure 7.4.19 Minimum Interval of Piles and Distance in Footing Edge

(f) Allowable Axial Pushing Bearing Capacity

Allowable axial push-in bearing capacity is obtained by the following equation[7];

$$Ra = \{qd\ A + U \sum (1i\ fi)\} / SF$$

Here,

Ra : Allowable Bearing Capacity of pile (kN)
qd : Ultimate Bearing Capacity Per Unit Area At Pile Top (kN/m2)
A : Pile Tip Area (m2)
U : Pile Perimeter (m)
1i : Layer Thickness (m)
fi : Maximum shaft resistance of soil layer considering pile shaft resistance: (kN/m2) See table below;

Table 7.4.13 Circumferential Friction Coefficient

Table C5.4.3.3-5 Maximum Shaft Resistance Intensity (kN/m^2)

Ground Type / Pile Installation Method	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-Hammer Method)	$2N$ ($\leq$ 100)	c or $10N$ ($\leq$ 150)
Cast-in-place RC pile method	$5N$ ($\leq$ 200)	c or $10N$ ($\leq$ 150)
Bored Pile Method	$2N$ ($\leq$ 100)	0.8 c or $8N$ ($\leq$ 100)
Pre-bored Pile Method	$5N$ ($\leq$ 150)	c or $10N$ ($\leq$ 100)
Steel Pipe Soil Cement Pile Method	$10N$ ($\leq$ 200)	c or $10N$ ($\leq$200)

Source: BSDS, 5-24

Here,

C : Cohesion of Ground (kN/m2),
N : N value
SF : Safety factor in the following table

[7] allowable bearing capacity of 12.4 piles of substructure section IV

Table 7.4.14 Safety Factor

Load condition	Safety factor	
	Support pile	Friction pile
Normal Condition	3	4
Seismic Condition	2	3

Source: Specifications for Highway Bridges IV Substructure Compilation Table 12.4 .1

The bearing pile refers to a pile in which the main element of bearing capacity is supported by the resistance force based on the characteristics of the pile tip ground, and the friction pile refers to a pile in which the main element of bearing capacity is supported only by the resistance force of the ground along the side of the pile of the penetration length. In the design of the foundation pile in this design, it is examined as a support pile.

In the case of driving Bearing Pile, the ultimate bearing capacity per unit area at the pile tip is calculated using the following figure. (In this figure, the pile tip ground is applied to cohesive soil, sand and gravel ground.)

In this design, the case of open-ended steel pipe pile in the figure below is applied to calculate the ultimate bearing capacity.

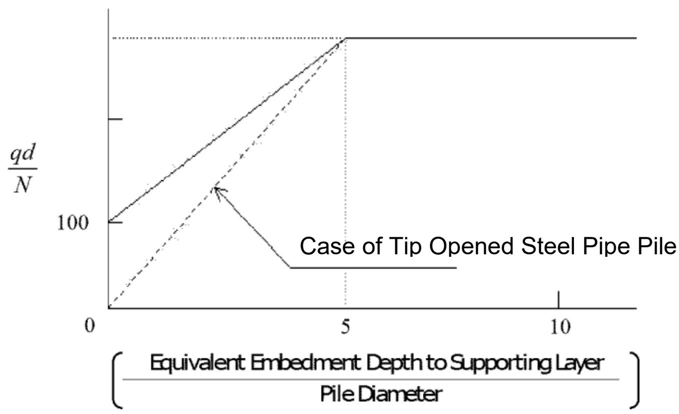


Figure C5.4.3.3-1 Evaluation Chart for Ultimate End Bearing Capacity Intensity (q_d)

Source: BCDS, P5-19

Figure 7.4.20 Calculation Diagram Of Ultimate Bearing Capacity qd of Pile Tip Ground

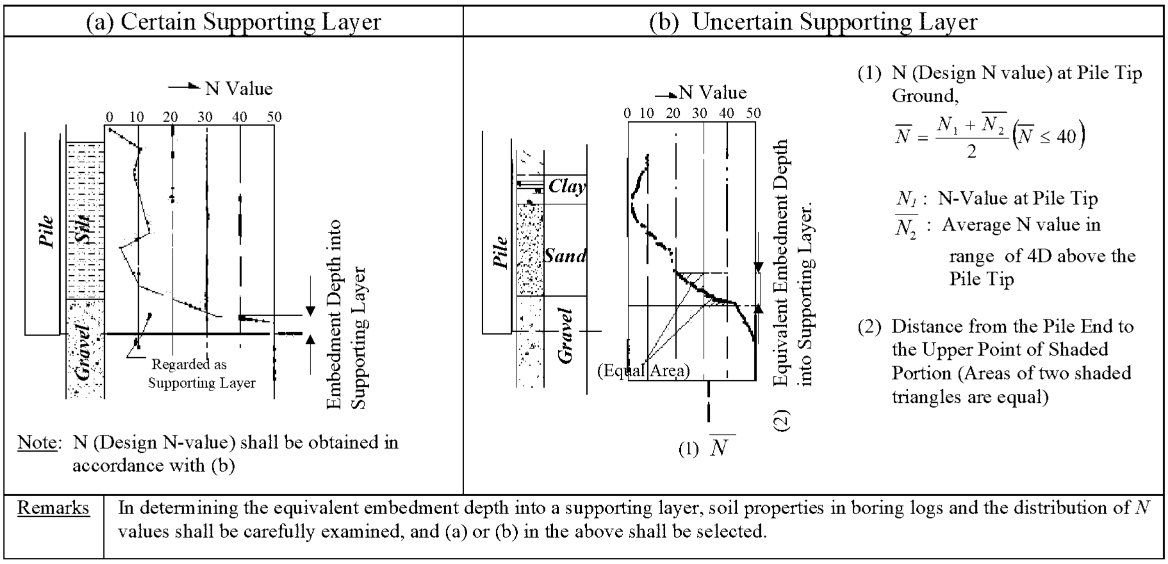


Figure C5.4.3.3-2 Determination Method of Equivalent Embedment Depth into Supporting Layer

Source: BSDS, P5-19

Figure 7.4.21 Method For Determining Reduced Depth of Penetration into Supporting Layer

The N value for calculating the ultimate bearing capacity is determined as follows.

$$N = \frac{N_1 + N_2}{2}$$

Here,

N1 : SPT N value at the pile tip
N2 : N value of SPT in the range of 4D above the pile top
D : diameter of round pile or side of square pile (m)

(g) Allowable Axial Pull-out Force[8]

The allowable Axial pull-out force of a pile shall be determined by the following formula.

Pa = Pu/SF + W

Here,

Pa : Allowable Axial Pull Force In Pile Head (kN)
Pu : Axial Pull of The Pile from Ground (kN)

P_u : $U \Sigma (1_i \quad f_i)$

W : Effective Pile Weight (kN)

The safety factor is as follows

SF = 6 (Normal Condition)

SF = 3 (Seismic Condition)

[8] Road bridge specifications and explanations IV Substructure 12.4. 2 Allowable axial pulling resistance of 1 pile

(h) Allowable Displacement of the Pile

Allowable displacement of pile foundation structure as a river structure is as follows.

Table 7.4.15 Allowable Displacement of Pile

Displacement type	During Normal Condition and earthquakes
Horizontal displacement	10 mm

Source: Technical Criteria for River Works: Practical Guide for Planning [I], Japan

2) Examination of Pile Arrangement

Results of economic comparison of pile arrangement for each member are shown in **Table 7.4.16** and **Table 7.4.17**. In addition, a layout plan of the adopted pile arrangement is shown in **Figure 7.4.22**.

Table 7.4.16 Comparison of Economics of Pile Arrangement (1/2)

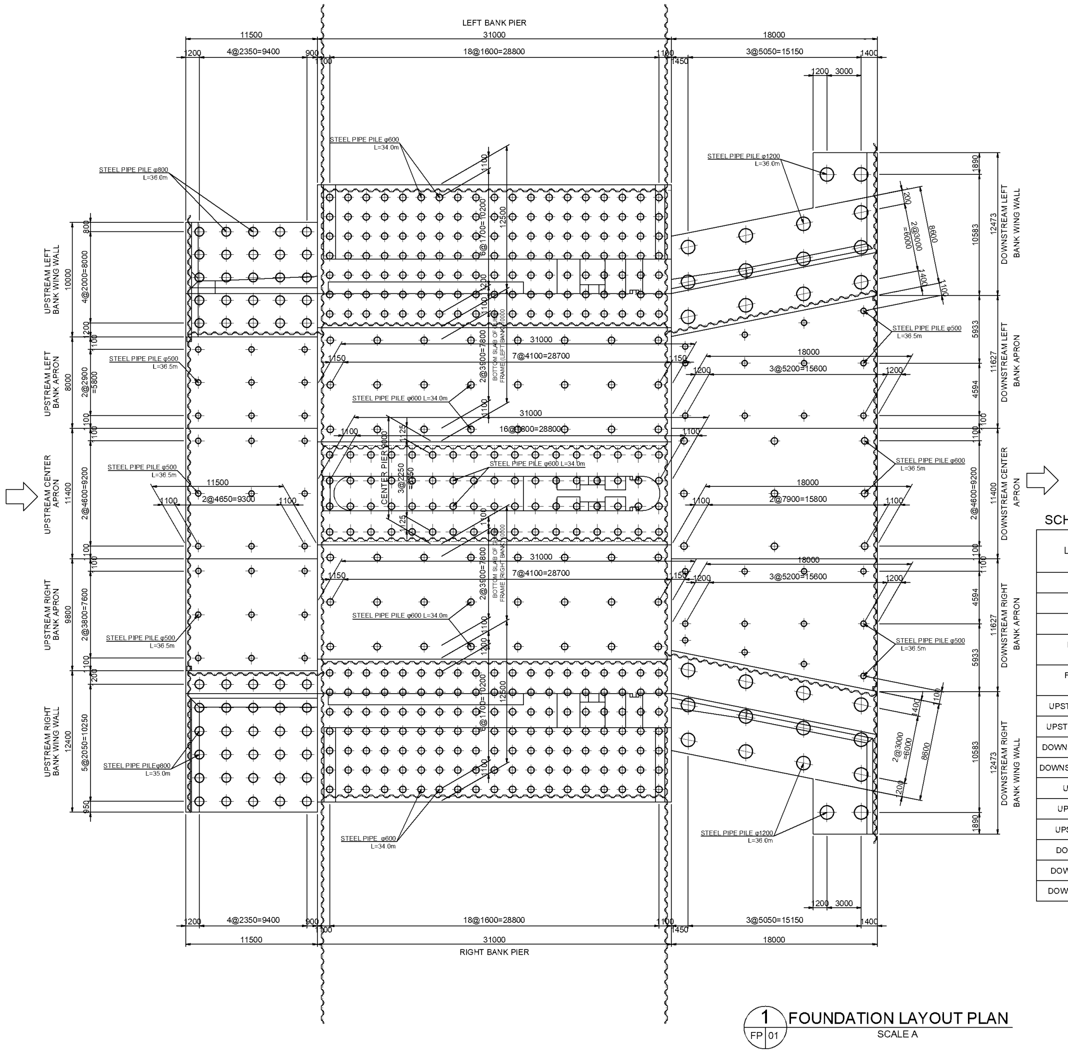
Member	Pile Type	Pile Diameter	Upper Pile			Lower Pile			Number Of Piles	Amount	Adoption	Remarks
			Plate Thicknes	Pile Length	Unit Price	Plate Thicknes	Pile Length	Unit Price	N			
			(mm)	(m)	(PhP/m)	(mm)	(m)	(PhP/m)	(pile)	(PhP)		
Downstream Side Wing Wall	Steel Pipe Pile	ϕ 1000	15	10.5	27,700	12	25.5	22,500	20	17,292,000		
		ϕ 1200	19	10.5	41,500	14	25.5	31,100	14	17,203,200	Adopted	
		ϕ 1400	14	10.5	36,100	14	25.5	36,100	14	18,194,400		
Upstream Side Right Bank Wing Wall	Steel Pipe Pile	ϕ 600	16	8.0	17,900	9	27.0	10,800	42	18,261,600		NG at maximum deployment
		ϕ 800	16	8.0	23,700	9	27.0	14,100	30	17,109,000		
		ϕ 1000	19	8.0	34,600	12	27.0	22,500	20	17,686,000		
Upstream Side Left Bank Wing Wall	Steel Pipe Pile	ϕ 600	16	8.0	17,900	9	28.0	10,800	30	13,368,000		
		ϕ 800	12	8.0	18,200	9	28.0	14,100	25	13,510,000		
		ϕ 1000	19	8.0	34,600	12	28.0	22,500	16	14,508,800		
Upstream Side Wing Walll Total	Steel Pipe Pile	ϕ 600								31,629,600		For the upstream wing wall: Pile diameters were unified at the right and left sides.
		ϕ 800								30,619,000	Adopted	
		ϕ 1000								32,194,800		
Center Pier	Steel Pipe Pile	ϕ 500	14	7.0	13,500	9	27.0	9,200	92	31,546,800		
		ϕ 600	16	7.0	17,900	9	27.0	10,800	68	28,349,200	Adopted	
		ϕ 800	16	7.0	23,700	9	27.0	14,100	52	28,423,200		
End Pier	Steel Pipe Pile	ϕ 500	14	7.0	13,500	9	27.0	9,200	176	60,350,400		
		ϕ 600	16	7.0	17,900	9	27.0	10,800	133	55,447,700	Adopted	
		ϕ 800	16	7.0	23,700	9	27.0	14,100	105	57,393,000		NG at maximum deployment
Floor Slab	Steel Pipe Pile	ϕ 500	12	5.5	11,700	9	28.5	9,200	36	11,755,800		
		ϕ 600	14	6.0	15,900	9	28.0	10,800	24	9,547,200	Adopted	

Source: Study team

Table 7.4.17 Comparison of Economics of Pile Arrangement (2/2)

Member	Pile Type	Pile Diameter	Upper Pile			Lower Pile			umber Of Pile	Amount	Adoption	Remarks
			Plate Thickness	Pile Length	Unit Price	Plate Thickness	Pile Length	Unit Price	N			
			(mm)	(m)	(PhP/m)	(mm)	(m)	(PhP/m)	(pile)	(PhP)		
Upstream Right Apron	Steel Pipe Pile	Φ 500	9	8.5	9200	9	28	9200	9	3022200	Adpted	
		Φ 600	9	8.5	10800	9	28	10800	9	3547800		
Upstream Left Apron	Steel Pipe Pile	Φ 500	9	8.5	9200	9	28	9200	9	3022200	Adpted	
		Φ 600	9	8.5	10800	9	28	10800	9	3547800		
Upstream Center Apron	Steel Pipe Pile	Φ 500	9	8.5	9200	9	28	9200	9	3022200	Adpted	
		Φ 600	9	8.5	10800	9	28	10800	9	3547800		
Downstream Left And Right Apron	Steel Pipe Pile	Φ 500	9	8.5	9200	9	28	9200	12	4029600	Adpted	
		Φ 600	9	8.5	10800	9	28	10800	12	4730400		
Downstream Center Apron	Steel Pipe Pile	Φ 500	9	8.5	9200	9	28	9200	12	4029600		
		Φ 600	9	8.5	10800	9	28	10800	9	3547800	Adpted	

Source: Study team



SCHEDULE OF FOUNDATION PILE

LOCATION OF DRIVING	TYPE・DIAMETER	THICKNESS/LENGTH					NOS.
		TOTAL	UPPER PILE	MIDDLE PILE		LOWER PILE	
				MIDDLE 2 (UPPER)	MIDDLE 1 (LOWER)		
CENTER PIER	STEEL PIPE PILE φ600	34.0m	t=16mm L=7.0m t=9mm L=2.0m	t=9mm L=8.0m	t=9mm L=8.5m	t=9mm L=8.5m	68
LEFT BANK PIER	STEEL PIPE PILE φ600	34.0m	t=16mm L=7.0m t=9mm L=2.0m	t=9mm L=8.0m	t=9mm L=8.5m	t=9mm L=8.5m	133
RIGHT BANK PIER	STEEL PIPE PILE φ600	34.0m	t=16mm L=7.0m t=9mm L=2.0m	t=9mm L=8.0m	t=9mm L=8.5m	t=9mm L=8.5m	133
FLOOR SLAB (LEFT BANK)	STEEL PIPE PILE φ600	34.0m	t=14mm L=6.0m t=9mm L=2.0m	t=9mm L=8.5m	t=9mm L=8.5m	t=9mm L=9.0m	24
FLOOR SLAB (RIGHT BANK)	STEEL PIPE PILE φ600	34.0m	t=14mm L=6.0m t=9mm L=2.0m	t=9mm L=8.5m	t=9mm L=8.5m	t=9mm L=9.0m	24
UPSTREAM LEFT BANK WING WALL	STEEL PIPE PILE φ800	36.0m	t=12mm L=8.0m t=9mm L=2.0m	t=9mm L=8.5m	t=9mm L=8.5m	t=9mm L=9.0m	25
UPSTREAM RIGHT BANK WING WALL	STEEL PIPE PILE φ800	35.0m	t=16mm L=8.0m t=9mm L=2.0m	t=9mm L=8.0m	t=9mm L=8.5m	t=9mm L=8.5m	30
DOWNSTREAM LEFT BANK WING WALL	STEEL PIPE PILE φ1200	36.0m	t=19mm L=10.5m t=14mm L=2.0m	t=14mm L=11.5m	-	t=14mm L=12.0m	14
DOWNSTREAM RIGHT BANK WING WALL	STEEL PIPE PILE φ1200	36.0m	t=19mm L=10.5m t=14mm L=2.0m	t=14mm L=11.5m	-	t=14mm L=12.0m	14
UPSTREAM CENTER APRON	STEEL PIPE PILE φ500	36.5m	t=9mm L=9.0m	t=9mm L=9.0m	t=9mm L=9.0m	t= 9mm L=9.5m	9
UPSTREAM LEFT BANK APRON	STEEL PIPE PILE φ500	36.5m	t=9mm L=9.0m	t=9mm L=9.0m	t=9mm L=9.0m	t= 9mm L=9.5m	9
UPSTREAM RIGHT BANK APRON	STEEL PIPE PILE φ500	36.5m	t=9mm L=9.0m	t=9mm L=9.0m	t=9mm L=9.0m	t= 9mm L=9.5m	9
DOWNSTREAM CENTER APRON	STEEL PIPE PILE φ600	36.5m	t=9mm L=9.0m	t=9mm L=9.0m	t=9mm L=9.0m	t= 9mm L=9.5m	9
DOWNSTREAM LEFT BANK APRON	STEEL PIPE PILE φ500	36.5m	t=9mm L=9.0m	t=9mm L=9.0m	t=9mm L=9.0m	t= 9mm L=9.5m	12
DOWNSTREAM RIGHT BANK APRON	STEEL PIPE PILE φ500	36.5m	t=9mm L=9.0m	t=9mm L=9.0m	t=9mm L=9.0m	t= 9mm L=9.5m	12

SCALE A 0 4.0 8.0 12.0 16.0 20.0m

R E V I S I O N S			
NO.	DESCRIPTION	APPROVED	DATE

NOTE :

PURSUANT TO ITEM 5 OF ANNEX "A" OF THE REVISED IMPLEMENTING RULES AND REGULATION OF R.A. 9184, THE APPROVAL BY THE AUTHORIZED GOVERNMENT OFFICIALS OF DETAILED ENGINEERING SURVEYS AND DESIGNS UNDERTAKEN BY CONSULTANTS NEITHER DIMINISHES THE RESPONSIBILITY OF THE LATTER FOR THE TECHNICAL INTEGRITY OF THE SURVEYS AND DESIGN NOR TRANSFER ANY PART OF THAT RESPONSIBILITY TO THE APPROVING OFFICIALS.

THE DESIGN CONSULTANT SHALL BE HELD FULLY RESPONSIBLE FOR THE FAILURE OF THE FACILITIES/STRUCTURES DUE TO FAULTY DESIGN EXCEPT FOR THE CHANGES MADE WITHOUT THE CONFORMITY OF THE CONSULTANTS.

KAZUTO SUZUKI
TEAM LEADER
JICA CONSULTANT

Source: Study team

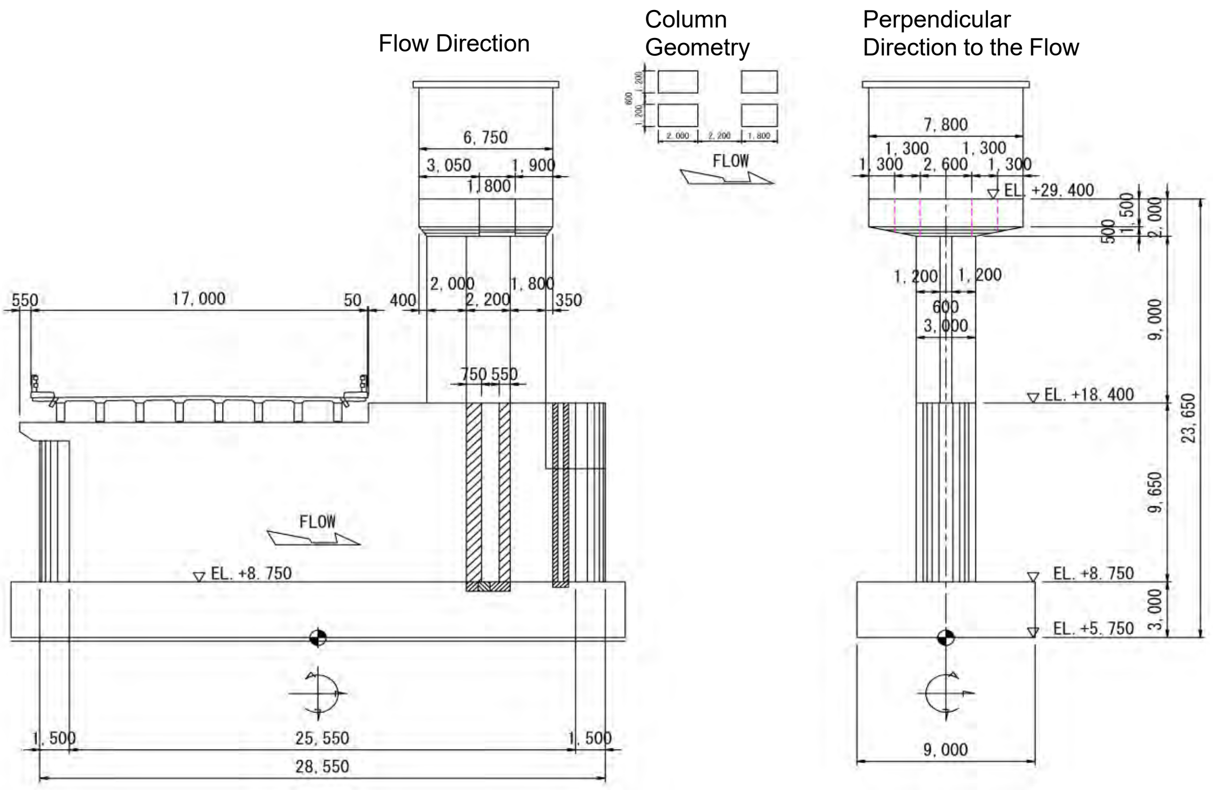
Figure 7.4.22 Pile Foundation Layout Plan

3) Study of Center Pier Pile Foundation

Design calculation of the pile foundation is performed for the adopted pile arrangement. The details of the calculation are indicated in **Vol.5A Structural Calculation for Contract Package-1**.

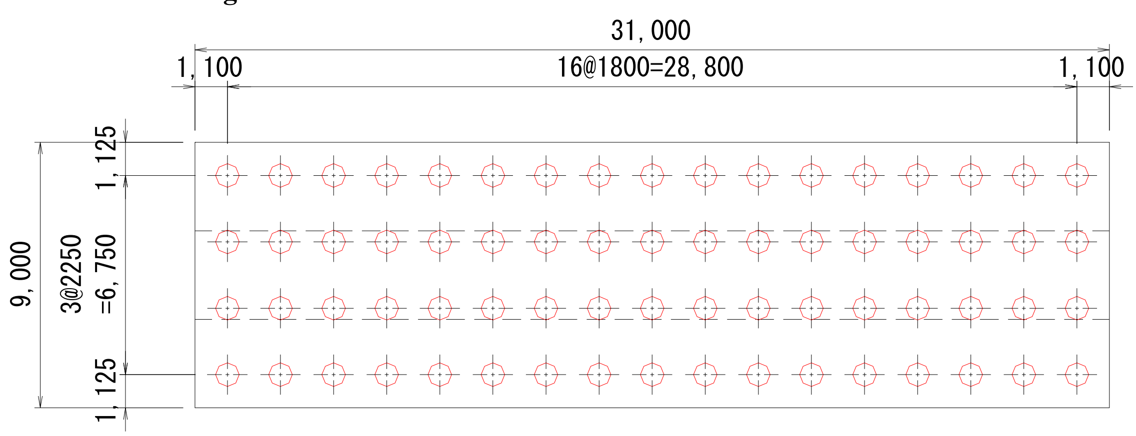
(a) Structural Dimension

The structural dimension drawing and pile arrangement of the center pier are shown.



Source: Study team

Figure 7.4.23 Dimension of Center Pier Structure



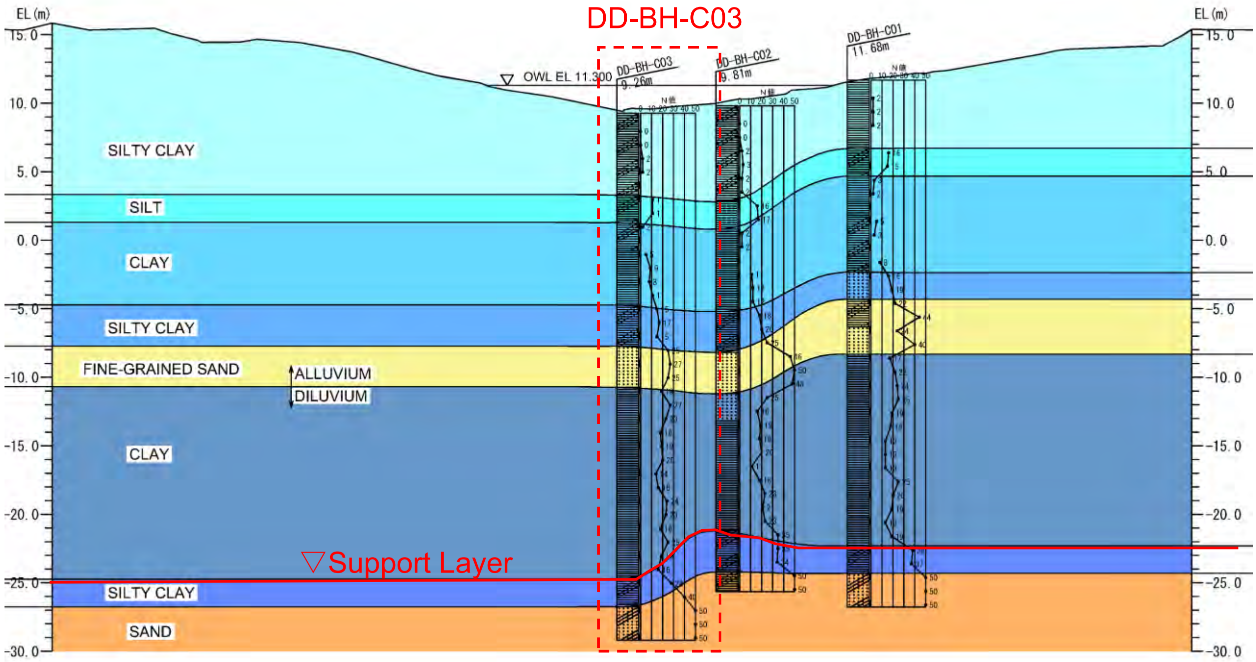
Source: Study team

Figure 7.4.24 Center Pier Pile Arrangement

(b) Ground Condition

The ground conditions used for the design of pile foundations are shown in below. At the Cainta Floodgate site, there are three geological surveys: BH-C01 to BH-C03. The geological survey

BH-C03, which is the most severe condition for pile foundation design, is adopted for the calculation because the N value of the stratum that appears is low and the depth of the support layer becomes deep.

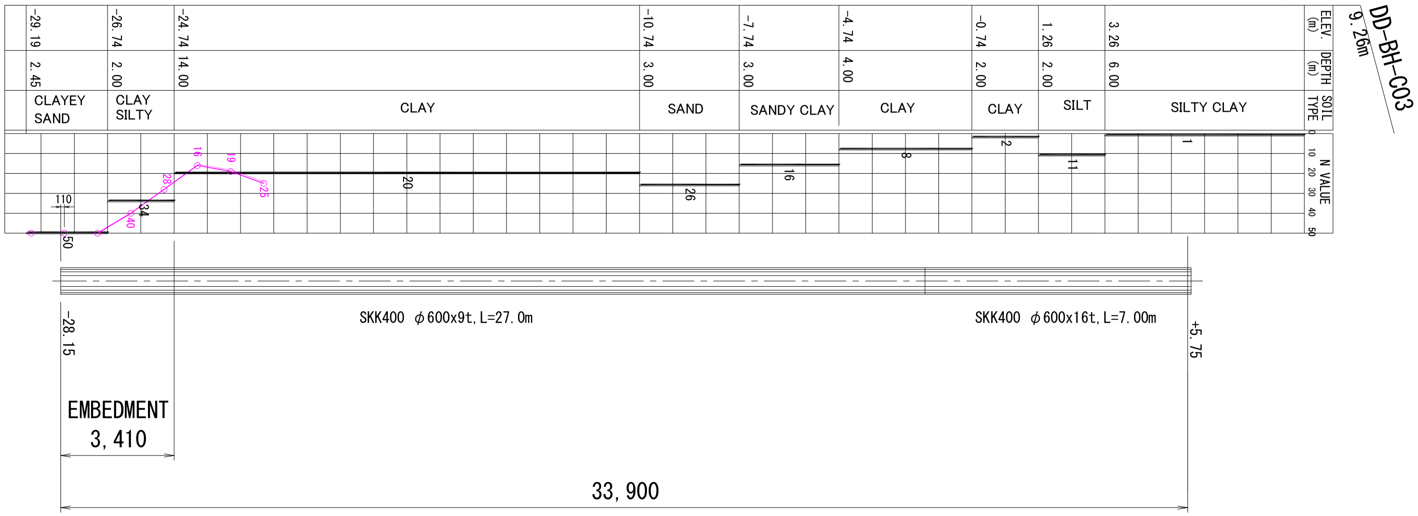


Source: Study Team

Figure 7.4.25 Assumed Geological Cross-Section

Table 7.4.18 List of soil properties (DD-BH-C03)

Stratum	Soil Type	N-value	Moisture content Wn (%)	Fine fraction content Fc (%)	Plasticity index Ip	Unit Weightγt (kN/m3)	Cohesion C (kN/m2)	Angle of shear resistance φ(°)	Deformation coefficient E 50 (MN/m2)	Consolidation settlement Target layer	Compression index Cc	Swelling index Cc
C1	Cohesive soil	1	67	93	21	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	Cohesive soil	11	43	90	16	17	130	0				
C3	Cohesive soil	2	37	70	17	(16)	(14)	(0)	(1.5)	○	(0.42)	(0.041)
C4	Cohesive soil	8	53	96	65	17	100	0				
C5	Cohesive soil	16	37	60	30	18	200	0				
S1	Sandy soil	26	19	13		20	0	36				
C6	Cohesive soil	20	46	94	45	18	250	0				
C7	Cohesive soil	34	35	74	23	19	420	0				
S2	Sandy soil	50	33	45	21	21	0	40				



Source: Study team

Figure 7.4.26 Pile Foundation Design Ground Condition

(c) Study Case

The calculation is made for the following cases.

Table 7.4.19 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

			1 Dead Load	2 Control House Load	3 Bridge Superstructure Load	4 Gate Load	5 Spiral Step Load	6 Temperature Load	7 Wind Load	8 Live Load	9 Earth Pressure	10 Upstream Water Pressure	11 Downstream Water Pressure	12 Embankment/Cover Soil Load	13 Upstream Water Weight	14 Downstream Water Weight	15 Uplift Pressure	16 Seismic Inertia Force	17 Hydrodynamic Pressure	Water Level Condition: Floodgate Main Body	Gate State	Additional Factor of Allowable Stress	Structures Subject to be Verified: End Pier	Center Pier	Wing Wall
Parpendicular Direction to Flow	CASE1	Normal	○	○	○	○	○	○		○	○	○		○	○	○	○			A	Open	1.00	○		○
	CASE2	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○		○	○	○	○			A	Open	1.25	○	○	
	CASE3	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○		○	○	○	○			B	Closed	1.25	○	○	○
	CASE4	Seismic	○	○	○	○	○				○	○		○	○	○	○	○	○	C	Open	1.33	○	○	○
	CASE 5	During construction (Left bank)	○	○	○	○	○		○		○									D	one side	1.50	○	○	○
	CASE 6	During construction (Right bank)	○	○	○	○	○		○		○									E	Open	1.50		○	
Flow Direction	CASE7	Normal	○	○	○	○	○	○		○	○	○	○	○	○	○	○			A	Open	1.00	○	○	
	CASE8	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○			A	Open	1.25	○	○	
	CASE9	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			B	Closed	1.25	○	○	
	CASE 10	Seismic	○	○	○	○	○				○	○	○	○	○	○	○	○	○	C	Open	1.33	○	○	
	CASE 11	During construction (Left bank)	○	○	○	○	○		○		○									D	Open	1.50	○	○	
	CASE 12	During construction (Right bank)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			F	Closed	1.25	○	○	

**Temperature loads are considered only for structural calculations.*
Source: Study team

(d) Load Condition

As for the load acting on the pile foundation, the load calculated in the stability calculation of the center pier is applied. The table below shows the results of the stability calculation of the center pier.

Table 7.4.20 Results of Calculation in Stability Analysis of Center pier (Perpendicular Direction to the Flow)

Case	Load name	V (kN)	H ((kN))	M ((kN)-m)
Case 1	Normal	-	-	-
Case 2	Normal + wind load	38708.59	1176.54	15096.35
Case 3	During floods (at Floodway DFL)	37748.70	767.02	12834.56
Case 4	Seismic	36368.68	9456.44	63645.19
Case 5	During construction (left bank)	44040.92	-1504.33	-19818.41
Case 6	During construction (right bank)	39832.13	11865.94	46205.27

Source: Study team

Table 7.4.21 Results of Calculation in Stability Analysis of Center Pier (Flow Direction)

Case	Load name		V (kN)	H (kN)	M (kNm)
Case 7	Normal condition		38708.59	0.00	-20902.64
Case 8 -1	Normal condition + wind load	(wind direction: upstream to downstream)	38708.59	1071.93	-40256.93
Case 8 -2	Normal condition + wind load	(wind direction: downstream to upstream)	38708.59	-1088.80	-1548.34
Case 9 -1	During floods (at Floodway DFL)	(Wind direction: upstream to downstream)	43515.20	-3126.59	-27414.58

Case	Load name		V (kN)	H (kN)	M (kNm)
Case 9 -2	During floods (at Floodway DFL)	(Wind direction: downstream to upstream)	43515.20	-4685.31	-1305.46
Case 10 -1	Seismic condition	(Inertial forces: upstream to downstream)	36368.68	9328.48	-94194.88
Case 10 -2	Seismic condition	(Inertia: downstream to upstream)	36368.68	-9328.48	34913.93
Case 11 -1	During construction	(Wind direction: upstream to downstream)	44040.92	1099.32	-58574.42
Case 11 -2	During construction	(Wind direction: downstream to upstream)	44040.92	-1116.20	-19378.00
Case 12 -1	During floods (at Tributary DFL)	(Wind direction: upstream to downstream)	46537.22	2885.57	-30249.19
Case 12 -2	During floods (at Tributary DFL)	(Wind direction: downstream to upstream)	46537.22	1207.22	-2792.23

Source: Study team

(e) Conditions for Consideration

The examination conditions of pile foundation are shown below.

- Pile Type : Steel Pipe Piles
- Construction Method : Driving Pile (Vibro hammer)
- Pile Cap Connection Condition : Rigid Ties and Hinges
- Pile Tip Condition : Type: Hinge
- Type of pile : Bearing Pile
- Allowable Displacement of Pile Normal Condition : 10.0 (mm)
  Seismic Condition : 10.0 (mm)
- Young's Modulus of Pile Body : 2.00 x 105 (N/mm2)
- Number of Piles : 68 (nos.)
- Pile Diameter : 600.0 (mm)
- Outside Corrosion Allowance : 1.0 (mm)
- Inside Corrosion Allowance : 0.0 (mm)
- Design Pile Length, Steel Pipe Thickness, Material : 33.90 (m)
  First Section: 6.90 (m) 16.0 (mm) SKK 400]
  Second Section: 27.00 (m) 9.0 (mm) SKK 400]

(f) Calculation Result

(i) Pile Foundation Calculation Result

The calculation result of the center pier pile foundation is shown in **Table 7.4.22** to **Table 7.4.24**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.22 Calculation Result of Pile Foundation of Center Pier (Perpendicular Direction To The Flow)

Load Case Number / Abbreviation		1 / Case1	2 / Case2	3 / Case3	4 / Case4
Origin Action Force					
Vo	kN	39041.10	39041.10	38081.20	36701.20
Ho	kN	0.00	1197.30	787.80	9585.00
Mo	kNm	0.00	15616.40	13328.40	65695.10
Origin Displacement					
δx	mm	0.00	1.29	0.93	6.04
δz	mm	3.74	3.74	3.65	3.52
α	rad	0.00000000	0.00025655	0.00021230	0.00116000
δf, δa	mm	0.00 ≦ 10.00	1.29 ≦ 10.00	0.93 ≦ 10.00	6.04 ≦ 10.00
Vertical Reaction Force					
PNmax, Ra	kN	574.13 ≦ 3395.00	706.97 ≦ 3395.00	669.94 ≦ 3395.00	1140.33 ≦ 5093.00
PNmin, Pa	kN	574.13 ≧ 0.00	441.30 ≧ 0.00	450.09 ≧ 0.00	-60.89 ≥ -2432.00
Horizontal Reaction Force					
PH	kN	0.00	17.61	11.59	140.96
Pile Moment					
Pile Head Mt	kNm	0.00	-19.41	-10.10	-160.04
Underground Mm	kNm	0.00	28.45	18.72	188.16
Pile Body Stress					
1 Section σ c, σ ca	N/mm2	-20.90 ≥ -140.00	-33.02 ≥ -140.00	-29.18 ≥ -140.00	-89.67 ≥ -186.20
1 Section σt, σta	N/mm2	-20.90 ≦ 140.00	-8.78 ≦ 140.00	-11.59 ≦ 140.00	50.38 ≦ 186.20
1 Section τ, τa	N/mm2	0.000 ≦ 80.000	0.641 ≦ 80.000	0.422 ≦ 80.000	5.131 ≦ 106.400
Evaluation		OK	OK	OK	OK

Load Case Number / Abbreviation		5 / case5	6 / case6
Origin Action Force			
Vo	kN	44373.40	40164.70
Ho	kN	-1525.10	11886.70
Mo	kNm	-19968.00	46189.30
Origin Displacement			
δx	mm	-1.64	9.69
δz	mm	4.25	3.85
α	rad	-0.00032787	0.00099622
δf, δa	mm	1.64 ≦ 10.00	9.69 ≦ 10.00
Vertical Reaction Force			
PNmax, Ra	kN	822.31 ≦ 3395.00	1106.47 ≦ 3395.00
PNmin, Pa	kN	482.79 ≧ 0.00	74.85 ≧ 0.00
Horizontal Reaction Force			
PH	kN	-22.43	174.80
Pile Moment			
Pile Head Mt	kNm	24.66	-287.89
Underground Mm	kNm	-36.24	282.42
Pile Body Stress			
1 Section σ c, σ ca	N/mm2	-39.21 ≥ -210.00	-113.97 ≥ -210.00
1 Section σt, σta	N/mm2	-8.30 ≦ 210.00	70.97 ≦ 210.00
1 Section τ, τa	N/mm2	0.816 ≦ 120.000	6.363 ≦ 120.000
Evaluation		OK	OK

Source: Study team

Table 7.4.23 Calculation Result of Pile Foundation of Center Pier (Flow Direction 1/2)

Load Case Number / Abbreviation			1	2	3	4
			case7	case 8 -1	case 8 -2	case 9 -1
Origin Action Force						
Vo		kN	39041.10	39041.10	39041.10	43849.10
Ho		kN	0.00	1075.70	-1088.80	-3122.80
Mo		kNm	-19130.10	-38650.70	390.40	-25871.00
Origin Displacement						
δx		mm	-0.05	0.61	-0.71	-2.10
δz		mm	3.74	3.74	3.74	4.20
α		rad	-0.00002346	-0.00004477	-0.00000219	-0.00003938
δf, δa		mm	0.05 ≦ 10.00	0.61 ≦ 10.00	0.71 ≦ 10.00	2.10 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra		kN	625.96 ≦ 3395.00	673.03 ≦ 3395.00	578.97 ≦ 3395.00	731.83 ≦ 3395.00
PNmin, Pa		kN	522.30 ≧ 0.00	475.23 ≧ 0.00	569.30 ≧ 0.00	557.85 ≧ 0.00
Horizontal Reaction Force						
PH		kN	0.00	15.82	-16.01	-45.92
Pile Moment						
Pile Head Mt		kNm	-1.44	-34.34	31.84	89.28
Underground Mm		kNm	0.05	25.56	-25.87	-74.20
Pile Body Stress						
1 Section	σ c, σ ca	N/mm2	-23.15 ≥ -140.00	-33.29 ≥ -140.00	-29.22 ≥ -140.00	-49.49 ≥ -140.00
	σt, σta	N/mm2	-18.64 ≦ 140.00	-8.51 ≦ 140.00	-12.57 ≦ 140.00	2.55 ≦ 140.00
	τ, τa	N/mm2	0.010 ≦ 80.000	0.576 ≦ 80.000	0.583 ≦ 80.000	1.672 ≦ 80.000
Evaluation			OK	OK	OK	OK
Load Case Number / Abbreviation			5	6	7	8
			case 9 -2	case 10 -1	case 10 -2	case 11 -1
Origin Action Force						
Vo		kN	43849.10	36701.20	36701.20	44373.40
Ho		kN	-4685.30	9395.00	-9395.00	1103.10
Mo		kNm	438.50	-93221.10	37802.20	-56798.00
Origin Displacement						
δx		mm	-3.06	3.79	-3.91	0.58
δz		mm	4.20	3.52	3.52	4.25
α		rad	-0.00001094	-0.00009427	0.00002637	-0.00006696
δf, δa		mm	3.06 ≦ 10.00	3.79 ≦ 10.00	3.91 ≦ 10.00	0.58 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra		kN	669.00 ≦ 3395.00	747.99 ≦ 5093.00	597.98 ≦ 5093.00	800.47 ≦ 3395.00
PNmin, Pa		kN	620.68 ≧ 0.00	331.46 ≥ -2432.00	481.47 ≥ -2432.00	504.63 ≧ 0.00
Horizontal Reaction Force						
PH		kN	-68.90	138.16	-138.16	16.22
Pile Moment						
Pile Head Mt		kNm	136.91	-246.28	241.35	-36.50
Underground Mm		kNm	-111.32	184.43	-184.43	26.21
Pile Body Stress						
1 Section	σ c, σ ca	N/mm2	-59.40 ≥ -140.00	-90.27 ≥ -186.20	-83.55 ≥ -186.20	-38.48 ≥ -210.00
	σt, σta	N/mm2	12.46 ≦ 140.00	50.98 ≦ 186.20	44.26 ≦ 186.20	-9.02 ≦ 210.00
	τ, τa	N/mm2	2.508 ≦ 80.000	5.029 ≦ 106.400	5.029 ≦ 106.400	0.590 ≦ 120.000
Evaluation			OK	OK	OK	OK

Source: Study team

Table 7.4.24 Calculation Result of Pile Foundation of Center Pier (Flow Direction 2/2)

Load Case Number Abbreviation			9	10	11
Origin Action Force			case 11 -2	case 12 -1	case 12 -2
Vo Ho Mo		kN kN kNm	44373.40 -1116.20 -17749.40	46871.10 2889.30 -28591.40	46871.10 1207.20 -937.40
Origin Displacement					
δx δz α		mm mm rad	-0.77 4.25 -0.00002450	1.82 4.49 -0.00002799	0.79 4.49 0.00000181
δf, δa		mm	0.77 ≦ 10.00	1.82 ≦ 10.00	0.79 ≦ 10.00
Vertical Reaction Force					
PNmax, Ra PNmin, Pa		kN kN	706.68 ≦ 3395.00 598.42 ≧ 0.00	751.11 ≦ 3395.00 627.45 ≧ 0.00	693.27 ≦ 3395.00 685.29 ≧ 0.00
Horizontal Reaction Force					
PH		kN	-16.41	42.49	17.75
Pile Moment					
Pile Head Mt Underground Mm		kNm kNm	31.27 -26.52	-86.56 68.65	-35.34 28.68
Pile Body Stress					
1 Section	σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-33.73 ≥ -210.00 -13.78 ≦ 210.00 0.597 ≦ 120.000	-49.50 ≥ -140.00 -0.68 ≦ 140.00 1.547 ≦ 80.000	-34.28 ≥ -140.00 -15.90 ≦ 140.00 0.646 ≦ 80.000
Evaluation			OK	OK	OK

Source: Study team

(ii) Calculation Result of Reinforcing Bar at Pile Head

The calculation condition of pile head reinforcing bar of the center pier pile foundation is shown in **Table 7.4.25** and **Table 7.4.26**. Of the calculation results, the case surrounded by the red line is the decision case.

・Pile Outside Diameter D = 600.00 (mm)
・Virtual RC Section Diameter Do = 850.00 (mm)
・Inner Diameter Ro = 0.00 (mm)
・Rebar D 25 -11 (@ 140) As = 55.74 (cm2) (Amount of reinforcing steel in JIS)

The pile head reinforcement is set to D-25 x 12 nos (As = 58.91 cm2) to exceed As = 55.74 (Amount of reinforcing steel in JIS).

Table 7.4.25 Verification of Center Pier Virtual Reinforced Concrete Section (Perpendicular Direction to the Flow)

No	Load name abbreviation	Axial force	Cross-sectional force		Neutral axis	Stress (N/mm2)		Allowance(N/mm2)		Evaluation
			M (kNm)	N (kN)	X (cm)	σ c	σ s	σ ca	σ sa	
1	Case1	Nmax Nmin	0.0	574.1 574.1	0.00 0.00	0.88 0.88	-13.23 -13.23	8.28	-168.00 -168.00	Ok Ok
2	Case2	Nmax Nmin	28.4	707.0 441.3	150.07 109.65	1.51 1.11	-20.00 -13.88	8.28	-168.00 -168.00	Ok Ok
3	Case3	Nmax Nmin	18.7	669.9 450.1	197.30 146.51	1.31 0.97	-17.88 -12.81	8.28	-168.00 -168.00	Ok Ok
4	Case4	Nmax Nmin	188.2	1140.3 -60.9	64.56 24.13	4.83 6.71	-52.25 178.71	11.01	-223.44 223.44	Ok Ok
5	Case5	Nmax Nmin	36.2	822.3 482.8	140.66 100.13	1.81 1.29	-23.67 -15.85	12.42	-252.00 -252.00	Ok Ok
6	Case6	Nmax Nmin	287.9	1106.5 74.8	46.58 25.98	7.71 9.99	-70.93 236.47	12.42	-252.00 252.00	Ok Ok

**M is the larger value of the pile head bending moment or the maximum bending moment of the underground part.*

**Positive values indicate tensile stress and negative values indicate compression.*
Source: Study team

Table 7.4.26 Verification of Center Pier Virtual Reinforced Concrete Section (Flow Direction)

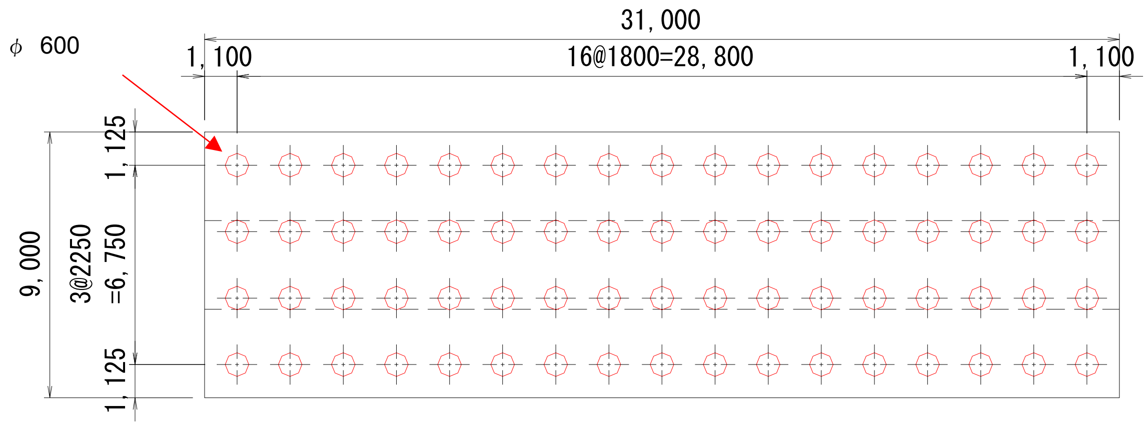
No	Load name abbreviation	Axial force	Cross-sectional force		Neutral axis	Stress (N/mm2)		Allowance(N/mm2)		Evaluation
			M (kNm)	N (kN)	X (cm)	σ c	σ s	σ ca	σ sa	
1	Case7	Nmax Nmin	1.4	626.0 522.3	1974.67 1654.60	0.98 0.82	-14.61 -12.22	8.28	-168.00 -168.00	Ok Ok
2	Case 8 -1	Nmax Nmin	34.3	673.0 475.2	127.29 102.37	1.55 1.25	-19.99 -15.43	8.28	-168.00 -168.00	Ok Ok
3	Case 8 -2	Nmax Nmin	31.8	579.0 569.3	121.18 119.86	1.37 1.35	-17.49 -17.27	8.28	-168.00 -168.00	Ok Ok
4	Case 9 -1	Nmax Nmin	89.3	731.8 557.9	77.41 65.88	2.48 2.30	-28.59 -25.09	8.28	-168.00 -168.00	Ok Ok
5	Case 9 -2	Nmax Nmin	136.9	669.0 620.7	55.43 52.46	3.51 3.54	-35.53 -34.89	8.28	-168.00 -168.00	Ok Ok
6	Case 10 -1	Nmax Nmin	246.3	748.0 331.5	40.20 30.26	7.00 8.00	69.97 145.76	11.01	223.44 223.44	Ok Ok
7	Case 10 -2	Nmax Nmin	241.3	598.0 481.5	36.38 33.52	7.18 7.47	90.68 111.92	11.01	223.44 223.44	Ok Ok
8	Case 11 -1	Nmax Nmin	36.5	800.5 504.6	137.27 102.24	1.78 1.33	-23.21 -16.39	12.42	-252.00 -252.00	Ok Ok
9	Case 11 -2	Nmax Nmin	31.3	706.7 598.4	140.06 125.11	1.56 1.39	-20.37 -17.88	12.42	-252.00 -252.00	Ok Ok
10	Case 12 -1	Nmax Nmin	86.6	751.1 627.4	79.77 72.24	2.47 2.31	-28.64 -25.99	8.28	-168.00 -168.00	Ok Ok
11	Case 12 -2	Nmax Nmin	35.3	693.3 685.3	127.37 126.39	1.60 1.59	-20.58 -20.40	8.28	-168.00 -168.00	Ok Ok

**M is the larger value of the pile head bending moment and the maximum bending moment of the underground part.*

**Positive values indicate tensile stress and negative values indicate compression.*
Source: Study team

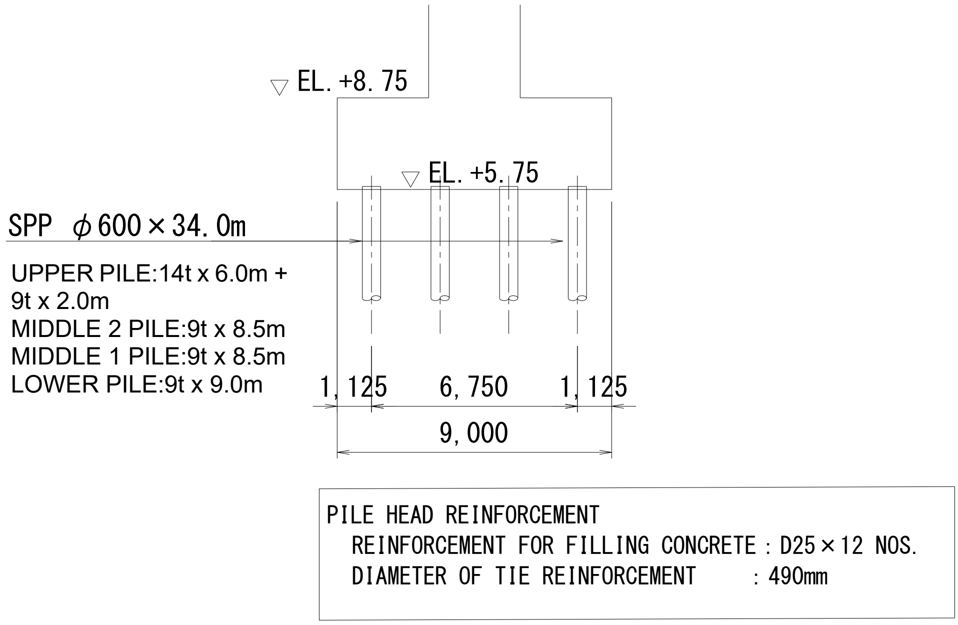
(g) Result of Examination

Results of study on the pile foundation of the center pier are shown in **Figure 7.4.27** and **Figure 7.4.28**. Details of the examination results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure 7.4.27 Center Pier Pile Arrangement



Source: Study team

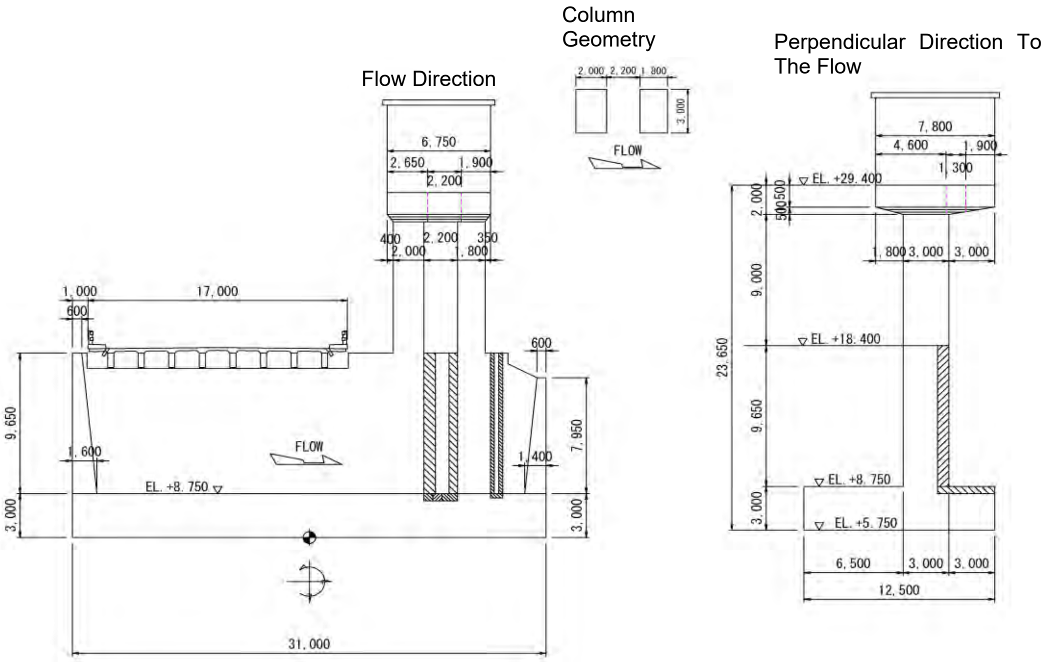
Figure 7.4.28 Pile Foundation Calculation Result

4) Study on the Pile Foundation of End Pier

The design calculation of the pile foundation is carried out for the end pier. The details of the calculation are shown in Vol.5A Structural Calculation for Contract Package-1.

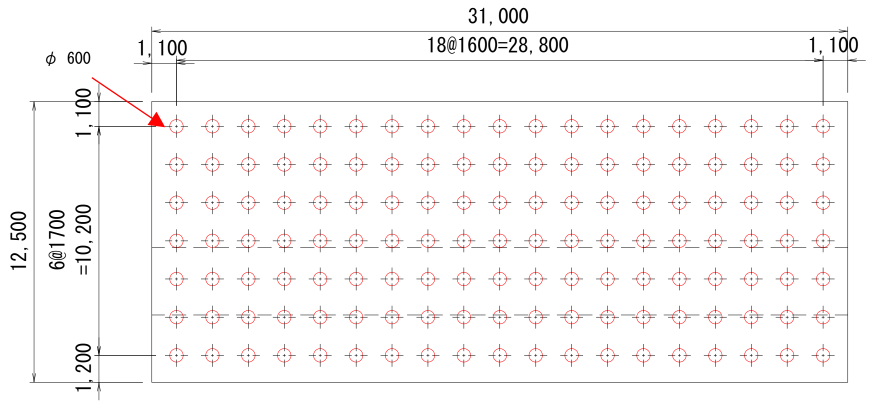
(a) Structural Dimension

The structural dimension of the end pier is shown.



Source: Study team

Figure 7.4.29 Dimension of End Pier Structure

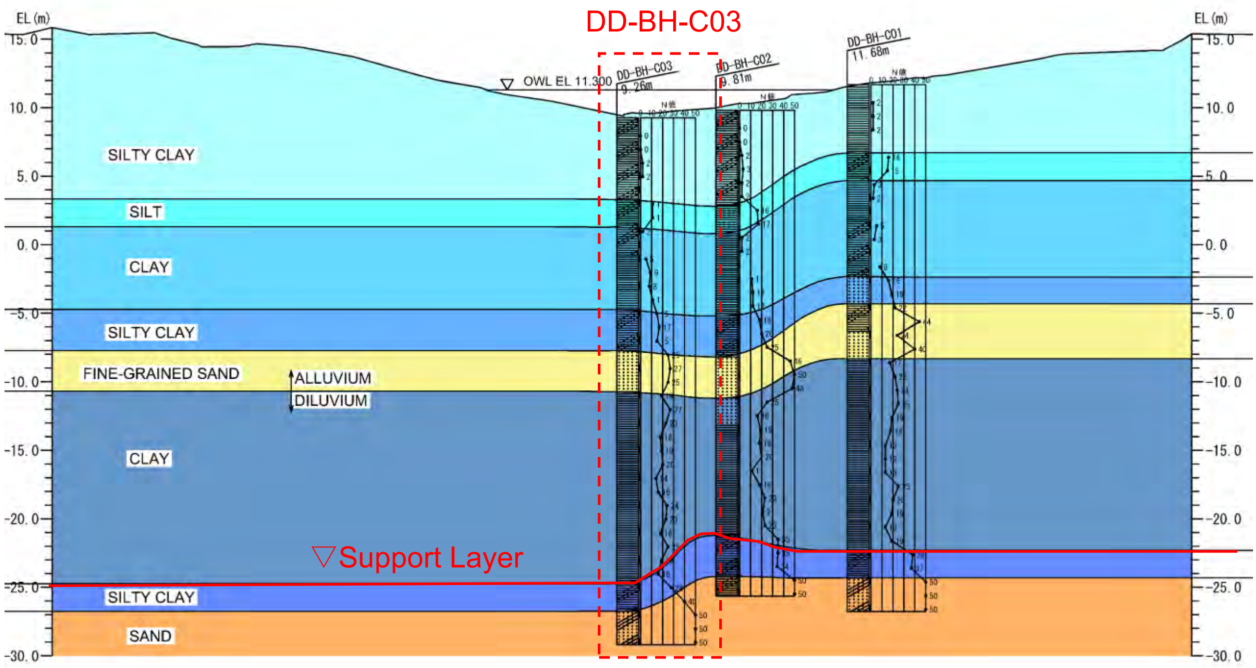


Source: Study team

Figure 7.4.30 End Pier Pile Arrangement

(b) Ground Condition

The ground conditions used for the design of pile foundations are shown in the next page. At the Cainta Floodgate site, there are three geological surveys: BH-C01 to BH-C03. The geological survey BH-C03, which is the most severe condition for pile foundation design, is adopted for the calculation because the N value of the stratum that appears is low and the depth of the support layer becomes deep.

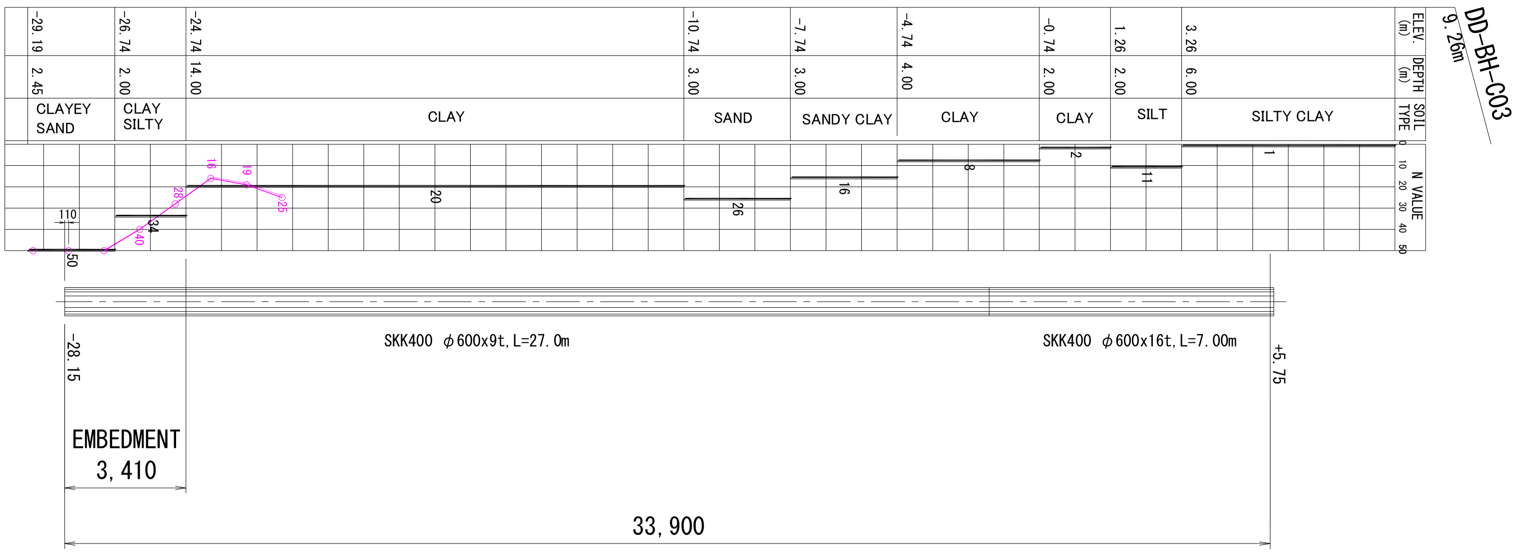


Source: Study team

Figure 7.4.31 Assumed Geological Cross-Section

Table 7.4.27 List of soil properties (DD-BH-C03)

Stratum	Soil Type	N-value	Moisture content Wn (%)	Fine fraction content Fc (%)	Plasticity index Ip	Unit Weightγt (kN/m3)	Cohesion C (kN/m2)	Angle of shear resistance ϕ(°)	Deformation coefficient E 50 (MN/m2)	Consolidation settlement Target layer	Compression index Cc	Swelling index Cc
C1	Cohesive soil	1	67	93	21	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	Cohesive soil	11	43	90	16	17	130	0				
C3	Cohesive soil	2	37	70	17	(16)	(14)	(0)	(1.5)	○	(0.42)	(0.041)
C4	Cohesive soil	8	53	96	65	17	100	0				
C5	Cohesive soil	16	37	60	30	18	200	0				
S1	Sandy soil	26	19	13		20	0	36				
C6	Cohesive soil	20	46	94	45	18	250	0				
C7	Cohesive soil	34	35	74	23	19	420	0				
S2	Sandy soil	50	33	45	21	21	0	40				



Source: Study team

Figure 7.4.32 Pile Foundation Design Ground Condition

(c) Study Case

The calculation is made for the following cases.

Table 7.4.28 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	Water Level Condition			Structures Subject to be Verified		
			Dead Load	Control House Load	Bridge Superstructure Load	Gate Load	Spiral Step Load	Temperature Load	Wind Load	Live Load	Earth Pressure	Upstream Water Pressure	Downstream Water Pressure	Embankment/Cover Soil Load	Upstream Water Weight	Downstream Water Weight	Uplift Pressure	Seismic Inertia Force	Hydrodynamic Pressure	Floodgate Main Body	Gate State	Additional Factor of Allowable Stress	End Pier	Center Pier	Wing Wall
Parpendicular Direction to Flow	CASE1	Normal	○	○	○	○	○	○		○	○	○		○	○	○	○			A	Open	1.00	○		○
	CASE2	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○		○	○	○	○			A	Open	1.25	○	○	
	CASE3	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○		○	○	○	○			B	Closed	1.25	○	○	○
	CASE4	Seismic	○	○	○	○	○				○	○		○	○	○	○	○	○	C	Open	1.33	○	○	○
	CASE 5	During construction (Left bank)	○	○	○	○	○		○		○									D	one side	1.50	○	○	○
	CASE 6	During construction (Right bank)	○	○	○	○	○		○		○									E	Open	1.50		○	
Flow Direction	CASE7	Normal	○	○	○	○	○	○		○	○	○	○	○	○	○	○			A	Open	1.00	○	○	
	CASE8	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○			A	Open	1.25	○	○	
	CASE9	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			B	Closed	1.25	○	○	
	CASE 10	Seismic	○	○	○	○	○				○	○	○	○	○	○	○	○	○	C	Open	1.33	○	○	
	CASE 11	During construction (Left bank)	○	○	○	○	○		○		○									D	Open	1.50	○	○	
	CASE 12	During construction (Right bank)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			F	Closed	1.25	○	○	

**Temperature loads are considered only for structural calculations.*

Source: Study team

(d) Load Condition

For the load acting on the pile foundation, the load calculated in the stability calculation of the end pier is applied. The table below shows the Calculation Results of Stability Analysis of the end pier.

Table 7.4.29 Calculation Results of Stability Analysis of End Pier (Perpendicular Direction to the Flow)

Case	Load name	V (kN)	H (kN)	M (kNm)
Case 1	Normal condition	79712.98	16146.08	6377.04
Case 2	Normal condition + wind load	79712.98	16591.55	-3188.52
Case 3	During floods (at Floodway DFL)	75951.57	20306.81	-18987.89
Case 4	Seismic condition	81001.29	38617.36	-136892.18
Case 5	Left bank construction	101565.53	13574.89	36563.59
Case 6	During construction on the right bank	-	-	-

Source: Study team

Table 7.4.30 Calculation Results of Stability Analysis of End Pier (Flow Direction)

Case	Load name		V (kN)	H (kN)	M (kNm)
Case 7	Normal condition		73345.12	0.00	-27137.69
Case 8 -1	Normal condition + wind load	(Wind direction: upstream to downstream)	73345.12	701.63	-41806.72
Case 8 -2	Normal condition + wind load	(Wind direction: downstream to upstream)	73345.12	-734.78	-12468.67
Case 9 -1	During floods (at Floodway DFL)	(Wind direction: upstream to downstream)	68062.99	-986.95	-20418.90
Case 9 -2	During floods (at Floodway DFL)	(Wind direction: downstream to upstream)	68062.99	-2146.81	2041.89
Case 10 -1	Seismic condition	(Inertial forces: upstream to downstream)	75698.79	18890.17	-158967.46
Case 10 -2	Seismic condition	(Inertia: downstream to upstream)	75698.79	-18890.17	98408.43
Case 11 -1	During construction	(Wind direction: upstream to downstream)	93985.25	701.63	-45112.92
Case 11 -2	During construction	(Wind direction: downstream to upstream)	93985.25	-734.78	-15977.49
Case 12 -1	During floods (at Tributary DFL)	(Wind direction: upstream to downstream)	72822.09	1398.03	-44421.47
Case 12 -2	During floods (at Tributary DFL)	(Wind direction: downstream to upstream)	72822.09	187.80	-21118.41

Source: Study team

(e) Conditions for consideration

The examination conditions of pile foundation are shown below.

- Pile Type : Steel Pipe Piles
- Construction Method : Driving Pile (Vibro hammer)
- Pile Cap Connection Condition : Rigid Ties and Hinges
- Pile Tip Condition : Type: Hinge
- Type of pile : Bearing Pile
- Allowable Displacement of Pile Normal Condition : 10.0 (mm)
  Seismic Condition : 10.0 (mm)
- Young's Modulus of Pile Body : 2.00 x 105 (N/mm2)
- Number of Piles : 68 (nos.)
- Pile Diameter : 600.0 (mm)
- Outside Corrosion Allowance : 1.0 (mm)
- Inside Corrosion Allowance : 0.0 (mm)
- Design Pile Length, Steel Pipe Thickness, Material : 33.90 (m)

First Section: 6.90 (m) 16.0 (mm) SKK 400]

Second Section: 27.00 (m) 9.0 (mm) SKK 400]

(f) Calculation Result

(i) Pile Foundation Calculation Result

The calculation result of the end pier pile foundation is shown in **Table 7.4.31** to **Table 7.4.33**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.31 Calculation Result of Foundation Pile of End Pier (Perpendicular Direction To The Flow)

Load Case Number / Abbreviation			1	2	3	4
			case1	case2	case3	case4
Origin Action Force						
Vo		kN	79814.50	79814.50	76054.00	81101.40
Ho		kN	16146.10	16591.50	20306.80	38637.40
Mo		kNm	-6385.20	3192.60	19013.50	137872.41
Origin Displacement						
δx		mm	5.59	5.83	7.25	9.78
δz		mm	3.92	3.92	3.74	4.02
α		rad	0.00012230	0.00016520	0.00025966	0.00085076
δf, δa		mm	5.59 ≦ 10.00	5.83 ≦ 10.00	7.25 ≦ 10.00	9.78 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra		kN	695.80 ≦ 3562.00	729.36 ≦ 3562.00	774.99 ≦ 3562.00	1275.43 ≦ 5344.00
PNmin, Pa		kN	504.42 ≧ 0.00	470.86 ≧ 0.00	368.68 ≧ 0.00	-55.86 ≥ -2432.00
Horizontal Reaction Force						
PH		kN	121.40	124.75	152.68	290.51
Pile Moment						
Pile Head Mt		kNm	-234.90	-238.96	-288.94	-441.66
Underground Mm		kNm	196.14	201.55	246.68	387.79
Pile Body Stress						
1 Section	σ c, σ ca	N/mm2	-85.46 ≥ -140.00	-87.72 ≥ -140.00	-102.18 ≥ -140.00	-159.49 ≥ -186.20
	σt, σta	N/mm2	41.77 ≦ 140.00	44.03 ≦ 140.00	60.55 ≦ 140.00	115.10 ≦ 186.20
	τ, τa	N/mm2	4.419 ≦ 80.000	4.541 ≦ 80.000	5.558 ≦ 80.000	10.574 ≦ 106.400
Evaluation			OK	OK	OK	OK

Load Case Number / Abbreviation			5
			case5
Origin Action Force			
Vo		kN	101662.80
Ho		kN	13574.90
Mo		kNm	-35582.00
Origin Displacement			
δx		mm	4.47
δz		mm	4.98
α		rad	-0.00001390
δf, δa		mm	4.47 ≦ 10.00
Vertical Reaction Force			
PNmax, Ra		kN	775.26 ≦ 3562.00
PNmin, Pa		kN	753.51 ≧ 0.00
Horizontal Reaction Force			
PH		kN	102.07
Pile Moment			
Pile Head Mt		kNm	-204.66
Underground Mm		kNm	164.91
Pile Body Stress			
1 Section	σ c, σ ca	N/mm2	-80.61 ≥ -210.00
	σt, σta	N/mm2	24.97 ≦ 210.00
	τ, τa	N/mm2	3.715 ≦ 120.000
Evaluation			OK

Source: Study team

Table 7.4.32 Result of foundation Calculation For End Pier Pile (Flow Direction 1/2)

Load Case Number Abbreviation / Origin Action Force			1 case7	2 case 8 -1	3 case 8 -2	4 case 9 -1
Vo Ho Mo		kN kN kNm	73446.60 0.00 -27175.30	73446.60 701.60 -41130.10	73446.60 -734.80 -12485.90	68165.40 -987.00 -20449.60
Origin Displacement						
δx δz α		mm mm rad	-0.03 3.60 -0.00001725	0.18 3.60 -0.00002522	-0.26 3.60 -0.00000886	-0.36 3.34 -0.00001423
δf, δa		mm	0.03 ≦ 10.00	0.18 ≦ 10.00	0.26 ≦ 10.00	0.36 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra PNmin, Pa		kN kN	590.34 ≦ 3562.00 514.12 ≧ 0.00	607.95 ≦ 3562.00 496.51 ≧ 0.00	571.80 ≦ 3562.00 532.66 ≧ 0.00	543.97 ≦ 3562.00 481.08 ≧ 0.00
Horizontal Reaction Force						
PH		kN	0.00	5.28	-5.52	-7.42
Pile Moment						
Pile Head Mt Underground Mm		kNm kNm	-1.06 0.04	-12.08 8.52	10.49 -8.93	13.94 -11.99
Pile Body Stress						
1 Section	σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-21.76 ≥ -140.00 -18.44 ≦ 140.00 0.007 ≦ 80.000	-25.22 ≥ -140.00 -14.98 ≦ 140.00 0.192 ≦ 80.000	-23.50 ≥ -140.00 -16.70 ≦ 140.00 0.201 ≦ 80.000	-23.37 ≥ -140.00 -13.94 ≦ 140.00 0.270 ≦ 80.000
Evaluation			OK	OK	OK	OK
Load Case Number Abbreviation / Origin Action Force			5 case 9 -2	6 case 10 -1	7 case 10 -2	8 case 11 -1
Vo Ho Mo		kN kN kNm	68165.40 -2146.80 2045.00	75798.90 18910.20 -159177.80	75798.90 -18910.20 99296.60	94082.50 701.60 -44218.80
Origin Displacement						
δx δz α		mm mm rad	-0.71 3.34 -0.00000142	3.93 3.71 -0.00008017	-3.99 3.71 0.00004219	0.18 4.61 -0.00002718
δf, δa		mm	0.71 ≦ 10.00	3.93 ≦ 10.00	3.99 ≦ 10.00	0.18 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra PNmin, Pa		kN kN	515.67 ≦ 3562.00 509.38 ≧ 0.00	747.03 ≦ 5344.00 392.80 ≥ -2432.00	663.13 ≦ 5344.00 476.71 ≥ -2432.00	767.44 ≦ 3562.00 647.34 ≧ 0.00
Horizontal Reaction Force						
PH		kN	-16.14	142.18	-142.18	5.28
Pile Moment						
Pile Head Mt Underground Mm		kNm kNm	32.14 -26.08	-252.22 189.80	249.46 -189.80	-12.20 8.52
Pile Body Stress						
1 Section	σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-27.00 ≥ -140.00 -10.31 ≦ 140.00 0.588 ≦ 80.000	-91.76 ≥ -186.20 50.27 ≦ 186.20 5.175 ≦ 106.400	-88.00 ≥ -186.20 46.51 ≦ 186.20 5.175 ≦ 106.400	-31.06 ≥ -210.00 -20.44 ≦ 210.00 0.192 ≦ 120.000
Evaluation			OK	OK	OK	OK

Source: Study team

Table 7.4.33 Result of foundation Calculation For End Pier Pile (Flow Direction 2/2)

Load Case Number Abbreviation		9 case 11 -2	10 case 12 -1	11 case 12 -2
Origin Action Force				
Vo Ho Mo	kN kN kNm	94082.50 -734.80 -15053.20	73033.40 1398.00 -44550.40	73033.40 187.80 -21910.00
Origin Displacement				
δx δz α	mm mm rad	-0.26 4.61 -0.00001049	0.41 3.58 -0.00002651	0.03 3.58 -0.00001367
δf, δa	mm	0.26 ≦ 10.00	0.41 ≦ 10.00	0.03 ≦ 10.00
Vertical Reaction Force				
PNmax, Ra PNmin, Pa	kN kN	730.56 ≦ 3562.00 684.22 ≧ 0.00	607.69 ≦ 3562.00 490.56 ≧ 0.00	579.33 ≦ 3562.00 518.92 ≧ 0.00
Horizontal Reaction Force				
PH	kN	-5.52	10.51	1.41
Pile Moment				
Pile Head Mt Underground Mm	kNm kNm	10.39 -8.93	-22.62 16.98	-3.66 2.28
Pile Body Stress				
1 Section σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-29.25 ≥ -210.00 -22.25 ≦ 210.00 0.201 ≦ 120.000	-27.91 ≥ -140.00 -12.07 ≦ 140.00 0.383 ≦ 80.000	-22.02 ≥ -140.00 -17.95 ≦ 140.00 0.051 ≦ 80.000
Evaluation		OK	OK	OK

Source: Study team

(ii) Calculation Result of Reinforcing Bar at Pile Head

The calculation result of pile head reinforcing bar of the end pier pile foundation is shown in **Table 7.4.34** and **Table 7.4.35**See. Of the calculation results, the case surrounded by the red line is the decision case.

- Pile Outside Diameter D = 600.00 (mm)
- Virtual RC Section Diameter Do = 850.00 (mm)
- Inner Diameter Ro = 0.00 (mm)
- Rebar D 32 -17 (@ 87) As 136.73 cm2

Table 7.4.34 Verification Of Virtual Reinforced Concrete Section of End Pier (Perpendicular Direction To The Flow)

No	Load name abbreviation	Axial force	Cross-sectional force		Neutral axis	Stress (N/mm2)		Allowance(N/mm2)		Evaluation
			M (kNm)	N (kN)	X (cm)	σ c	σ s	σ ca	σ sa	
1	Case1	Nmax Nmin	234.9	695.8 504.4	43.49 39.29	5.40 5.50	-45.63 56.14	8.28	-168.00 168.00	Ok Ok
2	Case2	Nmax Nmin	239.0	729.4 470.9	43.98 38.45	5.49 5.62	-46.74 60.42	8.28	-168.00 168.00	Ok Ok
3	Case3	Nmax Nmin	288.9	775.0 368.7	41.98 35.43	6.69 6.90	57.39 89.27	8.28	168.00 168.00	Ok Ok
4	Case4	Nmax Nmin	441.7	1275.4 -55.9	43.08 30.37	10.18 10.78	-85.31 189.61	11.01	-223.44 223.44	Ok Ok
5	Case5	Nmax Nmin	204.7	775.3 753.5	48.31 47.66	4.63 4.64	-42.15 -41.85	12.42	-252.00 -252.00	Ok Ok

**M is the larger value of the pile head bending moment and the maximum bending moment of the underground part.*

**Positive values indicate tensile stress and negative values indicate compression.*
Source: Study team

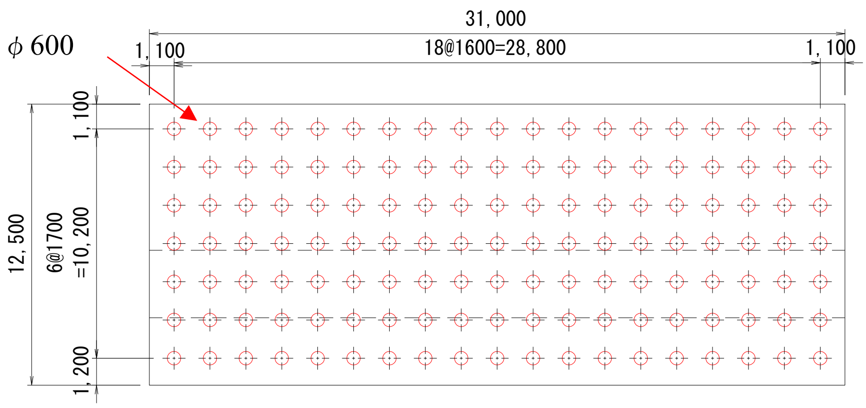
Table 7.4.35 Verification of virtual reinforced concrete section of end pier (Flow Direction)

No	Load name abbreviation	Axial force	Cross-sectional force		Neutral axis	Stress (N/mm2)		Allowance(N/mm2)		Evaluation
			M (kNm)	N (kN)	X (cm)	σ c	σ s	σ ca	σ sa	
1	Case7	Nmax Nmin	1.1	590.3 514.1	2218.12 1937.28	0.78 0.68	-11.62 -10.14	8.28	-168.00 -168.00	Ok Ok
2	Case 8 -1	Nmax Nmin	12.1	607.9 496.5	246.18 208.86	0.95 0.81	-13.21 -11.04	8.28	-168.00 -168.00	Ok Ok
3	Case 8 -2	Nmax Nmin	10.5	571.8 532.7	263.28 248.18	0.89 0.83	-12.33 -11.56	8.28	-168.00 -168.00	Ok Ok
4	Case 9 -1	Nmax Nmin	13.9	544.0 481.1	201.17 182.82	0.90 0.81	-12.17 -10.94	8.28	-168.00 -168.00	Ok Ok
5	Case 9 -2	Nmax Nmin	32.1	515.7 509.4	107.63 106.84	1.11 1.10	-13.67 -13.55	8.28	-168.00 -168.00	Ok Ok
6	Case 10 -1	Nmax Nmin	252.2	747.0 392.8	43.48 36.61	5.80 5.99	-48.99 72.10	11.01	-223.44 223.44	Ok Ok
7	Case 10 -2	Nmax Nmin	249.5	663.1 476.7	41.85 38.18	5.78 5.88	50.02 64.26	11.01	223.44 223.44	Ok Ok
8	Case 11 -1	Nmax Nmin	12.2	767.4 647.3	297.51 257.60	1.16 1.01	-16.33 -13.99	12.42	-252.00 -252.00	Ok Ok
9	Case 11 -2	Nmax Nmin	10.4	730.6 684.2	327.31 309.22	1.09 1.03	-15.41 -14.50	12.42	-252.00 -252.00	Ok Ok
10	Case 12 -1	Nmax Nmin	22.6	607.7 490.6	151.51 130.51	1.10 0.94	-14.39 -12.11	8.28	-168.00 -168.00	Ok Ok
11	Case 12 -2	Nmax Nmin	3.7	579.3 518.9	677.25 611.07	0.80 0.72	-11.70 -10.53	8.28	-168.00 -168.00	Ok Ok

**M is the larger value of the pile head bending moment and the maximum bending moment of the underground part.*
**Positive values indicate tensile stress and negative values indicate compression.*
Source: Study team

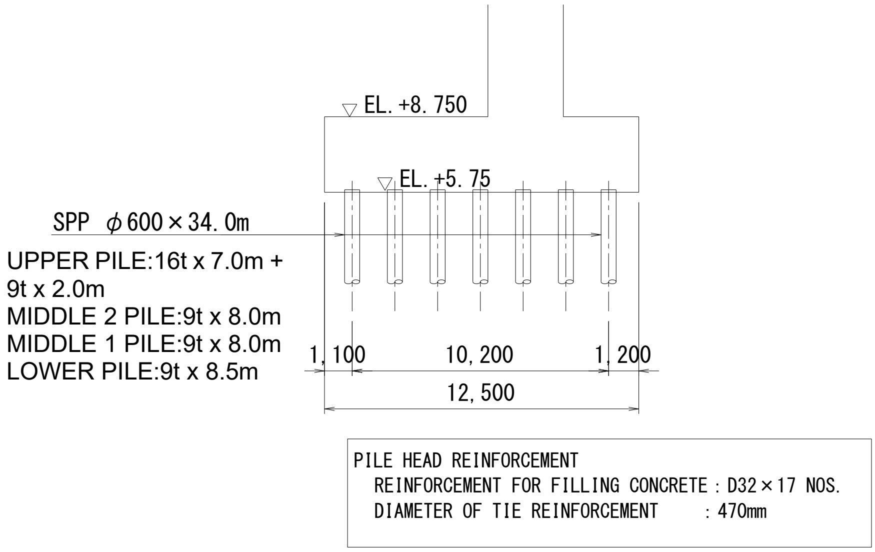
(g) Result Of Examination

Results of study on pile foundation of end pier are shown in **Figure 7.4.38** and **Figure 7.4.39**. Details of the examination results are shown in Vol.5A Structural Calculation for Contract Package-1.



Source: Study team

Figure 7.4.33 End Pier Pile Arrangement



Source: Study team

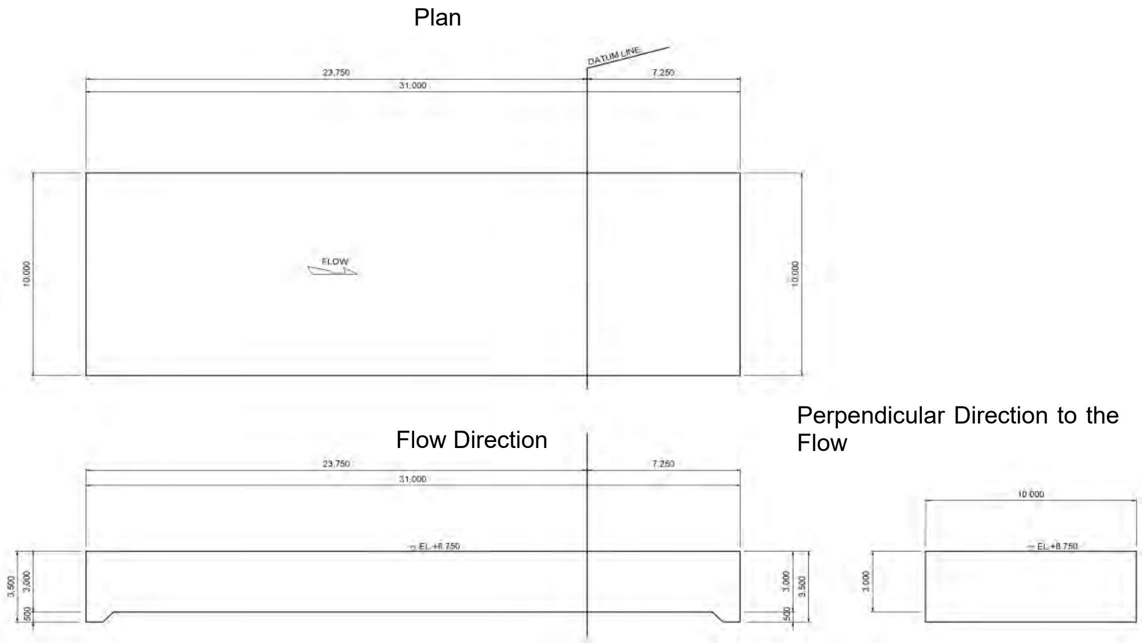
Figure 7.4.34 Pile Foundation Calculation Result

5) Study on Pile Foundation for Floor Slab

A pile foundation is designed and calculated for a floor slab. The details of the calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**.

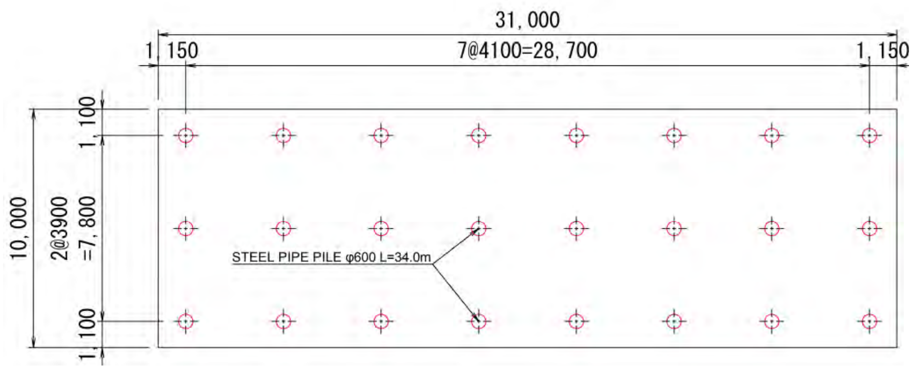
(a) Structural Dimension

The structural dimension drawing of the floor slab is shown.



Source: Study team

Figure 7.4.35 Structural Dimension of Floor slab



Source: Study team

Figure 7.4.36　Floor slab pile arrangement

(b) Ground Condition

As for the ground condition, the ground condition of DD-BH-C03 is used as in the case of the center pier and the end pier.

(c) Study Case

The stud cases indicated in **Table 7.4.36** are calculated.

Table 7.4.36　Load Case List

Member	Calculation Direction	Case	Case Name	Additional Factor of Allowable Stress
Floor slab	Flow Direction	1	During floods (at Floodway DFL)	1.0
		2	During floods (at Tributary DFL)	1.0
		3	During Construction	1.5
	Perpendicular Direction to the Flow	1	During Construction, Loading at the End	1.5
		2	During Construction, Loading at the Center	1.5
		3	During Flood	1.0

Source: Study team

(d) Load Condition

For loads acting on pile foundations, the load is summarized from the diagram set for each load case in **Table 7.4.36** . The crane load during construction is considered for the floor slab.

(i) Crane Load

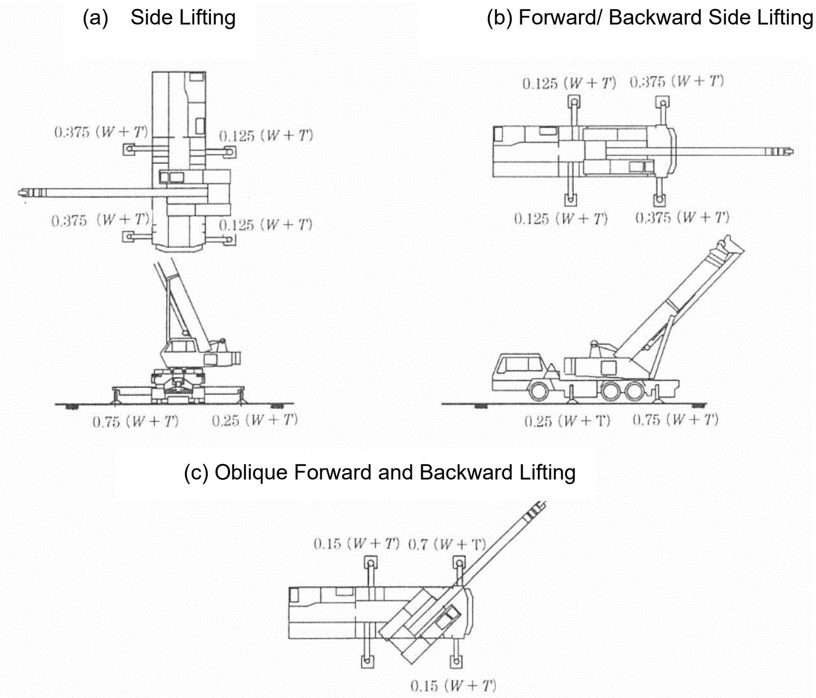
The crane load is assumed to be a 70 ton rafter crane capable of lifting the divided gate leaf (10 tons or less), and the reaction force at the outrigger position is loaded.

Crane Weight	: 41.3 t (Assumed from Manufacturer's Data)
Lifting load	: 10.0 t (Gate Leaf Dead Weight)
Hook Weight	: 0.7 t
Total	W = 52.0 t

The reaction force applied to the outrigger is from "Oblique forward and backward lifting" in the following document.

Crane Load 1: W × 0.70 = 52.0 × 0.70 = 36.4 t

Crane Load 2: W × 0.15 = 52.0 × 0.15 = 7.8 t



Source: Manufacturer

When the above load is applied and distributed with steel plate on the floor (1.5 m x 3.0 m),

Crane load 1: 36.4/(1.5 x 3.0) = 8.09 t/m2

= 8.09 × 9.8 = 79.28 kN/m2 ≒ 80 kN/m2

Crane load 2: 7.8/(1.5 x 3.0) = 1.73 t/m2

= 1.73 × 9.8 = 16.95 kN/m2 ≒ 20 kN/m2

The diagram set is shown on the following page.

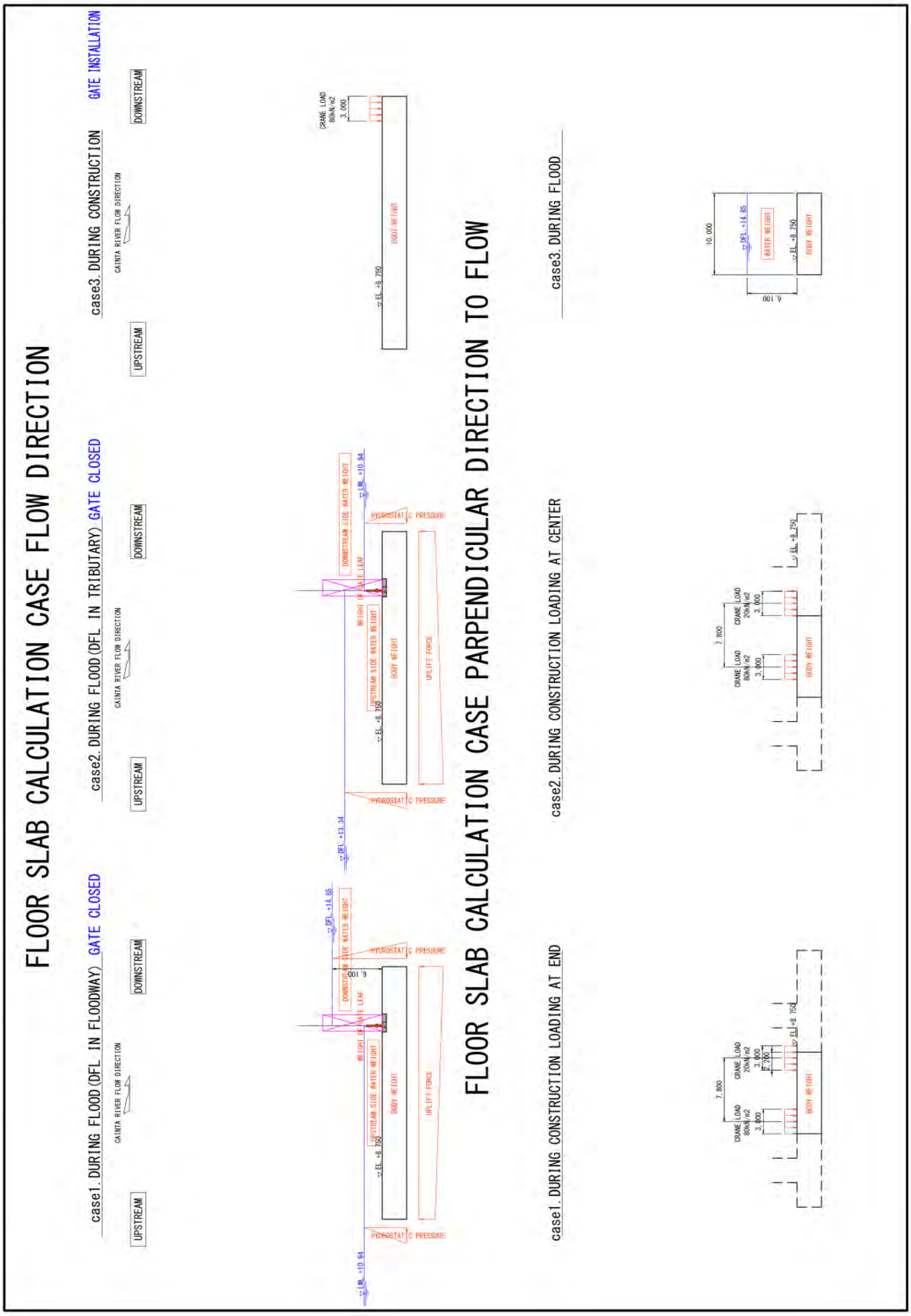


Figure 7.4.37 Load Diagram of Floor Slab

Source: Study team

(e) Conditions for Consideration

The examination conditions of pile foundation are shown below.

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 24 (nos.)
・Pile Diameter		: 600.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 33.90 (m)

First Section: 5.90 (m) 14.0 (mm) SKK 400]
Second Section: 28.00 (m) 9.0 (mm) SKK 400]

(f) Calculation Result

The calculation result of the floor slab pile foundation is shown in **Table 7.4.37** and **Table 7.4.38**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.37 Calculation Result of Pile Foundation for Floor Slab (Perpendicular Direction to the Flow)

Load Case Number Abbreviation		1 Perpendicular, Case1	2 Perpendicular, Case2	3 Perpendicular, Case3
Origin Action Force				
Vo	kN	22746.00	22680.00	36264.42
Ho	kN	0.00	0.00	0.00
Mo	kNm	909.84	453.60	0.00
Origin Displacement				
δx	mm	0.05	0.03	0.00
δz	mm	7.10	7.08	11.33
α	rad	0.00002692	0.00001342	0.00000000
δf, δa	mm	0.05 ≦ 10.00	0.03 ≦ 10.00	0.00 ≦ 10.00
Vertical Reaction Force				
PNmax, Ra	kN	961.76 ≦ 3395.00	951.98 ≦ 3395.00	1511.02 ≦ 3395.00
PNmin, Pa	kN	933.74 ≧ 0.00	938.02 ≧ 0.00	1511.02 ≧ 0.00
Horizontal Reaction Force				
PH	kN	0.00	0.00	0.00
Pile Moment				
Pile Head Mt	kNm	1.49	0.74	0.00
Underground Mm	kNm	-0.06	-0.03	0.00
Pile Body Stress				
Section 1 σ c, σ ca	N/mm2	-40.69 ≥ -210.00	-40.06 ≥ -210.00	-63.24 ≥ -140.00
Section 1 σt, σta	N/mm2	-38.65 ≦ 210.00	-39.04 ≦ 210.00	-63.24 ≦ 140.00
Section 1 τ, τa	N/mm2	0.012 ≦ 120.000	0.006 ≦ 120.000	0.000 ≦ 80.000
Evaluation		OK	OK	OK

Source: Study team

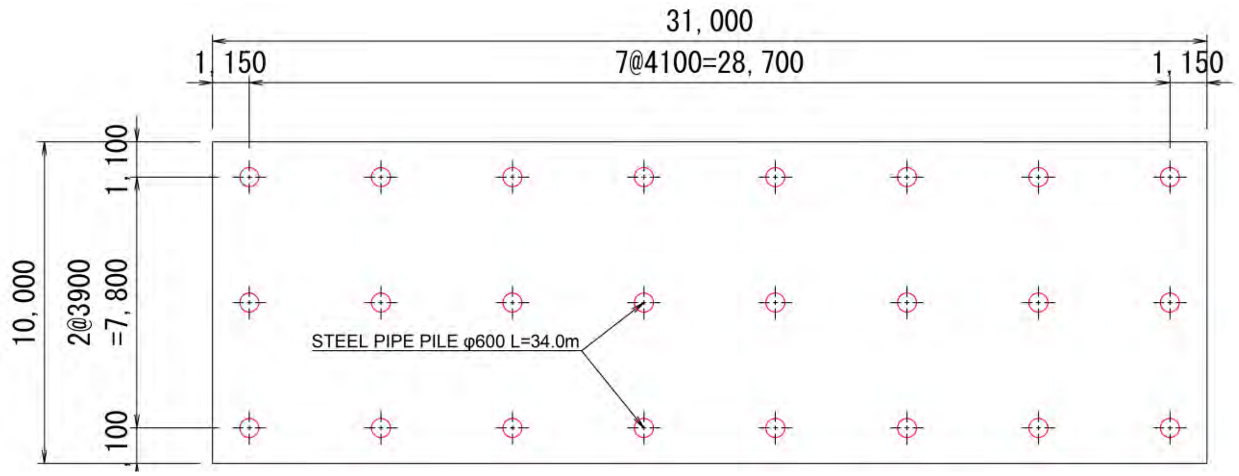
Table 7.4.38 Calculation Result of Pile Foundation For Floor Slab (Flow Direction)

Load Case Number Abbreviation		1 Flow, Case 1	2 Flow, Case 2	3 Flow, Case 3
Origin Action Force				
Vo	kN	20827.03	25038.39	22680.00
Ho	kN	-4052.41	2149.91	0.00
Mo	kNm	-5415.03	5758.83	-5216.40
Origin Displacement				
δx	mm	-7.89	4.21	-0.04
δz	mm	6.50	7.82	7.08
α	rad	-0.00004685	0.00003502	-0.00001837
δf, δa	mm	7.89 ≦ 10.00	4.21 ≦ 10.00	0.04 ≦ 10.00
Vertical Reaction Force				
PNmax, Ra	kN	957.48 ≦ 3395.00	1110.31 ≦ 3395.00	980.18 ≦ 3395.00
PNmin, Pa	kN	778.10 ≧ 0.00	976.22 ≧ 0.00	909.82 ≧ 0.00
Horizontal Reaction Force				
PH	kN	-168.85	89.58	0.00
Pile Moment				
Pile Head Mt	kNm	325.97	-172.37	-1.02
Underground Mm	kNm	-261.53	138.75	0.04
Pile Body Stress				
Section 1 σ c, σ ca	N/mm2	-135.39 ≧ -140.00	-96.88 ≧ -140.00	-41.32 ≧ -210.00
Section 1 σt, σta	N/mm2	62.75 ≦ 140.00	9.54 ≦ 140.00	-37.78 ≦ 210.00
Section 1 τ, τa	N/mm2	7.067 ≦ 80.000	3.749 ≦ 80.000	0.008 ≦ 120.000
Evaluation		OK	OK	OK

Source: Study team

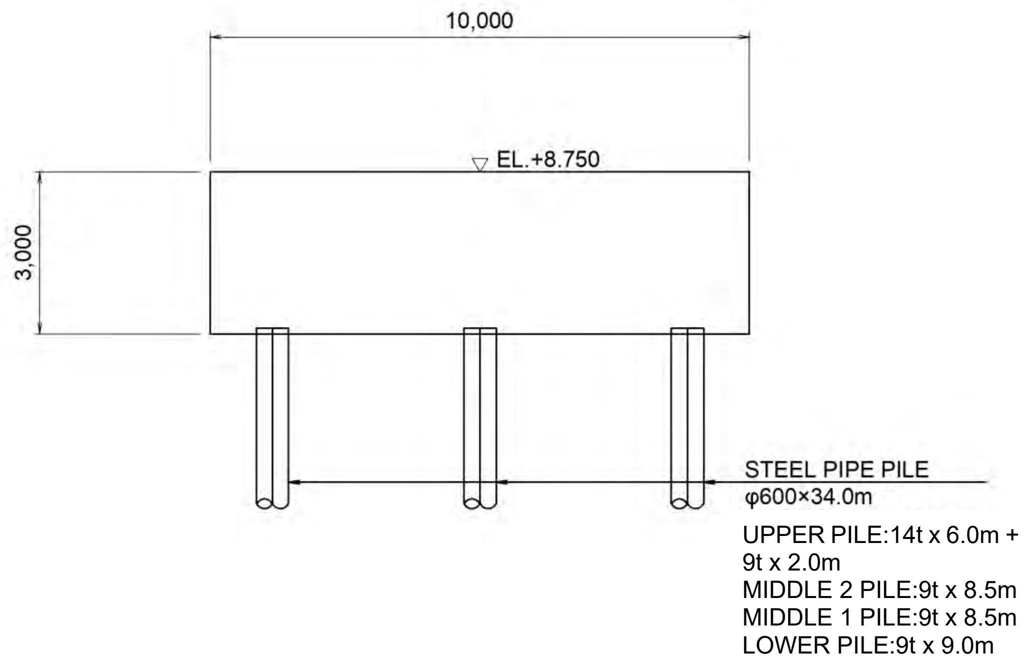
(g) Result Of Examination

Results of study on pile foundation of end pier are shown in **Figure 7.4.38** and **Figure 7.4.39**. Details of the examination results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure 7.4.38 Floor Slab Pile Arrangement



Source: Study team

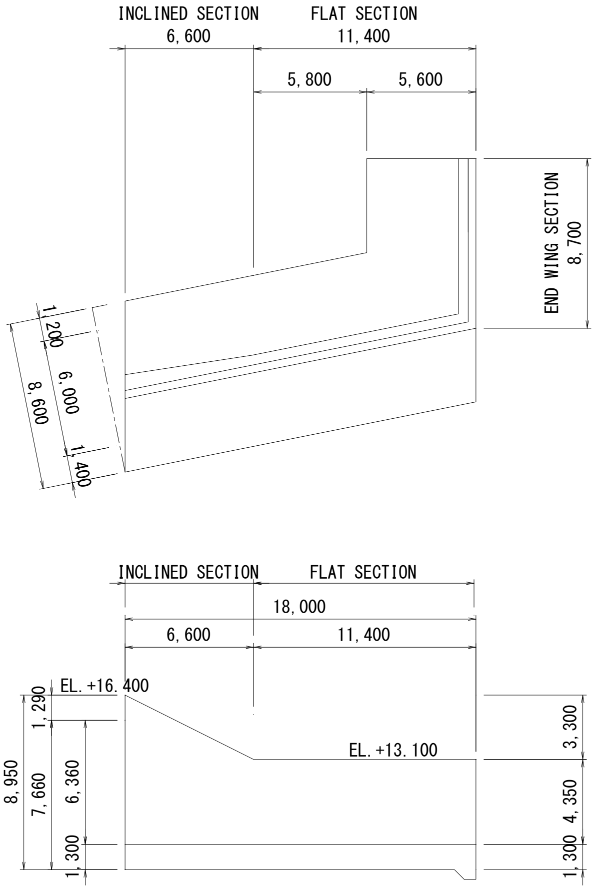
Figure 7.4.39 Pile Foundation Calculation Result

6) Study on Pile Foundation of Downstream Wing Wall

A pile foundation is designed and calculated for the Downstream wing wall. The details of the calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**. Since the downstream side wing wall has a symmetrical structure on the left and right banks, the calculation is performed using the left bank side wing wall as a representative.

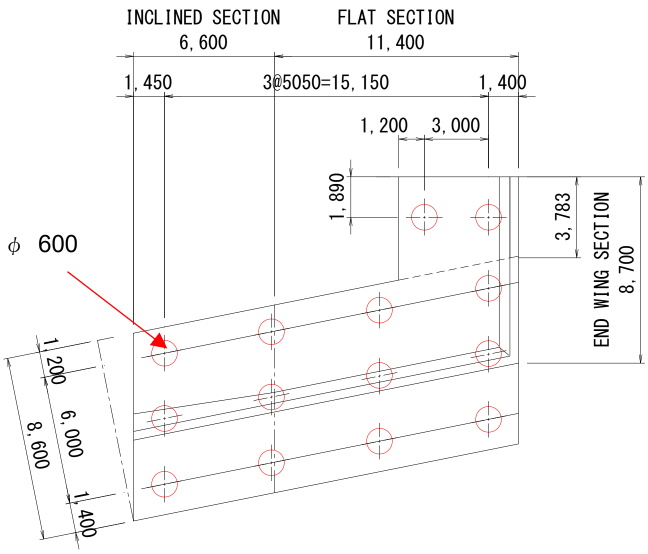
(a) Structural Dimension

The structural dimensions and pile arrangement of the Downstream wing wall are shown. The downstream wing wall is a retaining wall having an L-shaped surface whose wall height decreases in the flow direction.



Source: Study team

Figure 7.4.40 Structural Dimensions of the Downstream Wing Wall



Source: Study team

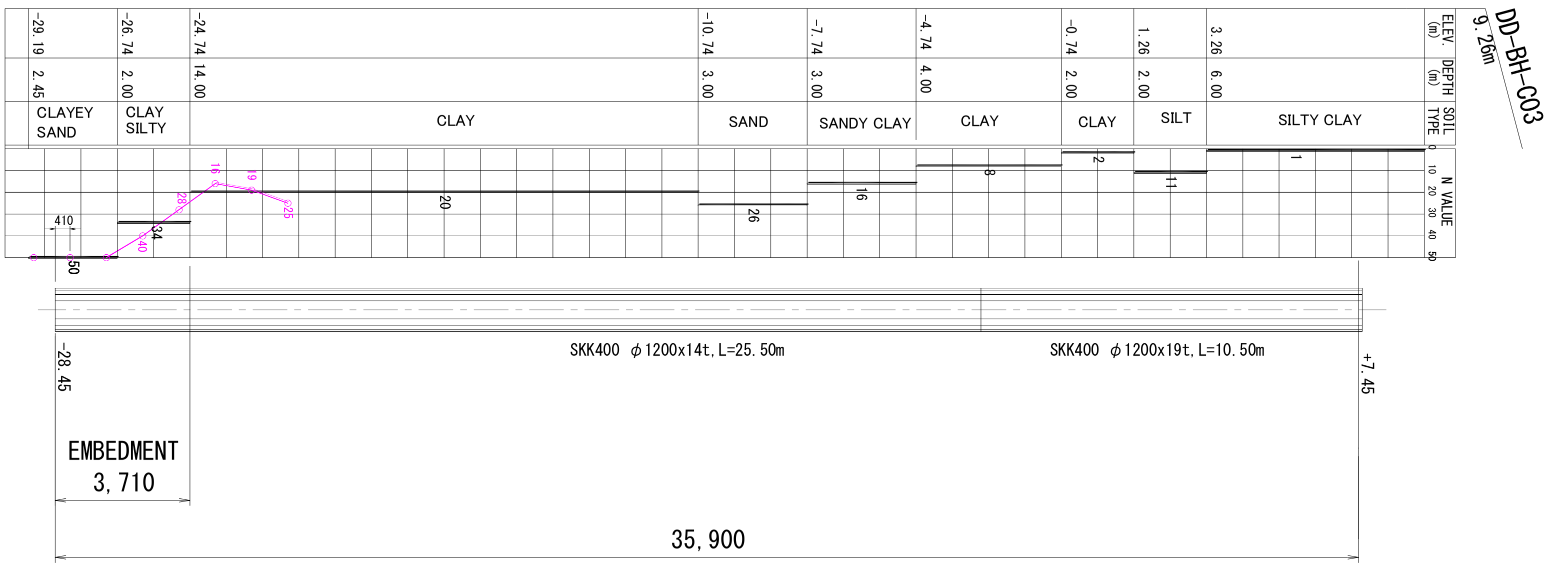
Figure 7.4.41 Downstream Wing Wall Pile Arrangement

(b) Ground Condition

The ground conditions used for the design of pile foundations are shown in the next page. At the Cainta Floodgate site, there are three geological surveys: BH-C01 to BH-C03. The calculation is based on BH-C03, which is the most unfavorable geological survey for pile foundation design.

Table 7.4.39 List of Soil Properties (DD-BH-C03)

Stratum	Soil Type	N-value	Moisture content Wn (%)	Fine fraction content Fc (%)	Plasticity index Ip	Unit Weightγt (kN/m3)	Cohesion C (kN/m2)	Angle of shear resistance φ(°)	Deformation coefficient E 50 (MN/m2)	Consolidation settlement Target layer	Compression index Cc	Swelling index Cc
C1	Cohesive soil	1	67	93	21	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	Cohesive soil	11	43	90	16	17	130	0				
C3	Cohesive soil	2	37	70	17	(16)	(14)	(0)	(1.5)	○	(0.42)	(0.041)
C4	Cohesive soil	8	53	96	65	17	100	0				
C5	Cohesive soil	16	37	60	30	18	200	0				
S1	Sandy soil	26	19	13		20	0	36				
C6	Cohesive soil	20	46	94	45	18	250	0				
C7	Cohesive soil	34	35	74	23	19	420	0				
S2	Sandy soil	50	33	45	21	21	0	40				



Source: Study team

Figure 7.4.42 Pile Foundation Design Ground Condition

(c) Study Case

The calculation is made for the following cases.

Table 7.4.40 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

Calculation Direction	Study Case	Load Condition	Water level Condition		Load Condition							
			Water level in rear side	Water level in front side	Body weight	Water Weight	Earth Pressure	Water pressure	Uplift pressure	Water Weight	Surcharge	Inertial force
Highest Section, Perpendicular Direction To The Flow	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual water level	WL = 13.55 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During Construction	WL = 12.81 Water Level for Cofferdam	WL = 7.45 Lower Side of Bottom Slab	○	○	○	○	○	-	○	-

The residual water level is 2/3 of the difference between HWL = 14.853 and the OWL = 10.94.
Source: Study team

Calculation Direction	Study Case	Load Condition	Water Level Condition		Load Condition							
			Water level in rear side	Water level in front side	Body Weight	Water Weight	Earth Pressure	Water Pressure	Uplift Pressure	Water Weight	Surcharge	Inertial Force
Lowest section, flow direction and perpendicular Direction to Flow	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual Water Level	WL = 12.38 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During construction	WL = 12.81 Water level for cofferdam	WL = 7.45 Lower side of bottom slab	○	○	○	○	○	-	○	-

The residual water level shall be 2/3 of the difference between upper wing wall height = 13.10 and the OWL = 10.94.
Source: Study team

(d) Load Condition

The load conditions acting on the pile foundation are as follows.

Design Horizontal Seismic Coefficient	: kh = 0.20
Back Earth Pressure	: Coulomb Earth Pressure
Soil Type of the Backfill	: backfill γ= 19.0 kN/m3 Internal friction angle ϕ = 30 ° , Cohesion c = 0 kN/m2
Surcharge (Normal Condition, During Construction)	: q0 = 10.0 kN/m2
Surcharge (Seismic Condition)	: q0 = 5.0 kN/m2

(e) Conditions for Consideration

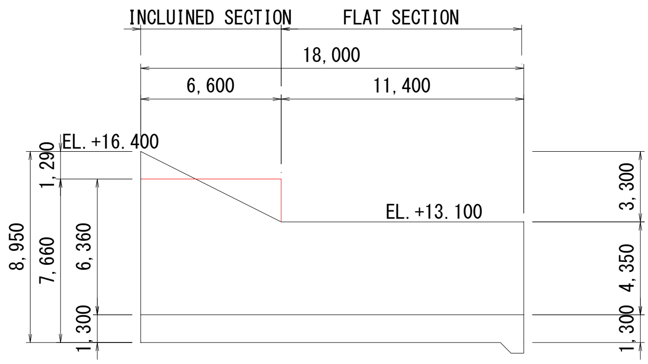
The pile foundation shall be examined under the following conditions. In addition, the thickness of the bottom slab which can be regarded as a rigid body is secured.

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 12 (nos.)
・Pile Diameter		: 1200.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 35.90 (m)

First Section: 10.40 (m) 19.0 (mm) SKK 400]
Second Section: 25.50 (m) 14.0 (mm) SKK 400]

(i) Examination Of The Perpendicular Direction To The Flow

In this retaining wall, the determining factor of pile is "pile head horizontal displacement". In this case, it is excessive to design the piles in the highest section of the wall. Therefore, the pile foundation in the perpendicular direction to the flow is considered not as the end of the wall height but as the total load. (Perform pile calculation in entire L = 18.0 m)

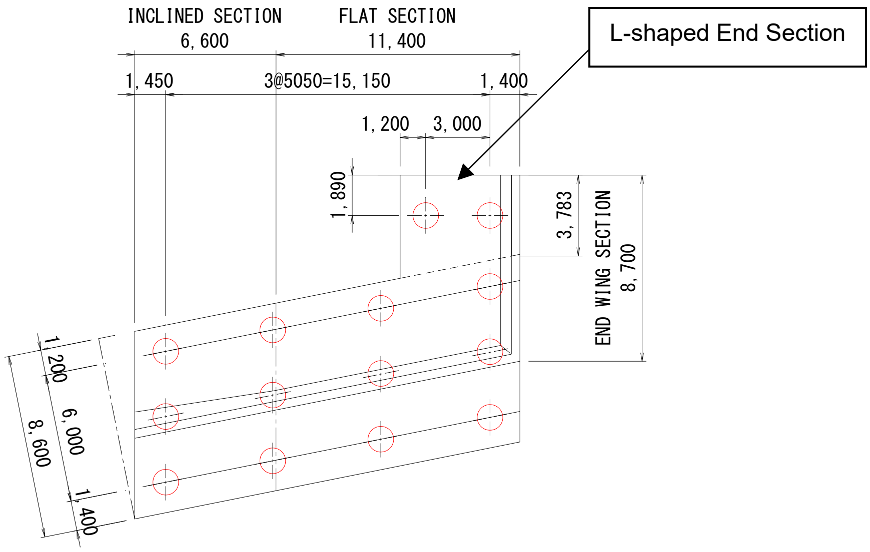


Source: Study team

Figure 7.4.43 Longitudinal Section of the Downstream Wing Wall

(ii) Consideration of Water Flow Direction

The L-shaped retaining wall at the end of the downstream wing wall is planned as a pile foundation that receives earth pressure independently. As shown in the figure below, the earth pressure working width is assumed to be B = 3.783 m. The horizontal displacement of the pile head of the L-shaped retaining wall is excluded from the inspection because the main body pile resists sufficiently.



Source: Study team

Figure 7.4.44 Layout Plan of the Downstream Wing Wall

(iii) Design Values Other Than the Horizontal Displacement of the Pile Head

The horizontal displacement of the pile head resists with the entire width (L = 18.0 m) in the direction perpendicular to the flow direction. For other design values (Upper pile length, pile head reinforcing bar), the most severe value among the values calculated for each part alone is used.

(f) Calculation Result

(i) Pile Head Horizontal Displacement

The horizontal displacement of the pile head is calculated for the entire wing wall. The calculation result of the downstream wing wall pile foundation is shown in **Table 7.4.41**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.41 Downstream side wall pile foundation calculation result (pile head waterside displacement)

Load Case Number / Abbreviation			1 Normal, Zero Water Level	2 Normal, RWL	3 Seismic, Zero Water Level	4 Seismic, LWL
Origin Action Force						
Vo		kN	16022.00	10596.00	15273.20	11143.80
Ho		kN	2175.10	3087.80	6304.20	6113.50
Mo		kNm	-17940.60	-9092.00	-4651.00	-853.00
Origin Displacement						
δx		mm	2.23	6.02	9.61	9.83
δz		mm	7.21	4.82	7.00	5.15
α		rad	-0.00051403	0.00012743	0.00081021	0.00096397
δf, δa		mm	2.23 ≦ 10.00	6.02 ≦ 10.00	9.61 ≦ 10.00	9.83 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra		kN	1618.69 ≦ 7726.00	953.29 ≦ 7726.00	1719.66 ≦ 11590.00	1460.35 ≦ 11590.00
PNmin, Pa		kN	1051.65 ≧ 0.00	812.71 ≧ 0.00	825.88 ≥ -4931.00	396.95 ≥ -4931.00
Horizontal Reaction Force						
PH		kN	181.26	257.32	525.35	509.46
Pile Moment						
Pile Head Mt		kNm	-794.49	-809.94	-1154.09	-1041.62
Underground Mm		kNm	482.44	684.88	1171.26	1135.83
Pile Body Stress						
1 Section	σ c, σ ca	N/mm2	-65.23 ≥ -140.00	-56.05 ≥ -140.00	-86.17 ≥ -186.20	-80.45 ≥ -186.20
	σt, σta	N/mm2	25.21 ≦ 140.00	29.58 ≦ 140.00	48.02 ≦ 186.20	52.62 ≦ 186.20
	τ, τa	N/mm2	2.716 ≦ 80.000	3.856 ≦ 80.000	7.873 ≦ 106.400	7.635 ≦ 106.400
Evaluation			OK	OK	OK	OK

Source: Study team

(ii) Highest Section

The calculation result of the highest section is shown in **Table 7.4.42** to **Table 7.4.45** Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.42 Calculation Results of Pile Foundation of the Highest section of the Downstream Wing Wall (Stability Calculation)

Load state (Water Level)	Amount of displacement (mm)		Pushing force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Normal condition 1 (Zero Water Level)	5.07 ≦	10.0 * 1	1774.011 ≦	8379.228	1363.566 >	0.000	○
Normal condition 2 (residual water level)	9.98 ≦	10.0 * 1	1354.328 ≦	8379.228	907.448 >	0.000	○
Seismic Condition 1 (Zero Water Level)	14.88 >	10.0 * 1	2382.086 ≦	12476.730	610.610 >	-5068.654	○
Seismic Condition 2 (LWL)	15.08 >	10.0 * 1	2157.070 ≦	12476.730	237.474 >	-5068.654	○
During construction (During construction)	10.89 >	0.0 * 1	1506.120 ≦	8379.228	1024.456 >	0.000	○

**1: Horizontal displacement is excluded from the evaluation because it is calculated for the entire wing wall.*
Source: Study team

Table 7.4.43 Calculation Results of Pile Foundation of the Highest section of the Downstream Wing Wall (Pile Body Stress)

Load state (Water Level)	Column Number	Type Of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2) Calculated		Compressive stress (N/mm2) Allowable	Tensile stress (N/mm2) Calculated		Tensile stress (N/mm2) Allowable	Evaluation
Normal Condition 1 (Zero Water Level)	3	P	Compress	1105.329	1774.011	83.581	≦	140.000	-30.409	≦	140.000	○
	1	P	Tensile	1105.329	1363.566	77.430	≦	140.000	-36.560	≦	140.000	○
Normal Condition 2 (Residual Water Level)	1	P	Compress	1172.870	1354.328	80.774	≦	140.000	-40.182	≦	140.000	○
	3	P	Tensile	1172.870	907.448	74.077	≦	140.000	-46.879	≦	140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	1652.021	2382.086	120.884	≦	186.200	-49.486	≦	186.200	○
	3	P	Tensile	1652.021	610.610	94.336	≦	186.200	-76.034	≦	186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	1622.546	2157.070	115.992	≦	186.200	-51.339	≦	186.200	○
	3	P	Tensile	1622.546	237.474	87.224	≦	186.200	-80.106	≦	186.200	○
During construction (During construction)	1	P	Compress	1283.732	1506.120	88.766	≦	210.000	-43.623	≦	210.000	○
	3	P	Tensile	1283.732	1024.456	81.547	≦	210.000	-50.842	≦	210.000	○

Legend: P···Perpendicular Direction to Flow

Source: Study team

Table 7.4.44 Calculation Results of Pile Foundation of the Highest section of the Downstream Wing Wall (Shear Stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2) Calculated τ		Shear Stress (N/mm2) Allowable τa1	Evaluation
Normal Condition 1 (Zero Water Level)	1	Perpendicular to Flow	290.377	4.352	<<	80.000	○
Normal Condition 2 (residual water level)	3	Perpendicular to Flow	397.188	5.952	<<	80.000	○
Seismic Condition 1 (Zero Water Level)	3	Perpendicular to Flow	740.983	11.105	<<	106.400	○
Seismic Condition 2 (LWL)	3	Perpendicular to Flow	727.762	10.906	<<	106.400	○
During construction (during construction)	3	Perpendicular to Flow	433.978	6.504	<<	120.000	○

Source: Study team

・Section Outer Radius R = 80.000 (cm) Inner Radius Ro = 0.000 (cm)

・Rebar D 25 - 34 (@ 98) As = 172.28 (cm2)

Table 7.4.45 Calculation Results of Pile Foundation of the Highest section of the Downstream Wing Wall (Pile Head Reinforcement)

Load state (Water Level)	Review State	Cross-sectional force M (kNm)	Cross-sectional force N (kN)	Neutral axis X (cm)	Compressive stress (N/mm2) σc		Compressive stress (N/mm2) σca	Tensile stress (N/mm2) σs		Tensile stress (N/mm2) σsa	Evaluation
Normal condition 1 (zero water level)	Nmax Nmin	1105.329	1774.011 1363.566	76.37 66.79	4.569 4.836	≦ ≦	8.280 8.280	50.809 71.901	≦ ≦	168.000 168.000	○ ○
Normal condition 2 (residual water level)	Nmax Nmin	1172.870	1354.328 907.448	64.98 57.23	5.193 5.478	≦ ≦	8.280 8.280	81.546 108.776	≦ ≦	168.000 168.000	○ ○
Seismic condition 1 (zero water level)	Nmax Nmin	1652.021	2382.086 610.610	71.96 50.61	7.000 8.082	≦ ≦	11.012 11.012	89.061 197.342	≦ ≦	223.440 223.440	○ ○
Seismic condition 2 (LWL)	Nmax Nmin	1622.546	2157.070 237.474	69.10 47.50	6.995 8.101	≦ ≦	11.012 11.012	97.023 218.749	≦ ≦	223.440 223.440	○ ○
During construction (during construction)	Nmax Nmin	1283.732	1506.120 1024.456	65.39 57.68	5.668 5.977	≦ ≦	12.420 12.420	87.892 117.069	≦ ≦	252.000 252.000	○ ○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As = 172.28 cm2, and D 28 -28 (As = 172.40 cm2) is arranged.

(iii) Lowest Part of Wall Height

The calculation result of the lowest part of the wall height is shown in **Table 7.4.46** to **Table 7.4.49**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.46 Calculation Results of Pile Foundation of the Lowest Part of the Downstream Wing Wall (Stability Calculation)

Load state (Water Level)	Amount of displacement (mm)		Pushing Force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Normal condition 1 (Zero Water Level)	1.53 ≦	10.0 * 1	1649.361 ≦	8379.228	1022.692 >	0.000	○
Normal condition 2 (residual water level)	4.54 ≦	10.0 * 1	877.340 ≦	8379.228	856.239 >	0.000	○
Seismic Condition 1 (Zero Water Level)	8.58 ≦	10.0 * 1	1622.586 ≦	12476.730	923.087 >	-5068.654	○
Seismic Condition 2 (LWL)	8.82 ≦	10.0 * 1	1334.505 ≦	12476.730	446.785 >	-5068.654	○
During construction (During construction)	8.98 ≦	10.0 * 1	1207.394 ≦	8379.228	691.277 >	0.000	○

**1: Horizontal displacement is excluded from the Evaluation because it is calculated for the entire wing wall.*

Source: Study team

Table 7.4.47 Calculation Results of Pile Foundation of the Lowest Part of the Downstream Wing (Pile Body Stress)

Load state (Water Level)	Column Number	Type of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2)		Tensile stress (N/mm2)		Evaluation
						Calculated	Allowable	Calculated	Allowable	
Normal Condition 1 (Zero Water Level)	3	P	Compress	735.055	1649.361	62.620 ≦	140.000	-13.185 ≦	140.000	○
	1	P	Tensile	735.055	1022.692	53.229 ≦	140.000	-22.576 ≦	140.000	○
Normal Condition 2 (Residual Water Level)	3	P	Compress	712.027	877.340	49.863 ≦	140.000	-23.567 ≦	140.000	○
	1	P	Tensile	712.027	856.239	49.547 ≦	140.000	-23.883 ≦	140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	1122.040	1622.586	82.174 ≦	186.200	-33.540 ≦	186.200	○
	3	P	Tensile	1122.040	923.087	71.691 ≦	186.200	-44.023 ≦	186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	1047.016	1334.505	73.988 ≦	186.200	-33.989 ≦	186.200	○
	3	P	Tensile	1047.016	446.785	60.684 ≦	186.200	-47.293 ≦	186.200	○
During construction (During construction)	1	P	Compress	963.786	1207.394	67.791 ≦	210.000	-31.602 ≦	210.000	○
	3	P	Tensile	963.786	691.277	60.056 ≦	210.000	-39.337 ≦	210.000	○

Legend: P…Perpendicular Direction to Flow
Source: Study team

Table 7.4.48 Calculation Results of Pile Foundation of the Lowest Part of the Downstream Wing (Shear Stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2)		Evaluation
				Calculated τ	Allowable τa1	
Normal Condition 1 (Zero Water Level)	1	Perpendicular to Flow	157.400	2.359 <<	80.000	○
Normal Condition 2 (residual water level)	1	Perpendicular to Flow	211.612	3.171 <<	80.000	○
Seismic Condition 1 (Zero Water Level)	3	Perpendicular to Flow	487.452	7.305 <<	106.400	○

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2)		Evaluation
				Calculated τ	Allowable τa1	
Seismic Condition 2 (LWL)	3	Perpendicular to Flow	469.619	7.038 <<	106.400	○
During construction (during construction)	3	Perpendicular to Flow	341.412	5.117 <<	120.000	○

Source: Study team

・Section Outer radius R = 80.000 (cm) Inner radius Ro = 0.000 (cm)

・Rebar D 25 - 34 (@ 98) As = 172.28 (cm2)

Table 7.4.49 Calculation Results of Pile Foundation of the Lowest Part of the Downstream Wing (Pile Head Reinforcement)

Load state (Water Level)	Review State	Cross-sectional force		Neutral axis	Compressive stress (N/mm2)		Tensile stress (N/mm2)		Evaluation
		M (kNm)	N (kN)	X (cm)	σ c	σ ca	σ s	σ sa	
Normal condition 1 (zero water level)	Nmax Nmin	735.055	1649.361 1022.692	95.81 70.65	2.816 ≦ 3.139 ≦	8.280 8.280	16.395 ≦ 41.545 ≦	168.000 168.000	○ ○
Normal condition 2 (residual water level)	Nmax Nmin	712.027	877.340 856.239	66.76 66.07	3.116 ≦ 3.130 ≦	8.280 8.280	46.376 ≦ 47.562 ≦	168.000 168.000	○ ○
Seismic condition 1 (zero water level)	Nmax Nmin	1122.040	1622.586 923.087	72.07 58.14	4.751 ≦ 5.207 ≦	11.012 11.012	60.252 ≦ 100.571 ≦	223.440 223.440	○ ○
Seismic condition 2 (LWL)	Nmax Nmin	1047.016	1334.505 446.785	67.76 51.46	4.552 ≦ 5.092 ≦	11.012 11.012	65.731 ≦ 121.016 ≦	223.440 223.440	○ ○
During construction (during construction)	Nmax Nmin	963.786	1207.394 691.277	67.24 56.22	4.204 ≦ 4.534 ≦	12.420 12.420	61.667 ≦ 92.878 ≦	252.000 252.000	○ ○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As = 172.28 cm2, and D 28 -28 (As = 172.40 cm2) is arranged.

(iv) L-shaped Tip

The calculation result of the L-shaped tip is shown in **Table 7.4.50** to **Table 7.4.53**.

Table 7.4.50 Calculation Result of Pile Foundation of L-Type Section at the End of the Downstream Wing Wall (Stability Calculation)

Load state (Water Level)	Amount of displacement (mm)		Pushing Force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Normal condition 1 (Zero Water Level)	7.43 ≦	10.0 * 1	1737.050 ≦	8374.225	1140.131 >	-2602.935	○
Normal condition 2 (residual water level)	10.74 >	10.0 * 1	1455.266 ≦	8374.225	531.985 >	-2602.935	×
Seismic Condition 1 (Zero Water Level)	16.78 >	10.0 * 1	2264.792 ≦	12473.081	457.840 >	-5068.244	×
Seismic Condition 2 (LWL)	16.15 >	10.0 * 1	1837.641 ≦	12473.081	99.491 >	-5068.244	×
During construction (During construction)	17.93 >	10.0 * 1	1945.774 ≦	8374.225	364.221 >	-2602.935	×

**1: Horizontal displacement is excluded from the Evaluation because it is calculated for the entire wing wall.*

Source: Study team

Table 7.4.51 Calculation Result of Pile Foundation of L-Type Section at the End of the Downstream Wing Wall (Pile Body Stress)

Load state (Water Level)	Column Number	Type of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2) Calculated	Compressive stress (N/mm2) Allowable	Tensile stress (N/mm2) Calculated	Tensile stress (N/mm2) Allowable	Evaluation
Normal Condition 1 (Zero Water Level)	1	P	Compress	468.209	1737.050	50.175	≦ 140.000	1.889	≦ 140.000	○
	2	P	Tensile	468.209	1140.131	41.229	≦ 140.000	-7.056	≦ 140.000	○
Normal Condition 2 (Residual Water Level)	1	P	Compress	632.437	1455.266	54.420	≦ 140.000	-10.802	≦ 140.000	○
	2	P	Tensile	632.437	531.985	40.584	≦ 140.000	-24.639	≦ 140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	1187.313	2264.793	95.164	≦ 186.200	-27.282	≦ 186.200	○
	2	P	Tensile	1187.313	457.840	68.084	≦ 186.200	-54.361	≦ 186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	1142.933	1837.641	86.474	≦ 186.200	-31.395	≦ 186.200	○
	2	P	Tensile	1142.933	99.491	60.425	≦ 186.200	-57.443	≦ 186.200	○
During construction (During construction)	1	P	Compress	1026.366	1945.774	82.084	≦ 210.000	-23.764	≦ 210.000	○
	2	P	Tensile	1026.366	364.221	58.382	≦ 210.000	-47.465	≦ 210.000	○

Source: Study team

Table 7.4.52 Calculation Result of Pile Foundation of L-Type Section at the End of the Downstream Wing Wall (Shear Stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2) Calculated τ	Shear Stress (N/mm2) Allowable τa1	Evaluation
Normal Condition 1 (Zero Water Level)	2	Perpendicular to Flow	175.918	2.636	<< 80.000	○
Normal Condition 2 (residual water level)	2	Perpendicular to Flow	237.623	3.561	<< 80.000	○
Seismic Condition 1 (Zero Water Level)	2	Perpendicular to Flow	532.547	7.981	<< 106.400	○
Seismic Condition 2 (LWL)	2	Perpendicular to Flow	512.641	7.683	<< 106.400	○
During construction (during construction)	2	Perpendicular to Flow	385.632	5.779	<< 120.000	○

Source: Study team

・Section Outer radius R = 80.000 (cm) Inner radius Ro = 0.000 (cm)

・Rebar D 25 - 34 (@ 98) As = 172.28 (cm2)

Table 7.4.53 Calculation Result of Pile Foundation of L-Type Section at the End of the Downstream Wing Wall (Pile Head Reinforcement)

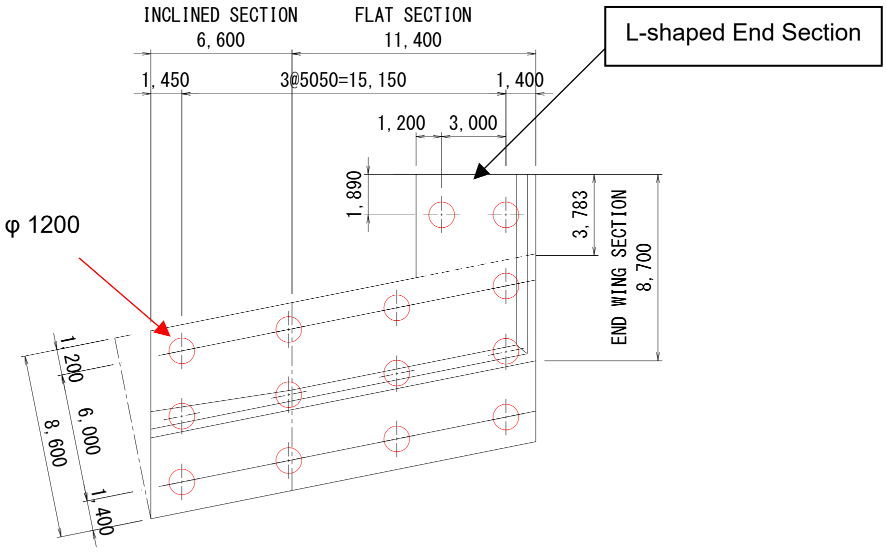
Load state (Water Level)	Review State	Cross-sectional force M (kNm)	Cross-sectional force N (kN)	Neutral axis X (cm)	Compressive stress (N/mm2) σ c	Compressive stress (N/mm2) σ ca	Tensile stress (N/mm2) σ s	Tensile stress (N/mm2) σ sa	Evaluation
Normal condition 1 (zero water level)	Nmax	468.209	1737.050	135.46	1.841	≦ 8.280	-0.502	≦ 168.000	○
	Nmin		1140.131	101.80	1.773	≦ 8.280	8.151	≦ 168.000	○
Normal condition 2 (residual water level)	Nmax	632.437	1455.266	97.62	2.413	≦ 8.280	13.122	≦ 168.000	○
	Nmin		531.985	58.49	2.928	≦ 8.280	55.948	≦ 168.000	○
Seismic condition 1 (zero water level)	Nmax	1187.313	2264.792	85.29	4.707	≦ 11.012	39.491	≦ 223.440	○
	Nmin		457.840	50.85	5.799	≦ 11.012	140.544	≦ 223.440	○
Seismic condition 2 (LWL)	Nmax	1142.933	1837.641	76.45	4.722	≦ 11.012	52.386	≦ 223.440	○
	Nmin		99.491	46.73	5.734	≦ 11.012	158.819	≦ 223.440	○
During construction (during construction)	Nmax	1026.366	1945.774	84.93	4.074	≦ 12.420	34.591	≦ 252.000	○
	Nmin		364.221	50.39	5.028	≦ 12.420	123.635	≦ 252.000	○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As = 172.28 cm2, and D 28 -2 (As = 172.40 cm2) is arranged.

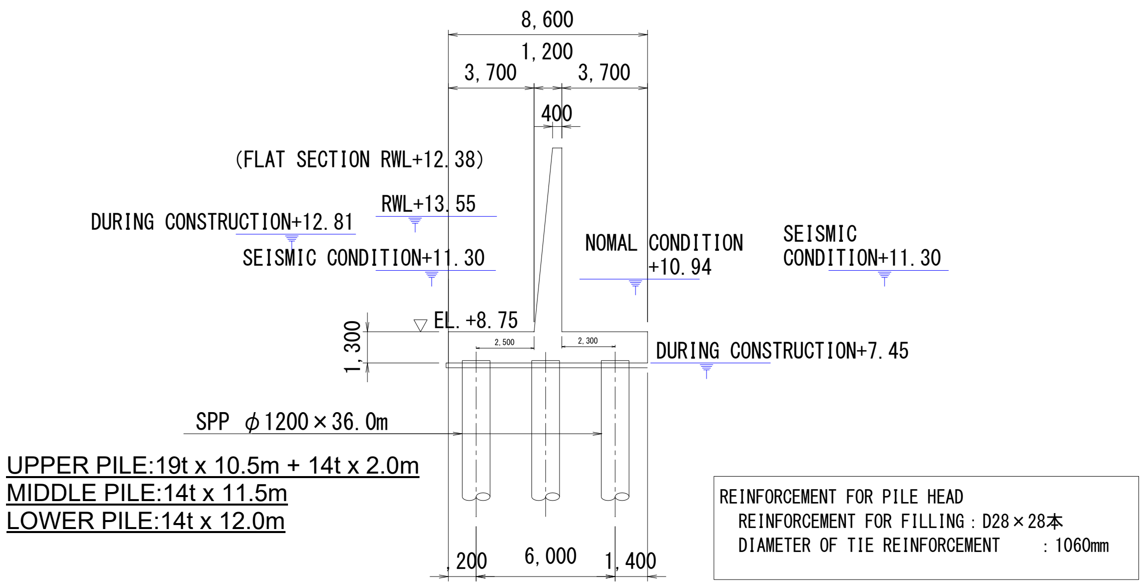
(g) Result of Examination

Study results of the pile foundation of the Downstream wing wall are shown in **Figure 7.4.45** and **Figure 7.4.46**. Details of the examination results are shown in Vol.5A Structural Calculation for Contract Package-1.



Source: Study team

Figure 7.4.45 Downstream Wing Wall Pile Arrangement



Source: Study team

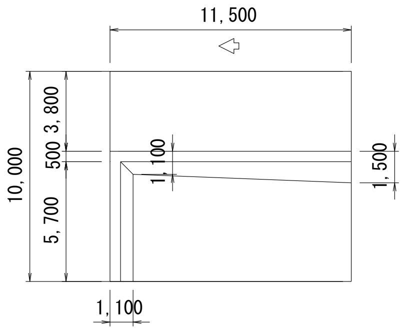
Figure 7.4.46 Pile Foundation Calculation Result

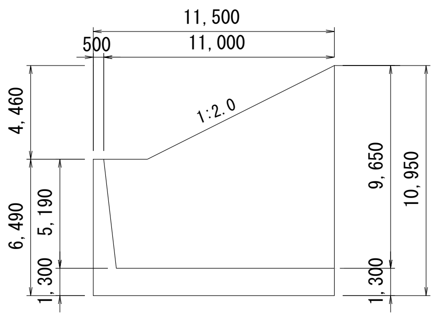
7) Study on the Pile Foundation of the Upstream left bank wing wall

A pile foundation design calculation is performed for the upstream left bank wing wall. The details of the calculation are shown in Vol.5A Structural Calculation for Contract Package-1.

(a) Structural Dimension

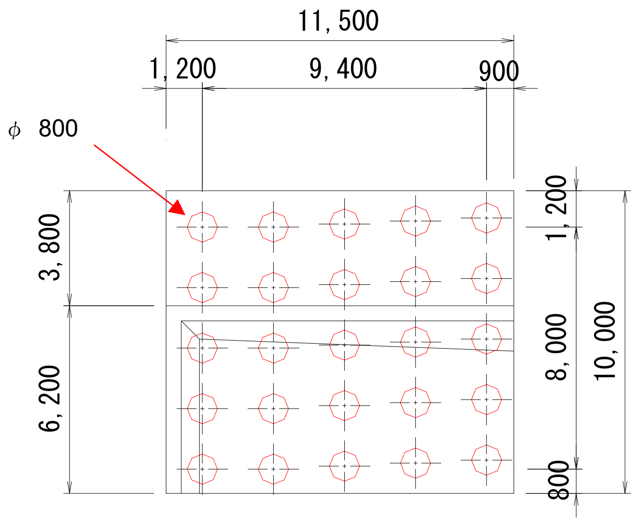
The structural dimension drawing and pile arrangement of the upstream left bank wing wall are shown. In below. The upstream left bank wing wall is a flat L-shaped retaining wall whose wall height decreases in the flow direction.





Source: Study team

Figure7.4.47 Upstream Left Bank Wing Structural Dimensions



Source: Study team

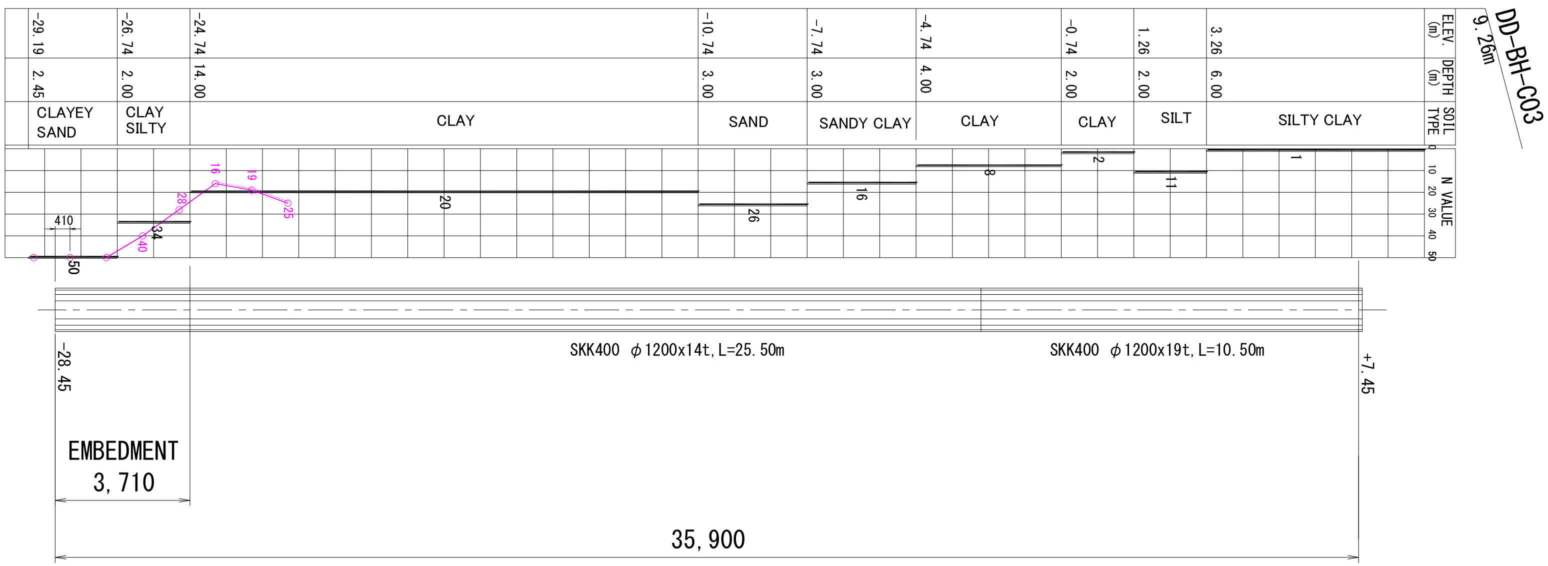
Figure7.4.48 Upstream Left Bank Wing Wall Pile Arrangement

(b) Ground Condition

The ground conditions used for the design of pile foundations are shown in the next page. At the Cainta Floodgate site, there are three geological surveys: BH-C01 to BH-C03. The calculation is based on BH-C03, which is the most unfavorable geological survey for pile foundation design.

Table 7.4.54 List of Soil Properties (DD-BH-C03)

Stratum	Soil Type	N-value	Moisture content Wn (%)	Fine fraction content Fc (%)	Plasticity index Ip	Unit Weightγt (kN/m3)	Cohesion C (kN/m2)	Angle of shear resistance φ(°)	Deformation coefficient E 50 (MN/m2)	Consolidation settlement Target layer	Compression index Cc	Swelling index Cc
C1	Cohesive soil	1	67	93	21	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	Cohesive soil	11	43	90	16	17	130	0				
C3	Cohesive soil	2	37	70	17	(16)	(14)	(0)	(1.5)	○	(0.42)	(0.041)
C4	Cohesive soil	8	53	96	65	17	100	0				
C5	Cohesive soil	16	37	60	30	18	200	0				
S1	Sandy soil	26	19	13		20	0	36				
C6	Cohesive soil	20	46	94	45	18	250	0				
C7	Cohesive soil	34	35	74	23	19	420	0				
S2	Sandy soil	50	33	45	21	21	0	40				



Source: Study team

Figure 7.4.49 Pile Foundation Design Ground Condition

(c) Study Case

The calculation is made for the following cases.

Table 7.4.55 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

Calculation Direction	Study Case	Load Condition	Water level Condition		Load Condition							
			Water level in rear side	Water level in front side	Body weight	Water Weight	Earth Pressure	Water pressure	Uplift pressure	Water Weight	Surcharge	Inertial force
Perpendicular Direction To The Flow Flow Direction	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual water level	WL = 12.54 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During Construction	WL = 12.81 Water Level for Cofferdam	WL = 7.45 Lower Side of Bottom Slab	○	○	○	○	○	-	○	-

The residual water level shall be 2/3 of the difference between DFL = 13.340 and the OWL = 10.94.
Source: Study team

(d) Load Condition

The load conditions acting on the pile foundation are as follows.

Design Horizontal Seismic Coefficient	: Kh = 0.20
Back Earth Pressure	: Coulomb earth pressure
Soil Type of the Backfill	: Backfill γ = 19.0 kN/m3
	Internal friction angle ϕ = 30 ° , Cohesion c = 0 kN/m2
Surcharge (Normal Condition, During Construction)	: q0 = 10.0 kN/m2
Surcharge (Seismic Condition)	: q0 = 5.0 kN/m2

(e) Conditions for consideration

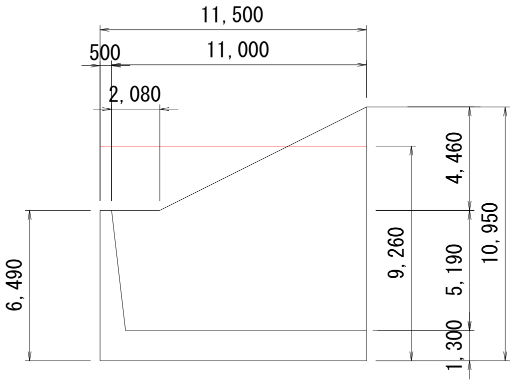
The pile foundation shall be examined under the following conditions. In addition, the thickness of the bottom slab which can be regarded as a rigid body is secured.

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 25 (nos.)
・Pile Diameter		: 800.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 35.90 (m)

First Section: 7.90 (m) 12.0 (mm) SKK 400]
Second Section: 28.00 (m) 9.0 (mm) SKK 400]

(i) Examination of the Perpendicular Direction to the Flow

In this retaining wall, the determining factor of pile is "pile head horizontal displacement". In this case, it is excessive to design the piles at the highest section of the wall. Therefore, the pile foundation in the Perpendicular Direction to the Flow is considered not as the highest section of the wall but as the total load. (Perform pile calculation in entire L = 11.5 m)

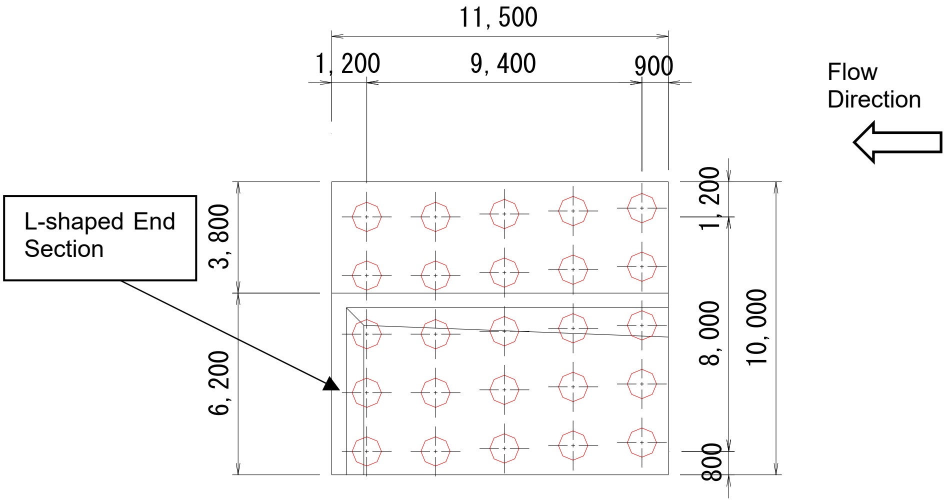


Source: Study team

Figure 7.4.50 Longitudinal Section of the Upstream Left Bank Wing Wall

(ii) Consideration of Water Flow Direction

The L-shaped retaining wall at the end of the upstream left bank wing wall is designed as a pile foundation that receives earth pressure independently. As shown in the figure below, the earth pressure working width is assumed to be B = 6.2 m. The horizontal displacement of the pile head of the L-shaped retaining wall is excluded from the inspection because the main body pile resists sufficiently.



Source: Study team

Figure 7.4.51 Layout Plan of the Upstream Left Bank Wing Wall

(iii) Design Values Other Than The Horizontal Displacement Of The Pile Head

The horizontal displacement of the pile head resists with the entire width (L = 11.5 m) in the direction perpendicular to the flow direction. For other design values (Upper pile length, pile head reinforcing bar), the most severe value among the values calculated for each part alone is used.

(f) Calculation Result

(i) Perpendicular Direction to the Flow (Consideration of the entire L = 11.5 m)

Calculation results in the perpendicular direction to the flow (L = entire 11.5 m) are shown in **Table 7.4.56** to **Table 7.4.59**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.56 Calculation Results of Pile Foundation of the Upstream Left Bank Wing Wall in Perpendicular Direction to the Flow (Stability Calculation)

Load State (Water Level)	Amount of displacement (mm)		Pushing force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Normal Condition 1 (Zero Water Level)	3.77 ≦	10.0	876.216 ≦	5238.819	483.272 >	-1701.392	○
Normal Condition 2 (residual water level)	5.14 ≦	10.0	609.282 ≦	5238.819	440.564 >	-1701.392	○
Seismic Condition 1 (Zero Water Level)	9.80 ≦	10.0	966.940 ≦	7815.542	334.978 >	-3344.931	○
Seismic Condition 2 (LWL)	9.86 ≦	10.0	881.611 ≦	7815.542	169.388 >	-3344.931	○
During construction (during construction)	6.95 ≦	10.0	573.662 ≦	5238.819	545.925 >	-1701.392	○

Source: Study team

Table 7.4.57 Calculation Results of Pile Foundation of the Upstream Left Bank Wing Wall in Perpendicular Direction to the Flow (Pile Body Stress)

Load State (Water Level)	Column Number	Type Of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2)		Tensile stress (N/mm2)		Evaluation
						Compressive stress (N/mm2)		Tensile stress (N/mm2)		
Normal Condition 1 (Zero Water Level)	5	P	Compress	296.731	876.216	88.435 ≦	140.000	-24.000 ≦	140.000	○
	1	P	Tensile	296.731	483.272	73.987 ≦	140.000	-38.448 ≦	140.000	○
Normal Condition 2 (residual water level)	5	P	Compress	314.093	609.282	81.910 ≦	140.000	-37.104 ≦	140.000	○
	1	P	Tensile	314.093	440.564	75.706 ≦	140.000	-43.308 ≦	140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	507.146	966.939	131.636 ≦	186.200	-60.529 ≦	186.200	○
	5	P	Tensile	507.146	334.978	108.399 ≦	186.200	-83.766 ≦	186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	488.323	881.611	124.932 ≦	186.200	-60.100 ≦	186.200	○
	5	P	Tensile	488.323	169.388	98.744 ≦	186.200	-86.288 ≦	186.200	○
During construction (during construction)	5	P	Compress	375.689	573.662	92.270 ≦	210.000	-50.084 ≦	210.000	○
	1	P	Tensile	375.689	545.925	91.250 ≦	210.000	-51.104 ≦	210.000	○

Legend: P···Perpendicular Direction to Flow
Source: Study team

Table 7.4.58 Calculation Results of Pile Foundation of the Upstream Left Bank Wing Wall in Perpendicular Direction to the Flow (Shear Stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2)		Evaluation
				Calculated τ	Allowable τa1	
Normal Condition 1 (Zero Water Level)	1	Perpendicular to Flow	107.399	3.949 <<	80.000	○

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2)		Evaluation
				Calculated τ	Allowable τa1	
Normal Condition 2 (residual water level)	1	Perpendicular to Flow	124.847	4.591 <<	80.000	○
Seismic Condition 1 (Zero Water Level)	5	Perpendicular to Flow	293.837	10.804 <<	106.400	○
Seismic Condition 2 (LWL)	5	Perpendicular to Flow	289.559	10.647 <<	106.400	○
During construction (during construction)	1	Perpendicular to Flow	157.191	5.780 <<	120.000	○

Source: Study team

・Section Outer radius R = 55.000 (cm) Inner radius Ro = 0.000 (cm)

・Rebar D 29 - 15 (@ 142) As = 96.36 (cm2)

Table 7.4.59 Calculation Results of Pile Foundation of the Upstream Left Bank Wing Wall in Perpendicular Direction to the Flow (Pile Head Reinforcement)

Load state (Water Level)	Review State	Cross-sectional force		Neutral axis	Compressive stress (N/mm2)		Tensile stress (N/mm2)		Evaluation
		M (kNm)	N (kN)	X (cm)	σ c	σca	σ s	σ sa	
Normal Condition 1 (Zero Water Level)	Nmax Nmin	296.731	876.216 483.272	61.07 44.98	3.555 ≦ 4.001 ≦	8.280 8.280	24.390 ≦ 58.734 ≦	168.000 168.000	○ ○
Normal Condition 2 (residual water level)	Nmax Nmin	314.093	609.282 440.564	48.22 42.84	4.113 ≦ 4.323 ≦	8.280 8.280	52.166 ≦ 69.862 ≦	168.000 168.000	○ ○
Seismic Condition 1 (Zero Water Level)	Nmax Nmin	507.146	966.940 334.978	47.86 36.93	6.662 ≦ 7.400 ≦	11.012 11.012	85.895 ≦ 156.531 ≦	223.440 223.440	○ ○
Seismic Condition 2 (LWL)	Nmax Nmin	488.323	881.611 169.388	46.78 34.85	6.476 ≦ 7.270 ≦	11.012 11.012	87.681 ≦ 169.452 ≦	223.440 223.440	○ ○
During construction (during construction)	Nmax Nmin	375.689	573.662 545.925	44.00 43.30	5.113 ≦ 5.147 ≦	12.420 12.420	78.454 ≦ 81.474 ≦	252.000 252.000	○ ○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As 96.36 = 28 cm2, and -16 D reinforcing rods (As = 98.53 cm2) are arranged.

(ii) Invert T Section (Highest section)

Calculation results of the invert T section are shown in **Table 7.4.60** to **Table 7.4.63**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.60 Calculation Results of Pile Foundation of Invert T Section of the Upstream Left Bank Wing Wall (Stability Calculation)

Load state (Water Level)	Amount of displacement (mm)		Pushing Force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Normal Condition 1 (Zero Water Level)	4.70 ≦	10.0 * 1	805.897 ≦	5238.819	516.155 >	-1701.392	○
Normal Condition 2 (residual water level)	5.83 ≦	10.0 * 1	585.153 ≦	5238.819	480.591 >	-1701.392	○
Seismic Condition 1 (Zero Water Level)	10.91 >	10.0 * 1	1069.404 ≦	7815.542	196.141 >	-3344.931	○
Seismic Condition 2 (LWL)	10.96 >	10.0 * 1	998.814 ≦	7815.542	59.176 >	-3344.931	○
During construction (during construction)	7.32 ≦	10.0 * 1	567.616 ≦	5238.819	555.705 >	-1701.392	○

**1: Horizontal displacement is excluded from the evaluation.*
Source: Study team

Table 7.4.61 Calculation Results of Pile Foundation of Invert T Section of the Upstream Left Bank Wing Wall (Pile Body Stress)

Load state (Water Level)	Column Number	Type of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2) Calculated		Compressive stress (N/mm2) Allowable	Tensile stress (N/mm2) Calculated		Tensile stress (N/mm2) Allowable	Evaluation
Normal Condition 1 (Zero Water Level)	5	P	Compress	320.552	805.897	90.363	≦	140.000	-31.099	≦	140.000	○
	1	P	Tensile	320.552	516.155	79.709	≦	140.000	-41.752	≦	140.000	○
Normal Condition 2 (residual water level)	5	P	Compress	334.900	585.153	84.965	≦	140.000	-41.934	≦	140.000	○
	1	P	Tensile	334.900	480.591	81.120	≦	140.000	-45.778	≦	140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	516.670	1069.404	137.208	≦	186.200	-58.566	≦	186.200	○
	5	P	Tensile	516.670	196.142	105.099	≦	186.200	-90.675	≦	186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	501.308	998.814	131.702	≦	186.200	-58.251	≦	186.200	○
	5	P	Tensile	501.308	59.176	97.152	≦	186.200	-92.801	≦	186.200	○
During construction (during construction)	1	P	Compress	385.783	567.616	93.960	≦	210.000	-52.219	≦	210.000	○
	5	P	Tensile	385.783	555.705	93.522	≦	210.000	-52.657	≦	210.000	○

Source: Study team

Table 7.4.62 Calculation Results of Pile Foundation of Invert T Section of the Upstream Left Bank Wing Wall (shear stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2) Calculated τ		Shear Stress (N/mm2) Allowable τa1	Evaluation
Normal Condition 1 (Zero Water Level)	1	Perpendicular to Flow	122.106	4.490	<<	80.000	○
Normal Condition 2 (residual water level)	1	Perpendicular to Flow	136.519	5.020	<<	80.000	○
Seismic Condition 1 (Zero Water Level)	5	Perpendicular to Flow	314.018	11.546	<<	106.400	○
Seismic Condition 2 (LWL)	5	Perpendicular to Flow	310.583	11.420	<<	106.400	○
During construction (during construction)	5	Perpendicular to Flow	163.238	6.002	<<	120.000	○

Source: Study team

・Section Outer radius R = 55.000 (cm) Inner radius Ro = 0.000 (cm)

・Rebar D 29 - 15 (@ 142) As = 96.36 (cm2)

Table 7.4.63 Calculation Results of Pile Foundation of Invert T Section of the Upstream Left Bank Wing Wall (Pile Head Reinforcement)

Load state (Water Level)	Review State	Cross-sectional force M (kNm)	Cross-sectional force N (kN)	Neutral axis X (cm)	Compressive stress (N/mm2) σ c		Compressive stress (N/mm2) σ ca	Tensile stress (N/mm2) σ s		Tensile stress (N/mm2) σ sa	Evaluation
Normal Condition 1 (Zero Water Level)	Nmax	320.552	805.897	55.13	3.977	≦	8.280	36.662	≦	168.000	○
	Nmin		516.155	44.80	4.330	≦	8.280	64.077	≦	168.000	○
Normal Condition 2 (residual water level)	Nmax	334.900	585.153	46.17	4.466	≦	8.280	62.131	≦	168.000	○
	Nmin		480.591	43.14	4.596	≦	8.280	73.295	≦	168.000	○

Load state (Water Level)	Review State	Cross-sectional force		Neutral axis	Compressive stress (N/mm2)			Tensile stress (N/mm2)			Evaluation
		M (kNm)	N (kN)	X (cm)	σ c		σ ca	σ s		σ sa	
Seismic Condition 1 (Zero Water Level)	Nmax Nmin	516.670	1069.404 196.141	49.68 35.06	6.682 7.677	≦ ≦	11.012 11.012	79.314 177.202	≦ ≦	223.440 223.440	○ ○
Seismic Condition 2 (LWL)	Nmax Nmin	501.308	998.814 59.176	48.81 33.46	6.531 7.560	≦ ≦	11.012 11.012	80.680 188.240	≦ ≦	223.440 223.440	○ ○
During construction (during construction)	Nmax Nmin	385.783	567.616 555.705	43.47 43.19	5.277 5.291	≦ ≦	12.420 12.420	82.896 84.200	≦ ≦	252.000 252.000	○ ○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As 96.36 = 28 cm2, and -16 D reinforcing rods (As = 98.53 cm2) are arranged.

(iii) L-shaped section

The calculation results of the L-shaped section are shown in **Table 7.4.64** to **Table 7.4.67**.

