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

FINAL REPORT

VOLUME-1B
MAIN REPORT

AUGUST 2020

JAPAN INTERNATIONAL COOPERATION AGENCY

CTI ENGINEERING INTERNATIONAL CO., LTD.
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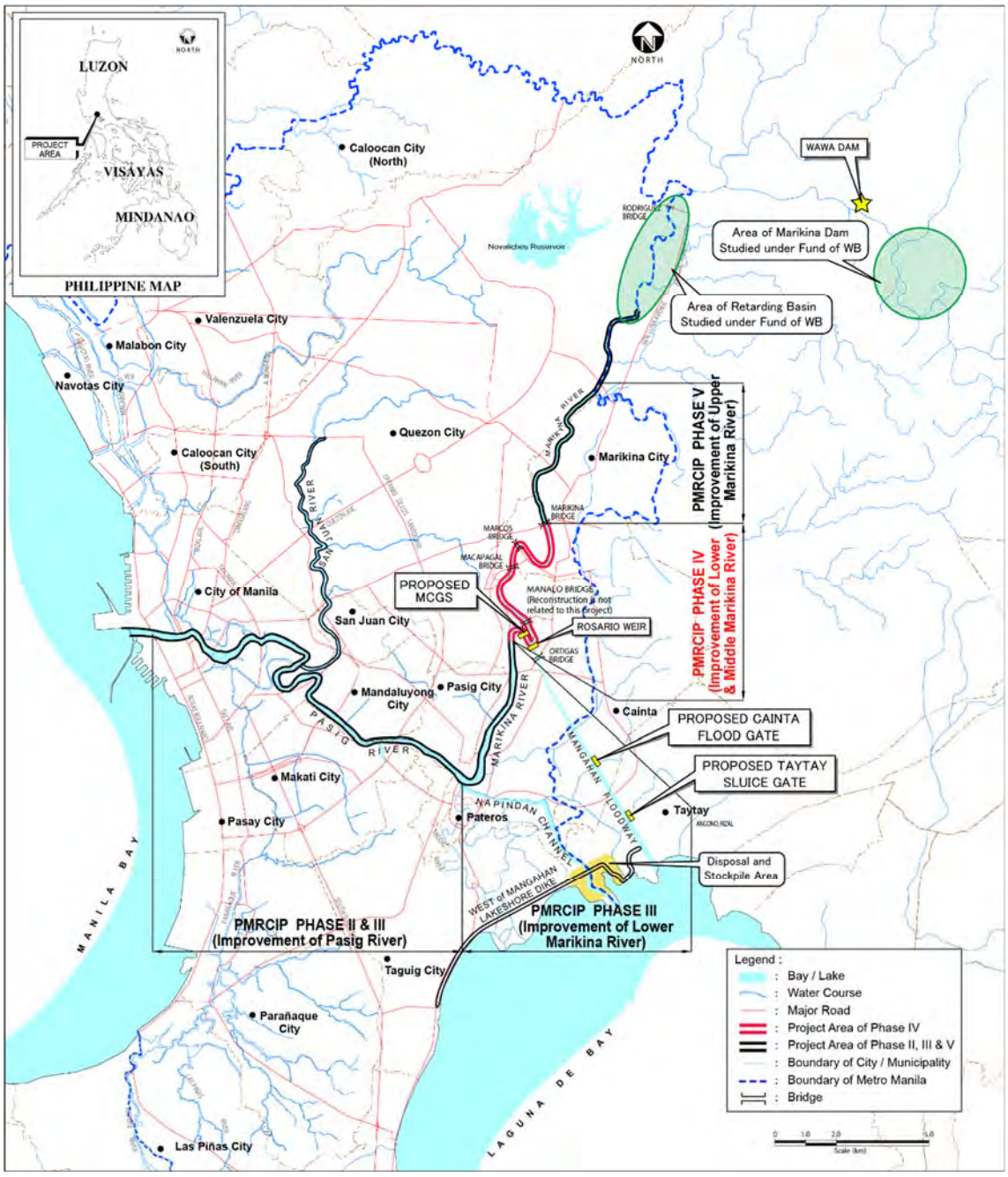
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COMPOSITION OF FINAL REPORT

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 14)**
VOLUME-2	**:**	**APPENDIX**
VOLUME-3	**:**	**APPENDIX FOR GEOTECHNICAL INVESTIGATION**
VOLUME-4A	**:**	**STRUCTURAL CALCULATION FOR CONTRACT PACKAGE-1**
VOLUME-4B	**:**	**STRUCTURAL CALCULATION FOR CONTRACT PACKAGE-2**
VOLUME-4C	**:**	**STRUCTURAL CALCULATION FOR CONTRACT PACKAGE-3**
VOLUME-5A	**:**	**QUANTITY CALCULATION FOR CONTRACT PACKAGE-1**
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VOLUME-6	**:**	**COST ESTIMATE**

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PROJECT LOCATION MAP

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

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ABBREVIATIONS AND ACRONYMS

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

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

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

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

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

Units of Measurement

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

CHAPTER 7 DETAILED DESIGN OF RIVER STRUCTURES

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

7.1.1 Outline of Detailed Design of River Channel

In this design, the centerline of the river was basically that of the existing river, but the position of the to the left of the right bank was adjusted based on the land-use situation along the river and the results of discussions with local governments. The centerline is created straight lines and circular arcs.

The structures consist of the followings.

- Low water channel, which is a normal channel
- High water channel and dike which is a channel in flood
- Parapet wall for the freeboard.

Low water channel constructed a revetment with self-supporting Steel Sheet Pile (SSP), and the centerline of the SSP revetment was parallel to the centerline of the river. In order to ensure the stability of revetment based on the structural calculation, riprap was installed in the front of the SSP as necessary. In the dike section, a concrete retaining wall revetment with a slope of 1: 0.5 is used for the embankment on the riverside, and a block retaining wall with a slope of 1: 0.5 is used for the embankment on the landside, due to the restriction of landform and site acquisition. In some sections, special dike with inverted T-shaped concrete retaining wall was adopted. The parapet wall at a higher place than DFL is a concrete wall.

The shape, dimensions, and reinforcement of each structure were determined based on the results of stability and structural calculations.

Hereinafter, the structure calculation result will be described in detail.

7.1.2 Detailed Design of SSP Revetment for Low Water Channel

7.1.2.1 Design Section

The design section for self-supporting SSP on the Marikina River was comprehensively determined through field surveys, project costs and consultations with relevant organizations such as DPWH. The sections and segmentation for low water channel are shown in **Table 7.1.1**.

Table 7.1.1 Sections and Segmentation for SSP Revetment on Marikina River

Left Bank

Block	Sta			Length (m)
L-1	6+035.3	-	6+080	42.80
L-2	6+080	-	6+362.8	266.29
L-3	6+753	-	6+800	61.44
L-4	6+800	-	7+180	409.61
L-5	7+180	-	7+480	302.83
L-6	7+480	-	7+820	359.77
L-7	7+820	-	7+940	120.00
L-8	7+940	-	8+120	180.00
L-9	8+120	-	8+300	180.11
L-10	8+300	-	8+600	293.86
L-11	8+600	-	8+835	211.24
L-12	9+205	-	9+320	107.58
L-13	9+320	-	9+560	230.49
L-14	9+560	-	9+800	267.21
L-15	9+800	-	9+900	100.98
L-16	9+900	-	10+020	120.99
L-17	10+020	-	10+360	332.08
L-18	10+360	-	10+520	160.22
L-19	10+520	-	10+580	60.10
L-20	10+580	-	10+640	60.00
L-21	10+640	-	10+760	86.91
L-22	10+760	-	11+040	230.40
L-23	11+040	-	11+180	140.19
L-24	11+180	-	11+460	282.50
L-25	11+460	-	11+640	182.49
L-26	11+640	-	11+800	164.63
L-27	11+800	-	12+040	266.26
L-28	12+040	-	12+280	260.87
L-29	12+280	-	12+520	249.09
L-30	12+520	-	12+820	304.65
L-31	12+820	-	13+000	179.78
L-32	13+000	-	13+320	322.81
L-33	13+320	-	13+360	31.22

Source: Study Team

Right Bank

Block	Sta			Length (m)
R-1	5+423	-	5+540	120.61
R-2	5+540	-	5+581.25	49.58
R-3	5+624	-	5+720	125.58
R-4	5+720	-	5+905.80	201.47
R-5	6+035.3	-	6+080	44.51
R-6	6+080	-	6+280	211.53
R-7	6+280	-	6+420	140.28
R-8	6+420	-	6+920	437.46
R-9	6+920	-	7+220	295.15
R-10	7+220	-	7+620	387.86
R-11	7+620	-	7+900	272.03
R-12	7+900	-	8+240	340.06
R-13	8+240	-	8+500	260.19
R-14	8+500	-	8+620	124.41
R-15	8+620	-	8+940	340.55
R-16	8+940	-	9+000	65.45
R-17	9+000	-	9+200	217.04
R-18	9+200	-	9+380	196.62
R-19	9+380	-	9+700	300.75
R-20	9+700	-	9+900	182.87
R-21	9+900	-	10+380	493.71
R-22	10+380	-	10+520	140.72
R-23	10+520	-	10+540	20.67
R-24	10+540	-	10+660	122.88
R-25	10+660	-	10+760	125.77
R-26	10+760	-	10+820	79.86
R-27	10+820	-	10+980	205.01
R-28	10+980	-	11+200	220.61
R-29	11+200	-	11+360	160.68
R-30	11+360	-	11+700	328.31
R-31	11+700	-	11+980	214.88
R-32	11+980	-	12+000	16.30
R-33	12+000	-	12+240	198.95
R-34	12+240	-	12+520	265.59
R-35	12+520	-	12+540	18.97
R-36	12+540	-	12+660	116.78
R-37	12+660	-	12+740	80.13
R-38	12+740	-	12+980	240.44
R-39	12+980	-	13+100	120.17
R-40	13+100	-	13+220	118.37
R-41	13+220	-	13+375	146.11

7.1.2.2 Design Criteria and Standard

SSP revetment design is based on the following criteria and standards.

Table 7.1.2 Design Criteria and Standard for SSP Revetment Design

No.	Name	supervision	Country
1	Design Guidelines of Disaster Recovery Works	Japan Disaster Prevention Association	Japan
2	SSP Revetment Design Guidelines	Kanto Regional Development Bureau Ministry of Land, Infrastructure, and Transport	Japan
3	SSP Revetment - From Design to Construction -	NIPPON STEEL & SUMITOMO METAL Co., Ltd.	Japan
4	SSP Revetment Q and A	Engineering Society of Steel Pipe Pile and Steel Sheet Pile	Japan
5	Cabinet Order Concerning Structural Standards for River Administration Facilities, etc.	Japan River Association	Japan
6	Technical Criteria for River Works: Practical Guide for Planning [I]	Japan River Association under the supervision of the River Bureau, Ministry of Construction	Japan
7	Road Earthwork Guideline	Japan Road Association	Japan
8	Specifications for Highway Bridges IV Substructures	Japan Road Association	Japan
9	Pile Foundation Design Manual	Japan Road Association	Japan
10	Standard Specifications for Concrete Structures [Design]	Japan Society of Civil Engineers	Japan
11	Guideline of River Planning Study	Kanto Regional Development Bureau, Ministry of Land, Infrastructure, and Transport	Japan
12	Civil Engineering Structure Design Manual – Sluiceway -	Ministry of Land, Infrastructure, and Transport	Japan
13	National Structural Code of the Philippines VOL.II (NSCP)		Philippine

Source: Study Team

Details of the design methodology are described in Chapter 11, Design Criteria 11.3.4.1 SSP revetment.

7.1.2.3 Design Condition

The design conditions of SSP revetment are shown in the following table.

Table 7.1.3 Design Condition of SSP Revetment

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

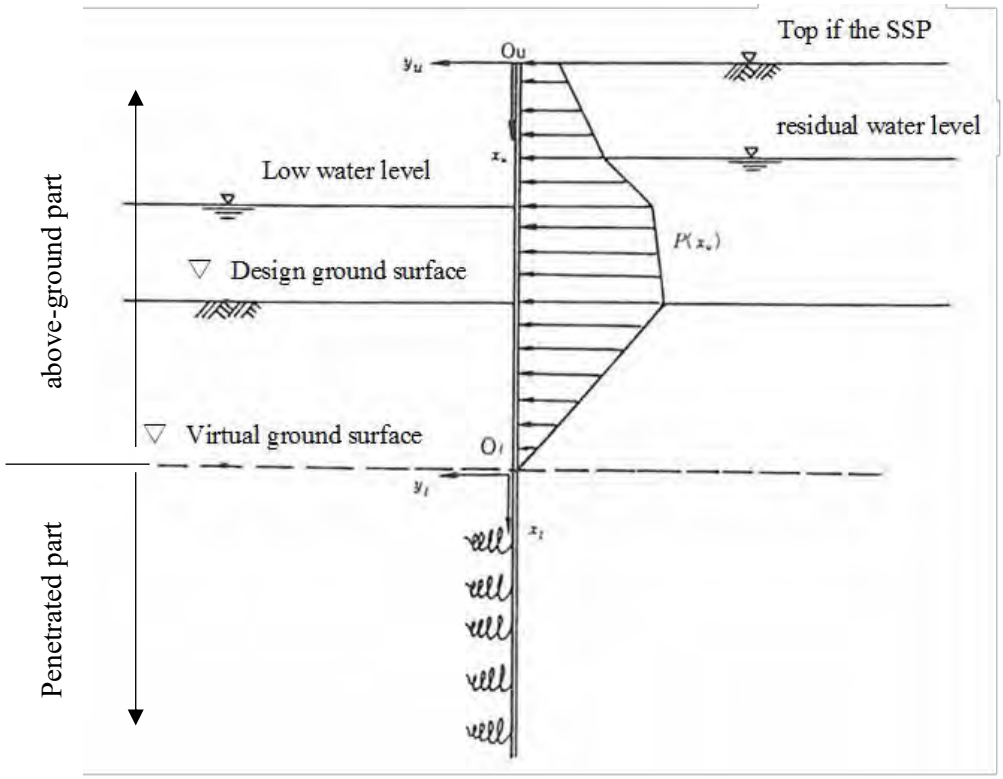
Source: Study Team

(1) Design Methodology

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

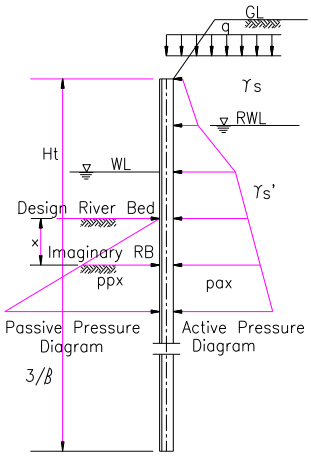
SSP revetment supports lateral load such as earth pressure by lateral support force of the intruded ground and flexural rigidity of the wall. The target external forces are lateral load (horizontal load) such as lateral soil pressure and water pressure. Vertical load will not be considered. Therefore,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

The Chang's equation will be applied for the structural calculation. In Chang's equation, when the SSP is divided into the upper part and the lower part with the virtual riverbed (A position where the sum of the active earth pressure strength and the residual hydraulic pressure equals the passive earth pressure strength) as the boundary as shown in **Figure 7.1.2**, it is assumed that the ground reaction force of the penetrated part is linear with respect to the displacement amount.



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

Figure 7.1.1 Virtual Ground Surface and Sheet Pile



Source: Study Team

Figure 7.1.2 Structure and Loads of a Sheet Pile

1) Calculation of Penetration Depth

The depth of sheet pile embedment from the riverbed, l_d, is determined using the following formula:

$l_d = x + 3/\beta$

Where,

X = distance from a design riverbed to a level where passive earth pressure becomes equal with the active earth pressure:

$\beta = (K_H D / 4Es I)^{0.25}$

K_H = coefficient of lateral subgrade reaction[1]

$$K_H = K_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

Where,

K_{H0} : the coefficient of horizontal subgrade reaction (kN / m^3) corresponds to the value of a flat plate loading test with a 0.3 m-diameter rigid disk. When it is estimated from deformation coefficients obtained by various soil tests or investigations, it is calculated by the following equation.

$$K_{H0} = \frac{1}{0.3}\alpha E_0$$

B_H : Converted load width of foundation orthogonal to the direction of loads action (m). For the case of a pile foundation, $\sqrt{D/\beta}$

D : load width of foundation orthogonal to the direction of loads action (m)

$1/\beta$: The depth (m) of the ground expected to have horizontal resistance. The depth shall be less than the effective penetration depth of the foundation.

β : Characteristic value of foundation $\sqrt[4]{\frac{K_H D}{4EI}}$ (m^{-1})

EI : Flexural rigidity of foundation (kN・m^2)

D : Width of SSP

Es : Yang's modules for SSP

I : Moment of inertia of area of SSP per unit width

2) Calculation of Maximum Moment

Formulae to analyze maximum moment and displacement of SSP (Chang's Formulae) are as follows:

$M_{MAX} = M \cdot \emptyset_m$

Where,

M : Bending moment of the imaginary riverbed

$$\emptyset_m = \frac{\sqrt{(1 + 2 \cdot \beta \cdot h_0)^2 + 1}}{2 \cdot \beta \cdot h_0} \cdot \exp\left(-tan^{-1}\frac{1}{1 + 2 \cdot \beta \cdot h_0}\right)$$

M_{MAX} : Maximum bending moment (kN・m) (t・m)

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4 \cdot E \cdot I}}\ (m^{-1})$$

[1] Specifications for Highway Bridges IV Substructures p285

K_h : Coefficient of lateral soil reaction (kN/m^3) (kg/cm^3)
B : Unit calculation width=1.0m = 100cm
E : Young Modulus = 2.0 ×10^8 (kN/m^2) = 2.1×10^6 (kg/cm^3)
I : Geometric moment of inertia = (m^4)
h_0 : Distance between imaginary riverbed and point of application force

3) Calculation of Deflection

$$\delta = \delta_1 + \delta_2 + \delta_3$$

Where,

δ : Deflection at top of a steel sheet pile (m) (cm)

δ_1 : Deflection on imaginary riverbed (m) (cm)

$$= \frac{(1 + \beta \cdot h_0) \cdot P}{2 \cdot E \cdot I \cdot \beta^3}$$

δ_2 : Deflection by incline (m) (cm)

$$= \frac{(1 + 2 + \beta \cdot h_0) \cdot P \cdot H}{2 \cdot E \cdot I \cdot \beta^2}$$

δ_3 : Deformation by bending of sheet pile (deflection as a cantilever) (m) (cm)

$$= \frac{B \cdot H^3}{6 \cdot E \cdot I} \cdot \sum (3 - \alpha_i) \cdot \alpha_i^2 \cdot P_i = \frac{B \cdot H^3}{E \cdot I} \cdot \sum q_i$$

H : Height of steel sheet pile from imaginary riverbed to top (m)
P : Total lateral force (kN) (t)
α_i : ratio of force point to total height from imaginary riverbed
q_i : Deformation coefficient

(2) Load

In the design of SSP revetment, the following load is considered.

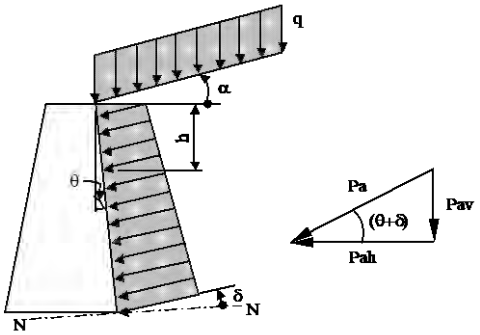
- Earth Pressure
- Hydraulic Pressure
- Seismic Load
- Surcharge Load
- Load from Inclined Wall

1) Earth Pressure

Earth pressure acting on movable walls is calculated by the following Coulomb's coefficient.

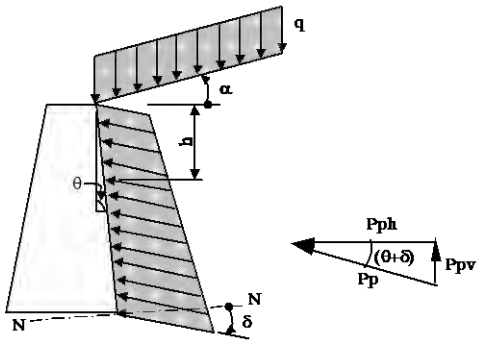
(a) Earth Pressure under Ordinary Condition

The following figures show the earth pressure acting wall.



Source: Study Team

Figure 7.1.3 Active Earth Pressure



Source: Study Team

Figure 7.1.4 Passive Earth Pressure

(i) For Sandy Soil

Pa = Ka γ h + Ka q
Pp = Kp γ h + Kp q

(ii) For Clayey Soil

Pa = Ka γ h - 2c Ka + Ka q

$$P_a = K_a\gamma h - 2c\sqrt{K_a} + K_a q$$

$$K_a = \frac{cos^2(\varphi - \theta)}{cos^2\theta \cos(\theta + \delta)\left[1 + \sqrt{\frac{\sin(\theta + \delta)\sin(\varphi - \alpha)}{\cos(\theta + \delta)\cos(\theta - \alpha)}}\right]^2}$$

$$P_p = K_p\gamma h + 2c\sqrt{K_p} + K_p q$$

$$K_p = \frac{cos^2(\varphi + \theta)}{cos^2\theta \cos(\theta + \delta)\left[1 - \sqrt{\frac{\sin(\theta - \delta)\sin(\varphi + \alpha)}{\cos(\theta + \delta)\cos(\theta - \alpha)}}\right]^2}$$

Where,

P_a	: Active earth pressure (kN/m²)
P_p	: Passive earth pressure (kN/m²)
γ	: Unit weight of soil (kN/m³)
K_a	: Coefficient of active earth pressure
K_p	: Coefficient of passive earth pressure
h	: Earth depth to acting point of earth pressures (m)
c	: Soil cohesion (kN/m²)
q	: Surcharge load in normal condition (kN/m²)
φ/ϕ	: internal friction angle of soil (degree)
θ	: angle between back side surface of wall and vertical plane (degree)
α	: angle between ground surface and horizontal plane (degree)
δ	: angle of wall friction (degree)

(b) Earth Pressure under Seismic Condition

Lateral earth pressure due to earthquake is calculated by the Mononobe-Okabe formula based on the Coulomb's theory in consideration of seismic factor.

$$Pea = K_{ea}\ \gamma\ h - 2c\ \sqrt{K_{ea}} + K_{ea}\ q'$$

$$Kea = \frac{\cos^2(\phi - \theta_o - \theta)}{\cos\theta_o \cos^2\theta \cos(\theta + \theta_o + \delta)\left(1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha - \theta_o)}{\cos(\theta + \theta_o + \delta)\cos(\theta - \alpha)}}\right)^2}$$

$$Pep = Kep\ \gamma\ h - 2c\ \sqrt{K_{ep}} + Kep\ q'$$

$$Kep = \frac{\cos^2(\phi - \theta_o + \theta)}{\cos\theta_o \cos^2\theta \cos(\theta - \theta_o + \delta)\left(1 - \sqrt{\frac{\sin(\phi - \delta)\sin(\phi + \alpha - \theta_o)}{\cos(\theta - \theta_o + \delta)\cos(\theta - \alpha)}}\right)^2}$$

Where,

Pea	: Active earth pressure under Seismic Condition (kN/m²)
Pep	: Passive earth pressure under Seismic Condition (kN/m²)
γ	: Unit weight of soil (kN/m³)
h	: Earth depth to acting point of earth pressures (m)
c	: Soil cohesion (kN/m²)
ϕ/φ	: internal friction angle of soil (degree)
θ	: angle between back side surface of wall and vertical plane (degree)
α	: angle between ground surface and horizontal plane (degree)
δ	: angle of wall friction (degree)
Kea	: Coefficient of active earth pressure under Seismic Condition
Kep	: Coefficient of passive earth pressure under Seismic Condition
q'	: Surcharge load in Seismic Condition (kN/m²)
θo	: Angle expressed by formula shown below (degree)

$$\tan\theta_o = \frac{Kh}{1 - Kv}$$

Kv	: seismic coefficient in vertical direction
Kh	: seismic coefficient in horizontal direction

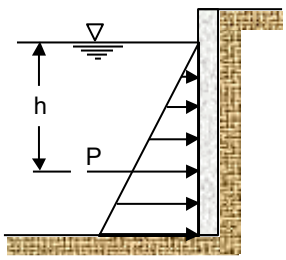
1) Hydraulic Pressure

Hydraulic pressure is hydrostatic pressure from the riverside daily water level, and a residual water pressure occurring from the landside after flood. A height of residual water is taken as same level of top of coping maximally in consideration of installation of drainage holes at the bottom of walls constructed on the coping.

(a) Hydrostatic Pressure

Hydrostatic pressure acting on the structure is calculated by the following formula:

$$P = \gamma_w h$$



Source: Study Team

Figure 7.1.5 Hydrostatic Pressure on Wall

Where,

P = hydrostatic pressure at "h" (kN/m²)
H = water depth (m)

γ_w = unit weight of water (9.8kN/m³)

(b) Dynamic Hydraulic Pressure due to Earthquake

Dynamic water pressure caused by earthquake acting on the wall structure facing on one side only is calculated by the following Westergaard's formula:

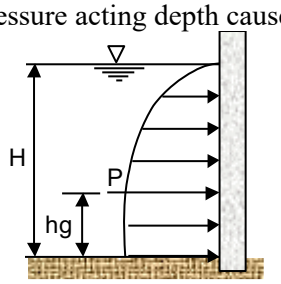
$$Pp = Kp\,\gamma\,h + Kp\,q$$

$$P = \frac{7}{12} Kh\,\gamma_w\,b\,H^2$$

$$hg = \frac{2}{5} H$$

Where,

P = dynamic water pressure caused by earthquake (kN)
Kh = coefficient of horizontal earthquake factor
γ_w = unit weight of water (kN/m³)
b = width of wall structure (m)
H = water depth (m)
h_g = dynamic water pressure acting depth caused by earthquake (m)



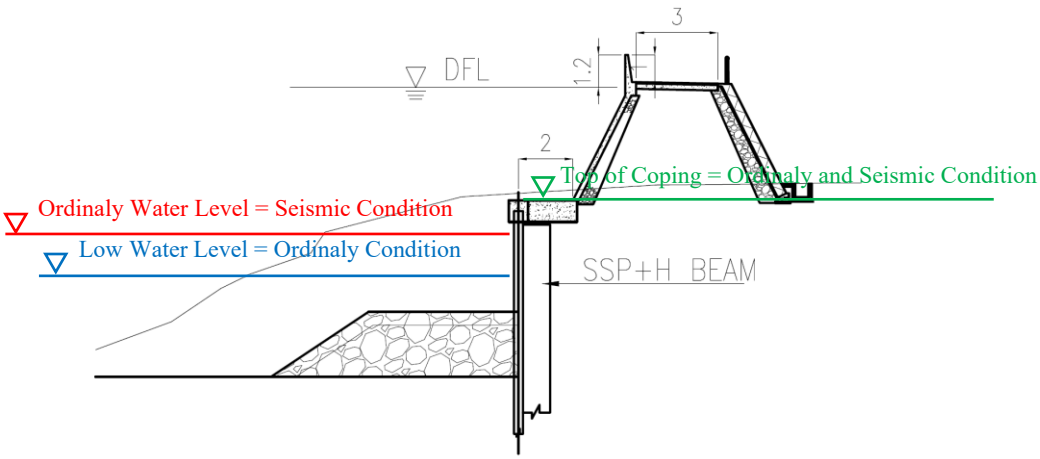
Source: Study Team

Figure 7.1.6 Dynamic Hydraulic Pressure on Wall

(c) Water Level and Residual Water Level

The water level on the front side shall be low water level at normal condition and normal water level at seismic condition.

The water level on the back side always considers the residual groundwater level. However, when a flood protection wall such as a returning wall is provided on the top of the coping, the residual groundwater level shall be the elevation of the top of the coping concrete in consideration of installing a drain hole at the bottom of the protection wall. The water level at seismic condition may be the same as the water level on the front side. However, considering the discontinuity of geological conditions, the water level shall be the same as at the top of coping. This concept is same as that of Phases 2 and 3.



Source: Study Team

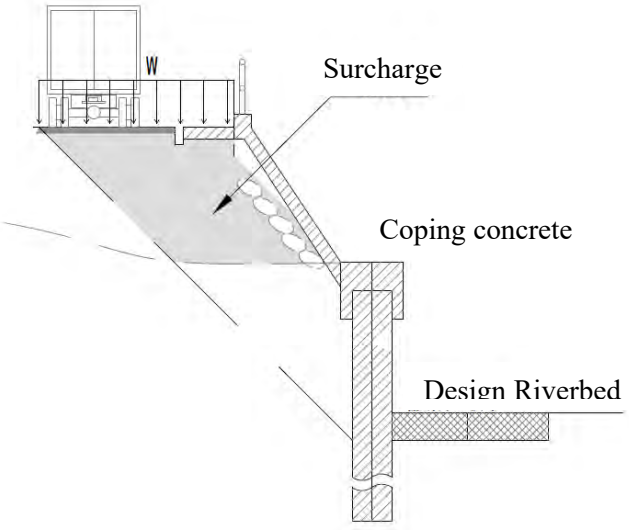
Figure 7.1.7 Water Level for SSP Revetment Calculation

2) Seismic Load

Seismic load is expressed as the increase in earth pressure. The earth pressure coefficient during earthquake is also computed using Mononobe-Okabe's formula revising Coulomb's formula, where horizontal seismic coefficient kh is set at 0.20

3) Surcharge Load

Surcharge is a load which acts on the land side above the SSPs. Surcharge load due to vehicle traffic shall be 10kN/m^2 in normal condition and 5kN/m^2 in seismic condition. For the area where pedestrians occupy, the surcharge load shall be 5kN/m2. Soil and other weight above the coping concrete shall be added to the surcharge load if ground height is higher than the coping concrete (**Figure 7.1.8**).

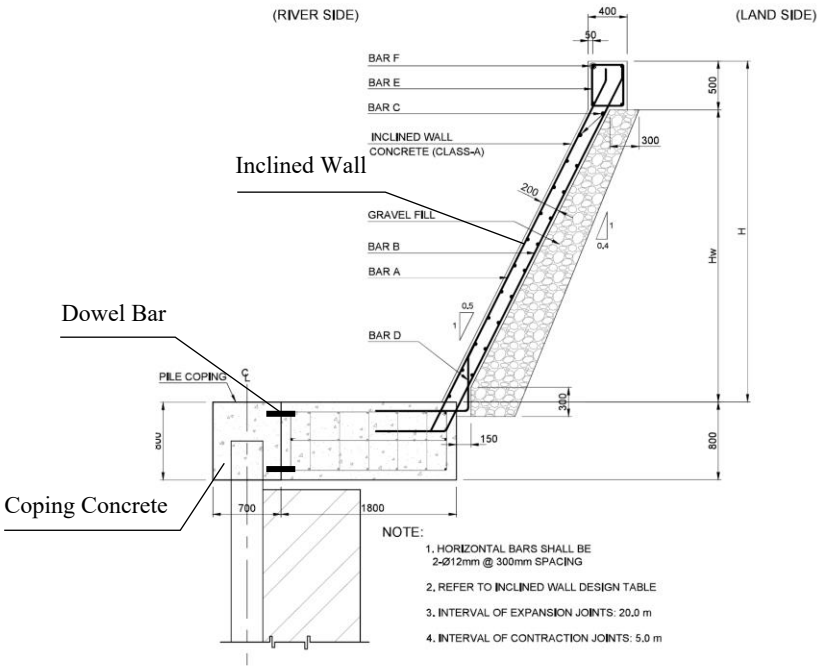


Source: Study Team

Figure 7.1.8 Upper Load Range Acting on an SSP

4) Load from Inclined Wall

In this design, inclined walls are installed in the revetment on the river channel side above SSP revetment. As shown in the figure below, at a position where the inclined wall and the SSP are close to each other, the toe portion of the inclined wall and the top coping of the revetment for low water channel are connected by dowel bars.



Source: Study Team

Figure 7.1.9 Connection Part between Inclined Wall and Coping Concrete of SSP Revetment

Since the inclined wall and the coping concrete of SSP are connected, if the stability of the inclined wall cannot be ensured, it is assumed that loads and moments will act on the coping concrete of SSP.

The stability of the inclined wall is usually evaluated by sliding, overturning, and bearing resistance. However, in this design, since the inclined wall is connected to the SSP revetment, and the vertical force is transmitted to SSP, bearing resistance isn't evaluated. The evaluation method for sliding and overturning and the method of calculating the load acting on the SSP are as follows.

(a) Sliding

Minimum safety factor against sliding should be as follows

$$SF = \frac{\text{Total Vertical forces x f}}{\text{Total Horizontal Force}}$$

SF ≥ 1.50 : in Normal Condition
SF ≥ 1.20 : in Seismic and Wind Condition

Where, f is as follows:

Concrete to clayey base : f = 0.5

If the inclined wall does not satisfy the minimum safety factor, the horizontal shortage is assumed to affect the coping (head) of revetment for low water channel, and the horizontal shortage calculated by the following formula is added to the design of SSP revetment.

$$SF = \frac{(\text{Total Vertical Forces of Inclined Wall}) \times f}{(\text{Total Horizontal Forces of Inclined Wall}) + (\text{Horizontal Shortage})}$$

Above formula is converted below,

$$(\text{Horizontal Shortage}) = \frac{(\text{Total Vertical Forces of Inclined Wall}) \times f}{(\text{Total Horizontal Forces of Inclined Wall}) \times SF}$$

Where,

SF = 1.50 : in Normal Condition
SF = 1.20 : in Seismic and Wind Condition

(b) Overturning

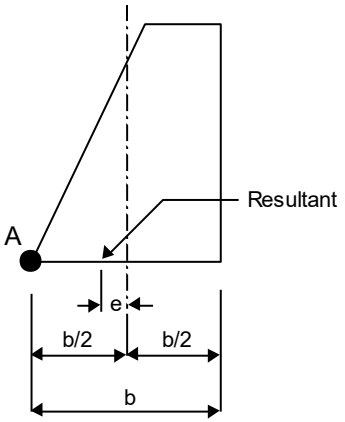
For stability of structures against overturning, the following conditions should be satisfied , :

$$e = \left|\frac{b}{2} - \frac{M}{N}\right| \le \frac{b}{6} \quad : \text{ in Normal Condition}$$

$$e = \left|\frac{b}{2} - \frac{M}{N}\right| \le \frac{b}{3} \quad : \text{ in Seismic, Wind and Flood Condition}$$

Where,

b	: Width of base (m)
M	: Total moment about point A (kN・m)
N	: Total vertical forces (kN)
e	: Eccentricity (m)



Source: Study Team

Figure 7.1.10 Point of Resultant Force

If the inclined wall does not satisfy the above formula, it is assumed that the moment acts on the coping of SSP revetment, and the moment shortage (M_L) calculated from the following formula is added to the design of revetment for low water channel.

$e = \left|\frac{b}{2} - \frac{M+M_L}{N}\right| = \frac{b}{6}$: in Normal Condition

Above formula is converted to $M_L = \left|\frac{b}{3}\right| \times N - M$

$e = \left|\frac{b}{2} - \frac{M+M_L}{N}\right| = \frac{b}{3}$ in Seismic Condition

Above formula is converted to $M_L = \left|\frac{b}{6}\right| \times N - M$

Where,

M_L : Moment Shortage （=Moment acting on SSP revetment）（kN・m）

The calculation results of the stability of the inclined wall and the moment and load acting on the SSP revetment are shown in the following page.

Table 7.1.4 Result of Stability Analysis of Inclined Wall and Force acting to SSP Revetment

Wall Height (m)	Condition	Over-Turning									Slide					
		Base Length (m)	Resistaince Moment at ToeΣMr (kN・m)	Over-Turning Moment at ToeΣMt (kN・m)	Vertical LoadΣV (kN)	Distance from toe to the working point of resultant force d (m)	Eccentric Distance (m)	Acceptable Eccentric Distance (m)	Judge※	Moment of Acting to SSP (KN*m)	Vertical Load ΣV (kN)	Horozontal Load ΣH (kN)	Safety Factor	Required Safety Factor	Judge※	Forth of Acting to SSP (KN)
3.0m	Steady	1.8	83.383	44.730	52.632	0.734	0.166	0.3	OK	0	52.632	31.214	0.843	1.5	NG	13.7
	Earthquake	1.8	83.383	85.061	52.632	-0.032	0.932	0.6	NG	17.5	52.632	60.613	0.434	1.2	NG	38.7
3.5m	Steady	2.223	142.439	68.150	75.856	0.979	0.133	0.37	OK	0	75.856	42.594	0.890	1.5	NG	17.4
	Earthquake	2.223	142.439	128.329	75.856	0.186	0.926	0.741	NG	14.0	75.856	81.831	0.463	1.2	NG	50.3
4.0m	Steady	2.273	167.356	90.416	83.484	0.922	0.215	0.379	OK	0	83.484	51.169	0.816	1.5	NG	23.4
	Earthquake	2.273	167.356	169.597	83.484	-0.027	1.164	0.758	NG	33.9	83.484	97.546	0.428	1.2	NG	62.8
4.5m	Steady	2.323	195.363	116.991	91.674	0.855	0.307	0.387	OK	0	91.674	60.523	0.757	1.5	NG	30.0
	Earthquake	2.323	195.363	218.298	91.674	-0.250	1.412	0.774	NG	58.5	91.674	114.406	0.401	1.2	NG	76.3
5.0m	Steady	2.373	226.618	148.319	100.426	0.780	0.407	0.395	NG	1.2	100.426	70.628	0.711	1.5	NG	37.2
	Earthquake	2.373	226.618	274.713	100.426	-0.479	1.666	0.791	NG	87.9	100.426	132.189	0.380	1.2	NG	90.4
5.5m	Steady	2.423	261.259	184.470	109.742	0.700	0.512	0.404	NG	11.9	109.742	81.372	0.674	1.5	NG	44.8
	Earthquake	2.423	261.259	339.833	109.742	-0.716	1.928	0.808	NG	122.9	109.742	151.164	0.363	1.2	NG	105.5
6.0m	Steady	2.473	299.511	225.595	119.619	0.618	0.619	0.412	NG	24.8	119.619	92.723	0.645	1.5	NG	52.9
	Earthquake	2.473	299.511	413.655	119.619	-0.954	2.191	0.824	NG	163.5	119.619	171.145	0.349	1.2	NG	121.4
6.5m	Steady	2.523	341.517	271.326	130.059	0.540	0.722	0.421	NG	39.2	130.059	104.356	0.623	1.5	NG	61.1
	Earthquake	2.523	341.517	521.717	130.059	-1.386	2.648	0.841	NG	234.9	130.059	201.654	0.322	1.2	NG	147.5
7.0m	Steady	2.573	387.398	322.795	141.062	0.458	0.829	0.429	NG	56.4	141.062	116.659	0.605	1.5	NG	69.7
	Earthquake	2.573	387.398	625.821	141.062	-1.690	2.977	0.858	NG	298.9	141.062	227.027	0.311	1.2	NG	168.3
6.5m (Long Bottom Concrete)	Steady	4.323	589.905	268.291	158.803	2.025	0.137	0.72	OK	0	158.803	103.189	0.769	1.5	NG	50.3
	Earthquake	4.323	589.905	518.814	158.803	0.448	1.714	1.441	NG	43.4	158.803	206.124	0.385	1.2	NG	140.0
7.0m (Long Bottom Concrete)	Steady	4.423	669.223	322.795	172.235	2.011	0.201	0.737	OK	0	172.235	116.659	0.738	1.5	NG	59.3
	Earthquake	4.423	669.223	627.153	172.235	0.244	1.968	1.474	NG	85.0	172.235	233.262	0.369	1.2	NG	161.5
3.5m (Short Bottom Concrete, Aroud Sta.7+200)	Steady	0.7	37.223	40.470	33.278	-0.098	0.448	0.117	NG	11.1	33.278	28.866	0.576	1.5	NG	17.8
	Earthquake	0.7	37.223	77.646	33.278	-1.215	1.565	0.233	NG	44.4	33.278	53.509	0.311	1.2	NG	39.7

Source: Study Team

(3) Geological Condition

1) Geological Classification

According to boring surveys, the geology along the Marikina River is largely divided into alluvial and diluvial layers. The alluvium is divided into alluvial clay layer (Ac) and alluvial sand layer (As), and the diluvial clay layer (Dc) and diluvial sand layer (Ds). The weathered layer (GFw) of the GF layer and the fresh rock (GFf) of the GF layer, which is composed of tuff rocks and is diluvial, are covered with these sedimentary layers.

In the design of the SSP revetment, the soil parameters are set according to the above classification. The outline of each soil classification is shown below.

Table 7.1.5 Geological Classification for SSP Design

Age		Geological Symbol / Photo	average N value	Thickness (m))	Characteristic
Quaternary	Holocene (Alluvium)	Ac 	6	5	Distributed in the middle to upper surface of the study area. Gray to dark brown clayey soil containing sand.
		As 	10	5-10	From the downstream end of the study area to the upstream end, it is partially covered by the Ac layer and distributed. Dark brown to brown, fine to coarse sand, the lower part consisting of gravel.
	Pleistocene (Diluvium)	Dc 	19	10-20	It fills the valley of the Marikina River and is widely distributed. Cohesive soil with brown, grey, dark blue-gray colors and high sand content. The surface layer is a little hard about 10m, but it is very hard at depths.
		Ds 	19	5-10	It is covered with Dc on the left bank of the Marikina River, and consists of dark blue - gray to brown coarse sand. The relative density is medium-dense.
		GFw 	50＜	5-10	Surface weathering of basement rock. The distribution is local and judged as gravel soil. Shows brown to gray.
		GFf 	50＜	10m<	Basement rocks in the study area. It consists of white to gray tuff, lapilli tuff, tuffaceous sandstone, conglomerate. Fresh parts are collected in the core.

Source: Study Team

2) Design Values

The design values are set as shown in **Table 7.1.6** based on the results of the existing survey, the new boring survey conducted in this study. Since it was assumed that the angle of internal friction As and Ds layers and the effective cohesion of the Ac and Dc layers depend on the location, the values were calculated from the standard penetration test results (N-value) in the vicinity boring.

The detail for the setting of each constant is described in Chapter 5. Natural conditions investigation 5.2.3.4 Soil test results (1) 2) Proposal of soil modulus.

Table 7.1.6 Design Values for SSP Design

Geological Classification		Unit Weight of Wet Soil	Unit Weight of Submerged Soil	Angle of Internal Fiction	Effective Cohesion	Reference:
		γ_t	γ_w	Φ	c	*Average N Value* ★
Name	Symbol	(kN/m^3)	(kN/m^3)	(degree)	(kN/m^2)	
Field Soil (Embankment)	F	18.0	8.0	30	0	15
Alluvial Sand	As	17.5	7.5	N≤9： 27 9<N： $15+\sqrt{15N}$ (Max: 45)	0	9.9
Diluvial Sand	Ds	19.0	9.0		0	21.1
Alluvial Clay	Ac	15.5	5.5	0	N≤4： 6 x N 4<N≦8： 25 8<N≦15： 50 15<N≦30： 100 30<N： 200	7.8
Diluvial Clay	Dc	18.0	8.0	0	N≤4： 8 x N 4<N≦8： 50 8<N≦15： 100 15<N≦30： 180 30<N： 250	22.6
Guadalupe Formation	GFw	16.5	6.5	20	200	63.5
	GFf	17.0	7.0	30	1,000	121.5

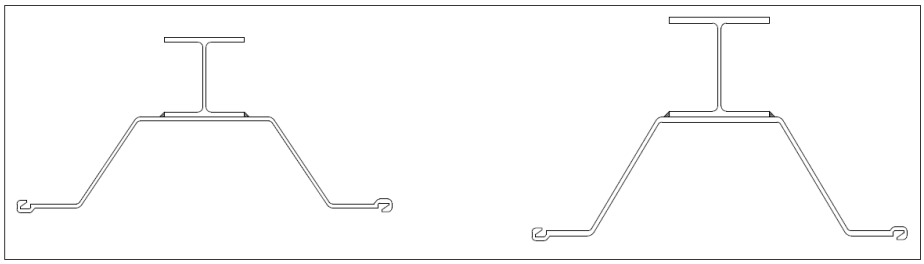
★ The average value of the unconsolidated layer includes existing boring data.

Source: Study Team

(4) Material

1) SSP

As SSP revetment, U-shaped SSP（SP-III～SPIVL）, Hat-shaped SSP(SP-10H、SP-25H、SP-45H、SP-50H）, and self-supporting Hat-Shaped Steel Sheet Pile + H-beam are used in the condition that the deformation is shorter than the allowance shown in the next item and minimize the weight.



Source: Study Team

Figure 7.1.11 Self-supporting Hat-Shaped Steel Sheet Pile + H Beam

The specifications of U-shaped SSP, Hat-shaped SSP, and self-supporting Hat-shaped steel SSP+ H-beam are shown below. The combination of Hat-shaped SSP with H-shaped steel was arranged by weight and moment of inertia because the moment of inertia was dominant in the stability calculation. Those with small moments of inertia and large weight were excluded. (See below)

Table 7.1.7 Properties of SSP

Type of SSP	Dimension (mm)			Per 1.0m (original condition)				Per 1.0m (after corrosion)	
	W	h	t	A (cm^2)	I (cm^4)	Z (cm^3)	Weight (kg/m)	I' (cm^4/m)	Z' (cm^3/m)
U-Shape									
SP-I_A	400	85	8.0	113.0	4500	529	89	3420	402
SP-II_W	600	130	10.3	131.2	13000	1000	103	10500	810
SP-III_W	600	180	13.4	173.2	32400	1800	136	27500	1530
SP-IV_W	600	210	18.0	225.5	56700	2700	177	49900	2380
SP-V_L	500	200	24.3	267.6	63000	3150	210	57300	2870
SP-VI_L	500	225	27.6	306.0	86000	3820	240	79100	3510
Hat-Shape									
SP-10H	900	230	10.8	122.2	10500	902	96	8300	713
SP-25H	900	300	13.2	160.4	24400	1610	126	20000	1320
SP-45H	900	368	15.0	207.8	45000	2450	163	38300	2080
SP-50H	900	370	17.0	236.3	51100	2760	186	44500	2400

I', Z' : are consider 2mm allowable corrosion depth (1mm at each side)
Source: Steel Sheet Pile Association Data

Table 7.1.8 Combinations of SSP and H-Beam

Hat + H sheet pile						Unit mass (kg/m2)	Sectional properties before corrosion				Sectional properties after corrosion (1mm, both side)		
Hat-type sheet pile	NSHYPER BEAMTM						Sectional area (cm2/m)	Moment of inertia (cm4/m)	Section modulus (cm3/m)	Class	Sectional area (cm2/m)	Moment of inertia (cm4/m)	Section modulus (cm3/m)
	H (mm)	B (mm)	t1 (mm)	t2 (mm)	r (mm)								
NS-SP-10H	400	200	9	12	13	169	214.7	87,800	2,320	3	175	71,000	1,890
NS-SP-10H	400	200	9	16	13	182	231.7	98,700	2,700	3	192	81,700	2,270
NS-SP-10H	450	200	9	16	13	186	236.7	119,100	2,970	3	195.8	98,600	2,500
NS-SP-10H	450	200	9	19	13	196	249.5	128,300	3,280	3	208.6	107,600	2,810
NS-SP-10H	450	250	9	16	13	200	254.5	132,600	3,420	3	212.5	110,200	2,890
NS-SP-10H	500	200	12	16	13	202	257.3	149,900	3,540	2	215.4	125,200	3,020
NS-SP-10H	550	200	12	16	13	207	264	177,600	3,900	3	221	148,400	3,320
NS-SP-10H	600	200	12	16	13	212	270.7	208,200	4,270	3	226.5	174,100	3,640
NS-SP-10H	650	200	12	16	13	218	277.3	242,100	4,670	3	232.1	202,500	3,970
NS-SP-10H	700	200	12	16	18	224	285.5	281,600	5,130	3	239.2	235,900	4,370
NS-SP-10H	700	200	12	19	18	234	298	299,700	5,540	3	251.7	254,400	4,790
NS-SP-10H	700	250	12	16	18	238	303.2	308,300	5,740	3	255.9	260,200	4,920
NS-SP-10H	700	200	12	22	18	244	310.5	317,400	5,960	3	264.3	271,700	5,210
NS-SP-10H	700	250	12	19	18	250	319.1	331,300	6,270	3	271.7	281,700	5,430
NS-SP-10H	700	200	12	25	18	254	323.1	334,800	6,370	3	276.8	288,700	5,620
NS-SP-10H	700	250	14	19	18	262	333.8	341,300	6,560	3	286.5	291,300	5,710
NS-SP-10H	700	250	12	22	18	263	335	353,900	6,810	3	287.6	303,900	5,970
NS-SP-10H	750	250	14	19	18	268	341.6	390,100	7,090	3	293.2	334,200	6,190
NS-SP-10H	750	250	14	22	18	280	357.3	414,700	7,640	3	308.9	358,300	6,740
NS-SP-10H	750	250	12	25	18	281	357.5	428,100	7,890	3	309	370,600	6,980
NS-SP-10H	800	250	14	22	18	287	365.1	471,700	8,240	3	315.6	407,000	7,260
NS-SP-10H	850	250	14	22	18	293	372.9	532,300	8,840	3	322.2	460,100	7,790
NS-SP-10H	900	250	14	22	18	299	380.7	597,600	9,470	3	328.9	516,600	8,340
NS-SP-10H	900	250	14	25	18	311	396.4	631,400	10,120	3	344.6	549,900	8,990
NS-SP-10H	950	250	16	22	18	321	408.6	688,700	10,570	3	355.7	596,800	9,340
NS-SP-10H	1000	250	16	22	18	328	417.5	766,300	11,270	3	363.5	665,100	9,970
NS-SP-10H	1000	250	16	25	18	340	433.1	806,700	11,980	3	379.1	704,900	10,680
NS-SP-10H	1000	250	16	28	18	352	448.7	845,700	12,670	3	394.7	743,300	11,370
NS-SP-25H	1000	250	16	22	18	358	455.7	892,400	12,240	3	398.3	778,400	10,850
NS-SP-10H	1000	250	16	32	18	369	469.5	896,800	13,590	3	415.5	794,700	12,300
NS-SP-25H	1000	250	16	25	18	370	471.3	935,900	12,980	3	413.9	821,200	11,580
NS-SP-25H	1000	300	16	22	18	377	480.1	962,300	13,420	3	421.6	842,700	11,920
NS-SP-25H	1000	250	16	28	18	382	486.9	977,700	13,690	3	429.5	862,300	12,290
NS-SP-25H	1000	300	16	25	18	392	499	1,014,300	14,310	3	440.5	892,900	12,800
NS-SP-25H	1000	250	16	32	18	399	507.7	1,032,400	14,640	3	450.3	917,200	13,250
NS-SP-25H	1000	300	16	28	18	407	518	1,064,500	15,180	3	459.5	943,400	13,680
NS-SP-25H	1000	250	19	32	18	423	538.9	1,068,100	15,400	2	481.5	950,400	13,990
NS-SP-25H	1000	300	16	32	18	426	543.2	1,129,800	16,330	3	484.7	1,007,700	14,830
NS-SP-45H	1000	250	19	25	18	432	550.3	1,137,900	15,010	3	490.3	1,010,500	13,510
NS-SP-45H	1000	250	16	32	18	436	555.1	1,197,700	15,840	3	495	1,071,200	14,370
NS-SP-45H	1000	300	16	28	18	444	565.4	1,232,100	16,400	3	504.2	1,098,900	14,810
NS-SP-45H	1000	250	19	32	18	460	586.3	1,240,400	16,710	3	526.3	1,111,400	15,210
NS-SP-45H	1000	300	16	32	18	464	590.6	1,302,900	17,590	3	529.4	1,168,800	16,000
NS-SP-50H	1000	250	19	32	18	483	614.8	1,301,500	17,050	3	554.7	1,174,400	15,570
NS-SP-50H	1000	300	16	32	18	486	619.1	1,364,500	17,920	3	557.8	1,232,000	16,340
NS-SP-45H	1000	300	19	36	18	508	646.8	1,409,300	19,580	3	585.7	1,272,900	17,980
NS-SP-50H	1000	300	19	32	18	510	650.3	1,408,100	18,800	3	589.1	1,274,700	17,220
NS-SP-50H	1000	250	19	40	18	515	655.8	1,415,900	18,960	3	595.7	1,288,400	17,490
NS-SP-45H	1000	300	19	40	18	527	671.8	1,474,300	20,720	3	610.7	1,337,900	19,130
NS-SP-50H	1000	300	19	36	18	530	675.3	1,476,300	19,960	3	614.1	1,342,000	18,370
NS-SP-50H	1000	300	19	40	18	550	700.3	1,543,500	21,110	3	639.1	1,409,400	19,540
NS-SP-50H	1000	300	22	40	18	574	730.9	1,581,400	21,920	2	669.8	1,445,200	20,320

Source : Steel Sheet Pile Association Data

Table 7.1.9 Example of Selecting Combinations of Hat-Shaped SSP and H-Beam

Combination	Moment of inertia	Unit Mass	Adaptation
A	45,000	250	Appropriate
B	47,000	260	Inappropriate (B unit mass is bigger than C unit mass)
C	48000	258	Appropriate
D	55,000	265	Inappropriate (D unit mass is bigger than F unit mass)
E	60,000	272	Inappropriate (E unit mass is bigger than F unit mass)
F	65000	264	Appropriate
G	70,000	270	-

Source: Study Team

2) Section Efficiency

As shown in **Table 7.1.10**, the 20% reduction of rigidness (e.g., Moment of Inertia of Area: I x 0.8) is applied to U-shape SSP. This reduction is caused by the joint efficiency of U-shape SSP during the bending load. On the other hand, there is no reduction of rigidness for Hat-shape SSP due to their connecting structural characteristic between SSPs.

Table 7.1.10 Moment of Inertia of Area and Efficient Ratio in SSP Wall

Item	Classification of Calculation	Efficient Ratio of Sectional Factor	
		Hat-shape	U-shape
Moment of Inertia of Area	Calculation of Penetration Depth	Full cross section is effective (100%)	
	Calculation of Dispalcement and Sectional Force	Full cross section is effective (100%)	80% of full cross section is effective
Sectional Factor	Stress Calculation	Full cross section is effective (100%)	

Source: Cantilever Steel Sheet Pile Design Manual, 2007 Dec.

3) Corrosion

The thickness of SSP shall consider corrosion allowance in 2 mm. It is assumed that the corrosion will process by 0.02 mm per year (actual value) during the entire service time of 50 years.

(5) Allowable Value

1) Allowable Stress

In this project, SYW 295 and SM 490 H-shaped steel will be used. The allowable stress of SYW 295 (SYW 390) for design calculation is 180 (235) N/mm^2. Similarly, the allowable stress of the SM 490 H-shaped steel is 185 N/mm2. The allowable stress is set to 185 N/mm2 in consideration that the tensile strength of the H-shaped steel placed on the land side is the dominant strength of the SSP with H shaped steel.

The seismic load works only temporarily during earthquake. Considering this point, the allowable stress level during an earthquake shall be increased by 50%. Therefore, the allowable stress levels (seismic) applied to SSPs and H-shaped SSPs shall be set as 270 N / mm2 and 277.5 N / mm2, respectively.

2) Allowable Displacement

The allowable displacement of SSPs shall be 50 mm at normal conditions.

In the case of an earthquake, the allowable displacement shall be increased by 50% because the period of time in which the seismic load acts is temporary. Therefore, the type of SSP shall be

decided so that the amount of displacement at a normal state is less than 50 mm and that during an earthquake is less than 75 mm. If SSP does not resist against external force and circular slip, or there is the fear of scouring, foot protection structures such as ripraps or gabions in front of SSP shall be installed.

7.1.2.4 Result of Calculation

The SSP specifications and length determined by the structural calculation of SSP are shown in **Table 7.1.11** and **Table 7.1.12**.

Table 7.1.11 SSP Specification (1/2)

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

Source: Study Team

Table 7.1.12 SSP Specification (2/2)

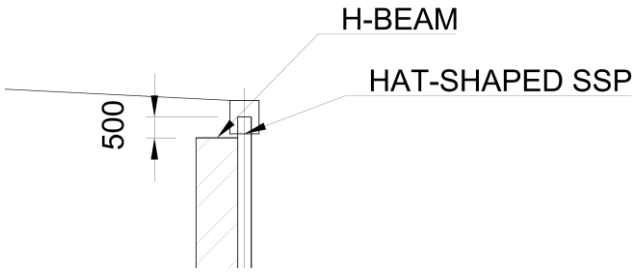
Bank	Block No.	Sta. From	-	To	Length in Longidinal direction	Specification					Length			RIPRAP (SIDE SLOPE: 1.5 :1)		
						Combined					Total	SSP	H-beam	FINISHED HEIGHT FROM DRB	FINISHED TOP WIDTH	CLASS
						No.	Name	Unit Mass	Section Modulus	Moment of Inertia						
					(m)			(kg/m2)	(cm3/m)	(cm4/m)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

Source: Study Team

7.1.2.5 Special Consideration about Hat-Shaped SSP and H-Beam

(1) Omission of Upper End of H-beam considering Construction

When a Vibro hammer is used for driving a Hat-shaped SSP + H-beam, the web of H-beam is usually gripped and driven by a chuck. However, in this construction, since the driving length of the SSP is long, it is assumed that a large Vibro hammer is used, and, the Hat-shaped SSP, which has already been driven, interferes with the chuck and the seismograph. Therefore, in the same concept as in phases 2 and 3, the upper end 50 cm of the hat-shaped SSP is omitted because the hat-shaped SSP is grasped by the chuck and the SSP is driven.

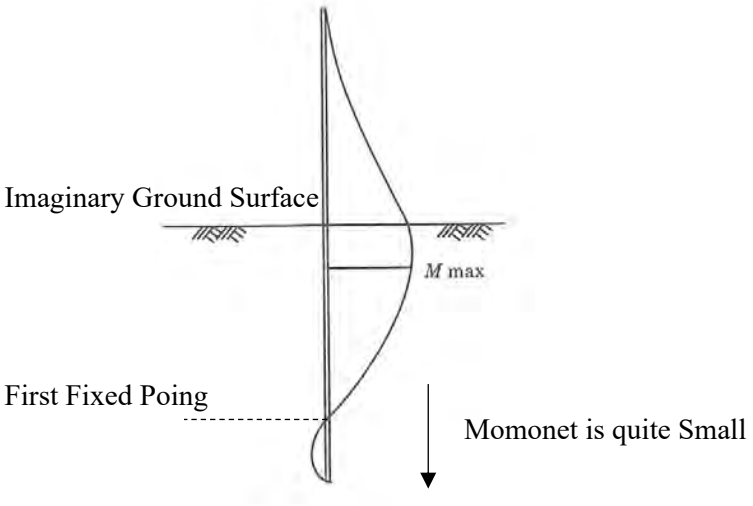


Source: Study Team

Figure 7.1.12 Omitting of Upper End of H-Beam

(2) Omission of Lower End of Hat-shaped SSP due to Reduction of the Quantity

As shown in the below figure, the moment of the self-supporting SSP becomes the maximum at the lower part of the imaginary ground surface, and the moment below the first fixed point is quite small.



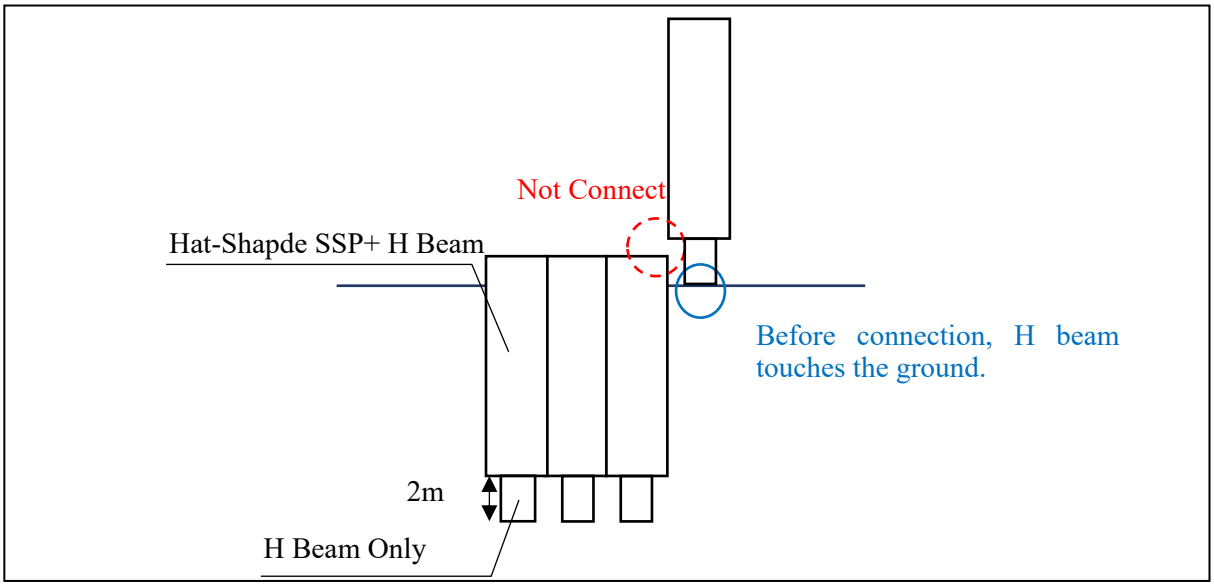
Source: Design Guidelines of Disaster Recovery Works

Figure 7.1.13 Moment Distribution of Self-Supporting SSP

Based on the above concept, in order to reduce the weight of steel materials in hat-shaped SSP + H beam, the hat-shaped SSP is shortened by 2 m at the lower end of the SSP where the moment is small. In this calculation, only the hat-shaped SSP section is omitted because the moment of inertia is dominant and that of H-beam is larger than that of hat-shaped SSP. Recently, there have been many similar cases of hat-shaped SSP + H-beam.

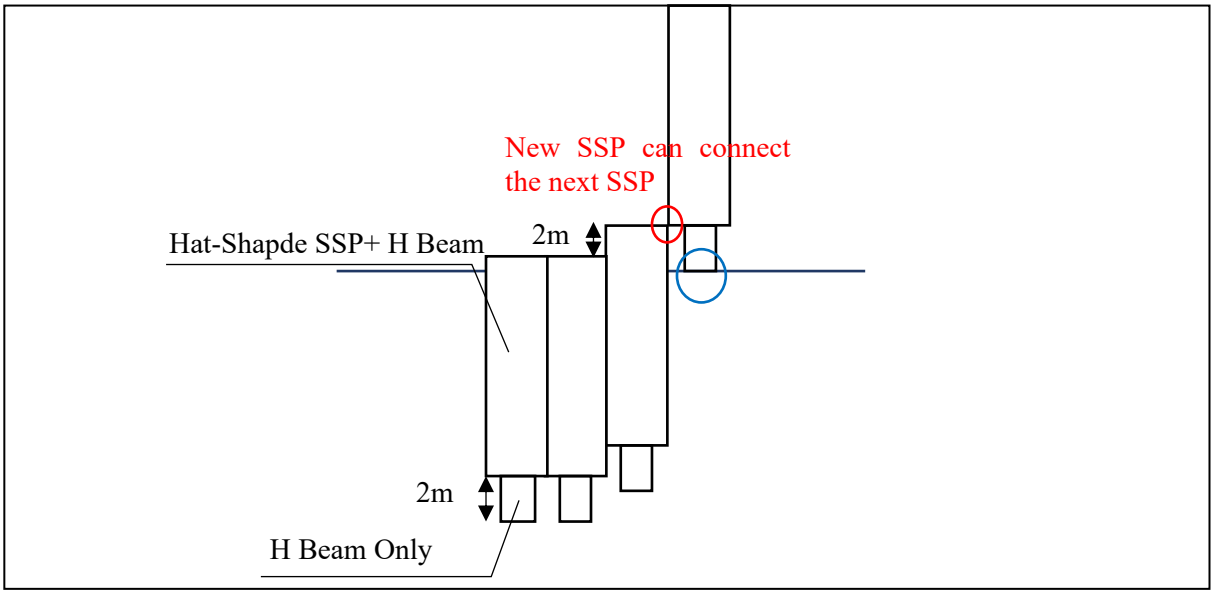
The omitted length was set considering workability. When the hat-shaped SSP at the lower end is

omitted, if the SSP is to be driven one by one as in the normal sequence of driving of the SSP, the hat-shaped SSP portion with the joint will come into contact with the ground of only the lower end of H-beam attached the ground before the joint which is attached hat-shaped SSP connect the next SSP, and it makes difficult that the joint connect portion (please refer to below figure). Therefore, in this construction, it is assumed that the driving of the SSP next to the SSP to be newly driven is stopped once at a position slightly higher than the specified driving depth, and the remaining driving is finished after connecting with the SSP to be newly driven. Assuming such a sequence, the connection work of the joint is adjusted by human hands, so it was set to 2 m, which is within reach of human hands.



（SSP is Driven One by One）

,



（SSP Driven Method in this Construction）

Source: Study Team

Figure 7.1.14 Method of SSP Driven

In addition, among the cases in ha-shaped SSP+ H beam was adopted, the cases in which the total

length of the sheet pile was the minimum (R -23) and the longest (R -40) were used, and the cross-section inspection was carried out at the point where the hat sheet pile was omitted. The moment generated at the point where the hat-shaped sheet pile was cut was calculated by the following formula derived from Chang's formula. The result of the calculation is shown in **Table 7.1.13**. the generated stress was about 10% at the maximum compared with the allowable stress.

$$M_{(x)} = \frac{P_0}{\beta} \times \exp^{-\beta x} \left(\beta h_0 \cdot \cos\beta x + (1+\beta h_0)\sin\beta x\right)$$

Here,

M (x) : bending moment at x (m) from the imaginary ground surface.
P_0 : earth pressure (Horizontal load)
β : Property Value

Where, $\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$

K_h : lateral subgrade reaction factor
B : Unit Width
E : Young's modulus
I : Moment of inertia
h_0 : working height of load

Table 7.1.13 Section Inspection at Omitted Place of Hat-shaped SSP

Block	Condition	β (m^{-1})	P_0 (kN/m)	h_0 (m)	c (m)	M(c) (kN*m/m)	Z_H (cm^3)	σ (N/mm)	σ_a (N/mm)	Judge
R-23	Ordinaly	0.41097	158.89	2.05	5.29	45.7	2900	15.8	180.0	OK
(Length Minimum)	Seismic	0.48873	235.48	3.31	5.29	0.3	2900	0.1	270.0	OK
R-40	Ordinaly	0.18179	167.69	2.73	14.94	10.1	7800	1.3	180.0	OK
(Length Maximum)	Seismic	0.21347	207.20	3.18	13.95	-19.6	7800	2.5	270.0	OK

Source: Study Team

Where,

C : The distance from the imaginary ground surface to the omitted place.
M (c) : Bending moment at c
Z_H : Section Modulus for H-beam
σ : Stress at c
σ a : Allowable Stress

7.1.3 Detailed Design of Revetment for Dike

7.1.3.1 Study on Inclined Wall and Parapet Wall

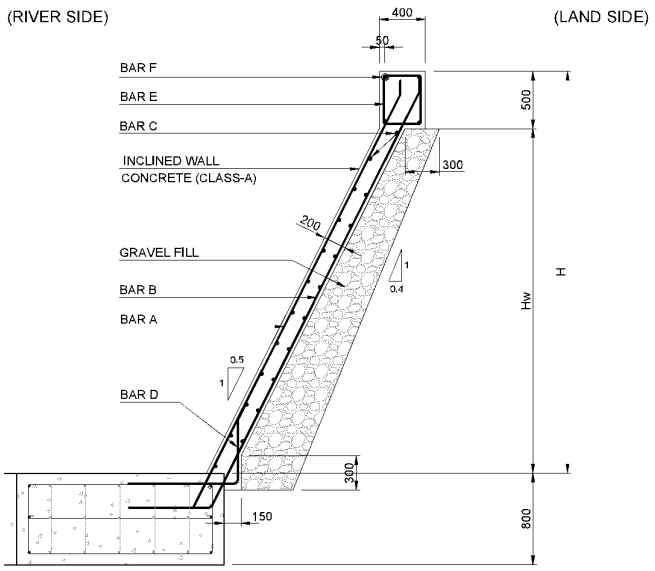
(1) Inclined Wall

The dimensions and reinforcement etc. of the inclined wall determined by structural calculation are shown in **Table 7.1.14**. 14. Also, the standard cross-section is shown in **Figure 7.1.15**.

Table 7.1.14 Dimension of Inclined Wall

HEIGHT RANGE		REINFORCEMENT											
		BAR A		BAR B		BAR C		BAR D		BAR E		BAR F	
Hw(m)	H(m)	Df (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)
0.00-1.50	0.50-2.00	16	250	16	250	12	300	12	250	12	250	12	-
1.50-2.50	2.00-3.00	16	125	16	125	12	300	12	125	12	125	12	-
2.50-2.75	3.00-3.25	16	125	16	125	12	300	12	125	12	125	12	-
2.75-3.00	3.25-3.50	20	125	20	125	12	300	12	125	12	125	12	-

Source: Study Team



Source: Study Team

Figure 7.1.15 Typical Cross-section of Inclined Wall

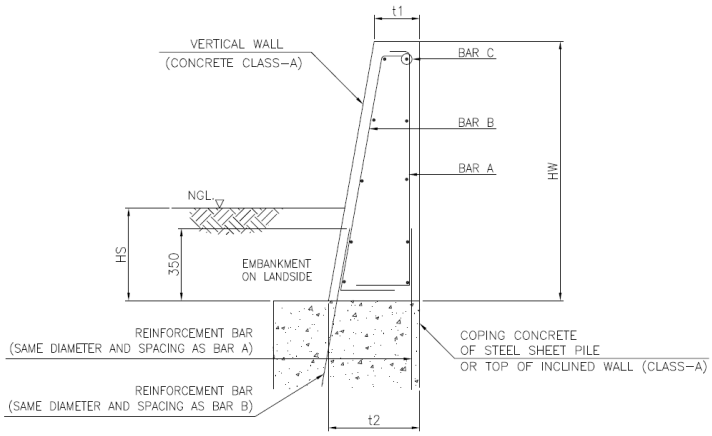
(2) Parapet Wall

The dimensions and reinforcement etc. of the parapet wall determined by structural calculation are shown in **Table 7.1.15**. 14. Also, the standard cross-section is shown in **Figure 7.1.16**.

Table 7.1.15 Dimension of Parapet Wall

HEIGHT RANGE		THICKNESS		REINFORCEMENT					
				BAR A		BAR B		BAR C	
Hw(m)	Hs(m)	t1 (mm)	t2 (mm)	Df] (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)	Df] (mm)	SPACING (mm)
0.00-1.50	0.00-0.50	0.20	t1+Hw x 0.1	12	250	12	250	12	300
0.00-1.50	0.50-1.00	0.20	t1+Hw x 0.1	12	250	12	250	12	300
0.00-1.50	1.00-1.50	0.20	t1+Hw x 0.1	12	250	12	250	12	300

Source: Study Team



Source: Study Team

Figure 7.1.16 Typical Cross-section of Parapet Wall

7.1.3.2 Material for Embankment and Backfill

(1) Consideration for Embankment Material

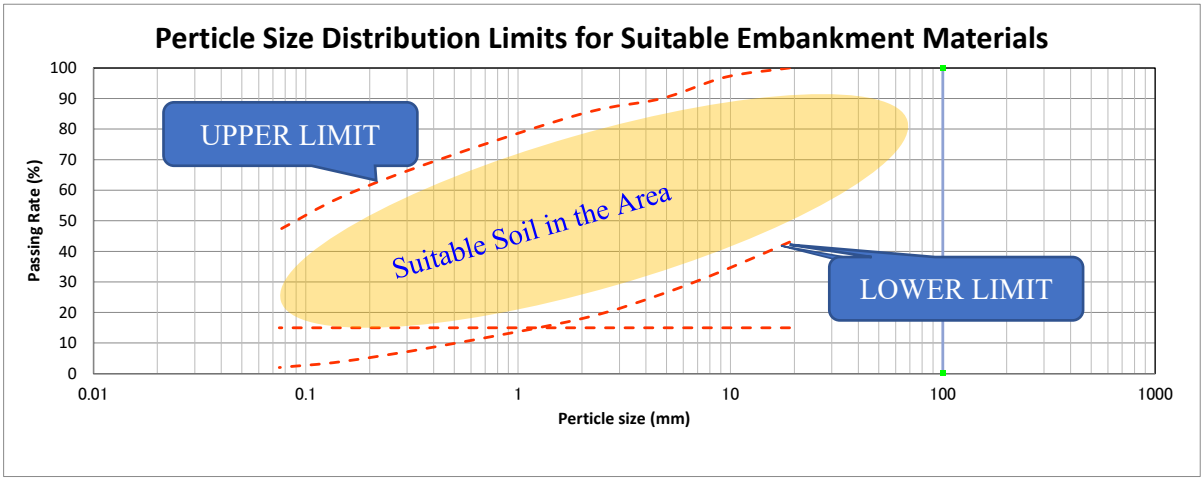
It is necessary for dike to have enough stability for a long time after the completion. The embankment material is the main material and related to the stability of the embankment directly Therefore, it is necessary to use suitable material for compaction. Specifically, it is necessary to select

1. Material that is sufficiently dense and doesn't soften or expand due to water intrusion: and
2. Material whose stability and bearing resistance of the embankment increase by compaction: and
3. Material with the smallest compressive settlement after completion of embankment

On the other hand, in the view of the cost reduction, 1.5 million m^3 of sediment in 3 packages will be totally transported to dumpsites in excavation work and dredging work. It is necessary to examine the effective utilization of these soil.

1) Standards for Embankment Material

In this design, there is no guideline of selection for the embankment material in Philippines. Therefore, the method for suitable embankment materials adopts "River earthwork Manual" in Japan. According to "River earthwork Manual", the suitable distribution of the particle size for embankment material shown in **Figure 7.1.17** is between the red dashed line.



Source: Study Team

Figure 7.1.17 Particle Size Distribution for Embankment Material

2) Particle Size Distribution of Soil Generated by Construction and Purchased Soil

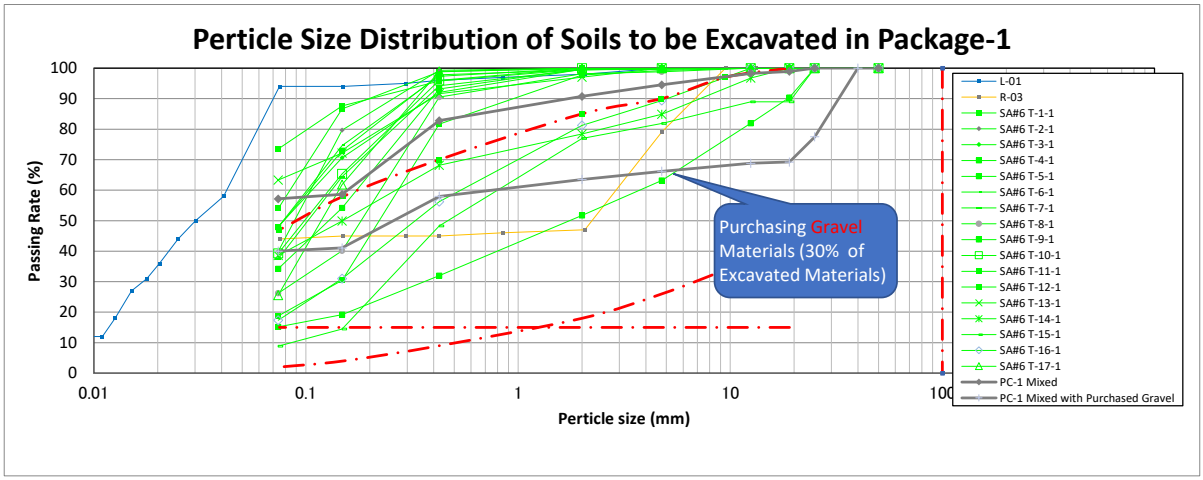
The soil generated by the excavation work and dredging work in all packages contains a lot of silt or sand because of the sediment deposited in the riverbed or riverbank. The particle size distribution exceeds the upper limit line. It is strong against infiltration, but its stability is weak, and consolidation settlement also occurs highly. Therefore, the trial calculation was conducted, and the following values were calculated for the generated soil in each package, by purchasing and mixing 20 ~ 40 mm of gravel (gravel) at the ratio shown in **Table 7.1.16**, to become a suitable particle size (**Figure 7.1.18~Figure 7.1.20**).

Therefore, the embankment is basically constructed using the soil generated at the site and mixed gravel (20 ~ 40 mm) with the ration, shown in **Table 7.1.16**.

Table 7.1.16 Ratio for Purchased Soil

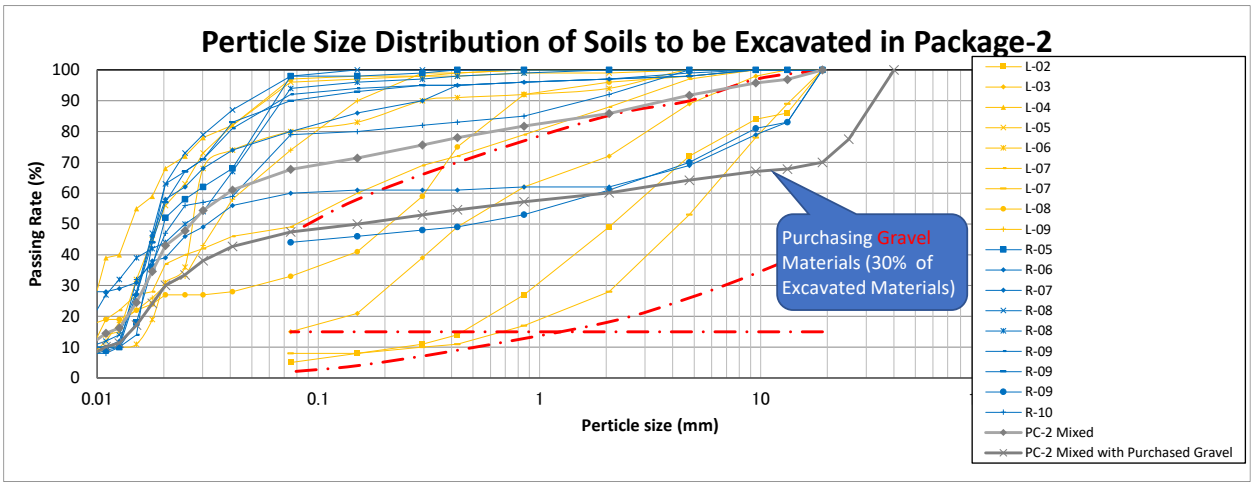
Item	CP-1	CP-2	CP-3
Ration of Purchased and Mixed Soil	30%	30%	10%

Source: Study Team



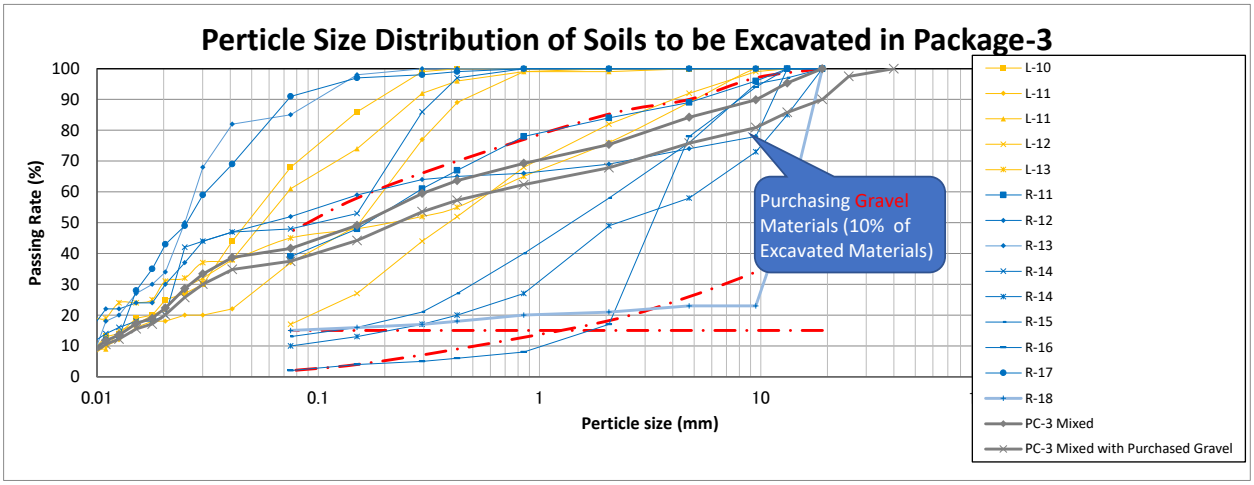
Source: Study Team

Figure 7.1.18 Particle Size Distribution of Generated Soil and Mixed Soil with 30% Gravel on CP-1



Source: Study Team

Figure 7.1.19 Partial Size Distribution of Generated Soil and Mixed Soil with 30% Gravel on CP-2



Source: Study Team

Figure 7.1.20 Particle Size Distribution of Generated Soil and Mixed Soil with 10% Gravel on CP-3

(2) Consideration for Backfill Material

In the place where the revetment is installed in front of the existing revetment, it is necessary to fill the space between the new revetment and the existing revetment with sand. For the following reason, the sand is used for backfill, same as in the previous phases (In Phases II and III use Free Draining Backfill).

- Improvement of drainage performance in the backfill of the newly constructed revetment
- Minimize the subsidence in the backfill area

7.1.3.3 Detailed Design of Embankment and Slope of Revetment

(1) Slope Stability Normal Condition and Seismic Condition

1) Methodology

In this section, stability analysis is carried out at the embankment and slope of the revetment under normal condition and level 1 seismic condition. The analysis was based on the slope stability analysis by the modified Fellenius method described in **Sub-section 11 11.5. 3**. The seismic coefficient on the horizontal direction is 0.20 (See **Sub-section 11. 11.4.2.2**).

In this design, as a result of the evaluation of the liquefaction risk, there is no place where a liquefaction layer exists under the embankment. Therefore, the circular slip (Δu) method considering the excess pore water pressure caused by liquefaction is not applicable. (Refer to **Sub-section 6.1.3.2** for the liquefaction judgment result.)

The safety factor is 1.2 under normal condition and 1.0 under seismic (see **Sub-section 11. 11.5. 3**).

2) Cross-Section for the Analysis

For each design section with different slope structures in revetment and foundation, the cross-section for the analysis are extracted in consideration with high slope height and weak ground etc. The extracted cross-section is shown in **Table 7.1.17**.

Table 7.1.17 Cross-Sections for Stability Analysis

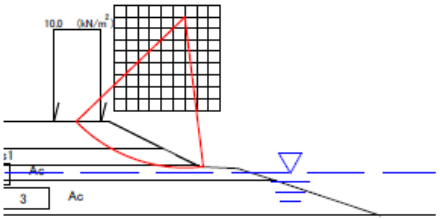
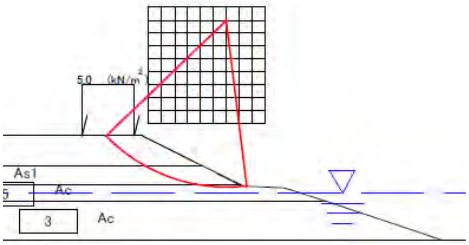
Bank	Cross Section	Structure	Reason for Selection
Left	L5 + 400	Riverside: Upper 20%, Lower 30% Slope Landside: Flat land (parking lot)	The sand layer and cohesive soil layer are the thickest and the ground behind is the weakest. It also has the highest wall height among sections with the same revetment structure.
	L5 + 780	Riverside: Upper 20%, Lower 20% Slope Landside: Flat land (administrative road)	The sand layer and cohesive soil layer are the thickest in the section with the same revetment structure, and the ground behind is the weakest.
	L6 + 340	Riverside : Upper 20% , Lower SSP revetment Landside: Flat land (administrative road)	The walls of the sections with the same revetment structure have the same height in all sections, but the section has the weakest ground.
	L7 + 820	Riverside : 0.5 retaining walls + SSP revetment Landside: Retaining wall 0.5 cm + SSP	It has the highest wall height among the sections with the same revetment structure.
Right	R6 + 060	Riverside : 0.5 retaining walls + SSP revetment Landside: 20% slope + embankment protection	It has the highest wall height among the sections with the same revetment structure.
	R 10 + 960	Riverside : 0.5 retaining walls + SSP revetment Landside: 20% slope + embankment protection	It has the highest wall height among the sections with the same revetment structure.

Source: Study Team

3) Analysis Result

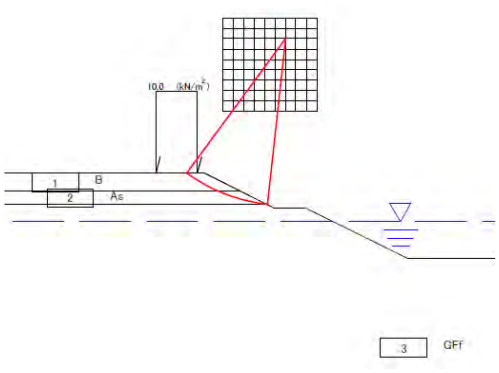
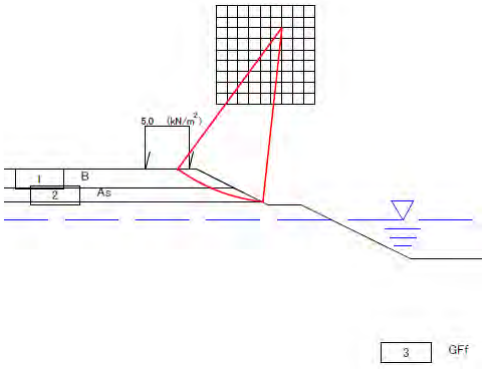
The result of the stability analysis of the slope is shown below.

Table 7.1.18 Result of Stability Analysis (L5+400)

Cross Section	L5+400	
Slope Height	3.84 (m)	
Crown Width	(flat ground)	
Case	Normal	Seismic
Water Level	14.415 (OWL)	14.415 (OWL)
Result	**Min. Fs=2.665** > 1.20 OK 	**Min. Fs=1.832** > 1.00 OK 

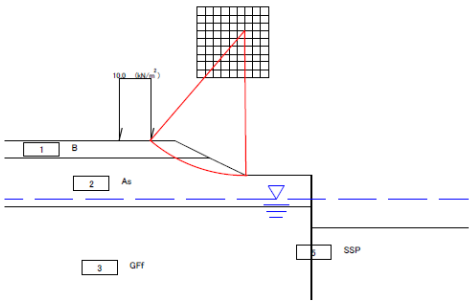
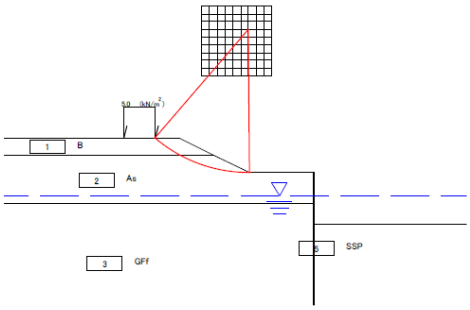
Source: Study Team

Table 7.1.19 Result of Stability Analysis (L5+780)

Cross Section	L5+780	
Dike Height	3.28 (m)	
Crown Width	(flat ground)	
Case	Normal	Seismic
Water Level	14.444 (OWL)	14.444 (OWL)
Result	**Min. Fs=6.128** > 1.20 OK 	**Min. Fs=4.435** > 1.00 OK 

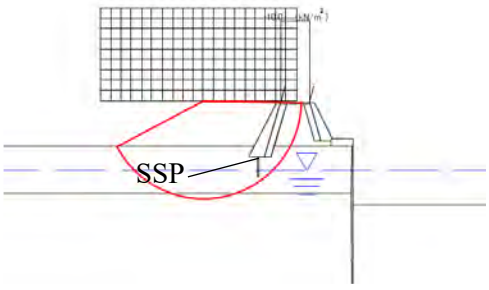
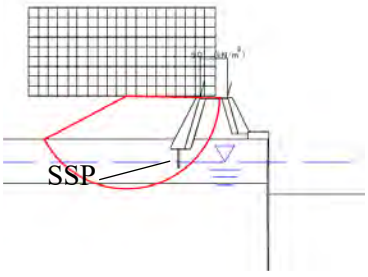
Source: Study Team

Table 7.1.20 Result of Stability Analysis (L6+340)

Cross Section	L6+340	
Dike Height	4.50 (m)	
Crown Width	(flat ground)	
Case	Normal	Seismic
Water Level	14.487 (OWL)	14.487 (OWL)
Result	**Min. Fs=1.642** > 1.20 OK 	**Min. Fs=1.001** > 1.00 OK 

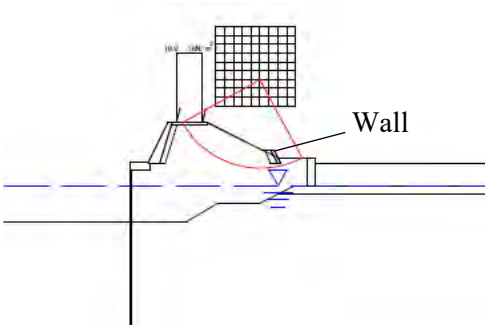
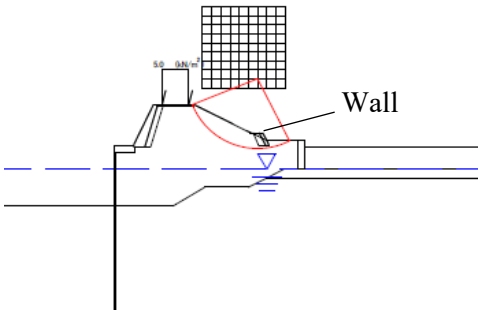
Source: Study Team

Table 7.1.21 Result of Stability Analysis (L7+820)

Cross Section	L7+820	
Dike Height	4.43 (m)	
Crown Width	3.70 (m)	
Case	Normal	Seismic
Water Level	11.601 (OWL)	11.601 (OWL)
Result	**Min. Fs=3.690** > 1.20 OK  The circular arc shown upper is the lower end of the investigation range, but the circular arc at the top edge position, which is the minimum safety factor, was extracted, because the slip occurred from the inside of embankment below this point and was inappropriate.	**Min. Fs=1.886** > 1.00 OK  The circular arc shown upper is the lower end of the investigation range, but the circular arc at the top edge position, which is the minimum safety factor, was extracted, because the slip occurred from the inside of embankment below this point and was inappropriate.

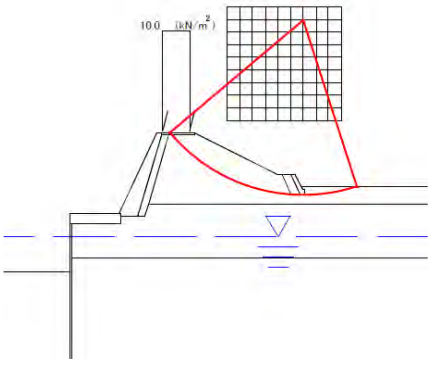
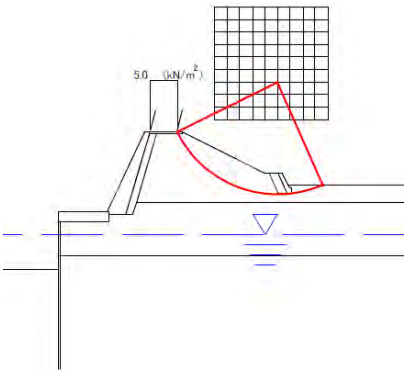
Source: Study Team

Table 7.1.22 Result of Stability Analysis (R6+060)

Cross Section	R6+060	
Dike Height	3.97 (m)	
Crown Width	4.30 (m)	
Case	Normal	Seismic
Water Level	11.465 (OWL)	11.465 (OWL)
Result	**Min. Fs=1.682** > 1.20 OK 	**Min. Fs=1.033** > 1.00 OK 

Source: Study Team

Table 7.1.23 Result of Stability Analysis (R10+960)

Cross Section	R10+960	
Dike Height	4.26 (m)	
Crown Width	3.00 (m)	
Case	Normal	Seismic
Water Level	11.843 (OWL)	11.843 (OWL)
Result	**Min. Fs=1.593** > 1.20 OK 	**Min. Fs=1.006** > 1.00 OK 

Source: Study Team

(2) Flood Condition（Slip by Infiltration）

1) Methodology and Cross-section for Infiltration Analysis

In this section, slip safety by infiltration against the designed flood is examined. The analysis used the modified Ferrenius method, described in Chapter 11, Section 11.5. 3. The safety factor shall be 1.2 (11. Chapter 11.5. 3).

Together with the slope stability, safety against the seepage (piping) at the bottom of the land-side embankment is checked. If the vertical and horizontal hydraulic gradients at the bottom of the embankment do not exceed 0.5 when the embankment is saturated, it is judged that there is no risk of piping.

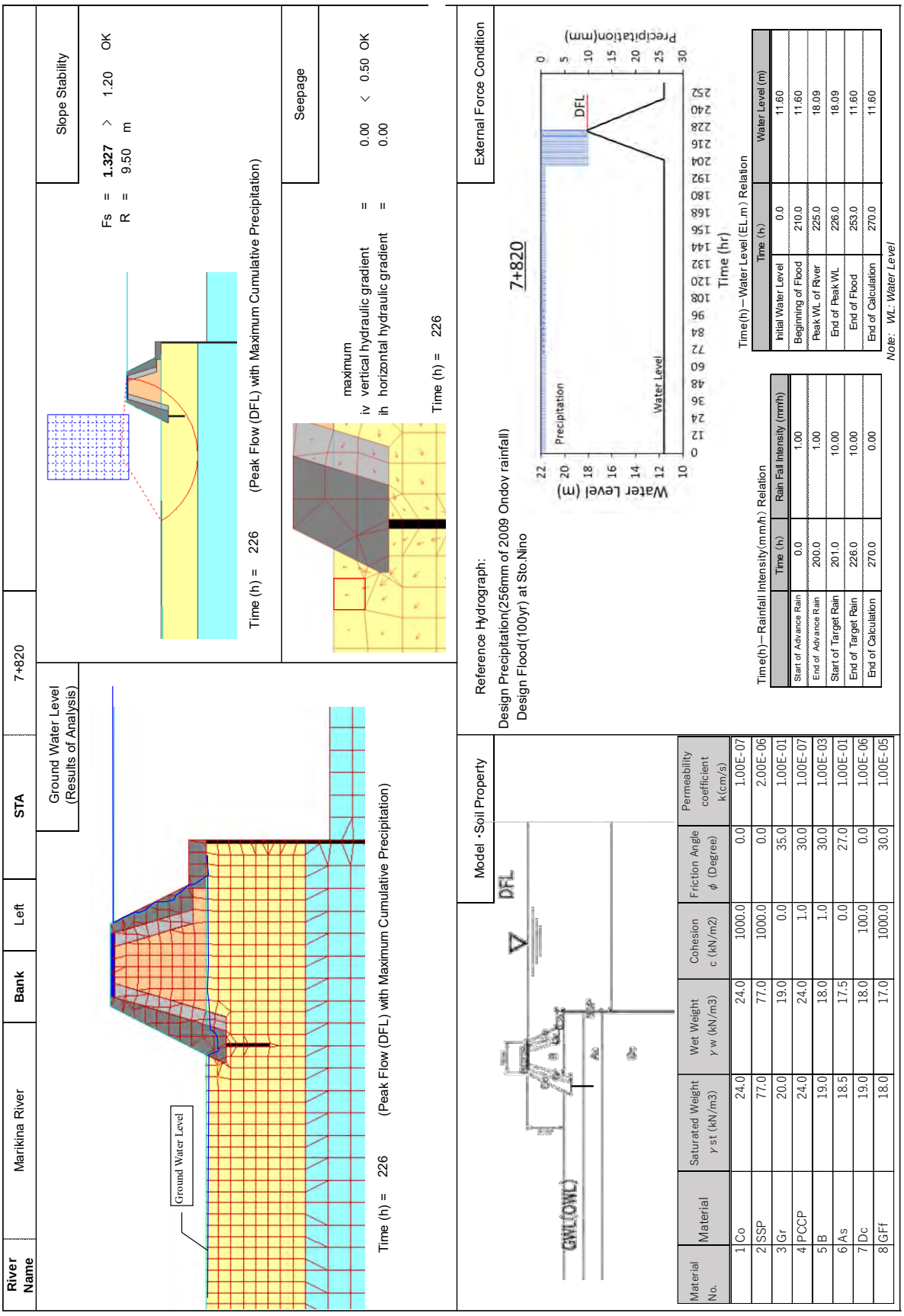
Table 7.1.24 Cross-Section for Infiltration Analysis

Bank	Cross Section	Structure	Reason for Selection
Left	L7+820	Riverside : 0.5 retaining walls + SSP revetment Landside: Retaining wall 0.5 cm + SSP	It has the highest wall height among the sections with the same revetment structure.
Right	R6+060	Riverside : 0.5 retaining walls + SSP revetment Landside: 20% slope + embankment protection	It has the highest wall height among the sections with the same revetment structure.
	R10+960	Riverside : 0.5 retaining walls + SSP revetment Landside: 20% slope + embankment protection	It has the highest wall height among the sections with the same revetment structure.

Source: Study Team

2) Analysis Result

The analysis conditions and results are shown below. The calculated safety factor of the cross-section was over the required.



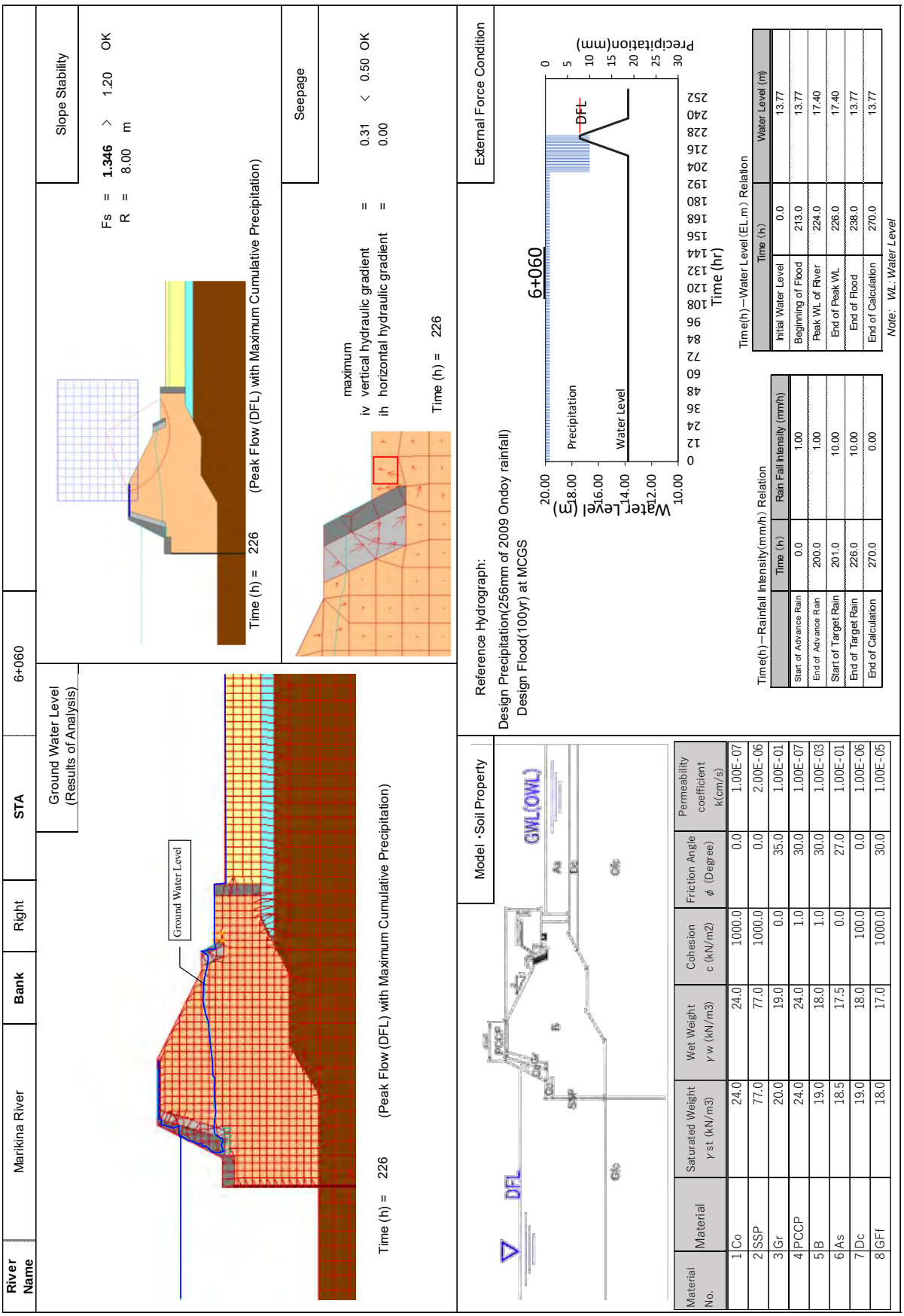
Material No.	Material	Saturated Weight γ st (kN/m3)	Wet Weight γ w (kN/m3)	Cohesion c (kN/m2)	Friction Angle φ (Degree)	Permeability coefficient k(cm/s)
1	Co	24.0	24.0	1000.0	0.0	1.00E-07
2	SSP	77.0	77.0	1000.0	0.0	2.00E-06
3	Gr	20.0	19.0	0.0	35.0	1.00E-01
4	PCCP	24.0	24.0	1.0	30.0	1.00E-07
5	B	19.0	18.0	1.0	30.0	1.00E-03
6	As	18.5	17.5	0.0	27.0	1.00E-01
7	Dc	19.0	18.0	100.0	0.0	1.00E-06
8	GFf	18.0	17.0	1000.0	30.0	1.00E-05

	Time (h)	Rain Fall Intensity (mm/h)
Start of Advance Rain	0.0	1.00
End of Advance Rain	200.0	1.00
Start of Target Rain	201.0	10.00
End of Target Rain	226.0	10.00
End of Calculation	270.0	0.00

	Time (h)	Water Level (m)
Initial Water Level	0.0	11.60
Beginning of Flood	210.0	11.60
Peak WL of River	225.0	18.09
End of Peak WL	226.0	18.09
End of Flood	253.0	11.60
End of Calculation	270.0	11.60

Source: Study Team

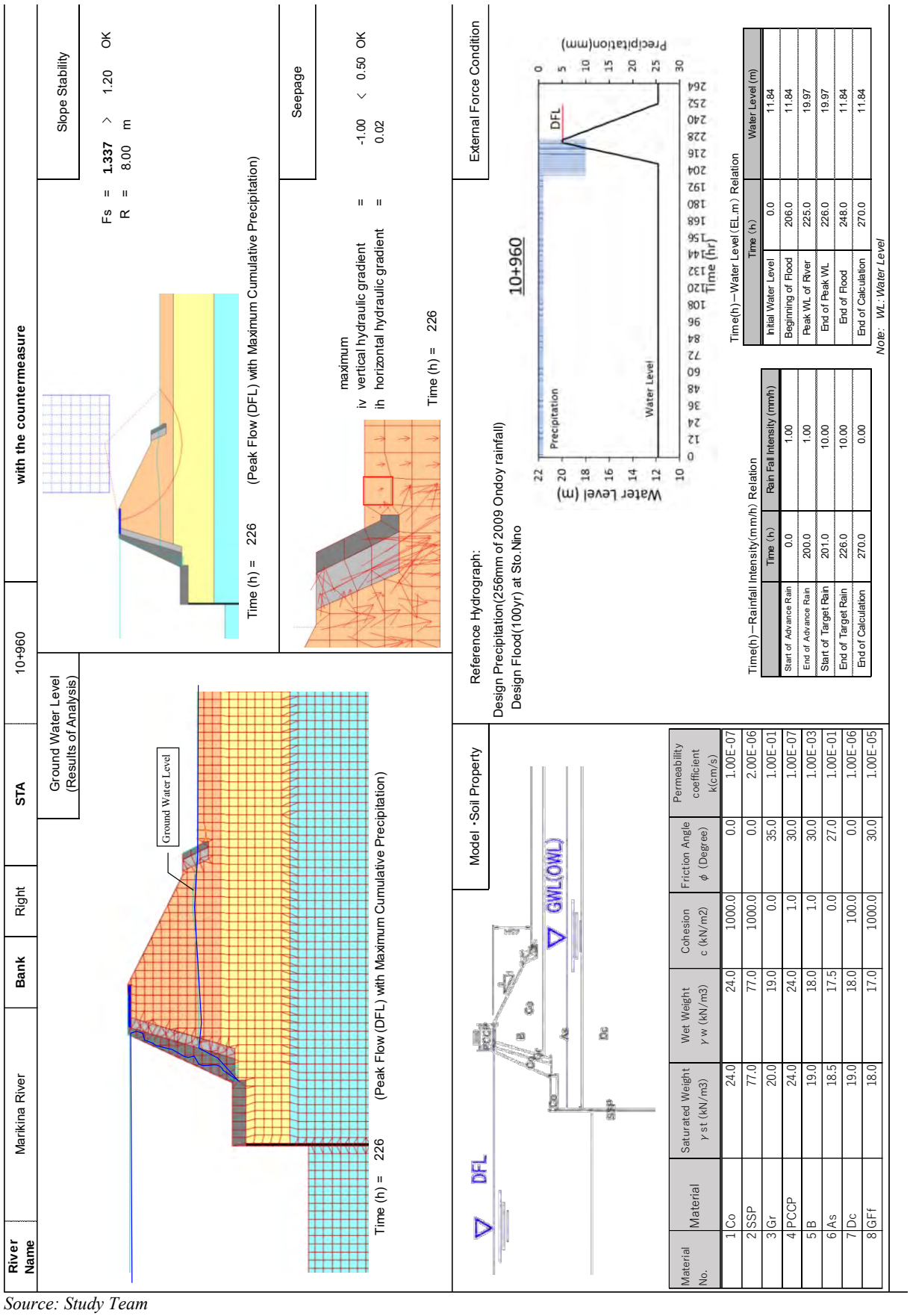
Figure 7.1.21 Result of Infiltration Analysis (L7+820)



Material No.	Material	Saturated Weight γ st (kN/m3)	Wet Weight γ w (kN/m3)	Cohesion c (kN/m2)	Friction Angle ϕ (Degree)	Permeability coefficient k(cm/s)
1	Co	24.0	24.0	1000.0	0.0	1.00E-07
2	SSP	77.0	77.0	1000.0	0.0	2.00E-06
3	Gr	20.0	19.0	0.0	35.0	1.00E-01
4	PCCP	24.0	24.0	1.0	30.0	1.00E-07
5	B	19.0	18.0	1.0	30.0	1.00E-03
6	As	18.5	17.5	0.0	27.0	1.00E-01
7	Dc	19.0	18.0	100.0	0.0	1.00E-06
8	GFf	18.0	17.0	1000.0	30.0	1.00E-05

Source: Study Team

Figure 7.1.22 Result of Infiltration Analysis (R6+060)



Source: Study Team

Figure 7.1.23 Result of Infiltration Analysis (R10+960)

7.1.3.4 Design of Slope Protection Work

(1) Extraction of Slope Protection Work

Slope protection is installed in the place where the slope is less than 20%. the slope is protected by grass sodding where the design flow rate is less than 2.0 m/s. Moreover, slope protection is installed to protect erosion in the following places.

- Upstream and downstream of weir, floodgate, and structures traversing dike
- Right bank of the downstream of MCGS

Slope protection work is extracted from the types introduced in DGCS, vol.3. From the evaluation is shown in **Table 7.1.25**, 5) Gabion (Spread Type) and 9) Reinforced Concrete Facing are applicable. From these two types, the type of slope protection shall be determined by taking into consideration the conditions required at each installation site.

Table 7.1.25 Evaluation of Slope Protection Work

Revetment form	Resistance to flow rate > 2.0 m/s	20% Slope Applicability	Durability	Evaluation
1) Sodded Riverbank with Pile Fence	NG	OK	Low	Not recommended
2) Dry Boulder Riprap	OK	OK	Low	Not recommended
3) Grouted Riprap (Spread Type)	OK	OK	middle position It's easy to crack.	Not recommended
4) Grouted Riprap (Wall Type)	OK	NG	middle position It's easy to crack.	Not recommended
5) Gabion (Spread Type)	OK	OK	middle position	OK
6) Gabion (Pile-up type)	OK	NG	middle position	Not recommended
7) Rubble Concrete (Spread Type)	OK	OK	middle position It's easy to crack.	Not recommended
8) Rubble Concrete (Wall Type)	OK	NG	middle position	Not recommended
9) Reinforced Concrete Facing	OK	OK	high	OK

Source: study team rearrangement from DGCS, vol.3, 2015, P5 -31

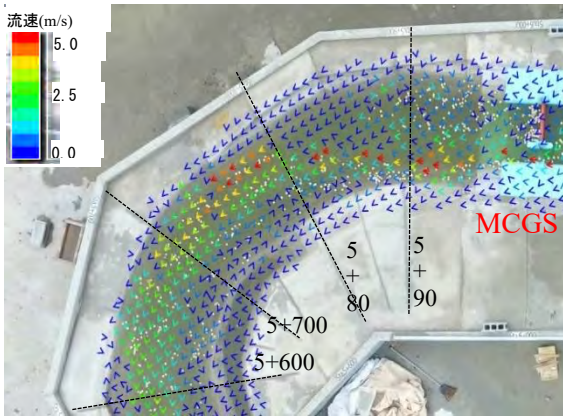
(2) Selection of Slope Protection Work

1) Upstream and Downstream of Weir, Floodgate, and Structures Traversing Dike

In the vicinity of the structure, it is necessary to ensure a sufficient strength no to become the end of the structure a weak point. Therefore, reinforced concrete facing is adopted by protected as the slope protection work around the structure.

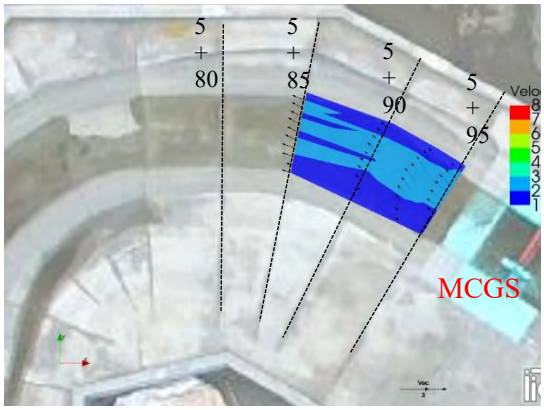
2) Right Bank of Downstream of MCGS

In the downstream of MCGS, complicated flow is generated by flow control. A sill is installed at the downstream of MCGS to stabilize the flow, and its effect has been confirmed through a hydraulic model test (refer to **Figure 7.1.24** and **Figure 7.1.25**). However, the reinforced concrete revetment of the MCGS downstream will be extended to the downstream in consideration of the conditions that it is located on the outer shore side and the flow is running to the right bank straightly.



Source: Study Team

Figure 7.1.24 Flow at the Downstream of MCGS before Sill Installation



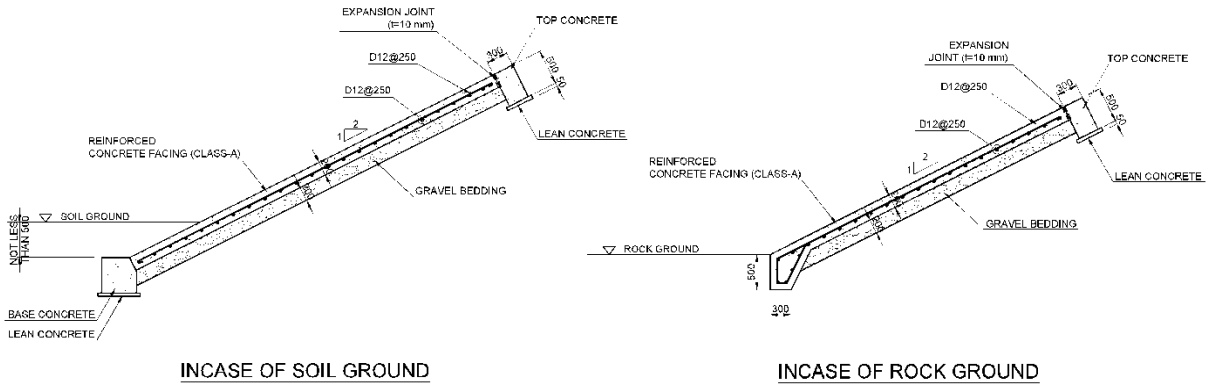
Source: Study Team

Figure 7.1.25 Flow at the Downstream of MCGS after Sill Installation

(3) Detail of Reinforced Concrete Facing

Figure 7.1.26 shows the typical cross-section of reinforced concrete facing. The thickness of reinforced concrete facing shall be 20 cm which is the minimum thickness shown in DGCS Vol. 3. If the ground of foundation is sand gravel, 0.5 m depth of embedding should be ensured. If the foundation is to be embedded in the riverbed, an embedding depth is more than 1.0 m. If the foundation ground or riverbed is rock, the cutoff point shall be integrated with the slope instead of embedding.

In order to prevent cracks due to expansion and contraction of concrete, expansion joints shall be installed at intervals of 1 in 20 m.



Source: Study Team

Figure 7.1.26 Typical Cross-section of Reinforced Concrete Facing

7.1.4 Detailed Design of Slope Protection in front of Existing Revetment（Left Bank, Sta.6+360～Sta6+600）

(1) Current Condition of Existing Revetment

On the left bank side from Sta. 6 + 360 to Sta. 6 + 600, the existing SSP revetment and slope protection is installed. At present, cracks and cave-in parts of the upper slope protection can be confirmed (Refer to **Picture 7.1.1** and **Picture 7.1.2**) However, the revetment can be considered to be an effective state without any fatal deformation such as the inclination to the top edge of the coping concrete or slippage of the slope protection. Based on an existing document, it can be assumed that the design reverbed of

SSP revetment was approximately EL. 10 m.

On the other hand, in this section, the river channel dredging work construction is being carried out, and the original planned was supposed to carry out the dredging work up to the front of the existing revetment. However, due to concerns about the stability of the existing revetment, the plan has been changed and the dredging area doesn't reach the riverside (Hearings with dredging work contractors). At present, no detailed information is available on how far away dredging work is from the existing revetment.



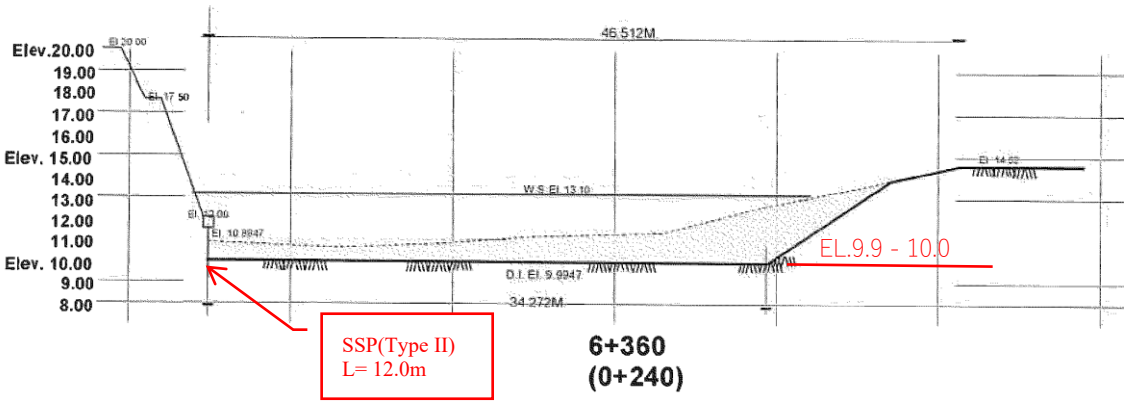
Source: Study Team

Picture 7.1.1 Damage of Slope Protection of Existing Slope (1)



Source: Study Team

Picture 7.1.2 Damage of Slope Protection of Existing Slope (2)



Note : The Type of SSP is supposed from the existing document
Source : DPWH-NCR, IROWR-PMO

Figure 7.1.27 Cross-Section of Existing Revetment

(2) Policy of Countermeasure

Under the current conditions described above, the repair work will be carried out in the following policy,

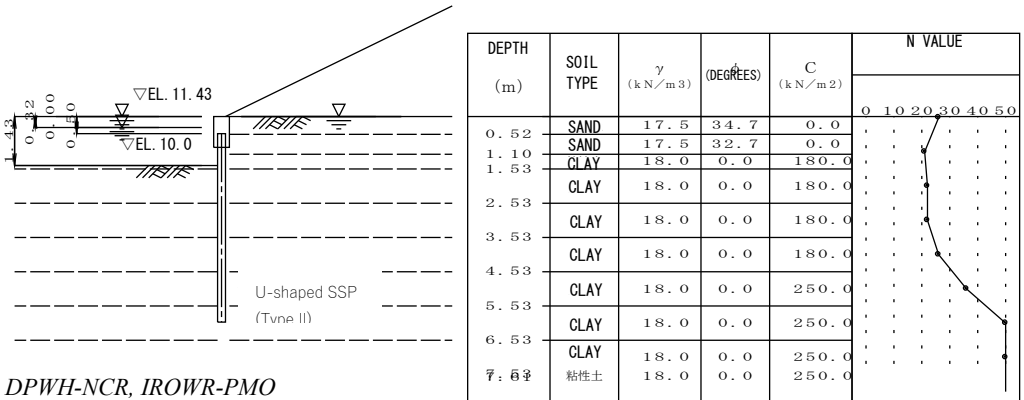
- Remain the existing SSP revetment
- As for the slope protection, remove and re-install it where it is installed by overhanging the maintenance road, described later.
- The foot protection will be installed so that the riverbed height in front of the existing revetment can be maintained at EL 10.0 m.
- The width of the foot protection is determined by the condition that the existing SSP revetment can ensure the required passive resistance.
- The dredging work in front of the foot protection is carried out up to the design riverbed.

(3) Stability Analysis of Existing SSP Revetment

The stability analysis results of the existing SSP revetment were shown below and the stability of the existing SSP is confirmed.

			Normal Condition	Seismic Condition
Inertia sectional moment	I (cm^4)	8740		
Sectional Modulus	Z (cm^3)	874		
Maximum Bending Moment	M_{max}(kN・m/m)		8.09	9.14
Stress Intensity	σ (N/mm^2)		11 (180)	13 (270)
Lateral Displacement	δ (mm)		3.52 (50.0)	3.08 (75.0)
Penetration Depths	D (m)		4.43	3.73
Whole Length of SS	L (m)	5.50		

Note: Values in ( ) means the Allowable Value



Source : DPWH-NCR, IROWR-PMO

Figure 7.1.28 Result of Stability Analysis of Existing SSP Revetment

(4) Examination of Foot Protection

1) Type of Foot Protection

In order to maintain the riverbed height at EL 10.0 m in front of the SSP, it is necessary to install the foot protection in the front of the revetment. On the other hand, the riverbed height in front of the SSP is below the normal water level, and the existing riverbed height is higher than EL. 10.0 m in some places and lower in other places. In addition, the planned dredging work far from the existing revetment will be conducted to the design riverbed. Therefore, it is necessary to protect the slope by the dredging work so that the scouring in front of the existing revetment does not advance from the point of dredging work. Based on the above conditions, polyester net gabions should be installed to protect the slope by the dredging work, not by RIPRAP which is used in other sections.

Polyester net gabion is the next applicable item, following RIPRAP, in the comparison shown in Table 6.1. 22.

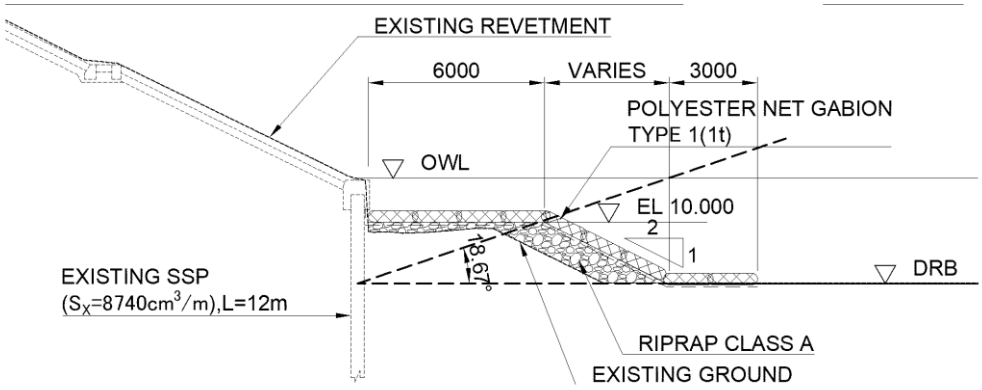
For areas where the existing riverbed is already lower than the EL. 10.0 m, fill using RIPRAP is installed and polyester net gabions are placed on the surface. **Figure 7.1.29** shows the typical cross-section.

2) Dimension of Foot Protection

Based on "6.1.3.6 Examination of the Structure of Foot Protection at the Low Water Channel", the design flow rate of this section is 1.9 m/s. Therefore, the type of polyester net gabions is 1 t type and the RIPRAP type is CLASS A.

3) Width of Foot Protection

In order to maintain the riverbed elevation EL. 10.0 m in front of the existing SSP, the width of the foot protection shall be the width that can ensure passive resistance. As shown in **Figure 7.1.29,** the point where the passive collapse line intersects with the EL. 10.0 m is set at 6.0 m from the design riverbed



Note: Passive collapse angle of Riprap is set to 35 degrees under normal condition.
Source: Study Team

Figure 7.1.29 Setting for Width and Typical Cross-section of Foot Protection

7.1.5 Detailed Design of Ancillary Facilities

(1) Maintenance Road

1) Pavement of Maintenance Road

When the maintenance road is used only for river management officers with the management vehicle, the crushed stone pavement alone is sufficient. However, the maintenance road along the Marikina River will be opened to the public as a walking path, the same as the maintenance road developed in the previous phases. Therefore, concrete pavement is installed at the top of the dike in consideration of the public use of rivers.

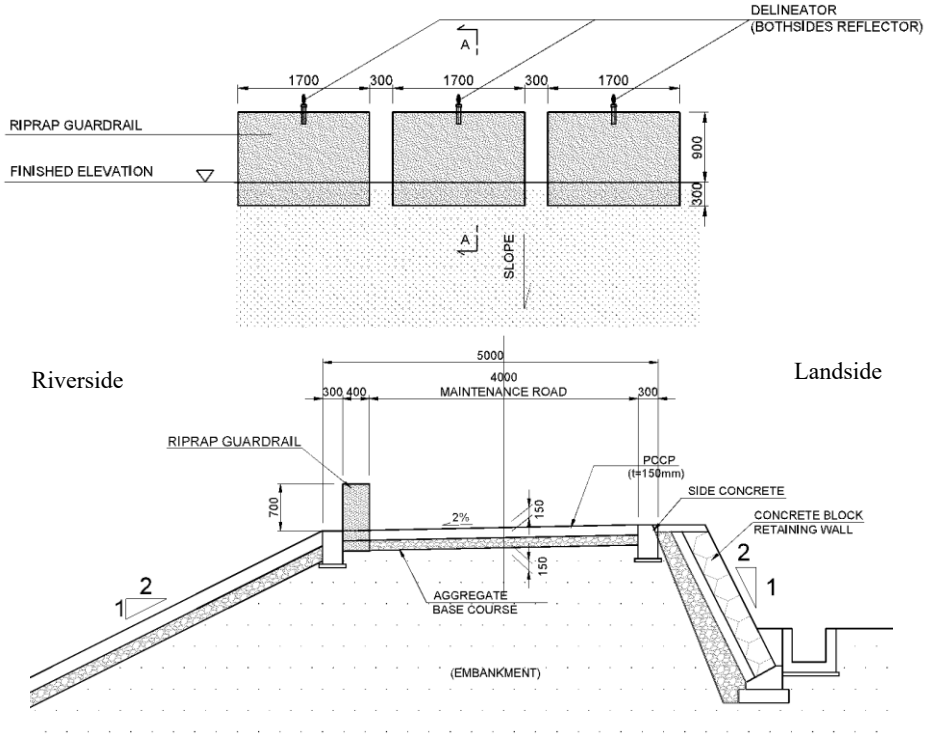
Table 7.1.26 Specification of Pavement for Maintenance Road

Item	Specifications	Reason for setting/Remarks
Type of Pavement	concrete pavement	Consideration for river use
Thickness of Pavement	150 mm	Since there is little vehicle traffic, the minimum thickness, shown by DO No. 11 Series of 2014.
Thickness of Crushed Stone	150 mm	Same thickness as concrete pavement

Source: Study Team

2) Riprap Guardrail (for 20% Slope)

When the slope of maintenance road is low (20% or less), guardrails for safety isn't installed mostly in Japan. However, DPWH-UPMO-FCMC requested guardrails, because vehicles have to run the maintenance road around MCGS and EFCOS under the condition of insufficient visibility at night or in stormy weather. Therefore, Riprap Guardrail shall be installed on the riverside. Delineators shall be also installed for nights.



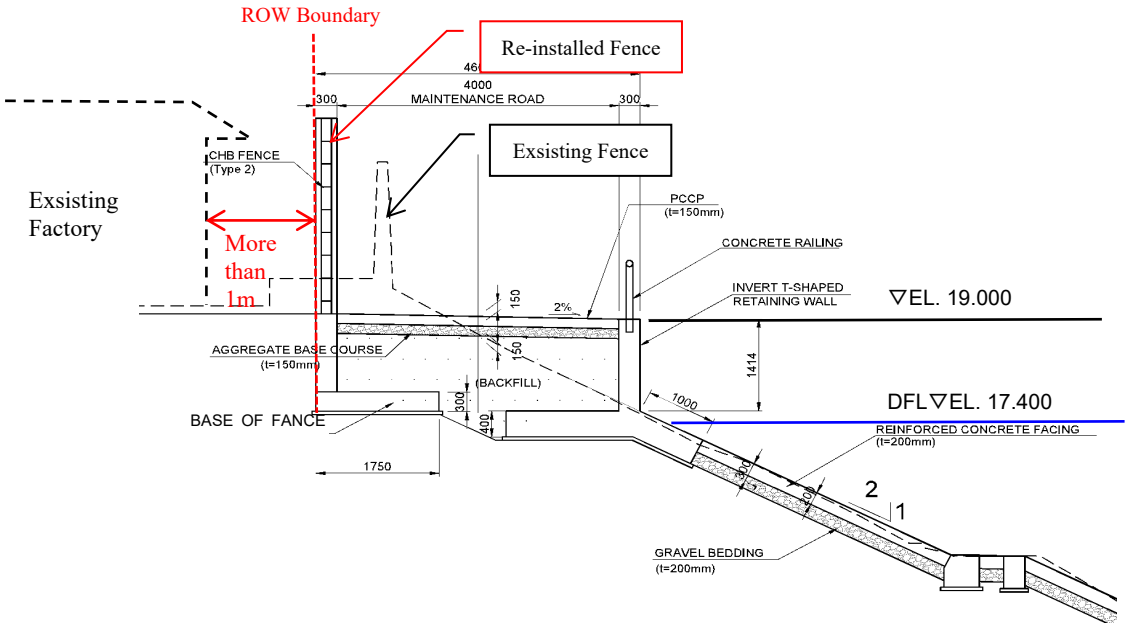
Source: Study Team

Figure 7.1.30 Typical Cross-section of Riprap Guardrail

3) From Sta.6+480 to Sta6+550 on Left Bank

In this section, the distance between the shoulder of existing revetment and the boundary of ROW is approximately 1 m at minimum. Moreover, since a large factory is located behind the boundary, it is difficult to acquire private land and to provide a management passage. However, if there is no maintenance road at this location, EFCOS and MCGS don't connect directly, and it is not appropriate for facility management.

Therefore, in this section, the maintenance road is provided overhanging from the existing revetment slope (**Figure 7.1.31**). Besides, although the existing SSP revetment and slope protection is installed in this section, some cracks and sinking are confirmed. To prevent further damage, by the load acting on the lining due to the installation of the maintenance road, the existing lining shall be removed, and the reinforced concrete lining shall be re-installed.



Source: Study Team

Figure 7.1.31 Typical Cross-section of Maintenance Road on Left Side from Sta.6+480 to 6+550

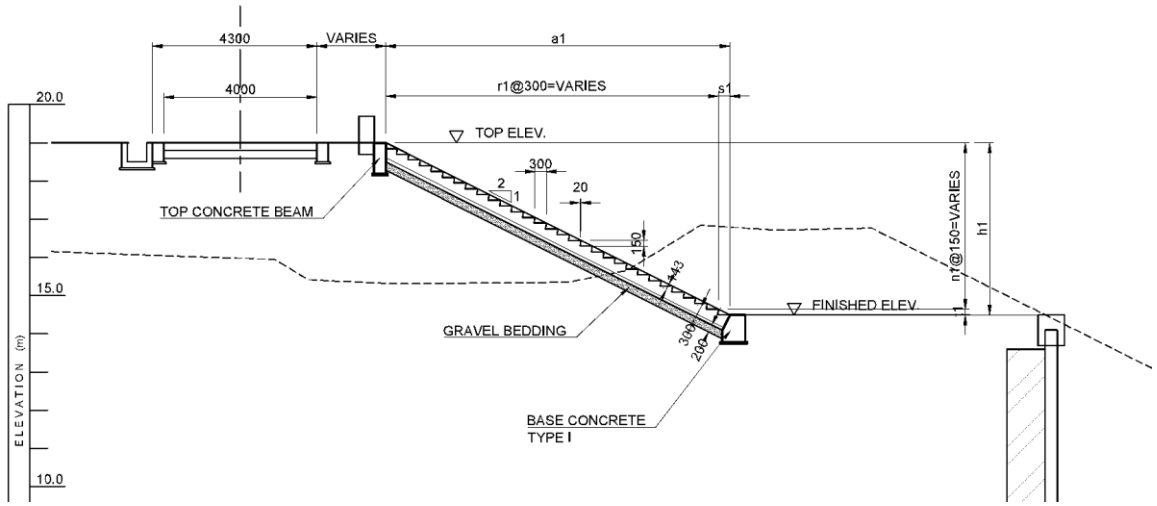
(2) Stairs

For slopes less than 20%, stairs shall be installed for river access. **Table 7.1.27** shows a list of stairs. The interval of the installation of the stair was approximately 100 m, referring to the interval of existing stairs near EFCOS.

Table 7.1.27 List of Stairs Installation

Package	Right bank/Left bank	STA.	Purpose, reasons for installation, etc.
CP-1	Right Bank	5+650	Downstream end of the right bank For river and revetment patrols
		5+710	For river and revetment patrols
		5+790	〃
		5+892	〃
		5+970	MCGS downstream For river and revetment patrols
	Left Bank	5+789	Left bank side revetment end For river and revetment patrols
		5+970	MCGS downstream For river and revetment patrols
		6+040	For river and revetment patrols
		6+120	〃
		6+220	〃
		6+320	〃
		6+475	Same location as the existing stairs For river and revetment patrols

Source: Study Team



Source: Study Team

Figure 7.1.32 Cross-section of Stairs (Sta.6+120, Left Bank)

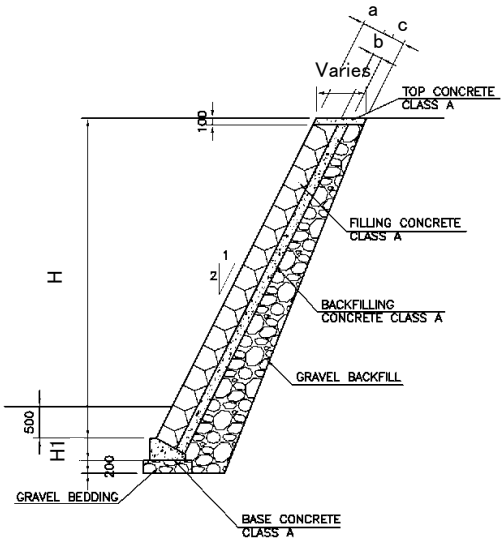
(3) Concrete Block Retaining Wall

1) Location

Concrete block retaining walls shall be installed at places where the landside slope is required to have 60 % slope due to site restrictions, etc.

2) Specification

Figure 7.1.33 and **Table 7.1.28** show the Typical cross-section and specification of concrete block retaining walls. The class of concrete shall be Class A based on the standard specifications of DPWH (ITEM 1728). In consideration of the use of non-reinforced concrete, the expansion joints are installed at intervals of 10 m.



Source: Study Team

Figure 7.1.33 Cross-section of Concrete Block Retaining Wall

Table 7.1.28 Specification of Concrete Block Retaining Wall

Height H (m)	Base Height H1 (mm)	Thickness of Block a (mm)	Thickness of Backfilling Concrete b (mm)	Thickness of Gravel Backfill c (mm)
1.50 - 3.00	300	350	100	200
3.00 - 5.00	350	350	150	200
5.00 - 7.00	400	350	200	200

Source: Study Team

(4) Channel at Foot and Ditch

Channel at the foot of dike and ditch will be installed to drain surface water from the dike and hinterland. In the section where the dike isn't developed, ditch is installed behind the maintenance road, if the following condition is confirmed.

- All or part of the slope on riverside is protected by grass sodding
  - Especially, there is a risk that the slope may be eroded by surface water flowing into the river from the hinterland until vegetation is established.
- There is a concrete wall in riverside and surface water cannot be drained directly to the riverside.

7.2 Detailed Design of Drainage Facility

7.2.1 Summary

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

7.2.2 Detailed Design of Outlet

7.2.2.1 Summary of Proposed Outlets

The summary of new proposed outlets is tabulated as below table. The size of proposed outlet is set in accordance with Guidelines, the design condition and design concept which are describe in section of Basic Design of Drainage Facility.

Table 7.2.1 The Draft Proposed Drainage Facility

Sr. No.	Bank	Station No.	Dia. or Size (Height x Width) (mm)	Invert Elevation	Sr. No.	Bank	Station No.	Dia. or Size (Height x Width) (mm)	Invert Elevation
ML104.5	LEFT	5+400	900	13.47	MR136	RIGHT	5+962	900	11.46
ML104.4	LEFT	5+700	900	12.94	MR124A	RIGHT	6+111	900	12.40
ML104.3	LEFT	5+800	900	13.17	MR124	RIGHT	6+114	500	14.61
ML104.2	LEFT	6+160	900	12.40	MR122	RIGHT	6+120	900	12.40
ML104.1	LEFT	6+330	900	12.40	MR120	RIGHT	6+170	1200	12.08
ML103	LEFT	6+500	900	15.59	MR116	RIGHT	6+202	1200	12.08
ML102	LEFT	6+509	900	13.41	MR115	RIGHT	6+214	300	13.10
ML101	LEFT	6+513	900	15.59	MR090	RIGHT	6+952	1050	12.24
ML100	LEFT	6+541	900	15.59	MR071	RIGHT	7+523	900	11.50
ML099	LEFT	6+758	1800	9.78	MR069	RIGHT	7+732	900	11.10
ML098	LEFT	7+178	1500	10.35	MR067.1	RIGHT	7+779	900	11.10
ML097A	LEFT	7+216	1000X1000	10.30	MR065	RIGHT	7+890	300	13.21
ML097.4	LEFT	7+406	1200	10.68	MR064	RIGHT	7+912	300	13.31
ML097.3	LEFT	7+658	1200	10.78	MR061	RIGHT	7+960	300	14.82
ML073.3	LEFT	8+137	1800	11.58	MR059	RIGHT	7+993	900	12.70
ML073.2	LEFT	8+245	900	12.80	MR058.1	RIGHT	8+128	900	12.52
ML073.1	LEFT	8+341	900	12.80	MR057	RIGHT	8+199	900	12.70
ML073A	LEFT	8+446	900	12.44	MR056	RIGHT	8+245	900	12.73
ML072A	LEFT	8+611	900	12.80	MR055	RIGHT	8+312	1500	12.15
ML071A	LEFT	8+791	900	12.40	MR054	RIGHT	8+328	900	12.80
ML070A	LEFT	8+848	600	12.09	MR053	RIGHT	8+443	900	12.80
ML069	LEFT	9+002	1440X1310	13.13	MR048	RIGHT	8+492	900	12.80
ML068.1	LEFT	9+177	900	11.66	MR047	RIGHT	8+507	900	12.80
ML066.4	LEFT	9+785	1050	12.94	MR047.1	RIGHT	8+521	900	12.80
ML066A	LEFT	9+896	3150X3080	11.72	MR043.39	RIGHT	8+563	1200	12.25
ML066.101	LEFT	9+899	900	12.40	MR032.501	RIGHT	8+603	1200	11.58
ML065A	LEFT	10+064	1050	12.84	MR030	RIGHT	8+665	900	11.30
ML064.2	LEFT	10+169	900	11.90	MR029	RIGHT	8+671	900	11.30
ML064.1	LEFT	10+266	900	11.86	MR028	RIGHT	8+674	900	11.30
ML062.3D	LEFT	10+345	1200	12.78	MR027	RIGHT	8+684	900	11.30
ML062A	LEFT	10+361	3X1200	11.87	MR025.1	RIGHT	9+120	1800	10.38
ML061.4	LEFT	10+405	900	13.30	MR025	RIGHT	9+202	2000X2000	10.15
ML061.3	LEFT	10+506	900	12.73	MR024	RIGHT	9+291	2000X2000	10.15
ML061.2	LEFT	10+607	900	11.60	MR023	RIGHT	9+453	900	13.00
ML058.2	LEFT	11+634	900	13.10	MR022	RIGHT	9+471	900	12.60
ML050.2	LEFT	11+782	900	12.63	MR021.3	RIGHT	9+569	900	12.32
ML022A	LEFT	12+011	1050	12.03	MR021.301	RIGHT	9+570	900	10.79
ML021.1	LEFT	12+052	1800X1800	12.27	MR021.302	RIGHT	9+571	900	12.45
ML020	LEFT	12+146	2x900	12.40	MR021.2	RIGHT	9+638	900	12.32
ML019	LEFT	12+251	2x1200	12.83	MR021.1	RIGHT	9+774	1050	12.94
ML018	LEFT	12+365	900	11.70	MR021	RIGHT	9+840	900	11.59
ML017.2	LEFT	12+388	900	12.10	MR020	RIGHT	9+886	900	11.78
ML017.1	LEFT	12+539	2000X2500	10.65	MR019	RIGHT	9+903	900	11.61
ML017	LEFT	12+542	900	11.96	MR018	RIGHT	10+221	3600x3200	12.85
ML016	LEFT	12+572	900	12.10	MR017	RIGHT	10+292	1500	11.29
ML015	LEFT	12+668	900	12.20	MR015.2	RIGHT	10+375	900	10.71
ML013	LEFT	12+764	900	12.20	MR015.1	RIGHT	10+414	900	10.64
ML012.3	LEFT	12+823	900	12.50	MR015.101	RIGHT	10+536	900	10.36
ML012.201	LEFT	12+878	900	11.80	MR015A	RIGHT	10+569	900	11.70
ML012.2	LEFT	12+942	900	12.20	MR014	RIGHT	10+610	900	11.42
ML012.1	LEFT	12+989	1200	10.36	MR010	RIGHT	10+726	1360X2000	11.01
ML012	LEFT	13+037	900	11.80	MR009.1	RIGHT	10+804	1800	10.48
ML011.1	LEFT	13+073	900	12.20	MR004.3	RIGHT	13+010	900	12.16
ML009	LEFT	13+151	1500X1500	11.55	MR004.1	RIGHT	13+033	300	10.96
ML005	LEFT	13+206	1000X1000	11.76	MR004A	RIGHT	13+046	1200	10.94
ML004	LEFT	13+261	900	11.73	MR003.105	RIGHT	13+075	300	10.81
MR151C	RIGHT	5+513	900	10.40	MR003.10	RIGHT	13+128	900	12.31
MR151D	RIGHT	5+524	900	10.60	MR003.5	RIGHT	13+158	300	10.50
MR151	RIGHT	5+788	900	10.70	MR003.1	RIGHT	13+215	900	11.92
MR147A	RIGHT	5+852	300	11.33	MR003	RIGHT	13+236	900	11.41
MR144	RIGHT	5+863	900	10.70	MR002A	RIGHT	13+283	900	11.86
MR141	RIGHT	5+907	300	11.38	MR001A	RIGHT	13+327	900	11.61
MR137	RIGHT	5+955	900	11.46					

Source: Study Team

7.2.2.2 Detailed Design of Drainage Outlet Facility

(1) General

1) Outlet

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

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

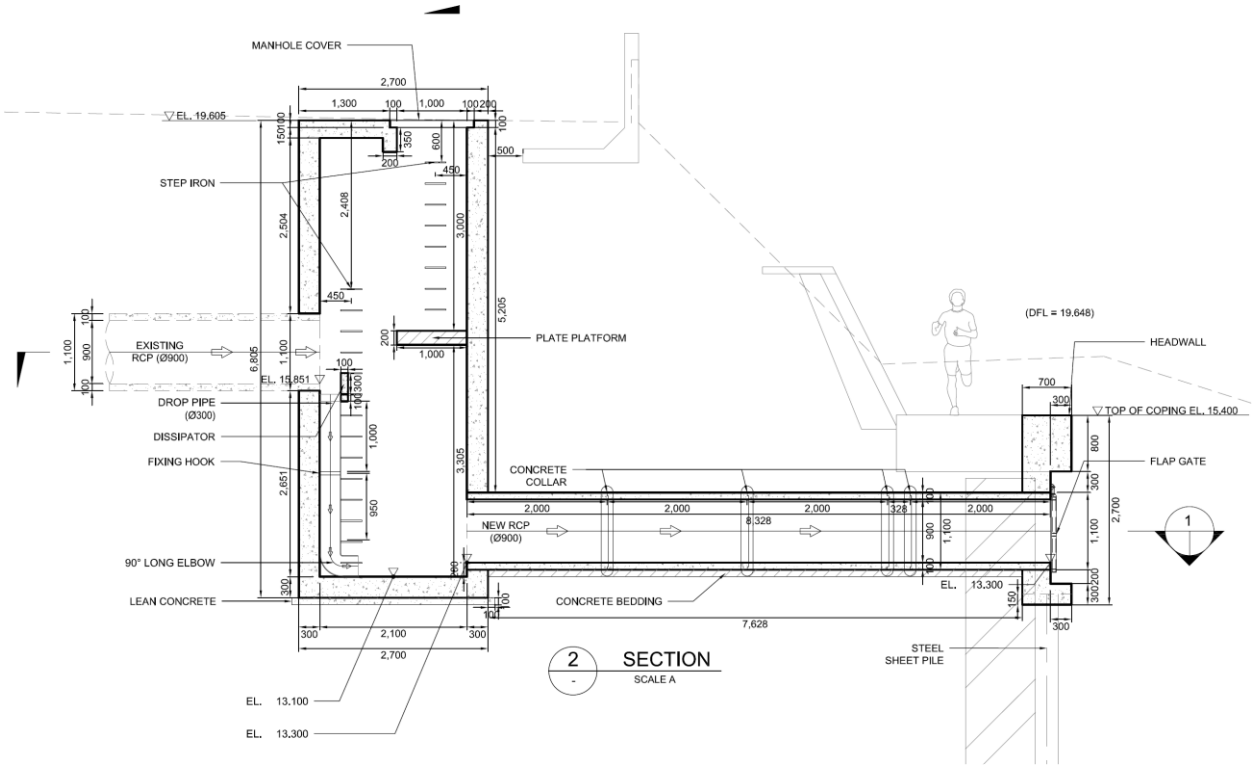
2) Manhole

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

In case that the manhole becomes deeper, a platform will be installed in consideration of the safety aspects of maintenance. Since there are no provisions for this platform in the DPWH Guideline, it is desirable to set up the platforms every 3.0 to 5.0 m in accordance with "Sewerage Facility Planning / Design Guidelines" (Japan Sewerage Association). Furthermore, there is provision what it should have the height space at least 2.0 m from bottom invert elevation of manhole to installed platform for maintenance works. Therefore, in this project, in case that the depth of manhole will be more than 5 m, every 3 m the platform will be set up in the manhole.

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

The general drawing of the Outlet having the above structure (1) and (2) is shown below.



Source: Study Team

Figure 7.2.1 Rainfall Intensity-Duration-Frequency Curves

(2) Structural Study on Manholes

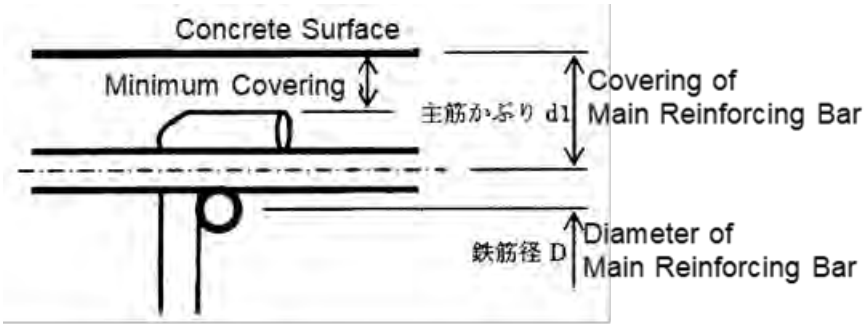
1) Concrete Cover

Minimum covering depth of reinforcing bar should be not less than 75mm for bottom slab and not less than 50mm for other members. To secure the minimum covering mentioned above, covering of main reinforcing bar (distance from concrete surface to center of main reinforcing bar) should be not less than the values shown in the table below.

Table 7.2.2 Minimum Coverage of Reinforcement Bars

Covering Item	Sign in Fig.	Top Slab	Side Wall, Bottom Slab
Minimum Covering of Reinforcing Bar	d_0	50mm	75mm
Covering of Main Reinforcing Bar (Concrete Surface to Centre of Main Reinforcing Bar)	d_1	75mm	100mm

Source: Study Team



Source: Study Team

Figure 7.2.2 Explanation of Covering of Main Reinforcing Bar

Minimum thickness of concrete members should be more than the values shown in the table below, to secure the covering of main reinforcing bar.

Table 7.2.3 Minimum Thickness of Slab

	Top Slab	Bottom Slab, Side Wall
Minimum Thickness	250mm (75mm x 2 +100mm)	300mm (100mm x 2 +100mm)

Source: Study Team

2) Conditions for Structural Calculation

Conditions for structural calculation are as follows.

(a) Condition

Materials	f'_{ck} (N/mm^2)	E_c 10^4 (N/mm^2)
20.7	20.7	2.380

Unit weight (Reinforced concrete) $\gamma c = 24.50$ (kN/m^3)
Unit weight (Reinforced concrete (submerged)) $\gamma c' = 14.50$ (kN/m^3)

(b) Specification of Reinforcing Bar

Grade	F_{yk} (N/mm^2)	$E_s \times 10^5$ (N/mm^2)
275	275	2.000

(c) Groundwater Level

Depth (H): 0.0 (m) from ground surface
Unit weight of water: 10.0 (kN/m^3)

(d) Conditions for Earth Pressure

Depth Z (m)	Unit weight (Wet) γ (kN/m^3)	Unit weight (saturated) γ_{sat} (kN/m^3)	Coefficient for earth pressure at rest K	Coefficient for vertical pressure α
10.000	18.000	20.000	0.5000	1.0000

(e) Live Load

Considering the usage of normal vehicles, following conditions are applied as live load.

Vehicle: T-14, 55.0 (kN)

Impact Coefficient: 0.300

Uniform Load: 10.0 (kN/m^2)

(f) Categorizing and Selection of Calculation Model Type

Manholes designed in this project can be categorized into several types of calculation models based on similarity in proportion, as all manholes have no soil coverage so that difference of load conditions is ignorable. Following tables explain categorization of all manholes and determination of dimensions of each calculation model in consideration of maximum sizes of manholes categorized.

Table 7.2.4 Grouping of Manholes and Selection of Calculation Model Type (1/2)

Name of MH	Values from Drawings				Dimensions of Calc. Model				Model Type
	Vd1 (mm)	Vd2 (mm)	H (mm)	Opening w (mm)	Vd1 (mm)	Vd2 (mm)	H (mm)	w (mm)	
MR141	1200	1200	1470	300	1500	2400	2000	300	a
MR124A	2000	1400	1650	900	1500	2400	2000	900	a
MR147A	1200	1200	1971	300	1500	2400	2000	300	a
ML104.5	1500	2350	2000	900	1500	2400	2000	900	a
MR015A	1200	1400	2248	900	1200	1400	3000	900	b
ML058.2	1200	1400	2300	900	1200	1400	3000	900	b
ML018	1200	1400	2300	900	1200	1400	3000	900	b
ML015	1200	1400	2300	900	1200	1400	3000	900	b
ML013	1200	1400	2300	900	1200	1400	3000	900	b
ML012.2	1200	1400	2300	900	1200	1400	3000	900	b
ML012	1200	1400	2300	900	1200	1400	3000	900	b
ML011.1	1200	1400	2300	900	1200	1400	3000	900	b
MR151C	1200	1400	2300	900	1200	1400	3000	900	b
ML017.2	1200	1400	2400	900	1200	1400	3000	900	b
ML016	1200	1400	2400	900	1200	1400	3000	900	b
ML012.201	1200	1400	2488	900	1200	1400	3000	900	b
MR003.10	1200	1400	2493	900	1200	1400	3000	900	b
ML017	1200	1400	2545	900	1200	1400	3000	900	b
ML050.2	1200	1400	2674	900	1200	1400	3000	900	b
MR136	1200	1400	2700	900	1200	1400	3000	900	b
ML004	1200	1400	2774	900	1200	1400	3000	900	b
MR137 & MR138	1200	1435	2782	900	1200	1400	3000	900	b
ML012.3	1200	1400	2801	900	1200	1400	3000	900	b
MR021.302	1200	1400	2948	900	1200	1400	3000	900	b
MR003.1	1200	1400	2976	900	1200	1400	3000	900	b
MR021.3	1200	1400	3080	900	1200	1400	3000	900	b
ML097A	1200	2300	2751	1000	1200	2600	3000	1000	c
ML022A	1200	2580	2967	1050	1200	2600	3000	1050	c
MR021.1	1200	1580	2915	1050	1500	2100	3000	1050	d
ML005	1200	1700	2740	1000	1500	2100	3000	1000	d
MR144	1200	1825	2320	900	1500	2100	3000	900	d
ML104.2	1500	2050	2891	900	1500	2100	3000	900	d
ML104.3	1500	2050	2968	900	1500	2100	3000	900	d
ML104.4	1500	2050	2993	900	1500	2100	3000	900	d
MR023	1200	1400	3134	900	1200	1400	4000	900	e
MR004.3	1200	1400	3239	900	1200	1400	4000	900	e
MR002A	1200	1400	3272	900	1200	1400	4000	900	e
MR021.2	1200	1400	3309	900	1200	1400	4000	900	e
MR022	1200	1400	3413	900	1200	1400	4000	900	e
MR003	1200	1400	3591	900	1200	1400	4000	900	e
ML100	1200	1400	3643	900	1200	1400	4000	900	e
MR001A	1200	1400	3794	900	1200	1400	4000	900	e
ML103	1200	1400	3817	900	1200	1400	4000	900	e
ML101	1200	1400	3817	900	1200	1400	4000	900	e
ML064.1	1200	1400	3959	900	1200	1400	4000	900	e
MR090	1000	1580	3203	1050	1200	2000	4000	1050	f
ML065A	1200	1580	3779	1050	1200	2000	4000	1050	f
ML062.3D	1200	1750	3122	1200	1200	2000	4000	1200	f
ML061.2	1200	2000	3700	900	1200	2000	4000	900	f
ML087.1	1200	2300	3256	1000	2000	2500	4000	1000	g
MR009.1	1200	2450	3600	1800	2000	2500	4000	1800	g
MR025.1	1200	2450	3715	1800	2000	2500	4000	1800	g
ML087.2	1200	2500	3649	1000	2000	2500	4000	1000	g
ML087A	1200	2500	3717	1000	2000	2500	4000	1000	g
ML087C	1200	2500	3717	1000	2000	2500	4000	1000	g
ML087D	1200	2500	3773	1000	2000	2500	4000	1000	g
MR125A	1400	2200	3525	1000	2000	2500	4000	1000	g
ML104.1	1500	2050	3182	900	2000	2500	4000	900	g
ML009	1500	2100	3115	1500	2000	2500	4000	1500	g
MR024	2000	2000	3857	1400	2000	2500	4000	1400	g

Source: Study Team

Table 7.2.5 Grouping of Manholes and Selection of Calculation Model Type (2/2)

Name of MH	Values from Drawings				Dimensions of Calc. Model			
	Vd1 (mm)	Vd2 (mm)	H (mm)	Opening w (mm)	Vd1 (mm)	Vd2 (mm)	H (mm)	w (mm)
MR003.5	1200	1200	4103	300	1200	1400	5000	300
MR003.105	1200	1200	4189	300	1200	1400	5000	300
MR004.1	1200	1200	4341	300	1200	1400	5000	300
MR151D	1200	1400	4415	900	1200	1400	5000	900
ML064.2	1200	1400	4418	900	1200	1400	5000	900
MR021.301	1200	1400	4596	900	1200	1400	5000	900
MR020	1200	1400	4637	900	1200	1400	5000	900
MR015.2	1200	1400	4886	900	1200	1400	5000	900
ML012.1	1200	1750	4138	1200	1200	2100	5000	1200
MR004A	1200	1750	4265	1200	1200	2100	5000	1200
MR059	1200	2130	4947	900	1200	2100	5000	900
MR073	1200	2340	4322	1000	2000	2700	5000	1000
MR072.1	1200	2340	4482	1000	2000	2700	5000	1000
ML099	1200	2450	4970	1800	2000	2700	5000	1800
ML087B	1200	2500	4298	1000	2000	2700	5000	1000
MR074	1200	2500	4877	1000	2000	2700	5000	1000
MR047	1200	2600	4977	900	2000	2700	5000	900
MR047.1	1200	2600	4977	900	2000	2700	5000	900
ML091.1	1200	2700	4211	1200	2000	2700	5000	1200
MR067A	1200	2700	4400	1200	2000	2700	5000	1200
ML017.1	2000	2500	4917	2000	2000	2700	5000	2000
MR025	2000	2700	4894	2000	2000	2700	5000	2000
ML061.3	900	1400	5979	900	1200	1400	6000	900
MR014	1000	1400	5663	900	1200	1400	6000	900
MR115	1200	1200	5230	300	1200	1400	6000	300
MR015.101	1200	1400	5040	900	1200	1400	6000	900
ML066.101	1200	1730	5106	900	2000	1400	6000	900
MR048	1200	2550	5011	900	2000	1400	6000	900
MR021	2000	1400	5188	900	2000	1400	6000	900
MR151	2000	1400	5290	900	2000	1400	6000	900
MR019	2000	1400	5315	900	2000	1400	6000	900
MR058.1	2000	1400	5788	900	2000	1400	6000	900
MR071	2000	1400	5905	900	2000	1400	6000	900
MR056	2000	1400	5979	900	2000	1400	6000	900
ML102	2000	1400	5994	900	2000	1400	6000	900
MR053	2000	1400	6001	900	2000	1400	6000	900
MR072	2000	2300	5385	1000	2100	2600	6000	1000
MR057	2000	2600	5464	900	2100	2600	6000	900
MR054	2100	1400	5568	900	2100	2600	6000	900
ML066.4	2100	1580	5528	1050	2100	2600	6000	1050
MR122	2100	1700	5916	900	2100	2600	6000	900
MR009A	2100	2300	5813	1000	2100	2600	6000	1000
MR069	2000	1400	6274	900	2100	1800	7000	900
MR116	2100	1750	6239	1200	2100	1800	7000	1200
MR043.39	2100	1750	6398	1200	2100	1800	7000	1200
MR032.501	2100	1750	6511	1200	2100	1800	7000	1200
ML068B	2000	2300	6384	1000	2100	2300	7000	1000
ML068A	2000	2300	6823	1000	2100	2300	7000	1000
ML068D	2000	2300	6905	1000	2100	2300	7000	1000
ML068C	2000	2300	6944	1000	2100	2300	7000	1000
MR120	2100	2050	6239	1200	2100	2300	7000	1200
ML068	2100	2300	6905	1000	2100	2300	7000	1000
ML072A	2100	2500	6100	900	1200	3000	7000	900
MR055	2100	2700	6268	1500	1200	3000	7000	1500
ML061.4	2100	2800	6505	900	1200	3000	7000	900
ML097.3	1200	2950	6393	1200	1200	3000	7000	1200
MR067.1	2000	1400	7306	900	2100	1400	8000	900
MR030	2000	1400	7634	900	2100	1400	8000	900
MR027	2000	1400	7646	900	2100	1400	8000	900
MR029	2100	1400	7646	900	2100	1400	8000	900
MR028	2100	1400	7646	900	2100	1400	8000	900
ML073.3	2100	3130	7123	1800	2100	3200	8000	1800
ML098	2100	3220	7687	1500	2100	3200	8000	1500
ML097.4	2100	2800	7945	1200	2100	3200	8000	1200
ML020 & ML021	1200	3200	2552	2x900	1200	2600	3000	2x900
ML019 & ML019.1	1200	3800	4697	2X1200	2000	4150	6000	2X1200
MR007 & MR008	2000	4150	5362	2x1200	2000	4150	6000	2x1200
ML062A ML062.1 M	1200	5850	4785	3X1200	1200	5850	5000	3X1200

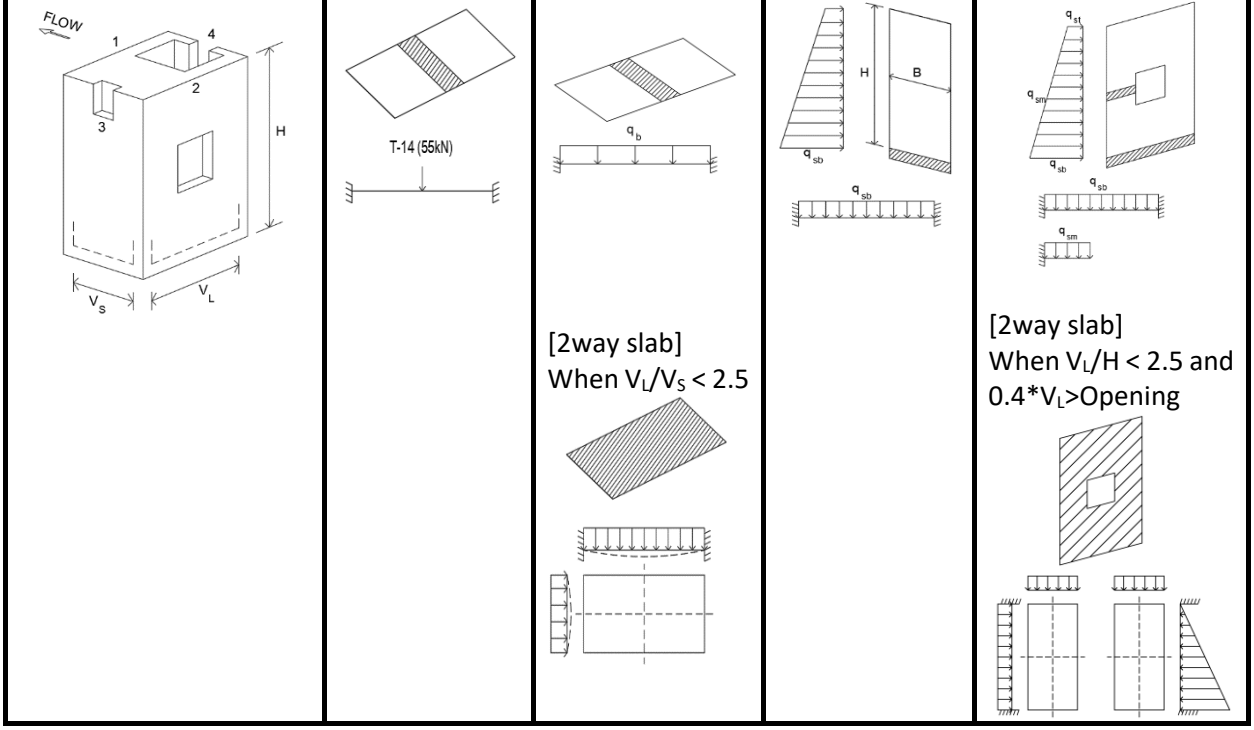
Source: Study Team

(g) Analysis Method of Manholes

In structural analysis, each member of manholes, namely top slab, bottom slab and side walls, can be calculated as an independent slab element. Slab analysis method can be selected considering proportion and support conditions of each member.