Table 7.4.64 Calculation Result of Pile Foundation of L-shaped Section of Upstream Left Bank Wing Wall (Stability Calculation)

Load State (Water Level)	Amount of displacement (mm)			Pushing Force (kN)			Pull-put Force (kN)			Evaluation
	Calculated		Allowable	Calculated		Allowable	Calculated		Allowable	
Normal condition 1 (Zero Water Level)	3.66	≦	10.0 * 1	564.879	≦	5238.819	519.880	>	-1701.392	○
Normal condition 2 (residual water level)	4.22	≦	10.0 * 1	481.361	≦	5238.819	375.153	>	-1701.392	○
Seismic Condition 1 (Zero Water Level)	7.89	≦	10.0 * 1	896.073	≦	7815.542	138.671	>	-3344.931	○
Seismic Condition 2 (LWL)	7.78	≦	10.0 * 1	783.401	≦	7815.542	47.473	>	-3344.931	○
During construction (During construction)	5.30	≦	10.0 * 1	595.537	≦	5238.819	351.513	>	-1701.392	○

**1: Horizontal displacement is excluded from the evaluation.*
Source: Study team

Table 7.4.65 Calculation Result of Pile Foundation of L-shaped Section of Upstream Left Bank Wing (Pile Body Stress)

Load state (Water Level)	Column Number	Type of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2)			Tensile stress (N/mm2)			Evaluation
						Compressive stress (N/mm2)			Tensile stress (N/mm2)			
Normal Condition 1 (Zero Water Level)	1	P	Compress	184.833	564.879	55.788	≦	140.000	-14.248	≦	140.000	○
	5	P	Tensile	184.833	519.880	54.133	≦	140.000	-15.902	≦	140.000	○
Normal Condition 2 (Residual Water Level)	1	P	Compress	201.562	481.361	55.886	≦	140.000	-20.488	≦	140.000	○
	5	P	Tensile	201.562	375.153	51.981	≦	140.000	-24.393	≦	140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	369.412	896.073	102.935	≦	186.200	-37.040	≦	186.200	○
	5	P	Tensile	369.412	138.671	75.087	≦	186.200	-64.889	≦	186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	366.957	783.401	98.328	≦	186.200	-40.718	≦	186.200	○
	5	P	Tensile	366.957	47.473	71.268	≦	186.200	-67.777	≦	186.200	○
During construction (During construction)	1	P	Compress	230.350	595.537	65.539	≦	210.000	-21.744	≦	210.000	○
	5	P	Tensile	230.350	351.513	56.566	≦	210.000	-30.717	≦	210.000	○

Legend: P···Perpendicular Direction to Flow
Source: Study team

Table 7.4.66 Calculation Result of Pile Foundation of L-shaped Section of Upstream Left Bank Wing (Shear Stress)

Load state (Water Level)	Column Number	Type of Row	Shear Force (kN)	Shear Stress (N/mm2) Calculated τ		Allowable τa1	Evaluation
Normal Condition 1 (Zero Water Level)	5	Perpendicular to Flow	79.681	2.930	<<	80.000	○
Normal Condition 2 (residual water level)	5	Perpendicular to Flow	89.088	3.276	<<	80.000	○
Seismic Condition 1 (Zero Water Level)	5	Perpendicular to Flow	225.811	8.303	<<	106.400	○
Seismic Condition 2 (LWL)	5	Perpendicular to Flow	223.416	8.215	<<	106.400	○
During construction (during construction)	5	Perpendicular to Flow	106.526	3.917	<<	120.000	○

Source: Study team

・Section outer radius R = 55.000 (cm) inner radius Ro = 0.000 (cm)

・Rebar D 29 - 15 (@ 142) As = 96.36 (cm2)

Table 7.4.67 Calculation Result of Pile Foundation of L-shaped Section of Upstream Left Bank Wing (Pile Head Reinforcement)

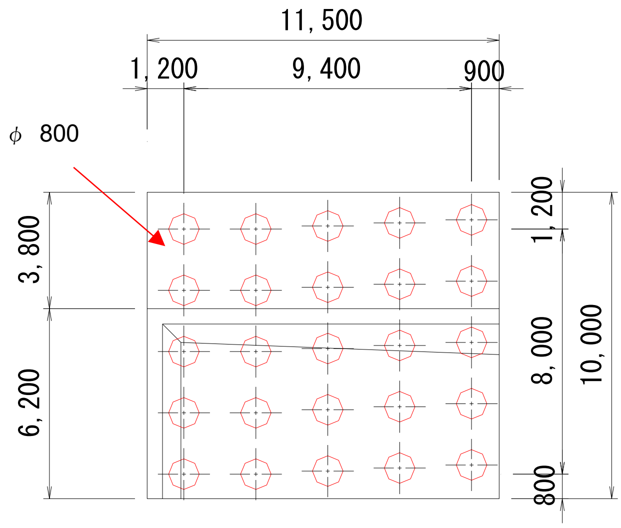
Load state (Water Level)	Review State	Cross-sectional force M (kNm)	Cross-sectional force N (kN)	Neutral axis X (cm)	Compressive stress (N/mm2) σ c		σ ca	Tensile stress (N/mm2) σ s		σ sa	Evaluation
Normal Condition 1 (Zero Water Level)	Nmax	184.833	564.879	62.52	2.199	≦	8.280	13.974	≦	168.000	○
	Nmin		519.880	59.12	2.237	≦	8.280	16.958	≦	168.000	○
Normal Condition 2 (Residual Water Level)	Nmax	201.562	481.361	53.52	2.530	≦	8.280	25.159	≦	168.000	○
	Nmin		375.153	47.37	2.659	≦	8.280	35.052	≦	168.000	○
Seismic Condition 1 (Zero Water Level)	Nmax	369.412	896.073	53.99	4.620	≦	11.012	44.941	≦	223.440	○
	Nmin		138.671	35.03	5.490	≦	11.012	126.887	≦	223.440	○
Seismic Condition 2 (LWL)	Nmax	366.957	783.401	50.44	4.716	≦	11.012	54.088	≦	223.440	○
	Nmin		47.473	33.53	5.531	≦	11.012	137.275	≦	223.440	○
During construction (during construction)	Nmax	230.350	595.537	56.06	2.840	≦	12.420	25.036	≦	252.000	○
	Nmin		351.513	43.99	3.135	≦	12.420	48.128	≦	252.000	○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As 96.36 = 28 cm2, and - 16 D reinforcing rods (As = 98.53 cm2) are arranged.

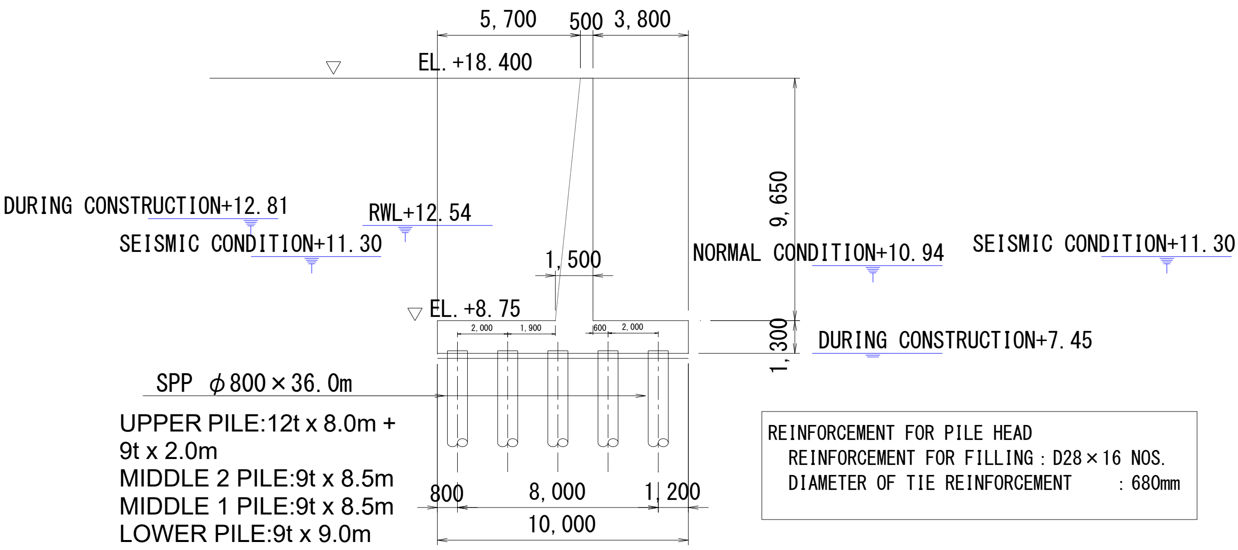
(g) Result of Examination

The results of the study on the pile foundation of the upstream left bank wing wall are shown in **Figure 7.4.52** and **Figure 7.4.53**. Details of the examination results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure 7.4.52 Upstream left bank wing wall Pile Arrangement



Source: Study team

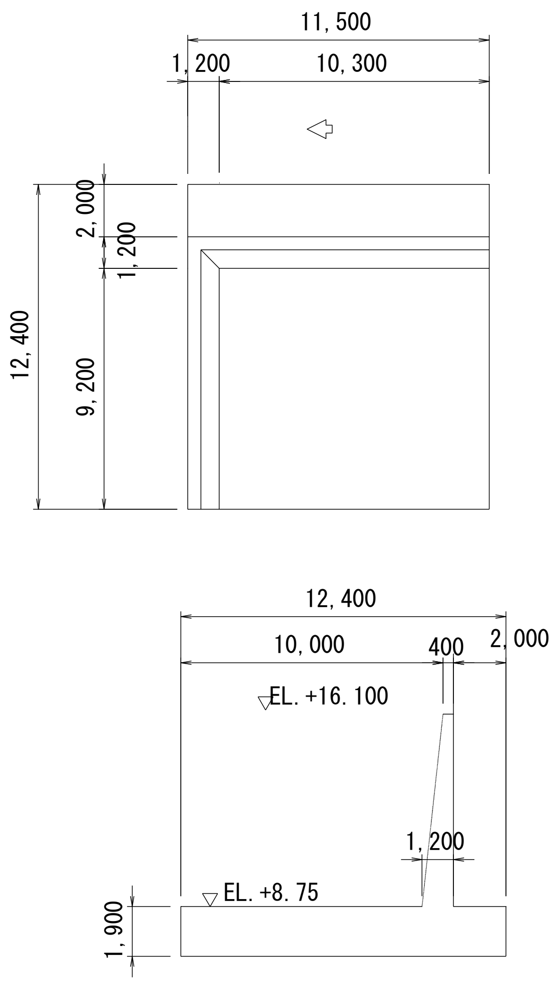
Figure 7.4.53 Pile Foundation Calculation Result

8) Study on Upstream Right Bank Wing Wall Pile Foundation

The pile foundation is designed and calculated for the upstream right bank wing wall. The details of the calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**.

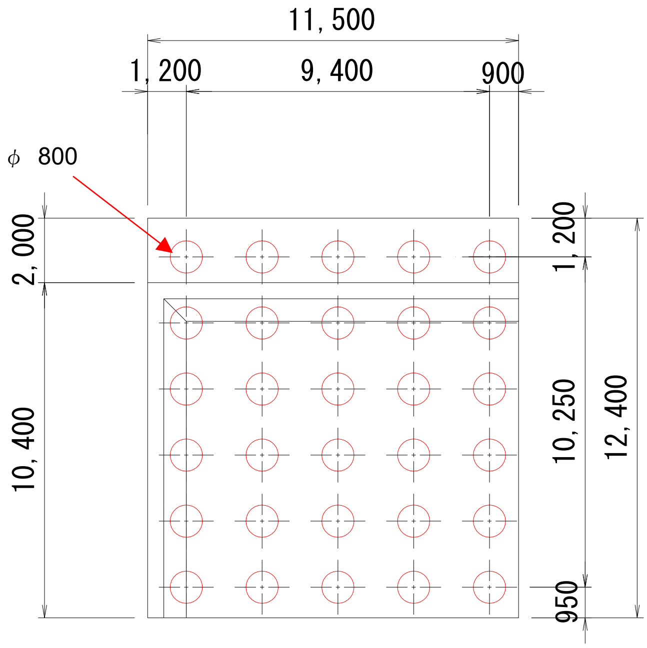
(a) Structural Dimension

The structural dimension drawing and pile arrangement of the upstream right bank wing wall are shown in below. The upstream right bank wing wall has the same wall height both in the flow direction and in the perpendicular direction to the flow and has a substantially square planar shape.



Source: Study team

Figure 7.4.54 Upstream Right Bank Wing Wall Structural Dimensions



Source: Study team

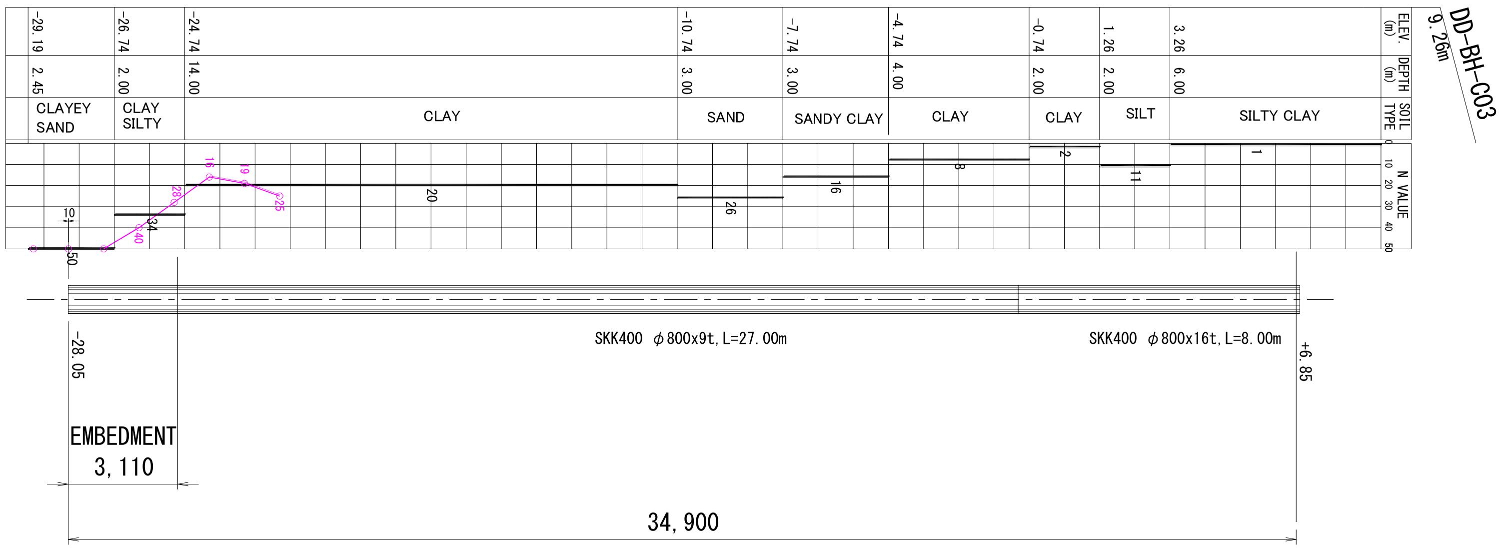
Figure 7.4.55 Upstream Right Bank Wing Wall Pile Arrangement

(b) Ground Condition

The ground conditions used for the design of pile foundations are shown in the next page. At the Cainta Floodgate site, there are three geological surveys: BH-C01 to BH-C03. The calculation is based on BH-C03, which is the most unfavorable geological survey for pile foundation design.

Table 7.4.68 List of soil properties (DD-BH-C03)

Stratum	Soil Type	N-value	Moisture content Wn (%)	Fine fraction content Fc (%)	Plasticity index Ip	Unit Weightγt (kN/m3)	Cohesion C (kN/m2)	Angle of shear resistance φ(°)	Deformation coefficient E 50 (MN/m2)	Consolidation settlement Target layer	Compression index Cc	Swelling index Cc
C1	Cohesive soil	1	67	93	21	(15)	(14)	(0)	(1.5)	○	(1.17)	(0.056)
C2	Cohesive soil	11	43	90	16	17	130	0				
C3	Cohesive soil	2	37	70	17	(16)	(14)	(0)	(1.5)	○	(0.42)	(0.041)
C4	Cohesive soil	8	53	96	65	17	100	0				
C5	Cohesive soil	16	37	60	30	18	200	0				
S1	Sandy soil	26	19	13		20	0	36				
C6	Cohesive soil	20	46	94	45	18	250	0				
C7	Cohesive soil	34	35	74	23	19	420	0				
S2	Sandy soil	50	33	45	21	21	0	40				



Source: Study team

Figure 7.4.56 Pile Foundation Design Ground Condition

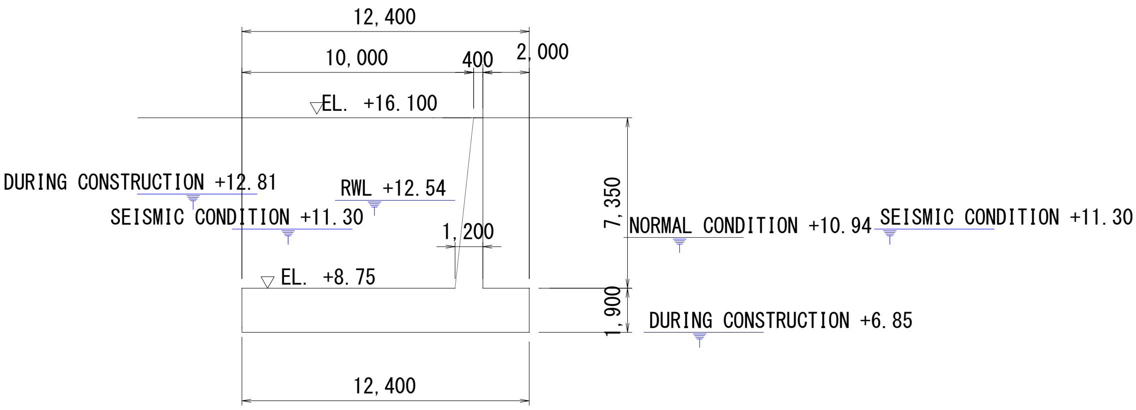
(c) Study Case

The calculation is made for the following cases.

Table 7.4.69 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

Calculation Direction	Study Case	Load Condition	Water Level Condition		Load Condition							
			Water Level in Rear Side	Water Level in Front Side	Body weight	Water Weight	Earth Pressure	Water pressure	Uplift pressure	Water Weight	Surcharge	Inertial Force
Perpendicular Direction To The Flow Flow Direction	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual water level	WL = 12.54 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During Construction	WL = 12.81 Water Level for Cofferdam	WL = 6.85 Lower Side of Bottom Slab	○	○	○	○	○	-	○	-

The residual water level shall be 2/3 of the difference between DFL = 13.340 and the OWL = 10.94.
Source: Study team



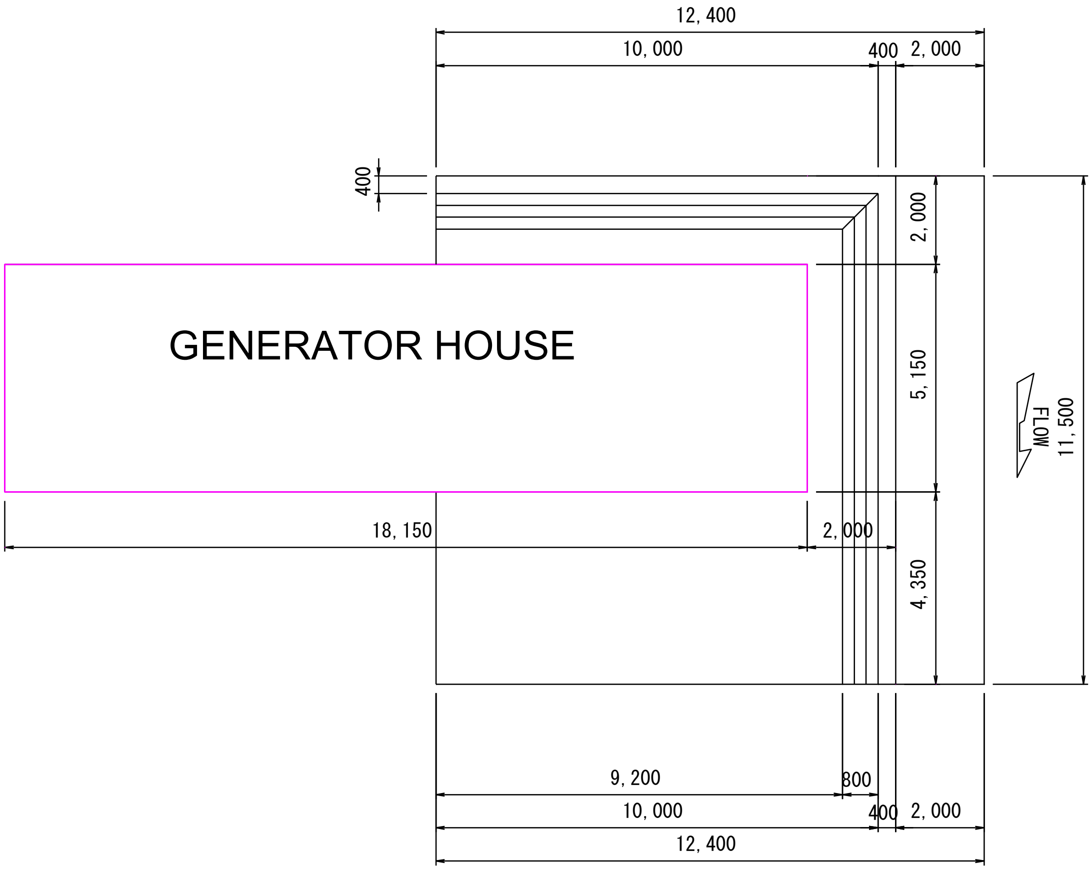
Source: Study team

Figure 7.4.57 Upstream Right Bank Wing Wall Water Level Condition

(d) Load Condition

The load conditions acting on the pile foundation are as follows. Since the generator house is located on the wing wall of the upstream right bank, the calculation is performed in consideration of the load (Approximately 47 kN/m2 rounded to 50.0 kN/m2) of the generator house.

Design Horizontal Seismic Coefficient	: Kh = 0.20
Back Earth Pressure	: Coulomb earth pressure
Soil Type of the Backfill	: Backfill γ = 19.0 kN/m3 Internal friction angle ϕ = 30 °, Cohesion c = 0 kN/m2
Surcharge (Normal Condition, During Construction)	: q0 = 10.0 kN/m2
Surcharge (Seismic Condition)	: q0 = 5.0 kN/m2
Surcharge (Generator House)	: q0 = 50.0 kN/m2 (Normal and Seismic Condition)



Source: Study team

Figure 7.4.58 Plan View of the Generator House

(e) Conditions For Consideration

The pile foundation shall be examined under the following conditions. In addition, the thickness of the bottom slab which can be regarded as a rigid body is secured. Since the upstream right bank wing wall has almost the same shape in both directions, it should be considered to stabilize in both directions independently.

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 30 (nos.)
・Pile Diameter		: 800.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 34.90 (m)

First Section: 7.90 (m) 62.0 (mm) SKK 400]
Second Section: 27.00 (m) 9.0 (mm) SKK 400]

(f) Calculation Result

(i) Perpendicular Direction to the Flow

The result of the calculation in the Perpendicular Direction to the Flow is shown in **Table 7.4.70** to **Table 7.4.73** . Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.70 Calculation Result of Pile Foundation of Upstream Right Bank Wing Wall in Perpendicular Direction to the Flow (Stability Calculation)

Load State (Water Level)	Amount of displacement (mm)		Pushing force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Normal Condition 1 (Zero Water Level)	3.57 ≦	10.0	1059.900 ≦	4991.016	845.515 > -1684.765		○
Normal Condition 2 (residual water level)	4.45 ≦	10.0	755.548 ≦	4991.016	738.301 > -1684.765		○
Seismic Condition 1 (Zero Water Level)	9.84 ≦	10.0	1471.503 ≦	7447.570	385.034 > -3308.030		○
Seismic Condition 2 (LWL)	9.80 ≦	10.0	1314.618 ≦	7447.570	180.648 > -3308.030		○
During construction (during construction)	6.13 ≦	10.0	922.089 ≦	4991.016	717.213 > -1684.765		○

Source: Study team

Table 7.4.71 Calculation Result of Pile Foundation of Upstream Right Bank Wing Wall in Perpendicular Direction to the Flow (Pile Body Stress)

Load State (Water Level)	Column Number	Type Of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2)	Tensile stress (N/mm2)	Evaluation
						Compressive stress (N/mm2)	Tensile stress (N/mm2)	
Normal Condition 1 (Zero Water Level)	6	P	Compress	280.342	1059.900	68.268 ≦ 140.000	-10.817 ≦ 140.000	○
	1	P	Tensile	280.342	845.515	62.457 ≦ 140.000	-16.628 ≦ 140.000	○
Normal Condition 2 (residual water level)	6	P	Compress	303.466	755.548	63.281 ≦ 140.000	-22.328 ≦ 140.000	○
	1	P	Tensile	303.466	738.301	62.813 ≦ 140.000	-22.795 ≦ 140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	652.472	1471.503	131.912 ≦ 186.200	-52.152 ≦ 186.200	○
	6	P	Tensile	652.472	385.034	102.467 ≦ 186.200	-81.597 ≦ 186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	639.013	1314.618	125.762 ≦ 186.200	-54.505 ≦ 186.200	○
	6	P	Tensile	639.013	180.648	95.029 ≦ 186.200	-85.238 ≦ 186.200	○
During construction (during construction)	1	P	Compress	376.173	922.089	78.050 ≦ 210.000	-28.070 ≦ 210.000	○
	6	P	Tensile	376.173	717.213	72.497 ≦ 210.000	-33.622 ≦ 210.000	○

Legend: P···Perpendicular Direction to Flow
Source: Study team

Table 7.4.72 Calculation Result of Pile Foundation of Upstream Right Bank Wing Wall in Perpendicular Direction to the Flow (Shear Stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2) Calculated τ		Allowable τa1	Evaluation
Normal Condition 1 (Zero Water Level)	1	Perpendicular to Flow	105.651	2.863	<<	80.000	○
Normal Condition 2 (residual water level)	1	Perpendicular to Flow	120.989	3.279	<<	80.000	○
Seismic Condition 1 (Zero Water Level)	6	Perpendicular to Flow	351.337	9.522	<<	106.400	○
Seismic Condition 2 (LWL)	6	Perpendicular to Flow	347.078	9.406	<<	106.400	○
During construction (during construction)	6	Perpendicular to Flow	156.953	4.254	<<	120.000	○

Source: Study team

・Section Outer radius R = 55.000 (cm) Inner radius Ro = 0.000 (cm)

・Rebar D 29 - 16 (@ 134) As = 102.78 (cm2)

Table 7.4.73 Calculation Result of Pile Foundation of Upstream Right Bank Wing Wall In Perpendicular Direction to the Flow (Pile Head Reinforcement)

Load state (Water Level)	Review State	Cross-sectional force M (kNm)	Cross-sectional force N (kN)	Neutral axis X (cm)	Compressive stress (N/mm2) σ c		σca	Tensile stress (N/mm2) σ s		σ sa	Evaluation
Normal Condition 1 (Zero Water Level)	Nmax Nmin	280.342	1059.900 845.515	72.83 62.22	3.210 3.305	≦ ≦	8.280 8.280	10.692 21.340	≦ ≦	168.000 168.000	○ ○
Normal Condition 2 (residual water level)	Nmax Nmin	303.466	755.548 738.301	55.21 54.49	3.717 3.734	≦ ≦	8.280 8.280	34.125 35.473	≦ ≦	168.000 168.000	○ ○
Seismic Condition 1 (Zero Water Level)	Nmax Nmin	652.472	1471.503 385.034	52.32 37.00	8.152 9.309	≦ ≦	11.012 11.012	85.730 196.231	≦ ≦	223.440 223.440	○ ○
Seismic Condition 2 (LWL)	Nmax Nmin	639.013	1314.618 180.648	50.02 35.00	8.124 9.280	≦ ≦	11.012 11.012	94.957 214.747	≦ ≦	223.440 223.440	○ ○
During construction (during construction)	Nmax Nmin	376.173	922.089 717.213	54.72 48.36	4.622 4.847	≦ ≦	12.420 12.420	43.428 61.083	≦ ≦	252.000 252.000	○ ○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As 102.78 = 28 cm2, and D28 x17 reinforcing rods (As = 104.69 cm2) are arranged.

(ii) Flow Direction

For the flow direction calculation is shown in **Table 7.4.74** to **Table 7.4.77**.

Table 7.4.74 Calculation Result of Pile Foundation of the Upstream Right Bank Wing Wall in the Flow Direction (Stability Calculation)

Load state (Water Level)	Amount of displacement (mm) Calculated		Allowable	Pushing Force (kN) Calculated		Allowable	Pull-put Force (kN) Calculated		Allowable	Evaluation
Normal Condition 1 (Zero Water Level)	3.71	≦	10.0	1104.147	≦	4991.016	683.489	>	-1684.765	○
Normal Condition 2 (residual water level)	4.40	≦	10.0	970.665	≦	4991.016	447.806	>	-1684.765	○

Load state (Water Level)	Amount of displacement (mm)		Pushing Force (kN)		Pull-put Force (kN)		Evaluation
	Calculated	Allowable	Calculated	Allowable	Calculated	Allowable	
Seismic Condition 1 (Zero Water Level)	8.79 ≦	10.0	1589.187 ≦	7447.570	151.390 >	-3308.030	○
Seismic Condition 2 (LWL)	8.61 ≦	10.0	1404.899 ≦	7447.570	2.745 >	-3308.030	○
During construction (during construction)	6.03 ≦	10.0	1168.634 ≦	4991.016	399.665 >	-1684.765	○

Source: Study team

Table 7.4.75 Calculation Result of Pile Foundation of the Upstream Right Bank Wing Wall in the Flow Direction (Pile Body Stress)

Load state (Water Level)	Column Number	Type of Row	Compress or Tensile	M (kNm)	N (kN)	Compressive stress (N/mm2)		Tensile stress (N/mm2)		Evaluation
						Calculated	Allowable	Calculated	Allowable	
Normal Condition 1 (Zero Water Level)	1	P	Compress	166.366	1104.147	53.390	≦ 140.000	6.458	≦ 140.000	○
	5	P	Tensile	166.366	683.489	41.990	≦ 140.000	-4.942	≦ 140.000	○
Normal Condition 2 (residual water level)	1	P	Compress	192.429	970.665	53.449	≦ 140.000	-0.836	≦ 140.000	○
	5	P	Tensile	192.429	447.806	39.279	≦ 140.000	-15.006	≦ 140.000	○
Seismic Condition 1 (Zero Water Level)	1	P	Compress	455.294	1589.187	107.289	≦ 186.200	-21.150	≦ 186.200	○
	5	P	Tensile	455.294	151.390	68.323	≦ 186.200	-60.117	≦ 186.200	○
Seismic Condition 2 (LWL)	1	P	Compress	448.140	1404.899	101.286	≦ 186.200	-25.136	≦ 186.200	○
	5	P	Tensile	448.140	2.745	63.285	≦ 186.200	-63.136	≦ 186.200	○
During construction (during construction)	1	P	Compress	252.985	1168.634	67.356	≦ 210.000	-4.012	≦ 210.000	○
	5	P	Tensile	252.985	399.665	46.515	≦ 210.000	-24.852	≦ 210.000	○

Legend: P···Perpendicular Direction to Flow

Source: Study team

Table 7.4.76 Calculation Result of Pile Foundation of the Upstream Right Bank Wing Wall in the Flow Direction (Shear Stress)

Load state (Water Level)	Column Number	Type Of Row	Shear Force (kN)	Shear Stress (N/mm2)		Evaluation
				Calculated τ	Allowable τa1	
Normal Condition 1 (Zero Water Level)	5	Perpendicular to Flow	80.763	2.189 <<	80.000	○
Normal Condition 2 (residual water level)	5	Perpendicular to Flow	94.636	2.565 <<	80.000	○
Seismic Condition 1 (Zero Water Level)	5	Perpendicular to Flow	279.773	7.582 <<	106.400	○
Seismic Condition 2 (LWL)	5	Perpendicular to Flow	274.769	7.447 <<	106.400	○
During construction (during construction)	5	Perpendicular to Flow	127.160	3.446 <<	120.000	○

Source: Study team

・Section Outer radius R = 55.000 (cm) Inner radius Ro = 0.000 (cm)

・Rebar D 29 - 16 (@ 134) As = 102.78 (cm2)

Table 7.4.77 Calculation Result of Pile Foundation of the Upstream Right Bank Wing Wall in the Flow Direction (Pile Head Reinforcement)

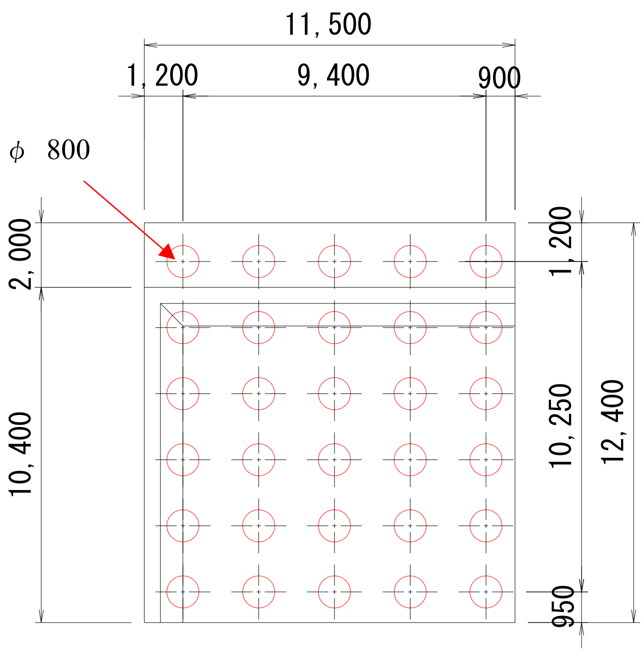
Load state (Water Level)	Review State	Cross-sectional force		Neutral axis	Compressive stress (N/mm2)			Tensile stress (N/mm2)			Evaluation
		M (kNm)	N (kN)	X (cm)	σ c		σ ca	σ s		σ sa	
Normal Condition 1 (Zero Water Level)	Nmax	166.366	1104.147	103.28	2.136	≦	8.280	-4.430	≦	168.000	○
	Nmin		683.489	77.20	1.902	≦	8.280	4.362	≦	168.000	○
Normal Condition 2 (residual water level)	Nmax	192.429	970.665	88.42	2.254	≦	8.280	0.223	≦	168.000	○
	Nmin		447.806	53.19	2.389	≦	8.280	24.133	≦	168.000	○
Seismic Condition 1 (Zero Water Level)	Nmax	455.294	1589.187	68.83	5.249	≦	11.012	23.078	≦	223.440	○
	Nmin		151.390	35.31	6.594	≦	11.012	150.378	≦	223.440	○
Seismic Condition 2 (LWL)	Nmax	448.140	1404.899	63.86	5.247	≦	11.012	30.977	≦	223.440	○
	Nmin		2.745	33.36	6.598	≦	11.012	165.089	≦	223.440	○
During construction (during construction)	Nmax	252.985	1168.634	83.58	2.919	≦	12.420	2.840	≦	252.000	○
	Nmin		399.665	45.04	3.353	≦	12.420	49.092	≦	252.000	○

Source: Study team

The pile head reinforcement has a reinforcement amount exceeding As 102.78 = 28 cm2, and -17 D reinforcing rods (As = 104.69 cm2) are arranged.

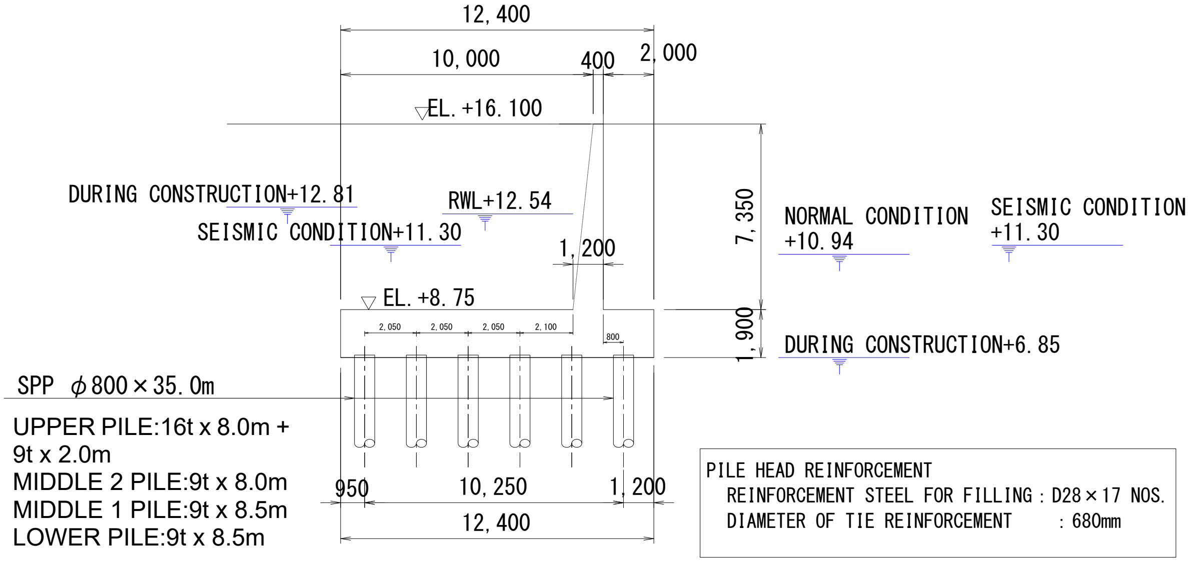
(g) Result of Examination

The results of the study on the pile foundation of the upstream right bank wing wall are shown in **Figure 7.4.59** and **Figure 7.4.60**. Details of the examination results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure 7.4.59 Upstream Right Bank Wing Wall Pile Arrangement



Source: Study team

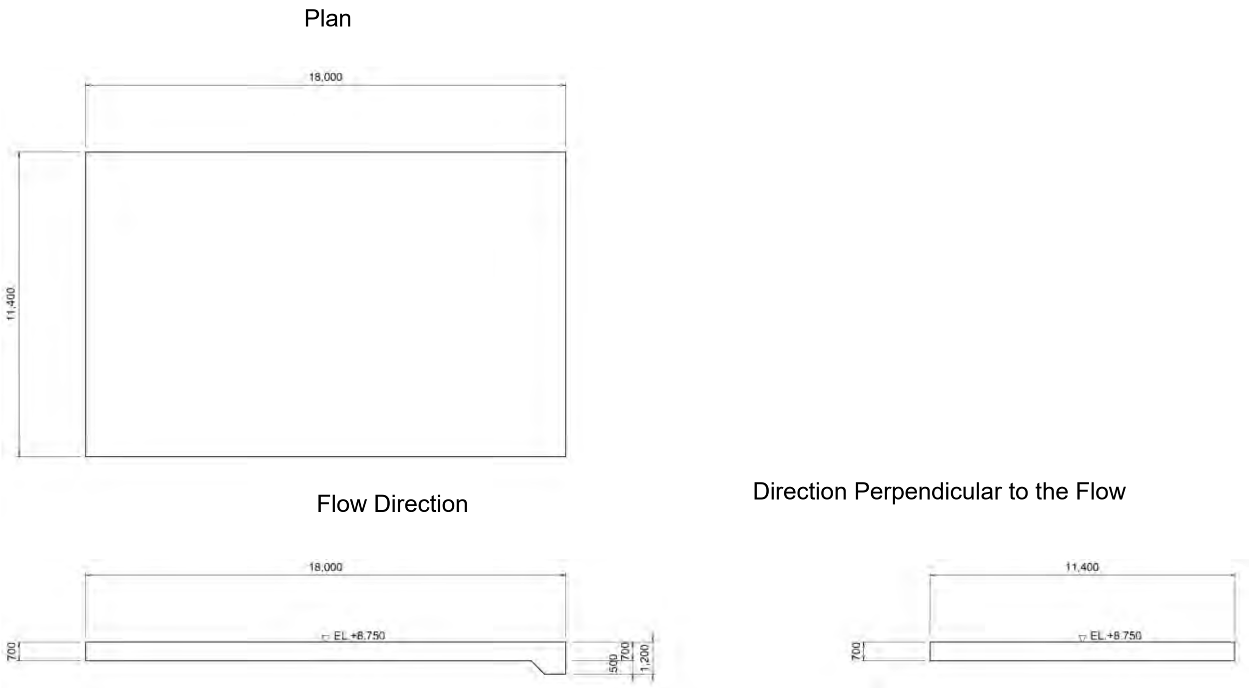
Figure 7.4.60 Pile Foundation Calculation Result

9) Examination of Downstream Apron

Design calculation of pile foundation is performed for downstream apron. The details of the calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**. As for the downstream apron, since the apron in the right and left banks is symmetry, the pile foundation is designed and calculated for the apron in the center and the apron in the left bank,

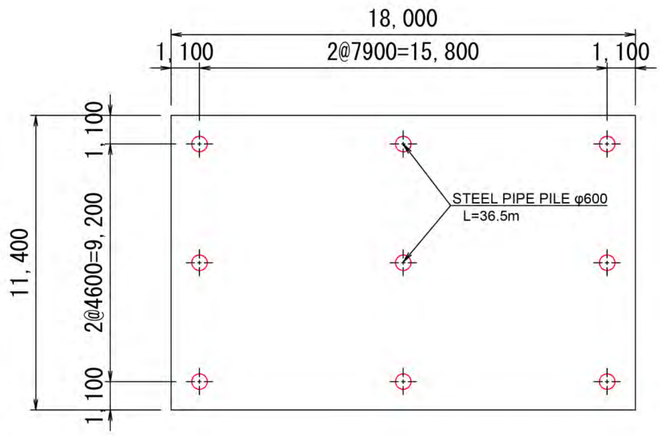
(a) Structural Dimension

The structure and dimensions of the downstream tap are shown in below.



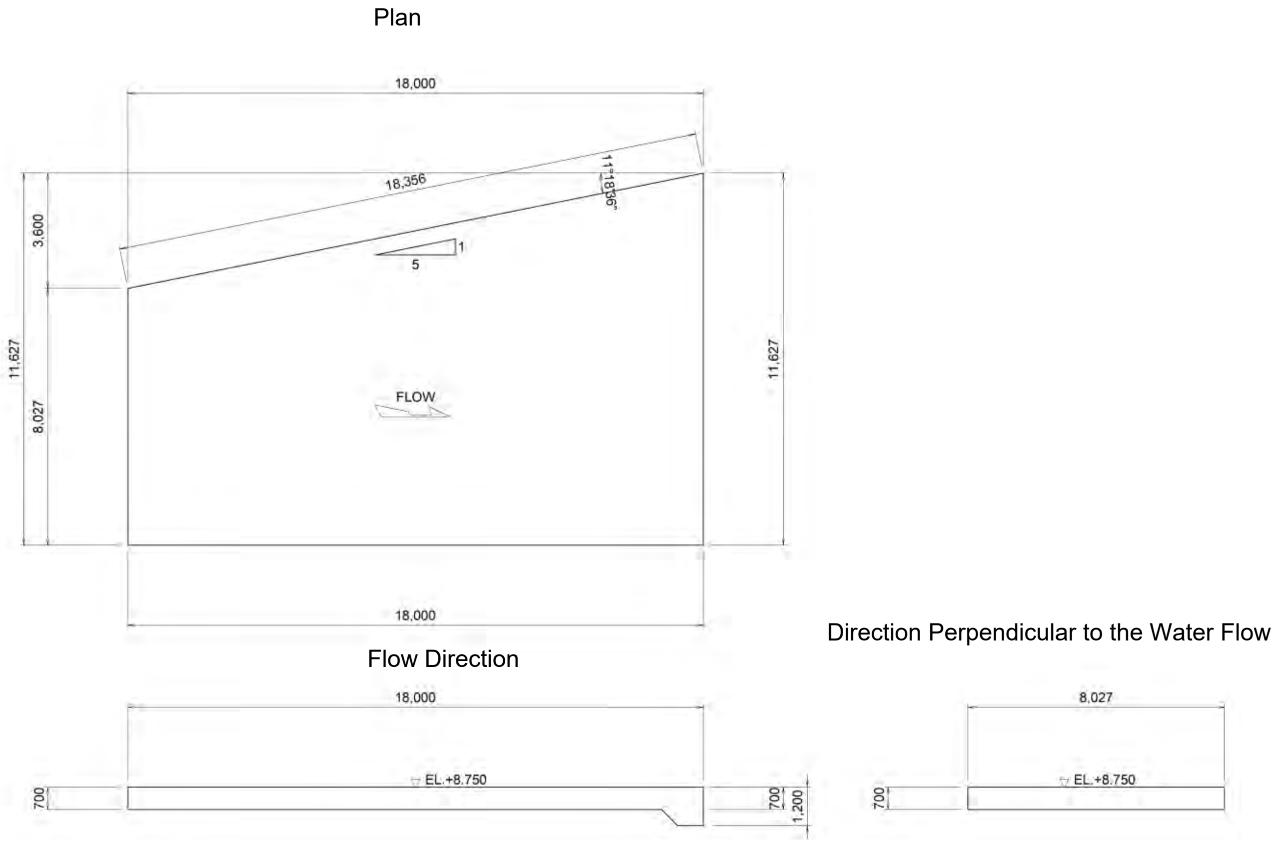
Source: Study team

Figure 7.4.61 Downstream Apron Structural Dimensions (Center)



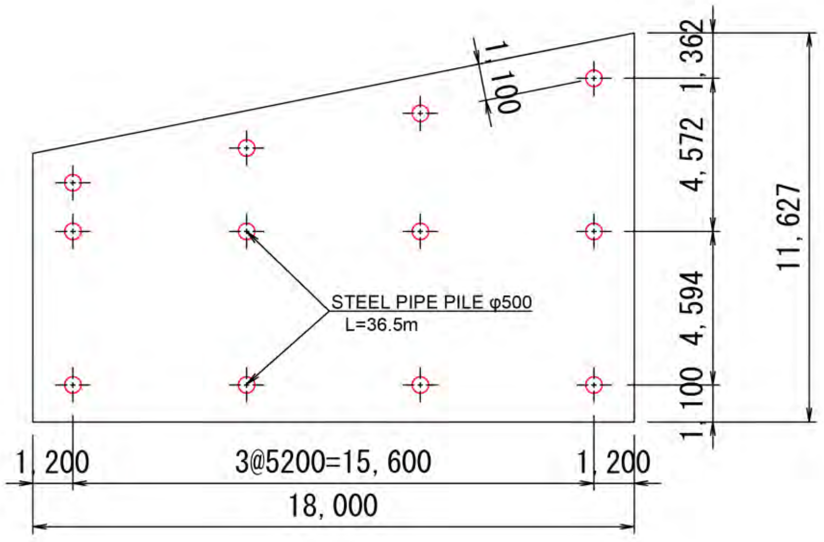
Source: Study team

Figure 7.4.62 Downstream Apron Pile Arrangement (Center)



Source: Study team

Figure 7.4.63 Downstream Apron Structural Dimensions (Left Bank)



Source: Study team

Figure 7.4.64 Downstream Apron Pile Arrangement (Left Bank)

(b) Ground Condition

As for the ground condition, the ground condition of DD-BH-C03 is used as in the case of the center pier and the end pier.

(c) Study Case

The calculation is made for the following cases.

Table 7.4.78 Load Case List

Member	Location	Calculation direction	Case	Case name	Additional factor of allowable stress
Downstream apron	Right and left bank	Perpendicular direction to the flow	1	During floods (at floodway DFL)	1.0
			2	At construction (1)	1.5
			3	At construction (2)	1.5
			4	At construction (3)	1.5
	Center	Perpendicular direction to the flow	1	During floods (at floodway DFL)	1.0
			2	During construction (cofferdam)	1.5
			3	At construction (1)	1.5
			4	At construction (2)	1.5
			5	At construction (3)	1.5

Source: Study team

(d) Load Condition

The load acting on the pile foundation is set from the load diagram set for each load case.

(i) Crane Load

In the downstream side apron part, a crane passes over the apron part during construction (during the installation of the gates). Therefore, the calculation is performed assuming the crane load. Crane load is using the value calculated in P7-500

(ii) Load of Cofferdam

For the apron part, a cofferdam is constructed on the apron in the left bank construction (During the second phase of construction). Therefore, the load on the cofferdam and the soil filling is considered in the calculation. The standard cross section of the cofferdam section is shown in **Figure 7.4.65**.

Not Presented Due to the Closed Information

Source: Study team

Figure 7.4.65 Cofferdam Part on Floor Slab

Cofferdam Load

Surcharge: **

SSP LOAD: **

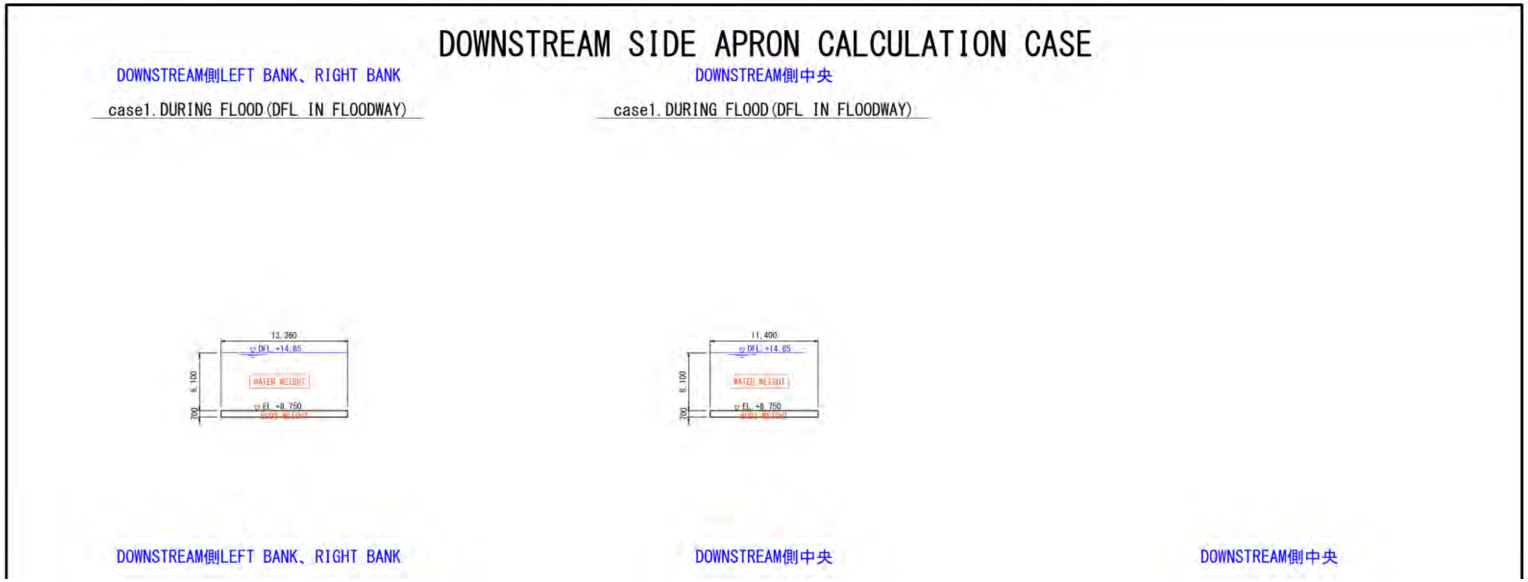
Soil Filling: **

Total Σ W = ***********

W = *********** = 125 kN/m2

Note: **** is not presented due to the closed information.

The diagram set on the next page is shown.



Not Presentaed Due to the Closed Information

Source: Study team

Figure 7.4.66 Downstream Apron Load Diagram

(e) Conditions for Consideration

The examination conditions of pile foundation are shown below.

・Downstream center apron

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 9 (nos.)
・Pile Diameter		: 600.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 36.40 (m)

First Section: 8.40 (m) 9.0 (mm) SKK 400]
Second Section: 28.00 (m) 9.0 (mm) SKK 400]

・Downstream left bank apron

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 12 (nos.)
・Pile Diameter		: 500.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 36.40 (m)

First Section: 8.40 (m) 9.0 (mm) SKK 400]
Second Section: 28.00 (m) 9.0 (mm) SKK 400]

(f) Calculation Result

(i) Center of the Downstream Apron

The calculation result at the center of the downstream apron is shown in **Table 7.4.79** and **Table 7.4.80**. Of the calculation results, the case surrounded by the red line is the decision case.

Table 7.4.79 Calculation Result of Pile Foundation of Downstream Apron Center In The Perpendicular Direction to the Flow (1/2)

Load Case Number Abbreviation / Origin Action Force			1 Perpendicular, Case1	2 Perpendicular, Case2	3 Perpendicular, Case3	4 Perpendicular, Case 4
Vo Ho Mo		kN kN kNm	15714.22 0.00 0.00	18920.38 0.00 -3973.28	3897.36 0.00 -1169.21	3897.36 0.00 935.37
Origin Displacement						
δx δz α		mm mm rad	0.00 21.08 0.00000000	-1.06 25.39 -0.00037056	-0.31 5.23 -0.00010904	0.25 5.23 0.00008723
δf, δa		mm	0.00 ≦ 10.00	0.38 ≦ 10.00	0.11 ≦ 10.00	0.09 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra PNmin, Pa		kN kN	1746.02 ≦ 3408.00 1746.02 ≧ 0.00	2243.42 ≦ 3408.00 1961.11 ≧ 0.00	474.58 ≦ 3408.00 391.50 ≧ 0.00	466.27 ≦ 3408.00 399.81 ≧ 0.00
Horizontal Reaction Force						
PH		kN	0.00	0.00	0.00	0.00
Pile Moment						
Pile Head Mt Underground Mm		kNm kNm	0.00 0.00	-8.60 -8.60	-2.53 0.12	2.02 2.02
Pile Body Stress						
1 Section	σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-117.75 ≥ -140.00 -117.75 ≦ 140.00 0.000 ≦ 80.000	-155.28 ≥ -210.00 -128.27 ≦ 210.00 0.127 ≦ 120.000	-33.18 ≥ -210.00 -25.23 ≦ 210.00 0.037 ≦ 120.000	-32.38 ≥ -210.00 -26.02 ≦ 210.00 0.030 ≦ 120.000
Evaluation			OK	OK	OK	OK

Source: Study team

Table 7.4.80 Calculation Result of Pile Foundation of Downstream Apron Center In The Perpendicular Direction to the Flow (2/2)

Load Case Number Abbreviation / Origin Action Force			5 Perpendicular, Case 5
Vo Ho Mo		kN kN kNm	3807.36 0.00 0.00
Origin Displacement			
δx δz α		mm mm rad	0.00 5.11 0.00000000
δf, δa		mm	0.00 ——
Vertical Reaction Force			
PNmax, Ra PNmin, Pa		kN kN	423.04 —— 423.04 ——
Horizontal Reaction Force			
PH		kN	0.00
Pile Moment			
Pile Head Mt Underground Mm		kNm kNm	0.00 0.00
Pile Body Stress			
1 Section	σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-28.53 ≥ -210.00 -28.53 ≦ 210.00 0.000 ≦ 120.000
Evaluation			OK

Source: Study team

(ii) Right and Left Sides of the Downstream Apron

The calculation result of the right and left sides of the downstream apron is shown in **Table 7.4.81**. Of the calculation results, the case surrounded by the red line is the decision case.

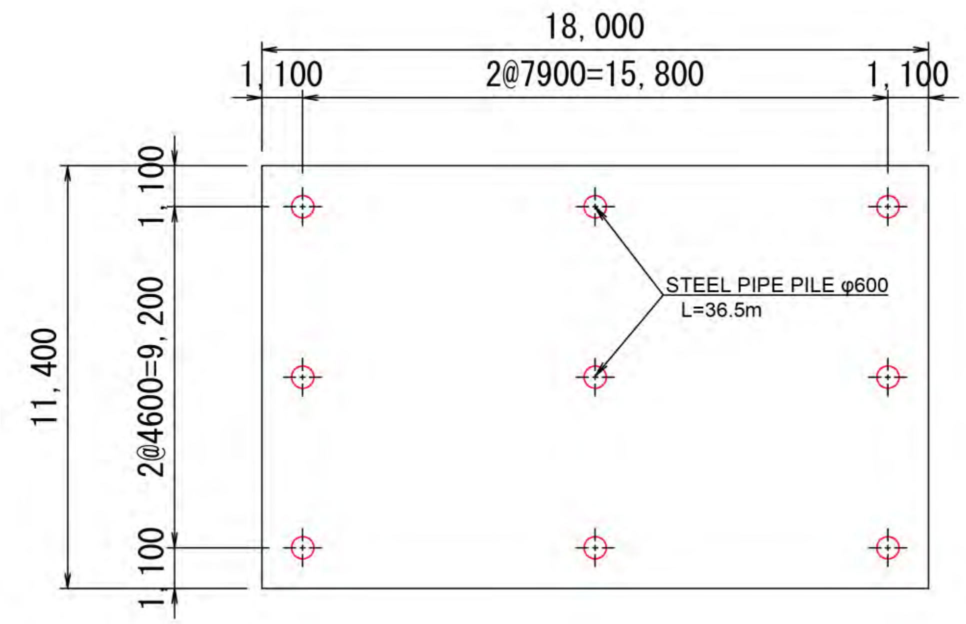
Table 7.4.81 Calculation Result of Pile Foundation of Downstream Right and Left Bank Apron (Perpendicular Direction to the Flow)

Load Case Number Abbreviation			1 Perpendicular, Case1	2 Perpendicular, Case3	3 Perpendicular, Case 4	4 Perpendicular, Case 5
Origin Action Force						
Vo		kN	13545.92	3421.68	3421.68	3331.68
Ho		kN	0.00	0.00	0.00	0.00
Mo		kNm	677.30	-684.34	1402.89	133.27
Origin Displacement						
δx		mm	2.44	0.40	0.94	0.59
δz		mm	14.24	3.54	3.67	3.50
α		rad	0.00091351	0.00014799	0.00034992	0.00022146
δf, δa		mm	0.81 ≦ 10.00	0.13 ≦ 10.00	0.31 ≦ 10.00	0.20 ≦ 10.00
Vertical Reaction Force						
PNmax, Ra		kN	1515.70 ≦ 2706.00	347.82 ≦ 2706.00	433.33 ≦ 2706.00	371.43 ≦ 2706.00
PNmin, Pa		kN	821.54 ≧ 0.00	235.36 ≧ 0.00	167.44 ≧ 0.00	203.15 ≧ 0.00
Horizontal Reaction Force						
PH		kN	0.00	0.00	0.00	0.00
Pile Moment						
Pile Head Mt		kNm	13.00	2.11	4.98	3.15
Underground Mm		kNm	13.00	2.11	4.98	3.15
Pile Body Stress						
1 Section	σ c, σ ca	N/mm2	-131.84 ≥ -140.00	-29.66 ≥ -210.00	-38.54 ≥ -210.00	-32.28 ≥ -210.00
	σt, σta	N/mm2	-57.95 ≦ 140.00	-17.69 ≦ 210.00	-10.24 ≦ 210.00	-14.37 ≦ 210.00
	τ, τa	N/mm2	0.259 ≦ 80.000	0.042 ≦ 120.000	0.099 ≦ 120.000	0.063 ≦ 120.000
Evaluation			OK	OK	OK	OK

Source: Study team

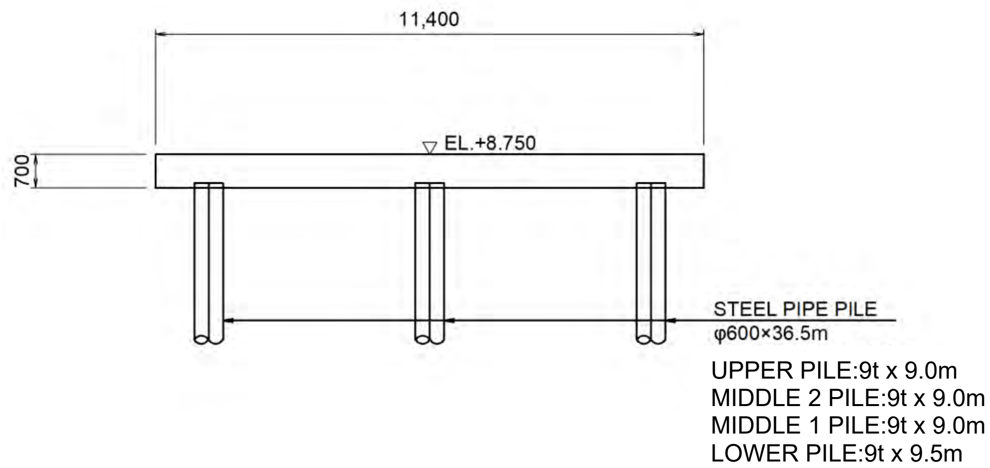
(g) Result of Examination

The examination result of the downstream apron pile foundation is shown below. Details of the examination results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



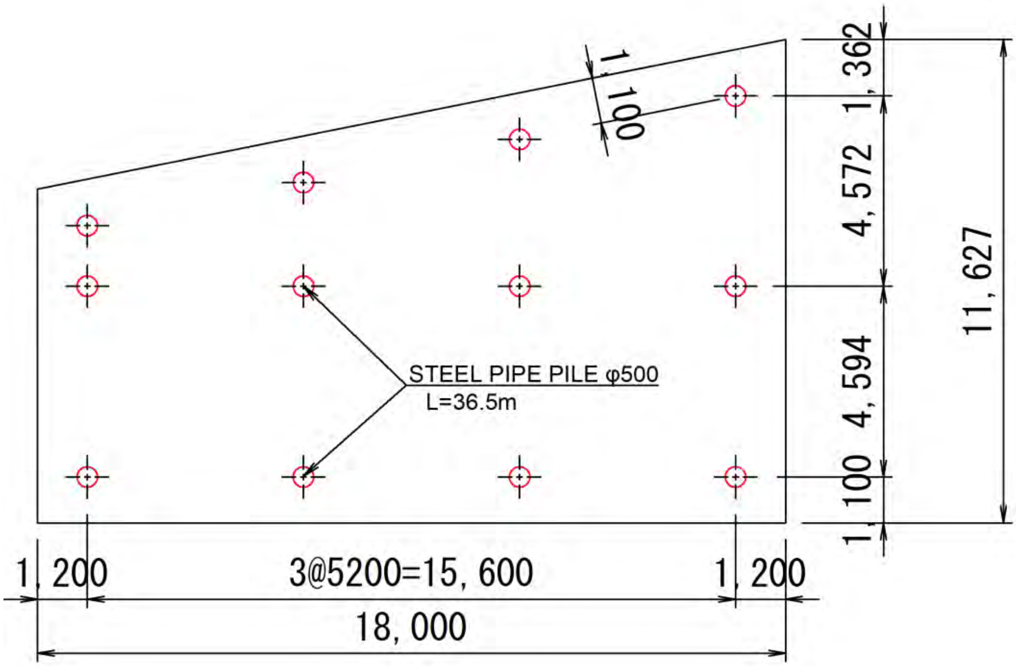
Source: Study team

Figure 7.4.67 Downstream Apron Pile Arrangement (Center)



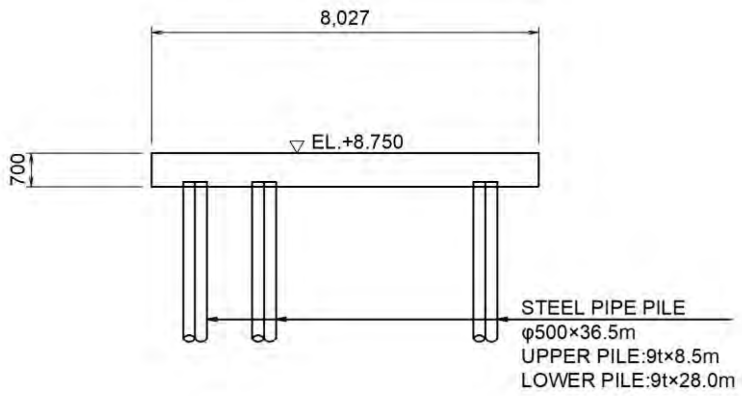
Source: Study team

Figure 7.4.68 Downstream Aproned Pile Foundation Calculation Result (Center)



Source: Study team

Figure 7.4.69 Downstream Apron Pile Arrangement (Left Bank)



Source: Study team

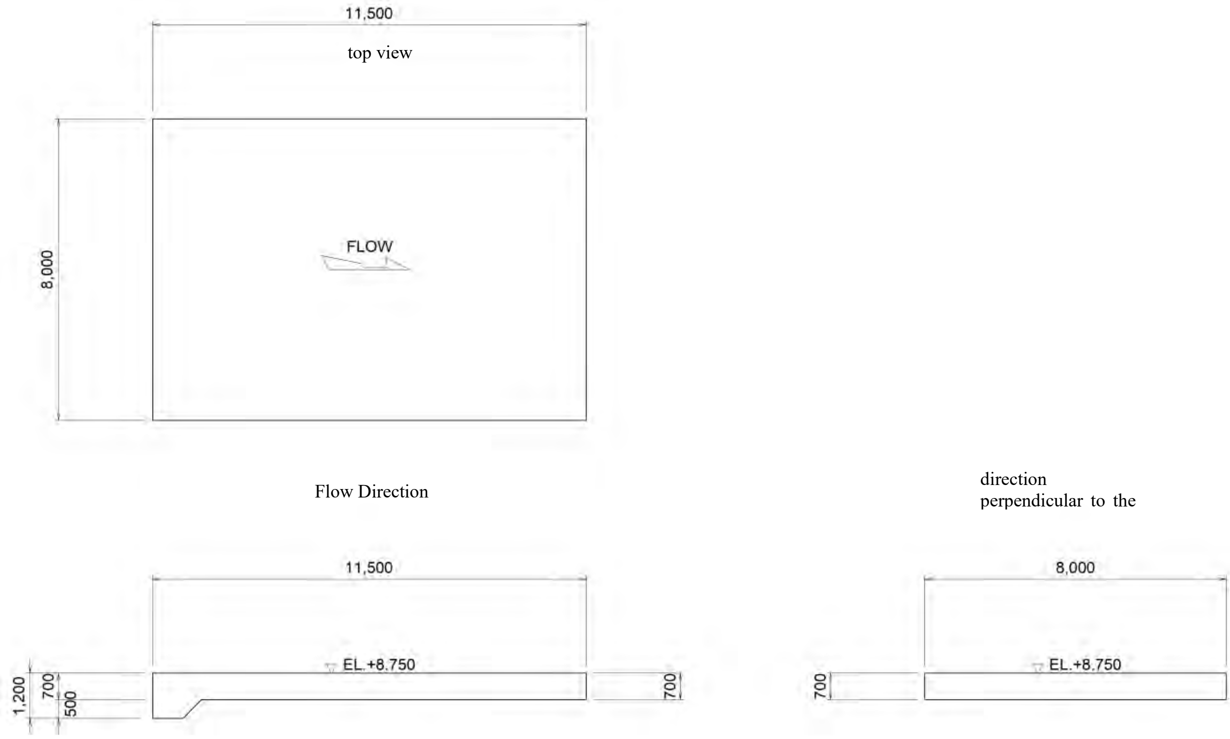
Figure 7.4.70 Downstream Aproned Pile Foundation Calculation Result (Left Bank)

10) Examination of Apron in the Upstream

Design calculation of pile foundation is performed for upstream apron. The details of the calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**.

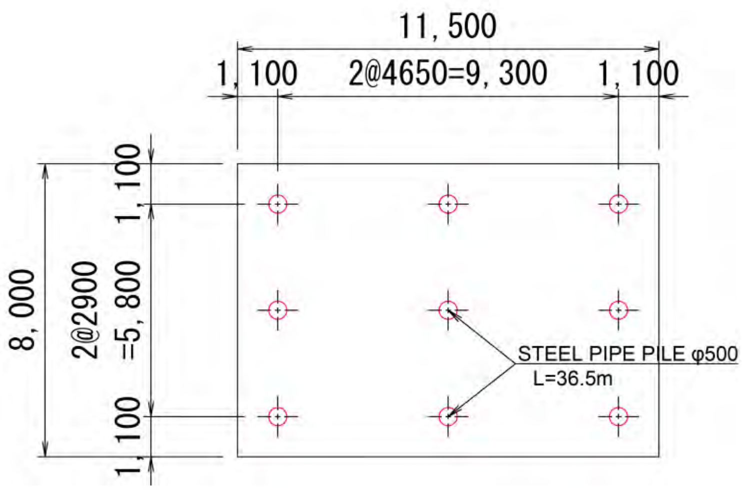
(a) Structural Dimension

The structural dimension of the upstream apron is shown.



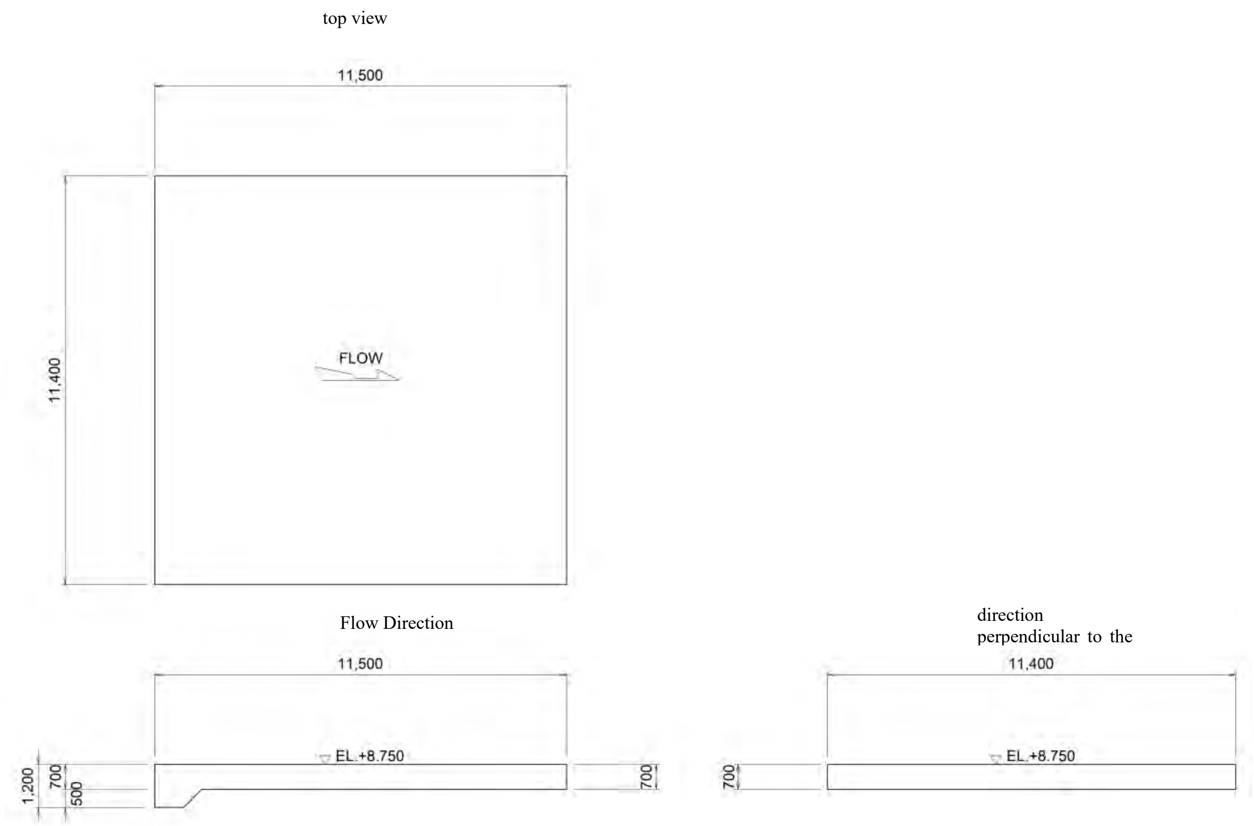
Source: Study team

Figure 7.4.71 Dimensions of Upstream Apron (Left Bank)



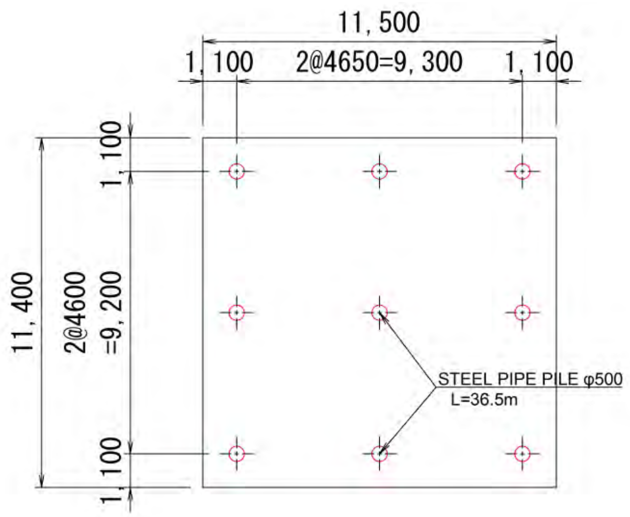
Source: Study team

Figure 7.4.72 Downstream Apron Pile Arrangement (Left Bank)



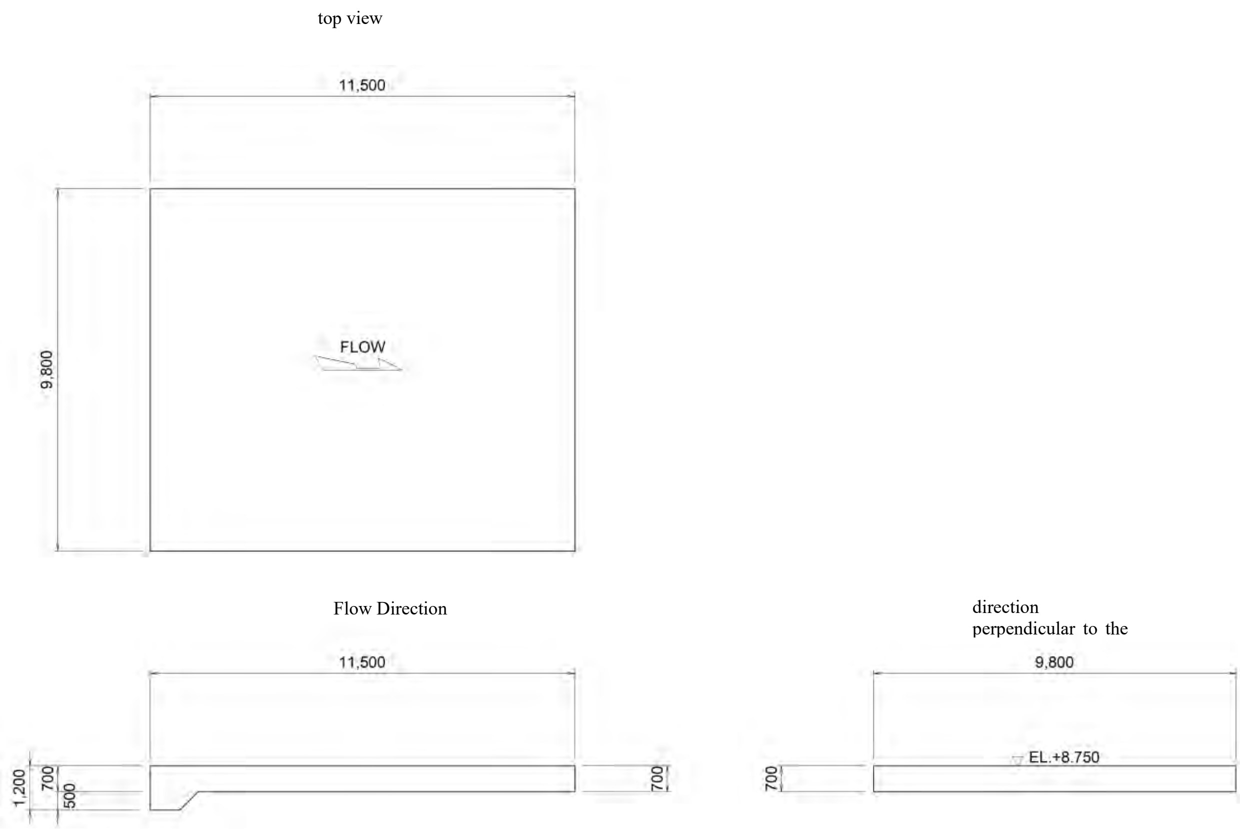
Source: Study team

Figure 7.4.73 Dimensions of Upstream Apron (Center)



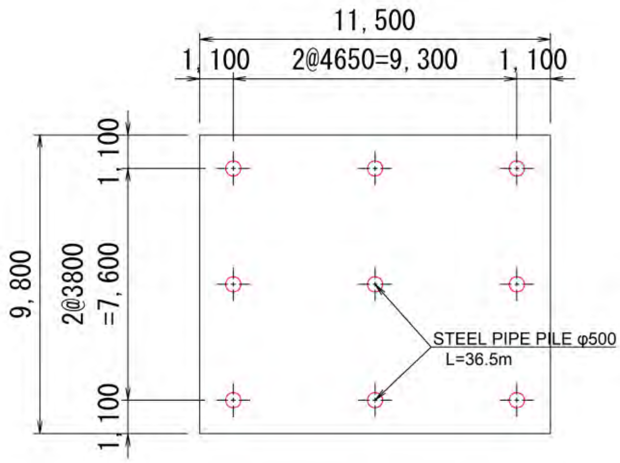
Source: Study team

Figure 7.4.74 Upstream Apron Pile Arrangement (Center)



Source: Study team

Figure 7.4.75 Dimensions of Upstream Apron (Right Bank)



Source: Study team

Figure 7.4.76 Upstream Apron Pile Arrangement (Right Bank)

(b) Ground Condition

As for the ground condition, the ground condition of DD-BH-C03 is used as in the case of the center pier and the end pier.

(c) Study Case

The calculation is made for the following cases.

Table 7.4.82 Load Case List

Member	Location	Calculation Direction	Case	Case Name	Additional Factor Of Allowable Stress
Upstream Apron	Left Bank	Perpendicular Direction To The Flow	1	During Floods (At Tributary DFL)	1.0
	Center	Perpendicular Direction To The Flow	1	During Floods (At Tributary DFL)	1.0
			2	During Construction (Cofferdam)	1.5
	Right Bank	Perpendicular Direction To The Flow	1	During Floods (At Tributary DFL)	1.0

Source: Study team

(d) Load Condition

The loads acting on pile foundations are set based on summarized load from the diagram for each load case shown in **Table 7.4.82**.

(i) Load of Cofferdam

For the apron central part, a cofferdam is constructed on the apron in the left bank construction (During the second phase of construction). Therefore, the load on the cofferdam and the soil filling is considered in the calculation. The standard cross section of the cofferdam section is shown in **Figure 7.4.77**.

Not Presentaed Due to the Closed Information

Source: Study team

Figure 7.4.77 Cofferdam Part on Floor Slab

・Cofferdam Load

Surcharge: **

SSP LOAD: **

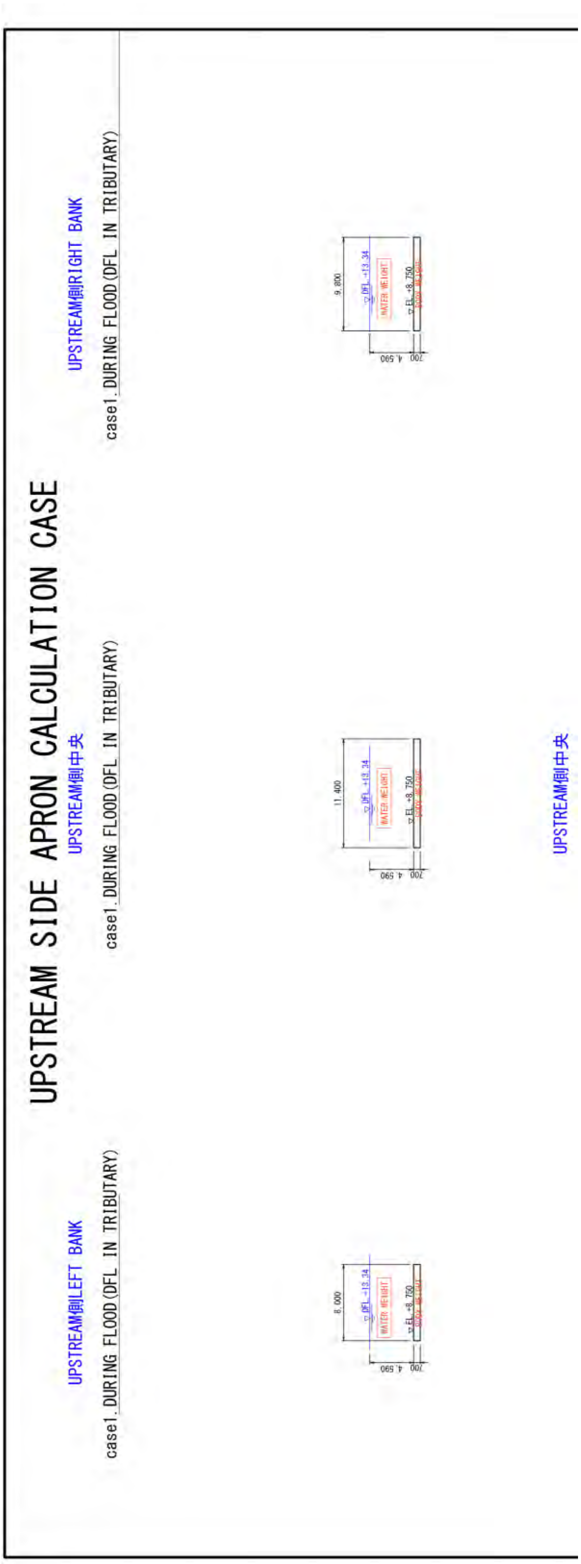
Soil Filling: **

Total ΣW = ***********

W = *********** = 125 kN/m2

Note: **** is not presented due to the closed information.

The diagram set on the next page is shown.



Not Presentaed Due to the Closed Information

Figure 7.4.78 Upstream Apron Load Diagram

Source: Study team

(e) Conditions for Consideration

The examination conditions of pile foundation are shown below. The following conditions are common to all members of the upstream apron.

・Upstream center apron

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 9 (nos.)
・Pile Diameter		: 600.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 36.40 (m)

First Section: 8.40 (m) 9.0 (mm) SKK 400]
Second Section: 28.00 (m) 9.0 (mm) SKK 400]

(f) Calculation Result

(i) Upstream Apron Center

The calculation result of the upstream apron center is shown in **Table7.4.83**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.83 Result of Calculation of Foundation of Central Pile of Upstream Apron (Perpendicular Direction to the Flow)

Load Case Number Abbreviation			1	2
Origin Action Force			Perpendicular, Case1	Perpendicular, Case2
Vo		kN	8099.62	12088.02
Ho		kN	0.00	0.00
Mo		kNm	0.00	-2538.48
Origin Displacement				
δx		mm	0.00	-0.64
δz		mm	10.88	16.23
α		rad	0.00000000	-0.00023877
δf, δa		mm	0.00 ≦ 10.00	0.21 ≦ 10.00
Vertical Reaction Force				
PNmax, Ra		kN	899.96 ≦ 2706.00	1433.98 ≦ 2706.00
PNmin, Pa		kN	899.96 ≧ 0.00	1252.25 ≧ 0.00
Horizontal Reaction Force				
PH		kN	0.00	0.00
Pile Moment				
Pile Head Mt		kNm	0.00	-3.40
Underground Mm		kNm	0.00	-3.40
Pile Body Stress				
1 Section	σ c, σ ca	N/mm2	-73.08 ≥ -140.00	-118.73 ≥ -210.00
	σt, σta	N/mm2	-73.08 ≦ 140.00	-99.40 ≦ 210.00
	τ, τa	N/mm2	0.000 ≦ 80.000	0.068 ≦ 120.000
Evaluation			OK	OK

Source: Study team

(ii) Upstream Apron Left Bank

The calculation result of the upstream apron left bank is shown in **Table7.4.84**.

Table7.4.84 Calculation Result of Pile Foundation on the Upstream Left Bank Apron (Perpendicular Direction to the Flow)

Load Case Number Abbreviation			1 Perpendicular, Case1
Origin Action Force			
Vo Ho Mo		kN kN kNm	5683.94 0.00 0.00
Origin Displacement			
δx δz α		mm mm rad	0.00 7.63 0.00000000
δf, δa		mm	0.00 ≦ 10.00
Vertical Reaction Force			
PNmax, Ra PNmin, Pa		kN kN	631.55 ≦ 2706.00 631.55 ≧ 0.00
Horizontal Reaction Force			
PH		kN	0.00
Pile Moment			
Pile Head Mt Underground Mm		kNm kNm	0.00 0.00
Pile Body Stress			
Section 1	σ c, σ ca σt, σta τ, τa	N/mm2 N/mm2 N/mm2	-51.28 ≥ -140.00 -51.28 ≦ 140.00 0.000 ≦ 80.000
Evaluation			OK

Source: Study team

(iii) Upstream Apron Right Bank

The calculation result of the upstream apron right bank is shown in **Table7.4.85**.

Table7.4.85 Calculation Result of Pile Foundation on the Upstream Right Bank Apron (Perpendicular Direction to the Flow)

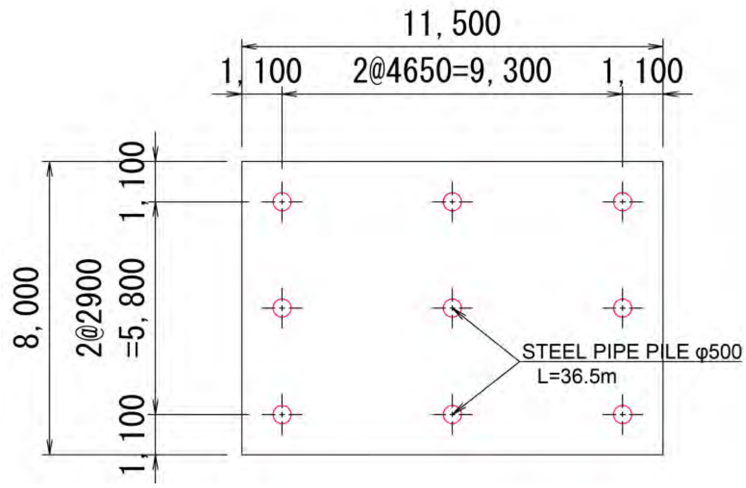
Load Case Number Abbreviation		1 Perpendicular, Case1
Origin Action Force		
Vo Ho Mo	kN kN kNm	6962.83 0.00 0.00
Origin Displacement		
δx δz α	mm mm rad	0.00 9.35 0.00000000
δf, δa	mm	0.00 ≦ 10.00
Vertical Reaction Force		
PNmax, Ra PNmin, Pa	kN kN	773.65 ≦ 2706.00 773.65 ≧ 0.00
Horizontal Reaction Force		
PH	kN	0.00
Pile Moment		
Pile Head Mt Underground Mm	kNm kNm	0.00 0.00
Pile Body Stress		

Load Case Number Abbreviation			1
Origin Action Force			Perpendicular, Case1
Section 1	σ c, σ ca	N/mm2	-62.82 ≥ -140.00
	σt, σta	N/mm2	-62.82 ≦ 140.00
	τ, τa	N/mm2	0.000 ≦ 80.000
Evaluation			OK

Source: Study team

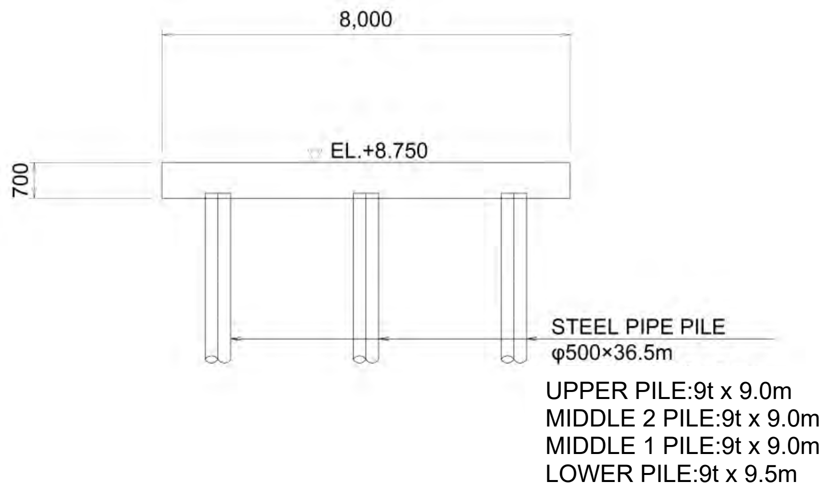
(g) Result of Examination

The results of examination on the upstream apron pile foundation are shown below. Details of the examination results are shown in **Vol.5A Structural Calculation for Contract Package-1.**



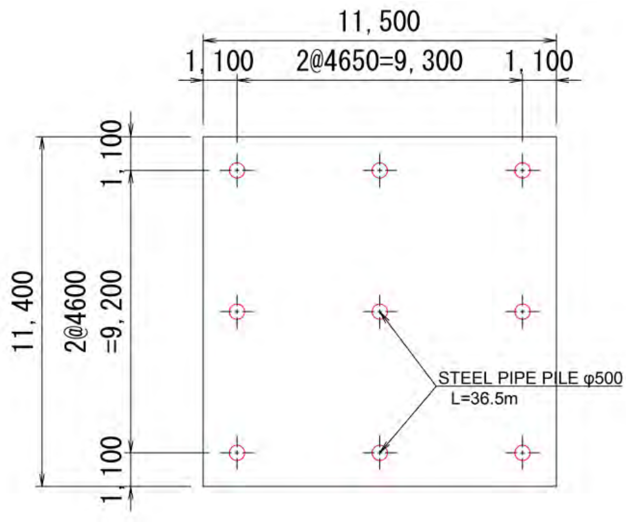
Source: Study team

Figure7.4.79 Upstream Apron Pile Arrangement (Left Bank)



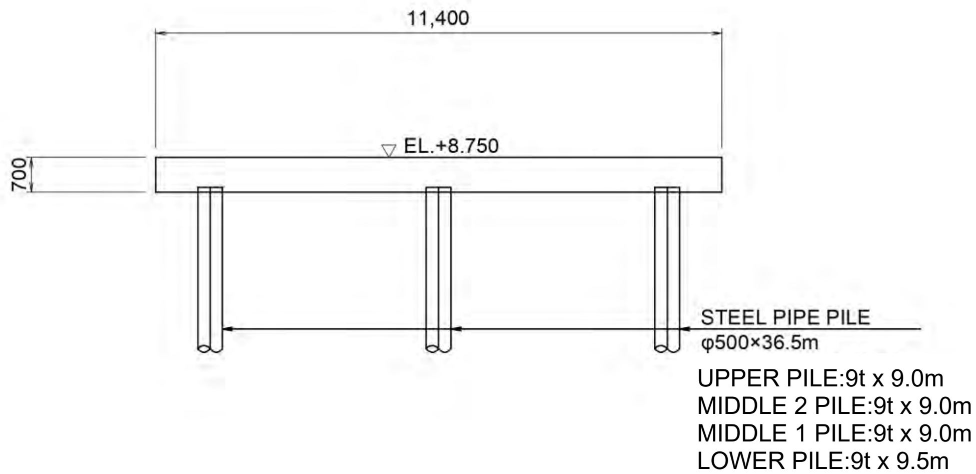
Source: Study team

Figure7.4.80 Upstream Aproned Pile Foundation Calculation Result (Left Bank)



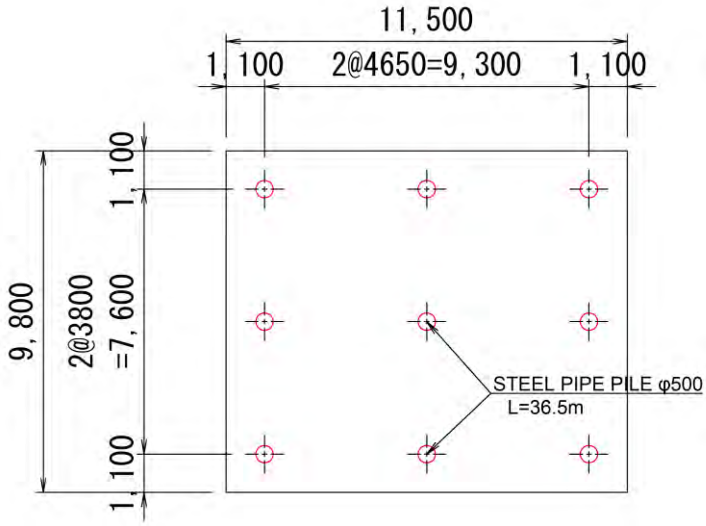
Source: Study team

Figure7.4.81 Upstream Apron Pile Arrangement (Center)



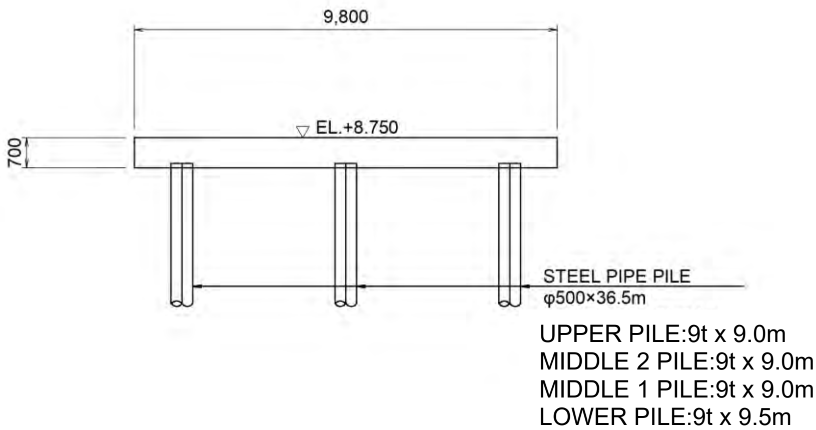
Source: Study team

Figure7.4.82 Upstream Aproned Pile Foundation Calculation Result (Center)



Source: Study team

Figure7.4.83 Upstream Apron Pile Arrangement (Right Bank)



Source: Study team

Figure7.4.84 Upstream Aproned Pile Foundation Calculation Result (Right Bank)

7.4.2.2 Main Body Work

(1) Stability Calculation

1) Study Policy

Since this facility has a pile foundation structure, its stability is examined in the examination of pile foundation. This section summarizes the load acting on the pile foundation by summing up the load such as earth and water pressure, wind load, seismic inertia force, uplift pressure, and load on the center pier and end pier in the design calculation of the pile foundation.

(a) Load Case

Load case lists for Cainta Floodgate in stability and structural calculations are shown in **Table7.4.86**.

Table7.4.86 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	Water Level Condition			Structures Subject to be Verified		
			Dead Load	Control House Load	Bridge Superstructure Load	Gate Load	Spiral Step Load	Temperature Load	Wind Load	Live Load	Earth Pressure	Upstream Water Pressure	Downstream Water Pressure	Embankment/Cover Soil Load	Upstream Water Weight	Downstream Water Weight	Uplift Pressure	Seismic Inertia Force	Hydrodynamic Pressure	Floodgate Main Body	Gate State	Additional Factor of Allowable Stress	End Pier	Center Pier	Wing Wall
Parpendicular Direction to Flow	CASE1	Normal	○	○	○	○	○	○		○	○	○		○	○	○	○			A	Open	1.00	○		○
	CASE2	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○		○	○	○	○			A	Open	1.25	○	○	
	CASE3	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○		○	○	○	○			B	Closed	1.25	○	○	○
	CASE4	Seismic	○	○	○	○	○				○	○		○	○	○	○	○	○	C	Open	1.33	○	○	○
	CASE 5	During construction (Left bank)	○	○	○	○	○		○		○									D	one side	1.50	○	○	○
	CASE 6	During construction (Right bank)	○	○	○	○	○		○		○									E	Open	1.50		○	
Flow Direction	CASE7	Normal	○	○	○	○	○	○		○	○	○	○	○	○	○	○			A	Open	1.00	○	○	
	CASE8	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○			A	Open	1.25	○	○	
	CASE9	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			B	Closed	1.25	○	○	
	CASE 10	Seismic	○	○	○	○	○				○	○	○	○	○	○	○	○	○	C	Open	1.33	○	○	
	CASE 11	During construction (Left bank)	○	○	○	○	○		○		○									D	Open	1.50	○	○	
	CASE 12	During construction (Right bank)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			F	Closed	1.25	○	○	

**Temperature loads are considered only for structural calculations.*
Source: Study team

(b) Water Level Condition

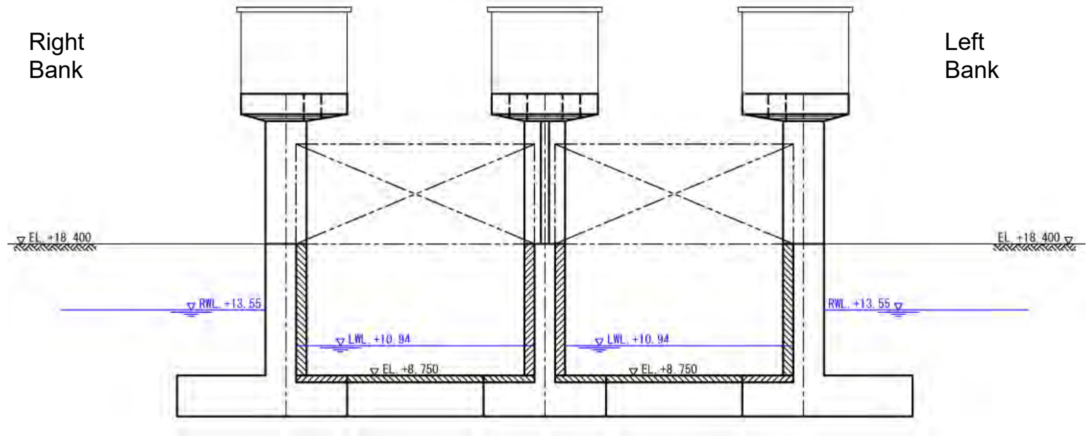
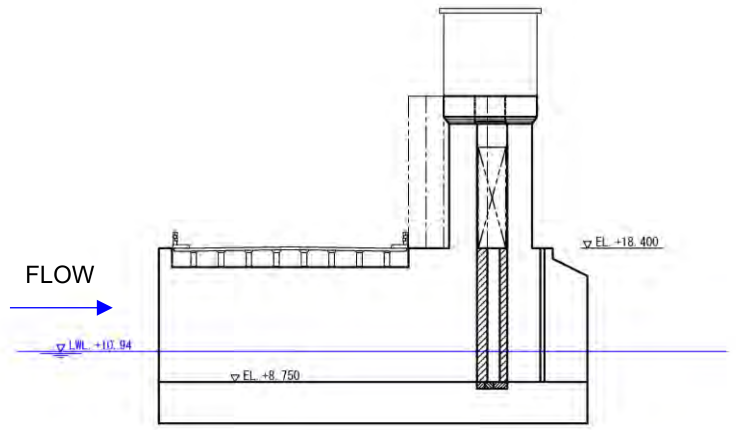
Table of design water level is shown in **Table7.4.87**. Also, a schematic diagrams of water level conditions is shown in **Table7.4.88** to **Table7.4.93**.

Table7.4.87 Design Water Table

No.	Water Level Condition	Water Level Condition (Raw Value)	Design Water Level	Grounds, Etc.
1	Floodway Side, Design Flood Level (DFL)	14.853	14.853	Calculated by interpolation from the As-built Drawing[2)]
2	Tributary Side, Design Flood Level (DFL)	13.340	13.340	Cainta River DFL [1)]
3	Design Riverbed	8.750	8.750	Cainta River STA. 0 + 000, Design Riverbed [1)]
4	Floodway Side, OWL	11.298	11.30	OWL of Manggahan Floodway
5	Floodway Side, Low Water Level (LWL)	10.942	10.94	LWL of Manggahan Floodway
6	Target Water Level for Cofferdam During Construction	14.40	14.45	Highest water level of Lake Laguna in the last 5 years (2014 ~ 2018) + 5 cm, considering the rise in water level due to cofferdam

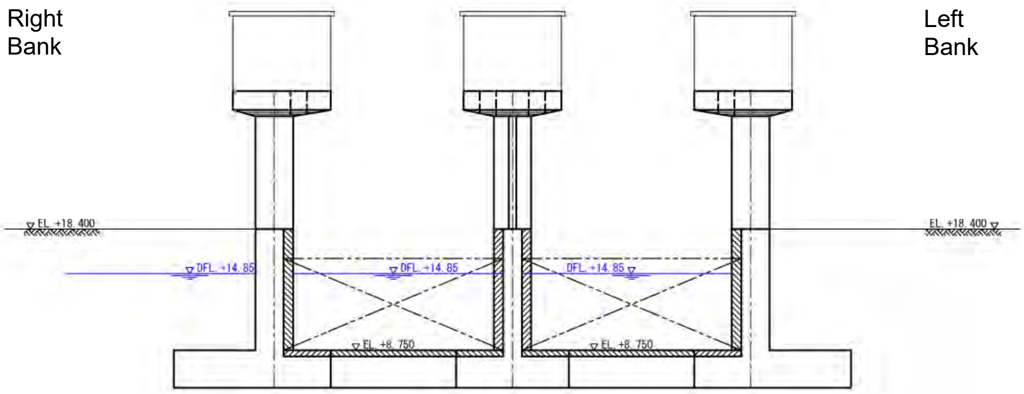
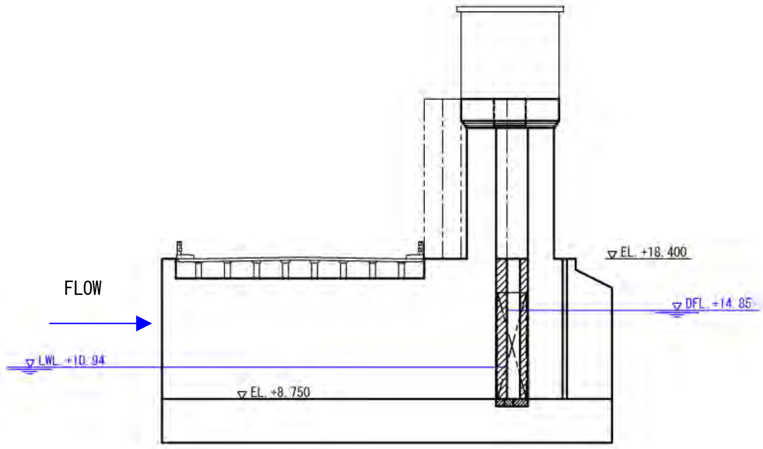
Source: 1) 2008 Pre-F/S
2) Final Report on Consulting Services for Manggahan Floodway Project

Table7.4.88 Water Level Condition A (Normal Condition)

<table>
<tr><th>Calculations Direction</th><th>Schematic Diagram of Water Level Condition</th></tr>
<tr><td>Perpendicular Direction to the Flow</td><td>
</td></tr>
<tr><td>Flow Direction</td><td>
</td></tr>
</table>

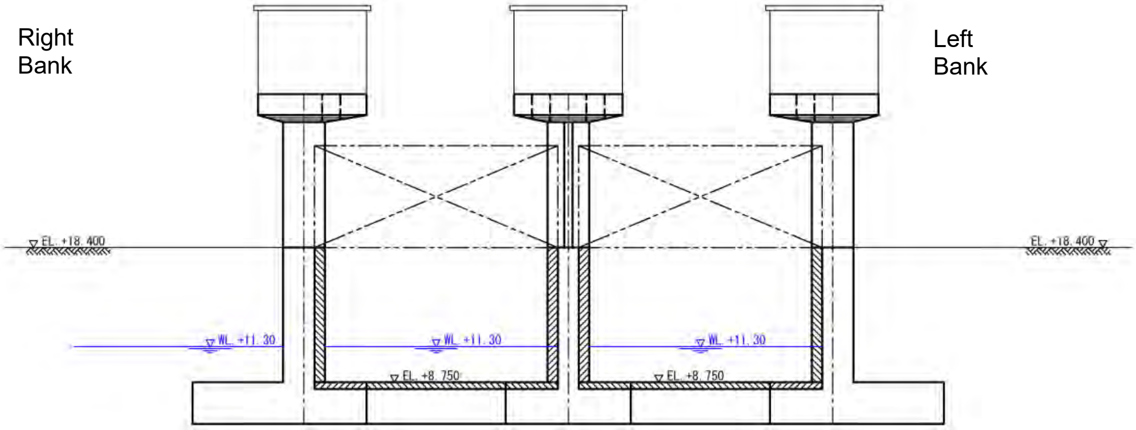
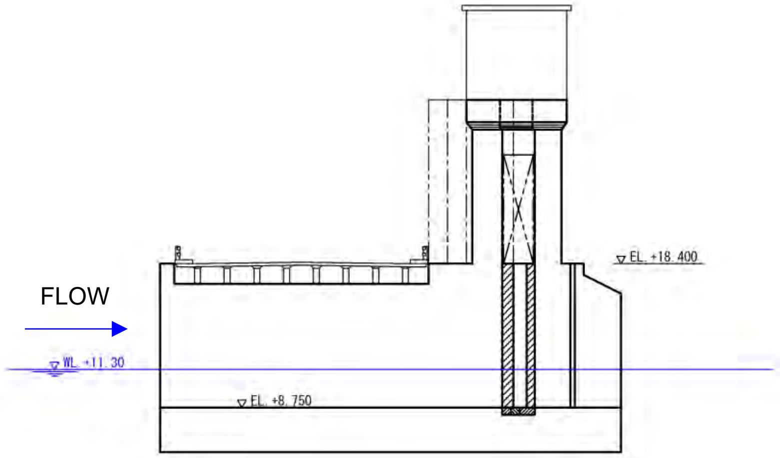
Source: Study team

Table7.4.89 Water Level Condition B (at DFL, Manggahan Floodway)

Calculations Direction	Schematic Diagram of Water Level Condition
Perpendicular Direction to the Flow	
Flow Direction	

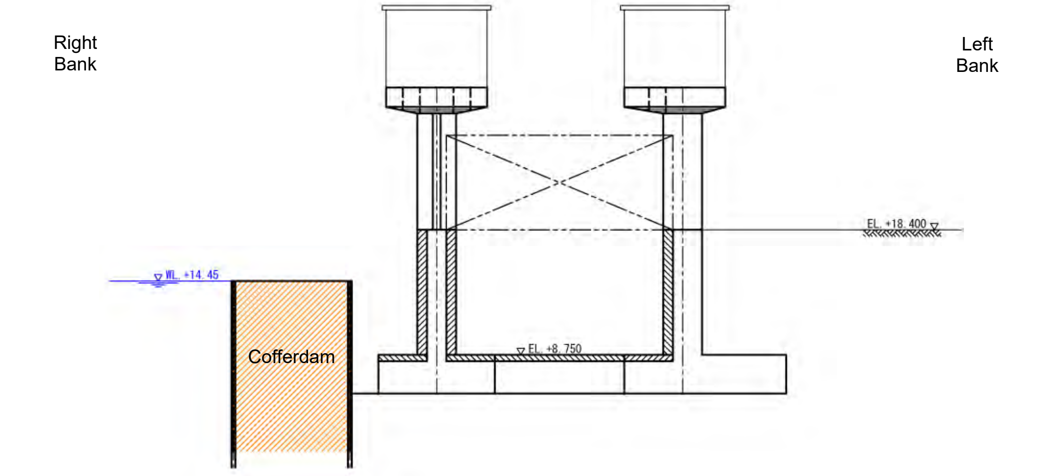
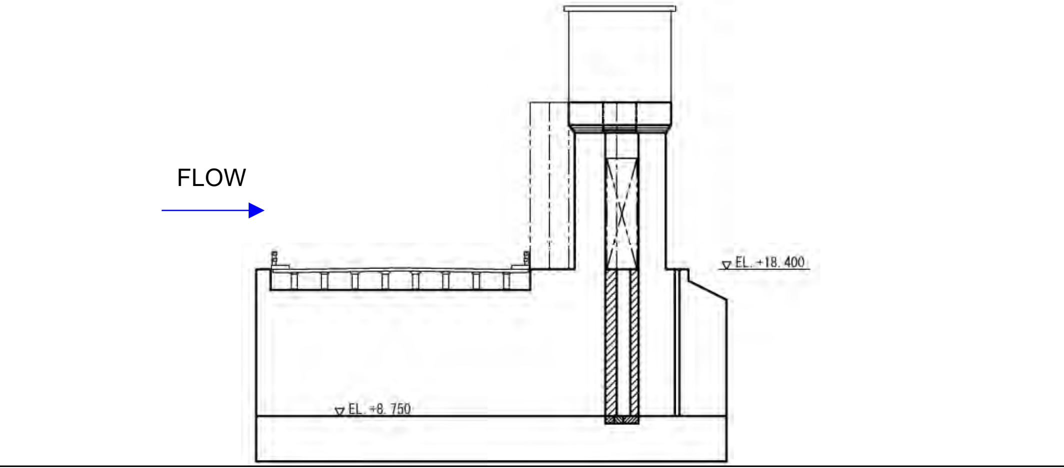
Source: Study team

Table7.4.90 Water Level Condition C (Seismic Condition)

Calculations Direction	Schematic Diagram of Water Level Condition
Perpendicular Direction to the Flow	
Flow Direction	

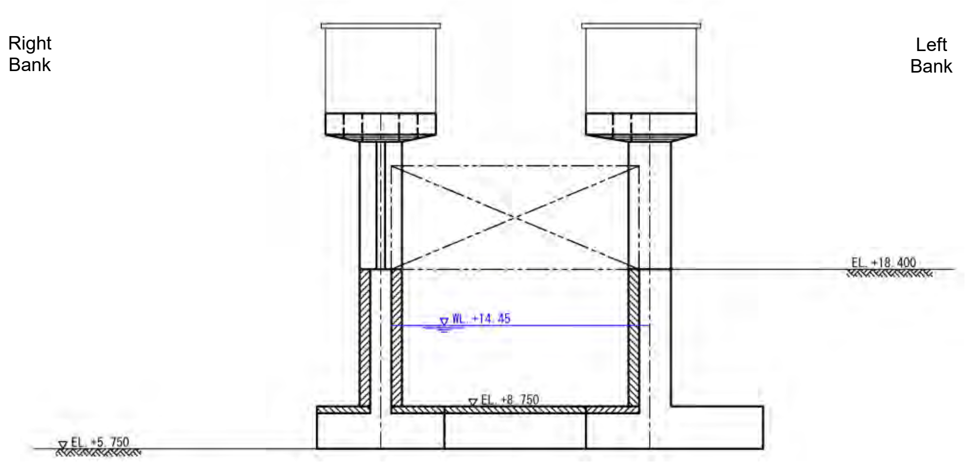
Source: Study team

Table7.4.91 Water Level Condition D (left bank construction)

Calculations Direction	Schematic Diagram of Water Level Condition
Perpendicular Direction to the Flow	
Flow Direction	

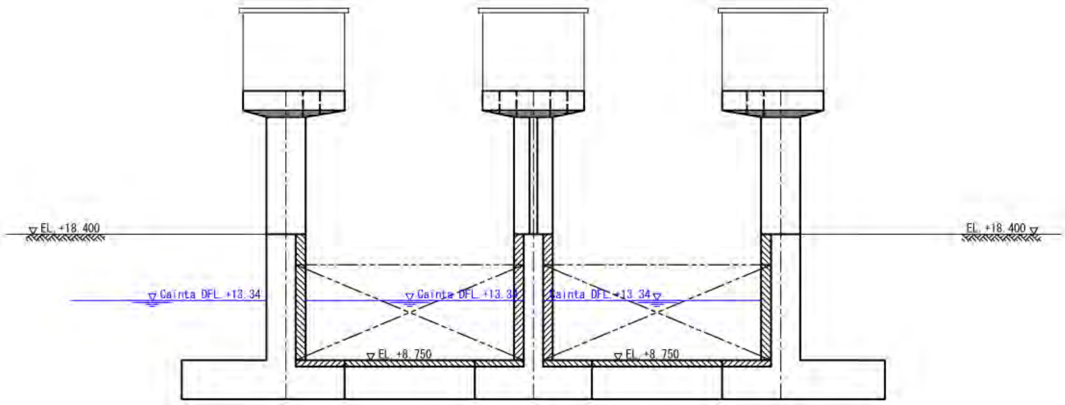
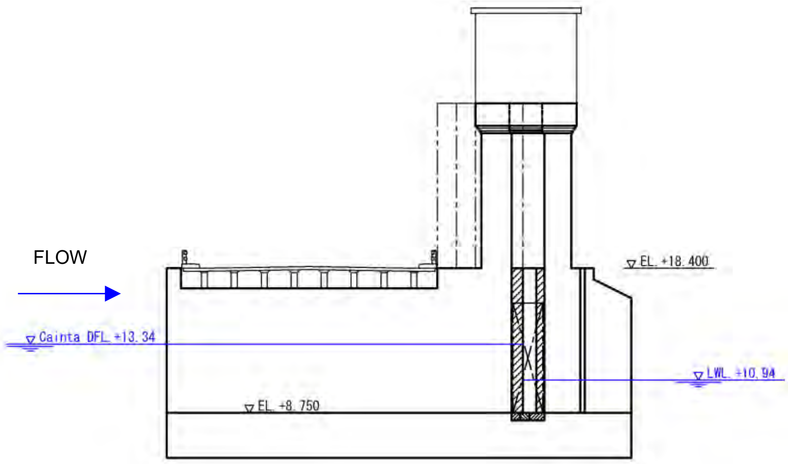
Source: Study team

Table7.4.92 Water level Condition E (During Construction on the Right Bank)

Calculations Direction	Schematic Diagram of Water Level Condition
Perpendicular Direction to the Flow	
Flow Direction	

Source: Study team

Table7.4.93 Water Level Condition F (at DFL, Cainta River)

Calculations Direction	Schematic Diagram of Water Level Condition
Perpendicular Direction to the Flow	
Flow Direction	

Source: Study team

2) Center Pier

For the stability calculation in the center pier, the details of the calculation such as calculation model and load conditions are indicated in **Vol.5A Structural Calculation for Contract Package-1**.

(a) Stability Calculation Result

A list of stability calculation results of the center pier is shown in **Table7.4.94** and **Table7.4.95**.

Table7.4.94 Results of Calculation in Stability Analysis of Center pier (Perpendicular Direction to the Flow)

Case	Load name	V (kN)	H ((kN))	M ((kN)-m)
Case 1	Normal	-	-	-
Case 2	Normal + wind load	38708.59	1176.54	15096.35
Case 3	During floods (at Floodway DFL)	37748.70	767.02	12834.56
Case 4	Seismic	36368.68	9456.44	63645.19
Case 5	During construction (left bank)	44040.92	-1504.33	-19818.41
Case 6	During construction (right bank)	39832.13	11865.94	46205.27

Source: Study team

Table7.4.95 Results of Calculation in Stability Analysis of Center Pier (Flow Direction)

Case	Load name		V (kN)	H (kN)	M (kNm)
Case 7	Normal condition		38708.59	0.00	-20902.64
Case 8 -1	Normal condition + wind load	(wind direction: upstream to downstream)	38708.59	1071.93	-40256.93
Case 8 -2	Normal condition + wind load	(wind direction: downstream to upstream)	38708.59	-1088.80	-1548.34
Case 9 -1	During floods (at Floodway DFL)	(Wind direction: upstream to downstream)	43515.20	-3126.59	-27414.58
Case 9 -2	During floods (at Floodway DFL)	(Wind direction: downstream to upstream)	43515.20	-4685.31	-1305.46
Case 10 -1	Seismic condition	(Inertial forces: upstream to downstream)	36368.68	9328.48	-94194.88
Case 10 -2	Seismic condition	(Inertia: downstream to upstream)	36368.68	-9328.48	34913.93
Case 11 -1	During construction	(Wind direction: upstream to downstream)	44040.92	1099.32	-58574.42
Case 11 -2	During construction	(Wind direction: downstream to upstream)	44040.92	-1116.20	-19378.00
Case 12 -1	During floods (at Tributary DFL)	(Wind direction: upstream to downstream)	46537.22	2885.57	-30249.19
Case 12 -2	During floods (at Tributary DFL)	(Wind direction: downstream to upstream)	46537.22	1207.22	-2792.23

Source: Study team

3) End Pier

For the stability calculation in the end pier, the details of the calculation such as calculation model and load conditions are indicated in **Vol.5A Structural Calculation for Contract Package-1**.

(a) Stability calculation result

A list of stability calculation results of the end pier is shown in **Table7.4.96** and **Table7.4.97**.

Table7.4.96 Calculation Results of Stability Analysis of End Pier (Perpendicular Direction to the Flow)

Case	Load name	V (kN)	H (kN)	M (kNm)
Case 1	Normal condition	79712.98	16146.08	6377.04
Case 2	Normal condition + wind load	79712.98	16591.55	-3188.52
Case 3	During floods (at Floodway DFL)	75951.57	20306.81	-18987.89
Case 4	Seismic condition	81001.29	38617.36	-136892.18
Case 5	Left bank construction	101565.53	13574.89	36563.59
Case 6	During construction on the right bank	-	-	-

Source: Study team

Table7.4.97 Calculation Results of Stability Analysis of End Pier (Flow Direction)

Case	Load name		V (kN)	H (kN)	M (kNm)
Case 7	Normal condition		73345.12	0.00	-27137.69
Case 8 -1	Normal condition + wind load	(Wind direction: upstream to downstream)	73345.12	701.63	-41806.72
Case 8 -2	Normal condition + wind load	(Wind direction: downstream to upstream)	73345.12	-734.78	-12468.67
Case 9 -1	During floods (at Floodway DFL)	(Wind direction: upstream to downstream)	68062.99	-986.95	-20418.90
Case 9 -2	During floods (at Floodway DFL)	(Wind direction: downstream to upstream)	68062.99	-2146.81	2041.89
Case 10 -1	Seismic condition	(Inertial forces: upstream to downstream)	75698.79	18890.17	-158967.46
Case 10 -2	Seismic condition	(Inertia: downstream to upstream)	75698.79	-18890.17	98408.43
Case 11 -1	During construction	(Wind direction: upstream to downstream)	93985.25	701.63	-45112.92
Case 11 -2	During construction	(Wind direction: downstream to upstream)	93985.25	-734.78	-15977.49

Case	Load name		V (kN)	H (kN)	M (kNm)
Case 12 -1	During floods (at Tributary DFL)	(Wind direction: upstream to downstream)	72822.09	1398.03	-44421.47
Case 12 -2	During floods (at Tributary DFL)	(Wind direction: downstream to upstream)	72822.09	187.80	-21118.41

Source: Study team

(2) Structural Calculation

1) Design Condition

The main design conditions are shown in **Table7.4.98**. The details will be provided separately.

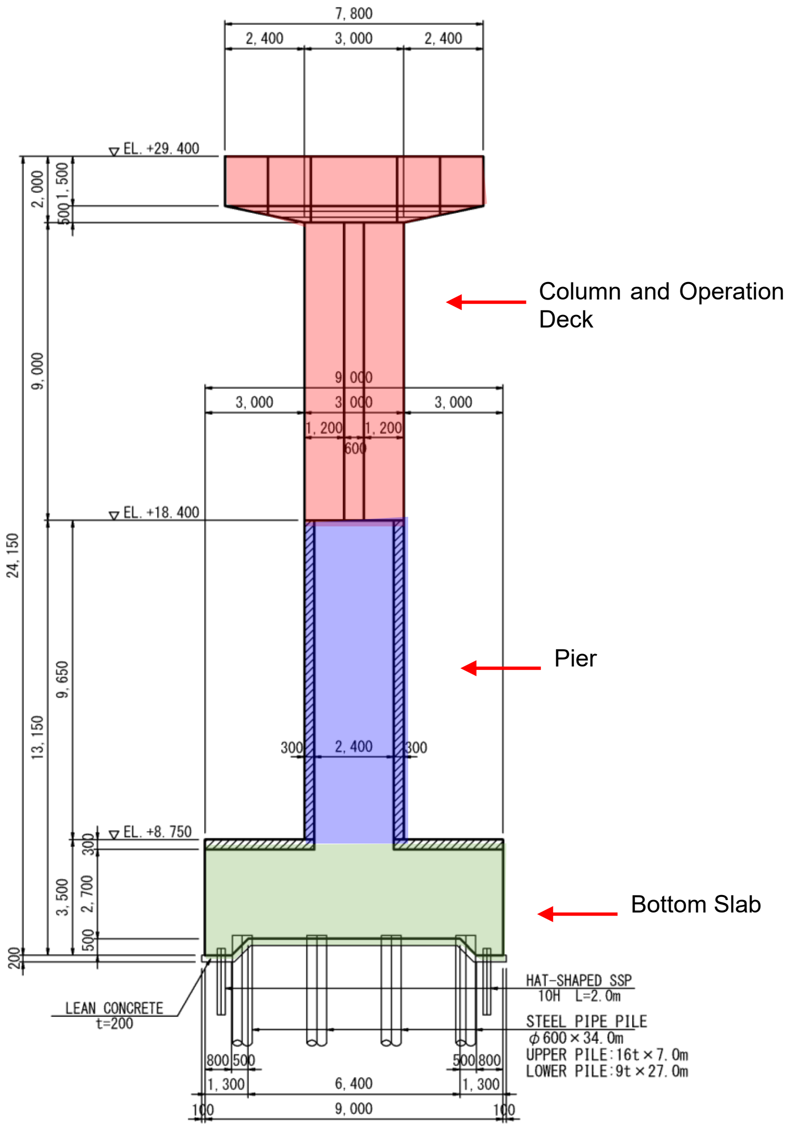
Table7.4.98 List of Cainta Floodgate design conditions

Item	List of Conditions		Reason for Establishment and Remarks
Standard	・ DGCS 2015 ・ Cabinet Order Concerning Structural Standards for River Administration Facilities, etc. ・ Technical Criteria for River Works: Practical Guide for Planning [I] ・ Design for Weirs (Japan Dam Engineering Center) ・ Guideline for Flexible Sluiceway ・ Specifications for Highway Bridges IV Substructures		The basic shape of the floodgate follows 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	Applying material properties in the Philippines
	Young's Modulus Ratio	n = 9	Same as above
	Linear Expansion Coefficient	10.8 x 10 -6	Same as above
Allowable Stress	Concrete	fc = 8.28 N/mm2 τ a = 0.36 N/mm2	Same as above
	Rebar	σ c = 168 N/mm2	Same as above
	Extra Factor	Wind Load 25% Temperature Change 25% Seismic Condition33% During construction50%	According to the setting method in the Philippines 40% premium for wind load + temperature change
Minimum Reinforcement	Member Receiving Bending	Mu ≥ Mb Mb ≥ 1.7 Mc	Specifications for Highway Bridges IV Substructures, 7.3
	Member Whose Axial Force Is Dominant	As ≧ 0.008 A′	Specifications for Highway Bridges IV Substructures, 7.3
	Crack Prevention	500 mm2 or more per 1 m of concrete surface with 300 mm spacing	Specifications for Highway Bridges IV Substructures, 7.3 (Required reinforcement in case of the Philippines to satisfy 500 mm2 or more per 1 meter is D 16 @ 250 or more.)
Reinforcement Specifications	Based on the arrangement that main bars are Outside, distributing bars are inside, it will be set for each part		General policy. In case of the column member such as column, the bar arrangement is reversed to the left, because the bar arrangement was made to be the restraint bar of earthquake resistance.

Source: Study team

2) Center Pier

The structural drawing of the center pier is shown in **Figure7.4.85**. Structural calculations of the center pier are carried out according to the classification shown in **Figure7.4.85**. The details of the structural calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure7.4.85 Structural Diagram of Center Pier

(a) Design of the Bottom slab

(i) Design Policy

Design the bottom slab follows "8.7 Design of Footing in Specifications for Highway Bridges IV Substructures ". The design of the bottom slab shall be carried out in the Perpendicular Direction to the Flow.

(ii) Load Condition

As for the load condition of the bottom slab, the load condition used for the design of the pile foundation is applied, and the calculated pile reaction force is applied to the bottom slab. Aside from pile reaction forces, the water weight and uplift shown in **Table7.4.99** are applied.

Table7.4.99 Center Pier Slab Arbitrary Load

Arbitrary Load in the Direction Perpendicular to Flow				L= 1.00			L= 31.00		
Number	Load Case	Category	Water Depth	Water Weight (kN/m)	Uplift Force (kN/m)		Water Weight (kN/m)	Uplift Force (kN/m)	
					Left	Right		Left	Right
Case 1	Normal Condition	Entire Area	2.190	21.5	50.9	50.9	666.5	1577.9	1577.9
Case 2	Normal + Wind Load	Entire Area	2.190	21.5	50.9	50.9	666.5	1577.9	1577.9
Case 3	During Flood (DFL in Floodway)	Upstream Side	6.100	59.8	89.2	89.2	1853.8	2765.2	2765.2
		Downstream Side	2.190	21.5	50.9	50.9	666.5	1577.9	1577.9
Case 4	Seismic Condition	Entire Area	2.550	25.0	54.4	54.4	775.0	1686.4	1686.4
Case 5	During Construction in Left Bank Side	Entire Area	0.000	0.0	0.0	0.0	0.0	0.0	0.0
Case 6	During Construction in Right Bank Side	Entire Area	5.700	55.9	85.3	0.0	1732.9	2644.3	0.0

Source: Study team

(iii) Calculation Result

The calculation result of the center pier base slab is shown in **Table7.4.100** to **Table7.4.102**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.100 Results of Bending Stress Check for Center Pier Slab

Units: M (kN m), σ (N/mm2)

Case	Left side			Right side			Allowable stress	
	M	σ c	σs	M	σc	σs	σ ca	σ sa
1	12148.57	0.98	73.91	12148.57	0.98	73.91	8.28	168.00
2	7914.53	0.64	48.15	16382.61	1.33	99.67	8.28	168.00
3	4093.52	0.33	24.90	11101.11	0.90	67.54	8.28	168.00
4	-8092.69	0.93	114.12	30196.16	2.44	183.71	11.01	223.44
5	15957.92	1.29	97.08	5135.64	0.42	31.24	12.42	252.00
6	-5088.38	0.58	71.75	26337.59	2.13	160.23	12.42	252.00

Source: Study team

Table7.4.101 Results of Shearing Stress Check for Center Pier Slab (Left Overhang)

Case	S (kN)	τm (N/mm2)	τa (N/mm2)	τa2 (N/mm2)	Sca (kN)	Cdc	Cds	a (mm)	Awreq (mm^2/m)
1	8222.29	0.095	0.360	1.600					0
2	5964.13	0.069	0.360	1.600					0
3	5088.30	0.059	0.360	1.600					0
4	-2573.05	0.029	0.470	2.128					0
5	11415.98	0.132	0.540	2.400					0
6	-1723.91	0.019	0.540	2.400					0

Source: Study team

Table7.4.102 Results of Shearing Stress Check For Center Pier Slab (right overhang)

Case	S (kN)	τm (N/mm2)	τa (N/mm2)	τa2 (N/mm2)	Sca (kN)	Cdc	Cds	a (mm)	Awreq (mm2/m)
1	8222.29	0.095	0.360	1.600					0
2	10480.44	0.122	0.360	1.600					0
3	8825.68	0.102	0.360	1.600					0
4	17847.67	0.207	0.470	2.128					0
5	5644.10	0.065	0.540	2.400					0
6	16432.58	0.191	0.540	2.400					0

Source: Study team

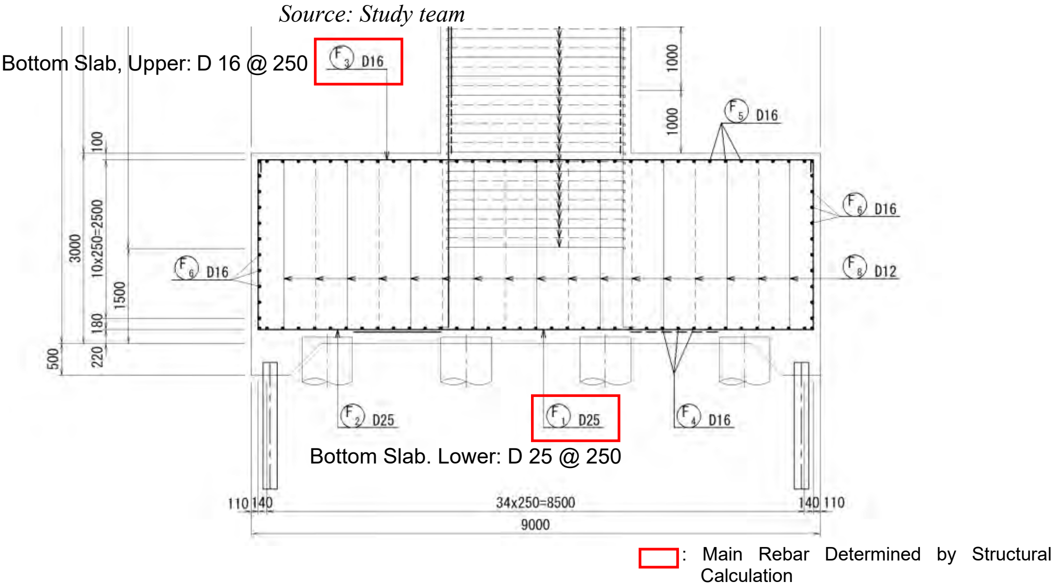
(iv) Result of Examination

The structural calculation result of the center pier base slab is shown below.

Table7.4.103 Calculation Result of Center Pier Structure

Perpendicular Direction to flow	Reinforcement	Cover
Lower Side of Bottom Slab	D25@250	220 mm
Upper Side of Bottom Slab	D16@250	100 mm
Flow Direction		
Both Side Distribution Bar	D16@250	

Source: Study team



Source: Study team

Figure7.4.86 Bar Arrangement of the Center Pier Slab

(b) Design of the Piers

(i) Design Policy

In accordance with " Technical Criteria for River Works: Practical Guide for Planning [I] ", structural calculation is performed for the pier as a cantilever fixed to the floor slab. Since the pier has a sufficient length in the flow direction, the check in the flow direction is omitted and the check is carried out in the direction perpendicular to the flow direction.

(ii) Load Condition

The load condition of the pier is the same as that of the load case in the stability calculation. The table below shows the cross sectional force generated at the base of the pier based on the load calculation of the stability calculation.

Table7.4.104 Cross Sectional Force at Base of Center Pier (Perpendicular Direction to the Flow)

Number	load name	V (kN)	H (kN)	M (kNm)
Case 1	Normal Condition	-	-	-
Case 2	Normal Condition + Wind Load	28838.69	1197.31	11823.86
Case 3	During floods (at Floodway DFL)	28458.69	787.78	13091.00
Case 4	Seismic Condition	26775.44	5567.35	43108.46
Case 5	left bank construction	24285.44	-1525.10	-19914.06
Case 6	During construction on the right bank	26775.44	5324.62	21152.60

Source: Study team

(iii) Calculation Result

The calculation result of the center pier is shown in **Table7.4.105**. Of the calculation results, the case surrounded by the red line is the decision case.

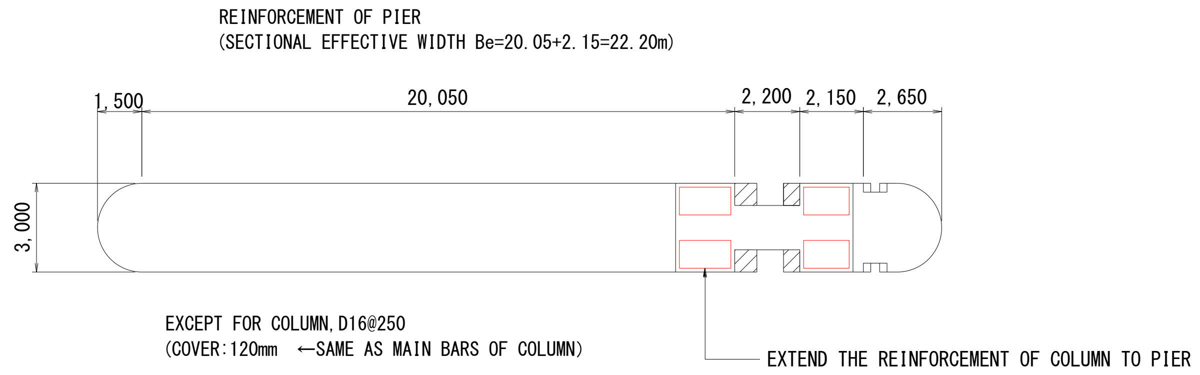
Table7.4.105 List of Calculation Results of Center Pier

Section No.		Center Pier				
		Weir Main Body				
State		Normal + Wind Load	During Floods (Floodway)	Seismic Condition	During Construction in Left Bank Side	During Construction in Right Bank Side
Dimensions						
b	cm	2220.0	2220.0	2220.0	2220.0	2220.0
h	cm	300.0	300.0	300.0	300.0	300.0
d	cm	288.0	288.0	288.0	288.0	288.0
d '	cm	12.0	12.0	12.0	12.0	12.0
Cross-sectional Force						
M	kN m	11,823.86	13,091.00	43,108.46	19,914.06	21,152.60
N	kN	28,838.69	28,458.69	26,775.44	24,285.44	26,775.44
S	kN	1,197.31	787.78	5,567.35	1,525.10	5,324.62
Reinforcement						
As	cm^2	282.780	282.780	282.780	282.780	282.780
As '	cm^2	282.780	282.780	282.780	282.780	282.780
n	-	9	9	9	9	9
Calculated Value						
e	cm	41.0	46.0	161.0	82.0	79.0
e-h/2	cm	-109.0	-104.0	11.0	-68.0	-71.0
A (x^3)	-	1	1	1	1	1
B (x^2)	-	-327.0000302	-311.9999726	33.00001793	-204.0000099	-212.9999731
C (x)	-	564.0313211	632.8159095	2214.855325	1128.062874	1086.792448
D (-)	-	-346590.4327	-356908.1209	-594214.0333	-431195.1655	-425004.6016
x	cm	328.495	313.611	66.846	208.508	217.016
G	cm^3	120687497.9	110003350.4	4536653.2	48555708.6	52617612.2
I	cm^4	26490296807.9	23057914225.0	353163695.3	6822464425.0	7683019170.3
Sectionals stress						
σc	N/mm^2	0.78	0.81	3.95	1.04	1.10
$\sigma s'$	N/mm^2	6.81	7.02	29.13	8.85	9.39
σs	N/mm^2	-0.87	-0.60	117.46	3.58	3.25
τm	N/mm^2	0.02	0.01	0.09	0.02	0.08
Allowance						
σca	N/mm^2	8.28	8.28	11.01	12.42	12.42
σsa	N/mm^2	168.00	168.00	223.44	252.00	252.00
τa	N/mm^2	0.36	0.36	0.48	0.54	0.54
σck	N/mm^2	20.70	20.70	20.70	20.70	20.70
σsy	N/mm^2	415.00	415.00	415.00	415.00	415.00
Mc	kN・m	72160.52	71970.52	71128.90	69883.90	71128.90
Mu	kN・m	74948.61	74417.90	72062.60	68565.16	72062.60
Evaluation	Minimum Reinforceme	Mb < Mu (Ok)	Mb < Mu (Ok)	Mb < Mu (Ok)	1.7 Mc < Mb (Ok)	Mb < Mu (Ok)
	Stress	OK	OK	OK	OK	OK
	Shear	OK	OK	OK	OK	OK
Bar Arrangement	Reinforcement	90 - D 20	90 - D 20	90 - D 20	90 - D 20	90 - D 20

Source: Study team

(iv) Result of Examination

The effective member width of the center pier is considered to be the length L = 22.20 m excluding the door contact part and the circular part. The structural calculation result of the pier section is shown below.



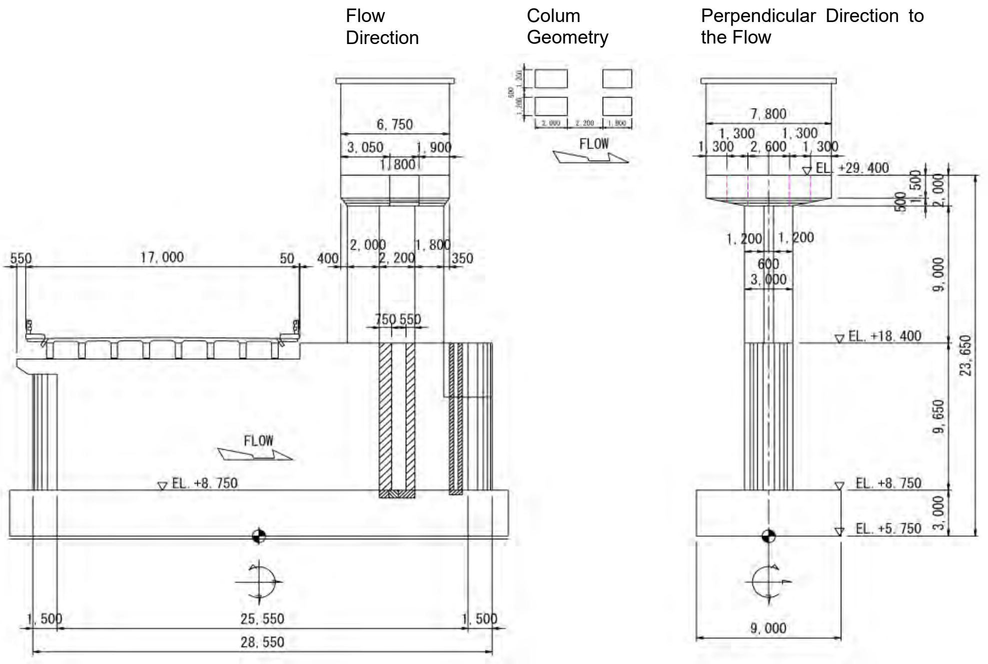
Source: Study team

Figure7.4.87 Bar Arrangement of Center Piers and Piers

(c) Design of Columns and Operation decks

(i) Design Policy

The structural dimensions of the column of the center pier are shown in **Figure7.4.86**. In consideration of access from the maintenance bridge side during gate maintenance work and gate inspection work, this floodgate has four columns at 1 location. In the structural calculation of the column, the structural calculation is carried out as a portal frame fixed to the pier both in the flow direction and in the perpendicular direction to the flow according to "Technical Criteria for River Works: Practical Guide for Planning [I]".



Source: Study team

Figure7.4.88 Dimension of Center Pier Structure

(ii) Study Case

A list of cases to be examined in the perpendicular direction to the flow and in the flow direction is shown in **Table7.4.106** and **Table7.4.107**.

Table7.4.106 Center Column Load Case (Perpendicular Direction to the Flow)

Calculation Direction	Study Case	Load Condition	Load Term								Extra Factor
			Body Weight	Equipment Load	Local Control House Load	Temperature Load	Wind Load	Operational Load	Inertial Force Right	Inertia Force Left	
Perpendicular Direction To The Flow	1	Normal Condition	○	○	○			○			1.00
	2	Normal Condition (Temperature Rise + 16.7 °)	○	○	○	○		○			1.25
	3	Normal Condition (Temperature Drop -22.2 °)	○	○	○	○		○			1.25
	4	Wind Load	○	○	○		○				1.25
	5	Wind Load + Temperature Rise	○	○	○	○	○				1.40
	6	Wind Load + Temperature Drop	○	○	○	○	○				1.40
	7	Seismic Condition	○	○	○				○		1.33

Source: Study team

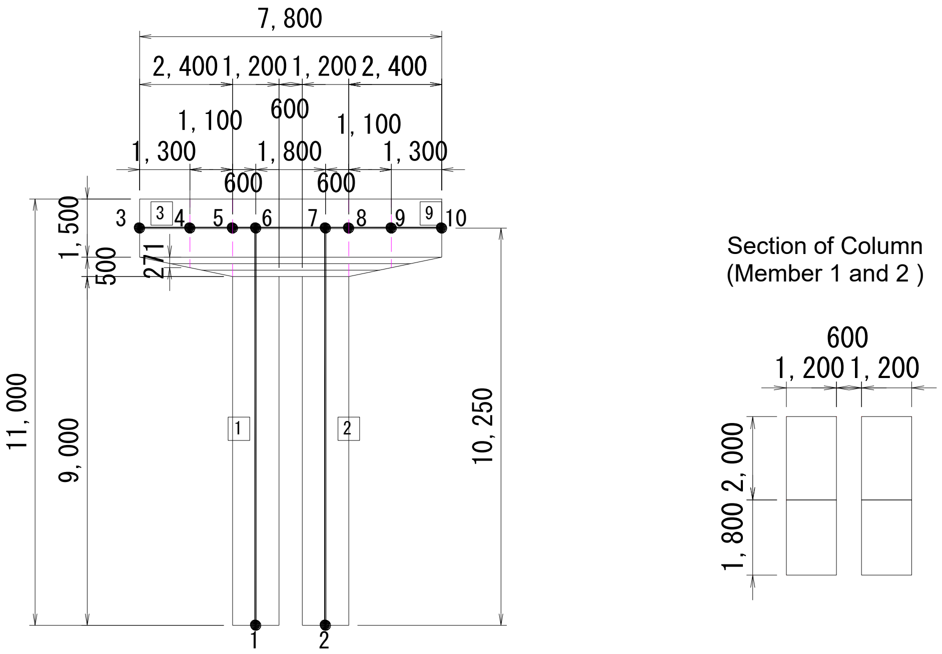
Table7.4.107 Center Column Load Case (Flow Direction)

Calculation Direction	Study Case	Load Condition	Load Term								Extra Factor
			Body Weight	Equipment Load	Local Control House Load	Temperature Load	Wind Load	Operational Load	Inertial Force Right	Inertia Force Left	
Flow Direction	1	Normal Condition	○	○	○			○			1.00
	2	Normal Condition (Temperature rise + 16.7 °)	○	○	○	○		○			1.25
	3	Normal Condition (Temperature drop -22.2 °)	○	○	○	○		○			1.25
	4	wind load	○	○	○		○				1.25
	5	Wind load + Temperature rise	○	○	○	○	○				1.40
	6	Wind load + Temperature drop	○	○	○	○	○				1.40
	7	Seismic Condition (Right and downstream directions)	○	○	○				○		1.33
	8	Seismic Condition (Left and upstream direction)	○	○	○					○	1.33

Source: Study team

(iii) Study Model

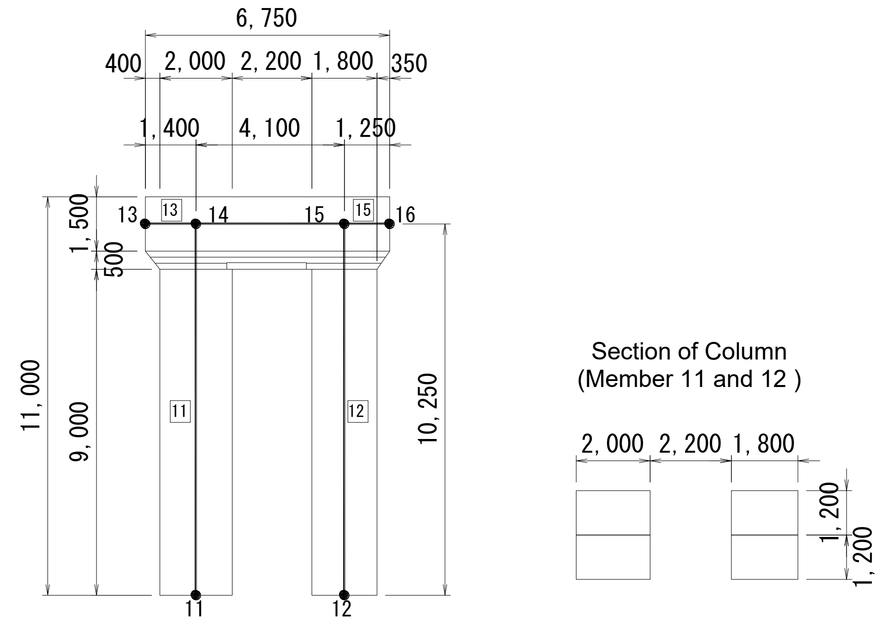
A model for perpendicular direction to the flow is shown in **Figure7.4.89**. The cross section of the column is calculated with the combined cross section of the upstream and downstream columns.



Source: Study team

Figure7.4.89 Center Pier Column Examination Model (Perpendicular Direction to the Flow)

A model for flow direction is shown in **Figure7.4.90**. The cross section of the column section is calculated as the combined cross section of the columns in the Perpendicular Direction to the Flow.



Source: Study team

Figure7.4.90 Center Pier Column Examination Model (Flow Direction)

(iv) Calculation Result

The calculation results for each member in the most severe load case are shown. Of the calculation results, the case surrounded by the red line is the decision case.

A. Column Flow Direction

The calculation results of in the column of the center pier in the flow direction are shown in **Table7.4.108** and **Table7.4.109**.

Table7.4.108 Results of Checking the Bending Stress of Center Pier Column (Flow Direction)

Seismic Condition(Right)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 11	5.25 < 11.01 OK	163.48 < 223.44 OK	33.78 < 223.44 OK
X = 0.000	3.38 < 11.01 OK	135.38 < 223.44 OK	21.72 < 223.44 OK
X = 5.125	0.66 < 6.21 OK	0.00 < 168.00 OK	5.90 < 168.00 OK
X = 10.250	5.25 < 11.01 OK	163.48 < 223.44 OK	33.78 < 223.44 OK
Seismic Condition(Left)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 12	4.60 < 11.01 OK	122.23 < 223.44 OK	30.80 < 223.44 OK
X = 0.000	3.20 < 11.01 OK	114.28 < 223.44 OK	20.30 < 223.44 OK
X = 5.125	0.89 < 7.76 OK	0.00 < 168.00 OK	6.29 < 168.00 OK
X = 10.250	4.60 < 11.01 OK	122.23 < 223.44 OK	30.80 < 223.44 OK

Source: Study team

Table7.4.109 Result of Shear Stress Check for Center Pier Column (Flow Direction)

Normal Condition (wind load)	τm (N/mm2)	Normal Condition (Wind + Temperature rise)	τm (N/mm2)
Ungrouped Member 11	0.20 < 0.48 OK (yp)	Ungrouped Member 12	0.22 < 0.50 OK (yp)
X = 0.000	0.11 < 0.48 OK (yp)	X = 0.000	0.10 < 0.48 OK (yp)
X = 5.125	0.15 < 0.48 OK (yp)	X = 5.125	0.15 < 0.48 OK (yp)
X = 10.250	0.20 < 0.48 OK (yp)	X = 10.250	0.22 < 0.50 OK (yp)

Source: Study team

B. Column in Perpendicular direction to the flow

Calculation results of the column of the center pier in the perpendicular direction to the flow at is shown in **Table7.4.110** and **Table7.4.111**.

Table7.4.110 Results of Checking the Bending Stress of Center Pier Column (Perpendicular Direction to the Flow)

Seismic Condition(Right)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 1	6.39 < 11.01 OK	210.66 < 223.44 OK	27.78 < 223.44 OK
X = 0.000	4.12 < 11.01 OK	176.35 < 223.44 OK	13.38 < 223.44 OK
X = 5.125	0.61 < 6.21 OK	9.23 < 223.44 OK	5.49 < 168.00 OK
X = 10.250	6.39 < 11.01 OK	210.66 < 223.44 OK	27.78 < 223.44 OK
Seismic Condition(Left)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 2	5.67 < 11.01 OK	59.71 < 223.44 OK	38.71 < 223.44 OK
X = 0.000	2.80 < 11.01 OK	11.68 < 223.44 OK	21.09 < 223.44 OK
X = 5.125	1.13 < 8.26 OK	0.00 < 168.00 OK	9.84 < 223.44 OK
X = 10.250	5.67 < 11.01 OK	59.71 < 223.44 OK	38.71 < 223.44 OK

Source: Study team

Table7.4.111 Result of Shear Stress Check For Center Pier Column (Perpendicular Direction to the Flow)

Seismic Condition(Right)	τm (N/mm2)	Seismic Condition(Right)	τm (N/mm2)
Ungrouped Member 1	0.43 < 0.48 OK (yp)	Ungrouped Member 2	0.48 < 0.48 OK (yp)
X = 0.000	0.26 < 0.48 OK (yp)	X = 0.000	0.21 < 0.48 OK (yp)

Seismic Condition(Right)	τm (N/mm2)	Seismic Condition(Right)	τm (N/mm2)
X = 5.125	0.34 < 0.48 OK (yp)	X = 5.125	0.32 < 0.48 OK (yp)
X = 10.250	0.43 < 0.48 OK (yp)	X = 10.250	0.48 < 0.48 OK (yp)

Source: Study team

C. Operation Deck in Flow direction

Calculation results of the operation deck of the center pier in the Perpendicular direction to the flow are shown in **Table7.4.112** and **Table7.4.113**.

Table7.4.112 Results of Checking the Bending Stress of Center Pier Operation Deck (Flow Direction)

Seismic Condition(Left)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 14	3.83 < 11.01 OK	208.75 < 223.44 OK	25.64 < 223.44 OK
X = 0.000	3.83 < 11.01 OK	208.75 < 223.44 OK	25.64 < 223.44 OK
X = 4.100	3.08 < 11.01 OK	172.94 < 223.44 OK	20.46 < 223.44 OK

Source: Study team

Table7.4.113 Result of Shearing Stress Check on Center Pier Operation Deck (Flow Direction)

Seismic Condition(Right)	τm (N/mm2)
Ungrouped Member 14	0.41 < 0.48 OK (yp)
X = 0.000	0.40 < 0.48 OK (yp)
X = 4.100	0.41 < 0.48 OK (yp)

Source: Study team

D. Operation Deck In Perpendicular Direction to the Flow

Calculation results of the operation deck of the center pier in the perpendicular direction to the flow are shown in **Table7.4.114** and **Table7.4.115**.

Table7.4.114 Results of Checking the Bending Stress of Center Pier Operation Deck (Perpendicular Direction to the Flow)

Normal Condition (+ Operating Loads)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 4	1.59 < 8.28 OK	87.06 < 168.00 OK	9.11 < 168.00 OK
X = 1.100	1.59 < 8.28 OK	87.06 < 168.00 OK	9.11 < 168.00 OK

Normal Condition (+ Operating Loads)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 5	1.77 < 8.28 OK	103.02 < 168.00 OK	10.29 < 168.00 OK
X = 0.000	1.03 < 8.28 OK	60.20 < 168.00 OK	6.01 < 168.00 OK
X = 0.600	1.77 < 8.28 OK	103.02 < 168.00 OK	10.29 < 168.00 OK
Seismic Condition(Right)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 6	2.71 < 11.01 OK	155.75 < 223.44 OK	15.80 < 223.44 OK
X = 0.000	1.70 < 8.28 OK	99.43 < 168.00 OK	9.85 < 168.00 OK
X = 1.800	2.71 < 11.01 OK	155.75 < 223.44 OK	15.80 < 223.44 OK

Normal Condition (+ Operating Loads)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 7	1.77 < 8.28 OK	103.02 < 168.00 OK	10.29 < 168.00 OK

X = 0.000	1.77 < 8.28 OK	103.02 < 168.00 OK	10.29 < 168.00 OK
X = 0.600	1.03 < 8.28 OK	60.20 < 168.00 OK	6.01 < 168.00 OK

Normal Condition (+ Operating Loads)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 8	1.59 < 8.28 OK	87.06 < 168.00 OK	9.11 < 168.00 OK
X = 0.000	1.59 < 8.28 OK	87.06 < 168.00 OK	9.11 < 168.00 OK

Source: Study team

Table7.4.115 Result of shearing stress check on Center Pier Operation Deck (Perpendicular Direction to the Flow)

Normal Condition (+ Operating Loads)	τm (N/mm2)
Ungrouped Member 4	0.18 < 0.36 OK (yp)
X = 1.100	0.18 < 0.36 OK (yp)

Normal Condition (+ Operating Loads)	τm (N/mm2)
Ungrouped Member 5	0.14 < 0.36 OK (yp)
X = 0.000	0.12 < 0.36 OK (yp)
X = 0.600	0.14 < 0.36 OK (yp)

Seismic Condition(Right)	τm (N/mm2)
Ungrouped Member 6	0.19 < 0.48 OK (yp)
X = 0.000	0.14 < 0.48 OK (yp)
X = 1.800	0.19 < 0.48 OK (yp)

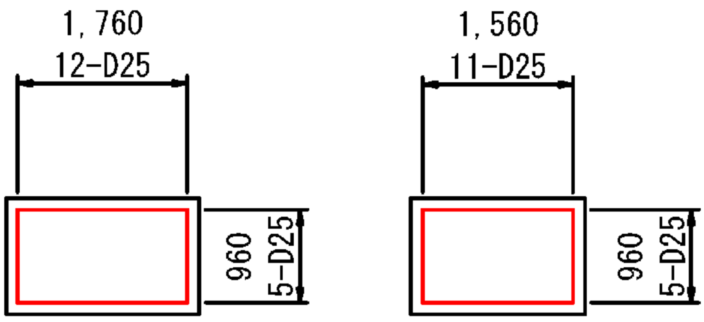
Normal Condition (+ Operating Loads)	τm (N/mm2)
Ungrouped Member 7	0.14 < 0.36 OK (yp)
X = 0.000	0.14 < 0.36 OK (yp)
X = 0.600	0.12 < 0.36 OK (yp)

Normal Condition (+ Operating Loads)	τm (N/mm2)
Ungrouped Member 8	0.18 < 0.36 OK (yp)
X = 0.000	0.18 < 0.36 OK (yp)

Source: Study team

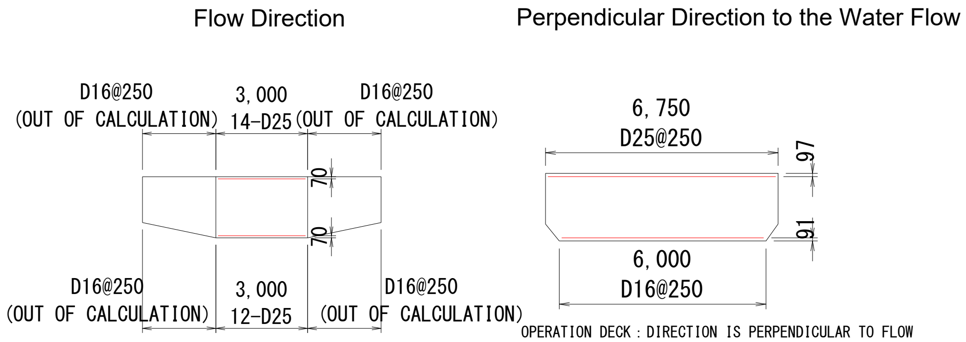
(i) Result of Examination

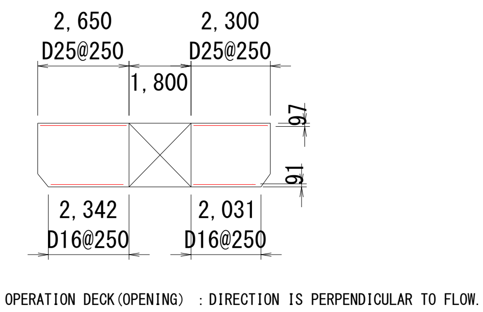
As a result of the structural calculation, the bar arrangement diagram is shown **Figure7.4.91** for column reinforcement procedures, **Figure7.4.92** for operation deck reinforcement are shown respectively. Detailed structural calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure7.4.91 Center Pier Column Reinforcement Point (Vertical Reinforcement)



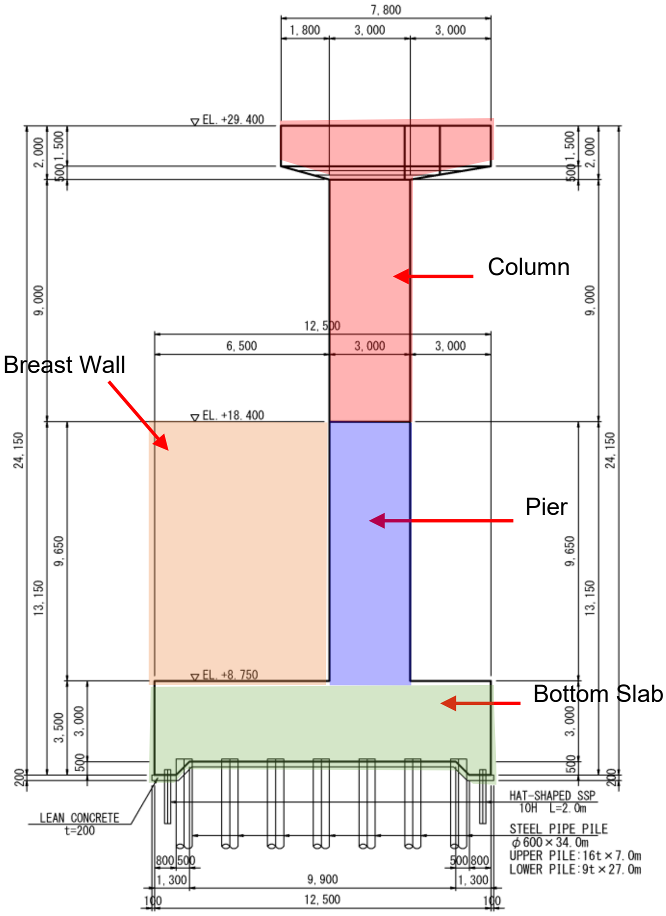


Source: Study team

Figure7.4.92 Center Pier Operation deck Reinforcement Work Procedure

3) End Pier

Since the left and right bank piers of this floodgate have symmetrical structures on the left and right sides, structural calculations will be carried out at one location as end piers. The structural drawing of the end pier is shown in **Figure7.4.93**. Structural calculations of end piers are carried out according to the classification shown in **Figure7.4.93**. The details of the structural calculation are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure7.4.93 Structural Drawing of End Pier

(a) Design of the Bottom Slab

(i) Design Policy

Design the bottom slab according to "8.7 Design of Footing in Specifications for Highway Bridges IV Substructures, Japan". The design of the bottom slab shall be carried out in the Perpendicular Direction to the Flow.

(ii) Load Condition

As for the load condition of the bottom slab, the load condition used for the design of the pile foundation is applied, and the calculated pile reaction force is applied to the bottom slab.

Aside from pile reaction forces, the arbitrary load shown in **Table7.4.116** is applied.

Table7.4.116 End Pier Slab Arbitrary Load

Number	Load name	Category	Water Level (m)		Rear Side Soil Weight (kN/m)	Front Side Water Weight (kN/m)	Uplift (kN/m)		Vertical Earth Pressure (kN, kN/m)	
			Back (m)	Front (m)			Back	Front	End pv	Distributed pv
Case 1	Normal Condition	Whole area	4.800	4.800	5834.2	1457.0	2368.4	2368.4	6348.5	1953.4
Case 2	Normal Condition + Wind Load	Whole area	4.800	4.800	5834.2	1457.0	2368.4	2368.4	6348.5	1953.4
Case 3	During Floods (At Floodway DFL)	Upstream side	6.100	6.100	5874.5	1853.8	2765.2	2765.2	5903.0	1816.3
		Downstream side	6.100	2.190	5874.5	666.5	2765.2	1577.9	5903.0	1816.3
Case 4	Seismic Condition	Whole area	2.550	2.550	5762.9	775.0	1686.4	1686.4	5302.5	1631.5
Case 5	Left Bank Construction	Whole area	0.000	0.0	5685.4	0.0	0.0	0.0	7580.3	2332.4

Source: Study team

(iii) Calculation Result

The calculation result of the end pier base slab is shown in **Table7.4.117** and **Table7.4.118**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.117 Results of Verification of Heel Slabs on the Bottom of Pier At the End

Section No.		End Pier				
		Heel Slab				
State		Normal Condition	Normal + Wind Load	During Floods (Floodway)	Seismic Condition	During Construction in Left Bank Side
Dimensions						
b	cm	28800.0	28800.0	28800.0	28800.0	28800.0
h	cm	300.0	300.0	300.0	300.0	300.0
d	cm	288.0	288.0	288.0	288.0	288.0
d '	cm	12.0	12.0	12.0	12.0	12.0
Cross-sectional Force						
M	kN m	26,599.64	34,760.65	38,786.95	82,340.11	30,138.60
N	kN	0.00	0.00	0.00	0.00	0.00
S	kN	11,057.70	11,503.17	13,065.57	19,686.13	9,686.57
Reinforcement						
As	cm^2	1477.920	1477.920	1477.920	1477.920	1477.920
As '	cm^2	377.040	377.040	377.040	377.040	377.040
n	-	9	9	9	9	9
Calculated Value						
e	cm	0.0	0.0	0.0	0.0	0.0
e-h/2	cm	-150.0	-150.0	-150.0	-150.0	-150.0
A (x^3)	-	0	0	0	0	0
B (x^2)	-	1	1	1	1	1
C (x)	-	1.15935	1.15935	1.15935	1.15935	1.15935
D (-)	-	-268.8534	-268.8534	-268.8534	-268.8534	-268.8534
x	cm	15.827	15.827	15.827	15.827	15.827
G	cm^3	-150.3	-150.3	-150.3	-150.3	-150.3
I	cm^4	1023443653.8	1023443653.8	1023443653.8	1023443653.8	1023443653.8
Sectionals stress						
σc	N/mm^2	0.41	0.54	0.60	1.27	0.47
σs '	N/mm^2	0.90	1.17	1.31	2.77	1.01
σs	N/mm^2	63.66	83.19	92.83	197.07	72.13
τm	N/mm^2	0.01	0.01	0.02	0.02	0.01
Allowance						
σca	N/mm^2	8.28	8.28	8.28	11.01	12.42
σ sa	N/mm^2	168.00	168.00	168.00	223.44	252.00
τa	N/mm^2	0.36	0.36	0.36	0.48	0.54
σ ck	N/mm^2	20.70	20.70	20.70	20.70	20.70
σsy	N/mm^2	415.00	415.00	415.00	415.00	415.00
Mc	kN・m	749074.73	749074.73	749074.73	749074.73	749074.73
Mu	kN・m	176269.82	176269.82	176269.82	176269.82	176269.82
Evaluation	Minimum Reinforceme	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)
	Stress	OK	OK	OK	OK	OK
	Shear	OK	OK	OK	OK	OK
Bar Arrangement	Reinforcement	D 28 @ 125	D 28 @ 125	D 28 @ 125	D 28 @ 125	D 28 @ 125

Source: Study team

Table7.4.118 Results of Checking the Toe Slab of the Bottom Slab of the End Pier

Case	Right Side			Allowable Stress	
	M	σ c	σs	σca	σ sa
1	19115.10	1.28	92.70	8.28	168.00
2	20305.33	1.36	98.47	8.28	168.00
3	22333.27	1.49	108.31	8.28	168.00
4	39678.64	2.65	192.42	11.01	223.44
5	17164.47	1.15	83.24	12.42	252.00

Source: Study team

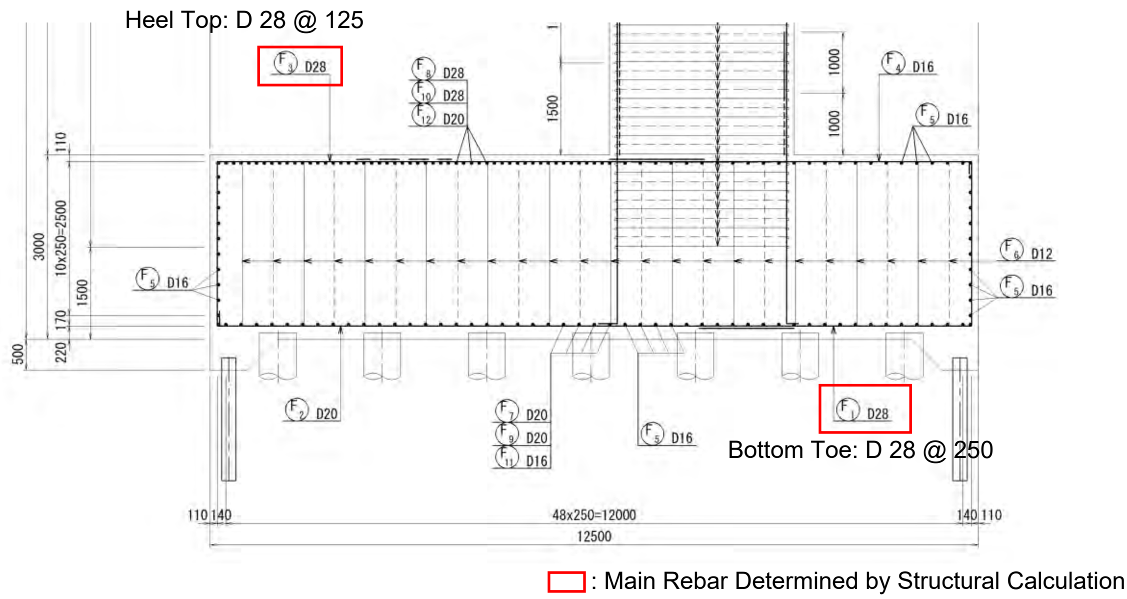
(iv) Result of Examination

The structural calculation result of the end pier base slab is shown below.

Table7.4.119 Calculation Result of End Pier Structure

Perpendicular Direction to flow	Reinforcement	Cover
Lower Side of Toe Slab	D28@250	220 mm
	(D16@250)	100 mm
Upper Side of Heel Slab	D28@125	110 mm
	(D20@250)	210 mm
Flow Direction		
Both Side Distribution Bar	D16@250	

Note: The value in () indicates compressive reinforcement.
Source: Study team



Source: Study team

Figure7.4.94 Bar Arrangement of End Pier Base Slab

(b) Design of the Piers

(i) Design Policy

In accordance with "Technical Criteria for River Works: Practical Guide for Planning [I]", structural calculation is performed for the pier as a cantilever fixed to the floor slab. Since the pier has a sufficient length in the flow direction, the check in the flow direction is omitted and the check is carried out in the direction perpendicular to the flow direction.

(ii) Load Condition

The load condition of the pier is the same as that of the load case in the stability calculation. The table below shows the cross sectional force generated at the base of the pier based on the load calculation of the stability calculation.

Table7.4.120 Cross Sectional Force At Base of End Pier (Perpendicular Direction to the Flow)

Number	Load name	V (kN)	H (kN)	M (kNm)
Case 1	Normal Condition	33176.41	11057.70	26599.64
Case 2	Normal Condition + Wind Load	33176.41	11503.17	34760.65
Case 3	During floods (at Floodway DFL)	32986.41	13065.57	38786.95
Case 4	Seismic Condition	32013.16	19686.13	82340.11
Case 5	left bank construction	32013.16	9686.57	30138.60

Source: Study team

(iii) Calculation Result

The calculation result of the end pier is shown in **Table7.4.121**. Of the calculation results, the case surrounded by the red line is the decision case.

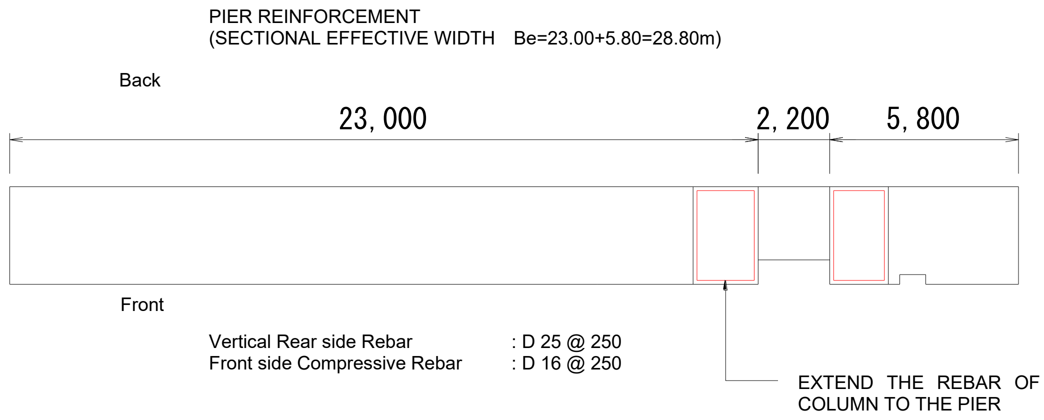
Table7.4.121 List of Calculation Results of End Pier

Section No.			End Pier				
State			Weir Main Body				
			Normal Condition	Normal + Wind Load	During Floods (Floodway)	Seismic Condition	During Construction in Left Bank Side
Dimensions							
	b	cm	28800.0	28800.0	28800.0	28800.0	28800.0
	h	cm	300.0	300.0	300.0	300.0	300.0
	d	cm	288.0	288.0	288.0	288.0	288.0
	d '	cm	12.0	12.0	12.0	12.0	12.0
Cross-sectional Force							
	M	kN m	26,599.64	34,760.65	38,786.95	82,340.11	30,138.60
	N	kN	33,176.41	33,176.41	32,986.41	32,013.16	32,013.16
	S	kN	11,057.70	11,503.17	13,065.57	19,686.13	9,686.57
Reinforcement							
	As	cm^2	589.080	589.080	589.080	589.080	589.080
	As '	cm^2	241.320	241.320	241.320	241.320	241.320
	n	-	9	9	9	9	9
Calculated Value							
	e	cm	80.2	104.8	117.6	257.2	94.1
	e-h/2	cm	-69.8	-45.2	-32.4	107.2	-55.9
	A (x^3)	-	1	1	1	1	1
	B (x^2)	-	-209.4709012	-135.6743994	-97.24609316	321.621202	-167.5667757
	C (x)	-	214.8175023	253.1178867	273.0621776	490.4543039	236.5657434
	D (-)	-	-69088.63529	-77047.16737	-81191.44116	-126363.8914	-73607.75638
	x	cm	210.014	137.891	102.331	18.566	168.750
	G	cm^3	635141277.5	273278145.7	150003346.3	3549424.5	409770712.1
	I	cm^4	89040784816.8	25323638747.3	10487586186.9	446406776.0	46260789003.9
Sectionals stress							
	σc	N/mm^2	0.11	0.17	0.23	1.67	0.13
	σs '	N/mm^2	0.93	1.38	1.79	5.33	1.10
	σs	N/mm^2	0.37	1.64	3.67	218.73	0.84
	τm	N/mm^2	0.01	0.01	0.02	0.02	0.01
Allowance							
	σca	N/mm^2	8.28	8.28	8.28	11.01	12.42
	σ sa	N/mm^2	168.00	168.00	168.00	223.44	252.00
	τa	N/mm^2	0.36	0.36	0.36	0.48	0.54
	σ ck	N/mm^2	20.70	20.70	20.70	20.70	20.70
	σsy	N/mm^2	415.00	415.00	415.00	415.00	415.00
	Mc	kN・m	765662.94	765662.94	765567.94	765081.31	765081.31
	Mu	kN・m	119843.83	119843.83	119560.98	118112.05	118112.05
Evaluation	Minimum Reinforceme		1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)	1.7 Mc < Mb (Ok)
	Stress		OK	OK	OK	OK	OK
	Shear		OK	OK	OK	OK	OK
Bar Arrangement	Reinforcement		120 - D 25	120 - D 25	120 - D 25	120 - D 25	120 - D 25

Source: Study team

(iv) Result of Examination

The effective member width of the center pier is considered to be the length L = 28.80 m excluding the door contact part. The structural calculation result of the pier section is shown below.



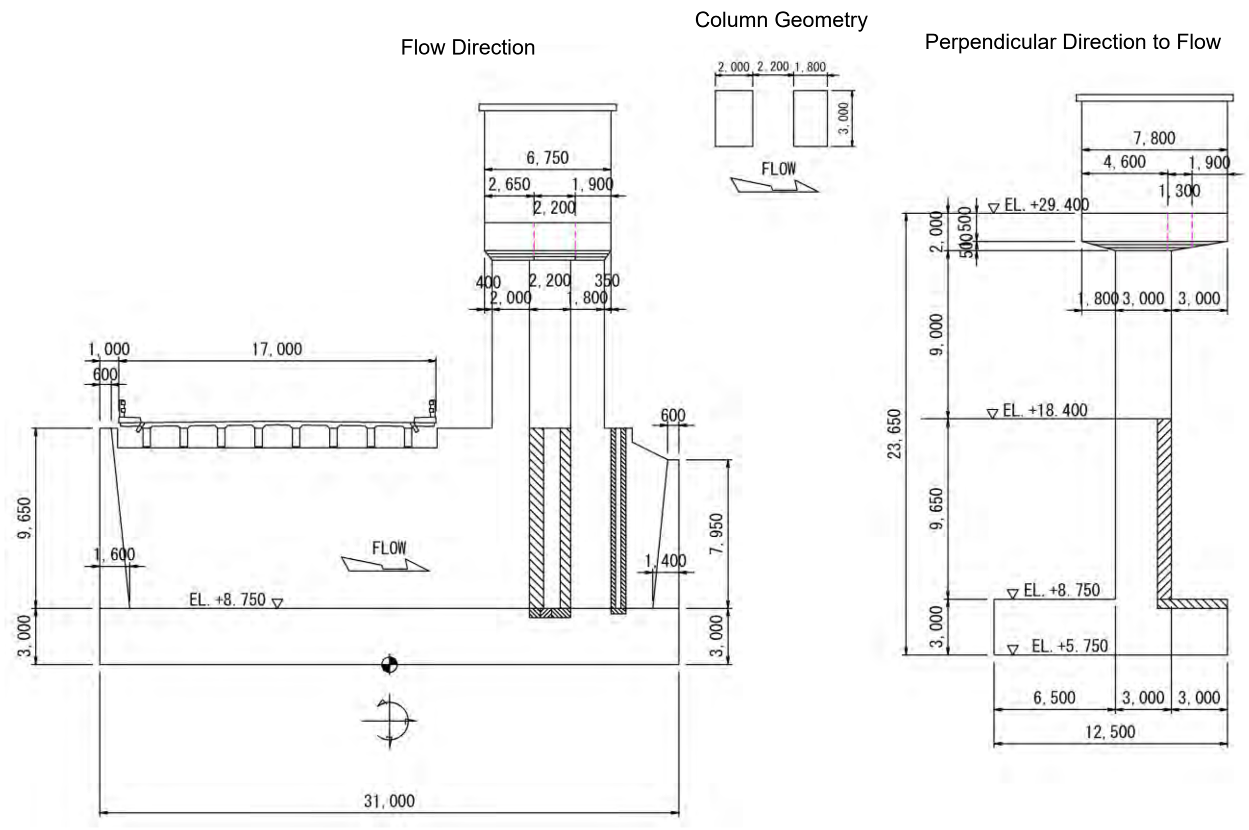
Source: Study team

Figure7.4.95 Bar Arrangement of End Pier And Piers

(c) Column Design

(i) Design Policy

The structural dimensions of the column of the end pier are shown in **Figure7.4.96**. Since it can be accessed from the side of the floodgate to the lower part of the column, the end pier column has two pillars. In the structural calculation of the column, in accordance with the "Technical Criteria for River Works: Practical Guide for Planning [I]", structural calculation is carried out as a portal frame fixed to the weir in the flow direction and a cantilever fixed to the weir in the perpendicular direction to the flow.



Source: Study team

Figure7.4.96 Structural Dimension of End Pier

(ii) Study Case

A list of cases to be examined in the perpendicular direction to the flow and in the flow direction is shown in **Table7.4.122** and **Table7.4.123**.

Table7.4.122 End Pier Column Load Case (Perpendicular Direction to the Flow)

Calculation Direction	Study Case	Load Condition	Load Term								Extra Factor
			Body Weight	Equipment Load	Local Control House Load	Temperature Load	Wind Load	Operational Load	Inertial Force Right	Inertia Force Left	
Perpendicular Direction To The Flow	1	Normal Condition	○	○	○			○			1.00
	2	Normal Condition (Temperature Rise + 16.7 °)	○	○	○	○		○			1.25
	3	Normal Condition (Temperature Drop -22.2 °)	○	○	○	○		○			1.25
	4	Wind Load	○	○	○		○				1.25
	5	Wind Load + Temperature Rise	○	○	○	○	○				1.40
	6	Wind Load + Temperature Drop	○	○	○	○	○				1.40
	7	Seismic Condition	○	○	○				○		1.33

Source: Study team

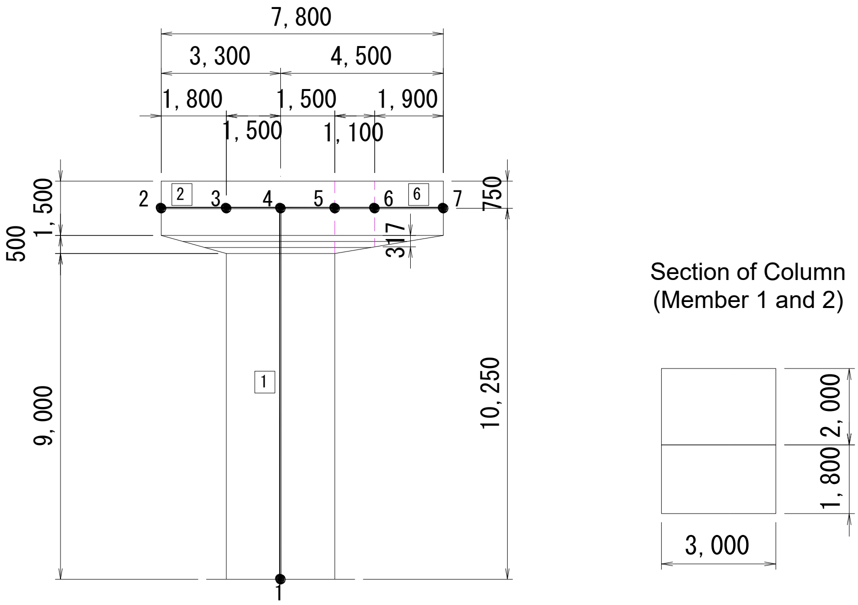
Table7.4.123 End Pier Column Load Case (Flow Direction)

Calculation Direction	Study Case	Load Condition	Load Term								Extra Factor
			Body Weight	Equipment Load	Local Control House Load	Temperature Load	Wind Load	Operational Load	Inertial Force Right	Inertia Force Left	
Flow Direction	1	Normal Condition	○	○	○			○			1.00
	2	Normal Condition (Temperature rise + 16.7 °)	○	○	○	○		○			1.25
	3	Normal Condition (Temperature drop -22.2 °)	○	○	○	○		○			1.25
	4	wind load	○	○	○		○				1.25
	5	Wind load + Temperature rise	○	○	○	○	○				1.40
	6	Wind load + Temperature drop	○	○	○	○	○				1.40
	7	Seismic Condition (Right and downstream directions)	○	○	○				○		1.33
	8	Seismic Condition (Left and upstream direction)	○	○	○					○	1.33

Source: Study team

(iii) Study Model

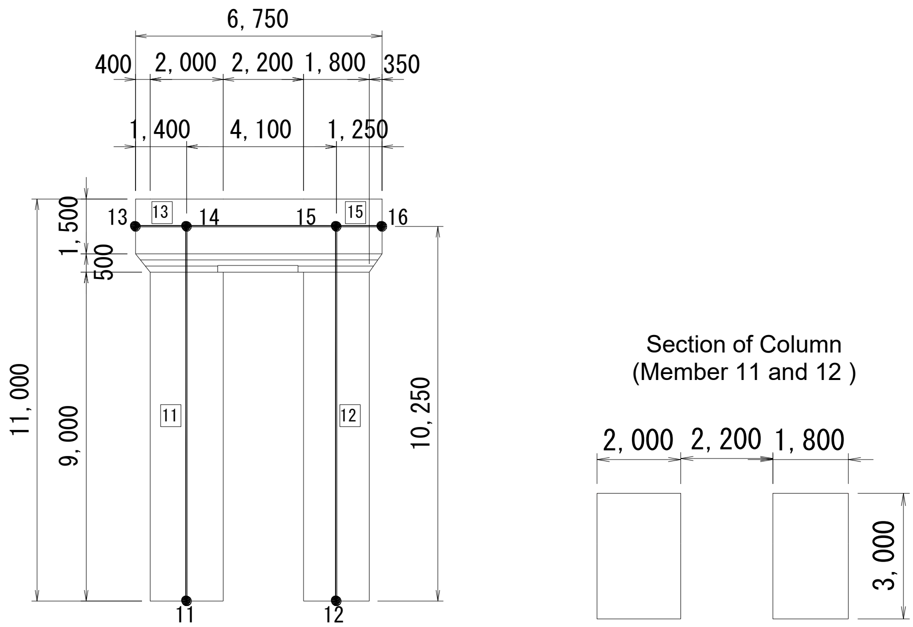
A model for appendicular direction to the flow direction is shown in **Figure7.4.97**. The cross section of the column is calculated as the cross section of the up-and-down column.



Source: Study team

Figure7.4.97 Study Model for End Pier Column (Perpendicular Direction to the Flow)

A model for flow direction is shown in **Figure7.4.98**. The cross section of the column section is calculated as the cross section of the column in the perpendicular direction to the flow.



Source: Study team

Figure7.4.98 Study Model for End Pier Column (Flow Direction)

(iv) Calculation Result

The calculation results for each member in the most severe load case are shown. Of the calculation results, the case surrounded by the red line is the decision case.

A. Column in Flow Direction

The calculation results of the flow direction in the end section column are shown in **Table7.4.124** and **Table7.4.125**.

Table7.4.124 Results of Bending Stress Check on End Pier Column (Flow Direction)

Seismic Condition(Right)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 11	5.72 < 11.01 OK	151.65 < 223.44 OK	39.02 < 223.44 OK
X = 0.000	3.90 < 11.01 OK	134.34 < 223.44 OK	24.71 < 223.44 OK
X = 5.125	0.77 < 6.21 OK	1.42 < 223.44 OK	6.68 < 168.00 OK
X = 10.250	5.72 < 11.01 OK	151.65 < 223.44 OK	39.02 < 223.44 OK

Seismic Condition(Left)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 12	3.69 < 11.01 OK	45.50 < 223.44 OK	28.70 < 223.44 OK
X = 0.000	2.84 < 11.01 OK	45.50 < 223.44 OK	22.21 < 223.44 OK
X = 5.125	0.78 < 6.21 OK	0.00 < 168.00 OK	6.84 < 168.00 OK
X = 10.250	3.69 < 11.01 OK	43.76 < 223.44 OK	28.70 < 223.44 OK

Source: Study team

Table7.4.125 Results of Shear Stress Check on End Pier Column (Flow Direction)

Seismic Condition(Right)	τm (N/mm2)	Seismic Condition(Left)	τm (N/mm2)
Ungrouped Member 11	0.18 < 0.48 OK (yp)	Ungrouped Member 12	0.19 < 0.48 OK (yp)
X = 0.000	0.10 < 0.48 OK (yp)	X = 0.000	0.10 < 0.48 OK (yp)
X = 5.125	0.14 < 0.48 OK (yp)	X = 5.125	0.14 < 0.48 OK (yp)
X = 10.250	0.18 < 0.48 OK (yp)	X = 10.250	0.19 < 0.48 OK (yp)

Source: Study team

B. Column in Perpendicular Direction to the Flow

Calculation results of the end section column section in the perpendicular direction to the flow are shown in **Table7.4.126** and **Table7.4.127**.

Table7.4.126 Results of Bending Stress Check on End Pier Column (Perpendicular Direction to the Flow)

Seismic Condition(Right)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 1	5.54 < 11.01 OK	146.37 < 223.44 OK	41.67 < 223.44 OK
X = 0.000	0.65 < 6.21 OK	0.00 < 168.00 OK	5.67 < 168.00 OK
X = 5.125	1.95 < 11.01 OK	17.08 < 223.44 OK	16.14 < 223.44 OK
X = 10.250	5.54 < 11.01 OK	146.37 < 223.44 OK	41.67 < 223.44 OK

Source: Study team

Table7.4.127 Results of Shear Stress Check on End Pier Column (Perpendicular Direction to the Flow)

Seismic Condition(Right)	τm (N/mm2)
Ungrouped Member 1	0.38 < 0.48 OK (yp)
X = 0.000	0.21 < 0.48 OK (yp)
X = 5.125	0.30 < 0.48 OK (yp)
X = 10.250	0.38 < 0.48 OK (yp)

Source: Study team

C. Operation Deck in Flow Direction

Calculation results of the end operation deck in the flow direction are shown in **Table7.4.128**, **Table7.4.129**.

Table7.4.128 Result of Checking Bending Stress on End Pier Operation Deck (Flow Direction)

Seismic Condition(Right)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 14	3.51 < 11.01 OK	211.56 < 223.44 OK	22.72 < 223.44 OK
X = 0.000	3.36 < 11.01 OK	196.02 < 223.44 OK	21.98 < 223.44 OK
X = 2.050	2.82 < 10.35 OK	184.93 < 210.00 OK	17.72 < 210.00 OK
X = 4.100	3.51 < 11.01 OK	211.56 < 223.44 OK	22.72 < 223.44 OK

Source: Study team

Table7.4.129 Results of Checking the Shear Stress on End Pier Operation Deck (Flow Direction)

Seismic Condition(Right)	τm (N/mm2)
Ungrouped Member 14	0.39 < 0.48 OK (yp)
X = 0.000	0.27 < 0.36 OK (yp)
X = 2.050	0.17 < 0.48 OK (yp)
X = 4.100	0.39 < 0.48 OK (yp)

Source: Study team

D. Operation Deck In the Perpendicular Direction to the Flow

Calculation results of the end operation deck in the perpendicular direction to the flow are shown in **Table7.4.130** and **Table7.4.131**.

Table7.4.130 Result of Checking Bending Stress on End Pier Operation Deck (Perpendicular Direction to the Flow)

Normal Condition (+ Operating Loads)	σ'c (N/mm2)	σs (N/mm2)	σ's (N/mm2)
Ungrouped Member 5	2.57 < 8.28 OK	142.48 < 168.00 OK	14.83 < 168.00 OK
X = 0.000	2.57 < 8.28 OK	142.48 < 168.00 OK	14.83 < 168.00 OK

Source: Study team

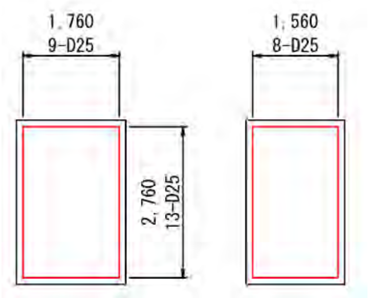
Table7.4.131 Results of Checking the Shear Stress on End Pier Operation Deck (Perpendicular Direction to the Flow)

Normal Condition (+ Operating Loads)	τm (N/mm2)
Ungrouped Member 5	0.24 < 0.36 OK (yp)
X = 0.000	0.24 < 0.36 OK (yp)

Source: Study team

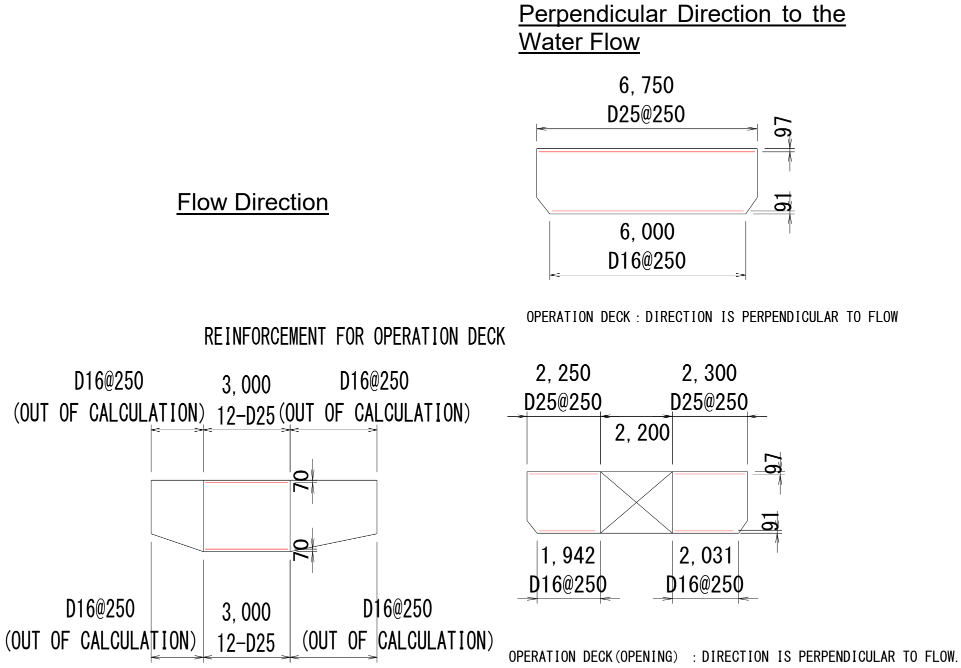
(v) Result of Examination

As a result of the structural calculation, the bar arrangement diagram is shown in **Figure7.4.99** for column reinforcement, and **Figure7.4.100** for operation deck. Detailed structural calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure7.4.99 Bar Arrangement of End Pier Column (Vertical Reinforcement)



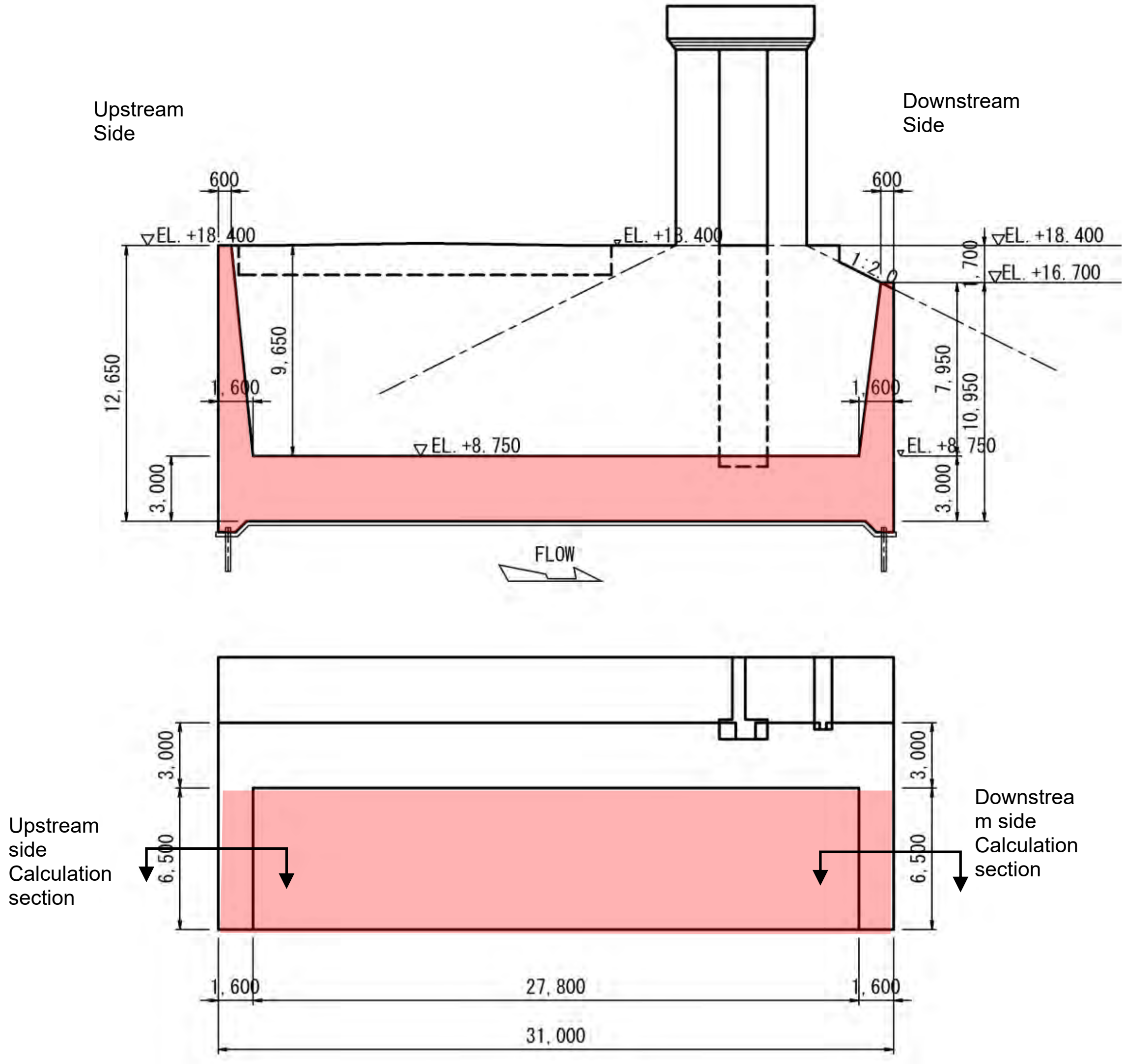
Source: Study team

Figure7.4.100 Bar Arrangement of Operation Deck of End Pier

(d) Breast wall Design

(i) Study Policy

The breast wall structural dimensions are shown in **Figure7.4.101**. In the case of this floodgate, since the overhanging length of the breast wall is 6.5 m, the structural calculation is performed assuming that each of the upstream and downstream sides is an independent inverted T retaining wall.



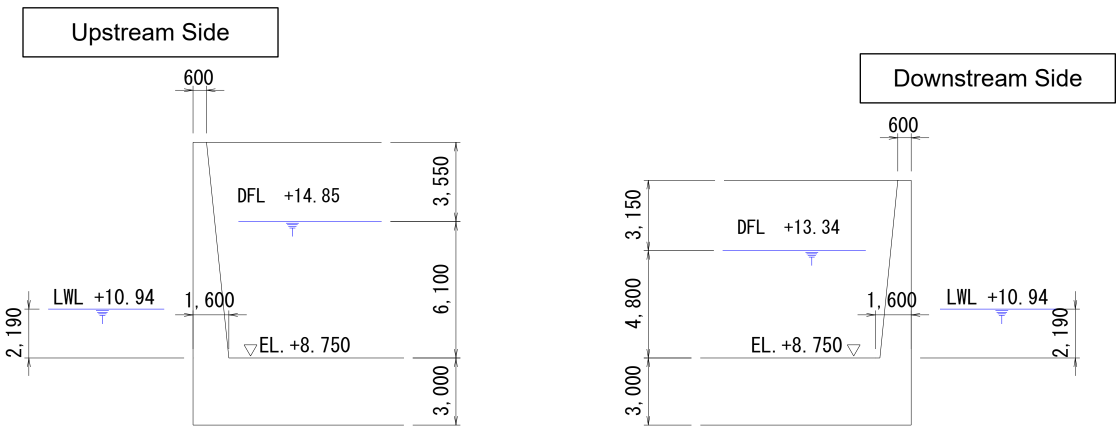
Source: Study team

Figure7.4.101 End Pier Breast Wall Structure

(ii) Load Condition

Structural calculation of the breast wall shall be carried out under constant load conditions where the difference in water level between the front and rear sides of the retaining wall is the biggest. The load conditions for calculating the breast wall are shown below.

Back Earth Pressure	: Normal, Earth Pressure at Rest
Soil Type of the Backfill	: Backfill γ = 19.0 kN/m3 Internal Friction Angle Φ = 30 ° , Cohesion C = 0 KN/m2
Rear Side Water Level (Upstream Side)	: El + 14.85 (Floodway DFL)
Rear Side Water Level (Downstream Side)	: El + 13.34 (Floodway DFL)
Front Side Water Level (Common in Upstream AND Downstream)	: El + 10.94 (Low Water Level)



Source: Study team

Figure7.4.102 Water Level Condition of the End Pier Breast Wall

(iii) Calculation Result

A. Breast wall in the Upstream Side

Calculation results of the end upstream breast wall are shown in **Table7.4.132** to **Table7.4.134**.

Table7.4.132 List of Calculated End Breast Wall Results

Section No.		End Pier Breastwall (Upstream)	
State		Vertical Wall	Bottom Slab
		Normal Condition	Normal Condition
Dimensions			
b	cm	100.0	100.0
h	cm	160.0	300.0
d	cm	150.0	285.0
d '	cm	9.0	23.0
Cross-sectional Force			
M	kN m	1,842.00	1,842.00
N	kN		
S	kN	567.40	
Reinforcement			
As	cm^2	81.430	49.264
As '	cm^2	19.636	12.568
n	-	9	9
Calculated Value			
e	cm	0.0	0.0
e-h/2	cm	-80.0	-150.0
A (x^3)	-	0	0
B (x^2)	-	1	1
C (x)	-	18.19188	11.12976
D (-)	-	-2230.42032	-2579.27472
x	cm	38.999	45.526
G	cm^3	-1.7	1.8
I	cm^4	11166042.9	28629297.0
Sectionals stress			
σc	N/mm^2	6.43	2.93
$\sigma s'$	N/mm^2	44.54	13.04
σs	N/mm^2	164.80	138.67
τm	N/mm^2	0.38	0.00
Allowance			
σca	N/mm^2	8.28	8.28
σsa	N/mm^2	168.00	168.00
τa	N/mm^2	0.36	0.36
σck	N/mm^2	20.70	20.70
σsy	N/mm^2	415.00	415.00
Mc	kN・m	739.83	2600.95
Mu	kN・m	4744.49	5707.92
Evaluation	Minimum Reinforceme	Mb < Mu (Ok)	Mb < Mu (Ok)
	Stress	OK	OK
	Shear	NG	OK
Bar Arrangement	Reinforcement	D 36 @ 125	D 28 @ 125

Source: Study team

From the above table, the shear stress value at the base of the vertical wall exceeds the allowable value. Therefore, it is necessary to install shear reinforcement.**Table7.4.133** shows the calculation of the requirement of shear reinforcement. In accordance with **Table7.4.133**, 2 pieces of D 12 @ 250 are arranged per 1 m.

Table7.4.133 Necessary Amount of Shear Reinforcement for the End-upstream Breast Wall

Shear Reinforcement Bars (Vertical Wall)			
Shear Force		567.4	kN (Normal Condition)
Shear Capacity	Concrete	536.4	kN
	Reinforcing Bar	196.9	kN
	Cross-Sectional Area	113.1	(D12)
	Number Of Trains	2	No. of Bars Per 1.0 m
	Pitch	250	Mm
	Subtotal	733.3	kN

Reinforcing bar capacity (kN) = Aw ・ σ sa ・ d/(1.15 s)
Source: Study team

Table7.4.134 shows the evaluation result of necessity of the shear reinforcement in the position of 0.5 m upper part from the base of vertical wall. Based on **Table7.4.134**, the height at which shear reinforcement becomes unnecessary is 0.50 m above the wall base. Therefore, the shear reinforcement range is set to be longer than the length obtained by adding the effective member length to 0.50 m from the wall base.

L = 0.50 + (1.548 -0.10) = 1.948 m ··· 2.5 m Above the Base of the Breast Wall.

Table7.4.134 Necessary Range of Shear Reinforcement of the End-upstream Breast Wall

Reinforcement height He =	0.500	m	Earth Pressure Coefficient	0.5	
Virtual Rear Height H =	9.150	m	γ s	19	(kN/m3)
Back water level Hw =	5.600	m	γ sub	10.2	(kN/m3)
Water Level in Front Side Hw =	-1.690	m	Surcharge	10.00	(kN/m2)

load term	Load Intensity kN/m2	Load Width m	Load Height m	Acting Height m	Horizontal Force kN	Moment kN m	Center of Load m
Earth Pressure 1	5.0	1.000	3.550	7.967	8.9	70.9	
Earth Pressure 2	38.7	1.000	3.550	6.783	68.7	466.0	
Earth Pressure 3	38.7	1.000	5.600	3.733	108.4	404.7	
Earth Pressure 4	67.3	1.000	5.600	1.867	188.4	351.7	
Rear Side Water Pressure	54.9	1.000	5.600	1.867	153.7	287.0	
Front Side Water Pressure	-16.6	1.000	1.690	0.563	-14.0	-7.9	
Subtotal					514.1	1572.4	3.059

Member Thickness	1.548	m
Shear Capacity (Ps)	521.3	kN
Shear Force (S)	514.1	kN
Evaluation	S < Ps	OK (No shear reinforcement is needed)

Source: Study team

B. Breast Wall in the Downstream Side

Calculation results of the end-downstream breast wall is shown in **Table7.4.135**.

Table7.4.135 List of Calculation Results of the End-Downstream Breast Wall

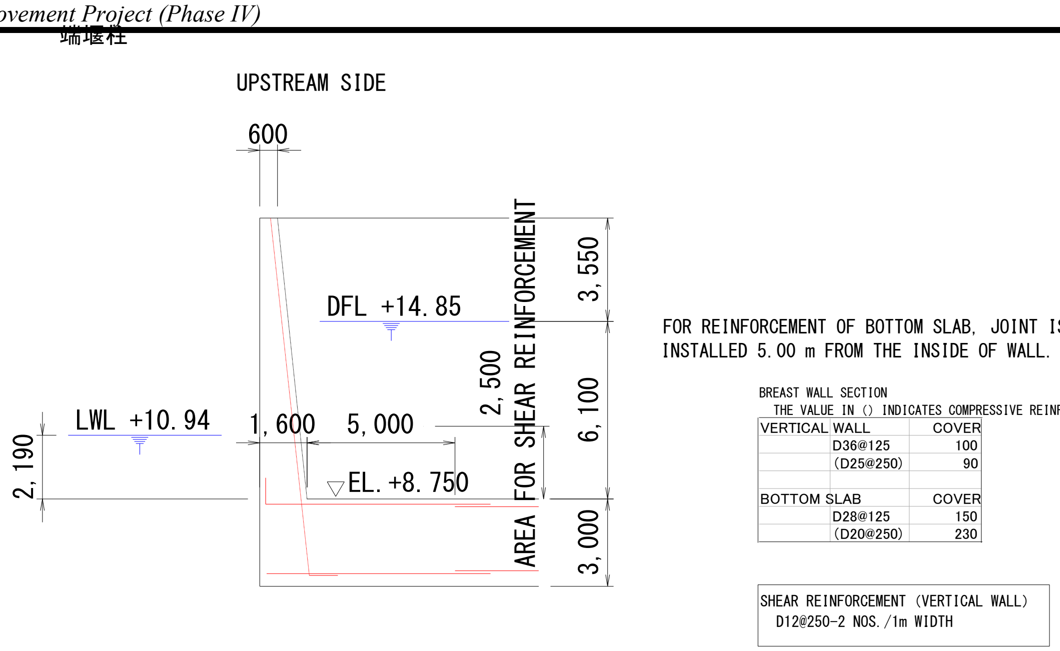
Section No.		hii bashira and chest wall (Downstre	
State		Vertical Wall	Bottom Slab
		Normal Condition	Normal Condition
Dimensions			
b	cm	100.0	100.0
h	cm	140.0	300.0
d	cm	130.0	285.0
d '	cm	9.0	22.8
Cross-sectional Force			
M	kN m	1,036.00	1,036.00
N	kN		
S	kN	378.60	
Reinforcement			
As	cm^2	64.344	24.632
As '	cm^2	12.568	8.044
n	-	9	9
Calculated Value			
e	cm	0.0	0.0
e-h/2	cm	-70.0	-150.0
A (x^3)	-	0	0
B (x^2)	-	1	1
C (x)	-	13.84416	5.88168
D (-)	-	-1526.00976	-1296.634176
x	cm	32.751	33.188
G	cm^3	1.4	0.5
I	cm^4	6711519.6	15283379.6
Sectionals stress			
σc	N/mm^2	5.06	2.25
σs '	N/mm^2	33.00	6.34
σs	N/mm^2	135.11	153.63
τm	N/mm^2	0.29	0.00
Allowance			
σca	N/mm^2	8.28	8.28
σ sa	N/mm^2	168.00	168.00
τa	N/mm^2	0.36	0.36
σ ck	N/mm^2	20.70	20.70
σsy	N/mm^2	415.00	415.00
Mc	kN・m	566.43	2600.95
Mu	kN・m	3268.73	2883.66
Evaluation	Minimum Reinforceme	Mb < Mu (Ok)	Mb < Mu (Ok)
	Stress	OK	OK
	Shear	OK	OK
Bar Arrangement	Reinforcement	D 32 @ 125	D 28 @ 250

Source: Study team

(iv) Result of Examination

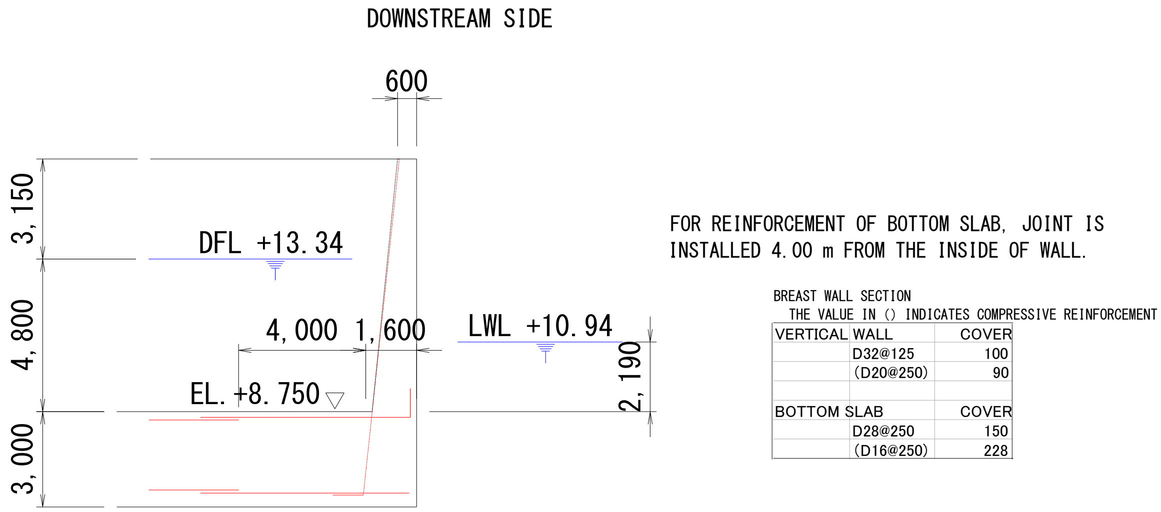
As a result of the structural calculation, the bar arrangement diagram is shown.

Figure7.4.103 shows the orientation of the end-upstream breast wall and **Figure7.4.104** shows the bar arrangement of the end-downstream breast wall. Detailed structural calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure7.4.103 Bar Arrangement of the Upstream Breast Wall of the End Pier



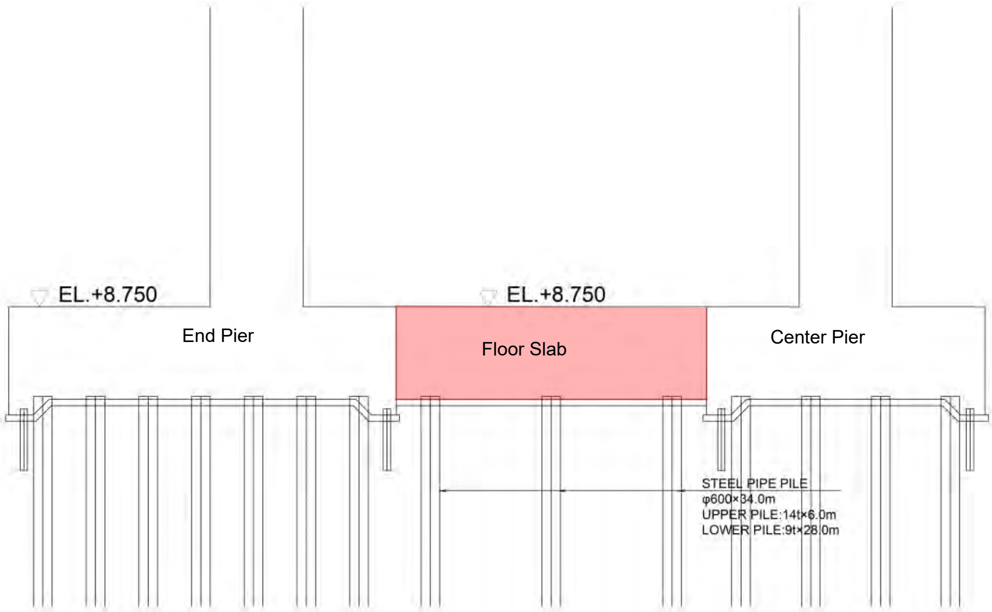
Source: Study team

Figure7.4.104 Bar Arrangement of the Downstream Breast Wall of the End Pier

4) Floor slab

(a) Study Policy

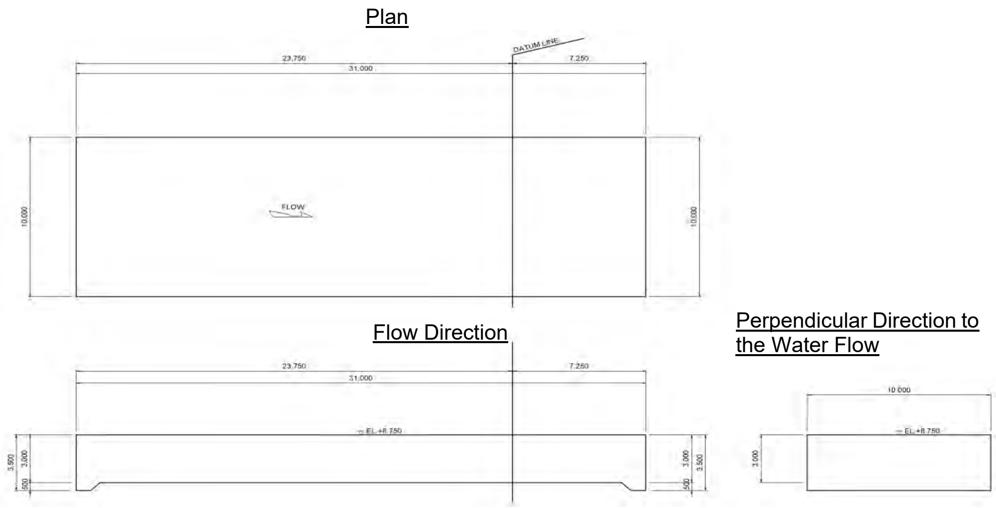
Since this floodgate has an inverted T-shaped floodgate structure, structural calculations are performed for the floor slab by determining the moment and axial force as beams supported by foundation piles.



Source: Study team

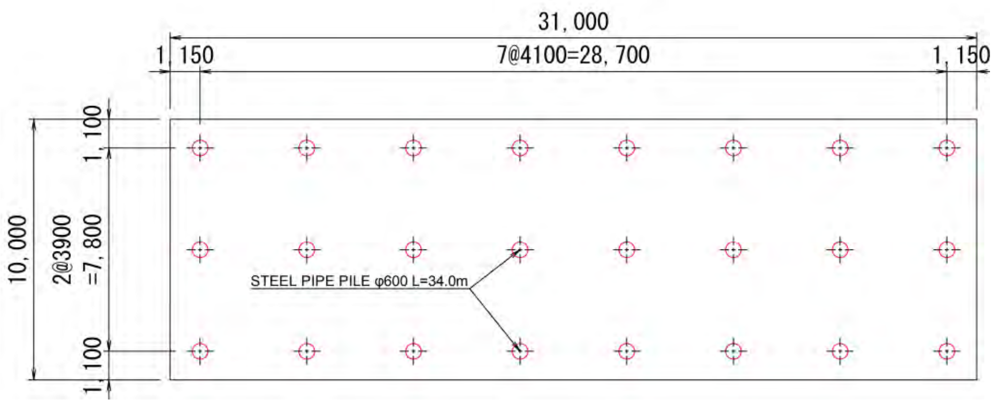
Figure7.4.105 Cross Section of Floor Slab

The structural dimensions of the floor slab are shown below. For calculation, perform structural calculation in the flow direction and the Perpendicular Direction to the Flow.



Source: Study team

Figure7.4.106 Structural Dimension of Floor Slab



Source: Study team

Figure7.4.107 Floor Slab Pile Arrangement

(b) Study Case

The calculation is made for the following cases.

Table7.4.136 Load Case List

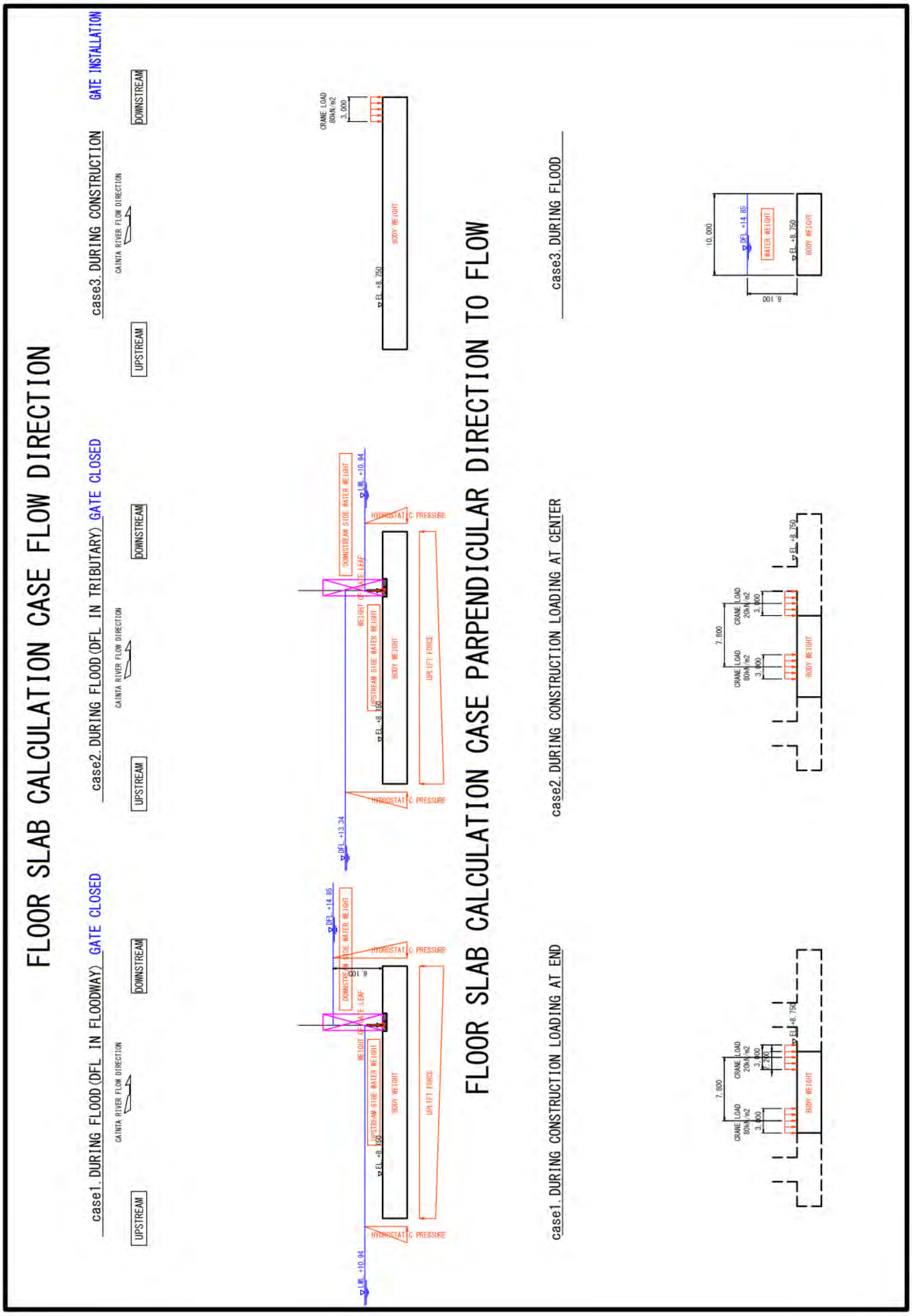
Member	Calculation Direction	Case	Case Name	Additional Factor Of Allowable Stress
Floor Slab	Flow Direction	1	During Floods (At Floodway DFL)	1.0
		2	During Floods (At Tributary DFL)	1.0
		3	During Construction	1.5
	Perpendicular Direction To The Flow	1	During Construction, Loading At The End	1.5
		2	During Construction, Loading At The Center	1.5
		3	During Flood	1.0

Source: Study team

(c) Load Condition

Load conditions used for structural calculation shall be the same as those used for pile foundation design.

The diagram set on the next page is shown.



Source: Study team

Figure7.4.108 Load Diagram of Floor Slab

(d) Calculation Result

(i) Flow Direction

The result of the bending stress degree check for the most severe load case at each check position is shown in **Table7.4.137**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.137 Results of Checking Bending Stress of Floor Slab (Flow Direction)

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom Slab Member 1 X = 1.100 Case 2 Flood (At Tributary DFL)	1.000 0.0 -31.4 0.0	0.07 < 8.28 0.34 < 168.00	x= -0.309, α= 0 8.95 < 168.00
Bottom Slab Member 2 X = 0.000 Case 2 Flood (At Tributary DFL)	1.000 0.0 -226.8 0.0	0.51 < 8.28 2.46 < 168.00	x= -0.309, α= 0 64.58 < 168.00
Bottom Slab Member 3 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -212.4 0.0	0.48 < 8.28 2.30 < 168.00	x= -0.309, α= 0 60.48 < 168.00
Bottom Slab Member 4 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -239.5 0.0	0.54 < 8.28 2.60 < 168.00	x= -0.309, α= 0 68.20 < 168.00
Bottom Slab Member 5 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -283.2 0.0	0.64 < 8.28 3.07 < 168.00	x= -0.309, α= 0 80.65 < 168.00
Bottom Slab Member 6 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -343.6 0.0	0.78 < 8.28 3.73 < 168.00	x= -0.309, α= 0 97.85 < 168.00
Bottom Slab Member 7 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -387.9 0.0	0.88 < 8.28 4.21 < 168.00	x= -0.309, α= 0 110.45 < 168.00
Bottom Slab Member 11 X = 2.036 Case 2 Flood (At Tributary DFL)	1.000 0.0 -370.3 0.0	0.84 < 8.28 4.02 < 168.00	x= -0.309, α= 0 105.45 < 168.00
Bottom Slab Member 9 X = 4.114 Case 2 Flood (At Tributary DFL)	1.000 0.0 -302.0 0.0	0.68 < 8.28 3.28 < 168.00	x= -0.309, α= 0 86.00 < 168.00
Bottom Slab Member 10 X = 0.000 Case 3 During Construction	1.500 0.0 -92.0 0.0	0.21 < 12.42 1.00 < 252.00	x= -0.309, α= 0 26.19 < 252.00

Source: Study team

(ii) Perpendicular Direction to the Flow

The result of the bending stress degree check for the most severe load case at each check position is shown in **Table7.4.138**.

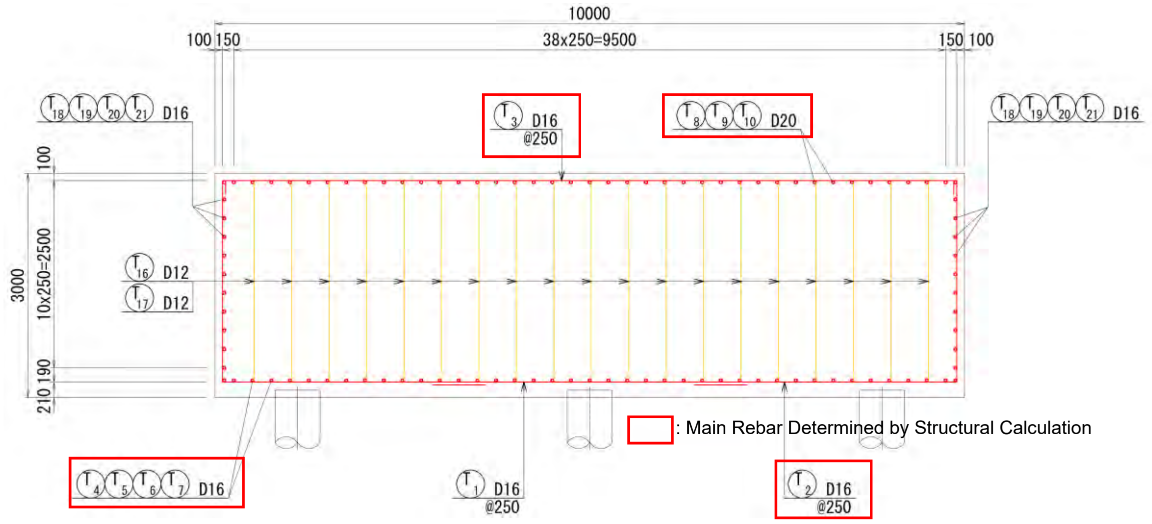
Table7.4.138 Results of Checking Bending Stress of Floor Slab (Perpendicular Direction to the Flow)

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom Slab Member 1 X = 1.100 Case 3 Flooding	1.000 0.0 -53.2 0.0	0.15 < 8.28 0.36 < 168.00	X = -0.251, α= 0 23.48 < 168.00
Bottom Slab Member 2 X = 0.000 Case 3 Flooding	1.000 0.0 -239.8 0.0	0.67 < 8.28 1.63 < 168.00	X = -0.251, α= 0 105.92 < 168.00
Bottom Slab Member 3 X = 3.900 Case 3 Flooding	1.000 0.0 -239.8 0.0	0.67 < 8.28 1.63 < 168.00	X = -0.251, α= 0 105.92 < 168.00
Bottom Slab Member 5 X = 0.000 Case 3 Flooding	1.000 0.0 -79.7 0.0	0.22 < 8.28 0.54 < 168.00	X = -0.251, α= 0 35.22 < 168.00

Source: Study team

(e) Result of Examination

The bar arrangement diagram is shown as the calculation result of the floor slab.



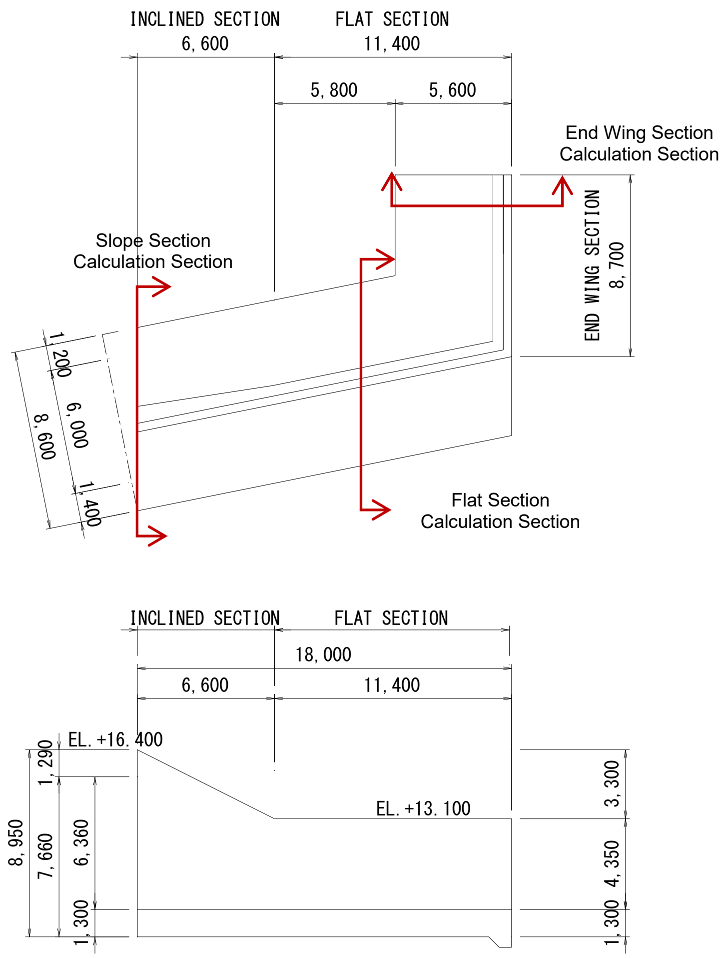
Source: Study team

Figure7.4.109 Bar Arrangement of Floor Slab

5) Downstream Wing wall

(a) Study Policy

The structural dimensions of the downstream wing wall are shown. The downstream wing wall is a retaining wall having an L-shaped surface whose wall height decreases in the flow direction. The structural calculation is performed in three sections shown in the following figure.



Source: Study team

Figure7.4.110 Structural Dimensions of the Downstream Wing Wall

(b) Study Case

The calculation is made for the following cases.

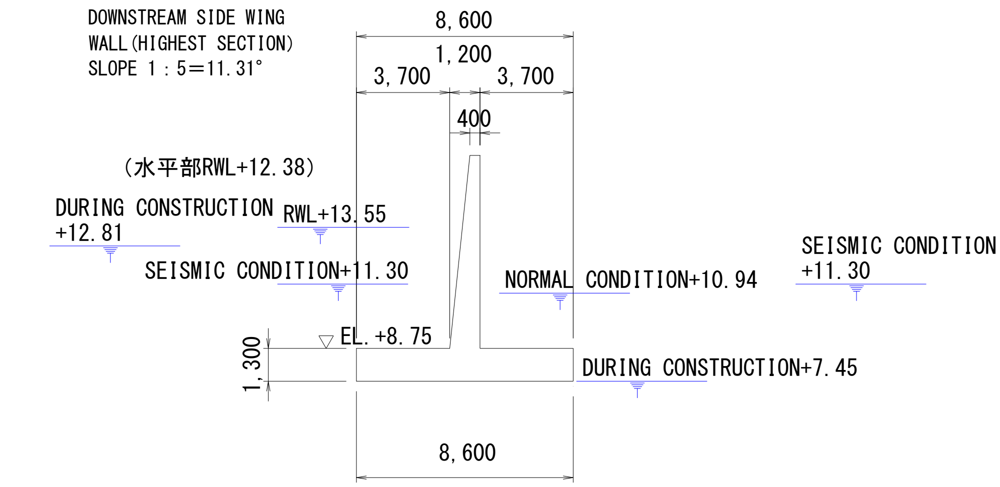
Table7.4.139 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

Calculation Direction	Study Case	Load Condition	Water level Condition		Load Condition							
			Water level in rear side	Water level in front side	Body weight	Water Weight	Earth Pressure	Water pressure	Uplift pressure	Water Weight	Surcharge	Inertial force
Highest Section, Perpendicular Direction To The Flow	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual water level	WL = 13.55 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During Construction	WL = 12.81 Water Level for Cofferdam	WL = 7.45 Lower Side of Bottom Slab	○	○	○	○	○	-	○	-

The residual water level is 2/3 of the difference between HWL = 14.853 and the OWL = 10.94.
Source: Study team

Calculation Direction	Study Case	Load Condition	Water Level Condition		Load Condition							
			Water level in rear side	Water level in front side	Body Weight	Water Weight	Earth Pressure	Water Pressure	Uplift Pressure	Water Weight	Surcharge	Inertial Force
Lowest section, flow direction and perpendicular Direction to Flow	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual Water Level	WL = 12.38 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During construction	WL = 12.81 Water level for cofferdam	WL = 7.45 Lower side of bottom slab	○	○	○	○	○	-	○	-

The residual water level shall be 2/3 of the difference between upper wing wall height = 13.10 and the OWL = 10.94.
Source: Study team



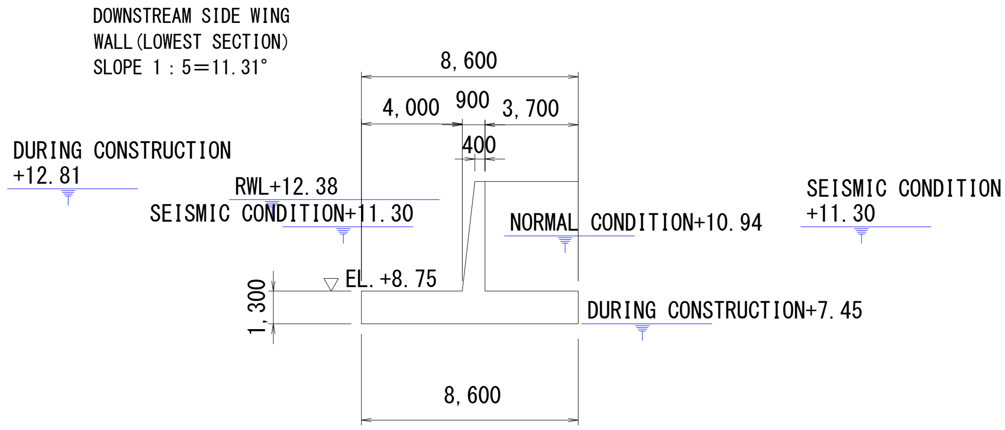


Figure7.4.111 Structural Dimensions of the Downstream Wing Wall

(c) Load Condition

The load conditions used for structural calculation are as follows.

Design Horizontal Seismic Coefficient	: kh = 0.20
Back Earth Pressure	: Coulomb Earth Pressure
Soil Type of the Backfill	: backfill γ= 19.0 kN/m3
	Internal friction angle ϕ = 30 ° , Cohesion c = 0 kN/m2
Surcharge (Normal Condition, During Construction)	: q0 = 10.0 kN/m2
Surcharge (Seismic Condition)	: q0 = 5.0 kN/m2

(d) Calculation Result

(i) Highest Section

The calculation results of the highest section are shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.140 Results of Bending Check of the Highest Section of the Downstream Wing Wall (Base of Vertica Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	577.930	0.000	26.616	4.026	<<	8.280	114.876	<<	168.000	○
Normal Condition 2 (Residual Water Level)	686.596	0.000	26.616	4.783	<<	8.280	136.476	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	896.608	0.000	26.616	6.246	<<	11.012	178.220	<<	223.440	○
Seismic Condition 2 (LWL)	888.612	0.000	26.616	6.190	<<	11.012	176.631	<<	223.440	○
During Construction (During Construction)	654.079	0.000	26.616	4.556	<<	12.420	130.013	<<	252.000	○

Source: Study team

Table7.4.141 Results of Shear Check of the Highest Section of the Downstream Wing Wall (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	213.716	111.000	0.193	≦	0.360	1.600			○
Normal Condition 2 (Residual Water Level)	268.855	111.000	0.242	≦	0.360	1.600			○
Seismic Condition 1 (Zero Water Level)	334.445	111.000	0.301	≦	0.470	2.128			○
Seismic Condition 2 (LWL)	325.106	111.000	0.293	≦	0.470	2.128			○
During Construction (During Construction)	269.977	111.000	0.243	≦	0.540	2.400			○

Source: Study team

B. Toe slab

Table7.4.142 Results of Bending Check of the Highest Section of the Downstream Wing Wall (Toe Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	574.427	29.421	3.783	<<	8.280	90.937	<<	168.000	○
Normal Condition 2 (residual water level)	681.401	29.421	4.488	<<	8.280	107.872	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1163.018	29.421	7.660	<<	11.012	184.116	<<	223.440	○
Seismic Condition 2 (LWL)	1120.189	29.421	7.378	<<	11.012	177.335	<<	223.440	○
During construction (during construction)	708.371	29.421	4.665	<<	12.420	112.141	<<	252.000	○

Source: Study team

Table7.4.143 Results of Shear Check of the Highest Section of the Downstream Wing Wall (Toe 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	247.445	108.000	0.229	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	297.814	108.000	0.276	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	503.354	108.000	0.466	≦	0.470	2.128				×
Seismic Condition 2 (LWL)	485.674	108.000	0.450	≦	0.470	2.128				×
During construction (during construction)	311.672	108.000	0.289	≦	0.540	1.600				○

Source: Study team

Table7.4.144 Results of Shear Check of the Highest Section of the Downstream Wing Wall (Toe Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	298.925	108.000	0.277	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	317.354	108.000	0.294	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	554.834	108.000	0.514	≦	0.470	2.128				×
Seismic Condition 2 (LWL)	516.133	108.000	0.478	≦	0.470	2.128				×
During construction (during construction)	340.728	108.000	0.315	≦	0.540	2.400				○

Source: Study team

The shearing force at the pile position exceeds the allowable shearing stress of concrete alone. Therefore, the shortage is reinforced with "shear reinforcement".

Table7.4.145 Maximum Shear Reinforcing Bar of Downstream Wing Wall

Shear Reinforcement Required (Toe)			
Shear Force		554.8	kN Seismic Condition)
Shear Capacity	Concrete	517.1	kN
	Reinforcing Bar	189.9	kN
	Cross-Sectional Area	113.1	(D12)
	Number Of Trains	2	No. of Bars Per 1.0 m
	Pitch	250	Mm
	Subtotal	707.0	kN

Reinforcing bar capacity (kN) = Aw ・ σ sa ・ d/(1.15 s)
Source: Study team

The reinforcing range shall be the length obtained by adding the effective member length from the pile tip position or more. L = 2.30 + (1.30 -0.22) = 3.40 m. Therefore, the whole area of the toe slab width 3.70 m is used.

C. Heel Slab

Table7.4.146 Results of Checking the Bending of the Highest Section of Downstream Wing Wall (Heel Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	474.316	25.803	3.240	<<	8.280	107.581	<<	168.000	○
Normal Condition 2 (residual water level)	607.404	25.803	4.149	<<	8.280	137.767	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	896.608	25.803	6.125	<<	11.012	203.362	<<	223.440	○
Seismic Condition 2 (LWL)	888.612	25.803	6.070	<<	11.012	201.549	<<	223.440	○
During construction (during construction)	618.611	25.803	4.226	<<	12.420	140.309	<<	252.000	○

Source: Study team

Table7.4.147 Results of Shear Check of the Highest Section of Downstream Wing Wall (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	245.716	121.000	0.203	≦	0.360	1.600				○
Normal Condition 2 (Residual Water Level)	286.013	121.000	0.236	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	491.665	121.000	0.406	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	475.022	121.000	0.393	≦	0.470	2.128				○
During Construction (During Construction)	296.424	121.000	0.245	≦	0.540	2.400				○

Source: Study team

Table7.4.148 Results of Shear Check of Downstream Wing Wall Height (Heel Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-153.195	121.000	0.127	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	-12.521	121.000	0.010	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	115.670	121.000	0.096	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	165.471	121.000	0.137	≦	0.470	2.128				○
During construction (during construction)	-28.836	121.000	0.024	≦	0.540	2.400				○

Source: Study team

(ii) Lowest Section of Wall Height

The calculation results of the lowest section of the wall height are shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.149 Results of Bending Check of the Lowest Section of the Downstream Wing Wall Height (Base of Vertical Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	121.228	0.000	14.985	2.053	<<	8.280	81.393	<<	168.000	○
Normal Condition 2 (residual water level)	158.255	0.000	14.985	2.680	<<	8.280	106.253	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	185.056	0.000	14.985	3.134	<<	11.012	124.246	<<	223.440	○
Seismic Condition 2 (LWL)	177.304	0.000	14.985	3.003	<<	11.012	119.042	<<	223.440	○
During construction (during construction)	197.040	0.000	14.985	3.337	<<	12.420	132.292	<<	252.000	○

Source: Study team

Table7.4.150 Results of Shear Check of the Lowest Section of the Downstream Wing Wall (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	76.182	81.000	0.094	≦	0.360	1.600			○
Normal Condition 2 (residual water level)	97.465	81.000	0.120	≦	0.360	1.600			○
Seismic Condition 1 (Zero Water Level)	117.819	81.000	0.145	≦	0.470	2.128			○
Seismic Condition 2 (LWL)	108.731	81.000	0.134	≦	0.470	2.128			○
During construction (during construction)	132.204	81.000	0.163	≦	0.540	2.400			○

Source: Study team

B. Toe Slab

Table7.4.151 Results of Bending Check of the Lowest Section of the Downstream Wing Wall Height (Toe Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	247.650	19.424	2.415	<<	8.280	99.135	<<	168.000	○
Normal Condition 2 (residual water level)	273.641	19.424	2.669	<<	8.280	109.540	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	518.191	19.424	5.054	<<	11.012	207.434	<<	223.440	○
Seismic Condition 2 (LWL)	475.476	19.424	4.638	<<	11.012	190.335	<<	223.440	○
Normal Condition 1 (Zero Water Level)	247.650	19.424	2.415	<<	8.280	99.135	<<	168.000	○

Source: Study team

Table7.4.152 Results of Shear Check of the Lowest Section of the Downstream Wing Wall (Toe 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	105.368	108.000	0.098	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	119.219	108.000	0.110	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	222.994	108.000	0.206	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	205.364	108.000	0.190	≦	0.470	2.128				○
During construction (during construction)	169.993	108.000	0.157	≦	0.540	2.400				○

Source: Study team

Table7.4.153 Results of Shear Check of the Lowest Section of the Downstream Wing Wall (toe slab pile position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	156.848	108.000	0.145	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	143.654	108.000	0.133	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	274.474	108.000	0.254	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	235.824	108.000	0.218	≦	0.470	2.128				○
During construction (during construction)	199.050	108.000	0.184	≦	0.540	2.400				○

Source: Study team

C. Heel Slab

Table7.4.154 Results of Bending Check of the Lowest Section of the Downstream Wing Wall Height (Heel Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	121.228	15.679	1.372	<<	8.280	82.955	<<	168.000	○
Normal Condition 2 (residual water level)	158.255	15.679	1.791	<<	8.280	108.292	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	185.056	15.679	2.095	<<	11.012	126.631	<<	223.440	○
Seismic Condition 2 (LWL)	177.304	15.679	2.007	<<	11.012	121.327	<<	223.440	○
During construction (during construction)	197.040	15.679	2.230	<<	12.420	134.832	<<	252.000	○

Source: Study team

Table7.4.155 Results of Shear Check of the Lowest Section of the Downstream Wing Wall (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	143.538	121.000	0.119	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	138.960	121.000	0.115	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	254.912	121.000	0.211	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	227.162	121.000	0.188	≦	0.470	2.128				○
During construction (during construction)	185.016	121.000	0.153	≦	0.540	2.400				○

Source: Study team

Table7.4.156 Results of Shear Check of the Lowest Section of the Downstream Wing Wall (Heel Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-147.522	121.000	0.122	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	-56.688	121.000	0.047	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	-18.597	121.000	0.015	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	30.857	121.000	0.026	≦	0.470	2.128				○
During construction (during construction)	-23.005	121.000	0.019	≦	0.540	2.400				○

Source: Study team

(iii) L-Shaped Section

The calculation results of the downstream wing wall L section are shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.157 Results of Checking Bending of the L-shaped Section of the Downstream Wing Wall (Base of Vertical Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	121.228	0.000	15.249	2.095	<<	8.280	81.316	<<	168.000	○
Normal Condition 2 (residual water level)	158.958	0.000	15.249	2.748	<<	8.280	106.624	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	185.056	0.000	15.249	3.199	<<	11.012	124.130	<<	223.440	○
Seismic Condition 2 (LWL)	177.143	0.000	15.249	3.062	<<	11.012	118.822	<<	223.440	○
During construction (during construction)	198.510	0.000	15.249	3.431	<<	12.420	133.155	<<	252.000	○

Source: Study team

Table7.4.158 Results of Shearing Check of the L-shaped Section of the Downstream Wing Wall (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	76.182	81.000	0.094	≦	0.360	1.600			○
Normal Condition 2 (residual water level)	97.853	81.000	0.121	≦	0.360	1.600			○
Seismic Condition 1 (Zero Water Level)	117.819	81.000	0.145	≦	0.470	2.128			○
Seismic Condition 2 (LWL)	108.544	81.000	0.134	≦	0.470	2.128			○
During construction (during construction)	133.290	81.000	0.165	≦	0.540	2.400			○

Source: Study team

B. Heel Slab

Table7.4.159 Results of Checking Bending of the L-shaped Section of the Downstream Wing wall (Heel Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	121.228	15.615	1.413	<<	8.280	84.213	<<	168.000	○
Normal Condition 2 (residual water level)	158.958	15.615	1.853	<<	8.280	110.423	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	185.056	15.615	2.157	<<	11.012	128.552	<<	223.440	○
Seismic Condition 2 (LWL)	177.143	15.615	2.065	<<	11.012	123.056	<<	223.440	○
During construction (during construction)	198.510	15.615	2.314	<<	12.420	137.898	<<	252.000	○

Source: Study team

Table7.4.160 Results of Shearing Check of the L-shaped Section of the Downstream Wing wall (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)			Correction Factor			Evaluation
			Calculated τ	Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	253.992	119.000	0.213 ≦	0.360	1.600				○
Normal Condition 2 (residual water level)	234.215	119.000	0.197 ≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	398.902	119.000	0.335 >	0.470	2.128				×
Seismic Condition 2 (LWL)	344.203	119.000	0.289 ≦	0.470	2.128				○
During construction (during construction)	317.196	119.000	0.267 ≦	0.540	2.400				○

Source: Study team

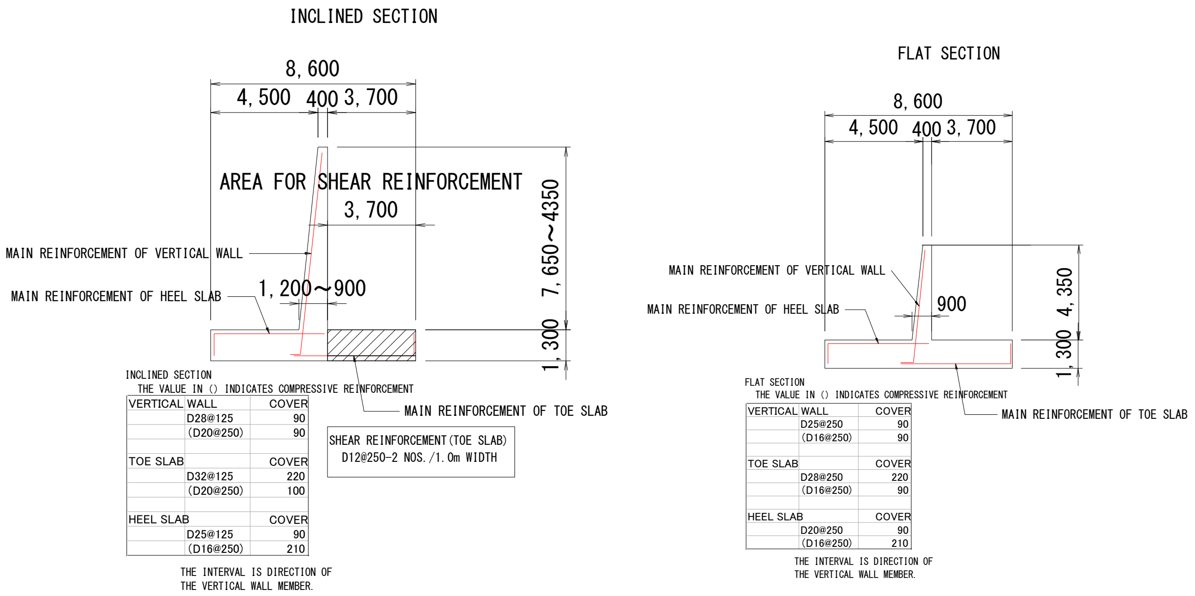
Table7.4.161 Results of Shearing Check of the L-shaped Section of the Downstream Wing Wall (Heel Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)			Correction Factor			Evaluation
			Calculated τ	Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-127.603	119.000	0.107 ≦	0.360	1.600				○
Normal Condition 2 (residual water level)	-27.756	119.000	0.023 ≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	39.413	119.000	0.033 ≦	0.470	2.128				○
Seismic Condition 2 (LWL)	89.029	119.000	0.075 ≦	0.470	2.128				○
During construction (during construction)	15.471	119.000	0.013 ≦	0.540	2.400				○

Source: Study team

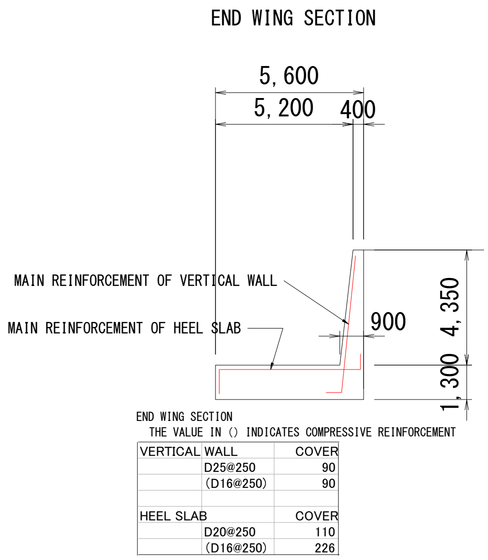
(e) Result of Examination

As a result of the structural calculation, the bar arrangement diagram is shown. **Figure7.4.112** to **Figure7.4.113** shows the bar arrangement of the downstream wing wall. Detailed structural calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



Source: Study team

Figure7.4.112 Downstream Wing Wall Bar Arrangement (1)



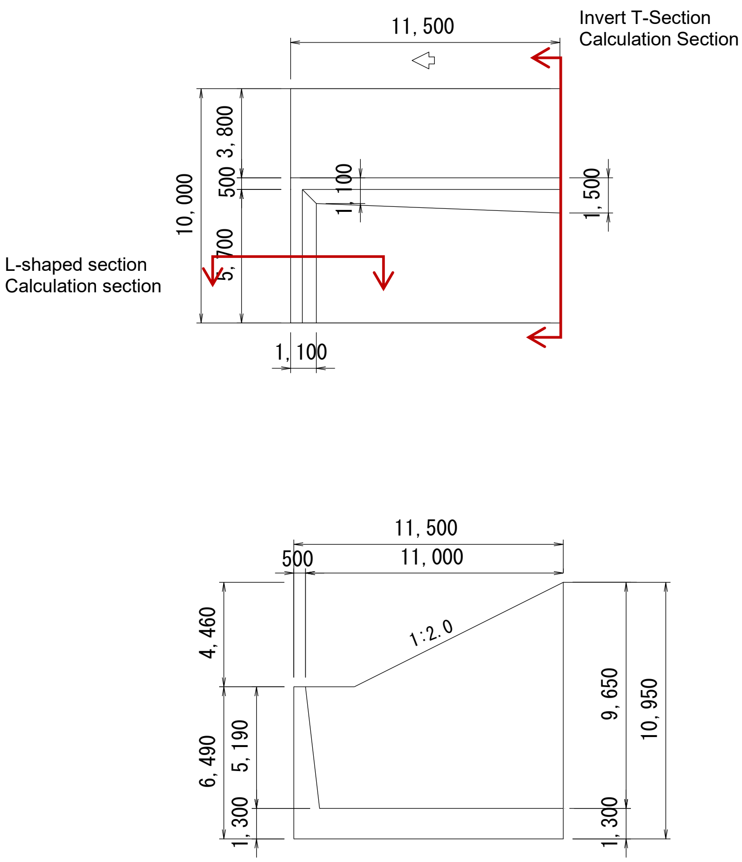
Source: Study team

Figure7.4.113 Downstream Wing Wall Bar Arrangement (2)

6) Upstream Left Bank Wing Wall

(a) Study Policy

The structural dimensions of the upstream left bank wing wall are shown. The upstream left bank wing wall is a flat L-shaped retaining wall whose wall height decreases in the flow direction. The structural calculation is carried out in the cross section shown in the figure below.



Source: Study team

Figure7.4.114 Upstream Left Bank Wing Structural Dimensions

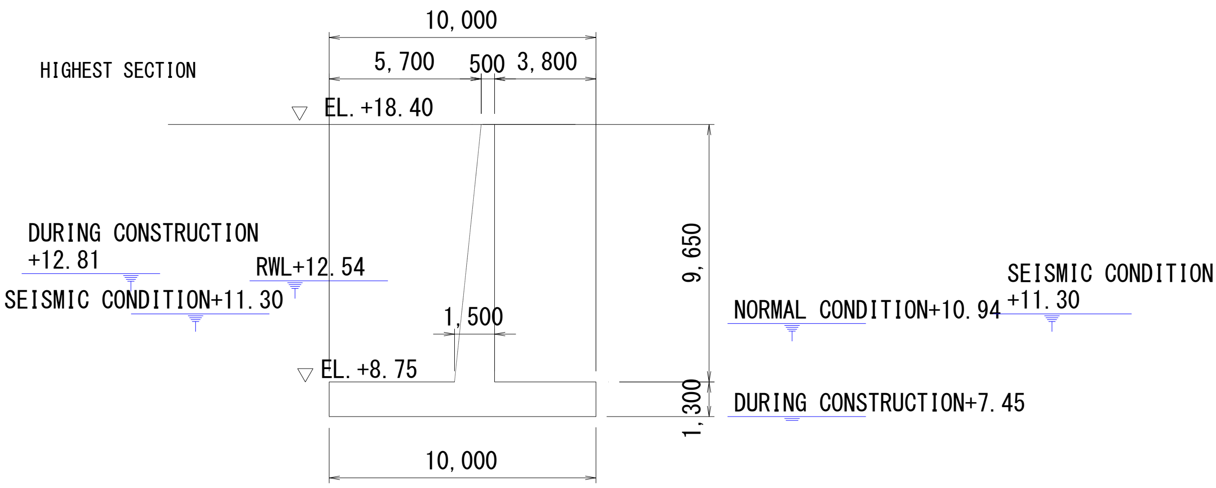
(b) Study Case

The calculation is made for the following cases.

Table7.4.162 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

Calculation Direction	Study Case	Load Condition	Water level Condition		Load Condition							
			Water level in rear side	Water level in front side	Body weight	Water Weight	Earth Pressure	Water pressure	Uplift pressure	Water Weight	Surcharge	Inertial force
Perpendicular Direction To The Flow Flow Direction	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual water level	WL = 12.54 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During Construction	WL = 12.81 Water Level for Cofferdam	WL = 7.45 Lower Side of Bottom Slab	○	○	○	○	○	-	○	-

The residual water level shall be 2/3 of the difference between DFL = 13.340 and the OWL = 10.94.
Source: Study team



Source: Study team

Figure7.4.115 Water Level Condition Diagram of Upstream left bank wing wall

(c) Load Condition

The load conditions used for structural calculation are as follows.

Design Horizontal Seismic Coefficient	: Kh = 0.20
Back Earth Pressure	: Coulomb earth pressure
Soil Type of the Backfill	: Backfill γ = 19.0 kN/m3
	Internal friction angle ϕ = 30 °, Cohesion c = 0 kN/m2
Surcharge (Normal Condition, During Construction)	: q0 = 10.0 kN/m2
Surcharge (Seismic Condition)	: q0 = 5.0 kN/m2

(d) Calculation Result

(i) Highest Section

The calculation results of the highest section are shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.163 Results of Checking the Bending at the Highest Section of the Upstream Left Bank Wing Wall (Base of Vertical Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	1117.920	0.000	34.186	4.845	<<	8.280	134.978	<<	168.000	○
Normal Condition 2 (Residual Water Level)	1162.715	0.000	34.186	5.039	<<	8.280	140.387	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1765.596	0.000	34.186	7.652	<<	11.012	213.179	<<	223.440	○
Seismic Condition 2 (LWL)	1757.308	0.000	34.186	7.617	<<	11.012	212.178	<<	223.440	○
During Construction (During Construction)	1194.084	0.000	34.186	5.175	<<	12.420	144.174	<<	252.000	○

Source: Study team

Table7.4.164 Results of Shear Check at the Highest Section of Upstream Left Bank Wing Wall (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	331.245	140.000	0.237	≦	0.360	1.600			○
Normal Condition 2 (residual water level)	356.791	140.000	0.255	≦	0.360	1.600			○
Seismic Condition 1 (Zero Water Level)	524.940	140.000	0.375	≦	0.470	2.128			○
Seismic Condition 2 (LWL)	515.235	140.000	0.368	≦	0.470	2.128			○
During construction (during construction)	387.529	140.000	0.277	≦	0.540	2.400			○

Source: Study team

B. Toe Slab

Table7.4.165 Results of Checking the Bending at the Highest Section of the Upstream Left Bank Wing Wall (Toe Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	666.923	32.151	3.963	<<	8.280	84.141	<<	168.000	○
Normal Condition 2 (residual water level)	698.730	32.151	4.152	<<	8.280	88.154	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1506.895	32.151	8.954	<<	11.012	190.115	<<	223.440	○
Seismic Condition 2 (LWL)	1474.749	32.151	8.763	<<	11.012	186.060	<<	223.440	○
During construction (during construction)	777.820	32.151	4.622	<<	12.420	98.133	<<	252.000	○

Source: Study team

Table7.4.166 Results of Shear Check at the Highest Section of Upstream Left Bank Wing Wall (Toe 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	173.380	108.000	0.161	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	202.573	108.000	0.188	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	464.564	108.000	0.430	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	467.542	108.000	0.433	≦	0.470	2.128				○
During construction (during construction)	226.526	108.000	0.210	≦	0.540	2.400				○

Source: Study team

Table7.4.167 Results of Shear Check at the Highest Section of Upstream Left Bank Wing Wall (Toe Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	234.220	108.000	0.217	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	231.920	108.000	0.215	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	525.404	108.000	0.486	≦	0.470	2.128				×
Seismic Condition 2 (LWL)	503.539	108.000	0.466	≦	0.470	2.128				○
During construction (during construction)	265.087	108.000	0.245	≦	0.540	2.400				○

Source: Study team

The allowable shearing stress exceeds the shearing force at the pile position with only concrete. Therefore, the shortage is reinforced with "shear reinforcement".

Table7.4.168 Shear Reinforcement For the Highest Section of Upstream Left Bank Wing Wall (Toe Slab)

Shear Reinforcement Required (Toe)			
Shear Force		554.8	kN (Seismic Condition)
Shear Capacity	Concrete	517.1	kN
	Reinforcing Bar	189.9	kN
	Cross-Sectional Area	113.1	(D12)
	Number Of Trains	2	No. of trains per 1.0 m
	Pitch	250	mm
	Subtotal	707.0	kN

Reinforcing bar capacity (kN) = Aw ・ σ sa ・ d/(1.15 s)
Source: Study team

The reinforcing range shall be the length obtained by adding the effective member length from the pile tip position or more. L = 2.60 + (1.30 -0.22) = 3.68 m. Therefore, the whole area of the toe slab width 3.80 m is used.

C. Heel Slab

Table7.4.169 Results of Checking the Bending at the Highest Section of the Upstream Left Bank Wing (Heel Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	673.785	34.373	3.560	<<	8.280	77.946	<<	168.000	○
Normal Condition 2 (residual water level)	766.309	34.373	4.049	<<	8.280	88.649	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1765.596	34.373	9.328	<<	11.012	204.250	<<	223.440	○
Seismic Condition 2 (LWL)	1757.308	34.373	9.284	<<	11.012	203.291	<<	223.440	○
During construction (during construction)	858.825	34.373	4.537	<<	12.420	99.352	<<	252.000	○

Source: Study team

Table7.4.170 Results of Shear Check At the Highest Section of Upstream Left Bank Wing Wall (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	281.213	118.000	0.238	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	298.780	118.000	0.253	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	704.810	118.000	0.597	≦	0.470	2.128				×
Seismic Condition 2 (LWL)	694.747	118.000	0.589	≦	0.470	2.128				×
During construction (during construction)	333.241	118.000	0.282	≦	0.540	2.400				○

Source: Study team

The allowable shearing stress exceeds the shearing force at the pile position with only concrete. Therefore, the shortage is reinforced with "shear reinforcement".

Table7.4.171 Shear Reinforcement For the Highest Section of Upstream Left Bank Wing Wall (Toe Slab)

Shear Reinforcement Required (Heel)			
Shear Force		704.8	kN (Seismic Condition)
Shear Capacity	Concrete	565	kN
	Reinforcing Bar	207.4	kN
	Cross-Sectional Area	113.1	(D12)
	Number Of Trains	2	No. of trains per 1.0 m
	Pitch	250	mm
	Subtotal	772.4	kN

Reinforcing bar capacity (kN) = Aw ・ σsa ・ d/(1.15 s)

Source: Study team

Shear from the base of the vertical wall at the first pile position 1 is OK. Therefore, the reinforcing range is set to be longer than the length obtained by adding the effective member length from the first pile. L = 1.90 + (1.30 -0.10) = 3.10 m. Therefore, the range up to the center of the vertical wall which is 3.850 m is set to the area to be reinforced.

Table7.4.172 Results of Shear Check of Upstream Left Bank Wing Wall(Heel Slab Pile Position 1)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-26.245	118.000	0.022	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	44.669	118.000	0.038	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	410.218	118.000	0.348	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	444.575	118.000	0.377	≦	0.470	2.128				○
During construction (during construction)	66.590	118.000	0.056	≦	0.540	2.400				○

Source: Study team

Table7.4.173 Results of Shear Check of Upstream Left Bank Wing Wall(Heel Slab Pile Position 2)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-186.735	118.000	0.158	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	-112.202	118.000	0.095	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	115.909	118.000	0.098	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	158.891	118.000	0.135	≦	0.470	2.128				○
During construction (during construction)	-97.999	118.000	0.083	≦	0.540	2.400				○

Source: Study team

(ii) L-shaped section

The calculation result of the L-shaped section is shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.174 Results of Checking the Bending of the L-Shaped Section of the Upstream Left Bank Wing (Base of Vertical Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	219.539	0.000	18.732	2.382	<<	8.280	94.159	<<	168.000	○
Normal Condition 2 (residual water level)	261.878	0.000	18.732	2.842	<<	8.280	112.317	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	469.466	0.000	18.732	5.094	<<	11.012	201.350	<<	223.440	○
Seismic Condition 2 (LWL)	464.376	0.000	18.732	5.039	<<	11.012	199.168	<<	223.440	○
During construction (during construction)	295.283	0.000	18.732	3.204	<<	8.280	126.645	<<	168.000	○

Source: Study team

Table7.4.175 Results of Shearing Check of the L-Shaped Section of the Upstream Left Bank Wing (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	126.901	101.000	0.126	≦	0.360	1.600			○
Normal Condition 2 (residual water level)	156.776	101.000	0.155	≦	0.360	1.600			○
Seismic Condition 1 (Zero Water Level)	265.140	101.000	0.263	≦	0.470	2.128			○
Seismic Condition 2 (LWL)	262.198	101.000	0.260	≦	0.470	2.128			○
During construction (during construction)	188.270	101.000	0.186	≦	0.540	2.400			○

Source: Study team

B. Heel Slab

Table7.4.176 Results of Checking the Bending of the L-Shaped Section of the Upstream Left Bank Wing (Heel Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	219.539	18.662	2.208	<<	8.280	102.476	<<	168.000	○
Normal Condition 2 (residual water level)	261.878	18.662	2.634	<<	8.280	122.240	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	469.466	18.662	4.722	<<	11.012	219.138	<<	223.440	○
Seismic Condition 2 (LWL)	464.376	18.662	4.670	<<	11.012	216.762	<<	223.440	○
During construction (during construction)	295.283	18.662	2.970	<<	8.280	137.833	<<	168.000	○

Source: Study team

Table7.4.177 Results of Shearing Check of the L-Shaped Section of the Upstream Left Bank Wing (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	194.396	114.900	0.169	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	185.755	114.900	0.162	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	467.917	114.900	0.407	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	441.062	114.900	0.384	≦	0.470	2.128				○
During construction (during construction)	222.596	114.900	0.194	≦	0.540	2.400				○

Source: Study team

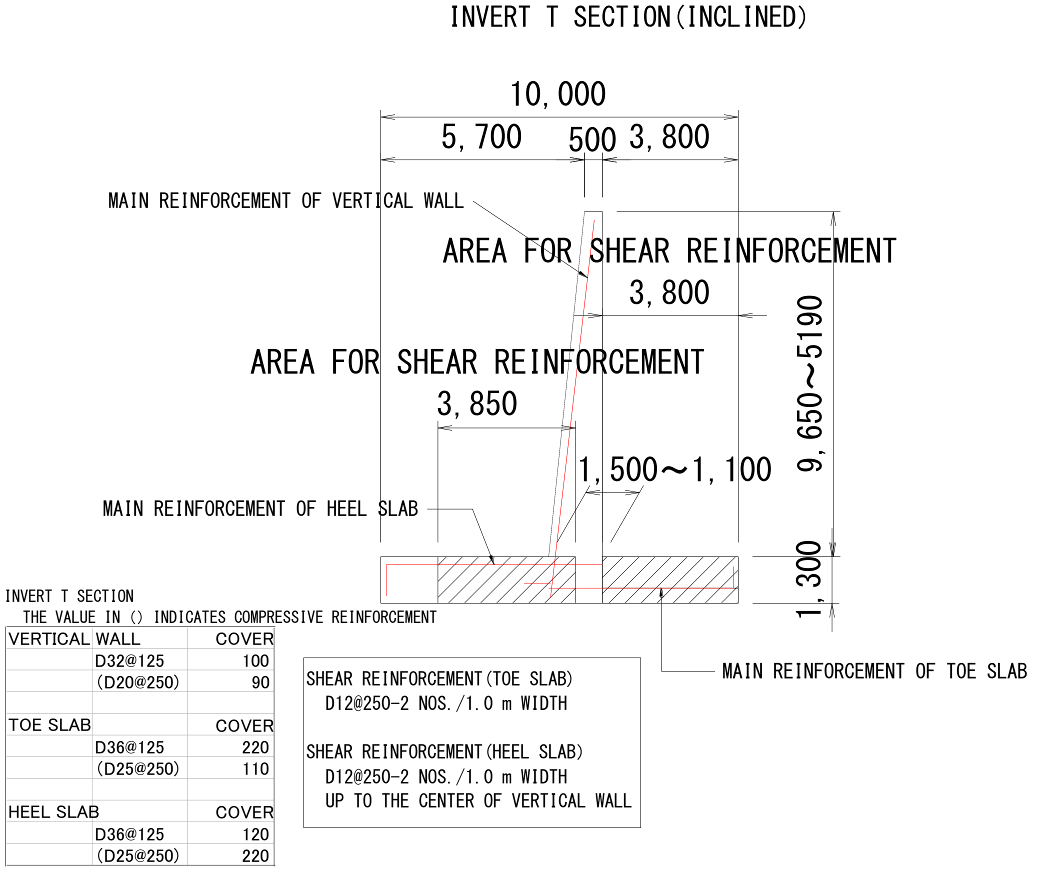
Table7.4.178 Results of Shearing Check of the L-Shaped Section of the Upstream Left Bank Wing (Heel Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-30.487	114.900	0.027	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	32.574	114.900	0.028	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	396.832	114.900	0.345	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	430.281	114.900	0.374	≦	0.471	2.128				○
During construction (during construction)	41.895	114.900	0.036	≦	0.540	2.400				○

Source: Study team

(e) Result of Examination

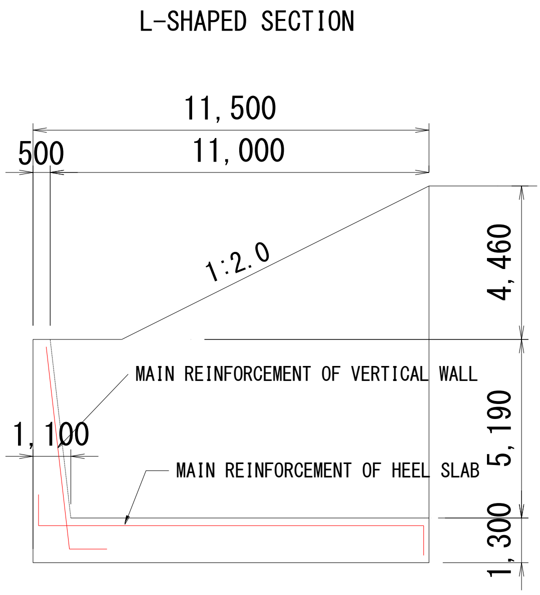
As a result of the structural calculation, the bar arrangement diagram is shown. **Figure7.4.116** to **Figure7.4.117** shows the bar arrangement of the upstream left bank wing wall. Detailed structural calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



VERTICAL	WALL	COVER
	D32@125	100
	(D20@250)	90
TOE SLAB		COVER
	D36@125	220
	(D25@250)	110
HEEL SLAB		COVER
	D36@125	120
	(D25@250)	220

Source: Study team

Figure7.4.116 Bar Arrangement of Upstream Left Bank Section (Inverted T Section)



VERTICAL WALL		COVER
	D28@250	100
	(D16@250)	90
HEEL SLAB		COVER
	D25@250	151
	(D16@250)	241

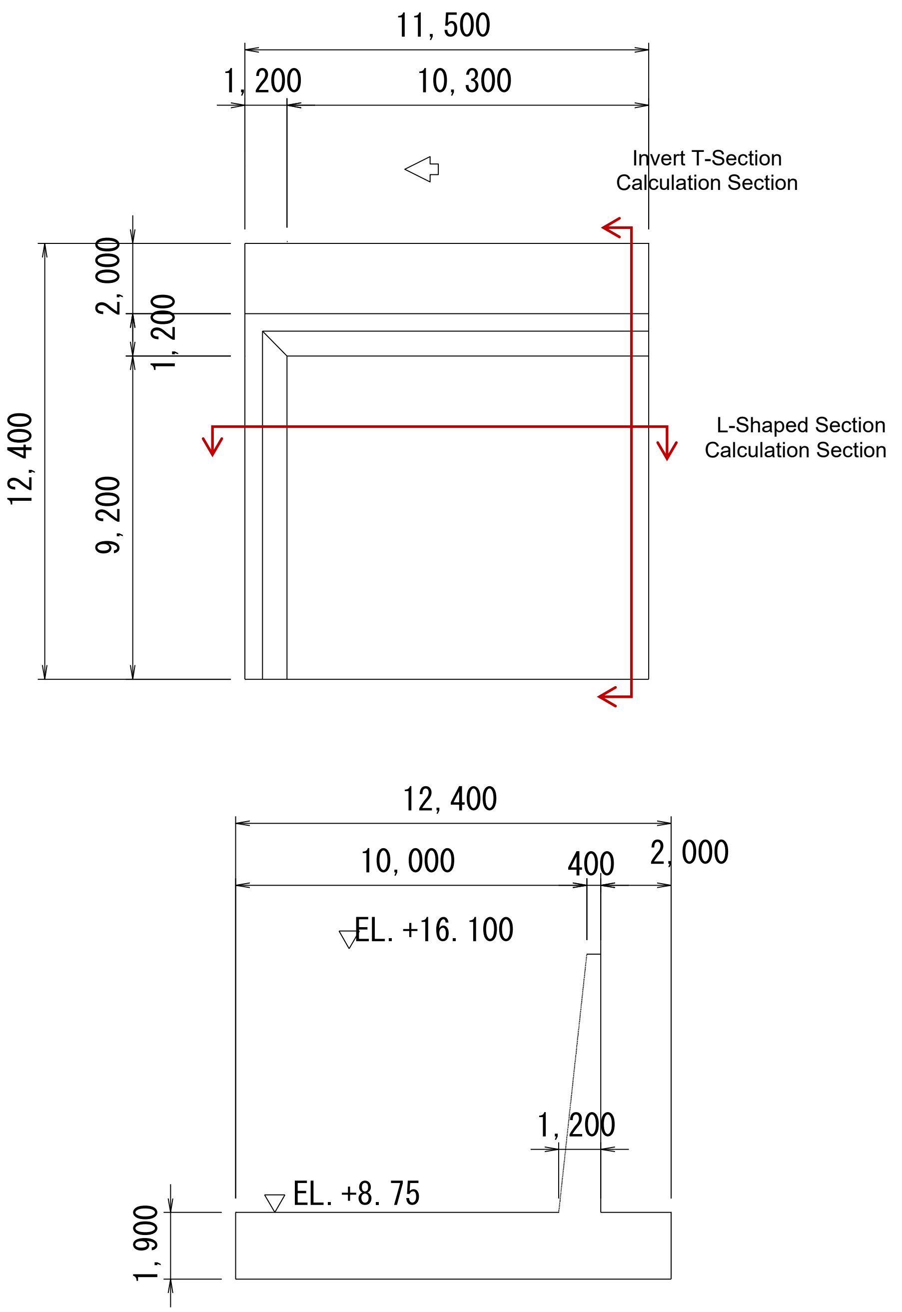
Source: Study team

Figure7.4.117 Bar Arrangement of Upstream Left Bank Section (L-Shaped Section)

7) Upstream Right Bank Wing Wall

(a) Study Policy

The structural dimensions of the upstream right bank wing wall are shown. The upstream right bank wing wall has the same wall height both in the flow direction and in the Perpendicular Direction to the Flow, and has a substantially square planar shape. The structure calculation is carried out in the cross section of the inverted T part and the L part shown in the figure below.



Source: Study team

Figure7.4.118 Upstream Right Bank Wing Wall Structural Dimensions

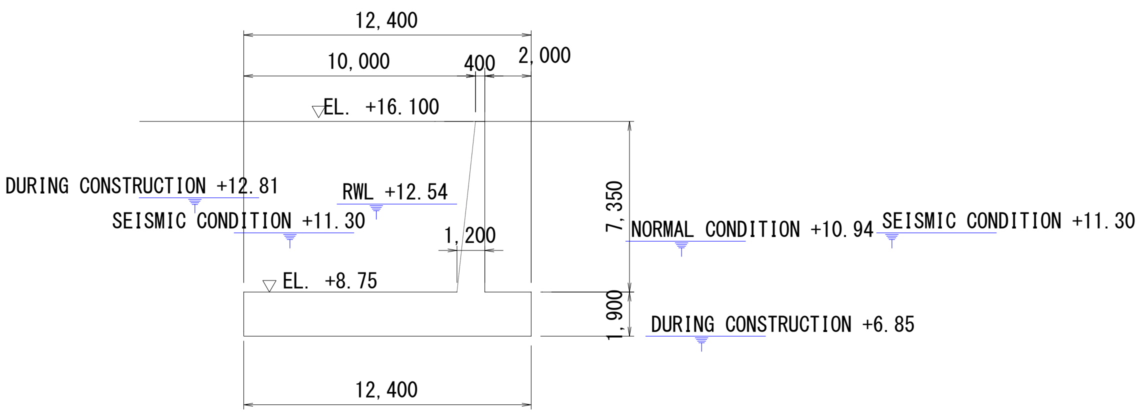
(b) Study Case

The calculation is made for the following cases.

Table7.4.179 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

Calculation Direction	Study Case	Load Condition	Water Level Condition		Load Condition							
			Water Level in Rear Side	Water Level in Front Side	Body weight	Water Weight	Earth Pressure	Water pressure	Uplift pressure	Water Weight	Surcharge	Inertial Force
Perpendicular Direction To The Flow Flow Direction	1	Normal Condition Zero Water Level	------	------	○	○	○	-	-	-	○	-
	2	Normal Condition Residual water level	WL = 12.54 Residual water level	WL = 10.94 OWL	○	○	○	○	○	○	○	-
	3	Seismic Condition Zero Water Level	------	------	○	○	○	-	-	-	○	○
	4	Seismic Condition LWL	WL = 11.30 LWL	WL = 11.30 LWL	○	○	○	○	○	○	○	○
	5	During Construction	WL = 12.81 Water Level for Cofferdam	WL = 6.85 Lower Side of Bottom Slab	○	○	○	○	○	-	○	-

The residual water level shall be 2/3 of the difference between DFL = 13.340 and the OWL = 10.94.
Source: Study team



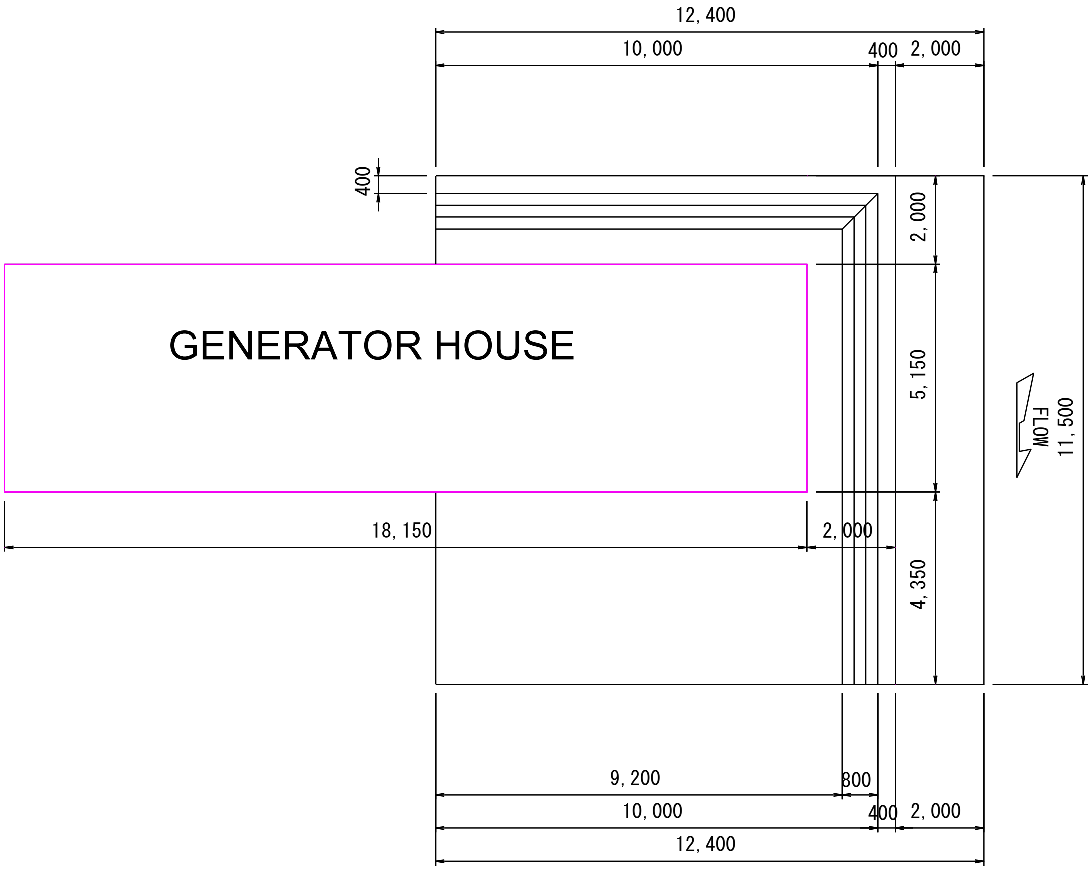
Source: Study team

Figure7.4.119 Upstream Right Bank Wing Wall Water Level Condition

(c) Load Condition

The load conditions acting on the pile foundation are as follows. Since the generator house is located on the wing wall of the upstream right bank, the calculation is performed in consideration of the load (Approximately 47 kN/m2 rounded to 50.0 kN/m2) of the generator house.

Design Horizontal Seismic Coefficient	: Kh = 0.20
Back Earth Pressure	: Coulomb earth pressure *As for the water flow direction, since the load on the top does not become the uniform load, the trial wedge pressure is adopted.
Soil Type of the Backfill	: Backfill γ = 19.0 kN/m3 Internal friction angle ϕ = 30 °, Cohesion c = 0 kN/m2
Surcharge (Normal Condition, During Construction)	: q0 = 10.0 kN/m2
Surcharge (Seismic Condition)	: q0 = 5.0 kN/m2
Surcharge (Generator House)	: q0 = 50.0 kN/m2 (Normal and Seismic Condition)



Source: Study team

Figure7.4.120 Dimensions of the Generator House

(d) Calculation Result

(i) Invert T section

The calculation result of the invert T section is shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.180 Results of Check of Invert T section Bending of Upstream Right Bank Wing Wall (Base of Vertical Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	682.271	0.000	26.616	4.753	<<	8.280	135.616	<<	168.000	○
Normal Condition 2 (residual water level)	726.952	0.000	26.616	5.064	<<	8.280	144.498	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1084.581	0.000	26.616	7.555	<<	11.012	215.584	<<	223.440	○
Seismic Condition 2 (LWL)	1076.029	0.000	26.616	7.496	<<	11.012	213.884	<<	223.440	○
During construction (during construction)	758.285	0.000	26.616	5.282	<<	12.420	150.726	<<	252.000	○

Source: Study team

Table7.4.181 Results of Shearing Check of Invert T Section of Upstream Right Bank Wing Wall (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	243.694	111.000	0.220	≦	0.36	1.600			○
Normal Condition 2 (residual water level)	269.135	111.000	0.242	≦	0.36	1.600			○
Seismic Condition 1 (Zero Water Level)	387.929	111.000	0.349	≦	0.47	2.128			○
Seismic Condition 2 (LWL)	377.963	111.000	0.341	≦	0.47	2.128			○
During construction (during construction)	299.857	111.000	0.270	≦	0.54	2.400			○

Source: Study team

B. Toe Slab

Table7.4.182 Results of Check of Invert T Section Bending of Upstream Right Bank Wing Wall (Toe Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	202.892	18.114	1.322	<<	8.280	99.134	<<	168.000	○
Normal Condition 2 (residual water level)	204.527	18.114	1.333	<<	8.280	99.932	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	420.627	18.114	2.741	<<	11.012	205.519	<<	223.440	○
Seismic Condition 2 (LWL)	403.299	18.114	2.628	<<	11.012	197.052	<<	223.440	○
During construction (during construction)	235.807	18.114	1.537	<<	12.420	115.216	<<	252.000	○

Source: Study team

Table7.4.183 Results of Shearing Check of Invert T Section of Upstream Right Bank Wing Wall (Toe 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-47.880	169.000	0.028	≦	0.36	1.600				○
Normal Condition 2 (residual water level)	-27.632	169.000	0.016	≦	0.36	1.600				○
Seismic Condition 1 (Zero Water Level)	-47.880	169.000	0.028	≦	0.47	2.128				○
Seismic Condition 2 (LWL)	-28.330	169.000	0.017	≦	0.47	2.128				○
During construction (during construction)	-45.283	169.000	0.027	≦	0.54	2.400				○

Source: Study team

C. Heel Slab

Table7.4.184 Results of bending check of invert T section of upstream right bank wing wall (heel slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2) Calculated		Compressive Stress (N/mm2) Allowance	Tensile Stress (N/mm2) Calculated		Tensile Stress (N/mm2) Allowance	Evaluation
Normal Condition 1 (Zero Water Level)	682.271	32.192	2.455	<<	8.280	102.119	<<	168.000	○
Normal Condition 2 (residual water level)	726.952	32.192	2.615	<<	8.280	108.807	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1084.581	32.192	3.902	<<	11.012	162.335	<<	223.440	○
Seismic Condition 2 (LWL)	1076.029	32.192	3.871	<<	11.012	161.055	<<	223.440	○
During construction (during construction)	758.285	32.192	2.728	<<	12.420	113.497	<<	252.000	○

Source: Study team

Table7.4.185 Results of shear check of invert T section of upstream right bank wing wall (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2) Calculated τ		Shear Stress (N/mm2) Allowable τa1	Shear Stress (N/mm2) Allowable τa2	Correction Factor Ce	Correction Factor Cpt	Correction Factor Cdc	Evaluation
Normal Condition 1 (Zero Water Level)	184.464	181.000	0.102	≦	0.36	1.600				○
Normal Condition 2 (residual water level)	204.240	181.000	0.113	≦	0.36	1.600				○
Seismic Condition 1 (Zero Water Level)	620.801	181.000	0.343	≦	0.47	2.128				○
Seismic Condition 2 (LWL)	608.518	181.000	0.336	≦	0.47	2.128				○
During construction (during construction)	250.648	181.000	0.138	≦	0.54	2.400				○

Source: Study team

Table7.4.186 Results of shear check of invert T section of upstream right bank wing wall (Heel slab pile position 2)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm)	Shear Stress (N/mm2) Calculated τ		Shear Stress (N/mm2) Allowable τa1	Shear Stress (N/mm2) Allowable τa2	Correction Factor Ce	Correction Factor Cpt	Correction Factor Cdc	Evaluation
Normal Condition 1 (Zero Water Level)	-123.399	181.000	0.068	≦	0.36	1.600				○
Normal Condition 2 (residual water level)	-41.039	181.000	0.023	≦	0.36	1.600				○
Seismic Condition 1 (Zero Water Level)	374.597	181.000	0.207	≦	0.47	2.128				○
Seismic Condition 2 (LWL)	418.056	181.000	0.231	≦	0.47	2.128				○
During construction (during construction)	-18.680	181.000	0.010	≦	0.54	2.400				○

Source: Study team

(ii) L-shaped Section

The calculation result of the L-shaped section is shown below. Of the calculation results, the case surrounded by the red line is the decision case.

A. Base of Vertical Wall

Table7.4.187 Result of Bending Check of L-Shaped Section of Upstream Right Bank Wing Wall (Base of Vertical Wall)

Load State (Water Level)	M (kNm)	N (Kn)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
				Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	692.875	0.000	26.616	4.827	<<	8.280	137.724	<<	168.000	○
Normal Condition 2 (residual water level)	712.110	0.000	26.616	4.961	<<	8.280	141.547	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1105.996	0.000	26.616	7.705	<<	11.012	219.841	<<	223.440	○
Seismic Condition 2 (LWL)	1080.940	0.000	26.616	7.530	<<	11.012	214.860	<<	223.440	○
During construction (during construction)	741.903	0.000	26.616	5.168	<<	12.420	147.469	<<	252.000	○

Source: Study team

Table7.4.188 Results of Shear Check of L-Shaped Section of Upstream Right Bank Wing Wall (Base of Vertical Wall)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor		Evaluation
			Calculated τ		Allowable τ a1	Allowable τ a2	Ce	Cpt	
Normal Condition 1 (Zero Water Level)	282.806	111.000	0.255	≦	0.36	1.600			○
Normal Condition 2 (residual water level)	308.249	111.000	0.278	≦	0.36	1.600			○
Seismic Condition 1 (Zero Water Level)	444.371	111.000	0.400	≦	0.47	2.128			○
Seismic Condition 2 (LWL)	434.144	111.000	0.391	≦	0.47	2.128			○
During construction (during construction)	338.972	111.000	0.305	≦	0.54	2.400			○

Source: Study team

B. Heel Slab

Table7.4.189 Result of Bending Check of L-Shaped Section of Upstream Right Bank Wing Wall (Heel Slab)

Load State (Water Level)	M (kNm)	X (Cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition 1 (Zero Water Level)	692.875	31.960	2.550	<<	8.280	105.223	<<	168.000	○
Normal Condition 2 (residual water level)	712.110	31.960	2.621	<<	8.280	108.144	<<	168.000	○
Seismic Condition 1 (Zero Water Level)	1105.996	31.960	4.070	<<	11.012	167.961	<<	223.440	○
Seismic Condition 2 (LWL)	1080.940	31.960	3.978	<<	11.012	164.156	<<	223.440	○
Normal Condition 1 (Zero Water Level)	692.875	31.960	2.550	<<	8.280	105.223	<<	168.000	○

Source: Study team

Table7.4.190 Results of shear check of L-shaped section of upstream right bank wing wall (Heel 1/2 H Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	131.877	178.500	0.074	≦	0.36	1.600				○
Normal Condition 2 (residual water level)	139.993	178.500	0.078	≦	0.36	1.600				○
Seismic Condition 1 (Zero Water Level)	413.397	178.500	0.232	≦	0.47	2.128				○
Seismic Condition 2 (LWL)	398.106	178.500	0.223	≦	0.47	2.128				○
During construction (during construction)	176.263	178.500	0.099	≦	0.54	2.400				○

Source: Study team

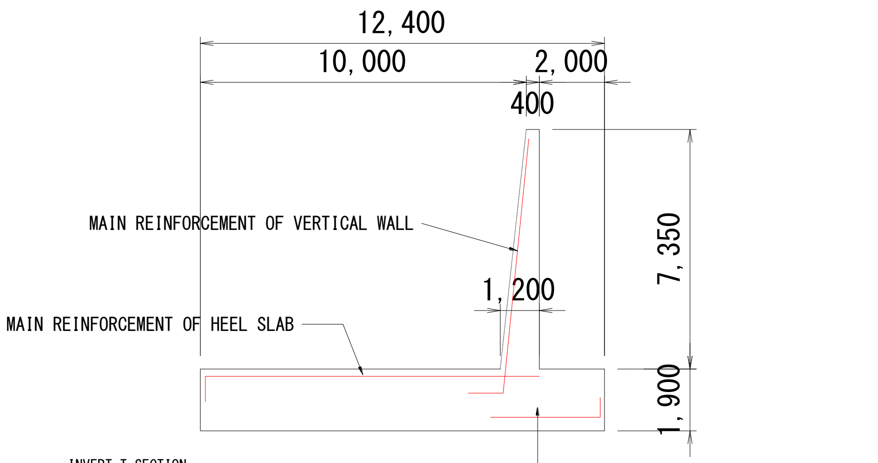
Table7.4.191 Results of Shear Check of L-Shaped Section of Upstream Right Bank Wing Wall (Heel Slab Pile Position)

Load State (Water Level)	Shear Force Sh (kN)	Effective Height D (cm	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition 1 (Zero Water Level)	-30.487	114.900	0.027	≦	0.360	1.600				○
Normal Condition 2 (residual water level)	32.574	114.900	0.028	≦	0.360	1.600				○
Seismic Condition 1 (Zero Water Level)	396.832	114.900	0.345	≦	0.470	2.128				○
Seismic Condition 2 (LWL)	430.281	114.900	0.374	≦	0.471	2.128				○
During construction (during construction)	41.895	114.900	0.036	≦	0.540	2.400				○

Source: Study team

(e) Result of Examination

As a result of the structural calculation, the bar arrangement diagram is shown.**Figure7.4.121** to **Figure7.4.122** shows the bar arrangement of the upstream right bank wing wall. Detailed structural calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**.



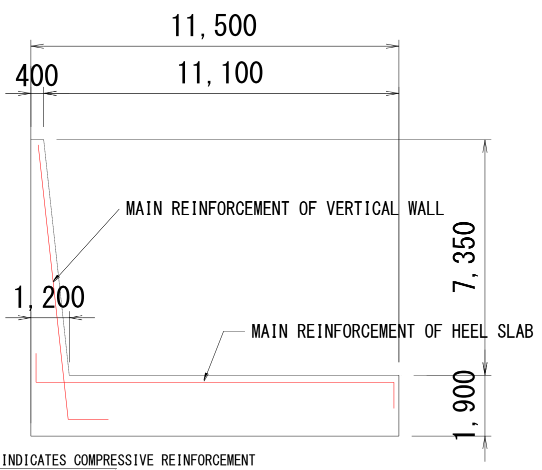
INVERT T SECTION
THE VALUE IN () INDICATES COMPRESSIVE REINFORCEMENT

VERTICAL WALL		COVER
	D28@125	90
	(D20@250)	90
TOE SLAB		COVER
	D20@250	210
	(D16@250)	90
HEEL SLAB		COVER
	D25@125	90
	(D16@250)	210

Source: Study team

Figure7.4.121 Bar Arrangement of Upstream Right Bank Wing Wall (Invert T Section)

L-SHAPED SECTION



L-SHAPED SECTION
THE VALUE IN () INDICATES COMPRESSIVE REINFORCEMENT

VERTICAL WALL		COVER
	D28@125	90
	(D20@250)	90
HEEL SLAB		COVER
	D25@125	115
	(D16@250)	226

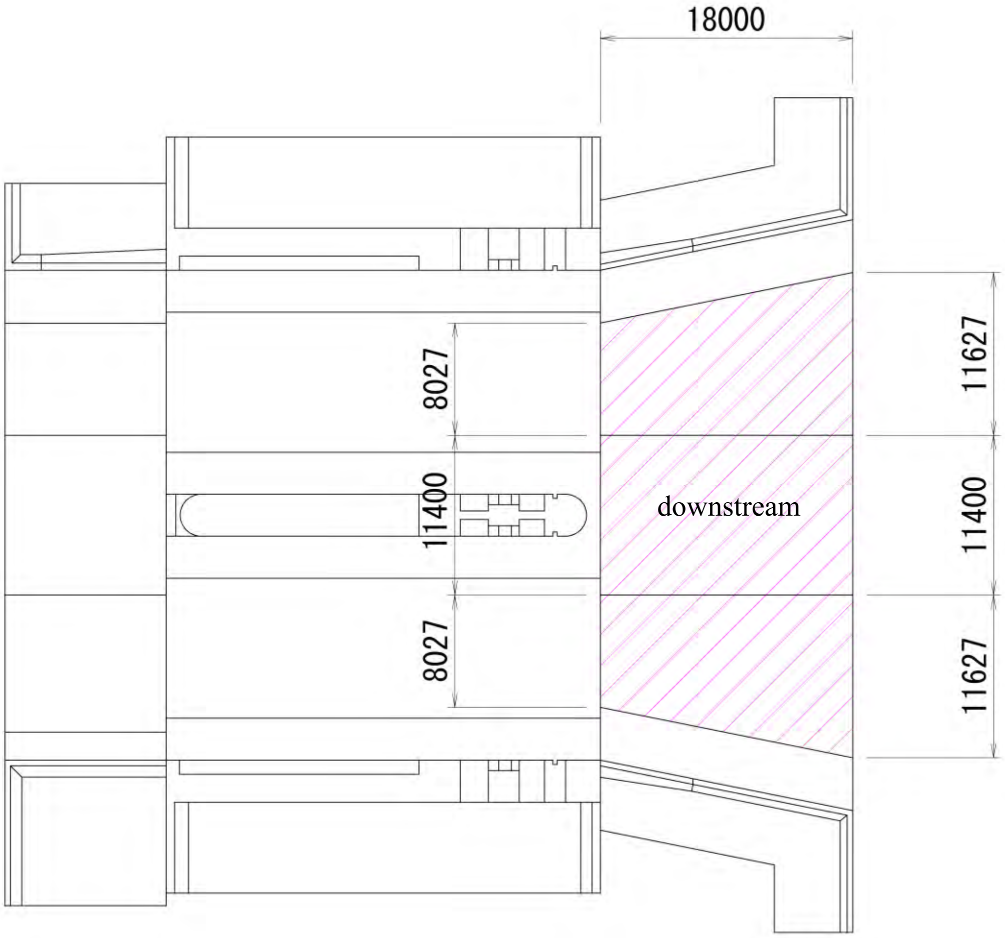
Source: Study team

Figure7.4.122 Bar Arrangement of Upstream Right Bank Wing Wall (L-Shaped Section)

8) Downstream Apron

(a) Study Policy

For downstream apron, the moment and axial force are calculated as a beam supported by a pile. For apron on the downstream side, since the perpendicular direction to the flow is the short side direction, the structure calculation is performed for the perpendicular direction to the flow.



Source: Study team

Figure7.4.123 Downstream Apron

(b) Study Case

The calculation is made for the following cases.

Table7.4.192 Load Case List

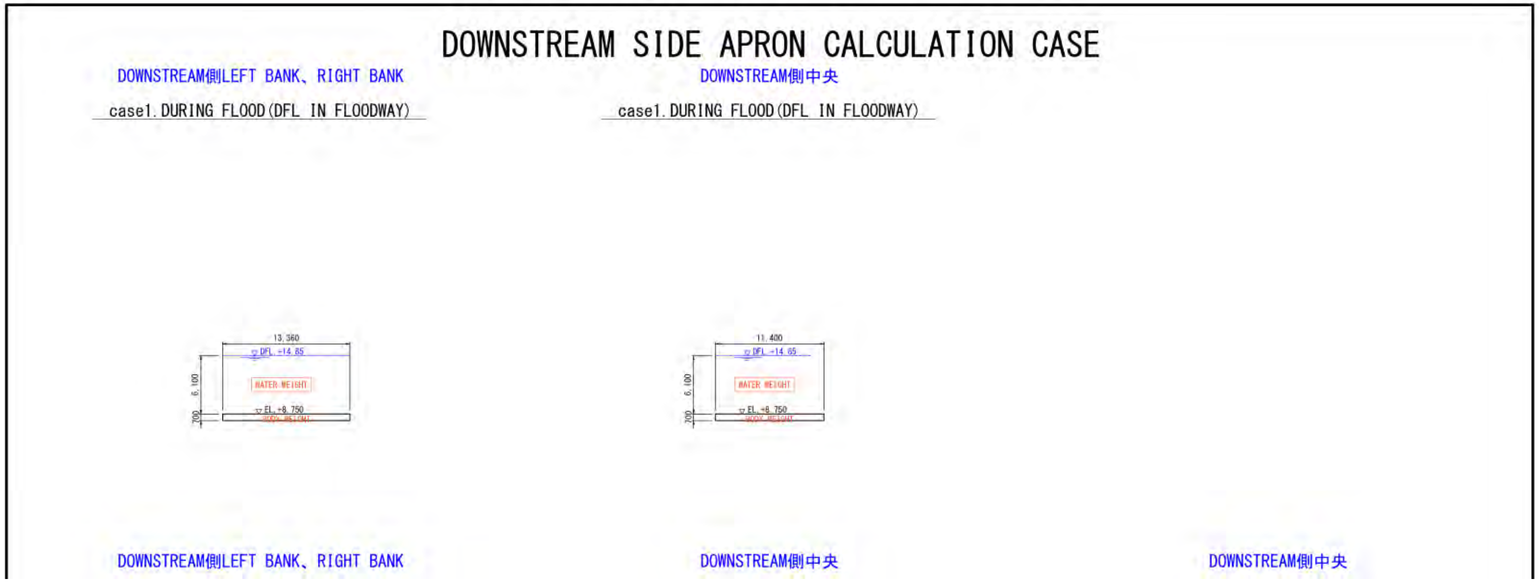
Member	Location	Calculation direction	Case	Case name	Additional factor of allowable stress
Downstream apron	Right and left bank	Perpendicular direction to the flow	1	During floods (at floodway DFL)	1.0
			2	At construction (1)	1.5
			3	At construction (2)	1.5
			4	At construction (3)	1.5
	Center	Perpendicular direction to the flow	1	During floods (at floodway DFL)	1.0
			2	During construction (cofferdam)	1.5
			3	At construction (1)	1.5
			4	At construction (2)	1.5
			5	At construction (3)	1.5

Source: Study team

(c) Load Condition

The load conditions used in the pile foundation calculation are used.

The diagram set on the next page is shown.



Not Presented Due to the Closed Information

Source: Study team

Figure7.4.124 Downstream Apron Load Diagram

(d) Calculation Result

(i) Downstream central apron

The calculation result of the downstream central apron is shown in **Table7.4.193**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.193 List of Bending Stress Check Results of the Downstream Center Apron

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom Slab Member 1 X = 1.100 Case 1 Flood (At Floodway DFL)	1.000 0.0 -30.9 0.0	0.55 < 8.28 0.00 < 168.00	x = -0.214, α= 0 14.85 < 168.00
Bottom Slab Member 2 X = 0.000 Cofferdam In Case 2 Construction	1.500 0.0 -346.9 0.0	6.17 < 12.42 0.00 < 252.00	x = -0.214, α= 0 166.75 < 252.00
Bottom Slab Member 3 X = 4.600 Cofferdam In Case 2 Construction	1.500 0.0 -480.5 0.0	8.54 < 12.42 0.00 < 252.00	x = -0.214, α= 0 230.94 < 252.00
Bottom Slab Member 4 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -46.3 0.0	0.82 < 8.28 0.00 < 168.00	x = -0.214, α= 0 22.27 < 168.00

Source: Study team

(ii) Downstream left and right bank apron

The calculation results of the downstream left and right apron are shown in **Table7.4.194**. Of the calculation results, the case surrounded by the red line is the decision case.

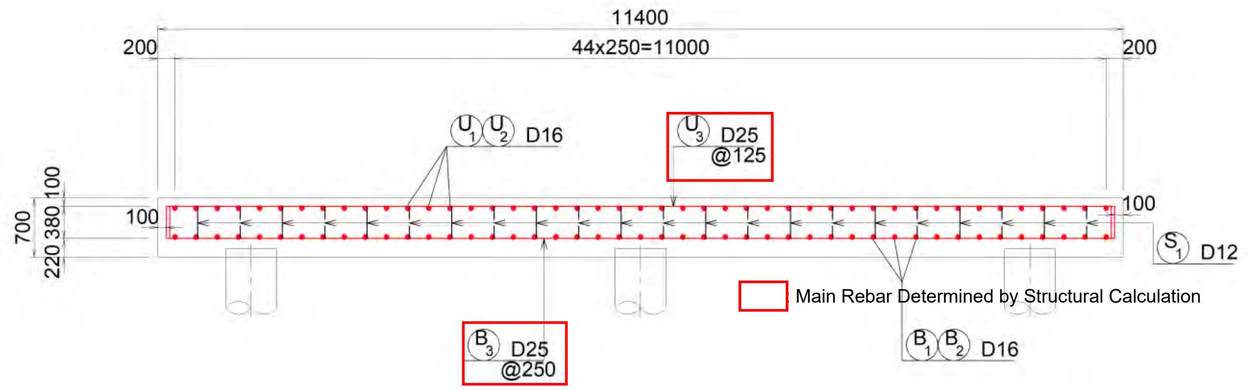
Table7.4.194 List of Bending Stress Check Results of the Downstream Left And Right Apron

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom Slab Member 1 X = 1.100 Case 1 Flood (At Floodway DFL)	1.000 0.0 -30.9 0.0	0.65 < 8.28 0.00 < 168.00	x = -0.182, α= 0 22.48 < 168.00
Bottom Slab Member 2 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -211.3 0.0	4.45 < 8.28 0.00 < 168.00	x = -0.182, α= 0 153.77 < 168.00
Bottom Slab Member 3 X = 4.713 Case 1 Flood (At Floodway DFL)	1.000 0.0 -211.3 0.0	4.45 < 8.28 0.00 < 168.00	x = -0.182, α= 0 153.77 < 168.00
Bottom Slab Member 4 X = 0.000 Case 1 Flood (At Floodway DFL)	1.000 0.0 -46.3 0.0	0.98 < 8.28 0.00 < 168.00	x = -0.182, α= 0 33.72 < 168.00

Source: Study team

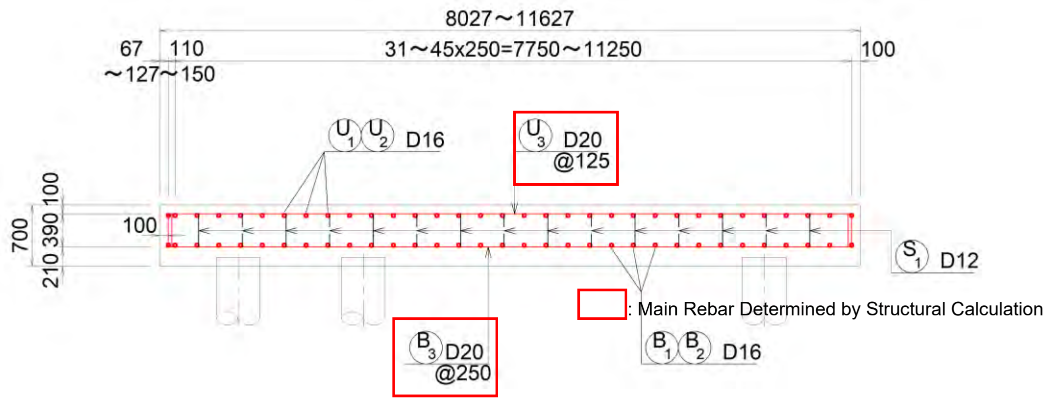
(e) Result of Examination

As the calculation result of the downstream apron, the bar arrangement plan is shown.



Source: Study team

Figure7.4.125 Bar Arrangement of Downstream Center Apron



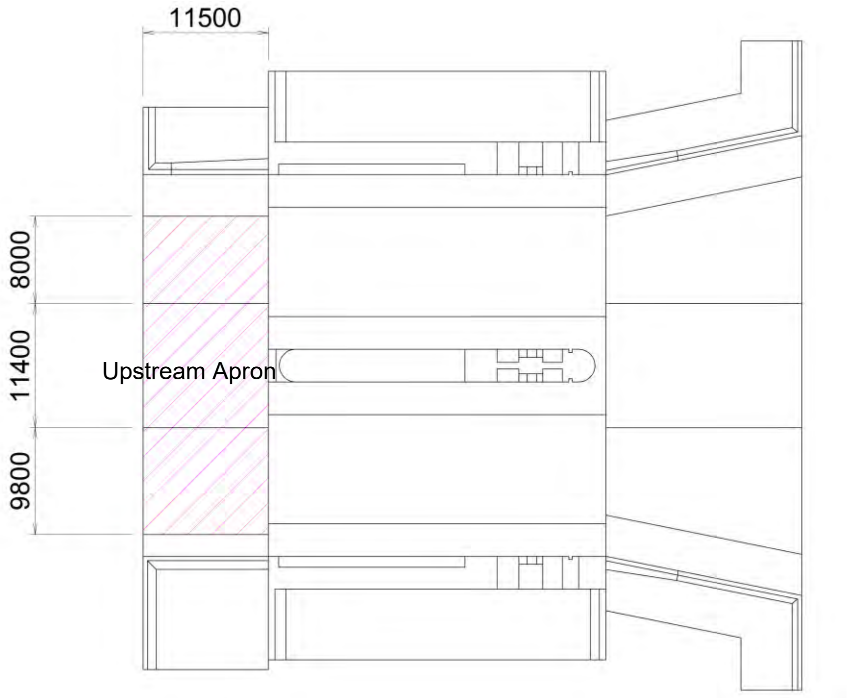
Source: Study team

Figure7.4.126 Bar Arrangement of Downstream Left And Right Apron

9) Upstream Apron

(a) Study Policy

For upstream apron, the moment and axial force are calculated as a beam supported by a pile. For upstream apron, perform structural calculation in the perpendicular direction to the flow.



Source: Study team

Figure7.4.127 Upstream Apron

(b) Study Case

The calculation is made for the following cases.

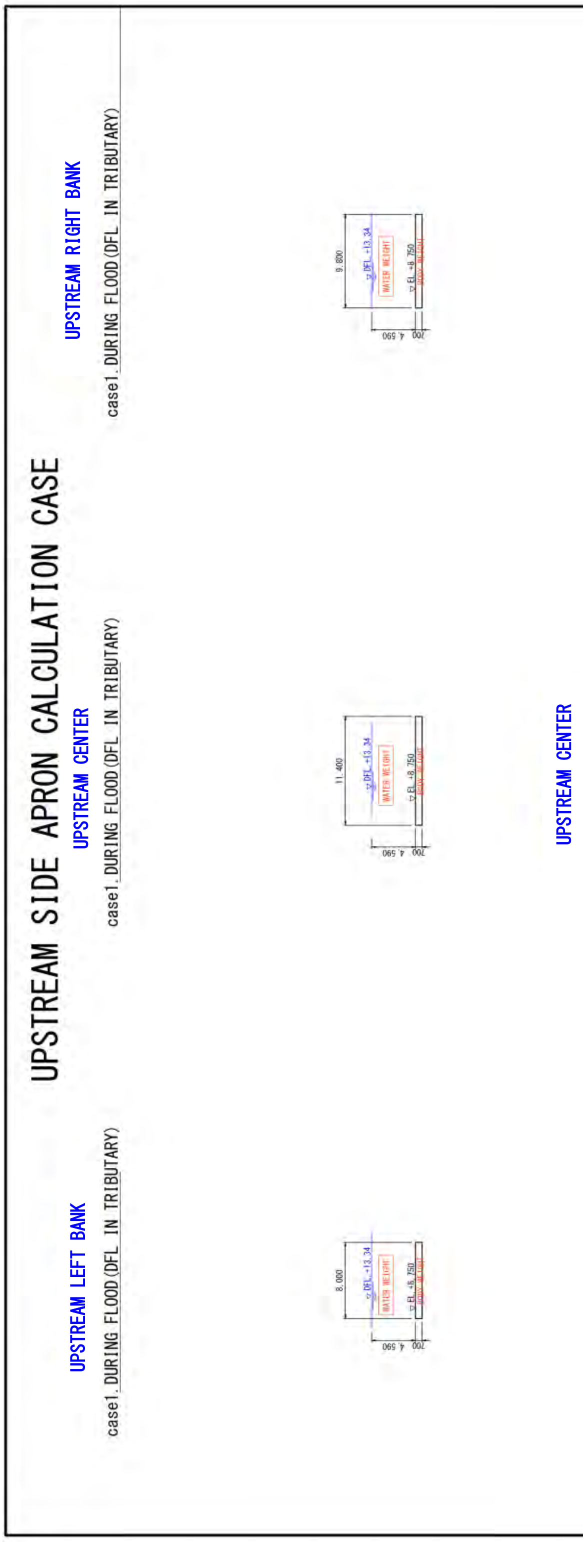
Table7.4.195 Load Case List

Member	Location	Calculation Direction	Case	Case Name	Additional Factor Of Allowable Stress
Upstream Apron	Left Bank	Perpendicular Direction To The Flow	1	During Floods (At Tributary DFL)	1.0
	Center	Perpendicular Direction To The Flow	1	During Floods (At Tributary DFL)	1.0
			2	During Construction (Cofferdam)	1.5
	Right Bank	Perpendicular Direction To The Flow	1	During Floods (At Tributary DFL)	1.0

Source: Study team

(c) Load Condition

The load conditions used in the pile foundation calculation are adopted. The diagram set on the next page is shown.



Not Presentaed Due to the Closed Information

Figure7.4.128 Upstream Apron Load Diagram

Source: Study team

(d) Calculation Result

(i) Upstream Central Apron

The calculation result of the upstream center apron is shown in **Table7.4.196**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.196 List of Bending Stress Check Results of Upstream Center Apron

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom slab member 1 X = 1.100 case 1 flood (at Floodway DFL)	1.000 0.0 -30.9 0.0	0.55 < 8.28 0.00 < 168.00	x = -0.214, α= 0 14.85 < 168.00
Bottom slab member 2 X = 0.000 cofferdam in case 2 construction	1.500 0.0 -346.9 0.0	6.17 < 12.42 0.00 < 252.00	x = -0.214, α= 0 166.75 < 252.00
Bottom slab member 3 X = 4.600 cofferdam in case 2 construction	1.500 0.0 -480.5 0.0	8.54 < 12.42 0.00 < 252.00	x = -0.214, α= 0 230.94 < 252.00
Bottom slab member 4 X = 0.000 case 1 flood (at Floodway DFL)	1.000 0.0 -46.3 0.0	0.82 < 8.28 0.00 < 168.00	x = -0.214, α= 0 22.27 < 168.00

Source: Study team

(ii) Upstream Left Bank Apron

The calculation result of the upstream left bank apron work is shown in **Table7.4.197**. Of the calculation results, the case surrounded by the red line is the decision case.

Table7.4.197 Results of Bending Stress Check For Upstream Left Bank Apron

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom slab member 1 X = 1.100 case 1 flood (at Tributary DFL)	1.000 0.0 -24.9 0.0	0.70 < 8.28 0.00 < 168.00	x = -0.139, α= 0 34.60 < 168.00
Bottom slab member 2 X = 0.000 case 1 flood (at Tributary DFL)	1.000 0.0 -104.0 0.0	2.90 < 8.28 0.00 < 168.00	x = -0.139, α= 0 144.46 < 168.00
Bottom slab member 3 X = 3.800 case 1 flood (at Tributary DFL)	1.000 0.0 -104.0 0.0	2.90 < 8.28 0.00 < 168.00	x = -0.139, α= 0 144.46 < 168.00
Bottom slab member 4 X = 0.000 case 1 flood (at Tributary DFL)	1.000 0.0 -37.4 0.0	1.04 < 8.28 0.00 < 168.00	x = -0.139, α= 0 51.89 < 168.00

Source: Study team

(iii) Upstream Right Bank Apron

The calculation result of the upstream right apron work is shown in **Table7.4.198**. Of the calculation results, the case surrounded by the red line is the decision case.

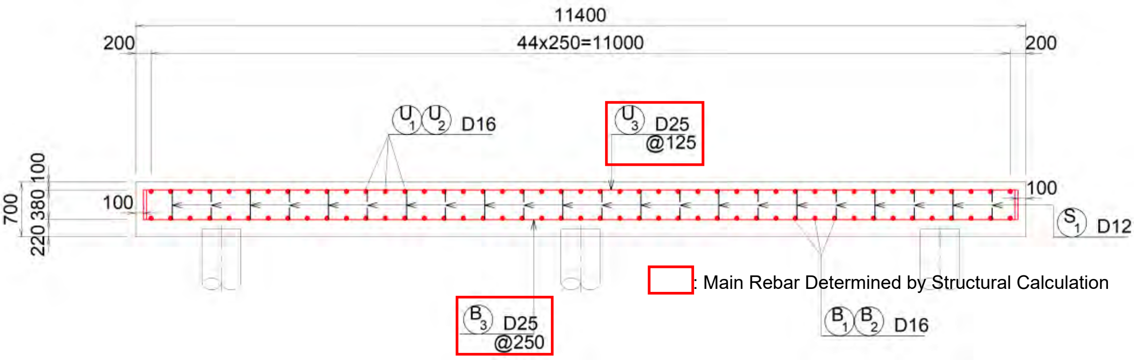
Table7.4.198 Results of Bending Stress Check for Upstream Right Apron

Member Names Checking Position Load Case Name	Extra Factor Section Force Myp (kNm) Section Force Mzp (kNm) Axial Force N '(kNm)	Compressive Stress Concreteσc '(N/mm2) Rebar σs' (N/mm2)	Tensile Stress (N/mm2) Neutral Axis X (m) Angle α(°) Rebar σs
Bottom slab member 1 X = 1.100 case 1 flood (at Tributary DFL)	1.000 0.0 -24.9 0.0	0.85 < 8.28 0.00 < 168.00	x = -0.119, α= 0 51.77 < 168.00
Bottom slab member 2 X = 0.000 case 1 flood (at Tributary DFL)	1.000 0.0 -53.5 0.0	1.83 < 8.28 0.00 < 168.00	x = -0.119, α= 0 111.16 < 168.00
Bottom slab member 3 X = 2.900 case 1 flood (at Tributary DFL)	1.000 0.0 -53.5 0.0	1.83 < 8.28 0.00 < 168.00	x = -0.119, α= 0 111.16 < 168.00
Bottom slab member 4 X = 0.000 case 1 flood (at Tributary DFL)	1.000 0.0 -37.4 0.0	1.28 < 8.28 0.00 < 168.00	x = -0.119, α= 0 77.66 < 168.00

Source: Study team

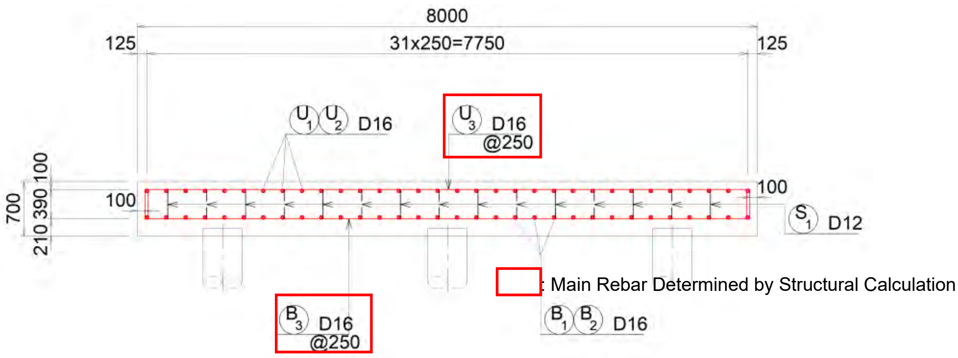
(e) Result of Examination

The bar arrangement procedure of the upstream apron is shown.



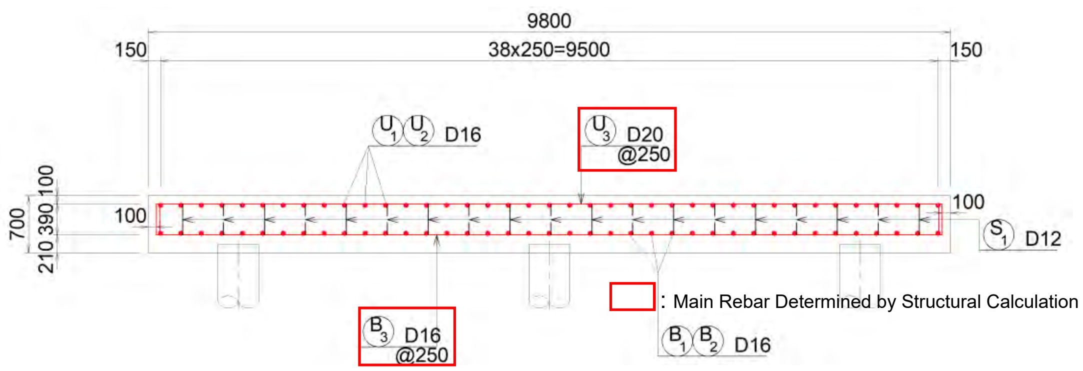
Source: Study team

Figure7.4.129 Bar Arrangement of Upstream Center Apron



Source: Study team

Figure7.4.130 Bar Arrangement of Upstream Left Bank Apron



Source: Study team

Figure7.4.131 Bar Arrangement of Upstream Right Bank Apron

7.4.2.3 Main Body Work (L2 Seismic Design)

(1) Design Criteria

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.

1) ***Level 1*** earthquake ground motion, considering seismic hazard from small to moderate earthquakes with high probability of occurrence during the bridge service life (100-year return), for ***seismic serviceability design objective*** to ensure normal bridge functions.

2) ***Level 2*** earthquake ground motion, considering a seismic hazard corresponding to an earthquake with return period event of 1,000 years (seven percent probability of exceedance in 75 years), for ***life safety performance objective*** under large earthquake.

Source: BSDS, DPWH, P2 -2

Since BSDS is a basic guideline to which the Japanese latest river structure design is applicable, the design will be conducted based on BSDS and Japanese “Performance Based Seismic Design Criteria for River Structures”. Technical codes and criteria for the seismic design are as shown in the following table (**Table7.4.199**).

Table7.4.199 Technical Codes and Criteria for Seismic Design

No.	Codes and Criteria	Year	Publisher
①	LRFD Bridge Seismic Design Specifications	2013	DPWH
②	Performance Based Seismic Design Criteria for River Structures I. Common	2012	MLIT (Japan)
③	Performance Based Seismic Design Criteria for River Structures II. Embankment	2012	MLIT (Japan)
④	Performance Based Seismic Design Criteria for River Structures III. Non-soil Embankment	2012	MLIT (Japan)
⑤	Performance Based Seismic Design Criteria for River Structures IV Floodgate, Sluiceway and Weir	2012	MLIT (Japan)
⑥	Guideline for Flexible Sluiceway	1998	Japan Institute of Country-ology and Engineering

No.	Codes and Criteria	Year	Publisher
⑦	Specifications for Highway Bridges I Common / IV Substructures	2012	Japan Road Association
⑧	Specifications for Highway Bridges V Seismic Design	2002	MLIT (Japan)

Source: Study Team

(2) Seismic Design Condition

1) Seismic Performance Setting

The "Performance Based Seismic Design Criteria for River Structures I. Common" states the following as a basis for the seismic performance evaluation.

"*In the seismic performance evaluation of river structures, the earthquake motions which is used to evaluate the seismic performance of river structures should be set appropriately, while a suitable evaluation method of seismic performance shall be applied*."

Source: Performance Based Seismic Design Criteria for River Structures I. Common

The Performance Based Seismic Design Criteria for River Structures IV (MLIT, Japan) sets the seismic performance of floodgates and weirs as shown in **Table7.4.200**. Important facilities for flood control or water use such as floodgates and weirs shall be under Seismic Performance 2, and the other of types floodgates shall be under Seismic Performance 3.

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

Table7.4.200 Seismic Performance

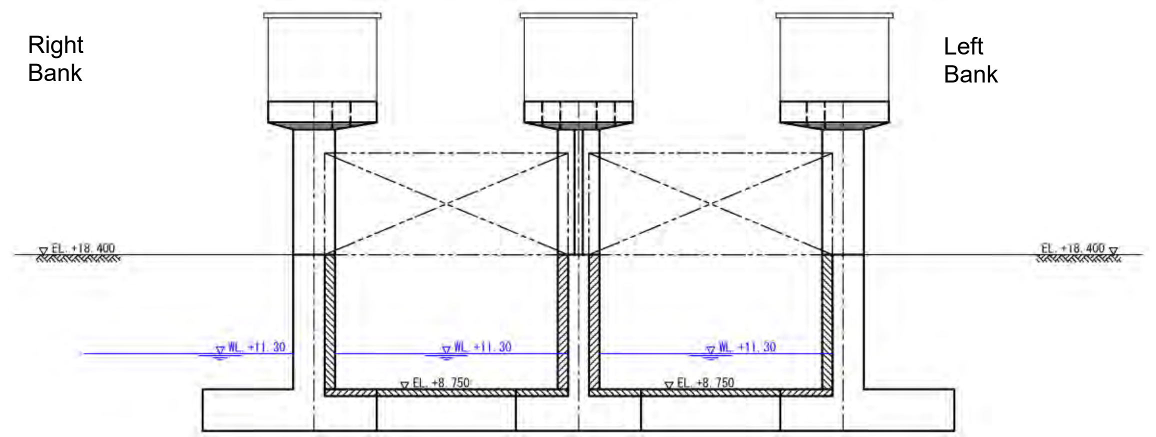
Seismic Performance	Performance Required
Seismic Performance 1	Soundness as floodgate, sluiceway or weir is not impaired by an earthquake
Seismic Performance 2	Function as floodgate, sluiceway or weir is maintained even after earthquake
Seismic Performance 3	Damage caused by earthquake is limited, and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

Source: Performance Based Seismic Design Criteria for River Structures IV

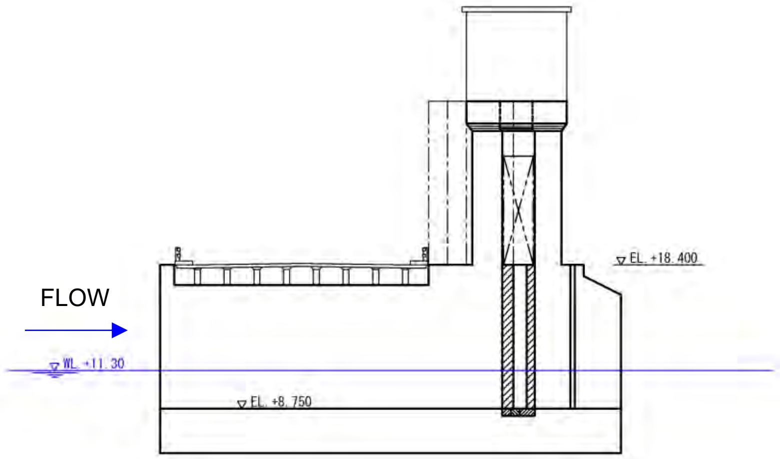
2) Water Level Condition

The water level conditions in L2 seismic condition are as follows: The water level seismic condition in the main body stability calculation shown in 7.4.2.1(4)1)(b) is adopted. A schematic diagram is shown in **Table7.4.201**.

Table7.4.201 Water Level Conditions (L2 seismic condition)

Calculations Direction	Schematic Diagram of Water Level Condition
Perpendicular Direction to the Flow	

Flow Direction



Source: Study team

3) Load Condition

In L2 seismic condition, from the load cases shown in **Table7.4.202**, load for seismic condition is considered.

Table7.4.202 Load Case Listing (Normal Condition, L1 Seismic Condition, During Construction)

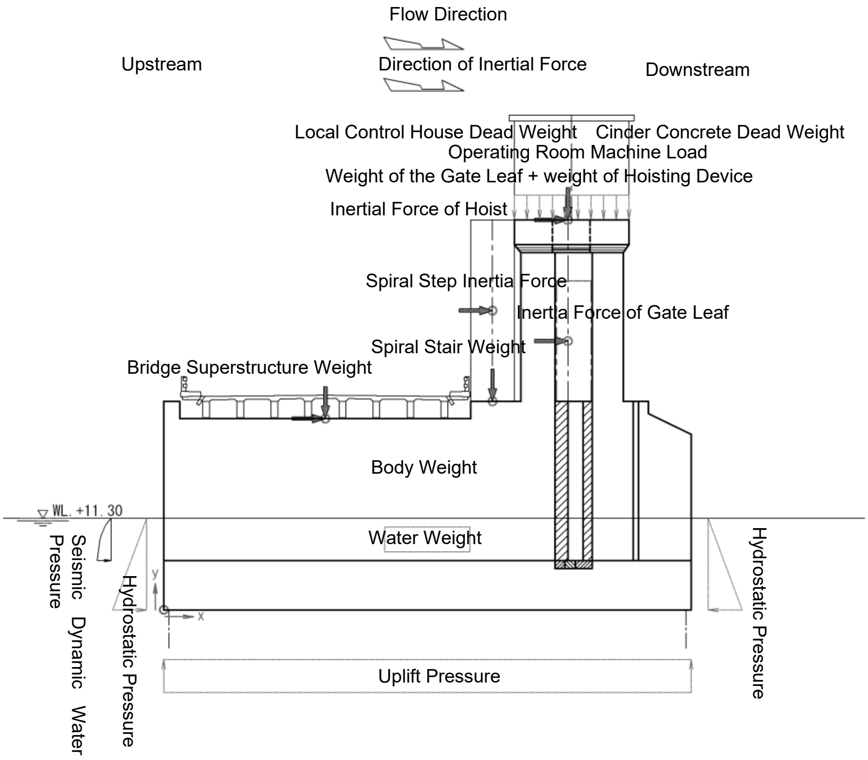
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	Water Level Condition			Structur Subject t Verifie	
			Dead Load	Control House Load	Bridge Superstructure Load	Gate Load	Spiral Step Load	Temperature Load	Wind Load	Live Load	Earth Pressure	Upstream Water Pressure	Downstream Water Pressure	Embankment/Cover Soil Load	Upstream Water Weight	Downstream Water Weight	Uplift Pressure	Seismic Inertia Force	Hydrodynamic Pressure	Floodgate Main Body	Gate State	Additional Factor of Allowable Stress	End Pier	Center Pier
Parpendicular Direction to Flow	CASE1	Normal	○	○	○	○	○	○		○	○	○		○	○	○	○			A	Open	1.00	○	
	CASE2	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○		○	○	○	○			A	Open	1.25	○	○
	CASE3	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○		○	○	○	○			B	Closed	1.25	○	○
	CASE4	Seismic	○	○	○	○	○				○	○		○	○	○	○	○	○	C	Open	1.33	○	○
	CASE 5	During construction (Left bank)	○	○	○	○	○		○		○									D	one side	1.50	○	○
	CASE 6	During construction (Right bank)	○	○	○	○	○		○		○									E	Open	1.50		○
Flow Direction	CASE7	Normal	○	○	○	○	○	○		○	○	○	○	○	○	○	○			A	Open	1.00	○	○
	CASE8	Normal + Wind Load	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○			A	Open	1.25	○	○
	CASE9	During floods (At Floodway DFL)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			B	Closed	1.25	○	○
	CASE 10	Seismic	○	○	○	○	○				○	○	○	○	○	○	○	○	○	C	Open	1.33	○	○
	CASE 11	During construction (Left bank)	○	○	○	○	○		○		○									D	Open	1.50	○	○
	CASE 12	During construction (Right bank)	○	○	○	○	○		○	○	○	○	○	○	○	○	○			F	Closed	1.25	○	○

x

Source: Study team

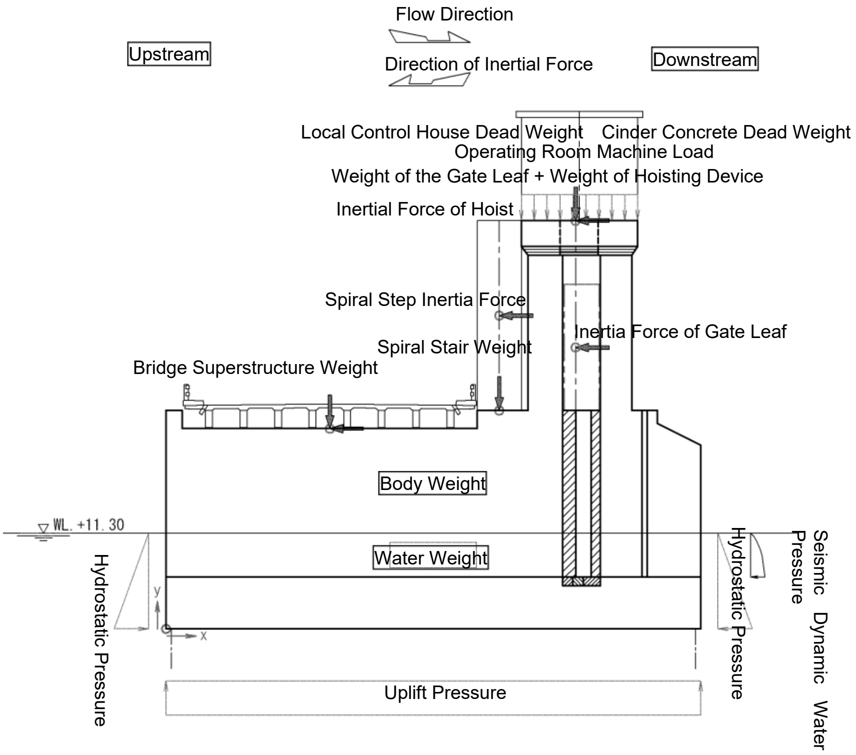
Load To Consider In L2 Earthquake

The following diagrams are shown on and after the next page.