Table 7.2.6 Adopted Slab Analysis Method by Members

Calc. Type	Dimension of Calculation Model						Calculation Method by Member											
	Vd(1)	Vd(2)	H	Opening		GL-WL	Top Slab			Bottom Slab			Side Slab (Short Side)			Side Slab (Long Side)		
	(mm)	(mm)	(mm)	width (mm)	height (mm)	(mm)	Short side	Long side	Method	Short side	Long side	Method	B	H	Method	B	H	Method
a	1,500	2,400	2000	300	300	1,162	1500	2400	2 side	1500	2400	4 side	1500	2000	4 side	2400	2000	4 side
b	1,200	1,400	3000	900	900	757	1200	1400	2 side	1200	1400	4 side	1200	3000	2 side	1400	3000	2 side
c	1,200	2,600	3000	1000	1000	616	1200	2600	2 side	1200	2600	4 side	1200	3000	2 side	2600	3000	4 side
d	1,500	2,100	3000	1050	1050	933	1500	2100	2 side	1500	2100	4 side	1500	3000	4 side	2100	3000	4 side
e	1,200	1,400	4000	900	900	1,141	1200	1400	2 side	1200	1400	4 side	1200	4000	2 side	1400	4000	2 side
f	1,200	2,000	4000	1050	1050	1,214	1200	2000	2 side	1200	2000	4 side	1200	4000	2 side	2000	4000	2 side
g	2,000	2,500	4000	1000	1000	616	2000	2500	2 side	2000	2500	4 side	2000	4000	4 side	2500	4000	4 side
h	1,200	1,400	5000	300	300	986	1200	1400	2 side	1200	1400	4 side	1200	5000	2 side	1400	5000	2 side
i	1,200	2,100	5000	1200	1200	1,274	1200	2100	2 side	1200	2100	4 side	1200	5000	2 side	2100	5000	2 side
j	2,000	2,700	5000	1000	1000	800	2000	2700	2 side	2000	2700	4 side	2000	5000	2 side	2700	5000	4 side
k	1,200	1,400	6000	900	900	1,744	1200	1400	2 side	1200	1400	4 side	1200	6000	2 side	1400	6000	2 side
l	2,000	1,400	6000	900	900	1,508	1400	2000	2 side	1400	2000	4 side	2000	6000	2 side	1400	6000	2 side
m	2,100	2,600	6000	1000	1000	1,700	2100	2600	2 side	2100	2600	4 side	2100	6000	2 side	2600	6000	2 side
n	2,100	1,800	7000	900	900	2,005	1800	2100	2 side	1800	2100	4 side	2100	7000	2 side	1800	7000	2 side
o	2,100	2,300	7000	1000	1000	1,382	2100	2300	2 side	2100	2300	4 side	2100	7000	2 side	2300	7000	2 side
p	1,200	3,000	7000	900	900	884	1200	3000	2 side	1200	3000	2 side	1200	7000	2 side	3000	7000	2 side
q	2,100	1,400	8000	900	900	2,012	1400	2100	2 side	1400	2100	4 side	2100	8000	2 side	1400	8000	2 side
r	2,100	3,200	8000	1800	1800	945	2100	3200	2 side	2100	3200	4 side	2100	8000	2 side	3200	8000	2 side
s	1,200	2,600	3000	2x900	2x900	1,685	1200	2600	2 side	1200	2600	4 side	1200	3000	2 side	2600	3000	4 side
t	2,000	4,150	6000	2X1200	2X1200	1,301	2000	4150	2 side	2000	4150	4 side	2000	6000	2 side	4150	6000	4 side
u	1,200	5,850	5000	3X1200	3X1200	1,478	1200	5850	2 side	1200	5850	2 side	1200	5000	2 side	5850	5000	4 side



Source: Study Team

Table 7.2.7 Adopted Slab Analysis Method and Reasons

Member	Adopted Analysis Method and Reasons
Top Slab	[Simple beam] Since one or two sides of top slabs are occupied by opening, it shall NOT be considered as 4 side supported. Therefore, regardless that the long-short ratio, top slabs will be calculated as simple beam.
Bottom Slab	[2way slab / Simple beam] If the long-short ratio of bottom slab is not less than 2.5, it can be calculated as 2way slab. If not, it shall be regarded as simple beam.
Side Wall 1,2 (left and right)	[Beam (Both ends fixed)] Since the long-short ratio of top slabs are not more than 2.5 in all calculation types except type a, it shall be calculated as beam (both ends fixed).
Side Wall 3,4 (front and back)	[2way slab / Simple beam] If the long-short ratio of bottom slab is not less than 2.5 and opening width is not more than 0.4 of side length, it can be calculated as 2way slab. If not, it shall be regarded as a beam with both ends fixed, and beam (cantilever) at openings.

Source: Study Team

3) Summary of Load Calculation

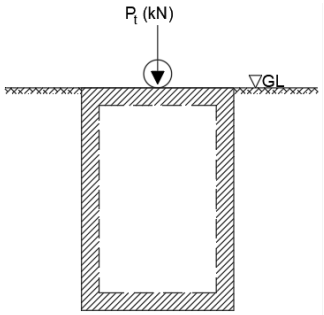
(a) Top Slab

Rear wheel load charged to manhole top slabs is considered as uniform load with soil coverage. However, in this project all manhole top slabs are exposed to road surface so that point load P can be loaded directly as rear wheel load.

Rear wheel Load :

$$P = \frac{(2 \text{ x Rear wheel (kN)})}{\text{Width of vehicle x } (1 + \text{Impact factor})}$$

$$= (2 \text{ x } 55) / 2.75 \text{ x } (1+0.30) = 52.0 \text{ kN}$$



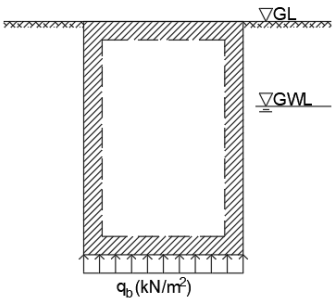
(b) Bottom Slab

For bottom slab, uniform load of ground reaction force shall be applied while sidewalls are functioned as perimeter support. Ground reaction force is obtained by summation of self-weight, soil coverage and live load at top slab by following equation.

$Wb = (W+Ws)/A+Pv1$

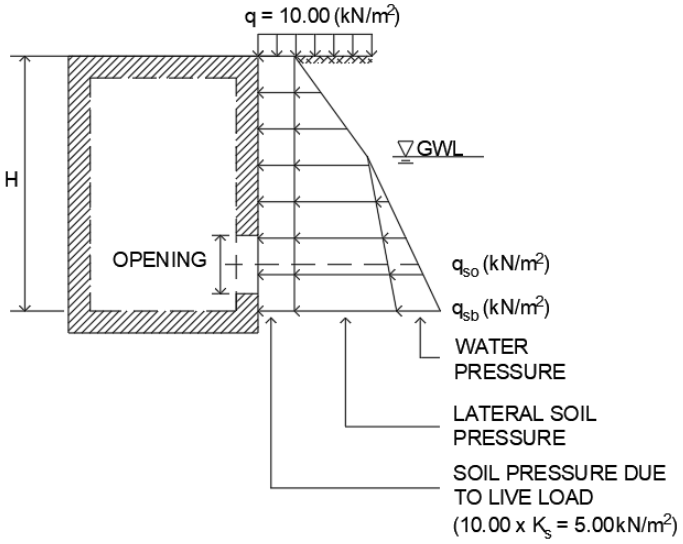
Where,

W : Self-Weight
Ws : Weight of Soil Coverage (=0.0(kN/m^2))
A : Area of Bottom Slab (m)
Pv1 : Live Load at Ground Surface(=10.0(kN/m^2))



(c) Side Wall

Lateral soil pressure and ground water pressure shall be considered for side walls. Effect of traffic load at ground surface shall be added to lateral soil pressure as shown in **Figure 7.2.3**. Since manholes are located at back of river dike, residual water level (RWL) is applied as ground water level.



Source: Study Team

Figure 7.2.3 Live load charged to Side Wall

(d) Middle Slab

Middle slab, or platform is constructed for maintenance staff to access safely to bottom floor of manhole. Thus, pedestrian Load of 5kN/m^2 shall be considered for middle slab.

4) Summerly of Calculation Results and Bar Arrangement

Summerly of calculation results are shown in the table below. Details of structural calculations of each calculation model type are shown in Appendix.

Table 7.2.8 Summary of Bar Schedule

Calc. Type	Thickness			Bar Schedule								
	Top Slab ts (mm)	Bottom Slab tb (mm)	Side Wall tw (mm)	Top Slab		Bottom Slab		Side Wall				
				Top Bar	Bottom Bar	Top Bar	Bottom Bar	Horizontal		Vertical		Shear Reinforcing Bar
								Outer Bar	Inner Bar	Outer Bar	Inner Bar	
a	250	300	300	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
b	250	300	300	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	-
c	250	300	300	D12 @250	D12 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
d	250	300	300	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
e	250	300	300	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	D12 @250	-
f	250	300	300	D12 @250	D12 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
g	250	350	350	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
h	250	300	300	D12 @250	D12 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
i	250	350	350	D12 @250	D12 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
j	250	400	400	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
k	250	300	300	D12 @250	D12 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
l	250	400	400	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
m	250	450	450	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
n	250	400	400	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
o	250	450	400	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
p	250	500	500	D16 @250	D16 @250	D25 @250	D25 @250	D25 @250	D25 @250	D25 @250	D25 @250	-
q	250	400	400	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
r	250	600	600	D16 @250	D16 @250	D25 @250	D25 @250	D25 @250	D25 @250	D25 @250	D25 @250	-
s	250	300	300	D12 @250	D12 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	D16 @250	-
t	250	550	550	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-
u	250	550	550	D16 @250	D16 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	D20 @250	-

Source: Study Team

In addition to the result of beam/slab analysis, minimum bar reinforcement as shown below shall be considered as well.

・Minimum reinforcement area per unit length against bending moment (mm^2)

＝Unit Width (mm) × Effective Depth (mm)×0.002

・Minimum reinforcement area per unit length against axial force (mm^2)

＝Axial force (N) / Allowable compressive stress （9.0N/mm^2） ×0.008 ⇒ 80(mm^2)

Additional reinforcement shall be applied along openings of manholes to prevent cracking. D12 is adopted as the minimum diameter of reinforcing bar.

5) Stability Analysis

(a) Stability Analysis against Buoyancy

Safety factor for buoyancy is obtained as a rate of total manhole weight load to buoyancy force due to ground water and the safety factor shall be above 1.2.

F =W/U>Fs=1.2

Where,

F : Safety factor for buoyancy
Fs : Allowable Safety factor against buoyancy
W : Self-weight (kN)
U : Buoyancy Force due to ground water(kN)

The result of analysis is shown in **Table 7.2.9**. Safety factors are above 1.2 for all calculation type.

Table 7.2.9 Result of Stability Analysis against Buoyancy

Calc. Type	Dimension (Outer)				Unit Weight of Water	Buoyancy Force	Self Weight	Safety Factor for Buoyancy		Allowable Safety Factor	
	Vd(1) (m)	Vd(2) (m)	Height (m)	WL depth (m)	(kN/m3)	(kN)	(kN)				
a	2.10	3.00	2.30	1.20	10	69.30	196.56	2.84	>	1.2	OK
b	1.80	2.00	3.30	0.80	10	90.00	174.24	1.94	>	1.2	OK
c	1.80	3.20	3.30	0.60	10	155.52	250.27	1.61	>	1.2	OK
d	2.10	2.70	3.30	0.90	10	136.08	241.16	1.77	>	1.2	OK
e	1.80	2.00	4.30	1.10	10	115.20	220.32	1.91	>	1.2	OK
f	1.80	2.60	4.30	1.20	10	145.08	266.98	1.84	>	1.2	OK
g	2.70	3.20	4.35	0.60	10	324.00	452.02	1.40	>	1.2	OK
h	1.80	2.00	5.30	1.00	10	154.80	266.40	1.72	>	1.2	OK
i	1.90	2.80	5.35	1.30	10	215.46	395.81	1.84	>	1.2	OK
j	2.80	3.50	5.40	0.80	10	450.80	654.48	1.45	>	1.2	OK
k	1.80	2.00	6.30	1.70	10	165.60	312.48	1.89	>	1.2	OK
l	2.80	2.20	6.40	1.50	10	301.84	559.78	1.85	>	1.2	OK
m	3.00	3.50	6.45	1.70	10	498.75	871.92	1.75	>	1.2	OK
n	2.90	2.60	7.40	2.00	10	407.16	740.38	1.82	>	1.2	OK
o	2.90	3.10	7.40	1.40	10	539.40	840.99	1.56	>	1.2	OK
p	3.10	4.00	7.50	0.90	10	818.40	1,230.80	1.50	>	1.2	OK
q	2.90	2.20	8.40	2.00	10	408.32	751.09	1.84	>	1.2	OK
r	3.30	4.40	8.60	0.90	10	1,118.04	1,767.37	1.58	>	1.2	OK
s	1.80	3.20	3.30	1.70	10	92.16	267.75	2.91	>	1.2	OK
t	3.10	5.25	6.55	1.30	10	854.44	1,437.95	1.68	>	1.2	OK
u	2.30	6.95	5.55	1.50	10	647.39	1,362.00	2.10	>	1.2	OK

Source: Study Team

(b) Stability Analysis against Soil Bearing Capacity

Stability against soil bearing capacity can be evaluated when soil mass of equivalent volume of where manhole structure occupies is self-weight of a manhole structure, summation of self-weight, soil coverage and live load at top slab. The result of analysis is shown in **Table 7.2.10**.

Table 7.2.10 Result of Stability Analysis against Soil Bearing Capacity

Calc. Type	Dimension (Outer) Vd(2)' (m)	Vd(1)' (m)	Height (m)	Unit Weight of Soil (kN/m3)	Soil Weight of Equivalent Volume (kN)		Self Weight (kN)	
a	2.10	3.00	2.30	18	260.82	>	196.56	OK
b	1.80	2.00	3.30	18	213.84	>	174.24	OK
c	1.80	3.20	3.30	18	342.144	>	250.27	OK
d	2.10	2.70	3.30	18	336.798	>	241.16	OK
e	1.80	2.00	4.30	18	278.64	>	220.32	OK
f	1.80	2.60	4.30	18	362.232	>	266.98	OK
g	2.70	3.20	4.35	18	676.512	>	452.02	OK
h	1.80	2.00	5.30	18	343.44	>	266.40	OK
i	1.90	2.80	5.35	18	512.316	>	395.81	OK
j	2.80	3.50	5.40	18	952.56	>	654.48	OK
k	1.80	2.00	6.30	18	408.24	>	312.48	OK
l	2.80	2.20	6.40	18	709.632	>	559.78	OK
m	3.00	3.50	6.45	18	1219.05	>	871.92	OK
n	2.90	2.60	7.40	18	1004.328	>	740.38	OK
o	2.90	3.10	7.40	18	1197.468	>	840.99	OK
p	3.10	4.00	7.50	18	1674	>	1,230.80	OK
q	2.90	2.20	8.40	18	964.656	>	751.09	OK
r	3.30	4.40	8.60	18	2247.696	>	1,767.37	OK
s	1.80	3.20	3.30	18	342.144	>	267.75	OK
t	3.10	5.25	6.55	18	1918.8225	>	1,437.95	OK
u	2.30	6.95	5.55	18	1596.9015	>	1,362.00	OK

Source: Study Team

7.2.3 Detailed Design of Sluiceways

7.2.3.1 Categorizing of Calculation Type

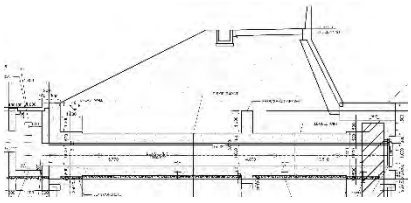
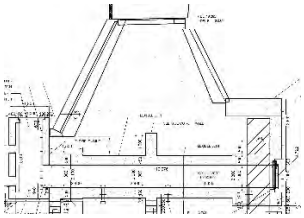
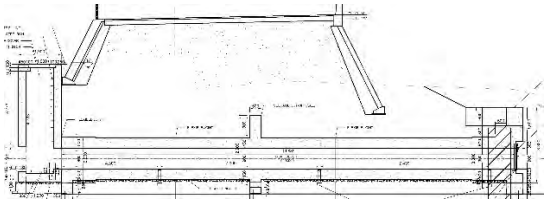
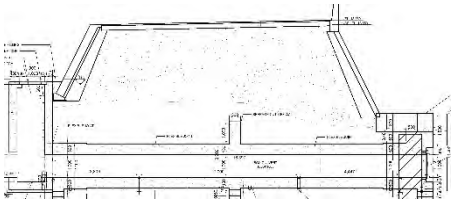
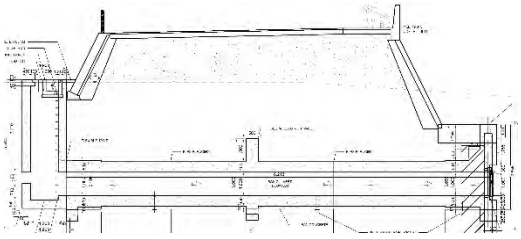
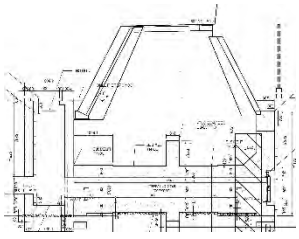
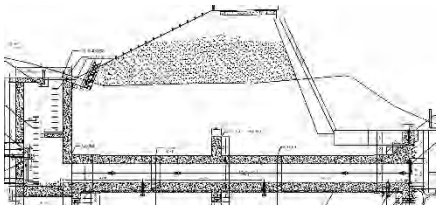
Sluiceways designed in this project can be categorized into several types of calculation models based on similarity in proportion and soil condition. Following tables explain categorization of all sluiceways.

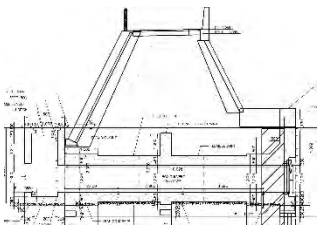
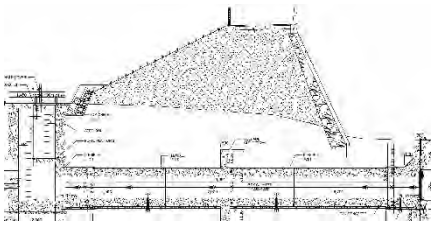
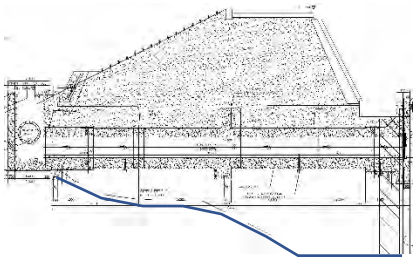
Table 7.2.11 Grouping of Sluiceway and Selection of Calculation Model Type

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

Source: Study Team

Table 7.2.12 Typical Model and Description of Each Type

Structural Type	Description	
Type A		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Levee slope at land side slope is not protected by concrete thus gentle, making the sluiceway longer as 16.3m.
Type B		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] ✓ Both side of Levee slopes concrete-protected and steep, thus sluiceway is relatively short as 9.9m -12.4m. ✓ Elevation of existing ground is relatively low which makes the height of embankment greater. Therefore, greater subsidence due to soil consolidation is expected in this type.
Type C		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] The length of sluiceway is 20.2m which makes the longest among all type, since levee crown is as wide as 10m and also levee is setting back from SSP retaining wall.
Type D		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Length of sluiceway is around 17.5m since levee crown is as wide as 10m.
Type E		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Length of sluiceway is around 19.5m since levee crown is as wide as 12m.
Type F		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Sluiceway is the shortest among all types as 7.0m – 7.7m, since coping concrete of SSP retaining wall is narrower than other types in addition to that the both side of Levee slopes are concrete-protected and steep.
Type G		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Shape of levee and length of sluiceway are similar to type A. However, since levee is setting back from SSP retaining wall, sluiceway is longer than Type A.

Type H		[Inner Size] 1.2m x 1.2m [Characteristic of Longitudinal Shape] Shape of levee and length of sluiceway are similar to type B, however, inner size of culvert is different.
Type I		[Inner Size] 1.2m x 1.2m (Double Barrel) [Characteristic of Longitudinal Shape] Shape of levee and length of sluiceway are similar to type G, however, inner size of culvert is different and double barrel.
Type J		[Inner Size] 1.0m x 1.0m [Characteristic of Longitudinal Shape] Shape of levee and length of sluiceway are similar to type A. However, since hard strata is shallow and inclined towards the river, longitudinal distribution of consolidation settlement must be unique.

Source: Study Team

7.2.3.2 Study on the ground settlement at the sluiceway site

The sluiceway shall be designed to allow ground settlement due to embankment, to minimize the gap of the settlement between sluiceway and adjoining levee. Therefore, first of all residual settlement due to the embankment at sluiceway sites shall be studied for design conditions of sluiceway. Residual settlement is obtained as the arithmetic summation of immediate settlement and consolidation settlement at the bottom level of sluiceway.

(1) Immediate settlement

Immediate settlement is calculated with the aid of the following formula as shown below, on the assumption that the ground is elastic.

$$S_x = \sum_{i=1}^{n} \frac{-3a_i \cdot q_i}{E_m \cdot \pi} \log \cdot \sin\left(\tan^{-1}\frac{a_i}{H}\right) \cdot \left\{1.0 - \frac{0.75}{\pi}\left[\left(1+\frac{x}{a_i}\right)\log\left|1+\frac{x}{a_i}\right| + \left(1-\frac{x}{a_i}\right)\log\left|1-\frac{x}{a_i}\right|\right]\right\}$$

Where,

S = Immediate Settlement on xi , m

qi = Load of Embankment, tf/m^2 or kN/m^2

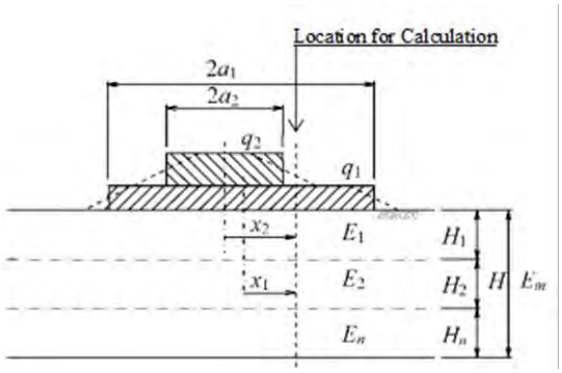
Em = Modulus of Deformation of the ground, tf/m^2 or kN/m^2

ai = Load width, m

H = Depth for considering the effect of immediate settlement, m

n = Number of uniform load

x = Distance from the center of the ach uniform load, m



Source: Study Team

Figure 7.2.4 Calculation Model for Immediate Settlement

$$E_m = \frac{\log \frac{(B+2h_n\tan\theta)L}{(L+2h_n\tan\theta)B}}{\sum_{i=1}^{n} \frac{1}{E_{mi}} \log \frac{(B+2h_i\tan\theta)(L+2h_{i-1}\tan\theta)}{(L+2h_i\tan\theta)(B+2h_{i-1}\tan\theta)}}$$

Where,

Em = Converted modulus of deformation (**Figure 7.2.5**) of the ground, in B ≠ L considering several layers, kgf/cm^2 or kN/m^2

B = Load width, m

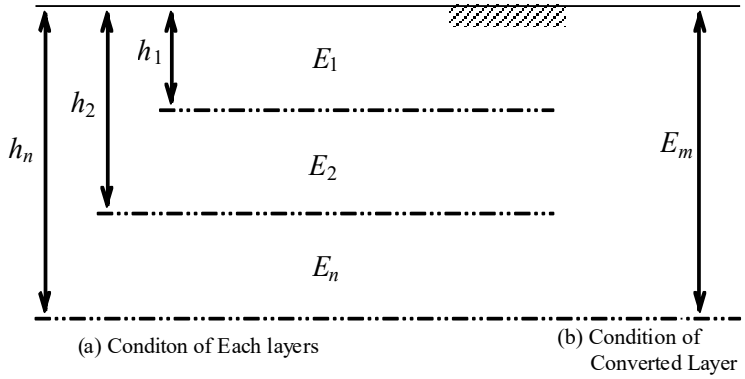
L = Load Length, m

Hn = Depth for considering the effect, more than 3 times of load width, m

Hi = Depth to the bottom of each layer, m

Emi = Modulus of deformation of the ground of the layer i, kgf/cm^2 or kN/m^2

θ = Angle of load distribution, θ = 30°



Source: Study Team

Figure 7.2.5 Converted Modulus of Deformation in case of each layer having different depth

(2) Consolidation Settlement

Consolidation settlement is calculated wherein the vertical stress in the ground is increased by the weight of the embankment and is the summation of the settlement of each silt layer as described below:

$$Sc = \frac{e1 - e2}{1 + e1} H$$

Where,

Sc : Total consolidation settlement (m)
e_1 : Initial void ratio
e_2 : Void ratio after consolidation, which is obtained from e-logP curb applying P0+ΔP
P0 : Initial compressive stress (kN/m^2)
Δp : Increased compressive stress (kN/m^2)
H : Thickness of compressive stratum (m)

(3) Conditions for Calculation of Ground Settlement

Soil properties used in calculation of sluiceway are shown in **Table 7.2.13**. Ground reaction factor Em used in calculation of immediate settlement can be calculated as Em=700N, where N is average N value of each layer.

Table 7.2.13 Mechanical Properties of Soil

Geological age		Holocene		Pleistocene			
Layer		As	Ac	Ds	Dc	GFw	GFf
N value		9.9	7.8	21.1	22.5	38.2	44.4
		0.0 - 30.0	0.0 - 44.0	3.0 - 48.0	0.0 - 50.0	7.0 - >50	12.0 - >50
Mechanical Properties	φ (degree)	27.2	-	32.8	-		
	c (kN/m^2)	-	46.9	-	135.0		
	qu	-	-	-	26.7	-	6,643
	(kN/m^2)	-	-	-	18.8 - 34.7	-	474.7 - 19,460.7
	Es	6,936	5,469	14,756	15,747	26,759	31,057
	(kN/m^2)	⇒6,900	⇒5,400	⇒14,700	⇒15,700	⇒26,700	
	Pc	-	-	-	100.9	-	-
	(kg/cm^2)	-	-	-	49.1 - 196.2	-	-

Source: Study Team

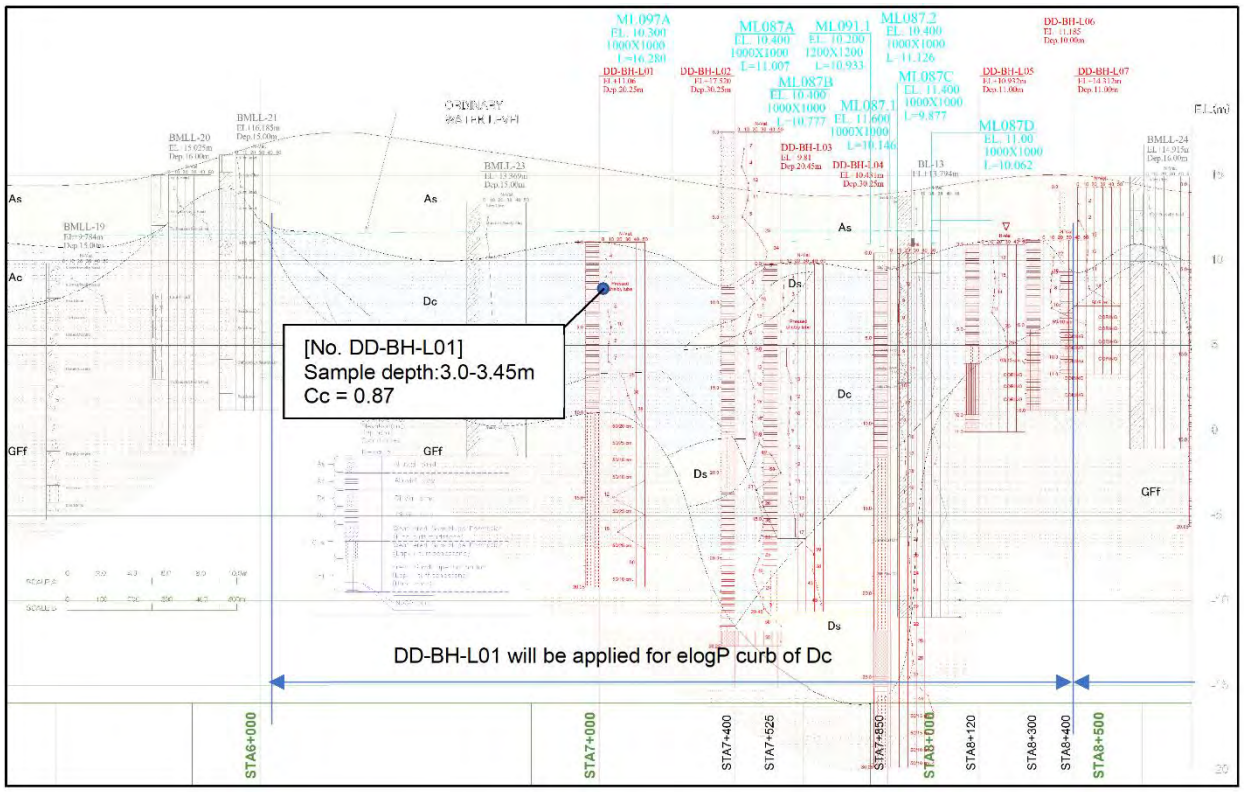
In this project, soil consolidation test for upper Marikina was conducted with 5 borehole samples. **Table 7.2.14** shows test samples and corresponding soil layer and station No. for each sample. Consolidation settlement of sluiceway is calculated using e-logP curb.

Table 7.2.14 Applied Consolidation Test Samples by Sluiceway Location

Location / Station No.		Sluiceway No.	Applied Test Sample	
			Ac	Dc
Left Bank	Downstream of STA8+400	ML097A～ML087D	-	DD-BH-L01
	Upstream of STA8+400	ML068A～ML068	DD-BH-L10	DD-BH-L13
Right Bank	Downstream of STA8+100	MR125A～MR067A	-	DD-BH-R03
	Upstream of STA8+100	MR009A, MR008	DD-BH-L10*	DD-BH-R13

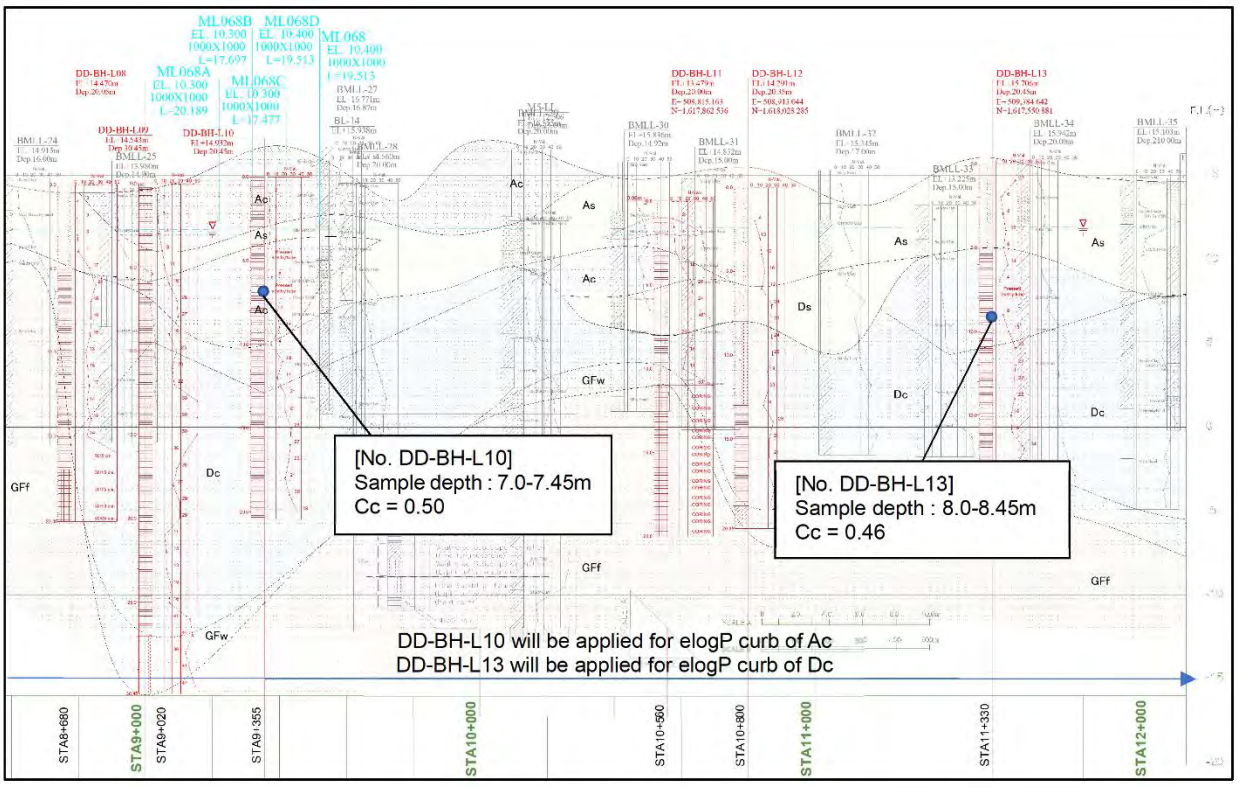
Note: Consolidation test was conducted only in left bank for Ac layer (DD-BH-L10). However, similarities are found in the result of two consolidation test samples of both bank at near station No., such as DD-BH-R03 to DD-BH-L01 and DD-BH-R13 to DD-BH-L13. Therefore, e-logP curb of DD-BH-L10 can be applied for Ac layer in right bank as well.

Source: Study Team



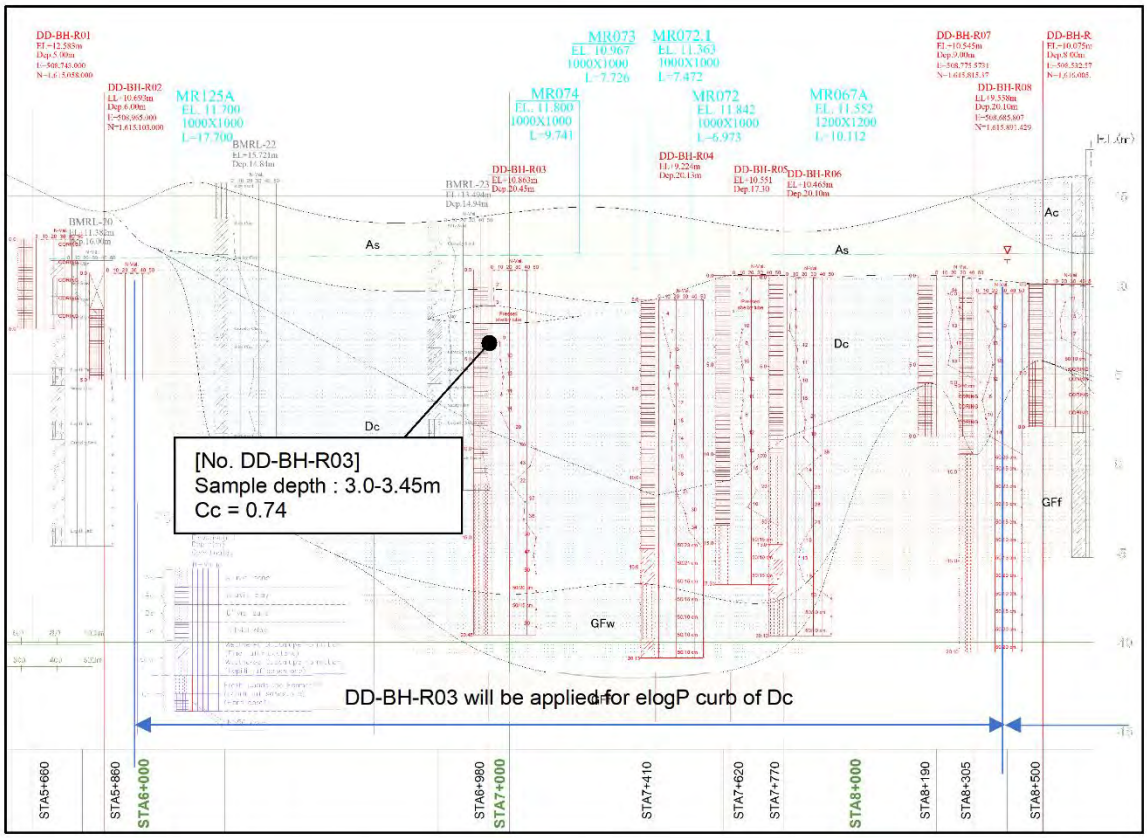
Source: Study Team

Figure 7.2.6 Location of Consolidation Test and Sluiceways Geological Profile (Left Bank 1/2)



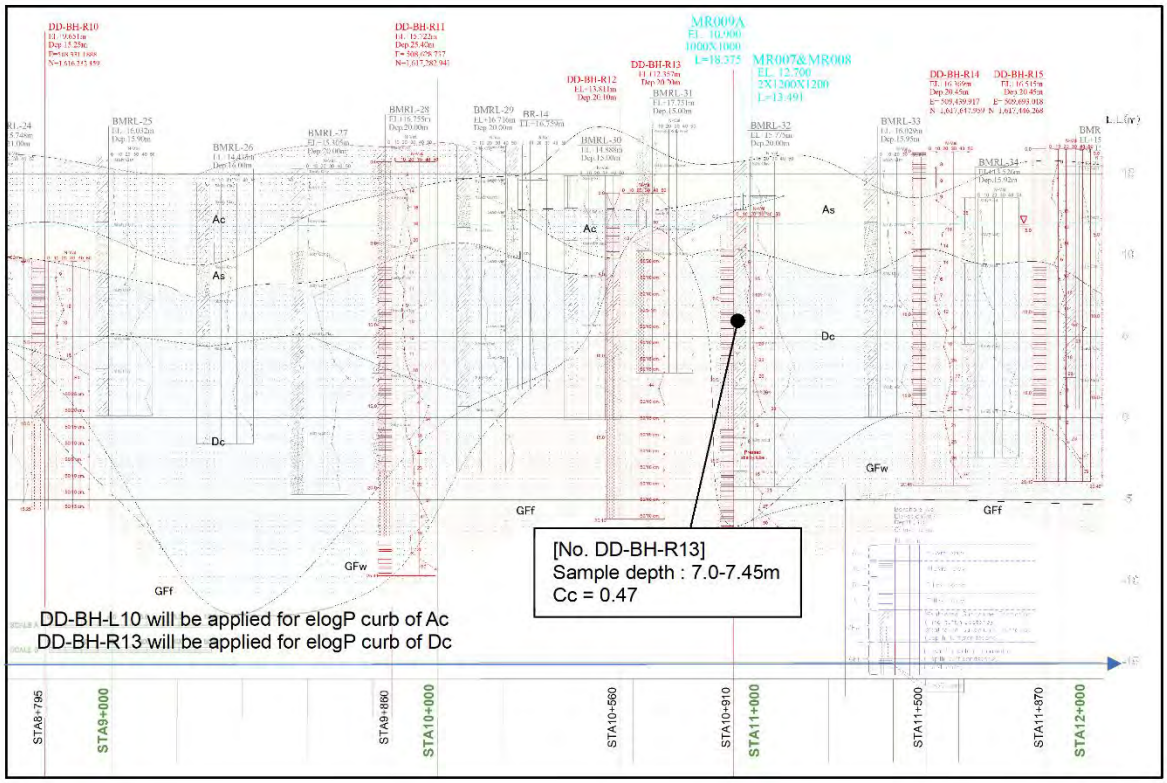
Source: Study Team

Figure 7.2.7 Location of Consolidation Test and Sluiceways Geological Profile (Left Bank 2/2)



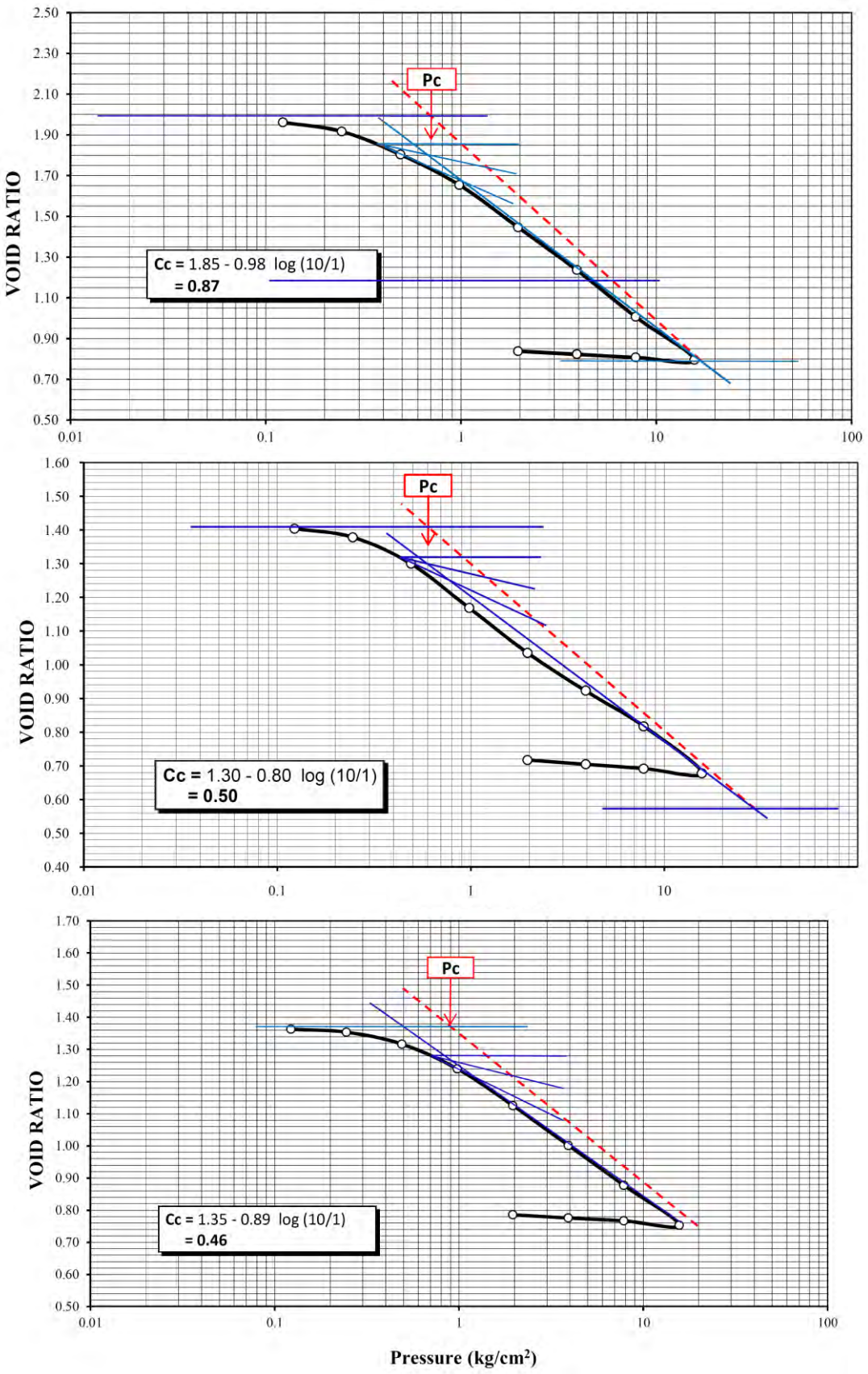
Source: Study Team

Figure 7.2.8 Location of Consolidation Test and Sluiceways Geological Profile (Right Bank 1/2)



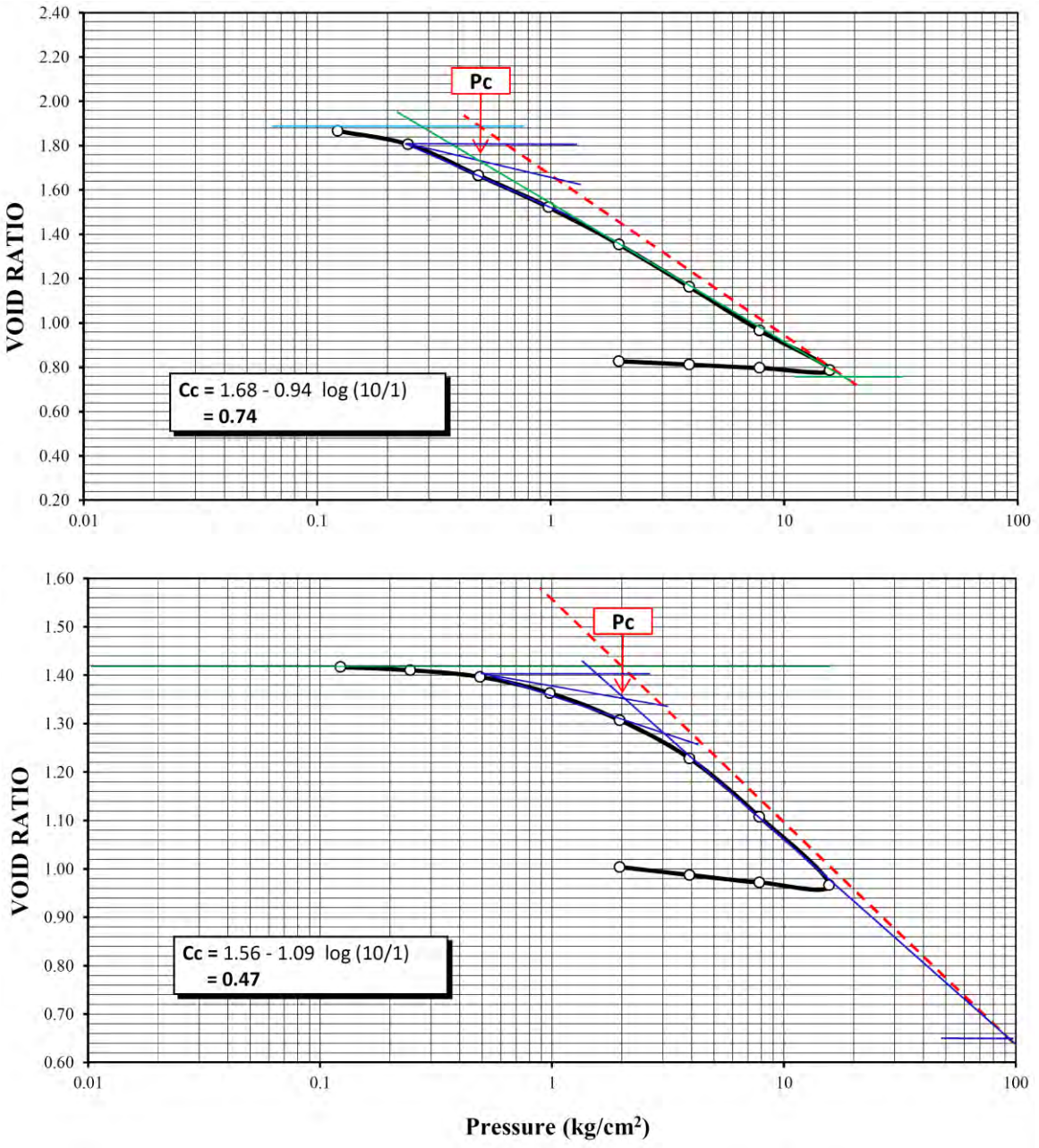
Source: Study Team

Figure 7.2.9 Location of Consolidation Test and Sluiceways Geological Profile (Right Bank 2/2)



Source: Study Team

Figure 7.2.10 e-log p Curbs of Test Samples at Left Bank



Source: Study Team

Figure 7.2.11 e-log p Curbs of Test Samples at Right Bank

(4) Summerly of Residual Settlement

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

Table 7.2.15 Calculation Results of Residual Settlement

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

Source: Study Team

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

7.2.3.3 Structural Details

(1) Seepage cut-off

Seepage cut-off is essential to avoid the adverse effect of piping and migration of soil particles.

1) Horizontal Direction

The proposed dikes are supported by SSP. The SSP also acts as horizontal seepage cutoff wall. For the horizontal direction, it is deemed that enough length of path against percolation is developed.

2) Vertical Direction

Regarding the vertical direction of seepage, the required creep length is distributed by the steel sheet piles which are installed at the wing wall, breast wall and seepage cut off wall.

The length of SSP for sealing is decided considering the following:

- When there is a thin layer of clayey soil under the subgrade and sandy soil layer is below the clayey soil, seepage would flow through the porous sandy soil layer. Sealing would not be effective.
- When there are alternate layers of sandy and clayey soil and SSP is driven down to deep clayey soil layer (Ac2), sealing would be most effective.

The required length is calculated based on Lane's Creep Theory as expressed by the following formula:

$$C = \frac{\sum H / 3 + \sum V}{\Delta H}$$

Where:

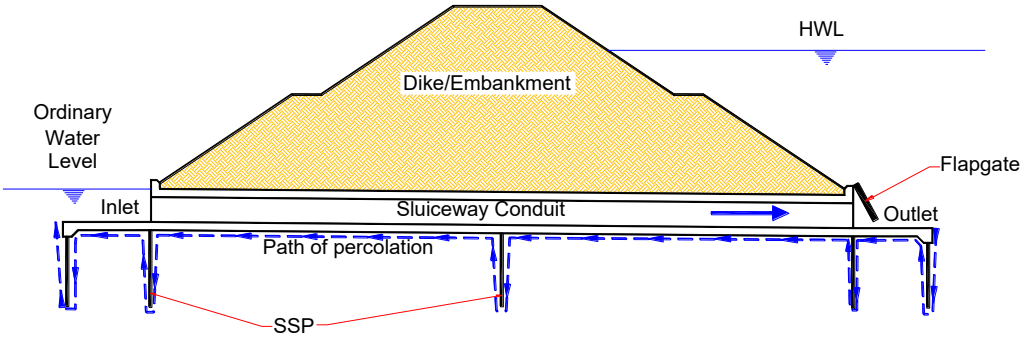
C	=	Creep Ratio as shown below.
H	=	Horizontal path length of percolation of main body and wing wall (m)
V	=	Vertical path length of percolation of sheet pile for sealing (m)
ΔH	=	Hydraulic head or the difference between headwater and tail water (m)

Table 7.2.16 Creep Ratio

Material	Ratio[1/]
- Very Fine sand or Silt	8.5
- Fine Sand	7.0
- Medium Sand	6.0
- Coarse Sand	5.0
- Fine Gravel	4.0
- Medium Gravel	3.5
- Coarse Gravel including Cobbles	3.0
- Boulders with Some Cobbles and Grave	2.5
- Soft Clay	3.0
- Medium Clay	2.0
- Hard Clay	1.8
- Very Hard Clay or Hardpan	1.6

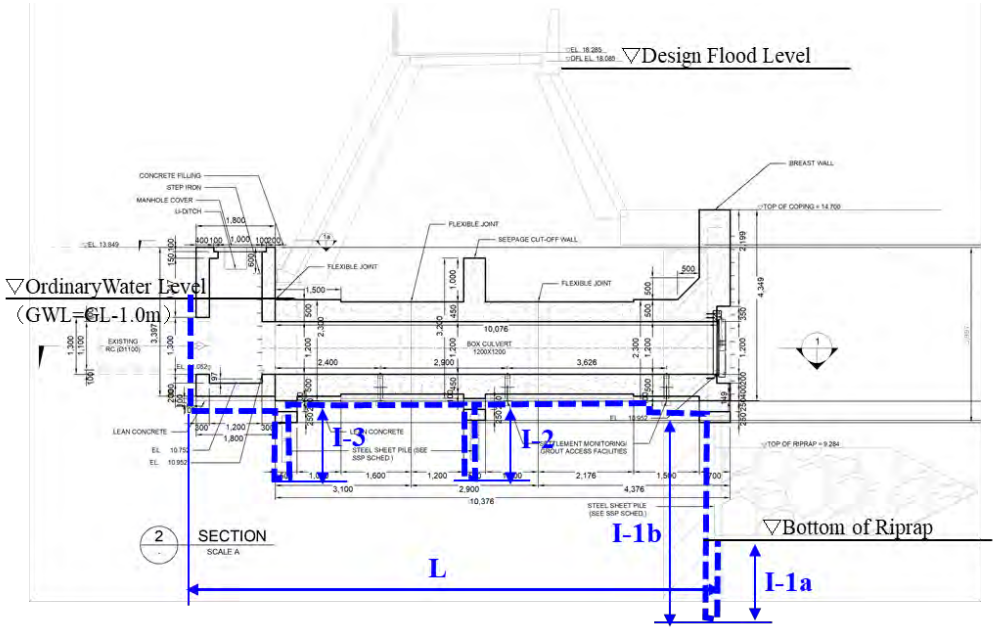
[1/] *"Design of Small Dams", US Bureau of Reclamation (USBR), 1974, p.341*
Source: Study Team

General idea of path of percolation of sluiceway is calculated from the river side end of wing wall to the end of the one at land side as shown in **Figure 7.2.12**. However, in this project the path can be measured from the river side end of sluiceway to land side end of manhole as shown in **Figure 7.2.13** since no wingwall is considered.



Source: Study Team

Figure 7.2.12 Path of Percolation of Sluiceway



Source: Study Team

Figure 7.2.13 Percolation Path of Sluiceway in this Project

As for vertical length of the percolation path, bottom end of manhole to the ground water level at back land shall be included.

The length of each SSP based on calculation is shown in **Table 7.2.17**.

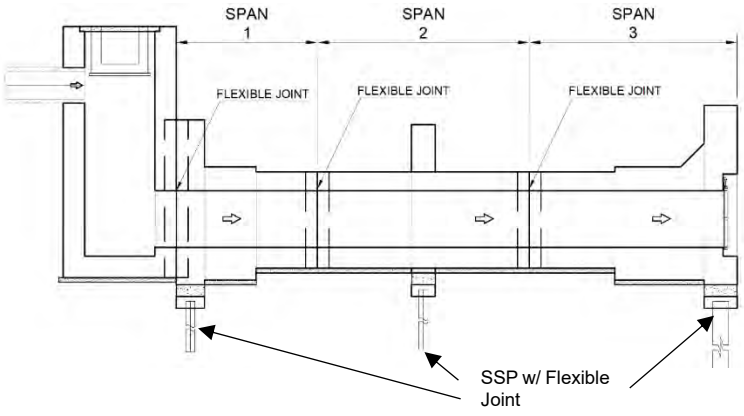
Table 7.2.17 Length of SSP Cut Off Wall

Location			Name	L1 Length of Sluiceway	L2 Length of Manhole	L/3 (L=L1+L2)	DFL	GWL	ΔH (ΔH=DFL - GWL)	Vertical Creep Distance				Total of Vertical Creep Distance: Σl	(L/3+Σl) /ΔH	Soil Condition	Creep Ratio: C	Safety against Seepage
										I-1a	I-1b	I-2	I-3					
Left Bank	CP2	7+216	ML097A	16.280	1.200	5.827	17.721	13.351	4.370	10.772	15.150	1.000	0.000	31.523	8.547	Fine Sand	7.000	OK
		7+680	ML087A	11.007	1.200	4.069	18.073	12.917	5.156	12.651	17.150	1.000	0.000	34.868	7.552	Fine Sand	7.000	OK
		7+780	ML087B	10.777	1.200	3.992	18.085	13.498	4.587	12.626	17.150	1.000	0.000	35.424	8.593	Fine Sand	7.000	OK
		7+800	ML091.1	10.933	1.200	4.044	18.000	13.211	4.789	12.621	16.950	1.000	0.000	35.132	8.180	Fine Sand	7.000	OK
		7+827	ML087.2	11.126	1.200	4.109	18.061	12.848	5.213	12.116	16.650	1.000	0.000	33.764	7.265	Fine Sand	7.000	OK
		7+856	ML087.1	10.146	1.200	3.782	18.109	13.657	4.452	12.111	16.650	1.000	0.000	33.368	8.345	Fine Sand	7.000	OK
		7+880	ML087C	9.877	1.200	3.692	18.121	13.917	4.204	12.101	16.150	1.000	0.000	33.318	8.804	Fine Sand	7.000	OK
		7+980	ML087D	9.560	1.200	3.587	18.182	14.573	3.609	12.076	16.650	1.000	0.000	33.849	10.373	Fine Sand	7.000	OK
		9+220	ML068A	20.189	2.000	7.396	18.933	15.923	3.010	4.766	9.650	1.000	0.000	22.589	9.962	Fine Sand	7.000	OK
	CP3	9+320	ML068B	17.697	2.000	6.566	18.659	15.484	3.175	12.741	17.650	1.000	0.000	38.125	14.076	Fine Sand	7.000	OK
		9+420	ML068C	17.477	2.000	6.492	19.054	16.044	3.010	12.716	17.650	1.000	0.000	38.660	15.001	Fine Sand	7.000	OK
		9+520	ML068D	19.513	2.000	7.171	19.115	16.104	3.011	12.691	15.650	1.000	0.000	36.595	14.535	Fine Sand	7.000	OK
		9+521	ML068	19.513	2.100	7.204	19.115	16.105	3.010	12.691	15.650	1.000	0.000	36.596	14.552	Fine Sand	7.000	OK
Right Bank	CP1	6+060	MR125A	17.700	1.400	6.367	17.400	14.025	3.375	8.049	12.150	1.000	0.000	25.074	9.316	Fine Sand	7.000	OK
	CP2	7+197	MR074	9.741	1.200	3.647	17.709	15.477	2.232	13.276	17.650	1.000	0.000	37.153	18.280	Fine Sand	7.000	OK
		7+356	MR073	7.726	1.200	2.975	17.806	14.089	3.717	13.736	17.317	1.000	0.000	36.725	10.681	Fine Sand	7.000	OK
		7+399	MR072.1	7.472	1.200	2.891	17.830	14.645	3.185	13.726	17.713	1.000	0.000	37.271	12.610	Fine Sand	7.000	OK
		7+509	MR072	6.973	2.000	2.991	17.891	15.667	2.224	13.696	17.652	1.000	0.000	38.163	18.504	Fine Sand	7.000	OK
		7+840	MR067A	10.162	1.200	3.787	18.097	13.400	4.697	9.611	13.950	2.000	2.000	35.311	8.324	Fine Sand	7.000	OK
	CP3	10+935	MR009A	18.375	2.100	6.825	19.956	15.512	4.444	8.841	14.150	1.000	0.000	30.153	8.321	Fine Sand	7.000	OK
		10+964	MR008	19.651	2.000	7.217	19.966	14.862	5.104	8.831	13.950	1.000	0.000	29.493	7.192	Fine Sand	7.000	OK

Source: Study Team

(2) Flexible Joint and SSP with Flexible Joint

To allow for the effect of ground settlement, flexible joint for box culvert and SSP with Flexible Joint are applied. Location of those joints are indicated in **Figure 7.2.14**.



Source: Study Team

Figure 7.2.14 Location of Flexible Joint and SSP with Flexible Joint

Capability of joint for will be selected among 100mm, 200mm and 300mm based on the results of settlement calculation. Selected flexible joint and SSP with flexible joint are shown in **Table 7.2.18** and **Table 7.2.19**.

Table 7.2.18 Selection of Flexible Joint Capacity

Structural Type of Sluiceway	Manhole - Span 1			Span 1 - Span 2			Span 1 - Span 3		
	Displacement (mm)	Capacity of Flexiblity	Type of Joint	Displacement (mm)	Capacity of Flexiblity	Type of Joint	Displacement (mm)	Capacity of Flexiblity	Type of Joint
TYPE-A	89.7	100.0	TYPE A	7.4	100.0	TYPE A	18.4	100.0	TYPE A
TYPE-B	240.8	300.0	TYPE C	1.8	100.0	TYPE A	2.6	100.0	TYPE A
TYPE-C	73.5	100.0	TYPE A	2.8	100.0	TYPE A	35.6	100.0	TYPE A
TYPE-D	152.3	200.0	TYPE B	12.4	100.0	TYPE A	15.7	100.0	TYPE A
TYPE-E	82.2	100.0	TYPE A	0.7	100.0	TYPE A	8.3	100.0	TYPE A
TYPE-F	119.6	200.0	TYPE B	0.5	100.0	TYPE A	23.5	100.0	TYPE A
TYPE-G	72.3	100.0	TYPE A	2.6	100.0	TYPE A	3.3	100.0	TYPE A
TYPE-H	241.9	300.0	TYPE C	1.5	100.0	TYPE A	2.2	100.0	TYPE A
TYPE-I	77.3	100.0	TYPE A	3.4	100.0	TYPE A	2.5	100.0	TYPE A
TYPE-J	42.6	100.0	TYPE A	19.6	100.0	TYPE A	1.2	100.0	TYPE A

Source: Study Team

Table 7.2.19 Capability of SSP with flexible joint

Structural Type of Sluiceway	Breast wall at Landside			Seepage cut-off wall			Breast wall at Riverside		
	Settlement (mm)	Capacity of Flexiblity	Type of Joint	Settlement (mm)	Capacity of Flexiblity	Type of Joint	Settlement (mm)	Capacity of Flexiblity	Type of Joint
TYPE-A	89.7	100.0	TYPE A	232.5	300.0	TYPE C	95.5	100.0	TYPE A
TYPE-B	240.8	300.0	TYPE C	266.6	300.0	TYPE C	175.5	200.0	TYPE B
TYPE-C	73.5	100.0	TYPE A	150.4	200.0	TYPE B	31.7	100.0	TYPE A
TYPE-D	152.3	200.0	TYPE B	252.1	300.0	TYPE C	79.6	100.0	TYPE A
TYPE-E	82.2	100.0	TYPE A	125.4	200.0	TYPE B	42.6	100.0	TYPE A
TYPE-F	119.6	200.0	TYPE B	137.7	200.0	TYPE B	126.9	200.0	TYPE B
TYPE-G	72.3	100.0	TYPE A	93.5	100.0	TYPE A	49.5	100.0	TYPE A
TYPE-H	241.9	300.0	TYPE C	226.9	300.0	TYPE C	182.2	200.0	TYPE B
TYPE-I	77.3	100.0	TYPE A	135.2	200.0	TYPE A	63.7	100.0	TYPE A
TYPE-J	42.6	100.0	TYPE A	153.1	200.0	TYPE B	55.9	100.0	TYPE A

Source: Study Team

(3) Box Culvert

1) Span Arrangement

Considering the basic concept of flexible sluiceway, which is to follow the settlement of adjoining dike, 3-span is reasonable arrangement as the center span beneath the levee crown and both river side and land side spans beneath levee slope. Specific length of those span may be adjusted when some spans are found over-stressed by bending moment in the longitudinal analysis.

2) Design Condition

The design conditions are shown below.

Table 7.2.20 Design Condition of Box Culvert

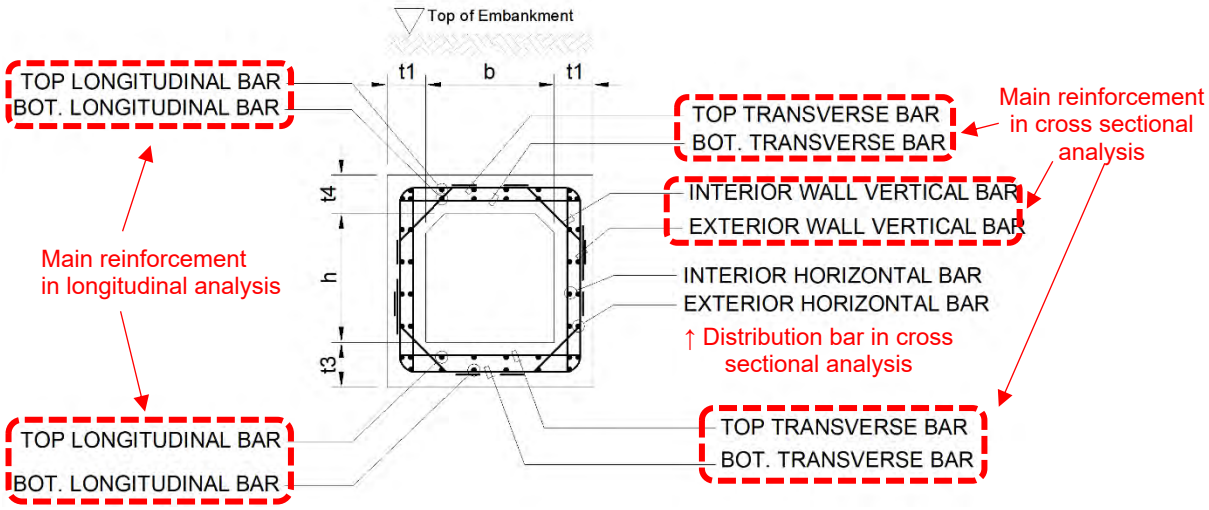
Item			Design Condition
Material Unit Weight	Concrete Reinforced		24.0 kN/m^3
	Concrete Plain		23.5kN/m^3
	Structural Steel		77.0 kN/m^3
	Water		9.8 kN/m^3
Soil Condition	Quality of Soil		
	Unit Weight	Damped and Wet Condition	19.0 kN/m^3
		Saturated Condition	20.0 kN/m^3
		Submerged Condition	10.0 kN/m^3
Surcharge on the Ground			Normal: 10 kN/m^2 Seismic: 5 kN/m^2 [1]
Live load on Top Slab			T-25

[1] *Seismic Condition is Considered only in calculation of longitudinal direction.*

Source: Study Team

3) Thickness of Culvert and Bar Arrangement

Thickness of culvert members and bar arrangement are designed by cross sectional analysis and longitudinal analysis. Longitudinal bars of top slab and bottom slab work as main reinforcement in longitudinal calculation, while transverse and wall vertical bars work as main reinforcement in cross sectional calculation. Horizontal bar of side wall are distribution bar in cross sectional calculation.



Source: Study Team

Figure 7.2.15 Typical Bar Arrangement of Box Culvert Cross Section

4) Calculation Result

Outline of calculation result is as follows. Details of calculation process are shown in appendix.

Table 7.2.21 Calculation Results of Box Culvert (Longitudinal Analysis)

Structural Type of Sluiceway	b	h	Span	Block	Bottom Slab					Top Slab				
					t3	Longitudinal				t4	Longitudinal			
						Outer Side		Inner Side			Outer Side		Inner Side	
						Dia.	Interval	Dia.	Interval		Dia.	Interval	Dia.	Interval
TYPE A	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
			3	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
				2	500	D22(A)	@250	D22(A)	@250	500	D22(A)	@250	D22(A)	@250
TYPE B	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			3	1	450	D19(A)	@250	D19(A)	@250	450	D19(A)	@250	D19(A)	@250
				2	700	D19(A)	@250	D19(A)	@250	500	D19(A)	@250	D19(A)	@250
TYPE C	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D19(A)	@250	D19(A)	@250	450	D19(A)	@250	D19(A)	@250
			3	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
				2	500	D22(A)	@250	D22(A)	@250	500	D22(A)	@250	D22(A)	@250
TYPE D	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			3	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
				2	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
TYPE E	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D19(A)	@250	D19(A)	@250	450	D19(A)	@250	D19(A)	@250
			3	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
				2	500	D22(A)	@250	D22(A)	@250	500	D22(A)	@250	D22(A)	@250
TYPE F	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			3	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
				2	500	D25(A)	@250	D25(A)	@250	500	D25(A)	@250	D25(A)	@250
TYPE G	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
			3	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
				2	500	D22(A)	@250	D22(A)	@250	500	D22(A)	@250	D22(A)	@250
TYPE H	1,200	1,200	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			3	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
				2	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
TYPE I	1,200	1,200	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			2	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
			3	1	450	D16(A)	@250	D16(A)	@250	450	D16(A)	@250	D16(A)	@250
				2	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
TYPE J	1,000	1,000	1	1	500	D16(A)	@250	D16(A)	@250	500	D16(A)	@250	D16(A)	@250
				2	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
			2	1	450	D22(A)	@250	D22(A)	@250	450	D22(A)	@250	D22(A)	@250
			3	1	450	D19(A)	@250	D19(A)	@250	450	D19(A)	@250	D19(A)	@250
				2	500	D19(A)	@250	D19(A)	@250	500	D19(A)	@250	D19(A)	@250

Source: Study Team

Table 7.2.22 Calculation Results of Box Culvert (Cross-Sectional Analysis)

Structural Type of Sluiceway	Span	Block	Side Wall				Bottom Slab				Top Slab			
			Vertical				Transverse				Transverse			
			Outer Side		Inner Side		Outer Side		Inner Side		Outer Side		Inner Side	
			Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval
TYPE A	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE B	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE C	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE D	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE E	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE F	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE G	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE H	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE I	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
TYPE J	1	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	2	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
	3	1	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250
		2	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250	D16(A)	@250	D12(A)	@250

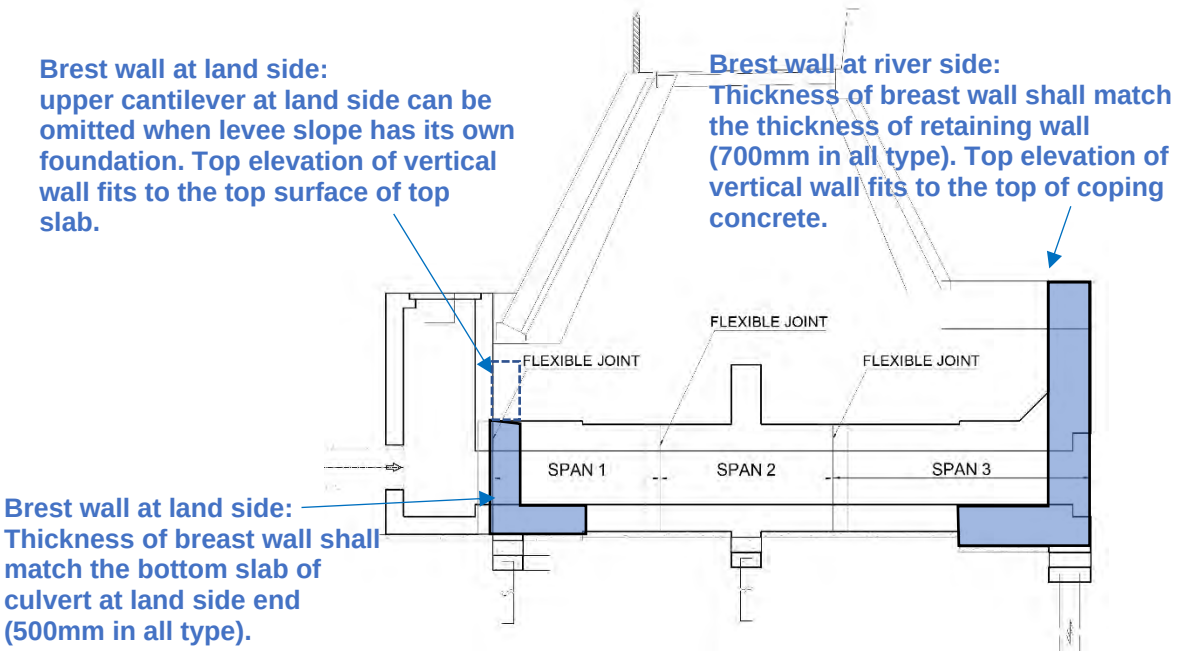
Source: Study Team

(4) Breast Wall

1) Dimension of Breast Wall

Brest walls at river side are part of continuous retaining wall of river structure. Therefore, thickness of breast wall shall match the thickness of adjoining concrete coping of river structure (700mm in all type). Top elevation of vertical wall fits to the top of coping concrete.

As for the breast walls at land side, thickness shall match that of bottom slab of culvert at land side end (500mm in all type). e top of coping concrete. Breast wall often has upper cantilever to serve as foundation of levee slope. However, since in this project levee slope has its own foundation, upper cantilever at land side can be omitted except Type A.



Source: Study Team

Figure 7.2.16 Dimension of Breast Wall

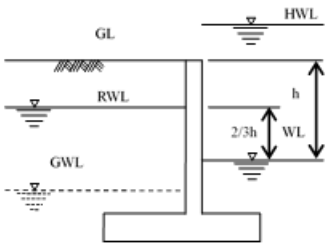
2) Design Condition

The Breast wall is considered as cantilever with the end fixed on the box culvert. Residual water level (RWL) is applied for back water level for the breast wall at river side as shown in Figure 7.2.16, while front water level is ordinary.

$$RWL = (GL - WL) \times 2/3$$

Where,

RWL	:	Residual water level
GWL	:	Natural ground water level
WL	:	Water level at front
GL	:	Ground level



Source: Study Team

Figure 7.2.17 Water Level Conditions for Breast Wall at River Side

As for the breast wall at land side, water level difference of front side and back side is ignorable since the top of wall is under ground water level.

3) Results of Calculation

The dimensions based on the results of structural calculation are shown below. Details of calculation process are shown in appendix.

Table 7.2.23 Calculation Results of Breast Wall (River Side)

SCHEDULE OF BREAST WALL AT RIVER SIDE																
Structural Type of Sluiceway	Vertical Wall								Bottom Slab							
	Horizontal				Vertical				Longitudinal				Transverse			
	Outer Side		Inner Side		Outer Side		Inner Side		Bottom		Top		Bottom		Top	
	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval
TYPE-A	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-B	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-C	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-D	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-E	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-F	D16(A)	@250	D22(A)	@250	D16(A)	@250	D22(A)	@250	D12(A)	@250	D12(A)	@250	D22(A)	@250	D22(A)	@250
TYPE-G	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-H	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-I	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-J	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250

Source: Study Team

Table 7.2.24 Calculation Results of Breast Wall (Land Side)

Structural Type of Sluiceway	Vertical Wall								Bottom Slab							
	Horizontal				Vertical				Longitudinal				Transverse			
	Outer Side		Inner Side		Outer Side		Inner Side		Bottom		Top		Bottom		Top	
	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval	Dia.	Interval
TYPE-A	D16(A)	@250	D16(A)	@250	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-B	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-C	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-D	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-E	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-F	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D22(A)	@250	D22(A)	@250
TYPE-G	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-H	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D19(A)	@250	D19(A)	@250
TYPE-I	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250
TYPE-J	D16(A)	@250	D16(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D12(A)	@250	D16(A)	@250	D16(A)	@250

Source: Study Team

7.3 MCGS Detailed Design

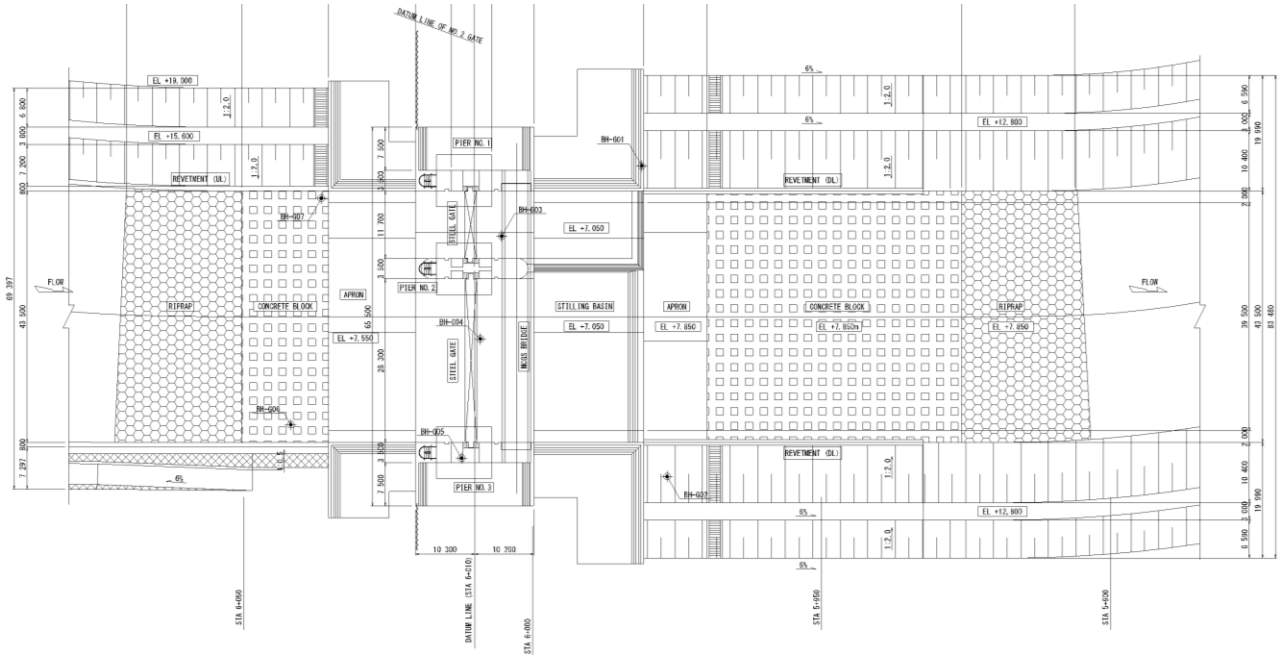
7.3.1 MCGS Detailed Design Overview

The detailed design was carried out based on the specifications in **6.3 MCGS Basic Design**. In the detailed design, the following assessment and designs were carried out.

- Structural design and Level 2 seismic design of civil engineering facilities (Main body, apron, and retaining walls.)
- Determination of structural design and specifications of gate equipment
- Designing temporary works (Temporary cofferdam and construction roads)
- Detailed design and specification setting of information and telecommunication facilities and electrical facilities

7.3.2 Civil Engineering Design

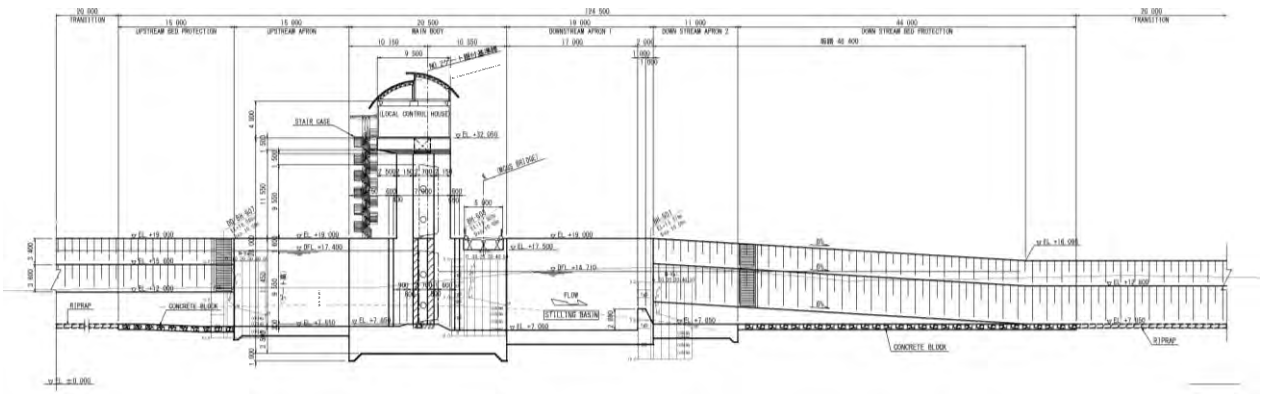
In the basic design of MCGS, civil engineering facilities such as stability calculation of foundation works, structural calculation of main body, L2 seismic-resistant design are conducted. **Figure 7.3.1** to **Figure 7.3.6** display structural drawings of the MCGS.



Source: Study Team

Figure 7.3.1 General Drawing of MCGS

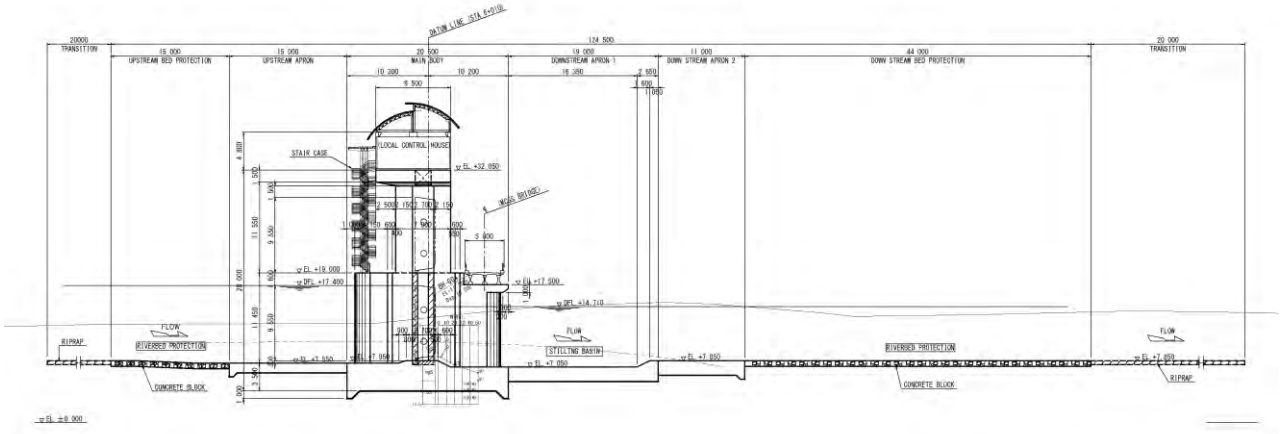
A-A



Source: Study Team

Figure 7.3.2 MCGS Profile -1

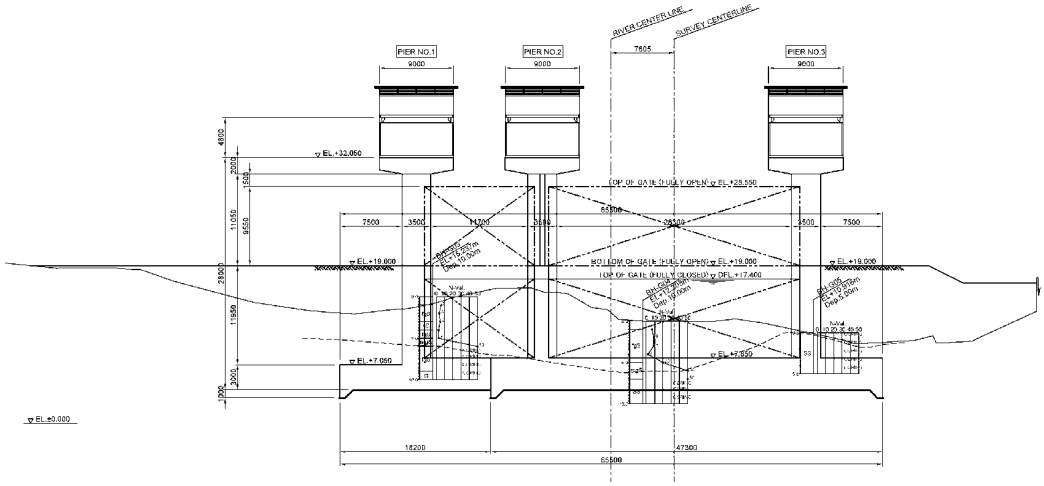
B-B



Source: Study Team

Figure 7.3.3 MCGS Profile View -2

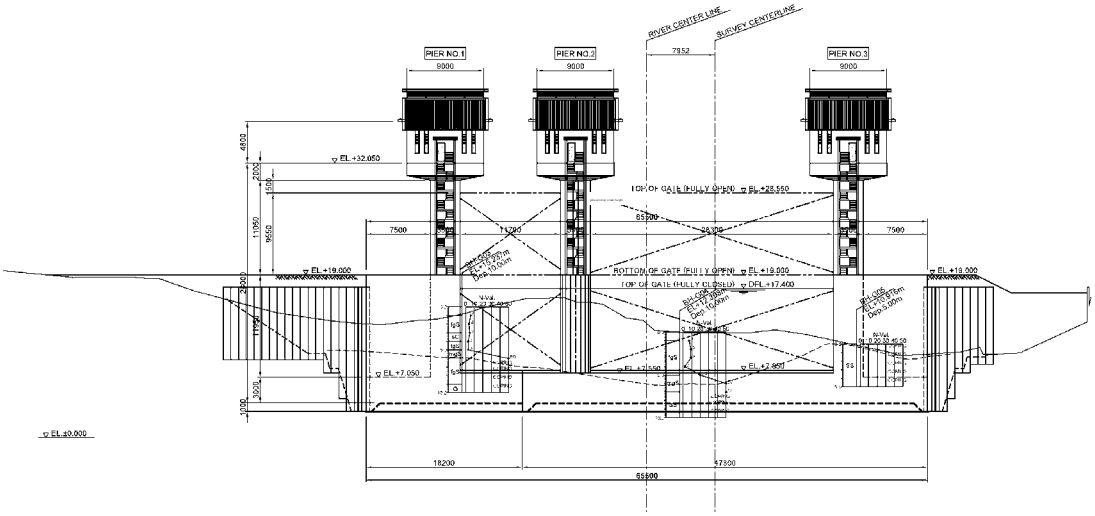
1-1



Source: Study Team

Figure 7.3.4 Cross Sectional View of MCGS-1

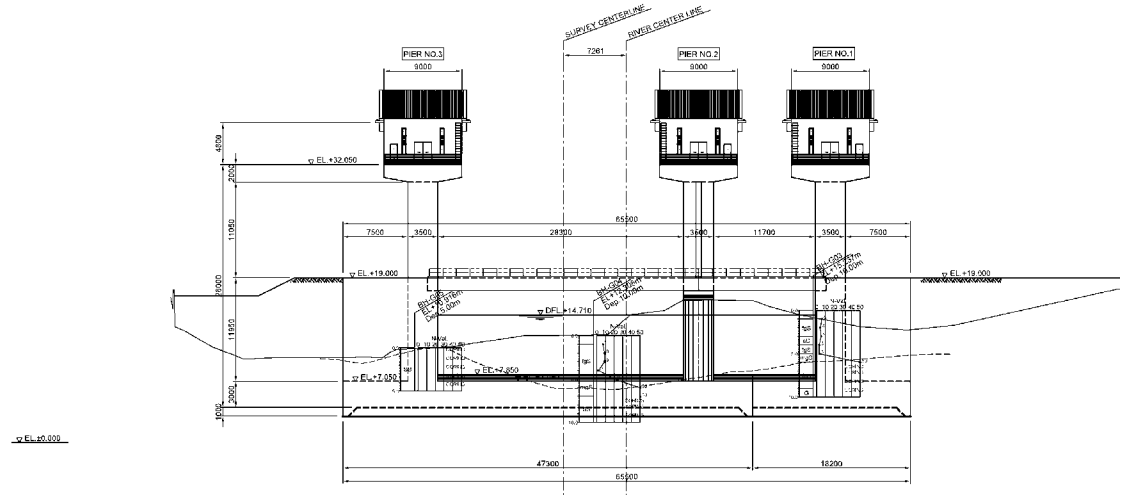
2−2



Source: Study Team

Figure 7.3.5 Cross Sectional View of MCGS -2

3 −3



Source: Study Team

Figure 7.3.6 Cross Sectional View of MCGS -3

7.3.2.1 Design Conditions

This section summarizes the necessary design conditions for the structural design of MCGS structural calculation and Load Conditions, ground conditions, water level conditions, and construction conditions.

(1) Structural Design Condition

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

Table 7.3.1 List of MCGS Structural Design Conditions

Item	List of conditions		Reason for setting
Compliant Criteria	・ DGCS 2015 ・ Revised Commentary and Order for the Construction of River Management Facilities ・ Ministry of Construction River Erosion Control Technical Standard (draft) Design Section [I] ・ weir design ・ A guide to flexible sluice design ・ Road bridge specifications IV substructure edition		The basic shape of the weir conforms to the standard on the left.
Material Specification	Concrete	Class A	Use materials from the Philippines
	Rebar	Grade 420	PNS: Philippine National Standard

Item	List of conditions		Reason for setting
Physical Constant	Young's modulus	200,000 MPa	Material properties in the Philippines
	Young's modulus ratio	n = 9	〃
	Linear expansion coefficient	10.8 x 10 -6	〃
Allowable Stress	Concrete	fc = 8.28 N/mm2, τa = 0.36 N/mm2	〃
	Rebar	σc = 168 N/mm2	〃
	Extra factor	Wind Load 25% Temperature change 25% 33% at the time of earthquake 50% during construction	According to the setting method in the Philippines 40% premium for wind load + temperature change
Minimum Member Thickness	Minimum	0.35 m	Normal
	Round value at premium	0.05 m	
Minimum Rebar Volume	Box lateral direction,	more than 0.2% of A	A: Effective cross-sectional area of concrete
	Longitudinal direction of box	more than 0.3% of A	〃
Rebar specifications	Major bars are set outside, and minor bars are set for each part according to the basic idea in.		General specifications. In order to use rebars as constraining bars for seismic resistance, rebars are arranged in the opposite direction from the left.

Source: Study Team

1) Basic Specifications

Basic dimensions (Location, Bed Height, Section) of MCGS arranged by basic design are shown in. **Table 7.3.2**.

Table 7.3.2 Basic Specifications of MCGS

Item	Specifications	Note
Structural Classification	Movable Weir	- Considering the certainty of the diversion during a flood - Wastewater from Laguna Lake through Manggahan Floodway
Location	Sta.6 + 010	- Positions as close as possible to EFCOS (Near STA.6 + 550) considering site restrictions and river channel curvature
Design High Water Level (DFL)	Upstream: EL. 17.400 M Downstream: EL. 14.711 M	
Water Level (Structural Study)	(Flood) Upstream: EL. 17.400 M Downstream: EL. 13.425 M	- The downstream water level is the water level at the time of the hydraulic model experiment.
	(Low Water Level) Upstream: EL. 17.400 M Downstream: EL. 11.003 M	- Downstream water level is calculated from the observed water level of Rosario dam on the Marikina River side.
Levee Height (Plan)	Marikina River Side: EL. + 18.600	- DFL + 1.2 m clearance
Number of Gates	Two Gates	- Consider operational flexibility, redundancy in case of failure.
Span Length	31.8 M + 15.2 M (Clear Span: 28.7 M + 11.3 M)	- Minimum span length 12.5 m (Avoidance of blockage of water passage due to falling objects) - The normal width of the water surface of 40 m was secured with 2 gates fully open. - Only the gate between small diameter should be fully opened in case of flood, and the discharge to the downstream in case of planned flood is limited to 500 m3/s.
Weir (Sil) Height	EL. 7.850 M	- planned bed height
Demineralizer	Reduced Reservoir System L = 26.4 M, EL. 7.050 M	- Method that does not interfere with normal operation of ships or flood runoff - Depth of about 20% of the maximum water level difference between upstream and downstream - Setting based on Japan's past case 1)
Main Body (Bottom Slab) Length	20.5 M	- Set from maintenance bridge, management stair, column width, and weir width
Apron Extension	Upstream: 15 M	- Extension of 1/2 downstream, referring to cases in Japan[1)]
	Downstream: 30 M	- Set extension according to required seepage path length
	Upstream: 15 M	- Same length as the upstream Apron

Item	Specifications	Note
Extension of Floor Protection Work	Downstream: 44 M	- Calculation according to the guideline for the study of floor stop structure
Main Body Crown Height	EL. 19.00 M	- Revetment construction crown height (overfill)
Column Crown Height	EL. 32.050 M	- Excess height of 1.5 m at the time of lifting is taken into consideration for the gate height.
Gate Top Height	EL. 17.400 M	- Match DFL
Gate Type	Pull-Up Roller Gate	- Selection based on maintenance, management and economic efficiency
Gate Structure/Material	Shell Structure/Alloy Saving Duplex Stainless Steel	- (Structure) Resilience to runoff, sedimentation, and cost advantages - (Materials) Life cycle costs, applicability in estuarine waters of the Pasig Marikina River;
Operating System	Hoist: Electric (Commercial Power Supply) Operation; Remote Control and Machine Side Operation	- Use of commercial power supply; however, two auxiliary power units are installed in case of power failure. - Installation of emergency operation panels in the generator room in addition to the remote control and equipment side
Maintenance Bridge	PC Bridge (Width: 4.0 M)	- It is assumed that only vehicles for management use pass through.

1)(Dam Engineering Center) "weir design"
Source: Study Team

2) Structural Calculation and Load Conditions

The stability calculation of the main body and the structural calculation of the wing wall and breast wall are outlined below.

(a) Main Body

The MCGS has 2 spans of 11.7 m and 28.3 m in span length and has an inverse T shape and a U shape structure consisting of an end pier, end piers and a central Pier. Stability analysis are conducted on the shape of the end pier (inverse-T structure) and the end pier + central Pier (U-shaped structure).

The reasons for adopting a U-shaped pier for No.2 and No.3 are:

- It is necessary satisfies the stability as a direct foundation, because the foundation ground is firm.
- Considering the order of construction and coffering method, it is difficult to adopt a U-shaped structure for No.1 and No.2.
- In case if the No.2 pier the inverse-T structure, the required safety against overturning is not satisfied.

Structure Type:	No. 1 Pier	: Inverse T Structure
	No. 2 and No. 3 Pier	: U Structure

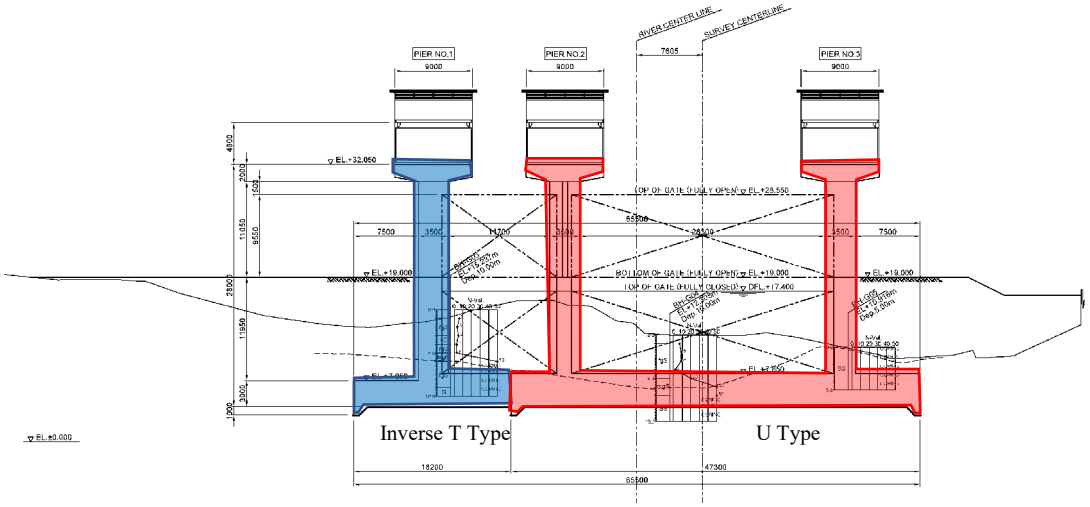


Figure 7.3.7 Structure Type of the Main Body

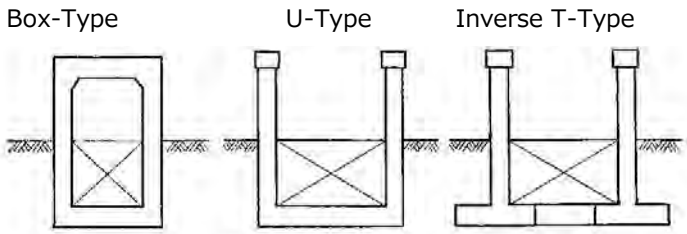


図 1-67 水門の本体の形式 *Figure 1-67 Main body type of a floodgate*

As for the type of the main body of the floodgate, box type is used for a small structure, while inverse T-type is used for a large scale, and U type is used for the intermediate scale. However, when selecting the type of structure, it is necessary to consider the quality of the foundation ground, workability (positional relationship with cofferdam), and cost.

Source: Guideline Checklist for Permitted Structures' Technical Review (MLIT, Japan)

Figure 7.3.8 Structure Type of the Piers

The foundation is directly supported by rock. Thus, it is the direct foundation. However, there is a possibility that the safety against overturning is not be satisfied in the stability analysis.

The Piers will receive a large horizontal force at an earthquake in the Direction Perpendicular to Flow. The piers are separated from each other, but they are strut structure because the body is constructed in the front. Therefore, it is not necessary to check overturning safety at the time of earthquake in this design.

The stability analysis is conducted for both in the flow direction and perpendicular to flow, for the case of normal, flood, seismic and at the time of construction, respectively.

In each case, the stability of sliding, overturning, and bearing capacity are calculated. **Table 7.3.3** shows the required safety factors.

Table 7.3.3 Safety Factor

Item	During normal times and floods ・during construction	Seismic	Remarks
Sliding	1.5	1.2	
Overturning	L/6	L/3	L: Foundation width
Bearing	3.0	2.0	Calculation of ultimate bearing capacity of foundation ground by Terzaghi' Concept.

Source: Reorganization of the investigation team

(b) Breast Wall

In principle, the breast wall shall be designed as a cantilever beam fixed to the body.

(c) Wing Wall

The cross-sectional direction of the wing wall is considered as an inverse T-type wall, and that is the flowing direction is considered as a L-type wall.

As for the inverse T-type part, the sliding check in the stability analysis is omitted because it is connected to the apron. Since it is necessary to consider the passive force of the apron, the apron should be constructed prior to the wing wall.

As for the L-type part, since the front surface of the retaining wall locates at a riverbank and a revetment is to be installed, the earth pressure acting on the front surface shall be considered when considering stability.

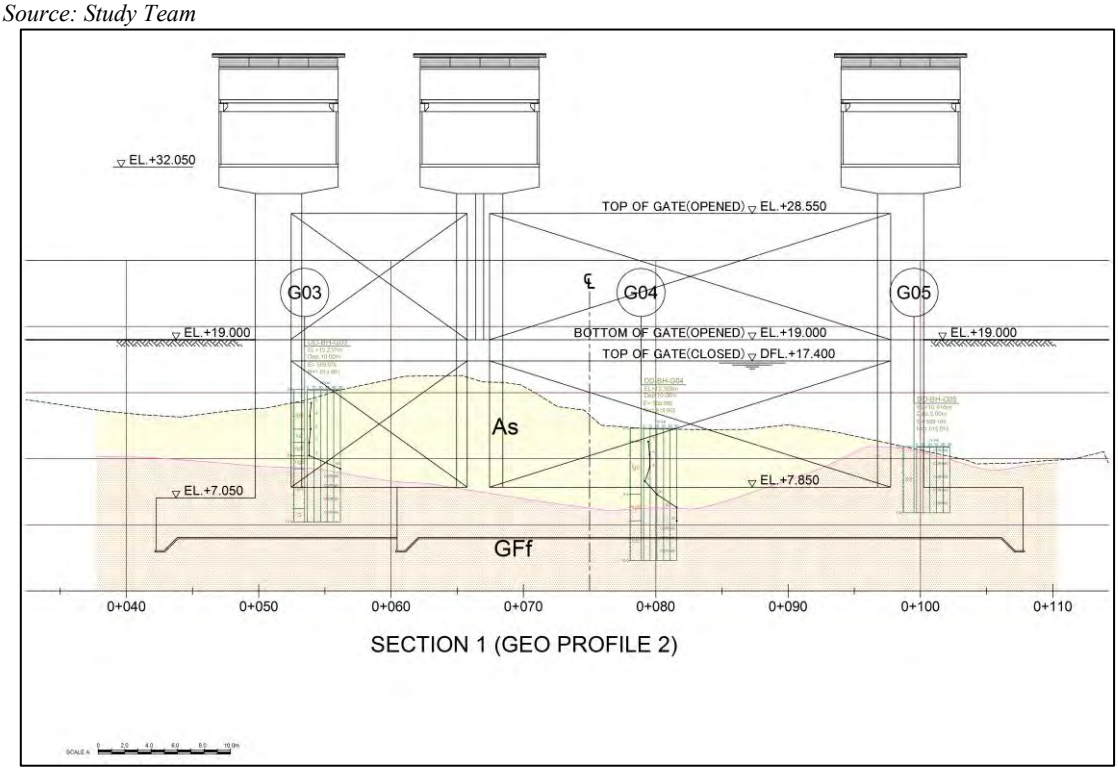
(2) Ground Condition

Based on the results of geological surveys conducted at the site, the ground conditions of the MCGS (formation components and soil constants) are set as follows:

Table 7.3.4 Soil Constants

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 (°)
As	Sandy Soil	9.9	30	30 to 20	9	17.5	0	30
GFf	Weathered Rock	121.5	8	-	-	17	1,000	30

Source: Study Team



Source: Study Team

Figure 7.3.9 Geological Map

(3) Water Level Condition

The design water level of the MCGS is shown in **Table 7.3.5**. The downstream water level at the time of flood used for structural examination is obtained by the hydraulic model experiment.

Table 7.3.5 Design Water Levels of MCGS

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

Source: Study Team

(4) Construction Condition

The main construction conditions of the MCGS are as follows.

Table 7.3.6 Construction Condition

Item	Condition	Remarks
Excavation Gradient	The excavation slope shall be a gentle slope of 1: 2.0, because there is no data on the soil properties of the embankment.	
Temporary Drain	Because it is difficult to operate the whole river due to the restriction of the site, the water is drained by the half river closing.	
Construction Road	On the right bank side, there are many houses on the bank, making it difficult to plan an access route. Land for the zaibatsu on the left bank side will be leased and an access road will be constructed.	
Construction	In the second stage of construction, water transportation is planned because land transportation is difficult.	

Source: Study Team

7.3.2.2 Foundation Work

As mentioned above in the overview of ground conditions, the rock layer is distributed at the site where MCGS is to be installed, and therefore, the direct foundation is adopted.

7.3.2.3 Detailed Design of The Main Body

(1) Stability Calculation

1) Design Policy of the Piers

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

Stability studies will be conducted on these shapes respectively.

In this weir, the foundation is directly supported by rock. Thus, it is the direct foundation. However, there is a possibility that the safety against overturning is not be satisfied in the stability analysis.

The Piers will receive a large horizontal force at an earthquake in the Direction Perpendicular to Flow. The piers are separated from each other, but they are strut structure because the body is constructed in the front. Therefore, it is not necessary to check overturning safety at the time of earthquake in this design.

(a) Load Case

The calculation is carried out in 8 cases in in the flow direction and 4 cases in the direction perpendicular to flow (**Table 7.3.7**).

Table 7.3.7 Load Cases (End Pier (No. 1))

			Body Weight	Topsoil of Parapet	Breast Wall Water Load	Internal Water Weight	Shed Load	Gate Load	Managed Bridge Load	Water Pressure (Upstream)	Water Pressure (Downstream)	Earth Pressure	Wind Load	Uplift Pressure	Seismic Inertia Force
A. Flow Direction	Case 1	Normal + wind load1 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○	
	Case 2	Normal + wind load2 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○	
	Case 3	Flood + wind load 1 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○	
	Case 4	Flood + wind load 2 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○	
	Case 5	Time of seismic 1 (upstream to downstream)	○	○	○	○	○	○	○	○	○			○	○
	Case 6	At the time of earthquake 2 (upstream to downstream)	○	○	○	○	○	○	○	○	○			○	○
	Case 7	Construction time 1 (upstream to downstream)	○	○			○	○	○				○		
	Case 8	Construction time 2 (upstream to downstream)	○	○			○	○	○				○		
B. Direction Perpendicular to Flow	Case 1	Normal + wind load (land side → river side)	○	○	○	○	○	○	○	○		○	○	○	
	Case 2	Flood + wind load (land side → river side)	○	○	○	○	○	○	○	○		○	○	○	
	Case 3	Seismic (land side → river side)	○	○	○	○	○	○	○	○		○		○	○
	Case 4	During construction (land side → river side)	○	○			○	○	○			○	○		

Source: Study Team

Table 7.3.8 Load Cases (Central Pier (No. 2) + End Pier (No. 3))

			Body Weight	Topsoil of Parapet	Breast Wall Water Load	Internal Water Weight	Shed Load	Gate Load	Managed Bridge Load	Water Pressure (Upstream)	Water Pressure (Downstream)	Earth Pressure	Wind Load	Uplift Pressure	Seismic Inertia Force	Hydrodynamic Pressure
A. Flow Direction	Case 1	Normal + wind load1 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○		
	Case 2	Normal + wind load2 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○		
	Case 3	Flood + wind load 1 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○		
	Case 4	Flood + wind load 2 (upstream to downstream)	○	○	○	○	○	○	○	○	○		○	○		
	Case 5	Time of seismic 1 (upstream to downstream)	○	○	○	○	○	○	○	○	○			○	○	○
	Case 6	At the time of earthquake 2 (upstream to downstream)	○	○	○	○	○	○	○	○	○			○	○	○
	Case 7	Construction time 1 (upstream to downstream)	○	○			○	○	○				○			
	Case 8	Construction time 2 (upstream to downstream)	○	○			○	○	○				○			
B. Direction Perpendicular to Flow	Case 1	Normal + wind load (land side → river side)	○	○	○	○	○	○	○	○		○	○	○		
	Case 2	Flood + wind load (land side → river side)	○	○	○	○	○	○	○	○		○	○	○		
	Case 3	Seismic (land side → river side)	○	○	○	○	○	○	○	○		○		○	○	○
	Case 4	During construction (land side → river side)	○	○			○	○	○			○	○			

Source: Study Team

(b) Design Water Level

Table 7.3.9 shows the design water levels.

Table 7.3.9 Design Water Levels

Item	Properties	Remarks
Design high water level (DFL)	Upstream: EL. 17.400 m Downstream: EL. 14.711 m	
Normal water level (OWL)	Weir shafts: EL. 11.090 m	
Low water level (LWL)	Weir shafts: EL. 10.800 m	
Water level conditions (structural study)	(flood) Upstream: EL. 17.400 m Downstream: EL. 13.425 m	- The downstream water level is the water level at the time of the hydraulic model experiment.
	(low water level) Upstream: EL. 17.400 m Downstream: EL. 11.003 m	- Downstream water level is calculated from the observed water level of Rosario dam on the Marikina River side.
	(residual water level outside the structure) Outside: EL. 16.670 m	- Highest in-hole water level (EL 15.20 m) and 2/3 water level of HWL in the vicinity boring

Source: Study Team

2) Stability Calculation of End Pier (No. 1 Pier)

(a) Loads

Loads for the end piers are shown in **Table 7.3.10**.

Table 7.3.10 Schedule of Load on End Pier

			Vertical Load V (kN)	Moment Mx (kN m)	Horizontal Load H (kN)	Moment My (kN m)
A. Flow Direction	Case 1	Normal + Wind Load1 (Upstream to downstream)	76405.13	770253.96	-945.21	-21087.16
	Case 2	Normal + Wind Load2 (Upstream to downstream)	76405.13	770253.96	698.39	20282.26
	Case 3	Flood + wind load 1 (Upstream to downstream)	64818.70	658568.51	-1747.44	-22686.66
	Case 4	Flood + wind load 2 (Upstream to downstream)	64818.70	658568.51	-103.84	18682.76
	Case 5	Time of seismic 1 (Upstream to downstream)	75424.58	758790.60	-19314.30	-162042.67
	Case 6	At the time of earthquake 2 (Upstream to downstream)	75424.58	758790.60	19047.04	161156.65
	Case 7	Construction time 1 (Upstream to downstream)	95527.47	964403.85	-821.80	-20684.71
	Case 8	Construction time 2 (Upstream to downstream)	95527.47	964403.85	821.80	20684.71
B. Direction Perpendicular to Flow	Case 1	Normal + Wind Load (Land side → River side)	81190.96	534959.38	-25035.89	-133234.82
	Case 2	Flood + wind load (Land side → River side)	70072.03	471081.95	-22335.20	-117119.91
	Case 3	Seismic (Land side → River side)	79725.45	529687.29	-48721.68	-297963.50
	Case 4	During construction (Land side → River side)	102446.12	689681.85	-12731.43	-81304.20

Source: Study Team

(b) Stability Check

Stability of the main body of MCGS was examined using the combined loads of the main body. The calculation results are shown in **Table 7.3.11** and **Table 7.3.12**.

Table 7.3.11 Result of stability calculation of End Pier (Flow Direction)

			Case 1	Case 2	Case 3	Case 4
Overturning	Point of action of resultant force	$d=\frac{(\Sigma w \cdot X-\Sigma H \cdot Y)}{\Sigma w}$	10.36	9.82	10.51	9.87
	Eccentric distance	e = B/2 - d (m)	0.11	0.43	0.26	0.38
	Tolerance	ea (m)	3.42	3.42	3.42	3.42
	Judgment		OK	OK	OK	OK
Sliding	Coefficient of friction	μ	0.7	0.7	0.7	0.7
	Cohesion of the ground	C (kN/m2)	13	13	13	13
	Safety factor	$F=\frac{\Sigma V \cdot \mu+c \cdot B}{\Sigma H}$	56.6	76.6	26.0	437.0
	Allowable safety factor	Fs	1.5	1.5	1.5	1.5
	Judgment		OK	OK	OK	OK
			Case 5	Case 6	Case 7	Case 8
Overturning	Point of action of resultant force	$d=\frac{(\Sigma w \cdot X-\Sigma H \cdot Y)}{\Sigma w}$	12.21	7.92	10.31	9.88
	Eccentric distance	e = B/2 - d (m)	1.96	2.33	0.06	0.37
	Tolerance	ea (m)	6.83	6.83	3.42	3.42
	Judgment		OK	OK	OK	OK
Sliding	Coefficient of friction	μ	0.7	0.7	0.7	0.7
	Cohesion of the ground	C (kN/m2)	13	13	13	13
	Safety factor	$F=\frac{\Sigma V \cdot \mu+c \cdot B}{\Sigma H}$	2.7	2.8	81.4	81.4
	Allowable safety factor	Fs	1.2	1.2	1.5	1.5
	Judgment		OK	OK	OK	OK

Source: Study Team

Table 7.3.12 Result of Stability Calculation of End Pier (Direction Perpendicular to Flow)

			Case 1	Case 2	Case 3	Case 4
Overturning	Point of action of resultant force	$d=\frac{(\Sigma w \cdot X-\Sigma H \cdot Y)}{\Sigma w}$	8.23	8.39	10.38	7.53
	Eccentric distance	e = B/2 - d (m)	0.87	0.71	1.28	1.57
	Tolerance	ea (m)	3.03	3.03	6.07	3.03
	Judgment		OK	OK	OK	OK
Sliding	Coefficient of friction	μ	0.7	0.7	0.7	0.7
	Cohesion of the ground	C (kN/m2)	13	13	0	13
	Safety factor	$F=\frac{\Sigma V \cdot \mu+c \cdot B}{\Sigma H}$	2.3	2.2	1.1	5.6
	Allowable safety factor	Fs	1.5	1.5	1.2	1.5
	Judgment		OK	OK	NG	OK

Source: Study Team

(c) Check of Bearing Capacity

(i) Calculation Method

The rock (GFf) soil constant of the supporting ground of this design is as follows.

Unit volume weight γ = 17.0 kN/m^3
Internal friction angle ϕ = 0 °
Cohesion C = 3500 kN/m^2

Allowable bearing capacity (Qa) is obtained by the ultimate bearing capacity using the safety factor shown below1,2.

$$Qa = \frac{Qu}{SF}$$

SF = 3 (normal)
SF = 2 (seismic)

Ultimate bearing capacity of foundation ground is calculated with the formula below.

$$Qu = A'\left\{\alpha\, k\, c\, N_c + k\, q\, N_q + \frac{1}{2}\gamma_1\, \beta\, B'\, N_r\right\}$$

Where,

Qu : Ultimate bearing capacity (kN)

A’ : Effective loading area on footing (m2) (Refer to following **Table 7.3.13**)

α , β : Shape factor of foundation as shown in the following table

Table 7.3.13 Shape Factor of Foundation

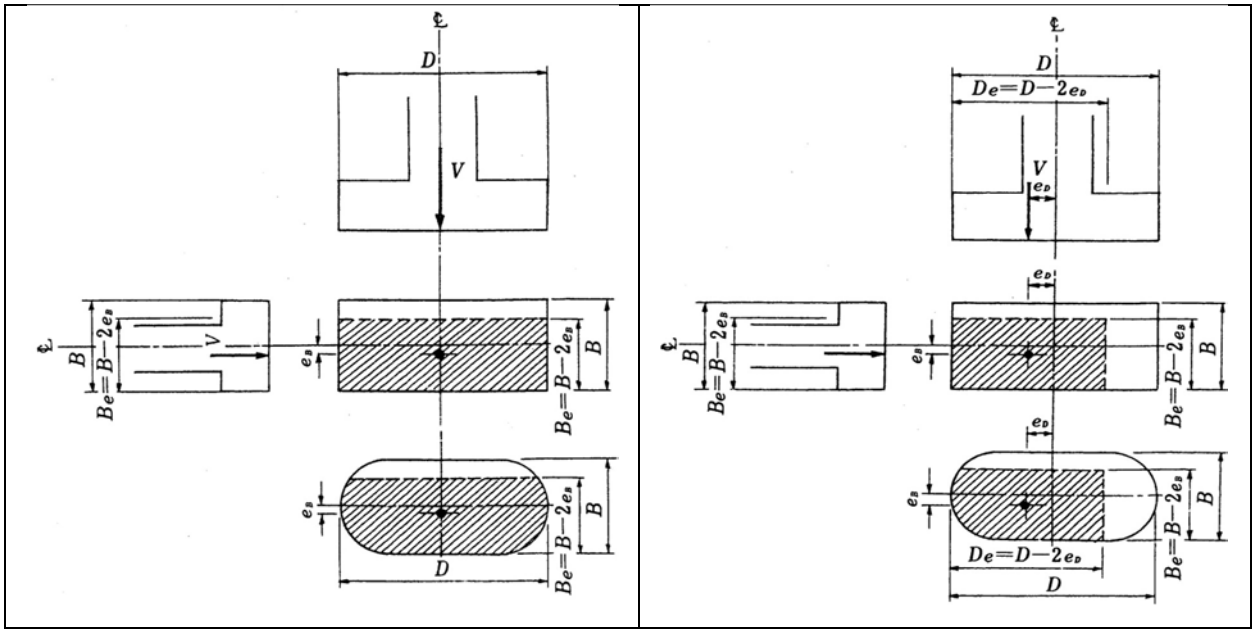
Shape of Foundation	α	β
Zonation	1.0	1.0
Circle or Square	1.3	0.6
Rectangle or ellipse	1+0.3B’/L’	1-0.4B’L’

Source: Specifications for Highway Bridges IV Substructures Table 10.3.3

C : Cohesion of foundation ground (kN/m2)
q : Surcharge (kN/m2)
= γ_2 · Df
γ_1, γ_2 : Unit weight of soil of bearing ground (kN/m³)
B’,L’ : Width and length of effective loading areas as shown in the following figure

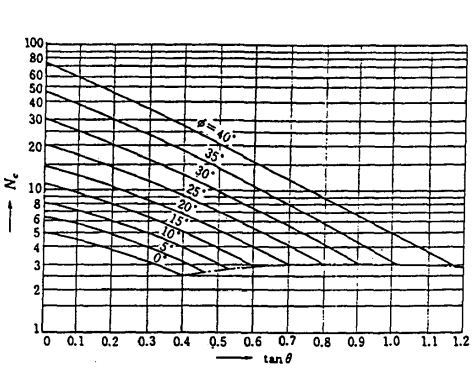
[1] NSCP Vol. II Bridges (ASD) 4.4.7.1 Bearing Capacity
[2] Specifications for Highway Bridges IV Substructures 10.3.1 Allowable Bearing Capacity of Foundation Ground

B’ = B – 2eH
L’ = L – 2eL
e : Distance from center of footing to point of resultant force on footing as illustrated in the following figure (m)
Df : Depth from ground surface to bottom of footing (m)
K : Coefficient (1+0.3 x Df’/B)
Df’ : Effective embedded depth of foundation (m)
Nc, Nq, Nr: Bearing capacity factors (refer to **Figure 7.3.11**)

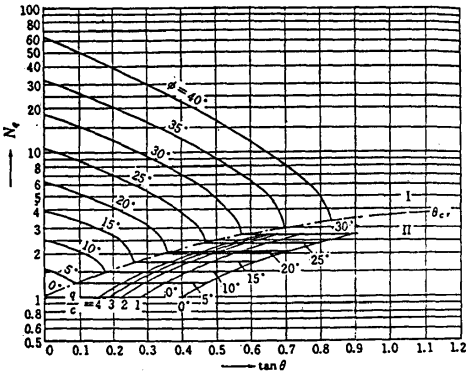


Source: Specifications for Highway Bridges, Volume IV Substructures, Japan Road Association, 2012, Table 10.3.4, 10.3.5

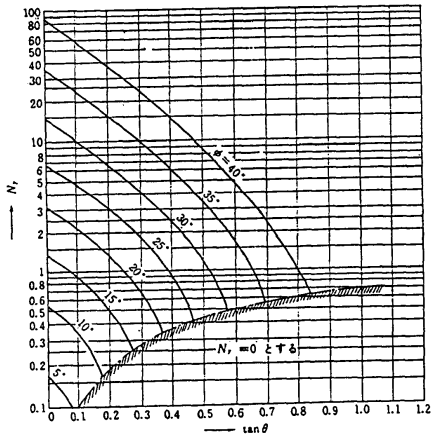
Figure 7.3.10 Effective Loading Area on Footing



Graph for Bearing Capacity Factor Nc



Graph for Bearing Capacity Factor Nq



Graph for Bearing Capacity Factor Nr

Source: Specifications for Highway Bridges, Volume IV Substructures, Japan Road Association, 2012, Table 10.3.1, 10.3.2, 10.3.3

Figure 7.3.11 Graphs for Bearing Capacity Factor

(ii) Check of Bearing Capacity

The allowable bearing capacity of the ground is calculated as follows.

Table 7.3.14 Axial Direction of Bridge

(B = 20.500 (m), D = 18.200 (m)

No	MB (kNm)	eB (m)	Be (m)	α	β	κ	HB (kN)	q (kN/m2)	Sp	Sr
1	8404.56	0.110	20.280	1.300	0.600	1.044	945.21	30.600	0.689	0.367
2	32854.21	0.430	19.640	1.300	0.600	1.046	698.39	30.600	0.689	0.371
3	16852.86	0.260	19.980	1.300	0.600	1.045	1747.44	30.600	0.689	0.369
4	24631.11	0.380	19.740	1.300	0.600	1.046	103.84	30.600	0.689	0.370
5	147832.18	1.960	16.580	1.273	0.636	1.054	19314.30	30.600	0.689	0.392
6	175739.27	2.330	15.840	1.261	0.652	1.057	19047.04	30.600	0.689	0.398
7	5731.65	0.060	20.380	1.300	0.600	1.044	821.80	30.600	0.689	0.366
8	35345.16	0.370	19.760	1.300	0.600	1.046	821.80	57.000	0.560	0.370

No	Load name	Load Conditions	Nc	Nq	Nr	Qu (kN)	Qa (kN)	V (kN)	Judg ment
1	Normal	Normal	5.08	1.00	0.00	4141115.41	1380371.80	76405.13	OK
2	Normal	Normal	5.09	1.00	0.00	4029529.74	1343176.58	76405.13	OK
3	Normal	Normal	5.00	1.00	0.00	4018711.39	1339570.46	64818.70	OK
4	Normal	Normal	5.13	1.00	0.00	4080408.85	1360136.28	64818.70	OK
5	Seismic	Seismic	3.49	1.00	0.00	2301736.59	1150868.30	75424.58	OK
6	Seismic	Seismic	3.51	1.00	0.00	2198862.85	1099431.43	75424.58	OK
7	During construction	Seismic	5.10	1.00	0.00	4177026.89	2088513.45	95527.47	OK
8	During construction	Seismic	5.10	1.00	0.00	4059400.69	2029700.35	95527.42	OK

Source: Study Team

Table 7.3.15 Direction Perpendicular to The Bridge Axis

(B = 18.200 (m), D = 20.500 (m)

No	MB (kNm)	eB (m)	Be (m)	α	β	κ	HB (kN)	q (kN/m2)	Sp	Sr
1	70636.14	0.870	16.460	1.241	0.679	1.055	25035.89	30.600	0.689	0.393
2	49751.14	0.710	16.780	1.246	0.673	1.054	22335.20	30.600	0.689	0.391
3	102048.58	1.280	15.640	1.229	0.695	1.058	48721.68	30.600	0.689	0.400
4	160840.41	1.570	15.060	1.220	0.706	1.060	12731.43	57.000	0.560	0.405

No	load name	Load Conditions	Nc	Nq	Nr	Qu (kN)	Qa (kN)	V (kN)	judgment
1	Normal	Normal	3.12	1.00	0.00	2245794.18	748598.06	81190.96	OK
2	Normal	Normal	3.05	1.00	0.00	2242787.94	747595.98	70072.03	OK
3	Seismic	Seismic	2.50	0.00	0.00	1692282.52	846141.26	79725.45	OK
4	During construction	Seismic	4.40	1.00	0.00	2864734.08	1432367.04	102446.12	OK

Source: Study Team

(iii) Result of Stability Calculations

Results of stability calculation to the maximum ground reaction are shown in **Table 7.3.16** and **Table 7.3.17**.

Table 7.3.16 Result of Stability Calculations (Axial Direction of Bridge)

No	Loads State	Acting force at the base center		Reaction force action width (m)	Subgrade reaction (kN/m2)		Maximum subgrade reaction (kN/m2)	Judgment
		M (kNm)	V (kN)					
1	Normal	8404.56	76405.13	20.500	211.378	≦	1000.000	OK
2	Normal	32854.21	76405.13	20.500	230.557	≦	1000.000	OK
3	Normal	16852.86	64818.70	20.500	186.951	≦	1000.000	OK
4	Normal	24631.11	64818.70	20.500	193.052	≦	1000.000	OK
5	Seismic	147832.18	75424.58	20.500	318.125	≦	1500.000	OK
6	Seismic	175739.27	75424.58	20.500	340.017	≦	1500.000	OK
7	Seismic	5731.65	95527.47	20.500	260.533	≦	1000.000	OK
8	Seismic	35345.16	95527.42	20.500	283.764	≦	1000.000	OK

Source: Study Team

Table 7.3.17 Result of Stability Calculations (Direction Perpendicular to the Bridge Axis)

No	Loads State	Acting force at the base center		Reaction force action width (m)	Subgrade reaction (kN/m2)		Maximum subgrade reaction (kN/m2)	Judgment
		M (kNm)	V (kN)					
1	Normal	70636.14	81190.96	18.200	280.026	≦	1000.000	OK
2	Normal	49751.14	70072.03	18.200	231.770	≦	1000.000	OK
3	Seismic	102048.58	79725.45	18.200	303.854	≦	1500.000	OK
4	Seismic	160840.41	102446.12	18.200	416.699	≦	1000.000	OK

Source: Study Team

Results of the analysis for the vertical subgrade reaction are shown in the following Tables.

Table 7.3.18 Result of Stability Calculations (Axial Direction of Bridge)

No	Loads State	Acting force at the base center			Acting Vertical Force (kN) Allowable Value (kN)	Judgment
		M(kN.m)	V(kN)	H(kN)		
1	Normal	8404.56	76405.13	945.21	76405.13 ≦ 1380371.80	OK
2	Normal	32854.21	76405.13	698.39	76405.13 ≦ 1343176.58	OK
3	Normal	16852.86	64818.70	1747.44	64818.70 ≦ 1339570.46	OK
4	Normal	24631.11	64818.70	103.84	64818.70 ≦ 1360136.28	OK
5	Seismic	147832.18	75424.58	19314.30	75424.58 ≦ 1150868.30	OK
6	Seismic	175739.27	75424.58	19047.04	75424.58 ≦ 1099431.43	OK
7	Seismic	5731.65	95527.47	821.80	95527.47 ≦ 2088513.45	OK
8	Seismic	35345.16	95527.42	821.80	95527.42 ≦ 2029700.35	OK

Source: Study Team

Table 7.3.19 Result of Stability Calculations (Direction Perpendicular to the Bridge Axis)

No	Loads State	Acting force at the base center			Acting Vertical Force (kN) Allowable Value (kN)	Judgment
		M(kN.m)	V(kN)	H(kN)		
1	Normal	70636.14	81190.96	25035.89	81190.96 ≦ 748598.06	OK
2	Normal	49751.14	70072.03	22335.20	70072.03 ≦ 747595.98	OK
3	Seismic	102048.58	79725.45	48721.68	79725.45 ≦ 846141.26	OK
4	Seismic	160840.41	102446.12	12731.43	102446.12 ≦ 1432367.04	OK

Source: Study Team

3) Stability Calculation of Central Pier (No. 2 pier) + End Pier (No. 3 Pier)

(a) Loads

Load schedule for the central pier + end pier is shown in **Table 7.3.20**.

Table 7.3.20 Schedule of Load on Central and End Piers

			Vertical Load V (kN)	Moment Mx (kN m)	Horizontal Load H (kN)	Moment My (kN m)
A. Flow Direction	Case 1	Normal + Wind Load1 (Upstream to downstream)	134012.22	1359818.74	-3338.01	-63817.43
	Case 2	Normal + Wind Load2 (Upstream to downstream)	134012.22	1359818.74	2213.63	60150.69
	Case 3	Flood + wind load 1 (Upstream to downstream)	118583.36	1201268.20	-12228.13	-101212.04
	Case 4	Flood + wind load 2 (Upstream to downstream)	118583.36	1201268.20	-8699.06	-18429.79
	Case 5	Time of seismic 1 (Upstream to downstream)	132738.33	1340337.04	-35777.52	-276733.00
	Case 6	At the time of earthquake 2 (Upstream to downstream)	132738.33	1340337.04	34575.13	272751.60
	Case 7	Construction time 1 (Upstream to downstream)	176417.23	1786023.62	-2793.48	-62266.67
	Case 8	Construction time 2 (Upstream to downstream)	176417.23	1786023.62	2793.48	62266.67
B. Direction Perpendicular to Flow	Case 1	Normal + Wind Load (Land side → River side)	138798.05	3961203.68	26484.86	160562.25
	Case 2	Flood + wind load (Land side → River side)	123836.69	3472429.98	23349.52	140322.65
	Case 3	Seismic (Land side → River side)	137039.20	3901386.67	68772.64	432018.80
	Case 4	During construction (Land side → River side)	183335.88	5130922.85	13711.63	103572.42

Source: Study Team

(b) Stability Check

In order to examine the stability of the main body of the weir, the stability was examined from the combined loads of the main body. The result of stability calculations are shown in **Table 7.3.21** and **Table 7.3.22**.

Table 7.3.21 Central Pier + End Pier (Flow Direction) Result of stability calculations

			Case 1	Case 2	Case 3	Case 4
Overturning	Point of action of resultant force	$d=\frac{(\Sigma w \cdot X-\Sigma H \cdot Y)}{\Sigma w}$	10.62	9.70	10.98	10.29
	Eccentric distance	e = B/2 - d (m)	0.37	0.55	0.73	0.04
	Tolerance	ea (m)	3.42	3.42	3.42	3.42
	Judgment		OK	OK	OK	OK
Sliding	Coefficient of friction	μ	0.7	0.7	0.7	0.7
	Cohesion of the ground	C (kN/m2)	13	13	13	13
	Safety factor	$F=\frac{\Sigma V \cdot \mu+c \cdot B}{\Sigma H}$	28.1	42.4	6.8	9.5
	Allowable safety factor	Fs	1.5	1.5	1.5	1.5
	Judgment		OK	OK	OK	OK
			Case 5	Case 6	Case 7	Case 8
Overturning	Point of action of resultant force	$d=\frac{(\Sigma w \cdot X-\Sigma H \cdot Y)}{\Sigma w}$	12.18	8.04	10.48	9.77
	Eccentric distance	e = B/2 - d (m)	1.93	2.21	0.23	0.48
	Tolerance	ea (m)	6.83	6.83	3.42	3.42
	Judgment		OK	OK	OK	OK
Sliding	Coefficient of friction	μ	0.7	0.7	0.7	0.7
	Cohesion of the ground	C (kN/m2)	13	13	13	13
	Safety factor	$F=\frac{\Sigma V \cdot \mu+c \cdot B}{\Sigma H}$	2.6	2.7	44.2	44.2
	Allowable safety factor	Fs	1.2	1.2	1.5	1.5
	Judgment		OK	OK	OK	OK

Source: Study Team

Table 7.3.22 End Pier (Flow Direction) Result of stability calculation

			Case 1	Case 2	Case 3	Case 4
Overturning	Point of action of resultant force	$d=\frac{(\Sigma w \cdot X-\Sigma H \cdot Y)}{\Sigma w}$	27.38	26.91	25.32	27.42
	Eccentric distance	e = B/2 - d (m)	3.73	3.26	1.67	3.77
	Tolerance	ea (m)	7.88	7.88	15.77	7.88
	Judgment		OK	OK	OK	OK
Sliding	Coefficient of friction	μ	0.7	0.7	0.7	0.7
	Cohesion of the ground	C (kN/m2)	13	13	0	13
	Safety factor	$F=\frac{\Sigma V \cdot \mu+c \cdot B}{\Sigma H}$	3.7	3.7	1.4	9.4
	Allowable safety factor	Fs	1.5	1.5	1.2	1.5
	Judgment		OK	OK	OK	OK

Source: Study Team

(c) Calculation of Bearing Capacity

(i) Calculation Method

The calculation method is explained in (**7.3.2.3 (1) 2) (c)**).

(ii) Check of Bearing Capacity

The allowable bearing capacity of the ground is calculated as follows.

Table 7.3.23 Axial Direction of Bridge

(B = 20.500 (m), D = 47.300 (m)

No	MB (kNm)	eB (m)	Be (m)	α	β	κ	HB (kN)	q (kN/m2)	Sp	Sr
1	49584.52	0.370	19.760	1.125	0.833	1.046	3338.01	30.600	0.689	0.370
2	73706.72	0.550	19.400	1.123	0.836	1.046	2213.63	30.600	0.689	0.372
3	86565.85	0.730	19.040	1.121	0.839	1.047	12228.13	30.600	0.689	0.374
4	4743.33	0.040	20.420	1.130	0.827	1.044	8699.06	30.600	0.689	0.366
5	256184.98	1.930	16.640	1.106	0.859	1.054	35777.52	30.600	0.689	0.392
6	293351.71	2.210	16.080	1.102	0.864	1.056	34575.13	30.600	0.689	0.396
7	40575.96	0.230	20.040	1.127	0.831	1.045	2793.48	30.600	0.689	0.368
8	84680.27	0.480	19.540	1.124	0.835	1.046	2793.48	57.000	0.560	0.371

No	Loads State	Nc	Nq	Nr	Qu (kN)	Qa (kN)	V (kN)	Judgment
1	Normal	5.01	1.00	0.00	8968705.13	2989568.38	134012.22	OK
2	Normal	5.05	1.00	0.00	8874846.45	2958282.15	134012.22	OK
3	Normal	4.54	1.00	0.00	7813340.96	2604446.99	118583.36	OK
4	Normal	4.73	1.00	0.00	8765670.68	2921890.23	118583.36	OK
5	Seismic	3.39	1.00	0.00	5072418.39	2536209.19	132738.33	OK
6	Seismic	3.46	1.00	0.00	4986665.05	2493332.52	132738.33	OK
7	Seismic	5.06	1.00	0.00	9194345.39	4597172.70	176417.23	OK
8	Seismic	5.06	1.00	0.00	8960083.25	4480041.62	176417.23	OK

Source: Study Team

Table 7.3.24 Direction Perpendicular to The Bridge Axis

(B = 47.300 (m), D = 20.500 (m)

No	MB (kNm)	eB (m)	Be (m)	α	β	κ	HB (kN)	q (kN/m2)	Sp	Sr
1	517716.73	3.730	39.840	1.300	0.600	1.023	26484.86	30.600	0.689	0.293
2	403707.61	3.260	40.780	1.300	0.600	1.022	23349.52	30.600	0.689	0.291
3	228855.46	1.670	43.960	1.300	0.600	1.020	68772.64	30.600	0.689	0.283
4	691176.27	3.770	39.760	1.300	0.600	1.023	13711.63	57.000	0.560	0.293

No	Loads State	Nc	Nq	Nr	Qu (kN)	Qa (kN)	V (kN)	Judgment
1	Normal	3.95	1.00	0.00	6978937.49	2326312.50	138798.05	OK
2	Normal	3.96	1.00	0.00	7168442.44	2389480.81	123836.69	OK
3	Seismic	2.50	0.00	0.00	4855469.19	2427734.60	137039.20	OK
4	Seismic	4.72	1.00	0.00	8329810.74	4164905.37	183335.88	OK

Source: Study Team

(iii) Result of Stability Calculations

Results of stability calculation to the maximum ground reaction are shown in **Table 7.3.25** and **Table 7.3.26**.

Table 7.3.25 Result of Stability Calculations (Axial Direction of Bridge)

No	Loads State	Acting force at the base center		Reaction force Action width (m)	Subgrade reaction (kN/m2)		Maximum subgrade reaction (kN/m2)	Judgment
		M (kNm)	V (kN)					
1	Normal	49584.52	134012.22	20.500	153.174	≦	1000.000	OK
2	Normal	73706.72	134012.22	20.500	160.455	≦	1000.000	OK
3	Normal	86565.85	118583.36	20.500	148.424	≦	1000.000	OK
4	Normal	4743.33	118583.36	20.500	123.727	≦	1000.000	OK
5	Seismic	256184.98	132738.33	20.500	214.221	≦	1500.000	OK
6	Seismic	293351.71	132738.33	20.500	225.439	≦	1500.000	OK
7	Seismic	40575.96	176417.23	20.500	194.187	≦	1000.000	OK
8	Seismic	84680.27	176417.23	20.500	207.499	≦	1000.000	OK

Source: Study Team

Table 7.3.26 Result of Stability Calculations (Direction Perpendicular to the Bridge Axis)

No	Loads State	Acting force at the base center		Reaction force Action width (m)	Subgrade reaction (kN/m2)	Maximum subgrade reaction (kN/m2)	Judgment
		M (kNm)	V (kN)				
1	Normal	517716.73	138798.05	47.300	210.870 ≦	1000.000	OK
2	Normal	403707.61	123836.69	47.300	180.526 ≦	1000.000	OK
3	Seismic	228855.46	137039.20	47.300	171.267 ≦	1500.000	OK
4	Seismic	691176.27	183335.88	47.300	279.494 ≦	1000.000	OK

Source: Study Team

Results of the analysis for the vertical subgrade reaction are shown in the following Tables.

Table 7.3.27 Result of Stability Calculations (Axial Direction of Bridge)

No	Loads State	Acting force at the base center			Acting Vertical Force (kN) Allowable Value (kN)	Judgment
		M(kN.m)	V(kN)	H(kN)		
1	Normal	49584.52	134012.22	3338.01	134012.22 ≦ 2989568.38	OK
2	Normal	73706.72	134012.22	2213.63	134012.22 ≦ 2958282.15	OK
3	Normal	86565.85	118583.36	12228.13	118583.36 ≦ 2604446.99	OK
4	Normal	4743.33	118583.36	8699.06	118583.36 ≦ 2921890.23	OK
5	Seismic	256184.98	132738.33	35777.52	132738.33 ≦ 2536209.19	OK
6	Seismic	293351.71	132738.33	34575.13	132738.33 ≦ 2493332.52	OK
7	Seismic	40575.96	176417.23	2793.48	176417.23 ≦ 4597172.70	OK
8	Seismic	84680.27	176417.23	2793.48	176417.23 ≦ 4480041.62	OK

Source: Study Team

Table 7.3.28 Result of Stability Calculations (Direction Perpendicular to the Bridge Axis)

No	Loads State	Acting force at the base center			Acting Vertical Force (kN) Allowable Value (kN)	Judgment
		M(kN.m)	V(kN)	H(kN)		
1	Normal	517716.73	138798.05	26484.86	138798.05 ≦ 2326312.50	OK
2	Normal	403707.61	123836.69	23349.52	123836.69 ≦ 2389480.81	OK
3	Seismic	228855.46	137039.20	68772.64	137039.20 ≦ 2427734.60	OK
4	Seismic	691176.27	183335.88	13711.63	183335.88 ≦ 4164905.37	OK

Source: Study Team

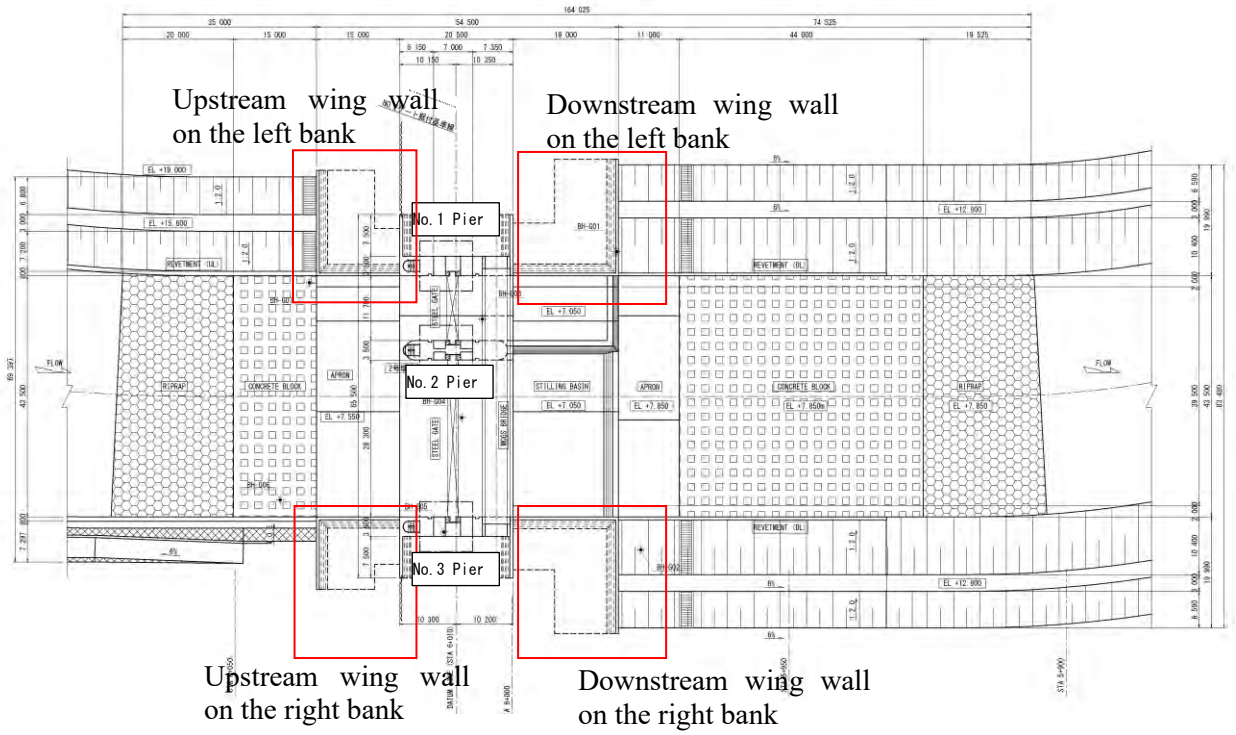
4) Design Policy of the Wing Wall

Location of the Wing walls are highlighted in **Figure 7.3.12** by the red lines. Also, cross section is shown in **Figure 7.3.13**. The transverse direction of the wing wall is considered as the inverse T retaining wall, and the flow direction wing is considered as the L-type retaining wall.

As for the inverse T retaining wall, the sliding check in the stability analysis is omitted because it is connected to the Apron.

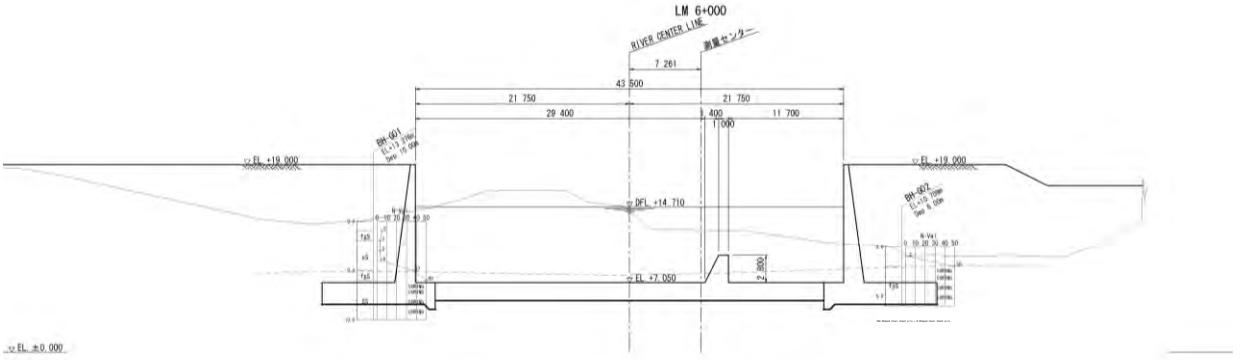
Since the front surface of the L-shaped retaining wall is a river channel, and a revetment is to be installed, the earth pressure acting on the front surface should be considered when considering stability.

PLAN



Source: Study Team

Figure 7.3.12 Wing Wall Plan



Source: Study Team

Figure 7.3.13 Cross Section Viewed from Downstream

(a) Loads

The loads for the case of normal, flood, seismic and at the time of construction are considered (**Table 7.3.29**).

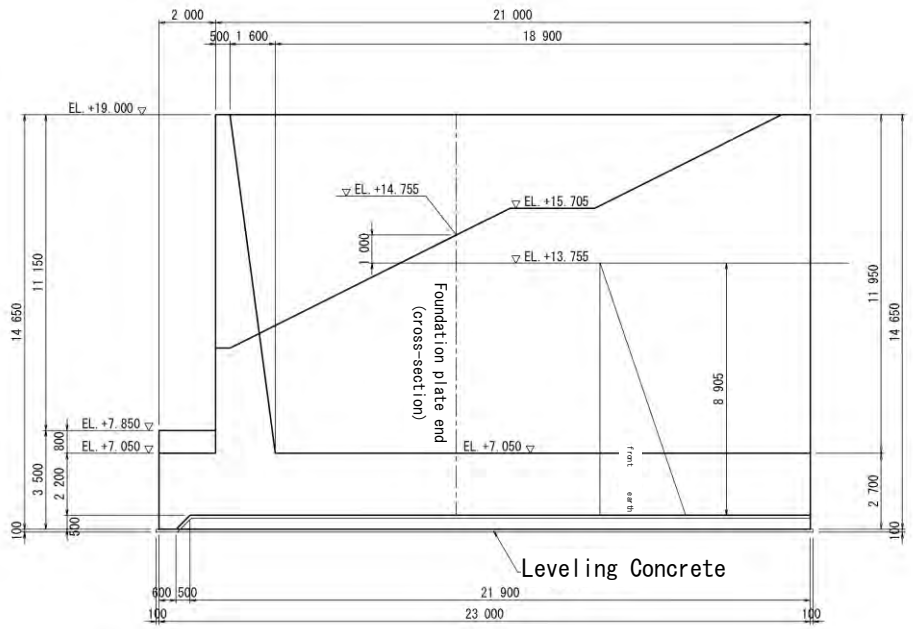
Table 7.3.29 List of Loads

Location		Study Case	Body weight	Wing-wall upper soil weight	Wing wall water weight	Surcharge	Surcharge (Generator room)	Water pressure (front)	Water pressure (back)	Earth pressure	Passive earth pressure	Uplift	Seismic inertia force
Downstream wing wall	L-type retaining wall	Normal (Water Level 1)	○	○	○	○		○	○	○	○		
		Seismic (Water Level 2)	○	○	○	○		○	○	○	○	○	○
		During construction	○	○	○	○		○	○	○	○		
	Inverse T retaining wall	Normal (Water Level 1)	○	○	○	○		○	○		○		
		Seismic (Water Level 2)	○	○	○	○		○	○		○	○	○
		During construction	○	○	○	○		○	○		○		
Upstream wing wall (Right Bank)	L-type retaining wall	Normal (Water Level 1)	○	○	○	○		○	○	○	○		
		Seismic (Water Level 2)	○	○	○	○		○	○	○	○	○	○
		During construction	○	○	○	○		○	○	○	○		
	Inverse T retaining wall	Normal (Water Level 1)	○	○	○	○		○	○		○		
		Seismic (Water Level 2)	○	○	○	○		○	○		○	○	○
		During construction	○	○	○	○		○	○		○		
Upstream wing wall (Left Bank)	L-type retaining wall	Normal (Water Level 1)	○	○	○	○	○	○	○	○	○		
		Seismic (Water Level 2)	○	○	○	○	○	○	○	○	○	○	○
		During construction	○	○	○	○	○	○	○	○	○		
	Inverse T retaining wall	Normal (Water Level 1)	○	○	○	○	○	○	○		○		
		Seismic (Water Level 2)	○	○	○	○	○	○	○		○	○	○
		During construction	○	○	○	○	○	○	○		○		

Source: Study Team

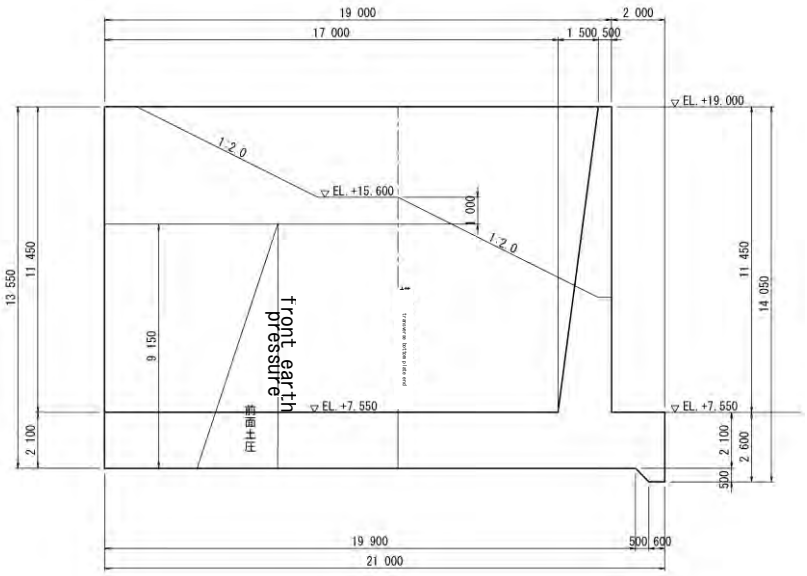
As shown in **Figure 7.3.14**, the examination of the L-shaped part is in the transverse direction (inverse T) to the right of the Bottom Slab end. For the upstream wall, the examination is carried out in the transverse direction (inverse T) in the left side from the Bottom Slab edge (**Figure 7.3.15**).

The ground heights of the receiving earth pressure are 14.755 m (downstream side) and 15.600 m (Upstream), which are the minimum heights on the left side of the Bottom Slab edge line, and the virtual ground level is set to 1.0 m from "Road Earthwork Guideline (Retaining Wall) p-114". The earth pressure is considered only for the study of sliding during the most severe seismic, and the value obtained by multiplying the earth pressure by 0.5 is taken as the resistance of the front ground.



Source: Study Team

Figure 7.3.14 Setting Passive Earth Pressure Height (Downstream L-Type Retaining Wall)



Source: Study Team

Figure 7.3.15 Setting Passive Earth Pressure Height (Upstream L-Type Retaining Wall)

(b) Design Water Level

Design water level is shown below.

Table 7.3.30 Design Water Levels

Item	Properties
Downstream L-type retaining wall Downstream Inverse T retaining wall	Front water level: 5.950 m (all cases)
	Back water level: 10.350 m (all cases)
Upstream L-type retaining wall Upstream Inverse T retaining wall	Front water level: 5.350 m (At all times during construction) 5.350 m (seismic)
	Back water level: 11.220 m (At all times during construction) 9.750 m (seismic)

Source: Study Team

5) Stability Calculation of Wing Wall

(a) Stability Check

The results of stability calculations are shown below. Detailed calculation sheets are attached in the separate piece of structural calculations.