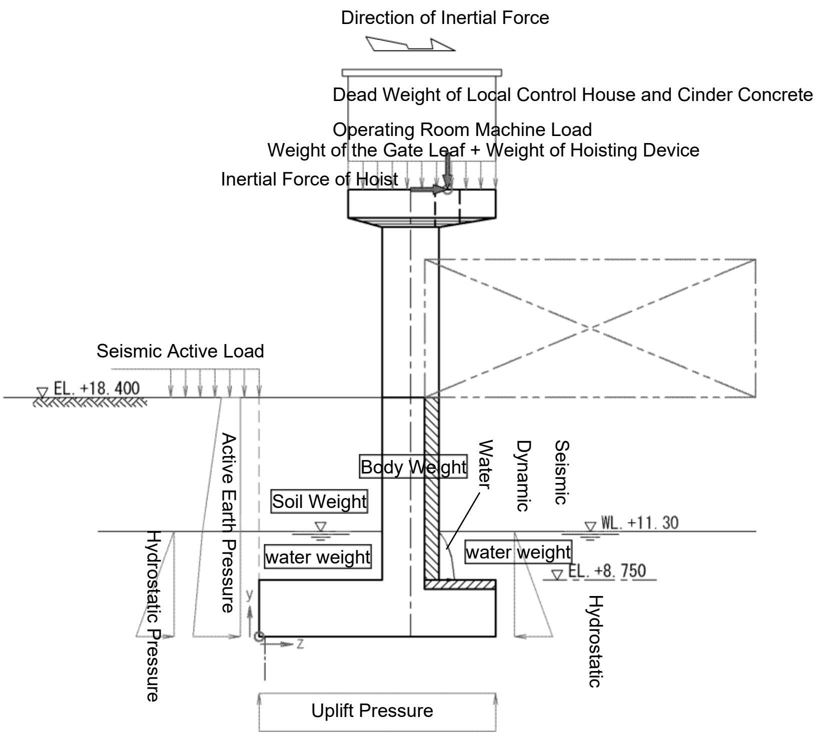
Source: Study team

Figure7.4.132 Load Diagram in Flow Direction (1/2) (Load from Upstream to Downstream)



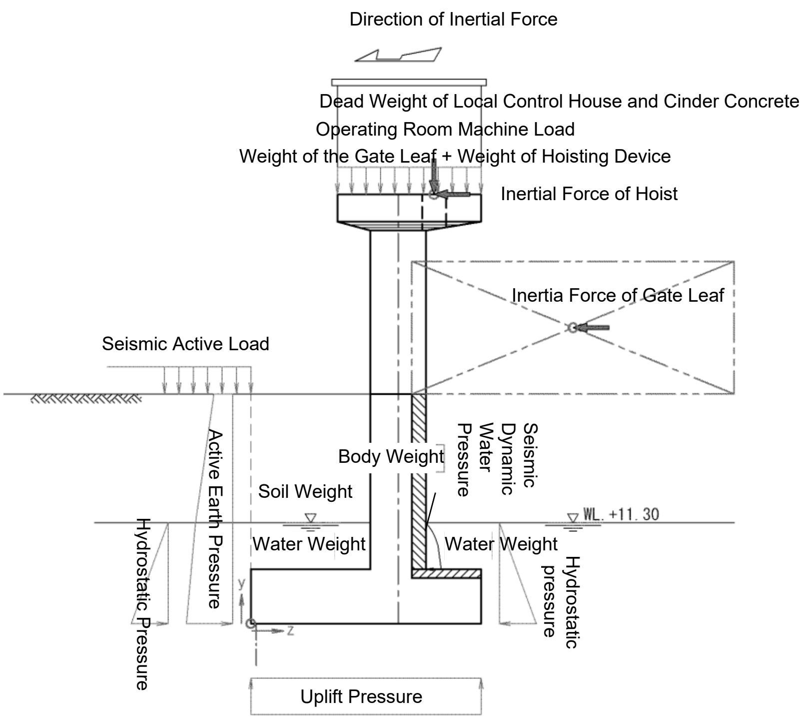
Source: Study team

Figure7.4.133 Load Diagram In Flow Direction (2/2) (Load from Upstream to Downstream)



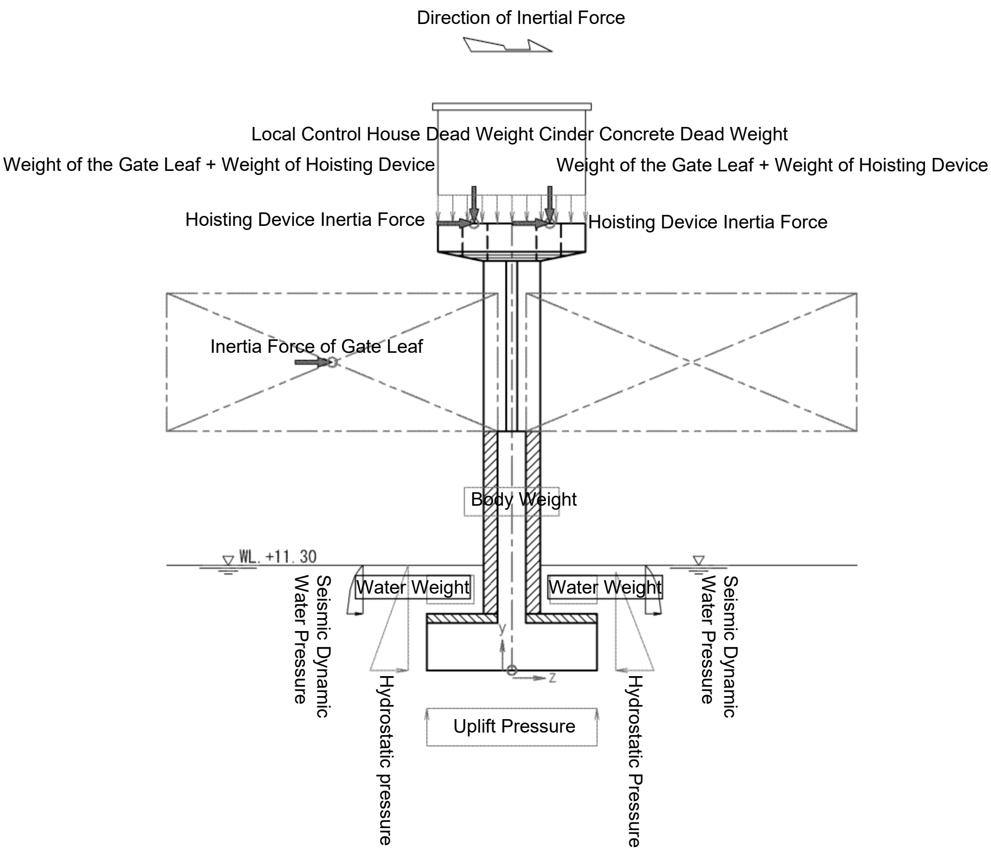
Source: Study team

Figure7.4.134 Load Diagram In Perpendicular Direction to the Flow (1/3) (End Pier (Load : Land Side → River Side))



Source: Study team

Figure7.4.135 Load Diagram In Perpendicular Direction to the Flow (2/3) (End Pier (Load : Land Side ← River Side))

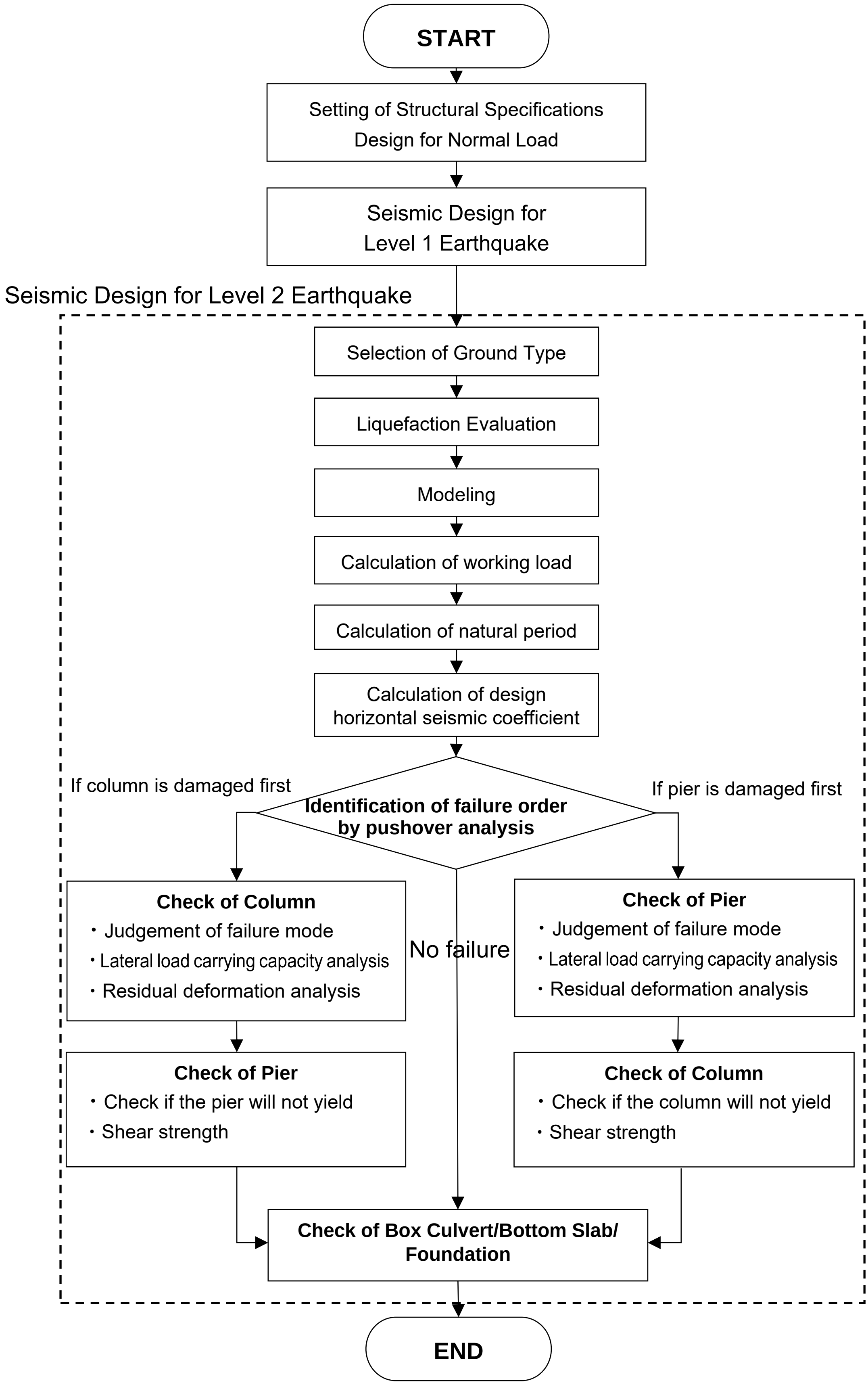


Source: Study team

Figure7.4.136 Load Diagram in Perpendicular Direction to the Flow (3/3) (Center Pier)

(3) Analysis Method

The seismic analysis method is summarized in **Figure7.4.137**.



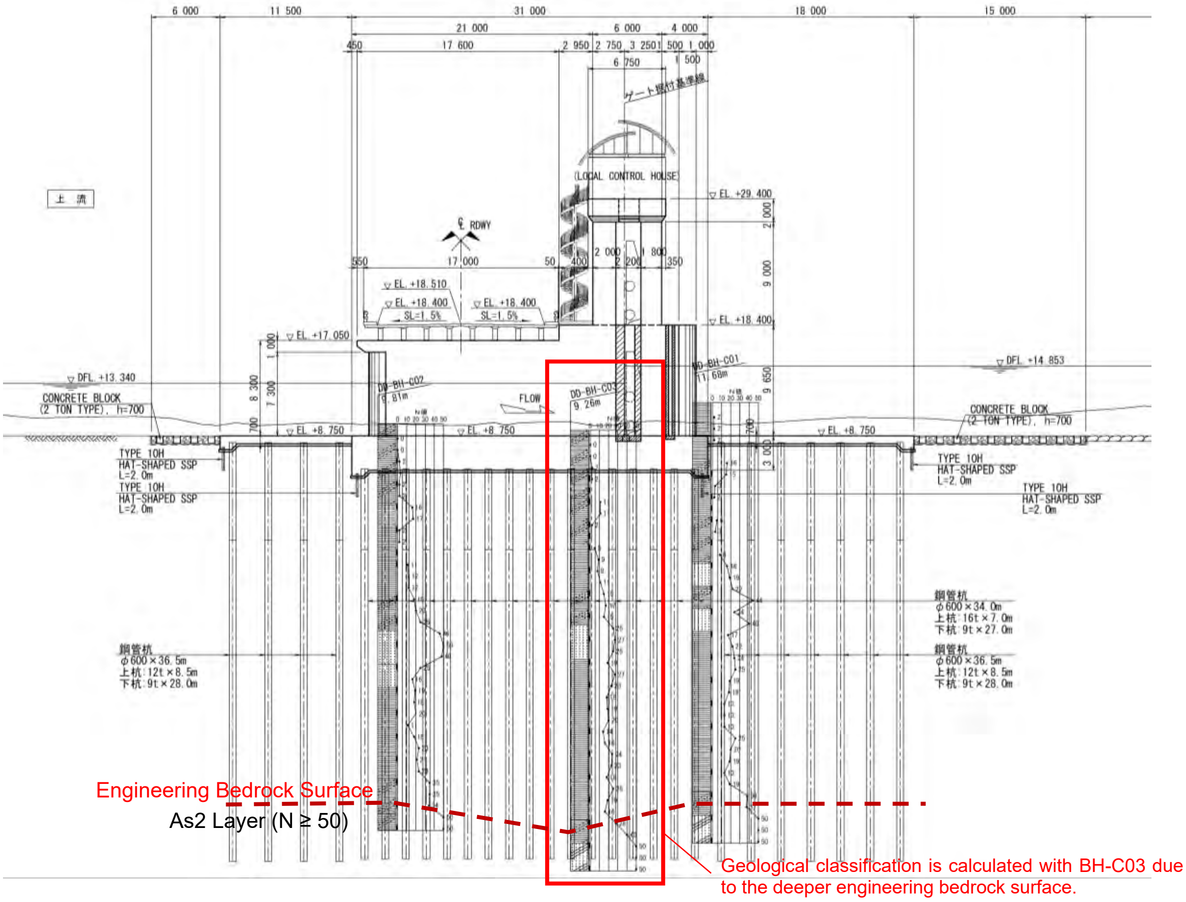
Source: Study team

Figure7.4.137 Flow of Seismic Analysis

(4) Content of Analysis

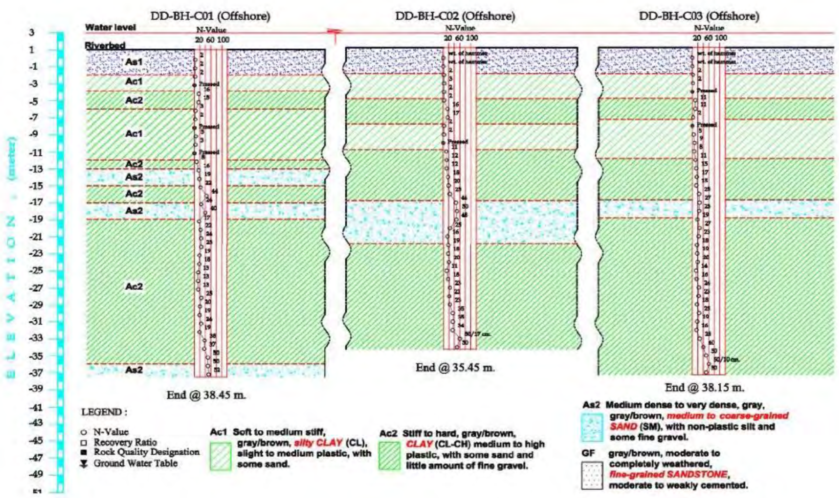
1) Selection of Ground Type

Geological profile of the Cainta Floodgate is shown in **Figure7.4.138** and the Soil Profile is shown in **Figure7.4.139** respectively. The engineering bedrock surface is the top surface of the As2 layer with an N value of 50.



Source: Study team

Figure7.4.138 General Drawing With Ground Condition



Source: Study team

Figure7.4.139 Soil Profile Representing BH-C01, BH-C02, BH-C03

The ground classification was calculated based on the following description in the BSDS.

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

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

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

Source: BSDS

The ground classification calculation results are shown in **Table7.4.203**. In the calculation, the shear elastic wave velocity Vs was calculated from the average N value based on the following description in the BSDS.

For cohesive soil layer,

$$V_{si} = 100N_i^{1/3} (1 \le N_i \le 25)$$

For sandy/cohesionless soil layer,

$$V_{si} = 80N_i^{1/3} (1 \le N_i \le 50) \quad \text{.......} \quad (C3.5.1\text{-}1)$$

where:

N_i : Average *N-value* of the *i-th* soil layer obtained from SPT

Source: BSDS, DPWH, P3 -30

As a result of calculating the ground classification, it was confirmed to correspond to the Type III ground, for the characteristic value T_G of the ground is 0.88 s ≧ 0.6 s.

Table7.4.203 Result of land classification calculation

Type of Ground		Elevation (m)	Thickness (m)	N-value	Vsi (m/s)	Hi/Vsi
Sandy Soil	As1	9.45	3.00	0	50	0.0600
Cohesive Soil	Ac1	6.45	3.19	1	100	0.0319
Cohesive Soil	Ac2	3.26	2.00	11	222	0.0090
Cohesive Soil	Ac2	1.26	2.00	2	126	0.0159
Cohesive Soil	Ac1	-0.74	0.41	8	200	0.0021
Cohesive Soil	Ac1	-1.15	3.59	8	200	0.0180
Cohesive Soil	Ac2	-4.74	3.00	16	252	0.0119
Sandy Soil	As2	-7.74	3.00	26	237	0.0127
Cohesive Soil	Ac2	-10.74	14.00	20	271	0.0517
Cohesive Soil	Ac2	-24.74	2.00	34	324	0.0062
		-26.74				
Subtotal			16.00			
4 Σ H/V						0.8776
Type of Ground					Type- III Ground	

Source: Study team

2) Liquefaction Analysis

Liquefaction Analysis should be performed for the soils listed below in the BSDS.

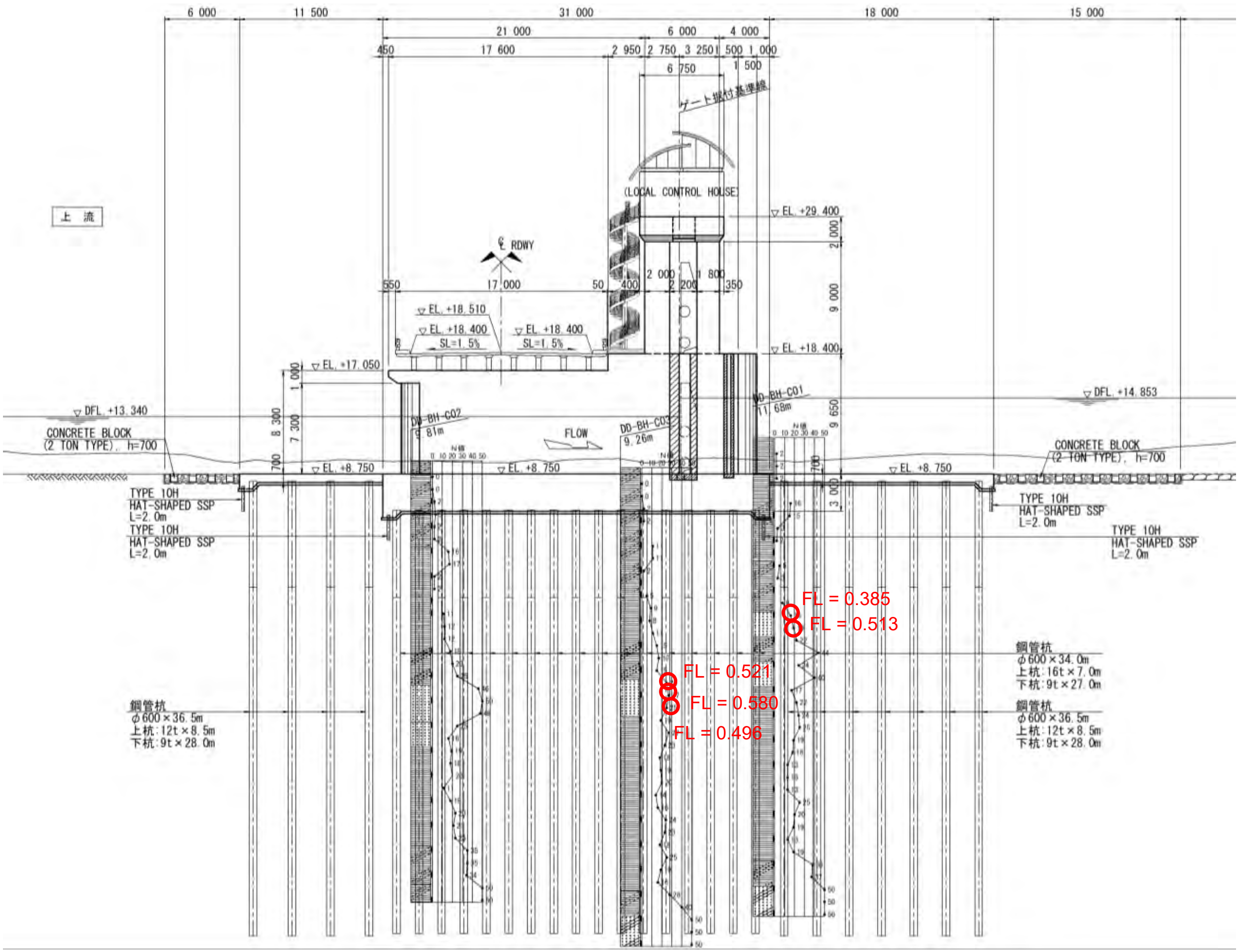
1) Saturated soil layer with depth less than 20 m below the ground surface and having ground water level higher than 10 m below the ground surface.

2) Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index, I_P, less than 15, even if FC is larger than 35%.

3) Soil layer having a mean particle size (D_{50}) of less than 10 mm and a particle size at 10% passing (D_{10}) (on the grading curve) is less than 1 mm.

Source: BSDS, DPWH, P6 -3

As a result of the liquefaction study shown in **7.4.2.1 (2)** , The F_L value is less than 1.0 at the position shown in **Figure7.4.140**. The detailed results of each boring are shown in **Table 7.4.7** to **Table 7.4.9**.



Source: Study team

Figure7.4.140 L2 Liquefaction Analysis Result

D0-BH-C03 boring was used for pile foundation design. At D0-BH-C03, liquefaction occurred in the sandy layer at a depth of -17 m from the top of the boring, and the F_L value is 0.496 ~ 0.580. Also, R = 0.368 ~ 0.497, as shown in **Table 7.4.9**. In designing the pile foundation, based on the following description in "Specifications for Highway Bridges V Seismic design edition", the reduction factor DE = 2/3, which is compatible with the depth of 10 m < x ≦ 20 m, 1/3 < FL ≦ 2/3, and 0.3 < R, is considered.

Table 6.2.4-1 Reduction Factor D_E for Geotechnical Parameters

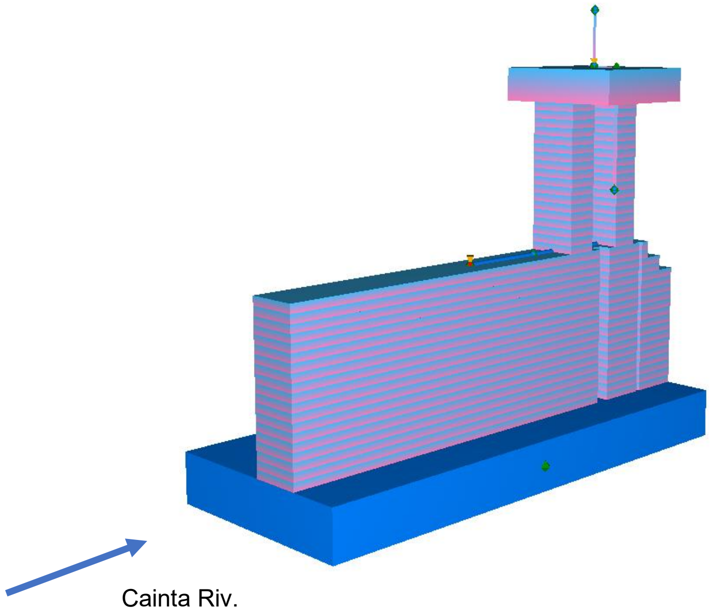
Range of F_L	Depth from Present Ground Surface x (m)	Dynamic Shear Strength Ratio, R	
		$R \leq 0.3$	$0.3 < R$
		Verification for Level 2 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion
$F_L \leq 1/3$	$0 \leq x \leq 10$	0	1/6
	$10 < x \leq 20$	1/3	1/3
$1/3 < F_L < 2/3$	$0 < x < 10$	1/3	2/3
	$10 < x \leq 20$	2/3	2/3
$2/3 < F_L \leq 1$	$0 \leq x \leq 10$	2/3	1
	$10 < x \leq 20$	1	1

Source: BSDS, P6-8 and "Specifications for Highway Bridges V Seismic design edition" (March 2012), p. 142

3) Analysis of Columns and Piers

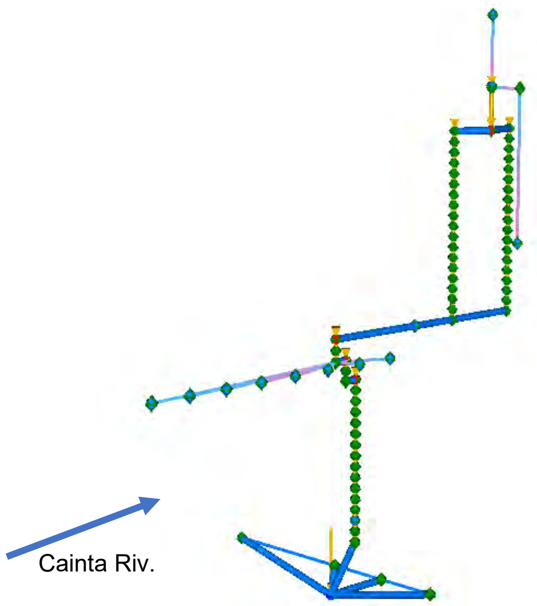
(a) Analytical Model Diagram

A three-dimensional beam element model considering nonlinearity was prepared for each of the end pier and center pier. A model diagram showing beam elements in solid form and a model diagram showing frame are indicated in **Figure7.4.141** to **Figure7.4.144**.



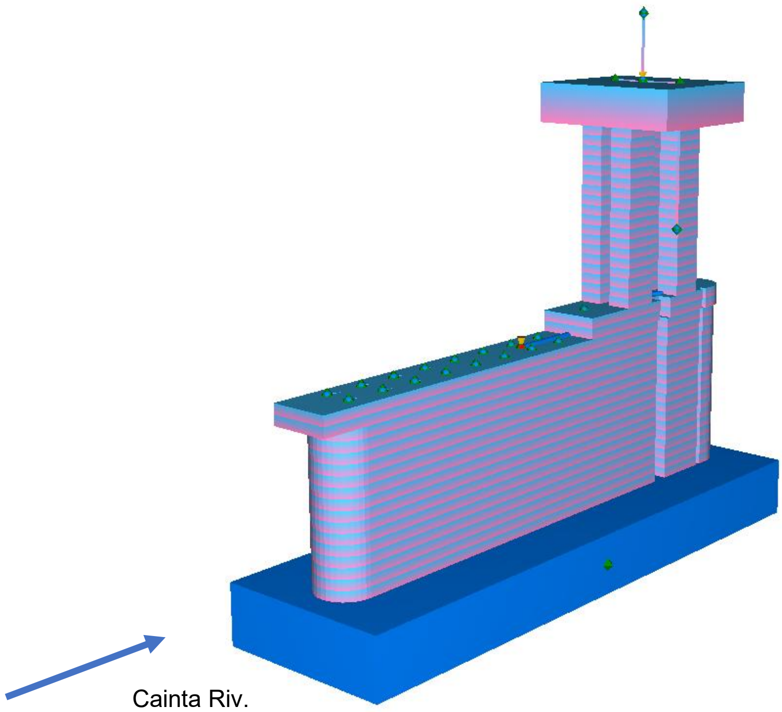
Source: Study team

Figure7.4.141 Analytical Model Diagram of End Pier (Solid Elements)



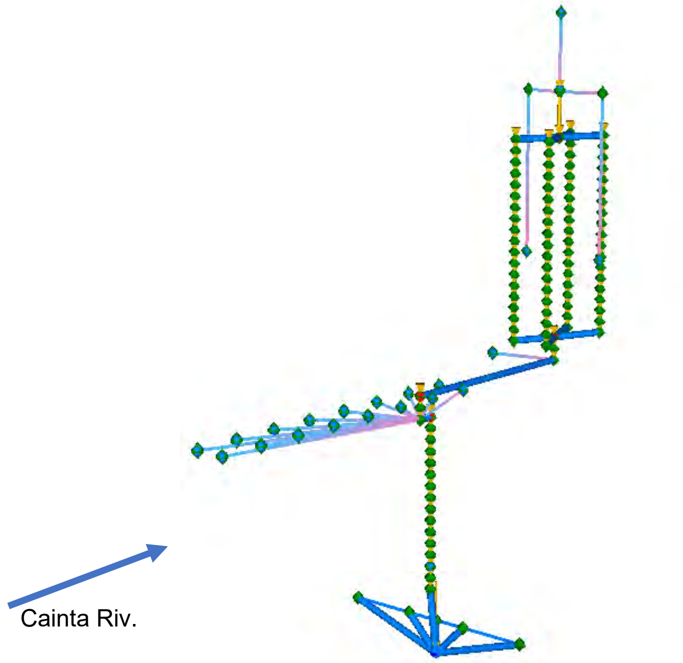
Source: Study team

Figure7.4.142 Analytical Model Diagram of End Pier (Presented In Frame)



Source: Study team

Figure7.4.143 Analytical Model Diagram of Center Pier (Solid Elements)



Source: Study team

Figure7.4.144 Analytical Model Diagram of Center Pier (Presented In Frame)

(b) Calculation of Working Load

(i) Summary

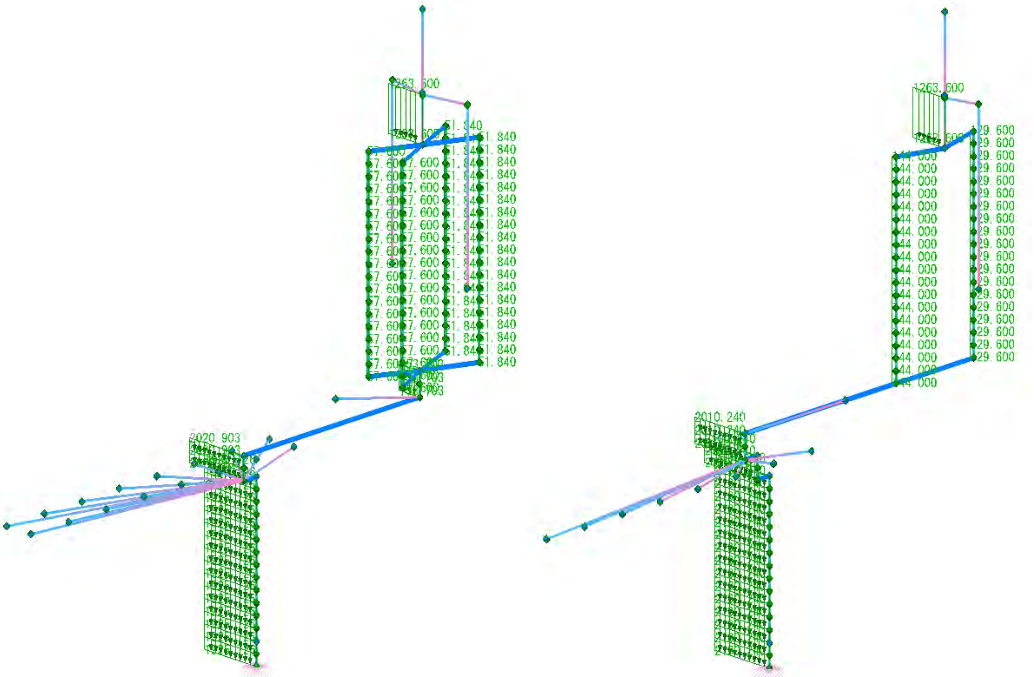
The loads to be considered in the analysis of columns and piers are shown below.

A. Dead Load

Consider the following weight as dead load.

- Dead Weight of Member
- Local Control House Weight
- Dead Weight of Cinder Concrete
- Gate Leaf Dead Weight
- Dead Weight of the Bridge
- Spiral Stair Weight

The load diagram of dead weight is shown in **Figure7.4.145**.

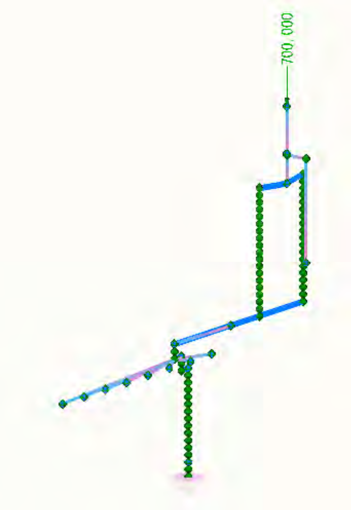


(Center Pier) (End Pier)

Source: Study team

Figure7.4.145 Load Diagram of Dead Weight

The self-weight of the local control house is 700 kN for both the end pier and the center pier. As a typical example, a load diagram of a pier is shown in **Figure7.4.146**.



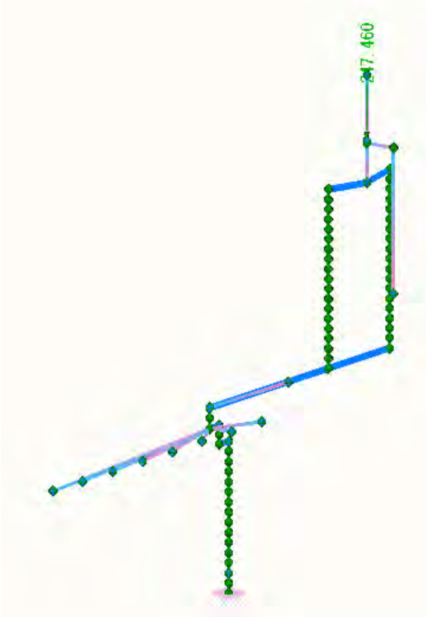
Source: Study team

Figure7.4.146 Load Diagram of Dead Weight of Local Control House (End Pier)

Since the wall thickness of cinder concrete in both the end piers and the center pier is 20 cm, the weight of the cinder concrete is calculated as follows.

W = 23.5 * 6.75 * 7.80 * 0.20 = 247.46 kN

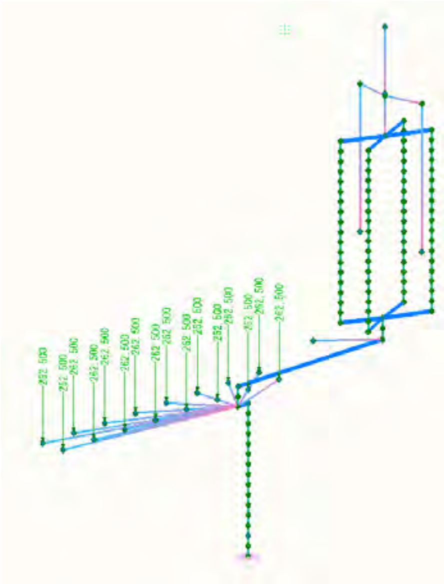
As a typical example, a load diagram in a pier is shown in **Figure7.4.147**.



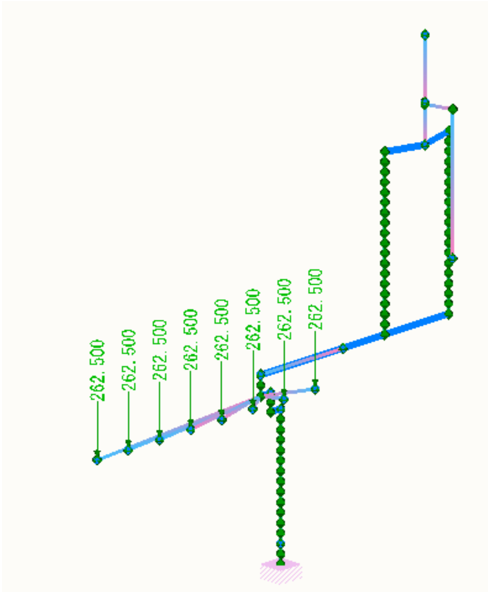
Source: Study team

Figure7.4.147 Load Diagram of Dead Weight of Cinder Concrete (End Pier)

The bridge's dead weight is 262.5 kN per 1 support (262.5 × 8 = 2,100 kN per 1 support). The load diagram is shown in **Figure7.4.148**.



(Center Pier)

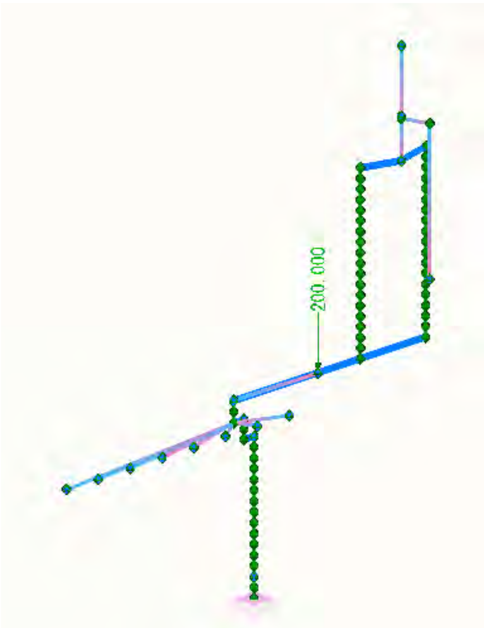


(End Pier)

Source: Study team

Figure7.4.148 Load Diagram of Dead Weight of Maintenance Bridge

The dead weight of the spiral stair is 200 kN per 1 location for both the end piers and the center pier. As a typical example, the load diagram of the pier is shown in **Figure7.4.149**.

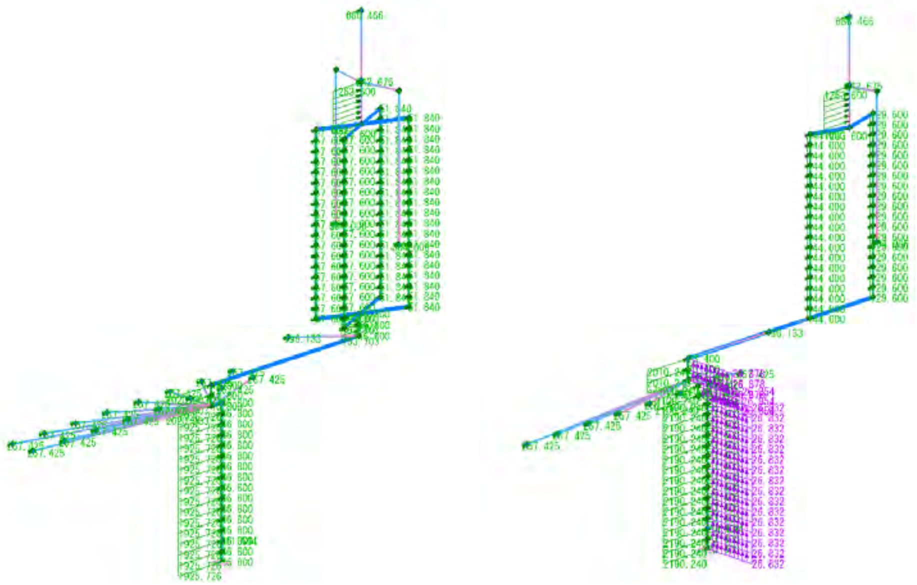


Source: Study team

Figure7.4.149 Load Diagram of Dead Weight of Spiral Stair

B. Inertial Force

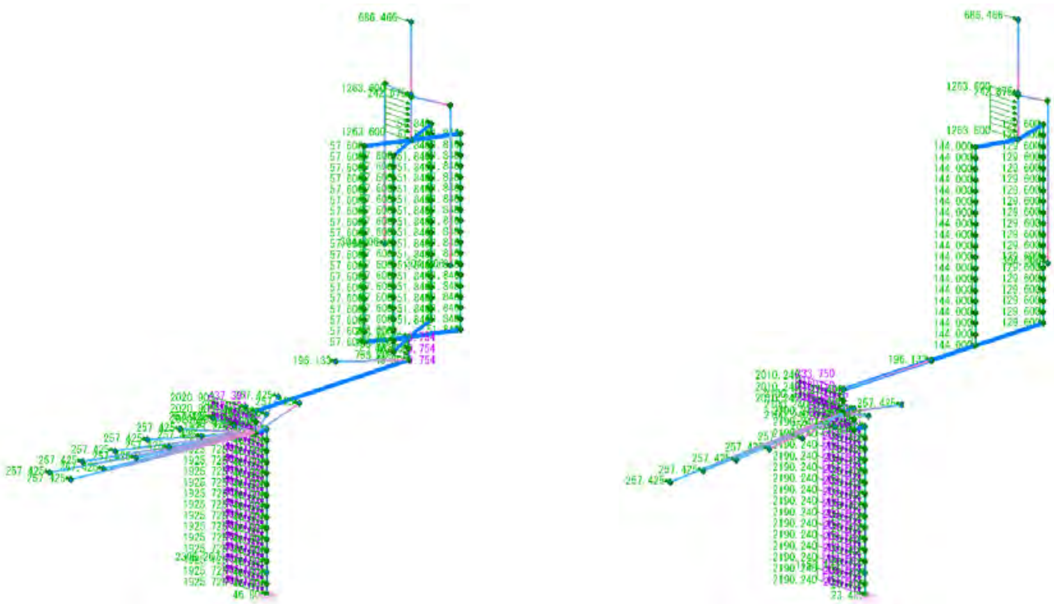
The dead load shown in the preceding clause is multiplied by the design horizontal seismic intensity to apply seismic inertia force. The inertial force of the gate acts on the gate position. Load diagram is indicated as follows. (Design horizontal seismic intensity kh is per 1.0 layer).



Source: Study team

(Center Pier) (End Pier)

Figure7.4.150 Load Diagram of Inertial Force in Flow Direction



(Center Pier) (End Pier)

. Source: Study team

Figure7.4.151 Load Diagram of Inertial Force in Perpendicular Direction to the Flow

C. Earth Pressure in Seismic Condition

The seismic earth pressure in seismic condition acting on the end pier in flow direction is calculated in accordance with "Seismic performance check guideline for river structures and its explanation I. Common section".

The designed horizontal seismic intensity is, since the gourd type is Type III based on p 7-659, khg = 0.47 is set.

Since it is assumed that "sandy soil" and the rear surface is soil and concrete, the active earth pressure coefficient in seismic condition is calculated as follows,

kEA = 0.24 + 1.08 ・ khg = 0.24 + 1.08 × 0.47 = 0.745

Here, kEA: Coefficient of Active Earth Pressure In Seismic Condition
khg: Design Horizontal Seismic Intensity At Ground Level

The earth pressure Seismic Condition is calculated by the following formula.

$PEA = \gamma \cdot x \cdot K_{EA} + q' \cdot K_{EA}$

Here, PEA: Active earth pressure in Seismic Condition (kN/m2)
γ: Unit weight of soil (kN/m2)
q ': Surcharge in Seismic Condition (kN/m2)

① Level 2 earthquake

	qL0 =	5.0	×	0.745						3.73	kN/m2
Groundwater Level	qL1 =	3.73	+	0.745	×	19.00	×	7.100		104.23	kN/m2
On The Bottom Slab	qL2 =	104.23	+	0.745	×	10.20	×	2.550		123.61	kN/m2

The earth pressure in seismic condition is applied as follows. The increment in seismic condition is applied only at the time of loading from the land side to the river side.

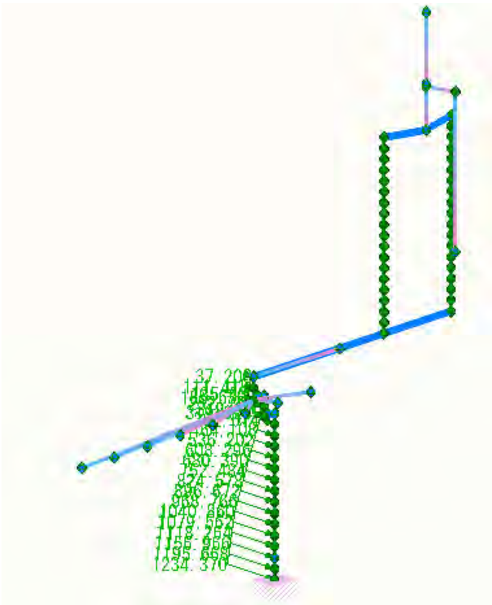
Earth Pressure In Seismic Condition = Earth Pressure In Normal Condition + Increment

The earth pressure in normal condition is calculated as follows, assuming that k_{hg} of k_{EA} = 0.24 + 1.08xkhg is 0.

qL0 =	5.0	×	0.240	×	31.000					=	37.20	kN/m2
qL1 =	37.20	+	0.240	×	19.00	×	7.100	×	31.000	=	1040.86	kN/m2
qL2 =	1040.86	+	0.240	×	10.20	×	2.550	×	31.000	=	1234.37	kN/m2

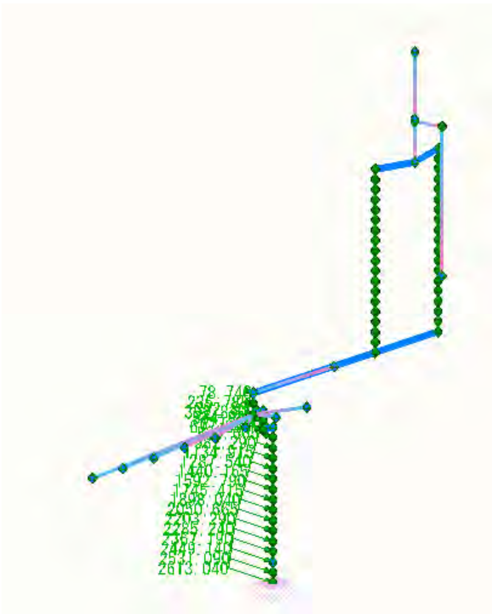
The increment in seismic condition was calculated as the value obtained by subtracting the active earth pressure at all times from the active earth pressure in seismic condition as follows.

qL0 =	3.73	×	31.000	-	37.20	=	78.43	kN/m2
qL1 =	104.23	×	31.000	-	1040.86	=	2190.27	kN/m2
qL2 =	123.61	×	31.000	-	1234.37	=	2597.54	kN/m2



. *Source: Study team*

Figure7.4.152 Load Diagram of Earth Pressure Acting on the End Pier in Perpendicular Direction to the Flow



. *Source: Study team*

Figure7.4.153 Load Diagram of Earth Pressure Increment Acting on the End Pier in the Perpendicular Direction to the Flow (Land Side → River Side)

D. Seismic Dynamic Water Pressure

For the dynamic water pressure in seismic condition, load using the dynamic water pressure described in "Example of calculation concerning the seismic performance analysis of floodgate gates and weirs based on the seismic horizontal capacity method in seismic condition, March 2008, Earthquake Resistance Research Group (Vibration), Public Works Research Institute ".

The height of the water level is as follows.

Water level height h = W.L. + 11.30 m - Bottom slab top height E.L. + 8.750 m = 2.55 m

The effect of seismic hydrodynamic pressure can be simulated by applying additional mass to the site where the seismic hydrodynamic pressure acts. Based on Performance Based Seismic Design Criteria for River (Solution 5.5 .1), the additional mass *md* (t/m) for simulating seismic dynamic water pressure is calculated by the following formula.

$$m_d = \int_{h_1}^{h_2} \frac{7}{8}\frac{\gamma_w}{g} b\sqrt{H \cdot h}\,dh = \frac{7}{12}\frac{\gamma_w}{g} b\left(\sqrt{H \cdot h_2^3} - \sqrt{H \cdot h_1^3}\right)$$

The results of calculations are shown in **Table7.4.204**.

Table7.4.204 Result of calculation of seismic dynamic water pressure

Item	Units	Calculated Value	
		Flow Direction	Perpendicular Direction To The Flow
md	t	11.4	117.6
γw	kN/m3	9.80	
g	m/s2	9.80	
b	m	3.000	31.000
H	m	2.550	
h1	m	0.000	
h2	m	2.550	
Acting Point (Load Centroid Height)	m	1.02	

. Source: Study team

(ii) Design Horizontal Seismic Coefficient

The design horizontal seismic intensity shall be set based on the following description in the BSDS.

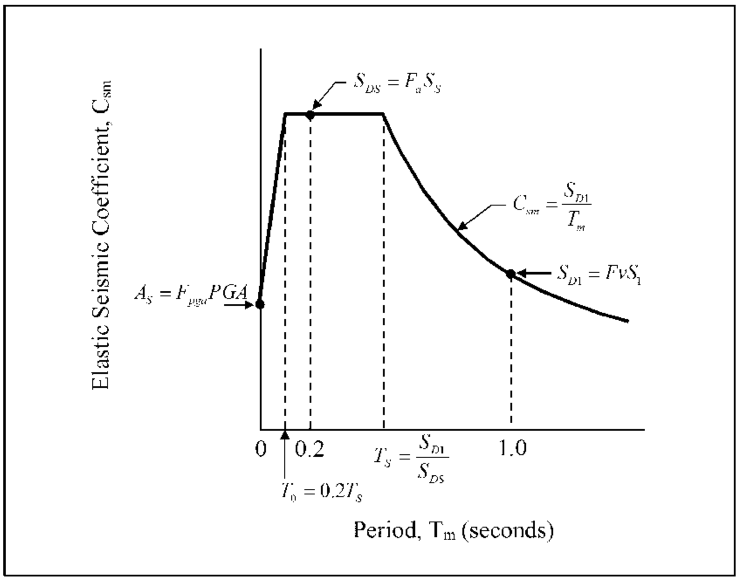


Figure 3.6.1-1 Design Response Spectrum

The five-percent-damped-design response spectrum shall be taken as specified in Figure 3.6.1-1. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients from Figures 3.4.1-1 to 3.4.1-6, scaled by the zero-, short-, and long-period site factors, F_{pga}, F_a, and F_v, respectively.

Source: BSDS, DPWH, P3 -36

Figure7.4.154 design response spectrum

The PGA, Ss and S1 shown above are set bed on **Figure7.4.155** to **Figure7.4.157** from BSDS.

PGA	:0.6
Ss	:1.2
S1	:0.45

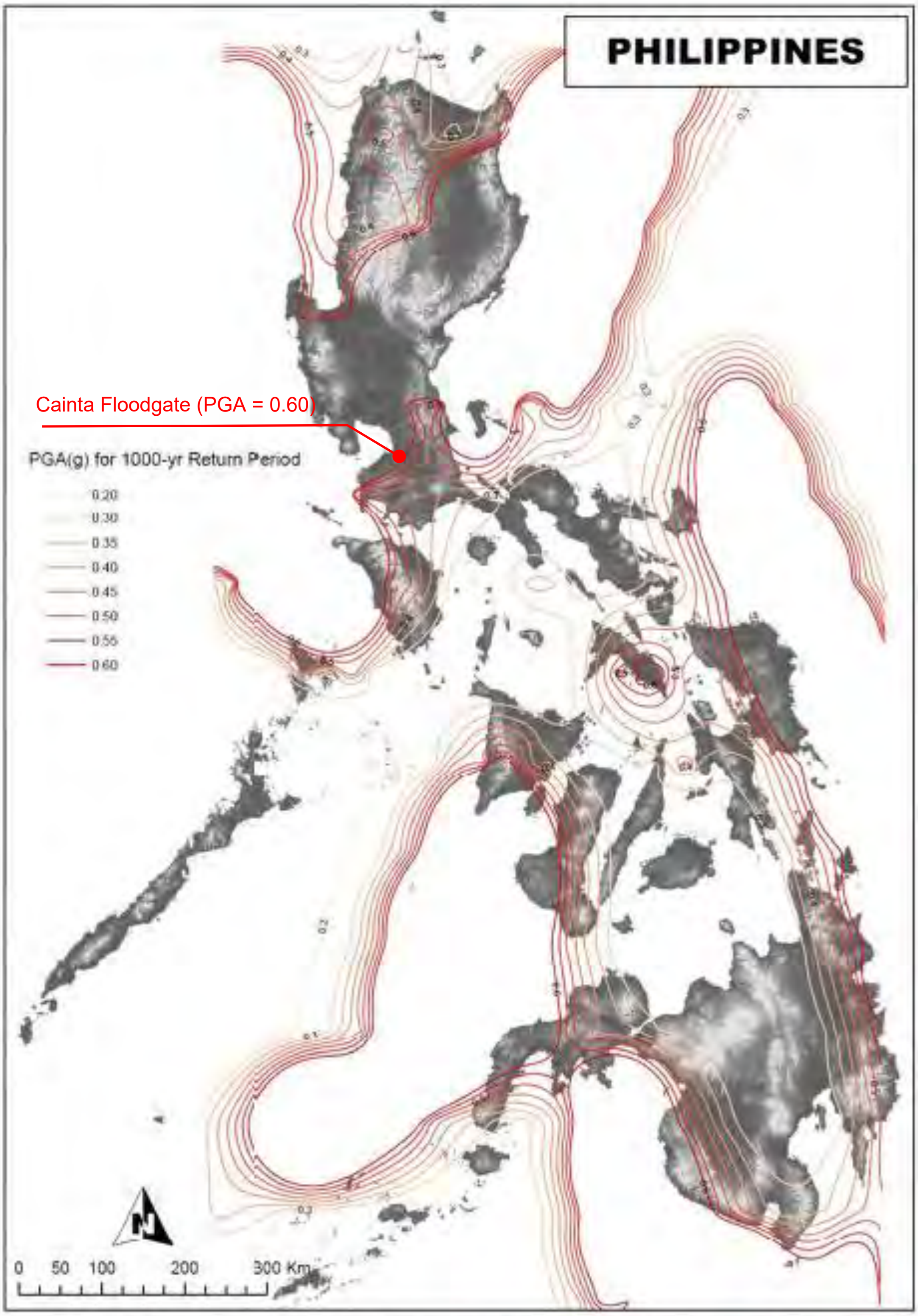


Figure 3.4.1-4 Horizontal Peak Ground Acceleration Coefficient (*PGA*) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground motion

Source: BSDS, DPWH

Figure7.4.155 L2 Earthquake Ground Motion Acceleration Response Spectrum Coefficient PGA (BSDS, P3 -21)

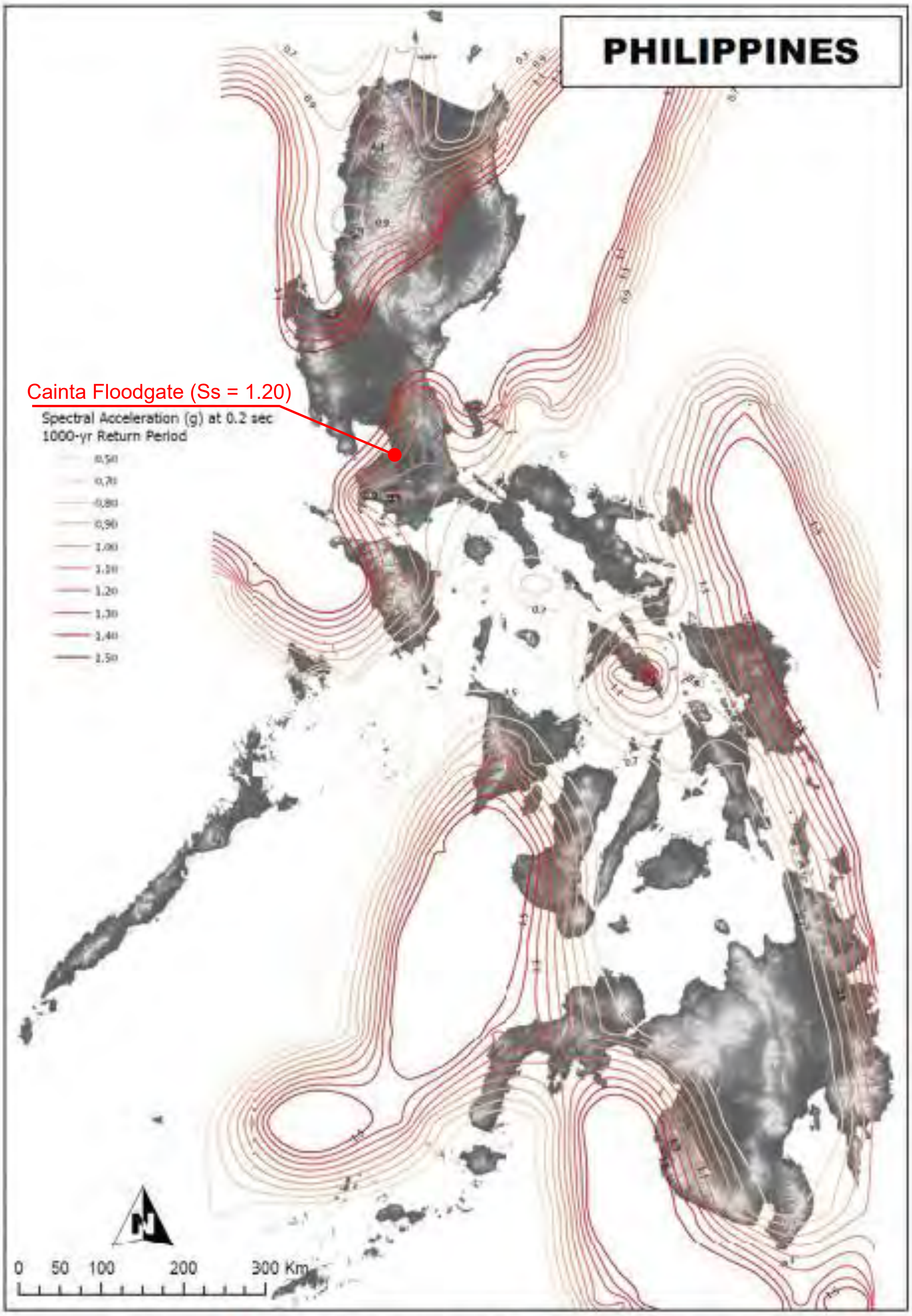


Figure 3.4.1-5 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_s) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground Motion

Source: BSDS, DPWH

Figure7.4.156 Acceleration Response Spectrum Coefficient Ss (BSDS Figure 3.4. 1. -5)

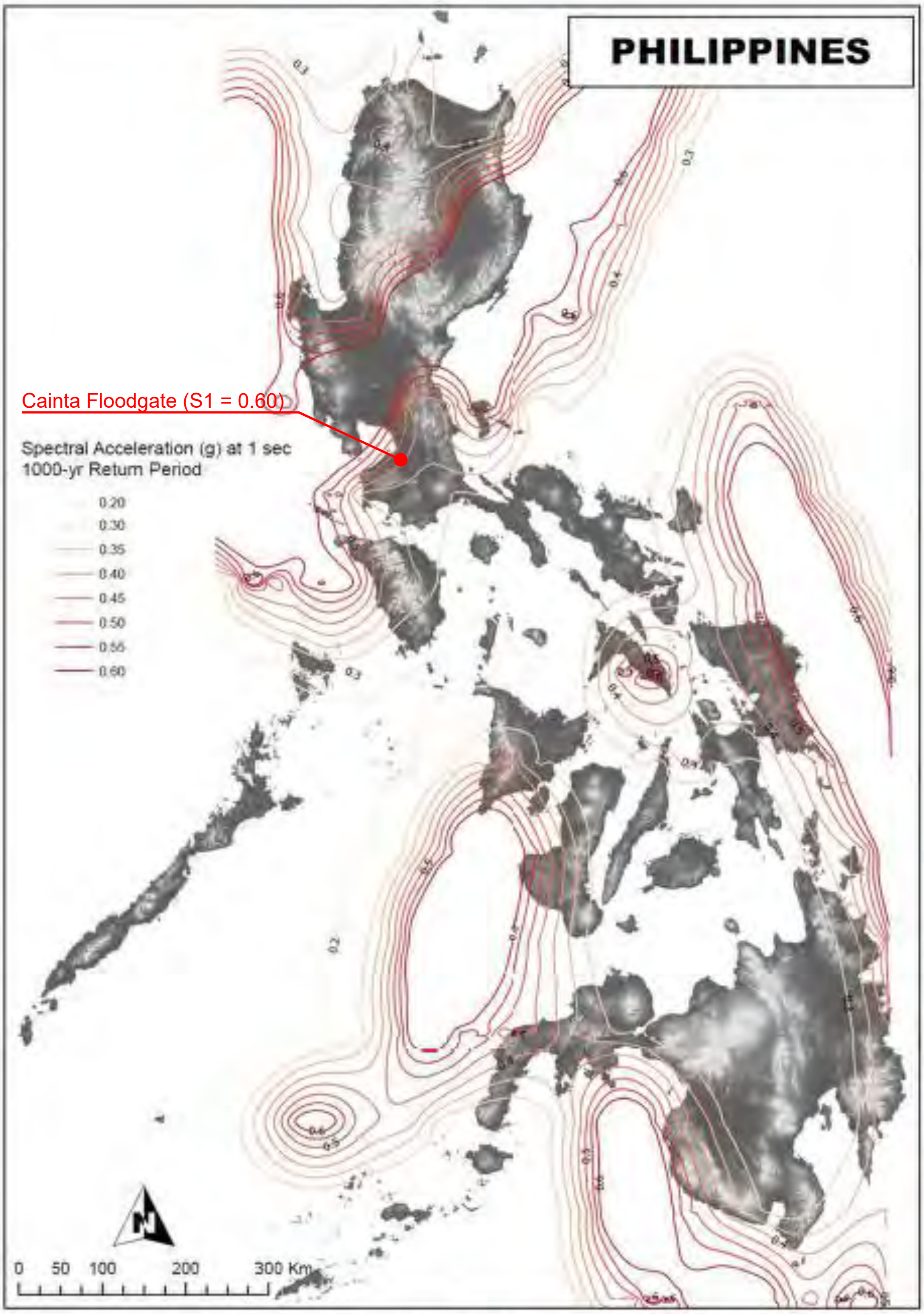


Figure 3.4.1-6 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground Motion

Source: BSDS, DPWH

Figure7.4.157 Acceleration response spectrum factor S1 (BSDS Figure 3.4. 1. -5)

The ground of the Cainta Floodgate corresponds to Type III. From the set PGA, Ss and S1, the F_{PGA}, F_a and F_v are set as follows based on the description of the BSDS.

F_{pga} :0.78
Fa :0.82
Fv :2.4

3.5.3 Site Factors

The Site Factors F_{pga}, F_a and F_v specified in Tables 3.5.3-1, 3.5.3-2, and 3.5.3-3 shall be used in the zero-period, short-period range, and long-period range, respectively for the elastic seismic response coefficient in the design response spectrum of Article 3.6 of this Section. These factors shall be determined using the Ground Types (Site Class) given in Table 3.5.1-1 and the mapped values of the coefficients PGA, S_S, and S_1 in Figures 3.4.1-1 to 3.4.1-6 and Appendix 3A and 3B.

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

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

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

PGA = 0.60 → F_{pga} = 0.78

Table 3.5.3-2 Values of Site Factor, F_a, for Short-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 2.0$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

Note:
[1] Use straight-line interpolation for intermediate values of S_S.

Ss = 1.2 → F_a = 0.82

Table 3.5.3-3 Values of Site Factor, F_v, for Long-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.80$
I	1.7	1.6	1.5	1.4	1.4	1.4
II	2.4	2.0	1.8	1.6	1.5	1.5
III	3.5	3.2	2.8	2.4	2.4	2.0

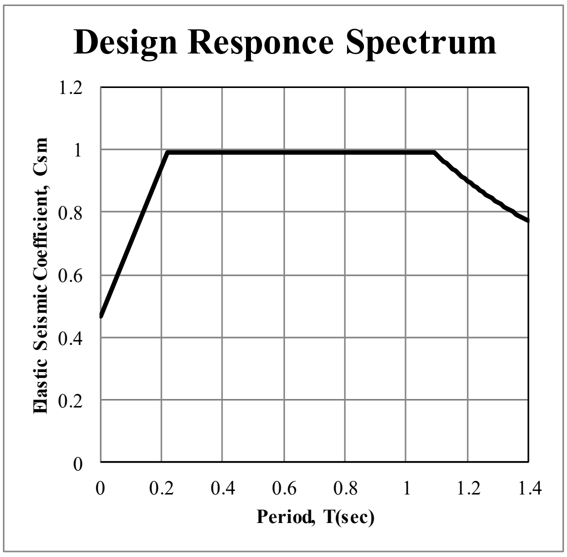
Note:
[1] Use straight-line interpolation for intermediate values of S_1.

S1 = 0.45 → F_v = 2.4

Source: BSDS, p3 -32 – 33

The set acceleration spectrum is shown in **Figure7.4.158**. The design horizontal seismic intensity kh_{gL} of the ground surface is as follows.

$kh_{gL} = F_{pga} \times PGA = 0.78 \times 0.60 = 0.47$



Source: Study team

Figure7.4.158 Cainta Floodgate Acceleration Spectrum

(c) Computation of Natural Period

The natural period in the flow direction and the perpendicular direction to the flow is calculated by eigenvalue analysis for the end pier and the center pier.

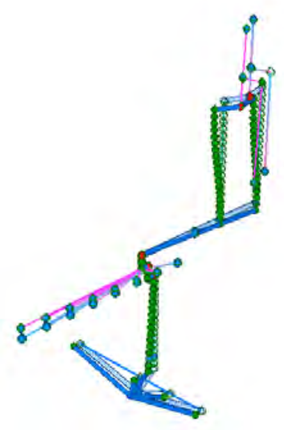
The end pier eigenvalue results are shown in **Figure7.4.159**. The sixth mode is dominant in the flow direction and the fourth mode is dominant in the perpendicular direction to the flow. Since the low-level mode is predominant in all cases, the seismic horizontal capacity method is used for checking.

Table7.4.205 End Pier Characteristic Analysis Result

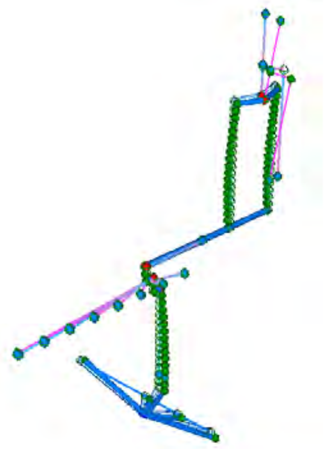
Mode Order	Frequency F (Hz)	Natural Period T (Sec)	Stimulus Coefficient			Effective Mass Ratio (%)			Mode Reduction Factor H
			X (Right Angle)	Y (Vertical)	Z (Water Flow)	X	Y	Z	
1	0.772	1.295	42.39	-4.32	-1.56	30%	0%	0%	0.08474
2	0.871	1.148	-1.90	-3.82	-44.22	0%	0%	33%	0.09874
3	2.479	0.403	24.65	-5.40	0.06	10%	1%	0%	0.06027
4	4.057	0.246	-42.48	4.88	-0.95	30%	0%	0%	0.06704
5	4.456	0.224	2.45	1.84	25.76	0%	0%	11%	0.05757
6	4.924	0.203	-4.36	-5.20	56.92	0%	0%	55%	0.09372
7	5.702	0.175	-41.26	-12.38	-3.00	28%	3%	0%	0.08391
8	7.360	0.136	2.26	1.12	0.10	0%	0%	0%	0.05022
9	8.561	0.117	5.26	18.14	0.42	0%	6%	0%	0.05418
10	10.141	0.099	-0.08	-3.60	-0.16	0%	0%	0%	0.05009
11	11.864	0.084	0.77	-49.81	-0.48	0%	45%	0%	0.07124
12	13.045	0.077	-1.39	47.40	0.18	0%	41%	0%	0.07371
13	14.306	0.070	1.33	-9.81	-0.11	0%	2%	0%	0.05159
14	18.068	0.055	0.45	-5.58	0.02	0%	1%	0%	0.05079

Legend
: Flow Direction Predominant Mode
: Flow Normal Predominant Mode

Source: Study team



Flow Direction 6th mode]



Perpendicular Direction to the Flow [4th mode]

Source: Study team

Figure7.4.159 End Pier Characteristic Analysis Result

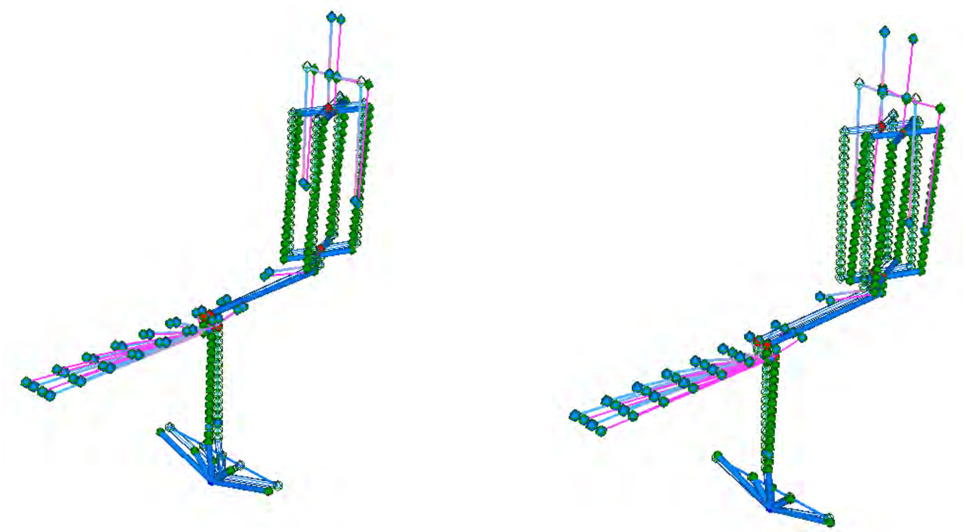
The eigenvalue analysis result of the center pier is shown in **Table7.4.206**. The result shows that the fourth mode was dominant in the flow direction and the first mode was dominant in the direction perpendicular to water flow. As the low-level mode is dominant as in the case of piers, the seismic horizontal capacity method during earthquakes is used for checking.

Table7.4.206 Results of Modal Analysis of Center pier

Mode Order	Frequency F (Hz)	Natural Period T (Sec)	Stimulus Coefficient			Effective Mass Ratio (%)			Mode Reduction Factor H
			X (Right Angle)	Y (Vertical)	Z (Water Flow)	X	Y	Z	
1	0.628	1.592	-45.02	0.00	0.00	39%	0%	0%	0.08959
2	0.658	1.520	0.00	-4.55	-43.83	0%	0%	39%	0.09916
3	2.382	0.420	-28.61	0.00	0.00	16%	0%	0%	0.05995
4	3.853	0.260	44.84	0.00	-0.05	39%	0%	0%	0.08132
5	3.884	0.257	0.05	-2.83	52.45	0%	0%	56%	0.09365
6	4.516	0.221	0.00	5.98	-16.37	0%	1%	5%	0.05714
7	5.225	0.191	-2.57	0.00	0.00	0%	0%	0%	0.05022
8	6.139	0.163	-17.01	0.00	0.00	6%	0%	0%	0.06508
9	8.321	0.120	1.60	0.01	0.00	0%	0%	0%	0.05032
10	9.276	0.108	1.54	-0.01	0.00	0%	0%	0%	0.05063
11	9.785	0.102	0.00	65.18	0.73	0%	98%	0%	0.09828
12	11.314	0.088	-2.46	0.00	0.00	0%	0%	0%	0.05183
13	16.590	0.060	0.88	0.00	0.00	0%	0%	0%	0.05060

Legend
: Flow Direction Predominant Mode
: Flow Normal Predominant Mode

Source: Study team



Flow Direction [4th mode]

Perpendicular Direction to the Flow [1st mode]

Source: Study team

Figure7.4.160 End Pier Characteristic Analysis Result

(d) Valuation Formula Based On Checking

(i) Formula For Calculating Shear Capacity

Shear strength Ps will be calculated by the following formula shown in the Specifications for Highway Bridges V Seismic Design[9]. P_{s0} is represented by Cc = 1.0 in the formula.

$$P_s = S_C + S_S$$

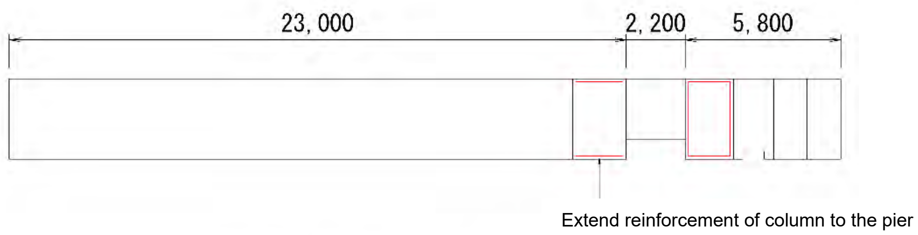
$$S_C = C_C C_e C_{pt} \tau_C bd$$

$$S_S = \frac{A_W \sigma_{sy} d(\sin\theta + \cos\theta)}{1.15a}$$

Where,

S_c	:	Shear capacity that concrete bears (kN)
τ_c	:	Average shear stress that concrete can bear (N/mm2)
C_c	:	Correction factor for the effect of reversed cyclic loading
C_e	:	Correction factor for effective height *d* of bridge cross section
C_{pt}	:	Correction factor for axial tensile bar ratio P_t
b	:	Width of the bridge cross section perpendicular to the direction of the shear capacity calculation (mm)
d	:	Effective height of bridge cross section parallel to the direction of the shear capacity calculation (mm)
P_t	:	The axial tensile rebar ratio, the value obtained by dividing the sum of the cross-sectional areas of the main rebar on the tensile side from the neutral axis by *bd* (%)
S_s	:	Shear capacity that the band rebar bears (kN)
Aw	:	Cross-sectional area of band rebars arranged at spacing *a* and angle θ (mm^2)
$\sigma\ s_y$	:	Yielding point of band rebar (N/mm2)
θ	:	Angle between the band rebar and the vertical axis (°)
a	:	Spacing between band rebars (mm)

Shear capacity was calculated for each pier and column. As a representative example, regarding a pier and a column, based on **Figure7.4.161** and **Figure7.4.162**, the results of the calculation of the shear capacity based on the reinforcement work procedure shown in **Table7.4.207**.



1 Main Reinforcement in Rear Side ··· Cover 120 mm(Same as Column)
D25 x 120 Nos/ 28.80 m (Equivalent to 250 mm interval)
D25@125

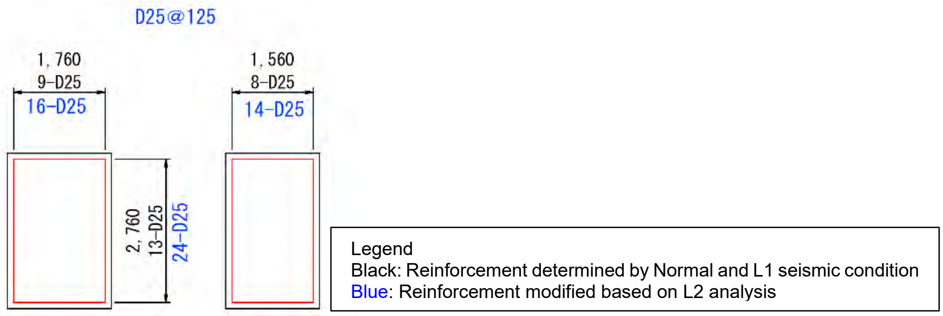
2 Compressive Reinforcement in Front Side ··· Cover 120 mm(Same as Column)
D16@250
D20@125
Lateral Tie D16 x 2 Nos.@150 (Flow Direction)
D16 x 20 Nos.@150 (Perpendicular Direction to the Flow)

Legend
Black: Reinforcement determined by Normal and L1 seismic condition
Blue: Reinforcement modified based on L2 analysis

Source: Study team

Figure7.4.161 Weir Reinforcement Method

[9] Specifications for Highway Bridges V Seismic Design P186



Lateral Tie D16 x 2 Nos.@150 (Flow Direction and Perpendicular Direction to the Flow)

Source: Study team

Figure7.4.162 Column Reinforcement Procedure

Table7.4.207 Shear Capacity Calculation Result

Item	Units	Pier		Column	
		Flow Direction	Perpendicular Direction to The Flow	Flow Direction	Perpendicular Direction to The Flow
		Value	Value	Value	Value
Ps	kN	38,035.2	42,045.2	5,182.6	3055.1
Sc	kN	8,045.2	13,022.6	1,545.1	1092.8
Ss	kN	29,990.0	29,022.6	3,637.5	1962.3
τc	N/mm2	0.36	0.36	0.360	0.36
Aw	mm2	402.12	4021.2	804.2	402.12
σ sy	N/mm2	415	415	415	345
b	m	2.25	28	3.0	1.8
d	m	31	3	1.88	2.44
Cc		0.8	0.8	0.8	0.8
Cdc		1	1	1	1
Cds		-	-	-	-
Ce		0.5	0.7	0.894	0.784
Cpt		0.801	0.769	1.064	1.102
CN		-	-	-	-
a	m	0.15	0.15	0.150	0.15
Angle	°	90	90	90	90

Source: Study team

(ii) Judgement of Failure Mode

In the seismic performance analysis for columns and piers of floodgates and weirs, it is necessary to identify the characteristics of failure that the columns and piers may suffer and to take appropriate rebars and measures to satisfy the seismic performance according to the form of the failure. Therefore, the failure mode is determined from the following formulae.

$P_u \leqq P_s$: Bending failure type

$P_s < P_u \leqq P_{s0}$: Transition type from bending failure to shear failure

$P_{s0} < P_u$: Shear failure type

Where,

P_u : Ultimate horizontal strength (kN)

P_s : Shear strength (kN)

P_{s0} : Shear strength calculated with a correction factor of 1.0 for the effect of reversed cyclic loading

(iii) Lateral Load Carrying Capacity Analysis

The lateral load bearing capacity at the time of an earthquake is determined by the following equation.

$$k_h W \leqq P_a$$

Where,

k_h	:	Horizontal Seismic Coefficient
W	:	Equivalent Mass Of A Structure
P_a	:	Lateral Load Carrying Capacity

The lateral load carrying capacity P_a will be calculated as follows according to the failure mode.

$P_a = P_u$: Bending failure type ($P_c < P_u$)
$P_a = P_u$: Transition type from bending failure to shear failure
$P_a = P_{s0}$: Shear failure type

Where,

P_c : Horizontal resistance against cracking (kN)

Regardless of failure mode, if $P_a \geqq k_{hc}W$ is satisfied, it is judged that the lateral load carrying capacity at the time of earthquake is adequate (OK). The constants required to check the lateral load carrying capacity are calculated as shown below.

A. Final Lateral Load Carrying Capacity Pu

The final lateral load carrying capacity P_u is calculated by the following equation in consideration of the plastic hinge which occurs at the damaged cross section.

$$P_u = \frac{M_u}{h}$$

Where,

M_u	:	Final bending moment (N・mm)
h	:	The height from the lower end to the upper structure to which inertia force works (mm)

B. Design Horizontal Seismic Coefficient Khc

Design horizontal seismic coefficient for Level 2 earthquake motion will be calculated by the following formulae.

$$k_{hc} = C_s C_z k_{hc0}$$

Where,

C_s	:	Correction factor of structure characteristics
$C_z\, k_{hc0}$	:	Horizontal seismic coefficient calculated from natural period and design response spectrum

Correction factor of structure characteristics C_s is obtained as follows.

$$C_S = \frac{1}{\sqrt{2\mu_a - 1}}$$

Where,

μ_a : Allowable plasticity rate of reinforced concrete columns

μ_a is calculated as follows according to failure mode.

【Bending failure type】

$$\mu_a = 1 + \frac{\delta_u - \delta_y}{\alpha \delta_y}$$

Where,

δ_u	:	Final displacement (mm)
δ_y	:	Yield displacement (mm)

α : Safety factor (**Table7.4.208**)

[Transition type from bending failure to shear failure]

$\mu_a = 1.0$

Table7.4.208 Safety Factor to Calculate Allowable Plasticity of Reinforced Concrete Piers (for Bending Failure Type)

Seismic Performance	Safety Factor α Used To Calculate Allowable Plasticity Rate For The Type I Earthquake Motion	Safety Factor α Used To Calculate Allowable Plasticity Rate For The Type II Earthquake Motion
Seismic Performance 2	3.0	1.5
Seismic Performance 3	2.4	1.2

Source : Specifications for Highway Bridges V Seismic Design

C. Equivalent Weight W

Equivalent weight W will be calculated as follows.

$W = W_u + C_p W_p$

Where,

C_p : Coefficient of equivalent weight
W_u : Weight of superstructure supported by the column concerned (N)
W_p : Weight of column or pier (N)

Table7.4.209 Coefficient of Equivalent Weight C_p

Transition Type From Bending Failure To Shear Failure	Shear Failure Type
0.5	1.0

Source : Specifications for Highway Bridges V Seismic Design

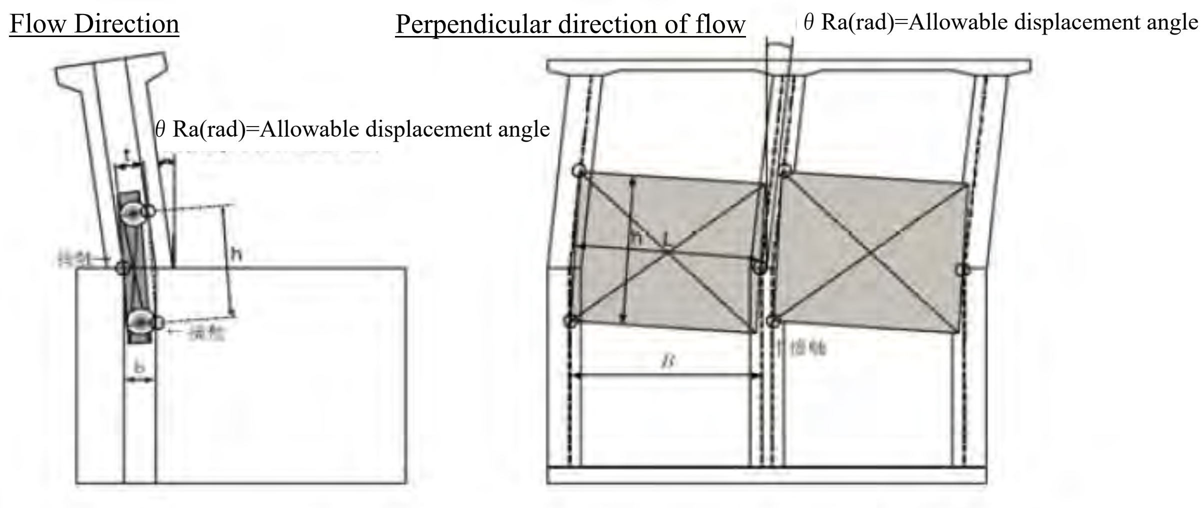
(e) Residual Displacement Analysis

$\delta_R \leqq \delta_{Ra}$

Where,

δ_R : Residual Displacement
δ_{Ra} : Allowable Residual Displacement (δ_{Ra1} or δ_{Ra2})
δ_{Ra1} : Allowable residual displacement (mm) of columns and piers, in principle, residual displacement that does not prevent the opening and closing of the gate (**Figure7.4.163**).
δ_{Ra2} : Allowable residual displacement (mm) of columns and piers : It shall be 1/100 of the height from the bottom of the pier or pier to the position of the inertia force of the upper structure.

Residual displacement in the seismic performance 2 is calculated from deformation angle (allowable residual deformation angle) that does not hinder opening and closing of the gate. It is calculated by the formula shown in **Figure7.4.163**. If the allowable residual displacement is less than h/100, the more severe value is used.



$$\theta Ra = 2\cos^{-1}\left(\frac{h^2-4b^2}{-4bt+h\sqrt{h^2+4t^2-4b^2}}\right)$$

h : Distance between rollers (or gate height)
L : Diameter of Roller(or gate thickness)
B : width between gate guide

$$\theta Ra = 2\cos^{-1}\left(\frac{h^2-4B^2}{-4BL+h\sqrt{h^2+4L^2-4B^2}}\right)$$

h : Gate height
L : Gate width
B : width between gate guide

Source: Performance Based Seismic Design Criteria for River Structures IV

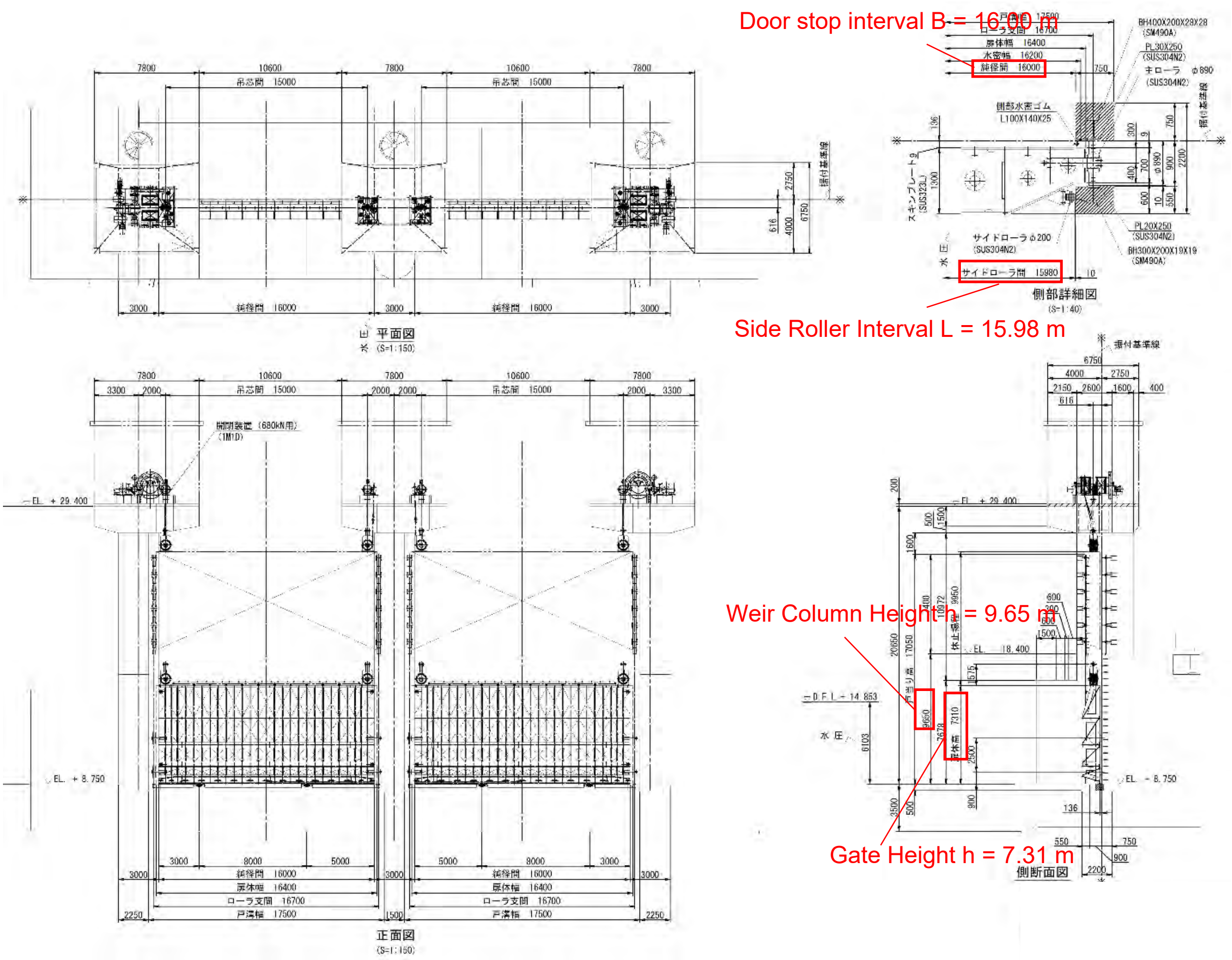
Figure7.4.163 Calculation Method of Deformation Angle (Allowable Residual Deformation Angle) That Does Not Hinder Opening and Closing of the Gate

Calculation results of allowable residual displacement and the calculation basis are shown in **Table7.4.210** and **Figure7.4.164**.

Table7.4.210 Calculation result of allowable residual displacement

Item		Units	Value	Remarks
Weir Height	H	m	9.650	
Door Height	h	m	7.310	
Side Roller Spacing	L	m	15.980	
Door Clearance	B	m	16.000	
Allowable Residual Angle (Calculated Value)	θra1	rad	0.011	
Allowable Residual Angle (1/100)	θRa2	rad	0.010	
Acceptable Residual Angle Adoption Value	θRa	rad	0.010	The smaller value of θRa1 and θRa2
Allowable Residual Displacement	δRa	m	0.097	H θRa

Source: Study team



Source: Study team

Figure7.4.164 Basis for Calculation of Allowable Residual Displacement

4) Analysis Result on Columns and Piers

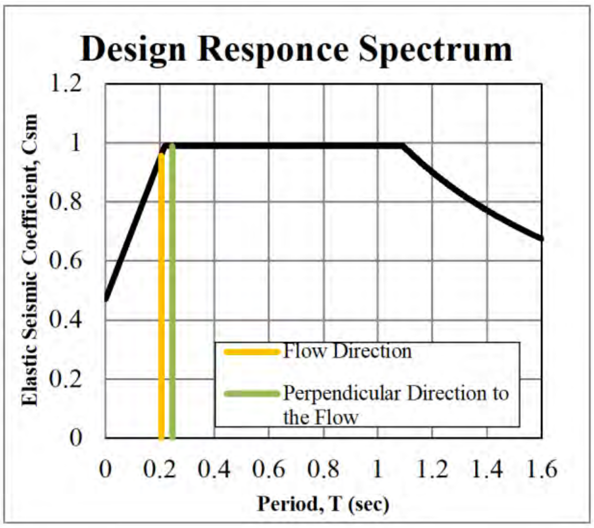
(a) Setting of Design Horizontal Seismic Intensity

The calculation results of horizontal seismic intensity is shown in **Table7.4.211** and **Figure7.4.165** and **Figure7.4.166** shows it in the acceleration spectrum.

Table7.4.211 Calculation Result of Design Horizontal Seismic Intensity

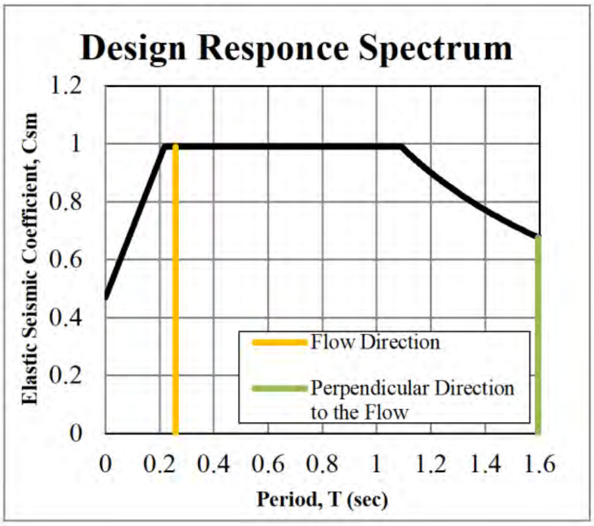
Item		Natural Period T (s)	Design Horizontal Seismic Coefficient khc0
End Pier	Flow Direction	0.203	0.95
	Perpendicular Direction To The Flow	0.246	0.99
Center Pier	Flow Direction	0.257	0.99
	Perpendicular Direction To The Flow	1.592	0.68

Source: Study team



Source: Study team

Figure7.4.165 Calculation Result of Horizontal Seismic Coefficient for End Pier Design



Source: Study team

Figure7.4.166 Results of Calculation of Horizontal Seismic Coefficient for Center Pier Design

(b) Results of Analysis by the Seismic Horizontal Capacity Method

Results of analysis of the end pier by the seismic horizontal capacity method in the flow direction and in the perpendicular direction are shown in **Table7.4.212** and **Table7.4.213**. It was confirmed that the seismic performance was satisfied by the reinforcement arrangement procedure shown in **Figure7.4.167** to **Figure7.4.168**. The pier in flow direction is a shear failure type. In general, it is desirable to design the pier to be a bending failure type in seismic design, but considering the following, however, the pier allows a shear failure type due to the consideration of the following items.

- The effective height in the water flow direction is large, and the bending strength is very large comparing to the shear strength, and it is not rational to make it the bending fracture type.
- The main plasticizing member is the column, which has sufficient shear capacity at the design horizontal seismic intensity $k_{hc} = 0.59$.

Table7.4.212 Results of Analysis by the Seismic Horizontal Capacity Method (End Pier Flow Direction)

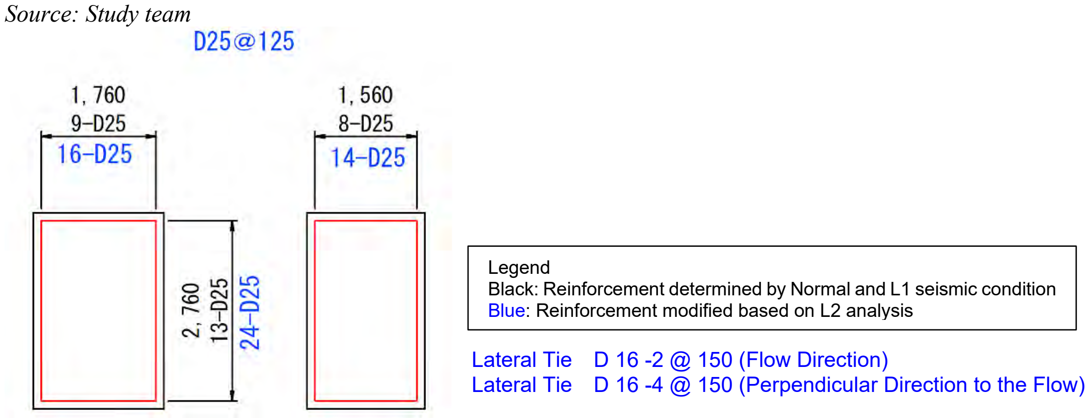
Checking Direction		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Main Plasticizing Member		Column	Column	Pier	Pier
Determination of Fracture Mode					
Response Shear Force	S (kN)	4,531.1	4,531.1	58,209.1	58,209.1
Shear Capacity	Ps (kN)	5,182.6	5,182.6	38,035.2	38,035.2
Shear Capacity	Ps0 (kN)	5,568.9	5,568.9	40,046.5	40,046.5
Decision Formula		S ≦ Ps	S ≦ Ps	Ps0 < S	Ps0 < S
Fracture Morphology		Bending Failure Type	Bending Failure Type	Shear Failure Type	Shear Failure Type
Checking By The Seismic Horizontal Load Bearing Capacity Method					
Safety Factor	1.5	1.5	1.5	1.5	1.5
Allowable Plasticity	μa	1.871	1.870	1.000	1.000
Structure Characteristic Correction Factor	cs	0.604	0.604	1.000	1.000
Regional Correction Factor	cz	1.00	1.00	1.00	1.00
Standard Value Of Design Horizontal Seismic Coefficient	khc0	0.95	0.95	0.95	0.95
Design Horizontal Seismic Coefficient	khc	0.57	0.57	0.95	0.95
Design Horizontal Seismic Intensity At The Horizontal Load Bearing Capacity During An Earthquake	kha	1.40	1.40	1.38	1.38
Result of Checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of Residual Displacement					
Residual Displacement	δR (mm)	0.0	0.0	0.0	0.0
Maximum Response Plasticity	μrT	0.729	0.730	0.739	0.738
Allowable Residual Displacement	δRa (mm)	90.0	90.0	96.5	96.5
Result of Checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study team

Table7.4.213 Results of Analysis by the Seismic Horizontal Capacity Method (End Pier, Perpendicular Direction to the Flow)

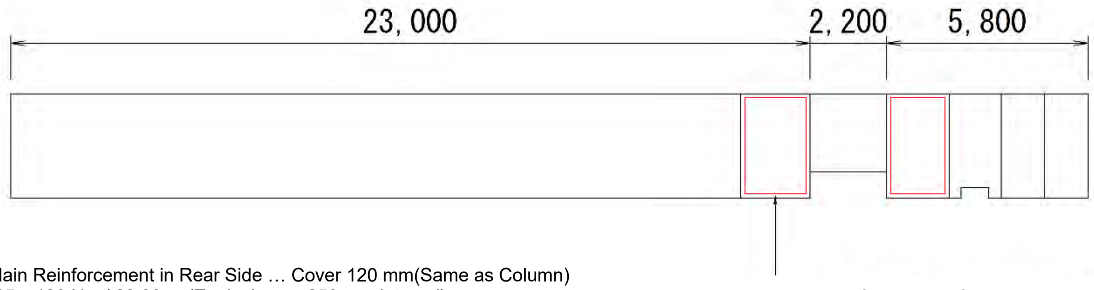
Checking Direction		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Main Plasticizing Member		Column	Column	Pier	Pier
Determination of Fracture Mode					
Response Shear Force	S (kN)	2,811.1	3,145.2	39,716.7	20,180.1
Shear Capacity	Ps (kN)	3,453.3	3,453.3	42,045.2	42,045.2
Shear Capacity	Ps0 (kN)	3,726.5	3,726.5	45,300.9	45,300.9
Decision Formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture Morphology		Bending Failure Type	Bending Failure Type	Bending Failure Type	Bending Failure Type
Checking By The Seismic Horizontal Load Bearing Capacity Method					
Safety Factor	1.5	1.5	1.5	1.5	1.5
Allowable Plasticity	μa	1.907	4.059	2.338	3.442
Structure Characteristic Correction Factor	Ccs	0.596	0.375	0.522	0.412
Regional Correction Factor	cz	1.00	1.00	1.00	1.00
Standard Value Of Design Horizontal Seismic Coefficient	khc0	0.990	0.990	0.990	0.990
Design Horizontal Seismic Coefficient	khc	0.59	0.40	0.52	0.41
Design Horizontal Seismic Intensity At The Horizontal Load Bearing Capacity During An Earthquake	kha	0.96	1.05	0.67	0.88
Result of Checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of Residual Displacement					
Residual Displacement	δR (mm)	6.1	0.0	7.1	1.6
Maximum Response Plasticity	μrT	1.037	0.948	1.608	1.13
Allowable Residual Displacement	δRa (mm)	90.0	90.0	96.5	96.5
Result of Checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study team



Source: Study team

Figure7.4.167 Bar Arrangement of End Pier Column



1 Main Reinforcement in Rear Side … Cover 120 mm(Same as Column)
D25 x 120 Nos/ 28.80 m (Equivalent to 250 mm interval)
→Equivalent to D25@125

2 Compressive Reinforcement in Front Side … Cover 120 mm(Same as Column)
D16@250
→D20@125
Lateral Tie D16 x 2 Nos.@150 (Flow Direction)
D16 x 20 Nos.@150 (Perpendicular Direction to the Flow)

Legend
Black: Reinforcement determined by Normal and L1 seismic condition
Blue: Reinforcement modified based on L2 analysis

Source: Study team

Figure7.4.168 Bar Arrangement of End Pier

Results of the analysis of the center pier by seismic horizontal capacity method in the flow direction and in the perpendicular direction to the flow are shown in **Table7.4.214** and **Table7.4.215**. It was confirmed that the seismic performance was satisfied by the reinforcement arrangement procedure shown in **Figure7.4.169** to **Figure7.4.170**. Like the end pier, the direction of flow of the pier is a shear failure type. In general, it is desirable to design the weir so as to have a bending failure type in the seismic design. However, the pier allows to have a shear failure type in consideration of the following as well as the end pier.

- The effective height in the water flow direction is large, and the bending strength is very large for the shear strength, and it is not rational to use the bending fracture type.
- The main plasticizing member is the column, which has sufficient shear capacity at the design horizontal seismic intensity k_{hc} = 0.71.

Table7.4.214 Results of Analysis by the Seismic Horizontal Capacity Method (Center Pier, Flow Direction)

Checking Direction		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Main Plasticizing Member		Column	Column	Pier	Pier
Determination of Fracture Mode					
Response Shear Force	S (kN)	2,378.4	2,380.1	55,714.5	55,714.5
Shear Capacity	Ps (kN)	2,560.9	2,560.9	43,478.1	43,478.1
Shear Capacity	Ps0 (kN)	2,746.5	2,746.5	44,692.2	44,692.2
Decision Formula		S ≦ Ps	S ≦ Ps	Ps0 < S	Ps0 < S
Fracture Morphology		Bending Failure Type	Bending Failure Type	Shear Failure Type	Shear Failure Type
Checking By The Seismic Horizontal Load Bearing Capacity Method					
Safety Factor	α	1.5	1.5	1.5	1.5
Allowable Plasticity	μa	1.468	1.467	1.000	1.000
Structure Characteristic Correction Factor	Ccs	0.719	0.719	1.000	1.000
Regional Correction Factor	cz	1.00	1.00	1.00	1.00
Standard Value Of Design Horizontal Seismic Coefficient	khc0	0.99	0.99	0.990	0.990
Design Horizontal Seismic Coefficient	khc	0.71	0.71	0.99	0.99
Design Horizontal Seismic Intensity At The Horizontal Load Bearing Capacity During An Earthquake	kha	1.53	1.53	1.60	1.60
Result of Checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of Residual Displacement					
Residual Displacement	δR (mm)	0.0	0.0	0.0	0.0
Maximum Response Plasticity	μrT	0.709	0.709	0.690	0.690
Allowable Residual Displacement	δRa (mm)	90.0	90.0	96.5	96.5
Result of Checking		OK (δ Ra ≧ δ R)	OK (δ Ra ≧ δ R)	OK (δ Ra ≧ δ R)	OK (δ Ra ≧ δ R)

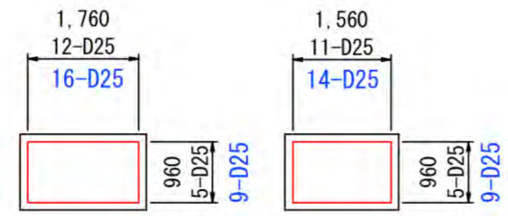
Source: Study team

Table7.4.215 Results of checking by the seismic horizontal capacity method during earthquakes (Center pier and the direction perpendicular to the stream)

Checking Direction		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Main Plasticizing Member		Column	Column	Pier	Pier
Determination of Fracture Mode					
Response Shear Force	S (kN)	1,431.9	1,431.9	22,719.2	22,719.2
Shear Capacity	Ps (kN)	1,816.2	1,816.2	25,776.5	25,776.5
Shear Capacity	Ps0 (kN)	2,009.1	2,009.1	28,592.8	28,592.8
Decision Formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture Morphology		Bending Failure Type	Bending Failure Type	Bending Failure Type	Bending Failure Type
Checking By The Seismic Horizontal Load Bearing Capacity Method					
Safety Factor	α	1.5	1.5	1.5	1.5
Allowable Plasticity	μa	1.870	1.865	3.156	3.155
Structure Characteristic Correction Factor	Ccs	0.604	0.605	0.434	0.434
Regional Correction Factor	cz	1.00	1.00	1.00	1.00
Standard Value Of Design Horizontal Seismic Coefficient	khc0	0.680	0.680	0.680	0.680
Design Horizontal Seismic Coefficient	khc	0.41	0.41	0.40	0.40
Design Horizontal Seismic Intensity At The Horizontal Load	kha	0.97	0.97	0.76	0.76

Checking Direction		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Bearing Capacity During An Earthquake					
Result of Checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of Residual Displacement					
Residual Displacement	δR (mm)	0.0	0.0	0.0	0.0
Maximum Response Plasticity	μrT	0.745	0.745	0.890	0.891
Allowable Residual Displacement	δRa (mm)	90.0	90.0	96.5	96.5
Result of Checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study team



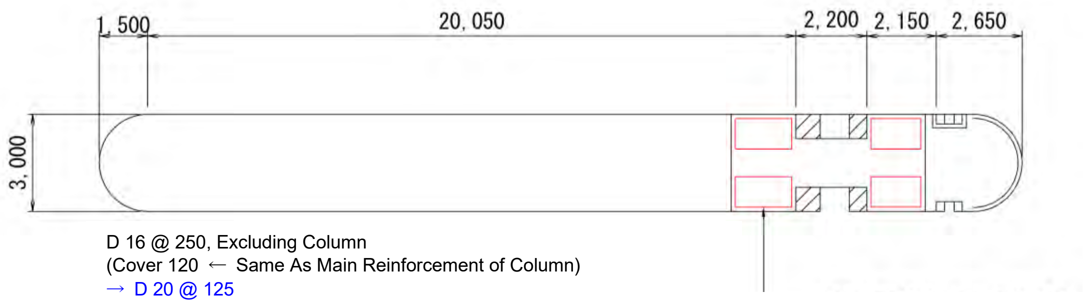
Legend
Black: Reinforcement determined by Normal and L1 seismic condition
Blue: Reinforcement modified based on L2 analysis

Lateral Tie D 16 x2 Nos. @ 150 (both flow and perpendicular to stream)

Source: Study team

Figure7.4.169 Bar Arrangement of Center Pier Column

Legend
Black: Reinforcement determined by Normal and L1 seismic condition
Blue: Reinforcement modified based on L2 analysis



D 16 @ 250, Excluding Column
(Cover 120 ← Same As Main Reinforcement of Column)
→ D 20 @ 125

Extend reinforcement of column to the pier

Lateral Tie D 20 X 2 Nos. @ 150 (Flow Direction)
D 16 X10 Nos. @ 150 (Perpendicular Direction to the Flow)

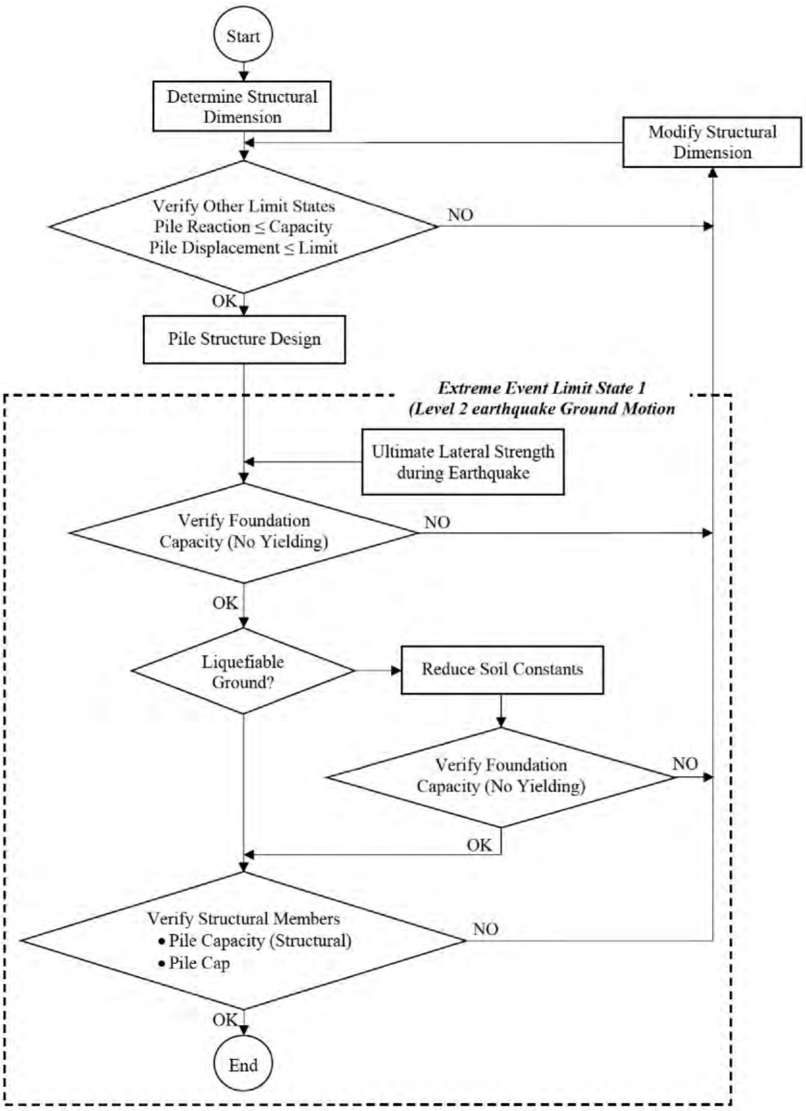
Source: Study team

Figure7.4.170 Bar Arrangement of Center Pier

5) Analysis on Pile Foundation

(a) Summary

The pile foundation is checked by the seismic horizontal capacity method in L2 earthquake. The flow of analysis is indicated in **Figure7.4.171**.



Source: BSDS.p5 -15

Figure7.4.171 L2 Analysis Flow of Pile Foundation

(b) Standard

Conform to the following standards.

- Road Bridge Specifications I Common (March 2012)
- Road Bridge Specifications III Concrete Bridge (March 2012)
- Road Bridge Specifications IV Substructure (March 2012)
- Road Bridge Specifications V Seismic Design (March 2012)
- Pile Foundation Design Manual (March 2015)
- Seismic Design of Road Bridges (March 1997)

(c) Materials used and allowable stress

The material used and allowable stress were set as follows.

Units: N/mm2

No	Extra Factor	Allowable Bending Compressive Stress σ ca		Allowable Bending Tensile Stress σ ta		Allowable Shear Stress τa	
		SKK 400	SKK 490	SKK 400	SKK 490	SKK 400	SKK 490
1	1.00	140.00	185.00	140.00	185.00	80.00	105.00

Source: Road Bridge Specifications IV Substructure, Table-4.4.1

(d) Content of the Analysis

(i) End Pier

A. Calculation Condition

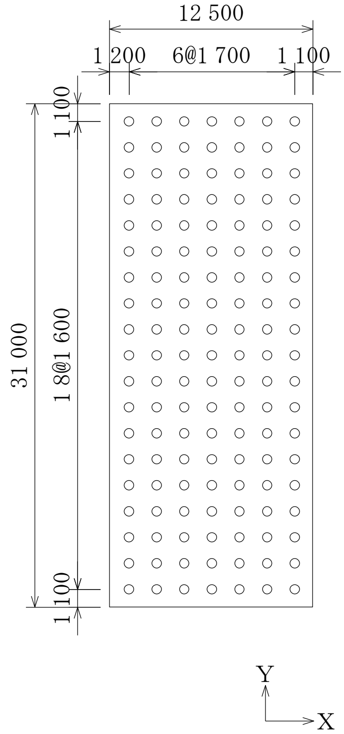
The calculation conditions are as follows.

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 133 (nos.)
・Pile Diameter		: 600.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 33.90 (m)

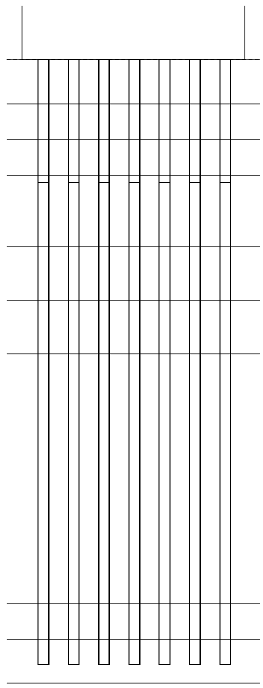
First Section: 6.90 (m) 16.0 (mm) SKK 400]

Second Section: 27.00 (m) 9.0 (mm) SKK 400]

The pile layout and side views are shown in **Figure7.4.172**, a list of soil properties is indicated in **Table7.4.216**.



(a)Layout Plan



--- Design Ground Plane (Normal Condition)
—-— Design Ground Plane (Seismic Condition)
(b)Side View (Perpendicular Direction to the Bridge Axis)

Source: Study team

Figure7.4.172 Pile Arrangement Plan And Side View

Table7.4.216 List of Soil Properties

layer No	Soil Type	Thickness (m)		Average N-value	α · Eo (kN/m2)		γ (kN/m3)		f (kN/m2)		Liquefaction Reduction Factor DE
		Normal	Seismic		Normal	Seismic	γ	γ′	f	fn	
1	Cohesive Soil	2.490	2.490	1.0	6,000	12,000	15.00	6.20	0.0	10.0	—
2	Cohesive Soil	2.000	2.000	11.0	30,800	61,600	17.00	8.20	110.0	110.0	—
3	Cohesive Soil	2.000	2.000	2.0	6,000	12,000	16.00	7.20	0.0	20.0	—
4	Cohesive Soil	4.000	4.000	8.0	22,400	44,800	17.00	8.20	80.0	80.0	—
5	Cohesive Soil	3.000	3.000	16.0	44,800	89,600	18.00	9.20	150.0	150.0	—
6	Sandy Soil	3.000	3.000	26.0	72,800	145,600	20.00	11.20	52.0	52.0	2/3
7	Cohesive Soil	14.000	14.000	20.0	56,000	112,000	18.00	9.20	150.0	150.0	—
8	Cohesive Soil	2.000	2.000	34.0	95,200	190,400	19.00	10.20	150.0	150.0	—
9	Sandy Soil	1.410	1.410	50.0	140,000	280,000	21.00	12.20	100.0	100.0	—

Source: Study team

B. Calculated Result

The stability calculation result in the flow direction and the analysis result of members are shown in **Table7.4.217**. As a result of the stability calculation, the foundation did not reach yield, and the seismic performance is satisfied.

Table7.4.217 Stability Calculation Results In Flow Direction (End Pier)

Liquefaction	Water Level	Evaluation	Yield of Foundation	Response Plasticity Rate		Displacement	
				Response Plasticity Rate	Allowable Plasticity	Rotation Angle (Rad)	Allowable Displacement (Rad)
Ignore	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Ignore	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	

Source: Study team

The result of checking the members (pile head) in the flow direction is shown in **Table7.4.218**. In the bar arrangement(D 32 -17 pieces) shown in **Figure7.4.173**, it is confirmed that the pile head bending moment is lower than the yield bending moment, My of the virtual RC cross section, and the seismic performance is satisfied.

Table7.4.218 Results of Checking Members in the Flow Direction (End Pier)

Liquefaction	Water Level	Evaluation	Pile Head		
			Pile Body My, Pile Head M (kNm/Pile)		Virtual RC Cross Section My (kNm/Pile)
Ignore	Buoyancy ignored	OK	421.62	≦	983.40
Ignore	Buoyancy Considered	OK	421.62	≦	983.40
Considered	Buoyancy ignored	OK	421.62	≦	983.40
Considered	Buoyancy Considered	OK	421.62	≦	983.40

Note: The shown result is the strictest against the limits
The shear check of the pile shows the sum of the shear force of the pile head and the sum of the shear capacity of the pile head.
The check of the pile head is evaluated OK, when all the piles satisfy the limit value.
Source: Study team

Detailed calculations are shown in V**ol.5A Structural Calculation for Contract Package-1**.

The results of stability calculation in the perpendicular direction to the flow and the results of checking the members are shown in **Table7.4.219** . As a result of stability calculation even in the perpendicular direction to the flow, the foundation does not reach yield, and the seismic performance is satisfied.

Table7.4.219 Stability Calculation Result In Perpendicular Direction to the Flow (End Pier)

Liquefaction	Water Level	Evaluation	Yield of Foundation	Response Plasticity Rate		Displacement	
				Response Plasticity Rate	Allowable Plasticity	Rotation Angle (Rad)	Allowable Displacement (Rad)
Ignore	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Ignore	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	

Source: Study team

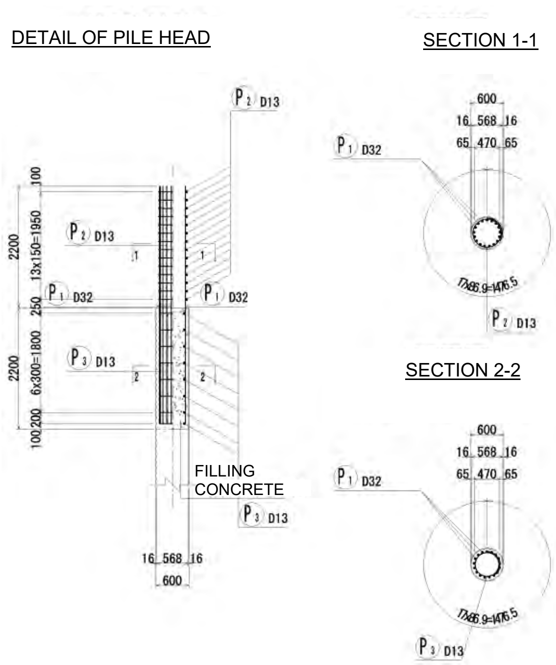
The result of checking the member (Pile head and bottom slab) in the perpendicular direction to the flow is shown in **Table7.4.220**. In case of the bar arrangement of pile head (D 32 -17 pieces) indicated in **Figure7.4.173**, and the bar arrangement specifications of the bottom slab

(Overhang section, upper side: D 28 @ 125, Lower D 20 @ 250) shown in **Figure7.4.174**, it is confirmed that the pile head bending moment is lower than the yield bending moment, My of the virtual RC cross section, and the seismic performance was satisfied.

Table7.4.220 Results of Checking Members in Perpendicular Direction to the Flow (End Pier)

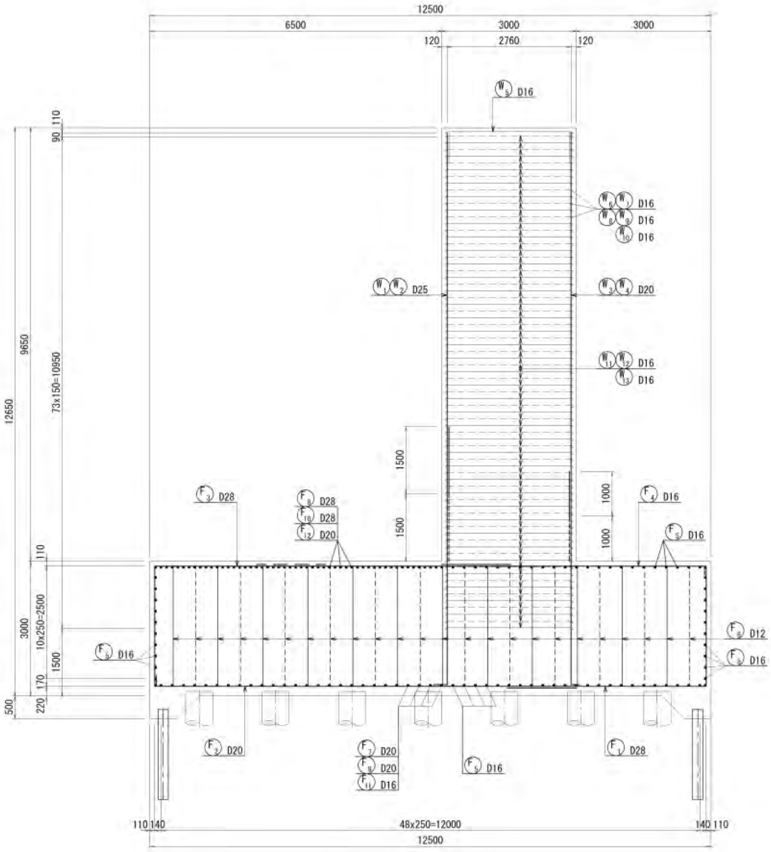
Liquefaction	Water Level	Evaluation	Pile Head	Bottom Slab
			Pile Body My, Pile Head M (kNm/pile) Virtual RC Cross Section My (kNm/pile)	Acting Force (kNm, kN) Bearing Capacity (kNm, kN)
Ignore	Buoyancy ignored	OK	372.90 ≦ 983.40	(1) -2145.47 ≥ -3686.48 (2) -844.50 ≦ 1131.88
Ignore	Buoyancy Considered	OK	372.90 ≦ 983.40	(1) -2145.47 ≥ -3686.48 (2) -844.50 ≦ 1131.88
Considered	Buoyancy ignored	OK	372.91 ≦ 983.40	(1) -2145.47 ≥ -3686.48 (2) -844.50 ≦ 1131.88
Considered	Buoyancy Considered	OK	372.91 ≦ 983.40	(1) -2145.47 ≥ -3686.48 (2) -844.50 ≦ 1131.88

Note: The shown result is the strictest against the limits
The shear check of the pile shows the sum of the shear force of the pile head and the sum of the shear capacity of the pile head.
The check of the pile head is evaluated OK, when all the piles satisfy the limit value.
The bottom slab check indicates (1) checking for bending, (2) checking for shear as a beam, and (3) checking for shear as a plate.
Source: Study team



Source: Study team

Figure7.4.173 Detailed of Pile Head



Source: Study team

Figure7.4.174 Bar Arrangement of Bottom Slab

(ii) Center Pier

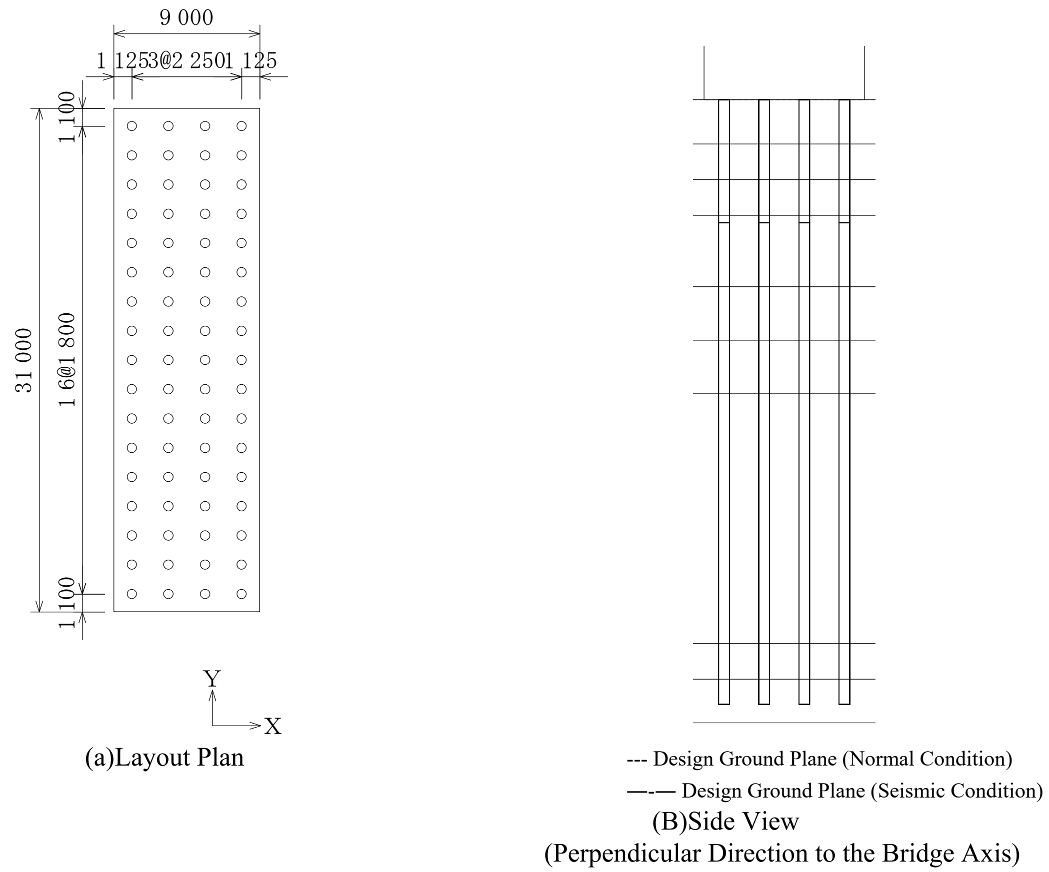
A. Calculation Condition

The calculation conditions are as follows.

・Pile Type		: Steel Pipe Piles
・Construction Method		: Driving Pile (Vibro hammer)
・Pile Cap Connection Condition		: Rigid Ties and Hinges
・Pile Tip Condition		: Type: Hinge
・Type of pile		: Bearing Pile
・Allowable Displacement of Pile	Normal Condition	: 10.0 (mm)
	Seismic Condition	: 10.0 (mm)
・Young's Modulus of Pile Body		: 2.00 x 105 (N/mm2)
・Number of Piles		: 68 (nos.)
・Pile Diameter		: 600.0 (mm)
・Outside Corrosion Allowance		: 1.0 (mm)
・Inside Corrosion Allowance		: 0.0 (mm)
・Design Pile Length, Steel Pipe Thickness, Material		: 33.90 (m)

First Section: 6.90 (m) 16.0 (mm) SKK 400]
Second Section: 27.00 (m) 9.0 (mm) SKK 400]

The pile layout and side views are shown in **Figure7.4.175**, a list of soil properties is shown in **Table7.4.221**.



Source: Study team

Figure7.4.175 Pile Arrangement Plan And Side View

Table7.4.221 List of Soil Properties

layer No	Soil Type	Thickness (m)		Average N-value	α · Eo (kN/m2)		γ (kN/m3)		f (kN/m2)		Liquefaction Reduction Factor DE
		Normal	Seismic		Normal	Seismic	γ	γ′	f	fn	
1	Cohesive Soil	2.490	2.490	1.0	6,000	12,000	15.00	6.20	0.0	10.0	—
2	Cohesive Soil	2.000	2.000	11.0	30,800	61,600	17.00	8.20	110.0	110.0	—
3	Cohesive Soil	2.000	2.000	2.0	6,000	12,000	16.00	7.20	0.0	20.0	—
4	Cohesive Soil	4.000	4.000	8.0	22,400	44,800	17.00	8.20	80.0	80.0	—
5	Cohesive Soil	3.000	3.000	16.0	44,800	89,600	18.00	9.20	150.0	150.0	—
6	Sandy Soil	3.000	3.000	26.0	72,800	145,600	20.00	11.20	52.0	52.0	2/3
7	Cohesive Soil	14.000	14.000	20.0	56,000	112,000	18.00	9.20	150.0	150.0	—
8	Cohesive Soil	2.000	2.000	34.0	95,200	190,400	19.00	10.20	150.0	150.0	—
9	Sandy Soil	1.410	1.410	50.0	140,000	280,000	21.00	12.20	100.0	100.0	—

Source: Study team

B. Calculation Result

The stability calculation result in the flow direction and the check result of members is shown in **Table7.4.222**. As a result of the stability calculation, the foundation does not reach yield, and the seismic performance is satisfied.

Table7.4.222 Flow Direction Stability Calculation Results (center pier)

Liquefaction	Water Level	Evaluation	Yield of Foundation	Response Plasticity Rate		Displacement	
				Response Plasticity Rate	Allowable Plasticity	Rotation Angle (Rad)	Allowable Displacement (Rad)
Ignore	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Ignore	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	

Source: Study team

The result of checking the members (pile head) in the flow direction is shown in **Table7.4.223**. In case of the bar arrangement (D 32 -17 pieces) shown in **Figure7.4.176**, it is confirmed that the pile head bending moment is lower than the yield bending moment, My of the virtual RC cross section, and the seismic performance is satisfied.

Table7.4.223 Results of checking members in the flow direction (center pier)

Liquefaction	Water Level	Evaluation	Pile Head		
			Pile Body My, Pile Head M (kNm/Pile)		Virtual RC Cross Section My (kNm/Pile)
Ignore	Buoyancy ignored	OK	751.05	≦	862.00
Ignore	Buoyancy Considered	OK	751.05	≦	862.00
Considered	Buoyancy ignored	OK	751.05	≦	862.00
Considered	Buoyancy Considered	OK	751.05	≦	862.00

Note: The shown result is the strictest against the limits
The shear check of the pile shows the sum of the shear force of the pile head and the sum of the shear capacity of the pile head.
The check of the pile head is evaluated OK, when all the piles satisfy the limit value.

Source: Study team

Detailed calculations are shown in **Vol.5A Structural Calculation for Contract Package-1**.

The results of stability calculation in the perpendicular direction to the flow and the results of checking the members are shown in **Table7.4.219**. As a result of stability calculation even in the perpendicular direction to the flow, the foundation does not reach yield, and the seismic performance is satisfied.

Table7.4.224 Calculation result of water flow stability in perpendicular direction

Liquefaction	Water Level	Evaluation	Yield of Foundation	Response Plasticity Rate		Displacement	
				Response Plasticity Rate	Allowable Plasticity	Rotation Angle (Rad)	Allowable Displacement (Rad)
Ignore	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Ignore	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy ignored	OK	The foundation has not reached yielding point.	——		——	
Considered	Buoyancy Considered	OK	The foundation has not reached yielding point.	——		——	

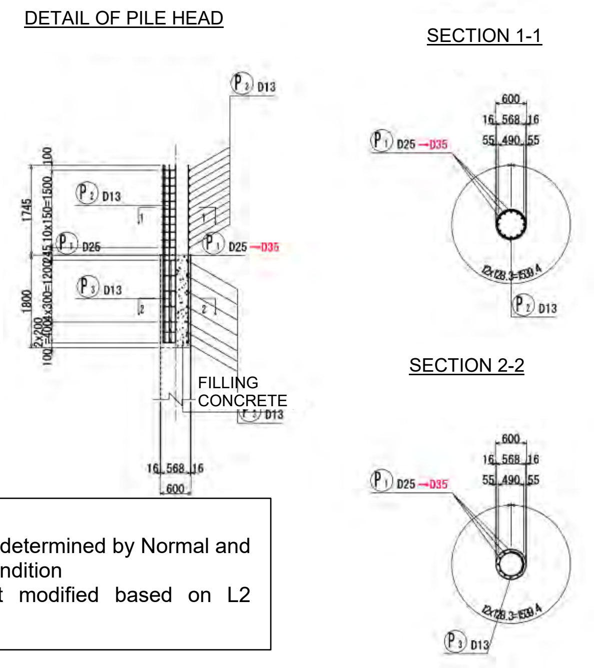
Source: Study team

The result of checking the member (Pile head and bottom slab) in the perpendicular direction to the flow is shown in **Table7.4.223**. In case of the pile head reinforcement (D 35 -12 pieces) shown in **Figure7.4.176** and the bar arrangement specifications of the bottom slab (Overhang section upper side: D 20 @ 250, Lower D 25 @ 250) shown in **Figure7.4.177**, it is confirmed that the pile head bending moment is lower than the yield bending moment, My of the virtual RC cross section, and the seismic performance is satisfied.

Table7.4.225 checking members in the Perpendicular Direction to the Flow

Liquefaction	Water Level	Evaluation	Pile Head: Pile Body My, Pile Head M (kNm/pile) ≦ Virtual RC Cross Section My (kNm/pile)	Bottom Slab: Acting Force (kNm, kN) vs Bearing Capacity (kNm, kN)
Ignore	Buoyancy ignored	OK	593.92 ≦ 862.00	(1) -1,145.46 ≥ -1,352.84 (2) 1,340.39 ≦ 2,503.16
Ignore	Buoyancy Considered	OK	593.92 ≦ 862.00	(1) -1,145.46 ≥ -1,352.84 (2) 1,340.39 ≦ 2,503.16
Considered	Buoyancy ignored	OK	593.92 ≦ 862.00	(1) -1,145.46 ≥ -1,352.84 (2) 1,340.39 ≦ 2,503.16
Considered	Buoyancy Considered	OK	593.92 ≦ 862.00	(1) -1,145.46 ≥ -1,352.84 (2) 1,340.39 ≦ 2,503.16

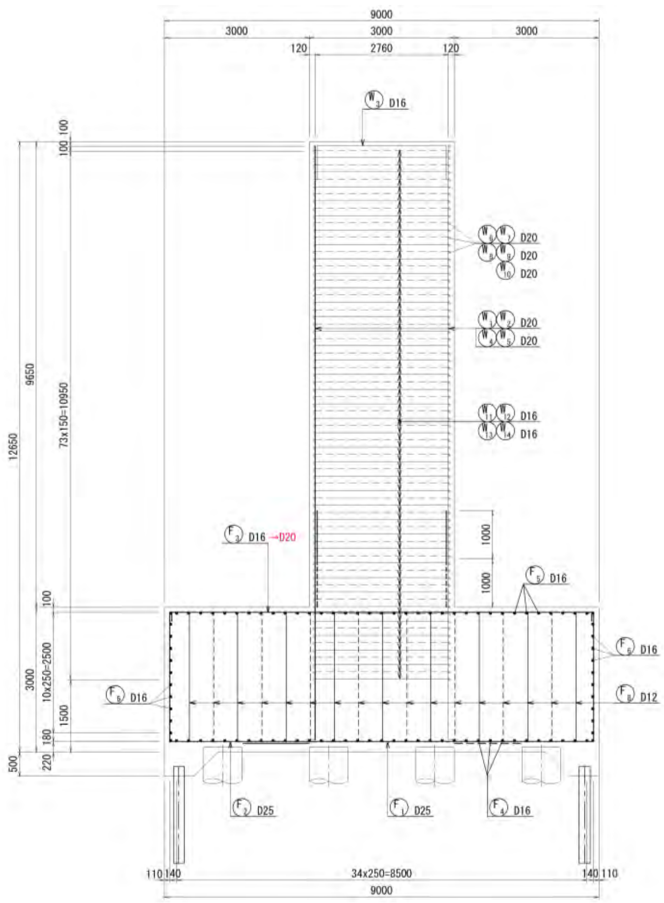
Note: The shown result is the strictest against the limits
The shear check of the pile shows the sum of the shear force of the pile head and the sum of the shear capacity of the pile head.
The check of the pile head is evaluated OK, when all the piles satisfy the limit value.
The bottom slab check indicates (1) checking for bending, (2) checking for shear as a beam, and (3) checking for shear as a plate.
Source: Study team



Legend
Black : Reinforcement determined by Normal and L1 Seismic Condition
Red : Reinforcement modified based on L2 analysis

Source: Study team

Figure7.4.176 Detailed of Pile Head



Legend
Black : Reinforcement determined by Normal and L1 Seismic Condition
Red : Reinforcement modified based on L2 analysis

Source: Study team

Figure7.4.177 Bar Arrangement of Bottom Slab

7.4.2.4 Detailed Design of Maintenance Bridge

The structural calculation is carried out based on the Cainta Floodgate Maintenance Bridge specifications determined in the basic design. The structure calculation results are shown below. Since the span length of this bridge is the same, the calculation results are the same.

(1) Design of the Main Girder

REFERENCE

VI. MOMENT AND SHEAR LOAD :

Unfactored Moment :

MOMENT per GIRDER							
Location	DC KN-m	DW KN-m	Lane Load KN-m	Truck/Tandem KN-m	Permit KN-m	Wind Load KN-m	Fatigue Load KN-m
0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.835	-317.53	-103.71	-139.78	-57.93	-114.20	30.53	-84.40
3.670	-568.34	-184.73	-248.98	-115.85	-211.22	54.38	-162.71
5.505	-752.43	-243.07	-327.60	-168.70	-274.90	71.55	-226.24
7.340	-861.46	-278.72	-375.65	-213.06	-338.58	82.05	-287.14
9.175	-899.58	-291.69	-393.12	-255.71	-368.28	85.86	-269.79
11.010	-861.46	-278.72	-375.65	-242.85	-339.46	82.05	-233.34
12.845	-752.43	-243.07	-327.60	-221.36	-305.35	71.55	-196.89
14.680	-568.34	-184.73	-248.98	-167.80	-236.66	54.38	-160.44
16.515	-317.53	-103.71	-139.78	-83.90	-123.10	30.53	-118.86
18.350	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Unfactored Shear :

SHEAR per GIRDER							
Location	DC KN	DW KN	Lane Load KN	Truck/Tandem KN	Permit KN	Wind Load KN	Fatigue Load KN
0.000	193.82	63.58	85.70	64.31	91.40	-18.72	82.33
1.835	156.42	50.87	68.56	30.35	41.28	-14.97	30.09
3.670	119.02	38.15	51.42	16.77	41.28	-11.23	30.09
5.505	81.62	25.43	34.28	-3.61	26.25	-7.49	30.09
7.340	38.76	12.72	17.14	-3.61	-3.82	-3.74	30.09
9.175	0.00	0.00	0.00	-11.80	-8.83	0.00	-22.16
11.010	-38.76	-12.72	-17.14	-11.80	-14.27	3.74	-22.16
12.845	-81.62	-25.43	-34.28	-11.80	-36.00	7.49	-27.20
14.680	-119.02	-38.15	-51.42	-11.80	-36.00	11.23	-34.77
16.515	-156.42	-50.87	-68.56	-11.80	-36.00	14.97	-34.77
18.350	-193.82	-63.58	-85.70	-11.80	-36.00	18.72	-34.77

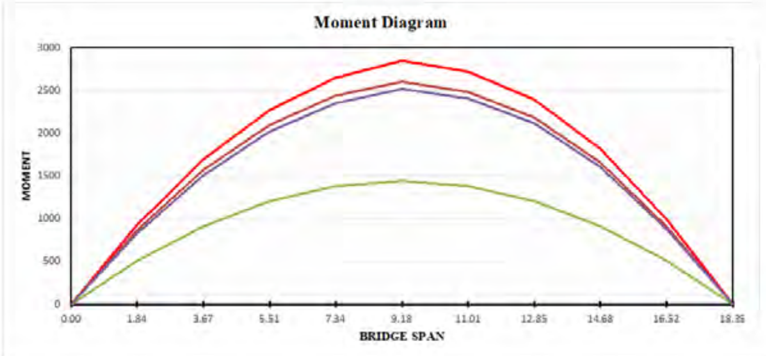
Factored Moment :

MOMENT per GIRDER (Strength I Limit State)						
Load Factor	1.25	1.50	1.75	1.75	1.75	Total
Location	DC KN-m	DW KN-m	Lane Load KN-m	Truck/Tandem KN-m	IM KN-m	KN-m
0.000	0.00	0.00	0.00	0.00	0.00	**0.00**
1.835	-396.91	-155.57	-244.61	-101.37	-33.45	**-931.91**
3.670	-710.43	-277.10	-435.71	-202.74	-66.91	**-1692.89**
5.505	-940.54	-364.61	-573.31	-295.23	-97.43	**-2271.11**
7.340	-1076.82	-418.08	-657.39	-372.86	-123.04	**-2648.19**
9.175	-1124.48	-437.53	-687.97	-447.50	-147.67	**-2845.15**
11.010	-1076.82	-418.08	-657.39	-424.98	-140.24	**-2717.52**
12.845	-940.54	-364.61	-573.31	-387.38	-127.84	**-2393.67**
14.680	-710.43	-277.10	-435.71	-293.66	-96.91	**-1813.81**
16.515	-396.91	-155.57	-244.61	-146.83	-48.45	**-992.37**
18.350	0.00	0.00	0.00	0.00	0.00	**0.00**

MOMENT per GIRDER (Strength II Limit State)						
Load Factor	1.25	1.50	1.35	1.35	1.35	Total
Location	DC KN-m	DW KN-m	Lane Load KN-m	Permit KN-m	IM KN-m	KN-m
0.000	0.00	0.00	0.00	0.00	0.00	**0.00**
1.835	-396.91	-155.57	-188.70	-78.20	-50.88	**-870.26**
3.670	-710.43	-277.10	-336.12	-156.40	-94.10	**-1574.15**
5.505	-940.54	-364.61	-442.26	-227.75	-122.47	**-2097.63**
7.340	-1076.82	-418.08	-507.13	-287.63	-150.84	**-2440.50**
9.175	-1124.48	-437.53	-530.72	-345.21	-164.07	**-2602.01**
11.010	-1076.82	-418.08	-507.13	-327.84	-151.23	**-2481.11**
12.845	-940.54	-364.61	-442.26	-298.84	-136.03	**-2182.28**
14.680	-710.43	-277.10	-336.12	-226.54	-105.43	**-1655.62**
16.515	-396.91	-155.57	-188.70	-113.27	-54.84	**-909.29**
18.350	0.00	0.00	0.00	0.00	0.00	**0.00**

MOMENT per GIRDER (Strength III Limit State)				
Load Factor	1.25	1.50	1.40	Total
Location	DC KN-m	DW KN-m	Wind Load KN-m	KN-m
0.000	0.00	0.00	0.00	**0.00**
1.835	-396.91	-155.57	42.74	**-509.74**
3.670	-710.43	-277.10	76.13	**-911.40**
5.505	-940.54	-364.61	100.17	**-1204.97**
7.340	-1076.82	-418.08	114.87	**-1380.04**
9.175	-1124.48	-437.53	120.21	**-1441.80**
11.010	-1076.82	-418.08	114.87	**-1380.04**
12.845	-940.54	-364.61	100.17	**-1204.97**
14.680	-710.43	-277.10	76.13	**-911.40**
16.515	-396.91	-155.57	42.74	**-509.74**
18.350	0.00	0.00	0.00	**0.00**

MOMENT per GIRDER (Strength V Limit State)							
Load Factor	1.25	1.50	1.35	1.35	1.35	0.40	Total
Location	DC	DW	Lane Load	Truck/Tanden	IM	Wind Load	
	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
0.000	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
1.835	-396.91	-155.57	-188.70	-78.20	-25.81	12.21	**-832.97**
3.670	-710.43	-277.10	-336.12	-156.40	-51.61	21.75	**-1509.91**
5.505	-940.54	-364.61	-442.26	-227.75	-75.16	28.62	**-2021.70**
7.340	-1076.82	-418.08	-507.13	-287.63	-94.92	32.82	**-2351.76**
9.175	-1124.48	-437.53	-530.72	-345.21	-113.92	34.35	**-2517.51**
11.010	-1076.82	-418.08	-507.13	-327.84	-108.19	32.82	**-2405.25**
12.845	-940.54	-364.61	-442.26	-298.84	-98.62	28.62	**-2116.24**
14.680	-710.43	-277.10	-336.12	-226.54	-74.76	21.75	**-1603.19**
16.515	-396.91	-155.57	-188.70	-113.27	-37.38	12.21	**-879.61**
18.350	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**



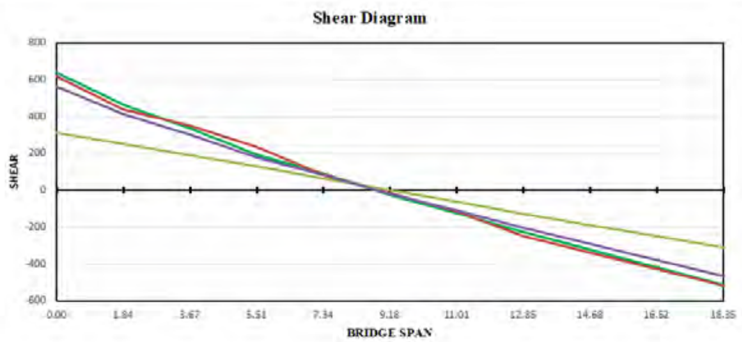
Factored Shear :

SHEAR per GIRDER (Strength I Limit State)						
Load Factor	1.25	1.50	1.75	1.75	1.75	Total
Location	DC	DW	Lane Load	Truck/Tanden	IM	
	KN	KN	KN	KN	KN	KN
0.000	242.27	95.37	149.97	112.55	37.14	**637.30**
1.835	195.53	76.30	119.97	53.12	17.53	**462.44**
3.670	148.78	57.22	89.98	29.35	9.68	**335.01**
5.505	102.03	38.15	59.99	-6.31	-2.08	**191.77**
7.340	48.45	19.07	29.99	-6.31	-2.08	**89.13**
9.175	0.00	0.00	0.00	-20.65	-6.82	**-27.47**
11.010	-48.45	-19.07	-29.99	-20.65	-6.82	**-124.99**
12.845	-102.03	-38.15	-59.99	-20.65	-6.82	**-227.64**
14.680	-148.78	-57.22	-89.98	-20.65	-6.82	**-323.45**
16.515	-195.53	-76.30	-119.97	-20.65	-6.82	**-419.27**
18.350	-242.27	-95.37	-149.97	-20.65	-6.82	**-515.08**

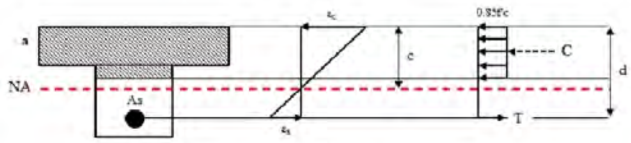
SHEAR per GIRDER (Strength II Limit State)						
Load Factor	1.25	1.50	1.35	1.35	1.35	Total
Location	DC	DW	Lane Load	Permit	IM	
	KN	KN	KN	KN	KN	KN
0.000	242.27	95.37	115.69	123.39	40.72	**617.45**
1.835	195.53	76.30	92.55	55.73	18.39	**438.50**
3.670	148.78	57.22	69.41	55.73	18.39	**349.54**
5.505	102.03	38.15	46.28	35.44	11.69	**233.58**
7.340	48.45	19.07	23.14	-5.16	-1.70	**83.80**
9.175	0.00	0.00	0.00	-11.93	-3.94	**-15.86**
11.010	-48.45	-19.07	-23.14	-19.26	-6.36	**-116.29**
12.845	-102.03	-38.15	-46.28	-48.60	-16.04	**-251.09**
14.680	-148.78	-57.22	-69.41	-48.60	-16.04	**-340.06**
16.515	-195.53	-76.30	-92.55	-48.60	-16.04	**-429.02**
18.350	-242.27	-95.37	-115.69	-48.60	-16.04	**-517.98**

SHEAR per GIRDER (Strength III Limit State)				
Load Factor	1.25	1.50	1.40	Total
Location	DC	DW	Wind Load	
	KN	KN	KN	KN
0.000	242.27	95.37	-26.20	**311.44**
1.835	195.53	76.30	-20.96	**250.86**
3.670	148.78	57.22	-15.72	**190.28**
5.505	102.03	38.15	-10.48	**129.70**
7.340	48.45	19.07	-5.24	**62.29**
9.175	0.00	0.00	0.00	**0.00**
11.010	-48.45	-19.07	5.24	**-62.29**
12.845	-102.03	-38.15	10.48	**-129.70**
14.680	-148.78	-57.22	15.72	**-190.28**
16.515	-195.53	-76.30	20.96	**-250.86**
18.350	-242.27	-95.37	26.20	**-311.44**

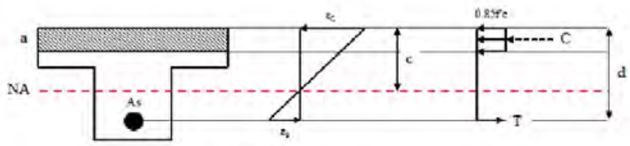
SHEAR per GIRDER (Strength V Limit State)							
Load Factor	1.25	1.50	1.35	1.35	1.35	0.40	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	Wind Load	
	KN	KN	KN	KN	KN	KN	KN
0.000	242.27	95.37	115.69	86.82	28.65	-7.49	**561.32**
1.835	195.53	76.30	92.55	40.98	13.52	-5.99	**412.88**
3.670	148.78	57.22	69.41	22.64	7.47	-4.49	**301.03**
5.505	102.03	38.15	46.28	-4.87	-1.61	-2.99	**176.98**
7.340	48.45	19.07	23.14	-4.87	-1.61	-1.50	**82.70**
9.175	0.00	0.00	0.00	-15.93	-5.26	0.00	**-21.19**
11.010	-48.45	-19.07	-23.14	-15.93	-5.26	1.50	**-110.36**
12.845	-102.03	-38.15	-46.28	-15.93	-5.26	2.99	**-204.65**
14.680	-148.78	-57.22	-69.41	-15.93	-5.26	4.49	**-292.11**
16.515	-195.53	-76.30	-92.55	-15.93	-5.26	5.99	**-379.58**
18.350	-242.27	-95.37	-115.69	-15.93	-5.26	7.49	**-467.04**



VII. DESIGN CALCULATIONS :



FLANGED SECTION



RECTANGULAR SECTION

DGCS 12.2.3	**STRENGTH LIMIT STATE**					
	Design for Flexural and Axial Force Effects :					
	Number of Steel Reinforcement,			ns	=	12 pc/s
	Area of Steel Reinforcement,			As	=	7,389.03 mm^2
	Distance from Extreme Compression Fiber to Neutral Axis,					
DGCS 12.4.3.1-7	Flanged Section, $c = \dfrac{Asfs - 0.85f'c(Weff - bw)ts}{0.85f'c\beta_1 bw}$			c	=	-557.663 mm
DGCS 12.4.3.1-8	Rectangular Section, $c = \dfrac{Asfs}{0.85f'c\beta_1 Weff}$			c	=	76.703 mm
	Depth of Equivalent Stress Block, $a = c\beta_1$			a	=	65.197 mm
	Rectangular Section Governs	a	<	ts	=	200 mm
	Adopt Depth of Equivalent Stress Block for Rectangular Section			a	=	65.197 mm
DGCS 12.4.3.1-8	Steel Yields	0.60	>	c/d	=	0.08599

DGCS 12.4.3.2-2	**Nominal Moment Resistance,**	Mn	=	3,598.08 KN-m
	$Mn = Apsfps\left(dp-\frac{a}{2}\right)+Asfs\left(ds-\frac{a}{2}\right)-A'sf's\left(d's-\frac{a}{2}\right)+0.85f'c(b-bw)ts\left(\frac{a}{2}-\frac{ts}{2}\right)$			
	Area of Prestressing Steel,	Aps	=	0 mm²
	Average Stress in Prestressing Steel at Nominal Bending Resistance,	fps	=	0 MPa
	Distance from Extreme Compression Fiber to Centroid of Prestressing Tendons,	dp	=	0 mm
	Area of Non-Prestressed Tension Reinforcement,	As	=	7,389.03 mm²
	Stress in Mild Steel Tension Reinforcement at Nominal Flexural Resistance,	fs	=	420 MPa
	Distance from Extreme Compression Fiber to Centroid of Non-Prestressed Reinforcement,	ds	=	1,192.00 mm
	Area of Compression Reinforcement,	A's	=	0 mm²
	Stress in Mild Steel Compression Reinforcement at Nominal Flexural Resistance,	f's	=	0 MPa
	Distance from Extreme Compression Fiber to Compression Reinforcement,	d's	=	0 mm
	Specified Compressive Strength of Concrete at 28 Days,	f'c	=	28 MPa
	Width of Compression Face of the Member,	b	=	2,000 mm
	Web Width,	bw	=	2,000 mm
DGCS 12.4.3.3	**Limits of Reinforcement,**	Mcr	=	624.70 KN-m
	1.33 Times the Factored Moment Required,	1.33*Ma	=	3,784.05 KN-m
DGCS 12.4.3.3-1	Cracking Moment, $Mcr = \gamma_3\left[(\gamma_1 fr+\gamma_2 fcpe)Sc - Mdnc\left(\frac{Sc}{Snc}-1\right)\right]$	Mcr	=	624.70 KN-m
	Modulus of Rupture,	fr	=	3.33 MPa
	Compressive Stress in Concrete due to Effective Prestress Forces,	fcpe	=	0 MPa
	Total Unfactored Dead Load Moment Acting on the Non-Composite Section,	Mdnc	=	899.58 KN-m
	Section Modulus for the Extreme Fiber of the Composite Section,	Sc	=	156,160,665 mm³
	Section Modulus for the Extreme Fiber of the Non-Composite Section,	Snc	=	66,666,667 mm³
	Flexural Cracking Variability Factor,	γ_1	=	1.60
	Prestress Variability Factor,	γ_2	=	1.00
	Ratio of Minimum Yield Strength to Ultimate Tensile Strength of the Reinforcement,	γ_3	=	0.75
	Capacity/Demand Ratio for Flexure,			
	Factored Flexural Resistance, $Mr = \phi Mn$	Mr	=	3,238.27 KN-m
	Actual Moment Required,	Ma	=	2,845.15 KN-m
	Capacity/Demand Ratio,	Mr/Ma	=	1.13817 **OK!**

Capacity Design Summary for Flexure,

Location	Number of Reinf. Bars	Reinf. Bar Diameter	Tension Side Location	Area of Reinf. Bars	c	a	Check if Steel Yields	Factored Flexural Resistance, Mr
	pc/s.	mm		mm²	mm	mm	0.6 > c/d	KN-m
0.000	6	28	Bottom	3694.51	38.351	32.599	OK!	1824.33
1.835	6	28	Bottom	3694.51	38.351	32.599	OK!	1824.33
3.670	8	28	Bottom	4926.02	51.135	43.465	OK!	2421.20
5.505	10	28	Bottom	6157.52	63.919	54.331	OK!	3012.45
7.340	12	28	Bottom	7389.03	76.703	65.197	OK!	3598.08
9.175	12	28	Bottom	7389.03	76.703	65.197	OK!	3598.08
11.010	12	28	Bottom	7389.03	76.703	65.197	OK!	3598.08
12.845	10	28	Bottom	6157.52	63.919	54.331	OK!	3012.45
14.680	8	28	Bottom	4926.02	51.135	43.465	OK!	2421.20
16.515	6	28	Bottom	3694.51	38.351	32.599	OK!	1824.33
18.350	6	28	Bottom	3694.51	38.351	32.599	OK!	1824.33

Capacity/Demand Ratio Design Summary for Flexure,

Location	Moment					Capacity/Demand Ratio	Remarks
	Mcr	1.33*Ma	Ma	Govern	Mr		
	KN-m	KN-m	KN-m	KN-m	KN-m		
0.000	624.70	0.00	0.00	0.00	1824.33	1.000	OK!
1.835	624.70	-1239.45	-931.91	931.91	1824.33	1.958	OK!
3.670	624.70	-2251.54	-1692.89	1692.89	2421.20	1.430	OK!
5.505	624.70	-3020.57	-2271.11	2271.11	3012.45	1.326	OK!
7.340	624.70	-3522.10	-2648.19	2648.19	3598.08	1.359	OK!
9.175	624.70	-3784.05	-2845.15	2845.15	3598.08	1.265	OK!
11.010	624.70	-3614.30	-2717.52	2717.52	3598.08	1.324	OK!
12.845	624.70	-3183.58	-2393.67	2393.67	3012.45	1.259	OK!
14.680	624.70	-2412.36	-1813.81	1813.81	2421.20	1.335	OK!
16.515	624.70	-1319.86	-992.37	992.37	1824.33	1.838	OK!
18.350	624.70	0.00	0.00	0.00	1824.33	1.000	OK!

Design for Shear :

	Number of Shear Leg Reinforcement,	nv	=	2 pc/s
	Area of Shear Reinforcement,	Av	=	402.12 mm²
DGCS 12.5.2.8	Effective Shear Depth,	dv	=	1,144.00 mm
DGCS 12.5.2.8-2	$de = \frac{Apsfpsdp + Asfyds}{Apsfps + Asfy}$	0.9de	=	1,072.80 mm
		0.72h	=	1,144.00 mm
DGCS 12.5.3.3-4	Net Longitudinal Tensile Strain in the Section at the Centroid of the Tension Reinforcement,	ε_s	=	0.00137
	$\varepsilon_s = \frac{\frac{\lvert Mu \rvert}{dv} + 0.5Nu + \lvert Vu - Vp \rvert - Apsfpo}{EsAs + EpAps}$			
	Factored Moment, not to be taken less than |Vu-Vp|dv,	|Mu|	=	580.99 KN-m
	Factored Axial Force,	Nu	=	0.00 KN
DGCS 12.5.3.3-5	Crack Spacing Parameter, $s_{xe} = s_x \frac{1.38}{a_g + 0.63}$	s_{xe}	=	783.87
	Maximum Aggregate Size,	a_g	=	25 mm
	Lesser of Either dv or the Maximum Distance of Longitudinal Crack Control Reinf.,	s_x	=	1144.00 mm
DGCS 12.5.3.3-2	Factor Indicating Ability of Diagonally Cracked Concrete, $\beta = \frac{4.8}{(1+750\varepsilon_s)}\frac{51}{(39+s_{xe})}$	β	=	0.14671

DGCS 12.5.3.2	**Nominal Shear Resistance,**		Vn	=	2605.65 KN
DGCS 12.5.3.2-1	The Nominal Shear Resistance shall be determined	$Vn = Vc + Vs$	Vn	=	2605.65 KN
DGCS 12.5.3.2-2	as the lesser of :	$Vn = 0.25f'cbvdv$	Vn	=	3203.20 KN
DGCS 12.5.3.2-3	Shear Resistance Provided of Concrete,	$Vc = 0.083\beta\sqrt{f'c}bvdv$	Vc	=	29.49 KN
DGCS 12.5.3.2-4	Shear Resistance Provided of Steel Reinforcement,	$Vs = \frac{Avfytdv}{s}$	Vs	=	2576.17 KN

Capacity/Demand Ratio for Flexure,

Factored Shear Resistance,	$Vr = \emptyset Vn$	Vr	=	2,605.65 KN-m	
Actual Shear Required,		Va	=	637.30 KN-m	
Capacity/Demand Ratio,		Vr/Va	=	4.08857	OK!

Capacity/Demand Ratio Design Summary for Shear,

Location	Number of Shear Legs	Spacing	Area of Shear Reinf.	Actual Shear Required, Va	Shear on Steel, Vs	Factored Shear, Vr	Capacity/Demand Ratio	Remarks
	pc/s.	mm	mm^2	KN	KN	KN		
0.000	2	50	402.12	637.30	2576.166	2605.652	4.08857	OK!
1.835	2	100	402.12	462.44	1288.083	1317.569	2.84914	OK!
3.670	2	100	402.12	349.54	1288.083	1317.569	3.76943	OK!
5.505	2	150	402.12	233.58	858.722	888.207	3.80254	OK!
7.340	2	150	402.12	89.13	858.722	888.207	9.96527	OK!
9.175	2	150	402.12	27.47	858.722	888.207	32.33257	OK!
11.010	2	150	402.12	124.99	858.722	888.207	7.10602	OK!
12.845	2	150	402.12	251.09	858.722	888.207	3.53734	OK!
14.680	2	100	402.12	340.06	1288.083	1317.569	3.87457	OK!
16.515	2	100	402.12	429.02	1288.083	1317.569	3.07114	OK!
18.350	2	50	402.12	517.98	2576.166	2605.652	5.03044	OK!

Transverse Reinforcement Specifications,

DGCS 12.5.2.4	Minimum Transverse Reinforcement,	$Av \geq 0.083\sqrt{f'c}\frac{bvs}{fy}$			
DGCS 12.5.2.4-1					
DGCS 12.5.2.8-1	Shear Stress on Concrete,	$v_u = \frac{\lvert V_u - \emptyset V_p \rvert}{\emptyset bvdv}$	v_u	=	1.39270 MPa
DGCS 12.5.2.6	Maximum Spacing of Transverse Reinforcement,		0.125f'c	=	3.50 MPa
			v_u	<	0.125f'c
DGCS 12.5.2.6-1		$smax = 0.8dv \leq 600mm$	smax	=	600.00 mm
DGCS 12.5.2.6-2		$smax = 0.4dv \leq 300mm$	smax	=	300.00 mm
	Governing Maximum Spacing of Transverse Reinforcement,		smax	=	600.00 mm

Capacity/Demand Ratio Design Summary for Transverse Reinforcement,

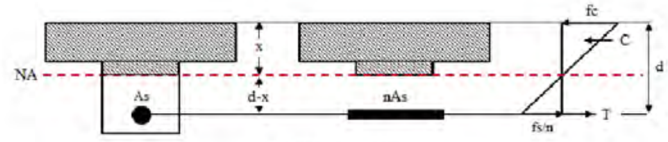
Location	Number of Shear Legs	Spacing	Area of Shear Reinf.	Minimum Shear Reinf.	Capacity/Demand Ratio	Maximum Spacing	Capacity/Demand Ratio	Remarks
	pc/s.	mm	mm^2	mm^2		mm		
0.000	2	50	402.12	31.37	12.818	600.000	12.00000	OK!
1.835	2	100	402.12	62.74	6.409	600.000	6.00000	OK!
3.670	2	100	402.12	62.74	6.409	600.000	6.00000	OK!
5.505	2	150	402.12	94.11	4.273	600.000	4.00000	OK!
7.340	2	150	402.12	94.11	4.273	600.000	4.00000	OK!
9.175	2	150	402.12	94.11	4.273	600.000	4.00000	OK!
11.010	2	150	402.12	94.11	4.273	600.000	4.00000	OK!
12.845	2	150	402.12	94.11	4.273	600.000	4.00000	OK!
14.680	2	100	402.12	62.74	6.409	600.000	6.00000	OK!
16.515	2	100	402.12	62.74	6.409	600.000	6.00000	OK!
18.350	2	50	402.12	31.37	12.818	600.000	12.00000	OK!

DGCS 12.5.4
DGCS 12.5.3.4-1

Longitudinal Reinforcement, $Apsfps + Asfy \geq \frac{|Mu|}{dv\emptyset f} + 0.5\frac{Nu}{\emptyset c} + \left(\left|\frac{Va}{\emptyset v} - Vp\right| - 0.5Vs\right)cot\theta$

Location	Asfy	Mu	Nu	Va	Vs	Limit for Longitudinal Reinf.	Capacity/Demand Ratio	Remarks
	N	KN-m	KN	KN	KN			
0.000	1551.70	0.00	0.00	637.30	2576.166	-650.782	2.38436	OK!
1.835	1551.70	-931.91	0.00	462.44	1288.083	723.526	2.14463	OK!
3.670	2068.93	-1692.89	0.00	349.54	1288.083	1349.719	1.53286	OK!
5.505	2586.16	-2271.11	0.00	233.58	858.722	2010.039	1.28662	OK!
7.340	3103.39	-2648.19	0.00	89.13	858.722	2231.828	1.39051	OK!
9.175	3103.39	-2845.15	0.00	27.47	858.722	2361.462	1.31418	OK!
11.010	3103.39	-2717.52	0.00	124.99	858.722	2335.029	1.32906	OK!
12.845	2586.16	-2393.67	0.00	251.09	858.722	2146.589	1.20478	OK!
14.680	2068.93	-1813.81	0.00	340.06	1288.083	1457.674	1.41933	OK!
16.515	1551.70	-992.37	0.00	429.02	1288.083	748.818	2.07219	OK!
18.350	1551.70	0.00	0.00	517.98	2576.166	-770.106	2.01491	OK!

DGCS 12.2.2 **FATIGUE LIMIT STATE**



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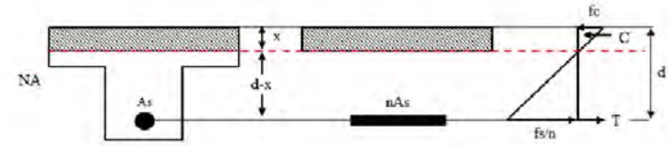


RECTANGULAR SECTION

Fatigue Investigations :

	Unfactored Moment for DC,		M_{DC}	=	899.58 KN-m
	Unfactored Moment for DW,		M_{DW}	=	291.69 KN-m
	Fatigue I Load Combination,	$Fatigue\ I = 1.50(LL + IM)$	Mfatigue	=	572.84 KN-m
	Total Moment for Fatigue Investigation,		M	=	1764.11 KN-m
	Stress at Bottom Fiber,		ft	=	11.30 MPa
	Limit Check for Crack Section Analysis,		$0.095\sqrt{f'c}$	=	0.50 MPa
			ft	>	$0.095\sqrt{f'c}$
	∴ The Section Properties for Fatigue Investigations shall be based on Cracked Sections				
	Transformed Area of Steel,	$n_{As} = As \cdot n$	n_{As}	=	55239.90 mm²
	Effective Depth,		d	=	1192.00 mm
	Effective Flange Width,		Weff	=	2000.00 mm
	Flange Thickness or Slab Thickness,		ts	=	200.00 mm
	Girder Web Width,		bw	=	400.00 mm
	Distance from Extreme Compression Fiber to Neutral Axis of the Cracked Section,				
	Flanged Section,		a	=	200.00
		$\frac{bw}{2}x^2 + (Weffts - bwts + nAs)x + (\frac{-Weffts^2}{2} + \frac{bwts^2}{2} - nAsd) = 0$	b	=	375239.90
			c	=	-97845955.98
			x+	=	232.05 mm
			x-	=	-2108.25 mm
	Rectangular Section,	$\frac{Weff}{2}x^2 + nAsx - nAsd = 0$	a	=	1000.00
			b	=	55239.90
			c	=	-65845955.98
			x+	=	230.47 mm
			x-	=	-285.71 mm
	Governing Distance of the Neutral Axis of the Cracked Section,		x	=	232.05 mm
	Moment of Inertia of Cracked Section,				
	Flanged Section,	$I_{cr} = \frac{Weffts^3}{12} + Weffts\left[\left(x - \frac{ts}{2}\right)^2\right] + \frac{bw(x - ts)^3}{3} + nAs(d - x)^2$	Icr	=	5.92176E+10 mm⁴
	Rectangular Section,	$I_{cr} = \frac{Weffts^3}{3} + nAs(d - x)^2$	Icr	=	5.92327E+10 mm⁴
	Governing Moment of Inertia of Cracked Section,		Icr	=	5.92176E+10 mm⁴
	Stress in Tension Steel Reinforcement,	$\frac{fs}{n} = \frac{Mfatigue(d - x)}{Icr}$	fs	=	69.42 MPa
DGCS 12.2.2	For Fatigue Considerations, Concrete Members shall satisfy :	$\gamma(\Delta f) \leq (\Delta F)_{TH}$	69.42	<	118.36 **OK!**
DGCS 10.3-1	Load Factor for the Fatigue I Load Combination,		γ	=	1.50
DGCS 12.2.2.1-1	Constant-Amplitude Fatigue Threshold,	$(\Delta F)_{TH} = 166 - 0.33fmin$	$(\Delta F)_{TH}$	=	118.36 MPa
	Minimum Stress Resulting from Fatigue I Load Combination,		fmin	=	144.37 MPa

Capacity/Demand Ratio Design Summary for Fatigue Limit State,

Location	As	Flanged Section			Rectangular Section			Distance of N.A. of Cracked Section, x
	mm²	a	b	c	a	b	c	
0.000	3694.51	200	347619.95	-64922978	1000.00	27619.95	-32922978	168.16
1.835	3694.51	200	347619.95	-64922978	1000.00	27619.95	-32922978	168.16
3.670	4926.02	200	356826.60	-75897304	1000.00	36826.60	-43897304	191.91
5.505	6157.52	200	366033.25	-86871630	1000.00	46033.25	-54871630	212.63
7.340	7389.03	200	375239.90	-97845956	1000.00	55239.90	-65845956	232.05
9.175	7389.03	200	375239.90	-97845956	1000.00	55239.90	-65845956	232.05
11.010	7389.03	200	375239.90	-97845956	1000.00	55239.90	-65845956	232.05
12.845	6157.52	200	366033.25	-86871630	1000.00	46033.25	-54871630	212.63
14.680	4926.02	200	356826.60	-75897304	1000.00	36826.60	-43897304	191.91
16.515	3694.51	200	347619.95	-64922978	1000.00	27619.95	-32922978	168.16
18.350	3694.51	200	347619.95	-64922978	1000.00	27619.95	-32922978	168.16

Location	As	Moment of Inertia C.S.	M_{DC-DW}	Mfatigue	$\gamma(\Delta f)$	$(\Delta F)_{TH}$	Capacity/Demand Ratio	Remarks
	mm^2	mm^4	KN-m	KN-m	Mpa	Mpa		
0.000	3694.51	32122693059	0.00	0.00	0.00	0.00	1.00000	OK!
1.835	3694.51	32122693059	421.24	168.37	40.12	100.37	2.50181	OK!
3.670	4926.02	41545185982	753.08	324.61	58.42	135.53	2.31992	OK!
5.505	6157.52	50561315203	995.50	451.35	65.36	144.16	2.20564	OK!
7.340	7389.03	59216389127	1140.18	572.84	69.42	138.18	1.99040	OK!
9.175	7389.03	59216389127	1191.27	538.22	65.23	144.37	2.21333	OK!
11.010	7389.03	59216389127	1140.18	465.51	56.42	138.18	2.44933	OK!
12.845	6157.52	50561315203	995.50	392.79	56.88	144.16	2.53446	OK!
14.680	4926.02	41545185982	753.08	320.07	57.60	135.53	2.35286	OK!
16.515	3694.51	32122693059	421.24	237.12	56.50	100.37	1.77650	OK!
18.350	3694.51	32122693059	0.00	0.00	0.00	0.00	1.00000	OK!

Reference	Description	Symbol		Value	Check
DGCS 12.2.1	**SERVICE LIMIT STATE**				
DGCS 12.4.3.4	The Spacing, s, of Mild Steel Reinforcement in the Layer Closest to the Tension Face shall Satisfy the following :				
DGCS 12.4.3.4-1	$s \leq \frac{123000\gamma_e}{\beta_s f_{ss}} - 2d_c$	50.00	<	172.94	**OK!**
	where :				
DGCS 12.4.3.4-2	$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$	βs	=	1.12943	
	Spacing of Mild Steel Reinforcement in the Layer Closes to the Tension Face,	s	=	50.00 mm	
	Exposure Factor,	γe	=	1.00	
	Thickness of Concrete Cover Measured from Extreme Tension Fiber to center of Flexural Reinforcement,	dc	=	108.00 mm	
	Tensile Stress in Steel Reinforcement at the Service Limit,	fss	=	280.00 MPa	
	Overall Thickness or Depth of the Component,	h	=	1300.00 mm	
DGCS 12.7.3	**SPACING OF REINFORCEMENT**				
DGCS 12.7.3.1	Minimum Spacing of Reinforcing Bars,	s	>	smin	**OK!**
	For precast-in-place concrete, the clear distance between parallel bars in a layer shall no be less than :				
	1.5 Times the Nominal Diameter of the Bars,	1.5db	=	42.00 mm	
	1.5 Times the Maximum Size of the Coarse Aggregates,	1.5dc	=	37.50 mm	
	38 mm,		=	38.00 mm	
	Governing Minimum Spacing of Reinforcing Bars,	smin	=	37.50 mm	
	Actual Spacing of Reinforcing Bars,	s	=	590.67 mm	

Location	n	Bar Diameter	Layer	Pcs. Per Layer	Actual Spacing	smin	Capacity/Demand Ratio	Remarks
	pc/s	mm		pc/s	mm	mm		
0.000	6	28	1	6	354.40	37.50	9.45067	OK!
1.835	6	28	1	6	354.40	37.50	9.45067	OK!
3.670	8	28	2	4	590.67	37.50	15.75111	OK!
5.505	10	28	2	5	443.00	37.50	11.81333	OK!
7.340	12	28	2	6	354.40	37.50	9.45067	OK!
9.175	12	28	2	6	354.40	37.50	9.45067	OK!
11.010	12	28	2	6	354.40	37.50	9.45067	OK!
12.845	10	28	2	5	443.00	37.50	11.81333	OK!
14.680	8	28	2	4	590.67	37.50	15.75111	OK!
16.515	6	28	1	6	354.40	37.50	9.45067	OK!
18.350	6	28	1	6	354.40	37.50	9.45067	OK!

Reference	Description	Formula	Symbol		Value	Check
DGCS 12.4.3.6	**DEFORMATIONS**					
	Deflection and Camber,					
DGCS 12.4.3.6-1	$Ie = \left(\frac{Mcr}{Ma}\right)^3 Ig + \left[1 - \left(\frac{Mcr}{Ma}\right)^3\right] Icr \leq Ig$		Ie	<	Ig	**OK!**
	where :					
DGCS 12.4.3.6-2	Cracking Moment,	$Mcr = fr\frac{Ig}{yt}$	Mcr	=	520.58 KN-m	
	Modulus of Rupture of Concrete,		fr	=	3.33 MPa	
	Distance from the Neutral Axis to the Extreme Tension Fiber,		yt	=	859.52 mm	
	Maximum Moment in a Component at the Stage for which Deformation is Computed,		Ma	=	899.58 KN-m	
	Effective Moment of Inertia,		Ie	=	7.37537E+10 mm^4	
	Gross Moment of Inertia,		Ig	=	1.34224E+11 mm^4	
	Moment of Inertia of Cracked Section,		Icr	=	5.92176E+10 mm^4	
	Instantaneous Deflection due to Dead Load,	$\Delta_{DI} = \frac{5wL^4}{384EcIe}$	Δ_{Di}	=	7.18 mm	
	Long Time Deflection due to Dead Load,	$\Delta_{DL} = \Delta_i\left[3.0 - 1.2\left(\frac{A's}{As}\right)\right] > 1.6$	Δ_{DL}	=	12.93 mm	
	Actual Deflection due to Dead Load,	$\Delta_{Da} = \Delta_{Di} + \Delta_{DL}$	Δ_{Da}	=	20.11 mm	
	Instantaneous Deflection due to Live Load,		Δ_{Li}	=	1.22 mm	
	Long Time Deflection due to Live Load,	$\Delta_{LL} = \Delta_i\left[3.0 - 1.2\left(\frac{A's}{As}\right)\right] > 1.6$	Δ_{LL}	=	2.19 mm	
	Actual Deflection due to Live Load,	$\Delta_{La} = \Delta_{Li} + \Delta_{LL}$	Δ_{La}	=	3.40 mm	
DGCS 10.1.2.6	Allowable Deflection for Vehicular Load,	$\Delta_{allowable} = \frac{L}{800}$	$\Delta_{allowable}$	=	22.94 mm	
			$\Delta_{allowable}$	>	Δ_{La}	**OK!**

(2) Design of the Floor Slab

1) Inside

	Design Base Width,	b	=	1000.00 mm
DGCS 14.4.1.1	Deck Slab Thickness,	h	=	200.00 mm
DGCS 12.9.2	Concrete Cover,	cc	=	40.00 mm
	Calculated Design Effective Depth,	deff	=	152.00 mm
	Girder Spacing,	Sg	=	2,000 mm
	Girder Width,	bw	=	400.000 mm
DGCS 14.4.2.2	Effective Length,	Leff	=	1200.00 mm
	Diameter of Main Bars,	db.1	=	16 mm
	Diameter of Distribution Bars,	db.2	=	16 mm
	Diameter of Temperature Bars,	db.3	=	16 mm
	Material Properties for Concrete :			
DGCS 12.1.1.1	Compressive Strength,	fc'	=	28.00 MPa
DGCS 12.1.1.4	Modulus of Elasticity, $Ec = 0.043K_1\rho c^{1.5}\sqrt{fc'}$ =	Ec	=	26,752.50 MPa
	Material Properties for Reinfocing Steel Bars :			
DGCS 12.1.2	Rebar Yield Strength (diameter ≥ 20mm),	fy	=	420.00 MPa
DGCS 12.1.2	Rebar Yield Strength (diameter ≤ 16mm),	fy	=	280.00 MPa
DGCS 12.1.2.1	Modulus of Elasticity,	Es	=	200,000.00 MPa
	Design Dead Load Moments,			
DGCS 10.3	Load Factors and Load Combinations,			
DGCS 10.3-1	Total Factored Force Effect, $Q = \sum \eta_i \gamma_i Q_i$			
	where :			
	For Loads for which a Maximum Value of γi is Appropriate :			
DGCS 10.3-2	$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$			
	For Loads for which a Minimum Value of γi is Appropriate :			
DGCS 10.3-3	$\eta_i = \dfrac{1}{\eta_D \eta_R \eta_I} \leq 1.0$			
	For Strength Limit State,			
	Factor Relating to Ductility,	η_D	=	1.05
	Factor Relating to Redundancy,	η_R	=	1.05
	Factor Relating to Operational Importance,	η_I	=	1.00
	Load Modifier for Maximum Value of γi,	ηi	=	1.10
	Load Modifier for Minimum Value of γi,	ηi	=	0.91
	For All Other Limit States,			
	Factor Relating to Ductility,	η_D	=	1.00
	Factor Relating to Redundancy,	η_R	=	1.00
	Factor Relating to Operational Importance,	η_I	=	1.00
	Load Modifier for Maximum Value of γi,	ηi	=	1.00
	Load Modifier for Minimum Value of γi,	ηi	=	1.00
	Deck Slab Dimensions and Other Properties,			
	Deck Slab Thickness,	h	=	0.20 m
	Unit Weight for Deck Slab Concrete,	γc	=	24.00 KN/m³
	Future Wearing Surface Thickness,	tws	=	0.05 m
	Unit Weight for FWS Concrete,	γws	=	22.00 KN/m³
	Weight of Deck Slab,	w	=	4.80 KN/m²
	Unfactored Deck Slab Positive or Negative Moment, $Mds = \dfrac{w * Leff^2}{10}$	Mds	=	0.69 KN-m/m
	Weight of FWS,	wws	=	1.10 KN/m²
	Unfactored Deck Slab Positive or Negative Moment, $Mws = \dfrac{wws * Leff^2}{10}$	Mws	=	0.16 KN-m/m
	Design of Deck Slab,			
	Design Factored Moment for IM = 33%			
	Dynamic Overload Factor,		=	1.50
	Strength I Limit State,			

Factor	Force	Moment Positive	Moment Negative	Unit
1.38	DC	0.95	0.95	KN-m/m
1.65	DW	0.26	0.26	KN-m/m
1.93	LL	36.88	53.42	KN-m/m
TOTAL		**38.10**	**54.63**	**KN-m/m**

Service I Limit State,

Factor	Force	Moment Positive	Moment Negative	Unit
1.00	DC	0.69	0.69	KN-m/m
1.00	DW	0.16	0.16	KN-m/m
1.00	LL	19.12	27.69	KN-m/m
TOTAL		**19.97**	**28.54**	**KN-m/m**

Design Factored Moment for IM = 75%

Dynamic Overload Factor, = 1.50

Strength I Limit State,

Factor	Force	Moment Positive	Moment Negative	Unit
1.38	DC	0.95	0.95	KN-m/m
1.65	DW	0.26	0.26	KN-m/m
1.93	LL	83.82	121.41	KN-m/m
TOTAL		**85.04**	**122.62**	**KN-m/m**

Service I Limit State,

Factor	Force	Moment Positive	Moment Negative	Unit
1.00	DC	0.69	0.69	KN-m/m
1.00	DW	0.16	0.16	KN-m/m
1.00	LL	43.45	62.93	KN-m/m
TOTAL		**44.29**	**63.78**	**KN-m/m**

DGCS 12.4.3.2 **Flexural Resistance,**

DGCS 12.4.3.3 **Limits of Reinforcement,** Mcr = 26.67 KN-m

1.33 Times the Factored Moment Required, 1.33*Ma = 84.82 KN-m

DGCS 12.4.3.3-1 Cracking Moment, $Mcr = \gamma_3 \gamma_1 fr Sc$ Mcr = 26.67 KN-m

Modulus of Rupture, fr = 3.33 MPa

Section Modulus for the Extreme Fiber of the Composite Section, $Sc = \frac{bh^2}{6}$ Sc = 6,666,667 mm^3

Flexural Cracking Variability Factor, γ_1 = 1.60

Ratio of Minimum Yield Strength to Ultimate Tensile Strength of the Reinforcement, γ_3 = 0.75

Reinforcement Parameters,

For Impact Factor IM = 33%

Positive Moment : Bottom Bars		
Rebar mm	**Spacing mm**	**Remarks**
16	150.00	ADEQUATE

Negative Moment : Top Bars		
Rebar mm	**Spacing mm**	**Remarks**
16	150.00	ADEQUATE

For Impact Factor IM = 75%

Positive Moment : Bottom Bars		
Rebar mm	**Spacing mm**	**Remarks**
16	100.00	ADEQUATE

Negative Moment : Top Bars		
Rebar mm	**Spacing mm**	**Remarks**
16	80.00	ADEQUATE

Design Calculation of Main Bars,

DGCS 12.2.3.1-1 **Strength Reduction Factor,**

	For Impact Factor IM=33%		**For Impact Factor IM=75%**	
	Pos. Moment	**Neg. Moment**	**Pos. Moment**	**Neg. Moment**
Flexure in Tension Controlled, Φf	0.90	0.90	0.90	0.90
Shear and Torsion, Φv	0.90	0.90	0.90	0.90

Flexural Reinforcement,

	For Impact Factor IM=33%		**For Impact Factor IM=75%**		**Units**
	Pos. Moment	**Neg. Moment**	**Pos. Moment**	**Neg. Moment**	
Total Flexural Bar Area, As	1340.41	1340.41	2010.62	2513.27	**mm^2**

DGCS 12.4.3 **Flexural Capacity Verification,**

	For Impact Factor IM=33%		**For Impact Factor IM=75%**		**Units**
	Pos. Moment	**Neg. Moment**	**Pos. Moment**	**Neg. Moment**	
Factored Applied	38.10	54.63	85.04	122.62	**KN-m/m**
Depth of Equivalent Sress Block	23.65	23.65	35.48	44.35	**mm**
Factored Flexural Resistance	71.02	71.02	102.04	123.34	**KN-m/m**
Mcrz ≤ Mrz	**OK!**	**OK!**	**OK!**	**OK!**	

DGCS 12.4.3.3 **Minimum Reinforcement Verification,**

	For Impact Factor IM=33%		**For Impact Factor IM=75%**		**Units**
	Pos. Moment	**Neg. Moment**	**Pos. Moment**	**Neg. Moment**	
Cracking Moment	26.67	26.67	26.67	26.67	**KN-m/m**
Mcrz ≤ Mrz	**OK!**	**OK!**	**OK!**	**OK!**	

Reference	Description	Formula	Symbol		Value
DGCS 14.4.3.1	**Distribution of Reinforcement,**				
	Percent Distribution,	$\frac{3840}{\sqrt{Leff}} \leq 67\%$	%distribution	=	67.00 %
	Area of Distribution Bars,	$As.dr = \%distribution * As.tension$	As.dr	=	898.08 mm²
	Spacing of Bottom Distribution Rebar,	$s = \frac{Ab * 1000}{As}$	s	=	220.00 mm
DGCS 12.7.8	**Shrinkage and Temperature Reinforcement,**				
	Least Width of Component Section,		b	=	1000.00 mm
	Least Thickness of Component Section,		h	=	200.00 mm
	Specified Yield Strength of Bar,		fy	=	420.00 MPa
	Reinforcement Area in Each Direction and Each Face, (mm2/mm)		As	=	0.23 mm²
DGCS 12.7.8-1		$As \geq \frac{0.75bh}{2(b+h)fy}$		=	0.15 mm²
DGCS 12.7.8-2		$0.233 \leq As \leq 1.27$		=	0.23 mm²
	Governing Area of Temperature Reinforcement, As			=	233.00 mm²
	Minimum Spacing for Temperature Rebar Verification,				
	Required Spacing Provided by Governing Temperature Reinforcement,			=	862.93 mm
	3.0 x The Component Thickness, or 450mm,			=	450.00 mm
	Required Minimum Spacing,			=	450.00 mm

Verification of Reinforcement,

Spacing Summary,

For Impact Factor IM = 33% (Adopt for Slabs Beyond End Regions)

Main Reinforcement,

Design Moment	Rebar Location	Rebar Size	Spacing	Remark
Positive Moment	Bottom Bars	16	150.00	Adopt Spacing
Negative Moment	Top Bars	16	150.00	Adopt Spacing

For Impact Factor IM = 75% (Adopt for First Three (3) Meters from every Start of Span)

Main Reinforcement,

Design Moment	Rebar Location	Rebar Size	Spacing	Remark
Positive Moment	Bottom Bars	16	100.00	
Negative Moment	Top Bars	16	80.00	Adopt Spacing

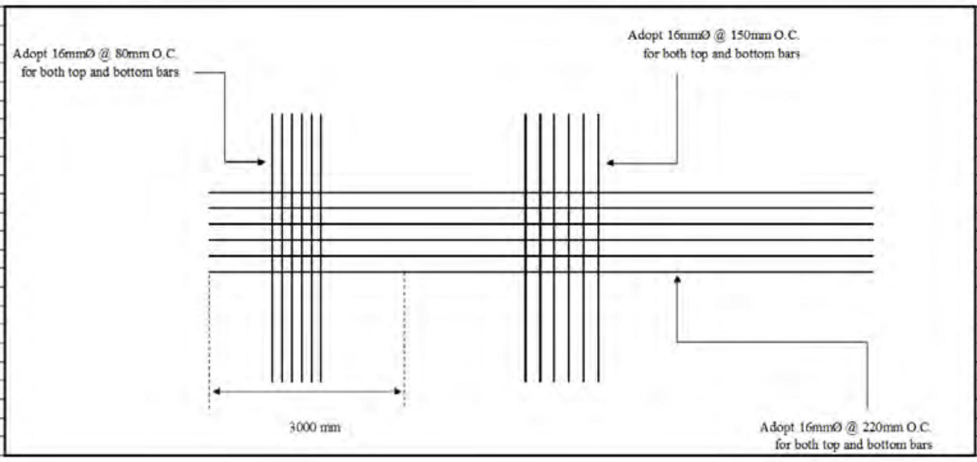
Distribution Reinforcement,

Adopt Distribution Reinforcement, Adopt 16mmØ @ 220.00 mm

Temperature and Shrinkage Reinforcement,

Adopt Distribution Reinforcement, Adopt 16mmØ @ 450.00 mm

Slab Reinforcement Layout,



2) Overhang

	Design Base Width,	b	=	1000.00 mm
DGCS 14.4.1.1	Section Overall Thickness,	h	=	250.00 mm
DGCS 12.9.2	Concrete Cover,	cc	=	40.00 mm
	Calculated Design Effective Depth,	deff	=	202.00 mm
	Width of Sidewalk,	bsw	=	1200.00 mm
	Thickness of Sidewalk,	tsw	=	250.00 mm
	Area of Sidewalk,	Asw	=	300000.00 mm²
	Diameter of Main Bars,	db.1	=	16 mm
	Diameter of Distribution Bars,	db.2	=	16 mm
	Diameter of Temperature Bars,	db.3	=	16 mm
	Material Properties for Concrete :			
DGCS 10.6-1	Density,	ρc	=	2,400.00 kg/m³
	Unit Weight,	γc	=	24.00 KN/m³
DGCS 12.1.1.1	Compressive Strength,	fc'	=	28.00 MPa
DGCS 12.1.1.4	Modulus of Elasticity, $Ec = 0.043K_1\rho c^{1.5}\sqrt{fc'}$ =	Ec	=	26,752.50 MPa
	Material Properties for Reinfocing Steel Bars :			
DGCS 12.1.2	Rebar Yield Strength (diameter ≥ 20mm),	fy	=	420.00 MPa
DGCS 12.1.2	Rebar Yield Strength (diameter < 16mm),	fy	=	280.00 MPa
DGCS 12.1.2.1	Modulus of Elasticity,	Es	=	200,000.00 MPa
	Dead Load Analysis,			
	Weight of Sidewalk,	Wsw	=	7.20 KN/m
	Weight of Post,	Wpost	=	0.74 KN/m
	Weight of Railings,	Wrail	=	1.92 KN/m
	Undactored Dead Load,			

Member	Weight KN/m	Lever Arm m	Moment KN-m
Sidewalk	7.20	0.60	4.32
Post	0.74	1.08	0.79
Railings	1.92	1.10	2.11
TOTAL DC	**9.86**		**7.22**

DGCS 18	**Live Load Analysis,**			
	Live Load Intensity,	LL	=	15.00 KN
	Live Load Moment Arm,	LLarm	=	700.00 mm
	Dynamic Load Allowance (For All Other Limit State),	IM	=	1.33
	Distance of Design Section to Face of Railing,	d	=	1000.00 mm
	Design Unfactored Shear with Impact Factor,	V	=	19.95 KN
	Design Unfactored Bending Moment with Impact Factor, Negative Moment,	M	=	13.97 KN-m
	Factored Shear and Bending Moments,			
DGCS 10.3	Load Factors and Load Combinations,			
DGCS 10.3-1	Total Factored Force Effect, $Q = \sum \eta_i \gamma_i Q_i$			
	where :			
	For Loads for which a Maximum Value of γi is Appropriate :			
DGCS 10.3-2	$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$			
	For Loads for which a Minimum Value of γi is Appropriate :			
DGCS 10.3-3	$\eta_i = \dfrac{1}{\eta_D \eta_R \eta_I} \leq 1.0$			
	For Strength Limit State,			
	Factor Relating to Ductility,	η_D	=	1.05
	Factor Relating to Redundancy,	η_R	=	1.05
	Factor Relating to Operational Importance,	η_I	=	1.00
	Load Modifier for Maximum Value of γi,	ηi	=	1.10
	Load Modifier for Minimum Value of γi,	ηi	=	0.91
	For All Other Limit States,			
	Factor Relating to Ductility,	η_D	=	1.00
	Factor Relating to Redundancy,	η_R	=	1.00
	Factor Relating to Operational Importance,	η_I	=	1.00
	Load Modifier for Maximum Value of γi,	ηi	=	1.00
	Load Modifier for Minimum Value of γi,	ηi	=	1.00
	Factored Shear and Bending Moments,			

Strength Event I Limit State			
Factor	Force	Shear KN	Moment KN-m
1.38	DC	13.58	9.95
1.93	LL	28.94	26.94
TOTAL		**42.52**	**36.90**

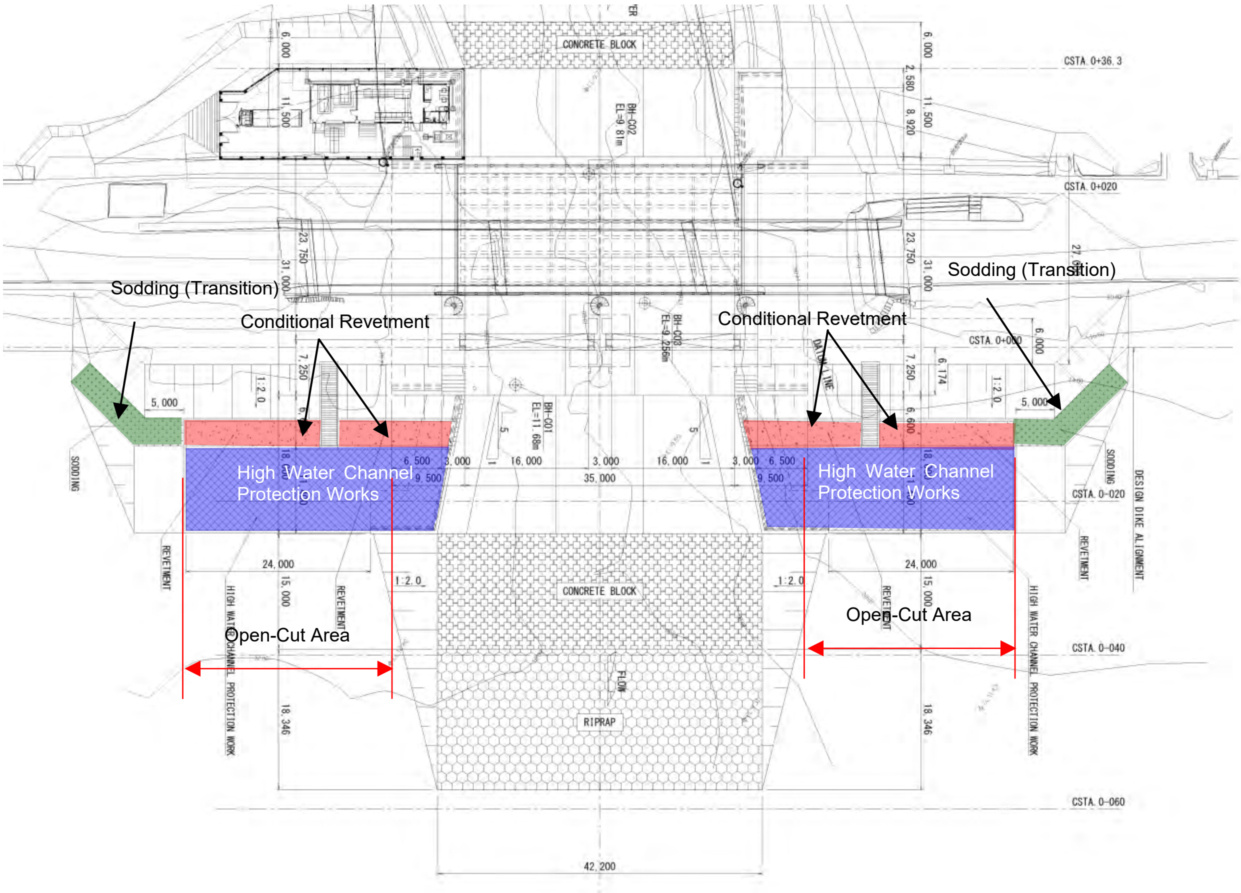
	Design of Main Reinforcement,				
	Maximum Factored Moment,		Mu	=	36.90 KN-m
DGCS 12.4.3.3	Limits of Reinforcement,		Mcr	=	41.67 KN-m
	1.33 Times the Factored Moment Required,		1.33*Mu	=	49.07 KN-m
DGCS 12.4.3.3-1	Cracking Moment,	$Mcr = \gamma_3 \gamma_1 frSc$	Mcr	=	41.67 KN-m
	Modulus of Rupture,		fr	=	3.33 MPa
	Section Modulus for the Extreme Fiber of the Composite Section,	$Sc = \frac{bh^2}{6}$	Sc	=	10,416,667 mm^3
	Flexural Cracking Variability Factor,		γ_1	=	1.60
	Ratio of Minimum Yield Strength to Ultimate Tensile Strength of the Reinforcement,		γ_3	=	0.75
	Verification of Governing Factored Flexural Resistance,				
	Governing Factored Flexural Resistance for Flexural Design,		Mu	=	41.67 KN-m
DGCS 12.2.3.1	Flexural Resistance Factor,		Φf	=	0.90
	Area of Flexural Reinforcement,		m	=	17.65
			x	=	0.003
			r	=	0.003
			As	=	559.41 mm^2
	Required Bar Spacing,	$s = Ab * b/As$	s	=	300.00 mm
DGCS 12.7.3.2	Maximum Spacing of Reinforcing Bars,				
	1.5 Times the Thickness of the Member,		smax1	=	375.00 mm
	450mm,		smax2	=	450.00 mm
DGCS 12.7.3.1	Check for Minimum Spacing,				
	1.5 Times the Nominal Diameter of the Bars,		smin1	=	24.00 mm
	1.5 Times the Maximum Size of the Coarse Aggregates,		smin2	=	37.50 mm
	38mm,		smin3	=	38.00 mm
	∴ Use 16 mmø Tension Reinforcement Bar spaced @ 300mm O.C.				
	Shear Verification,				
	Maximum Factored Shear,		Vu	=	42.52 KN
DGCS 12.5.3.3.1	Simplified Procedure for Non-Prestressed Section,				
	Verification of Overall Depth,		h	=	250.00 mm
					Simplified Procedure
	Calculation for Shear Depth,				
	Governing Effective Shear Depth,	$dv = de - 0.50a$	dv	=	197.06 mm
	Shear Depth,		de	=	202.00 mm
	Shall be Less Than the Greater of,				
	0.90*de	9.871945702		=	181.80 mm
	0.72*h			=	180.00 mm
	If Overall Depth is Less than 400 mm,				
	Beta,		β	=	2.00
	Theta,		θ	=	45.00 degrees
DGCS 12.5.3.2	Nominal Shear Resistance Verification,				
DGCS 12.5.3.2-3	Nominal Concrete Shear Resistance,	$Vc = 0.083 * \beta * \sqrt{f'c} * bv * dv$	Vc	=	173.10 KN
	Nominal Reinf. Shear Resistance,	$Vs = \frac{Av * fy * dv * cot\theta}{s}$	Vs	=	55.47 KN
	Nominal Shear Resistance,				
DGCS 12.5.3.2-1		$Vn1 = Vc + Vs$	Vn1	=	228.57 KN
DGCS 12.5.3.2-2		$Vn2 = 0.25 * f'c * bv * dv$	Vn2	=	1379.45 KN
	Governing Nominal Shear Resistance,		Vn	=	228.57 KN
	Factored Shear Resistance Verification,				
	Shear Resistance Factor,		Φv	=	0.90
	Factored Shear Resistance,	$Vr = \phi v * Vn$	Vr	=	205.71 KN
	∴ Shear Reinforcement is Adequate				

7.4.2.5 Revetment and Earth Work, Etc.

(1) Extent of Revetment

1) Area to Be Installed.

The installation range of revetment works is determined in the basic design. The range is shown in **Figure7.4.178**.



Source: Study team

Figure7.4.178 The Extent of Conditional Revetment

2) Revetment Structure

The revetment structure shall be reinforced concrete (t = 200 mm) as determined in the basic design.

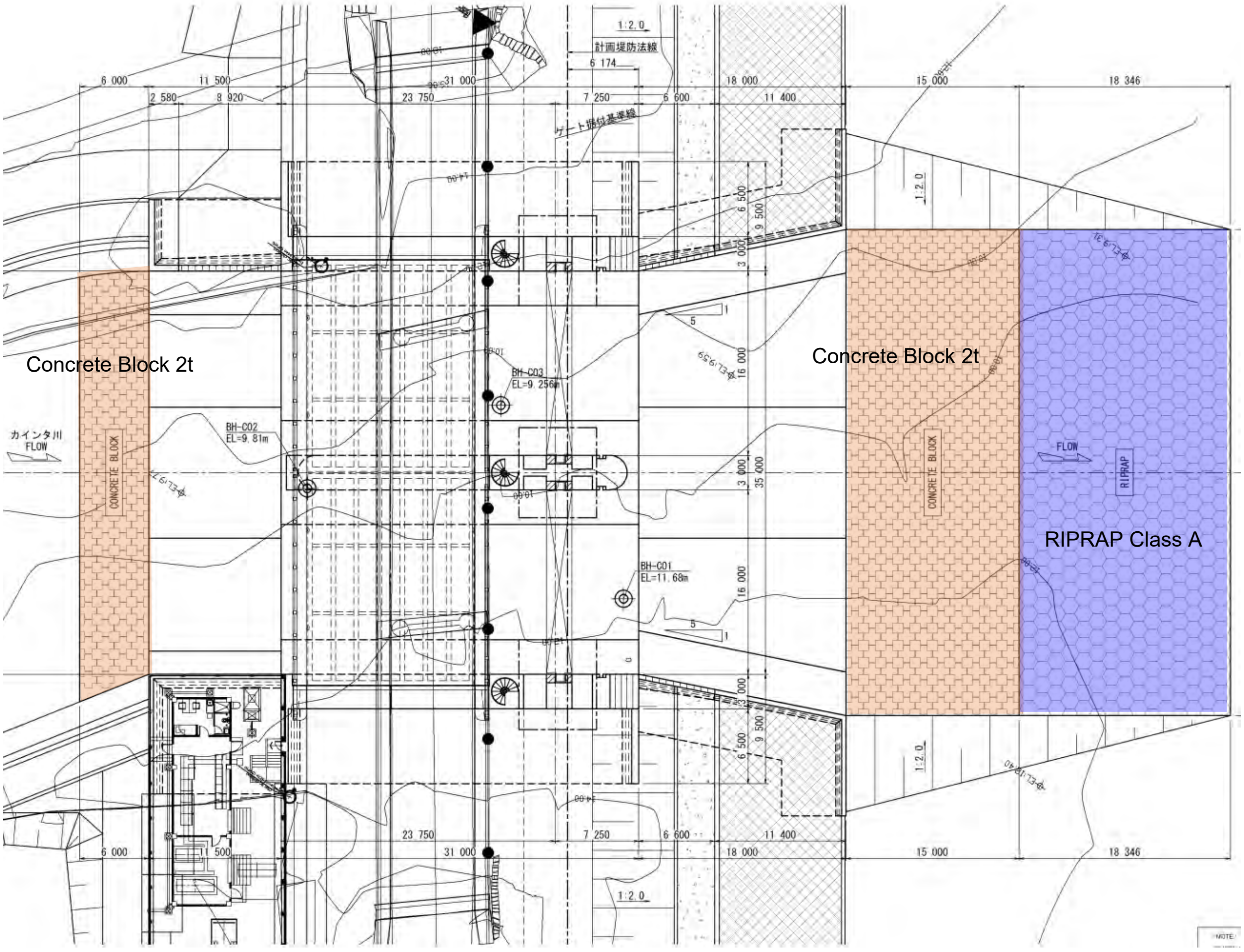
3) Partition Wall

The end part of the conditional revetment is provided with a partition wall so as not to be affected by erosion.

(2) Floor Protection Work

1) Installation Area

The scope of the floor protection work is shown below.



Source: Study team

Figure7.4.179 Area of Floor Protection Construction

2) Weight

(a) Concrete Block For Bed Protection

As determined in the basic design, the block weight of the floor protection work is 2t class.

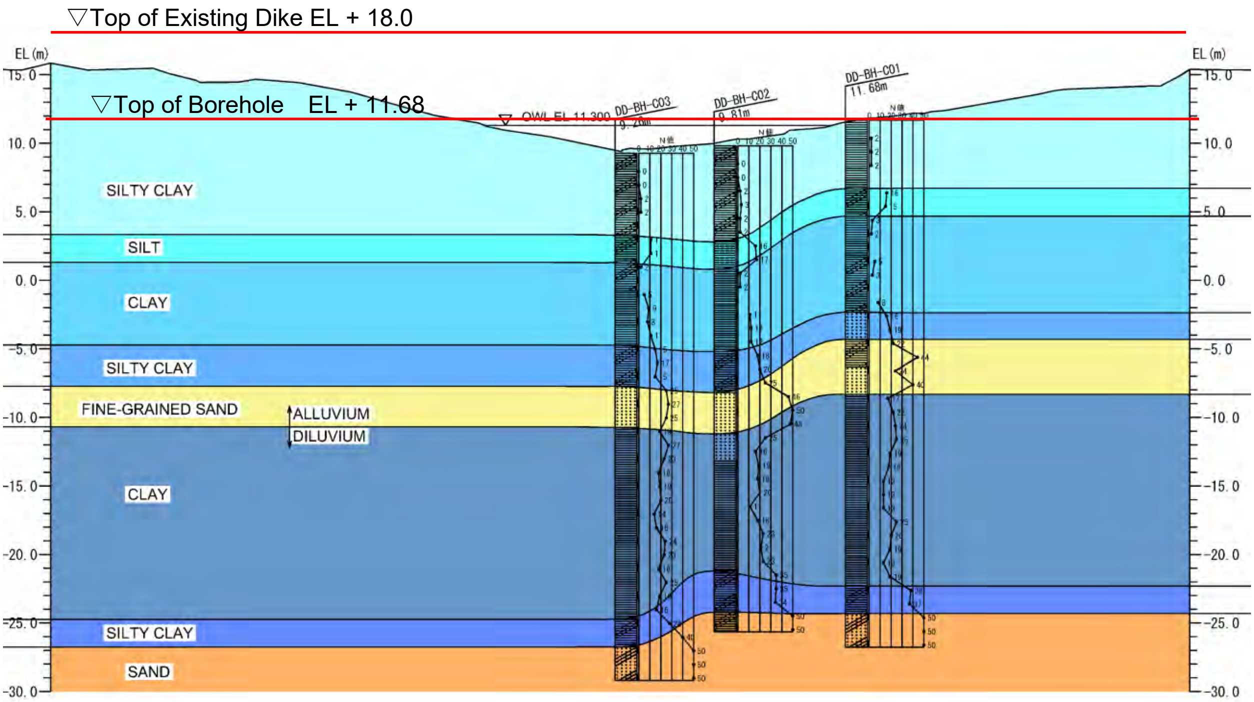
(b) RIPRAP

Use Riprap Class A as determined in the basic design.

(3) Earthwork

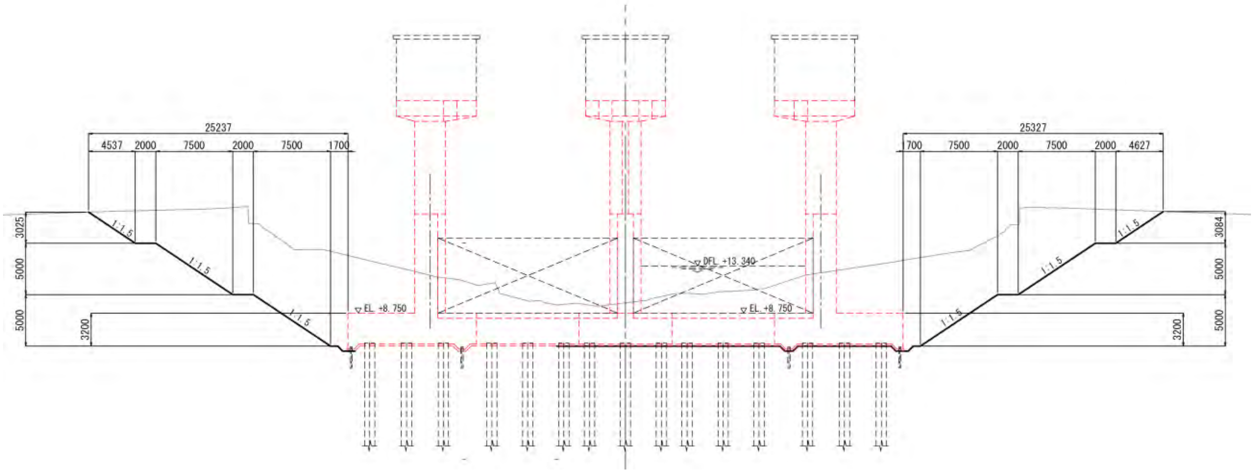
The excavation slope shall be set according to "Guideline for Quantity Calculation, Japan". The excavation slope is categorized by the type of the ground. Hence, the type of ground is confirmed.

The geological section of the Cainta Floodgate is shown in **Figure7.4.179**. According to the geological survey results, it is a clayey ground. However, the maximum height of the borehole elevation in the geological survey at 3 holes is EL + 11.68 m. The present embankment height is about EL + 18.0 m, and the formation structure above the borehole elevation is unknown, so the excavation slope of sandy soil is applied and it is 1.5:1. A cross sectional view of earthworks is shown in **Figure7.4.181**.



Source: Study Team

Figure7.4.180 Assumed Geological Section



Source: Study team

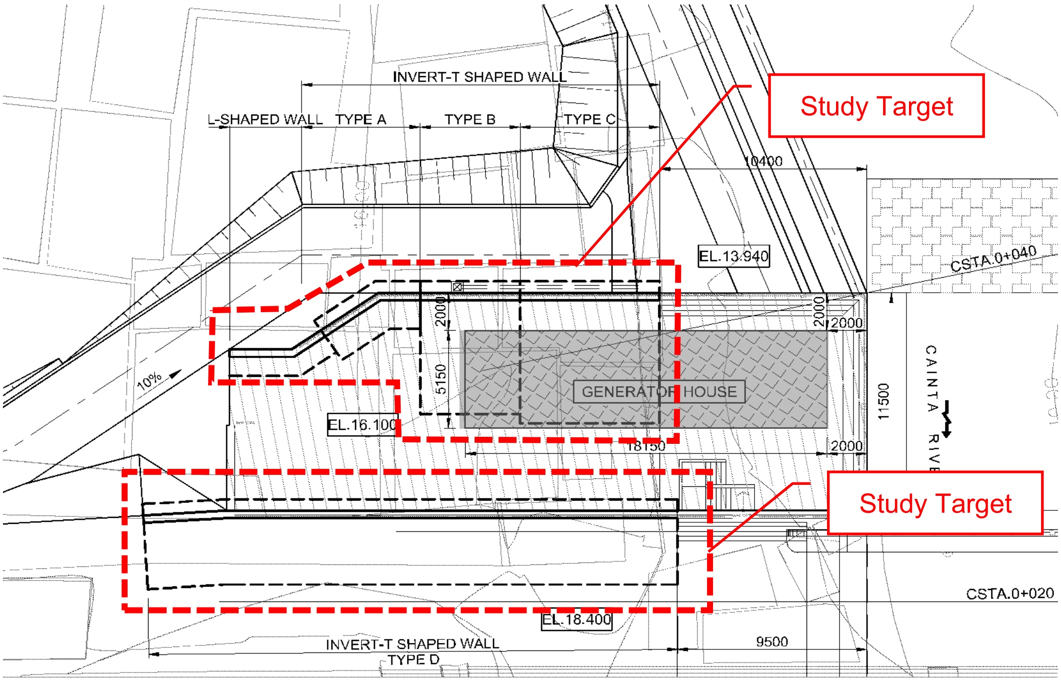
Figure7.4.181 Excavation Slope

7.4.2.6 Incidental Structure

(1) Retaining Wall for Generator House Area

1) Location of Retaining Walls

As it is stated in **Sub-section 6.4.3.11(6) Ground for Generator House**, a ground for a generator house would be developed at EL16.10. Hence, around the edge of the ground for the generator house needs retaining walls.



Source: Study team

Figure 7.4.182 Location of Retaining Walls for the Generator House Area

2) Design Condition

(a) Summary of Conditions

The summary of design conditions is shown in **Table 7.4.226**.

Table 7.4.226 Summary of Design Conditions

Item		Condition	Remarks
Material	Concrete	Class A f'_{ck}=20.7(N/mm2)	
	Re-bars	Grade 40 (275)	
	Soil	Sandy Soil γ=19.0(kN/m3), γ_{sat}=20.0(kN/m3)	
Water Level Condition	Normal	DFL > Lower Surface of the Retaining Wall RWL DFL ≦ Lower Surface of the Retaining Not Considered	DFL of Manggahan Floodway EL. 14.853
	Seismic	Not Considered	OWL EL.11.300 < Lower Surface of the Retaining Wall(The lowest is EL.13.100). The retaining walls and revetment have weep holes.
Load Condition	Normal	Ground in Rear Side : 10 kN/m2 Area of the Generator House Building : 50 kN/m2	
	Seismic	Ground in Rear Side : 5 kN/m2 Area of the Generator House Building : 50 kN/m2	

Source: Study team

(b) Load of Generator House Building

Load of Generator House Building is assumed as follows.

Table 7.4.227 Weight of Generator House Building

Item	Volume (m3)	Unit Weigh (t/m3)	Weight (t)
Structural Members Below Floor Slab	114.919	2.4	275.805
Structural Members Above Floor Slab	88.215	2.4	211.716
Non Structural RC Members	29.990	2.4	71.976
Cinder Concrete	26.750	1.7	45.474
CB wall	20.109	2.1	42.229
Sand Filling	60.025	1.9	114.048
Soil to be Excavated	-186.945	1.9	-355.196
		Sub-total	406.052

Source: Study team

<u>Weight of Structural Body</u> =Wight of Building / (Area of Foundation) x 9.8 m/s^2
= 406.052 t / (18.15m x 5.15m) x 9.8 m/s^2

= 42.572 (kN/m2)

- Weight of Finishing and Building Equipment =1.200 (kN/m2)
- Weight of Generator Set =Weight of Gen. Set / (Area of Foundation) x 9.8 m/s^2
  = (2.7 t + 1.3 t)/ (6.075 x 5.15m) x 9.8 m/s^2
  = 1.253 (kN/m2)
- Weight of Other Equipment = 2.400 (kN/m2)
- Total Weight = 42.572 + 1.200 + 1.253 + 2.400 = 47.425 (kN/m2) → 50 kN/m2

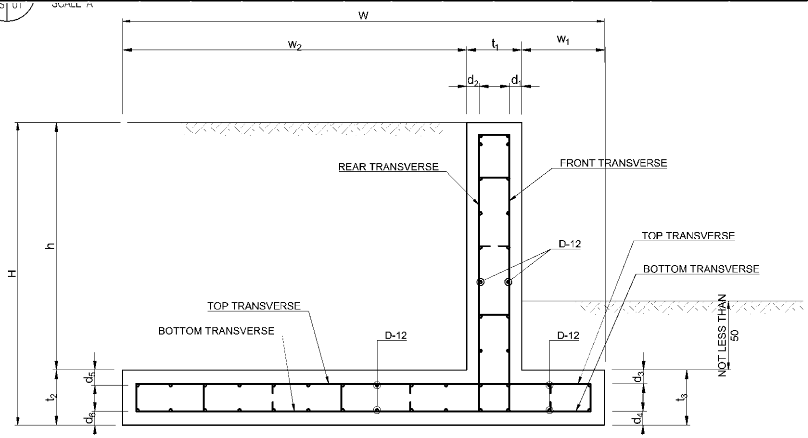
Accordingly, 50 kN/m2 would be assumed as the load acting on the area of the generator house building.

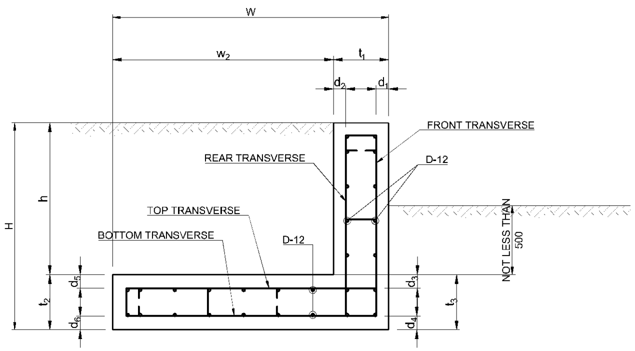
3) Study Results

In accordance with the above-mentioned condition, dimensions and bar arrangements of the retaining walls are determined and **Table 7.4.228** shows the summary. The detailed calculation is indicated in "**Vol.5A STRUCTURAL CALCULATION FOR CONTRACT PACKAGE 1**."

Table 7.4.228 Dimensions And Bar Arrangements of the Retaining Walls For Generator House Area

	Length L (m)	H (mm)	W (mm)	h (mm)	w_1 (mm)	w_2 (mm)	Vertical Wall t_1 (mm)	d_1 (mm)	d_2 (mm)	Rainforcing Bar Horizontal Extrior	Horizontal Interior	Vertical Front	Vertical Rear	Heal Slab t_2 (mm)	d_3 (mm)	d_4 (mm)	Rainforcing Bar Longditudial Bottom	Longditudial Top	Transverse Bottom	Transverse Top	Toe Slab t_3 (mm)	d_5 (mm)	d_6 (mm)	Rainforcing Bar Longditudial Bottom	Longditudial Top	Transverse Bottom	Transverse Top
L-shaped WALL	4.9	1500	2000	1100	-	1600	400	90	90	D12@250	D12@250	D12@250	D12@250	400	100	100	D12@250	D12@250	D12@250	D12@250	400	100	100				
Invert T-shaped WALL																											
TYPE A	6.0	2000	2500	1600	600	1500	400	90	90	D12@250	D12@250	D12@250	D12@250	400	100	100	D12@250	D12@250	D12@250	D12@250	400	100	100	D12@250	D12@250	D12@250	D12@250
TYPE B	5.0	2500	7000	2100	600	6000	400	90	90	D12@250	D12@250	D12@250	D16@250	400	100	100	D12@250	D12@250	D12@250	D16@250	400	100	100	D12@250	D12@250	D12@250	D12@250
TYPE C	7.0	3000	7500	2600	600	6500	400	90	90	D12@250	D12@250	D12@250	D20@250	400	100	100	D12@250	D12@250	D20@250	D20@250	400	100	100	D12@250	D12@250	D12@250	D12@250
TYEP D	26.8	3200	5000	2800	600	4000	400	90	90	D12@250	D12@250	D12@250	D20@250	400	100	100	D12@250	D12@250	D25@250	D20@250	400	100	100	D12@250	D12@250	D12@250	D12@250





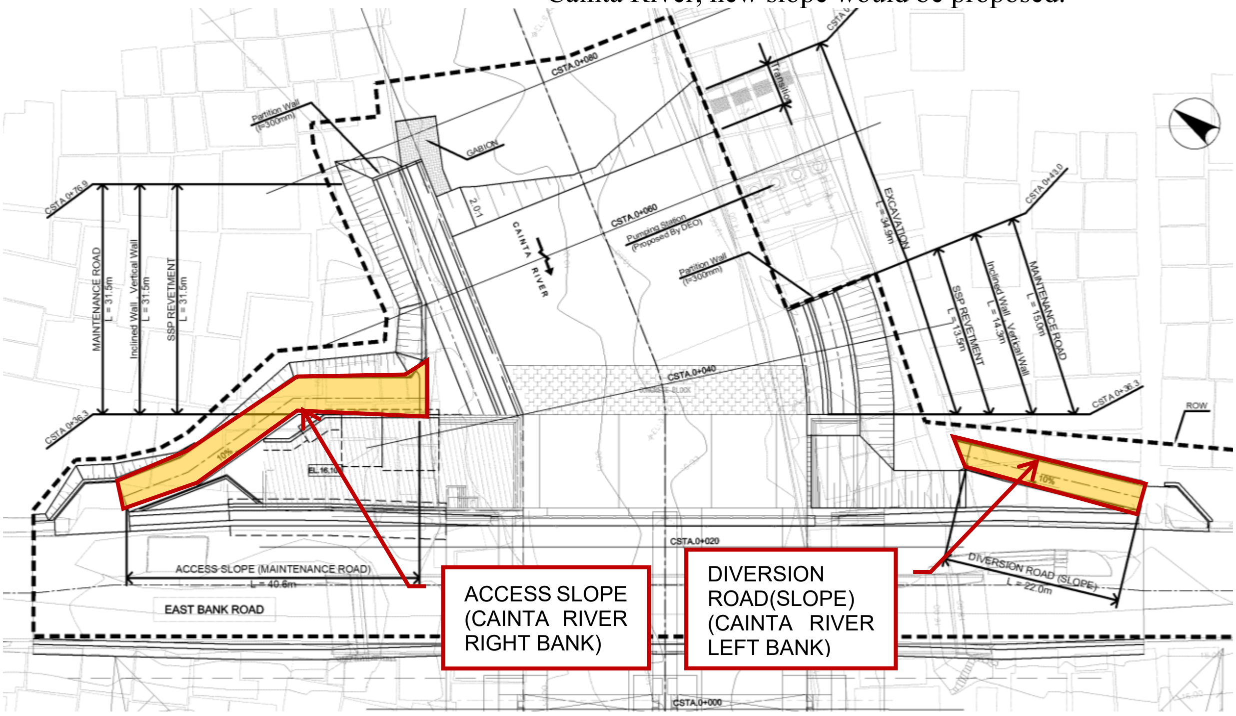
Source: Study team

(2) Slope

1) Locations and Purposes

The 2 locations of slopes indicated below would be proposed, and the purpose of each slope is also stated.

✓	Left Bank if Cainta River	In order to compensate the function of the existing slope, the diversion with same specification as the existing would be proposed.
✓	Right Bank if Cainta River	In order to access the generator house and revetment of Cainta River, new slope would be proposed.



Source: Study team

Figure 7.4.183 Locations of Slopes

2) Determination of Specification

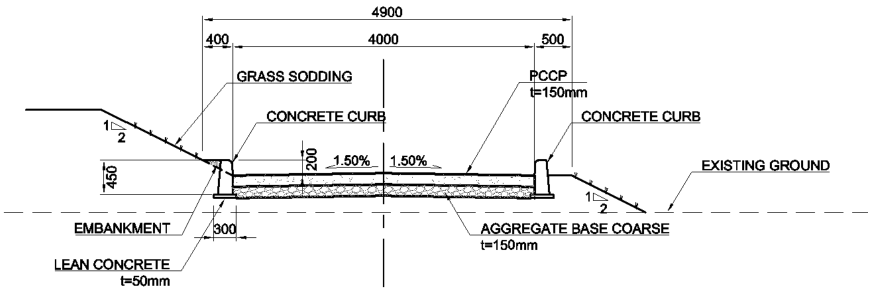
(a) Slope in the Right Bank of Cainta River

Specification of the slope in the right bank of Cainta River is shown in **Table 7.4.229**, and the standard section is indicated in **Figure 7.4.184**. This slope would be used only for the access of maintenance vehicles and the access to the generator house during construction and maintenance and it will not be opened to the public use. Hence, the pavement design is based on the minimum requirement stated in D.O. of DPWH.

Table 7.4.229 Specification of Slope in the Right Bank of Cainta River

Item	Value	Verification/ Remarks
Width	Carriage Way : 4.0 m	・ Not less than 3.0 m which is the minimum width of maintenance road ・ Considering the access of equipment and vehicles to the generator house during the maintenance and construction, set to 4.0 m.
Longitudinal Slope	10 %	・ In order to minimize the land acquisition area, the slope would be installed within the area between the edge of the existing side road and Cainta Riverbank. ・ Referring the longitudinal slope of the existing slope in the other side (about 10%)
Pavement	PCCP 150 mm	・ Minimum thickness indicated in DO No.11 ,Series of 2014 (FARM TO MARKET ROAD)
	Aggregate Subbase Course 150mm	・ Same thickness as PCCP ・ Considering the use only by maintenance vehicles

Source: Study team



Source: Study team

Figure 7.4.184 Standard Section of the Slope in the Right Bank of Cainta River

(b) Slope in the Left Bank of Cainta River

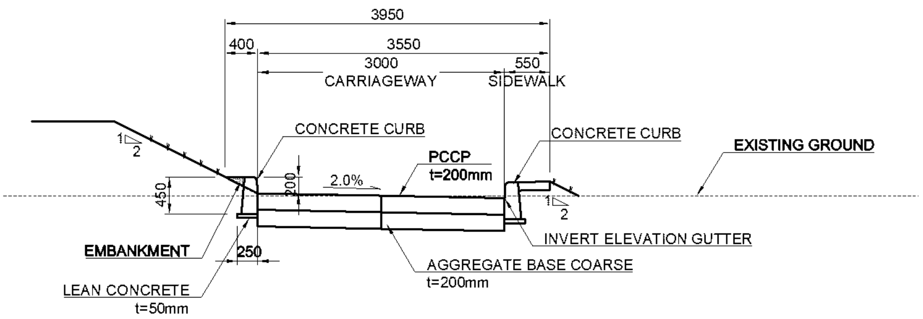
Specification of the slope in the left bank of Cainta River is shown in **Table 7.4.230**, and the standard section is indicated in**Figure 7.4.185**. The purpose of this slope is to divert the exiting slope. Since the width of carriageway of the existing slope is only 3m and there is little traffic volume, the proposed width is set to the same as the existing. Furthermore, the pavement design is determined based on the minimum requirement stated in D.O. of DPWH and considering that there will be some ordinary vehicles passing.

In addition, the required function sill be as same as the existing slope, it is recommended to confirm the actual thickness of the existing pavement before the construction and to change design to follow the existing pavement.

Table 7.4.230 Specification of Slope in the Left Bank of Cainta River

Item	Value	Verification/ Remarks
Width	Carriage Way :3.0 m	・ Same As the Existing
Longitudinal Slope	10 %	・ Following the longitudinal slope of the existing (About 10%)
Pavement	PCCP 200mm	・ Thicker than the minimum indicated in DO No.11 ,Series of 2014 ・ Considering some ordinary vehicles passing and the location in the urban area, 200mm is adopted.
	Aggregate Subbase Course 200mm	・ Same as PCCP ・ Referring FARM TO MARKET ROAD

Source: Study team



Source: Study team

Figure 7.4.185 Standard Section of the Slope in the Left Bank of Cainta River

(3) Drainage Facilities

Since the 3 locations of the existing outlets flowing into Cainta River will be affected by the proposed Cainta Floodgate and revetment along Cainta River, the diversion of these outlets shall be considered.

1) Drainage Planning

(a) Planning Condition

The planning condition is basically as same as the one adopted in the planning along Marikina River. The planning condition is summarized as follows. The details of each condition are sated in **Sub-section 6.2.3 Drainage Planning**.

Table 7.4.231 Summary of Drainage Planning Condition

Item	Condition		Remarks
Design Scale	Design Capacity : 15 years Check Capacity : 25 years		Based on DGCS Volume 3
Minimum Size of Pipe	900 mm		Based on DGCS Volume 3
Method for Discharge Calculation	Rational Formula		Based on DGCS Volume 3
Rainfall intensity Formula	$I = \frac{a}{Tc^n + b}$	I : Rainfall Intensity (mm/hr) Tc : Time of Concentration (min) n,a,b : regression constants (Refer to Table 6.2.7)	Same as the planning along Marikina River
Formula For Time of Concentration	Tc＝to + tg + td	to : Overland Flow tg : Curb and Gutter Flow td : Drain Flow	Based on DGCS Volume 3 Refer to 6.2.3.1 planning Condition for the calculation of each intel time

Source: Study team

(b) Catchment Area

The catchment area of each diverted drainage is shown in **Figure 7.4.186**.

(c) Calculation of Discharge

Table 7.4.233 shows the calculation results of the discharge from each catchment area. When the time of concentration is calculated, the hydraulic radius and longitudinal slope of the existing drainage is needed, the considered information of the existing drainage is also incorporated in **Table 7.4.233**.

2) Design of Drainage Facility

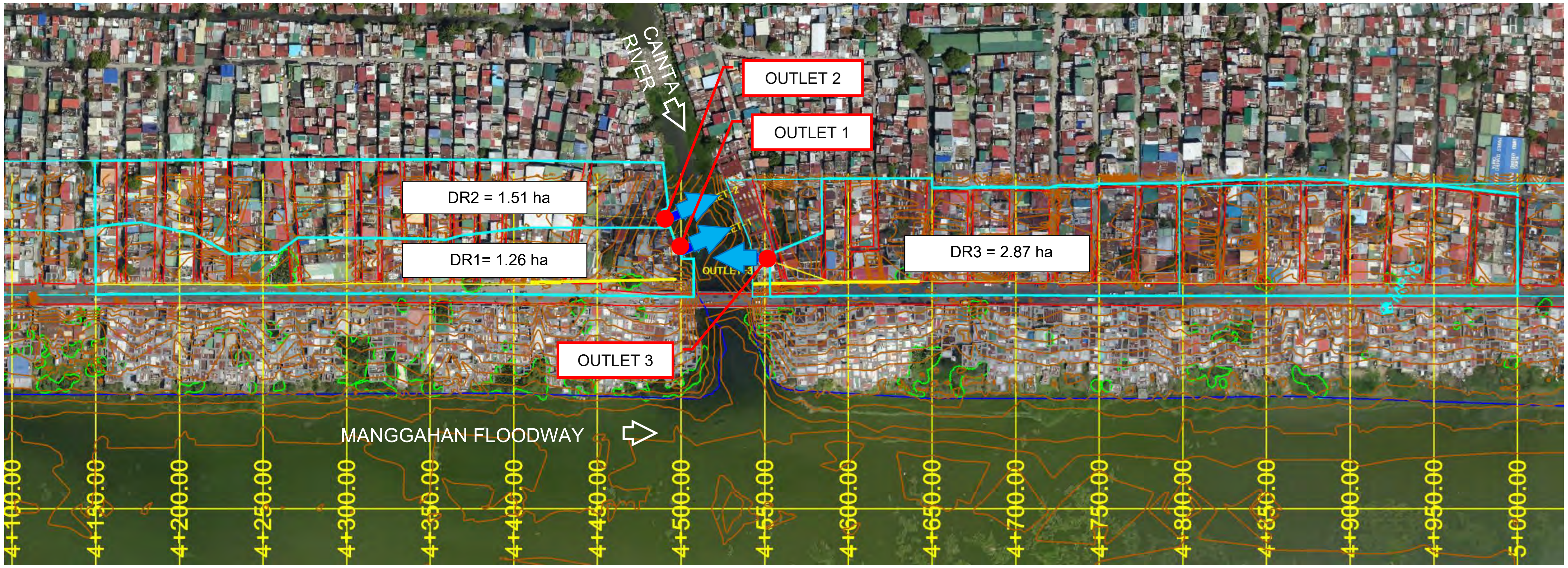
The sizes of drainage pipes would be determined to accommodate the discharge calculated in the previous sub-section based on the concepts stated in Sub-section 6.2.4.1 Basic Design of Outlet. The determined size of the drainage pipe is shown in **Table 7.4.232**, and the verification results of the flow capacity are indicated in **Table 7.4.234**. Furthermore, **Figure 7.4.187** and **Figure 7.4.188** show the standard drawings of the proposed outlets.

Since the ground elevation in the land side is higher than the DFL of Cainta River and this portion is so-called an excavated River channel, no flapgate to prevent backflow from the river is needed.

Table 7.4.232 Summary of Drainage Outlet (Cainta River)

	STA. (Cainta River)	Invert Elevation (EL.m)	Type /Size	Existing Drainage Size
Outlet 1	CSTA 0+049.9 / Right Bank	12.05	RCPC ϕ900	RCPC ϕ900
Outlet 2	CSTA 0+070.9 / Right Bank	11.95	RCPC ϕ1200	Box B1.0m x H1.2m
Outlet 3	CSTA 0+036.9/ Left Bank	11.95	RCPC ϕ1200	RCPC ϕ1100

Source: Study team



Source: Study team

Figure 7.4.186 Catchment Area (Around Cainta Floodgate Site)

Table 7.4.233 Results of Discharge Calculation

DRAIN DESIGNATION		Estimating Catchment Design Flows																			RCP FOR CHECKING								
		Catchment Area (A)	RUNOFF COEFFICIENT						Length of Drain (Ld)							R_{25}	R_{15}	Runoff Coeff.	Total Discharge	Total Discharge	Dimension	Slope	Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Rougness parameter	Velocity	Discharge
		Partial Accumulated	RESIDENTIAL 1	RESIDENTIAL 2	RESIDENTIAL 3	BUSINESS	Paved (road)	AGRICULTURAL	Partial Accumulated	Horton's roughness	Average Surface Pass	Surface Slope	Over Flow	Drain Flow	Tc	25-Year	15-Year	C	Q_{25}	Q_{15}	φ	I		A	S	R	n	V	Q
		Ap	0.35	0.50	0.65	0.80	0.95	0.30	Ld	n*	L	S	T0	Td		Rainfall Int.	Rainfall Int.												
		ha							m		m	%	min	min	min	mm/hr	mm/hr		m^3/s	m^3/s	m	‰		m^2	m	m		m/s	m/s
Cainta River	DR1	1.26			0.83		0.43		360.00																				
Right Bank 1		1.26	0.00	0.00	0.83	0.00	0.43	0.00	360.00	0.015	33.000	1.000	5.15	3.79	8.9	275.2	251.7	0.75	0.72	0.67	0.90	2.6	0.57	0.42	1.65	0.26	0.013	**1.58**	**0.67**
Cainta River	DR3	2.87			2.10		0.77		460.20																				
Left Bank		2.87	0.00	0.00	2.10	0.00	0.77	0.00	460.20	0.015	60.000	1.000	6.28	4.10	10.4	201.4	152.1	0.73	1.17	0.89	1.10	3.3	0.55	0.48	1.73	0.28	0.013	**1.87**	**0.89**

DRAIN DESIGNATION		Estimating Catchment Design Flows																			For Checking										
		Catchment Area (A)	RUNOFF COEFFICIENT						Length of Drain (Ld)							R_{25}	R_{15}	Runoff Coeff.	Total Discharge	Total Discharge	Dimension		Water Depth	Slope	Slope	Flow Area	Wetted Perimeter	Hydraulic Radius	Rougness parameter	Velocity	Discharge
		Partial Accumulated	RESIDENTIAL 1	RESIDENTIAL 2	RESIDENTIAL 3	BUSINESS	Paved (road)	AGRICULTURAL	Partial Accumulated	Horton's roughness	Average Surface Pass	Surface Slope	Over Flow	Drain Flow	Tc	25-Year Rainfall Int.	15-Year Rainfall Int.	C	Q_{25}	Q_{15}	W	H	D	1:N	I	A	S	R	n	V	Q
		Ap	0.35	0.50	0.65	0.80	0.95	0.30	Ld	n*	L	S	T0	Td																	
		ha							m		m	%	min	min	min	mm/hr	mm/hr		m^3/s	m^3/s	m	m	m		‰	m^2	m	m		m/s	m^3/s
Cainta River	DR2	1.52			1.30		0.22		358.20																						
Right Bank 2		1.52	0.00	0.00	1.30	0.00	0.22	0.00	358.20	0.015	19.900	1.000	4.35	4.58	8.9	275.4	199.6	0.69	0.81	0.59	1.00	1.20	0.45	0.00	2.6	0.45	1.91	0.24	0.015	1.30	0.59

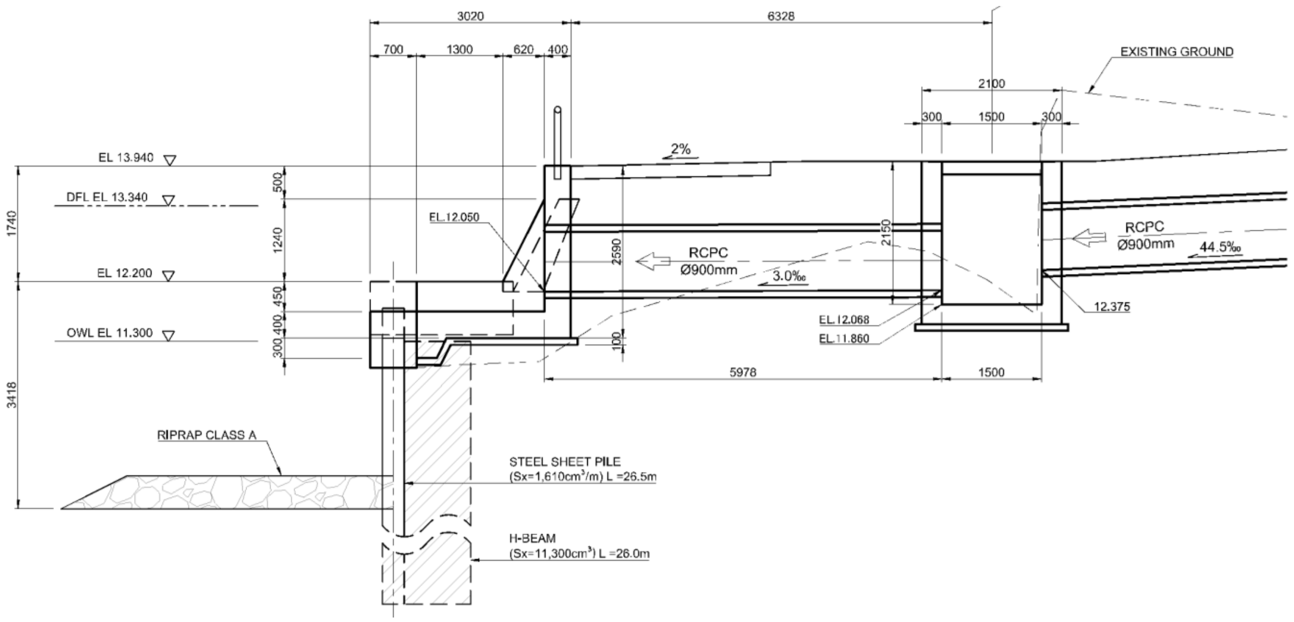
Source: Study team

Table 7.4.234 Verification Results of Flow Capacity

DRAIN DESIGNATION		Estimating Catchment Design Flows								Proposed Dimension (Pipe)									Proposed Dimension (Pipe)									Evaluation		Remarks
		Catchment Area (A)	R_{25}	R_{15}	Runoff Coeff.	Total Discharge		Total Discharge		Dimension	Slope	Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Rougness parameter	Velocity	Discharge	Dimension	Slope	Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Rougness parameter	Velocity	Discharge	Evaluation For Q25	Evaluation For Q15	
		Partial Accumulated Ap	25-Year Rainfall Int.	15-Year Rainfall Int.	C	Q_{25}	Sum Q	Q_{15}	Sum Q	ϕ	I	90%	A	S	R	n	V	Q	ϕ	I	100%	A	S	R	n	V	Q			
		ha	mm/hr	mm/hr		m^3/s	m^3/s	m^3/s	m^3/s	m		m	m^2	m	m		m/s	m^3/s	m		m	m^2	m	m		m/s	m^3/s			
Cainta River Right Bank 1	**DR1**	1.26								(EXSISTING RCPC)									(EXSISTING RCPC)											
		1.26	275.2	251.7	0.75	0.72	0.72	0.67	0.67	0.90	2.6	0.81	0.60	2.25	0.27	0.013	1.63	**0.98**	0.90	2.6	0.90	0.64	2.83	0.23	0.013	1.45	**0.92**	**OK**	**OK**	
	OUTLET 1			**Total Discharge**			**0.72**		**0.67**	0.90	3.0	0.81	0.60	2.25	0.27	0.013	1.75	**1.06**	0.90	3.0	0.90	0.64	2.83	0.23	0.013	1.56	**0.99**	**OK**	**OK**	
Cainta River Right Bank 2	**DRS**	0.00																												
		0.00																												
	OUTLET 2	**Total Discharge(EXISTING CAPACITY)**					**2.04**		**1.55**	1.20	2.5	1.08	1.07	3.00	0.36	0.013	1.94	**2.08**	1.20	2.5	1.20	1.13	3.77	0.30	0.013	1.72	**1.95**	**OK**	**OK**	
Cainta River Left Bank	**DR3**	2.87								(EXSISTING RCPC)									(EXSISTING RCPC)											
		2.87	201.4	152.1	0.73	1.17	1.17	0.89	0.89	1.10	3.3	0.99	0.90	2.75	0.33	0.013	2.10	**1.89**	1.10	3.3	1.10	0.95	3.46	0.28	0.013	1.87	**1.78**	**OK**	**OK**	
	OUTLET 3			**Total Discharge**			**1.17**		**0.89**	1.20	2.5	1.08	1.07	3.00	0.36	0.013	1.94	**2.08**	1.20	2.5	1.20	1.13	3.77	0.30	0.013	1.72	**1.95**	**OK**	**OK**	

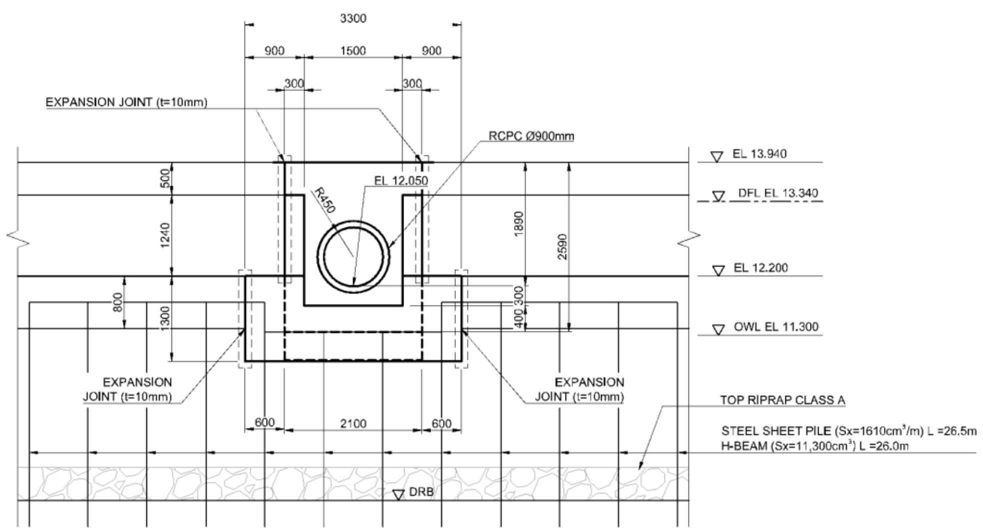
DRAIN DESIGNATION		Estimating Catchment Design Flows								Proposed Dimension (U-ditch or Box Culvert)											Proposed Dimension (U-ditch or Box Culvert)											Evaluation		Remarks
		Catchment Area (A)	R_{25}	R_{15}	Runoff Coeff.	Total Discharge		Total Discharge		Dimension		Water Depth	Slope	Slope	Flow Area	Wetted Perimeter	Hydraulic Radius	Rougness parameter	Velocity	Discharge	Dimension		Water Depth	Slope	Slope	Flow Area	Wetted Perimeter	Hydraulic Radius	Rougness parameter	Velocity	Discharge	Evaluation For Q25	Evaluation For Q15	
		Partial Accumulated Ap	25-Year Rainfall Int.	15-Year Rainfall Int.	C	Q_{25}	Sum Q	Q_{15}	Sum Q	W	H	D	1:N	I	A	S	R	n	V	Q	W	H	D	1:N	I	A	S	R	n	V	Q			
		ha	mm/hr	mm/hr		m^3/s	m^3/s	m^3/s	m^3/s	m	m	m		‰	m^2	m	m		m/s	m^3/s	m	m	m		‰	m^2	m	m		m/s	m^3/s			
Cainta River Right Bank 2	**DR2**	1.52								(EXISTING BOX CULVERT)											(EXISTING BOX CULVERT)													
		1.52	275.4	199.6	0.69	0.81	0.81	0.59	0.59	1.00	1.20	1.20	0.00	2.6	1.20	3.40	0.35	0.015	1.70	2.04	1.00	1.20	0.96	0.00	2.6	0.96	2.92	0.33	0.015	1.62	1.55	**OK**	**OK**	Box Culvert

Source: Study team



Source: Study team

Figure 7.4.187 Standard Profile Drawing of Drainage Outlet(Cainta River OUTLET 1)



Source: Study team

Figure 7.4.188 Standard Elevation Drawing of Drainage Outlet(Cainta River OUTLET 1)

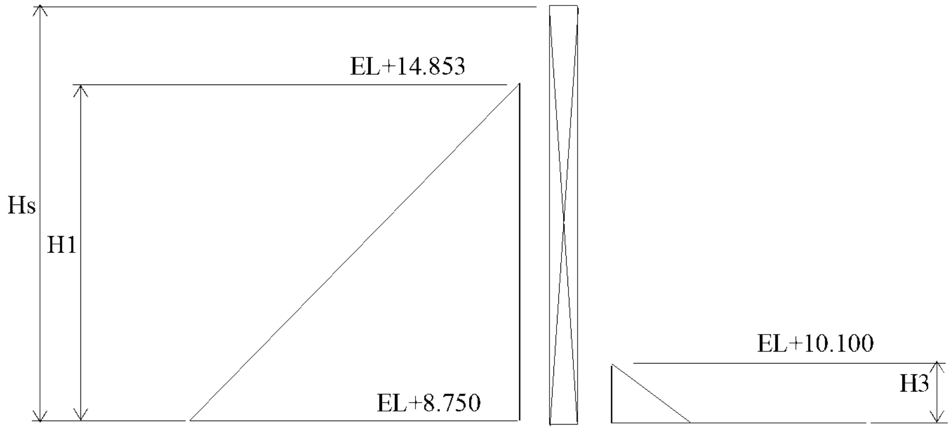
7.4.3 Gate Facility Design

7.4.3.1 Design Conditions

(1) Main Gate

Type	:	Duplex Stainless Steel Roller Gate							
Clear Span x Effective Height	:	W 16.000 m × H 7.310 m							
Number of Gates	:	2 Gate							
Water Sealing System	:	Rear three-way rubber Water Sealing							
Design Head	:	(Front)	6.103	m	(	EL	+	14.853)	→DFL
		(Rear)	1.350	m	(	EL	+	10.100)	→LWL * observed Lowest water level
Operating Head (Opening)	:	(Front)	4.190	m	(	EL	+	12.940)	→Tributary bank height -1.0 m
		(Rear)	5.190	m	(	EL	+	13.940)	→Tributary bank height
Operating Head (Closing)	:	(Front)	7.190	m	(	EL	+	15.940)	→Floodway Dike Height
		(Rear)	5.190	m	(	EL	+	13.940)	→Tributary bank height
Invert Elevation	:	EL + 8.750							
Type of Hoist	:	Wire drum winch (1M1D)							
Hoisting Height	:	9.650 m							
Main Materials	:	Gate Leaf : SUS 821 L1	Guide fram : SUS 304 N2 system etc. (exposed part)						
			SM 490 etc. (buried part)						
Water Quality	:	Outer Side : fresh water	Inner Side : fresh water						
Extra Thickness	:	Outer Side : one side	0.0 mm (fresh water)						
	:	Inner Side : one side	0.0 mm (fresh water)						

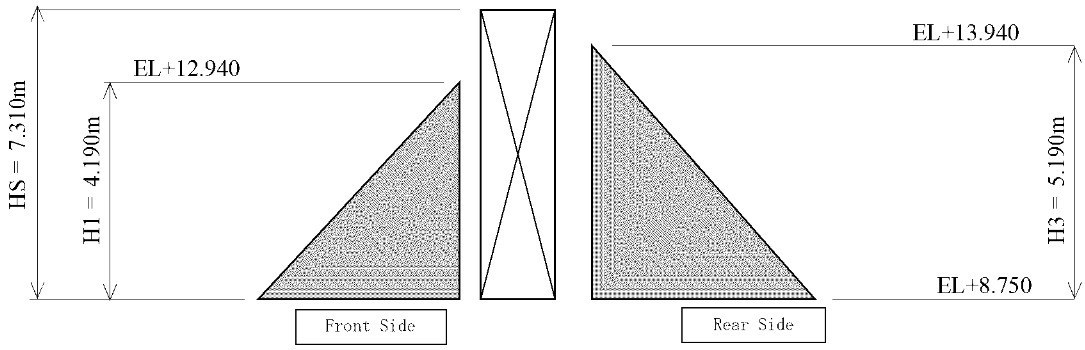
Standaı : Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan, ... 4th printing January 2014
Technical Specification for Dams and Weirs in Japan (draft)
(Explanation of Standard and Manual for Facility Planning) ... October 2016, Edition 1 published



P_S	:	Hydrostatic Load	...		kN
P_o	:	Front Side Hydraulic Load	...		kN
P_i	:	Rear Side Hydrostatic Load	...		kN
H_S	:	Water Sealing Height	...	7.310	m
H_1	:	Front Side Design Head	...	6.103	m
H_2	:	Front Side Water Depth at the Top of Gate Leaf	...	0.000	m
H_3	:	Design Water Depth on the Rear Side	...	1.350	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	...	0.000	m
B_S	:	Water Sealing Width	...	16.200	m
γ_1	:	Unit Weight of Water in the Front Side	...	9.810	kN/m3
γ_2	:	Unit Weight of Water in the Rear Side	...	9.810	kN/m3

Source: Study team

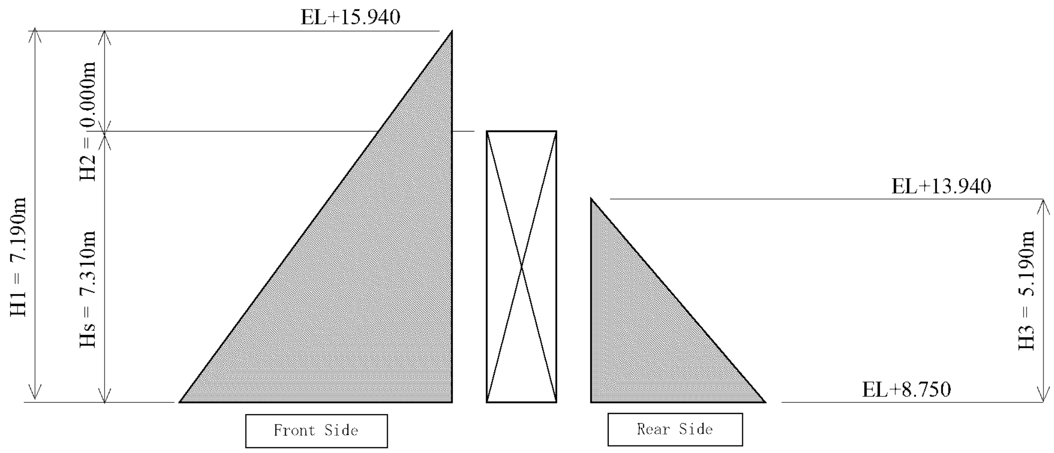
Figure7.4.189 Load Model Diagram (Design Load)



$P_{S\ (O)}$	:	Hydrostatic Pressure Load (Opening)	...		kN
P_o	:	Front Side Hydraulic Load	...		kN
P_i	:	Rear Side Hydrostatic Load	...		kN
H_S	:	Water Sealing Height	...	7.310	m
H_1	:	Front Side Operating Head	...	4.190	m
H_2	:	Front Side Water Depth at the Top of Gate Leaf	...	0.000	m
H_3	:	Operation Head in Rear Side	...	5.190	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	...	0.000	m
B_S	:	Water Sealing Width	...	16.200	m
γ_1	:	Unit Weight of Water in the Front Side	...	9.810	kN/m3
γ_2	:	Unit Weight of Water in the Rear Side	...	9.810	kN/m3

Source: Study team

Figure7.4.190 Load Model Diagram (Operational Load: Opening)



P_o	:	Front Side Hydraulic Load	...		kN
P_i	:	Rear Side Hydrostatic Load	...		kN
H_S	:	Water Sealing Height	...	7.310	m
H_1	:	Front Side Operating Head	...	7.190	m
H_2	:	Front Side Water Depth at the Top of Gate Leaf	...	0.000	m
H_3	:	Operation Head in Rear Side	...	5.190	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	...	0.000	m
B_S	:	Water Sealing Width	...	16.200	m
γ_1	:	Unit Weight of Water in the Front Side	...	9.810	kN/m3
γ_2	:	Unit Weight of Water in the Rear Side	...	9.810	kN/m3

Source: Study team

Figure7.4.191 Load Model Diagram (Operational Load: Closing)

(2) Stoplog

(1) Type — Steel Plate Girder Slide Gate (Strut Type Stoplog)

(2) Number of Gate — 1 gate

(3) Clear Span — 16.000 m (4 Split = 4.000 × 4 leaf)

(4) Height of Gate Leaf — 5.700 m (3 Split = 1.900 × 3 leaf)

(5) Design Head — Upstream Side 5.650 m (EL + 14.400) →Maximum water level for the past 5 years

Downstream Side 0.000 m (EL + 8.750)

(6) Invert Elevation — EL + 8.750

(7) Water Sealing System — Rear Side Three-way Rubber Sealing

(8) opening and Closing Method — By Crane

(9) Extra Thickness — 0.0 mm

(10) Deflection — 1/600 or less of Span

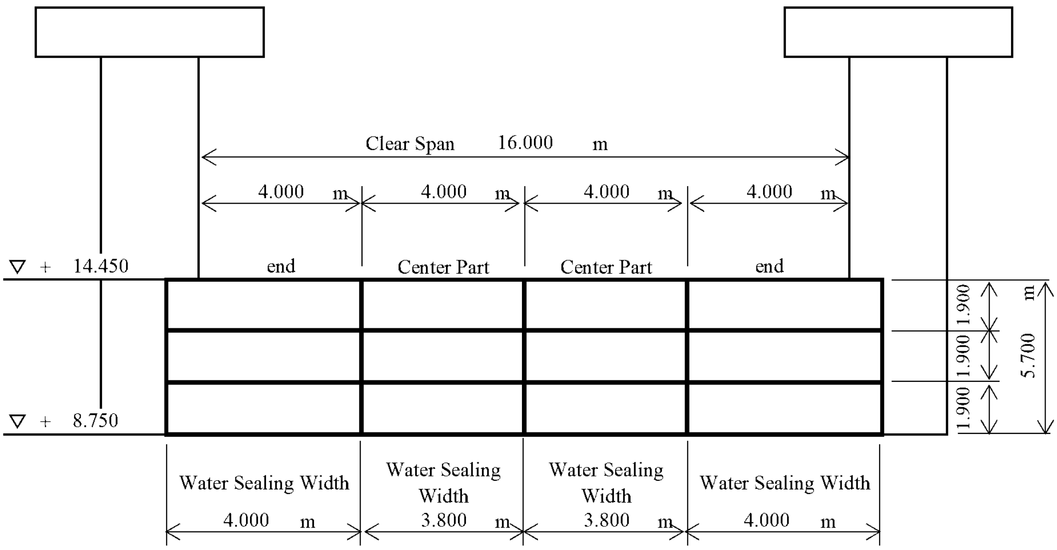
(11) Allowable stress intensity — Based on the Technical Specification for Dams and Weirs in Japan (Draft) [Gate for Repairing Work].

(12) Unit Weight of Water — 9.81 kN / m3

(13) Material — Stoplog SM 400

Guide Frame SUS 304 + SM 400

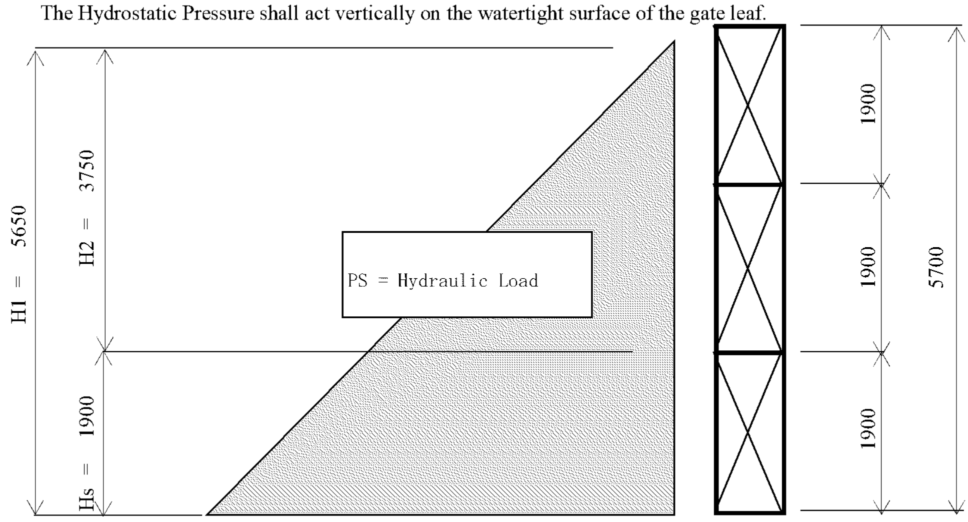
Strut SM 400



Source: Study team

Figure7.4.192 General Arrangement

The Hydrostatic Pressure shall act vertically on the watertight surface of the gate leaf.



H_S	:	Water Sealing Height	=	1.900 m
H_1	:	Upstream Design Head	=	5.650 m
H_2	:	Upstream Side Depth at the Top of Stoplog	=	3.750 m
B_{S1}	:	Water Sealing Width (End Stoplog)	=	4.000 m
B_{S2}	:	Water Sealing Width (Center Stoplog)	=	3.800 m
γ_1	:	Unit Weight of Water	=	9.81 kN / m3

Source: Study team

Figure7.4.193 Load Model Diagram

(3) Hoist

A self-weight lowering function is added to the Hoisting Device for emergency. Since it is not intended to hasten the closing operation, the self-weight lowering speed is set toe 1.0 m/min, which is the minimum value among the standard values described in “Technical Specification for Dams and Weirs in Japan (Draft) ".

[Sample Specification]

1. Lowering speed by the rapid closing device shall be about 1.0 to 2.0 m/min and the maximum should not be more than 9.0 m/min.
2. In case of mechanical hoist, the rapid closing device shall consist of device for releasing brake and for controlling speed. The device for releasing break has DC electromagnetic brake and DC power supply, etc. The device for controlling speed has centrifugal brake, fan brake, hydraulic brake, etc.

Source: Study Team translated from Draft technical standard for dams and weirs, p. 43, 28.10

Type of Hoist	1 motor/1 drum wire rope winding type
Numbers of Unit	2 Gates (1 Unit/1 Gate)
Opening and Closing Load	Opening and Closing Load W = 680 kN Load of Gate Leaf W ' ' = 635 kN
Opening and Closing Speed	Vm = 0.300 m/min Self-weight Lowering Speed V = 1.000 m/min
Hoisting Height	Normal Height 9.650 m Dogging Height 9.950 m Dogging Upper Limit Height 10.050 m Emergency Upper Limit Height 10.150 m
Operating System	Local Operation (Machine Side)
Power Source	220 V 60 Hz
Number of Wire Rope	6 Article (one side)
Reduction Gear	Differential Gear Reduction 1/500
Safety Factor	Technical Specification for Dams and Weirs in Japan (Draft) (July 2011 edition) (hereinafter referred to as [dam and weir (Draft)] Design Guideline for Hoist of Gate(Mechanical)(Draft)(August 2000 edition) (hereinafter referred to as [Design Guideline])

7.4.3.2 Design Calculation

In this section, only the calculation results of each facility are shown, and detailed design calculations are indicated in **Vol.5A Structural Calculation for Contract Package-1**.

Table7.4.235 Gate Calculation Results

1-1. Gate Leaf

			Calculation Results	Tolerance	Evaluation
Upper Girder	Maximum Bending Stress		21.9 N/mm2	158 N/mm2	OK
	Maximum Shear Stress		1.75 N/mm2	104 N/mm2	OK
	Deflection		1/1893	1/800	OK
Main Girder (G3, G4)	Maximum Bending Stress		94.5 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		31.6 N/mm2	104 N/mm2	OK
	Deflection		1/815	1/800	OK
Main Girder (G2, G5)	Maximum Bending Stress		94.7 N/mm2	159 N/mm2	OK
	Maximum Shear Stress		18.5 N/mm2	104 N/mm2	OK
	Deflection		1/815	1/800	OK
Skin Plate	Bending Stress	Parcel: (1)	26.2 N/mm2	180 N/mm2	OK
		Parcels: (2)	69.8 N/mm2	180 N/mm2	OK
		Parcels: (3)	104 N/mm2	180 N/mm2	OK
		Parcels: (4)	81.2 N/mm2	180 N/mm2	OK
Stringer	Bending Stress	Parcel: (1)	1.72 N/mm2	54.8 N/mm2	OK
		Parcels: (2)	2.15 N/mm2	54.8 N/mm2	OK
		Parcels: (3)	2.44 N/mm2	54.8 N/mm2	OK
		Parcels: (4)	1.30 N/mm2	54.8 N/mm2	OK
	Shear Stress	Parcel: (1)	0.64 N/mm2	104 N/mm2	OK
		Parcels: (2)	0.95 N/mm2	104 N/mm2	OK
		Parcels: (3)	1.31 N/mm2	104 N/mm2	OK
		Parcels: (4)	0.96 N/mm2	104 N/mm2	OK
End stringer (Portion where Roller Shaft does not penetrate)	Maximum Bending Stress	Compression Side	101 N/mm2	156 N/mm2	OK
		Tensile Side	101 N/mm2	200 N/mm2	OK
	Maximum Shear Stress		45.1 N/mm2	115 N/mm2	OK
	Composite Stress Intensity		128 N/mm2	172 N/mm2	OK
End stringer (Roller Shaft Mounting Part)	Maximum Bending Stress	Compression Side	89.1 N/mm2	156 N/mm2	OK
		Tensile Side	89.1 N/mm2	200 N/mm2	OK
	Maximum Shear Stress		80.2 N/mm2	115 N/mm2	OK
	Composite Stress Intensity		165 N/mm2	172 N/mm2	OK
Main Roller	Contact Stress Intensity		963 N/mm2	1030 N/mm2	OK
Main Roller Shaft (Point A)	Maximum Bending Stress		155 N/mm2	170 N/mm2	OK
	Maximum Shear Stress		25.4 N/mm2	100 N/mm2	OK
	Composite Stress Intensity		161 N/mm2	187 N/mm2	OK
Main Roller Shaft (Point B)	Maximum Bending Stress		112 N/mm2	170 N/mm2	OK
	Maximum Shear Stress		22.9 N/mm2	100 N/mm2	OK
	Composite Stress Intensity		119 N/mm2	187 N/mm2	OK
Bearing	Surface Pressure		16.3 N/mm2	23.0 N/mm2	OK

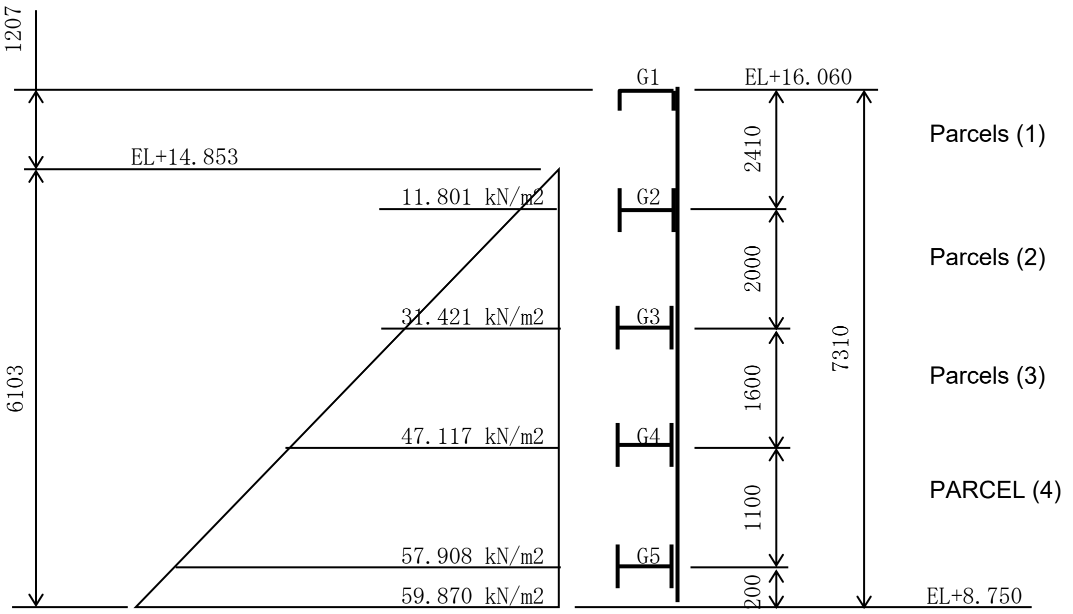
Source: Study team

Table7.4.236 Calculation Result of Guide Frame

1-2. Guide Frame

			Calculation Results	Tolerance	Evaluation
Roller rail	Concrete Bearing Stress		1.58 N/mm2	5.90 N/mm2	OK
	Concrete Shear Stress		0.24 N/mm2	0.40 N/mm2	OK
	Bending Stress in Roller Rail		95.3 N/mm2	160 N/mm2	OK
	Roller tread thickness		24.3 mm	30.0 mm	OK
	Local Stress at the Bottom Flange of Roller Rail		195 N/mm2	240 N/mm2	OK
	Bending Stress at Bottom Flange of Roller Rail		60.5 N/mm2	160 N/mm2	OK
	Composite Stress Intensity of Bottom Flange		136 N/mm2	176 N/mm2	OK
	K by Adjacent Rollers	K1	1.51 N/mm2	5.90 N/mm2	OK
		K2	1.58 N/mm2	5.90 N/mm2	OK
	Bearing Stress due to Adjacent Rollers		4.47 N/mm2	160 N/mm2	OK
	Compressive Stress by Adjacent Rollers		1.43 N/mm2	5.90 N/mm2	OK

Source: Study team



Source: Study team

Figure7.4.194 Section Shape (Main Gate)

Table7.4.237 Calculation Result of the Stoplog

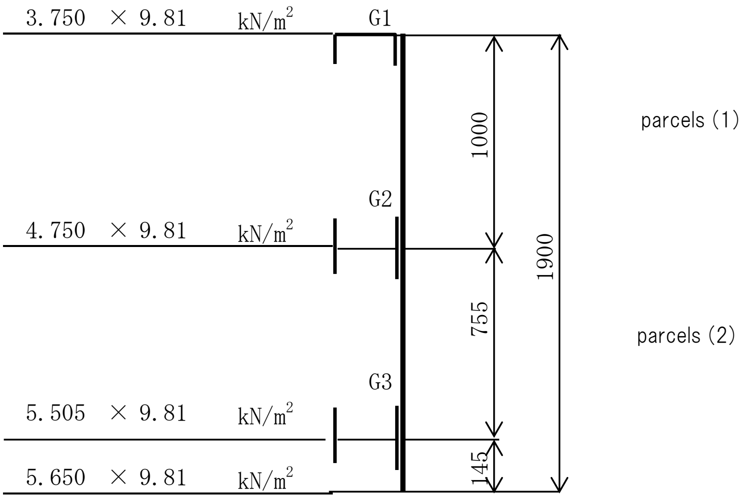
1-1. Gate Leaf

			Calculation Results	Tolerance	Evaluation
Main Girder (G1)	Maximum Bending Stress		113.0 N/mm2	165 N/mm2	OK
	Maximum Shear Stress		15.80 N/mm2	105 N/mm2	OK
	Deflection		1/649	1/600	OK
Main Girder (G2)	Maximum Bending Stress		121.0 N/mm2	179 N/mm2	OK
	Maximum Shear Stress		32.4 N/mm2	105 N/mm2	OK
	Deflection		1/607	1/600	OK
Main Girder (G3)	Maximum Bending Stress		114.0 N/mm2	176 N/mm2	OK
	Maximum Shear Stress		21.6 N/mm2	105 N/mm2	OK
	Deflection		1/640	1/600	OK
Skin Plate	Bending Stress	Parcel: (1)	114.0 N/mm2	180 N/mm2	OK
		Parcels: (2)	103.0 N/mm2	180 N/mm2	OK
Stringer	Bending Stress	Parcel: (1)	20.4 N/mm2	115 N/mm2	OK
		Parcels: (2)	10.9 N/mm2	134 N/mm2	OK
	Shear Stress	Parcel: (1)	3.9 N/mm2	105 N/mm2	OK
		Parcels: (2)	2.7 N/mm2	105 N/mm2	OK
Bearing Plate	Bearing Stress		9.2 N/mm2	225 N/mm2	OK

1-2. Guide Frame

		Calculation Results	Tolerance	Evaluation
Strut	Concrete Bearing Stress	2.20 N/mm2	5.90 N/mm2	OK
	Concrete Shear Stress	0.26 N/mm2	0.40 N/mm2	OK
	Bending Stress on Strut	77.5 N/mm2	154 N/mm2	OK
	Shear Stress on Strut	66.9 N/mm2	105 N/mm2	OK
	Deflection of Strut	1/1550	1/600	OK
	Diagonal Axial Tensile Stress	100.7 N/mm2	180 N/mm2	OK
	Fixed Anchor Shear Stress	50.7 N/mm2	90 N/mm2	OK
	Tensile Stress of Anchor in Fixed Part	119.4 N/mm2	150 N/mm2	OK

Source: Study team



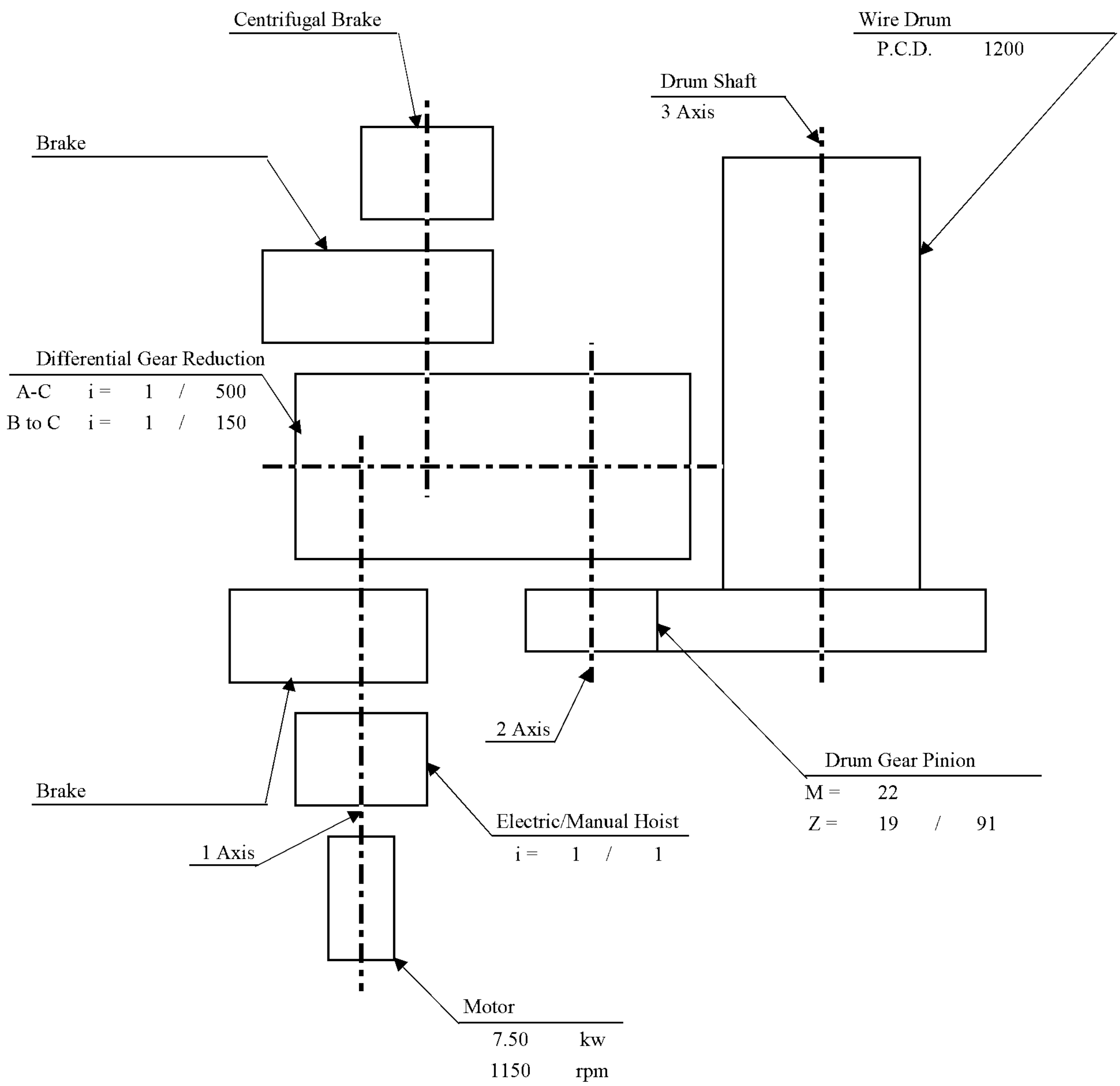
Source: Study team

Figure7.4.195 Section Shape (Stoplog)

Table7.4.238 Calculation Result of Hoist

			Calculation Results	Tolerance	Evaluation
Motor	Motor Output		6.05 kw	7.50 kw	OK
Hydraulic Push-up Brake	Safety Factor (During Electric Operation)		2.20	1.50	OK
Wire Rope	Safety Factor (From the Opening and Closing Load)		8.74	8.00	OK
	Tension from Motor Output (Maximum Torque)		250165 N	342225 N	OK
Wire Drum	Drum Diameter (Ratio to the Rope Diameter)		35.8	19.0	OK
Wire Sheave	Sheave Diameter (Ratio to the Rope Diameter)		19.7	17.0	OK
Wire Drum	Drum Plate Thickness		27.60 mm	33.25 mm	OK
Rope Presser	Allowable Tightening Force of the Tightening Bolt		45865 N	61560 N	OK
	Bending Stress of the Rope Pressing Metal		75.8 N/mm2	98.0 N/mm2	OK
Hydraulic push-up brake	Torque (Self Weight Lowering Device)		307 N. m	392 N. m	OK
Gear	Safety Factor at Rated Torque	Pinion	11.4	5.00	OK
		Drum Gear	13.6	5.00	OK
	Safety Factor at Maximum Torque	Pinion	3.50	1.00	OK
		Drum Gear	4.15	1.00	OK
	Surface Pressure Strength	Pinion/Drum Gear	1.07	1.00	OK
Fleet Angle	Fully Closed		1.81 °	4.00 °	OK
	Upper Limit		0.98 °	4.00 °	OK

Source: Study team



Source: Study team

Figure7.4.196 schematic Arrangement

7.4.3.3 Control Room Layout

(1) Control Room Components

Equipment relating to hoisting device is arranged in the operation room. Cainta Floodgate is in the 1M1D type, with the hoist body located in the end operating room. Components of equipment placed in the end and center operation room is shown in **Table7.4.239**.

Table7.4.239 control room components

End Operating Room	Central Control Room
Gate Hoisting Device	Gate Hoisting Device
・Wire Drum ・Wire Sheave ・Wire Terminal Device ・Motor ・Electric Manual Switching Device ・Centrifugal Brake ・Mew Lifter Brake ・Gear Reducer ・Drum Gear And Pinion Gear ・Dogging Device ・Limiting Switch ・Opening Meter ・Emergency Upper Limit Detection Device ・Machine Side Control Panel	・Wire Sheave ・Wire Terminal Device ・Dogging Device ・Limiting Switch

Source: Study team

(2) Control Room Layout

The layout of the control room shall be such that necessary space is secured in consideration of inspection and maintenance work based on the "Technical Specification for Dams and Weirs in Japan (Draft)". The layout of each console is shown in **Figure7.4.198** to **Figure7.4.199**.

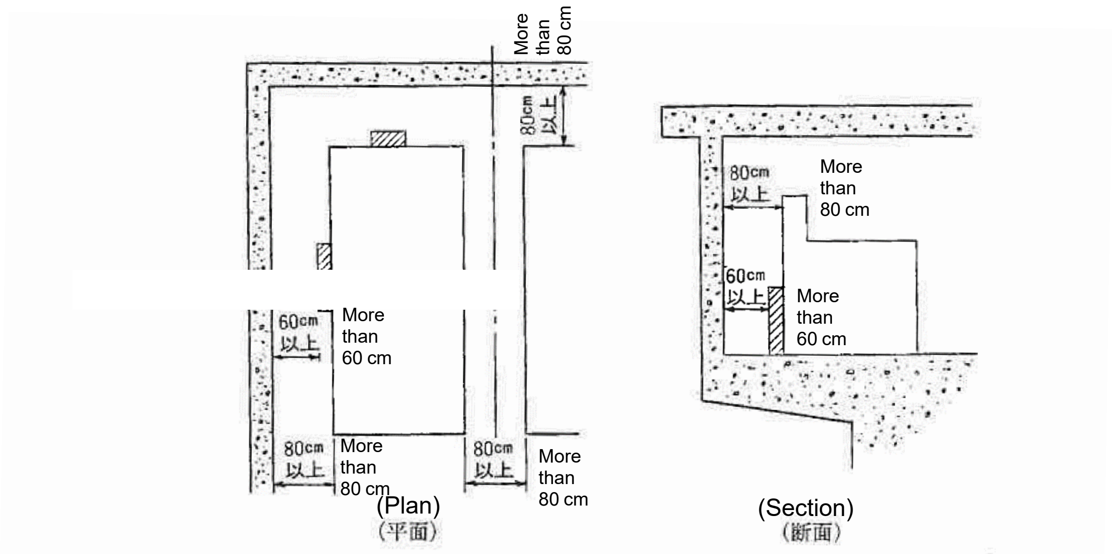
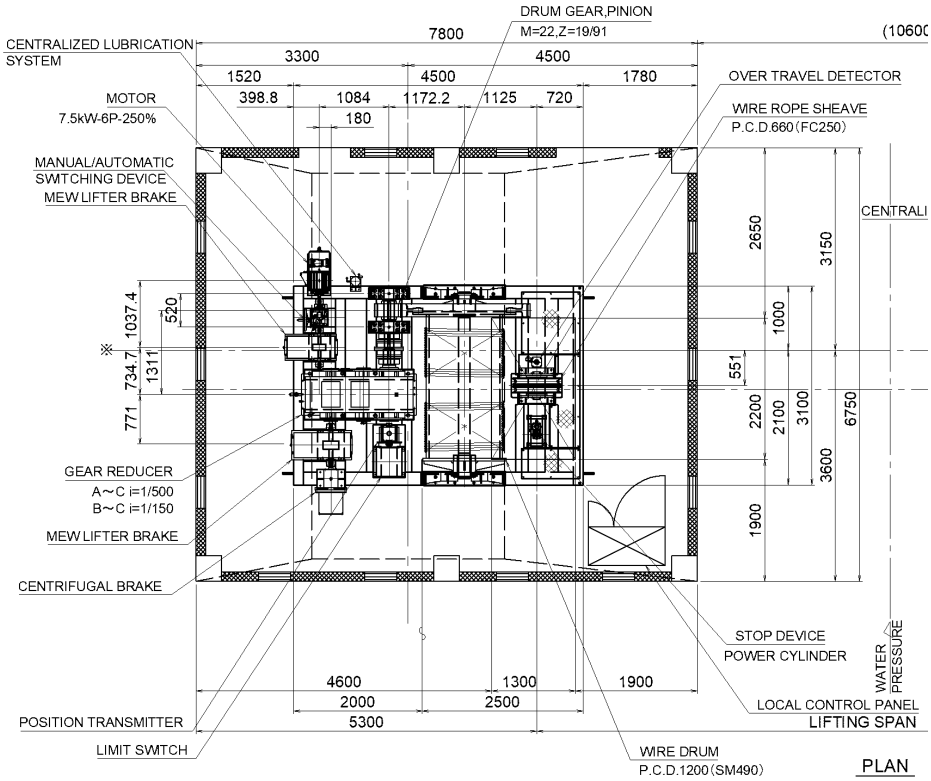


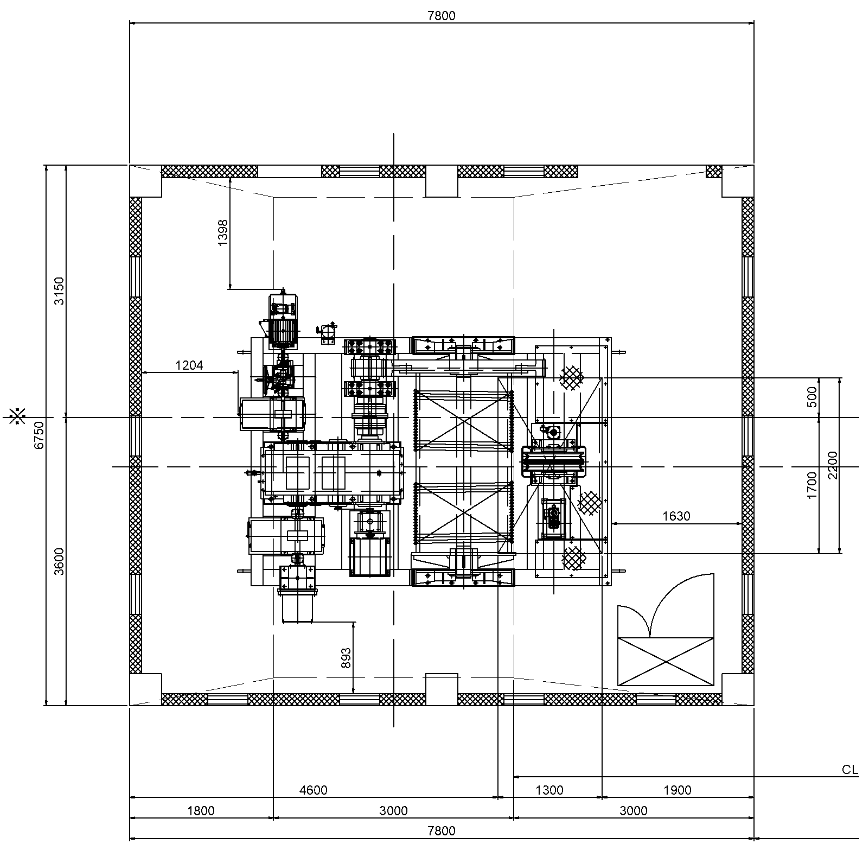
図4.4.2-1 開閉装置の端部から壁までの間隔

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

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

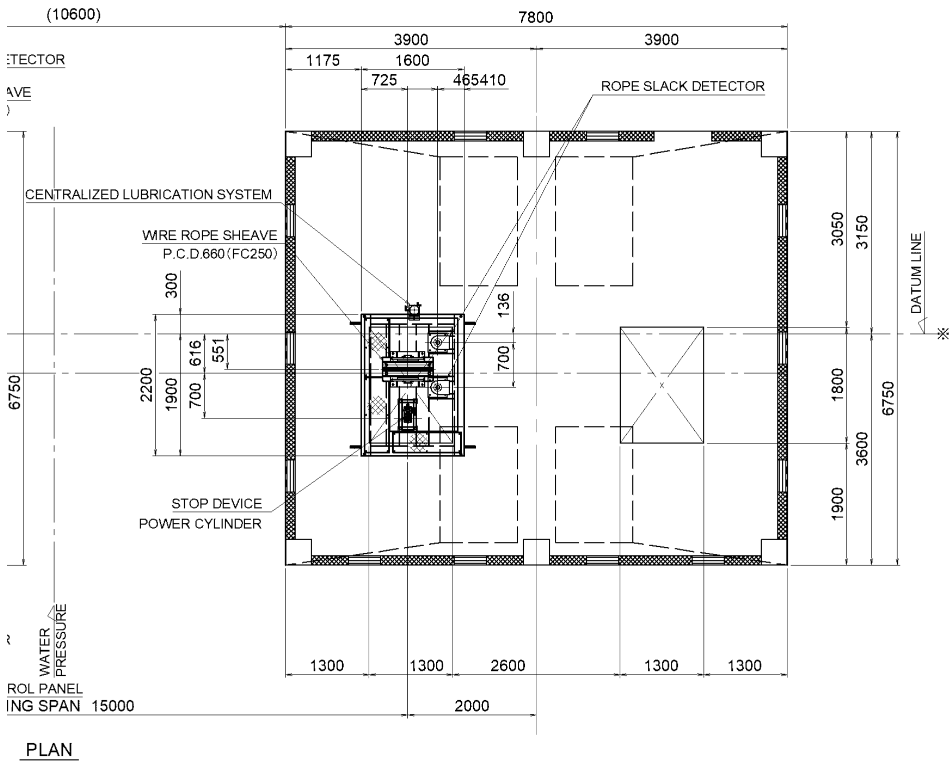
Figure7.4.197 Space to be Secured in Operating Room Space

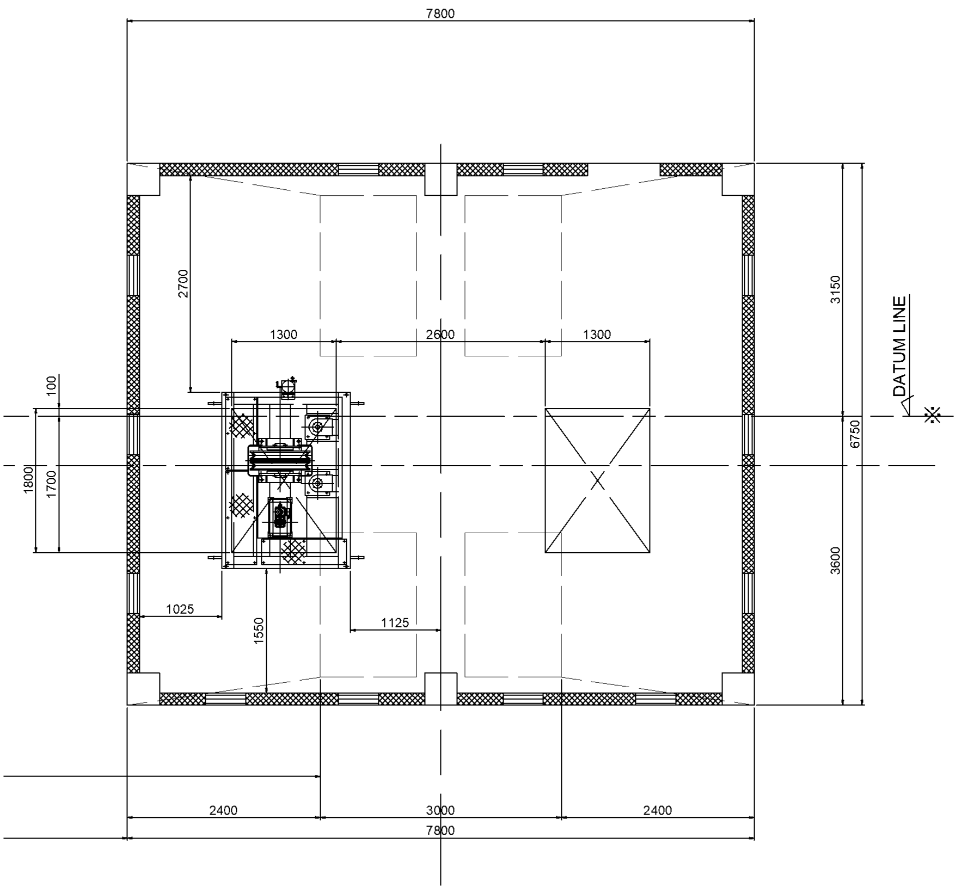




Source: Study team

Figure7.4.198 End Operation Room Layout





Source: Study team

Figure7.4.199 Central Control Room Layout Drawing

7.4.3.4 Specifications of the Gate Facility

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	
Standard	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)	

(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
	Rest Hook	Yes
Normal Lift	Normal H1	9.650 m
	Hibernate H2	9.950 m
Opening And Closing Speed	When Using An Electric Motor	0.30 About m/min
	During Self-Weight Descent	1.00 About m/min
Wire Rope	JIS 6 × 37 G type plating	
Power	200 VAC - 50 Hz	
Standard	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)	

(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
Standard	Technical Specification for Dams and Weirs in Japan (Draft) Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,

(4) Stoplog Facility

Gate Type	Plate Girder Structure Steel Slide (Strut Type Angle Drop)	
Clear Span (Gate Leaf Width)	Clear Span 16.00 m (4.0 m x 4 Leaves)	
Effective Height (Gate Leaf Height)	Effective Height: 5.70 m (1.9 m x 3 Leaves)	
Number of Gates	12 leaves (One gate)	
Design Depth	(Floodway Side)	EL + 14.853 (DFL)
	(Tributary Side)	EL + 10.100 (OWL in Tributary River)
Invert Elevation	(Plan)	EL + 8.750
Water Sealing System	Rear Three-Way Rubber Watertight	
Opening And Closing Method	Lifting By Crane	
Standard	Technical Specification for Dams and Weirs in Japan (Draft) Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,	

7.4.4 Building Facility Design

In **sub-section 7.6 Detailed Design,** building facilities, building machinery and equipment, and building electrical equipment of 3 facilities is stated.

7.4.5 Design of Information Facilities

7.4.5.1 Design of Instrumentation, Alarm Monitoring, and Remote Monitoring and Control Equipment

(1) Organizing Design Conditions

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

Table7.4.240 Design Condition List

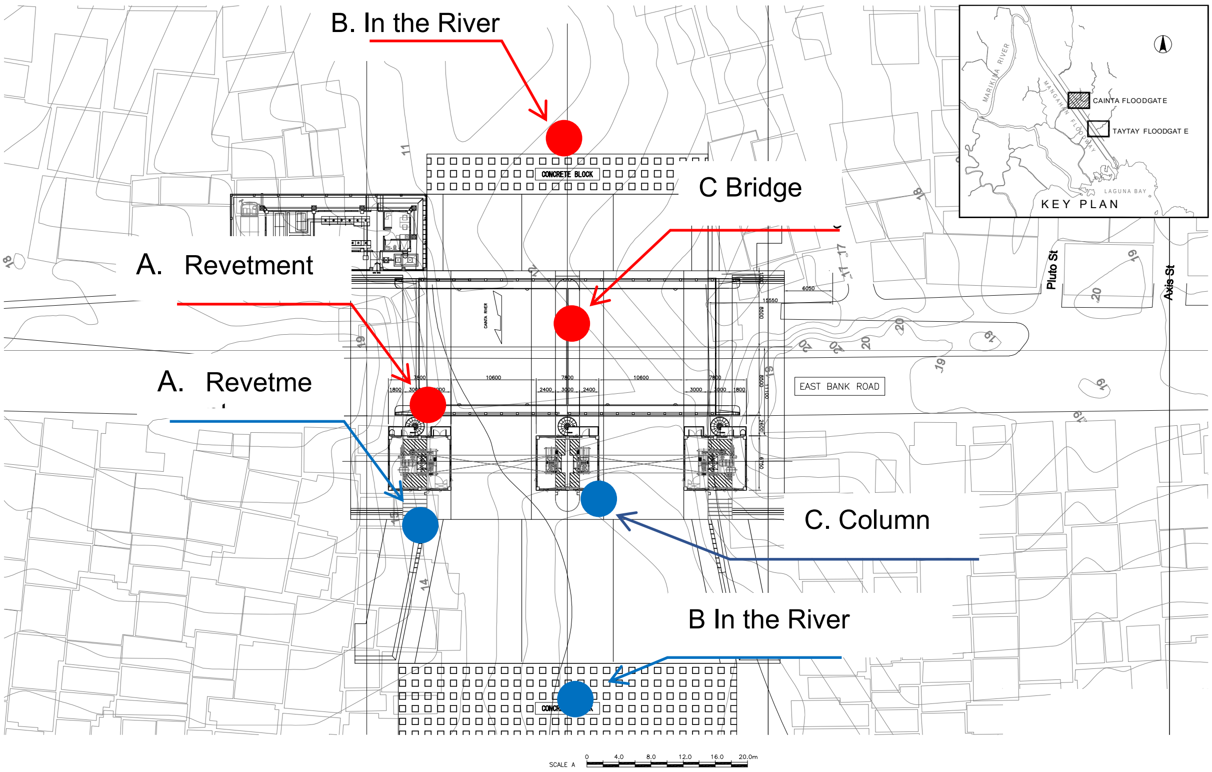
Facility	Equipment Classification	Installation Equipment	Design Conditions and Considerations	Installation Quantity
Cainta Floodgate	Instrumentation Facility	Water Level Gage	Measure the water level upstream and downstream of the gate for accurate gate operation. The water level shall be measured at 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

(2) Instrumentation Facility (Water Level Observation Equipment) Design

1) Study on the Layout of Water Level Gauge

Instrumentation equipment (water level observation equipment) installed in the Cainta Floodgate is to carry out the accurate gate operation during floods. In order to measure the water level according to the gate operation situation, the installation in the upstream (land side) and downstream (floodway side) of the gate is considered. Possible locations for the installation of water level gauges at Cainta Floodgate are set as shown in **Figure7.4.200**.



Source: Study team

Figure7.4.200 Alternative for Water Gauge Installation Position

About each alternative shown in **Figure7.4.200**, the following items are compared and examined. As a result of the comparison, the installation to the revetment of both upstream and downstream is selected.

[items to be compared]

1. Outline of Installation Location
2. Applicable Method of Measuring Water Level
3. Application of Observed Water Level to Facility Operation
4. Workability
5. Maintainability

Table7.4.241 Study on the Position For Installation of Water Level Gauges (Upstream of Upper Cainta Floodgate: Land Side)

Installation Position	A Revetment	B In the River	C Bridge
Outline of Installation Location	A water level gauge is installed in the upstream revetment of the Cainta floodgate to measure the water level.	A water level gauge is installed near the center of the river channel (center of flow) to measure the water level.	A water level gauge is installed on the bridge to measure the water level upstream of the gate.
Applicable Type of Measuring Water Level	Float Type Reed Switch Type Hydraulic (Quartz Hydraulic System) Ultrasonic And Radio Wave Type	Reed Switch Type Hydraulic (Quartz Hydraulic System)	Ultrasonic And Radio Wave Type
Application of Observed Water Level to Facility Operation	△ When the gate opening is different, it is not suitable for the facility operation, because accurate water level cannot be measured.	◎ Since it is observed near the center of the river channel (center of flow), it is hardly affected by gate operation.	△ When the gate opening is different, it is not suitable for the facility operation, because accurate water level cannot be measured.
Workability	◎ Its workability is good because it is installed on the revetment. However, depending on the water-level measurement method, it is necessary to install an observation well (float type) or a measuring column (reed switch type).	△ Since it is installed in a river channel, it is by a barge, and the workability is inferior. It is necessary to install multiple H-steels for water level gauge and refuse protection.	○ It can be installed with gate installation. installation is relatively easy.
Maintainability	◎ Maintenance from the land is possible, and the maintainability is good.	△ It is necessary to deal with driftwood and dust in case of flood.	○ Maintenance is possible from the bridge. However, an aerial work platform is required, and it is necessary to consider the effect on vehicle traffic.
Evaluation	Good Workability And Maintainability, And the Main Plan is Adopted.		

Source: Study team

Table7.4.242 Study on the Position for installation of water level gauges (Downstream of Cainta Floodgate: Floodway Side)

Installation Position	A Revetment Section	B Fluvial Part	C Gatepost
Outline Of Installation Location	A water level gauge is installed at the downstream revetment of the Cainta floodgate to measure the water level.	A water level gauge is installed near the center of the river channel (center of flow) to measure the water level.	A water level gauge is installed at the center column of the Cainta Floodgate to measure the water level downstream of the gate.
Applicable Method Of Measuring Water Level	Float Type Reed Switch Type Hydraulic (Quartz Hydraulic System) Ultrasonic And Radio Wave Type	Reed Switch Type Hydraulic (Quartz Hydraulic System)	Ultrasonic And Radio Wave Type
Application Of Observed Water Level To Facility Operation	△ When the gate opening is different, it is not suitable for the facility operation, because accurate water level cannot be measured.	◎ Since it is observed near the center of the river channel (center of flow), it is hardly affected by gate operation.	△ When the opening of the gate is different, it is not suitable for the facility operation, because accurate water level cannot be measured.
Workability	◎	△	○

Installation Position	A Revetment Section	B Fluvial Part	C Gatepost
	Its workability is good because it is installed on the revetment. However, depending on the water-level measurement method, it is necessary to install an observation well (float type) or a measuring column (reed switch type).	Since it is installed in a river channel, it is constructed on a ship by a barge, and the workability is inferior. It is necessary to install multiple H-steels for water level gauge and refuse protection.	It can be installed with the gate installation. The construction is comparatively easy, because it is installed in the external wall of the local control house.
Maintainability	◎	△	○
	Maintenance from the land is possible, and the maintainability is good.	It is necessary to deal with driftwood and dust in case of flood.	Maintenance from the local control house is possible, and the maintainability is good.
Evaluation	Good workability and maintainability, and the main plan is adopted.		

Source: Study team

2) Selection of Water Level Observation Method

As for the water level observation system, a light feed type quartz water pressure system is adopted as well as MCGS.

3) Instrumentation Configuration

The instrumentation structure is the same as MCGS.

4) Equipment Specification

The equipment specification is the same as MCGS.

(3) Alarm Facility Design

1) Siren

(a) Selection of siren system structure

As well as MCGS, inverter siren is adopted for the siren equipment structure.

(b) Determination of Inverter Siren Output

The siren installed at the Cainta Floodgate gate shall be for the purpose of making an alarm to notify when the gate is being operated. The Cainta Floodgate has a short span length, and its area to be notified by siren may be narrower than that of MCGS. The minimum output of the inverter siren installed at the Cainta floodgate is set to 0.75 kW.

Table7.4.243 Siren and Sound Distance (standard value)

Siren Output	Acoustic Distance (Radius)
0.75 kW	Approximately 500 m
2.2 kW	Approximately 800 m
3.7 kW	Approximately 1,100 m
5.5 kW	Approximately 1,400 m

Source: Study team

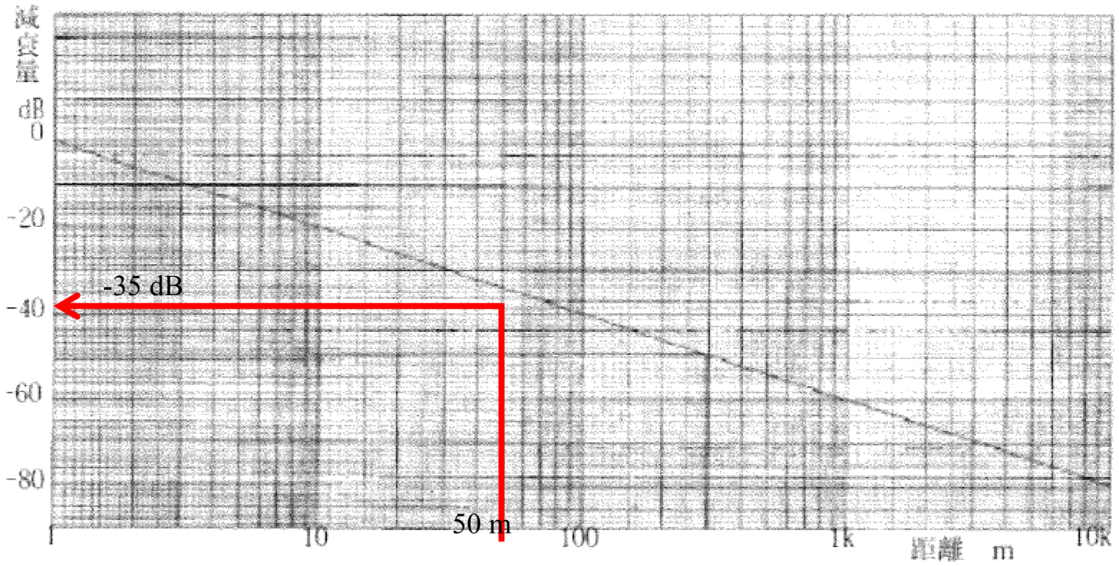
2) Loudspeaker Microphone

(a) Speakers

The loudspeakers to be installed at the Cainta Floodgate gate are for the purpose of making the alarm to notify the surrounding people when the gate is being operated. Therefore, the output of the loudspeakers is determined by setting the arrival distance at about 50 m so that the alarm sound reaches both banks of the Cainta River.

In accordance with **Figure7.4.201**, the attenuation corresponding to the arrival distance of 50 m is read out as -35 dB, so that the output sound pressure level which requires 113 dB (= 78 dB + 35 dB) is obtained by adding the attenuation of 35 dB to the target arrival level of 78 dB.

When the speaker output is selected from the output sound pressure level, it is equivalent to 25 W.



Source: Telecommunication Facilities Design Guidelines (Communications)

Figure7.4.201 Attenuation Due to Sound Distance

Table7.4.244 Speaker Output Sound Pressure Level (1 m Value)

Speaker Input	Output Sound Pressure Level
1W	110 dB
25W	124 dB
50W	127 dB
100 W	130 dB

Source: Study team

(b) Audio Amplifier

The audio amplifier is used for broadcasting by voice or broadcasting of the proceedings and is mounted on the alarm device. The rated output of the audio amplifier is 100 W as standard, and if the output is to be increased, an audio amplifier is added in units of 100 W.

(c) Sound Collection Microphone

One microphone is installed for each speaker.

(d) Warning Light

The warning lamp installed in the Cainta Floodgate is to add visual information in addition to the alarm by sound of siren and speaker during the gate operation. The light source of the alarm light shall be a LED system of long life and power saving type, and as a blinking system, a reflection mirror rotating system, or a lamp blinking system shall be used.

(e) Operating Equipment

As for the operation equipment, a display table system is selected like MCGS.

3) Study on Alarm System Configuration

The alarm equipment composition is similar to MCGS.

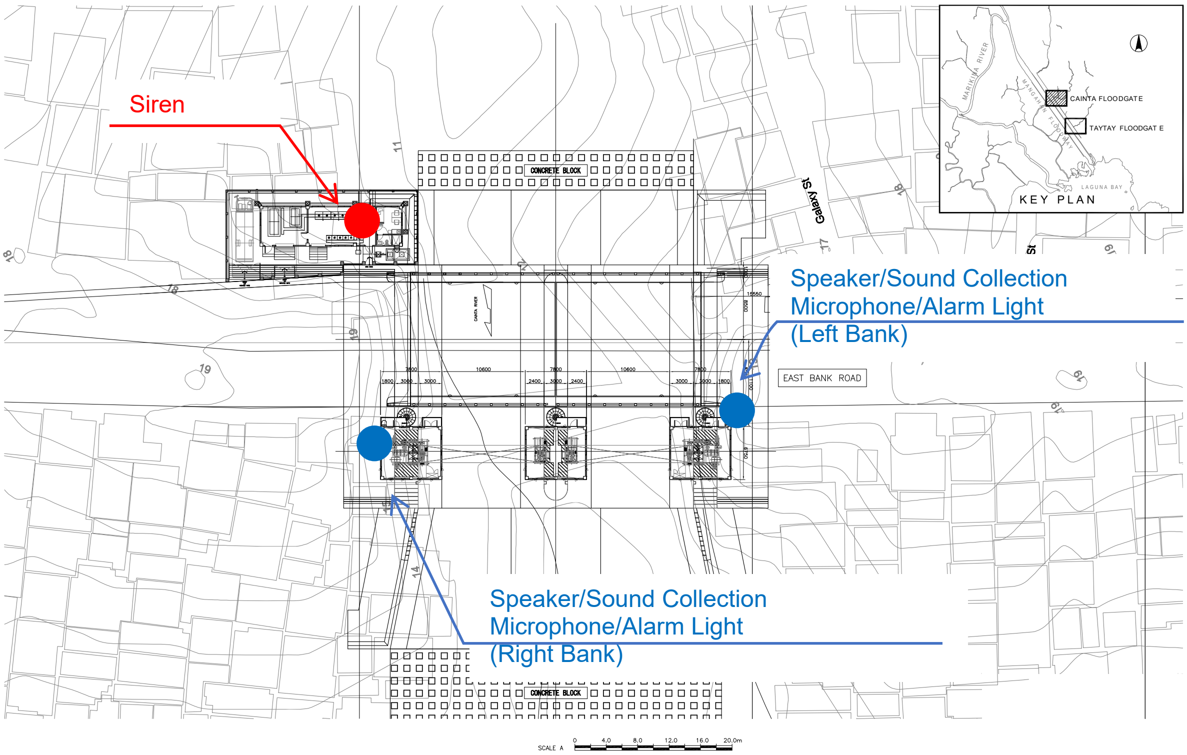
4) Consideration of Alarm Equipment Layout

The alarm equipment installed at the Cainta floodgate shall be arranged according to the following concept.

Table7.4.245 Arrangement of alarm equipment (Cainta Floodgate)

Alarm Facility	Placement Position	Quantity	Placement Criteria
Siren	Operating Room Roof	1	The sound range of the siren is about 500 m, and the alarm sound can be made to notify within a range of 500 m radius by the installation in the operation room of Cainta Floodgate.
Speakers	Machin Side Operation Room Left and Right Bank	2 (Right And Left Bank)	Speakers, sound collection microphones, and warning lights should be placed in the left and right bank side operation rooms so that they can blow and light on both banks of the Cainta River.
Sound Collection Microphone		2 (Right And Left Bank)	
Warning Light		2 (Right And Left Bank)	

Source: Study team



Source: Study team

Figure7.4.202 Alarm Facilities Layout

5) Equipment Specification

The equipment specification is the same as MCGS. However, the inverter siren output is 0.75 kW and the speaker output are 25 kW.

(4) Design of Monitoring Equipment (CCTV Camera)

1) Target to be Monitored

The camera equipment installed in the Cainta floodgate is aiming at quick and accurate situation grasp and facility operation during floods by remote monitoring of the field situation.

In this design, in addition to monitoring gate conditions (facility monitoring), grasping river conditions during floods (spatial monitoring) is carried out, and monitoring objects are set as follows.

Table7.4.246 Target to be Monitored

Monitored	Monitoring Classification	Concept of Monitoring
Gate Facility	Facility Monitoring	The angle of view is fixed toward the gate, and the opening and closing situation is monitored remotely.
Condition of The River	Spatial Monitoring	In the angle of view of river upstream (land side) and downstream (Floodway side), the situation is widely grasped. The monitoring direction can be arbitrarily changed by turning and zooming.

Source: Study team

2) Monitoring System

In this design, the HD camera is applied considering the certainty of manufacturer's guarantee and procurement of replacement parts, and the technological trend and market trend of the camera.

3) Selection of the Monitoring Method

As well as MCGS, HD simple type IP camera equipment is adopted.

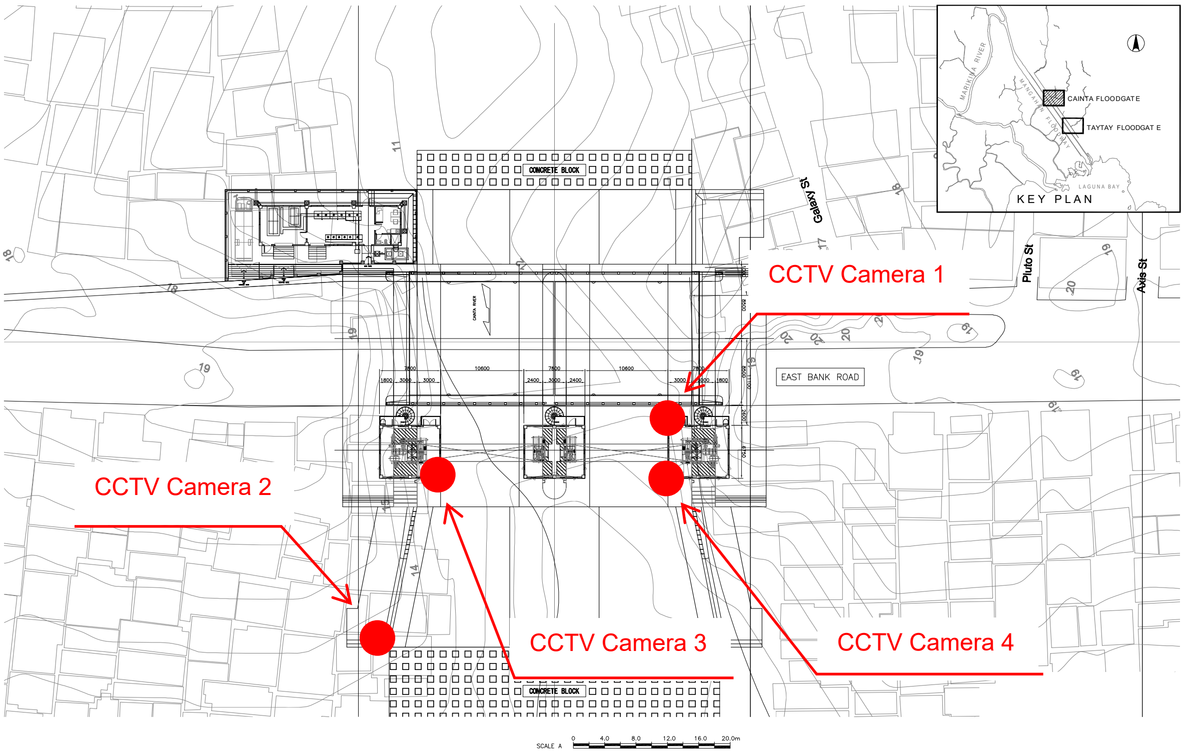
4) Study on Monitoring Equipment Layout

The monitoring equipment installed at the Cainta Floodgate gate would be arranged according to the following concept. The selected CCTV camera is a turning type, and it is possible to switch the monitoring object (gate ⇔ space) by turning the camera.

Table7.4.247 Arrangement of the Monitoring Facilities (Cainta Floodgate)

Monitored	Monitoring Classification	Quantity	Placement Criteria
Gate	Facility Monitoring	2	Install in the left and right bank operation rooms
Upstream of Cainta Floodgate (Land Side)	Spatial Monitoring	1	Install in left bank operation room
Downstream of Cainta Floodgate (Floodway Side)	Spatial Monitoring	1	Install on the right bank revetment at the confluence of the floodway

Source: Study team



Source: Study team

Figure7.4.203 Camera Equipment Layout

5) Configuration of Monitoring Equipment

The configuration of monitoring equipment is the same as MCGS.

6) Equipment Specification

The equipment specification is the same as MCGS.

(5) Remote Monitoring and Control Facility

The remote monitoring and control facilities will be stated in **Sub-section 7.3.6.10**.

7.4.5.2 Electrical Equipment (Emergency Power Supply) Design

(1) Design conditions

1) Operating Time of the Generator

The 1 time of opening (closing) time in Cainta Floodgate can be calculated about 20 minutes with rounding up of 17.3 minutes considering the opening/closing distance of 5.19 m and the opening/closing speed of 0.3 m/min.

20 minutes × (1 open + 1 closed) × 3 days = 120 minutes

In accordance with the above calculation, total fuel tank capacity for 2 hours is secured for gate opening/closing power. On the other hand, the generator for the control equipment shall secure fuel for 72 hours.

2) Creating a Load List

The equipment load in Cainta Floodgate is shown in **Table7.4.248**. The equipment is classified into generators for gate opening and closing and generators for control facilities.

Table7.4.248 Load List

Cainta

Category	Lload name	Three-phase load	Single-phase load	Units	Number of units	Subtotal
For gate hoisting power	Hoist motor	7.50		kW	2	15.00
	Machine side control panel control power supply		1.50	kVA	2	3.00
For control equipment	Distribution board for local control house 1 Control facility					5.29
	Speakers		0.02	kVA	1	0.02
	Sound collection microphone		0.01	kVA	1	0.01
	Warning light		0.03	kVA	1	0.03
	CCTV		0.15	kVA	2	0.30
	Control panel		2.77	kVA	1	2.77
	TC extension unit		0.11	kVA	1	0.11
	MC		0.01	kVA	5	0.05
	Backup		2.00	kVA	1	2.00
	Distribution board for local control house 3 Control facility					5.31
	Water level gage		0.01	kVA	2	0.02
	Speakers		0.02	kVA	1	0.02
	Sound collection microphone		0.01	kVA	1	0.01
	Warning light		0.03	kVA	1	0.03
	CCTV		0.15	kVA	2	0.30
	Control panel		2.77	kVA	1	2.77
	TC extension unit		0.11	kVA	1	0.11
	MC		0.01	kVA	5	0.05
	Backup		2.00	kVA	1	2.00
	Distribution board for generator house control equipment					9.35
	Optical receiver		0.01	kVA	2	0.02
	MC		0.01	kVA	21	0.21
	TC master unit (Include PLC)		0.22	kVA	1	0.22
	Operation switch panel		0.25	kVA	1	0.25
	L3SW		0.05	kVA	1	0.05
	Siren		1.60	kVA	1	1.60
	Generator board accessory power supply (150 kVA)		2.50	kVA	1	2.50
	Generator board accessory power supply (50 kVA)		2.50	kVA	1	2.50
	Backup		2.00	kVA	1	2.00
	Generator House Construction Panel (DB1)					7.61
	LIGHTINGS		1.30	kVA	1	1.30
	LIGHTINGS		0.95	kVA	1	0.95
	LIGHTINGS		1.41	kVA	1	1.41
	C.O., 4 UNITS		0.68	kVA	1	0.68
	C.O., 4 UNITS		0.68	kVA	1	0.68
	FACP		0.95	kVA	1	0.95
	ACCP&FCU, 1.5 HP		1.14	kVA	1	1.14
	SPARE		0.50	kVA	1	0.50
	Local Control House 1 Architectural distribution board (DB2)					1.96
	LIGHTINGS		1.28	kVA	1	1.28
	CONVENIENCE OUTLET, 1UNIT		0.18	kVA	1	0.18
	SPARE		0.50	kVA	1	0.50
	Local Control House 2 Architectural distribution board (DB3)					1.96
	LIGHTINGS		1.28	kVA	1	1.28
	CONVENIENCE OUTLET, 1UNIT		0.18	kVA	1	0.18
	SPARE		0.50	kVA	1	0.50
	Local Control House 3 Architectural distribution board (DB4)					1.96
	LIGHTINGS		1.28	kVA	1	1.28
	CONVENIENCE OUTLET, 1UNIT		0.18	kVA	1	0.18
	SPARE		0.50	kVA	1	0.50
						33.44

Source: Study team

(2) Design of Generator and Motor

1) Calculation of Generator Output

The output of the power generation equipment shall be calculated by taking into consideration the output of the load, the type and starting method, the presence or absence of a fire-fighting load, the type of engine, etc.

In this study, the output is calculated according to the "Output Calculation Software For Private Power Generation Facilities" based on the Design Guideline for Telecommunications Facilities (Electric).

The calculation method indicated in Design Guideline for Telecommunications Facilities (Electric) and the calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**. The calculation results are as follows.

Table7.4.249 Generator Calculation Result

For Gate Opening and Closing	For Control Equipment
Generator capacity 125.0 kVA Motor output 115.8 kW	Generator capacity 46.4 kVA Motor output 45.2 kW

Source: Study team

2) Selection of Motor

Based on the calculation results of "Output calculation software for private power generation facilities", the most recent generator efficiency is chosen from generator efficiency table shown in Design Guideline for Telecommunications Facilities (Electric). The generator efficiency is indicated in **Table7.4.251**.

Table7.4.250 Power Generating Capacity And Motor Output of the Nearest High-Order Generator

Gate Opening and Closing	For Control Equipment
Generator capacity 150 kVA Motor output 138 kW	Generator capacity 50.0 kVA Motor output 48.6 kW

Source: Study team

Table7.4.251 Generator Efficiency Table

Generator Output		Conventional Efficiency	Motor Output	Generator Output		Conventional Efficiency	Motor Output
(kVA)	(kw)(power factor:0.8)	ηG(%)	(kW)	(kVA)	(kw)(power factor:0.8)	ηG(%)	(kW)
5	4	74	5.5	200	160	87.9	182
10	8	75	10.7	250	200	88.9	225
15	12	76	15.8	300	240	89.5	269
20	16	77	20.8	375	300	90.3	333
37.5	30	80.7	37.2	500	400	91.0	440
50	40	82.3	48.6	625	500	91.7	546
62.5	50	82.4	60.0	750	600	92.1	652
75	60	84.3	71.2	875	700	92.3	759
100	90	85.6	93.6	1000	800	92.6	864
125	100	86.4	116	1250	1000	93.0	1076
150	120	87.0	138	1500	1200	93.3	1287

Note: The values with is not accordance with JEM-1354.*

Source: Design Guideline for Telecommunications Facilities (Electric)

(a) Selection of Generator

The basic conditions of the generator are as shown in the table below, based on the Design Guideline for Telecommunications Facilities (Electric).

Table7.4.252 Basic Requirement for Generators

Design Guideline	Applied To This Design
The generator shall be a horizontal synchronous generator.	Use a horizontal synchronous generator
Excitation type shall be brushless or static.	Use brushless or static excitation
For diesel engines, the protection type shall be JIS C 4034 (IP 20) or drip-proof type (IP 22 S). The gas turbine shall be of the protective type (IP 20).	Diesel engines, protected type (IP 20) or protected drip-proof type (IP 22 S)
The heat resistance class of insulation shall be class E or higher for low pressure and class B or higher for high pressure.	Class E or higher for low-pressure generators
The standard number of poles is 4, and a small-capacity machine (100 kVA or less) may have 2 poles.	Use the standard four poles
The generator rated voltage shall be as follows as a standard. 150 kVA or less 200 V/220 V (Production possible up to 200 kVA) 150 ~ 400 kVA 400 V/440 V (Production possible from 50 kVA to 500 kVA) 250 kVA or higher 3300 V/6600 V	The generator rated voltage shall be the standard.

Source: Study team

(b) Selection of Motor

The basic conditions of the motor are as shown in the table below based on the Design Guideline for Telecommunications Facilities (Electric). As described in the design guideline, a comparison between a diesel engine and a gas turbine is shown in **Table7.4.254**. As shown in **Table7.4.254**, the gas turbine system is not adopted unless the gas turbine is particularly advantageous.

Table7.4.253 Basic Requirement for Motors

Design Guideline	Applied To This Design
Diesel engines with high fuel consumption rates shall be used in principle. However, if a gas turbine is particularly advantageous under certain conditions, the selection of a gas turbine should be considered.	The standard diesel engine is used.
The starting system shall be electric or pneumatic. 1000 kVA or less: electric system Exceeding 1000 kVA: Electric or air system	The starting system is an electric system because it is 1000 kVA or less.
Starter The electric system is a system in which the cell motor is driven by a DC power supply. The storage battery is a control valve type stationary lead acid battery or a small control valve type lead acid battery.	Use a cell motor drive system. The storage battery is a control valve type stationary lead acid battery or a small control valve type lead acid battery.
The capacity of the storage battery shall be such that it can be started three times or more.	Capacity that can be started three times or more.
The standard cooling system for diesel engines is the radiator type.	Standard radiator type.

Source: Study team

Table7.4.254 Comparison of Diesel Engines and Gas Turbines

Item \ Motor	Diesel Engine	Gas Turbine
Operating Principle	Thermal energy of intermittent combustion and explosion combustion gas is once converted into reciprocating motion of piston, which is converted into rotary motion by crankshaft. (Reciprocating motion → Rotating motion)	Thermal energy of continuously burning combustion gas is directly converted into rotational motion by a turbine (rotational motion)
Output	The output is limited by the suction air temperature.	When the suction air temperature is high, the output is limited because the amount of air compressed by the compressor is reduced.
Fuel Consumption Rate	230 ~ 310 g/kWh	520 ~ 680 g/kWh
Spent Fuel	Light oil and heavy oil A	Kerosene, light oil, and heavy oil A
Excess Air Ratio	2.0~3.0	3.5~4.0
Instantaneous Frequency Variation	± 10% or less	± 5% or less in the case of uniaxial type ± 10% or less in the case of biaxial type
Frequency Dollop	Not more than 5%	Not more than 5%

Item \ Motor	Diesel Engine	Gas Turbine
Instantaneous Load Input Rate	In the case of no supercharge: 100% input is possible. In case of turbocharger: 70% input is possible. For high turbochargers: 50% input possible	In the case of uniaxial type: 100% input is possible. In the case of 2-shaft type: 70% input is possible.
Starting Time	5 to 40 seconds	20 － 40 seconds
Light Load Operation	It is difficult to obtain complete combustion of fuel. The amount of increase in lubricating oil increases and carbon adheres to the combustion chamber or the exhaust turbine (supercharger).	No problem.
Amount of NOx, Etc.	300 ~ 1000 ppm (O2 concentration of 13%)	20 ~ 150 ppm (O2 concentration of 16%)
Vibration	Vibration is generated due to the reciprocating engine, but it can be reduced from the load of the vibration isolator.	Because it is a rotating engine, there is no need for vibration isolators.
Volume And Mass	Large number of parts and heavy mass.	There are few mechanical parts, and the size and mass are small and light.
Installation	The installation area is large. I need a foundation. Intake/exhaust treatment equipment is small.	The installation area is small. The foundation is small and good. Intake and exhaust air treatment equipment becomes larger.
Cooling Water	Needed Radiators do not need to be replenished at all times.	Not required (For air cooling type)
Serviceability	There are few daily inspection items. (Equivalent to automobiles and construction equipment) Overhaul is basically possible on site. (Some exceptions) Replacement parts are relatively inexpensive	There are few daily inspection items. Overhaul must be carried out at the factory. Spare parts are more expensive than diesel
Response in the Event of a Failure	Depending on the severity of the failure, most can be recovered locally. If there is no major damage to the engine, it can be restored in one to three days.	Failures resulting from replacement of parts can be recovered at the site, but failures of the gas turbine itself require bringing the gas turbine into the factory. It takes about 3 days to 1 week for parts replacement and about 1 ~ 2 weeks for parts brought into the factory.
Extension of Operating Time Due to Load Limitation	Since the fuel consumption is almost proportional to the load, if the load is reduced to half of the rated output, the fuel consumption is reduced to half, and the operation time is doubled.	Since 50 ~ 60% of the fuel at the rated output is consumed even at no load, the fuel consumption is reduced by only about 20% even if the load is reduced to half of the rated output, so the effect of extending the operating time is small.

Source: Study team

3) Examination of Generator Specifications

(a) Electrical System Planning

From the calculation results of the load connected to the generator and the generator output and motor output, a single-wire connection diagram is prepared. The single wire diagram is shown in **Figure7.4.204**

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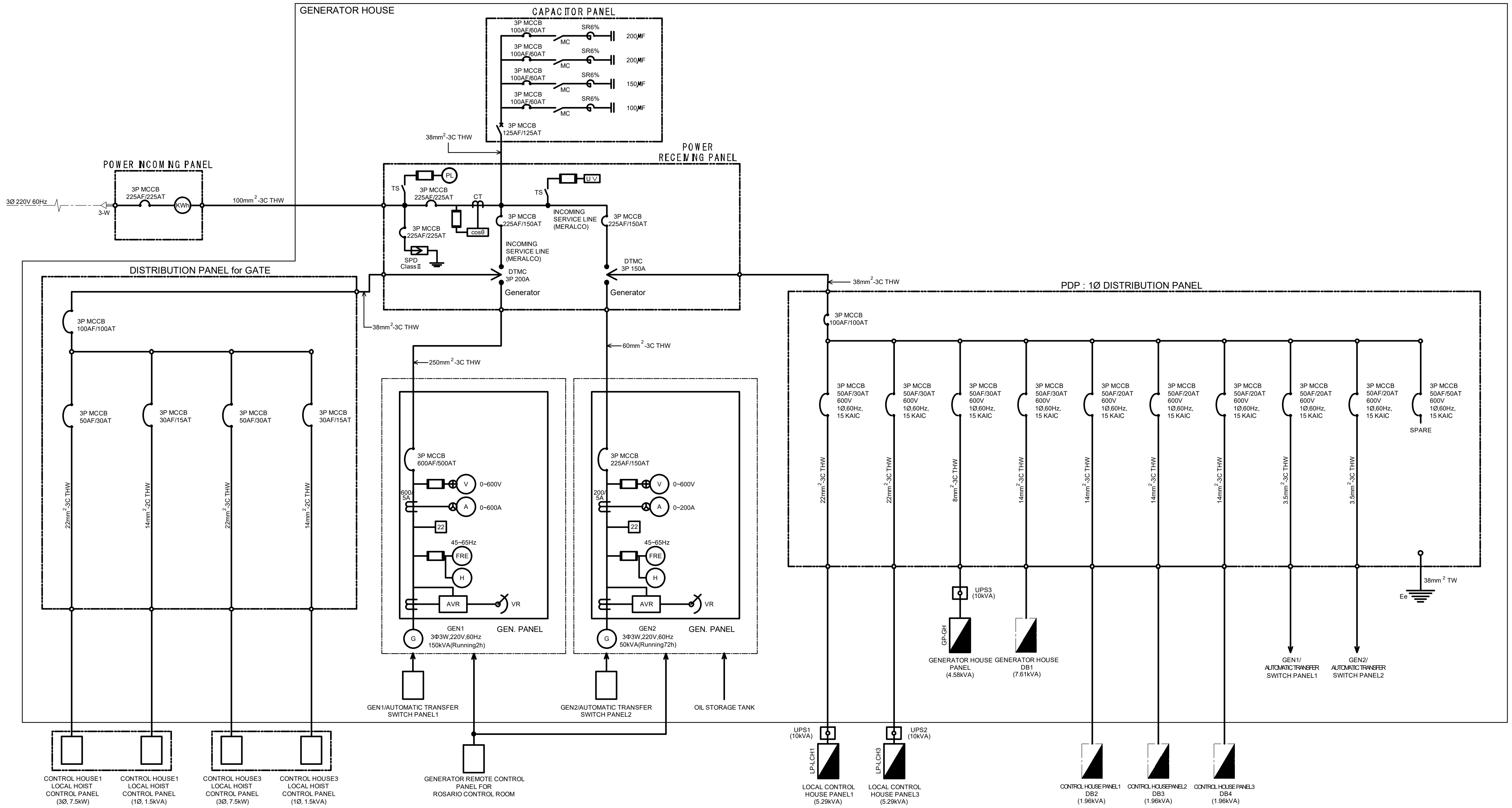


LEGEND

PAS :Pole mounted Air insulated Switch
PC :Primary Cutout
LA :Lightning Arrester
VCT :Voltage Current and Transformer
DS :Disconnecting Switch
DS TW :Hyperbolic switch
DSMC :Bipolar magnetic contactor
VCB :Vacuum insulated Circuit Breaker
VT :Voltage Transformars
CT :Current Transformars
ZCT :Zero-Phase-sequence Current Transformars
VTT :Voltage Testing Teminals
CTT :Current Test Terminal
TR :Transformer
MC :Electromagnetic Contactor
MCCB :Molded-Case Circuit Breakers
ZPD :Zero Phase potential Device
ELCB :Earth Leakage Circuit Breakers
EF :Enclosed Fuse
U=O :No-voltage relay
:Incoming Service Line (MERALCO)
:Fuse

A :AC Ammeter
V :AC Voltmeter
TS :Toggle switch
G :Generator
C :Capacitor
DEG :Diesel Generator
A :Ammeter
V :Voltmeter
W :Wattmeter
Hz :Frequency meter
kWh :Watt-hour meter
cosθ :Power-factor meter
TD :transducer
U< :Undervoltage Relays
I> :Over current relay
I≑> :Ground relay
I≑> :Ground direction relay
θ :Temperature relay
AVR :Automatic Voltage Regulater
SR :Series Reactor
⊕ :Voltmeter Change-over Switche

:Ammeter Change-over Switches
:Electric Panel, name as indicated
:Electric Panel, separate equipment
:Load Break Switches
:Molded-Case Circuit Breakers
:Surge Protective Device

Source: Study team

Figure7.4.204 Single Wire Diagram

(b) Determination of Cooling Method

From the Design Guideline for Telecommunications Facilities (Electric), the radiator type has been adopted as the standard cooling system for diesel engines. A schematic diagram of the radiator cooling system is shown below.

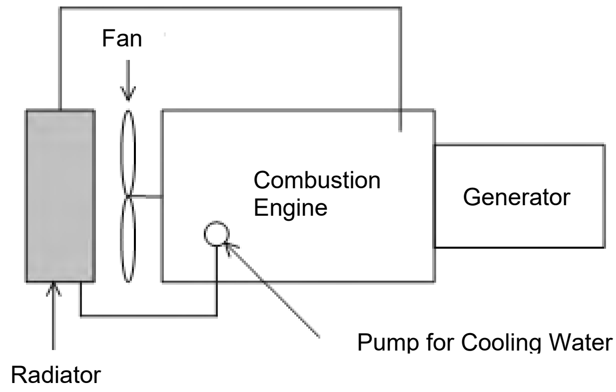
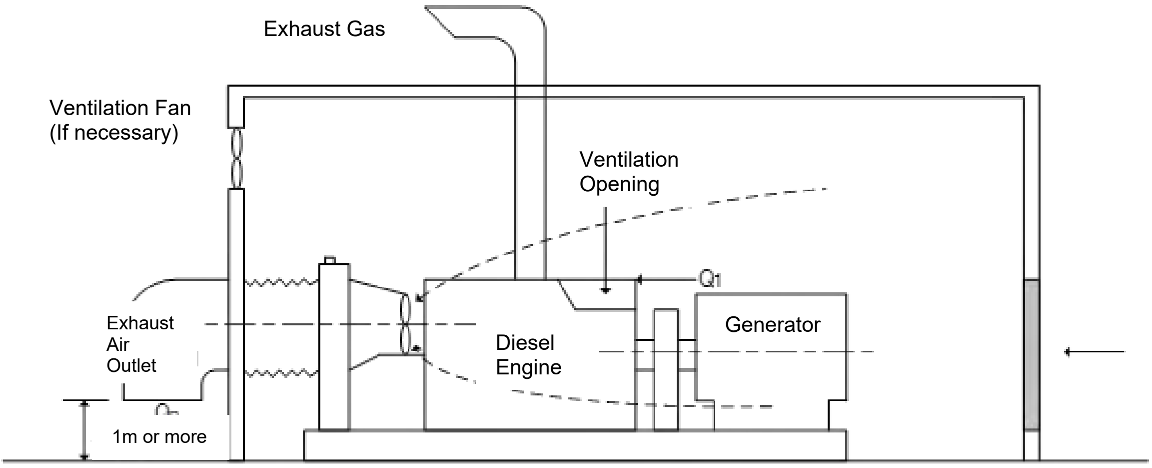


Figure7.4.205 Radiator Cooling Type

4) Ventilation Volume Calculation

The schematic diagram of ventilation in the radiator cooling system is shown below.