(i) Overturning

$$e_B = \frac{M_B}{V}$$

Here,

eB : Load eccentricity (m)
MB : Moment acting on foundation (kN・m)
V : Vertical load acting on foundation (kN)

$$e_a = B/n$$

Here,

e_a : Allowable load eccentricity (m)
B : Foundation Width (m), B = 10.500
n : Safety factor

Table 7.3.31 Calculation Result of Overturning Stability

Position	Load Conditions (water level)	MB (kNm)	V (kN)	eB (m) Ea (m)	Judgment
Downstream Inverse T retaining wall	Normal (Water Level 1)	1262.162	2087.091	0.605 ≦ 1.750	OK
	Seismic (Water Level 2)	6519.564	2058.695	3.167 ≦ 3.500	OK
	During construction	-761.920	2865.654	0.266 ≦ 1.750	OK
Downstream L-type retaining wall	Normal (Water Level 1)	3074.007	2692.121	1.142 ≦ 1.917	OK
	Seismic (Water Level 2)	9501.566	2663.725	3.567 ≦ 3.833	OK
	During construction	1210.363	3599.604	0.336 ≦ 1.917	OK
Upstream Inverse T retaining wall (Right Bank)	Normal (Water Level 1)	1776.143	1922.838	0.924 ≦ 1.750	OK
	Seismic (Water Level 2)	5699.781	1979.473	2.879 ≦ 3.500	OK
	During construction	-869.491	2729.240	0.319 ≦ 1.750	OK
Upstream L-type retaining wall (Right Bank)	Normal (Water Level 1)	3549.687	2312.478	1.535 ≦ 1.750	OK
	Seismic (Water Level 2)	8162.732	2366.173	3.450 ≦ 3.500	OK
	During construction	1122.808	3164.340	0.355 ≦ 1.750	OK
Upstream Inverse T retaining wall (Left Bank)	Normal (Water Level 1)	2167.855	1983.237	1.093 ≦ 1.750	OK
	Seismic (Water Level 2)	6174.314	2002.924	3.083 ≦ 3.500	OK
	During construction	-477.568	2789.640	0.171 ≦ 1.750	OK
Upstream L-type retaining wall (Left Bank)	Normal (Water Level 1)	3943.274	2470.815	1.596 ≦ 1.833	OK
	Seismic (Water Level 2)	8349.146	2466.978	3.384 ≦ 3.667	OK
	During construction	1447.701	3358.715	0.431 ≦ 1.833	OK

Source: Study Team

(ii) Sliding

$$Hu = cBAe + V\tan\varphi B$$

Here,

Hu : Shear resistance between the bottom of the foundation and the ground (kN)
cB : Cohesion between base and ground (kN/m2), cB = 0.000
tan φB : Friction coefficient between base bottom and ground, tan φB = 0.700
Ae : Effective loading area (m2), Ae = L, Be
V : Vertical load acting on base bottom (kN)
L : Foundation Depth (m), L = 1.000
Be : Effective loading width of footing (m), Be = B -2 eB

B : Foundation Width (m), B = 10.500
eB : Load eccentricity (m)

Table 7.3.32 Eccentricity Load Conditions (Inverse T Retaining Wall)

Position	Load Conditions (water level)	Eccentricity eB = M/V (m)	Enabled loading area Ae (m2)	Enabled loading width Be (m)	Vertical load V (kN)
Downstream Inverse T retaining wall	Normal (Water Level 1)	0.605	9.290	9.290	2087.091
	Seismic (Water Level 2)	3.167	4.166	4.166	2058.695
	During construction	-0.266	9.968	9.968	2865.654
Upstream Inverse T retaining wall (Right Bank)	Normal (Water Level 1)	0.924	8.652	8.652	1922.838
	Seismic (Water Level 2)	2.879	4.742	4.742	1979.473
	During construction	-0.319	9.862	9.862	2729.240
Upstream Inverse T retaining wall (Left Bank)	Normal (Water Level 1)	1.093	8.314	8.314	1983.237
	Seismic (Water Level 2)	3.083	4.334	4.334	2002.924
	During construction	-0.171	10.158	10.158	2789.640

Source: Study Team

Table 7.3.33 Eccentricity Load Conditions (L-type Retaining Wall)

Position	Load Conditions (water level)	Eccentricity eB = M/V (m)	Enabled load area A ′ (m2)	Enabled load width B ′ (m)	Vertical load V (kN)	Passive earth pressure Pp (kN)	Availability factor α
Downstream L-type retaining wall	Normal (Water Level 1)	1.142	9.216	9.216	2692.121	0.000	0.000
	Seismic (Water Level 2)	3.567	4.366	4.366	2663.725	2059.423	0.500
	During construction	0.336	10.828	10.828	3599.604	0.000	0.000
Upstream L-type retaining wall (Right Bank)	Normal (Water Level 1)	1.535	7.430	8.652	2312.478	0.000	0.000
	Seismic (Water Level 2)	3.450	3.600	4.742	2366.173	2307.064	0.500
	During construction	0.355	9.790	9.790	3164.340	0.000	0.000
Upstream L-type retaining wall (Left Bank)	Normal (Water Level 1)	1.596	7.808	7.808	2470.815	0.000	0.000
	Seismic (Water Level 2)	3.384	4.232	4.232	2466.978	2307.063	0.500
	During construction	0.431	10.138	10.138	3358.715	0.000	0.000

Source: Study Team

$$fs = Hu/Hb$$

Here,
fs : Safety factor against sliding
Hb : Horizontal load acting on base bottom (kN)

Table 7.3.34 Results of Sliding Stability Calculation

position	Load Conditions (water level)	Shear Resistance Hu (kN)	Action horizontal force Hb (kN)	Safety factor fs	Required safety factor fsa	Judgment
Downstream Inverse T retaining wall	Normal (Water Level 1)	1460.964	756.068	1.932 ≥	1.500	OK
	Seismic (Water Level 2)	1441.087	1641.053	0.878 <	1.200	NG
	During construction	2005.958	525.943	3.814 ≥	1.500	OK
Downstream L-type retaining wall	Normal (Water Level 1)	1884.485	756.068	2.492 ≥	1.500	OK
	Seismic (Water Level 2)	2894.319	1792.734	1.614 ≥	1.200	OK
	During construction	2519.722	525.943	4.791 ≥	1.500	OK

position	Load Conditions (water level)	Shear Resistance Hu (kN)	Action horizontal force Hb (kN)	Safety factor fs	Required safety factor fsa	Judgment
Upstream Inverse T retaining wall (Right Bank)	Normal (Water Level 1)	1345.987	817.814	1.646 ≥	1.500	OK
	Seismic (Water Level 2)	1385.631	1532.255	0.904 <	1.200	NG
	During construction	1910.468	483.764	3.949 ≥	1.500	OK
Upstream L-type retaining wall (Right Bank)	Normal (Water Level 1)	1618.735	817.814	1.979 ≥	1.500	OK
	Seismic (Water Level 2)	2809.853	1622.335	1.732 ≥	1.200	OK
	During construction	2215.038	483.764	4.579 ≥	1.500	OK
Upstream Inverse T retaining wall (Left Bank)	Normal (Water Level 1)	1388.266	922.430	1.505 ≧	1.500	OK
	Seismic (Water Level 2)	1402.047	1619.773	0.866 <	1.200	NG
	During construction	1952.748	588.381	3.319 ≧	1.500	OK
Upstream L-type retaining wall (Left Bank)	Normal (Water Level 1)	1729.571	922.430	1.875 ≧	1.500	OK
	Seismic (Water Level 2)	2880.416	1649.895	1.746 ≧	1.200	OK
	During construction	2351.101	588.381	3.996 ≧	1.500	OK

Note: Although the seismic stability is out of allowable value, it is not considered because the structure is connected to the front Apron.
Source: Study Team

(iii) Calculation of Bearing Capacity

A. Subgrade Reaction

1)A case which the load acting within the bottom nucleus (trapezoidal load distribution)

$$qmin, qmax = \frac{V}{LB} \pm \frac{6M_B}{LB^2}$$

2) A case which the load acting outside the bottom nucleus (triangular load distribution)

$$qmax = 2\frac{V}{L \cdot x}$$

Here,

V : Vertical load acting on base bottom (kN)
M_B : Moment acting on base center (kNm)
B : Load eccentricity (m)
x : Width of action of bottom reaction (m), x = 3 (B/2-eB)
If x is less than B, the distribution is triangular.
If x is greater than B, the distribution is trapezoidal.
qmax : Maximum grade reaction (kN/m2) at base bottom
qmin : Minimum subgrade reaction (kN) at base bottom
B : Foundation Width (m), B = 10.500
L : Foundation Depth (m), L = 1.000

Table 7.3.35 Calculation of Ground Reaction

Position	Case	Subgrade reaction Width of action (m)	Subgrade reaction Shape	Ground reaction force (kN/m^2) qmin	qmax	Tolerance	Judgment
Downstream Inverse T retaining wall	Normal (Water Level 1)	10.500	Platform type	130.081	267.460 ≦	1000.000	OK
	Seismic (Water Level 2)	6.249	Triangle	0.000	658.888 ≦	1500.000	OK
	During construction	10.500	Platform type	231.454	314.384 ≦	1000.000	OK
Downstream L-type retaining wall	Normal (Water Level 1)	11.500	Platform type	94.634	373.561 ≦	1000.000	OK
	Seismic (Water Level 2)	6.549	Triangle	0.000	813.475 ≦	1500.000	OK
	During construction	11.500	Platform type	258.096	367.921 ≦	1000.000	OK
Upstream	Normal (Water Level 1)	10.500	Platform	86.467	279.788 ≦	1000.000	OK

Position	Case	Subgrade reaction Width of action (m)	Subgrade reaction Shape type	Ground reaction force (kN/m^2) qmin	qmax	Tolerance	Judgment
Inverse T retaining Wall (Right Bank)	Seismic (Water Level 2)	7.113	Triangle	0.000	556.579	≦ 1500.000	OK
	During construction	10.500	Platform type	212.608	307.247	≦ 1000.000	OK
Upstream L-type retaining wall (Right Bank)	Normal (Water Level 1)	10.500	Platform type	27.056	413.416	≦ 1000.000	OK
	Seismic (Water Level 2)	5.400	Triangle	0.000	876.360	≦ 1500.000	OK
	During construction	10.500	Platform type	240.260	362.471	≦ 1000.000	OK
Upstream Inverse T retaining Wall (Left Bank)	Normal (Water Level 1)	10.500	Platform type	70.901	306.858	≦ 1000.000	OK
	Seismic (Water Level 2)	6.501	Triangle	0.000	616.189	≦ 1500.000	OK
	During construction	10.500	Platform type	239.690	291.670	≦ 1000.000	OK
Upstream L-type retaining wall (Left Bank)	Normal (Water Level 1)	11.000	Platform type	29.085	420.154	≦ 1000.000	OK
	Seismic (Water Level 2)	6.348	Triangle	0.000	777.246	≦ 1500.000	OK
	During construction	11.000	Platform type	233.551	377.125	≦ 1000.000	OK

Source: Study Team

B. Check of Bearing Capacity

The allowable bearing capacity of the foundation ground is summarized in the following Tables. The calculation method of allowable bearing capacity is the same as in **7.3.2.3 (1) 2) (c)** .

Table 7.3.36 Allowable Bearing Capacity

Position	MB Nm/m)	eB (m)	Be (m)	α	β	κ	HB (kN/m)	q (kN/m2)	Sp	Sr
Downstream Inverse T retaining wall	1262.16	0.605	9.291	1.000	1.000	1.071	756.06	22.440	0.764	0.476
	6519.56	3.167	4.166	1.000	1.000	1.158	1641.05	22.440	0.764	0.621
	761.92	0.266	9.968	1.000	1.000	1.066	525.94	41.800	0.621	0.465
Downstream L-type retaining wall	3074.00	1.142	9.216	1.000	1.000	1.072	756.06	22.440	0.764	0.477
	9501.56	3.567	4.366	1.000	1.000	1.151	1792.73	22.440	0.764	0.612
	1210.36	0.336	10.828	1.000	1.000	1.061	525.94	41.800	0.621	0.452
Upstream Inverse T retaining wall (Right Bank)	1776.14	0.924	8.653	1.000	1.000	1.073	817.81	21.420	0.776	0.487
	5699.78	2.879	4.741	1.000	1.000	1.133	1532.26	21.420	0.776	0.595
	869.49	0.319	9.863	1.000	1.000	1.064	483.76	39.900	0.630	0.466
Upstream L-type retaining wall (Right Bank)	3549.69	1.535	7.430	1.000	1.000	1.085	817.81	21.420	0.776	0.512
	8162.73	3.450	3.600	1.000	1.000	1.175	1622.34	21.420	0.776	0.652
	1122.81	0.355	9.790	1.000	1.000	1.064	483.76	39.900	0.630	0.467
Upstream Inverse T retaining wall (Left Bank)	2167.855	1.093	8.314	1.000	1.000	1.076	922.430	21.420	0.776	0.494
	6174.314	3.083	4.334	1.000	1.000	1.145	1619.773	21.420	0.776	0.613
	477.568	0.171	10.158	1.000	1.000	1.062	588.381	39.900	0.630	0.462
Upstream L-type retaining wall (Left Bank)	9646.211	1.596	7.808	1.000	1.000	1.081	922.430	21.420	0.776	0.504
	5219.235	3.667	4.232	1.000	1.000	1.149	1649.895	21.420	0.776	0.618
	17025.232	1.833	10.138	1.000	1.000	1.062	588.381	39.900	0.630	0.462

Position	Loads State	Nc	Nq	Nr	Qu (kN/m)	Qa (kN/m)	V (kN/m)	Judgment
Downstream Inverse T retaining wall	Normal	2.75	1.00	0.00	44616.13	14872.04	2087.09	OK
	Seismic	2.50	0.00	0.00	19601.50	9800.75	2058.69	OK
	Seismic	4.00	1.00	0.00	69296.21	34648.10	2865.65	OK
Downstream L-	Normal	3.31	1.00	0.00	53321.83	17773.94	2692.12	OK

Position	Loads State	Nc	Nq	Nr	Qu (kN/m)	Qa (kN/m)	V (kN/m)	Judgment
type retaining wall	Seismic	2.50	0.00	0.00	20412.33	10206.17	2663.72	OK
	Seismic	4.25	1.00	0.00	79690.25	39845.13	3599.60	OK
Upstream Inverse T retaining wall (Right Bank)	Normal	2.50	0.00	0.00	37700.21	12566.74	1922.84	OK
	Seismic	2.50	0.00	0.00	21814.15	10907.08	1979.47	OK
	Seismic	4.04	1.00	0.00	69147.54	34573.77	2729.24	OK
Upstream L-type retaining wall (Right Bank)	Normal	2.81	1.00	0.00	36898.73	12299.58	2312.48	OK
	Seismic	2.50	0.00	0.00	17181.59	8590.80	2366.17	OK
	Seismic	4.21	1.00	0.00	71501.93	35750.97	3164.34	OK
Upstream Inverse T retaining wall (Left Bank)	Normal	2.75	0.00	0.00	40092.52	13364.17	1983.237	OK
	Seismic	3.00	0.00	0.00	24184.61	12092.30	2002.924	OK
	Seismic	3.80	1.00	0.00	66846.29	33423.15	2789.640	OK
Upstream L-type retaining wall (Left Bank)	Normal	2.60	1.00	0.00	35768.82	11922.94	2470.815	OK
	Seismic	2.90	0.00	0.00	22898.08	11449.04	2466.978	OK
	Seismic	4.00	1.00	0.00	70219.81	35109.91	3358.715	OK

Source: Study Team

C. Result of Stability Calculations

The results of stability calculation against the maximum ground reaction force and the vertical support rate are shown in **Table 7.3.37** and **Table 7.3.38**, respectively.

Table 7.3.37 Checking for maximum ground reaction

Position	Loads State	Acting force at the base center		Reaction force action width (m)	Subgrade reaction (kN/m2)	Maximum subgrade reaction (kN/m2)	Judgment
		M (kN.m/m)	V (kN/m)				
Downstream Inverse T retaining wall	Normal	1262.16	2087.09	10.500	267.459	≦ 2376.990	○
	Seismic	6519.56	2058.69	6.249	658.838	≦ 1157.790	○
	Seismic	761.92	2865.65	10.500	314.384	≦ 4267.100	○
Downstream L-type retaining wall	Normal	3074.00	2692.12	11.500	373.561	≦ 2692.950	○
	Seismic	9501.56	2663.72	6.549	813.484	≦ 1417.930	○
	Seismic	1210.36	3599.60	11.500	367.921	≦ 4500.000	○
Upstream Inverse T retaining wall (Right Bank)	Normal	1776.14	1922.84	10.500	279.788	≦ 2005.450	○
	Seismic	5699.78	1979.47	7.112	556.683	≦ 1208.270	○
	Seismic	869.49	2729.24	10.500	307.247	≦ 4500.000	○
Upstream L-type retaining wall (Right Bank)	Normal	3549.69	2312.48	10.500	413.417	≦ 2436.100	○
	Seismic	8162.73	2366.17	5.401	876.245	≦ 1607.890	○
	Seismic	1122.81	3164.34	10.500	362.471	≦ 4500.000	○
Upstream Inverse T retaining wall (Left bank)	Normal	2167.855	1983.237	10.500	306.858	≦ 2005.450	○
	Seismic	6174.314	2002.924	6.501	616.189	≦ 1208.270	○
	Seismic	477.568	2789.640	10.500	291.670	≦ 4500.000	○
Upstream L-type retaining wall (Left bank)	Normal	9646.211	2470.815	11.000	420.154	≦ 2436.100	○
	Seismic	5219.235	2466.978	6.348	777.246	≦ 1607.890	○
	Seismic	17025.232	3358.715	11.000	377.125	≦ 4500.000	○

Source: Study Team

Table 7.3.38 Verification of vertical support rate

Position	Loads State	Acting force at the base center			Acting vertical force (kN/m)	Allowable vertical force (kN/m)	Judgment
		M (kN.m/m)	V (kN/m)	H (kN/m)			
Downstream Inverse T retaining wall	Normal	1262.16	2087.09	756.06	2087.09	≦ 14872.04	○
	Seismic	6519.56	2058.69	1641.05	2058.69	≦ 9800.75	○
	Seismic	761.92	2865.65	525.94	2865.65	≦ 34648.10	○
Downstream L-type retaining wall	Normal	3074.00	2692.12	756.06	2692.12	≦ 17773.94	○
	Seismic	9501.56	2663.72	1792.73	2663.72	≦ 10206.17	○
	Seismic	1210.36	3599.60	525.94	3599.60	≦ 39845.13	○
Upstream Inverse T retaining wall (Right Bank)	Normal	1776.14	1922.84	817.81	1922.84	≦ 12566.74	○
	Seismic	5699.78	1979.47	1532.26	1979.47	≦ 10907.08	○
	Seismic	869.49	2729.24	483.76	2729.24	≦ 34573.77	○
Upstream L-type retaining wall (Right Bank)	Normal	3549.69	2312.48	817.81	2312.48	≦ 12299.58	○
	Seismic	8162.73	2366.17	1622.34	2366.17	≦ 8590.80	○
	Seismic	1122.81	3164.34	483.76	3164.34	≦ 35750.97	○
Upstream Inverse T retaining wall (Left Bank)	Normal	2167.855	1983.237	922.430	1983.237	≦ 13364.17	○
	Seismic	6174.314	2002.924	1619.773	2002.924	≦ 12092.30	○
	Seismic	477.568	2789.640	588.381	2789.640	≦ 33423.15	○
Upstream L-type retaining wall (Left Bank)	Normal	9646.211	2470.815	922.430	2470.815	≦ 11922.94	○
	Seismic	5219.235	2466.978	1649.895	2466.978	≦ 11449.04	○
	Seismic	17025.232	3358.715	588.381	3358.715	≦ 35109.91	○

Source: Study Team

(2) Structural Calculation

1) Design Condition

The main design conditions are displayed in **Table 7.3.39**. Details are provided separately.

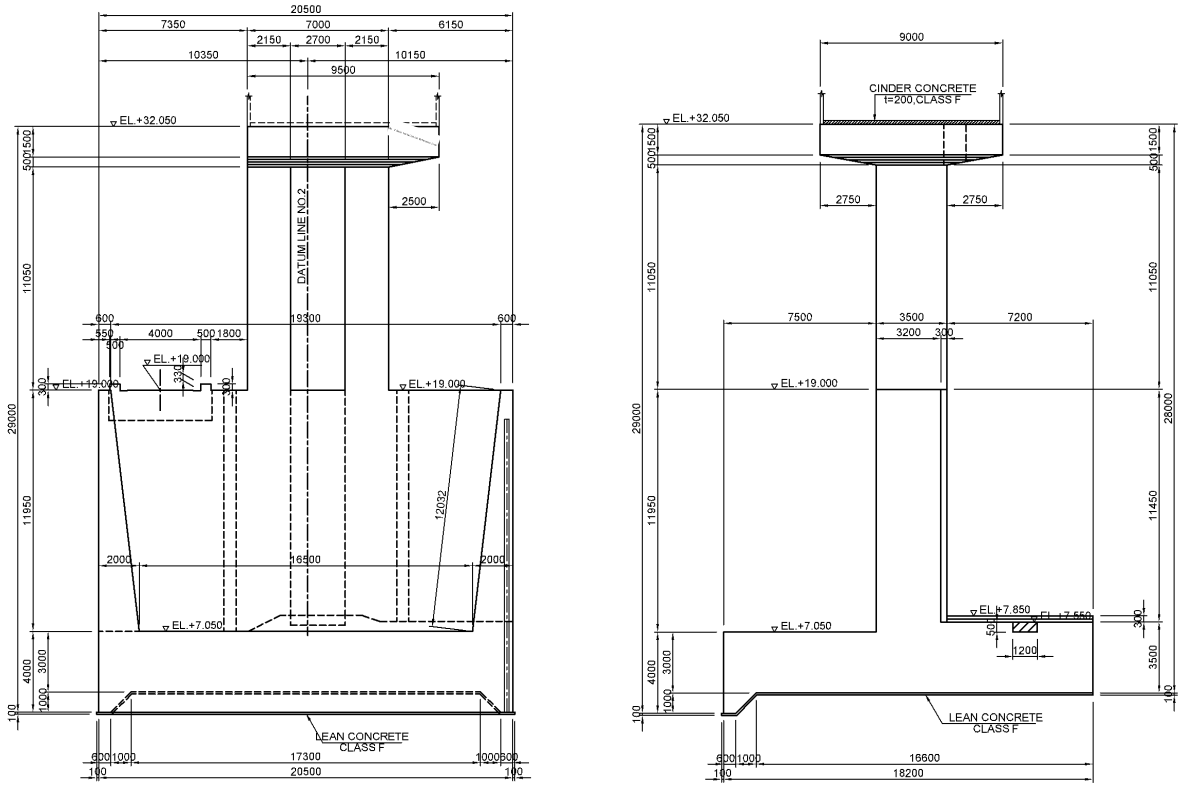
Table 7.3.39 List of MCGS Design Conditions

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

Source: Study Team

2) End Pier

The structural dimensions of the end pier (No. 1 Pier) are shown **Figure 7.3.16**.



(Flow Direction) (Direction Perpendicular to Flow)

Source: Study Team

Figure 7.3.16 Structural dimension of end pier

(a) Design Policy

Pier : is designed as a cantilever beam fixed to the Bottom Slab.

Bottom Slab : is designed as a cantilever beam fixed to pier.

Column : is designed as a cantilever beam fixed to the weir.

Operating deck : is designed as cantilever beam fixed to the column.

(b) Loads (Bottom Slab and Weir)

The loads for the case of normal, flood, seismic and at the time of construction are considered.

Since the pier has an enough length in the flow direction, the check in the flow direction is omitted and the check is carried out for the direction perpendicular to the flow.

Load schedules for end piers and Bottom Slabs are shown in **Table 7.3.40**.

Table 7.3.40 Schedule of Loads on End Pier and Bottom Slab

Direction	Study case	Vertical load V (kN)	Horizontal load H (kN)	e (m)	Moment My (kN m)
Flow Direction	Normal	77231.41	-15953.52	0.675	52131.20
	Flood	62599.73	-4299.67	1.195	74806.68
	Seismic	76014.56	-28876.61	-0.697	-52982.15

Source: Study Team

Table 7.3.41 Subgrade Reaction

Direction	Study Case	X = L (m)	Pier No.2 Side (kN/m2)	Backside of Pier No.1 (kN/m^2)
Flow Direction	Normal	18.200	3299.192	5187.776
	Flood	18.200	2084.516	4794.575
	Seismic	18.200	5136.330	3216.919

Source: Study Team

(c) Calculation Result

The stress calculation results of the Bottom Slab and pier in the most severe condition among the examination cases are shown below. The cross-sectional force diagram is attached in the separate structural calculation piece.

Table 7.3.42 Stress Calculation of Bottom Slab

Bottom Slab Stress Calculation

Member Name	Item		Units	Thickness T = 3.00 m	Thickness T = 3.50 m	Thickness T = 3.80 m	Thickness T = 3.00 m
	Position			Underside, Tension	Underside, Tension	Underside, Tension	Rear Upperside, Tension
	Load Condition			Seismic Condition (Short-term)	Seismic Condition (Short-term)	Seismic Condition (Short-term)	Seismic Condition (Short-term)
Cross-sectional Force	Bending Moment	M	kN m	5115.79	5115.79	5115.79	2860.79
	Axial Force	N	kN	1263.54	1263.54	1263.54	136.44
	Shear Stress	S	kN	1043.76	1043.76	1043.76	460.74
Member Cross-section	Member Width	b	mm	1000	1000	1000	1000
	Member Height	h	mm	3000	3500	3800	3500
	Tension Rebar Diameter	D	mm	32	32	32	28
	Tension Rebar Number of Pieces	n	rebar	8	8	8	8
	Tension Rebar Cover	d'	mm	120	120	120	610
	Compression Rebar Diameter	D	mm	28	28	28	32
	Compression Rebar Number of Pieces	n	rebar	4	4	4	4
	Compression Rebar Cover	d'	mm	610	610	610	110
Stress Intensity	Concrete	σc	N/mm2	7.36	5.66	4.91	4.26
	Re-bar	σs	N/mm2	206.8	162.6	141.9	196.1
	Shear Stress	τ	N/mm2	0.362	0.309	0.284	0.159
Allowable Stress	Concrete	σca	N/mm2	11.01	11.01	11.01	11.01
	Re-bar	σ sa	N/mm2	223.4	223.4	223.4	223.4
	Shear Stress	τa	N/mm2	0.470	0.470	0.940	0.940
Evaluationn	σc ≦ σca			OK	OK	OK	OK
	σs ≦ σsa			OK	OK	OK	OK
	τ ≦ τa			OK	OK	OK	OK

Source: Study Team

Table 7.3.43 Stress Calculation of No. 1 Pier (Direction Perpendicular to Flow)

Member Name	Item			Units	Thickness T = 3.50 m	Thickness T = 2.70 m
	Position				No. 1 Back Side Tension	No. 1 Back Side Tension
	Load Condition				Seismic	Seismic
					(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	6309.86	6309.86
	Axial Force		N	kN	1,489.71171	1,489.71171
	Shear Stress		S	kN	1,127.17073	1,127.17073
Member Cross-section	Member Width		b	mm	1000	1000
	Member Height		h	mm	3500	2700
	Tension Rebar	Diameter	D	mm	32	32
		Number of Rebar	n	rebar	8	16
		Cover	d'	mm	140	190
	Compression Rebar	Diameter	D	mm	32	32
		Number of Rebar	n	rebar	4	4
		Cover	d'	mm	140	140
Stress Intensity	Concrete		σc	N/mm2	6.80	8.42
	Re-bar		σs	N/mm2	203.9	161.8
	Shear Force		τ	N/mm2	0.335	0.449
Allowable Stress	Concrete		σca	N/mm2	11.01	11.01
	Re-bar		σ sa	N/mm2	223.4	223.4
	Shear Force		τa	N/mm2	0.940	0.940
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

(d) Loads (Operation Deck and Column)

The examination cases are four cases: normal, seismic, normal with temperature (+ 16.7 °C " and normal with temperature (-22.2 °C)". Summary is shown in **Table 7.3.44**. For columns, stress calculation in the flow direction and the direction perpendicular to flow are carried out.

Table 7.3.44 Schedule of Load on End Pier and Bottom Slab

Direction Calculation	Load Conditions	Loads						Extra factor
		Body weight	Equipment load	Shed load	Temperature load	Community load	Inertial force	
Direction perpendicular to the water flow Flow Direction	Ordinary time	○	○	○		○		1.00
	Normal (Temperature rise + 16.7 °)	○	○	○	○	○		1.25
	Normal (Temperature drop -22.2 °)	○	○	○	○	○		1.25
	Seismic	○	○	○			○	1.33

Source: Study Team

(e) Calculation Result

The results of stress calculation of the column and operating deck in the most critical conditions are shown below. The cross-sectional force diagram is attached in the separate structural calculation piece.

Table 7.3.45 Stress Calculation of the No. 1 Column (Flow Direction)

Member Name	Item			Units	Thickness T = 2.15 m
	Position				No.1 Column
	Load Condition				Seismic
					(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	6797.53
	Axial Force		N	kN	2568.93
	Shear Stress		S	kN	1037.78
Member Cross-section	Member Width		b	mm	3500
	Member Height		h	mm	2150
	Tension Rebar	Diameter	D	mm	25
		No.of Rebar	n	rebar	27
		Cover	d'	mm	130
	Compression Rebar	Diameter	D	mm	25
		No.of Rebar	n	rebar	27
		Cover	d'	mm	130
Stress Intensity	Concrete		σc	N/mm2	5.66
	Re-bar		σs	N/mm2	178.0
	Shear Force		τ	N/mm2	0.147
Allowable Stress	Concrete		σca	N/mm2	11.01
	Re-bar		σ sa	N/mm2	223.4
	Shear Force		τa	N/mm2	0.940
Evaluationn	σc ≦ σca				OK
	σs ≦ σsa				OK
	τ ≦ τa				OK

Source: Study Team

Table 7.3.46 Stress Calculation of the No. 1 Column (Direction Perpendicular to Flow)

Member Name	Item			Units	Thickness T = 3.50 m
	Position				No.1 Column
	Load Condition				Seismic Condition
					(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	8941.42
	Axial Force		N	kN	4684.73
	Shear Stress		S	kN	936.95
Member Cross-section	Member Width		b	mm	2150
	Member Height		h	mm	3500
	Tension Rebar	Diameter	D	mm	25
		No.of Pieces	n	rebar	9
		Cover	d'	mm	130
	Compression Rebar	Diameter	D	mm	25
		No.of Pieces	n	rebar	9
		Cover	d'	mm	130
Stress Intensity	Concrete		σc	N/mm2	5.63
	Re-bar		σs	N/mm2	149.9
	Shear		τ	N/mm2	0.129
Allowable Stress	Concrete		σca	N/mm2	11.01
	Re-bar		σ sa	N/mm2	223.4
	Shear		τa	N/mm2	0.940
Evaluationn	σc ≦ σca				OK
	σs ≦ σsa				OK
	τ ≦ τa				OK

Source: Study Team

Table 7.3.47 Stress Calculation of the No. 1 Operation Deck

Member Name	Item			Units	Thickness T = 2.00 m	Thickness T = 2.00 m
	Position				Flow Direction	Flow Direction
	Load Condition				Upperside, Tention	Underside, Tention
					Normal	Normal +Temperature
Cross-Sectional Force	Bending Moment		M	kN m	5919.1	2757.42
	Axial Force		N	kN	153.23	-480.82
	Shear Stress		S	kN	2214.20	0.00
Member Cross-section	Member Width		b	mm	6260	4330
	Member Height		h	mm	2000	2000
	Tension Rebar	Diameter	D	mm	25	20
		Number of Rebar	n	rebar	46	30
		Cover	d'	mm	70	70
	Compression Rebar	Diameter	D	mm	20	25
		Number of Rebar	n	rebar	30	46
		Cover	d'	mm	70	70
Stress Intensity	Concrete		σc	N/mm2	3.10	2.26
	Re-bar		σs	N/mm2	140.3	182.51
	Shear Force		τ	N/mm2	0.183	0.00
Allowable Stress	Concrete		σca	N/mm2	11.01	11.01
	Re-bar		σ sa	N/mm2	223.4	223.44
	Shear Force		τa	N/mm2	0.479	0.48
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

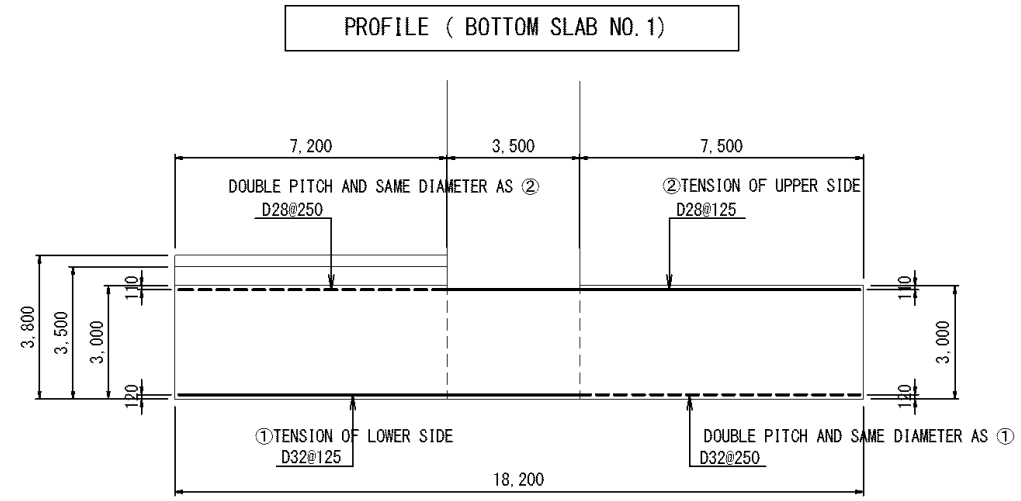
Table 7.3.48 Stress Calculation of the Overhang Part of No. 1 Operation Deck

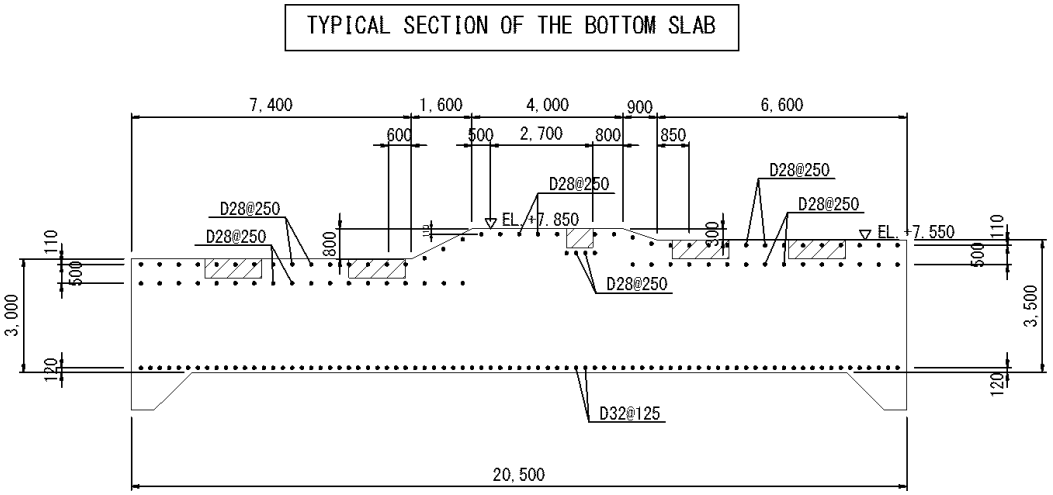
Member Name	Item			Units	Thickness T = 2.00 m
	Position				Weir Axial Direction
	Load Condition				Upperside, Tention
					Normal
Cross-Sectional Force	Bending Moment		M	kN・m	486.137
	Axial Force		N	kN	0
	Shear Stress		S	kN	225.0285
Member Cross-Section	Member Width		b	mm	1000
	Member Height		h	mm	2000
	Tension Rebar	Diameter	D	mm	25
		Number of Rebar	n	piece	4
		Amount of Rebar	As	mm^2	1963.5
		Cover	d'	mm	91
	Compression Rebar	Diameter	D	mm	16
		Number of Rebar	n	piece	4
		Amount of Rebar	As	mm^2	804.2
		Cover	d'	mm	88
Stress Intensity	Concrete		σ_c	N/mm^2	2.15
	Re-bar		σ_s	N/mm^2	135.4
	Shear Force		τ	N/mm^2	0.118
Allowable Stress	Concrete		σ_{ca}	N/mm^2	8.28
	Re-bar		σ_{sa}	N/mm^2	168.0
	Shear Force		τ_a	N/mm^2	0.360
Evaluationn	$\sigma c \leqq \sigma ca$				OK
	$\sigma s \leqq \sigma sa$				OK
	$\tau \leqq \tau a$				OK

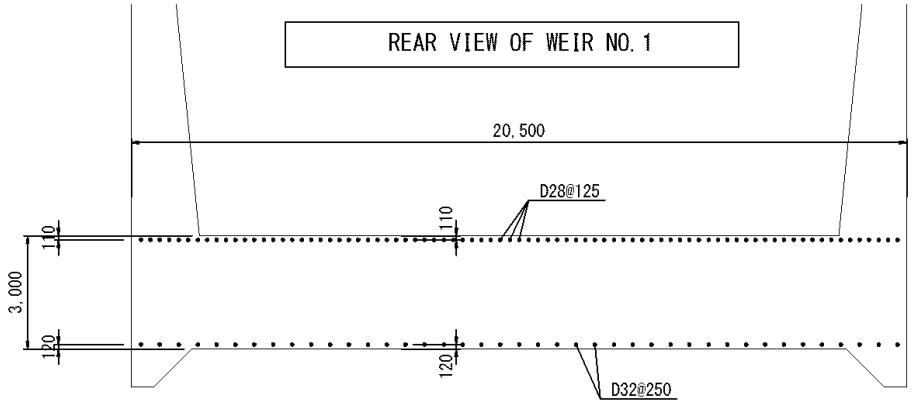
Note) The sectional force is the value calculated as a cantilever.

Source: Study Team

(f) Bar Arrangement Plan

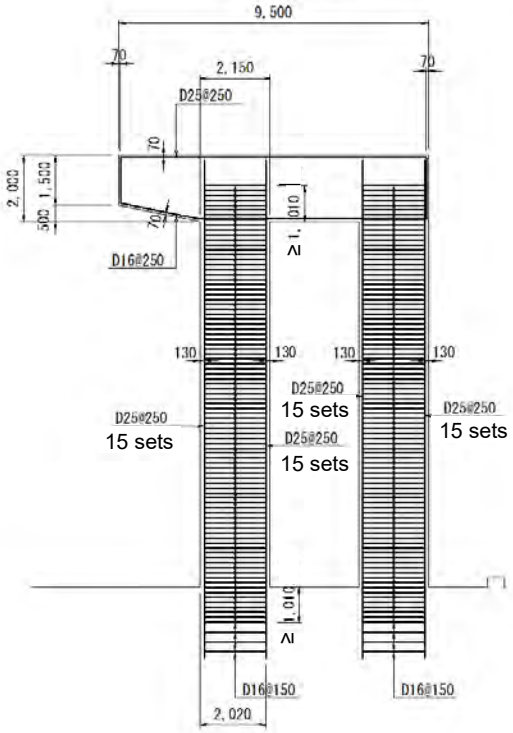




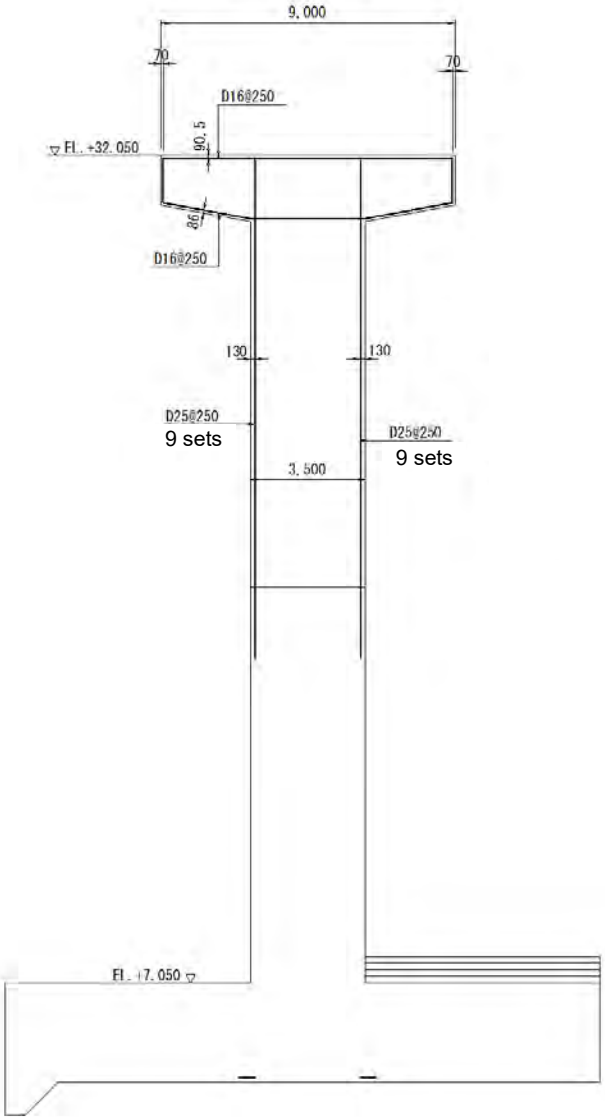


Profile of Operation Deck/Pier

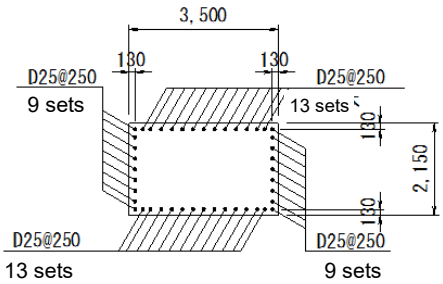
Direction Perpendicular to Flow



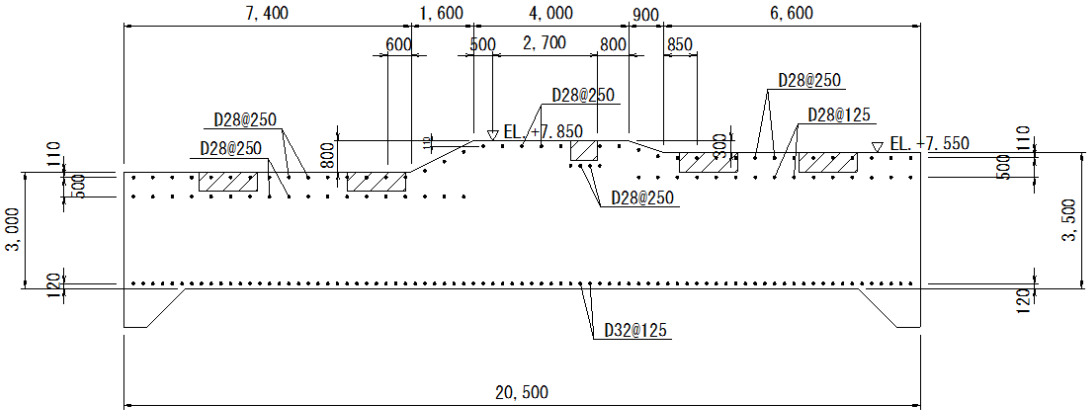
Flow Direction



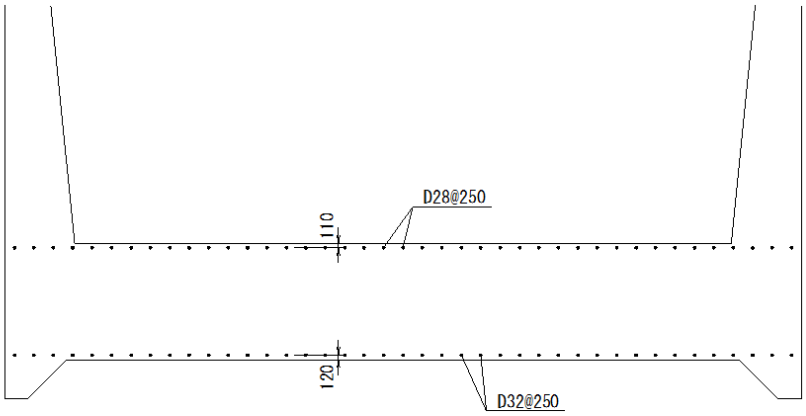
Pier Cross-section



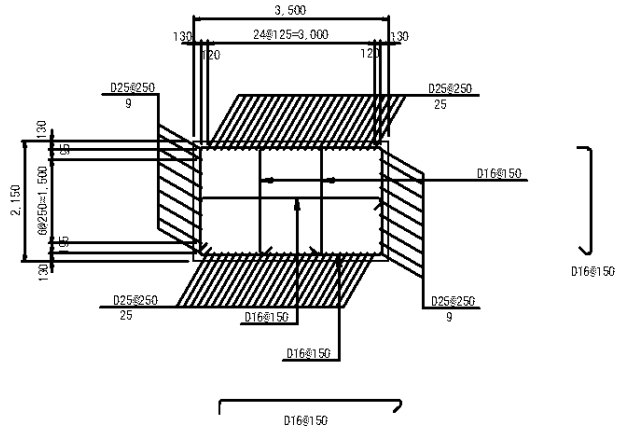
Normal Part of Bottom Slab



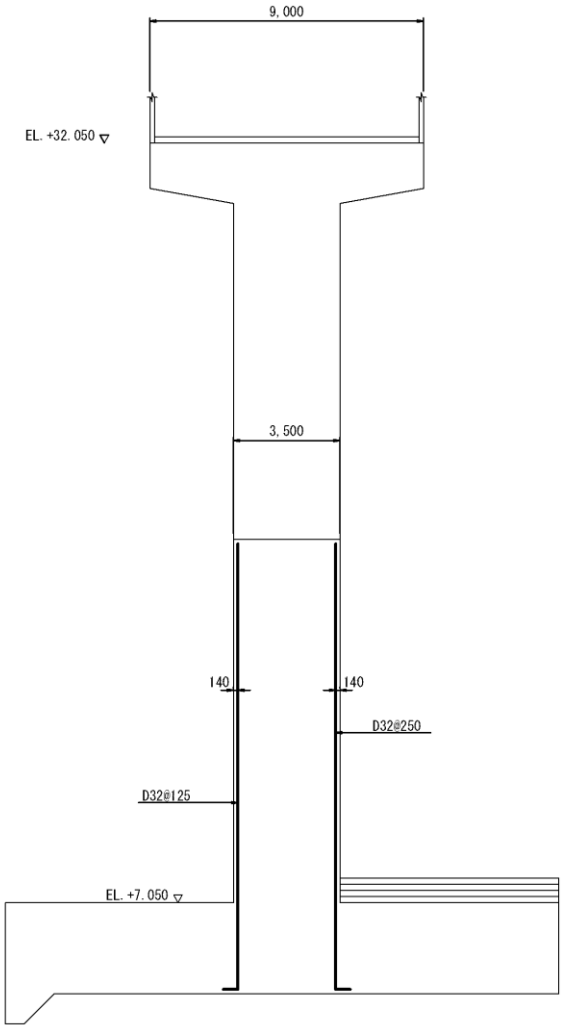
Backside of No.1 Pier



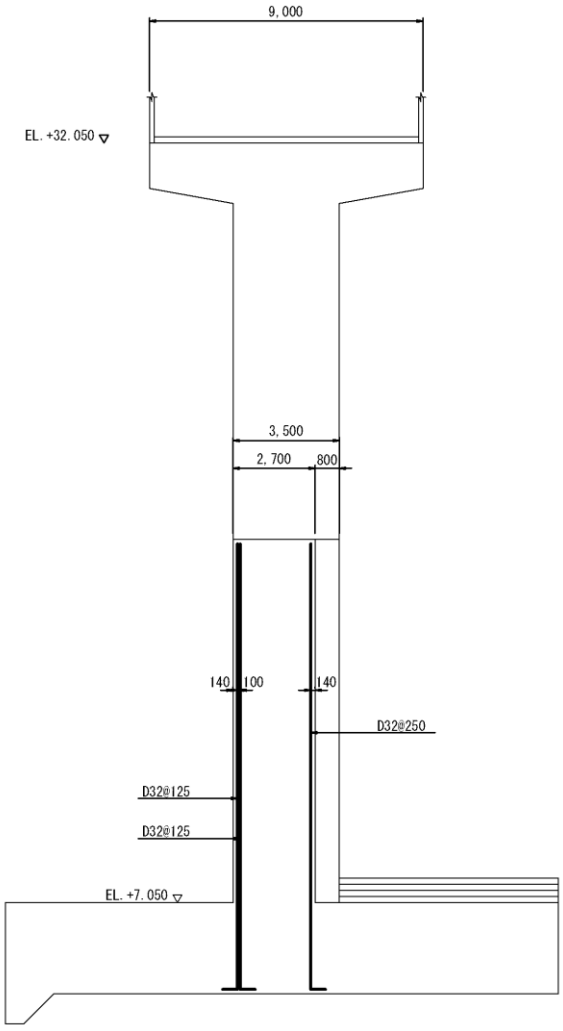
PIER CROSS-SECTION



TYPICAL SECTION OF THE PIER

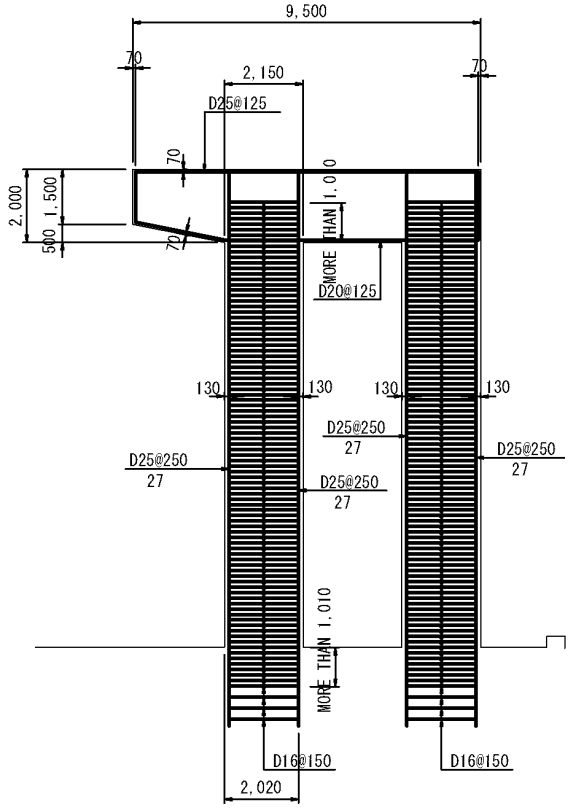


GATE PART OF THE PIER

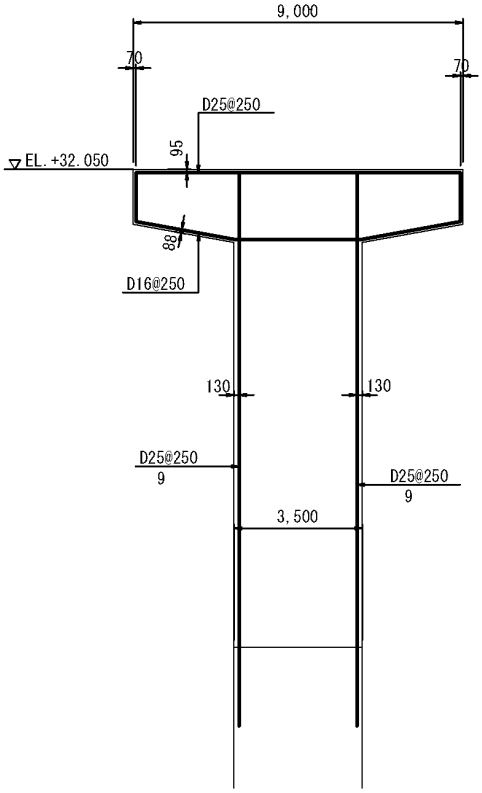


OPERATION DECK・CROSS-SECTION

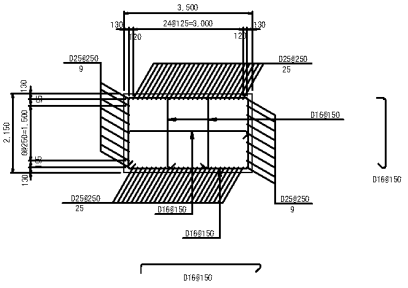
CROSS SECTION (FLOW DIRECTION)



CROSS SECTION (WEIR AXIAL DIRECTION)

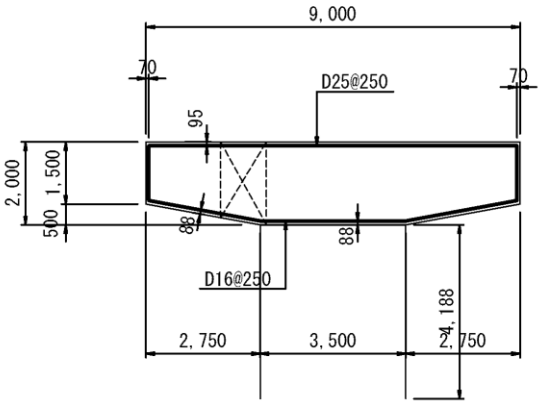


PIER CROSS-SECTION

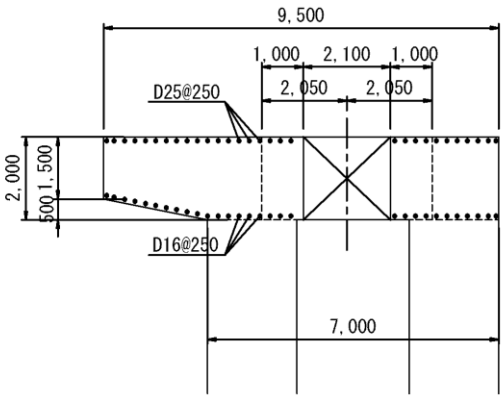


OPERATION DECK (WEIR AXIAL DIRECTION)

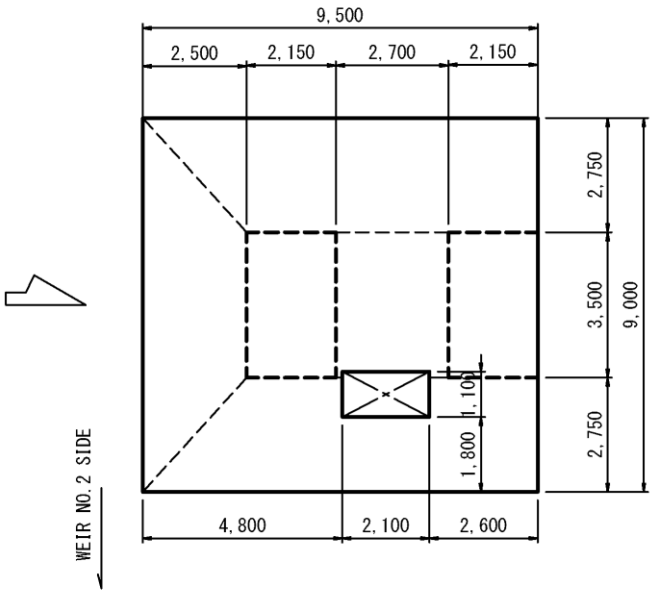
PROFILE (WEIR AXIAL DIRECTION)



CROSS SECTION (WEIR AXIAL DIRECTION)

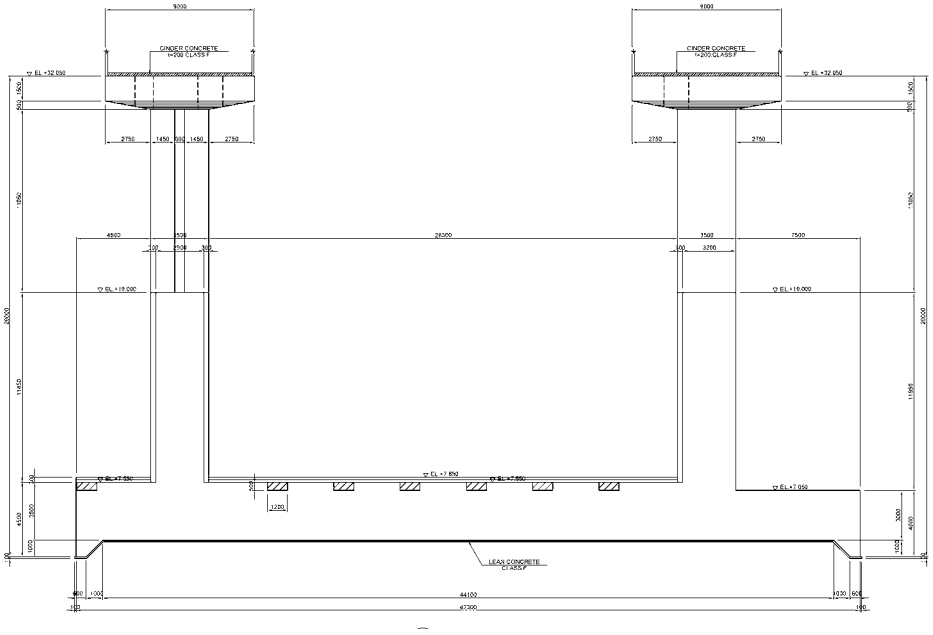


PLAN

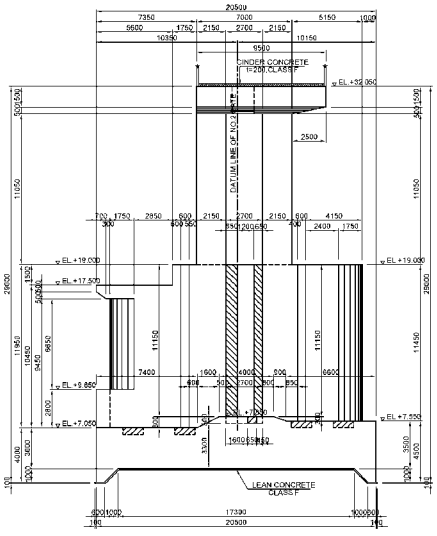


3) End Pier + Central Pier

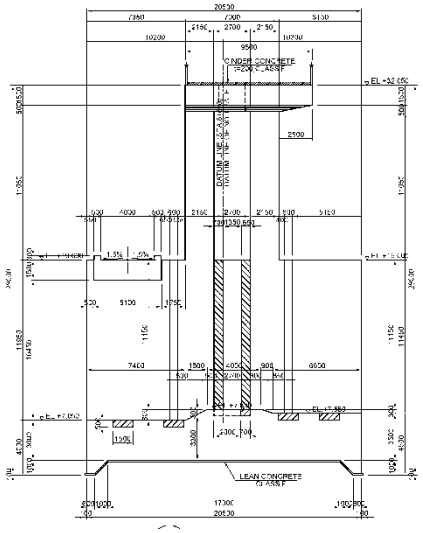
The structural dimensions of the end pier (No. 1 Pier) are shown in **Figure 7.3.17**.



(Direction Perpendicular to Flow)



(Central Pier: Flow Direction)



(End Pier: Flow Direction)

Source: Study Team

Figure 7.3.17 Structure of End Pier + Central Pier

(a) Design Policy

Pier and Bottom Slab : are designed as U-shaped rigid frame structure.

The dam posts and Operating decks : are treated as same as those in "end pier" in the previous section.

(b) Loads (Bottom Slab and Pier)

The loads for the case of normal, flood, and seismic are considered. Since the pier has an enough 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. Summary of loads for end pier + central pier and Bottom Slab are shown in **Table 7.3.49**.

Table 7.3.49 Schedule of Load on End Weir and Bottom Slab

Direction	Study case	Vertical load V (kN)	Horizontal load H (kN)	e (m)	Moment My (kN Nm)
Flow Direction	Normal	136154.14	-16056.220	3.300	449308.66
	Flood	117910.66	-4413.450	2.681	316118.48
	Seismic	134361.10	-45240.780	1.613	216724.45

Source: Study Team

Table 7.3.50 Subgrade Reaction

Direction	Study case	X = L (m)	Pier No.2 Side (kN/m2)	Backside of No.3 Pier (kN/m2)
Flow Direction	Normal	47.300	1673.560	4083.486
	Flood	47.300	1645.054	3340.597
	Seismic	47.300	2259.400	3421.830

Source: Study Team

(c) Calculation Result

The stress calculation results of the Bottom Slab and pier in the most severe condition among the examination cases are shown. The cross-sectional force diagram is shown in the separate structural calculation piece.

Table 7.3.51 Stress Calculation for No. 2 and No. 3 Bottom Slab

Member Name	Item			Units	Thickness T = 3.00 m	Thickness T = 3.00 m	Thickness T = 3.50 m	Thickness T = 3.50 m
	Position				Upperside, Tension	Underside, Tension	Upperside, Tension	Underside, Tension
	Load Condition				Normal	Seismic	Normal	Seismic
					(Long-term)	(Short-term)	(Long-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	73177.69	38812.19	65266.59	34616.28
	Axial Force		N	kN	5758.83	9484.81	5136.26	8459.42
	Shear Stress		S	kN	0	10518.5637	0	9211.09472
Member Cross-section	Member Width		b	mm	7400	7400	6600	6600
	Member Height		h	mm	3000	3000	3500	3500
	Tension Rebar	Diameter	D	mm	40	40	40	40
		Number of Rebar	n	rebar	182	118	130	52
		Amount of Rebar	As	mm2	13697.9	148283.5	163363.2	65345.3
		Cover	d'	mm	509	185	574	130
	Compression Rebar	Diameter	D	mm	40	40	40	40
		Number of Rebar	n	rebar	118	182	52	130
		Amount of Rebar	As	mm2	148283.5	228708.5	65345.3	163363.2
		Cover	d'	mm	185	509	130	570
Stress Intensity	Concrete		σc	N/mm2	8.13	4.55	7.34	4.59
	Rebar		σs	N/mm2	130.2	77.2	134.5	114.5
	Shear Force		τ	N/mm2	0.000	0.505	0.000	0.414
Allowable Stress	Concrete		σca	N/mm2	8.28	11.01	8.28	11.01
	Rebar		σ sa	N/mm2	168.0	223.4	168.0	223.4
	Shear Force		τa	N/mm2	0.360	2.130	0.360	0.470
Evaluationn	σc ≦ σca				OK	OK	OK	OK
	σs ≦ σsa				OK	OK	OK	OK
	τ ≦ τa				OK	OK	OK	OK

(Note) τa= 2.13 considers shear rainforcing bar

Source: Study Team

Table 7.3.52 Stress Calculation for No. 2 and No. 3 Gate Bottom Slab

Member Name	Item			Units	Thickness T = 3.80 m	Thickness T = 3.80 m
	Position				Upperside Tension	Underside, Tension
	Load Condition				Normal	Seismic
					(Long-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	39555.51	20979.56
	Axial Force		N	kN	3112.88	5126.92
	Shear Stress		S	kN	0.00	5520.54
Member Cross-section	Member Width		b	mm	4000	4000
	Member Height		h	mm	3800	3800
	Tension Rebar	Diameter	D	mm	40	40
		Number of Rebar	n	rebar	57	32
		Amount of Rebar	As	mm2	71628.5	40212.5
		Cover	d'	mm	239.6	130.0
	Compression Rebar	Diameter	D	mm	40	40
		Number of Rebar	n	rebar	32	57
		Amount of Rebar	As	mm2	40212.5	71628.5
		Cover	d'	mm	130.0	239.6
Stress Intensity	Concrete		σc	N/mm2	6.11	3.72
	Rebar		σs	N/mm2	147.8	94.5
	Shear Force		τ	N/mm2	0.000	0.376
Allowable Stress	Concrete		σca	N/mm2	8.28	11.01
	Rebar		σ sa	N/mm2	168.0	223.4
	Shear Force		τa	N/mm2	0.360	0.470
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

Table 7.3.53 Stress Calculation for Bottom Slab of No. 3 Back Side (Top Side Tension)

Member Name	Item			Units	Thickness T = 3.00 m
	Position				Upperside Tension
	Load Condition				Seismic
					(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	2268.81
	Axial Force		N	kN	108.00
	Shear Stress		S	kN	487.75
Member Cross-section	Member Width		b	mm	1000
	Member height		h	mm	3000
	Tension Rebar	Diameter	D	mm	40
		Number of Rebar	n	rebar	4
		Amount of Rebar	As	mm2	5026.56
		Cover	d'	mm	120
	Compression Rebar	Diameter	D	mm	40
		Number of Rebar	n	rebar	4
		Amount of Rebar	As	mm2	5026.56
		Cover	d'	mm	120
Stress Intensity	Concrete		σc	N/mm2	3.32
	Rebar		σs	N/mm2	154.77
	Shear Force		τ	N/mm2	0.17
Allowable Stress	Concrete		σca	N/mm2	11.01
	Rebar		σ sa	N/mm2	223.40
	Shear Force		τa	N/mm2	0.47
Evaluationn	σc ≦ σca				OK
	σs ≦ σsa				OK
	τ ≦ τa				OK

Source: Study Team

Table 7.3.54 Stress Calculation at the Column of No.2 and No.3 (Direction Perpendicular to Flow)

Member Name	Item			Units	Thickness T = 3.50 m	Thickness T = 1.70 m	Thickness T = 3.50 m	Thickness T = 2.50 m
	Position				No.2 pier	No.2 pier	No.3 pier Backside tension	No.3 pier Backside tension
	Load Condition				Seismic (Short-term)	Seismic (Short-term)	Seismic (Short-term)	Seismic (Short-term)
Cross-sectional Force	Bending Moment		M	kN m	3629.80	3629.80	6651.93	6651.93
	Axial Force		N	kN	1448.86	1448.86	1580.27	1580.27
	Shear Stress		S	kN	316.52	316.52	1145.28	1145.28
Member Cross-section	Member Width		b	mm	1000	1000	1000	1000
	Member Height		h	mm	3500	1700	3500	2500
	Tension Rebar	Diameter	D	mm	32	32	32	32
		Number of Rebar	n	rebar	4	16	8	16
		Amount of Rebar	As	mm2	3217.0	12868.0	6434.0	12868.0
		Cover	d'	mm	140	190	140	190
	Compression Rebar	Diameter	D	mm	32	32	32	32
		Number of Rebar	n	rebar	4	16	4	4
		Amount of Rebar	As	mm2	3217.0	12868.0	3217.0	3217.0
		Cover	d'	mm	140	190	140	140
Stress Intensity	Concrete		σc	N/mm2	4.73	9.89	7.17	10.13
	Rebar		σs	N/mm2	145.1	156.2	214.3	190.3
	Shear Force		τ	N/mm2	0.094	0.210	0.341	0.496
Allowable Stress	Concrete		σca	N/mm2	11.01	11.01	11.01	11.01
	Rebar		σ sa	N/mm2	223.4	223.4	223.4	223.4
	Shear Force		τa	N/mm2	0.400	0.470	0.470	2.130
Evaluationn	σc ≦ σca				OK	OK	OK	OK
	σs ≦ σsa				OK	OK	OK	OK
	τ ≦ τa				OK	OK	OK	OK

(Note) τa= 2.13 considers shear rainforcing bar

Source: Study Team

(d) Loads (Operation Deck and Column)

The examination cases are four cases: normal, seismic, normal with temperature (+ 16.7 °C " and normal with temperature (-22.2 °C)". Summary is shown in **Table 7.3.55**. For columns, stress calculation in the flow direction and the direction perpendicular to flow are carried out.

Table 7.3.55 Schedule of Load on End Weir and Bottom Slab

Direction Calculation	Conditions Load	Load Term						Extra factor
		Body weight	Equipment load	Shed load	Load Temperature	Load Community	Inertial force	
Direction Perpendicular to Flow Flow Direction	Ordinary time	○	○	○		○		1.00
	Normal (Temperature rise + 16.7 °)	○	○	○	○	○		1.25
	Normal (Temperature drop -22.2 °)	○	○	○	○	○		1.25
	Seismic	○	○	○			○	1.33

Source: Study Team

(e) Calculation Result

The stress calculation results of the column and operating deck in the most critical conditions of the study cases are shown. The cross-sectional force diagram is attached in the separate structural calculation piece.

Table 7.3.56 Stress Calculation of theNo.2 Column (Flow Direction)

Member Name	Item			Units	Thickness T = 2.15 m	Thickness T = 2.15 m
	Position				Compression side	Tension side
	Load Condition				Seismic	Seismic
					(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	3830.15	3690.05
	Axial Force		N	kN	4109.15	1344.96
	Shear Stress		S	kN	562.61	528.15
Member Cross-section	Member Width		b	mm	1450	1450
	Member Height		h	mm	2150	2150
	Tension Rebar	Diameter	D	mm	32	32
		Number of Rebar	n	rebar	11	11
		Amount of Rebar	As	mm2	8846.8	8736.20
		Cover	d'	mm	130	130
	Compression Rebar	Diameter	D	mm	32	32
		Number of Rebar	n	rebar	11	11
		Amount of Rebar	As	mm2	8846.8	8736.20
		Cover	d'	mm	130	130
Stress Intensity	Concrete		σc	N/mm2	5.61	6.05
	Rebar		σs	N/mm2	49.2	153.1
	Shear Force		τ	N/mm2	0.192	0.180
Allowable Stress	Concrete		σca	N/mm2	11.01	11.01
	Rebar		σ sa	N/mm2	223.4	223.4
	Shear Force		τa	N/mm2	0.940	0.940
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

Table 7.3.57 Stress Calculation of the No.3 Column (Flow Direction)

Member Name	Item			Units	Thickness T = 2.15 m	Thickness T = 2.15 m
	Position				Compression side	Tension Side
	Load Condition				Seismic	Seismic
					(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	7660.30	7380.09
	Axial Force		N	kN	8218.29	2689.91
	Shear Stress		S	kN	1125.22	1056.3
Member Cross-section	Member Width		b	mm	3500	3500
	Member Height		h	mm	2150	2150
	Tension Rebar	Diameter	D	mm	32	32
		Number of Rebar	n	rebar	15	15
		Amount of Rebar	As	mm2	12063.8	11913.0
		Cover	d'	mm	130	130
	Compression Rebar	Diameter	D	mm	32	32
		Number of Rebar	n	rebar	15	15
		Amount of Rebar	As	mm2	12063.8	11913.0
		Cover	d'	mm	130	130
Stress Intensity	Concrete		σc	N/mm2	5.32	6.45
	Rebar		σs	N/mm2	58.9	217.4
	Shear Force		τ	N/mm2	0.159	0.149
Allowable Stress	Concrete		σca	N/mm2	11.01	11.01
	Rebar		σ sa	N/mm2	223.4	223.4
	Shear Force		τa	N/mm2	0.940	0.940
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

Table 7.3.58 Stress Calculation of the No. 2 and 3 Columns (Perpendicular to Flow)

Member Name	Item			Units	Thickness T =1.45 m	Thickness T = 3.50 m
	Position				No. 2 pier	No. 3 pier
	Load Condition				Seismic	Seismic
					(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	5627.75	10461.75
	Axial Force		N	kN	2570.06	5267.23
	Shear Stress		S	kN	555.46	1053.45
Member Cross-section	Member Width		b	mm	2150	2150
	Member Height		h	mm	1450	3500
	Tension Rebar	Diameter	D	mm	36	32
		Number of Rebar	n	rebar	17	9
		Amount of Rebar	As	mm2	17303.96	7238.25
		Cover	d'	mm	140.0	140.0
	Compression Rebar	Diameter	D	mm	36	32
		Number of Rebar	n	rebar	17	9
		Amount of Rebar	As	mm2	17304.0	7238.3
		Cover	d'	mm	140.0	140.0
Stress Intensity	Concrete		σc	N/mm2	10.88	5.74
	Rebar		σs	N/mm2	204.1	130.1
	Shear Force		τ	N/mm2	0.197	0.146
Allowable Stress	Concrete		σca	N/mm2	11.01	11.01
	Rebar		σ sa	N/mm2	223.4	223.4
	Shear Force		τa	N/mm2	0.940	0.940
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

Table 7.3.59 Stress Calculation of the No. 2 Operation Deck

Member Name	Item			Units	Thickness T = 2.00 m	Thickness T = 2.00 m
	Position				Flow Direction	Flow Direction
	Load Condition				upper tension	lower tension
					Seismic	Normal +Temperature
Cross-sectional Force	Bending Moment		M	kN m	7077.55	3561.6
	Axial Force		N	kN	379.12	-323.08
	Shear Stress		S	kN	3315.15	0.00
Member Cross-section	Member Width		b	mm	7360	3930
	Member Height		h	mm	2000	2000
	Tension Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	36	21
		Amount of Rebar	As	mm2	17671.7	10308.5
		Cover	d'	mm	70	70
	Compression Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	21	21
		Amount of Rebar	As	mm2	10308.5	10308.5
		Cover	d'	mm	70	70
Stress Intensity	Concrete		σc	N/mm2	3.77	3.24
	Rebar		σs	N/mm2	207.0	202.33
	Shear Force		τ	N/mm2	0.233	0.00
Allowable Stress	Concrete		σca	N/mm2	11.01	10.35
	Rebar		σ sa	N/mm2	223.4	210.00
	Shear Force		τa	N/mm2	0.479	0.45
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

Table 7.3.60 Stress Calculation of the No. 3 Operation Deck

Member Name	Item			Units	Thickness T = 2.00 m	Thickness T = 2.00 m
	Position				Flow Direction	Flow Direction
	Load Condition				upper tension	lower tension
					Seismic	Normal + Temperature
Cross-sectional Force	Bending Moment		M	kN m	7077.55	3561.6
	Axial Force		N	kN	379.12	-323.08
	Shear Stress		S	kN	3315.15	0.00
Member Cross-section	Member Width		b	mm	5860	3930
	Member Height		h	mm	2000	2000
	Tension Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	46	30
		Amount of Rebar	As	mm2	22580.5	14726.4
		Cover	d'	mm	70	70
	Compression Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	30	30
		Amount of Rebar	As	mm2	14726.4	14726.4
		Cover	d'	mm	70	70
Stress Intensity	Concrete		σc	N/mm2	3.78	2.71
	Rebar		σs	N/mm2	163.7	142.31
	Shear Force		τ	N/mm2	0.293	0.00
Allowable Stress	Concrete		σca	N/mm2	11.01	10.35
	Rebar		σ sa	N/mm2	223.4	210.00
	Shear Force		τa	N/mm2	0.479	0.45
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

Source: Study Team

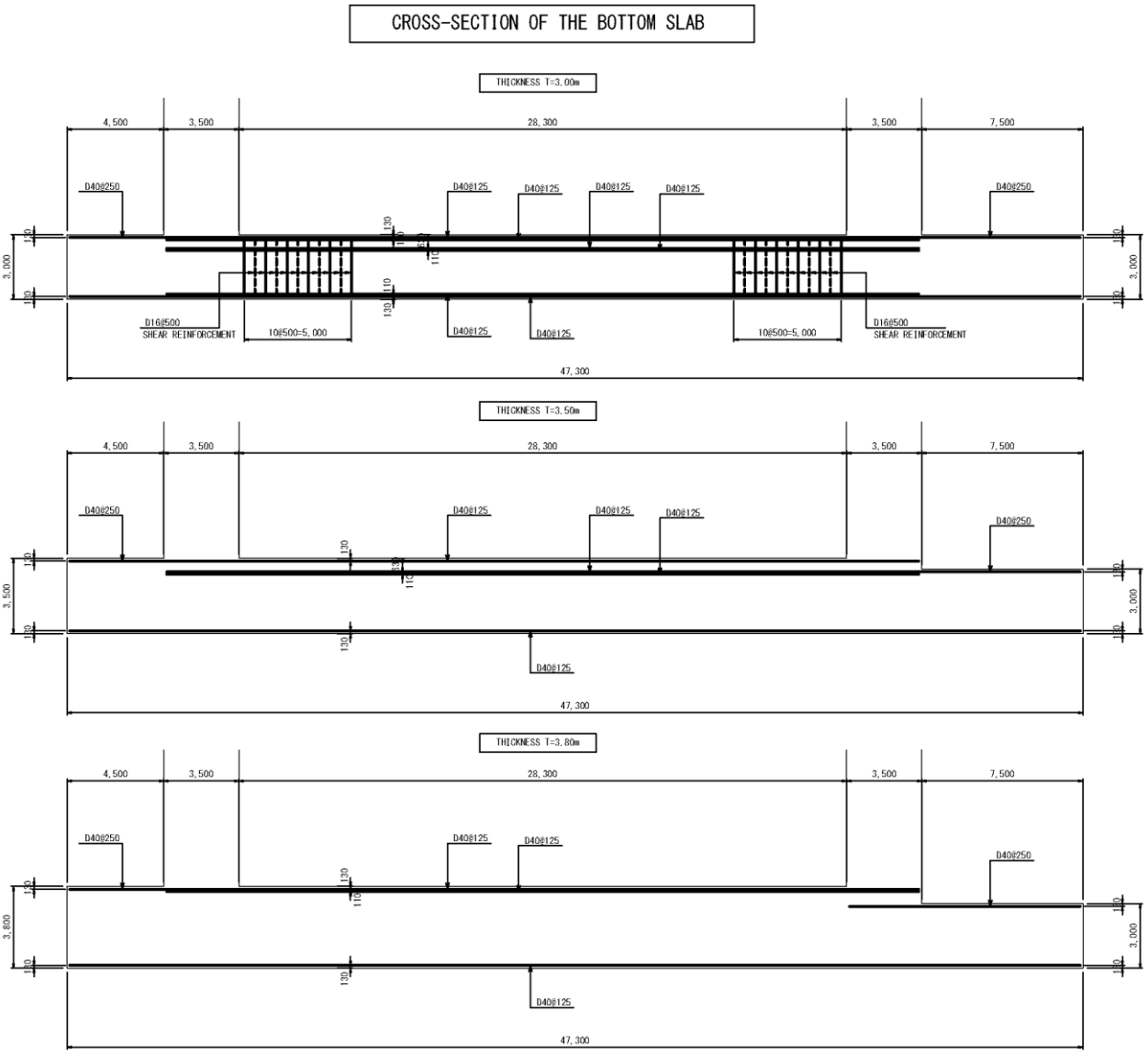
Table 7.3.61 Stress Calculation of the Overhang Part of No. 2 and No.3 Operation Decks

Member Name	Item			Units	Thickness T = 2.00 m
	Position				Weir Axial Direction
	Load Condition				Upperside, Tention
					Normal
Cross-Sectional Force	Bending Moment		M	kN・m	474.28
	Axial Force		N	kN	0
	Shear Stress		S	kN	219.54
Member Cross-Section	Member Width		b	mm	1000
	Member Height		h	mm	2000
	Tension Rebar	Diameter	D	mm	25
		Number of Rebar	n	piece	8
		Amount of rebar	As	mm^2	3927.0
		Cover	d'	mm	91
	Compression Rebar	Diameter	D	mm	16
		Number of Rebar	n	piece	4
		Amount of rebar	As	mm^2	804.2
		Cover	d'	mm	86
Stress Intensity	Concrete		σ_c	N/mm^2	1.55
	Re-bar		σ_s	N/mm^2	67.1
	Shear Force		τ	N/mm^2	0.115
Allowable Stress	Concrete		σ_{ca}	N/mm^2	8.28
	Re-bar		σ_{sa}	N/mm^2	168.0
	Shear Force		τ_a	N/mm^2	0.360
Evaluationn	$\sigma c \leqq \sigma ca$				OK
	$\sigma s \leqq \sigma sa$				OK
	$\tau \leqq \tau a$				OK

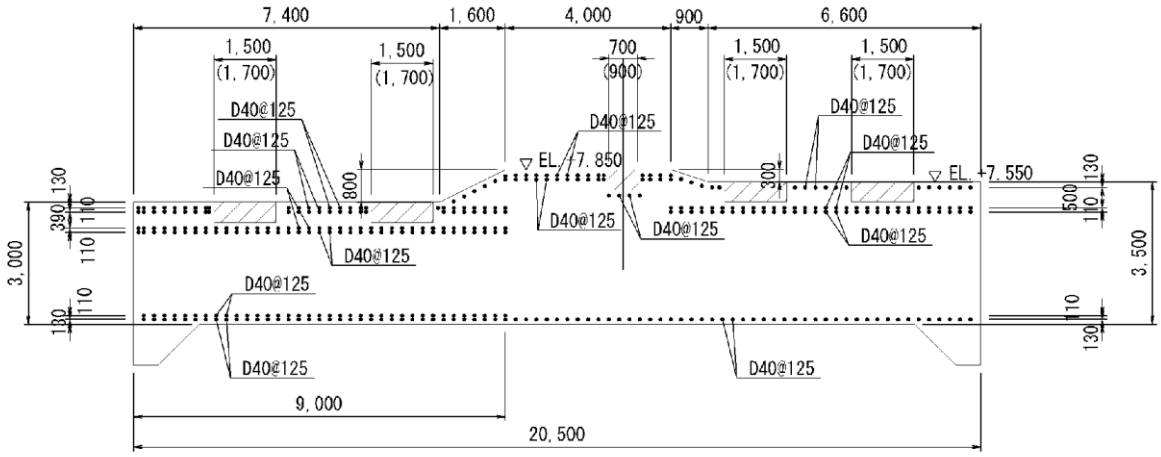
Note) The sectional force is the value calculated as a cantilever.

Source: Study Team

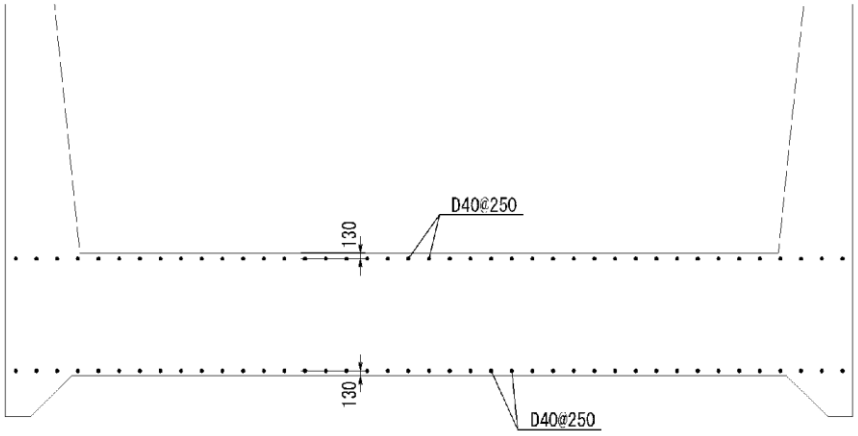
(f) Bar Arrangement Plan



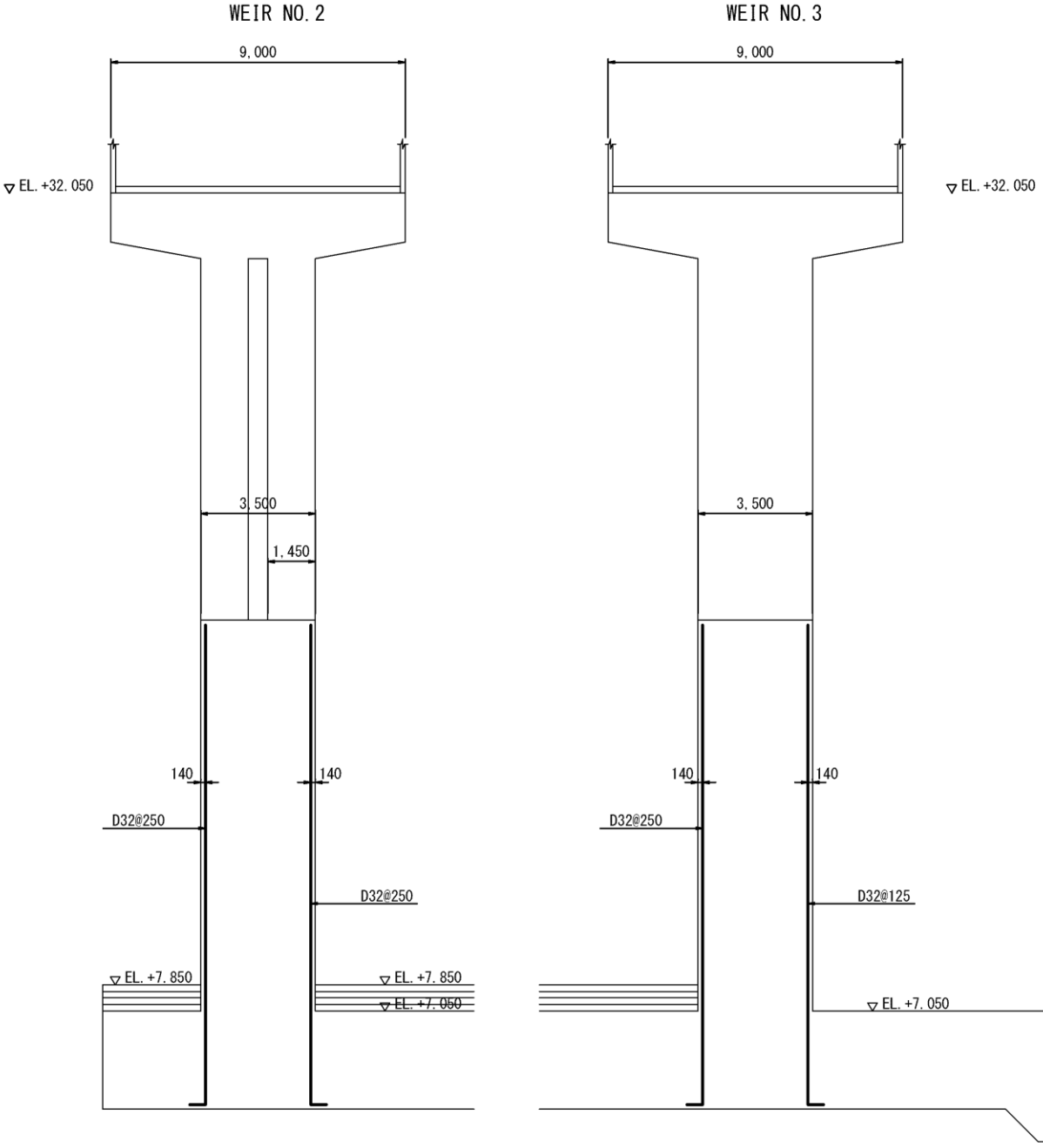
Cross-Section of Bottom Slab

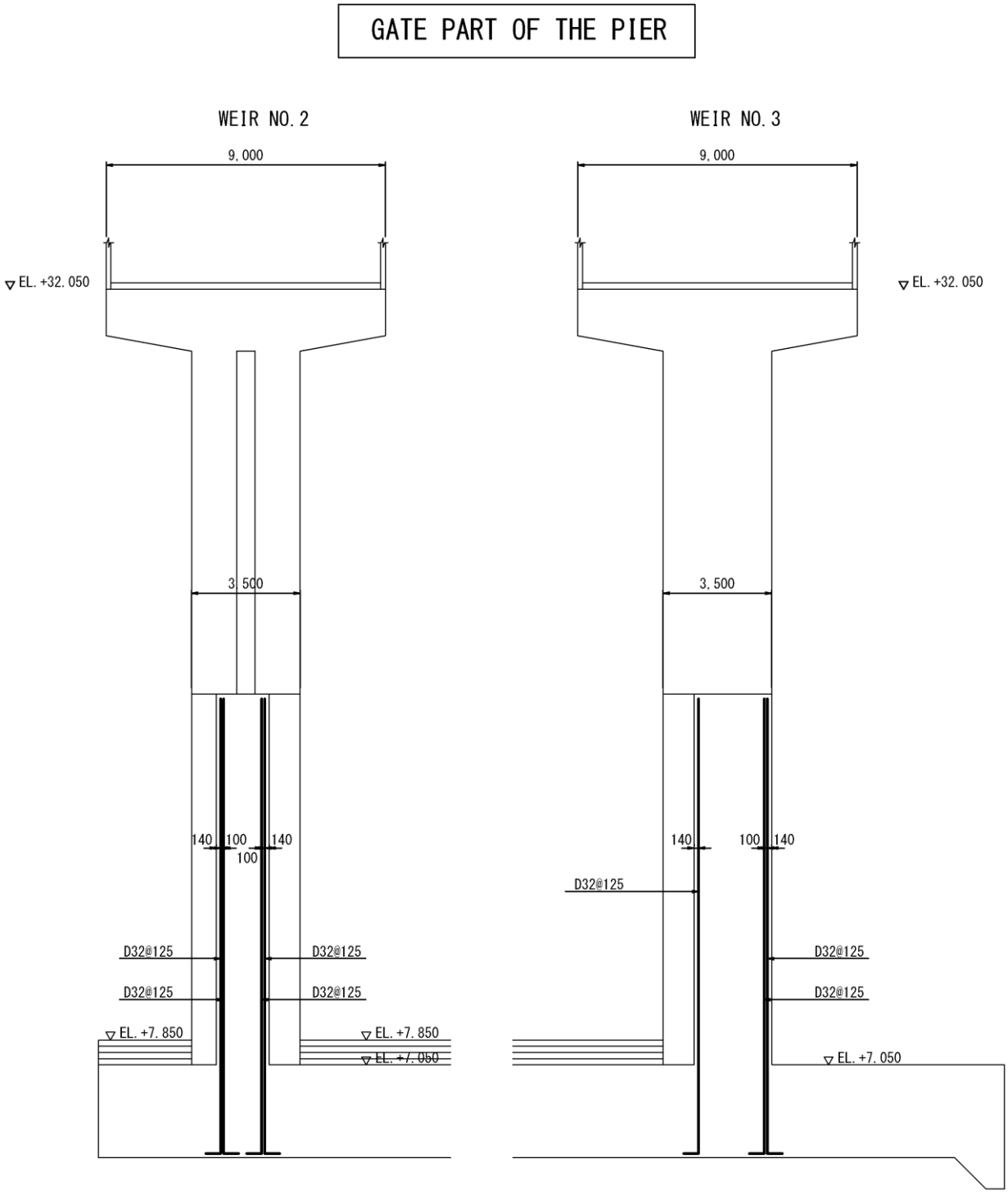


REAR VIEW of No.2 No.3 Piers



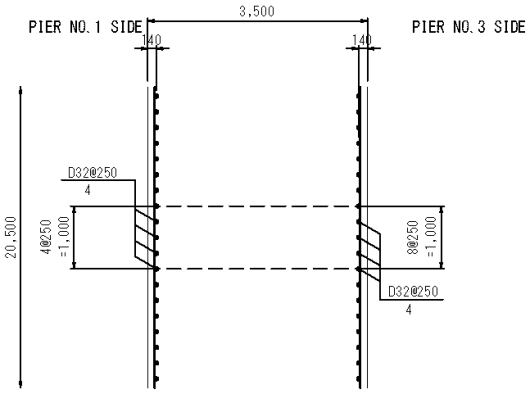
TYPICAL SECTION OF THE PIER



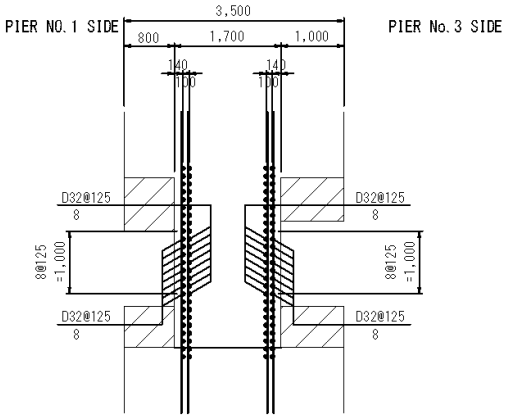

GATE PART OF THE PIER
WEIR NO. 2
WEIR NO. 3
9,000
9,000
EL. +32.050
EL. +32.050
3,500
3,500
140
100
140
100
140
100
140
D32@125
D32@125
D32@125
D32@125
D32@125
D32@125
D32@125
EL. +7.850
EL. +7.850
EL. +7.050
EL. +7.050

PIER NO.2

STANDARD CROSS-SECTION OF THE PIER

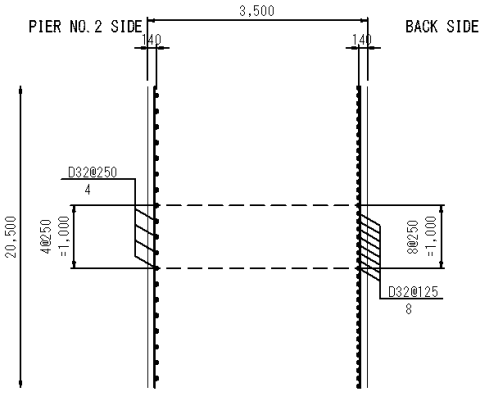


CROSS-SECTION OF THE GATE PART

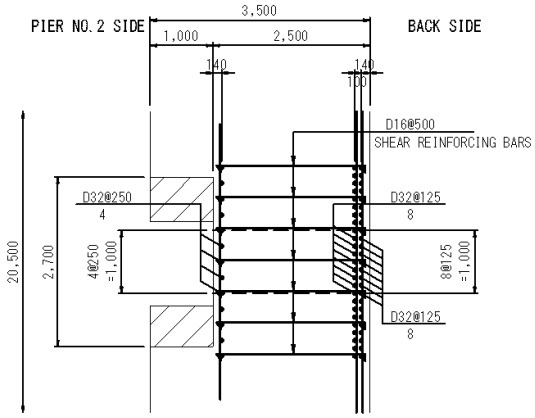


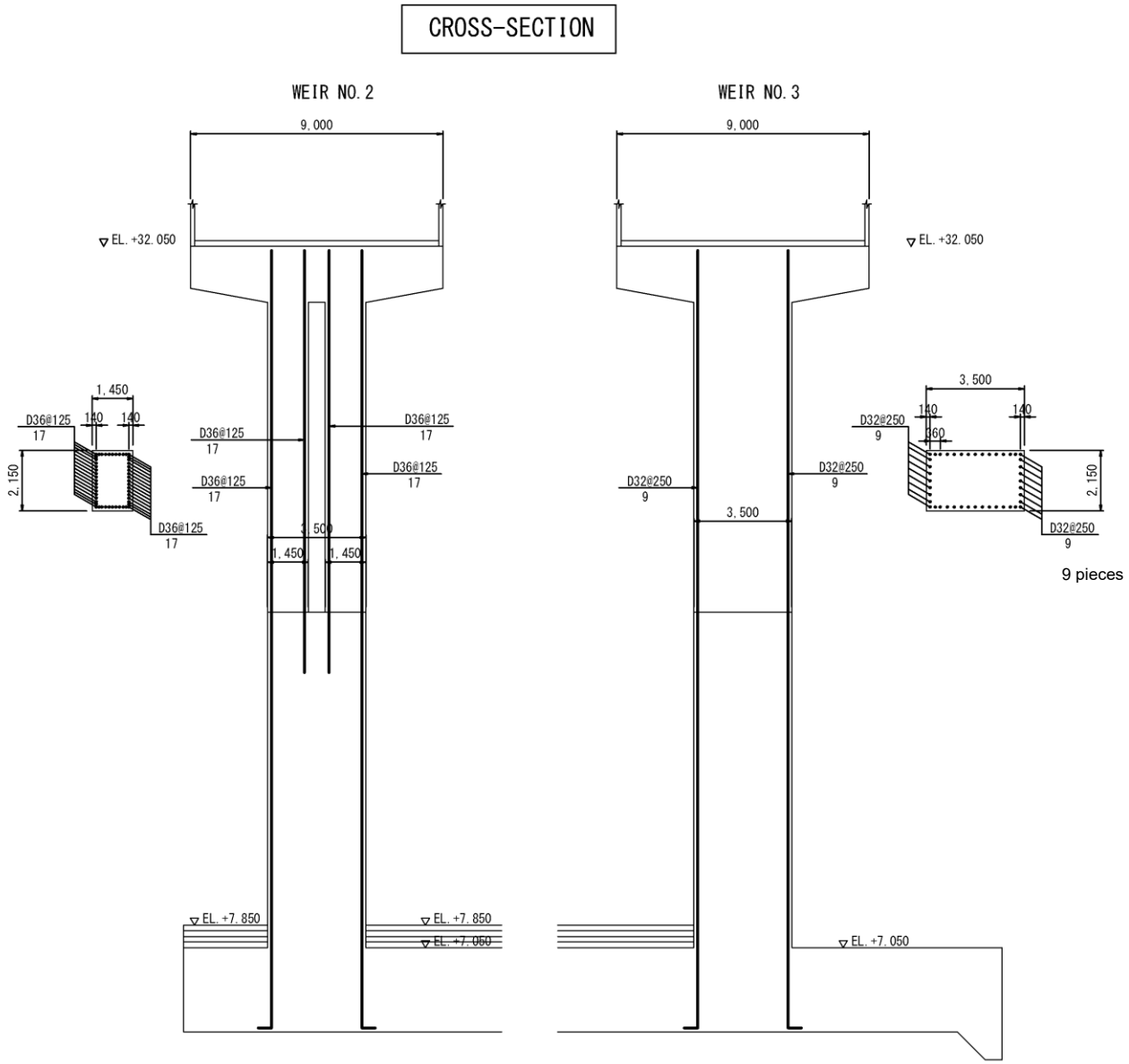
PIER NO.3

STANDARD CROSS-SECTION OF THE PIER



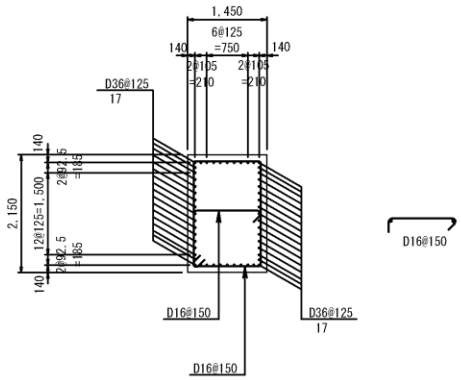
CROSS-SECTION OF THE GATE PART



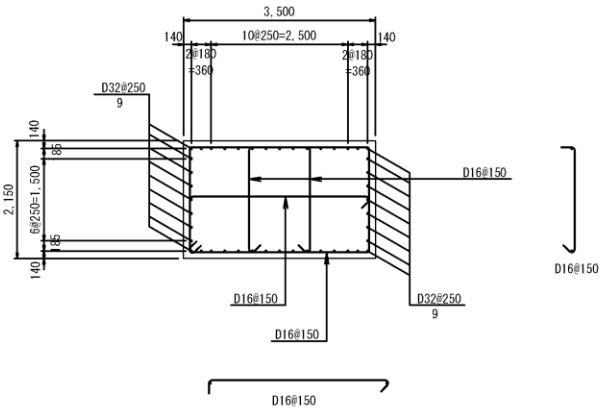

CROSS-SECTION
WEIR NO. 2
WEIR NO. 3
9,000
9,000
EL. +32.050
EL. +32.050
1,450
140
140
D36@125
17
D36@125
17
D36@125
17
D36@125
17
D36@125
17
2,150
3,500
1,450
1,450
3,500
D32@250
9
D32@250
9
D32@250
9
D32@250
9
140
360
140
2,150
9 pieces
EL. +7.850
EL. +7.850
EL. +7.060
EL. +7.050

CROSS-SECTION

WEIR NO.2



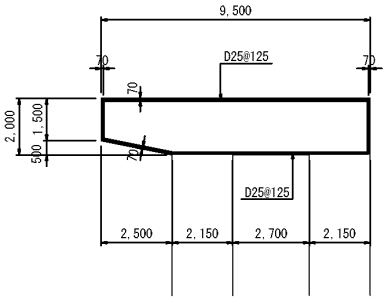
WEIR NO.3



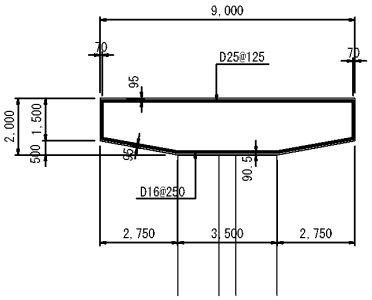
CROSS-SECTION OF THE PIER OF OPERATION DECK

NO.2 OPERATION DECK

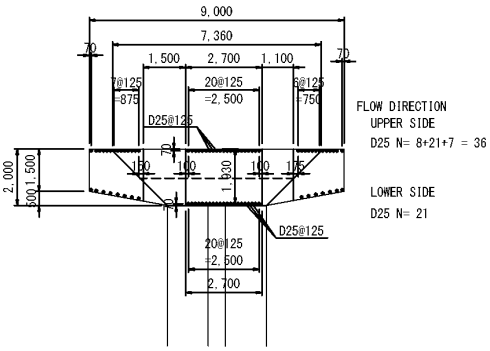
FLOW DIRECTION



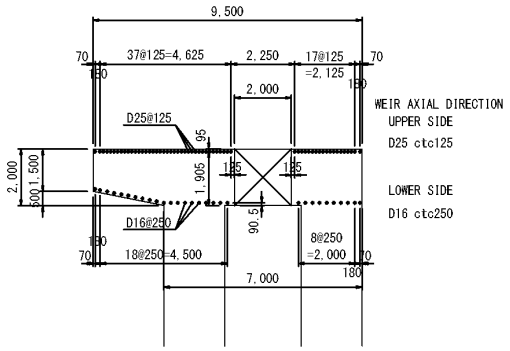
WEIR AXIAL DIRECTION



FLOW DIRECTION



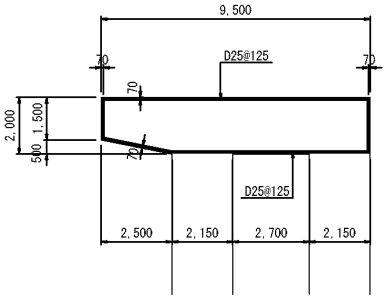
WEIR AXIAL DIRECTION



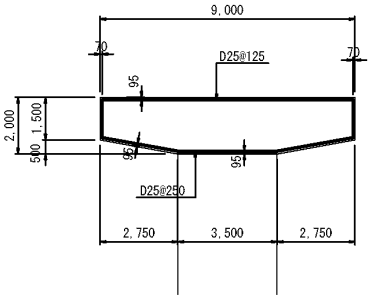
CROSS-SECTION OF THE PIER OF OPERATION DECK

NO.2 OPERATION DECK

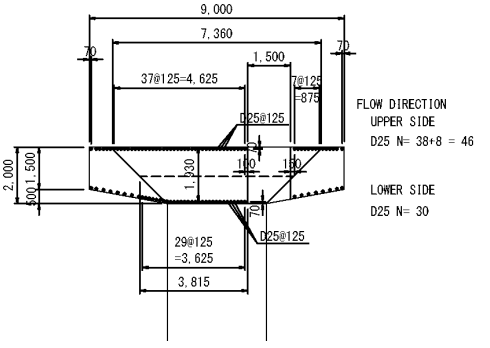
FLOW DIRECTION



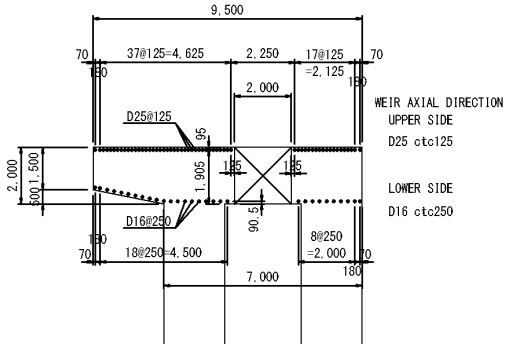
WEIR AXIAL DIRECTION



FLOW DIRECTION



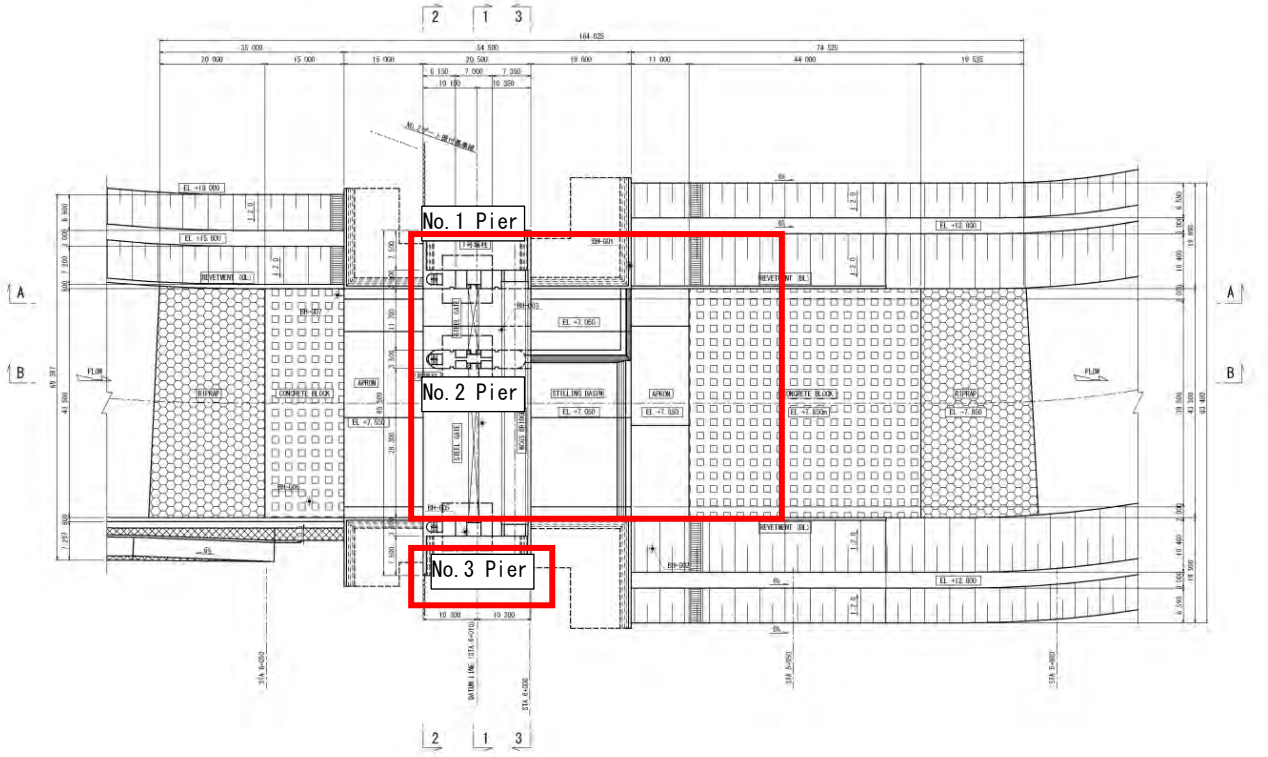
WEIR AXIAL DIRECTION



4) Breast Wall

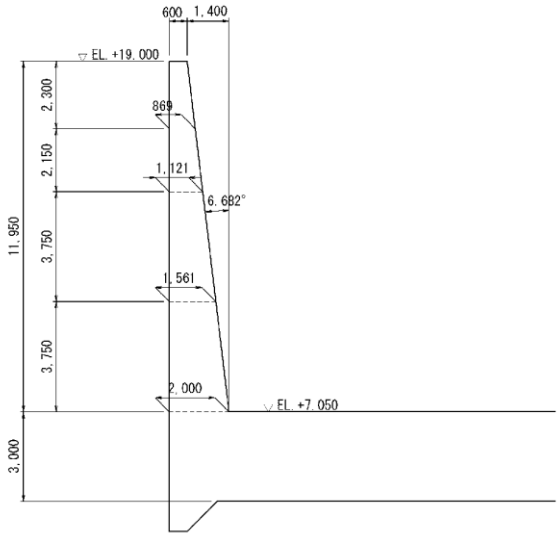
The object is the breast wall connected with No. 1 and No. 3 piers. Location of the breast wall is shown in **Figure 7.3.18**.

PLAN



Source: Study Team

Figure 7.3.18 Plan of Breast Wall



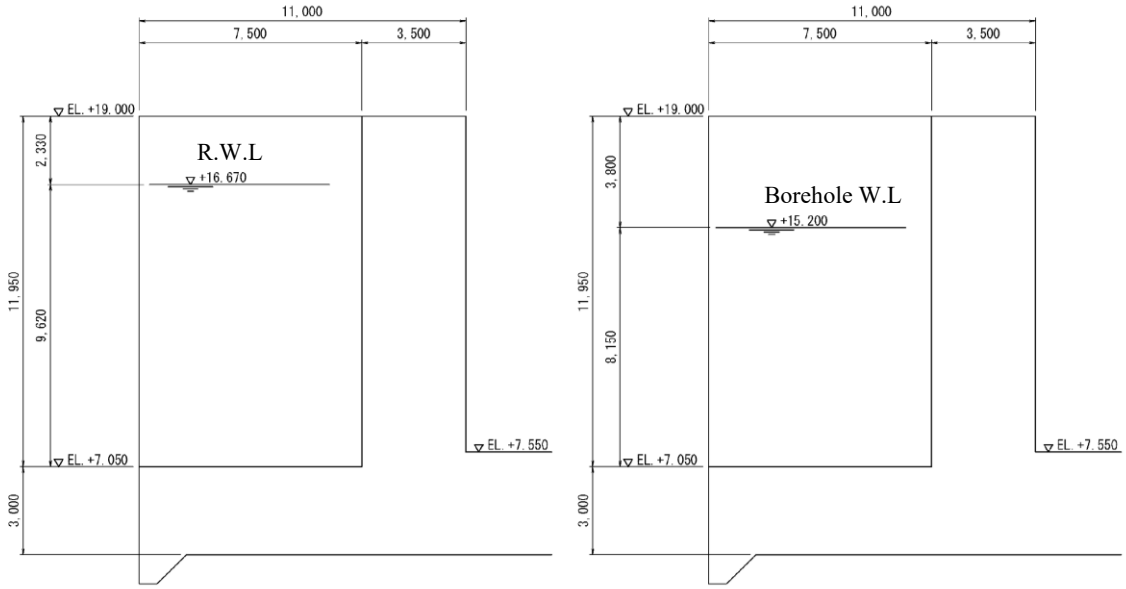
Source: Study Team

Figure 7.3.19 Dimensional of Breast Wall

(a) Study Case

Normal and seismic is considered for design. Structural calculation of the breast wall shall be carried out under constant Load Conditions when the water level difference between the front and back sides of the wall becomes the largest. The Load Conditions for calculating the breast wall are shown below.

Back earth pressure	: Constant and static earth pressure
Soil property of the back soil	: Backfill γ = 19.0 kN/m^3
	Internal friction angle ϕ = 30 ° ,
	cohesion c = 0 kN/m^2
Back side water level (normal)	: EL + 16.67 (residual water level)
Back side water level (seismic)	: EL + 15.20 (water level in the borehole)
Front water level	: in case of empty



(Normal) (Seismic)

Source: Study Team

Figure 7.3.20 Water Level Condition of Breast Wall

(b) Calculation Result

The stress calculation result of the breast wall is shown below.

Table 7.3.62 Stress Calculation of Breast Wall (1)

Member Name	Item			Units	Thickness T = 2.00 m	Thickness T = 2.00 m	Thickness T = 2.00 m	Thickness T = 2.00 m
	Position				B-D	B-D	C-D	C-D
	Load Condition				Normal (Residual W.L)	Seismic (Borehole W.L)	Normal (Residual W.L)	Normal (Residual W.L)
					(Long-term)	(Short-term)	(Long-term)	(Long-term)
Cross-sectional Force	Bending Moment		M	kN m	2205.00	2595.38	795.66	940.55
	Axial Force		N	kN	0	0	0	0
	Shear Stress		S	kN	715.47	847.69	455.49	540.53
Member Cross-section	Member Width		b	mm	1000	1000	1000	1000
	Member Height		h	mm	2000	2000	2000	2000
	Tension Rebar	Diameter	D	mm	40	40	40	40
		Number of	n	rebar	8	8	4	4
		Amount of Rebar	As	mm2	10053.12	10053.12	5026.56	5027
		Cover	d'	mm	166.0	170.0	170.0	170.0
	Compression Rebar	Diameter	D	mm	25	25	25	25
		Number of	n	rebar	4	4	4	4
		Amount of Rebar	As	mm2	1963.52	1963.52	1964	1964
		Cover	d'	mm	135.0	135.0	135.0	135.0
Stress Intensity	Concrete		σc	N/mm2	5.19	6.13	2.47	2.92
	Re-bar		σs	N/mm2	131.0	154.5	92.5	109.37
	Shear Force		τ	N/mm2	0.390	0.463	0.249	0.30
Allowable Stress	Concrete		σca	N/mm2	8.28	11.01	8.28	11.01
	Re-bar		σ sa	N/mm2	168.0	223.4	168.00	223.44
	Shear Force		τa	N/mm2	0.360	0.48	0.36	0.48
Evaluationn	σc ≦ σca				OK	OK	OK	OK
	σs ≦ σsa				OK	OK	OK	OK
	τ ≦ τa				NG	OK	OK	OK

Note) For B-D Section at Normal (residual), shear reinforcing bars are allocated.

Source: Study Team

Table 7.3.63 Stress Calculation of Breast Wall (2)

Member Name	Item			Units	Thickness T = 1.121 m	Thickness T = 1.121 m	Thickness T = 1.561 m	Thickness T = 1.561 m
	Position				B-B	B-B	C-C	C-C
	Load Condition				Normal (Residual W.L)	Seismic (Borehole W.L)	Normal (Residual W.L)	Normal (Residual W.L)
					(Long-term)	(Short-term)	(Long-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	1249.03	1428.47	678.87	794.67
	Axial Force		N	kN	0	0	0	0
	Shear Stress		S	kN	333.08	380.93	362.06	423.83
Member Cross-section	Member Width		b	mm	1000	1000	1000	1000
	Member Height		h	mm	1121	1121	1561	1561
	Tension Rebar	Diameter	D	mm	40	40	40	40
		Number of Rebar	n	rebar	8	8	4	4
		Amount of Rebar	As	mm2	10053.12	10053.12	3217.00	3217
		Cover	d'	mm	130	130	130	130
	Compression Rebar	Diameter	D	mm	25	25	25	25
		Number of Rebar	n	rebar	4	4	4	4
		Amount of Rebar	As	mm2	1963.52	1963.52	1964	1964
		Cover	d'	mm	110	110	110	110
Stress Intensity	Concrete		σc	N/mm2	7.95	9.09	3.72	4.35
	Re-bar		σs	N/mm2	141.2	161.5	156.9	183.63
	Shear Force		τ	N/mm2	0.336	0.384	0.253	0.30
Allowable Stress	Concrete		σca	N/mm2	8.28	11.01	8.28	11.01
	Re-bar		σ sa	N/mm2	168.0	223.4	168.00	223.44
	Shear Force		τa	N/mm2	0.360	0.48	0.36	0.48
Evaluationn	σc ≦ σca				OK	OK	OK	OK
	σs ≦ σsa				OK	OK	OK	OK
	τ ≦ τa				OK	OK	OK	OK

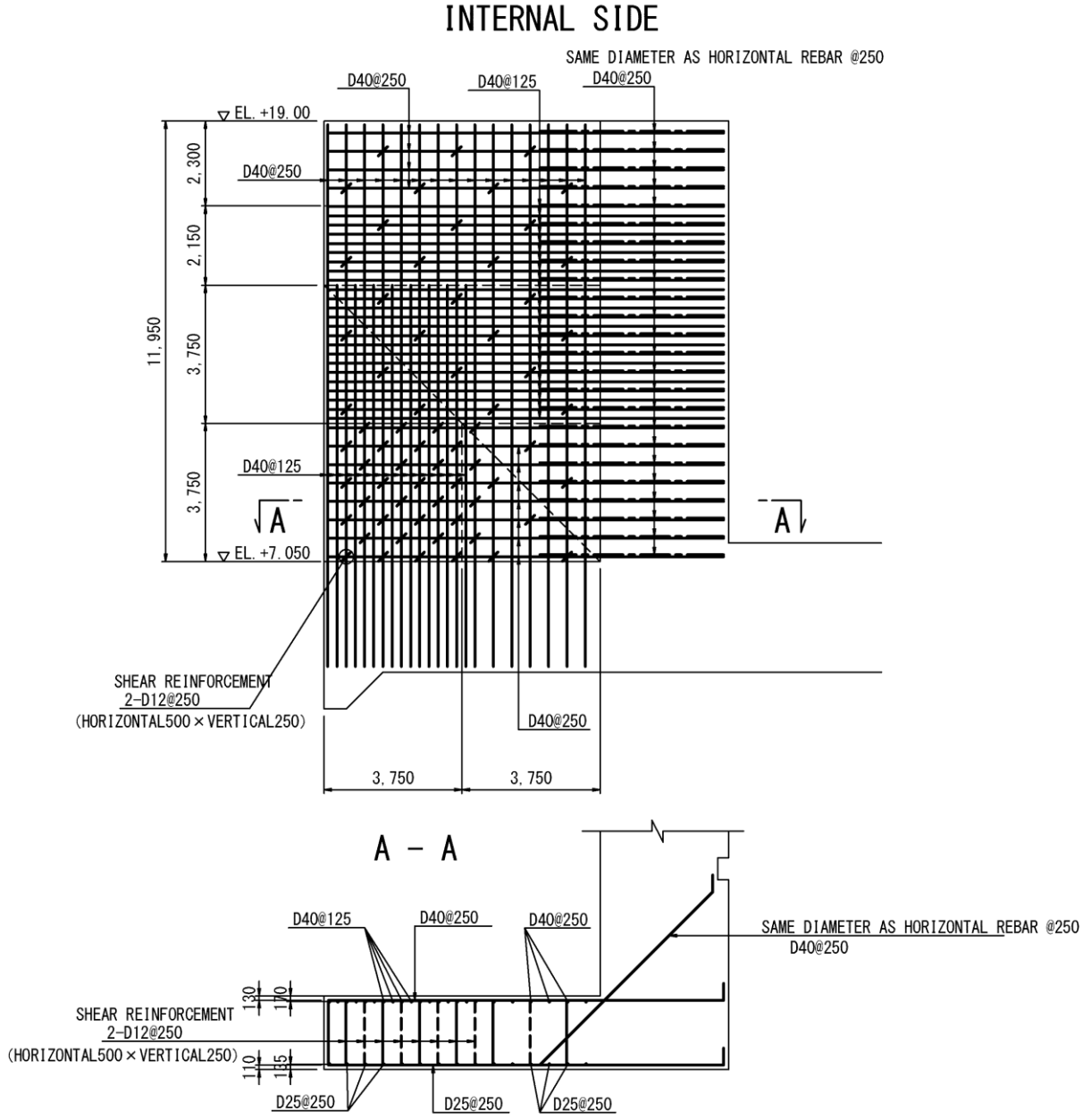
Source: Study Team

Table 7.3.64 Stress Calculation of Breast Wall (3)

Member Name	Item			Units	Thickness T = 869 m	Thickness T = 869 m
	Position				E-E	E-E
	Load Condition				Normal (RWL)	Seismic (GWL)
					(Long-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	470.25	709.59
	Axial Force		N	kN	0	0
	Shear Stress		S	kN	125.4	189.23
Member Cross-section	Member Width		b	mm	1000	1000
	Member Height		h	mm	869	869
	Tension Rebar	Diameter	D	mm	40	40
		Number of Rebar	n	rebar	4	4
		Amount of Rebar	As	mm2	5026.56	5026.56
		Cover	d'	mm	130	130
	Compression Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	4	4
		Amount of Rebar	As	mm2	1963.52	1963.52
		Cover	d'	mm	110	110
Stress Intensity	Concrete		σc	N/mm2	6.21	9.38
	Re-bar		σs	N/mm2	140.5	212.0
	Shear Foece		τ	N/mm2	0.170	0.256
Allowable Stress	Concrete		σca	N/mm2	8.28	11.01
	Re-bar		σ sa	N/mm2	168.0	223.4
	Shear Foece		τa	N/mm2	0.360	0.48
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	OK
	τ ≦ τa				OK	OK

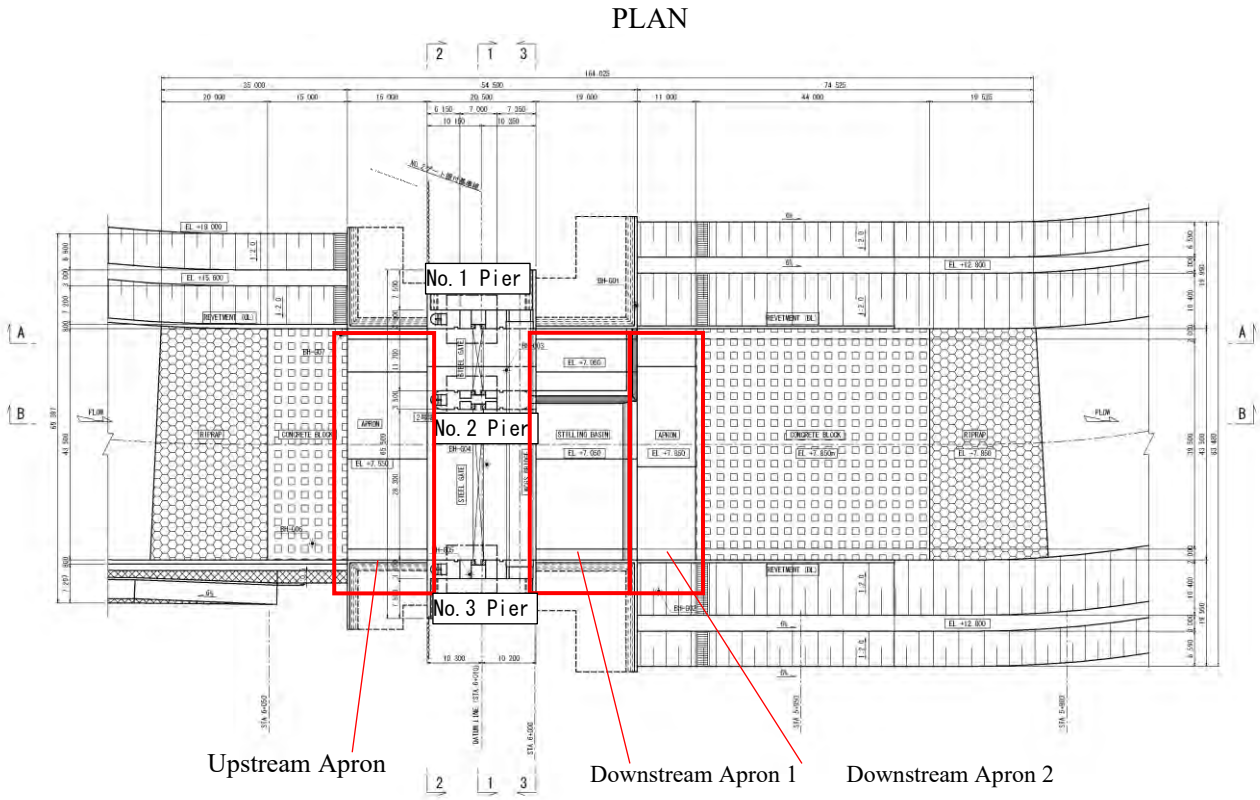
Source: Study Team

(c) Bar Arrangement Plan



5) Upstream Apron

As shown in **Figure 7.3.21**, apron consists of "upstream Apron", "downstream Apron 1", and "downstream Apron 2".



Source: Study Team

Figure 7.3.21 Allocation of Aprons

(a) Study Case

The examination cases are normal, construction (no heavy machineries), and during construction (with heavy machineries)".

Table 7.3.65 Loads of Upstream Apron

Direction	Study Case	vertical load V (kN)	Moment My (kNm)
Direction Perpendicular to Flow	Normal	1031.05	8763.93
	During construction (No crane load)	489.60	4161.60
	During construction (with crane load)	922.76	7904.62
Flow Direction	Normal	909.75	6823.13
	During construction (No crane load)	432.00	3240.00
	During construction (with crane load)	865.16	6593.17

Source: Study Team

Table 7.3.66 Subgrade Reaction

Direction	Study Case	X = L (m)	qmin (kN/m2)	qmax (kN/m2)
Direction Perpendicular to Flow	Normal	17.000	60.650	60.650
	Flood	17.000	28.800	28.800
	Seismic	17.000	53.016	55.544
Flow Direction	Normal	15.000	60.650	60.650
	Flood	15.000	28.800	28.800
	Seismic	15.000	54.886	60.469

Source: Study Team

Crane Load

The 70t crane is assumed to calculate its load capacity with the reaction force at the outrigger position.

Self-weight of the crane : 41.3t (Assumption from technical report of manufacturers)
Hanging weight :10.0t (Gate Weight)

Hook weight :0.7t

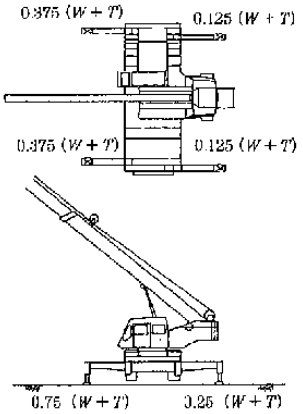
Total *W=52.0t*

Reaction force acts in the outrigger is calculated as follows © Diagonally front/back hanging).

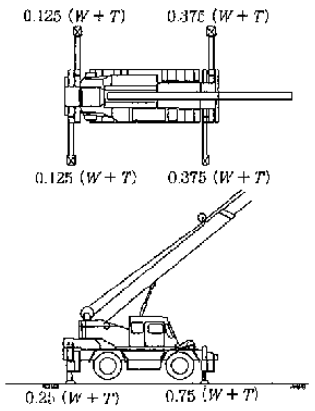
Crane load1 :W×0.70=52.0×0.70 = 36.40t

Crane load2 : W×0.15=52.0×0.15 = 7.80t

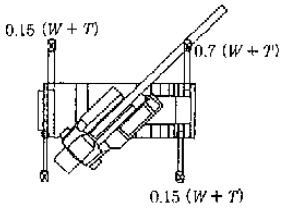
(a) Side hanging



b) Front/back hanging



c) Diagonally in front/back hanging



Normal (residual water)

Crane load on the Bottom Slab is not considered

At construction (Outrigger distance L= 7.600m)

P1 = 36.40×9.800

load acting position x1 = 7.300m

P2= 7.80×9.800

load acting position x2 = 14.900m

(b) Result of Examination

Stress calculation result of upstream Apron (L = 17.0m in direction perpendicular to flow), L = 15.0m in flow direction) is shown. The cross-sectional force diagram is shown in the separate structural calculation piece.

Table 7.3.67 Stress Analysis of Upstream Apron (Direction Perpendicular to Flow)

Member Name	Item			Units	Thickness T = 1.20 m	Thickness T = 1.20 m	Thickness T = 1.20 m
	Position				Upperside, Tension	Upperside, Tension	Underside, Tension
	Load Condition				Normal	During construction	During construction
					(Long-term)	(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	1462.7	2092.72	656.43
	Axial Force		N	kN	0	0	0
	Shear Stress		S	kN	344.17	492.41	180.96
Member Cross-section	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1200	1200	1200
	Tension Rebar	Diameter	D	mm	32	32	32
		Number of Rebar	n	rebar	16	16	4
		Cover	d'	mm	170	170	120
	Compression Rebar	Diameter	D	mm	32	32	32
		Number of Rebar	n	rebar	4	4	16
		Cover	d'	mm	120	120	170
Stress Intensity	Concrete		σc	N/mm2	7.86	11.24	5.36
	Re-bar		σs	N/mm2	125.4	179.4	205.1
	Shear Forces		τ	N/mm2	0.334	0.478	0.168
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Re-bar		σ sa	N/mm2	168.0	252.0	252.0
	Shear Forces		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

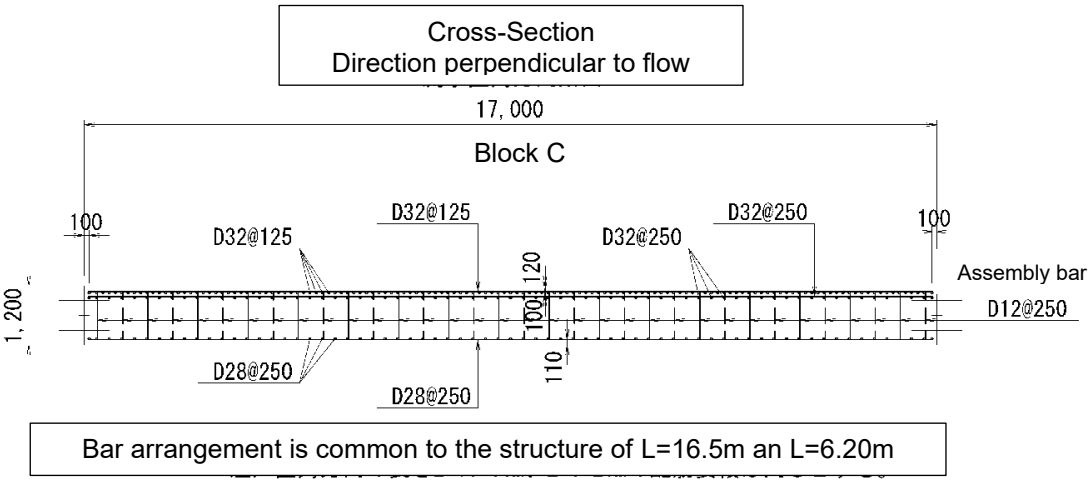
Source: Study Team

Table 7.3.68 Stress Analysis of Upstream Apron (Flow Direction)

Member Name	Item			Units	Thickness T = 1.20 m	Thickness T = 1.20 m	Thickness T = 1.20 m
	Position				Upperside, Tension	Upperside, Tension	Underside, Tension
	Load Condition				Normal	During Construction	During Construction
					(Long-term)	(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	1138.78	1629.28	551.78
	Axial Force		N	kN	0	0	0
	Shear Stress		S	kN	303.67	434.48	174.78
Member Cross-section	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1200	1200	1200
	Tension Rebar	Diameter	D	mm	32	32	28
		Number of Rebar	n	rebar	12	12	4
		Cover	d'	mm	185	185	150
	Compression Rebar	Diameter	D	mm	28	28	32
		Number of Rebar	n	rebar	4	4	12
		Cover	d'	mm	150	150	185
Stress Intensity	Concrete		σc	N/mm2	7.09	10.14	5.67
	Re-bar		σs	N/mm2	130.9	187.3	228.1
	Shear Force		τ	N/mm2	0.299	0.428	0.166
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Re-bar		σ sa	N/mm2	168.0	252.0	252.0
	Shear Force		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

Source: Study Team

(c) Bar Arrangement Plan



6) Downstream Apron 1

(a) Study Case

The examination cases are normal, construction (no heavy machineries), and during construction (with heavy machineries)".

Table 7.3.69 Downstream Apron 1 Load Schedule

Direction	Study Case	Vertical Load V (kN)	Moment My (kN m)
Direction Perpendicular to Flow (L=17.0)	Normal	1318.35	11205.98
	During construction (No crane load)	693.60	5895.60
	During construction (With crane load)	1126.76	9638.62
Direction Perpendicular to Flow (L=16.3)	Normal	1378.31	10839.10
	During construction (No crane load)	779.28	5957.01
	During construction (With crane load)	1212.44	9570.07
Flow Direction (L=19.0)	Normal	1473.45	13997.78
	During construction (No crane load)	775.20	7364.40
	During construction (With crane load)	1208.36	11562.23

Source: Study Team

Table 7.3.70 Subgrade Reaction

Direction	Study Case	X = L (m)	qmin (kN/m2)	qmax (kN/m2)
Direction Perpendicular to Flow (L=17.0)	Normal	17.000	77.550	77.550
	Flood	17.000	40.800	40.800
	Seismic	17.000	65.017	67.543
Direction Perpendicular to Flow (L=16.3)	Normal	16.300	75.657	93.461
	Flood	16.300	38.904	56.713
	Seismic	16.300	67.346	81.420
Flow Direction (L=19.0)	Normal	19.000	77.550	77.550
	Flood	19.000	40.800	40.800
	Seismic	19.000	62.212	64.984

Source: Study Team

(b) Calculation Result

Stress calculation result of downstream Apron1 (L = 17.0m in direction perpendicular to flow), L = 19.0m in flow direction) is shown. The cross-sectional force diagram is shown in the separate structural calculation piece.

Table 7.3.71 Stress Calculation for Downstream Apron 1 (Direction Perpendicular to Flow L=17.0m)

Member Name	Item			Units	Thickness T = 1.70 m	Thickness T = 1.70 m	Thickness T = 1.70 m
	Position				top tension	top tension	undersurface tension
	Load Condition				Normal (Long-term)	during construction (Short-term)	during construction (Short-term)
Cross-sectional Force	Bending Moment		M	kN m	1206.21	2013.25	656.43
	Axial Force		N	kN	0	0	0
	Shear Stress		S	kN	283.82	473.71	180.96
Member Cross-section	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1700	1700	1700
	Tension Rebar	Diameter	D	mm	25	25	25
		Number of Rebar	n	rebar	12	12	4
		Cover	d'	mm	143	143	110
	Compression Rebar	Diameter	D	mm	25	25	25
		Number of Rebar	n	rebar	4	4	12
		Cover	d'	mm	110	110	143
Stress Intensity	Concrete		σc	N/mm2	4.52	7.54	3.65
	Rebar		σs	N/mm2	142.0	237.1	221.2
	Shear Force		τ	N/mm2	0.182	0.304	0.114
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Rebar		σ sa	N/mm2	168.0	252.0	252.0
	Shear Force		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

Source: Study Team

Table 7.3.72 Stress Calculation for Downstream Apron 1 (Direction Perpendicular to Flow L=16.3m)

Member Name	Item			Units	Thickness T = 1.70 m	Thickness T = 1.70 m	Thickness T = 1.70 m
	Position				top tension	top tension	undersurface tension
	Load Condition				Normal (Long-term)	during construction (Short-term)	during construction (Short-term)
Cross-sectional Force	Bending Moment		M	kN・m	856.71	1592.13	684.41
	Axial Force		N	kN	0.00	0.00	0.00
	Shear Stress		S	kN	190.83	372.90	149.14
member section	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1700	1700	1700
	Tension Rebar	Diameter	D	mm	25	25	25
		Number of Rebar	n	rebar	12	12	4
		Amount of rebar	As	mm2	5890.56	5890.56	1963.52
		Cover	d'	mm	143.3	143.3	110.0
	Compression Rebar	Diameter	D	mm	25	25	25
		Number of Rebar	n	rebar	4	4	12
		Amount of rebar	As	mm2	1963.52	1963.52	5890.56
		Cover	d'	mm	110.0	110.0	143.3
Stress Intensity	Concrete		σc	N/mm2	3.21	5.96	3.81
	Rebar		σs	N/mm2	100.9	187.5	230.6
	Shear Force		τ	N/mm2	0.123	0.240	0.094
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Rebar		σsa	N/mm2	168.0	252.0	252.0
	Shear Force		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

Source: Study Team

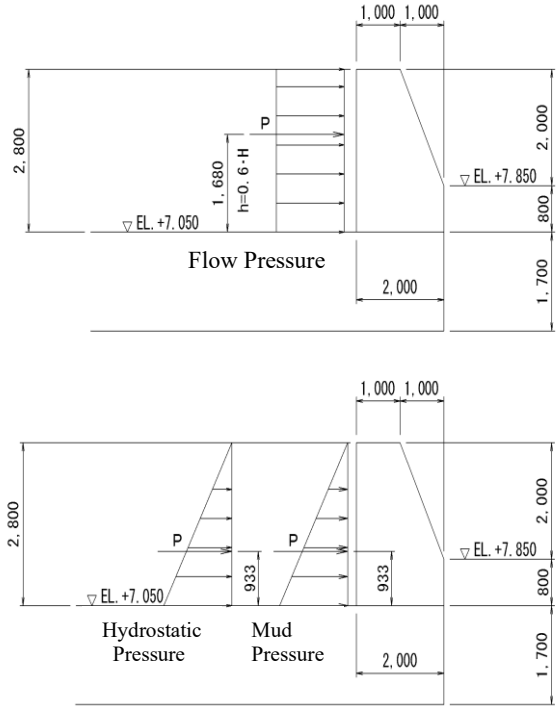
Table 7.3.73 Stress Calculation for Downstream Apron 1 (Flow Direction)

Member Name	Item			Units	Thickness T = 1.70 m	Thickness T = 1.70 m	Thickness T = 1.70 m
	Position				top tension	top tension	undersurface tension
	Load Condition				Normal	during construction	during construction
					(Long-term)	(Short term)	(Short term)
Cross-sectional Force	Bending Moment		M	kN・m	1506.72	2514.82	763.05
	Axial Force		N	kN	0.00	0.00	0.00
	Shear Stress		S	kN	317.21	529.44	184.16
member sectio	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1700	1700	1700
	Tension Rebar	Diameter	D	mm	32	32	28
		Number of Rebar	n	rebar	12	12	4
		Amount of Rebar	As	mm2	9651.00	9651.00	2463.00
		Cover	d'	mm	171.8	171.8	136.5
	Compression Rebar	Diameter	D	mm	28	28	32
		Number of Rebar	n	rebar	4	4	12
		Amount of Rebar	As	mm2	2463.00	2463.00	9651.00
		Cover	d'	mm	136.5	136.5	171.8
Stress Intensity	Concrete		σc	N/mm2	4.79	8.00	3.91
	Rebar		σs	N/mm2	112.5	187.8	210.2
	Shear Force		τ	N/mm2	0.208	0.346	0.118
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Rebar		σsa	N/mm2	168.0	252.0	252.0
	Shear Force		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

Source: Study Team

(c) Study Case (Sill)

The working load shall be the hydraulic pressure, the water pressure + mud pressure, and the horizontal force due to the ship impact load.



1. flow pressure

$P = K \cdot \upsilon 2 \cdot A$

K = 0.7 Use the coefficient 0.7 in Table −2.2 on page 50 of the Road Bridge Specifications I Common Edition.

υ = 8.50 m/s It is assumed that the flow velocity before the installation of the reduction facility in the hydraulic experiment is 14.85 m/s.

A = H L Vertical Projection Area

= 2.800 × 1.000 = 2.80 m2

P = 0.70 × 8.50 × 8.50 × 2.80 = 141.61 kN

bending moment

M = P − h

P = 141.61 kN

h = 0.6 H The working position is 0.6 H from the bottom.

= 0.6 × 2.800 = 1.68 m

M = 141.61 × 1.68 = 237.90 kN m

shear force

S = 141.61 kN

2. mud pressure

For a deposition thickness h = 2.800 m

mud pressure coe 0.5

Unit weight in wat 1.0 t/m3 × 9.8 = 9.80 kN/m3

Pe = Cek・γ・h

P = 1/2・Pe・h・L

kh = 0.21 seismic design horizontal seismic intensity

V = 1/2・(B1 + B2)・H・L・γc

Pe = 0.5 × 9.80 × 2.800 = 13.72 kN

P = 1/2 × 13.72 × 2.800 × 1.000 = 19.21 kN

bending moment

M = P − h

P = 19.21 kN

h = 2.800 × 1/3 = 0.933 m

M = 19.21 × 0.933 = 17.92 kN m

shear force

S = 19.21 kN

3. hydrostatic pressure

Calculations are made on the safe side, assuming that there is no downstream water level.

hydraulic load

P = 1/2・γw・h2

P = 1/2 × 9.80 × 2.800 × 2.800 = 38.42 kN/m

Action Position Y = 2.800 /3 = 0.933 m

M = 38.42 × 0.933 = 35.85 kN m

shear force

S = 38.42 kN

4. Total horizontal load = mud pressure + hydrostatic pressure

bending moment

M = 17.92 + 35.85 = 53.77 kN m

shear force

S = 19.21 + 38.42 = 57.63 kN m

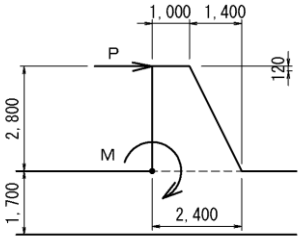
5. Impact Load by Ship

Specification of Ferry Boat

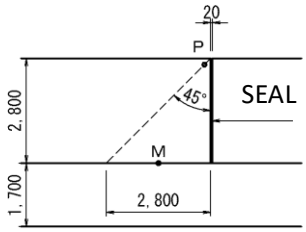
Item	Ferry boat	
	Type-1	Type-2
Weight (t)	19.40	16.4
Length (m)	15.10	11.9
Width (m)	5.00	2.5
Minimum Water Level (m)	1.90	1.3
Required clearance from the water surface (m)	3.00	3.00

source: The Phase I Report Vol.VII, P2-2, 2002

CROSS-SECTION



FRONT VIEW



Sailing Speed :

12 knot = 6.17m/s

Virtual Weight W

W = Wo + W '= (19.40 x 9.8) × 3 + (pi/4) × 1.902 × 5.00 × 9.8

= 709.22 kN ≒ 710.0 kN

Impact Energy E

E = Eo · V2/(2 g) = 710.0 × 6.172/(2 x 9.8)

= 1379.03 kj ≒ 1380.0 kj

Navigational Speed V 6.17 m/s

Calculation of the absorbed energy

The horizontal force by which concrete is bent and destroyed is calculated.

Failure bending moment M (Per meter of unit)

M = 1474.9 kN m/m (From calculation of yield stress of next page member)

Horizontal force P ' = M/h = 526.75 kN/m

Working height h = 2.80 m

Here, the load dispersion angle (45 °) is taken into consideration.

Horizontal force P = P '–h 526.75 × 2.800 = 1474.90 kN

Wall Height h = 2.800 m

The dispersion width B = h = 2.80 m is used as the expansion joint side end.

Aabsorbed energy

E = P = 1474.90 kj

Verification of the amount of energy absorbed

It is determined whether the partition (concrete structure) exceeds collision energy by the ship.

Verification of energy

Absorbed Energy	Impact Energy	Evaluation
1474.90 kj	≧ 1,380.000 kj	OK

(d) Calculation Result (Sill)

The stress calculation result of the sill is shown below.

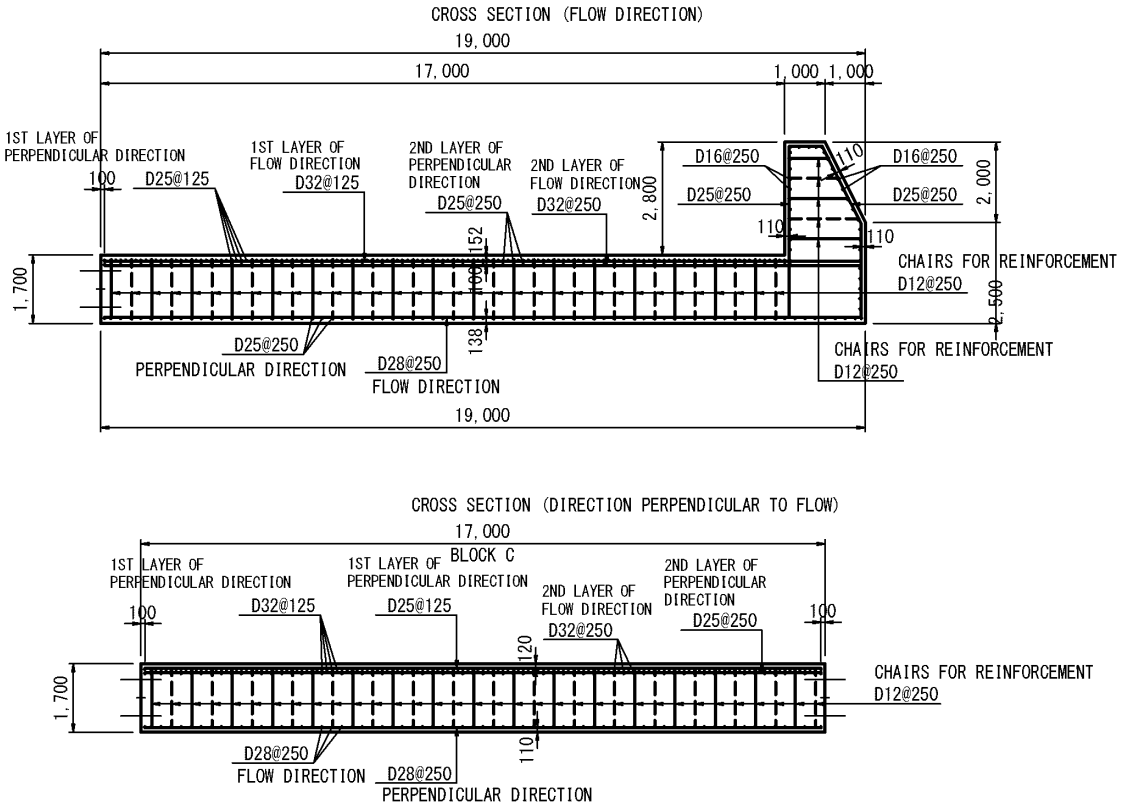
Table 7.3.74 Stress Calculation of the Sill

Member Name	Item			Units	Thickness T = 2.00 m	Thickness T = 2.00 m
	Position				upstream crown	upstream crown
	Load Condition				h = 2.80 m	h = 2.80 m
					flexural failure	flexural failure
Cross-sectional Force	Bending Moment		M	kN m	1474.90	1474.90
	Axial Force		N	kN	0	0
	Shear Stress		S	kN	0	0
Member Cross-section	Member Width		b	mm	1000	1000
	Member Height		h	mm	2000	2000
	Tension Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	4	4
		Amount of Rebar	As	mm2	1963.52	1963.52
		Cover	d'	mm	100	100
	Compression Rebar	Diameter	D	mm	25	25
		Number of Rebar	n	rebar	4	4
		Amount of Rebar	As	mm2	1963.52	1963.52
		Cover	d'	mm	100	100
Stress Intensity	Concrete		σc	N/mm2	6.48	6.48
	Rebar		σs	N/mm2	415.00	415.02
	Shear Force		τ	N/mm2	0.000	0.000
Allowable Stress	Concrete		σca	N/mm2	20.70	20.70
	Rebar		σsa	N/mm2	415.0	415.0
	Shear Force		τa	N/mm2		
Evaluationn	σc ≦ σca				OK	OK
	σs ≦ σsa				OK	NG
	τ ≦ τa					

From above calculation, moment of bending fracture is evaluated as 1474.9kN/m

Source: Study Team

(e) Bar Arrangement Plan



NOTE) THE BAR ARRANGEMENT IS THE SAME WHEN THE LENGTH L=16.3m AND L=6.20m IN THE PERPENDICULAR DIRECTION.

7) Downstream Apron 2

(a) Study Case

The examination cases are normal, construction (no heavy machineries), and during construction (with heavy machineries)".

Table 7.3.75 Loads of Downstream Apron 2

Direction	Study Case	Vertical Load V (kN)	Moment My (kN m)
Direction Perpendicular to Flow	Normal	1355.67	12743.29
	During construction (No crane load)	812.16	7634.30
	During construction (consideration of crane load)	1245.32	11788.81
Flow Direction	Normal	793.21	4362.66
	During construction (No crane load)	475.20	2613.60
	During construction (consideration of crane load)	831.92	4575.56

Source: Study Team

Table 7.3.76 Subgrade Reaction

Direction	Study Case	X = L (m)	qmin (kN/m2)	qmax (kN/m2)
Direction Perpendicular to Flow	Normal	18.800	72.110	72.110
	Flood	18.800	43.200	43.200
	Seismic	18.800	64.845	67.636
Flow Direction	Normal	16.500	72.110	72.110
	Flood	16.500	43.200	43.200
	Seismic	16.500	75.629	75.629

Source: Study Team

(b) Calculation Result

Stress calculation result of downstream Apron2 (L = 18.80m in direction perpendicular to flow), L = 11.0m in flow direction) is shown.

Table 7.3.77 Stress Calculation of the Downstream Apron 2 (Direction Perpendicular to Flow)

Member Name	Item			Units	Thickness T = 1.80 m	Thickness T = 1.80 m	Thickness T = 1.80 m
	Position				top tension	top tension	undersurface tension
	Load Condition				Normal	during construction	during construction
					(Long-term)	(Short-term)	(Short-term)
Cross-sectional Force	Bending Moment		M	kN m	1412.44	2053.05	752.05
	Axial Force		N	kN	0	0	0
	Shear Stress		S	kN	300.52	436.82	183.83
Member Cross-section	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1800	1800	1800
	Tension Rebar	Diameter	D	mm	32	32	28
		Number of Rebar	n	rebar	8	8	4
		Cover	d'	mm	120	120	110
	Compression Rebar	Diameter	D	mm	28	28	32
		Number of Rebar	n	rebar	4	4	8
		Cover	d'	mm	110	110	120
Stress Intensity	Concrete		σc	N/mm2	4.48	6.51	3.30
	Rebar		σs	N/mm2	141.0	205.0	190.2
	Shear Force		τ	N/mm2	0.179	0.260	0.109
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Rebar		σ sa	N/mm2	168.0	252.0	252.0
	Shear Force		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

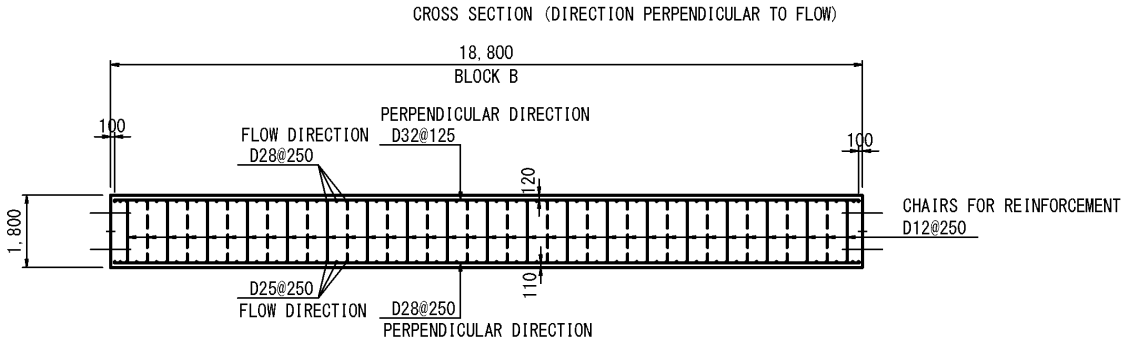
Source: Study Team

Table 7.3.78 Stress Calculation of the Downstream Apron 2 (Flow Direction)

Member Name	Item			Units	Thickness T = 1.80 m	Thickness T = 1.80 m	Thickness T = 1.80 m
	Position				top tension	top tension	undersurface tension
	Load Condition				Normal (Long-term)	during construction (Short-term)	during construction (Short-term)
Cross-sectional Force	Bending Moment		M	kN m	483.55	702.86	491.25
	Axial Force		N	kN	0	0	0
	Shear Stress		S	kN	175.84	255.59	178.5
Member Cross-section	Member Width		b	mm	1000	1000	1000
	Member Height		h	mm	1800	1800	1800
	Tension Rebar	Diameter	D	mm	28	28	25
		Number of Rebar	n	rebar	4	4	4
		Cover	d'	mm	150	150	137
	Compression Rebar	Diameter	D	mm	25	25	28
		Number of Rebar	n	rebar	4	4	4
		Cover	d'	mm	137	137	150
Stress Intensity	Concrete		σc	N/mm2	2.40	3.48	2.66
	Rebar		σs	N/mm2	125.4	182.2	157.8
	Shear Force		τ	N/mm2	0.107	0.155	0.107
Allowable Stress	Concrete		σca	N/mm2	8.28	12.42	12.42
	Rebar		σ sa	N/mm2	168.0	252.0	252.0
	Shear Force		τa	N/mm2	0.360	0.540	0.540
Evaluationn	σc ≦ σca				OK	OK	OK
	σs ≦ σsa				OK	OK	OK
	τ ≦ τa				OK	OK	OK

Source: Study Team

(c) Bar Arrangement Plan

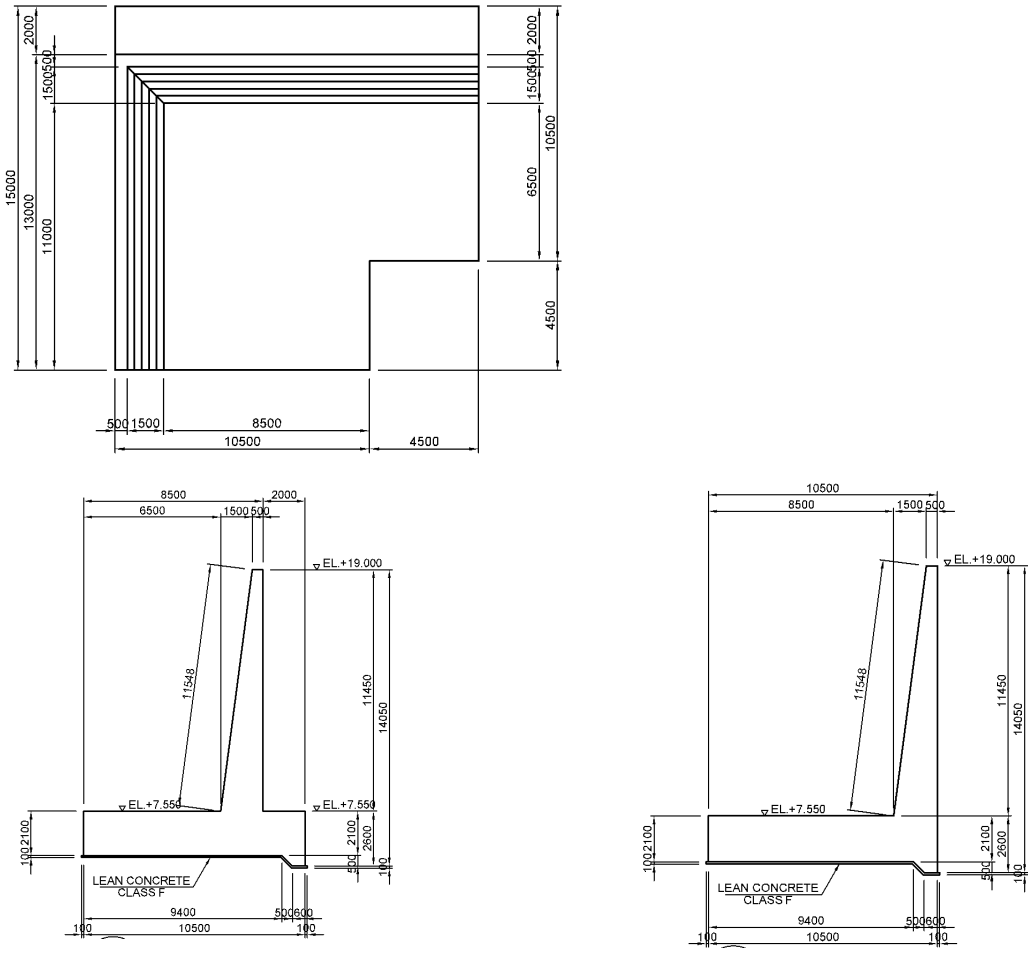


Assembly bar

8) Downstream Wing wall

(a) Study Policy

The cross-sectional direction of the wing wall is considered as the inverse T-type retaining wall, and the flow direction wing wall is considered as the L-type retaining wall.



Source: Study Team

Figure 7.3.22 Structural Dimensions of the Downstream Wing Wall

(b) Calculation Case

The calculation cases are normal, seismic, and construction. Load cases are the same as the stability calculation.

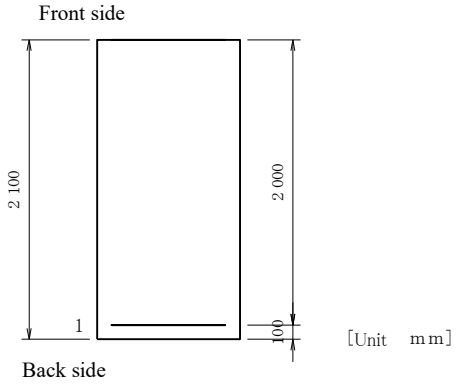
(c) Results of Section Calculation (Inverse T-type Retaining Wall)

The results of cross section design are shown below. Detailed design sheets are attached in the separate piece of structural calculation.

(i) Results of Section Calculation

A. Vertical Wall

[Bar Arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Front	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Back	1	10.0	D40	12.566	8.000	100.528
	2	-	-	-	-	-

Required rebar amount in tension side 98.144 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
				Calculated value Tolerance	Calculated value Tolerance	
Normal (Water Level 1)	2646.666	0.000	64.036	4.627 ≦ 8.280	147.365 ≦ 168.000	○
Seismic (Water Level 2)	3921.740	0.000	64.036	6.856 ≦ 11.010	218.361 ≦ 223.400	○
During construction	2124.291	0.000	64.036	3.714 ≦ 11.010	118.280 ≦ 223.400	○

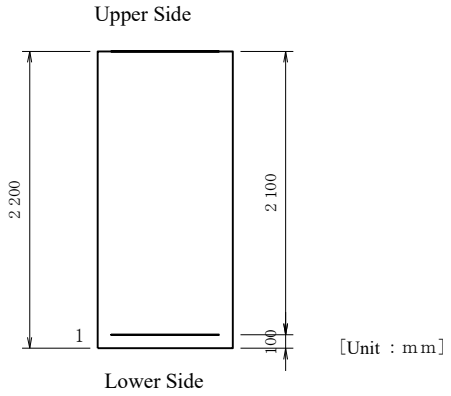
[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)	Judgment
			Calculated value τ Allowable value τa1 Allowable value τa2	
Normal (Water Level 1)	667.632	200.000	0.334 ≦ 0.360 1.700	○
Seismic (Water Level 2)	982.240	200.000	0.491 > 0.470 2.260	×
During construction	512.548	200.000	0.256 ≦ 0.470 2.260	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

B. Bottom Toe

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Underside	1	10.0	D32	8.043	4.000	32.172
	2	-	-	-	-	-

Required rebar amount in tension side 25.339 (cm2)

[Checking of Bending Stress]

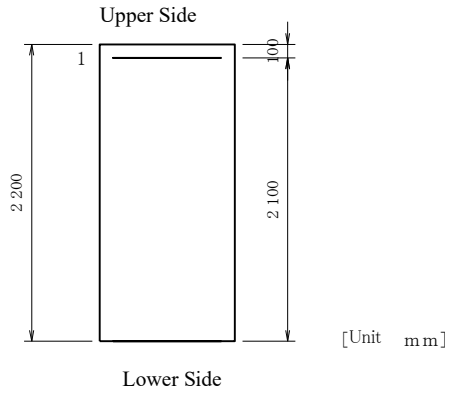
Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
			Calculated value Tolerance	Calculated value tolerance	
Normal (Water Level 1)	460.471	40.444	1.158 ≦ 8.280	72.836 ≦ 168.000	○
Seismic (Water Level 2)	1120.186	40.444	2.818 ≦ 11.010	177.189 ≦ 223.400	○
during construction	367.839	40.444	0.925 ≦ 11.010	58.184 ≦ 223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)	Judgment
			Calculated value τ Allowable value τa1	
Normal (Water Level 1)	208.962	2100.000	0.100 ≦ 0.360	○
Seismic (Water Level 2)	523.844	2100.000	0.249 ≦ 0.470	○
During construction	163.987	2100.000	0.078 ≦ 0.470	○

C. Bottom Heel

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1	10.0	D50	19.635	8.000	157.080
	2	-	-	-	-	-
Underside	1'	-	-	-	-	-
	2 '	-	-	-	-	-

Required rebar amount in tension side 115.144 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	1966.256	78.687	2.721 ≦	8.280	68.104 ≦	168.000	○
Seismic (Water Level 2)	4803.992	78.687	6.647 ≦	11.010	166.392 ≦	223.400	○
During construction	1138.205	78.687	1.575 ≦	11.010	39.423 ≦	223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	Shear Force Sh (kN)	Effective Height d (mm)	Shear Stress (N/mm2)		Judgment
			Calculated value τ	Allowable value τa1	
Normal (Water Level 1)	453.027	2100.000	0.216 ≦	0.360	○
Seismic (Water Level 2)	1214.148	2100.000	0.578 >	0.470	×
During construction	275.582	2100.000	0.131 ≦	0.470	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

(ii) Examination of Shear Rebar

A. Vertical Wall

Calculation of diagonal Tensile Rebar
Downstream Inverse T-type wall
Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed.
The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm2
d: effective height of member section d = 2000 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar σ Sa = 223.4 N/mm2
S: shear force acting on the cross section of the member S = 982240 N
Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 360{,}000 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 622240 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 400.4 \text{ mm2}$$

stirrup D16 = 2 pieces
D16 are arranged at 250 pitches in the axial direction.
As = 4.022 cm > Aw = 4.004 cm OK

B. Bottom Heel

Downstream Inverse T-type Bottom Heel
Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed.
The necessary quantity of tensile rebar is calculated by the following formula.

τ a1: Allowable shear stress of concrete τ a1 = 0.36 N/mm2
d: effective height of member section d = 2100 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σ Sa: Allowable Tensile Stress of diagonal tensile rebar σ Sa = 223.4 N/mm2
S: shear force acting on the cross section of the member S = 1214148 N
Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 378000 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 836148 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

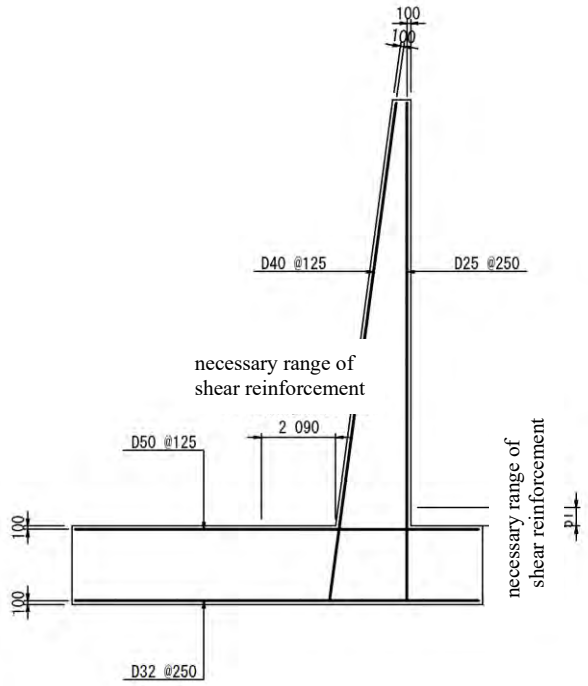
$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 512.4 \text{ mm2}$$

stirrup D16 = 4 pieces

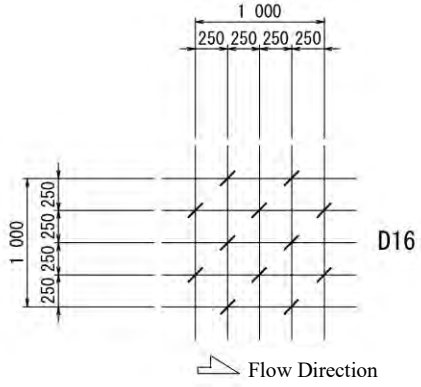
D16 are arranged at 250 pitches in the axial direction.
As = 8.004 cm > Aw = 5.124 cm OK

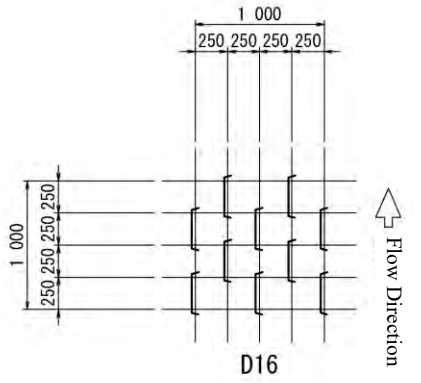
(d) Bar Scheduling (Inverse T-Type Retaining Wall)

(i) Bar Arrangement Plan



(ii) Shear Rebar Arrangement





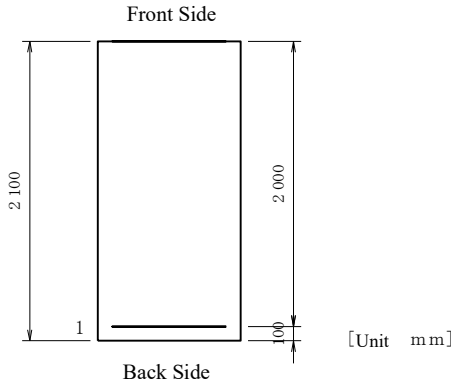
(e) Results of Section Calculation (L-Type Retaining Wall)

The results of cross section design are shown below. Detailed design sheets are attached in the separate piece of structural calculation.

(i) Results of Section Calculation

A. Vertical Wall

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Front	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Back	1	10.0	D40	12.566	8.000	100.528
	2	-	-	-	-	-

Required rebar amount in tension side 98.144 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
				Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	2646.666	0.000	64.036	4.627	≦ 8.280	147.365	≦ 168.000	○
Seismic (Water Level 2)	3921.740	0.000	64.036	6.856	≦ 11.010	218.361	≦ 223.400	○
During construction	2124.291	0.000	64.036	3.714	≦ 11.010	118.280	≦ 223.400	○

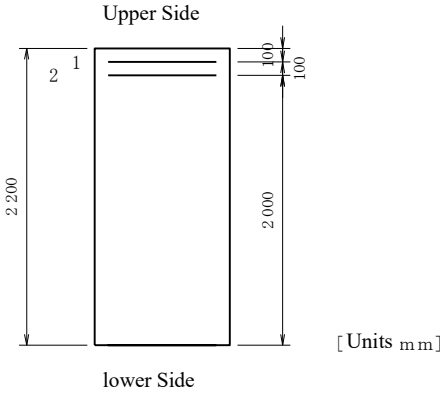
[Check of Shear Stress]

Load Conditions (Water Level)	shear force Sh (kN)	effective height d (cm)	Shear Stress (N/mm2)			Judgment
			Calculated value τ	Allowable value τa1	Allowable value τa2	
Normal (Water Level 1)	667.632	200.000	0.334	> 0.360	1.600	○
Seismic (Water Level 2)	982.240	200.000	0.491	> 0.470	2.130	×
During construction	512.548	200.000	0.256	≦ 0.470	2.130	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

B. Bottom Slab

[Bar Arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1	10.0	D50	19.635	8.000	157.080
	2	20.0	D40	12.566	8.000	100.528
Underside	1'	-	-	-	-	-
	2 '	-	-	-	-	-

Required rebar amount in tension side 220.855 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
			Calculated value Tolerance	Calculated value Tolerance	
Normal (Water Level 1)	3129.030	93.350	3.827 ≦ 8.280	71.730 ≦ 168.000	○
Seismic (Water Level 2)	8445.607	93.350	10.329 ≦ 11.010	193.607 ≦ 223.400	○
During construction	1983.775	93.350	2.426 ≦ 11.010	45.476 ≦ 223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)	Judgment
			Calculated value τ Allowable value τa1	
Normal (Water Level 1)	370.196	2060.976	0.180 ≦ 0.360	○
Seismic (Water Level 2)	1225.021	2060.976	0.594 > 0.470	×
During construction	234.104	2060.976	0.114 ≦ 0.470	○

・In the shear check, since the seismic is not satisfied, the shear rebar is used.

(ii) Examination of Shear Rebar

A. Vertical Wall

Calculation of Oblique Tensile Rebar

downstream L-side wall

seismic

Since shearing force cannot be borne only by concrete, diagonal tension rebar is calculated by the following formula.

τ a1: Allowable shear stress of concrete τ a1 = 0.36 N/mm2

d: effective height of member section d = 2000 mm

b: member cross section width b = 1000 mm

a: axial spacing of diagonal tension rebar a = 250 mm

σ Sa: Allowable Tensile Stress of Oblique Tensile Rebar σ Sa = 223.4 N/mm2

S: shear force acting on the cross section of the member S = 982240 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 360{,}000 \text{ N}$$

Sh ′ : Shear force borne by diagonal tension rebar placed at spacing a

$$Sh' = S - Sc \quad = \quad 622240 \text{ N}$$

Aw: Cross-sectional area of diagonal tension rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 400.4 \text{ mm2}$$

stirrup D16 2 sticks

16 pieces of D are arranged at 250 pitches in the axial direction.

As = 4.022 cm > Aw = 4.004 cm OK

B. Bottom Slab

downstream L-bottom heel

seismic

Since shearing force cannot be borne only by concrete, diagonal tension rebar is calculated by the following formula.

τ a1: Allowable shear stress of concrete τ a1 = 0.36 N/mm2

d: effective height of member section d = 2063 mm

b: member cross section width b = 1000 mm

a: axial spacing of diagonal tension rebar a = 250 mm

σ Sa: Allowable Tensile Stress of Oblique Tensile Rebar σ Sa = 223.4 N/mm2

S: shear force acting on the cross section of the member S = 1225021 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 371340 \text{ N}$$

Sh ′ : Shear force borne by diagonal tension rebar placed at spacing a

$$Sh' = S - Sc \qquad = \quad 853681 \text{ N}$$

Aw: Cross-sectional area of diagonal tension rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \qquad = \quad 532.5 \text{ mm2}$$

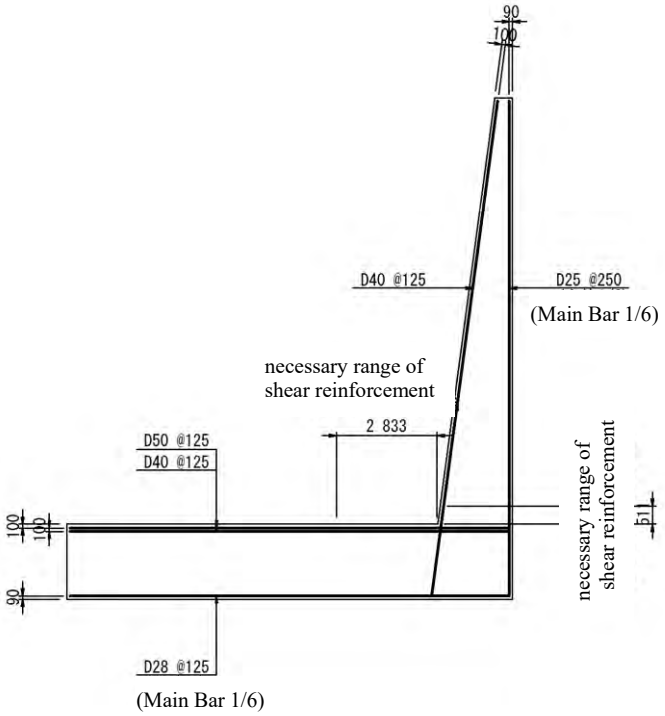
stirrupD16 4 sticks

16 pieces of D are arranged at 250 pitches in the axial direction.

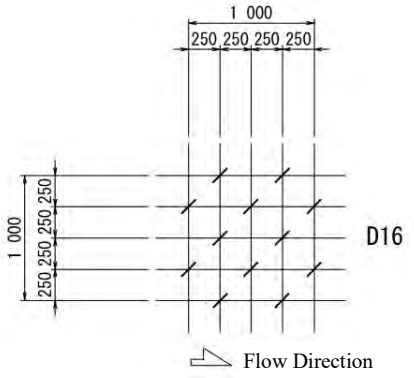
As = 8.044 cm > Aw = 5.325 cm OK

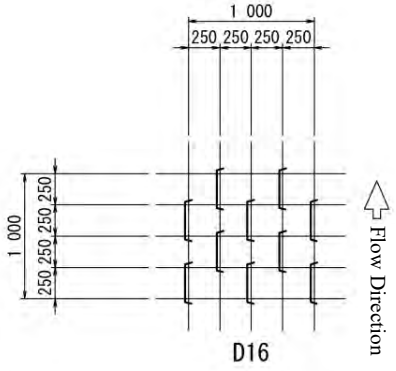
(f) Bar Scheduling (L-type retaining wall)

(i) Bar Arrangement Plan



(ii) Shear Rebar Arrangement

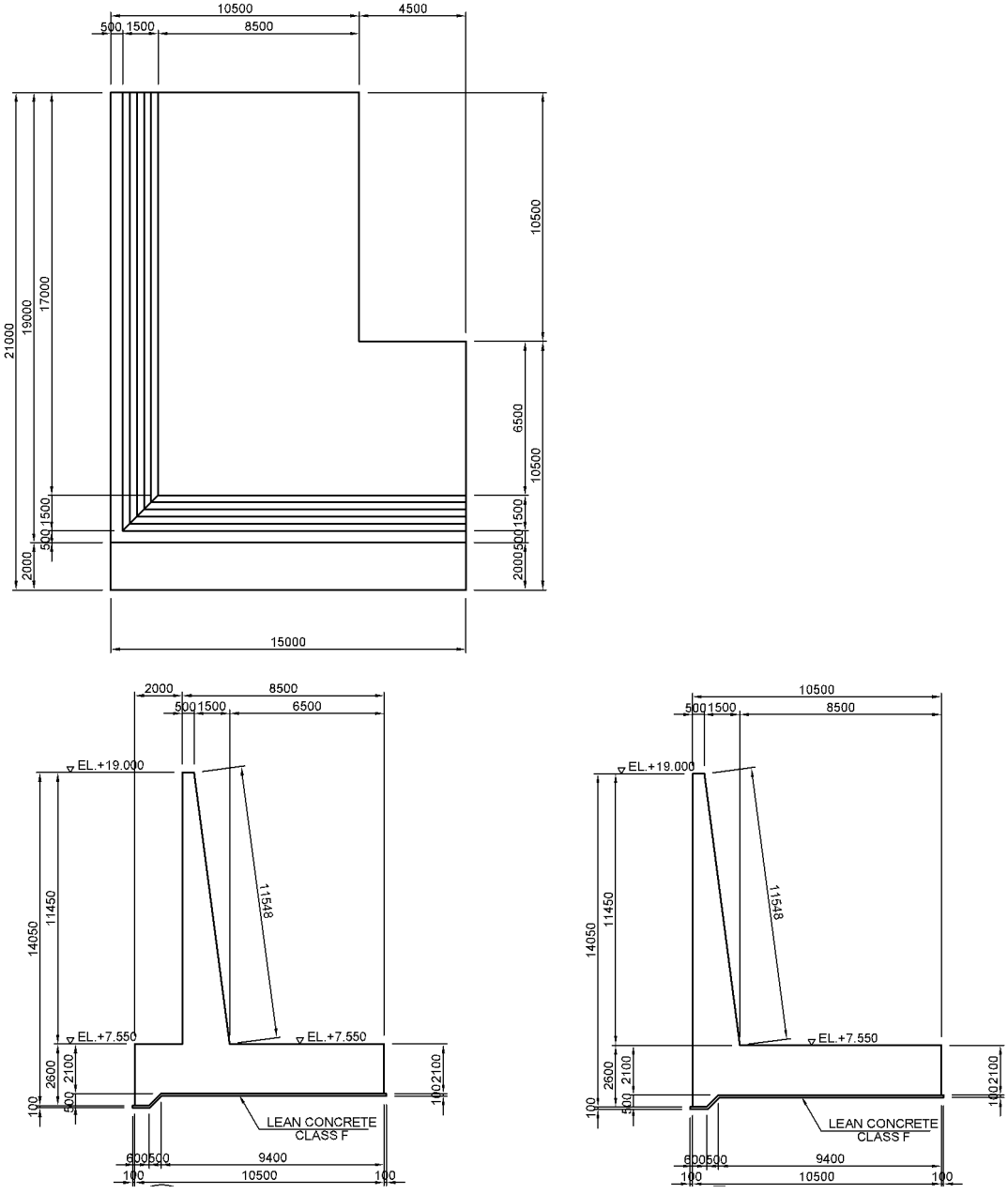




9) Upstream Wing Wall (Right Bank)

(a) Study Policy

The cross-sectional direction of the wing wall is considered as the inverse T-type retaining wall, and the wing wall at the flow direction is considered as the L-type retaining wall.



(Inverse T-Type Retaining Wall) (L-Type Retaining Wall)

Source: Study Team

Figure7.3.23 Structural Dimensions of the Upstream Wing Wall

(b) Calculation Case

The calculation cases are normal, seismic, and construction. Load cases are the same as the stability calculation.

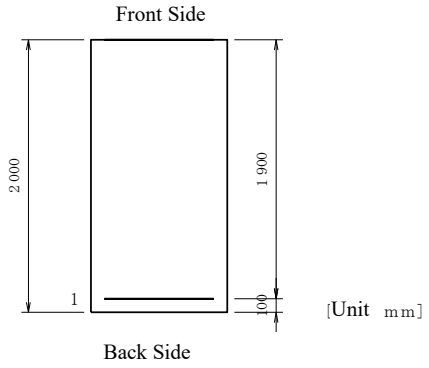
(c) Section Calculation Results (Inverse T-Type Retaining Wall)

The results of cross section design are shown below. Detailed design sheets are attached in the separate piece of structural calculation.

(i) Results of Section Calculation

A. Vertical Wall

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Front	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Back	1	10.0	D40	12.566	8.000	100.528
	2	-	-	-	-	-

Required rebar amount in tension side 93.562 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive Stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
				Calculated value Tolerance	Calculated value Tolerance	
Normal (Water Level 1)	2670.846	0.000	62.109	5.081 ≦ 8.280	156.927 ≦ 168.000	○
Seismic (Water Level 2)	3441.258	0.000	62.109	6.546 ≦ 11.010	202.192 ≦ 223.400	○
During construction	1873.239	0.000	62.109	3.563 ≦ 11.010	110.063 ≦ 223.400	○

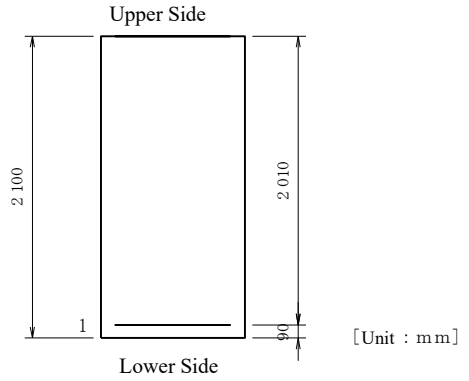
[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)	Judgment
			Calculated value τ Allowable value τa1 Allowable value τa2	
Normal (Water Level 1)	700.037	190.000	0.368 > 0.360 1.700	×
Seismic (Water Level 2)	904.774	190.000	0.476 > 0.470 2.260	×
During construction	470.978	190.000	0.248 ≦ 0.470 2.260	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

B. Bottom Toe

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Underside	1	9.0	D28	6.158	4.000	24.632
	2	-	-	-	-	-

Required rebar amount in tension side 22.515 (cm2)

[Checking of Bending Stress]

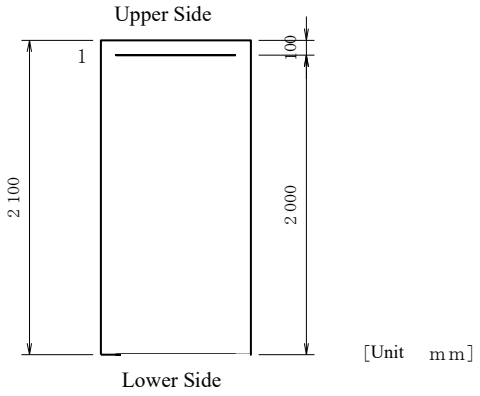
Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
			Calculated value Tolerance	Calculated value Tolerance	
Normal (Water Level 1)	482.692	35.017	1.456 ≦ 8.280	103.508 ≦ 168.000	○
Seismic (Water Level 2)	954.663	35.017	2.879 ≦ 11.010	204.716 ≦ 223.400	○
During construction	336.434	35.017	1.015 ≦ 11.010	72.144 ≦ 223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)	Judgment
			Calculated value τ Allowable value τa1	
Normal (Water Level 1)	231.633	2010.000	0.115 ≦ 0.360	○
Seismic (Water Level 2)	466.964	2010.000	0.232 ≦ 0.470	○
During construction	158.165	2010.000	0.079 ≦ 0.470	○

C. Sole Plate Heel

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1	10.0	D50	19.635	8.000	157.080
	2	-	-	-	-	-
Underside	1'	-	-	-	-	-
	2 '	-	-	-	-	-

Required rebar amount in tension side 114.042 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	2176.631	76.340	3.267 ≦	8.280	79.383 ≦	168.000	○
Seismic (Water Level 2)	4523.467	76.340	6.790 ≦	11.010	164.973 ≦	223.400	○
During construction	1003.951	76.340	1.507 ≦	11.010	36.614 ≦	223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judgment
			Calculated value τ	Allowable value τa1	
Normal (Water Level 1)	493.815	2000.000	0.247 ≦	0.360	○
Seismic (Water Level 2)	1087.237	2000.000	0.544 ≦	0.470	×
During construction	246.202	2000.000	0.123 ≦	0.470	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

(ii) Examination of Shear Rebar

A. Vertical wall

Calculation of diagonal Tensile Rebar

Upstream Inverse T-type wall

Normal

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm^2

d: effective height of member section d = 1900 mm

b: member cross section width b = 1000 mm

a: axial spacing of diagonal tensile rebar a = 250 mm

σSa: Allowable Tensile Stress of diagonal tensile rebar σSa = 168 N/mm2

S: shear force acting on the cross section of the member S = 700037 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 342000 \text{ N}$$

Sh′: Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 358037 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 322.5 \text{ mm2}$$

stirrup D16 2 pieces

D16 are arranged at 250 pitches in the axial direction.

As = 4.022 cm > Aw = 3.225 cm OK

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm2

d: effective height of member section d = 1900 mm

b: member cross section width b = 1000 mm

a: axial spacing of diagonal tensile rebar a = 250 mm

σSa: Allowable Tensile Stress of diagonal tensile rebar σSa = 223.4 N/mm2

S: shear force acting on the cross section of the member S = 904774 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 342000 \text{ N}$$

Sh′: Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 562774 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 381.2 \text{ mm}^2$$

stirrup D16 2 pieces

D16 pieces of D are arranged at 250 pitches in the axial direction.

As = 4.022 cm > Aw = 3.812 cm OK

B. Bottom Heel

Downstream Inverse T-type Bottom Heel

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm^2

d: effective height of member section d = 2000 mm

b: member cross section width b = 1000 mm

a: axial spacing of diagonal tensile rebar a = 250 mm

σ Sa: Allowable Tensile Stress of diagonal tensile rebar σ Sa = 223.4 N/mm^2

S: shear force acting on the cross section of the member S = 1087237 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 360{,}000 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc \quad = \quad 727237 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 468 \text{ mm2}$$

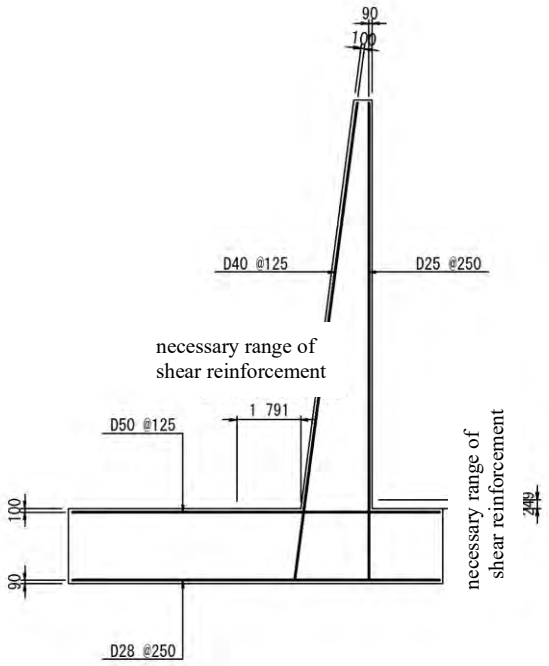
stirrup D16 4 pieces

D16 are arranged at 250 pitches in the axial direction.

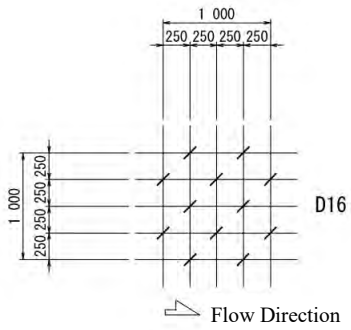
As = 8.044 cm > Aw = 4.680 cm OK

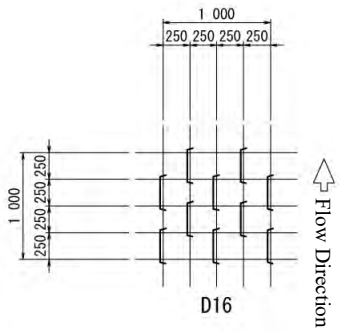
(d) Bar Scheduling (Inverse T-Type Retaining Wall)

(i) Bar Arrangement Plan



(ii) Shear Rebar Arrangement





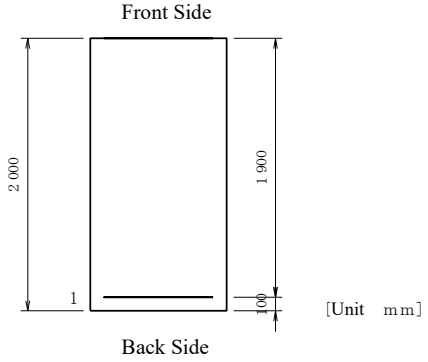
(e) Results of Section Calculation (L-type Retaining Wall)

The results of cross section design are shown below. Detailed design sheets are attached in the separate piece of structural calculation.

(i) Results of Section Calculation

A. Vertical Wall

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Front	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Back	1	10.0	D40	12.566	8.000	100.528
	2	-	-	-	-	-

Required rebar amount in tension side 93.562 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
				Calculated value Tolerance	Calculated value Tolerance	
Normal (Water Level 1)	2670.846	0.000	62.109	5.081 ≦ 8.280	156.927 ≦ 168.000	○
Seismic (Water Level 2)	3441.258	0.000	62.109	6.546 ≦ 11.010	202.192 ≦ 223.400	○
During construction	1873.239	0.000	62.109	3.563 ≦ 11.010	110.063 ≦ 223.400	○

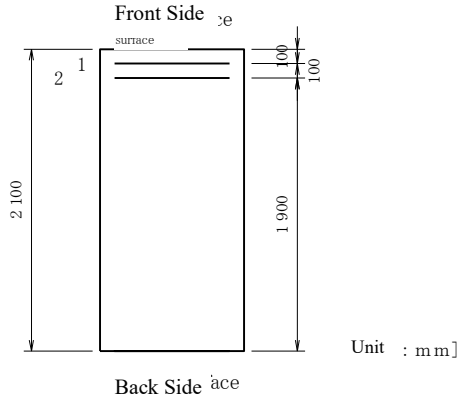
[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Judgment
			Calculated value τ	Allowable value τa1	Allowable value τa2	
Normal (Water Level 1)	700.037	190.000	0.368	> 0.360	1.700	×
Seismic (Water Level 2)	904.774	190.000	0.476	> 0.470	2.130	×
During construction	470.978	190.000	0.248	≦ 0.470	2.130	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

B. Bottom Slab

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1	10.0	D50	19.635	8.000	157.080
	2	20.0	D40	12.566	8.000	100.528
underside	1'	-	-	-	-	-
	2 '	-	-	-	-	-

Required rebar amount in tension side 194.791 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)	Tensile Stress (N/mm2)	Judgment
			Calculated value Tolerance	Calculated value Tolerance	
Normal (Water Level 1)	3331.691	90.388	4.436 ≦ 8.280	80.691 ≦ 168.000	○
Seismic (Water Level 2)	7114.140	90.388	9.472 ≦ 11.010	172.299 ≦ 223.400	○
During construction	1752.945	90.388	2.334 ≦ 11.010	42.455 ≦ 223.400	○

[Check of Shear Stress]

Load Conditions (water level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)	Judgment
			Calculated value τ Allowable value τa1	
Normal (Water Level 1)	442.824	1960.976	0.226 ≦ 0.360	○
Seismic (Water Level 2)	1230.753	1960.976	0.628 ≦ 0.470	×
During construction	232.829	1960.976	0.119 ≦ 0.470	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

(ii) Examination of Shear Rebar

A. Vertical Wall

Calculation of diagonal Tensile Rebar

Upstream L-type wall

Normal

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm^2
d: effective height of member section d = 1900 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar σSa = 168 N/mm2
S: shear force acting on the cross section of the member S = 700037 N
Sc: Shear force borne by concrete

$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d$ = 342000 N

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$Sh' = S - Sc$ = 358037 N

Aw: Cross-sectional area of diagonal tensile rebar with spacing a

$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d}$ = 322.5 mm^2

stirrup D16 2 pieces
D16 are arranged at 250 pitches in the axial direction.
As = 4.022 cm > Aw = 3.225 cm OK

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm2
d: effective height of member section d = 1900 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar σSa = 223.4 N/mm2
S: shear force acting on the cross section of the member S = 904774 N
Sc: Shear force borne by concrete

$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d$ = 342000 N

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$Sh' = S - Sc$ = 562774 N

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d}$ = 381.2 mm^2

stirrup D16 2 pieces
D16 pieces of D are arranged at 250 pitches in the axial direction.
As = 4.022 cm > Aw = 3.812 cm OK

B. Bottom Slab

Upstream L-type Bottom Heel

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm^2
d: effective height of member section d = 1960.976 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σ Sa: Allowable Tensile Stress of diagonal tensile rebar σ Sa = 223.4 N/mm2
S: shear force acting on the cross section of the member S = 1230753 N
Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 352976 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc \quad = \quad 877777 \text{ N}$$

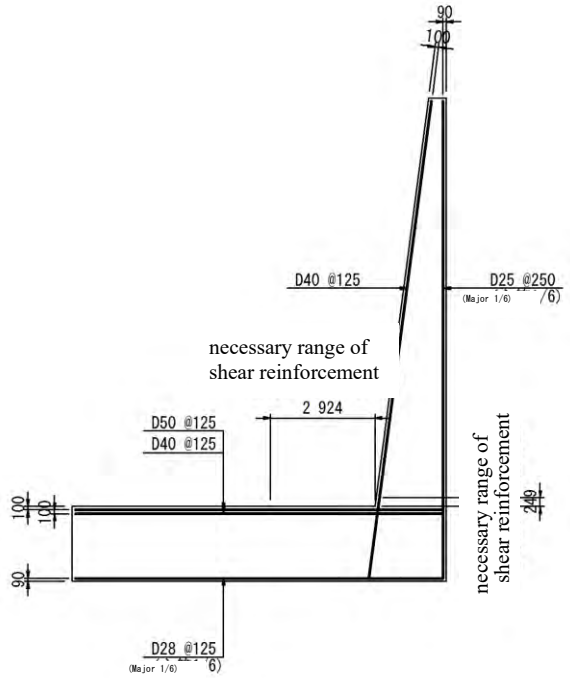
Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 576.1 \text{ mm2}$$

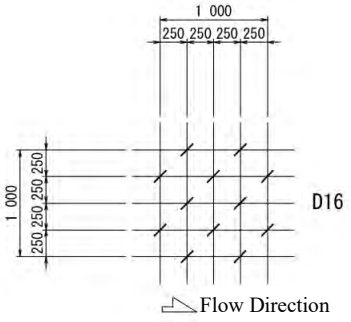
stirrup D16 4 pieces
D16 are arranged at 250 pitches in the axial direction.
As = 8.044 cm > Aw = 5.761 cm OK

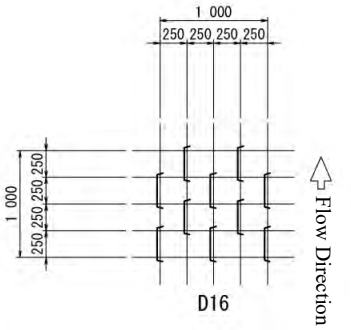
(f) Bar Scheduling (L-Type Retaining Wall)

(i) Bar Arrangement Plan



(ii) Shear Rebar Arrangement

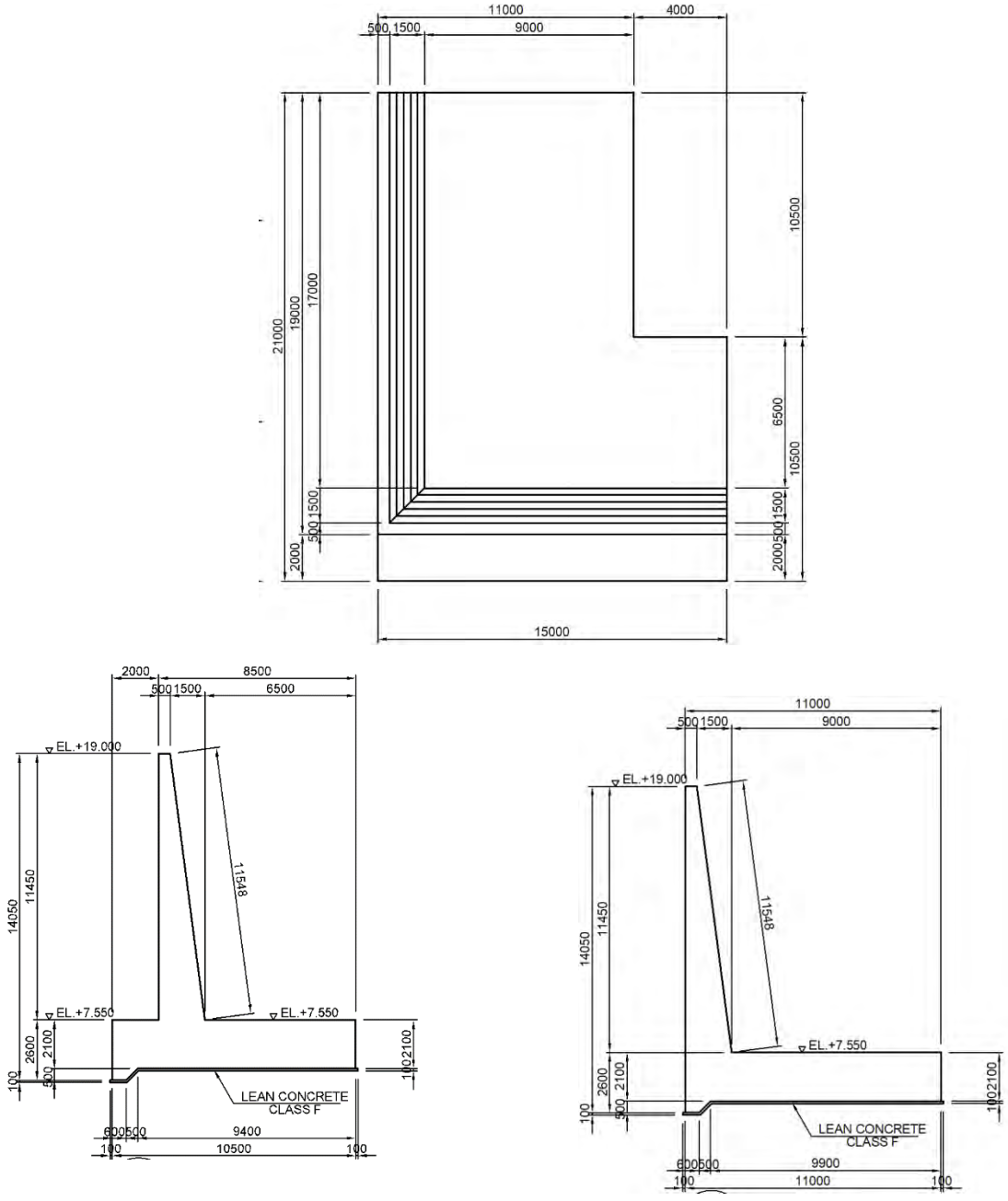




10) Upstream Wing Wall (Left Bank)

(a) Study Policy

The cross-sectional direction of the wing wall is considered as the inverse T-type retaining wall, and the wing wall at the flow direction is considered as the L-type retaining wall.



(Inverse T-Type Retaining Wall) (L-Type Retaining Wall)

Source: Study Team

Figure7.3.24 Structural Dimensions of the Upstream Wing Wall

(b) Calculation Case

The calculation cases are normal, seismic, and construction. Load cases are the same as the stability calculation.

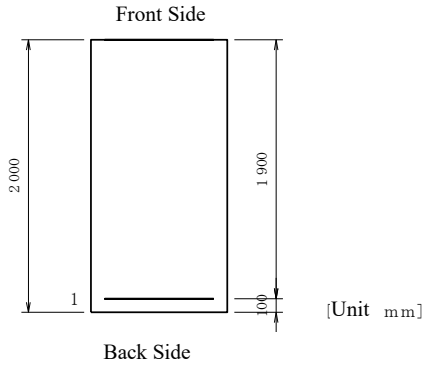
(c) Section Calculation Results (Inverse T-Type Retaining Wall)

The results of cross section design are shown below. Detailed design sheets are attached in the separate piece of structural calculation.

(i) Results of Section Calculation

A. Vertical Wall

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Front	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Back	1	10.0	D40	12.566	8.000	100.528
	2	-	-	-	-	-

Required rebar amount in tension side 93.562 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive Stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
				Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	2670.846	0.000	62.109	5.081	≦ 8.280	156.927	≦ 168.000	○
Seismic (Water Level 2)	3441.258	0.000	62.109	6.546	≦ 11.010	202.192	≦ 223.400	○
During construction	1873.239	0.000	62.109	3.563	≦ 11.010	110.063	≦ 223.400	○

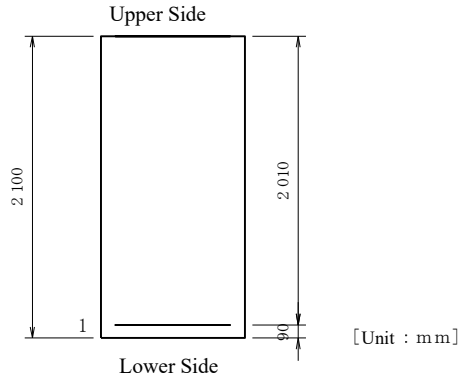
[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)				Judgment
			Calculated value τ		Allowable value τa1	Allowable value τa2	
Normal (Water Level 1)	700.037	190.000	0.368	>	0.360	1.700	×
Seismic (Water Level 2)	904.774	190.000	0.476	>	0.470	2.260	×
During construction	470.978	190.000	0.248	≦	0.470	2.260	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

B. Bottom Toe

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Underside	1	9.0	D32	7.942	4.000	31.768
	2	-	-	-	-	-

Required rebar amount in tension side 24.877 (cm2)

[Checking of Bending Stress]

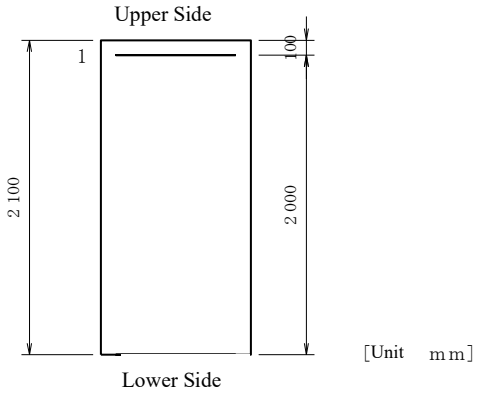
Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	31.418	39.272	1.441 ≦	8.280	89.014 ≦	168.000	○
Seismic (Water Level 2)	1051.835	39.272	2.852 ≦	11.010	176.186 ≦	223.400	○
During construction	385.181	39.272	1.044 ≦	11.010	64.519 ≦	223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judgment
			Calculated value τ	Allowable value τa1	
Normal (Water Level 1)	255.517	2010.000	0.127 ≦	0.360	○
Seismic (Water Level 2)	516.132	2010.000	0.257 ≦	0.470	○
During construction	182.059	2010.000	0.091 ≦	0.470	○

C. Sole Plate Heel

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1	10.0	D50	19.635	8.000	157.080
	2	-	-	-	-	-
Underside	1'	-	-	-	-	-
	2 '	-	-	-	-	-

Required rebar amount in tension side 117.94 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	2319.605	76.340	3.482 ≦	8.280	84.597 ≦	168.000	○
Seismic (Water Level 2)	4669.763	76.340	7.009 ≦	11.010	170.308 ≦	223.400	○
During construction	1147.060	76.340	1.722 ≦	11.010	41.834 ≦	223.400	○

[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judgment
			Calculated value τ	Allowable value τa1	
Normal (Water Level 1)	518.345	2000.000	0.259 ≦	0.360	○
Seismic (Water Level 2)	1153.968	2000.000	0.577 ＞	0.470	×
During construction	270.759	2000.000	0.135 ≦	0.470	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

(ii) Examination of Shear Rebar

A. Vertical wall

Calculation of diagonal Tensile Rebar

Upstream Inverse T-type wall

Normal

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete	τa1 =	0.36 N/mm^2
d: effective height of member section	d =	1900 mm
b: member cross section width	b =	1000 mm
a: axial spacing of diagonal tensile rebar	a =	250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar	σSa =	168 N/mm2
S: shear force acting on the cross section of the member	S =	700037 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 342000 \text{ N}$$

Sh′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc \quad = \quad 358037 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 322.5 \text{ mm2}$$

stirrup D16 2 pieces

D16 are arranged at 250 pitches in the axial direction.

As = 4.022 cm > Aw = 3.225 cm OK

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete	τa1 =	0.36 N/mm2
d: effective height of member section	d =	1900 mm
b: member cross section width	b =	1000 mm
a: axial spacing of diagonal tensile rebar	a =	250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar	σSa =	223.4 N/mm2
S: shear force acting on the cross section of the member	S =	904774 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 342000 \text{ N}$$

Sh′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc \quad = \quad 562774 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 381.2 \text{ mm}^2$$

stirrup D16 2 pieces

D16 pieces of D are arranged at 250 pitches in the axial direction.

As = 4.022 cm > Aw = 3.812 cm OK

B. Bottom Heel

Downstream Inverse T-type Bottom Heel

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm^2

d: effective height of member section d = 2000 mm

b: member cross section width b = 1000 mm

a: axial spacing of diagonal tensile rebar a = 250 mm

σ Sa: Allowable Tensile Stress of diagonal tensile rebar σ Sa = 223.4 N/mm^2

S: shear force acting on the cross section of the member S = 1153969 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d \quad = \quad 360{,}000 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc \quad = \quad 793969 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} \quad = \quad 510.9 \text{ mm2}$$

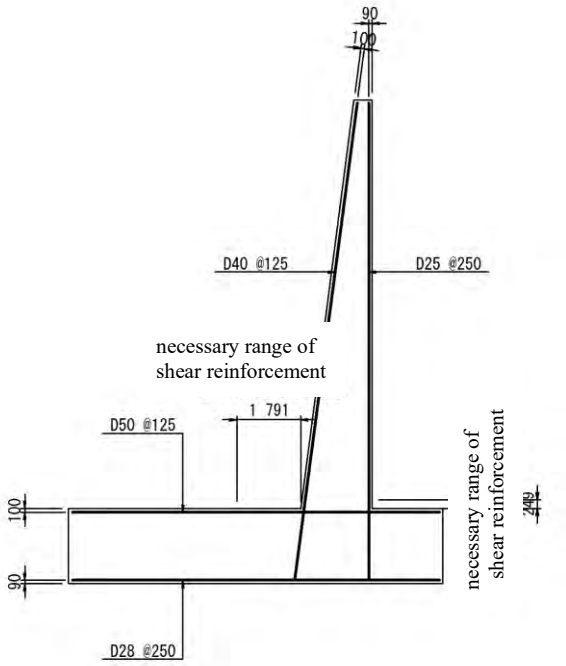
stirrup D16 4 pieces

D16 are arranged at 250 pitches in the axial direction.

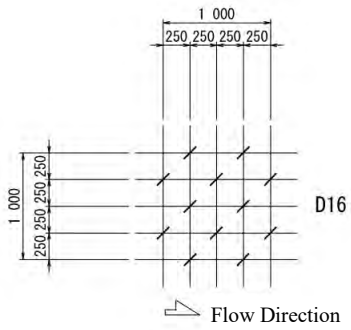
As = 8.044 cm > Aw = 4.680 cm OK

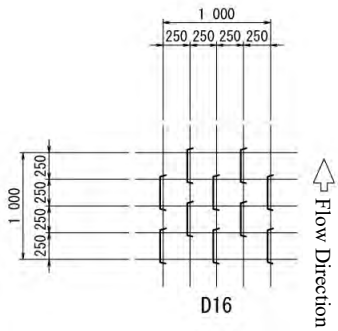
(d) Bar Scheduling (Inverse T-Type Retaining Wall)

(i) Bar Arrangement Plan



(ii) Shear Rebar Arrangement





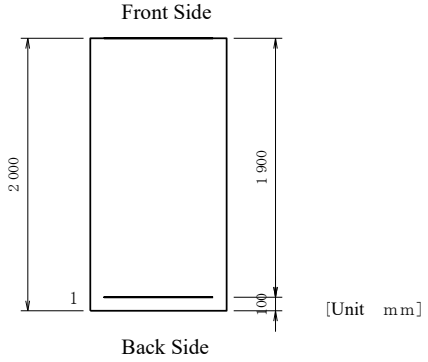
(e) Results of Section Calculation (L-type Retaining Wall)

The results of cross section design are shown below. Detailed design sheets are attached in the separate piece of structural calculation.

(i) Results of Section Calculation

A. Vertical Wall

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Front	1'	-	-	-	-	-
	2 '	-	-	-	-	-
Back	1	10.0	D50	20.270	8.000	162.160
	2	-	-	-	-	-

Required rebar amount in tension side 93.562 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
				Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	3351.881	0.000	74.854	5.427 ≦	8.280	125.227 ≦	168.000	○
Seismic (Water Level 2)	3954.629	0.000	74.854	6.403 ≦	11.010	147.746 ≦	223.400	○
During construction	2554.252	0.000	74.854	4.136 ≦	11.010	95.428 ≦	223.400	○

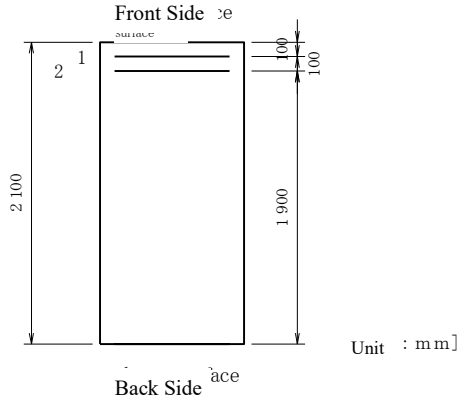
[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Judgment
			Calculated value τ	Allowable value τa1	Allowable value τa2	
Normal (Water Level 1)	818.996	190.000	0.431 >	0.360	1.600	×
Seismic (Water Level 2)	994.109	190.000	0.523 >	0.470	2.130	×
During construction	589.937	190.000	0.310 ≦	0.470	2.130	○

In the shear stress check, allowable stress is not satisfied under the seismic condition. Thus, the shear rebar shall be applied.

B. Bottom Slab

[Bar arrangement]



Single Rebar

Location		Cover (cm)	Rebar Diameter	Rebar Area (cm2/piece)	Number of Pieces	Amount of Rebar (cm2)
Top	1	10.0	D50	20.270	8.000	157.080
	2	20.0	D50	20.270	8.000	157.080
underside	1'	-	-	-	-	-
	2 '	-	-	-	-	-

Required rebar amount in tension side 288.12 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal (Water Level 1)	5070.432	96.387	6.448 ≦	8.280	103.974 ≦	168.000	○
Seismic (Water Level 2)	8453.393	96.387	10.750 ≦	11.010	173.344 ≦	223.400	○
During construction	3465.530	96.387	4.407 ≦	11.010	71.064 ≦	223.400	○

[Check of Shear Stress]

Load Conditions (water level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judgment
			Calculated value τ	Allowable value τa1	
Normal (Water Level 1)	711.009	1950.000	0.365 ＞	0.360	×
Seismic (Water Level 2)	1347.748	1950.000	0.691 ＞	0.470	×
During construction	514.084	1950.000	0.264 ≦	0.470	○

In the shear stress check, allowable stress is not satisfied under the normal and seismic conditions. Thus, the shear rebar shall be applied.

(ii) Examination of Shear Rebar

A. Vertical Wall

Calculation of diagonal Tensile Rebar

Upstream L-type wall

Normal

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm^2
d: effective height of member section d = 1900 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar σSa = 168 N/mm2
S: shear force acting on the cross section of the member S = 994109 N
Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 342000 \text{ N}$$

Sh′: Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 652109 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 587.3 \text{ mm}^2$$

stirrup D20 2 pieces
D20 are arranged at 250 pitches in the axial direction.
As = 6.284 cm > Aw = 4.296 cm OK

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete τa1 = 0.36 N/mm2
d: effective height of member section d = 1900 mm
b: member cross section width b = 1000 mm
a: axial spacing of diagonal tensile rebar a = 250 mm
σSa: Allowable Tensile Stress of diagonal tensile rebar σSa = 223.4 N/mm2
S: shear force acting on the cross section of the member S = 904774 N
Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 342000 \text{ N}$$

Sh′: Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 562774 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 381.2 \text{ mm}^2$$

stirrup D16 2 pieces
D16 pieces of D are arranged at 250 pitches in the axial direction.
As = 4.022 cm > Aw = 3.812 cm OK

B. Bottom Slab

Upstream L-type Bottom Heel

Normal

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed.

τa1: Allowable shear stress of concrete	τa1 =	0.36 N/mm^2
d: effective height of member section	d =	1960.976 mm
b: member cross section width	b =	1000 mm
a: axial spacing of diagonal tensile rebar	a =	250 mm
σ Sa: Allowable Tensile Stress of diagonal tensile rebar	σ Sa =	168.0 N/mm2
S: shear force acting on the cross section of the member	S =	994109 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 352976 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 641133 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 559.5 \text{ mm2}$$

stirrup D16 4 pieces

D16 are arranged at 250 pitches in the axial direction.

As = 8.044 cm > Aw = 5.761 cm OK

Seismic

Since shear stress cannot be handled only by concrete, diagonal tensile rebar shall be installed. The necessary quantity of tensile rebar is calculated by the following formula.

τa1: Allowable shear stress of concrete	τa1 =	0.36 N/mm^2
d: effective height of member section	d =	1960.976 mm
b: member cross section width	b =	1000 mm
a: axial spacing of diagonal tensile rebar	a =	250 mm
σ Sa: Allowable Tensile Stress of diagonal tensile rebar	σ Sa =	223.4 N/mm2
S: shear force acting on the cross section of the member	S =	1347748 N

Sc: Shear force borne by concrete

$$Sc = \frac{1}{2} \cdot \tau a1 \cdot b \cdot d = 352976 \text{ N}$$

Sh ′ : Shear force borne by diagonal tensile rebar placed at spacing a

$$Sh' = S - Sc = 994772 \text{ N}$$

Aw: Cross-sectional area of diagonal tensile rebar reinforced with spacing a

$$Aw = \frac{1.15 \cdot Sh' \cdot a}{\sigma Sa \cdot d} = 652.8 \text{ mm2}$$

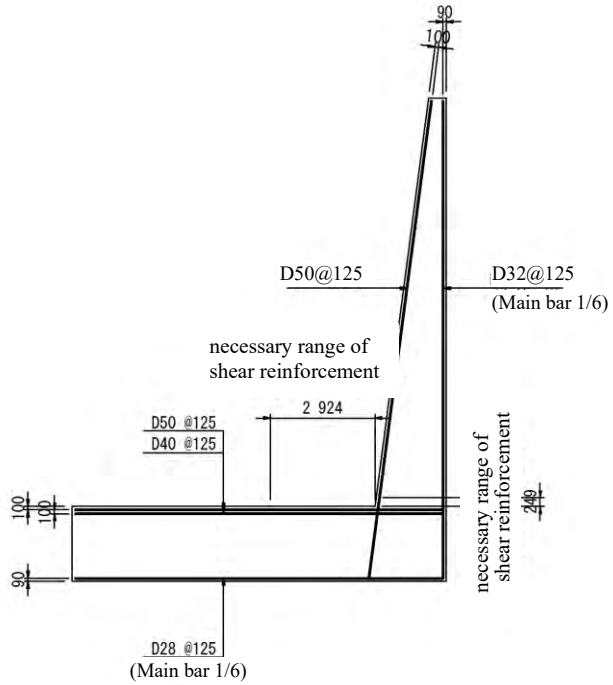
stirrup D16 4 pieces

D16 are arranged at 250 pitches in the axial direction.

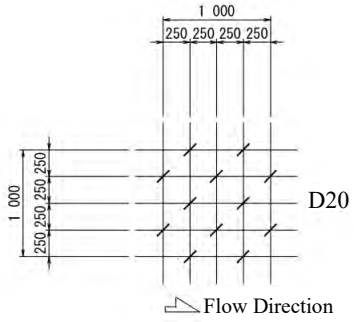
As = 8.044 cm > Aw = 5.761 cm OK

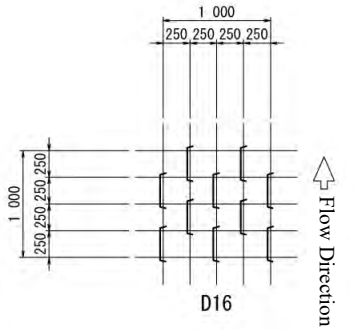
(f) Bar Scheduling (L-Type Retaining Wall)

(i) Bar Arrangement Plan



(ii) Shear Rebar Arrangement





7.3.2.4 L2 Seismic Design of the MCGS Main Body

(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.

The BSDS describes the term of Level 1 earthquake motion and Level 2 earthquake motion as follows. In comparison to the Japanese seismic methodologies, they have common objectives.

"*Level 1* Earthquake *Ground Motion - The design earthquake considering seismic hazard from small to moderate earthquake with high probability of occurrence during the bridge service life (taken as 100-year return), for seismic serviceability design objective to ensure normal bridge functions."*

"Level 2 Earthquake *Ground Motion - The design earthquake considering a seismic hazard corresponding to an earthquake with a return period of 1000 years (seven percent probability of exceedance in 75 years), for life safety performance objective under a 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 (**Table 7.3.79**).

Table 7.3.79 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
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 **Table 7.3.80**. 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.

Table 7.3.80 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) Design Water Level

The water level at the time of the L2 seismic uses the water level at the time of low water level of the structural calculation. A summary of design water level is shown in **Table 7.3.81**.

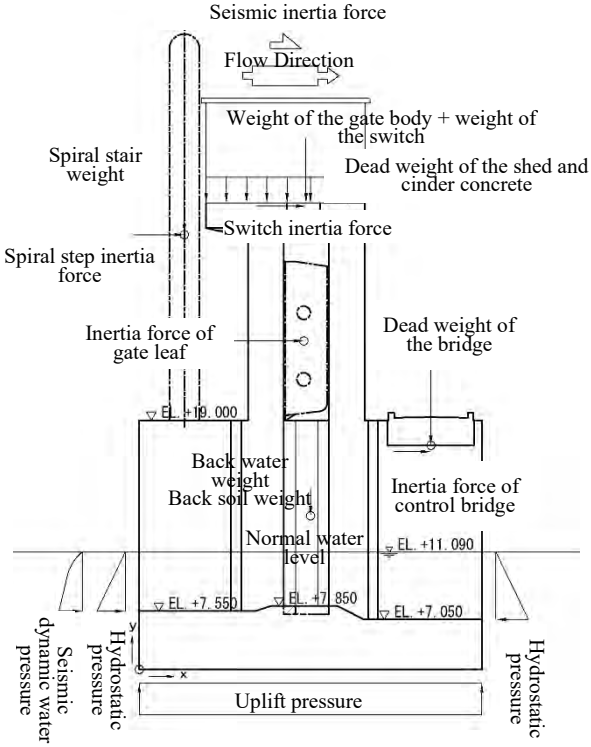
Table 7.3.81 Design Water Level for Level 2 Seismic Assessment

Item	Properties	Remarks
Design Water Level	(Low Water Level) Upstream: EL. 17.400 m Downstream: EL. 11.003 m	- Downstream water level is calculated from the observed water level of Rosario dam on the Marikina River side.

Source: Study Team

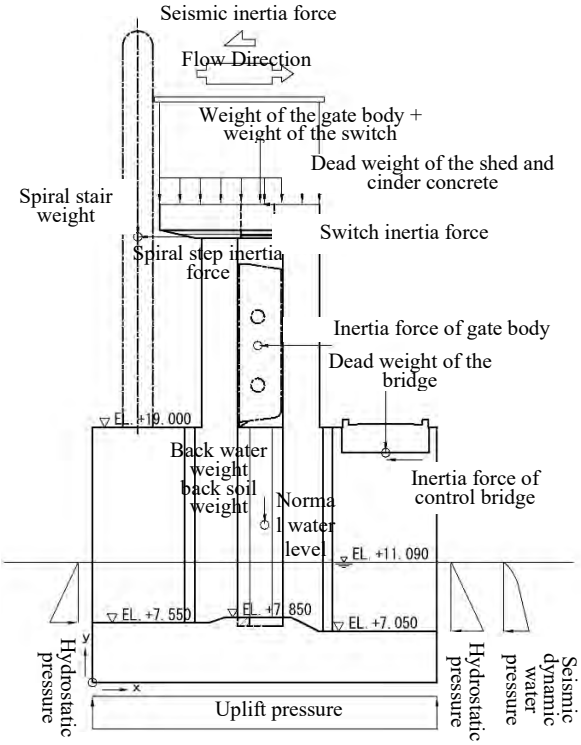
3) Load Conditions

Load diagrams of flow direction are illustrated in **Figure 7.3.25** and **Figure 7.3.22**.



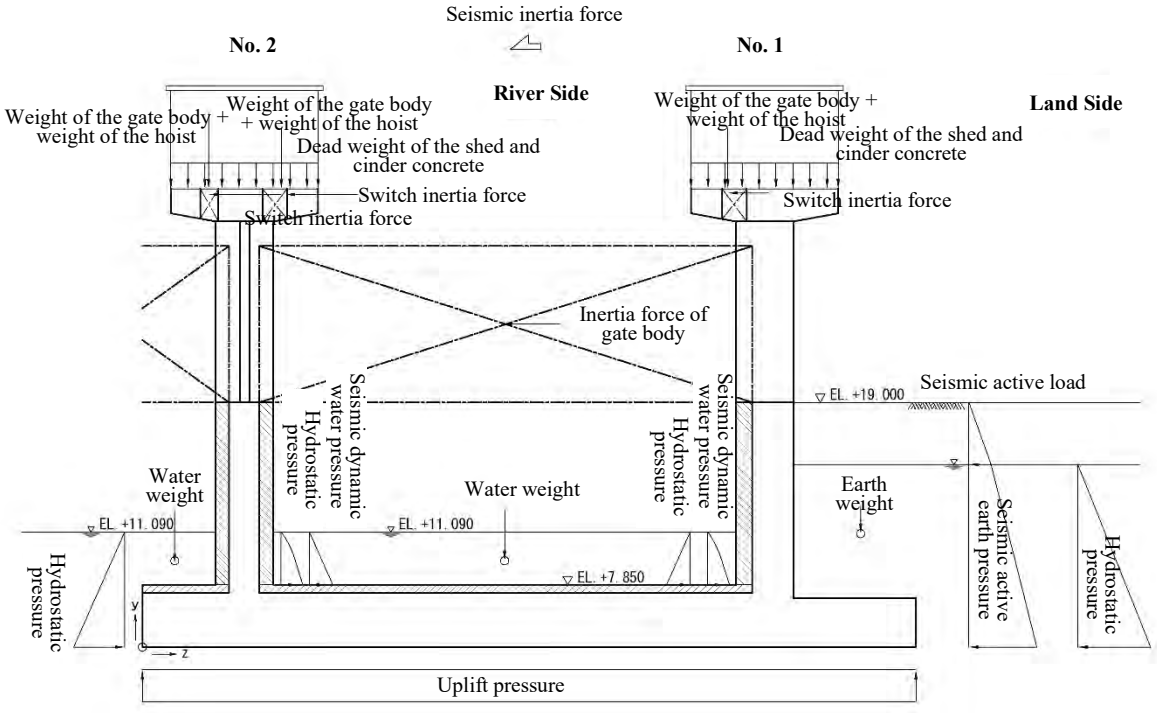
Source: Study Team

Figure 7.3.25 Load Diagram of Flow Direction (Load from Upstream to Downstream)



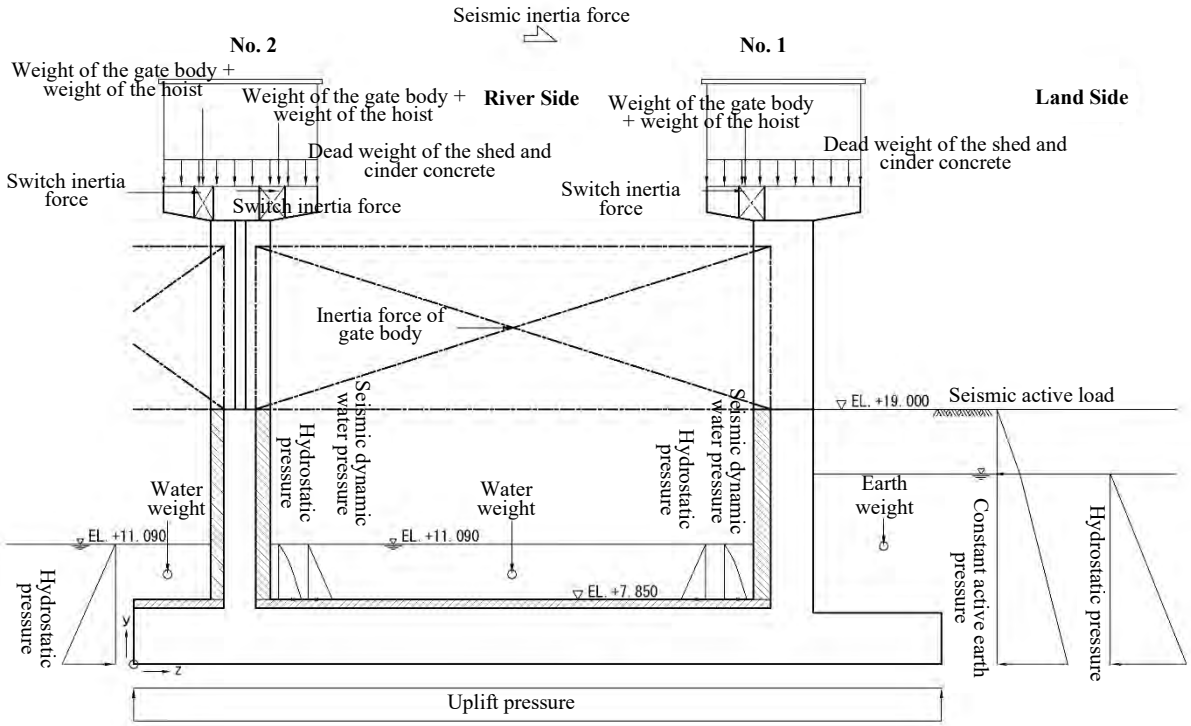
Source: Study Team

Figure 7.3.26 Load Diagram of Flow Direction (Load from Downstream to Upstream)



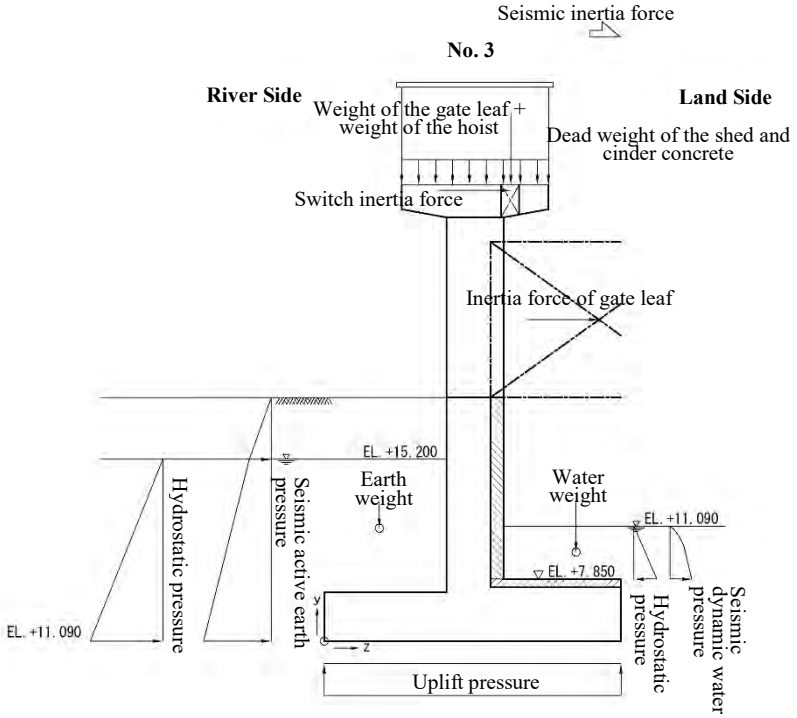
Source: Study Team

Figure 7.3.27 Load Diagram of Flow Direction in the No.1 and No.2 Piers (Load from No.1 Pier to No.2 Pier)



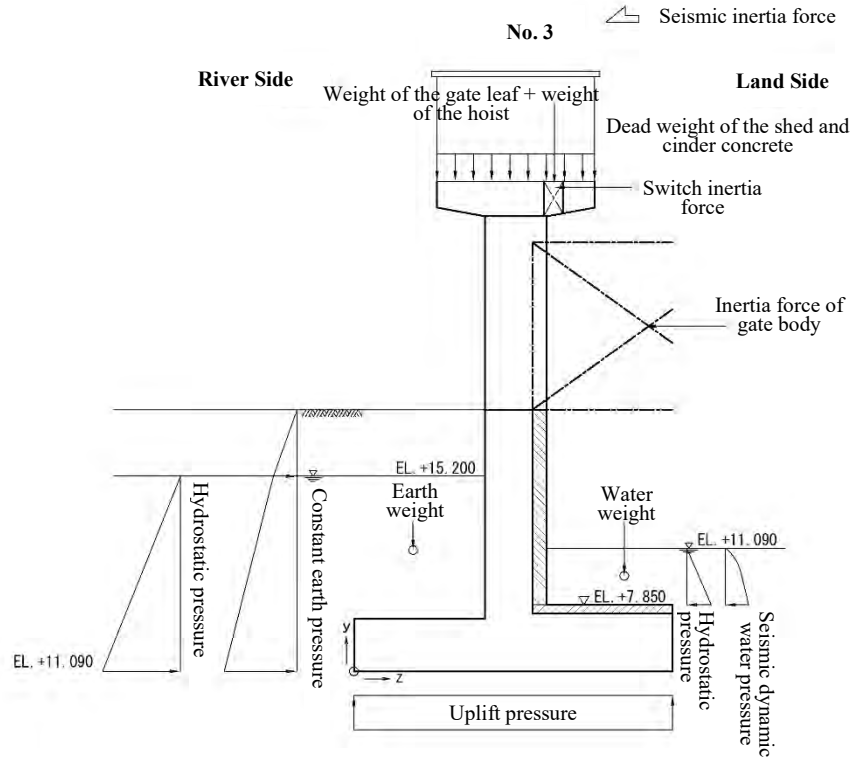
Source: Study Team

Figure 7.3.28 Load Diagram of Direction Perpendicular to Flow in the No.1 and No.2 Piers (Load from No.2 Pier to No.1 Pier)



Source: Study Team

Figure 7.3.29 Load Diagram of Perpendicular direction of Flow in No. 3 Pier (Load from River to Land)

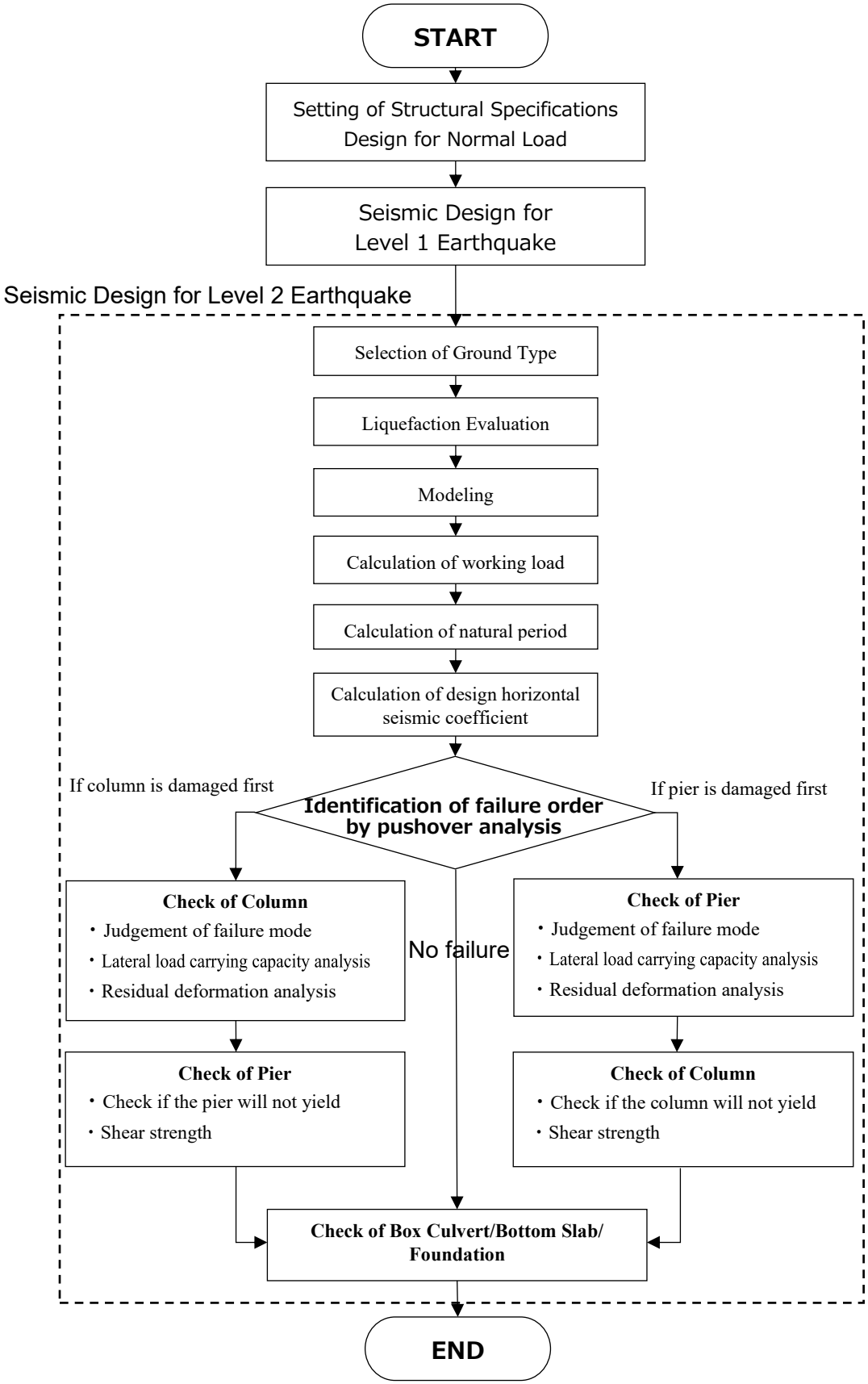


Source: Study Team

Figure 7.3.30 Load Diagram of Perpendicular direction of Flow in No. 3 Pier (Load from River to Land)

(3) Analysis Method

The seismic analysis method is summarized in **Figure 7.3.31**.



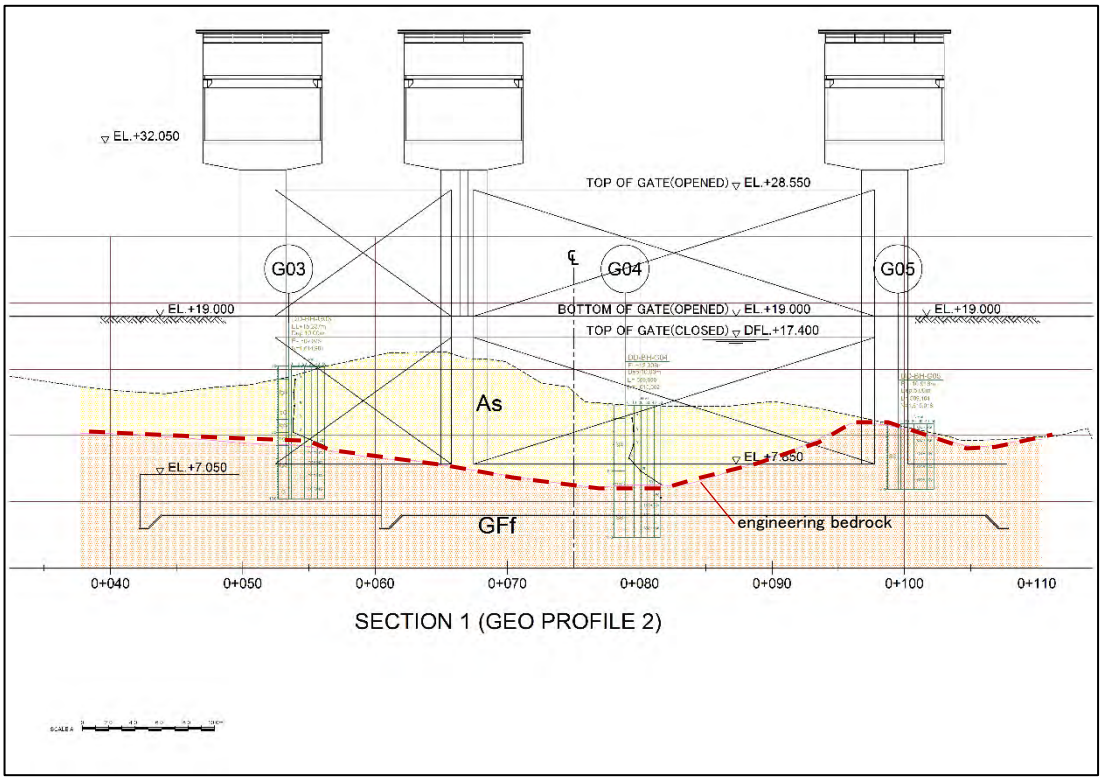
Source: Study Team

Figure 7.3.31 Flow of Seismic Analysis

(4) Contents of The Analysis

1) Selection of Ground Type

To the MCGS geological profile **Figure 7.3.32** See. The engineering substrate surface is the upper surface of the GF layer having an N value of 50.



Source: Study Team

Figure 7.3.32 General Drawing with Ground Conditions

The ground type was assessed based on the following description in the BSDS.

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

Where,

V_{si} : Primary Shear-Wave Velocity (sandy soil $V_s=80N^{1/3}$, clayey soil $V_s=100N^{1/3}$)
N : N Value of each soil layer
i : i th soil layer number

Table 7.3.82 Ground Type

Ground Type	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 (DPWH) / Performance Based Seismic Design Criteria for River Structures

The result of the ground characteristic value T_G in DD-BH-G04 (middle of the supposed MCGS allocation) are shown in **Table 7.3.83**. The ground at the MCGS site can be judged as type I ground, considering that it can be assumed that bedrock appears directly under the foundation of the majority part of MCGS.

Table 7.3.83 Calculation of Ground Characteristic value T_G (DD-BH-G04)

DD-BH-G04

Soil No.	Soil Thickness	Depth	Soil	Average N Value	Sand Layer	Clayey Layer	Vsi(m/s)	Hi/Vsi
1	2.00	2.00	As	7.0	○			0.027
2	3.00	5.00	As	6.7	○			0.012
3	2.00	7.00	As	35.0	○			0.006
4	3.00	10.00	GFf	50.0	○			
						Σ (Hi/Vsi)		0.045
						4* Σ (Hi/Vsi)		0.179
						Ground Type		Type I

2) Liquefaction Assessment

Liquefaction assessment 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.

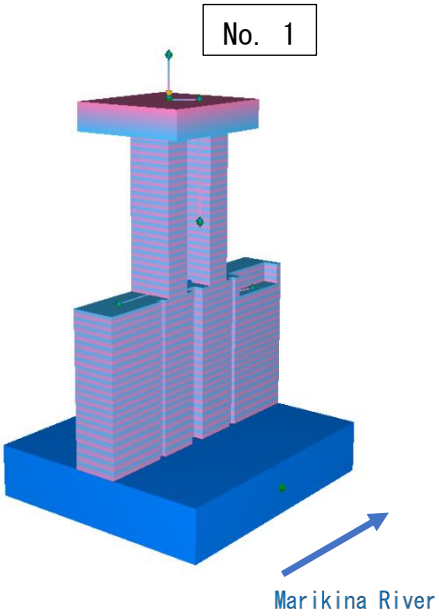
Source: BSDS, DPWH, P6 -3, provisional translation by the Mission

Figure 7.3.32 shows the assumed geological profile at the MCGS. Since the foundation ground was touched with the bedrock (GF layer), it is not necessary to conduct a liquefaction assessment.

3) Modelling

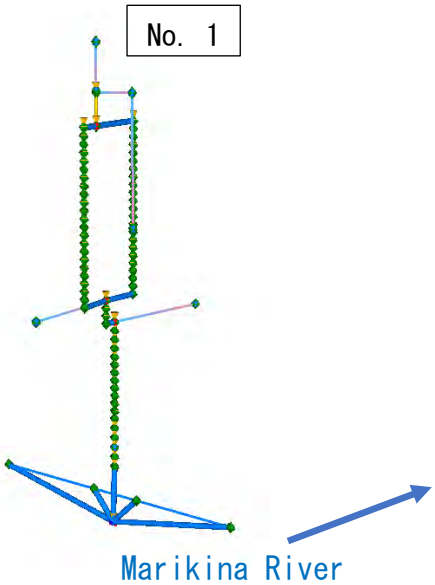
(a) Modeling of Column and Pier

Two types of analytical models for columns and piers were developed: No. 1 pier, and No. 2 to No. 3 piers. In each model, a three-dimensional beam element model considering nonlinearity is made. Model diagrams expressing beam elements in a solid form and model diagrams expressing a frame is displayed in **Figure 7.3.33** to **Figure 7.3.36**.



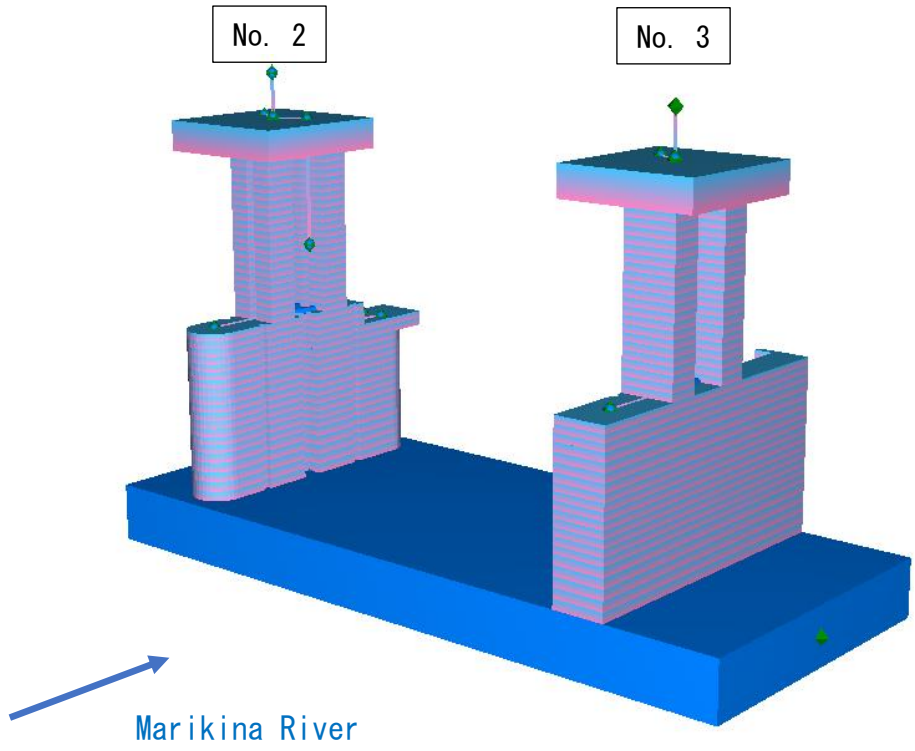
Source: Study Team

Figure 7.3.33 Analytical Model Diagram of No. 1 Pier (Solid Elements)



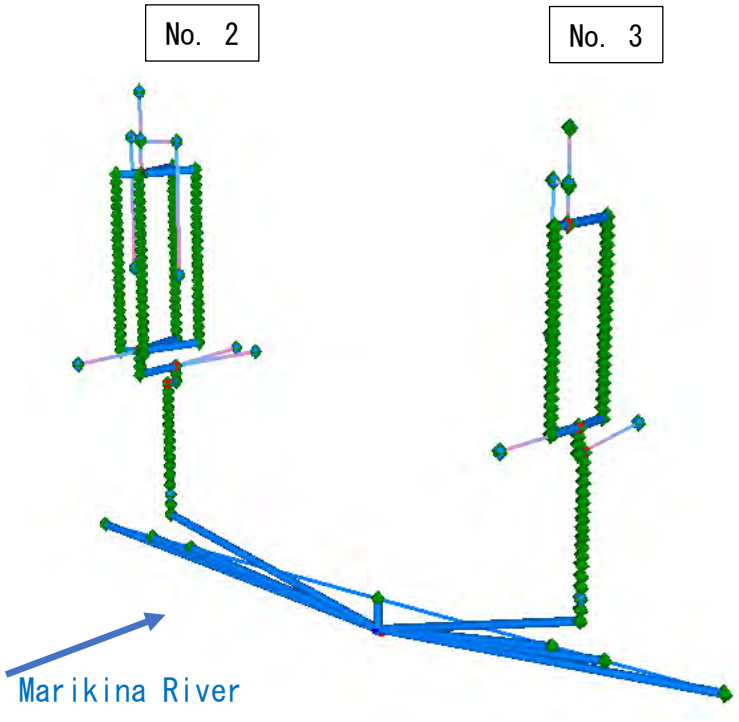
Source: Study Team

Figure 7.3.34 Analytical Model Diagram of No. 1 Pier (Skeleton)



Source: Study Team

Figure 7.3.35 Analytical Model Diagram of No. 2 to No. 3 Piers (Solid Elements)



Source: Study Team

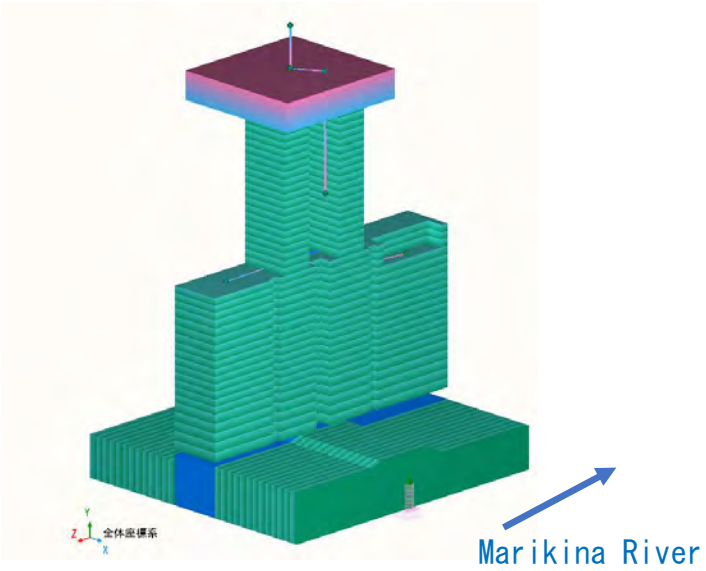
Figure 7.3.36 Analytical Model Diagram of No. 2 to No. 3 Piers (Skeleton)

(b) Analytical Model of Bottom Slab

Similar to the column and piers, two analytical models of the Bottom Slab, for that of No. 1 pier, and No. 2 to No. 3 piers were made. Distribution springs are installed at the below the Bottom Slab, and in order to calculate the load acting on the Bottom Slab, the upper column and pier were

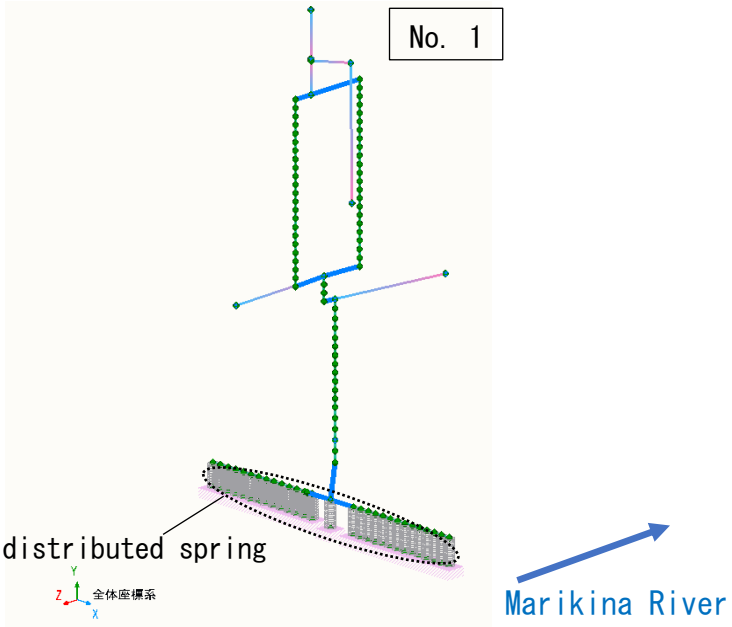
also integrated into the models in the same way as the analytical model of the column and pier. Model diagrams expressing beam elements in a solid form and model diagrams expressing a frame is displayed in **Figure 7.3.37** to **Figure 7.3.38**.

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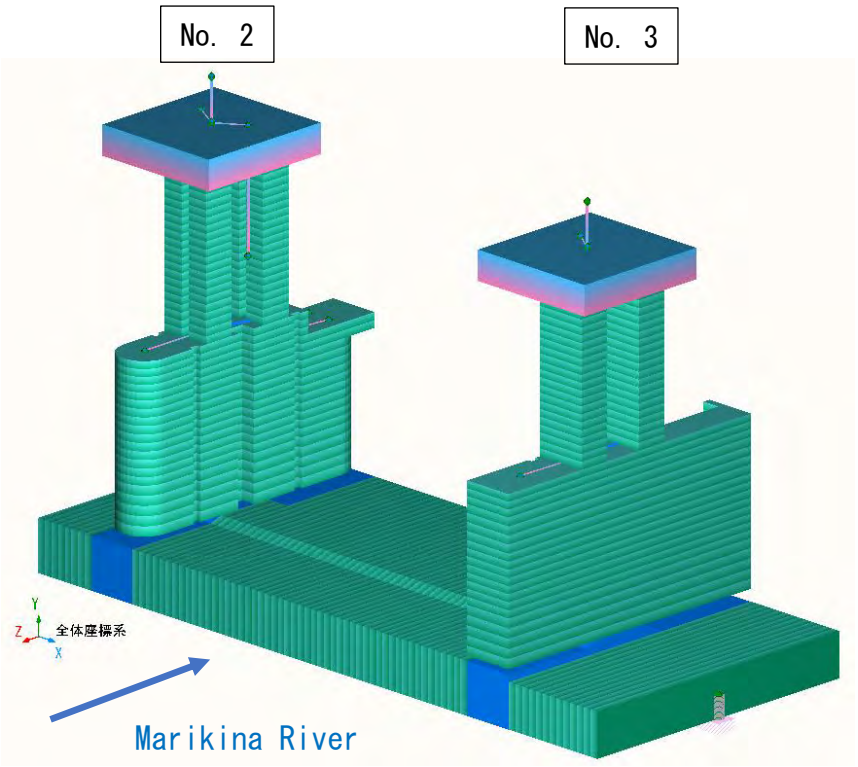
Source: Study Team

Figure 7.3.37 Analytical Model Diagram of No. 1 Pier Bottom Slab (Solid Elements)



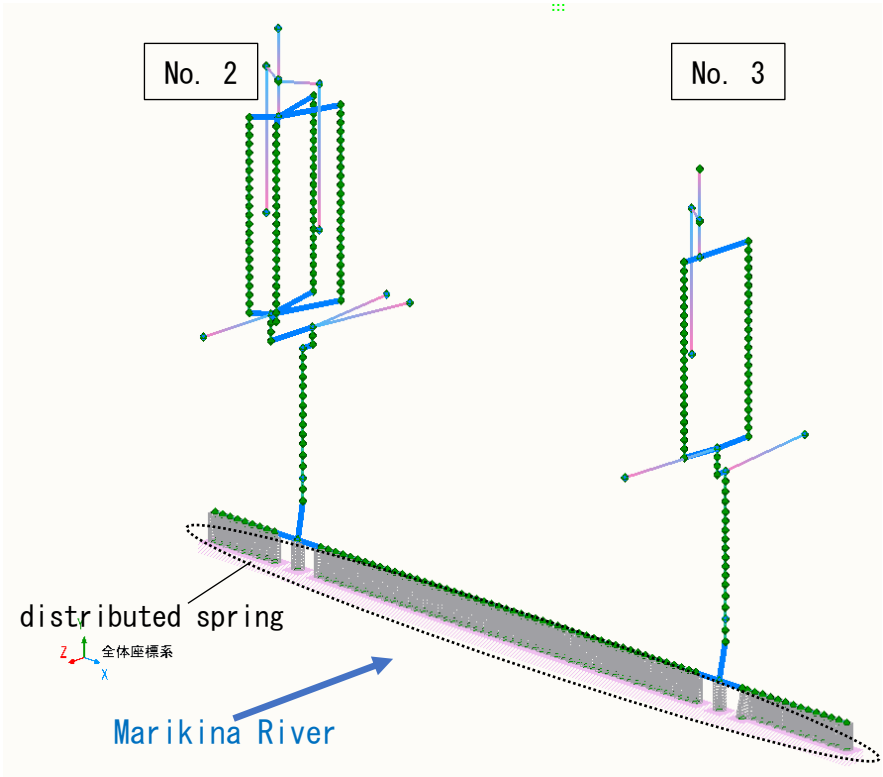
Source: Study Team

Figure 7.3.38 Analytical Model Diagram of No. 1 Pier Bottom Slab (Skeleton)



Source: Study Team

Figure 7.3.39 Analytical Model Diagram of Bottom Slab of Piers No. 2 to No. 3 (Solid Elements)



Source: Study Team

Figure 7.3.40 Analytical Model Diagram of Bottom Slab of Piers No. 2 to No. 3 (Skeleton)

4) Calculation of Working Load

(a) Summary

Table 7.3.84 summarizes loads considered to the assessment. The details of the load setting are shown on the following pages.

Table 7.3.84 Loads to Consider

Permanent load	Dead load	Body weight
		Weight of the Control room
		Dead weight of cinder concrete
		Machine weight (gate and hoist)
		Spiral stair weight
		Dead weight of the bridge
	Hydrostatic pressure	
	Internal water weight	
	Uplift pressure	
	Normal earth pressure	
	Topsoil weight	
Seismic load	Inertial force	
	Earth pressure during earthquake	
	Seismic dynamic water pressure	

Source: Study Team

(b) Normal Load

The following loads shall be considered as normal loads.

- Dead load (weight of body, control room, cinder concrete gate, maintenance bridge, stair, and hoist)
- Hydrostatic pressure, internal water weight, and uplift
- Normal earth pressure

The calculation results of the load and the load action diagram are listed on the following pages.

(i) Dead Load

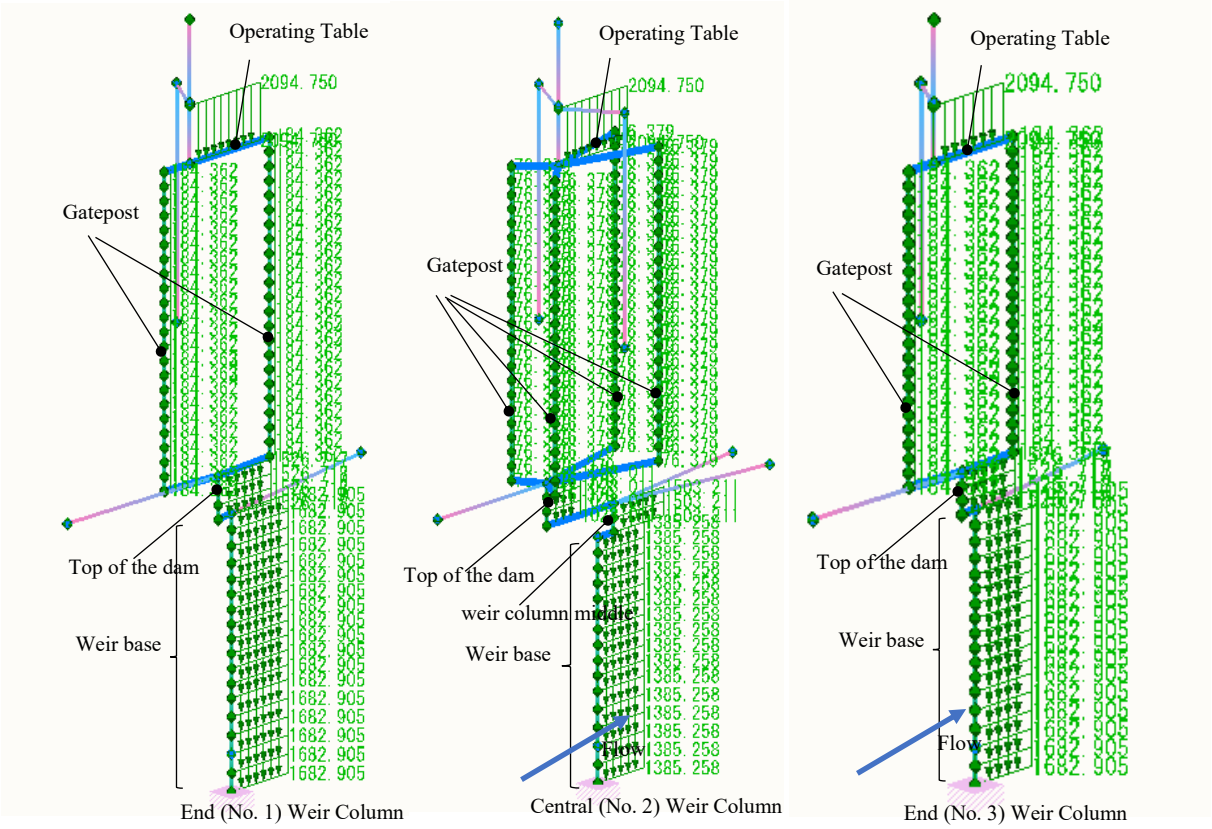
A. Body Weight

The following loads shall be considered for the self-weight of the body.

Table 7.3.85 List of Self-Weight

Member			Section A (m2)	Unit Volume Weight γ (kN/m3)	Weight per Unit Length P (kN/m)
Operating deck			85.500	24.5	2,094.750
Column	No. 1 and No. 3		7.525	24.5	184.363
	No. 2		3.118	24.5	76.379
Pier	No. 1 and No. 3	Base	68.690	24.5	1,682.905
		Top	62.315	24.5	1,526.718
	No. 2	Base	56.541	24.5	1,385.255
		Chubu	61.356	24.5	1,503.222
		Top	41.756	24.5	1,023.022
Bottom Slab		General Section	73.221	24.5	1,793.913
		End	61.500	24.5	1,506.750

Source: Study Team

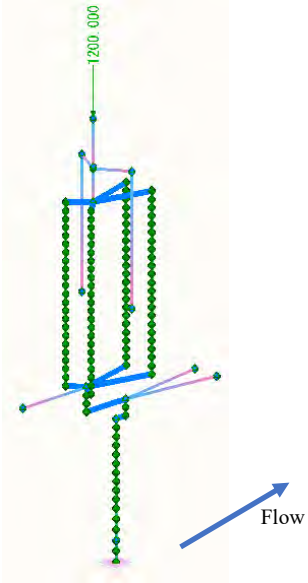


Source: Study Team

Figure 7.3.41 Self-Weight Diagram

B. Weight of Control room

The weight of the control room is 1,200 kN at each pier. As a typical example, the action diagram of the weight of a control room in the No. 2 pier is shown **Figure 7.3.42**.



Source: Study Team

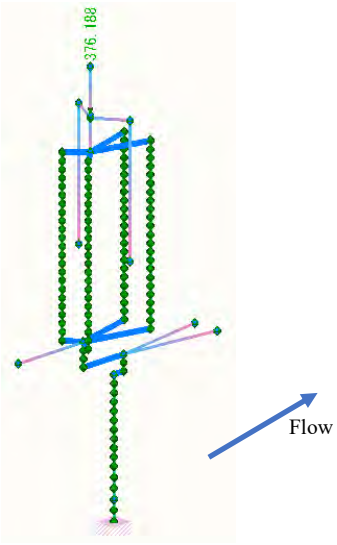
Figure 7.3.42 Action Diagram of Shed Weight

① Self-Weight of Cinder Concrete

The weight of cinder concrete was calculated as follows, considering the thickness of each pier was 20 cm.

W = 23.5 * 9.20 * 8.70 * 0.20 = 376.188 kN

As a typical example, the action diagram of the weight of cinder concrete in the No. 2 pier is shown in **Figure 7.3.43**.

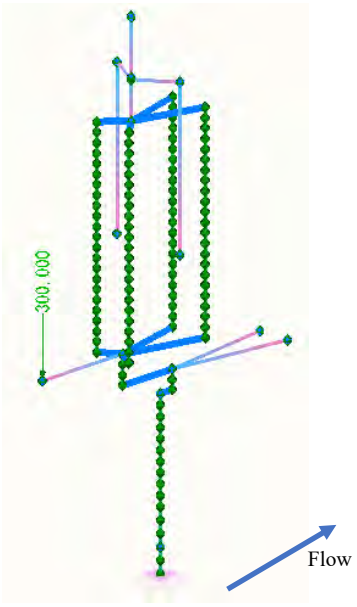


Source: Study Team

Figure 7.3.43 Action Diagram of Cinder Concrete Weight

C. Stair Weight

The weight of the stairs shall be 300 kN at each pier. As a typical example, the action diagram of the stair weight in No. 2 pier is shown in **Figure 7.3.44**.

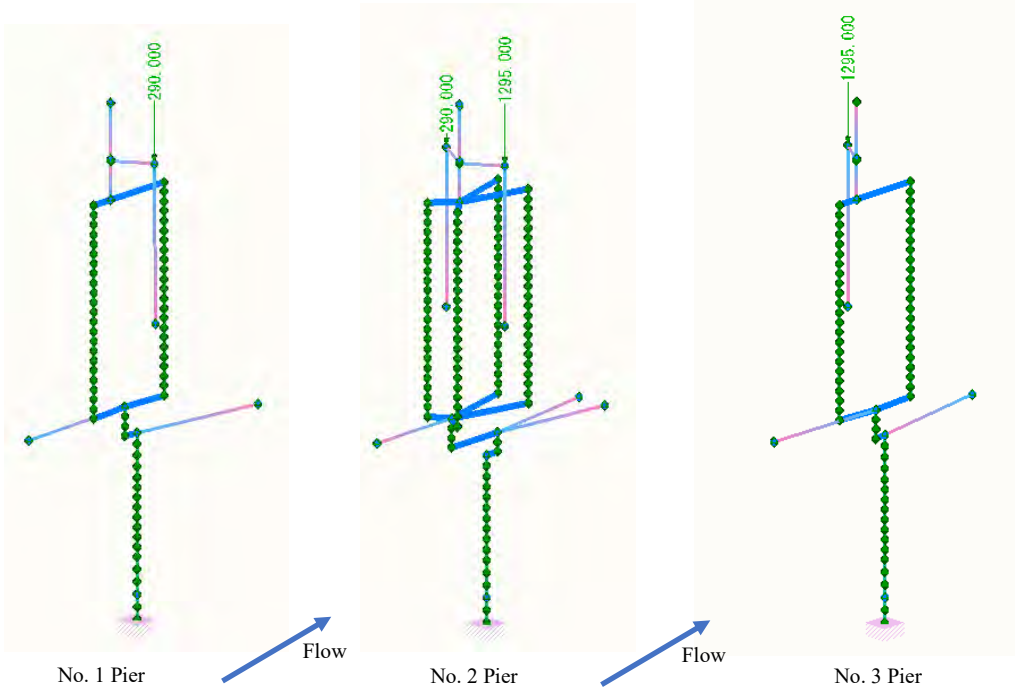


Source: Study Team

Figure 7.3.44 Action Diagram of Stair Weight

D. Gate Weight

The gate weight is 290 kN per 1 fulcrum for No. 1 to No. 2 piers and 1295 kN per 1 fulcrum for No. 2 to No. 3 piers. Action diagram of each pier section is shown in **Figure 7.3.45**.

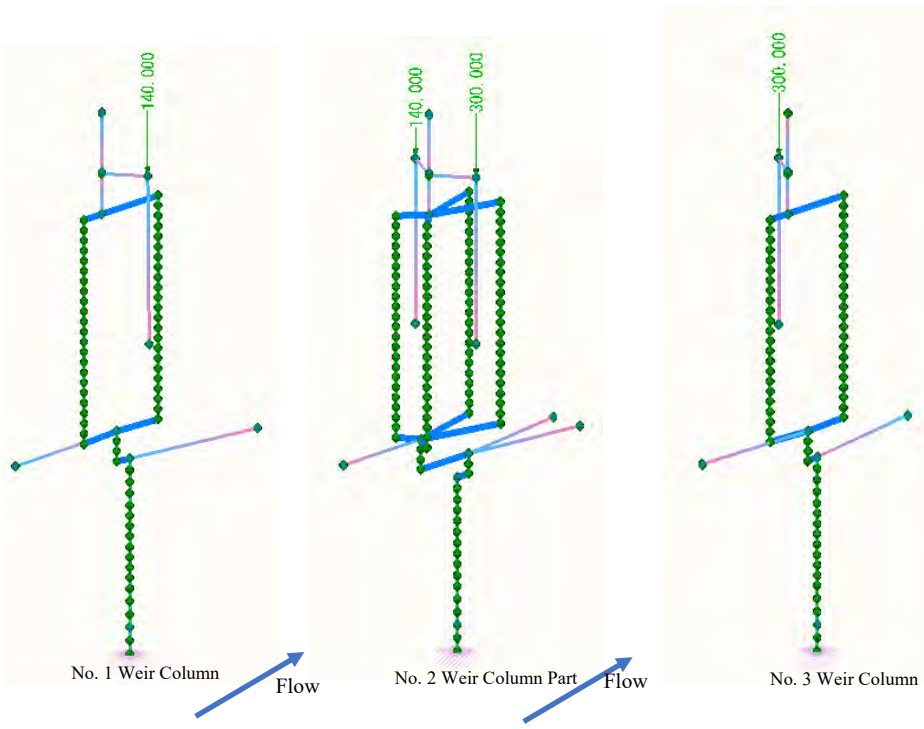


Source: Study Team

Figure 7.3.45 Action Diagram of Gate Weight

E. Hoist Weight

The weight of the hoist shall be 300 kN per 1 fulcrum for No.1 to No.2 piers and 140 kN per 1 fulcrum for No.2 to No.3 piers. The load action diagram of each pier is shown in **Figure 7.3.46**.

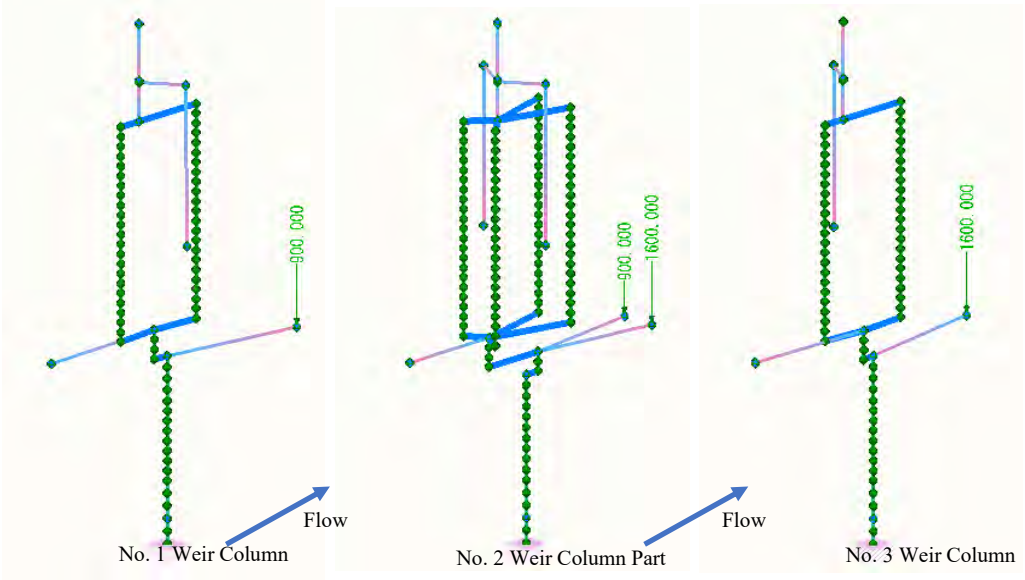


Source: Study Team

Figure 7.3.46 Action Diagram of Hoist Weight

F. Weight of Maintenance Bridge

The weight of the maintenance bridge shall be 900 kN per 1 fulcrum for No. 1 to No. 2 piers and 1295 kN per 1 fulcrum for No. 2 to No. 3 piers. The load action diagram of each pier is shown in **Figure 7.3.47**.



Source: Study Team

Figure 7.3.47 Action Diagram of Maintenance Bridge Weight

(ii) Hydraulic Pressure

A. Hydrostatic Pressure (Internal Pressure) and Internal Water Weight

The equation for calculating the hydrostatic pressure is shown below. For calculation results and diagrams are displayed in **Table 7.3.86**.

Normal water = +	11.09	m	(Inner Water Level)
Bottom Slab top = +	7.850	m	

horizontal load

$ph = \gamma w \cdot h$

ph	:	Hydrostatic Pressure (kN/m^2)		
H1	:	Depth (m)	=	3.240
γw	:	Unit Weight of Water (kN/m^3)	=	9.80
h	:	Depth from Water Surface (m)		
l	:	Sidewall Sharing Width (m)	=	20.50

Table 7.3.86 hydrostatic pressure calculation result

Number	Name	Depth y (m)	Hydrostatic Pressure (kN/m2)	Hydrostatic Pressure Acting on the Side Walls (kN/m)
1		0.00	0.000	0.000
2	On the Bottom Slab	3.240	31.752	650.916

Source: Study Team

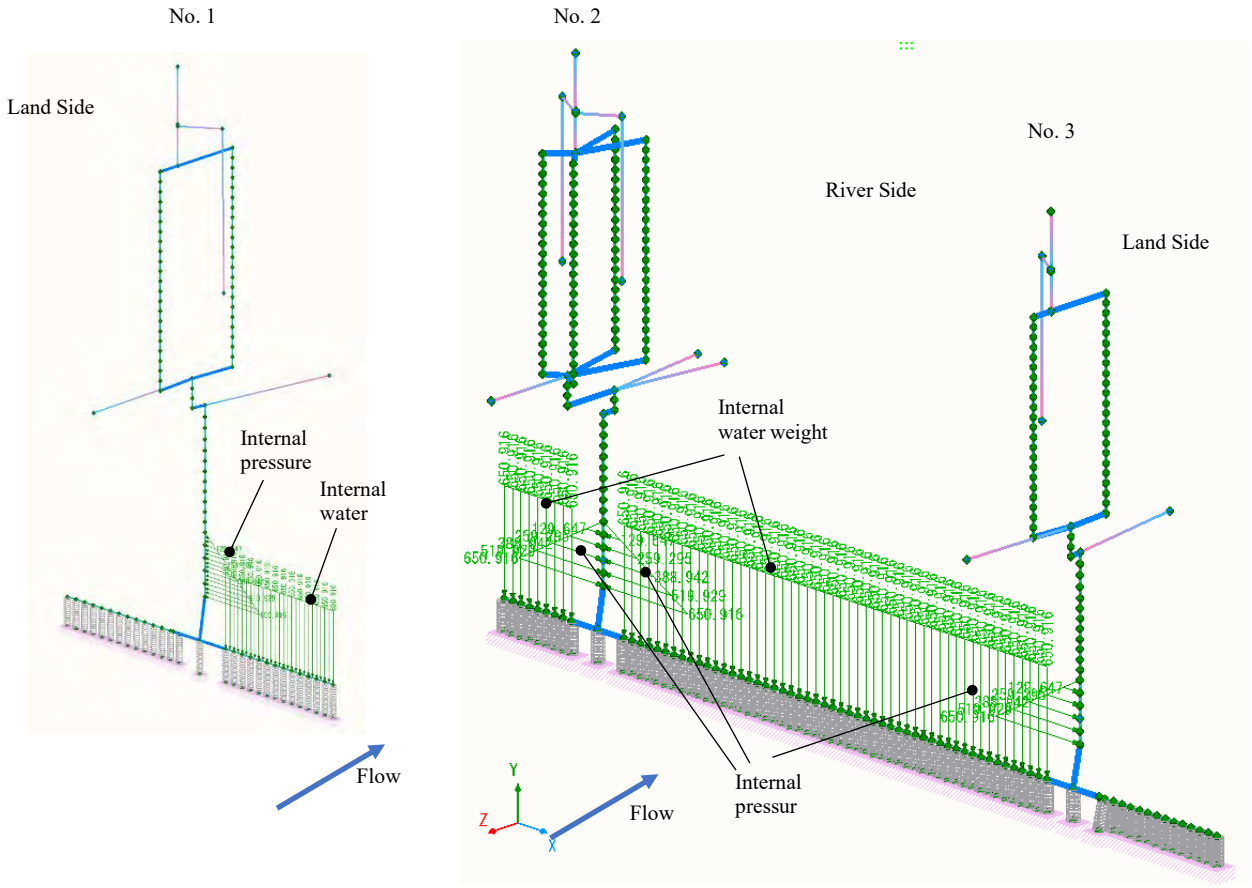
The formula for calculating the internal water weight is shown below. The internal water weight shall be considered when checking the Bottom Slab.

$Ww = \gamma w \cdot H \cdot a \cdot b$

$= 650.916$ kN/m

γw	:	Unit Weight of Water (kN/m3)	=	9.80
H	:	Depth (m)	=	3.240
a	:	Action Width	=	20.50
b	:	Length of Acting	=	1.00

Hydrostatic pressure (internal pressure) and internal water weight are illustrated in **Figure 7.3.48**.



Source: Study Team

Figure 7.3.48 Calculation Results of Internal Water Pressure and Internal Water Weight

B. Hydrostatic Pressure (External Water Pressure)

The equation for calculating hydrostatic pressure (external water pressure) is shown below. The calculation results and a typical load action diagram (No. 1 pier analytical model) are shown in **Figure 7.3.82** and **Figure 7.3.49**.

Normal Water = +	15.2	m	(Outside Water Level)
Bottom Slab Bottom = +	4.050	m	

Horizontal Load

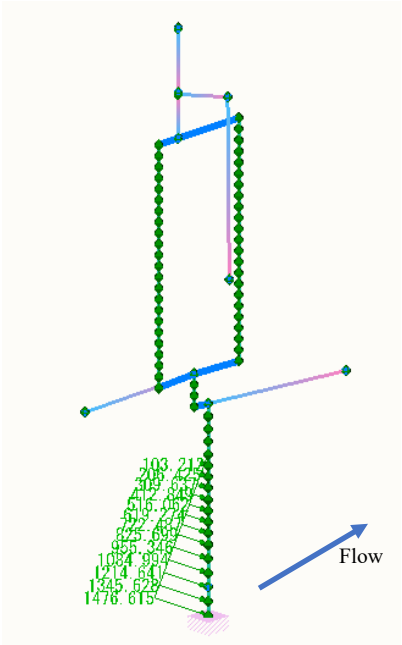
$$ph = \gamma w \cdot h$$

ph	:	Hydrostatic pressure (kN/m2)	
H1	:	Depth (m)	= 11.15
γw	:	Unit weight of water (kN/m3)	= 9.80
h	:	Depth from Water Surface (m)	
l	:	Sidewall sharing width (m)	= 20.50

Table 7.3.87 Calculation Result of Hydrostatic Pressure

Number	Name	Depth y (m)	Hydrostatic Pressure (kN/m^2)	Hydrostatic Pressure Acting on the Side Walls (kN/m)
1		0.00	0.000	0.000
2	On the Bottom Slab	7.350	72.030	1476.615
3	Bottom Slab	11.150	109.270	2240.035

Source: Study Team



Source: Study Team

Figure 7.3.49 Action Diagram of Hydrostatic Pressure (External Water Pressure) (No. 1 Pier Analytical Model)

C. Uplift

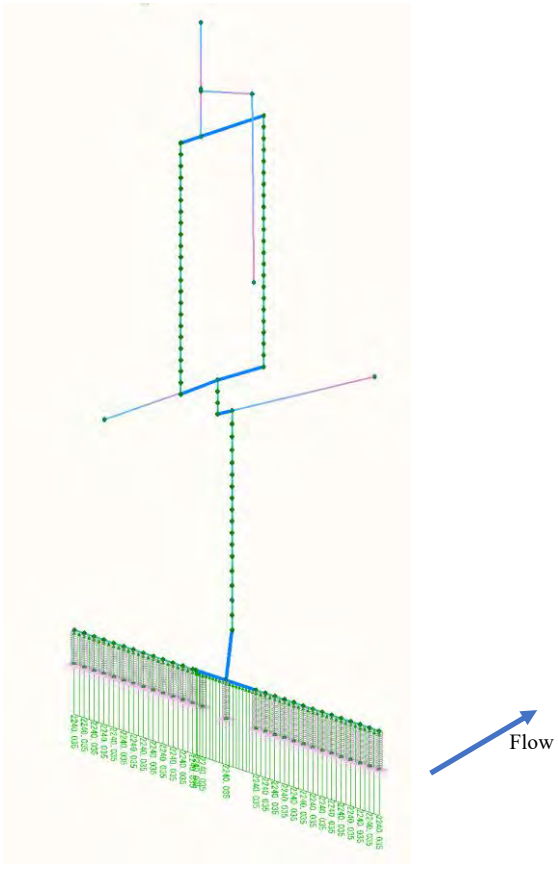
The formula for calculating the uplift is shown below. Uplift shall be considered in the assessment of the Bottom Slab.

$$U = \gamma w \cdot H \cdot a \cdot b$$

$$= 2240.035 \text{ kN/m}$$

γw	:	Unit weight of water (kN/m3)	=	9.80
H1	:	Depth (m)	=	11.150
a	:	Action width	=	20.50
b	:	Prolonged action	=	1.00

As a representative example of the action diagram, the action diagram of the uplift of the Bottom Slab of the No. 1 pier analytical model is shown in **Figure 7.3.50**.



Source: Study Team

Figure 7.3.50 Action Diagram of Uplift (Bottom Slab Analytical Mode of No. 1 Pier)

(iii) Earth Pressure

A. Normal Earth Pressure

The normal earth pressure shall be considered together with the seismic earth pressure. Details are given in the section of seismic earth pressure.

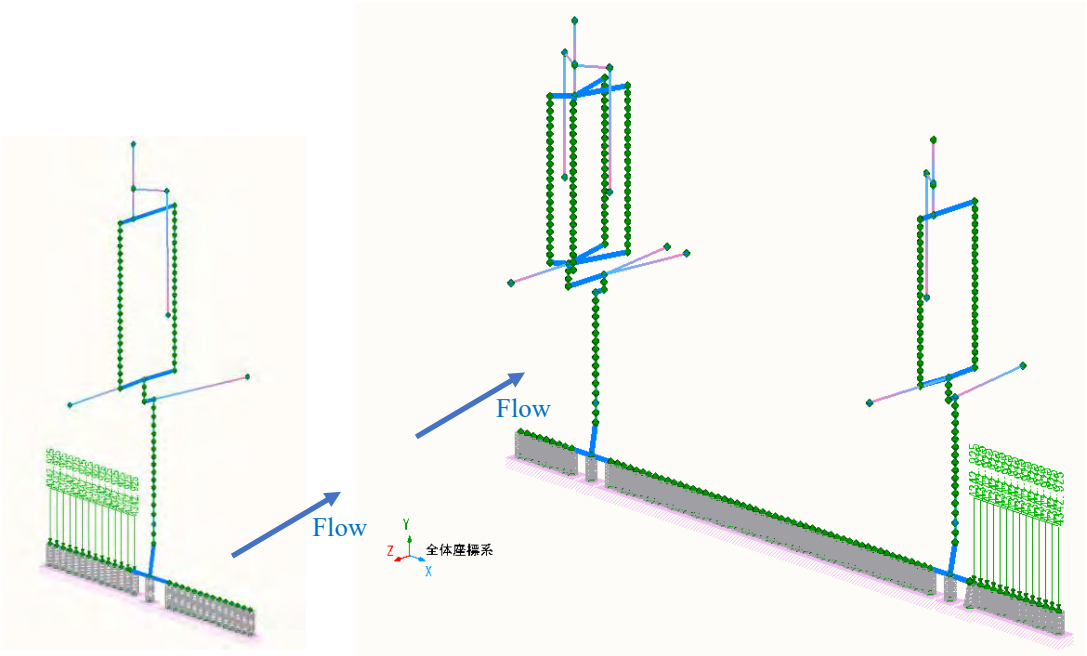
B. Soil Weight

The formula for calculating the soil weight on the overhang of the end pier is shown below.

$$W = \gamma w \cdot H \cdot a \cdot b$$

$$= 4342.925 \text{ kN/m}$$

γ	:	Unit volume weight of soil (kN/m3)	=	19.00
H	:	Depth of soil (m)	=	11.150
a	:	Action width	=	20.50
b	:	Prolonged action	=	1.00



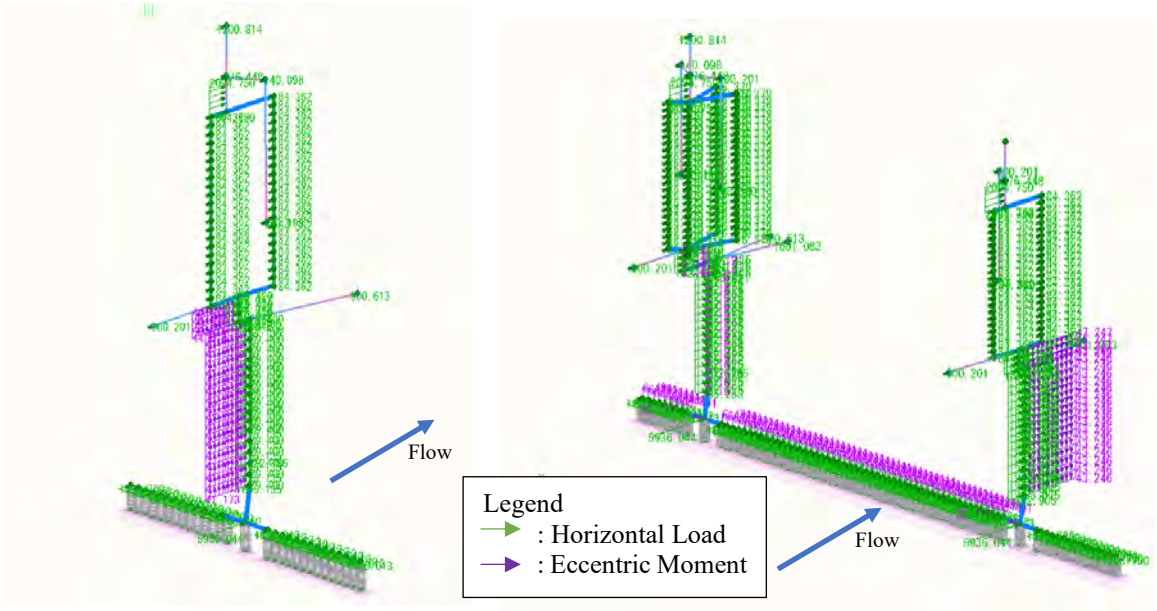
Source: Study Team

Figure 7.3.51 Action Diagram of Weight of Soil

(c) Seismic Load

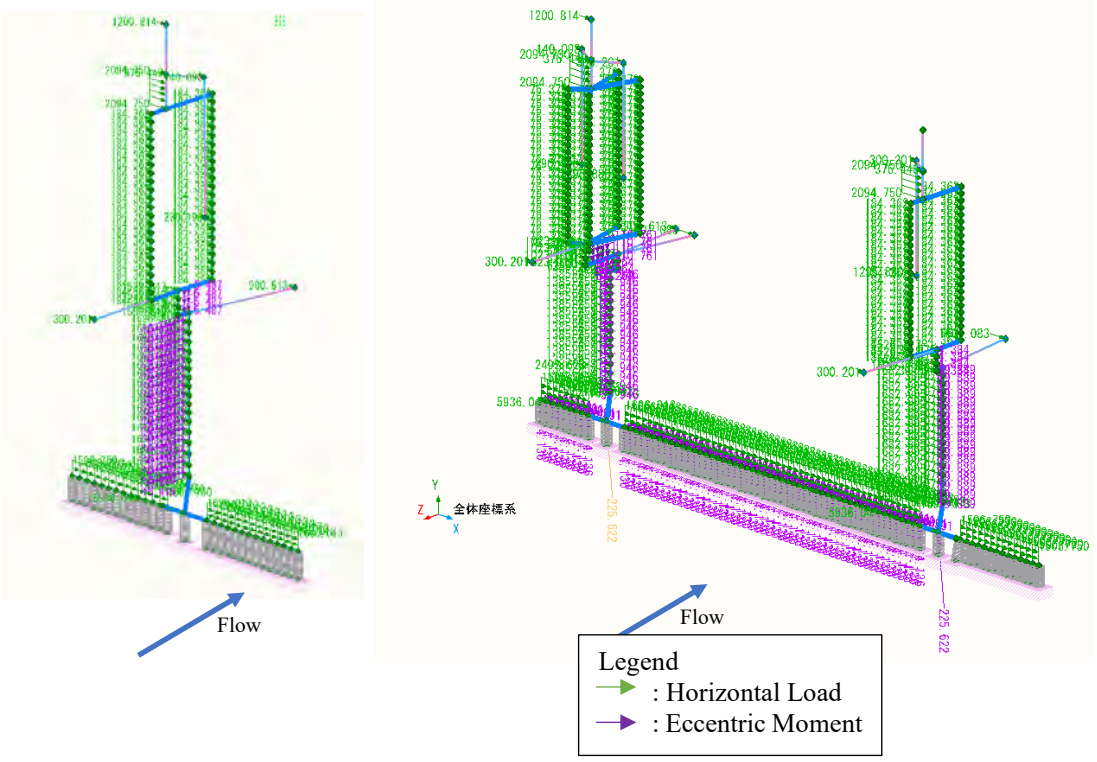
A. Inertial Force

Action diagrams of inertial force in the flow direction and in the direction perpendicular to the flow are shown in **Figure 7.3.52** and **Figure 7.3.53**.



Source: Study Team

Figure 7.3.52 Action Diagram of Inertia
(Flow Direction (Upstream to Downstream) Kh = 1.0 during loading)



Source: Study Team

Figure 7.3.53 Action Diagram of Inertia
(Perpendicular to Flow (Left Bank to Right Bank) Kh = 1.0 during loading)

B. Seismic Earth Pressure

According to “Performance Based Seismic Design Criteria for River Structures I. Common MLIT, Japan 2012”, the "earth pressure at the time of earthquake is calculated.

① Calculation of design Horizontal Seismic Coefficient

The ground type is type I. As will be described later in **7.3.2.4 (4) 5)** , the acceleration spectrum of the MCGS was estimated consideration of the ground type and the location of the MCGS.

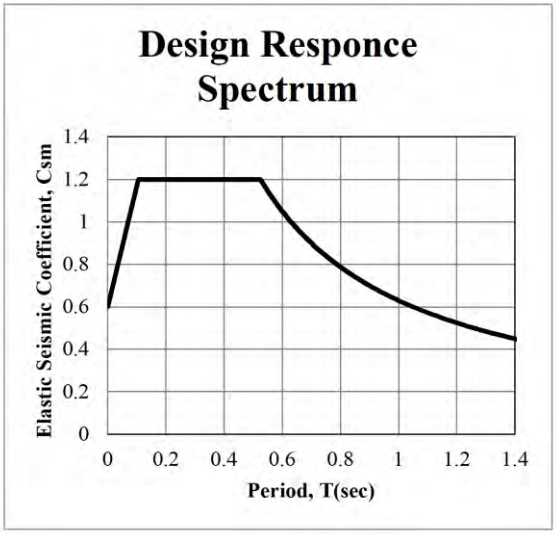
① PGA: 0.6
F_{PGA}: 1 (Soil Type I)
AS= 0.6

② Ss: 1.2
Fa 1 (Soil Type I)
S_{DS}= 1.2

③ S1: 0.45
Fv 1.4 (Soil Type I)
S_{D1}= 0.63

$T_S = S_{D1}/S_{DS}$ = 0.525

$T_0 = 0.2 * T_S$ = 0.105



From the above spectrum, khg = 0.60.

② Calculation of active earth pressure coefficient at the time of earthquake

If the back is soil and concrete, it is assumed that "sandy soil".

kEA = 0.24 + 1.08 khg

Here, kEA: Coefficient of active earth pressure at the time of earthquake

khg: Design Horizontal Seismic Coefficient at ground level

kEA =	0.24	+	1.08	×	0.60	=	0.888	

③ Calculation of seismic active earth pressure

The Earth Pressure at The Time of Earthquake Is Calculated by The Following Formula.

PEA = Γ ・ X ・ KEA + Q '・ KEA

Here, PEA: Active Earth Pressure at The Time of Earthquake (KN/m^2)

Γ: Unit Volume Weight of Soil (KN/m^2)

Q ': Ground Surface Loading at The Time of Earthquake (KN/m^2)

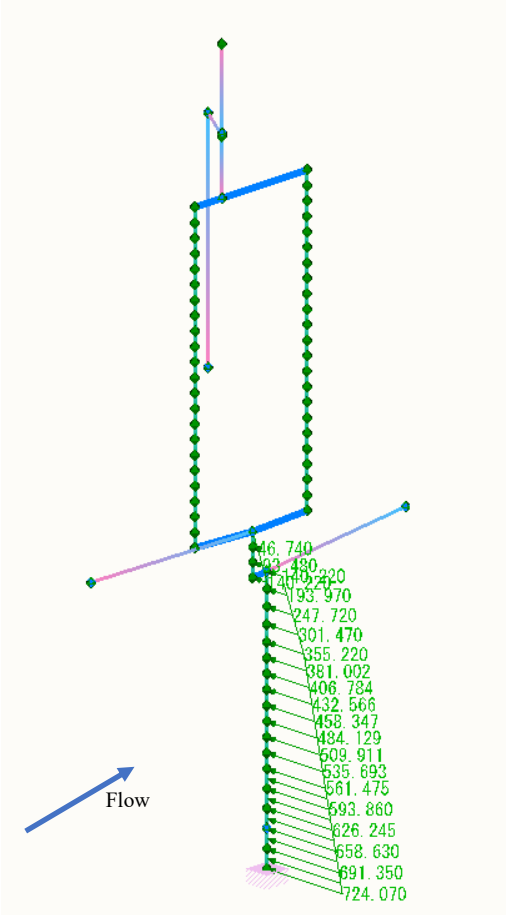
	Ql0 =	0.00	×	0.888				=	0.00	KN/m^2
Groundwater Level	Ql1 =	0.00	+	0.888	×	19.00	× 3.800	=	64.11	KN/m^2
On the Bottom Slab	Ql2 =	64.11	+	0.888	×	10.20	× 7.350	=	130.68	KN/m^2
Bottom Slab	Ql3 =	130.68	+	0.888	×	10.20	× 3.800	=	165.10	KN/m^2

④ Calculation of normal active earth pressure

The normal earth pressure is the value when khg of kEA = 0.24 + 1.08 · khg is 0.

	kEA =		0.240										
qL0 =	0.0	×	0.240	×	31.000					=	0.00	KN/m^2
qL1 =	0.00	+	0.240	×	19.00	×	3.800	×	20.500	=	355.22	KN/m^2
qL2 =	355.22	+	0.240	×	10.20	×	7.350	×	20.500	=	724.07	KN/m^2
qL3 =	724.07	+	0.240	×	10.20	×	3.800	×	20.500	=	914.77	KN/m^2

As a typical action diagram of the normal active earth pressure, the action diagram in the No. 3 pier Analytical Model is shown in **Figure 7.3.54**.



Source: Study Team

Figure 7.3.54 Action Diagram of Normal Earth Pressure (No. 3 Pier Analytical Model)

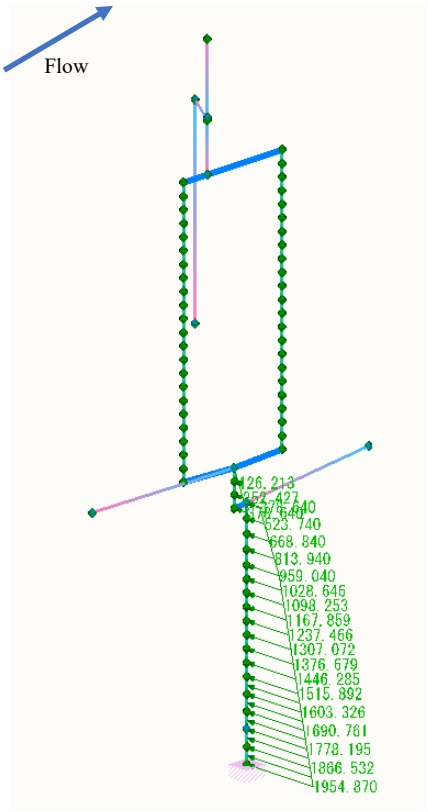
⑤ Calculation of increment by seismic active earth pressure

The earth pressure increment at the time of earthquake shall be the value obtained by subtracting the active earth pressure at all times from the active earth pressure at the time of earthquake.

① Level 2 seismic

qL0 =	0.00	×	20.500	-	0.00	=	0.00 kN/m
qL1 =	64.11	×	20.500	-	355.22	=	959.04 kN/m
qL2 =	130.68	×	20.500	-	724.07	=	1954.87 kN/m
qL3 =	165.10	×	20.500	-	914.77	=	2469.78 kN/m

As a typical action diagram of the increment by seismic active earth pressure, the action diagram in the No. 3 pier Analytical Model is shown in **Figure 7.3.55**.



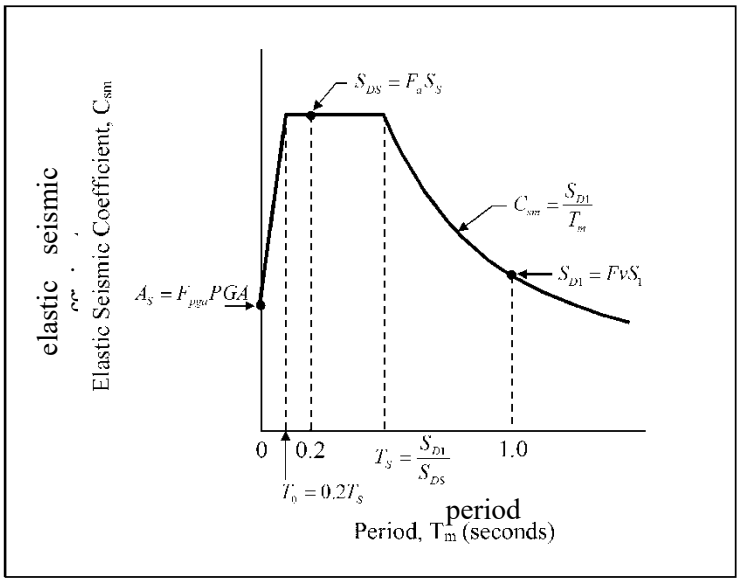
Source: Study Team

Figure 7.3.55 Action Diagram of Increment by Seismic Earth Pressure (No. 3 Pier Analytical Model)

5) Design Horizontal Seismic Coefficient

The design Horizontal Seismic Coefficient shall be set based on the following description in the BSDS.

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.



design response spectrum

Figure 3.6.1-1 Design Response Spectrum

Source: BSDS, DPWH, P3 -36

Figure 7.3.56 Design Response Spectrum

Referring to **Figure 7.3.57** to **Figure 7.3.59**, the PGA, Ss and S1 indicated above were determined as follows.

PGA	: 0.6
Ss	: 1.2
S_1	: 0.45

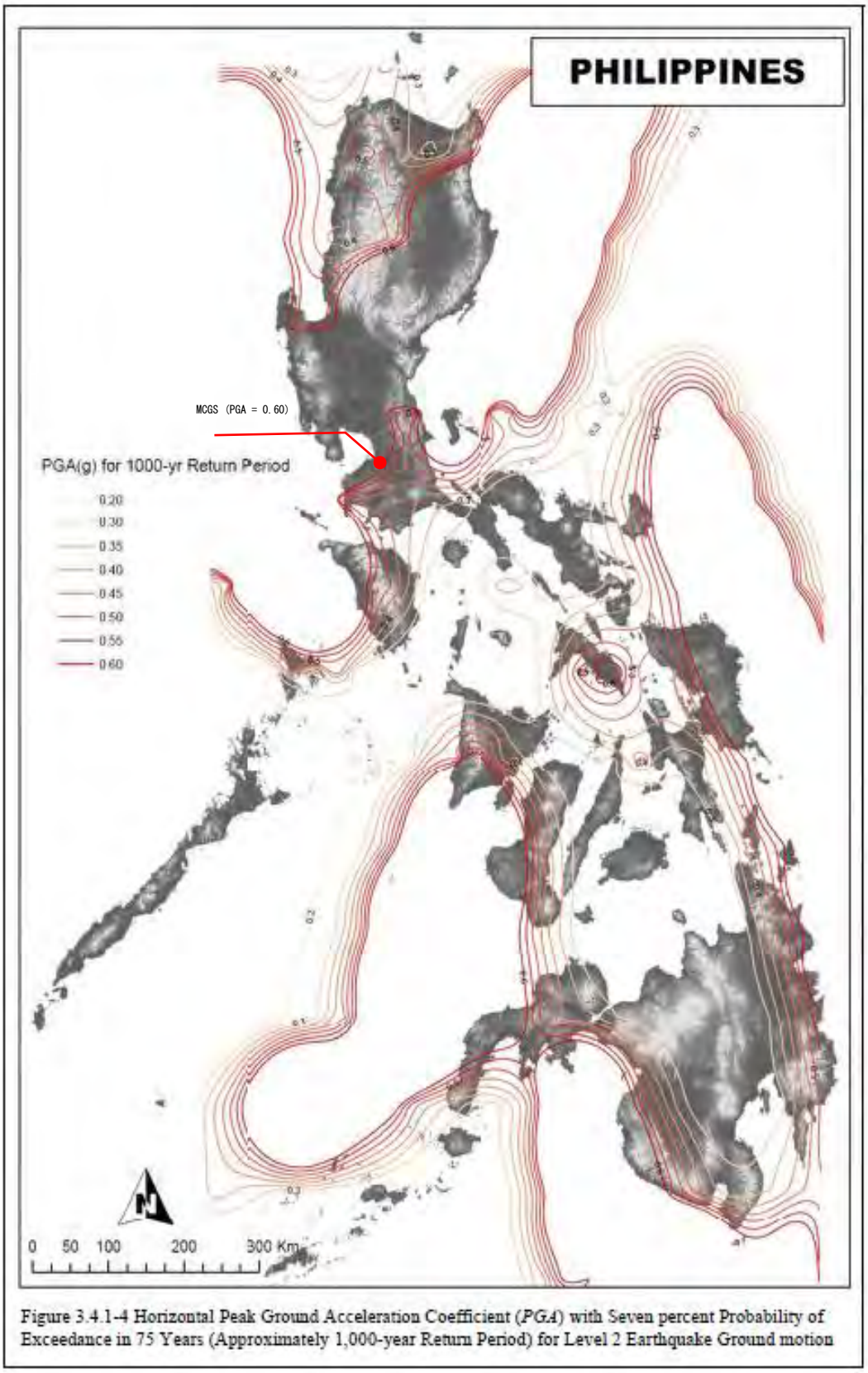


Figure 3.4.1-4 Horizontal Peak Ground Acceleration Coefficient (*PGA*) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground motion

Figure 7.3.57 L2 Horizontal Peak Ground Acceleration Coefficient PGA (BSDS, p3 -21)

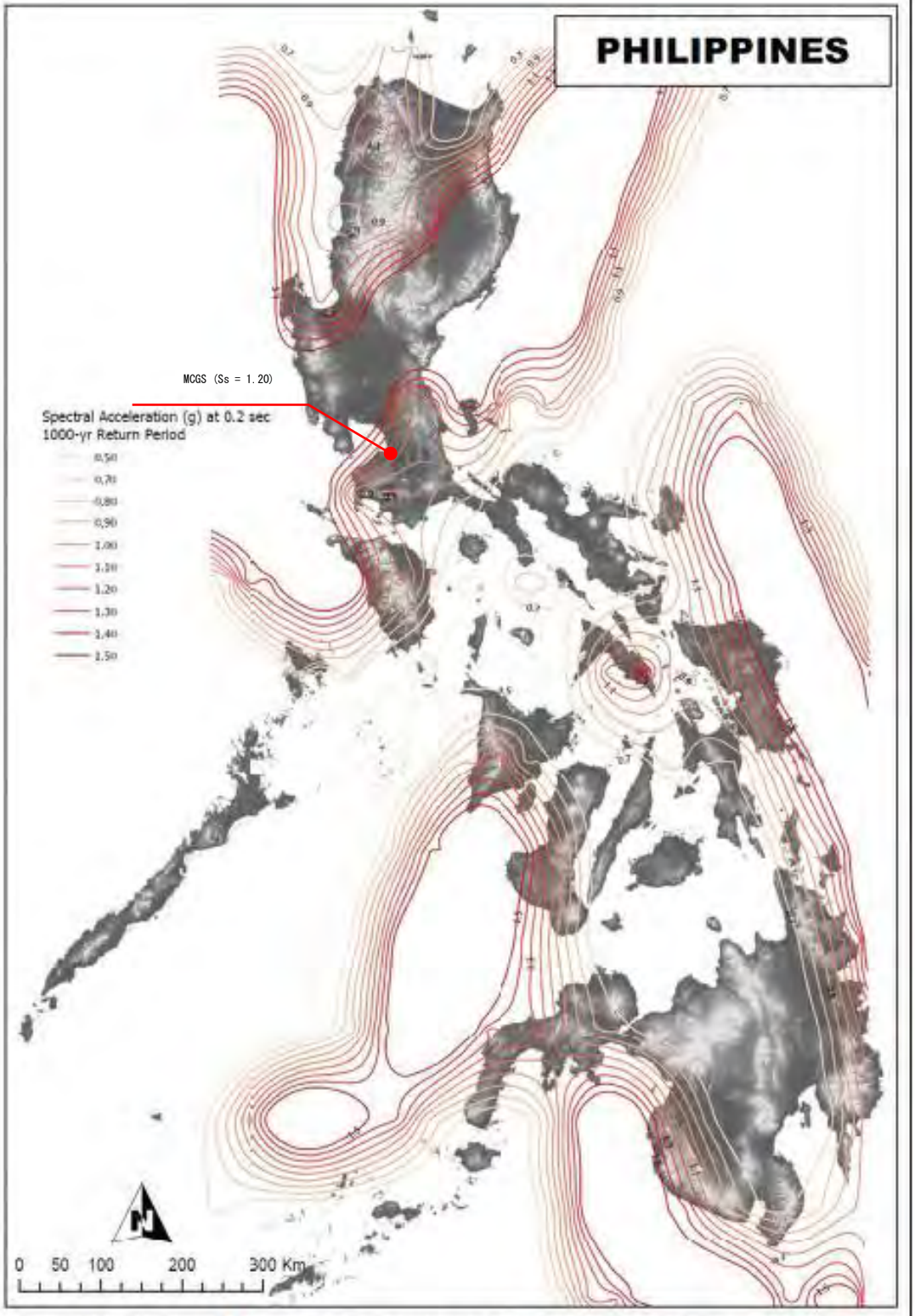


Figure 3.4.1-5 Horizontal Response Spectral Acceleration Coefficient at Period of 0.20-sec (S_s) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground Motion

Figure 7.3.58 Horizontal Response Spectral Acceleration Coefficient Ss (BSDS Figure 3.4. 1. -5)

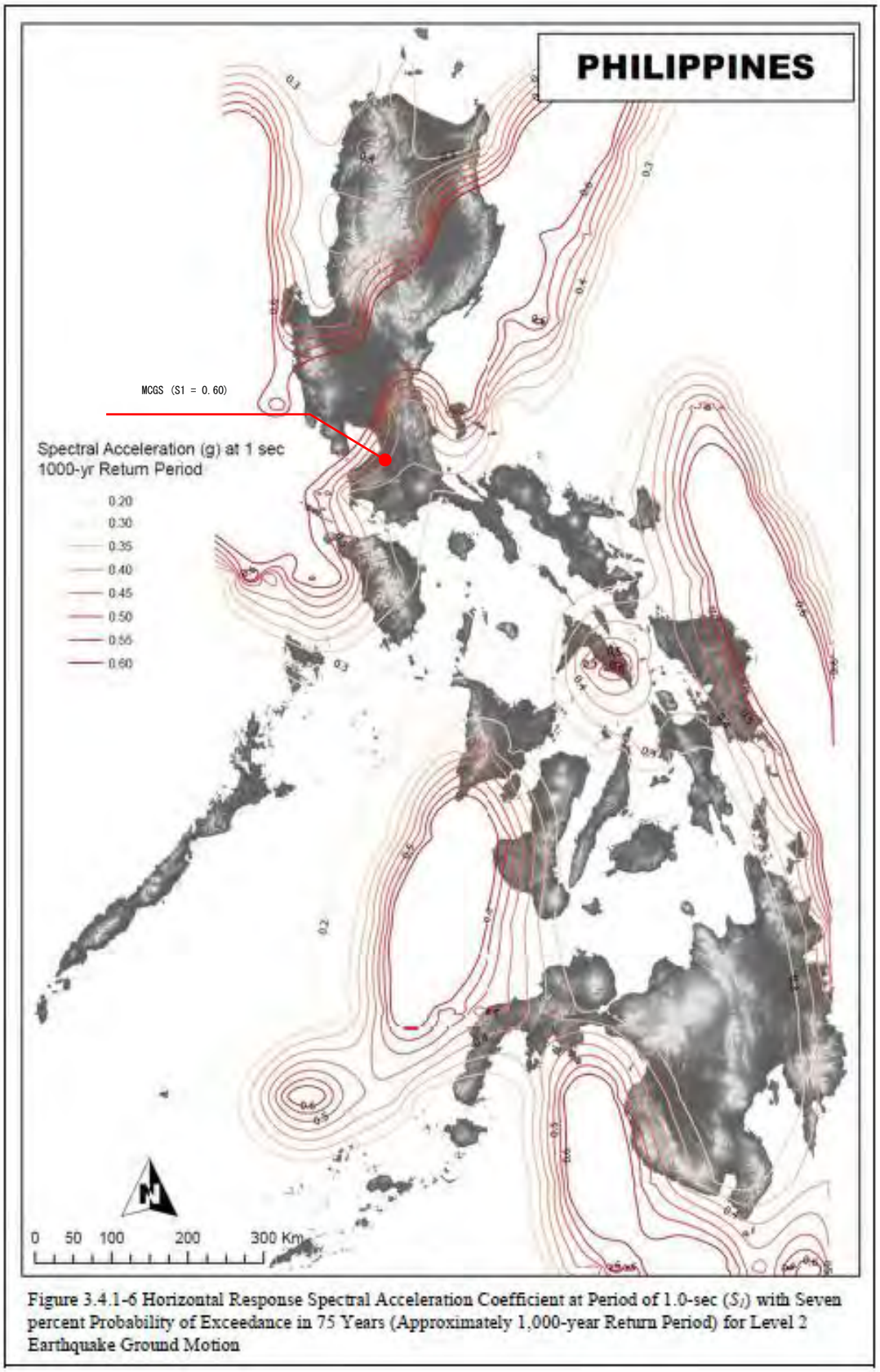


Figure 3.4.1-6 Horizontal Response Spectral Acceleration Coefficient at Period of 1.0-sec (S_1) with Seven percent Probability of Exceedance in 75 Years (Approximately 1,000-year Return Period) for Level 2 Earthquake Ground Motion

Figure 7.3.59 Horizontal Response Spectral Acceleration Coefficient S1 (BSDS Figure 3.4. 1. -5)

The ground of MCGS corresponds to Type I ground. The FPGA, Fa and Fv were set from the PGA, Ss and S1 based on the description of the BSDS.

F_{PGA} : 1.0
Fa : 1.0
Fv : 1.4

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]					
	$PGA \le 0.10$	$PGA = 0.20$	$PGA = 0.30$	$PGA = 0.40$	$PGA = 0.50$	$PGA \ge 0.80$
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}=1.0

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]					
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \ge 2.0$
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→Fa=1.0

Table 3.5.3-3 Values of Site Factor, F_v, for Long-Period Range on Acceleration Spectrum

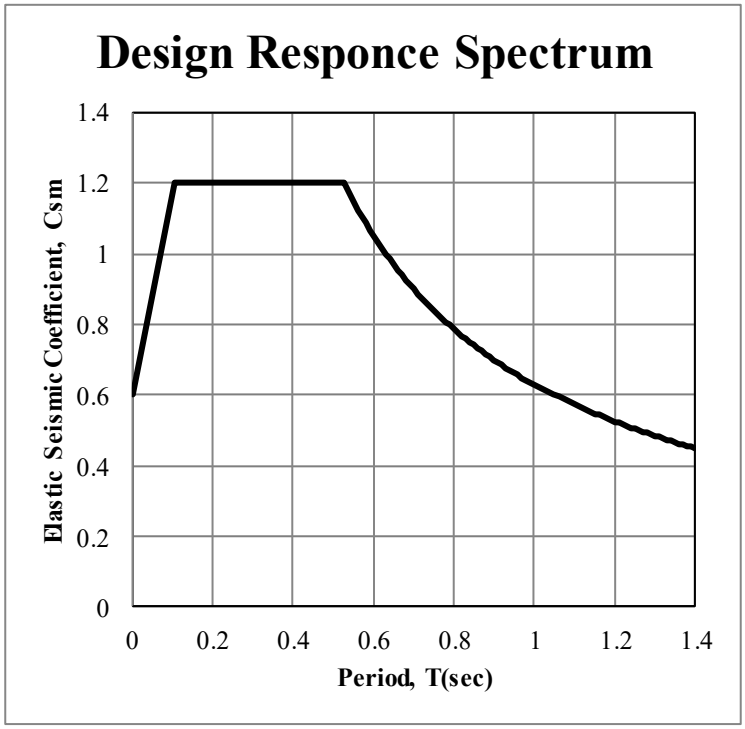
Ground Type (Site Class)	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \ge 0.80$
I	1.7	1.6	1.5	1.4	1.4	1.4
II	2.4	2.0	1.8	1.6	1.5	1.5
III	3.5	3.2	2.8	2.4	2.4	2.0

Note:
[1] Use straight-line interpolation for intermediate values of S_1.

S_1=0.45→Fv=1.4

Source: BSDS, p3 -32 - 33

The acceleration spectrum set in this study is shown in **Figure 7.3.60**.



Source: Study Team prepared

Figure 7.3.60 MCGS Acceleration Spectrum

6) Checking of Columns and Piers

(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[3]. 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/mm^2)
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)
σ_{sy}	:	Yielding point of band rebar (N/mm^2)
θ	:	Angle between the band rebar and the vertical axis (°)
a	:	Spacing between band rebars (mm)

(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.

$$k_h W \quad \leqq \quad P_a$$

[3] Specifications for Highway Bridges V Seismic Design P186

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 (**Table 7.3.88**)

[Transition type from bending failure to shear failure]

$\mu_a = 1.0$

Table 7.3.88 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.

$W=W_u+C_pW_p$

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)

Table 7.3.89 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

(b) Residual Displacement Analysis

$$\delta_R \leqq \delta_{Ra}$$

Where,

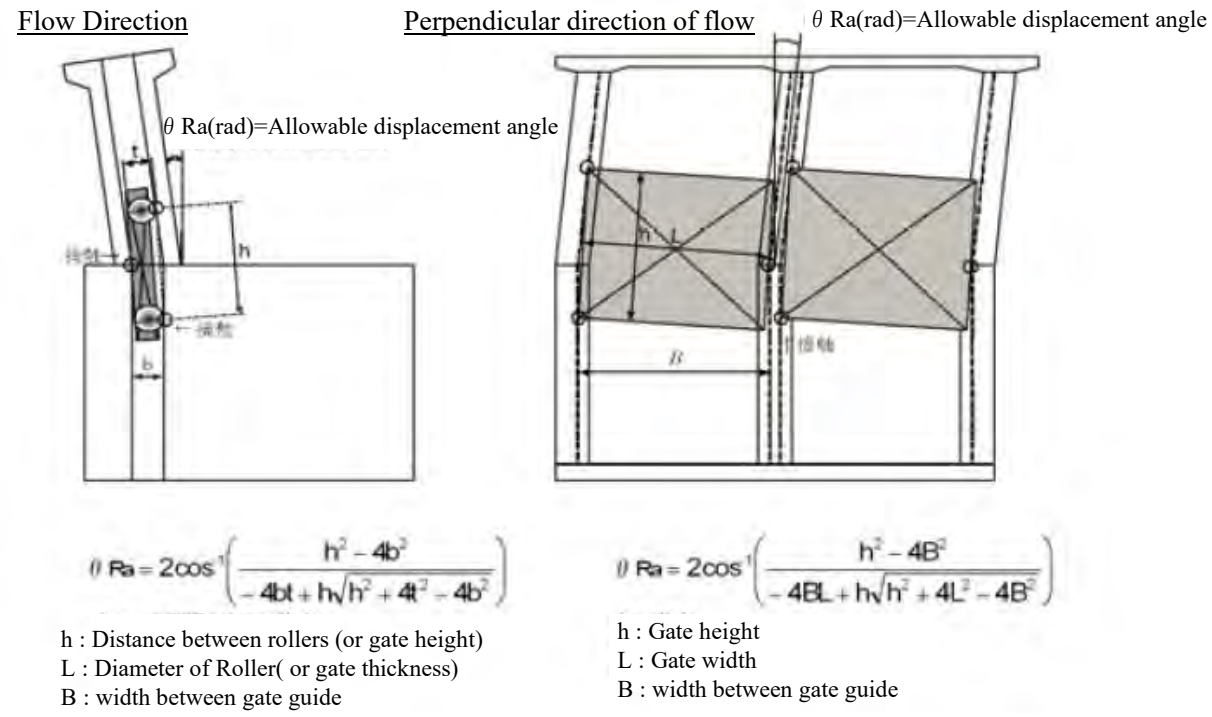
δ_R : Residual Displacement

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 (**Figure 7.3.61**).

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 **Figure 7.3.61**. If the allowable residual displacement is less than h/100, the more severe value is used.



Source: Performance Based Seismic Design Criteria for River Structures IV

Figure 7.3.61 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 (Gate 1 and Gate 2) and the calculation basis are shown in **Table 7.3.90** and **Figure 7.3.62**.

Table 7.3.90 Calculation Result of Allowable Residual Displacement (No.1 Gate)

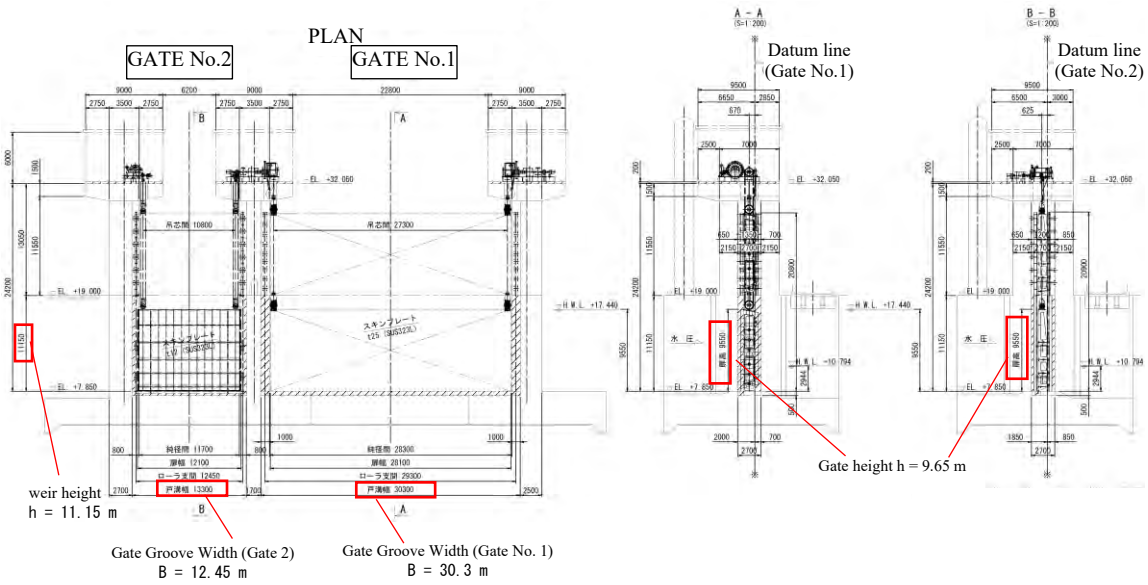
Term		Units	Value	Remarks
Weir height	H	m	11.150	
Gate Height	h	m	9.550	
Between side rollers	L	m	28.300	
Gate Groove Width	B	m	28.500	
Allowable residual angle (calculated value)	θ ra1	rad	0.098	
Allowable residual angle (1/100)	θ Ra2	rad	0.010	
Acceptable residual angle adoption value	θ Ra	rad	0.010	The smaller value of θRa1 and θRa2
Allowable residual displacement	δ Ra	m	0.112	H θRa

Source: Study Team

Table 7.3.91 Calculation Result of Allowable Residual Displacement (No.2 Gate)

Term		Units	Value	Remarks
Weir height	H	m	11.150	
Gate Height	h	m	9.550	
roller support	L	m	12.100	
Gate Groove Width	B	m	12.450	
Allowable residual angle (calculated value)	θ ra1	rad	0.164	
Allowable residual angle (1/100)	θ Ra2	rad	0.010	
Acceptable residual angle adoption value	θ Ra	rad	0.010	The smaller value of θRa1 and θRa2
Allowable residual displacement	δ Ra	m	0.112	H θRa

Source: Study Team



Source: Study Team

Figure 7.3.62 Calculation Basis of Allowable Residual Displacement

(5) Result of Level 2 Seismic Analysis on Columns and Piers

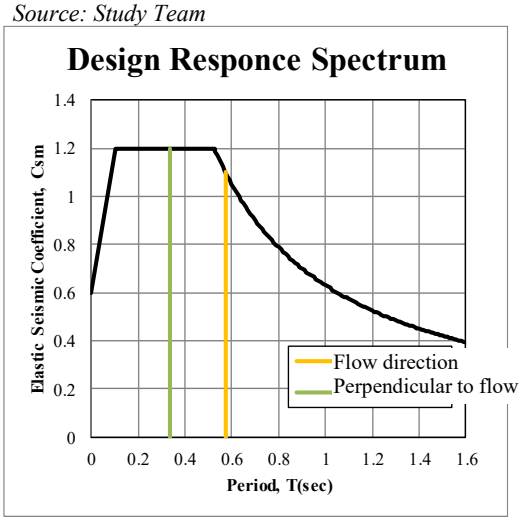
1) Setting of Design Horizontal Seismic Coefficient

Table 7.3.92 describes the horizontal seismic coefficient. These results are illustrated on the acceleration spectrum in **Figure 7.3.63** and **Figure 7.3.64**.