Source: Study Team

Figure7.4.206 Schematic Diagram of the Radiator Cooling System

The ventilation rate of the radiator type is obtained from the following.

Required Air Volume Supply Q (m3/min) = Q1 + QR

Q1: Amount of air required for combustion of the motor

$$Q1 = \frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho} \quad \text{(m3/min)}$$

A′ : Amount of air required to burn 1 kg of fuel (m3/kg)

(Heavy oil A: 14.6, light oil: 14.7)

be: Engine fuel consumption rate (kg/kWh)

In the case of a radiator type, the value added by an increase of 7%

Pe: Motor output (kW)

ε : Excess air ratio (Non-supercharged engine = 2.0, engine with supercharger = 2.5)

ρ : Air density (= 1.165 at 30 °C) (kg/m3).

QR: Radiator fan ventilation (m3/min)

Table7.4.255 Amount of Ventilation by the Radiator Fan

Output Power		By The Radiator Fan Ventilation (M3/Min)	Galari Area (m2)
Motor (kW)	Generator (kVA)		
37	37.5	150	0.8
49	50	175	1.0
60	62.5	186	1.0
71	75	191	1.1
94	100	250	1.4
116	125	311	1.7
138	150	375	2.1
182	200	400	2.2
225	250	500	2.8
268	300	600	3.3
332	375	750	4.2
440	500	1000	5.6
545	625	1250	6.9
652	750	1500	8.3
759	875	1750	9.7
864	1000	2000	11.1

Source Design Guideline for Telecommunications Facilities (Electric), p 3 -48

Each value and calculation result in this design are shown below.

Table7.4.256 Calculated Ventilation Rate

Gate Opening And Closing	For Control Equipment
・A '= 1.47 ・be = 0.32 (7% increase of 0.30 kg/kWh for 22 ~ 184 kW diesel engines) ・Pe = 138 ・ε = 2.0 ・ ρ = 1.165 ・QR = 375 (Table7.4.255) Q1 = $\frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho}$ = $\frac{1.47 \times 0.32 \times 138 \times 2.0}{60 \times 1.165}$ Size = 1.86 Q = 1.86 + 375 = 376.86 (m3/min) = 22611 (m3/h)	・A '= 1.47 ・be = 0.32 (7% increase of 0.30 kg/kWh for 22 ~ 184 kW diesel engines) ・Pe = 49 ・ε = 2.0 ・ ρ = 1.165 ・QR = 175 (Table7.4.255) Q1 = $\frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho}$ = $\frac{1.47 \times 0.32 \times 49 \times 2.0}{60 \times 1.165}$ Size = 0.66 Q = 0.66 + 175 = 175.66 (m3/min) = 10539.6 (m3/h)

Source: Study team

(a) Examination of fuel consumption

Diesel oil is used as fuel.

(b) Calculation of fuel and lubricating oil consumption

(i) Calculation of fuel consumption

The fuel consumption per 1 hour is as follows based on the calculation shown in Design Guideline for Telecommunications Facilities (Electric).

$$Fuel\ Consumption\ \left(\frac{L}{h}\right) = \frac{Motor\ Output\ (kw) \times Fuel\ Consumption\ Rate\ (g/kWh)}{1000\ \times Specific\ Gravity\ of\ Fuel}$$ [13]

The fuel consumption rate and specific gravity of fuel are as follows according to the Design Guideline for Telecommunications Facilities (Electric).

Table7.4.257 Fuel Consumption Rate (Unit: g/kWh)

Motor Output (kW)		Less than 22	Not less than 22 to less than 184	Not less than 184 to less than 331	Not less than 331 to less than 552	Not less than 552
Fuel Consumption Rate	Diesel Engine	310	300	270	250	230
	Gas Turbine		680	660	590	520

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3 -39

Table7.4.258 Specific Gravity of Fuel

Fuel to be Used	Specific Gravity
A Heavy Oil	0.85
Light Oil	0.83
kerosene	0.79

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3, -39

Each value and calculation result in this design are shown below.

[13] Design Guideline for Telecommunications Facilities (Electric)

For Gate Opening And Closing	For Control Equipment
・Motor output: 138 kW ・Fuel consumption rate: 300 (diesel engine) ・Specific gravity of fuel: 0.83 (light oil) Fuel consumption (L/h) $=\frac{Motor\ Output\ (kw)\times Fuel\ Consumption\ Rate\ (g/kWh)}{1000\times Specific\ Gravity\ of\ Fuel}$ $=\frac{138\times 300}{1000\times 0.83}$ =49.88 7% increase due to radiator cooling system 49.88 × 1.07 = 53.37 (L/h)	・Motor output: 48.6 kW ・Fuel consumption rate: 300 (diesel engine) ・Specific gravity of fuel: 0.83 (light oil) Fuel consumption (L/h) $=\frac{Motor\ Output\ (kw)\times Fuel\ Consumption\ Rate\ (g/kWh)}{1000\times Specific\ Gravity\ of\ Fuel}$ $=\frac{48.6\times 300}{1000\times 0.83}$ =17.57 7% increase due to radiator cooling system 17.57 × 1.07 = 18.80 (L/h)

(ii) Amount of Fuel to be Stored

The required amount of fuel oil storage is calculated by the following formula based on Design Guideline for Telecommunications Facilities (Electric).

Amount of Fuel to be Stored (L) = Fuel Consumption (L/h) x Operating Time (h)[14]

The operating time is calculated as 2 hours for the gate opening/closing generator and 72 hours for the control equipment generator according to the design conditions. The amount of fuel to be stored is as follows.

For Gate Opening And Closing	For Control Equipment
Amount of fuel to be stored = 53.37 x 2 = 107 (L)	Amount of fuel to be stored = 18.88 x 72 = 1353 (L)

5) Examination of the Installation Location

From the design results up to the preceding paragraph, the equipment to be installed is as follows.

- Generator for Gate Operation : 150 kVA
- Generator for Control Equipment : 50 kVA
- Fuel Tank

(a) Generator for Gate Operation

The generator for gate operation is installed to enable sure gate operation in the power failure. The generator for gate opening and closing is planned in accordance with the layout plan of the generator house (See **Figure 7.4.207**).

(b) Generator for Control Equipment

The generator for the control equipment is installed to enable the sure facility control in the power failure. The generator for the control equipment is prepared together with the fuel tank to satisfy the power failure compensation of 3 days (72 hours) which is set in the previous section. The generator for gate opening and closing is planned in accordance with the layout plan of the generator house (See **Figure 7.4.207**).

(c) Fuel Storage

Fuel of the generator for gate opening and closing is supplied by a service tank built in the generator. In order to install two generators for gate opening and closing, specification is made to satisfy required quantity by two generators for normal use and emergency use.

[14] Design Guideline for Telecommunications Facilities (Electric)

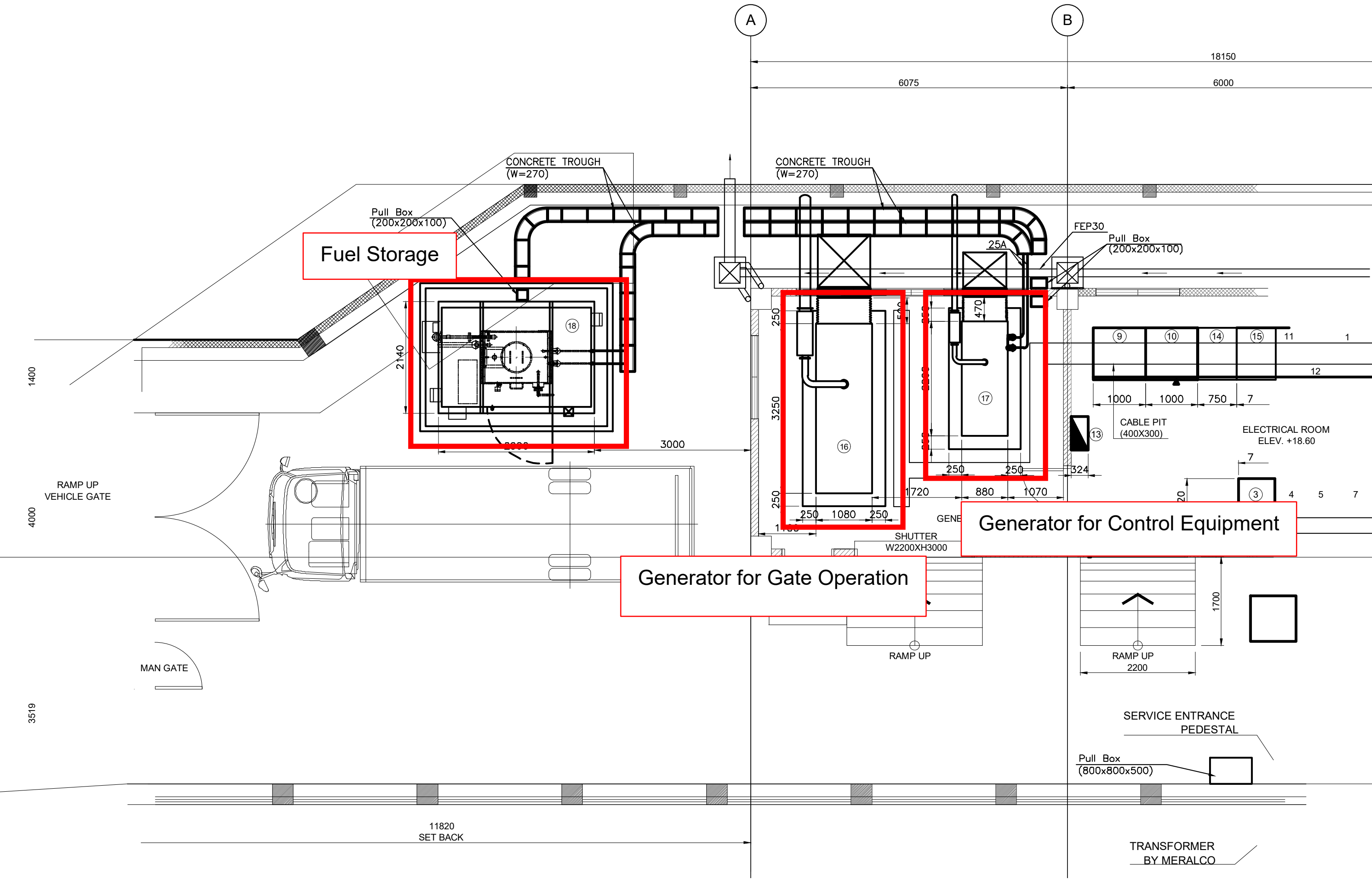
Since it is difficult to store the fuel for the generator for the control facility in the service tank, a fuel storage tank is installed outside the generator building. The arrangement of the storage (clearance) is based on the installation standard in the Philippines, and ensures a clearance of 3 m from the generator building.

Table 7.4.259 Clearance of Combustible Liquid Type and Capacity from Building

Table 20: STORAGE OF FLAMMABLE OR COMBUSTIBLE LIQUIDS IN CLOSED CONTAINERS OUTSIDE OF BUILDINGS

QUANTITY IN LITERS	DISTANCE FROM BUILDING OR LINE OF ADJOINING PROPERTY WHICH MAY BE BUILT UPON IN METERS
CLASS I	
1 to 568 (3 drums)	4.5
568 to 1892 (3 to 10 drums)	7.5
1893 to 18925 (10 to 100 drums)	15
CLASS II or III	
1 to 568 (3 drums)	1.5
568 to 1892 (3 to 10 drums)	3
1893 to 18925 (10 to 100 drums)	9

Source: FIRE CODE OF THE PHILIPPINES OF 2008



Source: Study Team

Figure 7.4.207 Arrangement of Generators and Oil Storage

6) Generator Room Plan

(a) Ventilation System

The natural ventilation system is adopted as the ventilation system of the power generation facilities.

When natural ventilation is used, the required area A of the air supply gallery is determined by the following formula.

$$A = \frac{Q1 + QR}{60 \times V \times \alpha} \quad (m2)$$

Here,

A : Galari area (m2)
Q1 : Amount of Air Required For Combustion of Fuel (m3/min)
QR : Radiator Fan Ventilation (m3/min)
V : Wind Velocity 4 m/sec
α : Light Transmittance (Size = 0.3), when a wire gauze is installed on the light transmittance (Size = 0.27)

Each value and calculation result in this design are shown below.

For gate opening and closing	For control equipment
$A = \dfrac{1.86 + 375}{60 \text{ x } 4 \text{ x } 0.27} = 5.82 \text{ m } 2$	$A = \dfrac{0.66 + 175}{60 \text{ x } 4 \text{ x } 0.27} = 2.71 \text{ m } 2$

(b) Separation Distance Between Devices

The separation distance between the components of the generator is based on **Table7.4.260** described in Design Guideline for Telecommunications Facilities (Electric).

Table7.4.260 Separation Distance Between Devices

<table>
<tr><th colspan="3">Portion To Secure The Separation Distance</th><th>Separation Distance</th></tr>
<tr><td colspan="2" rowspan="3">Cubicle Type</td><td>Operation Side</td><td>More than 1.0m</td></tr>
<tr><td>Maintenance Side</td><td>More than 0.6 m
In case of transformers, storage batteries and portion facing to a building, 1.0m</td></tr>
<tr><td>Ventilation Side</td><td>More than 0.2 m</td></tr>
<tr><td rowspan="9">Not Cubicle Type</td><td rowspan="2">Generator</td><td>Between Generators</td><td>More than 1.0 m</td></tr>
<tr><td>Around Generators</td><td>More than 0.6 m</td></tr>
<tr><td rowspan="3">Generator Control Board</td><td>Operation Side</td><td>More than 1.0 m
(In case of facing each other, 1.2m)</td></tr>
<tr><td>Maintenance Side</td><td>More than 0.6 m
(In case of facing each other, 1.0m)</td></tr>
<tr><td>Ventilation Side</td><td>More than 0.2 m</td></tr>
<tr><td rowspan="2">Fuel Tank Of Small Lots</td><td>Internal - combustion engine</td><td>More than 0.6 m
More than 2.0 m</td></tr>
<tr><td>Inside surface of Oil Retaining Wall</td><td>According to Local Law</td></tr>
<tr><td rowspan="2">DC Power Supply Equipment</td><td>Operation Side</td><td>More than 1.0 m</td></tr>
<tr><td>Maintenance Side</td><td>More than 0.6 m</td></tr>
</table>

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3-65

(c) Seismic Measures for Power Generation Equipment

The generator and prime mover shall be directly connected to each other and fixed to a common trestle having a vibration-proof and earthquake-proof structure. The generator shall be equipped with a fall prevention device to prevent the generator from falling off due to an earthquake.

(i) Basic Dimensions of the Generator

Basic dimensions of power generation equipment shall be examined in accordance with the Telecommunications Facility Design Guideline for Telecommunications Facilities (Electric) p. 3 -53.

(1) Foundation
1) Dimensions of Foundation
The dimensions of foundation are as follows.
Width => (Width of Common Bed) + 0.3 (m) Note
Length => (Length of Common Bed) + 0.3 (m) Note
Bottom to Surface of Foundation =>0.1 (m)

Note: In case of adhesive anchor, 0.3m. 0.5 m or more shall be secured in case of blackout anchor.

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3 -53

The installation dimensions of the generator used for the study are set with reference to a manufacturer's generator. The generator dimensions and foundation dimensions are indicated in **Table7.4.261**.

Table7.4.261 Generator Dimensions and Foundation Dimensions

Generator	Model Number	Generator Dimensions (mm)	Foundation Dimensions (Mm)
150 kVA	DCA -150 ESK	Width: 1080 Length: 3250	Width: 1080 + 300 = 1380 Length: 3250 + 300 = 3550
50 kVA	DCA -60 ESI2	Width: 880 Length: 2200	Width: 880 + 300 = 1180 Length: 2200 + 300 = 2500

Source: Study team

(ii) Seismic calculation of anchor bolts

The anchor bolt size of the generator and the fuel tank shall be selected based on the "Guidelines for earthquake-resistant design and construction of building equipment 2014". As a result of the calculation, the following anchor bolts were selected. The detailed calculations are shown in **Vol.5A Structural Calculation for Contract Package-1**.

A generator for opening and closing the gate: The anchor bolt W 3/4 (8 sticks) does not cause insufficient strength.

Generator for control facility: Using anchor bolt W 5/8 (8 sticks) does not cause insufficient strength.

7) Setting Display Items

The generator board mounted on the generator shall be equipped with a status alarm indication by lamp so that the generator status and fault alarm can be visually confirmed. The contents and items to be indicated are shown in **Table 7.4.262**.

Table 7.4.262 Contents and Items to be Indicated

(1) The light source shall be a light-emitting diode. LED lights must be replaced easily. When it is not easy to replace LED lights on printed wiring board, the spare of the board shall be stored.
(2) Condition indicator (monitoring items and control items) and external connection terminal are as shown in the following Table 1.
(3) Protective device shall be attached in accordance with Table 2.
  1) In case of failure, the contents must be notified with lamp indicators and alarm.
  2) When a failure occurs, self-holding shall be performed, and it shall be released by the operation of a push bottom for "Failure Recovery"

Table 1

Item to be Displayed		Outside Contact		Remarks
		Monitoring	Control	
1	Commercial	○	-	Lighting while commercial power is normal
2	Abnormal in Commercial Power	○	-	
3	Power Generation	○	-	Lighting when the build up voltage of generator
4	Power Supply	○	○	Lighting In Circuit Breaker on or Switcher on
5	Power Generation and Supply	○	○	Lighting In Circuit Breaker on or Switcher on
6	Machine Side	○		Based on the switch of "Machine Side" or "Remote"
7	Remote	○	-	
8	Manual Start-up	-	-	Based on the switch of "Manual", "Automatic" or "Test"
9	Automatic Start up	-	-	
10	Starting	-	○	
11	Stop	-	○	

Source: Study team translated from" Emergency Power Generation Device Specifications (Draft) January 2017, p. 10".

Table 2

Type	Item to be Indicated	Engine Stop	Main Circuit Break	Detector	Outside Contact
Major Failure	Start Failure	○	-	Timer/Switch To Detect Start Failure	○
	Lubricating Oil Pressure Decrement	○	○	Switch To Detect Hydraulic Pressure	○
	Cooling Water Cut-off/Temperature Rising	○	○	Switch To Detect Water Cut-Off/Water Temperature	○
	Over Rotating	○	○	Switch To Detect Over Rotating	○
	Over Current		○	Over Current Relay	○
	Minimum Amount of Fuel	○	○	Equipment To Detect Oil Surface	○
	Emergency Stop	○	○	Manual	○
	Control Power Abnormality	○	○	DC Undervoltage Relay	○
Minor Failure	Oil Surface of Fuel Decrement	-	-	Equipment To Detect Oil Surface	○
	Battery Temperature Rising	-	-	Alert To Indicate Battery Temperature Rising	○
	Auxiliary Device Failure	-	-	Over Current Relay/Open-Phase Relay	○

Note: "○" indicates "must be adopted".
Source: Study team translated from" Emergency Power Generation Device Specifications (Draft) January 2017, p. 11"

7.5 Detailed Design of Taytay Sluiceway

7.5.1 Outline of Detailed Design Results of Taytay Sluiceway

The detailed design is carried out based on the dimensions set in **Sub-Section 6.4.4 Basic Design of Taytay Sluiceway**. In the detailed design, the following examination is carried out.

- Structural Design of Civil Engineering Facilities (Foundation works, body works, columns, breast walls, wing walls, etc.) and Level 2 Seismic Design of Main body
- Determination of structural design and specifications of Gate Facility
- Designing Temporary Equipment (detour road and cofferdam)
- Detailed examination and specification determination of information and telecommunications facilities and electrical facilities

7.5.2 Civil Engineering Design

7.5.2.1 Dimensions of Major Structure

The result of setting the dimensions major structures of Taytay Sluiceway is shown in **Table 7.5.1**. Items that is not set in the basic design are described in detail in the following pages.

Table 7.5.1 Dimensions of Major Structure of Taytay Sluiceway

Item	This Design	
	Conclusion	Reason for Setting
Length of Main Body	7.50 m	Set the river side edge of the East Bank Road as the "Dike alignment" and extend the 2.0:1 slope which is standard slope for dike embankment from that point t. The structural drawings are prepared under the conditions of the invert elevation, the inner height and the breast wall height of 0.5 m, and the length of main body is set to 7.5 m.
Span Allocation	1 span	Since the length of the main body as short as 7.5 m , it has 1 span.
Structure of End of Culvert	**Figure 7.5.1**	Portion around columns and piers should be reinforced with increasing the thickness of members. Usually, an area from the column base is reinforced, but the entire Taytay Sluiceway is reinforced because its longitudinal length is as short as 7.5 m.
Seepage Cut-Off Wall	Not Installed	Since Taytay Sluiceway does not reach the center of the dike embankment where the seepage cut-off wall is placed, the wall is not to considered. The land side is a culvert structure of 145 m in length. Since the section of Manggahan Floodway where the culvert is placed is the excavated river channel, the seepage cut-off wall is unnecessary[1].
Column	Operation Deck Height EL + 16.20 m	The top of the operation deck is EL + 16.20 m, considering gate maintenance.
Operation Deck	**Figure7.5.77**	The plane dimensions of the operation deck are set based on the layout of various facilities such as the hoisting device and power generator.
Breast Wall	Invert T-shaped-Shaped Retaining Wall	The breast wall shall be 1 m long in the transverse direction of the culvert, and the structural type shall be invert T-shaped.
Seepage Control Work	Installation of SSP For Seepage Controls	An SSPs for seepage control are provided in the range of dike to be excavated. SSP with flexible joint are installed at the joint with the main body. Since the culvert is 145 m in length, piping calculation is not performed, and the minimum length is set to 2 m.
Grout Hole	**Figure7.5.2**	Since it shall be installed 5 m interval in standard, place it in one location.
Wing Wall	U shaped Type	The structure shall be U-shaped RC structure. The Invert T-shaped type needs many pieces of SSPs for seepage controls and is uneconomical. The dimensions are set considering the stability against uplift force by attaching overhang.
Connecting Water Channel	Gabion Mattress	Gabion Mattress which follows the deformation of the ground is applied to the revetment of the connecting water channel.

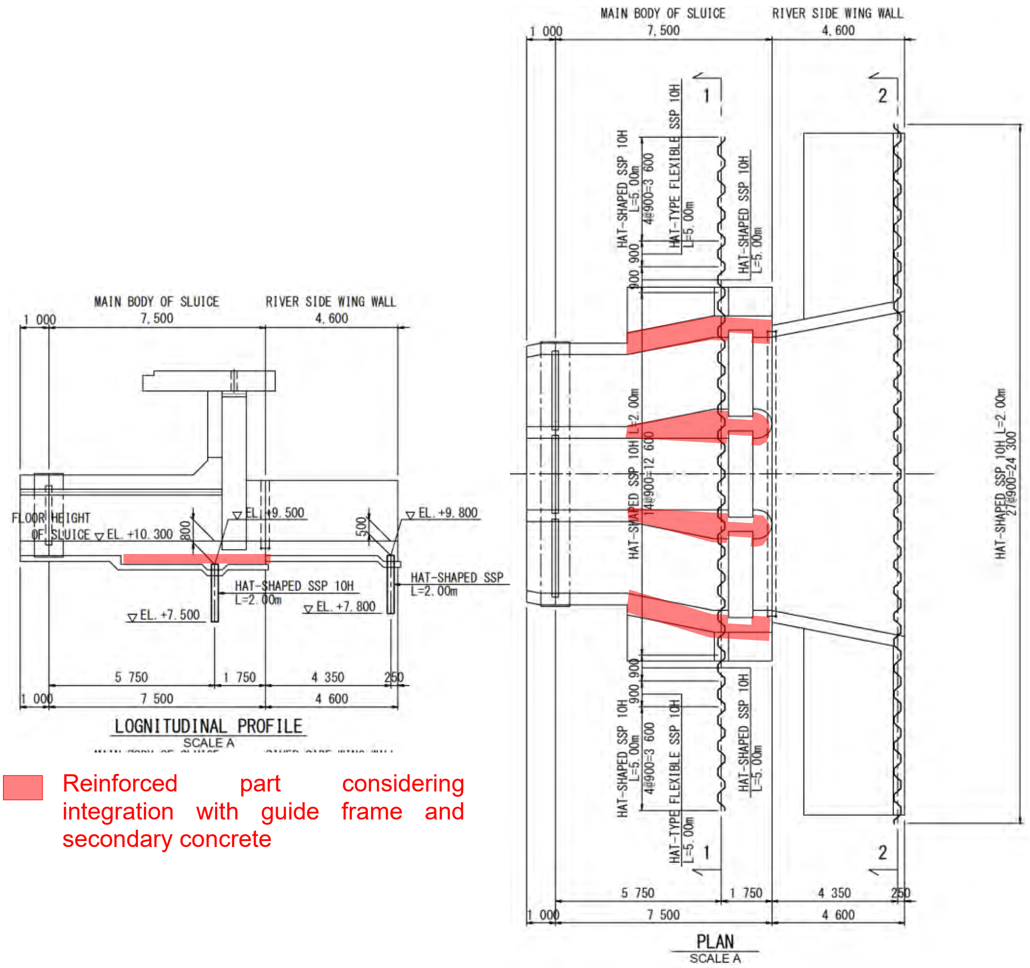
[1] Technical Criteria for River Works: Practical Guide for Planning [I], P100

Item	This Design	
	Conclusion	Reason for Setting
Maintenance Bridge and Abutment	Refer to the Standard Design of Ministry of Construction, Japan	Prepare a plan for a maintenance bridge of 1.0 m width in accordance with the standard design.
Dike Revetment	Reinforced Concrete Lining (t = 200 mm)	Reinforced concrete facing (t = 200 mm) is adopted, which is a local revetment style. Joint material is placed in 20 m interval. Revetments shall be placed to cover the area to be excavated.
Stair	Cast-in-place Concrete	
Transition	**Figure7.5.18**	Fit to the upstream and downstream side smoothly.
Guard House	L shaped Retaining Wall	An L-shaped retaining wall will be placed in the riverside and a foundation ground is built for the Guard house.

Source: Study Team

(1) Culvert End

The end of the box and culvert shall be integrated with the column and the breast wall and shall have a structure that is safe against loads such as the breast wall. At the end of the box, the following points are considered. The dimensions are shown in **Figure 7.5.1**.

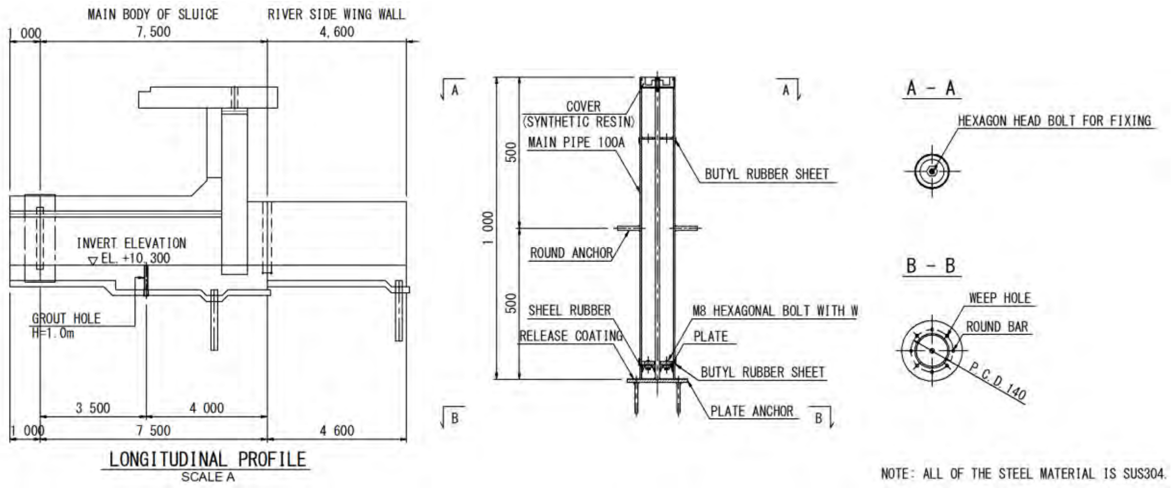


Source: Study Team

Figure 7.5.1 Structural Detail of the Culvert End

(2) Grout Hole

Grout holes are installed in sluiceways regardless of the type of foundation, as voids is easily formed below the bottom surface of the sluiceway due to residual settlement. The standard installation interval shall be 5 m or less considering the diffusion range of the filler material.



Source: Study Team

Figure7.5.2 Grout Hole Layout and Structure(Sample Only)

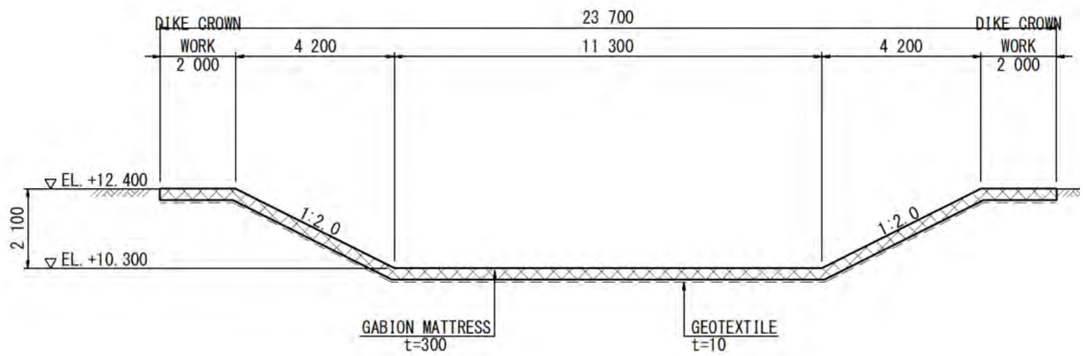
(3) Operation deck

The plane dimension of the operation deck is determined based on the wall of the operation room and the layout of equipment to be installed in the room. The results of the equipment layout are shown in the **Figure7.5.77**.

(4) Connecting Water Channel

Gabion Mattress (basket) is applied to the resentment of the water channel. The cross-sectional shape is determined as shown in **Figure7.5.3**.

The low channel width of 11.3 m was determined considering the expansion of the wing wall with 5.0;1. In addition, the side slope is set to 2.0:1 which is the standard vales for a gentle slope and is also used in the dike embankment slope. The width of shoulder protection is set to 2 m.

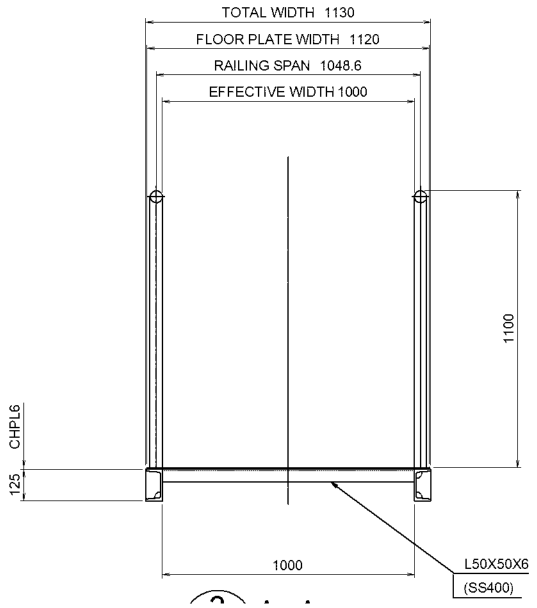


Source: Study Team

Figure7.5.3 Cross Section of Connecting Water Channel

(5) Maintenance bridge and Abutment

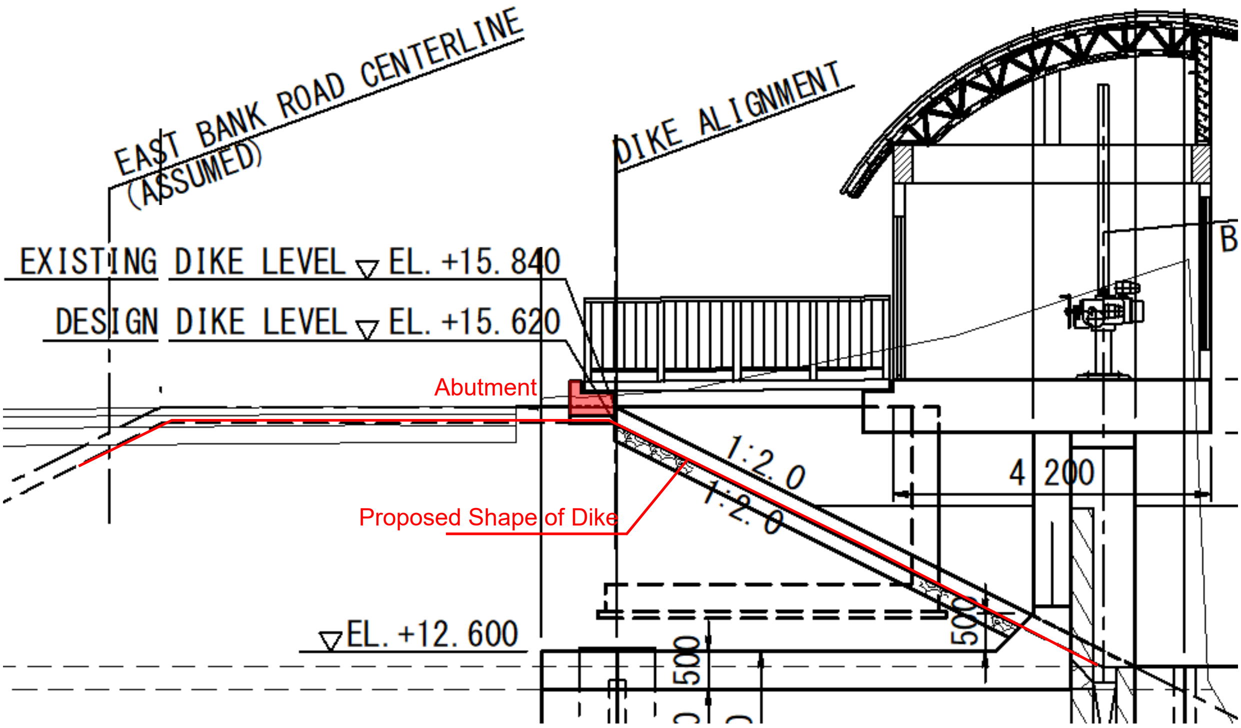
The maintenance bridge of the sluiceway shall have an effective width of 1 m or more, and the girder height shall be above design dike crown. The results of the drawing are shown in **Figure7.5.4**. The girder height is set to EL + 16.045 m, which is higher than design dike crown EL + 15.62 m. The dimensions of the main members of the bridge refers to the standard design of the Ministry of Construction, Japan.



Source: Study Team

Figure7.5.4 Cross Section of the Maintenance Bridge

Generally, the abutment of sluiceway shall have a concrete structure and shall not be installed within the proposed section of the dike. The results of the drawing are shown in **Figure7.5.5**.

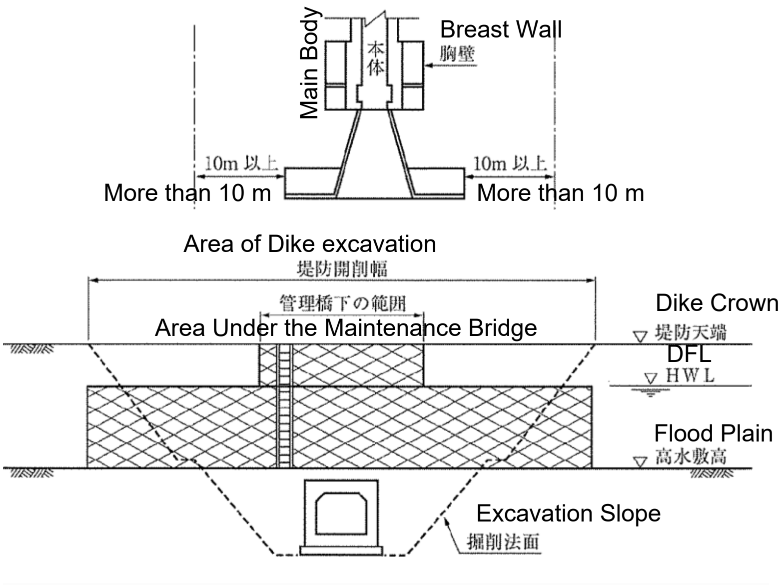


Source: Study Team

Figure7.5.5 Relationship between Abutment and Proposed Shape of Dike

(6) Revetment of Dike

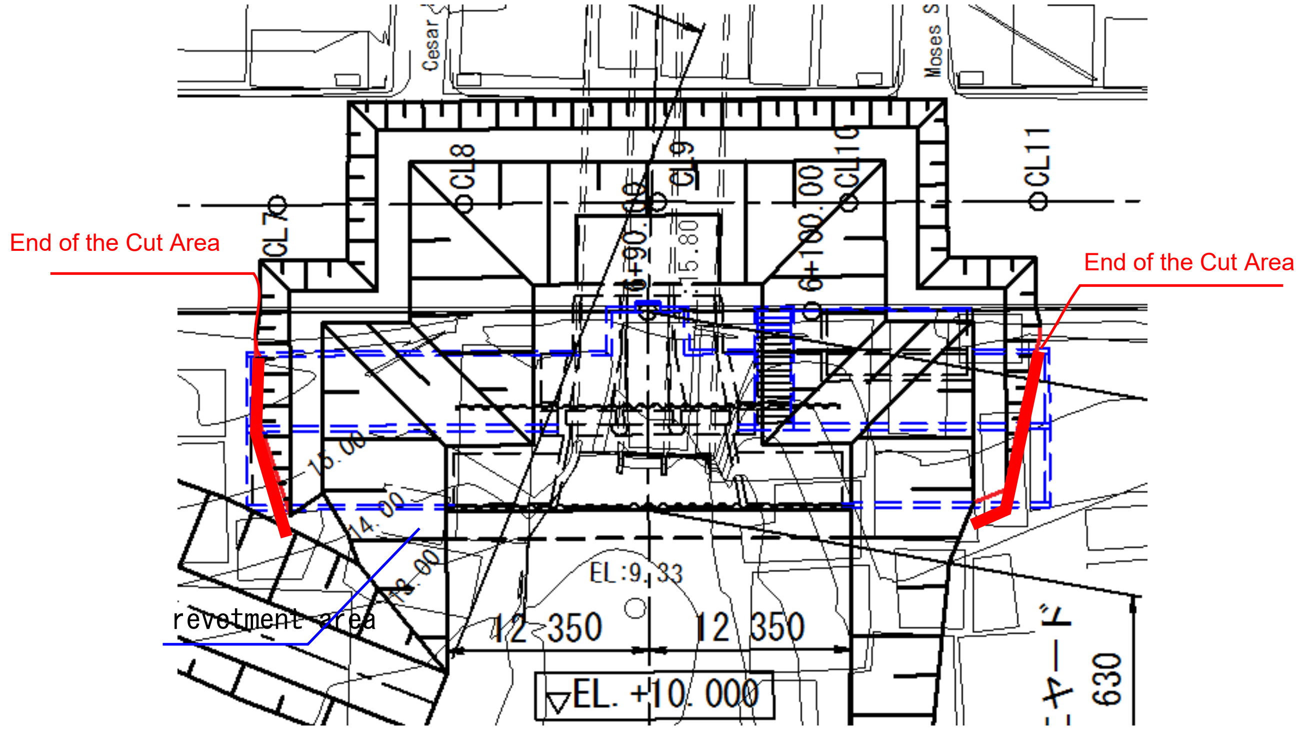
The revetment on the river side of the sluiceway is arranged up to DFL and within a range of 10 m from the end of the sluiceway to cover the area of dike to be excavated.



Source: Study Team

Figure7.5.6 Extent of Revetment

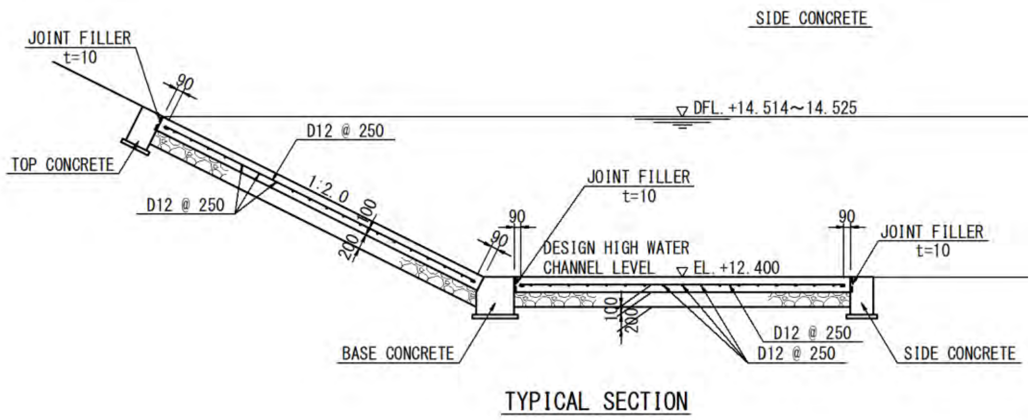
The revetment area for Taytay Sluiceway is shown in **Figure7.5.7**. The area is set to cover the area of dike to be excavated.



Source: Study Team

Figure7.5.7 Extent of Dike Excavation and Revetment

The revetment structure shall be reinforced concrete facing (t = 200 mm). (**Figure7.5.8**).

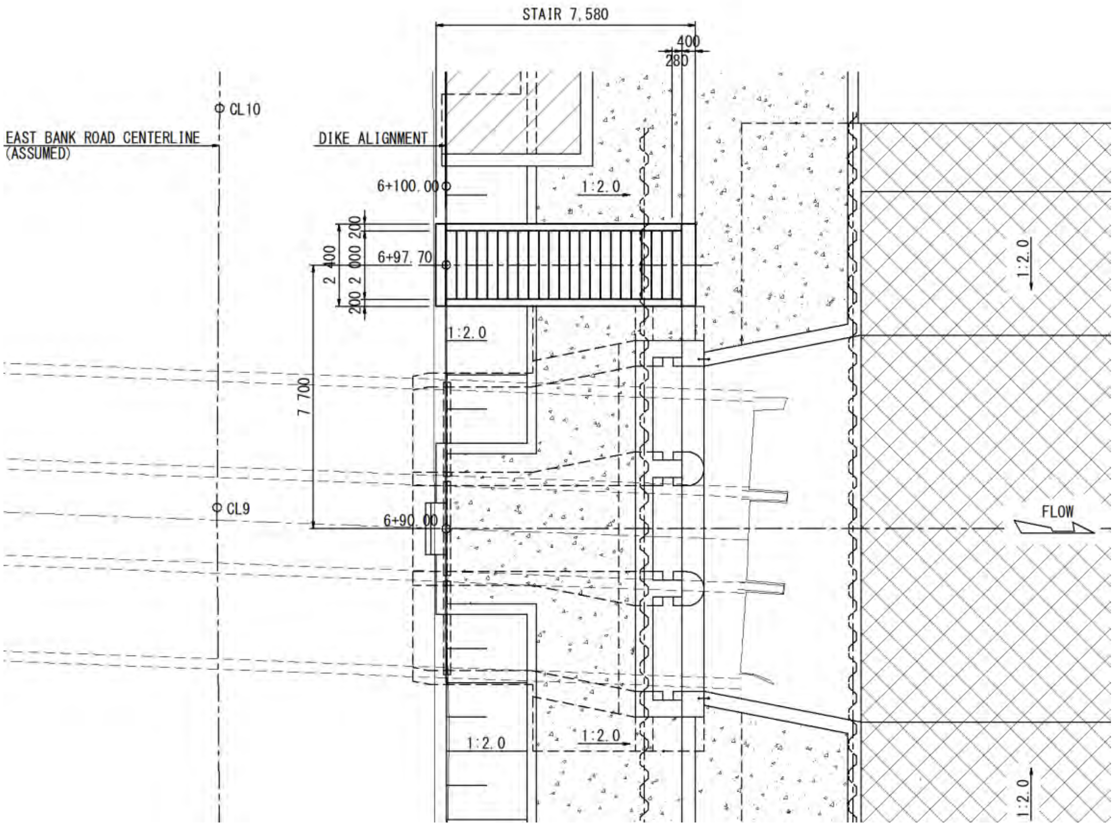


Source: Study Team

Figure7.5.8 Revetment Structure

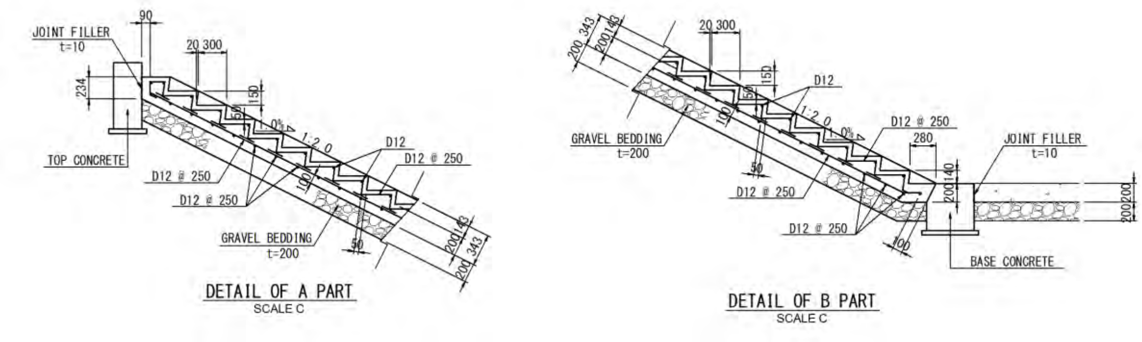
(7) Stair

The sluiceway is provided with stairs for maintenance on the slope of river side. Generally, the stairway should be of concrete construction, with an effective width of 1 m or more and a height of about 15 ~ 20 cm per stairway and be located in the downstream side. The plan of the stairway is shown in **Figure7.5.9** and **Figure7.5.10**.



Source: Study Team

Figure7.5.9 Stairway Plan (1)



Source: Study Team

Figure7.5.10 Stair Work Plan (2)

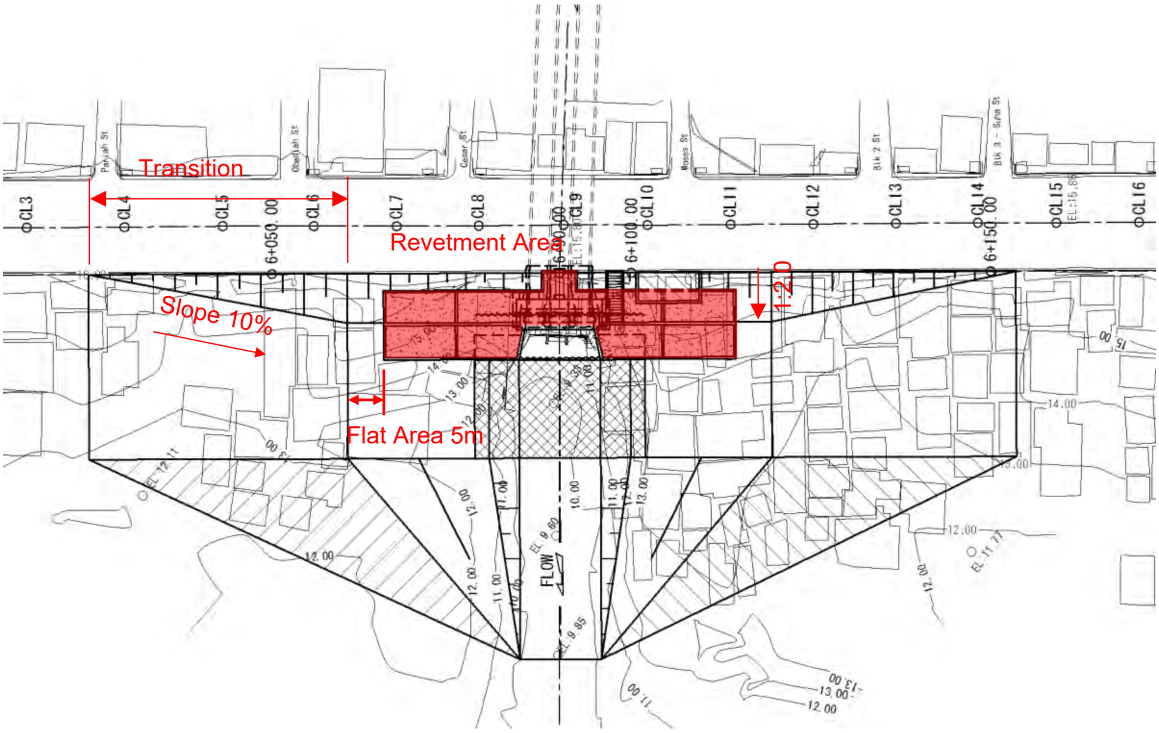
(8) Transition to the Existing Dike

In front of the river side edge of the East Bank Road, which is the dike alignment banks, there is a gentle slope on which houses are built (see **Figure7.5.11**). due to the excavation based on the proposed side slope of 2.0:1, transition to the existing dike would be needed. The transition is shown in **Figure7.5.12**. The concepts are as follows.

- In consideration of the layout of heavy equipment during revetment construction, a flat area of 5 m from the upstream and downstream ends of the revetment is considered.
- A slope of 2.0:1 is constructed along the embankment from the end, and transition from the road surface to the floodplain surface with a slope of 10%. The slope of 10% is designed to allow entry of heavy maintenance equipment.

Source: Study Team

Figure7.5.11 View of Existing Culvert Outlet

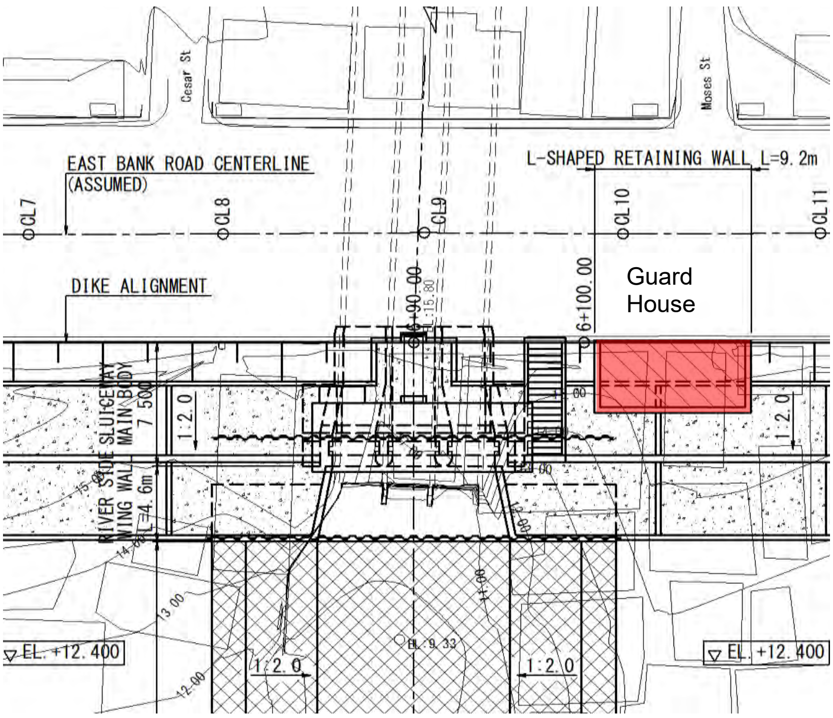


Source: Study Team

Figure7.5.12 Setting the Transition Area

(9) Guard House

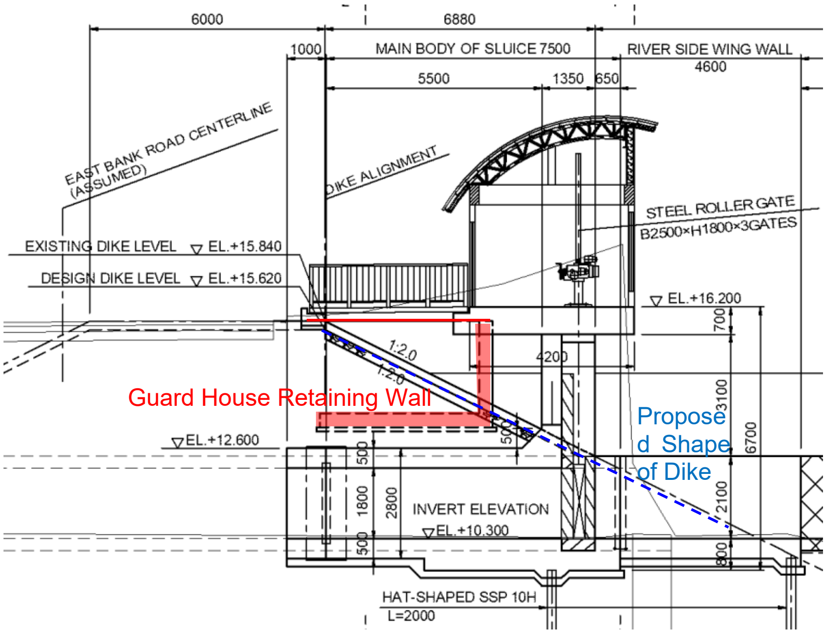
In order to prevent the vandalization of the sluiceway equipment, a guard house would be placed. The floor dimensions and location of the guard house are in the downstream of the stairway. It is shown in **Figure7.5.13**.



Source: Study Team

Figure7.5.13 Location of Guard House

The guardhouse is constructed with an L-shaped retaining wall. Since it is installed in the river side, it will cut into a proposed shape of the dike(see **Figure7.5.14**). However, since this section of Manggahan Floodway is an excavated river channel and the shortage of sectional area won't directly lead to instability of the dike embankment, this can be allowed.



Source: Study Team

Figure7.5.14 Cross-Section of Guard house

(10) Detail of Connection with the Existing Culvert

The member thickness of the existing culvert is different from it of the new culvert. Furthermore, its bar detail could not be grasped. Hence, the treatment between the existing and new member would be proposed in accordance with the structure shown in **Figure 7.5.15**.

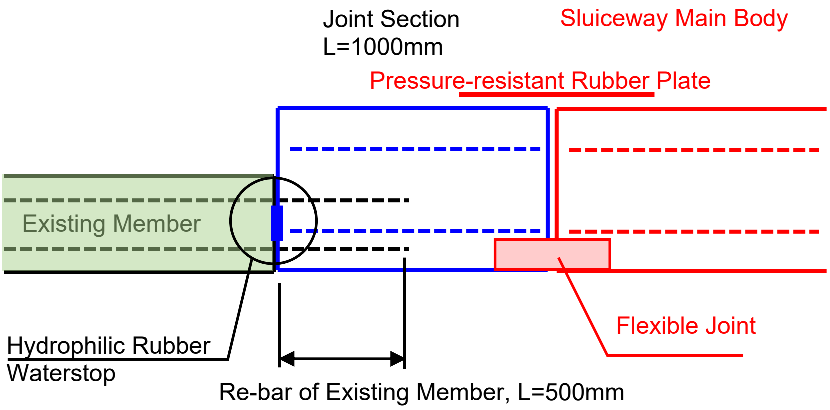


Figure 7.5.15 Detail of Connection Between the Existing and New Culvert

7.5.2.2 Confirmation of Design Conditions

In this section, necessary items such as design conditions, structural calculation and load conditions, ground conditions, water level conditions, and construction conditions for the structural design of Taytay Sluiceway are summarized.

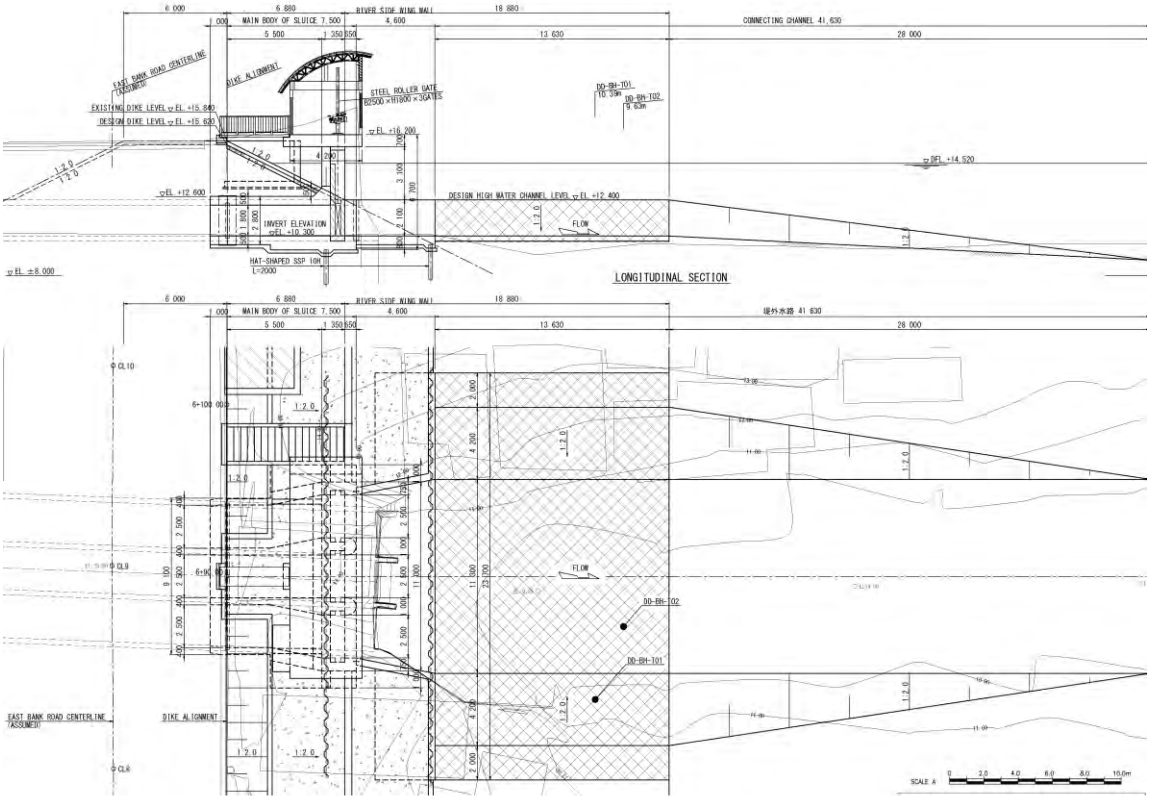
(1) Design Condition

The major design conditions are shown in **Table7.5.2**. The general drawing of the sluiceway, which is also the basic condition of the design, is shown in **Figure 7.5.1**.

Table7.5.2 Taytay Sluiceway Design Conditions List

Item	List of Conditions		Reason
Standard	Guideline for Flexible Sluiceway Sluiceway Reinforcement Manual (Draft)		The design theory of the sluiceway of the target facilities is based on the standard shown at left in Japan. It is also considered to be an extension of the existing structure.
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 property values in the Philippines are applied.
	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 design 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 Reinforcement	Box lateral direction, etc.	more than 0.2% of A	A: Effective cross-sectional area of concrete
	Longitudinal direction of culvert	more than 0.3% of A	〃
Reinforcement Specifications	Major bars are set outside, and minor bars are set for each part according to the basic idea in.		General arrangement. In case of column, since the distributing bars are lateral tie to seismic reinforcement the main bars are inside, and the distributing bars are outside.

Source: Study Team



Source: Study Team

Figure7.5.16 General Drawing of Taytay

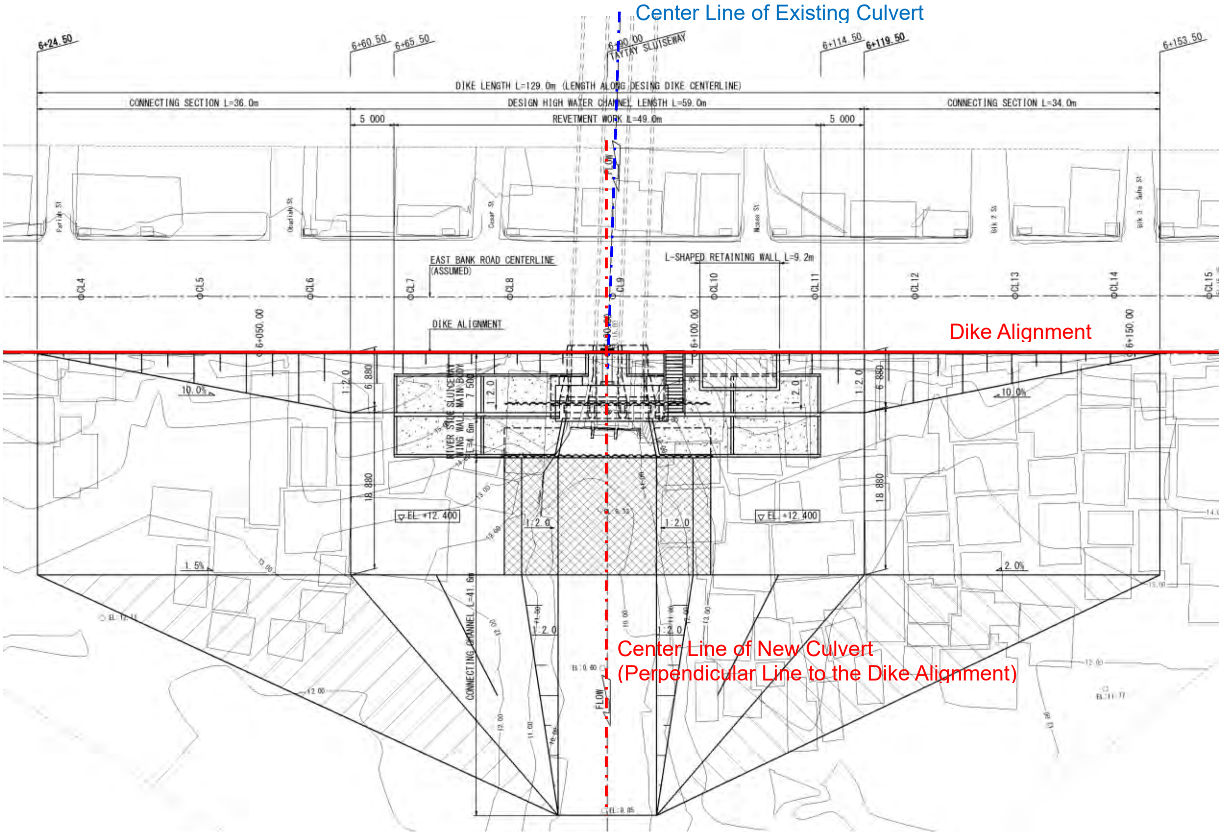
(2) Basic Specifications

This section describes the basic specifications (Location, Invert elevation, Section) of Taytay Sluiceway described in the basic design. The summary is shown in **Table7.5.3**.

Table7.5.3 Basic Specifications of Taytay Sluiceway

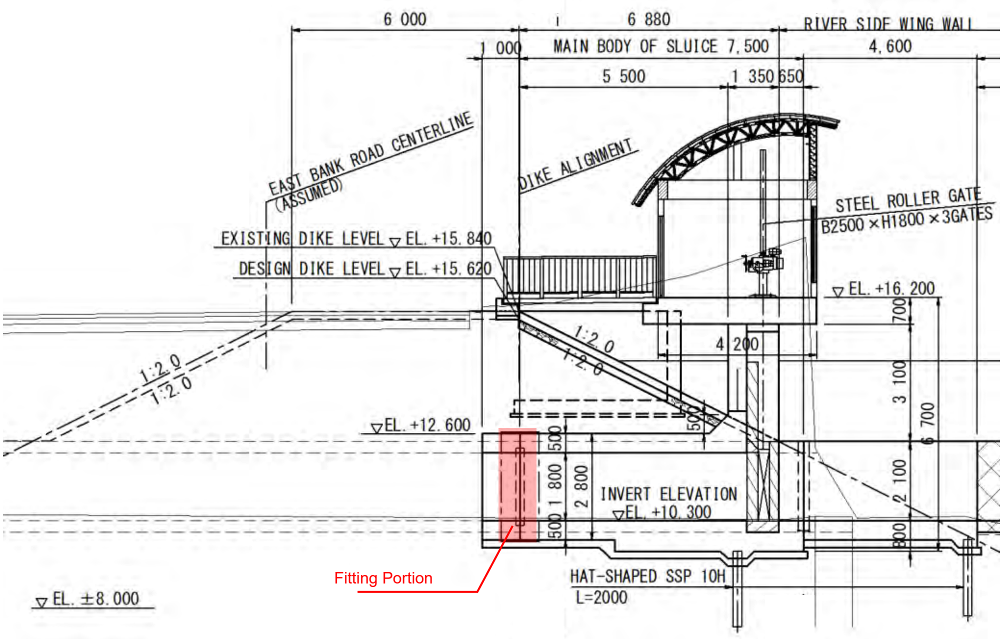
Item		Contents	Reason
Location		No. 6 + 090	Sta. of the center line of the existing culvert
Invert elevation		EL + 10.300	Present invert elevation of cutting position of existing culvert
Sectional Size		B 2.50 × H 1.80 × 3	As same as the existing cross section
Other	Dike section	Dike Crown Width of 6m Side Slope: 2.0 : 1	Same as the proposed shape of the dike in Manggahan Floodway
	Bank Alignment Embankment shoulder	Floodway Side of East bank Road	Since the existing road was developed as a dike at the same time as the Manggahan Floodway, the edge of the road is regarded as the shoulder of the dike which is the dike alignment
	Inclination of the existing culvert alignment	Fit within 1 m between the new section and the existing section	The existing culvert has an inclination of 2 degrees to the perpendicular line of the dike alignment. In addition, since the thickness of the members differs between existing and new sections, 1 m section for fitting from the existing end to the new section is provided.

Source: Study Team



Source: Study Team

Figure7.5.17 Inclination of the Existing Culvert Relative to the Dike Alignment



Source: Study Team

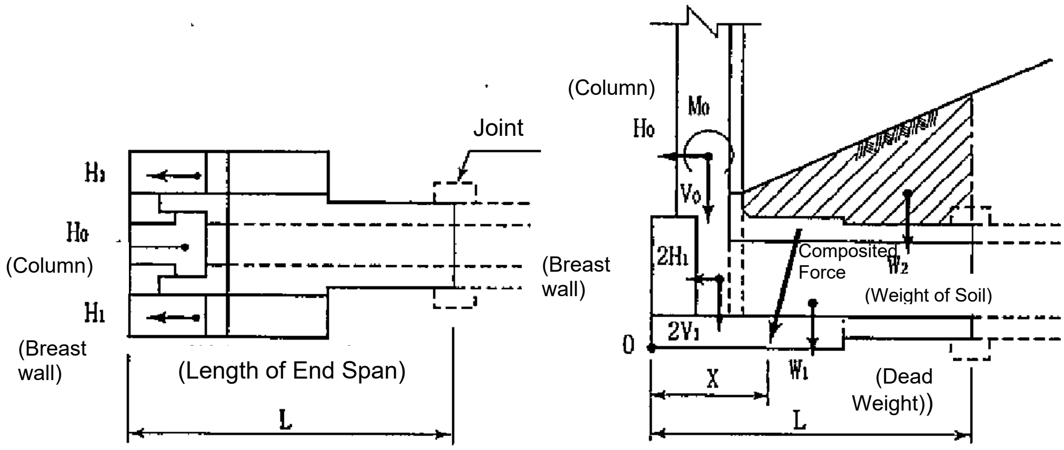
Figure7.5.18 Fitting Portion

(3) Structural Calculation Method

The main body stability calculation and the structural calculation of the culvert in transverse and longitudinal direction, wing wall, and breast wall are outlined below.

1) Main Body Stability analysis

The main structure of Taytay Sluiceway is an extended structure to the existing culvert in the landside, and the stability of sliding, overturning, and bearing capacity is verified under the condition that the horizontal load due to the embankment of the slope and the earthquake is predominant. The required safety factor is shown in **Table7.5.4**.



Source: Study Team

Figure7.5.19 Load Diagram for Calculating Stability of Main Body (Extension)

Table7.5.4 Safety Factor

Item	Normal Condition	Seismic Condition	Remarks
Sliding	1.5	1.2	
Overturn	L/6	L/3	L: Foundation width
Bearing Capacity	3.0	2.0	Calculation of ultimate bearing capacity of foundation ground by Taraghi

Source: Study Team

2) Box Lateral Direction

The load combination in the lateral direction of the culvert is summarized in **Table7.5.5**. The structural calculation is carried out assuming multiple load conditions based on the loading conditions of the vehicle, which act as live load.

Table7.5.5 Load Combination in the Transverse Direction of the Box Culvert

Load Classification		Lateral Design of The Culvert
Dead load	Dead Weight of Culvert	○
Live load	Vehicle Load	○
Earth pressure	Vertical Earth Pressure and Horizontal Earth Pressure	○
Hydraulic pressure	Groundwater Pressure	Depending on the conditions
	Internal Water Pressure, Etc.	Depending on the conditions

Source: Study Team

3) Longitudinal Direction of Culvert

The load in the longitudinal direction of the culvert is summarized in **Table7.5.6**. In this design, "Beam on elastic floor considering ground displacement" is applied.

Table7.5.6 Load Combination in Longitudinal Direction of Culvert

Load Classification		Longitudinal Design Model of The Box Culvert	
		Beam on an Elastic Floor	Beam on an Elastic Floor Considered Considering the Effect of Ground Displacement
Dead Load	Body weight (including columns and breast walls)	Considered	Considered
	Internal water weight	Considered	Considered
Influence of Ground Displacement	Ground displacement (settlement)	Not Considered	Considered
	Ground displacement (lateral displacement)	Not Considered	Depending on the Condition
Live Load	Vehicle load	Considered	Depending on the Condition
Earth Pressure	Vertical earth pressure	Considered	Not Considered
	Earth pressure acting on the breast wall	Considered	Considered
Hydraulic Pressure	Earth pressure acting on the breast wall	Considered	Considered
Effect of Negative Friction Force		Depending on the Condition	Depending on the Condition
Prestress	Pc culvert	Depending on the Condition	Depending on the Condition
Influence of Earthquake		Considered	Considered

Source: Study Team

4) Column

(a) Transverse Direction (Perpendicular Direction to the Flow)

The transverse direction of the column (perpendicular direction to the flow) is calculated as a portal rigid frame structure fixed to the end of the box culvert. Load conditions such as normal and earthquake is studied. load combinations is shown in below.

Table7.5.7 Lateral Load Combination

Load case		1	2	3	4	5	Remarks
Normal Condition	Ordinary Time	○	○	○	○		
	Wind Load		○		○		4.17 kN/m2
	Temperature Change			○	○		16.7℃。 −22.2℃
Seismic Condition						○	

Legend: ○···Considered.
Source: Study Team

(b) Longitudinal Direction (Flow Direction)

The longitudinal direction of the column (Flow Direction) is calculated as a cantilever beam fixed to the end of the box culvert under load conditions of total 3 cases such as normal condition (wind load, front side), normal condition (wind load, Rear side) and seismic condition. Since Taytay Sluiceway has a triple barrel structure, the bar arrangement specifications of the end pier and the center pier are examined.

5) Breast wall

The breast wall is designed as a cantilever beam fixed to the culvert.

6) Wing Wall

The wing wall shall be separated from the main body and stability analysis and structural calculation in normal condition and seismic condition shall be performed.

(4) Load Condition

1) Weight of the Local Control House

The weight of the local control house is based on the results in **Sub-section 7.5 structural design of Building Structure** and is shown in **Table7.5.8**. The summary is W = 54.96 t, and the load of the local control house is assumed to be W = 60 t because of the load increase in the construction stage may be expected.

Table7.5.8 Load of Local Control House

Name of the facility	Weight (t)	Remarks
Switchboard	0.20	
Receiving panel	0.25	
Water level gauge	0.5	
ITV	0.5	
Remote control	0.5	
Lightning protection transformer	0.15	
Uninterruptible power supply	0.15	
Generator	1.10	Main body and fuel tank
Operating room	51.61	= 7.28 t + 44.33 t, **Table7.5.9** Browse
	54.96	≒ 60 t

Source: Study Team

Table7.5.9 Control Room Weight List

Steel member

Item	Spec	Quantity		Number	Unit weight	Sub Total
		Area/Length	Unit		Unit	(in ton)
I-Column	200 X 186 X 14.5 X 24	1.500	m	4	kg/m	0.534
I-Beam	200 X 186 X 14.5 X 24	12.500	m	2	kg/m	2.223
I-Beam	200 X 186 X 14.5 X 24	3.800	m	2	kg/m	0.676
Tube (Bottom)	D=76mm, t=6mm	3.870	m	4	kg/m	0.161
Tube (Top)	D=76mm, t=6mm	6.060	m	4	kg/m	0.253
Tube (truss1)	D=44.5mm, t=3mm	0.470	m	56	kg/m	0.081
Tube (truss2)	D=44.5mm, t=3mm	0.340	m	28	kg/m	0.030
Perlin	75 X 50 t=2mm	12.500	m	16	kg/m	0.640
Roof sheet	t=1.6mm	79.638	m2	1	kg/m2	1.020
Z-purlin (louver)	100 X 50 X 15 t=2mm	12.500	m	10	kg/m	0.450
Sub Total						6.068
other parts	20% of structural members					1.214
Total						7.282

RC / CHB

Item	Dimension	Quantity		Number	Unit weight	Sub Total
		Area/Length	Unit		Unit	(in ton)
Column	400 X 400	3.100	m	4	t/m	4.762
Column	300 X 400	3.100	m	4	t/m	3.572
Beam (X)	300 X 500	3.200	m	2	t/m	1.920
Beam (X)	300 X 500	3.600	m	2	t/m	2.592
Beam (Y)	300 X 500	3.450	m	4	t/m	4.968
Beam (Y)	300 X 500	3.000	m	2	t/m	2.160
Deck Slab	t=150	3.100	m2	1	t/m2	1.116
Deck Slab	t=150	4.200	m2	1	t/m2	1.512
Deck Slab beam	t=150	0.150	m2	5	t/m2	0.270
Wall (X)	H=2600, t=150	3.200	m	2	t/m	5.242
Wall (Y)	H=2600, t=150	3.450	m	4	t/m	11.303
Wall (Y)	H=2600, t=150	3.000	m	2	t/m	4.914
Sub Total						44.331
Total						44.331

Source: Study Team

2) Weight of the Gate Equipment

The weight of the gate facility is based on the results of the gate facility design in **Sub-section 7.5.3 Gate Facility Design** and it is decided as **Table7.5.10**.

Table7.5.10 Load of the Gate Equipment

Item	Loads	Remarks
Hoisting Force	30.0 kN	
Hoisting Device Weight (Include a rack bar)	12.0 kN	
Load acting on the guide frame	40.0 kN	
Weight of gate leaf	5 kN/m	
Weight of Guide Frame	7 kN	

Source: Study Team

3) Weight of the Guard house

The weight of the guardhouse is based on the results in **Sub-section 7.5 structural design of Building Structure**. As a result, it is 24.228 tons (see **Table7.5.11**). The weight per unit length used in the design is set to w = 30 kN/m2 considering that the calculated weight per unit weight is W = 2.8 t/m^2.

Table7.5.11 Weight of the Guard house

Item	Dimension	Quantity		Number	Unit weight		Sub Total (in ton)
		Area/Length	Unit		weight	Unit	
Column	250 X 250	2.750	m	4	0.15	t/m	1.650
Beam (X)	250 X 400	3.100	m	2	0.24	t/m	1.488
Beam (Y)	250 X 400	1.900	m	2	0.24	t/m	0.912
Roof Slab	t=100	15.640	m2	1	0.24	t/m2	3.754
Roof Parapet	t=100	3.600	m2	1	0.24	t/m2	0.864
Wall (X)	H=2300, t=150	3.100	m	2	0.7245	t/m	4.492
Wall (Y)	H=2300, t=150	1.900	m	2	0.7245	t/m	2.754
Wall (Y)	H=2600, t=100	3.200	m	1	0.546	t/m	1.748
Grade Beam (X)	300 X 500	3.000	m	2	0.36	t/m	2.160
Grade Beam (Y)	300 X 500	2.400	m	2	0.36	t/m	1.728
Floor Slab	t=200	5.400	m2	1	0.48	t/m2	2.592
Pedestorian Load		8.640	m2	1	0.35	t/m2	3.024
[Deduction of Soil Weight]							
Grade Beam (X)	300 X 400	3.000	m	-2	0.192	t/m	-1.152
Grade Beam (Y)	300 X 400	2.400	m	-2	0.192	t/m	-0.922
Floor Slab	t=100	5.400	m2	-1	0.16	t/m2	-0.864
Sub Total							24.228
Total							24.228

Source: Study Team

(5) Ground Condition

The ground conditions of Taytay Sluiceway (formation of strata and soil parameters) are set as shown in **Table7.5.12** and **Table7.5.21** in accordance with the results of geological surveys conducted in this study.

Table7.5.12 Soil Constant

Stratum	Soil Quality	N-value	Water Content Wn (%)	Fine Grain Fraction Fc (%)	Plasticity Index Ip	Unit Weight γt (kN/m3)	Cohesion c (kN/m2)	Shear Resistance Angle ϕ(°)	Deformation Coefficient E 50 (MN/m2)	Consolidation Settlement Target Layer	Compression Coefficient Cc	Swelling Coefficient Cc
As2	sandy soil	21	17	4		20	0	37				
Ac1 -1	cohesive soil	1	57	80	27	(15)	(14)	(0)	(1.5)	○	(1.17)	
Ac1 -2	cohesive soil	12	33	75	41	17	150	0				
Ac1 -3	cohesive soil	13	35	62	15	17	160	0				
Dc1 -1	cohesive soil	12	40	85	25	17	150	0				(0.056)
Dc1 -2	cohesive soil	10	60	88	33	17	120	0				
Dc1 -3	cohesive soil	25	27	82	21	18	310	0				
Dc1 -4	cohesive soil	35	32	64	21	19	430	0				
Dc1 -5	cohesive soil	50	50	60	37	19	620	0				

Source: Study Team

(6) Water Level Condition

The design water level of Taytay Sluiceway is summarized in **Table7.5.13**. The height of the flood plain required for setting the residual water level for the structural calculation of wing walls, etc. is set to EL+12.40 m from the present sand bar height.

Table7.5.13 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

Table7.5.14 Water Level of Manggahan Floodway

No	Station No.	Distance	Interval	DFL		Ordinary Water Level	Low Water Level	
					Annual (year)			
				DFL	WL _ 95 - Day	WL _ 185 - Day	WL _ 275 - Day	WL _ 355 - Day
		(m)	(m)	(E.L.m.)				
1	Sta. 10 + 000	-1000	200		11.77	11.32	10.94	10.74
2	Sta. 9 + 800	-800	200		11.77	11.31	10.94	10.73
3	Sta. 9 + 600	-600	200		11.77	11.31	10.94	10.73
4	Sta. 9 + 400	-400	200		11.77	11.31	10.94	10.73
5	Sta. 9 + 200	-200	200		11.77	11.31	10.94	10.73
6	Sta. 9 + 000	0	200		11.77	11.31	10.94	10.73
7	Sta. 8 + 800	200	200		11.76	11.31	10.94	10.72
8	Sta. 8 + 600	400	200		11.76	11.31	10.94	10.72
9	Sta. 8 + 400	600	200		11.76	11.31	10.94	10.72
10	Sta. 8 + 200	800	200	14.065	11.76	11.31	10.94	10.72
11	Sta. 8 + 000	1000	200	14.109	11.76	11.31	10.94	10.72
12	Sta. 7 + 800	1200	200	14.135	11.76	11.31	10.94	10.71
13	Sta. 7 + 600	1400	200	14.195	11.75	11.31	10.94	10.71
14	Sta. 7 + 400	1600	200	14.238	11.75	11.31	10.94	10.71
15	Sta. 7 + 200	1800	200	14.282	11.75	11.31	10.94	10.71
16	Sta. 7 + 000	2000	200	14.325	11.75	11.31	10.94	10.71
17	Sta. 6 + 800	2200	200	14.368	11.75	11.31	10.94	10.71
18	Sta. 6 + 600	2400	200	14.411	11.75	11.30	10.94	10.70
19	Sta. 6 + 400	2600	200	14.453	11.74	11.30	10.94	10.70
20	Sta. 6 + 200	2800	200	14.496	11.74	11.30	10.94	10.70
	Sta. 6 + 90		110	14.520	11.74	11.30	10.94	10.70
21	Sta. 6 + 000	3000	200	14.539	11.74	11.30	10.94	10.70
22	Sta. 5 + 800	3200	200	14.582	11.74	11.30	10.94	10.70
23	Sta. 5 + 600	3400	200	14.625	11.74	11.30	10.94	10.69
24	Sta. 5 + 400	3600	200	14.667	11.74	11.30	10.94	10.69

Source: Study Team

(7) Construction Condition

The major construction conditions for Taytay Sluiceways are shown in **Table7.5.15** and **Table7.5.19**.

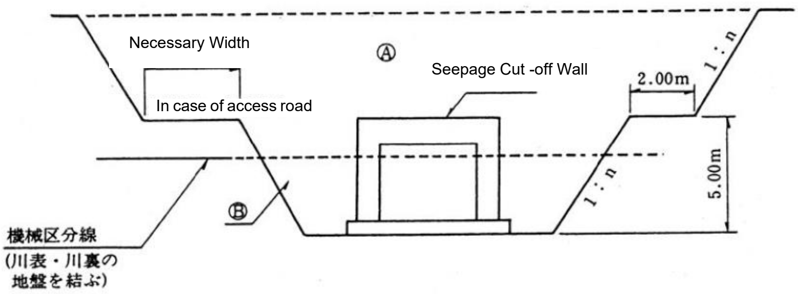
Table7.5.15 Construction Condition

Item	Condition	Remarks
Excavation Slope	Since there were many houses on the slope and the location where it was avoided was on the East Bank Road, geological survey could not be carried out on the existing embankment. Therefore, the excavation slope cannot be set by circular slip calculation. Accordingly with reference to "Guideline for Quantity Calculation, Japan", the excavation slope is set to 1.5:1 which is the value for sandy ground (see **Figure7.5.21**)	
Detour Road	The existing culvert (A) for erecting Taytay Sluiceway crosses the East Bank road. It is necessary to secure the traffic function during the construction.	
Houses Around the Site	In addition to the above road layout, the extent of improvement of Taytay Sluiceway in the longitudinal and transverse direction depends on the situation of surrounding houses. In the land side of the East Bank Road, (B) is not the target of relocation. On the other hand, in the floodway side, since they are located within the river channel, (C) is target of the house relocation	
Temporary Drainage	Not presented due to the closed information	

Source: Study Team

Source: Study Team

Figure7.5.20 Major Existing Structures around Taytay Sluiceway



Note) 1. The clearance is 0.5 m as a standard from the seepage cut off wall (In case of no seepage cut-off wall, it is 0.5m as a standard.
2. Side Slope (n) Sandy Ground 1.5:1
Clayey Ground 1.0 : 1
The above mentioned value is a standard, is case it is difficult to apply, it can be set based on the slip circle calculation
3. When snow fence is used, the necessary width shall be calculated

Source: Study Team Translated from Guideline for quantity calculation of civil works, 2019.4, p. 1.2. 8

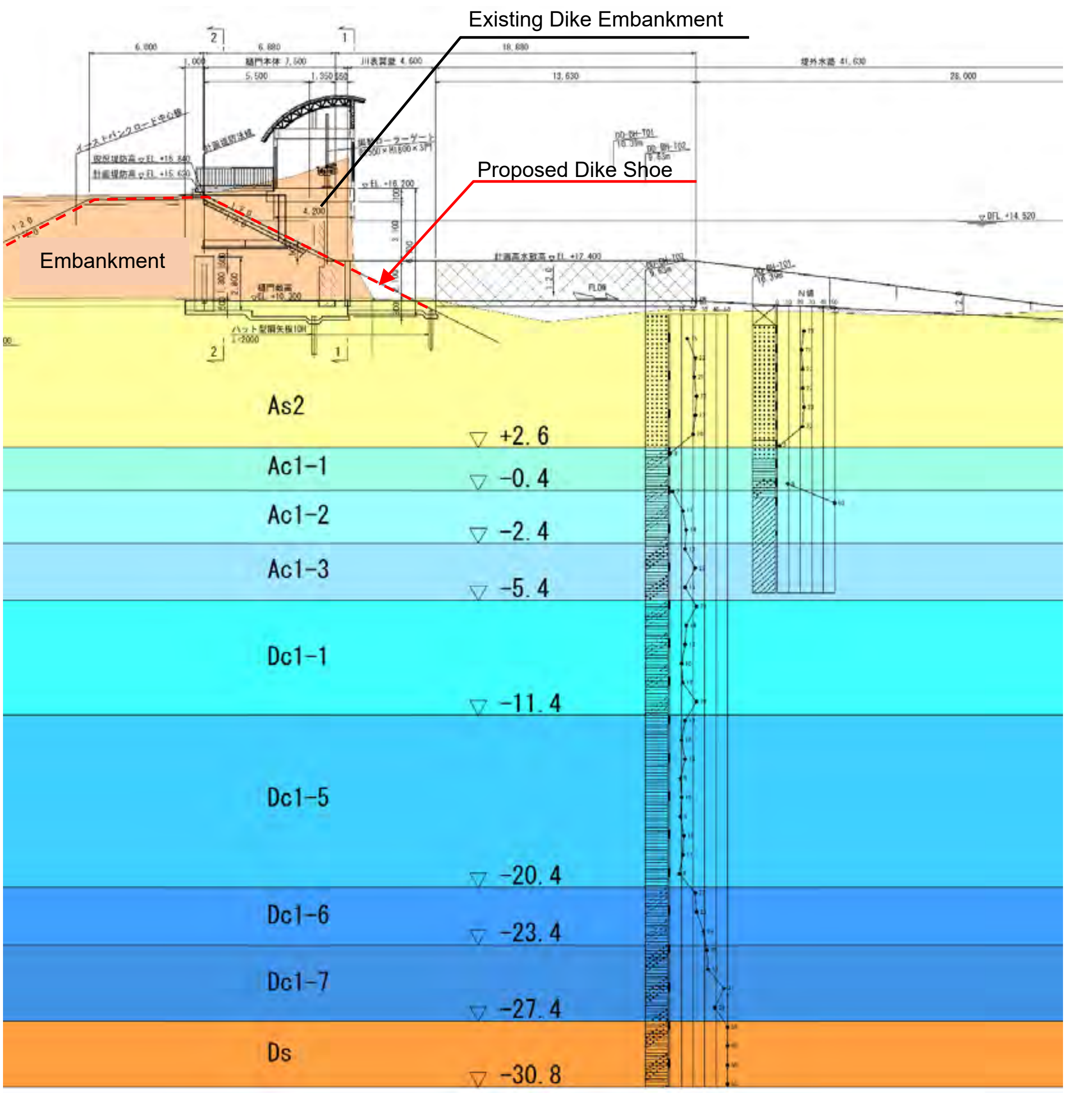
Figure7.5.21 Basis of Excavation Slope

7.5.2.3 Foundation Work

(1) Calculation policy for residual settlement

According to "Guideline for flexible Sluiceway", the settlement and lateral displacement are calculated by a simplified method. Since no new embankment load is generated and no consolidation settlement is generated in this sluiceway, only the immediate settlement is calculated.

□ Sandy soil: Immediate settlement and lateral displacement



Source: Study Team

Figure7.5.22 Existing Embankment

(2) Equation for Calculating Immediate Settlement

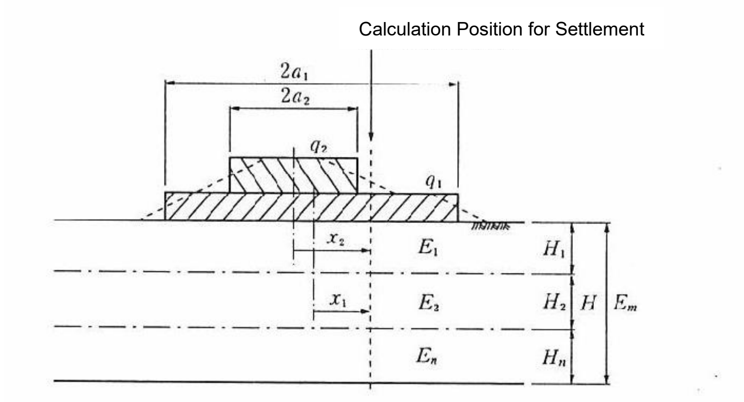
The formula for calculating the amount of immediate settlement is as follows.

$$Sx=\sum_{i=1}^{n}\frac{-3ai\cdot qi}{Em\cdot\pi}\log\cdot\sin\left(\tan^{-1}\frac{ai}{H}\right)\cdot\left\{1.0-\frac{0.75}{\pi}\left[\left(1+\frac{x}{ai}\right)\log\left|1+\frac{x}{ai}\right|+\left(1-\frac{x}{ai}\right)\log\left|1-\frac{x}{ai}\right|\right]\right\}$$ [2]

Where,

Sx	: Immediate settlement on x (m)
q_i	: Load of embankment (kN/m^2)
E_m	: Modulus of deformation of the ground (kN/m^2)
$2a_i$	: Load width (m)
H	: Depth for considering the effect of immediate settlement (m)
n	: Number of uniform loads
x	: Distance from the center of the ach uniform load (m)

[2] Guideline for flexible Sluiceway, H 10.11, p. 84



Source: Study Team Translated from Guideline for flexible Sluiceway, National Land Japan Institute of Country-ology and Engineering, H 10.11, p. 84 to 85 Guideline for flexible Sluiceway

Figure7.5.23 Formula for Calculating the Amount Of Immediate Settlement

(3) Formula for Calculating Lateral Displacement

Lateral displacement is horizontal displacement due to shear deformation of the ground caused by embankment. A formula for calculating the lateral displacement is given by **Figure7.5.24**.

$$R_{ix}=\sum_{i=1}^{n}\frac{-(1+v)(1-2v)q_i\cdot a_i}{E_m\cdot\pi}\left[\frac{b_i}{2a_i}\log\frac{(a_i-x)^2+b_i^2}{(a_i+x)^2+b_i^2}+\frac{a_i-x}{a_i}\tan^{-1}\frac{b_i}{a_i-x}-\frac{a_i+x}{a_i}\tan^{-1}\frac{b_i}{a_i+x}\right] \quad (1\text{-}5\text{-}13)$$

Where,

R_{ix}	: Lateral Displacement of Ground at x in the Culvert Axis Direction (m) {m}
q_i	: Embankment Load (tf/m2) {ken/m2}
E_m	: Coverted Deformation Coefficient of Ground (tf/m2) {ken/m2}
v	: Poisson Ratio, Normally about 0.3 to 0.45.
2 a_i	: Load Width (m) {m}, Width of Embankment B= 2 a_i
2 b_i	: Depth of Load) (m) {m}, Average Excavation Width L=2 b_i
n	: Number of Distribution Load
x	: Distance from Each Distribution Load (m) {m}

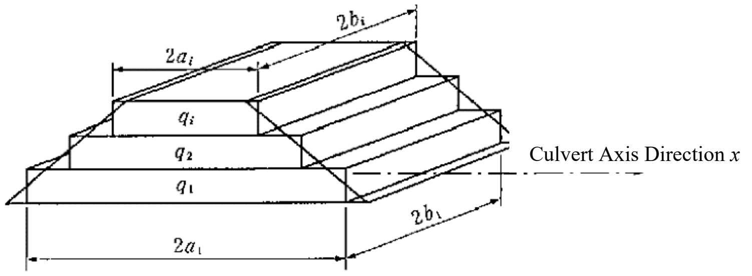


Figure 1-5-5 Calculation model for Lateral Displacement

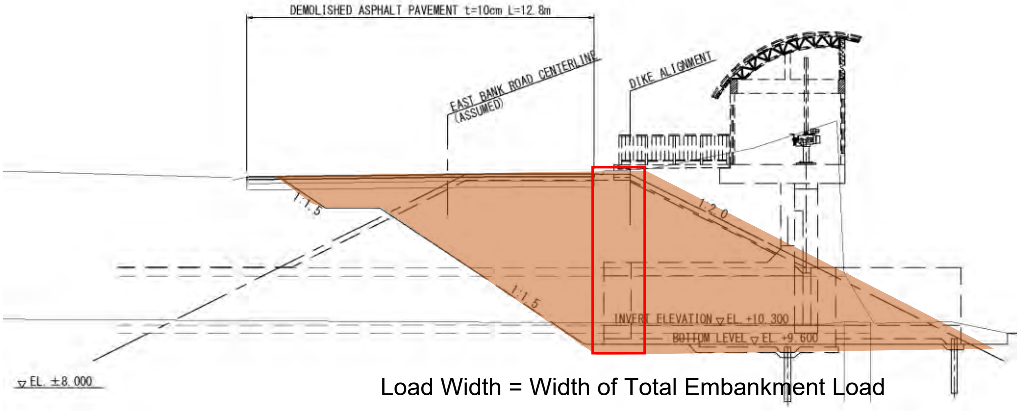
Source: Study Team Translated from Guideline for flexible Sluiceway, p. 87 to 88

Figure7.5.24 Formula for Calculating Lateral Displacement

(4) Setting the Settlement Target Layer

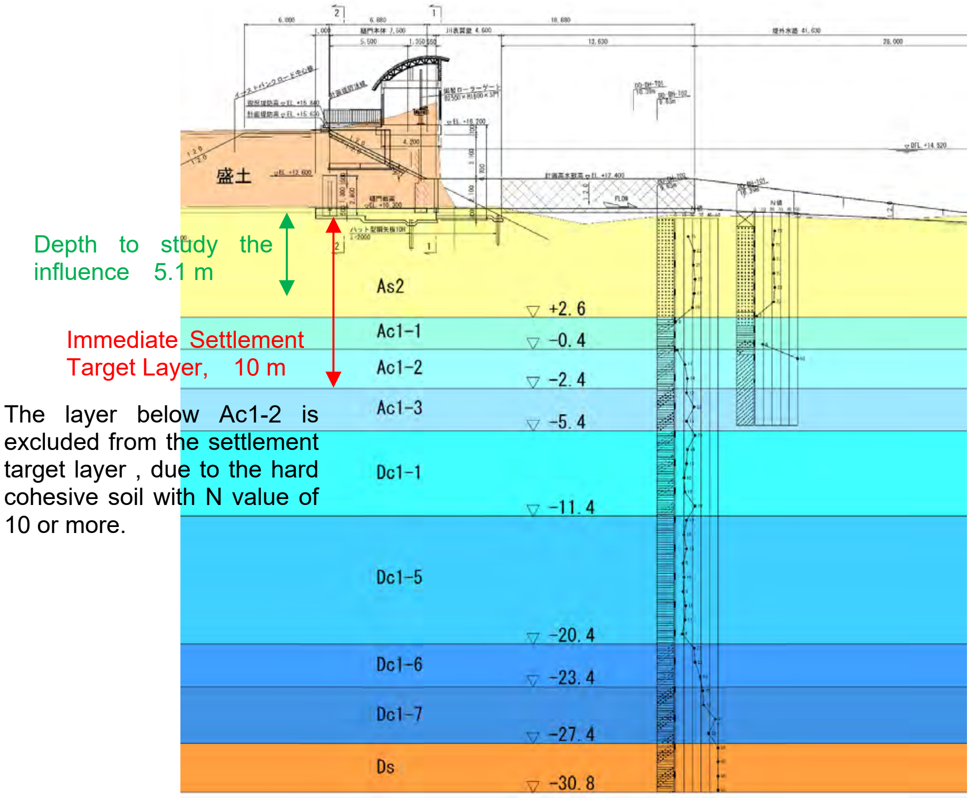
The target layer for immediate settlement is about 3 times of the load width, and the width where the total load acts from the top of the embankment to the bottom is 1.7 m in this study. Therefore, 1.7 m × 3 = 5.1 m or more is the depth to be considered.

Based on the above, the target layer for immediate settlement is as shown in **Figure7.5.26**. 2 layers below the subgrade which are the sand layer As2 (H = 7.0 m) and the cohesive soil Ac1 -1 (3.0 m) distributed below the sand layer As2 are considered as the settlement target layer. The cohesive soil layer below these layers has N value of about 10, which is harder than the Ac1 -1 layer. Hence, it was evaluated that settlement would not be expected.



Source: Study Team

Figure7.5.25 Area of Immediate Settlement



Source: Study Team

Figure7.5.26 Setting the Settlement Target Layer

(5) Calculation of the Converted Deformation Coefficient

The deformation coefficient of the ground is calculated as a converted deformation coefficient for the multi-layer ground as follows. About the deformation coefficient of the settlement target layer, E = 700 N x N value is used for the sand layer As2, and uniaxial compressive strength E 50 is used for the cohesive soil Ac1, and the converted deformation coefficient is calculated for the lower end of the Ac1-1 layer(Subgrade – about 10 m).

$$E_m = \frac{\log \dfrac{(B+2h_n \tan\theta)\cdot L}{(L+2h_n \tan\theta)\cdot B}}{\sum_{i=1}^{n} \dfrac{1}{E_{mi}} \log \dfrac{(B+2h_i \tan\theta)(L+2h_{i-1}\tan\theta)}{(L+2h_i \tan\theta)(B+2h_{i-1}\tan\theta)}}$$

Here,

E_m : Converted Deformation Coefficient Considering the Change of Ground when $B \neq L$ (kgf/cm^2) {kN/m2}

B : Load Width (m)

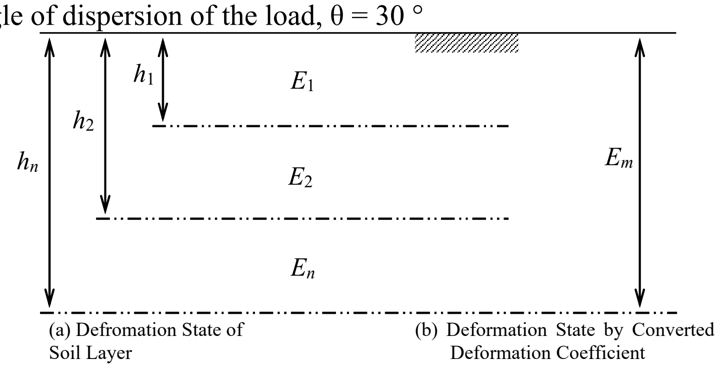
L : Load Depth (m)

h_n : Depth at which the Effect must be examined, 3 Times or More of the Loading Width B (m)

hi : Depth up to the Bottom of Each Layer (m)

E_{mi} : Converted Deformation Coefficient of i-th layer (kgf/cm2) {kN/m2}

θ : Angle of dispersion of the load, θ = 30 °



Source: Study Team Translated from Guideline for flexible Sluiceway, p. 79 to 80

Figure7.5.27 Deformation Factor when the Soil Layer Changes in the Depth Direction

The calculation results of the converted coefficient are shown in **Table7.5.16**. It is set to E_m = 5,400 kN/m^2.

Table7.5.16 Conversion Deformation Coefficient Calculation Table

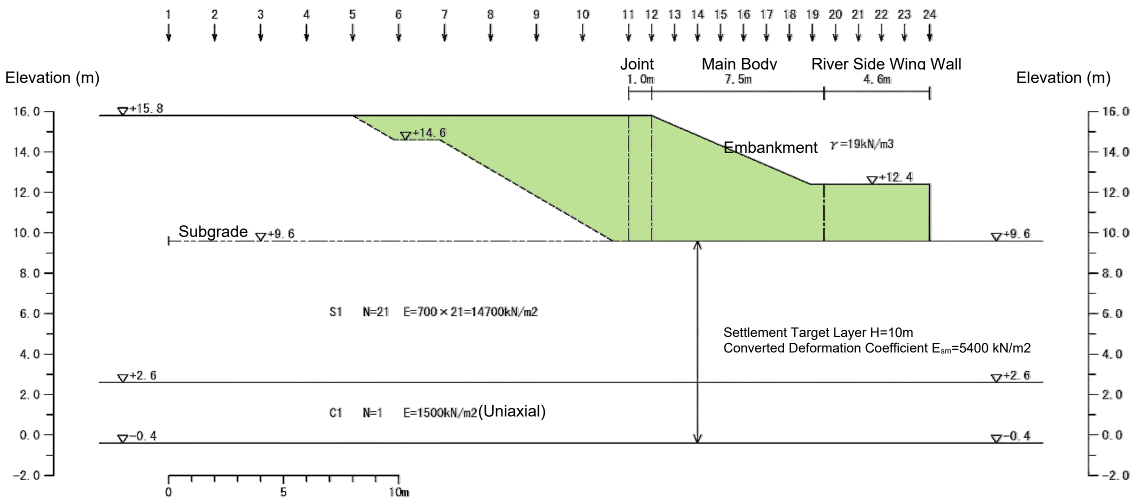
Layer Number	soil name	E_{oi} (kN/m2)	hi (m)	Numerator	Fraction
1	Sandy soil (As2)	14700	7.0	0.00000	
2	Cohesive soil (Ac1)	1500	10.0	-0.00001	
Subtotal			10.0	-0.00001	-0.05552
Loading Width B (m)		*Maximum load width (lowest load)			15.90
Loading Depth L (m)		*Excavation Width			14.00
Converted Deformation Coefficient E_{sm} (kN/m2)					5420.8
					(5400)

Source: Study Team

(6) Calculation Model

The settlement calculation model is shown in **Figure7.5.28**.

As2

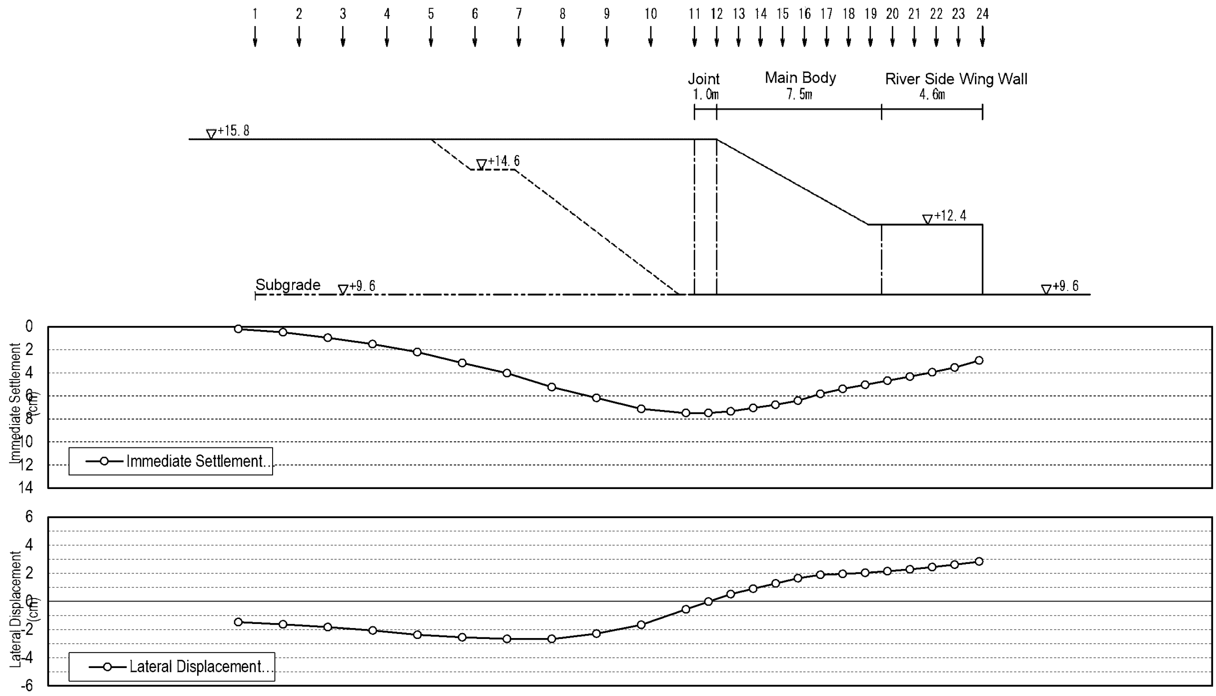


Source: Study Team

Figure7.5.28 Overall Model Diagram

(7) Calculation of Residual Settlement

The residual settlement compose of only the immediate settlement. The settlement is calculated as shown in **Figure7.5.29** the vertical displacement (settlement) is below 10 cm at 7.6 cm.



calculation point	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Coordinates (m)	-20.0	-18.0	-16.0	-14.0	-12.0	-10.0	-8.0	-6.0	-4.0	-2.0	0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.1
Immediate Settlement (cm)	0.2	0.5	1.0	1.5	2.2	3.2	4.0	5.3	6.2	7.1	7.5	7.5	7.4	7.1	6.8	6.4	5.8	5.4	5.1	4.7	4.3	4.0	3.6	2.9
Lateral Displacement (cm)	-1.5	-1.6	-1.8	-2.1	-2.4	-2.6	-2.7	-2.7	-2.3	-1.7	-0.6	0.0	0.5	0.9	1.3	1.6	1.9	2.0	2.0	2.1	2.3	2.4	2.6	2.8

Source: Study Team

Figure7.5.29 Settlement Diagram

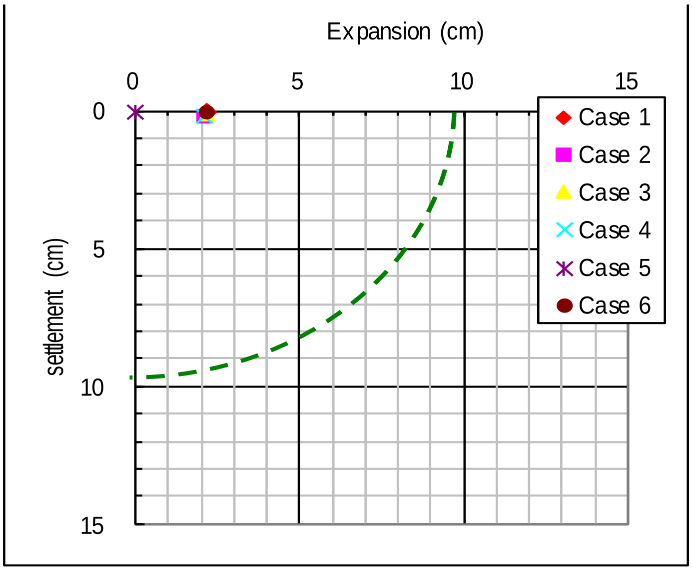
(8) Design of Foundation Work

Since the maximum settlement is 10 cm or less, direct foundation is adopted, and its structure is set to a flexible sluiceway.

(9) Joint Structure

The existing box is assumed to be a pile foundation structure, and the new culvert becomes a direct foundation. Since the foundation structure is likely to be different, a flexible joint is arranged in the connection part.

From the result of the structural calculation of box culvert in the longitudinal direction, as shown in **Figure7.5.30**, since the settlement and opening of the culvert are 10 cm or less, the required capacity of flexible joint is set to 10 cm (minimum specification).



Source: Study Team

Figure7.5.30 Verification Results of Flexible Joint

7.5.2.4 Main Body Work

(1) Stability Analysis of Main Body

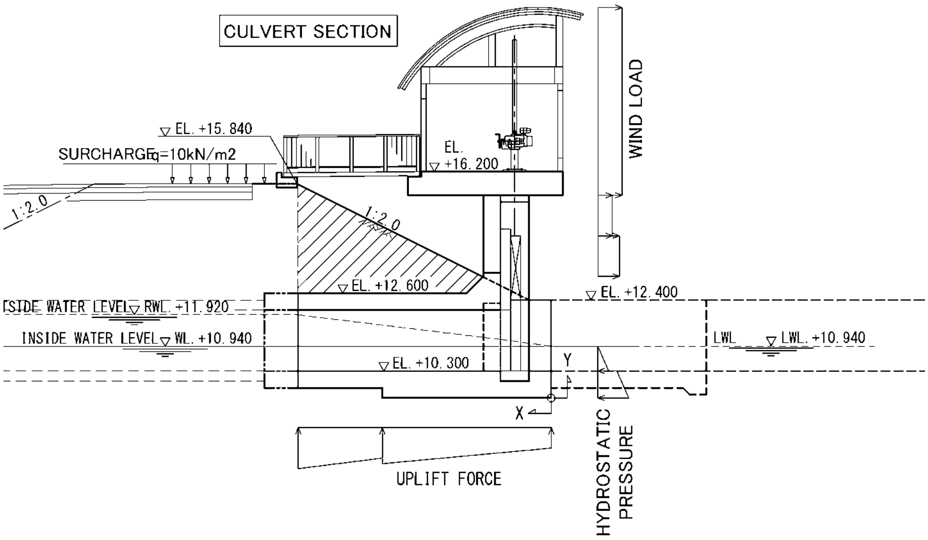
1) Study Policy

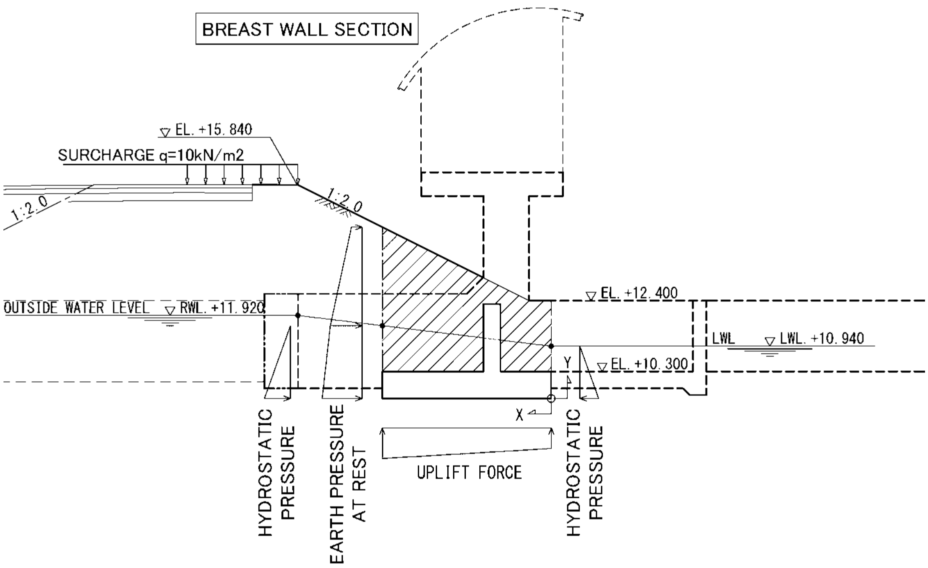
Since the span length of the sluiceway main body is short, the safety against overturning, sliding and bearing capacity is checked. The load acting on the foundation shall be the composited load of the column and the breast wall.

2) Load Case

The calculation model diagram is shown in **Figure7.5.31**, the calculation case is indicated in **Table7.5.17**.

＜Normal Condition＞

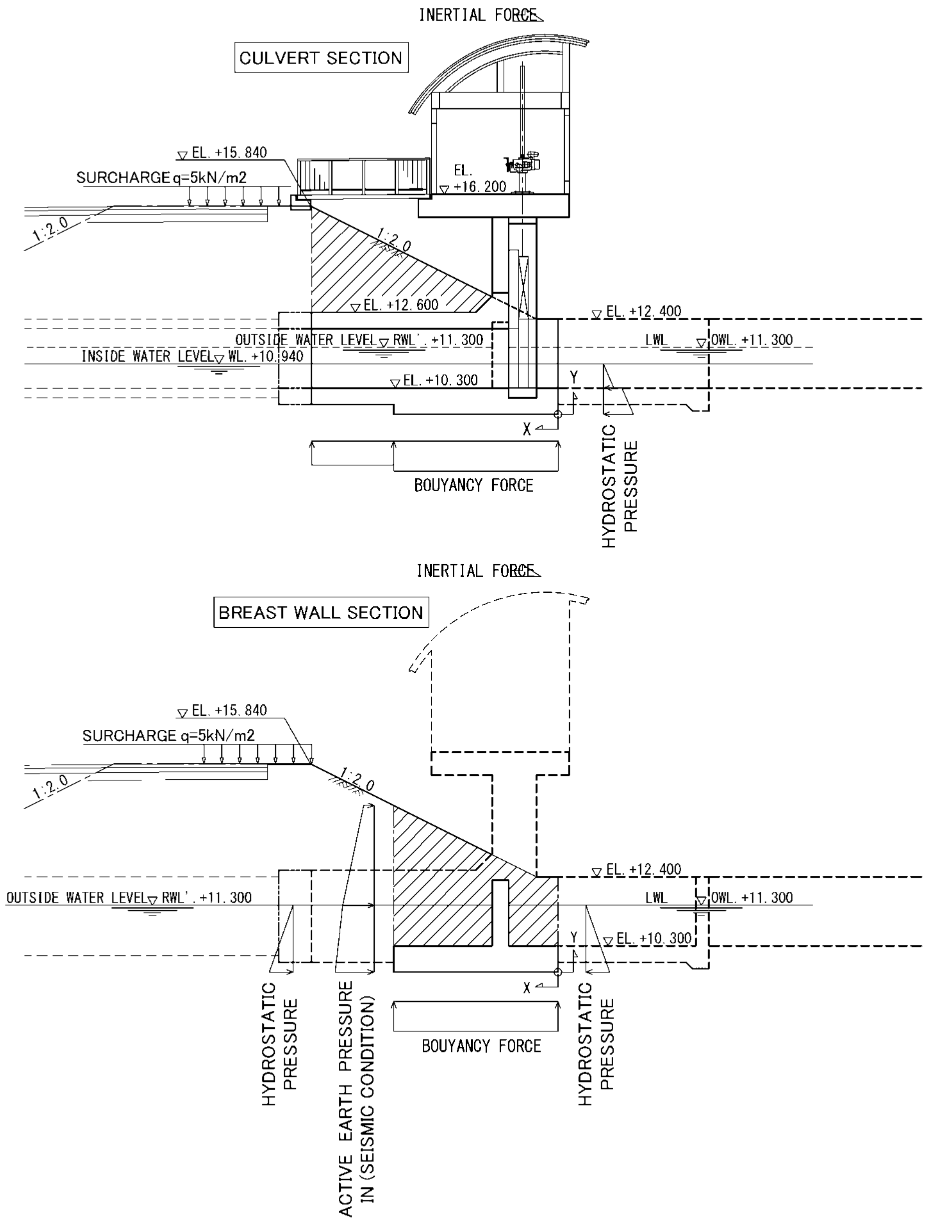




Source: Study Team

Figure7.5.31 Main Body Stability Analysis Model Diagram (Normal Condition)

<Seismic Condition>



Source: Study Team

Figure7.5.32 Main Body Stability Analysis Model Diagram (Seismic Condition)

Table7.5.17 List of Calculation Cases (Normal Condition, L1 Seismic Condition)

		1	2	3	4	5	6	7	8	9
		Dead weight	Local control house load	Gate load	Wind load	Hydrostatic pressure	Earth pressure at rest	Seismic active earth pressure	Uplift pressure	Internal water weight
Normal Condition	CASE1	○	○	○	○	○	○		○	○
L1 Seismic Condition	CASE2	○	○	○		○		○	○	○

Legend : ○···Considered
Source: Study Team

3) Design Water Level

The design water level is set based on the basic water level information (see**Table7.5.18**).

Table7.5.18 List of Design Water Levels

<table>
<tr><th colspan="3">Item</th><th>Water Level</th><th>Remarks</th></tr>
<tr><td rowspan="6">Basic water level</td><td colspan="2">Design Flood Level</td><td>DFL = + 14.52</td><td></td></tr>
<tr><td colspan="2">Natural Groundwater Level</td><td>GWL = + 10.94</td><td></td></tr>
<tr><td colspan="2">Ordinary Water Level</td><td>OWL = + 11.30</td><td></td></tr>
<tr><td colspan="2">Low Water Level</td><td>LWL = + 10.94</td><td>Same as GWL</td></tr>
<tr><td rowspan="2">Water Level in Front Side</td><td>Normal Condition</td><td>WL = + 11.03</td><td>Same as LWL</td></tr>
<tr><td>Seismic Condition</td><td>WL = + 11.30</td><td>Same as OWL</td></tr>
<tr><td colspan="3">Ground Height GL Behind Structures</td><td>GL = + 12.40</td><td></td></tr>
<tr><td rowspan="5">On the outer structure surface (Rear) design water level</td><td rowspan="2">Normal Condition (Aside from Breast Wall)</td><td rowspan="2">Residual Water Level</td><td>REL = (GL-LWL) × 2/3 + LWL</td><td>The case of HWL > GL</td></tr>
<tr><td>= (12.40 -10.94) x 2/3 + 10.94 = + 11.92</td><td></td></tr>
<tr><td rowspan="2">Normal Condition (For Breast Wall)</td><td rowspan="2">Residual Water Level</td><td>RWL= (GL－LWL)×2/3 ＋ LWL</td><td>The case of HWL>GL</td></tr>
<tr><td>= (13.08－10.94)×2/3＋10.94 = +12.37</td><td></td></tr>
<tr><td>Seismic Condition</td><td>Water Level in Rear Side′</td><td>RWL '= + 11.30</td><td>Since GWL < OWL, set to OWL</td></tr>
<tr><td rowspan="2">Design Water Level in Front of The Structure (Breast Wall)</td><td>Normal Condition</td><td>Low Water Level</td><td>LWL = + 10.94</td><td></td></tr>
<tr><td>Seismic Condition</td><td>Ordinary Water Level</td><td>OWL = + 11.30</td><td></td></tr>
<tr><td>Design Water Level Inside the Structure</td><td>Normal Condition
Seismic Condition</td><td>Low Water Level</td><td>LWL = + 10.94</td><td>In the stability Analysis, LWL is used considering the safe side both in Normal and Seismic Condition.</td></tr>
</table>

**Low water level in the floodway (Manggahan Floodway) = 10.94*
**Ordinary water level of the floodway (Manggahan Floodway) = 11.30*
Source: Study Team

4) Summary of Loads

In order to examine the stability of the sluiceway main body, the loads acting on the culvert, on the breast wall and on the column are summarized and composited to examine the stability of CASE1: Normal Condition and CASE2: Seismic Condition. The summary of the loads is shown in **Table7.5.19**. The details are given in **Vol.5A Structural Calculation for Contract Package-1**.

Table7.5.19 Summary of Load

CASE1 Normal Condition

Composited Load Schedule

	Vertical Force W (kN)	Horizontal Force H (kN)	X (d) (m)	Y (m)	W · X (kN · m)	H · Y (kN · m)
Culvert Section	3873.29	296.78	4.13	0.00	15996.69	0.00
Breast Wall section (per 2 Locations)	616.96	393.84	1.47	0.00	906.93	0.00
Column Section	2070.78	383.03	1.41	3.10	2919.80	1187.39
Σ	6561.03	1073.65			19823.42	1187.39

Column portion lower end moment M = 1141.07 kN · m (clockwise)

CASE2 Seismic Condition (Culvert Section: Inertial Force of Body and Embankment is Considered.)

Composited Load Schedule

	Vertical Force W (kN)	Horizontal Force H (kN)	X (d) (m)	Y (m)	W · X (kN · m)	H · Y (kN · m)
Culvert Section	4035.20	1211.19	3.67	0.00	14809.18	0.00
Breast Wall section (per 2 Locations)	697.62	469.34	1.72	0.00	1199.91	0.00
Column Section	1788.08	378.34	1.41	3.10	2521.19	1172.85
Σ	6520.90	2058.87			18530.28	1172.85

Moment at Lower End of Column M = 825.46 kN · m (clockwise)

Source: Study Team

5) Stability Analysis

Stable analysis of the direct foundation shall be performed for overturning, sliding and bearing capacity. Since the unit of the allowable bearing capacity is kN, the value converted into the bearing capacity is also used. The list of results is shown in **Table7.5.20**. All the calculated values satisfies the allowable value.

Table7.5.20 List of Results of Stability Analysis

Case			CASE1 Normal Condition (+Wind Load)	CASE2 Seismic Condition	Remarks
Load	Vertical Force	ΣW (kN)	6561.03	6520.90	Composited Load
	Horizontal Force	ΣH (kN)	1073.65	2058.87	
	Moment by Vertical Force	ΣW · X (kN · m)	19823.42	18530.28	
	Moment by Horizontal Force	ΣH · Y (kN · m)	1187.39	1172.85	
	Moment at the Lower End of Column	M (kN · m)	1141.07	825.46	Clockwise
Shape	Foundation width	B (m)	7.50		
	Foundation length	L (m)	11.70		Equality in terms of length
Overturning	The acting point of the Composited Force	$d = \frac{\Sigma W \cdot X - \Sigma H \cdot Y - \text{Column } M}{\Sigma W}$	2.67	2.54	
	Eccentric Distance	e = B / 2-d (m)	1.08	1.21	Front Side
	Allowable Value	ea (m)	1.25	2.50	
	Evaluation (Absolute Value)		**OK**	**OK**	
Sliding	Coefficient of Friction	f	0.60	0.60	
	Cohesion	C (kN / m2)	0	0	
	Effective Load Width	B '= B-2e (m)	5.34	5.08	
	Effective Load Area	A '= B' · L (m2)	62.48	59.44	
	Safety Factor	F = (ΣW · f + C · A ') / ΣH	3.67	1.90	
	Acceptable safety factor	Fs	1.5	1.2	
	Determine constant		**OK**	**OK**	
Bearing	Vertical Force on Bottom Surface: e ≦ B / 6, $q = \frac{\Sigma W}{B \cdot L} \cdot (1 \pm 6e / B)$	q1 (kN / m2)	139.37	146.25	
		q2 (kN / m2)	10.17	2.38	
		ΣW (kN)	6561.03	6520.90	
	Allowable Bearing Capacity	qa (kN / m2)	318	232	
		Qa (kN)	19856	13798	
	Evaluation		**OK**	**OK**	

Allowable Capacity	Item		Symbol	Unit	Formula	Value	Remarks
	The amount of eccentricity	Normal Condition	ea	m	B / 6	1.25	
		Seismic Condition			B / 3	2.50	
	Support force	CASE1 Normal Condition	Qa	kN	From Calculation of Allowable Bearing Capacity (taking into account the slope ratio of load)	19856	
		CASE2 Seismic Condition				13798	
	Support force of	CASE1 Normal Condition	qa	kN / m2	Bearing Force / Effective Loading Area = Qa / A '	318	A'1 = 62.48
		CASE2 Seismic Condition				232	A'2 = 59.44

※ A'1 ~ A'2 → from the calculation of the allowable bearing capacity of each case

Foundation Length L: setting to the converted length to have equal area.

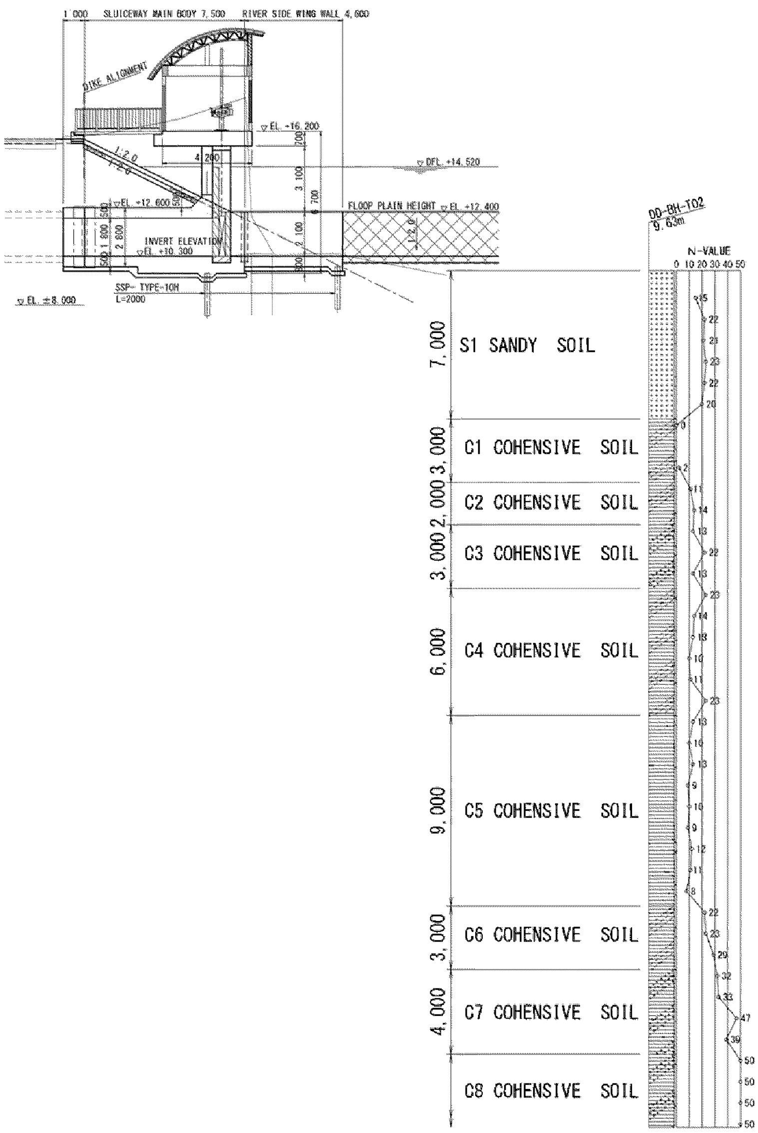
→ A = 2.50 × 9.10 + 5.00 × 13.00 = 87.75m2

∴ L = A / B = 87.75 / 7.50 = 11.70m

Source: Study Team

6) Bearing Capacity

(a) Design Soil Constant



Geological Survey Data

Borehole: DD-BH-T02

Strata		Design N value	Unit Weight Saturated)	Cohesion	Angle of internal friction	Remarks
			γ (kN / m3)	C (kN / m2)	φ (degrees)	
1	S1 sandy soil	21	20	0	37	
2	C1 cohesive soil	1	15	14	0	
3	C2 cohesive soil	12	17	150	0	
4	C3 cohesive soil	13	17	160	0	
5	C4 cohesive soil	12	17	150	0	
6	C5 cohesive soil	10	17	120	0	
7	C6 cohesive soil	25	18	310	0	
8	C7 cohesive soil	35	19	430	0	
9	C8 cohesive soil	50	19	620	0	

(b) Calculated Result

CASE1 Normal Condition (+Wind Load)

Item	Symbol	Unit	Formula	Value	Remarks
N value of the ground	N			21	S1 sandy soil
Internal Friction Angle of the Ground	φ	°		37	
Cohesion	C	kN / m2		0.00	
Depth from ground surface to bottom of footing	Df	m	Riverbed high - bottom version of the bottom surface height = 10.30 - 9.50	0.80	
Effective Embedded Depth of Foundation	D'f	m		0.00	
Unit Weight of Soil of support ground	γ1	kN / m3	20.0 - 9.8	10.2	S1 sandy soil water weight
Unit Weight of Soil embedment ground	γ2	kN / m3	20.0 - 9.8	10.2	
Foundation width	B	m		7.50	
Depth in Longitudinal Direction of the foundation	D	m		11.70	Equality in terms of width
Horizontal Force acting on the Bottom Surface	HB	kN		1073.65	
Vertical Force acting on the Bottom surface	V	kN		6561.03	
Eccentricity of the Load	eB	m		1.08	
Eccentricity of the Load in Longitudinal Depth	eD	m		0.00	
Effective Width of Foundation	Be	m	B - 2eB	5.34	
Effective Longitudinal Depth of Foundation	De	m	D - 2eD	11.70	
Shape factor	α		1.0 + 0.3 · Be / De	1.14	Be / De> 1 → Be / De = 1
	β		1.0 - 0.4 · Be / De	0.82	
Coefficient of Embedment	κ		1.0 + 0.3 · D'f / Be	1.00	
Slope ratio of the load	tanθ		HB / V	0.16	
Surcharge	q	kN / m2	γ2 · Df	8.16	
Effective loading area	A '	m2	Be · De	62.48	
Bearing capacity factors	Nc			45.0	
	Nq			32.0	
	Nr			31.0	
Ultimate bearing capacity	Qu	kN	A '(α · κ · c · Nc + κ · q · Nq + 1/2 · γ1 · β · Be · Nr)	59569	
Bearing Capacity in Normal Condition	Qa	kN	1/3 · Qu	19856	

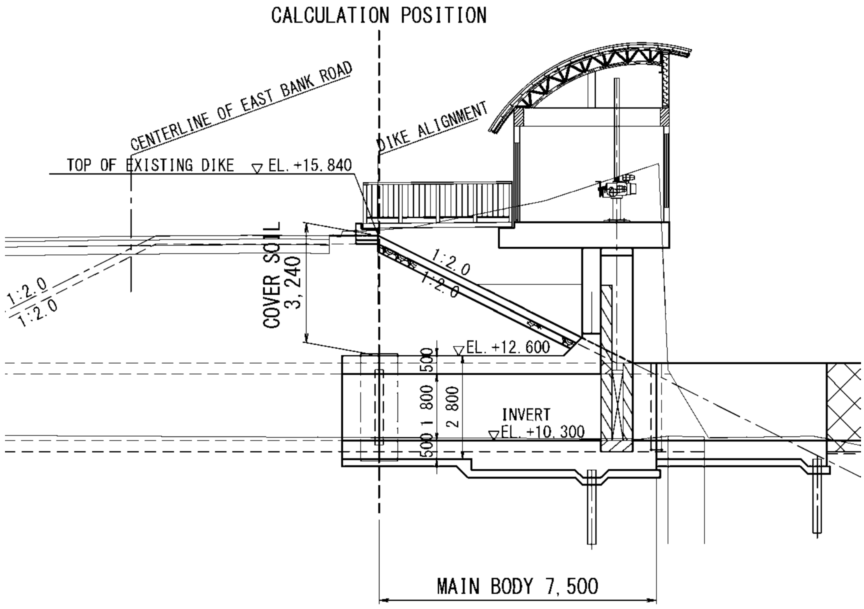
CASE2 Seismic Condition

Item	Symbol	Unit	Formula	Value	Remarks
N value of the ground	N			21	S1 sandy soil
Internal Friction Angle of the Ground	φ	°		37	
Cohesion	C	kN / m2		0.00	
Depth from ground surface to bottom of footing	Df	m	Riverbed high - bottom version of the bottom surface height = 10.30 - 9.50	0.80	
Effective Embedded Depth of Foundation	D'f	m		0.00	
Unit Weight of Soil of support ground	γ1	kN / m3	20.0 - 9.8	10.2	S1 sandy soil water weight
Unit Weight of Soil embedment ground	γ2	kN / m3	20.0 - 9.8	10.2	
Foundation width	B	m		7.50	
Depth in Longitudinal Direction of the foundation	D	m		11.70	Equality in terms of width
Horizontal Force acting on the Bottom Surface	HB	kN		2058.87	
Vertical Force acting on the Bottom surface	V	kN		6520.90	
Eccentricity of the Load	eB	m		1.21	
Eccentricity of the Load in Longitudinal Depth	eD	m		0.00	
Effective Width of Foundation	Be	m	B - 2eB	5.08	
Effective Longitudinal Depth of Foundation	De	m	D - 2eD	11.70	
Shape factor	α		1.0 + 0.3 · Be / De	1.13	Be / De> 1 → Be / De = 1
	β		1.0 - 0.4 · Be / De	0.83	
Coefficient of Embedment	κ		1.0 + 0.3 · D'f / Be	1.00	
Slope ratio of the load	tanθ		HB / V	0.32	
Surcharge	q	kN / m2	γ2 · Df	8.16	
Effective loading area	A '	m2	Be · De	59.44	
Bearing capacity factors	Nc			30.0	
	Nq			20.0	
	Nr			14.0	
Ultimate bearing capacity	Qu	kN	A '(α · κ · c · Nc + κ · q · Nq + 1/2 · γ1 · β · Be · Nr)	27595	
Bearing Capacity in Seismic Condition	Qa	kN	1/2 · Qu	13798	

(2) Design of Box Culvert in Transverse Direction

1) Design Policy

The box culvert in the transverse direction is designed as a box frame. In addition, this sluiceway is the one that flows down naturally from the Taytay Creek to the Manggahan Floodway, so the cross section is calculated at the position where maximum thickness of soil covers. The position of the cross section for calculation is shown in **Figure7.5.33**.

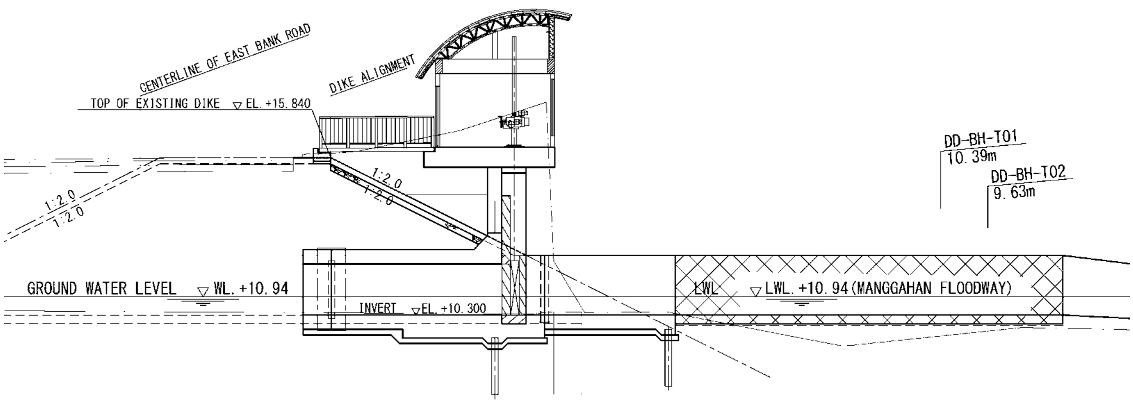


Source: Study Team

Figure7.5.33 Section Checking Position

2) Design Water Level

Since the boring carried out in this sluiceway design is water boring and there is no water level in the hole, the low water level of the Manggahan Floodway is set as the groundwater level.



Source: Study Team

Figure7.5.34 Design Water Level

3) Calculation Case

The calculation cases are total seven cases shown in **Table7.5.21**.

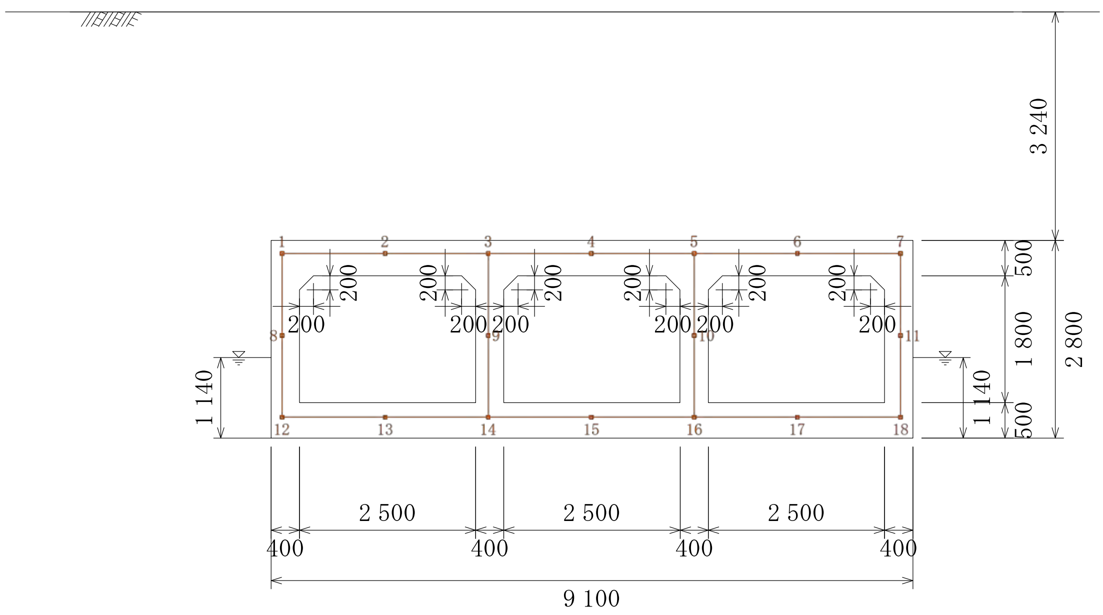
Table7.5.21 Calculation Case

Calculation case	Load Condition
Case 1	Dead load
Case 2	Dead load + Live Load (Lateral Pressure)
Case 3	Dead load + Live Load (Load of Rear Wheel Acts on Left Side Center)
Case 4	Dead load + Live Load (Load of Rear Wheel Acts on Center)
Case 5	Dead load + Live Load (Load of Rear Wheel Acts on the Left Side Wall)
Case 6	Dead load + Live Load (Load of Rear Wheel Acts on the Left Side Separation Wall)
Case 7	Dead load + Live Load (Load of Rear Wheel Acts on the Right Side Separation Wall)

Source: Study Team

4) Study Model

The study model is shown in **Figure7.5.35**.



Source: Study Team

Figure7.5.35 Study Model

5) Calculated Result

(a) Bending Stress

Bedding stress of the top slab, left wall, left side separation wall, right separation wall, right wall, and bottom slab is indicated in **Table7.5.23**.

Table7.5.22 Bending Stress (1)

Top Slab

		Unit	Left Corner	Left Span	Left Center Conner	Center Span	Right Center Conner	Right Span	Right Conner
Thickness		mm	500						
Cover Concrete		mm	90						
Section Force	Bending Moment	KN・m	32.309	43.746	69.110	27.894	72.015	44.767	32.309
	Axial Force	KN	47.537	44.538	44.538	45.513	40.790	40.790	47.537
Reinforcement As		mm2	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566
Stress Intensity	σc	N/mm2	1.94	2.66	4.22	1.66	4.40	2.72	1.94
	σs	N/mm2	47.29	72.22	124.98	39.01	132.65	75.92	47.29
Allowable Stress Intensity	σca	N/mm2	8.28	8.28	8.28	8.28	8.28	8.28	8.28
	σsa	N/mm2	168.00	168.00	168.00	168.00	168.00	168.00	168.00
Critical Case			Case 4	Case 3	Case 3	Case 6	Case 7	Case 7	Case 4

Left Side Wall

		Unit	Upper Corner	Span	Bottom Corner
Thickness		mm	400		
Cover Concrete		mm	90		
Section Force	Bending Moment	KN・m	32.309	1.584	37.525
	Axial Force	KN	104.590	102.095	111.287
Reinforcement As		mm2	D20@250 12.566	D16@250 8.042	D20@250 12.566
Stress Intensity	σc	N/mm2	2.85	0.32	3.35
	σs	N/mm2	44.78	1.94	56.06
Allowable Stress Intensity	σca	N/mm2	8.28	8.28	6.21
	σsa	N/mm2	168.00	168.00	168.00
Critical Case			Case 4	Case 2	Case 2

Left Separation Wall

		Unit	Upper Conner		Bottom Corner	
			Outside Tensile	Inside Tensile	Outside Tensile	Inside Tensile
Thickness		mm	400			
Cover Concrete		mm	90			
Section Force	Bending Moment	KN・m	7.381	0.339	5.021	2.894
	Axial Force	KN	226.087	262.646	285.619	243.367
Reinforcement As		mm2	D16@250 8.042	D16@250 8.042	D16@250 8.042	D16@250 8.042
Stress Intensity	σc	N/mm2	0.85	0.68	0.91	0.72
	σs	N/mm2	3.59	5.65	5.31	4.77
Allowable Stress Intensity	σca	N/mm2	8.28	8.28	6.21	6.21
	σsa	N/mm2	168.00	168.00	168.00	168.00

Source: Study Team

Table7.5.23 Bending Stress (2)

Right Separation Wall

		Unit	Upper Corner		Bottom Corner	
			Outside Tensile	Inside Tensile	Outside Tensile	Inside Tensile
Thickness		mm	400			
Cover Concrete		mm	90			
Section Force	Bending Moment	KN・m	0.339	9.895	2.400	5.370
	Axial Force	KN	262.646	249.036	266.316	279.926
Reinforcement As		mm2	D16@250 8.042	D16@250 8.042	D16@250 8.042	D16@250 8.042
Stress Intensity	σc	N/mm2	0.68	1.00	0.76	0.91
	σs	N/mm2	5.65	3.64	5.36	5.13
Allowable Stress Intensity	σca	N/mm2	8.28	8.28	6.21	6.21
	σsa	N/mm2	168.00	168.00	168.00	168.00
Critical Case			Case 4	Case 7	Case 7	Case 4

Right Side Wall

		Unit	Upper Corner	Span	Bottom Corner
Thickness		mm	400		
Cover Concrete		mm	90		
Section Force	Bending Moment	KN・m	32.309	1.584	39.226
	Axial Force	KN	104.590	102.095	128.300
Reinforcement As		mm2	D20@250 12.566	D16@250 8.042	D20@250 12.566
Stress Intensity	σc	N/mm2	2.85	0.32	3.45
	σs	N/mm2	44.78	1.94	53.86
Allowable Stress Intensity	σca	N/mm2	8.28	8.28	6.21
	σsa	N/mm2	168.00	168.00	168.00
Critical Case			Case 4	Case 2	Case 7

Bottom Slab

		Unit	Left Corner	Left Span	Left Center Conner	Center Span	Right Center Conner	Right Span	Right Conner
Thickness		mm	500						
Cover Concrete		mm	90						
Section Force	Bending Moment	KN・m	37.414	46.972	75.213	28.208	74.682	47.119	39.226
	Axial Force	KN	63.611	57.084	57.084	59.993	55.986	58.273	62.733
Reinforcement As		mm2	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566	D20@250 12.566
Stress Intensity	σc	N/mm2	2.23	2.84	4.59	1.65	4.55	2.85	2.34
	σs	N/mm2	51.29	73.69	132.37	33.92	131.74	73.51	55.37
Allowable Stress Intensity	σca	N/mm2	6.21	8.28	6.21	8.28	6.21	8.28	6.21
	σsa	N/mm2	168.00	168.00	168.00	168.00	168.00	168.00	168.00

Source: Study Team

(b) Shear Stress

The shear stress of the top plate, side wall, partition, and bottom plate is indicated in **Table7.5.24**.

Table7.5.24 Shear Stress

Top Slab

		Unit	Left Corner, Left τ	Left Center Corner, Right τ	Left Center Coner, Left τ	Right Center Corner, Right τ	Right Center Coner, Left τ	Right Corner, Right τ
Thickness		mm	500					
Cover Concrete		mm	90					
Shear Force		KN	44.271	68.783	54.095	56.416	71.410	44.271
Stress Intensity	τm	N/mm2	0.108	0.168	0.132	0.138	0.174	0.108
Allowable Stress Intensity	τa	N/mm2	0.36	0.36	0.36	0.36	0.36	0.36
Critical Case			Case 4	Case 3	Case 6	Case 6	Case 7	Case 4

Side Wall

		Unit	Left Wall		Right Wall	
			Upper τ Point	Lower τ Point	Upper τ Point	Lower τ Point
Thickness		mm	400			
Cover Concrete		mm	90			
Shear Force		KN	25.155	29.392	25.155	28.514
Stress Intensity	τm	N/mm2	0.081	0.095	0.081	0.092
Allowable Stress Intensity	τa	N/mm2	0.36	0.36	0.36	0.36
Critical Case			Case 4	Case 7	Case 4	Case 7

Separation Wall

		Unit	Left Wall		Right Wall	
			Upper τ Point	Lower τ Point	Upper τ Point	Lower τ Point
Thickness		mm	400			
Cover Concrete		mm	90			
Shear Force		KN	4.467	4.467	5.346	5.346
Stress Intensity	τm	N/mm2	0.014	0.014	0.017	0.017
Allowable Stress Intensity	τa	N/mm2	0.36	0.36	0.36	0.36
Critical Case			Case 7	Case 7	Case 7	Case 7

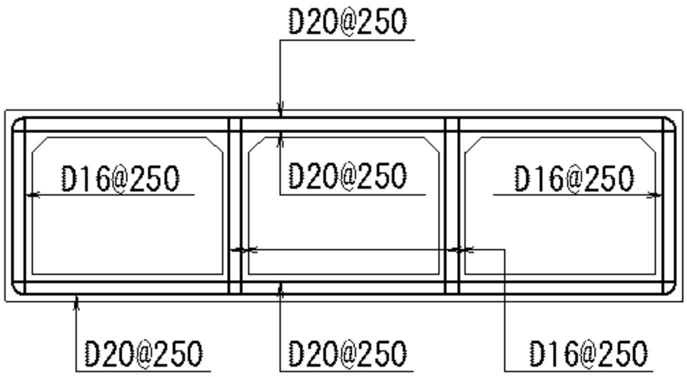
Bottom Slab

		Unit	Left Corner, Left τ	Left Center Corner, Right τ	Left Center Coner, Left τ	Right Center Corner, Right τ	Right Center Coner, Left τ	Right Corner, Right τ
Thickness		mm	500					
Cover Concrete		mm	90					
Shear Force		KN	46.377	74.620	58.313	56.701	74.408	48.927
Stress Intensity	τm	N/mm2	0.113	0.182	0.142	0.138	0.181	0.119
Allowable Stress Intensity	τa	N/mm2	0.36	0.36	0.36	0.36	0.36	0.36
Critical Case			Case 3	Case 5	Case 5	Case 5	Case 6	Case 7

Source: Study Team

6) Bar Arrangement

The bar arrangement based on the calculation result is shown.



Source: Study Team

Figure7.5.36 Bar Arrangement

(3) Longitudinal Design of the Box

The calculation of the longitudinal direction of the culvert was carried out according to the Guideline for Flexible Sluiceway.

1) Design Policy

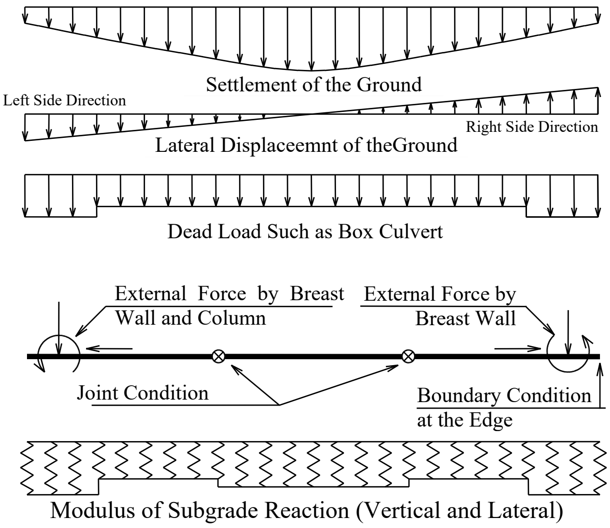
Since the amount of settlement was 5 cm or more, the following description was given: "A beam on an elastic floor considering ground displacement.[3]is modeled as.

$$\frac{EI}{B}\cdot\frac{d^4w}{dx^4}+k_v(w-w_g)=q \quad \text{(Swag)}$$

$$\frac{EA}{U}\cdot\frac{d^2u}{dx^2}+k_s(u-u_g)=p \quad \text{(Displacement in axial direction of Box culvert)}$$

Where,

w	: Displacement of box culvert (Swag : m)
w_g	: Displacement of the ground (residual settlement : m)
k_v	: Modulus of subgrade reaction in vertical direction (kN/m^3)
B	: Width of box culvert (m)
EI	: Rigidness of box culvert (kN・m^2)
q	: Vertical external force (kN/m^2)
u	: Displacement on axial direction of box culvert (m)
w_g	: Displacement of ground (lateral displacement : m)
k_s	: Modulus of lateral shear subgrade reaction (kN/m^3)
U	: Circumference of box culvert (m)
EA	: Rigidness of box culvert in axial direction (kN)
p	: External force in axial direction (kN/m^2)



Source: Study Team Translated from Guideline for Flexible Sluiceway 7.6.2 Figure 1-7-17

Figure 7.5.37 Calculation Model of a Beam on Elastic Foundation in Consideration of Ground Displacement

2) Calculation Case

The calculation case was set to 6 cases in total, consisting of 3 cases when the box was empty, when

[3] Guide to the Design of Flexible Sluice Gates 7.6 Longitudinal Design of the Box

the box was full of water, and 2 cases of wind load at the time of earthquake.

Table7.5.25 Calculation Case

Study Case	Load Condition	Water Level Condition		Direction of Inertial Force	Wind Load Direction	Load Condition								
		Inside Water Level	Rear Side of Breast Wall Residual Water Level			Dead Weight of Culvert	Dead Load of Column	Dead Weight of River Side Breast Wall	Earth Pressure and Water Pressure acts on River Side Breast Wall in the River Side	Internal Water Weight	Live Load (Operation Deck)	Live Load (Dike Crown)	Wind Load	Earth Weight
1	Normal Condition	+10.94	+10.94		→	○	○	○	○	○	○	○	○	-
2	Normal Condition	+10.94	+10.94		←	○	○	○	○	○	○	○	○	-
3	Normal Condition	+12.10	+11.92		→	○	○	○	○	○	○	○	○	-
4	Normal Condition	+12.10	+11.92		←	○	○	○	○	○	○	○	○	-
5	Seismic Condition	+11.30	+11.30	→		○	◎	◎	◆	○	-	○	-	-
6	Seismic Condition	+11.30	+11.30	←		○	◎	◎	◆	○	-	○	-	-

○: Considered
◎- Also consider inertia forces
◆: Consider the active earth pressure in Seismic Condition
◇: Consider earth pressure at rest
-: Not considered

Source: Study Team

(a) Water Level Condition

The water level conditions are shown in **Table7.5.26**.

Table7.5.26 Water Level Conditions for Longitudinal Calculation

Item	Water Level	Remarks
Natural Groundwater Level GWL	+10.94	LWL of Manggahan Floodway
Ordinary Water Level	+11.30	OWL of Manggahan Floodway
Culvert Full Water Level	+12.10	Lower Side of Top Slab of Culvert (Invert Elevation + 10.3 m) + (Inner Height 1.80) = + 12.100
Residual Water Level	+11.94	Residual Water Level (Normal Condition) 2/3 Between the Ground Level Behind the Structure And the Ground Water Level (Low Water Level) 2/3 {(Groundwater level + 10.94) - (Background + 12.40)} + (+ 10.94) = + 11.92 m

Source: Study Team

(b) Inertial Force

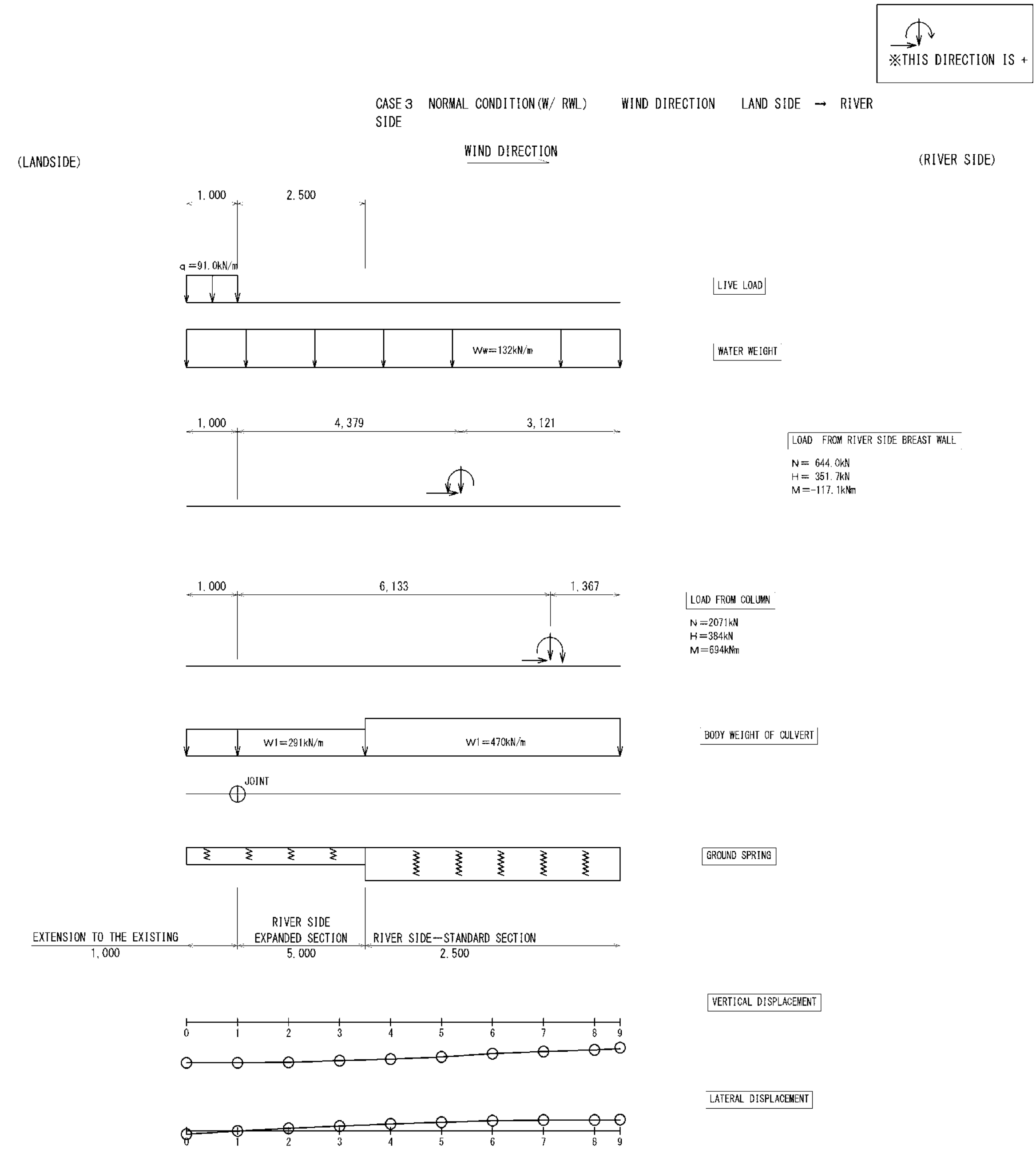
The designed horizontal seismic coefficient is kh = 0.2. The direction of inertia force is set to ← : From the River Side to the Land Side, → : From the Land Side to the River Side.

(c) Wind Direction

The direction of the wind load is from the river side to the land side, and from the land side to the river side.

3) Calculation Model

The calculation model is shown in **Figure7.5.38**.

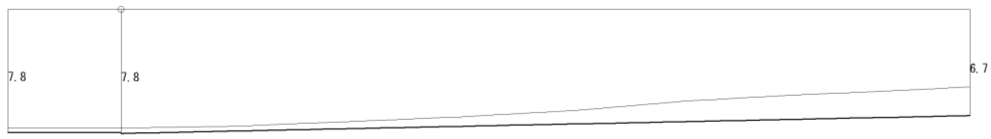


Source: Study Team

Figure7.5.38 Calculation Model Diagram (Case 3)

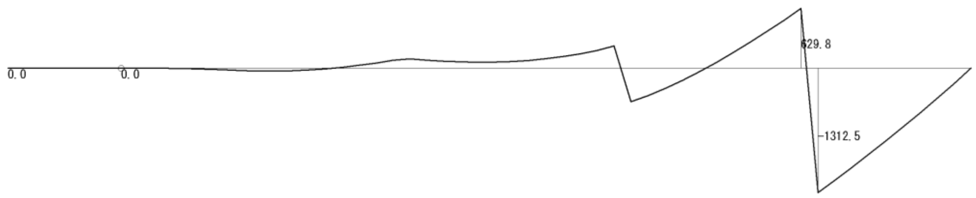
4) Calculation of Cross-Sectional Force

As a result of the calculation of the cross-sectional force, the critical case is shown in **Figure7.5.39**.



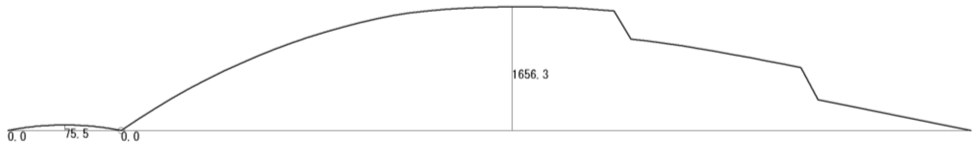
Settlement of Culvert / Settlement of Ground (cm)

Bending Moment (kNm))



Shear Force (kN)

Displacement in Culvert Axis Direction/ Ground Displacement



Axial Force (kN)

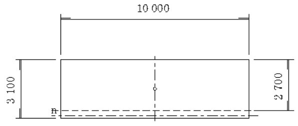
Source: Study Team

Figure7.5.39 Calculation Result of Cross Section Force Diagram (Case 3)

5) Stress Check Result

As a result of stress degree checking, a result of the critical case is shown in **Figure7.5.40**.

Title: Case- 2 : Maximum Axial Force Load- 3 : Section Force Case3



A (m^2)	31.000
A' (m^2)	0.000
yu (m)	1.550
yl (m)	-1.550
Iz (m^4)	24.826
Iy (m^4)	258.333
Wu (m^3)	16.017
Wl (m^3)	-16.017
J (m^4)	79.924
Ao (m^2/m)	26.200
Ai (m^2/m)	0.000

Section Force M (kN.m)	71.000
N (kN)	-1656.000
S (kN)	1313.000
Web Width bw (m)	10.000
Effective Height d (m)	0.400
Shear Span s (m)	0.000
Stress Intensity σc σca	8.843 < 10.350
(N/mm^2) σs1 σsa1	198.233 < 210.000
Neutral Axis X (m)	R0.1146
Ratio of Young's Modulus	n = 9.00
Allowance τa (N/mm2)	0.450
Average τm (N/mm2)	0.33 < 0.45
Averageτamax (N/mm2)	3.200
Suc Crushing Strength (kN)	12800.000 > 1313.000
τmax (N/mm^2)	0.060
σl σla (N/mm^2)	-0.10 < -0.80
Vo (m)	1.550
Shear Asreq (cm2)	19.029
Interval of Diagonal Tension Bara (cm)	100.000
Area Aw (cm2)	0.000
Angel θ (°)	90.000
Diagonal Tensile Fracture Sc (kN)	0.000
Diagonal Tensile Fracture Ss (kN)	0.000
Diagonal Tensile Fracture Strength Sus (kN)	0.000 < 1313.000

Type of Steel	Position (m)	Diameter (mm)	Nos. of Rebars (Piece)	Reinforcement As (cm^2)
D1	2.700	20.00	108.000	339.120
Total Reinforcement Σ			339.120	

《Legend of Type of Steel》
D:Deformed 1(φ:Round) E:Defromed2(φ:Round)
P:PC steel1 R:PC steel2
S:Steel Plate Q:Outer Cable C:Carbon Fiber
1:Upper Edge to Height 0:Perimeter
-1:Upper and Lower Cover -2:Left and Right Cover

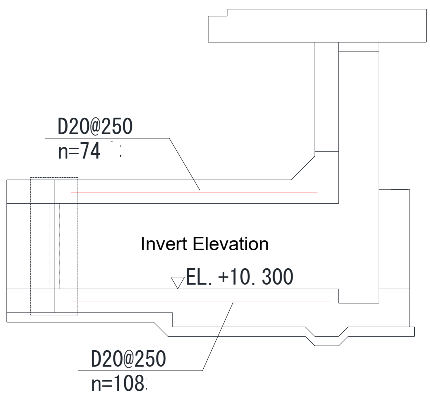
Note: R means turn around of section.

Source: Study Team

Figure7.5.40 Stress Check Result

6) Bar Arrangement Procedure

The bar arrangement is shown in **Figure7.5.41**. The required amount of reinforcement was similar to the minimum amount of reinforcement.

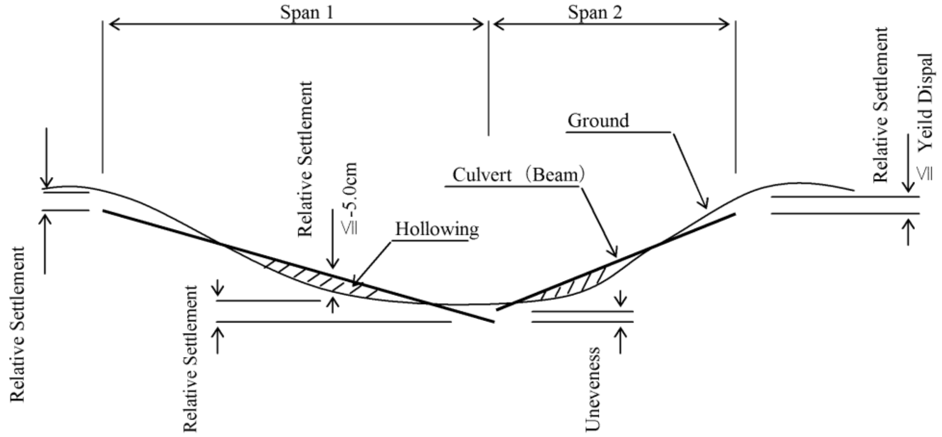


Source: Study Team

Figure7.5.41 Bar Arrangement

7) Checking the Cavity and Sinking

The stability of the sluiceway main body against the bearing is checked based on the calculation results in the longitudinal direction of the main body. There are positive (sinking) and negative (hollowing out) values for the relative settlement between the sluiceway main body and the ground, and it is checked that each value is within the allowable value. As a result of checking, since the relative settlement satisfies the allowable value, the foundation does not need any treatment.



Positive Relative Displacement
Yielding Displacement of Ground ≦ 5.0 cm
≦ 1.0% of Width of Foundation

Negative Relative Displacement
Amount of Hollowing ≦ -5.0 cm

Source: Study Team Summarized Based on "Guideline for Flexible Sluiceway"

Figure7.5.42 Conceptual Drawing of Checking the Amount of Cavity and Sinking

Table7.5.27 Verification of Bearing Capacity of Foundation Ground (Case 3)

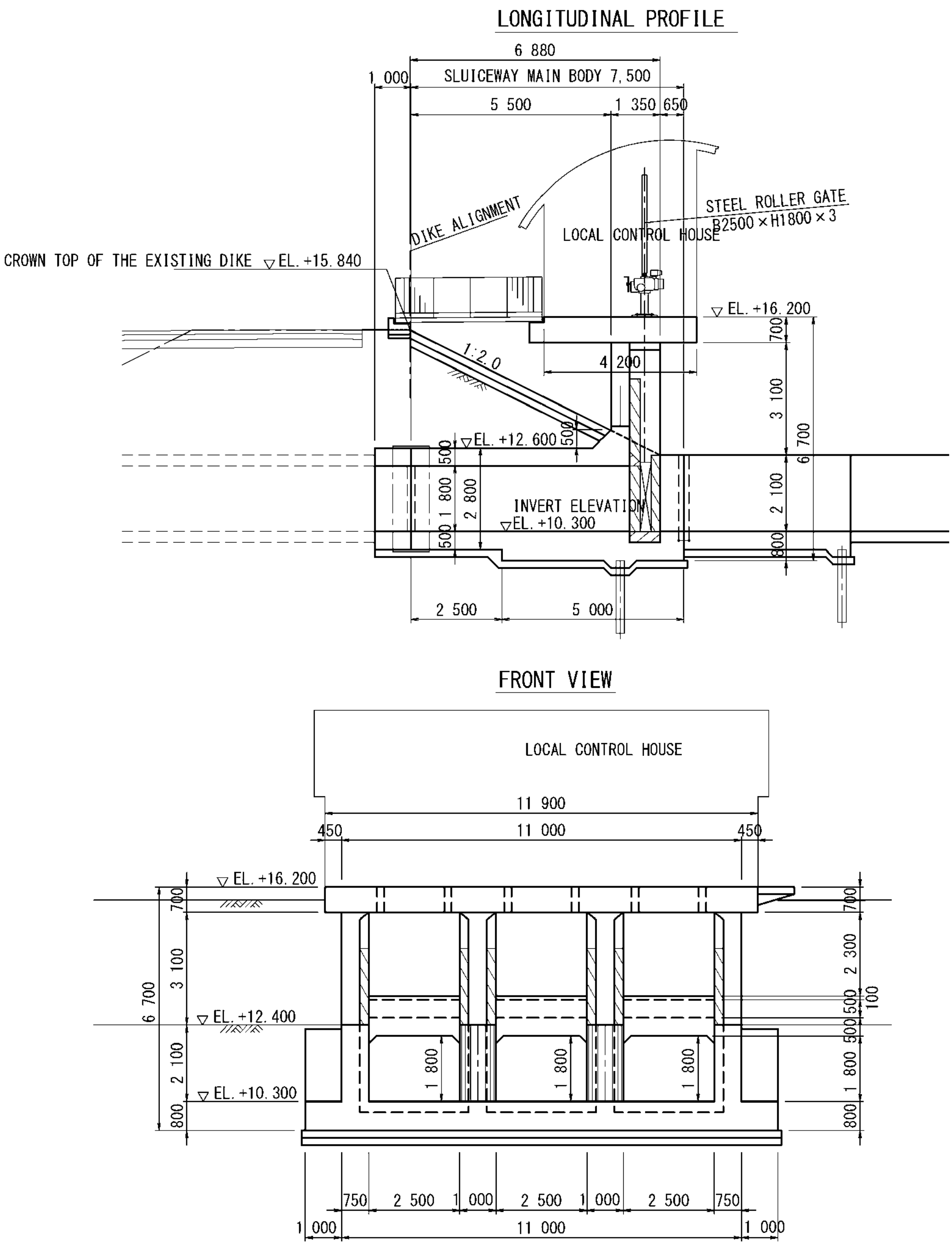
	Fulcrum number	Position (m)	Case (3) normal condition Settlement (I) Ground (cm)	Settlement (II) Culvert (cm)	Relative settlement (II) - (I) (cm)	Allowance for relative Displacement (cm) Positive (Sinking)	Allowance for relative Displacement (cm) Negative (Hollowing out)	Result Positive (Sinking)	Result Negative (Hollowing out)
Joint Part	1	0.000	7.5	7.8	0.3	5.0	-5.0	OK	OK
	2	0.500	7.5	7.8	0.3	5.0	-5.0	OK	OK
	3	1.000	7.5	7.8	0.3	5.0	-5.0	OK	OK
Main Body	1	0.000	7.5	7.8	0.3	5.0	-5.0	OK	OK
	2	0.750	7.4	7.7	0.3	5.0	-5.0	OK	OK
	3	1.500	7.2	7.6	0.4	5.0	-5.0	OK	OK
	4	2.250	7.0	7.5	0.5	5.0	-5.0	OK	OK
	5	3.000	6.8	7.4	0.6	5.0	-5.0	OK	OK
	6	3.450	6.6	7.3	0.7	5.0	-5.0	OK	OK
	7	3.750	6.5	7.3	0.8	5.0	-5.0	OK	OK
	8	4.500	6.1	7.2	1.1	5.0	-5.0	OK	OK
	9	5.250	5.7	7.1	1.4	5.0	-5.0	OK	OK
	10	6.000	5.4	6.9	1.5	5.0	-5.0	OK	OK
	11	6.150	5.4	6.9	1.5	5.0	-5.0	OK	OK
	12	6.750	5.2	6.8	1.6	5.0	-5.0	OK	OK
	13	7.500	4.7	6.7	2.0	5.0	-5.0	OK	OK

Source: Study Team

(4) Column Design

1) Design Policy

The column is designed with the top plate of the culvert as a fixed end, the lateral direction as a portal frame, and the vertical direction as a cantilever. Normal Condition examine the case of wind load, temperature change, and earthquake. The gate shall be in the suspended state in normal condition and at the time of earthquake. The standard load conditions are shown on and after the next page.



Source: Study Team

Figure7.5.43 Dimensions of Column

2) Calculation of Transverse Direction (Perpendicular direction to the flow)

As a portal frame fixed to the top plate of the culvert, the following conditions are examined in normal, seismic, wind load, and temperature change conditions.

(a) Calculation Case

The 5 calculation cases is are shown in **Table7.5.28**

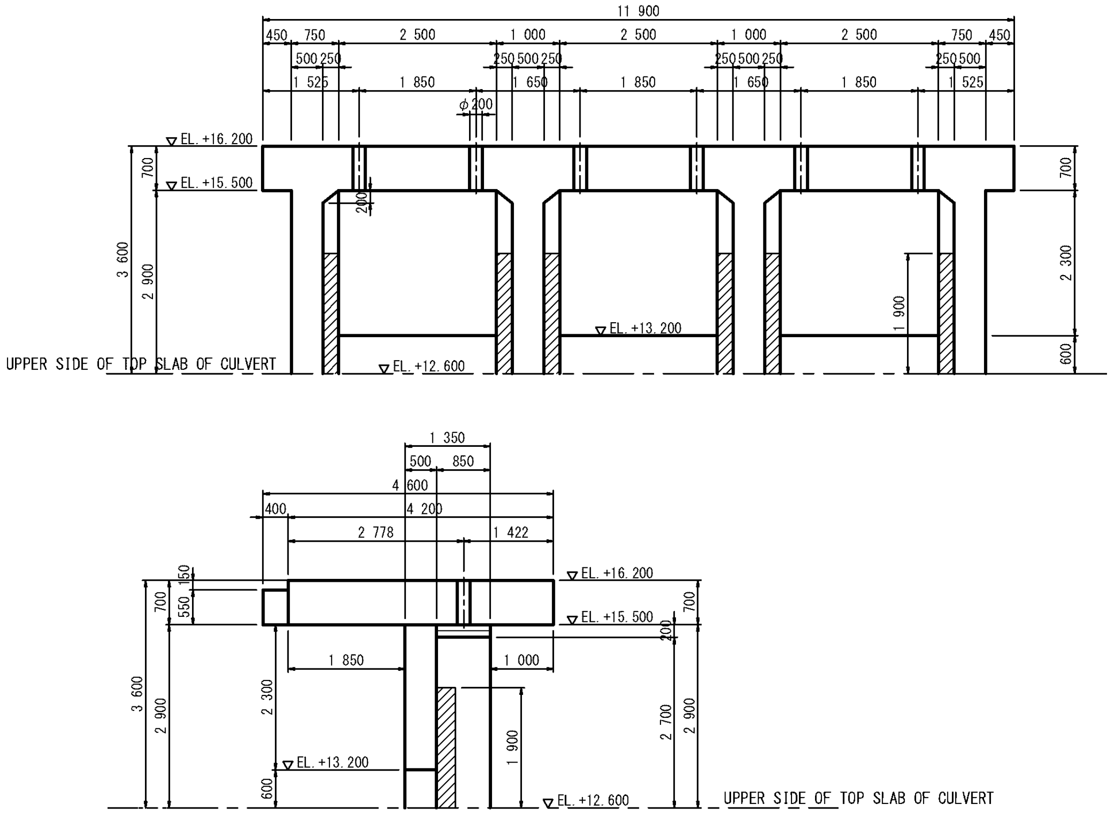
Table7.5.28 Load Combination in Transverse Direction

load case		1	2	3	4	5	Remarks
Normal Condition	Ordinary Time	○	○	○	○		
	Wind Load		○		○		4.17 kN/m2
	Temperature change			○	○		+16.7℃、－22.2℃
Seismic Condition						○	

Legend : ○···Considered
Source: Study Team

(b) Computational Model

The geometric dimensions are shown in **Figure7.5.44**.

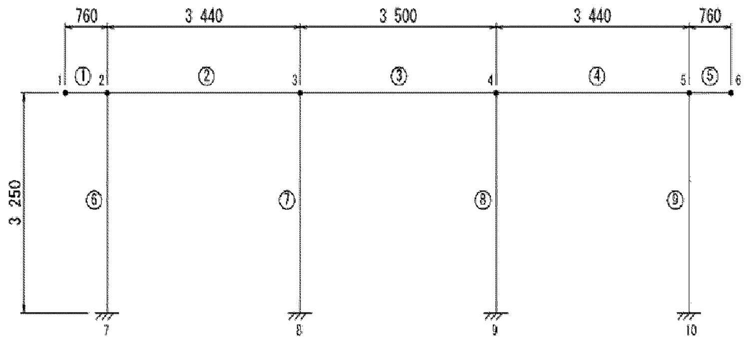


Source: Study Team

Figure7.5.44 Member Dimensions in Calculation in Transverse Direction

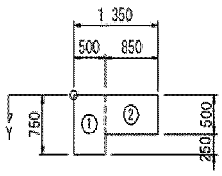
The member specifications are as shown in**Figure7.5.45** and **Figure7.5.46**

① Rigid Frame Axis



② Cross-sectional Area and Moment of Inertia of Member

・End Column



Cross section specification

Symbol	Vertical dimension h m	Horizontal dimension b m	Area A = h, b m2	arm Y m	A. Y m3	A. Y2 m4	Moment of Inertia of Area In m4
①	0.75	0.50	0.375	0.375	0.1406	0.0527	0.01758
②	0.50	0.85	0.425	0.250	0.1063	0.0266	0.00885
Σ			0.800		0.2469	0.0793	0.02643

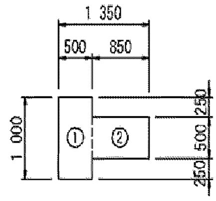
Calculation table of cross section area and moment of inertia

Name	Symbol	Units	Formula	Value	Remarks
Cross-sectional area	A	m2		0.800	
Distance from Centroid to Top	y	m	ΣA/Y/ΣA	0.30863	
Horizontal axis Moment of inertia of area	Ix	m4	ΣA・Y2 + ΣIo-ΣA・y2	0.02953	

Source: Study Team

Figure7.5.45 Component Specifications (1)

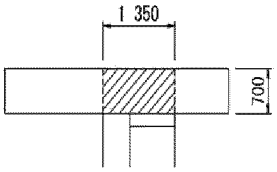
・Center Column



Calculation table of member cross section and moment of inertia

Item		Symbol	Units	Formula	Value	Remarks
①	Horizontal dimension	b	m		0.50	
	Vertical dimension	h	m		1.00	
	Cross-sectional area	A1	m2	b ・ h	0.500	
	Horizontal Axis, Moment of inertia of area	I1	m4	1/12・b・h3	0.04167	
②	Horizontal dimension	b	m		0.85	
	Vertical dimension	h	m		0.50	
	Cross-sectional area	A2	m2	b ・ h	0.425	
	Moment of inertia of area	I2	m4	1/12・b・h3	0.00885	
Subtotal	Cross-sectional area	A	m2	A1 + A2	0.925	
	Horizontal Axis, Moment of inertia of area	I	m4	I1 + I2	0.05052	

・Operation Deck



The shaded area is valid.

Calculation table of member cross section and moment of inertia

Item	Symbol	Units	Formula	Value	Remarks
Horizontal dimension	b	m		1.35	
Vertical dimension	h	m		0.70	
Cross-sectional area	A	m2	b ・ h	0.945	
Moment of inertia of area	I	m4	1/12・b・h3	0.03859	

③ Member characteristic value

Elastic modulus : E = 2.14 x 104 N/mm2 = 2.14 x 107 kN/m2

Linear expansion coeffi : ε = 1.08 x 10 -5/℃

Source: Study Team

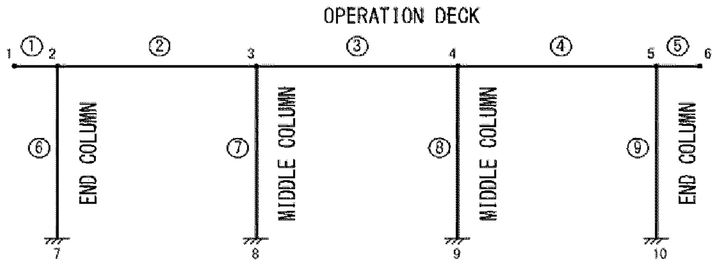
Figure7.5.46 Component Specifications (2)

(c) Calculated Result

(i) Adopted Sectional Force

The cross-sectional force of each case sated in the preceding paragraph is summarized and the adopted cross-sectional force is shown in the table below.

Table7.5.29 Constant Equivalent Cross-Sectional Force



Converted section force

Panel Point		Point to be Focuced	CASE		M kN m	S kN	N kN	Remarks
Operation deck	2,5	end fulcrum Upper	9	Normal Condition + Wind Load + temperature rise	228.87	153.09	128.31	
	3,4	Middle fulcrum Upper	10 (8)	Normal Condition + Wind Load + temperature drop	260.35	202.41	-16.71	In ( ) is S
	2-3,3-4 ,4-5	Span Bottom	10	Normal Condition + Wind Load + temperature drop	164.96	–	-16.71	
end column	2,5	Outside Top	9 (10)	Normal Condition + Wind Load + temperature rise	193.12	130.00	333.28	In ( ) is S
	2,5	Top Inside	10	Normal Condition + Wind Load + temperature drop	167.25	130.00	144.01	
	7,10	Bottom Outside	10	Normal Condition + Wind Load + temperature drop	272.16	141.66	186.16	
	7,10	Bottom Inside	9 (10)	Normal Condition + Wind Load + temperature rise	240.80	141.66	375.44	In ( ) is S
Center Colum	3,4	Top	10	Normal Condition + Wind Load + temperature drop	189.03	124.66	481.88	
	8,9	lower end	10	Normal Condition + Wind Load + temperature drop	233.03	136.32	532.65	

*At each point to be focuced on, the maximum value is adopted as a representative, and a common cross-sectional force is used.

*The shearing force used for the examination of the vicinity of the column of the operation deck is determined by the standard concept and the force is the one at a position 1/2 of the height of the operation decke member away from the column surface.

(ii) Degree of Stress

・Operation Deck

Shape name Title		[Rectangle Shape] Upper Side, Edge Fulcrum of the Operation Deck	[Rectangle Shape] Upper Side, Middle Fulcrum of the Operation Deck	[Rectangle Shape] Lower Side of the Operation deck Span
Section dimension b1 h1 (m) b2 h2 B3 h3		1.350 0.700 0.000 0.000 ------- 0.000	1.350 0.700 0.000 0.000 ------- 0.000	1.350 0.700 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		1.3500 0.6200 0.0000	1.3500 0.6200 0.0000	1.3500 0.6300 0.0000
Amount of reinforcement	D1/as1	0.6200 8-D 25	0.6200 8-D 25	0.6300 25.133
	Total cm2	39.270	39.270	25.133
Cross-sectional force M kNm N kN S kN		228.870 128.310 153.090	260.350 -16.710 202.410	164.960 -16.710 0.000
Stress intensity σc σca N/mm2 σs σsa σs' σsa		3.3 < 8.28 90.0 < 168.00 -------	3.6 < 8.28 121.3 < 168.00 -------	2.6 < 8.28 117.1 < 168.00 -------
Neutral axis x m		0.21853	0.19023	0.15773
Young's modulus ratio		n = 9.00	n = 9.00	n = 9.00
Sh kN Sh ' ' kN Mean τ m n/mm2		153.090 0.000 0.18 < 0.360	202.410 0.000 0.24 < 0.360	0.000 0.000 0.00 < 0.360

x, Vo indicate the distance from the compression edge.

*Operation deck span lower side : Rebar 8-D 20 = 25.133 cm^2

・Middle Column (1) (2)

Shape name Title		[Rectangle Shape] Middle column upper both sides (1)	[Rectangle Shape] Middle column bottom both sides (1)	[Rectangle Shape] Middle column top both sides (2)	[Rectangle Shape] Middle column bottom both sides (2)
Section dimension b1 h1 (m) b2 h2 b3 h3		0.500 1.000 0.000 0.000 ------- 0.000	0.500 1.000 0.000 0.000 ------- 0.000	0.850 0.500 0.000 0.000 ------- 0.000	0.850 0.500 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		0.5000 0.8700 0.0000	0.5000 0.8700 0.0000	0.8500 0.3700 0.0000	0.8500 0.3700 0.0000
Amount of reinforcement	D1/as1	0.8700 9.425	0.8700 9.425	0.3700 7-D 16	0.3700 7-D 16
	Total cm2	9.425	9.425	14.074	14.074
Cross-sectional force M kNm n kN s kN		155.950 260.700 67.440	192.250 288.160 73.750	33.080 221.180 57.220	40.780 244.490 62.570
Stress intensity σc σca N/mm2 σs σsa σs' σsa		3.8 < 8.28 80.3 < 168.00 -------	4.7 < 8.28 112.5 < 168.00 -------	1.7 < 8.28 3.8 < 168.00 -------	2.1 < 8.28 8.6 < 168.00 -------
Neutral axis x m		0.35861	0.33531	0.32061	0.29014
Young's modulus ratio		n = 9.00	n = 9.00	n = 9.00	n = 9.00

Shape name Title	[Rectangle Shape] Middle column upper both sides (1)	[Rectangle Shape] Middle column bottom both sides (1)	[Rectangle Shape] Middle column top both sides (2)	[Rectangle Shape] Middle column bottom both sides (2)
Sh kN Sh ’ ’ kN Mean τ m n/mm2	67.440 0.000 0.16 < 0.360	73.750 0.000 0.17 < 0.360	57.220 0.000 0.18 < 0.360	62.570 0.000 0.20 < 0.360

x, Vo indicate the distance from the compression edge.

*Middle Column Top Both Sides (1) : Rebar 3-D 20 = 9.425 cm^2

*Middle Column Bottom Both Sides (1) : Rebar 3-D 20 = 9.425 cm^2

・End column (1)

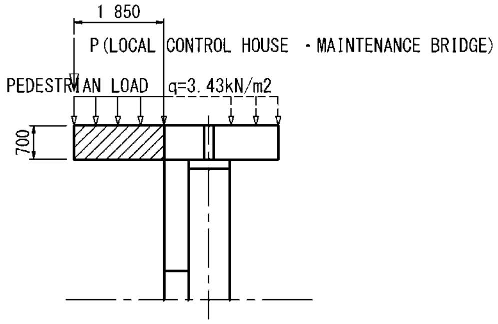
Shape name Title		[Rectangle Shape] End Column, upper outside (1)	[Rectangle Shape] End Column, top inside (1)	[Rectangle Shape] End Column, bottom outside (1)	[Rectangle Shape] End Column, column lower end inside (1)
Section dimension b1 h1 (m) b2 h2 b3 h3		0.500 0.750 0.000 0.000 ------- 0.000	0.500 0.750 0.000 0.000 ------- 0.000	0.500 0.750 0.000 0.000 ------- 0.000	0.500 0.750 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		0.5000 0.6200 0.0000	0.5000 0.6200 0.0000	0.5000 0.6200 0.0000	0.5000 0.6200 0.0000
Amount of reinforcement	D1/as1	0.6200 3-D 25	0.6200 3-D 25	0.6200 18.473	0.6200 3-D 25
	Total cm2	14.726	14.726	18.473	14.726
Cross-sectional force M kNm n kN s kN		128.420 156.310 60.970	111.220 67.540 60.970	180.990 87.310 66.440	160.130 176.080 66.440
Stress intensity σc σca N/mm2 σs σsa σs’ σsa		4.9 < 8.28 105.4 < 168.00 -------	4.2 < 8.28 113.1 < 168.00 -------	6.3 < 8.28 155.1 < 168.00 -------	6.1 < 8.28 137.3 < 168.00 -------
Neutral axis x m		0.25457	0.22226	0.23573	0.24798
Young's modulus ratio		n = 9.00	n = 9.00	n = 9.00	n = 9.00
Sh kN Sh ’ ’ kN Mean τ m n/mm2		60.970 0.000 0.20 < 0.360	60.970 0.000 0.20 < 0.360	66.440 0.000 0.21 < 0.360	66.400 0.000 0.21 < 0.360

x, Vo indicate the distance from the compression edge.

*End Column Bottom Outside (1) : Rebar 3-D 28 = 18.473 cm2

3) Operation Table Overhang

Since the overhanging length from the column surface is as long as 1.85 m in the land side, the upper reinforcement in the flow direction is examined in normal condition.



Source: Study Team

Figure7.5.47 Load Diagram

① Load Calculation

・Normal Condition

Item		Symbol	Unit	Formula	Value	Remarks
Distributed Load	Dead Weight	W1	kN/m2	24.0 x 0.70	16.80	
	Pedestrian Load	W2	kN/m2		3.43	
	Total	ΣW	kN/m2		20.23	
Concentrated Load	Local Control House	P1	kN/m	600 x 1/(11.90 + 4.20) x 2	18.63	
	Maintenace Bridge	P2	kN/m	From the previous FRAME diagram CASE1	13.19	
	Total	ΣP	kN/m		31.82	

*local control house load = P = 600 kN according to Building Design (including equipment)

② Section Force Calculation (Per meter of unit width)

Calculate as cantilevered beams fixed to columns.

Item	Symbol	Units	Formula	Value	Remarks
Distributed Load	W	kN/m		20.23	
Concentrated Load	P	kN		31.82	
Span	L1	m		1.85	
Point of Concentrated Load	L2	m		1.85	
Moment	M	kN m	1/2 W L 12 + P. L2	93.49	
Shear force	S	kN	W L1 + P	69.25	

③ Verification of Section

Check by elastic theoretical formula based on allowable stress method.

・Design Section Force (Per meter of unit width)

case	Category	Units	Section Force (Top)	Renarks
Normal Condition	M	kN m	93.49	
	S	kN	69.25	
	N	kN	—	

・Stress Intensity Calculation Operation Table Overhang

Shape name Title		[Rectangle Shape] Upper side of the operation deck
Section Dimension B1 H1 (m) B2 H2 B3 H3		1.000 0.700 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		1.0000 0.6200 0.0000
Amount of reinforcement	d1/As1	0.6200 12.566
	Total cm2	12.566
Cross-sectional force M kNm N kN S kN		93.490 0.000 69.250
Stress intensity σc σca N/mm2 σs σsa σs' σsa		2.4 < 8.28 129.4 < 168.00 -------
neutral axis x m		0.13518
Young's modulus ratio		n = 9.00
Sh kN Sh ' ' kN mean τ m N/mm2		69.250 0.000 0.11 < 0.360

x, Vo indicate the distance from the compression edge.

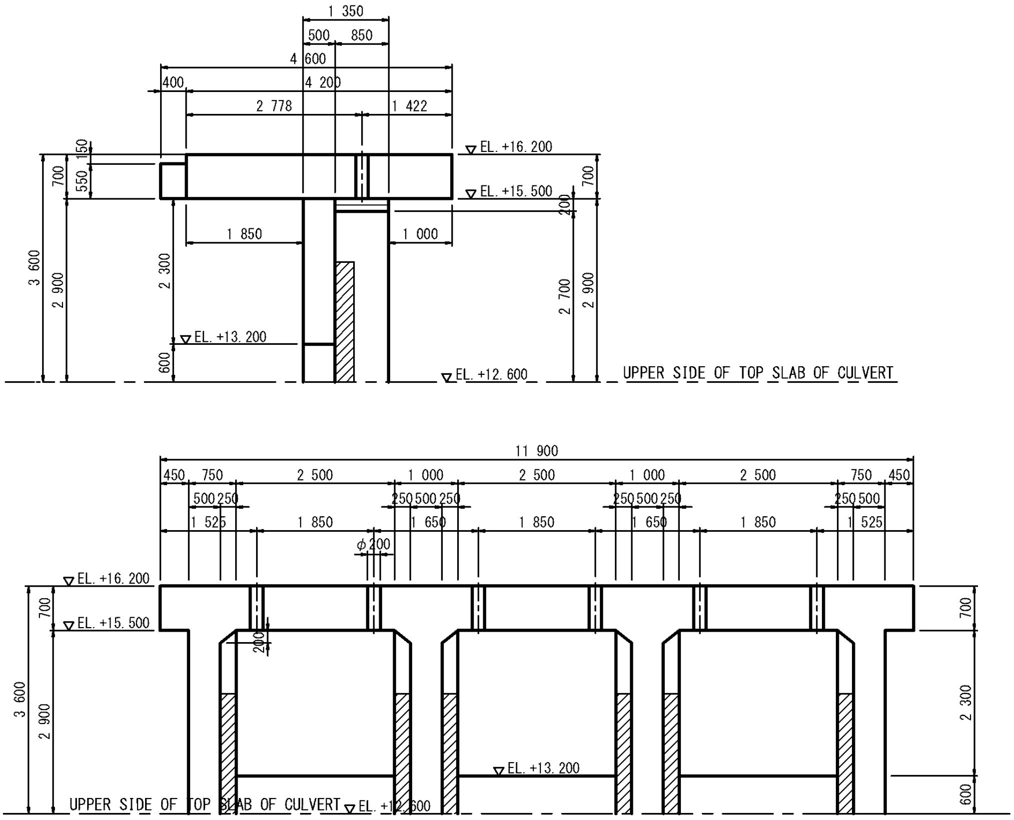
*Reinforcement : 4 - D 20 = 12.566 cm2 on the upper side of the overhanging portion of the operation deck.

4) Vertical (Flow Direction) Calculation

As a cantilever fixed to the top plate of the culvert, constant and earthquake conditions are examined. The cross sectional force is obtained at the center of gravity of the column.

(a) Geometrical Dimension

The shape and dimensions are as follows.



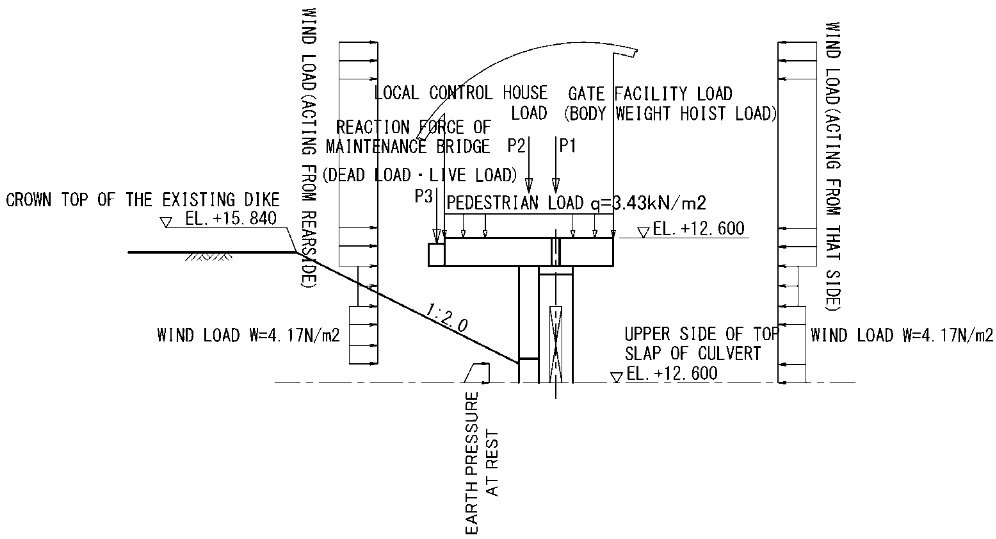
Source: Study Team

Figure7.5.48 Geometrical Diagram

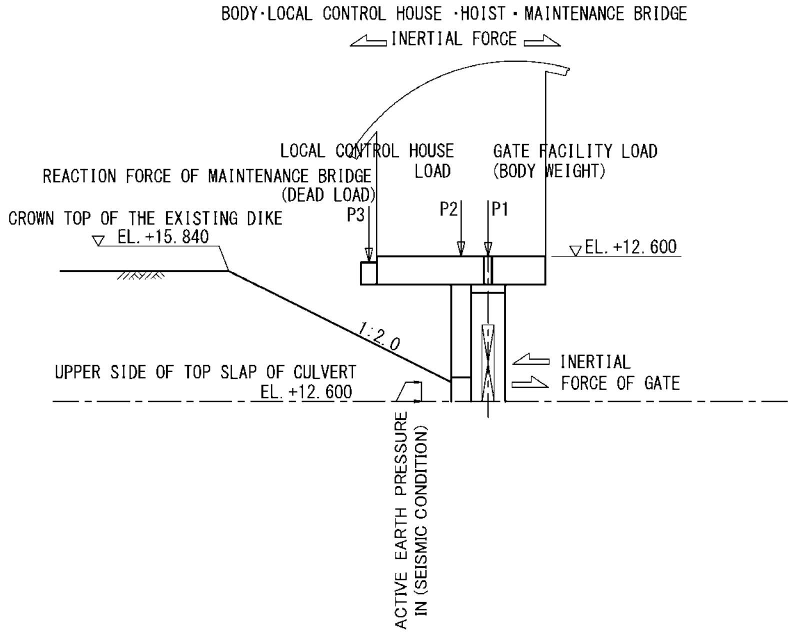
(b) Calculation Case

The calculation cases are indicated in **Figure7.5.49**.

CASE 1 : Normal Condition



CASE 2 : Seismic Condition



Source: Study Team

Figure7.5.49 Calculation Case

(c) Calculated Result

The cross-sectional force at the centroid position of the lower end of the column is obtained. For the direction of the horizontal force, two cases are considered: from the land side to river side and from the river side to land side.

(i) Cross-Sectional Force

① Total cross-section force

Normal Condition -1 (When wind load is applied, H: Land Side → River Side)

Item			V (kN)	H (kN)	X (m)	Y (m)	V. X (kN m)	H. Y (kN m)
Body Weight		Pv1	1105.08	-	-0.31	-	-342.57	-
Gate Facility	Gate and Hoist ･Hoisting Force	Pv2	168.00	-	0.34	-	57.12	-
Local Contorol House Load		Pv3	600.00	-	-0.34	-	-204.00	-
Pedestrian Load		Pv4	185.84	-	-0.34	-	-63.19	-
Maintenace Bridge		Pv5	11.86	-	-2.57	-	-30.48	-
Wind Load		Hw	-	358.95	-	4.79	-	1719.37
Earth Pressure		Hd	-	24.08	-	0.20	-	4.82
Σ			2070.78	383.03			-583.12	1724.19

Axial Force V = N = 2070.78 kN

Moment M = -583.12 + 1724.19 = 1141.07 kN m (Rear Side, Tension)

Shear Force H = S = 383.03 kN

Normal Condition -2 (When wind load is applied, H: Land Side ← Riverside)

Item			V (kN)	H (kN)	X (m)	Y (m)	V. X (kN m)	H. Y (kN m)
Body Weight		Pv1	1105.08	-	-0.31	-	-342.57	-
Gate Facility	Gate and Houst ･Hoisting force	Pv2	168.00	-	0.34	-	57.12	-
Local Contorol House Load		Pv3	600.00	-	-0.34	-	-204.00	-
Pedestrian Load		Pv4	185.84	-	-0.34		-63.19	-
Maintenace Bridge		Pv5	11.86	-	-2.57	-	-30.48	-
Wind Load		Hw	-	-380.47	-	4.53	-	-1723.53
Earth Pressure		Hd	-	24.08	-	0.20	-	4.82
Σ			2070.78	-356.39			-583.12	-1718.71

Axial Force V = N = 2070.78 kN

Moment M = -583.12 -1718.71 = -2301.83 kN m (front side tension)

Shear Force H = S = -356.39 kN

Seismic Condition: -1 (H: Land Side → River Side)

Item			V (kN)	H (kN)	X (m)	Y (m)	V. X (kN m)	H. Y (kN m)
Body Weight		Pv1	1105.08	221.02	-0.31	2.77	-342.57	612.23
Gate Facility	Vertical Force of Gate and Hoist	Pv2	78.00	-	0.34	-	26.52	-
	Inertia Force of Gate	Phg	-	8.40	-	0.90	-	7.56
	Inertia Force of Hoist	Ph2	-	7.20	-	3.60	-	25.92
Local Contorol House Load		Pv3Ph3	600.00	120.00	-0.34	5.85	-204.00	702.00
Pedestrian Load		Pv4	-	-	-	-	-	-
Maintenace Bridge		Pve5	5.00	2.00	-2.57	3.45	-12.85	6.90
Wind Load		Hw	-	-	-	-	-	-
Earth Pressure		Hd	-	19.72	-	0.19	-	3.75
Σ			1788.08	378.34			-532.90	1358.36

Axial Force V = N = 1788.08 kN

Moment M = -532.90 + 1358.36 = 825.46 kN m (Rear Side, Tension)

Shear Force H = S = 378.34 kN

Seismic Condition: -2 (H: Land Side - River Side)

Item			V (kN)	H (kN)	X (m)	Y (m)	V. X (kN m)	H. Y (kN m)
Body Weight		Pv1	1105.08	-221.02	-0.31	2.77	-342.57	-612.23
Gate Facility	Vertical Force of Gate and Hoist	Pv2	78.00	-	0.34	-	26.52	-
	Inertia Force of Gate	Phg	-	-8.40	-	0.90	-	-7.56
	Inertia Force of Hoist	Ph2	-	-7.20	-	3.60	-	-25.92
Local Contorol House Load		Pv3Ph3	600.00	-120.00	-0.34	5.85	-204.00	-702.00
Pedestrian Load		Pv4	-	-	-	-	-	-
Maintenace Bridge		Pve5	5.00	-2.00	-2.57	3.45	-12.85	-6.90
Wind Load		Hw	-	-	-	-	-	-
Earth Pressure		Hd	-	19.72	-	0.19	-	3.75
Σ			1788.08	-338.90			-532.90	-1350.86

Axial Force V = N = 1788.08 kN

Moment M = -532.90 -1350.86 = -1883.76 kN m (front side tension)

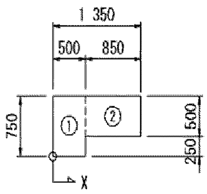
Shear Force H = S = -338.90 kN

② Section Force per Column

The design cross sectional force acting on each column is distributed from the total cross sectional force by the rigid ratio considering the area of each column the total cross sectional force.

・Member Cross-Sectional Area, Center of Gravity and Moment of Inertia of Area

End Column

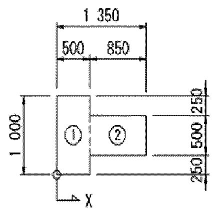


Symbol	Vertical Dimension h m	Horizontal Dimension b m	Cross-sectional Area A = h, b m2	Arm X m	A. X m3	A. X2 m4	Moment of Inertia of Area In m4
①	0.75	0.50	0.375	0.250	0.0938	0.0234	0.00781
②	0.50	0.85	0.425	0.925	0.3931	0.3636	0.02559
Σ			0.800		0.4869	0.3870	0.03340

Calculation Table of Cross Section Area, Center of Gravity, and Moment of Inertia of Area

Name	Symbol	Units	Formula	Value	Remarks
Cross-sectional Area	A	m2		0.800	
Distance from Centroid to Edge (Center of Gravity)	x	m	ΣA/X/ΣA	0.60863	
Vertical Axis Moment of Inertia of Area	Iy	m4	ΣA・X2 + ΣIo-ΣA・x2	0.12406	

Middle Column



Symbol	Vertical Dimension h m	Horizontal Dimension b m	Cross-sectional Area A = h, b m2	Arm X m	A. X m3	A. X2 m4	Moment of Inertia of Area In m4
①	1.00	0.50	0.500	0.250	0.1250	0.0313	0.01042
②	0.50	0.85	0.425	0.925	0.3931	0.3636	0.02559
Σ			0.925		0.5181	0.3949	0.03601

Calculation Table of Cross Section Area, Center of Gravity, and Moment of Inertia of Area

Name	Symbol	Units	Formula	Value	Remarks
Cross-sectional Area	A	m2		0.925	
Distance from Centroid to Edge (Center of Gravity)	x	m	ΣA/X/ΣA	0.56011	
Vertical Axis Moment of Inertia of Area	Iy	m4	ΣA・X2 + ΣIo-ΣA・x2	0.14072	

・Distribution Coefficient

Distribution Coefficient Calculation Sheet

Item		Symbol	Units	Formula	Value	Remarks
Moment of Inertia of Area	End Column(Left)	I1	m4		0.12406	
	Middle Column(Left)	I2	m4		0.14072	
	Middle Column (Right)	I2	m4		0.14072	
	End Column Right	I1	m4		0.12406	
	Subtotal	ΣI	m4		0.52956	
Distribution coefficient	End Column	α1		I1/ΣI	0.234	
	Middle Column	α2		I2/ΣI	0.266	

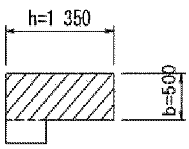
・Section Force per Column

Distributed Cross-sectional Force Calculation Table

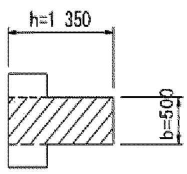
CASE	Total cross-sectional force			Distribution coefficient		Design Area Force per Column		Remarks
				End Cplumn α1	Middle Column α2	End Column	Middle Column	
Normal Condition -1 (When wind load is applied, H: Land Side → River Side)	M	kN m	1141.07	0.234	0.266	267.01	303.52	Rear Side, Tension
	N	kN	2070.78			484.56	550.83	
	S	kN	383.03			89.63	101.89	
Normal Condition -2 (When wind load is applied, H: Land Side ←	M	kN m	-2301.83	0.234	0.266	-538.63	-612.29	Front Side, Tension
	N	kN	2070.78			484.56	550.83	
	S	kN	356.39			83.40	94.80	
Seismic Condition-1 (H: Land Side → River Table)	M	kN m	825.46	0.234	0.266	193.16	219.57	Rear Side, Tension
	N	kN	1788.08			418.41	475.63	
	S	kN	378.34			88.53	100.64	
Seismic Condition-2 (H: Land Side ← River Side)	M	kN m	-1883.76	0.234	0.266	-440.80	-501.08	Front Side, Tension
	N	kN	1788.08			418.41	475.63	
	S	kN	338.90			79.30	90.15	

(ii) Degree of Stress

End column



Middle Colum



End Column

Shape name Title		[Rectangle Shape] End column Normal 1 Rear	[Rectangle Shape] End column Normal 2 Front	[Rectangle Shape] End column Seismic 1 Rear	[Rectangle Shape] End column Seismic 2 Front
Section Dimension B1 H1 (m) B2 H2 B3 H3		0.500 1.350 0.000 0.000 ------- 0.000	0.500 1.350 0.000 0.000 ------- 0.000	0.500 1.350 0.000 0.000 ------- 0.000	0.500 1.350 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		0.5000 1.2200 0.0000	0.5000 1.2200 0.0000	0.5000 1.2200 0.0000	0.5000 1.2200 0.0000
Reinforceme nt	d1/As1	1.2200 3-D 16	1.2200 3-D 25	1.2200 3-D 16	1.2200 3-D 25
	Total cm2	6.032	14.726	6.032	14.723
Cross-sectional force M kNm N kN S kN		 267.010 484.560 89.630	 538.630 484.560 83.400	 193.160 418.410 88.530	 440.800 418.410 79.300
Stress intensity σc σca N/mm2 σs σsa σs' σsa		3.5 < 10.35 56.7 < 210.00 -------	6.8 < 10.35 179.1 < 210.00 -------	2.3 < 11.01 23.0 < 223.40 -------	5.5 < 11.01 140.7 < 223.40 -------
Neutral axis x m		0.58869	0.44173	0.73715	0.45248
Young's modulus ratio		n = 9.00	n = 9.00	n = 9.00	n = 9.00
Sh kN Sh ' ' kN mean τm N/mm2		89.630 0.000 0.15 < 0.450	83.400 0.000 0.14 < 0.450	88.530 0.000 0.15 < 0.470	79.300 0.000 0.13 < 0.470

Middle Column

Shape name Title		[Rectangle Shape] Middle Column Normal 1 Rear	[Rectangle Shape] Middle Column Normal 2/ Front	[Rectangle Shape] Middle Column Seismic 1 Rear	[Rectangle Shape] Middle Column Seismic 2/ Front
Section Dimension B1 H1 (m) B2 H2 B3 H3		0.500 1.350 0.000 0.000 ------- 0.000	0.500 1.350 0.000 0.000 ------- 0.000	0.500 1.350 0.000 0.000 ------- 0.000	0.500 1.350 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		0.5000 1.2200 0.0000	0.5000 1.2200 0.0000	0.5000 1.2200 0.0000	0.5000 1.2200 0.0000
Reinforceme nt	d1/As1	1.2200 3-D 16	1.2200 3-D 25	1.2200 3-D 16	1.2200 3-D 25
	Total cm2	6.032	14.726	6.032	14.723
Cross-sectional force M kNm N kN S kN		 303.520 550.830 101.890	 612.290 550.830 94.800	 219.570 475.630 100.640	 501.080 475.630 90.150
Stress intensity σc σca N/mm2 σs σsa σs' σsa		4.0 < 10.35 64.5 < 210.00 -------	7.7 < 10.35 203.6 < 210.00 -------	2.7 < 11.01 26.2 < 223.40 -------	6.3 < 11.01 159.9 < 223.40 -------
Neutral axis x m		0.58870	0.44173	0.73719	0.45251
Young's modulus ratio		n = 9.00	n = 9.00	n = 9.00	n = 9.00
Sh kN Sh ' ' kN mean τm N/mm2		101.890 0.000 0.17 < 0.450	94.800 0.000 0.16 < 0.450	100.640 0.000 0.16 < 0.470	90.150 0.000 0.15 < 0.470

5) Minimum Reinforcement

The minimum reinforcement is checked for the operation deck as the member in which the bending moment is dominant, and for the end column and middle column as the member in which the axial force is dominant. The axial force used for checking the end column and the middle column shall be the maximum value determined in the horizontal and vertical directions of the column.

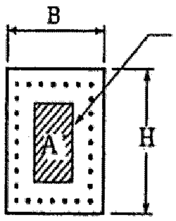
① Operating Dexk

As members dominated by bending moment

Member	Item	Symbol	Units	Formula	Value	Remarks
Top of Operation Table	Width	b	cm		135	
	Effective Height	d	cm		62	
	Minimum Reinforcement	As m	cm2	0.2% ・ b ・ d	16.74	
	Bar Arrangement	As	cm2	D 25 -8	39.27	
	Evaluation	As ・ m = 16.74 cm2 < As = 39.27 cm2 -> OK				
Lower Surface of the Operation Table	Width	b	cm		135	
	Effective Height	d	cm		63	
	Minimum Reinforcement	As m	cm2	0.2% ・ b ・ d	17.01	
	Bar Arrangement	As	cm2	D 20 -8	25.13	
	Evaluation	As ・ m = 17.01 cm2 < As = 25.13 cm2 -> OK				
operating dexk Flow Direction Top	Width	b	cm		100	
	Effective Height	d	cm		62	
	Minimum Reinforcement	As m	cm2	0.2% ・ b ・ d	12.40	
	Bar Arrangement	As	cm2	D 20 -4	12.57	@250
	Evaluation	As ・ m = 12.40 cm2 < As = 12.57 cm2 -> OK				

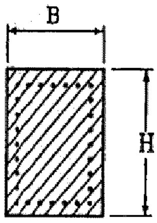
② End Column

The formula specified in Specifications for Highway Bridges, Japan and Concrete Standard Specifications, Japan would be used.



Cross Section Area Required in Calculation

$$\frac{TOTAL\ REINFORCEMENT}{A'} \geq 0.8\ (\%)$$



$$\frac{TOTAL\ REINFORCEMENT}{B \times H} \geq 0.15\ (\%)$$

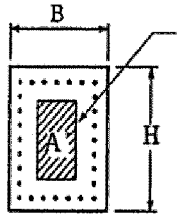
As an axial force dominant component

Category	Item	Symbol	Units	Formula	Value	Remarks
For Cross Section Area Required in Calculation	Axial Force (Maximum)	N	kN		484.56	From Longitudinal Calculation
	Allowanble Compressive Stress of	sigma sa	N/mm2		168.00	
	Allowable Axial Compressive Stress of	σca	N/mm2		6.21	
	Cross-sectional Area Required in Calculation	A'	mm2	N/(0.008 σsa + σca)	64146	
	Minimum Reinforcement 1	As1	cm2	0.8%・A'	5.13	
For Cross Section Area which is larger than it Required in Calculation	Section (1)	A1	cm2	50 x 75	3750.00	
	Section (2)	A2	cm2	50 x 85	4250.00	
	Total Cross-sectional Area	ΣA	cm2		8000.00	
	Minimum Reinforcement2	As2	cm2	0.15%・A	12.00	

Verification of Minimum Reinforcement of End Column				
Minimum Rainforcement (cm2)			Evaluation	All the Axial Reinforcement (Per Location)
For Cross Section Area Required in Calculation	As1 =	5.13	<	D 25 -18, D 16 -8 As = 104.45 cm2
For Cross Section Area which is larger than it Required in Calculation	As2 =	12.00	<	
As a result, the reinforcement of the end column as a member in which the axial force is dominant satisfies the minimum reinforcment..				

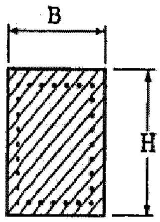
③ Middle Column

The formula specified in Specifications for Highway Bridges, Japan and Concrete Standard Specifications, Japan would be used.



Cross Section Area Required in Calculation

$$\frac{TOTAL\ REINFORCEMENT}{A'} \geq 0.8\ (\%)$$



$$\frac{TOTAL\ REINFORCEMENT}{B \times H} \geq 0.15\ (\%)$$

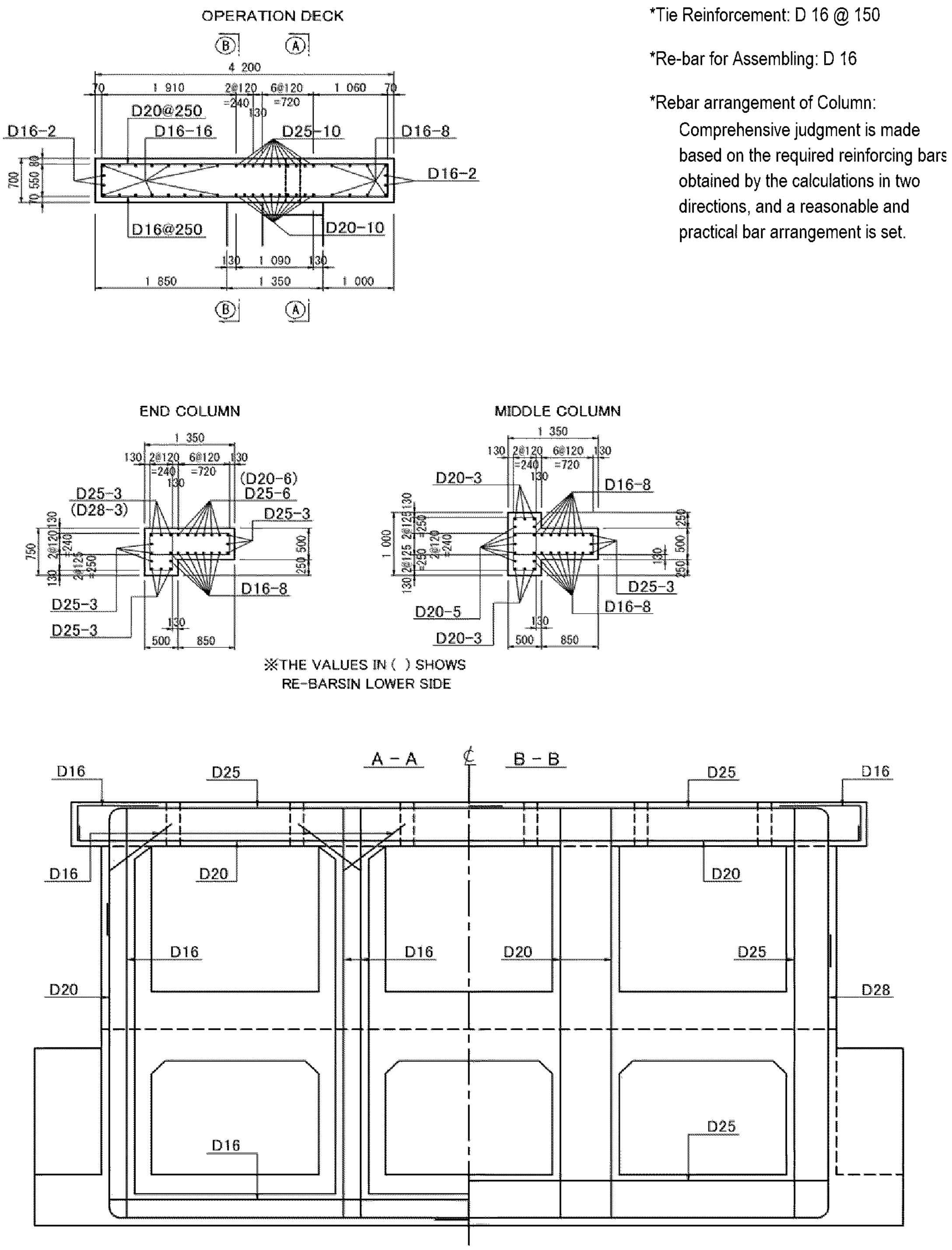
As an axial force dominant component

Category	Item	Symbol	Units	Formula	Value	Remarks
For Cross Section Area Required in Calculation	Axial Force (Maximum)	N	kN		629.00	From Transverse Calculation
	Allowanble Compressive Stress of	sigma sa	N/mm2		168.00	
	Allowable Axial Compressive Stress of	σca	N/mm2		6.21	
	Cross-sectional Area Required in Calculation	A’	mm2	N/(0.008 σsa + σca)	83267	
	Minimum Reinforcement 1	As1	cm2	0.8% ・A '	6.66	
For Cross Section Area which is larger than it Required in Calculation	Section (1)	A1	cm2	50 x 100	5000.00	
	Section (2)	A2	cm2	50 x 85	4250.00	
	Total Cross-sectional Area	ΣA	cm2		9250.00	
	Minimum Reinforcement2	As2	cm2	0.15% ・A	13.88	

Verification of Minimum Reinforcement of Middle Column				
Minimum Rainforcement (cm2)			Evaluation	All the Axial Reinforcement (Per Location)
For Cross Section Area Required in Calculation	As1 =	6.66	<	D 25 -3, D 20 -11, D 16 -16 As = 81.47 cm2
For Cross Section Area which is larger than it Required in Calculation	As2 =	13.88	<	
As a result, the reinforcement of the middle column as a member in which the axial force is dominant satisfies the minimum reinforcment..				

6) Bar Arrangement Procedure

Based on the above calculation results, the column reinforcement arrangement plan is as follows.



Source: Study Team

Figure7.5.50 bar arrangement plan

(5) Design of Riverside Breast walls

1) Design Policy

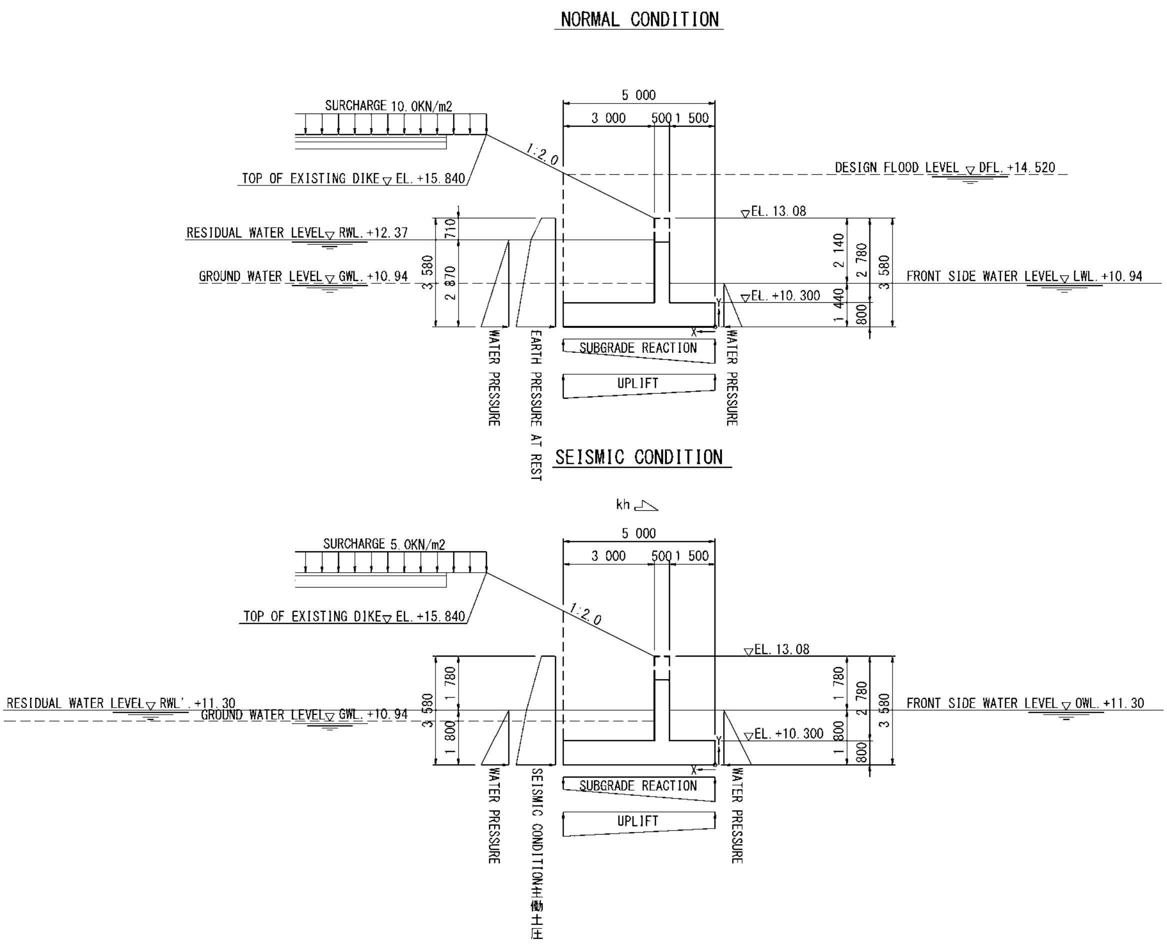
The vertical wall and the bottom slab of the breast wall are designed as cantilevers fixed to the main body.

2) Design Water Level

The design water level is the same as that described above mentioned values described in **Table7.5.18**.

3) Calculation Case

The calculation cases shall be examined in normal condition and seismic condition. In the passive side, only water pressure shall be considered, and earth pressure shall not be considered.



Source: Study Team

Figure7.5.51 Calculation Model Diagram

4) Calculated Result

(a) Vertical Wall

Table7.5.30 Stress Intensity in Normal Condition

Section Information

	Symbol	Units	Vertical wall
Section Width	B	m	1.000
Section height	H	m	0.500
Rebar Information	Front	mm2	D 16 -250 804.2
	Rear	mm2	D 16 -250 804.2

	Symbol	Units	Base
Bending moment	M	kNm	25.41
Rebar	d1 As1	mm mm2	410.0 804.2
Required reinforcement	dorsum	mm2	384.6
Neutral axis	X	m	0.070
Stress intensity	σ c	N/mm2	1.87
	σ s	N/mm2	81.74
Allowance	σ ca	N/mm2	8.28
	σ sa	N/mm2	168.00

Source: Study Team

Table7.5.31 Shear Stress in Normal Condition

	Symbol	Units	Base
Shear force	S	kN	50.83
Rebar used	d1 As1	mm mm2	410.0 804.2
Effective height	d	mm	410.0
Section width	b	mm	1000.0
Mean shear stress	τm	N/mm2	0.124
Allowance	τa	N/mm2	0.360
	τa2	N/mm2	1.600

Source: Study Team

Table7.5.32 Bending Stress in Seismic Condition

Section Information

	Symbol	Units	Vertical wall
Section Width	B	m	1.000
Section Height	H	m	0.500
Rebar	Front	mm2	D 16 -250 804.2
	Rear	mm2	D 16 -250 804.2

	Symbol	Units	Base
Bending Moment	M	kNm	21.40
Rebar Used	d1 As1	mm mm2	410.0 804.2
Required Reinforcement	dorsum	mm2	241.5
Neutral Axis	X	m	0.070
Degree of Stress	σ c	N/mm2	1.58
	σ s	N/mm2	68.84
Allowance	σ ca	N/mm2	12.42
	σ sa	N/mm2	223.40

Source: Study Team

Table7.5.33 Shear Stress in Seismic Condition

	Symbol	Units	Base
Shear Force	S	kN	42.81
Rebar	d1 As1	mm mm2	410.0 804.2
Effective Height	d	mm	410.0
Section Width	b	mm	1000.0
Mean Shear Stress	τm	N/mm2	0.104
Allowance	τa	N/mm2	0.540
	τa2	N/mm2	2.400

Source: Study Team

(b) Toe Slab

Table7.5.34 Bending Stress in Normal Condition

Section Information

	Symbol	Units	Toe slab
Section width	B	m	1.000
Section height	H	m	0.800
Rebar information	Top	mm2	D 16 -250 804.2
	Bottom	mm2	D 16 -250 804.2

	Symbol	Units	Base	End
Bending Moment	M	kNm	49.66	59.99
Rebar Used	d1 As1	mm mm2	710.0 804.2	710.0 804.2
Required Reinforcement	Bottom	mm2	430.5	521.8
Neutral Axis	X	m	0.094	0.094
Degree of Stress	σ c	N/mm2	1.55	1.87
	σ s	N/mm2	90.99	109.92

Allowance	σ ca	N/mm2	8.28	8.28
	σ sa	N/mm2	168.00	168.00

Source: Study Team

Table7.5.35 Shear Stress in Normal Condition

	Symbol	Units	Base	End
Shear force	S	kN	99.31	119.97
Rebar used	d1 As1	mm mm2	710.0 804.2	710.0 804.2
Effective height	d	mm	710.0	710.0
Section width	b	mm	1000.0	1000.0
Mean shear stress	τm	N/mm2	0.140	0.169
Allowance	τa	N/mm2	0.360	0.360
	τa2	N/mm2	1.600	1.600

Source: Study Team

Table7.5.36 Seismic Bending Stress

Section Information

	Symbol	Units	Toe slab
Section width	B	m	1.000
Section height	H	m	0.800
Rebar information	Top	mm2	D 16 -250 804.2
	Bottom	mm2	D 16 -250 804.2

	Symbol	Units	Base	End
Bending Moment	M	kNm	42.40	53.81
Rebar Used	d1 As1	mm mm2	710.0 804.2	710.0 804.2
Required Reinforcement	Bottom	mm2	274.6	349.7
Neutral Axis	X	m	0.094	0.094
Degree of Stress	σ c	N/mm2	1.32	1.68
	σ s	N/mm2	77.69	98.61
Allowance	σ ca	N/mm2	12.42	12.42
	σ sa	N/mm2	223.40	223.40

Source: Study Team

Table7.5.37 Shear Stress in Seismic Condition

	Symbol	Units	Base	End
Shear Force	S	kN	84.80	107.62

Rebar Used	d1 As1	mm mm2	710.0 804.2	710.0 804.2
Effective Height	d	mm	710.0	710.0
Section Width	b	mm	1000.0	1000.0
Mean Shear Stress	τm	N/mm2	0.119	0.152
Allowance	τa	N/mm2	0.540	0.540
	τa2	N/mm2	2.400	2.400

(c) Heel Slab

Table7.5.38 Bending Stress in Normal condition

Section Information

	Symbol	Units	Heel slab
Section Width	B	m	1.000
Section height	H	m	0.800
Rebar information	Top	mm2	D 16 -250 804.2
	Bottom	mm2	D 16 -250 804.2

	Symbol	Units	Base	end
Bending Moment	M	kNm	21.90	-13.00
Rebar Used	d1 As1	mm mm2	710.0 804.2	90.0 804.2
Required Reinforcement	Top	mm2		110.9
	Bottom	mm2	187.8	
Neutral Axis	X	m	0.094	0.094
Stress Intensity	σ c	N/mm2	0.68	0.41
	σ s	N/mm2	40.14	23.83
Allowance	σ ca	N/mm2	8.28	8.28
	σ sa	N/mm2	168.00	168.00

Source: Study Team

Table7.5.39 Constant Shear Stress

	Symbol	Units	Base	end
Shear Force	S	kN	43.81	-26.00
Rebar Used	d1 As1	mm mm2	710.0 804.2	90.0 804.2
Effective Height	d	mm	710.0	710.0
Section Width	b	mm	1000.0	1000.0
Mean Shear Stress	τm	N/mm2	0.062	0.037
Allowance	τa	N/mm2	0.360	0.360
	τa2	N/mm2	1.600	1.600

Source: Study Team

Table7.5.40 Bending Stress in Seismic Condition

Section Information

	Symbol	Units	Heel slab
Section Width	B	m	1.000
Section height	H	m	0.800
Rebar information	Top	mm2	D 16 -250 804.2
	Bottom	mm2	D 16 -250 804.2

	Symbol	Units	Base	end
Bending Moment	M	kNm	16.58	-20.49
Rebar Used	d1 As1	mm mm2	710.0 804.2	90.0 804.2
Required Reinforcement	Top	mm2		131.6
	Bottom	mm2	106.3	
Neutral Axis	X	m	0.094	0.094
Stress Intensity	σ c	N/mm2	0.52	0.64
	σ s	N/mm2	30.39	37.55
Allowance	σ ca	N/mm2	12.42	12.42
	σ sa	N/mm2	223.40	223.40

Source: Study Team

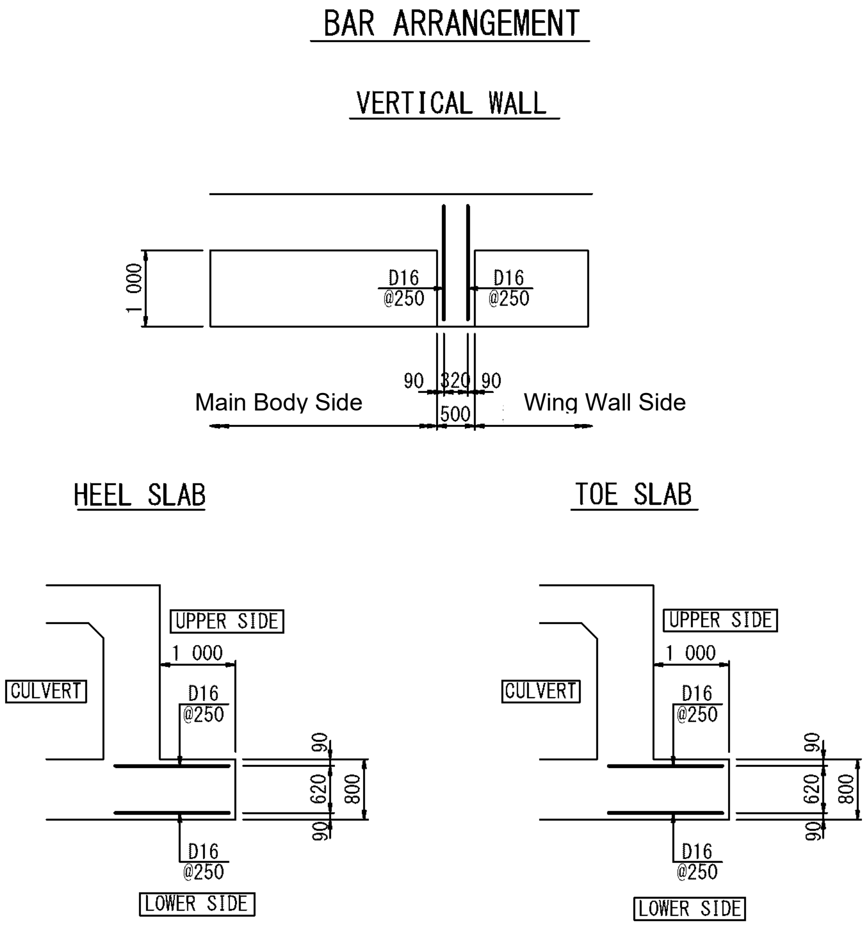
Table7.5.41 Shear Stress in Seismic Condition

	Symbol	Units	Base	end
Shear force	S	kN	33.17	-40.98
Rebar used	d1 As1	mm mm2	710.0 804.2	90.0 804.2
Effective height	d	mm	710.0	710.0
Section width	b	mm	1000.0	1000.0
Mean shear stress	τm	N/mm2	0.047	0.058
Allowance	τa	N/mm2	0.540	0.540
	τa2	N/mm2	2.400	2.400

Source: Study Team

5) Bar Arrangement

The bar arrangement of the breast wall is determined as follows.

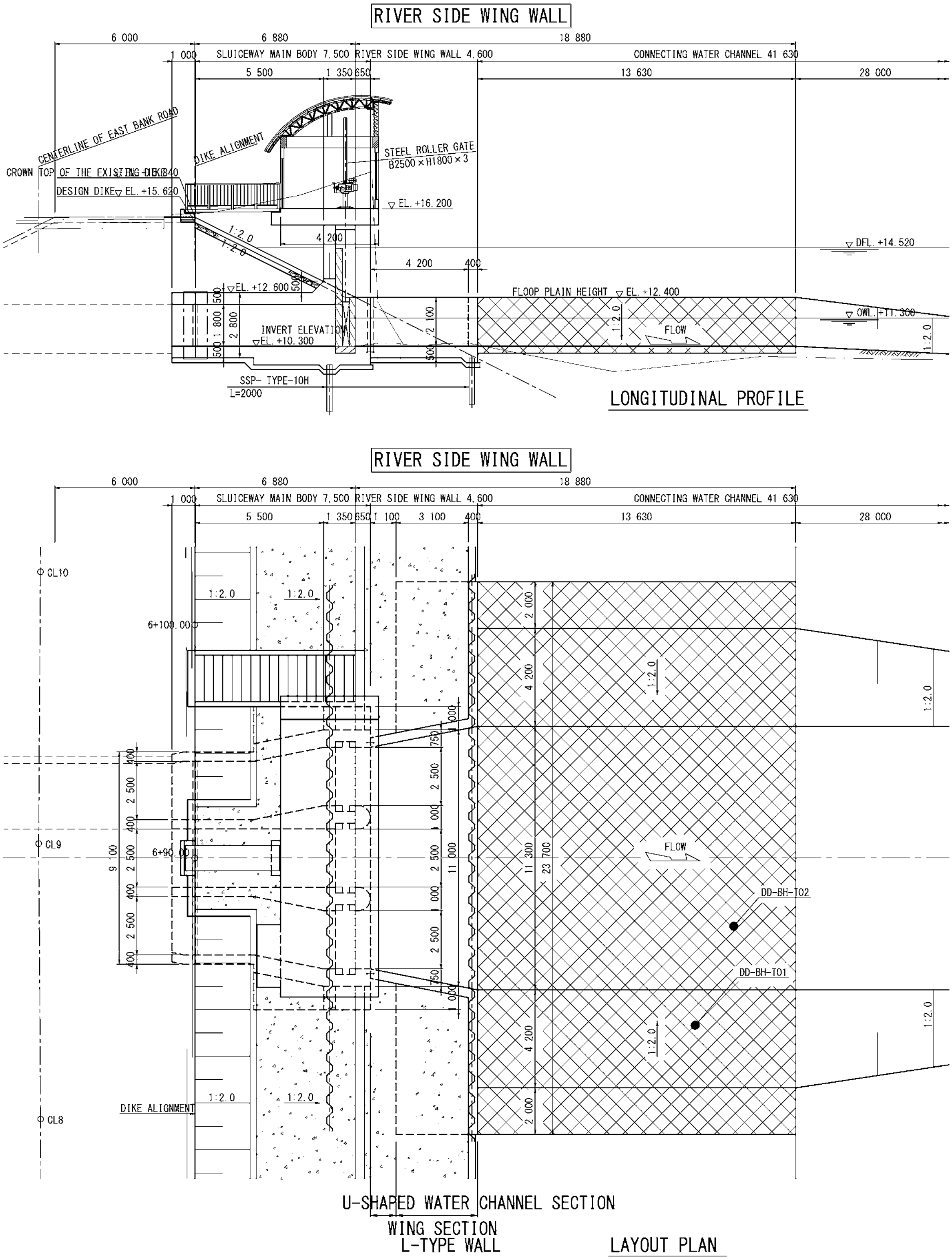


Source: Study Team

Figure7.5.52 Bar Arrangement of Breast Wall

(6) Design of the Wing Wall in Riverside

For river side wing wall, stability calculation and structural calculation of members are performed. The details of the design policy are shown in the following pages.



Source: Study Team

Table7.5.42 Dimensions of Wing Wall

1) Design Policy

(a) Check Item

The table below shows the check items of the U-shaped channel and the L-shaped retaining wall of the wing. Details of the shape, dimensions, calculation case, etc. are separately indicated in the design of each facility.

Table7.5.43 List of Stable Calculation Check Items

<table>
<tr><th rowspan="2">Type of Facility</th><th colspan="2">Calculation Case</th><th colspan="4">Study Item</th><th rowspan="2" colspan="2">Remarks</th></tr>
<tr><th>Normal Condition</th><th>Seismic Condition</th><th>Bearing</th><th>Overturning</th><th>Sliding</th><th>Floating</th></tr>
<tr><td rowspan="3">U-shaped Channel</td><td rowspan="3">×</td><td rowspan="3">×</td><td rowspan="3">×</td><td rowspan="3">×</td><td rowspan="3">×</td><td rowspan="3">×</td><td>Bearing</td><td>Load(dead weight, etc.) acting area on the foundation is small. Therefore, The study can be omitted.</td></tr>
<tr><td>Overturning , Sliding</td><td>Horizontal load acting on the left and right walls is balanced, Hence, this is omitted.</td></tr>
<tr><td>Floating</td><td>Since the length in flow direction is short, the influence of wing section in whole wing wall is significant. Therefore, the stability against uplift is studied with whole wing wall, and no review would be conducted in each section.</td></tr>
<tr><td rowspan="3">Wing section L type Retaining Wall</td><td rowspan="3">○</td><td rowspan="3">○</td><td rowspan="3">○</td><td rowspan="3">○</td><td rowspan="3">○</td><td rowspan="3">×</td><td>Bearing</td><td>Since the composited force does not balance, the subgrade reaction by the eccentricity becomes large. Hence, this item shall be studied.</td></tr>
<tr><td>Overturning , Sliding</td><td>Earth pressure acts from the rear side and passive earth pressure in front side is not considered. The horizontal force does not balance. Hence, this item shall be studied.</td></tr>
<tr><td>Floating</td><td>Since soil is loaded on the rear side, the dead wight becomes large. The Floating is not issue.</td></tr>
<tr><td>Whole Wing Wall</td><td>○</td><td>×</td><td>×</td><td>×</td><td>×</td><td>○</td><td>Floating</td><td>External (rear side) water level: Normal Condition > Seismic condition,
Inner (front side) water level: Normal Condition < Seismic Condition, only in normal condition, this item shall be checked.</td></tr>
</table>

Source: Study Team

(b) Member Section Calculation

The sections of the two facilities shown in the table below shall be examined for normal and earthquake conditions. Details of the shape, dimensions, calculation case, etc. are separately indicated in the design of each facility.

Table7.5.44 Member Section Calculation Case List

<table>
<tr><th rowspan="2">Type of Facility</th><th colspan="2">calculation case</th><th rowspan="2">Calculation Model</th><th rowspan="2">Remarks</th></tr>
<tr><th>Normal Condition</th><th>Seismic Condition</th></tr>
<tr><td>U-shaped channel</td><td>○</td><td>○</td><td>U-shaped Frame considering the vertical ground spring on the foundation ground.</td><td rowspan="2">Calculation per unit width of 1 m</td></tr>
<tr><td>Wing Section L-shaped Retaining Wall</td><td>○</td><td>○</td><td>Vertical wall: cantilevered beam fixed to bottom slab
Bottom slab: cantilevered beam fixed to vertical wall</td></tr>
</table>

Source: Study Team

2) Design Water Level

Table7.5.18 The design water level is set based on the above.

3) U-Shaped Channel

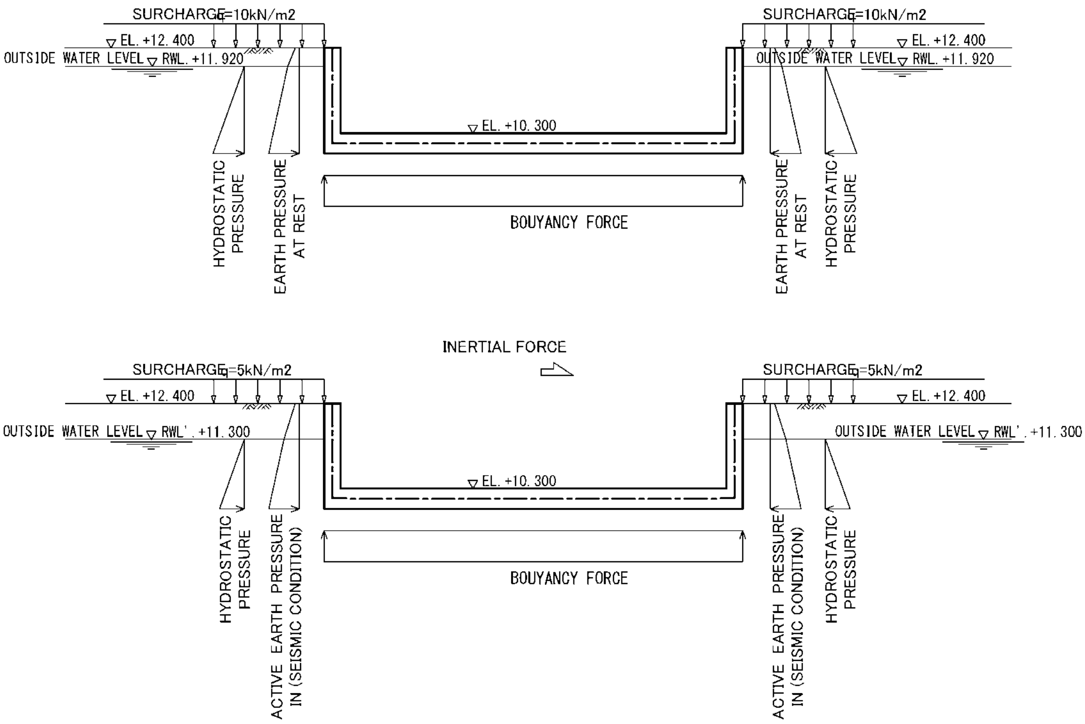
(a) Stability analysis

Table7.5.43 Therefore, the stability calculation of the U-shaped channel is omitted.

(b) Cross Section Calculation

(i) Calculation Case

The calculation cases are as follows.

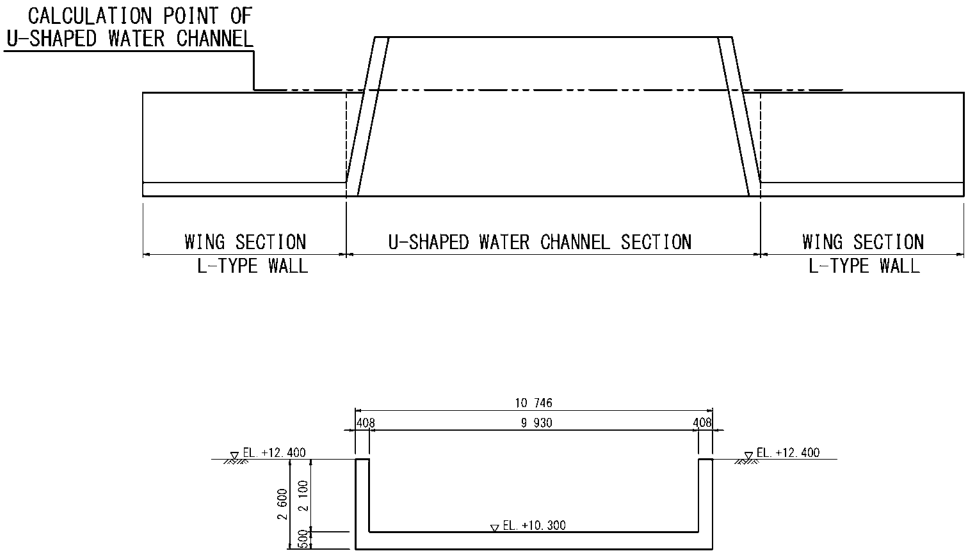


Source: Study Team

Figure7.5.53 Calculation Case

(ii) Computational Model

The calculation model was a U-shaped frame considering a vertical spring in the foundation ground. The calculation section is out of the wing section and the maximum sectional force is expected due to the surcharge load acting on the rear side.



Source: Study Team

Figure7.5.54 Computational Model

(iii) Calculated Result

Table7.5.45 Normal Condition: Bending Stress of Sidewall Bottom (Outside)

Section Information

	Symbol	Units	Normal Condition: Bottom Side Wall (Outside)
Section Width	B	m	1.000
Section Height	H	m	0.400
Rebar Information	Outside	cm2	D 20 -250 12.566

	Symbol	Units	
Bending Moment	M	kNm	40.24
Rebar Used	d1 As1	mm cm2	310.0 12.566
Neutral Axis	X	m	0.091
Stress Intensity	σ c	N/mm2	3.2
	σ s	N/mm2	114.5
Allowance	σ ca	N/mm2	8.28
	σ sa	N/mm2	168.00

Source: Study Team

Table7.5.46 Normal Condition: Shear Stress of Sidewall Bottom (Outside)

	Symbol	Units	
Shear Force	S	kN	47.43
Rebar Used	d1 As1	mm cm2	310.0 12.566
Effective Height	d	mm	310.0
Section Width	b	mm	1000.0
Mean Shear Stress	τm	N/mm2	0.15
Allowance	τa	N/mm2	0.360

Source: Study Team

Table7.5.47 Normal Condition: Bending Stress of Bottom Plate End (Underside)

Section Information

	Symbol	Units	Normal Condition: Bottom Slab End (Underside)
Section Width	B	m	1.000
Section Height	H	m	0.500
Rebar Information	Outside	cm2	D 16 -250 8.042

	Symbol	Units	
Bending Moment	M	kNm	40.24
Rebar Used	d1 As1	mm cm2	410.0 8.042
Neutral Axis	X	m	0.108
Stress Intensity	σ c	N/mm2	2.4
	σ s	N/mm2	100.0
Allowance	σ ca	N/mm2	8.28
	σ sa	N/mm2	168.00

Source: Study Team

Table7.5.48 Normal Condition: Shear Stress of Bottom Plate End (Underside)

	Symbol	Units	
Shear Force	S	kN	23.12
Rebar Used	d1 As1	mm cm2	410.0 8.042
Effective Height	d	mm	410.0
Section Width	b	mm	1000.0
Mean Shear Stress	τm	N/mm2	0.06
Allowance	τa	N/mm2	0.360

Source: Study Team

Table7.5.49 Regular: Bending Stress at Bottom Plate Span (Upper Side)

Section Information

	Symbol	Units	Normal Condition: Bottom Slab Span (Upper)
Section Width	B	m	1.000
Section Height	H	m	0.500
Rebar Information	Outside	cm2	D 16 -250 8.042

	Symbol	Units	
Bending Moment	M	kNm	9.32
Rebar Used	d1 As1	mm cm2	410.0 8.042
Neutral Axis	X	m	0.240
Stress Intensity	σ c	N/mm2	0.4
	σ s	N/mm2	4.6
Allowance	σ ca	N/mm2	8.28
	σ sa	N/mm2	168.00

Source: Study Team

4) Wing Section (L-Type Retaining Wall)

(a) Stability Analysis

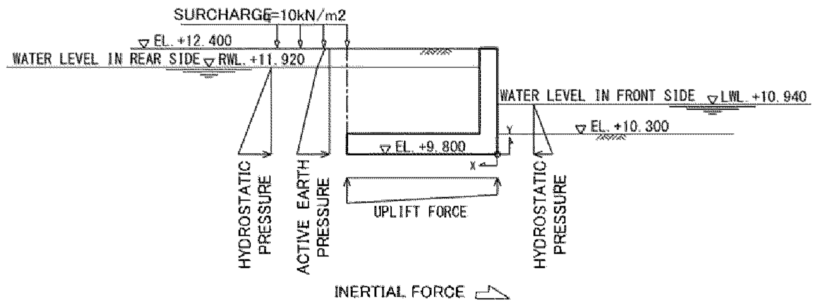
(i) Load Case

The load cases are examined for 2 cases such as normal and seismic condition. The case diagram is shown in **Figure7.5.55**.

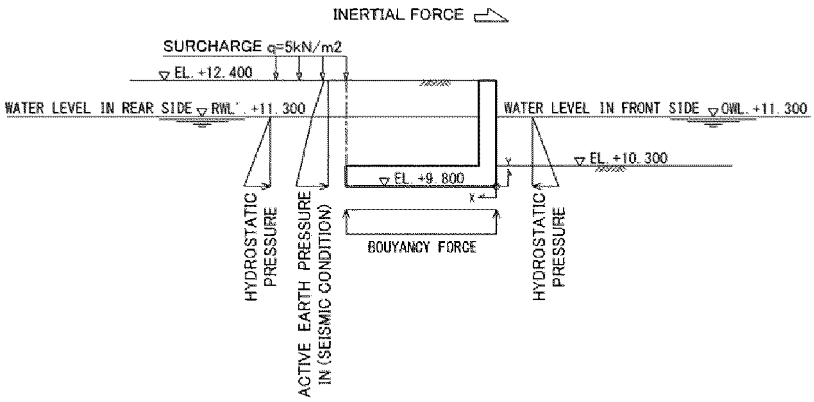
(ii) Design Water Level

The design water level is indicated in **Figure7.5.55**.

CASE1 Normal Condition



CASE 2 Seismic Condition



Source: Study Team

Figure7.5.55 Load Case Diagram

(iii) Load Schedule

The load calculation results for the two load cases are as shown in **Table7.5.50** and **Table7.5.51**.

Table7.5.50 Permanent Load

Item	Vertical Force ni (kN)	Horizontal Force hi (kN)	Arm Length		Moment of Rotation (kNm)	
			Xi (m)	Yi (m)	Mxi = Ni/Xi	Myi = Hi Yi
Dead Weight	190.872	0.000	1.721	0.000	328.520	0.000
Uplift Force	-55.909	0.000	1.925	0.000	-107.645	0.000
Hydrostatic Pressure in Rear side	0.000	22.023	0.000	0.707	0.000	15.563
Hydrostatic Pressure in Front side	0.000	-6.368	0.000	0.380	0.000	-2.420
Earth Pressure	10.467	18.129	3.500	1.071	36.634	19.420
Total	145.430	33.784	———	———	257.510	32.563

Source: Study Team

Table7.5.51 Seismic Load

Item	Vertical Force ni (kN)	Horizontal Force hi (kN)	Arm Length		Moment of Rotation (kNm)	
			Xi (m)	Yi (m)	Mxi = Ni/Xi	Myi = Hi Yi
Dead Weight	188.950	37.790	1.719	1.252	324.773	47.314
Uplift Force	-51.450	0.000	1.750	0.000	-90.038	0.000
Hydrostatic Pressure in Rear side	0.000	11.025	0.000	0.500	0.000	5.513
Hydrostatic Pressure in Front side	0.000	-11.025	0.000	0.500	0.000	-5.513
Earth Pressure	8.100	30.229	3.500	0.990	28.350	29.936
Total	145.600	68.019	———	———	263.085	77.250

Source: Study Team

Table7.5.52 Summary of Load

Load State (Water Level)	No (kN)	Ho (kN)	Mo (kNm)
permanent load (constant water level)	145.430	33.784	224.947
seismic load (earthquake water level)	145.600	68.019	185.836

Source: Study Team

(iv) Stability Checking

The results of the stability calculation are shown in **Table7.5.53** to **Table7.5.55**.

Table7.5.53 Verification Results of Overturning

Load State (Water Level)	Σ Mr (kNm)	ΣMt (kNm)	ΣV (kN)	d (m)	e (m)		Ea (m)	Evaluation
Load in Normal Condition (Water Level in Normal Condition	257.510	32.563	145.430	1.547	0.203	<<	0.583	OK
Load in Seismic Condition (Water Level in Seismic Condition)	263.085	77.250	145.600	1.276	0.474	<<	1.167	OK

Table7.5.54 Verification Results of Sliding

Load State (Water Level)	vertical Load ΣV (kN)	Horizontal Load ΣH (kN)	Safety Factor Fs		Required Safety Factor Fsa	Evaluation
Load in Normal Condition (Water Level in Normal Condition	145.430	33.784	2.583	≧	1.500	OK
Load in Seismic Condition (Water Level in Seismic Condition)	145.600	68.019	1.284	≧	1.200	OK

Source: Study Team

Table7.5.55 Verification Results of Allowable Bearing Capacity

Load State (Water Level)	Effective Loading Area Ae (m2)	Allowable Bearing Capacity qa (kN/m2)	Vertical Force V (kN)		Bearing Capacity Qa (kN)	Evaluation
Load in Normal Condition (Water Level in Normal Condition	3.094	145.085	145.430	≦	448.827	OK
Load in Seismic Condition (Water Level in Seismic Condition)	2.553	72.511	145.600	≦	185.099	OK

Source: Study Team

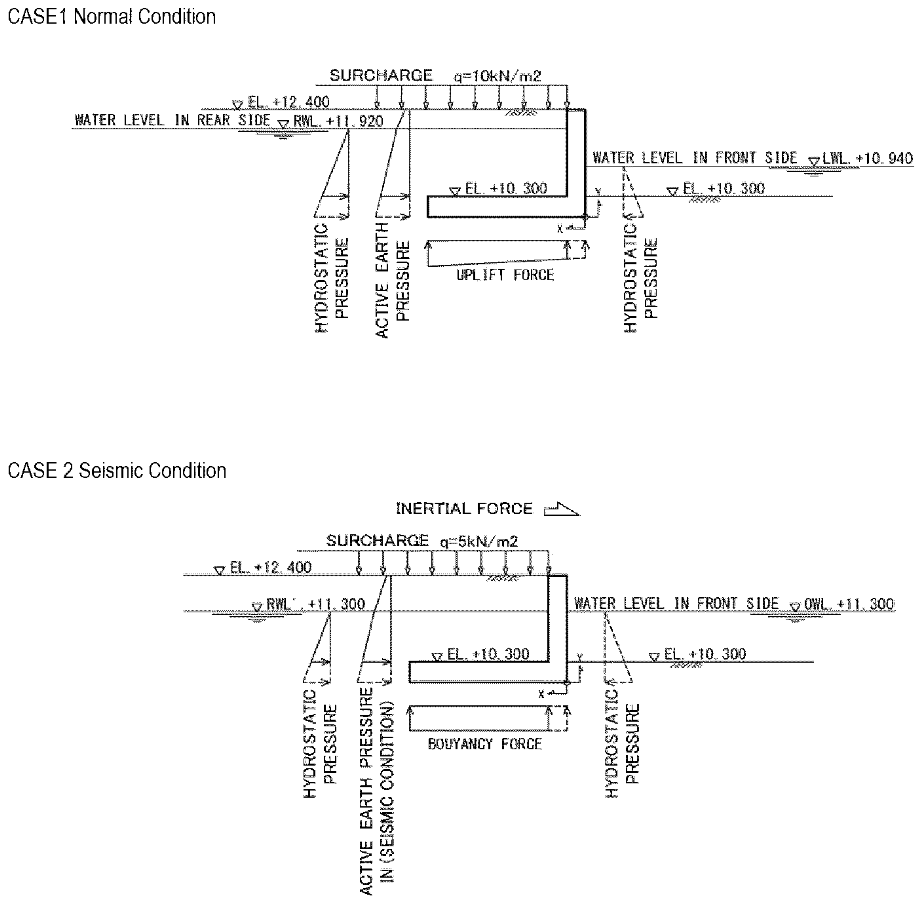
(b) Cross Section Calculation

(i) Design Policy

In the section calculation, the vertical wall is considered as a cantilever beam fixed to the bottom slab, and the bottom slab is also considered as a cantilever beam fixed to the vertical wall.

(ii) Calculation Case

Similar to the stability calculation described above, the calculation cases are two cases such as normal condition and seismic condition.



Source: Study Team

Figure7.5.56 Load Cases for Cross Section Calculations

(iii) Calculated Result

The verification results of stress intensity are shown in **Table7.5.56** and **Table7.5.57**.

Table7.5.56 Verification of Stress in Vertical Wall

Shape Name Title		[Rectangle Shape] Vertical Wall Base (Rear)	[Rectangle Shape] Seismic Vertical Wall Base (Rear)
Section Dimension B1 H1 (m) B2 H2 B3 H3		1.000 0.400 0.000 0.000 ------- 0.000	1.000 0.400 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		1.0000 0.3100 0.0000	1.0000 0.3100 0.0000
Reinforcement	d1/As1	0.3100 D 16 -4 8.042	0.3100 D 16 -4 8.042
	Total cm2	8.042	8.042
Cross-sectional force M kNm N kN S kN		 20.85 0.00 28.69	 25.19 0.00 33.43
Degree of stress σc σca N/mm2 σs σsa σs' σsa		1.9 < 8.28 91.0 < 168.00 -------	2.3 < 11.01 109.9 < 223.40 -------
Neutral axis x m		0.0752	0.0752
Young's modulus ratio		n = 9.00	n = 9.00
Sh kN Sh ' ' kN mean τ m N/mm2		28.69 0.000 0.09 < 0.360	33.43 0.000 0.11 < 0.470

Source: Study Team

Table7.5.57 Verification of Stress in Bottom Slab

shape name Title		[Rectangle Shape] Heel Slab (Top)	[Rectangle Shape] Seismic Heel Slab (Top)
Section Dimension B1 H1 (m) B2 H2 B3 H3		1.000 0.500 0.000 0.000 ------- 0.000	1.000 0.500 0.000 0.000 ------- 0.000
Web width bw m Effective height d m Digit height change tan γ		1.0000 0.4100 0.0000	1.0000 0.4100 0.0000
Reinforcement	d1/As1	0.4100 D 16 -4 8.042	0.4100 D 16 -4 8.042
	Total cm2	8.042	8.042
Cross-sectional force M kNm N kN S kN		 20.85 0.00 32.55	 25.19 0.00 30.46
Degree of stress σc σca N/mm2 σs σsa σs' σsa		1.2 < 8.28 68.1 < 168.00 -------	1.5 < 11.01 82.3 < 223.40 -------
Neutral axis x m		0.0881	0.0881
Young's modulus ratio		n = 9.00	n = 9.00
Sh kN Sh ' ' kN mean τ m N/mm2		32.55 0.000 0.08 < 0.360	30.46 0.000 0.07 < 0.470

Source: Study Team

5) Examination of surfacing (To the entire blade wall)

(a) Study Policy

Since the section of the U-shaped channel in the direction of flowing water is short, the influence of the L-shaped retaining wall load on the wing becomes large as a whole. Therefore, for levitation, the load of the U-shaped channel and wing is regarded as the total load, and the load acting downward is checked against the uplift pressure of the whole bottom slab.

(b) Load Case

Since the load cases are the outer surface water level in normal condition > at the time of earthquake and the inner surface water level in normal condition < at the time of earthquake, the load cases should be examined only in normal condition, omitting the time of earthquake. The diagrams are shown in**Figure7.5.57**It is as follows.

(c) Design Water Level

The design water level is **Figure7.5.57**It is as follows.

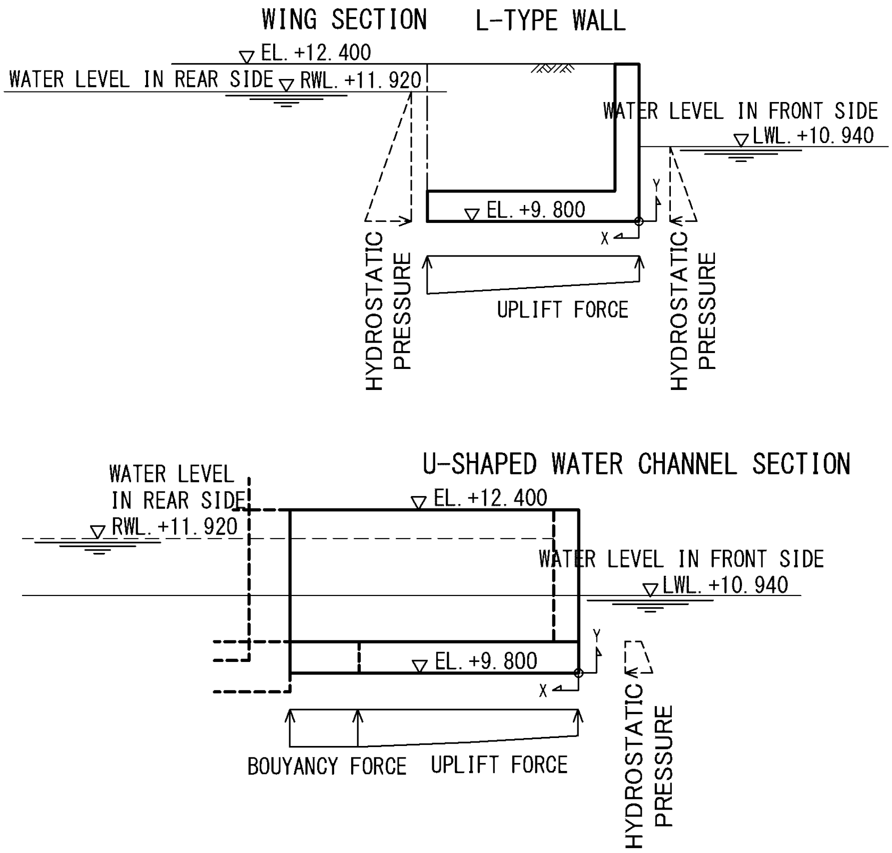


Figure7.5.57 Normal Load Condition

(d) Load Schedule

The load schedule results are displayed in the **Figure7.5.59**

Table7.5.58 Summary of Load Calculation

term eye			Symbol	Units	Formula/ Computation	Value	Remarks
Downward Load	Body Weight	Bottom Slab (1)	P	kN	x 0.50 x 3.50 x 23.70 x 24.0	995.40	
		Bottom Slab (2)	P	kN	24.0 × 0.50 × 1.10 × (10.75 + 10.32) × 1/2	139.06	
		Vertical Wall (3)	P	kN	24.0 x 0.40 x 2.10 x 5.87 x 2	236.68	
		Vertical Wall (4)	P	kN	24.0 x 0.41 x 2.10 x 4.60 x 2	190.11	
		Subtotal	P1	kN		1561.25	
	Inner Water Weight		P2	kN	9.8 × 0.64 × 4.60 × (9.50 + 11.30) × 1/2	300.05	U-shaped Channel
	Cover Soil		P3	kN	19.0 x 2.10 x 3.10 x (5.87 + 6.48)2 x 1/2	1527.57	
	Weight of Water in Soil		P4	kN	1.0 x 1.62 x 3.10 x (5.87 + 6.48)2 x 1/2	62.02	
	Total		Pv	kN	P1 + P2 + P3 + P4	3450.89	
Upward Load	Buoyancy and Uplift Pressure		Pu	kN	9.8 × 2.12 × 1.10 × (10.75 + 10.32) × 1/2 + 9.8 × (2.12 + 1.14)× 1/2 × 3.50 × 23.70	1565.81	

(e) Stability Checking

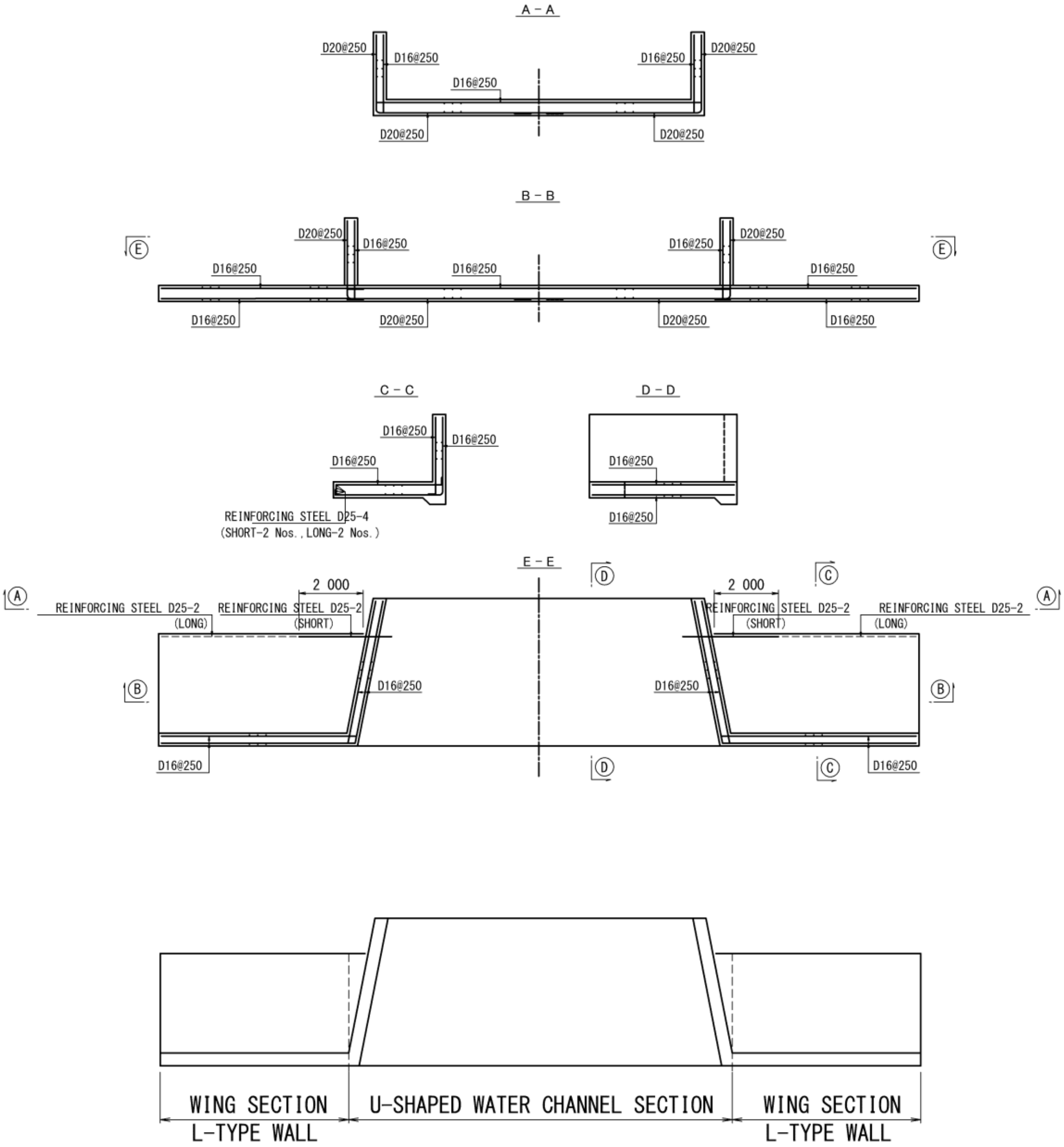
The results of the stability check for surfacing are as follows:

Table7.5.59 Results of the Stability Check

case			CASE1 Normal Condition	Remarks
Floating Stability	Composited Force of Downward Load	Pv (kN)	3450.89	
	Upward Load(Composited Force of Uplift Pressure and Buoyancy	Pu (kN)	1565.81	
	Safety Factor F = Pv/Pu		2.20	
	Allowable Safety Factor Fa		1.33	
	Evaluation		F > Fa -> OK	

6) Bar Arrangement Procedure

The following is a schematic diagram of reinforcement arrangement based on the calculation results.



Source: Study Team

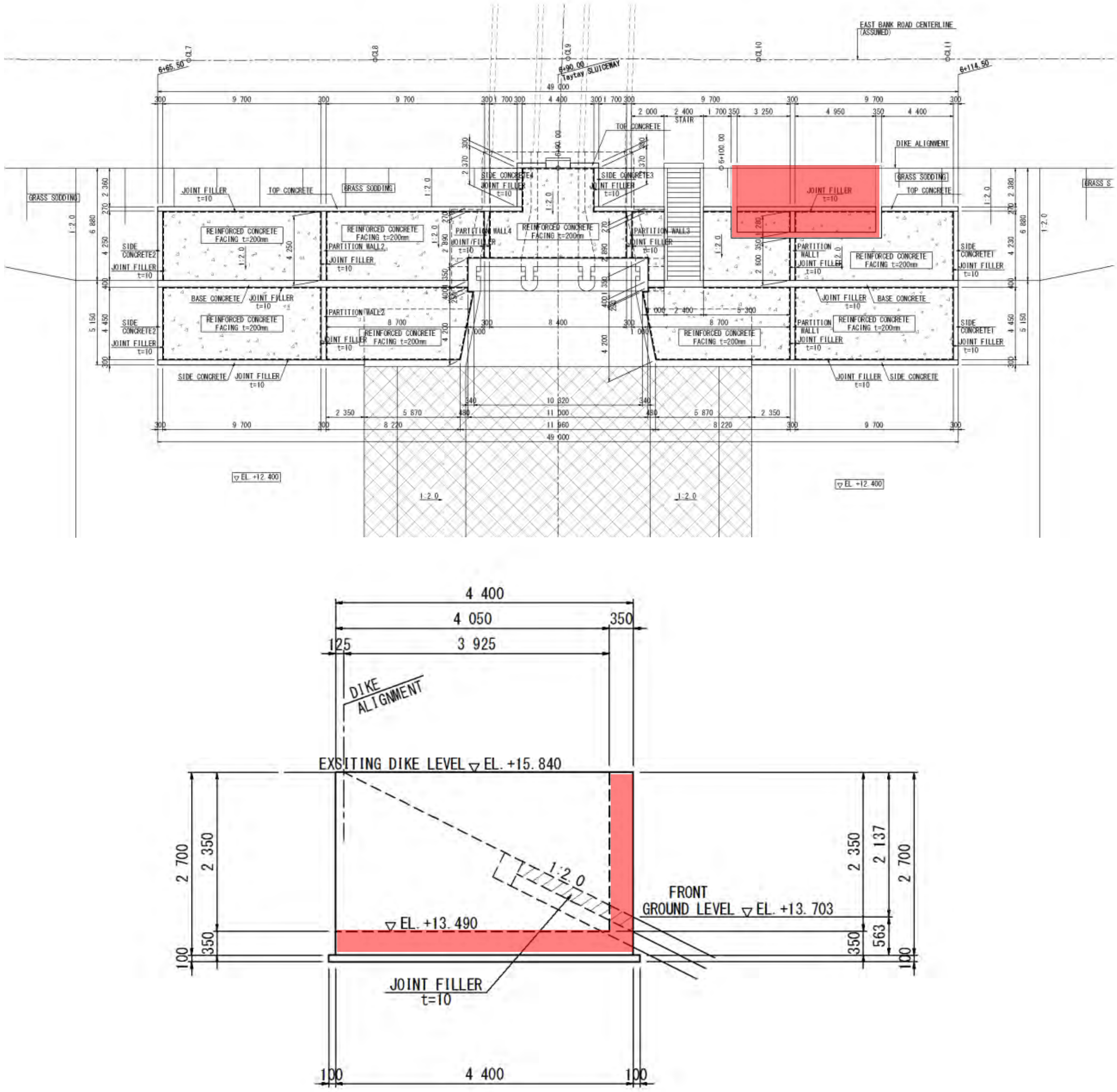
Figure7.5.58 Bar Arrangement

(7) Design of the Retaining Wall for Guard House

1) Study Policy

The retaining wall for guard house has an L-shaped retaining wall structure. The overall stability and structural dimensions are determined, and each member of the vertical wall and the bottom slab is designed as a cantilever.

For the location and structural dimensions are shown in **Figure7.5.59**



Source: Study Team

Figure7.5.59 Structural Dimensions of the Retaining Wall for Guard House

2) Calculation Case

While the main body of sluiceway, the breast wall and the wing wall are affected by the water level due to the waterside location, this retaining wall is above the ordinary water level and groundwater level and is about DFL, so the effect of water level is not considered.

Two cases such as normal and seismic condition are used for calculation.

3) Load Schedule

The load in each case is indicated in **Table7.5.60** and **Table7.5.61** The summary of load are shown in **Table7.5.62**.

Table7.5.60 Permanent Load

Item	Vertical Force Ni (kN)	Horizontal Force Hi (kN)	Arm Length		Moment of Rotation (kNm)	
			Xi (m)	Yi (m)	Mxi = Ni/Xi	Myi = Hi Yi
Dead Weight	233.392	0.000	2.112	0.000	492.858	0.000
Surcharge	118.500	0.000	2.325	0.000	275.513	0.000
Earth Pressure	0.000	33.260	4.300	0.900	0.000	29.934
Subtotal	351.892	33.260	———	———	768.371	29.934

Source: Study Team

Table7.5.61 Seismic Load

Item	Vertical Force Ni (kN)	Horizontal Force Hi (kN)	Arm Length		Moment of Rotation (kNm)	
			Xi (m)	Yi (m)	Mxi = Ni/Xi	Myi = Hi Yi
Dead Weight	233.392	46.679	2.112	1.312	492.858	61.230
Surcharge	98.750	19.750	2.325	2.700	229.594	53.325
Earth Pressure	19.530	40.612	4.300	0.900	83.979	36.551
Subtotal	351.672	107.041	———	———	806.431	151.105

Source: Study Team

Table7.5.62 Summary of Load

Load State (Water Level)	No (kN)	Ho (kN)	Mo (kNm)
Normal Condition	351.892	33.260	738.437
Seismic Condition	351.672	107.041	655.326

Source: Study Team

4) Calculated Result

(a) Stability analysis

The results of the stability calculation are shown in **Table7.5.63** to **Table7.5.66**

Table7.5.63 Verification Results for Overturning

Load State (Water Level)	Σ Mr (kNm)	ΣMt (kNm)	ΣV (kN)	d (m)	e (m)		Ea (m)	Evaluation
Normal Condition	768.371	29.934	351.892	2.098	0.052	<<	0.717	○
Seismic Condition	806.431	151.105	351.672	1.863	0.287	<<	1.433	○

Source: Study Team

Table7.5.64 Verification Results for Sliding

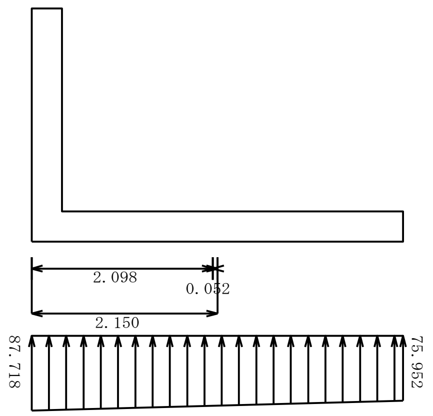
Load State (Water Level)	Eccentricity e (m)	Effective Loading Width B '(m)
Normal Condition	0.052	4.196
Seismic Condition	0.287	3.726

Load State (Water Level)	Vertical Load ΣV (kN)	Horizontal Load ΣH (kN)	Safety Factor Fs		Required Safety Factor Fsa	Evaluation
Normal Condition	351.892	33.260	6.348	≧	1.500	OK
Seismic Condition	351.672	107.041	1.971	≧	1.200	OK

Source: Study Team

Table7.5.65 Verification Results for Bearing Capacity

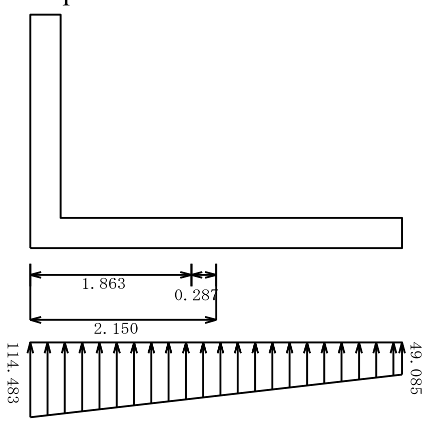
Normal Condition



Subgrade Reaction Width of Action (m)	Subgrade Reaction Shape	Subgrade Reaction (kN/m2)				Evaluation
		qmin	qmax		Allowance	
4.300	Trapezoid	75.952	87.718	≦	300.000	OK

Source: Study Team

earthquake time



Subgrade Reaction Width of Action (m)	Subgrade Reaction Shape	Subgrade Reaction (kN/m2)				Evaluation
		qmin	qmax		Allowance	
4.300	Trapezoid	49.085	114.483	≦	450.000	OK

Source: Study Team

Table7.5.66 Verification Results for Bearing Capacity

Load State (Water Level)	Effect Loading Area Ae (m2)	Allowable Bearing Capacity qa (kN/m2)	Acting Vertical Force V (kN)		Allowable Bearing Capacity Qa (kN)	Evaluation
Normal Condition	4.196	139.899	351.892	≦	587.018	OK
Seismic Condition	3.726	95.183	351.672	≦	354.651	OK

Source: Study Team

(b) Calculation Result of Vertical Wall and Bottom Plate

The calculation results of the vertical wall and the bottom slab were as follows, with details in the design calculation sheet.

Table7.5.67 Verification Result of Bending Stress of Vertical Wall And Bottom Slab

Cover (cm)	Re-bar Diameter	Rebar Area (cm2/piece)	Number of Re-bars	Reinforcement (cm2)
11.00	D20	3.871	4.000	12.566

Load State (Water Level)	M (kNm)	x (cm)	Compressive Stress (N/mm2)			Tensile Stress (N/mm2)			Evaluation
			Calculated		Allowance	Calculated		Allowance	
Normal Condition	26.899	7.810	3.218	<<	8.280	100.060	<<	168.000	OK
Seismic Condition	42.755	7.810	5.115	<<	11.010	159.040	<<	223.400	OK

Source: Study Team

Table7.5.68 Verification Result of Shear Stress of the Vertical Wall

Load State (Water Level)	Shear Force Sh (kN)	Effective Height d (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	CN	
Normal Condition	34.354	25.000	0.137	≦	0.360	1.600	1.40	1.20	1.00	OK
Seismic Condition	52.586	25.000	0.210	≦	0.470	2.130	1.40	1.20	1.00	OK

Source: Study Team

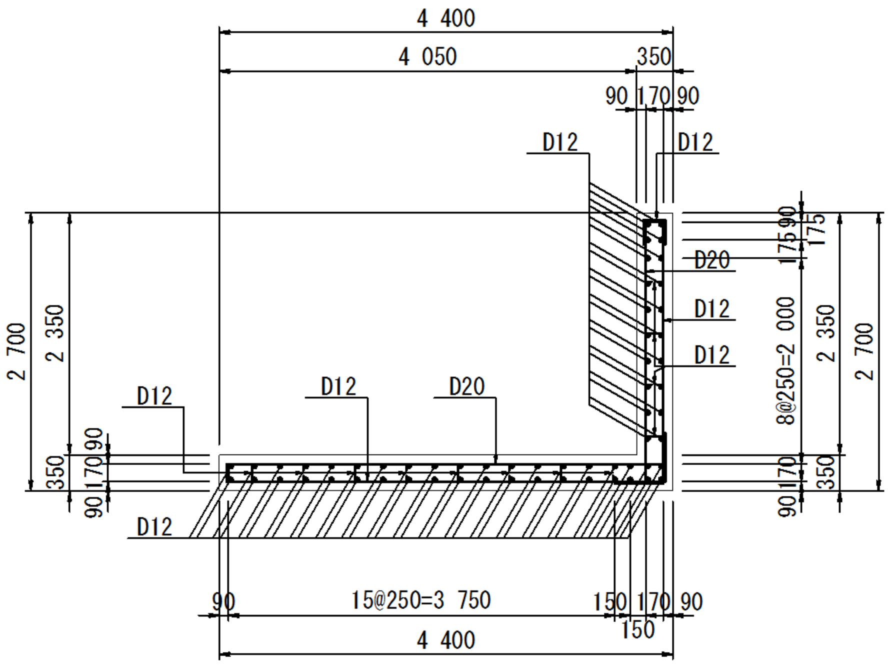
Table7.5.69 Verification Result of Shear Stress of the Bottom Plate

Load State (Water Level)	Shear Force Sh (kN)	Effective Height d (cm)	Shear Stress (N/mm2)				Correction Factor			Evaluation
			Calculated τ		Allowable τa1	Allowable τa2	Ce	Cpt	Cdc	
Normal Condition	7.959	24.000	0.033	≦	0.360	1.600	1.40	1.21	1.00	○
Seismic Condition	21.128	24.000	0.088	≦	0.470	2.130	1.40	1.21	1.00	○

Source: Study Team

5) Bar Arrangement

The bar arrangement of the guard house is shown in **Figure7.5.60**. The major bars is designated D 20 and the minor bars is designated D 12. The bar arrangement of the gable wall is the same.



Source: Study Team

Figure7.5.60 Bar Arrangement of Retaining Wall for Guard House

7.5.2.5 Main body Work (L2 Seismic Design)

(1) Design Criteria

Seismic performance verification of Taytay Sluiceways against L2 earthquake ground motions would be conducted in accordance with the "Performance Based Seismic Design Criteria for River Structures, 2012.2".

(2) Seismic Design Condition

1) Target for Verification and Setting of Seismic Performance

The aforementioned standard specifies the performance of sluiceway for L1 and L2 earthquake ground motions. The breakdown is indicated in **Table7.5.70**.

Taytay Sluiceway will be improved to provide a backflow prevention function that is not currently possessed. Therefore, it will be positioned as an important sluiceway for flood control and will require "Seismic Performance 2". The functions required for "Seismic Performance 2" and the method of verifying seismic performance is summarized in **Table7.5.71**.

Table7.5.70 Seismic Motion, Seismic Performance, and Applicable Facility

Earthquake Motion	Seismic Performance		Applicable Facility
	Category	Performance	
L1 Earthquake Ground Motion	Seismic Performance 1	Performance that it does not lose the soundness of sluiceway, floodgate or weirs due to an earthquake	All Facilities
L2 Earthquake Ground Motion	Seismic Performance 2 **Apply to this Facility**	Performance to maintain the function of sluiceway even after an earthquake	Floodgate/ Sluiceway Which is Important For Flood Control or Water Utilization
	Seismic Performance 3	Performance in which the damage caused by an earthquake is limited and the functions of sluiceway can be recovered quickly	Other Sluiceway and Sluiceway

Source: Study Team

Table7.5.71 Seismic Performance and Seismic Verification Items to be Secured

Structure	Sluiceway Longitudinal Direction	Column
Assumed Damage	 The section on a sluiceway coming off and 10 cm gap in joint occurs (15 Tokachi-oki Earthquake, Tokachi-gawa Otsu City Sluiceway)	 Damage to Columns Caused by the earthquake (16 Niigata Chuetsu Earthquake, Shinano-gawa River Myoken Weir)
Standard	Joint and Column of sluiceway main body: "Performance Based Seismic Design Criteria for River Structures, 2012.2", Flood Control Division of MLIT, Japan	
Required Performance to	To Prevent Secondary Disaster by Flood Even If It is Damaged by An Earthquake. => Functions to Prevent Backflow from the Main River Channel and Ensure Function of Dike	
Seismic performance	**Seismic performance 2**: To Maintain the Function as a Sluiceway Even after an Earthquake	

Structure	Sluiceway Longitudinal Direction	Column
Seismic Performance Verification Method	**Dikes**: Static FEM Analysis Considering Liquefaction, Deformation of Joints *Normally, "Analysis by beam on elastic floor considering ground settlement and horizontal displacement due to Liquefaction" is carried out in the longitudinal direction of the sluiceway. However even if only the newly installed part has the capability, it does not make sense in the situation, when the existing part is damaged. Therefore, it is verified that the deformation of the joint does not cause damage on the sluiceway function, when the existing part and the new part are not damaged in the longitudinal direction. **Column**: Seismic Horizontal Bearing Capacity Method	

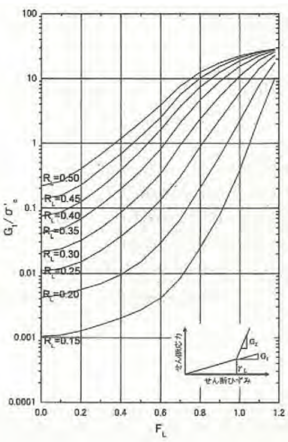
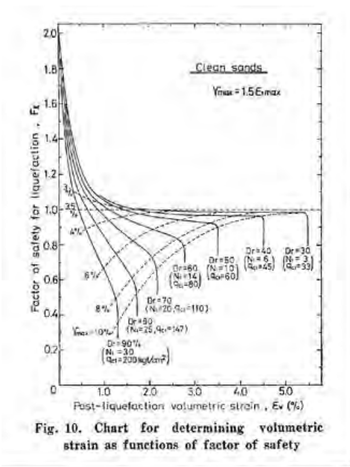
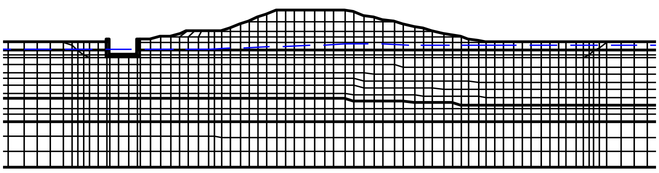
Source: Study Team

2) Examination Technique

(a) Checking the Culvert

An overview of the L2 earthquake analysis in the longitudinal direction of a culvert is shown in **Table7.5.72**.

Table7.5.72 Deformation Analysis of Foundation Ground

Item	Specifications	Remarks
Target Ground Motion	Level 2 earthquake ground motion	BSDS, see below
F_L Value	Calculated in the Gravity Deformation Analysis	The calculation method follows "Performance Based Seismic Design Criteria for River Structures."
Analytical Technique (Code Name: ALID)	**[Settlement due to Decreased Rigidity]** Self-Weight Deformation Analysis Using Finite Element Method Using the rigidity lowered by the liquefaction layer, the settlement and deformation by the self-weight analysis (static FEM) is analyzed. The rigidity decreasing due to liquefaction is determined by the relationship shown in Figure- (1). **[Settlement After Liquefaction]** Calculation of Settlement due to Volumetric Compression of Liquefied Layer The relationship shown in Fig. -(2) is used to calculate the settlement caused by the dissipation of excess pore water pressure. This relationship is an example of an experiment on the volumetric strain associated with the dissipation of excess pore water pressure. The volumetric strain is obtained from F^L and D^r (relative density).	*Source: Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes(Ishihara, K and Yoshi mine, M), Soils and Foundations, vol.28, No.1,pp41-50,1998* Figure - (1) (Method by Yasuda and Inagaki) Figure - (2)
Settlement Calculation Method	**[Sluiceway]** Settlement due to Rigidity Reduction during Liquefaction + Settlement due to Volumetric Compression after Liquefaction	The deformation amount of the sluiceway lower part is calculated from the result of the self-weight deformation analysis.  Side: Vertical roller (Horizontal Fixed) Bottom: fixed

Source: Study Team

(b) Evaluation of Columns

The columns shall be checked for the following two conditions: (1) the seismic horizontal capacity of the column should not be less than the inertial force acting on the column, and (2) the residual displacement of the column should be less than the allowable residual displacement.

(i) Flow Direction

To conduct the verification in flow direction, calculate the inertia force shared by each member of the end column and the middle column, and confirm the seismic bearing capacity and residual displacement as a single column type concrete bridge pier.

$$\begin{cases} Pa \geqq khcW & \cdots \text{ Verification of Seismic Horizontal Bearing Capacity} \\ \delta Ra \geqq \delta R & \text{ Verification of Residual displacement} \end{cases}$$

Here,

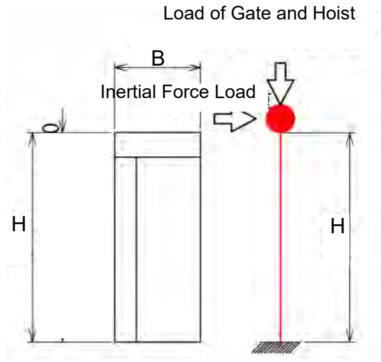
Pa : Horizontal Bearing Capacity [kN]

khc : Design Horizontal Seismic Coefficient

W : Equivalent Weight [kN]

δ Ra : Allowable Residual Displacement [m]

δ R : Residual Displacement [m]



Source: Study Team

Figure7.5.61 Flow Direction Model Diagram

(ii) Direction Perpendicular to Flow

In the verification in the direction perpendicular to water flow, the column system is evaluated by the double rigid frame, and the horizontal bearing capacity and residual displacement in the earthquake are checked according to the failure type.

$$\begin{cases} khu \geqq khc & \text{ Verification of Seismic Horizontal Bearing Capacity} \\ \delta Ra \geqq \delta R & \text{ Verification of Residual Displacement} \end{cases}$$

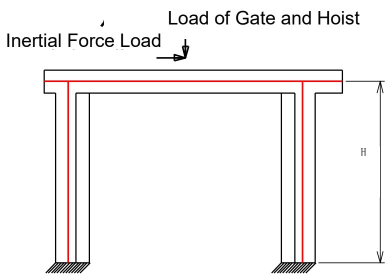
Here.

Khc : Design Horizontal Seismic Coefficient

khu : Horizontal Seismic Coefficient at the Ultimate State

δ Ra : Allowable Residual Displacement [m]

δ R : residual Displacement [m]



Source: Study Team

Figure7.5.62 Flow Right Angle Model Diagram in perpendicular Direction to Flow(In the case of single-strand ramen)

The pushover analysis in the rigid frame model, which is performed by checking the flow direction and the direction perpendicular to the flow direction, is as follows.

■Pushover Analysis

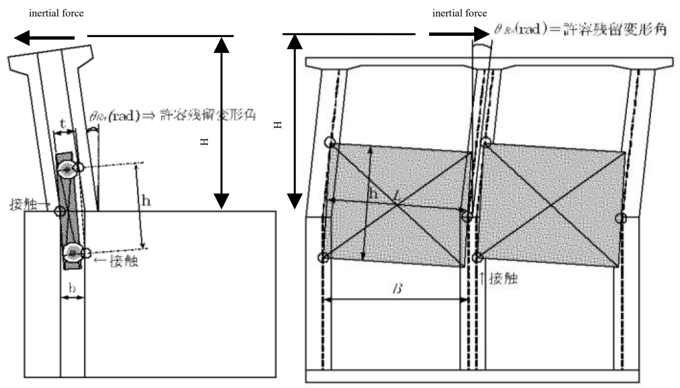
The calculation method of the horizontal bearing capacity of rigid frame structures using pushover analysis is stated in Specifications for Highway Bridges V Seismic Design (page: 179 to 189). The analysis procedure is as follows.

1. The M- ϕ relationship is calculated for each member (columns and beams) constituting the rigid frame structure. (Plastic hinge points at both ends of the member)

2. As the dimensions of the rigid frame structure, Mu, ϕu, and ϕy of each member end determined from the M- ϕ diagram are inputted. The M- ϕ relationship is trilinear (double bend) in accordance with the provisions of Specifications for Highway Bridges V Seismic Design, but the analytical model is input as bilinear, ignoring the crack time, in the same way as the P- δ curve.
3. The horizontal force loaded on the rigid frame structure is gradually increased (Number of analysis steps 10000 ~ 30000), and each plastic hinge is focused on, and the location where the initial yield and the final failure occur and the load value at that time are calculated.
4. The obtained bearing capacity is compared with the inertia force kh· W applied to the whole rigid frame structure.

(iii) Evaluation of Gate

For verification of gates, an "Allowable residual displacement determined from reparability" is set as H/100 with the aim of limiting damage due to earthquakes and restoring functions as sluiceway and weirs.



H: the height from the lower end of the column to the position where the inertial force of the superstructure acts
Source: Performance Based Seismic Design Criteria for River Structures IV. Floodgates, Sluiceway and Weirs

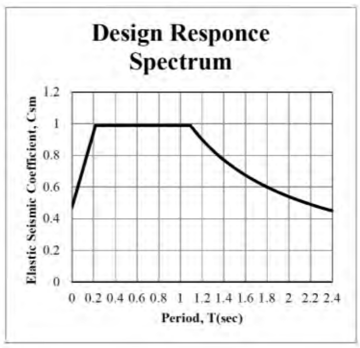
Figure7.5.63 Schematic Diagram for Verification of the Gate

3) Design Horizontal Seismic Coefficient

The design horizontal seismic coefficient k_{hgL} is as follows, based on the fact that the ground concerned is a type III ground, and PGA = 0.60, Ss = 1.20, and S1 = 0.45 are obtained by reading the figure in the BSDS. In Japanese standard, the Level 2 earthquake motion has a plate boundary type and inland direct type and the earthquake to be used in this design is categorized under the inland direct type.

F_{pga} = 0.78, F_a = 0.82, F_v = 2.4

① PGA:		0.6 (BSDS図3. 4. 1−4より)
F_{PGA}:		0.78 (Soil Type Ⅲ)
	AS=	0.47
② Ss:		1.2 (BSDS図3. 4. 1−5より)
Fa		0.82 (Soil Type Ⅲ)
	S_{DS}=	0.99
③ S1:		0.45 (BSDS図3. 4. 1−5より)
Fv		2.4 (Soil Type Ⅲ)
	S_{D1}=	1.08



Therefore, the design horizontal seismic coefficient k_{hgL} is as follows.

$k_{hgL} = F_{pga} \times PGA = 0.78 \times 0.60 = 0.468 \fallingdotseq$ **<u>0.47 (Level 2 Design Horizontal Seismic Coefficient at Ground Level)</u>**

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

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

0.78

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

Table 3.5.3-2 Values of Site Factor, F_a, for Short-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 2.0$
I	1.2	1.2	1.1	1.0	1.0	1.0
II	1.6	1.4	1.2	1.0	0.9	0.85
III	2.5	1.7	1.2	0.9	0.8	0.75

0.82

Note:
[1] Use straight-line interpolation for intermediate values of S_S.

Table 3.5.3-3 Values of Site Factor, F_v, for Long-Period Range on Acceleration Spectrum

Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.80$
I	1.7	1.6	1.5	1.4	1.4	1.4
II	2.4	2.0	1.8	1.6	1.5	1.5
III	3.5	3.2	2.8	2.4	2.4	2.0

2.4

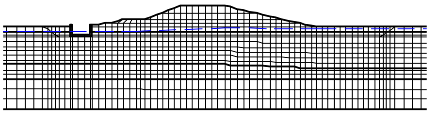
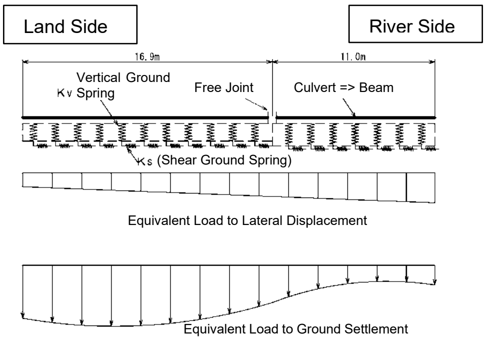
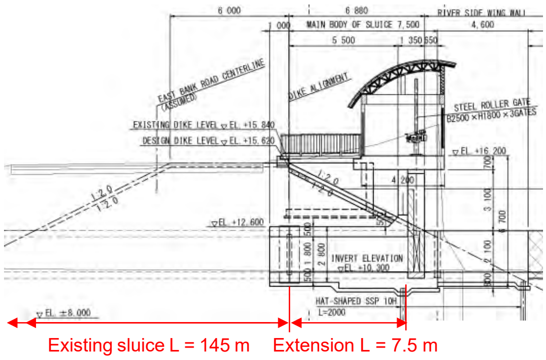
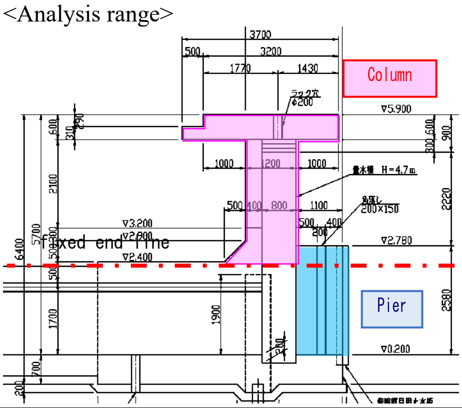
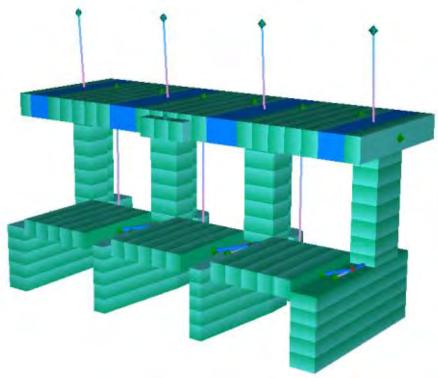
Note:
[1] Use straight-line interpolation for intermediate values of S_1.

4) Seismic Performance Verification of Taytay Sluiceway against L2 Earthquake Ground Motion

The seismic performance verification for L2 earthquake ground motion is carried out at ordinary sluiceway is described in **2) Examination Technique**, **Table7.5.73** shown it in the diagram.

On the other hand, Taytay Sluiceway is connected to the existing culvert (L = 145 m), which is different from a general sluiceway. The culvert does not have detailed information such as reinforcement arrangement specifications and joint structure. Considering this situation, Seismic performance of Taytay Sluiceways is checked as shown in **Table7.5.73**. Although the flow of verification is different from that of usual sluiceway, the object of checking member is covered by this verification.

Table7.5.73 Method of Seismic Performance Verification (Ordinary Sluiceway and Taytay Sluiceway)

		Ordinary Sluiceway	Taytay Sluiceway
Box Culver	Schematic Diagram	① Self-weigh Deformation Analysis  ② Beam Analysis Considering Ground Deformation  	<Profile View> 
	Concept	Using the rigidity reduced by the liquefied layer, the land settlement/horizontal displacement is calculated based on the settlement deformation and the settlement caused by the dissipation of the excess pore water pressure by the dead weight analysis (static FEM) ((1)). Then, the ground displacement is applied to the beam as a load through the ground spring, and the longitudinal direction of the culvert is modeled and analyzed to the beam on the elastic floor, and the sectional force ,the displacement and the deviation of the joint are checked ((2)). The box culvert has a frame structure with high rigidity, and the sluiceway has not been damaged by the L2 earthquake, so this was excluded.	Taytay Sluiceway cannot be studied in the same way as the ordinary sluiceway shown on the left, because it is a connecting the existing sluiceway and the detailed information on the reinforcement and joint of is not available. Therefore, only the static FEM analysis was carried out to check the displacement and the amount of deviation of the joint.
Column	Schematic Diagram	<Analysis range>  	<Model diagram> 
	Concept	The column of the sluiceway is separated from the pier, and the upper part of the sluiceway separated from the pier is studied in accordance with **(i)** Flow Direction, **(ii)** Perpendicular Direction to Flow and **(iii)** Verification of gate (p7-842~7-843).	In the above, only the displacement and the amount of deviation are checked in the longitudinal direction of the box culvert, so there is a concern about the stability of the box culvert against L2 earthquakes. Therefore, as shown above, a three-dimensional model of the column and the pier is developed to verify the stability against the L2 earthquake motion.

Source: Study Team

(3) Calculation in Longitudinal Direction of Sluiceway

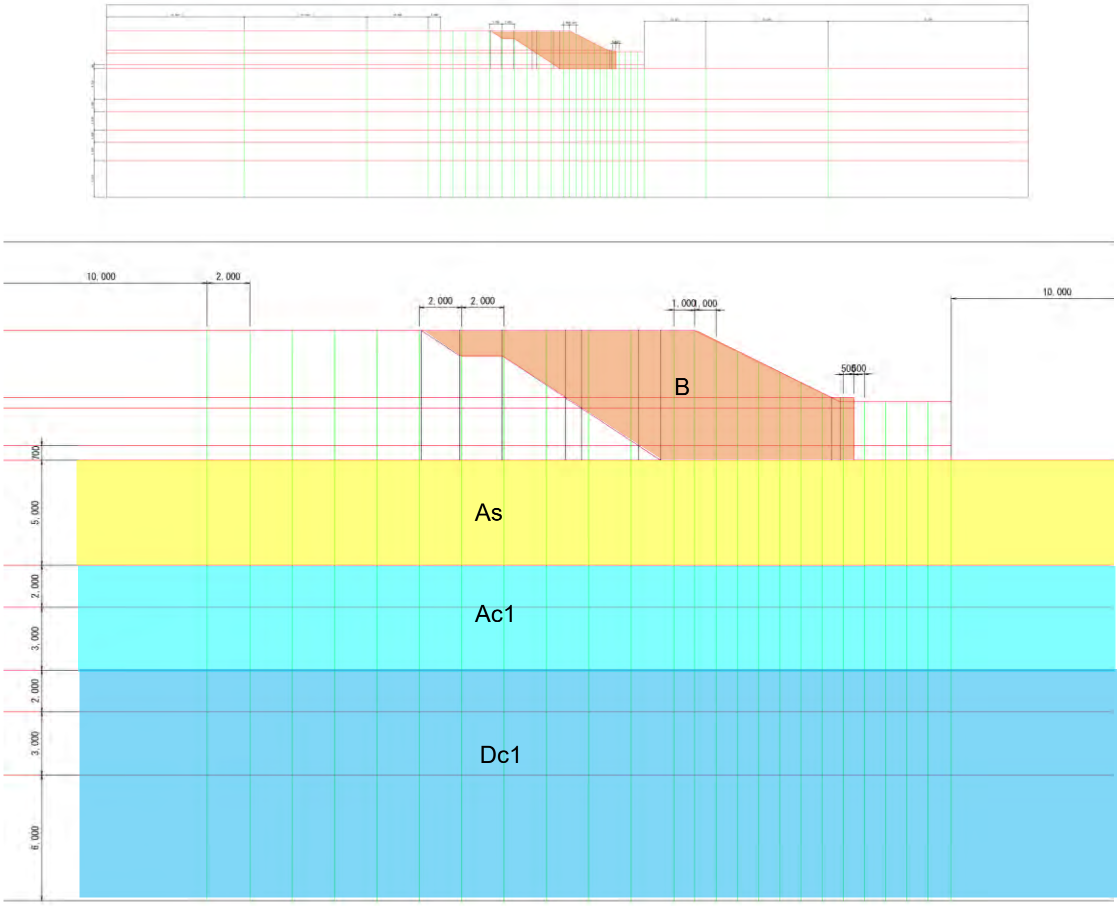
In the calculation in longitudinal direction, the ground deformation analysis by the static FEM is carried out, and the ground deformation of the joint is calculated, and the specification in the case that large deformation in the joint is expected is set.

1) Analytical Model

The analytical model is shown in **Figure7.5.64**. Soil constants are established based on the geological survey in DD-BH-T02.

- Boundary Conditions:

Bottom	Both XY Fixed
Both End	Fixed in X Direction, and Ground Constant of Roller In Y Direction



Source: Study Team

Figure7.5.64 Analytical Model (Upper: Entire Model, Lower: Enlarged Model)

Table7.5.74 Soil Constant

土質定数(地震時)

NO.	Name		Liquefaction Target Layer	N-Value	Wet Density γt (kN/m3)	Deformation Coefficent E (kN/m2)	Cohesion C (kN/m2)	Internal Friction Angle ϕ (°)	Poisson Ratio ν	Dilatancy ψ (°)	Relative Density Dr (%)	Shear modulus G (kN/m2)	Deformation Charactoristic	Type of Non-linerar	Fc (%)	Cyclic triaxial shear stress ratio RL	D50 (mm)	σv'	σm Lower Limit
1	Embankment	B		15	19	10,500	0	30	0.35	5	—	7,778	Undrained Deformation	MC/DPelasto-plastic	80.0	—	0.020	—	9.8
2	Sandy	As	○	21	20	14,700	0	37	0.35	12	103.54	10,889	Liquefaction Element	MC/DPelasto-plastic	4.0	2.154	0.440	28.0	9.8
3	Sandy	As	○	21	20	14,700	0	37	0.35	12	93.96	10,889	Liquefaction Element	MC/DPelasto-plastic	4.0	0.641	0.500	58.0	9.8
4	Clayey	Ac1		1	15	1,500	14	0	0.45	0	62.69	1,034	Undrained Deformation	MC/DPelasto-plastic	80.0	0.128	0.025	76.5	9.8
5	Clayey	Ac1		12	17	8,400	150	0	0.40	0	83.04	6,000	Undrained Deformation	MC/DPelasto-plastic	75.0	0.352	0.025	90.6	9.8
6	Clayey	Ac1		13	17	9,100	160	0	0.40	0	84.13	6,500	Undrained Deformation	MC/DPelasto-plastic	62.0	0.705	0.040	108.1	9.8
7	Clayey	Ac1		12	17	8,400	150	0	0.40	0	83.75	6,000	Undrained Deformation	MC/DPelasto-plastic	85.0	0.567	0.025	139.6	9.8
8	Clayey	Dc1		10	17	7,000	120	0	0.40	0	74.76	5,000	Undrained Deformation	MC/DPelasto-plastic	88.0	0.234	0.025	192.1	9.8
9	Clayey	Dc1		25	18	17,500	310	0	0.35	0	84.80	12,963	Undrained Deformation	MC/DPelasto-plastic	82.0	0.379	0.025	235.4	9.8
10	Clayey	Dc1		35	19	24,500	430	0	0.35	0	90.02	18,148	Undrained Deformation	MC/DPelasto-plastic	64.0	1.230	0.040	265.2	9.8
11	Clayey	Dc1		50	19	35,000	620	0	0.35	0	94.64	25,926	Undrained Deformation	MC/DPelasto-plastic	60.0	2.562	0.040	301.2	9.8

Source: Study Team

2) Liquefaction Assessment

The results of the liquefaction assessment based on the results of the geological survey in DD-BH-T02 are shown in **Table7.5.75**. Since the FL value of the As layer is less than 1, the As layer becomes a layer to be liquefied.

Table7.5.75 Liquefaction Judgment Result

<Soil Condition>

Depth of GWL(m)	GL-	0.000	(m)
Ground Elvation	TP	+9.63	(m)
Unit Weigth of Water	γw=	9.80	(kN/m^3)

Layer	Unit Weight γ(kN/m^3)	Upper Limit Depth (m)	Loaer Limit Depth (m)	σ_v at the Bottom (kN/m^2)
1	20.00	0.00	5.00	100.00
2	20.00	5.00	7.00	140.00
3	15.00	7.00	10.00	185.00
4	17.00	10.00	12.00	219.00
5	17.00	12.00	15.00	270.00
6	17.00	15.00	21.00	372.00
7	17.00	21.00	30.00	525.00
8	18.00	30.00	33.00	579.00
9	19.00	33.00	37.00	655.00
10	19.00	37.00	40.45	720.55

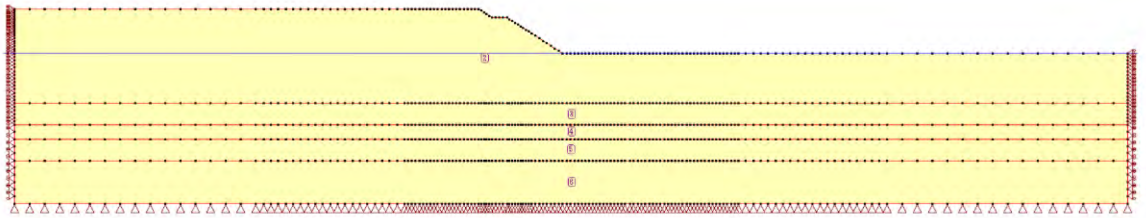
Layer	Elev. (m)	Depth (m)	N-Value	Fc (%)	Cfc	D50 (mm)	σ_v (kN/m^2)	σ_v' (kN/m^2)	In Case of Sand C_1	C_2	N_1	N_a	N_a	R_L	C_w	r_d	R	L	F_L
1	8.33	1.30	15.0	4.0	1.00	0.400	26.0	13.3	1.00	0.00	30.63	30.63	30.63	0.873	1.00	0.981	0.873	0.904	0.966
2	7.33	2.30	22.0	4.0	1.00	0.400	46.0	23.5	1.00	0.00	40.02	40.02	40.02	4.167	1.00	0.966	4.167	0.890	4.683
3	6.33	3.30	21.0	4.0	1.00	0.480	66.0	33.7	1.00	0.00	34.44	34.44	34.44	1.660	1.00	0.951	1.660	0.876	1.895
4	5.33	4.30	23.0	4.0	1.00	0.480	86.0	43.9	1.00	0.00	34.34	34.34	34.34	1.632	1.00	0.936	1.632	0.862	1.893
5	4.33	5.30	22.0	4.0	1.00	0.500	106.0	54.1	1.00	0.00	30.15	30.15	30.15	0.808	1.00	0.921	0.808	0.848	0.953
6	3.33	6.30	20.0	4.0	1.00	0.500	126.0	64.3	1.00	0.00	25.32	25.32	25.32	0.429	1.00	0.906	0.429	0.834	0.514
7	2.33	7.30	0.0	80.0	5.33	0.025	144.5	73.0	3.00	3.89	0.00	0.00	0.00	0.098	1.00	0.891	0.098	0.829	0.118
8	1.33	8.30	1.0	80.0	5.33	0.025	159.5	78.2	3.00	3.89	1.15	1.93	1.93	0.131	1.00	0.876	0.131	0.840	0.156
9	0.33	9.30	2.0	80.0	5.33	0.025	174.5	83.4	3.00	3.89	2.22	3.74	3.74	0.155	1.00	0.8605	0.155	0.847	0.184
10	-0.67	10.30	11.0	75.0	4.92	0.025	190.1	89.2	2.75	3.61	11.75	19.80	19.80	0.305	1.00	0.846	0.305	0.847	0.360
11	-1.67	11.30	14.0	75.0	4.92	0.025	207.1	96.4	2.75	3.61	14.31	24.11	24.11	0.385	1.00	0.831	0.385	0.839	0.459
12	-2.67	12.30	13.0	62.0	3.83	0.040	224.1	103.6	2.10	2.89	12.73	20.52	20.52	0.314	1.00	0.816	0.314	0.829	0.378
13	-3.67	13.30	22.0	62.0	3.83	0.040	241.1	110.8	2.10	2.89	20.69	33.35	33.35	1.376	1.00	0.801	1.376	0.819	1.680
14	-4.67	14.30	13.0	62.0	3.83	0.040	258.1	118.0	2.10	2.89	11.76	18.95	18.95	0.297	1.00	0.786	0.297	0.808	0.367
15	-5.67	15.30	23.0	85.0	5.75	0.025	275.1	125.2	3.25	4.17	20.03	33.76	33.76	1.478	1.00	0.771	1.478	0.796	1.856
16	-6.67	16.30	14.0	85.0	5.75	0.025	292.1	132.4	3.25	4.17	11.76	19.82	19.82	0.306	1.00	0.756	0.306	0.784	0.390
17	-7.67	17.30	13.0	85.0	5.75	0.025	309.1	139.6	3.25	4.17	10.55	17.77	17.77	0.286	1.00	0.741	0.286	0.771	0.371
18	-8.67	18.30	10.0	85.0	5.75	0.025	326.1	146.8	3.25	4.17	7.84	13.22	13.22	0.247	1.00	0.726	0.247	0.758	0.326
19	-9.67	19.30	11.0	85.0	5.75	0.025	343.1	154.0	3.25	4.17	8.35	14.07	14.07	0.254	1.00	0.7105	0.254	0.744	0.341
20	-10.67	20.30	23.0	85.0	5.75	0.025	360.1	161.2	3.25	4.17	16.91	28.50	28.50	0.631	1.00	0.6955	0.631	0.730	0.864
21	-11.67	21.30	13.0	88.0	6.00	0.025	377.1	168.4	3.40	4.33	9.27	15.62	15.62	0.267	1.00	0.6805	0.267	0.716	0.373
22	-12.67	22.30	10.0	88.0	6.00	0.025	394.1	175.6	3.40	4.33	6.92	11.67	11.67	0.234	1.00	0.6655	0.234	0.702	0.334
23	-13.67	23.30	13.0	88.0	6.00	0.025	411.1	182.8	3.40	4.33	8.74	14.73	14.73	0.260	1.00	0.6505	0.260	0.688	0.378

Source: Study Team

3) Analysis Step

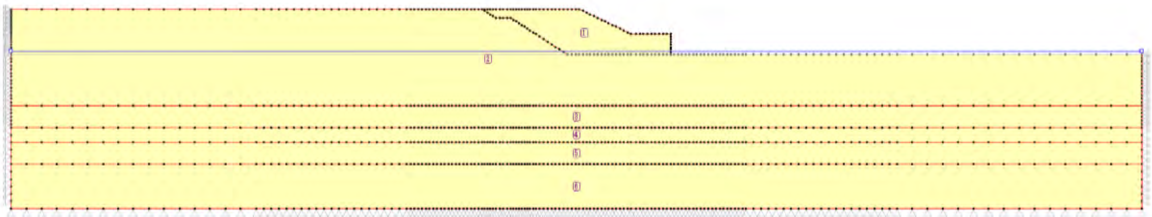
The analysis steps are shown in **Figure7.5.65**.

1) Initial Ground



2) Culvert Addition

3) Water Level Fluctuation



4) Liquefaction Flow

5) Liquefaction Settlement

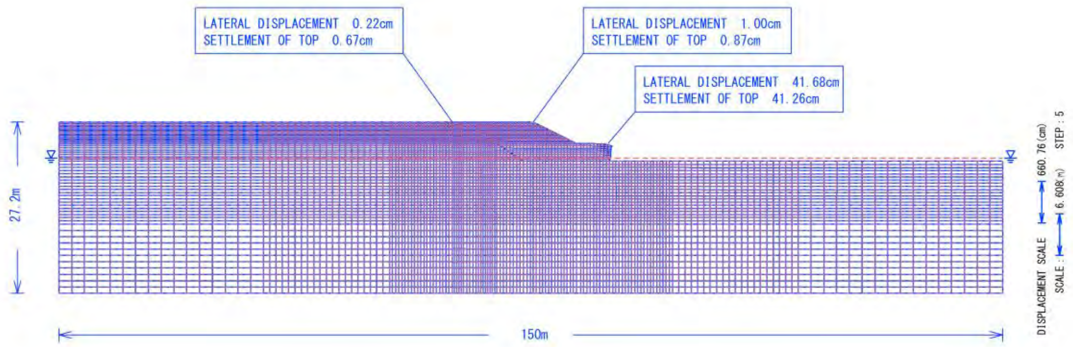
Source: Study Team

Figure7.5.65 Analysis Step Diagram

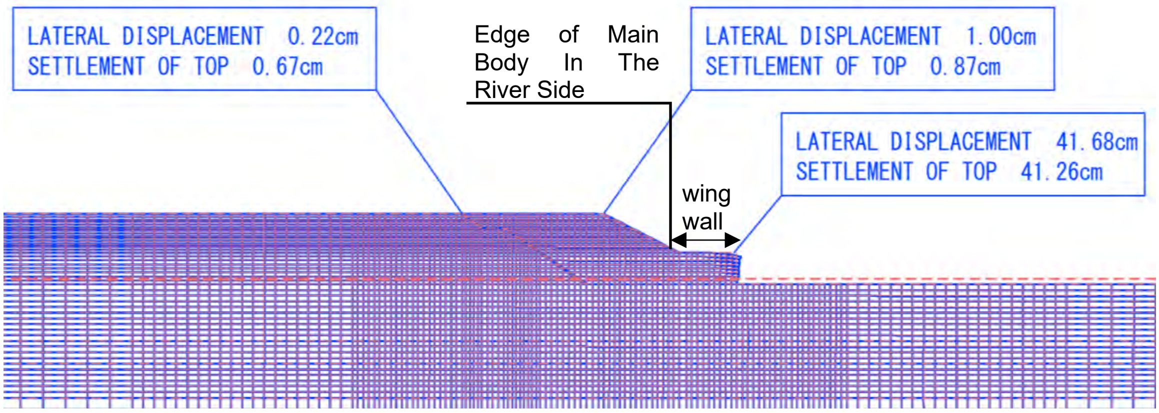
4) Analysis Result

As a result of the analysis of the L2 earthquake motion under the above-mentioned conditions, the ground deformation (lateral displacement, settlement) was very small, 1 cm at the dike crown in the river side and 0.5 cm at the end of dike excavation, and the effect of the earthquake was very small. In the mesh diagram, deformation of about 40 cm occurs at the end of the wing wall. However, since the ground is tied by the breast wall, wing wall, etc., which are difficult to input into the mesh conditions, this deformation is unlikely to occur.

■Computation Result (Entire)



■Computation Result (Close up)

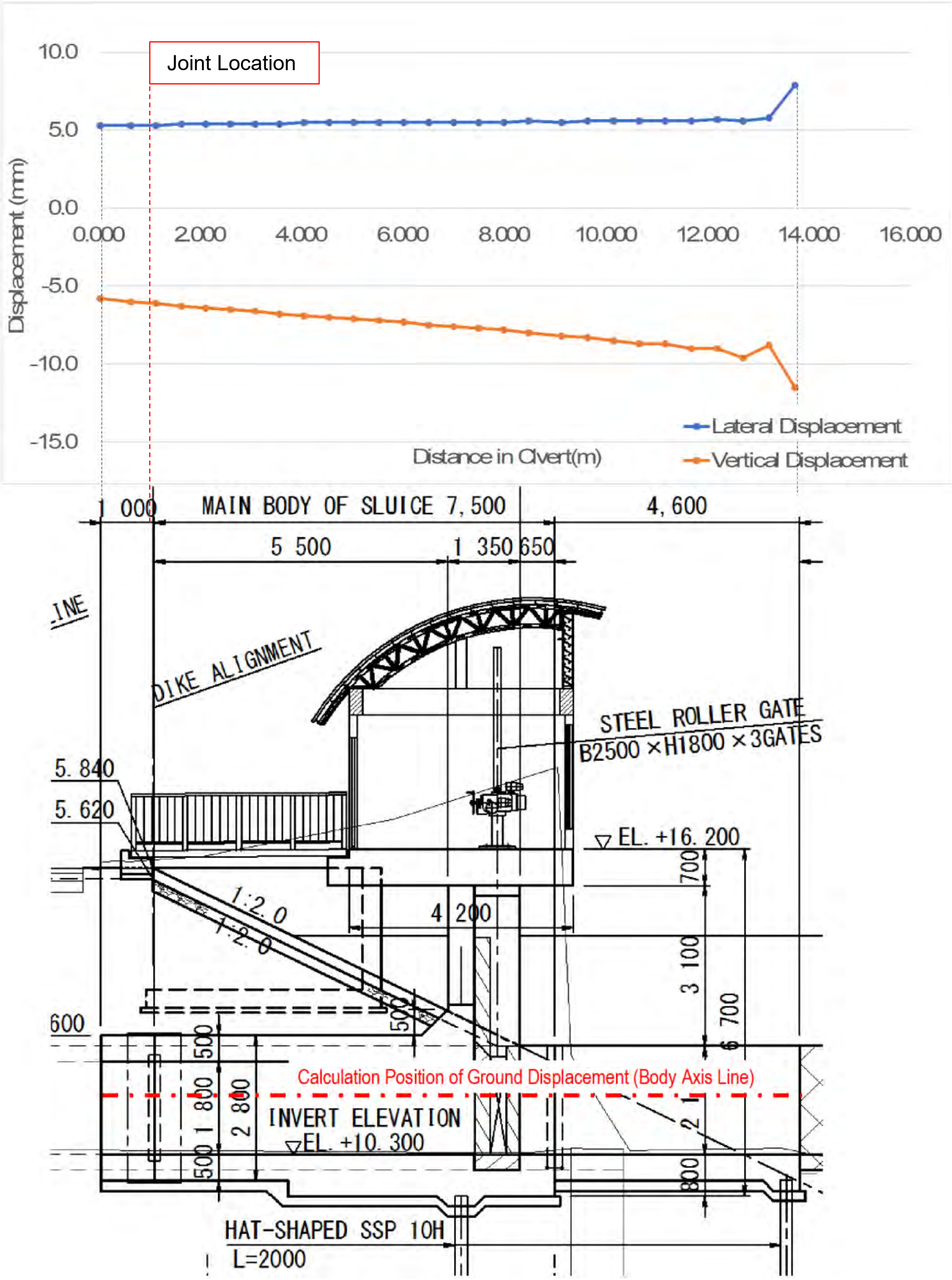


Source: Study Team

Figure7.5.66 FEM Deformation Quantity

The ground deformation due to an earthquake on the installation axis of Taytay Sluiceways is as shown in **Figure7.5.67** Both the horizontal and vertical displacements are about 5 mm at the existing and new joint positions.

z



Source: Study Team

Figure7.5.67 Ground Deformation at Main Body

5) Verification of Joints

The results of verification the opening of the joints and calculating the alignment gap after the L2 earthquake are shown in **Table7.5.76**, this result is calculated considering both immediate and consolidation settlement.

In both cases, the computed values are lower than the capacity of the flexible joints. Therefore, the joints will not be damaged even if an L2 earthquake occurred.

Table7.5.76 Verification Results of Joint

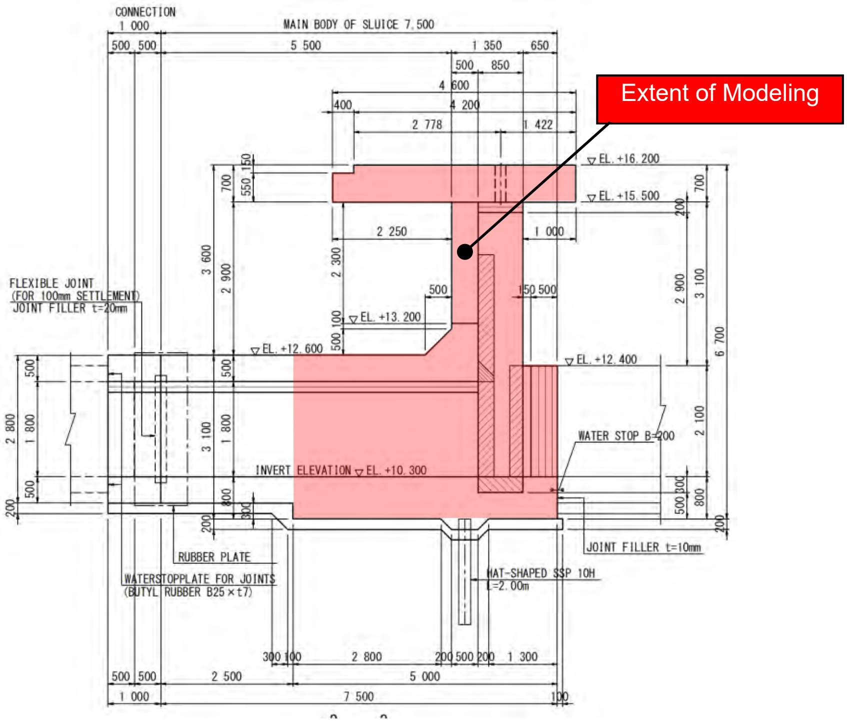
Item	Calculated Result	Capacity	Calculated Result
Opening	16.5 mm	100 mm	Within the Capacity
False alarm	Start 131.0 mm, End 132.1 mm 1.1 mm gap	200 mm	Within the Capacity

Source: Study Team

(4) Verification of Column

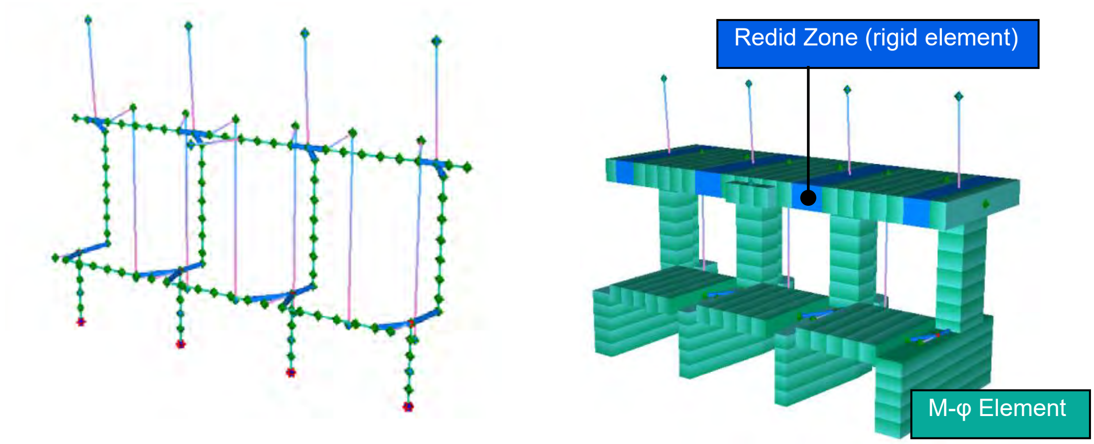
1) Analytical Model Diagram

About the analytical model, the solid model shown in **Figure7.5.69** is prepared, and the same model is used for evaluation by adjusting the direction of inertia force in both the flow direction and the direction perpendicular to the flow direction.



Source: Study Team

Figure7.5.68 Extent of Modeling



Source: Study Team

Figure7.5.69 Frame Model (Left) and Solid Model (Right)

2) Calculation of Working Load

The working load is shown in **Table7.5.77** The detailed calculation process is described in **Vol.5A Structural Calculation for Contract Package-1**.

Table7.5.77 Working Load List

Item	Loads	Remarks
Hydrostatic Pressure	2.94 ~ 20.58 kN/m	The water level for calculating the residual displacement of the wing wall is set to 12.4.
External Water Pressure	21.88 ~ 29.72 kN/m	The groundwater level is the maximum measured groundwater level of 12.53 at the site.
Buoyancy	148.62 kN/m	The bottom surface of the slab is 9.50 and the ground water level of 12.53. Hence, the water level difference is 3.03 m.
Inner Water Weight	88.2 kN/m	Water Depth: 2.10 meters
Cover Soil	57.00 kN/m	Average Depth of Soil 1 m
Earth Pressure in Seismic Condition	15.16 ~ 38.30 kN/m2	
Local Control House Load	150.0 kN	4 Columns Loaded
Gate Load	13.0 kN	Shared with Six Rack Bars
Maintenance Bridge	5.0 kN	

Source: Study Team

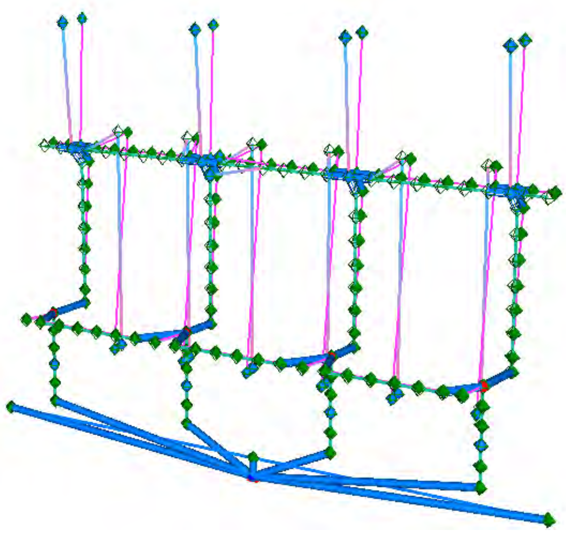
3) Computation of Natural Period

The natural period is calculated as shown in **Table7.5.78**, **Figure7.5.70** and **Figure7.5.71**. Since the primary mode is dominant, the calculation results are summarized below by applying the pushover analysis.

Table7.5.78 Results of Modal Analysis

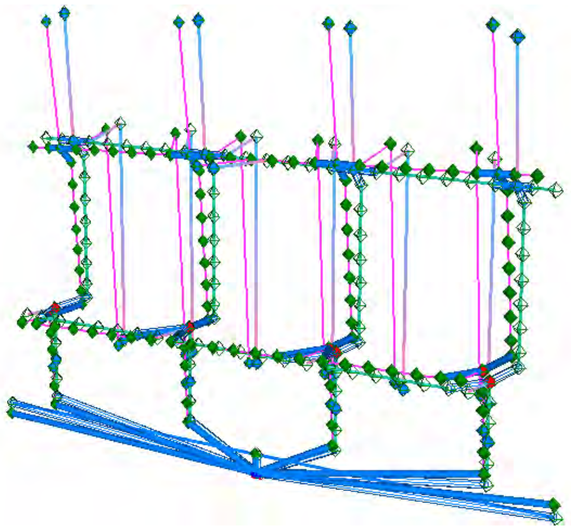
Mode Order	Frequency f (Hz)	Natural Period T(sec)	Stimulus Coefficient			Effective Mass (%)			Effective Mass Ratio (%)			Mode Reduction Factor h
			X (Right Angel)	Y (Vertical)	Z (Flow)	X^2	Y^2	Z^2	X	Y	Z	
1	2.403	0.416	-14.70	0.00	0.00	216.04	0.00	0.00	50%	0%	0%	0.08874
2	3.218	0.311	0.00	-0.38	11.53	0.00	0.14	132.89	0%	0%	47%	0.05582
3	7.178	0.139	0.59	0.00	0.00	0.34	0.00	0.00	0%	0%	0%	0.05025
4	7.690	0.130	-11.09	0.00	0.00	122.98	0.00	0.00	29%	0%	0%	0.08298
5	9.882	0.101	-9.23	-0.01	0.00	85.23	0.00	0.00	20%	0%	0%	0.07390
6	9.923	0.101	-0.06	1.12	-0.58	0.00	1.25	0.34	0%	0%	0%	0.05014
7	11.805	0.085	-2.39	-0.01	0.00	5.73	0.00	0.00	1%	0%	0%	0.05228
8	12.264	0.082	0.00	18.02	2.42	0.00	324.73	5.86	0%	94%	2%	0.09669
9	15.464	0.065	0.00	-3.82	8.25	0.00	14.62	68.20	0%	4%	24%	0.06848

Source: Study Team



Source: Study Team

Figure7.5.70 Vibration Mode Diagram in Flow Direction



Source: Study Team

Figure7.5.71 Vibration Mode Diagram in the Direction Perpendicular to the Water Flow

4) Result of Verification

The verification result of column is summarized in the flow direction and the direction perpendicular to the water flow for 2 members of middle column and end column. The results are shown in **Table7.5.79** and **Table7.5.80**. Both the horizontal bearing capacity and residual displacement during the earthquake are below the allowable values, and the failure mode is judged to be bending failure type.

In other words, since the deformation of the structure was within the elastic region, seismic reinforcement became unnecessary and fatal damage could be avoided even if L2 earthquake ground motion occurs under the condition of bar arrangement set by the calculation result in L1 earthquake ground motion.