Table 7.3.92 Calculation Result of Design Horizontal Seismic Coefficient

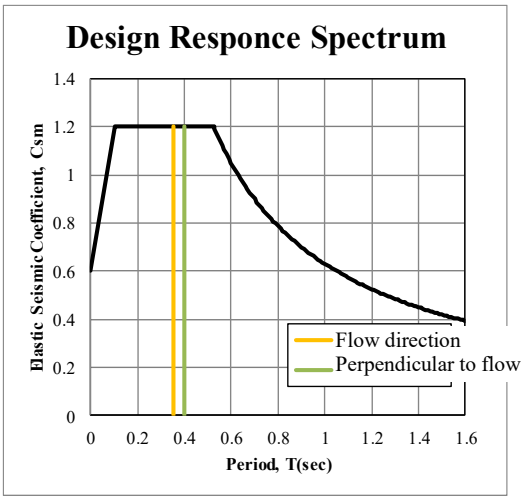
Item		Natural period T (s)	Design Horizontal Seismic Coefficient khc0
No. 1 Pier	Flow Direction	0.573	1.10
	Direction perpendicular to the water flow	0.334	1.20
No. 2 to No. 3 Weir Pillars	Flow Direction	0.351	1.20
	Direction perpendicular to the water flow	0.399	1.20

Source: Study Team



Source: Study Team

Figure 7.3.63 Calculated Design Horizontal Seismic Coefficient of No.1 Pier



Source: Study Team

Figure 7.3.64 Calculated Design Horizontal Seismic Coefficient of No. 2 to No. 3 Piers

2) Results of Lateral Load Carrying Capacity Analysis at the Time of Earthquake

The lateral load carrying capacity analysis at the time of earthquake were conducted using the two models: No.1 pier, and No.2 to No.3 piers, in the flow direction and perpendicular to flow. The results are shown in **Table 7.3.93** to **Table 7.3.96** . In addition, seismic performance under the design bar arrangements in **Figure 7.3.65** to **Figure 7.3.70** were checked and confirmed that the seismic performance was satisfied.

Table 7.3.93 Results of Lateral Load Carrying Capacity Analysis (No. 1 Column, Pier; Flow Direction)

Reference items/Direction		Flow 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)	6,422	6,411	59,173	59,158
Shear capacity	Ps (kN)	7,135	7,135	55,409	55,409
Shear capacity	Ps0 (kN)	7,703	7,703	57,641	57,641
Decision formula		S ≦ Ps	S ≦ Ps	Ps0 < S	Ps0 < S
Fracture morphology		Flexural fracture type	Flexural fracture 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	1.445	1.443	1.000	1.000
Structure characteristic correction factor	Ccs	0.727	0.728	1.000	1.000
Regional correction factor	cz	1.00	1.00	1.00	1.00
Standard value of design Horizontal Seismic Coefficient	khc0	1.100	1.100	1.100	1.100
Design horizontal seismic coefficient	khc	0.80	0.80	1.10	1.10
Horizontal Seismic Coefficient kha designed for horizontal load bearing capacity during earthquakes		1.23	1.23	1.95	1.95
Result of checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of residual displacement					
Residual displacement	δ R (mm)	0.0	0.0	0.0	0.0
Maximum response plasticity	μrT	0.897	0.899	0.659	0.659
Allowable residual displacement	δ Ra (mm)	110.5	110.5	111.5	111.5
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

Table 7.3.94 Results of Lateral Load Carrying Capacity Analysis (No. 1 Column, Pier; Perpendicular to Flow)

Reference items/Direction		Direction perpendicular to flow			
		Positive direction	Negative direction	Positive direction	Negative direction
Main plasticizing member		Column	Column	Pier	Pier
Determination of fracture mode					
Response shear force	S (kN)	5,600	5,689	52,804	21,646
Shear capacity	Ps (kN)	6,586	6,586	58,321	58,321
Shear capacity	Ps0 (kN)	7,014	7,014	61,658	61,658
Decision formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture morphology		Flexural fracture type	Flexural fracture type	Flexural fracture type	Flexural fracture type
Checking by the seismic horizontal load bearing capacity method					
Safety factor	α	1.5	1.5	1.5	1.5
Allowable plasticity	μa	1.351	2.501	1.450	2.313
Structure characteristic correction factor	Ccs	0.766	0.500	0.725	0.525
Regional correction factor	cz	1.00	1.00	1.00	1.00
Standard value of design Horizontal Seismic Coefficient	khc0	1.200	1.200	1.200	1.200
Design horizontal seismic coefficient	khc	0.92	0.60	0.87	0.63
Horizontal seismic coefficient kha designed for horizontal load bearing capacity during earthquakes		1.05	1.04	1.07	0.97
Result of checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of residual displacement					
Residual displacement	δR (mm)	11.7	16.6	2.8	6.1
Maximum response plasticity	μrT	1.153	1.170	1.125	1.264
Allowable residual displacement	δRa (mm)	110.5	110.5	111.5	111.5
Result of checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team

Table 7.3.95 Results of Lateral Load Carrying Capacity Analysis (No. 2 Column, Pier; Flow Direction)

Reference items/Direction		Flow 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)	3,643	4,141	57,926	57,926
Shear capacity	Ps (kN)	5,703	7,222	39,310	39,310
Shear capacity	Ps0 (kN)	5,990	7,509	40,635	40,635
Decision formula		S ≦ Ps	S≦Ps	S≦Ps	Ps0＜S
Fracture morphology		Flexural fracture type	Flexural fracture type	Shear fracture type	Shear fracture type
Checking by the seismic horizontal load bearing capacity method					
Safety factor	α	1.5	1.5	1.5	1.5
Allowable plasticity	μa	1.133	1.134	1.000	1.000
Structure characteristic correction factor	Ccs	0.888	0.888	1.000	1.000
Regional correction factor	cz	1.00	1.00	1.00	1.00
Standard value of design Horizontal Seismic Coefficient	khc0	1.200	1.200	1.200	1.200
Design horizontal seismic coefficient	khc	1.07	1.07	1.20	1.20
Horizontal Seismic Coefficient kha designed for horizontal load bearing capacity during earthquakes		1.46	1.46	1.40	1.40
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	0.836	0.839	0.866	0.866
Allowable residual displacement	δRa (mm)	110.5	110.5	111.5	111.5
Result of checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team

Table 7.3.96 Results of Lateral Load Carrying Capacity Analysis (No. 2 Column, Pier; Perpendicular to Flow)

Checking Direction		Direction perpendicular to flow			
		Positive direction	Negative direction	Positive direction	Negative direction
Main plasticizing member		Column	Column	Pier	Pier
Determination of fracture mode					
Response shear force	S (kN)	3,139	3,045	29,600	29,181
Shear capacity	Ps (kN)	3,153	3,153	31,858	31,858
Shear capacity	Ps0 (kN)	3,446	3,446	34,201	34,201
Decision formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture morphology		Flexural fracture type	Flexural fracture type	Flexural fracture type	Flexural fracture type
Checking by the seismic horizontal load bearing capacity method					
Safety factor	α	1.5	1.5	1.5	1.5
Allowable plasticity	μa	1.192	1.205	1.418	1.557
Structure characteristic correction factor	Ccs	0.850	0.842	0.738	0.688
Regional correction factor	cz	1.00	1.00	1.00	1.00
Standard value of design Horizontal Seismic Coefficient	khc0	1.200	1.200	1.200	1.200
Design horizontal seismic coefficient	khc	1.02	1.01	0.89	0.83
Horizontal Seismic Coefficient kha designed for horizontal load bearing capacity during earthquakes		1.21	1.10	0.95	0.93
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	17.9	8.2	9.3
Maximum response plasticity	μrT	0.993	1.093	1.303	1.325
Allowable residual displacement	δRa (mm)	110.5	110.5	111.5	111.5
Result of checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team

Table 7.3.97 Results of Lateral Load Carrying Capacity Analysis (No. 3 Column, Pier; Flow Direction)

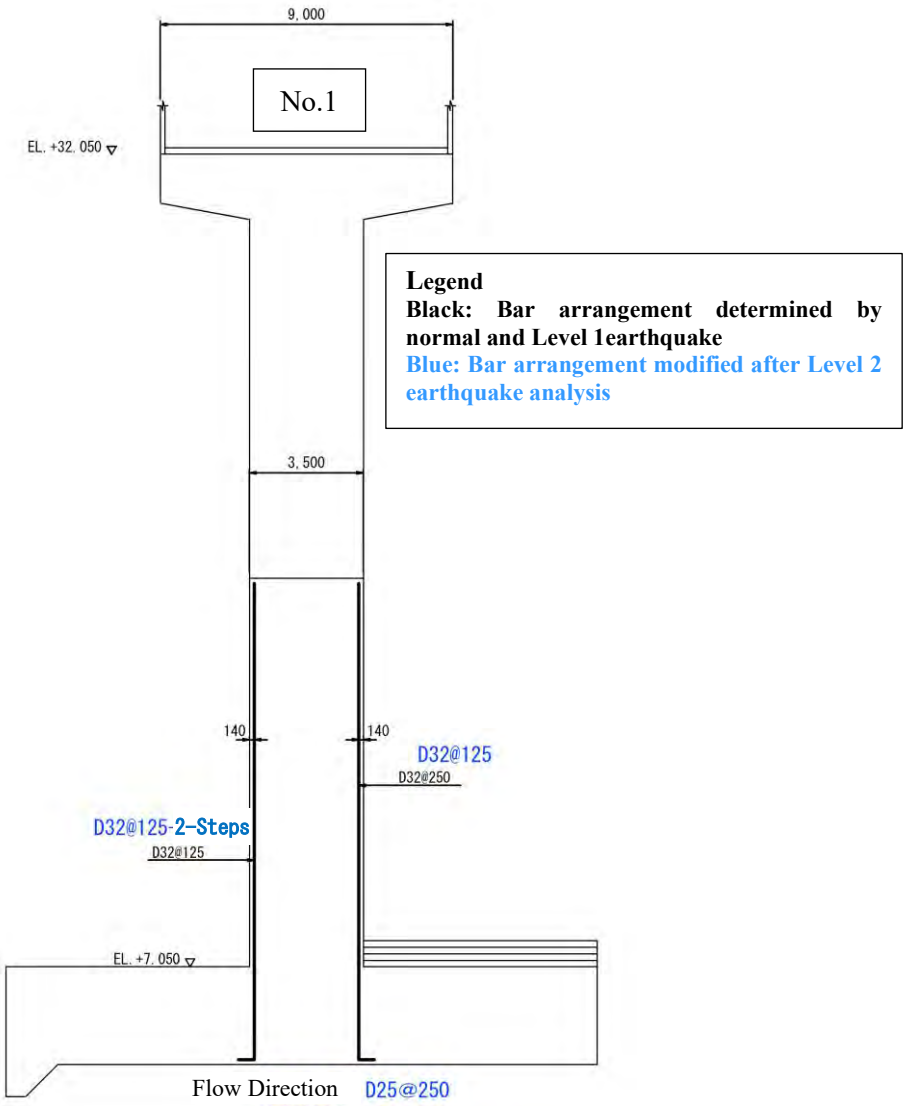
Checking Direction		Flow direction			
		Positive direction	Negative direction	Positive direction	Negative direction
Main plasticizing member		Column	Column	Pier	Pier
Determination of fracture mode					
Response shear force	S (kN)	6,496	6,485	60,286	60,303
Shear capacity	Ps (kN)	9,875	9,875	81,197	81,197
Shear capacity	Ps0 (kN)	10,445	10,445	83,339	83,339
Decision formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture morphology		Flexural fracture type	Flexural fracture type	Flexural fracture type	Flexural fracture type
Checking by the seismic horizontal load bearing capacity method					
Safety factor	α	1.5	1.5	1.5	1.5
Allowable plasticity	μa	1.426	1.425	1.001	1.001
Structure characteristic correction factor	Ccs	0.735	0.735	0.999	0.999
Regional correction factor	cz	1.00	1.00	1.00	1.00
Standard value of design Horizontal Seismic Coefficient	khc0	1.200	1.200	1.200	1.200
Design horizontal seismic coefficient	khc	0.88	0.88	1.20	1.20
Horizontal Seismic Coefficient kha designed for horizontal load bearing capacity during earthquakes		1.25	1.25	2.00	2.00
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	0.958	0.959	0.680	0.680
Allowable residual displacement	δRa (mm)	110.5	110.5	111.5	111.5
Result of checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team

Table 7.3.98 Results of Lateral Load Carrying Capacity Analysis (No. 3 Column, Pier; Perpendicular to Flow)

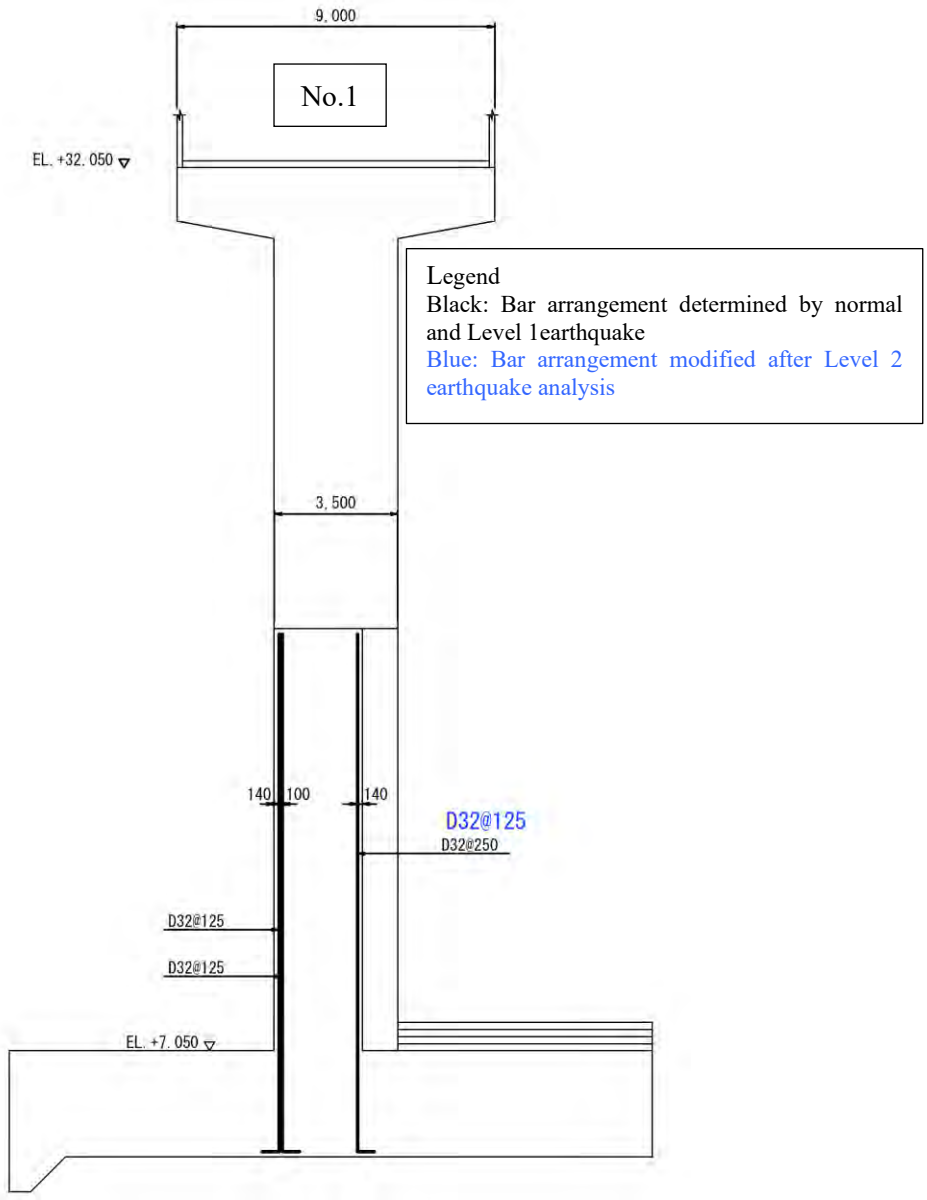
Checking Direction		Direction perpendicular to flow			
		Positive direction	Negative direction	Positive direction	Negative direction
Main plasticizing member		Column	Column	Pier	Pier
Determination of fracture mode					
Response shear force	S (kN)	6,738	6,351	23,387	54,929
Shear capacity	Ps (kN)	6,789	6,789	81,742	81,742
Shear capacity	Ps0 (kN)	7,217	7,217	84,611	84,611
Decision formula		S ≦ Ps	S ≦ Ps	S ≦ Ps	S ≦ Ps
Fracture morphology		Flexural fracture type	Flexural fracture type	Flexural fracture type	Flexural fracture type
Checking by the seismic horizontal load bearing capacity method					
Safety factor	α	1.5	1.5	1.5	1.5
Allowable plasticity	μa	1.785	1.454	1.738	1.433
Structure characteristic correction factor	Ccs	0.624	0.724	0.636	0.732
Regional correction factor	cz	1.00	1.00	1.00	1.00
Standard value of design Horizontal Seismic Coefficient	khc0	1.200	1.200	1.200	1.200
Design horizontal seismic coefficient	khc	0.75	0.87	0.76	0.88
Horizontal Seismic Coefficient kha designed for horizontal load bearing capacity during earthquakes		1.17	1.27	0.93	1.18
Result of checking		OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)	OK (khc ≦ kha)
Checking of residual displacement					
Residual displacement	δR (mm)	8.2	0.0	7.4	0.3
Maximum response plasticity	μrT	1.027	0.946	1.334	1.014
Allowable residual displacement	δRa (mm)	110.5	110.5	111.5	111.5
Result of checking		OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)	OK (δRa ≧ δR)

Source: Study Team



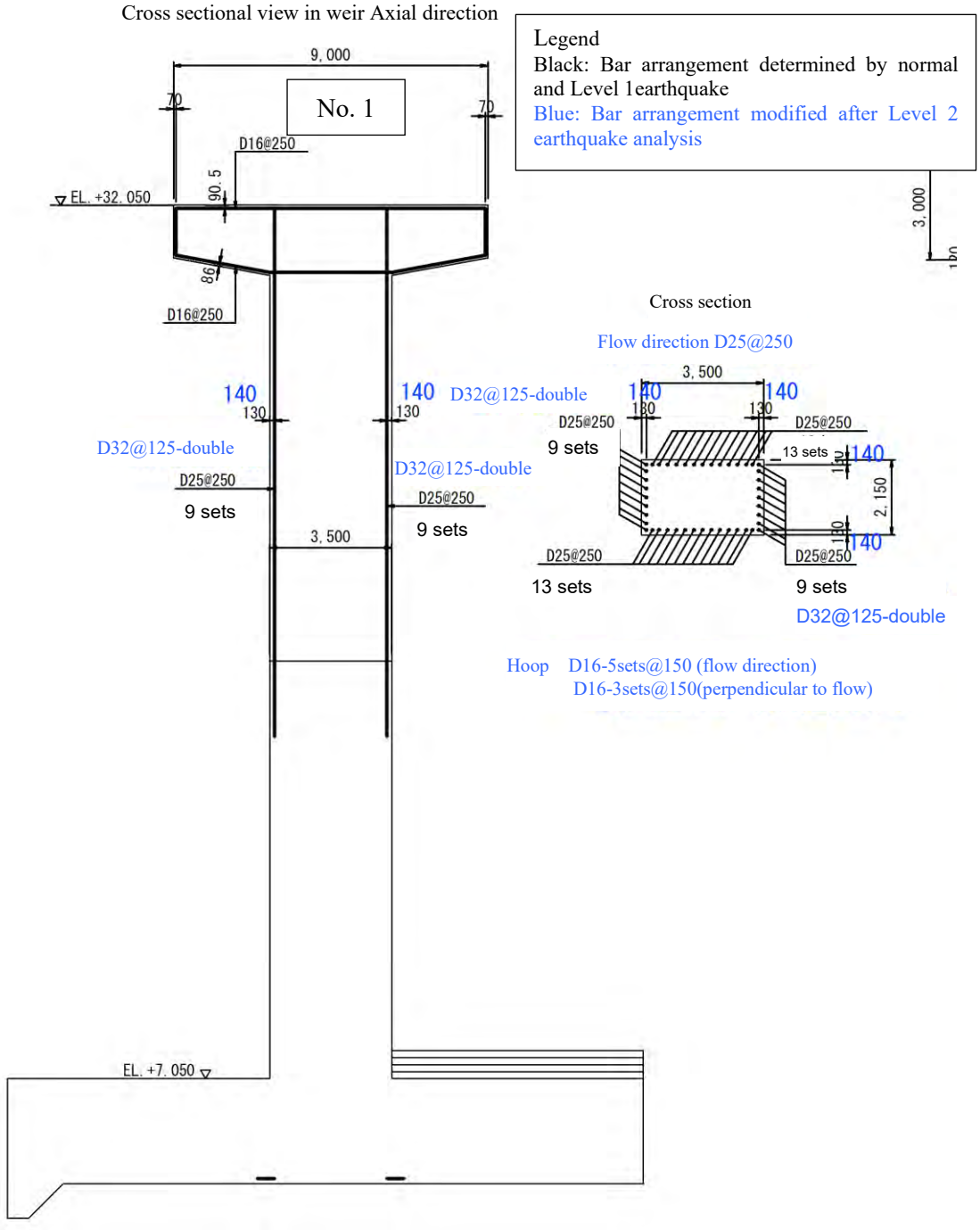
Source: Study Team

Figure 7.3.65 Bar Arrangement of No. 1 Pier (Standard Part)



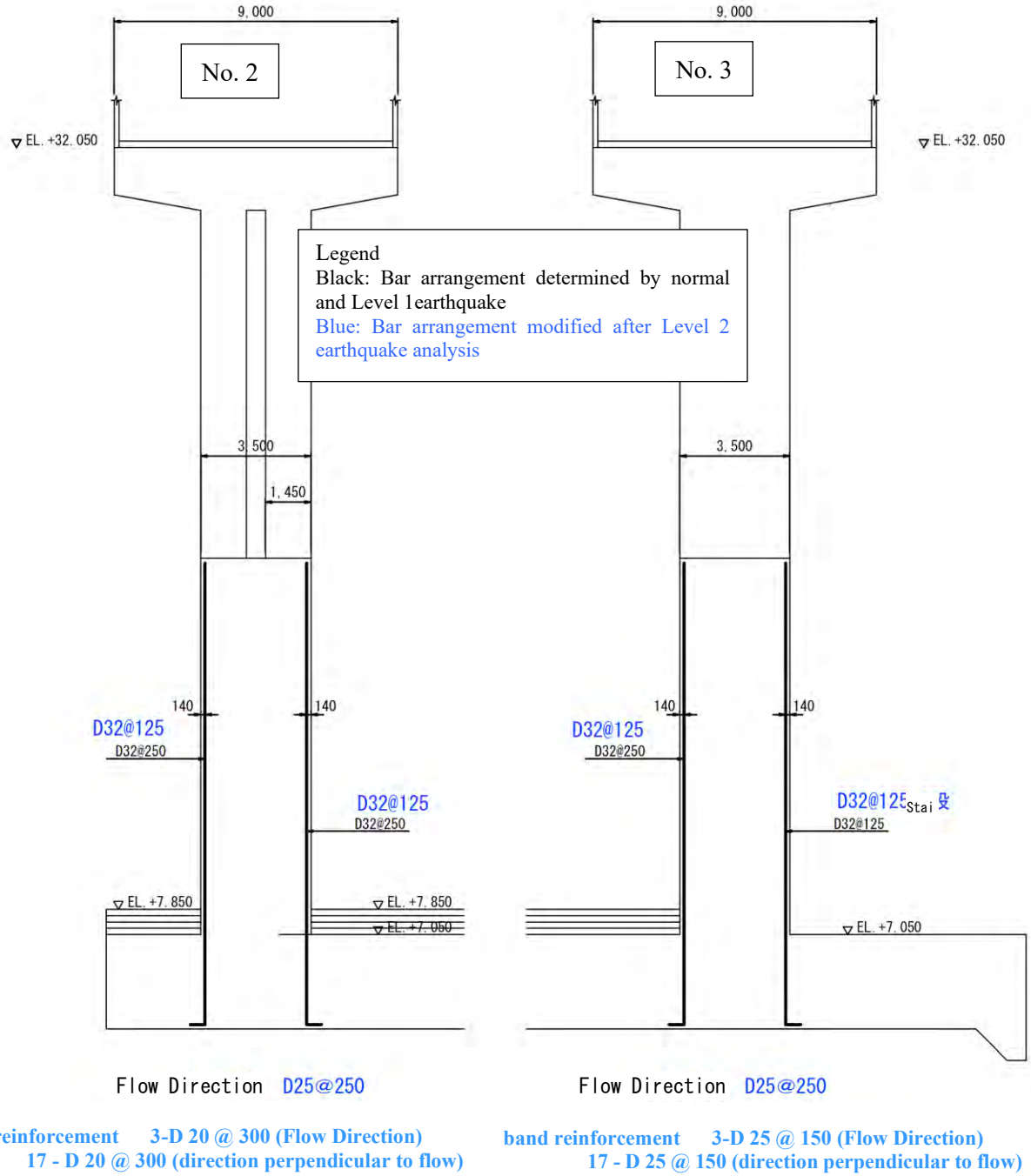
Source: Study Team

Figure 7.3.66 Bar Arrangement of No. 1 Pier (Gate Part)



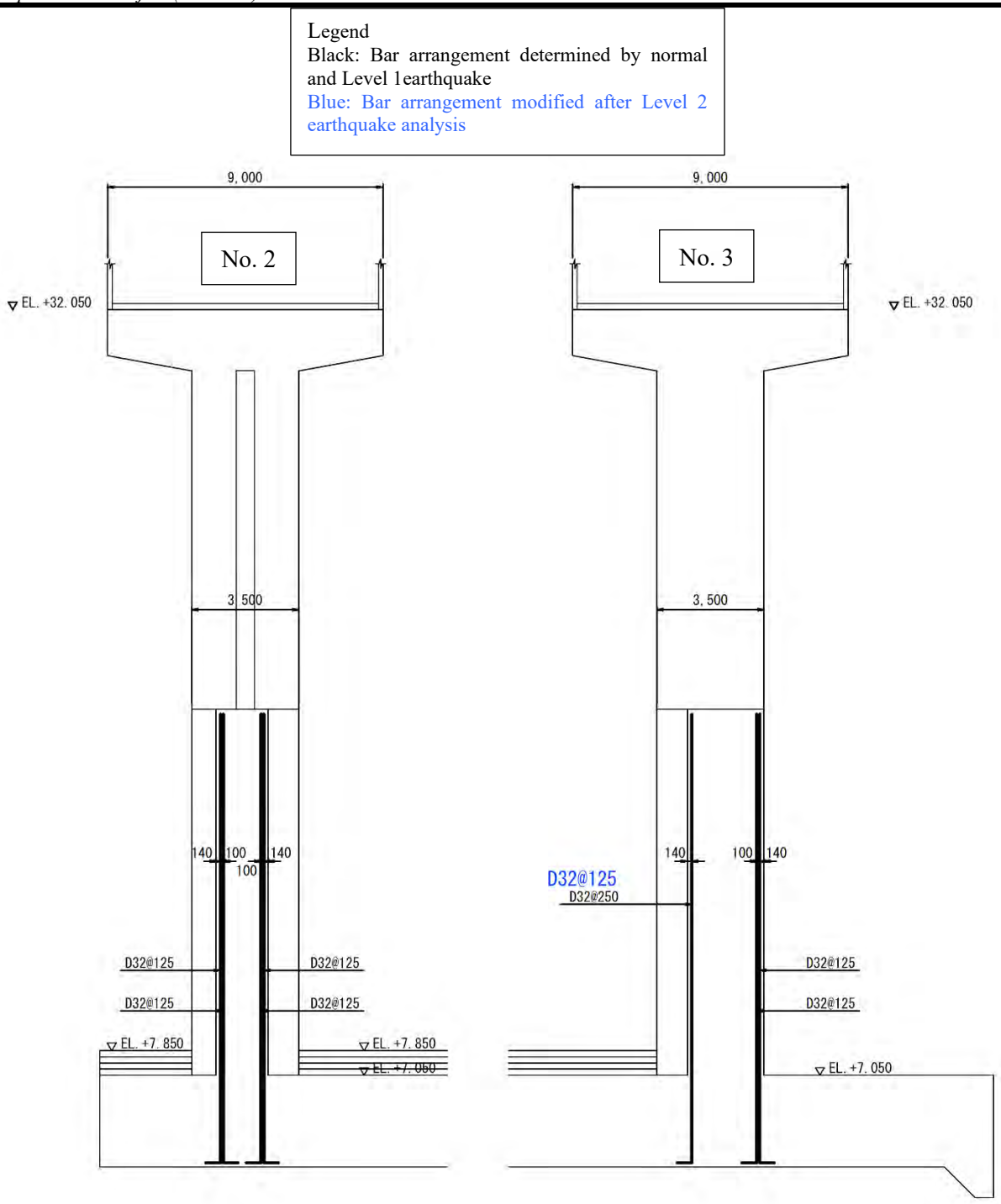
Source: Study Team

Figure 7.3.67 Bar Arrangement of No. 1 Pier (Column)



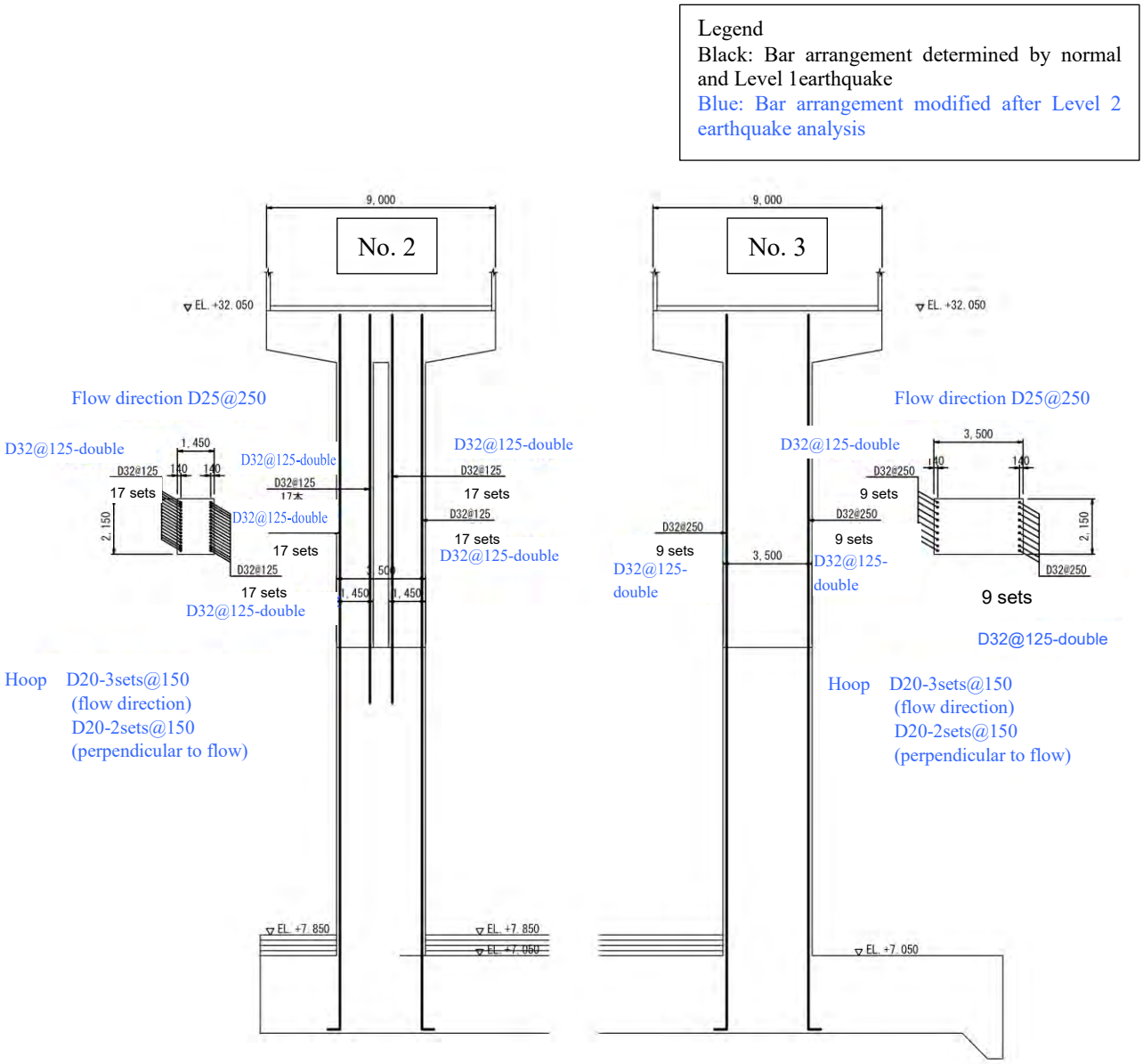
Source: Study Team

Figure 7.3.68 Bar Arrangement of No. 2 To No. 3 Piers (Standard Part)



Source: Study Team

Figure 7.3.69 Bar Arrangement of No. 2 To No. 3 Piers (Gate)



Source: Study Team

Figure 7.3.70 Bar Arrangement of No. 2 To No. 3 Piers (Column)

(6) Result of Level 2 Seismic Analysis on Foundation

1) Summary

The analytical model diagram of No.1 Part is **Figure 7.3.71,** similarly, that of No.2 and No.3 Part is **Figure 7.3.72**.

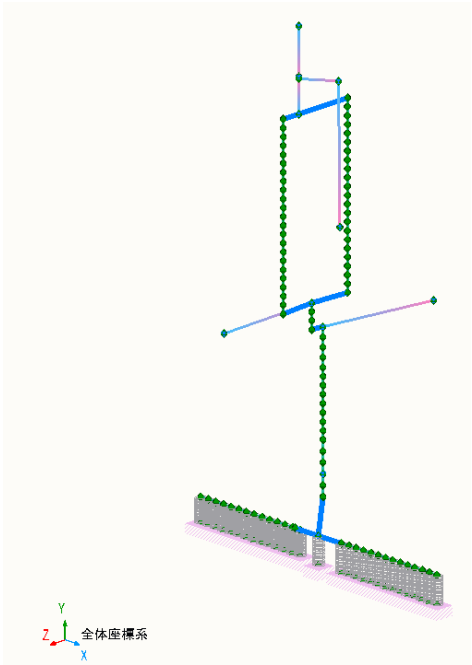
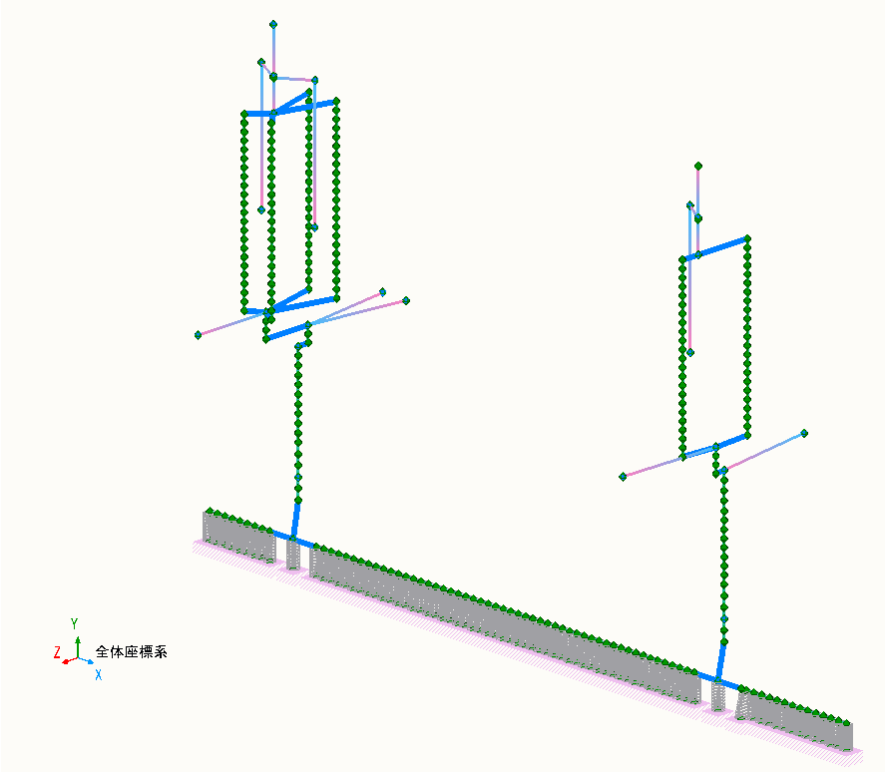


Figure 7.3.71 Analytical Model Diagram of Bottom Slab (No. 1)



Source: Study Team

Figure 7.3.72 Analytical Model Diagram of Bottom Slab (2 and 3)

2) Analysis Case

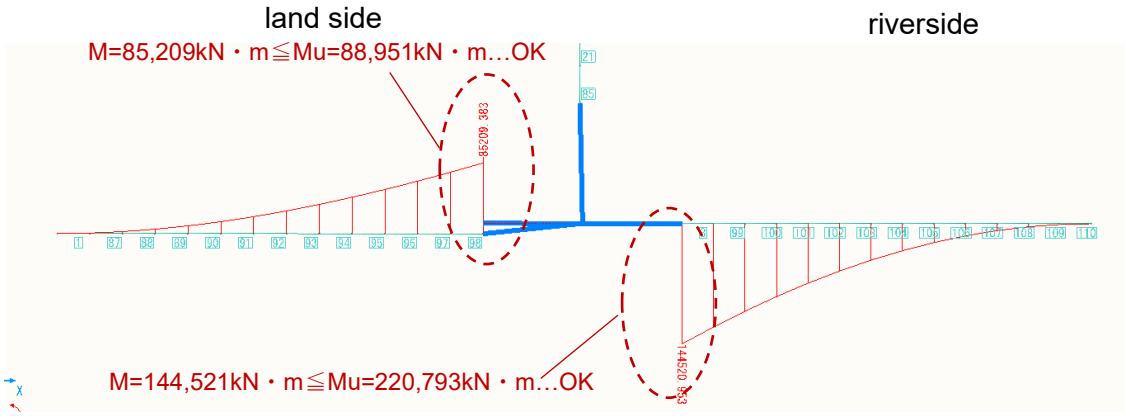
The analysis cases are described as follows.

- Direction perpendicular to flow (Left bank to right bank)
- Direction perpendicular to flow (Right bank to Left bank)

Analysis in the flow direction is excluded because the effective height of the Bottom Slab is large that has sufficient strength.

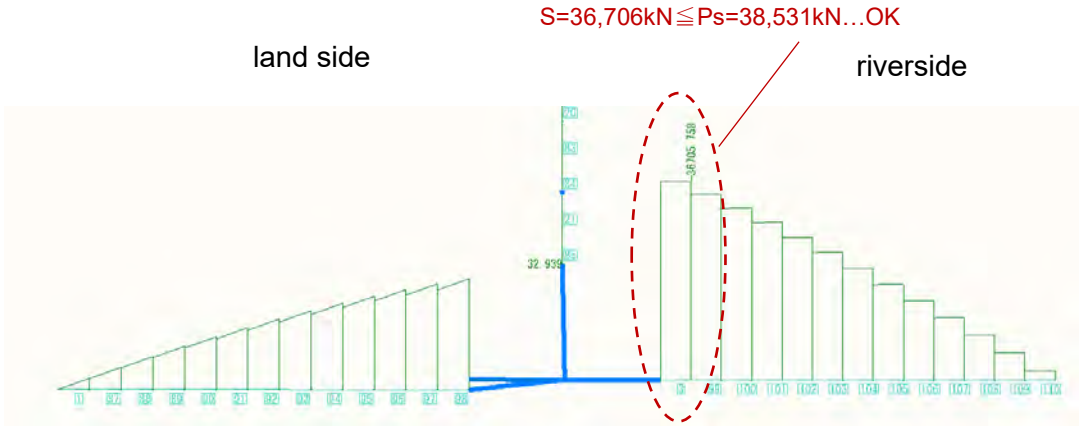
3) Result of Analysis

The cross-sectional force distribution diagrams are shown below. The generated force was checked under the bar arrangement conditions illustrated in **Figure7.3.81** to **Figure 7.3.84**. It was confirmed that the structures have enough seismic performance because the strength exceeded the generated force for both bending and shearing.



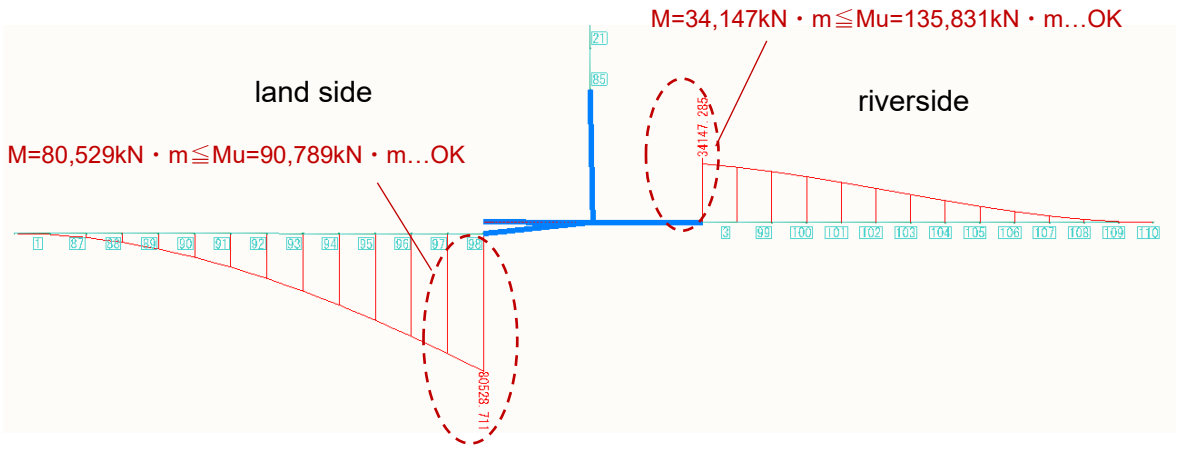
Source: Study Team

Figure 7.3.73 Bending Moment Distribution
(No. 1 Pier Bottom Slab (Load acts from left to right))



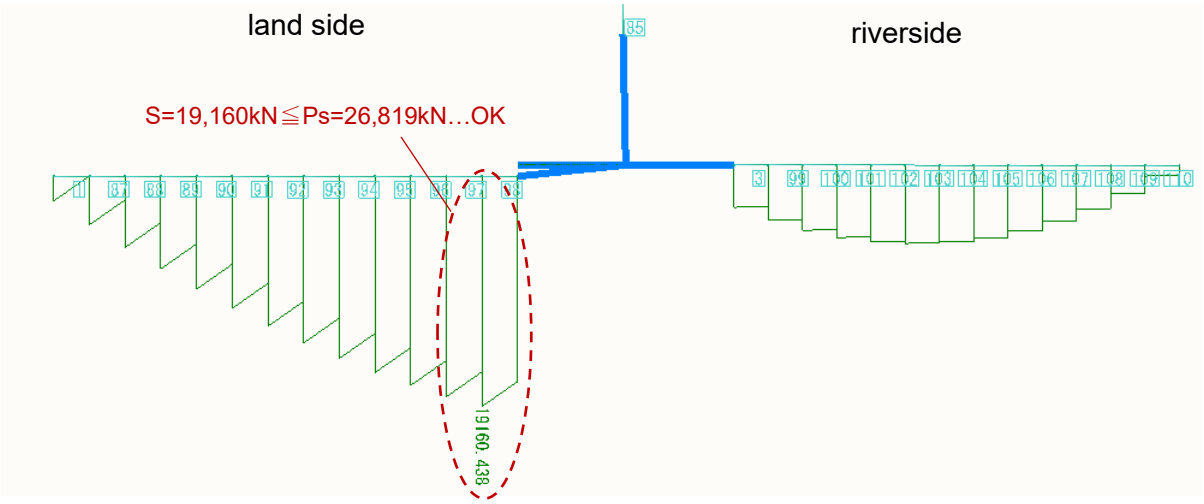
Source: Study Team

Figure 7.3.74 Shear Force Distribution
(No. 1 Pier Bottom Slab (Load acts from left to right))



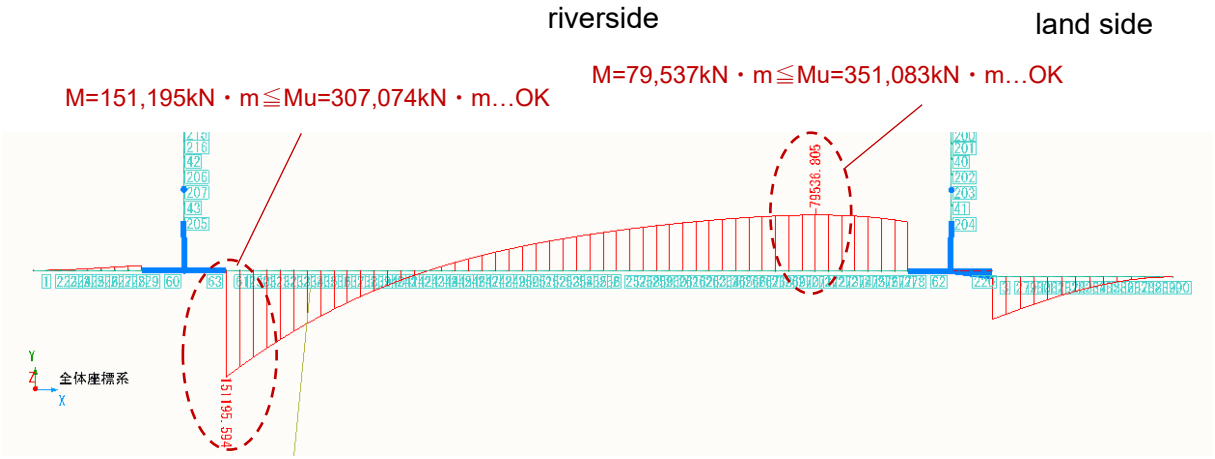
Source: Study Team

Figure 7.3.75 Bending Moment Distribution
(No. 1 Pier Bottom Slab (Load acts from right to left))



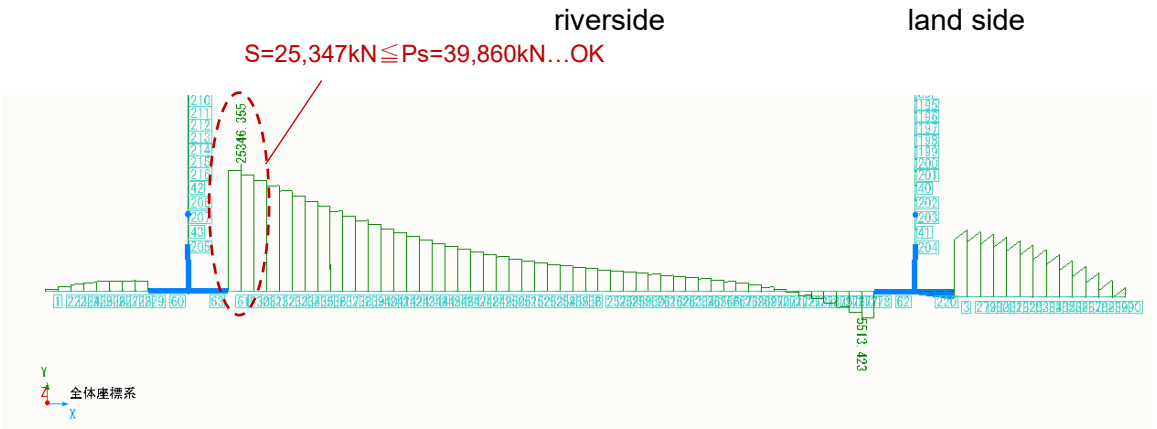
Source: Study Team

Figure 7.3.76 Shear Force Distribution
(No. 1 Pier Bottom Slab (Load acts from right to left))



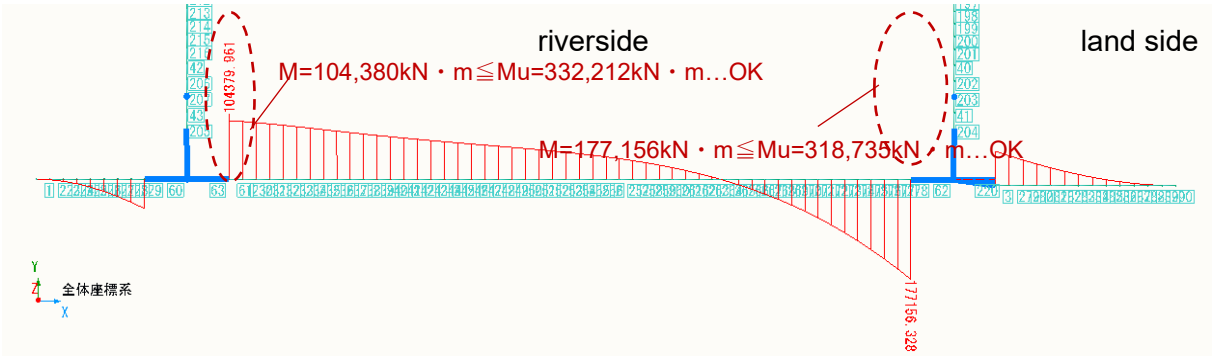
Source: Study Team

Figure 7.3.77 Bending Moment Distribution
(Bottom Slab of No. 2 and No. 3 Piers (Load acts from left to right))



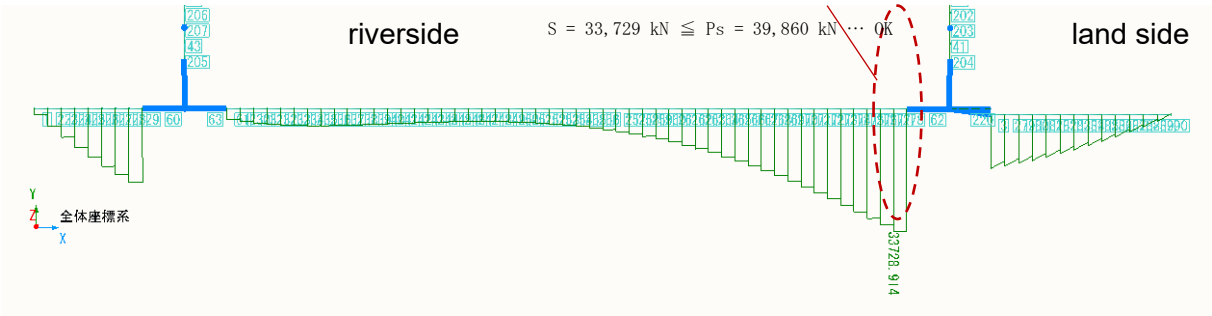
Source: Study Team

Figure 7.3.78 Shear Force Distribution
(Bottom Slab of No. 2 and No. 3 Piers (Load acts from left to right))



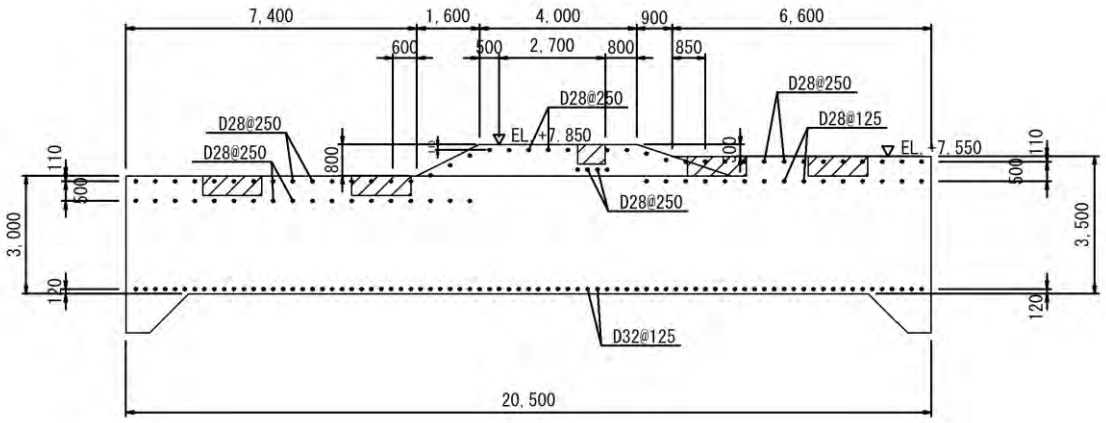
Source: Study Team

Figure 7.3.79 Bending Moment Distribution
(Bottom Slab of No. 2 and No. 3 Piers (Load acts from right to left))



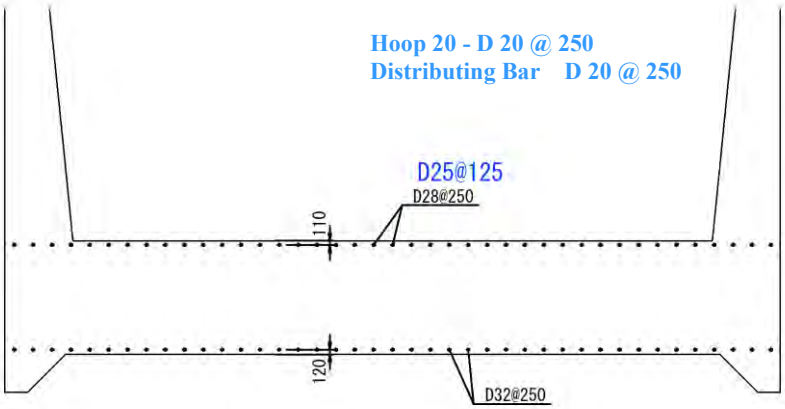
Source: Study Team

Figure 7.3.80 Shear Force Distribution
(Bottom Slab of No. 2 and No. 3 Piers (Load acts from right to left))



Source: Study Team

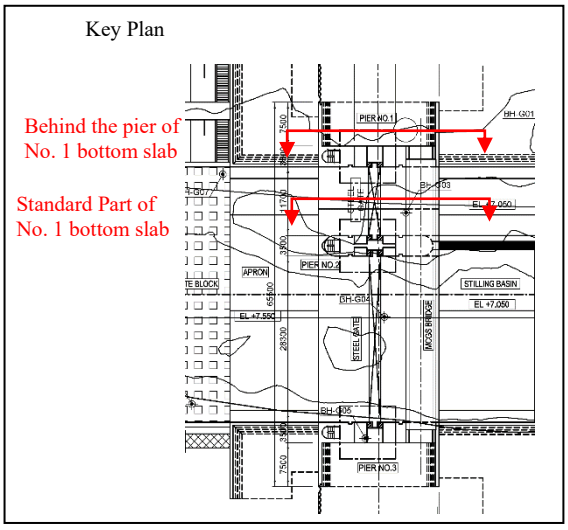
Figure7.3.81 Bar Arrangement of No.1 Bottom Slab Standard Part

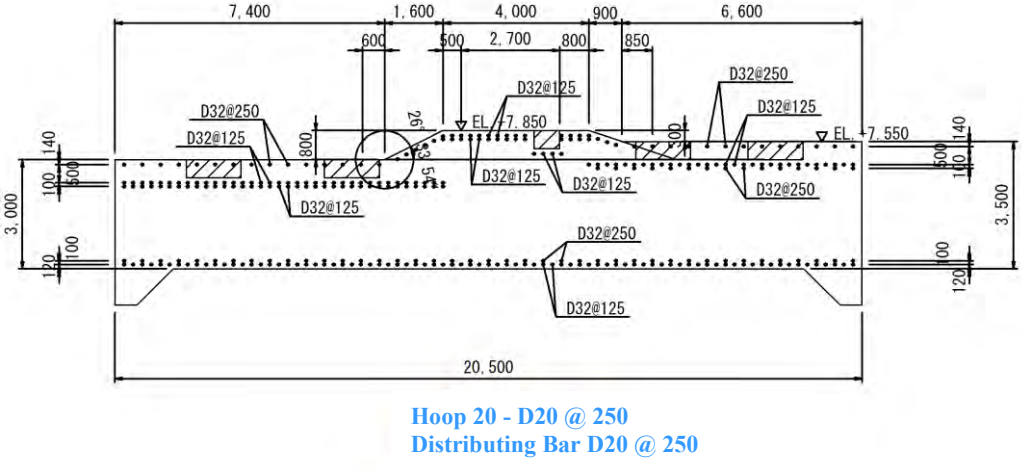


Source: Study Team

Figure 7.3.82 Bar Arrangement of No.1 Bottom Slab Behind the Pier

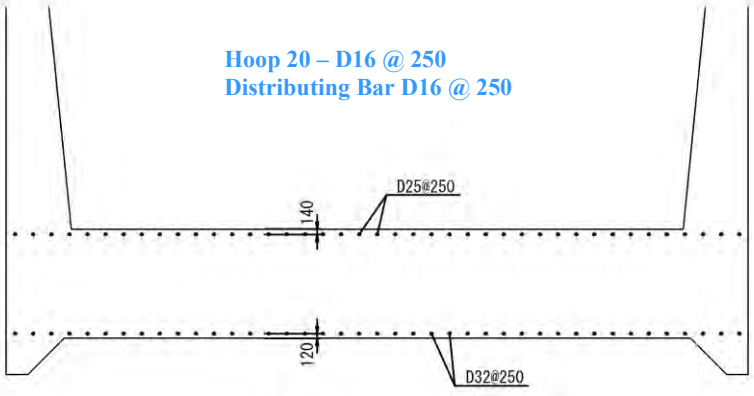
Legend
Black: Bar arrangement determined by normal and Level 1earthquake
Blue: Bar arrangement modified after Level 2 earthquake analysis





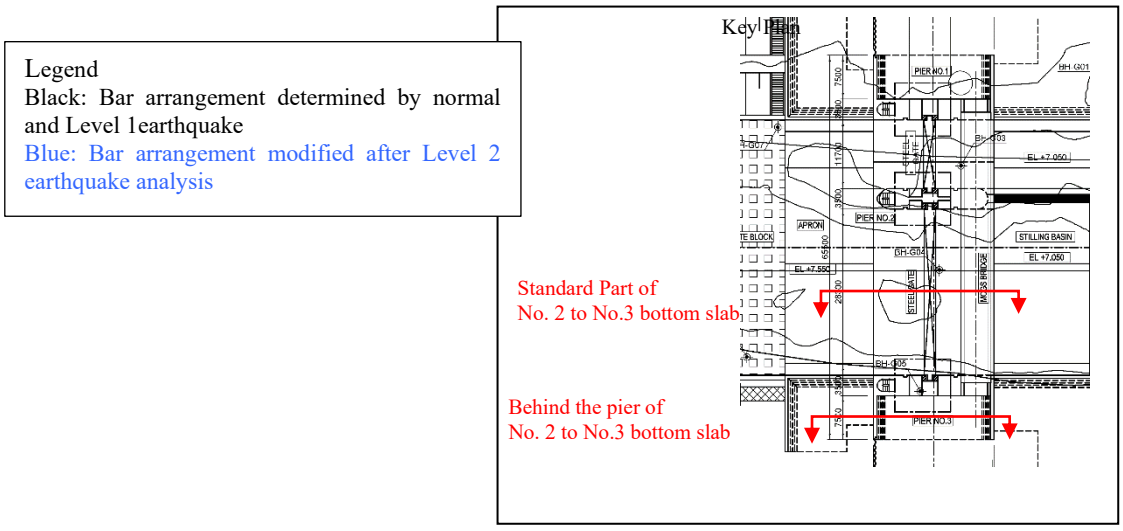
Source: Study Team

Figure 7.3.83 Bar Arrangement of No.2 to No.3 Bottom Slab Standard Part



Source: Study Team

Figure 7.3.84 Bar Arrangement of No.2 to No.3 Bottom Slab Behind the Pier



7.3.2.5 Maintenance Bridge

The following Table is the structural calculation results of the MCGS maintenance bridge determined in the basic design.

(1) Short Span: L = 14.40 m

1) Design of the Main Girder

DGCS 12.6.3 **Stresses Limitations for Prestressing Tendons,**

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars (MPa)	Tendon Type: Low Relaxation Strand (MPa)	Tendon Type: Deformed High-Strength Bars (MPa)
Pretensioning			
Immediately prior to transfer *(fpbt)*	1302.00	1395.00	-
At service limit state after all losses *(fpe)*	1339.20	1339.20	1339.20
Post-Tensioning			
Prior to seating-short-term *fpbt* may be allowed	1506.60	1506.60	1506.60
At anchorages and couplers immediately after anchor set	1302.00	1302.00	1302.00
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	1302.00	1376.40	1302.00
At service limit state after losses *(fpe)*	1339.20	1339.20	1339.20

DGCS 12.6.4.1 **For Temporary Stresses Before Losses-Fully Prestressed Components,**

Compression Stresses, 0.60 fci = 21.00 MPa

DGCS 12.6.3.1-1 **Tension Stresses,**

Bridge Type	Location	Stress Limit (MPa)
Other Than Segmentally-Constructed Bridges	In precompressed tensile zone without bonded reinforcement	N/A
	In areas other than the precompressed tensile zone and without bonded reinforcement	1.38
	In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.50 fy, not to exceed 210 MPa.	3.73
	For handling stresses in prestressed piles	2.46
Segmentally-Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	
	Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of 0.5 fy, with internal tendons or external tendons	1.48
	Joints without minimum bonded auxiliary reinforcement through the joints	0.00
	Transverse Stresses Through Joints For any type of joint	1.48
	Stresses in Other Areas	
	For areas without bonded non-prestressed reinforcement	0.00
	In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.50 fy, not to exceed 210 MPa	2.96
	Principal Tensile Stress at Neutral Axis in Web	
	All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures	1.71

DGCS 12.6.4.2 **For Stresses at Service Limit State after Losses-Fully Prestressed Components,**

DGCS 12.6.4.2-1 **Compression Stresses,**

Location		Stress Limit (MPa)
In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	Girder	20.25
	Deck	12.60
In segmentally constructed bridges due to the sum of effective prestress and permanent loads	Girder	20.25
	Deck	12.60
Due to the sum of effective prestress, permanent loads, and transient loads and during shipping and handling	Girder	27.00
	Deck	16.80

DGCS 12.6.4.2-2 **Tension Stresses,**

Bridge Type	Location	Stress Limit (MPa)
Other Than Segmentally-Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	3.35
	For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	1.68
	For components with unbonded prestressing tendons	0.00
Segmentally Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	
	Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 fy, internal tendons or external tendons	1.68
	Joints without the minimum bonded auxiliary reinforcement through joints	0.00
	Transverse Stresses Through Joints Tension in the transverse direction in precompressed tensile zone	1.68
	Stresses in Other Areas	
	For areas without bonded reinforcement	0.00
	In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 fy, not to exceed 205 Mpa.	3.35
	Principal Tensile Stress at Neutral Axis in Web	
	All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures.	1.94

Estimate of Required Number of Strands,

Estimated Center of Gravity of Strands from Bottom of Girder, = 130.00 mm

= 130.00 mm

Limit Cracking due to Tensile Stress,

Tensile Stress Limit for Concrete at Service Load, = 3.35 MPa

Required Pre-Compressive Stress at the Bottom Fiber, = 1.11 MPa

Required Effective Prestressing Force, $P = \frac{\left|\left(\frac{M}{St}\right) - \sigma\right|}{\left[\left(\frac{e}{St}\right) + \frac{1}{A}\right]}$ P = 1014.86 KN

No Tensile Stress during Service Load,

Required Effective Prestressing Force, $P = \frac{\left|\left(\frac{M}{Sb}\right)\right|}{\left[\left(\frac{e}{Sb}\right) + \frac{1}{A}\right]}$ P = 848.52 KN

Estimated Prestress Loss vs. Actual Prestress Loss,

Estimated Percentage of Prestress Loss, = 36.67358 %

Actual Percentage of Prestress Loss, = 36.67358 %

Estimated vs. Actual Percentage of Prestress Loss, = 0.00000 %

Required Number of Strands,

Estimated Effective Jacking Force, = 1602.59 KN

Allowable Stress for Prestressing Steel near Anchorage,

Estimated Area of Prestressing Steel, = 1230.87 mm^2

Estimated Number of Strands Required, = 12.47 strands

= 13.00 strands

Number of Strands, = 30.00 strands

Number of Identical Tendons, N = 3.00

Pattern of Strands at Midspan

Row	Centroid yb	No. of Strand n	n*yb	e_{mid}
1	190.00	10.00	1900.00	438.240
2	100.00	10.00	1000.00	528.240
3	100.00	10.00	1000.00	528.240
4	0.00	0.00	0.00	0.000
5	0.00	0.00	0.00	0.000
6	0.00	0.00	0.00	0.000
7	0.00	0.00	0.00	0.000
8	0.00	0.00	0.00	0.000
9	0.00	0.00	0.00	0.000
10	0.00	0.00	0.00	0.000
TOTAL		30.00	3900.00	

Computed Centroid at Mid-Span vs. Estimated Centroid at Mid-Span,

Computed Centroid at Mid-Span, $y_{incomputed}$ = 130.00 mm

Estimated Centroid at Mid-Span, $y_{inestimated}$ = 130.00 mm

Relative Error, %error = 0.00 %

Computed Total Area of Prestressing Steel, Aps = 2961.00 mm^2

Computed Eccentricity at Mid-Span, $e_{incomputed}$ = 498.24 mm

PRESTRESS LOSS,

Summary of Losses,

Ducts	Jacking Stage		
	Po (N)	Aps	fpj
1	3858000.00	987.00	1674.00
2	3858000.00	987.00	1674.00
3	3858000.00	987.00	1674.00
4	0.00	0.00	0.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
Whole Girder	11574000.00	2961.00	1674.00

Ducts	Total Prestress Losses at Transfer		
	Δfpa (MPa)	Δfpf (MPa)	ΔfpES (MPa)
1	134.06	37.55	17.69
2	134.06	31.79	17.69
3	134.06	21.26	17.69
4	0.00	0.00	0.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
Whole Girder	134.06	37.55	53.08

Total Prestress Loss at Service,

Ducts	Prestress Loss between Time of Transfer and Deck Placement		
	ΔfpSR (MPa)	ΔfpCR (MPa)	ΔfpR1 (MPa)
1	17.24	42.85	15.26
2	17.24	42.85	15.48
3	17.24	42.85	15.90
4	0.00	0.00	0.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
Whole Girder	51.73	128.55	15.90

Ducts	Prestress Loss between After Deck Placement			
	ΔfpSD (MPa)	ΔfpCD (MPa)	ΔfpR2 (MPa)	ΔfpSS (MPa)
1	23.45	40.22	15.26	-4.60
2	23.45	40.21	15.48	-4.60
3	23.45	40.18	15.90	-4.60
4	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00
6	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00
8	0.00	0.00	0.00	0.00
9	0.00	0.00	0.00	0.00
10	0.00	0.00	0.00	0.00
Whole Girder	70.34	120.61	15.90	-13.81

Ducts	Pe
	(KN)
1	1,317,672.07
2	1,322,921.31
3	1,332,506.21
4	-
5	-
6	-
7	-
8	-
9	-
10	-
Whole Girder	3,973,099.59

TABLE SUMMARY OF TENSION FORCES,

Cable No.	Jacking Force (Po) (N)	Effective Prestressing Force (Pe) (N)
1	3,858,000.00	1,317,672.07
2	3,858,000.00	1,322,921.31
3	3,858,000.00	1,332,506.21

		Jacking Stress per Strand,		=	3908.81	MPa	
		Total Prestress Loss at Transfer,		=	224.69	MPa	
		Total Prestress Loss at Service,		=	389.22	MPa	
		Stress per Strand at Transfer,		=	3684.12	MPa	
		Stress per Strand at Service,		=	3294.90	MPa	
		Total Prestress Losses at Service between Time of Transfer and Deck Placement,					
		$\Delta fpId = \Delta fpSR + \Delta fpCR + \Delta fpR1$	Δfpid	=	196.18	MPa	
		Total Prestress Losses at Service After Deck Placement,					
		$\Delta fpdf = \Delta fpSD + \Delta fpCD + \Delta fpR2 - \Delta fpSS$	Δfpdf	=	193.04	MPa	
		Total Prestress Loss,		=	613.92	MPa	
		Percentage of Total Losses,		=	36.67	%	
		Check for Prestressing Limit,					
		Allowable Prestress Limit at Service,		=	1339.20	MPa	
		Stress in Strands after Losses,		=	1060.08	MPa	**OK!**
		Final Effective Prestressing Force after Losses,		=	3138.91	KN	
DGCS 12.6.5.2		**Instantaneous Losses,**					
		Anchorage Set,	Δfpa	=	134.06	MPa	
		Friction Losses,					
DGCS 12.6.5.2-1		Between the Prestressing Tendons and the Duct Wall, $\Delta fpF = fpj(1 - e^{-(Kx+\mu\alpha)})$					
DGCS 12.6.5.2-2		Between the External Tendons across a Single Deviator Pipe, $\Delta fpF = fpj(1 - e^{-\mu'\alpha - 0.04})$					
		Wobble Friction,	K	=	6.60E-07		
		Coefficient of Friction,	μ	=	0.23		

Stress in Prestressing Steel During Jacking,

Duct	Jacking Force (Pj)	No. of Strands (N)	Jacking Stress (fpj)	
1	3,858,000.00	10.00	YIELD	1674.00
2	3,858,000.00	10.00	YIELD	1674.00
3	3,858,000.00	10.00	YIELD	1674.00
4	0.00	0.00		
5	0.00	0.00		
6	0.00	0.00		
7	0.00	0.00		
8	0.00	0.00		
9	0.00	0.00		
10	0.00	0.00		
TOTAL	**11,574,000.00**			

Sum of Absolute Values of Angular Change of Prestressing Steel,

Duct	Centroid at Ends	Centroid at Mid	SAG	x	a
1	900.00	190.00	710.00	7.247	0.099
2	700.00	100.00	600.00	7.233	0.083
3	500.00	100.00	400.00	7.215	0.056
4	0.00	0.00	0.00		
5	0.00	0.00	0.00		
6	0.00	0.00	0.00		
7	0.00	0.00	0.00		
8	0.00	0.00	0.00		
9	0.00	0.00	0.00		
10	0.00	0.00	0.00		

Computed Prestress Loss due to Friction,

Duct	Jacking Stress (fpj)	$1 - e^{-(Kx+\mu\alpha)}$	Friction Loss (Δfpf)
1	1674.00	0.022	37.55
2	1674.00	0.019	31.79
3	1674.00	0.013	21.26
4			
5			
6			
7			
8			
9			
10			

DGCS 12.6.5.2-4		Elastic Shortening, $\Delta fpES = \frac{N-1}{2N}\frac{Ep}{Eci} fcgp$					
		Sum of Concrete Stresses at Center of Gravity of Tendon after Transfer, $fcgp = \left(\frac{Pi}{A}\right) + \left[\frac{(Pi \cdot ec^2)}{I}\right] \left[\frac{(Mg \cdot ec)}{I}\right]$	fcgp	=	24.18	MPa	
		NOTE :					
		fcgp values may be calculated using a steel stress reduced below the initial value by a margin					
		dependent on elastic shortening, relaxation, and friction effects,					
		Estimated Prestress Loss,	$Loss_{est}$	=	1.313721		
		Eccentricity of Strands from Bottom of Girder at Midspan,	ec	=	550.00	mm	
		Total Prestressing Force at Release (w/ Anchorage Loss),	Pi	=	1.20E+07	N	
		Moment due to Selfweight at Midspan,	Mg	=	702.77	KN-m	
		Computed Loss due to Elastic Shortening vs. Estimated Loss,					
		Computed Loss,	ΔfpES	=	53.08	MPa	
		Percentage of Loss,	$Loss_{per}$	=	1.313721	%	
		Relative Error,	$Error_{rel}$	=	0.00		**OK!**

DGCS 12.6.5.4 **Refined Estimates of Time-Dependent Losses,**

DGCS 12.6.5.4-1 The change in prestressing steel stress due to time dependent loss, ΔfpLT, shall be determined as follows :

$$\Delta fpLT = (\Delta fpSR + \Delta fpCR + \Delta fpR1)_{id} + (\Delta fpSD + \Delta fpCD + \Delta fpR2 - \Delta fpSS)_{df}$$

where :

ΔfpSR	=	prestress loss due to shrinkage of girder concrete between transfer and deck placement (Mpa)
ΔfpCR	=	prestress loss due to creep of girder concrete between transfer and deck placement (MPa)
ΔfpR1	=	prestress loss due to relaxation of prestressing strands between time of transfer and deck placement (MPa)
ΔfpR2	=	prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (MPa)
ΔfpSD	=	prestress loss due to shrinkage of girder concrete between time of deck placement and final time (MPa)
ΔfpCD	=	prestress loss due to creep of girder concrete between time of deck placement and final time (MPa)
ΔfpSS	=	prestress gain due to shrinkage of deck in composite section (MPa)
$(\Delta fpSR + \Delta fpCR + \Delta fpR1)_{id}$	=	sum of time-dependent prestress losses between transfer and deck placement (MPa)
$(\Delta fpSD + \Delta fpCD + \Delta fpR2 - \Delta fpSS)_{df}$	=	sum of time-dependent prestress losses after deck placement (MPa)

Accounts for Losses and Gains between Time of Deck Placement to Final Time,

Duct	ΔfpSR + ΔfpCR + ΔfpR1	ΔfpSD + ΔfpCD + ΔfpR2 - ΔfpSS
1	75.35	112.01
2	75.58	112.22
3	75.99	112.62
4	0.00	0.00
5	0.00	0.00
6	0.00	0.00
7	0.00	0.00
8	0.00	0.00
9	0.00	0.00
10	0.00	0.00

Losses: Time of Transfer to Time of Deck Placement,

Shrinkage of Girder Concrete,

Prestress Loss due to Shrinkage of Girder Concrete between Transfer and Deck Placement, ΔfpSR = 51.73 MPa

DGCS 12.6.5.4-2 $\Delta fpSR = \varepsilon_{bid} E_p K_{id}$

in which :

Transformed Section Coefficient that Accounts for Time-Dependent interaction between Concrete and Bonded Steel in the Section being Considered for Time Period between Transfer and Deck Placement, Kid = 0.87

DGCS 12.6.5.4-3 $$K_{id} = \frac{1}{1 + \frac{E_p A_{ps}}{E_{ci} A_g}\left(1 + \frac{A_g e_{pg}^2}{I_g}\right)\left[1 + 0.7\varphi_b(t_f, t_i)\right]}$$

where :

Concrete Shrinkage Strain of Girder between the Time of Transfer and Deck Placement, εbid = 0.000303

Eccentricity of Prestressing Force with Respect to Centroid of Girder; Positive in Common Construction where it is Below Girder Centroid, epg = 498.24 mm

Girder Creep Coefficient at Final Time due to Loading Introduced at Transfer, Ψb(tf,ti) = 1.25

Final Age tf = 9000.00 day/s

Age at Transfer ti = 3.00 day/s

Creep of Girder Concrete,

Prestress Loss due to Creep of Girder Concrete between Transfer and Deck Placement, ΔfpCR = 128.55 MPa

DGCS 12.6.5.4-4 $$\Delta fpCR = \frac{E_p}{E_{ci}} f_{cgp} \varphi_b(t_f, t_i) K_{id}$$

where :

Girder Creep Coefficient at Time of Deck Placement due to Loading Introduced at Transfer, Ψb(td,ti) = 0.93

Age at Deck Placement (days), td = 120.00 day/s

Relaxation of Prestressing Strands,

Prestress Loss due to Relaxation of Prestressing Strands between Time of Transfer and Deck Placement, ΔfpR1

DGCS 12.6.5.4-5 $$\Delta fpR1 = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)$$

where :

Stress in Prestressing Strands Immediately After Transfer, Taken Not Less Than 0.55fpy; fp

30 for Low Relaxation Strands and 7 for Other Prestressing Steel, Unless More Accurate Manufacturer's Data are Available, K_L = 30.00

Computed Prestress Loss due to Relaxation,

Duct	fp (MPa)	ΔfpR1 (MPa)
1	1449.31	15.26
2	1455.07	15.48
3	1465.59	15.90
4	0.00	0.00
5	0.00	0.00
6	0.00	0.00
7	0.00	0.00
8	0.00	0.00
9	0.00	0.00
10	0.00	0.00

Losses: Time of Deck Placement to Final Time,

Shrinkage of Girder Concrete,

Prestress Loss due to Shrinkage of Girder Concrete between
Time of Deck Placement and Final Time, ΔfpSD = 70.34 MPa

DGCS 12.6.5.4-6 $\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df}$

in which :

Transformed Section Coefficient that Accounts for Time-Dependent Interaction between Concrete and Bonded Steel in the Section
being Considered for Time Period between Deck Placement and Final Time, Kdf = 0.88

DGCS 12.6.5.4-7 $K_{df} = \dfrac{1}{1 + \dfrac{E_p A_{ps}}{E_{ci} A_c}\left(1 + \dfrac{A_c e_{pc}^2}{I_c}\right)\left|1 + 0.7\varphi_b(t_f, t_d)\right|}$

where :

Shrinkage Strain of Girder between Time of Deck Placement and Final Time, εbdf = 0.000407
Eccentricity of Prestressing Force with Respect to Centroid of Composite Section; Positive in Typical Construction
where Prestressing Force is Below Centroid of Section, epc = 857.97 mm
Area of Section Calculated using the Gross Composite Concrete Section Properties of the Girder and the Deck
and the Deck-to-Girder Modular Ratio, Ac = 833074.65 mm^2
Moment of Inertia of Section Calculated using the Gross Composite Concrete Section Properties of the Girder
and the Deck and the Deck-to-Girder Modular Ratio at Service, Ic = 2.79733E+11 mm^4

Creep of Girder Concrete,

Prestress Loss due to Creep of Girder Concrete between
Time of Deck Placement and Final Time, ΔfpCD

DGCS 12.6.5.4-8 $\Delta f_{pCD} = \dfrac{E_p}{E_{ci}} f_{cgp}\varphi_b\left[(t_f, t_i) - \varphi_b(t_d, t_i)\right]K_{df} + \dfrac{E_p}{E_c}\Delta f_{cd}\varphi_b(t_f, t_d)K_{df}$

where :

Change in Concrete Stress at Centroid of Prestressing Strands due to Long-Term Losses between
Transfer and Deck Placement, Combined with Deck Weight and Superimposed Loads, Δfcd

$\Delta f_{cd} = \left(\Delta f_{pSR} + \Delta_{CR} + \Delta f_{pR1}\right)\dfrac{A_{ps}}{A_g}\left(1 + \dfrac{A_g e_{pg}^2}{I_g}\right) \quad \left(\dfrac{M_s e_{ptf}}{I_{tf}} + \dfrac{(M_h + M_{ws})e_{pte}}{I_{te}}\right)$

Girder Creep Coefficient at Final Time due to Loading at Deck Placement, Ψb(tf,td) = 0.81

Duct	ΔfpSR + ΔfpCR + ΔfpR1	Δfcd (MPa)	ΔfpCD (MPa)
1	75.35	-0.92	40.22
2	75.58	-0.92	40.21
3	75.99	-0.93	40.18
4	0.00	0.00	0.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00

Relaxation of Prestressing Strands,

Prestress Loss due to Relaxation of Prestressing Strands in Composite Section between
Time of Deck Placement and Final Time, ΔfpR2

DGCS 12.6.5.4-9 $\Delta f_{pR2} = \Delta f_{pR1}$

Duct	ΔfpR2 (MPa)
1	15.26
2	15.48
3	15.90
4	0.00
5	0.00
6	0.00
7	0.00
8	0.00
9	0.00
10	0.00

Shrinkage of Deck Concrete,

Prestress Gain due to Shrinkage of Deck in Composite Section

DGCS 12.6.5.4-10 $\Delta f_{pSS} = \dfrac{E_p}{E_c}\Delta f_{cdf} K_{df}\left|1 + 0.7\varphi_b(t_f, t_d)\right|$ ΔfpSS = -13.81 MPa

in which :

Change in Concrete Stress at Centroid of Prestressing Strands due to
Shrinkage of Deck Concrete, Δfcdf = -1.53 MPa

DGCS 12.6.5.4-11 $\Delta f_{cdf} = \dfrac{\varepsilon_{ddf} A_d E_{cd}}{\left|1 + 0.7\varphi_b(t_f, t_d)\right|}\left(\dfrac{1}{A_c} - \dfrac{e_{pc} e_d}{I_c}\right)$

where :

Shrinkage Strain of Deck Concrete between Placement and Final Time, εddf = 0.000407
Area of Deck Concrete, Ad = 283971.83 mm^2
Modulus of Elasticity of Deck Concrete, Ecd = 26752.50 MPa
Eccentricity of Deck with Respect to the Gross Composite Section, Positive in Typical Construction
where Deck is Above Girder, ed = 643.36 mm
Creep Coefficient of Deck Concrete at Final Time due to Loading Introduced Shortly
after Deck Placements (i.e. overlays, barriers, etc.), Ψb(tf,td) = 0.81

TRANSFORMED SECTION PROPERTIES,

Modulus of Elasticity of Beam at Transfer,	Eci	=	29910.20	MPa
Modulus of Elasticity of Beam at Service,	Ec	=	33914.98	MPa
Modulus of Elasticity of Prestressing Strands,	Eps	=	197000.00	MPa
Modular Ratio of Strands to Beam at Transfer,	n_{pi}	=	6.59	
Modular Ratio of Strands to Beam at Service,	n_{ps}	=	5.81	
Area of Prestressing Strands,	Aps	=	98.70	mm^2
Transformed Area of Prestressing Strands at Transfer,	(Aps (n_{pi}-1))	=	16541.28	mm^2
Transformed Area of Prestressing Strands at Service,	(Aps (n_{ps}-1))	=	14238.39	mm^2
Midspan Center of Gravity of Strands,	Ybot	=	130.00	mm

Girder and Strands (Initial, Transfer),

Element	A	Ybot	Ay	Io	D	AD^2
Girder	5.0903E+05	6.2824E+02	3.1979E+08	1.0862E+11	1.5681E+01	1.2517E+08
Strands	1.6541E+04	1.3000E+02	2.1504E+06	0.0000E+00	4.8256E+02	3.8518E+09
TOTAL	5.2557E+05		3.2194E+08	1.0862E+11		3.9770E+09

Transformed Initial :

Neutral Axis from Extreme Bottom Fiber,	Ybot	=	612.56	mm
Neutral Axis from Extreme Top Fiber,	Ytop	=	759.04	mm
Moment of Inertia About X-axis,	Ix	=	1.13E+11	mm^4
Section Modulus from Bottom Neutral Axis,	Sxb	=	1.84E+08	mm^3
Section Modulus from Top Neutral Axis,	Sxt	=	1.48E+08	mm^3
Eccentricity,	e	=	482.56	mm

Girder and Strands (Final),

Element	A	Ybot	Ay	Io	D	AD^2
Girder	5.0903E+05	6.2824E+02	3.1979E+08	1.0862E+11	1.3557E+01	9.3560E+07
Strands	1.4238E+04	1.3000E+02	1.8510E+06	0.0000E+00	4.8468E+02	3.3448E+09
TOTAL	5.2327E+05		3.2164E+08	1.0862E+11		3.4384E+09

Transformed Service :

Neutral Axis from Extreme Bottom Fiber,	Ybot	=	614.68	mm
Neutral Axis from Extreme Top Fiber,	Ytop	=	756.92	mm
Moment of Inertia About X-axis,	Ix	=	1.12E+11	mm^4
Section Modulus from Bottom Neutral Axis,	Sxb	=	1.82E+08	mm^3
Section Modulus from Top Neutral Axis,	Sxt	=	1.48E+08	mm^3
Eccentricity,	e	=	484.68	mm

Composite Girder and Deck,

Element	A	Ybot	Ay	Io	D	AD^2
Girder	5.0903E+05	6.2824E+02	3.1979E+08	1.0862E+11	3.0779E+02	4.8223E+10
Strands	1.4238E+04	1.3000E+02	1.8510E+06	0.0000E+00	8.0603E+02	9.2504E+09
Slab	2.8397E+05	1.5716E+03	4.4629E+08	9.4657E+08	6.3557E+02	1.1471E+11
Haunch	4.0072E+04	6.2824E+02	2.5175E+07	3.3393E+07	3.0779E+02	3.7961E+09
TOTAL	8.4731E+05		7.9311E+08	1.0960E+11		1.7598E+11

Transformed Service Composite :

Neutral Axis from Extreme Bottom Fiber,	Ybot	=	936.03	mm
Neutral Axis from Extreme Top Fiber,	Ytop	=	535.57	mm
Neutral Axis from Extreme Top Fiber for Girder,	Ytopg	=	435.57	mm
Moment of Inertia About X-axis,	Ix	=	2.86E+11	mm^4
Section Modulus from Bottom Neutral Axis,	Sxb	=	3.05E+08	mm^3
Section Modulus from Top Neutral Axis,	Sxt	=	5.33E+08	mm^3
Section Modulus from Top Neutral Axis of Girder,	Sxtg	=	6.56E+08	mm^3
Eccentricity,	e	=	806.03	mm

SUMMARY OF SECTION PROPERTIES,

	NON-COMPOSITE SECTION			COMPOSITE SECTION	
	Gross	Transformed at Transfer	Transformed at Service	Gross	Transformed
Area, mm^2	509031.24	525572.52	5.2327E+05	833074.65	847313.04
Neutral Axis from Extreme Bottom Fiber, mm	628.24	612.56	614.68	987.97	936.03
Neutral Axis from Extreme Top Fiber, mm	743.36	759.04	756.92	383.63	535.57
Moment of Inertia, mm^4	1.09E+11	1.13E+11	1.12E+11	2.80E+11	2.86E+11
Eccentricity, mm	498.24	482.56	484.68	857.97	806.03

STRESS AT TRANSFER,

Force per Strand at Transfer,

Stress per Strand at Transfer, fpi = 1113.16 MPa

Total Prestressing Force at Transfer, $Pi = fpi \times Aps$ Pi = 3296.08 KN

DGCS 12.6.4 **Stress Limits for Concrete,**

DGCS 12.6.4.1 Compression Stresses,

Compressive Stress Limit for Pretensioned and Post-Tensioned Concrete, 0.60 fci = 21.00 MPa

Tension Stresses, = 3.73 MPa

Stress at Transfer Length,

DGCS 12.8.4 Transfer Length, Ltransfer = 762.00 mm

Moment due to Beam Selfweight at Transfer, Mg = 63.48 KN-m

Due to Camber of the Beam, The Effective Beam Length shall be Taken as the Whole Beam Length,

$$Mg = \frac{Wx}{2}(L - x)$$

Beam Uniform Load, W = 12.22 KN/m

Transfer Length, x = 0.76 m

Beam Span, Lspan = 14.40 m

Stress at the Beam,

Top Stress,

$$ft = \frac{Pi}{At.transformed} + \frac{Pi \times e}{St.transformed} - \frac{Mg}{St.transformed}$$

ft = 16.57 MPa

ft < 21.00 MPa **OK!**

Area of Girder, A = 525572.52 mm^2

Eccentricity of Prestressing Strand, e = 482.56 mm

Section Modulus, St.transformed = 1.48E+08 mm^3

Bottom Stress,

$$fb = \frac{Pi}{At.transformed} - \frac{Pi \times e}{Sb.transformed} + \frac{Mg}{Sb.transformed}$$

fb = -2.04 MPa

fb > -3.73 MPa **OK!**

Section Modulus, St.transformed = 1.84E+08 mm^3

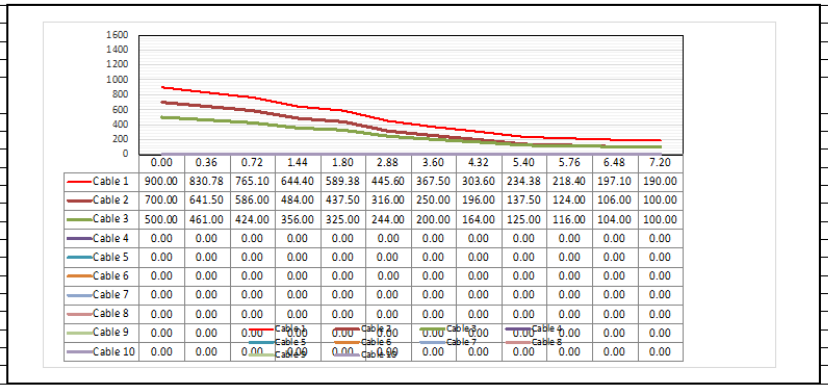
Stress at Transfer Length (Considering Effect of Harping),

Cable Profile,

$$y = \frac{4 \times Sag}{L^2}(0.50L - x)^2 + y_{midspan}$$

Point	Distance	1	2	3	4	5
0.000	0.000	900.00	700.00	500.00	0.00	0.00
0.025	0.360	830.78	641.50	461.00	0.00	0.00
0.050	0.720	765.10	586.00	424.00	0.00	0.00
0.100	1.440	644.40	484.00	356.00	0.00	0.00
0.125	1.800	589.38	437.50	325.00	0.00	0.00
0.200	2.880	445.60	316.00	244.00	0.00	0.00
0.250	3.600	367.50	250.00	200.00	0.00	0.00
0.300	4.320	303.60	196.00	164.00	0.00	0.00
0.375	5.400	234.38	137.50	125.00	0.00	0.00
0.400	5.760	218.40	124.00	116.00	0.00	0.00
0.450	6.480	197.10	106.00	104.00	0.00	0.00
0.500	7.200	190.00	100.00	100.00	0.00	0.00

Point	Distance	6	7	8	9	10
0.000	0.000	0.00	0.00	0.00	0.00	0.00
0.025	0.360	0.00	0.00	0.00	0.00	0.00
0.050	0.720	0.00	0.00	0.00	0.00	0.00
0.100	1.440	0.00	0.00	0.00	0.00	0.00
0.125	1.800	0.00	0.00	0.00	0.00	0.00
0.200	2.880	0.00	0.00	0.00	0.00	0.00
0.250	3.600	0.00	0.00	0.00	0.00	0.00
0.300	4.320	0.00	0.00	0.00	0.00	0.00
0.375	5.400	0.00	0.00	0.00	0.00	0.00
0.400	5.760	0.00	0.00	0.00	0.00	0.00
0.450	6.480	0.00	0.00	0.00	0.00	0.00
0.500	7.200	0.00	0.00	0.00	0.00	0.00



	0.00	0.36	0.72	1.44	1.80	2.88	3.60	4.32	5.40	5.76	6.48	7.20
Cable 1	900.00	830.78	765.10	644.40	589.38	445.60	367.50	303.60	234.38	218.40	197.10	190.00
Cable 2	700.00	641.50	586.00	484.00	437.50	316.00	250.00	196.00	137.50	124.00	106.00	100.00
Cable 3	500.00	461.00	424.00	356.00	325.00	244.00	200.00	164.00	125.00	116.00	104.00	100.00
Cable 4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Angular Change of Cable per Point,

$$\frac{dy}{dx} = \frac{8 * Sag}{L^2}(0.50L - x)1 = \alpha$$

Point	Distance	1	2	3	4	5
0.000	0.000	0.197	0.167	0.111	0.000	0.000
0.025	0.360	0.187	0.158	0.106	0.000	0.000
0.050	0.720	0.178	0.150	0.100	0.000	0.000
0.100	1.440	0.158	0.133	0.089	0.000	0.000
0.125	1.800	0.148	0.125	0.083	0.000	0.000
0.200	2.880	0.118	0.100	0.067	0.000	0.000
0.250	3.600	0.099	0.083	0.056	0.000	0.000
0.300	4.320	0.079	0.067	0.044	0.000	0.000
0.375	5.400	0.049	0.042	0.028	0.000	0.000
0.400	5.760	0.039	0.033	0.022	0.000	0.000
0.450	6.480	0.020	0.017	0.011	0.000	0.000
0.500	7.200	0.000	0.000	0.000	0.000	0.000

Point	Distance	6	7	8	9	10
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.025	0.360	0.000	0.000	0.000	0.000	0.000
0.050	0.720	0.000	0.000	0.000	0.000	0.000
0.100	1.440	0.000	0.000	0.000	0.000	0.000
0.125	1.800	0.000	0.000	0.000	0.000	0.000
0.200	2.880	0.000	0.000	0.000	0.000	0.000
0.250	3.600	0.000	0.000	0.000	0.000	0.000
0.300	4.320	0.000	0.000	0.000	0.000	0.000
0.375	5.400	0.000	0.000	0.000	0.000	0.000
0.400	5.760	0.000	0.000	0.000	0.000	0.000
0.450	6.480	0.000	0.000	0.000	0.000	0.000
0.500	7.200	0.000	0.000	0.000	0.000	0.000

Cable Centroids at Transfer Length,

Tendon	Centroid (Yb)	No. of Strands (N)	N * Yb
1	757.67	10	7576.69
2	579.72	10	5797.20
3	419.81	10	4198.14
4	0.00	0	0.00
5	0.00	0	0.00
6	0.00	0	0.00
7	0.00	0	0.00
8	0.00	0	0.00
9	0.00	0	0.00
10	0.00	0	0.00
TOTAL		30	17572.03

Center of Gravity of Tendons at Transfer Length, = 585.73 mm

Eccentricity at Transfer Length, = 26.82 mm

Stress at the Beam,

Top Stress,

$$ft = \frac{Pi}{At.transformed} + \frac{Pi * e}{St.transformed} - \frac{Mg}{St.transformed}$$

ft = 6.44 MPa

ft < 21.00 MPa **OK!**

Area of Girder, A = 525572.52 mm^2

Eccentricity of Prestressing Strand, e = 26.82 mm

Section Modulus, St.transformed = 1.48E+08 mm^3

Bottom Stress,

$$fb = \frac{Pi}{At.transformed} - \frac{Pi * e}{Sb.transformed} + \frac{Mg}{Sb.transformed}$$

fb = 6.14 MPa

fb < 21.00 MPa **OK!**

Section Modulus, St.transformed = 1.84E+08 mm^3

Stress at Midspan,

Moment due to Beam Selfweight at Transfer, Mg = 316.66 KN-m

Due to Camber of the Beam, The Effective Beam Length shall be Taken as the Whole Beam Length,

$$M_\mu = \frac{Wx}{2}(L - x)$$

Beam Uniform Load, W = 12.22 KN/m

Transfer Length, x = 7.20 m

Beam Span, Lspan = 14.40 m

Stress at the Beam,

Top Stress,

$$ft = \frac{Pi}{At.transformed} + \frac{Pi * e}{St.transformed} - \frac{Mg}{St.transformed}$$

ft = 14.86 MPa

ft < 21.00 MPa **OK!**

Area of Girder, A = 525572.52 mm^2

Eccentricity of Prestressing Strand, e = 482.56 mm

Section Modulus, St.transformed = 1.48E+08 mm^3

Bottom Stress,

$$fb = \frac{Pi}{At.transformed} - \frac{Pi * e}{Sb.transformed} + \frac{Mg}{Sb.transformed}$$

fb = -0.66 MPa

fb > -3.73 MPa **OK!**

Section Modulus, St.transformed = 1.84E+08 mm^3

STRESS AT SERVICE,

Force per Strand at Transfer,

Stress per Strand at Service, fpe = 1339.20 MPa

Total Prestressing Forces at Service, $Pe = fpi \cdot Aps$ Pe = 3965.37 KN

Stress Limits for Concrete,

Compressive Stress Limits under Unfactored Permanent Loads and Prestressing Force,

Girder, = 20.25 MPa

Deck Slab, = 12.60 MPa

Compressive Stress Limits under Unfactored Permanent Loads, Transient Loads and Prestressing Force,

Girder, = 27.00 MPa

Deck Slab, = 16.80 MPa

Tension Limits, = 3.35 MPa

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 $\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$

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

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 1.48E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 5.33E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength I Limit State)

Location	Load Factor	1.25 DC	1.50 DW	1.75 Lane Load	1.75 Truck/Tandem	Total
		MPa	MPa	MPa	MPa	MPa
0.000		0.00	0.00	0.00	0.00	**0.00**
1.440		1.67	0.16	0.28	0.28	**2.39**
2.880		2.97	0.29	0.50	0.53	**4.30**
4.320		3.92	0.38	0.66	0.65	**5.62**
5.760		4.51	0.44	0.76	0.72	**6.43**
7.200		4.75	0.46	0.79	0.79	**6.79**
8.640		4.51	0.44	0.76	0.66	**6.38**
10.080		3.92	0.38	0.66	0.52	**5.49**
11.520		2.97	0.29	0.50	0.37	**4.14**
12.960		1.67	0.16	0.28	0.19	**2.30**
14.400		0.00	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 1.82E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 3.05E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength I Limit State)

Location	Load Factor	1.25 DC	1.50 DW	1.75 Lane Load	1.75 Truck/Tandem	Total
		MPa	MPa	MPa	MPa	MPa
0.000		0.00	0.00	0.00	0.00	**0.00**
1.440		1.35	0.29	0.49	0.49	**2.62**
2.880		2.41	0.51	0.88	0.92	**4.73**
4.320		3.19	0.67	1.16	1.14	**6.15**
5.760		3.67	0.77	1.33	1.26	**7.02**
7.200		3.85	0.80	1.39	1.38	**7.43**
8.640		3.67	0.77	1.33	1.16	**6.92**
10.080		3.19	0.67	1.16	0.91	**5.93**
11.520		2.41	0.51	0.88	0.65	**4.46**
12.960		1.35	0.29	0.49	0.34	**2.47**
14.400		0.00	0.00	0.00	0.00	**0.00**

Compressive Stress Limits (STRENGTH II Limit State),

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 1.48E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 5.33E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength II Limit State)

Load Factor	1.25	1.50	1.35	1.35	Total
Location	DC	DW	Lane Load	Permit	
	MPa	MPa	MPa	MPa	MPa
0.000	0.00	0.00	0.00	0.00	**0.00**
1.440	1.67	0.16	0.22	0.22	**2.26**
2.880	2.97	0.29	0.39	0.41	**4.06**
4.320	3.92	0.38	0.51	0.50	**5.32**
5.760	4.51	0.44	0.59	0.56	**6.09**
7.200	4.75	0.46	0.61	0.61	**6.43**
8.640	4.51	0.44	0.59	0.51	**6.05**
10.080	3.92	0.38	0.51	0.40	**5.22**
11.520	2.97	0.29	0.39	0.29	**3.94**
12.960	1.67	0.16	0.22	0.15	**2.20**
14.400	0.00	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 1.82E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 3.05E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength II Limit State)

Load Factor	1.25	1.50	1.35	1.35	Total
Location	DC	DW	Lane Load	Permit	
	MPa	MPa	MPa	MPa	MPa
0.000	0.00	0.00	0.00	0.00	**0.00**
1.440	1.35	0.29	0.38	0.38	**2.40**
2.880	2.41	0.51	0.68	0.71	**4.31**
4.320	3.19	0.67	0.89	0.88	**5.63**
5.760	3.67	0.77	1.02	0.97	**6.43**
7.200	3.85	0.80	1.07	1.07	**6.80**
8.640	3.67	0.77	1.02	0.90	**6.35**
10.080	3.19	0.67	0.89	0.70	**5.45**
11.520	2.41	0.51	0.68	0.50	**4.11**
12.960	1.35	0.29	0.38	0.26	**2.28**
14.400	0.00	0.00	0.00	0.00	**0.00**

Compressive Stress Limits (STRENGTH III Limit State),

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 1.48E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 5.33E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength III Limit State)

Load Factor	1.25	1.50	1.40	Total
Location	DC	DW	Wind Load	
	MPa	MPa	MPa	MPa
0.000	0.00	0.00	0.00	**0.00**
1.440	1.67	0.16	-0.04	**1.79**
2.880	2.97	0.29	-0.07	**3.20**
4.320	3.92	0.38	-0.09	**4.22**
5.760	4.51	0.44	-0.10	**4.85**
7.200	4.75	0.46	-0.11	**5.10**
8.640	4.51	0.44	-0.10	**4.85**
10.080	3.92	0.38	-0.09	**4.22**
11.520	2.97	0.29	-0.07	**3.20**
12.960	1.67	0.16	-0.04	**1.79**
14.400	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 1.82E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 3.05E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength III Limit State)

Load Factor	1.25	1.50	1.40	Total
Location	DC	DW	Wind Load	
	MPa	MPa	MPa	MPa
0.000	0.00	0.00	0.00	**0.00**
1.440	1.35	0.29	-0.07	**1.57**
2.880	2.41	0.51	-0.12	**2.80**
4.320	3.19	0.67	-0.16	**3.70**
5.760	3.67	0.77	-0.18	**4.25**
7.200	3.85	0.80	-0.19	**4.47**
8.640	3.67	0.77	-0.18	**4.25**
10.080	3.19	0.67	-0.16	**3.70**
11.520	2.41	0.51	-0.12	**2.80**
12.960	1.35	0.29	-0.07	**1.57**
14.400	0.00	0.00	0.00	**0.00**

Compressive Stress Limits (STRENGTH V Limit State),

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 1.48E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 5.33E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength V Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.35 Lane Load MPa	1.35 Truck/Tandem MPa	0.40 Wind Load MPa	Total MPa
0.000	0.00	0.00	0.00	0.00	0.00	**0.00**
1.440	1.67	0.16	0.22	0.22	-0.01	**2.25**
2.880	2.97	0.29	0.39	0.41	-0.02	**4.04**
4.320	3.92	0.38	0.51	0.50	-0.03	**5.29**
5.760	4.51	0.44	0.59	0.56	-0.03	**6.07**
7.200	4.75	0.46	0.61	0.61	-0.03	**6.40**
8.640	4.51	0.44	0.59	0.51	-0.03	**6.02**
10.080	3.92	0.38	0.51	0.40	-0.03	**5.19**
11.520	2.97	0.29	0.39	0.29	-0.02	**3.92**
12.960	1.67	0.16	0.22	0.15	-0.01	**2.18**
14.400	0.00	0.00	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 1.82E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 3.05E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength V Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.35 Lane Load MPa	1.35 Truck/Tandem MPa	0.40 Wind Load MPa	Total MPa
0.000	0.00	0.00	0.00	0.00	0.00	**0.00**
1.440	1.35	0.29	0.38	0.38	-0.02	**2.38**
2.880	2.41	0.51	0.68	0.71	-0.03	**4.28**
4.320	3.19	0.67	0.89	0.88	-0.05	**5.58**
5.760	3.67	0.77	1.02	0.97	-0.05	**6.38**
7.200	3.85	0.80	1.07	1.07	-0.05	**6.74**
8.640	3.67	0.77	1.02	0.90	-0.05	**6.30**
10.080	3.19	0.67	0.89	0.70	-0.05	**5.41**
11.520	2.41	0.51	0.68	0.50	-0.03	**4.07**
12.960	1.35	0.29	0.38	0.26	-0.02	**2.26**
14.400	0.00	0.00	0.00	0.00	0.00	**0.00**

Stress at Midspan,

STRENGTH I		STRENGTH II		STRENGTH III		STRENGTH V	
Top Fiber MPa	Bottom Fiber MPa	Top Fiber MPa	Bottom Fiber MPa	Top Fiber MPa	Bottom Fiber MPa	Top Fiber MPa	Bottom Fiber MPa
6.79	7.43	6.43	6.80	5.10	4.47	6.40	6.74

Stress at the Girder,

Stress at the Top Fiber,

$$ft = \frac{Pi}{Atransformed} - \frac{Pi * e}{St.transformed} + ft.service$$

Area of Girder, Atransformed = 523269.63 mm^2

Eccentricity of Prestressing Strand, e = 484.68 mm

Section Modulus from Bottom of Transformed Service, St.transformed = 1.48E+08 mm^3

Stress at the Top Fiber for Strength I, ft1 = 1.39 MPa

ft1 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength II, ft2 = 1.03 MPa

ft2 < 27.00 MPa **OK!**

Stress at the Top Fiber for Strength III, ft3 = -0.31 MPa

ft3 > -3.35 MPa **OK!**

Stress at the Top Fiber for Strength V, ft5 = 0.99 MPa

ft5 < 27.00 MPa **OK!**

Stress at the Bottom Fiber,

$$fb = \frac{Pi}{Atransformed} + \frac{Pi * e}{St.transformed} - fb.service$$

Section Modulus from Bottom of Transformed Service, Sb.transformed = 1.82E+08 mm^3

Stress at the Top Fiber for Strength I, fb1 = 10.69 MPa

fb1 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength II, fb2 = 11.32 MPa

fb2 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength III, fb3 = 13.65 MPa

fb3 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength V, fb5 = 11.38 MPa

fb5 < 20.25 MPa **OK!**

DGCS 12.2.2 Fatigue Limit,

Stress at Bottom Fiber due to Permanent Loads and Prestress,

Bottom Fiber, = 7.43 MPa

Fatigue Load,

Bending Moment due to Fatigue Load, = 144.32 KN-m

Factor for Fatigue Load, = 1.50

Stress due to Fatigue Load at Bottom Fiber, = -0.71 MPa

Half the Sum of Stress due to Effective Prestress and Permanent Load at the Bottom Fiber, = 3.71 MPa

Stress due to Prestress, Permanent Load and Fatigue I Load, ff = 3.01 MPa

ff < 20.25 MPa **OK!**

STRENGTH LIMIT STATE (FLEXURE),

Ultimate Bending Moment :

MOMENT per GIRDER (Strength I Limit State)

Location	Load Factor	1.38 DC	1.65 DW	1.93 Lane Load	1.93 Truck/Tandem	1.93 IM	Total
		KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
0.000		0.000	0.000	0.000	0.000	0.000	**0.00**
1.440		-271.875	-96.171	-166.075	-165.391	-54.579	**-754.09**
2.880		-485.293	-171.304	-295.821	-310.083	-102.327	**-1364.83**
4.320		-640.254	-225.400	-389.240	-382.325	-126.167	**-1763.39**
5.760		-736.759	-258.458	-446.328	-423.516	-139.760	**-2004.82**
7.200		-774.807	-270.479	-467.088	-464.708	-153.354	**-2130.44**
8.640		-736.759	-258.458	-446.328	-390.325	-128.807	**-1960.68**
10.080		-640.254	-225.400	-389.240	-307.317	-101.415	**-1663.63**
11.520		-485.293	-171.304	-295.821	-219.312	-72.373	**-1244.10**
12.960		-271.875	-96.171	-166.075	-112.987	-37.286	**-684.39**
14.400		0.000	0.000	0.000	0.000	0.000	**0.00**

MOMENT per GIRDER (Strength II Limit State)

Location	Load Factor	1.38 DC	1.65 DW	1.49 Lane Load	1.49 Permit	1.49 IM	Total
		KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
0.000		0.000	0.000	0.000	0.000	0.000	**0.00**
1.440		-271.875	-96.171	-128.115	-127.588	-58.138	**-681.89**
2.880		-485.293	-171.304	-228.205	-239.207	-103.120	**-1227.13**
4.320		-640.254	-225.400	-300.271	-294.936	-125.322	**-1586.18**
5.760		-736.759	-258.458	-344.310	-326.713	-138.093	**-1804.33**
7.200		-774.807	-270.479	-360.325	-358.489	-150.864	**-1914.96**
8.640		-736.759	-258.458	-344.310	-301.108	-142.604	**-1783.24**
10.080		-640.254	-225.400	-300.271	-237.073	-114.675	**-1517.67**
11.520		-485.293	-171.304	-228.205	-169.184	-80.783	**-1134.77**
12.960		-271.875	-96.171	-128.115	-87.162	-46.891	**-630.21**
14.400		0.000	0.000	0.000	0.000	0.000	**0.00**

MOMENT per GIRDER (Strength III Limit State)

Location	Load Factor	1.38 DC	1.65 DW	1.54 Wind Load	Total
		KN-m	KN-m	KN-m	KN-m
0.000		0.000	0.000	0.000	**0.00**
1.440		-271.875	-96.171	22.760	**-345.29**
2.880		-485.293	-171.304	40.541	**-616.06**
4.320		-640.254	-225.400	53.343	**-812.31**
5.760		-736.759	-258.458	61.167	**-934.05**
7.200		-774.807	-270.479	64.012	**-981.27**
8.640		-736.759	-258.458	61.167	**-934.05**
10.080		-640.254	-225.400	53.343	**-812.31**
11.520		-485.293	-171.304	40.541	**-616.06**
12.960		-271.875	-96.171	22.760	**-345.29**
14.400		0.000	0.000	0.000	**0.00**

MOMENT per GIRDER (Strength V Limit State)

Location	Load Factor	1.38 DC	1.65 DW	1.49 Lane Load	1.49 Truck/Tandem	1.49 IM	0.44 Wind Load	Total
		KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
0.000		0.000	0.000	0.000	0.000	0.000	0.000	**0.00**
1.440		-271.875	-96.171	-128.115	-127.588	-42.104	6.503	**-659.35**
2.880		-485.293	-171.304	-228.205	-239.207	-78.938	11.583	**-1191.36**
4.320		-640.254	-225.400	-300.271	-294.936	-97.329	15.241	**-1542.95**
5.760		-736.759	-258.458	-344.310	-326.713	-107.815	17.476	**-1756.58**
7.200		-774.807	-270.479	-360.325	-358.489	-118.301	18.289	**-1864.11**
8.640		-736.759	-258.458	-344.310	-301.108	-99.366	17.476	**-1722.52**
10.080		-640.254	-225.400	-300.271	-237.073	-78.234	15.241	**-1465.99**
11.520		-485.293	-171.304	-228.205	-169.184	-55.831	11.583	**-1098.23**
12.960		-271.875	-96.171	-128.115	-87.162	-28.763	6.503	**-605.58**
14.400		0.000	0.000	0.000	0.000	0.000	0.000	**0.00**

Governing Load Case :

Design Moment, Mu = 2130.44 KN-m

DGCS 12.4.3.1.3 Flexural Strength Design (Strain Compatibility Approach),

PCI BDM 2003 Assumptions in this Approach

- Plane Sections Remain Plane After Bending.
- Compatibility of Strains, i.e. Full Bond between Steel and Concrete at the Section being Considered.
- Equilibrium of Forces within a Section.
- Concrete Stress at Ultimate Flexure is Represented by a Rectangular Stress Block with Intensity of 0.85fc and Depth a = β1*c where c is the Neutral Axis Depth and β1 is Determined.

DGCS 12.4.2.1

- Steel Stress Relationship Adopted for a 1863N/mm^2 (270kip/in^2) Low Relaxation Strand is :

PCI BDM 8.2.2.5-1

$$fst = \varepsilon st \left\{ \frac{887 + 27613}{\left[1 + (112.4\varepsilon st)^{7.36}\right]^{\frac{1}{7.36}}} \right\} \leq 270ksi$$

- For Mild Reinforcements, an Elastic Plastic Stress-Strain Relationship is Assumed :

PCI BDM 8.2.2.5-2 $fsi = Es * \varepsilon si \leq fy$

- For Each Concrete Component, the Force Would Be :

PCI BDM 8.2.2.5-5 $Fcj = 0.85 * fcj * Acj$

Computation,

Assume Neutral Axis Depth c and Substitute in the Equation Below,

PCI BDM 8.2.2.5-3

$$\varepsilon si = 0.003\left[\left(\frac{di}{c}\right)-1\right]+\left(\frac{fsc}{Es}\right)$$

Assumed Neutral Axis Depth, c = 200.00 mm

Depth of Steel Layer from Extreme Compression Fiber, di

Effective Prestress, fse. For Partially Tensioned Tendons or for Non-Tension Reinforcing Bars, This May be Assumed As,

PCI BDM 8.2.2.5-5

$$fsc = fpi - 172.4\ MPa$$

Where fpi is Initial Tension

Layer	db	n	As	di	fpi
	mm	pc/s	mm^2	mm	MPa
1	28	3	1,847.26	90.00	0.00
2	16	12	2,412.74	100.00	0.00
3	28	4	2,463.01	360.00	0.00
4	12	10	1,130.97	1,028.00	0.00
5	28	4	2,463.01	1,732.00	0.00

Steel Stresses and Forces,

Layer	di	fse	esi	fsi	Asi*fsi
1	90.00	-172.40	-0.003	-420.00	-775.85
2	100.00	-172.40	-0.002	-280.00	-675.57
3	360.00	-172.40	0.002	307.60	757.62
4	1,028.00	-172.40	0.012	280.00	316.67
5	1,732.00	-172.40	0.022	420.00	1034.46
TENDON	1,441.60	1060.08	0.024	1840.04	5448.35

Stresses in Concrete Components with Corresponding Forces,

Averaged Value for β due to Composite Construction,

PCI BDM 8.2.2.5-6

$$\beta 1.ave = \frac{\sum(f'c * Ac * \beta 1)_j}{\sum(f'c * Ac)_j}$$

β1.ave = 0.85

Assumed Average β1, = 0.85

Assumed Compression Block Depth, a = 170.00 mm

Component	Compression Area, Ac	Compressive Strength, f'c	β1	f'c*Ac*β1	f'c*Ac
Deck	306000.00	28.00	0.85	7,282,800.00	8,568,000.00
Flange	0.00	45.00	0.73	0.00	0.00
Web	0.00	45.00	0.73	0.00	0.00
			TOTAL	**7,282,800.00**	**8,568,000.00**

Revised Compression Block Depth, a = 170.00 mm

Component	Compression Area, Ac	Compressive Strength, f'c	β1	f'c*Ac*β1	f'c*Ac	Fcj
Deck	306000.00	28.00	0.85	7,282,800.00	8,568,000.00	-7,282.80
Flange	0.00	45.00	0.73	0.00	0.00	0.00
Web	0.00	45.00	0.73	0.00	0.00	0.00
			TOTAL	**7,282,800.00**	**8,568,000.00**	**-7,282.80**

Check for Equilibrium,

PCI BDM 8.2.2.5-4

$$\sum Asi * fsi + \sum Fcj = 0$$

Summation of Forces on Steel Components, = 657.34 KN

Summation of Forces on Concrete Components, = -7282.80 KN

Permissible Error Set by Designer, =

Summation of Forces, = -6625.46 KN

Computation for Flexural Strength,

PCI BDM 8.2.2.5-7

$$Mn = \sum Asi * fsi * di + \sum Fcj * dj$$

Moment Arm from Top Fiber,

Component		Asi*fsi	di	Fcj	dj	Moment about Top Fiber
		KN	mm	KN	mm	KN-m
Deck		-	-	-7,282.80	100.00	-728.28
Flange		-	-	0.00	0.00	0.00
Web		-	-	0.00	0.00	0.00
For Steel	Layer 1	-775.85	90.00	-	-	-69.83
	Layer 2	-675.57	100.00	-	-	-67.56
	Layer 3	757.62	360.00	-	-	272.74
	Layer 4	316.67	1,028.00	-	-	325.54
	Layer 5	1,034.46	1,732.00	-	-	1,791.69
TENDON		5,448.35	1,441.60	-	-	7,854.34
					TOTAL	**9,378.65**

DGCS 12.4.3.3	**Limits of Reinforcement,**		Mcr	=	1,171.98	KN-m
	1.33 Times the Factored Moment Required,		1.33*Ma	=	2,833.48	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	=	1,171.98	KN-m
	Modulus of Rupture, $fr = 0.63\sqrt{fc'}$		fr	=	4.23	MPa
	Compressive Stress in Concrete due to Effective Prestress Forces,		fcpe	=	0.02	MPa
	Total Unfactored Dead Load Moment Acting on the Non-Composite Section,		Mdnc	=	562.22	KN-m
	Section Modulus for the Extreme Fiber of the Composite Section,		Sc	=	1.73E+08	mm^3
	Section Modulus for the Extreme Fiber of the Non-Composite Section,		Snc	=	2.83E+08	mm^3
	Flexural Cracking Variability Factor,		γ_1	=	1.60	
	Prestress Variability Factor,		γ_2	=	1.10	
	Ratio of Minimum Yield Strength to Ultimate Tensile Strength of the Reinforcement,		γ_3	=	1.00	
	Checking for Flexural Capacity,	9,378.65	>	1171.98	KN-m	**OK!**

STRENGTH LIMIT STATE (SHEAR),

Ultimate Shear :

SHEAR per GIRDER (Strength I Limit State)

Load Factor	1.38	1.65	1.93	1.93	1.93	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	
	KN	KN	KN	KN	KN	KN
0.000	212.00	75.13	129.75	180.18	59.46	**656.52**
1.440	170.24	60.11	103.80	80.00	26.40	**440.54**
2.880	128.49	45.08	77.85	80.00	26.40	**357.81**
4.320	86.73	30.05	51.90	19.89	6.56	**195.14**
5.760	44.98	15.03	25.95	-20.19	-6.66	**59.11**
7.200	3.22	0.00	0.00	-20.19	-6.66	**-23.62**
8.640	-44.98	-15.03	-25.95	-25.02	-8.26	**-119.24**
10.080	-86.73	-30.05	-51.90	-44.37	-14.64	**-227.70**
11.520	-128.49	-45.08	-77.85	-44.37	-14.64	**-310.43**
12.960	-170.24	-60.11	-103.80	-44.37	-14.64	**-393.16**
14.400	-212.00	-75.13	-129.75	-44.37	-14.64	**-475.89**

SHEAR per GIRDER (Strength II Limit State)

Load Factor	1.38	1.65	1.49	1.49	1.49	Total
Location	DC	DW	Lane Load	Permit	IM	
	KN	KN	KN	KN	KN	KN
0.000	212.00	75.13	100.09	185.54	61.23	**633.99**
1.440	170.24	60.11	80.07	117.11	38.65	**466.17**
2.880	128.49	45.08	60.05	71.48	23.59	**328.70**
4.320	86.73	30.05	40.04	71.48	23.59	**251.89**
5.760	44.98	15.03	20.02	25.86	8.53	**114.42**
7.200	3.22	0.00	0.00	-42.58	-14.05	**-53.41**
8.640	-44.98	-15.03	-20.02	-42.58	-14.05	**-136.65**
10.080	-86.73	-30.05	-40.04	-42.58	-14.05	**-213.45**
11.520	-128.49	-45.08	-60.05	-79.68	-26.29	**-339.59**
12.960	-170.24	-60.11	-80.07	-104.41	-34.45	**-449.28**
14.400	-212.00	-75.13	-100.09	-104.41	-34.45	**-526.08**

SHEAR per GIRDER (Strength III Limit State)

Load Factor	1.38	1.65	1.54	Total
Location	DC	DW	Wind Load	
	KN	KN	KN	KN
0.000	212.00	75.13	-17.78	**269.35**
1.440	170.24	60.11	-14.22	**216.13**
2.880	128.49	45.08	-10.67	**162.90**
4.320	86.73	30.05	-7.11	**109.68**
5.760	44.98	15.03	-3.56	**56.45**
7.200	3.22	0.00	0.00	**3.22**
8.640	-44.98	-15.03	3.56	**-56.45**
10.080	-86.73	-30.05	7.11	**-109.68**
11.520	-128.49	-45.08	10.67	**-162.90**
12.960	-170.24	-60.11	14.22	**-216.13**
14.400	-212.00	-75.13	17.78	**-269.35**

SHEAR per GIRDER (Strength V Limit State)

Load Factor	1.38	1.65	1.49	1.49	1.49	0.44	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	Wind Load	
	KN	KN	KN	KN	KN	KN	KN
0.000	212.00	75.13	100.09	139.00	45.87	-5.08	**567.01**
1.440	170.24	60.11	80.07	61.71	20.37	-4.06	**388.44**
2.880	128.49	45.08	60.05	61.71	20.37	-3.05	**312.65**
4.320	86.73	30.05	40.04	15.34	5.06	-2.03	**175.20**
5.760	44.98	15.03	20.02	-15.57	-5.14	-1.02	**58.30**
7.200	3.22	0.00	0.00	-15.57	-5.14	0.00	**-17.49**
8.640	-44.98	-15.03	-20.02	-19.30	-6.37	1.02	**-104.68**
10.080	-86.73	-30.05	-40.04	-34.23	-11.29	2.03	**-200.31**
11.520	-128.49	-45.08	-60.05	-34.23	-11.29	3.05	**-276.10**
12.960	-170.24	-60.11	-80.07	-34.23	-11.29	4.06	**-351.88**
14.400	-212.00	-75.13	-100.09	-34.23	-11.29	5.08	**-427.66**

Governing Load Case :

Design Shear, Vu = 656.52 KN

Shear Strength Design,

Assumptions in this Approach :

- Plane Sections Remains Plane after Bending.

DGCS 12.5.2.3-1 Computation, $Vu > 0.50 * \phi v (Vc + Vp)$

DGCS 12.5.2.8-1 Shear Stress on Concrete, vu $vu = \frac{(Vu - \phi v * Vp)}{(\phi v * bv * dv)}$ vu = 0.07 MPa

Critical Section for Shear,

Effective Shear Depth, $dv = de - \frac{a}{2}$ dv = 987.55 mm

where :

Depth of Compression Block at Midspan, a = 170.00 mm

Effective Shear Depth of the Beam at the End, dv = 586.60 mm

Need Not be Taken Less Than the Greater of,

0.90*de = 604.44 mm

0.72*h = 987.55 mm

Effective Depth at the End of Beam,

DGCS 12.5.2.8-2 $de = \frac{(Aps * fps * dp)}{(Aps * fps)} = dp$ de.end = 671.60 mm

Tendon Centroid at Beam Support,

Tendon	Centroid	No. of Strands	N*Yb
1	900.00	10.00	9,000.00
2	700.00	10.00	7,000.00
3	500.00	10.00	5,000.00
4	0.00	0.00	0.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
TOTAL		**30.00**	**21,000.00**

Centroid of Prestressing Steel at End of Beam, Ybps = 700.00 mm

Distance Measured from Extreme Compression Fiber dp = 671.60 mm

to the Centroid of Prestressing Steel at End of Beam,

Contribution of Prestressing Force to Nominal Shear Resistance, $Vp = Pj * \alpha$ Vp = 694.44 KN

Prestressing Force before Losses, Pj = 11574.00 KN

Angle Change at Point of Critical Shear, α = 0.06 radians

Contribution of Concrete to Nominal Shear Resistance,

DGCS 12.5.3.2-4 Nominal Shear Resistance of Concrete, $Vc = 0.083\beta\sqrt{f'c} * bv * dv$ Vc = 1876.06 KN

Strain in Flexural Tension Reinforcement, εs = -0.00290

DGCS 12.5.3.3-4 $\varepsilon s = \frac{\left(\frac{|Mu|}{dv} + 0.5Nu + |Vu - Vp| - \Delta psfpo\right)}{(EsAs + EpAps)}$

Applied Factored Bending Moment at the Specified Sections, Mu = 37.45 KN-m

Applied Factored Shear at the Specified Section, Vu = 656.52 KN

Applied Factored Normal Force at the Specified Section, Nu = 0.00 KN

Parameter Taken as Modulus of Elasticity of Prestressing fpo = 1302.00 MPa

Tendons Multiplied by the Locked in Difference in Strain

between the Prestressing Tendons and Surrounding Concrete,

Area of Non-Prestressed Steel on the Flexural Tension Side As = 3593.98 mm^2

of the Member at the Section Under Consideration,

Calculation for Negative Strain,

Applicable if Initial Strain Calculation is Negative, Should Not be Less than -0.004, εs = -0.00038

$$\varepsilon s = \frac{\left(\frac{|Mu|}{dv} + 0.5Nu + |Vu - Vp| - \Delta psfpo\right)}{(EsAs) + (EpAps + EcAct)}$$

Area of Concrete on the Flexural Tension Side of the Act = 254515.62 mm^2

Member, where the Flexural side of the Member shall

be taken as the Half-Depth containing the Flexural

Tension Zone,

Strain Used in Calculation for Contribution of Concrete to Shear, εs = -0.0004

DGCS 12.5.3.3.2 For Sections containing at least the Minimum Amount of Transverse Reinforcement, β = 6.72

DGCS 12.5.3.3-1 $\beta = \frac{4.8}{(1 + 750\varepsilon s)}$

DGCS 12.5.3.3-3 Angle of Inclination, $\theta = 29 + 3500\varepsilon s$ θ = 27.67 degrees

Contribution of Reinforcement to Nominal Shear Resistance,

Transverse Reinforcement will be Provided where,

DGCS 12.5.2.3-1 $Vu > 0.50 * \phi v (Vc + Vp)$

Location	Factored Shear	0.50*φv*(Vc + Vp)	Remarks
0.000	656.52	1,156.73	**OK!**
1.440	466.17	1,156.73	**OK!**
2.880	357.81	1,156.73	**OK!**
4.320	251.89	1,156.73	**OK!**
5.760	114.42	1,156.73	**OK!**
7.200	3.22	1,156.73	**OK!**

Requireed Strength of Shear Reinforcement, As

Required Shear Strength for Steel, Vs

DGCS 12.5.3.2-4 $Vu \leq \phi v \cdot Vn = \phi v(Vc + Vs + Vp)$

$$Vu = \frac{Vu}{\phi v} - (Vc + Vp)$$

Required Spacing of Transverse Shear Reinforcement, s

DGCS 12.5.3.2-4 $$Vs = \frac{Avfysin\alpha(cot\theta + cot\alpha)sin\alpha}{s}$$

Angle of Inclination of Transverse Reinforcement to Longitudinal Axis,	α	=	90.00 degrees
Transverse Shear Reinforcement Specifications :			
Diameter of Transverse Bar,	db.transverse	=	12.00 mm
Yield Strength of Bar,	fy	=	280.00 MPa
Number of Legs,	Nlegs	=	2.00
Area of Transverse Shear Reinforcement,	Av	=	226.19 mm^2

$Vu > 0.50 \cdot \phi v(Vc + Vp)$

Location	Factored Shear	Required Transverse Shear Strength of Steel	Required Spacing	Av/s
0.000	656.52	-1,841.04	300.00	**MINIMUM**
1.440	466.17	-2,052.53	300.00	**MINIMUM**
2.880	357.81	-2,172.93	300.00	**MINIMUM**
4.320	251.89	-2,290.62	300.00	**MINIMUM**
5.760	114.42	-2,443.38	300.00	**MINIMUM**
7.200	3.22	-2,566.92	300.00	**MINIMUM**

Minimum Transverse Reinforcements,

Except for Segmental Post-Tensioned Concrete Box Girder Bridges,

DGCS 12.5.2.4-1 $$Av \geq 0.083\sqrt{f'c}\frac{bvs}{fy}$$

Required Area of Transverse Shear per Spacing,		=	0.40 mm^2/mm
Spacing for Minimum Transverse Reinforcement,		=	559.80 mm
Maximum Spacing of Transverse Reinforcement,	0.125fc	=	5.63 MPa
	v_u	>	0.125fc
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	=	300.00 mm
Maximum Nominal Shear Resistance, $Vn = 0.25f'cbvdv + Vp$	Vn	=	2951.98 KN

Design for Shear,

Location	Required Spacing	Spacing	Computed Shear Resistance	Exceeds Max. Shear Resistance	Design Shear Resistance	Factored Shear	C/D Ratio	Remarks
0.000	300.00	150.00	3,365.79	YES	2,656.79	656.52	4.05	**OK!**
1.440	300.00	150.00	3,365.79	YES	2,656.79	466.17	5.70	**OK!**
2.880	300.00	250.00	3,047.67	YES	2,656.79	357.81	7.43	**OK!**
4.320	300.00	250.00	3,047.67	YES	2,656.79	251.89	10.55	**OK!**
5.760	300.00	300.00	2,968.15	YES	2,656.79	114.42	23.22	**OK!**
7.200	300.00	300.00	2,968.15	YES	2,656.79	3.22	823.86	**OK!**

DEFLECTION AND CAMBER,

Deflections due to Dead Loads,

Deflection due to Prestressing Force at Transfer,

PCI BDM 8.7-1 $$\Delta p = \frac{5wL^4}{384EI}$$ Δp = 36.14 mm

Equivalent Moment or Load,

$$w = \frac{8Pe}{L^2}$$ w = 209.69 KN/m

Eccentricity, e = 0.50 mm

Prestressing Force at Transfer, P = 10908.69 KN

Deflection due to Beam Self Weight,

PCI BDM 8.7-1 $$\Delta p = \frac{5wL^4}{384EI}$$ Δp = 2.11 mm

PCI Multiplier,

Deflection due to Diaphragm Weight, ΔD = 0.02 mm

$$\Delta D = \frac{PL^3}{28EI}$$ (For Two Intermediate Diaphragm)

$$\Delta D = \frac{PL^3}{48EI}$$ (For One Intermediate Diaphragm)

Weight of Diaphragm 1, P1 = 7.02 KN

Weight of Diaphragm 2, P2 = 0.00 KN

Distance of Intermediate Diaphragms from End Face of Girder, a = 0.00 m

Deflection due to Slab and Haunch Weights,

PCI BDM 8.7-1 $$\Delta p = \frac{5wL^4}{384EI}$$ Δp = 1.70 mm

Deflection due to Rail/Barrier and Future Wearing Surface,

PCI BDM 8.7-1 $$\Delta p = \frac{5wL^4}{384EI}$$ Δp = 0.90 mm

Deflection due to Live Load,

DGCS 10.7.4 Deflection Limits,

DGCS 10.1.2.6	Vehicular Load General	ΔLL1	=	18.00	mm
DGCS 10.1.2.6	Vehicular and Pedestrian Loads	ΔLL2	=	14.40	mm
DGCS 10.1.2.6	Vehicular Load on Cantilever Arms	ΔLL3	=	48.00	mm
DGCS 10.1.2.6	Vehicular and Pedestrian Loads on Cantilever Arms	ΔLL4	=	38.40	mm

Live Load Deflections,

Deflection due to Lane Load,

$$\Delta LL = \frac{5wL^4}{384EI}$$

ΔLL = 1.08 mm

Deflection due to Truck Load,

Location of Resultant to Mid-Span: N = 0.50

Vehicle Load Multiplier for Wheel Load, User Guide

Point Load per Axle, P

Span Length, L

Location where Deflection is Needed, x

Distance from End of Beam to Location of Point Load, a

P	a	b	x	Δmidspan
72.50	3.628	10.77	7.20	0.03
72.50	7.928	6.47	7.20	0.47
17.50	12.228	2.17	7.20	0.05
	TOTAL			0.55

Total Deflection with IM and DFM, ΔTL = 1.08 mm

Deflection at Time of Erection,

Initial Prestress,

Time Dependent Multiplier for Deflection, $TDM_{initial\ prestress}$ = 1.93

PCI BDM 15.5-1 $TDM_{initial\ prestress} = 1 + \varphi_{b(td,ti)}$

Deflection at Midspan, = 69.84 mm

Self Weight,

Time Dependent Multiplier for Deflection, $TDM_{selfweight}$ = 1.93

PCI BDM 15.5-1 $TDM_{selfweight} = 1 + \varphi_{b(td,ti)}$

Deflection at Midspan, = 4.07 mm

Dead Load on Plain Beam,

Time Dependent Multiplier for Deflection, $TDM_{DLofplainb.}$ = 1.00

$TDM_{DL\ of\ plain\ beam} = 1.00$

Deflection at Midspan, = 1.72 mm

Dead Load on Composite Beam,

Time Dependent Multiplier for Deflection, $TDM_{DLofcomp.b.}$ = 1.00

$TDM_{DL\ of\ composite\ beam} = 1.00$

Deflection at Midspan, = 0.90 mm

Total Deflection at Erection before Deck Placement, = 65.77 mm

Total Deflection after Deck Placement, = 64.06 mm

Total Deflection due to Dead Loads on Composite Section, = 63.16 mm

Deflection Check,

Type of Live Load Traffic, Vehicular and Pedestrian Loads

Deflection Limit, = 14.40 mm

Maximum Live Load Deflector, = 1.08 mm

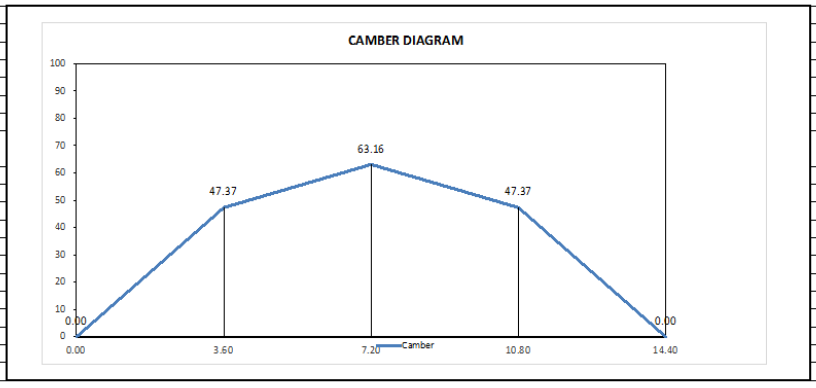
14.40 > 1.08 **OK!**

Camber Diagram,

$$y = \frac{-4\Delta_{mid}}{L^2}\left(x - \frac{L}{2}\right)^2 + \Delta_{mid}$$

Deflection at Midspan due to Permanent Loads, Δmid = 63.16 mm

Point	Distance	Deflection
0.00	0.00	0.00
0.25	3.60	47.37
0.50	7.20	63.16
0.75	10.80	47.37
1.00	14.40	0.00



DGCS 12.5.5 **SHEAR INTERFACE DESIGN,**

Shear Interface Reinforcement Specifications,

Shear Interface Reinforcement Bar,

Diameter of Bar,	db.interface	=	12.00	mm
Area of Bar,	As.interface	=	113.10	mm^2
No. of Legs Crossing Interface,	Nlegs	=	2.00	pc/s
Total Area of Shear Interface Reinforcement,	Avf	=	226.19	mm^2

Computation of the Factored Interface Shear Force for Girder/Slab Bridges,

DGCS 12.5.4.1-1 $vui = \frac{Vui}{bvdv}$

Distance between the Centroid of the Tension Steel and the Mid	dv	=	987.55	mm
Thickness of the Slab to Compute a Factored Interface Shear Stress,				
Width of Cross Section at the Contact Surface being	bv	=	508.00	mm

Investigated for Horizontal Shear,

Factored Shear Force at Specified Section

(Critical Section for Shear in this Case, Vui)

PCI BDM 8.5.3-1 Required Nominal Shear Interface Shear Resistance, Vni

$Vni = \frac{vui * Acv\ per\ m}{\phi v}$

Point	Factored Shear	Factored Interface Shear Force	Required Nominal Interface Resistance	Wave Reinforcement Provision?
	KN	MPa	KN/m	
0.00	656.52	1.309	738.661	APPLICABLE
0.05	466.17	0.929	524.500	APPLICABLE
0.15	357.81	0.713	402.583	APPLICABLE
0.30	251.89	0.502	283.411	APPLICABLE

NOTE: Minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces roughened to an amplitude of 6mm where the Factored Interface shear stress is less than and all vertical shear reinforcement required is extended across the interface and adequately anchored in the slab. = 1.40 MPa

DGCS 12.5.4-3 Nominal Shear Resistance,

$Vni = cAv + \mu(Avfy + Pc)$

Area of Concrete to be Engaged in Shear Transfer,	Acv	=	508000.00	mm^2
Area of Shear Reinforcement Crossing the Shear Plane,	Avf	=	635.00	mm^2
Yield Strength of Reinforcement,	fy	=	280.00	MPa
Permanent Net Compressive Force Normal to Shear Plane,	Pc	=	0.00	N

Cohesion and Friction Factors,

Condition 1

Cast-in-Place Concrete Slab on Clean Concrete Girder with Surface Roughened to an Amplitude of 6mm. (Normal Weight Concrete)

a. c	=	1.9	(N/mm2)
b. μ	=	1	
c. K1	=	0.3	
d. K2	=	12.4	

Condition 2

Cast-in-Place Concrete Slab on Clean Concrete Girder with Surface Roughened to an Amplitude of 6mm. (Light Weight Concrete)

a. c	=	1.9	(N/mm2)
b. μ	=	1	
c. K1	=	0.3	
d. K2	=	9	

Condition 3

Normal Weight Concrete Placed Monolithically

a. c	=	2.8	(N/mm2)
b. μ	=	1.4	
c. K1	=	0.25	
d. K2	=	10.3	

Condition 4

For Light Weight Concrete Placed Monolithically or Non Monolithically against a Clean Concrete Surface, Free of Laitance with Surface Roughened to an Amplitude of 6mm.

a. c	=	1.7	(N/mm2)
b. μ	=	1	
c. K1	=	0.25	
d. K2	=	6.9	

Condition 5

For Normal Weight Concrete Placed against a Clean Concrete Surface Free of Laitance with Roughened to an Amplitude of 6mm.

a. c	=	1.7	(N/mm2)
b. μ	=	1	
c. K1	=	0.25	
d. K2	=	10.3	

Condition 6

For Concrete Placed against a Clean Concrete Surface, Free of Laitance but not Intentionally Roughened.

a. c	=	0.52	(N/mm2)
b. μ	=	0.6	
c. K1	=	0.2	
d. K2	=	5.5	

Condition Applicable to Design of Slab, Condition 1

c	=	1.90
μ	=	1.00
K1	=	0.30
K2	=	12.40

Required Area of Shear Reinforcement,

$$Av = \frac{(Vnl \quad cAcv) \quad Pc}{\mu fy \qquad fy}$$

Point	Required Nominal Interface Resistance	Shear Resistance Provided by Concrete	Required Area of Reinforcement
	KN/m	KN/m	mm^2/m
0.00	738.66	965.20	Minimum
0.05	524.50	965.20	Minimum
0.15	402.58	965.20	Minimum
0.30	283.41	965.20	Minimum

MINIMUM AREA OF INTERFACE SHEAR REINFORCEMENT,

DGCS 12.5.4-3-1

$$Avf \geq \frac{0.35Acv}{fy}$$

For Cast in Place Concrete Slabs on Clean Concrete Girder Surface Free of Liatance, the following Provision shall Apply,

Minimum Interface Shear Reinforcement, Avf

Amount Required to Resist 1.33vui/Φv

Point	1.33vui/Φv	Required Area of Reinforcement	Minimum Interface Reinforcement	Maximum Spacing
	KN/m	mm^2/m	mm^2/m	mm
0.00	982.42	3508.64	635.00	356.21
0.05	697.59	2491.38	635.00	356.21
0.15	535.44	1912.27	635.00	356.21
0.30	376.94	1346.20	635.00	356.21

2) Design of the Bottom Slab

(a) 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	=	1,800	mm
	Girder Width,		bw	=	508.00	mm
DGCS 14.4.2.2	Effective Length,		Leff	=	784.00	mm
	Diameter of Main Bars,		db1	=	16	mm
	Diameter of Distribution Bars,		db2	=	16	mm
	Diameter of Temperature Bars,		db3	=	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	$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$					
	For Strength Limit State,					
	Factor Relating to Ductility,		ηD	=	1.05	
	Factor Relating to Redundancy,		ηR	=	1.05	
	Factor Relating to Operational Importance,		ηI	=	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,		ηD	=	1.00	
	Factor Relating to Redundancy,		ηR	=	1.00	
	Factor Relating to Operational Importance,		ηI	=	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,	$Mds = \frac{w \cdot Leff^2}{10}$	Mds	=	0.30	KN-m/m
	Weight of FWS,		wws	=	1.10	KN/m²
	Unfactored Deck Slab Positive or Negative Moment,	$Mws = \frac{wws \cdot Leff^2}{10}$	Mws	=	0.07	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		Unit
		Positive	Negative	
1.38	DC	0.41	0.41	KN-m/m
1.65	DW	0.11	0.11	KN-m/m
1.93	LL	54.58	37.29	KN-m/m
TOTAL		**55.10**	**37.80**	**KN-m/m**

Service I Limit State,

Factor	Force	Moment		Unit
		Positive	Negative	
1.00	DC	0.30	0.30	KN-m/m
1.00	DW	0.07	0.07	KN-m/m
1.00	LL	28.29	19.33	KN-m/m
TOTAL		**28.65**	**19.69**	**KN-m/m**

Design Factored Moment for IM = 75%

Dynamic Overload Factor, = 1.50

Strength I Limit State,

Factor	Force	Moment		Unit
		Positive	Negative	
1.38	DC	0.41	0.41	KN-m/m
1.65	DW	0.11	0.11	KN-m/m
1.93	LL	124.04	84.74	KN-m/m
TOTAL		**124.56**	**85.26**	**KN-m/m**

Service I Limit State,

Factor	Force	Moment		Unit
		Positive	Negative	
1.00	DC	0.30	0.30	KN-m/m
1.00	DW	0.07	0.07	KN-m/m
1.00	LL	64.29	43.92	KN-m/m
TOTAL		**64.65**	**44.28**	**KN-m/m**

DGCS 12.4.3.2 **Flexural Resistance,**

DGCS 12.4.3.3 **Limits of Reinforcement,**

1.33 Times the Factored Moment Required, Mcr = 1.33*Ma = 85.99 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	Spacing	Remarks
mm	mm	
16	150.00	ADEQUATE

Negative Moment : Top Bars		
Rebar	Spacing	Remarks
mm	mm	
16	200.00	ADEQUATE

For Impact Factor IM = 75%

Positive Moment : Bottom Bars		
Rebar	Spacing	Remarks
mm	mm	
16	70.00	ADEQUATE

Negative Moment : Top Bars		
Rebar	Spacing	Remarks
mm	mm	
16	100.00	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	1005.31	2872.31	2010.62	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	55.10	37.80	124.56	85.26	KN-m/m
Depth of Equivalent Sress Block	23.65	17.74	50.69	35.48	mm
Factored Flexural Resistance	71.02	54.39	137.51	102.04	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!	

DGCS 14.4.3.1 **Distribution of Reinforcement,**

Percent Distribution, $\frac{3840}{\sqrt{Leff}} \leq 67\%$ — %distribution = 67.00 %

Area of Distribution Bars, $As.dr = \%distribution \times As.tension$ — As.dr = 673.56 mm²

Spacing of Bottom Distribution Rebar, $s = \frac{Ab \times 1000}{As}$ — 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	200.00	

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	70.00	Adopt Spacing
Negative Moment	Top Bars	16	100.00	

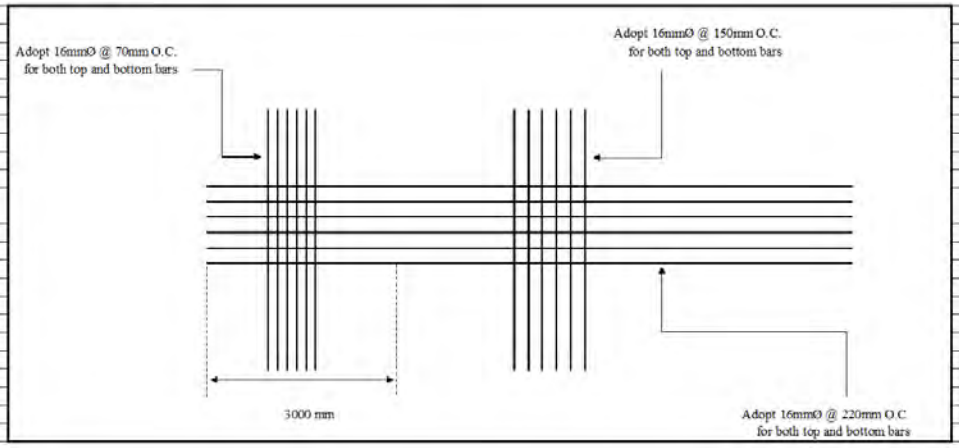
Distribution Reinforcement,

Adopt Distribution Reinforcement, Adopt 16mmØ @ 220.00 mm

Temperature and Shrinkage Reinforcement,

Adopt Distribution Reinforcement, Adopt 16mmØ @ 450.00 mm

Slab Reinforcement Layout,



(b) 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	=	500.00	mm
	Thickness of Sidewalk,		tsw	=	250.00	mm
	Area of Sidewalk,		Asw	=	125000.00	mm^2
	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/m^3
	Unit Weight,		γc	=	24.00	KN/m^3
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	=	26,752.90	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	=	3.00	KN/m
	Weight of Post,		Wpost	=	0.09	KN/m
	Weight of Railings,		Wrail	=	0.30	KN/m
	Undactored Dead Load,					

Member	Weight KN/m	Lever Arm m	Moment KN-m
Sidewalk	3.00	0.25	0.75
Post	0.09	0.48	0.04
Railings	0.30	0.48	0.14
TOTAL DC	**3.40**		**0.94**

DGCS 18	**Live Load Analysis,**					
	Live Load Intensity,		LL	=	15.00	KN
	Live Load Moment Arm,		LLarm	=	150.00	mm
	Dynamic Load Allowance (For All Other Limit State),		IM	=	1.33	
	Distance of Design Section to Face of Railing,		d	=	450.00	mm
	Design Unfactored Shear with Impact Factor,		V	=	19.95	KN
	Design Unfactored Bending Moment with Impact Factor, Negative Moment,		M	=	2.99	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	$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$					
	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	4.68	1.29
1.93	LL	28.94	5.77
TOTAL		**33.62**	7.07

	Design of Main Reinforcement,				
	Maximum Factored Moment,	Mu	=	7.07	KN-m
DGCS 12.4.3.3	Limits of Reinforcement,	Mcr	=	9.40	KN-m
	1.33 Times the Factored Moment Required,	1.33*Mu	=	9.40	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_2	=	0.75	
	Verification of Governing Factored Flexural Resistance,				
	Governing Factored Flexural Resistance for Flexural Design,	Mu	=	9.40	KN-m
DGCS 12.2.3.1	Flexural Resistance Factor,	Φf	=	0.90	
	Area of Flexural Reinforcement,	m	=	17.65	
		x	=	0.001	
		r	=	0.001	
		As	=	123.77	mm^2
	Required Bar Spacing, $s = Ab * b/As$	s	=	1500.00	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 mmo Tension Reinforcement Bar spaced @ 375mm O.C.				
	Shear Verification,				
	Maximum Factored Shear,	Vu	=	33.62	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	=	200.91	mm
	Shear Depth,	de	=	202.00	mm
	Shall be Less Than the Greater of,				
	0.90*de		=	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	=	176.48	KN
	Nominal Reinf. Shear Resistance, $Vs = \frac{Av * fy * dv * cot\theta}{s}$	Vs	=	45.24	KN
	Nominal Shear Resistance,				
DGCS 12.5.3.2-1	$Vn1 = Vc + Vs$	Vn1	=	221.72	KN
DGCS 12.5.3.2-2	$Vn2 = 0.25 * f'c * bv * dv$	Vn2	=	1406.36	KN
	Governing Nominal Shear Resistance,	Vn	=	221.72	KN
	Factored Shear Resistance Verification,				
	Shear Resistance Factor,	Φv	=	0.90	
	Factored Shear Resistance, $Vr = \phi v * Vn$	Vr	=	199.55	KN
	∴ Shear Reinforcement is Adequate				

(2) Long Span: L = 31.00 m

1) Design of the Main Girder

DGCS 12.6.3 **Stresses Limitations for Prestressing Tendons,**

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars (MPa)	Tendon Type: Low Relaxation Strand (MPa)	Tendon Type: Deformed High-Strength Bars (MPa)
Pretensioning			
Immediately prior to transfer *(fpbt)*	1302.00	1395.00	-
At service limit state after all losses *(fpe)*	1339.20	1339.20	1339.20
Post-Tensioning			
Prior to seating-short-term *fpbt* may be allowed	1506.60	1506.60	1506.60
At anchorages and couplers immediately after anchor set	1302.00	1302.00	1302.00
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	1302.00	1376.40	1302.00
At service limit state after losses *(fpe)*	1339.20	1339.20	1339.20

DGCS 12.6.4.1 **For Temporary Stresses Before Losses-Fully Prestressed Components,**

Compression Stresses, 0.60 f'ci = 21.00 MPa

DGCS 12.6.3.1-1 **Tension Stresses,**

Bridge Type	Location	Stress Limit (MPa)
Other Than Segmentally-Constructed Bridges	In precompressed tensile zone without bonded reinforcement	N/A
	In areas other than the precompressed tensile zone and without bonded reinforcement	1.38
	In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.50 fy, not to exceed 210 MPa.	3.73
	For handling stresses in prestressed piles	2.46
Segmentally-Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	
	Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of 0.5 fy, with internal tendons or external tendons	1.48
	Joints without minimum bonded auxiliary reinforcement through the joints	0.00
	Transverse Stresses Through Joints For any type of joint	1.48
	Stresses in Other Areas	
	For areas without bonded non-prestressed reinforcement	0.00
	In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.50 fy, not to exceed 210 MPa	2.96
	Principal Tensile Stress at Neutral Axis in Web	
	All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures	1.71

DGCS 12.6.4.2 **For Stresses at Service Limit State after Losses-Fully Prestressed Components,**

DGCS 12.6.4.2-1 **Compression Stresses,**

Location		Stress Limit (MPa)
In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	Girder	20.25
	Deck	12.60
In segmentally constructed bridges due to the sum of effective prestress and permanent loads	Girder	20.25
	Deck	12.60
Due to the sum of effective prestress, permanent loads, and transient loads and during shipping and handling	Girder	27.00
	Deck	16.80

DGCS 12.6.4.2-2 **Tension Stresses,**

Bridge Type	Location	Stress Limit (MPa)
Other Than Segmentally-Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	3.35
	For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	1.68
	For components with unbonded prestressing tendons	0.00
Segmentally Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	
	Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0.5 fy; internal tendons or external tendons	1.68
	Joints without the minimum bonded auxiliary reinforcement through joints	0.00
	Transverse Stresses Through Joints Tension in the transverse direction in precompressed tensile zone	1.68
	Stresses in Other Areas	
	For areas without bonded reinforcement	0.00
	In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 fy; not to exceed 205 Mpa.	3.35
	Principal Tensile Stress at Neutral Axis in Web	
	All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures.	1.94

Estimate of Required Number of Strands,

Estimated Center of Gravity of Strands from Bottom of Girder, = 210.00 mm

= 210.00 mm

Limit Cracking due to Tensile Stress,

Tensile Stress Limit for Concrete at Service Load, = 3.35 MPa

Required Pre-Compressive Stress at the Bottom Fiber, = 12.82 MPa

Required Effective Prestressing Force, P = 2068.61 KN

$$P = \frac{\left|\left(\frac{M}{St}\right) - \sigma\right|}{\left|\left(\frac{e}{St}\right) + \frac{1}{A}\right|}$$

No Tensile Stress during Service Load,

Required Effective Prestressing Force, P = 1413.76 KN

$$P = \frac{\left|\left(\frac{M}{Sb}\right)\right|}{\left|\left(\frac{e}{Sb}\right) + \frac{1}{A}\right|}$$

Estimated Prestress Loss vs. Actual Prestress Loss,

Estimated Percentage of Prestress Loss, = 51.93790 %

Actual Percentage of Prestress Loss, = 51.93790 %

Estimated vs. Actual Percentage of Prestress Loss, = 0.00000 %

Required Number of Strands,

Estimated Effective Jacking Force, = 4304.04 KN

Allowable Stress for Prestressing Steel near Anchorage,

Estimated Area of Prestressing Steel, = 3305.72 mm^2

Estimated Number of Strands Required, = 33.49 strands

= 34.00 strands

Number of Strands, = 48.00 strands

Number of Identical Tendons, N = 4.00

Pattern of Strands at Midspan

Row	Centroid yb	No. of Strand n	n*yb	e_{mid}
1	300.00	12.00	3600.00	511.208
2	300.00	12.00	3600.00	511.208
3	120.00	12.00	1440.00	691.208
4	120.00	12.00	1440.00	691.208
5	0.00	0.00	0.00	0.000
6	0.00	0.00	0.00	0.000
7	0.00	0.00	0.00	0.000
8	0.00	0.00	0.00	0.000
9	0.00	0.00	0.00	0.000
10	0.00	0.00	0.00	0.000
TOTAL		48.00	10080.00	

Computed Centroid at Mid-Span vs. Estimated Centroid at Mid-Span,

Computed Centroid at Mid-Span, $y_{mcomputed}$ = 210.00 mm

Estimated Centroid at Mid-Span, $y_{mestimated}$ = 210.00 mm

Relative Error, %error = 0.00 %

Computed Total Area of Prestressing Steel, Aps = 4737.60 mm^2

Computed Eccentricity at Mid-Span, $e_{mcomputed}$ = 601.21 mm

PRESTRESS LOSS,

Summary of Losses,

Ducts	Jacking Stage		
	Po (N)	Aps	fpj
1	6173000.00	1,184.40	1674.00
2	6173000.00	1,184.40	1674.00
3	6173000.00	1,184.40	1674.00
4	6173000.00	1,184.40	1674.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
Whole Girder	24692000.00	4737.60	1674.00

Ducts	Total Prestress Losses at Transfer		
	Δfpa (MPa)	Δfpf (MPa)	ΔfpES (MPa)
1	62.28	19.77	27.89
2	62.28	14.85	27.89
3	62.28	14.36	27.89
4	62.28	9.43	27.89
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
Whole Girder	62.28	19.77	111.56

Total Prestress Loss at Service,

Ducts	Prestress Loss between Time of Transfer and Deck Placement		
	ΔfpSR (MPa)	[illegible]	ΔfpR1 (MPa)
1	12.59	58.43	16.50
2	12.59	58.43	16.70
3	12.59	58.43	16.72
4	12.59	58.43	16.92
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
Whole Girder	50.35	233.74	16.92

Ducts	Prestress Loss between After Deck Placement			
	ΔfpSD (MPa)	[illegible]	ΔfpR2 (MPa)	ΔfpSS (MPa)
1	17.02	74.73	16.50	-2.26
2	17.02	74.71	16.70	-2.26
3	17.02	74.71	16.72	-2.26
4	17.02	74.70	16.92	-2.26
5	0.00	0.00	0.00	0.00
6	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00
8	0.00	0.00	0.00	0.00
9	0.00	0.00	0.00	0.00
10	0.00	0.00	0.00	0.00
Whole Girder	68.09	298.85	16.92	-9.04

Ducts	Pe (KN)
1	1,623,284.15
2	1,628,648.85
3	1,629,186.08
4	1,634,565.89
5	-
6	-
7	-
8	-
9	-
10	-
Whole Girder	6,515,684.97

TABLE SUMMARY OF TENSION FORCES,

Cable No.	Jacking Force (Po) (N)	Effective Prestressing Force (Pe) (N)
1	6,173,000.00	1,623,284.15
2	6,173,000.00	1,628,648.85
3	6,173,000.00	1,629,186.08
4	6,173,000.00	1,634,565.89

	Jacking Stress per Strand,			=	5211.92 MPa	
	Total Prestress Loss at Transfer,			=	193.61 MPa	
	Total Prestress Loss at Service,			=	675.83 MPa	
	Stress per Strand at Transfer,			=	5018.31 MPa	
	Stress per Strand at Service,			=	4342.48 MPa	
	Total Prestress Losses at Service between Time of Transfer and Deck Placement,					
	$\Delta fpid = \Delta fpSR + \Delta fpCR + \Delta fpR1$	Δfpid	=	301.01 MPa		
	Total Prestress Losses at Service After Deck Placement,					
	$\Delta fpdf = \Delta fpSD + \Delta fpCD + \Delta fpR2 \quad \Delta fpSS$	Δfpdf	=	374.83 MPa		
	Total Prestress Loss,			=	869.44 MPa	
	Percentage of Total Losses,			=	51.94 %	
	Check for Prestressing Limit,					
	Allowable Prestress Limit at Service,			=	1339.20 MPa	
	Stress in Strands after Losses,			=	804.56 MPa	**OK!**
	Final Effective Prestressing Force after Losses,			=	3811.68 KN	
DGCS 12.6.5.2	**Instantaneous Losses,**					
	Anchorage Set,	Δfpa	=	62.28 MPa		
	Friction Losses,					
DGCS 12.6.5.2-1	Between the Prestressing Tendons and the Duct Wall, $\Delta fpF = fpj(1 - e^{-(Kx+\mu\alpha)})$					
DGCS 12.6.5.2-2	Between the External Tendons across a Single Deviator Pipe, $\Delta fpF = fpj(1 \quad e^{-\mu(\alpha+0.03)})$					
	Wobble Friction,	K	=	6.60E-07		
	Coefficient of Friction,	μ	=	0.23		

Stress in Prestressing Steel During Jacking,

Duct	Jacking Force (Pj)	No. of Strands (N)	Jacking Stress (fpj)	
1	6,173,000.00	12.00	YIELD	1674.00
2	6,173,000.00	12.00	YIELD	1674.00
3	6,173,000.00	12.00	YIELD	1674.00
4	6,173,000.00	12.00	YIELD	1674.00
5	0.00	0.00		
6	0.00	0.00		
7	0.00	0.00		
8	0.00	0.00		
9	0.00	0.00		
10	0.00	0.00		
TOTAL	**24,692,000.00**			

Sum of Absolute Values of Angular Change of Prestressing Steel,

Duct	Centroid at Ends	Centroid at Mid	SAG	x	a
1	1100.00	300.00	800.00	15.527	0.052
2	900.00	300.00	600.00	15.515	0.039
3	700.00	120.00	580.00	15.514	0.037
4	500.00	120.00	380.00	15.506	0.025
5	0.00	0.00	0.00		
6	0.00	0.00	0.00		
7	0.00	0.00	0.00		
8	0.00	0.00	0.00		
9	0.00	0.00	0.00		
10	0.00	0.00	0.00		

Computed Prestress Loss due to Friction,

Duct	Jacking Stress (fpj)	$1 - e^{-(Kx+\mu\alpha)}$	Friction Loss (Δfpf)
1	1674.00	0.012	19.77
2	1674.00	0.009	14.85
3	1674.00	0.009	14.36
4	1674.00	0.006	9.43
5			
6			
7			
8			
9			
10			

DGCS 12.6.5.2-4	Elastic Shortening, $\Delta fpES = \frac{N-1}{2N}\frac{Ep}{Eci} fcgp$					
	Sum of Concrete Stresses at Center of Gravity of Tendon after Transfer, $fcgp = \left(\frac{Pi}{A}\right) + \left[\left(\frac{Pi * ec^2}{I}\right)\right] \left[\frac{(Mg * ec)}{I}\right]$	fcgp	=	45.17 MPa		
	NOTE :					
	fcgp values may be calculated using a steel stress reduced below the initial value by a margin					
	dependent on elastic shortening, relaxation, and friction effects,					
	Estimated Prestress Loss,	$Loss_{est}$	=	2.158960		
	Eccentricity of Strands from Bottom of Girder at Midspan,	ec	=	550.00 mm		
	Total Prestressing Force at Release (w/ Anchorage Loss),	Pi	=	2.45E+07 N		
	Moment due to Selfweight at Midspan,	Mg	=	3929.75 KN-m		
	Computed Loss due to Elastic Shortening vs. Estimated Loss,					
	Computed Loss,	ΔfpES	=	111.56 MPa		
	Percentage of Loss,	$Loss_{per}$	=	2.158960 %		
	Relative Error,	$Error_{rel}$	=	0.00		**OK!**

DGCS 12.6.5.4 **Refined Estimates of Time-Dependent Losses,**

DGCS 12.6.5.4-1 The change in prestressing steel stress due to time dependent loss, ΔfpLT, shall be determined as follows :

$$\Delta fpLT = (\Delta fpSR + \Delta fpCR + \Delta fpR1)_{id} + (\Delta fpSD + \Delta fpCD + \Delta fpR2 \quad \Delta fpSS)_{df}$$

where :

ΔfpSR	=	prestress loss due to shrinkage of girder concrete between transfer and deck placement (Mpa)
ΔfpCR	=	prestress loss due to creep of girder concrete between transfer and deck placement (MPa)
ΔfpR1	=	prestress loss due to relaxation of prestressing strands between time of transfer and deck placement (MPa)
ΔfpR2	=	prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time (MPa)
ΔfpSD	=	prestress loss due to shrinkage of girder concrete between time of deck placement and final time (MPa)
ΔfpCD	=	prestress loss due to creep of girder concrete between time of deck placement and final time (MPa)
ΔfpSS	=	prestress gain due to shrinkage of deck in composite section (MPa)
$(\Delta fpSR + \Delta fpCR + \Delta fpR1)_{id}$	=	sum of time-dependent prestress losses between transfer and deck placement (MPa)
$(\Delta fpSD + \Delta fpCD + \Delta fpR2 - \Delta fpSS)_{df}$	=	sum of time-dependent prestress losses after deck placement (MPa)

Accounts for Losses and Gains between Time of Deck Placement to Final Time,

Duct	ΔfpSR + ΔfpCR + ΔfpR1	ΔfpSD + ΔfpCD + ΔfpR2 - ΔfpSS
1	87.52	150.28
2	87.72	150.47
3	87.74	150.48
4	87.94	150.67
5	0.00	0.00
6	0.00	0.00
7	0.00	0.00
8	0.00	0.00
9	0.00	0.00
10	0.00	0.00

Losses: Time of Transfer to Time of Deck Placement,

Shrinkage of Girder Concrete,

Prestress Loss due to Shrinkage of Girder Concrete between Transfer and Deck Placement, ΔfpSR = 50.35 MPa

DGCS 12.6.5.4-2 $\Delta fpSR = \varepsilon_{bid} E_p K_{id}$

in which :

Transformed Section Coefficient that Accounts for Time-Dependent interaction between Concrete and Bonded Steel in the Section being Considered for Time Period between Transfer and Deck Placement, Kid = 0.84

DGCS 12.6.5.4-3 $$K_{id} = \frac{1}{1 + \frac{E_p A_{ps}}{E_{ci} A_g}\left(1 + \frac{A_g e^2_{pg}}{I_g}\right)[1 + 0.7\varphi_b(t_f, t_i)]}$$

where :

Concrete Shrinkage Strain of Girder between the Time of Transfer and Deck Placement, εbid = 0.000303

Eccentricity of Prestressing Force with Respect to Centroid of Girder; Positive in Common Construction where it is Below Girder Centroid, epg = 601.21 mm

Girder Creep Coefficient at Final Time due to Loading Introduced at Transfer, Ψb(tf,ti) = 1.25

Final Age tf = 9000.00 day's

Age at Transfer ti = 3.00 day's

Creep of Girder Concrete,

Prestress Loss due to Creep of Girder Concrete between Transfer and Deck Placement, ΔfpCR = 233.74 MPa

DGCS 12.6.5.4-4 $$\Delta fpCR = \frac{E_p}{E_{ci}} f_{cgp} \varphi_b(t_f, t_i) K_{id}$$

where :

Girder Creep Coefficient at Time of Deck Placement due to Loading Introduced at Transfer, Ψb(td,ti) = 0.93

Age at Deck Placement (days), td = 120.00 day's

Relaxation of Prestressing Strands,

Prestress Loss due to Relaxation of Prestressing Strands between Time of Transfer and Deck Placement, ΔfpR1

DGCS 12.6.5.4-5 $$\Delta fpR1 = \frac{f_{pt}}{K_L}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)$$

where :

Stress in Prestressing Strands Immediately After Transfer, Taken Not Less Than 0.55fpy, fp

30 for Low Relaxation Strands and 7 for Other Prestressing Steel, Unless More Accurate Manufacturer's Data are Available, K_L = 30.00

Computed Prestress Loss due to Relaxation,

Duct	fp (MPa)	ΔfpR1 (MPa)
1	1480.39	16.50
2	1485.31	16.70
3	1485.80	16.72
4	1490.73	16.92
5	0.00	0.00
6	0.00	0.00
7	0.00	0.00
8	0.00	0.00
9	0.00	0.00
10	0.00	0.00

Losses: Time of Deck Placement to Final Time,

Shrinkage of Girder Concrete,

Prestress Loss due to Shrinkage of Girder Concrete between
Time of Deck Placement and Final Time, $\Delta fpSD$ = 68.09 MPa

DGCS 12.6.5.4-6 $\Delta fpSD = \varepsilon_{bdf} E_p K_{df}$

in which :

Transformed Section Coefficient that Accounts for Time-Dependent Interaction between Concrete and Bonded Steel in the Section
being Considered for Time Period between Deck Placement and Final Time, Kdf = 0.85

DGCS 12.6.5.4-7 $$K_{df} = \frac{1}{1 + \frac{E_p A_{ps}}{E_{ci} A_c}\left(1 + \frac{A_c e_{pc}^2}{I_c}\right)\left[1 + 0.7\varphi_b(t_f, t_i)\right]}$$

where :

Shrinkage Strain of Girder between Time of Deck Placement and Final Time, εbdf = 0.000407
Eccentricity of Prestressing Force with Respect to Centroid of Composite Section; Positive in Typical Construction
where Prestressing Force is Below Centroid of Section, epc = 945.20 mm
Area of Section Calculated using the Gross Composite Concrete Section Properties of the Girder and the Deck
and the Deck-to-Girder Modular Ratio, Ac = 1021669.23 mm^2
Moment of Inertia of Section Calculated using the Gross Composite Concrete Section Properties of the Girder
and the Deck and the Deck-to-Girder Modular Ratio at Service, Ic = 4.33754E+11 mm^4

Creep of Girder Concrete,

Prestress Loss due to Creep of Girder Concrete between
Time of Deck Placement and Final Time, $\Delta fpCD$

DGCS 12.6.5.4-8 $$\Delta fpCD = \frac{E_p}{E_{ci}} f_{cgp} \varphi_b\left[(t_f, t_i) - \varphi_b(t_d, t_i)\right] K_{df} + \frac{E_p}{E_c} \Delta f_{cd} \varphi_b(t_f, t_d) K_{df}$$

where :

Change in Concrete Stress at Centroid of Prestressing Strands due to Long-Term Losses between
Transfer and Deck Placement, Combined with Deck Weight and Superimposed Loads, Δfcd

$$\Delta f_{cd} = -(\Delta f_{pSR} + \Delta_{cR} + \Delta f_{pR1})\frac{A_{ps}}{A_g}\left(1 + \frac{A_g e_{pg}^2}{I_g}\right) - \left(\frac{M_s e_{psf}}{I_{cf}} + \frac{(M_b + M_{ws}) e_{pc}}{I_c}\right)$$

Girder Creep Coefficient at Final Time due to Loading at Deck Placement, Ψb(tf,td) = 0.81

Duct	ΔfpSR + ΔfpCR + ΔfpR1	Δfcd (MPa)	ΔfpCD
1	87.52	-1.28	74.73
2	87.72	-1.28	74.71
3	87.74	-1.28	74.71
4	87.94	-1.29	74.70
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00

Relaxation of Prestressing Strands,

Prestress Loss due to Relaxation of Prestressing Strands in Composite Section between
Time of Deck Placement and Final Time, $\Delta fpR2$

DGCS 12.6.5.4-9 $\Delta fpR2 = \Delta fpR1$

Duct	ΔfpR2 (MPa)
1	16.50
2	16.70
3	16.72
4	16.92
5	0.00
6	0.00
7	0.00
8	0.00
9	0.00
10	0.00

Shrinkage of Deck Concrete,

Prestress Gain due to Shrinkage of Deck in Composite Section

DGCS 12.6.5.4-10 $$\Delta fpSS = \frac{E_p}{E_c} \Delta f_{cdf} K_{df}\left[1 + 0.7\varphi_b(t_f, t_d)\right]$$ $\Delta fpSS$ = -9.04 MPa

in which :

Change in Concrete Stress at Centroid of Prestressing Strands due to
Shrinkage of Deck Concrete, Δfcdf = -1.03 MPa

DGCS 12.6.5.4-11 $$\Delta f_{cdf} = \frac{\varepsilon_{ddf} A_d E_{cd}}{\left[1 + 0.7\varphi_b(t_f, t_d)\right]}\left(\frac{1}{A_c} - \frac{e_{pc} e_d}{I_c}\right)$$

where :

Shrinkage Strain of Deck Concrete between Placement and Final Time, εddf = 0.000407
Area of Deck Concrete, Ad = 283971.83 mm^2
Modulus of Elasticity of Deck Concrete, Ecd = 26752.50 MPa
Eccentricity of Deck with Respect to the Gross Composite Section, Positive in Typical Construction
where Deck is Above Girder, ed = 688.99 mm
Creep Coefficient of Deck Concrete at Final Time due to Loading Introduced Shortly
after Deck Placements (i.e. overlays, barriers, etc.), Ψb(tf,td) = 0.81

TRANSFORMED SECTION PROPERTIES,

Modulus of Elasticity of Beam at Transfer,	Eci	=	29910.20 MPa
Modulus of Elasticity of Beam at Service,	Ec	=	33914.98 MPa
Modulus of Elasticity of Prestressing Strands,	Eps	=	197000.00 MPa
Modular Ratio of Strands to Beam at Transfer,	n_{pi}	=	6.59
Modular Ratio of Strands to Beam at Service,	n_{pc}	=	5.81
Area of Prestressing Strands,	Aps	=	98.70 mm^2
Transformed Area of Prestressing Strands at Transfer,	(Aps (n_{pi}-1))	=	26466.04 mm^2
Transformed Area of Prestressing Strands at Service,	(Aps (n_{pc}-1))	=	22781.42 mm^2
Midspan Center of Gravity of Strands,	Ybot	=	210.00 mm

Girder and Strands (Initial, Transfer),

Element	A	Ybot	Ay	Io	D	AD^2
Girder	6.5355E+05	8.1121E+02	5.3016E+08	2.1664E+11	2.3399E+01	3.5782E+08
Strands	2.6466E+04	2.1000E+02	5.5579E+06	0.0000E+00	5.7781E+02	8.8360E+09
TOTAL	6.8001E+05		5.3572E+08	2.1664E+11		9.1939E+09

Transformed Initial :

Neutral Axis from Extreme Bottom Fiber,	Ybot	=	787.81 mm
Neutral Axis from Extreme Top Fiber,	Ytop	=	812.39 mm
Moment of Inertia About X-axis,	Ix	=	2.26E+11 mm^4
Section Modulus from Bottom Neutral Axis,	Sxb	=	2.87E+08 mm^3
Section Modulus from Top Neutral Axis,	Sxt	=	2.78E+08 mm^3
Eccentricity,	e	=	577.81 mm

Girder and Strands (Final),

Element	A	Ybot	Ay	Io	D	AD^2
Girder	6.5355E+05	8.1121E+02	5.3016E+08	2.1664E+11	2.0251E+01	2.6802E+08
Strands	2.2781E+04	2.1000E+02	4.7841E+06	0.0000E+00	5.8096E+02	7.6890E+09
TOTAL	6.7633E+05		5.3495E+08	2.1664E+11		7.9570E+09

Transformed Service :

Neutral Axis from Extreme Bottom Fiber,	Ybot	=	790.96 mm
Neutral Axis from Extreme Top Fiber,	Ytop	=	809.24 mm
Moment of Inertia About X-axis,	Ix	=	2.25E+11 mm^4
Section Modulus from Bottom Neutral Axis,	Sxb	=	2.84E+08 mm^3
Section Modulus from Top Neutral Axis,	Sxt	=	2.78E+08 mm^3
Eccentricity,	e	=	580.96 mm

Composite Girder and Deck,

Element	A	Ybot	Ay	Io	D	AD^2
Girder	6.5355E+05	8.1121E+02	5.3016E+08	2.1664E+11	2.5578E+02	4.2757E+10
Strands	2.2781E+04	2.1000E+02	4.7841E+06	0.0000E+00	8.5699E+02	1.6731E+10
Slab	2.8397E+05	1.8002E+03	5.1121E+08	9.4657E+08	7.3321E+02	1.5266E+11
Haunch	8.4150E+04	8.1121E+02	6.8263E+07	7.0125E+07	2.5578E+02	5.5054E+09
TOTAL	1.0445E+06		1.1144E+09	2.1766E+11		2.1766E+11

Transformed Service Composite :

Neutral Axis from Extreme Bottom Fiber,	Ybot	=	1066.99 mm
Neutral Axis from Extreme Top Fiber,	Ytop	=	633.21 mm
Neutral Axis from Extreme Top Fiber for Girder,	Ytopg	=	533.21 mm
Moment of Inertia About X-axis,	Ix	=	4.35E+11 mm^4
Section Modulus from Bottom Neutral Axis,	Sxb	=	4.08E+08 mm^3
Section Modulus from Top Neutral Axis,	Sxt	=	6.87E+08 mm^3
Section Modulus from Top Neutral Axis of Girder,	Sxtg	=	8.16E+08 mm^3
Eccentricity,	e	=	856.99 mm

SUMMARY OF SECTION PROPERTIES,

	NON-COMPOSITE SECTION			COMPOSITE SECTION	
	Gross	Transformed at Transfer	Transformed at Service	Gross	Transformed
Area, mm^2	653547.08	680013.12	6.7633E+05	1021669.23	1044450.65
Neutral Axis from Extreme Bottom Fiber,	811.21	787.81	790.96	1155.20	1066.99
Neutral Axis from Extreme Top Fiber, mm	788.99	812.39	809.24	445.00	633.21
Moment of Inertia, mm^4	2.17E+11	2.26E+11	2.25E+11	4.34E+11	4.35E+11
Eccentricity, mm	601.21	577.81	580.96	945.20	856.99

STRESS AT TRANSFER,

Force per Strand at Transfer,

Stress per Strand at Transfer, fpi = 916.12 MPa

Total Prestressing Force at Transfer, $Pi = fpi * Aps$ Pi = 4340.21 KN

DGCS 12.6.4 **Stress Limits for Concrete,**

DGCS 12.6.4.1 Compression Stresses,

Compressive Stress Limit for Pretensioned and Post-Tensioned Concrete, 0.60 f'ci = 21.00 MPa

Tension Stresses, = 3.73 MPa

Stress at Transfer Length,

DGCS 12.8.4 Transfer Length, Ltransfer = 762.00 mm

Moment due to Beam Selfweight at Transfer, Mg = 180.70 KN-m

Due to Camber of the Beam, The Effective Beam Length shall be Taken as the Whole Beam Length,

$$Mg = \frac{Wx}{2}(L - x)$$

Beam Uniform Load, W = 15.69 KN/m

Transfer Length, x = 0.76 m

Beam Span, Lspan = 31.00 m

Stress at the Beam,

Top Stress,

$$ft = \frac{Pi}{At.transformed} + \frac{Pi * e}{St.transformed} - \frac{Mg}{St.transformed}$$

ft = 14.75 MPa

ft < 21.00 MPa **OK!**

Area of Girder, A = 680013.12 mm^2

Eccentricity of Prestressing Strand, e = 577.81 mm

Section Modulus, St.transformed = 2.78E+08 mm^3

Bottom Stress,

$$fb = \frac{Pi}{At.transformed} - \frac{Pi * e}{Sb.transformed} + \frac{Mg}{Sb.transformed}$$

fb = -1.74 MPa

fb > -3.73 MPa **OK!**

Section Modulus, St.transformed = 2.87E+08 mm^3

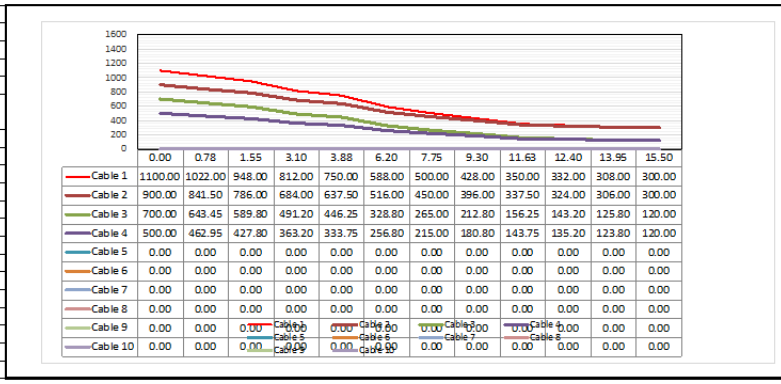
Stress at Transfer Length (Considering Effect of Harping),

Cable Profile,

$$y = \frac{4 * Sag}{L^2}(0.50L - x)^2 + y_{mid.span}$$

Point	Distance	1	2	3	4	5
0.000	0.000	1100.00	900.00	700.00	500.00	0.00
0.025	0.775	1022.00	841.50	643.45	462.95	0.00
0.050	1.550	948.00	786.00	589.80	427.80	0.00
0.100	3.100	812.00	684.00	491.20	363.20	0.00
0.125	3.875	750.00	637.50	446.25	333.75	0.00
0.200	6.200	588.00	516.00	328.80	256.80	0.00
0.250	7.750	500.00	450.00	265.00	215.00	0.00
0.300	9.300	428.00	396.00	212.80	180.80	0.00
0.375	11.625	350.00	337.50	156.25	143.75	0.00
0.400	12.400	332.00	324.00	143.20	135.20	0.00
0.450	13.950	308.00	306.00	125.80	123.80	0.00
0.500	15.500	300.00	300.00	120.00	120.00	0.00

Point	Distance	6	7	8	9	10
0.000	0.000	0.00	0.00	0.00	0.00	0.00
0.025	0.775	0.00	0.00	0.00	0.00	0.00
0.050	1.550	0.00	0.00	0.00	0.00	0.00
0.100	3.100	0.00	0.00	0.00	0.00	0.00
0.125	3.875	0.00	0.00	0.00	0.00	0.00
0.200	6.200	0.00	0.00	0.00	0.00	0.00
0.250	7.750	0.00	0.00	0.00	0.00	0.00
0.300	9.300	0.00	0.00	0.00	0.00	0.00
0.375	11.625	0.00	0.00	0.00	0.00	0.00
0.400	12.400	0.00	0.00	0.00	0.00	0.00
0.450	13.950	0.00	0.00	0.00	0.00	0.00
0.500	15.500	0.00	0.00	0.00	0.00	0.00



	0.00	0.78	1.55	3.10	3.88	6.20	7.75	9.30	11.63	12.40	13.95	15.50
Cable 1	1100.00	1022.00	948.00	812.00	750.00	588.00	500.00	428.00	350.00	332.00	308.00	300.00
Cable 2	900.00	841.50	786.00	684.00	637.50	516.00	450.00	396.00	337.50	324.00	306.00	300.00
Cable 3	700.00	643.45	589.80	491.20	446.25	328.80	265.00	212.80	156.25	143.20	125.80	120.00
Cable 4	500.00	462.95	427.80	363.20	333.75	256.80	215.00	180.80	143.75	135.20	123.80	120.00
Cable 5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cable 10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Angular Change of Cable per Point,

$$\frac{dy}{dx} = \frac{8 \cdot Sag}{L^2}(0.50L - x)\mathbf{1} = \alpha$$

Point	Distance	1	2	3	4	5
0.000	0.000	0.103	0.077	0.075	0.049	0.000
0.025	0.775	0.098	0.074	0.071	0.047	0.000
0.050	1.550	0.093	0.070	0.067	0.044	0.000
0.100	3.100	0.083	0.062	0.060	0.039	0.000
0.125	3.875	0.077	0.058	0.056	0.037	0.000
0.200	6.200	0.062	0.046	0.045	0.029	0.000
0.250	7.750	0.052	0.039	0.037	0.025	0.000
0.300	9.300	0.041	0.031	0.030	0.020	0.000
0.375	11.625	0.026	0.019	0.019	0.012	0.000
0.400	12.400	0.021	0.015	0.015	0.010	0.000
0.450	13.950	0.010	0.008	0.007	0.005	0.000
0.500	15.500	0.000	0.000	0.000	0.000	0.000

Point	Distance	6	7	8	9	10
0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.025	0.775	0.000	0.000	0.000	0.000	0.000
0.050	1.550	0.000	0.000	0.000	0.000	0.000
0.100	3.100	0.000	0.000	0.000	0.000	0.000
0.125	3.875	0.000	0.000	0.000	0.000	0.000
0.200	6.200	0.000	0.000	0.000	0.000	0.000
0.250	7.750	0.000	0.000	0.000	0.000	0.000
0.300	9.300	0.000	0.000	0.000	0.000	0.000
0.375	11.625	0.000	0.000	0.000	0.000	0.000
0.400	12.400	0.000	0.000	0.000	0.000	0.000
0.450	13.950	0.000	0.000	0.000	0.000	0.000
0.500	15.500	0.000	0.000	0.000	0.000	0.000

Cable Centroids at Transfer Length,

Tendon	Centroid (Yb)	No. of Strands (N)	N * Yb
1	1023.28	12	12279.30
2	842.46	12	10109.48
3	644.37	12	7732.50
4	463.56	12	5562.67
5	0.00	0	0.00
6	0.00	0	0.00
7	0.00	0	0.00
8	0.00	0	0.00
9	0.00	0	0.00
10	0.00	0	0.00
TOTAL		48	35683.95

Center of Gravity of Tendons at Transfer Length, = 743.42 mm

Eccentricity at Transfer Length, = 44.39 mm

Stress at the Beam,

Top Stress,

$$ft = \frac{Pi}{At.transformed} + \frac{Pi \cdot e}{St.transformed} - \frac{Mg}{St.transformed}$$

ft = 6.43 MPa

ft < 21.00 MPa **OK!**

Area of Girder, A = 680013.12 mm^2

Eccentricity of Prestressing Strand, e = 44.39 mm

Section Modulus, St.transformed = 2.78E+08 mm^3

Bottom Stress,

$$fb = \frac{Pi}{At.transformed} - \frac{Pi \cdot e}{Sb.transformed} + \frac{Mg}{Sb.transformed}$$

fb = 6.34 MPa

fb < 21.00 MPa **OK!**

Section Modulus, St.transformed = 2.87E+08 mm^3

Stress at Midspan,

Moment due to Beam Selfweight at Transfer, Mg = 1884.18 KN-m

Due to Camber of the Beam, The Effective Beam Length shall be Taken as the Whole Beam Length,

$$Mg = \frac{Wx}{2}(L - x)$$

Beam Uniform Load, W = 15.69 KN/m

Transfer Length, x = 15.50 m

Beam Span, Lspan = 31.00 m

Stress at the Beam,

Top Stress,

$$ft = \frac{Pi}{At.transformed} + \frac{Pi \cdot e}{St.transformed} - \frac{Mg}{St.transformed}$$

ft = 8.63 MPa

ft < 21.00 MPa **OK!**

Area of Girder, A = 680013.12 mm^2

Eccentricity of Prestressing Strand, e = 577.81 mm

Section Modulus, St.transformed = 2.78E+08 mm^3

Bottom Stress,

$$fb = \frac{Pi}{At.transformed} - \frac{Pi \cdot e}{Sb.transformed} + \frac{Mg}{Sb.transformed}$$

fb = 4.21 MPa

fb < 21.00 MPa **OK!**

Section Modulus, St.transformed = 2.87E+08 mm^3

STRESS AT SERVICE ,

Force per Strand at Transfer,

Stress per Strand at Service, fpe = 1339.20 MPa

Total Prestressing Forces at Service, $Pe = fpi * Aps$ Pe = 6344.59 KN

Stress Limits for Concrete,

Compressive Stress Limits under Unfactored Permanent Loads and Prestressing Force,

Girder, = 20.25 MPa

Deck Slab, = 12.60 MPa

Compressive Stress Limits under Unfactored Permanent Loads, Transient Loads and Prestressing Force,

Girder, = 27.00 MPa

Deck Slab, = 16.80 MPa

Tension Limits, = 3.35 MPa

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 $\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$

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

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 2.78E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 6.87E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength I Limit State)

Load Factor	1.25	1.50	1.75	1.75	Total
Location	DC	DW	Lane Load	Truck/Tandem	
	MPa	MPa	MPa	MPa	MPa
14.400	0.00	0.00	0.00	0.00	**0.00**
17.500	5.02	0.63	1.02	0.34	**7.01**
20.600	8.96	1.13	1.81	0.67	**12.57**
23.700	11.81	1.48	2.38	1.01	**16.68**
26.800	13.54	1.70	2.73	1.25	**19.22**
29.900	14.16	1.78	2.86	1.33	**20.12**
33.000	13.54	1.70	2.73	1.13	**19.10**
36.100	11.81	1.48	2.38	0.88	**16.56**
39.200	8.96	1.13	1.81	0.59	**12.48**
42.300	5.02	0.63	1.02	0.29	**6.96**
45.400	0.00	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 2.84E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 4.08E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength I Limit State)

Load Factor	1.25	1.50	1.75	1.75	Total
Location	DC	DW	Lane Load	Truck/Tandem	
	MPa	MPa	MPa	MPa	MPa
14.400	0.00	0.00	0.00	0.00	**0.00**
17.500	4.91	1.07	1.71	0.57	**8.25**
20.600	8.76	1.90	3.05	1.13	**14.84**
23.700	11.55	2.50	4.01	1.70	**19.75**
26.800	13.23	2.87	4.60	2.10	**22.80**
29.900	13.84	3.00	4.81	2.23	**23.88**
33.000	13.23	2.87	4.60	1.91	**22.61**
36.100	11.55	2.50	4.01	1.48	**19.54**
39.200	8.76	1.90	3.05	0.99	**14.69**
42.300	4.91	1.07	1.71	0.49	**8.18**
45.400	0.00	0.00	0.00	0.00	**0.00**

Compressive Stress Limits (STRENGTH II Limit State),

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 2.78E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 6.87E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength II Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.35 Lane Load MPa	1.35 Permit MPa	Total MPa
14.400	0.00	0.00	0.00	0.00	**0.00**
17.500	5.02	0.63	0.78	0.26	**6.70**
20.600	8.96	1.13	1.40	0.52	**12.00**
23.700	11.81	1.48	1.84	0.78	**15.91**
26.800	13.54	1.70	2.11	0.96	**18.31**
29.900	14.16	1.78	2.20	1.02	**19.16**
33.000	13.54	1.70	2.11	0.87	**18.22**
36.100	11.81	1.48	1.84	0.68	**15.81**
39.200	8.96	1.13	1.40	0.45	**11.93**
42.300	5.02	0.63	0.78	0.23	**6.66**
45.400	0.00	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 2.84E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 4.08E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength II Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.35 Lane Load MPa	1.35 Permit MPa	Total MPa
14.400	0.00	0.00	0.00	0.00	**0.00**
17.500	4.91	1.07	1.32	0.44	**7.73**
20.600	8.76	1.90	2.35	0.87	**13.88**
23.700	11.55	2.50	3.09	1.31	**18.45**
26.800	13.23	2.87	3.55	1.62	**21.27**
29.900	13.84	3.00	3.71	1.72	**22.27**
33.000	13.23	2.87	3.55	1.47	**21.12**
36.100	11.55	2.50	3.09	1.14	**18.28**
39.200	8.76	1.90	2.35	0.76	**13.77**
42.300	4.91	1.07	1.32	0.38	**7.68**
45.400	0.00	0.00	0.00	0.00	**0.00**

Compressive Stress Limits (STRENGTH III Limit State),

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 2.78E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 6.87E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength III Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.40 Wind Load MPa	Total MPa
14.400	0.00	0.00	0.00	**0.00**
17.500	5.02	0.63	-0.14	**5.52**
20.600	8.96	1.13	-0.25	**9.84**
23.700	11.81	1.48	-0.33	**12.97**
26.800	13.54	1.70	-0.37	**14.87**
29.900	14.16	1.78	-0.39	**15.55**
33.000	13.54	1.70	-0.37	**14.87**
36.100	11.81	1.48	-0.33	**12.97**
39.200	8.96	1.13	-0.25	**9.84**
42.300	5.02	0.63	-0.14	**5.52**
45.400	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 2.84E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 4.08E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength III Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.40 Wind Load MPa	Total MPa
14.400	0.00	0.00	0.00	**0.00**
17.500	4.91	1.07	-0.23	**5.74**
20.600	8.76	1.90	-0.42	**10.24**
23.700	11.55	2.50	-0.55	**13.50**
26.800	13.23	2.87	-0.63	**15.47**
29.900	13.84	3.00	-0.66	**16.18**
33.000	13.23	2.87	-0.63	**15.47**
36.100	11.55	2.50	-0.55	**13.50**
39.200	8.76	1.90	-0.42	**10.24**
42.300	4.91	1.07	-0.23	**5.74**
45.400	0.00	0.00	0.00	**0.00**

Compressive Stress Limits (STRENGTH V Limit State),

Stress at Top Fiber of Girder,

Section Modulus from Top of Transformed Service, Sxt = 2.78E+08 mm^3

Section Modulus from Top of Transformed Service Composite, Sxt = 6.87E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength V Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.35 Lane Load MPa	1.35 Truck/Tandem MPa	0.40 Wind Load MPa	Total MPa
14.400	0.00	0.00	0.00	0.00	0.00	**0.00**
17.500	5.02	0.63	0.78	0.26	-0.04	**6.66**
20.600	8.96	1.13	1.40	0.52	-0.07	**11.93**
23.700	11.81	1.48	1.84	0.78	-0.09	**15.82**
26.800	13.54	1.70	2.11	0.96	-0.11	**18.20**
29.900	14.16	1.78	2.20	1.02	-0.11	**19.05**
33.000	13.54	1.70	2.11	0.87	-0.11	**18.11**
36.100	11.81	1.48	1.84	0.68	-0.09	**15.72**
39.200	8.96	1.13	1.40	0.45	-0.07	**11.86**
42.300	5.02	0.63	0.78	0.23	-0.04	**6.62**
45.400	0.00	0.00	0.00	0.00	0.00	**0.00**

Stress at Bottom Fiber of Girder,

Section Modulus from Bottom of Transformed Service, Sxb = 2.84E+08 mm^3

Section Modulus from Bottom of Transformed Service Composite, Sxb = 4.08E+08 mm^3

COMPRESSIVE STRESS per GIRDER (Strength V Limit State)

Location	Load Factor 1.25 DC MPa	1.50 DW MPa	1.35 Lane Load MPa	1.35 Truck/Tandem MPa	0.40 Wind Load MPa	Total MPa
14.400	0.00	0.00	0.00	0.00	0.00	**0.00**
17.500	4.91	1.07	1.32	0.44	-0.07	**7.66**
20.600	8.76	1.90	2.35	0.87	-0.12	**13.76**
23.700	11.55	2.50	3.09	1.31	-0.16	**18.29**
26.800	13.23	2.87	3.55	1.62	-0.18	**21.09**
29.900	13.84	3.00	3.71	1.72	-0.19	**22.08**
33.000	13.23	2.87	3.55	1.47	-0.18	**20.94**
36.100	11.55	2.50	3.09	1.14	-0.16	**18.12**
39.200	8.76	1.90	2.35	0.76	-0.12	**13.65**
42.300	4.91	1.07	1.32	0.38	-0.07	**7.61**
45.400	0.00	0.00	0.00	0.00	0.00	**0.00**

Stress at Midspan,

STRENGTH I		STRENGTH II		STRENGTH III		STRENGTH V	
Top Fiber MPa	Bottom Fiber MPa	Top Fiber MPa	Bottom Fiber MPa	Top Fiber MPa	Bottom Fiber MPa	Top Fiber MPa	Bottom Fiber MPa
20.12	14.84	19.16	13.88	15.55	16.18	19.05	13.76

Stress at the Girder,

Stress at the Top Fiber,

$$ft = \frac{Pi}{Atransformed} - \frac{Pi * e}{St.transformed} + ft.service$$

Area of Girder, Atransformed = 676328.50 mm^2

Eccentricity of Prestressing Strand, e = 945.20 mm

Section Modulus from Bottom of Transformed Service, St.transformed = 2.78E+08 mm^3

Stress at the Top Fiber for Strength I, ft1 = 7.89 MPa

ft1 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength II, ft2 = 6.94 MPa

ft2 < 27.00 MPa **OK!**

Stress at the Top Fiber for Strength III, ft3 = 3.32 MPa

ft3 < 27.00 MPa **OK!**

Stress at the Top Fiber for Strength V, ft5 = 6.83 MPa

ft5 < 27.00 MPa **OK!**

Stress at the Bottom Fiber,

$$fb = \frac{Pi}{Atransformed} + \frac{Pi * e}{St.transformed} - fb.service$$

Section Modulus from Bottom of Transformed Service, Sb.transformed = 2.84E+08 mm^3

Stress at the Top Fiber for Strength I, fb1 = 15.66 MPa

fb1 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength II, fb2 = 16.62 MPa

fb2 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength III, fb3 = 14.32 MPa

fb3 < 20.25 MPa **OK!**

Stress at the Top Fiber for Strength V, fb5 = 16.74 MPa

fb5 < 20.25 MPa **OK!**

DGCS 12.2.2 Fatigue Limit,

Stress at Bottom Fiber due to Permanent Loads and Prestress,

Bottom Fiber, = 23.88 MPa

Fatigue Load,

Bending Moment due to Fatigue Load, = 365.30 KN-m

Factor for Fatigue Load, = 1.50

Stress due to Fatigue Load at Bottom Fiber, = -1.34 MPa

Half the Sum of Stress due to Effective Prestress and Permanent Load at the Bottom Fiber, = 11.94 MPa

Stress due to Prestress, Permanent Load and Fatigue I Load, ff = 10.60 MPa

ff < 20.25 MPa **OK!**

STRENGTH LIMIT STATE (FLEXURE),

Ultimate Bending Moment :

MOMENT per GIRDER (Strength I Limit State)

Load Factor	1.38	1.65	1.93	1.93	1.93	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	
	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
14.400	0.000	0.000	0.000	0.000	0.000	**0.00**
17.500	-1536.762	-479.601	-769.670	-254.334	-83.930	**-3124.30**
20.600	-2741.731	-854.290	-1370.975	-508.668	-167.860	**-5643.52**
23.700	-3614.905	-1124.066	-1803.914	-763.002	-251.791	**-7557.68**
26.800	-4142.958	-1288.929	-2068.489	-945.139	-311.896	**-8757.41**
29.900	-4332.554	-1348.880	-2164.697	-1004.360	-331.439	**-9181.93**
33.000	-4142.958	-1288.929	-2068.489	-859.613	-283.672	**-8643.66**
36.100	-3614.905	-1124.066	-1803.914	-665.721	-219.688	**-7428.29**
39.200	-2741.731	-854.290	-1370.975	-443.814	-146.459	**-5557.27**
42.300	-1536.762	-479.602	-769.670	-221.907	-73.229	**-3081.17**
45.400	0.000	0.000	0.000	0.000	0.000	**0.00**

MOMENT per GIRDER (Strength II Limit State)

Load Factor	1.38	1.65	1.49	1.49	1.49	Total
Location	DC	DW	Lane Load	Permit	IM	
	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
14.400	0.000	0.000	0.000	0.000	0.000	**0.00**
17.500	-1536.762	-479.601	-593.746	-196.200	-99.404	**-2905.71**
20.600	-2741.731	-854.290	-1057.610	-392.401	-198.808	**-5244.84**
23.700	-3614.905	-1124.066	-1391.590	-588.601	-289.892	**-7009.06**
26.800	-4142.958	-1288.929	-1595.691	-729.107	-354.901	**-8111.59**
29.900	-4332.554	-1348.880	-1669.909	-774.792	-393.521	**-8519.66**
33.000	-4142.958	-1288.929	-1595.691	-663.130	-367.282	**-8057.99**
36.100	-3614.905	-1124.066	-1391.590	-513.556	-308.012	**-6952.13**
39.200	-2741.731	-854.290	-1057.610	-342.371	-207.058	**-5203.06**
42.300	-1536.762	-479.602	-593.746	-171.185	-103.529	**-2884.82**
45.400	0.000	0.000	0.000	0.000	0.000	**0.00**

MOMENT per GIRDER (Strength III Limit State)

Load Factor	1.38	1.65	1.54	Total
Location	DC	DW	Wind Load	
	KN-m	KN-m	KN-m	KN-m
14.400	0.000	0.000	0.000	**0.00**
17.500	-1536.762	-479.601	105.479	**-1910.88**
20.600	-2741.731	-854.290	187.885	**-3408.14**
23.700	-3614.905	-1124.066	247.217	**-4491.75**
26.800	-4142.958	-1288.929	283.476	**-5148.41**
29.900	-4332.554	-1348.880	296.661	**-5384.77**
33.000	-4142.958	-1288.929	283.476	**-5148.41**
36.100	-3614.905	-1124.066	247.217	**-4491.75**
39.200	-2741.731	-854.290	187.885	**-3408.14**
42.300	-1536.762	-479.602	105.480	**-1910.88**
45.400	0.000	0.000	0.000	**0.00**

MOMENT per GIRDER (Strength V Limit State)

Load Factor	1.38	1.65	1.49	1.49	1.49	0.44	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	Wind Load	
	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m	KN-m
14.400	0.000	0.000	0.000	0.000	0.000	0.000	**0.00**
17.500	-1536.762	-479.601	-593.746	-196.200	-64.746	30.137	**-2840.92**
20.600	-2741.731	-854.290	-1057.610	-392.401	-129.492	53.681	**-5121.84**
23.700	-3614.905	-1124.066	-1391.590	-588.601	-194.238	70.634	**-6842.77**
26.800	-4142.958	-1288.929	-1595.691	-729.107	-240.605	80.993	**-7916.30**
29.900	-4332.554	-1348.880	-1669.909	-774.792	-255.681	84.760	**-8297.06**
33.000	-4142.958	-1288.929	-1595.691	-663.130	-218.833	80.993	**-7828.55**
36.100	-3614.905	-1124.066	-1391.590	-513.556	-169.474	70.634	**-6742.96**
39.200	-2741.731	-854.290	-1057.610	-342.371	-112.982	53.681	**-5055.30**
42.300	-1536.762	-479.602	-593.746	-171.185	-56.491	30.137	**-2807.65**
45.400	0.000	0.000	0.000	0.000	0.000	0.000	**0.00**

Governing Load Case :

Design Moment, Mu = 9181.93 KN-m

DGCS 12.4.3.1.3
PCI BDM 2003

Flexural Strength Design (Strain Compatibility Approach),

Assumptions in this Approach

- Plane Sections Remain Plane After Bending.
- Compatibility of Strains, i.e. Full Bond between Steel and Concrete at the Section being Considered.
- Equilibrium of Forces within a Section.
- Concrete Stress at Ultimate Flexure is Represented by a Rectangular Stress Block with Intensity of 0.85fc and

DGCS 12.4.2.1

Depth a = β1 *c where c is the Neutral Axis Depth and β1 is Determined.

- Steel Stress Relationship Adopted for a 1863N/mm^2 (270kip/in^2) Low Relaxation Strand is :

PCI BDM 8.2.2.5-1

$$fst = \varepsilon st\left\{\frac{887 + 27613}{\left[1 + (112 \cdot 4 \cdot \varepsilon st)^{7.36}\right]^{\frac{1}{7.36}}}\right\} < 270ksi$$

- For Mild Reinforcements, an Elastic Plastic Stress-Strain Relationship is Assumed :

PCI BDM 8.2.2.5-2

$$fst = Es \cdot \varepsilon st < fy$$

- For Each Concrete Component, the Force Would Be :

PCI BDM 8.2.2.5-5

$$Fcj = 0.85 \cdot fcj \cdot Acj$$

Computation,

Assume Neutral Axis Depth c and Substitute in the Equation Below,

PCI BDM 8.2.2.5-3

$$\varepsilon si = 0.003\left(\frac{di}{c}\right) - 1 + \left(\frac{fse}{Es}\right)_i$$

Assumed Neutral Axis Depth, c = 200.00 mm

Depth of Steel Layer from Extreme Compression Fiber, di

Effective Prestress, fse. For Partially Tensioned Tendons or for Non-Tension Reinforcing Bars, This May be Assumed As,

PCI BDM 8.2.2.5-5

$$fse = fpi \quad 172.4\,MPa$$

Where fpi is Initial Tension

Layer	db mm	n pc/s	As mm^2	di mm	fpi MPa
1	28	3	1,847.26	90.00	0.00
2	16	12	2,412.74	100.00	0.00
3	28	4	2,463.01	360.00	0.00
4	12	10	1,130.97	1,028.00	0.00
5	28	4	2,463.01	1,732.00	0.00

Steel Stresses and Forces,

Layer	di	fse	εsi	fsi	Asi*fsi
1	90.00	-172.40	-0.003	-420.00	-775.85
2	100.00	-172.40	-0.002	-280.00	-675.57
3	360.00	-172.40	0.002	307.60	757.62
4	1,028.00	-172.40	0.012	280.00	316.67
5	1,732.00	-172.40	0.022	420.00	1034.46
TENDON	1,590.20	804.56	0.025	1845.89	8745.11

Stresses in Concrete Components with Corresponding Forces,

Averaged Value for β due to Composite Construction,

PCI BDM 8.2.2.5-6

$$\beta 1.ave = \frac{\sum(f'c \cdot Ac \cdot \beta 1)_i}{\sum(f'c \cdot Ac)_i}$$

β1.ave = 0.85

Assumed Average β1, = 0.85

Assumed Compression Block Depth, a = 170.00 mm

Component	Compression Area, Ac	Compressive Strength, f'c	β1	f'c*Ac*β1	f'c*Ac
Deck	306000.00	28.00	0.85	7,282,800.00	8,568,000.00
Flange	0.00	45.00	0.73	0.00	0.00
Web	0.00	45.00	0.73	0.00	0.00
			TOTAL	**7,282,800.00**	**8,568,000.00**

Revised Compression Block Depth, a = 170.00 mm

Component	Compression Area, Ac	Compressive Strength, f'c	β1	f'c*Ac*β1	f'c*Ac	Fcj
Deck	306000.00	28.00	0.85	7,282,800.00	8,568,000.00	-7,282.80
Flange	0.00	45.00	0.73	0.00	0.00	0.00
Web	0.00	45.00	0.73	0.00	0.00	0.00
			TOTAL	**7,282,800.00**	**8,568,000.00**	**-7,282.80**

Check for Equilibrium,

PCI BDM 8.2.2.5-4

$$\sum Asi \cdot fsi + \sum Fcj = 0$$

Summation of Forces on Steel Components, = 657.34 KN

Summation of Forces on Concrete Components, = -7282.80 KN

Permissible Error Set by Designer, =

Summation of Forces, = -6625.46 KN

Computation for Flexural Strength,

PCI BDM 8.2.2.5-7

$$Mn = \sum Asi \cdot fsi \cdot di + \sum Fcj \cdot dj$$

Moment Arm from Top Fiber,

Component		Asi*fsi KN	di mm	Fcj KN	dj mm	Moment about Top Fiber KN-m
Deck		-	-	-7,282.80	100.00	-728.28
Flange		-	-	0.00	0.00	0.00
Web		-	-	0.00	0.00	0.00
For Steel	Layer 1	-775.85	90.00	-	-	-69.83
	Layer 2	-675.57	100.00	-	-	-67.56
	Layer 3	757.62	360.00	-	-	272.74
	Layer 4	316.67	1,028.00	-	-	325.54
	Layer 5	1,034.46	1,732.00	-	-	1,791.69
TENDON		8,745.11	1,590.20	-	-	13,906.47
					TOTAL	**15,430.78**

DGCS 12.4.3.3	**Limits of Reinforcement,**					
	1.33 Times the Factored Moment Required,	1.33*Ma	=	12,211.97	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	=	1,810.09	KN-m	
	Modulus of Rupture, $fr = 0.63\sqrt{fc'}$	fr	=	4.23	MPa	
	Compressive Stress in Concrete due to Effective Prestress Forces,	fcpe	=	0.01	MPa	
	Total Unfactored Dead Load Moment Acting on the Non-Composite Section,	Mdnc	=	3,143.80	KN-m	
	Section Modulus for the Extreme Fiber of the Composite Section,	Sc	=	2.67E+08	mm^3	
	Section Modulus for the Extreme Fiber of the Non-Composite Section,	Snc	=	3.75E+08	mm^3	
	Flexural Cracking Variability Factor,	γ_1	=	1.60		
	Prestress Variability Factor,	γ_2	=	1.10		
	Ratio of Minimum Yield Strength to Ultimate Tensile Strength of the Reinforcement,	γ_3	=	1.00		
	Checking for Flexural Capacity,	15,430.78	>	1810.09	KN-m	**OK!**

STRENGTH LIMIT STATE (SHEAR),

Ultimate Shear :

SHEAR per GIRDER (Strength I Limit State)

Load Factor	1.38	1.65	1.93	1.93	1.93	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	
	KN	KN	KN	KN	KN	KN
14.400	556.89	174.05	279.32	14.10	4.65	**1029.01**
17.500	446.80	139.24	223.45	14.10	4.65	**828.25**
20.600	336.71	104.43	167.59	14.10	4.65	**627.49**
23.700	222.76	69.62	111.73	14.10	4.65	**422.86**
26.800	110.09	34.81	55.86	14.10	4.65	**219.52**
29.900	0.00	0.00	0.00	14.10	4.65	**18.76**
33.000	-110.09	-34.81	-55.86	14.10	4.65	**-182.00**
36.100	-222.76	-69.62	-111.73	7.49	2.47	**-394.14**
39.200	-336.71	-104.43	-167.59	-2.44	-0.81	**-611.98**
42.300	-446.80	-139.24	-223.45	-57.27	-18.90	**-885.67**
45.400	-556.89	-174.05	-279.32	-139.52	-46.04	**-1195.82**

SHEAR per GIRDER (Strength II Limit State)

Load Factor	1.38	1.65	1.49	1.49	1.49	Total
Location	DC	DW	Lane Load	Permit	IM	
	KN	KN	KN	KN	KN	KN
14.400	556.89	174.05	215.47	33.18	10.95	**990.54**
17.500	446.80	139.24	172.38	33.18	10.95	**802.55**
20.600	336.71	104.43	129.28	33.18	10.95	**614.56**
23.700	222.76	69.62	86.19	33.18	10.95	**422.70**
26.800	110.09	34.81	43.09	33.18	10.95	**232.12**
29.900	0.00	0.00	0.00	33.18	10.95	**44.13**
33.000	-110.09	-34.81	-43.09	24.72	8.16	**-155.11**
36.100	-222.76	-69.62	-86.19	-9.12	-3.01	**-390.69**
39.200	-336.71	-104.43	-129.28	-55.94	-18.46	**-644.82**
42.300	-446.80	-139.24	-172.38	-87.15	-28.76	**-874.33**
45.400	-556.89	-174.05	-215.47	-165.19	-54.51	**-1166.11**

SHEAR per GIRDER (Strength III Limit State)

Load Factor	1.38	1.65	1.54	Total
Location	DC	DW	Wind Load	
	KN	KN	KN	KN
14.400	556.89	174.05	-38.28	**692.66**
17.500	446.80	139.24	-30.62	**555.42**
20.600	336.71	104.43	-22.97	**418.18**
23.700	222.76	69.62	-15.31	**277.06**
26.800	110.09	34.81	-7.66	**137.24**
29.900	0.00	0.00	0.00	**0.00**
33.000	-110.09	-34.81	7.66	**-137.24**
36.100	-222.76	-69.62	15.31	**-277.06**
39.200	-336.71	-104.43	22.97	**-418.18**
42.300	-446.80	-139.24	30.62	**-555.42**
45.400	-556.89	-174.05	38.28	**-692.66**

SHEAR per GIRDER (Strength V Limit State)

Load Factor	1.38	1.65	1.49	1.49	1.49	0.44	Total
Location	DC	DW	Lane Load	Truck/Tandem	IM	Wind Load	
	KN	KN	KN	KN	KN	KN	KN
14.400	556.89	174.05	215.47	10.88	3.59	-10.94	**949.94**
17.500	446.80	139.24	172.38	10.88	3.59	-8.75	**764.14**
20.600	336.71	104.43	129.28	10.88	3.59	-6.56	**578.34**
23.700	222.76	69.62	86.19	10.88	3.59	-4.37	**388.66**
26.800	110.09	34.81	43.09	10.88	3.59	-2.19	**200.28**
29.900	0.00	0.00	0.00	10.88	3.59	0.00	**14.47**
33.000	-110.09	-34.81	-43.09	10.88	3.59	2.19	**-171.33**
36.100	-222.76	-69.62	-86.19	5.78	1.91	4.37	**-366.51**
39.200	-336.71	-104.43	-129.28	-1.88	-0.62	6.56	**-566.37**
42.300	-446.80	-139.24	-172.38	-44.18	-14.58	8.75	**-808.43**
45.400	-556.89	-174.05	-215.47	-107.63	-35.52	10.94	**-1078.62**

Governing Load Case :

Design Shear, Vu = 1029.01 KN

Shear Strength Design,

Assumptions in this Approach :

• Plane Sections Remains Plane after Bending.

DGCS 12.5.2.3-1 Computation, $Vu > 0.50 * \phi v (Vc + Vp)$

DGCS 12.5.2.8-1 Shear Stress on Concrete, vu $vu = \frac{(Vu - \phi v * Vp)}{(\phi v * bv * dv)}$ vu = -0.28 MPa

Critical Section for Shear,

Effective Shear Depth, $dv = de - \frac{a}{2}$ dv = 1152.14 mm

where :

Depth of Compression Block at Midspan, a = 170.00 mm

Effective Shear Depth of the Beam at the End, dv = 715.20 mm

Need Not be Taken Less Than the Greater of,

0.90*de = 720.18 mm

0.72*h = 1152.14 mm

Effective Depth at the End of Beam,

DGCS 12.5.2.8-2 $de = \frac{(Aps * fps * dp)}{(Aps * fps)} = dp$ de.end = 800.20 mm

Tendon Centroid at Beam Support,

Tendon	Centroid	No. of Strands	N*Yb
1	1,100.00	12.00	13,200.00
2	900.00	12.00	10,800.00
3	700.00	12.00	8,400.00
4	500.00	12.00	6,000.00
5	0.00	0.00	0.00
6	0.00	0.00	0.00
7	0.00	0.00	0.00
8	0.00	0.00	0.00
9	0.00	0.00	0.00
10	0.00	0.00	0.00
TOTAL		**48.00**	**38,400.00**

Centroid of Prestressing Steel at End of Beam, Ybps = 800.00 mm

Distance Measured from Extreme Compression Fiber to the Centroid of Prestressing Steel at End of Beam, dp = 800.20 mm

Contribution of Prestressing Force to Nominal Shear Resistance, $Vp = Pj * \alpha$ Vp = 1481.52 KN

Prestressing Force before Losses, Pj = 24692.00 KN

Angle Change at Point of Critical Shear, α = 0.06 radians

Contribution of Concrete to Nominal Shear Resistance,

DGCS 12.5.3.2-4 Nominal Shear Resistance of Concrete, $Vc = 0.083\beta\sqrt{f'c} * bv * dv$ Vc = 4692.64 KN

Strain in Flexural Tension Reinforcement, εs = -0.00319

DGCS 12.5.3.3-4 $\varepsilon s = \frac{\left(\frac{|Mu|}{dv} + 0.5Nu + |Vu - Vp| - ApsfpO\right)}{(EsAs + EpAps)}$

Applied Factored Bending Moment at the Specified Sections, Mu = 521.35 KN-m

Applied Factored Shear at the Specified Section, Vu = 1029.01 KN

Applied Factored Normal Force at the Specified Section, Nu = 0.00 KN

Parameter Taken as Modulus of Elasticity of Prestressing Tendons Multiplied by the Locked in Difference in Strain between the Prestressing Tendons and Surrounding Concrete, fpo = 1302.00 MPa

Area of Non-Prestressed Steel on the Flexural Tension Side of the Member at the Section Under Consideration, As = 3593.98 mm^2

Calculation for Negative Strain,

Applicable if Initial Strain Calculation is Negative, Should Not be Less than -0.004, εs = -0.00041

$\varepsilon s = \frac{\left(\frac{|Mu|}{dv} + 0.5Nu + |Vu - Vp| - ApsfpO\right)}{(EsAs) + (EpAps + EcAct)}$

Area of Concrete on the Flexural Tension Side of the Member, where the Flexural side of the Member shall be taken as the Half-Depth containing the Flexural Tension Zone, Act = 326773.54 mm^2

Strain Used in Calculation for Contribution of Concrete to Shear, εs = -0.0004

DGCS 12.5.3.3.2 For Sections containing at least the Minimum Amount of Transverse Reinforcement, β = 6.86

DGCS 12.5.3.3-1 $\beta = \frac{4.8}{(1 + 750\varepsilon s)}$

DGCS 12.5.3.3-3 Angle of Inclination, $\theta = 29 + 3500\varepsilon s$ θ = 27.60 degrees

Contribution of Reinforcement to Nominal Shear Resistance,

Transverse Reinforcement will be Provided where,

DGCS 12.5.2.3-1 $Vu > 0.50 * \phi v (Vc + Vp)$

Location	Factored Shear	0.50*φv*(Vc + Vp)	Remarks
14.400	1,029.01	2,778.37	**OK!**
17.500	828.25	2,778.37	**OK!**
20.600	627.49	2,778.37	**OK!**
23.700	422.86	2,778.37	**OK!**
26.800	232.12	2,778.37	**OK!**
29.900	44.13	2,778.37	**OK!**

Requireed Strength of Shear Reinforcement, As

Required Shear Strength for Steel, Vs

DGCS 12.5.3.2-4 $Vn \le \phi v \cdot Vn = \phi v(Vc + Vs + Vp)$

$Vu = \frac{Vu}{\phi v} \quad (Vc + Vp)$

Required Spacing of Transverse Shear Reinforcement, s

DGCS 12.5.3.2-4 $Vs = \frac{Avfysin\alpha(cot\theta + cot\alpha)sin\alpha}{s}$

Angle of Inclination of Transverse Reinforcement to Longitudinal Axis,	α	=	90.00	degrees
Transverse Shear Reinforcement Specifications :				
Diameter of Transverse Bar,	db.transverse	=	12.00	mm
Yield Strength of Bar,	fy	=	280.00	MPa
Number of Legs,	Nlegs	=	2.00	
Area of Transverse Shear Reinforcement,	Av	=	226.19	mm^2

$Vu > 0.50 * \phi v(Vc + Vp)$

Location	Factored Shear	Required Transverse Shear Strength of Steel	Required Spacing	Av/s
14.400	1,029.01	-5,030.81	300.00	**MINIMUM**
17.500	828.25	-5,253.88	300.00	**MINIMUM**
20.600	627.49	-5,476.95	300.00	**MINIMUM**
23.700	422.86	-5,704.32	300.00	**MINIMUM**
26.800	232.12	-5,916.24	300.00	**MINIMUM**
29.900	44.13	-6,125.12	300.00	**MINIMUM**

Minimum Transverse Reinforcements,

Except for Segmental Post-Tensioned Concrete Box Girder Bridges,

DGCS 12.5.2.4-1 $Av \ge 0.083\sqrt{f'c}\frac{bvs}{fy}$

Required Area of Transverse Shear per Spacing,	=	0.40	mm^2/mm
Spacing for Minimum Transverse Reinforcement,	=	559.80	mm

DGCS 12.5.2.6	Maximum Spacing of Transverse Reinforcement,	0.125f'c	=	5.63	MPa
		v_u	>	0.125f'c	
DGCS 12.5.2.6-1	$smax = 0.8dv \le 600mm$	smax	=	600.00	mm
DGCS 12.5.2.6-2	$smax = 0.4dv \le 300mm$	smax	=	300.00	mm
	Governing Maximum Spacing of Transverse Reinforcement,	smax	=	300.00	mm
	Maximum Nominal Shear Resistance, $Vn = 0.25f'cbvdv + Vp$	Vn	=	4115.32	KN

Design for Shear,

Location	Required Spacing	Spacing	Computed Shear Resistance	Exceeds Max. Shear Resistance	Design Shear Resistance	Factored Shear	C/D Ratio	Remarks
14.400	300.00	150.00	7,104.69	YES	3,703.79	1,029.01	3.60	**OK!**
17.500	300.00	150.00	7,104.69	YES	3,703.79	828.25	4.47	**OK!**
20.600	300.00	250.00	6,732.48	YES	3,703.79	627.49	5.90	**OK!**
23.700	300.00	250.00	6,732.48	YES	3,703.79	422.86	8.76	**OK!**
26.800	300.00	300.00	6,639.43	YES	3,703.79	232.12	15.96	**OK!**
29.900	300.00	300.00	6,639.43	YES	3,703.79	44.13	83.92	**OK!**

DEFLECTION AND CAMBER,

Deflections due to Dead Loads,

Deflection due to Prestressing Force at Transfer,

PCI BDM 8.7-1 $\Delta p = \frac{5wL^4}{384EI}$ Δp = 220.81 mm

Equivalent Moment or Load,

$w = \frac{8Pe'}{L^2}$ w = 118.99 KN/m

Eccentricity,	e	=	0.60 mm
Prestressing Force at Transfer,	P	=	23774.77 KN

Deflection due to Beam Self Weight,

PCI BDM 8.7-1 $\Delta p = \frac{5wL^4}{384EI}$ Δp = 29.11 mm

PCI Multiplier,

Deflection due to Diaphragm Weight, ΔD = 0.18 mm

$\Delta D = \frac{PL^3}{28EI}$ (For Two Intermediate Diaphragm)

$\Delta D = \frac{PL^3}{48EI}$ (For One Intermediate Diaphragm)

Weight of Diaphragm 1,	P1	=	7.02 KN
Weight of Diaphragm 2,	P2	=	7.02 KN
Distance of Intermediate Diaphragms from End Face of Girder,	a	=	10.33 m

Deflection due to Slab and Haunch Weights,

PCI BDM 8.7-1 $\Delta p = \frac{5wL^4}{384EI}$ Δp = 20.78 mm

Deflection due to Rail/Barrier and Future Wearing Surface,

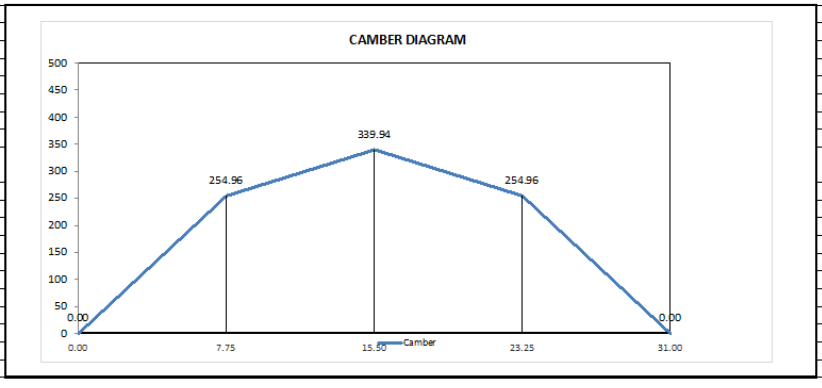
PCI BDM 8.7-1 $\Delta p = \frac{5wL^4}{384EI}$ Δp = 9.63 mm

	Deflection due to Live Load,				
DGCS 10.7.4	Deflection Limits,				
DGCS 10.1.2.6	Vehicular Load General	ΔLL1	=	38.75	mm
DGCS 10.1.2.6	Vehicular and Pedestrian Loads	ΔLL2	=	31.00	mm
DGCS 10.1.2.6	Vehicular Load on Cantilever Arms	ΔLL3	=	103.33	mm
DGCS 10.1.2.6	Vehicular and Pedestrian Loads on Cantilever Arms	ΔLL4	=	82.67	mm
	Live Load Deflections,				
	Deflection due to Lane Load,				
	$\Delta_{LL} = \frac{5wL^4}{384EI}$	ΔLL	=	8.30	mm
	Deflection due to Truck Load,	e	=	0.73	m
	Location of Resultant to Mid-Span	N	=	0.50	
	Vehicle Load Multiplier for Wheel Load, User Guide				
	Point Load per Axle, P				
	Span Length, L				
	Location where Deflection is Needed, x				
	Distance from End of Beam to Location of Point Load, a				

P	a	b	x	Δmidspan
72.50	11.928	19.07	15.50	1.36
72.50	16.228	14.77	15.50	3.05
17.50	20.528	10.47	15.50	0.63
	TOTAL			5.04

	Total Deflection with IM and DFM,	ΔTL	=	5.62	mm
	Deflection at Time of Erection,				
	Initial Prestress,				
	Time Dependent Multiplier for Deflection,	$TDM_{initial\ prestress}$	=	1.93	
PCI BDM 15.5-1	$TDM_{initial\ prestress} = 1 + \varphi_{b(td,ti)}$				
	Deflection at Midspan,		=	426.79	mm
	Self Weight,				
	Time Dependent Multiplier for Deflection,	$TDM_{selfweight}$	=	1.93	
PCI BDM 15.5-1	$TDM_{selfweight} = 1 + \varphi_{b(td,ti)}$				
	Deflection at Midspan,		=	56.26	mm
	Dead Load on Plain Beam,				
	Time Dependent Multiplier for Deflection,	$TDM_{DLofplainb.}$	=	1.00	
	$TDM_{DL\ of\ plain\ beam} = 1.00$				
	Deflection at Midspan,		=	20.96	mm
	Dead Load on Composite Beam,				
	Time Dependent Multiplier for Deflection,	$TDM_{DLofcomp.b.}$	=	1.00	
	$TDM_{DL\ of\ composite\ beam} = 1.00$				
	Deflection at Midspan,		=	9.63	mm
	Total Deflection at Erection before Deck Placement,		=	370.53	mm
	Total Deflection after Deck Placement,		=	349.57	mm
	Total Deflection due to Dead Loads on Composite Section,		=	339.94	mm
	Deflection Check,				
	Type of Live Load Traffic,			Vehicular and Pedestrian Loads	
	Deflection Limit,		=	31.00	mm
	Maximum Live Load Deflector,		=	8.30	mm
		31.00	>	8.30	**OK!**
	Camber Diagram,				
	$y = \frac{4\Delta_{mid}}{L^2}\left(x - \frac{L}{2}\right)^2 + \Delta_{mid}$				
	Deflection at Midspan due to Permanent Loads,	Δmid	=	339.94	mm

Point	Distance	Deflection
0.00	0.00	0.00
0.25	7.75	254.96
0.50	15.50	339.94
0.75	23.25	254.96
1.00	31.00	0.00



DGCS 12.5.5 **SHEAR INTERFACE DESIGN,**

Shear Interface Reinforcement Specifications,

Shear Interface Reinforcement Bar,

Diameter of Bar,	db.interface	=	12.00	mm
Area of Bar,	As.interface	=	113.10	mm^2
No. of Legs Crossing Interface,	Nlegs	=	2.00	pc/s
Total Area of Shear Interface Reinforcement,	Avf	=	226.19	mm^2

Computation of the Factored Interface Shear Force for Girder/Slab Bridges,

DGCS 12.5.4.1-1 $$vui = \frac{Vui}{bvdv}$$

Distance between the Centroid of the Tension Steel and the Mid	dv	=	1152.14	mm
Thickness of the Slab to Compute a Factored Interface Shear Stress,				
Width of Cross Section at the Contact Surface being	bv	=	1066.80	mm

Investigated for Horizontal Shear,

Factored Shear Force at Specified Section

(Critical Section for Shear in this Case, Vui)

Required Nominal Shear Interface Shear Resistance, Vni

PCI BDM 8.5.3-1 $$Vni = \frac{vui \cdot Acv\ per\ m}{\phi v}$$

Point	Factored Shear	Factored Interface Shear Force	Required Nominal Interface Resistance	Wave Reinforcement Provision?
	KN	MPa	KN/m	
0.00	1,029.01	0.837	992.365	APPLICABLE
0.05	828.25	0.674	798.755	APPLICABLE
0.15	627.49	0.511	605.143	APPLICABLE
0.30	422.86	0.344	407.801	APPLICABLE

NOTE: Minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces roughened to an amplitude of 6mm where the Factored Interface shear stress is less than and all vertical shear reinforcement required is extended across the interface and adequately anchored in the slab. = 1.40 MPa

DGCS 12.5.4-3 Nominal Shear Resistance,

$$Vni = cAv + \mu(Avfy + Pc)$$

Area of Concrete to be Engaged in Shear Transfer,	Acv	=	1066800.00	mm^2
Area of Shear Reinforcement Crossing the Shear Plane,	Avf	=	1333.50	mm^2
Yield Strength of Reinforcement,	fy	=	280.00	MPa
Permanent Net Compressive Force Normal to Shear Plane,	Pc	=	0.00	N

Cohesion and Friction Factors,

Condition 1

Cast-in-Place Concrete Slab on Clean Concrete Girder with Surface Roughened to an Amplitude of 6mm. (Normal Weight Concrete)

a. c	=	1.9	(N/mm2)
b. μ	=	1	
c. K1	=	0.3	
d. K2	=	12.4	

Condition 2

Cast-in-Place Concrete Slab on Clean Concrete Girder with Surface Roughened to an Amplitude of 6mm. (Light Weight Concrete)

a. c	=	1.9	(N/mm2)
b. μ	=	1	
c. K1	=	0.3	
d. K2	=	9	

Condition 3

Normal Weight Concrete Placed Monolithically

a. c	=	2.8	(N/mm2)
b. μ	=	1.4	
c. K1	=	0.25	
d. K2	=	10.3	

Condition 4

For Light Weight Concrete Placed Monolithically or Non Monolithically against a Clean Concrete Surface, Free of Laitance with Surface Roughened to an Amplitude of 6mm.

a. c	=	1.7	(N/mm2)
b. μ	=	1	
c. K1	=	0.25	
d. K2	=	6.9	

Condition 5

For Normal Weight Concrete Placed against a Clean Concrete Surface Free of Laitance with Roughened to an Amplitude of 6mm.

a. c	=	1.7	(N/mm2)
b. μ	=	1	
c. K1	=	0.25	
d. K2	=	10.3	

Condition 6

For Concrete Placed against a Clean Concrete Surface, Free of Laitance but not Intentionally Roughened.

a. c	=	0.52	(N/mm2)
b. μ	=	0.6	
c. K1	=	0.2	
d. K2	=	5.5	

Condition Applicable to Design of Slab, Condition 1

c	=	1.90
μ	=	1.00
K1	=	0.30
K2	=	12.40

Required Area of Shear Reinforcement,

$$Avf = \frac{(Vui - c\lambda cv)}{\mu fy} - \frac{Pc}{fy}$$

Point	Required Nominal Interface Resistance	Shear Resistance Provided by Concrete	Required Area of Reinforcement
	KN/m	KN/m	mm^2/m
0.00	992.37	2026.92	Minimum
0.05	798.75	2026.92	Minimum
0.15	605.14	2026.92	Minimum
0.30	407.80	2026.92	Minimum

MINIMUM AREA OF INTERFACE SHEAR REINFORCEMENT,

DGCS 12.5.4-3-1

$$Avf \geq \frac{0.35Acv}{fy}$$

For Cast in Place Concrete Slabs on Clean Concrete Girder Surface Free of Liatance, the following Provision shall Apply,

Minimum Interface Shear Reinforcement, Avf

Amount Required to Resist 1.33vui/Φv

Point	1.33vui/Φv	Required Area of Reinforcement	Minimum Interface Reinforcement	Maximum Spacing
	KN/m	mm^2/m	mm^2/m	mm
0.00	1319.85	4713.74	1333.50	169.62
0.05	1062.34	3794.08	1333.50	169.62
0.15	804.84	2874.43	1333.50	169.62
0.30	542.37	1937.05	1333.50	169.62

2) Design of the Bottom Slab

(a) 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	=	1,800	mm
	Girder Width,		bw	=	1,066.80	mm
DGCS 14.4.2.2	Effective Length,		Leff	=	733.20	mm
	Diameter of Main Bars for Impact Factor IM = 33%,		db 1	=	16	mm
	Diameter of Main Bars for Impact Factor IM = 75%,		db 2	=	20	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,	$Ec = 0.043 K_1 \rho c^{1.5} \sqrt{fc'}$	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,	$Q = \sum \eta_i \gamma_i Q_i$				
DGCS 10.3-1	Total Factored Force Effect,					
	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	$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$					
	For Strength Limit State,					
	Factor Relating to Ductility,		ηD	=	1.05	
	Factor Relating to Redundancy,		ηR	=	1.05	
	Factor Relating to Operational Importance,		ηI	=	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,		ηD	=	1.00	
	Factor Relating to Redundancy,		ηR	=	1.00	
	Factor Relating to Operational Importance,		ηI	=	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,	$Mds = \frac{w * Leff^2}{10}$	Mds	=	0.26	KN-m/m
	Weight of FWS,		wws	=	1.10	KN/m²
	Unfactored Deck Slab Positive or Negative Moment,	$Mws = \frac{wws * Leff^2}{10}$	Mws	=	0.06	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		Unit
		Positive	Negative	
1.38	DC	0.36	0.36	KN-m/m
1.65	DW	0.10	0.10	KN-m/m
1.93	LL	83.93	73.23	KN-m/m
TOTAL		**84.38**	**73.68**	**KN-m/m**

Service I Limit State,

Factor	Force	Moment		Unit
		Positive	Negative	
1.00	DC	0.26	0.26	KN-m/m
1.00	DW	0.06	0.06	KN-m/m
1.00	LL	43.50	37.95	KN-m/m
TOTAL		**43.82**	**38.27**	**KN-m/m**

Design Factored Moment for IM = 75%

Dynamic Overload Factor, = 1.50

Strength I Limit State,

Factor	Force	Moment		Unit
		Positive	Negative	
1.38	DC	0.36	0.36	KN-m/m
1.65	DW	0.10	0.10	KN-m/m
1.93	LL	152.60	133.14	KN-m/m
TOTAL		**153.05**	**133.60**	**KN-m/m**

Service I Limit State,

Factor	Force	Moment		Unit
		Positive	Negative	
1.00	DC	0.26	0.26	KN-m/m
1.00	DW	0.06	0.06	KN-m/m
1.00	LL	79.09	69.01	KN-m/m
TOTAL		**79.41**	**69.33**	**KN-m/m**

DGCS 12.4.3.2 **Flexural Resistance,**

DGCS 12.4.3.3 **Limits of Reinforcement,**

1.33 Times the Factored Moment Required, Mcr = 26.67 KN-m; 1.33*Ma = 105.62 KN-m

DGCS 12.4.3.3-1 Cracking Moment, $Mcr = \gamma_3\gamma_1 frSc$ 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	Spacing	Remarks
mm	mm	
16	120.00	ADEQUATE

Negative Moment : Top Bars		
Rebar	Spacing	Remarks
mm	mm	
16	130.00	ADEQUATE

For Impact Factor IM = 75%

Positive Moment : Bottom Bars		
Rebar	Spacing	Remarks
mm	mm	
20	70.00	ADEQUATE

Negative Moment : Top Bars		
Rebar	Spacing	Remarks
mm	mm	
16	70.00	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	1675.52	1546.63	4487.99	4487.99	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	84.38	73.68	153.05	133.60	KN-m/m
Depth of Equivalent Sress Block	29.57	27.29	79.20	79.20	mm
Factored Flexural Resistance	86.91	80.88	190.68	190.68	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!	

DGCS 14.4.3.1 **Distribution of Reinforcement,**

Percent Distribution, $\frac{3840}{\sqrt{Leff}} \leq 67\%$ — %distribution = 67.00 %

Area of Distribution Bars, $As.dr = \%distribution * As.tenston$ — As.dr = 1036.24 mm²

Spacing of Bottom Distribution Rebar, $s = \frac{Ab * 1000}{As}$ — s = 170.00 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	120.00	Adopt Spacing
Negative Moment	Top Bars	16	130.00	

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	20	70.00	Adopt Spacing
Negative Moment	Top Bars	16	70.00	Adopt Spacing

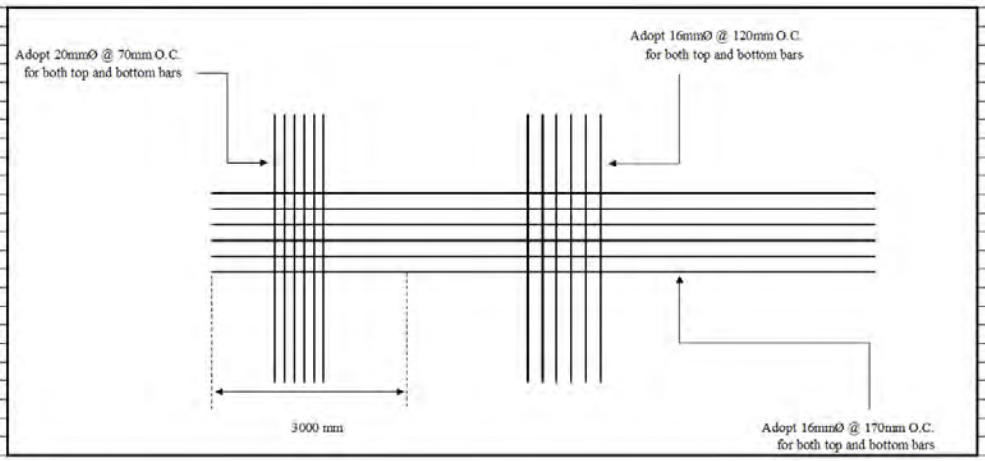
Distribution Reinforcement,

Adopt Distribution Reinforcement, — Adopt 16mmØ @ 170.00 mm

Temperature and Shrinkage Reinforcement,

Adopt Distribution Reinforcement, — Adopt 16mmØ @ 450.00 mm

Slab Reinforcement Layout,



(b) 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	=	500.00	mm
	Thickness of Sidewalk,	tsw	=	250.00	mm
	Area of Sidewalk,	Asw	=	125000.00	mm^2
	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/m^3
	Unit Weight,	γc	=	24.00	KN/m^3
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	=	3.00	KN/m
	Weight of Post,	Wpost	=	0.09	KN/m
	Weight of Railings,	Wrail	=	0.30	KN/m

Undactored Dead Load,

Member	Weight KN/m	Lever Arm m	Moment KN-m
Sidewalk	3.00	0.25	0.75
Post	0.09	0.48	0.04
Railings	0.30	0.48	0.14
TOTAL DC	**3.39**		**0.94**

DGCS 18	**Live Load Analysis,**				
	Live Load Intensity,	LL	=	15.00	KN
	Live Load Moment Arm,	LLarm	=	150.00	mm
	Dynamic Load Allowance (For All Other Limit State),	IM	=	1.33	
	Distance of Design Section to Face of Railing,	d	=	450.00	mm
	Design Unfactored Shear with Impact Factor,	V	=	19.95	KN
	Design Unfactored Bending Moment with Impact Factor, Negative Moment,	M	=	2.99	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	$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$				
	For Strength Limit State,				
	Factor Relating to Ductility,	ηD	=	1.05	
	Factor Relating to Redundancy,	ηR	=	1.05	
	Factor Relating to Operational Importance,	ηI	=	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,	ηD	=	1.00	
	Factor Relating to Redundancy,	ηR	=	1.00	
	Factor Relating to Operational Importance,	ηI	=	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	4.68	1.29
1.93	LL	28.94	5.77
TOTAL		**33.62**	7.07

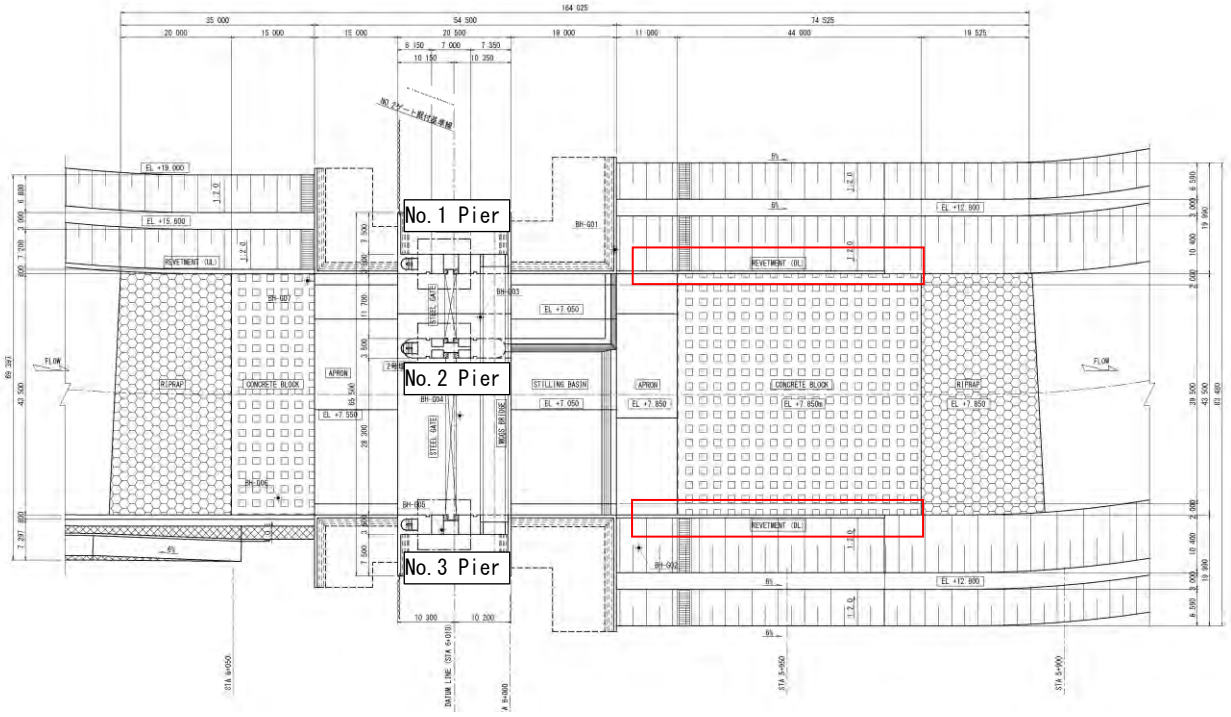
	Design of Main Reinforcement,				
	Maximum Factored Moment,	Mu	=	7.07	KN-m
DGCS 12.4.3.3	Limits of Reinforcement,	Mcr	=	9.40	KN-m
	1.33 Times the Factored Moment Required,	1.33*Mu	=	9.40	KN-m
DGCS 12.4.3.3-1	Cracking Moment, $Mcr = \gamma_3\gamma_1 fr Sc$	Mcr	=	41.67	KN-m
	Modulus of Rupture,	fr	=	3.33	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	=	9.40	KN-m
DGCS 12.2.3.1	Flexural Resistance Factor,	Φf	=	0.90	
	Area of Flexural Reinforcement,	m	=	17.65	
		x	=	0.001	
		r	=	0.001	
		As	=	123.74	mm^2
	Required Bar Spacing, $s = Ab * b/As$	s	=	1500.00	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 mmo Tension Reinforcement Bar spaced @ 375mm O.C.				
	Shear Verification,				
	Maximum Factored Shear,	Vu	=	33.62	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	=	200.91	mm
	Shear Depth,	de	=	202.00	mm
	Shall be Less Than the Greater of,				
	0.90*de		=	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	=	176.48	KN
	Nominal Reinf. Shear Resistance, $Vs = \frac{Av * fy * dv * cot\theta}{s}$	Vs	=	45.24	KN
	Nominal Shear Resistance,				
DGCS 12.5.3.2-1	$Vn1 = Vc + Vs$	Vn1	=	221.72	KN
DGCS 12.5.3.2-2	$Vn2 = 0.25 * f'c * bv * dv$	Vn2	=	1406.36	KN
	Governing Nominal Shear Resistance,	Vn	=	221.72	KN
	Factored Shear Resistance Verification,				
	Shear Resistance Factor,	Φv	=	0.90	
	Factored Shear Resistance, $Vr = \phi v * Vn$	Vr	=	199.55	KN
	∴ Shear Reinforcement is Adequate				

7.3.2.6 Connecting Wall

(1) Downstream Connecting Wall

Location of the downstream connecting wall of MCGS is highlighted in **Figure 7.3.85**. In addition, the section view is shown in **Figure 7.3.86**.

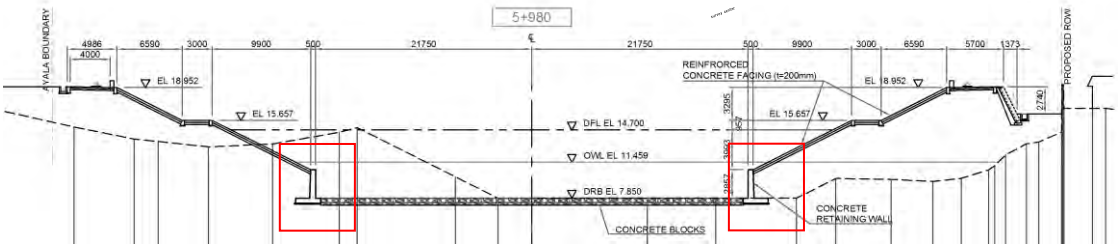
PLAN



Source: Study Team

Figure 7.3.85 Plan of the Downstream Connecting Wall

CROSS SECTION



Source: Study Team

Figure 7.3.86 Section View (STA.5 + 980 Cross Section)

(2) Design Policy

The section direction is examined as an inverse T-type retaining wall. The length of the retaining wall is 48.40m, and it is divided into the following sections considering the joint division installed every 15 m to 20 m.

Section A: The area where the Bottom Slab of the apron is located in front (Length: 11.00 m)

Section B: Range of Front Concrete Blocks are located in front (Length 20.00 m)

Section C: Range of Front Concrete Blocks are located in front (Length 17.40 m)

As for section A, the retaining wall is connected to the Apron Bottom Slab, so the sliding check in the stability study is omitted. As for sections B and C, the earth pressure from the back side is large and stability against sliding cannot meet the allowable value. Since the foundation ground in these sections is rock, it was decided to increase sliding resistance by attaching projections under the Bottom Slab of the retaining wall. The projection had a square cross-sectional shape and a minimum member thickness of 350 mm. If the allowable shear stress of concrete is not satisfied, the member thickness is increased by 50 mm.

In addition, since both the left and right bank sides are rock formations and the backside is backfilling soil, the cross-sectional shape of each section was the same, and only the left bank side was used for stability calculation and cross section calculation.

(3) Body Design

1) Calculation Case

The calculation case is as follows.

Table 7.3.99 List of Downstream Side Retaining Wall Examination Cases

Study Case	Condition
Case 1	Normal + Low Water Level
Case 2	Normal + Design High Water Level
Case 3	Normal + When Water Level Drops
Case 4	Seismic + Low Water Level
Case 5	During Construction

Source: Study Team

2) Design Condition

(a) Friction Coefficient

Friction coefficient ($\mu = \tan\varphi$)for the retaining wall is 0.70, considering that the foundation is rock[4].

(b) Bearing Capacity of Foundation Ground

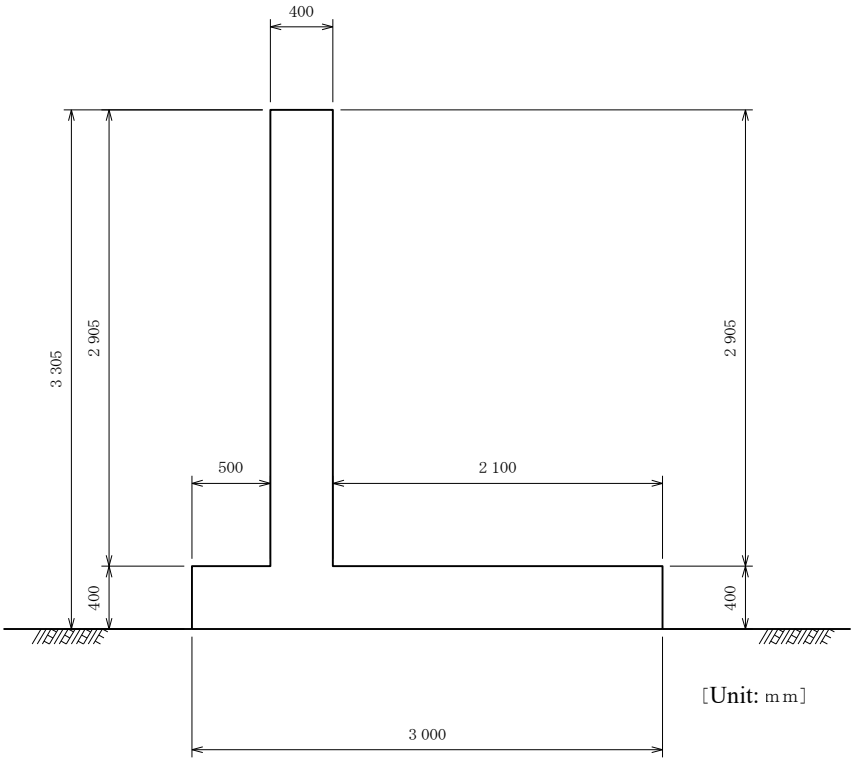
Bearing capacity (qu)for the retaining wall is 1,000 kN/m^2, considering that the foundation is a hard rock with few cracks[5].

[4] Road Earthwork Guideline (Retaining Wall) p70

[5] Road Earthwork Guideline (Retaining Wall) p69

3) Section A

(a) Body Shape



Depth Direction Width (Block Length) B = 11000 (Mm)

(b) Result of Stability Calculation

(i) Overturning

$$d = \frac{\Sigma Mr - \Sigma Mt}{\Sigma V}$$

Here,

d: distance from toe to point of action of resultant force (m)

Σ Mr: resistance moment around the point (kNm)

ΣMt: tipping moment (kNm)

ΣV: total vertical load (kN) at Bottom Slab bottom

$$e = \frac{B}{2} - d$$

Here,

e: the eccentric distance (m) from the center of the Bottom Slab of the point of action of the resultant force

B: Bottom Slab width (m), B = 3.000

ea = B/n

Here,

ea: allowable eccentric distance (m)

n: ratio of allowable eccentricity to base plate width

Load Conditions (Water Level)	Σ Mr (kNm)	ΣMt (kNm)	ΣV (kN)	d (m)	e (m)		Ea (m)	Judgement
Normal LWL (at Normal LWL)	311.546	106.798	151.442	1.352	0.148	≦	0.500	○
Normal HWL (at Normal HWL)	253.713	69.697	130.138	1.414	0.086	≦	0.500	○
Normal at the Top (Normal-on crown height)	358.345	166.145	186.626	1.030	0.470	≦	0.500	○
Seismic Low Water Level (Low water level during earthquake)	538.502	351.438	227.094	0.824	0.676	≦	1.000	○
Temporary (Time of temporary construction)	492.198	151.611	245.303	1.388	0.112	≦	0.500	○

(ii) Bearing Capacity

1)When the resultant force acts within the Bottom Slab width of 1/3 (middle-third) at the center of the Bottom Slab

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

2)When the resultant force acts within the Bottom Slab width of 2/3 at the center of the Bottom Slab

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

Here,

ΣV: total vertical load acting on lower surface of Bottom Slab (kN)

B: Bottom Slab width (m), B = 3.000

e: Eccentricity (m)

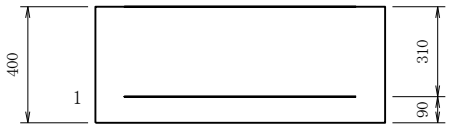
Load Conditions	Action width of ground reaction force (m)	Subgrade reaction shape	Subgrade reaction (kN/m2)			Judgement
			qmin	qmax	Tolerance	
Normal LWL (at Normal LWL)	3.000	trapezoid	35.538	65.424 ≦	1000.000	○
Normal HWL (at Normal HWL)	3.000	trapezoid	35.918	50.841 ≦	1000.000	○
Normal at the Top (Normal-on crown height)	3.000	trapezoid	3.716	120.701 ≦	1000.000	○
Seismic Low Water Level (Low water level during earthquake)	2.472	triangle	0.000	183.733 ≦	1500.000	○
Temporary (Time of temporary construction)	3.000	trapezoid	63.523	100.013 ≦	1000.000	○

(c) Section Calculation Results

(i) Vertical Wall

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar area (cm2/piece)	Number of Piece	Amount of Rebar (cm2)
Front	1'	—	—	—	—	—
	2'	—	—	—	—	—
Back	1	9.0	D25	4.909	4.000	19.635
	2	—	—	—	—	—

required rebar quantity in tension side 14.873 (cm2)

[Checking of Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgement
				Calculated Value	Tolerance	Calculated Value	Tolerance	
Normal LWL (at Normal LWL)	30.689	0.000	8.848	2.473 ≦	8.280	55.722 ≦	168.000	○
Normal HWL (at Normal HWL)	20.528	0.000	8.848	1.654 ≦	8.280	37.273 ≦	168.000	○
Normal at the Top (Normal-on crown height)	70.909	0.000	8.848	5.714 ≦	8.280	128.748 ≦	168.000	○
Seismic Low Water Level (Low water level during earthquake)	121.125	0.000	8.848	9.760 ≦	11.012	219.924 ≦	223.440	○
Temporary (Time of temporary construction)	48.292	0.000	8.848	3.891 ≦	8.280	87.682 ≦	168.000	○

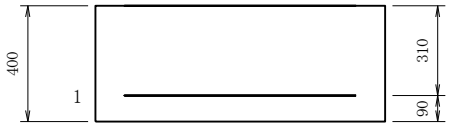
[Check of Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Judgement
			Calculated value τ	Allowable value τa1	allowable value τa2	
Normal LWL (at Normal LWL)	31.704	31.000	0.102 ≦	0.360	1.600	○
Normal HWL (at Normal HWL)	21.207	31.000	0.068 ≦	0.360	1.600	○
Normal at the top (Normal-on crown height)	73.239	31.000	0.236 ≦	0.360	1.600	○
Seismic Low water level (Low water level during earthquake)	122.338	31.000	0.395 ≦	0.479	2.128	○
Temporary (Time of temporary construction)	49.888	31.000	0.161 ≦	0.360	1.600	○

(ii) Toe Plate

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar area (cm2/piece)	Number of piece	Amount of rebar (cm2)
Front	1'	—	–	—	—	—
	2'	—	–	—	—	—
Back	1	9.0	D20	3.142	4.000	12.566
	2	—	–	—	—	—

required rebar quantity in tension side 2.658 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgement
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	7.261	7.319	0.695 ≦	8.280	20.231 ≦	168.000	○
Normal HWL (at Normal HWL)	5.542	7.319	0.530 ≦	8.280	15.441 ≦	168.000	○
Normal at the top (Normal-on crown height)	13.300	7.319	1.273 ≦	8.280	37.058 ≦	168.000	○
Seismic Low water level (Low water level during earthquake)	20.709	7.319	1.982 ≦	11.012	57.700 ≦	223.440	○
Temporary (Time of temporary construction)	11.048	7.319	1.057 ≦	8.280	30.784 ≦	168.000	○

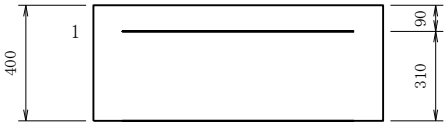
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judgement
			Calculated value τ	Allowable value τa1	
Normal LWL (at Normal LWL)	17.475	310.000	0.056 ≦	0.360	○
Normal HWL (at Normal HWL)	13.324	310.000	0.043 ≦	0.360	○
Normal at the top (Normal-on crown height)	32.061	310.000	0.103 ≦	0.360	○
Seismic Low water level (Low water level during earthquake)	50.072	310.000	0.162 ≦	0.479	○
Temporary (Time of temporary construction)	26.577	310.000	0.086 ≦	0.360	○

(iii) Heel Plate

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of Rebar (cm2)
Front	1	9.0	D25	4.909	4.000	19.635
	2	—	—	—	—	—
Back	1'	—	—	—	—	—
	2'	—	—	—	—	—

Required rebar quantity in tension side 14.873 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgement
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	30.689	8.848	2.473 ≦	8.280	55.722 ≦	168.000	○
Normal HWL (at Normal HWL)	20.528	8.848	1.654 ≦	8.280	37.273 ≦	168.000	○
Normal at the top (Normal-on crown height)	70.909	8.848	5.714 ≦	8.280	128.748 ≦	168.000	○
Seismic Low water level (Low water level during earthquake)	121.125	8.848	9.760 ≦	11.012	219.924 ≦	223.440	○
Temporary (Time of temporary construction)	48.292	8.848	3.891 ≦	8.280	87.682 ≦	168.000	○

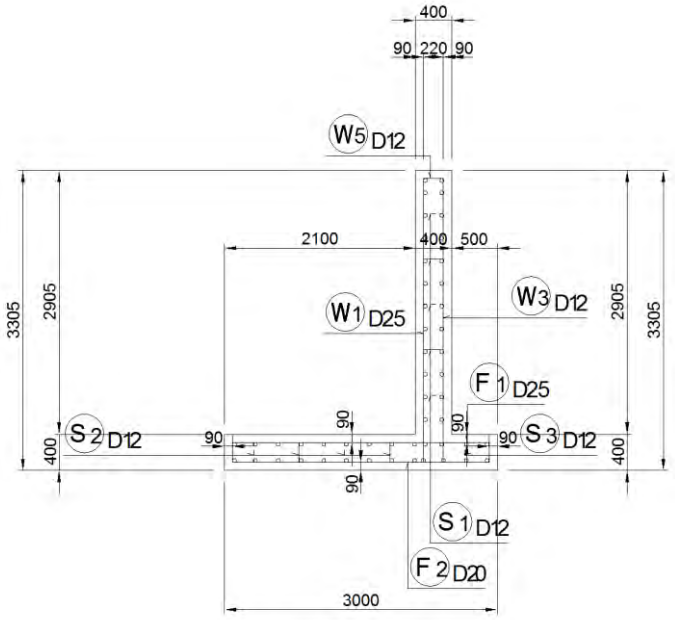
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judgement
			Calculated value τ	Allowable value τa1	
Normal LWL (at Normal LWL)	36.695	310.000	0.118 ≦	0.360	○
Normal HWL (at Normal HWL)	23.928	310.000	0.077 ≦	0.360	○
Normal at the top (Normal-on crown height)	65.131	310.000	0.210 ≦	0.360	○
Seismic Low water level (Low water level during earthquake)	127.209	310.000	0.410 ≦	0.479	○
Temporary (Time of temporary construction)	52.508	310.000	0.169 ≦	0.360	○

(d) Bar Arrangement

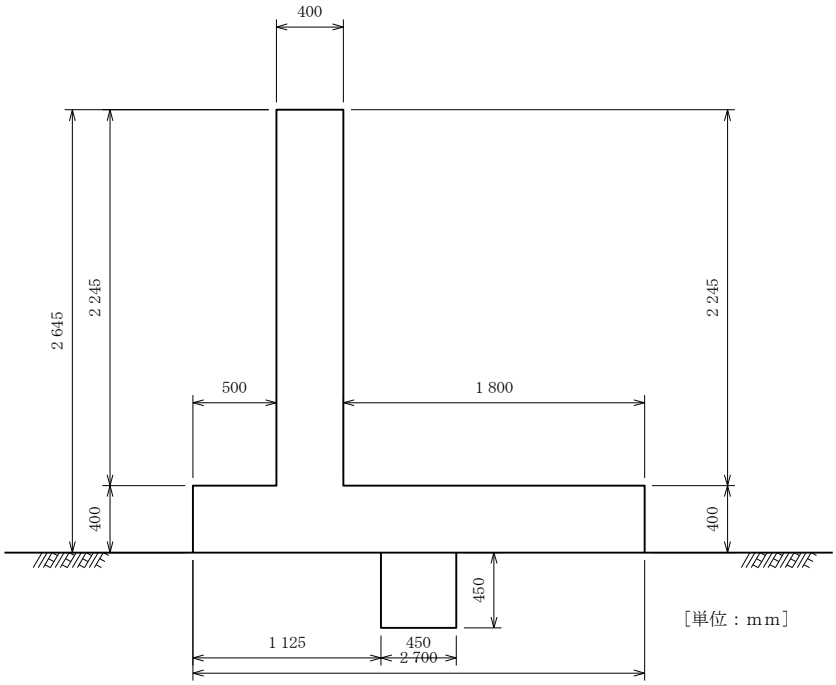
The bar arrangement diagram based on the calculation results is shown below.

Vertical main bar: D 25 @ 250, Main bar of toe: D 25 @ 250, Main bar of heel: D 20 @ 250



4) Section B

(a) Body Shape



Depth direction width (block length) B = 20,000 (mm)

(b) Result of Stability Calculation

The results of stability calculations are shown below. Detailed calculations are shown in the separate book of structural calculations.

(i) Overturning

Load Conditions (Water Level)	Σ Mr (kNm)	ΣMt (kNm)	ΣV (kN)	d (m)	e (m)		Ea (m)	Judge ment
Normal LWL (at Normal LWL)	183.991	55.120	100.852	1.278	0.072	≦	0.450	○
Normal HWL (at Normal HWL)	162.245	36.601	92.793	1.354	0.004	≦	0.450	○
Normal at the top (Normal-on crown height)	232.018	89.998	133.553	1.063	0.287	≦	0.450	○
Seismic Low water level (Low water level during earthquake)	340.399	207.261	158.781	0.839	0.511	≦	0.900	○
Temporary (Time of temporary construction)	319.441	86.735	176.249	1.320	0.030	≦	0.450	○

(ii) Sliding

$$Hk = c \cdot b1 + v1 \cdot \tan\varphi + (v2 + v3) \cdot \tan\varphi B$$

Here,

Hk : the shear resistance acting between the bottom of the foundation and the ground (kN/m)

B ': effective load width (m), provided that the eccentricity of the load is calculated at the actual base position.

b1: Distance (m) from the front end of the bottom surface of the retaining wall to the front surface of the projection within the effective loading width

b2: protrusion width (m) within the effective load width

b3: Distance (m) from the back of the protrusion to the rear of the effective load width

v1 to v3: Calculate the vertical load (kN) acting on b1 to b3 by vi = bi/B '· Vo.

Vo: Total vertical load at base bottom (kN/m)

tan φB: friction coefficient between base bottom and ground, tan φB = 0.700

tan φ: friction coefficient of foundation ground, tan φ = 0.577

c : Cohesion of foundation ground (kN/m2), c = 1000.000

Load Conditions (Water Level)	Effective Load width B ' (m)	Width of projection front b1 (m)	Width of projection base b2 (m)	Width of projection back b3 (m)	Vertical Load v1 (kN)	Vertical Load v2 (kN)	Vertical Load v3 (kN)
Normal LWL (at Normal LWL)	2.556	1.125	0.450	0.981	44.389	17.756	38.707
Normal HWL (at Normal HWL)	2.692	1.117	0.450	1.125	38.503	15.511	38.779
Normal at the top (Normal-on crown height)	2.126	1.125	0.450	0.551	70.671	28.268	34.613
Seismic Low water level (Low water level during earthquake)	1.678	1.125	0.450	0.103	106.453	42.581	9.746
Temporary (Time of temporary construction)	2.640	1.125	0.450	1.065	75.106	30.042	71.100

Fs = Hk/Ho

Here,

Fs : Safety factor against sliding

Ho : The sum of the horizontal components of each load acting on bottom of the retaining wall (kN/m)

Load Conditions (Water Level)	Shear Resistance Hk (kN/m)	Horizontal force Ho (kN/m)	Safety Factor Fs		Required Safety factor Fsa	Judgement
Normal LWL (at Normal LWL)	1190.152	46.633	25.522	≧	1.500	○
Normal HWL (at Normal HWL)	1177.233	30.965	38.018	≧	1.500	○
Normal at the top (Normal-on crown height)	1209.819	84.851	14.258	≧	1.500	○
Seismic Low water level (Low water level during earthquake)	1223.090	168.938	7.240	≧	1.200	○
Temporary (Time of temporary construction)	1239.163	73.380	16.887	≧	1.500	○

(iii) Bearing Capacity

The formula is shown in P7-308.

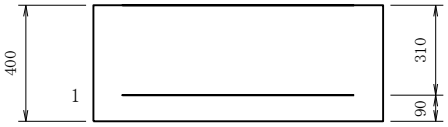
	Action width of ground reaction force (m)	Subgrade reaction Shape	Subgrade Reaction (kN/m2)			Judgement
			qmin	qmax	Tolerance	
Normal LWL (at Normal LWL)	2.700	Trapezoid	31.361	43.344 ≦	1000.000	○
Normal HWL (at Normal HWL)	2.700	Trapezoid	34.060	34.675 ≦	1000.000	○
Normal at the top (Normal-on crown height)	2.700	Trapezoid	17.961	80.967 ≦	1000.000	○
Seismic Low water level (Low water level during earthquake)	2.517	Triangle	0.000	126.167 ≦	1500.000	○
Temporary (Time of temporary construction)	2.700	Trapezoid	60.973	69.582 ≦	1000.000	○

(c) Section Calculation Results

(i) Vertical Wall

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of Rebar (cm2)
Front	1'	—	—	—	—	—
	2'	—	—	—	—	—
Back	1	9.0	D20	3.142	4.000	12.566
	2	—	—	—	—	—

required rebar quantity in tension side 10.133 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgement
				Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	12.572	0.000	7.319	1.203	≦ 8.280	35.028	≦ 168.000	○
Normal HWL (at Normal HWL)	9.473	0.000	7.319	0.907	≦ 8.280	26.396	≦ 168.000	○
Normal at the top (Normal-on crown height)	32.727	0.000	7.319	3.132	≦ 8.280	91.186	≦ 168.000	○
Seismic Low water level (Low water level during earthquake)	65.162	0.000	7.319	6.235	≦ 11.012	181.562	≦ 223.440	○
Temporary (Time of temporary construction)	23.103	0.000	7.319	2.211	≦ 8.280	64.371	≦ 168.000	○

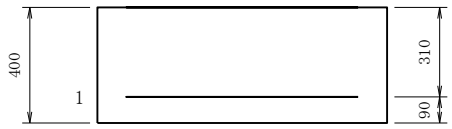
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Judgement
			Calculated value τ	Allowable value τa1	Allowable value τa2	
Normal LWL (at Normal LWL)	16.807	31.000	0.054 ≦	0.360	1.600	○
Normal HWL (at Normal HWL)	12.665	31.000	0.041 ≦	0.360	1.600	○
Normal at the top (Normal-on crown height)	43.741	31.000	0.141 ≦	0.360	1.600	○
Seismic Low water level (Low water level during earthquake)	84.957	31.000	0.274 ≦	0.479	2.128	○
Temporary (Time of temporary construction)	30.886	31.000	0.100 ≦	0.360	1.600	○

(ii) Toe Plate

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of rebar (cm2)
Front	1'	—	–	—	—	—
	2'	—	–	—	—	—
Back	1	9.0	D16	1.986	4.000	8.042
	2	—	–	—	—	—

required rebar quantity in tension side 2.097 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2) Calculated value Tolerance	Tensile Stress (N/mm2) Calculated value Tolerance	Judgement
Normal LWL (at Normal LWL)	4.616	6.016	0.530 ≦ 8.280	19.794 ≦ 168.000	○
Normal HWL (at Normal HWL)	3.552	6.016	0.408 ≦ 8.280	15.234 ≦ 168.000	○
Normal at the top (Normal-on crown height)	8.635	6.016	0.991 ≦ 8.280	37.028 ≦ 168.000	○
Seismic Low water level (Low water level during earthquake)	14.017	6.016	1.608 ≦ 11.012	60.108 ≦ 223.440	○
Temporary (Time of temporary construction)	7.431	6.016	0.853 ≦ 8.280	31.867 ≦ 168.000	○

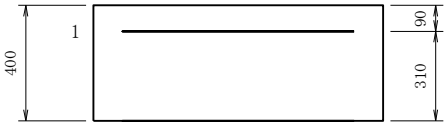
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2) Calculated value τ Allowable value τa1	Judgement
Normal LWL (at Normal LWL)	11.100	310.000	0.036 ≦ 0.360	○
Normal HWL (at Normal HWL)	8.524	310.000	0.027 ≦ 0.360	○
Normal at the top (Normal-on crown height)	20.792	310.000	0.067 ≦ 0.360	○
Seismic Low water level (Low water level during earthquake)	33.891	310.000	0.109 ≦ 0.479	○
Temporary (Time of temporary construction)	17.851	310.000	0.058 ≦ 0.360	○

(iii) Heel Plate

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar Size	Rebar Area (cm2/piece)	Number of Piece	Amount of rebar (cm2)
Front	1	9.0	D20	2.865	4.000	12.566
	2	—	—	—	—	—
Back	1'	—	—	—	—	—
	2'	—	—	—	—	—

required rebar quantity in tension side 10.133 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judge ment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	12.572	7.319	1.203 ≦	8.280	35.028 ≦	168.000	○
Normal HWL (at Normal HWL)	9.473	7.319	0.907 ≦	8.280	26.396 ≦	168.000	○
Normal at the top (Normal-on crown height)	32.727	7.319	3.132 ≦	8.280	91.186 ≦	168.000	○
Seismic Low water level (Low water level during earthquake)	65.162	7.319	6.235 ≦	11.012	181.562 ≦	223.440	○

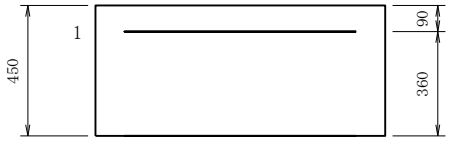
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (mm)	Shear Stress (N/mm2)		Judge ment
			Calculated value τ	Allowable value τa1	
Normal LWL (at Normal LWL)	21.556	310.000	0.070 ≦	0.360	○
Normal HWL (at Normal HWL)	13.735	310.000	0.044 ≦	0.360	○
Normal at the top (Normal-on crown height)	39.165	310.000	0.126 ≦	0.360	○
Seismic Low water level (Low water level during earthquake)	84.304	310.000	0.272 ≦	0.479	○
Temporary (Time of temporary construction)	33.326	310.000	0.108 ≦	0.360	○

(iv) Projection

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of rebar (cm2)
Front	1	9.0	D16	1.986	4.000	8.042
	2	—	—	—	—	—
Back	1'	—	—	—	—	—
	2'	—	—	—	—	—

required rebar quantity in tension side 4.626 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	9.980	6.530	0.904 ≦	8.280	36.698 ≦	168.000	○
Normal HWL (at Normal HWL)	6.647	6.530	0.602 ≦	8.280	24.443 ≦	168.000	○
Normal at the top (Normal-on crown height)	17.928	6.530	1.623 ≦	8.280	65.929 ≦	168.000	○
Seismic Low water level (Low water level during earthquake)	35.483	6.530	3.213 ≦	11.012	130.484 ≦	223.440	○
Temporary (Time of temporary construction)	15.147	6.530	1.371 ≦	8.280	55.700 ≦	168.000	○

[Shear Stress]

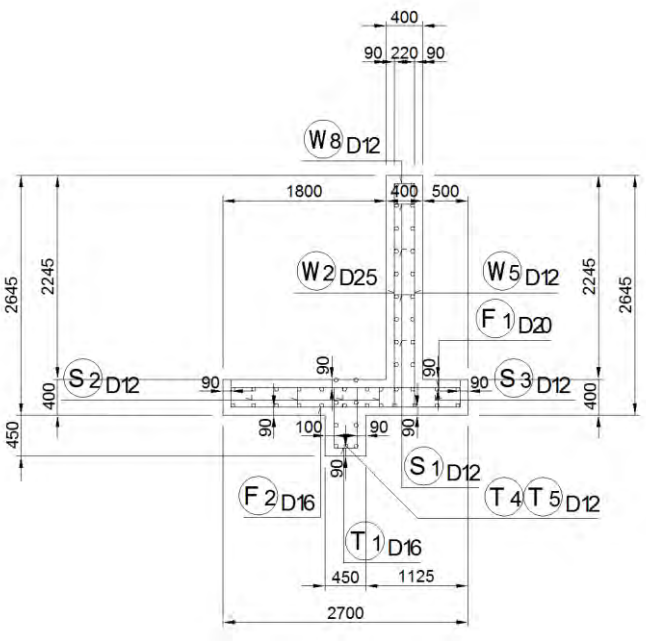
Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Judgment
			Calculated value τ	Allowable value τ a1	Allowable value τ a2	
Normal LWL (at Normal LWL)	44.354	36.000	0.123 ≦	0.360	1.600	○
Normal HWL (at Normal HWL)	29.542	36.000	0.082 ≦	0.360	1.600	○
Normal at the top (Normal-on crown height)	79.682	36.000	0.221 ≦	0.360	1.600	○
Seismic Low water level (Low water level during earthquake)	157.703	36.000	0.438 ≦	0.479	2.128	○
Temporary (Time of temporary construction)	67.319	36.000	0.187 ≦	0.360	1.600	○

(d) Bar Arrangement

The bar arrangement diagram based on the calculation results is shown below.

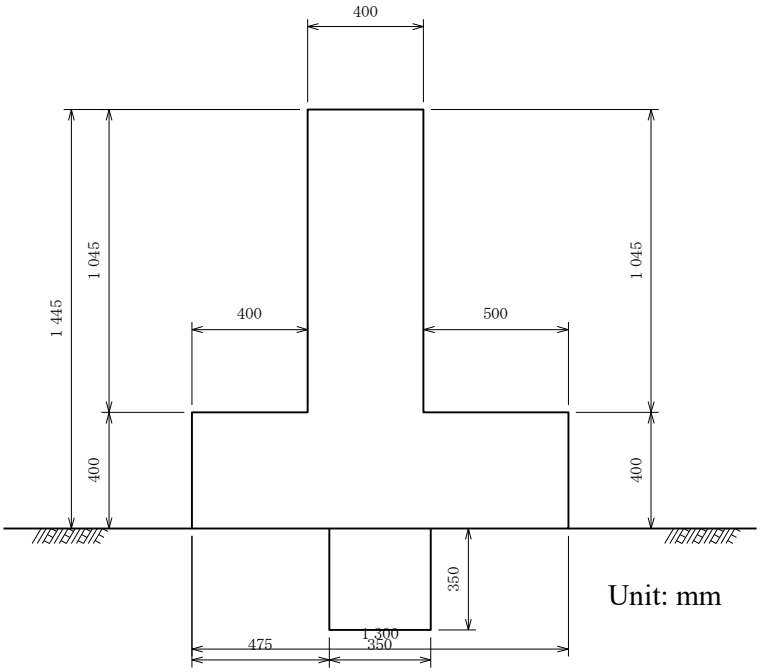
Vertical main bar: D 20 @ 250, Toe main bar: D 20 @ 250

Heel main bar: D 16 @ 250, Protrusion main bar: D 16 @ 250



5) Section C

(a) Body Shape



Unit: mm

Depth direction width (block length) B = 17400 (mm)

(b) Result of Stability Calculation

(i) Overturning

Load Conditions (Water Level)	Σ Mr (kNm)	ΣMt (kNm)	ΣV (kN)	d (m)	e (m)		Ea (m)	Judgement
Normal LWL (at Normal LWL)	19.664	4.332	23.121	0.663	0.013	≦	0.217	○
Normal HWL (at Normal HWL)	19.244	3.973	22.803	0.670	0.020	≦	0.217	○
Normal at the top (Normal-on crown height)	25.796	11.295	30.579	0.474	0.176	≦	0.217	○
Seismic Low water level (Low water level during earthquake)	54.757	34.829	50.116	0.398	0.252	≦	0.433	○
Temporary (Time of temporary construction)	37.999	10.516	42.934	0.640	0.010	≦	0.217	○

(ii) Sliding

Calculation formula is shown in P7-313.

Load Conditions (Water Level)	Effective Load width B ' (m)	Width of projection front b1 (m)	Width of projection base b2 (m)	Width of projection back b3 (m)	Vertical load v1 (kN)	Vertical load v2 (kN)	Vertical load v3 (kN)
Normal LWL (at Normal LWL)	1.274	0.449	0.350	0.475	8.149	6.352	8.621
Normal HWL (at Normal HWL)	1.260	0.435	0.350	0.475	7.872	6.334	8.596
Normal at the top (Normal-on crown height)	0.948	0.475	0.350	0.123	15.322	11.290	3.968
Seismic Low water level (Low water level)	0.796	0.475	0.321	0.000	29.906	20.210	0.000
Temporary	1.280	0.475	0.350	0.455	15.933	11.740	15.262

Load Conditions (Water Level)	Shear Resistance Hk (kN/m)	Horizontal force Ho (kN/m)	Safety factor Fs		Required safety factor Fsa	Judgement
Normal LWL (at Normal LWL)	464.186	7.668	60.535	≧	1.500	○
Normal HWL (at Normal HWL)	449.997	7.031	64.002	≧	1.500	○
Normal at the top (Normal-on crown height)	494.526	21.499	23.002	≧	1.500	○
Seismic Low water level (Low water level)	506.414	60.239	8.407	≧	1.200	○
Temporary	503.100	18.613	27.029	≧	1.500	○

(iii) Bearing capacity

The formula is shown in7-308.

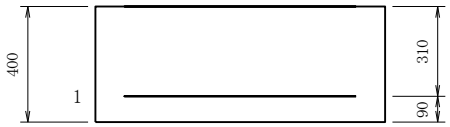
Load Conditions (Water Level)	Action width of ground reaction force (m)	Subgrade reaction Shape	Subgrade Reaction (kN/m2)			Judgement
			qmin	qmax	Tolerance	
Normal LWL (at Normal LWL)	1.300	Trapezoid	16.712	18.859 ≦	981.000	○
Normal HWL (at Normal HWL)	1.300	Trapezoid	15.946	19.136 ≦	981.000	○
Normal at the top (Normal-on crown height)	1.300	Trapezoid	4.440	42.606 ≦	981.000	○
Seismic Low water level (Low water level)	1.194	Triangle	0.000	83.947 ≦	1470.000	○
Temporary	1.300	Trapezoid	31.520	34.532 ≦	981.000	○

(c) Section Calculation Results

(i) Vertical Wall

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar Size	Rebar Area (cm2/piece)	Number of Piece	Amount of Rebar (cm2)
Front	1'	—	—	—	—	—
	2'	—	—	—	—	—
Back	1	9.0	D16	1.986	4.000	8.042
	2	—	—	—	—	—

required rebar quantity in tension side 1.841 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	N (kN)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgment
				Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	0.955	0.000	6.016	0.110 ≦	8.280	4.095 ≦	168.000	○
Normal HWL (at Normal HWL)	0.955	0.000	6.016	0.110 ≦	8.280	4.095 ≦	168.000	○
Normal at the top (Normal-on crown height)	3.300	0.000	6.016	0.379 ≦	8.280	14.150 ≦	168.000	○
Seismic Low water level (Low water level)	12.330	0.000	6.016	1.415 ≦	11.012	52.873 ≦	223.440	○
Temporary	2.500	0.000	6.016	0.287 ≦	8.280	10.722 ≦	168.000	○

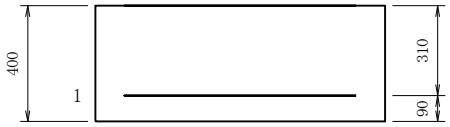
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Correction factor			Judgment
			Calculated value τ	Allowable value τ a1	Allowable value τ a2	Ce	Cpt	CN	
Normal LWL (at Normal LWL)	2.744	31.000	0.009 ≦	0.482	1.600	1.39	0.96	1.00	○
Normal HWL (at Normal HWL)	2.744	31.000	0.009 ≦	0.482	1.600	1.39	0.96	1.00	○
Normal at the top (Normal-on crown height)	9.477	31.000	0.031 ≦	0.482	1.600	1.39	0.96	1.00	○
Seismic Low water level (Low water level)	34.424	31.000	0.111 ≦	0.385	2.128	1.39	0.96	1.00	○
Temporary	7.185	31.000	0.023 ≦	0.482	1.600	1.39	0.96	1.00	○

(ii) Toe Plate

[Bar Arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of rebar (cm2)
Front	1'	—	–	——	—	——
	2'	—	–	——	—	——
Back	1	9.0	D16	1.986	4.000	8.042
	2	—	–	——	—	——

required rebar quantity in tension side 0.814 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgement
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	0.900	6.016	0.103 ≦	8.280	3.861 ≦	168.000	○
Normal HWL (at Normal HWL)	0.848	6.016	0.097 ≦	8.280	3.636 ≦	168.000	○
Normal at the top (Normal-on crown height)	2.444	6.016	0.280 ≦	8.280	10.478 ≦	168.000	○
Seismic Low water level (Low water level)	5.512	6.016	0.632 ≦	11.012	23.635 ≦	223.440	○
Temporary	1.970	6.016	0.226 ≦	8.280	8.447 ≦	168.000	○

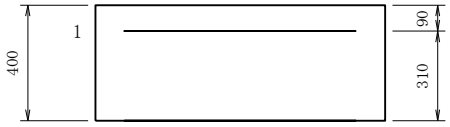
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d '(cm)	Shear Stress (N/mm2)			Correction Factor			Judgement
			Calculated value τ	Allowable value τ a1	Allowable value τ a2	Ce	Cpt	Cdc	
Normal LWL (at Normal LWL)	2.240	31.000	0.007 ≦	2.761	1.600	1.39	0.96	5.73	○
Normal HWL (at Normal HWL)	2.102	31.000	0.007 ≦	2.768	1.600	1.39	0.96	5.75	○
Normal at the top (Normal-on crown height)	6.232	31.000	0.020 ≦	2.684	1.600	1.39	0.96	5.57	○
Seismic Low water level (Low water level)	14.247	31.000	0.046 ≦	2.110	2.128	1.39	0.96	5.48	○
Temporary	4.940	31.000	0.016 ≦	2.737	1.600	1.39	0.96	5.68	○

(iii) Heel Plate

[Bar Arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of rebar (cm2)
Front	1	9.0	D16	1.986	4.000	8.042
	2	—	—	—	—	—
Back	1'	—	—	—	—	—
	2'	—	—	—	—	—

required rebar quantity in tension side 1.762 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judgement
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	0.955	6.016	0.110 ≦	8.280	4.095 ≦	168.000	○
Normal HWL (at Normal HWL)	0.955	6.016	0.110 ≦	8.280	4.095 ≦	168.000	○
Normal at the top (Normal-on crown height)	3.300	6.016	0.379 ≦	8.280	14.150 ≦	168.000	○
Seismic Low water level (Low water level)	11.812	6.016	1.355 ≦	11.012	50.654 ≦	223.440	○
Temporary	2.500	6.016	0.287 ≦	8.280	10.722 ≦	168.000	○

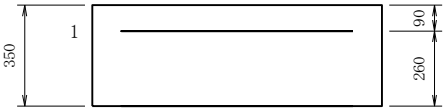
[Shear Stress]

Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d '(cm)	Shear Stress (N/mm2)			Correction factor			Judgement
			Calculated value τ	Allowable value τ a1	Allowable value τ a2	Ce	Cpt	Cdc	
Normal LWL (at Normal LWL)	3.075	31.000	0.010 ≦	0.957	1.600	1.39	0.96	1.99	○
Normal HWL (at Normal HWL)	2.760	31.000	0.009 ≦	0.959	1.600	1.39	0.96	1.99	○
Normal at the top (Normal-on crown height)	8.439	31.000	0.027 ≦	1.043	1.600	1.39	0.96	2.16	○
Seismic Low water level (Low water level)	30.011	31.000	0.097 ≦	0.783	2.128	1.39	0.96	2.03	○
Temporary	8.092	31.000	0.026 ≦	0.972	1.600	1.39	0.96	2.02	○

(iv) Projection

[Bar arrangement]

Front



Unit: mm

Back

Single Rebar

Location		Cover (cm)	Rebar size	Rebar Area (cm2/piece)	Number of piece	Amount of rebar (cm2)
Front	1	9.0	D16	1.986	4.000	8.042
	2	—	—	—	—	—
Back	1'	—	—	—	—	—
	2'	—	—	—	—	—

required rebar quantity in tension side 1.806 (cm2)

[Bending Stress]

Load Conditions (Water Level)	M (kNm)	x (cm)	Compressive stress (N/mm2)		Tensile Stress (N/mm2)		Judge ment
			Calculated value	Tolerance	Calculated value	Tolerance	
Normal LWL (at Normal LWL)	1.308	5.453	0.198 ≦	8.280	6.727 ≦	168.000	○
Normal HWL (at Normal HWL)	1.199	5.453	0.182 ≦	8.280	6.166 ≦	168.000	○
Normal at the top (Normal-on crown height)	3.660	5.453	0.555 ≦	8.280	18.823 ≦	168.000	○
Seismic Low water level (Low water level)	10.106	5.453	1.533 ≦	11.012	51.979 ≦	223.440	○
Temporary	3.116	5.453	0.473 ≦	8.280	16.026 ≦	168.000	○

[Shear Stress]

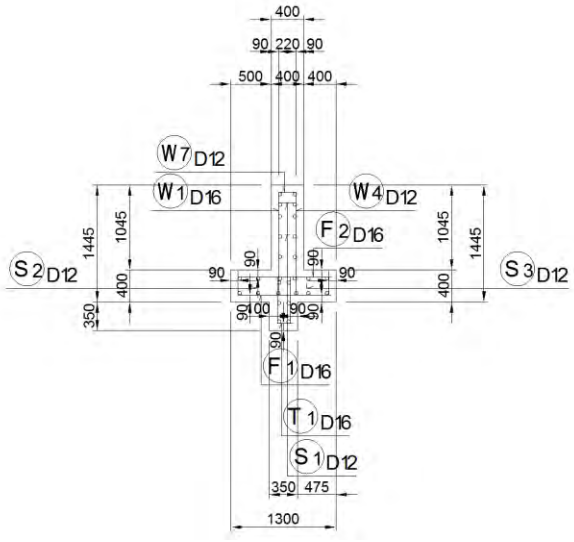
Load Conditions (Water Level)	Shear force Sh (kN)	Effective height d (cm)	Shear Stress (N/mm2)			Correction factor		Judge ment
			Calculated value τ	Allowable value τ a1	Allowable value τ a2	Ce	Cpt	
Normal LWL (at Normal LWL)	7.474	26.000	0.029 ≦	0.509	1.600	1.40	1.01	○
Normal HWL (at Normal HWL)	6.851	26.000	0.026 ≦	0.509	1.600	1.40	1.01	○
Normal at the top (Normal-on crown height)	20.912	26.000	0.080 ≦	0.509	1.600	1.40	1.01	○
Seismic Low water level (Low water level)	57.749	26.000	0.222 ≦	0.407	2.128	1.40	1.01	○
Temporary	17.805	26.000	0.068 ≦	0.509	1.600	1.40	1.01	○

(d) Bar Arrangement

The bar arrangement diagram based on the calculation results is shown below.

Vertical main bar: D 16 @ 250, Toe main bar: D 16 @ 250

Heel main bar: D 16 @ 250, Protrusion main bar: D 16 @ 250



7.3.3 Gate Facility Design

7.3.3.1 Design Conditions

(1) Permanent Gate

1) No.1 Gate

1. Design Condition

1-1. Design Specification

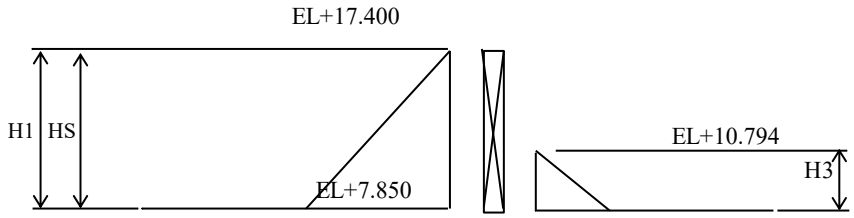
Type	:	Duplex Stainless Steel Shell Structure Roller Gate
Clear Span x Effective Height	:	W 28.300 m × H 9.550 m
Number of Gates	:	1 Gate
Water Sealing System	:	3 Way Front Rubber Watertight
Design Head	:	(Front) 9.550 m (EL + 17.400)
		(Rear) 2.944 m (EL + 10.794)
		*Case 1: Evaluation by Allowable Stress
		(Rear) 2.150 m (EL + 10.000)
		*Case 2: Evaluation by Ultimate Strength
Operating Head (Opening)	:	(Front) 4.636 m (EL + 12.486)
		(Rear) 3.636 m (EL + 11.486)
Operating Head (Closing)	:	(Front) 3.636 m (EL + 11.486)
* It is assumed that the water level is balanced during operation.		
		(Rear) 3.636 m (EL + 11.486)
Invert Elevation	:	EL + 7.850
Type of Hoist	:	Wire drum winch (2M2D)
Hoisting Height	:	11.150 m
Main Materials	:	Gate Leaf : SUS 323 L Guide frame : 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)
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

2) No. 2Gate

1. Design Condition

1-1. Design Specification

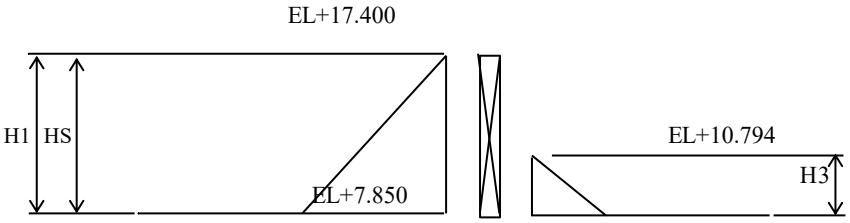
Type	:	Duplex Stainless Steel Roller Gate
Clear Span x Effective Height	:	W 11.700 m × H 9.550 m
Number of Gates	:	1 gate
Water Sealing System	:	Three (3) Way Front Rubber Watertight
Design Head	:	(Front) 9.550 m (EL + 17.400)
		(Rear) 2.944 m (EL + 10.794)
		*Case 1: Evaluation by Allowable Stress
		(Rear) 2.150 m (EL + 10.000)
		*Case 2: Evaluation by Ultimate Strength
Operating Head (Opening)	:	(Front) 4.636 m (EL + 12.486)
		(Rear) 3.636 m (EL + 11.486)
Operating Head (Closing)	:	(Front) 3.636 m (EL + 11.486)
*Operation when water level is balanced Therefore, it is assumed.		(Rear) 3.636 m (EL + 11.486)
Invert Elevation	:	EL + 7.850
Type of Hoist	:	Wire drum winch (1M 1D)
Hoisting Height	:	11.150 m
Main Materials	:	Gate Leaf : SUS 323 L Guide fram : SUS 304 N2 syster 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)
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



P_S	:	Hydrostatic Load	···		kN
P_o	:	Front Side Hydraulic Load	···		kN
P_i	:	Rear Side Hydrostatic Load	···		kN
H_S	:	Water Sealing Height	···	9.550	m
H_1	:	Front Side Design Head	···	9.550	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	···	2.944	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	···	0.000	m
B_S	:	Water Sealing Width	···	28.500	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

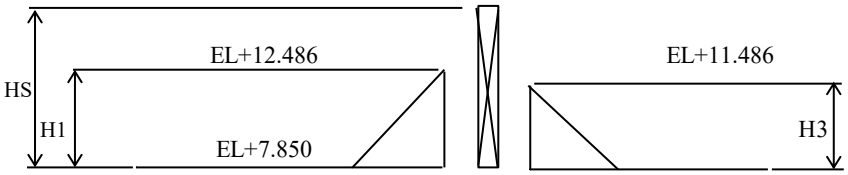
Figure 7.3.87 Load Model Diagram (Design Load: Case 1)



P_S	:	Hydrostatic load	···		kN
P_o	:	Front side hydraulic load	···		kN
P_i	:	Rear side hydrostatic load	···		kN
H_S	:	Water Sealing Height	···	9.550	m
H_1	:	Front Side Design Head	···	9.550	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	···	2.944	m
H_4	:	Rear side water depth at the top of gate leaf	···	0.000	m
B_S	:	Water Sealing Width	···	11.900	m
γ_1	:	Unit wight of water in the front side	···	9.810	kN/m3
γ_2	:	Unit wight of water in the rear side	···	9.810	kN/m3

Source: Study Team

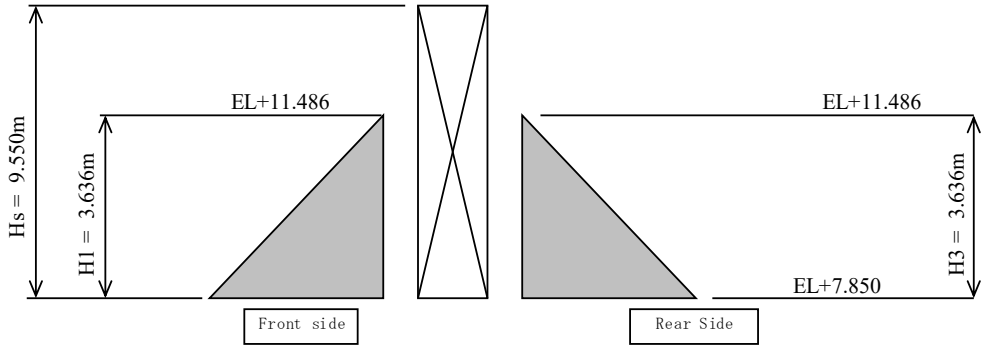
Figure 7.3.88 Load Model Diagram (Design Load: Case 2)



P_2	:	Hydrostatic Load	···		kN
P_o	:	Front Side Hydraulic Load	···		kN
P_i	:	Rear Side Hydrostatic Load	···		kN
H_S	:	Water Sealing Height	···	9.550	m
H_1	:	Front Side Design Head	···	4.636	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	···	3.636	m
H_4	:	Rear Side Water Depth at the Top of Gate Leaf	···	0.000	m
B_S	:	Water Sealing Width	···	28.500	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

Figure7.3.89 Load Model Diagram (working loads: open)



Po	:	Front side hydraulic load	···		kN
Pi	:	Rear side hydrostatic load	···		kN
HS	:	Water Sealing Height	···	9.550	m
H1	:	Front Side Operating Head	···	3.636	m
			*Assumptions (water level balance)		
H2	:	Front side water depth at the top of Gate Leaf	···	0.000	m
H3	:	Operation Head in Rear Side	···	3.636	m
			*Assumptions (water level balance)		
H4	:	Rear side water depth at the top of gate leaf	···	0.000	m
BS	:	Water Sealing Width	···	11.900	m
γ 1	:	Unit wight of water in the front side	···	9.810	kN/m3
γ2	:	Unit wight of water in the rear side	···	9.810	kN/m3

Source: Study Team

Figure7.3.90 Load Model Diagram (working loads: when closed)

(2) Stoplog

1) Upstream

1.Design Conditions (Upstream Side of MCGS)

(1) Type Steel Plate Girder Slide Gate (Strut Type Stoplog)

(2) Number of Gate 1 gate

(3) Clear Span

No.1 Gate 28.300 m (7 Split = 4.150 × 2 leaf + 4.000 × 5 leaf)

NO.2 Gate 11.700 m (3 Split = 3.850 × 2 leaf + 4.000 × 1 leaf)

(4) Height of Gate Leaf 5.950 m (3 Split = 2.150 × 2 leaf + 1.650 × 1 leaf)

(5) Design Head Upstream Side 5.850 m (EL + 13.400)

Downstream Side 0.000 m (EL + 7.550)

(6) Invert Elevation ∇ EL + 7.550

(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 + SS 400

Strut SM 400

2) Downstream

1.Design Conditions (Downstream side of the MSGS)

(1) Type Steel plate girder slide gate (Strut type Stoplog)

(2) Number of Gate 1 gate

(3) Clear Span

No.1 Gate 28.300 m (7 Split = 4.150 × 2 leaf + 4.000 × 5 leaf)

NO.2 Gate 11.700 m (3 Split = 3.850 × 2 leaf + 4.000 × 1 leaf)

(4) Height of Gate Leaf 6.450 m (3 Split = 2.150 × 3 leaf)

(5) Design Head Upstream Side 6.350 m (EL + 13.400)

Downstream Side 0.000 m (EL + 7.050)

(6) Invert Elevation EL + 7.050

(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].

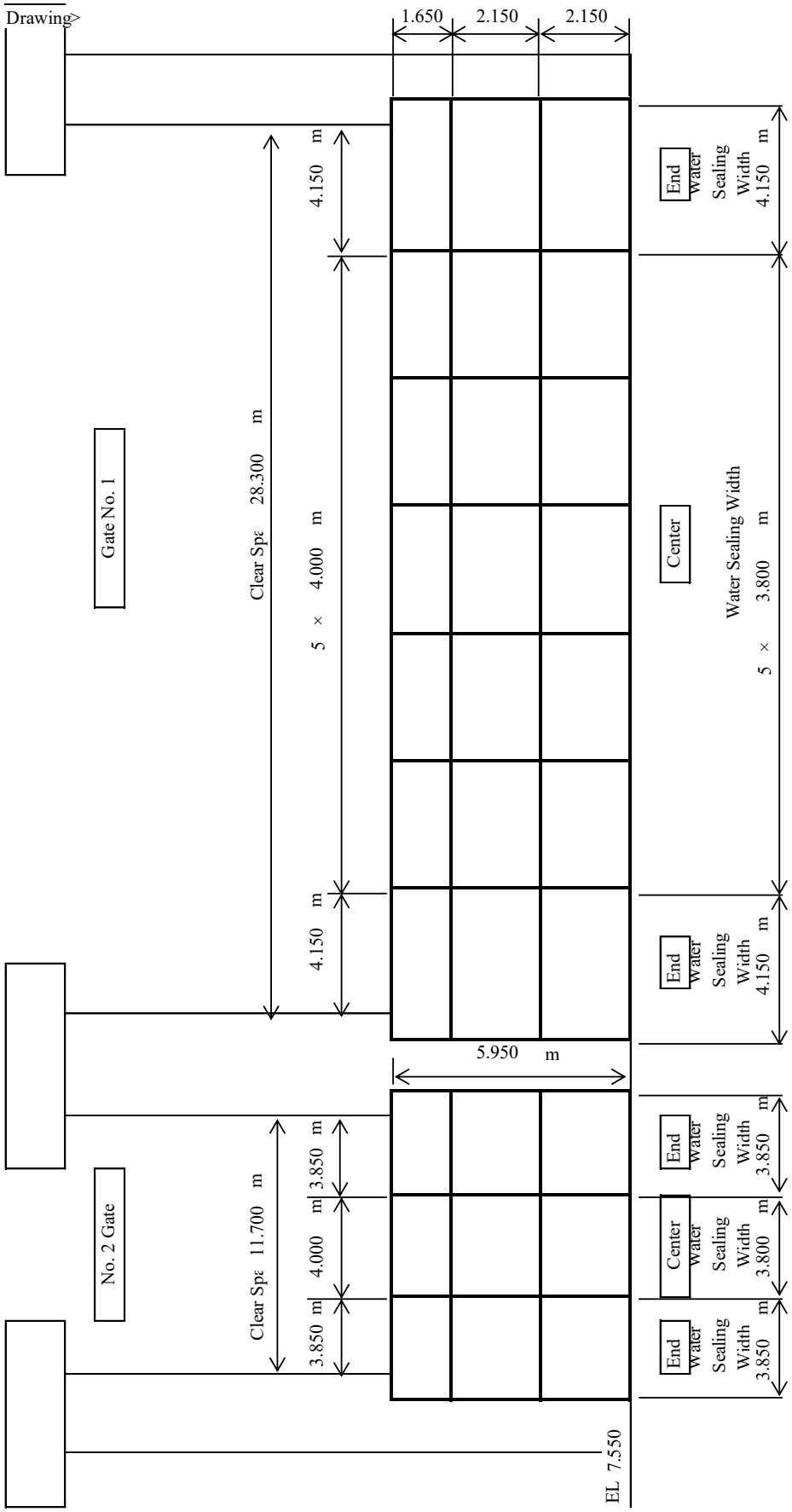
(12) Unit Weight of Water 9.81 kN / m3

(13) Material Stoplog SM 400

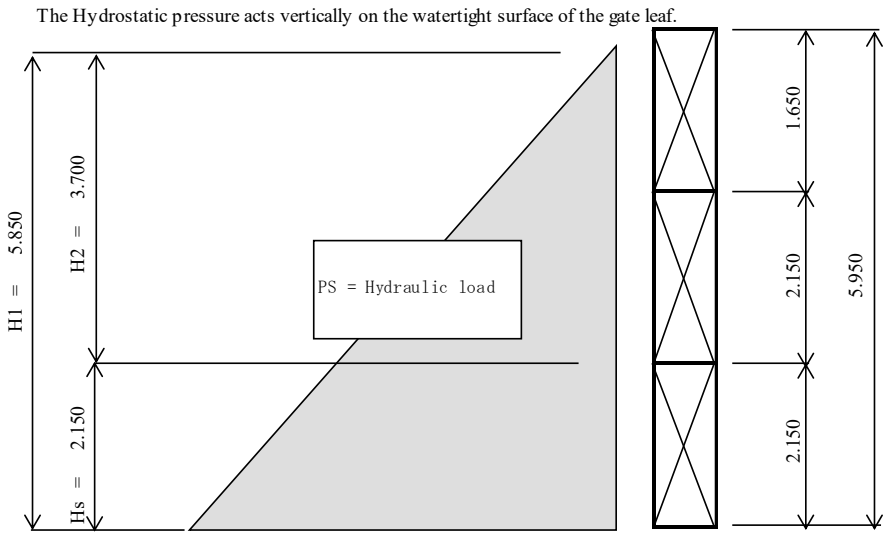
Guide Frame SUS 304 + SS 400

Strut SM 400

<General Arrangement Drawing>



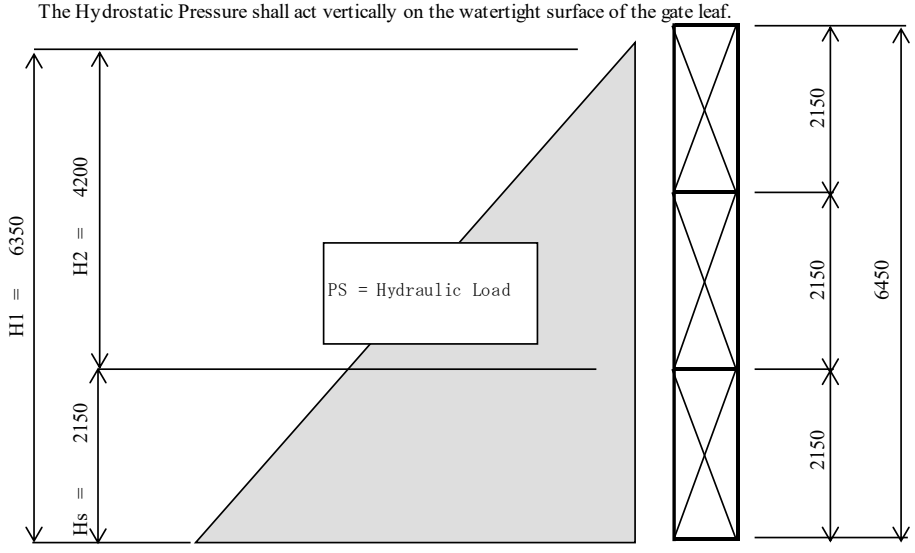
The following calculation is performed at the end of the No. 1 gate, which is the maximum span.
Since the height of the gate leaf is different, the study is divided in to the case of Lower part (Gate leaf height: 2.150 m) and the upper part (Gate leaf height: 1.650 m).



H_S	: Water Sealing Height	=	2.150	m
H_1	: Upstream Design Head	=	5.850	m
H_2	: Upstream side Depth at the Top of Stoplog	=	3.700	m
B_{S1}	: Water Sealing Width	=	4.150	m
γ_1	: Unit Weight of Water	=	9.81	kN / m3

Source: Study Team

Figure7.3.91 Load Model Diagram (Upstream)



H_S	: Water Sealing Height	=	2.150	m
H_1	: Upstream Design Head	=	6.350	m
H_2	: Upstream Side Depth at the Top of Stoplog	=	4.200	m
B_{S1}	: Water Sealing Width (End Stoplog)	=	4.150	m
γ_1	: Unit Weight of Water	=	9.81	kN / m^3

Source: Study Team

Figure7.3.92 Load Model Diagram (Downstream)

(3) Hoist

1) No.1 Gate

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) ".

Specification example
1. The descent speed by the rapid closing hoist shall be 1.0 to 2.0 m/ min, and at the maximum not more than 9.0 m / min.
2. In the case of mechanical switchgear, the self-weight lowering function is composed of a brake release device and a speed control device. Examples of the braking release device include a DC power supply device. A centrifugal brake, a fan brake, a hydraulic brake, or the like is used for the speed control device.

Source: Draft technical standard for dams and weirs, p. 43, 28.10

Type of Hoist	2 motor/2 drum wire rope winch type
Numbers of Unit	1 Unit/1 Gate
Opening and Closing Load	Opening and Closing Load W = 2700 kN Load of Gate Leaf W' ' = 2600 kN
Opening and Closing Speed	Vm = 0.300 m/min Self-weight descent speed V = 1.000 m/min
Hoisting Height	Normal Height 11.150 m Dogging Height 11.450 m Dogging Upper Limit Height 11.550 m Emergency Upper Limit Height 11.650 m
Operating System	Local Operation (Machine Side)
Power Source	220 V 60 Hz
Number of Wire Rope	8 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])

2) No. 2 Gate

The self -weight closing function is added in the same way as the No. 1 gate.

Type of Hoist	1 motor/1 drum wire rope winding type
Numbers of Unit	1 Unit/1 Gate
Opening and Closing Load	Opening and Closing Load W = 610 kN Load of Gate Leaf W' ' = 570 kN
Opening and Closing Speed	Vm = 0.300 m/min
Hoisting Height	Normal Height 11.150 m Dogging Height 11.450 m Dogging Upper Limit Height 11.550 m Emergency Upper Limit Height 11.650 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 1/400
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])

7.3.3.2 Design Calculation

In this section, only the calculation results of each facility are shown. Detailed design calculations are summarized in a separate structural calculation piece.