Table7.5.79 Verification Results of Middle Column

Reference items/direction		Flow Direction		Direction Perpendicular to the Water Flow	
		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Main Plasticizing Member		Column	Column	Column	Column
Response Shear Force	S (kN)	419.6	641.2	827.3	837.7
Shear Capacity	Ps (kN)	1327.9	1327.9	848.5	848.5
Shear Capacity	Ps0 (kN)	1388.5	1388.5	909.8	909.8
Decision Formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture Morphology		Bending Failure Type	Bending Failure Type	Bending Failure Type	Bending Failure Type
Checking by The Seismic Horizontal Bearing Capacity Method					
Safety Factor	α	1.5	1.5	1.5	1.5
Allowable Plasticity	μa	1.868	1.230	1.701	1.678
Structure Characteristic Correction Factor	Ccs	0.605	0.827	0.645	0.651
Regional Correction Factor	cz	1.00	1.00	1.00	1.00
Standard Value of Design Horizontal Seismic Coefficient	khc0	1.071	1.071	1.000	1.000
Design Horizontal Seismic Coefficient	khc	0.60	0.83	0.65	0.65
Design Horizontal Seismic Coefficient at The Horizontal Bearing Capacity During An Earthquake	kha	1.11	1.59	2.12	2.02
Result of Checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking Of Residual Displacement					
Residual Displacement	δR (mm)	0.0	0.0	0.0	0.0
Maximum Response Plasticity	μrT	0.907	0.698	0.611	0.623
Allowable Residual Displacement	δRa (mm)	32.0	32.0	21.0	21.0
Result of Checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team

Table7.5.80 Verification Results of End Posts

Reference Items/Direction		Flow Direction		Direction Perpendicular to The Water Flow	
		Positive Direction	Negative Direction	Positive Direction	Negative Direction
Main Plasticizing Member		Column	Column	Column	Column
Response Shear Force	S (kN)	478.2	757.6	817.2	811.7
Shear Capacity	Ps (kN)	1345.2	1345.2	856.9	856.9
Shear Capacity	Ps0 (kN)	1410.1	1410.1	920.3	920.3
Decision Formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture Morphology		Bending Failure Type	Bending Failure Type	Bending Failure Type	Bending Failure Type
Checking by the Seismic Horizontal Bearing Capacity Method					
Safety Factor	α	1.5	1.5	1.5	1.5
Allowable Plasticity	μa	1.479	1.177	1.534	1.523
Structure Characteristic Correction Factor	Ccs	0.715	0.859	0.695	0.699
Regional Correction Factor	cz	1.00	1.00	1.00	1.00
Standard Value of Design Horizontal Seismic Coefficient	khc0	1.000	1.000	1.000	1.000
Design Horizontal Seismic Coefficient	khc	0.71	0.86	0.70	0.70
Design Horizontal Seismic Coefficient at The Horizontal Bearing Capacity During An Earthquake	kha	0.96	1.53	2.28	2.22
Result of Checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of Residual Displacement					
Residual Displacement	δR (mm)	0.3	0.0	0.0	0.0
Maximum Response Plasticity	μrT	1.045	0.714	0.596	0.601
Allowable Residual Displacement	δRa (mm)	32.0	32.0	32.0	32.0
Result of Checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team

7.5.3 Gate Facility Design

7.5.3.1 Organizing Design Conditions

The design conditions of the gate are as follows.

Type	:	Duplex Stainless Steel Roller Gate
Clear Span x Effective Height	:	W 2.500 m × H 1.800 m
Number of Gates	:	3 Gates
Water Sealing System	:	Rear Side Four-Way Rubber Water Sealing
Design Head	:	(Front) 4.220 m (EL + 14.520)
		(Rear) 0.000 m (EL + 10.300)
Operating Head (Opening)	:	(Front) 2.800 m (EL + 13.100)
		(Rear) 3.800 m (EL + 14.100)
Operating Head (Closing)	:	(Front) 5.320 m (EL + 15.620)
		(Rear) 3.800 m (EL + 14.100)
Invert Elevation	:	EL + 10.300
Type of Hoist	:	Electric rack type (2-sling)
Hoisting Height	:	Normal 1.900 m
	:	At the time 3.000 m
Main Materials	:	Gate Leaf : SUS 821 L1 Guide fram : SUS 304 N2 system etc. (exposed part) SS 400 etc. (buried part)
Water Quality	:	Outer Side : fresh water Inner Side : fresh water
Extra Thickness	:	Outer Side : one side 0.0 mm (fresh water)
	:	Inner Side : one side 0.0 mm (fresh water)

Standard : Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan, ... 4th printing January 2014
Technical Specification for Dams and Weirs in Japan (draft)
(Explanation of Standard and Manual for Facility Planning) ... October 2016, Edition 1 published

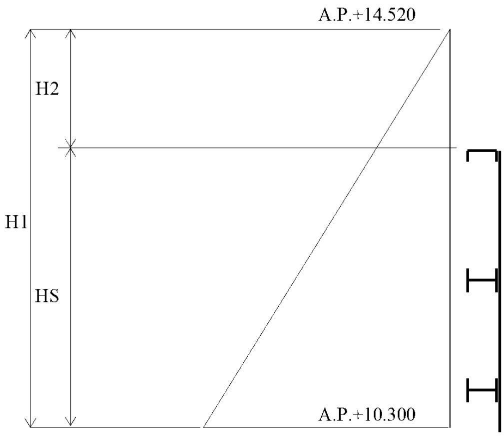
The values described in the design head etc. are as follows.

Table7.5.81 List of Design Water Levels

Item	Water level	Remarks
Floodway Side Design Dike Crown	EL + 15.620 m	
DFL	EL + 14.520 m	
Gate Height	EL + 10.30 m	
Height of Riverbank in Tributary	EL + 14.100 m	
Height of Riverbank in Tributary -1.0 m.	EL + 13.100 m	

Source: Study Team

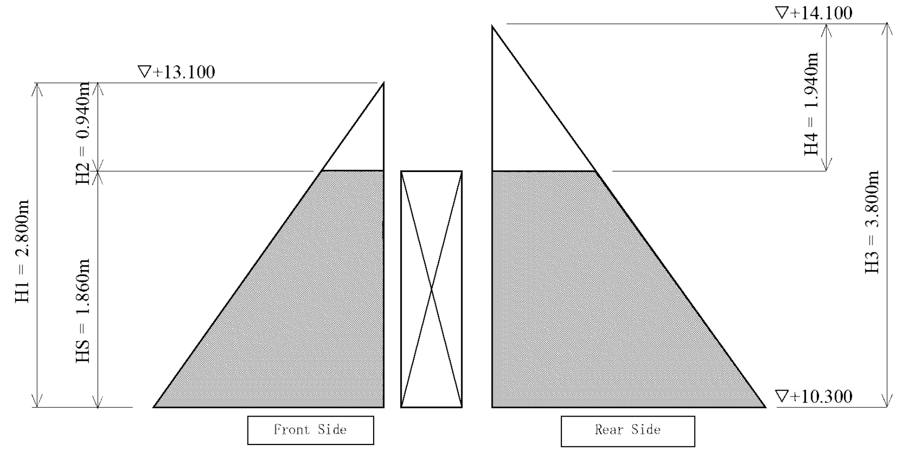
The load diagrams of the above-mentioned loads ((1) Design load, (2) Operating load: Open, (3) Operating load: Closed: 3 cases) are as follows.



P_S	:	Hydrostatic Load	···		kN
P_o	:	Front Side Hydraulic Load	···		kN
P_i	:	Rear Side Hydrostatic Load	···		kN
H_S	:	Water Sealing Height	···	1.860	m
H_1	:	Front Side Design Head	···	4.220	m
H_2	:	Front Side Water Depth at the Top of Gate Leaf	···	2.360	m
H_3	:	Design Water Depth on the Rear Side	···	0.000	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	···	0.000	m
B_S	:	Water Sealing Width	···	2.620	m
γ_1	:	Unit Weight of Water in the Front Side	···	9.810	kN/m3
γ_2	:	Unit Weight of Water in the Rear Side	···	9.810	kN/m3

Source: Study Team

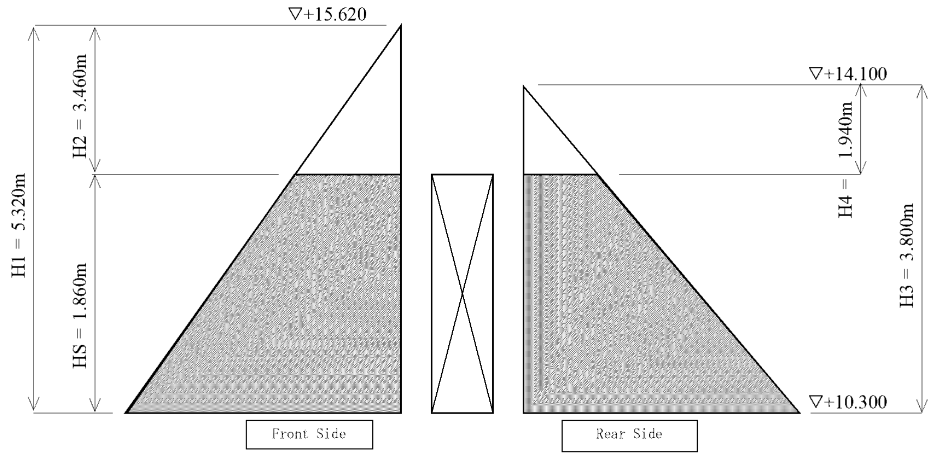
Figure7.5.72 Load Model Diagram ((1) Design Load)



$P_{S\ (O)}$	:	Hydrostatic Pressure Load (Opening)	...		kN
P_o	:	Front Side Hydraulic Load	...		kN
P_i	:	Rear Side Hydrostatic Load	...		kN
H_S	:	Water Sealing Height	...	1.860	m
H_1	:	Front Side Operating Head	...	2.800	m
H_2	:	Front Side Water Depth at the Top of Gate Leaf	...	0.940	m
H_3	:	Operation Head in Rear Side	...	3.800	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	...	1.940	m
B_S	:	Water Sealing Width	...	2.620	m
γ_1	:	Unit Weight of Water in the Front Side	...	9.810	kN/m^3
γ_2	:	Unit Weight of Water in the Rear Side	...	9.810	kN/m^3

Source: Study Team

Figure7.5.73 Load Model Diagram ((2) Operating load: Open)



P_o	:	Front Side Hydraulic Load	...		kN
P_i	:	Rear Side Hydrostatic Load	...		kN
H_S	:	Water Sealing Height	...	1.860	m
H_1	:	Front Side Operating Head	...	5.320	m
H_2	:	Front Side Water Depth at the Top of Gate Leaf	...	3.460	m
H_3	:	Operation Head in Rear Side	...	3.800	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	...	1.940	m
B_S	:	Water Sealing Width	...	2.620	m
γ_1	:	Unit Weight of Water in the Front Side	...	9.810	kN/m^3
γ_2	:	Unit Weight of Water in the Rear Side	...	9.810	kN/m^3

Source: Study Team

Figure7.5.74 Load Model Diagram ((3) Operating load: when closed)

7.5.3.2 Design Calculation

In this section, only the calculation results of each facility are shown, and detailed design calculations are indicated in **Vol.5A Structural Calculation for Contract Package-1**. The result of the calculation for gate leaf, guide frame are indicated in **Table7.5.82** and **Table7.5.83** respectively. Furthermore, the division of the parcel of gate leaf and the load, etc. are shown in **Figure7.5.75**.

Table7.5.82 Gate Calculation Results

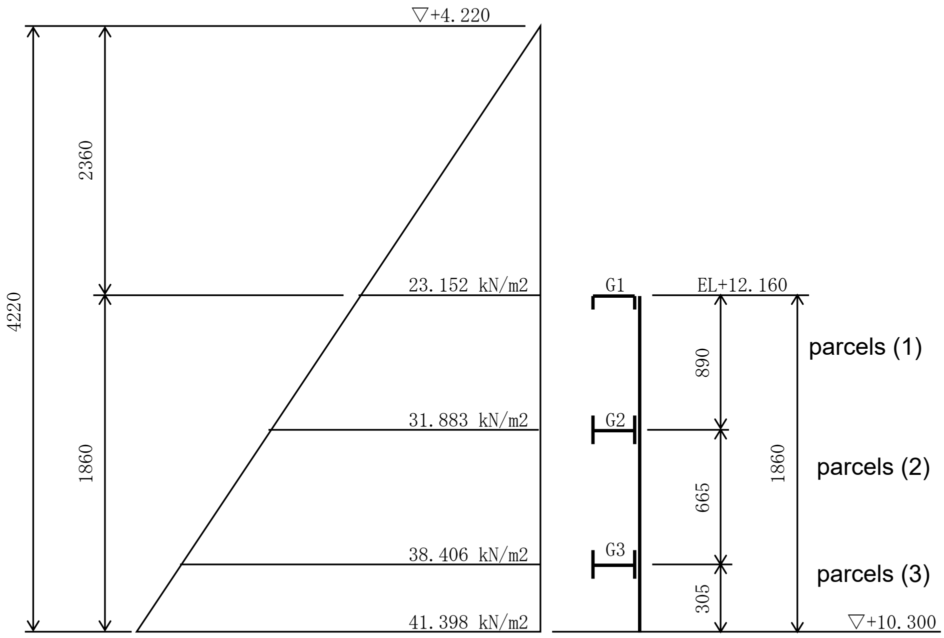
			Calculation Results	Tolerance	Evaluation
Upper Girder	Maximum Bending Stress		57.0 kN/mm2	194.7 kN/mm2	OK
	Maximum Shear Stress		10.3 kN/mm2	115.0 kN/mm2	OK
	Deflection		1/1270	1/600	OK
Main Girder	Maximum Bending Stress		119.1 kN/mm2	194.7 kN/mm2	OK
	Maximum Shear Stress		21.6 kN/mm2	115.0 kN/mm2	OK
	Deflection		1/608	1/600	OK
Skin Plate (Dam & Weir)	Bending Stress	Parcel: (1)	36.8 kN/mm2	200.0 kN/mm2	OK
		Parcels: (2)	42.0 kN/mm2	200.0 kN/mm2	OK
		Parcels: (3)	21.2 kN/mm2	200.0 kN/mm2	OK
Stringer	Bending Stress	Parcel: (1)	22.4 kN/mm2	125.2 kN/mm2	OK
		Parcels: (2)	14.7 kN/mm2	148.2 kN/mm2	OK
		Parcels: (3)	1.8 kN/mm2	185.0 kN/mm2	OK
	Shear Stress	Parcel: (1)	3.6 kN/mm2	115.0 kN/mm2	OK
		Parcels: (2)	3.0 kN/mm2	115.0 kN/mm2	OK
		Parcels: (3)	0.8 kN/mm2	115.0 kN/mm2	OK
End Stringer (Portion where roller shaft does not penetrate)	Maximum Bending Stress	ompression Sic	47.4 kN/mm2	175.1 kN/mm2	OK
		Tension Side	47.4 kN/mm2	200.0 kN/mm2	OK
	Maximum Shear Stress		21.6 kN/mm2	115.0 kN/mm2	OK
	Composite Stress Intensity		60.4 kN/mm2	220.0 kN/mm2	OK
End Stringer (Roller shaft mounting part)	Maximum Bending Stress	ompression Sic	39.1 kN/mm2	172.0 kN/mm2	OK
		Tension Side	39.1 kN/mm2	200.0 kN/mm2	OK
	Maximum Shear Stress		24.3 kN/mm2	115.0 kN/mm2	OK
	Composite Stress Intensity		57.5 kN/mm2	220.0 kN/mm2	OK
Main Roller	Contact Stress Intensity		447.8 kN/mm2	679.2 kN/mm2	OK
Main Roller Shaft	Maximum Bending Stress		131.0 kN/mm2	170.0 kN/mm2	OK
	Maximum Shear Stress		20.2 kN/mm2	100.0 kN/mm2	OK
Bearing	Surface Pressure		13.2 kN/mm2	23.0 kN/mm2	OK

Source: Study Team

Table7.5.83 Calculation Result Of Guide Frame

			Calculation Results	Tolerance	Evaluation
Roller Rail	Concrete Bearing Stress		0.46 kN/mm2	5.90 kN/mm2	OK
	Concrete Shear Stress		0.12 kN/mm2	0.40 kN/mm2	OK
	Bending Stress in Roller Rail		29.2 kN/mm2	120.0 kN/mm2	OK
	Roller Thread Thickness		3.836 mm	6.000 mm	OK
	Local Stress at the Bottom Flange of Roller Rail		103.8 kN/mm2	180.0 kN/mm2	OK
	Bending Stress at Bottom Flange of Roller Rail		66.5 kN/mm2	120.0 kN/mm2	OK
	Composite Stress Intensity of Bottom Flange		85.0 kN/mm2	132.0 kN/mm2	OK
	K by Adjacent Rollers	K1	0.46 kN/mm2	5.9 kN/mm2	OK
		K2	0.45 kN/mm2	5.9 kN/mm2	OK
	Bearing Stress due to Adjacent Rollers		0.9 kN/mm2	120.0 kN/mm2	OK
	Compressive Stress by Adjacent Rollers		0.37 kN/mm2	5.9 kN/mm2	OK

Source: Study Team



Source: Study Team

Figure7.5.75 Partition of Gate Leaf, Load, etc.

7.5.3.3 Control Room Equipment Layout

(1) Component Device

Hoisting device-related equipment, generators, and information equipment are arranged in the operation room. Taytay Sluiceway has pin rack type hoisting devices. The equipment arranged in the control room is shown in **Table7.5.84**.

Table7.5.84 Control Room Components

	Hoisting device	Information Equipment Related
Itemized Component	・Wire Drum ・Wire Sheave ・Wire Terminal Device ・Motor ・Electric Manual Hoisting Device ・Centrifugal Brake ・Mew Lifter Brake ・Gear Reducer ・Drum Gear And Pinion Gear ・Dogging Device ・Limiting Switch ・Opening Meter ・Emergency Upper Limit Detection Device ・Machine Side Control Panel	・Power Switching Panel ・Power Receiving Panel ・TC Sub Unit ・Control Panel ・Isolation Transformer ・UPS ・Emergency Generator

Source: Study Team

(2) Equipment Layout

The layout of the control room shall be such that necessary space is secured in consideration of inspection and maintenance work based on the "Technical Specification for Dams and Weirs in Japan (Draft)".

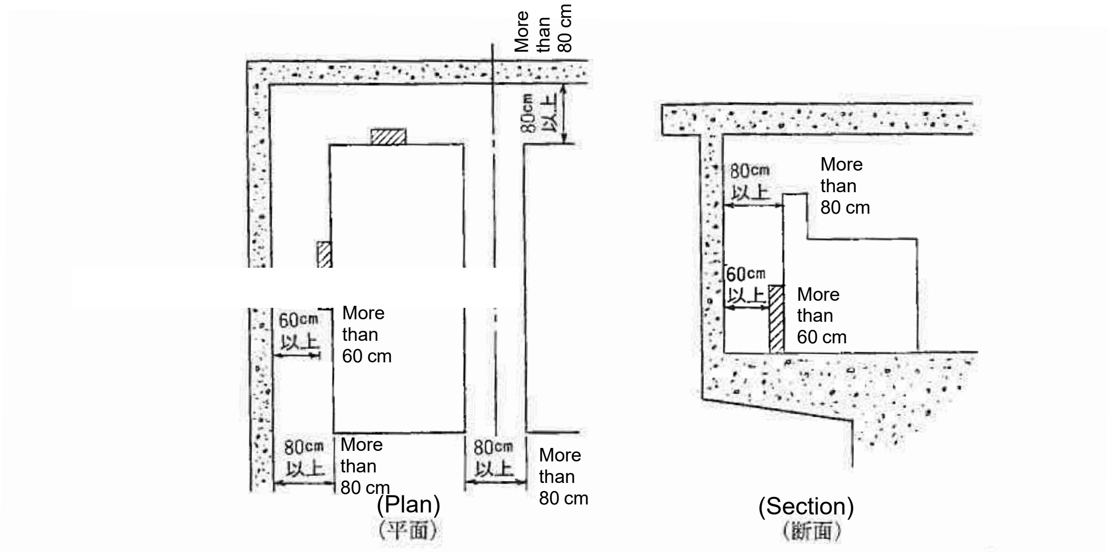
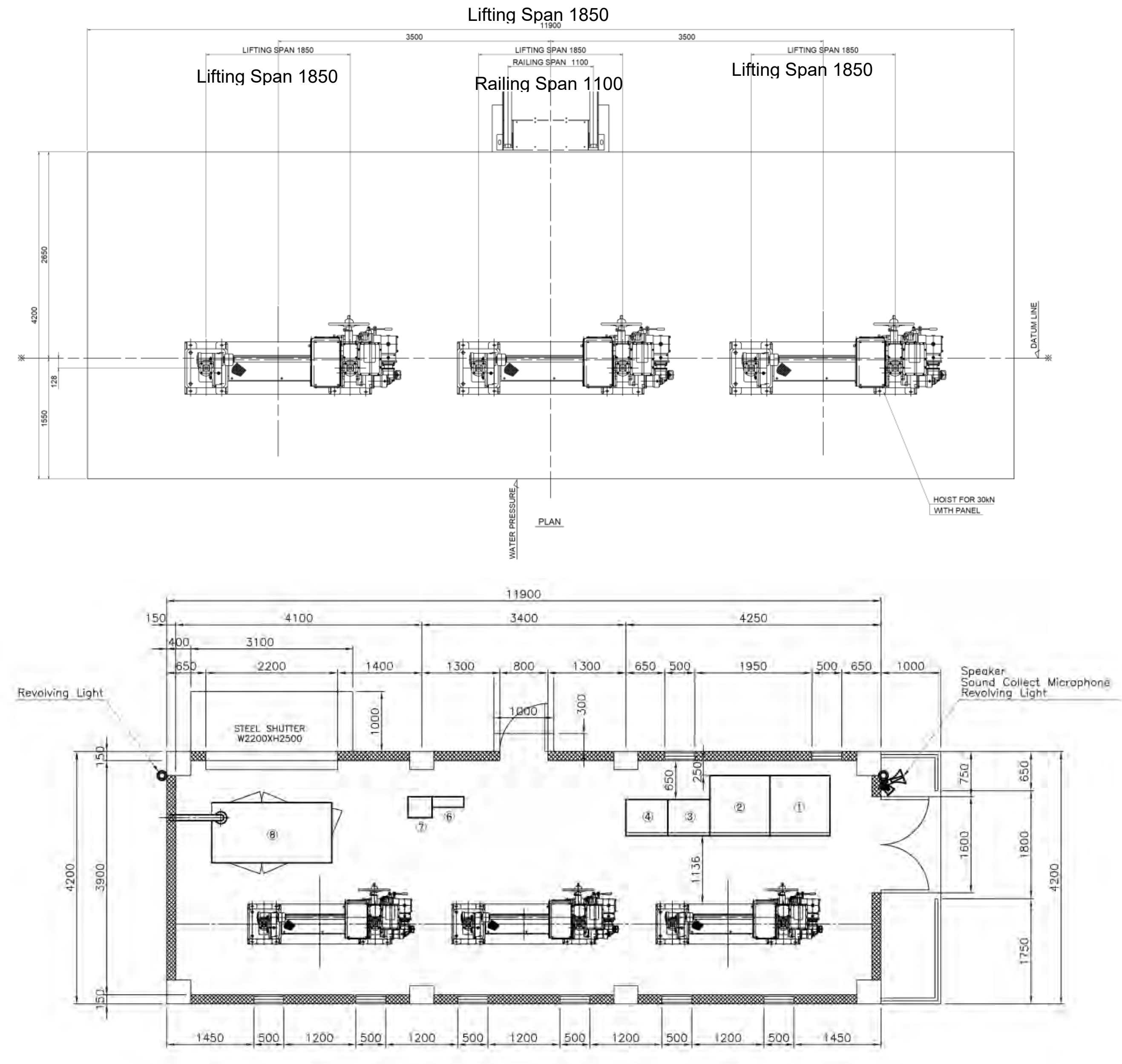


図4.4.2-1 開閉装置の端部から壁までの間隔

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

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

Figure7.5.76 Space to be Secured In Operating Room Space



LEGEND
1 Power Switching Panel
2 Power Receiving Panel
3 TC Sub Unit
4 Control Panel
5
6 Isolation Transformer
7 UPS
8 Emergency Generator

Source: Study Team

Figure7.5.77 Control Room Layout

7.5.3.4 Specifications of the Gate Facility

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	(Riverbank Elevation of Tributary River: -1 m)
	(Tributary Side)	EL + 14.100	(Riverbank 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		
Standard	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)		

(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	
Standard	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)	

(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
Standard	Technical Specification for Dams and Weirs in Japan (Draft) Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,

7.5.4 Building Facility Design

In **sub-section 7.6 Detailed Design,** building facilities, building machinery and equipment, and building electrical equipment of 3 facilities is stated.

7.5.5 Information Equipment Design

7.5.5.1 Design of Instrumentation, Alarm Monitoring, and Remote Monitoring and Control Equipment

(1) Design Conditions

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

Table7.5.85 Design Condition List

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

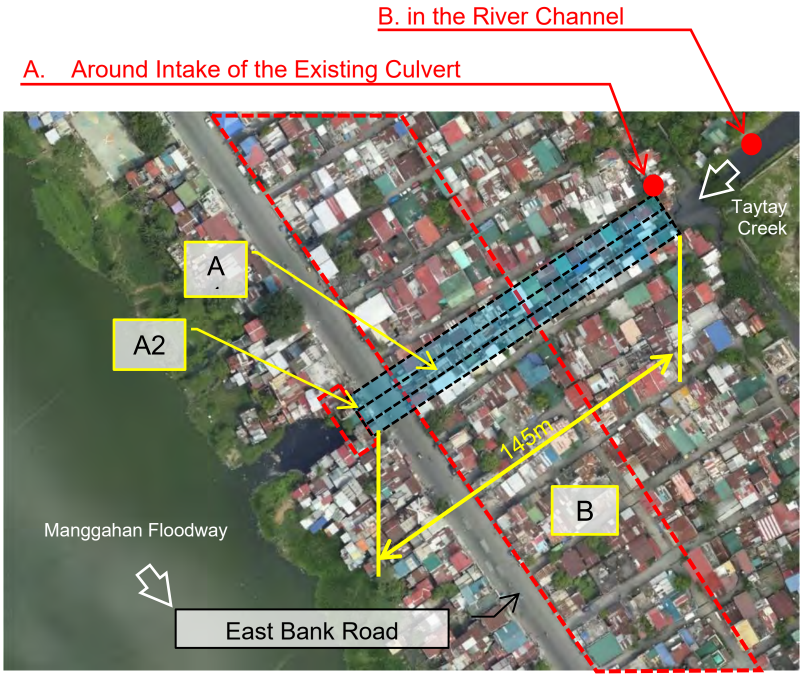
Source: Study Team

(2) Instrumentation (Water Level Observation Equipment) Design

1) Review of Water Level Gauge Layout

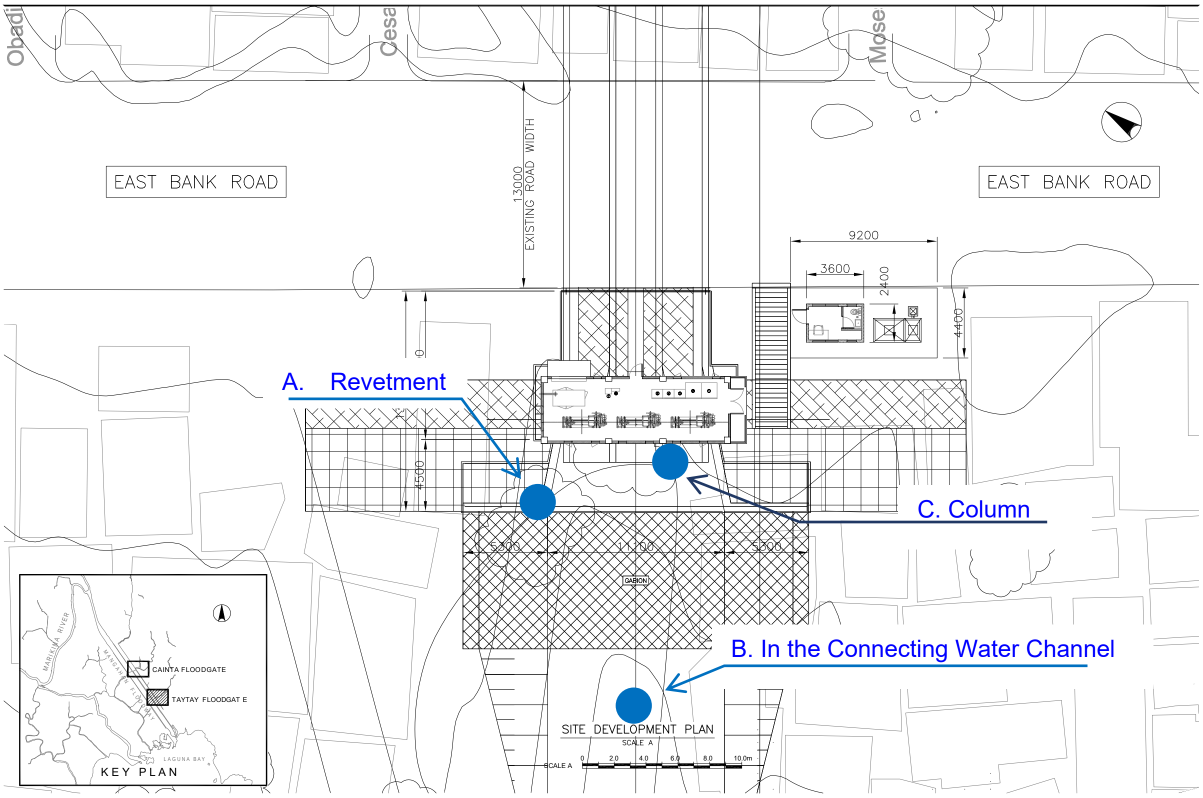
The instrumentation facility (water level observation equipment) installed in Taytay Sluiceway is aiming at caring out the proper gate operation during floods. In order to measure the water level according to the gate operation situation, the installation in the upstream (land side) and downstream (river side) of the gate is considered. Possible locations for installation of water level gauges in Taytay Sluiceways are shown in **Figure7.5.77**

There is an existing box culvert with 145 m length in the upstream of the sluiceway and the sluiceway will be installed connecting with this existing culvert. Hence, an alternative location for the water level gauge in the upstream side of the gate(land side) would be around the inlet of this existing box culvert.



Symbol [1)]	Item	Description
A1	Existing Box Culvert	A box culvert of about 145 m in length has been installed. Considering the N-value according to the past boring data, it can be assumed that it has pile foundations. (The As-built drawing is not available.).
A2	Existing Outlet of Box Culvert	End of the Box Culvert Wing walls are installed on the right and left sides of the existing box culvert, and earth retaining walls are installed on the top slab.
B	Houses in the landside	Houses in the land side from the East Bank Road are not the original targets of relocation.

Figure 7.5.78 Alternate Locations for Water Level Gauge Installation in the Upstream Side



Source: Study Team

Figure7.5.79 Alternate Positions for Water Level Gauge Installation

For each alternative position shown in **Figure7.5.77**, the following are compared and examined. As a result of the comparative examination, the installation to the revetment in the river side is chosen.

[Items to Be Compared]

1. Outline of installation location
2. Applicable method of measuring water level
3. Application of observed water level to facility operation
4. Workability
5. Maintainability

Table7.5.86 Comparison of Alternative Locations for Installation of Water Level Gauges (Upstream side of Taytay Sluiceway: Land Side)

Installation Position	A. Around Intake of the Existing Culvert	B. In the River Channel
Outline of The Installation Location	A water level gauge is installed at the revetment upstream of the around the intake of the existing box culvert	A water level gauge is installed near the center of the channel (center of flow)
Applicable Water Level Measurement Method	Float Type Reed Switch Type Hydraulic (Quartz Hydraulic System) Ultrasonic And Radio Wave Type	Reed Switch Type Hydraulic (Quartz Hydraulic System)
Application of Observed Water Level To Facility Operation	△ When the gate opening is different, it is not possible to measure the accurate water level, and it is inferior to the facility operation.	◎ Since it is observed near the river channel center (center of flow), it is not affected by the gate operation.
Workability	◎ Since it is installed on the revetment, the workability is good. However, depending on the water level measurement method, installation of observation wells (float type) and measurement pillars (reed switch type) are required.	△ Since it is installed in the river channel, it becomes the shipboard construction by the base ship, and it is inferior in the workability. It is necessary to install a plurality of H-steels for a water level gauge and a dust guard.
Maintainability	◎ Maintenance from the land is possible, and the maintainability is good.	△ When driftwood and refuse adhere in the flood, it is necessary to deal with them sequentially.
Evaluation	It is excellent in workability and maintainability, and the main plan is adopted.	

Source: Study Team

Table7.5.87 Comparison of Alternative Locations for Installation of Water Level Gauges (Downstream side of Taytay Sluiceway: External water)

Installation Position	A. Revetment	B. In the Connecting Water Channel	C. Column
Outline of Installation Location	A water level gauge is installed on the revetment in the Manggahan Floodway around Taytay Sluiceway to measure the water level.	A water level gauge is installed near the center of the connecting water channel (center of flow) to measure the water level.	A water level gauge is installed at the middle column of Taytay Sluiceway to measure the water level in the downstream of the gate.
Applicable Method of Measuring Water Level	Float Type Reed Switch Type Hydraulic (Quartz Hydraulic System) Ultrasonic And Radio Wave Type	Reed Switch Type Hydraulic (Quartz Hydraulic System)	Ultrasonic And Radio Wave Type
Application of Observed Water Level To Facility Operation	△ When the gate opening is different, it is not suitable for the facility operation, because accurate water level cannot be measured.	◎ Since it is observed near the center of the channel (center of flow), it is hardly affected by gate operation.	△ When the opening of the gate is different, it is not suitable for the facility operation, because accurate water level cannot be measured.

Installation Position	A. Revetment	B. In the Connecting Water Channel	C. Column
Workability	◎ Its workability is good because it is installed on the revetment. However, depending on the water-level measurement method, it is necessary to install an observation well (float type) or a measuring column (reed switch type).	△ Since it is installed in a water channel, it is constructed by using a barge, and the workability is inferior. It is necessary to install multiple H-beams with water level gauge for protection.	○ It can be installed with the gate construction. The construction is comparatively easy, because it is installed on the external wall part of the local control house.
Maintainability	◎ Maintenance from the land is possible, and the maintainability is good.	△ It is necessary to deal with driftwood and garbage after flood events.	○ Maintenance from the local control house is possible, and the maintainability is good.
Evaluation	Good workability and maintainability, and this position is selected.		

Source: Study Team

2) Selection of Water Level Observation Method

As for the water level observation system, a light feed type quartz water pressure system is adopted as well as MCGS.

3) Instrumentation Configuration

The instrumentation structure is the same as MCGS.

4) Equipment Specification

The equipment specification is the same as MCGS.

(3) Alarm Facility Design

1) Siren

In Taytay Sluiceway, the siren is not installed, and it is substituted by the pseudo sound of the speaker.

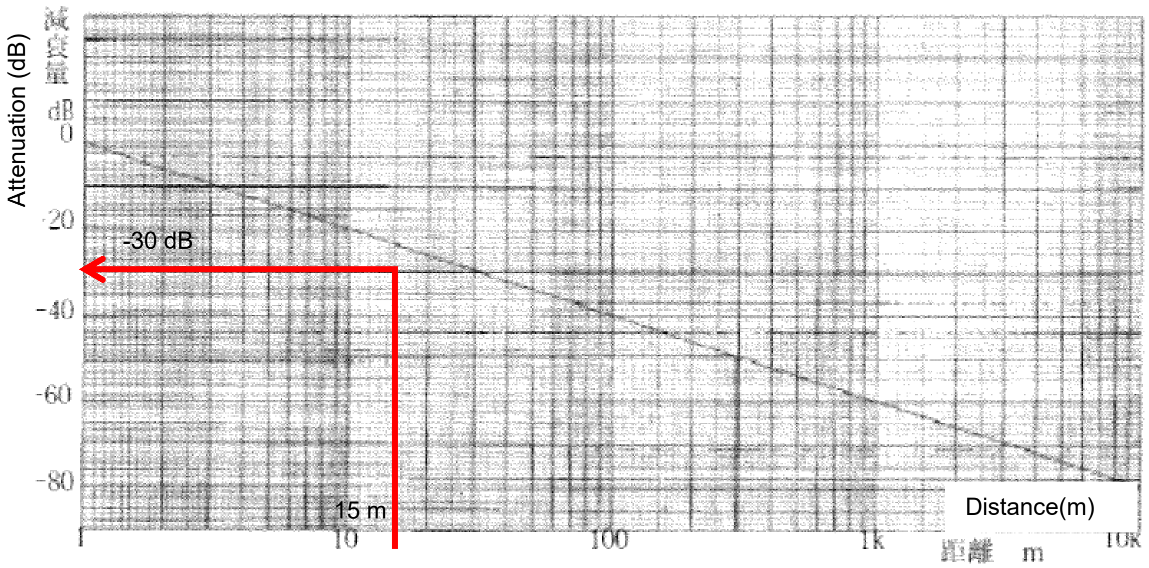
2) Loudspeaker Microphone

(a) Speakers

The speakers to be installed at Taytay Sluiceway are aiming at making the alarm to advise the surroundings when the gate is operated, the output of the speakers is determined by setting the arrival distance to be about 15 m so that the alarm sound reaches around Taytay Sluiceway.

Based on **Figure7.5.80**, the attenuation corresponding to the arrival distance of 15 m is read out as -35 dB. Hence, the output sound pressure level which requires 108 dB (= 78 dB + 30 dB) is obtained by adding the attenuation of 30 dB to the target arrival level of 78 dB.

When the speaker output is selected from the output sound pressure level, the output sound pressure level is equivalent to 1 W, but since the output sound pressure level is close to the threshold of 1 W and 25 W in the output sound pressure level standard value table, the safe side is taken and 25 W is set.



Source: Design Guideline for Telecommunications Facilities (Communications)

Figure7.5.80 Attenuation Due to Sound Distance

Table7.5.88 Speaker Output Sound Pressure Level (1 M Value)

Speaker Input	Output Sound Pressure Level
1W	110 dB
25W	124 dB
50W	127 dB
100 W	130 dB

Source: Study Team

(b) Audio Amplifier

The audio amplifier is used for broadcasting by voice or broadcasting of the proceedings and is mounted on the alarm device. The rated output of the audio amplifier is 100 W as standard, and if the output is to be increased, an audio amplifier is added in units of 100 W.

(c) Sound Collection Microphone

One microphone is installed for each speaker.

(d) Warning Light

The warning lamp installed in Taytay Sluiceway is installed to add visual information in addition to the warning by sound of siren and speaker in the gate operation. The light source of the alarm light shall be a LED system of long life and power saving type, and as a blinking system, a reflection mirror rotating system, or a lamp blinking system shall be used.

(e) Operating Equipment

Since there is no separated operation room in Taytay Sluiceway, the alarm equipment operation except for the machine side is carried out from Cainta Floodgate operation room.

3) Study on Alarm System Configuration

The alarm equipment composition is the same as MCGS.

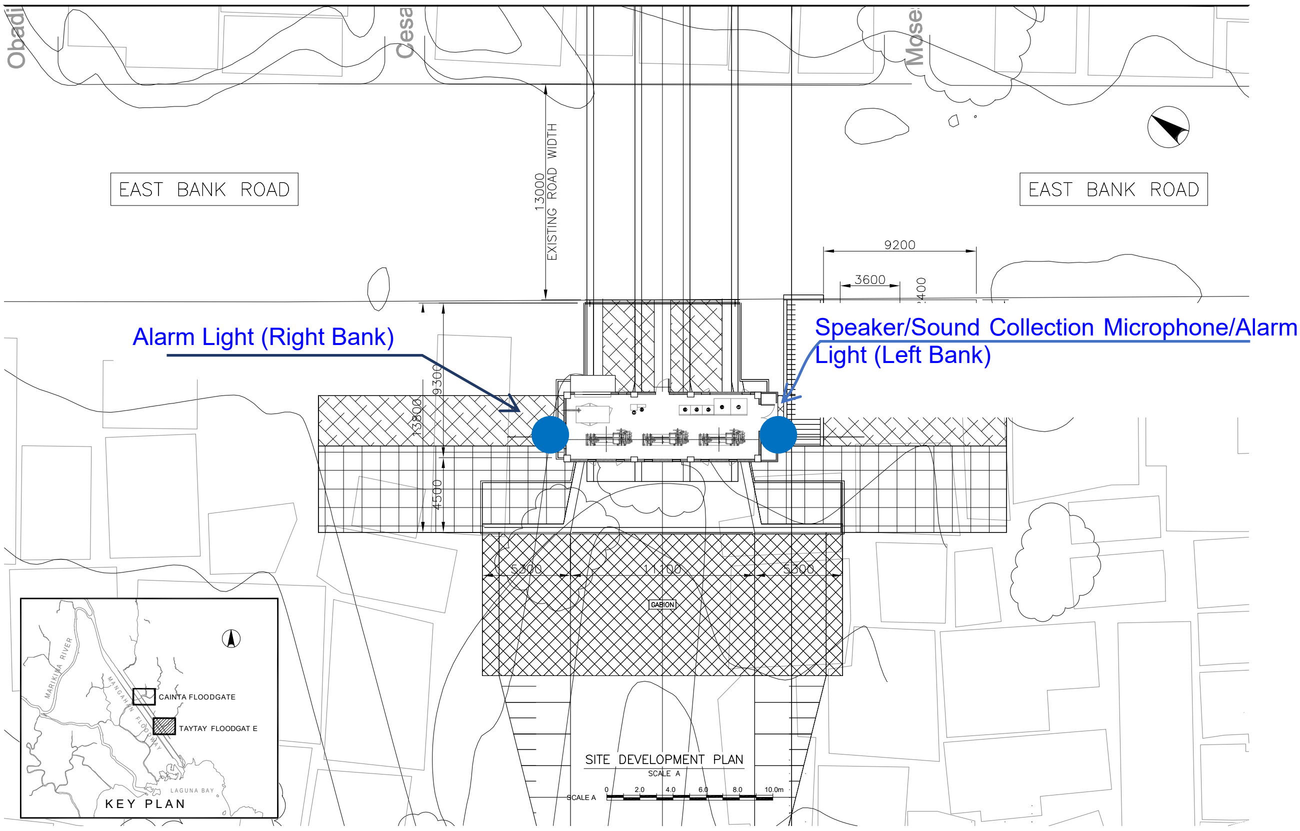
4) Consideration of Alarm Equipment Layout

The alarm equipment installed in Taytay Sluiceway is arranged according to the following concept.

Table7.5.89 Arrangement of Alarm Equipment (Taytay Sluiceway)

Alarm facility	Installation Position	Quantity	Installation criteria
Speakers	Left Bank Side of the Local Control House	1 (Left Bank Side)	Speakers, sound collection microphones and warning lights shall be installed on the left bank side of the local control house so that they can blow and light on both bank sides of Taytay Creek. The warning light is also installed in the right bank side of the local control house, considering the visual recognition from the right bank side of Taytay Creek.
Sound Collection Microphone			
Warning Light	Left and Right Bank Side of the Local Control House	2 (Right and Left Bank Side)	

Source: Study Team



Source: Study Team

Figure7.5.81 Alarm Installation Position

5) Equipment Specification

The equipment specification is the same as MCGS. However, the speaker output is 25 kW.

(4) Design of Monitoring Equipment (CCTV Camera)

1) Object to be Monitored

The camera equipment installed in Taytay Sluiceway is to quick and accurate situation grasp and facility operation in the flood by remote monitoring of the field situation.

In this design, in addition to monitoring of gate conditions (facility monitoring), grasping of river conditions at the time of flood (spatial monitoring) is carried out, and monitoring objects are set as follows.

Table7.5.90 Object to be Monitored

Object	Monitoring Classification	Concept on Monitoring
Gate Facility	Facility Monitoring	The angle of view is fixed in the gate direction, and the opening and closing situation is monitored remotely.
Condition of The River	Spatial Monitoring	For the angle of view of the land side and the river side, the situation is widely grasped. The monitoring direction can be arbitrarily changed by turning and zooming.

Source: Study Team

2) Monitoring System

In this design, the HD camera is applied considering the certainty of manufacturer's warranty and procurement of replacement parts, and the technological trend and market trend of the camera.

3) Selection of the Monitoring Method

As well as MCGS, HD simple type IP camera equipment is adopted.

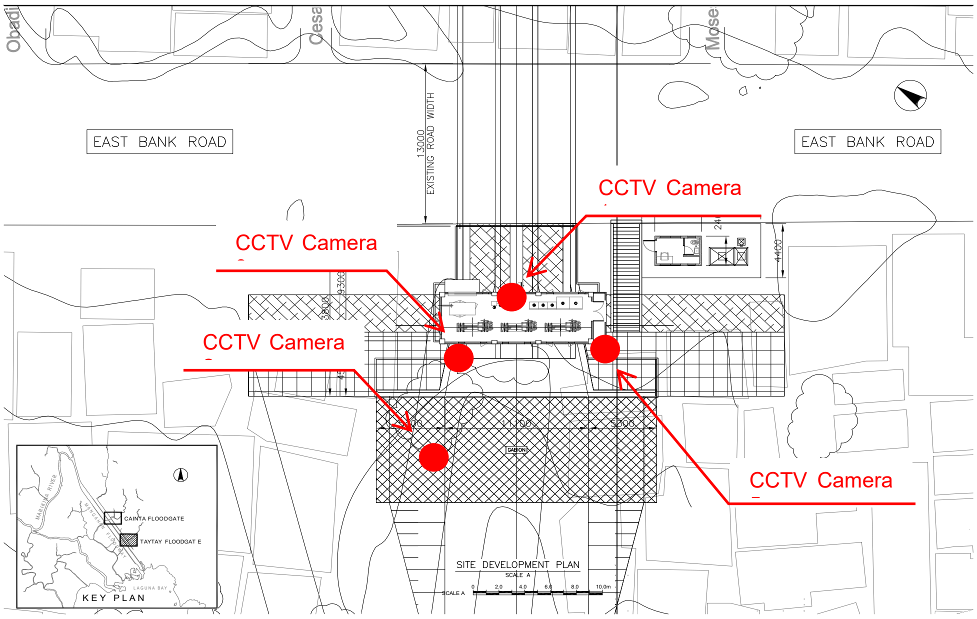
4) Study on Monitoring Equipment Layout

The monitoring equipment installed at Taytay Sluiceway would be arranged according to the following concept. The selected CCTV camera is a turning type, and it is possible to switch the monitoring object (gate ⇔ space) by turning the camera.

Table7.5.91 Arrangement of the Monitoring Facilities (Taytay Sluiceway)

Object	Monitoring Classification	Quantity	Placement Criteria
Gate	Facility monitoring	2	Install in the left and right bank side of local control house
Land Side	Spatial monitoring	1	Install in left bank side of local control room
River Side	Spatial monitoring e	1	Install on the right bank revetment at the confluence of the floodway

Source: Study Team



Source: Study Team

Figure7.5.82 Position of Camera Equipment

5) Configuration of Monitoring Equipment

The configuration of monitoring equipment is the same as MCGS.

6) Equipment Specification

The equipment specification is the same as MCGS.

(5) Remote Monitoring and Control Facility

The remote monitoring and control facilities will be organized in **Sub-Section 7.3.6.6 Examination of Remote Monitoring and Control Equipment**.

7.5.5.2 Electrical Equipment (Emergency Power Supply) Design

(1) Design Conditions

1) Operating time of the Generator

In order to ensure the reliability of gate opening and closing of Taytay Sluiceway, a standby power generator is permanently installed as a standby power source for power facilities. Regarding Taytay Sluiceway, the staff standing by at the nearby Cainta Floodgate generator house will operate the equipment during power outages, so it is not considered to have the remote control equipment stand by for the entire duration of power outages. The power supply for opening and closing the gate shall be secured so that it can be opened and closed once a day for three days assuming a power failure.

As for Taytay Sluiceway, since the required output of the power for opening and closing the gate is not so much different from that of the control equipment, it is assumed that one generator is used for both purposes.

The operation time of the gate opening/closing power can be calculated to be 6 minutes, and this is rounded up to about 10 minutes for 1 time opening (closing) time from the opening/closing height of 1.8 m and the opening/closing speed of 0.3 m/min. 10 minutes × (1 open + 1 closed) × 3 days = 60 minutes in total, that is, a fuel tank capacity for 1 hour operation in total is secured.

2) Creating a Load List

The load capacity of the equipment in Taytay Sluiceway is shown in **Table7.5.92**. The equipment is classified into generators for gate opening and closing and generators for control facilities.

Table7.5.92 Load List

Category	Load name	Three-phase load	Single-phase load	Units	Number of units	Subtotal
Generator for gate opening and closing power and control facility	Hoist motor	0.33		kW	3	0.99
	Machine side control panel control power supply		0.50	kVA	3	1.50
	Distribution Board for Local Control House					8.33
	Speakers		0.02	kVA	1	0.02
	Sound collection microphone		0.01	kVA	1	0.01
	Warning light		0.03	kVA	2	0.06
	CCTV		0.15	kVA	5	0.75
	Control panel		2.77	kVA	1	2.77
	TC extension unit		0.11	kVA	1	0.11
	MC		0.01	kVA	11	0.11
	Generator panel auxiliary power supply		2.50	kVA	1	2.50
	backup		2.00	kVA	1	2.00
	Distribution board for building					2.95
	LIGHTINGS		0.99	kVA	1	0.99
	CONVENIENCE OUTLET, 3UNIT		0.51	kVA	1	0.51
	FACP		0.95	kVA	1	0.95
	SPARE		0.50	kVA	1	0.50

Source: Study Team

(2) Design of Generator and Motor

1) Calculation of Generator Capacity

The output of the power generation equipment shall be calculated by taking into consideration the output of the load, the type and starting method, the presence or absence of a fire-fighting load, the type of engine, etc.

In this work, the output was calculated according to the "Output calculation software for private power generation facilities" based on the Design Guideline for Telecommunications Facilities (Electric).

The calculation method indicated in Design Guideline for Telecommunications Facilities (Electric) and the calculation results are shown in **Vol.5A Structural Calculation for Contract Package-1**. The calculation results are as follows.

Table7.5.93 Generator Calculation Result

Taytay Sluiceway
Generator capacity 19.3 kVA Motor output 23.7 kW

Source: Study Team

2) Selection of Motor

Based on the calculation results of "Output calculation software for private power generation facilities", the most recent generator efficiency is chosen from generator efficiency table shown in Design Guideline for Telecommunications Facilities (Electric). The generator efficiency is indicated in **Table7.5.95**.

Table7.5.94 Power Generating Capacity and Motor output of the Nearest High-Order Generator

Taytay Sluiceway
Generator capacity 37.5 kVA Motor output 37.2 kW

Source: Study Team

Table7.5.95 Generator Efficiency Table

Generator Output		Conventional Efficiency	Motor Output	Generator Output		Conventional Efficiency	Motor Output
(kVA)	(kw)(power factor:0.8)	ηG(%)	(kW)	(kVA)	(kw)(power factor:0.8)	ηG(%)	(kW)
5	4	74	5.5	200	160	87.9	182
10	8	75	10.7	250	200	88.9	225
15	12	76	15.8	300	240	89.5	269
20	16	77	20.8	375	300	90.3	333
37.5	30	80.7	37.2	500	400	91.0	440
50	40	82.3	48.6	625	500	91.7	546
62.5	50	82.4	60.0	750	600	92.1	652
75	60	84.3	71.2	875	700	92.3	759
100	90	85.6	93.6	1000	800	92.6	864
125	100	86.4	116	1250	1000	93.0	1076
150	120	87.0	138	1500	1200	93.3	1287

Note: The values with is not accordance with JEM-1354.*
Source: Study Team Translated from Design Guideline for Telecommunications Facilities (Electric)

(a) Selection of Generator

The basic conditions of the generator are as shown in the table below, based on the Design Guideline for Telecommunications Facilities (Electric).

Table7.5.96 Basic Requirement for Generators

Design Guideline	Applied to This Design
The generator shall be a horizontal synchronous generator.	Use a horizontal synchronous generator
Excitation type shall be brushless or static.	Use brushless or static excitation
For diesel engines, the protection type shall be JIS C 4034 (IP 20) or drip-proof type (IP 22 S). The gas turbine shall be of the protective type (IP 20).	Diesel engines, protected type (IP 20) or protected drip-proof type (IP 22 S)
The heat resistance class of insulation shall be class E or higher for low pressure and class B or higher for high pressure.	Class E or higher for low-pressure generators
The standard number of poles is 4, and a small-capacity machine (100 kVA or less) may have 2 poles.	Use the standard four poles
The generator rated voltage shall be as follows as a standard. 150 kVA or less 200 V/220 V (Production possible up to 200 kVA) 150 ~ 400 kVA 400 V/440 V (Production possible from 50 kVA to 500 kVA) 250 kVA or higher 3300 V/6600 V	The generator rated voltage shall be the standard.

Source: Study Team

(b) Selection of Motor

The basic conditions of the motor are as shown in the table below based on the Design Guideline for Telecommunications Facilities (Electric). As described in the design guideline, a comparison between a diesel engine and a gas turbine is shown in **Table7.5.98**. The gas turbine system is not adopted unless the gas turbine is particularly advantageous.

Table7.5.97 Basic Requirements for Motors

Design Guideline	Applied to This Design
Diesel engines with high fuel consumption rates shall be used in principle. However, if a gas turbine is particularly advantageous under certain conditions, the selection of a gas turbine should be considered.	The standard diesel engine is used.
The starting system shall be electric or pneumatic. 1000 kVA or less: electric system Exceeding 1000 kVA: Electric or air system	The starting system is an electric system because it is 1000 kVA or less.
Starter The electric system is a system in which the cell motor is driven by a DC power supply. The storage battery is a control valve type stationary lead acid battery or a small control valve type lead acid battery.	Use a cell motor drive system. The storage battery is a control valve type stationary lead acid battery or a small control valve type lead acid battery.
The capacity of the storage battery shall be such that it can be started three times or more.	Capacity that can be started three times or more.
The standard cooling system for diesel engines is the radiator type.	Standard radiator type.

Source: Study Team

Table7.5.98 Comparison of Diesel Engines and Gas Turbines

Item \ Motor	Diesel Engine	Gas Turbine
Operating Principle	Thermal energy of intermittent combustion and explosion combustion gas is once converted into reciprocating motion of piston, which is converted into rotary motion by crankshaft. (Reciprocating motion → Rotating motion)	Thermal energy of continuously burning combustion gas is directly converted into rotational motion by a turbine (rotational motion)
Output	The output is limited by the suction air temperature.	When the suction air temperature is high, the output is limited because the amount of air compressed by the compressor is reduced.
Fuel Consumption Rate	230 ~ 310 g/kWh	520 ~ 680 g/kWh
Spent Fuel	Light oil and heavy oil A	Kerosene, light oil, and heavy oil A
Excess Air Ratio	2.0~3.0	3.5~4.0
Instantaneous Frequency Variation	± 10% or less	± 5% or less in the case of uniaxial type ± 10% or less in the case of biaxial type
Frequency Droop	Not more than 5%	Not more than 5%
Instantaneous Load Input Rate	In the case of no supercharge: 100% input is possible. In case of turbocharger: 70% input is possible. For high turbochargers: 50% input possible	In the case of uniaxial type: 100% input is possible. In the case of 2-shaft type: 70% input is possible.
Starting Time	5 to 40 seconds	20 - 40 seconds
Light Load Operation	It is difficult to obtain complete combustion of fuel. The amount of increase in lubricating oil increases and carbon adheres to the combustion chamber or the exhaust turbine (supercharger).	No problem.
Amount of NOx, Etc.	300 ~ 1000 ppm (O2 concentration of 13%)	20 ~ 150 ppm (O2 concentration of 16%)
Vibration	Vibration is generated due to the reciprocating engine, but it can be reduced from the load of the vibration isolator.	Because it is a rotating engine, there is no need for vibration isolators.
Volume And Mass	Large number of parts and heavy mass.	There are few mechanical parts, and the size and mass are small and light.
Installation	The installation area is large. I need a foundation. Intake/exhaust treatment equipment is small.	The installation area is small. The foundation is small and good. Intake and exhaust air treatment equipment becomes larger.

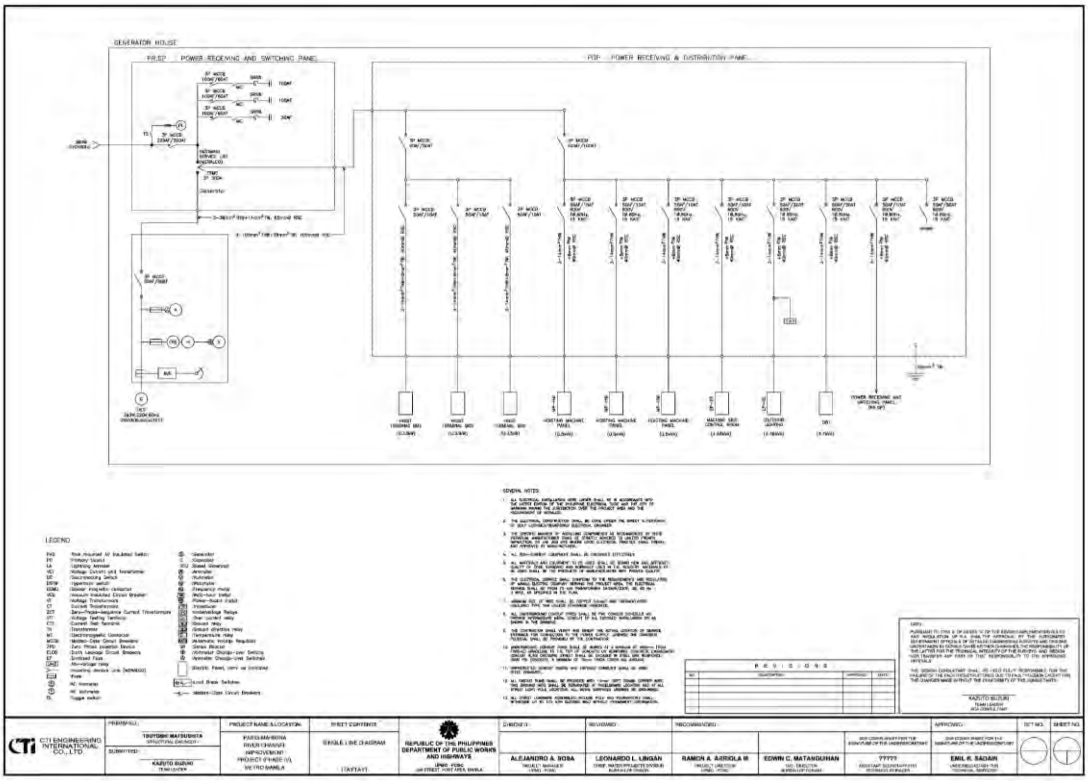
Motor / Item	Diesel Engine	Gas Turbine
Cooling Water	Needed Radiators do not need to be replenished in normal condition.	Not required (For air cooling type)
Serviceability	There are few daily inspection items. (Equivalent to automobiles and construction equipment) Overhaul is basically possible on site. (Some exceptions) Replacement parts are relatively inexpensive	There are few daily inspection items. Overhaul must be carried out at the factory. Spare parts are more expensive than diesel
Response in the Event of a Failure	Depending on the severity of the failure, most can be recovered locally. If there is no major damage to the engine, it can be restored in one to three days.	Failures resulting from replacement of parts can be recovered at the site, but failures of the gas turbine itself require bringing the gas turbine into the factory. It takes about 3 days to 1 week for parts replacement and about 1 ~ 2 weeks for parts brought into the factory.
Extension of Operating Time Due to Load Limitation	Since the fuel consumption is almost proportional to the load, if the load is reduced to half of the rated output, the fuel consumption is reduced to half, and the operation time is doubled.	Since 50 ~ 60% of the fuel at the rated output is consumed even at no load, the fuel consumption is reduced by only about 20% even if the load is reduced to half of the rated output, so the effect of extending the operating time is small.

Source: Study Team

3) Examination of Generator Specifications

(a) Electrical System Planning

From the calculation results of the load connected to the generator and the generator output and motor output, a single-wire connection diagram is prepared. The single wire diagram is shown in **Figure7.5.83**.

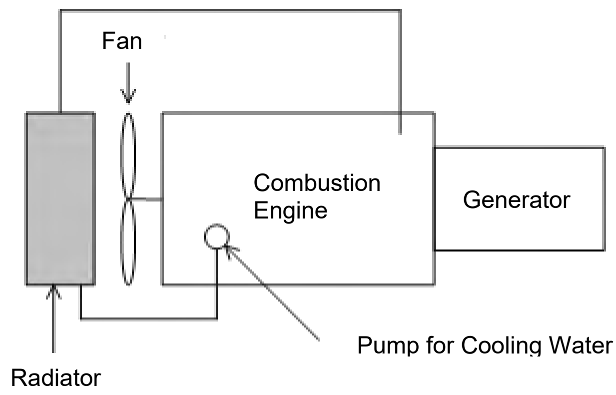


Source: Study Team

Figure7.5.83 Single Wire Diagram

(b) Determination of Cooling Method

From the Design Guideline for Telecommunications Facilities (Electric), the radiator type has been adopted as the standard cooling system for diesel engines. A schematic diagram of the radiator cooling system is shown below.

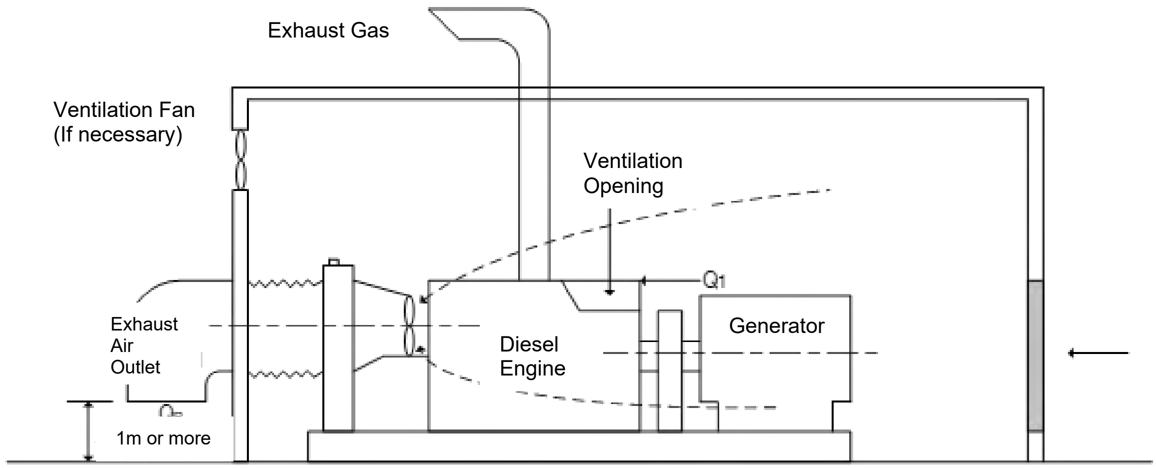


Source: Study Team

Figure7.5.84 Radiator Cooling Type

4) Ventilation Volume Calculation

The schematic diagram of ventilation in the radiator cooling system is shown below.



Source: Study Team

Figure7.5.85 Schematic Diagram of the Radiator Cooling System

The ventilation rate of the radiator type is obtained from the following.

Supply air volume required Q (m3/min) = Q1 + QR

Q1: Amount of air required for combustion of the prime mover

$$Q1 = \frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho} \quad \text{(m3/min)}$$

A′ : the amount of air required to burn 1 kg of fuel (m3/kg)

(Heavy oil A: 14.6, light oil: 14.7)

be: Engine fuel consumption rate (kg/kWh)

In the case of a radiator type, the value added by an increase of 7%

Pe: motor output (kW)

ε : excess air ratio (Non-supercharged engine = 2.0, engine with supercharger = 2.5)

ρ : air density (= 1.165 at 30 °C) (kg/m3).

QR: Radiator fan ventilation (m3/min)

Table7.5.99 Amount of Ventilation by the Radiator Fan

Output Power		By The Radiator Fan Ventilation (m^3/Min)	Louver Area (m^2)
Power engines (kW)	Generator (kVA)		
37	37.5	150	0.8
49	50	175	1.0
60	62.5	186	1.0
71	75	191	1.1
94	100	250	1.4
116	125	311	1.7
138	150	375	2.1
182	200	400	2.2
225	250	500	2.8
268	300	600	3.3
332	375	750	4.2
440	500	1000	5.6
545	625	1250	6.9
652	750	1500	8.3
759	875	1750	9.7
864	1000	2000	11.1

4.1 (m3/min) per 1 kW for motors of 21 kW or less.
Source: Design Guideline for Telecommunications Facilities (Electric), page 3 -48

Each value and calculation result in this design are shown below.

Table7.5.100 Calculated Ventilation Rate

Taytay Sluiceway
・A '= 1.47 ・be = 0.32 (7% increase of 0.30 kg/kWh for 22 ~ 184 kW diesel engines) ・Pe = 37.0 ・ε = 2.0 ・ρ = 1.165 ・QR = 150 (**Table7.5.99**) Q1 $=\frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho}$ $=\frac{1.47 \times 0.32 \times 37 \times 2.0}{60 \times 1.165}$ Size = 0.50 Q = 0.50 + 150 = 150.50 (m3/min) = 9030 (m3/h)

Source: Study Team

(a) Examination of Fuel to be used

Diesel oil is used as fuel.

(b) Calculation of Fuel and Lubricating Oil Consumption

(i) Calculation of fuel consumption

The fuel consumption per 1 hour is as follows based on the calculation shown in Design Guideline for Telecommunications Facilities (Electric).

$$Fuel\ Consumption\ \left(\frac{L}{h}\right) = \frac{Motor\ Output\ (kw) \times Fuel\ Consumption\ Rate\ (g/kWh)}{1000\ \times Specific\ Gravity\ of\ Fuel}$$ [4]

The fuel consumption rate and specific gravity of fuel are as follows according to the Design Guideline for Telecommunications Facilities (Electric).

Table7.5.101 Fuel Consumption Rate (Unit: g/kWh)

Motor Output (kW)		Less than 22	Not less than 22 to less than 184	Not less than 184 to less than 331	Not less than 331 to less than 552	Not less than 552
Fuel Consumption Rate	Diesel Engine	310	300	270	250	230
	Gas Turbine		680	660	590	520

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3 -39

Table7.5.102 Specific Gravity of Fuel

Fuel to be Used	Specific Gravity
A Heavy Oil	0.85
Light Oil	0.83
kerosene	0.79

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3, -39

Each value and calculation result in this design are shown below.

Taytay Sluiceway
・Motor output: 37 kW ・Fuel consumption rate: 300 (diesel engine) ・Specific gravity of fuel: 0.83 (light oil) Fuel consumption (L/h) = $\frac{Motor\ Output\ (kw) \times Fuel\ Consumption\ Rate\ (g/kWh)}{1000\ \times Specific\ Gravity\ of\ Fuel}$ $= \frac{37 \times 300}{1000 \times 0.83}$ =13.37 7% increase due to radiator cooling system 13.37 × 1.07 = 14.31 (L/h)

(ii) Amount of Fuel Oil Stored

The required amount of fuel oil storage is calculated by the following formula based on Design Guideline for Telecommunications Facilities (Electric).

Amount of Fuel to be Stored(L) = Fuel Consumption (L/h) x Operating Time (h) [5]

The operation time is calculated as one hour from the design condition.

Fuel oil storage amount (L) = 14.50 × 1 = 14.50 (L)

[4] Design Guideline for Telecommunications Facilities (Electric)

[5] Design Guideline for Telecommunications Facilities (Electric)

5) Equipment to be Installed

From the design results up to the preceding paragraph, the equipment to be installed is as follows.

- Generator 37.5 kVA

6) Generator Room Plan

(a) Ventilation System

The natural ventilation system is adopted as the ventilation system of the power generation facilities.

When natural ventilation is used, the required area A of the air supply gallery is determined by the following formula.

$$A = \frac{Q1 + QR}{60 \times V \times \alpha} \ (m^2)$$

where:

A : Louver area (m2)
Q1 : Amount of Air Required For Combustion of Fuel (m3/min)
QR : Radiator Fan Ventilation (m3/min)
V : Wind Velocity 4 m/sec
α : Light Transmittance (Size = 0.3), when a wire gauze is installed on the light transmittance (Size = 0.27)

Each value and calculation result in this design are shown below.

Taytay Sluiceway
$A = \frac{0.50 + 150}{60 \text{ x } 4 \text{ x } 0.27} = 2.32 \text{ m}^2$

(b) Separation Distance Between Devices

The separation distance between the components of the generator is based on **Table7.5.103** described in Design Guideline for Telecommunications Facilities (Electric).

Table7.5.103 Holding Distance Between Devices

Portion To Secure The Separation Distance			Separation Distance
Cubicle Type		Operation Side	More than 1.0m
		Maintenance Side	More than 0.6 m In case of transformers, storage batteries and portion facing to a building, 1.0m
		Ventilation Side	More than 0.2 m
Not Cubicle Type	Generator	Between Generators	More than 1.0 m
		Around Generators	More than 0.6 m
	Generator Control Board	Operation Side	More than 1.0 m (In case of facing each other, 1.2m)
		Maintenance Side	More than 0.6 m (In case of facing each other, 1.0m)
		Ventilation Side	More than 0.2 m
	Fuel Tank Of Small Lots	Internal - combustion engine	More than 0.6 m More than 2.0 m
		Inside surface of Oil Retaining Wall	According to Local Law
	DC Power Supply Equipment	Operation Side	More than 1.0 m
		Maintenance Side	More than 0.6 m

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3-65

(c) Seismic Measures for Power Generation Equipment

The generator and prime mover shall be directly connected to each other and fixed to a common trestle having a vibration-proof and earthquake-proof structure. The generator shall be equipped with a fall prevention device to prevent the generator from falling off due to an earthquake.

(i) Basic Dimensions of the Generator

Basic dimensions of power generation equipment shall be examined in accordance with the Telecommunications Facility Design Guideline for Telecommunications Facilities (Electric) p. 3 -53.

(1) Foundation
1) Dimensions of Foundation
The dimensions of foundation is as follows.
Width => (Width of Common Bed) + 0.3 (m) Note
Length => (Length of Common Bed) + 0.3 (m) Note
Bottom to Surface of Foundation =>0.1 (m)

Note: In case of adhesive anchor, 0.3m. 0.5 m or more shall be secured in case of blackout anchor.

Source: Study Team translated based on Design Guideline for Telecommunications Facilities (Electric), P3 -53

The installation dimensions of the generator used for the study are set with reference to a manufacturer's generator. The generator dimensions and foundation dimensions is indicated in **Table7.5.104**.

Table7.5.104 Generator dimensions and foundation dimensions

Generator	Model Number	Generator Dimensions (mm)	Foundation Dimensions (Mm)
37.5 kVA	DCA -20 LSK	Width: 650 length: 1540	Width: 650 + 300 = 950 Length: 1540 + 300 = 1840

(ii) Seismic Calculation of Anchor Bolts

The anchor bolt size of the generator and the fuel tank shall be selected based on the "Guidelines for earthquake-resistant design and construction of building equipment 2014". As a result of the calculation, the following anchor bolts were selected. The detailed calculations are shown in **Vol.5A Structural Calculation for Contract Package-1**. As a result of the calculation, there is no shortage of strength in the use of anchor bolts W 5/8 (8 sticks).

7) Setting Display Items

The generator board mounted on the generator shall be equipped with a status alarm indication by lamp so that the generator status and fault alarm can be visually confirmed. The contents and items to be indicated are shown in **Table 7.5.105**.

Table 7.5.105 Contents and Items to be Indicated

(1) The light source shall be a light-emitting diode. LED lights must be replaced easily. When it is not easy to replace LED lights on printed wiring board, the spare of the board shall be stored.
(2) Condition indicator(monitoring items and control items) and external connection terminal are as shown in the following Table 1.
(3) Protective device shall be attached in accordance with Table 2.
  1) In case of failure, the contents must be notified with lamp indicators and alarm.
  2) When a failure occurs, self-holding shall be performed, and it shall be released by the operation of a push bottom for "Failure Recovery"

Table 1

Item to be Displayed		Outside Contact		Remarks
		Monitoring	Control	
1	Commercial	○	-	Lighting while commercial power is normal
2	Abnormal in Commercial Power	○	-	
3	Power Generation	○	-	Lighting when the build up voltage of generator
4	Power Supply	○	○	Lighting In Circuit Breaker on or Switcher on
5	Power Generation and Supply	○	○	Lighting In Circuit Breaker on or Switcher on
6	Machine Side	○		Based on the switch of "Machine Side" or "Remote"
7	Remote	○	-	
8	Manual Start-up	-	-	Based on the switch of "Manual", "Automatic" or "Test"
9	Automatic Start up	-	-	
10	Starting	-	○	
11	Stop	-	○	

Source: Study team translated from" Emergency Power Generation Device Specifications (Draft) January 2017, p. 10".

Table 2

Type	Item to be Indicated	Engine Stop	Main Circuit Break	Detector	Outside Contact
Major Failure	Start Failure	○	-	Timer/Switch To Detect Start Failure	○
	Lubricating Oil Pressure Decrement	○	○	Switch To Detect Hydraulic Pressure	○
	Cooling Water Cut-off/Temperature Rising	○	○	Switch To Detect Water Cut-Off/Water Temperature	○
	Over Rotating	○	○	Switch To Detect Over Rotating	○
	Over Current		○	Over Current Relay	○
	Minimum Amount of Fuel	○	○	Equipment To Detect Oil Surface	○
	Emergency Stop	○	○	Manual	○
	Control Power Abnormality	○	○	DC Undervoltage Relay	○
Minor Failure	Oil Surface of Fuel Decrement	-	-	Equipment To Detect Oil Surface	○
	Battery Temperature Rising	-	-	Alert To Indicate Battery Temperature Rising	○
	Auxiliary Device Failure	-	-	Over Current Relay/Open-Phase Relay	○

Note: "○" indicates "must be adopted".

Source: Study team translated from" Emergency Power Generation Device Specifications (Draft) January 2017, p. 11"

7.6 Structural Design of Buildings

7.6.1 Conditions for Structural Design of Buildings

7.6.1.1 Load

(1) Floor Live Load

Floor live load is set forth by type of occupancy in NSCP as shown in **Table 7.6.3**. **Table 7.6.1** shows the determined live load for generator house in this project. Floor live load for exterior deck of local control house will be referred by the one for electrical room of generator house since the deck will bear similar equipment such as control panels. Interior floor slab of local control house is not included in building design since it is a part of civil sub structure.

Table 7.6.1 Applied Floor Live Load in Generator House / Exterior Deck of Local Control House

Occupancy	Category	Uniform Load (kN/m2)	Remarks
Generator Room	Storage - Heavy	12.0	
Electric Room	Storage - Light	6.0	
Staff Room	Office - Offices	2.4	
Toilet & Bathroom	Restrooms	2.4	
Exterior Deck of Local Control House	Storage - Light	6.0	Equivalent to Electrical Room

Source: Study Team

For generator rooms, uniform load of 12.0(kN/m2) by the category "Storage – Heavy", which is the largest floor live load suggested in NSCP, is applied since those room shall bear heavy weight of generators. Estimated weight of generators with fuel fully filled is shown in **Table 7.6.2**. Since the weight of generators per area varies between 6.0 to 9.0 (kN/m2), 12.0(kN/m2) is reasonable considering allowance.

Table 7.6.2 Weight of Generators including Fuel (kg/m2)

Generator Type		Reference Model	Self Weight (kg)	Built-in Tank Cap. (L)	Fuel Weight (kg)	Weight in Total (kg)	Size (m)		Occupancy Area (m2)	Weight per Area (kg/m2)
							L	W		
MCGS	Gate Hoisting	*DENYO DCA-300*	*4160*	*490*	*416.5*	*4577*	*3.75*	*1.4*	*5.25*	*872*
	Others	*YANMAR YH280*	*885*	*85+750 option*	*709.75*	*1595*	*2.1*	*0.98*	*2.058*	*776*
Cainta	Gate Hoisting	*DENYO DCA-150*	*2390*	*250*	*212.5*	*2603*	*3.25*	*1.08*	*3.51*	*742*
	Others	*YANMAR YH220*	*870*	*85+330 option*	*352.75*	*1223*	*2.1*	*0.98*	*2.058*	*595*

Source: Study Team

For electrical rooms, uniform load of 6.0(kN/m2) by the category "Storage – Light" is applied since those room shall bear relatively light equipment such as control panels. "The guideline of structural design for public building" by MLIT of Japan suggested 4.9(kN/m2) for electric room, which implies 6.0(kN/m2) is reasonable condition with enough allowance.

Table 7.6.3 List of Floor Live Load in NSCP

Use or Occupancy		Uniform Load[1]	Concentrated Load
Category	Description	kPa	kN
13. Office	Call Centers & BPO	2.9	9.0
	Lobbies & ground floor corridors	4.8	9.0
	Offices	2.4	9.0[2]
	Building corridors above ground floor	3.8	9.0
14. Printing plants	Press rooms	7.2	11.0[2]
	Composing and linotype rooms	4.8	9.0[2]
15. Residential[8]	Basic floor area	1.9	0[6]
	Exterior balconies	2.9[4]	0
	Decks	1.9[4]	0
	Storage	1.9	0
16. Restrooms[9]	--	--	--
17. Reviewing stands, grandstands, Bleachers, and folding and telescoping seating	--	4.8	0
18. Roof decks	Same as area served or Occupancy	--	--
19. Schools	Classrooms	1.9	4.5[2]
	Corridors above ground floor	3.8	4.5
	Ground floor corridors	4.8	4.5
20. Sidewalks and driveways	Public access	12.0	--[7]
21. Storage	Light	6.0	--
	Heavy	12.0	--
22. Stores	Retail	4.8	4.5[2]
	Wholesale	6.0	13.4[2]
23. Pedestrian bridges and walkways	--	4.8	--

NOTES FOR TABLE 205-1

[1] *See Section 205.5 for live load reductions.*
[2] *See Section 205.3.3, first paragraph, for area of load application.*
[3] *Assembly areas include such occupancies as dance halls, drill rooms, gymnasiums, playgrounds, plazas, terraces and similar occupancies that are generally accessible to the public.*
[4] *For special-purpose roofs, see Section 205.4.4.*
[5] *Exit facilities shall include such uses as corridors serving an occupant load of 10 or more persons, exterior exit balconies, stairways, fire escapes and similar uses.*
[6] *Individual stair treads shall be designed to support a 1.3 kN concentrated load placed in a position that would cause maximum stress. Stair stringers may be designed for the uniform load set forth in the table.*
[7] *See Section 205.3.3, second paragraph, for concentrated loads. See Table 205-2 for vehicle barriers.*
[8] *Residential occupancies include private dwellings, apartments and hotel guest rooms.*
[9] *Restroom loads shall not be less than the load for the occupancy with which they are associated, but need not exceed 2.4 kPa.*

Source: NSCP VOL. II (2010)

Roof live load may vary between 0.70(kN/m2) as set forth by tributary area and roof slope shown in **Table 7.6.3**.

Table 7.6.4 List of Roof Live Load in NSCP

Table 205-3 Minimum Roof Live Loads [1]

ROOF SLOPE	METHOD 1			METHOD 2		
	Tributary Area (m^2)			Uniform Load [2] (kPa)	Rate of Reduction, r	Maximum Reduction R (percentage)
	0 to 20	20 to 60	Over 60			
	Uniform Load (kPa)					
1. Flat [3] or rise less than 4 units vertical in 12 units horizontal (33.3% slope). Arch and dome with rise less than one-eighth of span.	1.00	0.75	0.60	1.00	0.08	40
2. Rise 4 units vertical to less than 12 units vertical in 12 units horizontal (33.3% to less than 100% slope). Arch and dome with rise one-eighth of span to less than three-eighths of span.	0.75	0.70	0.60	0.75	0.06	25
3. Rise 12 units vertical in 12 units horizontal (100% slope) and greater. Arch or dome with rise three-eighths of span or greater.	0.60	0.60	0.60	0.60	*No reduction permitted*	
4. Awnings except cloth covered. [4]	0.25	0.25	0.25	0.25		
5. Greenhouses, lath houses and agricultural buildings. [5]	0.50	0.50	0.50	0.50		

Source: NSCP VOL. II (2010)

(2) Wind Load

In NSCP VOL. II (2010), two methods are available to obtain design wind pressure, namely Simple Procedure and Analytical Procedure. Simple Procedure can be applied for structures which satisfy the conditions as defined in NCSP Section 207.5. In Simple Procedure, design wind pressure for the Main Wind-Force Resisting System is given by following equations:

$$ps = \lambda K_{zt}\, I_w\, p_{s9}$$

where,

ps	: design wind pressure by Simple Procedure
λ	: adjustment factor for building height and exposure as in NSCP Figure 207-2
p_{s9}	: wind pressure as defined in NSCP Figure 207-2 at average height 9m and exposure B
Iw	: importance factor as defined in NSCP Table 207-3
Kzt	: topographic factor as defined in NSCP Section 207.5.7

The wind pressure p_{s9} shall be in accordance with **Table 7.6.5**, under the conditions that basic wild speed is 200(kph) as project sites are within Zone2 and roof angle is 0 to 5 degree.

Table 7.6.5 Design Wind Pressures for Main Wind -Force Resisting System

Basic Wind Speed (kph)	Roof Angle (°)	Load Case	Horizontal Pressures, kPa				Vertical Pressures, kPa				Overhangs	
			A	B	C	D	E	F	G	H	E_{oh}	G_{oh}
150	0 to 5	1	0.66	-0.34	0.44	-0.21	-0.79	-0.45	-0.55	-0.35	-1.11	-0.87
	10	1	0.75	-0.31	0.50	-0.18	-0.79	-0.48	-0.55	-0.37	-1.11	-0.87
	15	1	0.83	-0.28	0.55	-0.16	-0.79	-0.52	-0.55	-0.40	-1.11	-0.87
	20	1	0.92	-0.24	0.61	-0.13	-0.79	-0.55	-0.55	-0.42	-1.11	-0.87
	25	1	0.83	0.13	0.60	0.14	-0.37	-0.50	-0.27	-0.40	-0.69	-0.59
		2	0.00	0.00	0.00	0.00	-0.14	-0.27	-0.04	-0.18	0.00	0.00
	30 to 45	1	0.74	0.51	0.59	0.41	0.06	-0.45	0.02	-0.39	-0.26	-0.30
		2	0.74	-0.08	0.59	0.41	0.29	-0.22	0.25	-0.16	-0.26	-0.30
200	0 to 5	1	1.18	-0.62	0.79	-0.36	-1.42	-0.81	-0.99	-0.63	-2.00	-1.57
	10	1	1.34	-0.56	0.89	-0.32	-1.42	-0.87	-0.99	-0.67	-2.00	-1.57
	15	1	1.49	-0.49	0.99	-0.28	-1.42	-0.93	-0.99	-0.71	-2.00	-1.57
	20	1	1.64	-0.43	1.10	-0.24	-1.42	-0.99	-0.99	-0.75	-2.00	-1.57
	25	1	1.48	0.24	1.08	0.24	-0.66	-0.90	-0.48	-0.72	-1.23	-1.05
		2	-	-	-	-	-0.25	-0.49	-0.07	-0.31	-	-
	30 to 45	1	1.34	0.91	1.06	0.73	0.11	-0.81	0.04	-0.69	-0.47	-0.54
		2	1.34	0.91	1.06	0.73	0.51	-0.40	0.45	-0.29	-0.47	-0.54
250	0 to 5	1	1.84	-0.95	1.22	-0.57	-2.21	-1.26	-1.54	-0.97	-3.09	-2.42
	10	1	2.07	-0.86	1.38	-0.50	-2.21	-1.35	-1.54	-1.04	-3.09	-2.42
	15	1	2.31	-0.77	1.54	-0.44	-2.21	-1.44	-1.54	-1.10	-3.09	-2.42
	20	1	2.54	-0.67	1.70	-0.37	-2.21	-1.54	-1.54	-1.17	-3.09	-2.42
	25	1	2.31	0.37	1.67	0.38	-1.03	-1.40	-0.74	-1.12	-1.91	-1.63
		2	-	-	-	-	-0.39	-0.76	-0.11	-0.49	-	-
	30 to 45	1	2.07	1.41	1.65	1.13	0.16	-1.26	0.05	-1.08	-0.73	-0.83
		2	2.07	1.41	1.65	1.13	0.79	-0.62	0.69	-0.44	-0.73	-0.83

Figure 207-2 Design Wind Pressures on Walls and Roofs of Enclosed Buildings with h ≤ 18m, Main Wind-Force Resisting System – Method 1

Source: NSCP VOL. II (2010)

I_w =1.15 and λ=1.0 shall be applied in accordance with **Table 7.6.6**. K_{zt} shall be 1.0 as defined in NSCP Section 207.5.7

Table 7.6.6 Factors for Main Wind -Force Resisting System

Table 207-3 Importance Factor, I_w (Wind Loads)

Occupancy Category[1]	Description	I_w
I	Essential	1.15
II	Hazardous	1.15
III	Special Occupancy	1.15
IV	Standard Occupancy	1.00
V	Miscellaneous	0.87

Adjustment Factor for Building Height and Exposure λ

Mean roof height (m)	Exposure		
	B	C	D
4.5	1.00	1.21	1.47
6.0	1.00	1.29	1.55
7.5	1.00	1.35	1.61
9.0	1.00	1.40	1.66
11.0	1.05	1.45	1.70
12.0	1.09	1.49	1.74
13.7	1.12	1.53	1.78
15.2	1.16	1.56	1.81
16.8	1.19	1.59	1.84
18.0	1.22	1.62	1.87

WALLS AND ROOFS

Source: NSCP VOL. II (2010)

(3) Seismic Load

In NSCP VOL. II (2010), two methods are available to obtain base shear, namely Static Force Procedure and Dynamic Analysis Procedure. The buildings with following conditions as defined in NCSP Section 208.4.8.3 have to use Dynamic Analysis;

1. Structures 75m or more in height,
2. Structures having a stiffness, weight or geometric vertical irregularity of Type1,2,3 as defined in NSCP Table 208-9,
3. Structures over 5 stories or 20m in height in Seismic Zone 4 not having same structure system throughout their height except as permitted by Section 208.6.2, and
4. Structures located on Soil Profile Type S_F and having a period greater than 0.7 second.

In this project, both generator houses and local control house do not match above condition so that Static Force can be applied. In Static Analysis, shear base V is given by following equations:

$V = \frac{C_v I}{RT} W$ (Eq. 208-4 of NSCP)

$V \leq \frac{2.5 C_a I}{R} W$ (Eq. 208-5 of NSCP)

$\geq 0.11 C_a I W$ (Eq. 208-6 of NSCP)

$\geq \frac{0.8 Z N_v I}{R} W$ in Zone 4 (Eq. 208-7 of NSCP)

Where,

V : design base shear

I : importance factor as defined in NSCP Table 208-1

Z : seismic zone factor as defined in NSCP Table 208-3 and Zone4 shall be applied in this project

N_v : near-source factor as defined in NSCP Table 208-5

N_a : near-source factor as defined in NSCP Table 208-6

C_v : seismic coefficient as defined in NSCP Table 208-7

C_a : seismic coefficient as defined in NSCP Table 208-8

R : numerical coefficient representing the inherent overstrength and global ductility as defined in NSCP Table 208-11

T : elastic fundamental period of vibration

W : the total dead load

Elastic period T shall be calculated by following equations;

$$T = C_t\,(h_n)^{3/4} \qquad \text{(Eq. 208-8 of NSCP)}$$

where h_n is height from ground at level n, and C_t=0.0731 is given for reinforced concrete structures.

For importance factor, I=1.5 shall be applied as structures in this project are regarded highly important for disaster response.

Seismic zone factor shall be Z=0.4 as NCR and Rizal are classified as ZONE4.

Soil profile types in NSCP Table 208-1 are required to determine Near-source factor N_v and N_v. Considering the average soil properties for top 30m at each site, S_D (Stiff Soil) is applied for MCGS and S_E (Soft Soil) is applied for Cainta/Taytay.

Since West Valley Fault runs along Marikina River near the project sites, Type A is applied for seismic source and distance to known seismic source shall be less than 5km.

Table 7.6.7 Selected Coefficients for Static Seismic Load (1/2)

Table 208-1 - Seismic Importance Factors

Occupancy Category [1]	Seismic Importance Factor, I	Seismic Importance [2] Factor, I_p
I. Essential Facilities [3]	1.50	1.50
II. Hazardous Facilities	1.25	1.50
III. Special Occupancy Structures [4]	1.00	1.00
IV. Standard Occupancy Structures [4]	1.00	1.00
V. Miscellaneous structures	1.00	1.00

Table 208-2 - Soil Profile Types

Soil Profile Type	Soil Profile Name / Generic Description	Average Soil Properties for Top 30 m of Soil Profile: Shear Wave Velocity, V_s (m/s)	SPT, N (blows/ 300 mm)	Undrained Shear Strength, S_U (kPa)
S_A	Hard Rock	> 1500		
S_B	Rock	760 to 1500		
S_C	Very Dense Soil ad Soft Rock	360 to 760	> 50	> 100
S_D	Stiff Soil Profile	180 to 360	15 to 50	50 to 100
S_E [1]	Soft Soil Profile	< 180	< 15	< 50
S_F	Soil Requiring Site-specific Evaluation. See Section 208.4.3.1			

Sc for MCGS

Sd for Cainta Taytay

Table 208-3 Seismic Zone Factor Z

ZONE	2	4
Z	0.20	0.40

Table 208-4 Near-Source Factor N_a [1]

Seismic Source Type	Closest Distance To Known Seismic Source[2]: ≤ 5 km	≥ 10 km
A	1.2	1.0
B	1.0	1.0
C	1.0	1.0

Table 208-5 Near-Source Factor, N_v [1]

Seismic Source Type	Closest Distance To Known Seismic Source[2]: ≤ 5 km	10 km	≥ 15 km
A	1.6	1.2	1.0
B	1.2	1.0	1.0
C	1.0	1.0	1.0

Table 208-6 - Seismic Source Types [1]

Seismic Source Type	Seismic Source Description	Seismic Source Definition Maximum Moment Magnitude, M
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	$M \geq 7.0$
B	All faults other than Types A and C.	$6.5 \leq M < 7.0$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$

[1] *Subduction sources shall be evaluated on a site-specific basis.*

Table 208-7 - Seismic Coefficient, C_a

Soil Profile Type	Seismic Zone Z	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	*See Footnote 1 of Table 208-8*	

Table 208-8 - Seismic Coefficient, Cv

Soil Profile Type	Seismic Zone Z	
	Z=0.2	Z=0.4
S_A	0.16	$0.32N_V$
S_B	0.20	$0.40N_V$
S_C	0.32	$0.56N_V$
S_D	0.40	$0.64N_V$
S_E	0.64	$0.96N_V$
S_F	*See Footnote 1 of Table 208-8*	

[1] *Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients.*

Source: NSCP VOL. II (2010)

For ductility coefficient, R=3.5 can be applied as both generator house and local control house are Moment-Resisting Frame System consists of concrete members.

Table 7.6.8 Selected Coefficients for Static Seismic Load (2/2)

Table 208-11A Earthquake-Force-Resisting Structural Systems of Concrete

Basic Seismic-Force Resisting System	R	Ω_0	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
• Special reinforced concrete shear walls	4.5	2.8	NL	50
• Ordinary reinforced concrete shear walls	4.5	2.8	NL	NP
B. Building Frame Systems				
• Special reinforced concrete shear walls or braced frames	5.5	2.8	NL	75
• Ordinary reinforced concrete shear walls or braced frames	5.6	2.2	NL	NP
• Intermediate precast shear walls or braced frames	5.5	2.8		
C. Moment-Resisting Frame Systems				
• Special reinforced concrete moment frames	8.5	2.8	NL	NL
• Intermediate reinforced concrete moment frames	5.5	2.8	NL	NP
• Ordinary reinforced concrete moment frames	3.5	2.8	NL	NP
D. Dual Systems				
• Special reinforced concrete shear walls	8.5	2.8	NL	NL
• Ordinary reinforced concrete shear walls	6.5	2.8	NP	NP
E. Dual System with Intermediate Moment Frames				
• Special reinforced concrete shear walls	6.5	2.8	NL	50
• Ordinary reinforced concrete shear walls	4.2	2.8	NL	50
• Shear wall frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	4.2	2.8	NP	NP
F. Cantilevered Column Building Systems				
• Cantilevered column elements	2.2	2.0	NL	10
G. Shear Wall- Frame Interaction Systems	5.5	2.8	NL	50

Source: NSCP VOL. II (2010)

7.6.1.2 Seismic Design Policy for Local Control House of Floodgates

Since local control houses will be built on top of the gate structures, amplification of seismic force shall be considered. NSCP VOL. II (2010) provides following equations to calculate the forces at each level:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \text{(Eq. 208-15 of NSCP)}$$

$$V = F_t + \sum_{i=1}^{n} F_i \qquad \text{(Eq. 208-13 of NSCP)}$$

$$F_t = 0.07\ TV \qquad \text{(Eq. 208-14 of NSCP)}$$

where,

$F_{x,i}$: lateral seismic force at level x or i

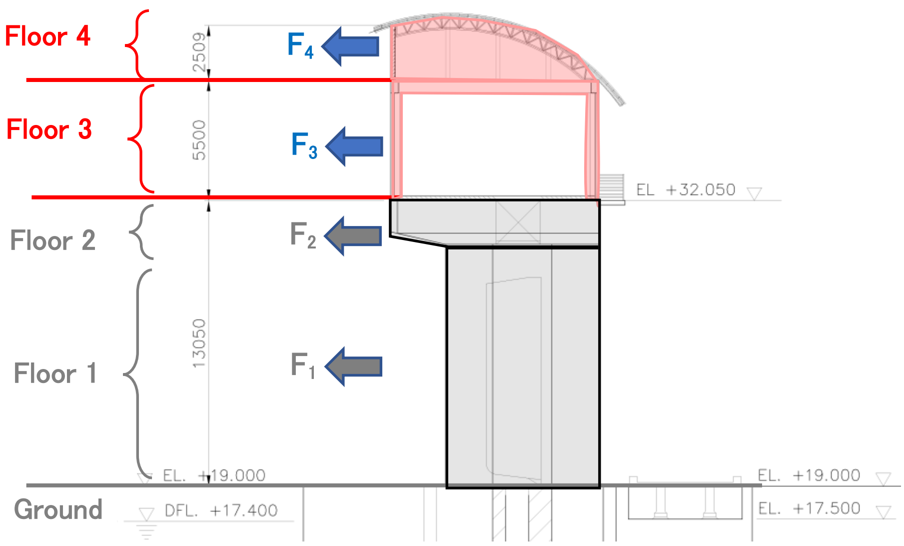
V : the total design lateral force

F_t : concentrated force at the top

$w_{x,i}$: weight at level x or i

$h_{x,i}$: height at level x or i

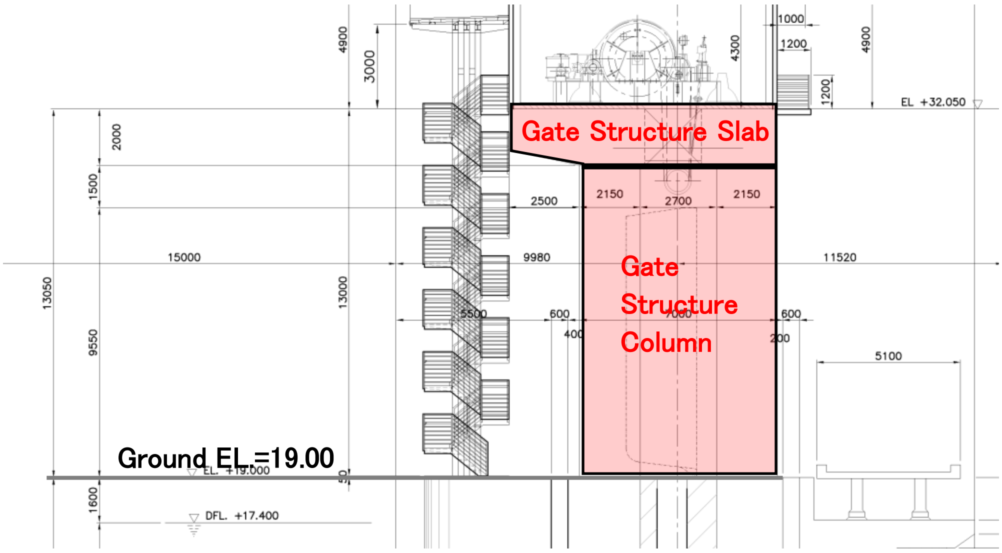
Vertical Distribution for local control house shall be calculated integrally with civil sub-structure as described in **Figure 7.6.1**, with assuming flood gate structure consists of 4 levels which are; gate column for level 1, gate slab for level 2, RC frame of local control house for level 3 and steel roof structure for level 4. In seismic design of local control house, only the force for level 2 and level 3 will be used.



Source: Study Team

Figure 7.6.1 Calculation Model for Vertical Distribution

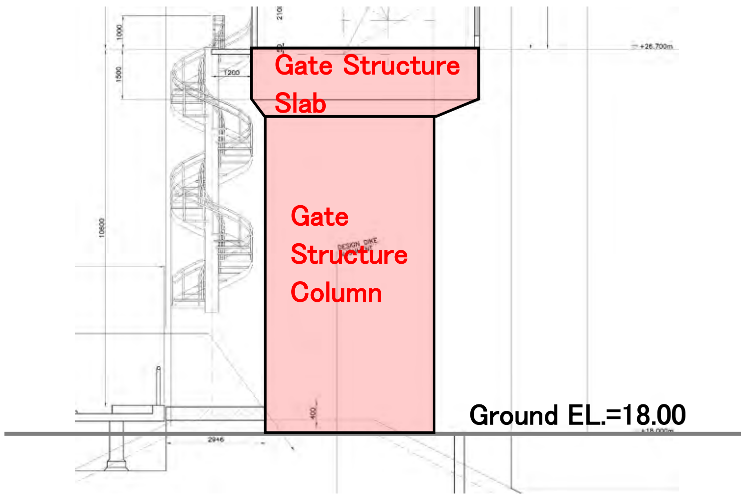
For calculation of Fx and T, w_1 (weight for gate structure column) and w_2 (weight for gate structure slab) are given as follows, with assuming the unit weight of 24.0(kN/m3) for reinforced concrete.



		W	D	H	n	V	Weight
Gate Slab	Slab	9.00	9.50	1.50	1	128.25	
	Tapering	9.00	9.50	0.25	1	21.38	
		4.50	7.00	0.25	1	7.88	w2 =
	Total					157.50	378.00
Gate Column	Column	4.50	7.00	11.50	1	362.25	
	Hollow	1.00	2.70	11.50	-2	-62.10	w1 =
	Total					300.15	720.36

Source: Study Team

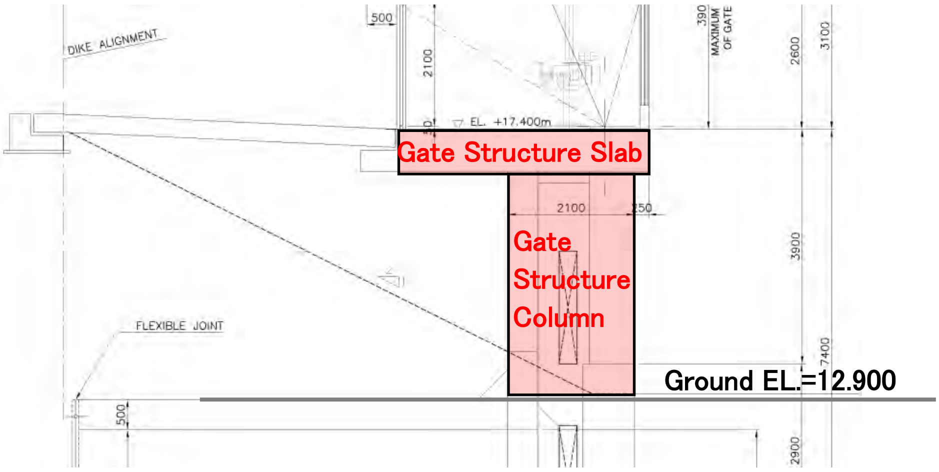
Figure 7.6.2 Gate Column and Gate Slab of MCGS



		W	D	H	n	V	Weight
Gate Slab	Slab	7.80	6.75	1.50	1	78.98	
	Tapering	7.80	6.75	0.25	1	13.16	
		3.00	6.00	0.25	1	4.50	w2 =
	Total					96.64	231.93
Gate Column	Column	3.00	6.00	11.50	1	207.00	
	Hollow	3.00	0.90	11.50	-1	-31.05	w1 =
	Total					175.95	422.28

Source: Study Team

Figure 7.6.3 Gate Column and Gate Slab of Cainta



		W	D	H	n	V	Weight
Gate Slab	Slab	11.90	4.20	0.70	1	34.99	w2 =
	Total					34.99	83.97
Gate Column	Column1	0.40	0.85	3.80	4	5.17	
	Column2	0.65	0.50	3.80	2	2.47	
	Column3	0.90	0.50	3.80	2	3.42	w1 =
	Total					7.64	18.33

Source: Study Team

Figure 7.6.4 Gate Column and Gate Slab of Taytay

7.6.1.3 Soil Bearing Capacity for Foundation Design of Generator Houses

Both MCGS and Cainta generator houses will be built on backfill of non-consolidation. The backfill is laid on hard strata in MCGS and laid on pile-supported slab of sub-structure in Cainta. Therefore, subsidence due to soft soil consolidation is ignorable.

The backfill material shall be specified as DPWH Pay Item 104(2)d "Embankment from Borrow - Granular Coarse Material". Soil factors for backfill are assumed as shown in **Table 7.6.9**. Ground water table can be assumed enough below the bottom of foundation slab with proper drainage of surrounding retaining wall.

Table 7.6.9 Soil Factors for Backfill

	Effective Friction Angle (degree)	Cohesion (kN/m2)	Effective Unit Weight (kN/m3)
Value	30.0	0.0	19.3

Source: Study Team

Soil bearing capacity shall be calculated by following equations provided in DGCS Volume 6 and DGCS Volume 2 Geotechnical and Geological. Further modification may be applied for consideration of the shallow foundation in accordance with DGCS Volume 2 Geotechnical and Geological.

The general soil bearing capacity equation for shallow strip footings is:

$$q_{ult} = cN_c + qN_q + 0.5\gamma BN_\gamma$$

where:

q_{ult} = ultimate bearing capacity

c = cohesion of soil

q = overburden pressure at footing base

γ = soil unit weight beneath footing

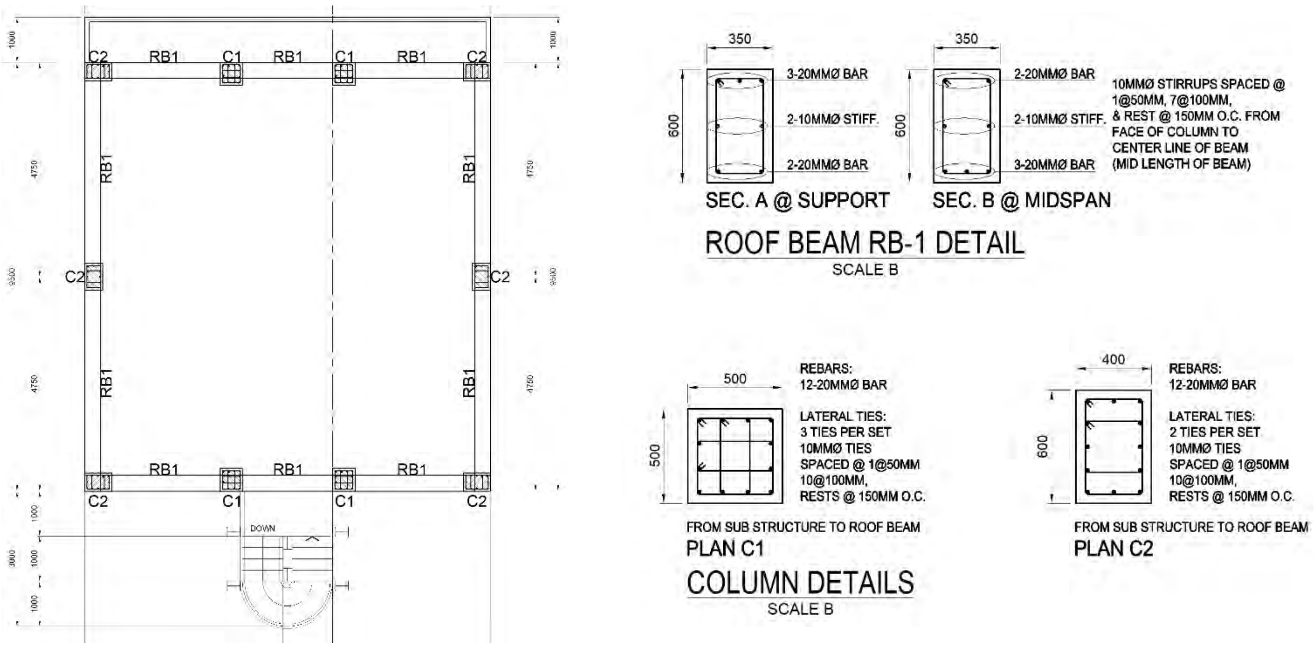
B = footing width

$N_c.N_q N_y$ = bearing capacity factors

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right) e^{\pi \tan\phi'}$$
$$N_y = 2(N_q + 1)\tan\phi'$$
$$N_c = (N_q - 1)\cot\phi'$$

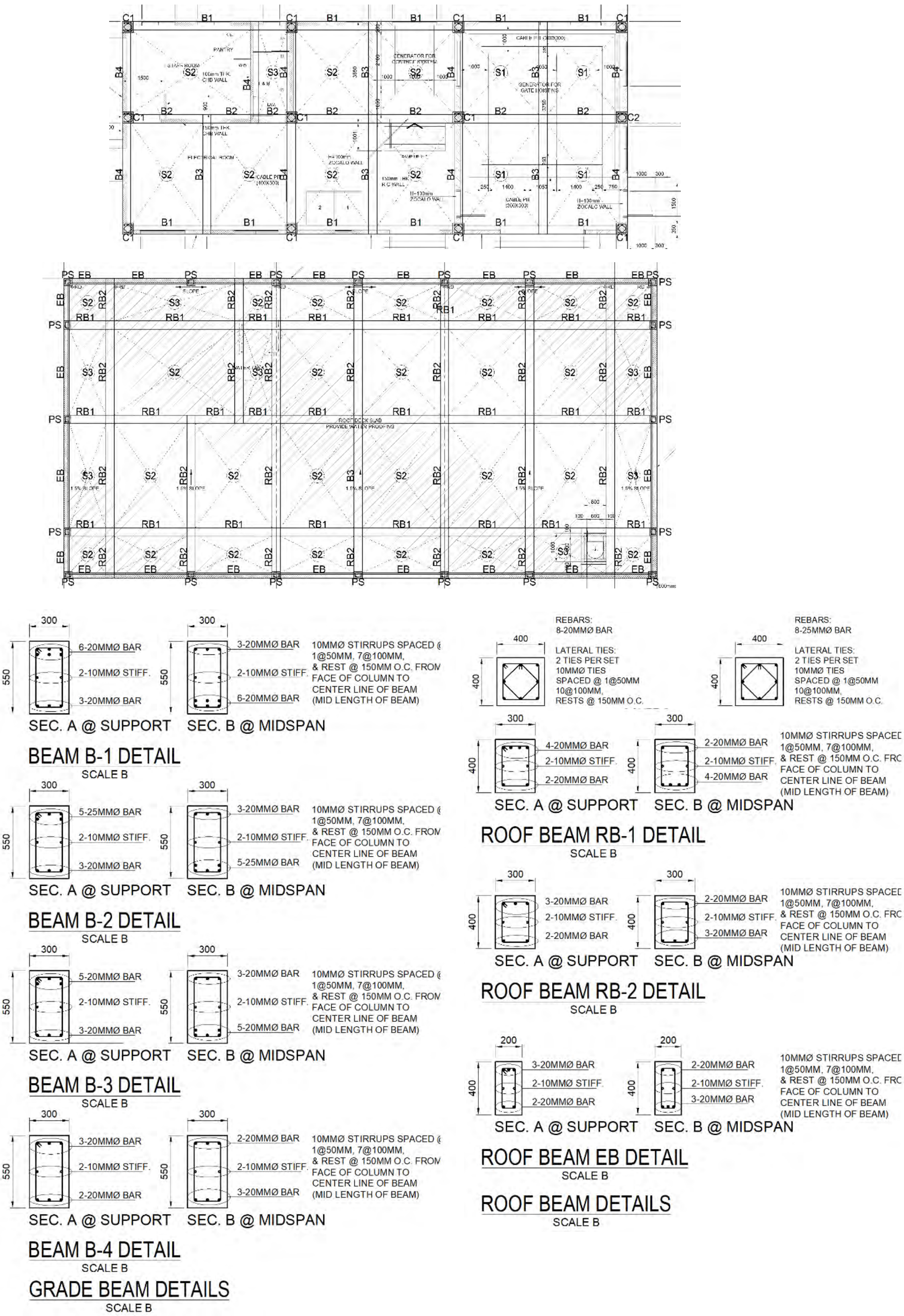
7.6.1.4 Structural Calculation Result

Section dimensions determined from the results of the structural calculation and the member checkout of each building are shown. For more information about structural calculations, see Volume. Section III -2.



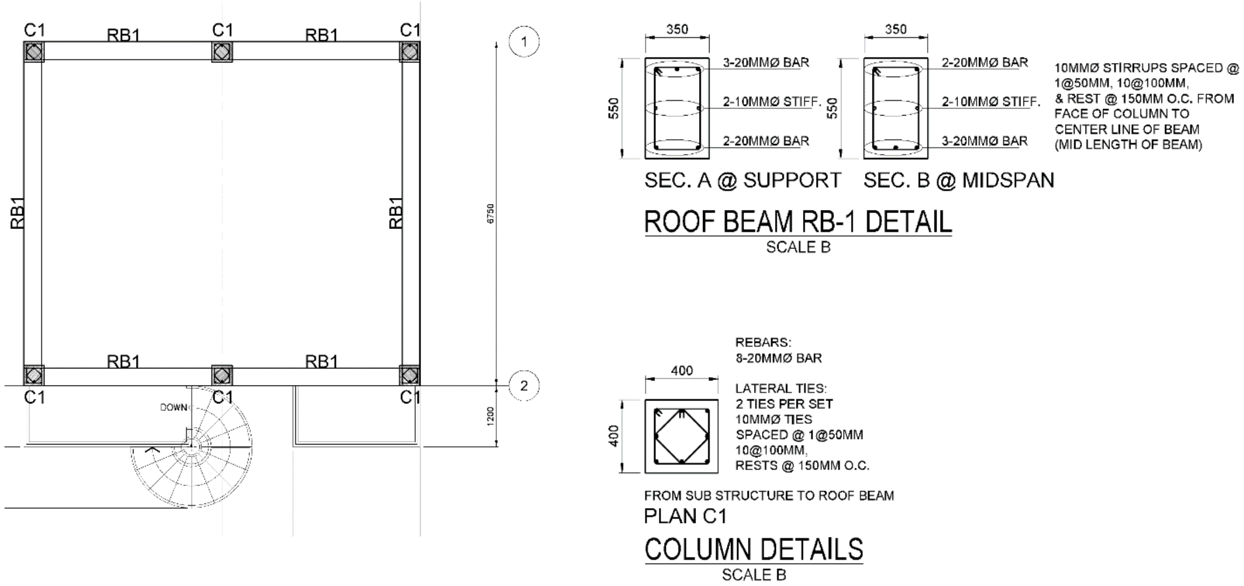
Source: Study Team

Figure7.6.5 Typical Member Section for MCGS Local Control House



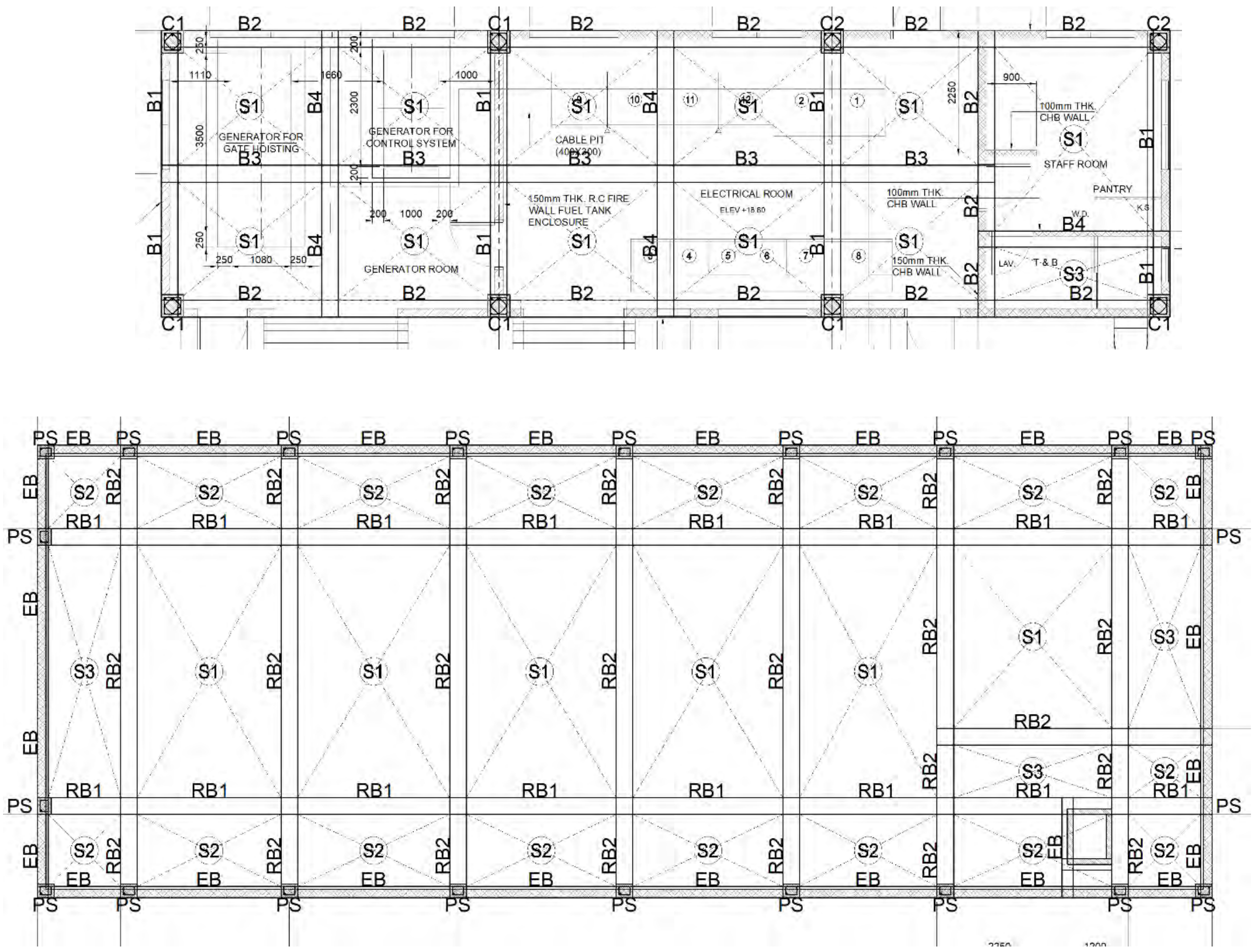
Source: Study Team

Figure7.6.6 Typical Member Section for MCGS Generator House



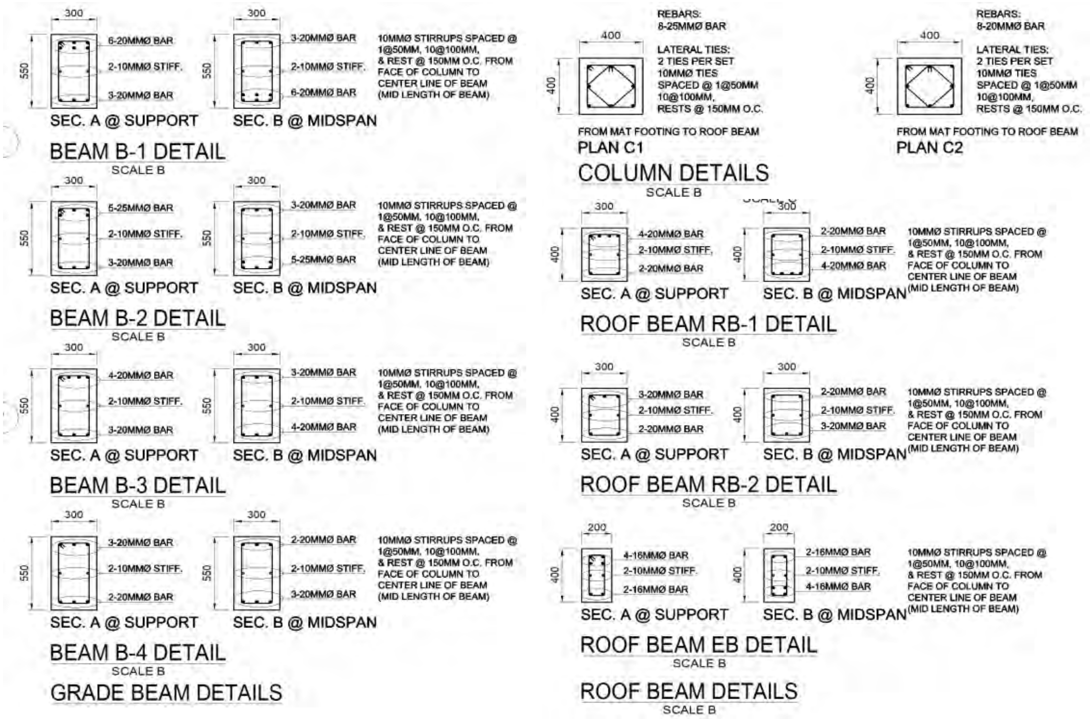
Source: Study Team

Figure7.6.7 Typical Member Section for Cainta Local Control House



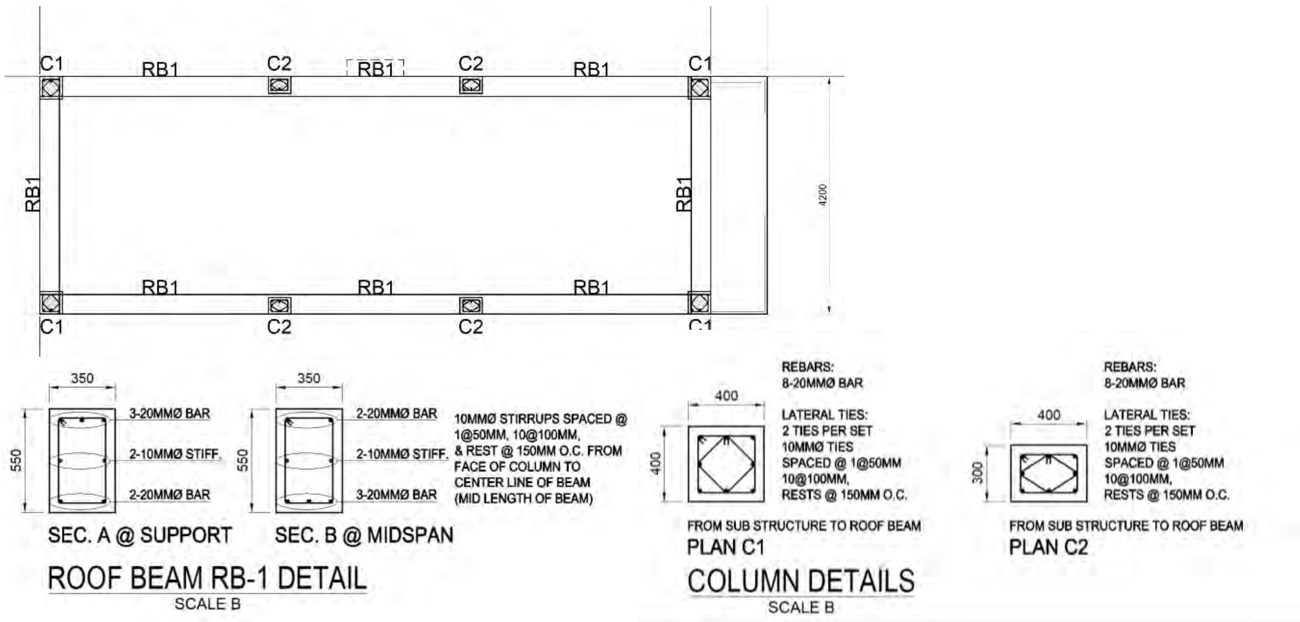
Source: Study Team

Figure7.6.8 Typical Member Section for Cainta Generator House (1/2)



Source: Study Team

Figure7.6.9 Typical Member Section for Cainta Generator House (2/2)



Source: Study Team

Figure7.6.10 Typical Member Section for Taytay Local Control House

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". This section features matters with unique to this project.

(1) Water Supply to the MCGS Generator House

The MCGS site does not face to public road so that a water line of more than 600 m shall be installed from EFCOS site. Considering the length of waterline, the head loss shall be examined to determine the diameter of water supply pipe.

According to DGCS Volume 6 – "Public Buildings and Other Related Structures", minimum requirement of water pressure for public pipe network is 1.7 (kg/cm2). Meanwhile, minimum water pressure at demand side pipe end water tap is 0.03 MPa to supply water to the rooftop tank. The table below shows conversion results of those water pressure to water head.

Table7.6.10 Conversion of Water Supply Pressure to Water Head

	Water Supply Pressure	Required Pressure at Tap
Water Pressure (kg/cm2)	1.7	-
Water Pressure (MPa)	0.1666	0.03
Water Head (m)	17.00	3.06

Source: Study Team

The head loss of straight pipes can be calculated by following equation when the pipe diameter is 50 mm or less. Toilet bowl flushing (about 20 L/mi) is the expected as the type of water usage with maximum flow rate Q.

$$\mathrm{h} = \left(0.0126 + \frac{0.01739 - 0.1087 * D}{\sqrt{\mathrm{V}}}\right) * \frac{L}{D} * \frac{V\text{^}2}{2g} \qquad V = Q * \frac{4}{\pi * D\text{^}2}$$

where:

V: Average flow velocity in the pipe (m/s)

L: Length of pipe (m)

D: Inside diameter of pipe (m)

g: Acceleration of gravity (9.8 m/s 2)

Q: Flow rate (m3/s)

Table7.6.11 Head Loss of Straight Pipes by Diameter

Dia.	40mm	32mm	25mm
h1 : Head Loss by Straight Pipe (m)	2.087	5.722	17.483
V : Average Flow Speed (m/s)	0.263	0.41	0.672
L : Length of Pipe (m)	622	622	622
D : Inner Diameter (m)	0.04	0.032	0.025
g : Gravity (9.8m/s2)	9.8	9.8	9.8
Q : Flow Volume (m3/s)	0.00033	0.00033	0.00033
Q : Flow Volume (L/min)	20	20	20

Source: Study Team

In addition to the head loss of straight pipes, the vertical height from the water supply pipe position to the roof tank water surface is 7.2 m (Water supply pipe embedded depth - ground surface 0.8 m, ground surface - roof floor surface 5.4 m, roof floor surface - roof tank water surface 1.0 m) and head loss of 1.0 m due to allowance bending parts and equipment shall be considered.

With those conditions, the water head allowances at the roof top tank is calculated as shown in **Table7.6.12**. From this calculation result, diameter of the water supply pipe from the EFCOS site to the MCGS shall be 32mm.

Table7.6.12 Calculation of Water Head at Roof Top Tank

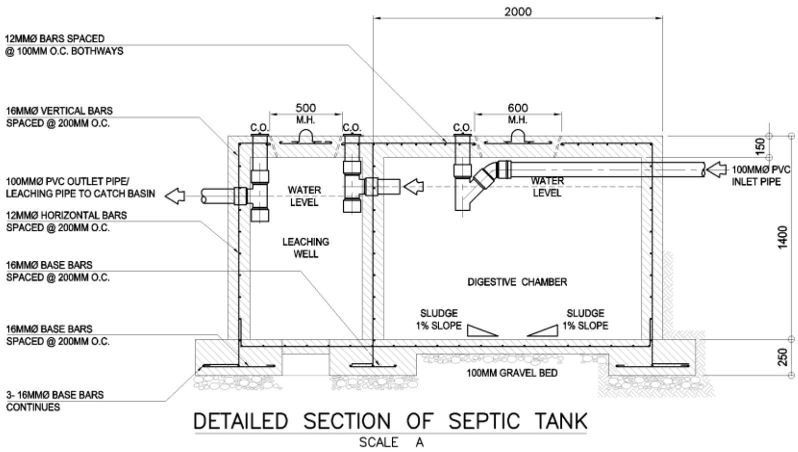
Dia.	40mm	32mm	25mm
H : Water Pressure (Water Head)	17	17	17
h1 : Head Loss by Straight Pipe (m)	2.087	5.722	17.483
h2 : Height of Tank from U.G.Pipe (m)	7.2	7.2	7.2
h3 : Head Loss by Equipment (m)	1	1	1
H' : Allowance (m)	6.713	3.078	-8.683
Required Water Head (m)	3.060	3.060	3.060
Result	OK	OK	NG

Source: Study Team

Roof top tank is good for the facility for emergency purpose where staff will stay in the event of a disaster because the stored water in the rooftop tank can be used even when water supply is cut. On the other hand, if the pressure of the public water supply became very low to reach the rooftop, direct piping can offer minimum water supply to the ground floor. Therefore, for MCGS generator house both roof top water tank and bypass pipe from waterline to faucets will be installed for redundancy.

(2) Sewage Treatment

MCGS is located distant from the public road, and the site of Cainta floodgate generator house is 2.3m lower than the front road, making them difficult to connect to the public sewerage system. For this reason, septic tanks shall be installed to treat wastewater from toiletry of generator houses, and the treated water can be discharged into the ground. Since each facility has only one toilet, the dimensions of the septic tank can be minimum specified in the standards. Typical detail of septic tank is shown below.



Source: Study Team

Figure7.6.11 Septic Tank Cross Section

7.6.2.2 Ventilation and Air Conditioning

(1) Policy for Installation of Ventilation and Air Conditioning Equipment

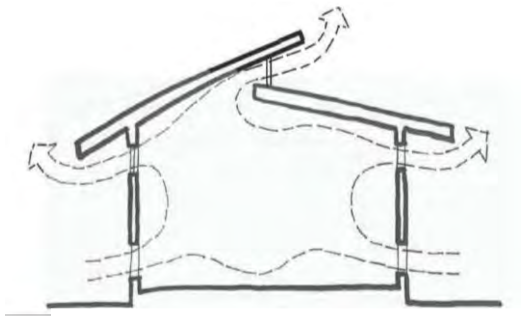
The installation place and policy of ventilation and air conditioning in this project building are as follows.

Table7.6.13 Installation Policy of Ventilation and Air Conditioning Equipment in Each Facility

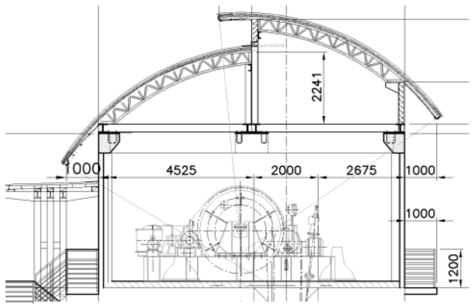
Facilities		Room name	Design Policy for Ventilation and Air conditioning
MCGS	Local Control House	-	Same as existing flood gates in the Philippines, No mechanical ventilation installed, while passive ventilation is considered in architectural design.
	Generator House	staff room	Air conditioners will 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.)
Cainta flood gate	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 flood gate	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

Although there is no mechanical ventilation system in the local control houses, they are desirable to be designed to maximize natural ventilation effect. With reference to the roof shape considering tropical climate shown in DGGS Volume 6 -- 3.10.1.16 "Passive Ventilation", louvers and sloped roofs are considered in design.



Concept of passive ventilation in DGGS Volume 6



Typical cross-section of local control house in this project

Source: Study Team

Figure7.6.12 Roof Shape to Promote Natural Ventilation

(2) Ventilation Equipment

Ventilation capacity calculations are based on the National Mechanical Code of the Philippines and DGCS Vol. 6- "Public Buildings and Other Related Structures".

RAF= $\frac{V*ACH}{60\ (min/hr)}$

where:

RAF: Required Air Flow Required Air Flow (m3/hour)

ACH: Air Change Rate required air change rate

V: Room area (m3)

The required ventilation rate is 6 times per hour for the generator room, and 4 times per hour for the staff room.

Table7.6.14 Capacity and Number of Fan

Location	Capacity and Number of Fan	Remarks
MCGS Generator Room	500(cfm) x 3	
MCGS Staff Room	500(cfm) x 1	
Cainta Generator Room	500(cfm) x 2	
Cainta Staff Room	500(cfm) x 1	
Taytay Local Control Room	500(cfm) x 2	

Source: Study Team

The ventilation volume calculations for each room are shown below.

MCGS: GENERATOR HOUSE VENTILATION

1. Generator Room for control system

Floor area 11.78 m2
Volume : 58.90 m2

RAF gate housing = 58.90 x (ACH = 6) x 100%
= 353.4 m3/h
= say 400 m3/h < 780 m3/h (Selected Model)

2. Generator Room for gate housing

Floor area 46.48 m2
Volume : 232.40 m2

RAF gate housing = 232.40 x (ACH = 6) x 100%
= 1394.4 m3/h
= say 1400 m3/h < 1560 m3/h (Selected Model)
(780 x 2)

3. Staff house

Floor area 22.18 m2
Volume : 110.90 m2

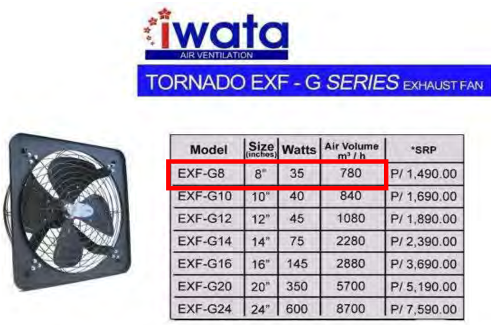
RAF gate housing = 110.90 x (ACH = 6) x 100%
= 665.4 m3/h
= say 700 m3/h < 780 m3/h (Selected Model)

From the computation of the required air ventilation, select from brochures the needed ventilation fan fit for aforementioned rooms. Select the fan with the higher rating higher than what was computed.

SAMPLE OF APPLIED PRODUCT :

Air Volume : 780 cu.m./hour
Input power : 35 watts
Voltage : 220 V
Phase : single
Hertz : 60
Number of units in Total : 4

*Air Volume in CFM :
780 cu.m./hour
= 459.42 cfm
= say 500 cfm



Model	Size (inches)	Watts	Air Volume m^3 / h	*SRP
EXF-G8	8"	35	780	P/ 1,490.00
EXF-G10	10"	40	840	P/ 1,690.00
EXF-G12	12"	45	1080	P/ 1,890.00
EXF-G14	14"	75	2280	P/ 2,390.00
EXF-G16	16"	145	2880	P/ 3,690.00
EXF-G20	20"	350	5700	P/ 5,190.00
EXF-G24	24"	600	8700	P/ 7,590.00

Source: Study Team

CAINTA: GENERATOR HOUSE VENTILATION

1. Generator Room

Floor area	31.29	m2
Volume :	156.45	m2

RAF gate housing =	156.45	x (ACH = 6) x 100%			
=	938.7	m3/h			
= say	1000	m3/h	<	1560 m3/h (780 x 2)	(Selected Model)

2. Staff house

Floor area	17.77	m2
Volume :	88.85	m2

RAF gate housing =	88.85	x (ACH = 6) x 100%			
=	533.1	m3/h			
= say	600	m3/h	<	780 m3/h	(Selected Model)

From the computation of the required air ventilation, select from brochures the needed ventilation fan fit for aforementioned rooms. Select the fan with the higher rating higher than what was computed.

SAMPLE OF APPLIED PRODUCT :

Air Volume : 780 cu.m./hour
Input power : 30 watts
Voltage : 220 V
Phase : single
Hertz : 60
Number of units in Total : 3

*Air Volume in CFM :
780 cu.m./hour
= 459.42 cfm
= say 500 cfm

Model	Size (inches)	Watts	Air Volume m³/h	*SRP
EXF-G8	8"	35	780	P/ 1,490.00
EXF-G10	10"	40	840	P/ 1,690.00
EXF-G12	12"	45	1080	P/ 1,890.00
EXF-G14	14"	75	2280	P/ 2,390.00
EXF-G16	16"	145	2880	P/ 3,690.00
EXF-G20	20"	350	5700	P/ 5,190.00
EXF-G24	24"	600	8700	P/ 7,590.00

TAYTAY: LOCAL CONTROL HOUSE VENTILATION

1. Local Control Room

Floor area	49.98	m2
Volume :	249.90	m2

RAF gate housing =	249.90	x (ACH = 6) x 100%			
=	1499.4	m3/h			
= say	1500	m3/h	<	1560 m3/h (780 x 2)	(Selected Model)

From the computation of the required air ventilation, select from brochures the needed ventilation fan fit for aforementioned rooms. Select the fan with the higher rating higher than what was computed.

SAMPLE OF APPLIED PRODUCT :

Air Volume : 780 cu.m./hour
Input power : 30 watts
Voltage : 220 V
Phase : single
Hertz : 60
Number of units in Total : 2

*Air Volume in CFM :
780 cu.m./hour
= 459.42 cfm
= say 500 cfm

Model	Size (inches)	Watts	Air Volume m³/h	*SRP
EXF-G8	8"	35	780	P/ 1,490.00
EXF-G10	10"	40	840	P/ 1,690.00
EXF-G12	12"	45	1080	P/ 1,890.00
EXF-G14	14"	75	2280	P/ 2,390.00
EXF-G16	16"	145	2880	P/ 3,690.00
EXF-G20	20"	350	5700	P/ 5,190.00
EXF-G24	24"	600	8700	P/ 7,590.00

Source: Study Team

(3) Air Conditioning Equipment

Since the area of the staff room is small and general-purpose air conditioning equipment for home use is installed, the recommended room area in the product specifications is referred to, and the capacity calculation is omitted. The selected air conditioners are used as follows.

Table7.6.15 Capacity and Number of Air Conditioner

Location	Capacity and Number of Fan	Remarks
MCGS Staff Room	1.5 (HP) x 1	
Cainta Staff Room	1.5 (HP) x 1	

Source: Study Team

The ventilation volume calculations for each room are shown below.

Qr = A*Lc*1.2

Where,

Lc : Unit Cooling Load : (kcal/m2・h)
Qn : Nominal Cooling Cap. (kcal/h)
Qr : Required Cooling Cap. (kcal/h) or (kW)

Note :

1. Unit Cooling Load : Meeting Room 140～170 kcal/m2・h
   Ref. : HASS 109, The Society of Heating, Air-Conditioning an Sanitary Engineering of Japan
   Here, 155(kcal/m2*h) will be applied as the median value of above mentioned range.

2.Margin Factor : 1.2

3.Conversion of Cooling Cap. from kcal/h to kW : 1 kW= 900 (kcal/h)

4.Conversion of Cooling Cap. to horse power : 1 hp = 2.8 (kW)

MCGS: GENERATOR HOUSE AIR-CONDITION

Name of Room	W (m)	D (m)	A (m2)	Qn : Nominal Cooling Cap. (kcal/m2*h)	Qr : Required Cooling Cap. (kcal/m2*h)	Qr : Required Cooling Cap. (kW)	**Horse Power (hp)**
staff room	3.65	4.60	16.79	2,602	3,123	3.5	**1.2**

CAINTA: GENERATOR HOUSE AIR-CONDITION

Name of Room	W (m)	D (m)	A (m2)	Qn : Nominal Cooling Cap. (kcal/m2*h)	Qr : Required Cooling Cap. (kcal/m2*h)	Qr : Required Cooling Cap. (kW)	**Horse Power (hp)**
staff room	3.45	5.15	17.7675	2,754	3,305	3.7	**1.3**

SAMPLE OF APPLIED PRODUCT :

Mx-FDC32-INV Matrix 1.5HP Full DC Split Type Aircondition
1.5HP Capacity
Power Supply: 220-240.50,1
Cooling Capacity: 12,000Btu
Cooling Input: 975w
Cooling Current: 4.2A

Source: Study Team

7.6.3 Building Electrical Equipment

7.6.3.1 Lightning Protection

Since the National Standards do not specify details of lightning arresters, design of lightning arresters in this project are based on NFC -17 -102 "Early streamer emission lightning protection systems". The protection radius of a lightning rod is given by following equation according to the height from the tip to the object to be protected.

$$R_p(h) = \sqrt{2rh - h^2 + \Delta(2r + \Delta)}$$

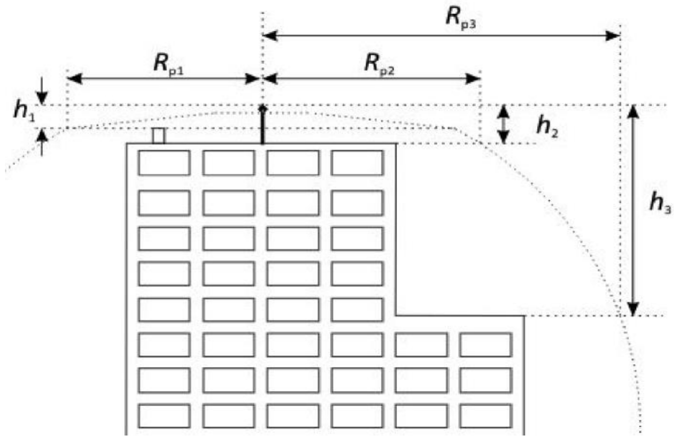
where

R_p (h) (m) is the protection radius at a given height h

h (m) is the height of the ESEAT tip over the horizontal plane through the furthest point of the object to be protected

r (m) 20 m for protection level I
30m for protection level II
45m for protection level III
60m for protection level IV

Δ(m) $\Delta = \Delta T \times 10^6$
Field experience has proved that Δ is equal to the efficiency obtained during the ESEAT evaluation tests



Source: NF C-17-102 "Early streamer emission lightning protection systems

Figure7.6.13 Protection Range of the Lightning Arrester

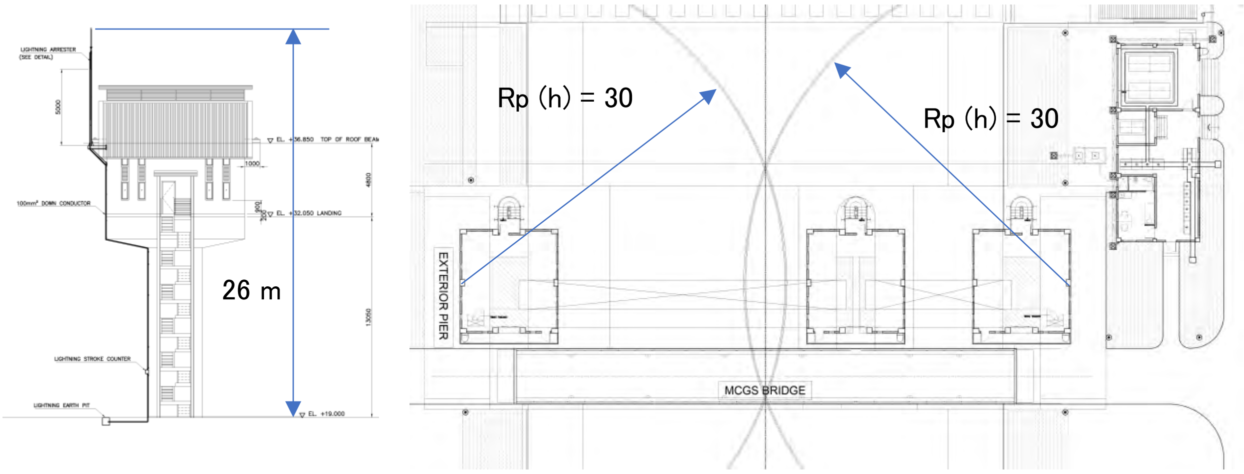
⊿ T can be determined by the performance test, however, in this design 10 µs is applied considering safe side since it is the minimum value within the allowed range. The protection level is Level I, so that r shall be 20 (m).

C.2.2 Requirements for early streamer emission

The early streamer emission of the ESEAT (ΔT) shall be determined according to the procedures of clause C.3.5.
It shall range between 10 µs and 60 µs.
If the result of ΔT is lower than 10 µs, then the air terminal will not be considered as an ESEAT.

Source: NF C-17-102 "Early streamer emission lightning protection systems

Difference of height (h) is 26m as shown in **Figure 7.6.14**. The protection radius for MCGS is calculated as 30m. In order to cover the three local control houses and generator house, lightning arrester shall be installed at control housed at both side piers.

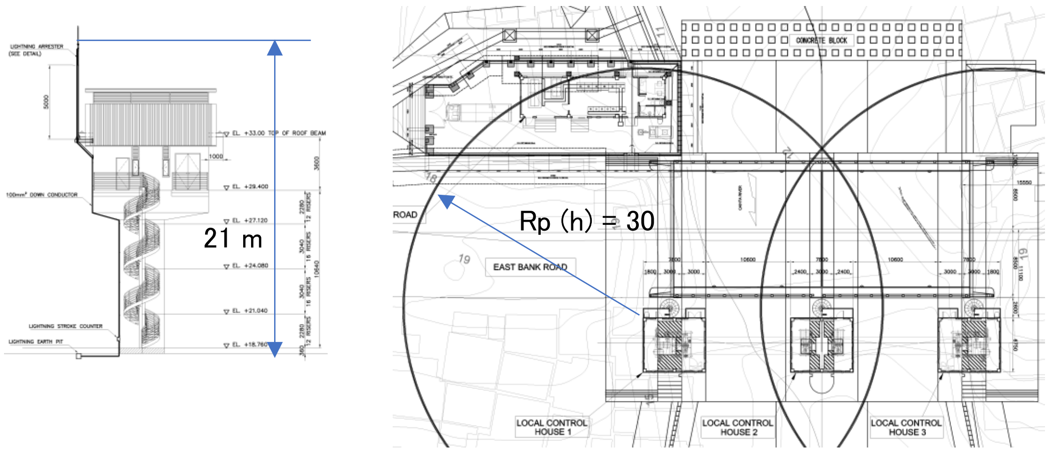


Height of top of lightning rod from GL is 26 m and the height of generator house is 6m so that 26m -6 m = 20 m shall be applied for “h”, as height difference of rod tip to the furthest object to be protected. From the calculation formula on the previous page, Rp (h) = 30 m.

Source: Study Team

Figure 7.6.14 Protection Radius of Lightning Arrester (MCGS)

The protective radius at the Cainta floodgate is 30m too as shown in **Figure7.6.15**. In order to cover the three local control houses and generator house, lightning arrester shall be installed at control housed at both side piers.

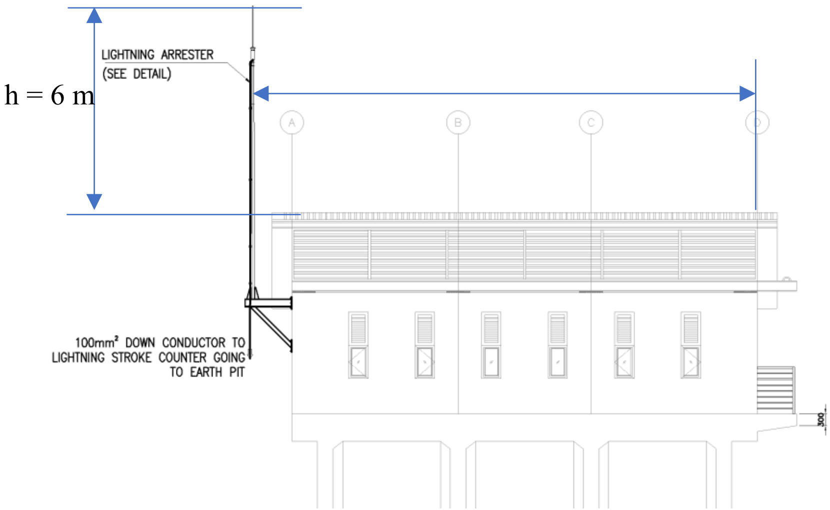


Height of top of lightning rod from GL is 21 m and the height of generator house is 6m, plus site elevation of generator house is -2.3m from the front road, so that 21m -6m + 2.3m = 17.3m shall be applied for “h”, as height difference of rod tip to the furthest object to be protected. From the calculation formula on the previous page, Rp (h) = 30 m.

Source: Study Team

Figure7.6.15 Protection Radius of Lightning Arrester (Cainta)

The protective radius at Taytay floodgate is 26m too as shown in **Figure7.6.15**. The whole area of local control house is within the protection radius.



h is 6m, then Rp (h) is calculated as 26m.

Source: Study Team

Figure7.6.16 Protection Radius of Lightning Arrester (Taytay)

7.6.3.2 Lighting Equipment

(1) Lighting of Generator House

According to DGCS Vol. 6 – "Public Buildings and Other Related Structures", design illuminance level of generator house can be 200 ~ 300 (lux) since it is regarded as "Interior (infrequent reading and writing)".

Table7.6.16 Recommended Illuminance by Room Type

Task	Minimum and Maximum (Lux)	Application
Lighting for Infrequently Used	50-150	Circulation area and Corridors
	100-200	Stairs
	100-200	Escalators
Lighting for Working Interiors	200-300	Infrequent reading and writing
	300-750	General offices, typing and computing
	300-750	Conference room
	500-1000	Deep-plan general offices
	500-1000	Drawing offices
Localized lighting for Exacting tasks	500-1000	Proofreading
	750-1500	Designing, architecture and machine engineering
	1000-2000	Detailed and precise work

Source: Guide ines on Energy Conserving Design of Bui dings (2007 Ed)

Source: DGCS Vol. 6 - Public Buildings and Other Related Structures

LED type (1 x 35 W) will be the lighting fixture for generator house, which is easy to maintain and has a long service life. **Table7.6.17** shows efficacy ranges of various lamps. 88 (lumen/watt) can be applied for LED as intermediate of 80 -95 (lumen/watt), thus the luminous flux per 1 unit is 35 × 88 = 3080 ⇒ 3000 (lumen).

Table7.6.17 Luminous flux by lighting type

Table 5-6 Efficacy Ranges of Various Lamps

Lamp Type	Rated Power Ranges (watts)	Efficacy Ranges (lumens/watt)
Linear/Tubular Fluorescent Lamp		
Halophosphate	10 – 40	55 – 70
Triphosphor	14 – 65	60 – 83
Compact Fluorescent Lamp (CFL)	3 – 125	41 – 65
Light Emitting Diode (LED)	3 – 100	80 – 95
ncandescent Lamp	10 – 100	10 – 25
Mercury Vapor Lamp	50 – 2000	40 – 63
Metal Halide lamp	Up to 1000	75 – 95
Low Pressure Sodium Lamp	20 – 200	100 – 180
High Pressure Sodium Lamp	50 – 250	80 – 130

Source: Guide ines on Energy Conserving Design of Bui dings (2007 Ed)

1 X 35W T16 LAMP(S)
IP65 DUST AND MOISTURE
RESISTANT LIGHTING FIXTURE

Source: DGCS Vol. 6 - Public Buildings and Other Related Structures

The required number of lamps is given by the following formula using the luminous flux method.

$$N = E * \frac{A}{F} * U * M$$

where:

N: Required number of lamps
E: Design illuminance, 200 (lux).
A: Room area (m2)
F: Flux of luminous, 3000 (lumen).
U: Utility factor, 0.5.
M: Maintenance factor, 0.8 is applied for M considering decrease of flux after 10 years due to is 20% for LED.

The calculation results in for MCGS generator house are shown below. Layout of lighting fixtures shall be based on those numbers.

Table7.6.18 Recommended Number of Lighting Fixtures in Generator House

	Stuff Room	Generator Room	Electrical Room
	スタッフルーム	発電機室	電気室
E	200	200	200
A	12.19	46.47	58.22
F	3000	3000	3000
U	0.5	0.5	0.5
M	0.8	0.8	0.8
N	2	8	10

Source: Study Team

(2) Lighting for Local Control House

Although there are no lightings in local control houses of existing Rosario weir, it is better to have consider installation of lightings in this project for emergency operation and maintenance at night. Since the steel roof of local control house is designed to be detachable, the lighting fixtures are supported by the side of the RC beam without being suspended from the roof.

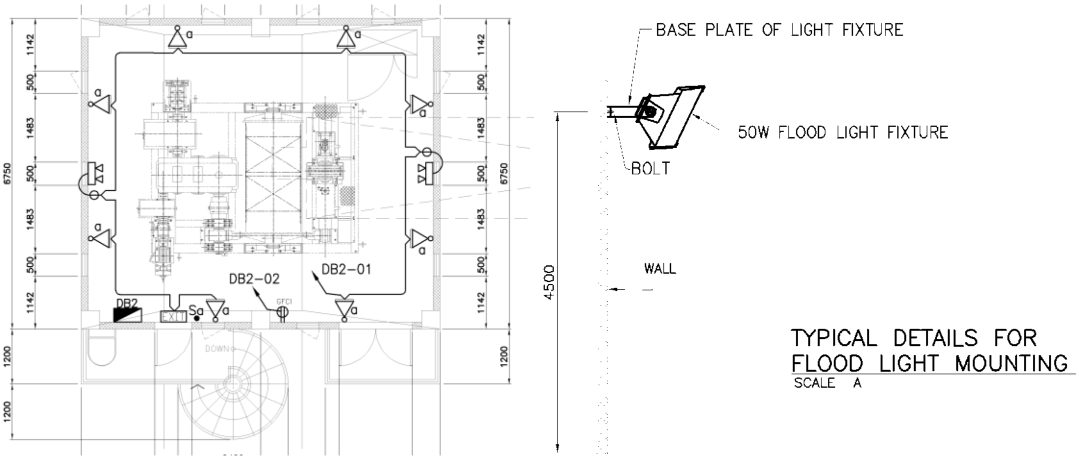


Figure7.6.17 Example of Lighting Fixtures for Local Control House

7.6.4 Other Conditions

7.6.4.1 Design Conditions for Stairs

Section 1207 of NBCP specifies the structural details of stairs. Width of the stair steps shall be not less than 750 mm width as buildings in this project can be classified as "Private stairs serving less than 10 persons".

The height of riser shall be 200 mm or less, and depth of tread shall be 250 mm or more. With regards to spiral staircases, the tread length shall be 150 mm or more on the center side and 300 mm or more on the peripheral side.

5. *Stairways.* Except stairs or ladders used only to access equipment, every stairway serving any building or portion thereof shall conform to the following requirements:

   a. *Width.* Stairways serving an occupant load of more than fifty (50) shall not be less than 1.10 meters. Stairways serving an occupant load of fifty (50) or less may be 900 millimeters wide. Private stairways serving an occupant load of less than ten (10) may be 750 millimeters. Trim and handrails shall not reduce the required width by more than 100 millimeters.

   b. *Rise and Run.* The rise of every step in a stairway shall not exceed 200 millimeters and the run shall not be less than 250 millimeters. The maximum variations in the height of risers and the width of treads in any one flight shall be 5 millimeters: *Except,* in case of private stairways serving an occupant load of less than ten (10), the rise may be 200 millimeters and the run may be 250 millimeters, except as provided in sub-paragraph (c) below.

   c. *Winding Stairways.* In Group A Occupancy and in private stairways in Group B Occupancies, winders may be used if the required width of run is provided at a point not more than 300 millimeters from the side of the stairway where the treads are narrower but in no case shall any width of run be less than 150 millimeters at any point.

Source: IRR of NBCP

7.6.4.2 Restriction for Rooms Handling Flammable Liquids

(1) Fire Resistance Measure

Section 10.3.4.2.1 of Fire Codes of the Philippines specifies the conditions required for buildings handling hazardous materials including diesel fuel. MCGS generator house and the Cainta generator house shall be designed to conform the conditions specified in the codes.

Maximum floor area is depending on amount of fuels, sprinklers system and fire resistance rate as shown below.

vi. Inside storage and handling room shall comply with approved, supervised sprinkler system as shown in **Table 19** below:

Table 19: SPRINKLER SYSTEMS FOR INSIDE STORAGE AND HANDLING ROOMS OF FLAMMABLE LIQUIDS

SPRINKLER SYSTEM PROVIDED	FIRE RESISTANCE RATING	MAXIMUM SIZE (Floor Area)	TOTAL LITERS ALLOWED
Yes	2 hours	46.5 sq. m	18, 925
No	2 hours	46.5 sq. m	7, 570
Yes	1 hour	13.9 sq. m	3,785
No	1 hour	13.9 sq. m	1,893

Source: IRR of FC

Since it would be difficult to accommodate generators with room area of 13.9 m2, fire resistance rate shall be 2 house to secure 46.5 m2 for maximum floor area. Sprinklers are not necessary since 7,570L is enough for generators.

(2) Oil Leakage Prevention

Opening of generator room will be designed with 10cm raise from the floor level to prevent oil discharge to other rooms in case of the spillage from generator.

i. Opening to other rooms or buildings shall be provided with noncombustible liquid-tight raised sills or ramps at least ten centimeters (10 cm) in height or the floor in the room shall be at least ten centimeters (10 cm) below the surrounding floors. . A permissible alternate to the sill or ramp is an open-grated trench inside the room. A downgraded flooring shall be provided for spillage which drains to a safe location or an open-grated trench.

Source: IRR of FC

It is also stipulated that the separation of the Any burning equipment including generators shall be distant at least 0.9 m from the electrical equipment including control panels and at least 1.5 m from the unsealed fuel tank.

1. Oil-burning appliances and equipment shall be installed so that a minimum of nine tenths meter (0.9 m) separation is maintained from any electrical panel board and a minimum of one and a half meters (1.5 m) separation is maintained from any unenclosed fuel oil tank.

Source: IRR of FC

7.6.5 Consideration of Architectural design

7.6.5.1 Example of Flood Gate Design in the Philippines

Examples of large floodgates and weirs similar to MCGS include Rosario weir located at the upper end of Manggahan floodway and Kamanava floodgate in Nabotas, Marabon district located in the northern suburb of the metropolitan area from storm surges.

	
Rosario Weir	Kamanava Flood Gate

Source: Study Team

Figure7.6.18 Existing Samples of Large Span Flood Gates in Metro Manila

The Taytay/Cainta floodgate will be constructed at the mouth of tributary to prevent back flow from Manggahan floodway. Similar backflow prevention gates are observed many in Metro Manila as shown in **Figure7.6.19**.

		
Gate of Tapayan Pumping Station	Gate of Napindan Pumping Station	Gate of Binondo Pumping Station
		
Gate of Escolta Pumping Station	Gate of San Francisco Pumping Station	Gate of Valenzuela Pumping Station

Source: Study Team

Figure7.6.19 Example of Existing Floodgates in Metro Manila

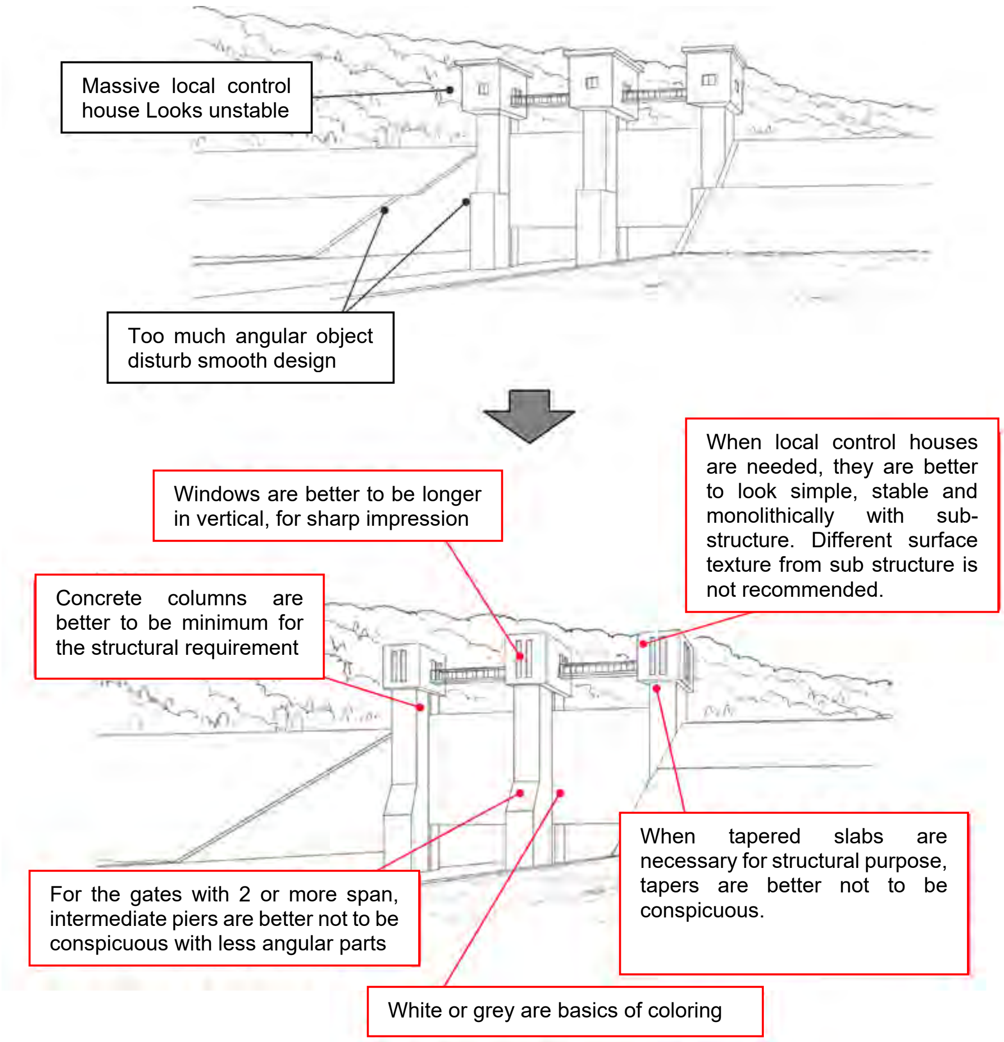
All of those local control houses are simple RC box with minimum windows are for ventilation. Those are often painted same as adjacent pumping stations or bare surface of concrete with no painting.

7.6.5.2 Design Policy in this Project

MCGS will be the first floodgate crossing over the Pasig / Marikina River. Therefore, while the function is the first priority, following factors shall be considered in the architectural design.

1. Continuity of river scape along upper and downer streams shall be considered, to not to give the impression of 'blocking' the flow of mother river.
2. Since ferry service will go through under the gates, the design shall not be too massive and intimidating.
3. The design should harmonize with the adjacent scape along the river as much as possible.

In order to prevent giving oppressive impression, a vertically long windows are adopted as one of the elements in the design of the local control houses, in refence to "Guidance for Landscape Consideration in the Restoration of River and Coastal Structures" (2011: Water Management and National Land Conservation Bureau, Ministry of Land, Infrastructure and Transport) as shown below.



Source: Guidance for Landscape Consideration in the Restoration of River and Coastal Structures

Figure7.6.20 Example of Flood Gate Design in Japan

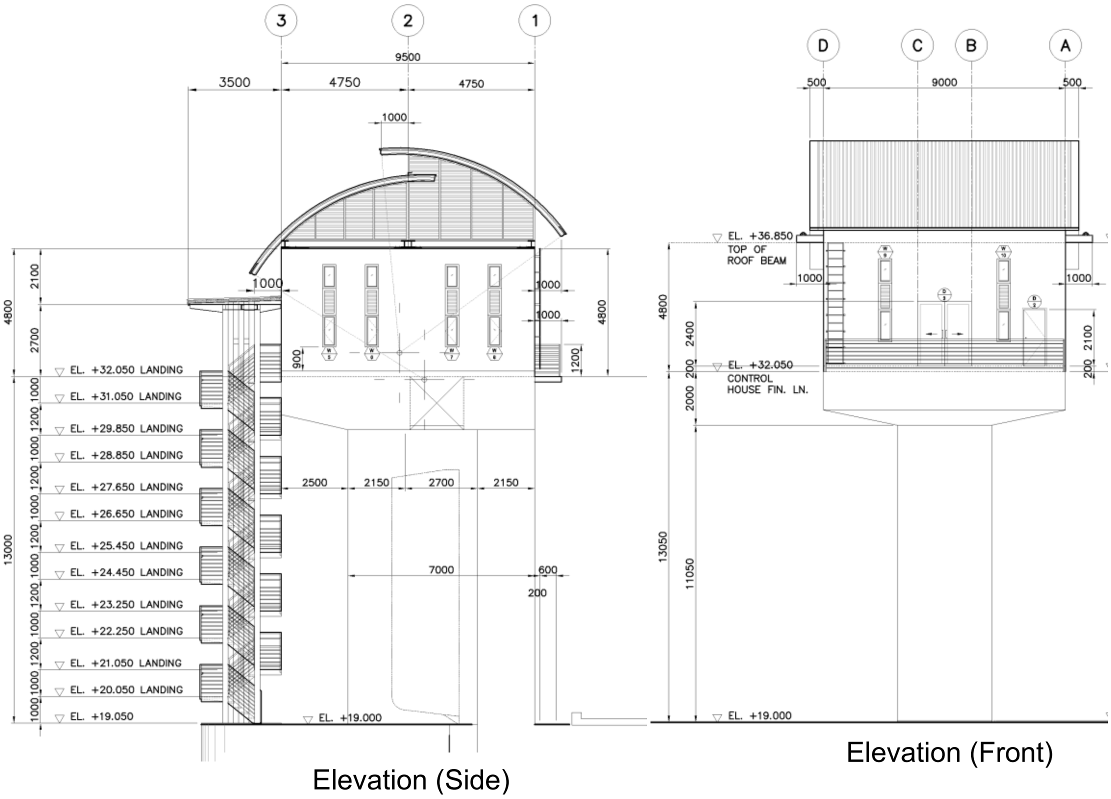
Ferry stations operated by MMDA can be seen continuously through the upper and lower stream of the Pasig Marikina River. The common design elements of these ferry stations are arcuate roof with deep eaves, the bright colored wall surface with of windows reaching roof etc. One idea for providing continuity to the riverside landscape is to borrow those design elements from ferry stations.

		
Valenzuela Ferry Station	Guadalupe Ferry Station	San Joaquin Ferry Station

Source: Study Team

Figure7.6.21 Design Example of Ferry Terminal

Based on the above factors, the design of the MCGS local control house is shown below. The columns, beams and walls of RC and CHB will be finished with bright grey plaster to look similar surface with sub structure concrete, and the steel roof structure will be painted white. The same design shall be applied to the roofs of the Cainta and Taytay flood gates.



Source: Study Team

Figure7.6.22 MCGS Local Control Design