(1) Permanent Gate

Table 7.3.100 Calculation Result of No.1 Gate (1)

1-1. Gate Leaf

Target for Calculation	Case	Calculation Item		Calculation Results		Tolerance		Evaluation
Shell Part	Fully Closed	A – Stress Intensity		74	N/mm2	105	N/mm2	OK
		B – Stress Intensity		74	N/mm2	105	N/mm2	OK
		C – Stress Intensity		74	N/mm2	105	N/mm2	OK
		D – Stress Intensity		74	N/mm2	105	N/mm2	OK
		E – Stress Intensity		80	N/mm2	119	N/mm2	OK
		F – Stress Intensity		80	N/mm2	119	N/mm2	OK
	During Operation	A – Stress Intensity		14	N/mm2	105	N/mm2	OK
		B – Stress Intensity		14	N/mm2	105	N/mm2	OK
		C – Stress Intensity		7	N/mm2	105	N/mm2	OK
		D – Stress Intensity		0	N/mm2	105	N/mm2	OK
		E – Stress Intensity		14	N/mm2	124	N/mm2	OK
		F – Stress Intensity		8	N/mm2	124	N/mm2	OK
	Fully Closed	Deflection (Horizontal)		1/918		1/800		OK
	During Operation	Deflection (Horizontal)		1/9185		1/800		OK
		Deflection (Vertical Direction)		1/49636		1/800		OK
		Deflection (Synthesis)		1/9032		1/600		OK
	–	Rigidity of Horizontal Girder (S2)		3423	cm4	44112	cm4	OK
		Rigidity of Horizontal Girder (S4)		3037	cm4	44112	cm4	OK
		Rigidity of Horizontal Girder (S8)		4016	cm4	44112	cm4	OK
		Rigidity of Horizontal Girder (S6)		3019	cm4	44112	cm4	OK
		Sum of Shear Stress and Torsional Stress		72.3	N/mm2	104	N/mm2	OK
Skin Plate	Fully Closed	Bending Stress		37	N/mm2	180	N/mm2	OK
	During Operation	Bending Stress		6	N/mm2	180	N/mm2	OK
Horizontal girder	Fully Closed	Bending Stress	MS: Between Fulcrum (Skin Plate Side)	3	N/mm2	180	N/mm2	OK
			MF: Between Fulcrum(Flange Side)	8	N/mm2	173	N/mm2	OK
			ES: On Between Fulcrum (Skin Plate Side)	10	N/mm2	180	N/mm2	OK
			EF: On Fulcrum (Flange Side)	16	N/mm2	180	N/mm2	OK
	During Operation	Bending Stress	MS: Between Fulcrum (Skin Plate Side)	1	N/mm2	180	N/mm2	OK
			MF: Between Fulcrum (Flange Side)	1	N/mm2	173	N/mm2	OK
			ES: On Between Fulcrum (Skin Plate Side)	1	N/mm2	180	N/mm2	OK
			EF: On Fulcrum (Flange Side)	2	N/mm2	180	N/mm2	OK

Source: Study Team

Table 7.3.101 Calculation Result of No.1 Gate (2)

1-1. Gate Leaf

Target for Calculation	Case	Calculation Item		Calculation Results		Tolerance		Evaluation
Diaphragm	Fully Closed	Bending Stress		14	N/mm2	180	N/mm2	OK
		Shear Stress		27	N/mm2	104	N/mm2	OK
	During Operation	Bending Stress		2	N/mm2	180	N/mm2	OK
		Shear Stress		4	N/mm2	104	N/mm2	OK
Evaluation of Total of Shell Stress Intensity and Coaxial Stress	Fully Closed	Stress Intensity (On Diaphragm)		100	N/mm2	105	N/mm2	OK
		Stress Intensity (Diaphragm Middle)		85	N/mm2	105	N/mm2	OK
	During Operation	Stress Intensity (On Diaphragm)		17	N/mm2	105	N/mm2	OK
		Stress Intensity (Diaphragm Middle)		16	N/mm2	105	N/mm2	OK
Evaluation of Combined Stress Intensity of Diaphragm and Shell	Fully Closed	Composite Stress Intensity (Intermediate Plate Portion)		81.0	N/mm2	116	N/mm2	OK
		Composite Stress Intensity (Center Part)		103	N/mm2	116	N/mm2	OK
		Composite Stress Intensity (Bottom)		83.2	N/mm2	116	N/mm2	OK
	During Operation	Composite Stress Intensity (Intermediate Plate Portion)		8.1	N/mm2	116	N/mm2	OK
		Composite Stress Intensity (Center Part)		14.5	N/mm2	116	N/mm2	OK
		Composite Stress Intensity (Bottom)		14.8	N/mm2	116	N/mm2	OK
Main Roller	Fully Closed	Contact Stress Intensity		974	N/mm2	1030	N/mm2	OK
Main Roller Shaft	Fully Closed	Bending Stress		165	N/mm2	170	N/mm2	OK
		Shear Stress		63.2	N/mm2	100	N/mm2	OK
		Bearing Surface Pressure		31.7	N/mm2	45.0	N/mm2	OK
Rocker beam	Fully Closed	Point A	Bending Stress	51.2	N/mm2	200	N/mm2	OK
			Shear Stress	25.9	N/mm2	115	N/mm2	OK
		Point B	Bending Stress	113	N/mm2	200	N/mm2	OK
			Shear Stress	49.7	N/mm2	115	N/mm2	OK
		Point C	Shear Stress	94.9	N/mm2	115	N/mm2	OK
		Point A	Composite Stress Intensity	0.12		1.20		OK
		Point B	Composite Stress Intensity	0.51		1.20		OK
		Axis Point A	Bending Stress	98.0	N/mm2	170	N/mm2	OK
			Shear Stress	23.4	N/mm2	100	N/mm2	OK
		Axis Point B	Bending Stress	162	N/mm2	170	N/mm2	OK
			Shear Stress	17.3	N/mm2	100	N/mm2	OK
		Bearing Surface Pressure		19.3	N/mm2	45.0	N/mm2	OK

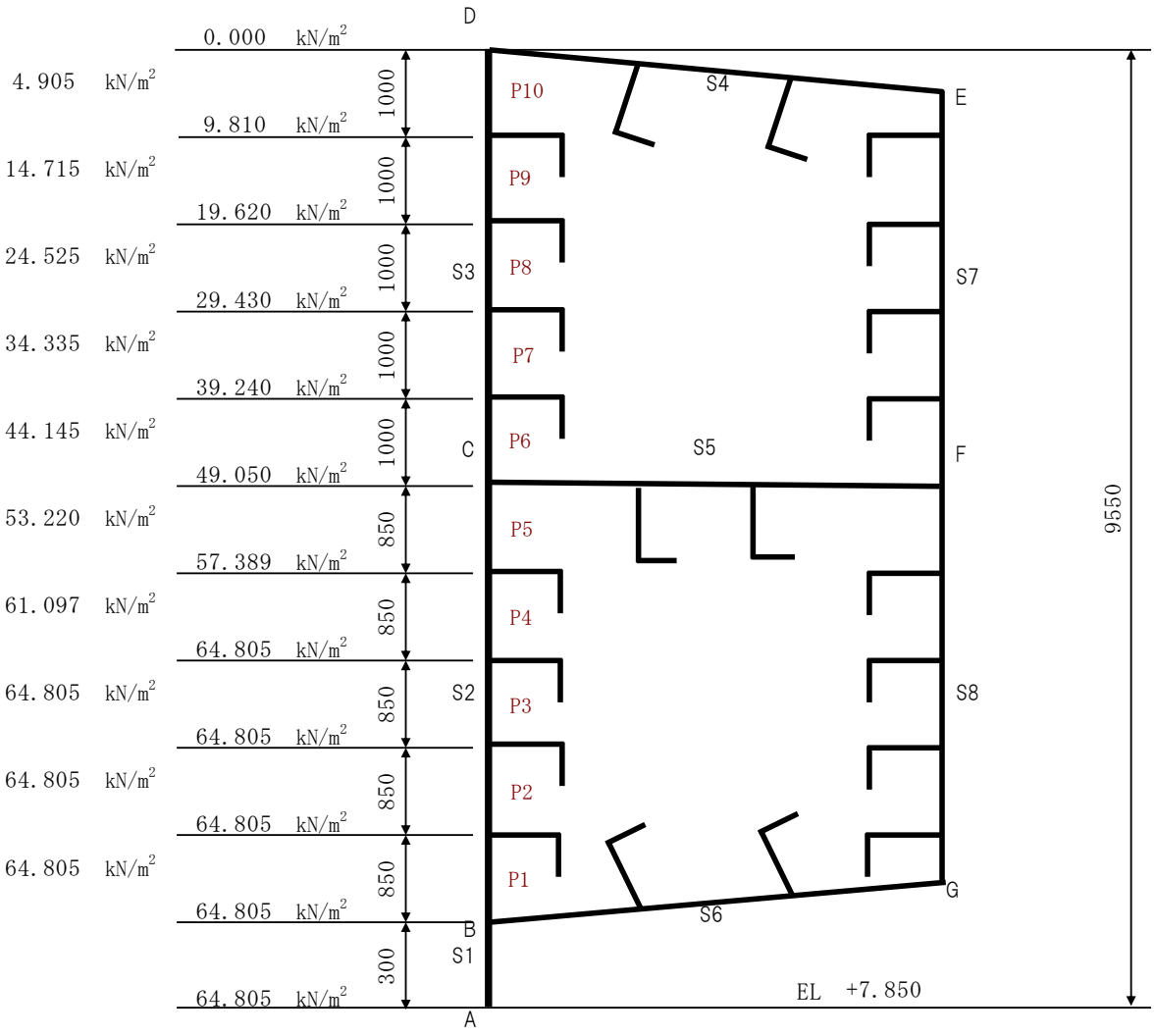
Source: Study Team

Table 7.3.102 Calculation Result of No.1 Gate (Guide Frame)

1-2. Guide Frame

Target for Calculation	Case	Calculation Item		Calculation Results		Tolerance		Evaluation
Roller Rail	Fully Closed	Concrete Bearing Stress Intensity (Maximum)		2.05	N/mm2	5.90	N/mm2	OK
		Concrete Shear Stress Intensity (Maximum)		0.35	N/mm2	0.40	N/mm2	OK
		Roller Rail Bending Stress (Maximum)		115	N/mm2	150	N/mm2	OK
		Roller Thread Thickness		37.2	mm	40.0	mm	OK
		Local Stress at the Web Plate of Roller Rail		195	N/mm2	226	N/mm2	OK
		Bending Stress at Bottom Flange of Roller Rail		68.3	N/mm2	150	N/mm2	OK
		Composite Stress Intensity of Bottom flange		160	N/mm2	165	N/mm2	OK
		K by Adjacent Rollers	K1	1.43	N/mm2	5.90	N/mm2	OK
			K2	2.05	N/mm2	5.90	N/mm2	OK
		Bearing Stress due to Adjacent Rollers		45.2	N/mm2	150	N/mm2	OK
		Compressive Stress by Adjacent Rollers		2.81	N/mm2	5.90	N/mm2	OK

Source: Study Team



Source: Study Team

Figure7.3.93 No. 1 Gate Cross-Sectional Diagram

Table 7.3.103 Calculation Result of No. 2 Gate

1-1. Gate Leaf

			Calculation Results	Tolerance	Evaluation
Upper Girder	Maximum Bending Stress		84.7 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		8.79 N/mm2	104 N/mm2	OK
	Deflection		1/841	1/800	OK
Main Girder	Maximum Bending Stress		118.8 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		30.8 N/mm2	104 N/mm2	OK
	Deflection		1/836	1/800	OK
Lower Girder	Maximum Bending Stress		121.6 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		24.5 N/mm2	104 N/mm2	OK
	Deflection		1/819	1/800	OK
Skin Plate	Bending Stress	Parcel: (1)	106.0 N/mm2	180 N/mm2	OK
		Parcels: (2)	161 N/mm2	180 N/mm2	OK
		Parcels: (3)	134 N/mm2	180 N/mm2	OK
		Parcels: (4)	142 N/mm2	180 N/mm2	OK
Stringer	Bending Stress	Parcel: (1)	159.45 N/mm2	200 N/mm2	OK
		Parcels: (2)	102.93 N/mm2	200 N/mm2	OK
		Parcels: (3)	81.11 N/mm2	200 N/mm2	OK
		Parcels: (4)	86.03 N/mm2	200 N/mm2	OK
	Shear Stress	Parcel: (1)	11.16 N/mm2	115 N/mm2	OK
		Parcels: (2)	12.81 N/mm2	115 N/mm2	OK
		Parcels: (3)	12.20 N/mm2	115 N/mm2	OK
		Parcels: (4)	12.94 N/mm2	115 N/mm2	OK
End stringer (Portion where roller shaft does not penetrate)	Maximum Bending Stress	Compression Side	150.7 N/mm2	180 N/mm2	OK
		Tension side	150.7 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		43.7 N/mm2	104 N/mm2	OK
	Composite Stress Intensity		168.6 N/mm2	198 N/mm2	OK
End stringer (Roller Shaft Mounting Part)	Maximum Bending Stress	Compression Side	139.6 N/mm2	200 N/mm2	OK
		Tension side	139.6 N/mm2	200 N/mm2	OK
	Maximum Shear Stress		76.3 N/mm2	115 N/mm2	OK
	Composite Stress Intensity		192 N/mm2	198 N/mm2	OK
Main Roller	Contact Stress Intensity		945 N/mm2	1030 N/mm2	OK
Main Roller Shaft (Point A)	Maximum Bending Stress		158 N/mm2	170 N/mm2	OK
	Maximum Shear Stress		26.7 N/mm2	100 N/mm2	OK
	Composite Stress Intensity		165 N/mm2	187 N/mm2	OK
Main Roller Shaft (Point B)	Maximum Bending Stress		159 N/mm2	170 N/mm2	OK
	Maximum Shear Stress		29.1 N/mm2	100 N/mm2	OK
	Composite Stress Intensity		167 N/mm2	187 N/mm2	OK
Bearing	Surface Pressure		18.1 N/mm2	23.0 N/mm2	OK

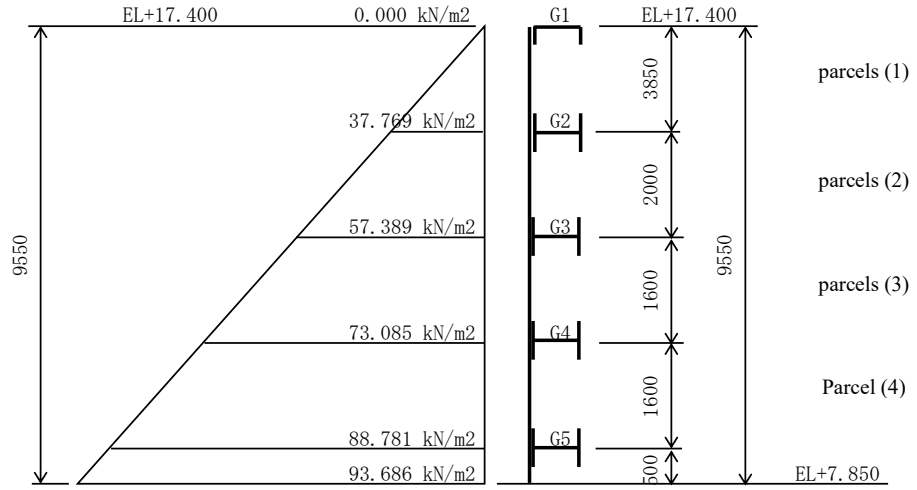
Source: Study Team

Table 7.3.104 Calculation Result of No. 2 Gate (Guide Frame)

1-2. Guide Frame

			Calculation Results	Tolerance	Evaluation
Roller Rail	Concrete Bearing Stress		1.54 N/mm2	5.90 N/mm2	OK
	Concrete Shear Stress		0.27 N/mm2	0.40 N/mm2	OK
	Bending Stress in Roller Rail		92.0 N/mm2	160 N/mm2	OK
	Roller Thread Thickness		31.8 mm	35.0 mm	OK
	Local Stress at the Bottom Flange of Roller Rail		221 N/mm2	240 N/mm2	OK
	Bending Stress at Bottom Flange of Roller Rail		52.6 N/mm2	160 N/mm2	OK
	Composite Stress Intensity of Bottom Flange		127 N/mm2	176 N/mm2	OK
	K by Adjacent Rollers	K1	1.53 N/mm2	5.90 N/mm2	OK
		K2	1.54 N/mm2	5.90 N/mm2	OK
	Bearing Stress due to Adjacent Rollers		0.81 N/mm2	160 N/mm2	OK
	Compressive Stress by Adjacent Rollers		0.86 N/mm2	5.90 N/mm2	OK

Source: Study Team



Source: Study Team

Figure 7.3.94 No. 2 Gate Cross-Sectional Diagram

(2) Stoplog

Table 7.3.105 Calculation Result of The Upstream Stoplog

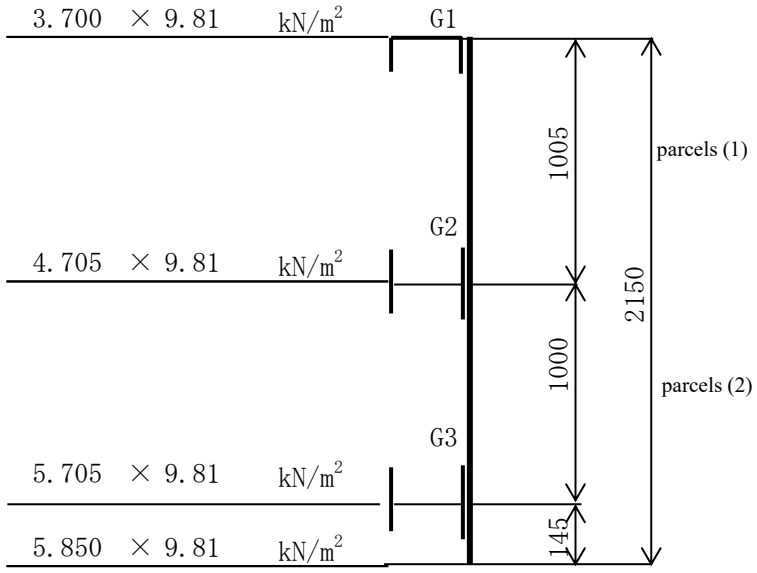
1-1. Gate Leaf

			Calculation Results	Tolerance	Evaluation
Lower Main girder (G1)	Maximum Bending Stress		97.0 N/mm2	165 N/mm2	OK
	Maximum Shear Stress		13.8 N/mm2	105 N/mm2	OK
	Deflection		1/852	1/600	OK
Lower Main girder (G2)	Maximum Bending Stress		136.0 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		32.1 N/mm2	105 N/mm2	OK
	Deflection		1/606	1/600	OK
Lower Main girder (G3)	Maximum Bending Stress		126.0 N/mm2	176 N/mm2	OK
	Maximum Shear Stress		24.0 N/mm2	105 N/mm2	OK
	Deflection		1/651	1/600	OK
Top Main girder (G1)	Maximum Bending Stress		17.0 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		4.1 N/mm2	105 N/mm2	OK
	Deflection		1/4782	1/600	OK
Top Main girder (G2)	Maximum Bending Stress		22.0 N/mm2	176 N/mm2	OK
	Maximum Shear Stress		4.1 N/mm2	105 N/mm2	OK
	Deflection		1/3825	1/600	OK
Lower Skin Plate	Bending Stress	Parcel: (1)	114.0 N/mm2	180 N/mm2	OK
		Parcels: (2)	140.0 N/mm2	180 N/mm2	OK
Top Skin Plate	Bending Stress	Parcel: (1)	8.0 N/mm2	180 N/mm2	OK
		Parcels: (2)	12.0 N/mm2	180 N/mm2	OK
Lower stringer	Bending Stress	Parcel: (1)	15.6 N/mm2	111 N/mm2	OK
		Parcels: (2)	19.1 N/mm2	111 N/mm2	OK
	Shear Stress	Parcel: (1)	3.3 N/mm2	105 N/mm2	OK
		Parcels: (2)	4.1 N/mm2	105 N/mm2	OK
Top stringer	Bending Stress	Parcels: (3)	0.9 N/mm2	123 N/mm2	OK
		Parcels: (4)	0.7 N/mm2	141 N/mm2	OK
	Shear Stress	Parcels: (3)	0.2 N/mm2	105 N/mm2	OK
		Parcels: (4)	0.2 N/mm2	105 N/mm2	OK
Lower bearing plate	Bearing Stress		9.9 N/mm2	225 N/mm2	OK
Upper bearing plate	Bearing Stress		1.2 N/mm2	225 N/mm2	OK

1-2. Guide Frame

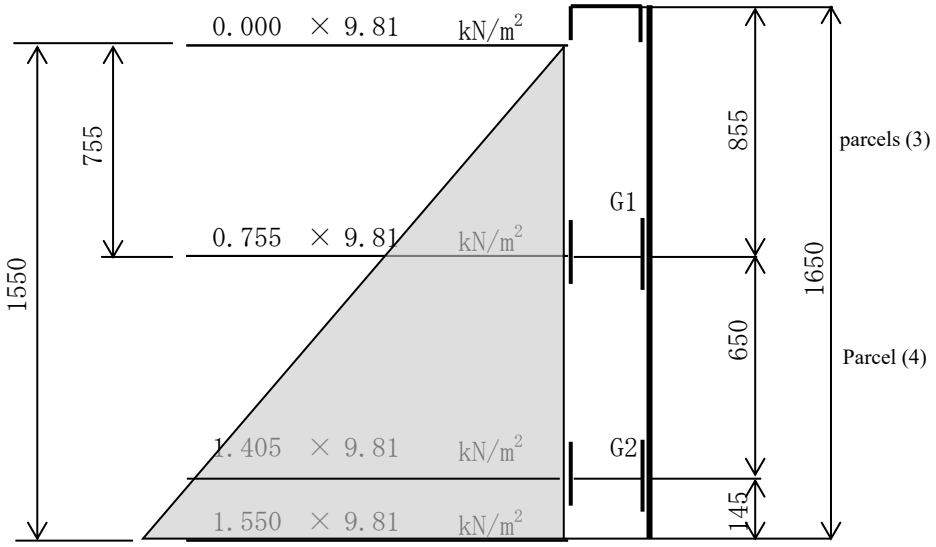
		Calculation Results	Tolerance	Evaluation
Strut	Concrete Bearing Stress	2.40 N/mm2	5.90 N/mm2	OK
	Concrete Shear Stress	0.28 N/mm2	0.40 N/mm2	OK
	Bending Stress on Strut	75.8 N/mm2	152 N/mm2	OK
	Shear Stress on Strut	62.4 N/mm2	105 N/mm2	OK
	Deflection of Strut	1/1618	1/600	OK
	Diagonal Axial Tensile Stress	75.9 N/mm2	180 N/mm2	OK
	Fixed Anchor Shear Stress	77.7 N/mm2	90 N/mm2	OK
	Tensile Stress of anchor in fixed part	120.0 N/mm2	150 N/mm2	OK

Source: Study Team



Source: Study Team

Figure 7.3.95 Sectional Shape Diagram (Lower Part)



Source: Study Team

Figure 7.3.96 Sectional Shape Diagram (Upper Part)

Table 7.3.106 Calculation Result of the Downstream Stoplog

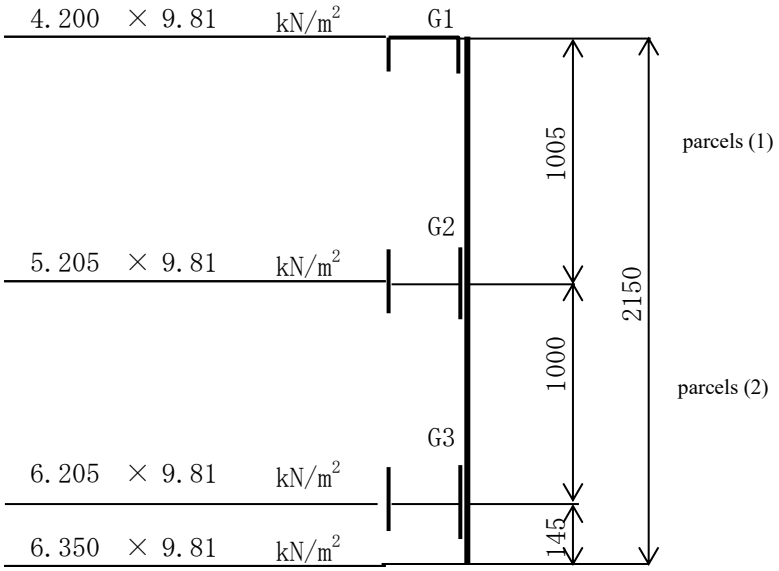
1-1. Gate Leaf

			Calculation Results	Tolerance	Evaluation
Main girder (G1)	Maximum Bending Stress		108.0 N/mm2	165 N/mm2	OK
	Maximum Shear Stress		15.5 N/mm2	105 N/mm2	OK
	Deflection		1/758	1/600	OK
Main girder (G2)	Maximum Bending Stress		122.0 N/mm2	180 N/mm2	OK
	Maximum Shear Stress		36.2 N/mm2	105 N/mm2	OK
	Deflection		1/677	1/600	OK
Main girder (G3)	Maximum Bending Stress		113.0 N/mm2	176 N/mm2	OK
	Maximum Shear Stress		26.7 N/mm2	105 N/mm2	OK
	Deflection		1/726	1/600	OK
Skin Plate	Bending Stress	Parcel: (1)	127.0 N/mm2	180 N/mm2	OK
		Parcels: (2)	153.0 N/mm2	180 N/mm2	OK
Stringer	Bending Stress	Parcel: (1)	17.5 N/mm2	111 N/mm2	OK
		Parcels: (2)	21.0 N/mm2	111 N/mm2	OK
	Shear Stress	Parcel: (1)	3.7 N/mm2	105 N/mm2	OK
		Parcels: (2)	4.5 N/mm2	105 N/mm2	OK
Bearing Plate	Bearing Stress		10.7 N/mm2	225 N/mm2	OK

1-2. Guide Frame

		Calculation Results	Tolerance	Evaluation
Strut	Concrete Bearing Stress	2.60 N/mm2	5.90 N/mm2	OK
	Concrete Shear Stress	0.31 N/mm2	0.40 N/mm2	OK
	Bending Stress on Strut	92.6 N/mm2	149 N/mm2	OK
	Shear Stress on Strut	74.2 N/mm2	105 N/mm2	OK
	Deflection of Strut	1/1190	1/600	OK
	Diagonal Axial Tensile Stress	79.8 N/mm2	114 N/mm2	OK
	Fixed Anchor Shear Stress	63.3 N/mm2	90 N/mm2	OK
	Tensile Stress of Anchor in Fixed Part	81.0 N/mm2	150 N/mm2	OK

Source: Study Team



Source: Study Team

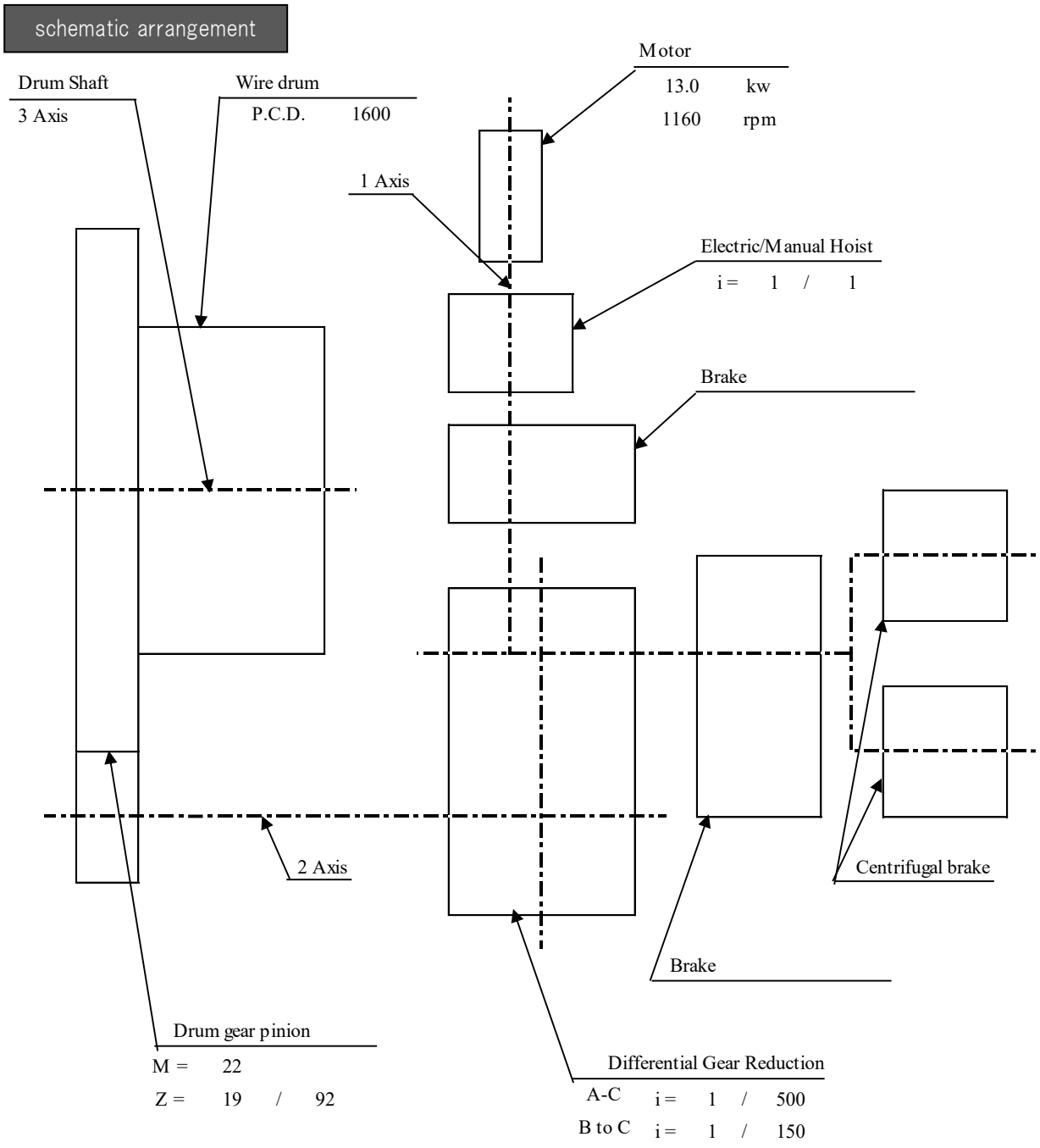
Figure 7.3.97 Cross-Sectional Diagram

(3) Hoist

Table 7.3.107 Calculation Result of No. 1 Gate Hoist

			Calculation Results	Tolerance	Evaluation
Motor	Motor Output		12.4 kw	13.0 kw	OK
Hydraulic push-up brake	Safety Factor (during electric operation)		1.94	1.50	OK
Wire Rope	Safety Factor (From the opening and closing load)		8.37	8.00	OK
	Tension from Motor Output (maximum torque)		658205 N	982800 N	OK
Wire Drum	Drum Diameter (ratio to the rope diameter)		28.6	19.0	OK
Wire Sheave	Sheave Diameter (ratio to the rope diameter)		17.9	17.0	OK
Wire Drum	Drum Plate Thickness		44.8 mm	47.0 mm	OK
Rope Presser	Allowable Tightening Force of the Tightening Bolt		80448 N	108744 N	OK
	Bending Stress of the Rope Pressing Metal		78.1 N/mm2	98.0 N/mm2	OK
Hydraulic push-up brake	Torque (Self Weight Lowering Device)		527 N. m	617 N. m	OK
Gear	Safety Factor at Rated Torque	Pinion	7.51	0.00	OK
		Drum gear	9.04	0.00	OK
	Safety Factor at Maximum Torque	Pinion	2.30	0.00	OK
		Drum gear	2.75	0.00	OK
	Surface Pressure Strength Safety Factor	Pinion/Drum Gear	1.03	1.00	OK
Drum Shaft	Safety Factor at Rated Torque	Bending Stress	8.62	5.00	OK
		Shear Stress	63.1	8.70	OK
	Maximum Torque Stress	Bending Stress	165 N/mm2	311 N/mm2	OK
		Shear Stress	22.6 N/mm2	179 N/mm2	OK
	Bearing C	Surface Pressure at Rated Torque	5.25 N/mm2	6.00 N/mm2	OK
		Surface Pressure at Maximum Torque	13.1 N/mm2	28.0 N/mm2	OK
	Bearing D	Surface Pressure at Rated Torque	4.57 N/mm2	6.00 N/mm2	OK
		Surface Pressure at Maximum Torque	11.4 N/mm2	28.0 N/mm2	OK
Fleet Angle	Fully Closed		2.94 °	4.00 °	OK
	Upper Limit		0.94 °	4.00 °	OK

Source: Study Team



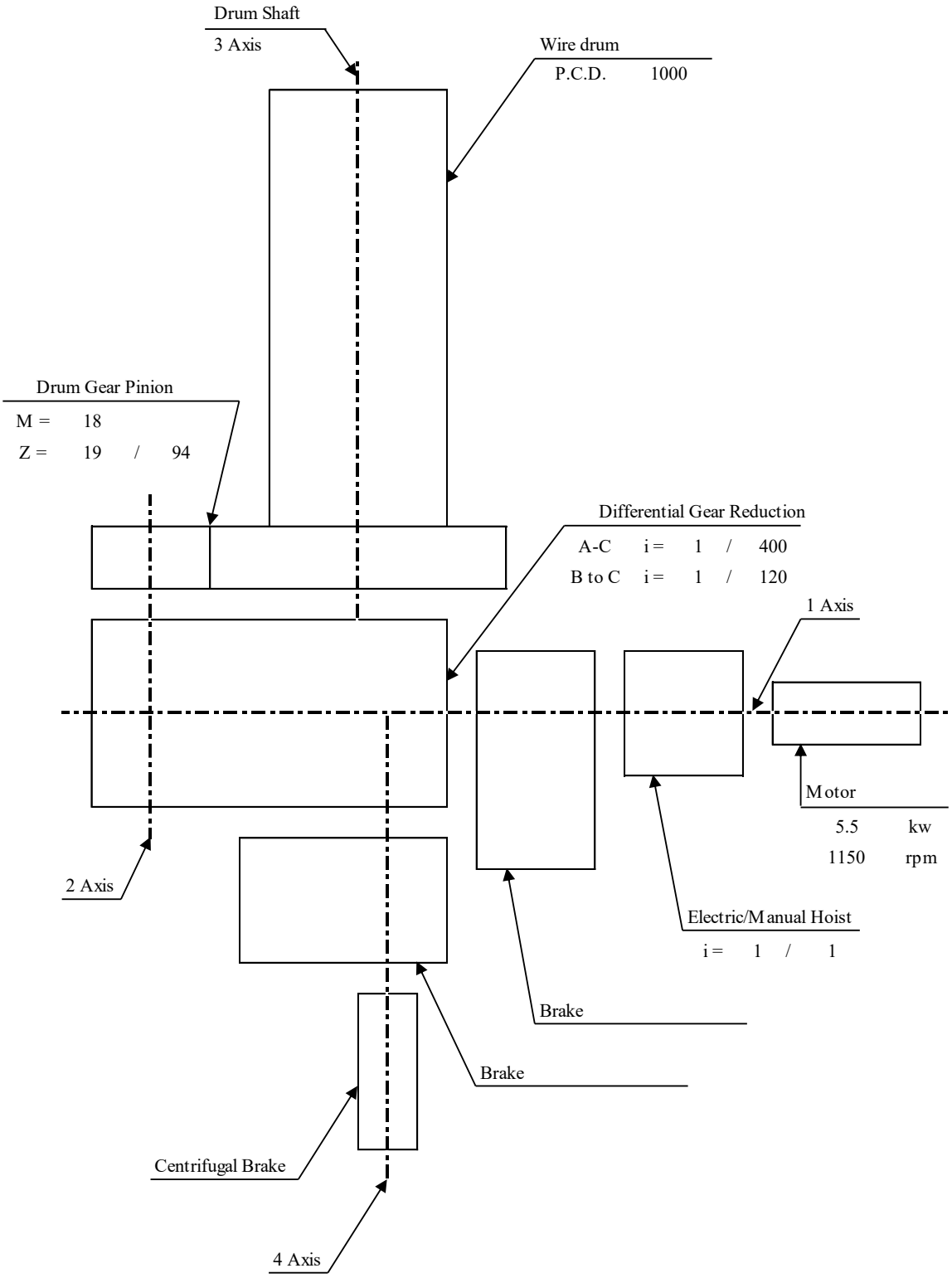
Source: Study Team

Figure 7.3.98 Schematic Arrangement of No. 2 Gate Hoist

Table 7.3.108 Calculation Result of No. 2 Gate Hoist

			Calculation Results	Tolerance	Evaluation
Motor	Motor Output		5.18 kw	5.50 kw	OK
Hydraulic Push-up Brake	Safety Factor (during electric operation)		2.14	1.50	OK
Wire rope	Safety Factor (From the opening and closing load)		8.61	8.00	OK
	Tension from Motor Output (maximum torque)		181971 N	302445 N	OK
Wire drum	Drum Diameter (Ratio to the rope diameter)		31.7	19.0	OK
Wire sheave	Sheave Diameter (Ratio to the rope diameter)		18.1	17.0	OK
Wire drum	Drum Plate Thickness		21.20 mm	25.00 mm	OK
Rope presser	Allowable Tightening Force of the Tightening Bolt		22241 N	35152 N	OK
	Bending Stress of the Rope Pressing Metal		47.9 N/mm2	98.0 N/mm2	OK
Hydraulic Push-up Brake	Torque (Self Weight Lowering Device)		225 N. m	294 N. m	OK
Gear	Safety Factor at Rated Torque	Pinion	13.0	5.00	OK
		Drum Gear	15.6	5.00	OK
	Safety Factor at Maximum Torque	Pinion	3.99	1.00	OK
		Drum Gear	4.73	1.00	OK
	Surface Pressure Strength	Pinion/Drum Gear	1.23	1.00	OK
Fleet Angle	Fully Closed		2.11 °	4.00 °	OK
	Upper Limit		1.07 °	4.00 °	OK

Source: Study Team



Source: Study Team

Figure 7.3.99 Schematic Arrangement of No. 2 Gate Hoist

7.3.3.3 Control Room Layout

(1) Components of Control Room

In the control room, a device related to the hoist of the No. 1 gate and a device related to the hoist of the No. 2 gate are arranged. Since the No. 1 gate is the 2M2D type, the hoisting device is installed at both the end pier and the central pier. **Table 7.3.109** describes the components installed in the end and central control room.

Table 7.3.109 Control Room Components

End Control Room (No. 1 Gate Side)	Central Control Room	End Control Room (No. 2 Gate Side)
No. 1 Gate Hoist	No. 1 Gate Hoist	No. 2 Gate Hoist
・wire drum ・wire sheave ・wire terminal device ・motor ・electric manual hoisting device ・centrifugal brake ・mew lifter brake ・gear reducer ・drum gear and pinion gear ・stop device ・limiting hoist ・position transmitter ・local operation panel	・wire drum ・wire sheave ・wire terminal device ・motor ・electric manual hoisting device ・centrifugal brake ・mew lifter brake ・gear reducer ・stop device ・limiting hoist ・position transmitter ・emergency upper limit detection device	・wire drum ・wire sheave ・motor ・electric manual hoisting device ・centrifugal brake ・mew lifter brake ・gear reducer ・drum gear and pinion gear ・stop device ・limiting hoist ・position transmitter ・local operation panel
	No. 2 Gate Hoist	
	・wire sheave ・wire terminal device ・stop device	

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 **Figure 7.3.101** to **Figure 7.3.103**.

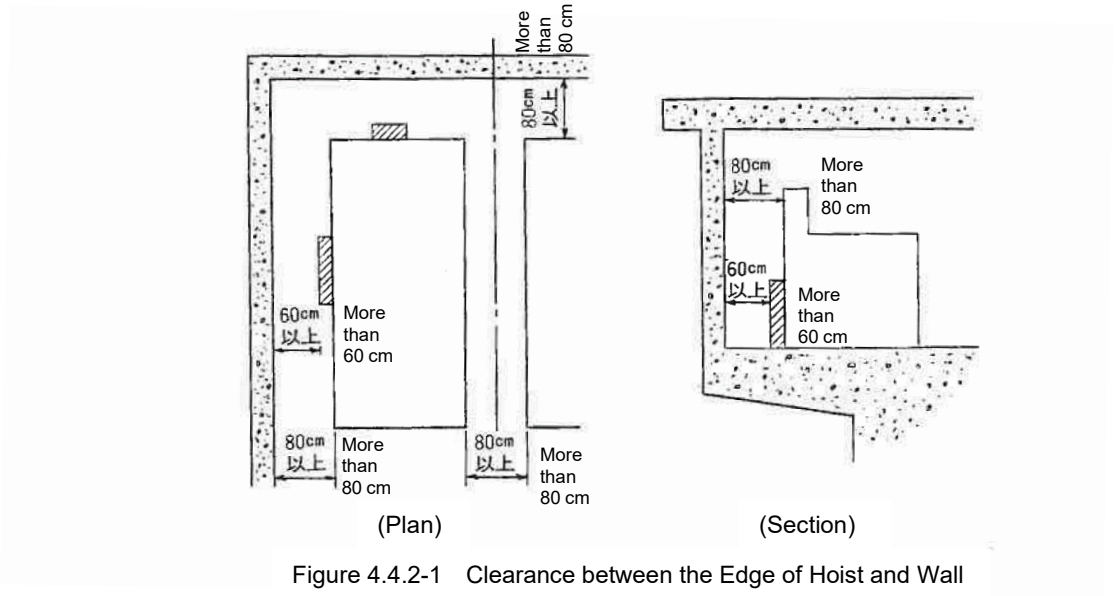
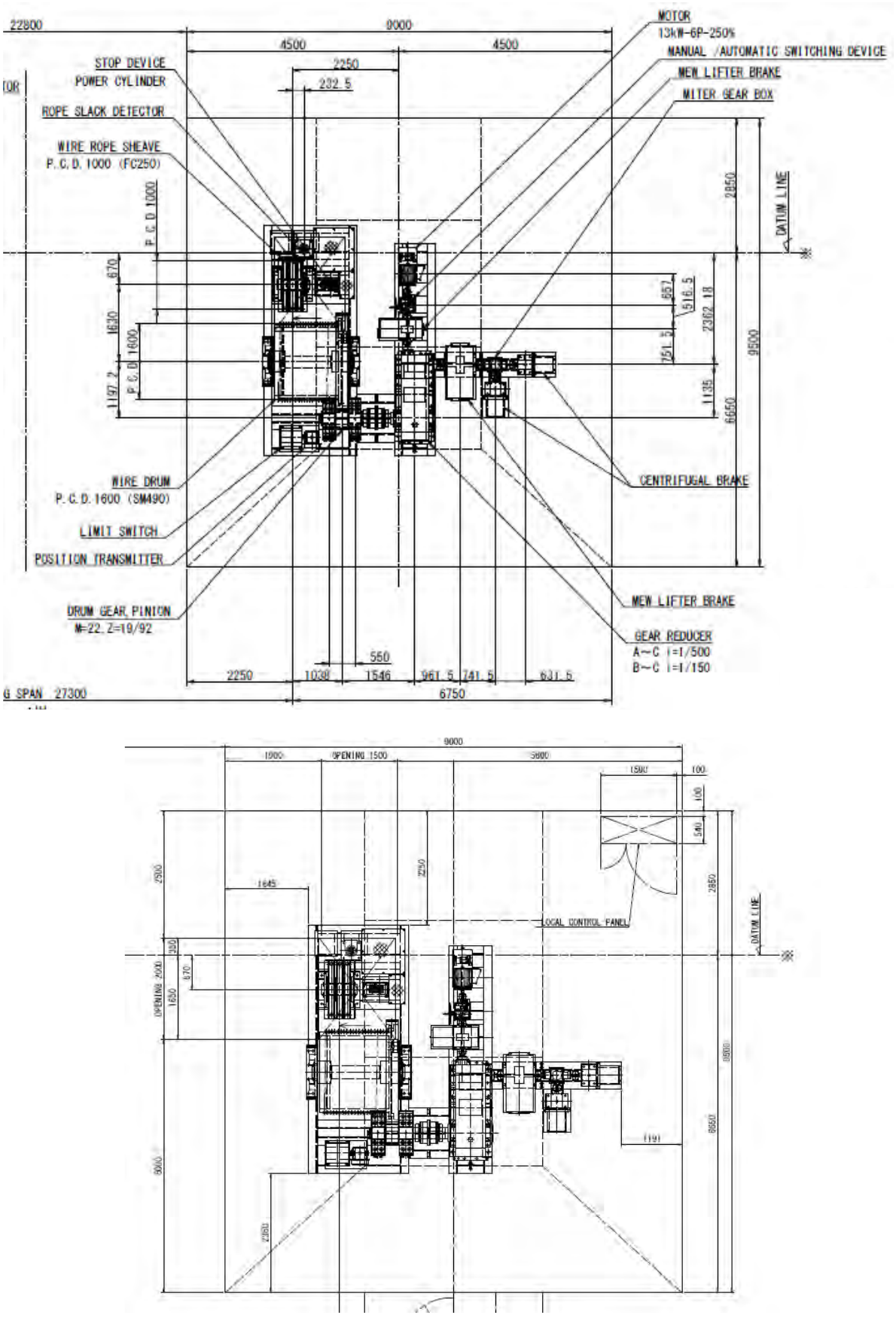


Figure 4.4.2-1 Clearance between the Edge of Hoist and Wall

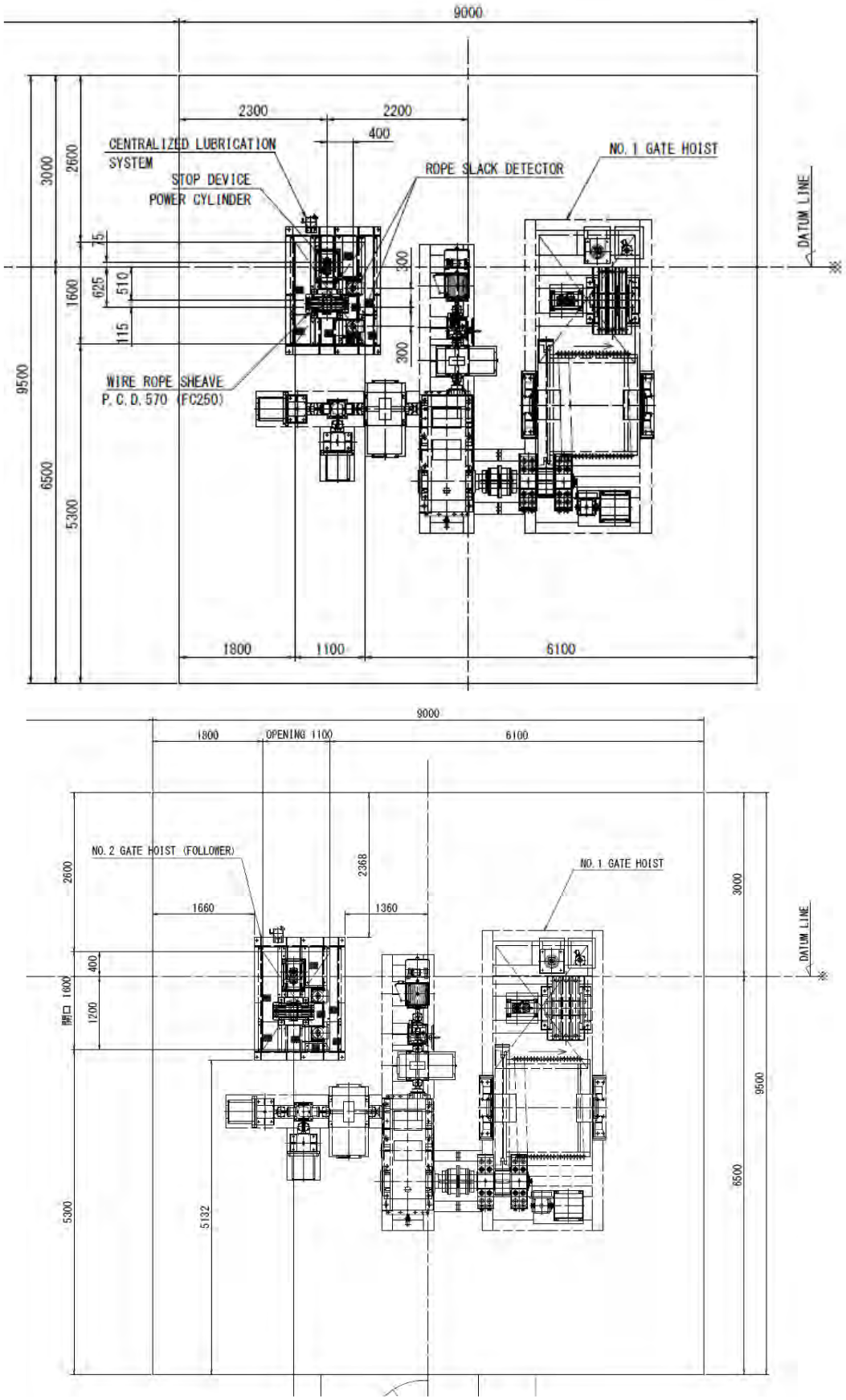
Note: Translated by JICA Study Team from Technical Specification for Dams and Weirs in Japan (Draft) [Reference Commentary and Manual]

Figure 7.3.100 Space to be Secured in Operating Room Space



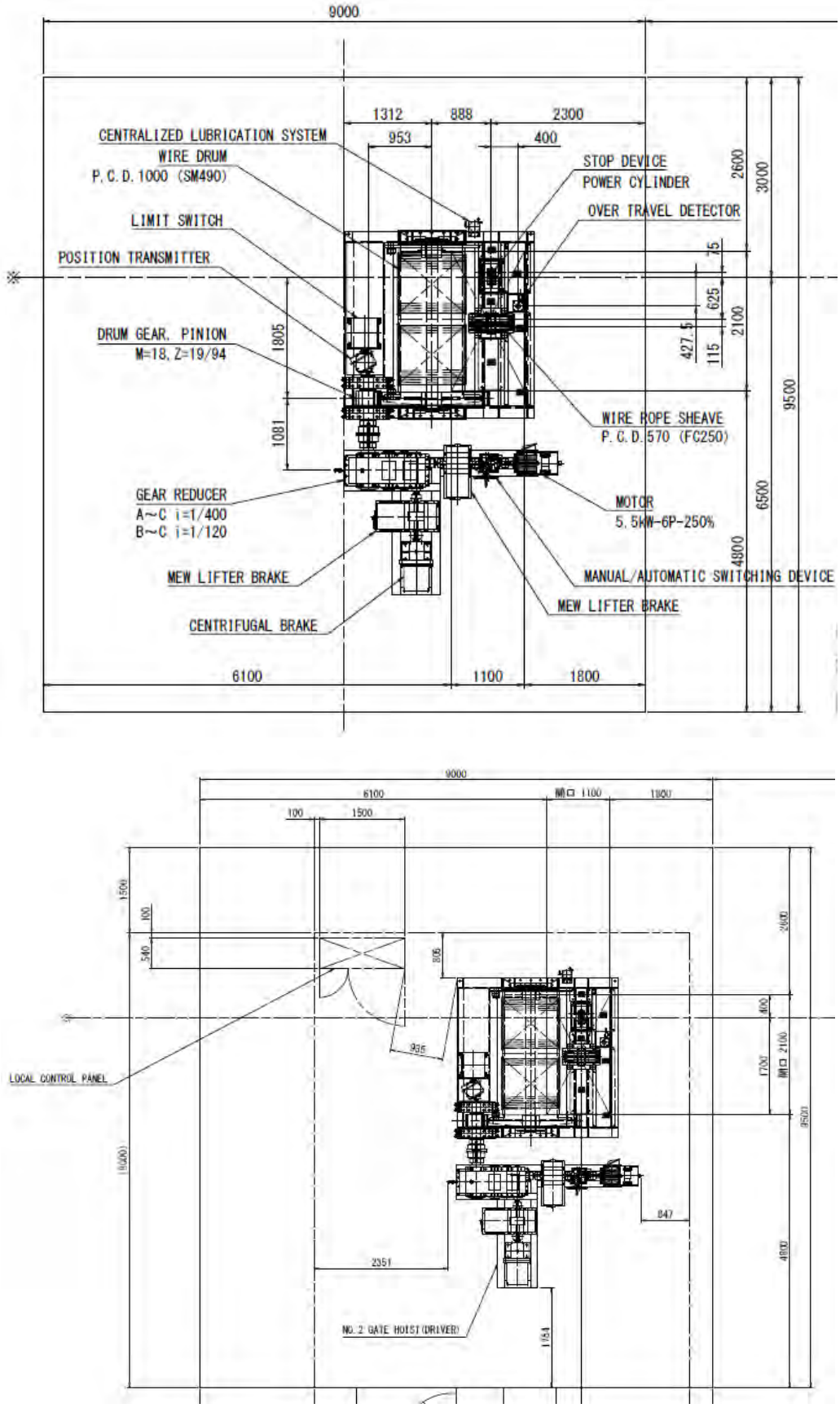
Source: Study Team

Figure 7.3.101 Layout of the End Control Room (No. 1 Gate)



Source: Study Team

Figure 7.3.102 Layout of the Central Control Room



Source: Study Team

Figure 7.3.103 Layout of the End Control Room (No. 2 Gate)

7.3.3.4 Specifications of the Gate Equipment

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

(1) Gate Facilities (Gate and Guide Frame)

1) No.1 Gate

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

Gate Type	Plate Girder Structure Two-Phase Stainless Roller Gate		
Clear Span x Effective Height	Clear Span 11.70 m x Effective Height 9.55 m		
Number of Gate	1 Gate		
Design Depth	(River Side) (Seaward)	EL + 17.400 EL + 10.794	(Design High Water Level) (Normal Water Level)
Operating Depth (opening time)	(River Side) (Seaward)	EL + 12.486 EL + 11.486	(Start water level of -1 m) (Diverting Start Water Level)
Operating Depth (closing time)	(River Side) (Seaward)	EL + 11.486 EL + 11.486	(Diverting Start Water Level) (Diverting Start Water Level)
Bed Height	(Plan)	EL + 7.850	
Water sealing System	3 Way Front Rubber Watertight		
Operation Method	Machine side operation and remote control		
Compliance 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)

1) No.1 Gate

Hoist Type	2M2D Wire Rope Winch	
Rated Opening Capacity	2650 KN,	
Number of Installations	One	
Additional Function	Self weight lowering function Rest hook	Yes Yes
Normal Lift	Normal H1 Dogging H2	11.150 m 11.450 m
Opening and Closing Speed	When using an electric motor During self-weight Lowering	ca.0.30 m/min ca.1.00 m/min
Wire Rope	JIS 6 × 37 G Type Plating	
Power	200 VAC - 50 Hz	
Compliance 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) No. 2 Gate

Hoist TYPE	1M1D Wire Rope Winch Type	
Rated Opening Capacity	610 KN,	
Number of Installations	One	
Additional Function	Self weight lowering function	Yes
	Rest hook	Yes
Normal Lift	Normal H1	11.150 m
	Dogging H2	11.450 m
Opening and closing speed	When using an electric motor	ca.0.30 m/min
	During self-weight descent	ca.1.00 m/min
Wire Rope	JIS 6 × 37 G Type plating	
Power	220 VAC - 60 Hz	
Compliance 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	Ingate closing Self-Standing Type Steel Plate
Number of Installations	1 Face
Outline Dimensions	Width: 1.000 m x Height: 2.000 m x Depth: 0.500 m
Compliance Standard	Technical Specification for Dams and Weirs in Japan (Draft) Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,

(4) Stoplog Facility (Upstream)

Format	Plate Girder Structure Steel Slide (Strut Type Angle Drop)	
Clear Span (Gate leaf Width)	Clear Span 28.30 m (4.15 m x 2 leaves + 4.00 m x 5 leaves)	
	Clear Span 11.70 m (3.85 m x 2 leaves + 4.00 m x 1 leaves)	
Effective Height (Gate Height)	Effective height: 5.95 m (2.15 m x 2 leaves + 1.65 m x 1 leaves)	
Number of Gate	9 leaves (One gate of No. 2 gate)	
	21 leaves (One gate at No. 1)	
Design Depth	(Front)	EL + 13.400 (HWL)
	(Posterior Surface)	EL + 7.550 (Bed Height)
Bed Height	(Plan)	EL + 7.550
Water sealing System	Rear three-way rubber watertight	
Opening and Closing Method	Suspension system by crane	
Compliance Standard	Technical Specification for Dams and Weirs in Japan (Draft) Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,	

(5) Stoplog Facility (Downstream)

Format	Plate Girder Structure Steel Slide (Strut Type Angle Drop)	
Clear Span (Gate leaf Width)	Clear Span 28.30 m (4.15 m x 2 leaves + 4.00 m x 5 leaves)	
	Clear Span 11.70 m (3.85 m x 2 leaves + 4.00 m x 1 leaves)	
Effective Height (Gate Height)	Effective height: 6.45 m (2.15 m x 3 leaves)	
Number of Gate	9 leaves (One gate of No. 2 gate)	
	21 leaves (One gate at No. 1)	
Design Depth	(Front)	EL + 13.400 (HWL)
	(Posterior Surface)	EL + 7.050 (Bed height)
Bed Height	(Plan)	EL + 7.050
Water sealing System	Rear three-way rubber watertight	
Opening and Closing Method	Suspension system by crane	
Compliance Standard	Technical Specification for Dams and Weirs in Japan (Draft) Design Guideline for Floodgate and Sluiceway Gate (Draft). Japan,	

7.3.4 Design of Building Facilities

The detailed design of the three building facilities, machine facilities, and electric facilities, . is described in the Section 7.6.

7.3.5 Design of Temporary Facilities

7.3.5.1 Water Level Setting for Coffering

The water level at the time of coffering work of MCGS was set from the highest water level in past 5 years (2014 – 2018 years). Calculation conditions are summarized in **Table 7.3.110**.

Table 7.3.110 Conditions for Calculating the Water Level at the Time of Coffering Work

River Name	Marikina
Applicable Section	Sta. 0 + 000 to Sta. 6 + 650 (NHCS – Rosario Weir)
Calculation Method	Unequal current calculation
Cross Section Used for Calculation	2015 IV & V Existing Section + Phase III Dredging Reflected
Roughness Coefficient	Sta. 0 + 000 to Sta. 6+650:0.028
Flow Condition	437 m3/s (From the hydraulic model experiment)
Starting Water Level	EL. + 12.60 m (Napindan JS 5-year high water level)

Source: Study Team

The calculation method and results are as follows. The water level for cofferdam is calculated as EL. + 14.0 m.

1. The highest water level at Rosario JS 5 year was extracted: EL. + 15.83 m (2014/9/19 12:00:00)
2. H-Q Curve was calculated from the observed water level EL. + 19.61 m at Sto.Nino: 2,147 m3/s
3. Diversion quantity to lower Marikina River was estimated by a hydraulic model experiment: 437 m3/s
4. Water level at MCGS was calculated under the Condition shown in **Table 7.3.110**: EL. + 13.94 m
5. Round the above result and set the water level during coffering: EL. + 14.0 m

On the other hand, based on the verification result of the hydraulic model experiment, the cofferdam target water level is EL. + 14.2 in the construction of the left bank side (water flows in the right bank side).

For reference, the several water levels (2, 5, 10 y -return period) at the MCGS site were calculated and compared with the water level during coffering. Conditions and results of calculation of water levels are shown in **Table 7.3.111** and **Table 7.3.112** respectively. The difference in water level between the 2-year return period and the 10-year return period was about 60 cm, and there was no significant difference.

Table 7.3.111 Conditions for Calculating the Probable Water Level

River Name	Marikina	
Applicable section	Sta. 0 + 000 to Sta. 6 + 650 (NHCS – Rosario Weir)	
Calculation Method	Unequal current calculation	
Cross Section Used for Calculation	2015 IV & V Existing Section + Phase III Dredging Reflected	
Roughness Coefficient	Sta. 0 + 000 to Sta. 6+650:0.028	
Flow Condition	2-year probability: 343 m3/s 5-year probability: 427 m3/s 10-year probability: 501 m3/s	(Sto. From the stochastic discharge at the Nino site and the fraction ratio of hydraulic model experiments.)
Starting Water Level	EL. + 12.60 m (Napindan JS 5-year high water level)	

Source: Study Team

Table 7.3.112 Comparison of Discharge and Water Levels for the Cofferdam

	Discharge (m3/s)	Water Level (EL.m)
2-year	343	13.565
5-year	427	13.901
10 year	501	14.193
Highest in past 5 years (2014 -2018)	437	13.941

Source: Study Team

The calculated water level corresponding to the 10-year probable discharge of 501 m3/s is EL+14.193 m, and the target water level for the cofferdam is EL.+14.2, which satisfy the definition of "Natural Disaster" in FIDIC. Therefore, the validity of the water level for the cofferdam based on the maximum water level

for the past 5 years has been verified.

(1) Double Sheet-Pile Temporary Cofferdam

It is difficult to divert the entire Marikina River during the construction of MCGS. Therefore, it is decided to close half of the river. Since a cofferdam height becomes high, a double steel sheet pile (SSP) is used to construct a temporary cofferdam.

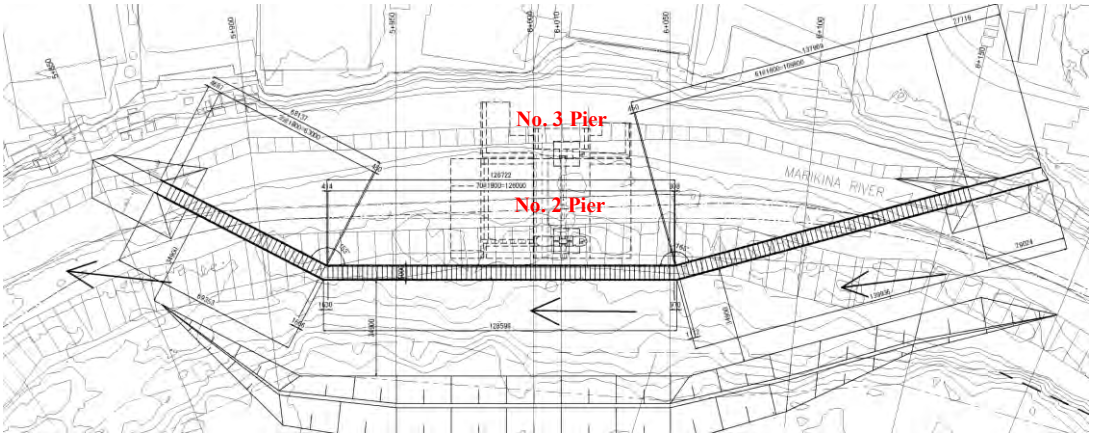
1) Layout Planning

The construction is divided into two periods. The double SSP temporary cofferdam will be arranged to make it possible to construct each part within the cofferdam. Particularly, upstream from the MCGS construction point shall be connected to the revetment at an angle of 45 degrees to the flow direction so as not to disturb the water flow. The connection with the exiting ground will be reinforced with sandbags, etc., depending on local conditions.

・Parts of the 1st stage construction: right bank pier of Marikina River (No. 3 Pier), central pier (No. 2 pier), central Bottom Slab, wing wall

・Parts of the 2nd stage construction: left bank pier of the Cainta River (No. 1 Pier), central Bottom Slab, wing wall

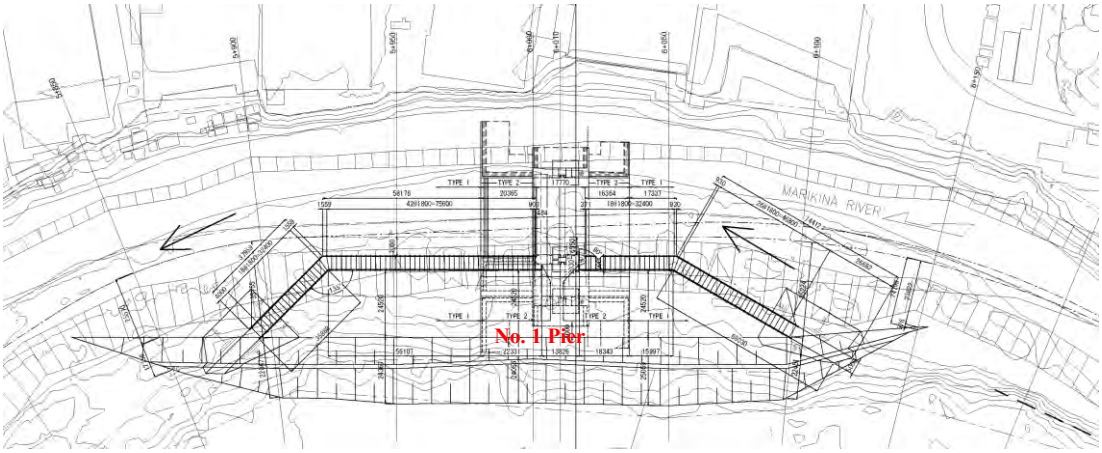
[1st Stage Construction]



Source: Study Team

Figure 7.3.104 Installation Location of Double SSP Temporary Cofferdam (1st Stage Construction)

[Second Stage Construction]



Source: Study Team

Figure 7.3.105 Installation Location of Double SSP Temporary Cofferdam (2nd Stage Construction)

2) Structural Design

(a) Calculation condition

(i) Standard

"Design manual of "Double sheet pile temporary cofferdam" National Land and Technology Research Center, May 2001, Japan.

(ii) Crown Height

The design height is assumed to be EL + 14.20 m according to the following condition.

・The maximum discharge of 440 m3/s for the past 5 years (2014/9/19 12: 00) was set by the hydraulic model experiment (Existing river channel before construction).

Freeboard is not provided because it does not require any levee excavation.

(iii) Design Ground Height

・1st stage construction : Excavated section +2.85

・2nd stage construction : Bottom Slab part +7.85 (design riverbed height)

(iv) Water Level

The design height shall be EL + 14.20 m.

(v) Soil Conditions

The soil constants arranged in the basic design are adopted. The soil constant of the filling soil is 19.0 kN/m^3 (unit weight) and 30 ° (internal friction angle ϕ).

Table 7.3.113 Unit Weight of the Filling Soil

Filling Soil type	Unit Weight of Soil			ϕ (Degrees)	Cohesion	
	Wer kN/m3	In Water kN/m3	Saturated kN/m3		Co kN/m2	Increment k kN/m3
Sandy Soil	19.0	9.0	19.0	30.0	0.0	0.0

Source: Study Team

Table 7.3.114 Soil Unit Weight Outside the Embankment and the Embankment (Ground Surface G.L. 7.850 m)

No	Layer Thickness m	Ground Type	Average N-Value	Unit Weight of Soil			ϕ (Degrees)	Cohesion		Deformation Coefficient αEo kN/m2
				Wet kN/m3	In Water kN/m3	Saturated kN/m3		Co kN/m2	Increment k kN/m3	
1	2.540	Sandy Soil	5.0	17.0	8.0	18.0	30.0	0.0	0.0	14000
2	20.000	Cohesive Soil	50.0	18.0	9.0	19.0	0.0	999.0	0.0	2000000

Source: Study Team

Table 7.3.115 Soil Unit Weight in the Embankment section (Ground Surface G.L. 2.850 m)

No	Layer Thickness m	Ground Type	Average N-Value	Unit Weight of Soil			ϕ (Degrees)	Cohesion		Deformation Coefficient αEo kN/m2
				Wet kN/m3	In Water kN/m3	Saturated kN/m3		Co kN/m2	Increment k kN/m3	
1	17.540	Cohesive Soil	50.0	18.0	9.0	19.0	0.0	999.0	0.0	200000

Source: Study Team

(vi) Overhead Load

・10.0 kN/m^2 (normal)

・5.0 kN/m^2 (seismic)

(vii) Seismic Intensity for Calculation of Inertia Force

The depth to calculate inertial force shall be as follows in accordance with " Design manual of Double sheet pile temporary cofferdam "(hereafter “the Cofferdam Manual”).

⑤ seismic intensity for calculation of inertia force

Seismic Inertial force shall be considered only in the horizontal direction, and seismic intensity shall be as follows:

In principle, it shall be as follows.

Horizontal Seismic Coefficient in air Kh = 0.1

apparent Horizontal Seismic Coefficient in water

$$Kh' = \Gamma sat / \Gamma sat - \Gamma w \, Kh$$

Γ sat: the unit volume weight (kN/m3) of the soil saturated with water in the air.

Γ w: Unit volumetric weight of water (kN/m3) $\gamma w = 10$ kN/m3

Refer to “Reference 1” if liquefaction is considered for the design seismic intensity .

Source: " Design manual of Double sheet pile temporary cofferdam " p. 16, 17

(viii) Allowable Stress

The allowable stress shall follow the " the Cofferdam Manual " **(Table 7.3.116)**. The value of allowable stress at the time of earthquake is 1.5 times from the normal (and flood).

Table 7.3.116 Allowable Stress of Main Steel Materials (Axial Tensile Stress, Bending Tensile Stress) (N/Mm^2)

Material name			Normal (High Water Time)	Seismic
SSP	SY 295		180	270
	SY 300		235	353
Steel Pipe Sheet Pile	SKY 400		140	210
	SKY 490		185	278
Tie Rod	SS 400	40 mm or less in diameter	94	141
		Greater than 40 mm in diameter	86	129
	SS 490	40 mm or less in diameter	110	165
		Greater than 40 mm in diameter	102	153
	HT 690		176	264
Tie Wire	SWRH (hard steel wire rod)		The allowable stress shall be determined so as to have a safety factor of 3.8 or more for the rupture strength at all times and 2.5 or more at the time of an earthquake.	
	SWRS (Piano wire rod)			
Antler	SS 400		140	210

Source: "steel sheet pile double temporary cofferdam design manual" p 30 -31

(ix) Safety Factor

The safety factor is as follows based on the " the Cofferdam Manual ".

Table 7.3.117 Safety Factor of the Stability Check

Check Item	Normal (High Water Time)	Seismic	Source
Shear Deformation Failure of Wall Body	1.2 or more	1.0 or more	Double cofferdam manual p 33
Sliding	1.2 or more	1.0 or more	Double cofferdam manual p 45
Bearing Capacity of Foundation Ground	1.2 or more	1.0 or more	Double cofferdam manual p 47
Circular Slip	1.2 or more	1.0 or more	Double cofferdam manual p 52

Source: Study Team

Table 7.3.118 Safety Factor of Sheet Pile Embedded Pile Length

Ground	Normal (High Water Time)	Seismic	Source
Sandy Soil	1.5 or more	1.2 or more	Double cofferdam manual p 58
Cohesive Soil	1.2 or more	1.2 or more	ditto

Source: Study Team

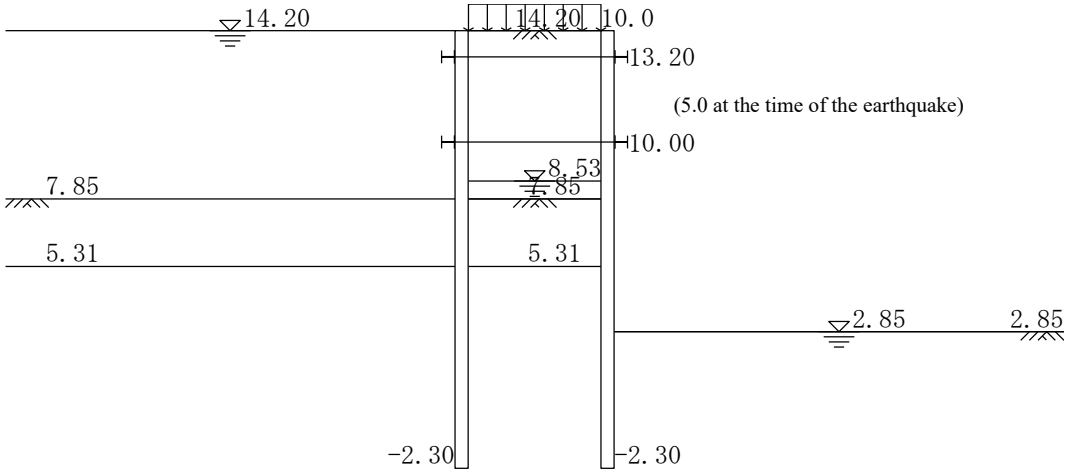
Table 7.3.119 Safety Factor of Seepage Path Length

Ground	Normal (High Water Time)	Source
Sandy Soil	3.5 or more	Double cofferdam manual p 67
Cohesive Soil	3.0 or more	ditto

Source: Study Team

(b) Calculation Model (General Section)

The model diagram is shown below.



Source: Study Team

Figure7.3.106 Model Diagram (1st Stage Construction)

(c) Calculation Result

(i) Result of Stability Calculation

The following is the calculation result with the condition that the wall width B = 5.000 (m), the inside sheet-pile length LR = 16.50 (m), and the outside sheet-pile length LL = 16.50 (m).

・Shear deformation failure

Study Case: Normal

Check point	Checking Surface G.l. (m)	Depth for Checking d	Deformation Moment Md (kNm)	Resistive Moment Mr. (kNm)	Safety Factor F
Pile tip	-2.300	10.150	0.00	17829.34	999.99 ≥ 1.20
Stratum Interface	5.310	2.540	1178.28	1740.42	1.48 ≥ 1.20
Minimum Safety Factor	5.310	2.540	1178.28	1740.42	1.48 ≥ 1.20
Ground Level	7.850	0.000	426.75	1223.36	2.87 ≥ 1.20

Case study: Seismic

Check Point	Checking Surface G.l. (m)	Depth for Checking d	Deformation Moment Md (kNm)	Resistive Moment Mr. (kNm)	Safety Factor F
Pile Tip	-2.300	10.150	0.00	18208.95	999.99 ≥ 1.00
Stratum Interface	5.310	2.540	410.49	2131.89	5.19 ≥ 1.00
Minimum Safety Factor	5.310	2.540	410.49	2131.89	5.19 ≥ 1.00
Ground Level	7.850	0.000	207.41	1483.28	7.15 ≥ 1.00

・Sliding

The check was carried out only at the pile tip.

Study case	Checking Surface G.l. (m)	Depth for Checking d	Horizontal Force Sum Fd (kn)	Resistance Sum Fr (kn)	Safety Factor F
Normal Seismic	-2.300 -2.300	10.150 10.150	1237.24 128.46	15404.05 15404.05	12.45 ≥ 1.20 119.91 ≥ 1.00

・Bearing capacity of foundation ground

The check should be made only at the tip of the rooting.

Study Case	Checking Surface G.L. (m)	Depth for Checking d	Ultimate Bearing Capacity Qu (kN)	V-γ2.Df.Be (kN)	Safety Factor F
Normal Seismic	-2.300 -2.300	10.150 10.150	25682.31 25682.31	619.50 628.25	41.46 ≥ 1.20 40.88 ≥ 1.00

■Embedding pile length

・results of the limit equilibrium method

Land side SSP

Total length = 25.500 m (G.L. -11.500 m)

Study Case	Required Length (m)	Decided Length (m)	Active Moment (kNm)	Passive Moment (kNm)	Safety Factor F
Normal Seismic	11.470 11.410	16.500 16.500	4169.52 619.10	101330.47 101330.47	24.30 ≥ 1.20 163.67 ≥ 1.20

・Study results on water barrier effect

Study Case	Seepage Path Length 1		
	L1 (m)	h1 (m)	Safety Factor F1
Normal	35.520	11.350	3.13>3.00

・Study on 4c > Σγh

4c > Σ γ h is not considered.

(ii) Cross-Sectional Force Calculation Result

・Bend, shear, and displacement

Land side SSP

Total length = 25.500 m (G.L. -11.500 m)

Study Case	Moment		Shear Force		Amount of Variation	
	Moment (kNm)	Generating Position (GL.m)	Shear Force (kn)	Generating Position (GL.m)	Deformation (mm)	Generating Position (GL.m)
Normal seismic	-232.18 -156.20	6.200 6.200	178.37 -137.45	3.200 10.000	26.81 15.61	6.200 6.200

・Tensile Material Reaction

Land side SSP

Study case	Upper Reaction Force (kN/m)	Lower Reaction Force (kN/m)
Normal Seismic	0.00 0.00	-234.48 -226.29

・Elastic region length

The elastic region is not examined.

(iii) Member Stress (Main body, tension member, tumbling member)

・Wall Body

Section Type: Steel Sheet Pile

Unit (N/mm^2)

	Land Side SSP	
Steel Material	SP -25 H SYW 295	
Study Case	Bending Stress	Shear Stress
Normal Seismic	144.2 ≦ 180.0 97.0 ≦ 270.0	11.1 ≦ 83.0 8.6 ≦ 125.0

・Tension Material

1)Upper Tension Member

Diameter : ϕ 25 (mm)

Material: High tensile steel 690

Installation interval: 1.800 (m)

Number of sticks: 1

Unit (N/mm^2)

Study Case	Bending Stress of Land Side SSP
Normal Seismic	0.0 ≦ 176.0 0.0 ≦ 264.0

2)Lower Tension Member

Diameter : ϕ 55 (mm)

Material: High tensile steel 690

Installation interval: 1.800 (m)

Number of sticks: 1

Unit (N/mm^2)

Study Case	Bending Stress of Land Side SSP
Normal Seismic	149.3 ≦ 176.0 144.1 ≦ 264.0

・Waling Beam

1)Upper Waling Beam

Steel: [x 75 x 6.5 x 10 x 150]

Material: SS 400

Unit (N/mm^2)

Study Case	Bending Stress of Land Side SSP
Normal Seismic	0.0 ≦ 140.0 0.0 ≦ 210.0

2) Lower Waling Beam

Steel: [250 × 90 × 9 × 13]

Material: SS 400

Unit (N/mm^2)

Study Case	Bending Stress of Land Side SSP
Normal Seismic	113.7 ≦ 140.0 109.8 ≦ 210.0

(iv) Examination of Water Barrier Effect

・checking method

The examination of water barrier effect (seepage path length) is carried out for following 2 seepage paths.

The water level condition: Normal at the time of stability calculation (checking of the land side SSP).

1)Seepage path length 1 (along sheet piles)

$$F1 = \frac{L1}{h1} \geqq FS$$

2)Seepage path length 2 (In the case of penetrating through the bottom of the excavation surface to the land side: If shape of the excavation bottom is not available, this study is not conducted)

$$F2 = \frac{L2}{h2} \geqq FS$$

Here,

FS: Required Safety Factor (Viscous ground: 3.00)

Fi: safety factor

L1: Seepage path length 1 (Along sheet piles)

h1: water level difference 1 (From the H.W.L. to the land side excavation surface)

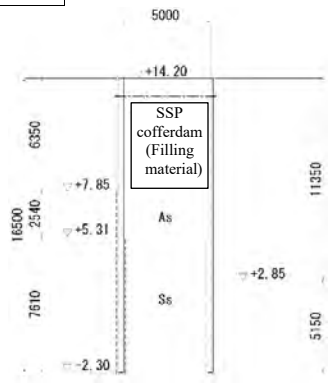
L2: Seepage path length 2 (from the bottom of excavation surface to the land side)

h2: Water level difference 2 (From the H.W.L. to the land side excavation surface)

・List of calculation results

Study Case	Seepage Path Length 1		
	L1 (m)	h1 (m)	Safety Factor F1
Normal	35.520	11.350	3.13>3.00

・Seepage path length 1 (along sheet piles)



L1 = D1 + Lb + D2

Here,

D1: Embedded length (m) of river side SSP

D2: Embedded length (m) of land side SSP

Lb: distance between sheet piles (m)

$Lb = \sqrt{\ } (B2 + \Delta L2)$

B: levee Width (5.000 m)

ΔL: difference between the lengths of the land side and river side SSP (0.000 m)

L1 = 2.540 + 7.610 × 3 + 5.000 + 5.150

= 35.520 (m)

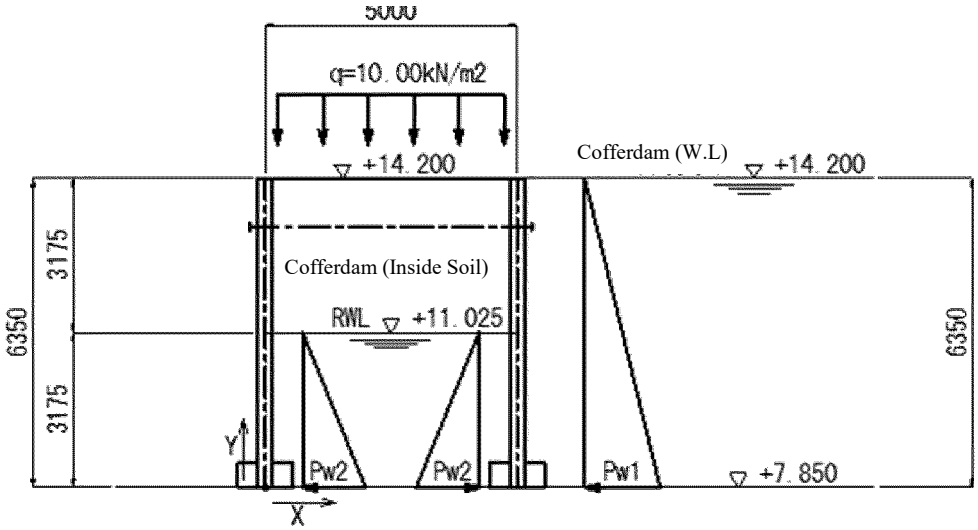
・Seepage path length 2 (from the bottom of excavation surface to the land side)

This case is omitted because there is no excavation shape is available

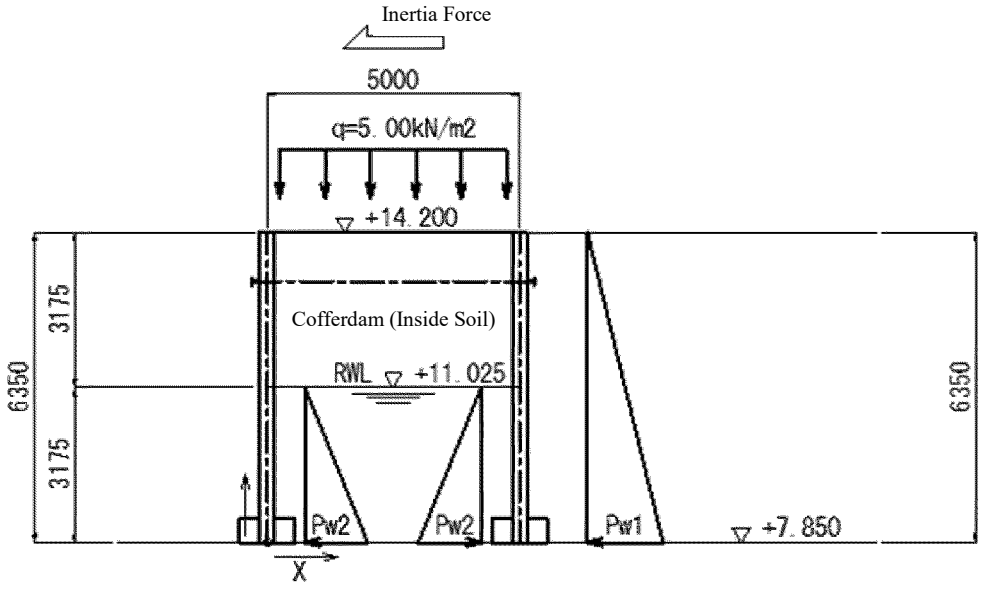
(d) Calculation Model (Bottom Slab Section: 2nd Stage Construction)

The model is shown below.

Normal



Seismic



Source; investigation team

Figure 7.3.107 Model Diagram (2nd stage construction)

(e) Calculation Result

(i) Stability Calculation

Normal

Load Calculation

	Vertical load	Horizontal load	Distance		Moment	
	V (kN)	H (kN)	X (m)	Y (m)	VX (kN m)	Hy (kN m)
Soil Filling	460.38		2.50		1150.95	
Water weight	158.75		2.50		396.88	
Surcharge	50.00		2.50		125.00	
Hydraulic pressure		201.61		2.117		426.81
Subtotal	669.13	201.61			1672.83	426.81

Acting position and eccentric distance of total load

$$x = \frac{\Sigma Mx + My}{\Sigma V} = \frac{1{,}672.83 - 426.81}{669.13} = 1.862 \text{ m}$$

$$e = \frac{L}{2} - x = \frac{5.00}{2} - 1.862 = 0.638 \text{ m} < \frac{L}{6} = 0.833 \text{ m}$$

... OK

Load Calculation

	Vertical load	Horizontal load	Distance		Moment	
	V (kN)	H (kN)	X (m)	Y (m)	VX (kN m)	Hy (kN m)
Soil Filling	460.38		2.50		1150.95	
Water weight	158.75		2.50		396.88	
Hydraulic pressure		201.61		2.117		426.81
Subtotal	619.13	201.61			1547.83	426.81

Acting position and eccentric distance of total load

$$x = \frac{\Sigma Mx + My}{\Sigma V} = \frac{1{,}547.83 - 426.81}{619.13} = 1.811 \text{ m}$$

$$e = \frac{L}{2} - x = \frac{5.00}{2} - 1.811 = 0.689 \text{ m} < \frac{L}{6} = 0.833 \text{ m}$$

... OK

Seismic

Load Calculation

	Vertical load	Horizontal load	Distance		Moment	
	V (kN)	H (kN)	X (m)	Y (m)	VX (kN m)	Hy (kN m)
Soil Filling (dry	301.63	30.16	2.50	4.763	754.08	143.65
Soil Filling (wet	158.75	31.75	2.50	1.588	396.88	50.4
Water weight	158.75		2.50		396.88	
Surcharge	50.00		2.50		125.00	
Hydrostatic Pressure		201.61		2.117		426.75
Hydrodynamic pressure		23.52		2.540		59.74
Subtotal	669.13	287.05			1672.84	680.59

Acting position and eccentric distance of total load

$$x = \frac{\Sigma Mx + My}{\Sigma V} = \frac{1{,}672.84 - 680.59}{669.13} = 1.483 \text{ m}$$

$$e = \frac{L}{2} - x = \frac{5.00}{2} - 1.483 = 1.017 \text{ m} < \frac{L}{3} = 1.667 \text{ m}$$

... OK

(ii) Examination of Sheet Pile Cross Section

Tensile stress

(1) Examination of Tensile Material

1) Dimension

Diameter		:	φ 60 (mm)
Materials		:	High tensile strength steel 690
Allowable stress		:	176 (N/mm2)
Tensile material installation interval	L	:	1.80 (m)
Number of tensile materials used	n	:	1 (set)
Area of tension material	A	:	60 ^ 2 × π/4 (N/mm2)

2) Tension calculation

$P = R \times L$

Tnesion material reaction force R kN/m	Interval L m	Tensile strength P kN/set
76.47	1.80	137.65

3) Stress intensity

$$\sigma = \frac{P \times 103}{n \times A} \leqq \sigma sa$$

Stress intensity σ N/mm2	Allowable stress σsa N/mm2	Evaluation
49	176	○

Stress intensity of waling material

(1) Examination of waling material

1) Dimension

Steel	:	[-150 × 75 × 6.5 × 10
Materials	:	SS 400
Allowable stress	:	140 (N/mm2)
Installation interval	:	1.80 m

2) Calculation of moment

$$M = \frac{P \times L}{10}$$

Tensile strength P kN/piece	Interval L m	Moment M kN m/m
76.47	1.80	13.76

3) Stress Intensity

$$\sigma = \frac{106\ M}{103 \times Z} \leqq \sigma sa$$

Z: Section modulus (= 115×2 cm3)

Stress Intensity σ N/mm2	Allowable Stress σ sa N/mm2	Evaluation
60	140	o

(iii) Shear Stress Check of Installing Point of the Bottom Slab

The horizontal force at the lower end based on the cross section calculation reaction force is

H = 140 kN

b = 400 mm

Shear checking with concrete only

$$\tau = \frac{H}{b \times d} = \frac{140.02 \times 1000}{1000 \times 400} = 0.35 < \tau a = 0.45\ N/mm2$$

... OK

The shearing force of concrete alone is satisfied, but since there are some portions where concrete is casted on the floor slab, the amount of re-bars which satisfies the shearing force only with re-bars is determined.

$$\tau = \frac{P}{A}$$

P: Horizontal force acting on lower end of sheet pile = H

A: Rebar Area

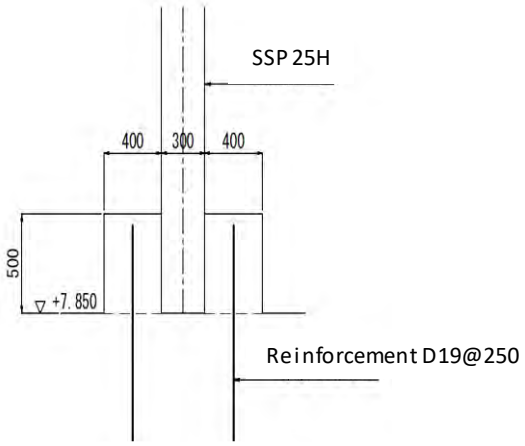
D19 @250 n = 2 x 4 = 8 piece

A = 286.5×8 = 2292 mm2

$$\tau = \frac{140.02 \times 1000}{2292} = 61\ N/mm2 < \tau a = 80\ N/mm2$$

... OK

Therefore, the pitch is D19 @250



(iv) Study on Anchors attached with the Temporary Cofferdam

(1) Design policy

Since the width of the main body and the width of the cofferdam are different at the part where the cofferdam is connected to the pier in the two-stage construction, the sheet pile is installed to the side wall of the pier with bending perpendicularly.

A tie rod is to be attached to the corner in the 45 ° direction, and it is assumed to be a cantilever in the pier side.

Therefore, considering the safety side, the shearing force of the anchor bolt is checked with the fulcrum reaction force at the tie rod position obtained by the sheet pile calculation.

(2) Fulcrum reaction force

Considering the safety side, it is assumed that 1/2 of 2.03m is shared by the weir body.

fulcrum reaction force

R = 76.47 kN/m (Fulcrum reaction force at tie rod position)

Shared load

P = 76.47×2.03/2 = 77.62 kN

Since the post-installed anchor is to be fixed the H-steel, four anchors are to be installed.

On the other hand, the reaction force acting on one anchor is

Po = 77.62/4 = 19.405 kN

Since anchor bolt which has the strength more than 19.41kN can be used, M16 is adopted。

o = 19.41 kN < 45.0 kN OK

(2) Slope for Construction

During construction, it is necessary to carry heavy machines such as cranes to the bottom surface in order to excavate river channels and install gates. Therefore, a slope for construction will be installed to connect the top of the embankment with the excavation ground.

1) Allocation

A slope will be installed inside the cofferdam to connect the top of the embankment with the excavation ground.

2) Width

The roadway width shall be 8.0 m with reference to the following standards, and the total width of the slope shall be 9.0 m with 0.5 m of guard fences on both sides.

Table 7.3.120 Width of Construction Road, Steepest Gradient, Specifications of Base Course (References)

Item	Specifications
1. Total width	3.5 m to 5.0 m
2. Total width (two lane)	8.0 m
3. Effective width (gravel road width)	3.0 m to 4.0 m
4. Effective width (two lane)	7.0 m
5. Steepest Gradient	10% to 15% or less
6. Sidewalk width	0.75 m
7. Crushed stone	W = 3.0 m to 4.0 m, t = 30 cm

Source: Temporary Planning Guidepiece (II) p.7

3) Slope

The slope gradient shall be 8% considering the allowable gradient of the vehicle used.

The next page shows the calculation result of the slope.

■calculation of the slope hitting the bottom of the vehicle

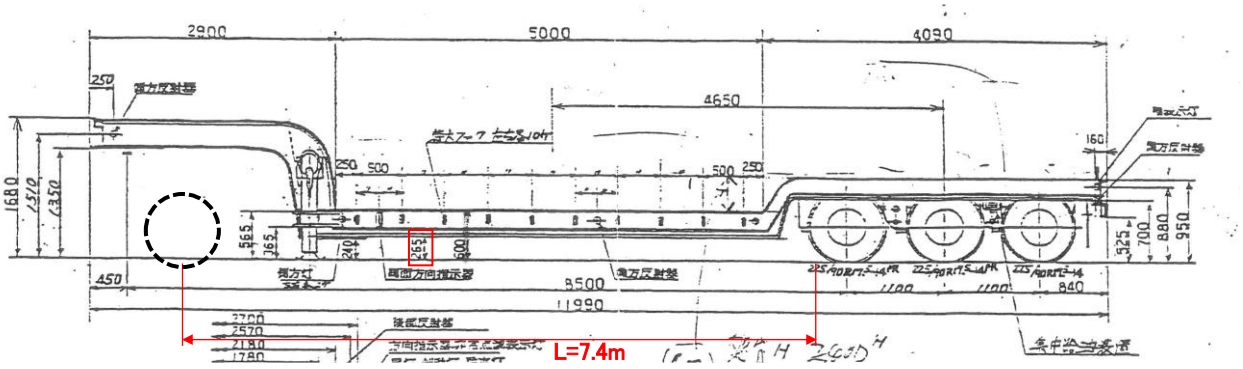
<calculation condition>

・Target vehicle: 25t low-floor semi-trailer

*It affords to carry 70 t suspension crawler crane (24.3 t: Weight when disassembled) used for gate installation.

・Wheel span L = 7.4 m (Read value)

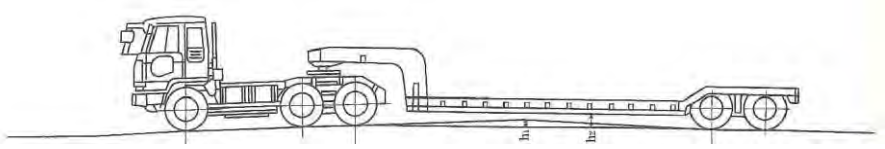
・bogie bottom height h = 265 mm



Source: Technical Report of Nishimitsu Unyu Co., Ltd.

The height of the convex part of the base course from the bottom of the bogie must be kept below h1 = 150 mm (100 mm margin) in order for vehicles to pass through.

When L = 7.4 m and h1 = 150 mm, the allowable gradient is calculated.

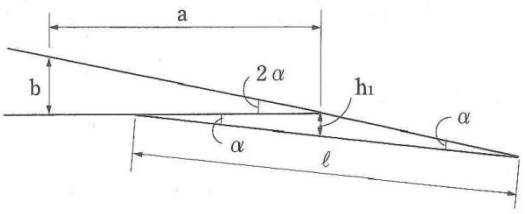


If slope = b/a = tan2 α , L = 7.4 m, h1 = 150 mm

$\tan\alpha$ = h1/(L/2) = 150/3400 = 0.041, α = 2.3 °

2 α = 4.6 ° , tan 4.6 ° = 0.08 = 1/1.3 = b/a

i.e. Slopes gradient shall be 1/1.3 (8%) or less.



7.3.5.2 Transportation Route Plan (MCGS) and Temporary Bridge Design

(1) Need for Transportation Routes and the Temporary Bridge Design

During MCGS construction, the following routes from public roads are accessible to the construction site: There are three possible routes shown in **Figure 7.3.108**.

1. On the right bank of the Marikina River, approaching from the Manggahan Road.
2. On the left bank of the Marikina River, through the administrative passageway of the EFCOS office
3. Entry from the development area of Ayala



Source: Study Team

Figure 7.3.108 Transportation Route Plan for MCGS Construction

Regardless of the route, it is necessary to install a temporary bridge on the current river channel or the diversion channel to secure a means of transportation for construction vehicles to move across the left and right banks of the construction site.

In this section, a temporary bridge is designed as a part of the transportation route at the MCGS construction.

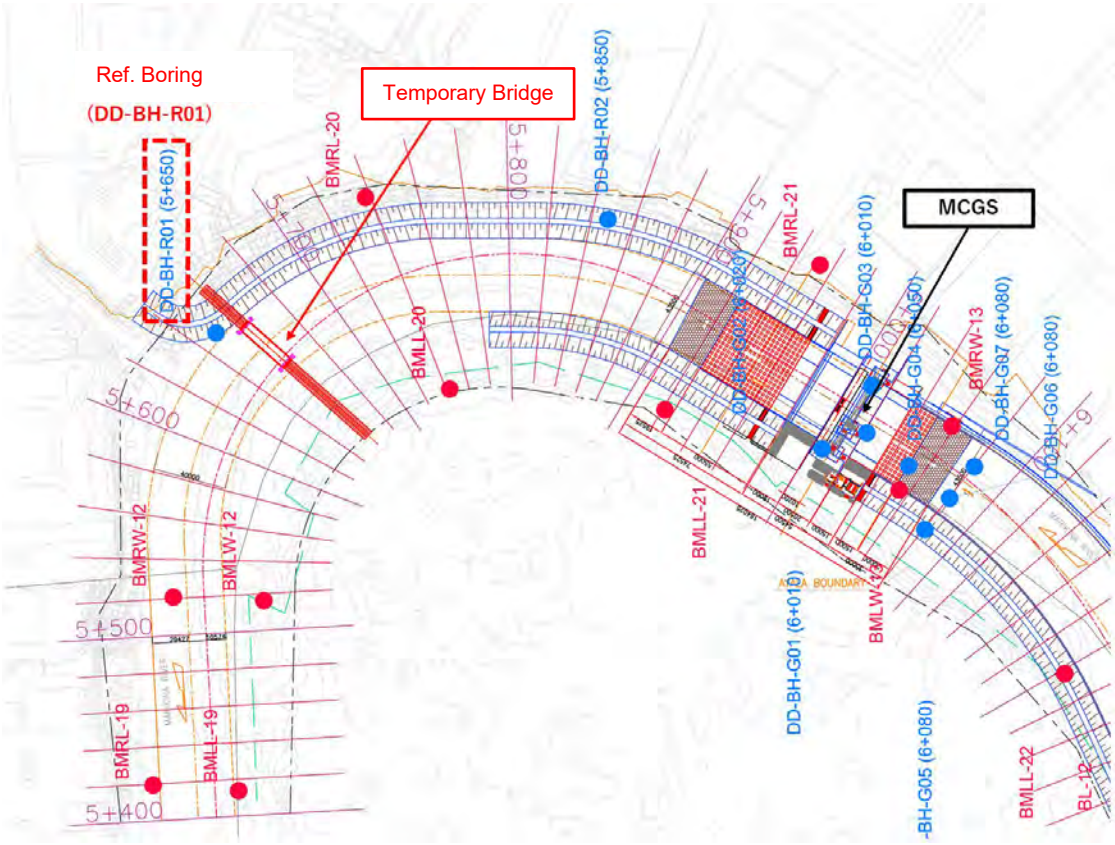
(2) Design of Temporary Bridge

1) Design Condition

(a) Basic Condition

(i) Location of Temporary Bridge

As shown in **Figure 7.3.108**, there are three possible routes for MCGS during the construction. In this section, the bridge position is set on the route of the first plan (STA.5+660), which is the point where the bridge length is the longest (the current river cross width is the widest). The geological conditions are set using the borehole results of DD-BH-R01 near the bridge location of the temporary bridge.

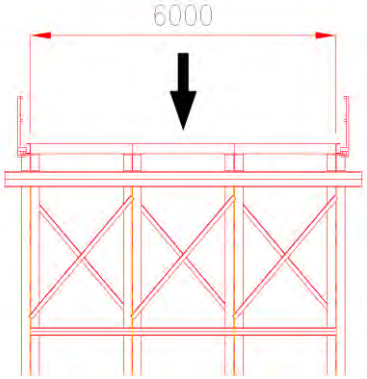


Source: Study Team

Figure 7.3.109 Installation Position of the Temporary Bridge

(ii) Road Width

According to the guidelines for temporary works in Japan, the standard width of the temporary bridge for construction is 4m to 6m. The bridge will be set at 6m in consideration of the travel allowance of construction equipment to be used.

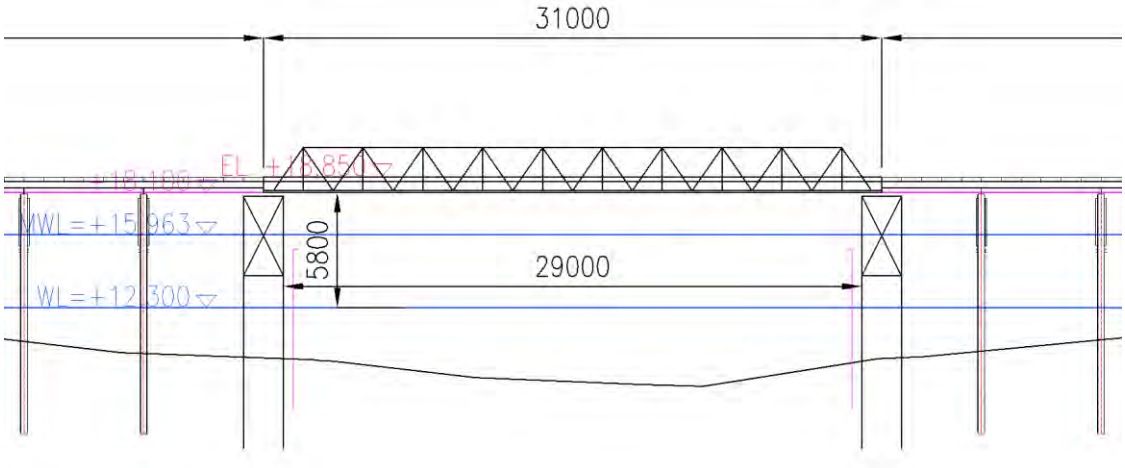


Source: Study Team

Figure 7.3.110 Cross section of temporary bridge (MCGS temporary bridge)

(iii) Road Surface Elevation

As shown in **Figure 7.3.110**, the elevation of the road surface is set to EL + 18.85m, in consideration of the current elevation of the right and left banks and the maximum elevation of the construction barges and various construction machines on the barges that go under the girder after the erection.



Source: Study Team

Figure 7.3.111 Control Points for Determining Road Surface Elevation

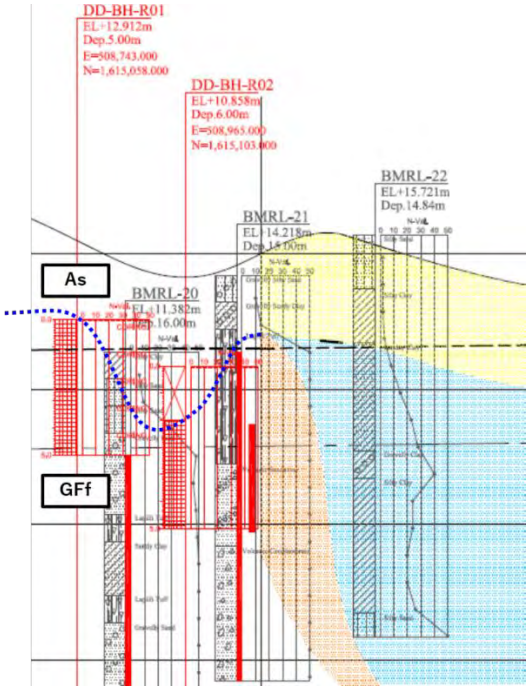
(iv) Span Length

As with the temporary bridge on Cainta River, a span length of 6m or more shall be secured.

However, in the central part of the river, as mentioned above, a span length approximately 30m was secured to allow the construction barges to safely navigate.

(b) Ground Conditions

Figure 7.3.112 shows the geological profile. The design soil constants are the same as the constants for the design of the revetment on the Marikina River (refer to **Figure 7.3.112**).



Source: Study Team

Figure 7.3.112 Geological Profile- Temporary Bridge on Marikina River

Table7.3.121 Design Soil Constants (MCGS Temporary Bridge)

Geological Classification		Unit Weight of Wet Soil γ_t	Unit Weight of Submerged Soil γ_w	Angle of Internal Fiction Φ		Effective Cohesion		(Reference)
Name	Symbol	(kN/m³)	(kN/m³)	(degree)		c (kN/m²)		Average N Value
Field Soil (Embankment)	F	18	8.2	30		0		15
Alluvial Sand	As	17.5	7.7	N≤9：	27	0		9.9
				9<N：	15+(15N)1/2			
Diluvial Sand	Ds	19	9.2		(Max: 45)	0		21.1
Alluvial Clay	Ac	15.5	5.7	0		N≤4：	6*N	7.8
						5<N≦8：	25	
						8<N≦15：	50	
						15<N≦30：	100	
						30<N：	200	
Diluvial Clay	Dc	18	8.2	0		N≤4：	8*N	22.6
						5<N≦8：	50	
						8<N≦15：	100	
						15<N≦30：	180	
						30<N：	250	
Guadalupe Formation	GFw	16.5	6.7	20		200		63.5
	GFf	17	7.2	30		1000		121.5

Source: Study Team

2) Load Conditions

The load shall be A live load, and the following will be considered as various construction machines used during the construction.

- Rough terrain crane (class 25t)
- Crawler crane (class 65t)

3) Materials

The bailey bridge will be applied to the 30m span of the central part where the construction barges will sail, and the behind and ahead of the bailey bridge will be designed as general temporary steel materials.

(3) Results of Detailed Design and General Drawings

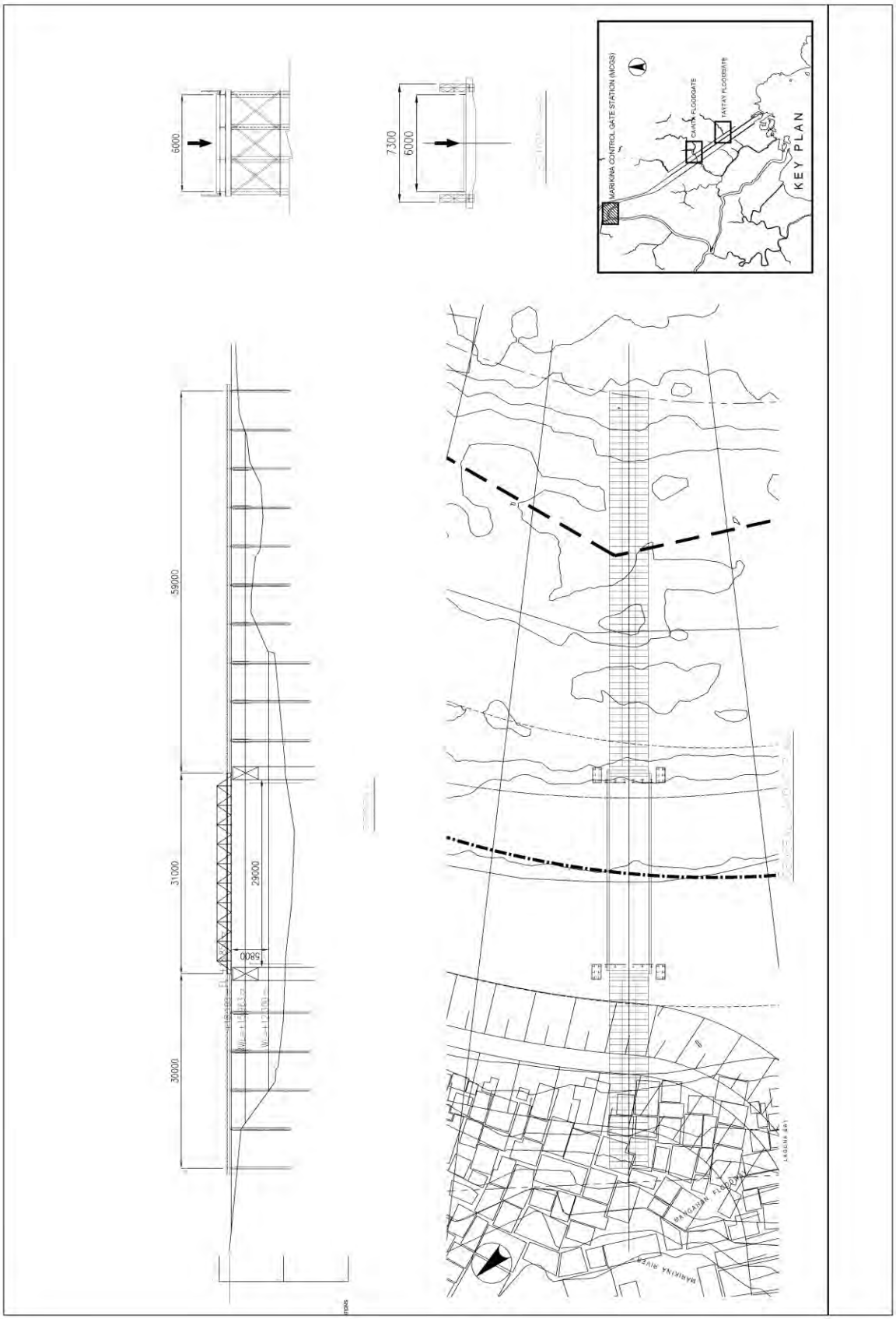
(1) Design Result

1) Result of Detailed Design

	Item	MD(M)-1.0×2.0
Deck Slab	Bending Moment(Max) Mmax σ	Truck road (parallel) 61.250 (kN.m) 79.167 ≦ 210.000 (N/mm²)
	Shearing force(Max) Smax τ	Truck road (parallel) 126.875 (kN) 63.148 ≦ 120.000 (N/mm²)

Main Girder (Longitudinal Direction)	Applied Member	H-350x350x12x19
	Bending Moment (Max) Mmax σ	Crawler crane moving (parallel) 311.134 (kN.m) 136.462 ≦ 164.486 (N/mm²)
	Shearing force (Max) Smax τ	Crawler crane moving (parallel) 207.427 (kN) 55.403 ≦ 120.000 (N/mm²)
	Deflection δ	Crawler crane moving (parallel) 1.353 ≦ 1.500 (cm)
Main Girder (Transverse Direction)	Applied Member	H-300x300x10x15
	Bending Moment (Max) Mmax σ	Truck road (parallel) 0.456 (kN.m) 0.338 ≦ 202.200 (N/mm²)
	Shearing force (Max) Smax τ	Crawler crane moving (parallel) 298.373 (kN) 110.508 ≦ 120.000 (N/mm²)
	Deflection δ	Truck road (parallel) 0.000 ≦ 0.500 (cm)
Pile	Applied Member	H-350x350x12x19 (Minor axis)
	Load Condition (cross-section) Load Condition (bearing capacity)	Crawler crane moving (parallel) Crawler crane moving (parallel)
	Acting force	N = 307.891 (kN) M = 107.142 (kN.m) S = 31.492 (kN) σc = 17.911 σb = 138.070 (N/mm²) τ = 2.368 ≦ τa = 120.000 (N/mm²)
	Verification of backling	Formula－1 ―― 0.850 ≦ 1.000 Formula－2 ―― 169.210 ≦ 210.000 (N/mm²)
	Bearing Capacity	307.891 ≦ 700.000 (kN)
Horizontal stiffener	Applied Member	[-150x75x6.5x10
	Compressive stress σc	26.565 ≦ 123.770 (N/mm²) (N= 62.985kN)
Joint part of horizontal stiffener	Required weld length	29.993 (cm)
Vertical brace	Applied Member	L-90x90x10
	Compressive stress σc	44.529 ≦ 73.767 (N/mm²) (T= 75.699kN)
Joint part of vertical brace	Required number	N= 2 ≦ 4 (M22)

2) General Drawing



7.3.6 Detailed Design of Information Equipment

7.3.6.1 Design Conditions of Information Equipment

In the information equipment design, the design conditions are summarized below from the basic design in Chapter 6.

Table 7.3.122 Design Conditions

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

Source: Study Team

7.3.6.2 Instrumentation (Water Level Observation Equipment) Design

(1) Examination of Instrumentation Facilities (Water Level Observation Equipment)

1) Water Level Observation System

There are following methods for water level observation by a water level gage, and the features of each method are summarized as follows.

(a) Float type water gauge

(b) Reed hoist type water gauge

(c) Hydraulic (quartz hydraulic system) water gauge

(d) Ultrasonic water gauge and radio wave type water gauge

(i) Float Type Water Gauge

The water level is gauged by observing the vertical movement of a float on the water surface. The vertical movement is converted to horizontal movement or angle through the internal mechanism of the water gauge with the rotation of a pulley.

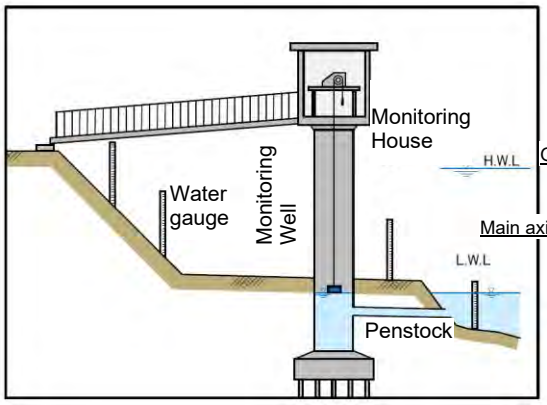
The basic configuration of the float type water gauge is as follows: It is installed and operated at Sto.Nino.

[Equipment whose composition is determined by the manufacturer]

- Measuring instruments (float type water gauge)
- observation well
- penstock
- maintenance bridge

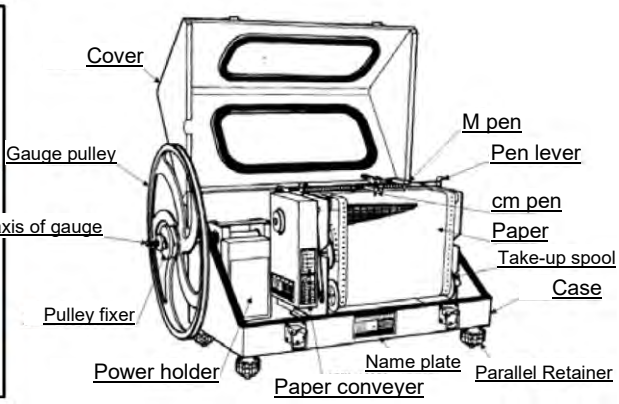
[Common items regardless of the type of water gauge]

- automatic recorder
- Automatic data transmission apparatus (Telemeter, .)
- Power Supply (Commercial power sources, solar cells, .)
- Observation hut (bureau)
- water mark
- labeling



Source: Study Team

Figure 7.3.113 Image of Float Type Water Gauge



Source: Study Team

Figure 7.3.114 Mechanism diagram of Float Type Water Gauge



Source: Study Team

Figure 7.3.115 Installation Example of a Float Type Water Gauge (Japan)





Source: Study Team

Figure 7.3.116 Installation Example of a Float Type Water Gauge (Sto Nino)

(ii) Reed Hoist Type Water Gauge

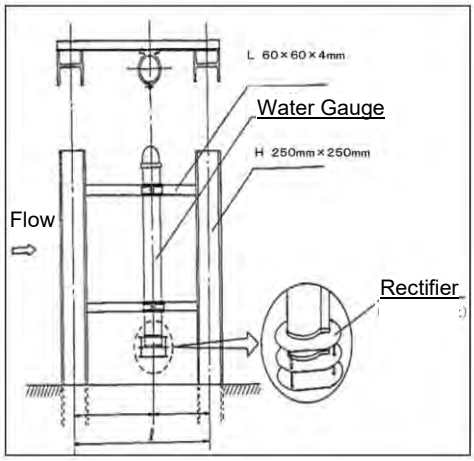
In the airtight chamber of the measuring pier installed under the water of the observation site, the measuring plate in which the reed hoist is arranged at 1cm interval is stored. The float with the permanent magnet is built in the adjacent measuring tube. When the hoist moves up and down according to the water level fluctuations, the reed hoist becomes conductive (ON), corresponding to the water level. The water level is measured by detecting the ON position of the reed hoist. The basic configuration of the reed hoist water gauge is shown below.

[Equipment whose composition is determined by the manufacturer]

- Measuring instruments (measuring pier)
- Radio equipment (as Required)
- Strut

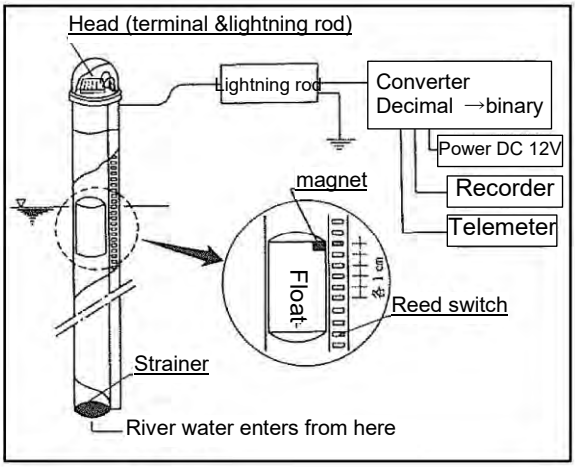
[Common items regardless of the type of water gauge]

- Automatic recorder
- Automatic data transmission apparatus (Radio, telemeter, .)
- Power Supply (Commercial power sources, solar cells, .)
- Water mark
- Labeling



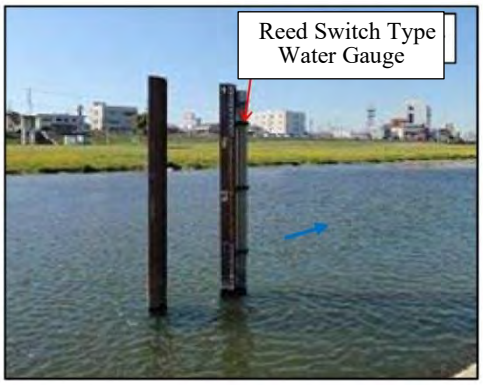
Source: Study Team

Figure 7.3.117 Image of reed hoist type water gauge

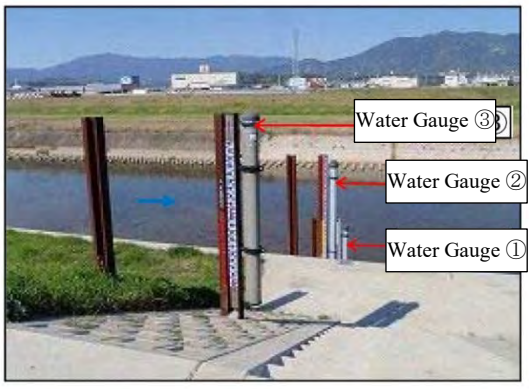


Source: Study Team

Figure 7.3.118 Mechanism diagram of reed hoist type water gauge



Source: Study Team



Source: Study Team

Figure 7.3.119 Installation Example of a Reed Hoist Type Water Gauge (Japan)

(iii) Hydraulic (Quartz Hydraulic System) Water Gauge

With the change of water, the change of water pressure received by the pressure receiving part installed under the water is measured mechanically (quartz type). Alternatively, the water level is measured by measuring the water depth by converting it into an electric signal by a pressure-sensitive element (semiconductor type). There are two kinds of the crystal type gauges according to the type of cable used for power supply and data transmission between sensor and water level converter. In addition to the conventional crystal-type water gauge, there is a light supply type which is hardly affected by the lightning. The basic configuration of a hydraulic (quartz hydraulic system) water gauge is shown below.

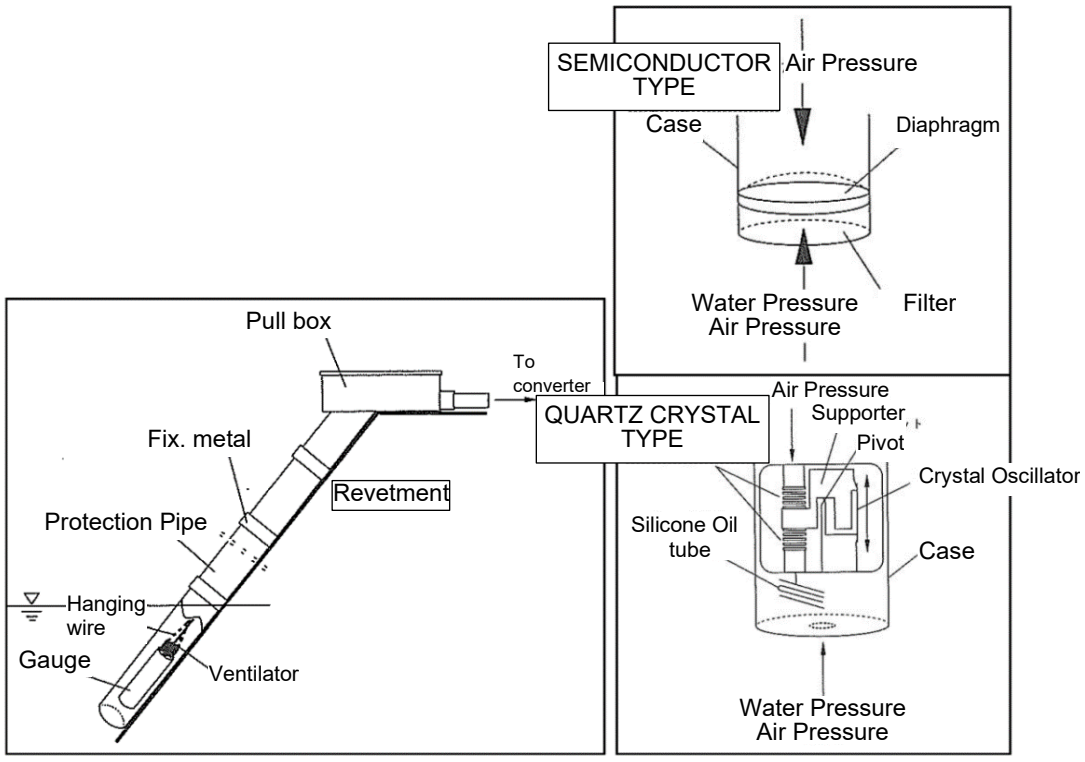
[Equipment whose composition is determined by the manufacturer]

- Measuring instruments (pressure receiving part)
- Protective tube
- Radio equipment (as Required)

[Common matters regardless of the type of water gauge]

Automatic Recorder

- Automatic data transmission apparatus (Telemeter, .)
- Power Supply (Commercial power sources, solar cells, .)
- WATER mark
- labeling

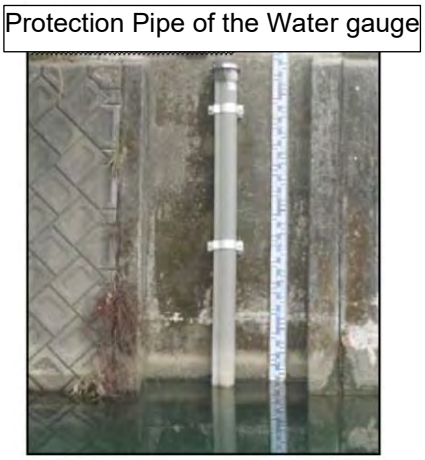


Source: Study Team

Figure 7.3.120 Image of Hydraulic (Quartz Hydraulic System) Water Gauge

Source: Study Team

Figure 7.3.121 Mechanism Diagram of Hydraulic (Quartz Hydraulic System) Water Gauge



Source: Study Team

Figure 7.3.122 Installation Example of a Hydraulic Type (Quartz Hydraulic System) (Japan)

(iv) Ultrasonic Water Gauge and Radio Wave Type Water Gauge

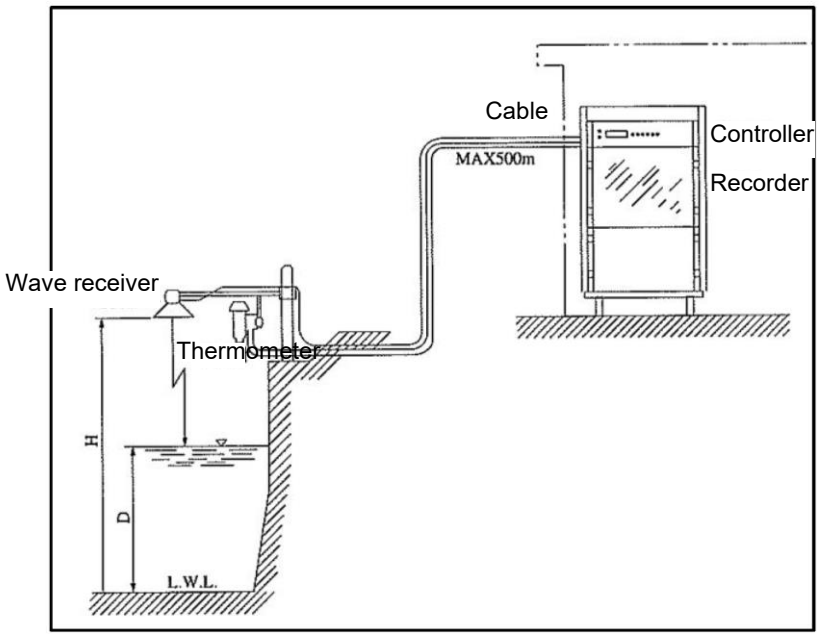
It measures the distance between the water surface and the transducer by measuring the time until the ultrasonic wave or radio wave (microwave) strikes the water surface and returns. The characteristic of the gauge is that it can measure water level without contacting with the water surface.

[Equipment whose composition is determined by the manufacturer]

- Measuring instruments (transducer)

[Common Items regardless of the type of water gauge]

- Automatic Recorder
- Automatic Data Transmission Apparatus (Telemeter, etc.)
- Power Supply (Commercial Power Sources, Solar Cells, etc.)
- Water Mark
- Labeling



Source: Study Team

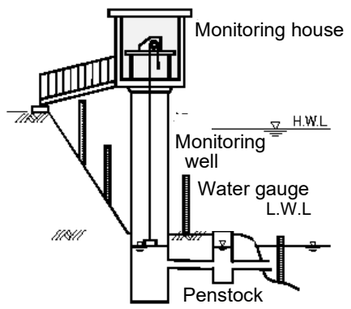
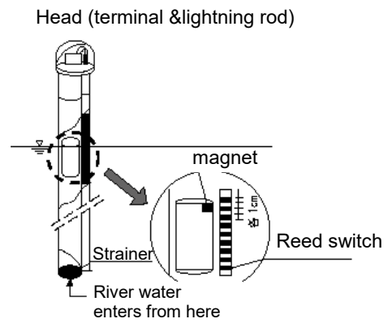
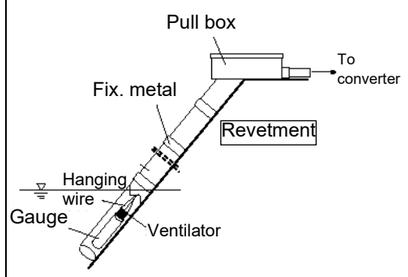
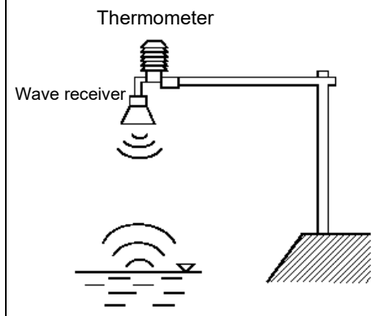
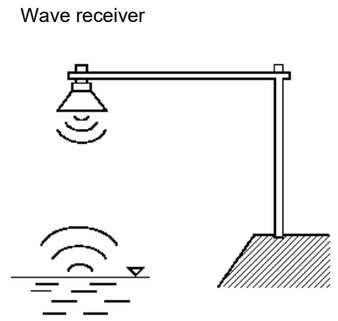
Figure 7.3.123 Images of Ultrasonic and Radio Wave Water Gauges



Source: Study Team *Source: Study Team*

Figure 7.3.124 Installation Example of an Ultrasonic and Radio Wave Water Gauges (Japan)

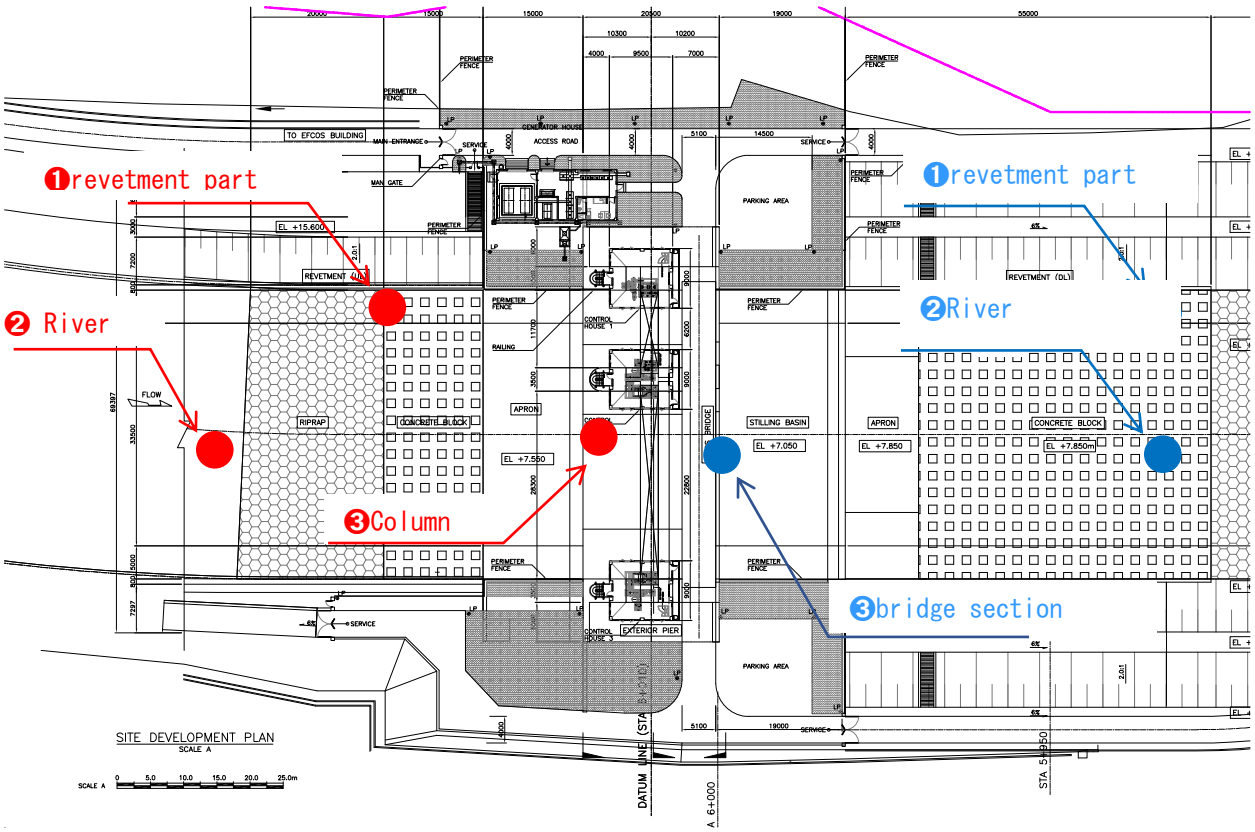
Table 7.3.123 List of Characteristics and Applicability by Type of Water Gauge

Item		Float Type Water Gauge	Reed Hoist Type Water Gauge	Hydraulic (Quartz Hydraulic System) Water Gauge	Ultrasonic Water Gauge	Radio Water Gauge
Outline Diagram of Measurement Principle						
Measurement Principle		A float is attached to one end of the wire and a weight (counterweight) is attached to the other end, and the wire is put on a pulley (Pulley). When the float moves up and down with the rise and fall of the water surface, the pulley rotates, and the water level is detected from the rotation amount.	A measuring pier is erected in the water, and a float with a magnet and a reed hoist arranged at a fixed Section Are arranged in the pier, and the water level is measured by the ON/OFF of the hoist by the top and bottom of the float.	A signal of a pressure sensor installed in a river channel is electrically converted to measure the water level. There are various types of sensors such as semiconductor type and crystal-type.	The water level is obtained by measuring the distance between the water surface and the ultrasonic wave receiver by mounting the ultrasonic wave receiver vertically above the water surface, shooting the ultrasonic wave toward the water surface, correcting the speed of the ultrasonic wave by the air temperature, and measuring the time until the reflected ultrasonic wave returns.	The distance between the water surface and the microwave transmitter/receiver is measured to obtain the water level by mounting the microwave transmitter/receiver vertically above the water surface, and by measuring the time until the microwave is emitted toward the water surface and reflected back.
Location Where It can be Installed		In a River Channel	In a River Channel	In a River Channel	Superstructures such as bridges, weirs, sluice gates, ., overhang rods	Superstructures such as bridges, weirs, sluice gates, ., overhang rods
Machine Characteristics	Measurement Accuracy	Within ± 1 cm	Within ± 1 cm	Semiconductor Formula: F.S ± 0.1% Quartz Type : F.S ± 0.05%	Within ± 1 cm	Within ± 1 cm
	Measurement Range (Vertical Direction)	to 100 m	to 5 m (per bottle) (Up to 50 m)	to 100 m	About 10 m	About 10 m
	Workability	Construction of structures such as observation wells and horizontal water pipes for low water is required when installing in rivers. Workability: Low (Needs an observation well)	It can be installed on the concrete wall surface of the waterway, but in the case of the river, it is necessary to place H-steel for support and install the measuring pier. Workability: Medium (Measurement pier required)	The sensor protection pipe is fixed to the revetment, and the sensor is dropped in it, and it is installed. It does not require a large structure. Workability: High (protective tube only)	A bridge or mounting pole is built, and a transducer is mounted on the tip of the arm. It is necessary to consider the effect of wind and vibration. Workability: Medium (Bridge mounting and gate machine room mounting possible)	A bridge or mounting pole is built, and a transducer is mounted on the tip of the arm. It is necessary to consider the effect of wind and vibration. Workability: Medium (Bridge mounting and gate machine room mounting possible)
	Maintainability	It is necessary to remove clogging of the water pipeline due to sediment runoff. Maintainability: Medium	It is necessary to remove the entanglement on the measuring pole and the supporting pole by sediment runoff, .etc. Since it is normal arranged in the place where the water surface is covered, it becomes an underwater maintenance work. Low Maintenance	(Especially of low wettability cohesive soil) Sediment needs to be removed if it accumulates. Maintainability: High	A scaffold for maintenance is required for the transducer. Maintainability: Medium	A scaffold for maintenance is required for the transducer. Maintainability: Medium
Environment of Installation Site	Areas with Large Riverbed Fluctuations	It cannot be observed when the headrace runs dry. Cleaning is necessary when sediment is accumulated in the water pipe.	When the sensor dries up, it cannot be observed.	It cannot be observed when the sensory part dries up.	When the irradiation area dries up, the bed height is measured.	It cannot be observed when the irradiation range dries up.
	a place where the flow velocity is high, and the water is turbulent	No Impact	If it is installed on H steel, it is affected by raising of dam by running water (Upstream), lowering of dam (downstream side),etc..	Suction (drop in water level in pipe) may occur at the pipe tip (sensitive area), lowering the observed water level.	Error is caused by weir raising (Upstream), weir lowering (downstream side), unevenness of water surface, air bubbles, . when installed near a structure such as a pier.	Error is caused by weir raising (Upstream), weir lowering (downstream side), unevenness of water surface, air bubbles, . when installed near a structure such as a pier.
	Outside Air Environment	No Impact	No Impact	No Impact	Since air is used as a medium, it is affected by air temperature and wind velocity.	No Impact

Source: Study Team

2) Study of the Water Gauge Layout

The purpose of the instrumentation equipment (water level observation equipment) installed in MCGS is to carry out accurate gate operation during a flood. In order to measure water levels according to the gate operation situation, it shall be installed in the upstream and downstream of the gate. Three possible locations for installation of water gauges are shown in **Figure 7.3.125**.



Source: Study Team

Figure 7.3.125 Water Gauge Installation Candidate Position

For each candidate position shown in **Figure 7.3.125**, the following aspects were compared and examined. As a result, the installation to the revetment of both upstream and downstream was adopted.

[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

Table 7.3.124 Candidate Sites for Installation of Water Gauges (Upstream MCGS)

Installation Position	❶Revetment Part	❷River	❸Column
Outline of Installation Location	A water gauge is installed at the upstream seawall of the MCGS to measure the water level.	A water gauge is installed near the center of the river channel (center of flow) to measure the water level.	A water gauge is installed in the central column of the MCGS to measure the water level upstream of the gate.
Applicable Method of Measuring Water Level	Float type Reed hoist type Hydraulic (quartz hydraulic system) Ultrasonic and radio wave type	Reed hoist type Hydraulic (quartz hydraulic system)	Ultrasonic and radio wave type
Application of Observed Water Level to facility Operation	○ Since the water level observed by the operation of the large span gate or the small span gate is different in the right and left banks, it is necessary to observe the water level in both banks and use it for the facility operation.	◎ Since it is observed in the position near the river channel center (center of flow), it is hardly affected by gate operation, and it is suitable for facility operation.	△ When the opening of the large span gate is different from that of the small span gate, the water level cannot be measured accurately, so that it is inferior to the facility operation.
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 pier (reed hoist 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 gauge and refuse protection.	○ It can be installed according to gate construction. The construction is comparatively easy, because it is installed in the center machine side control room external wall part.
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 control room of the central machine side is possible, and the maintainability is good.
Evaluation	Although it is necessary to install multiple water gauges, this method has good workability and maintainability, and the main plan is adopted.		

Source: Study Team

Table 7.3.125 Candidate Sites for Installation of Water Gauges (Downstream of the MCGS)

Installation Position	❶Revetment Part	❷River	❸Column
Outline of Installation Location	A water gauge is installed at the seawall downstream of the MCGS to measure the water level.	A water gauge is installed near the center of the river channel (center of flow) to measure the water level.	A water gauge is installed at the bridge girder of the MCGS maintenance bridge to measure the water level downstream of the gate.
Applicable Method of Measuring Water Level	Float type Reed hoist type Hydraulic (quartz hydraulic system) Ultrasonic and radio wave type	Reed hoist type Hydraulic (quartz hydraulic system)	Ultrasonic and radio wave type
Application of Observed Water Level to facility Operation	○ Since the water level observed by the operation of the large span gate or the small span gate is different in the right and left banks, it is necessary to observe the water level in both banks and use it for the facility operation.	◎ Since it is observed in the position near the river channel center (center of flow), it is hardly affected by gate operation, and it is suitable for facility operation.	△ When the opening of the large span gate is different from that of the small span gate, the water level cannot be measured accurately, so that it is inferior to the facility operation.
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 pier (reed hoist 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 gauge and refuse protection.	◎ It can be installed according to gate construction. Since it is installed in the maintenance bridge, the construction is comparatively 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 from the maintenance bridge is possible, and the maintainability is good.
Evaluation	Although it is necessary to install multiple water gauges, this method has good workability and maintainability, and the main plan is adopted.		

Source: Study Team

3) Selection of Water Level Observation Method

On the water gauge installed in the revetment, the optimum system was selected. From the result of the comparative examination shown in **Table 7.3.126**, the hydraulic system (quartz hydraulic system) is adopted.

Generally, a hydraulic type (quartz hydraulic system) gage is connected between the sensor and the converter by exclusive communication cable (metal cable). The countermeasure to lightning is required. In this design, as a safe connection against lightning, an optical power supply type crystal water pressure system which connects between a sensor and a transducer by an optical fiber is adopted.

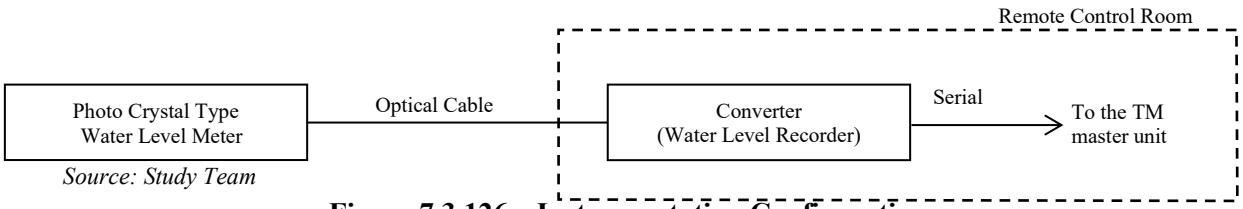
Table 7.3.126 Selection of Water Level Observation Method

Item	Float Type	Reed Hoist Type	Hydraulic (Quartz Hydraulic System)	Ultrasonic and Radio Wave Type
Measurement Accuracy	◎ Necessary and Sufficient	◎ Necessary and Sufficient	◎ Necessary and Sufficient	◎ Necessary and Sufficient
Measurement Range	◎ Necessary and Sufficient	◎ Necessary and Sufficient	◎ Necessary and Sufficient	◎ Necessary and Sufficient
Workability	△ Require the installation of observation wells	△ Need to install measuring poles	◎ The revetment is strong and has sufficient space.	○ It is necessary to install the arm for the sensor.
Maintainability	△ Sediment tends to accumulate in the water conducting part, requiring periodic removal of soil and sand.	△ A work ship is required for the maintenance in the river channel, and the maintainability is inferior.	◎ It is only pulled out from the protective tube, and maintenance is good.	◎ The sensor arm is only rotated, and maintenance is good.
Environmental Suitability	○ Since the sensor and observation equipment are connected by a metal cable, countermeasures against induced lightning are essential.	△ The transmission system of observation data is limited to the radio system, which may cause interference with other radio equipment.	◎ By adopting the light feeding type sensor, the sensor and the observation equipment are connected by the optical cable, and the resistance against the induced lightning is high.	○ Since the sensor and observation equipment are connected by a metal cable, countermeasures against induced lightning are essential.
Evaluation	△ Installation of an observation well is required, and large-scale construction is required.	△ It is necessary to install the measuring pole, and the workability is poor even in the periodic inspection.	◎ Both workability and maintainability are good and optimum.	○ Though both workability and maintainability are good, countermeasures against induced lightning are necessary and inferior to other plans.

Source: Study Team

(2) Instrumentation Configuration

The instrumentation configuration is shown below.



Source: Study Team

Figure 7.3.126 Instrumentation Configuration

(3) Equipment Specification

The equipment specifications are as follows.

1) Summary

The optical crystal-type water gauge is an instrument to measure the water levels in the upstream and downstream parts of MCGS.

2) Equipment Specification

(a) Sensors

- Measuring range 0 to 20 m
- Accuracy ± 0.05% FS
- Adaptable optical connector SM

(b) Water Gauge Coder

- Structure rack mount type
- Input optical signal
- Output BCD
- Operation touch panel type

7.3.6.3 Alarm Facility Design

(1) Consideration of Alarm Facility

1) Siren

(a) Selection of Siren System

The siren equipment is composed of siren and siren control panel. There are motor siren and inverter siren as types of siren equipment.

Though the conventional motor siren is used for the existing facility, the inverter type siren is becoming major recently.

(i) Type of Motor Siren

There are the following types of motor siren capacity.

Table 7.3.127 Types of Motor Siren

Power	Directivity	Motor Siren Capacity (kW)				
		0.75	2.2	3.7	5.5	7.5
200 VAC 3 Phase	Omnidirectional Type	○	○	○	○	○
	Directional Form	○	○	○	○	○
100 VAC Single Phase	Omnidirectional Type	○	-	-	-	-

Source: Study Team

(ii) Type of Inverter Siren

A. Capacity

There are the following types of inverter siren capacity.

Table 7.3.128 Types of Inverter Siren

Power	Directivity	Inverter Siren Capacity (kW)				
		0.75	2.2	3.7	5.5	7.5
200 VAC 3 Phase	Omnidirectional Type	○	○	○	○	○
	Directional Form	○	○	○	○	○
200 VAC Single Phase *	Omnidirectional Type	○	○	−	−	−
	Directional Form	○	○	−	−	−
100 VAC Single Phase *	Omnidirectional Type	○	−	−	−	−
	Directional Form	○	−	−	−	−

*Since the main body of the inverter siren is a three-phase input, the siren control panel converts a single phase into a three-phase.

Source: Study Team

B. Features

Features of the Inverter Siren are as Follows.

- Small size and light weight

Since the motor siren is rotated at high speed of 100 Hz instead of 50 to 60 Hz, the sounding part is smaller. In addition, the electromagnetic shutter to stop motor rotation is eliminated by turning off the power supply through the inverter. Consequently, the structure of the siren is more simplified. Therefore, it becomes possible to reduce the wind receiving surface and to

simplify the panzer mast and concrete pier by making them smaller and lighter. Besides, the installation work is possible without using the heavy machine by dividing the siren facility.

Table 7.3.129 Siren Capacity

Type / Capacity	Motor Siren	Inverter Siren
0.75 kW	105 kg	35 kg (Split up to about 20 kg)
2.2 kW	150 kg	50 kg (Split up to about 25 kg)
3.7 kW	175 kg	65 kg (Split up to about 35 kg)
5.5 kW	240 kg	75 kg (Split up to about 42 kg)
7.5 kW	280 kg	86 kg (Split up to about 50 kg)

Source: Study Team

- Silent inspection is possible

Using the multistage speed change function of the inverter, it is possible to detect the result by rotating the siren at low speed. As a result, the inspection of the siren at the same level as the actual sound test can be carried out without making a sound.

(iii) Economy

Comparison of reference prices for motor sirens and inverter sirens is shown in **Table 7.3.130** and **Table 7.3.131**. Though the inverter siren is cheaper, the control panel is a little expensive, and though there is little difference, the inverter siren is a little cheaper.

Table 7.3.130 Reference Price of Motor Siren (1000 yen)

Motor Siren Specifications	Siren	Control Panel	Total
Siren (Omnidirectional-Type), 3-phase, 200 V AC, 2.2 kW	399	280	679
Siren (Directional-Type), 3-phase, 200 V AC, 2.2 kW	472	280	752
Siren (Omnidirectional-Type), 3-phase, 200 V AC, 3.7 kW	480	280	760
Siren (Directional-Type), 3-phase, 200 V AC, 3.7 kW	590	280	870

Source: Study Team

Table 7.3.131 Reference Price of Inverter Siren (1000 yen)

Inverter Siren Specification	Siren	Control Panel	Total
Inverter Siren (Omnidirectional-Type), 3-phase, 200 V AC, 2.2 kW	289	362	651
Inverter Siren (Directional-Type), 3-phase, 200 V AC, 2.2 kW	362	362	724
Inverter Siren (Omnidirectional-Type), 3-phase, 200 V AC, 3.7 kW	320	400	720
Inverter Siren (Directional-Type), 3-phase, 200 V AC, 3.7 kW	415	400	815

Source: Study Team

(b) Comparison of Sirens

Control systems of motor siren and inverter siren are compared (**Table 7.3.132**). As a result of the comparison, it is recommended to adopt the inverter siren.

Table 7.3.132 Comparison of Motor Siren and Inverter Siren

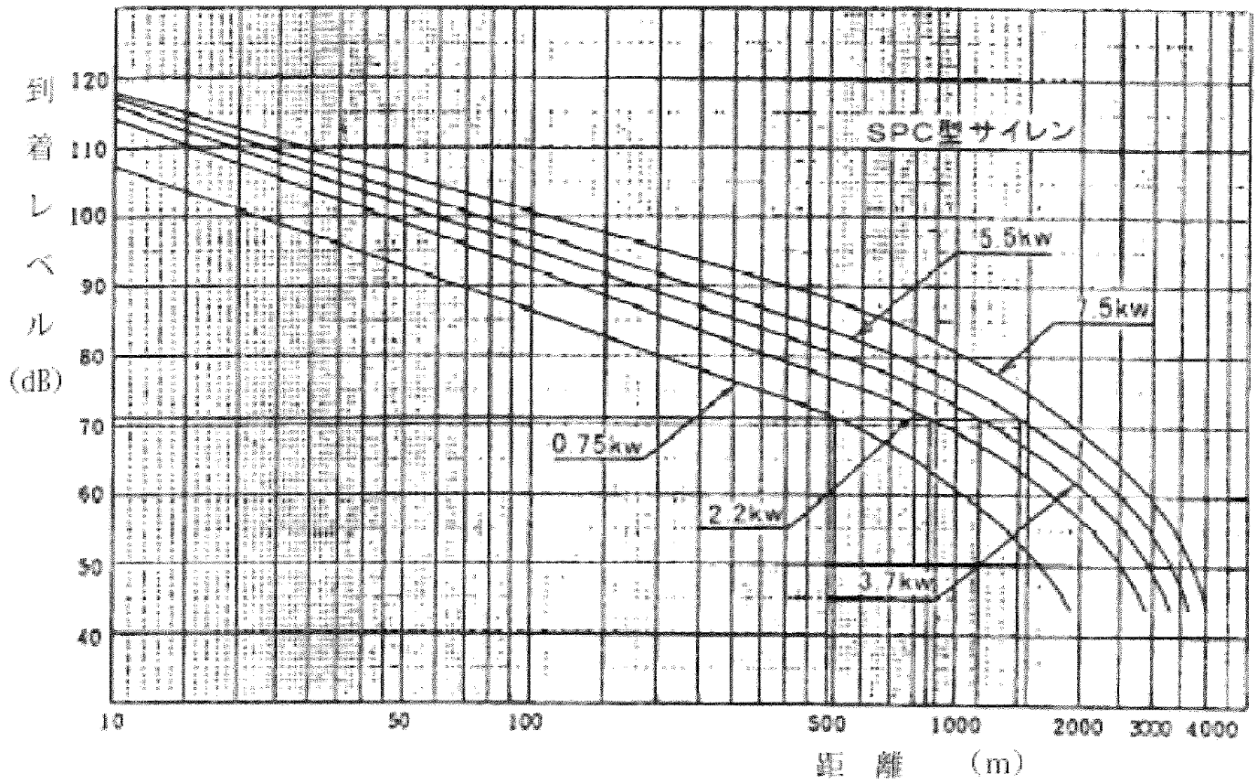
Item	Motor Siren	Inverter Siren
Capacity	0.75 kW, 2.2 kW, 3.7 kW, 5.5 kW, 7.5 kW	0.75 kW, 2.2 kW, 3.7 kW, 5.5 kW, 7.5 kW
Power	3 ϕ 200 V $\pm$ 10%, 50/60 Hz (0.75 kW has 1 ϕ 100 V)	3 ϕ 200 V $\pm$ 10%, 50/60 Hz (0.75 and 2.2 kW have 1 ϕ 100 V)
Structure	・Non-directional, directional, rain proof ・At the time of stopping, the afterglow is prevented by the electromagnetic shutter.	・Non-directional, directional, rain proof ・In stopping the sound, the number of revolutions of the siren is reduced to prevent the afterglow.
Method	・Commercial control system (Motor: Three-phase induction motor)	・Inverter control system (Motor: Three-phase induction motor)
Action	・When the siren is activated: the motor is activated and the sound is emitted ・When the siren stops: The motor and the electromagnetic shutter stop in conjunction with each other.	・When the siren is activated: the motor is activated and the sound is emitted ・When the siren is stopped: Reduce the number of revolutions of the siren by using the inverter control function ・Sounding sound pressure levels at night and daytime can be set by control commands from monitoring stations or alarm stations.
Serviceability	・Since the individual control of the electromagnetic shutter operation and the motor operation is impossible, the inspection by silence is impossible. △	・Silence inspection by low rotation is possible. ○
Siren Weight	・150 kg (With electromagnetic shutter) ・Robust and heavy to withstand the impact load of electromagnetic shutters during operation △	・50 kg (Split up to about 25 kg) (No electromagnetic shutter) ○
Starting Time	・Approximately 5 seconds	・Approximately 4.1 seconds
Stop Time	・Inertial rotation after shutter closure	・Approximately 1.5 seconds
Equipment Cost	Refer **Table 7.3.130** -(Not much different)	Refer **Table 7.3.131** -(Not much different)
Support Pier	・Support pier with larger load capacity than inverter siren is required. ○	・By reducing the weight of the siren, the weight load of the support pier is reduced, and the pier with small load capacity can be used. ○
Evaluation	△	○

Source: Study Team

(c) Determination of Inverter Siren Output

Inverter siren output is determined according to "Guidelines for telecommunications facility design (draft) and its explanation (Communications)".

The standard sound distance of a siren is obtained from the relation of attenuation quantity according to the sound distance. The sound distance curve is shown in the design guideline **(Figure 7.3.127**).



Source: Telecommunication Facilities Design Guidelines (Communications)

Figure 7.3.127 Relation between Sound Level and Distance at Which the Siren Sound Can be Heard

The target level of the siren is as follows, and the sound distance corresponding to the siren output satisfying this level is shown in **Table 7.3.133**.

The target S/N is the sound pressure difference obtained from the sorrowed noise level and the sound pressure of the siren, and the standard value of 6 dB shown in the design guideline. The sorrowed noise level is set at 65 dB as an intermediate value of noise level corresponding to the city residential area from the standard of noise level indicated in the design guideline.

Target level (standard value) of the siren = target S/N (6 dB) + sorrowed noise level (65 dB) = 71 dB

Table 7.3.133 Siren and Distance at which the Sound to be Heard(Standard Value)

Siren Output	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

Table 7.3.134 Approximate Surround Noise Level

Noise Level	Decibel	Situation Example
Conversation is impossible	120dB	Max. hearing level (with pain)
		Near the aircraft engine
		Lightning at close range
		Noisy subway station
	110dB	Near the factory siren
	100dB	Under the overpass when the train passes
Conversation is difficult		Inside the subway car/ train station
	90dB	Machine workshop/Air conditioning machine room/
	80dB	Printing factory/Intersection
		Waiting room/Market
Need to speak loud to have	70dB	Theater/Department store/Bank lobby
a conversation	65 dB	Busy office
	60dB	Restaurant/Large Shop/
		Normal conversation
		Urban residual area/Office
		Hotel lobby
Easy to have a conversation	50dB	The crowd of the theater audience
	40dB	Normal house (average)
		Quiet residential area
	30dB	Suburbs/Radio broadcast studio
	20dB	The sound of the leaves rubbing
	10dB	Whisper
	0dB	Min. hearing level

Source: Study Team

The purpose of this siren installed in MCGS is to inform the alarm at the gate operation. In this design, the output magnitude is determined so that the alarm sound can be heard by EFCOS which is a remote base.

The direct distance between the MCGS planned site and EFCOS is about 570m. The magnitude of sound reaches to this distance are 2.2 kW (**Table 7.3.135**).

Table 7.3.135 Siren and Distance at which the Sound to be Heard

Siren Output	Distance (Radius)
0.75 kW	ca. 500 m
2.2 kW	ca. 800 m
3.7 kW	ca. 1,100 m
5.5 kW	ca. 1,400 m

Source: Study Team

Thus, the inverter siren output of the MCGS is set at 2.2 kW.



Source: Study Team

Figure 7.3.128 Location of MCGS and EFCOS

2) Loudspeaker and Microphone

(a) Loudspeaker

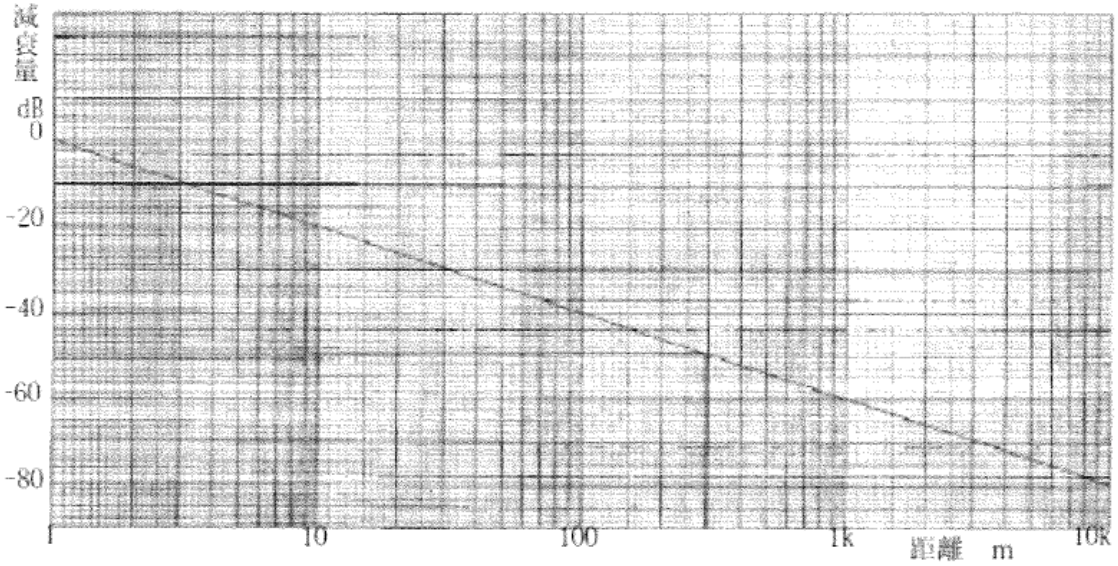
(i) Standard Loudspeaker Range

The standard hearable distance of the loudspeaker is shown in "Guidelines for telecommunications facility design (draft) and its explanation (Communications)".

The target level of a loudspeaker for audio broadcasting is defined as follows. The 3dB added here is the value for setting the service area (sound hearable area) in the sector of the half angle = 60 ° of the loudspeaker.

Target Achievement Level = Target S/N (10 dB) + Surround Noise Level (65 dB) + 3 dB = 78 dB

Assuming that the loudspeaker input is 50W, the output sound pressure level (1m value) is 127 dB **(Table 7.3.136)**. Thus, the allowable attenuation is 127 dB -78 dB = 49dB. From the attenuation by the sound distance shown in **Figure 7.3.129**, the sound hearable distance is estimated to be about 300 m.



Source: Telecommunication Facilities Design Guidelines (Communications)

Figure 7.3.129 Attenuation due to Sound Distance

Table 7.3.136 Output Sound Pressure Level of a Loudspeaker (1m Value)

Loudspeaker Input	Output Sound Pressure Level
1W	110 dB
25W	124 dB
50W	127 dB
100 W	130 dB

Source: Study Team

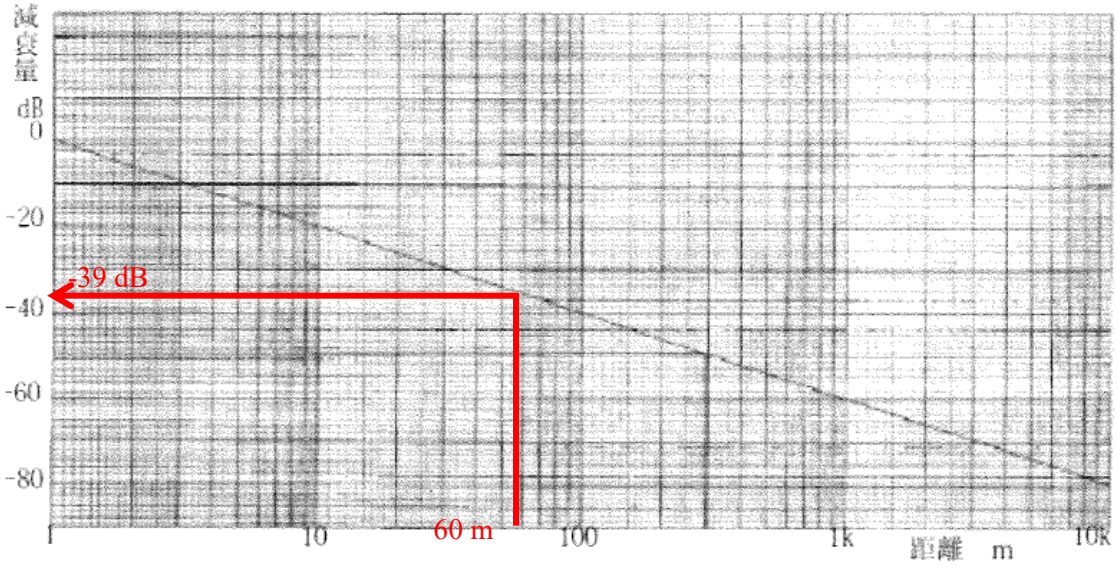
(ii) Setting the Loudspeaker Output Level

The purpose of the loudspeaker installed in the MCGS is to notice the gate operation to the surrounded area.

Since two gates (the small diameter and the large diameter) are installed in MCGS, loudspeakers corresponding to each gate operation will be installed. Since the loudspeaker sound shall reach to the opposite shore at the maximum, the output is determined with the hearable distance of about 60 m in this design.

The attenuation corresponding to the hearable distance of 60 m is read out as -39 dB, so that the output sound pressure level which requires 117 dB (= 78 dB + 39 dB) is obtained by adding the attenuation of 39 dB to the target arrival level of 78 dB.

If the speaker output is selected from this output sound pressure level, it will be equivalent to 25W. However, the range in which the speaker can hear the sound is a sector of about 60 ° with the hearable distance as the radius, and the actual hearable distance will be longer than the straight distance to the opposite shore. Therefore, 50W is set which is one rank higher than the speaker output obtained from the calculation.



Source: Telecommunication Facilities Design Guidelines (Communications)

Figure 7.3.130 Attenuation due to Sound Distance

Table 7.3.137 Loudspeaker Output Sound Pressure Level (1m Value)

Loudspeaker 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 sound broadcasting. It is installed in the alarm device. The standard rated output of the audio amplifier is 100W. To increase the output, an audio amplifier is added in units of 100W.

(c) Sound Collection Microphone

One microphone is installed for each loudspeaker.

3) Warning Light

In addition to the audible warning by the siren and the loudspeaker, the warning light installed on the MCGS adds visual information when operating the gate. The source of the warning light shall be a long-life, power-saving LED system. The blinking system shall be either a rotating reflector system or a blinking lamp system.

4) Operating Equipment

In "Ministry of Land, Infrastructure, Transport and Tourism Standard Specifications of Discharge Warning System" (No. 27), the operation of alarm facility shall mount either a dedicated console (hard) or a display console. The dedicated operation (hard) console system and the display console are as follows.

(a) Dedicated Console (Hard Table)

It was made exclusively for alarm facility operation. It is similar to the existing console of Rosario Weir alarm facility in EFCOS.

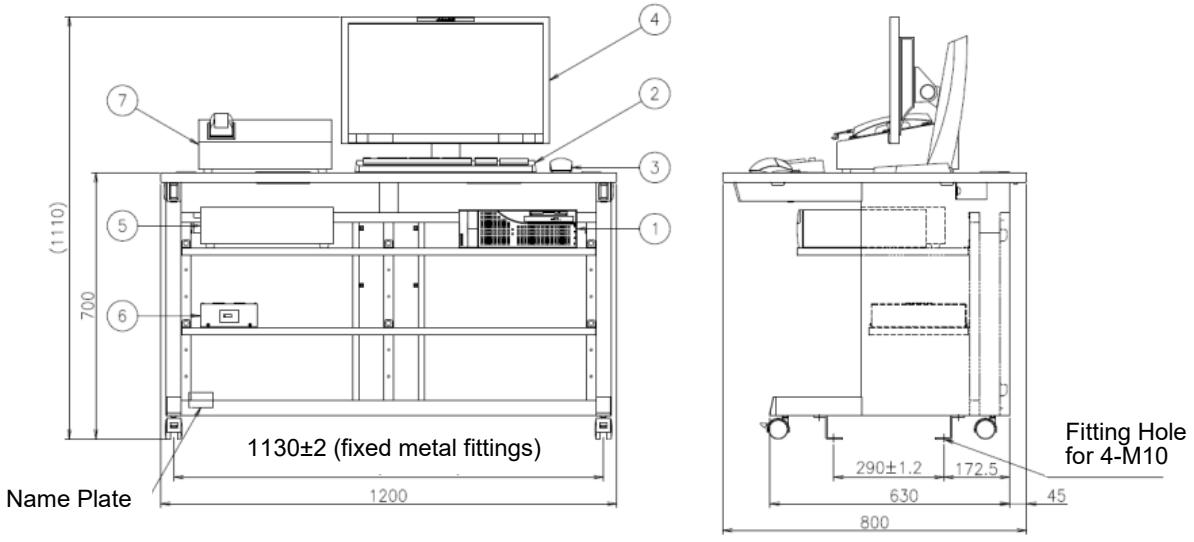


Source: Study Team

Figure 7.3.131 Existing Console for Rosario Weir Alarm System

(b) Display Console

On the console, a control monitoring screen is displayed, and the operation is performed by a mouse.



Source: Study Team

Figure 7.3.132 Example of Display Console

The following is a comparison of the dedicated console and the display console.

"dedicated console system" will be renewed as an exclusive console similar to the existing console. It is an exclusive operation machine. It is able to operate similarly to the existing facility. There is little confusion in operation. However, it is more expensive than the "display console system" because the console (housing) shall also be updated.

On the other hand, the "display console system" shows the control monitoring screen on display and is operated by a mouse. Though the way of operation changes, it is economically superior and easier to expand in comparison with the "console system."

In this study, the "display system" is recommended from the viewpoint of economy and expansibility.

Table 7.3.138 Comparison of Operating Facilities

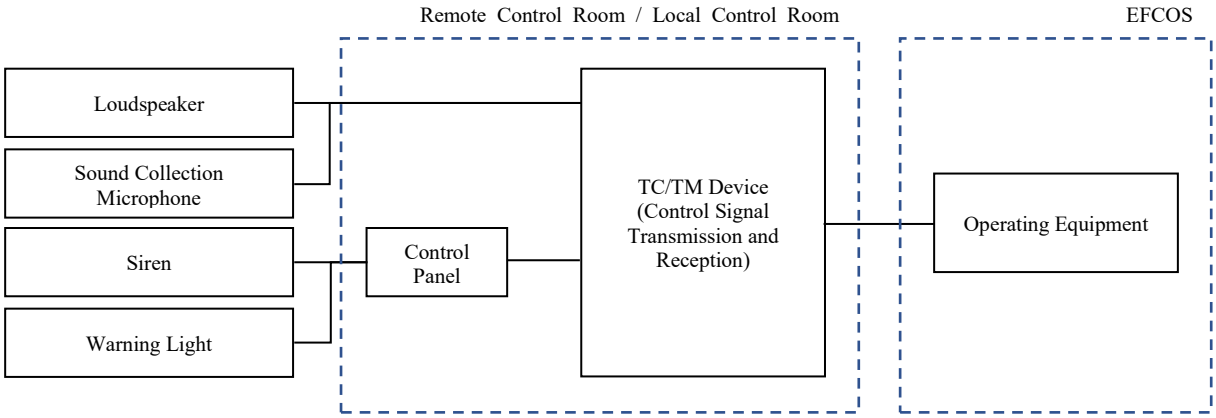
Comparison Item	Dedicated Console System	Display Console System
Summary	It is installed as a console of the same type as the existing one.	The control screen is displayed on the display, and the operation is carried out by mouse and keyboard.
Size	Console: W: 1,600 mm × D: about 900 mm	Installation (desk) space: W: 1,200 mm × D: about 800 mm
	△	○
	The console needs more space.	
Usability	It is a dedicated operation machine, and the operation (button) is clear. There are few mistakes.	Since it is a mouse operation, contrivance such as attaching the "Confirm" button is necessary in order to prevent the erroneous operation.
	○	△
	The console is operated by a button, and the display console is operated by a mouse. Though the operation console is superior in the intuitive operation, it is considered that the security of the operability is possible even in the display console system.	
Economy (Initial Cost)	22 million yen	18 million yen
	(△)	(○)
	Hardware and software costs.	
Economy (10-Year Total Expenses)	-1000 yen	¥1 million (500,000-yen x 2 times)
	In the case of the display table system, it is calculated that the hard disk of the control section (PC) is replaced once in 5 years.	
Scalability	Since it is a dedicated table (equipment), addition of buttons (Function), . is difficult for remodeling.	Since the operation part is a PC, the addition of the function can be dealt with by the remodeling of the software.
	(△)	(○)
	The addition of buttons in conjunction with the addition of functions requires the modification of the chassis in the case of the console system. The function addition in the case of the display system is the modification of the software.	
Overall Evaluation	(△)	(○)

Source: Study Team

(2) Study of Alarm System Configuration

The alarm system configuration is shown in **Figure 7.3.133**.

In the remote-control room, only EFCOS will introduce the operation equipment (display console), because the button operation is possible by the control panel.



Source: Study Team

Figure 7.3.133 Configuration of Alarm Facility

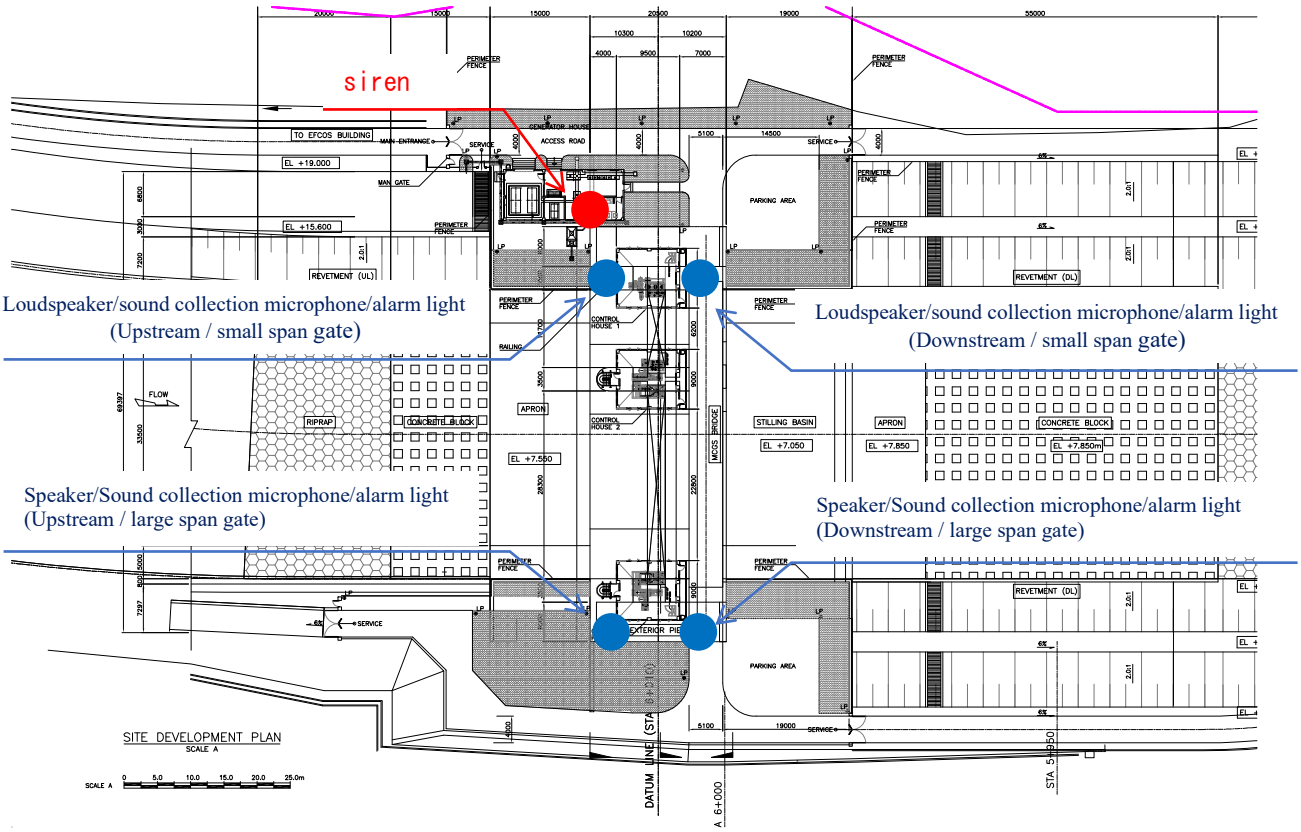
(3) Alarm Facility Layout

Alarm facility installed in MCGS shall be arranged according to the following concept.

Table 7.3.139 Arrangement of Alarm Facility (MCGS)

Alarm Facility	Placement Position	Quantity	Placement Criteria
Siren	Control Room Roof	1	The sound range of the siren is about 800 m, and the alarm sound can be known within a radius of 800 m by placing it in the MCGS control room.
Loudspeakers	Cockpit control room Upstream and downstream surfaces (left and right bank)	1 (Upstream of Left Bank) 1 (Downstream of Left Bank) 1 (Upstream of Right Bank) 1 (Downstream of Right Bank)	Loudspeakers, sound collection microphones and alarm lights are arranged in the left bank machine side control room and the right bank machine side control room so that they can sound and light in the operation of the small span gate and the large span gate respectively. It is arranged on the Upstream and the downstream side of the machine side control room so that it can be known up and down stream.
Sound Collection Microphone			
Warning Light			

Source: Study Team



Source: Study Team

Figure 7.3.134 Alarm Installation Position

(4) Equipment Specification

The equipment specifications are as follows.

1) Siren

(a) Summary

The siren performs operations such as siren sounding, and vocal broadcasting by a call-control from the control station, and it returns the operation status to the control station.

(b) Equipment Specification

(a) Structure :outgate type
(b) Capacity : 2.2 kW
(c) Directivity :omnidirectional

2) Loudspeaker

(a) Summary

The loudspeaker functions except for the siren sounding operation described in the above paragraph.

(b) Equipment Specification

(a) Structure :outgate type
(b) Format :reflex horn type
(c) Rated input :50 W
(d) maximum input :70 W
(e)output sound pressure level : Sound pressure at 1m on the front axis at input 1W > 104 dB
(f)wind resistance speed : Maximum instantaneous wind speed of 60 m/s

3) Sound Collection Microphone

(a) Summary

The sound collection microphone is used for the collection of siren sounding, pseudo sound sounding and chime broadcasting.

(b) Equipment Specification

(a) Structure :outgate type
(b) Format :horn type
(c) Playback frequency band : 0.4 to 5 KHz
(d) Nominal impedance : 8Ω ± 15%

4) Alarm Light

(a) Summary

The alarm lamp emits light according to the following standards and functions by the LED system.

(b) Equipment Specification

(a) Light source : LED lamp
(b) flash system : Reflection mirror rotary type or lamp flashing type

5) Control Panel

(a) Summary

An inverter siren control panel controls sirens and alarm lights by an inverter system.

(b) Equipment Specification

(a) Structure : Wall hanging type made of ingate steel plate
(b) operational function : turning on and off the siren motor
(c) protection function : Prevention of overcurrent, detection of missing phase and reverse phase, and detection of operation time over

(c) Functional Specification

(a) Power supply control of siren by inverter system
(b) inspection control by low rotation
(c) Phase failure of siren power supply and power supply detection
(d) Power control of siren by manual button
(e) alarm lamp control function

6) TM Slave Unit

(a) Summary

Th slave unit shall be connected with loudspeakers, sirens, alarm lights, and sound collecting microphones and control them.

(b) Equipment Specification

It consists of the following equipment.

No.	Item
1	Alarm Device
-1	Basic Control Unit
-2	Modulation and Demodulation Unit
-3	Transmission Control Unit
-4	Sound Collecting and Returning Part

-5	Sound Collection Level Determination Part
-6	Alarm Control Unit
-7	Chime Sound Output Part
-8	Impedance Determination Part
-9	Test Department
-10	Power Supply Section
-11	Monitoring Information Input Unit
-12	Housing
2	Audio Amplifier

(c) Functional Specification

(i) Basic Control Unit

The basic control unit is the center of the alarm facility and controls each input/output control unit and judges the reception code.

(ii) Modulation and Demodulation Unit

The modulation/demodulation unit is composed of a modulation circuit for converting a pulse code into a frequency modulation symbol and a demodulation circuit for converting a frequency modulation signal into a pulse code.

(iii) Transmission Control Unit

The transmission control unit manages a data transmission procedure with the control monitoring device and transmits and receives codes.

(iv) Sound Collecting and Returning Unit

The sound collecting and returning unit shall have the function of returning audible sounds for approximately 5 seconds after activation.

(v) Sound Collection Level Determination Unit

A sound collection level determination unit determines the level collected by the sound collection microphone.

(vi) Alarm Control Unit

The alarm control unit shall have a siren control function (Not applicable to loudspeaker alarm stations), a pseudo sound control function, and a broadcast control function.

① The siren sounding and pseudo sound sounding patterns shall satisfy the following standards. The operation time and pause time shall be constant for each operation.
- operating time ; Up to approximately 1 minute can be set in 5 second increments.
- Downtime ; Up to approximately 1 minute can be set in 5 second increments.
- Number of operations; You can set up to 9 times.

① The frequency for the pseudo sound sounding is a mixed sound of 523.5 Hz and 526.5 Hz. Frequency accuracy shall be within ± 0.1%.

② protection circuit
The alarm control unit shall have a transmission protection circuit and a protection circuit for sirens and loudspeakers.
The time length of the transmission protection circuit shall be about 10 minutes, and the time length of the protection circuit of the siren and the loudspeaker shall be configurable within a maximum of 10 minutes.

(vii) Chime Sound Output Unit

The chime shall be of an electronic circuit and the standard tone shall be 4 beats (Do Mi Do and Do Mi Do).

(viii) Impedance Judgment Unit

The impedance judgment unit shall judge the impedance of the loudspeaker during inspection control and satisfy the following standards.

① normal impedance range specified value ± 50%
② inspection sound frequency 1000 Hz ± 10%
③ inspection sound output 1W or less
④ supervisory information return bit loudspeaker 1 to 3 error

(ix) Test Department

The test section is provided with functions necessary for system maintenance.

(x) Power Supply Section

The power supply supplies necessary power to each part in the alarm device.

(xi) Monitoring Information Input Unit

The monitoring information input unit inputs the following monitoring information.

① inspection error
② 100 VAC power failure
③ 100 VAC power failure
④ power supply error
⑤ audio amplifier error
⑥ siren error
⑦ Audio amplifier ON
⑧ Siren ON

(xii) Box

The box shall contain the equipment that constitutes th slave unit and shall be drip-proof.

(xiii) Audio Amplifier

The specifications and functions of the audio amplifier are as follows.

① Output 100 W
② current consumption 18 A or less
③ clipping distortion 10% or less at 100 W output
④ crossover distortion Power supply voltage: 10.8 V, surround temperature: + 40 ° C
⑤ frequency characteristic Within 3 dB at 0.3 to 3 kHz
⑥ output impedance Match loudspeaker impedance
⑦ signal-to-noise ratio 40 dB or greater
⑧ input level 0 dBm or -4 dBm (Deviation ± 3 dB)
⑨ input impedance 600 ω ± 20%
⑩ Operation and display functions a. Power on/off
b. Adjust the output level
c. Measurement of output by meter
d. Microphone connection function

7) TM Master Unit

(a) Summary

Th master unit shall remotely control loudspeakers, sirens, alarm lights and sound collection microphones.

(b) Equipment Specification

It consists of the following equipment.

No.	Item
1	Control Monitor
-1	Basic Control Unit
-2	Transmission Control Unit
-3	Modulation and Demodulation Unit
-4	Clock Unit
-5	Time Control Unit
-6	Test Unit
-7	Print Control Unit
-8	Power Supply Unit
-9	Box
-10	Alarm Control Unit
-11	Monitoring Information Determination Unit
2	Printer

(c) Functional Specification

(i) Basic Control Unit

The basic control unit is the center of the control and monitoring equipment and has the total control and monitoring function of the system. The maximum number of alarm stations that can be controlled by the basic control unit is 30.

(ii) Transmission Control Unit

The transmission control unit has a function of managing transmission procedures such as control calls of alarm stations, collection of state monitoring information, and transmitting and receiving codes.

Individual specifications such as the number of alarm stations are specified in the special specifications.

(iii) Modulation and Demodulation Unit

The modulation/demodulation section is composed of a modulation circuit for converting a pulse code into a frequency modulation signal and a demodulation circuit for converting the frequency modulation signal into a pulse code.

(iv) Clock Unit

The clock unit is driven by the internal clock and outputs the year, month, and hour data to each section. The year, month and day are automatically move forward.

① accuracy Within ± 3 seconds of daily difference

② guaranteed outage time Within 30 minutes

It should be able to calibrate the clock by LAN connection or 1-minute pulse.

(v) Time Control Unit

The time control unit inputs time data from the clock part and controls the time.

(vi) Test Unit

The test unit is provided with functions necessary for system maintenance.

(vii) Print Control Unit

The print control unit controls the printer to output a document or print a list of alarm control operations.

① Cable length to the printer Up to 15 m (For RS 232 C connection)

② print item the number of print column is fixed.

(viii) Power Supply Unit

The power supply unit provides necessary power to each part in the control and monitoring device.

(ix) Box

The box shall be a housing for the equipment that constitutes th master unit.

(x) Alarm Control Unit

The alarm control unit has a function of performing control management for the alarm station and collation determination of the control answer signal returned from the alarm station. The following six items can be controlled and judged.

① Siren 1
② Pseudo-Sound 11
③ microphone broadcasting
④ stop broadcasting
⑤ inspection
⑥ Monitoring

(xi) Monitoring Information Determination Unit

The monitoring information determination unit has a function of determining the control confirmation signal returned from the alarm station. The following nine points of monitoring information can be determined.

① inspection error
② 100 VAC power failure
③ 200 VAC power failure
④ power supply error
⑤ Sound amplifier 1 error
⑥ Loudspeaker 1 error
⑦ siren error
⑧ Audio amplifier 1 ON
⑨ Siren ON

7.3.6.4 Design of Monitoring Equipment (CCTV Camera)

(1) Monitoring Object

The camera installed in MCGS remotely monitors accurate situation of the facility and operation during the flood. In this design, in addition to monitoring of gate conditions (facility monitoring), watching of river conditions at the time of flood (spatial surveillance) will be carried out. Monitoring objects are as follows.

Table 7.3.140 Monitoring Objects

Objects	Monitoring Classification	Principle of Monitoring
Gate Facility	Facility Monitoring	The angle of view is fixed in the gate direction, and the opening and closing situation is monitored remotely.
River Condition	Spatial Surveillance	For the angle of view of river upstream and downstream, the situation is widely grasped. The monitoring direction can be arbitrarily changed by turning and zooming.

Source: Study Team

(2) Monitoring System

There are SD camera and HD camera in CCTV camera. 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

1) Camera Specification

In the Ministry of Land, Infrastructure, Transport and Tourism, "CCTV Camera Equipment Specifications (September 2018)" which is the standard equipment specification of the CCTV camera equipment is published, and the standard specification of the HD camera is decided in it. The camera apparatus described in the specification is shown in **Table 7.3.141**.

Table 7.3.141 List of CCTV Camera Equipment Specifications (Draft) of the Ministry of Land, Infrastructure, Transport and Tourism

No	Image Quality Standard	Standard Equipment Specification	Remarks
1	HD	HD Camera Apparatus	H 30.9
2		HD Camera Equipment (High Sensitivity)	
3		HD Camera Equipment (High Magnification)	
4		HD IP Camera Apparatus	
5		HD IP Camera Equipment (High Sensitivity)	
6		HD Simple IP Camera Device	
7		HD Simple IP Camera (Non-Swivel Type)	

Source: Study Team prepared based on

In the CCTV camera equipment specification (draft), camera equipment is classified into three types: "Camera apparatus", "IP camera apparatus", and "Simple IP camera apparatus". Since the transmission from the CCTV camera to the control room and EFCOS is the IP transmission, "Ip camera apparatus" and "Simple IP camera apparatus" are selected for this design, which can mount the encoder in the CCTV camera main body.

2) Nighttime Monitoring (Minimum Subject Illuminance)

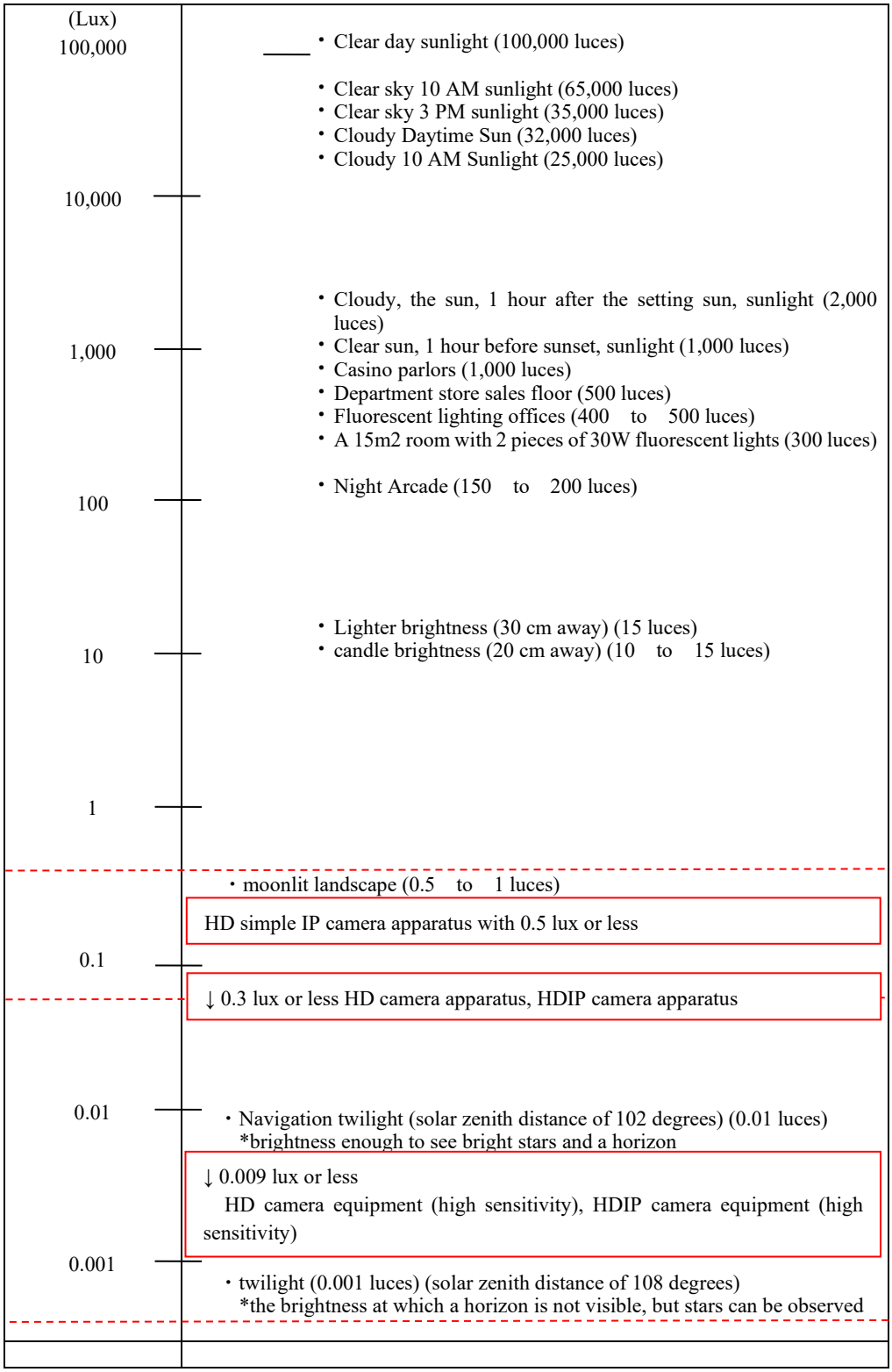
It is necessary to consider the possibility of night monitoring, when facilities monitoring, and space monitoring are carried out by CCTV camera. As the MCGS will install the site lighting, the illuminance of the moonlight level is ensured.

For reference, **Table 7.3.141** shows the standard of brightness. Since "moonlit landscape" is equivalent to 0.5 lux, nighttime monitoring is possible without a floodlight even in the present

condition, if the minimum subject illuminance is 0.5 lux or less.

When the minimum subject illuminance is checked in the CCTV camera equipment specifications (draft) (September 2018), a camera that meets the conditions of 0.5 lux or less is equivalent to a swivel camera. Therefore, this design selects "swivel camera".

Table 7.3.142 Standard for brightness



Source: Study Team

3) Model Selection

Though the performance (minimum subject illuminance) of the HD simple type IP camera equipment is inferior to other two models, it is possible to ensure the brightness in the environment where the outside illumination is installed, and it has an excellent economic efficiency.

Considering economic efficiency, HD simple type IP camera equipment shall be adopted.

Table 7.3.143 Comparison of IP Camera Equipment

Item			HDIP Camera Device	HDIP Camera Device (High Sensitivity)	HD Simple IP Camera Apparatus
performance (minimum subject illuminance)		Electron Sensitivity OFF	0.3 Lx or less	0.009 Lx or less	0.5 Lx or less
		Electron Sensitivity ON	0.02 Lx or less	0.0003 Lx or less	−
Economy			4.2 million yen	7 million yen	1.9 million yen
	Breakdown	Camera Apparatus	(3.2 million yen)	(5.5 million yen)	(1.1 million yen)
		Machine Side Device	(1 million yen)	(1.5 million yen)	(800,000 yen)
Evaluation			△	×	○

Source: Study Team

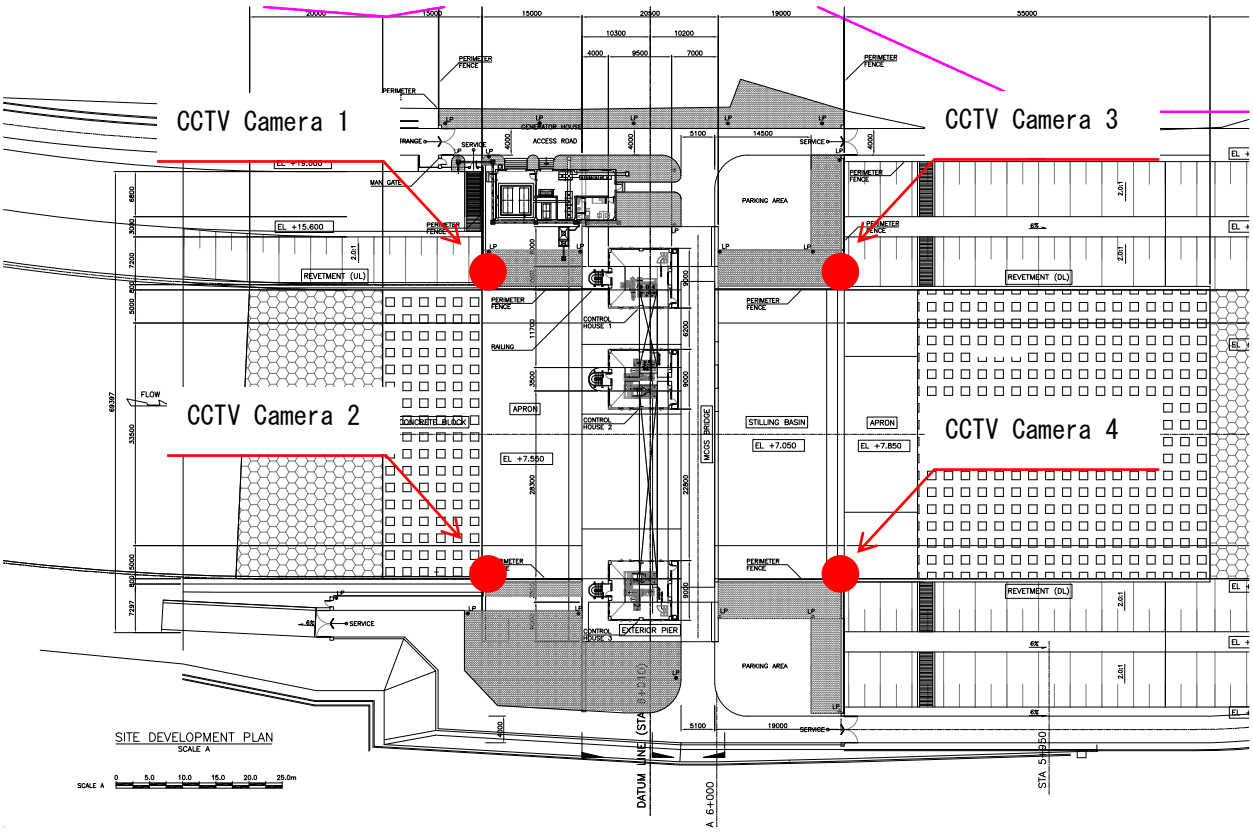
(4) Study of Monitoring Equipment Layout

The monitoring equipment installed in MCGS is arranged according to the following concept. The selected CCTV camera is a turning type, and it is possible to hoist the monitoring object (gate ⇔ space) by turning the camera.

Table 7.3.144 Arrangement of the Monitoring Facilities (MCGS)

Monitoring Objects	Monitoring Classification	Quantity	Allocation Criteria
Small Span Gate	Facility Monitoring	1	The camera will be installed near the small span gate and inside the fences at the seawall. (Marking locations of CCTV Camera 1 and CCTV Camera 3 in **Figure 7.3.135**) By turning, the camera 1 can monitor the space downstream of Marikina River, and camera 3 can monitor the space upstream of Marikina River.
Large Span Gate	Facility Monitoring	1	The camera will be installed near the large span gate and inside the fences at the seawall. (Marking locations of CCTV Camera 2 and CCTV Camera 4 in **Figure 7.3.135**) By turning, the camera 2 can monitor the space downstream of Marikina River, and camera 4 can monitor the space upstream of Marikina River.
Upper Marikina River	Spatial Monitoring	1	Same as the small span gates and large span gates
Lower Marikina River	Spatial Monitoring	1	Same as to the small-span gates and large-span gates

Source: Study Team

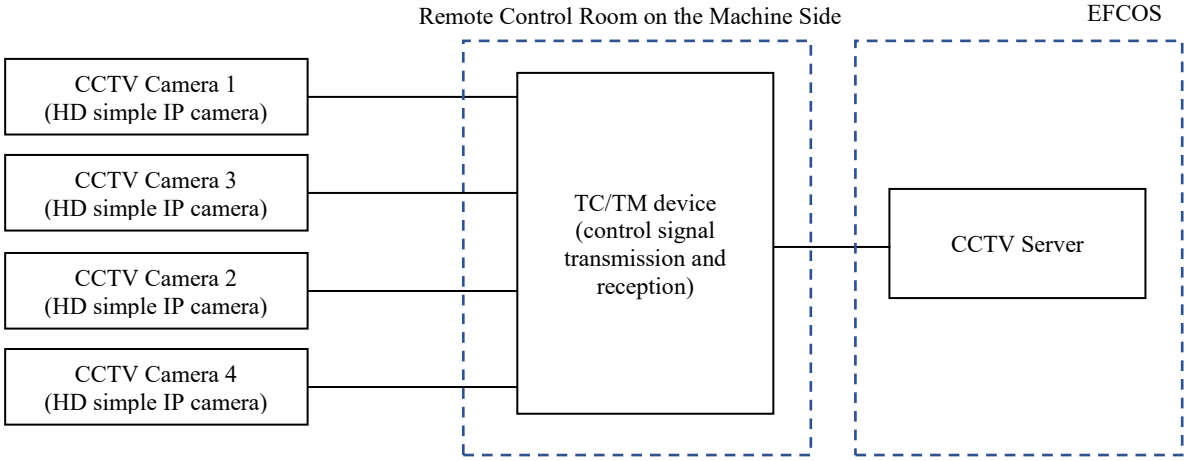


Source: Study Team

Figure 7.3.135 Position of Camera Equipment

(5) Configuration of Monitoring Equipment

The configuration of monitoring equipment is shown in **Figure 7.3.136**.



Source: Study Team

Figure 7.3.136 Configuration of Monitoring Equipment

(6) Equipment Specification

1) CCTV Camera

(a) Summary

The camera takes images of the river space and management facilities. A webcam that can be connected to an IP network shall have the following specifications and functions.

(b) CCTV Camera Specifications

(a)	Type	Dome type camera integrated with outgate housing
(b)	Imaging element	1/4 type Interline transfer type CCD
(c)	Effective pixels	2 million pixels or more
(d)	Min Illuminance	0.5 lx (Color Mode) F 1.2 0.01 lx (black and white mode) F 1.2
(e)	Structure	IPX5 (jet proof type)
(f)	Horizontal rotation range	360 ° (endless turn)
(g)	Vertical rotation range	± 90 ° (From a horizontal plane)
(h)	Zoom ratio	30 (When using electronic zoom: about 12 times)
(i)	angle of view	Horizontal: 41° to 5° Vertical: 31° to 4°
(j)	Presets	256 presets

(c) IP Encoding Specification

(a)	Image Encoding	H. 264
(b)	Video distribution method	multicast delivery
(c)	Multiplex system	Mpeg 2 system Tts The stream specifications shall conform to the following specifications. CCTV camera equipment specifications (draft) September 2018 Ministry of Land, Infrastructure and Transport
(d)	Frame rate	29.97 fps (Fixed)
(k)	Output	100 Base TX Ethernet

(d) Functional Specification

(a)	camera control function	Remote control from a CCTV control server. Control specifications shall conform to the following specifications. CCTV facility control interface specification (draft) January 2017: Ministry of Land, Infrastructure, Transport and Tourism
(b)	Others	・To have an optical shake correction function. ・Human detection and voice detection are built-in as intelligent video analysis programs.

2) CCTV Server

(a) Summary

The CCTV server is a device having a control function for controlling a CCTV camera and displaying a CCTV camera image.

(b) Equipment Specification

(a)	Type	Rackmount
(b)	CPU	1 or more CPU of Xeon Gold 5122 (4 Cores/3.6 GHz)
(c)	Memory	32 GB or more
(d)	HDD	3 or more HDD of 300 GB (RAID 1 + Hotspare)
(e)	LAN	2 or more LAN ports (1 Gbps)
(f)	Optical drive	Internal DVD drive unit
(g)	Others	Power supply units shall be redundant.

(c) Functional Specification

(a) Camera selection function (List/MAP)
(b) Camera control function (PTZW)

Remote control to the CCTV camera is possible.

Control specifications shall conform to the following.

"CCTV facility control interface specification (draft) January 2017 Ministry of Land, Infrastructure and Transport"

(c) Video selection display/ control screen (Intra Web)

7.3.6.5 Remote Monitoring and Control Facility

(1) Control Items and Control Functions

1) Control Items

Control items related to various instrumentation equipment, alarm facility, monitoring equipment, and gate equipment targeted at remote monitoring and control equipment are shown in Table7.3.138 to Table7.3.140. Since the remote monitoring control equipment operates MCGS and backflow prevention floodgate (Cainta sluice gate/Taytay sluice gate), this paper arranges control items for all facilities.

Table 7.3.145 List of Control Items (MCGS)

Station name (Name of the facility)	facility classification	control item	amount of data: number of units	amount of data: Data volume per unit	amount of data: Total	data I/O delivery conditions: input-output signal	data I/O delivery conditions: number of digits	data I/O delivery conditions: minimum unit	data I/O delivery conditions: measurement range	transmission s: direct delivery	transmission s: Deliveries (telemeter)	Site (cockpit control room): Show: Digital (Numeric)	Site (cockpit control room): Show: Analog (instrument type)	Site (cockpit control room): Show: Lamp (indicating lamp)	Site (cockpit control room): operation and control: manual operation	Site (cockpit control room): operation and control: manual set value control	Site (cockpit control room): operation and control: automatic control	Generator building/control room/EFCOS: large display unit: Show: Digital (Numeric)	Generator building/control room/EFCOS: large display unit: Show: Analog (instrument type)	Generator building/control room/EFCOS: large display unit: Show: Symbol	Generator building/control room/EFCOS: console: Show: Digital (Numeric)	Generator building/control room/EFCOS: console: Show: Analog (instrument type)	Generator building/control room/EFCOS: console: Show: Symbol	Generator building/control room/EFCOS: console: operation and control: manual operation	Generator building/control room/EFCOS: console: operation and control: manual set value control	Generator building/control room/EFCOS: Integrated Management AP Server/Integrated Management Control Server: Display	operation and control: manual operation	operation and control: manual set value control	operation and control: constant setting	operation Processing: arithmetic processing	operation Processing: aggregate value processing	automatic control: automatic control processing	automatic control: operation quantity calculation	alarm Process: alarm	alarm Process: audible alarm	record processing: daily report	record processing: monthly report record	record processing: notification alarm record	record processing: operation record	information: XML Delivery	information: browser delivery	information: mail delivery	Remarks
MCGS	instru-mentation	upper Marikina River	2	1	2	optical signal	5	0.01	0 ~ 20.0 m	○		△						◎			◎					◎				◎	◎					◎	◎				◎		
		Lower Marikina River	2	1	2	optical signal	5	0.01	0 ~ 20.0 m	○		△						◎			◎					◎				◎	◎					◎	◎				◎		
	Monitoring	CCTV Camera 1	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 2	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 3	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 4	1	1	1	H. 264				○											◎					◎														◎		
	alarm	siren	1	1	1	contact point				○				△										◎		◎	◎							◎	◎				◎		◎		B
		Speakers	4	1	4	contact point				○				△										◎		◎	◎							◎	◎				◎		◎		B
		Sound collection microphone	4	1	4	contact point				○				△										◎		◎	◎							◎					◎		◎		
		warning light	4	1	4	contact point				○				△										◎		◎	◎							◎					◎		◎		
	Gate (small and large span)	Operation items and control signals	2	1	2	contact point				○					△									◎			◎												◎				
		Open (descent)	2	1	2	contact point				○					△									◎			◎												◎				
		Stopped	2	1	2	contact point				○					△									◎																			
		alarm stop	2	1	2	contact point				○					△									◎																			
		lamp test	2	1	2	contact point				○					△									◎																			
		emergency stop	2	1	2	contact point				○					△									◎			◎												◎				
		fault reset	2	1	2	contact point				○					△									◎			◎												◎				
		gate status, operation display, and monitoring signal	2	1	2	contact point				○				△									◎			◎												◎		◎		
		AC controlled power supply	2	1	2	contact point				○				△									◎			◎												◎		◎		
		Remote	2	1	2	contact point				○				△									◎			◎												◎		◎		
		Local	2	1	2	contact point				○				△									◎			◎												◎		◎		
		normal operation	2	1	2	contact point				○				△									◎			◎												◎		◎		
		maintenance operation	2	1	2	contact point				○				△									◎			◎												◎		◎		
		normal circuit	2	1	2	contact point				○				△									◎			◎												◎		◎		
		emergency circuit	2	1	2	contact point				○				△									◎			◎												◎		◎		
		Open (rise)	2	1	2	contact point				○				△								◎			◎												◎		◎		
		Closed (descent)	2	1	2	contact point				○				△								◎			◎												◎		◎		
		Stopped	2	1	2	contact point				○				△									◎			◎												◎				
		gate pause	2	1	2	contact point				○				△									◎			◎												◎				
		full opening	2	1	2	contact point				○				△								◎			◎												◎		◎		
		fully closed	2	1	2	contact point				○				△								◎			◎												◎		◎		
		hook withdrawal upper limit	2	1	2	contact point				○				△									◎			◎												◎		◎		
		Hook Deflection Lower Limit	2	1	2	contact point				○				△									◎			◎												◎		◎		
		gate operation	2	1	2	contact point				○				△								◎			◎												◎		◎		
		gate fault indication/ monitoring signal 3E operation	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		rope overload	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		slack rope	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		hydraulic brake overload	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		electromagnetic brake overload	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		extreme upper limit	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		emergency stop	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		contact welding	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		MCCB Trip	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		Heavy failure, B
		electrical leakage	2	1	2	contact point				○				△									◎			◎									◎	◎				◎		◎		minor fault, B

Source: Survey team

(※ 1) Audible alarm legend C : Chime (ping-pong) , B : Buzzer (boo-boo-hoo) , E : Electrophone (Poro Poro)

(※ 2) XML distribution refers to the distribution of data in the form of an XML file to a related organization or device. The data format to be distributed is determined through consultation.
Browser distribution refers to the browsing of data by an operator or administrator from a remote location via Internet Explore on a personal computer or a smartphone.
Mail delivery refers to the function of sending an abnormality notification mail from a mobile phone of an operator or a manager when a failure occurs.

Table 7.3.146 List of Control Items (Cainta)

Station name (Name of the facility)	facility classification	control item	amount of data I/O delivery conditions: number of units	Data volume per unit	Total	input-output signal	number of digits	minimum unit	measurement range	transmission system: direct delivery	Deliveries (telemeter)	Site (cockpit control room): Show: Digital (Numeric)	Analog (instrument type)	Lamp (indicating lamp)	operation and control: manual operation	manual set value control	automatic control	Generator building/control room/EFCOS: large display unit: Show: Digital (Numeric)	Analog (instrument type)	Symbol	console: Show: Digital (Numeric)	Analog (instrument type)	Symbol	operation and control: manual operation	manual set value control	Integrated Management AP Server/Integrated Management Control Server: Display	operation and control: manual operation	manual set value control	constant setting	operation Processing: arithmetic processing	aggregate value processing	automatic control: automatic control processing	operation quantity calculation	alarm Processing: alarm	audible alarm	record processing: daily report	monthly report record	notification alarm record	operation record	information: XML Delivery	browser delivery	mail delivery	Remarks
Cainta	instr	upper Marikina River	1	1	1	optical signal	5	0.01	0 ~ 20.0 m	○		△						◎			◎					◎				◎	◎					◎	◎				◎			
	instr	Lower Marikina River	1	1	1	optical signal	5	0.01	0 ~ 20.0 m	○		△						◎			◎					◎				◎	◎					◎	◎				◎			
	Monitorin	CCTV Camera 1	1	1	1	H. 264				○											◎					◎														◎			
	Monitorin	CCTV Camera 2	1	1	1	H. 264				○											◎					◎														◎			
	Monitorin	CCTV Camera 3	1	1	1	H. 264				○											◎					◎														◎			
	Monitorin	CCTV Camera 4	1	1	1	H. 264				○											◎					◎														◎			
	alarm	siren	1	1	1	contact point				○				△										◎		◎	◎							◎	◎				◎		◎		B
	alarm	Speakers	2	1	2	contact point				○				△										◎		◎	◎							◎	◎				◎		◎		B
	alarm	Sound collection microphone	2	1	2	contact point				○				△										◎		◎	◎							◎					◎		◎		
	alarm	warning light	2	1	2	contact point				○				△										◎		◎	◎							◎					◎		◎		
	Gate (small and large span)	Operation items and	2	1	2	contact point				○					△									◎			◎												◎				
	Gate (small and large span)	Open (descent)	2	1	2	contact point				○					△									◎			◎												◎				
	Gate (small and large span)	Stopped	2	1	2	contact point				○					△									◎																			
	Gate (small and large span)	alarm stop	2	1	2	contact point				○					△									◎																			
	Gate (small and large span)	lamp test	2	1	2	contact point				○					△									◎																			
	Gate (small and large span)	emergency stop	2	1	2	contact point				○					△									◎			◎													◎				
	Gate (small and large span)	fault reset	2	1	2	contact point				○					△																												
	Gate (small and large span)	gate status, operation display, and monitoring signal	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	AC controlled power supply	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	Remote	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	Local	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	normal operation	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	maintenance operation	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	normal circuit	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	emergency circuit	2	1	2	contact point				○				△							◎					◎														◎		◎		
	Gate (small and large span)	Open (rise)	2	1	2	contact point				○				△						◎					◎														◎		◎		
	Gate (small and large span)	Closed (descent)	2	1	2	contact point				○				△						◎					◎														◎		◎		
	Gate (small and large span)	Stopped	2	1	2	contact point				○				△											◎														◎				
	Gate (small and large span)	gate pause	2	1	2	contact point				○				△											◎														◎				
	Gate (small and large span)	full opening	2	1	2	contact point				○				△						◎					◎														◎		◎		
	Gate (small and large span)	fully closed	2	1	2	contact point				○				△						◎					◎														◎		◎		
	Gate (small and large span)	hook withdrawal upper limit	2	1	2	contact point				○				△											◎														◎		◎		
	Gate (small and large span)	Hook Deflection Lower Limit	2	1	2	contact point				○				△											◎														◎		◎		
	Gate (small and large span)	gate operation	2	1	2	contact point				○				△						◎					◎														◎		◎		
	Gate (small and large span)	gate fault indication/ monitoring signal 3E operation	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	rope overload	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	slack rope	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	hydraulic brake overload	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	electromagnetic brake overload	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	extreme upper limit	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	emergency stop	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	contact welding	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	MCCB Trip	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		Heavy failure, B
	Gate (small and large span)	electrical leakage	2	1	2	contact point				○				△											◎										◎	◎				◎		◎		minor fault, B

(※ 1) Audible alarm legend C : Chime (ping-pong) , B : Buzzer (boo-boo-hoo) , E : Electrophone (Poro Poro)

(※ 2) XML distribution refers to the distribution of data in the form of an XML file to a related organization or device. The data format to be distributed is determined through consultation.
Browser distribution refers to the browsing of data by an operator or administrator from a remote location via Internet Explore on a personal computer or a smartphone.
Mail delivery refers to the function of sending an abnormality notification mail from a mobile phone of an operator or a manager when a failure occurs.

Table 7.3.147 List of Control Items (Taytay)

Station name (Name of the facility)	facility classification	control item	amount of data I/O delivery conditions: number of units	Data volume per unit	Total	input-output signal	number of digits	minimum unit	measurement range	transmission system: direct delivery	Deliveries (telemeter)	Site (cockpit control room): Show: Digital (Numeric)	Analog (instrument type)	Lamp (indicating lamp)	operation and control: manual operation	manual set value control	automatic control	Generator building/control room/EFCOS: large display unit: Show: Digital (Numeric)	Analog (instrument type)	Symbol	console: Show: Digital (Numeric)	Analog (instrument type)	Symbol	operation and control: manual operation	manual set value control	Integrated Management AP Server/Integrated Management Control Server: Display	ration and cor: manual operation	manual set value control	constant setting	operation Processing: arithmetic processing	aggregate value processing	automatic control: automatic control processing	operation quantity calculation	alarm Processing: alarm	audible alarm	record processing: daily report	monthly report record	notification alarm record	operation record	information: XML Delivery	browser delivery	mail delivery	Remarks
Taytay	instrumentation	Upper Cainta floodgate	1	1	1	optical signal	5	0.01	0 ~ 20.0 m	○		△						◎			◎					◎				◎	◎					◎	◎				◎		
		Lower Cainta sluice	1	1	1	optical signal	5	0.01	0 ~ 20.0 m	○		△						◎			◎					◎				◎	◎					◎	◎				◎		
	Monitoring	CCTV Camera 1	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 2	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 3	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 4	1	1	1	H. 264				○											◎					◎														◎		
		CCTV Camera 5	1	1	1	H. 264				○											◎					◎														◎		
	alarm	siren	0	1	0	contact point				○				△										◎		◎	◎							◎	◎				◎	◎		
		Speakers	1	1	1	contact point				○				△										◎		◎	◎							◎	◎				◎	◎		B
		Sound collection microphone	1	1	1	contact point				○				△										◎		◎	◎							◎					◎	◎		
		warning light	2	1	2	contact point				○				△										◎		◎	◎							◎					◎	◎		
	Gate (Nos. 1 to 3)	Operation items and control signals	3	1	3	contact point				○					△									◎			◎												◎		
		Open (descent)	3	1	3	contact point				○					△									◎			◎												◎		
		Stopped	3	1	3	contact point				○					△									◎																		
		alarm stop	3	1	3	contact point				○					△									◎																		
		lamp test	3	1	3	contact point				○					△									◎																		
		emergency stop	3	1	3	contact point				○					△									◎			◎												◎		
		fault reset	3	1	3	contact point				○					△										◎																		
		gate status, operation display, and	3	1	3	contact point				○				△									◎				◎												◎		
		AC controlled power supply	3	1	3	contact point				○				△									◎				◎												◎		
		Remote	3	1	3	contact point				○				△									◎				◎												◎		
		Local	3	1	3	contact point				○				△									◎				◎												◎		
		normal operation	3	1	3	contact point				○				△									◎				◎												◎		
		maintenance operation	3	1	3	contact point				○				△									◎				◎												◎		
		normal circuit	3	1	3	contact point				○				△									◎				◎												◎		
		emergency circuit	3	1	3	contact point				○				△									◎				◎												◎		
		Open (rise)	3	1	3	contact point				○				△							◎			◎			◎												◎		
		Closed (descent)	3	1	3	contact point				○				△							◎			◎			◎												◎		
		Stopped	3	1	3	contact point				○				△										◎			◎												◎		
		gate pause	3	1	3	contact point				○				△										◎			◎												◎		
		full opening	3	1	3	contact point				○				△							◎			◎			◎												◎		
		fully closed	3	1	3	contact point				○				△							◎			◎			◎												◎		
		hook withdrawal upper limit	3	1	3	contact point				○				△										◎			◎												◎		
		Hook Deflection Lower Limit	3	1	3	contact point				○				△										◎			◎												◎		
		gate operation	3	1	3	contact point				○				△							◎			◎			◎												◎		
		gate fault indication/monitoring signal	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		rope overload	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		slack rope	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		hydraulic brake overload	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		electromagnetic brake overload	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		extreme upper limit	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		emergency stop	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		contact welding	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		MCCB Trip	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		Heavy failure, B
		electrical leakage	3	1	3	contact point				○				△										◎			◎							◎	◎				◎	◎		minor fault, B

(※ 1) Audible alarm legend C : Chime (ping-pong) , B : Buzzer (boo-boo-hoo) , E : Electrophone (Poro Poro)

(※ 2) XML distribution refers to the distribution of data in the form of an XML file to a related organization or device. The data format to be distributed is determined through consultation.
Browser distribution refers to the browsing of data by an operator or administrator from a remote location via Internet Explore on a personal computer or a smartphone.
Mail delivery refers to the function of sending an abnormality notification mail from a mobile phone of an operator or a manager when a failure occurs.

Table 7.3.148 Operation Items and Control Signals

Operation items and control signals (local control panel)

Item	Local control panel (Display)	control signal		Note
		presence or absence of signals	signal type	
Open (rise) Operation	○	○	continuous output	
Close (descent) operation	○	○	continuous output	
Stop	○	-		
Alarm stop	○	-		
Lamp test	○	-		
Emergency stop	○	○	pulse signal	
Fault reset	○	-		

Source: Study Team

Table 7.3.149 Gate status and Operation Display Items, Monitoring Signals

Gate status and operation display items and monitoring signals

Item	Local control panel (Display)	supervisory signal	
		presence or absence of signals	signal type
Power source	○	○	continuous output
AC controlled power supply	○	○	continuous output
Remote	○	○	continuous output
Local	○	○	continuous output
Normal operation	○	○	continuous output
Maintenance operation	○	○	continuous output
normal circuit	○	○	continuous output
Emergency circuit	○	○	continuous output
Open (rise)	○	○	continuous output
Close (descent)	○	○	continuous output
Stop	○		
Gate pause	○		
Full open	○	○	continuous output
Full close	○	○	continuous output
Hook take-off upper limit	○	○	continuous output
Hook take-off Lower Limit	○	○	continuous output
Gate operation	○	○	continuous output

Source: Study Team

Table 7.3.150 Gate Fault Indication Items And Monitoring Signals

Gate fault indication items and monitoring signals

Item	Local operation panel (Display) failure category		supervisory signal	
	severe failure	minor fault	presence or absence of signals	signal form
3E operation	○		○	continuous output
Rope overload	○		○	continuous output
Slack rope	○		○	continuous output
Hydraulic brake overload	○		○	continuous output
Electromagnetic brake overload	○		○	continuous output
Emergency upper limit	○		○	continuous output
Emergency stop	○		○	continuous output
Contact welding	○		○	continuous output
MCCB Trip	○		○	continuous output
Electrical leakage		○	○	continuous output

Source: Study Team

2) Control Function

Remote monitoring and control equipment are summarized in **Table 7.3.151**. Details of each function are described below.

Table 7.3.151 Control Functions to be Provided in Control Facilities

Function Name	Feature Summary
Systems Management	Execution management of various processing programs constituting software, program management for operation management of hardware, time and schedule management, event (alarm and notification) management. are performed.
Data Collection	Instrumentation data (water level) and gate opening data are collected and used as basic data for facility operation.
Alarm Determination	The decision processing for the alarm notification is carried out using instrumentation data (water level) and SV signal outputted from the mechanical equipment, and the alarm is sounded.
Operation	The discharge is calculated using instrumentation data (water level).
Display for Monitoring	Various contents for system monitoring are prepared and provided to users through Web screens.
Data Storage	Instrumentation data, operation data, system operation record, . are stored in a magnetic disk device and an auxiliary storage device in a system device.
Maintenance	The following are carried out: Compensation and correction processing of abnormal value, data substitution.

Source: Study Team

(a) Systems Management

(i) Program Management

It performs an initial start processing and stop processing of a system and operation information transmission/reception processing of a program on operation conditions between various programs.

(ii) Buffer Management

It manages a common area memory used between various programs.

(iii) RAS Management

Operation information of various programs is transmitted and received, and notification messages from multiple applications are tracked and displayed. In addition, a process for monitoring the state of the distributed processing apparatus associated with the processing apparatus is performed. (RAS stands for Reliability, Availability and Serviceability. It expresses a measure of the total reliability of the system.)

(iv) Time and Schedule Management

It processes tasks such as a periodic start and scheduled starts of various programs.

(v) Business Processing Sequence Management

It performs processing such as a distribution and an integrated control of various program operation information and operation sequence managements.

(vi) Event Management

It stores changes and history information of an event.

(b) Data Collection

(i) Water Level Data

The value for each positive minute is extracted from the primary smooth water storage level processed in the input/output processing of th slave unit. It is used as data subjected to the secondary smoothing processing in this processing.

(ii) Opening Data

The opening and discharge information of each minute are extracted from the measured values processed in the input/output processor of th slave unit.

(c) Alarm Determination

According to the observation data, it is decided if an alarm notification is required. If yes, an alarm sounding is carried out. The items for decision are as follows. Each constant in the data can be changed arbitrarily by the user.

Table 7.3.152 Condition for Alarm Sounding

Category	Items	Judgment Condition
Water Level	Upper limit	Water Level ≧ Upper Limit Set Value
	Lower limit	Water Level ≦ Upper Limit Set Value
Discharge	Upper limit	Discharge ≧ Upper Limit Set Value
	Lower limit	Discharge ≦ Upper Limit Set Value
Gate Sv	ON	Severe Failure SV ON
Common	Missing Measurement	Input Data Missing

Source: Study Team

3) Operation

(a) Water Level Calculation

The water level *Hri* is the water level input by each station.

Hri : 10-minute fixed water level of station *i* [m]

(b) Discharge Calculation

Discharge *Qri* (m^3/s) is calculated from the water-level-flow relationship specified for each station. The discharge calculation formula shall be prepared with several sets of calculation constants in accordance with the characteristics of the river channel so that it can be hoisted for each water level.

$$Qri\ (m) = Cij \cdot (Hri\ (m) - dij)\ 2\ [m^3/s]$$

However, Qri : river discharge of station *i* [m^3/s]

Cij, dij : *j* th approximation constant of station *i*

4) Display for Monitoring

The following various contents shall be created and provided.

① general view of water level and discharge of the entire basin
② Water level table/graph (MCGS, Cainta, Taytay)
③ machine status list
④ alarm history
⑤ Data output (CSV Output)
⑥ Maintenance Items (Alarm constant/arithmetic constant/data substitution)
⑦ Data Fill/Correction
⑧ control screen 1 (Loudspeakers/Sirens/Alarm Lights)
⑨ Control screen 2 (gate facility)
⑩ CCTV screen at each point
⑪ Large monitor display method setting (IPch control)

5) Accumulation

The following methods shall be used.

(a) The Online Files

A method of reading and writing data online using the auxiliary storage device (Mainly magnetic disk devices) of an integrated management server. Data exceeding the retention period will be deleted from the old period.

Table7.3.153 Online Retention Period for Each Data

Item	Retention Period
Fixed Time (10 minutes) Value Data	180 days
Hour Data	2 years
Error and Judgment Record Data	20,000 projects

Source: Study Team

(b) Offline Files

A method of storing data on an external storage medium using an auxiliary storage device connected to an integrated management server. The offline file recording is carried out when the data is stored for a long period of time or when the stored data is utilized in another system. In offline files, information such as file type, data items, the period to be stored are specified and stored in the external storage medium. The external medium will be DVDs, .

As for the data to be stored in the external storage medium in the offline file recording, the screen display data (fixed time) shall be stored in CSV format.

6) Maintenance

(a) Fill/Adjustment

Data compensation and correction should be possible in the event of equipment failure, missing data, abnormal value detection. A monitoring PC shall perform compensation and correction work with access control.

The target data shall be fixed time data and regular time data and is recalculated after data replenishment or correction.

(b) Data Substitution

In order to prevent failure of water gauge and gate opening and missing data due to inspection, it has a function to substitute data. The following data shall be substituted.

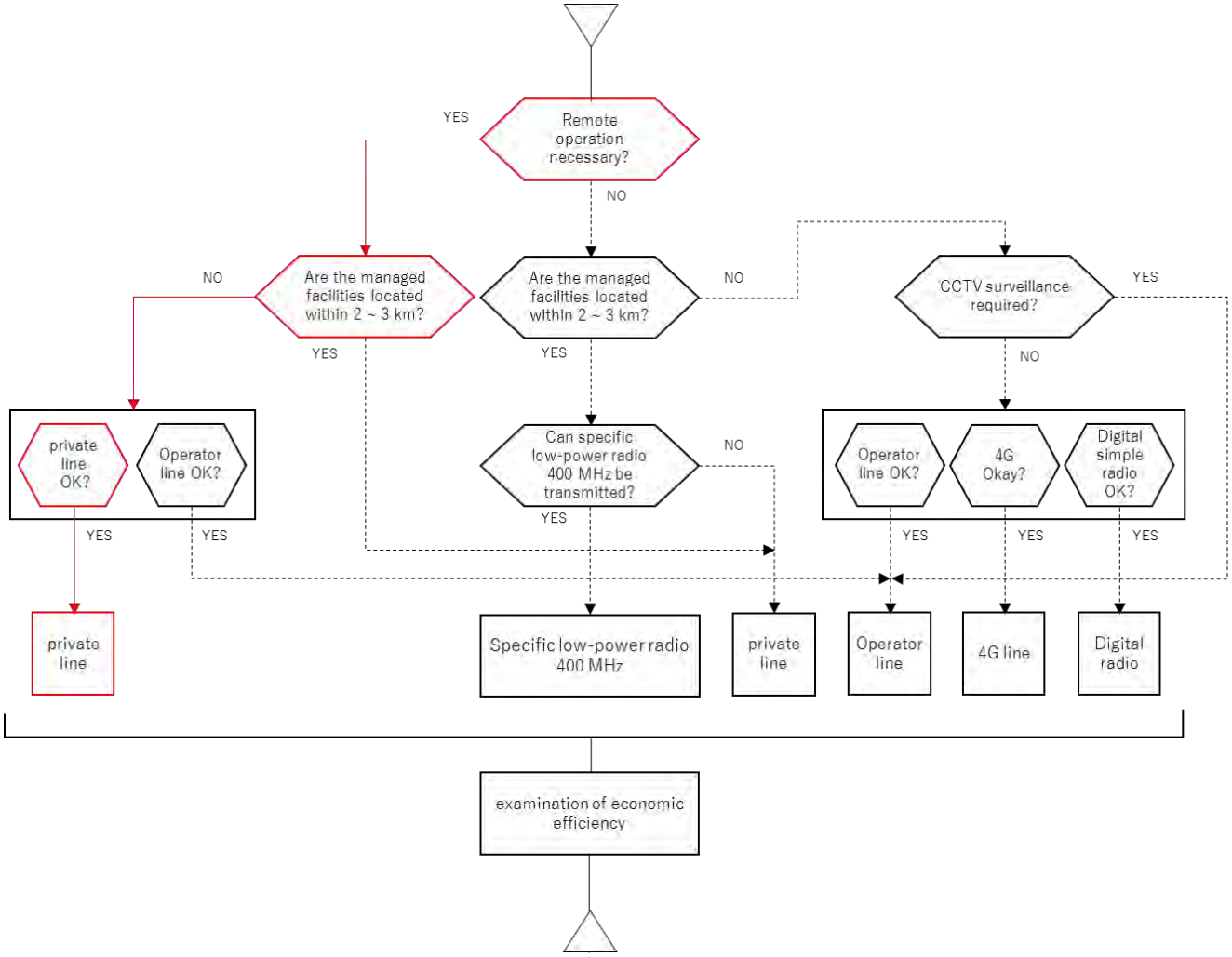
MCGS	Normal water level and secondary water level
	small span gate opening and large span gate opening
Cainta	Normal water level and secondary water level
	Gate openings No. 1 and No. 2
Taytay	Normal water level and secondary water level
	No. 1 gate opening, No. 2 gate opening, and No. 3 gate opening

(2) Examination of Transmission Paths (Network)

1) Determination of Transmission Path

The network is selected based on the use, economic efficiency, from the following lines which are generally used: "private line," "Operator Line (domestic carrier optical line)," "Cellular (4G line)", "Specific low-power radio (400 MHz)", and "Digital simple radio circuit." ,

As shown in **Figure 7.3.137**, a self-managed circuit is newly equipped in order to carry out more reliable data transmission.



Source: Study Team

Figure 7.3.137 Selection Flow of a Transmission Line

In MCGS, Cainta sluice, and Taytay sluice to be an object of this design, it is necessary to transmit instrumentation data (water level observation data), monitoring data (CCTV video footage), gate conditions (Operation display items, monitoring signals, and failure signals), . using EFCOS as a base. If we calculate the transmission bandwidth required here, **Table 7.3.154**. 120 Mbps for MCGS and 270 Mbps for Cainta/Taytay. From the above, an optical line shall be selected as a self-run line, and necessary equipment shall be selected.

Table 7.3.154 Estimation of Transmission Bandwidth

Section	Authority	Point	Transmission Volume (Presumption)	Remarks
1	EFCOS	MCGS	120 Mbps	
2	EFCOS	Cainta Sluice	270 Mbps	Include section 3 transmission volume
3	Cainta Sluice	Taytay Sluice Gate	150 Mbps	

Source: Study Team

2) Optical Cable Path

(a) Design Policy

(i) Buried Piping

Design policy was decided referring to the "Optical fiber cable installation procedure and explanation" (hereinafter, "construction procedure)".

A. Selection of Pipeline

The pipe type shall be selected by considering the mechanical load, strength, cable laying performance, workability, maintainability, and economy during construction and after completion.

As for the selection of pipe types, it is necessary to use different types depending on the installation site, . In general, these types shall be selected according to the criteria shown in **Table 7.3.155**.

Table 7.3.155 Pipeline Selection Standards

Installation Site / Pipeline Material Name	Abbreviation	Single Pipe System (ϕ 50)							Article
		Open Cutting			Non-Open Cutting		Attached Rack		
		Sidewalk Part	Roadway Section	Rump Shoulders and Slope	Tramway Overpass	Transverse Part Intersections, .	Bridge Section	Tunnel Section	
Steel Pipeline	PL-PS	◎	◎	○			○	○	
	PLC and PLP	○	◎	○	◎	◎	○	◎	
	SGP						○	○	Be treated to prevent corrosion
Synthetic Resin Pipe (rigid vinyl chloride pipe)	PV	◎	◎	◎	○	○	○		Hard vinyl chloride pipe for communication
	VP	◎	◎	◎	○	○	○		
	VE	◎	◎	◎	◎	◎	◎	◎	Rigid vinyl pipeline
	SVP	◎	◎	○	○	○			
Fiber-Reinforced Plastic Tube	FRP						◎	◎	
Reinforced Plastic Composite Tube	FRPM	○	○	○	◎	◎			100 mm or more
Corrugated Hard Polyethylene Tube	FEP	◎	○	◎				○	

Flexible Tube	Stainless Flexible Tube	SUS		○				◎	○	
	Synthetic Resin Flexible Pipe	PF CD						○	○	
Concrete Trough			○		○					

Note 1: The symbol ◎ indicates the optimum type, and the symbol ○ indicates the applicable type of pipe.
Note 2: Depending on the installation place, it is necessary to select the steel pipe with corrosion resistance.
Note 3: The description of synthetic resin pipes (PV, VP, VE, SVP, .) from this table is generically referred to as "rigid vinyl chloride pipe". When selecting this piping type, select the piping type suitable for the installation conditions.
Note 4: When laying corrugated rigid polyethylene pipes, due attention should be paid to the linearity of the pipes, since the pipes tend to rise and meander.
Note 5: Attention should be paid to the fact that this type of pipe to be applied differs depending on the jacking method according to the installation distance and soil quality.

Source: Study Team

[Annotation]
Material used for underground installation shall be JIS C 8305 "steel pipeline", JIS C 8430 "rigid vinyl pipeline", or pipeline material with equivalent or superior performance.
In order to ensure cable laying performance, pipeline materials having the following performance including joints shall be used in the selection of pipeline materials.

1. Passability : There are no projections on the inner surface, and the required inner diameter is maintained, so that it does not interfere with the installation and removal of cables.
2. Strength : To ensure required strength against the weight, earth pressure, ., of a vehicle, ., after burial, and to have sufficient strength against flatness.
3. water tightness : Soil and water must not enter the pipe.
4. impact resistance : The pipes buried directly in the ground shall have the required strength against impact during construction.
5. durability Materials must not deteriorate over a long period of time.
6. flame resistance: To be incombustible or self-extinguishing.
7. heat resistance : To ensure required strength against temperature change due to the influence of soil.
8. serviceability the repair material and construction method suitable for the pipeline material are secured in consideration of the repair due to accidents after installation.

B. Number of Lines in the Pipeline

The number of stripes is determined in consideration of the capacity of the optical fiber cable contained therein.

1. The number of lines for accommodating the optical fiber cable for the communication trunk line is determined in consideration of the allowable range of the cable occupancy rate with respect to the inner cross-sectional area of the line.
2. The composite cable and the power cable in which the optical fiber cable is housed together with the control cable are not housed in the same pipe.
3. The number of lines consists of the actual number of lines and the number of spare lines. It is desirable that at least 1 line should be constructed in consideration of the response to cable failure (cutting accident).
4. The actual number of stripes shall be the design quantity plus the estimated capacity of equipment increase.

C. Pipe Diameter in the Pipeline

① Among pipeline facilities, the diameter of a pipeline used in a single pipeline system differs depending on the linear shape, span length, and cable outside diameter, . However, it is desirable to select a cable with a diameter equal to or smaller than "Pipeline inner diameter ≥

1.5 × cable external diameter" when the selecting a standard pipe diameter for general construction. Therefore, the external diameter of the cable relative to the internal diameter of the pipeline is as follows.

Pipe Type and Pipe Diameter		External Diameter of Cable that can be Routed
Rigid PVC Pipes (PV)	ϕ 50 mm (Internal Diameter 56 mm)	37 mm or less
Rigid PVC Pipes (PV)	ϕ 75 mm (Internal Diameter 83 mm)	55 mm or less
Corrugated Rigid Polyethylene Pipe (FEP)	ϕ 50 mm (Internal Diameter 50 mm)	34 mm or less
Corrugated Rigid Polyethylene Pipe (FEP)	ϕ 80 mm (Internal Diameter 80 mm)	54 mm or less

Source: Study Team

② For the specifications of the core structure and external diameter of the optical fiber cable, refer to Construction Procedure 4 -2 -2 "classification of fiber optic cables".

The dimensions of the rigid vinyl chloride pipes and corrugated rigid polyethylene pipes used as standard piping materials are shown in **Table 7.3.156** and **Table 7.3.157**.

Table 7.3.156 Standard Dimensions of Rigid PVC Pipes (PV)

Nominal Diameter	ϕ 25	ϕ 50	ϕ 75
Internal Diameter (mm)	28.0	52.9	83.0
Outside Diameter (mm)	34.0	60.5	96.0
Internal Diameter Cross Section	615.6	2206.1	5410.6

Source: Study Team

Table 7.3.157 Standard Dimensions of Corrugated Rigid Polyethylene Tubes (FEP)

Nominal Diameter	ϕ 30	ϕ 50	ϕ 80
Internal Diameter (mm)	30.0	50.0	80.0
Outside Diameter (mm)	40.0	65.0	102.0
Internal Diameter Cross Section	706.5	1962.5	5024.0

Source: Study Team

D. Buried Position

I. The buried position of the pipeline in the underground part shall be determined in consideration of the safety, workability, and economical efficiency of the pipeline.

II. The position shall be such that there is little competition with other buried portions or structures and the specified depth and alignment can be ensured.

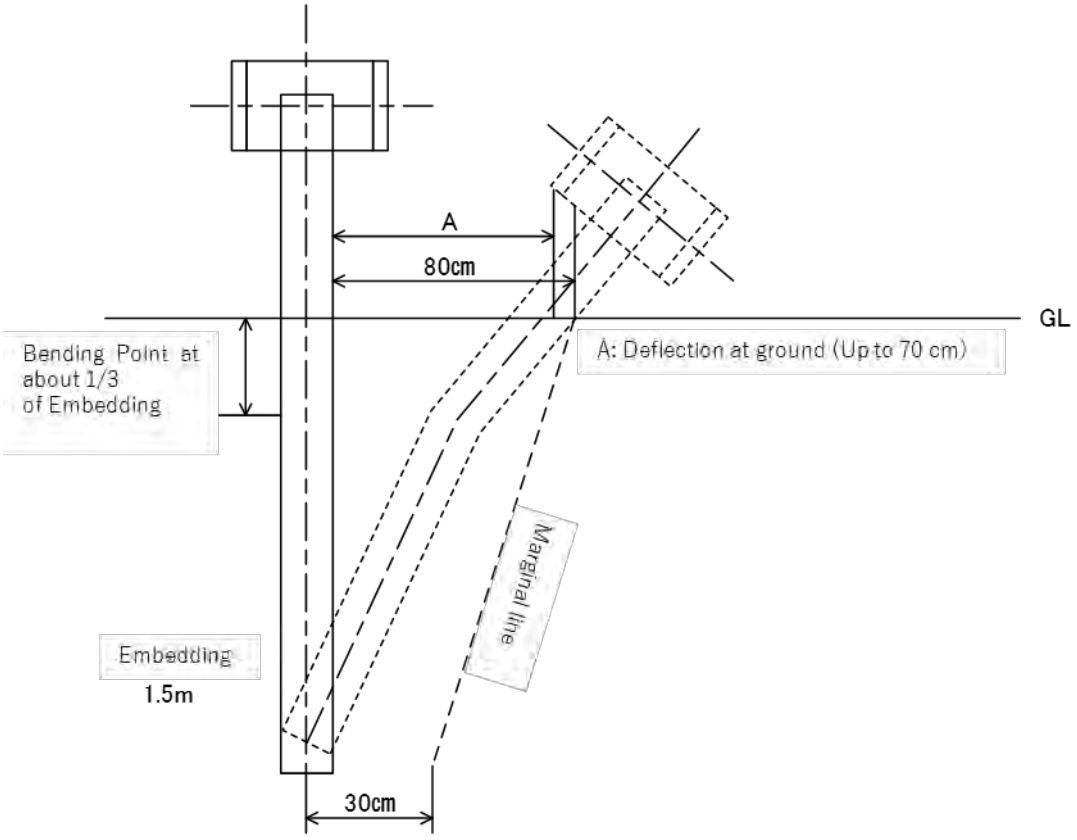
III. It is desirable that the pipeline is buried at a position where the construction can be carried out with the same cross section, so that the straight-line construction can be carried out.

IV. In burying at a road shoulder, the burying depth shall be lowered as deep as possible to a position where the pipeline is not damaged, assuming that a shoulder is damaged by heavy rain.

V. The location of buried pipelines is often determined in relation to other underground structures under the jurisdiction of the MLIT (Communications, electricity, water, gas, .), but in principle, the location is on the sidewalk, and in some circumstances, the location is also buried on the median of roadways, shoulders, .

VI. To avoid damage to the pipeline by hammering of guardrail posts by vehicle collisions, the buried position of the pipeline is, in principle, away from the limit line in the deformation of the guardrail post (**Figure 7.3.138**).

VII. The road-crossing pipelines shall cross at right angles to the road centerline.



Source: Study Team

Figure 7.3.138 Deformation of the Guardrail Post

E. Burial Depth

The burial depth of a pipe shall be designed in accordance with "Road Act Enforcement Order", "Technical standards for electrical equipment" and other guidelines.

Generally, the load on the buried pipe is divided into dead load, live load, and impact load. For

example, in a shallow buried pipe, the dead load such as earth pressure is ignorable, but when a running vehicle or a construction vehicle passes over the buried pipe, a large live load or impact load is generated.

In consideration of the above points, the standard burying depth of the pipelines shall refer to the following values (**Table 7.3.158)**.

As a preventive measure against cable cutting accidents, the location of the buried cable should be clearly indicated with a buried marker sheet.

Table 7.3.158 Standard Buried Depth of Pipeline

Burial Site		Minimum Buried Depth (References)	Remarks
Roadway		0.6 m or more (0.3 m + Pavement Thickness)	
Sidewalk		0.6 m or more	
Riverside	Embankment Crown Land Side Slope (Step) Land Side Slope (Foot)	0.3 m or more	
	High Water Channel Low Water Channel	1.0 m or more	

Note: To decide the depth of burial, further instructions may be received from each administrator. In such cases, decisions shall be made through negotiations and consultations with each administrator.
Source: Study Team

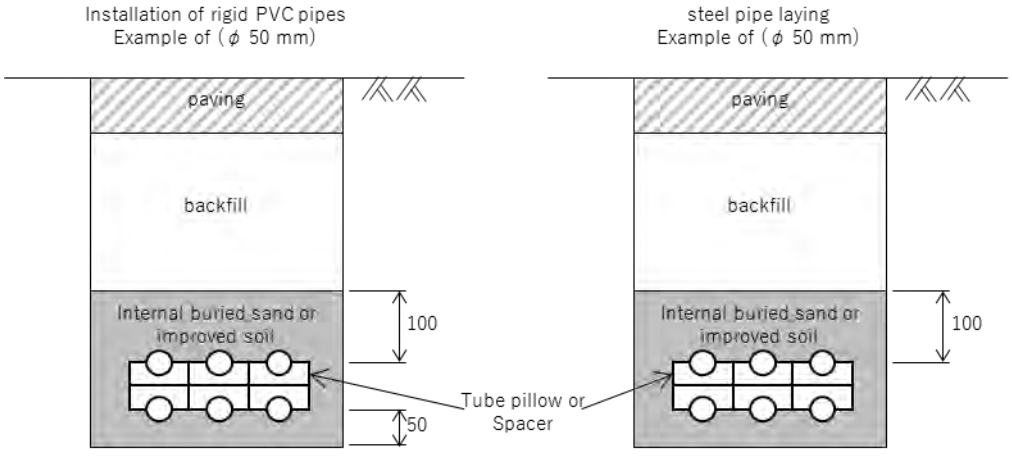
F. Arrangement Interval of Protective Soil and Piping

① Excavated section shall be such that the circumference of the pipe can be filled with sufficient protective soil (Medium buried sand and granular improved soil) and the uppermost part of the pipe shall satisfy the specified depth.

If the excavated ground is of good quality (sandy soil), the generated soil can be used for foundation work.

② When the pipeline is laid in multiple stages, a pipe pillow or spacer is installed to prevent the step collapse.

③ To a standard buried section is shown in **Figure 7.3.139**.



Source: Study Team

Figure 7.3.139 Standard Section of Buried Pipe (General Section)

G. Distance from Other Buried Objects

In case there are a communication line crosses or locates near the pipeline, or there are other regulations, the distance from other underground objects shall be set. In general, the following shall apply (**Table 7.3.159**).

Table 7.3.159 Distance from Other Buried Objects (Unit: cm)

Buried Object \ Positional Relation		Crossing	Parallel
Gas, Water and Sewage		15 or more	30 or more
Underground Pipeline Tube	Low Pressure and High Pressure	30 or more * 1	30 or more * 1
	Extra High Pressure	60 or more * 1	60 or more * 1
Orbit		100 or more	100 or more * 2

*1: The distance from the underground electric line is determined by "Electrical Equipment Technical Standards and Interpretation Article 139" and "Article 14 of the Wire Telecommunications Equipment Ordinance".
In the case of optical fiber cables and power protection communication lines , refer to paragraphs 1 to 5 of Article 139 of the "Technical standards and interpretations for electrical equipment) ".

*2: Distance parallel to the orbit shall be based on "Technical Requirements and Interpretation for Electrical Equipment Article 267".

Source: "Electrical Equipment Technical Standards and Interpretation (H. 9.6. 1 Implementation)" (Order of the Ministry of International Trade and Industry)

The type of voltage is shown in **Table 7.3.160**.

Table 7.3.160 Voltage Type

	Low Pressure	High Pressure	Extra high Pressure
DC	750 V or less	Above 750 V 7,000 V or less	Be greater than 7,000 V Voltage
AC	600 V or less	Above 600 V 7,000 V or less	

Source: Study Team

In the case of protecting underground-installed objects of other enterprises, the protective measures for the underground-installed objects shall be implemented though consultation with each manager.

(ii) Exposed Pipe

To determine the exposed piping system, "Construction procedure and explanation of optical fiber cable" and (hereinafter referred to as "construction procedure") were checked.

A. General

Exposed piping shall be selected considering the conditions of the installation site of the pipelines, in compliance with the relevant standards, in conformity with other structures, and without structural mechanical hindrance.

[Explanation]

When the underground burial method cannot be adopted for special locations such as a bridge attachment and a tunnel, exposed piping shall be used. When piping is exposed, follow the procedure below.

① Depending on the type of bridge to be attached (Steel bridges, PC bridges, concrete bridges, .), the attachment space, allowable attachment weight, rebar, sun cover, fire protection,

necessity of the bridge floor manhole, pipe type must be sufficiently examined to determine the attachment system.

② Depending on the tunnel type (Mountain, shield, .) for the exposed piping, the piping system shall be determined after careful consideration of the piping space (building limit) for the cross-section, the piping type, the piping support method, the necessity of the pull box, .
In the case of an exposed installation (Rack system, clip stop system, .), a sun cover and a fireproof protection work will be installed as necessary.

③ The weight of the future extension cable and sheath pipe is considered in the calculation of the support hardware in the case of the exposed piping in the bridge attachment, tunnel, .

B. Bridge Attachment

When designing a pipeline attached to a bridge, the type and structure of the bridge to be connected and the type, arrangement, number of lines, construction, and maintenance of the pipeline shall be considered. Also, the attachment position and attachment method should be selected in consideration of other occupancy items.

[Explanation]

Application of a bridge to be attached

The bridge to be attached shall be determined in conformity with the future concept of the infrastructure and in consideration of its stability, replacement and decommissioning plan.

① Select a bridge with less vibration.

② Avoid bridges constructed by DC electric railroads (Passage of trains) that may suffer from electric corrosion.

③ Avoid bridges to be rebuilt in the near future

④ Avoid bridges with damaged by floods in the past or bridges predicted to be damaged by floods.

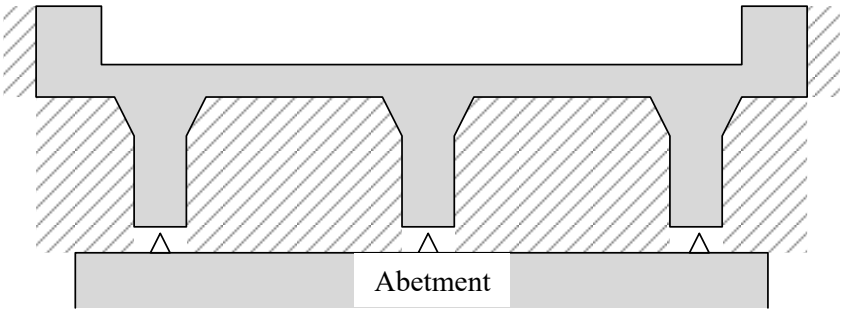
⑤ Avoid bridges that are not seismic-resistant (Including prevention of falling bridges).

If it is inevitable to attach a planned piping with an above-mentioned bridge, protective measures shall be taken.

C. Selection of Mounting Position

The piping should be installed on both sides of the girder of the road bridge or under the Bottom Slab to avid natural physical and chemical effects such as external force due to running water or direct sunlight during a flood.

①The following Figure shows the range of the road bridge at which the piping can be attached.



: Range that a pipeline can be attached

Figure 7.3.140 Range at which the Pipeline can be Attached to A Road Bridge

② When the pipeline is exposed to the outside of the bridge girder due to a side-attachment, . it should be attached on the downstream side as much as possible to make it safe against flooding.

③ Bridges are often designed in harmony with the sorrowing environment and scenery, the installed pipeline shall not damage the aesthetic appearance of bridges.

[Note]
When attaching the pipeline to the outer wall of the handrail, consult with the bridge administrator after enough consideration to beauty and security.

D. Selection of Attachment Type

The attachment type shall be selected after considering the construction and maintenance workability, type of bridge, structure, type of pipeline, arrangement, number of lines, and fire protection.

① To a diagram of attachment is shown below.

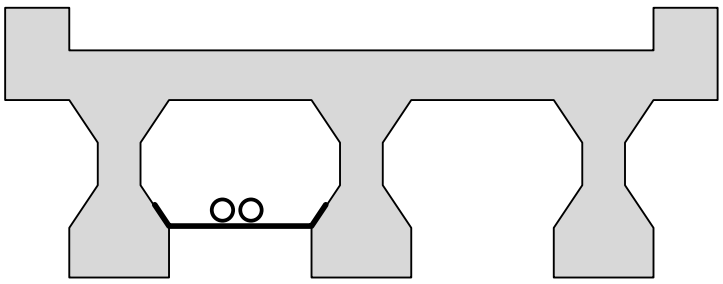
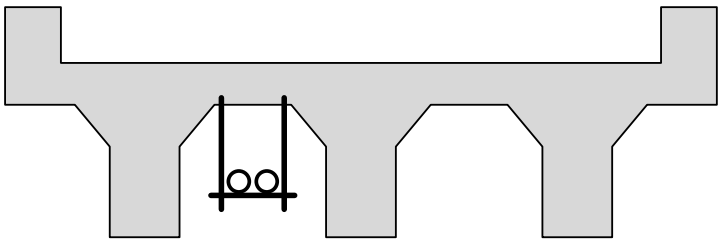
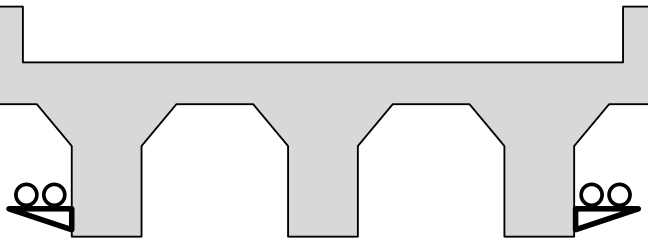
Format	Diagram
Beam Receiving System * Using girders on both sides	
Suspension System *Hanging from the Bottom Slab	
Bracket System * Using the sides of the girders	

Figure 7.3.141 Attached rack type

② In addition to use girders and Bottom Slabs, there is also a type in which the superstructure of a bridge (Pipe beam bridges, .) is used by utilizing abutments and girders of road bridges.

③ If the bridge has an asphalt-paved sidewalk, burying in the sidewalk is also an option.

However, it is necessary to examine if the pavement thickness is sufficient or if the pavement thickness can be added.

E. Attached Pipeline

Selection of pipe type

Rigid vinyl chloride pipes (VP), thick steel pipelines (GP), polyethylene-coated steel pipes (PLP), synthetic resin-coated steel pipes (PLC) for cable protection, fiber reinforced plastic pipes (FRP) are used as the pipe attached to bridges.

If rigid vinyl chloride pipes or steel pipes are not applicable due to weather conditions or salt damage, consider fiber reinforced plastic pipes (FRP).

・ When external forces such as temperature expansion, vibration due to a braking load of an automobile, and vibration of an earthquake are applied to a rigid vinyl chloride pipe with an inserted joint, there is a possibility of joint disengagement. Therefore, when a rigid vinyl chloride pipe is used, a rigid vinyl chloride pipe (No joint) and a rigid vinyl chloride pipe socket (adhesive type) are options.

However, when it is difficult to joint disengagement in a short section (Approximately 5 m), a rigid vinyl chloride pipe with an insertion joint may be used.

・ FRP tubes are superior to rigid vinyl chloride tubes in strength, durability, and salt damage resistance, and have less thermal expansion and contraction.

Support Point Design

The pipeline is supported by a general support point and a special support point.

*Structure of the support point

・general support points: structures that allow movement in the tube axis direction

・special support points: structure constrained to move in a pipe axis direction

*The support spacing of the pipelines is determined by considering the type of pipe, horizontal movement of the pipe, axial deflection, allowable load, and resonance with bridge vibration.

The pipeline support spacing is shown in the following Figure.

Table 7.3.161 Support Spacing (Examples of Rigid Vinyl Chloride Pipes)

Type of Support Point	Applicable Interval	Support Spacing	Remarks
General Support Point	General Part of Bridge Girder	Spacing between support points : 1.5 m or less	
Special Support Point	Abutment	The distance between the support point adjacent to the abutment and the abutment front face : 2.5 m or less	
	Discontinuity Part of Bridge Girder	Distance between support points on both sides adjacent to the bridge girder discontinuity : 2.5 m or less	
	General Part of Bridge Girder	Spacing between support points : 20 m or less	

(iii) Installation of Handholes and Manholes

A. General

To decide the installation method of the hand hole, "Construction procedure and explanation of optical fiber cable" and (hereinafter referred to as "construction procedure") were referred.

The location, capacity, structure of the handholes and manholes shall be set by considering the connection and branching points of the optical fiber cables, the pipeline shape, restrictions due to underground space, types of cables to be laid.

[Explanation]

B. Classification

① The handhole shall be installed in a place where a structure shallower than 2m from the sidewalk to the bottom floor. In addition, a manhole shall be installed on the roadway with paying attention to prevent accidents during a cable installation inspection.

② The handhole is provided with a cable connecting pipe and a wire passing pipe. They can take out cable branching pipes.

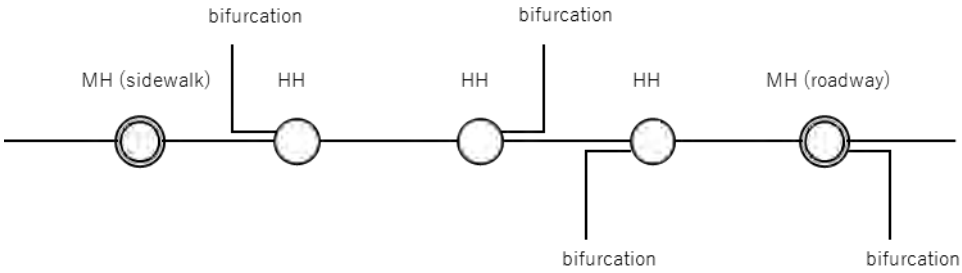
③ The standard shape of the handhole is a straight line type, however, when a window type is appropriate depending on the road form and the pipeline branching direction, this type is applied considering the possibility of cable accommodation.

[Annotation]

(1) The handhole has a structure in which a square lid or a round lid is attached to a box-shaped skeleton and is installed on a sidewalk. If it is installed on a roadway, use manholes.

(2) The manhole has a structure in which a round lid is attached to a box type skeleton having a slab and is installed in a roadway in principle.

[Image]



(3) A window type handhole is a type for absorbing a bending space of a cable in the handhole by a projecting part and not enlarging a dimension of a skeleton and is suitable for pipeline installation such as the crossing of a road. In the application, the cable laying line, shelf position, closure space shall be determined.

C. Installation Position

The location of the handholes shall be determined by taking cable connections, branch points, pipeline types, restrictions due to aboveground and underground occupancy into account.

[Annotation]

1. The location of the handholes, shall be determined by considering the cable connection and branch points, the road configuration, the horizontal and vertical alignments of the pipeline, the span length, the presence or absence of special sections, the presence or absence of existing occupancies, and the presence or absence of additional facilities for road management.

2. The installation of hand holes shall be determined by conducting a survey of buried objects (Test excavation, investigation of buried objects) in advance, as necessary, so as to minimize the transfer of existing proprietary objects.
3. Access points with road attached facilities shall be investigated in advance, and the installation position shall be determined after examining the capacity of handholes. and the installation position of access pipelines.
4. Generally, at mountainous roads with a risk of landslides, the facility should be installed on the mountain side.
5. When planning the installation of hand holes, consider the cable connection points.
In addition, each span length shall be within a range of approximately 250 m in consideration of the type of linear (intersection angle, radius of curvature) accommodation cable, allowable traction tension of the cable, workability.
In a continuous curve section of a bridge, the span length is shortened in consideration of the cable laying tension.

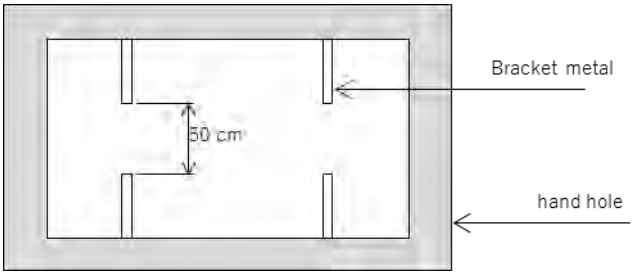
D. Capacity and Structure

① The capacity of the handhole is determined by the allowable bending radius of the cable, the closure size, the work space of the receiver size, the excess length of the cable, the pipeline mounting position, .

② A precast product made of cement concrete is used as a standard for a handhole , but a product made of other materials can be used as long as it satisfies a prescribed function and strength.

[Annotation]

1. The capacity of a handhole is determined by the sidewalk and roadway division, the standard part, the applicable division of the intersection part, the number of pipeline attachment. In determining the internal dimensions, necessary capacity is determined by the number of cable lines, type, allowable bending radius, closure dimensions.
   (1) The cable connection in the handhole is a street (ground) connection. Therefore, the excess length necessary for cable connection work, closure movement, storage is secured.
2. determination of the internal dimensions of the handhole
   The standard values for accommodating optical fiber cables with 1000 or fewer cores are as follows. When housing multi-core cables, metal cables, metal composite cables and power cables, design them individually according to the allowable bending radius.
   (1) The allowable bending radius of an optical fiber cable housed in a handhole is 10 times or more of the cable external diameter.
   (2) The handhole shall be mounted at a position where the uppermost stage is 20 cm or more from the top of the body and the lowermost stage is 65 cm or more from the Bottom Slab. The horizontal interval is 80 cm.
   (3) The working space width of the handhole should be at least about 50 cm.



(4) When connecting and accommodating an optical fiber cable in a handhole length. In general, it will be state shown in **Figure 7.3.142**. The required effective length L of the handhole at the state can be obtained by the following equation.

$$L = 2 (R + b + c) + a$$

Here.

L : required effective length of the handhole
a : closure length
b : required length of cable straight part
R : Cable Tolerance Bend Radius
c : cable to wall separation

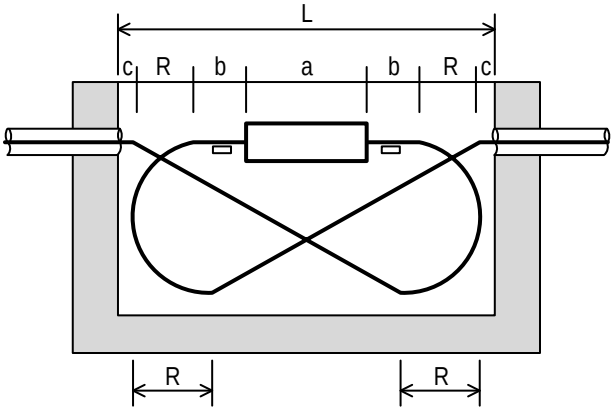


Figure 7.3.142 Required Effective Length of a Handhole

Calculation example (In case of 1000 core/cable external diameter 30 mm)

$$L = 2 (0.3 + 0.1 + 0.05) + 0.70 = 1.60$$

a: closure length……0.70 m
b: required length of cable straight part ……0.10 m
R: Cable Tolerance Bend Radius ……0.30 m
c: cable to wall separation ……0.05 m

(5) The size of the handhole applied only to the pull-through can be reduced by the size of the closure since the closure is not installed, but the size of the inner space of the handhole is not reduced unnecessarily because of the connection with the cable turning, ., and is set to the size equivalent to the No. 6 handhole used by the Ministry of Land, Infrastructure, Transport and Tourism.

(6) To the handhole, necessary accessories such as a brace, a pulling bolt, a step bolt and an insert bolt are attached.

③ Iron lid of Handhole

a. For the iron cover of the handhole, a square or round iron cover shall be used, and a manager's mark and information box indication shall be provided.
To provide a simple waterproof structure capable of locking a keyhole for securing security.

b. The iron lid of the handhole is of a fully open type in consideration of road connection. In addition, measures shall be taken to prevent the iron lid from falling.

④ Countermeasures against pooled water in hand holes (waterproofing and drainage)

a. need for waterproof measures

Most of the cable closures housed in the handholes are waterproof, but a large amount of water is not desirable for cable connection work and maintenance of facilities. In addition, a large amount of reservoir water causes flooding failure from the closure, and

in the lower hand hole of the normal sloping land, a fountain from the iron lid and a floating accident of the iron lid may occur due to rainfall flooding.

b. Water pooled in the handhole

The main causes of the pooled water in the handhole are rainwater inundation from the road surface due to rainfall, cracks in the handhole and the pipeline body, inundation of underground water due to defects and dew condensation due to temperature difference in the handhole and the pipeline. The most significant cause of the pooled water is rainwater inundation.

c. Waterproofing of Handholes (Measures against Rainwater Inflow from Iron Lids)

The iron cover of the handhole is generally of simple waterproof specification, and no perfect waterproof measure is taken against road surface flooding.

In general, it is desirable to implement a combination of waterproofing and drainage measures.

- Examples of waterproof measures
  - i. A waterproof rubber packing is installed all around between the receiving frame and the lid and is fastened with bolts.
  - ii. A waterproof cap is installed on the whole surface of the receiving frame and fastened by a rubber ring.
- Examples of Wastewater Measures
  - i. Installation of natural drainage holes ……general land
  - Pit with check valve……Location with high groundwater level
  - ii. Installation of forced drainage pump……a place where there is normal reserved water in a depression
  - iii. Sewage discharge system using water distribution pipes Lowest point of consecutive slopes

E. Mounting of Pipeline to Handhole.

① In addition to earth pressure and vehicle load, seismic stress and expansion/contraction force of the pipe itself are applied to the pipe line mounting part to the handhole, . (including manholes), but in any case, separation of the handhole from the pipe must not occur. Therefore, a duct sleeve for absorbing them is installed in the duct opening.

For precast handholes and the like, the duct sleeve should be installed in the factory in principle.

② When the nominal diameter is 50 mm, the center interval of the pipe attached to the handhole is 95 mm.

F. Pipeline Slope and Handhole Location

The gradient of the pipeline indicates a state of inclination of the pipeline between the handholes and is used for preventing water and sand from staying in the handholes and the pipeline.

The gradient is divided according to the ground height of the road and the depth of the underground buried object as follows.

G. Horizontal Condition

Since there is a risk that water or soil may remain in the pipeline with no slope, a water stop valve shall be provided as necessary to prevent water or soil from entering.

H. Half Slope

Since the pipeline has a linear slope to the handhole in one direction, and water and soil in the pipeline flow out to the handhole, the slope is ideal.

I. Medium to High GRADIENT

The central part of the pipeline is high and has a downward slope toward both handholes and has a good slope.

J. Medium to Low Gradient

Since the central part of the pipeline is low and has an upward slope toward both slopes, and water and sand are accumulated in the pipeline, such a pipeline is avoided as much as possible. If it is unavoidable, a water stop should be installed to prevent the intrusion of water and soil.

K. Half Slope/Medium to Low Gradient

At a part of the center of the pipeline, there is a part lower than a straight line connecting the pipe line extraction ports of both the hand holes, and from the part, a downward slope is provided to one hand hole and an upward slope is provided to the other hand hole to provide a good slope.

(b) Transmission Path

Remote monitoring and control equipment will be installed in remote control rooms of MCGS and Cainta Weir with EFCOS. The transmission path for information communication is a path that connects these points as shown in Table7.3.55 and the Figure in the next page.in.

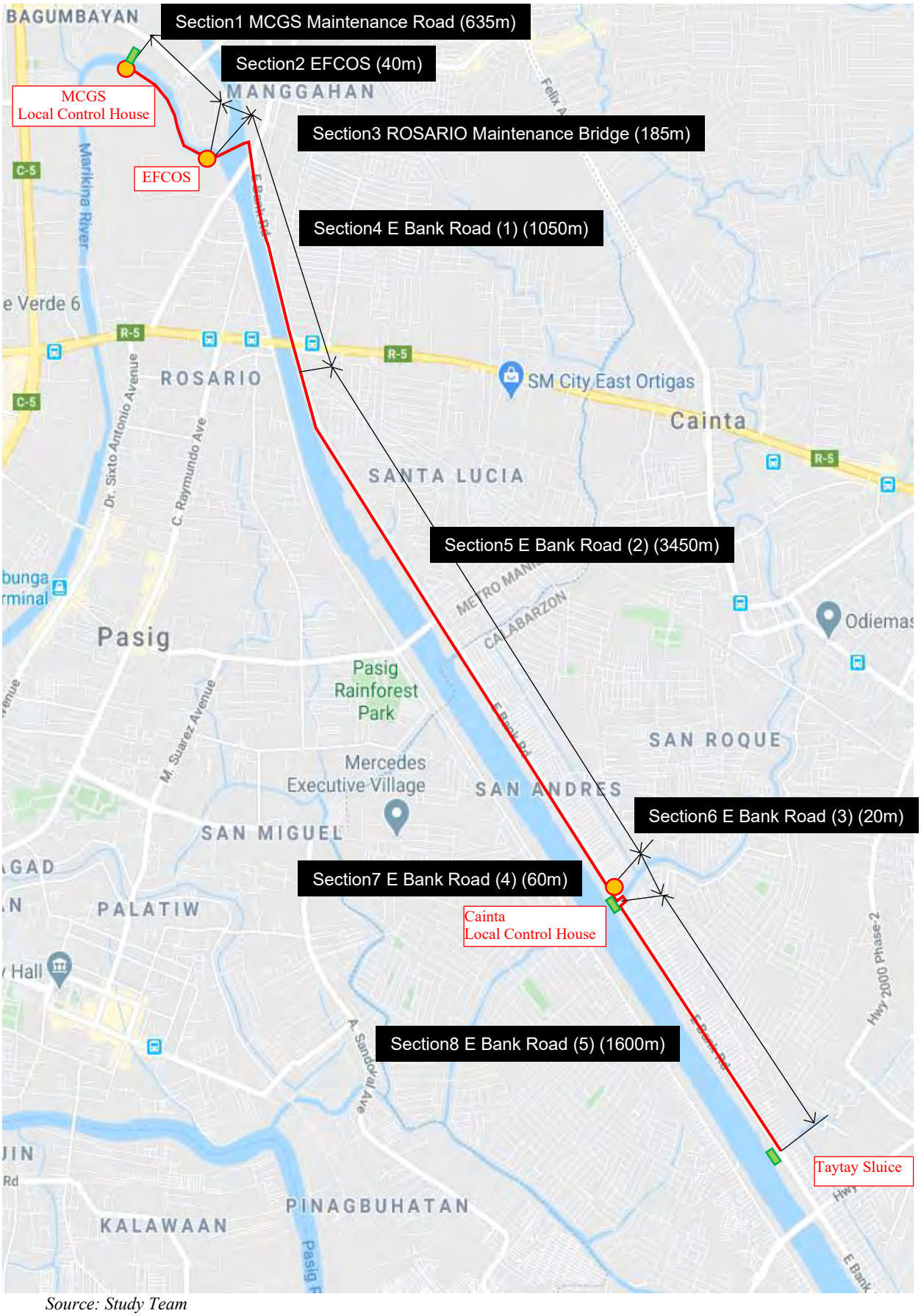
Table 7.3.162 Transmission Path

Section Number	Section Name	Origin	Endpoint	Section Extension (m)
1	MCGS Management Passage	EFCOS	MCGS Administration Building	635
2	EFCOS	EFCOS	Right bank of ROSARIO weir maintenance bridge	40
3	ROSARIO Weir Maintenance Bridge	Right bank of ROSARIO weir maintenance bridge	ROSARIO weir maintenance bridge left bank	185
4	E Bank Rd (1)	ROSARIO weir maintenance bridge left bank	Ortigas Bridge	1,050
5	E Bank Rd (2)	Ortigas Bridge	Cainta Administrative Building	3,450
6	E Bank Rd (3)	Cainta Administrative Building	Right bank of the Cainta Maintenance bridge	20
7	E Bank Rd (4)	Right bank of the Cainta Maintenance bridge	Left bank of the Cainta Maintenance bridge	60
8	E Bank Rd (5)	Left bank of the Cainta Maintenance bridge	Taytay sluice gate	1,600

Source: Study Team

To ensure the reliability of data transmission over optical lines, a star network, that is, a single site (EFCOS in this design), is connected to multiple nodes (MCGS control building and Cainta weir control building, in this design) is not desirable. Instead, it is necessary to construct whether a mesh network (a network that connects multiple sites and nodes to each other and can communicate by bypassing a communication path even when a failure occurs in a specific section), or a loop network (A network that connects all bases in a loop and communicates by taking a reverse route when a failure occurs in some sections).

I this design, however, it is difficult to set the optical line laying route for constructing the mesh type network or the loop type network because the field condition and road condition are not suitable. Thus, the reliability of the data transmission is secured by improving the redundancy of the laying line.



Source: Study Team

Figure 7.3.143 Transmission Path

(c) Construction Design by Section

(i) Section 1 (MCGS management passage)

A. Summary

Section 1 (MCGS management passage) is a section of about 635 m in length from EFCOS to the MCGS administration building.

B. Installation Policy

In section 1, the left dike of the Marikina River is newly constructed up to MCGS. Based on this situation, the pipeline will be buried under the maintenance road. Vehicle traffic to the MCGS is also scheduled on the maintenance road. Considering these situations, the number of lines, pipe diameter, embedded position and depth, and piping arrangement are determined according to the "Optical fiber cable installation procedure and explanation."

C. Laying Track

A communication line (fiber-optic cable) and a power supply line (from the MCGS low-voltage receiving position to the MCGS receiving panel) are installed in the section 1.

D. Communication Line

In view of the estimated amount of transmission (**Table 7.3.154**) and the extension of in section 1, the SM -4 C will be laid. Install two sets of SMs -4 C based on the certainty of data communication.

E. Power Line

It is assumed that low voltage power reception (3 Phase 3 Wire 220 V) is conducted in EFCOS and it is drawn to MCGS. According to the extension of installation from the power receiving position to the MCGS management building power receiving panel and the amount of voltage drop, the power supply line to be installed has 4 lines of CV -325 req -3 C.

F. Laying Pipeline

Communication line pipelines are specified by the internal line regulation by dividing into cases where the number of insulated wires is 10 or less and 10 or more, and cases where thick steel pipelines, thin steel pipelines, and unthreaded pipelines. When the bending of the pipe is small and the wire can be easily inserted and replaced, the total cross sectional area including the insulation covering the wire is selected to be 48% or less of the inner cross sectional area of the pipe when the wire has the same thickness, and 32% or less when the wire has a different thickness. Based on the communication line specifications and power line specifications, the pipeline specifications in section 1 are as follows.

Table 7.3.163 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 30	2
CV -325 rp -3 C (600 V)	4	65	336.22	1344.88	FEP 80	2

Source: Study Team

G. Laying Method

The Pipeline in section 1 shall be buried more than 0.6 m as burying in the roadway part according to the "Optical fiber cable installation procedure and explanation".

H. Laying of Handholes

According to "Optical fiber cable installation procedure and explanation", 1 place of handhole

is laid in the range of about 250 m in the section1.

(ii) Section 2 (EFCSOS)

A. Summary

Section 2 (EFCSOS) is a section of about 40 m in length from EFCOS to the right bank of the Rosario weir maintenance bridge.

B. Laying Policy

Section 2 is a communication path in the EFCOS premises. In the same section, the number of lines, pipe diameter, buried position, buried depth, and piping arrangement shall be determined following the "Optical fiber cable installation procedure and explanation."

C. Laying Track

The line laid in section 2 is only a communication line (fiber-optic cable).

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**)) and the extension of in section 2, the SM -4 C will be laid. Install two sets of SMs -4 C based on the certainty of data communication.

D. Laying Pipeline

The specifications of the pipeline in section 2 are as follows.

Table 7.3.164 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 30	2

Source: Study Team

E. Laying Method

The Pipeline in section 2 shall be buried more than 0.6 m as burying in the roadway part according to the "Optical fiber cable installation procedure and explanation".

(iii) Section 3 (Rosario Weir Maintenance bridge)

A. Summary

Section 3 (Rosario Weir Maintenance bridge) is about 185 m extends to the right bank and the left bank of the Rosario weir maintenance bridge.

B. Laying Policy

Section 3 crosses the Rosario Weir maintenance bridge from EFCOS to the Cainta Weir control building. In the same section, the cable will be attached to the main girder of the Rosario Weir maintenance bridge, and the laying method will be decided according to "Optical fiber cable installation procedure and explanation."



Source: Study Team

Figure 7.3.144 Route to Section 3 (Bridge Attachment for Rosario Weir Maintenance Bridge)

Section 3 is the communication path from EFCOS to the Cainta Weir control building. In the same section, the number of lines, pipe diameter, buried position, buried depth and piping arrangement shall be determined in accordance with the "Optical fiber cable installation procedure and explanation".

C. Laying Track

The line laid in section 3 is only a communication line (fiber-optic cable).

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**)) and the extension of in section 3, the SM -4 C will be laid. Install two sets of SMs -4 C based on the certainty of data communication.

D. Laying Pipeline

The specifications of the pipeline in section 3 are as follows.

Table 7.3.165 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 30	2

Source: Study Team

E. Laying Method

Laying in section 3 shall be carried by the pipeline to the bridge main girder in accordance with "Optical fiber cable installation procedure and explanation". As for the attachment type, the support interval is in accordance with the same standard, and the general support point (Set the distance between the bridge girder general part and the supporting point to be 1.5m or less.).

(iv) Section 4 (E Bank Rd # 1)

A. Summary

Section 4 (E Bank Rd # 1) is about 1,050 m in length from the left bank of the Rosario Weir maintenance bridge to the Ortigas Bridge.

B. Laying Policy

Section 4 lies in the left bank revetment of Manggahan floodway from EFCOS to the Cainta Weir control building. In this section, the pipeline will be laid on the surface of the parapet revetment, and the laying method will be decided according to "Optical fiber cable installation procedure and explanation."



Source: Study Team

Figure 7.3.145 Route to Section 4 (Revetment Attachment)



Source: Study Team

Figure 7.3.146 Route to Section 4 (Revetment Attachment)

C. Laying Track

The line laid in section 4 is only a communication line (fiber-optic cable).

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**) and the extension of in section 4, the SM -4 C will be laid. Install two sets of SMs -4 C based on the certainty of data communication.

D. Laying Pipeline

The specifications of the pipeline in section 4 are as follows.

Table 7.3.166 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 30	2

Source: Study Team

E. Laying Method

Pipeline in section 4 shall be attached to the main girder of the bridge following the "Optical fiber cable installation procedure and explanation." As for the attachment type, the support interval follows the same standard, and the general support point (Set the distance between the bridge girder general part and the supporting point to be 1.5 m or less.).

(v) Section 5 (E Bank Rd # 2)

A. Summary

Section 5 (E Bank Rd # 2) is about 3,450 m in length from the Ortigas Bridge to the Cainta Weir control building.

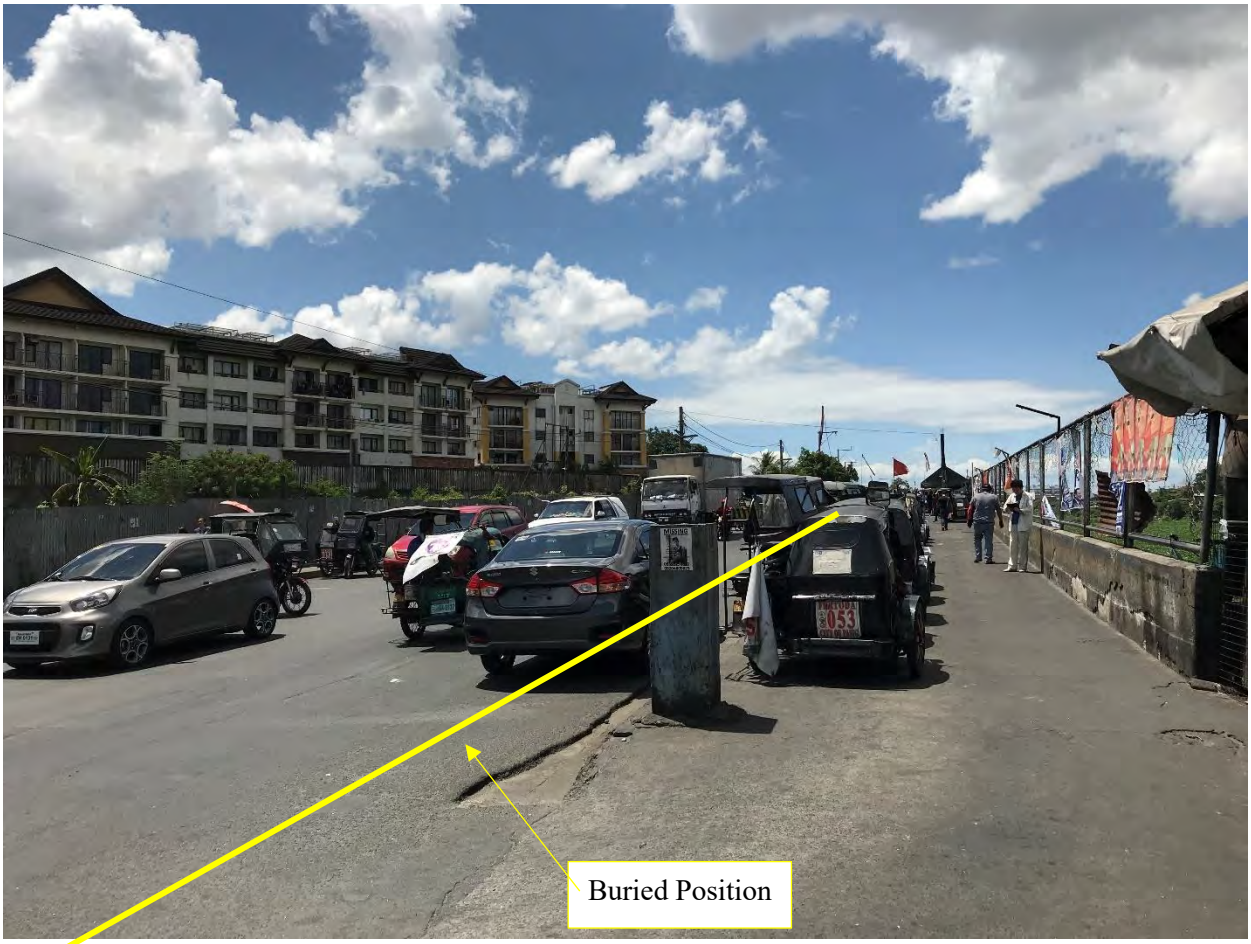
B. Laying Policy

Section 5 stretches from EFCOS to the Cainta Weir control building. The pipeline from the left bank of the Rosario Weir maintenance bridge to the Ortigas Bridge can be attached to the revetment. At the section beyond Ortigas Bridge, construction works of revetment and river park by Pasig City are currently carried out, and it is difficult to attach the pipeline to the revetment. From the above, the pipeline in the section shall be buried under the road in E Bank Rd. The laying method determines the number of lines, pipe diameter, embedded position, buried depth, and piping arrangement according to "Optical fiber cable installation procedure and explanation."



Source: Study Team

Figure 7.3.147 Section 5 Revetment and River Park Construction by Pasig City



Source: Study Team

Figure 7.3.148 Route to Section 5 (Road Burial)

C. Laying Track

The line laid in section 5 is only a communication line (fiber-optic cable). In this section, the pipeline for the communication channel is laid.

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**) and the extension of in section 5, the SM -4 C will be laid. Install two sets of SMs -4 C based on the certainty of data communication.

D. Laying Pipeline

The specifications of the pipeline in section 5 are as follows.

Table 7.3.167 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 30	2

Source: Study Team

E. Laying Method

The Pipeline in section 5 shall be buried more than 0.6 m as burying in the roadway part according to the "Optical fiber cable installation procedure and explanation".

(vi) Section 6 (E Bank Rd # 3)

A. Summary

Section 6 (E Bank Rd # 3) is about 20m in length from the Cainta administration building to the right bank of the Cainta administration bridge.

B. Laying Policy

Section 6 is part is from the Cainta administration building to the Taytay sluice gate. The pipeline in this section shall be buried under the road in E Bank Rd. The laying method determines the number of lines, pipe diameter, buried position, buried depth, and piping arrangement according to "Optical fiber cable installation procedure and explanation."



Source: Study Team

Figure 7.3.149 Route to Section 6 (Road Burial)

C. Laying Track

(vii) The line laid in section 6 is a communication line (optical fiber cable, cable for Cainta sluice water gauge and cable for Taytay sluice water gauge). In this section, the pipeline for the communication channel is laid.

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**) and the extension of in section 6, the SM -4 C and an exclusive cable for the crystal water gauge of the optical power supply type (ϕ 12.5) will be laid. Two sets of SMs -4 C, and four sets for water gauge (two for Cainta Weir and two for Taytay Weir) will be installed based on the certainty of data communication.

A. Laying Pipeline

The specifications of the pipeline in section 6 are as follows.

Table 7.3.168 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 50	2
Optical-Crystal Sensor	4	39.27 *	122.72	490.88		

**Calculated from optical-crystal sensor cable ϕ 12.5*

Source: Study Team

B. Laying Method

Laying in section 6 shall be done by embedding over 0.6 m in the roadway in accordance with "Optical fiber cable installation procedure and explanation".

(viii) Section 7 (E Bank Rd # 4)

A. Summary

Section 7 (E Bank Rd # 4) is about 60 m in length from the right bank of the Cainta Maintenance bridge to the left bank of the Cainta Maintenance bridge.

B. Laying Policy

Section 7 is part of a series of sections from the Cainta administration building to the Taytay sluice gate. In this section, the pipeline will be laid on the main girder of Cainta sluice maintenance bridge to be newly constructed. The laying method will be decided according to "Optical fiber cable installation procedure and explanation".

C. Laying Track

The line laid in section 7 is a communication line (Optical fiber Cable, Cable for Cainta sluice water gauge and Cable for Taytay sluice water gauge). In this section, the pipeline for the communication channel is laid.

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**) and the extension of in section 7, the SM -4 C and an exclusive cable for the crystal water gauge of the optical power supply type (ϕ 12.5) will be laid. Two sets of SMs -4 C, and four sets for water gauge (two for Cainta Weir and two for Taytay Weir) will be installed based on the certainty of data communication.

D. Laying Pipeline

The specifications of the pipeline in section 7 are as follows.

Table 7.3.169 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 50	2
Optical-Crystal Sensor	4	39.27 *	122.72	490.88		

**Calculated from optical-crystal sensor cable ϕ 12.5*

Source: Study Team

E. Laying Method

Laying in section 7 shall be carried by the pipeline to the bridge main girder in accordance

with "Optical fiber cable installation procedure and explanation". As for the attachment type, the support interval follows the same standard, and the general support point (Set the distance between the bridge girder general part and the supporting point to be 1.5 m or less.).

(ix) Section 8 (E Bank Rd # 5)

A. Summary

Section 8 (E Bank Rd # 5) is about 1,600 m in length from the left bank of the Cainta maintenance bridge to the Taytay Weir.

B. Laying Policy

Section 8 is from the Cainta Administration Building to the Taytay Weir. The pipeline in this section from the left bank of the Cainta Maintenance bridge to the Taytay Weir shall be buried under the road in E Bank Rd as in Section 5. The laying method determines the number of lines, pipe diameter, buried position, buried depth and piping arrangement according to "Optical fiber cable installation procedure and explanation".



Source: Study Team

Figure 7.3.150 Route to Section 8 (Road Burial)

C. Laying Track

The line laid in section 8 is a communication line (Optical fiber Cable and Cable for Taytay sluice water gauge). In this section, the pipeline for the communication channel is laid.

[Communication Line]

In view of the estimated amount of transmission (**Table 7.3.154**) and the extension of in section 8, the SM -4 C and an exclusive cable for the crystal water gauge of the optical power supply type (ϕ 12.5) will be laid. Two sets of SMs -4 C, and four sets for water gauge (two for Cainta

Weir and two for Taytay Weir) will be installed based on the certainty of data communication.

D. Laying Pipeline

The specifications of the pipeline in section 8 are as follows.

Table 7.3.170 Pipeline Specification

By Species	Number of Cable Lines	External Diameter (mm)	Section Area (mm2)	In-Pipe Cable Section Area (mm2)	Pipe Size (FEP)	Number of Pipelines
SM -4 C	2	10.5	8.77	17.54	FEP 40	2
Optical-Crystal Sensor	2	39.27 *	122.72	245.44		

*Calculated from optical-crystal sensor cable ϕ 12.5

Source: Study Team

E. Laying Method

The pipeline in section 8 shall be buried more than 0.6 m as burying in the roadway part according to the "Optical fiber cable installation procedure and explanation".

3) Selection of Network Equipment

(a) Standard Network Equipment

In IP transmission equipment design, a network hoist is selected based on the following conditions. The L3-SW and L2-SW device specifications are shown on the next page for reference.

① Extraction of specifications required for network configuration

② "Specifications of IP transmission equipment (draft): March 2015; Ministry of Land, Infrastructure, Transport and Tourism" is applied to the equipment specifications of network equipment (L3 and L2).

Table 7.3.171 Comparison of L3-SW Standard Specifications

Item		Modul Type			Fixed type		
		Type A	Type B	Type C	Type A	Type B	Type C
Usage	Backbone network core switch	○	○	-	-	-	-
	Main line and in-house networks distribution switch	-	○	○	○	○	○
	Office network access switch	-	-	○	-	-	-
Configuration		chassis configuration	chassis configuration	chassis configuration	fixed configuration	fixed configuration	fixed configuration
basic part	duplexing	○	○	○	×	×	×
power supply part	duplexing	○ Mixed AC/DC/AC/DC	○ AC/DC	○ AC/DC	○ Mixed AC/DC/AC/DC	○ Mixed AC/DC/AC/DC	○ Mixed AC/DC/AC/DC
Power consumption		4 slots: 2700 W 6 slots: 4000 W 9 slots: 4000 W	7 slots: 1400 W	7 slots: 1400 W	AC/DC: 300 W	AC: 250 W, DC: 640 W	AC: 250 W, DC: 640 W
Connecton Port	10/100/1000 BASE-T	48 + ports/modules	48 + ports/modules	48 + ports/modules	48 + ports	48 + ports	24 + ports
	1000 BASE-X (SFP)	24 +/Module	12 +/Module	12 +/Module	Four or more SFPs or SFP + 4 +	Two or more SFPs or SFP + 2 +	Two or more SFPs or SFP + 2 +
	10 GBASE-R (SFP +)	8 +/Module	12 +/Module	12 +/Module			
Backplane capacity		2080 Gbps or higher	848 Gbps or higher	520 Gbps or higher	176 Gbps or higher	176 Gbps or higher	88 Gbps or higher
Packet forwarding capability		4 slots: up to 180 Mpps or more 6 slots: up to 300 Mpps or more 9 slots: up to 480 Mpps or more	Up to 250 Mpps	Up to 225 Mpps	Up to 131 Mpps	Up to 130 Mpps	Up to 65 Mpps
Iteme quivalent to CISCO		Catalyst 6504 E, etc.	Catalyst 4510 R + E, etc.	Catalyst 4507 R + E, etc.	Catalyst 4948 E-E, etc.	Catalyst 3650 -48 TQ-E, etc.	Catalyst 3650 -24 TD-E, etc.
Price equivalent to CISCO					3,600,000	1,800,000	1,200,000
Number of MAC registered addresses equivalent to CISCO		128,000	55,000	55,000	55,000	32,000	32,000
Maximum number of routes equivalent to CISCO		1,000,000	256,000	256,000	57,000	24,000	24,000

Source: Study Team

Table 7.3.172 Comparison of L2-SW Standard Specifications

Item		Fixed type						
		Type A	Type B	Type C	Type D	Type E	Type F	Type G
Usage		Core L2 switch for VoIP in pipe (For branch offices), Access switch for aggregation of in-house networks						
Configuration		Rackmount	Rackmount	Rackmount	Rackmount	Rackmount	Rackmount	Rackmount
Power supply section		100 Vac	100 Vac	100 Vac	100 Vac	100 Vac	100 Vac	100 Vac
Power consumption		about 48 W	about 50 W	about 32 W	about 38 W	about 20 W	about 20 W	about 36 W (Body only)
Connecton Port	10/100/1000 BASE-T	48 + ports	48 + ports	–	24 + ports	–	8 + ports	–
	10/100 BASE-TX	–	–	48 + ports	–	24 + ports	–	24 + ports
	Others	Two or more SFPs or SFP + 2 +	Four or more SFPs	1000 BASE-T2 ports or higher or 2 or more SFPs	Four or more SFPs	1000 BASE-T2 ports or higher or 2 or more SFPs	1000 BASE-T2 ports or higher or 2 or more SFPs	1000 BASE-T2 ports or higher or 2 or more SFPs
Backplane capacity		108 Gbps or higher	108 Gbps or higher	14 Gbps or higher	108 Gbps or higher	9 Gbps or higher	10 Gbps or higher	16 Gbps or higher
Packet forwarding capability		Up to 102 Mpps	Up to 100 Mpps	Up to 10 Mpps	Up to 71 Mpps	Up to 6 Mpps	Up to 14 Mpps	Up to 6 Mpps
PoE power supply		–	–	–	–	–	–	○(All Ports) (15.4 Watts/Port)
Item equivalent to CISCO		Catalyst 2960 X -48 TD-L, etc.	Catalyst 2960 X -48 TS-L, etc.	Catalyst 2960 Plus 48 TC-L, etc.	Catalyst 2960 X -24 TS-L, etc.	Catalyst 2960 Plus 24 TC-L, etc.	Catalyst 2960 CG -8 TC-L, etc.	Catalyst 2960 – Plus 24 PC-L, etc.

Source: Study Team

(b) Selection of Network Equipment

In EFCOS, MCGS (control room), and Cainta Weir (control room), the system equipment related to the core of the management facility will be improved. Also, the measured water level data in each gate facility, CCTV camera image, alarm facility and gate state will be transmitted. Connections with related established facilities (Monitoring and control equipment for Rosario Weir) and transmission of monitoring information to upper organizations (DPWH and MMDA) will be made. Network equipment should be controllable in terms of security.

Network equipment installed in EFCOS, MCGS, and Cainta Weir should avoid bottlenecking of the network. Thus, L3-SW is arranged, which has a large packet transfer capacity and can enhance safety and stability by duplicating the power supply.

There are six types of L3-SW specifications: modular types A to C and fixed types A to C. These specifications are high enough to form a backbone network. Among them, "Fixed Type C" was adopted because of its relatively low specification in connection port number and data transfer capacity. The safety and stability of the power supply will be enhanced by duplexing, and 100 VAC will be supplied in two systems.

Regarding the optical line connecting MCGS and EFCOS and the optical line connecting Cainta Weir and EFCOS, the SFP port of intermediate distance type shall be added as an option because the distance between the bases is about 0.6 km and about 6 km, respectively,

(c) Network equipment selection

The model selection results of L3-SW are summarized below.

Table 7.3.173 Selection of L3-SW Models

Base Point	Selection Hoist	Characteristics
MCGS	Catalyst 3650 -24 TD-E L3 Fixed Type C (24 ports)	Packet transmission capacity up to 65 Mbps 1000 BAS compatible models Dual power supplies available 24 LAN ports Add Medium SFP (For EFCOS)
Cainta Sluice	Catalyst 3650 -24 TD-E L3 Fixed Type C (24 ports)	Packet transmission capacity up to 65 Mbps 1000 BAS compatible models Dual power supplies available 24 LAN ports Add Medium SFP (For EFCOS and Taytay sluice gates)
EFCOS	Catalyst 3650 -24 TD-E L3 Fixed Type C (48 ports)	Packet transmission capacity up to 65 Mbps 1000 BAS compatible models Dual power supplies available 48 LAN ports Add Medium SFP (For MCGS and Cainta sluice gates)

Source: Study Team

(3) System Configuration

Based on the results of the previous study, the summary of the entire system diagram in the **Figure 7.3.151**.

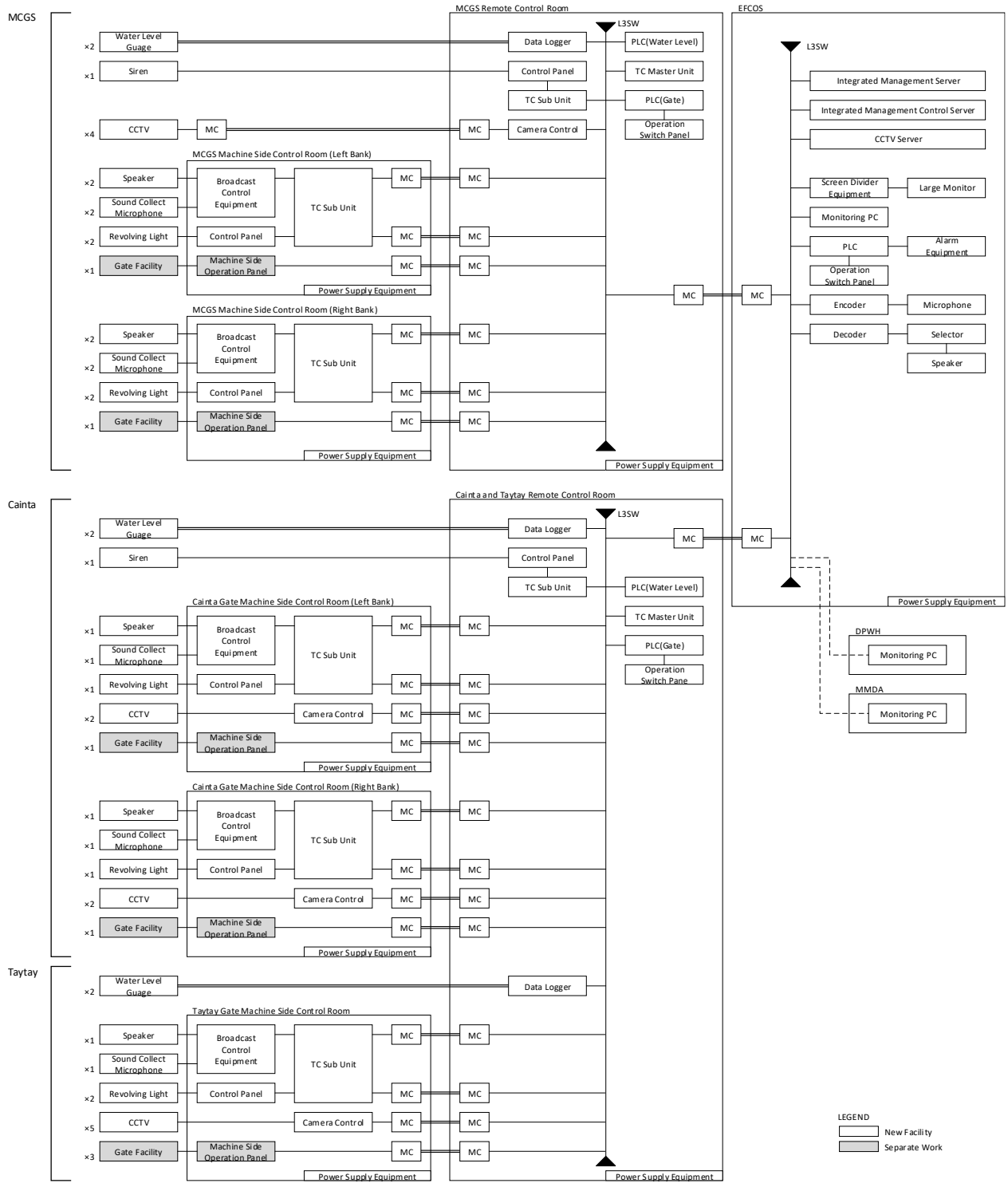


Figure 7.3.151 Entire System Diagram

(4) Examination of Remote Monitoring and Control Equipment

1) Integrated Management Server

(a) Summary

The integrated management server is the equipment which shoulders central functions such as calculation, decision, display of the system.

(b) Equipment Specification

a	Type	Rackmount
b	CPU	1 + Xeon Gold 5122 (4 Cores/3.6 GHz)
c	Memory	32 GB or more
d	HDD	3 + 300 GB (RAID 1 + Hotspare)
e	LAN	2 + LAN (1 Gbps) ports
f	Optical Drive	Internal DVD drive unit
g	Other	Power supply units shall be redundant.

2) Integrated Management Control Server

(a) Summary

The integrated management control server is used to control the MCGS and Cainta equipment from the remote-control room. It also controls the large monitor.

(b) Equipment Specification

a	Type	Rackmount
b	CPU	1 + Xeon Gold 5122 (4 Cores/3.6 GHz)
c	Memory	32 GB or more
d	HDD	3 + 300 GB (RAID 1 + Hotspare)
e	LAN	2 + LAN (1 Gbps) ports
f	Optical drive	Internal DVD drive unit
g	Other	Power supply units shall be redundant.

(c) Functional Specification

a	Gate Distance Control
b	Siren/Loudspeaker/Alarm Light Remote Control
c	Sound Collection Microphone Reproduction

3) Monitoring Terminal Equipment (Monitoring PC)

(a) Summary

The monitoring terminal equipment (Monitoring PC) displays the Web contents created by the integrated management server.

(b) Equipment Specification

a	Type	Desktop
b	Liquid Crystal	at least 21 inch wide LCD (1920 x 1080)
c	CPU	Core (TM) i3 -7100 x 1 or higher
d	HDD	500 GB or more
e	Memory	4 GB or more
f	Optical Drive	Super Multi
g	OS	Windows 10 or the latest
h	Others	Keyboard, including mouse Install anti-virus software Multicast video stream is displayed by this equipment. The implementation of software (soft decoder) to
i	Implementation of Microsoft Office software	
j	Ability to view web content	

4) Screen Divider

(a) Summary

The screen divider outputs video and Web contents to a 55-inch monitor by a display control signal from an integrated management control server.

(b) Equipment Specification

a	Video Output	One DVI or HDMI
b	LAN	LAN1 port or more
c	Split Pattern	1, 3, 4, 6, 9 or more
d	Multiplexed Form	MPEG -2: MPEG -2 System PS H. 264: MPEG -2 System TS and TTS
e	Video Decoding System	HD: ITU-T H. 264 (ISO/IEC 144496 -10) At least MP @ L4

(c) Functional Specification

a	Video Display	Decodes an IP stream (multicast delivery) from a CCTV camera installed at a site and to display it on a large monitor. Display image must be able to register 100 cameras or more.
b	Web View	Web content can be displayed at the specified URL
c	Cyclic Display	The number of channels for which video and Web content can be cyclically displayed at specified intervals is 100 or more for each channel.

5) Large Monitor

(a) Summary

The large monitor shall display necessary information such as water level, present condition of gate facilities or CCTV image. The large monitor is made to be the composition of 4 plane multi.

(b) Equipment Specification

a	Screen Size	55 v type or higher
b	Structure	55-inch TFT LCD
c	Backlight	LED
d	Resolution	1920 × 1080 dots or more
e	Luminance	700 cd/m2
f	Input Terminal	1 or more DVI-D 24 pins
g	One or more HDMI Type A connectors	
h	Normal Operating Time	24 hours
i	Others	Including wall-mounting installation fittings

6) Alarm Device

(a) Summary

The alarm device sounds the alarm after judgement of the integrated management server.

(b) Equipment Specification

a	Lamp	1 Color
b	Buzzer	Yes (Volume can be adjusted)
c	Shape	Table

7) Operation Hoist Panel

(a) Summary

The operation hoist panel has a function of manually controlling the gate from a remote/remote location by a push button. The operation hoist board outputs control signals to the machine side

operation board via the PLC.

(b) Equipment Specification

a	Device Configuration	Key SW, push button SW, alarm, power supply
b	Connection I/F	Contact signal
c	Equipment to be Operated	MCGS Remote Control Room: MCGS Cainta Remote Control Room: Cainta EFCOS remote control room: MCGS, Cainta
d	Safety Mechanism	Prevents erroneous operation by key SW.

7.3.6.6 Electrical Equipment (Emergency Power Supply) Design

(1) Design Conditions

1) Operating time of the Generator

MCGS must be able to operate at all times, even during floods generated by large-scale typhoon accompanied by power outages. The emergency power supply for control equipment shall be designed to ensure its function, even in the case of normal power failure (72 hours). The operating time was determined regarding the following factors.

-Existing flood control facilities in Metro Manila

-Design standards for disaster prevention facilities in Japan

(a) Establishment of existing flood control facilities in Metro Manila

The emergency power source of each drainage pump station constructed in the "Kamanaba District Flood Control and Drainage System Improvement Project," which started in 2000, is planned to be operated for 72 hours. In the Rosario Weir located at the upstream end of the Manggahan Floodway, the operation time estimated from the fuel consumption of 5 L/h (from hearings with the MMDA) for the tank capacity of 390 L is 78 hours. Still, this fuel consumption is a value that does not consider the gate opening/closing load, and there is a possibility that the fuel for the 72-hour operation of the control equipment + several times of gate opening/closing operations is secured.

Next, the power outage caused by Typhoon Ondoy in 2009, which brought record-breaking rainfall, said to occur only once in 180 years, in the Metro Manila area, is reviewed. The state of blackout in Metro Manila during Typhoon Ondoy in 2009 is shown in **Table 7.3.174**. Descriptions of power outages were based on statements made by a Meralco spokesperson in local news at the time.

It took more than a week for the entire area to recover from the blackout because it took a long time to restorer the inundated area. It seemed that the restoration of distribution networks was almost completed on the second day after the power failure, except for the severely inundated regions. Considering the blackout recovery after the typhoon Ondoy, the expected outage time shall be set as 72 hours.

Table 7.3.174 Blackouts during Typhoon Ondoy

Date	Typhoons and Power Outages
9/26 (Sat) 2009	Typhoon Ondoy passed by, and power failure occurred in various places in the city. Number of power outages: approximately 1.2 million (1/4 of Meralco's 4.6 million total subscriptions) Number of blackout districts: 139 (Approximately 1/5 of the total 683 distribution districts)
9/27 (Sun) 2009	More than 1 day has passed since the power outage, but 13% of the affected areas have not yet recovered.
Monday, 9/28, 2009	Number of power outages: approximately 132000 (3% of Meralco's 4.6 million total subscriptions) Number of blackout districts: 27 (Approximately 4% of the total 683 distribution districts)
10/2 (Fri) 2009	Number of blackout districts: 6 (Approximately 1% of the total 683 distribution districts) Restoration work has been difficult in areas that have been flooded since the typhoon passed.
Monday, 10/6, 2009	All areas have been restored (Some isolated areas are still considering how to restore.).

Source: Study Team

(b) Design Standards for Disaster Prevention Facilities In Japan

As an approach to securing power sources for disaster prevention facilities in Japan, the "Guidelines for Designing Telecommunications Facilities and Explanations (electric (al) knitting)" stipulates that "Fuel stockpiles that can be operated for seven consecutive days (168 hours) or more at critical facilities and three consecutive days (72 hours) or more at other facilities" in the communication facilities of disaster prevention facilities (Id. at p. 3 -5.). Here, "Important facilities requiring seven days of storage" refers to dams located in mountainous areas that are difficult to supply, and this facility is not considered to be located in urban areas.

(c) Setting of Fuel For Gate Opening and Closing Power

The gate opening/closing power will need to be operated more than two times a day during a power outage. Thus, the fuel tank capacity shall be secured for a total of 480 minutes, that is, 8 hours, for one opening (closing) time of about 40 minutes × (1 open + one closed) × two times × three days.

2) Creating a Load List

To MCGS equipment load capacity is shown in **Table 7.3.175**. The equipment was classified into generators for gate opening and closing and generators for control facilities.

Table 7.3.175 Load List

Category	Loads	Three-phase	Single-phase	Unit	Set	Total
Gate Hoist	Hosit Motor of large span gate （2 M 2 D）	13.00		kVA	2	26.00
	Hosit Motor of smalle span gate （1 M 1 D）	5.50		kW	1	5.50
	local control panel control switch		1.50	kW	2	3.00
					合計	34.50
Control Device	Local Control House1 Distribution board for control equipment					6.99
	Water Gague		0.01	kVA	2	0.02
	Loudspeaker		0.02	kVA	2	0.04
	Sound collection mike		0.01	kVA	2	0.02
	Siren		1.60	kVA	1	1.60
	Alarm lamp		0.03	kVA	2	0.06
	CCTV		0.15	kVA	2	0.30
	Control panel		2.77	kVA	1	2.77
	Tc extension		0.11	kVA	1	0.11
	MC		0.01	kVA	7	0.07
	Spare		2.00	kVA	1	2.00
	Local Control House3 Distribution board for control equipment					5.37
	Loudspeaker		0.02	kVA	2	0.04
	Sound collection mike		0.01	kVA	2	0.02
	Alarm lamp		0.03	kVA	2	0.06
	CCTV		0.15	kVA	2	0.30
	Control panel		2.77	kVA	1	2.77
	Tc extension		0.11	kVA	1	0.11
	MC		0.01	kVA	7	0.07
	Spare		2.00	kVA	1	2.00
	Generator House Distribution board for control equipment					14.79
	Optical receiver		0.01	kVA	1	0.01
	MC		0.01	kVA	15	0.15
	TC main device （incl. PLC）		0.22	kVA	1	0.22
	Tc extension		0.11	kVA	1	0.11
	Operation SW panel		0.25	kVA	1	0.25
	L3SW		0.05	kVA	1	0.05
	Miscellaneous power supply in the panel		3.00	kVA	1	3.00
	Generator panel auxiliary power supply (250kVA)		3.00	kVA	2	6.00
	Generator panel auxiliary power supply (50kVA)		3.00	kVA	1	3.00
	Spare		2.00	kVA	1	2.00
	Generator House Distribution board for building (DB1)					6.75
	LIGHTINGS		1.12	kVA	1	1.12
	LIGHTINGS		1.39	kVA	1	1.39
	LIGHTINGS		0.46	kVA	1	0.46
	C.O.,4 UNITS		0.68	kVA	1	0.68
	C.O.,3 UNITS		0.51	kVA	1	0.51
	FACP		0.95	kVA	1	0.95
	ACCP&FCU, 1.5HP		1.14	kVA	1	1.14
	SPARE		0.50	kVA	1	0.50
	Local Control House1 Distribution board for building (DB2)					1.96
	LIGHTINGS		1.28	kVA	1	1.28
	CONVENIENCE OUTLET, 1 UNIT		0.18	kVA	1	0.18
	SPARE		0.50	kVA	1	0.50
	Local Control House2 Distribution board for building (DB3)					1.96
	LIGHTINGS		1.28	kVA	1	1.28
	CONVENIENCE OUTLET, 1 UNIT		0.18	kVA	1	0.18
	SPARE		0.50	kVA	1	0.50
	Local Control House3 Distribution board for building(DB4)					1.96
	LIGHTINGS		1.28	kVA	1	1.28
	CONVENIENCE OUTLET, 1 UNIT		0.18	kVA	1	0.18
	SPARE		0.50	kVA	1	0.50
					Total	39.78

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.

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.

Table 7.3.176 Generator Calculation Result

Gate Opening and Closing	For Control Equipment
Generator Capacity 216.7 kVA Motor Output 195.0 kW	Generator Capacity 38.6 kVA Motor Output 37.6 kW

Source: Study Team

2) Selection of Motor

Motor Output based on the calculation results of "Output calculation software for private power generation facilities", the most recent generator efficiency table was selected from among those shown in Telecommunications Facility Design Guidelines and Explanations (electric (al) knitting).The generator efficiency is shown in **Table 7.3.178**.

Table 7.3.177 Power Generating Capacity and Motor Output of the Nearest High-Order Generator

Gate Opening And Closing	For Control Equipment
Generator Capacity 250 kVA Motor Output 225 kW	Generator Capacity 50.0 kVA Motor Output 48.6 kW

Source: Study Team

Table 7.3.178 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)

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

Generator Output		Conventional Efficiency	Motor Output	Generator Output		Conventional Efficiency	Motor Output
(kVA)	(kw)(power factor:0.8)	$\eta G(\%)$	(kW)	(kVA)	(kw)(power factor:0.8)	$\eta G(\%)$	(kW)
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)

(i) Selection of Generator

The basic conditions of the generator are as shown in the table below, based on the Guidelines for the Design of Telecommunications Facilities and the Explanation thereof (electric (al) knitting).

Table 7.3.179 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 to 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

(ii) Selection of Motor

The basic conditions of the motor are as shown in the table below based on the Guidelines for the Design of Telecommunications Facilities and their Explanation (electric (al) knitting). As described in the design procedure, a diesel engine and a gas turbine were compared (**Table 7.3.181**). The gas turbine system shall not be adopted unless the gas turbine is particularly advantageous.

Table 7.3.180 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.
The electric system of the starter 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.	The electric system of the starter 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.
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

Table 7.3.181 Comparison of Diesel Engines and Gas Turbines

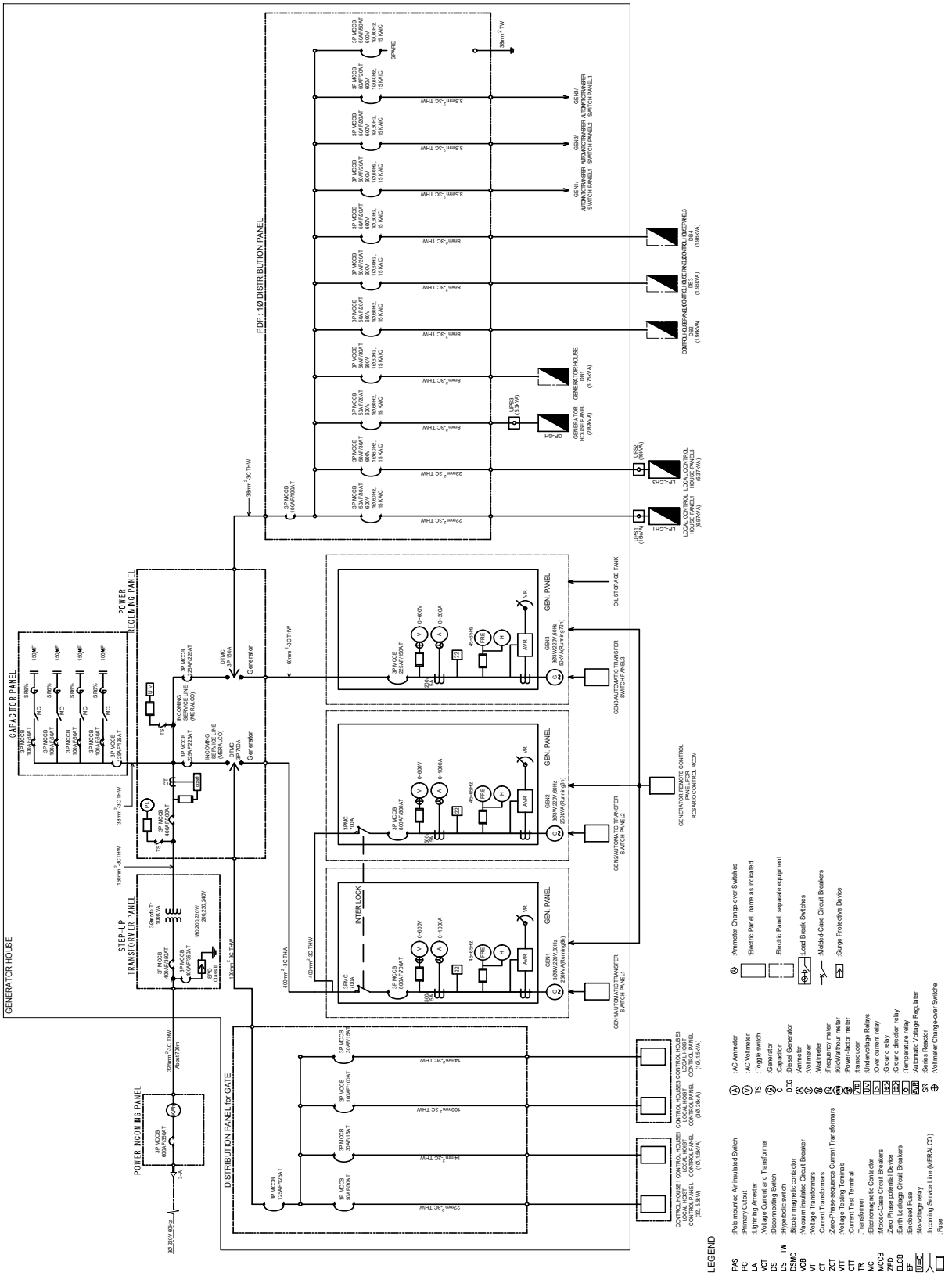
Item \ Motor	Diesel Engine	Gas Turbine
Operating Principle	Thermal energy of intermittent combustion and explosion combustion gas is once converted into reciprocating motion of piston, which is converted into rotary motion by crankshaft. (Reciprocating motion → Rotating motion)	Thermal energy of normally 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 to 310 g/kWh	520 to 680 g/kWh
Spent Fuel	Light oil and heavy oil A	Kerosene, light oil, and heavy oil A
Excess Air Ratio	2.0 to 3.0	3.5 to 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 Speed Loop	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 to 1000 ppm (O_2 concentration of 13%)	20 to 150 ppm (O_2 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	Required 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 to 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 to 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) Study on 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 was prepared (**Figure 7.3.152**).

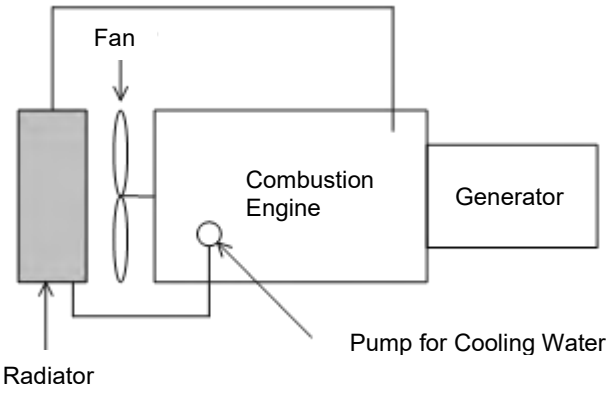


Source: Study Team

Figure 7.3.152 Single Wire Diagram

(b) Determination of Cooling Method

From the Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting), page 3 -32 of the 29 edition, 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.

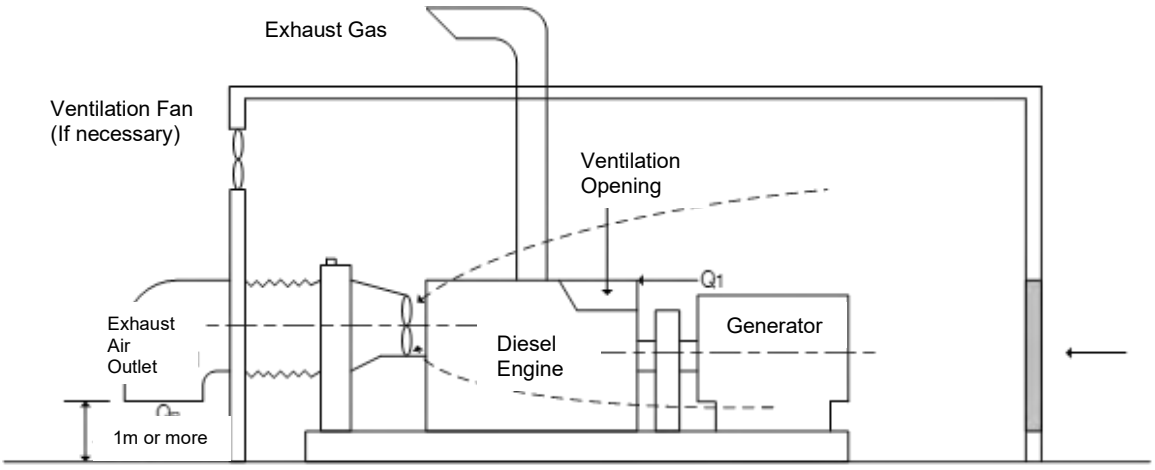


Source: Study Team

Figure 7.3.153 Radiator Cooling Type

4) Ventilation Volume Calculation

The schematic diagram of ventilation in the radiator cooling system is shown below.



Source: Study Team

Figure 7.3.154 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 motor

$$Q1 = (m3/min) \frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho}$$

A ′ : the amount of air required to burn 1 kg of fuel (m^3/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/m^3).

QR: Radiator fan ventilation (m3/min)

Table 7.3.182 Ventilation Amount by the Radiator Fan

Output Power		Ventilation by the Radiator Fan (m^3/min)	Louver Area (m^2)
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: Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting), page 3 -48

Each value and calculation result in this design are shown below.

Table 7.3.183 Calculated Ventilation Rate

Gate Opening And Closing	For Control Equipment
・A '= 1.47 ・be = 0.29 (7% increase of 0.27 kg/kWh for 184 to 331 kW diesel engines) ・Pe = 225 ・ε = 2.0 ・ ρ = 1.165 ・QR = 500 (Ref. **Table 7.3.182**) Q1 $=\frac{A' \times be \times Pe \times \varepsilon}{60 \times \rho}$ $=\frac{1.47 \times 0.29 \times 225 \times 2.0}{60 \times 1.165}$ Size = 2.74 Q = 2.74 + 500 = 502.74 (m3/min) = 30164.4 (m3/h)	・A '= 1.47 ・be = 0.32 (7% increase of 0.30 kg/kWh for 22 to 184 kW diesel engines) ・Pe = 49 ・ε = 2.0 ・ ρ = 1.165 ・QR = 175 (Ref. **Table 7.3.182**) 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) Fuel Type

Diesel oil is used.

(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 the Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting).

$$Fuel\ Consumption\ (L/h) = \frac{Motor\ Output\ (kW) \times Fuel\ Consumption\ Rate\ (g/kWh)}{1000 \times Specific\ Weight\ of\ Fuel}$$

Source: Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting), 29 edition, P3, -39

The fuel consumption rate and specific gravity of fuel are as follows according to the Guidelines for the Design of Telecommunications Facilities and the Explanation (electric (al) knitting).

Table 7.3.184 Fuel Consumption Rate (Unit: g/kWh)

Motor Output (kW)		≦22	22 to <184	184 to 331	331 to 552	552≦
Consumption Rate	Diesel	310	300	270	250	230
	Gas Turbine	-	680	660	590	520

Source: Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting), 29 edition, P3, -39

Table 7.3.185 Specific Gravity of Fuel

Fuel Type	Specific Gravity
A Heavy Oil	0.85
Light Oil	0.83
Kerosene	0.79

Source: Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting), 29 edition, P3, -39

Each value and calculation result in this design are shown below.

Gate Opening and Closing	For Control Equipment
・Motor output: 225 kW ・Fuel consumption rate: 270 (diesel engine) ・Specific gravity of fuel: 0.83 (light oil) Fuel consumption (L/h) $=\frac{Motor\ output(\mathrm{kW})\times Fuel\ consumption\ rate(g/\mathrm{kWh})}{1000\times Specific\ gravity\ of\ fuel}$ $=\frac{225\times270}{1000\times0.83}$ =73.2 7% increase due to radiator cooling system 73.2 × 1.07 = 78.3 (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) $=\frac{Motor\ output(\mathrm{kW})\times Fuel\ consumption\ rate(g/\mathrm{kWh})}{1000\times Specific\ gravity\ of\ fuel}$ $=\frac{48.6\times300}{1000\times0.83}$ =17.57 7% increase due to radiator cooling system 17.57 × 1.07 = 18.80 (L/h)

(ii) Amount of Fuel Oil Stored

The required amount of fuel oil storage is calculated by the following formula based on the Telecommunication Facilities Design Guidelines and the Electrical Manual (H 29).

Amount of Stored Fuel Oil (L) = Fuel Consumption (L/h) × Operation Time (h)

Source: Telecommunications Facilities Design Guidelines and Explanations (electric (al) knitting), 29 edition

The operating time was calculated as 8 hours for the gate opening/closing generator and 72 hours for the control equipment generator according to the design conditions. The amount of stored fuel oil is as follows.

Gate Opening And Closing	For Control Equipment
Amount of fuel oil stored Size = 78.3 x 8 = 627 (L)	Amount of fuel oil stored Size = 18.80 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.

Gate Open/Close Generator 250 kVA

50 kVA Generator for Control Equipment

・ Bunker

(a) Generator for Gate Operation

The generator for gate operation is installed in 2 types of normal use and emergency use in order to enable sure gate operation in the power failure and to ensure the power supply even in the equipment failure. The generator for gate opening and closing is planned in accordance with the layout plan of the generator house (See **Figure 7.3.155**).

(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 7.3.155**).

(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.

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 7.3.186 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

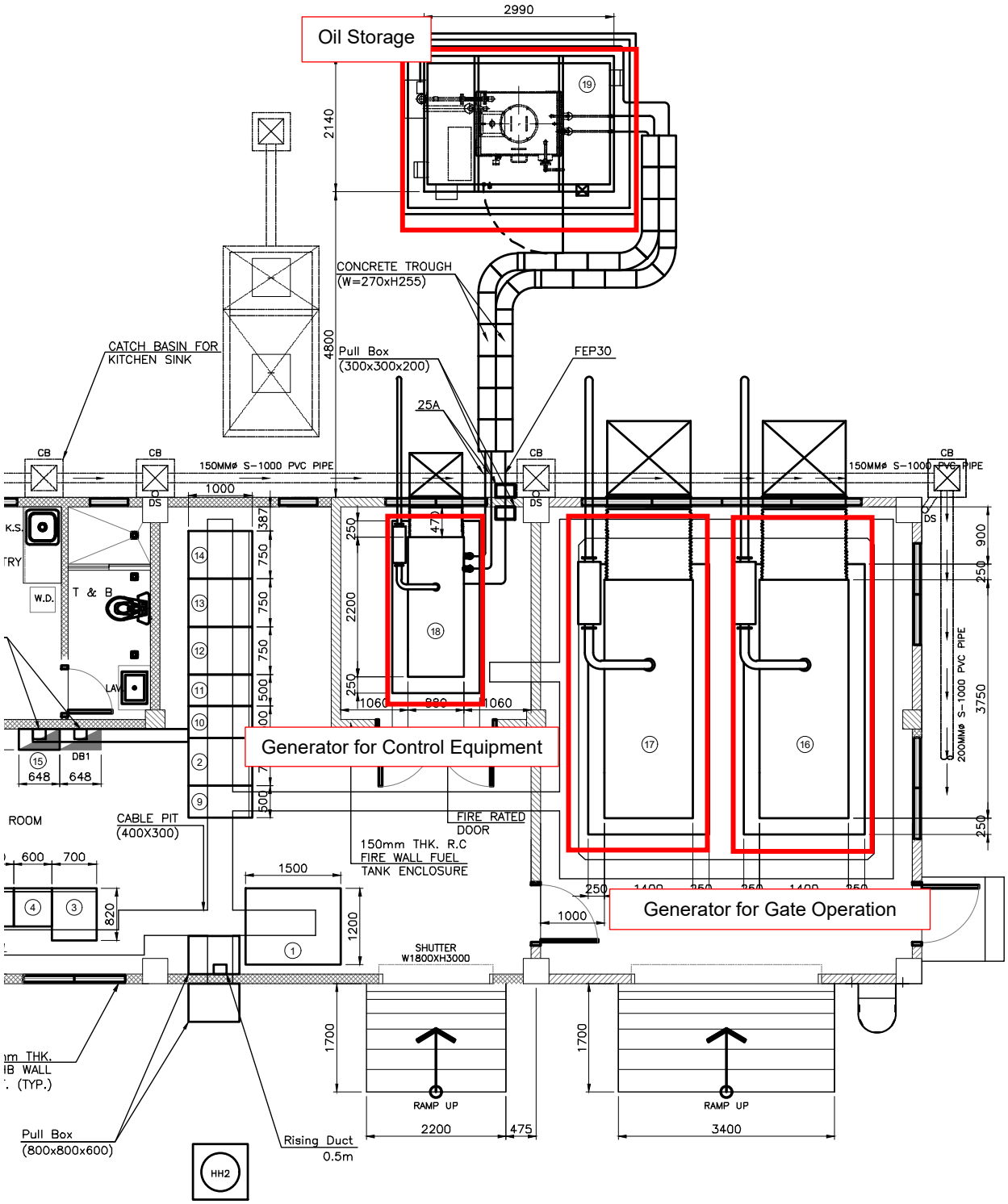


Figure 7.3.155 Arrangement of Generators and Oil Storage

6) Generator Room Plan

(a) Ventilation System

The natural ventilation system was 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.

$$A = \frac{Q1 + QR}{60 \times V \times \alpha} \text{ (m2)}$$

where:

A : gallery area (m2)

Q1 : Amount of air required for combustion of fuel oil (m3/min)

QR : Radiator fan ventilation (m3/min)

V : wind speed 4 m/sec

A : Permeability of the collar (Size = 0.3), and transmittance when a wire gauze is installed on the collar (Size = 0.27)

Each value and calculation result in this design are shown below.

Gate Opening and Closing	For control Equipment
$A = \frac{2.75 + 500}{60 \times 4 \times 0.27} = 7.76$ m 2	$A = \frac{0.66 + 175}{60 \times 4 \times 0.27} = 2.71$ m 2

(b) Minimum Distance between Devices

The minimum distance between the components of the generator is described in the Telecommunication Facilities Design Guidelines and Explanations (electric (al) knitting), 29, page 3 -65 (**Table 7.3.187**).

Table 7.3.187 Minimum Distance between Devices

Portion Toto 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 motor shall be directly connected to each other and fixed to a common trestle having a vibration-proof and seismic-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. To generator dimensions and foundation dimensions (**Table 7.3.188**).

Table 7.3.188 Generator Dimensions and Foundation Dimensions

Generator	Model Number	Generator Dimensions (mm)	Foundation Dimensions (Mm)
250 kVA	DCA -300 SPK3	Width: 1400 length: 3750	Width: 1400 + 300 = 1700 Length: 3750 + 300 = 4050
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 size of anchor bolt of the generator and the fuel tank shall be selected based on the "Guidelines for seismic-resistant design and construction of building equipment 2014". As a result of the calculation, the following anchor bolts were selected. The detailed calculation is indicated in **Vol.4A Structural Calculation for Contact 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 7.3.189**.

Table 7.3.189 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"